REPORT

Preliminary Foundation Investigation and Design Report

Holland River East Branch Structure
Highway 400 to Highway 404 Link (Bradford Bypass)
Simcoe County and York Region
Assignment No. 2019-E-0048

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HIGHWAY 400 TO HIGHWAY 404 LINK (BRADFORD BYPASS)
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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 East Branch as shown on the Key Plan of Drawing 1.

2.0 SITE DESCRIPTION

The proposed bridge will span across the Holland River East Branch, which is located within the Region of York, between Bathurst Street and Yonge Street. The site is generally flat and rises slightly to the east towards Yonge Street located about 500 m east of the river. There is a private marina located on the west side of Holland River East Branch and just north of the proposed new highway alignment and west approach embankment footprint. The marina contains numerous boat storage buildings on land as well as a dredged harbour (up to about 3 m deep) with boat docks that runs parallel (and about 100 m away) from the new highway alignment. There is a cleared area south of the marina which transitions into a heavily treed area within the footprint of the proposed bridge and approach embankment (see Photograph 1). The east side of Holland River East Branch consists of predominantly flat residential and forested areas, with a public golf course located on the east riverbank north of the proposed bridge structure location. 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. The Holland River East Branch is considered a navigable waterway and is up to about 2 m deep in this area. The river flows from south to north and joins the Holland River about 3 km north before the Holland River outlets into Lake Simcoe.

Photograph 1 - West side of Holland River East Branch looking northwest at HRE-1 drilling location with marina in background

3.0 INVESTIGATION PROCEDURES

The field work for the current investigation was carried out between September 29, 2021 and March 1, 2022, during which time four boreholes (designated HRE-1 to HRE-4) and one Seismic Cone Penetration Test (designated SCPT21-HSE-1) were advanced at the locations shown on Drawing 1. Boreholes HRE-1, HRE-2 and SCPT21-HSE-1 were advanced on the west side of the Holland River East Branch and Boreholes HRE-3 and HRE-4 were advanced on the east side of the Holland River East Branch.

All boreholes were advanced using 210 mm outside diameter (O.D.) hollow-stem augers generally set a depth of approximately 3.0 m followed by wash-rotary techniques (advancement of tricone with water/drilling mud) using a Diedrich D50 track-mounted drill supplied and operated by the drilling subcontractor, 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)[1], and using 76 mm O.D. thin-walled 'Shelby' Tube samplers (ASTM D1587)[2] 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 in general accordance with ASTM D2573[3].

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

The groundwater conditions in the open boreholes were generally observed during and immediately following the drilling operations, although any recorded water levels in the boreholes are likely not representative of actual groundwater levels due to the introduction of water/slurry as part of the mud rotary drilling technique. A standpipe piezometer was installed in Borehole HRE-3 to allow monitoring of the groundwater level(s). A standpipe piezometer was not installed on the west side of the river at the request of the property owner. The installed piezometer consists of a 50 mm diameter PVC pipe, with a 3.0 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 in general accordance with Regulation 903 (as amended). The monitoring well was capped with an above grade (steel monument) casing. Flowing artesian conditions were temporarily encountered during drilling in Borehole HRE-4; however the groundwater level stabilized to 3.3 m below ground surface subsequent to borehole completion and removal of augers/rods. Borehole HRE-4 was backfilled and sealed with bentonite in general accordance with Regulation 903 (as amended).

The field work was monitored on a full-time basis by a member of Golder's engineering staff who located the boreholes and sCPT 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

[1] ASTM D1586 – Standard Test Method for Standard Penetration Tests and Split Barrel Sampling of Soils.

[2] ASTM D1587 - Standard Practice for Thin-Walled Tube Sampling of Fine-Grained Soils for Geotechnical Purposes.

[3] ASTM D2573 - Standard Test Method for Field Vane Shear Test in Saturated Fine-Grained Soils.

transported to Golder's laboratory in Mississauga for further visual review and geotechnical laboratory testing. Index and classification testing consisting of natural moisture 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.

Two soil samples, one from each side of the Holland River East Branch, 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 soluble 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 generally achieved a horizontal accuracy of less than 50 cm and vertical accuracy of less than 10 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 / sCPT Depth (m)
	Northing (m) (Latitude)	Easting (m) (Longitude)		
HRE-1	4,888,459.3 (44.136129)	303,648.6 (-79.514389)	219.5	50.9
HRE-2	4,888,413.9 (44.135719)	303,260.8 (-79.519236)	218.9	31.1
HRE-3	4,888,657.1 (44.137919)	304,531.7 (-79.503353)	220.0	52.2
HRE-4	4,888,345.7 (44.135107)	304,339.4 (-79.505756)	219.9	52.4
SCPT21-HSE-01	4,888,472.8 (44.136250)	303,652.6 (-79.514340)	219.5	50.0

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)[4].

The Simcoe Lowlands physiographic region covers the central portion of the County of Simcoe and northern portion of York Region. Following the retreat of the last glacial ice sheet, the lowland was flooded by the now

[4] 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.

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 are also provided in Appendix A. The detailed 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 in the boreholes advanced near the Holland River East Branch consist of surficial layers of fill and organics underlain by a loose to very dense silty sand to sandy silt deposit. On the west side of the river, an upper cohesive deposit consisting of stiff to very stiff clayey silt to clayey silt-silt with frequent silt/sand seams was encountered between the fill/organics and the silty sand to sandy silt deposit. The silty sand to sandy silt deposit is underlain by a lower cohesive deposit consisting of stiff to hard clayey silt to clayey silt-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 Borehole HRE-3 at ground surface.

4.2.2 Cohesive Fill

An approximately 1.2 m to 2.2 m thick layer of surficial clayey silt-silt to sandy clayey silt-silt (fill) was encountered west of Holland River East Branch in Boreholes HRE-1 and HRE-2. The fill typically contained trace organics / rootlets. The base of the fill layer extended to Elevations 218.3 m and 216.7 m.

The SPT 'N'-values measured within the cohesive fill range from 2 to 13 blows per 0.3 m of penetration, suggesting a soft to stiff consistency.

Grain size distribution testing was carried out on a sample of the cohesive fill and the results are shown on Figure B1 in Appendix B.

Atterberg limits testing was carried out on two selected samples of the cohesive fill layer and measured liquid limits of 19% and 20%, plastic limits of 16% and 17%, and plasticity indices of 2% and 4%. These results, which are plotted on a plasticity chart on Figure B2, indicate that the deposit consists of clayey silt to clayey silt-silt of low plasticity

The natural water content measured on selected samples of the cohesive fill was about 21% and 24%.

4.2.3 Non-Cohesive Fill

An approximately 0.7 m to 1.6 m thick layer of silty sand fill was encountered east of Holland River East Branch in Boreholes HRE-3 and HRE-4. The fill was encountered below the topsoil in HRE-3 and at ground surface in HRE-4 and typically contained trace organics. The base of the fill layer extended to Elevations 218.2 m and 219.2 m in HRE-3 and HRE-4 respectively.

The SPT ‘N’-values measured within the non-cohesive fill range from 5 to 10 blows per 0.3 m of penetration, indicating a loose to compact level of compactness.

Grain size distribution testing was carried out on a sample of the non-cohesive fill layer and the results are shown on Figure B3 in Appendix B.

The natural water content measured on a selected sample of the cohesive fill was about 20%.

4.2.4 Sandy Peat

A deposit of sandy peat was encountered below the fill layer in Borehole HRE-1. The deposit was about 0.4 m thick and extended to a depth of 1.6 m (Elevation 217.9 m). A field vane test carried out within the organic deposit measured an undrained shear strength of 85 kPa and remoulded strength of about 55 kPa. Based on the field vane data, the deposit is considered to generally have a stiff consistency.

4.2.5 Silty Sand to Sandy Silt

A deposit of silty sand to sandy silt was encountered underlying the fills or organic deposit in all boreholes. The cohesionless deposit was generally 10.8 m to 28.6 m thick and extended to depths ranging from of 13.0 m to 29.3 m (Elevations 206.0 m to 190.6 m). The deposit contained cohesive interlayers comprising of clayey silt to clayey silt-silt which ranged from 0.3 m to 1.5 m in thickness. On the west side of the river (i.e. Boreholes HRE-1 and HRE-2) the deposit was observed to be interlayered with a thicker cohesive layer (i.e. the upper cohesive deposit described in the following section).

The SPT ‘N’-values measured in the silty sand to sandy silt range from 5 to 95 blows per 0.3 m of penetration, indicating a loose to very dense state of compactness. The SPT ‘N’-values measured in the clayey silt to clayey silt-silt interlayers range from 4 to 34 blows per 0.3 m of penetration, suggesting a firm to hard consistency.

Grain size distribution testing was carried out on eighteen samples of the silty sand to sandy silt deposit and the results are shown on Figure B4A and B4B in Appendix B.

Atterberg limits testing was carried out on twelve selected samples of the silty sand to sandy silt deposit. Four tests showed that the deposit is non-plastic and the remaining tests measured liquid limits ranging between 13% and 18%, plastic limits ranging between 12% and 16%, and plasticity indices ranging between 1% and 3%. These results, which are plotted on a plasticity chart on Figure B5, indicate that the deposit contains regions of silt with slight plasticity.

The natural water content measured on selected samples of the silty sand to sandy silt deposit ranges between about 15% and 23%.

Grain size distribution testing was carried out on two samples of the cohesive interlayers and the results are shown on Figure B6 in Appendix B.

Atterberg limits testing was carried out on three selected samples of the cohesive interlayers and measured liquid limits ranging between 18% and 31%, plastic limits ranging between 14% and 19%, and plasticity indices ranging between 4% and 12%. These results, which are plotted on a plasticity chart on Figure B7, indicate that the interlayers consist of clayey silt to clayey silt-silt of low plasticity.

The natural water content measured on selected samples of the cohesive interlayers ranges between about 19% and 28%.

4.2.6 Clayey Silt to Clayey Silt-Silt (Upper Cohesive Deposit)

An upper cohesive deposit of clayey silt to clayey silt-silt was encountered within the sandy silt to silty sand deposit in both boreholes west of Holland River East Branch (i.e. Boreholes HRE-1 and HRE-2). The thickness of the deposit was approximately 10.9 m and 1.9 m and extended to depths of 13.9 m and 4.9 m (Elevations 205.6 m and 214.0 m) in Boreholes HRE-1 and HRE-2. The cohesive deposit is characterized as having sand and silt seams/layers throughout.

The SPT 'N'-values measured within the upper cohesive deposit typically range from 4 to 26 blows per 0.3 m of penetration. Three in-situ field vane tests carried out within the upper cohesive deposit in Borehole HRE-1 measured undrained shear strengths of about 75 kPa, 90 kPa and >96 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 two samples of the upper cohesive deposit and the results are shown on Figure B8.

Atterberg limits testing was carried out on two selected samples of the upper cohesive deposit and measured liquid limits of 21% and 22%, plastic limits of 14% and 17%, and plasticity indices of 5% and 7%. These results, which are plotted on a plasticity chart on Figure B9, 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 14% and 27%.

Two Shelby tubes were collected in the upper cohesive deposit (including a Shelby tube obtained in a borehole advanced adjacent to HRE-1 following completion of drilling as indicated on the Record of Borehole). Laboratory consolidation (oedometer) testing was carried out on two vertically trimmed specimens of the clayey silt to clayey silt-silt to assess the compressibility characteristics of the deposit. The details of the test results are shown on Figures B10 to B17 and are summarized below.

Borehole / Sample No.	Sample Depth / Elevation (m)	σ_{vo}' (kPa)	σ_p' (kPa)	σ_p' - σ_{vo}' (kPa)	OCR (avg.)	C_c	C_r	e_o	c_v (cm^2/s)
HRE-1 / TO-4	4.6 / 214.9	65	235 to 338	170 to 270	3 to 5	0.127	0.007- 0.012	0.64	0.0175 to 0.0340
HRE-2 / 6	3.8 / 215.1	60	421 to 530	360 to 475	7 to 9	0.130	0.002 – 0.007	0.62	0.0092 to 0.0107

Where:
σ_p' = Estimated preconsolidation stress (using Casagrande construction and Work interpretation methods)
c_v = Coefficient of consolidation (vertical) for approximate overconsolidated stress range 50 kPa ≤ σ_v' ≤ 300 kPa
C_c = Compression index
C_r = Recompression index
e_o = Initial void ratio
OCR = Overconsolidation ratio
σ_{vo}' = Calculated existing vertical effective stress

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 all boreholes. The deposit was encountered at depths ranging from of 13.0 m to 29.3 m (Elevations 206.8 m to 190.6 m) and all boreholes were terminated within the deposit at depths of 31.1 m to 52.4 m (Elevations 187.8 m to 167.5 m). The lower cohesive deposit contained frequent sand and silt seams in all boreholes with thicker interlayers of sandy silt to silt encountered on the west side of Holland River East Branch (i.e. Boreholes HRE-1 and HRE-2). An interlayer of clay was encountered within the lower cohesive deposit in Borehole HRE-4 on the east side of Holland River East Branch. The sandy silt to silt interlayers were approximately 1.6 m and 3.8 m thick and extended to depths of 24.5 m and 20.1 m (Elevations 194.9 m and 198.8 m) in Boreholes HRE-1 and HRE-2. The clay interlayer was approximately 3.0 m in thickness and extended to a depth of 41.4 m (Elevation 178.5 m).

The SPT ‘N’-values measured in the lower cohesive deposit generally range from 9 to 42 blows per 0.3 m of penetration. Two lower SPT ‘N’-values of 3 and 6 were measured in HRE-2 and HRE-3, and one distinct SPT ‘N’-value of 166 blows per 0.3 m of penetration was measured in the final split-spoon upon termination of Borehole HRE-3. Three in-situ field vane tests carried out within the lower cohesive deposit in both boreholes measured undrained shear strengths ranging from about 65 kPa to >96 kPa. The undrained shear strength value of 65 kPa was measured directly below the sample where the SPT ‘N’-value of 3 was measured. Based on the SPT ‘N’-values and field vane tests, the deposit is considered to generally have a stiff to hard consistency.

The SPT ‘N’-values measured in the sandy silt to silt interlayers range from 13 to 45 blows per 0.3 m of penetration, indicating a compact to dense state of compactness. The SPT ‘N’-value measured in the clay interlayer was 17 blows per 0.3 m of penetration suggesting a very stiff consistency.

Grain size distribution tests were carried out on six samples of the lower cohesive deposit and the results are shown on Figure B18.

Atterberg limits testing was carried out on nine selected samples of the lower cohesive deposit and measured liquid limits ranging between 19% and 29%, plastic limits ranging between 14% and 18%, and plasticity indices ranging between 4% and 13%. These results, which are plotted on a plasticity chart on Figure B19, 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 18% and 27%, with one distinct higher water content measurement of 42% in a sample of the clayey silt near a silt pocket in HRE-1.

A grain size distribution test was carried out on a sample of the sandy silt to silt interlayer and the results are shown on Figure B20. A grain size distribution test was carried out on a sample of the clay interlayer and the results are shown on Figure B21.

Atterberg limits testing was carried out on a selected sample of the sandy silt to silt interlayer in HRE-2 and the results showed that the cohesionless interlayer is non-plastic. Atterberg limits testing was carried out on a selected sample of the clay interlayer and measured a liquid limit of 52%, a plastic limit of 20%, and a plasticity index of 32%. These results, which are plotted on a plasticity chart on Figure B22, indicate that the interlayer consists of clay of high plasticity.

The water content measured on samples of the sandy silt to silt interlayers generally ranges between 13% and 15%. The water content measured on a sample of the clay interlayer was 40%.

Four Shelby tube samples were attempted in the lower cohesive deposit: two Shelby tube samples were collected at depths of 31.2 m and 31.8 m in Borehole HRE-1 and HRE-3, and two Shelby tube attempts resulted in limited sample recovery at depths of 31.2 m and 35.0 m in HRE-1 and HRE-3. The limited sample recovery is likely due to disturbance of attempting to collect samples in the stiff to hard clayey soils combined with the presence of frequent sand seams/layers. Laboratory consolidation (oedometer) testing was carried out on one vertically trimmed specimen of the lower clayey silt to clayey silt-silt from Borehole HRE-3 to assess the compressibility characteristics of the deposit. The details of the test results are shown on Figures B23 to B26 and are summarized below.

Borehole / Sample No.	Sample Depth / Elevation (m)	σ_{vo}' (kPa)	σ_p' (kPa)	σ_p' - σ_{vo}' (kPa)	OCR (avg.)	C_c	C_r	e_o	c_v (cm^2/s)
HRE-3 / 23	38.1 / 181.8	400	440 to 460	40 - 60	1.1	0.399	0.054 to 0.082	1.2	0.0006 – 0.0009

Where:
σ_p' = Estimated preconsolidation stress (using Casagrande construction and Work interpretation methods)
c_v = Coefficient of consolidation (vertical) for approximate overconsolidated stress range 50 kPa ≤ σ_v' ≤ 300 kPa
C_c = Compression index
C_r = Recompression index
e_o = Initial void ratio
OCR = Overconsolidation ratio
σ_{vo}' = Calculated existing vertical effective stress

4.3 Seismic Cone Penetration Test Results

The sCPT was advanced from ground surface to 50 m below ground surface 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 sCPT are presented in the Cone Penetration Test Report in Appendix A along with the results of sCPT's advanced at the Holland River, and the profile of the shear wave velocity at the cone penetration test located at the Holland River East Branch 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 frequently encountered from ground surface to a depth of about 35 mbgs. Below a depth of 35 mbgs, these higher permeability layers were not evident based on pore water pressure measurements. 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.

sCPT Designation	Soil Strata	C_u (kPa)	σ_p' (kPa)	$\sigma_p' - \sigma_{vo}'$ (kPa)	OCR (avg.)	c_h (cm²/s)
sCPT21-HSE-1	Upper cohesive deposit	55-400	250-1800	200-1600	5 to >10	0.02 to 0.32 (Note 1)
	Lower cohesive deposit	110-190	500-850	350-750	1.5 – 2.5	0.06

Where:
$\sigma_p' = \frac{q_t - \sigma_{vo}}{3.4}$ = range of estimated preconsolidation stress after "smooth fit" line interpretation of raw data (Demers and Leroueil, 2002)
q_t = corrected tip resistance
σ_{vo} = vertical stress
c_h = estimated horizontal coefficient of consolidation (Houlsby and Teh, 1991)
Note 1: Based on 3 test results. Another dissipation test was attempted but surrounding soil was too permeable and pore pressure dissipation could not be measured accurately.

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 generally not considered representative of the hydrostatic water levels at the site due to the addition of drilling fluids/water into the boreholes and/or considering the water levels did not have sufficient time to stabilize.

A standpipe piezometer was installed in Borehole HRE-3 to allow monitoring of the groundwater level at this site. The groundwater levels recorded during drilling and in the piezometer 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	Comments
HRE-1	219.5	2.5	217.0	Sept. 29, 2021	Open borehole
HRE-2	218.9	2.3	216.6	Dec. 3, 2021	Open borehole
HRE-3	220.0	1.5 1.0	218.5 219.0	Jan.13, 2021 May 13, 2022	Open borehole Piezometer
HRE-4	219.9	0.7 (-2.4)* 3.3	219.2 222.3 216.6	Feb. 22, 2022 Feb. 24-25 Mar. 1, 2022	Open borehole Open borehole Caved borehole

*Flowing artesian conditions encountered during drilling. Groundwater observed to be flowing out of the top of casing at a height of 2.4 m above ground surface when drilling below a depth of 12.2 m below ground surface.

In borehole HRE-4, flowing artesian groundwater conditions were first observed during drilling operations within the silty sand to sandy silt deposit at a depth of about 12.2 m (Elevation 207.7 m) and the artesian pressures were observed to increase with drilling depth within the inferred aquifer.

The groundwater level and hydrostatic head at depth 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 East Branch that is reported to have a high water level at El. 219.8 m.

4.5 Analytical Testing of Soil

Three 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)	Sulphide (mg/kg)
HRE-1, SA 1	7.60	4800	209	<20[1]	<20[1]	2.5
HRE-3, SA 3	7.72	9000	111	<20[1]	<20[1]	5.1
HRE-4, SA 4	7.86	7200	138	<20[1]	27	5.6

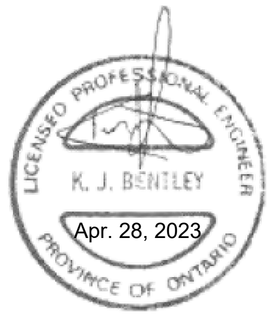
Note 1: Less than reportable detection limit

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.

Signature Page

Golder Associates Ltd.



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PART B

PRELIMINARY FOUNDATION DESIGN REPORT
HOLLAND RIVER EAST BRANCH 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 East Branch bridge crossing near Bradford, Ontario. The recommendations are based on interpretation of the factual data obtained from the boreholes and sCPT advanced on the east and west side of the Holland River East Branch during the current subsurface investigation.

The Preliminary 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 Preliminary 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 and preliminary design of the project and for which special provisions may be required in the future 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 Bradford Bypass mainline alignment and profile drawings provided by AECOM (preliminary draft dated September 2022), the proposed bridge will consist of twin structures to carry the Bradford Bypass eastbound and westbound lanes over the Holland River East Branch. The total span of each bridge structure is anticipated to be a minimum length of about 765 m to 790 m with conceptual 8-span to 10-span configurations (with spans up to 135 m across the Holland River East Branch) being considered. 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 configuration to accommodate four lanes in each direction (eight lanes total). The approach embankments are proposed to be about 8 m and 11 m high at the west and east approach embankments, respectively. Given the size of the structures and according to the structural designers, the twin Holland River East Branch structures are classified as “lifeline” bridges (according to the importance category designation in the CHBDC) at this preliminary stage.

The bridges will span over provincially significant wetlands and floodplain, and any piers (shafts/columns) are proposed to be placed outside of the current Holland River East Branch (i.e. no piers within the river), although temporary excavations and construction access within or near 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 East Branch 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, Ψ, and geotechnical resistance factors, ϕ_{gu} and ϕ_{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 CPT 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. assuming existing ground surface). The average $\bar{V}_s$ on the west side of the river was measured to be about 330 m/s and the average $\bar{N}_{60}$ on the east side of the river was measured to be about 30. Based on the results, the site may be classified as Site Class D in accordance with Table 4.1 of the CHBDC (2019). Additional site-specific study is recommended during detail design to evaluate the subsurface conditions closer to the foundation footprint and specifically for the piers located closer to the river to determine if more than one Site Class is applicable for different portions of the bridge, possibly with different foundation support systems.

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 D were obtained for the bridge site using the NBCC website (*earthquakescanada.nrcan.gc.ca*) and are summarized below.

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	0.098
$S_a(0.5)$ (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	0.025	0.040
$S_a(10.0)$ (g)	0.001	0.003	0.004

The values given above are for the reference ground condition Site Class D and must be modified (as appropriate) to the site-specific seismic site classification to be determined during detail design to obtain applicable design spectral values. 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).

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. 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. 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 boreholes advanced at the site were generally located greater 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, possible “perched” within approach embankments (after sufficient preloading), are considered to be an alternative from a geotechnical/foundations perspective based on the available subsurface information, however, the relatively low geotechnical resistance and challenges associated with subexcavating up to 4 m (with high groundwater level) in silts and sands combined with the anticipated high structural loads may eliminate this option. Given that structural loads are anticipated to be much higher than conventional highway bridge design and the fact that 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 majority of the west side during detail design.

Alternatively, driven steel H-piles (friction piles) 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. Caissons could also be considered but would need to be designed using predominantly skin friction given that there was no competent (100-blow) end-bearing soil deposit encountered within a 50 m depth during the investigation. The presence of artesian groundwater conditions on the east side of the river also makes caisson installation more challenging compared to driven piles.

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, although 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 to about 4 mbgs and below groundwater level is required to remove the existing fills, peat, and unsuitable very loose to loose / soft to firm soils. The presence and thickness of the peat 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:

Foundation Element	Founding Stratum	Footing Width	Founding[1] Elevation	Factored Geotechnical Resistance	
				Ultimate Limits State, f-ULS	Serviceability Limits State, f-SLS (for 25 mm of settlement)[2]
East Abutment and Surrounding Piers	Compact silty sand to silt	3 m	216 m	500 kPa	200 kPa
	3 m thick granular pad on compact silty sand to silt	3 m	219 m (min. 3 m of granular pad above El. 216 m)	750 kPa	300 kPa
	Compact silty sand	10 m	216 m	800 kPa	85 kPa
	10 m thick granular pad above compact silty sand to silt[3]	10 m	226 m (min. 10 m thick granular pad)	1200 kPa	100 kPa
West Abutment and surrounding Piers	stiff to very stiff clayey silt-silt	3 m	216 m	150 kPa	>150 kPa
	3 m thick granular pad on stiff to very stiff clayey silt-silt	3 m	219 m (min. 3 m thick granular pad above El. 216 m)	400 kPa	225 kPa
	Stiff to very stiff clayey silt-silt	10 m	216	150 kPa	60 kPa
	10 m thick granular pad above stiff to very stiff clayey silt-silt[3]	10 m	226 (min. 10 m thick granular pad)	800 kPa	75 kPa

Notes:

1. Subexcavation to about 4 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. 10 m thick Granular Pad = 4 m (subexcavation and replacement with Granular ‘A’) + 6 m (Granular ‘A’ pad above ground surface). This configuration is considered applicable for the abutment foundations only.

The factored ultimate and serviceability geotechnical resistances are dependent on the footing width, founding elevation and thickness of granular pad (as applicable) 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. In general, for larger footing sizes, higher factored ultimate and lower factored serviceability geotechnical resistances would apply. The preliminary factored geotechnical resistances should also be reevaluated using geotechnical resistance factors for a typical degree of understanding once further investigation data is available at the foundation elements.

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 clayey silt to clayey silt-silt deposit on the west side and silty sand to sandy silt deposit or lower clayey silt to clayey silt-silt deposit on the east 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 during detail design. 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 axial geotechnical resistances may be used for preliminary design:

Foundation Element	Pile Type	Approximate Pile Length[1]	Estimated Pile Tip Elevation	Factored Ultimate Geotechnical Resistance[2]	Factored Serviceability Geotechnical Resistance for 25 mm of Settlement[2]
East Abutment and Surrounding Piers	HP 310x110	30 m	190 m	800 kPa	>800 kPa
		40 m	180 m	1,000 kPa	>1,000 kPa
West Abutment and Surrounding Piers	HP 310x110	30 m	190 m	800 kPa	>800 kPa
		40 m	180 m	1000 kPa	>1000 kPa

Notes:
1. Measured from approximate existing ground surface at closest borehole location (approx. Elevation 220 m).
2. 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 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% using larger HP 360x132 piles or 460 mm outer diameter pipe 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 during detail design to confirm or adjust axial geotechnical resistances for design based on the use of a typical rather than low degree of understanding;
- 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 future 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 friction caissons (>30 m) are likely required 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 lower cohesive deposit.

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

Foundation Element	Caisson Diameter	Approximate Caisson Length[1]	Estimated Caisson Base Elevation [1]	Factored Ultimate Geotechnical Resistance[2]	Factored Serviceability Geotechnical Resistance for 25 mm of Settlement[2]
East Abutment and Surrounding Piers	0.9 m	30 m	186 m	2,400 kN	>2,400 kN
	1.5 m	30 m	186 m	3,900 kN	>3,900 kN
West Abutment and Surrounding Piers	0.9 m	30 m	186 m	2,400 kN	>2,400 kN
	1.5 m	30 m	186 m	3,900 kN	>3,900 kN

Notes:
1. Measured from approximate existing ground surface at closest borehole location (approx. Elevation 220 m).
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 within 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:

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

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 (Firm)	-	50
	Silty Sand (Compact to Loose)	15,000 – 7,500	-
	Sandy Silt (Compact to Very Dense)	15,000 – 25,000	-
	Silty Sand (Compact to Dense)	15,000 – 20,000	-
	Clayey Silt to Clayey Silt-Silt (Very Stiff)	-	125 - 190
West Abutment	New Granular Fill (Granular ‘A’ or ‘B’ Type II)	40,000 – 50,000	-
	Sandy Silt (Loose)	7,500	-
	Clayey Silt to Clayey Silt-Silt (Stiff to Hard)	-	55 - 400[3]
	Silty Sand to Sandy Silt (Very Dense)	20,000 – 25,000	-
	Clayey Silt to Clayey Silt-Silt (Very Stiff)	-	200 - 115[3]
	Sandy Silt to Silt (Compact)	15,000	-
	Clayey Silt to Clayey Silt-Silt (Very Stiff)	-	125 - 190

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 Table 3 for more details.

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 8 m to 11 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 East Branch. 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 11 m high on the east side and up to 8 m high on the west side of Holland River East Branch, 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 about 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 and sCPT nearest to the east and west approach embankments are summarized in Table 2 and Table 3. A summary plot of the engineering parameters and design line used for the cohesive deposits (upper cohesive deposit and lower cohesive deposit) is provided on Figure 1. For stability and settlement analysis, the groundwater is assumed to be at about the existing ground surface.

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 analyses, 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, Ψ, and the geotechnical resistance factor, ϕ_{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 8 m and 11 m of new fill over the native subgrade materials at the west and east approach embankments, respectively. A 2 m wide mid-height bench was modelled along the embankment slopes as required for embankment heights greater than 8 m in height as per OPSD 202.010 (*Slope Flattening*).

The global stability analyses indicate that for the short-term (undrained) condition, the approach embankments at the abutments will have a global Factor of Safety of greater than or equal to 1.6, and for the long-term (permanent) conditions, the approach embankments at the abutments will have a global Factor of Safety greater than or equal to 1.8. The results of the stability analyses 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
East Approach Embankment	HRE-3, HRE-4	Temporary (Undrained) Condition	>1.6
		Permanent (Drained) Condition	>1.8
West Approach Embankment	HRE-1, HRE-2 sCPT21-HSE-1	Temporary (Undrained) Condition	>1.6
		Permanent (Drained) Condition	>1.8

Slightly lower factors of safety (equal to 1.6) were calculated for shallow slip surfaces located within the lower portion of the embankment itself, however, when more detailed foundation investigation is completed at the site (typical or high level of understanding), the resistance factor can be increased and the target Factor of Safety for the temporary and permanent conditions can be decreased accordingly.

6.7.2 Settlement

Settlement analyses were carried out for the proposed maximum height of the east and west approach embankments 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 near the abutments.

The settlement analyses assume that topsoil, peat, 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 sCPT 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 (cm^2/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 sCPT 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 east and west 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 [1]	Estimated Settlement over a 20-Year Period[2] (mm)	
East Approach Embankment	HRE-3, HRE-4	11 m	$\delta_{Immediate}$ =	85 - 100
			$\delta_{Primary}$ =	30 - 40
			$\delta_{Secondary}$ =	<5
			δ_{Total} =	115 - 145
West Approach Embankment	HRE-1, HRE-2 sCPT21-HSE-1	8 m	$\delta_{Immediate}$ =	30 - 40
			$\delta_{Primary}$ =	70 - 85
			$\delta_{Secondary}$ =	<5
			δ_{Total} =	100 - 130

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 September 2022. Embankment heights are approximate and are relative to original ground surface.
2. The total settlement (δ_{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. Embankments were modelled independently to represent interim conditions and larger settlements should be anticipated for the ultimate configuration.

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, thickness and depth of the compressible d0eposits 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 Slag or Cellular Concrete**: Various lightweight fill materials are available, from lightweight slag with a unit weight of approximately 14 kN/m^3, to cellular concrete with a unit weight between 4 and 7 kN/m^3. However, for the volume of fill required for the new embankments, a similar preloading period to using conventional fill materials may still be required to achieve the settlement performance criterion. Floatation concerns within the floodplain will also need to be considered.

- **Lightweight 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. Given the relatively short preload time anticipated with using conventional fill (see next section), the impact on the construction schedule may not be significant and given the high cost of EPS compared to other lightweight and conventional granular fills, this option may not be practical. 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 x 10^{-2} cm^2/s for the majority of the compressible cohesive deposits, 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[1] (days)
East Approach Embankment	11	120 - 150
West Approach Embankment	8	120 - 150

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.

We understand that consideration is being given to constructing one bridge to accommodate the interim 4-lane configuration with construction of the second bridge in the future to accommodate the ultimate 8-lane highway configuration. It is recommended that the approach embankment geometry for the ultimate bridge(s) configuration be designed and constructed at the interim stage to induce the majority of all anticipated settlement at the approach embankments (interim and future). The advantages of constructing the ultimate configuration of the approach embankments as early as possible includes reduced future construction staging and more importantly reduce the impacts of settlement / differential settlement on the interim approach embankment configuration as a result of the future adjacent embankment loading.

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 depends on the soil resistivity / electrical conductivity, hydrogen ion concentration (pH), and salts (chloride and sulphate) concentrations. The analytical results for the soil 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 samples 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 soil analytical test results indicate a pH of 7.6 to 7.9 and a resistivity of 4,800 to 9,000 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 (4500 ohm-cm < R < 6000 ohm-cm) to Very Low (6000 ohm-cm < R < 10,000 ohm-cm) as per Table 3.2 (MTO, 2014), and limited corrosion protection is anticipated to be required for the foundation elements / materials.

Although the soluble chloride concentrations were low in the samples tested, given that the foundations are located adjacent to and below the future 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 anticipated to remove the unsuitable soils at the approach embankments, although this will need to be reassessed when boreholes are advanced closer the actual approach embankment footprints.

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 applied 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/organics, existing surficial fills and any loose/soft alluvial soils, temporary excavations extending up to about 2 m deep (for stripping below approach embankments) and up to 4 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/organics and any loosened/softened (alluvial) soils are classified as Type 4 soils, and the existing fill materials, loose to compact silt and sand deposits and the firm to 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. Given the interlayered nature and alluvial deposition of the native soils, very loose / soft seams and interlayers may be present which could cause sloughing and unstable conditions, thus, dewatering is likely required to achieve 1H:1V slopes in Type 3 soils located below the groundwater table.

Dewatering is likely not required for stripping / subexcavation below embankment footprints provided Granular A or Granular 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.

The shallow groundwater table was measured to be about 1 m to 2 m below the ground surface at the time of the investigation (about Elevation 219 m and 217 m at the east and west side of the river). It is noted that flowing artesian groundwater conditions were encountered on the east side of the river (Borehole HRE-4) with a hydrostatic head measured to be greater than 2.4 m above ground surface (higher than Elevation 222.3 m). The groundwater was observed to flow out of the drill casing when drilling below a depth of 12.2 m below ground surface and the artesian flow appeared to increase with depth within the silty sand to sandy deposit. Temporary excavations within this area or any area above confined aquifers with artesian pressures will need to be assessed during detail design such that base heave / instability at the base or sides of any excavations (or near any temporary or permanent foundation elements) is adequately controlled by balancing the hydrostatic pressures, providing adequate filtering to reduce migration of soil fines and the potential to undermine foundation elements, and/or providing adequate dewatering / depressurization of the aquifer.

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 East Branch 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 subgrade for granular fill pads or approach embankment construction 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 for construction of the foundation elements.

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 be required depending on the proximity to the river bank, river water level and required flood protection during construction. Conventional sheet pile or soldier pile and lagging temporary protection systems are considered feasible options.

6.9.4 Temporary Access Routes

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 and challenges accessing and supporting heavy equipment near the river 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 four boreholes and single cone penetration test advanced near (but typically more than 100 m away) from the proposed bridge and approach embankments. 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 (including potential artesian 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). It is anticipated that difficult access (tree clearing and temporary access roads) and/or specialized drilling equipment (e.g. cranes, barges, and/or portable equipment) will be required to advance additional foundation investigation near the anticipated pier locations, especially on the west side of the river. Boreholes and seismic CPTs should be advanced below the anticipated pile tip elevations (preferably beyond 50 m depth) in order to investigate foundation soils and potential end-bearing stratum to increase pile capacities, confirm or update geotechnical resistances for the deep foundation options and determine if any drainage layers are present in the lower portion of the clayey soil stratum to improve estimates for rates of settlement. In particular, a seismic CPT should be advanced on the east side of the river. The use of GSC 5th Generation or 6th 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 of settlement and any mitigation measures (including estimated preload durations) will need to be reassessed, especially if the ultimate configuration of the bridge approach embankments is to be constructed at the interim stage as this will increase the magnitude and time-rate of settlements. The design high water level of Holland River East Branch needs to be established for detail design. When more details are known on actual loading for the bridge, the foundation types and bearing resistance values will need to be reassessed and revised as necessary.

