



**REPORT**

# **Preliminary Foundation Investigation and Design Report**

*Holland River Structure*

*Highway 400 to Highway 404 Link (Bradford Bypass)*

*Simcoe County and York Region*

*Assignment No. 2019-E-0048*

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# PART A

**PRELIMINARY FOUNDATION INVESTIGATION REPORT  
HOLLAND RIVER STRUCTURE  
HIGHWAY 400 TO HIGHWAY 404 LINK (BRADFORD BYPASS)  
SIMCOE COUNTY AND YORK REGION  
ASSIGNMENT NO. 2019-E-0048**

## 1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by AECOM on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the proposed Bradford Bypass (BBP), a 16.2 km rural controlled access freeway connecting Highway 400 to Highway 404, in the County of Simcoe and Regional Municipality of York. This report addresses the foundation investigation carried out for planning and preliminary design of the bridge (twin structures) to carry the Bradford Bypass eastbound and westbound lanes over the Holland River as shown on the Key Plan of Drawing 1.

## 2.0 SITE DESCRIPTION

The proposed bridge will span across the Holland River, which divides the County of Simcoe and Region of York and is located between the 8<sup>th</sup> Line and 9<sup>th</sup> Line. The site is generally flat and rises slightly to the west towards to the CN Railway located about 400 m west of the river. The west side of Holland River is swampy near the river and transitions to a forested area to the west (see Photograph 1). The east side of Holland River consists of predominantly flat farmland with an apparent berm / dyke running along the eastern edge of the river with water-filled ditches and channels present along Hochreiter Road and on the east side of the apparent berm / dyke (see Photograph 2). It is understood that the ground surface east of Holland River is near or below the river water level during certain periods of the year, thus the need for the protection berm / dyke and based on conversations with the hydraulics team, active pumping of surface water / groundwater from designated collection points in the ditches / channels (near or within the proposed bridge abutment / approach embankment location) back into the river is required to facilitate farming activities in the low-lying floodplain area. Although details are not known at this preliminary stage, it is recommended that the dyke protection and surface water / groundwater collection system and active dewatering be investigated further during detail design.



*Photograph 1: West side of Holland River looking at swampy area from edge of forest.*



*Photograph 2: East side of Holland River looking west from end of Hochreiter Road (near HRW-4)*

The Holland River is up to about 2 m deep in this area and flows from south to north and outlets into Lake Simcoe. There is no existing infrastructure along the riverbank in this area other than some farm related sheds / barns, although the river is considered a navigable waterway. There were no underground utilities identified at or near the boreholes, although buried utilities may be present at private properties located along the proposed

highway alignment. Overhead hydro was visually confirmed just south of the proposed alignment (along 8<sup>th</sup> Line and Hochreiter Road) and crosses Holland River by a submerged hydro cable south of the proposed bridge location.

### 3.0 INVESTIGATION PROCEDURES

The field work for the current investigation was carried out between October 4 and December 21, 2021, during which time two boreholes (designated HRW-1 and HRW-4) and two Seismic Cone Penetration Tests (designated sCPT21-HRW-1 and sCPT21-HRW-4) were advanced at the site, at the locations shown on Drawing 1. Boreholes HRW-1 and sCPT21-HRW-1 were advanced on the west side of the Holland River and Boreholes HRW-4 and sCPT21-HRW-4 were advanced on the east bank of the Holland River.

All boreholes were advanced using 210 mm outside diameter (O.D.) hollow-stem augers generally set a depth of approximately 2.5 m followed by wash-rotary techniques (advancement of tricone with water/drilling mud) using a Diedrich D50 track-mounted drill supplied and operated by Walker Drilling Inc. of Utopia, Ontario. The wash-rotary technique was used to counter-balance hydrostatic forces and reduce disturbance at the sampling and testing interval. Water used for the drilling operation was brought to site in totes (portable plastic tanks) by the drilling subcontractor.

Soil samples were generally obtained at 0.75 m, 1.5 m, and 3.0 m intervals of depth using a 50 mm O.D. split-spoon sampler driven with an automatic hammer in general accordance with Standard Penetration Test (SPT) procedure (ASTM D1586)<sup>1</sup>, and using 76 mm O.D. thin-walled 'Shelby' Tube samplers (ASTM D1587)<sup>2</sup> to obtain relatively undisturbed samples in the soil. Given the consistency of the cohesive strata at this site (i.e. generally stiff to very stiff), the use of a Piston-sampler, which is typically used to achieve better recovery in very soft to soft soils, was not used at this site. In-situ field vane shear tests were carried out using an MTO 'N'-vane in the cohesive soils to assess shear strength (ASTM D2573)<sup>3</sup>.

The Seismic Cone Penetration Test (sCPT) holes were advanced using a track-mounted CPT rig equipped with 25-ton rig cylinders and a 150 MPa sCPT cone, which were all supplied and operated by ConeTec Investigations Ltd. (ConeTec) of Richmond Hill, Ontario.

The groundwater conditions in the open boreholes were observed during and immediately following the drilling operations. A standpipe piezometer was installed in Boreholes HRW-1 and HRW-4 to allow monitoring of the water level(s). The installed piezometers consist of a 50 mm diameter PVC pipe, with a 1.5 m long slotted screen within a filter sand pack. The borehole and annulus surrounding the piezometer pipe above the filter sand pack were backfilled to near ground surface with bentonite pellets. The monitoring wells were capped with above grade (steel monument) casings.

The field work was monitored on a full-time basis by a member of Golder's engineering staff who located the boreholes and sCPTs in the field, directed the sampling and in-situ testing operations, logged the boreholes and examined the soil samples. The soil samples were identified in the field, placed in labelled containers and transported to Golder's laboratory in Mississauga for further visual review and geotechnical laboratory testing.

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<sup>1</sup> ASTM D1586 – Standard Test Method for Standard Penetration Tests and Split Barrel Sampling of Soils.

<sup>2</sup> ASTM D1587 - Standard Practice for Thin-Walled Tube Sampling of Fine-Grained Soils for Geotechnical Purposes.

<sup>3</sup> ASTM D2573 - Standard Test Method for Field Vane Shear Test in Saturated Fine-Grained Soils.

Index and classification testing consisting of natural moisture content, organic content, Atterberg limits and grain size distribution were conducted on selected samples. In addition, one-dimensional consolidation (oedometer) testing was carried out on selected soil specimens obtained from the Shelby tube samples. All laboratory tests were carried out in general accordance with MTO and / or ASTM Standards, as applicable.

Selected soil samples were submitted to a specialist analytical laboratory (Bureau Veritas Laboratories of Mississauga, Ontario) under chain of custody procedures for testing of conductivity / resistivity, pH, and chemical analysis of sulphate and chloride content, to assess the potential for the soil to cause deterioration to buried concrete and corrosion to steel.

The borehole and sCPT locations were surveyed in the field by Golder personnel using a Trimble Geo 7X Global Positioning System (GPS) unit. The Trimble Geo 7X achieved a horizontal accuracy of 2 cm to 5 cm and vertical accuracy of 4 cm while in use at the site. The locations given on the Record of Borehole sheets and shown on Drawing 1 are positioned relative to MTM NAD 83 (Zone 10) northing and easting coordinates and the ground surface elevations are referenced to Geodetic datum (CGVD28 datum). The borehole locations, including the geographic (Latitude / Longitude) coordinates, the ground surface elevations, and borehole / sCPT depths are summarized below.

Borehole / sCPT No.	MTM NAD83		Ground Surface Elevation (m)	Borehole / CPT Depth (m)
	Northing (m) (Latitude)	Easting (m) (Longitude)		
HRW-1	4,887,870.6 (44.13082)	300,852.7 (-79.549328)	219.0	39.9
sCPT21-HRW-1	4,887,864.5 (44.130765)	300,859.7 (-79.549240)	219.0	19.8
HRW-4	4,888,043.9 (44.132381)	301,230.5 (-79.544608)	217.4	49.2
sCPT21-HRW-4	4,888,040.9 (44.132335)	301,224.1 (-79.544689)	217.4	42.1

Note: sCPT21-HRW-1 is designated as sCPT21-HRW-01b in the ConeTec report in Appendix A.

## 4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

### 4.1 Regional Geology

This section of the Bradford Bypass is located in an area defined as the Simcoe Lowlands physiographic region, as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, 1984)<sup>4</sup>.

<sup>4</sup> Chapman, L.J. and Putnam, D.F. 1984. *The Physiography of Southern Ontario*, Ontario Geological Survey, Special Volume 2, Third Edition. Accompanied by Map P. 2715, Scale 1:600,000.



The Simcoe Lowlands physiographic region covers the central portion of the County of Simcoe. Following the retreat of the last glacial ice sheet, the lowland was flooded by the now extinct post-glacial Lake Algonquin. This past post-glacial lacustrine environment is marked by deep sand, silt and clay beds overlying glacial ground moraine material.

## 4.2 General Overview of Subsurface Conditions

The detailed subsurface soil and groundwater conditions encountered in the boreholes from the current investigation including piezometer installation details and water level readings, and the results of the in-situ and laboratory tests are provided on the Record of Borehole Sheets in Appendix A. The results of the in-situ field tests (i.e., SPT “N”-values and shear strengths from the field vanes) as presented on the borehole records and in Section 4 are uncorrected. The results of the sCPT testing are also provided in Appendix A. The results of the geotechnical laboratory testing on soil samples are presented on the laboratory test figures in Appendix B. The results of the analytical testing are provided in Appendix C.

The stratigraphic boundaries shown on the borehole records and on the stratigraphic profile on Drawing 1 are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Variation in the stratigraphic boundaries between and beyond boreholes will exist and is to be expected.

In general, the subsurface soils encountered near the banks of the Holland River consist of surficial layers of fill and organics underlain by a very loose to loose silty sand to sandy silt deposit, further underlain by an upper cohesive deposit consisting of stiff to very stiff clayey silt to clayey silt-silt. The upper cohesive deposit is underlain by a non-cohesive deposit of compact to very dense silty sand to sandy silt. This layer is underlain or interlayered with a lower cohesive deposit consisting of stiff clayey silt to clayey silt-silt which is in turn underlain by a deposit consisting of very stiff to hard clayey silt-silt till in borehole HRW-4. Below the clayey silt-silt till, borehole HRW-4 encountered a deposit consisting of very dense sandy silt.

Detailed descriptions of the major layers encountered in the boreholes are provided in the following sections.

### 4.2.1 Topsoil

An approximate 200 mm thick layer of topsoil was encountered in both boreholes at ground surface. The water content measured on a sample of the topsoil was 18% and the result of an Atterberg Limits test indicates the topsoil is non-plastic.

### 4.2.2 Fill

An approximately 0.5 m to 0.7 m thick layer of silty sand to sand (fill) was encountered in both boreholes below the topsoil. The base of the fill layer extended to Elevations 218.2 m and 216.7 m.

The SPT ‘N’-values measured within the silty sand to sand fill range from 11 to 12 blows per 0.3 m of penetration, indicating a compact state.

The natural water content measured on a selected sample of the fill was about 10%.

### 4.2.3 Sandy Peat to Organic Silt

A deposit of sandy peat to organic silt was encountered below the fill layer in both boreholes. The deposit was about 0.1 m and 1.0 m thick and extended to depths of 0.9 m and 1.7 m (Elevations 218.1 m and 215.7 m) in

HRW-1 and HRW-4. In Borehole HRW-4, the organic deposit was interlayered with the sandy silt to silty sand deposit described in the next section.

Atterberg limits testing was carried out on a selected sample of the organic deposit indicates that the material is non-plastic.

The natural water content measured on a selected sample of the sandy peat was about 61%.

#### **4.2.4 Silty Sand to Sandy Silt**

A deposit of silty sand to sandy silt was encountered underlying the organic deposit in both boreholes. The deposit was 1.3 m and 0.2 m thick and extended to a depth of 2.2 m and 1.8 m (Elevations 216.8 m and 215.5 m) in HRW-1 and HRW-4 respectively. As mentioned in the previous section, an approximate 0.5 interlayer of sandy silt was also encountered within the organic deposit described above in HRW-4.

The SPT 'N'-values measured in the silty sand to sandy silt range from 1 to 10 blows per 0.3 m of penetration, indicating a very loose to loose state of compactness.

Grain size distribution testing was carried out on a sample of the silty sand to sandy silt and the results are shown on Figure B1 in Appendix B.

Atterberg limits testing was carried out on a selected sample of the silty sand to sandy silt deposit and the results indicate that the sample is non-plastic.

The natural water content measured on selected samples of the silty sand to sandy silt deposit ranges between about 19% and 22%. A laboratory organic content test measured an organic content of 0.4% on a sample of the sandy silt interlayer encountered in Borehole HRW-4.

#### **4.2.5 Clayey Silt to Clayey Silt-Silt (Upper Cohesive Deposit)**

An upper deposit of clayey silt to clayey silt-silt was encountered underlying the sandy silt to silty sand deposit in both boreholes. The thickness of the deposit was approximately 0.8 m and 9.9 m and extended to depths of 3.0 m and 11.7 m (Elevations 216.0 m and 205.6 m) in HRW-1 and HRW-4. The cohesive deposit is characterized as having sand and silt seams/layers throughout.

The SPT 'N'-values measured within the clayey silt-silt deposit typically range from 7 to 18 blows per 0.3 m of penetration. Three in-situ field vane tests carried out within the upper cohesive deposit in Borehole HRW-4 measured undrained shear strengths ranging from about 86 kPa to >100 kPa. The results of the SPT 'N'-values and field vane tests suggests the clayey silt to clayey silt-silt deposit exhibits a stiff to very stiff consistency.

Grain size distribution tests were carried out on four samples of the upper cohesive deposit, and the results are shown on B2.

Atterberg limits testing was carried out on three selected samples of the upper cohesive deposit and measured liquid limits ranging between 18% and 24%, plastic limits ranging between 14% and 16%, and plasticity indices ranging between 4% and 8%. These results, which are plotted on a plasticity chart on Figure B3, indicate that the deposit consists of clayey silt to clayey silt-silt of low plasticity.

The water content measured on samples of the upper cohesive deposit generally ranges between 18% and 22%. A higher moisture content of 27% was measured in HRW-4 near the interface with the overlying silty sand and peat layers.

Three Shelby tubes were attempted in the upper cohesive deposit and two Shelby tubes collected samples at depths of 2.1 m and 5.2 m in Borehole HRW-4. An attempt was made to collect a Shelby tube from this deposit in HRW-1 however the sample was highly disturbed and limited recovery collected, likely due to the presence of frequent sand seams/layers. Laboratory consolidation (oedometer) testing was carried out on a vertically trimmed specimen of the clayey silt-silt obtained from the Shelby tube sample at 5.2 m depth to assess the compressibility characteristics of the deposit. The details of the test results are shown on Figures B10 to B13 and are summarized below.

Borehole / Sample No.	Sample Depth / Elevation (m)	$\sigma_{vo}'$ (kPa)	$\sigma_p'$ (kPa)	$\sigma_p' - \sigma_{vo}'$ (kPa)	OCR (avg.)	$C_c$	$C_r$	$e_o$	$C_v$ (cm <sup>2</sup> /s)
HRW-4 / TO-8	5.2 / 212.2	59	453 to 564	394 to 505	8.6	0.147	0.004 to 0.009	0.65	0.0091 to 0.0274

Where:  $\sigma_p'$  = Estimated preconsolidation stress (using Casagrande construction and Work interpretation methods)  $c_v$  = Coefficient of consolidation (vertical) for approximate overconsolidated stress range  $50 \text{ kPa} \leq \sigma_v' \leq 300 \text{ kPa}$

$C_c$  = Compression index  $C_r$  = Recompression index

$e_o$  = Initial void ratio OCR = Overconsolidation ratio

$\sigma_{vo}'$  = Calculated existing vertical effective stress

## 4.2.6 Sandy Silt to Silty Sand

A deposit of sandy silt to silty sand was encountered underlying the upper clayey silt to clayey silt-silt deposit in both boreholes. The deposit was 12.2 m and 4.6 m thick and extended to depths 15.2 m and 16.3 m (Elevations 203.8 m and 201.0 m) in HRW-1 and HRW-4 respectively. Silt, gravel, and clay seams/interlayers were encountered throughout the deposit. Grinding of the auger/tri-cone bit was observed at a depth of 9.1 m suggesting the presence of gravel pockets and/or cobbles.

The SPT 'N'-values measured in the sandy silt to silty sand range from 8 to 74 blows per 0.3 m of penetration, suggesting a loose to very dense (and generally compact to dense) state of compactness.

Grain size distribution tests were carried out on four samples of the sandy silt to silty sand deposit, and the results are shown on Figure B4.

The natural water content measured on samples of the sandy silt to silty sand range between 8% and 21%.

## 4.2.7 Clayey Silt to Clayey Silt-Silt – Lower Cohesive Deposit

A lower cohesive deposit of clayey silt to clayey silt-silt was encountered underlying the sandy silt to silty sand deposit in both boreholes. The deposit was 6.1 m thick in HRW-1 and 18.7 m thick in HRW-4. The deposit extends to a depth of 21.3 m and 35 m (Elevation 197.7 and 182.3 m) in HRW-1 and HRW-4 respectively. The lower cohesive deposit contained frequent sand silt seams / layers.

The SPT 'N'-values measured in the lower cohesive deposit range from 8 to 17 blows per 0.3 m of penetration. One lower 'N'-value of 3 was measured near the interface with the overlying silty sand in HRW-1 and may have

been influenced by sampling disturbance (i.e. blowing sands). Four in-situ field vane tests carried out within the lower cohesive deposit in both boreholes measured undrained shear strengths ranging from about 75 kPa to >100 kPa, with one lower value of 14 kPa measured near the interface with the overlying silty sand in HRW-1 and may have been influenced by sampling disturbance. Based on the SPT 'N'-values and field vane tests, the deposit is considered to generally have a stiff to very stiff consistency.

Grain size distribution tests were carried out on three samples of the lower cohesive deposit and the results are shown on Figure B5.

Atterberg limits testing was carried out on four selected samples of the lower cohesive deposit and measured liquid limits ranging between 17% and 25%, plastic limits ranging between 12% and 15%, and plasticity indices ranging between 5% and 10%. These results, which are plotted on a plasticity chart on Figure B6, indicate that the deposit consists of clayey silt to clayey silt-silt of low plasticity.

The water content measured on samples of the cohesive deposit generally ranges between 12% and 26%.

Two Shelby tube samples were attempted at depths of 17 m and 25 m below ground surface in HRW-1 and HRW-4 respectively; however, there was limited sample recovered from the Shelby tubes due to the stiff to very stiff consistency and presence of silt and sand seams.

#### **4.2.8 Silty Sand**

On the west side of the river, a silty sand layer was encountered below the lower cohesive deposit in HRW-1. The silty sand layer was 12.3 m thick and extended to a depth of 33.6 m (Elevation 185.4 m).

The SPT 'N'-values measured in the silty sand range from 100 blows per 0.3 m of penetration to 121 blows per 0.26 m of penetration, indicating the deposit is very dense.

Grain size distribution tests were carried out on two samples of the silty sand deposit and the results are shown on Figure B4.

The natural water content measured on samples of the silty sand range between 16% and 19%.

#### **4.2.9 Clayey Silt-Silt**

On the west side of the river, a clayey silt-silt deposit was encountered below the silty sand layer in HRW-1. Borehole HRW-1 was terminated within this deposit at a depth of 39.9 m (Elevation 179.1 m), after penetrating 6.3 m into the deposit. The deposit typically contained trace sand and trace gravel.

The SPT 'N'-values measured in the clayey silt-silt range from 100 blows per 0.26 m of penetration to 100 blows per 0.05 m of penetration, suggesting the deposit is hard.

A grain size distribution test was carried out on a sample of the clayey silt-silt deposit and the results are shown on Figure B5.

Atterberg limits testing was carried out on a selected sample of the clayey silt-silt deposit and the results indicate a liquid limit of 21%, plastic limit of 15%, and plasticity index of 6%. These results, which are plotted on a plasticity chart on Figure B6, indicate that the deposit is a clayey silt-silt of low plasticity.

The natural water content measured on samples of the clayey silt-silt were 8% and 17%, with the lower moisture content measured near the interface with the overlying silty sand.

#### 4.2.10 Clayey Silt-Silt Till

On the east side of the river, a deposit of clayey silt-silt till was encountered underlying the lower cohesive deposit in Borehole HRW-4. The deposit was 12.2 m thick and extended to a depth of 47.2 m (Elevation 170.1 m). Auger/tri-cone grinding was observed between a depth of 40 m and 42 m suggesting the presence of gravel pockets and/or cobbles.

The SPT 'N'-values measured within this layer range from 15 to 100 blows per 0.3 m of penetration suggesting a very stiff to hard consistency.

Grain size distribution testing was carried out on one sample of the cohesive till and the results are shown on Figure B7.

Atterberg limits testing was carried out on one sample of the cohesive till and measured a liquid limit of 17%, a plastic limit of 12%, and a plasticity index of 5%. These results, which are plotted on a plasticity chart on Figure B8, indicate that the deposit consists of clayey silt-silt of low plasticity.

The natural water content measured on a sample of the cohesive till was about 13%.

#### 4.2.11 Sandy Silt

On the east side of the river, a lower deposit of sandy silt was encountered below the cohesive till deposit in Borehole HRW-4. The deposit was encountered for a thickness of 2.0 m, at which point the borehole was terminated within the deposit at a depth of 49.2 m (Elevation 168.1 m). Auger/tri-cone grinding was observed during drilling between a depth of 48.0 m and 48.8 m suggesting the presence of gravel pockets and/or cobbles.

The SPT 'N'-value measured within this layer was 103 blows per 0.3 m penetration indicating a very dense state of compactness.

Grain size distribution testing was carried out on one sample of the lower sandy silt deposit and the results are shown on Figure B9.

Atterberg limits testing carried out on a selected sample of the lower sandy silt deposit indicates that the material is non-plastic.

The natural water content measured on a selected sample of the sandy silt was about 10%.

### 4.3 Seismic Cone Penetration Test Results

The sCPTs were advanced from ground surface to practical cone refusal with continuous measurement of tip resistance ( $q_t$ ), sleeve friction ( $f_s$ ), and dynamic pore water pressure ( $u$ ). As part of the cone penetration testing, pore water pressure dissipation tests were carried out at select depths within the cohesive deposits. In addition, shear wave velocity ( $V_s$ ) testing was performed at approximately 1 m intervals of depth. The results of the sCPTs are presented in the Cone Penetration Test Report in Appendix A along with the results of the sCPT advanced at the Holland River East Branch site, and the profile of the shear wave velocity at each cone penetration test location at the Holland River site is shown on Drawing 1.

The pore pressure response measured during cone penetration testing indicates layers of higher and lower drainage capacity exist within the upper and lower cohesive deposits. These higher permeability layers were generally encountered at a high frequency within the full thickness of the upper cohesive deposit (2 m to 3 m depth in sCPT21-HRW-1 and 2 m to 12 m depth in sCPT21-HRW-4) and majority (16 m to 30 m depth) of the



lower cohesive deposit in sCPT21-HRW-4. The high permeability layers were less frequent or non-existent in the upper cohesive deposit in sCPT21-HRW-1 (14 m to 19 m depth) and bottom portion of the lower cohesive deposit in sCPT21-HRW-4 (30 m to 42 m depth). The pore water pressure response (i.e. frequent build-up and dissipation during advancement of the cone) is representative of the presence of frequent sand / silt lenses and layers encountered in the upper and lower cohesive deposits in the adjacent boreholes.

The coefficient of consolidation ( $c_h$ ) value and preconsolidation stress within the upper and lower cohesive deposits were also evaluated from the results of the sCPT testing (i.e. dissipation tests and  $q_t$ ) using equations presented by Houlsby and Teh (1991) and Demers and Leroueil (2002). The results of the calculated preconsolidation stress with depth are shown on Figure 1 and summarized below along with estimated undrained shear strength, OCR, and  $c_h$  values.

Location	sCPT Designation	Soil Strata	$C_u$ (kPa)	$\sigma_p'$ (kPa)	$\sigma_p' - \sigma_{vo}'$ (kPa)	OCR (avg.)	$c_h$ (cm <sup>2</sup> /s)
West Side of River (near HRW-1)	sCPT21-HRW-1	Upper cohesive deposit	150-200	700-900	675-850	>10	Note 1
		Lower cohesive deposit	85-300	400-1,300	230-1,200	2-9	0.01 (Note 1)
East Side of River (near HRW-4)	sCPT21-HRW-4	Upper cohesive deposit	65-190	300-900	125-400	3-9	0.06 to 1.37
		Lower cohesive deposit	85-175	400-800	50-125	1.1-1.4	0.007 to 0.05

Where:

$$\sigma_p' = \frac{q_t - \sigma_{vo}}{3.4} = \text{range of estimated preconsolidation stress after "smooth fit" line interpretation of raw data (Demers and Leroueil, 2002)}$$

$q_t$  = corrected tip resistance

$\sigma_{vo}$  = vertical stress

$c_h$  = estimated horizontal coefficient of consolidation (Houlsby and Teh, 1991)

Note 1: Another dissipation test was attempted but surrounding soil too permeable and no pore pressure dissipation measured.

In general, the sCPT results indicate the upper and lower cohesive deposits are overconsolidated and exhibit a similar consistency / strength in comparison to the borehole in-situ and laboratory test results.

## 4.4 Groundwater Conditions

The groundwater levels in the open boreholes were measured during and upon completion of drilling operations as noted on the borehole records in Appendix A. The water levels measured in the open boreholes at the time of the investigation are not considered representative of the hydrostatic water levels at the site due to the addition of drilling fluids/water into the boreholes and considering the water levels did not have sufficient time to stabilize.

A standpipe piezometer was installed in Boreholes HRW-1 and HRW-4 to allow monitoring of the groundwater level at this site. The groundwater levels recorded in the piezometers are shown on the borehole records in Appendix A and are summarized below.

Borehole No.	Ground Surface Elevation (m)	Depth to Water Level (m)	Groundwater Elevation (m)	Date
HRW-1	219.0	0.44	218.6	February 7, 2022
		0.67	218.3	May 13, 2022
HRW-4	217.4	1.7	215.7	Oct 6, 2021
		0.5	216.9	Dec 23, 2021
		0.55	216.8	May 13, 2022

The groundwater level observations at this site will be subject to seasonal fluctuations and precipitation events; the water levels should be expected to be higher during the spring season or during and following periods of heavy precipitation and snow melt. Groundwater levels will also be influenced by the water level in the Holland River that is reported to have a high water level at El. 219.7 m (100-year design event).

## 4.5 Analytical Testing of Soil

Two soil samples were submitted for analysis of parameters used to assess the potential corrosivity of the site soil to steel and concrete. Detailed analytical test results are included in Appendix C and the test results are summarized below:

Borehole No. - Sample No.	pH	Resistivity (ohm-cm)	Electrical Conductivity (µmho/cm)	Soluble Chloride (µg/g)	Soluble Sulphate (µg/g)
HRW-4, Sa#1	7.59	4700	213	34	<20 <sup>1</sup>
HRW-1, Sa#5	7.94	6000	167	59	<20 <sup>1</sup>

Notes: 1. Measured value less than reportable detection limit

## 5.0 CLOSURE

This Foundation Investigation Report was prepared by Mr. Carter Comish, P.Eng., and reviewed by Mr. Kevin Bentley, P.Eng., a Geotechnical Engineer with WSP Golder and MTO Foundations Designated Contact.

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[https://golderassociates.sharepoint.com/sites/120387/project files/6 deliverables/foundations/holland river \(i.e. holland river west\)/final/19136074-r-rev0-hr fidr\\_28april2023 - final.docx](https://golderassociates.sharepoint.com/sites/120387/project%20files/6%20deliverables/foundations/holland%20river%20(i.e.%20holland%20river%20west)/final/19136074-r-rev0-hr%20fidr_28april2023-final.docx)

# **PART B**

**FOUNDATION DESIGN REPORT  
HOLLAND RIVER STRUCTURE  
HIGHWAY 400 TO HIGHWAY 404 LINK (BRADFORD BYPASS)  
SIMCOE COUNTY AND YORK REGION  
ASSIGNMENT NO. 2019-E-0048**

## 6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

### 6.1 General

This section of the report provides foundation recommendations for planning and preliminary design of the Bradford Bypass / Holland River bridge crossing near Bradford, Ontario. The recommendations are based on interpretation of the factual data obtained from the boreholes and sCPTs advanced on the west and east side of the Holland River during the current subsurface investigation.

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

### 6.2 Project Understanding

Based on the latest preliminary Bradford Bypass mainline alignment and profile drawings provided by AECOM (dated August 2022), the proposed bridge will consist of twin structures to carry the Bradford Bypass eastbound and westbound lanes over the Holland River. The total span of each bridge structure is anticipated to be a minimum length of 580 m, with preferred alternatives currently consisting of 6-span and 7-span options (with spans up to 115 m to 120 m long across Holland River). Each bridge structure will accommodate two lanes of traffic in the eastbound and westbound direction (four lanes total) for the interim configuration, with an ultimate future configuration to accommodate four lanes in each direction (eight lanes total). The approach embankments are proposed to be about 9 m and 14 m high at the east and west approach embankments, respectively. Given the size of the structures and according to the structural designers, the Holland River bridges are classified as a “lifeline” structures (according to the importance category designation in the CHBDC) at this preliminary stage.

The bridges will span over provincially significant wetlands and floodplain, and the piers (shafts/columns) are proposed to be placed outside of the current Holland River (i.e. no piers within the river), although temporary excavations and construction access within the river is anticipated.

### 6.3 General Foundation Design Context

#### 6.3.1 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the *Canadian Highway Bridge Design Code CAN/CSA S6-19* (CHBDC, 2019) and its *Commentary*, the bridge structure and its foundation system may be designed for applications essential to post-disaster recovery (i.e. lifeline) and having large societal or economic impacts, resulting in a “high consequence level” associated with exceeding limit states design.

In addition, based on the preliminary level of foundation investigation completed to date at this location (see Part A of this report) in comparison to the degree of site understanding, the level of confidence for design of the Holland River bridge foundation elements and approach embankments has been assessed as a “low degree of site and prediction model understanding”. At the time of issue of this report, the locations of the abutments and



pier foundations were not confirmed. As such, the recommendations contained in the report are generalized for planning and ongoing preliminary design and further investigation will be required when actual locations of the abutments and piers are known.

Accordingly, the appropriate corresponding ultimate limit state (ULS) and serviceability limit state (SLS) consequence factor,  $\Psi$ , and geotechnical resistance factors,  $\phi_{gu}$  and  $\phi_{gs}$ , from Tables 6.1 and 6.2 of the CHBDC (2019) have been used for the concept design. During detail design, additional investigation and testing must be performed to increase the level of confidence and modify the geotechnical resistance factors as appropriate. In addition, reference is made to the MTO Material Engineering and Research Office (MERO) Memorandum #2020-01 (dated March 23, 2020) for future selection of geotechnical resistance factors for analyses during detail design, as applicable.

### 6.3.2 Seismic Design

#### 6.3.2.1 Seismic Site Classification

The subsurface conditions for seismic site characterization were assessed based on the results of the field investigation, specifically the energy-corrected average shear wave velocity ( $\bar{V}_s$ ) from the seismic CPTs and penetration resistance ( $\bar{N}_{60}$ ) from the boreholes within the upper 30 m of the soil layers below the approximate founding level (i.e. existing ground surface). The average  $\bar{V}_s$  on the east and west sides of the river were measured to be about 290 m/s and 400 m/s respectively. It is noted that the sCPT achieved refusal on the west side at about 20 m below ground surface, and the  $\bar{N}_{60}$  values measured in the lower 10 m of soil were consistently greater than 100 blows per 0.3 m. Based on the results, the east and west sides of the river may be classified as Site Class D and Site Class C in accordance with Table 4.1 of the CHBDC (2019). Additional site-specific study will be required during detail design to evaluate the subsurface conditions and site-superstructure response to determine if the use of more than one Site Class is warranted. Recommendations for additional foundation investigation are provided in Section 6.10.

The CHBDC (2019) states that the seismic hazard values associated with the design earthquakes should be those established for the National Building Code of Canada (NBCC) by the Geological Survey of Canada (GSC). The 2015 seismic hazard maps (referred to as the 5th generation seismic hazard maps) have been used for preliminary design for this project, as referenced in the CHBDC (2019).

#### 6.3.2.2 Spectral Response Values and Seismic Performance Category

In accordance with Section 4.4.3.1 of the CHBDC (2019), the peak ground acceleration ( $PGA$ ), peak ground velocity ( $PGV$ ) and 5% damped spectral response acceleration ( $S_a(T)$ ) values for Site Class C and D were obtained for the bridge site using the NBCC website ([earthquakescanada.nrcan.gc.ca](http://earthquakescanada.nrcan.gc.ca)) and are summarized below.

Seismic Hazard Values for Site Class C	10% Exceedance in 50 years (475-year return period)	5% Exceedance in 50 years (975-year return period)	2% Exceedance in 50 years (2,475-year return period)
$PGA$ (g)	0.030	0.046	0.074
$PGV$ (m/s)	0.027	0.042	0.067
$S_a(0.2)$ (g)	0.053	0.079	0.122
$S_a(0.5)$ (g)	0.037	0.054	0.082

Seismic Hazard Values for Site Class C	10% Exceedance in 50 years (475-year return period)	5% Exceedance in 50 years (975-year return period)	2% Exceedance in 50 years (2,475-year return period)
$S_a(1.0)$ (g)	0.021	0.032	0.049
$S_a(2.0)$ (g)	0.010	0.016	0.025
$S_a(5.0)$ (g)	0.002	0.004	0.006
$S_a(10.0)$ (g)	0.001	0.002	0.003

Seismic Hazard Values for Site Class D	10% Exceedance in 50 years (475-year return period)	5% Exceedance in 50 years (975-year return period)	2% Exceedance in 50 years (2,475-year return period)
$PGA$ (g)	0.039	0.059	0.095
$PGV$ (m/s)	0.040	0.062	0.098
$S_a(0.2)$ (g)	0.066	0.098	0.151
$S_a(0.5)$ (g)	0.054	0.079	0.121
$S_a(1.0)$ (g)	0.033	0.050	0.076
$S_a(2.0)$ (g)	0.016	0.025	0.039
$S_a(5.0)$ (g)	0.003	0.006	0.009
$S_a(10.0)$ (g)	0.001	0.003	0.004

The values given above are for the reference ground conditions for Site Class C and D and must be confirmed or modified on each side of the river during the site-specific seismic site classification to be determined during detail design. The design spectral values will need to be assessed along with the importance category of the bridge structure (defined as “lifeline” by the structural designer and to be confirmed by the owner as per Section 4.4.2 (CHBDC)) and actual structure periods to determine the Seismic Performance Category and level of seismic analysis required during detail design as per Table 4.10 of the CHBDC (2019). Depending on the Seismic Performance Category determined during detail design, a special study may be required.

### 6.3.2.3 Soil Liquefaction

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

In general, the soils at this bridge site consist of interlayered compact to very dense silty sand to sandy silt and stiff to very stiff clayey silt to clayey silt-silt over hard glacial till or very dense silty sand. Based on the compactness and consistency of the soils and the relatively low site-specific PGA, the soils at this site are considered to have a low potential for liquefaction during a seismic event as per Section 6.14.8 of the CHBDC (2019). The potential for liquefaction (especially if looser near surface cohesionless soils are encountered below embankments) will need to be reassessed when more site-specific foundation soil information is available during detail design.

## 6.4 Foundation Types

Based on the proposed multi-span twin structure configuration and subsurface conditions encountered at the site, both shallow and deep foundation options have been considered for support of the new abutments and piers. It is noted that the borehole and sCPT on the west side of the river are located more than 100 m from the proposed bridge footprint and the preliminary recommendations provided herein will be subject to change when more detailed soil information and actual location of the abutments and piers is known.

A summary of the general advantages and disadvantages associated with each option is provided below and a comparison of the alternative foundation options based on advantages, disadvantages, risks, and relative costs is provided in Table 1 following the text of this report.

Shallow foundations “perched” within approach embankments (after sufficient preloading) are considered to be a valid alternative from a geotechnical/foundations perspective based on the available subsurface information, however, the foundation size, geotechnical resistance, and minimum depth of granular pad will need to be reassessed when actual structure loads are known. Given that structural loads are anticipated to be high and deeper unsuitable soils (e.g. organic and/or soft alluvial deposits) are anticipated to be encountered closer to the river on the west side, shallow foundations may not be feasible on the west side during detail design.

Alternatively, driven steel H-piles (friction piles on the east side and both friction and end-bearing piles on the west side) are preferred for the abutments and piers from a geotechnical/foundations perspective as they provide higher geotechnical resistance and lower risk of settlement or significant design changes if variable shallow soil conditions (e.g. deeper peat or soft/loose soil deposits) are encountered during detail design.

### 6.4.1 Shallow Foundations

Strip or spread footings founded on the native stiff to very stiff clayey silt-silt and/or compact to very dense silty sand soils on the east and west side of the river could be considered for support of the piers and abutments. The feasibility of using shallow foundations will need to be reassessed when actual structure loads and footing sizes are known. Based on the existing boreholes, subexcavation up to about 3 m below ground surface (bgs) and below groundwater level is required to remove the existing fills, organic silt / peat, and very loose to loose / firm soils (unsuitable soils). The thickness of the peat / organic silt is anticipated to increase closer to the river on the west side (i.e., near the piers), but this will need to be confirmed during detail design.

Consideration could be given to subexcavating the unsuitable soils and placing engineering fill such that spread footings could be ‘perched’ within the approach embankments or within granular fill pads at the pier locations to increase geotechnical resistances.

The following geotechnical resistances may be used for preliminary design, assuming a 3 m or 10 m wide footing:

Foundation Element	Founding Stratum	Footing Width	Founding <sup>1</sup> Elevation	Factored Geotechnical Resistance	
				Ultimate Limits State, f-ULS	Serviceability Limits State, f-SLS (for 25 mm of settlement) <sup>2</sup>
East Abutment and Surrounding Piers	Stiff to very stiff clayey silt to clayey silt-silt	3 m	215 m	200 kPa	>200 kPa
	3 m thick granular pad on stiff to very stiff clayey silt to clayey silt-silt	3 m	218 m (min. 3 m of granular fill above El. 215 m)	600 kPa	300 kPa
	Stiff to very stiff clayey silt to clayey silt-silt	10 m	215 m	225 kPa	60 kPa
	10 m thick granular pad above stiff to very stiff clayey silt to clayey silt-silt	10 m	225 m (min. 10 m granular pad)	1100 kPa	85 kPa
West Abutment and surrounding Piers	Compact to dense silty sand	3 m	216 m	500 kPa	300 kPa <sup>3</sup>
	3 m thick granular pad on compact to dense silty sand	3 m	219 m (min. 3 m of granular fill above El. 216 m)	750 kPa	400 kPa <sup>3</sup>
	Compact to dense silty sand	10 m	216 m	900 kPa	90 kPa
	10 m thick granular pad above compact to dense silty sand	10 m	226 m (min. 10 m granular pad)	1200 kPa	95 kPa

## Notes:

1. Subexcavation to about 3 m below ground surface (and below groundwater) is required to remove unsuitable soils to a competent founding stratum.
2. Factored SLS does not include settlement due to loading from approach embankment or granular fill pad above existing ground surface which will govern settlement at foundation location. See settlement section 6.7.2 for more details.
3. Reduced to account for clay interlayers encountered within silty sand

The factored ultimate and serviceability geotechnical resistances are dependent on the footing width and founding elevation and as such, the geotechnical resistances must be reviewed if the footing width varies from that specified above or if the founding soils differ from that given in the previous section. If a larger footing width is required due to the anticipated high loads, the f-SLS loads provided above will decrease and a thicker granular pad will be required to effectively distribute the loads to the native founding subgrade. For preliminary design, the granular pad should be at least as thick as the width of the footing.

Resistance to lateral loads / sliding resistance between the new concrete footing and the subgrade should be calculated in accordance with Section 6.10.4 of *CHBDC* (2019), applying the appropriate consequence and degree of site understanding factors as applicable during detailed design.

## 6.4.2 Deep Foundations

### 6.4.2.1 Steel H-Pile Foundations

Driven steel-H-piles founded within the lower silty clay to clayey silt or clayey silt till deposit on the east side and silty sand deposit on the west side are considered feasible for the support of the new abutments or piers. Consideration should be given to “perched” pile caps within the embankment fill or granular pads at pier locations to reduce subexcavation and dewatering requirements, although settlement due to the embankment or raised granular pads will need to be assessed and mitigated. The factored ultimate and serviceability geotechnical axial resistances for driven steel HP 310x110 piles for two different pile lengths (with corresponding pile tip elevations) at the east and west side are provided below for preliminary design purposes.

The following geotechnical resistances may be used for preliminary design:

Foundation Element	Pile Type	Approximate Pile Length <sup>1</sup>	Estimated Pile Tip Elevation	Factored Ultimate Geotechnical Resistance	Factored Serviceability Geotechnical Resistance for 25 mm of Settlement
East Abutment and Surrounding Piers	HP 310x110	30 m	187 m	900	>900
		35 m	182 m	1,100	>1,100
West Abutment and Surrounding Piers	HP 310x110	20 m	199 m	500	>500
		25 m <sup>2</sup>	134 m	1,300	>1,300

Notes:

1. Measured from approximate existing ground surface at closest borehole location.
2. Pile tip founded in “100-blow” silty sand.
3. Resistance values assume single pile and do not take into account pile group efficiency.

The estimated factored ultimate geotechnical resistances provided above are calculated on both shaft and tip resistances, but predominantly shaft (except for the west abutment option founded on end-bearing “100-blow” soil) and assume piles have had sufficient time to “set-up” and allow pore pressures to dissipate after initial driving in order to achieve the design geotechnical resistances. If higher capacities are required, consideration can be given to increasing the size of the piles to HP 360x132 or using larger area pipe piles. For preliminary design



purposes, the geotechnical resistances provided above could be increased by about 15% (assuming predominantly shaft resistance) using larger HP 360x132 piles or 460 mm outer diameter piles.

Considering the anticipated high loads, pile groups at each foundation element are likely required. For preliminary design, driven steel piles spaced at 3 pile diameters (centre-to-centre) can be assumed to act as single piles with no group interaction effects with regards to axial resistance. For piles spaced less than 3 diameters, the total pile axial resistance should be reduced by a group reduction factor ( $R_A$ ) (Reese, 2006) as follows:

Pile Spacing ( $d$ = Pile Diameter)	Pile Axial Resistance Group Reduction Factor ( $R_A$ )
3.0 $d$	1.0
1.5 $d$	0.7
1.25 $d$	0.55

Note: Reduction factors for other pile spacings may be interpolated from the values above.

Pile installation should be carried out in accordance with Ontario Provincial Standard Specification (OPSS). PROV 903 (*Deep Foundations*) as amended by Special Provision 109F57.

In order to optimize the design and reduce the risk of piles not achieving the design geotechnical resistance at the design tip elevation during construction, the design-builder or contractor can consider a combination of the following options:

- Advanced site-specific investigation to achieve a minimum “typical” degree of understanding and confirm or adjust axial geotechnical resistances for design;
- High-strain dynamic testing (PDA) on all piles at end-of-initial drive (EOID) and at a specified number of piles on beginning-of-restrike (BOR) or retap;
- Advanced static pile load test as per ASTM D-1143, and
- Evaluation of strength gain with time (via PDA testing or static pile load testing or both) to ascertain that geotechnical resistance will ultimately be achieved.

The selected design and testing method(s) must consider logistical challenges and potential schedule impacts as part of the detailed design and planned construction, and optimized design and testing methods must be incorporated into SP109F57 and the contract documents.

The subsequent pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile and length of pile; the criteria must therefore be established at the time of construction after the piling equipment is known to ensure that the piles are not overdriven, to avoid possible damage to the piles, and to calibrate with the results of the high-strain dynamic testing or advanced static pile load testing. Alternatively, high-strain dynamic testing could be performed on all piles.

#### **6.4.2.2 Drilled Shafts (Caissons)**

Caissons are considered feasible for supporting the bridge structure abutments and piers on the east and west side of the river. Long caissons (greater than 30 m) are likely required for the east abutment as no consistent thickness of “100-blow” soil was encountered at depth and consideration must be given to the presence of potential gravel pockets / cobbles that may be present within the glacially derived till.

The following geotechnical resistances may be used for preliminary design of the caissons:

Foundation Element	Caisson Diameter	Approximate Caisson Length <sup>1</sup>	Estimated Caisson Base Elevation <sup>1</sup>	Factored Ultimate Geotechnical Resistance	Factored Serviceability Geotechnical Resistance for 25 mm of Settlement
East Abutment and Surrounding Piers	0.9 m	30 m	187 m	1,600 kN	>1,600 kN
	1.5 m	30 m	187 m	3,300 kN	>3,300 kN
West Abutment and Surrounding Piers	0.9 m	25 m	194 m	3,800 kN	>3,800 kN
	1.5 m	25 m	194 m	7,500 kN	>7,500 kN

Notes:

1. Measured from approximate existing ground surface at closest borehole location.
2. Resistance values assume single caisson and do not take into account caisson group efficiency.

For preliminary design, bored caisson piles spaced at 8 pile diameters (centre-to-centre) can be assumed for design purposes to act as single caisson piles, with no group interaction effects with regards to axial resistance. For caissons spaced less than 8 diameters, the total caisson axial resistance should be reduced by a group reduction factor ( $R_A$ ) (Reese, 2006) as follows:

Caisson Spacing (d = Pile Diameter)	Caisson Axial Resistance Group Reduction Factor ( $R_A$ )
9 d	1.0
6 d	0.9
4 d	0.75
3 d	0.7

Note: Reduction factors for other caisson pile spacings may be interpolated from the values above.

If caisson foundations are adopted for support of any of the foundation elements, a temporary or permanent liner is likely required (at least in the upper zone) to support the soils during construction, to reduce disturbance and loss of ground in the water-bearing cohesionless soils and cohesive soils containing silt and sand interlayers. If a permanent liner is used, the design geotechnical resistances provided above may need to be revised to account for the reduced adhesion between the liner material and surrounding soil along the length of the liner compared to the adhesion between concrete and surrounding soil if temporary liners are used. Specialized construction techniques would be required during advancement of the caisson to maintain a sufficient head of water and/or drilling fluid (polymer slurry) within the liner to prevent basal heave and disturbance of water-bearing cohesionless layers/interlayers (along the shaft and at the base). Given that the above drilled shaft capacities have both a shaft friction and end-bearing component, the performance of the drilled shafts in compression will depend to a large degree upon the final cleaning and verification of the condition of the drilled shaft. Following cleaning to remove all loose cuttings, the base should be inspected by a qualified geotechnical engineer using a shaft inspection

device (SID) or given the use of polymer slurry, a shaft quantitative inspection device (SQUID); because of the likelihood of water with entrained sediment or the presence of polymer slurry, it is recommended that SQUID testing be specified on some or all caissons to demonstrate and verify base cleaning. Should the inspection indicate that loosened material is present at the base of the drilled shaft, the base would need to be re-cleaned and re-inspected.

Alternatively, a design based solely on shaft friction may be considered provided the design geotechnical resistances are reduced accordingly and appropriate quality assurance procedures are adopted by the design-builder / contractor. The consistency and characteristics of the drilling slurry (particularly if a bentonite slurry is used) will have an impact on the design geotechnical resistances and this will need to be considered during detail design.

In order to optimize the design, the design-builder or contractor can consider a combination of the following options:

- Advanced site-specific investigation during detail design to confirm or adjust axial geotechnical resistances for design based on the use of a typical or high rather than low degree of understanding;
- Advanced static pile load test as per ASTM D-1143, bi-directional static load ("Osterberg Cell") test (CFEM, 2006), or Statnamic Load Test (CFEM, 2006).

Caisson installation must be in accordance with OPSS.PROV 903 (Deep Foundations) and MTO's recent special provision should be included in the future contract documents to address the requirements for supply and installation of drilled shafts (caissons) including the use of temporary or permanent liners/casings and slurry, the placement of concrete by tremie methods, cleaning and inspection of the shafts and base of the drilled shafts as applicable, and quality control testing.

Non-destructive post-construction testing in selected drilled shafts is recommended to verify the integrity of the concrete given the groundwater conditions, presence of saturated cohesionless soils, and specialized installation methods to counterbalance the hydrostatic pressures.

#### **6.4.2.3 Resistance to Lateral Loads**

The design of piles or caissons subjected to lateral loads should take into account such factors as the relative rigidity of the pile / caisson to the surrounding soil, the fixity condition at the head of the pile / caisson (i.e., at the pile / caisson cap level), the structural capacity of the pile / caisson to withstand bending moments and shear, the soil resistance that can be mobilized, the tolerable lateral deflections at the head of the pile / caisson and group effects. Lateral loading could be resisted fully or partially by the use of battered piles. For vertical piles or caissons, the resistance to lateral loading will have to be derived from the soil in front of the piles.

For design purposes, both the structural and geotechnical resistances should be evaluated to establish the governing case. Lateral pile / caisson analysis for detail design should be carried out using non-linear methods (such as p-y curves) when the pile / caisson group configuration is established as per the CHBDC (2019).

For preliminary design, the resistance to static lateral loading in front of the piles / caissons may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction,  $k_h$  (kPa/m), is based on the following equations (CFEM, 2002 as referenced in CHBDC, 2006):

For non-cohesive soils:

$$k_h = \frac{n_h z}{B}$$

Where  $n_h$  is the constant of subgrade reaction (kPa/m);  
 $z$  is the depth (m); and  
 $B$  is the pile / caisson diameter or width (m).

For cohesive soils:

$$k_h = \frac{67S_u}{B}$$

Where  $S_u$  is the undrained shear strength of the soil (kPa); and  
 $B$  is the pile / caisson diameter or width (m).

Considering the subgrade reaction equations provided above model linear behaviour, they are only considered appropriate where the maximum pile deflections are small (less than 1% of the pile/caisson diameter), where the loading is static (no cycling) and where the pile/caisson material is linear.

The following values of  $n_h$  and  $S_u$  may be assumed in the structural analyses for a single vertical pile or caisson, using the interpreted stratigraphic conditions from the boreholes. The range in the values reflect the variability of the subsurface conditions, the soil properties and groundwater level, and the approximate nature of the linear-elastic subgrade reaction analysis. The groundwater level is assumed to be at ground surface.

Location	Idealized Soil Unit	$n_h$ (kPa/m)	$S_u$ (kPa)
East Abutment	New Granular Fill (Granular 'A' or 'B' Type II)	40,000 – 50,000	-
	Clayey Silt to Clayey Silt-Silt (Stiff to Very Stiff)	-	66 to 198
	Sandy Silt (Dense to Very Dense)	20,000 – 25,000	-
	Clayey Silt-Silt to Sandy Clayey Silt to Clayey Silt (Stiff to Very Stiff)	-	88 to 148
	Clayey Silt-Silt Till (Very Stiff to Hard)	20,000 – 25,000	148 to 209
	Sandy Silt (Very Dense)	20,000 – 25,000	-
West Abutment	New Granular Fill (Granular 'A' or 'B' Type II)	40,000 – 50,000	-
	Sandy Silt to Silty Sand (Very Loose to Compact)	7,500 – 15,000	-
	Clayey Silt-Silt (Stiff to Very Stiff)	-	100
	Silty Sand (Loose to Dense)	7,500 – 15,000	-
	Clayey Silt to Clayey Silt-Silt (Stiff to Very Stiff)	-	88 to 198
	Silty Sand (Very Dense)	20,000 – 25,000	-
	Clayey Silt-Silt (Hard)	-	200

Notes:

1. Although parameters are provided for the full depth of the soil stratigraphy, lateral resistance in the upper 1.5 m should be neglected to account for frost action.
2. Where both  $n_h$  and  $S_u$  parameters are provided, the structural assessment should be completed for both undrained and drained conditions, and the selected design should be based on the more conservative approach.
3. Refer to Tables 2 and 3 for more detailed stratigraphy.

Group action for lateral loading should also be evaluated by reducing the coefficient of horizontal subgrade reaction either in the direction of loading or perpendicular to the direction of loading by relevant group pile / caisson efficiency factors as outlined in Section C6.11.3.4 of the *Commentary to the CHBDC* (2019).

#### **6.4.2.4 Downdrag Loads on Piles / Caissons**

Based on the preliminary design, the east and west approach embankments are to be 9 m to 14 m high with total settlements in the foundation soils estimated to be greater than 100 mm due to the embankment loading (see Section 6.7.2). In addition, granular pads and/or scour protection may be required at the piers that may result in settlement of the foundation soils surrounding the piles / caissons. The downdrag loads must be assessed during detail design and mitigated accordingly (e.g. preloading prior to pile/caisson installation and/or accounting for the additional structural load in the pile/caisson design).

### **6.5 Scour Protection**

The proposed abutments and specifically the piers will be located within the floodplain and may be near or within the erosion hazard limit of the Holland River. Shallow foundations and any pile / caisson caps must be founded below the maximum anticipated depth of scour, erosion, or undermining. Scour / erosion protection must be provided as necessary as per Section 6.10.1.6 of the CHBDC (2019) and will require design liaison between the Foundation Engineer, Hydraulics Engineer and Bridge/Structural Engineer. The scour analysis and protection measures should be designed by the Hydraulics Engineer. During detail design, the Foundation Engineer is to provide soil parameters for the scour analysis and should have the opportunity to review the scour protection measures to check foundations are adequately designed and protected from active toe erosion and projected meandering of the river throughout the lifespan of the bridge.

### **6.6 Frost Protection**

The spread / strip footing(s) and pile / caisson caps should be founded at a minimum depth of 1.5 m below the lowest surrounding final grade, including any distance measured perpendicular to the sloping ground surface to provide adequate protection against frost penetration (as interpreted from OPSD 3090.101 – *Foundation Frost Penetration Depths for Southern Ontario*).

### **6.7 Approach Embankments**

As previously mentioned, the approach embankments to the proposed twin structure bridge are up to 14 m high on the west side and up to 9 m high on the east side of Holland River, relative to the existing ground surface. Accordingly, a 2 m wide mid-height bench along the embankment slopes is required for embankment heights greater than 8 m in height as per OPSD 202.010 (*Slope Flattening*).

For preliminary design, it is assumed that prior to construction of the new approach embankments, all topsoil, peat/organic soil, existing fill materials and any soft/loose surficial alluvial deposits are considered unsuitable foundation soils and will be stripped from the footprint of the new embankments and replaced with suitable granular fill. Based on the borehole information, stripping of unsuitable soil is assumed to extend up to 2 m below existing ground surface at the east and west approach embankments. Additional details regarding stripping, subgrade preparation and embankment construction are provided in Section 6.9.1.

Global stability and settlement analyses were carried out for the proposed preliminary east and west approach embankment configurations using the current borehole and sCPT information. The foundation engineering parameters for the soil types encountered in the boreholes nearest to the east and west approach embankments are summarized in Table 2 and Table 3. For stability and settlement analysis, the groundwater elevation is assumed to be the highest measured water level (i.e., about 0.5 m below existing ground surface) in both Boreholes HRW-4 and HRW-1 for the east and west abutments, respectively.

### 6.7.1 Global Stability

Limit equilibrium global stability analyses were carried out for the proposed approach embankments using the commercially available program Slide2 (version 9.017), developed by Rocscience Inc., employing the Morgenstern-Price method of analysis. For all analysis, the Factors of Safety of numerous potential circular failure surfaces were computed to establish the minimum Factor of Safety. The Factor of Safety is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. The Factor of Safety is equal to the inverse of the product of the consequence factor,  $\Psi$ , and the geotechnical resistance factor,  $\phi_{gu}$  (i.e.,  $FoS = 1/(\Psi \cdot \phi_{gu})$ ). Accordingly, given the limited geotechnical information at the site and high consequence level, minimum target Factors of Safety of 1.6 and 1.8 have been used for the preliminary design of the approach embankment slopes for the temporary (short-term) and permanent (long-term) conditions, respectively, as per Table 6.2 of CHBDC (2019) and MERO (2020). Both total stress and effective stress analyses were carried out at each approach embankment location.