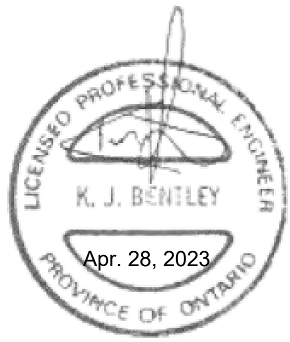
Additional foundation investigation and design should meet the general requirements outlined in the latest version of the *Guideline for MTO Foundation Engineering Services*. Additional piezometers / monitoring wells should be installed on both sides of the river, especially on the west side where no current piezometers exist. The artesian conditions encountered on the east side of the river will need to be investigated further, and it is recommended that several deep and shallow monitoring wells be installed at selected depths to assess the actual hydrostatic head and characteristics of the artesian aquifer to safely design temporary excavations, permanent foundation elements, and/or dewatering / depressurization requirements in this area. The existing standpipe piezometer (installed in Borehole HRE-3) should be maintained and remain operational to allow for continued monitoring of the groundwater level during detail design and up to construction, at which time the piezometer will need to be decommissioned in accordance with Ontario Regulation 903 (as amended).

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.

Signature Page

Golder Associates Ltd.



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REFERENCES

Bowles, J.E., 1984. Physical and Geotechnical Properties of Soils, Second Edition. McGraw Hill Book Company, New York.

Canadian Geotechnical Society. 2006. Canadian Foundation Engineering Manual (CFEM), 4th Edition. The Canadian Geotechnical Society, BiTech Publisher Ltd., British Columbia.

Canadian Standards Association (CSA) Group, 2019. Canadian Highway Bridge Design Code (CHBDC) and Commentary on CAN/CSA-S6-19.

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.

Demers, D., and Leroueil, S. 2002. Evaluation of preconsolidation pressure and the overconsolidaiton ratio from piezocone tests of clay deposits in Quebec. Canadian Geotechnical Journal, 39: 174-192.

Kulhawy, F.H. and Mayne, P.W. 1990. Manual on Estimating Soil Properties for Foundation Design. EL 6800, Research Project 1493 6. Prepared for Electric Power Research Institute, Palo Alto, California.

National Resources Canada, 2017. Earthquake Hazard. http://www.earthquakescanada.nrcan.gc.ca/hazard-alea/interpolat/index_2015-en.php.

Peck, R.B., Hanson, W.E., and Thornburn, T.H. 1974. Foundation Engineering, 2nd Edition, John Wiley and Sons, New York.

Terzaghi, K.V., 1955. Evaluation of Coefficient of Subgrade Reaction. Getechnique, 5(4): 297-326.

Terzaghi, K. and Peck, R.B., 1967. Soil Mechanics in Engineering Practice, 2nd Edition, John Wiley and Sons, New York.

Rendon-Herero, O., 1980. Universal Compression Index Equation, Journal of the Geotechnical Engineering

Division, A.S.C.E., 106(11), 1179-1200

Sowers, George B, and George F. Sowers. Introductory Soil Mechanics and Foundations, 1970. Print.

Unified Facilities Criteria, U.S. Navy. 1986. NAVFAC Design Manual 7.02. Soil Mechanics, Foundation and Earth Structures. Alexandria, Virginia.

Westergaard, H.M. 1938. A problem of elasticity suggested by a problem in soil mechanics: Soft material reinforced by numerous strong horizontal sheets. Contributions to the Mechanics of Solids, Stephen Timoshenko 60th birthday anniversary volume, Macmillan, New York. Pages 268-277.

Wroth, C.P., & Wood, D.M. 1978. THE CORRELATION OF INDEX PROPERTIES WITH SOME BASIC ENGINEERING PROPERTIES OF SOILS. Canadian Geotechnical Journal, 15, 137-145.

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 East Branch Bridge

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risk / Consequences
Spread footings founded on native compact silty sand to silt (east side) or stiff to very stiff clayey silt-silt (west side)	■ Marginally feasible at the site	■ Conventional construction ■ Relatively lightweight equipment to access wet, swampy areas anticipated on west side of river for piers	■ Anticipated high loading requires large foundation widths and native foundation soils can only offer a low geotechnical resistance at f-SLS and likely not feasible. ■ Anticipated settlement / consolidation of foundation soils due to embankment loading (or granular pads at piers) will exceed tolerable limits (25 mm) at abutments and will need to be mitigated (e.g. ground improvement such as preloading). ■ Subexcavation up to 4 m below ground surface (with high groundwater and potential artesian conditions on east side) will be challenging, especially in close proximity to river and/or environmentally sensitive areas. Temporary protection systems (possibly cofferdams) with dewatering/depressization and excavation/backfill controls likely required to limit footprint and control stability / unbalanced hydrostatic pressures. ■ Low geotechnical resistance compared to deep foundations ■ Less competent near surface soils (presence of and thicker peat or compressible soils) may exist at actual abutment and/or pier locations, especially on west side near river.	■ Lower cost than deep foundations although additional costs related to ground improvement / mitigation costs to address settlement, dewatering and temporary protection systems / cofferdams will need to be considered.	■ High anticipated structure loads will require large footing widths (up to 10 m) resulting in reduced f-SLS geotechnical resistances (compared to smaller footing widths) that will govern design. ■ Risk of excess total and differential settlement due to anticipated high foundation loads, approach embankment loads, and variable soil conditions. Settlement mitigation and monitoring required. ■ 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. ■ High risk of disturbance to founding soils during construction due to subexcavation in saturated cohesionless soils / layers (alluvial deposits) with high groundwater table, and likely under artesian pressures on the east side. ■ Significant dewatering / depressurization likely required for excavation and placement of concrete for footings.
"Perched" abutment spread footings founded on a compacted granular pad	■ Marginally feasible at the site	■ Conventional construction ■ Granular pad can be constructed within approach embankment for abutment locations. ■ Granular pad could also be considered for pier locations ■ Relatively lightweight equipment to access wet, swampy areas anticipated on west side of river ■ Increased geotechnical resistance at f-ULS but marginal increase in f-SLS compared to spread footings directly on native ground. Given high loads and anticipated spread footing widths up to 10 m, the granular pad thickness would need to be significant to lower the contact stress on the native foundation soils. The thickness of the granular pad is limited by the practical subexcavation limits and actual embankment heights.	■ Anticipated high loading requires large foundation widths and native foundation soils can only offer a low geotechnical resistance at f-SLS and likely not feasible. ■ Anticipated settlement / consolidation of foundation soils due to embankment loading (or granular pads at piers) will exceed tolerable limits (25 mm) at abutments and will need to be mitigated (e.g. ground improvement such as preloading). ■ Subexcavation up to 4 m below ground surface (with high groundwater and potential artesian conditions on east side) and replacement with granular pad will be challenging, especially in close proximity to river and/or environmentally sensitive areas. Temporary protection systems (possibly cofferdams) with dewatering/depressurization and excavation/backfill controls likely required to limit footprint and control stability / unbalanced hydrostatic pressures. ■ 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 ■ Lower geotechnical resistance compared to deep foundations ■ Less competent near surface soils may exist at actual abutment and/or pier locations, especially on west side.	■ Lower cost than deep foundations although additional costs related to ground improvement / mitigation costs to address settlement, dewatering and temporary protection systems / cofferdams will need to be considered.	■ High anticipated structure loads will require large footing widths (up to 10 m) and increased granular pad thickness to limit the reduction of the f-SLS geotechnical resistances (compared to smaller footing widths) that will govern design. ■ Logistics of increased granular pad thickness (practical subexcavation depth, height, and footprint) will need to be considered. ■ Risk of excess total and differential settlement due to anticipated high foundation loads, approach embankment and granular pad loads, and variable soil conditions. Settlement mitigation and monitoring required. ■ 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. ■ High risk of disturbance to founding soils during construction due to subexcavation in saturated cohesionless soils / layers (alluvial deposits) with high groundwater table, and likely artesian conditions on the east side. ■ Significant dewatering / depressurization likely required for excavation and replacement with compacted granular pad to support foundations.

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risk / Consequences
Foundation Option	**Feasibility**	**Advantages**	**Disadvantages**	**Relative Costs**	**Risk / Consequences**
Driven Steel Piles	■ Feasible for all foundation elements	■ Conventional construction methods for driven H-pile foundations. ■ Higher axial resistances compared to shallow footings. ■ Larger H-piles or tube piles can be considered to increase axial resistance. ■ Perched abutments can be considered to reduce dewatering / subexcavation for pile caps. ■ Likely feasible and preferred if deeper unsuitable soil deposits are encountered near ground surface within footprint.	■ Dewatering measures may be required for pile caps if they cannot be perched. ■ Heavy cranes / construction equipment may create access / laydown challenges in wet, swampy area anticipated on west side of river. ■ Relatively long (greater than 30 m) piles will be required and will be designed mainly on skin friction as there was no confirmed hard / very dense end-bearing stratum encountered within a 50 m depth.	■ Lower relative cost than drilled shafts (caissons) and may be comparable to spread footings if dewatering and subexcavation of unsuitable soils can be reduced. ■ Higher cost for access roads / platforms for pile driving equipment compared to shallow foundations.	■ Variable soil conditions (deeper peat or unsuitable soil deposits) on west side of river may lead to longer pile lengths. ■ Risk of lower geotechnical resistances during installation for friction pile design in predominantly silty soils with possible artesian groundwater conditions. A longer wait time to allow pore-water pressures to dissipate may be required when testing production piles. Alternatively, advanced static load testing may be considered. ■ Settlement of approach embankments (and any raised granular pads near piers) will cause potential downdrag loads on piles (reduced capacity) unless mitigation and monitoring is provided during construction. ■ Risk of water seepage / soil migration along piles on east side where artesian groundwater conditions were encountered.
Drilled Shafts (Caissons)	■ Feasible to marginally feasible	■ Offers higher geotechnical resistance compared to driven steel piles, requiring fewer foundation elements. ■ Larger diameter caissons can be considered to increase axial resistance. ■ Pile/caisson caps can likely be eliminated to reduce dewatering and subexcavation requirements.	■ Long drilled shafts (in excess of 30 m) likely required and may be challenging from constructability perspective, especially where artesian groundwater condition was encountered on the east side. ■ Given that there was no confirmed hard / very dense end-bearing stratum encountered within a 50 m depth and artesian groundwater conditions present, the caisson design is based mainly on skin friction and offers limited increase in resistance compacted to driven piles. ■ Temporary or permanent casings will be required, plus special measures such as use of polymer slurry to counterbalance groundwater pressures, reduce risk of blow-out in cohesionless layers/soils, and minimize disturbance. ■ Spoils will need to be properly contained and protected within floodplain and environmentally sensitive area.	■ Higher relative cost than driven piles. ■ Higher cost for access roads / platforms for caisson equipment compared to shallow foundations.	■ Variable soil conditions (deeper peat or unsuitable soil deposits) on west side of river may lead to longer caisson lengths. ■ Risk of lower geotechnical resistances during installation for friction caisson design in predominantly silty soils with possible artesian groundwater conditions. Higher geotechnical capacities could be considered if advanced load testing is considered (e.g. Osterberg Cell Test or Static Load Test). ■ Settlement of approach embankments (and any raised granular pads near piers) will cause potential downdrag loads on piles (reduced capacity) unless mitigation and monitoring is provided during construction. ■ Risk of water seepage / soil migration along caissons (with risk of impacting caisson integrity) on east side where artesian groundwater conditions were encountered. ■ Generation of soil cuttings and slurry will need to be contained in wetland and near river, and may required special permits / approvals. ■ Challenges associated with inspection of shaft walls and base may lead to conservative friction design and longer caissons.

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^3)	E (MPa)	Friction Angle (Degrees)	S_u (kPa)	P_c' (kPa)	e_0	C_c	C_r	C_v
Clayey Silt (Firm)	0.7	219.2	1.9	218	1.2	19	20	30	50	--	--	--	--	--
Silty Sand (Compact to Loose)	1.9	218	6.4	213.5	4.5	20	35	32	--	--	--	--	--	--
Sandy Silt (Compact to Very Dense)	6.4	213.5	17.8	202.1	11.4	20	80	35	--	--	--	--	--	--
Silty Sand (Compact to Dense)	17.8	202.1	24.5	195.4	6.7	20	65	33	--	--	--	--	--	--
Clayey Silt to Clayey Silt-Silt (Very Stiff)	24.5	195.4	34.9	185	10.4	20	90	31	125 to 170	570 to 770	0.6	0.1	0.01	0.0134
	34.9	185	52.4	167.5	17.5	19	90	31	170 to 190	770 to 865	0.6	0.2	0.02	0.0134

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^3)	E (MPa)	Friction Angle (Degrees)	S_u (kPa)	P_c' (kPa)	e_0	C_c	C_r	C_v (cm^2/sec)
Sandy Silt (Loose)	1.6	217.9	3.1	216.4	1.5	19	20	28	--	--	--	--	--	--
Clayey Silt to Clayey Silt-Silt (Stiff to Hard)	3.1	216.4	8.5	211	5.4	20	30 to 130	32	55 to 400	250 to 1800	0.6	0.1	0.01	0.0134
	8.5	211	13.9	205.6	5.4	20	130	33	400 to 245	1800 to 1100	0.6	0.1	0.01	0.0134
Silty Sand to Sandy Silt (Very Dense)	13.9	205.6	15.6	203.9	1.7	20	80	35	--	--	--	--	--	--
Clayey Silt to Clayey Silt-Silt (Very Stiff)	15.6	203.9	18.5	201	2.9	20	130	33	200 to 115	900 to 525	0.6	0.1	0.01	0.0134
	18.5	201	22.9	196.6	4.4	20	130	33	115 to 124	525 to 565	0.6	0.1	0.01	0.0134
Sandy Silt to Silt (Compact)	22.9	196.6	24.6	194.9	1.7	20	80	32	--	--	--	--	--	--
Clayey Silt to Clayey Silt-Silt (Very Stiff)	24.6	194.9	34.5	185	9.9	20	90	31	125 to 170	570 to 770	0.6	0.1	0.01	0.0134
	34.5	185	50.9	168.6	16.4	19	90	31	170 to 190	770 to 865	0.6	0.2	0.02	0.0134

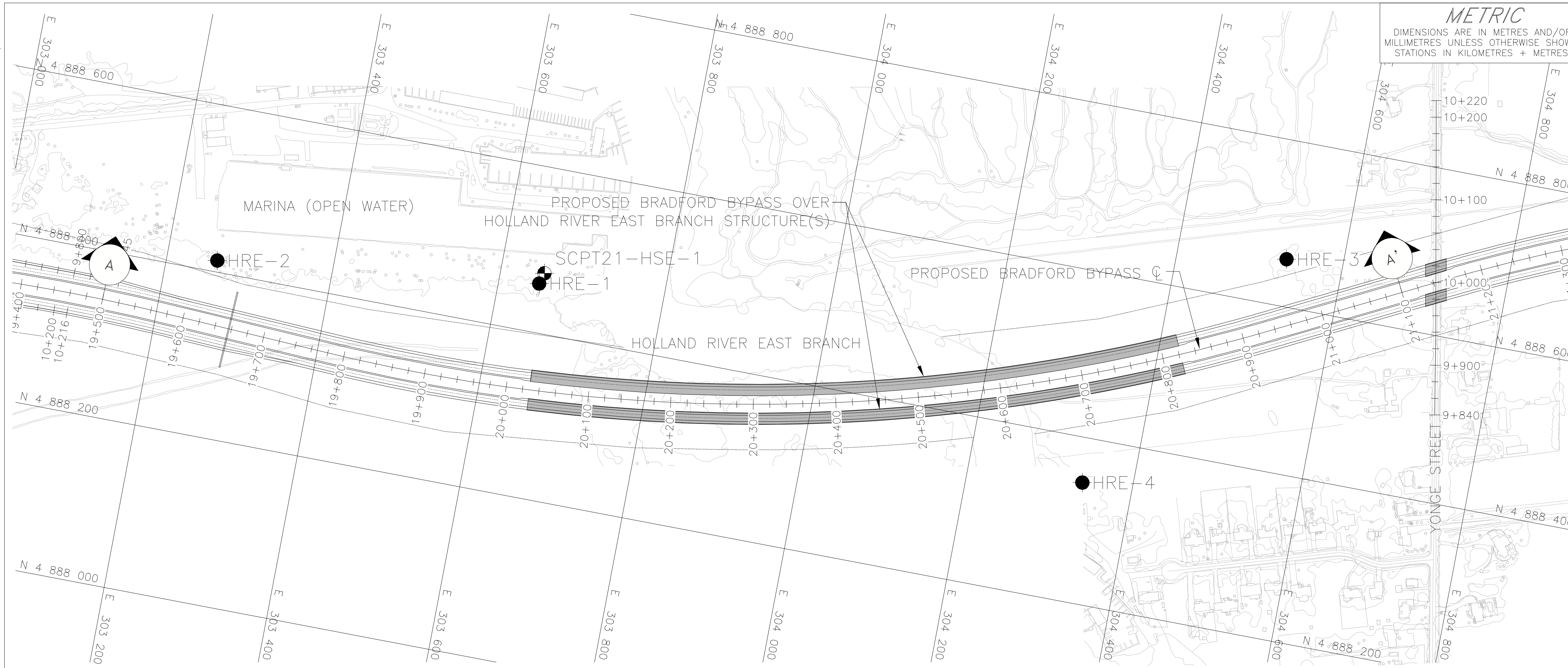
Drawings

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

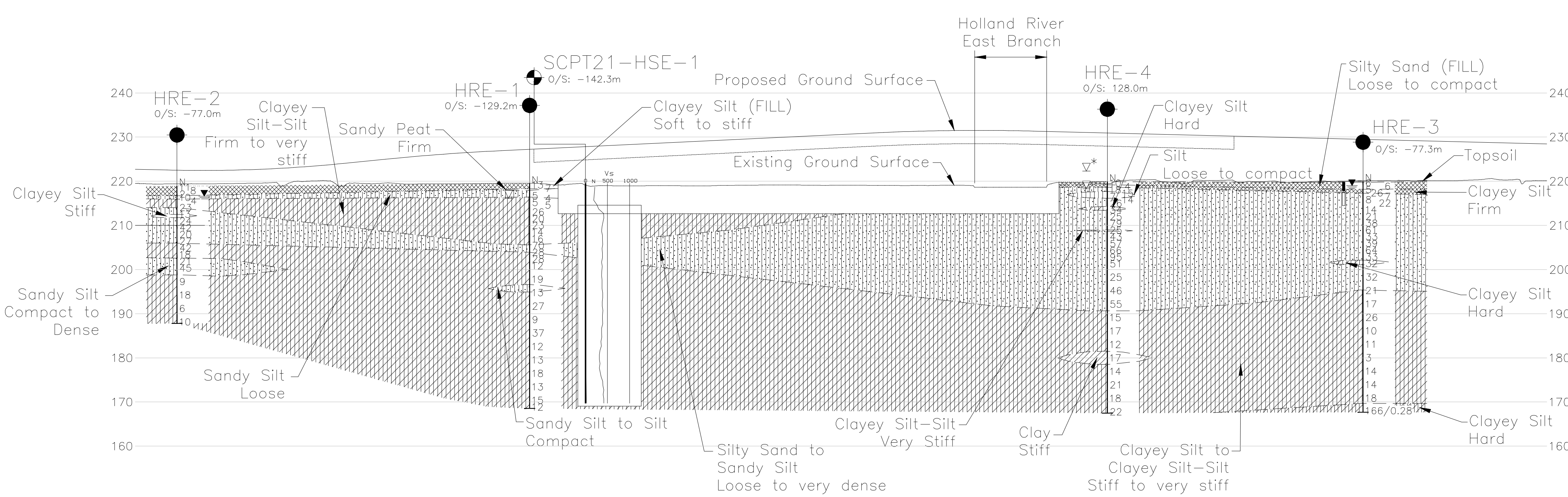
CONT No.
WP No.

BRADFORD BYPASS
HOLLAND RIVER EAST BRANCH CROSSINGS
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET



PLAN
SCALE
60 0 60 120 m



A-A' PROFILE BRADFORD BYPASS ℄

HORIZONTAL SCALE
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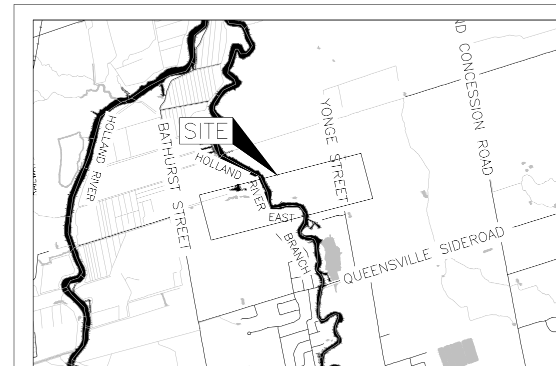
VERTICAL SCALE
12 0 12 24 m



KEY PLAN
SCALE
1 0 1 2 km

LEGEND

●	Borehole – Current Investigation
◐	Seismic Cone Penetration Test
▮	Seal
▯	Piezometer
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
Vs	Shear Wave Velocity (m/s)
▼	WL in piezometer, measured on May 13, 2022
▽	WL upon completion of drilling
▽*	Artesian WL encountered during drilling

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
HRE-1	219.5	4888459.3	303648.6
HRE-2	218.9	4888413.9	303260.8
HRE-3	220.0	4888657.1	304531.7
HRE-4	219.9	4888345.7	304339.4
SCPT21-HSE-1	219.5	4888472.8	303652.6

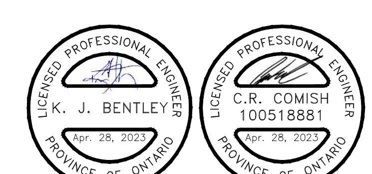
NOTES

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.



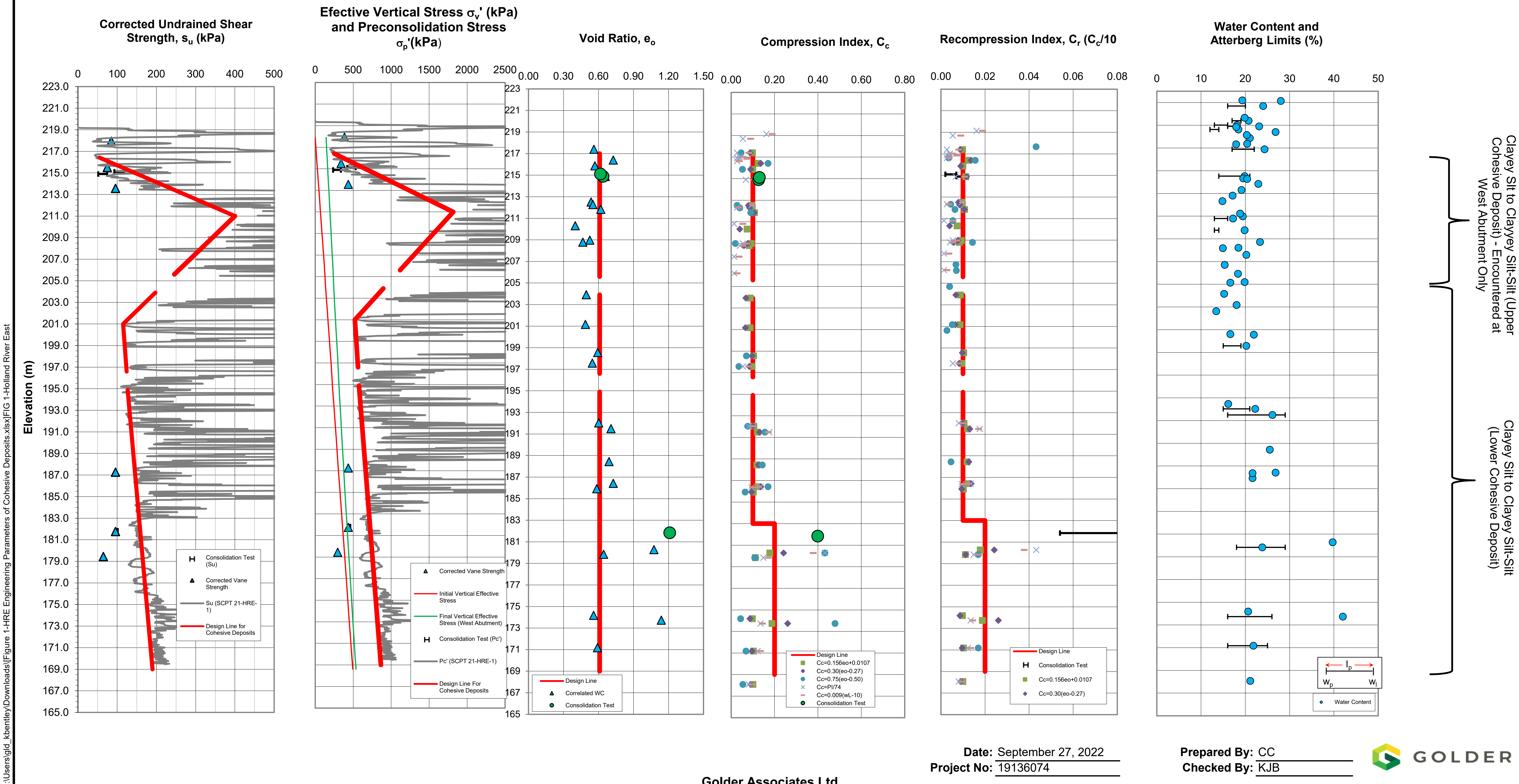
NO.	DATE	BY	REVISION

Geocres No. 31D-812			
HWY.	PROJECT NO. 19136074		DIST.
SUBM'D. MA	CHKD. MA	DATE: 04/25/2023	SITE:
DRAWN: DD/SA	CHKD. CC	APPD. KJB	DWG. 1

Figures

SUMMARY PLOT OF ENGINEERING PARAMETERS FOR COHESIVE DEPOSIT
HOLLAND RIVER EAST BRANCH CROSSING

FIGURE 1



Date: September 27, 2022
Project No: 19136074

Prepared By: CC
Checked By: KJB

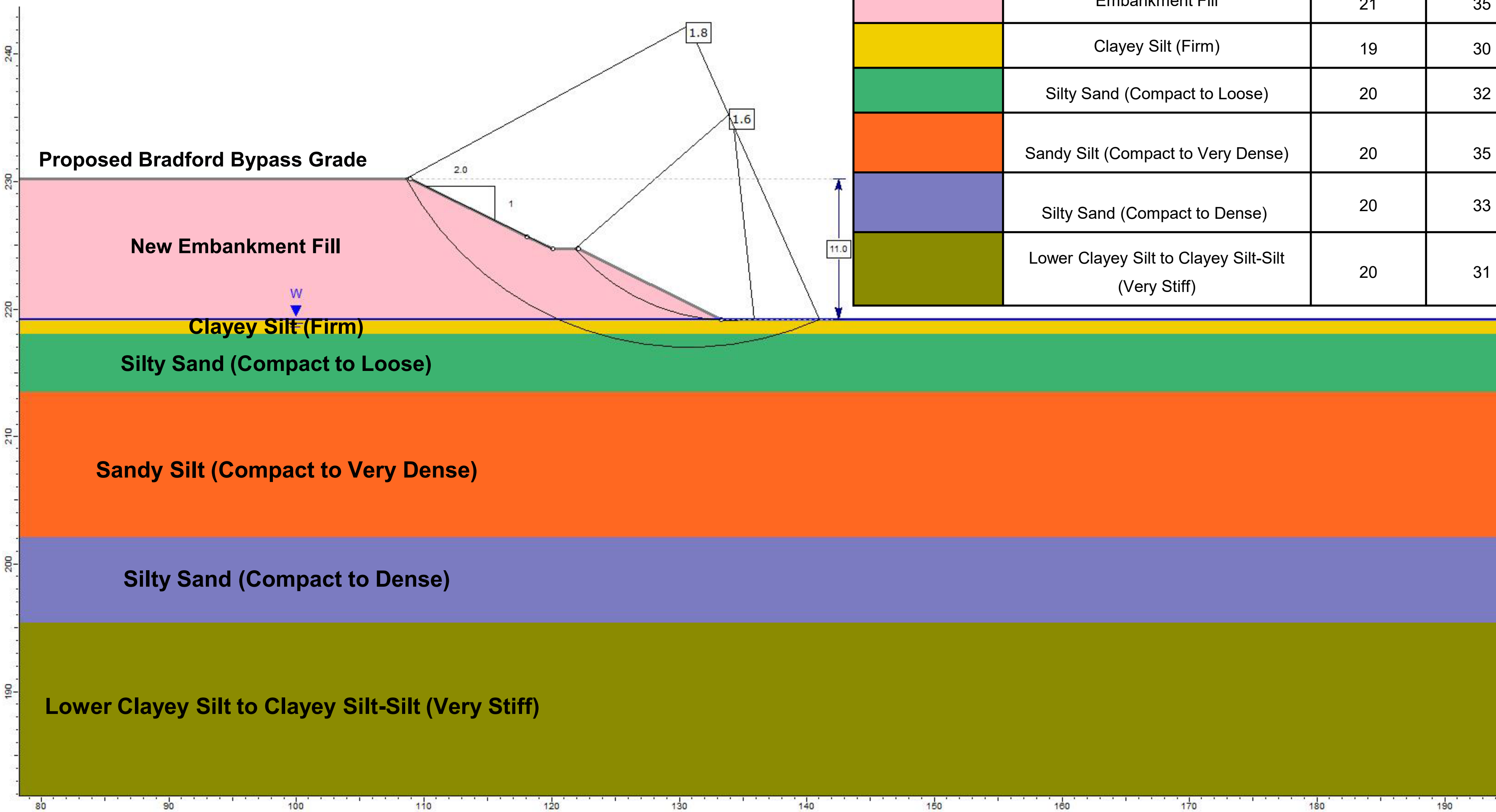
GOLDER

Golder Associates Ltd.

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

Figure 2

Colour	Soil Description	Unit Weight (kN/m³)	Internal Angle of Friction, ϕ (Degrees)
	Embankment Fill	21	35
	Clayey Silt (Firm)	19	30
	Silty Sand (Compact to Loose)	20	32
	Sandy Silt (Compact to Very Dense)	20	35
	Silty Sand (Compact to Dense)	20	33
	Lower Clayey Silt to Clayey Silt-Silt (Very Stiff)	20	31



Date: September 2022
Project No: 19136074

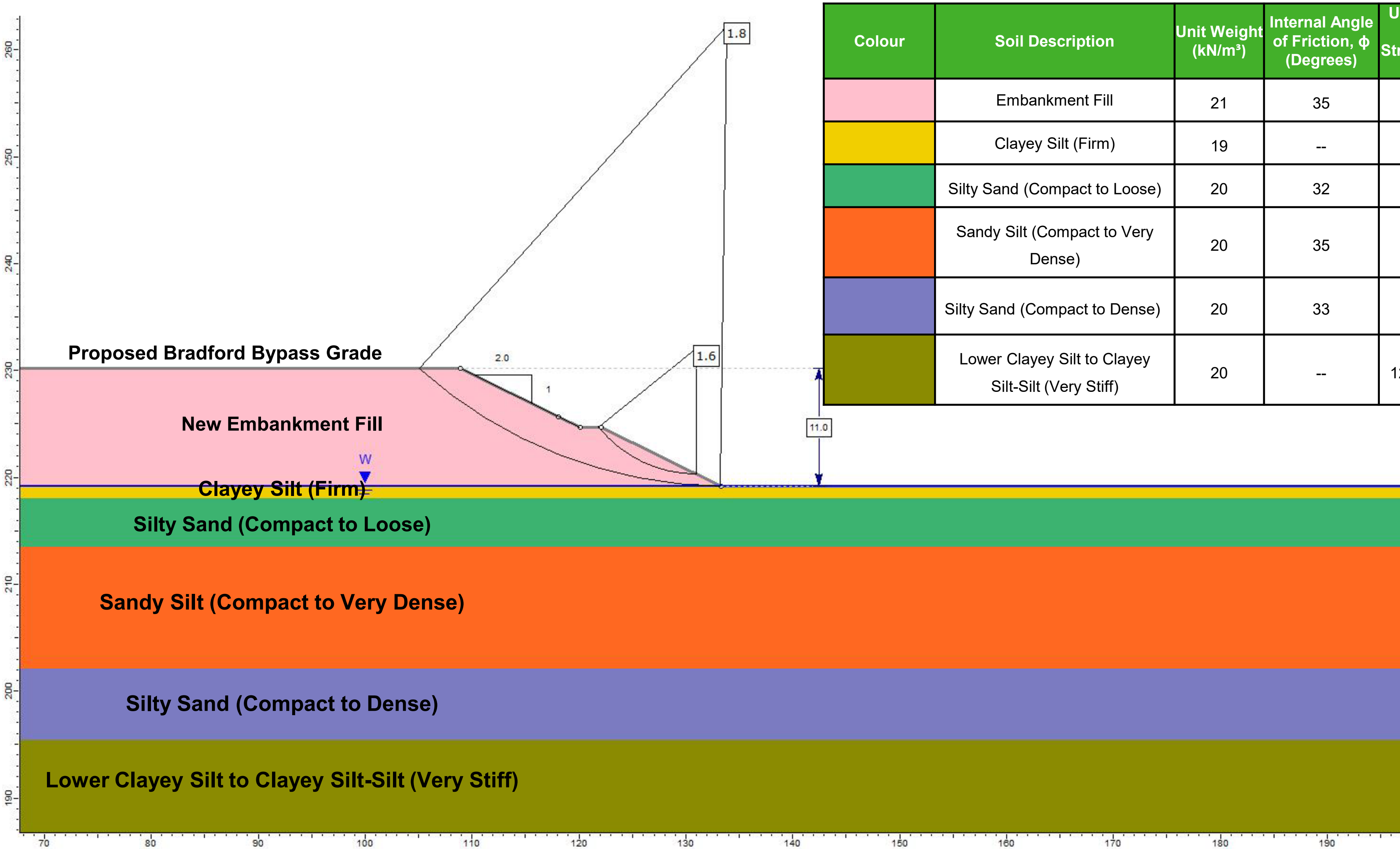
Analysis By: CC **Reviewed By:** KJB

GOLDER

Bradford Bypass– Holland River East Branch (East Abutment)
Stability Analysis Results (Undrained Condition)

Figure 3

Colour	Soil Description	Unit Weight (kN/m³)	Internal Angle of Friction, ϕ (Degrees)	Undrained Shear Strength, Su (kPa)
	Embankment Fill	21	35	--
	Clayey Silt (Firm)	19	--	50
	Silty Sand (Compact to Loose)	20	32	--
	Sandy Silt (Compact to Very Dense)	20	35	--
	Silty Sand (Compact to Dense)	20	33	--
	Lower Clayey Silt to Clayey Silt-Silt (Very Stiff)	20	--	125 to 190



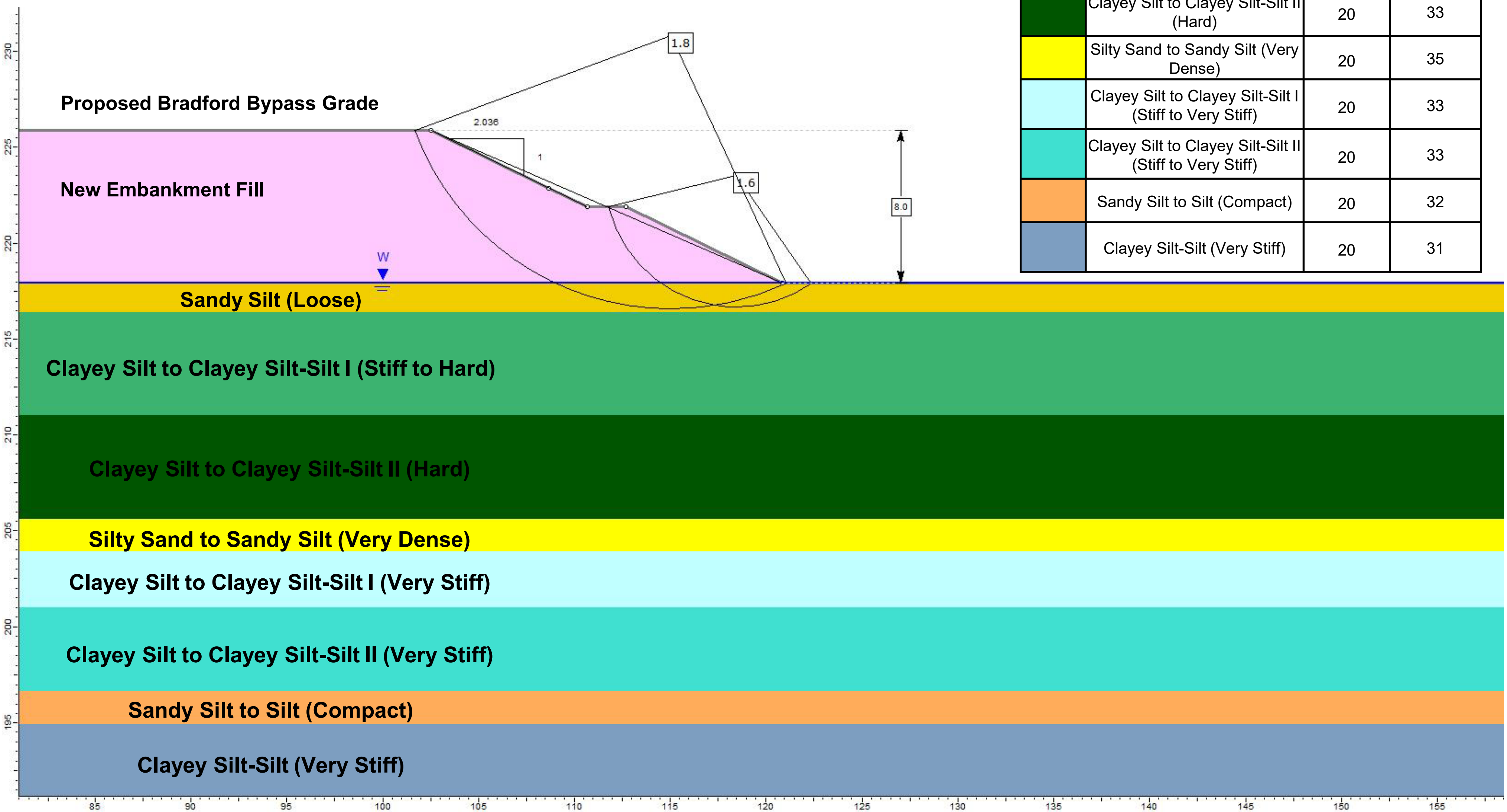
Date: September 2022
Project No: 19136074