For preliminary design, the embankment side slopes are assumed be inclined at 2 horizontal to 1 vertical (2H:1V) with an overall height of about 9.0 m and 14.0 m of new fill over the native subgrade materials at the east and west approach embankments, respectively. A 2 m wide bench was modelled along the embankment slopes as required by for embankment heights greater than 8 m in height as OPSD 202.010 (*Slope Flattening*)

The global stability analyses indicate that for the short-term (undrained) conditions, the approach embankments at the abutments will have a global Factor of Safety of greater than 1.6, and for the long-term (permanent) conditions, the approach embankments at the abutments will have a global Factor of Safety greater than 1.6. The results of the stability analysis are summarized below and are shown on Figures 2 to 5 following the text of this report.

Location	Relevant Borehole / sCPT	Static Global Stability Limit State	Factor of Safety
West Approach Embankment	HRW-1, sCPT21-HRW-1	Temporary (Undrained) Condition	>1.6
		Permanent (Drained) Condition	>1.6
East Approach Embankment	HRW-4, sCPT21-HRW-4	Temporary (Undrained) Condition	>1.6
		Permanent (Drained) Condition	>1.6

Although the calculated factor of safety for the permanent (drained) condition does meet the target Factor of Safety of 1.8 at this preliminary stage, the global stability is not considered to be concern. When more detailed foundation investigation is completed at the site during detail design (typical or high level of understanding), the resistance factor can be increased and the target Factor of Safety for the permanent (and temporary) conditions can be decreased to meet the calculated factors of safety at this stage accordingly.

### 6.7.2 Settlement

Settlement analyses were carried out for the proposed maximum height of the west and east approach embankments, be near the abutment locations. The thickness of the compressible foundation soils and the height of the approach embankments will vary along the approach embankment alignment, and as such the settlements

along the length of the alignment will similarly vary; however, the settlements estimated from the settlement analysis represent the maximum anticipated value at the approach embankments.

The settlement analyses assume that topsoil, peat/organics, and any surficial deposits containing excessive organic material, or any other deleterious materials have been removed and replaced with suitable granular fill. The settlement analyses were carried out using the commercially available program Settle3 (Version 5.015), developed by Rocscience Inc. The stress distribution calculations used in the settlement analyses were based on Westergaard's (1938) solution.

The sources of total settlement are considered to include the following:

- Immediate settlement of the granular soils (short-term);
- Primary time-dependent consolidation of the cohesive deposits (using Terzaghi's one-dimensional consolidation theory – long-term); and,
- Secondary time dependent (creep) consolidation of the cohesive deposits (long term).

The immediate compression of the non-cohesive deposits were modelled by estimating an elastic modulus of deformation based on the shear wave velocity profiles from the sCPTS and the SPT "N"-values using correlations proposed by Bowles (1984), Kulhawy and Mayne (1990), and Peck et al. (1974), as well engineering judgement from experience with similar soils in this region of Ontario. The modulus of deformation ( $E'$ ) estimated from the sCPTS was adjusted accordingly to account for large strain effects.

The consolidation settlement of the cohesive deposits was assessed using the results of the laboratory consolidation tests near the site, along with the results of the in-situ field vane tests to estimate the stress history and deformation parameters for the cohesive deposits. In addition, the results of the laboratory index tests were employed to further assess deformation parameters (i.e., compression and recompression indices) using empirical correlations proposed in literature by Rendon-Herrero (1980), Bowles (1984), Sowers (1970), Wood and Wroth (1978), and Terzaghi and Peck (1967).

The coefficient of consolidation,  $c_v$  ( $\text{cm}^2/\text{s}$ ), required in the time-rate settlement analysis was estimated using the results of the laboratory consolidation tests and the results were also checked with the dissipation tests from the sCPTs and correlation from the U.S. Navy (1986) with liquid limit assuming normally consolidated or over-consolidated soils, as applicable.

The target settlement performance criteria for design of approach embankments are outlined in MTO's "Embankment Settlement Criteria for Design", dated July 2, 2010. In general, new embankments approaching structural elements such as bridge abutments are to be designed such that total settlement and rate of differential settlement do not exceed 25 mm, over a 20-year period following completion of construction.

A summary of the estimated magnitude of settlement for the west and east approach embankment is presented below, assuming the use of conventional granular fill for construction. The estimated settlements do not account for settlement of the embankment fill itself, which would need to be assessed during detail design.



Location	Relevant Boreholes / sCPTs	Proposed Maximum Embankment Height <sup>1</sup>	Estimated Settlement over a 20-Year Period <sup>2</sup> (mm)	
West Approach Embankment	HRW-1, sCPT21-HRW-1	14 m	$\delta_{Immediate} =$	100 - 140
			$\delta_{Primary} =$	15 - 20
			$\delta_{Secondary} =$	<5
			$\delta_{Total} =$	115 - 165
East Approach Embankment	HRW-4, sCPT21-HRW-4	9 m	$\delta_{Immediate} =$	40 - 60
			$\delta_{Primary} =$	60 - 85
			$\delta_{Secondary} =$	<15
			$\delta_{Total} =$	100 - 160

## Notes:

1. The proposed maximum embankment height is based on centreline profiles of the proposed highway alignment and existing ground surface profiles provided in AECOM's Mainline Profile dated January 2022. Embankment heights are approximate and are relative to original ground surface.
2. The total settlement ( $\delta_{Total}$ ) is defined as the sum of the immediate settlement ( $\delta_{Immediate}$ ) due to elastic compression of the non-cohesive deposits as well as primary ( $\delta_{Primary}$ ) and secondary ( $\delta_{Secondary}$ ) settlements due to time dependent consolidation of the cohesive deposits.

Based on the estimated magnitude of settlement above, settlement mitigation options will be required to meet the settlement performance criterion.

### 6.7.2.1 Mitigation Options

Several settlement mitigation options have been considered to meet the settlement performance criterion and a brief discussion on these alternatives is provided below. Other ground improvement measures such as full subexcavation and replacement, surcharging, wick drains, rammed aggregate piers, deep soil mixing, and dynamic compaction are not considered suitable or cost effective due to the composition and thickness of the deposits and such options are not discussed further for preliminary assessment.

- **Preloading:** Due to the frequent drainage boundaries (i.e. cohesionless seams / layers) observed throughout the cohesive deposits at the site, preloading is expected to be effective in reaching the settlement performance criterion in a relatively short period of time. A settlement instrumentation and monitoring plan would be required during construction to assess when the settlement performance criterion has been achieved.
- **Lightweight Fill (Lightweight Slag or Cellular Concrete):** Various lightweight fill materials are available, from lightweight slag with a unit weight of approximately 14 kN/m<sup>3</sup>, to cellular concrete with a unit weight between 4 and 7 kN/m<sup>3</sup>. However, for the volume of fill required for the new embankments, a preloading period may still be required to achieve the settlement performance criterion, with similar (although lesser) disadvantages compared to preloading using conventional fill materials. Floatation concerns within the floodplain will also need to be considered.

- **Lightweight Fill (Expanded Polystyrene):** The use of expanded polystyrene (EPS) is another alternative that can be considered to significantly reduce the magnitude of consolidation settlement. Where required, EPS can be used to achieve the settlement performance criterion without preloading and therefore, will reduce the length of time for construction. Another advantage with EPS is that it allows for a rapid construction of an embankment and can, therefore, shorten the construction schedule. However, the cost of EPS is higher than other lightweight and granular fills. Floatation concerns within the floodplain will also need to be considered.

Based on the above considerations, preloading is considered the technically preferred alternative to mitigate long-term post-construction settlement at this site.

### 6.7.2.2 Preloading

Based on the estimated coefficient of consolidation ( $c_v$ ) of  $1.34 \times 10^{-2} \text{ cm}^2/\text{s}$ , it is estimated that the following preload periods will be required for each approach embankment area to meet the settlement performance criterion assuming the embankments are constructed of granular fill.

Location	Height of Embankment (m)	Estimated Preload Period <sup>1</sup> (days)
West Approach Embankment	14	< 30
East Approach Embankment	9	60 – 90

Notes:

1. Time for preload to remain in place to reduce future consolidation settlements to less than 25 mm

The design-builder / contractor will need to monitor actual settlements upon completion of the preload period so that the embankment is constructed to the design geometric requirements. Considering the size of the embankment (to accommodate twin structures) and length of the preload period, if this alternative is to be adopted, the magnitude and time-rate of settlement during and after construction of the preload embankment should be assessed by a monitoring program consisting of settlement plates (SPs) and vibrating wire piezometers (VWPs) to confirm the end of the preload period.

As mentioned in Section 6.4.1 and 6.4.2.4, the settlement of the foundation soils due to the approach embankment loading (and any other foundation locations where the grade is to be raised) will need to be considered for design of any spread footings (excess settlement in addition to the f-SLS geotechnical resistance) and/or deep foundations (i.e. associated downdrag forces).

### 6.7.3 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment walls and any associated wingwalls should be designed in accordance with Section 6 of the CHBDC (2019) and will depend on the type and method of placement of the backfill materials, the nature of the soils behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls. The following recommendations are made concerning the design of the abutment walls and wingwalls:

- Free-draining granular fill meeting the specifications of OPSS.PROV 1010 (*Aggregates*) Granular A or Granular B Type II should be used as backfill behind the walls. Longitudinal drains or weep holes should be installed to provide positive drainage of the granular backfill. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS.PROV 501 (*Compacting*). Other aspects of the granular backfill requirements with respect to subdrains and frost taper should be in general accordance with OPSD 3101.150 (*Walls, Abutment, Backfill, Minimum Granular Requirement*), OPSD 3121.150 (*Walls, Retaining, Backfill, Minimum Granular Requirement*), and 3190.100 (*Walls, Retaining and Abutment, Wall Drain*).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with CHBDC (2019) Section 6.12.3 and Figure 6.8. Care must be taken during the compaction operation not to overstress the wall, with limitations required on heavy construction equipment and requirements for the use of hand-operated compaction equipment per OPSS.PROV 501 (*Compacting*). Other surcharge loadings should be accounted for in the design, as required.
- For restrained walls, granular fill should be placed in a zone with the width equal to at least 1.5 m behind the back of the wall in accordance with Figure C6.31(a) of the *Commentary to the CHBDC* (2019). For unrestrained walls, fill should be placed within the wedge-shaped zone defined by a line drawn at flatter than 1 horizontal to 1 vertical (1H:1V) extending up and back from the rear face of the footing or pile cap in accordance with Figure C6.31(b) of the *Commentary to the CHBDC* (2019).

## 6.8 Corrosion Assessment and Protection

Soil corrosivity may affect the concrete or steel elements (e.g. reinforcing steel) of foundations or related structures buried in the soil. The long-term performance and durability of the foundations are directly related to their respective corrosion resistance. Generally, the corrosivity potential to a structure can be estimated from indicators such as the soil resistivity / electrical conductivity, hydrogen ion concentration (pH), and salts (chloride and sulphate) concentrations. The analytical results for the samples submitted for testing are summarized in Section 4.5 and the analytical laboratory test reports are included in Appendix C.

### 6.8.1 Potential for Sulphate Attack

The analytical test results were compared to CSA Standard, CAN/CSA-A23.1-14 Table 3 (*"Additional requirements for concrete subjected to sulphate attack"*) to assess potential sulphate attack on concrete. The sulphate concentrations measured in the tested samples are below the exposure class of S-3 (Moderate). Therefore, based on the sample of soil tested, when the designer is selecting the exposure class for foundations or buried structures, the effects of sulphates may not need to be considered.

### 6.8.2 Potential for Corrosion

The test results indicate a pH of 7.59 to 7.94 and a resistivity of 4,700 to 6,000 ohm-cm. According to the Gravity Pipe Design Guidelines (MTO, 2014), the pH is not considered detrimental to concrete durability. The resistivity indicates that the soil corrosiveness is Low (6000 ohm-cm > R > 4500 ohm-cm), as per Table 3.2 (MTO, 2014). Given that the foundations are located adjacent to and below the highway and will be exposed to de-icing salt, consideration should be given to selection of a "C" type exposure class for any concrete as defined by CSA A23.1 Table 1.

These recommendations are provided as guidance only; the design-builder should take the results of the laboratory testing into consideration for selecting appropriate materials and corrosion susceptibility for design

service of the structure foundations. Ultimately, it is the designer's decision to determine the appropriate exposure class and to ensure that all aspects of CSA A23.1 Section 4.1.1 (Durability Requirements) are satisfied.

## 6.9 Construction Considerations

### 6.9.1 Subgrade Preparation and Approach Embankment Construction

Prior to construction of the new approach embankments, it is recommended that all unsuitable soils such as topsoil, peat/organic soil, and existing surficial fill materials or loose/soft alluvial soils be stripped from the embankment footprint and be replaced with OPSS SSM, Granular A or Granular B. If stripping extends below the groundwater, Granular A or Granular B Type II is preferred to reduce or eliminate dewatering efforts provided the temporary excavation remains stable. Based on the boreholes, stripping up to about 2 m below ground surface is required to remove the unsuitable soils at the approach embankments.

Engineered fill for construction of the new embankments should consist of OPSS.PROV 1010 (*Aggregates*) granular materials (i.e., SSM, Granular A or Granular B). The embankment fill should be placed and compacted in accordance with OPSS.PROV 501 (*Compacting*) and OPSS.PROV 206 (*Grading*). Permanent embankment side slopes should be constructed no steeper than 2 horizontal to 1 vertical (2H:1V) in granular fill.

In accordance with MTO's standard practice, a minimum 2 m wide bench should be provided where embankment slopes are greater than 8 m in height, such that the uninterrupted slope height does not exceed 8 m, consistent with OPSD 202.010 (*Slope Flattening*).

To reduce surface water erosion on the granular embankment side slopes, topsoil and seeding as per OPSS.PROV 804 (*Seed and Cover*) or pegged sod should be carried out as soon as possible after construction of the embankments.

### 6.9.2 Temporary Excavation and Control of Groundwater and Surface Water

During the stripping and removal of the topsoil, peat/organic soil, existing surficial fills and any loose/soft alluvial soils, temporary excavations extending up to 2 m deep (for stripping below approach embankments) and up to 3 m deep (for shallow footings or subexcavation and replacement with granular pad for shallow foundations, if considered) will be required on both the east and west sides of the river.

All temporary excavations must be carried out in accordance with Ontario Regulation 213 of the Ontario Occupational Health and Safety Act for Construction Projects (OHSA), as amended. As per OHSA, the topsoil, peat/organic silt and very loose silty sand to sandy silt (alluvial) soils are classified as Type 4 soils, and the existing compact silty sand fill materials, loose to compact silt and sand deposits and the stiff clayey silt to clayey silt-silt deposits are classified as Type 3 soils. As such, temporary excavations (i.e., those that are open for a relatively short time period where personnel are required to enter) within Type 4 soils should be made with side slopes no steeper than 3H:1V, while those within Type 3 soils should be made with side slopes no steeper than 1H:1V.

Based on results of the borehole investigation, dewatering is likely not required for stripping / subexcavation below embankment footprints provided Granular A or B Type II is used with adequate preload duration of the embankment fill, however, advanced dewatering will likely be required for the abutment (and pier) foundation locations, especially if spread footings are to be considered. Temporary excavations for construction of shallow footings (or pile caps) or for placement and compaction of a granular pad to support "perched" spread footings will require a dry and stable subgrade during construction. It is recommended that the groundwater level be lowered to at least 1 m below the base of the subexcavation level, resulting in temporary groundwater lowering of up to 4

m. Dewatering operations should be in general accordance with OPSS.PROV 517 (*Dewatering*) as referenced in OPSS.PROV 902 (*Excavation and Backfilling – Structures*). Inclusion of a special provision for foundation dewatering will need to be considered in the future contract documents during detail design to address potential instability / base heave of the foundation subgrade, temporary flow diversion or cofferdams and condition survey requirements, especially if temporary protections systems or other temporary works may be impacted by dewatering.

Construction water takings in excess of 50,000 L/day are regulated by the Ministry of the Environment, Conservation and Parks (MECP). Certain takings of groundwater for construction dewatering purposes with a combined total less than 400,000 L/day qualify for self-registration on the MECP's Environmental Activity and Sector Registry (EASR), requiring a "Water Taking Plan" and a "Discharge Plan" (to be developed by the Design-Builder or Contractor). A Category 3 PTTW would be required for water takings in excess of 400,000 L/day. The design/builder or contractor will be responsible for obtaining any required discharge approvals.

Surface water must be directed away from the excavations and foundation subgrade at all times. In particular, the anticipated water level in the Holland River and surrounding floodplain must be properly assessed during construction and diverted / controlled such that the integrity of any abutment / pier footing (or pile cap) subgrade and/or temporary excavation for granular fill pads is maintained. For this reason, it is anticipated that temporary cofferdams or temporary protection systems / flow diversion embankments may be needed by the Design-Builder / Contractor.

The existing dyke / berm located on the east side of Holland River and existing groundwater / surface water collection system and water pumping / dewatering system within the field east of Holland River needs to be investigated further during detail design and possible use during construction to aid in groundwater / surface water control for construction of foundations.

### **6.9.3 Temporary Protection Systems**

Where property limits or environmental constraints (e.g. proximity to the river) do not allow for open cut excavations, temporary protection systems must be designed and constructed in accordance with OPSS.PROV 539 (*Temporary Protection System*) and Special Provision 105S09. Although bridge piers are not proposed to be in the river, cofferdams may also be required depending on the river water level and required flood protection. Conventional sheet pile or soldier pile and lagging temporary protection systems are considered feasible options.

### **6.9.4 Temporary Access Routes**

Raised access roads / platforms to access and construct piers and provide laydown areas will be required for construction and must be carefully designed near / within the river and on the floodplain. As previously noted, the west side of the Holland River is swampy and the ground surface on the east side is near or below the river water level and protected by earth berms / dykes along the riverbank, and challenges accessing and supporting heavy equipment near the river and/or during periods of high groundwater / surface water (Spring season) is anticipated. Stability and settlement of the access roads and slopes should be carried out during detail design when further details are known.

## **6.10 Recommendations for Additional Work**

The preliminary foundation recommendations provided in this report are based on the limited available subsurface information in the two boreholes and two cone penetration tests advanced near the bridge. Additional foundation

investigation and assessment is recommended to be carried out such that the level of confidence for design meets a minimum “typical degree of site and prediction model understanding” for the ultimate bridge configuration.

The additional investigation will need to explore the subsurface soil and groundwater conditions at the location of the actual bridge foundation elements (abutments and pier locations), approach embankments and associated retaining walls, and temporary access roads / laydown areas or protection systems (e.g. near the river). In particular, additional investigation is required closer to the west abutment and piers (west side of Holland River) given that the existing borehole / CPT is located more than 150 m away from the proposed west abutment and more than 250 m from the west bank of the Holland River (i.e. piers). It is anticipated that difficult access (tree clearing and temporary access roads) and specialized drilling equipment (e.g. cranes, barges, and/or portable equipment) will be required to advance additional foundation investigation near the anticipated pier locations on the west side of Holland River. Boreholes and seismic CPTs should be advanced below the anticipated pile tip elevations (preferably beyond 30 m depth on west side and beyond 50 m depth on east side) in order to investigate foundation soils to confirm or update geotechnical resistances for the deep foundation options (e.g. confirm end-bearing stratum). Additional geophysical testing such as Multichannel Analysis of Surface Waves (MASW), Vertical Seismic Profiling (VSP) and/or additional seismic CPTs (with dissipation tests) should be performed on both sides of the river (and in two directions) to further characterize the Site Class (currently the Site Class is different on the east and west side of river based on limited information) and to provide additional soil parameters for detail design. The use of GSC 5<sup>th</sup> Generation or 6<sup>th</sup> Generation seismic hazard maps to define the Site Class should be confirmed for detail design.

The global stability of the approach embankments, any raised granular pads or cofferdams near piers (if applicable), and any retaining walls will need to be checked and the magnitude and time-rate of settlement and any mitigation measures (including estimated preload durations) will need to be reassessed. The interim and ultimate configuration of the bridge approach embankments (and staging of such configurations) will need to be assessed in more detail as the magnitude and time-rate of settlement predictions will depend on the selected configuration. When more details are known on actual loading for the 580 m+ long bridge, the foundation types and bearing resistance values will need to be reassessed and revised as necessary.

The fluctuation of the Holland River water level (and design high water level) needs to be established and details of the apparent protective dyke / berm on the east side of Holland River, and groundwater / surface water collection and active water pumping system in the fields on the east side of Holland River needs to be provided such that foundation and construction risks can be appropriately assessed and mitigated.

Additional foundation investigation and design should meet the general requirements outlined in the latest version of the *Guideline for MTO Foundation Engineering Services*. The existing standpipe piezometers (installed in Boreholes HRW-1 and HRW-4) should be maintained operational to allow for continued monitoring of the groundwater level during detail design and up to construction, at which time the piezometers will need to be decommissioned in accordance with Ontario Regulation 903 (as amended). Additional piezometers / monitoring wells should be installed on both sides of the river, especially on the west side where no current subsurface information is available near the abutment or piers.

## 7.0 CLOSURE

This Foundation Design Report was prepared by Mr. Carter Comish, P.Eng., and reviewed by Mr. Kevin Bentley, P.Eng., a Geotechnical Engineer with WSP Golder and MTO Foundations Designated Contact.

### Golder Associates Ltd.



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CC/KJB/al

[https://golderassociates.sharepoint.com/sites/120387/project files/6 deliverables/foundations/holland river \(i.e. holland river west\)/final/19136074-r-rev0-hr fidr\\_28april2023 - final.docx](https://golderassociates.sharepoint.com/sites/120387/project%20files/6%20deliverables/foundations/holland%20river%20(i.e.%20holland%20river%20west)/final/19136074-r-rev0-hr%20fidr_28april2023-final.docx)



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### ASTM International

- |            |  |
|------------|--|
| ASTM D1586 | Standard Test Method for Standard Penetration Tests and Split Barrel Sampling of Soils.          |
| ASTM D1587 | Standard Practice for Thin-Walled Tube Sampling of Fine-Grained Soils for Geotechnical Purposes. |
| ASTM D2573 | Standard Test Method for Field Vane Shear Test in Saturated Fine-Grained Soils.                  |

**Commercial Software:**

Settle3 (Version 5.015) by Rocscience Inc.

Slide2 (Version 9.017) by Rocscience Inc.

**Ontario Provisional Standard Drawing:**

OPSD 3090.101	Foundation Frost Penetration Depths for Southern Ontario
OPSD 3101.150	Walls, Abutments, Backfill, Minimum Granular Requirements
OPSD 3121.150	Walls, Retaining, Backfill, Minimum Granular Requirements

**Ontario Provincial Standard Specifications (OPSS)**

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.PROV 539	Construction Specification for Temporary Protection Systems
OPSS.PROV 804	Construction Specification for Seed and Cover
OPSS.PROV 903	Construction Specification for Deep Foundations
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material
Special Provision 109F57	Amendment to OPSS.PROV 903

**Ontario Regulations**

Ontario Regulation 213	Construction Projects (as amended)
Ontario Regulation 903	Wells (as amended)

**Ministry of Transportation, Ontario**

MTO Gravity Pipe Design Guidelines, Circular Culverts and Storm Sewers, April 2014.

MTO Foundations Guideline, Embankment Settlement Criteria for Design, July 2010.

Provincial Engineering Memorandum #20201, Material Engineering and Research Office (MERO), March 23, 2020.

Guideline for MTO Foundation Engineering Services, Version 3, dated April 2022.

Table 1: Comparison of Foundation Alternatives – Holland River Bridge

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risk / Consequences
Spread footings founded on native stiff to very stiff clayey silt-silt (east side) or compact silty sand (west side)	■ Marginally feasible at the site	<ul style="list-style-type: none"><li>■ Conventional construction</li><li>■ Relatively lightweight equipment to access wet, swampy areas anticipated on west side of river</li></ul>	<ul style="list-style-type: none"><li>■ Anticipated settlements / consolidation of foundation soils will exceed tolerable limits (25 mm) at abutments and will need to be mitigated (e.g. ground improvement).</li><li>■ Less appealing for piers given large granular pad footprint, risk of less competent soils present at ground surface near river, and proximity to river and/or environmentally sensitive areas</li><li>■ Low geotechnical resistance compared to deep foundations</li><li>■ Subexcavation of 3 m to 4 m of unsuitable soils and below groundwater table. Temporary protection systems / cofferdams and dewatering likely required.</li><li>■ Less competent near surface soils may exist at actual abutment and/or pier locations, especially on west side.</li></ul>	■ Lower cost than deep foundations although additional ground improvement / mitigation costs to address settlement, dewatering and temporary protection systems / cofferdams will need to be considered	<ul style="list-style-type: none"><li>■ High anticipated structure loads may require very large footing widths and reduced geotechnical resistances.</li><li>■ Risk of excess total and differential settlement due to anticipated high foundation loads and variable soil conditions. Settlement mitigation and monitoring required.</li><li>■ High risk of variable soil conditions and increased subexcavation depth of unsuitable soils (e.g. peat and organics) near west side of river may eliminate this option during detail design.</li><li>■ High risk of disturbance to founding soils during construction due to dewatering and subexcavation in saturated cohesionless soils / layers</li></ul>
"Perched" abutment spread footings founded on a compacted granular pad	■ Marginally feasible at the site	<ul style="list-style-type: none"><li>■ Conventional construction</li><li>■ Granular pad can be constructed within approach embankment for abutment locations.</li><li>■ Granular pad could also be considered for pier locations</li><li>■ Relatively lightweight equipment to access wet, swampy areas anticipated on west side of river</li></ul>	<ul style="list-style-type: none"><li>■ Anticipated settlements / consolidation of foundation soils will exceed tolerable limits (25 mm) at abutments and will need to be mitigated (e.g. ground improvement).</li><li>■ Less appealing for piers given large granular pad footprint, risk of less competent soils present at ground surface near river, and proximity to river and/or environmentally sensitive areas</li><li>■ Lower geotechnical resistance compared to deep foundations</li><li>■ Subexcavation of 3 m to 4 m of unsuitable soils and below groundwater table. Temporary protection systems / cofferdams and dewatering likely required.</li><li>■ Less competent near surface soils may exist at actual abutment and/or pier locations, especially on west side.</li></ul>	■ Lower cost than deep foundations although additional ground improvement / mitigation costs to address settlement, dewatering and temporary protection systems / cofferdams will need to be considered.	<ul style="list-style-type: none"><li>■ High anticipated structure loads may require very large footing widths increased granular pad thickness. Logistics of increased granular pad height / footprint will need to be considered.</li><li>■ Risk of excess total and differential settlement due to anticipated high foundation loads and variable soil conditions. Settlement mitigation and monitoring required.</li><li>■ High risk of variable soil conditions and increased subexcavation (and granular fill replacement) depth if thicker unsuitable soils (e.g. peat and organics) encountered near west side of river may eliminate this option during detail design.</li><li>■ High risk of disturbance to founding soils during construction due to dewatering and subexcavation in saturated cohesionless soils / layers</li></ul>
Driven Steel H-piles	■ Feasible for all foundation elements	<ul style="list-style-type: none"><li>■ Conventional construction methods for driven H-pile foundations.</li><li>■ Higher axial resistances compared to shallow footings.</li><li>■ Perched abutments can be considered to reduce dewatering / subexcavation for pile caps.</li><li>■ Likely feasible and preferred if deeper unsuitable soil deposits are encountered within footprint (especially on west side)</li></ul>	<ul style="list-style-type: none"><li>■ Dewatering measures may be required for pile caps if they cannot be perched</li><li>■ Heavy cranes / construction equipment will create access / laydown challenges in wet, swampy area on west side of river.</li><li>■ Relatively long (greater than 30 m) piles will be required on east side of river.</li></ul>	<ul style="list-style-type: none"><li>■ Lower relative cost than drilled shafts (caissons) and may be comparable to spread footings if dewatering and subexcavation of unsuitable soils can be reduced.</li><li>■ Higher cost for access roads / platforms for pile driving equipment compared to shallow foundations.</li></ul>	<ul style="list-style-type: none"><li>■ Variable soil conditions on west side may lead to longer pile lengths and friction pile design similar to east side of river.</li><li>■ Risk of settlement and lower geotechnical resistances for friction pile design.</li><li>■ Settlement will cause potential downdrag loads on piles (reduced capacity) unless mitigation and monitoring provided during construction.</li></ul>
Drilled Shafts (Caissons)	■ Feasible to marginally feasible	<ul style="list-style-type: none"><li>■ Offers higher geotechnical resistance compared to driven steel piles, requiring fewer foundation elements.</li><li>■ Pile/caisson caps can likely be eliminated to reduce dewatering and subexcavation requirements.</li></ul>	<ul style="list-style-type: none"><li>■ Long drilled shafts (in excess of 25 m to 30 m) likely required and may be challenging from constructability perspective.</li><li>■ Temporary or permanent casings will be required, plus special measures such as use of slurry to counterbalance groundwater pressures, reduce risk of blow-out in cohesionless layers/soils, and minimize disturbance.</li></ul>	<ul style="list-style-type: none"><li>■ Higher relative cost than driven piles.</li><li>■ Higher cost for access roads / platforms for caisson equipment compared to shallow foundations.</li></ul>	<ul style="list-style-type: none"><li>■ Generation of soil cuttings and slurry will need to be contained in wetland and near river, and may required special permits / approvals.</li><li>■ Challenges associated with inspection of shaft walls and base may lead to conservative friction design and longer caissons.</li></ul>

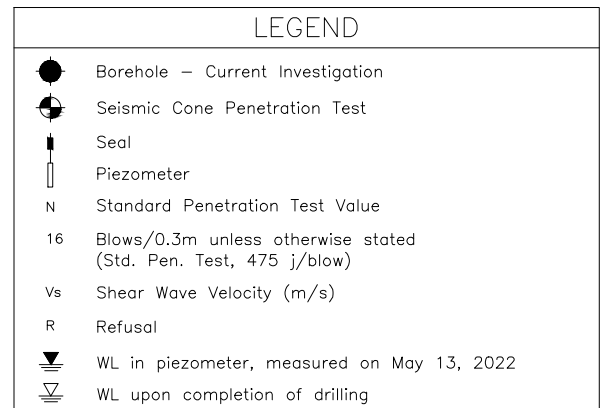
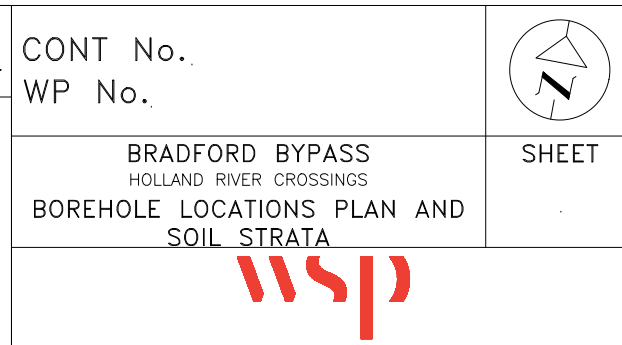
Table 2: Summary of Idealized Soil Model - East Abutment

Soil Unit	Top Depth (m)	Top Elevation (m)	Bottom Depth (m)	Bottom Elevation (m)	Thickness (m)	Unit Weight (kN/m <sup>3</sup> )	E (MPa)	Friction Angle (Degrees)	S <sub>u</sub> (kPa)	P <sub>c</sub> ' (kPa)	e <sub>0</sub>	C <sub>c</sub>	Cr	C <sub>v</sub> (cm <sup>2</sup> /sec)
Silty Sand to Sandy Silt (Compact)	0.0	217.4	1.8	215.6	1.8	20	10	32	--	--	--	--	--	--
Clayey Silt to Clayey Silt-Silt I (Stiff to Very Stiff)	1.8	215.6	6.4	211.0	4.6	20	80	30	66 to 198	300 to 900	0.61	0.1	0.01	0.0134
Clayey Silt to Clayey Silt-Silt II (Very Stiff)	6.4	211.0	11.7	205.7	5.3	20	80	30	198 to 110	900 to 500	0.61	0.1	0.01	0.0134
Sandy Silt (Dense to Very Dense)	11.7	205.7	16.3	201.1	4.6	20	90	34	--	--	--	--	--	--
Clayey Silt-Silt to Sandy Clayey Silt to Clayey Silt (Stiff to Very Stiff)	16.3	201.1	35.0	182.4	18.7	19	80	33	88 to 148	400 to 675	0.61	0.1	0.01	0.0134
Clayey Silt-Silt Till (Very Stiff to Hard)	35.0	182.4	47.2	170.2	12.2	19	200	35	148 to 209	675 to 950	0.61	0.1	0.01	0.0134
Sandy Silt (Very Dense)	47.2	170.2	49.2	168.2	2.0	19	200	35	--	--	--	--	--	--

Table 3: Summary of Idealized Soil Model - West Abutment

Soil Unit	Top Depth (m)	Top Elevation (m)	Bottom Depth (m)	Bottom Elevation (m)	Thickness (m)	Unit Weight (kN/m <sup>3</sup> )	E (MPa)	Friction Angle (Degrees)	S <sub>u</sub> (kPa)	P <sub>c</sub> ' (kPa)	e <sub>0</sub>	C <sub>c</sub>	Cr
Sandy Silt to Silty Sand (Very loose to Compact)	0.0	219.0	2.2	216.8	2.2	20	10	28	--	--	--	--	--
Clayey Silt-Silt (Stiff to Very Stiff)	2.2	216.8	3.0	216.0	0.8	20	21	32	100	--	--	--	--
Silty Sand (Loose to Dense)	3.0	216.0	15.2	203.8	12.2	20	150	30	--	--	--	--	--
Clayey Silt to Clayey Silt-Silt (Stiff to Very Stiff)	15.2	203.8	19.0	200.0	3.8	19	140	30	198 to 88	900 to 400	0.61	0.1	0.01
	19.0	200.0	21.3	197.7	2.3	19	140	30	88 to 100	400 to 454	0.61	0.1	0.01
Silty Sand (Very Dense)	21.3	197.7	33.6	185.4	12.3	21	200	35	--	--	--	--	--
Clayey Silt-Silt (Hard)	33.6	185.4	39.9	179.1	6.3	21	200	35	200	--	--	--	--

## Drawings



BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
HRW-1	219.0	4887870.6	300852.7
HRW-4	217.4	4888043.9	301230.5
SCPT21-HRW-1	219.0	4887864.5	300859.7
SCPT21-HRW-4	217.4	4888040.9	301224.1

**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.

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

## REFERENCE

Base plans provided in digital format by Aecom, drawing file no. X-Base\_Bradford Bypass.dwg and BRADFORD BY-PASS OG\_Combined.xml, received January 11, 2022.

Vertical alignment provided in digital format by Aecom, drawing file no. ACAD-X-60636190-C-DES-BBP Mainline Align and Profile.dwg, received September 9, 2022.

Horizontal alignment provided in digital format by Aecom, drawing file no. X-60636190-C-DES-All Alignments.xml, received September 14, 2022.

Design plan provided in digital format by Aecom, drawing file no. X-60636190-C-DES-BBP Overall Plan.dwg, received September 14, 2022.



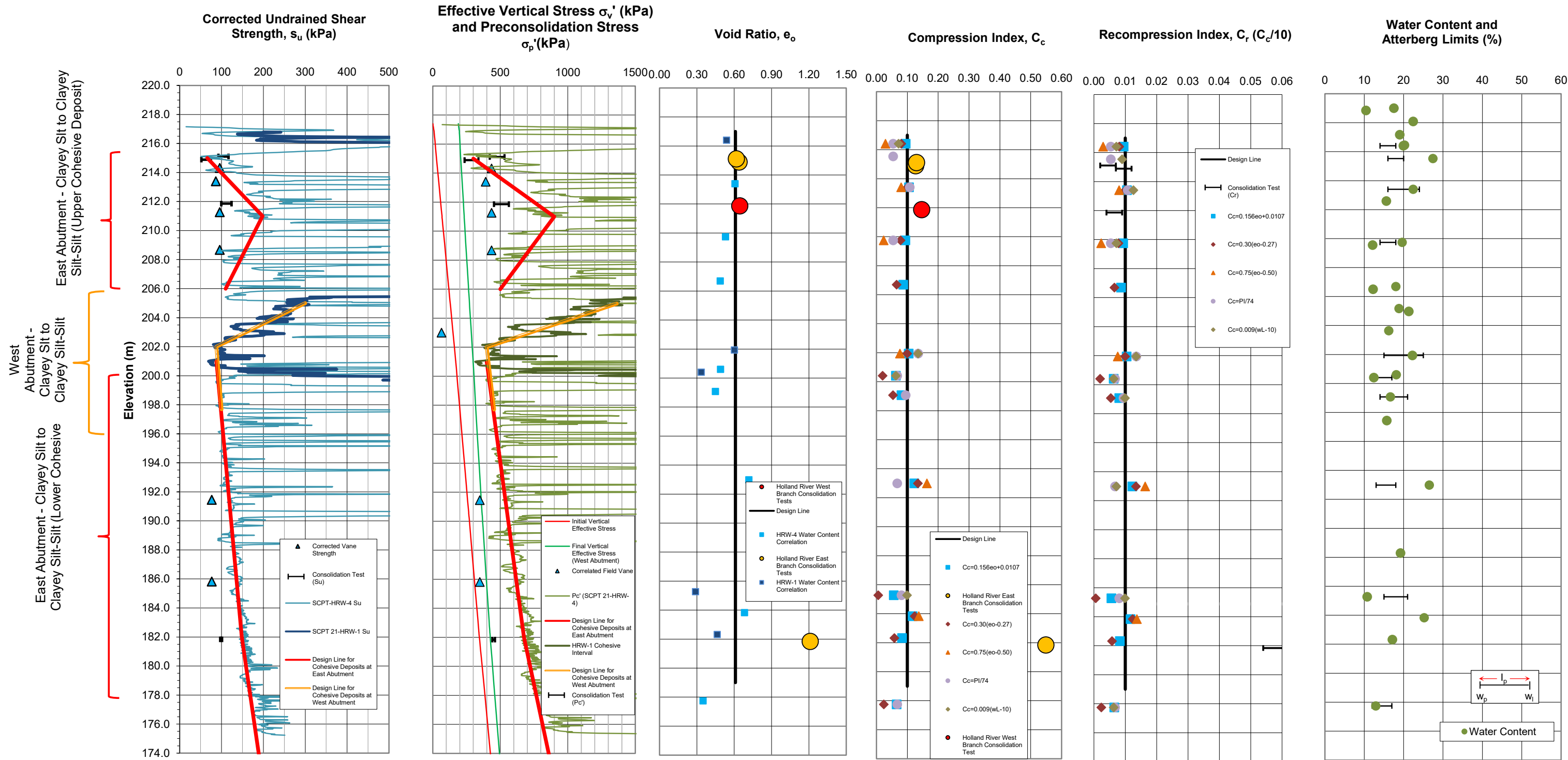
NO.		DATE		BY		REVISION	
Geocres No. 31D-809							
HWY. . . . .			PROJECT NO. 19136074			DIST. . . . .	
SUBM'D. MA		CHKD. MA		DATE: 04/27/2023		SITE: . . . . .	
DRAWN: DD/SA		CHKD. CC		APPD. KJB		DWG. 1	

## Figures



SUMMARY PLOT OF ENGINEERING PARAMETERS FOR  
COHESIVE DEPOSITS  
HOLLAND RIVER CROSSING

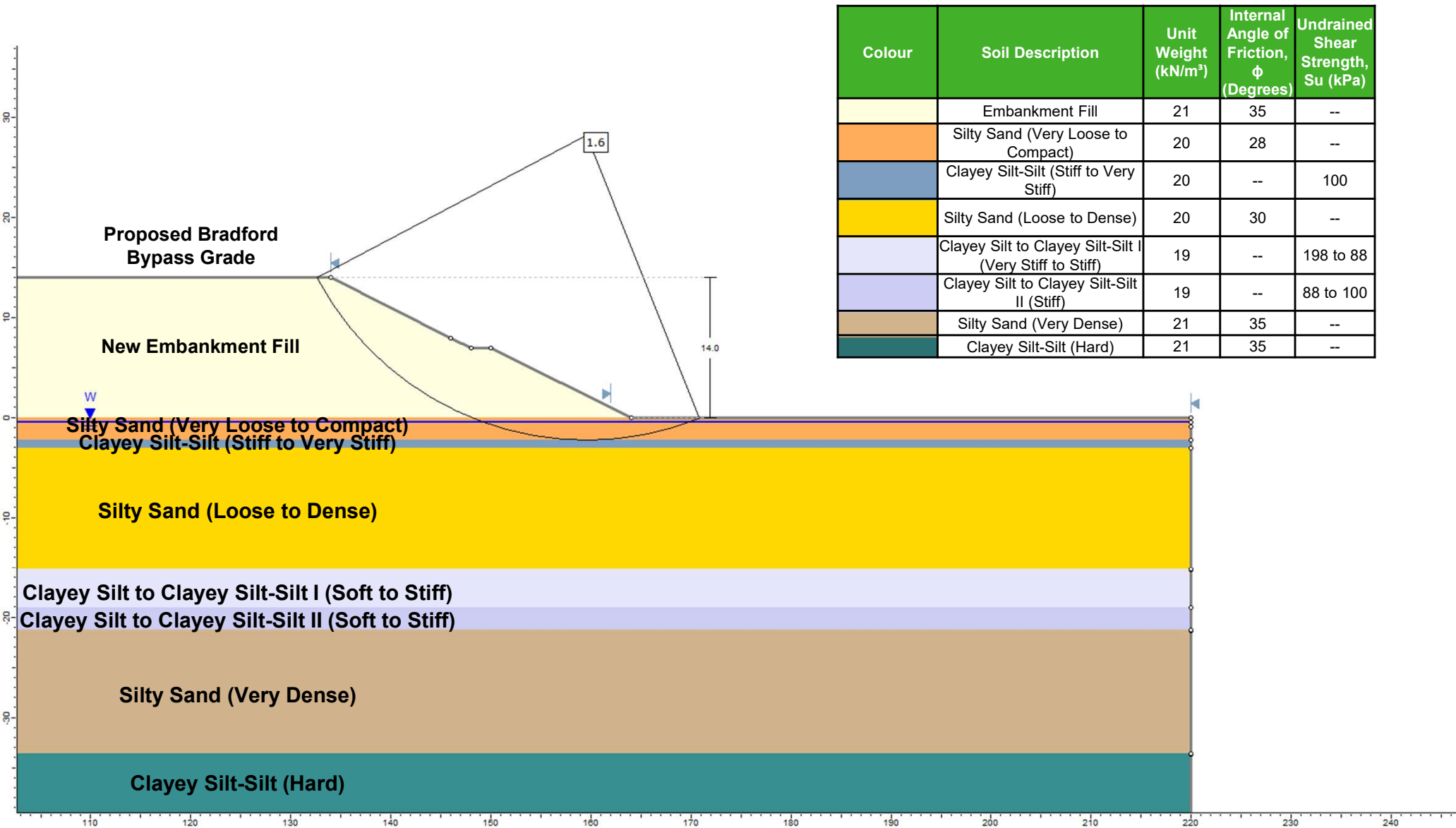
FIGURE 1





# Bradford Bypass– Holland River (West Abutment) Stability Analysis Results (Undrained Condition)

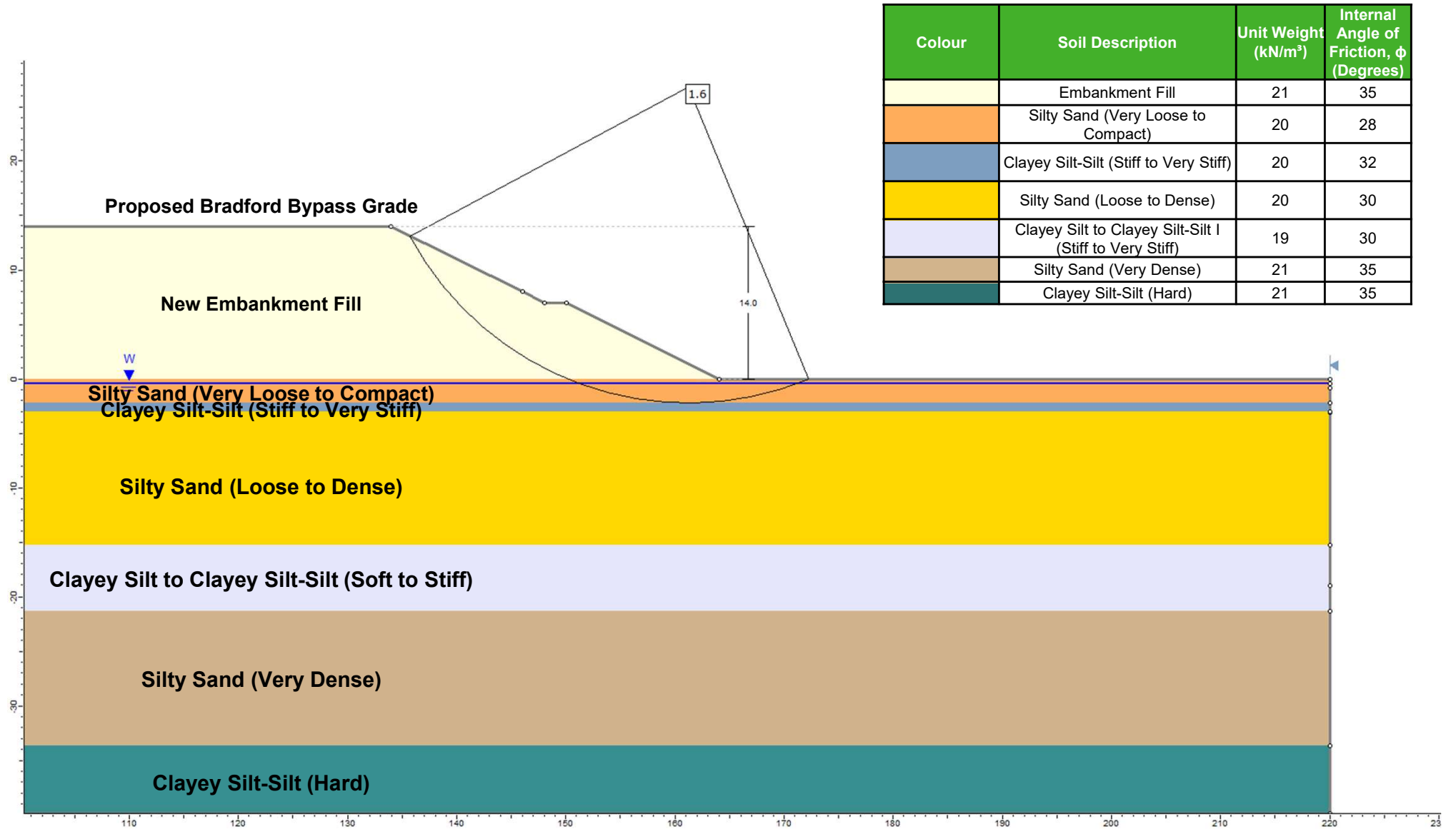
**Figure 2**





# Bradford Bypass– Holland River (West Abutment) Stability Analysis Results (Drained Condition)

**Figure 3**

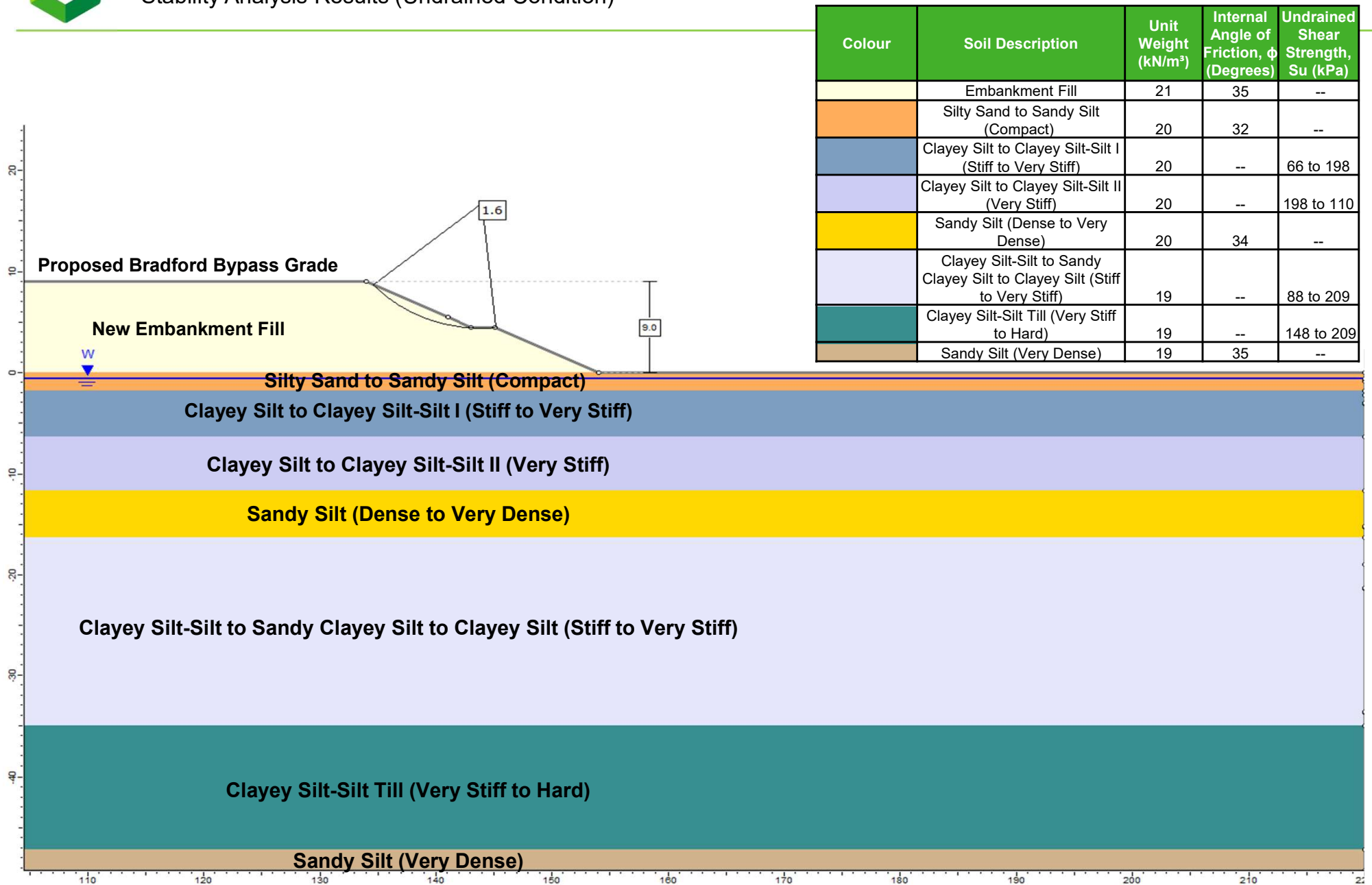




## Bradford Bypass– Holland River (East Abutment)

### Stability Analysis Results (Undrained Condition)

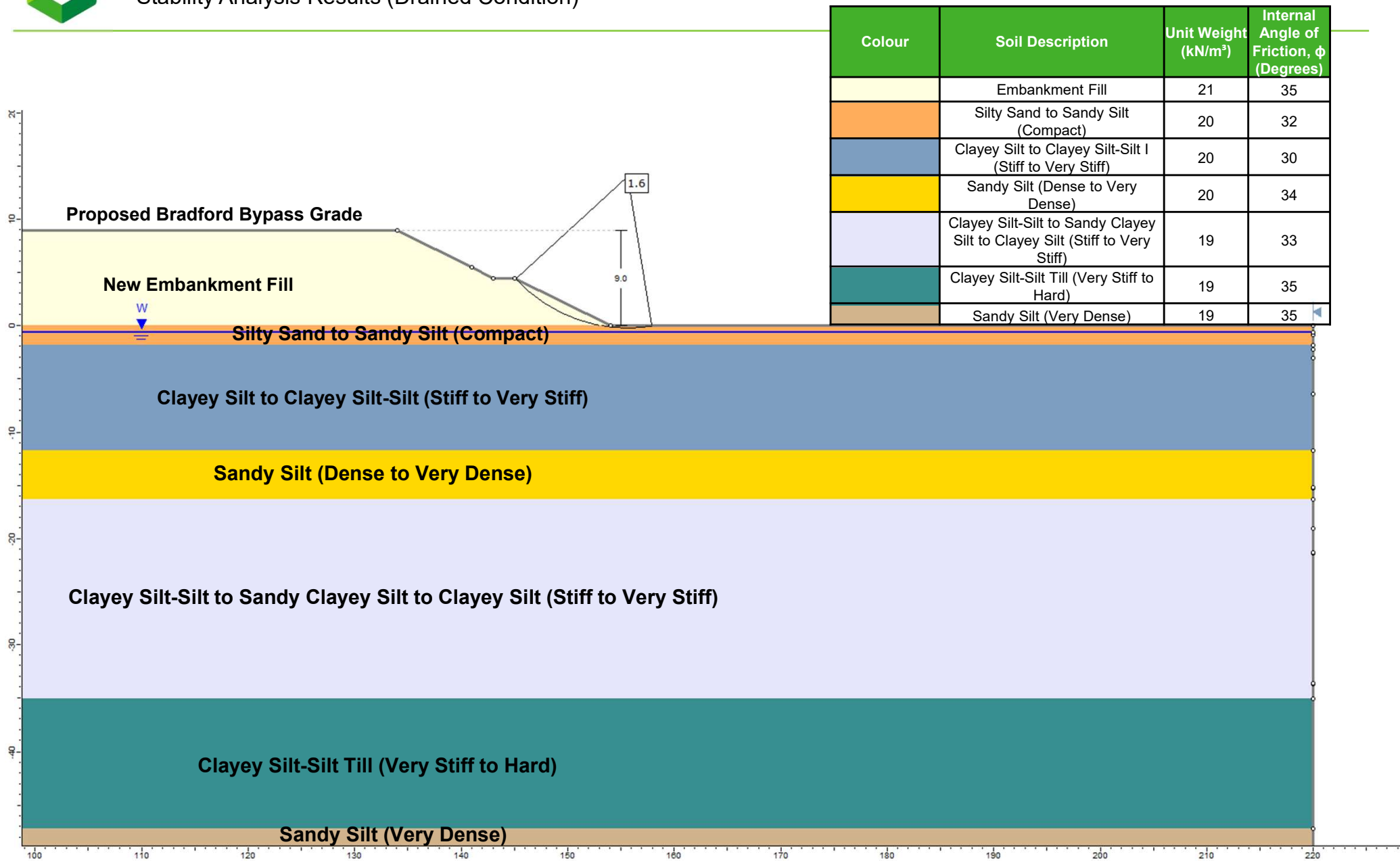
**Figure 4**





# Bradford Bypass– Holland River (East Abutment) Stability Analysis Results (Drained Condition)

**Figure 5**



APPENDIX A

# Borehole Records and SCPT Results

PROJECT	19136074	RECORD OF BOREHOLE	No. HRW-1	Sheet 1 of 5	METRIC
G.W.P.	Assignment No 2019-E-0048	LOCATION	N 4887870.6; E 300852.7 NAD83 / MTM Zone 10 (LAT. 44.13082; LONG. -79.549328)	ORIGINATED BY	AM
DIST	Central HWY BBP	BOREHOLE TYPE	210 mm Hollow Stem Auger; Mud Rotary	COMPILED BY	MA/MTI
DATUM	CGVD28 Surface Elevation:219.0 m	DATE	Nov 11, 2021 - Nov 15, 2021	CHECKED BY	KJB

SOIL PROFILE			SAMPLES			GROUNDWATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					WATER CONTENT (%)			UNIT WEIGHT Y kN/m³	GR SA SI CL				REMARKS	
ELEV. ----- DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)					PL W <sub>p</sub>	NMC W	LL W <sub>L</sub>							
								Field Vane Remoulded Pocket Pen Quick Triaxial Unconfined														
							20	40	60	80	100											
0.0	SANDY SILT (ML), trace clay, trace organic includes roots, (TOPSOIL)		1A												NP							
218.8 0.2	Loose Brown Moist		1B	SS	11																	
218.2	SILTY SAND (SM) to SAND (SP), trace gravel (FILL)		2A																			
0.8	Compact to loose Brown		2B												NP							
218.1 0.9	Moist to wet PEAT (PT), sandy Very loose Blackish brown Wet		2C	SS	4																	
	SILTY SAND to SANDY SILT (SM-ML), contains clay seams Very loose to loose Brown Wet		3	SS	1											NP						
216.8																						
2.2	CLAYEY SILT-SILT (CL-ML), some sand Stiff to Very Stiff Brown Moist		4	SS	15													0	14	66	20	
216.0																						
3.0	SILTY SAND (SM), trace clay, trace gravel Loose to dense Brown to grey Wet		5	SS	8																	
	- 3.9 m: 75mm clay layer (Elev. 215.1 m)		6	SS	17																	
			7	SS	23																	
			8	SS	23													1	82	15	2	
	- 7.9 m: silt seams encountered (Elev. 211.1 m).		9	SS	27																	
	- 9.1 m: tricone grinding noted																					
			10	SS	43																	
	- 9.8 m: gravel seam / layer at tip of spoon																					

Continued on Next Page

+3, x3 : Numbers refer to Sensitivity o3% STRAIN AT FAILURE



PROJECT	19136074	RECORD OF BOREHOLE	No. HRW-1	Sheet 2 of 5	METRIC
G.W.P.	Assignment No 2019-E-0048	LOCATION	N 4887870.6; E 300852.7 NAD83 / MTM Zone 10 (LAT. 44.13082; LONG. -79.549328)	ORIGINATED BY	AM
DIST	Central HWY BBP	BOREHOLE TYPE	210 mm Hollow Stem Auger; Mud Rotary	COMPILED BY	MA/MTI
DATUM	CGVD28 Surface Elevation:219.0 m	DATE	Nov 11, 2021 - Nov 15, 2021	CHECKED BY	KJB

SOIL PROFILE			SAMPLES			GROUNDWATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					WATER CONTENT (%)			UNIT WEIGHT  Y  kN/m³					REMARKS
ELEV. ----- DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)					PL	NMC	LL						
								Field Vane Remoulded Pocket Pen Quick Triaxial Unconfined	20	40	60	80	100	W <sub>p</sub>	W		W <sub>i</sub>				
														NP Nonplastic			----- ----- -----				
	SILTY SAND (SM), trace clay, trace gravel Loose to dense Brown to grey Wet - 10.0 to 12.0 m: gravelly silty sand encountered. Difficulties advancing tricone.						209														
			11	SS	31		208							O				32	56	11	1
							207														
			12	SS	24		206							O							
	- 13.7 to 14.3 m: no sample recovered						205							O							
			13	SS	19		204														
203.8							203														
15.2	CLAYEY SILT (CL) to CLAYEY SILT-SILT (CL-ML), trace to some sand, trace to some gravel Soft to stiff Moist Grey		14	SS	3		202														
	- 16.8 to 17.4 m: no sample recovered in shelby tube. Obtained disturbed sample with split spoon. Contains sand seams/ layers		15	TO			201														
	- 18.3 m: contains sand layers/seams		16	SS	9		200														

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+<sup>3</sup>, x<sup>3</sup> : Numbers refer to Sensitivity o<sup>30</sup>% STRAIN AT FAILURE

PROJECT	19136074	RECORD OF BOREHOLE	No. HRW-1	Sheet 3 of 5	METRIC
G.W.P.	Assignment No 2019-E-0048	LOCATION	N 4887870.6; E 300852.7 NAD83 / MTM Zone 10 (LAT. 44.13082; LONG. -79.549328)	ORIGINATED BY	AM
DIST	Central HWY BBP	BOREHOLE TYPE	210 mm Hollow Stem Auger; Mud Rotary	COMPILED BY	MA/MTI
DATUM	CGVD28 Surface Elevation:219.0 m	DATE	Nov 11, 2021 - Nov 15, 2021	CHECKED BY	KJB

SOIL PROFILE			SAMPLES			GROUNDWATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					WATER CONTENT (%)			UNIT WEIGHT					REMARKS
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)					PL W <sub>p</sub>	NMC W	LL W <sub>L</sub>		GR	SA	SI	CL	
								Field Vane Remoulded Pocket Pen Quick Triaxial Unconfined													
								NP Nonplastic													
									20	40	60	80	100	20	40	60					
21.3	CLAYEY SILT (CL) to CLAYEY SILT-SILT (CL-ML), trace to some sand, trace to some gravel Soft to stiff Moist Grey						199														
197.7	SILTY SAND (SM), trace clay, trace gravel Very dense Grey Wet		17	SS	106		198														
							197														
							196														
							195														
			18	SS	121/0.26		194														
							193														
							192														
			19	SS	101/0.23		191														
							190														

Continued on Next Page

+<sup>3</sup>, x<sup>3</sup> : Numbers refer to Sensitivity o<sup>3</sup>% STRAIN AT FAILURE

PROJECT	19136074	RECORD OF BOREHOLE	No. HRW-1	Sheet 4 of 5	METRIC
G.W.P.	Assignment No 2019-E-0048	LOCATION	N 4887870.6; E 300852.7 NAD83 / MTM Zone 10 (LAT. 44.13082; LONG. -79.549328)	ORIGINATED BY	AM
DIST	Central HWY BBP	BOREHOLE TYPE	210 mm Hollow Stem Auger; Mud Rotary	COMPILED BY	MA/MTI
DATUM	CGVD28 Surface Elevation:219.0 m	DATE	Nov 11, 2021 - Nov 15, 2021	CHECKED BY	KJB

SOIL PROFILE			SAMPLES			GROUNDWATER CONDITIONS	ELEVATION SCALE <div><div><div>○</div><div>●</div><div>⊗</div><div>⊠</div><div>×</div></div><div>Field Vane Remoulded Pocket Pen Quick Triaxial Unconfined</div></div>	DYNAMIC CONE PENETRATION RESISTANCE PLOT					WATER CONTENT (%)			UNIT WEIGHT  Y  kN/m³	GR	SA	SI	CL	REMARKS
ELEV. ----- DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)					PL	NMC	LL						
								20	40	60	80	100	W <sub>p</sub>	W	W <sub>i</sub>						
													NP Nonplastic								
	SILTY SAND (SM), trace clay, trace gravel Very dense Grey Wet						189														
			20	SS	100							○			0	60	36	4			
							188														
							187														
							186														
185.4 33.6	- 33.5 m: gravelly silty sand layer encountered (Elev. 185.5 m). CLAYEY SILT-SILT (CL-ML), trace sand, trace gravel Hard Grey Moist		21	SS	100/0.26		185					○	○		1	16	73	10			
							184														
							183														
			22	SS	100/0.05							○									
							182														
							181														
							180														

Continued on Next Page

+<sup>3</sup>, x<sup>3</sup> : Numbers refer to Sensitivity o<sup>3</sup>% STRAIN AT FAILURE

PROJECT	19136074	RECORD OF BOREHOLE	No. HRW-1	Sheet 5 of 5	METRIC
G.W.P.	Assignment No 2019-E-0048	LOCATION	N 4887870.6; E 300852.7 NAD83 / MTM Zone 10 (LAT. 44.13082; LONG. -79.549328)	ORIGINATED BY	AM
DIST	Central HWY BBP	BOREHOLE TYPE	210 mm Hollow Stem Auger; Mud Rotary	COMPILED BY	MA/MTI
DATUM	CGVD28 Surface Elevation:219.0 m	DATE	Nov 11, 2021 - Nov 15, 2021	CHECKED BY	KJB

SOIL PROFILE			SAMPLES			GROUNDWATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					WATER CONTENT (%)			UNIT WEIGHT					REMARKS
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)					PL W <sub>p</sub>	NMC W	LL W <sub>L</sub>		GR	SA	SI	CL	
								Field Vane Remoulded Pocket Pen Quick Triaxial Unconfined													
								NP Nonplastic													
									20	40	60	80	100	20	40	60					
39.9	CLAYEY SILT-SILT (CL-ML), trace sand, trace gravel Hard Grey Moist		23	SS	100/0.13																
179.1	End of Borehole						179														
	Notes: 1. Hollow stem augers to 2.4 m (Elev. 216.6) and then switched to mud rotary. 2. Water level measured at a depth of 1.98 m after completion of drilling. 3. Water level in piezometer measured as follows: Depth(m) El. (m) Date 0.44 218.6 02/07/22 0.67 218.3 05/013/22						178														
							177														
							176														
							175														
							174														
							173														
							172														
							171														

+<sup>3</sup>, x<sup>3</sup> : Numbers refer to Sensitivity o<sup>3</sup>% STRAIN AT FAILURE

METRIC

CHECKED BY            KJB

SOIL PROFILE			SAMPLES			GROUNDWATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					WATER CONTENT (%)			UNIT WEIGHT  Y kN/m³					REMARKS
ELEV. ----- DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	N° VALUES			SHEAR STRENGTH (kPa)					PL W <sub>p</sub>	NMC W	LL W <sub>L</sub>		GR	SA	SI	CL	
								Field Vane Remoulded Pocket Pen Quick Triaxial Unconfined	20	40	60	80	100	NP Nonplastic							
0.0	TOPSOIL																				
217.2 0.2	SILTY SAND (SM), trace clay (FILL) Compact Brown Dry		1	SS	12																
216.7 0.7	ORGANIC SILT (OL) Loose Brown Moist		2A																		
216.4 0.9			2B	SS	10																
215.9 1.4	SANDY SILT (ML), trace clay Compact Brown Wet		3A																		
215.7 1.7			3B	SS	2																
215.5 1.8	SANDY PEAT (PT)		3C																		
	SILTY SAND (SM) to SANDY SILT (ML) Very loose Brown Wet		4	TO																	
	CLAYEY SILT (CL) to CLAYEY SILT-SILT (CL-ML), trace sand Firm to very stiff Brown Moist to wet																				
	- 3.3 m: 75 mm thick sandy layer encountered (Elev. 214.1 m).		5	SS	11																
			6	SS	7																
			7	SS	12																
			8	TO																	
			9	SS	18																
			10	SS	10																
			11	SS	11																
										</											

<sup>+</sup>, x<sup>3</sup> : Numbers refer to Sensitivity    o<sup>3%</sup> STRAIN AT FAILURE

PROJECT	19136074	RECORD OF BOREHOLE	No. HRW-4	Sheet 2 of 5	METRIC
G.W.P.	Assignment No 2019-E-0048	LOCATION	N 4888043.9; E 301230.5 NAD83 / MTM Zone 10 (LAT. 44.132381; LONG. -79.544608)	ORIGINATED BY	AM
DIST	Central HWY BBP	BOREHOLE TYPE	210 mm Hollow Stem Auger; Mud Rotary	COMPILED BY	MA/MTI
DATUM	CGVD28 Surface Elevation:217.4 m	DATE	Oct 04, 2021 - Oct 08, 2021	CHECKED BY	KJB

SOIL PROFILE			SAMPLES			GROUNDWATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					WATER CONTENT (%)			UNIT WEIGHT  Y  kN/m³	GR SA SI CL				REMARKS
ELEV. ----- DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	N* VALUES			SHEAR STRENGTH (kPa)					PL	NMC	LL						
								Field Vane Remoulded Pocket Pen Quick Triaxial Unconfined	20	40	60	80	100	W <sub>p</sub>	W		W <sub>l</sub>				
														NP Nonplastic					----- ----- -----		
	CLAYEY SILT (CL) to CLAYEY SILT-SILT (CL-ML), trace sand Firm to very stiff Brown Moist to wet		12	SS	14		206														
205.6																					
11.7	SANDY SILT (ML), trace clay Dense to very dense Brown Moist to wet		13	SS	34		205										0	21	70	9	
							204														
			14	SS	35		203														
							202										0	23	69	8	
201.0							201														
16.3	Sandy CLAYEY SILT (CL) to CLAYEY SILT (CL) Very stiff Grey Moist to wet		16	SS	17		200														
199.5							199										0	1	71	28	
17.8	CLAYEY SILT (CL) to CLAYEY SILT-SILT (CL-ML), trace to some sand Stiff Grey Wet		17	SS	12		198														
							197														

Continued on Next Page

+<sup>3</sup>, x<sup>3</sup> : Numbers refer to Sensitivity o<sup>30</sup>% STRAIN AT FAILURE

PROJECT	19136074	RECORD OF BOREHOLE	No. HRW-4	Sheet 3 of 5	METRIC
G.W.P.	Assignment No 2019-E-0048	LOCATION	N 4888043.9; E 301230.5 NAD83 / MTM Zone 10 (LAT. 44.132381; LONG. -79.544608)	ORIGINATED BY	AM
DIST	Central HWY BBP	BOREHOLE TYPE	210 mm Hollow Stem Auger; Mud Rotary	COMPILED BY	MA/MTI
DATUM	CGVD28 Surface Elevation:217.4 m	DATE	Oct 04, 2021 - Oct 08, 2021	CHECKED BY	KJB

SOIL PROFILE			SAMPLES			GROUNDWATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					WATER CONTENT (%)			UNIT WEIGHT Y kN/m <sup>3</sup>	GR	SA	SI	CL	REMARKS
ELEV. ----- DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)					PL W <sub>p</sub>	NMC W	LL W <sub>L</sub>						
								Field Vane Remoulded Pocket Pen Quick Triaxial Unconfined													
								20 40 60 80 100					20 40 60								
	CLAYEY SILT (CL) to CLAYEY SILT-SILT (CL-ML), trace to some sand Stiff Grey Wet - 21.5 m: 75 mm thick wet silt layer encountered		18	SS	13		195														
							194														
							193														
			19	SS	8								H	O			0	14	60	26	
			20	TO			192														
							191														
							190														
			21	SS	13																
							189														
							188														
							187														
			22	SS	11																
							186														

Continued on Next Page

+<sup>3</sup>, x<sup>3</sup> : Numbers refer to Sensitivity o<sup>30</sup>% STRAIN AT FAILURE



PROJECT	19136074	<b>RECORD OF BOREHOLE</b>		<b>No. HRW-4</b>	Sheet 4 of 5	<b>METRIC</b>
G.W.P.	Assignment No 2019-E-0048	LOCATION	N 4888043.9; E 301230.5 NAD83 / MTM Zone 10 (LAT. 44.132381; LONG. -79.544608)			ORIGINATED BY AM
DIST	Central HWY BBP	BOREHOLE TYPE	210 mm Hollow Stem Auger; Mud Rotary			COMPILED BY MA/MTI
DATUM	CGVD28 Surface Elevation:217.4 m	DATE	Oct 04, 2021 - Oct 08, 2021			CHECKED BY KJB

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<sup>+</sup>, x<sup>3</sup> : Numbers refer to Sensitivity    o<sup>3</sup>% STRAIN AT FAILURE

PROJECT	19136074		RECORD OF BOREHOLE		No. HRW-4	Sheet 5 of 5	METRIC
G.W.P.	Assignment No 2019-E-0048		LOCATION	N 4888043.9; E 301230.5 NAD83 / MTM Zone 10 (LAT. 44.132381; LONG. -79.544608)			ORIGINATED BY AM
DIST	Central	HWY BBP	BOREHOLE TYPE	210 mm Hollow Stem Auger; Mud Rotary			COMPILED BY MA/MTI
DATUM	CGVD28 Surface Elevation:217.4 m		DATE	Oct 04, 2021 - Oct 08, 2021			CHECKED BY KJB

SOIL PROFILE			SAMPLES			GROUNDWATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					WATER CONTENT (%)			UNIT WEIGHT					REMARKS
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)					PL W <sub>p</sub>	NMC W	LL W <sub>L</sub>		GR	SA	SI	CL	
								Field Vane Remoulded Pocket Pen Quick Triaxial Unconfined													
								20	40	60	80	100	20	40	60						
	CLAYEY SILT-SILT (CL-ML), some sand to sandy, trace to some gravel, (TILL) Very stiff to hard Grey Moist to wet		26	SS	100		174														
							173														
							172														
			27	SS	15		171														
170.1							170														
47.2	Sandy SILT (ML), some gravel to gravelly, trace to some clay Very dense Grey Wet - 48.0 to 48.8 m: tricone grinding noted						169														
168.1			28	SS	103								○			NP	20	20	50	10	
49.2	End of Borehole Notes: 1. Water level measured at a depth of 1.67 m (Elev. 215.7) prior to introducing water for mud rotary. 2. Water level measured at a depth of 0.55 m (Elev. 216.9) on May 13, 2022.						168														
							167														
							166														
							165														
							164														

+<sup>3</sup>, x<sup>3</sup> : Numbers refer to Sensitivity    σ<sup>3</sup>% STRAIN AT FAILURE

# PRESENTATION OF SITE INVESTIGATION RESULTS

## Bradford Bypass

*Prepared for:*

Golder Associates

ConeTec Job No: 21-05-23424

Project Start Date: 20-Dec-2021

Project End Date: 22-Dec-2021

Report Date: 07-Jan-2022



*Prepared by:*

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## Introduction

The enclosed report presents the results of the site investigation program conducted by ConeTec Investigations Ltd. for Golder Associates along Holland River ~~East Branch~~<sup>1</sup> in Bradford, ON. The program consisted of three seismic cone penetration tests (SCPTu). Please note that this report, which also includes all accompanying data, are subject to the 3<sup>rd</sup> Party Disclaimer and Client Disclaimer that follow in the 'Limitations' section of this report.

## Project Information

Project	
Client	Golder Associates
Project	Bradford Bypass
ConeTec project number	21-05-23424

An aerial overview from Google Earth including the CPTu test locations is presented below.



<sup>1</sup> For clarity, both Holland River and Holland River East Branch sites were investigated as shown on the above figure (KJB, WSP Golder)



Rig Description	Deployment System	Test Type
CPT track rig (TC14)	25 ton rig cylinder	SCPTu

Coordinates		
Test Type	Collection Method	EPSG Number
SCPTu	Consumer grade GPS	26917

Cone Penetrometers Used for this Project						
Cone Description	Cone Number	Cross Sectional Area (cm <sup>2</sup> )	Sleeve Area (cm <sup>2</sup> )	Tip Capacity (bar)	Sleeve Capacity (bar)	Pore Pressure Capacity (bar)
765:T1500F15U35	EC765	15	225	1500	15	35
Cone EC765 was used for all CPTu soundings.						

Cone Penetration Test (CPTu)	
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	<ul style="list-style-type: none"> <li>Standard plots with expanded range</li> <li>Advanced plots with <math>I_c</math>, <math>S_u</math>, <math>\phi</math> and <math>N1(60)</math></li> <li>Seismic plots with <math>V_s</math></li> <li>Soil Behaviour Type (SBT) scatter plots</li> </ul>

Calculated Geotechnical Parameter Tables	
Additional information	<p>The Normalized Soil Behaviour Type Chart based on <math>Q_{tn}</math> (SBT <math>Q_{tn}</math>) (Robertson, 2009) was used to classify the soil for this project. A detailed set of calculated CPTu parameters have been generated and are provided in Excel format files in the release folder. The CPTu parameter calculations are based on values of corrected tip resistance (<math>q_t</math>) sleeve friction (<math>f_s</math>) and pore pressure (<math>u_2</math>).</p> <p>Soils were classified as either drained or undrained based on the <math>Q_{tn}</math> Normalized Soil Behaviour Type Chart (Robertson, 2009). Calculations for both drained and undrained parameters were included for materials that classified as silt mixtures (zone 4).</p>

## Limitations

### 3rd Party Disclaimer

This report titled “Bradford Bypass”, referred to as the (“Report”), was prepared by ConeTec for Golder Associates. The Report is confidential and may not be distributed to or relied upon by any third parties without the express written consent of ConeTec. Any third parties gaining access to the Report do not acquire any rights as a result of such access. Any use which a third party makes of the Report, or any reliance on or decisions made based on it, are the responsibility of such third parties. ConeTec accepts no responsibility for loss, damage and/or expense, if any, suffered by any third parties as a result of decisions made, or actions taken or not taken, which are in any way based on, or related to, the Report or any portion(s) thereof.

### Client Disclaimer

ConeTec was retained by Golder Associates to collect and provide the raw data (“Data”) which is included in this report titled “Bradford Bypass”, which is referred to as the (“Report”). ConeTec has collected and reported the Data in accordance with current industry standards. No other warranty, express or implied, with respect to the Data is made by ConeTec. In order to properly understand the Data included in the Report, reference must be made to the documents accompanying and other sources referenced in the Report in their entirety. Any analysis, interpretation, judgment, calculations and/or geotechnical parameters (collectively “Interpretations”) included in the Report, including those based on the Data, are outside the scope of ConeTec’s retainer and are included in the Report as a courtesy only. Other than the Data, the contents of the Report (including any Interpretations) should not be relied upon in any fashion without independent verification and ConeTec is in no way responsible for any loss, damage or expense resulting from the use of, and/or reliance on, such material by any party.

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 two geophone sensors for recording seismic signals. All signals are amplified and measured with minimum sixteen-bit resolution down hole within the cone body, and the signals are sent to the surface using a high bandwidth, error corrected digital interface through a shielded cable.

ConeTec penetrometers are manufactured with various tip, friction and pore pressure capacities in both 10 cm<sup>2</sup> and 15 cm<sup>2</sup> 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<sup>2</sup> penetrometers do not require friction reducers as they have a diameter larger than the deployment rods. The 10 cm<sup>2</sup> piezocones use a friction reducer consisting of a rod adapter extension behind the main cone body with an enlarged cross sectional area (typically 44 millimeters diameter over a length of 32 millimeters with tapered leading and trailing edges) located at a distance of 585 millimeters 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<sub>2</sub>" position ([ASTM Type 2](#)). The filter is six millimeters 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](#).



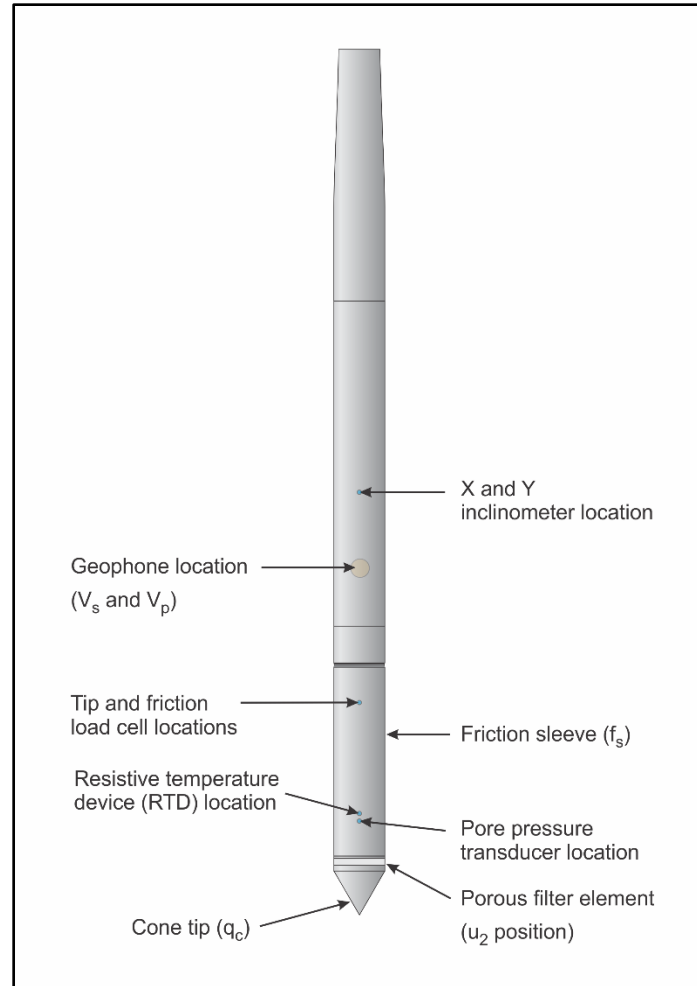


Figure CPTu. Piezocone Penetrometer (15 cm<sup>2</sup>)

The ConeTec data acquisition systems consist of a Windows based computer and a signal interface box and power supply. The signal interface combines depth increment signals, seismic trigger signals and the downhole digital data. This combined data is then sent to the Windows based computer for collection and presentation. 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 centimeters; 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 CPTu 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 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 two centimeters per second, within acceptable tolerances. Typically one meter length rods with an outer diameter of 38.1 millimeters 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
- 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. \(1986\)](#):

$$q_t = q_c + (1-a) \cdot u_2$$

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 ( $R_f$ ) 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 \(2012\)](#).

## References

ASTM D5778-20, 2020, "Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils", ASTM International, West Conshohocken, PA. DOI: [10.1520/D5778-20](#).

Lunne, T., Robertson, P.K. and Powell, J. J. M., 1997, "Cone Penetration Testing in Geotechnical Practice", Blackie Academic and Professional.

Mayne, P.W., 2013, "Evaluating yield stress of soils from laboratory consolidation and in-situ cone penetration tests", Sound Geotechnical Research to Practice (Holtz Volume) GSP 230, ASCE, Reston/VA: 406-420. DOI: [10.1061/9780784412770.027](#).

Mayne, P.W. and Peuchen, J., 2012, "Unit weight trends with cone resistance in soft to firm clays", Geotechnical and Geophysical Site Characterization 4, Vol. 1 (Proc. ISC-4, Pernambuco), CRC Press, London: 903-910.

Mayne, P.W., 2014, "Interpretation of geotechnical parameters from seismic piezocone tests", CPT'14 Keynote Address, Las Vegas, NV, May 2014.

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. DOI: [10.1139/T90-014](#).

Robertson, P.K., 2009, "Interpretation of cone penetration tests – a unified approach", Canadian Geotechnical Journal, Volume 46: 1337-1355. DOI: [10.1139/T09-065](#).

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 ( $t$ ).

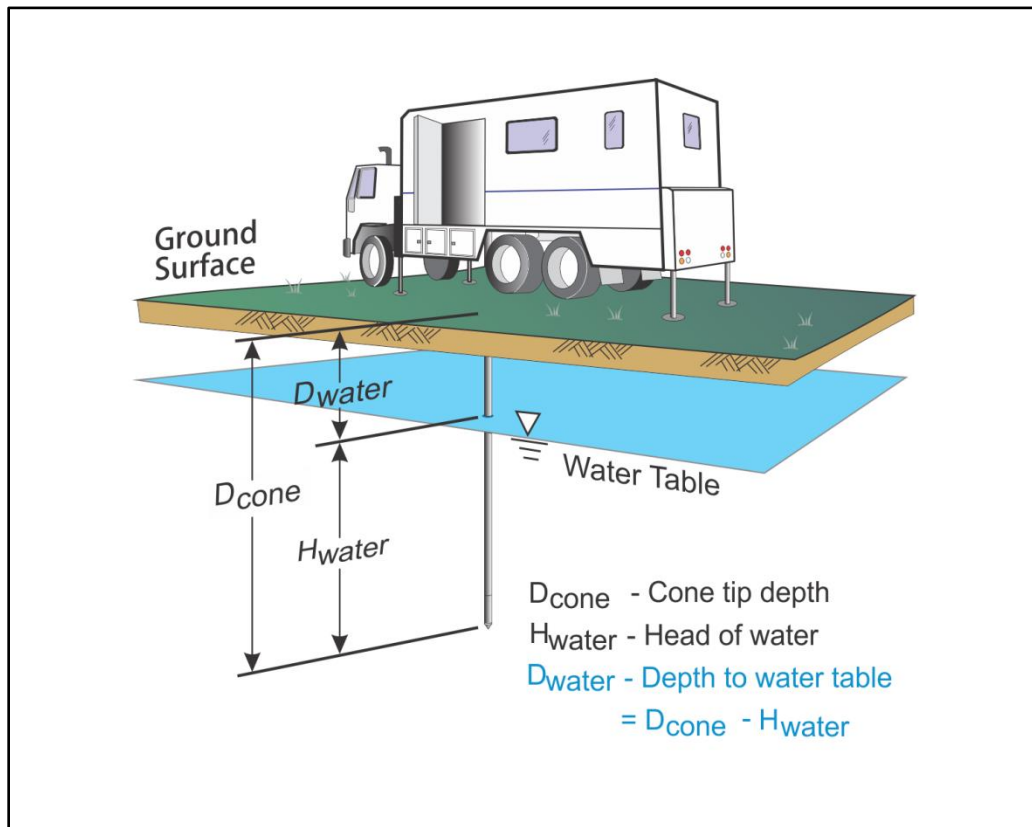


Figure PPD-1. Pore pressure dissipation test setup

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.

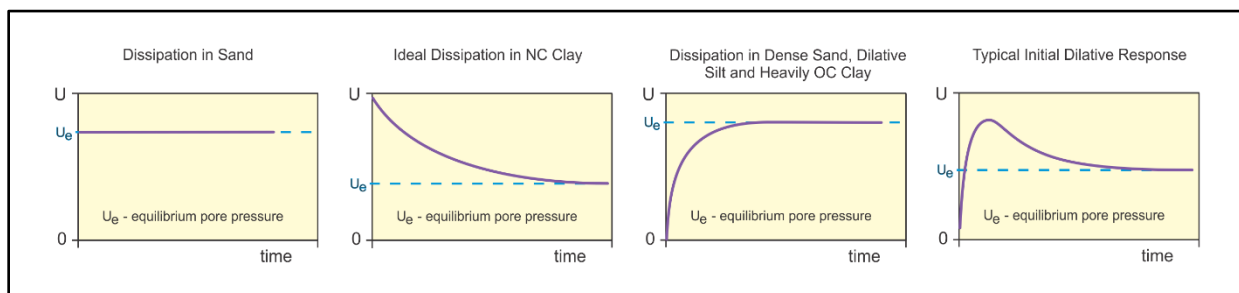


Figure PPD-2. Pore pressure dissipation curve examples

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 dilatatory 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 dilatatory 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.

### References

Burns, S.E. and Mayne, P.W., 1998, "Monotonic and dilatatory pore pressure decay during piezocone tests", Canadian Geotechnical Journal 26 (4): 1063-1073. DOI: [1063-1073/T98-062](https://doi.org/10.1139/T98-062).

Burns, S.E. and Mayne, P.W., 2002, "Analytical cavity expansion-critical state model cone dissipation in fine-grained soils", Soils & Foundations, Vol. 42(2): 131-137.

Jones, G.A. and Van Zyl, D.J.A., 1981, "The piezometer probe: a useful investigation tool", Proceedings, 10<sup>th</sup> International Conference on Soil Mechanics and Foundation Engineering, Vol. 3, Stockholm: 489-495.

Robertson, P.K., Sully, J.P., Woeller, D.J., Lunne, T., Powell, J.J.M. and Gillespie, D.G., 1992, "Estimating coefficient of consolidation from piezocone tests", Canadian Geotechnical Journal, 29(4): 539-550. DOI: [10.1139/T92-061](https://doi.org/10.1139/T92-061).

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. DOI: [10.1139/T98-105](https://doi.org/10.1139/T98-105).

Teh, C.I., and Houlsby, G.T., 1991, "An analytical study of the cone penetration test in clay", Geotechnique, 41(1): 17-34. DOI: [10.1680/geot.1991.41.1.17](https://doi.org/10.1680/geot.1991.41.1.17).

Shear wave velocity ( $V_s$ ) testing is performed in conjunction with the piezocone penetration test (SCPTu) in order to collect interval velocities. For some projects seismic compression wave velocity ( $V_p$ ) testing is also performed.

ConeTec's piezocone penetrometers are manufactured with one horizontally active geophone (28 hertz) and one vertically active geophone (28 hertz). Both geophones are rigidly mounted in the body of the cone penetrometer, 0.2 meters behind the cone tip. The vertically mounted geophone is more sensitive to compression waves.

Shear waves are typically generated by using an impact hammer horizontally striking a beam that is held in place by a normal load. In some instances, an auger source or an imbedded impulsive source may be used for both shear waves and compression waves. The hammer and beam act as a contact trigger that initiates the recording of the seismic wave traces. For impulsive devices an accelerometer trigger may be used. The traces are recorded in the memory of the cone using a fast analog to digital converter. The seismic trace is then transmitted digitally uphole to a Windows based computer through a signal interface box for recording and analysis. An illustration of the shear wave testing configuration is presented in [Figure SCPTu-1](#).

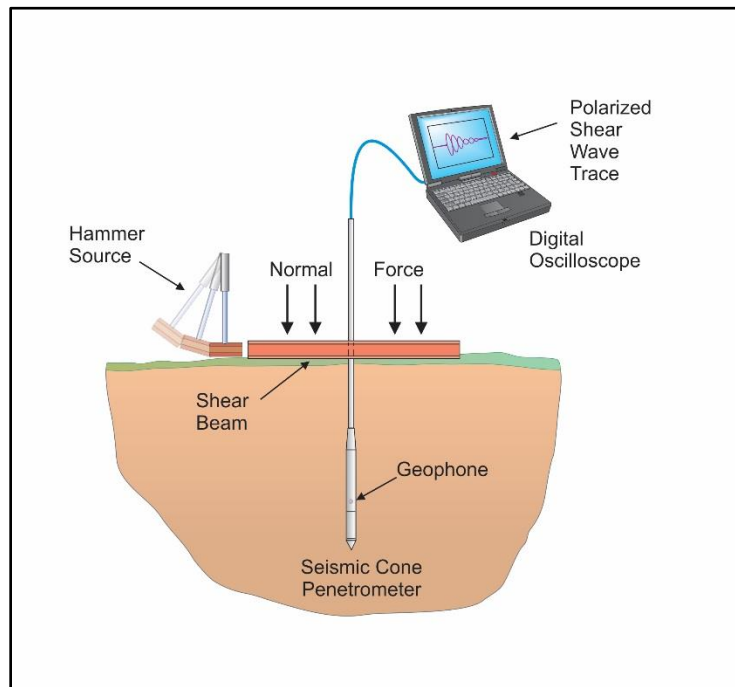


Figure SCPTu-1. Illustration of the SCPTu system

All testing is performed in accordance to ConeTec's SCPTu operating procedures which are in general accordance with the current [ASTM D5778](#) and [ASTM D7400](#) standards.

Prior to the start of a SCPTu sounding, the procedures described in the Cone Penetration Test section are followed. In addition, the active axis of the geophone is aligned parallel to the beam (or source) and the horizontal offset between the cone and the source is measured and recorded.

Prior to recording seismic waves at each test depth, cone penetration is stopped and the rods are decoupled from the rig to avoid transmission of rig energy down the rods. Typically, five wave traces for



each orientation are recorded for quality control and uncertainty analysis purposes. After reviewing wave traces for consistency the cone is pushed to the next test depth (typically one meter intervals or as requested by the client). [Figure SCPTu-2](#) presents an illustration of a SCPTu test.

For additional information on seismic cone penetration testing refer to [Robertson et al. \(1986\)](#).

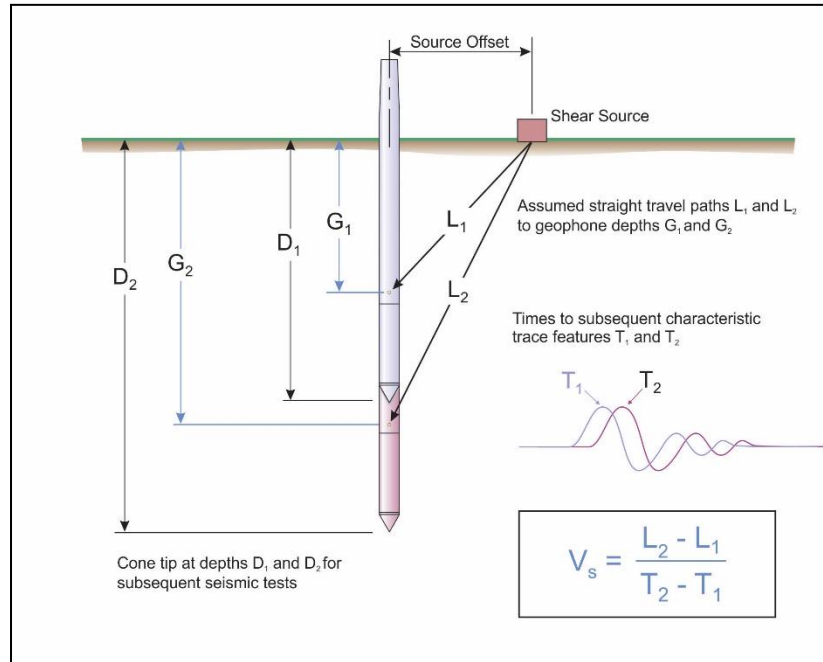


Figure SCPTu-2. Illustration of a seismic cone penetration test

Calculation of the interval velocities are performed by visually picking a common feature (e.g. the first characteristic peak, trough, or crossover) on all of the recorded wave sets and taking the difference in ray path divided by the time difference between subsequent features. Ray path is defined as the straight line distance from the seismic source to the geophone, accounting for beam offset, source depth and geophone offset from the cone tip.

The average shear wave velocity to a depth of thirty meters ( $V_{s30}$ ) has been calculated and provided for all applicable soundings using an equation presented in [Crow et al. \(2012\)](#).

$$V_{s30} = \frac{\text{total thickness of all layers (30m)}}{\sum(\text{layer travel times})}$$

The layer travel times refers to the travel times propagating in the vertical direction, not the measured travel times from an offset source.

Tabular results and SCPTu plots are presented in the relevant appendix.

## References

ASTM D5778-20, 2020, "Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils", ASTM International, West Conshohocken, PA. DOI: [10.1520/D5778-20](https://doi.org/10.1520/D5778-20).

ASTM D7400/D7400M-19, 2019, "Standard Test Methods for Downhole Seismic Testing", ASTM International, West Conshohocken, PA. DOI: [10.1520/D7400\\_D7400M-19](https://doi.org/10.1520/D7400_D7400M-19).

Crow, H.L., Hunter, J.A., Bobrowsky, P.T., 2012, "National shear wave measurement guidelines for Canadian seismic site assessment", GeoManitoba 2012, Sept 30 to Oct 2, Winnipeg, Manitoba.

Robertson, P.K., Campanella, R.G., Gillespie D and Rice, A., 1986, "Seismic CPT to Measure In-Situ Shear Wave Velocity", Journal of Geotechnical Engineering ASCE, Vol. 112, No. 8: 791-803. DOI: [10.1061/\(ASCE\)0733-9410\(1986\)112:8\(791\)](https://doi.org/10.1061/(ASCE)0733-9410(1986)112:8(791)).

The appendices listed below are included in the report:

- Cone Penetration Test Summary and Standard Cone Penetration Test Plots
- Standard Cone Penetration Test Plots with Expanded Range
- Advanced Cone Penetration Test Plots with  $I_c$ ,  $S_u(N_{kt})$ ,  $\Phi$  and  $N1(60)I_c$
- Seismic Cone Penetration Test Plots
- Seismic Cone Penetration Test Shear Wave ( $V_s$ ) Tabular Results
- Seismic Cone Penetration Test Shear Wave ( $V_s$ ) Traces
- Soil Behaviour Type (SBT) Scatter Plots
- Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots
- Description of Methods for Calculated CPT Geotechnical Parameters

# Cone Penetration Test Summary and Standard Cone Penetration Test Plots

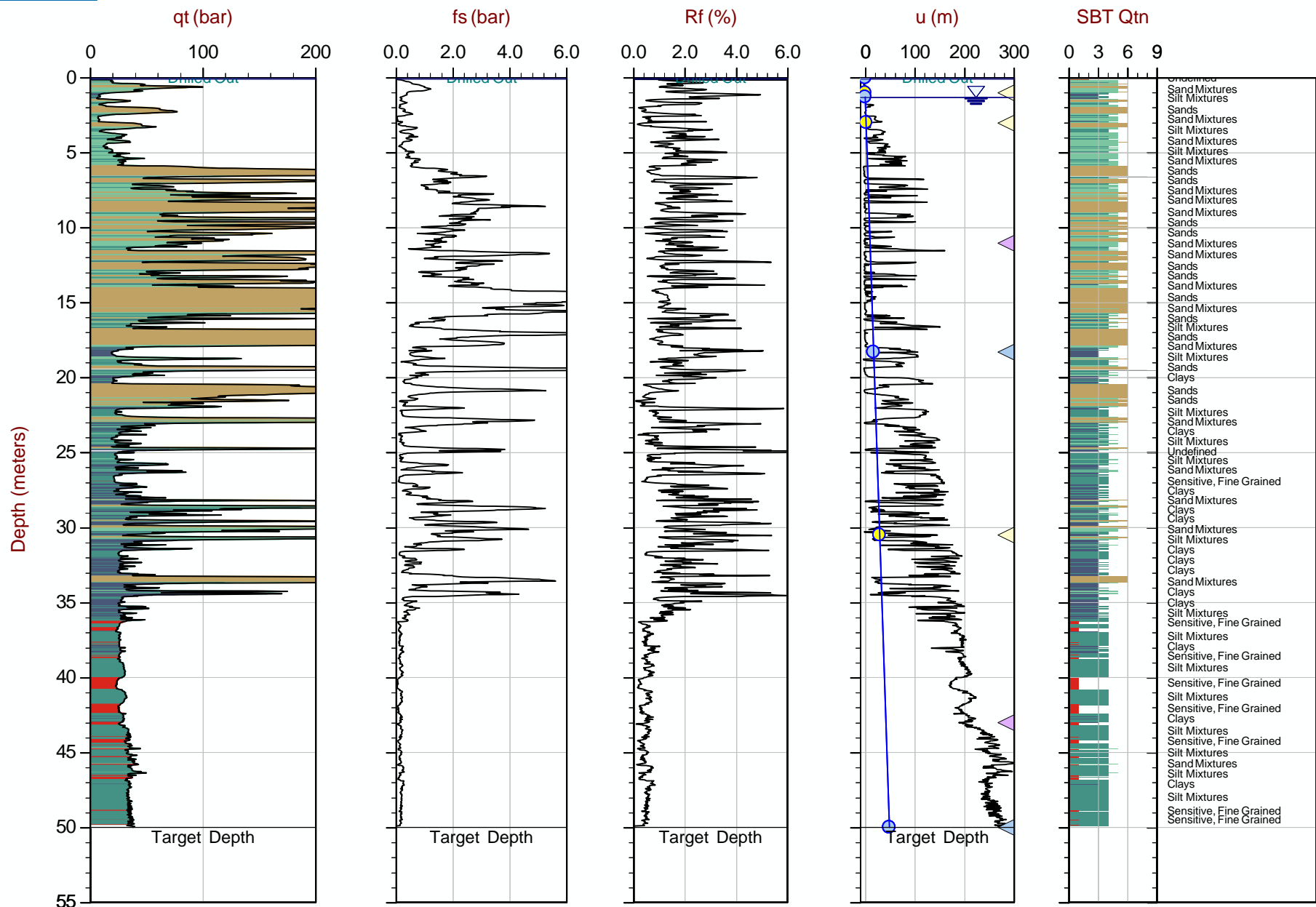


Job No: 21-05-23424  
Client: Golder Associates  
Project: Bradford Bypass  
Start Date: 20-Dec-2021  
End Date: 22-Dec-2021

### ***CONE PENETRATION TEST SUMMARY***

Sounding ID	File Name	Date	Cone	Cone Area (cm <sup>2</sup> )	Assumed Phreatic Surface <sup>1</sup> (m)	Final Depth (m)	Northing <sup>2</sup> (m)	Easting <sup>2</sup> (m)	Refer to Notation Number
SCPT21-HSE-01	21-05-23424_SP-HSE-01	20-Dec-2021	765:T1500F15U35	15	1.3	50.000	4888079	618839	
SCPT21-HRW-01B	21-05-23424_SP-HRW-01B	22-Dec-2021	765:T1500F15U35	15	0.9	19.775	4887420	616058	
SCPT21-HRW-04	21-05-23424_SP-HRW-04	21-Dec-2021	765:T1500F15U35	15	0.3	42.125	4887603	616419	

1. The assumed phreatic surface was based on pore pressure dissipation tests. Hydrostatic conditions were assumed for the calculated parameters.
2. Coordinates were collected with a consumer grade GPS device, datum: NAD83 / UTM Zone 17N.



SBT: Robertson, 2009 and 2010  
 Coords: UTM17N:4888079mE:618839m  
 SheetNo: 1 of 1

Overplot Item: ● Ueq ● Assumed Ueq ◀ Dissipation, Ueq achieved ◀ Dissipation, Ueq not achieved ◀ Dissipation, Ueq assumed — Ueq Line — Hydrostatic Line

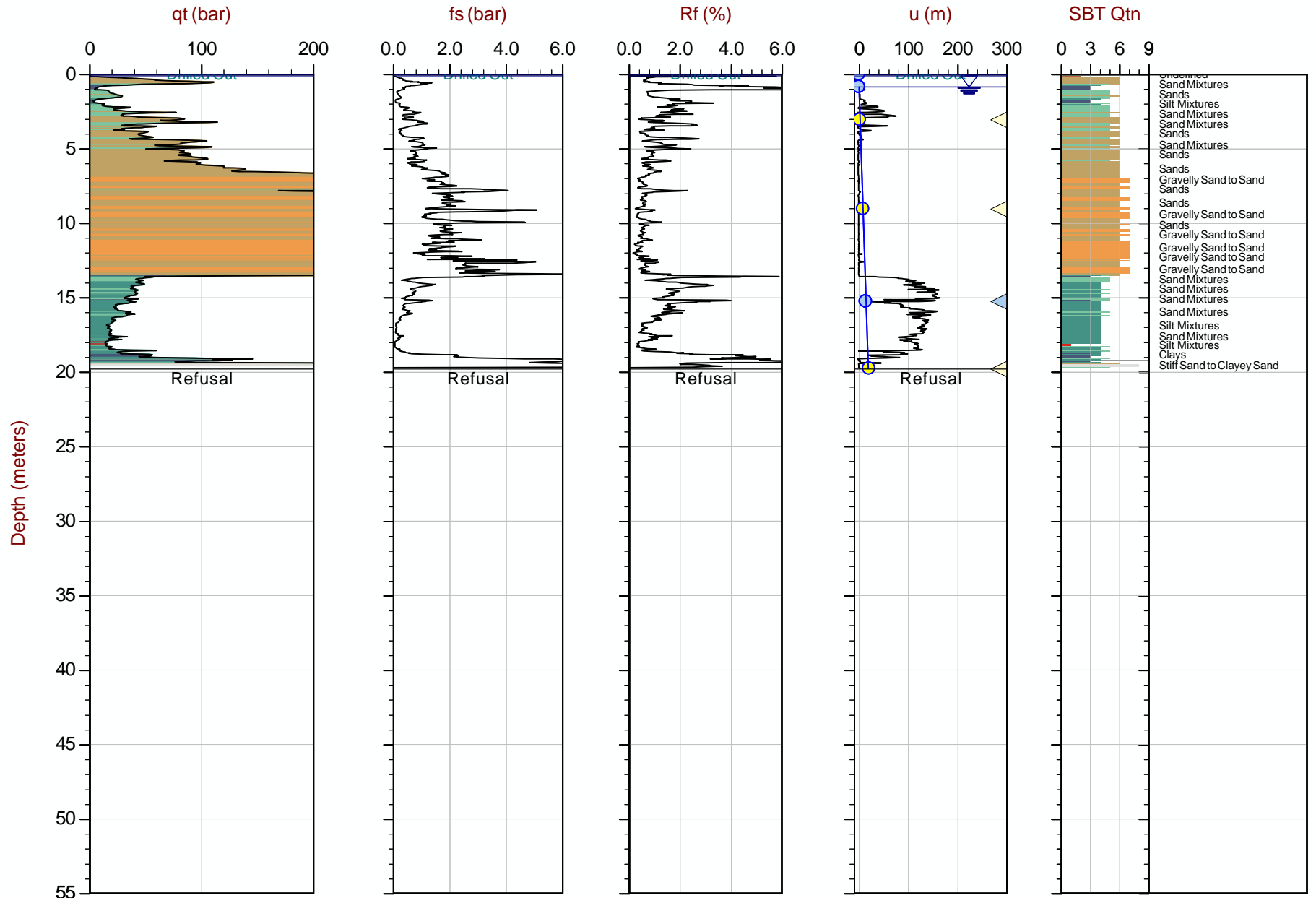
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



Golder

Job No: 21-05-23424  
Date: 2021-12-22 09:45  
Site: East Holland River

Sounding: SCPT21-HRW-01B  
Cone: 765:T1500F15U35 Area= 15cm2



Max Depth: 19.775 m / 64.88 ft  
Depth Inc: 0.025 m / 0.082 ft  
Avg Int: Every Point

File: 21-05-23424\_SP-HRW-01B.COR  
Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010  
Coords: UTM 17N N: 4887420m E: 616058m  
Sheet No: 1 of 1

Overplot Item: ● Ueq    ● Assumed Ueq    ▲ Dissipation, Ueq achieved    ▲ Dissipation, Ueq not achieved    ▲ Dissipation, Ueq assumed    — Ueq Line    — Hydrostatic Line

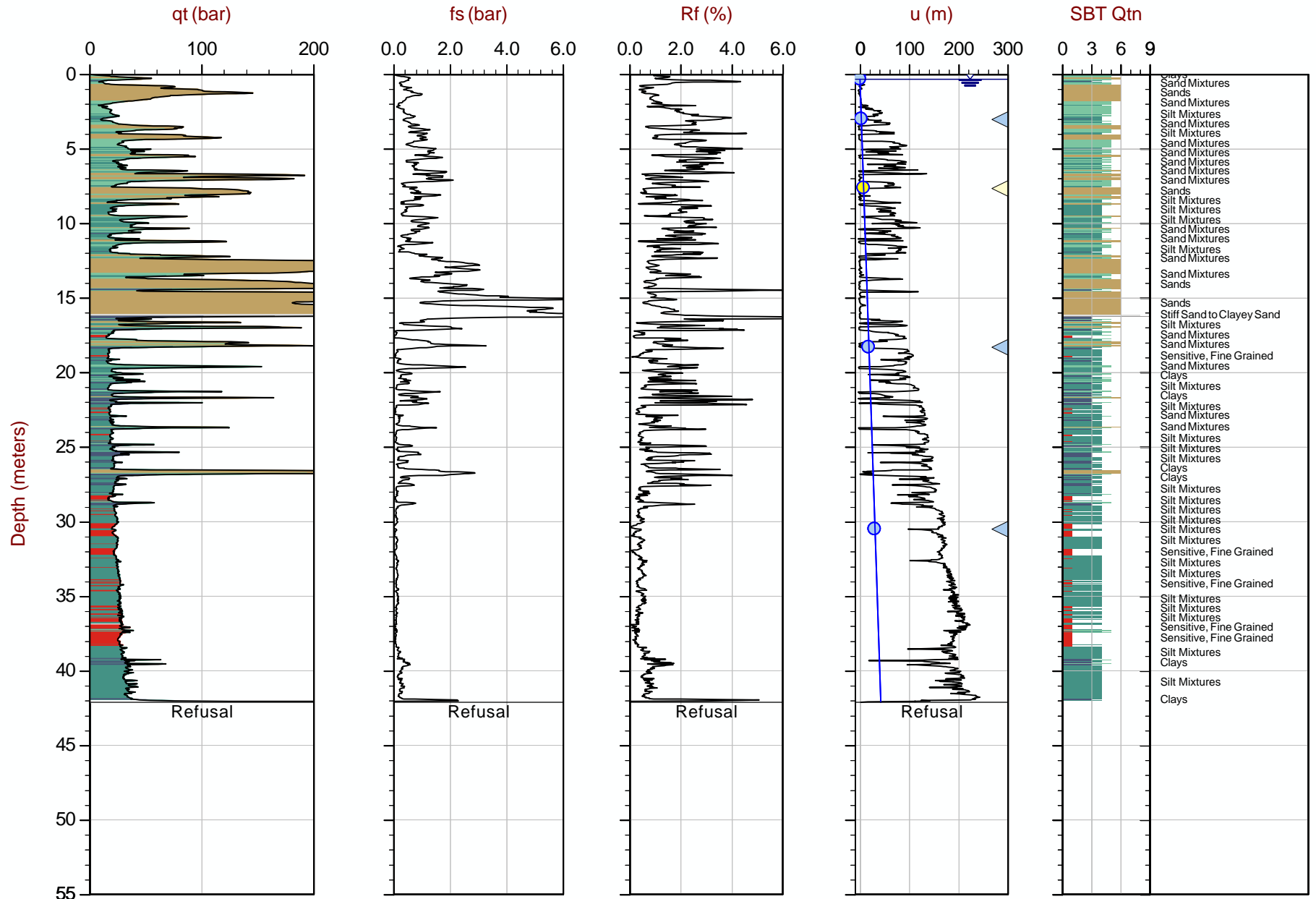
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



# Golder

Job No: 21-05-23424  
Date: 2021-12-21 11:38  
Site: East Holland River

Sounding: SCPT21-HRW-04  
Cone: 765:T1500F15U35 Area= 15cm2



Max Depth: 42.125 m / 138.20 ft  
Depth Inc: 0.025 m / 0.082 ft  
Avg Int: Every Point

File: 21-05-23424\_SP-HRW-04.COR  
Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010  
Coords: UTM 17N N: 4887603m E: 616419m  
Sheet No: 1 of 1

Overplot Item: ● Ueq    ● Assumed Ueq    ▲ Dissipation, Ueq achieved    ▲ Dissipation, Ueq not achieved    ▲ Dissipation, Ueq assumed    — Ueq Line    — Hydrostatic Line  
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



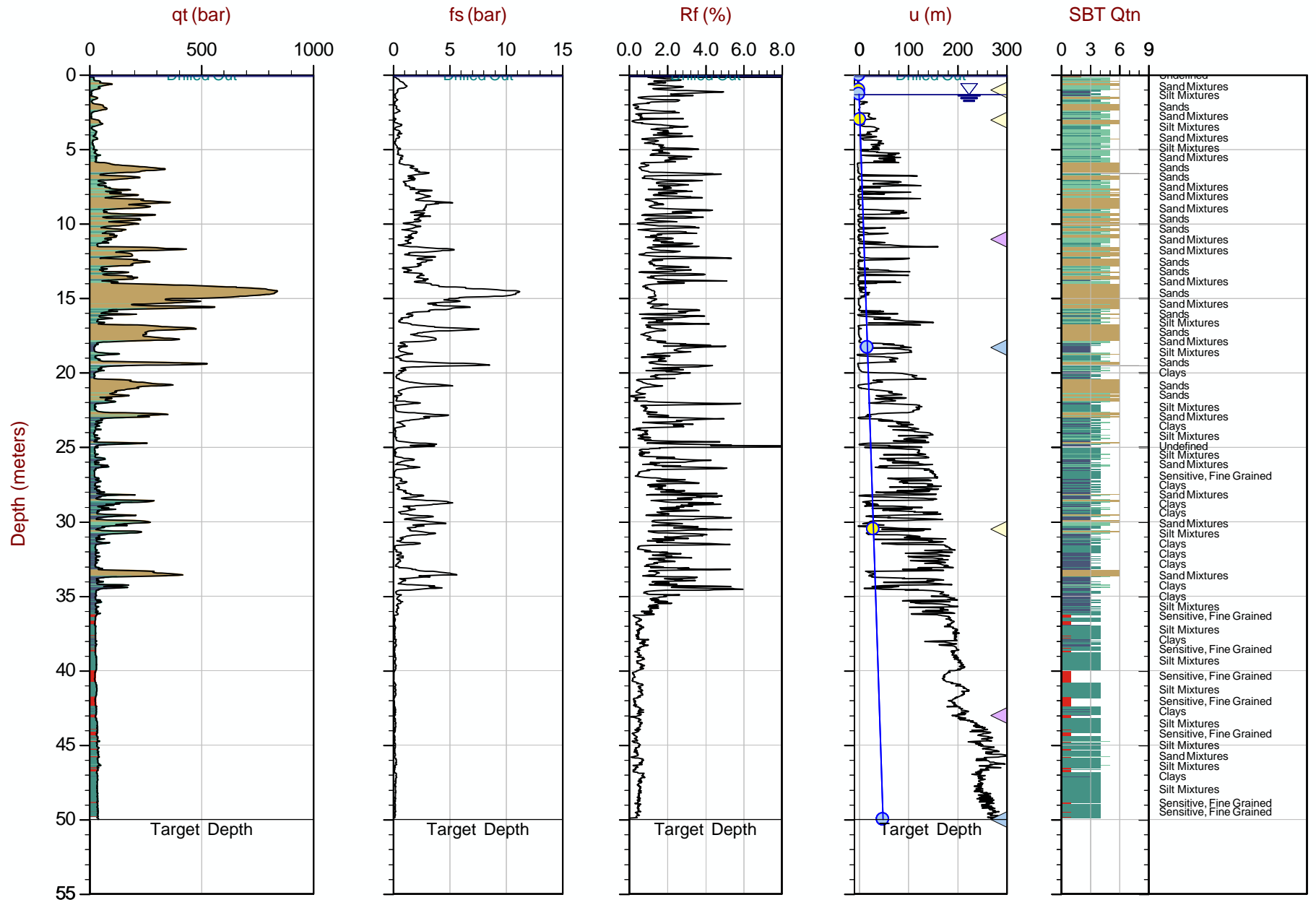
## Standard Cone Penetration Test Plots with Expanded Range