Analysis By: CC **Reviewed By:** KJB

Figure 4

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

Colour	Soil Description	Unit Weight (kN/m³)	Internal Angle of Friction, ϕ (Degrees)
Pink	Embankment Fill	21	35
Gold	Sandy Silt (Loose)	19	28
Green	Clayey Silt to Clayey Silt-Silt I (Stiff to Hard)	20	32
Dark green	Clayey Silt to Clayey Silt-Silt II (Hard)	20	33
Yellow	Silty Sand to Sandy Silt (Very Dense)	20	35
Light cyan	Clayey Silt to Clayey Silt-Silt I (Stiff to Very Stiff)	20	33
Turquoise	Clayey Silt to Clayey Silt-Silt II (Stiff to Very Stiff)	20	33
Orange	Sandy Silt to Silt (Compact)	20	32
Blue-grey	Clayey Silt-Silt (Very Stiff)	20	31



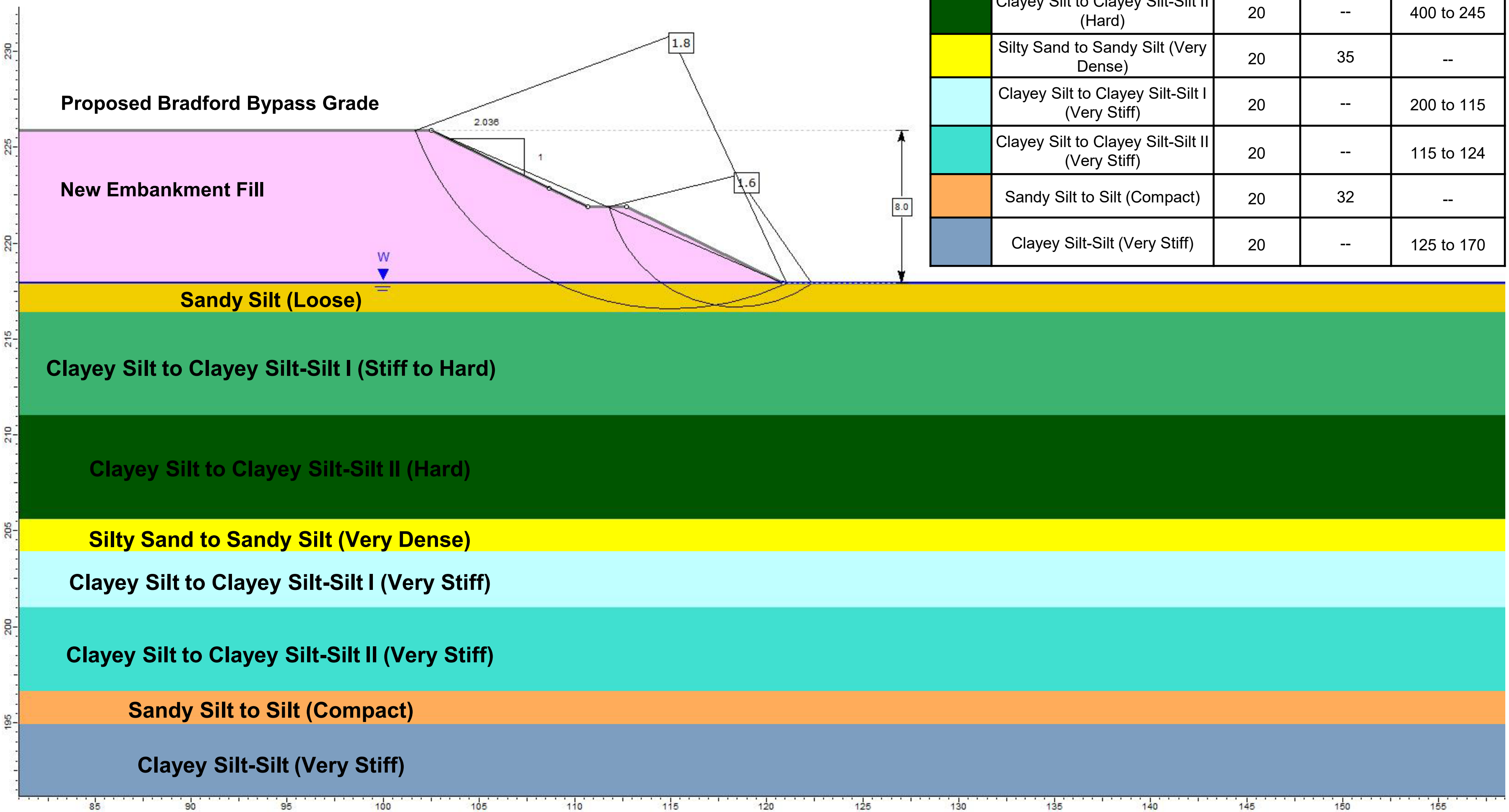
Date: September 2022
Project No: 19136074

Analysis By: CC **Reviewed By:** KJB

Figure 5

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

Colour	Soil Description	Unit Weight (kN/m³)	Internal Angle of Friction, ϕ (Degrees)	Undrained Shear Strength, Su (kPa)
	Embankment Fill	21	35	--
	Sandy Silt (Loose)	19	28	--
	Clayey Silt to Clayey Silt-Silt I (Stiff to Hard)	20	--	55 to 400
	Clayey Silt to Clayey Silt-Silt II (Hard)	20	--	400 to 245
	Silty Sand to Sandy Silt (Very Dense)	20	35	--
	Clayey Silt to Clayey Silt-Silt I (Very Stiff)	20	--	200 to 115
	Clayey Silt to Clayey Silt-Silt II (Very Stiff)	20	--	115 to 124
	Sandy Silt to Silt (Compact)	20	32	--
	Clayey Silt-Silt (Very Stiff)	20	--	125 to 170



Date: September 2022
Project No: 19136074

Analysis By: CC **Reviewed By:** KJB

APPENDIX A

Borehole Records and SCPT Results

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

PARTICLE SIZES OF CONSTITUENTS

Soil Constituent	Particle Size Description	Millimetres	Inches (US Std. Sieve Size)
BOULDERS	Not Applicable	>200	>8
COBBLES	Not Applicable	75 to 200	3 to 8
GRAVEL	Coarse Fine	19 to 75 4.75 to 19	0.75 to 3 (4) to 0.75
SAND	Coarse Medium Fine	2.00 to 4.75 0.425 to 2.00 0.075 to 0.425	(10) to (4) (40) to (10) (200) to (40)
FINES	Classified by plasticity	<0.075	< (200)

MODIFIERS FOR SECONDARY COMPONENTS[1,2]

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

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

PENETRATION RESISTANCE

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

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

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

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

SAMPLES

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

SOIL TESTS

w	water content
PL , w_p	plastic limit
LL , w_L	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, Gs)
DS	direct shear test
GS	specific gravity
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V (FV)	field vane (LV-laboratory vane test)
γ	unit weight

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

COARSE-GRAINED SOILS

Compactness[1]

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

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

FINE-GRAINED SOILS

Consistency

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

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

Field Moisture Condition

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

LIST OF SYMBOLS
MINISTRY OF TRANSPORTATION, ONTARIO

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

I. GENERAL

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

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

Ontario

wsp

PROJECT 19136074	**RECORD OF BOREHOLE No. HRE-1**	Sheet 1 of 6	**METRIC**
G.W.P. Assignment No 2019-E-0048	LOCATION N 4888459.3; E 303648.6 NAD83 / MTM Zone 10 (LAT. 44.136129; LONG. -79.514389)		ORIGINATED BY AM
DIST Central HWY BBP	BOREHOLE TYPE 210 mm Hollow Stem Auger; Mud Rotary		COMPILED BY MA/ MTI
DATUM CGVD28 Surface Elevation:219.5 m	DATE Sep 29, 2021 - Oct 01, 2021		CHECKED BY KJB

ELEV. / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	SHEAR STRENGTH (kPa)	WATER CONTENT	GR	SA	SI	CL
0.0	CLAYEY SILT-SILT (CL-ML), some sand to sandy; trace rootlets (FILL) Stiff to firm Brown Moist	1	SS	13						
		2A	SS	7			0	34	56	10
218.3 / 1.2	Sandy PEAT (PT), trace rootlets Firm Blackish brown Moist	2B								
217.9 / 1.6	Sandy SILT (ML), some clay Loose Brown Wet	3	TO							
		4	SS	5			0	22	64	14
216.4 / 3.0	CLAYEY SILT -SILT (CL-ML) to CLAYEY SILT (CL), trace sand Firm to very stiff Brown Moist	5	SS	4						
	- 4.0 m: - contains silt pockets/ seams throughout	6	SS	5						
	- 4.6 m: - Following borehole completion, an additional borehole adjacent to HRE-1 was advanced to a depth of 4.6 m and a shelby tube (designated as HRE-1 TO-4) was obtained.	7	SS	5			0	1	78	21
					> 96 kPa					
		8	SS	26						
	- 7.9 to 8.0 m: - sand seam; becoming sandy at 9.73 m	9	SS	20						
	- 8.3 m: - colour changes from brown to greyish brown									
		10	SS	23						

Column headings: SOIL PROFILE; STRATA PLOT; SAMPLES; GROUNDWATER CONDITIONS; ELEVATION SCALE (219, 218, 217, 216, 215, 214, 213, 212, 211, 210); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH (kPa): × Field Vane, ⊕ Remoulded, ■ Pocket Pen, ● Quick Triaxial, ○ Unconfined; WATER CONTENT (%) PL W_p, NMC W, LL W_l, NP Nonplastic, 20 40 60; UNIT WEIGHT γ kN/m³; GR SA SI CL; REMARKS

Continued on Next Page

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

Ontario

wsp

PROJECT	19136074	**RECORD OF BOREHOLE No. HRE-1**		Sheet 2 of 6	**METRIC**
G.W.P.	Assignment No 2019-E-0048	LOCATION	N 4888459.3; E 303648.6 NAD83 / MTM Zone 10 (LAT. 44.136129; LONG. -79.514389)	ORIGINATED BY	AM
DIST	Central HWY BBP	BOREHOLE TYPE	210 mm Hollow Stem Auger; Mud Rotary	COMPILED BY	MA/ MTI
DATUM	CGVD28 Surface Elevation:219.5 m	DATE	Sep 29, 2021 - Oct 01, 2021	CHECKED BY	KJB

ELEV. / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL	REMARKS
	CLAYEY SILT -SILT (CL-ML) to CLAYEY SILT (CL), trace sand Firm to very stiff Brown Moist				209						
		11	SS	14			0	4	70	26	
					208						
	- 12.2 m: -silt seam	12	SS	16	207						
					206						
205.6 / 13.9	SILT (ML) and SAND (SM), trace clay Very dense Greyish brown Wet	13 A / 13 B	SS	70			0	46	48	6	
					205						
203.9 / 15.5	CLAYEY SILT (CL), some sand Very stiff Greyish brown Wet	14 A / 14 B	SS	28	204						
203.2 / 16.3	CLAYEY SILT-SILT (CL-ML), trace sand Stiff to very stiff Greyish brown Wet				203						
		15	SS	28	202						
	- 18.3 m: -silt pockets from 18.3 m to 18.9 m	16	SS	12	201						
					200						

Column headings also printed: STRATA PLOT; GROUNDWATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH (kPa): ○ Unconfined, ● Quick Triaxial, ■ Pocket Pen, ⊕ Remoulded, X Field Vane; WATER CONTENT (%): PL W_p, NMC W, LL W_l, NP Nonplastic, 20 40 60; UNIT WEIGHT γ kN/m³.

Continued on Next Page

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

Ontario

RECORD OF BOREHOLE No. HRE-1

METRIC

PROJECT	19136074				
G.W.P.	Assignment No 2019-E-0048	LOCATION	N 4888459.3; E 303648.6 NAD83 / MTM Zone 10 (LAT. 44.136129; LONG. -79.514389)	ORIGINATED BY	AM
DIST	Central HWY BBP	BOREHOLE TYPE	210 mm Hollow Stem Auger; Mud Rotary	COMPILED BY	MA/ MTI
DATUM	CGVD28 Surface Elevation:219.5 m	DATE	Sep 29, 2021 - Oct 01, 2021	CHECKED BY	KJB

ELEV. / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR SA SI CL	REMARKS
	CLAYEY SILT-SILT (CL-ML), trace sand Stiff to very stiff Greyish brown Wet				199				
		17	SS	19	198				
					197				
196.6 22.9	Sandy SILT to SILT (ML) Compact Greyish brown Wet				196				
194.9 24.5	CLAYEY SILT-SILT (CL-ML) to CLAYEY SILT (CL) Stiff to very stiff Greyish brown Wet	18A 18B	SS	13	195				
					194				
					193				
	- 27.8 m: -75 mm thick silt layer	19	SS	27	192	PL–NMC plotted (~20)		0 0 73 27	
					191				

Column headings: SOIL PROFILE (ELEV. DEPTH, DESCRIPTION, STRATA PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUNDWATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH (kPa): ○ Unconfined, ● Quick Triaxial, ■ Pocket Pen, ⊕ Remoulded, × Field Vane; WATER CONTENT (%): PL W_p, NMC W, LL W_l (20 40 60), NP Nonplastic; UNIT WEIGHT γ kN/m³; GR SA SI CL; REMARKS.

Continued on Next Page

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

Ontario

PROJECT	19136074		
G.W.P.	Assignment No 2019-E-0048	LOCATION	N 4888459.3; E 303648.6 NAD83 / MTM Zone 10 (LAT. 44.136129; LONG. -79.514389)
ORIGINATED BY	AM		
DIST	Central HWY BBP	BOREHOLE TYPE	210 mm Hollow Stem Auger; Mud Rotary
COMPILED BY	MA/ MTI		
DATUM	CGVD28 Surface Elevation:219.5 m	DATE	Sep 29, 2021 - Oct 01, 2021
CHECKED BY	KJB		

ELEV. / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH (kPa)	WATER CONTENT (%)
	CLAYEY SILT-SILT (CL-ML) to CLAYEY SILT (CL) Stiff to very stiff Greyish brown Wet				189		
		20	SS	9			
188.2 / 31.2	CLAYEY SILT (CL) trace sand Stiff to hard Grey Wet	21	TO		188		
						× > 96 kPa	
					187		
					186		
		22	SS	37			o
	- 34.0 m: - 300 mm thick silty sand layer				185		
					184		
	- 36.5 m: - contains silt pockets				183		
		23	SS	12			
					182	× > 96 kPa	
	- 38.1 m: - attempted shelby tube extraction but no recovery				181		

Legend: Field Vane (×), Remoulded (⊕), Pocket Pen (■), Quick Triaxial (●), Unconfined (○). DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100. WATER CONTENT (%): PL (W_p), NMC (W), LL (W_l); NP Nonplastic; 20 40 60. UNIT WEIGHT γ kN/m³. GR SA SI CL. REMARKS.

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

Ontario

PROJECT 19136074

RECORD OF BOREHOLE No. HRE-1

METRIC

G.W.P. Assignment No 2019-E-0048

LOCATION N 4888459.3; E 303648.6 NAD83 / MTM Zone 10 (LAT. 44.136129; LONG. -79.514389)

ORIGINATED BY AM

DIST Central HWY BBP

BOREHOLE TYPE 210 mm Hollow Stem Auger; Mud Rotary

COMPILED BY MA/ MTI

DATUM CGVD28 Surface Elevation:219.5 m

DATE Sep 29, 2021 - Oct 01, 2021

CHECKED BY KJB

ELEV. / DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES	GROUNDWATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH (kPa)	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR SA SI CL	REMARKS
	CLAYEY SILT (CL) trace sand Stiff to hard Grey Wet						180					
			24	SS	13						0 1 59 40	
							179					
							178					
							177					
			25	SS	18							
							176					
							175					
							174					
	- 45.7 m: - silt pocket at a depth of 45.7 m											
			26	SS	13						0 0 64 36	
							173					
							172					
							171					

Legend: ✕ Field Vane; ⊕ Remoulded; ■ Pocket Pen; ● Quick Triaxial; ○ Unconfined. DCPT scale: 20 40 60 80 100. Water content: PL (W_p), NMC (W), LL (W_l); NP Nonplastic; scale 20 40 60.

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

Ontario

PROJECT	19136074	RECORD OF BOREHOLE No. HRE-1	Sheet 6 of 6	METRIC	
G.W.P.	Assignment No 2019-E-0048	LOCATION	N 4888459.3; E 303648.6 NAD83 / MTM Zone 10 (LAT. 44.136129; LONG. -79.514389)	ORIGINATED BY	AM
DIST	Central HWY BBP	BOREHOLE TYPE	210 mm Hollow Stem Auger; Mud Rotary	COMPILED BY	MA/ MTI
DATUM	CGVD28 Surface Elevation:219.5 m	DATE	Sep 29, 2021 - Oct 01, 2021	CHECKED BY	KJB

ELEV. / DEPTH	DESCRIPTION (SOIL PROFILE)	STRATA PLOT	NUMBER (SAMPLES)	TYPE	"N" VALUES	GROUNDWATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH (kPa) ○ Unconfined ● Quick Triaxial ■ Pocket Pen ⊕ Remoulded × Field Vane	WATER CONTENT (%) PL W_p NMC W LL W_l NP Nonplastic 20 40 60	UNIT WEIGHT γ kN/m³	GR SA SI CL	REMARKS
	CLAYEY SILT (CL) trace sand Stiff to hard Grey Wet		27	SS	15		170					
	- 50.3 m: - trace gravel encountered in sample		28	SS	12		169					
168.6 50.9	End of Borehole Notes: 1. Water observed at a depth of 2.5 m below ground surface (El. 217.0 m) prior to mud-rotary. 2. Following borehole completion, an additional borehole adjacent to HRE-1 was advanced to a depth of 4.6 m and a shelby tube sample (designated HRE-1, Sample No. TO-4) was collected.						168 167 166 165 164 163 162 161					

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

Ontario — WSP GOLDER

RECORD OF BOREHOLE No. HRE-2

METRIC

PROJECT	19136074		
G.W.P.	Assignment No 2019-E-0048	LOCATION	N 4888413.9; E 303260.8 NAD83 / MTM Zone 10 (LAT. 44.135719; LONG. -79.519236)
ORIGINATED BY	MTI		
DIST	Central HWY BBP	BOREHOLE TYPE	210 mm Hollow Stem Auger; Mud Rotary
COMPILED BY	MA/ MTI		
DATUM	CGVD28 Surface Elevation:218.9 m	DATE	Dec 03, 2021 - Dec 06, 2021
CHECKED BY	KJB		

ELEV. / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUNDWATER CONDITIONS	WATER CONTENT	GR SA SI CL	REMARKS
0.0	Sandy CLAYEY SILT-SILT (CL-ML), trace rootlets, trace gravel (possible FILL) soft to stiff Brown to greenish grey Moist - 0.4 m: - sand seam layer (130 mm thick) - 0.8 m: - silty sand layer (75 mm thick)	1	SS	11				
		2	SS	8				
		3	SS	2		plotted		
216.7 / 2.2	Sandy SILT (ML), some clay, trace organics in upper zone Loose Greyish brown (light brown) Wet	4	SS	10	▽	plotted	0 32 55 13	
216.0 / 3.0	CLAYEY SILT-SILT (CL-ML), trace organics Firm to very stiff Greyish brown (light brown) Wet	5	SS	4		plotted		
		6	TO					
		7A	SS	23				
214.0 / 4.9	Sandy SILT (ML) Compact Brownish grey Wet - 6.1 m: - 75mm thick clayey layer	7B						
		8A	SS	13				
212.5 / 6.4	CLAYEY SILT-SILT (CL-ML), some sand Stiff Grey Moist	8B				plotted		
211.8 / 7.2	Sandy SILT (ML) to SILT (ML), some clay Compact Grey Moist to Wet	9	SS	24		plotted	0 32 56 12	
209.3 / 9.6	SILT (ML), some clay, some sand to sandy Dense to compact Grey Moist to wet	10	SS	42				

Elevation scale: 218, 217, 216, 215, 214, 213, 212, 211, 210

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH (kPa): ○ Unconfined, ● Quick Triaxial, ■ Pocket Pen, ⊕ Remoulded, × Field Vane
WATER CONTENT (%): PL W_p, NMC W, LL W_l; NP Nonplastic; 20 40 60
UNIT WEIGHT γ kN/m³

Continued on Next Page

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

Ontario | WSP GOLDER

PROJECT 19136074 **RECORD OF BOREHOLE No. HRE-2** **METRIC**

G.W.P. Assignment No 2019-E-0048 | LOCATION N 4888413.9; E 303260.8 NAD83 / MTM Zone 10 (LAT. 44.135719; LONG. -79.519236) | ORIGINATED BY MTI

DIST Central HWY BBP | BOREHOLE TYPE 210 mm Hollow Stem Auger; Mud Rotary | COMPILED BY MA/ MTI

DATUM CGVD28 Surface Elevation:218.9 m | DATE Dec 03, 2021 - Dec 06, 2021 | CHECKED BY KJB

ELEV. / DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR SA SI CL	REMARKS
	SILT (ML), some clay, some sand to sandy Dense to compact Grey Moist to wet				209				
		11	SS	20	208				
					207				
		12	SS	27		○ NP		0 18 70 12	
206.0 / 13.0	CLAYEY SILT-SILT (CL-ML), trace to some sand Very stiff Grey Moist to wet				206				
		13	SS	42	205				
					204				
	- 15.2 to 15.4 m: -sand laye	14	SS	18	203				
202.6 / 16.3	Sandy SILT (ML), trace clay Compact to dense Grey Moist to wet								
		15	SS	21	202	○ NP		0 20 72 8	
					201				
	- 18.3 m: - clayey silt layer (125 mm thick)	16	SS	45		○			
					200				

Plot legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH (kPa): ○ Unconfined, ● Quick Triaxial, ■ Pocket Pen, ⊕ Remoulded, X Field Vane; WATER CONTENT (%): PL NMC LL, W_p W W_l (20 40 60); NP Nonplastic

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

Ontario

WSP GOLDER

PROJECT 19136074	**RECORD OF BOREHOLE No. HRE-2**	Sheet 3 of 4		**METRIC**
G.W.P. Assignment No 2019-E-0048	LOCATION N 4888413.9; E 303260.8 NAD83 / MTM Zone 10 (LAT. 44.135719; LONG. -79.519236)		ORIGINATED BY	MTI
DIST Central HWY BBP	BOREHOLE TYPE 210 mm Hollow Stem Auger; Mud Rotary		COMPILED BY	MA/ MTI
DATUM CGVD28 Surface Elevation:218.9 m	DATE Dec 03, 2021 - Dec 06, 2021		CHECKED BY	KJB

ELEV. / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL	REMARKS
	Sandy SILT (ML), trace clay Compact to dense Grey Moist to wet				199						
198.8 / 20.1	CLAYEY SILT-SILT (CL-ML), interbedded with silt and sand pockets / layers Stiff to very stiff Grey Moist				198						
		17	SS	9							
	- 21.9 to 22.0 m: -sand layer				197						
					196						
					195						
	- 24.3 m: -silt pockets encountered										
		18	SS	18							
					194						
					193						
192.7 / 26.2	CLAYEY SILT (CL) Firm to stiff Grey Moist				192						
		19	SS	6			0	0	56	44	
					191						
					190						

Chart headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH (kPa): ○ Unconfined, ⊕ Field Vane, ✕ Remoulded, ● Quick Triaxial, ■ Pocket Pen; WATER CONTENT (%): PL W_p, NMC W, LL W_l (20 40 60), NP Nonplastic; UNIT WEIGHT γ kN/m³; GROUNDWATER CONDITIONS; STRATA PLOT.

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

Ontario

wsp GOLDER

PROJECT	19136074	RECORD OF BOREHOLE No. HRE-2	Sheet 4 of 4	METRIC
G.W.P.	Assignment No 2019-E-0048	LOCATION: N 4888413.9; E 303260.8 NAD83 / MTM Zone 10 (LAT. 44.135719; LONG. -79.519236)	ORIGINATED BY	MTI
DIST	Central HWY BBP	BOREHOLE TYPE: 210 mm Hollow Stem Auger; Mud Rotary	COMPILED BY	MA/ MTI
DATUM	CGVD28 Surface Elevation:218.9 m	DATE: Dec 03, 2021 - Dec 06, 2021	CHECKED BY	KJB

ELEV. / DEPTH	SOIL PROFILE DESCRIPTION	STRATA PLOT	SAMPLES NUMBER	SAMPLES TYPE	"N" VALUES	GROUNDWATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH (kPa) (○ Unconfined, ● Quick Triaxial, ■ Pocket Pen, ⊕ Remoulded, X Field Vane) 20 40 60 80 100	WATER CONTENT (%) PL (W_p) NMC (W) LL (W_l), NP Nonplastic 20 40 60	UNIT WEIGHT γ kN/m³	GR SA SI CL	REMARKS
	CLAYEY SILT (CL) Firm to stiff Grey Moist						189					
			20	SS	10		188		○			
187.8 31.1	End of Borehole Note: 1. Groundwater encountered at a depth of 2.3 m (El. 216.6 m) during advancement of hollow stem auger and prior to introducing water for mud rotary.						187					
							186					
							185					
							184					
							183					
							182					
							181					
							180					

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

Ontario | WSP GOLDER

PROJECT 19136074	**RECORD OF BOREHOLE No. HRE-3** Sheet 1 of 6		**METRIC**
G.W.P. Assignment No 2019-E-0048	LOCATION N 4888657.1; E 304531.7 NAD83 / MTM Zone 10 (LAT. 44.13791; LONG. -79.503353)	ORIGINATED BY	DP
DIST Central HWY BBP	BOREHOLE TYPE 210 mm Hollow Stem Auger; Mud Rotary	COMPILED BY	MA/ MTI
DATUM CGVD28 Surface Elevation:220.0 m	DATE Jan 13, 2022 - Jan 25, 2022	CHECKED BY	KJB

SOIL PROFILE

ELEV. / DEPTH	DESCRIPTION
0.0	SILTY SAND (SM), trace organics including rootlets, (TOPSOIL) Dark Brown Dry to moist
219.8 / 0.2	SILTY SAND (SM), trace clay, trace gravel, trace organics (FILL) Loose Brown Dry to moist
218.2 / 1.8	CLAYEY SILT-SILT (CL-ML), trace sand Firm Brown Moist
217.7 / 2.3	SILTY SAND (SM) Compact to loose Brown to grey Moist to wet - 4.8 m: sample contains silt seams
213.5 / 6.5	SANDY SILT (ML), trace clay Compact to very dense Grey Moist

SAMPLES

NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
1	SS	6				
2	SS	6	0	80	16	4
3	SS	5				
4	SS	26	0	65	29	6
5	SS	7				
6	SS	8	0	47	50	3
7	SS	22	0	82	16	2
8A / 8B	SS	14				
9	SS	21				
10	SS	38				

ELEVATION SCALE: 219, 218, 217, 216, 215, 214, 213, 212, 211

Column headings: STRATA PLOT; GROUNDWATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH (kPa): ○ Unconfined, ● Quick Triaxial, ■ Pocket Pen, ⊕ Remoulded, X Field Vane; WATER CONTENT (%) PL NMC LL, W_p W W_l, NP Nonplastic (20 40 60); UNIT WEIGHT γ kN/m³; GR SA SI CL; REMARKS

Continued on Next Page

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

Ontario

wsp GOLDER

PROJECT 19136074	RECORD OF BOREHOLE No. HRE-3	Sheet 2 of 6	METRIC
G.W.P. Assignment No 2019-E-0048	LOCATION N 4888657.1; E 304531.7 NAD83 / MTM Zone 10 (LAT. 44.13791; LONG. -79.503353)		ORIGINATED BY DP
DIST Central HWY BBP	BOREHOLE TYPE 210 mm Hollow Stem Auger; Mud Rotary		COMPILED BY MA/ MTI
DATUM CGVD28 Surface Elevation:220.0 m	DATE Jan 13, 2022 - Jan 25, 2022		CHECKED BY KJB

ELEV. / DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES	GROUNDWATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH (kPa)	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR SA SI CL	REMARKS
	SANDY SILT (ML), trace clay Compact to very dense Grey Moist						210					
			11	SS	61		209					
							208					
			12	SS	33						0 21 71 8	
							207					
			13	SS	39		206					
							205					
			14	SS	64						0 19 76 5	
							204					
			15	SS	33		203					
202.1 17.8	CLAYEY SILT-SILT (CL-ML), some sand Hard Grey Wet - 18.3 to 18.9 m: lenses of clayey silt-silt						202					
			16	SS	32							
201.1 18.9	SILTY SAND (SM), trace clay Dense to compact Grey Wet						201					

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH (kPa): X Field Vane, ⊕ Remoulded, ■ Pocket Pen, ● Quick Triaxial, ○ Unconfined; WATER CONTENT (%): PL W_p, NMC W, LL W_l, NP Nonplastic, 20 40 60

Continued on Next Page

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

Ontario

WSP GOLDER

PROJECT 19136074	RECORD OF BOREHOLE No. HRE-3	Sheet 3 of 6	METRIC
G.W.P. Assignment No 2019-E-0048	LOCATION N 4888657.1; E 304531.7 NAD83 / MTM Zone 10 (LAT. 44.13791; LONG. -79.503353)	ORIGINATED BY	DP
DIST Central HWY BBP	BOREHOLE TYPE 210 mm Hollow Stem Auger; Mud Rotary	COMPILED BY	MA/ MTI
DATUM CGVD28 Surface Elevation:220.0 m	DATE Jan 13, 2022 - Jan 25, 2022	CHECKED BY	KJB

ELEV. --------- DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%) W	GR	SA	SI	CL
	SILTY SAND (SM), trace clay Dense to compact Grey Wet				200					
					199					
		17	SS	32		○	0	75	24	1
					198					
					197					
					196					
195.4 24.6	CLAYEY SILT (CL), trace to some sand Stiff to very stiff Grey Moist to wet	18 A 18 B	SS	21	195					
					194					
					193					
		19	SS	17	192					
					191					

Column headings also include: STRATA PLOT; GROUNDWATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH (kPa) — X Field Vane, ⊕ Remoulded, ■ Pocket Pen, ● Quick Triaxial, ○ Unconfined; WATER CONTENT (%) PL W_p, NMC W, LL W_l, NP Nonplastic (20 40 60); UNIT WEIGHT γ kN/m³; REMARKS.