Golder

Job No: 21-05-23424  
Date: 2021-12-20 13:32  
Site: East Holland River

Sounding: SCPT21-HSE-01  
Cone: 765:T1500F15U35 Area= 15cm2



Max Depth: 50.000 m / 164.04 ft  
Depth Inc: 0.025 m / 0.082 ft  
Avg Int: Every Point

File: 21-05-23424\_SP-HSE-01.COR  
Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010  
Coords: UTM17N N: 4888079m E: 618839m  
Sheet No: 1 of 1

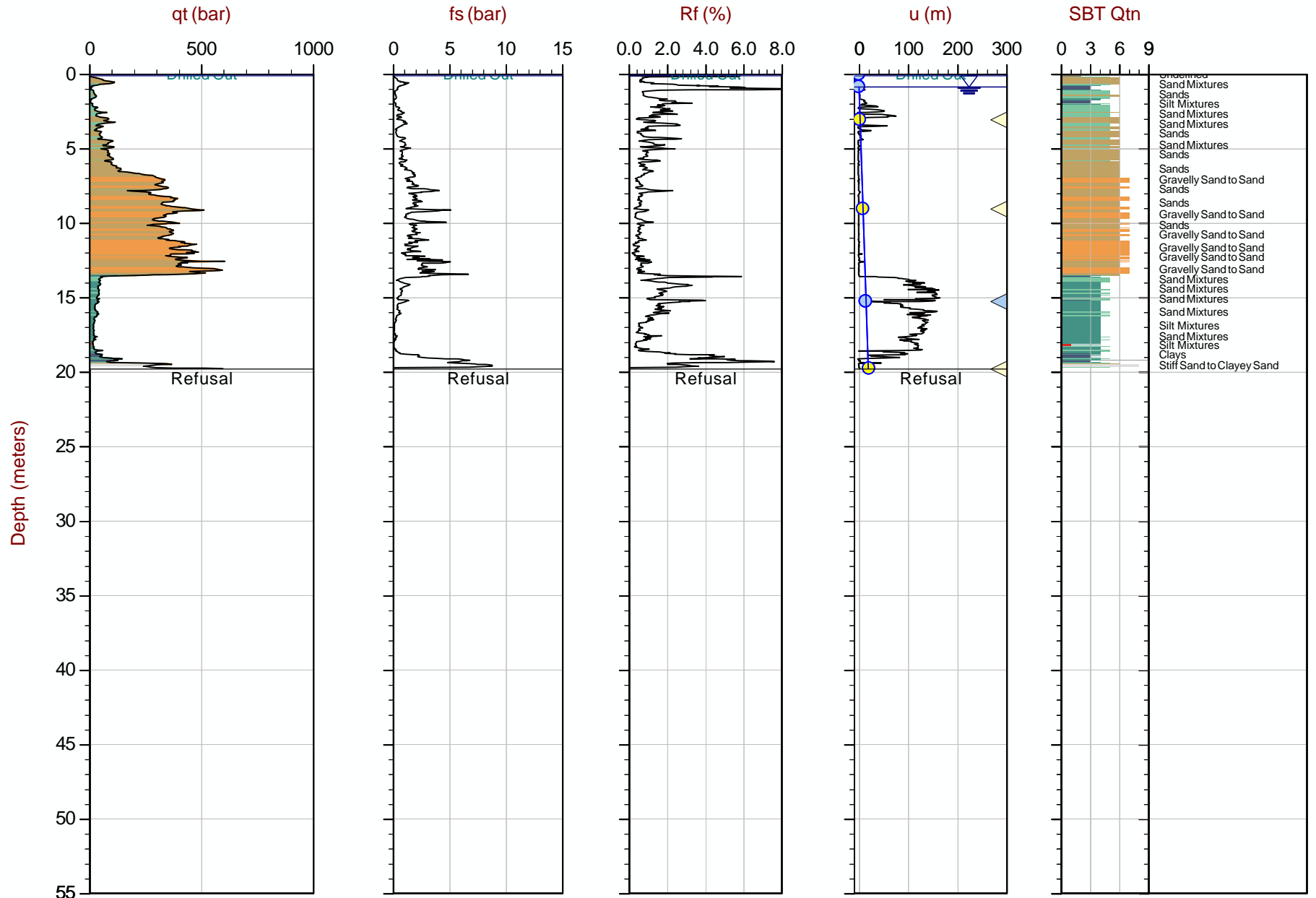
Overplot Item: ● Ueq ● Assumed Ueq ▲ Dissipation, Ueq achieved ▲ Dissipation, Ueq not achieved ▲ Dissipation, Ueq assumed — Ueq Line — Hydrostatic Line  
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



Golder

Job No: 21-05-23424  
Date: 2021-12-22 09:45  
Site: East Holland River

Sounding: SCPT21-HRW-01B  
Cone: 765:T1500F15U35 Area= 15cm2



Max Depth: 19.775 m / 64.88 ft  
Depth Inc: 0.025 m / 0.082 ft  
Avg Int: Every Point

File: 21-05-23424\_SP-HRW-01B.COR  
Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010  
Coords: UTM 17N N: 4887420m E: 616058m  
Sheet No: 1 of 1

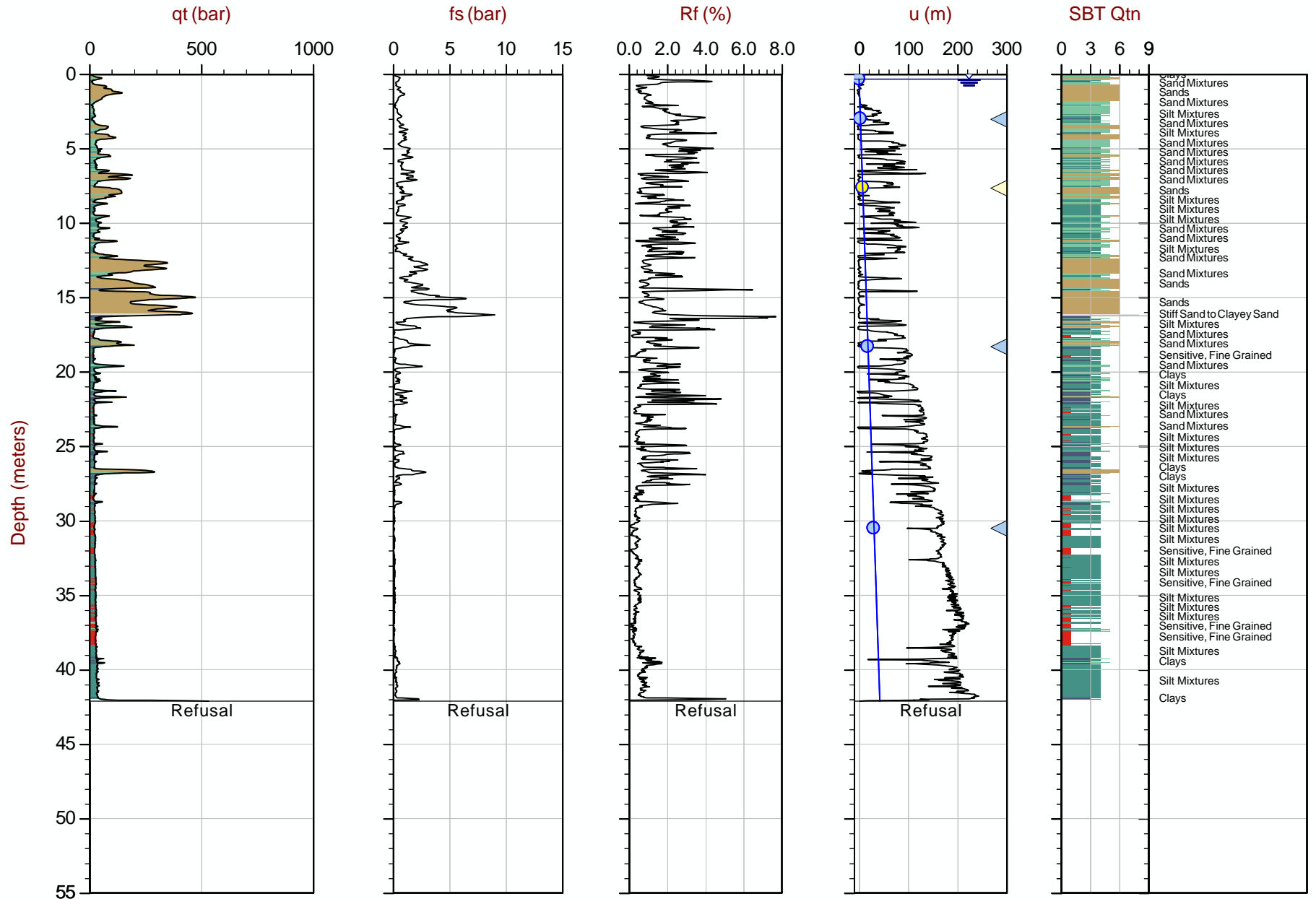
Overplot Item: ● Ueq    ● Assumed Ueq    ▲ Dissipation, Ueq achieved    ▲ Dissipation, Ueq not achieved    ▲ Dissipation, Ueq assumed    — Ueq Line    — Hydrostatic Line  
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



Golder

Job No: 21-05-23424  
Date: 2021-12-21 11:38  
Site: East Holland River

Sounding: SCPT21-HRW-04  
Cone: 765:T1500F15U35 Area= 15cm2



Max Depth: 42.125 m / 138.20 ft  
Depth Inc: 0.025 m / 0.082 ft  
Avg Int: Every Point

File: 21-05-23424\_SP-HRW-04.COR  
Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010  
Coords: UTM17N: 4887603m E: 616419m  
Sheet No: 1 of 1

Overplot Item: ● Ueq    ● Assumed Ueq    ▲ Dissipation, Ueq achieved    ▲ Dissipation, Ueq not achieved    ▲ Dissipation, Ueq assumed    — Ueq Line    — Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

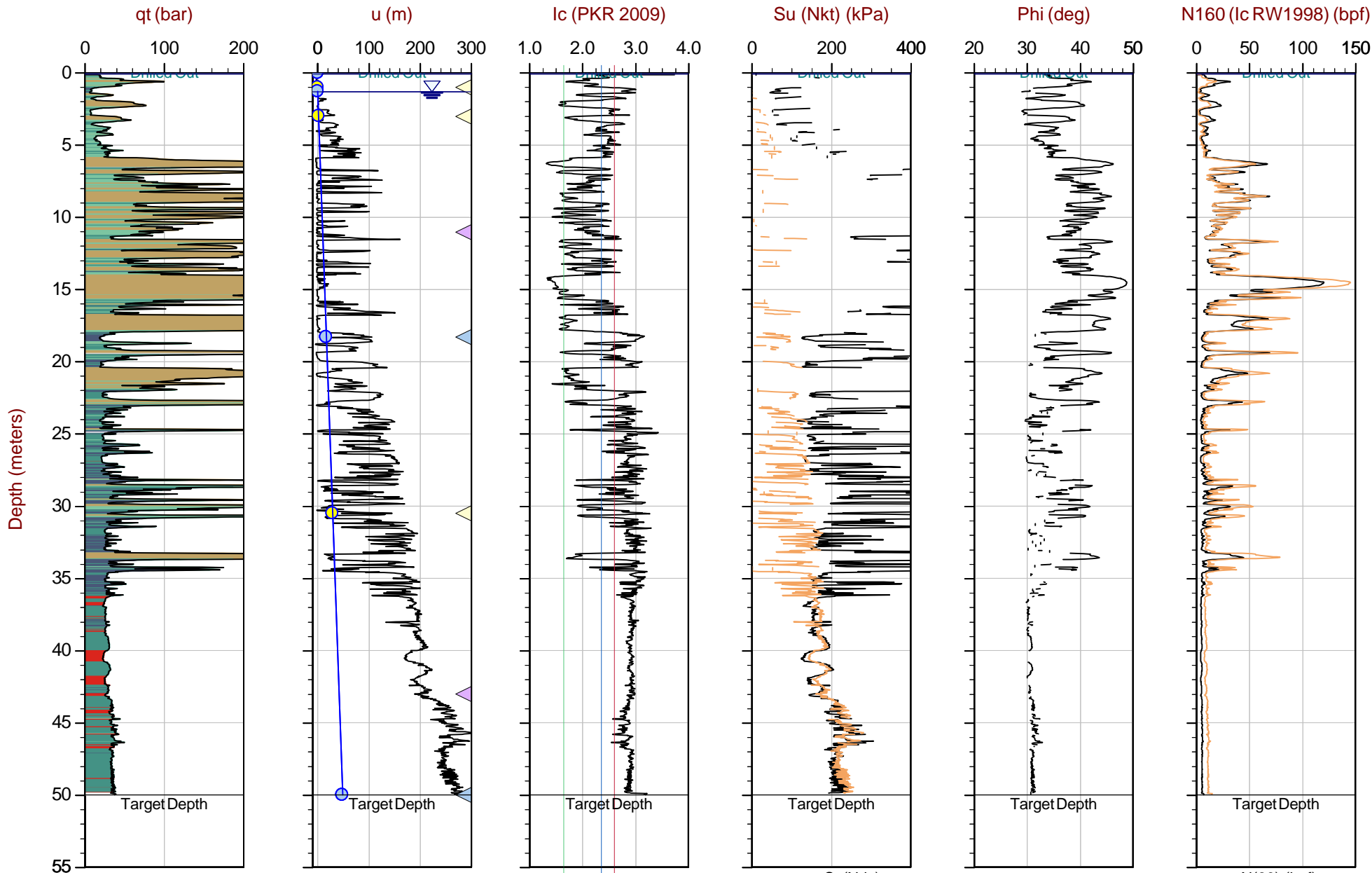
Advanced Cone Penetration Plots with  $I_c$ ,  $S_u(N_{kt})$ ,  $\Phi$  and  $N_{1(60)I_c}$



*Golder*

Job No: 21-05-23424  
Date: 2021-12-20 13:32  
Site: East Holland River

Sounding: SCPT21-HSE-01  
Cone: 765:T1500F15U35 Area= 15cm2



Max Depth: 50.000 m / 164.04 ft  
Depth Inc: 0.025 m / 0.082 ft  
Avg Int: EveryPoint

File: 21-05-23424\_SP-HSE-01.COR  
Unit Wt: SBTQtn(PKR2009)  
SuNkt/Ndu: 12.0 / 9.0

SBT: Robertson, 2009 and 2010  
Coords: UTM17N N: 4888079m E: 618839m  
Sheet No: 1 of 1

Overplot Item: ● Ueq ● Assumed Ueq ▲ Dissipation, Ueq achieved ▲ Dissipation, Ueq not achieved ▲ Dissipation, Ueq assumed — Ueq Line — Hydrostatic Line

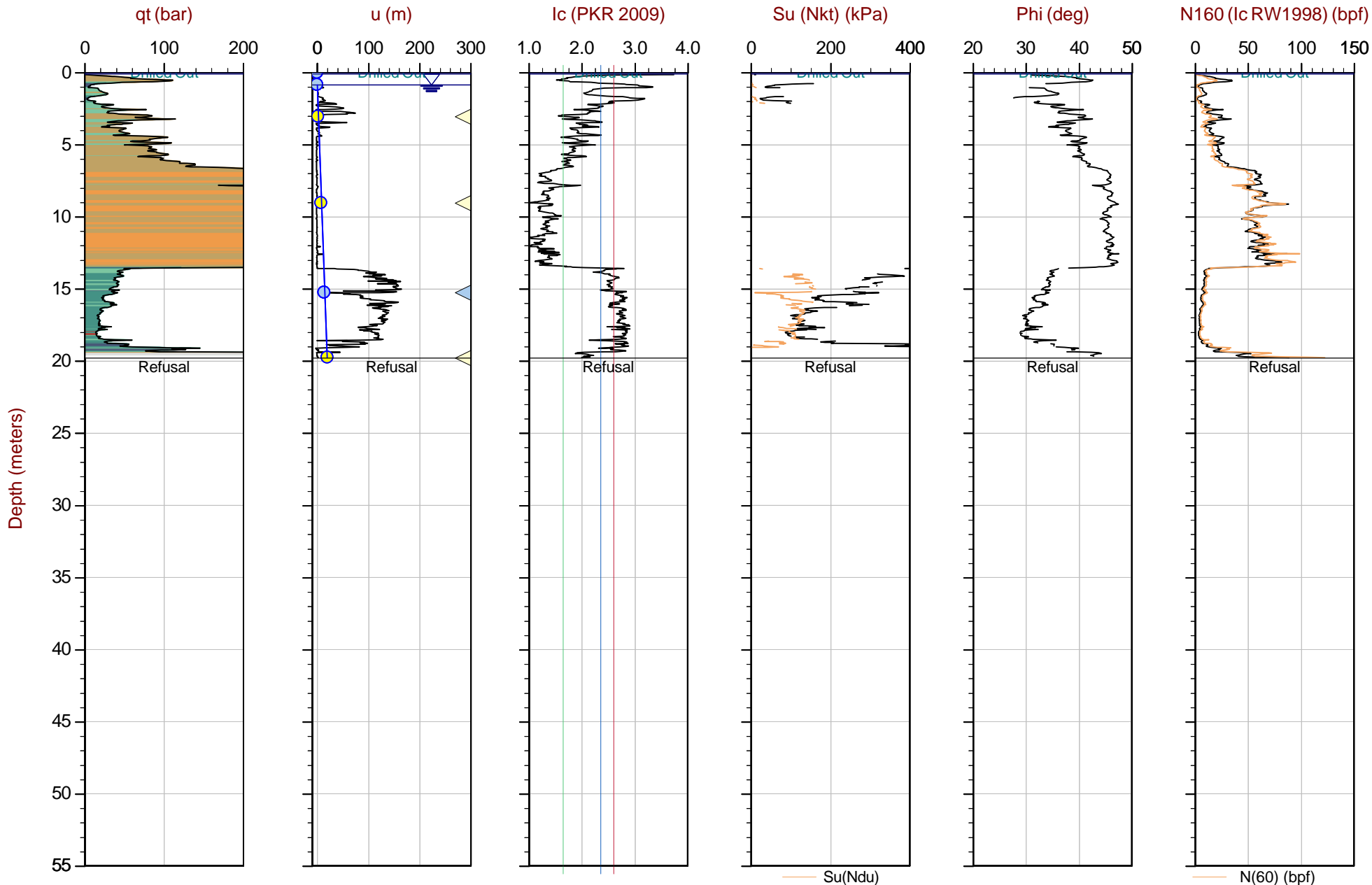
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



*Golder*

Job No: 21-05-23424  
Date: 2021-12-22 09:45  
Site: East Holland River

Sounding: SCPT21-HRW-01B  
Cone: 765:T1500F15U35 Area= 15cm2



Max Depth: 19.775 m / 64.88 ft  
Depth Inc: 0.025 m / 0.082 ft  
Avg Int: Every Point

File: 21-05-23424\_SP-HRW-01B.COR  
Unit Wt: SBTQtn(PKR2009)  
SuNkt/Ndu: 12.0 / 9.0

SBT: Robertson, 2009 and 2010  
Coords: UTM17N N: 4887420m E: 616058m  
Sheet No: 1 of 1

Overplot Item: ● Ueq ● Assumed Ueq ▲ Dissipation, Ueq achieved ▼ Dissipation, Ueq not achieved ◀ Dissipation, Ueq assumed — Ueq Line — Hydrostatic Line

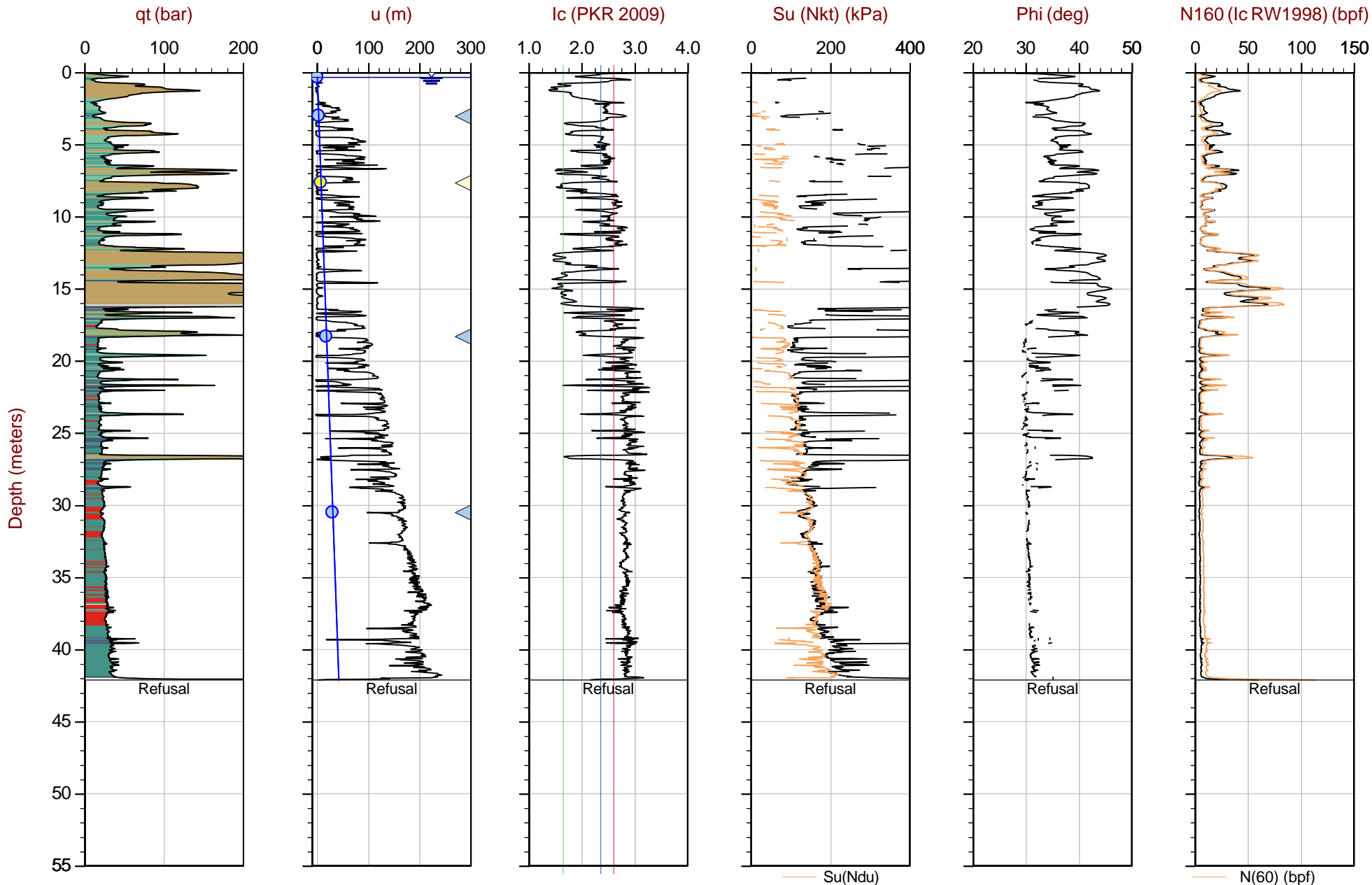
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



*Golder*

Job No: 21-05-23424  
Date: 2021-12-21 11:38  
Site: East Holland River

Sounding: SCPT21-HRW-04  
Cone: 765:T1500F15U35 Area= 15cm2



Max Depth: 42.125 m / 138.20 ft  
Depth Inc: 0.025 m / 0.082 ft  
Avg Int: EveryPoint

File: 21-05-23424\_SP-HRW-04.COR  
Unit Wt: SBTQtn(PKR2009)  
SuNkt/Ndu: 12.0 / 9.0

SBT: Robertson, 2009 and 2010  
Coords: UTM17N:4887603mE:616419m  
Sheet No: 1 of 1

OverplotItem: ● Ueq ● Assumed Ueq ▲ Dissipation, Ueq achieved ▲ Dissipation, Ueq not achieved ▲ Dissipation, Ueq assumed — Ueq Line — Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



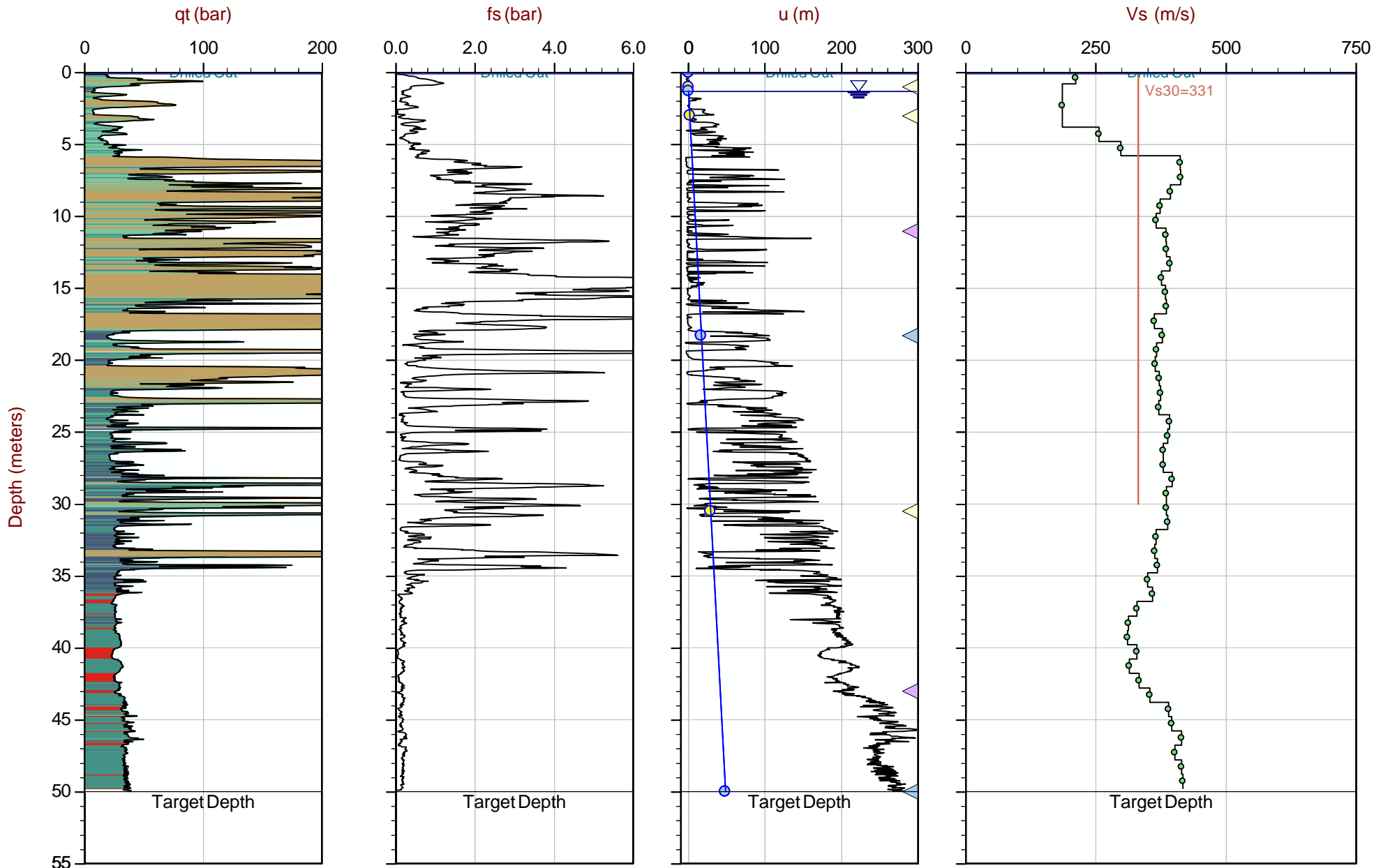
## Seismic Cone Penetration Test Plots



*Golder*

Job No: 21-05-23424  
Date: 2021-12-20 13:32  
Site: East Holland River

Sounding: SCPT21-HSE-01  
Cone: 765:T1500F15U35 Area= 15cm<sup>2</sup>



Max Depth: 50.000 m / 164.04 ft  
Depth Inc: 0.025 m / 0.082 ft  
Avg Int: Every Point

File: 21-05-23424\_SP-HSE-01.COR  
Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010  
Coords: UTM17N: 4888079mE: 618839m  
Sheet No: 1 of 1

Overplot Item: ● Ueq ● Assumed Ueq ◀ Dissipation, Ueq achieved ◀ Dissipation, Ueq not achieved ◀ Dissipation, Ueq assumed — Ueq Line — Hydrostatic Line

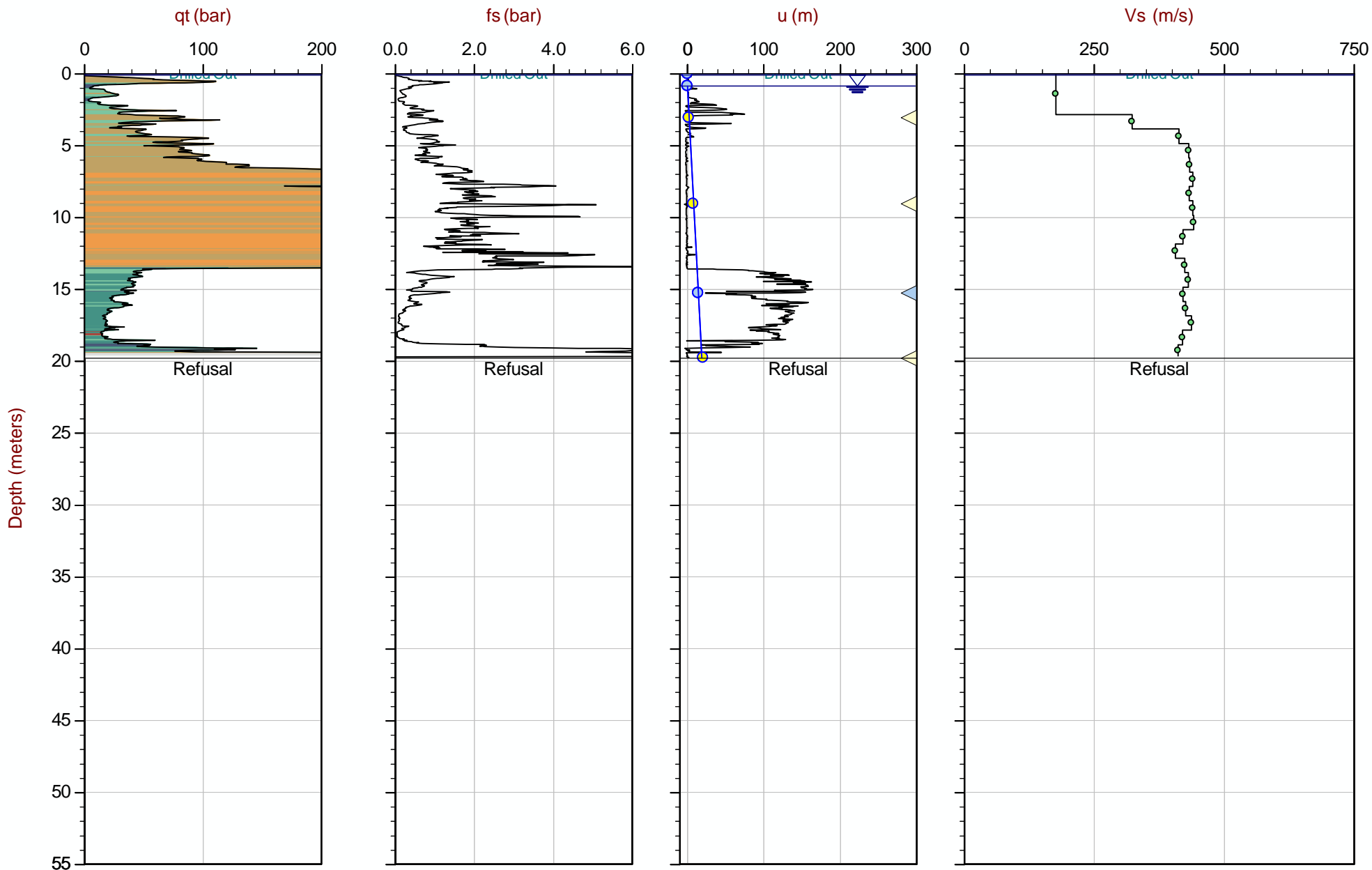
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



*Golder*

Job No: 21-05-23424  
Date: 2021-12-22 09:45  
Site: East Holland River

Sounding: SCPT21-HRW-01B  
Cone: 765:T1500F15U35 Area= 15cm2



Max Depth: 19.775 m / 64.88 ft  
Depth Inc: 0.025 m / 0.082 ft  
Avg Int: Every Point

File: 21-05-23424\_SP-HRW-01B.COR  
Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010  
Coords: UTM17N: 4887420m E: 616058m  
Sheet No: 1 of 1

Overplot Item: ● Ueq ○ Assumed Ueq ◀ Dissipation, Ueq achieved ▶ Dissipation, Ueq not achieved ◄ Dissipation, Ueq assumed — Ueq Line — Hydrostatic Line

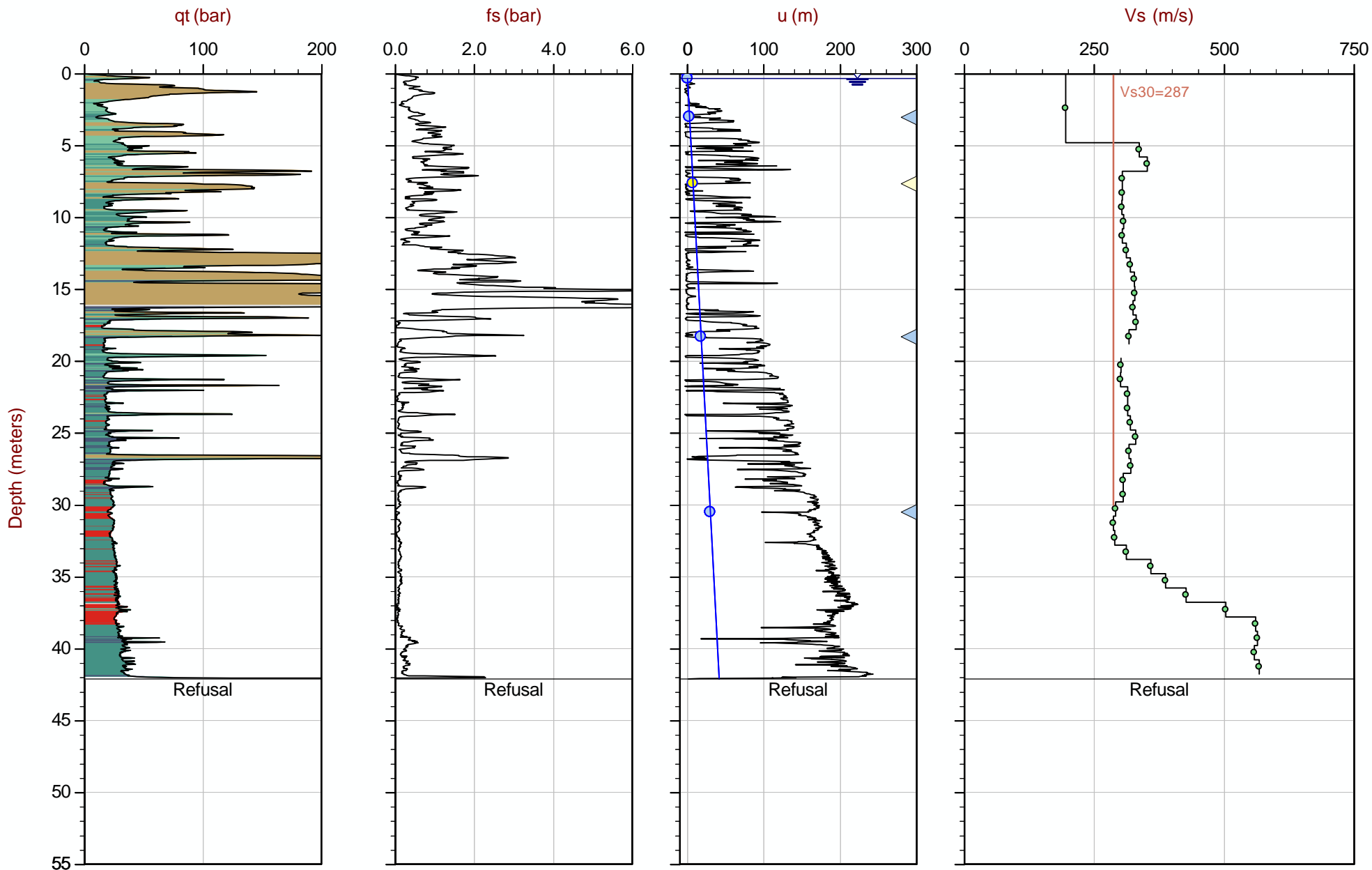
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



*Golder*

Job No: 21-05-23424  
Date: 2021-12-21 11:38  
Site: East Holland River

Sounding: SCPT21-HRW-04  
Cone: 765:T1500F15U35 Area= 15cm2



Max Depth: 42.125 m / 138.20 ft  
Depth Inc: 0.025 m / 0.082 ft  
Avg Int: Every Point

File: 21-05-23424\_SP-HRW-04.COR  
Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010  
Coords: UTM17N: 4887603m E: 616419m  
Sheet No: 1 of 1

Overplot Item: ● Ueq    ● Assumed Ueq    ▲ Dissipation, Ueq achieved    ▲ Dissipation, Ueq not achieved    ▲ Dissipation, Ueq assumed    — Ueq Line    — Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

## Seismic Cone Penetration Test Shear Wave ( $V_s$ ) Tabular Results



Job No: 21-05-23424  
 Client: Golder Associates  
 Project: Bradford Bypass  
 Sounding ID: SCPT21-HSE-01  
 Date: 20-Dec-2021

Seismic Source: Beam  
 Seismic Offset (m): 3.65  
 Source Depth (m): 0.00  
 Geophone Offset (m): 0.20

### SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
1.00	0.80	3.74	3.74	17.72	211
4.00	3.80	5.27	1.53	8.24	186
5.00	4.80	6.03	0.76	2.97	257
6.00	5.80	6.85	0.82	2.76	298
7.00	6.80	7.72	0.87	2.10	412
8.00	7.80	8.61	0.89	2.16	414
9.00	8.80	9.53	0.92	2.33	393
10.00	9.80	10.46	0.93	2.49	374
11.02	10.82	11.42	0.96	2.63	366
12.00	11.80	12.35	0.93	2.43	385
13.00	12.80	13.31	0.96	2.48	386
14.00	13.80	14.28	0.97	2.46	392
15.00	14.80	15.24	0.97	2.57	376
16.00	15.80	16.22	0.97	2.53	384
17.00	16.80	17.19	0.98	2.53	385
18.00	17.80	18.17	0.98	2.70	363
19.00	18.80	19.15	0.98	2.59	378
19.98	19.78	20.11	0.96	2.62	367
21.00	20.80	21.12	1.00	2.76	364
22.00	21.80	22.10	0.99	2.65	372
23.00	22.80	23.09	0.99	2.64	374
24.00	23.80	24.08	0.99	2.66	372
25.00	24.80	25.07	0.99	2.52	392
26.00	25.80	26.06	0.99	2.55	388
27.00	26.80	27.05	0.99	2.61	379
28.00	27.80	28.04	0.99	2.61	380
29.00	28.80	29.03	0.99	2.50	397
30.00	29.80	30.02	0.99	2.58	386
31.00	30.80	31.02	0.99	2.58	386
32.00	31.80	32.01	0.99	2.56	388
33.00	32.80	33.00	0.99	2.71	366



Job No: 21-05-23424  
Client: Golder Associates  
Project: Bradford Bypass  
Sounding ID: SCPT21-HSE-01  
Date: 20-Dec-2021

Seismic Source: Beam  
Seismic Offset (m): 3.65  
Source Depth (m): 0.00  
Geophone Offset (m): 0.20

### ***SCPT<sub>u</sub> SHEAR WAVE VELOCITY TEST RESULTS - V<sub>s</sub>***

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
34.00	33.80	34.00	1.00	2.74	364
35.00	34.80	34.99	0.99	2.70	369
36.00	35.80	35.99	1.00	2.84	350
37.00	36.80	36.98	1.00	2.77	359
38.00	37.80	37.98	1.00	3.02	329
39.00	38.80	38.97	1.00	3.18	313
40.00	39.80	39.97	1.00	3.20	311
41.00	40.80	40.96	1.00	3.03	329
42.00	41.80	41.96	1.00	3.17	314
43.00	42.80	42.96	1.00	2.98	334
44.00	43.80	43.95	1.00	2.82	354
45.00	44.80	44.95	1.00	2.55	390
46.00	45.80	45.95	1.00	2.52	396
47.00	46.80	46.94	1.00	2.40	415
48.00	47.80	47.94	1.00	2.48	402
49.00	48.80	48.94	1.00	2.40	415
50.00	49.80	49.93	1.00	2.39	417



Job No: 21-05-23424  
Client: Golder Associates  
Project: Bradford Bypass  
Sounding ID: SCPT21-HRW-01B  
Date: 22-Dec-2021

Seismic Source: Beam  
Seismic Offset (m): 3.65  
Source Depth (m): 0.00  
Geophone Offset (m): 0.20

### ***SCPT<sub>u</sub> SHEAR WAVE VELOCITY TEST RESULTS - Vs***

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
3.05	2.85	4.63	4.63	26.31	176
4.05	3.85	5.31	0.67	2.09	323
5.05	4.85	6.07	0.77	1.85	414
6.05	5.85	6.90	0.83	1.91	432
7.05	6.85	7.76	0.87	2.00	434
8.05	7.85	8.66	0.90	2.04	440
9.05	8.85	9.57	0.92	2.12	433
10.05	9.85	10.51	0.93	2.12	440
11.05	10.85	11.45	0.94	2.14	441
12.05	11.85	12.40	0.95	2.26	421
13.05	12.85	13.36	0.96	2.36	406
14.05	13.85	14.32	0.97	2.27	425
15.05	14.85	15.29	0.97	2.25	431
16.05	15.85	16.27	0.97	2.31	421
17.05	16.85	17.24	0.98	2.29	426
18.05	17.85	18.22	0.98	2.24	437
19.08	18.88	19.23	1.01	2.41	420
19.83	19.63	19.97	0.74	1.79	411





Job No: 21-05-23424  
 Client: Golder Associates  
 Project: Bradford Bypass  
 Sounding ID: SCPT21-HRW-04  
 Date: 21-Dec-2021

Seismic Source: Beam  
 Seismic Offset (m): 3.65  
 Source Depth (m): 0.00  
 Geophone Offset (m): 0.20

### SCPT<sub>u</sub> SHEAR WAVE VELOCITY TEST RESULTS - V<sub>s</sub>

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
5.00	4.80	6.03	6.03	30.84	196
6.00	5.80	6.85	0.82	2.44	337
7.00	6.80	7.72	0.87	2.46	352
8.00	7.80	8.61	0.89	2.94	304
9.00	8.80	9.53	0.92	3.01	305
10.00	9.80	10.46	0.93	3.07	303
11.00	10.80	11.40	0.94	3.07	307
12.00	11.80	12.35	0.95	3.13	304
13.00	12.80	13.31	0.96	3.07	312
14.00	13.80	14.28	0.97	3.02	320
15.00	14.80	15.24	0.97	2.96	328
16.00	15.80	16.22	0.97	2.97	328
17.00	16.80	17.19	0.98	3.01	325
18.00	17.80	18.17	0.98	2.96	331
19.00	18.80	19.15	0.98	3.09	317
20.00	19.80	20.13			
21.00	20.80	21.12	0.98	3.26	302
22.00	21.80	22.10	0.99	3.28	301
23.00	22.80	23.09	0.99	3.13	315
24.00	23.80	24.08	0.99	3.14	315
25.00	24.80	25.07	0.99	3.10	320
26.00	25.80	26.06	0.99	3.00	330
27.00	26.80	27.05	0.99	3.13	317
28.02	27.82	28.06	1.01	3.15	321
29.00	28.80	29.03	0.97	3.18	306
30.00	29.80	30.02	0.99	3.25	306
31.00	30.80	31.02	0.99	3.41	291
32.00	31.80	32.01	0.99	3.46	287
33.00	32.80	33.00	0.99	3.42	290
34.00	33.80	34.00	1.00	3.19	312
35.00	34.80	34.99	0.99	2.77	359



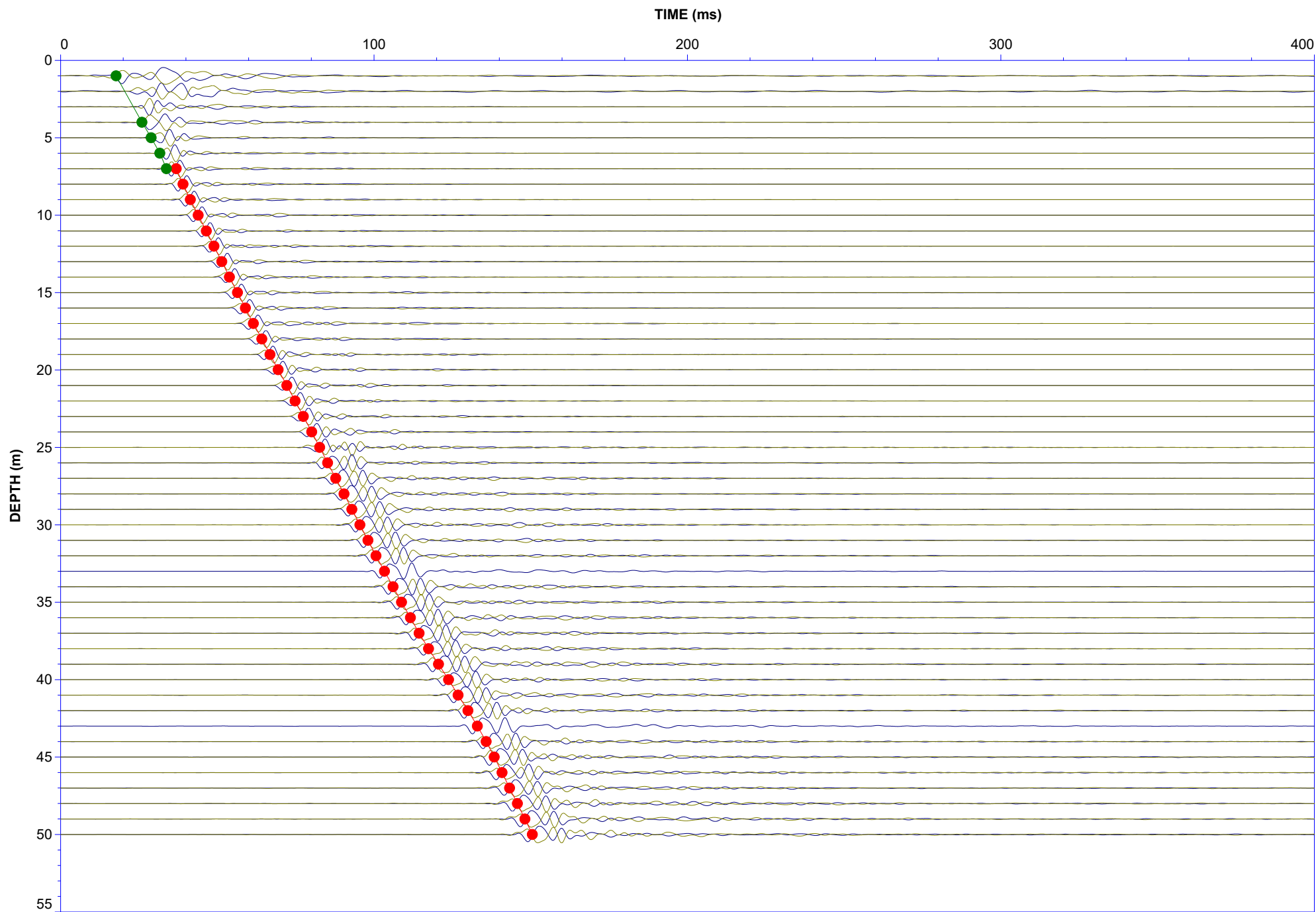
Job No: 21-05-23424  
Client: Golder Associates  
Project: Bradford Bypass  
Sounding ID: SCPT21-HRW-04  
Date: 21-Dec-2021

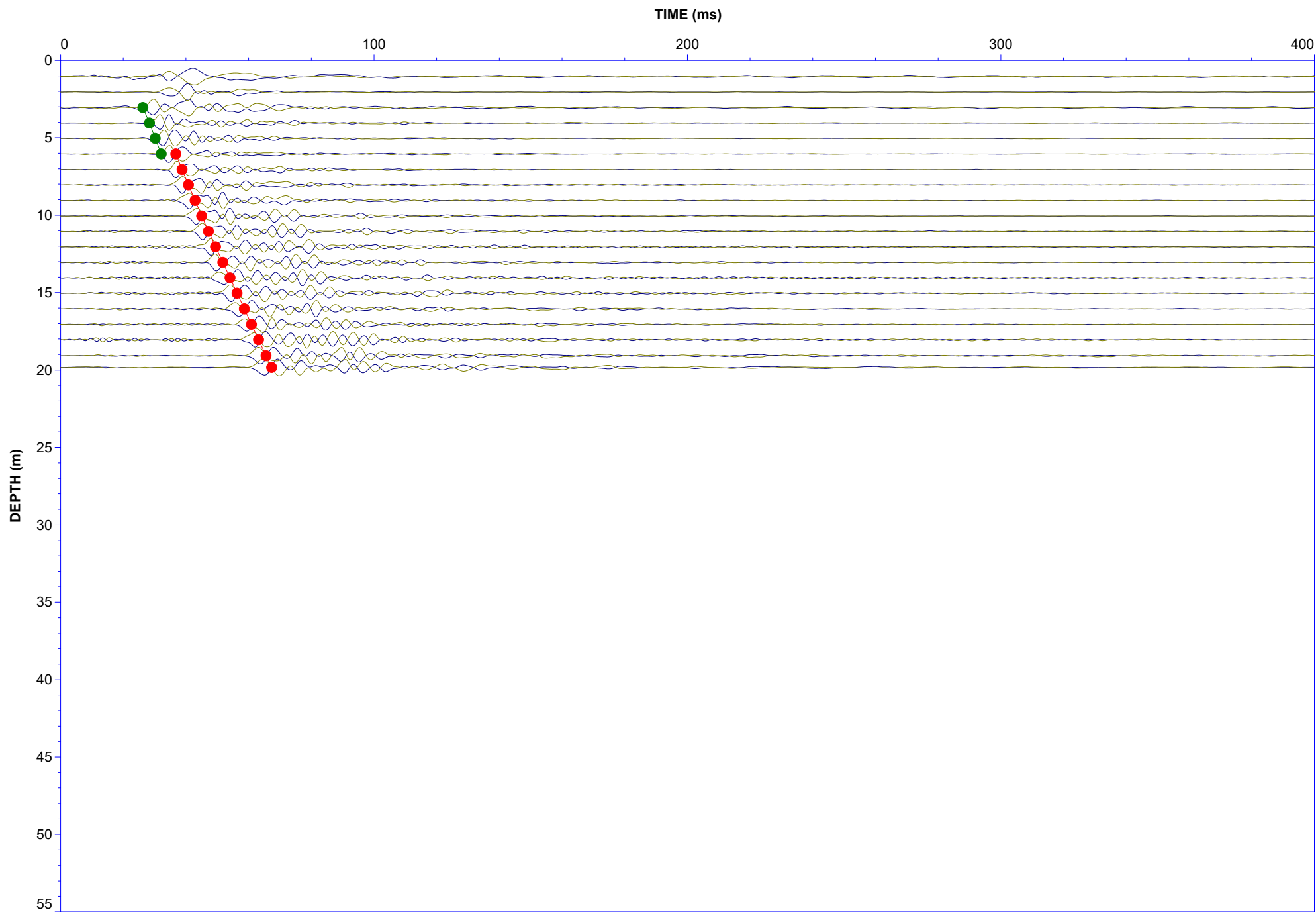
Seismic Source: Beam  
Seismic Offset (m): 3.65  
Source Depth (m): 0.00  
Geophone Offset (m): 0.20

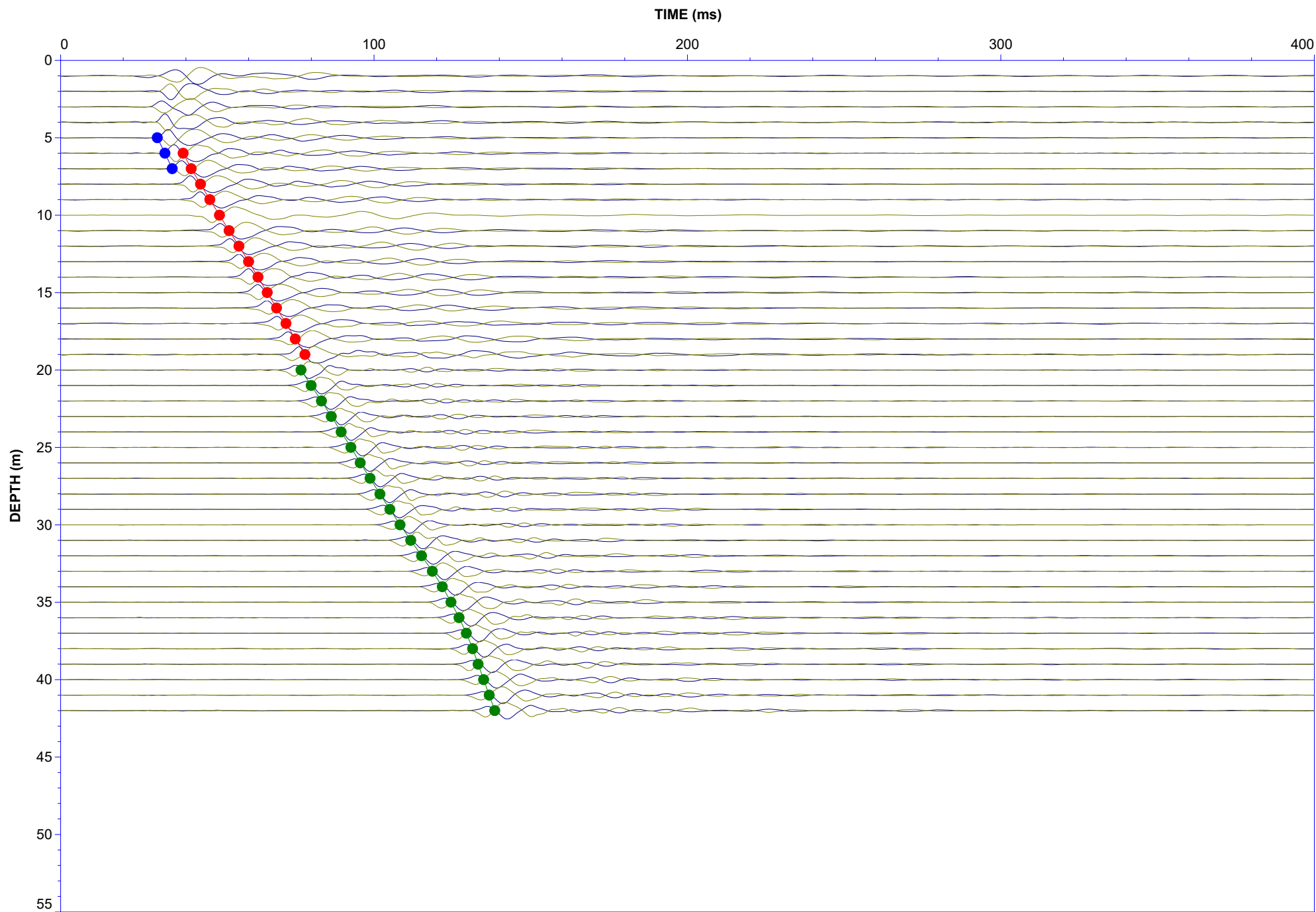
### ***SCPT<sub>u</sub> SHEAR WAVE VELOCITY TEST RESULTS - V<sub>s</sub>***