Continued on Next Page

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

Ontario | WSP GOLDER

PROJECT 19136074 **RECORD OF BOREHOLE No. HRE-3** **METRIC**

G.W.P. Assignment No 2019-E-0048 LOCATION N 4888657.1; E 304531.7 NAD83 / MTM Zone 10 (LAT. 44.13791; LONG. -79.503353) ORIGINATED BY DP

DIST Central HWY BBP BOREHOLE TYPE 210 mm Hollow Stem Auger; Mud Rotary COMPILED BY MA/ MTI

DATUM CGVD28 Surface Elevation:220.0 m DATE Jan 13, 2022 - Jan 25, 2022 CHECKED BY KJB

ELEV. / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	GR	SA	SI	CL	REMARKS
	CLAYEY SILT (CL), trace to some sand Stiff to very stiff Grey Moist to wet				190					
		20	SS	26	189					
					188					
					187					
		21	SS	10	186					
	- 35.0 m: attempted to obtain shelby tube sample but no recovery				185					
					184					
		22	SS	11	183	0	0	67	33	
					182					
		23	TO		181					

Column headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH (kPa): ○ Unconfined, ● Quick Triaxial, ■ Pocket Pen, ⊕ Remoulded, X Field Vane; WATER CONTENT (%): PL W_p, NMC W, LL W_l (20 40 60), NP Nonplastic; GROUNDWATER CONDITIONS; STRATA PLOT; UNIT WEIGHT γ kN/m³

Continued on Next Page

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

Ontario | WSP GOLDER

PROJECT 19136074	RECORD OF BOREHOLE No. HRE-3	Sheet 5 of 6	METRIC
G.W.P. Assignment No 2019-E-0048	LOCATION N 4888657.1; E 304531.7 NAD83 / MTM Zone 10 (LAT. 44.13791; LONG. -79.503353)		ORIGINATED BY DP
DIST Central HWY BBP	BOREHOLE TYPE 210 mm Hollow Stem Auger; Mud Rotary		COMPILED BY MA/ MTI
DATUM CGVD28 Surface Elevation:220.0 m	DATE Jan 13, 2022 - Jan 25, 2022		CHECKED BY KJB

SOIL PROFILE			SAMPLES			GROUNDWATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH (kPa) ○ Unconfined ● Quick Triaxial ■ Pocket Pen ⊕ Remoulded × Field Vane 20 40 60 80 100	WATER CONTENT (%) PL W_p NMC W LL W_l NP Nonplastic 20 40 60	UNIT WEIGHT γ kN/m³	GR SA SI CL	REMARKS
ELEV. --------- DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES							
	CLAYEY SILT (CL), trace to some sand Stiff to very stiff Grey Moist to wet											
			24	SS	3		180					
								⊕ ×				
							179					
							178					
			25	SS	14		177					
							176					
							175					
			26	SS	14		174					
							173					
							172					

Continued on Next Page

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

Ontario

WSP GOLDER

RECORD OF BOREHOLE No. HRE-3

METRIC

PROJECT	19136074				
G.W.P.	Assignment No 2019-E-0048	LOCATION	N 4888657.1; E 304531.7 NAD83 / MTM Zone 10 (LAT. 44.13791; LONG. -79.503353)	ORIGINATED BY	DP
DIST	Central HWY BBP	BOREHOLE TYPE	210 mm Hollow Stem Auger; Mud Rotary	COMPILED BY	MA/ MTI
DATUM	CGVD28 Surface Elevation:220.0 m	DATE	Jan 13, 2022 - Jan 25, 2022	CHECKED BY	KJB

ELEV. / DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES	GROUNDWATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH (kPa) (20 40 60 80 100)	WATER CONTENT (%) W_p W W_l (20 40 60)	UNIT WEIGHT γ kN/m³	GR SA SI CL	REMARKS
	CLAYEY SILT (CL), trace to some sand Stiff to very stiff Grey Moist to wet		27	SS	18		170					
169.7 50.3	CLAYEY SILT (CL), some sand to sandy, trace to some gravel Hard Grey Wet						169					
52.2			28	SS	166/0.28		168					
167.7	End of Borehole Notes: 1. Groundwater level was measured at 1.5 m (El. 218.5 m) inside hollow stem auger during drilling. 2. Groundwater level was measured at 1.0 m (El. 219.0 m) inside the monitoring well on May 13, 2022.						167 166 165 164 163 162					

Shear strength legend: X Field Vane; ⊕ Remoulded; ■ Pocket Pen; ● Quick Triaxial; ○ Unconfined

NP Nonplastic

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

Ontario

WSP GOLDER

RECORD OF BOREHOLE No. HRE-4

METRIC

PROJECT	19136074		
G.W.P.	Assignment No 2019-E-0048	LOCATION	N 4888345.7; E 304339.4 NAD83 / MTM Zone 10 (LAT. 44.135107; LONG. -79.505756)
DIST	Central HWY BBP	BOREHOLE TYPE	210 mm Hollow Stem Auger; Mud Rotary
DATUM	CGVD28 Surface Elevation:219.9 m	DATE	Feb 22, 2022 - Mar 01, 2022
ORIGINATED BY	MTI		
COMPILED BY	MA/ MTI		
CHECKED BY	KJB		

SOIL PROFILE; SAMPLES; GROUNDWATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH (kPa): ○ Unconfined, ● Quick Triaxial, ■ Pocket Pen, ⊕ Remoulded, X Field Vane (20 40 60 80 100); WATER CONTENT (%): PL W_p, NMC W, LL W_l (20 40 60), NP Nonplastic; UNIT WEIGHT γ kN/m³; GR SA SI CL; REMARKS

ELEV. / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL	REMARKS
0.0	SILTY SAND (SM), trace gravel, trace organics, rootlets, (FILL) Compact Grey to brown Moist	1	SS	10					Artesian groundwater levels encountered - see notes.
219.2 / 0.7	CLAYEY SILT (CL), trace sand, trace gravel Soft to firm Mottled brown / grey Moist	2	SS	4					
218.5 / 1.4	SILTY SAND (SM), Compact Brown Wet	3	SS	18					
217.7 / 2.2	SILT (ML), some clay, trace sand, silty sand seams Loose to compact Brown Moist to wet - contains clayey silt pockets	4	SS	15					
		5	SS	7	0	5	78	17	
216.2 / 3.7	SILTY SAND (SM), trace gravel, slight plasticity Compact Brown Moist to wet - 3.8 to 4.9 m: - contains clayey silt pockets	6	SS	14					
		7	SS	26	0	36	(64)		
214.3 / 5.6	CLAYEY SILT (CL), trace sand Hard Brown Moist - contains clayey silt pockets	8A	SS	34	0	5	73	22	
213.4 / 6.4	Sandy SILT (ML) Dense Brown Moist	8B							
212.7 / 7.2	CLAYEY SILT (CL), trace sand Very Stiff Brown Moist	9	SS	25					
211.2 / 8.7	SILTY SAND (SM) to SAND (SP), Very dense Grey Moist to wet	10	SS	79					

Elevation scale: 219, 218, 217, 216, 215, 214, 213, 212, 211

Continued on Next Page

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

Ontario | WSP GOLDER

PROJECT	19136074	RECORD OF BOREHOLE No. HRE-4	Sheet 2 of 6	METRIC	
G.W.P.	Assignment No 2019-E-0048	LOCATION	N 4888345.7; E 304339.4 NAD83 / MTM Zone 10 (LAT. 44.135107; LONG. -79.505756)	ORIGINATED BY	MTI
DIST	Central HWY BBP	BOREHOLE TYPE	210 mm Hollow Stem Auger; Mud Rotary	COMPILED BY	MA/ MTI
DATUM	CGVD28 Surface Elevation:219.9 m	DATE	Feb 22, 2022 - Mar 01, 2022	CHECKED BY	KJB

Column headings: SOIL PROFILE (ELEV. / DEPTH; DESCRIPTION; STRATA PLOT); SAMPLES (NUMBER; TYPE; "N" VALUES); GROUNDWATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH (kPa): ○ Unconfined, ● Quick Triaxial, ■ Pocket Pen, ⊕ Remoulded, X Field Vane; WATER CONTENT (%) PL W_p, NMC W, LL W_l (20 40 60), NP Nonplastic; UNIT WEIGHT γ kN/m³; GR SA SI CL; REMARKS

ELEV. / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT	GR	SA	SI	CL
	SILTY SAND (SM) to SAND (SP), Very dense, Grey, Moist to wet				210					
209.0 / 10.9	CLAYEY SILT-SILT (CL-ML), trace sand, Very Stiff, Grey, Moist	11 A	SS	25	209	○	0	82	(18)	
208.7 / 11.2	Sandy SILT (ML) trace clay, Dense to very dense, Grey, Moist to wet	11 B				├○	0	4	76	20
					208					
	- 12.2 m: - Artesian groundwater conditions encountered at depth of 12.2 m below ground surface. Groundwater observed to rise and flow out top of casing at 1.5 m above ground surface (El. 221.4 m).	12	SS	43						
					207					
		13	SS	57	206	○ NP	0	18	74	8
					205					
		14	SS	66	204					
		15	SS	95	203	○ NP	0	20	73	7
					202					
	- 18.3 m: - sand layer (50 mm thick)	16	SS	51	201					

Continued on Next Page

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

Ontario | wsp GOLDER

PROJECT 19136074	**RECORD OF BOREHOLE No. HRE-4**	Sheet 3 of 6	**METRIC**
G.W.P. Assignment No 2019-E-0048	LOCATION N 4888345.7; E 304339.4 NAD83 / MTM Zone 10 (LAT. 44.135107; LONG. -79.505756)		ORIGINATED BY MTI
DIST Central HWY BBP	BOREHOLE TYPE 210 mm Hollow Stem Auger; Mud Rotary		COMPILED BY MA/ MTI
DATUM CGVD28 Surface Elevation:219.9 m	DATE Feb 22, 2022 - Mar 01, 2022		CHECKED BY KJB

ELEV. / DEPTH	DESCRIPTION (SOIL PROFILE)	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI CL	REMARKS
	Sandy SILT (ML) trace clay Dense to very dense Grey Moist to wet				200					
199.8 / 20.1	Sandy SILT (ML) Compact to dense Grey Moist to wet				199					
		17	SS	25	198	o NP	0	25	(75)	
					197					
					196					
	- 24.3 m: Artesian groundwater flow out of top of casing increased from 12.4 m depth to 24.3 m depth below ground surface. The water level inside casing was measured to be 2.4 m above ground surface (El. 222.3 m) at 24.3 m depth.	18	SS	46	195					
					194					
193.7 / 26.2	SILTY SAND (SM), trace clay Very dense Grey Moist				193					
		19	SS	55	192	o	0	70	(30)	
					191					
190.6 / 29.3	CLAYEY SILT -SILT (CL-ML) to CLAYEY SILT (CL), trace sand, contains sand seams Stiff to very stiff Grey Moist to wet									

Continued on Next Page

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

Ontario

WSP GOLDER

PROJECT 19136074 **RECORD OF BOREHOLE No. HRE-4** Sheet 4 of 6 **METRIC**

G.W.P. Assignment No 2019-E-0048 LOCATION N 4888345.7; E 304339.4 NAD83 / MTM Zone 10 (LAT. 44.135107; LONG. -79.505756) ORIGINATED BY MTI

DIST Central HWY BBP BOREHOLE TYPE 210 mm Hollow Stem Auger; Mud Rotary COMPILED BY MA/ MTI

DATUM CGVD28 Surface Elevation:219.9 m DATE Feb 22, 2022 - Mar 01, 2022 CHECKED BY KJB

ELEV. / DEPTH	SOIL PROFILE: DESCRIPTION	STRATA PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUNDWATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH (kPa) (○ Unconfined, ● Quick Triaxial, ■ Pocket Pen, ⊕ Remoulded, X Field Vane) 20 40 60 80 100	WATER CONTENT (%) PL W_p, NMC W, LL W_l; NP Nonplastic 20 40 60	UNIT WEIGHT γ kN/m³	GR SA SI CL	REMARKS
	CLAYEY SILT -SILT (CL-ML) to CLAYEY SILT (CL), trace sand, contains sand seams Stiff to very stiff Grey Moist to wet						190					
			20	SS	15		189					
							188					
							187					
			21	SS	17		186		H-O		0 1 68 31	
							185					
							184					
			22	SS	12		183					
							182					
181.5 38.4	CLAY (CH), some silt, trace sand Stiff Grey Moist						181					

Continued on Next Page

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

Ontario | WSP GOLDER

PROJECT 19136074 **RECORD OF BOREHOLE No. HRE-4** Sheet 5 of 6 **METRIC**

G.W.P. Assignment No 2019-E-0048 LOCATION N 4888345.7; E 304339.4 NAD83 / MTM Zone 10 (LAT. 44.135107; LONG. -79.505756) ORIGINATED BY MTI

DIST Central HWY BBP BOREHOLE TYPE 210 mm Hollow Stem Auger; Mud Rotary COMPILED BY MA/ MTI

DATUM CGVD28 Surface Elevation:219.9 m DATE Feb 22, 2022 - Mar 01, 2022 CHECKED BY KJB

ELEV. / DEPTH	SOIL PROFILE – DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUNDWATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL	REMARKS
	CLAY (CH), some silt, trace sand Stiff Grey Moist												
		23	SS	17		180	W_p ≈ 20, w ≈ 40, W_L ≈ 52		0	0	25	75	
						179							
178.5 41.4	CLAYEY SILT-SILT (CL-ML) to CLAYEY SILT (CL), trace sand, contains sand seams / layers Stiff to very stiff Grey Moist to wet					178							
		24	SS	14		177							
						176							
						175							
	- 45.7 m: -sand layer (25 mm thick)	25	SS	21		174	w ≈ 20						
						173							
						172							
						171							

Chart headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT; SHEAR STRENGTH (kPa): × Field Vane, ⊕ Remoulded, ■ Pocket Pen, ● Quick Triaxial, ○ Unconfined; scale 20 40 60 80 100. WATER CONTENT (%): PL W_p, NMC W, LL W_l; NP Nonplastic; scale 20 40 60.

Continued on Next Page

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

Ontario | WSP GOLDER

PROJECT 19136074		RECORD OF BOREHOLE No. HRE-4	Sheet 6 of 6		METRIC
G.W.P. Assignment No 2019-E-0048	LOCATION	N 4888345.7; E 304339.4 NAD83 / MTM Zone 10 (LAT. 44.135107; LONG. -79.505756)		ORIGINATED BY	MTI
DIST Central HWY BBP	BOREHOLE TYPE	210 mm Hollow Stem Auger; Mud Rotary		COMPILED BY	MA/ MTI
DATUM CGVD28 Surface Elevation:219.9 m	DATE	Feb 22, 2022 - Mar 01, 2022		CHECKED BY	KJB

ELEV. / DEPTH	DESCRIPTION (SOIL PROFILE)	STRATA PLOT	NUMBER	TYPE	"N" VALUES	GROUNDWATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH (kPa) (20 40 60 80 100)	WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	GR SA SI CL	REMARKS
	CLAYEY SILT-SILT (CL-ML) to CLAYEY SILT (CL), trace sand, contains sand seams / layers Stiff to very stiff Grey Moist to wet		26	SS	18							
							170					
							169					
							168					
	- 52.0 m: -sand layer (25 mm thick)		27	SS	22							
167.5 52.4	End of Borehole											
							167					
							166					
							165					
							164					
							163					
							162					

Legend (shear strength): × Field Vane; ⊕ Remoulded; ■ Pocket Pen; ● Quick Triaxial; ○ Unconfined

Water content: PL W_p — NMC W — LL W_l; NP Nonplastic

Notes: 1. Groundwater first encountered at a depth of 0.7 m below ground surface (El. 219.2 m) inside hollow stem augers.
2. Artesian groundwater conditions (up to 2.4 m above ground surface, El. 222.3 m) encountered from 12.2 m (El. 207.7 m) to the borehole termination depth.
3. Borehole caved to a depth of 14.0 m (El. 205.9 m) upon completion of drilling on February 28.
4. Water level measured at a depth of 3.3 m (El. 216.6 m) and borehole caved to 3.9 m depth (El. 216 m) on March 1, 2022 prior to borehole backfilling.

+³, x³ : Numbers refer to Sensitivity o³% 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~~[1] 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 3rd 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.

[1] For clarify, 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^2)	Sleeve Area (cm^2)	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	• Standard plots with expanded range • Advanced plots with Ic, Su, phi and N1(60) • Seismic plots with Vs • Soil Behaviour Type (SBT) scatter plots

Calculated Geotechnical Parameter Tables	
Additional information	The Normalized Soil Behaviour Type Chart based on Q_{tn} (SBT Q_{tn}) (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 (q_t) sleeve friction (f_s) and pore pressure (u_2). Soils were classified as either drained or undrained based on the Q_{tn} 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).

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^2 and 15 cm^2 tip base area configurations in order to maximize signal resolution for various soil conditions. The specific piezocone used for each test is described in the CPT summary table presented in the first appendix. The 15 cm^2 penetrometers do not require friction reducers as they have a diameter larger than the deployment rods. The 10 cm^2 piezocones use a friction reducer consisting of a rod adapter extension behind the main cone body with an enlarged cross sectional area (typically 44 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_2" 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.

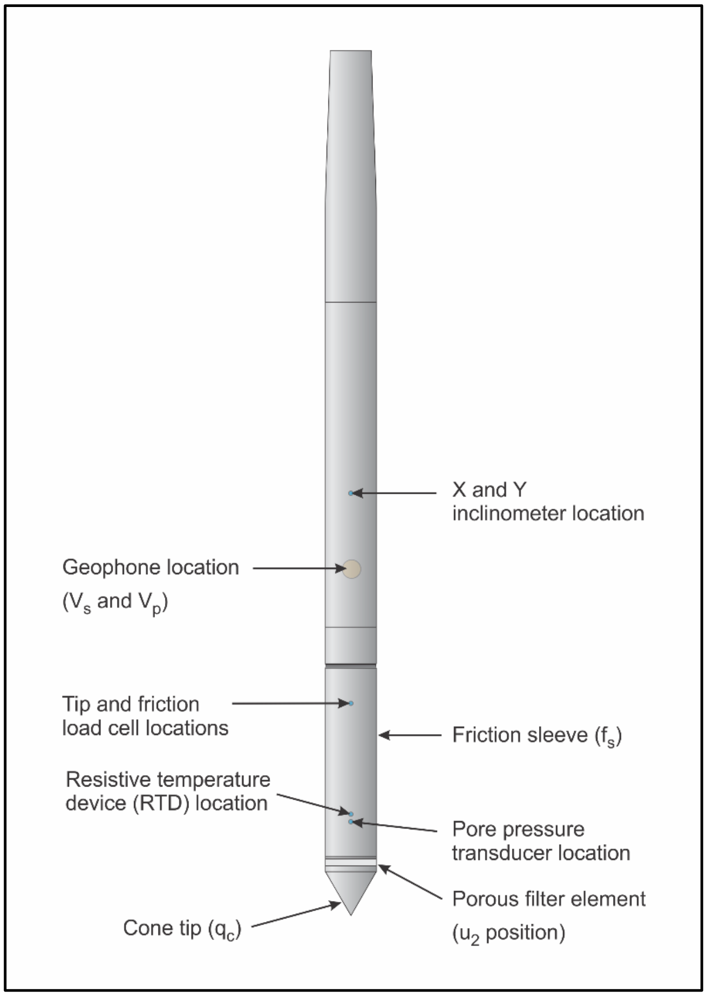


Figure CPTu. Piezocone Penetrometer (15 cm^2)

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):

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

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

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

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

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

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

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

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).

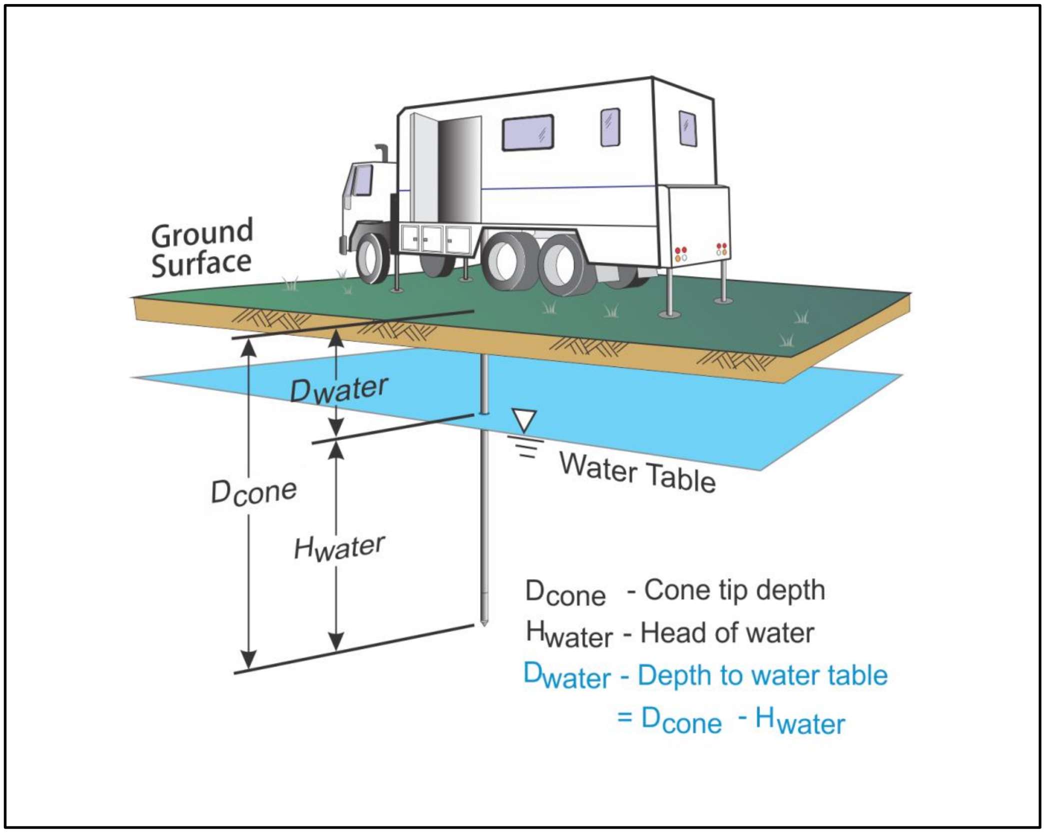


Figure PPD-1. Pore pressure dissipation test setup

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.

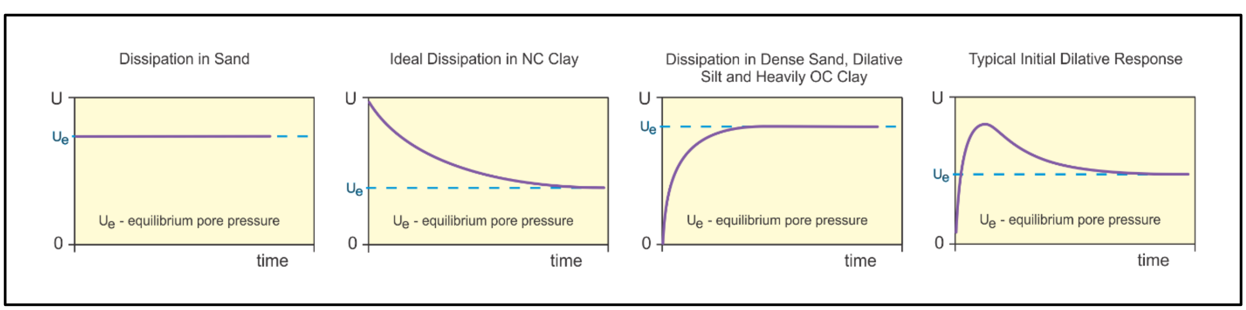


Figure PPD-2. Pore pressure dissipation curve examples

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 dilatory response in which an initial rise in pore pressure occurs before reaching a peak, the relative time from the peak value is used in determining t_{50}. In cases where the time to peak is excessive, t_{50} values are not calculated.

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

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

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

References

Burns, S.E. and Mayne, P.W., 1998, “Monotonic and dilatory pore pressure decay during piezocone tests”, Canadian Geotechnical Journal 26 (4): 1063-1073. DOI: 1063-1073/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, 10th 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.

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.

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.

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

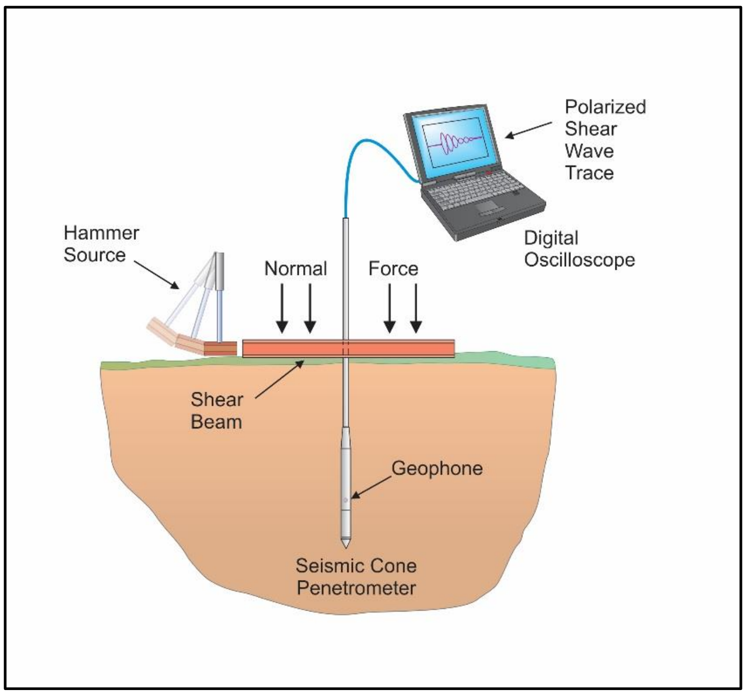


Figure SCPTu-1. Illustration of the SCPTu system

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).

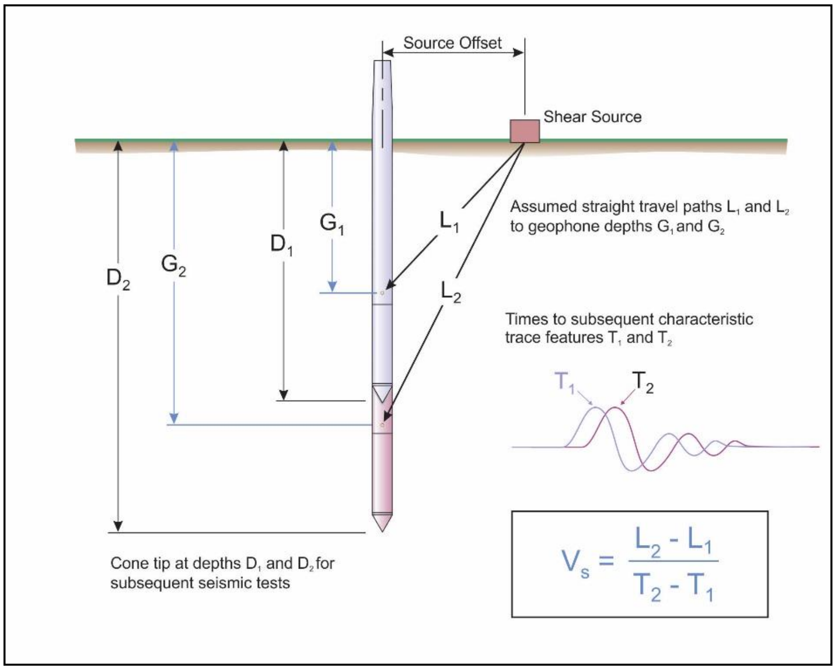


Figure SCPTu-2. Illustration of a seismic cone penetration test

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).

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

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.

ASTM D7400/D7400M-19, 2019, "Standard Test Methods for Downhole Seismic Testing", ASTM International, West Conshohocken, PA. DOI: 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).

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 Ic, Su(Nkt), Phi and N1(60)Ic
- Seismic Cone Penetration Test Plots
- Seismic Cone Penetration Test Shear Wave (Vs) Tabular Results
- Seismic Cone Penetration Test Shear Wave (Vs) 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

ConeTec

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^2)	Assumed Phreatic Surface[1] (m)	Final Depth (m)	Northing[2] (m)	Easting[2] (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	

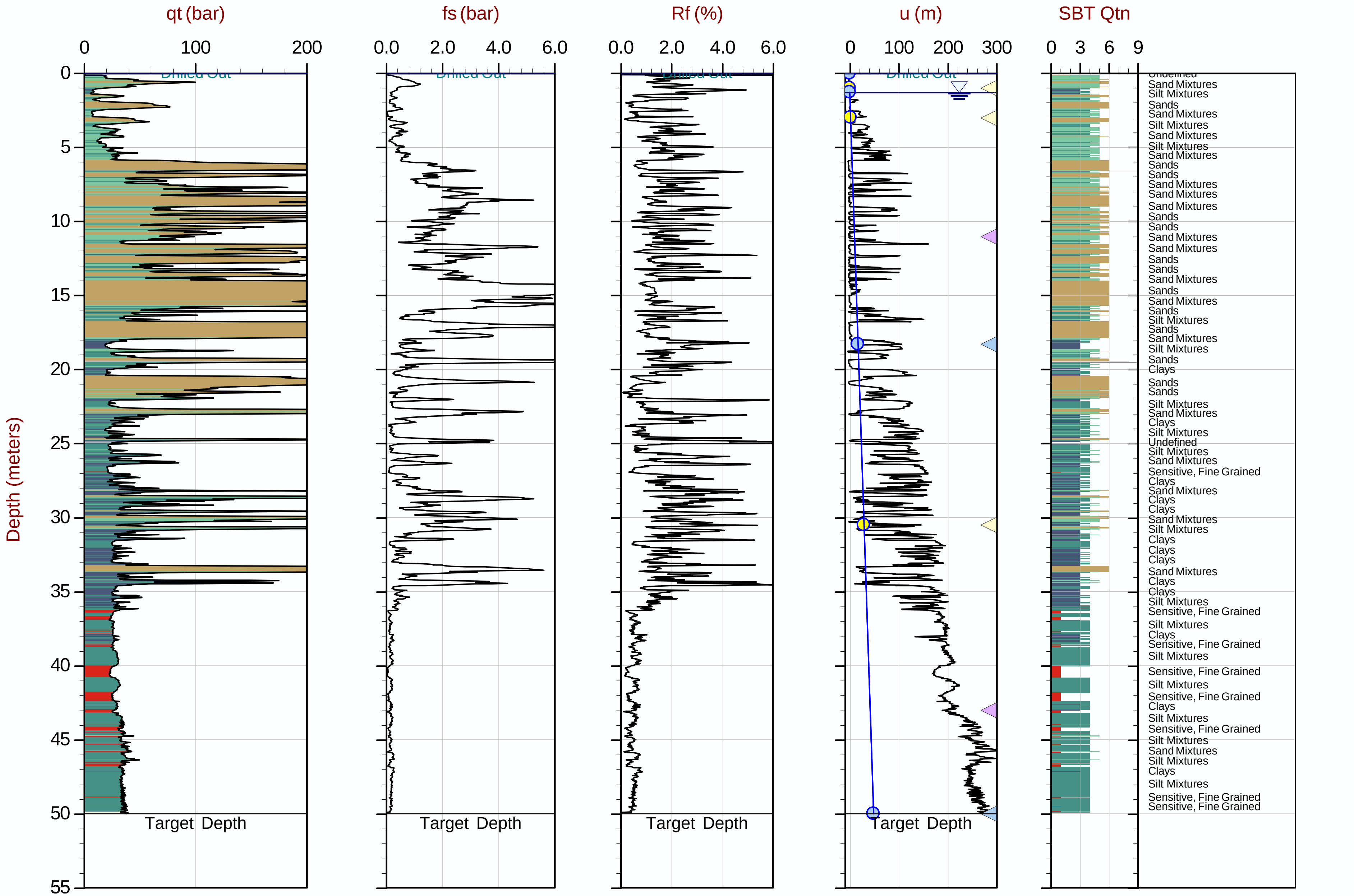
1. The assumed phreatic surface was based on pore pressure dissipation tests. Hydrostatic conditions were assumed for the calculated parameters.
2. Coordinates were collected with a consumer grade GPS device, datum: NAD83 / UTM Zone 17N.

CONETEC

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: UTM 17N 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

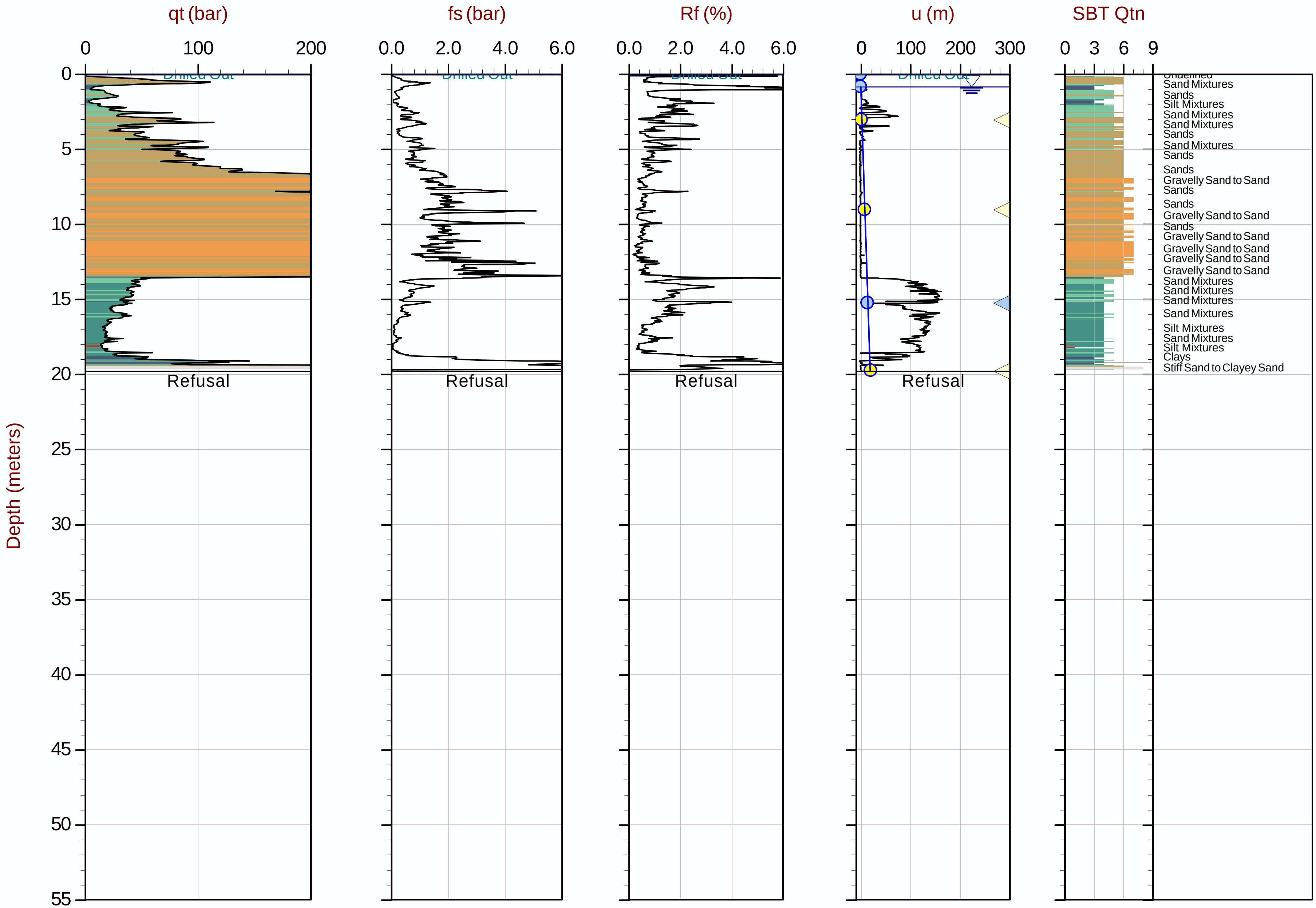
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

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

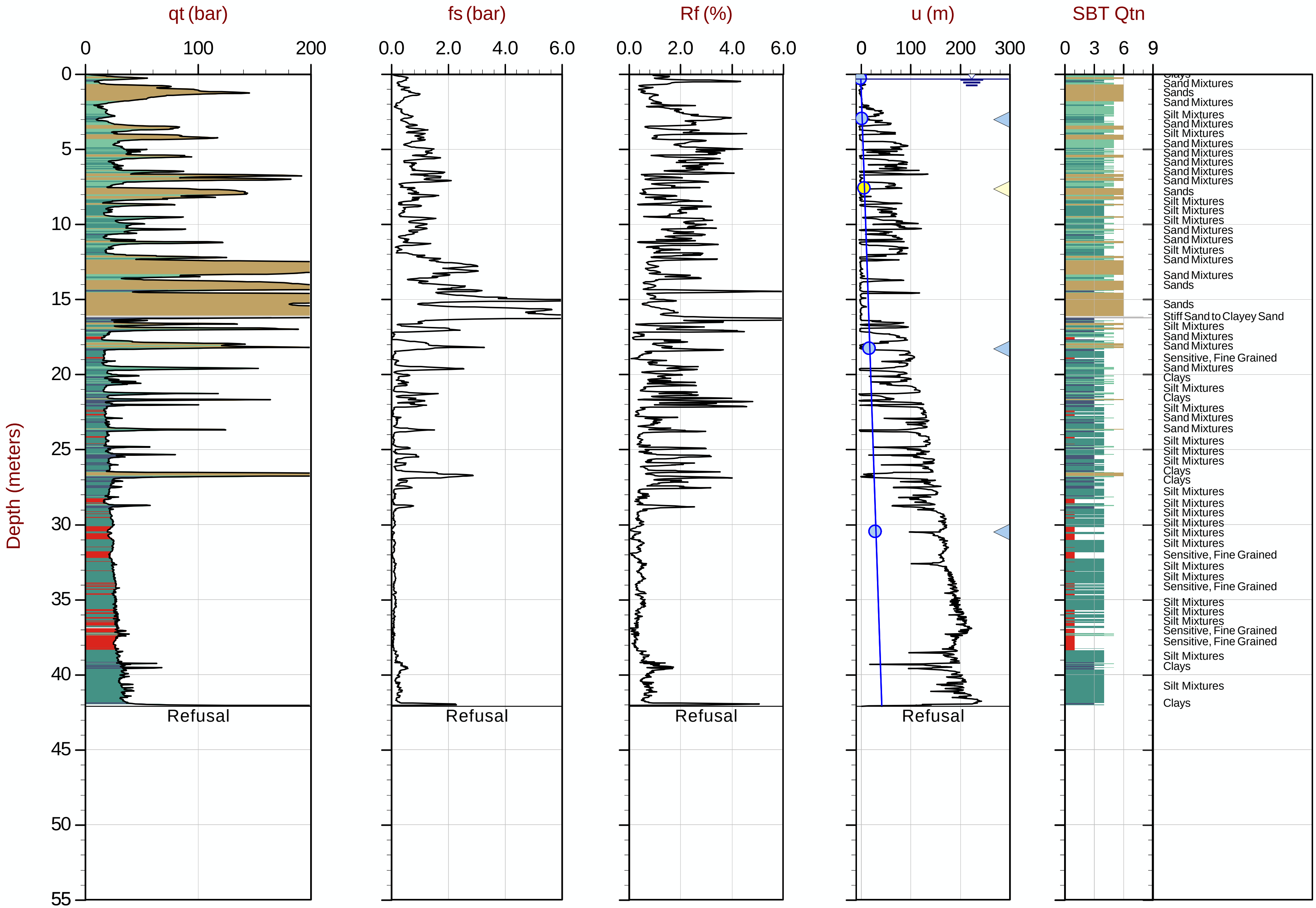
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

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.