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
36.00	35.80	35.99	1.00	2.57	387
37.00	36.80	36.98	1.00	2.33	427
38.00	37.80	37.98	1.00	1.98	504
39.00	38.80	38.97	1.00	1.78	561
40.00	39.80	39.97	1.00	1.77	564
41.00	40.80	40.96	1.00	1.79	558
42.00	41.80	41.96	1.00	1.75	568

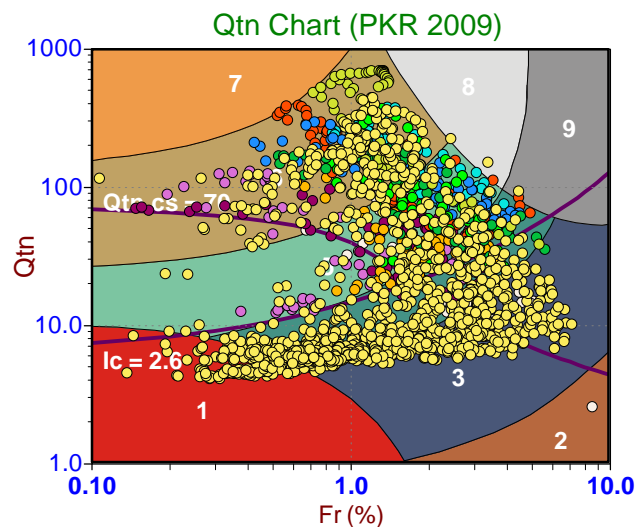
## Seismic Cone Penetration Test Shear Wave ( $V_s$ ) Traces







## Soil Behaviour Type (SBT) Scatter Plots

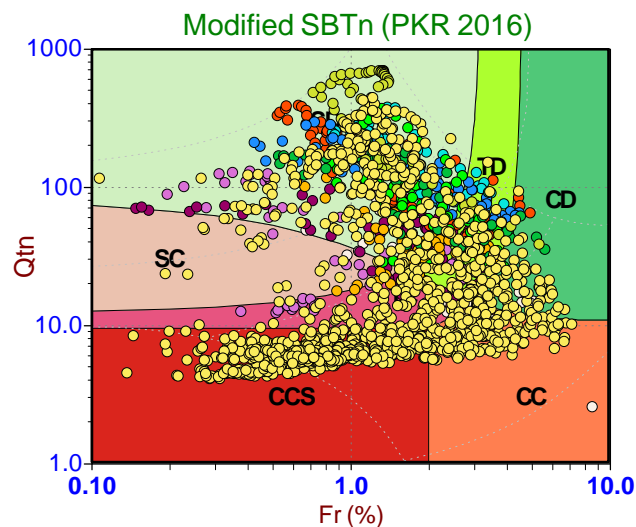


#### Depth Ranges

- >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

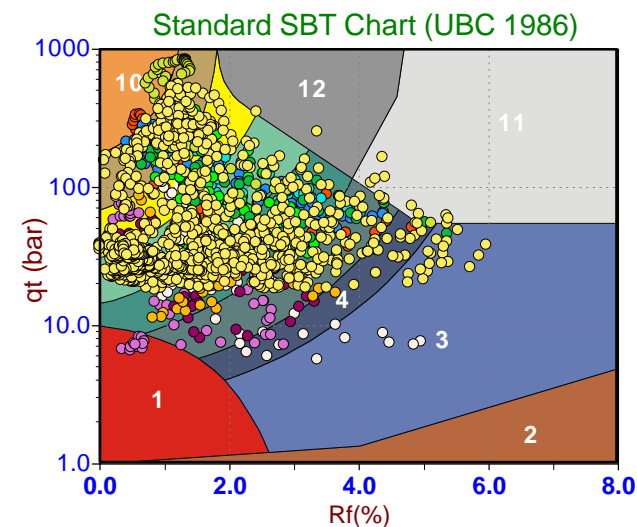
#### Legend

- 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

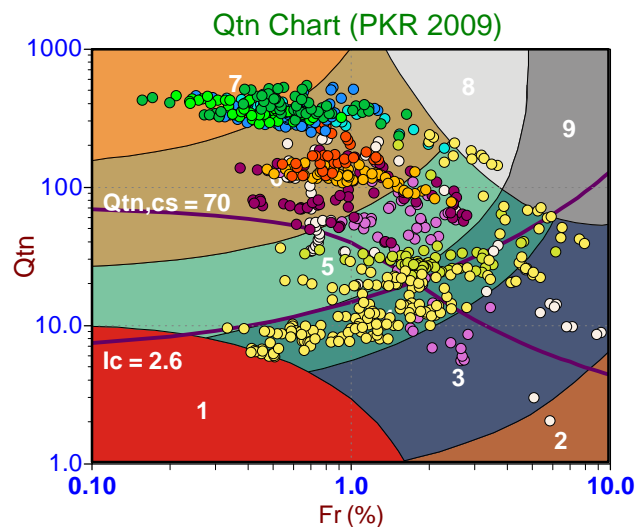
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)



#### Legend

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



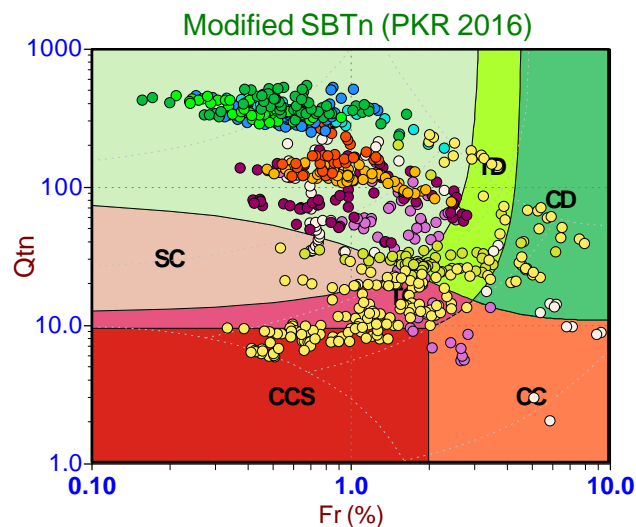


#### Depth Ranges

- >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

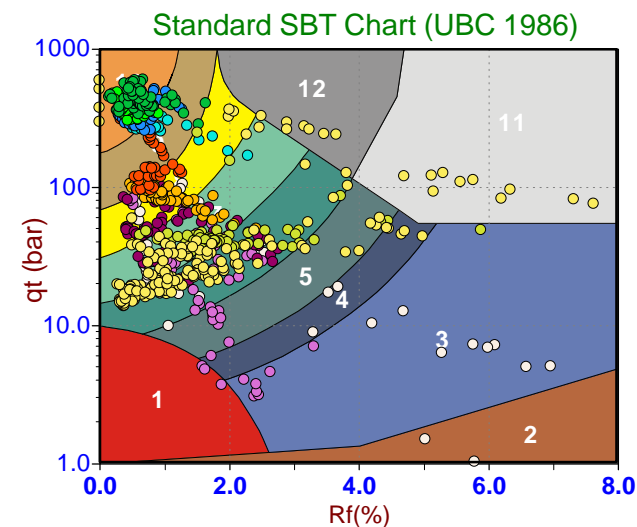
#### Legend

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



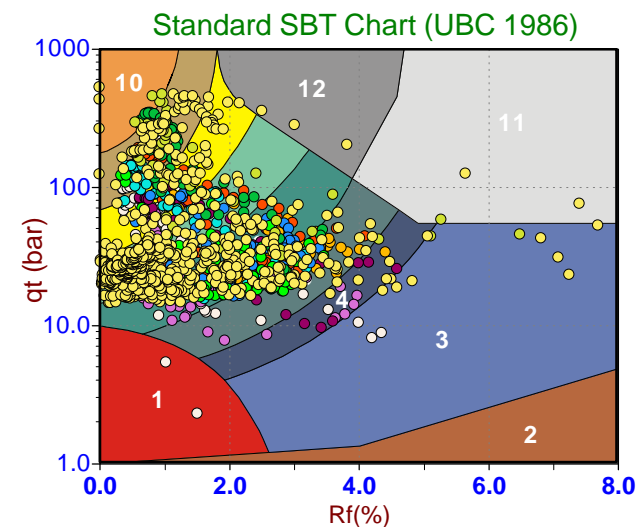
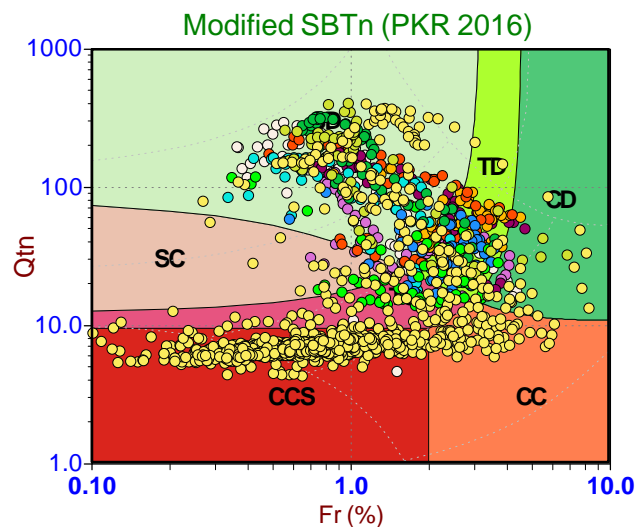
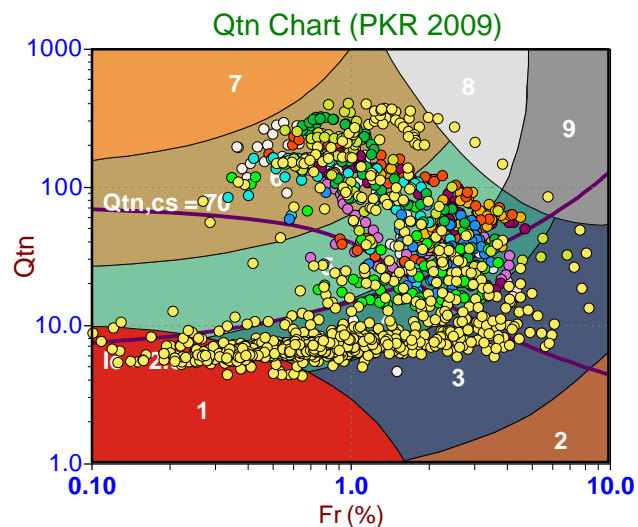
#### Legend

- 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



## 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: 21-05-23424  
 Client: Golder Associates  
 Project: Bradford Bypass  
 Start Date: 20-Dec-2021  
 End Date: 22-Dec-2021

### CPTu PORE PRESSURE DISSIPATION SUMMARY

Sounding ID	File Name	Cone Area (cm <sup>2</sup> )	Duration (s)	Test Depth (m)	Equilibrium Pore Pressure U <sub>eq</sub> (m)	Estimated Equilibrium Pore Pressure U <sub>eq</sub> (m)	Estimated Phreatic Surface (m)	t <sub>50</sub> <sup>1</sup> (s)	Assumed Rigidity Index (I <sub>r</sub> )	C <sub>h</sub> <sup>2</sup> (cm <sup>2</sup> /min)
SCPT21-HSE-01	21-05-23424_SP-HSE-01	15	930	1.000	0.0		1.0			
SCPT21-HSE-01	21-05-23424_SP-HSE-01	15	300	3.000	1.7		1.3	17	100	41.3
SCPT21-HSE-01	21-05-23424_SP-HSE-01	15	300	11.025	Not Achieved					
SCPT21-HSE-01	21-05-23424_SP-HSE-01	15	5580	18.300	Not Achieved	16.6	1.7	497	100	1.4
SCPT21-HSE-01	21-05-23424_SP-HSE-01	15	1000	30.500	28.8		1.7	37	100	18.9
SCPT21-HSE-01	21-05-23424_SP-HSE-01	15	355	43.000	Not Achieved					
SCPT21-HSE-01	21-05-23424_SP-HSE-01	15	1100	50.000	Not Achieved	48.3	1.7	195	100	3.6
SCPT21-HRW-01B	21-05-23424_SP-HRW-01B	15	600	3.050	2.2		0.9			
SCPT21-HRW-01B	21-05-23424_SP-HRW-01B	15	300	9.050	7.9		1.2			
SCPT21-HRW-01B	21-05-23424_SP-HRW-01B	15	1500	15.250		14.4	0.9	1113	100	0.6
SCPT21-HRW-01B	21-05-23424_SP-HRW-01B	15	620	19.775	20.4		-0.6			
SCPT21-HRW-04	21-05-23424_SP-HRW-04	15	600	3.000	Not Achieved	2.7	0.3	196	100	3.6
SCPT21-HRW-04	21-05-23424_SP-HRW-04	15	600	7.625	7.3		0.3	9	100	82.2
SCPT21-HRW-04	21-05-23424_SP-HRW-04	15	720	18.300	Not Achieved	18.0	0.3	228	100	3.1
SCPT21-HRW-04	21-05-23424_SP-HRW-04	15	2250	30.500	Not Achieved	30.2	0.3	1668	100	0.4

1. Time is relative to where umax occurred.

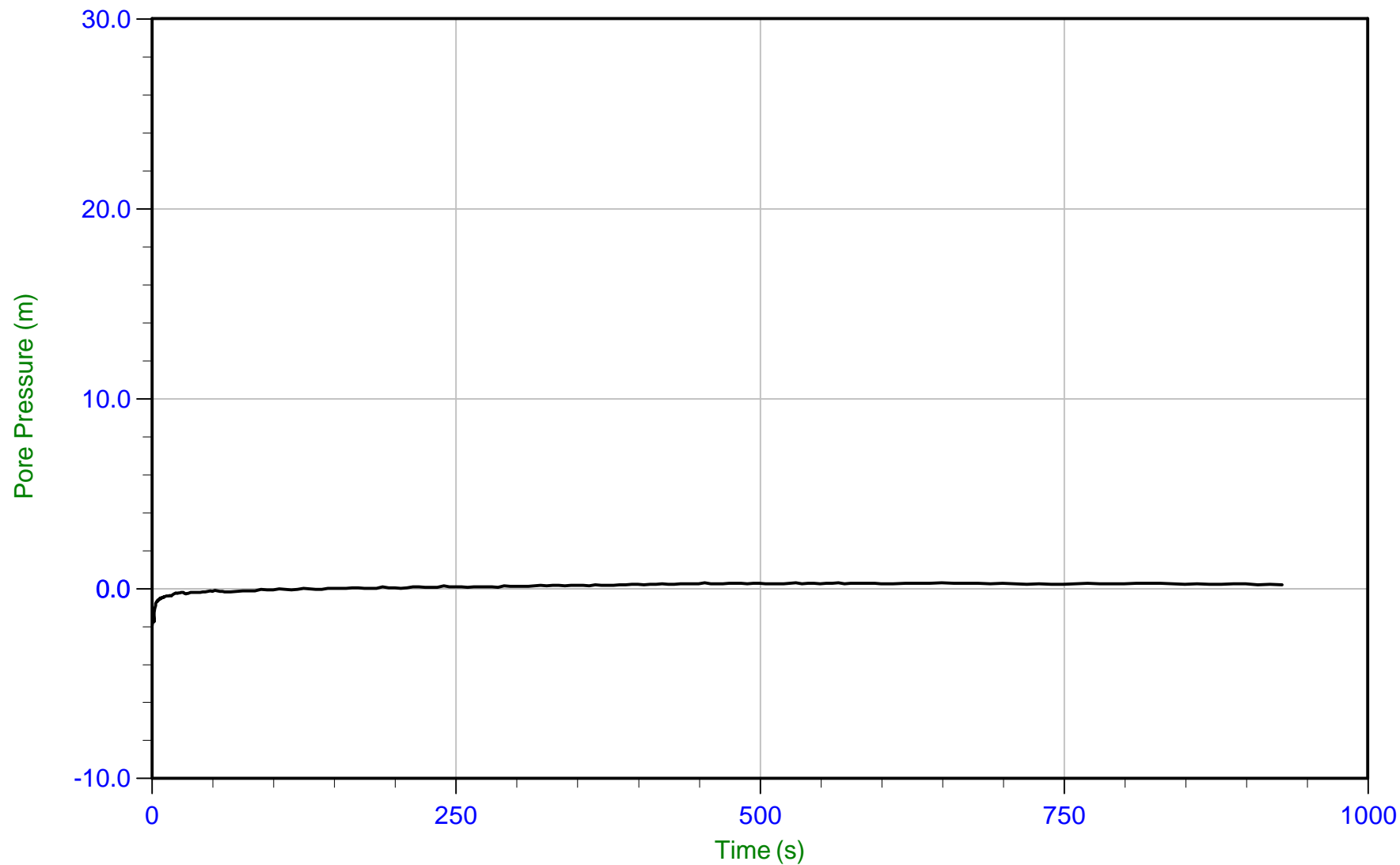
2. Houlsby and Teh, 1991.



*Golder*

Job No: 21-05-23424  
Date: 12/20/2021 13:32  
Site: East Holland River

Sounding: SCPT21-HSE-01  
Cone: 765:T1500F15U35 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 21-05-23424\_SP-HSE-01.PPF  
Depth: 1.000 m / 3.281 ft  
Duration: 930.0 s

u Min: -1.9 m  
u Max: 0.3 m  
u Final: 0.2 m

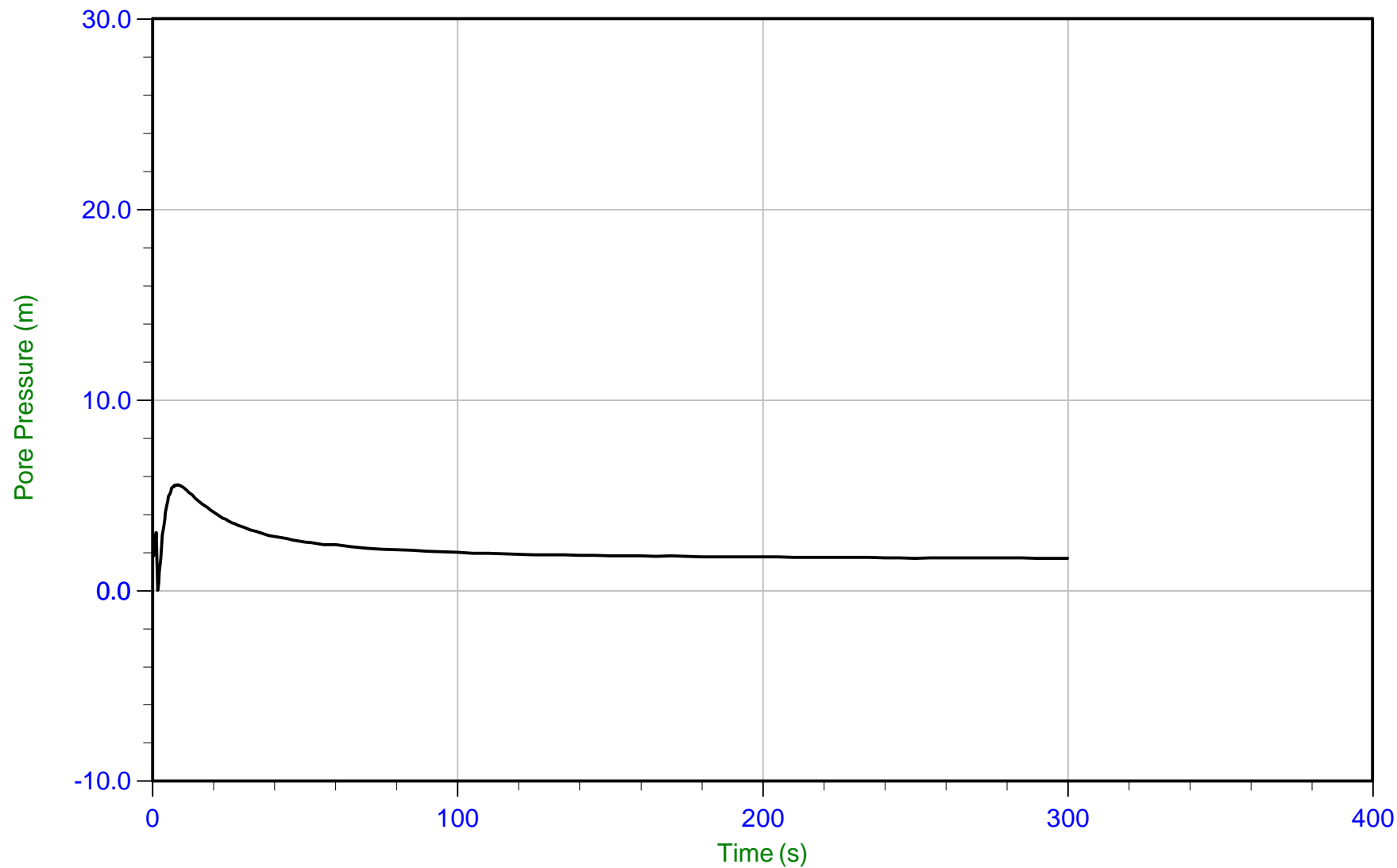
WT: 1.000 m / 3.281 ft  
Ueq: 0.0 m



*Golder*

Job No: 21-05-23424  
Date: 12/20/2021 13:32  
Site: East Holland River

Sounding: SCPT21-HSE-01  
Cone: 765:T1500F15U35 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 21-05-23424\_SP-HSE-01.PPF  
Depth: 3.000 m / 9.842 ft  
Duration: 300.0 s

u Min: 0.0 m  
u Max: 5.6 m  
u Final: 1.7 m

WT: 1.300 m / 4.265 ft  
Ueq: 1.7 m  
U(50): 3.63 m

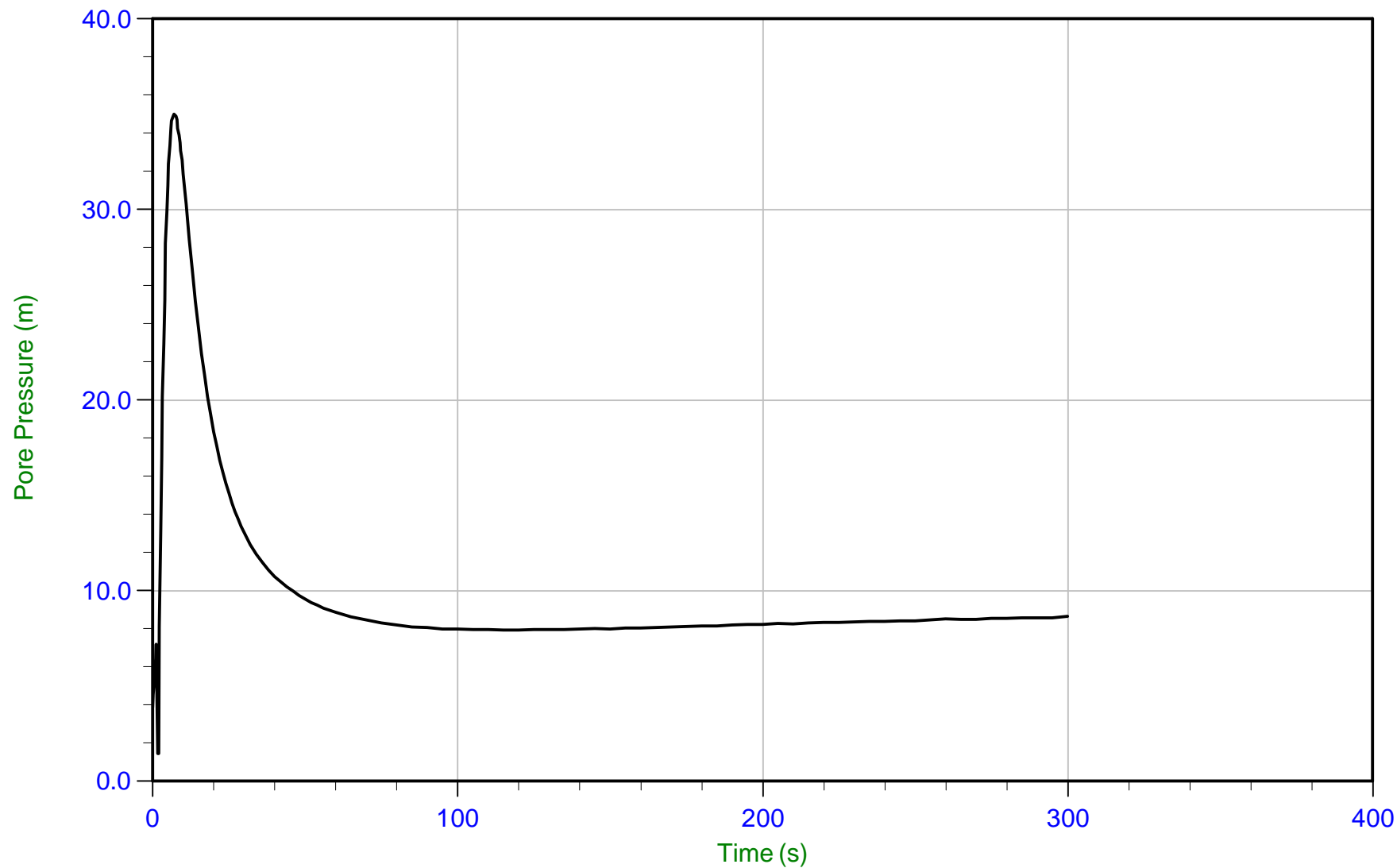
T(50): 17.0 s  
Ir: 100  
Ch: 41.3 cm<sup>2</sup>/min



*Golder*

Job No: 21-05-23424  
Date: 12/20/2021 13:32  
Site: East Holland River

Sounding: SCPT21-HSE-01  
Cone: 765:T1500F15U35 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 21-05-23424\_SP-HSE-01.PPF  
Depth: 11.025 m / 36.171 ft  
Duration: 300.0 s

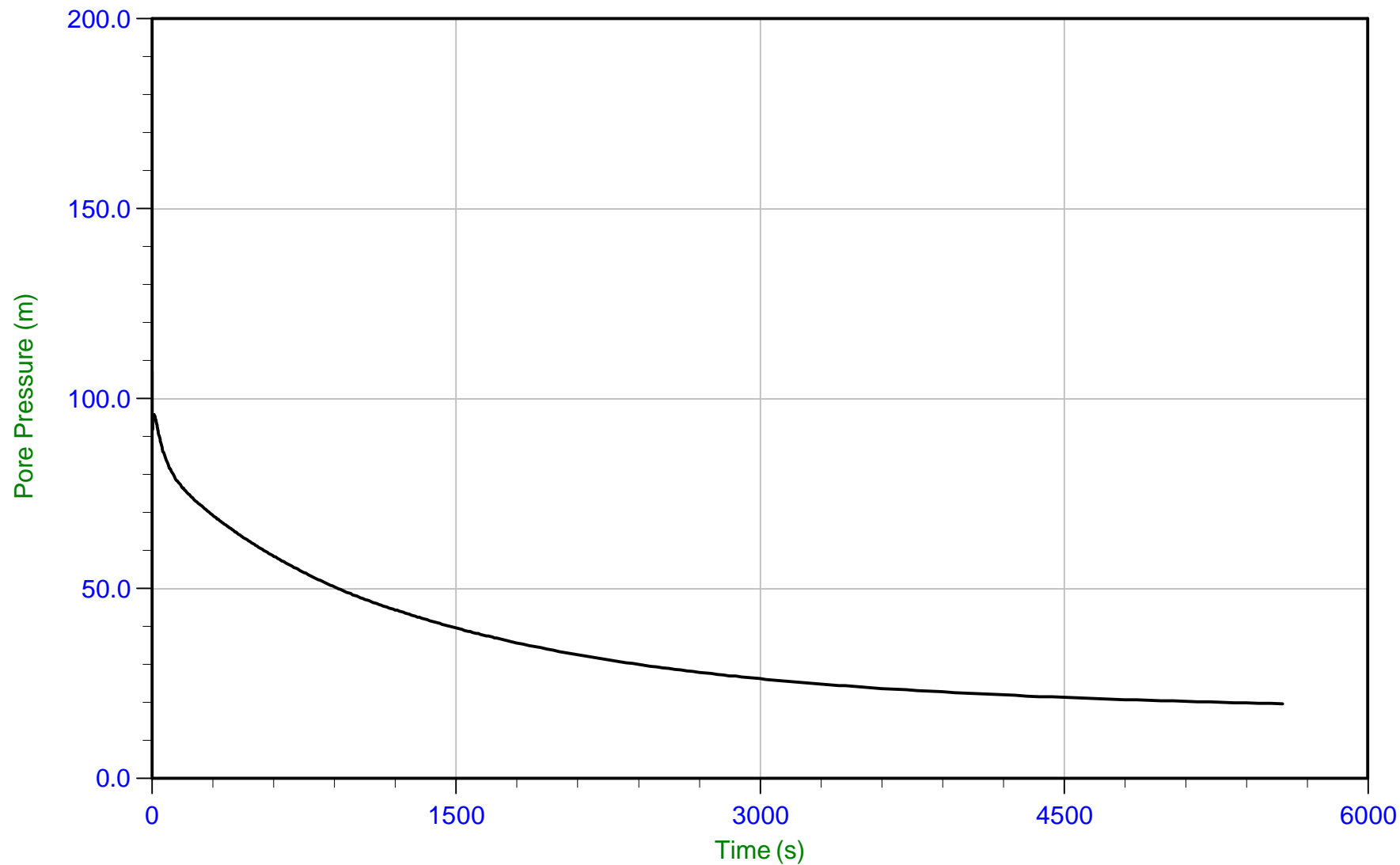
u Min: 1.5 m  
u Max: 35.0 m  
u Final: 8.6 m



*Golder*

Job No: 21-05-23424  
Date: 12/20/2021 13:32  
Site: East Holland River

Sounding: SCPT21-HSE-01  
Cone: 765:T1500F15U35 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 21-05-23424\_SP-HSE-01.PPF  
Depth: 18.300 m / 60.039 ft  
Duration: 5580.0 s

u Min: 19.7 m  
u Max: 107.1 m  
u Final: 19.7 m

WT: 1.700 m / 5.577 ft  
Ueq: 16.6 m  
U(50): 61.84 m

T(50): 497.3 s  
Ir: 100  
Ch: 1.4 cm<sup>2</sup>/min

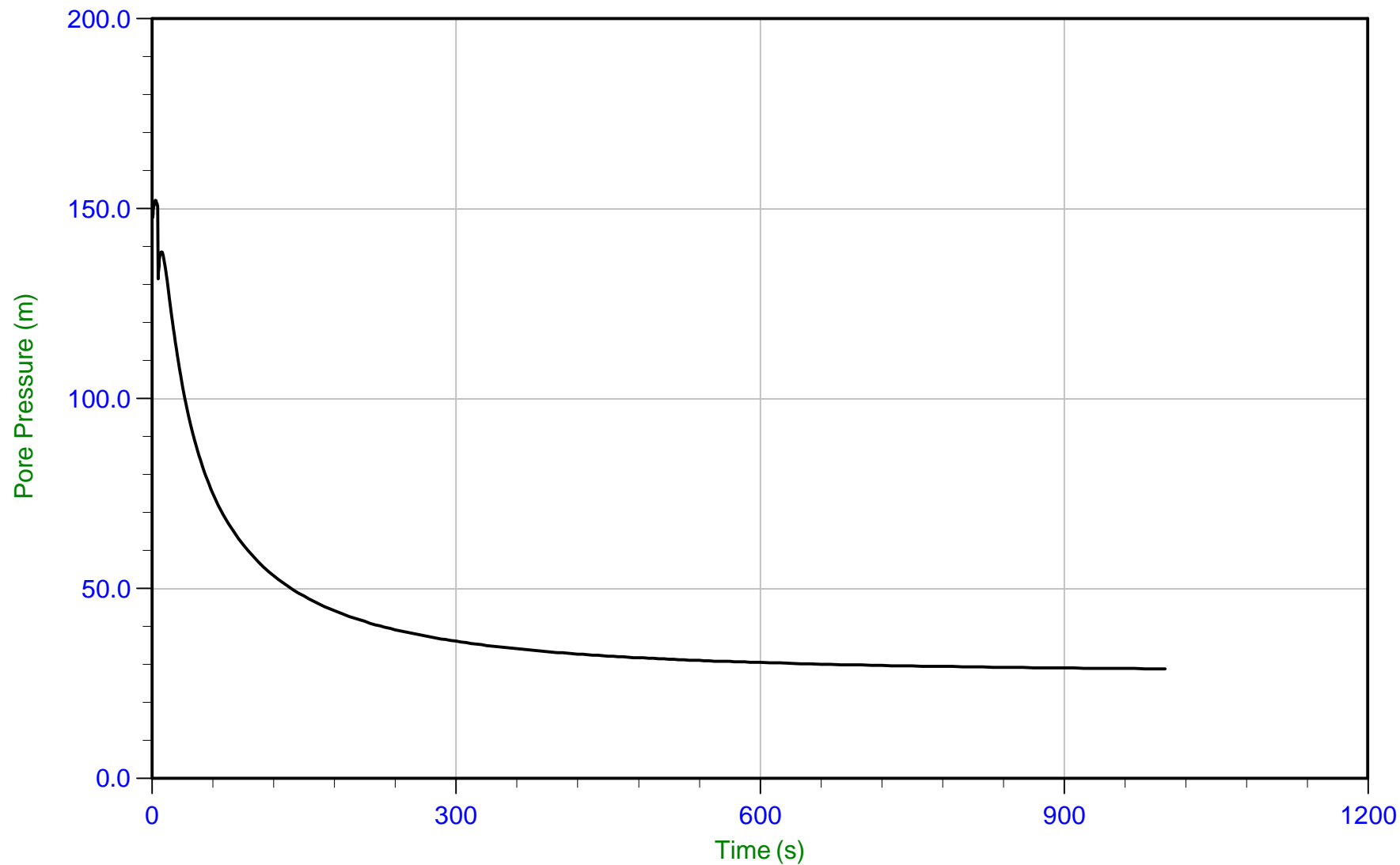




*Golder*

Job No: 21-05-23424  
Date: 12/20/2021 13:32  
Site: East Holland River

Sounding: SCPT21-HSE-01  
Cone: 765:T1500F15U35 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 21-05-23424\_SP-HSE-01.PPF  
Depth: 30.500 m / 100.064 ft  
Duration: 1000.0 s

u Min: 28.9 m  
u Max: 152.2 m  
u Final: 28.9 m

WT: 1.700 m / 5.577 ft  
Ueq: 28.8 m  
U(50): 90.48 m

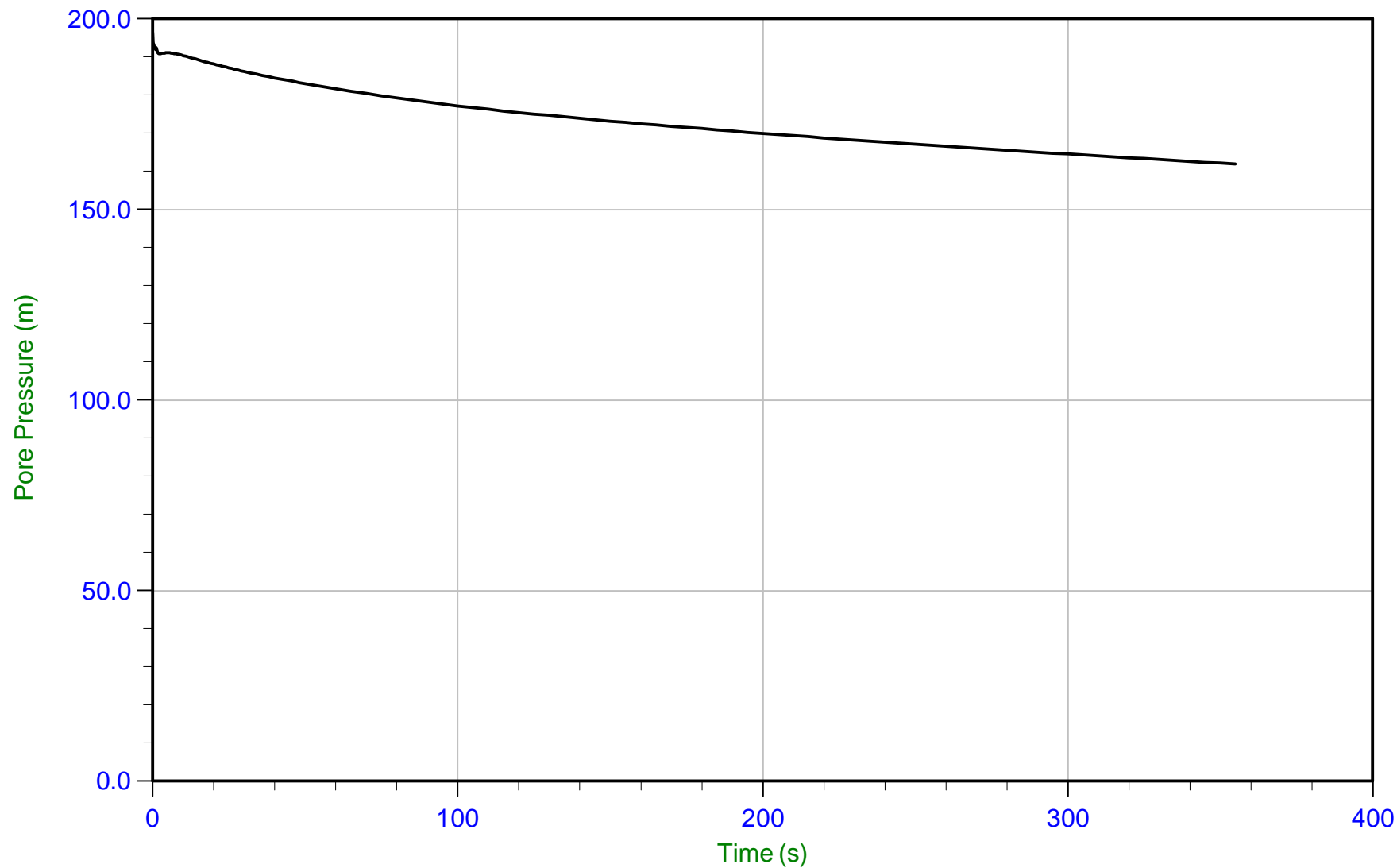
T(50): 37.2 s  
Ir: 100  
Ch: 18.9 cm<sup>2</sup>/min



*Golder*

Job No: 21-05-23424  
Date: 12/20/2021 13:32  
Site: East Holland River

Sounding: SCPT21-HSE-01  
Cone: 765:T1500F15U35 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 21-05-23424\_SP-HSE-01.PPF  
Depth: 43.000 m / 141.074 ft  
Duration: 355.0 s

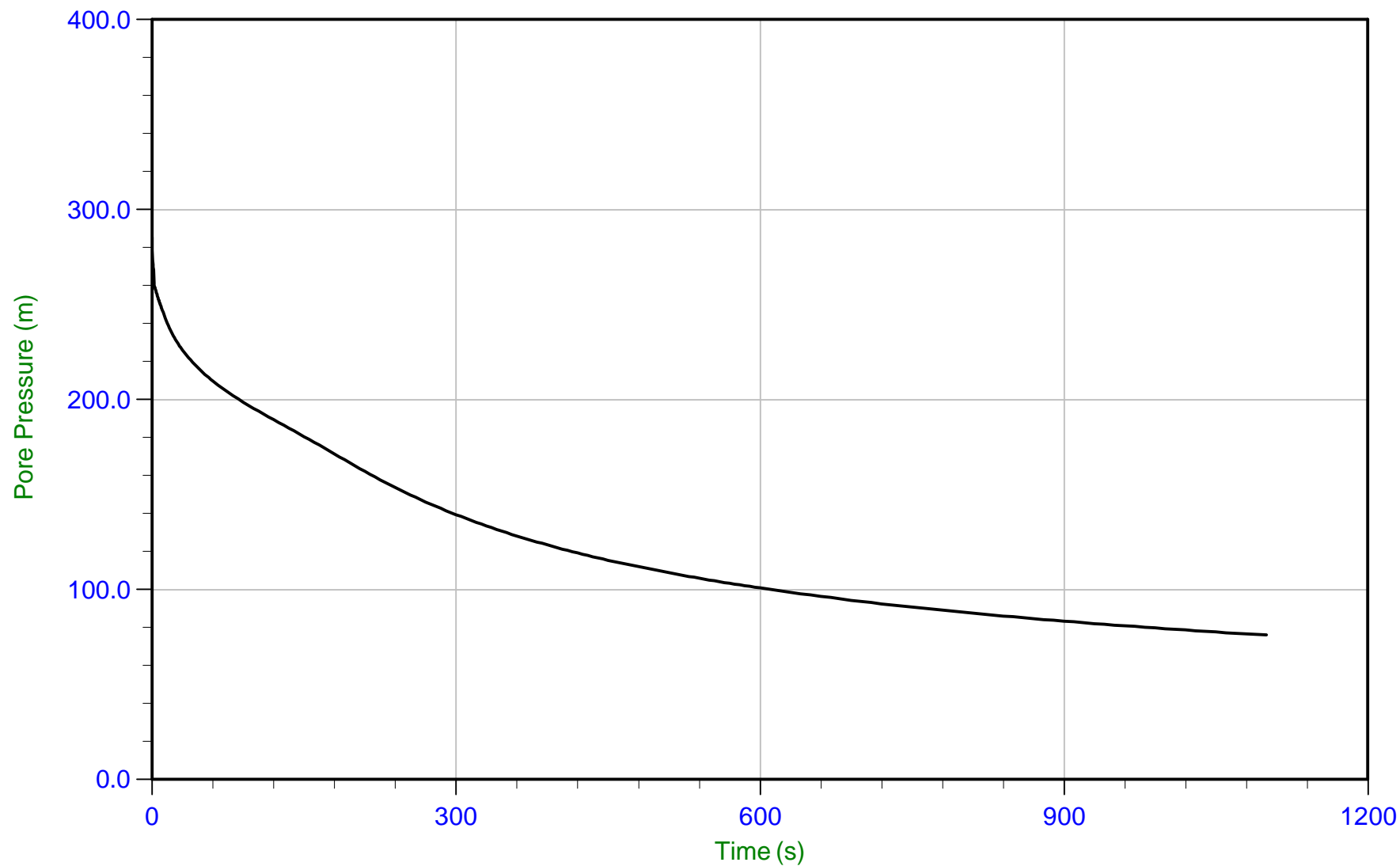
u Min: 161.9 m  
u Max: 196.5 m  
u Final: 161.9 m



*Golder*

Job No: 21-05-23424  
Date: 12/20/2021 13:32  
Site: East Holland River

Sounding: SCPT21-HSE-01  
Cone: 765:T1500F15U35 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 21-05-23424\_SP-HSE-01.PPF  
Depth: 50.000 m / 164.040 ft  
Duration: 1100.0 s

u Min: 76.0 m  
u Max: 285.4 m  
u Final: 76.0 m

WT: 1.700 m / 5.577 ft  
Ueq: 48.3 m  
U(50): 166.84 m

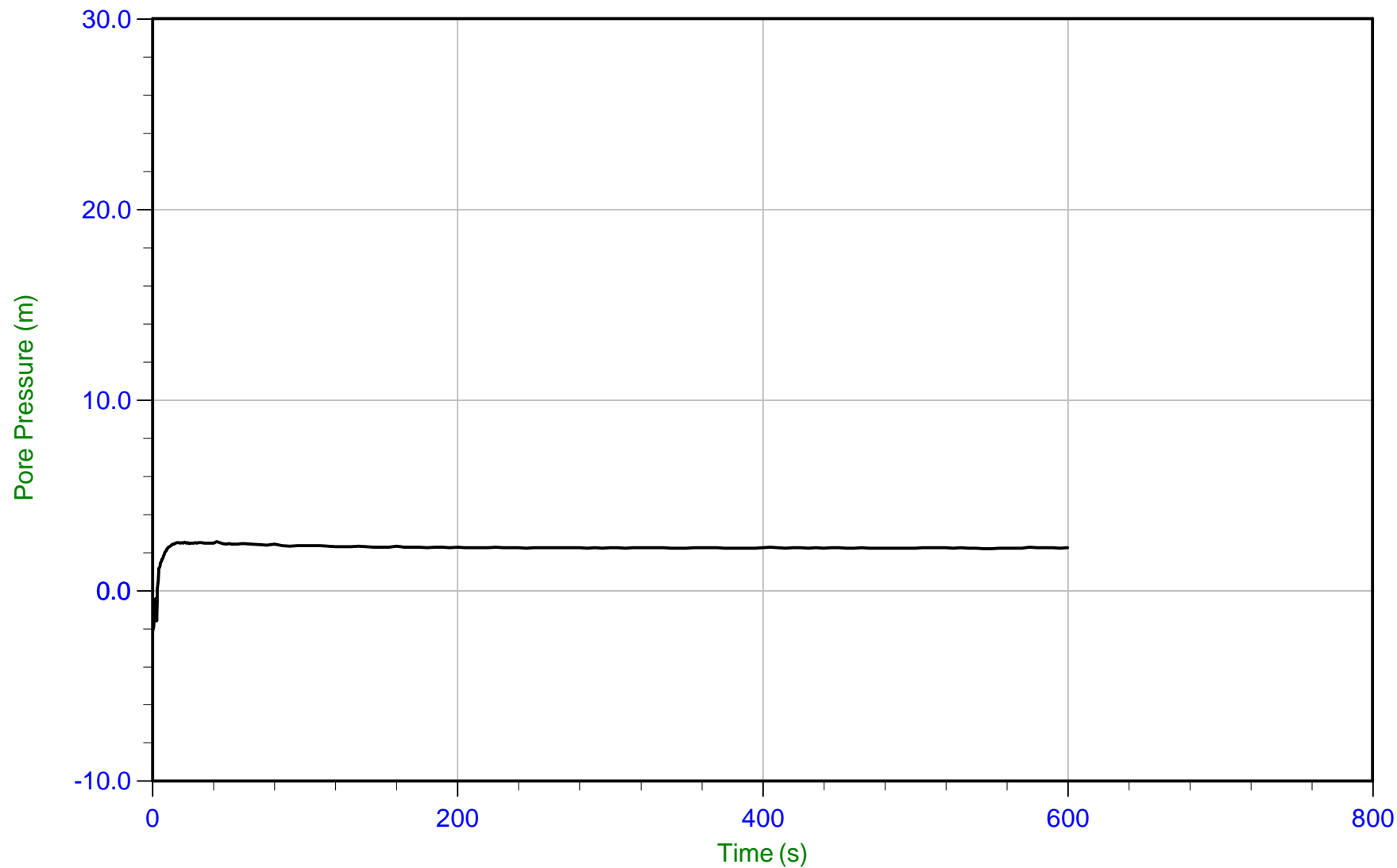
T(50): 194.7 s  
Ir: 100  
Ch: 3.6 cm<sup>2</sup>/min



*Golder*

Job No: 21-05-23424  
Date: 12/22/2021 09:45  
Site: East Holland River

Sounding: SCPT21-HRW-01B  
Cone: 765:T1500F15U35 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 21-05-23424\_SP-HRW-01B.PPF  
Depth: 3.050 m / 10.006 ft  
Duration: 600.0 s

u Min: -2.1 m  
u Max: 2.6 m  
u Final: 2.2 m

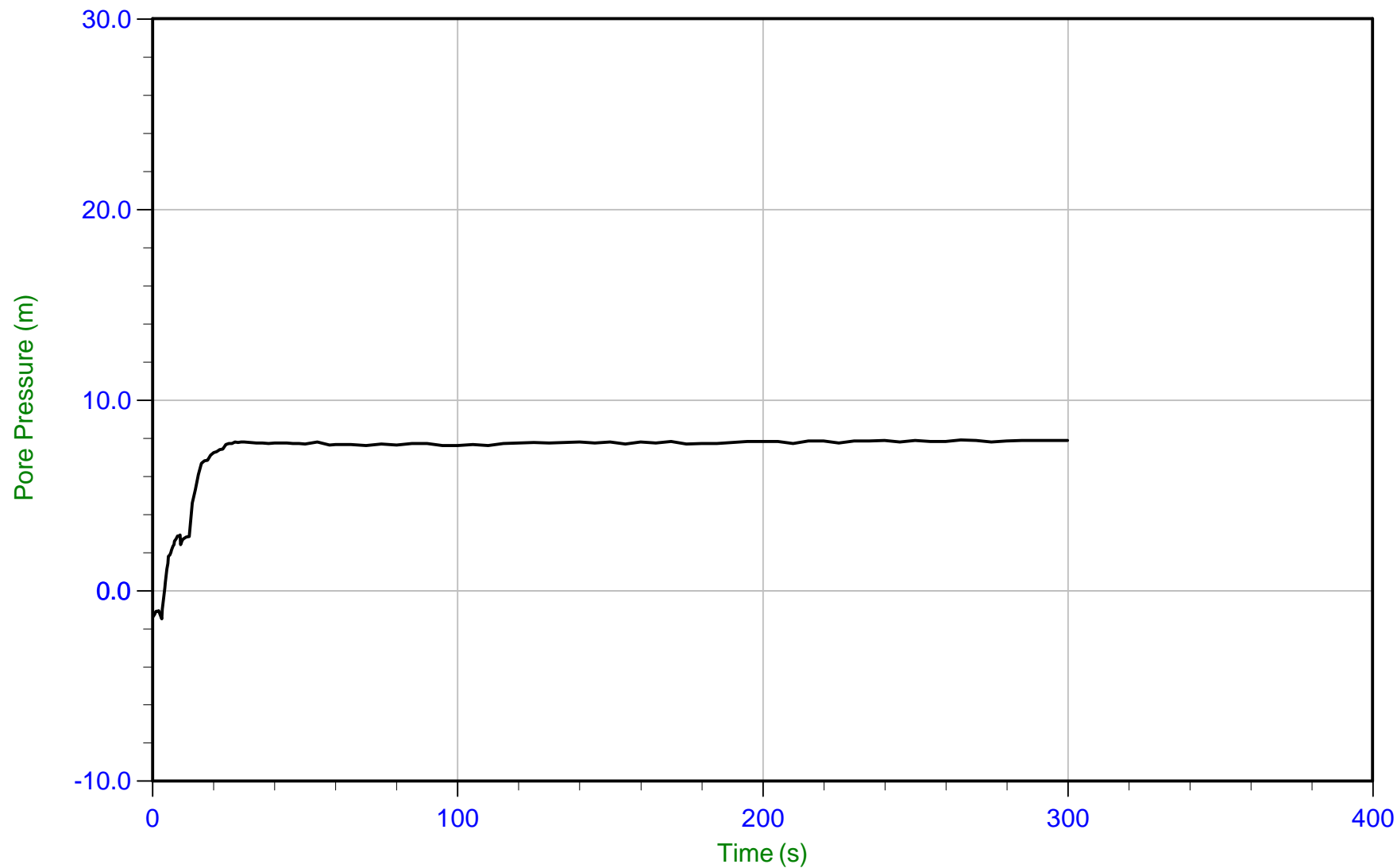
WT: 0.850 m / 2.789 ft  
Ueq: 2.2 m



*Golder*

Job No: 21-05-23424  
Date: 12/22/2021 09:45  
Site: East Holland River

Sounding: SCPT21-HRW-01B  
Cone: 765:T1500F15U35 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 21-05-23424\_SP-HRW-01B.PPF  
Depth: 9.050 m / 29.691 ft  
Duration: 300.0 s

u Min: -1.5 m  
u Max: 7.9 m  
u Final: 7.9 m

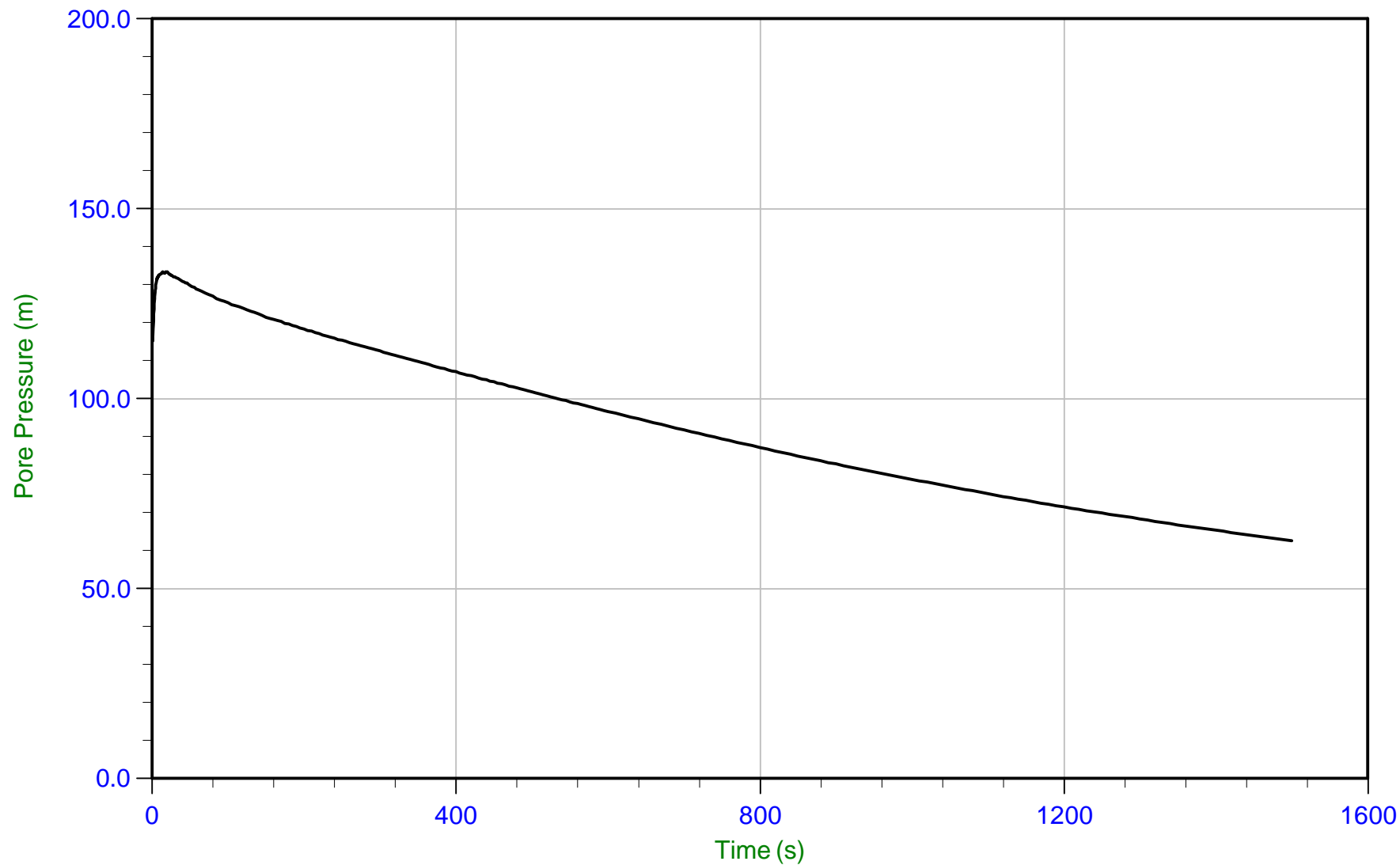
WT: 1.150 m / 3.773 ft  
Ueq: 7.9 m



*Golder*

Job No: 21-05-23424  
Date: 12/22/2021 09:45  
Site: East Holland River

Sounding: SCPT21-HRW-01B  
Cone: 765:T1500F15U35 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 21-05-23424\_SP-HRW-01B.PPF  
Depth: 15.250 m / 50.032 ft  
Duration: 1500.0 s

u Min: 62.6 m  
u Max: 133.4 m  
u Final: 62.6 m

WT: 0.850 m / 2.789 ft  
Ueq: 14.4 m  
U(50): 73.92 m

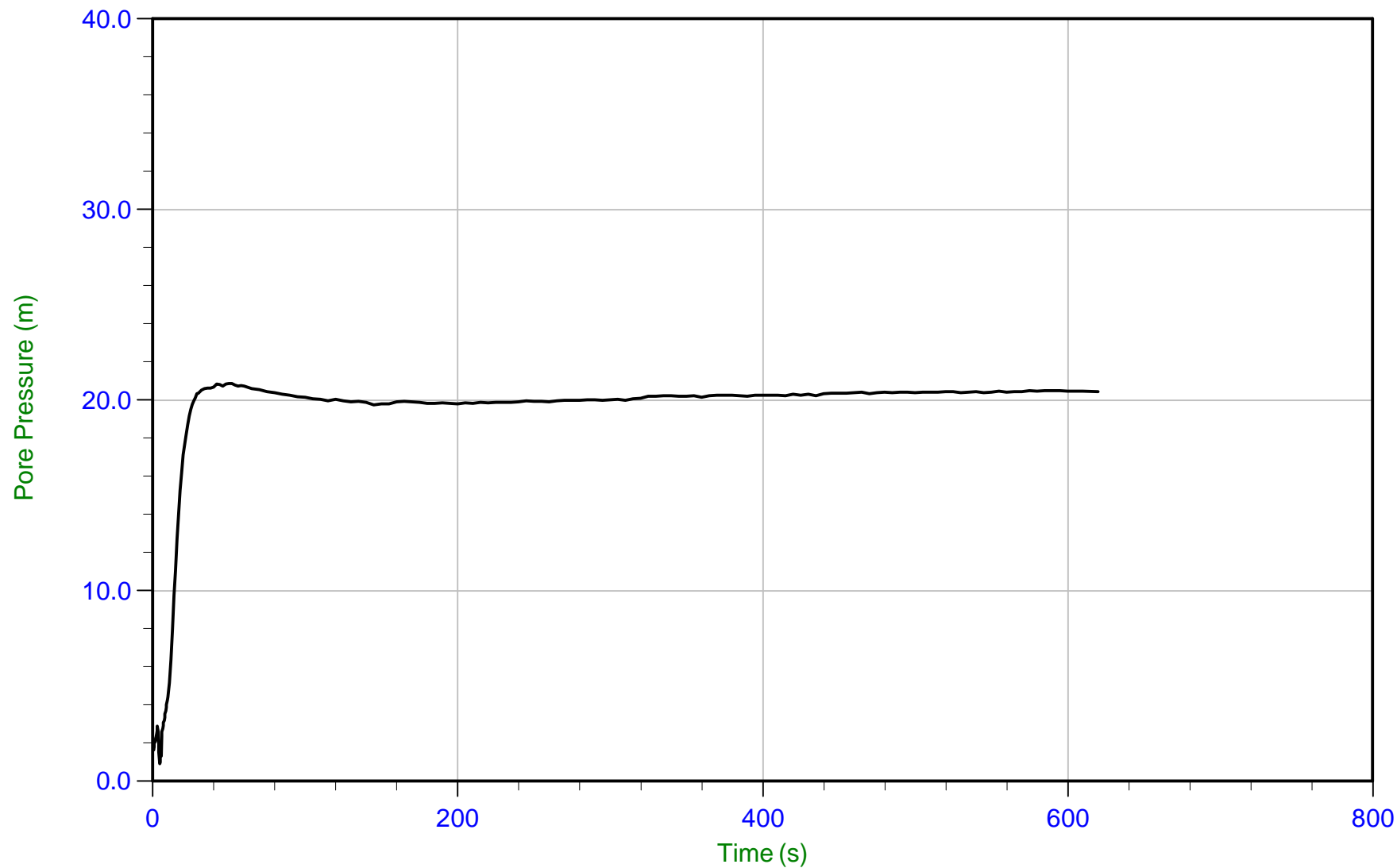
T(50): 1112.7 s  
Ir: 100  
Ch: 0.6 cm<sup>2</sup>/min