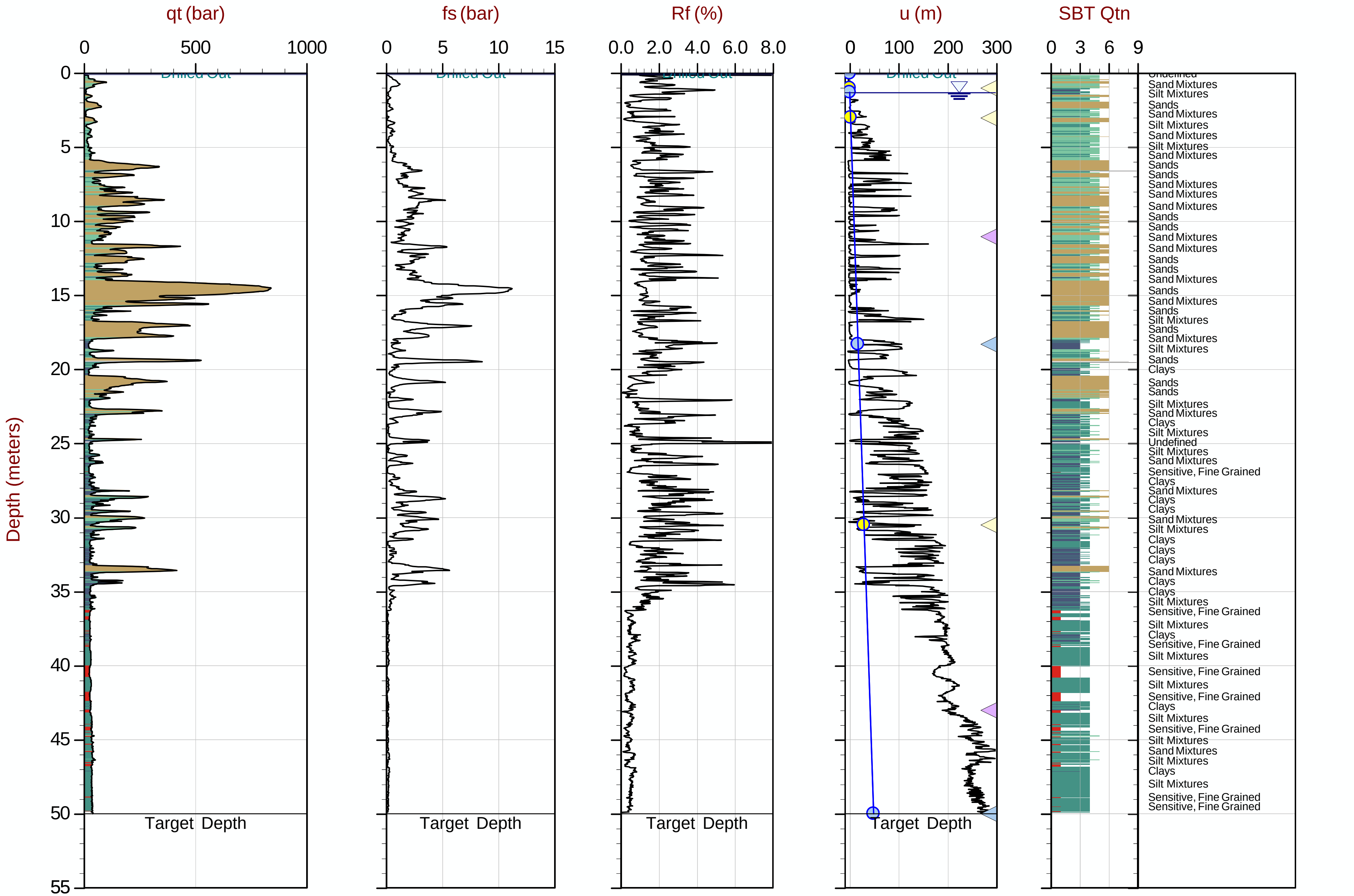
Standard Cone Penetration Test Plots with Expanded Range

CONETEC

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: UTM 17N 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

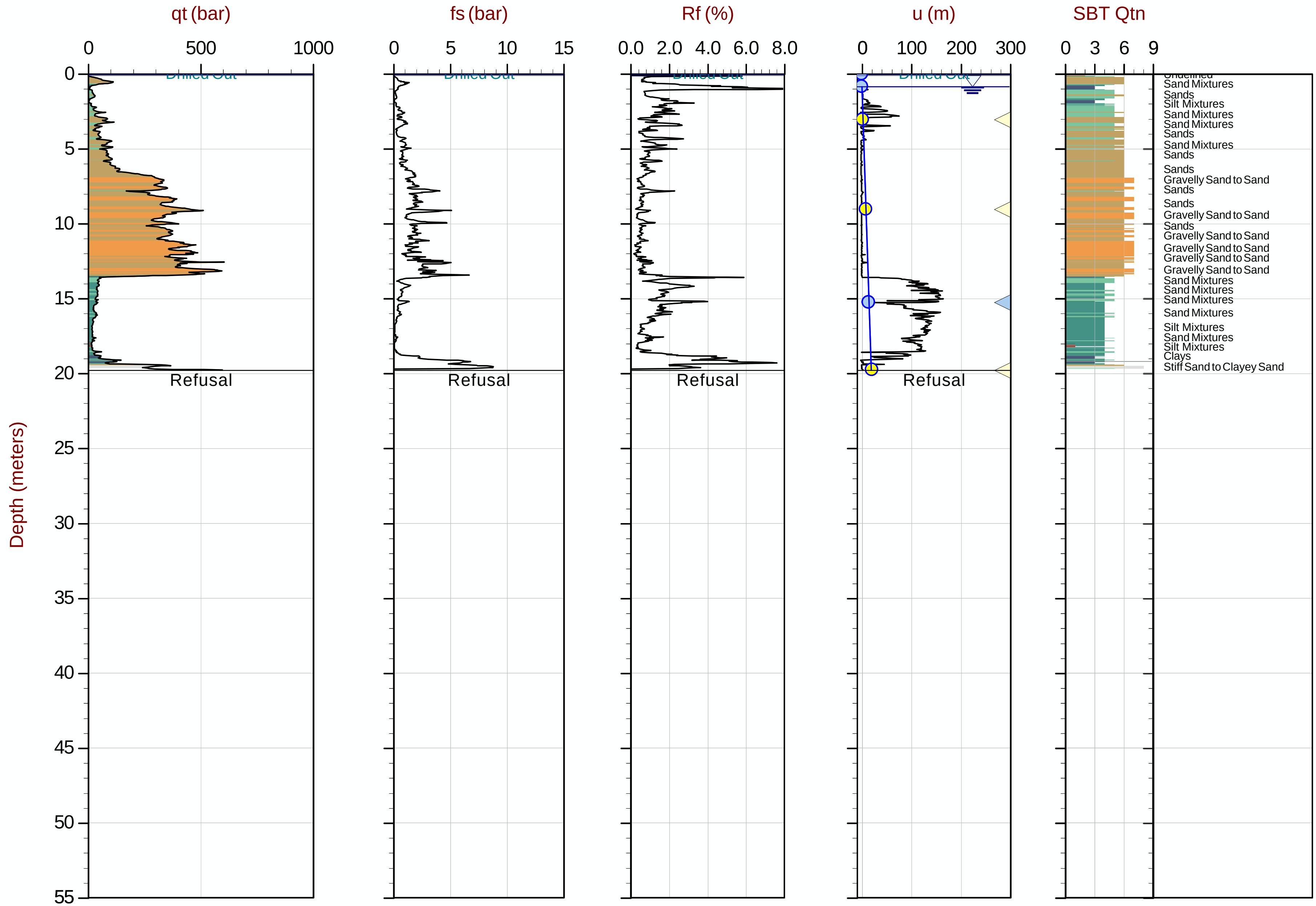
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

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.

CONETEC

Golder

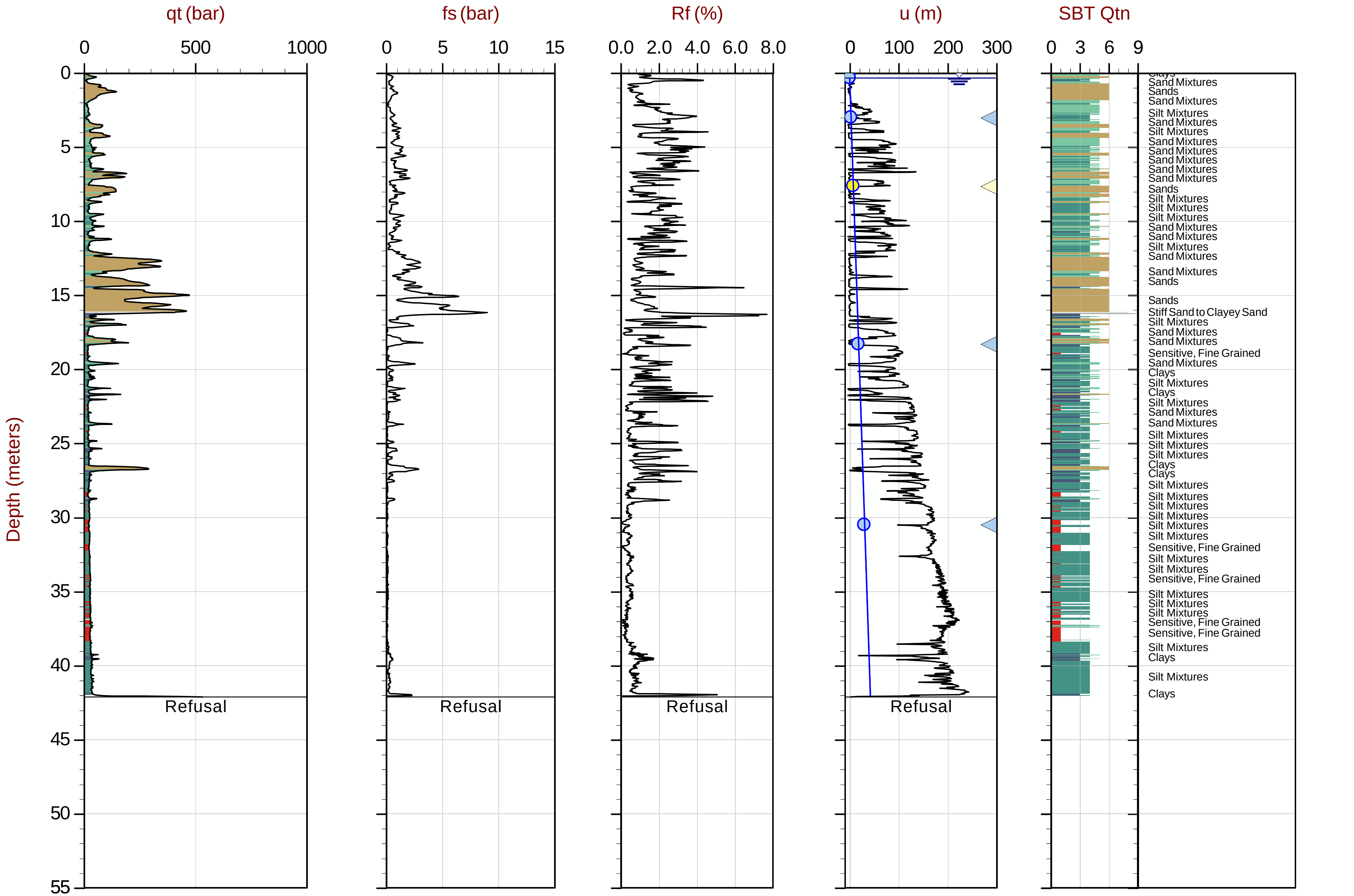
Job No: 21-05-23424

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.

Advanced Cone Penetration Plots with Ic, Su(Nkt), Phi and N1(60)Ic

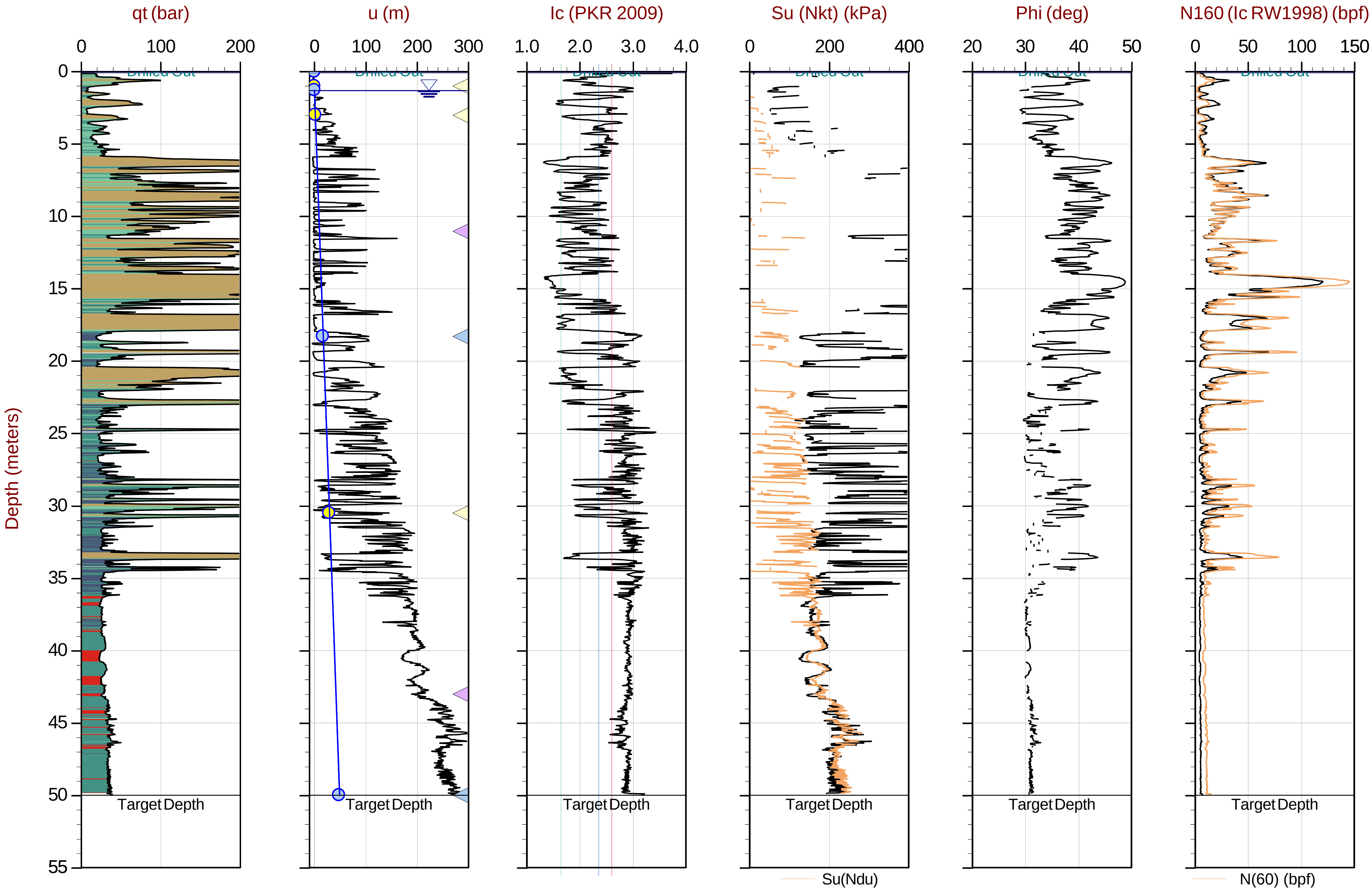
CONETEC

CONETEC

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)
Su Nkt/Ndu: 12.0 / 9.0

SBT: Robertson, 2009 and 2010
Coords: UTM 17N 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

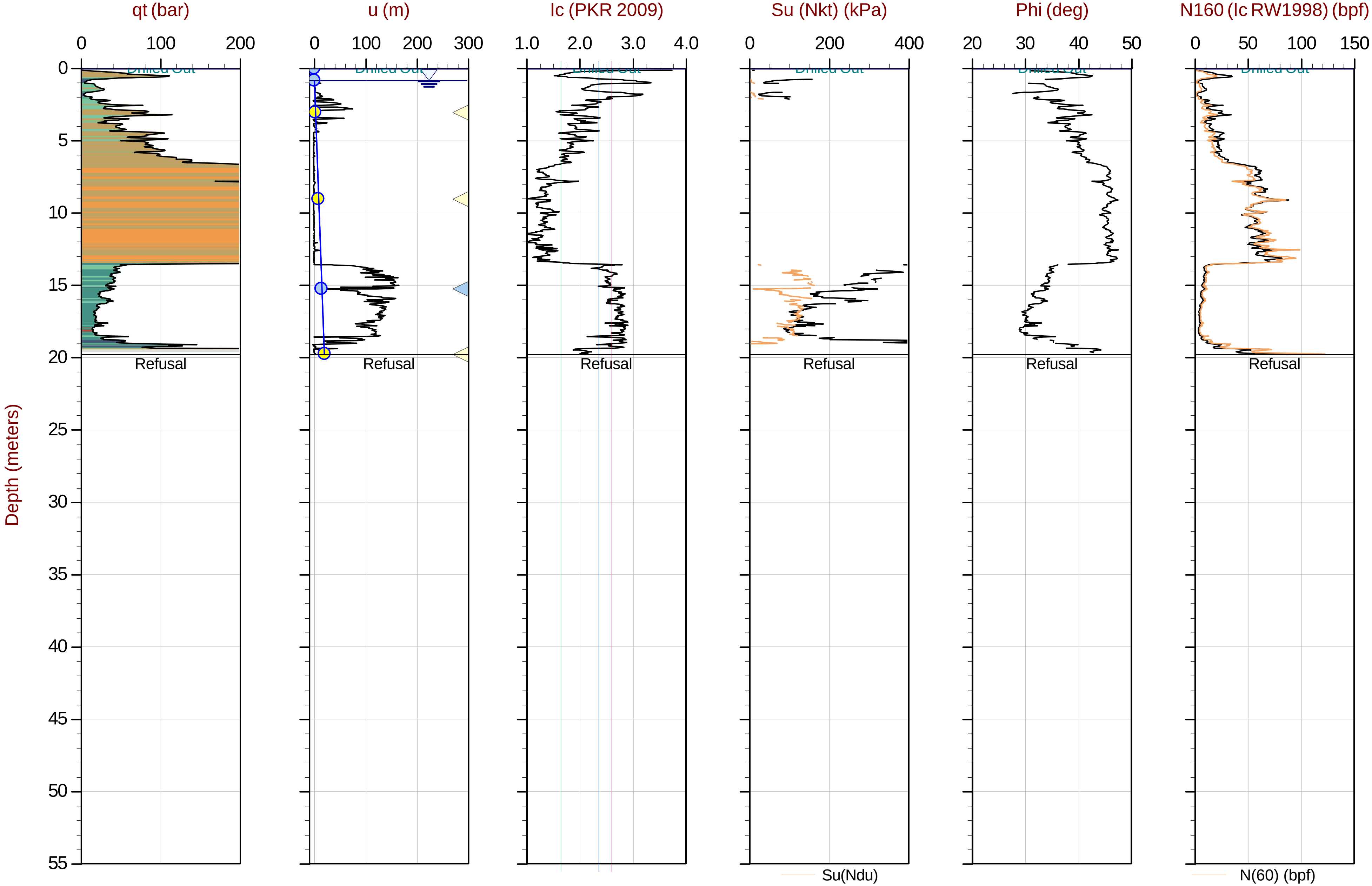
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

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)
Su Nkt/Ndu: 12.0 / 9.0

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

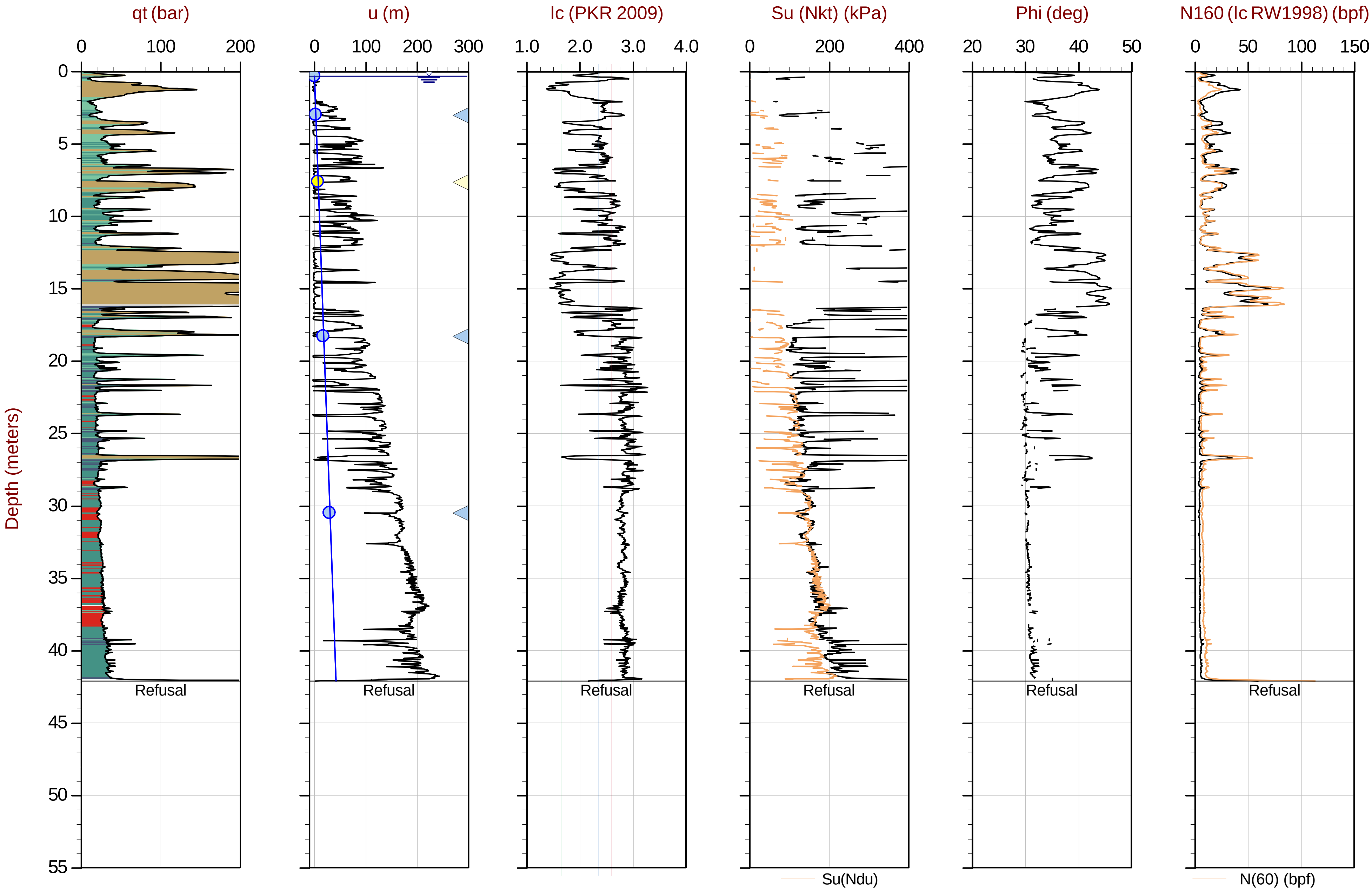
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

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)
Su Nkt/Ndu: 12.0 / 9.0

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.

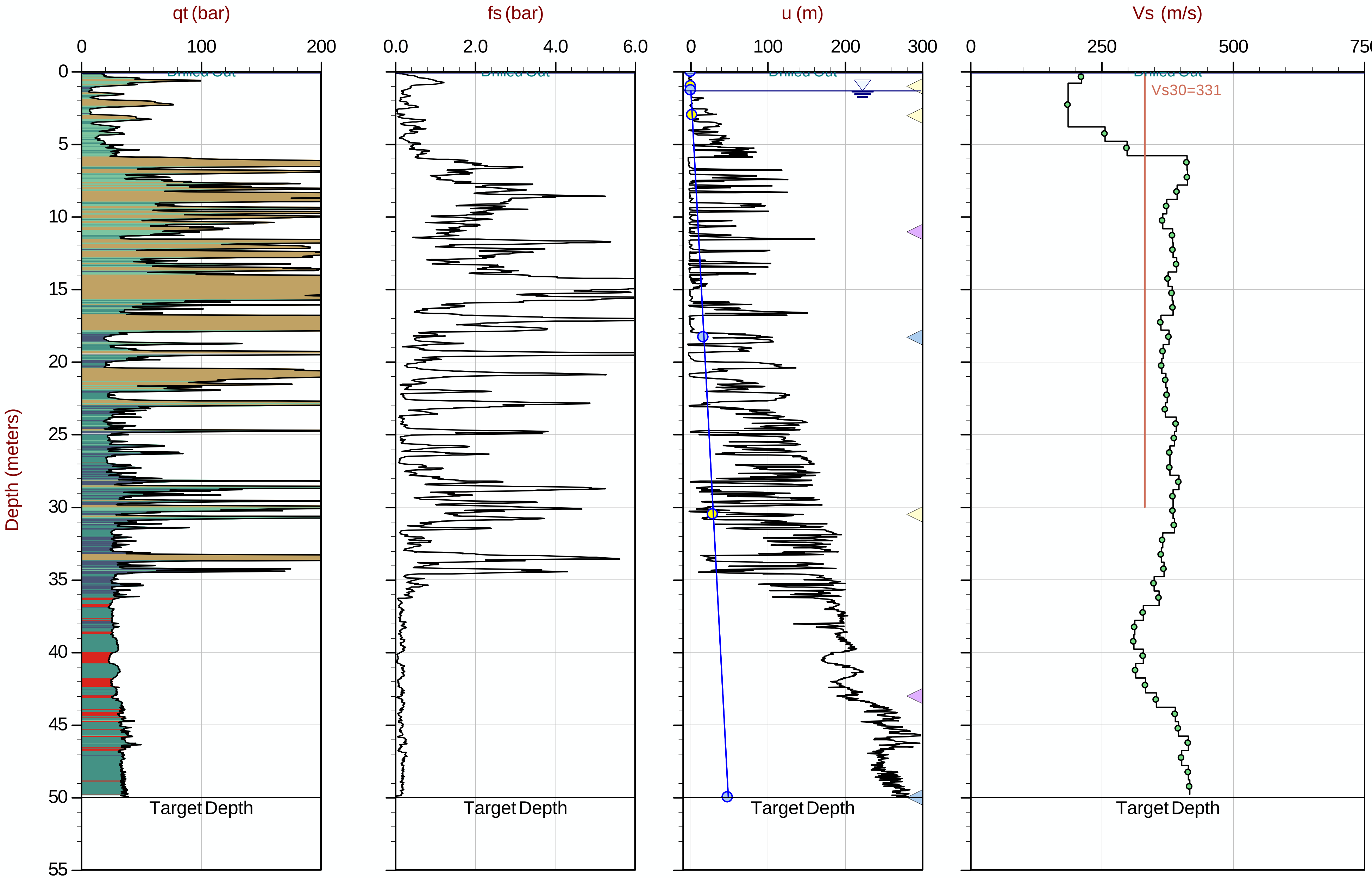
Seismic Cone Penetration Test Plots

CONETEC

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: UTM 17N N: 4888079m E: 618839m
Sheet No: 1 of 1

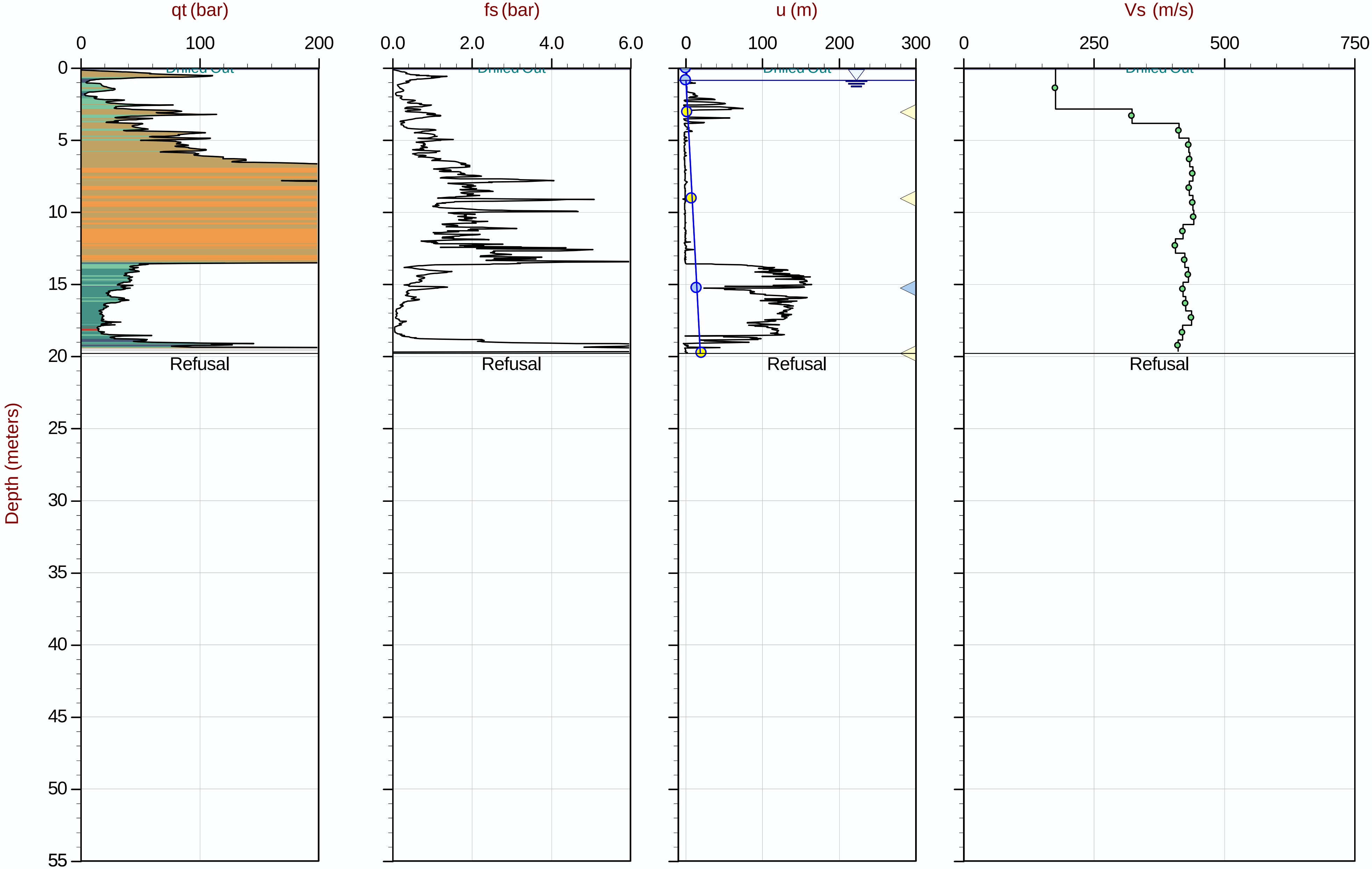
Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

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

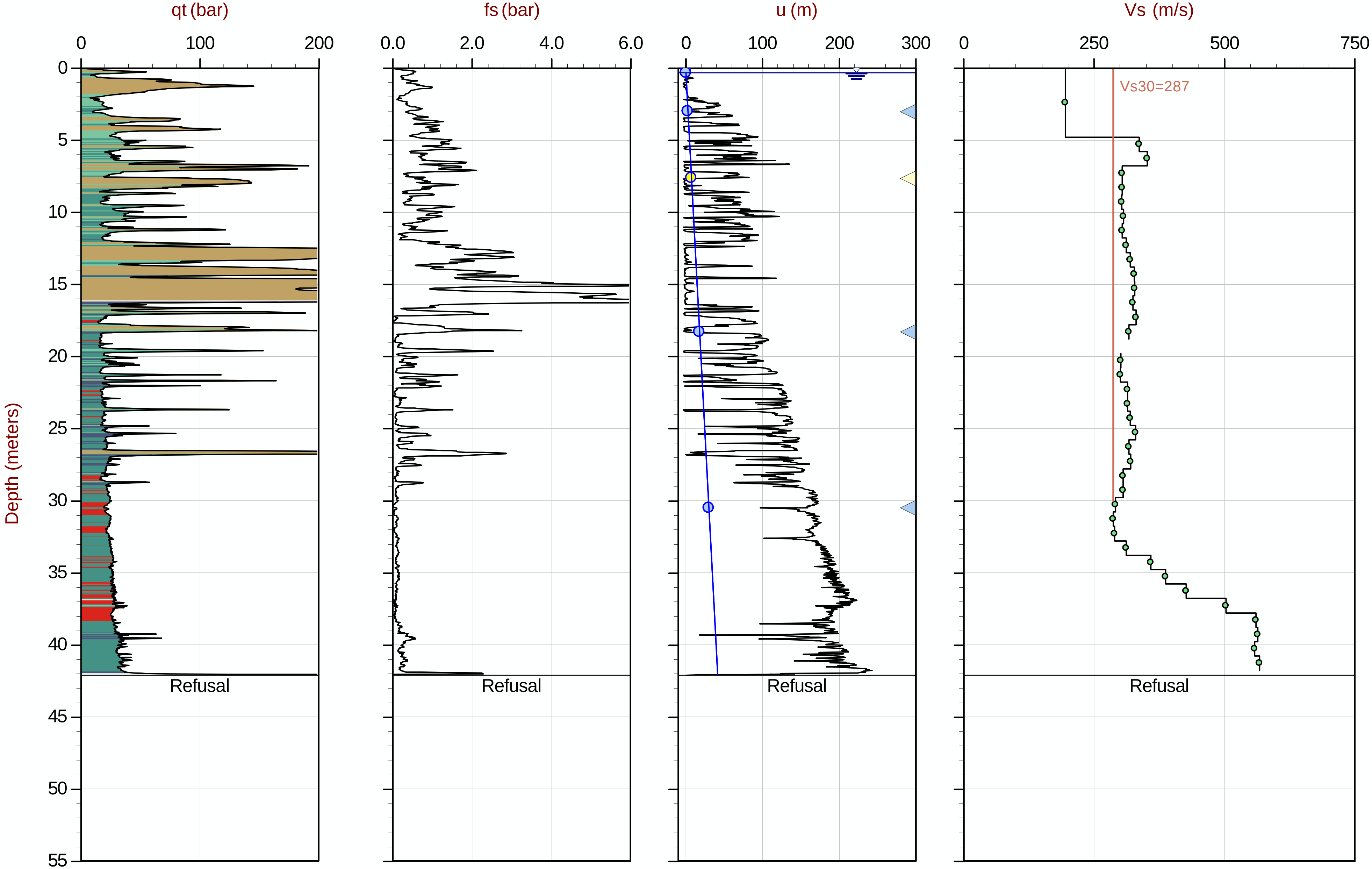
Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

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: 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.

Seismic Cone Penetration Test Shear Wave (Vs) 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

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)
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

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)
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

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)
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

SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

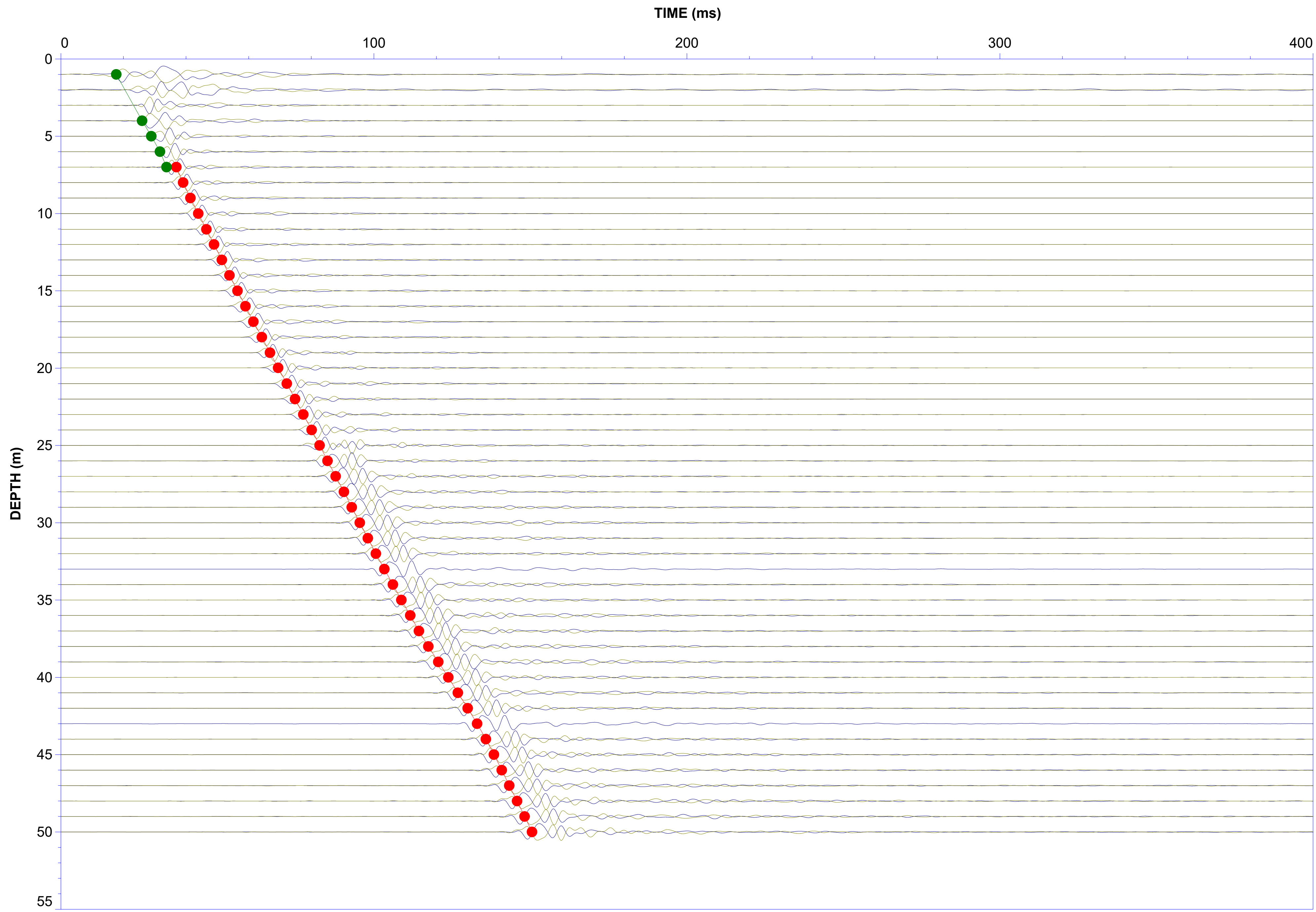
Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
36.00	35.80	35.99	1.00	2.57	387
37.00	36.80	36.98	1.00	2.33	427
38.00	37.80	37.98	1.00	1.98	504
39.00	38.80	38.97	1.00	1.78	561
40.00	39.80	39.97	1.00	1.77	564
41.00	40.80	40.96	1.00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

Seismic Cone Penetration Test Shear Wave (Vs) Traces

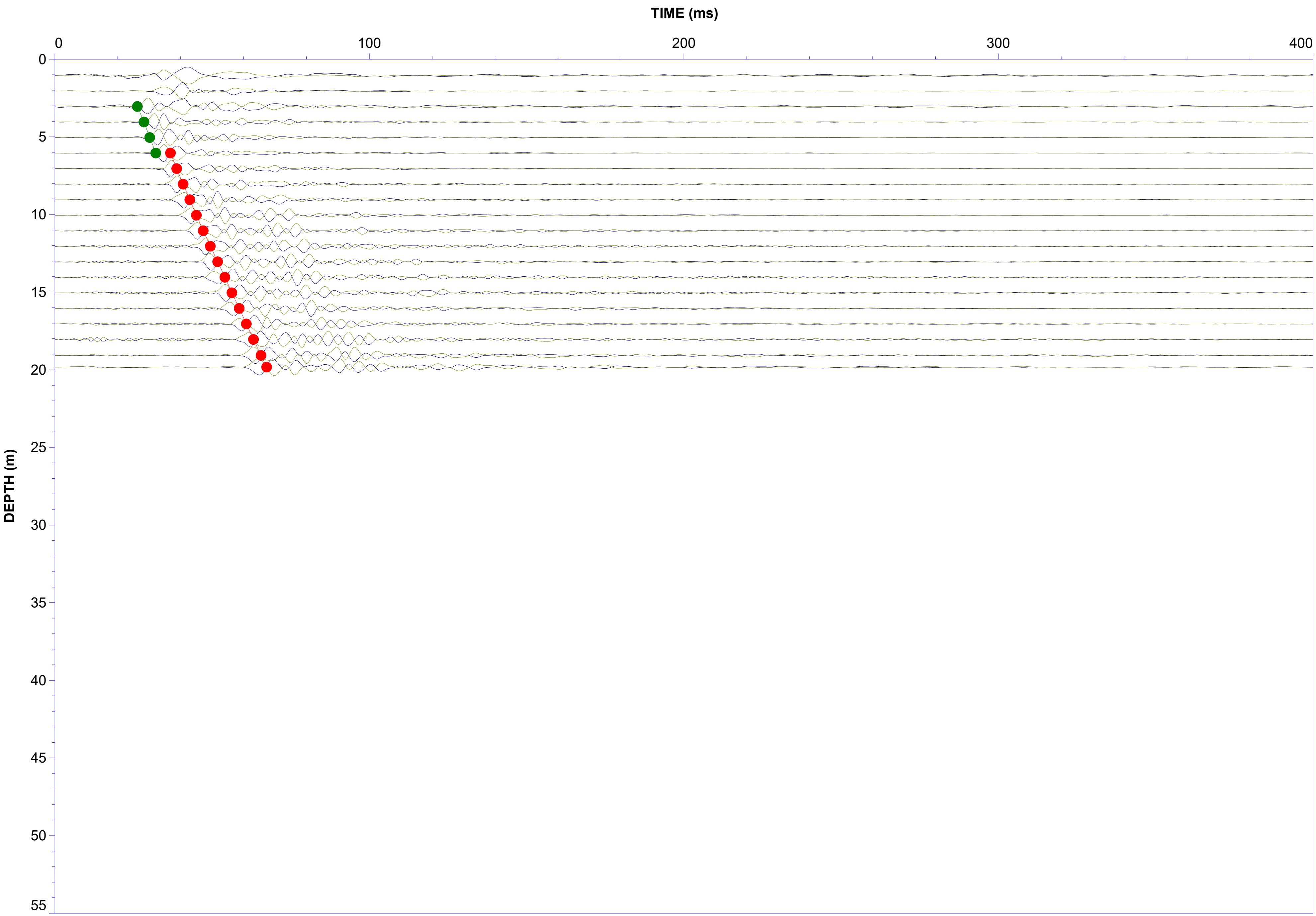
ConeTec

CONETEC

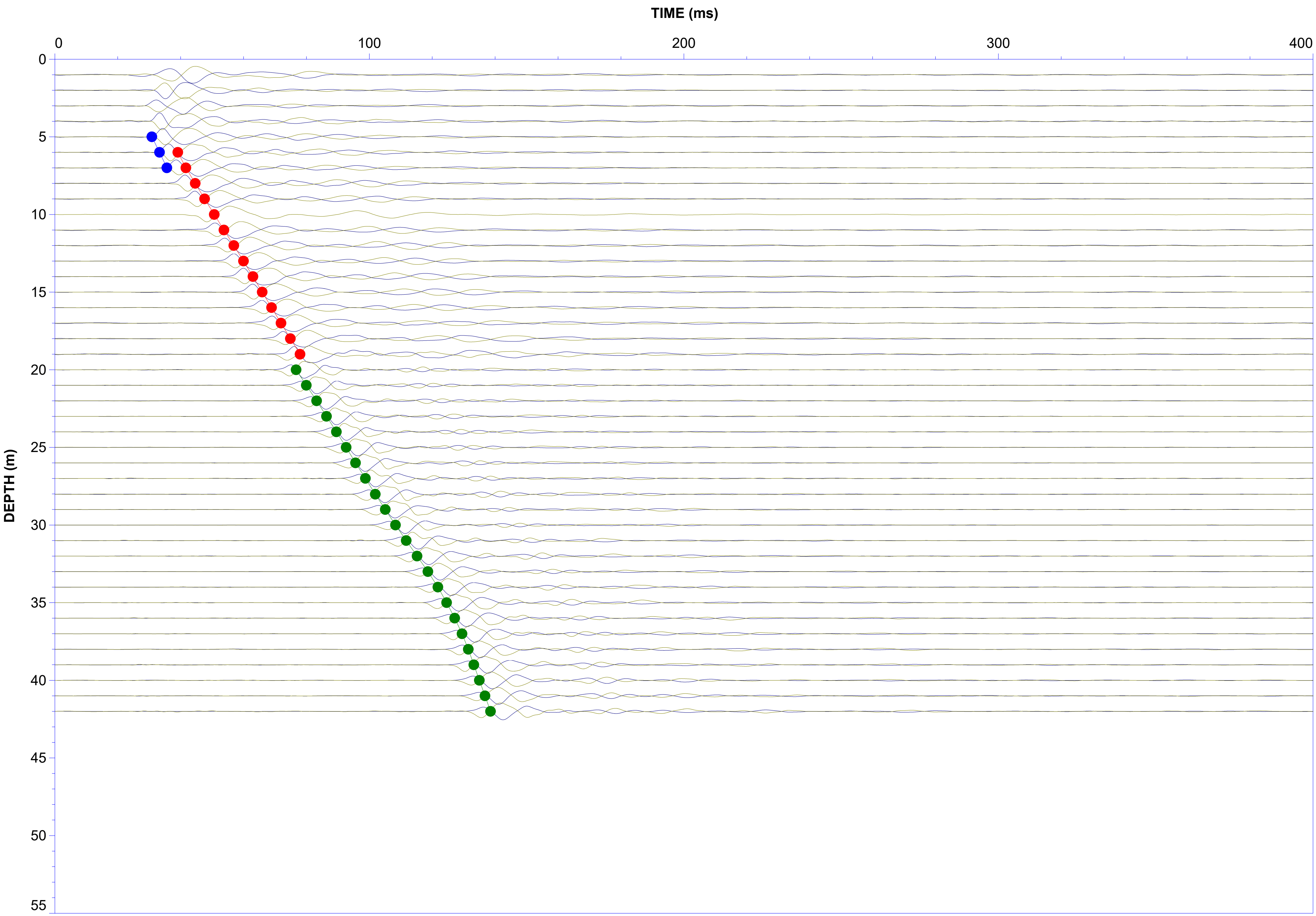
Job No: 21-05-23424 | Client: Golder Associates | Project: Bradford Bypass | Filter: 0 - 300Hz | Sounding: SCPT21-HSE-01 | Date: 20-Dec-2021



Job No: 21-05-23424 Client: Golder Associates Project: Bradford Bypass Filter: 0 - 300Hz Sounding: SCPT21-HRW-01B Date: 22-Dec-2021



Job No: 21-05-23424 Client: Golder Associates Project: Bradford Bypass Filter: 0 - 300Hz Sounding: SCPT21-HRW-04 Date: 21-Dec-2021



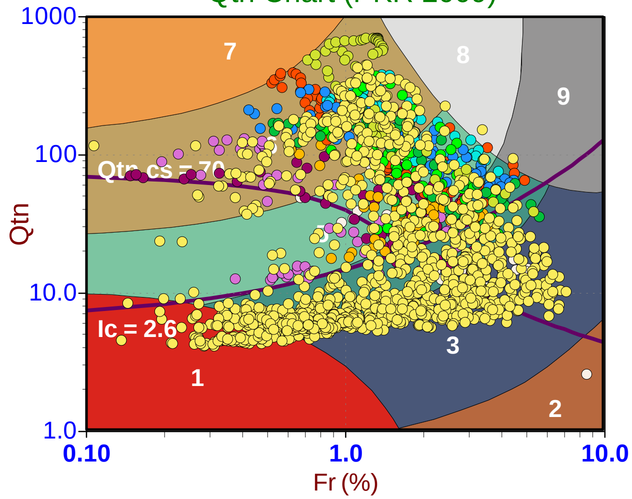
Soil Behaviour Type (SBT) Scatter Plots

ConeTec

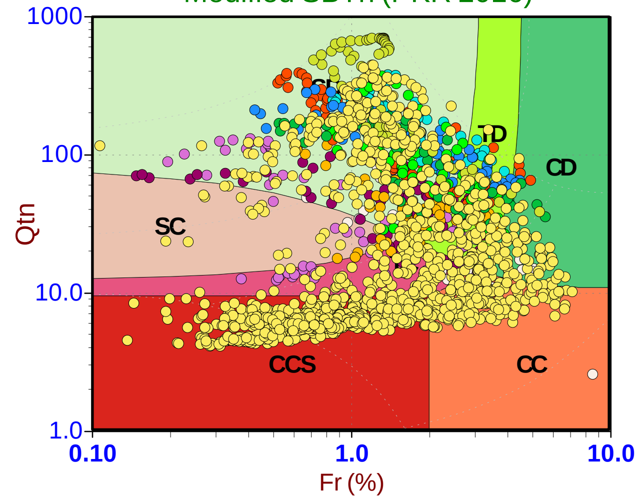
Golder

Job No: 21-05-23424	Sounding: SCPT21-HSE-01
Date: 2021-12-20 13:32	Cone: 765:T1500F15U35
Site: East Holland River	

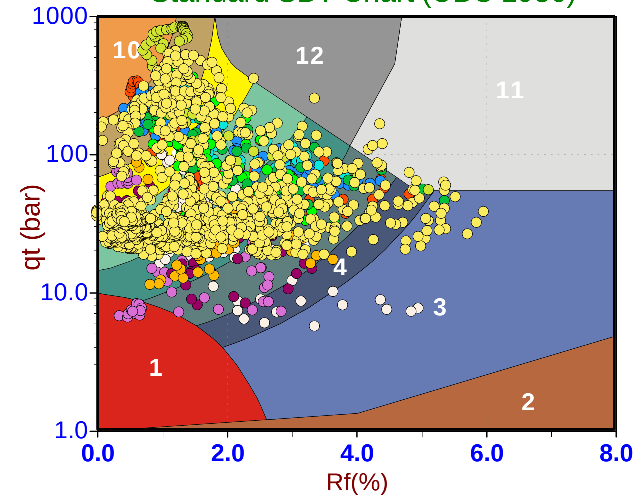
Qtn Chart (PKR 2009)



Modified SBTn (PKR 2016)



Standard SBT Chart (UBC 1986)



Depth Ranges

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

Legend

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

Legend

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

Legend

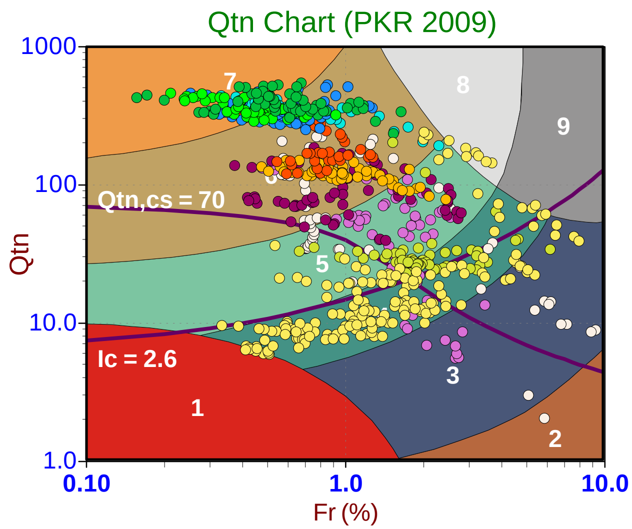
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

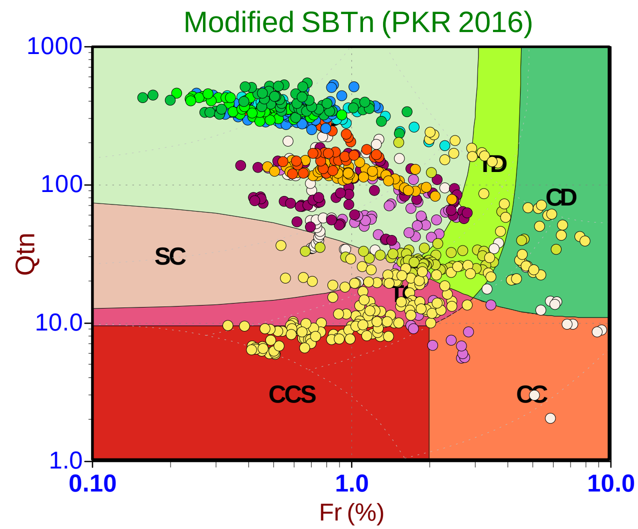
CONETEC

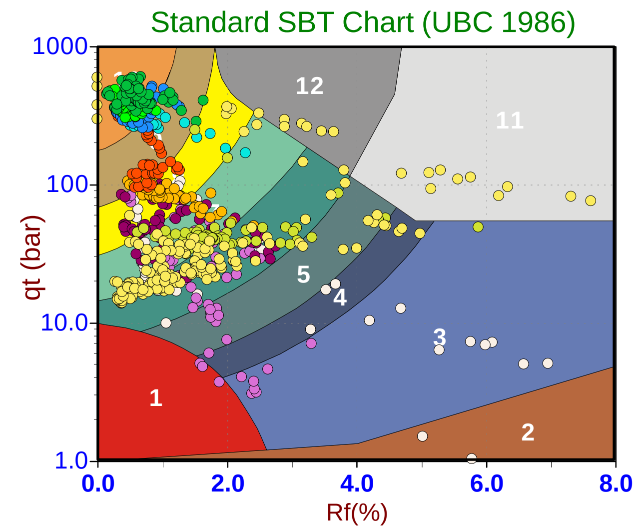
Golder

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

Sounding: SCPT21-HRW-01B
Cone: 765:T1500F15U35







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

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

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

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

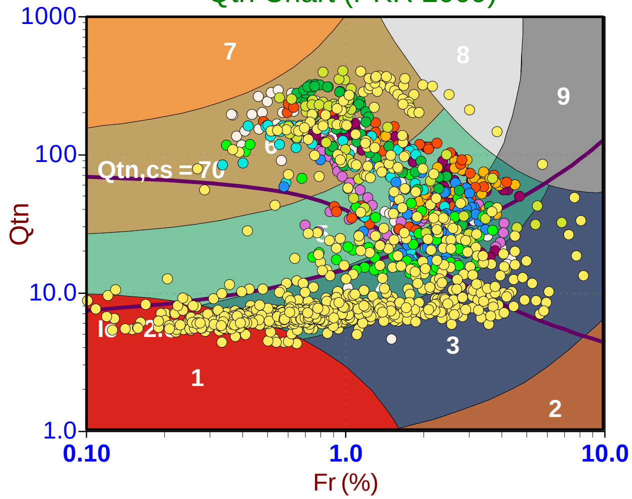
CONETEC

Golder

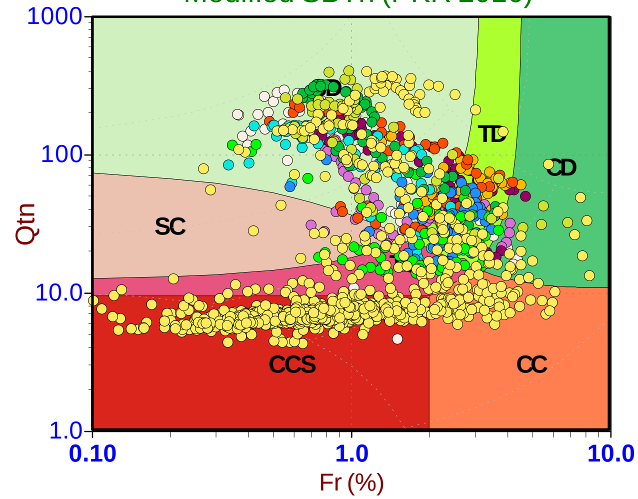
Job No: 21-05-23424
Date: 2021-12-21 11:38
Site: East Holland River

Sounding: SCPT21-HRW-04
Cone: 765:T1500F15U35

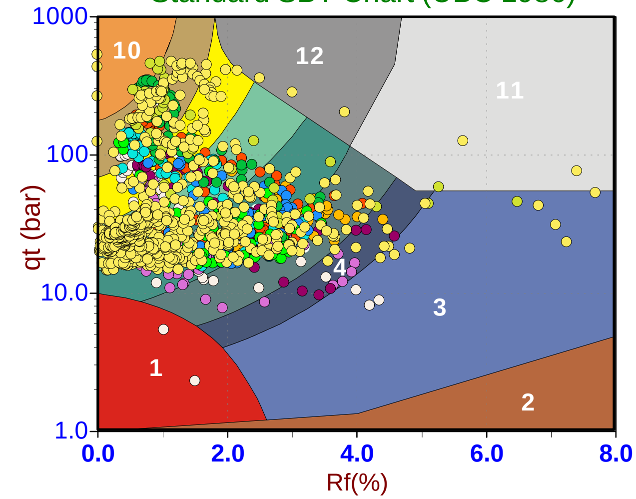
Qtn Chart (PKR 2009)



Modified SBTn (PKR 2016)



Standard SBT Chart (UBC 1986)



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

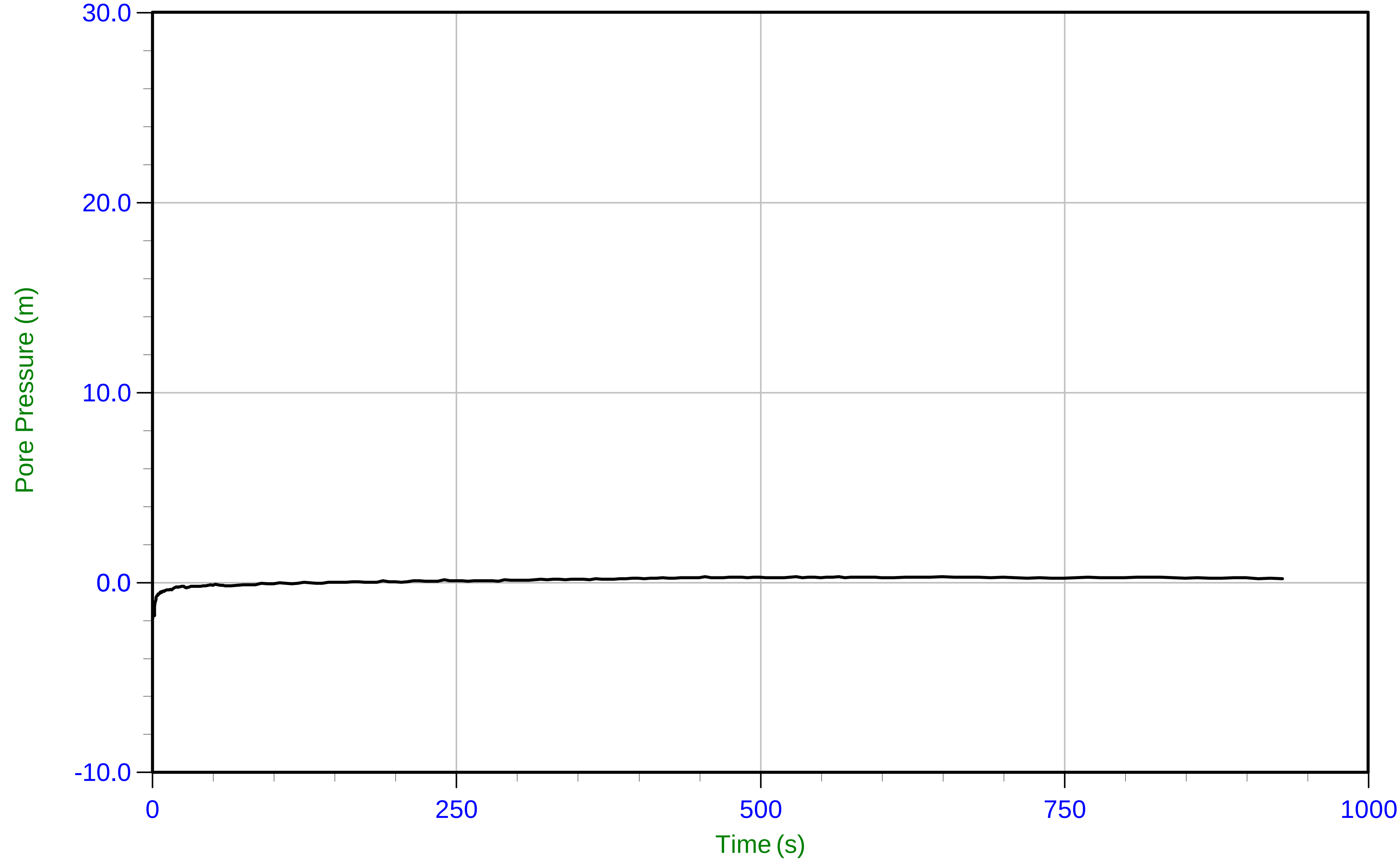
Sounding ID	File Name	Cone Area (cm^2)	Duration (s)	Test Depth (m)	Equilibrium Pore Pressure U_{eq} (m)	Estimated Equilibrium Pore Pressure U_{eq} (m)	Estimated Phreatic Surface (m)	t_{50} [1] (s)	Assumed Rigidity Index (I_r)	c_h [2] (cm^2/min)
SCPT21-HSE-01	21-05-23424_SP-HSE-01	15	930	1.000	0.0		1.0			
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 cm²



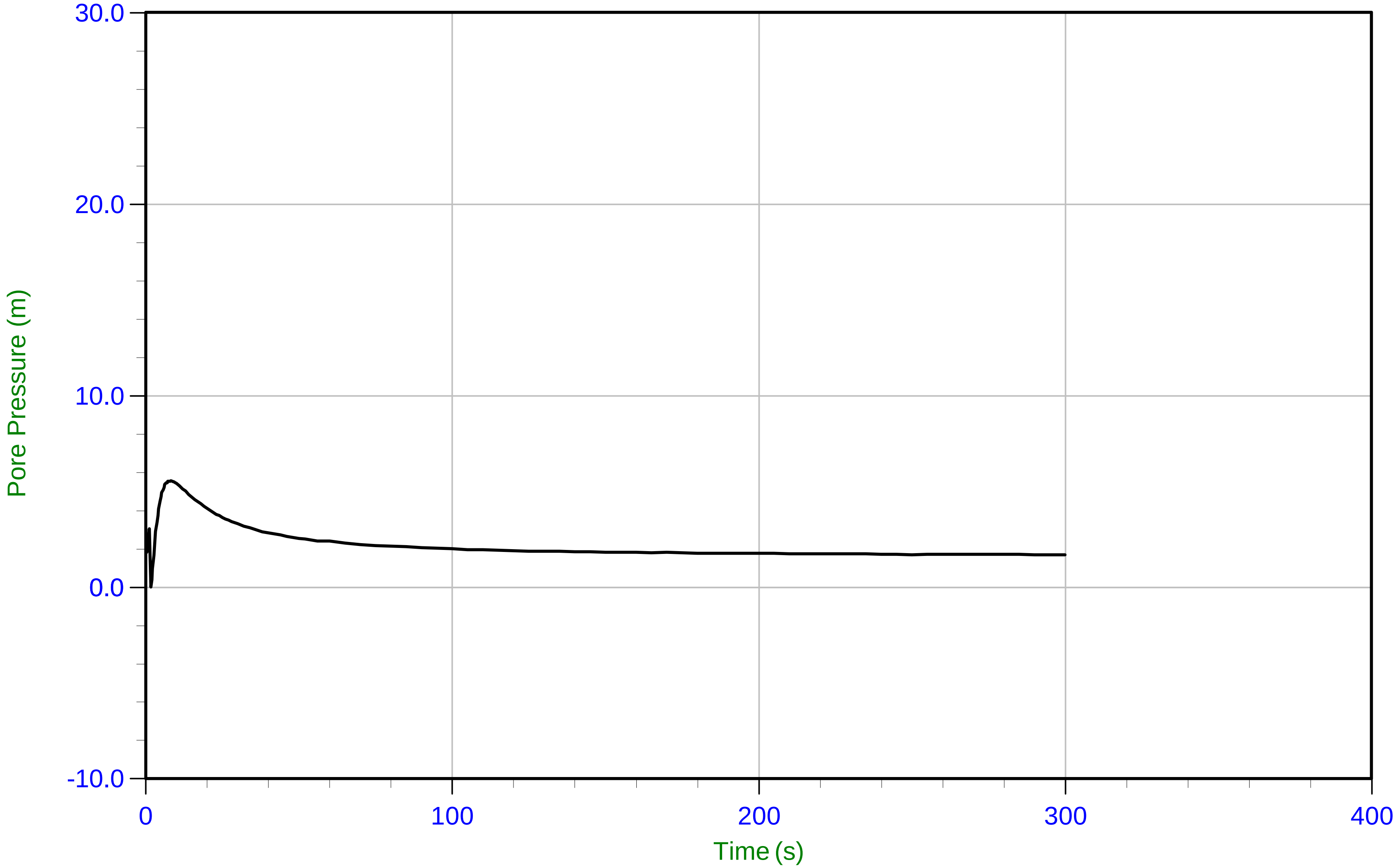
Trace Summary:

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 cm²



Trace Summary:

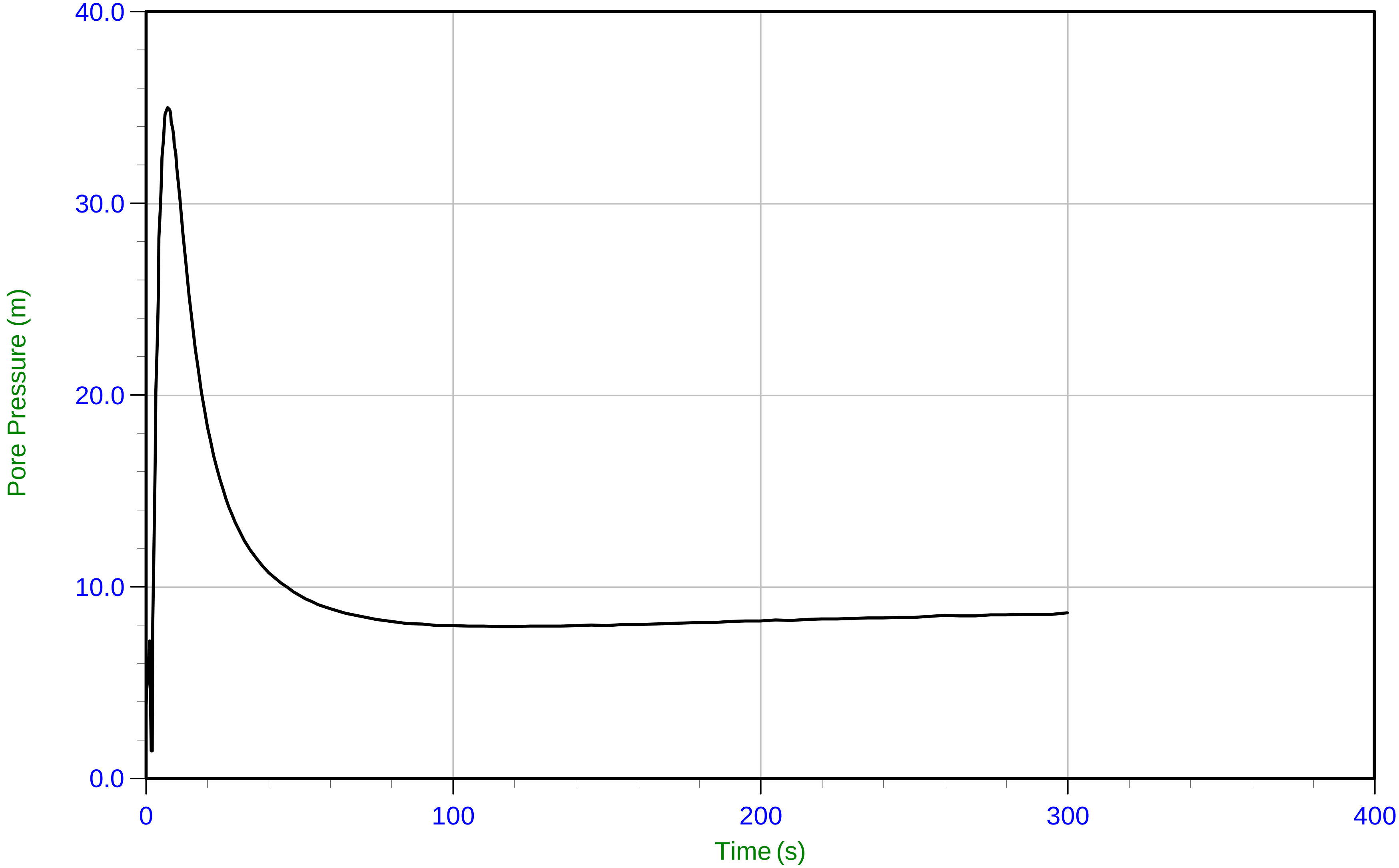
Filename: 21-05-23424_SP-HSE-01.PPF	u Min: 0.0 m	WT: 1.300 m / 4.265 ft	T(50): 17.0 s
Depth: 3.000 m / 9.842 ft	u Max: 5.6 m	Ueq: 1.7 m	Ir: 100
Duration: 300.0 s	u Final: 1.7 m	U(50): 3.63 m	Ch: 41.3 cm²/min

ConeTec

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 cm²



Trace Summary:

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

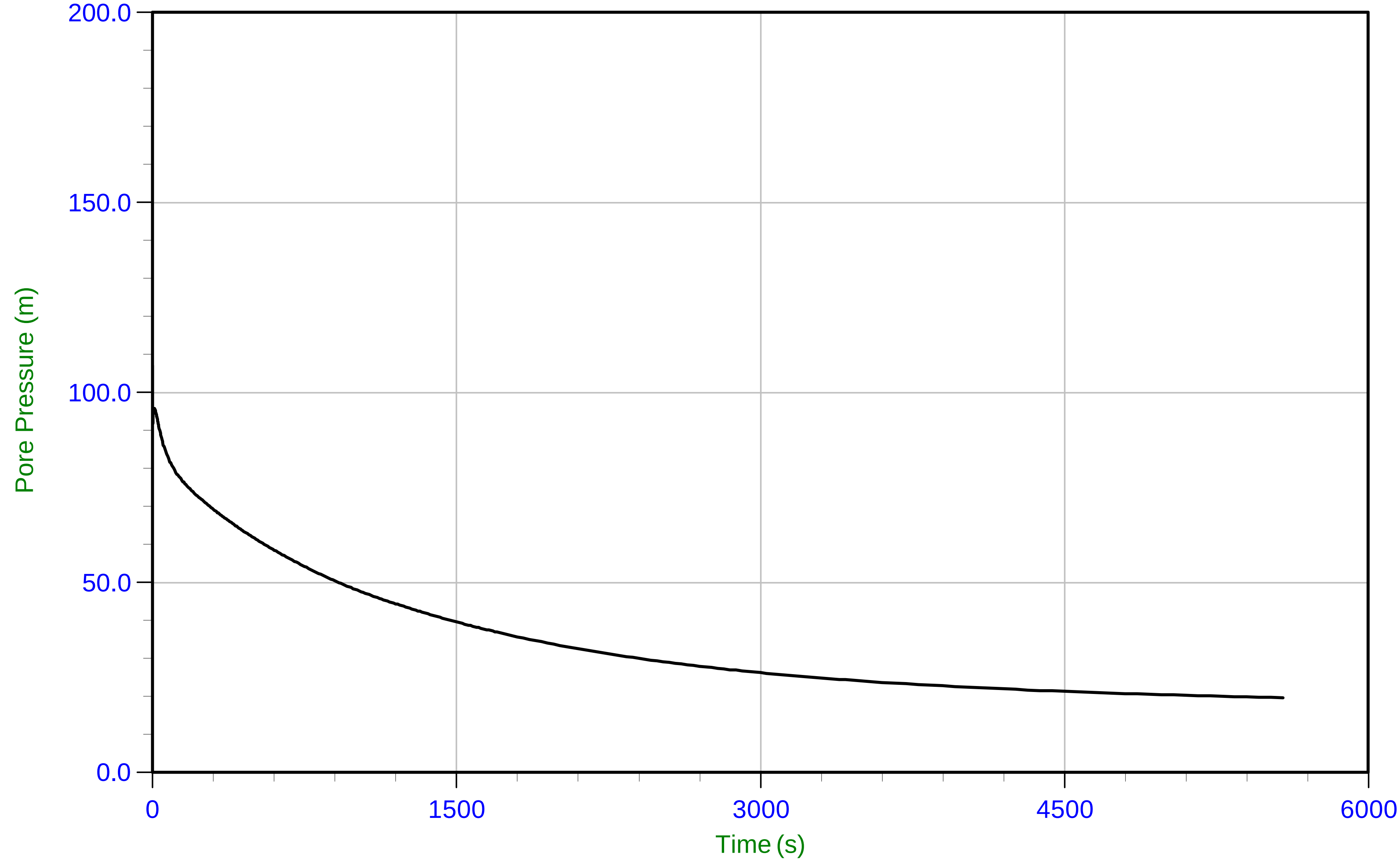
u Min: 1.5 m
u Max: 35.0 m
u Final: 8.6 m