*Golder*

Job No: 21-05-23424  
Date: 12/22/2021 09:45  
Site: East Holland River

Sounding: SCPT21-HRW-01B  
Cone: 765:T1500F15U35 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 21-05-23424\_SP-HRW-01B.PPF  
Depth: 19.775 m / 64.878 ft  
Duration: 620.0 s

u Min: 0.9 m  
u Max: 20.9 m  
u Final: 20.4 m

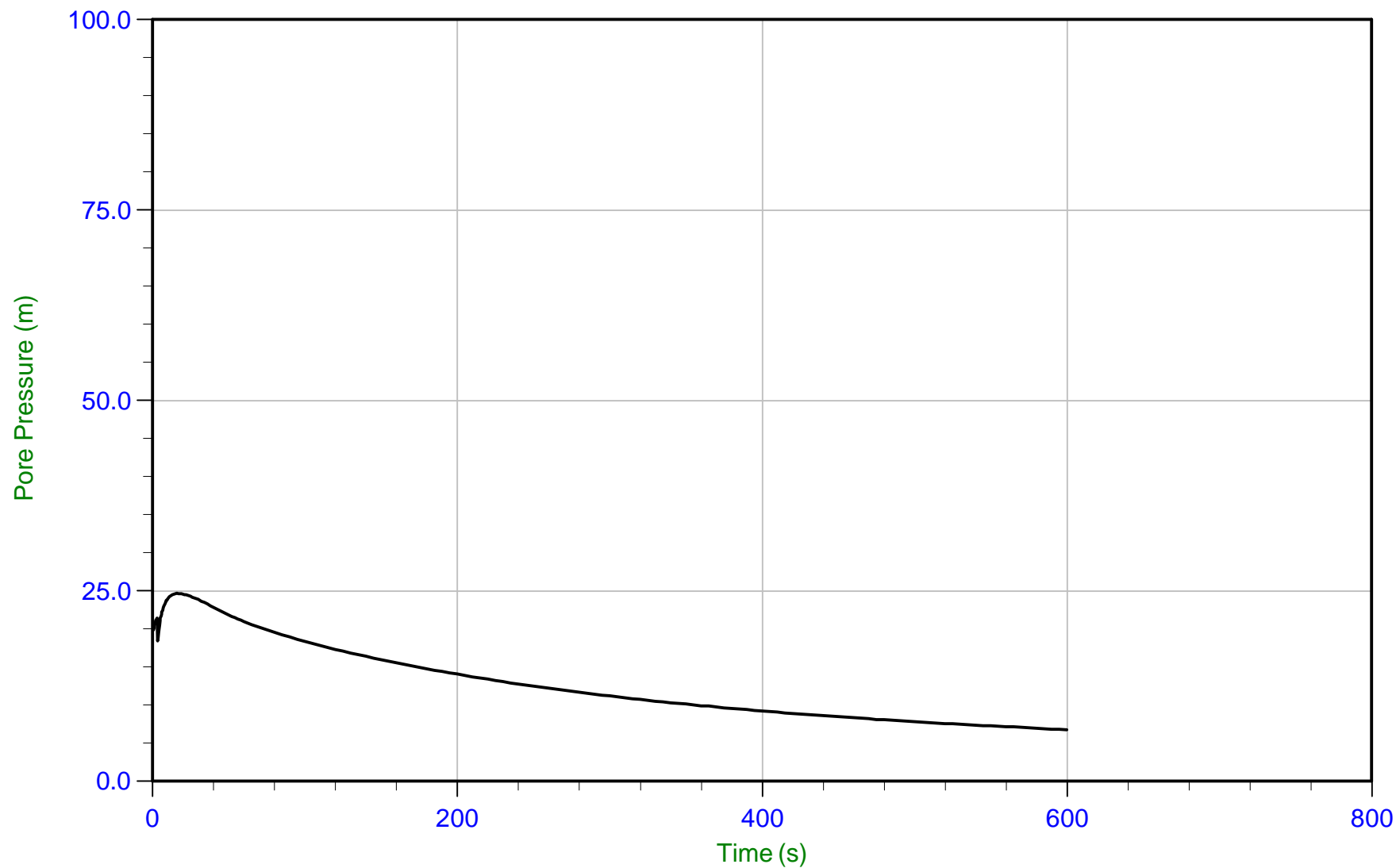
WT: -0.625 m / -2.050 ft  
Ueq: 20.4 m



*Golder*

Job No: 21-05-23424  
Date: 12/21/2021 11:38  
Site: East Holland River

Sounding: SCPT21-HRW-04  
Cone: 765:T1500F15U35 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 21-05-23424\_SP-HRW-04.PPF  
Depth: 3.000 m / 9.842 ft  
Duration: 600.0 s

u Min: 6.8 m  
u Max: 24.7 m  
u Final: 6.8 m

WT: 0.325 m / 1.066 ft  
Ueq: 2.7 m  
U(50): 13.68 m

T(50): 195.8 s  
Ir: 100  
Ch: 3.6 cm<sup>2</sup>/min

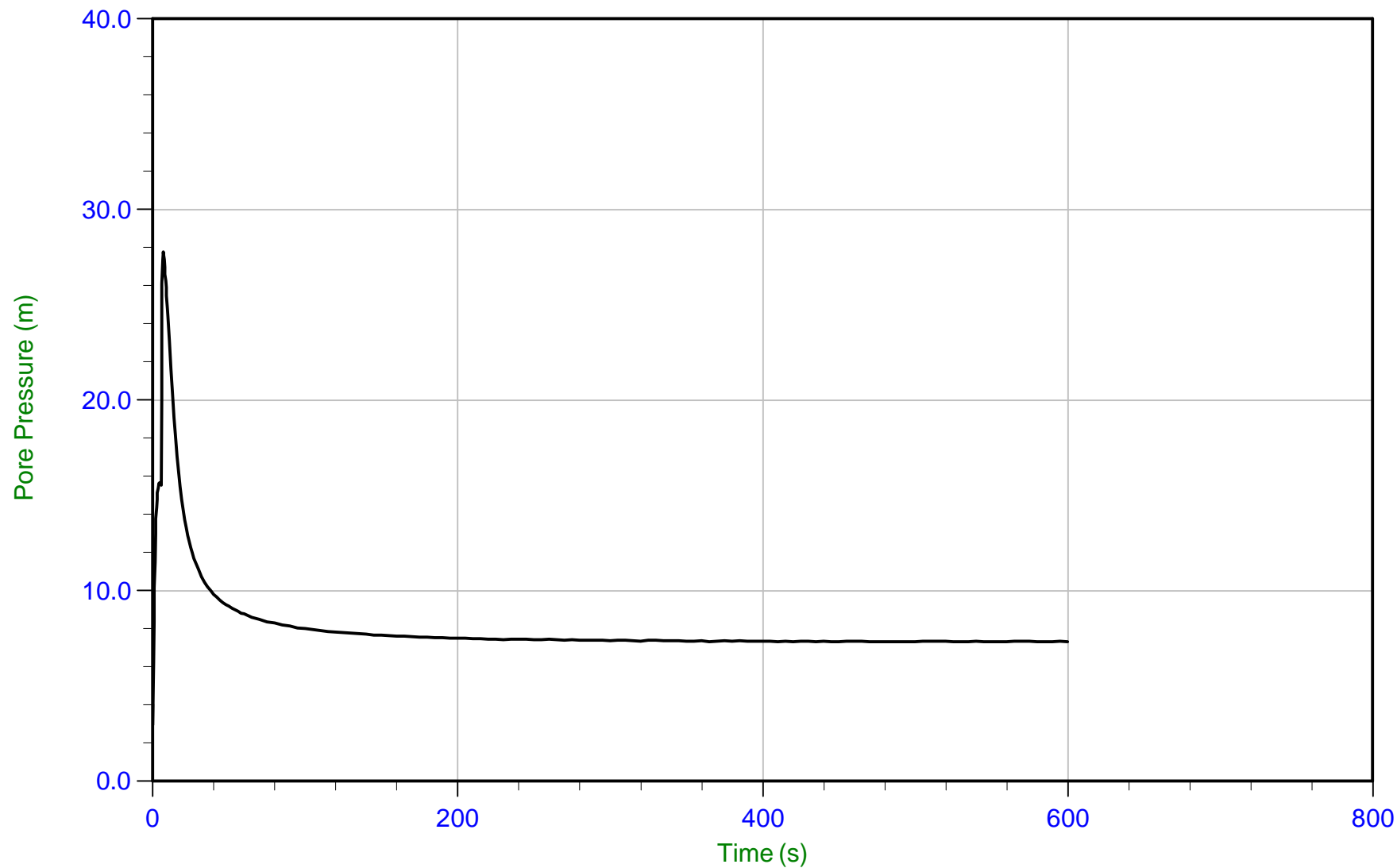




*Golder*

Job No: 21-05-23424  
Date: 12/21/2021 11:38  
Site: East Holland River

Sounding: SCPT21-HRW-04  
Cone: 765:T1500F15U35 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 21-05-23424\_SP-HRW-04.PPF  
Depth: 7.625 m / 25.016 ft  
Duration: 600.0 s

u Min: 3.0 m  
u Max: 27.8 m  
u Final: 7.3 m

WT: 0.325 m / 1.066 ft  
Ueq: 7.3 m  
U(50): 17.53 m

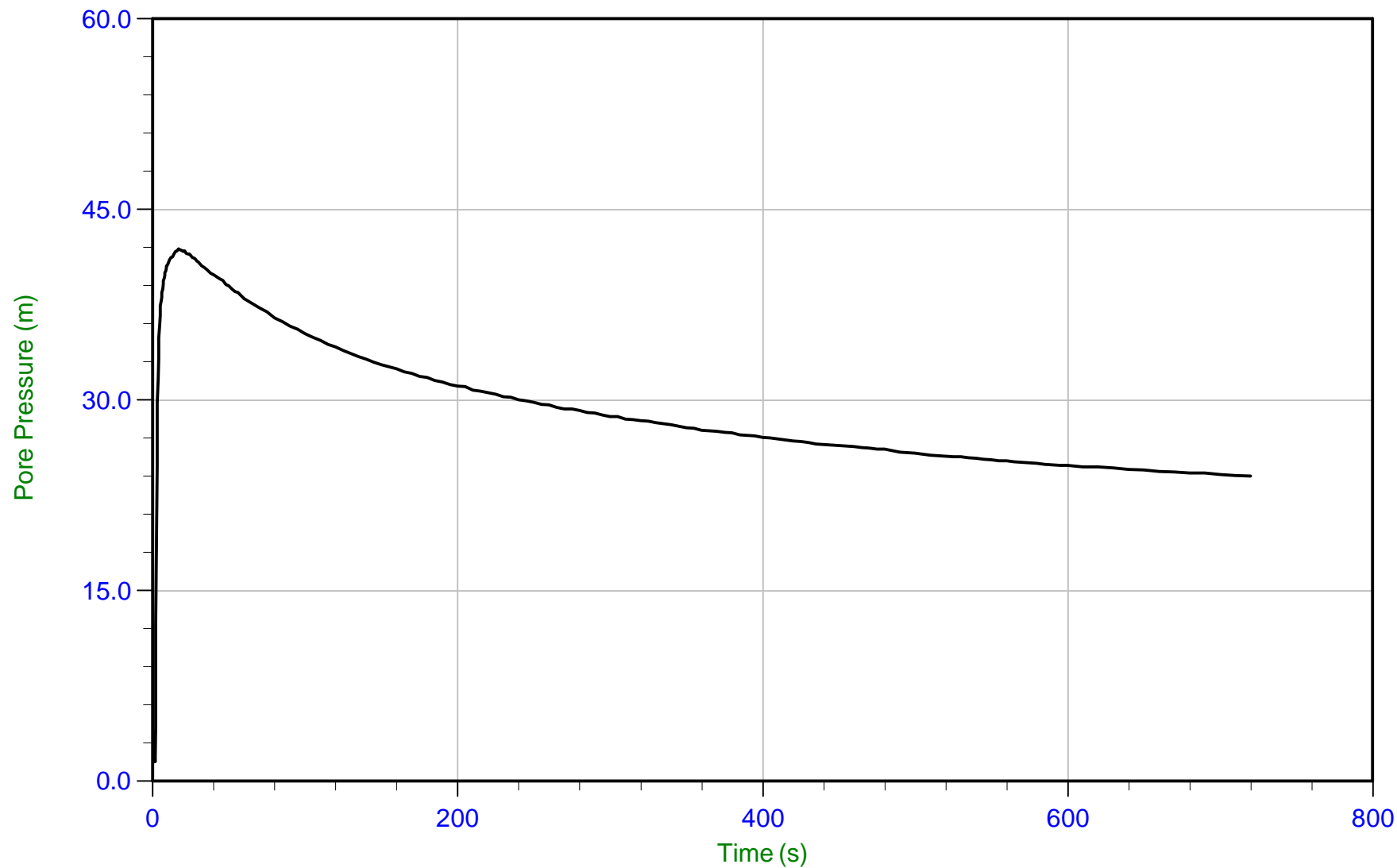
T(50): 8.5 s  
Ir: 100  
Ch: 82.2 cm<sup>2</sup>/min



*Golder*

Job No: 21-05-23424  
Date: 12/21/2021 11:38  
Site: East Holland River

Sounding: SCPT21-HRW-04  
Cone: 765:T1500F15U35 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 21-05-23424\_SP-HRW-04.PPF  
Depth: 18.300 m / 60.039 ft  
Duration: 720.0 s

u Min: 1.4 m  
u Max: 41.9 m  
u Final: 24.0 m

WT: 0.325 m / 1.066 ft  
Ueq: 18.0 m  
U(50): 29.93 m

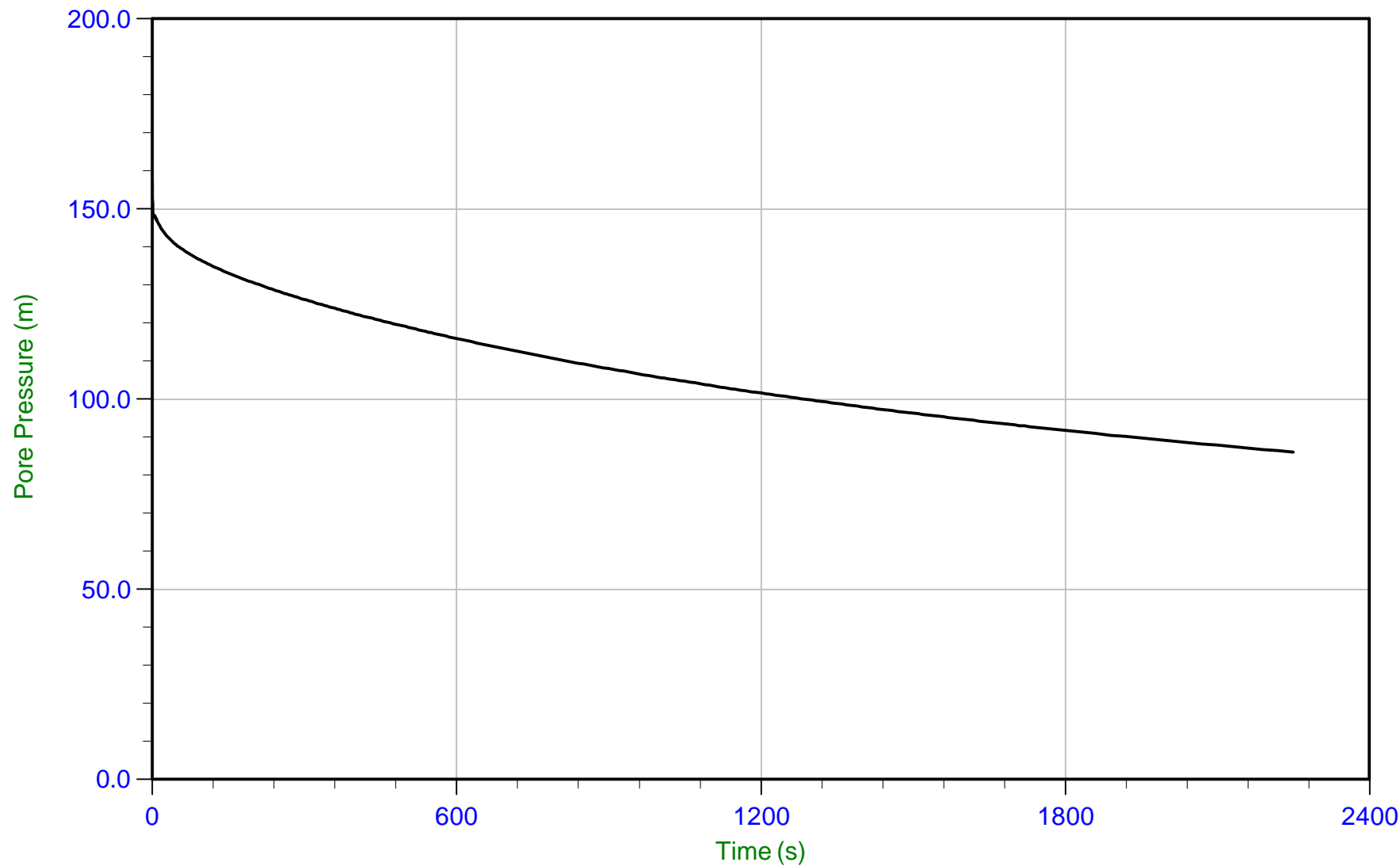
T(50): 228.5 s  
Ir: 100  
Ch: 3.1 cm<sup>2</sup>/min



*Golder*

Job No: 21-05-23424  
Date: 12/21/2021 11:38  
Site: East Holland River

Sounding: SCPT21-HRW-04  
Cone: 765:T1500F15U35 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 21-05-23424\_SP-HRW-04.PPF  
Depth: 30.500 m / 100.064 ft  
Duration: 2250.0 s

u Min: 86.1 m  
u Max: 157.2 m  
u Final: 86.1 m

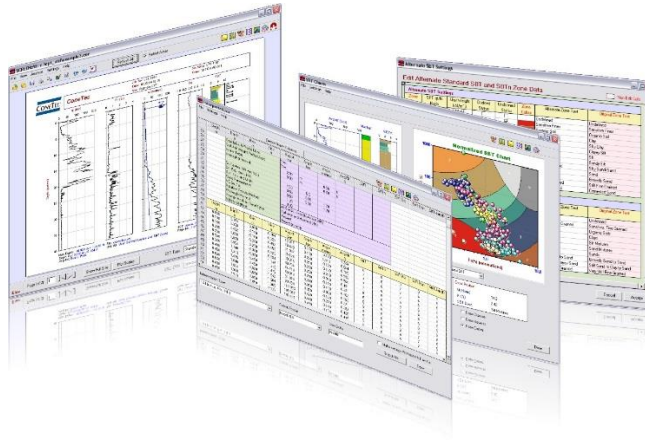
WT: 0.325 m / 1.066 ft  
Ueq: 30.2 m  
U(50): 93.68 m

T(50): 1668.2 s  
Ir: 100  
Ch: 0.4 cm<sup>2</sup>/min

## Description of Methods for Calculated CPT Geotechnical Parameters

# CALCULATED CPT GEOTECHNICAL PARAMETERS

## A Detailed Description of the Methods Used in ConeTec's CPT Geotechnical Parameter Calculation and Plotting Software



Revision SZW-Rev 14

Revised November 26, 2019

Prepared by Jim Greig, M.A.Sc, P.Eng (BC)



### Limitations

The geotechnical parameter output was prepared specifically for the site and project named in the accompanying report subject to objectives, site conditions and criteria provided to ConeTec by the client. The output may not be relied upon by any other party or for any other site without the express written permission of ConeTec Group (ConeTec) or any of its affiliates. For this project, ConeTec has provided site investigation services, prepared factual data reporting and produced geotechnical parameter calculations consistent with current best practices. No other warranty, expressed or implied, is made.

To understand the calculations that have been performed and to be able to reproduce the calculated parameters the user is directed to the basic descriptions for the methods in this document and the detailed descriptions and their associated limitations and appropriateness in the technical references cited for each parameter.

## ConeTec's Calculated CPT Geotechnical Parameters as of November 26, 2019

ConeTec's CPT parameter calculation and plotting routine provides a tabular output of geotechnical parameters based on current published CPT correlations and is subject to change to reflect the current state of practice. Due to drainage conditions and the basic assumptions and limitations of the correlations, not all geotechnical parameters provided are considered applicable for all soil types. The results are presented only as a guide for geotechnical use and should be carefully examined for consideration in any geotechnical design. Reference to current literature is strongly recommended. ConeTec does not warranty the correctness or the applicability of any of the geotechnical parameters calculated by the program and does not assume liability for any use of the results in any design or review. For verification purposes we recommend that representative hand calculations be done for any parameter that is critical for design purposes. The end user of the parameter output should also be fully aware of the techniques and the limitations of any method used by the program. The purpose of this document is to inform the user as to which methods were used and to direct the end user to the appropriate technical papers and/or publications for further reference.

The geotechnical parameter output was prepared specifically for the site and project named in the accompanying report subject to objectives, site conditions and criteria provided to ConeTec by the client. The output may not be relied upon by any other party or for any other site without the express written permission of ConeTec Group (ConeTec) or any of its affiliates.

The CPT calculations are based on values of tip resistance, sleeve friction and pore pressures considered at each data point or averaged over a user specified layer thickness (e.g. 0.20 m). Note that  $q_t$  is the tip resistance corrected for pore pressure effects and  $q_c$  is the recorded tip resistance. The corrected tip resistance (corrected using  $u_2$  pore pressure values) is used for all of the calculations. Since all ConeTec cones have equal end area friction sleeves pore pressure corrections to sleeve friction,  $f_s$ , are not required.

The tip correction is:  $q_t = q_c + (1-a) \cdot u_2$  (consistent units are implied)

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 cone (typically 0.80 for ConeTec cones)

The total stress calculations are based on soil unit weight values that have been assigned to the Soil Behavior Type (SBT) zones, from a user defined unit weight profile, by using a single uniform value throughout the profile, through unit weight estimation techniques described in various technical papers or from a combination of these methods. The parameter output files indicate the method(s) used.

Effective vertical overburden stresses are calculated based on a hydrostatic distribution of equilibrium pore pressures below the water table or from a user defined equilibrium pore pressure profile (typically obtained from CPT dissipation tests) or a combination of the two. For over water projects the stress effects of the column of water above the mudline have been taken into account as has the appropriate unit weight of water. How this is done depends on where the instruments were zeroed (i.e. on deck or at the mudline). The parameter output files indicate the method(s) used.

A majority of parameter calculations are derived or driven by results based on material types as determined by the various soil behavior type charts depicted in Figures 1 through 5. The parameter output files indicate the method(s) used.

The Soil Behavior Type classification chart shown in Figure 1 is the classic non-normalized SBT Chart developed at the University of British Columbia and reported in Robertson, Campanella, Gillespie and Greig (1986). Figure 2 shows the original normalized (linear method) SBT chart developed by Robertson (1990). The Bq classification charts shown in Figures 3a and 3b incorporate pore pressures into the SBT classification and are based on the methods described in Robertson (1990). Many of these charts have been summarized in Lunne, Robertson and Powell (1997). The



Jefferies and Davies SBT chart shown in Figure 3c is based on the techniques discussed in Jefferies and Davies (1993) which introduced the concept of the Soil Behavior Type Index parameter,  $I_c$ . Please note that the  $I_c$  parameter developed by Robertson and Fear (1995) and Robertson and Wride (1998) is similar in concept but uses a slightly different calculation method than that used by Jefferies and Davies (1993) as the latter incorporates pore pressure in their technique through the use of the  $B_q$  parameter. The normalized  $Q_{tn}$  SBT chart shown in Figure 4 is based on the work by Robertson (2009) utilizing a variable stress ratio exponent,  $n$ , for normalization based on a slightly modified redefinition and iterative approach for  $I_c$ . The boundary curves drawn on the chart are based on the work described in Robertson (2010).

Figure 5 shows a revised behavior based chart by Robertson (2016) depicting contractive-dilative zones. As the zones represent material behavior rather than soil gradation ConeTec has chosen a set of zone colors that are less likely to be confused with material type colors from previous SBT charts. These colors differ from those used by Dr. Robertson.

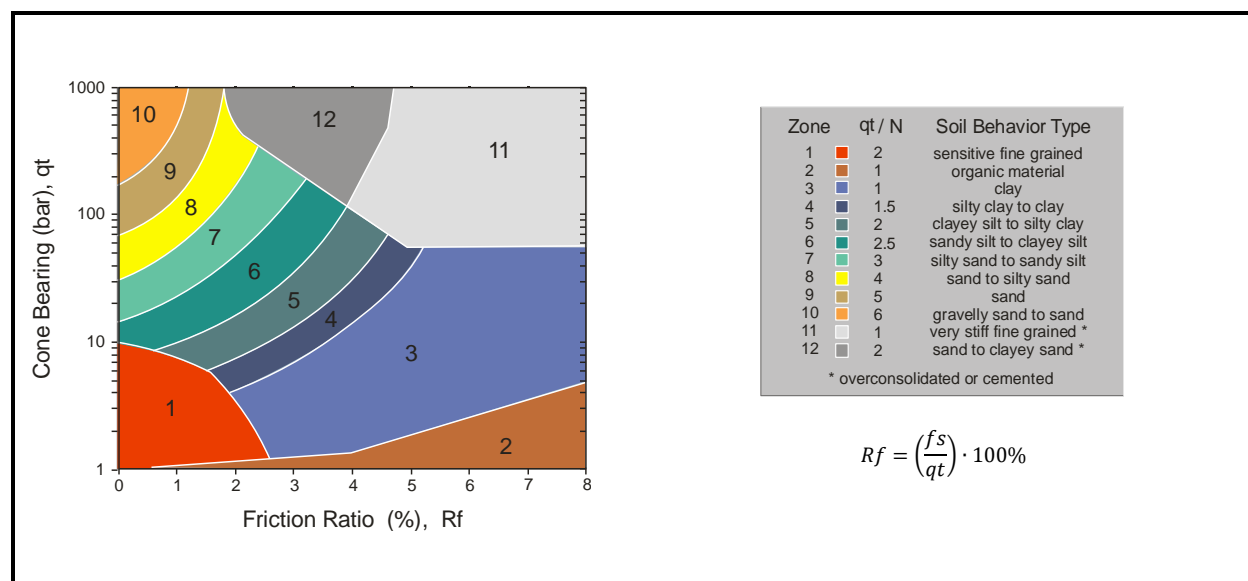


Figure 1. Non-Normalized Soil Behavior Type Classification Chart (SBT)

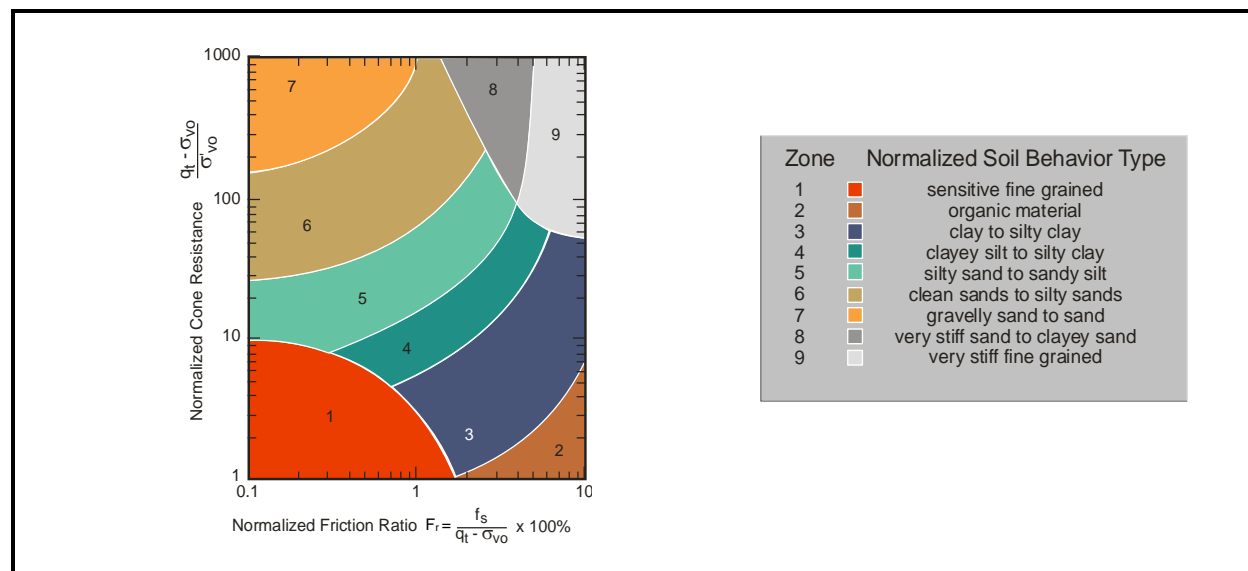


Figure 2. Normalized Soil Behavior Type Classification Chart (SBTn)

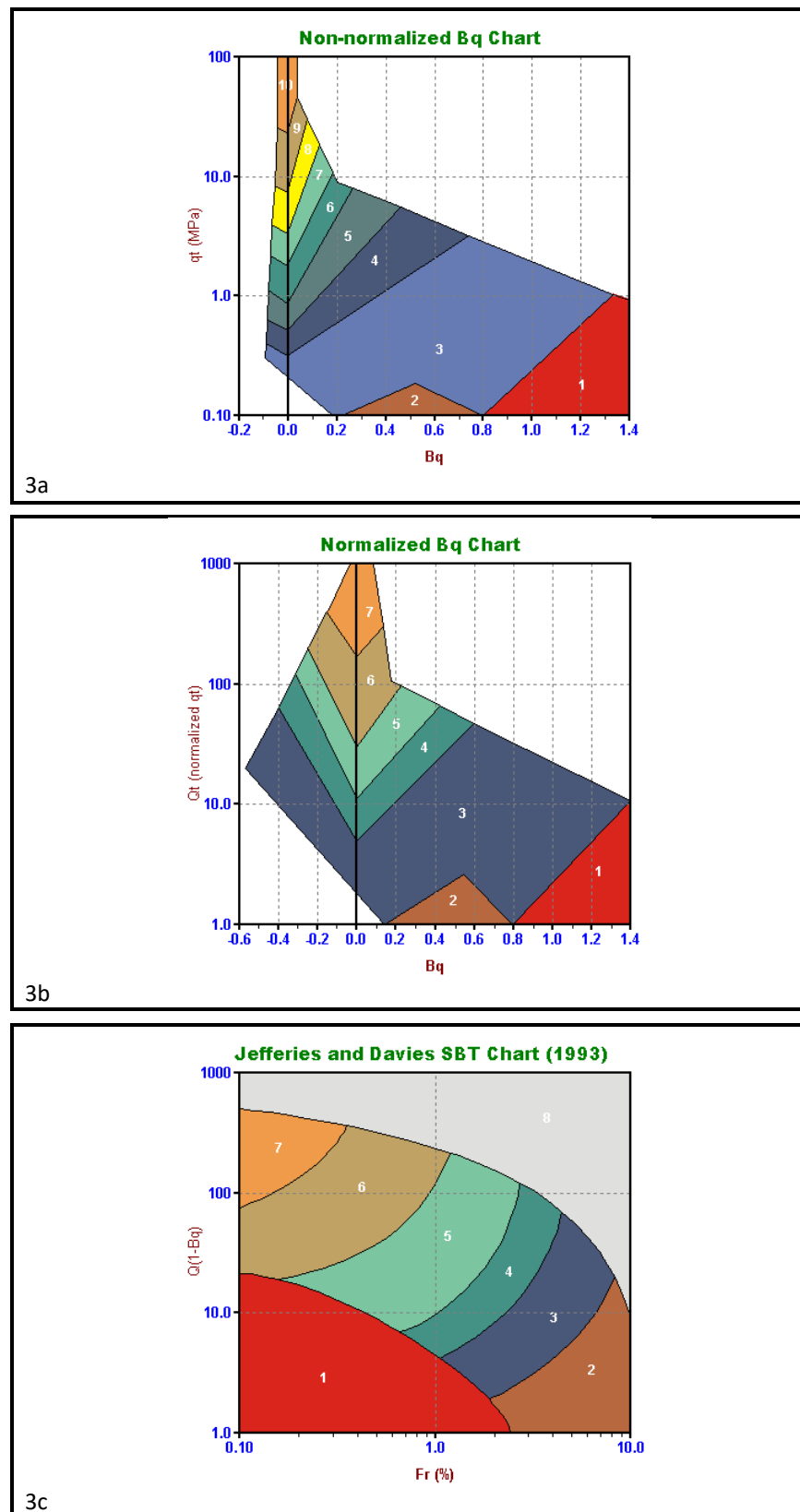


Figure 3. Alternate Soil Behavior Type Charts



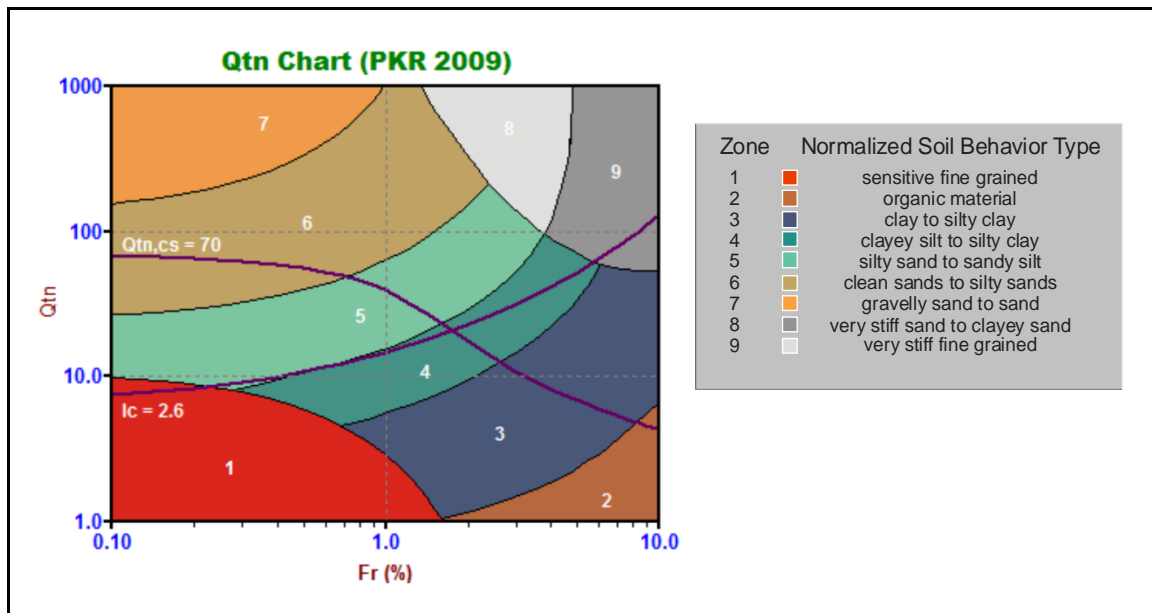
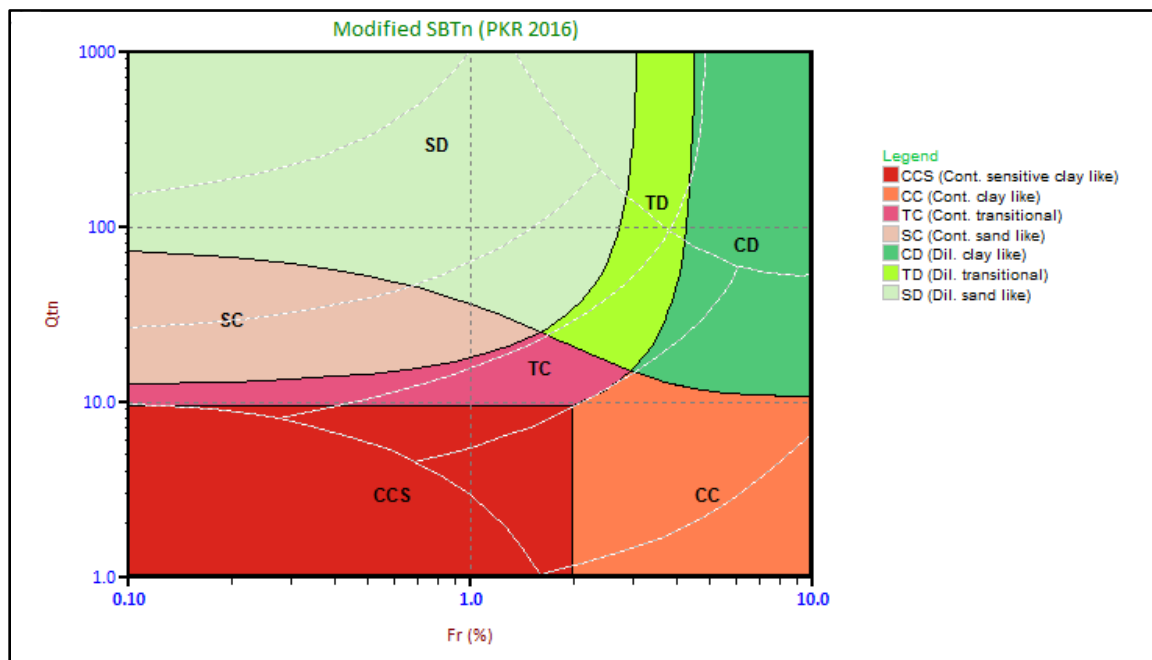
Figure 4. Normalized Soil Behavior Type Chart using  $Q_{tn}$  (SBT  $Q_{tn}$ )

Figure 5. Modified SBTn Behavior Based Chart

Details regarding the geotechnical parameter calculations are provided in Tables 1a and 1b. The appropriate references cited are listed in Table 2. Non-liquefaction specific parameters are detailed in Table 1a and liquefaction specific parameters are detailed in Table 1b.

Where methods are based on charts or techniques that are too complex to describe in this summary the user should refer to the cited material. Specific limitations for each method are described in the cited material.

Where the results of a calculation/correlation are deemed 'invalid' the value will be represented by the text strings "-9999", "-9999.0", the value 0.0 (Zero) or an empty cell. Invalid results will occur because of (and not limited to) one or a combination of:

1. Invalid or undefined CPT data (e.g. drilled out section or data gap).
2. Where the calculation method is inappropriate, for example, drained parameters in a material behaving as an undrained material (and vice versa).
3. Where input values are beyond the range of the referenced charts or specified limitations of the correlation method.
4. Where pre-requisite or intermediate parameter calculations are invalid.

The parameters selected for output from the program are often specific to a particular project. As such, not all of the calculated parameters listed in Table 1 may be included in the output files delivered with this report.

The output files are typically provided in Microsoft Excel XLS or XLSX format. The ConeTec software has several options for output depending on the number or types of calculated parameters desired or requested by the client. Each output file is named using the original COR file base name followed by a three or four letter indicator of the output set selected (e.g. BSC, TBL, NLI, NL2, IFI, IFI2) and possibly followed by an operator selected suffix identifying the characteristics of the particular calculation run.

**Table 1a. CPT Parameter Calculation Methods – Non liquefaction Parameters**

Calculated Parameter	Description	Equation	Ref
Depth	Mid Layer Depth <i>(where calculations are done at each point then Mid Layer Depth = Recorded Depth)</i>	$[Depth (Layer Top) + Depth (Layer Bottom)] / 2.0$	CK*
Elevation	Elevation of Mid Layer based on sounding collar elevation supplied by client or through site survey	Elevation = Collar Elevation - Depth	CK*
Avg qc	Averaged recorded tip value ( $q_c$ )	$Avg qc = \frac{1}{n} \sum_{i=1}^n q_c$ <i>n=1 when calculations are done at each point</i>	CK*
Avg qt	Averaged corrected tip ( $q_t$ ) where: $q_t = q_c + (1 - \alpha) \bullet u_2$	$Avg qt = \frac{1}{n} \sum_{i=1}^n q_t$ <i>n=1 when calculations are done at each point</i>	1
Avg fs	Averaged sleeve friction ( $f_s$ )	$Avg fs = \frac{1}{n} \sum_{i=1}^n f_s$ <i>n=1 when calculations are done at each point</i>	CK*
Avg Rf	Averaged friction ratio ( $R_f$ ) where friction ratio is defined as: $R_f = 100\% \bullet \frac{f_s}{q_t}$	$Avg Rf = 100\% \bullet \frac{Avg fs}{Avg qt}$ <i>n=1 when calculations are done at each point</i>	CK*
Avg u	Averaged dynamic pore pressure ( $u$ )	$Avg u = \frac{1}{n} \sum_{i=1}^n u_i$ <i>n=1 when calculations are done at each point</i>	CK*

Calculated Parameter	Description	Equation	Ref
Avg Res	Averaged Resistivity (this data is not always available since it is a specialized test requiring an additional module)	$AvgRes = \frac{1}{n} \sum_{i=1}^n Resistivity_i$ <i>n=1 when calculations are done at each point</i>	CK*
Avg UVIF	Averaged UVIF ultra-violet induced fluorescence (this data is not always available since it is a specialized test requiring an additional module)	$AvgUVIF = \frac{1}{n} \sum_{i=1}^n UVIF_i$ <i>n=1 when calculations are done at each point</i>	CK*
Avg Temp	Averaged Temperature (this data is not always available since it requires specialized calibrations)	$AvgTemp = \frac{1}{n} \sum_{i=1}^n Temperature_i$ <i>n=1 when calculations are done at each point</i>	CK*
Avg Gamma	Averaged Gamma Counts (this data is not always available since it is a specialized test requiring an additional module)	$AvgGamma = \frac{1}{n} \sum_{i=1}^n Gamma_i$ <i>n=1 when calculations are done at each point</i>	CK*
SBT	Soil Behavior Type as defined by Robertson et al 1986 (often referred to as Robertson and Campanella, 1986)	See Figure 1	1, 5
SBTn	Normalized Soil Behavior Type as defined by Robertson 1990 (linear normalization)	See Figure 2	2, 5
SBT-Bq	Non-normalized Soil Behavior type based on the Bq parameter	See Figure 3	1, 2, 5
SBT-Bqn	Normalized Soil Behavior based on the Bq parameter	See Figure 3	2, 5
SBT-JandD	Soil Behavior Type as defined by Jeffries and Davies	See Figure 3	7
SBT Qtn	Soil Behavior Type as defined by Robertson (2009) using a variable stress ratio exponent for normalization based on $I_c$	See Figure 4	15
Modified SBTn (contractive /dilative)	Modified SBTn chart as defined by Robertson (2016) indicating zones of contractive/dilative behavior.	See Figure 5	30
Unit Wt.	<p>Unit Weight of soil determined from one of the following user selectable options:</p> <ol style="list-style-type: none"> <li>1) uniform value</li> <li>2) value assigned to each SBT zone</li> <li>3) value assigned to each SBTn zone</li> <li>4) value assigned to SBTn zone as determined from Robertson and Wride (1998) based on <math>q_{c1n}</math></li> <li>5) values assigned to SBT Qtn zones</li> <li>6) Mayne <math>f_s</math> (sleeve friction) method</li> <li>7) Robertson 2010 method</li> <li>8) user supplied unit weight profile</li> </ol> <p>The last option may co-exist with any of the other options</p>	See references	3, 5, 15, 21, 24, 29

Calculated Parameter	Description	Equation	Ref
<p>TStress</p> <p><math>\sigma_v</math></p>	<p>Total vertical overburden stress at Mid Layer Depth</p> <p><i>A layer is defined as the averaging interval specified by the user where depths are reported at their respective mid-layer depth.</i></p> <p><i>For data calculated at each point layers are defined using the recorded depth as the mid-point of the layer. Thus, a layer starts half-way between the previous depth and the current depth unless this is the first point in which case the layer start is at zero depth. The layer bottom is half-way from the current depth to the next depth unless it is the last data point.</i></p> <p><i>Defining layers affects how stresses are calculated since the unit weight attributed to a data point is used throughout the entire layer. This means that to calculate the stresses the total stress at the top and bottom of a layer are required. The stress at mid layer is determined by adding the incremental stress from the layer top to the mid-layer depth. The stress at the layer bottom becomes the stress at the top of the subsequent layer. Stresses are NOT calculated from mid-point to mid-point.</i></p> <p><i>For over-water work the total stress due to the column of water above the mud line is taken into account where appropriate.</i></p>	$TStress = \sum_{i=1}^n \gamma_i h_i$ <p>where <math>\gamma_i</math> is layer unit weight <math>h_i</math> is layer thickness</p>	CK*
<p>EStress</p> <p><math>\sigma_v'</math></p>	Effective vertical overburden stress at mid-layer depth	$\sigma_v' = \sigma_v - u_{eq}$	CK*
<p>Equil u</p> <p><math>u_{eq}</math> or <math>u_0</math></p>	<p>Equilibrium pore pressure determined from one of the following user selectable options:</p> <ol style="list-style-type: none"> <li>1) hydrostatic below water table</li> <li>2) user supplied profile</li> <li>3) combination of those above</li> </ol> <p>When a user supplied profile is used/provided a linear interpolation is performed between equilibrium pore pressures defined at specific depths. If the profile values start below the water table then a linear transition from zero pressure at the water table to the first defined point is used.</p> <p>Equilibrium pore pressures may come from dissipation tests, adjacent piezometers or other sources. Occasionally, an extra equilibrium point ("assumed value") will be provided in the profile that does not come from a recorded value to smooth out any abrupt changes or to deal with material interfaces. These "assumed" values will be indicated on our plots and in tabular summaries.</p>	<p>For hydrostatic option:</p> $u_{eq} = \gamma_w \cdot (D - D_{wt})$ <p>where <math>u_{eq}</math> is equilibrium pore pressure <math>\gamma_w</math> is unit weight of water <math>D</math> is the current depth <math>D_{wt}</math> is the depth to the water table</p>	CK*
$K_0$	Coefficient of earth pressure at rest, $K_0$	$K_0 = (1 - \sin \Phi') OCR^{\sin \Phi'}$	17
$C_n$	Overburden stress correction factor used for $(N_1)_{60}$ and older CPT parameters	$C_n = (P_a / \sigma_v')^{0.5}$ <p>where <math>0.0 &lt; C_n &lt; 2.0</math> (user adjustable, typically 1.7) <math>P_a</math> is atmospheric pressure (100 kPa)</p>	12
$C_q$	Overburden stress normalizing factor	$C_q = 1.8 / (0.8 + (\sigma_v' / P_a))$ <p>where <math>0.0 &lt; C_q &lt; 2.0</math> (user adjustable) <math>P_a</math> is atmospheric pressure (100 kPa)</p>	3, 12

Calculated Parameter	Description	Equation	Ref
$N_{60}$	SPT N value at 60% energy calculated from $q_t/N$ ratios assigned to each SBT zone. This method has abrupt N value changes at zone boundaries.	<i>See Figure 1</i>	5
$(N_1)_{60}$	SPT $N_{60}$ value corrected for overburden pressure	$(N_1)_{60} = C_n \cdot N_{60}$	4
$N_{60lc}$	SPT $N_{60}$ values based on the $I_c$ parameter [as defined by Roberston and Wride 1998 (5), or by Robertson 2009 (15)].	$(q_t/P_a)/N_{60} = 8.5 (1 - I_c/4.6)$ $(q_t/P_a)/N_{60} = 10^{(1.1268 - 0.2817I_c)}$ $P_a$ being atmospheric pressure	5 15, 31
$(N_1)_{60lc}$	SPT $N_{60}$ value corrected for overburden pressure (using $N_{60lc}$ ). User has 3 options.	1) $(N_1)_{60lc} = C_n \cdot (N_{60lc})$ 2) $q_{c1n}/(N_1)_{60lc} = 8.5 (1 - I_c/4.6)$ 3) $(Q_{tn})/(N_1)_{60lc} = 10^{(1.1268 - 0.2817I_c)}$	4 5 15, 31
$S_u$ or $S_u$ (Nkt)	Undrained shear strength based on $q_t$ $S_u$ factor $N_{kt}$ is user selectable	$S_u = \frac{q_t - \sigma_v}{N_{kt}}$	1, 5
$S_u$ or $S_u$ (Ndu)	Undrained shear strength based on pore pressure $S_u$ factor $N_{du}$ is user selectable	$S_u = \frac{u_2 - u_{eq}}{N_{du}}$	1, 5
$D_r$	Relative Density determined from one of the following user selectable options: a) Ticino Sand b) Hokksund Sand c) Schmertmann (1978) d) Jamiolkowski (1985) - All Sands e) Jamiolkowski et al (2003) (various compressibilities, $K_o$ )	See reference (methods a through d) Jamiolkowski et al (2003) reference	5 14
$\Phi$ $\phi$	Friction Angle determined from one of the following user selectable options (methods a through d are for sands and method e is for silts and clays): a) Campanella and Robertson b) Durgunoglu and Mitchel c) Janbu d) Kulhawy and Mayne e) NTH method (clays and silts)	See appropriate reference	5 5 5 11 23
Delta U/ $q_t$	Differential pore pressure ratio (older parameter used before $B_q$ was established)	$= \frac{\Delta u}{q_t}$  where: $\Delta u = u - u_{eq}$ and $u$ = dynamic pore pressure $u_{eq}$ = equilibrium pore pressure	CK*
$B_q$	Pore pressure parameter	$B_q = \frac{\Delta u}{q_t - \sigma_v}$  where: $\Delta u = u - u_{eq}$ and $u$ = dynamic pore pressure $u_{eq}$ = equilibrium pore pressure	1, 2, 5
Net $q_t$ or $q_{tNet}$	Net tip resistance (used in many subsequent correlations)	$q_t - \sigma_v$	CK*
$q_e$	Effective tip resistance (using the dynamic pore pressure $u_2$ and not equilibrium pore pressure)	$q_t - u_2$	CK*

Calculated Parameter	Description	Equation	Ref
qeNorm	Normalized effective tip resistance	$\frac{qt - u_2}{\sigma_v}$	CK*
$Q_t$ or Norm: $Q_t$	Normalized $q_t$ for Soil Behavior Type classification as defined by Robertson (1990) using a linear stress normalization. Note this is different from $Q_{tn}$ .	$Q_t = \frac{qt - \sigma_v}{\sigma_v}$	2, 5
$F_r$ or Norm: $F_r$	Normalized Friction Ratio for Soil Behavior Type classification as defined by Robertson (1990)	$Fr = 100\% \cdot \frac{fs}{qt - \sigma_v}$	2, 5
$Q(1-Bq)$	$Q(1-Bq)$ grouping as suggested by Jefferies and Davies for their classification chart and the establishment of their $I_c$ parameter	$Q \cdot (1 - Bq)$  <i>where <math>Bq</math> is defined as above and <math>Q</math> is the same as the normalized tip resistance, <math>Q_t</math>, defined above</i>	6, 7
qc1	Normalized tip resistance, $q_{c1}$ , using a fixed stress ratio exponent, $n$ (this method has stress units)	$q_{c1} = q_t \cdot (P_a / \sigma_v')^{0.5}$ where: $P_a$ = atmospheric pressure	21
qc1 (0.5)	Normalized tip resistance, $q_{c1}$ , using a fixed stress ratio exponent, $n$ (this method is unit-less)	$q_{c1} (0.5) = (q_t / P_a) \cdot (P_a / \sigma_v')^{0.5}$ where: $P_a$ = atmospheric pressure	5
qc1 ( $C_n$ )	Normalized tip resistance, $q_{c1}$ , based on $C_n$ (this method has stress units)	$q_{c1}(C_n) = C_n \cdot q_t$	5, 12
qc1 ( $C_q$ )	Normalized tip resistance, $q_{c1}$ , based on $C_q$ (this method has stress units)	$q_{c1}(C_q) = C_q \cdot q_t$ (some papers use $q_c$ )	5, 12
qc1n	normalized tip resistance, $q_{c1n}$ , using a variable stress ratio exponent, $n$ (where $n=0.0, 0.70, 1.0$ ) (this method is unit-less)	$q_{c1n} = (q_t / P_a) (P_a / \sigma_v')^n$ where: $P_a$ = atm. Pressure and $n$ varies as described below	3, 5
$I_c$ or $I_c$ (RW1998)	Soil Behavior Type Index as defined by Robertson and Fear (1995) and Robertson and Wride (1998) for estimating grain size characteristics and providing smooth gradational changes across the SBTn chart	$I_c = [(3.47 - \log_{10} Q)^2 + (\log_{10} Fr + 1.22)^2]^{0.5}$  <i>Where:</i> $Q = \left( \frac{qt - \sigma_v}{P_a} \right) \left( \frac{P_a}{\sigma_v'} \right)^n$  <i>Or</i> $Q = q_{c1n} = \left( \frac{qt}{P_a} \right) \left( \frac{P_a}{\sigma_v'} \right)^n$  <i>depending on the iteration in determining <math>I_c</math></i>  <i>And <math>Fr</math> is in percent <math>P_a</math> = atmospheric pressure</i>  <i><math>n</math> varies between 0.5, 0.70 and 1.0 and is selected in an iterative manner based on the resulting <math>I_c</math></i>	3, 5, 21
$I_c$ (PKR 2009)	Soil Behavior Type Index, $I_c$ (PKR 2009) based on a variable stress ratio exponent $n$ , which itself is based on $I_c$ (PKR 2009). An iterative calculation is required to determine $I_c$ (PKR 2009) and its corresponding $n$ (PKR 2009).	$I_c \text{ (PKR 2009)} = [(3.47 - \log_{10} Q_{tn})^2 + (1.22 + \log_{10} Fr)^2]^{0.5}$	15