CONETEC

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 cm²



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

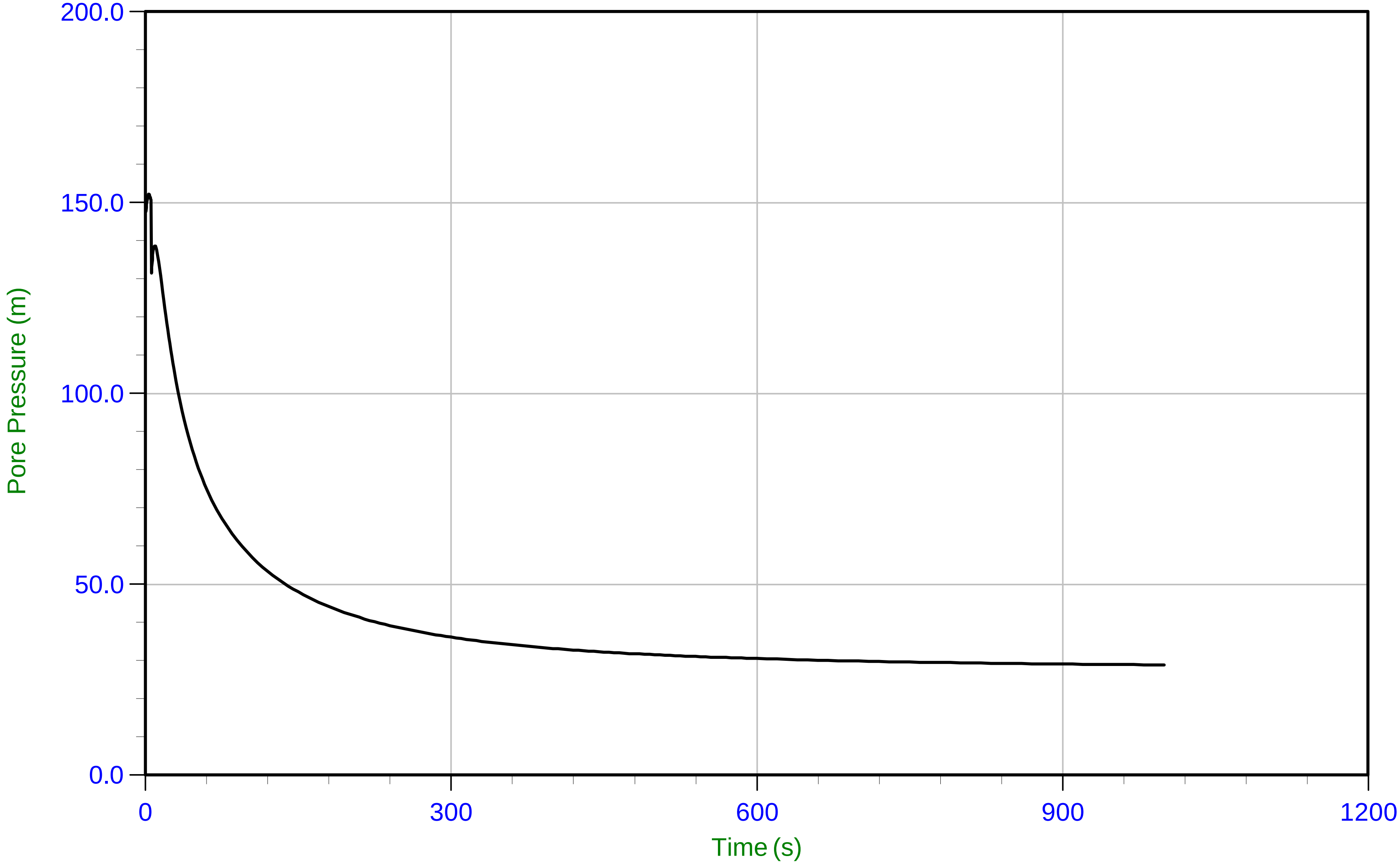
WT: 1.700 m / 5.577 ft
Ueq: 16.6 m
U(50): 61.84 m

T(50): 497.3 s
Ir: 100
Ch: 1.4 cm²/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 cm²



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

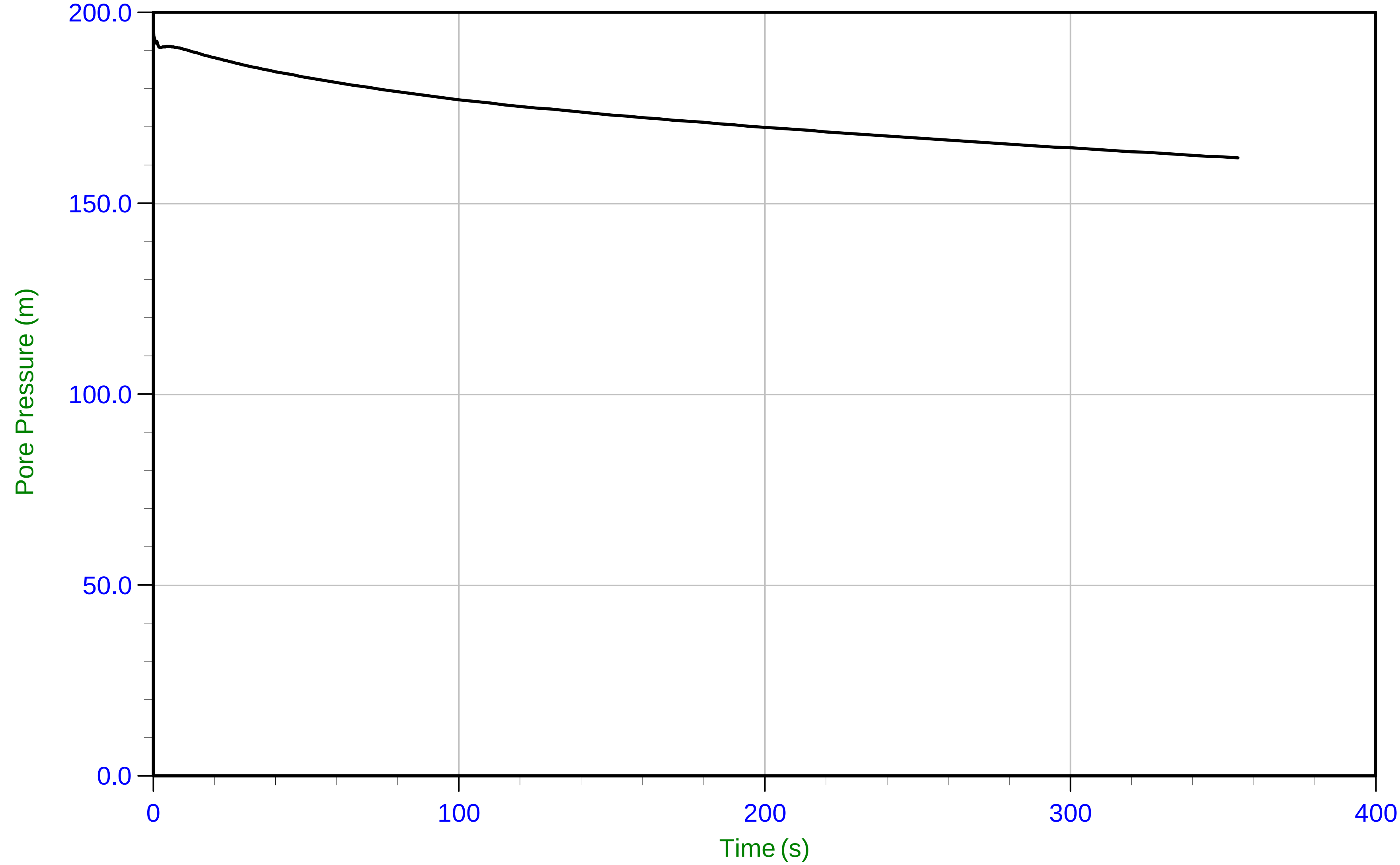
T(50): 37.2 s
Ir: 100
Ch: 18.9 cm²/min

CONETEC

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 cm²



Trace Summary:

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

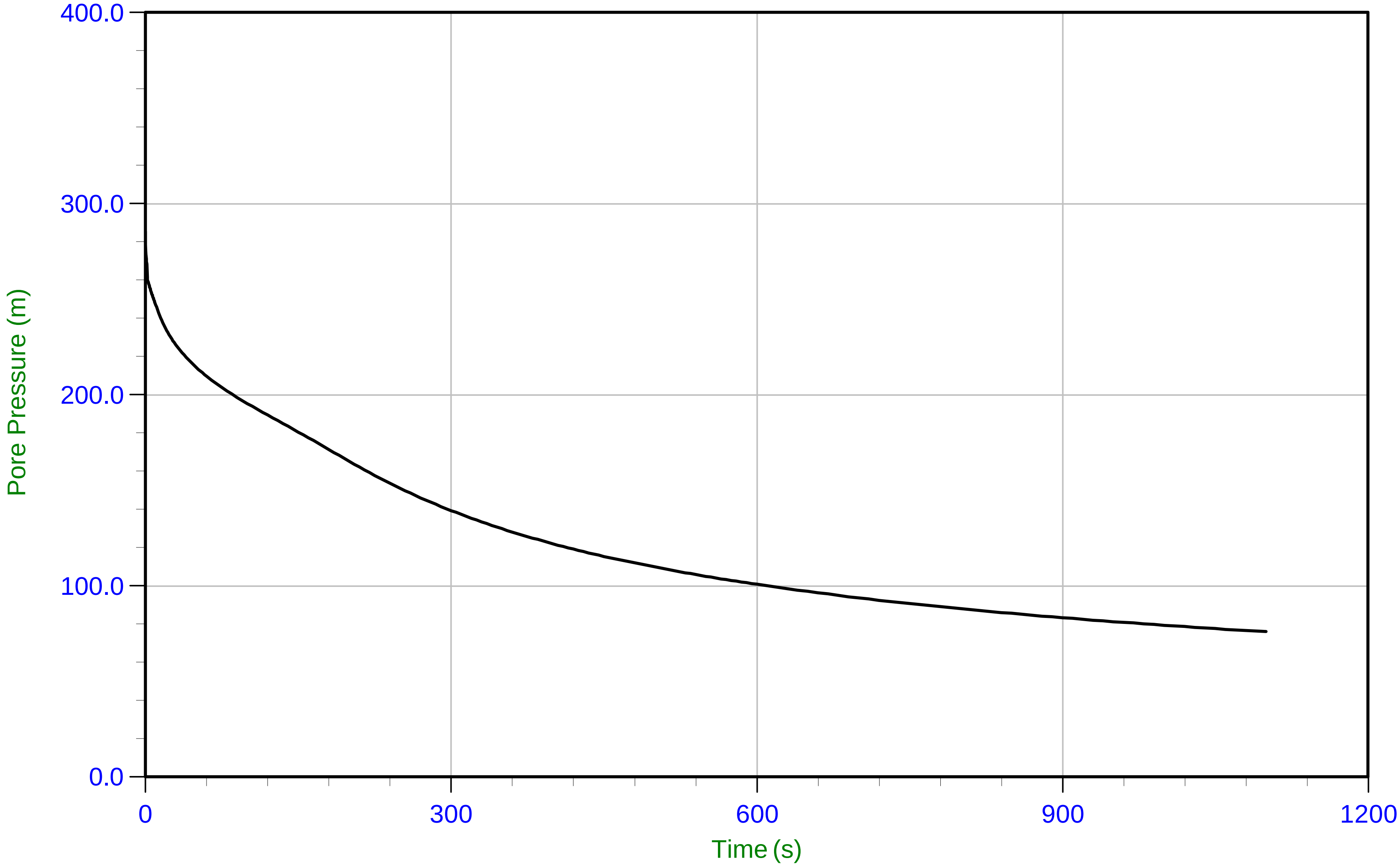
u Min: 161.9 m
u Max: 196.5 m
u Final: 161.9 m

ConeTec

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 cm²



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

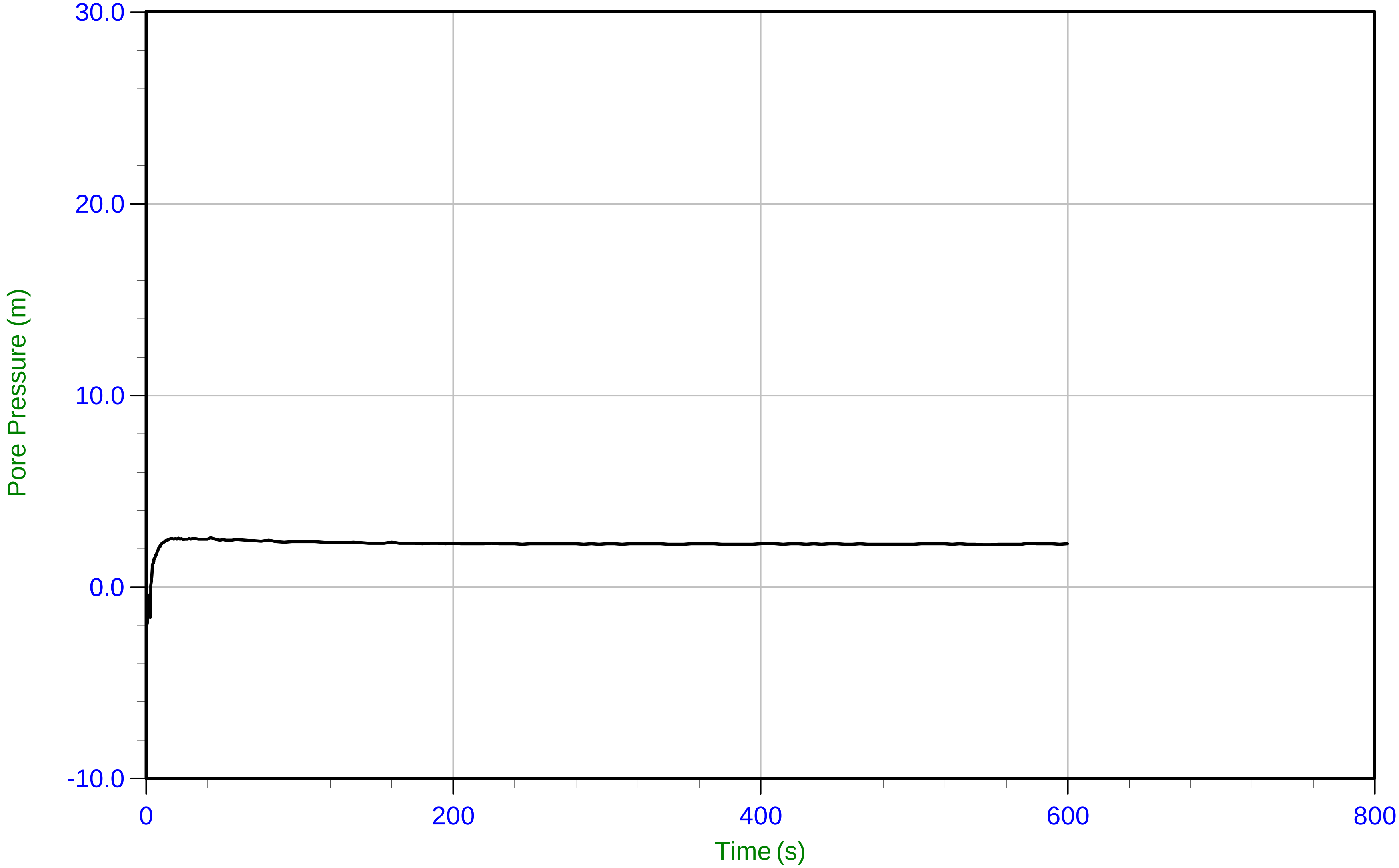
T(50): 194.7 s
Ir: 100
Ch: 3.6 cm²/min

CONETEC

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 cm²



Trace Summary:

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

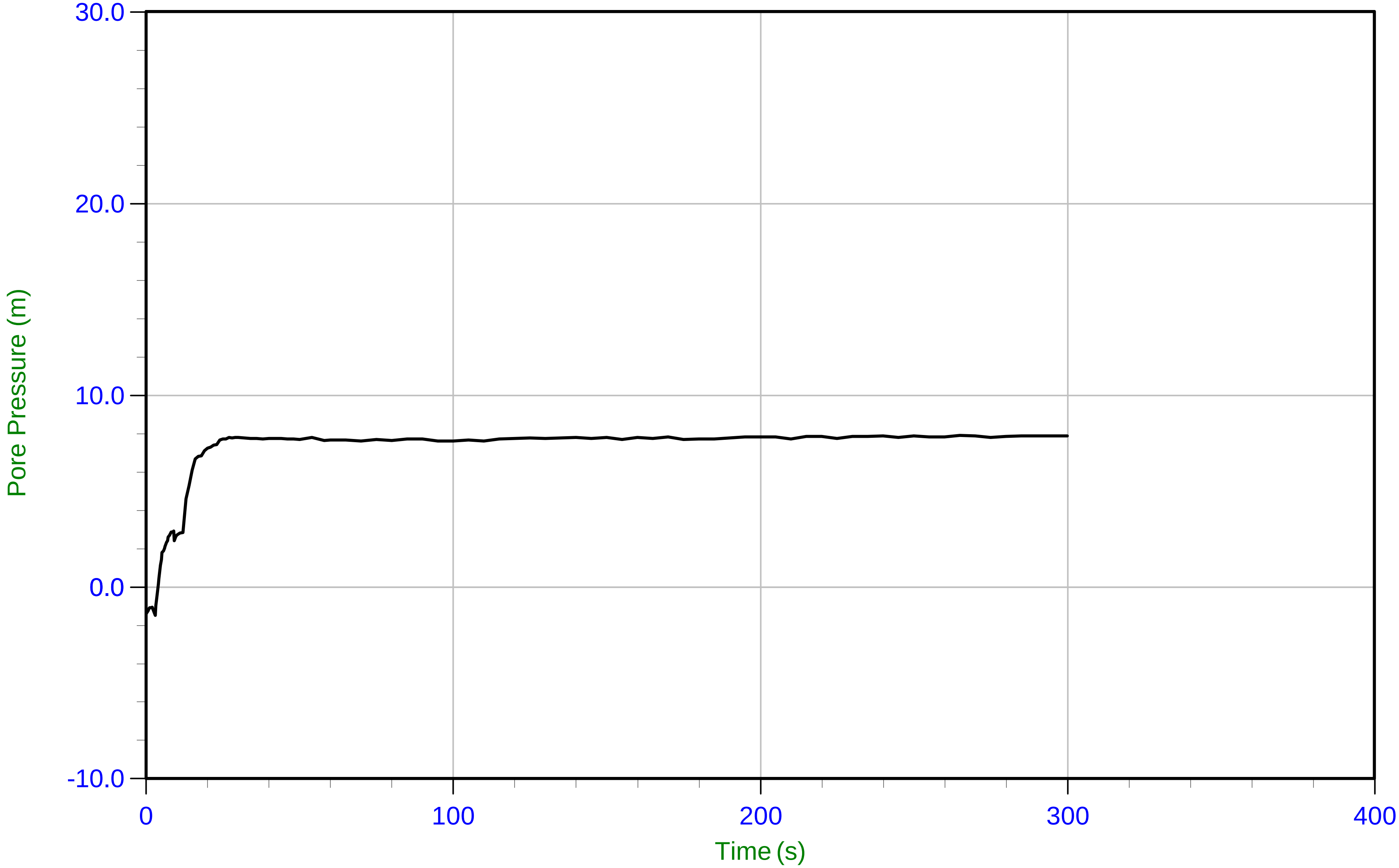
u Min: -2.1 m
u Max: 2.6 m
u Final: 2.2 m

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 cm²



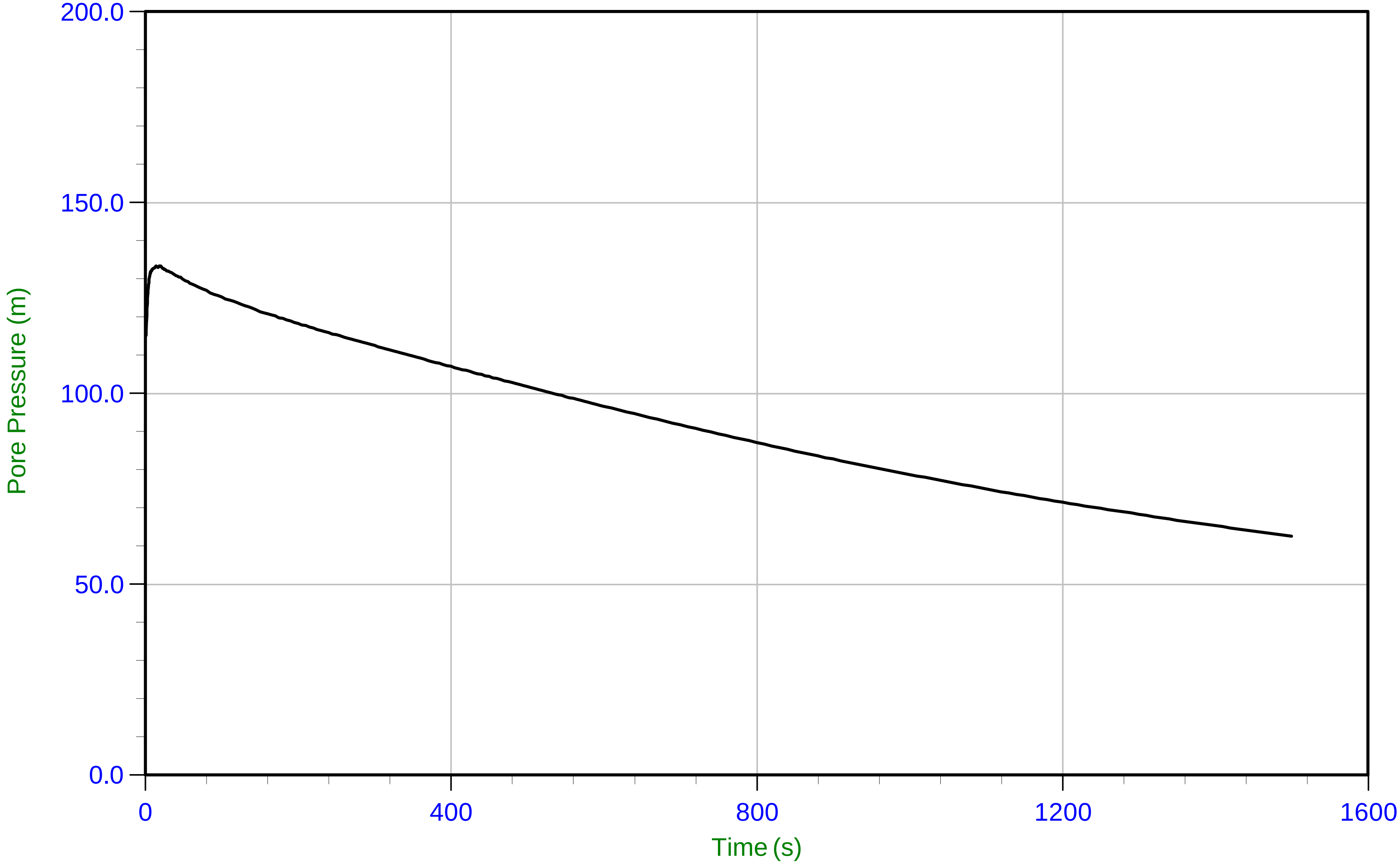
Trace Summary:

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 cm²



Trace Summary:

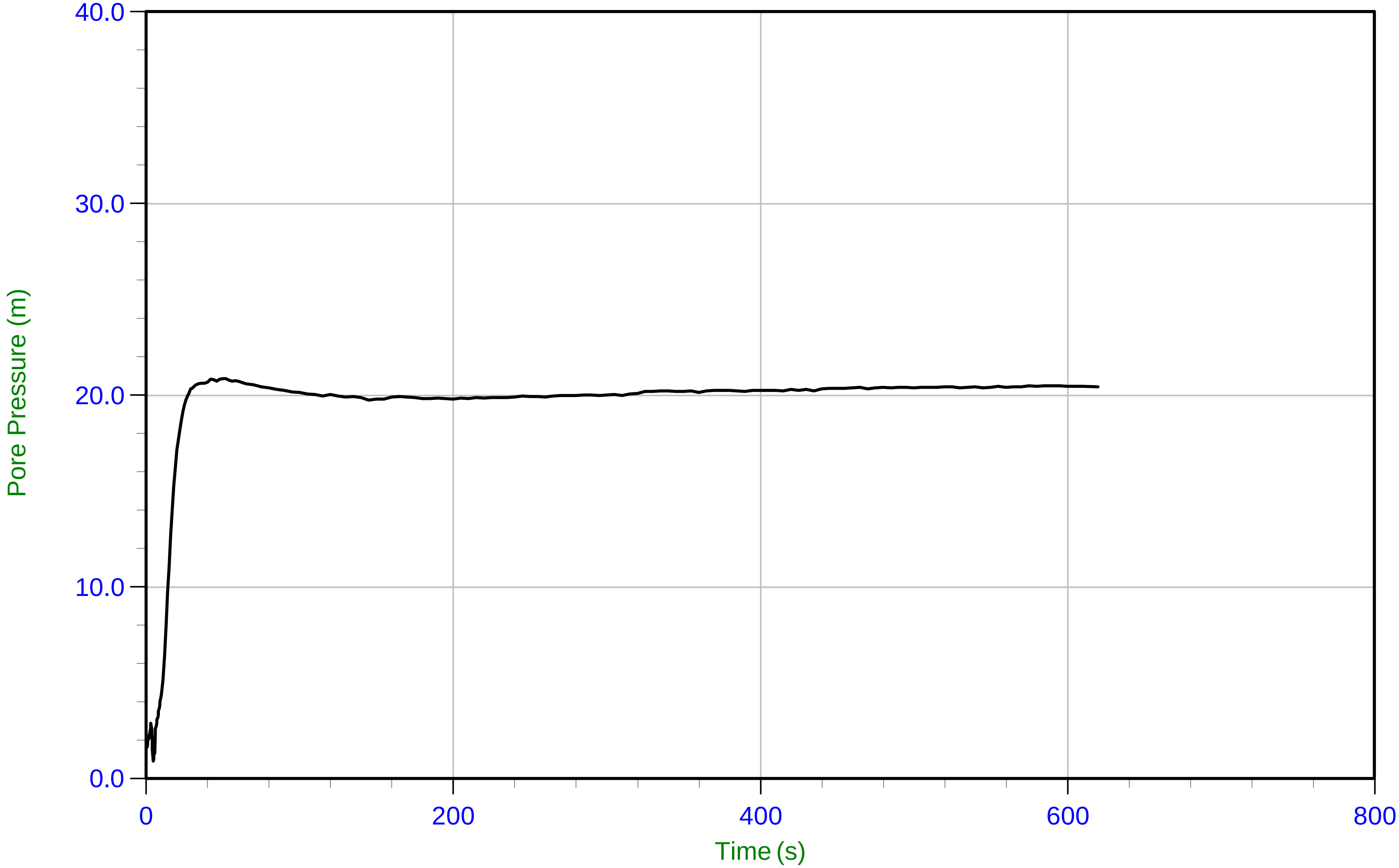
Filename: 21-05-23424_SP-HRW-01B.PPF	u Min: 62.6 m	WT: 0.850 m / 2.789 ft	T(50): 1112.7 s
Depth: 15.250 m / 50.032 ft	u Max: 133.4 m	Ueq: 14.4 m	Ir: 100
Duration: 1500.0 s	u Final: 62.6 m	U(50): 73.92 m	Ch: 0.6 cm²/min

ConeTec

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 cm²



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

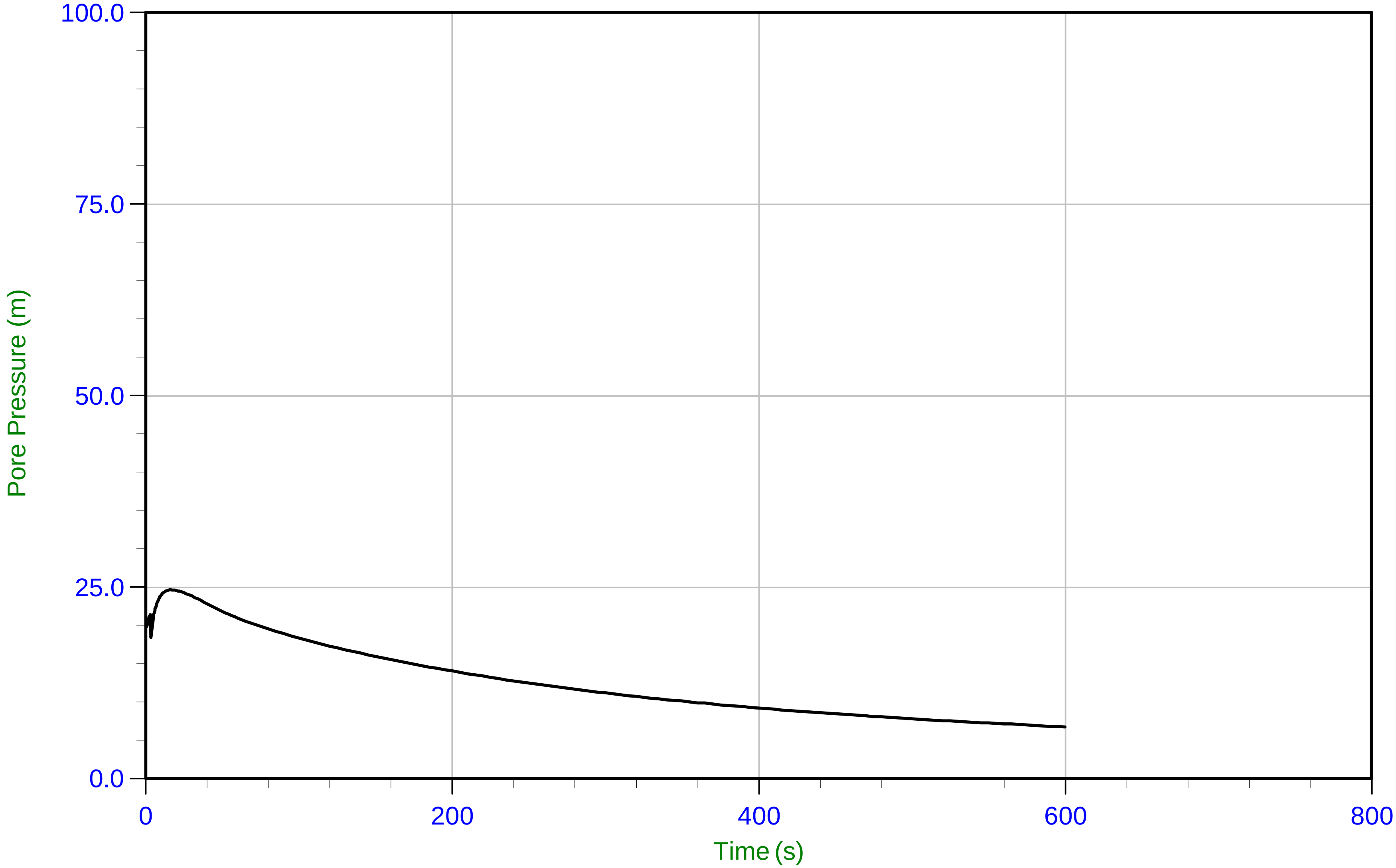
WT: -0.625 m / -2.050 ft
Ueq: 20.4 m

ConeTec

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 cm²



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

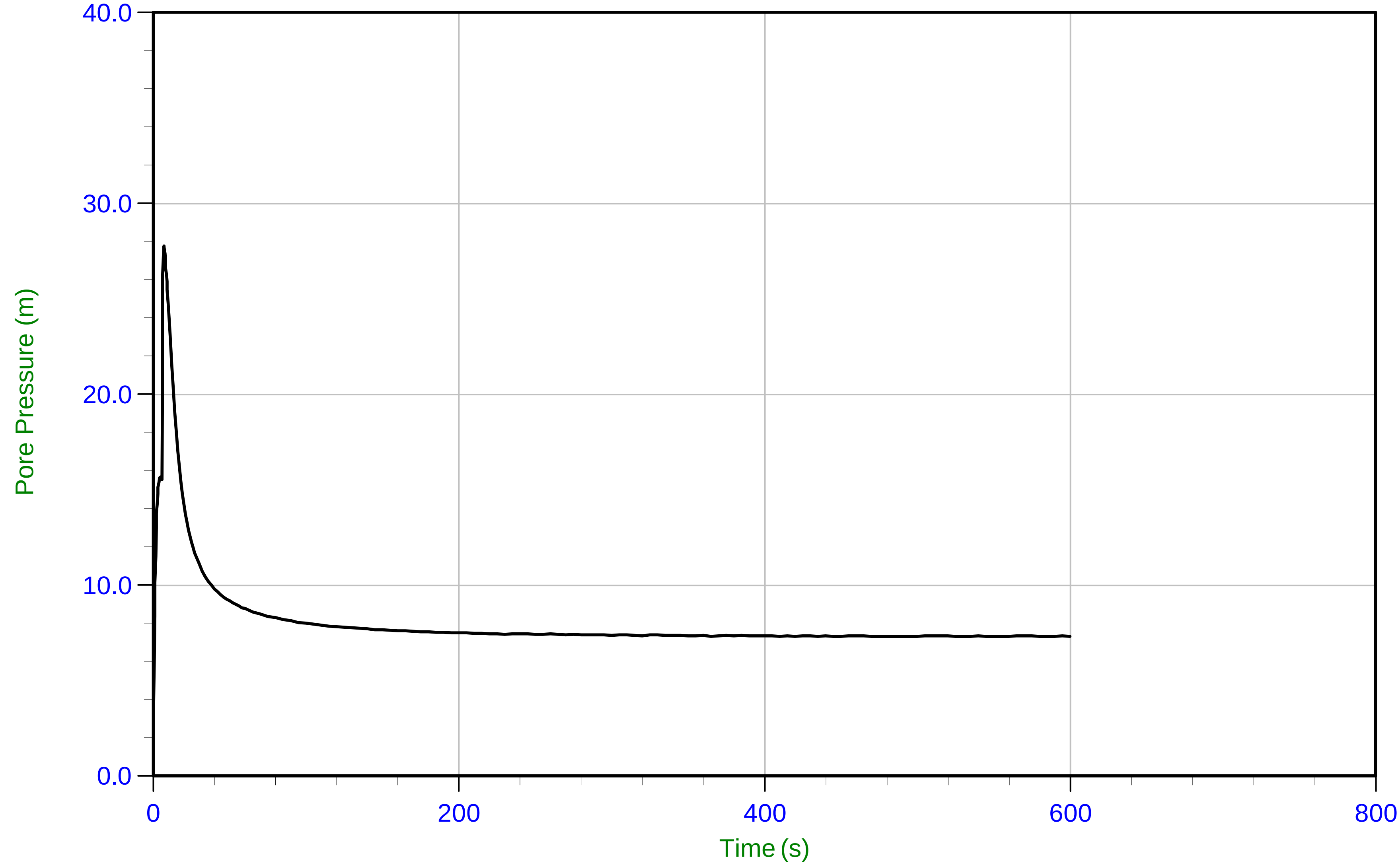
T(50): 195.8 s
Ir: 100
Ch: 3.6 cm²/min

ConeTec

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 cm²



Trace Summary:

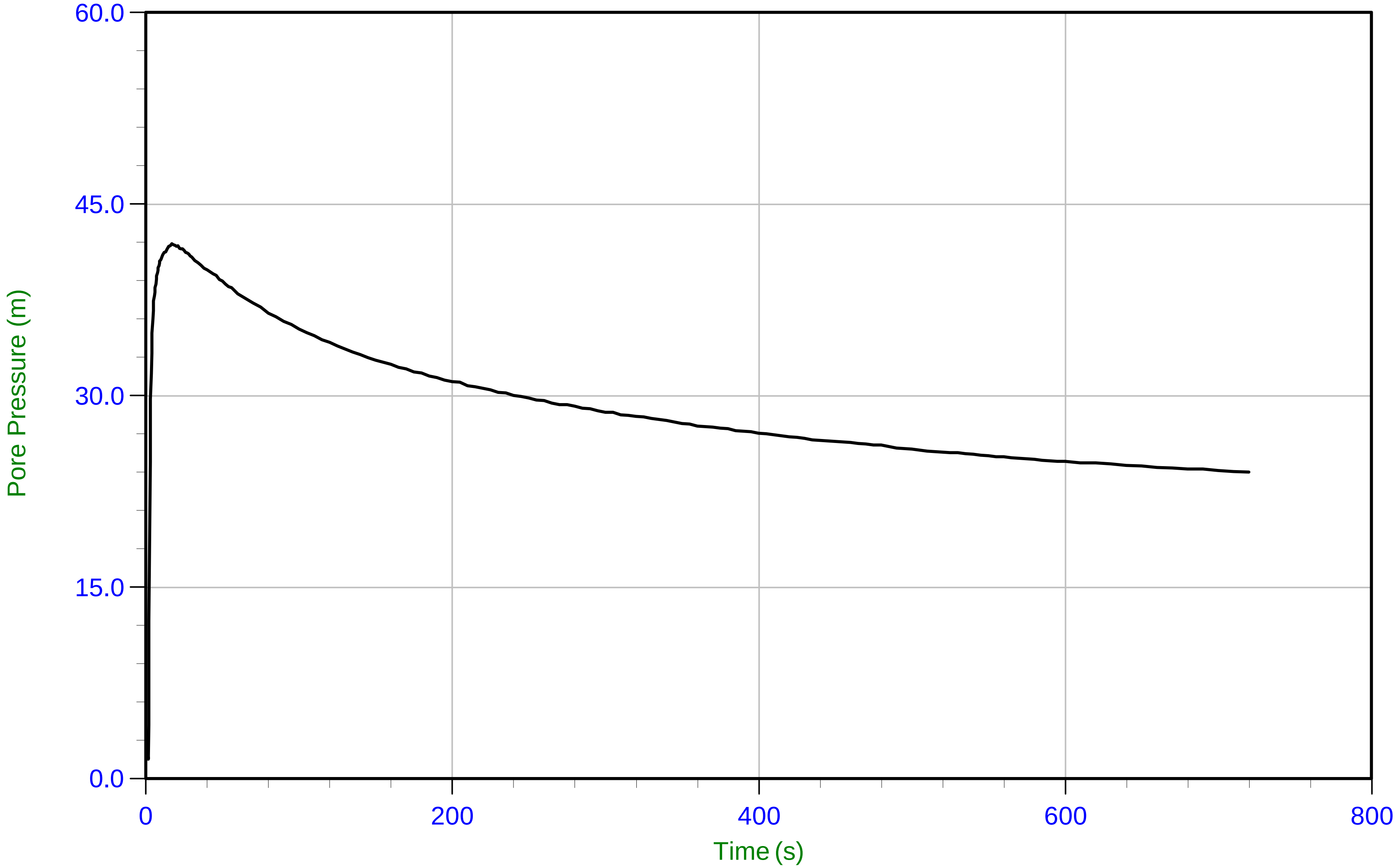
Filename: 21-05-23424_SP-HRW-04.PPF
Depth: 7.625 m / 25.016 ft
Duration: 600.0 s