Calculated Parameter	Description	Equation	Ref
n (PKR 2009)	Stress ratio exponent n, based on $I_c$ (PKR 2009). An iterative calculation is required to determine n (PKR 2009) and its corresponding $I_c$ (PKR 2009).	$n \text{ (PKR 2009)} = 0.381 (I_c) + 0.05 (\sigma_v'/P_a) - 0.15$	15
Qtn (PKR 2009)	Normalized tip resistance using a variable stress ratio exponent based on $I_c$ (PKR 2009) and n (PKR 2009). An iterative calculation is required to determine Qtn (PKR 2009).	$Q_{tn} = [(qt - \sigma_v)/P_a](P_a/\sigma_v')^n$ where $P_a$ = atmospheric pressure (100 kPa) $n$ = stress ratio exponent described above	15
FC	Apparent fines content (%)	$FC = 1.75(I_c^{3.25}) - 3.7$ $FC = 100$ for $I_c > 3.5$ $FC = 0$ for $I_c < 1.26$ $FC = 5\%$ if $1.64 < I_c < 2.6$ AND $F_r < 0.5$	3
$I_c$ Zone	This parameter is the Soil Behavior Type zone based on the $I_c$ parameter (valid for zones 2 through 7 on SBTn or SBT Qtn charts)	$I_c < 1.31$ Zone = 7 $1.31 < I_c < 2.05$ Zone = 6 $2.05 < I_c < 2.60$ Zone = 5 $2.60 < I_c < 2.95$ Zone = 4 $2.95 < I_c < 3.60$ Zone = 3 $I_c > 3.60$ Zone = 2	3
State Param or State Parameter or $\psi$	The state parameter index, $\psi$ , is defined as the difference between the current void ratio, $e$ , and the critical void ratio, $e_c$ . Positive $\psi$ - contractive soil Negative $\psi$ - dilative soil  This is based on the work by Been and Jefferies (1985) and Plewes, Davies and Jefferies (1992)  - vertical effective stress is used rather than a mean normal stress	See reference	6, 8
Yield Stress $\sigma_p'$	Yield stress is calculated using the following methods  a) General method  b) 1 <sup>st</sup> order approximation using $q_t$ Net (clays) c) 1 <sup>st</sup> order approximation using $\Delta u_2$ (clays) d) 1 <sup>st</sup> order approximation using $q_e$ (clays)	All stresses in kPa  a) $\sigma_p' = 0.33 \cdot (q_t - \sigma_v)^{m'} (\sigma_{atm}/100)^{1-m'}$  where $m' = 1 - \frac{0.28}{1 + (I_c / 2.65)^{2.5}}$  b) $\sigma_p' = 0.33 \cdot (q_t - \sigma_v)$ c) $\sigma_p' = 0.54 \cdot (\Delta u_2)$ $\Delta u_2 = u_2 - u_0$ d) $\sigma_p' = 0.60 \cdot (q_t - u_2)$	19  20 20 20
OCR  OCR(JS1978)  OCR(Mayne2014) OCR (qtNet) OCR (deltaU) OCR (qe) OCR (Vs) OCR (PKR2015)	Over Consolidation Ratio based on  a) Schmertmann (1978) method involving a plot of $S_u/\sigma_v' / (S_u/\sigma_v')_{NC}$ and OCR  b) based on Yield stresses described above c) approximate version based on qtNet d) approximate version based on $\Delta u$ e) approximate version based on effective tip, $q_e$ f) approximate version based on shear wave velocity, $V_s$ g) based on $Q_t$	a) requires a user defined value for NC $S_u/P_c'$ ratio  b through f) based on yield stresses  g) $OCR = 0.25 \cdot (Q_t)^{1.25}$	9  19 20 20 20 18 32

Calculated Parameter	Description	Equation	Ref
Es/qt	Intermediate parameter for calculating Young's Modulus, E, in sands. It is the Y axis of the reference chart.	Based on Figure 5.59 in the reference	5
Es Young's Modulus E	<p>Young's Modulus based on the work done in Italy. There are three types of sands considered in this technique. The user selects the appropriate type for the site from:</p> <p>a) OC Sands b) Aged NC Sands c) Recent NC Sands</p> <p>Each sand type has a family of curves that depend on mean normal stress. The program calculates mean normal stress and linearly interpolates between the two extremes provided in the Es/qt chart. Es is evaluated for an axial strain of 0.1%.</p>	<p>Mean normal stress is evaluated from:</p> $\sigma'_m = \frac{1}{3}(\sigma'_v + \sigma'_h + \sigma'_h)$ <p>where <math>\sigma'_v</math> = vertical effective stress <math>\sigma'_h</math> = horizontal effective stress</p> <p>and <math>\sigma'_h = K_o \cdot \sigma'_v</math> with <math>K_o</math> assumed to be 0.5</p>	5
Delta U/TStress	Differential pore pressure ratio with respect to total stress	$= \frac{\Delta u}{\sigma_v}$ where: $\Delta u = u - u_{eq}$	CK*
Delta U/Estress, P Value, Excess Pore Pressure Ratio	Differential pore pressure ratio with respect to effective stress. Key parameter (P, Normalized Pore Pressure Parameter, Excess Pore Pressure Ratio) in the Winckler et. al. static liquefaction method.	$= \frac{\Delta u}{\sigma'_v}$ where: $\Delta u = u - u_{eq}$	25, 25a, CK*
Su/Estress	Undrained shear strength ratio with respect to vertical effective overburden stress using the Su (N <sub>kt</sub> ) method	$= Su (N_{kt}) / \sigma'_v$	CK*
Gmax	G <sub>max</sub> determined from SCPT shear wave velocities (not estimated values)	$G_{max} = \rho V_s^2$ where $\rho$ is the mass density of the soil determined from the estimated unit weights at each test depth	27
qtNet/Gmax	Net tip resistance ratio with respect to the small strain modulus G <sub>max</sub> determined from SCPT shear wave velocities (not estimated values)	$= (qt - \sigma_v) / G_{max}$ where $G_{max} = \rho V_s^2$ and $\rho$ is the mass density of the soil determined from the estimated unit weights at each test depth	15, 28, 30

\*CK – common knowledge



**Table 1b. CPT Parameter Calculation Methods – Liquefaction Parameters**

Calculated Parameter	Description	Equation	Ref
$K_{SPT}$	Equivalent clean sand factor for $(N_1)_{60}$	$K_{SPT} = 1 + ((0.75/30) \cdot (FC - 5))$	10
$K_{CPT}$ or $K_C$ (RW1998)	Equivalent clean sand correction for $q_{c1N}$	$K_{cpt} = 1.0$ for $l_c \leq 1.64$ $K_{cpt} = f(l_c)$ for $l_c > 1.64$ (see reference) $K_c = -0.403 l_c^4 + 5.581 l_c^3 - 21.63 l_c^2 + 33.75 l_c - 17.88$	3, 10
$K_C$ (PKR 2010)	Clean sand equivalent factor to be applied to $Q_{tn}$	$K_c = 1.0$ for $l_c \leq 1.64$ $K_c = -0.403 l_c^4 + 5.581 l_c^3 - 21.63 l_c^2 + 33.75 l_c - 17.88$ for $l_c > 1.64$	16
$(N_1)_{60cs} I_C$	Clean sand equivalent SPT $(N_1)_{60} I_C$ . User has 3 options.	1) $(N_1)_{60cs} I_C = \alpha + \beta((N_1)_{60} I_C)$ 2) $(N_1)_{60cs} I_C = K_{SPT} * ((N_1)_{60} I_C)$ 3) $(q_{c1ncs}) / ((N_1)_{60cs} I_C) = 8.5 (1 - l_c/4.6)$  FC $\leq$ 5%: $\alpha = 0, \beta = 1.0$ FC $\geq$ 35%: $\alpha = 5.0, \beta = 1.2$ 5% < FC < 35%: $\alpha = \exp[1.76 - (190/FC^2)]$ $\beta = [0.99 + (FC^{1.5}/1000)]$	10 10 5
$q_{c1ncs}$	Clean sand equivalent $q_{c1n}$	$q_{c1ncs} = q_{c1n} \cdot K_{cpt}$	3
$Q_{tn,cs}$ (PKR 2010)	Clean sand equivalent for $Q_{tn}$ described above - $Q_{tn}$ being the normalized tip resistance based on a variable stress exponent as defined by Robertson (2009)	$Q_{tn,cs} = Q_{tn} \cdot K_C$ (PKR 2016)	16
$Su(Liq)/ESv$	Liquefied shear strength ratio as defined by Olson and Stark	$\frac{Su(Liq)}{\sigma_v'} = 0.03 + 0.0143(q_{c1})$  Note: $\sigma_v'$ and $s_v'$ are synonymous	13
$Su(Liq)/ESv$ (PKR 2010)	Liquefied shear strength ratio as defined by Robertson (2010)	$\frac{Su(Liq)}{\sigma_v'}$ Based on a function involving $Q_{tn,cs}$	16
$Su(Liq)$ (PKR 2010)	Liquefied shear strength derived from the liquefied shear strength ratio and effective overburden stress		16
Cont/Dilat Tip	Contractive / Dilative qc1 Boundary based on $(N_1)_{60}$	$(\sigma_v')_{boundary} = 9.58 \times 10^{-4} [(N_1)_{60}]^{4.79}$ $qc1$ is calculated from specified qt(MPa)/N ratio	13
CRR	Cyclic Resistance Ratio (for Magnitude 7.5)	$q_{c1ncs} < 50$ : $CRR_{7.5} = 0.833 [q_{c1ncs}/1000] + 0.05$  $50 \leq q_{c1ncs} < 160$ : $CRR_{7.5} = 93 [q_{c1ncs}/1000]^3 + 0.08$	10
$K_g$	Small strain Stiffness Ratio Factor, $K_g$	$[G_{max}/qt]/[q_{c1n}^{-m}]$ $m$ = empirical exponent, typically 0.75	26

Calculated Parameter	Description	Equation	Ref
SP Distance	State Parameter Distance, Winckler static liquefaction method	Perpendicular distance on Qtn chart from plotted point to state parameter $\Psi = -0.05$ curve	25
URS NP Fr	Normalized friction ratio point on $\Psi = -0.05$ curve used in SP Distance calculation		25
URS NP Qtn	Normalized tip resistance (Qtn) point on $\Psi = -0.05$ curve used in SP Distance calculation		25

**Table 2. References**

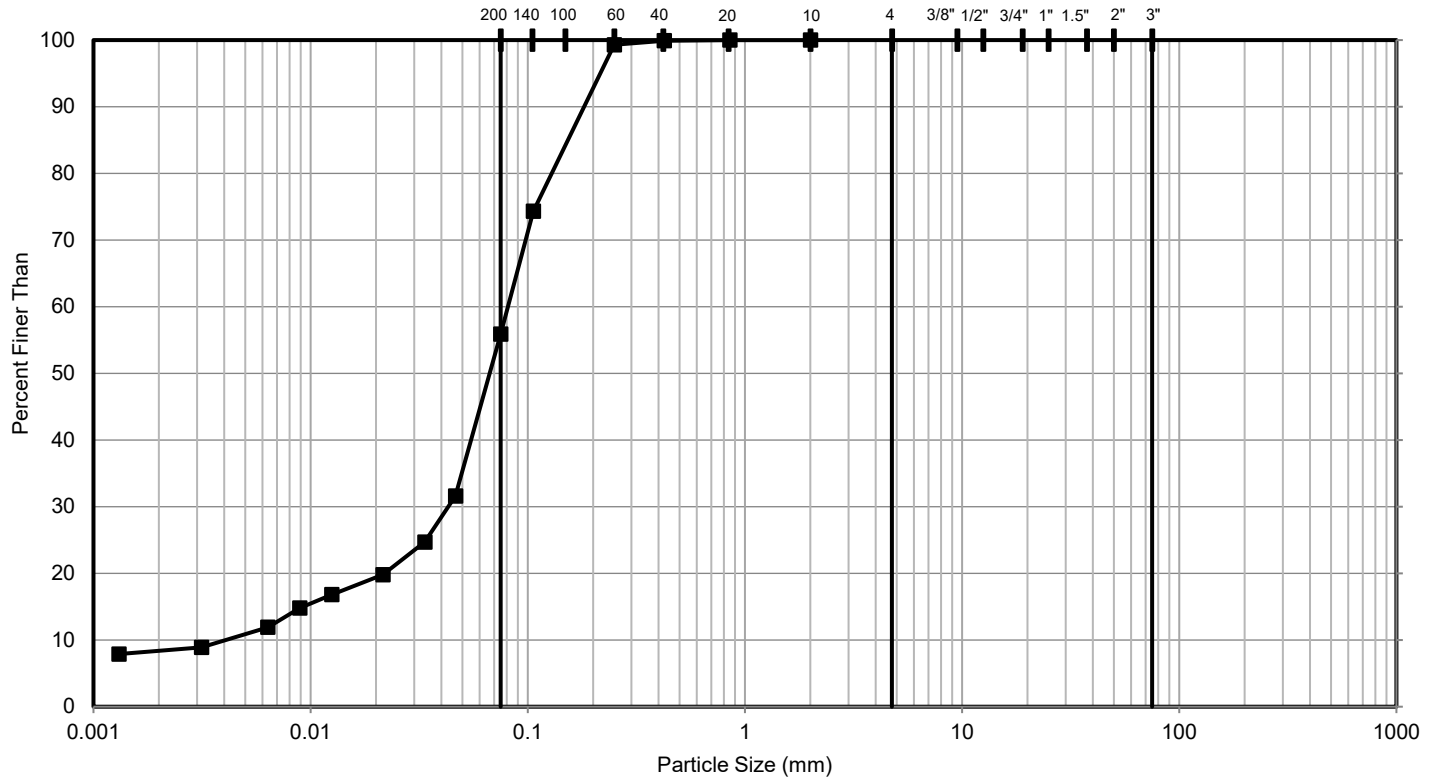
No.	Reference
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APPENDIX B

# Geotechnical Laboratory Test Results

# GRAIN SIZE DISTRIBUTION- SILTY SAND (SM) TO SANDY SILT (ML)



FINES (Silt, Clay)	SAND			GRAVEL		COBBLES	BOULDERS
	Fine	Medium	Coarse	Fine	Coarse		

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	HRW-4	2B	0.9 - 1.4	216.5 to 216.0

CLIENT

AECOM / MTO

CONSULTANT



YYYY-MM-DD 2022-05-10

DESIGNED AM

PREPARED MA

REVIEWED MA/MTI

APPROVED KJB

PROJECT

BRADFORD BYPASS - Holland River

TITLE

Grain Size Distribution - SILTY SAND (SM) TO SANDY SILT (ML) (ML)

PROJECT NO.

19136074

CONTROL

1000

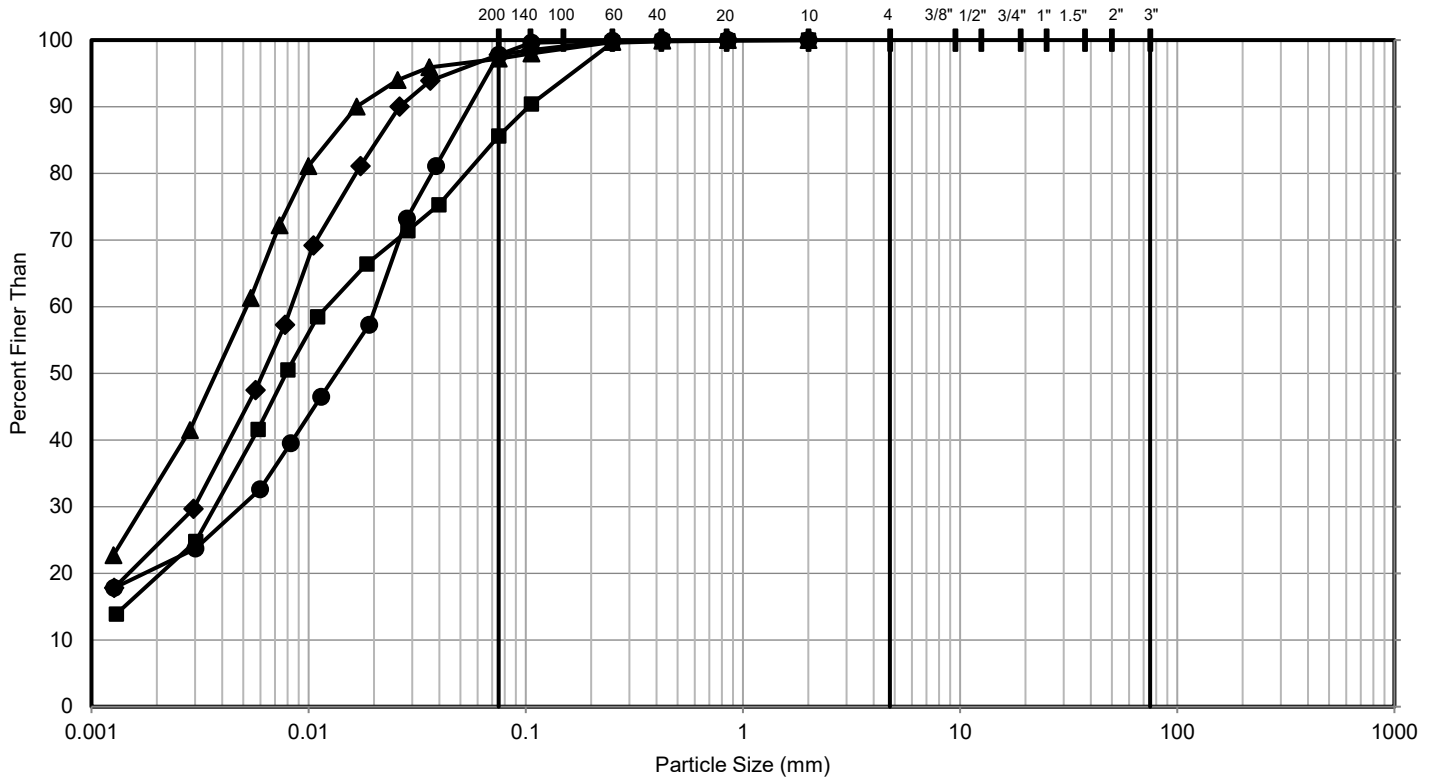
REV.

A

FIGURE

B-1

# GRAIN SIZE DISTRIBUTION - UPPER COHESIVE DEPOSIT CLAYEY SILT (CL) TO CLAYEY SILT-SILT (CL-ML)



FINES (Silt, Clay)	SAND			GRAVEL		COBBLES	BOULDERS
	Fine	Medium	Coarse	Fine	Coarse		

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	HRW-1	4	2.3 - 2.9	216.7 to 216.1
◆	HRW-4	3C	1.8 - 2.1	215.5 to 215.2
▲	HRW-4	6	4.0 - 4.6	213.4 to 212.8
●	HRW-4	10	7.6 - 8.2	209.7 to 209.1

CLIENT

AECOM / MTO

CONSULTANT



YYYY-MM-DD 2022-05-10

DESIGNED AM

PREPARED MA

REVIEWED MA/MTI

APPROVED KJB

PROJECT

BRADFORD BYPASS - Holland River

TITLE

Grain Size Distribution - UPPER COHESIVE DEPOSIT  
CLAYEY SILT (CL) TO CLAYEY SILT-SILT (CL-ML)

PROJECT NO.

19136074

CONTROL

1000

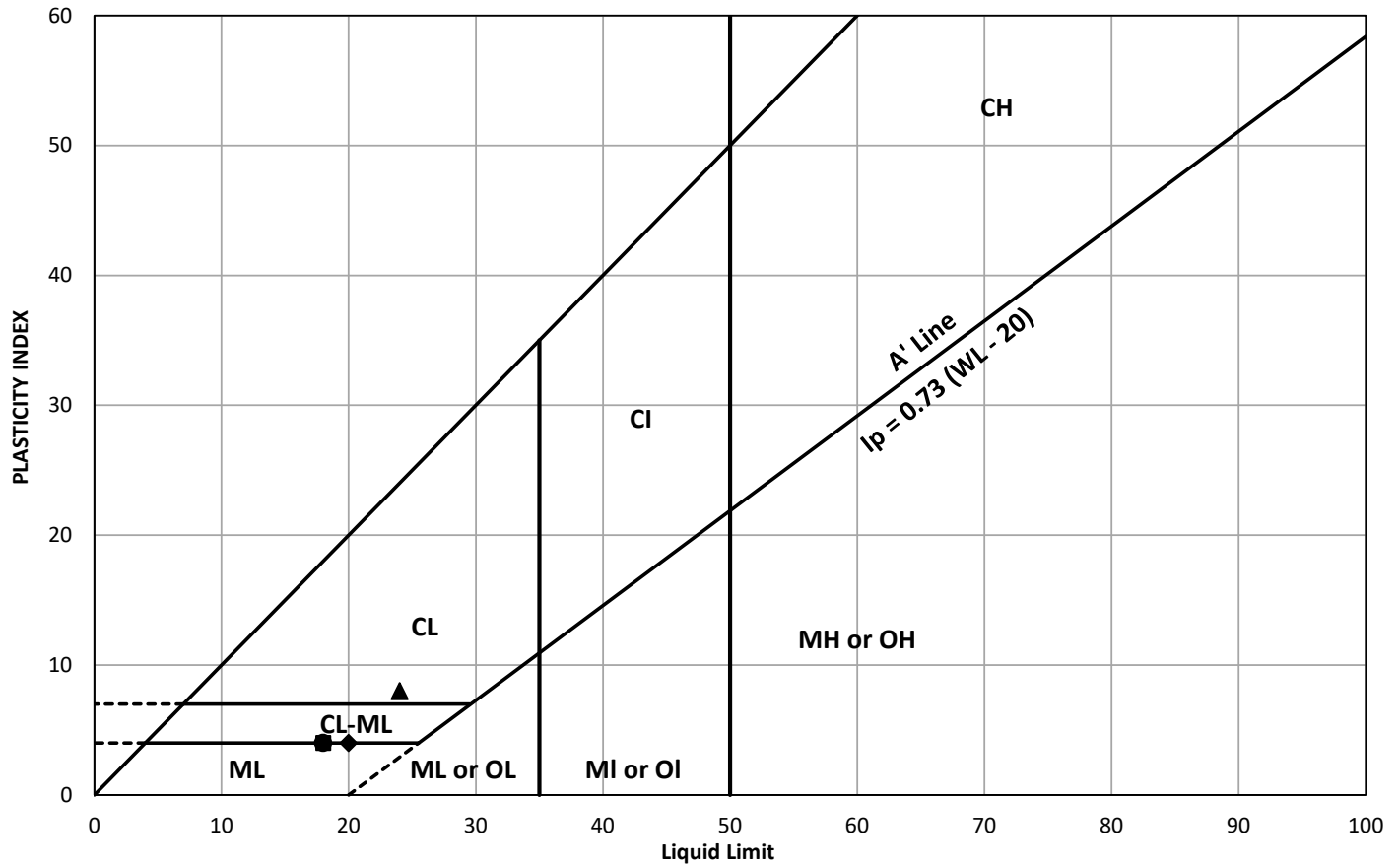
REV.

A

FIGURE

B-2

# PLASTICITY CHART - UPPER COHESIVE DEPOSIT CLAYEY SILT (CL) TO CLAYEY SILT-SILT (CL-ML)

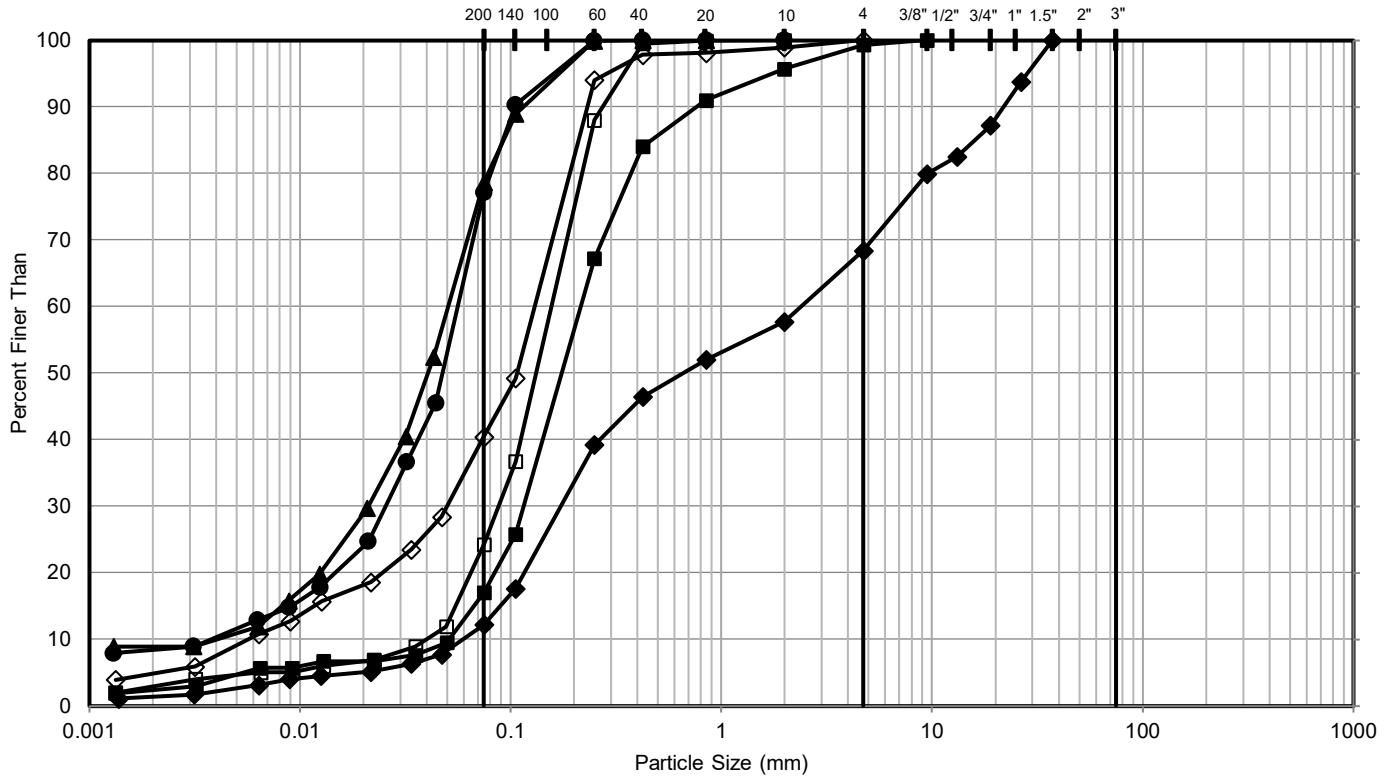


	Sample Location	Sample / Specimen Number	Depth (m)	Natural Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
■	HRW-1	4	2.3 - 2.9	19.9	18	14	4	1.48
◆	HRW-4	3C	1.8 - 2.1	27.4	20	16	4	2.85
▲	HRW-4	6	4.0 - 4.6	22.4	24	16	8	0.80
●	HRW-4	10	7.6 - 8.2	19.6	18	14	4	1.40

CLIENT			PROJECT			
AECOM / MTO			Bradford Bypass - Holland River			
CONSULTANT			TITLE			
2022-05-10			Plasticity Chart - UPPER COHESIVE DEPOSIT			
DESIGNED AM			CLAYEY SILT (CL) TO CLAYEY SILT-SILT (CL-ML)			
PREPARED MA			PROJECT NO.			
REVIEWED MA/MTI			CONTROL			
APPROVED KJB			REV.			
			FIGURE			
			19136074			
			1000			
			A			
			B-3			



# GRAIN SIZE DISTRIBUTION - SILTY SAND (SM) to SANDY SILT (ML)



FINES (Silt, Clay)	SAND			GRAVEL		COBBLES	BOULDERS
	Fine	Medium	Coarse	Fine	Coarse		

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	HRW-1	8	6.1 - 6.7	212.9 to 212.3
◆	HRW-1	11	10.7 - 11.3	208.3 to 207.7
▲	HRW-4	13	12.2 - 12.8	205.2 to 204.6
●	HRW-4	15	15.2 - 15.9	202.1 to 201.5
□	HRW-1	17	21.3 - 21.8	197.7 to 197.2
◇	HRW-1	20	30.5 - 30.9	188.5 to 188.1

CLIENT

Central

CONSULTANT



YYYY-MM-DD 2022-07-23

DESIGNED CC

PREPARED CC

REVIEWED KJB

APPROVED KJB

PROJECT

Bradford Bypass - Holland River

TITLE

Grain Size Distribution  
SILTY SAND (SM) to SANDY SILT (ML)

PROJECT NO.

19136074

CONTROL

1000

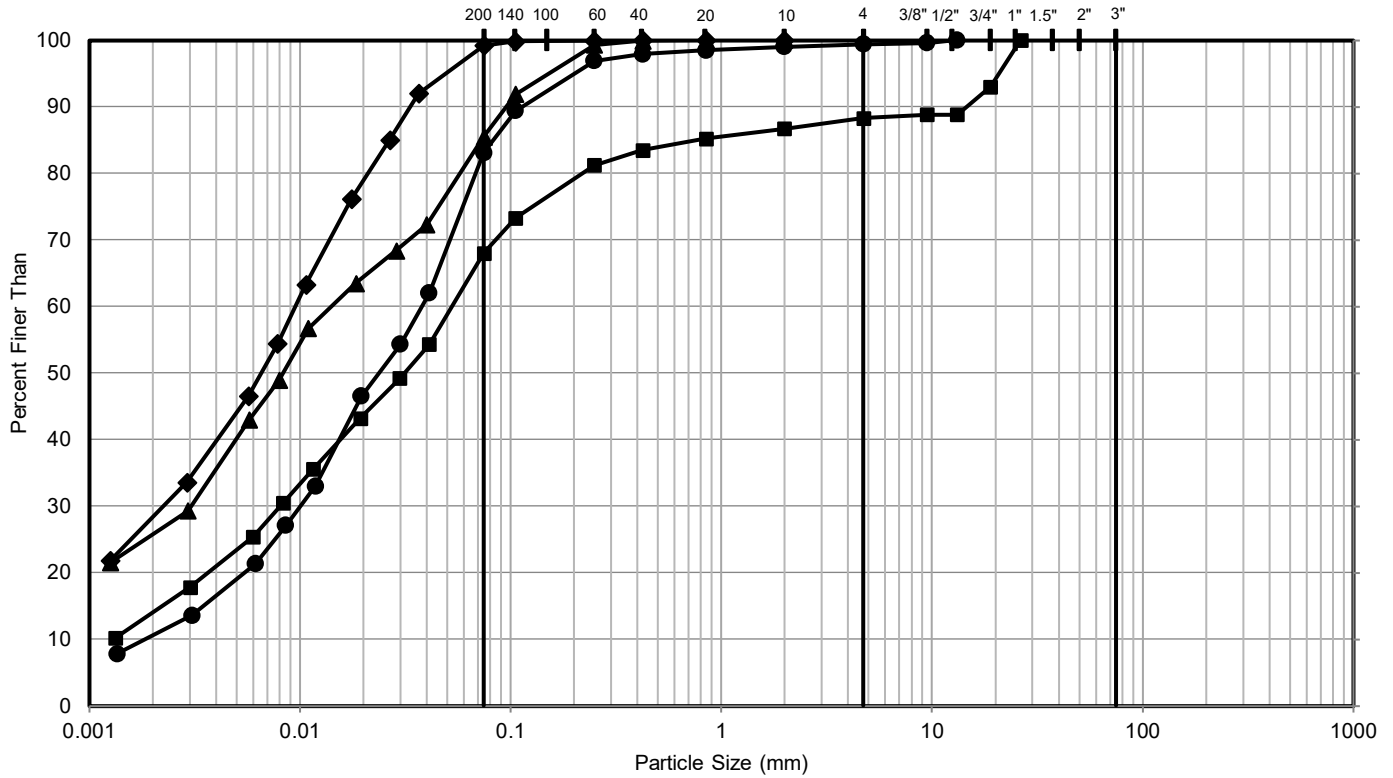
REV.

A

FIGURE

B-4

# GRAIN SIZE DISTRIBUTION - CLAYEY SILT (CL) to CLAYEY SILT-SILT (CL-ML)



FINES (Silt, Clay)	SAND			GRAVEL		COBBLES	BOULDERS
	Fine	Medium	Coarse	Fine	Coarse		

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	HRW-1	16	18.3 - 18.9	200.7 to 200.1
◆	HRW-4	17	18.3 - 18.9	199.1 to 198.5
▲	HRW-4	19	24.4 - 25.0	193.0 to 192.4
●	HRW-1	21	33.5 - 33.9	185.5 to 185.1

CLIENT

Central

CONSULTANT



YYYY-MM-DD 2022-07-23

DESIGNED CC

PREPARED CC

REVIEWED KJB

APPROVED KJB

PROJECT

Bradford Bypass - Holland River

TITLE

Grain Size Distribution LOWER COHESIVE DEPOSIT  
CLAYEY SILT (CL) to CLAYEY SILT-SILT (CL-ML)

PROJECT NO.

19136074

CONTROL

1000

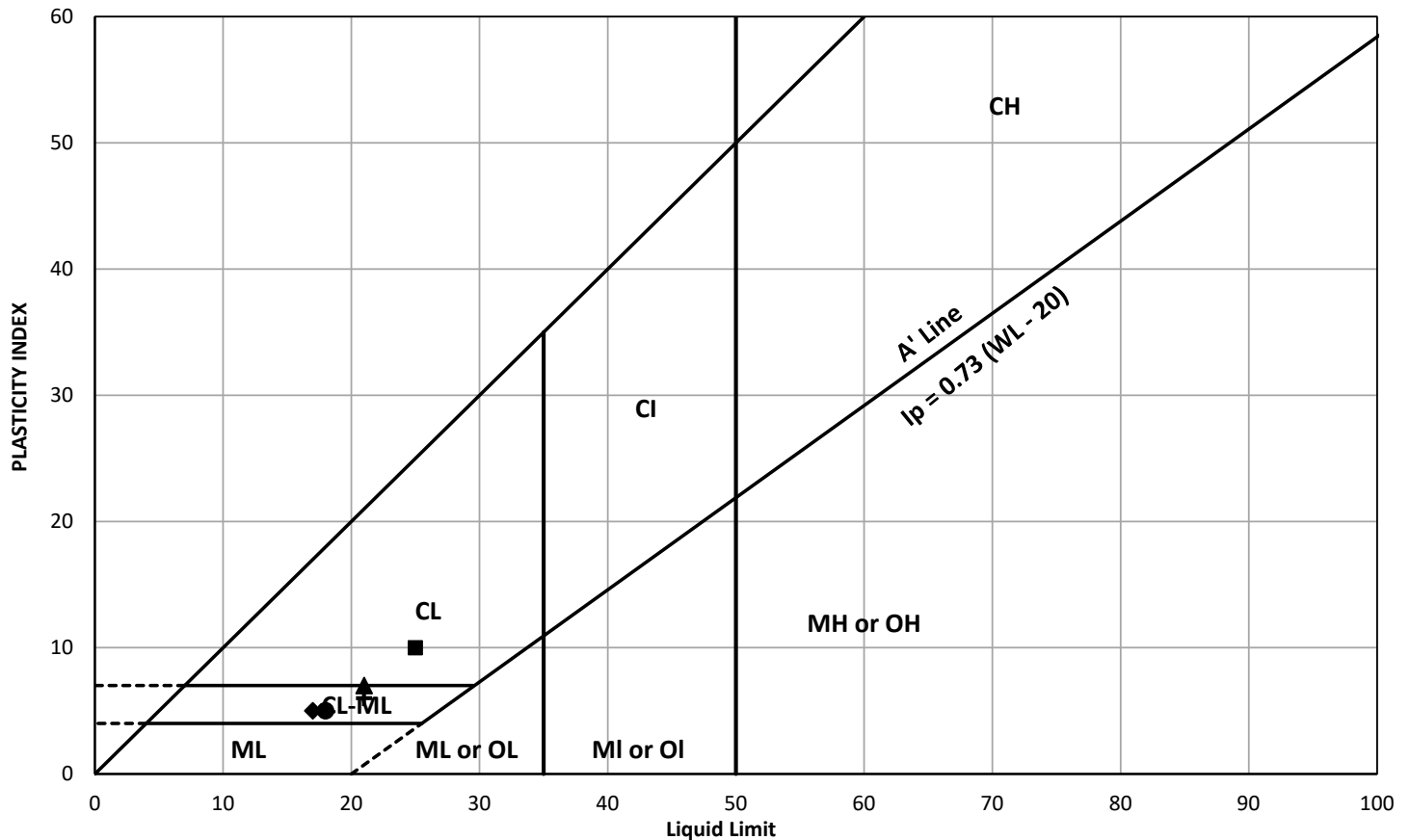
REV.

A


FIGURE

B-5

# PLASTICITY CHART - CLAYEY SILT (CL) to CLAYEY SILT-SILT (CL-ML)

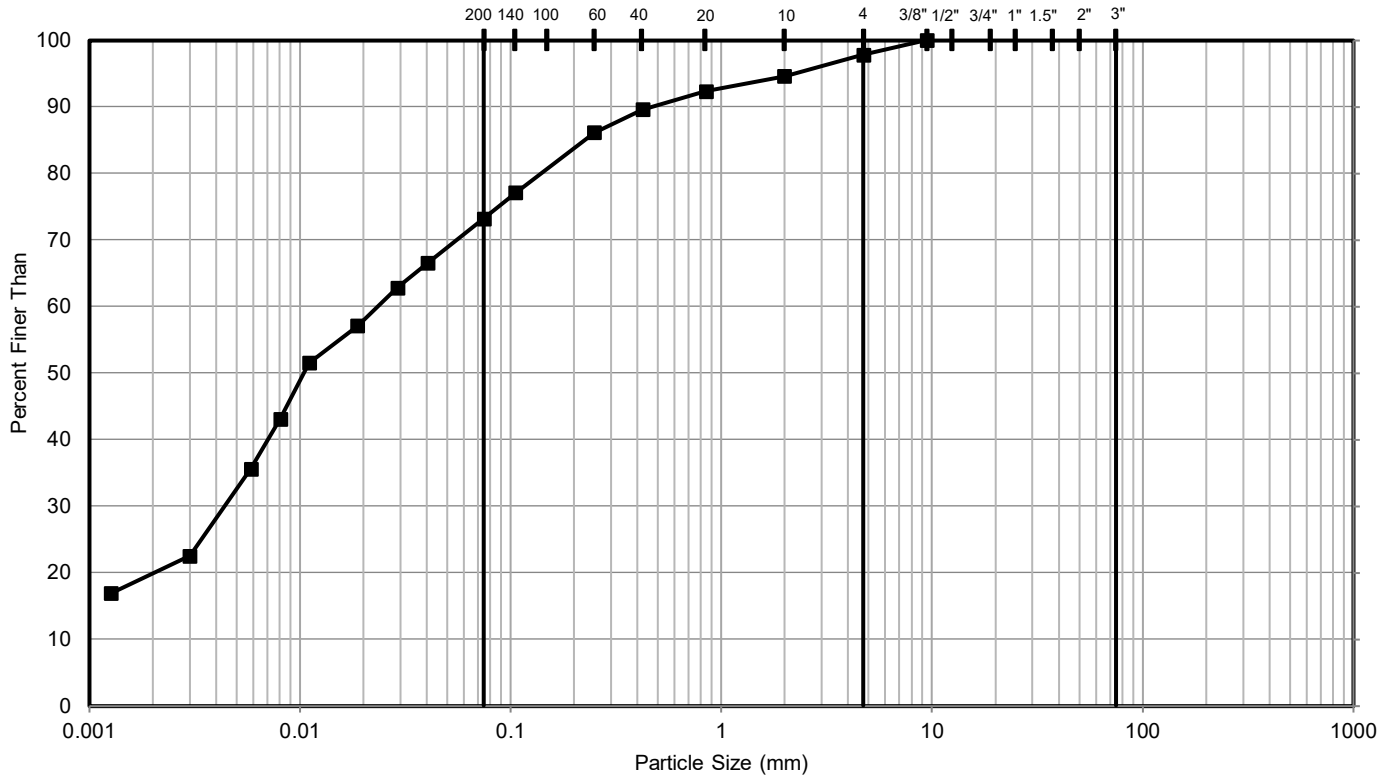


Symbol	Sample Location	Sample / Specimen Number	Elevation (m)	Natural Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
■	HRW-1	15	202.2 to 201.6	22.2	25	15	10
◆	HRW-1	16	200.7 to 200.1	9.4	17	12	5
▲	HRW-4	17	199.1 to 198.5	16.6	21	14	7
●	HRW-4	19	193.0 to 192.4	26.5	18	13	5
+	HRW-1	21	185.5 to 185.1	8.4	21	15	6

CLIENT		
Central		
CONSULTANT		
 <b>GOLDER</b> MEMBER OF WSP	YYYY-MM-DD	2022-07-22
	DESIGNED	CC
	PREPARED	CC
	REVIEWED	KJB
	APPROVED	KJB

PROJECT			
Bradford Bypass - Holland River			
TITLE			
Plasticity Chart - LOWER COHESIVE DEPOSIT Clayey Silt (CL) to CLAYEY SILT-SILT (CL-ML)			
PROJECT NO.	CONTROL	REV.	FIGURE
19136074	1000	A	B-6

## Grain Size Distribution - CLAYEY SILT-SILT (CL-ML) TILL



FINES (Silt, Clay)	SAND			GRAVEL		COBBLES	BOULDERS
	Fine	Medium	Coarse	Fine	Coarse		

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	HRW-4	25	39.6 - 40.2	177.7 to 177.1

CLIENT

Central

CONSULTANT



YYYY-MM-DD 2022-07-23

DESIGNED CC

PREPARED CC

REVIEWED KJB

APPROVED KJB

PROJECT

Bradford Bypass - Holland River

TITLE

Grain Size Distribution  
CLAYEY SILT-SILT (CL-ML) TILL

PROJECT NO.

19136074

CONTROL

1000

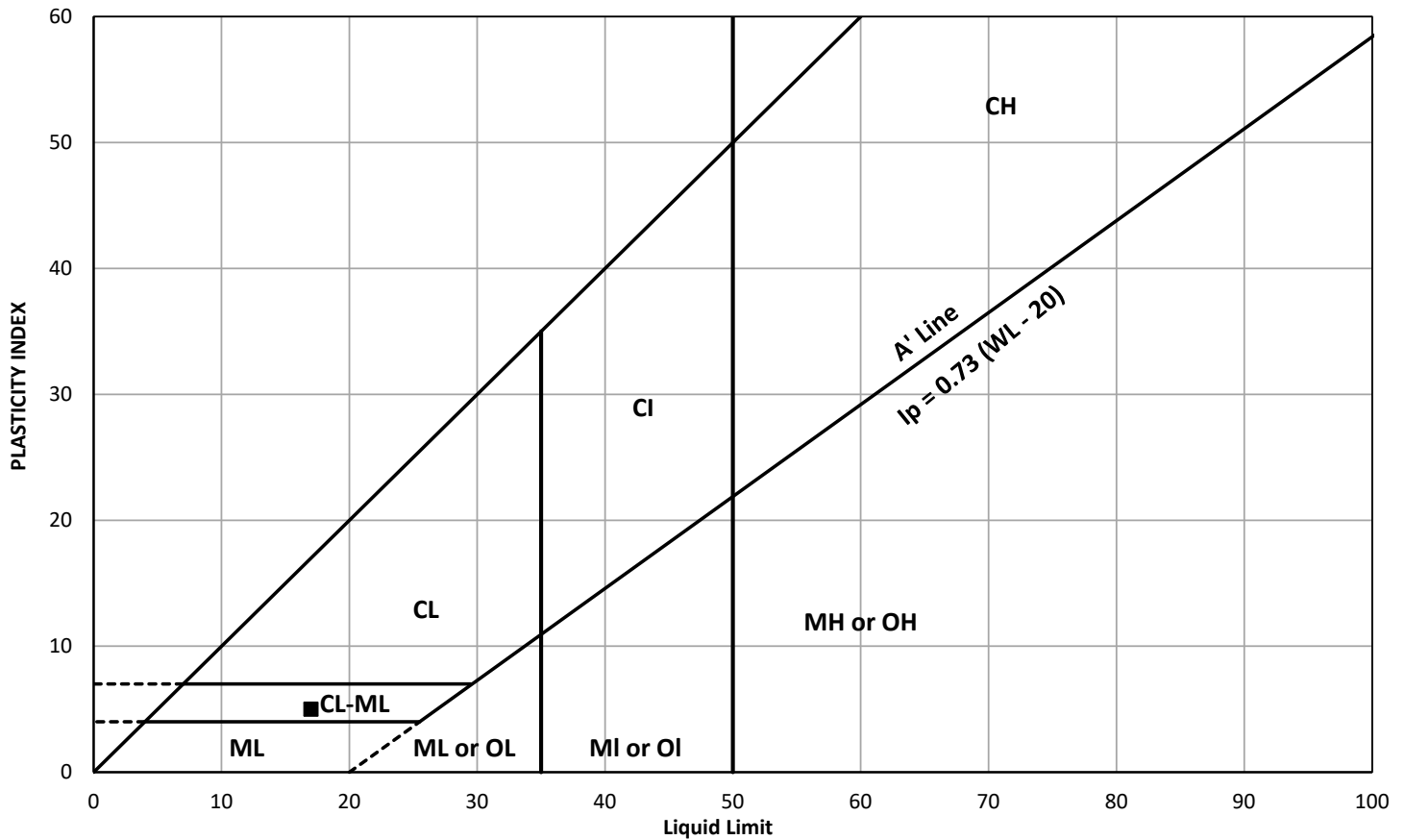
REV.

A


FIGURE

B-7

# PLASTICITY CHART - CLAYEY SILT-SILT (CL-ML) TILL

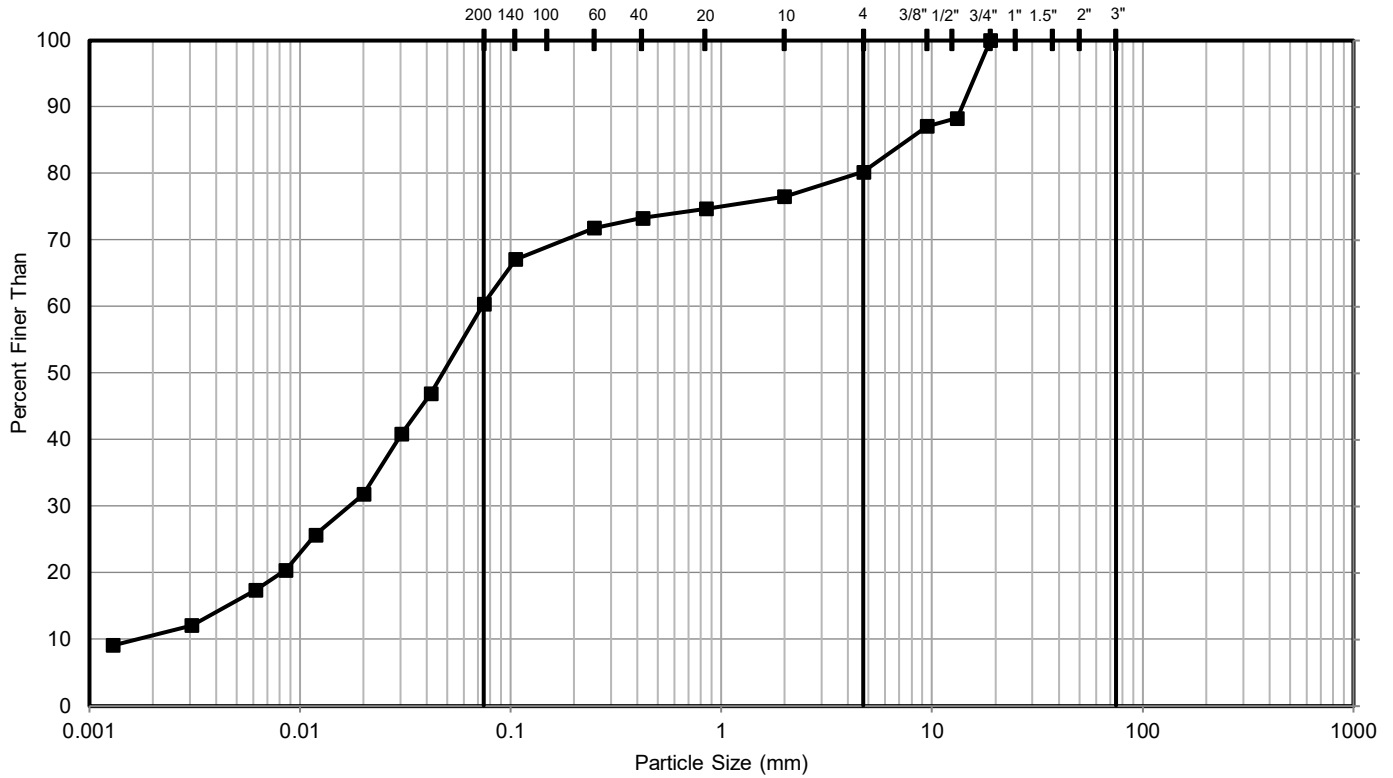


Symbol	Sample Location	Sample / Specimen Number	Elevation (m)	Natural Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
■	HRW-4	25	177.7 to 177.1	12.9	17	12	5

CLIENT		
Central		
CONSULTANT		
 <b>GOLDER</b> MEMBER OF WSP	YYYY-MM-DD	2021-03-25
	DESIGNED	CC
	PREPARED	CC
	REVIEWED	KJB
	APPROVED	KJB

PROJECT			
Bradford Bypass - Holland River			
TITLE			
Plasticity Chart - CLAYEY SILT-SILT (CL-ML) TILL			
PROJECT NO.	CONTROL	REV.	FIGURE
19136074	1000	A	B-8

## Grain Size Distribution - LOWER NON-COHESIVE DEPOSIT SANDY SILT (ML)



FINES (Silt, Clay)	SAND			GRAVEL		COBBLES	BOULDERS
	Fine	Medium	Coarse	Fine	Coarse		

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	HRW-4	28	48.8 - 49.2	168.6 to 168.1

CLIENT

Central

CONSULTANT



YYYY-MM-DD 2022-07-23

DESIGNED CC

PREPARED CC

REVIEWED KJB

APPROVED KJB

PROJECT

Bradford Bypass - Holland River

TITLE

Grain Size Distribution - LOWER NON-COHESIVE DEPOSIT  
SANDY SILT (ML)

PROJECT NO.

19136074

CONTROL

1000

REV.

A

FIGURE

B-9

**CONSOLIDATION TEST SUMMARY****ASTM D2435/D2435M****FIGURE B-10****SAMPLE IDENTIFICATION**

Project Number	19136074	Sample Number	TO8
Borehole Number	HRW4	Sample Depth, m	5.18-5.87

**TEST CONDITIONS**

Test Type	Laboratory Standard	Load Duration, hr	24
Oedometer Number	2		
Date Started	11/3/2021		
Date Completed	11/17/2021		

**SAMPLE DIMENSIONS AND PROPERTIES - INITIAL**

Sample Height, cm	2.54	Unit Weight, kN/m <sup>3</sup>	19.83
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m <sup>3</sup>	16.13
Area, cm <sup>2</sup>	31.52	Specific Gravity, measured	2.71
Volume, cm <sup>3</sup>	80.19	Solids Height, cm	1.544
Water Content, %	22.95	Volume of Solids, cm <sup>3</sup>	48.66
Wet Mass, g	162.12	Volume of Voids, cm <sup>3</sup>	31.53
Dry Mass, g	131.86	Degree of Saturation, %	96.0

**TEST COMPUTATIONS**

Stress kPa	Corr. Height cm	Void Ratio	Average		t <sub>90</sub> sec	cv. cm <sup>2</sup> /s	mv m <sup>2</sup> /kN	k cm/s
	Height cm		Height cm					
0.00	2.544	0.648	2.544					
5.93	2.544	0.648	2.544	29	4.73E-02	1.47E-05	6.83E-08	
10.73	2.543	0.647	2.543	25	5.48E-02	8.83E-05	4.74E-07	
20.68	2.538	0.644	2.540	109	1.26E-02	1.94E-04	2.38E-07	
39.91	2.528	0.638	2.533	73	1.86E-02	2.00E-04	3.66E-07	
79.05	2.516	0.630	2.522	66	2.04E-02	1.24E-04	2.47E-07	
40.24	2.516	0.630	2.516					
10.73	2.517	0.631	2.517					
40.24	2.517	0.630	2.517	38	3.53E-02	7.99E-06	2.77E-08	
78.93	2.513	0.628	2.515	49	2.74E-02	4.17E-05	1.12E-07	
156.58	2.497	0.617	2.505	118	1.13E-02	8.00E-05	8.83E-08	
311.92	2.471	0.601	2.484	144	9.08E-03	6.48E-05	5.77E-08	
622.79	2.438	0.580	2.455	101	1.26E-02	4.16E-05	5.16E-08	
1244.32	2.389	0.548	2.414	94	1.31E-02	3.11E-05	4.01E-08	
2487.34	2.327	0.507	2.358	101	1.17E-02	1.97E-05	2.25E-08	
622.79	2.337	0.514	2.332					
156.60	2.356	0.526	2.346					
40.00	2.369	0.535	2.362					
10.76	2.388	0.547	2.378					

Note:

Consolidation loading and unloading schedule assigned by the client.

cv and k are approximate only based on t<sub>90</sub> estimated from Square Root of Time Method (ASTMD2435/2435M)

Specimen taken 15-20cm from top of the tube.

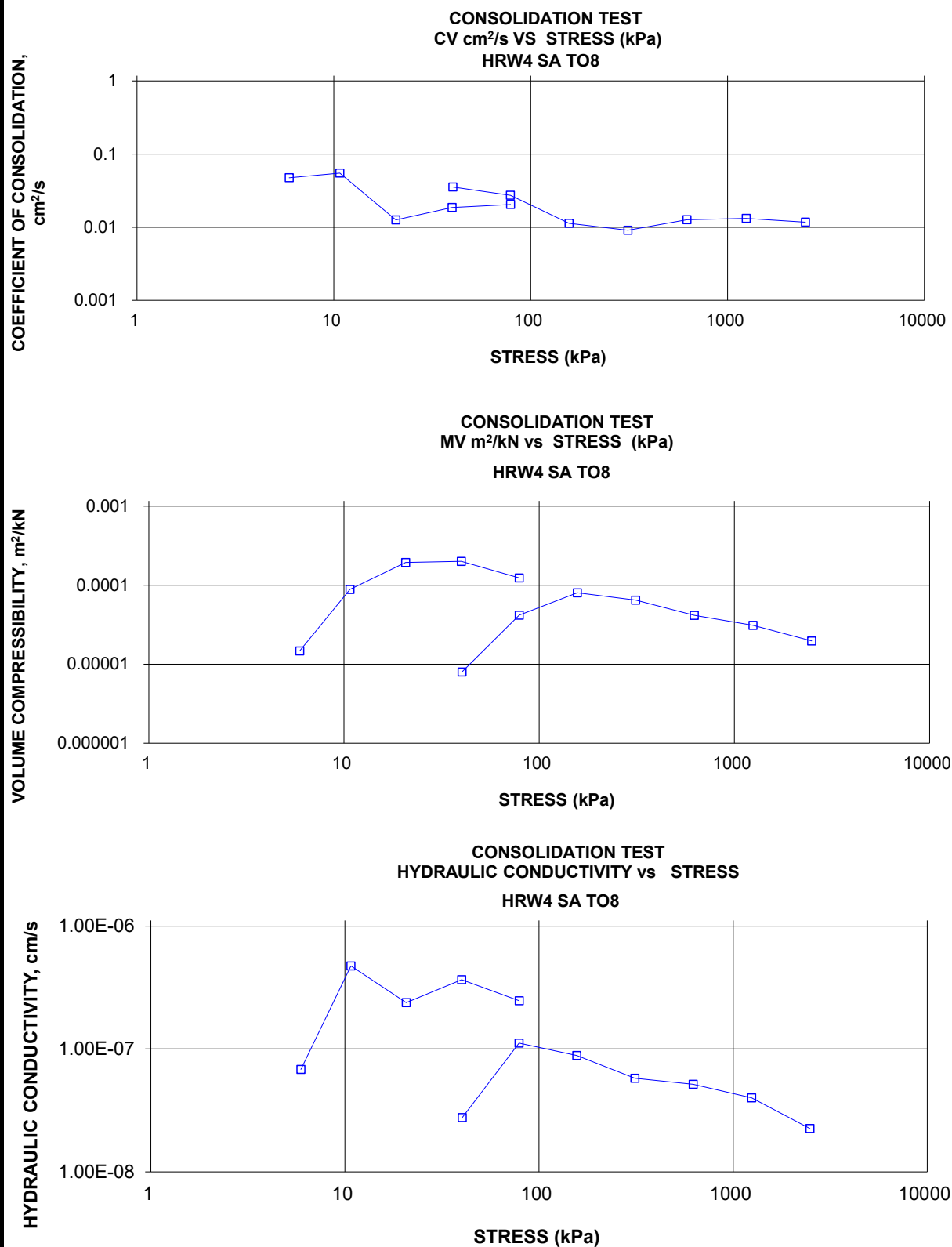
**SAMPLE DIMENSIONS AND PROPERTIES - FINAL**

Sample Height, cm	2.39	Unit Weight, kN/m <sup>3</sup>	20.73
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m <sup>3</sup>	17.18
Area, cm <sup>2</sup>	31.52	Specific Gravity, measured	2.71
Volume, cm <sup>3</sup>	75.25	Solids Height, cm	1.544
Water Content, %	20.65	Volume of Solids, cm <sup>3</sup>	48.66
Wet Mass, g	159.09	Volume of Voids, cm <sup>3</sup>	26.60
Dry Mass, g	131.86		

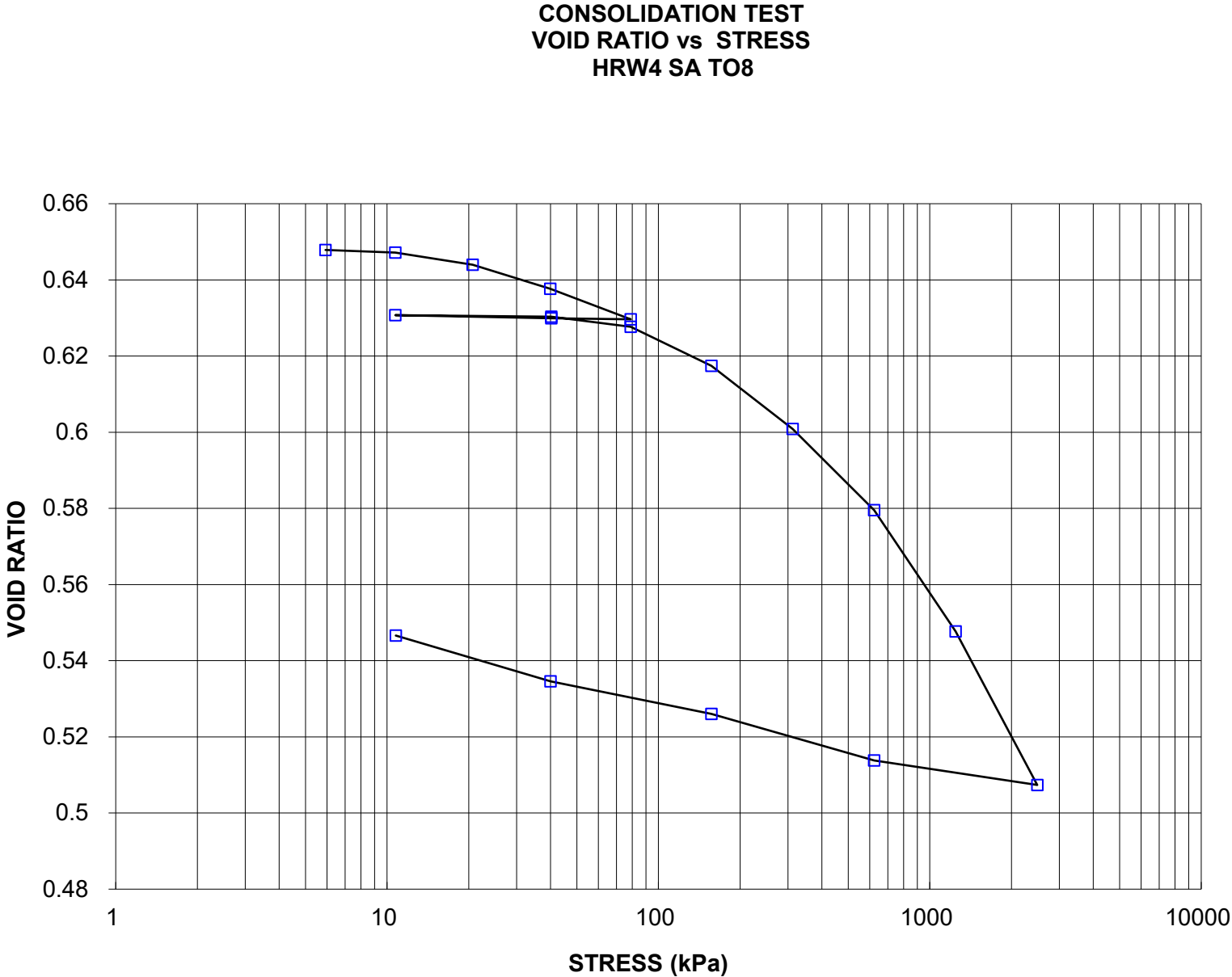
Prepared By: LH

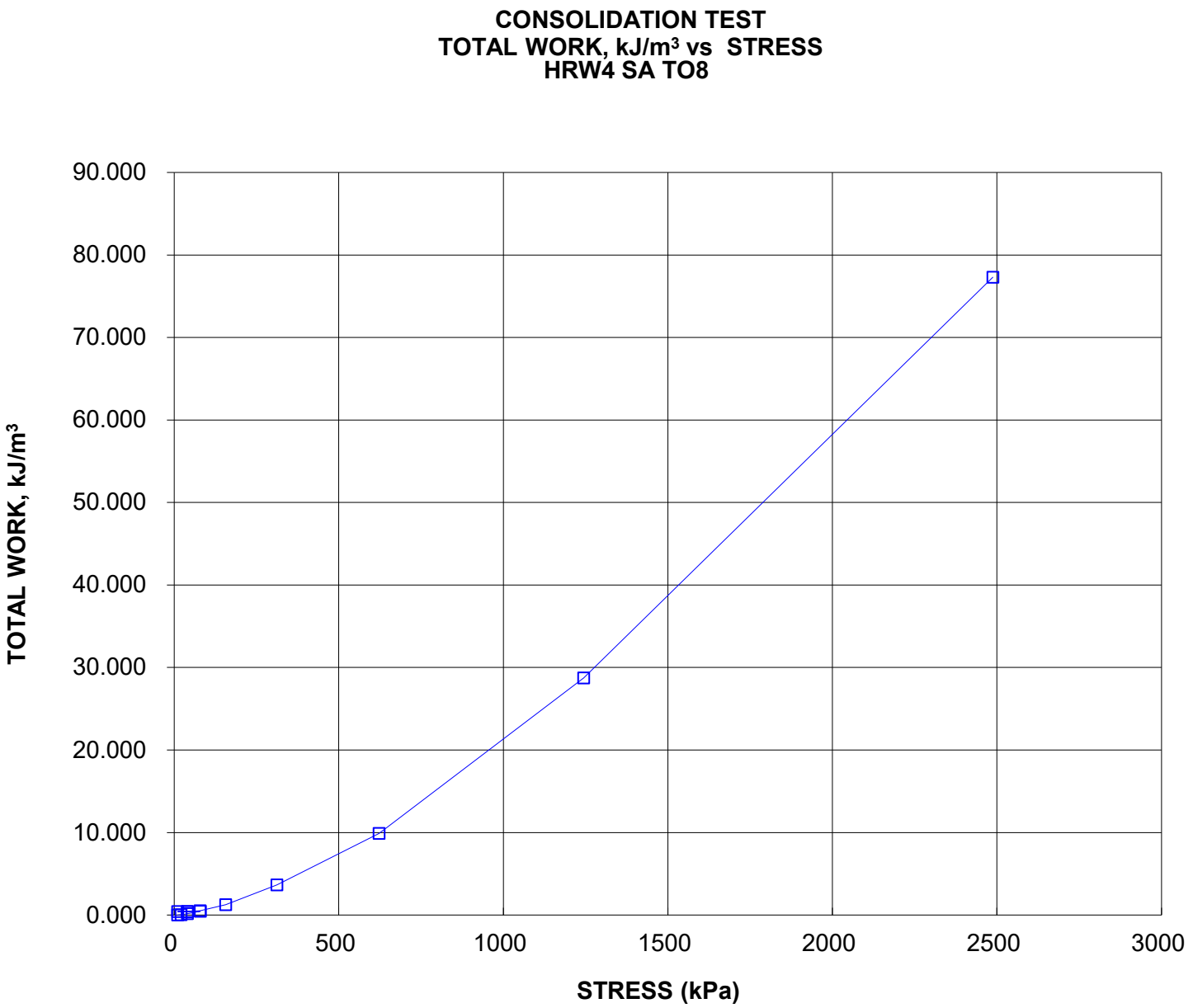
**Golder Associates**

Checked By: MM









APPENDIX C

# Analytical Chemical Test Results



Your P.O. #: 19136074  
 Your Project #: 19136074  
 Site Location: BRADFORD  
 Your C.O.C. #: n/a

**Attention: Manisha Ahuja**

Golder Associates Ltd  
 6925 Century Ave  
 Suite 100  
 Mississauga, ON  
 CANADA L5N 7K2

**Report Date: 2021/10/26**  
 Report #: R6873206  
 Version: 1 - Final

**CERTIFICATE OF ANALYSIS**

**BV LABS JOB #: C1U4495**

**Received: 2021/10/19, 17:12**

Sample Matrix: Soil  
 # Samples Received: 2

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Analytical Method
Chloride (20:1 extract)	2	2021/10/25	2021/10/26	CAM SOP-00463	SM 23 4500-Cl E m
Conductivity	2	2021/10/25	2021/10/26	CAM SOP-00414	OMOE E3530 v1 m
Moisture (Subcontracted) (1, 2)	2	N/A	2021/10/25	AB SOP-00002	CCME PHC-CWS m
Sulphide in Soil (1)	2	N/A	2021/10/25	AB SOP-00080	EPA9030B/SM4500S2-DF
pH CaCl2 EXTRACT	1	2021/10/22	2021/10/22	CAM SOP-00413	EPA 9045 D m
pH CaCl2 EXTRACT	1	2021/10/25	2021/10/25	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	2	2021/10/20	2021/10/26	CAM SOP-00414	SM 23 2510 m
Sulphate (20:1 Extract)	2	2021/10/25	2021/10/26	CAM SOP-00464	EPA 375.4 m

**Remarks:**

Bureau Veritas is accredited to ISO/IEC 17025 for specific parameters on scopes of accreditation. Unless otherwise noted, procedures used by Bureau Veritas are based upon recognized Provincial, Federal or US method compendia such as CCME, MELCC, EPA, APHA.

All work recorded herein has been done in accordance with procedures and practices ordinarily exercised by professionals in Bureau Veritas' profession using accepted testing methodologies, quality assurance and quality control procedures (except where otherwise agreed by the client and Bureau Veritas in writing). All data is in statistical control and has met quality control and method performance criteria unless otherwise noted. All method blanks are reported; unless indicated otherwise, associated sample data are not blank corrected. Where applicable, unless otherwise noted, Measurement Uncertainty has not been accounted for when stating conformity to the referenced standard.

Bureau Veritas liability is limited to the actual cost of the requested analyses, unless otherwise agreed in writing. There is no other warranty expressed or implied. Bureau Veritas has been retained to provide analysis of samples provided by the Client using the testing methodology referenced in this report. Interpretation and use of test results are the sole responsibility of the Client and are not within the scope of services provided by Bureau Veritas, unless otherwise agreed in writing. Bureau Veritas is not responsible for the accuracy or any data impacts, that result from the information provided by the customer or their agent.

Solid sample results, except biota, are based on dry weight unless otherwise indicated. Organic analyses are not recovery corrected except for isotope dilution methods.

Results relate to samples tested. When sampling is not conducted by Bureau Veritas, results relate to the supplied samples tested.

This Certificate shall not be reproduced except in full, without the written approval of the laboratory.

Reference Method suffix "m" indicates test methods incorporate validated modifications from specific reference methods to improve performance.

\* RPDs calculated using raw data. The rounding of final results may result in the apparent difference.

(1) This test was performed by Bureau Veritas Calgary (19th), 4000 19th Street NE, Calgary, AB, T2E 6P8

(2) Offsite analysis requires that subcontracted moisture be reported.



Your P.O. #: 19136074  
Your Project #: 19136074  
Site Location: BRADFORD  
Your C.O.C. #: n/a

**Attention: Manisha Ahuja**

Golder Associates Ltd  
6925 Century Ave  
Suite 100  
Mississauga, ON  
CANADA L5N 7K2

**Report Date: 2021/10/26**  
Report #: R6873206  
Version: 1 - Final

**CERTIFICATE OF ANALYSIS**

**BV LABS JOB #: C1U4495**  
**Received: 2021/10/19, 17:12**

**Encryption Key**

Please direct all questions regarding this Certificate of Analysis to your Project Manager.  
Ema Gitej, Senior Project Manager  
Email: emese.gitej@bureauveritas.com  
Phone# (905)817-5829

=====

This report has been generated and distributed using a secure automated process.

BV Labs has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per ISO/IEC 17025, signing the reports. For Service Group specific validation please refer to the Validation Signature Page.



BUREAU  
VERITAS

Bureau Veritas Job #: C1U4495

Report Date: 2021/10/26

Golder Associates Ltd

Client Project #: 19136074

Site Location: BRADFORD

Your P.O. #: 19136074

Sampler Initials: MTI

### SOIL CORROSIVITY PACKAGE (SOIL)

<b>Bureau Veritas ID</b>		QZC417			QZC417			QZC418		
<b>Sampling Date</b>		2021/09/29			2021/09/29			2021/10/06		
<b>COC Number</b>		n/a			n/a			n/a		
	<b>UNITS</b>	<b>HRE-1 SS #01</b>	<b>RDL</b>	<b>QC Batch</b>	<b>HRE-1 SS #01 Lab-Dup</b>	<b>RDL</b>	<b>QC Batch</b>	<b>HRW-4 SA-01</b>	<b>RDL</b>	<b>QC Batch</b>
<b>Calculated Parameters</b>										
Resistivity	ohm-cm	4800		7648263				4700		7648263
<b>Inorganics</b>										
Soluble (20:1) Chloride (Cl-)	ug/g	<20	20	7657911	<20	20	7657911	34	20	7657911
Conductivity	umho/cm	209	2	7658077	199	2	7658077	213	2	7658077
Available (CaCl2) pH	pH	7.60		7653694	7.64		7653694	7.59		7657473
Soluble (20:1) Sulphate (SO4)	ug/g	<20	20	7657917				<20	20	7657917
Sulphide	mg/kg	2.5 (1)	0.5	7659305				<0.5 (1)	0.5	7659305
<b>Physical Testing</b>										
Moisture-Subcontracted	%	15	0.30	7659304				15	0.30	7659304
RDL = Reportable Detection Limit QC Batch = Quality Control Batch Lab-Dup = Laboratory Initiated Duplicate (1) Analyzed past method specified hold time										



BUREAU  
VERITAS

Bureau Veritas Job #: C1U4495

Report Date: 2021/10/26

Golder Associates Ltd

Client Project #: 19136074

Site Location: BRADFORD

Your P.O. #: 19136074

Sampler Initials: MTI

## TEST SUMMARY

**Bureau Veritas ID:** QZC417  
**Sample ID:** HRE-1 SS #01  
**Matrix:** Soil

**Collected:** 2021/09/29  
**Shipped:**  
**Received:** 2021/10/19

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	7657911	2021/10/25	2021/10/26	Alina Dobreanu
Conductivity	AT	7658077	2021/10/25	2021/10/26	Kien Tran
Moisture (Subcontracted)	BAL	7659304	N/A	2021/10/25	Salini Vidhyadharan
Sulphide in Soil	SPEC	7659305	N/A	2021/10/25	Preetleen Kathuria
pH CaCl2 EXTRACT	AT	7653694	2021/10/22	2021/10/22	Taslina Aktar
Resistivity of Soil		7648263	2021/10/26	2021/10/26	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	7657917	2021/10/25	2021/10/26	Avneet Kour Sudan

**Bureau Veritas ID:** QZC417 Dup  
**Sample ID:** HRE-1 SS #01  
**Matrix:** Soil

**Collected:** 2021/09/29  
**Shipped:**  
**Received:** 2021/10/19

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	7657911	2021/10/25	2021/10/26	Alina Dobreanu
Conductivity	AT	7658077	2021/10/25	2021/10/26	Kien Tran
pH CaCl2 EXTRACT	AT	7653694	2021/10/22	2021/10/22	Taslina Aktar

**Bureau Veritas ID:** QZC418  
**Sample ID:** HRW-4 SA-01  
**Matrix:** Soil

**Collected:** 2021/10/06  
**Shipped:**  
**Received:** 2021/10/19

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	7657911	2021/10/25	2021/10/26	Alina Dobreanu
Conductivity	AT	7658077	2021/10/25	2021/10/26	Kien Tran
Moisture (Subcontracted)	BAL	7659304	N/A	2021/10/25	Salini Vidhyadharan
Sulphide in Soil	SPEC	7659305	N/A	2021/10/25	Preetleen Kathuria
pH CaCl2 EXTRACT	AT	7657473	2021/10/25	2021/10/25	Taslina Aktar
Resistivity of Soil		7648263	2021/10/26	2021/10/26	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	7657917	2021/10/25	2021/10/26	Avneet Kour Sudan



BUREAU  
VERITAS

Bureau Veritas Job #: C1U4495

Report Date: 2021/10/26

Golder Associates Ltd

Client Project #: 19136074

Site Location: BRADFORD

Your P.O. #: 19136074

Sampler Initials: MTI

### GENERAL COMMENTS

Each temperature is the average of up to three cooler temperatures taken at receipt

Package 1	12.7°C
-----------	--------

**Results relate only to the items tested.**





BUREAU  
VERITAS

Bureau Veritas Job #: C1U4495

Report Date: 2021/10/26

## QUALITY ASSURANCE REPORT

Golder Associates Ltd

Client Project #: 19136074

Site Location: BRADFORD

Your P.O. #: 19136074

Sampler Initials: MTI

QC Batch	Parameter	Date	Matrix Spike		SPIKED BLANK		Method Blank		RPD	
			% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
7653694	Available (CaCl <sub>2</sub> ) pH	2021/10/22			100	97 - 103			0.52	N/A
7657473	Available (CaCl <sub>2</sub> ) pH	2021/10/25			100	97 - 103			1.4	N/A
7657911	Soluble (20:1) Chloride (Cl <sup>-</sup> )	2021/10/26	113	70 - 130	101	70 - 130	<20	ug/g	NC	35
7657917	Soluble (20:1) Sulphate (SO <sub>4</sub> )	2021/10/26	145 (1)	70 - 130	107	70 - 130	<20	ug/g	NC	35
7658077	Conductivity	2021/10/26			100	90 - 110	<2	umho/cm	4.8	10
7659304	Moisture-Subcontracted	2021/10/25					<0.30	%	3.4	20
7659305	Sulphide	2021/10/25	90	75 - 125	98	75 - 125	<0.5	mg/kg	NC	30

N/A = Not Applicable

Duplicate: Paired analysis of a separate portion of the same sample. Used to evaluate the variance in the measurement.

Matrix Spike: A sample to which a known amount of the analyte of interest has been added. Used to evaluate sample matrix interference.

Spiked Blank: A blank matrix sample to which a known amount of the analyte, usually from a second source, has been added. Used to evaluate method accuracy.

Method Blank: A blank matrix containing all reagents used in the analytical procedure. Used to identify laboratory contamination.

NC (Duplicate RPD): The duplicate RPD was not calculated. The concentration in the sample and/or duplicate was too low to permit a reliable RPD calculation (absolute difference <= 2x RDL).

(1) Recovery or RPD for this parameter is outside control limits. The overall quality control for this analysis meets acceptability criteria.



BUREAU  
VERITAS

Bureau Veritas Job #: C1U4495

Report Date: 2021/10/26

Golder Associates Ltd

Client Project #: 19136074

Site Location: BRADFORD

Your P.O. #: 19136074

Sampler Initials: MTI

### VALIDATION SIGNATURE PAGE

The analytical data and all QC contained in this report were reviewed and validated by:

Veronica Falk, B.Sc., P.Chem., QP, Scientific Specialist, Organics

Maria Magdalena Florescu, Ph.D., P.Chem., QP, Inorganics Manager

Ewa Pranjić, M.Sc., C.Chem, Scientific Specialist

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6740 Campobello Road, Mississauga, Ontario L5N 2L8  
Phone: 905-817-5700 Fax: 905-817-5779 Toll Free: 800-563-6266  
CAM FCD-01191/5

## WORK ORDER

### CHAIN OF CUSTODY RECORD

Page 1 of 1

Invoice Information		Report Information (if differs from invoice)		Project Information (where applicable)		Turnaround Time (TAT) Required	
Company Name: <b>Golder Associates Ltd.</b>	Company Name: <b>same</b>	Quotation #: <b>Golder rates</b>	<input checked="" type="checkbox"/> Regular TAT (5-7 days) Most analyses		PLEASE PROVIDE ADVANCE NOTICE FOR RUSH PROJECTS		
Contact Name: <b>Manisha Ahuja</b>	Contact Name:	P.O. #/ AFE#: <b>19136074</b>	<input type="checkbox"/> Rush TAT (Surcharges will be applied)		<input type="checkbox"/> 1 Day <input type="checkbox"/> 2 Days <input type="checkbox"/> 3-4 Days		
Address: <b>6925 Century Ave., Suite 100</b>	Address:	Project #: <b>19136074</b>	Site Location: <b>Bradford</b>		Date Required:		
Mississauga, ON	Phone: <b>647-239-0174</b> Fax:	Site #:	Site Location Province: <b>Ontario</b>		Rush Confirmation #:		
Email: <b>canadaaccounts@payableinvoices@golder.com; Manisha_Ahuja@golder.com</b>	Email: <b>Manisha_Ahuja@golder.com; 120387@golder.com</b>	Sampled By:	Analysis Requested		LABORATORY USE ONLY		
MOE REGULATED DRINKING WATER OR WATER INTENDED FOR HUMAN CONSUMPTION MUST BE SUBMITTED ON THE BUREAU VERITAS LABORATORIES' DRINKING WATER CHAIN OF CUSTODY						CUSTODY SEAL Y/N	
Regulation 153 <input type="checkbox"/> Table 1 <input type="checkbox"/> Res/Park <input type="checkbox"/> Med/ Fine <input type="checkbox"/> Table 2 <input type="checkbox"/> Ind/Comm <input type="checkbox"/> Coarse <input type="checkbox"/> Table 3 <input type="checkbox"/> Agri/ Other <input type="checkbox"/> Table <input type="checkbox"/> FOR RSC (PLEASE CIRCLE) Y / N		Other Regulations <input type="checkbox"/> CCME <input type="checkbox"/> Sanitary Sewer Bylaw <input type="checkbox"/> MISA <input type="checkbox"/> Storm Sewer Bylaw <input type="checkbox"/> PWQO <input type="checkbox"/> Region <input type="checkbox"/> Other (Specify) <input type="checkbox"/> REG 558 (MIN. 3 DAY TAT REQUIRED)		Analysis Requested FIELD FILTERED (CIRCLE) Metals / Ig / Cr / V BTEX / PHC / FL PHC / F2 / F4 VOCs REG 153 METALS & INORGANICS REG 153 ICPMS METALS REG 153 METALS (Hg, Cr, V, ICPMS Metals, HWS, B) CORROSIVITY PACKAGE (+ SULPHIDE) HOLD - DO NOT ANALYZE		COOLER TEMPERATURES Present Intact N N 11/14/13	
Include Criteria on Certificate of Analysis: Y / N		SAMPLES MUST BE KEPT COOL (< 10 °C) FROM TIME OF SAMPLING UNTIL DELIVERY TO BUREAU VERITAS				COOLING MEDIA PRESENT: (Y) N	
SAMPLE IDENTIFICATION		DATE SAMPLED (YYYY/MM/DD)	TIME SAMPLED (HH:MM)	MATRIX	COMMENTS		
1 HRE-1 SS #01	2021-09-29	PM	Soil	2	2 JARS. NO REDOX POTENTIAL.		
2 HRW-4 SA-01	2021-10-06	PM	Soil	2	2 JARS. NO REDOX POTENTIAL.		
3							
4							
5							
6							
7							
8							
9							
10							
RELINQUISHED BY: (Signature/Print)	DATE: (YYYY/MM/DD)	TIME: (HH:MM)	RECEIVED BY: (Signature/Print)	DATE: (YYYY/MM/DD)	TIME: (HH:MM)		
Muhammad Talha Irshad	2021-10-19		Y. Thompson	21/10/19	17:12		

MSA with BV Signed May 18, 2020.  
Golder standing offer rates in email from Julie Clement dated Sept 20, 2021.  
Corrosivity package including chloride, conductivity, resistivity, pH, sulphate, sulphide is \$98.60/sample.

Unless otherwise agreed to in writing, work submitted on this Chain of Custody is subject to Bureau Veritas Laboratories' standard Terms and Conditions. Signing of this Chain of Custody document is acknowledgment of the above terms and conditions at <http://www.bv-labs.com/terms-and-conditions>

19-Oct-21 17:12  
Ema Gitej  
CIU4495

ENV 970



Your P.O. #: 19136074  
Your Project #: 19136074  
Site Location: BRADFORD  
Your C.O.C. #: n/a

**Attention: Mohammed Taha**

Golder Associates Ltd  
6925 Century Ave  
Suite 100  
Mississauga, ON  
CANADA L5N 7K2

**Report Date: 2022/09/06**  
Report #: R7285025  
Version: 1 - Final

**CERTIFICATE OF ANALYSIS**

**BUREAU VERITAS JOB #: C205551**

**Received: 2022/08/26, 15:58**

Sample Matrix: Soil  
# Samples Received: 9

Analyses	Quantity	Date	Date	Laboratory Method	Analytical Method
		Extracted	Analyzed		
Chloride (20:1 extract)	9	2022/09/01	2022/09/02	CAM SOP-00463	SM 23 4500-Cl E m
Conductivity	9	2022/08/31	2022/09/01	CAM SOP-00414	OMOE E3530 v1 m
Moisture (Subcontracted) (1, 2)	9	N/A	2022/09/02	AB SOP-00002	CCME PHC-CWS m
Sulphide in Soil (1)	9	N/A	2022/09/02	AB SOP-00080	EPA9030B/SM4500S2-DF
pH CaCl2 EXTRACT	9	2022/08/31	2022/08/31	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	9	2022/08/29	2022/09/01	CAM SOP-00414	SM 23 2510 m
Sulphate (20:1 Extract)	9	2022/09/01	2022/09/02	CAM SOP-00464	EPA 375.4 m

**Remarks:**

Bureau Veritas is accredited to ISO/IEC 17025 for specific parameters on scopes of accreditation. Unless otherwise noted, procedures used by Bureau Veritas are based upon recognized Provincial, Federal or US method compendia such as CCME, MELCC, EPA, APHA.

All work recorded herein has been done in accordance with procedures and practices ordinarily exercised by professionals in Bureau Veritas' profession using accepted testing methodologies, quality assurance and quality control procedures (except where otherwise agreed by the client and Bureau Veritas in writing). All data is in statistical control and has met quality control and method performance criteria unless otherwise noted. All method blanks are reported; unless indicated otherwise, associated sample data are not blank corrected. Where applicable, unless otherwise noted, Measurement Uncertainty has not been accounted for when stating conformity to the referenced standard.

Bureau Veritas liability is limited to the actual cost of the requested analyses, unless otherwise agreed in writing. There is no other warranty expressed or implied. Bureau Veritas has been retained to provide analysis of samples provided by the Client using the testing methodology referenced in this report. Interpretation and use of test results are the sole responsibility of the Client and are not within the scope of services provided by Bureau Veritas, unless otherwise agreed in writing. Bureau Veritas is not responsible for the accuracy or any data impacts, that result from the information provided by the customer or their agent.

Solid sample results, except biota, are based on dry weight unless otherwise indicated. Organic analyses are not recovery corrected except for isotope dilution methods.

Results relate to samples tested. When sampling is not conducted by Bureau Veritas, results relate to the supplied samples tested.

This Certificate shall not be reproduced except in full, without the written approval of the laboratory.

Reference Method suffix "m" indicates test methods incorporate validated modifications from specific reference methods to improve performance.

\* RPDs calculated using raw data. The rounding of final results may result in the apparent difference.

(1) This test was performed by Bureau Veritas Calgary (19th), 4000 19th Street NE, Calgary, AB, T2E 6P8

(2) Offsite analysis requires that subcontracted moisture be reported.



Your P.O. #: 19136074  
Your Project #: 19136074  
Site Location: BRADFORD  
Your C.O.C. #: n/a

**Attention: Mohammed Taha**

Golder Associates Ltd  
6925 Century Ave  
Suite 100  
Mississauga, ON  
CANADA L5N 7K2

**Report Date: 2022/09/06**  
Report #: R7285025  
Version: 1 - Final

**CERTIFICATE OF ANALYSIS**

**BUREAU VERITAS JOB #: C205551**

**Received: 2022/08/26, 15:58**

**Encryption Key**

Please direct all questions regarding this Certificate of Analysis to your Project Manager.

Ankita Bhalla, Project Manager

Email: Ankita.Bhalla@bureauveritas.com

Phone# (905) 817-5700

=====

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For Service Group specific validation please refer to the Validation Signature Page.



BUREAU  
VERITAS

Bureau Veritas Job #: C205551  
Report Date: 2022/09/06

Golder Associates Ltd  
Client Project #: 19136074  
Site Location: BRADFORD  
Your P.O. #: 19136074  
Sampler Initials: MTI

### SOIL CORROSIVITY PACKAGE (SOIL)

<b>Bureau Veritas ID</b>		TOL232			TOL232			TOL233		
<b>Sampling Date</b>		2021/06/15			2021/06/15			2021/06/17		
<b>COC Number</b>		n/a			n/a			n/a		
	<b>UNITS</b>	<b>404-2 SA#4</b>	<b>RDL</b>	<b>QC Batch</b>	<b>404-2 SA#4 Lab-Dup</b>	<b>RDL</b>	<b>QC Batch</b>	<b>404-3 SA#3</b>	<b>RDL</b>	<b>QC Batch</b>
<b>Calculated Parameters</b>										
Resistivity	ohm-cm	3200		8193773				810		8193773
<b>Inorganics</b>										
Soluble (20:1) Chloride (Cl-)	ug/g	130	20	8201208				570	20	8201208
Conductivity	umho/cm	316	2	8198559				1240	2	8198559
Available (CaCl2) pH	pH	7.87		8199338				8.02		8199338
Soluble (20:1) Sulphate (SO4)	ug/g	46	20	8201217				250	20	8201217
Sulphide	mg/kg	6.4 (1)	0.5	8205122	5.2	0.5	8205122	3.7 (2)	0.5	8205122
<b>Physical Testing</b>										
Moisture-Subcontracted	%	18	0.30	8205121				3.9	0.30	8205121
RDL = Reportable Detection Limit QC Batch = Quality Control Batch Lab-Dup = Laboratory Initiated Duplicate (1) Sample extracted past method-specified hold time. Sample contained greater than 10% headspace at time of extraction. Analyzed past method specified hold time (2) Sample extracted past method-specified hold time. Analyzed past method specified hold time										

BUREAU  
VERITAS

Bureau Veritas Job #: C2O5551

Report Date: 2022/09/06

Golder Associates Ltd  
 Client Project #: 19136074  
 Site Location: BRADFORD  
 Your P.O. #: 19136074  
 Sampler Initials: MTI

**SOIL CORROSIVITY PACKAGE (SOIL)**

<b>Bureau Veritas ID</b>		TOL233			TOL234	TOL235	TOL236	TOL237		
<b>Sampling Date</b>		2021/06/17			2021/06/10	2021/12/21	2021/07/07	2022/01/13		
<b>COC Number</b>		n/a			n/a	n/a	n/a	n/a		
	<b>UNITS</b>	<b>404-3 SA#3 Lab-Dup</b>	<b>RDL</b>	<b>QC Batch</b>	<b>404-4 SA#4</b>	<b>2-1 SA#4</b>	<b>L-4B SA#4</b>	<b>HRE-3 SA#3B</b>	<b>RDL</b>	<b>QC Batch</b>

<b>Calculated Parameters</b>										
Resistivity	ohm-cm				700	2800	990	9000		8193773
<b>Inorganics</b>										
Soluble (20:1) Chloride (Cl-)	ug/g	570	20	8201208	780	65	540	<20	20	8201208
Conductivity	umho/cm				1430	359	1010	111	2	8198559
Available (CaCl2) pH	pH				7.88	7.82	8.04	7.72		8199338
Soluble (20:1) Sulphate (SO4)	ug/g				100	210	53	<20	20	8201217
Sulphide	mg/kg				4.9 (1)	89.9 (1)	4.8 (1)	5.1 (1)	0.5	8205122
<b>Physical Testing</b>										
Moisture-Subcontracted	%				18	11	11	21	0.30	8205121
RDL = Reportable Detection Limit QC Batch = Quality Control Batch Lab-Dup = Laboratory Initiated Duplicate (1) Sample extracted past method-specified hold time. Analyzed past method specified hold time										

<b>Bureau Veritas ID</b>		TOL238			TOL238			TOL239		
<b>Sampling Date</b>		2022/02/22			2022/02/22			2021/11/11		
<b>COC Number</b>		n/a			n/a			n/a		
	<b>UNITS</b>	<b>HRE-4 SA#4</b>	<b>RDL</b>	<b>QC Batch</b>	<b>HRE-4 SA#4 Lab-Dup</b>	<b>RDL</b>	<b>QC Batch</b>	<b>HRW-1B SA#5</b>	<b>RDL</b>	<b>QC Batch</b>
<b>Calculated Parameters</b>										
Resistivity	ohm-cm	7200		8193773				6000		8193773
<b>Inorganics</b>										
Soluble (20:1) Chloride (Cl-)	ug/g	<20	20	8201208				59	20	8201208
Conductivity	umho/cm	138	2	8198559				167	2	8198559
Available (CaCl2) pH	pH	7.86		8199338				7.94		8199338
Soluble (20:1) Sulphate (SO4)	ug/g	27	20	8201217	24	20	8201217	<20	20	8201217
Sulphide	mg/kg	5.6 (1)	0.5	8205122				1.7 (1)	0.5	8205122
<b>Physical Testing</b>										
Moisture-Subcontracted	%	18	0.30	8205123				14	0.30	8205121
RDL = Reportable Detection Limit QC Batch = Quality Control Batch Lab-Dup = Laboratory Initiated Duplicate (1) Sample extracted past method-specified hold time. Analyzed past method specified hold time										



### SOIL CORROSIVITY PACKAGE (SOIL)

<b>Bureau Veritas ID</b>		TOL239			TOL240		
<b>Sampling Date</b>		2021/11/11			2022/05/20		
<b>COC Number</b>		n/a			n/a		
	<b>UNITS</b>	<b>HRW-1B SA#5 Lab-Dup</b>	<b>RDL</b>	<b>QC Batch</b>	<b>FD-02 SA#3</b>	<b>RDL</b>	<b>QC Batch</b>
<b>Calculated Parameters</b>							
Resistivity	ohm-cm				5800		8193773
<b>Inorganics</b>							
Soluble (20:1) Chloride (Cl <sup>-</sup> )	ug/g				<20	20	8201208
Conductivity	umho/cm				173	2	8198559
Available (CaCl <sub>2</sub> ) pH	pH				7.71		8199338
Soluble (20:1) Sulphate (SO <sub>4</sub> )	ug/g				<20	20	8201217
Sulphide	mg/kg				2.0 (1)	0.5	8205122
<b>Physical Testing</b>							
Moisture-Subcontracted	%	14	0.30	8205121	10	0.30	8205123
RDL = Reportable Detection Limit QC Batch = Quality Control Batch Lab-Dup = Laboratory Initiated Duplicate (1) Sample extracted past method-specified hold time. Analyzed past method specified hold time							





BUREAU  
VERITAS

Bureau Veritas Job #: C205551  
Report Date: 2022/09/06

Golder Associates Ltd  
Client Project #: 19136074  
Site Location: BRADFORD  
Your P.O. #: 19136074  
Sampler Initials: MTI

## TEST SUMMARY

**Bureau Veritas ID:** TOL232  
**Sample ID:** 404-2 SA#4  
**Matrix:** Soil

**Collected:** 2021/06/15  
**Shipped:**  
**Received:** 2022/08/26

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	8201208	2022/09/01	2022/09/02	Chandra Nandlal
Conductivity	AT	8198559	2022/08/31	2022/09/01	Roya Fathitil
Moisture (Subcontracted)	BAL	8205121	N/A	2022/09/02	Eric Tse
Sulphide in Soil	SPEC	8205122	N/A	2022/09/02	Bailey Morrison
pH CaCl2 EXTRACT	AT	8199338	2022/08/31	2022/08/31	Taslina Aktar
Resistivity of Soil		8193773	2022/09/01	2022/09/01	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	8201217	2022/09/01	2022/09/02	Samuel Law

**Bureau Veritas ID:** TOL232 Dup  
**Sample ID:** 404-2 SA#4  
**Matrix:** Soil

**Collected:** 2021/06/15  
**Shipped:**  
**Received:** 2022/08/26

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Sulphide in Soil	SPEC	8205122	N/A	2022/09/02	Bailey Morrison

**Bureau Veritas ID:** TOL233  
**Sample ID:** 404-3 SA#3  
**Matrix:** Soil

**Collected:** 2021/06/17  
**Shipped:**  
**Received:** 2022/08/26

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	8201208	2022/09/01	2022/09/02	Chandra Nandlal
Conductivity	AT	8198559	2022/08/31	2022/09/01	Roya Fathitil
Moisture (Subcontracted)	BAL	8205121	N/A	2022/09/02	Eric Tse
Sulphide in Soil	SPEC	8205122	N/A	2022/09/02	Bailey Morrison
pH CaCl2 EXTRACT	AT	8199338	2022/08/31	2022/08/31	Taslina Aktar
Resistivity of Soil		8193773	2022/09/01	2022/09/01	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	8201217	2022/09/01	2022/09/02	Samuel Law

**Bureau Veritas ID:** TOL233 Dup  
**Sample ID:** 404-3 SA#3  
**Matrix:** Soil

**Collected:** 2021/06/17  
**Shipped:**  
**Received:** 2022/08/26

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	8201208	2022/09/01	2022/09/02	Chandra Nandlal

**Bureau Veritas ID:** TOL234  
**Sample ID:** 404-4 SA#4  
**Matrix:** Soil

**Collected:** 2021/06/10  
**Shipped:**  
**Received:** 2022/08/26

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	8201208	2022/09/01	2022/09/02	Chandra Nandlal
Conductivity	AT	8198559	2022/08/31	2022/09/01	Roya Fathitil
Moisture (Subcontracted)	BAL	8205121	N/A	2022/09/02	Eric Tse
Sulphide in Soil	SPEC	8205122	N/A	2022/09/02	Bailey Morrison



**BUREAU  
VERITAS**

Bureau Veritas Job #: C2O5551

Report Date: 2022/09/06

Golder Associates Ltd

Client Project #: 19136074

Site Location: BRADFORD

Your P.O. #: 19136074

Sampler Initials: MTI

## TEST SUMMARY

**Bureau Veritas ID:** TOL234  
**Sample ID:** 404-4 SA#4  
**Matrix:** Soil

**Collected:** 2021/06/10  
**Shipped:**  
**Received:** 2022/08/26

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
pH CaCl2 EXTRACT	AT	8199338	2022/08/31	2022/08/31	Taslina Aktar
Resistivity of Soil		8193773	2022/09/01	2022/09/01	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	8201217	2022/09/01	2022/09/02	Samuel Law

**Bureau Veritas ID:** TOL235  
**Sample ID:** 2-1 SA#4  
**Matrix:** Soil

**Collected:** 2021/12/21  
**Shipped:**  
**Received:** 2022/08/26

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	8201208	2022/09/01	2022/09/02	Chandra Nandlal
Conductivity	AT	8198559	2022/08/31	2022/09/01	Roya Fathitil
Moisture (Subcontracted)	BAL	8205121	N/A	2022/09/02	Eric Tse
Sulphide in Soil	SPEC	8205122	N/A	2022/09/02	Bailey Morrison
pH CaCl2 EXTRACT	AT	8199338	2022/08/31	2022/08/31	Taslina Aktar
Resistivity of Soil		8193773	2022/09/01	2022/09/01	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	8201217	2022/09/01	2022/09/02	Samuel Law

**Bureau Veritas ID:** TOL236  
**Sample ID:** L-4B SA#4  
**Matrix:** Soil

**Collected:** 2021/07/07  
**Shipped:**  
**Received:** 2022/08/26

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	8201208	2022/09/01	2022/09/02	Chandra Nandlal
Conductivity	AT	8198559	2022/08/31	2022/09/01	Roya Fathitil
Moisture (Subcontracted)	BAL	8205121	N/A	2022/09/02	Eric Tse
Sulphide in Soil	SPEC	8205122	N/A	2022/09/02	Bailey Morrison
pH CaCl2 EXTRACT	AT	8199338	2022/08/31	2022/08/31	Taslina Aktar
Resistivity of Soil		8193773	2022/09/01	2022/09/01	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	8201217	2022/09/01	2022/09/02	Samuel Law

**Bureau Veritas ID:** TOL237  
**Sample ID:** HRE-3 SA#3B  
**Matrix:** Soil

**Collected:** 2022/01/13  
**Shipped:**  
**Received:** 2022/08/26

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	8201208	2022/09/01	2022/09/02	Chandra Nandlal
Conductivity	AT	8198559	2022/08/31	2022/09/01	Roya Fathitil
Moisture (Subcontracted)	BAL	8205121	N/A	2022/09/02	Eric Tse
Sulphide in Soil	SPEC	8205122	N/A	2022/09/02	Bailey Morrison
pH CaCl2 EXTRACT	AT	8199338	2022/08/31	2022/08/31	Taslina Aktar
Resistivity of Soil		8193773	2022/09/01	2022/09/01	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	8201217	2022/09/01	2022/09/02	Samuel Law



BUREAU  
VERITAS

Bureau Veritas Job #: C205551

Report Date: 2022/09/06

Golder Associates Ltd

Client Project #: 19136074

Site Location: BRADFORD

Your P.O. #: 19136074

Sampler Initials: MTI

## TEST SUMMARY

**Bureau Veritas ID:** TOL238  
**Sample ID:** HRE-4 SA#4  
**Matrix:** Soil

**Collected:** 2022/02/22  
**Shipped:**  
**Received:** 2022/08/26

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	8201208	2022/09/01	2022/09/02	Chandra Nandlal
Conductivity	AT	8198559	2022/08/31	2022/09/01	Roya Fathitil
Moisture (Subcontracted)	BAL	8205123	N/A	2022/09/02	Simranjeet Batth
Sulphide in Soil	SPEC	8205122	N/A	2022/09/02	Bailey Morrison
pH CaCl2 EXTRACT	AT	8199338	2022/08/31	2022/08/31	Taslina Aktar
Resistivity of Soil		8193773	2022/09/01	2022/09/01	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	8201217	2022/09/01	2022/09/02	Samuel Law

**Bureau Veritas ID:** TOL238 Dup  
**Sample ID:** HRE-4 SA#4  
**Matrix:** Soil

**Collected:** 2022/02/22  
**Shipped:**  
**Received:** 2022/08/26

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Sulphate (20:1 Extract)	KONE/EC	8201217	2022/09/01	2022/09/02	Samuel Law

**Bureau Veritas ID:** TOL239  
**Sample ID:** HRW-1B SA#5  
**Matrix:** Soil

**Collected:** 2021/11/11  
**Shipped:**  
**Received:** 2022/08/26

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	8201208	2022/09/01	2022/09/02	Chandra Nandlal
Conductivity	AT	8198559	2022/08/31	2022/09/01	Roya Fathitil
Moisture (Subcontracted)	BAL	8205121	N/A	2022/09/02	Eric Tse
Sulphide in Soil	SPEC	8205122	N/A	2022/09/02	Bailey Morrison
pH CaCl2 EXTRACT	AT	8199338	2022/08/31	2022/08/31	Taslina Aktar
Resistivity of Soil		8193773	2022/09/01	2022/09/01	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	8201217	2022/09/01	2022/09/02	Samuel Law

**Bureau Veritas ID:** TOL239 Dup  
**Sample ID:** HRW-1B SA#5  
**Matrix:** Soil

**Collected:** 2021/11/11  
**Shipped:**  
**Received:** 2022/08/26

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Moisture (Subcontracted)	BAL	8205121	N/A	2022/09/02	Eric Tse

**Bureau Veritas ID:** TOL240  
**Sample ID:** FD-02 SA#3  
**Matrix:** Soil

**Collected:** 2022/05/20  
**Shipped:**  
**Received:** 2022/08/26

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	8201208	2022/09/01	2022/09/02	Chandra Nandlal
Conductivity	AT	8198559	2022/08/31	2022/09/01	Roya Fathitil
Moisture (Subcontracted)	BAL	8205123	N/A	2022/09/02	Simranjeet Batth
Sulphide in Soil	SPEC	8205122	N/A	2022/09/02	Bailey Morrison



BUREAU  
VERITAS

Bureau Veritas Job #: C2O5551  
Report Date: 2022/09/06

Golder Associates Ltd  
Client Project #: 19136074  
Site Location: BRADFORD  
Your P.O. #: 19136074  
Sampler Initials: MTI

## TEST SUMMARY

**Bureau Veritas ID:** TOL240  
**Sample ID:** FD-02 SA#3  
**Matrix:** Soil

**Collected:** 2022/05/20  
**Shipped:**  
**Received:** 2022/08/26

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
pH CaCl2 EXTRACT	AT	8199338	2022/08/31	2022/08/31	Taslima Aktar
Resistivity of Soil		8193773	2022/09/01	2022/09/01	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	8201217	2022/09/01	2022/09/02	Samuel Law



BUREAU  
VERITAS

Bureau Veritas Job #: C2O5551

Report Date: 2022/09/06

Golder Associates Ltd

Client Project #: 19136074

Site Location: BRADFORD

Your P.O. #: 19136074

Sampler Initials: MTI

### GENERAL COMMENTS

Each temperature is the average of up to three cooler temperatures taken at receipt

Package 1	4.7°C
-----------	-------

**Results relate only to the items tested.**



**BUREAU**  
**VERITAS**

Bureau Veritas Job #: C205551

Report Date: 2022/09/06

## QUALITY ASSURANCE REPORT

Golder Associates Ltd

Client Project #: 19136074

Site Location: BRADFORD

Your P.O. #: 19136074

Sampler Initials: MTI

QC Batch	Parameter	Date	Matrix Spike		SPIKED BLANK		Method Blank		RPD	
			% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
8198559	Conductivity	2022/09/01			99	90 - 110	<2	umho/cm	0.34	10
8199338	Available (CaCl <sub>2</sub> ) pH	2022/08/31			100	97 - 103			1.2	N/A
8201208	Soluble (20:1) Chloride (Cl <sup>-</sup> )	2022/09/02	NC	70 - 130	103	70 - 130	<20	ug/g	0.69	35
8201217	Soluble (20:1) Sulphate (SO <sub>4</sub> )	2022/09/02	NC	70 - 130	103	70 - 130	<20	ug/g	12	35
8205121	Moisture-Subcontracted	2022/09/02					<0.30	%	0.73	20
8205122	Sulphide	2022/09/02	120	75 - 125	115	75 - 125	<0.5	mg/kg	21	30
8205123	Moisture-Subcontracted	2022/09/02					<0.30	%		

N/A = Not Applicable

Duplicate: Paired analysis of a separate portion of the same sample. Used to evaluate the variance in the measurement.

Matrix Spike: A sample to which a known amount of the analyte of interest has been added. Used to evaluate sample matrix interference.

Spiked Blank: A blank matrix sample to which a known amount of the analyte, usually from a second source, has been added. Used to evaluate method accuracy.

Method Blank: A blank matrix containing all reagents used in the analytical procedure. Used to identify laboratory contamination.

NC (Matrix Spike): The recovery in the matrix spike was not calculated. The relative difference between the concentration in the parent sample and the spike amount was too small to permit a reliable recovery calculation (matrix spike concentration was less than the native sample concentration)



BUREAU  
VERITAS

Bureau Veritas Job #: C2O5551  
Report Date: 2022/09/06

Golder Associates Ltd  
Client Project #: 19136074  
Site Location: BRADFORD  
Your P.O. #: 19136074  
Sampler Initials: MTI

### VALIDATION SIGNATURE PAGE

The analytical data and all QC contained in this report were reviewed and validated by:

Anastassia Hamanov, Scientific Specialist

Ghayasuddin Khan, M.Sc., P.Chem., QP, Scientific Specialist, Inorganics

Gita Pokhrel, Senior Analyst

---

Bureau Veritas has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per ISO/IEC 17025, signing the reports. For Service Group specific validation please refer to the Validation Signature Page.





26-Aug-22 15:58

Ankita Bhalla



C205551

L8

Free: 800-563-6266

## WORK ORDER

## CHAIN OF CUSTODY RECORD

Page \_\_\_ of 1

Report Information (if differs from invoice)		Project Information (where applicable)		Turnaround Time (TAT) Required	
Company Name: <b>Golder Associates Ltd.</b>	Company Name: <b>Golder Associates Ltd.</b>	Quotation #: <b>Golder rates</b>	<input checked="" type="checkbox"/> Regular TAT (5-7 days) Most analyses		
Contact Name: <b>Kevin Bentley</b>	Contact Name: <b>Muhammad Talha Irshad</b>	P.O. #/ AFE#: <b>19136074</b>	PLEASE PROVIDE ADVANCE NOTICE FOR RUSH PROJECTS		
Address: <b>6925 Century Ave., Suite 100</b>	Address: <b>6925 Century Ave., Suite 100</b>	Project #: <b>19136074</b>	Rush TAT (Surcharges will be applied)		
<b>Mississauga, ON</b>	<b>Mississauga, ON</b>	Site Location: <b>Bradford</b>	<input type="checkbox"/> 1 Day <input type="checkbox"/> 2 Days <input type="checkbox"/> 3-4 Days		
Phone: <b>905 567-6100</b> Fax: _____	Phone: <b>778 228-5756</b> Fax: _____	Site #: _____	Date Required: _____		
Email: <b>canadaaccounts@bureauveritas.com; Kevin_Bentley@golder.com</b>	Email: <b>mirshad@golder.com; 120387@golder.com</b>	Site Location Province: <b>Ontario</b>	Rush Confirmation #: _____		
MOE REGULATED DRINKING WATER OR WATER INTENDED FOR HUMAN CONSUMPTION MUST BE SUBMITTED ON THE BUREAU VERITAS LABORATORIES' DRINKING WATER CHAIN OF CUSTODY		Sampled By: _____	LABORATORY USE ONLY		
<b>Regulation 153</b> <input type="checkbox"/> Table 1 <input type="checkbox"/> Res/Park <input type="checkbox"/> Med/ Fine <input type="checkbox"/> Table 2 <input type="checkbox"/> Ind/Comm <input type="checkbox"/> Coarse <input type="checkbox"/> Table 3 <input type="checkbox"/> Agri/ Other <input type="checkbox"/> Table _____ FOR RSC (PLEASE CIRCLE) Y / N		<b>Other Regulations</b> <input type="checkbox"/> CCME <input type="checkbox"/> Sanitary Sewer Bylaw <input type="checkbox"/> MISA <input type="checkbox"/> Storm Sewer Bylaw <input type="checkbox"/> PWQO <input type="checkbox"/> Region <input type="checkbox"/> Other (Specify) _____ <input type="checkbox"/> REG 558 (MIN. 3 DAY TAT REQUIRED)		<b>Analysis Requested</b>	
Include Criteria on Certificate of Analysis: Y / N		<b>Analysis Requested</b>		CUSTODY SEAL Y / N	
SAMPLES MUST BE KEPT COOL (< 10 °C) FROM TIME OF SAMPLING UNTIL DELIVERY TO BUREAU VERITAS		<b>Analysis Requested</b>		COOLER TEMPERATURES	
SAMPLE IDENTIFICATION		DATE SAMPLED (YYYY/MM/DD)		TIME SAMPLED (HH:MM)	
MATRIX		# OF CONTAINERS SUBMITTED		FIELD FILTERED (CIRCLE) Metals / Hg / Cr VI	
FIELD FILTERED (CIRCLE) Metals / Hg / Cr VI		BTEX / PHC F1		PHC F2 - F4	
VOCS		REG 153 METALS & INORGANICS		REG 153 CPMS METALS	
REG 153 METALS (Hg, Cr VI, CPMS METALS, HWS - B)		CORROSIVITY PACKAGE (P-SULPHIDE)		HOLD-DO NOT ANALYZE	
1 404-2 SA#4		2021-06-15		AM	
2 404-3 SA#3		2021-06-17		AM	
3 404-4 SA#4		2021-06-10		AM	
4 2-1 SA#4		2021-12-21		AM	
5 L-4B SA#4		2021-07-07		AM	
6 HRE-3 SA#3B		2022-01-13		AM	
7 HRE-4 SA#4		2022-02-22		AM	
8 HRW-1B SA#5		2021-11-11		AM	
9 FD-02 SA#3		2022-05-20		AM	
10					
RELINQUISHED BY: (Signature/Print)		DATE: (YYYY/MM/DD)		TIME: (HH:MM)	
Muhammad Talha Irshad		2022-08-26		15:58	
RECEIVED BY: (Signature/Print)		DATE: (YYYY/MM/DD)		TIME: (HH:MM)	
Ankita Bhalla		2022/08/26		15:58	
BV JOB #					

Unless otherwise agreed to in writing, work submitted on this Chain of Custody is subject to Bureau Veritas Laboratories' standard Terms and Conditions. Signing of this Chain of Custody document is acknowledgment and acceptance of our terms available at <http://www.bvlabs.com/terms-and-conditions>





[golder.com](http://golder.com)