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 cm²/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 cm²



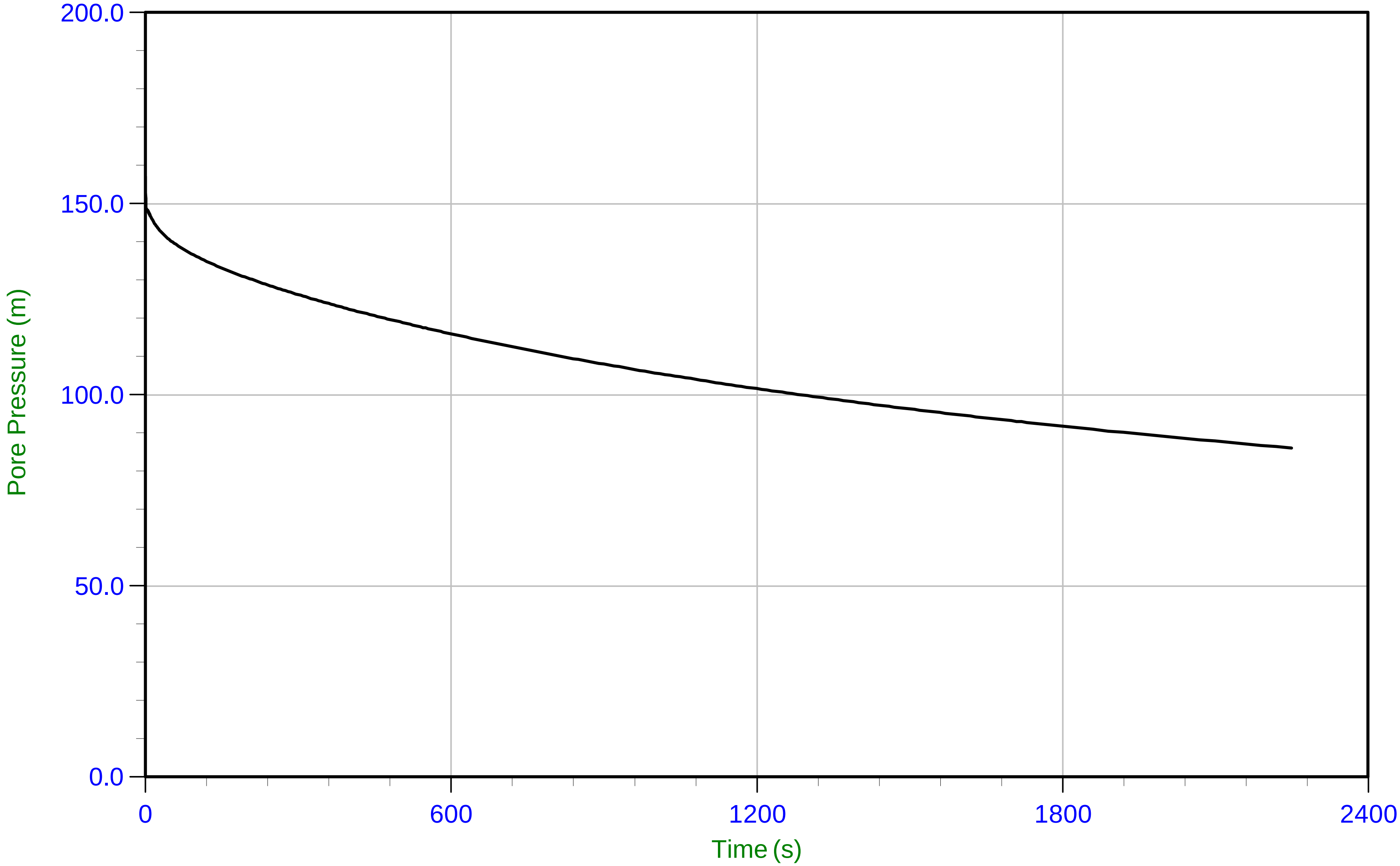
Trace Summary:				
	Filename: 21-05-23424_SP-HRW-04.PPF	u Min: 1.4 m	WT: 0.325 m / 1.066 ft	T(50): 228.5 s
	Depth: 18.300 m / 60.039 ft	u Max: 41.9 m	Ueq: 18.0 m	Ir: 100
	Duration: 720.0 s	u Final: 24.0 m	U(50): 29.93 m	Ch: 3.1 cm²/min

ConeTec

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 cm²



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 cm²/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: $q_t = q_c + (1-a) \bullet u_2$ (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.

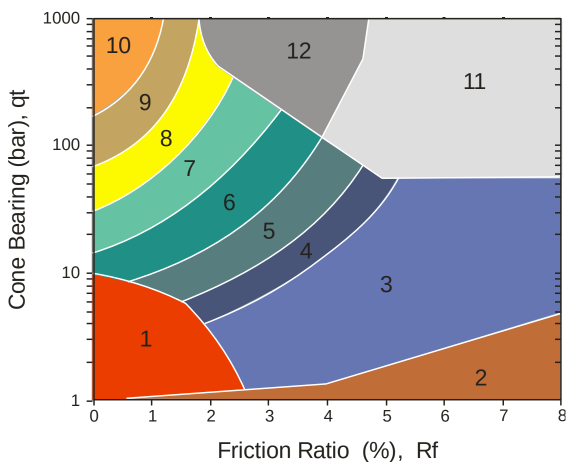
Effective vertical overburden stresses are calculated based on a hydrostatic distribution of equilibrium pore pressures below the water table or from a user defined equilibrium pore pressure profile (typically obtained from CPT dissipation tests) or a combination of the two. For over water projects the 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 Bq parameter. The normalized Qtn 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.

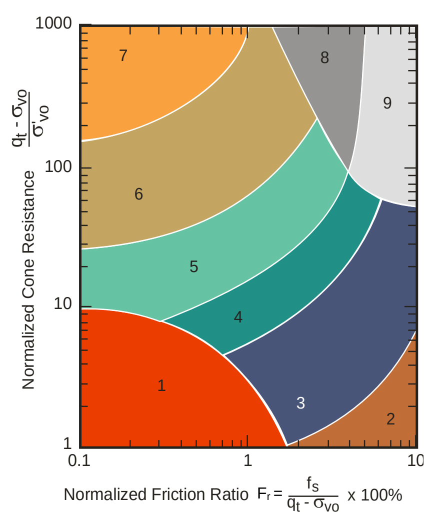


Zone	qt / N	Soil Behavior Type
1	2	sensitive fine grained
2	1	organic material
3	1	clay
4	1.5	silty clay to clay
5	2	clayey silt to silty clay
6	2.5	sandy silt to clayey silt
7	3	silty sand to sandy silt
8	4	sand to silty sand
9	5	sand
10	6	gravelly sand to sand
11	1	very stiff fine grained *
12	2	sand to clayey sand *

* overconsolidated or cemented

$$Rf = \left(\frac{fs}{qt}\right) \cdot 100\%$$

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



Zone	Normalized Soil Behavior Type
1	sensitive fine grained
2	organic material
3	clay to silty clay
4	clayey silt to silty clay
5	silty sand to sandy silt
6	clean sands to silty sands
7	gravelly sand to sand
8	very stiff sand to clayey sand
9	very stiff fine grained

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

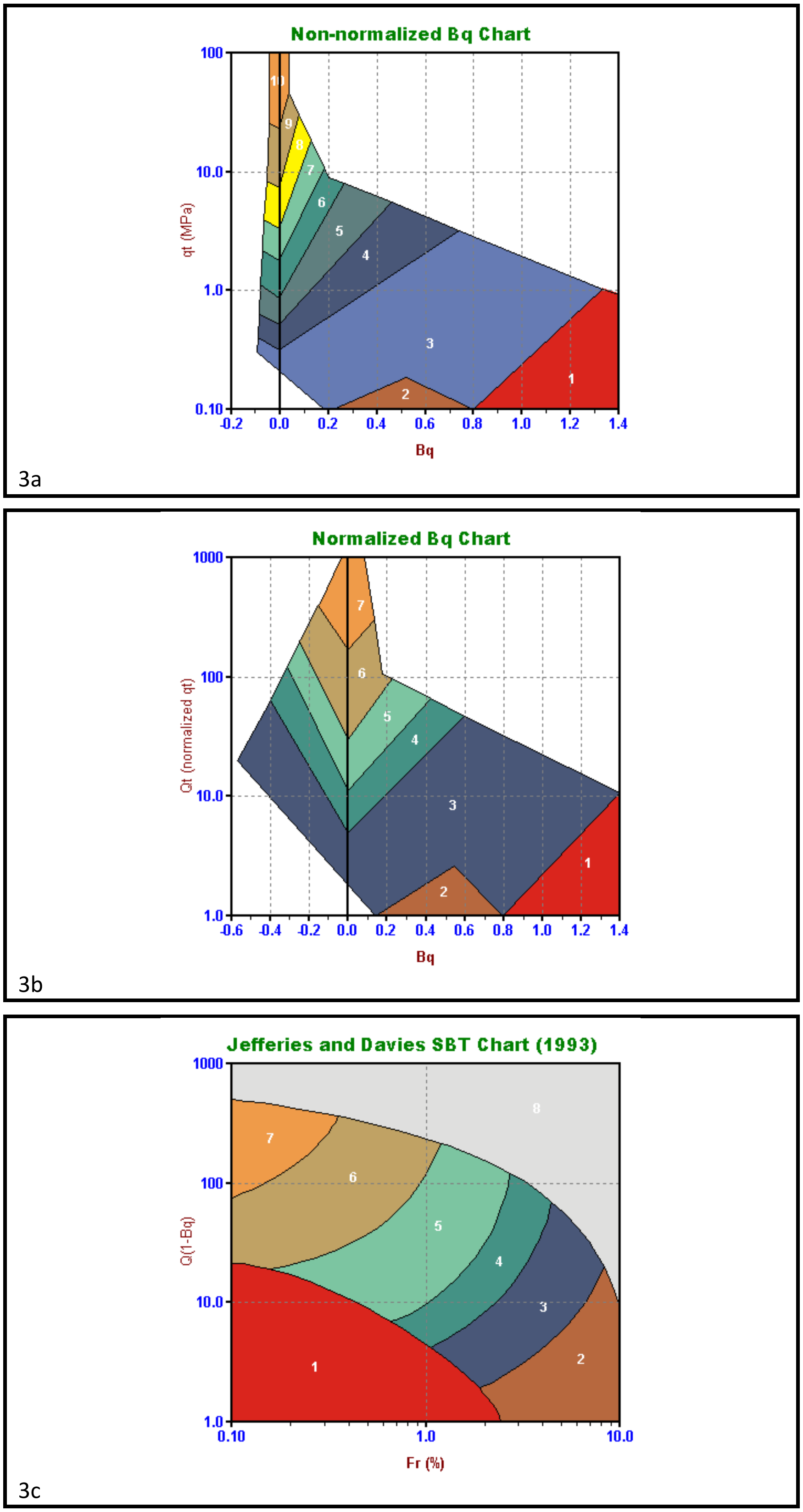


Figure 3. Alternate Soil Behavior Type Charts

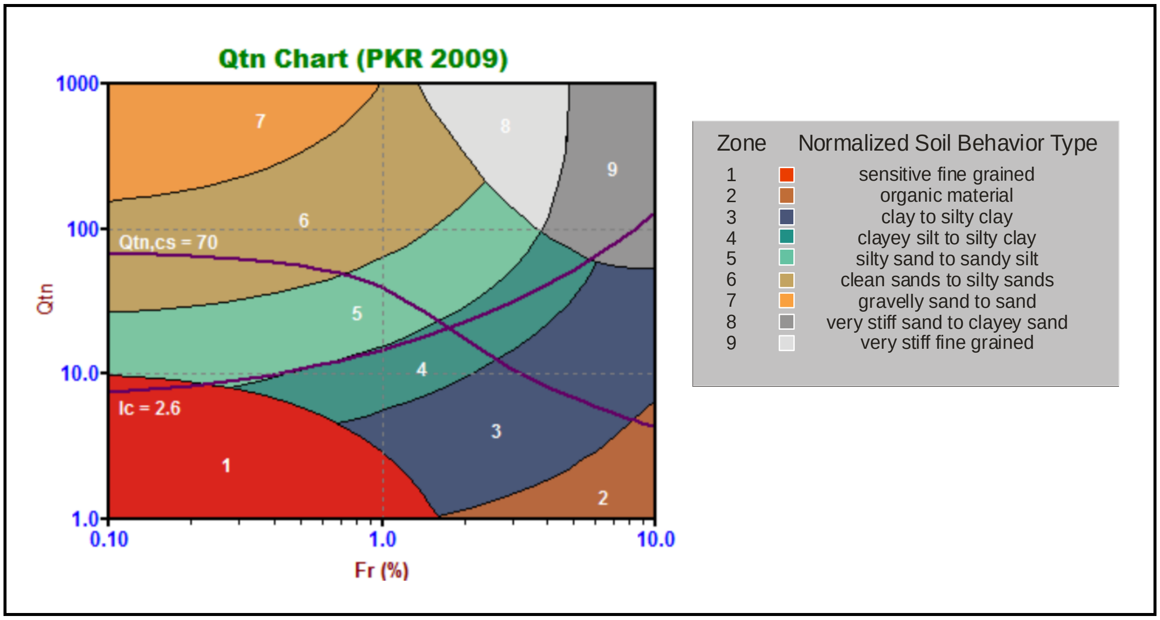


Figure 4. Normalized Soil Behavior Type Chart using Q_{tn} (SBT Qtn)

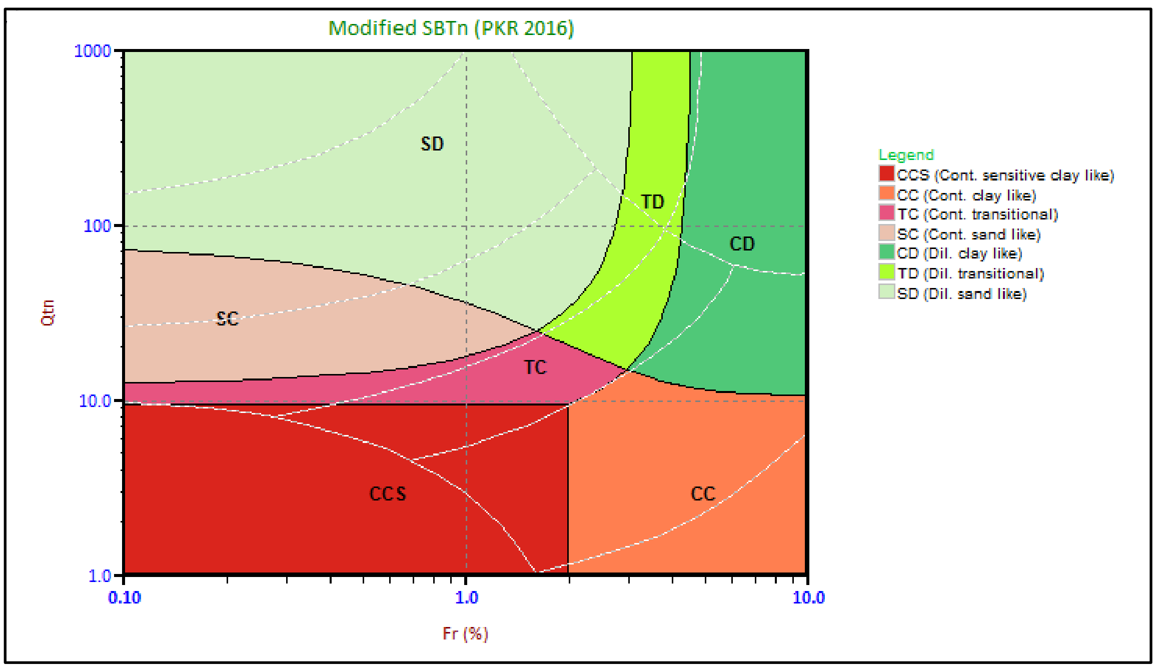


Figure 5. Modified SBTn Behavior Based Chart

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 *(where calculations are done at each point then Mid Layer Depth = Recorded Depth)*	*[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)	$Avgqc = \frac{1}{n}\sum_{i=1}^{n} q_c$ *n=1 when calculations are done at each point*	CK*
Avg qt	Averaged corrected tip (q_t) where: $q_t = q_c + (1-a)\bullet u_2$	$Avgqt = \frac{1}{n}\sum_{i=1}^{n} q_t$ *n=1 when calculations are done at each point*	1
Avg fs	Averaged sleeve friction (f_s)	$Avgfs = \frac{1}{n}\sum_{i=1}^{n} fs$ *n=1 when calculations are done at each point*	CK*
Avg Rf	Averaged friction ratio (R_f) where friction ratio is defined as: $Rf = 100\% \bullet \frac{fs}{q_t}$	$AvgRf = 100\% \cdot \frac{Avgfs}{Avgqt}$ *n=1 when calculations are done at each point*	CK*
Avg u	Averaged dynamic pore pressure *(u)*	$Avgu = \frac{1}{n}\sum_{i=1}^{n} u_i$ *n=1 when calculations are done at each point*	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)	$AvgRes = \frac{1}{n}\sum_{i=1}^{n} Resistivity_i$ *n=1 when calculations are done at each point*	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)	$AvgUVIF = \frac{1}{n}\sum_{i=1}^{n} UVIF_i$ *n=1 when calculations are done at each point*	CK*
Avg Temp	Averaged Temperature (this data is not always available since it requires specialized calibrations)	$AvgTemp = \frac{1}{n}\sum_{i=1}^{n} Temperature_i$ *n=1 when calculations are done at each point*	CK*
Avg Gamma	Averaged Gamma Counts (this data is not always available since it is a specialized test requiring an additional module)	$AvgGamma = \frac{1}{n}\sum_{i=1}^{n} Gamma_i$ *n=1 when calculations are done at each point*	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.	Unit Weight of soil determined from one of the following user selectable options: 1) uniform value 2) value assigned to each SBT zone 3) value assigned to each SBTn zone 4) value assigned to SBTn zone as determined from Robertson and Wride (1998) based on q_{c1n} 5) values assigned to SBT Qtn zones 6) Mayne fs (sleeve friction) method 7) Robertson 2010 method 8) user supplied unit weight profile The last option may co-exist with any of the other options	See references	3, 5, 15, 21, 24, 29

Calculated Parameter	Description	Equation	Ref
TStress σ_v	Total vertical overburden stress at Mid Layer Depth *A layer is defined as the averaging interval specified by the user where depths are reported at their respective mid-layer depth.* *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.* *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.* *For over-water work the total stress due to the column of water above the mud line is taken into account where appropriate.*	$TStress = \sum_{i=1}^{n} \gamma_i h_i$ *where* γ_i is layer unit weight h_i is layer thickness	CK*
EStress σ_v'	Effective vertical overburden stress at mid-layer depth	$\sigma_v' = \sigma_v - u_{eq}$	CK*
Equil u u_{eq} or u_0	Equilibrium pore pressure determined from one of the following user selectable options: 1) hydrostatic below water table 2) user supplied profile 3) combination of those above 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 pointed is used. 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.	For hydrostatic option: $u_{eq} = \gamma_w \cdot (D - D_{wt})$ where u_{eq} is equilibrium pore pressure γ_w is unit weight of water D is the current depth D_{wt} is the depth to the water table	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	$C_n = (P_a/\sigma_v')^{0.5}$ where $0.0 < C_n < 2.0$ *(user adjustable, typically 1.7)* P_a *is atmospheric pressure (100 kPa)*	12
C_q	Overburden stress normalizing factor	$C_q = 1.8 / (0.8 + (\sigma_v'/P_a))$ where $0.0 < C_q < 2.0$ *(user adjustable)* P_a *is atmospheric pressure (100 kPa)*	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.	*See Figure 1*	5
(N1)60	SPT N_{60} value corrected for overburden pressure	$(N_1)_{60} = C_n \bullet N_{60}$	4
N60Ic	SPT N_{60} values based on the I_c parameter [as defined by Roberston and Wride 1998 (5), or by Robertson 2009 (15)].	$(q_t/P_a)/\ N_{60} = 8.5\ (1 - I_c/4.6)$ $(q_t/P_a)/\ N_{60} = 10^{(1.1268 - 0.2817Ic)}$ Pa being atmospheric pressure	5 15, 31
(N1)60Ic	SPT N_{60} value corrected for overburden pressure (using N_{60} I_c). User has 3 options.	1) $(N_1)_{60}Ic = C_n \bullet (N_{60}\, I_c)$ 2) $q_{c1n}/\ (N_1)_{60}I_c = 8.5\ (1 - I_c/4.6)$ 3) $(Q_{tn})/\ (N_1)_{60}I_c = 10^{(1.1268 - 0.2817Ic)}$	4 5 15, 31
Su or Su (Nkt)	Undrained shear strength based on q_t S_u factor N_{kt} is user selectable	$Su = \frac{qt - \sigma_v}{N_{kt}}$	1, 5
Su or Su (Ndu)	Undrained shear strength based on pore pressure S_u factor $N_{\Delta u}$ is user selectable	$Su = \frac{u_2 - u_{eq}}{N_{\Delta u}}$	1, 5
Dr	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 ϕ	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/qt	Differential pore pressure ratio (older parameter used before B_q was established)	$= \frac{\Delta u}{qt}$ *where:* $\Delta u = u - u_{eq}$ *and* *u = dynamic pore pressure* u_{eq} *= equilibrium pore pressure*	CK*
Bq	Pore pressure parameter	$Bq = \frac{\Delta u}{qt - \sigma_v}$ where : $\Delta u = u - u_{eq}$ *and* *u = dynamic pore pressure* u_{eq} *= equilibrium pore pressure*	1, 2, 5
Net qt or qtNet	Net tip resistance (used in many subsequent correlations)	$qt - \sigma_v$	CK*
qe	Effective tip resistance (using the dynamic pore pressure u_2 and not equilibrium pore pressure)	$qt - u_2$	CK*

Calculated Parameter	Description	Equation	Ref
qeNorm	Normalized effective tip resistance	$\frac{qt - u_2}{\sigma_v'}$	CK*
Q_t or Norm: Qt	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}.	$Qt = \frac{qt - \sigma_v}{\sigma_v'}$	2, 5
F_r or Norm: Fr	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	$Q \cdot (1 - Bq)$ *where Bq is defined as above and Q is the same as the normalized tip resistance, Q_t, defined above*	6, 7
qc1	Normalized tip resistance, q_{c1}, using a fixed stress ratio exponent, n (this method has stress units)	$q_{c1} = q_t \cdot (Pa/\sigma_v')^{0.5}$ where: Pa = atmospheric pressure	21
qc1 (0.5)	Normalized tip resistance, q_{c1}, using a fixed stress ratio exponent, n (this method is unit-less)	$q_{c1}\ (0.5) = (q_t/P_a) \cdot (Pa/\sigma_v')^{0.5}$ where: Pa = atmospheric pressure	5
qc1 (Cn)	Normalized tip resistance, q_{c1}, based on C_n (this method has stress units)	$q_{c1}(Cn) = C_n * q_t$	5, 12
qc1 (Cq)	Normalized tip resistance, q_{c1}, based on C_q (this method has stress units)	$q_{c1}\ (Cq) = C_q * q_t$ (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 Ic (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	$I_c = [(3.47 - \log_{10}Q)^2 + (\log_{10} Fr + 1.22)^2]^{0.5}$ *Where:* $Q = \left(\frac{qt - \sigma_v}{P_a}\right)\left(\frac{P_a}{\sigma_v'}\right)^n$ *Or* $Q = q_{c1n} = \left(\frac{qt}{P_a}\right)\left(\frac{P_a}{\sigma_v'}\right)^n$ *depending on the iteration in determining I_c* *And Fr is in percent* *P_a = atmospheric pressure* *n varies between 0.5, 0.70 and 1.0 and is selected in an iterative manner based on the resulting I_c*	3, 5, 21
Ic (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 Ic (PKR 2009) and its corresponding n (PKR 2009).	$I_c\ (PKR\ 2009) = [(3.47 - \log_{10}Q_{tn})^2 + (1.22 + \log_{10}F_r)^2]^{0.5}$	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 Ic (PKR 2009).	*n (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 (%)	$FC=1.75(Ic^{3.25}) - 3.7$ $FC=100$ 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 ψ	The state parameter index, ψ, is defined as the difference between the current void ratio, e, and the critical void ratio, e_c. Positive ψ - contractive soil Negative ψ - 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 σ_p'	Yield stress is calculated using the following methods a) General method b) 1st order approximation using q_tNet (clays) c) 1st order approximation using Δu_2 (clays) d) 1st 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 $m' = 1 - \frac{0.28}{1+(I_c/2.65)^{25}}$ 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 plot of S_u/σ_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 Δu e) approximate version based on effective tip, q_e f) approximate version based on shear wave velocity, V_s g) based on Qt	 a) requires a user defined value for NC S_u/P_c' ratio b through f) *based on yield stresses* g) OCR = $0.25\cdot(Qt)^{1.25}$	 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	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: a) OC Sands b) Aged NC Sands c) Recent NC Sands 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 E_s/q_t chart. Es is evaluated for an axial strain of 0.1%.	Mean normal stress is evaluated from: $\sigma'_m = \frac{1}{3}(\sigma'_v + \sigma'_h + \sigma'_h)^3$ where σ_v'= vertical effective stress σ_h'= horizontal effective stress *and* $\sigma_h = K_o \bullet \sigma_v'$ with K_o assumed to be 0.5	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 S_u (N_{kt}) method	$= Su\ (N_{kt}) / \sigma_v'$	CK*
Gmax	G_{max} determined from SCPT shear wave velocities (not estimated values)	$G_{max} = \rho V_s^2$ where ρ 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_{max} determined from SCPT shear wave velocities (not estimated values)	$= (qt - \sigma_v) / G_{max}$ where $G_{max} = \rho V_s^2$ and ρ 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 $(N_1)60$	$K_{SPT} = 1 + ((0.75/30) \bullet (FC – 5))$	10
K_{CPT} or K_C (RW1998)	Equivalent clean sand correction for q_{c1N}	$K_{cpt} = 1.0$ for $I_c \leq 1.64$ $K_{cpt} = f(I_c)$ for $I_c > 1.64$ (see reference) $K_c = – 0.403\, I_c^4 + 5.581\, I_c^3 – 21.63 I_c^2 + 33.75\, I_c – 17.88$	3, 10
Kc (PKR 2010)	Clean sand equivalent factor to be applied to Q_{tn}	$K_c = 1.0$ for $I_c \leq 1.64$ $K_c = – 0.403\, I_c^4 + 5.581\, I_c^3 – 21.63 I_c^2 + 33.75\, I_c – 17.88$ for $I_c > 1.64$	16
$(N_1)_{60cs}Ic$	Clean sand equivalent SPT $(N_1)_{60}I_c$. User has 3 options.	1) $(N_1)_{60cs}Ic = \alpha + \beta((N_1)_{60}I_c)$ 2) $(N_1)_{60cs}Ic = K_{SPT} * ((N_1)_{60}I_c)$ 3) $(q_{c1ncs})/ (N_1)_{60cs}I_c = 8.5\, (1 – I_c/4.6)$ $FC \leq 5\%$: $\alpha = 0$, $\beta=1.0$ $FC \geq 35\%$ $\alpha = 5.0$, $\beta=1.2$ $5\% < FC < 35\%$ $\alpha = exp[1.76 – (190/FC^2)]$ $\beta = [0.99 + (FC^{1.5}/1000)]$	10 10 5
q_{c1ncs}	Clean sand equivalent q_{c1n}	$q_{c1ncs} = q_{c1n} \bullet K_{cpt}$	3
Qtn,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	$\dfrac{Su(Liq)}{\sigma_v'} = 0.03 + 0.0143(q_{c1})$ Note: σ_v' and s_v' are synonymous	13
Su(Liq)/ESv (PKR 2010)	Liquefied shear strength ratio as defined by Robertson (2010)	$\dfrac{Su(Liq)}{\sigma_v'}$ 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
Kg	Small strain Stiffness Ratio Factor, Kg	$[Gmax/qt]/[qc1n^{-m}]$ 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 Ψ = -0.05 curve	25
URS NP Fr	Normalized friction ratio point on Ψ = -0.05 curve used in SP Distance calculation		25
URS NP Qtn	Normalized tip resistance (Qtn) point on Ψ = -0.05 curve used in SP Distance calculation		25

Table 2. References

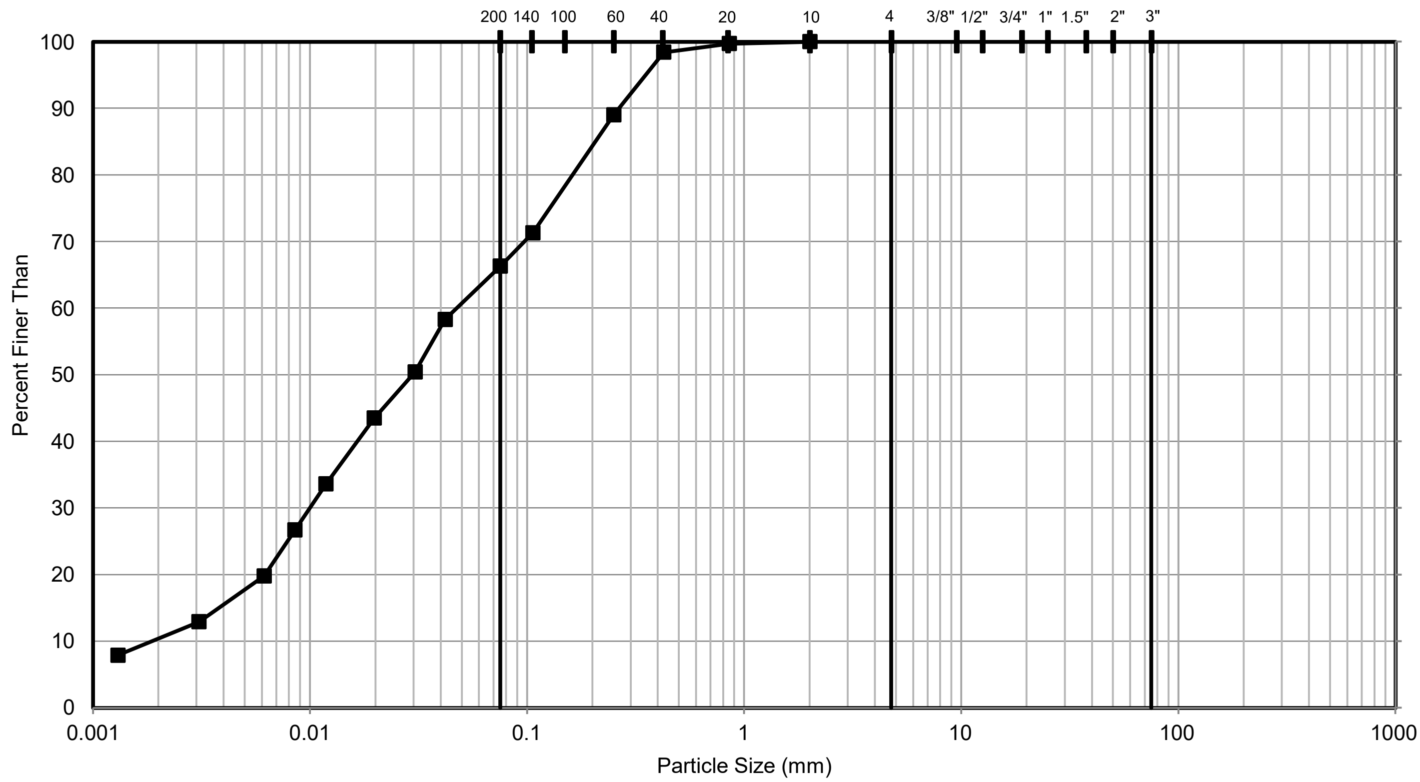
No.	Reference
1	Robertson, P.K., Campanella, R.G., Gillespie, D. and Greig, J., 1986, “Use of Piezometer Cone Data”, Proceedings of InSitu 86, ASCE Specialty Conference, Blacksburg, Virginia.
2	Robertson, P.K., 1990, “Soil Classification Using the Cone Penetration Test”, Canadian Geotechnical Journal, Volume 27. This includes the discussions and replies.
3	Robertson, P.K. and Wride (Fear), C.E., 1998, “Evaluating cyclic liquefaction potential using the cone penetration test”, Canadian Geotechnical Journal, 35: 442-459.
4	Robertson, P.K. and Wride, C.E., 1997, “Cyclic Liquefaction and its Evaluation Based on SPT and CPT”, NCEER Workshop Paper, January 22, 1997.
5	Lunne, T., Robertson, P.K. and Powell, J. J. M., 1997, “ Cone Penetration Testing in Geotechnical Practice,” Blackie Academic and Professional.
6	Plewes, H.D., Davies, M.P. and Jefferies, M.G., 1992, “CPT Based Screening Procedure for Evaluating Liquefaction Susceptibility”, 45th Canadian Geotechnical Conference, Toronto, Ontario, October 1992.
7	Jefferies, M.G. and Davies, M.P., 1993, “Use of CPTu to Estimate equivalent N_{60}”, Geotechnical Testing Journal, 16(4): 458-467.
8	Been, K. and Jefferies, M.P., 1985, “A state parameter for sands”, Geotechnique, 35(2), 99-112.
9	Schmertmann, 1978, “Guidelines for Cone Penetration Test Performance and Design”, Federal Highway Administration Report FHWA-TS-78-209, U.S. Department of Transportation.
10	Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils, Salt Lake City, 1996, chaired by Leslie Youd.
11	Kulhawy, F.H. and Mayne, P.W., 1990, “Manual on Estimating Soil Properties for Foundation Design, Report No. EL-6800″, Electric Power Research Institute, Palo Alto, CA, August 1990, 306 p.
12	Olson, S.M. and Stark, T.D., 2002, “Liquefied strength ratio from liquefied flow failure case histories”, Canadian Geotechnical Journal, 39: 951-966.
13	Olson, Scott M. and Stark, Timothy D., 2003, “Yield Strength Ratio and Liquefaction Analysis of Slopes and Embankments”, Journal of Geotechnical and Geoenvironmental Engineering, ASCE, August 2003.
14	Jamiolkowski, M.B., Lo Presti, D.C.F. and Manassero, M., 2003, “Evaluation of Relative Density and Shear Strength of Sands from CPT and DMT”, Soil Behaviour and Soft Ground Construction, ASCE, GSP N0. 119, 201-238.
15	Robertson, P.K., 2009, “Interpretation of cone penetration tests – a unified approach”, Canadian Geotechnical Journal, 46: 1337-1355.
16	Robertson, P.K., 2010, “Evaluation of Flow Liquefaction and Liquefied Strength Using the Cone Penetration Test”, Journal of Geotechnical and Geoenvironmental Engineering, ASCE, June 2010.
17	Mayne, P.W. and Kulhawy, F.H., 1982, "Ko-OCR Relationships in Soil", Journal of the Geotechnical Engineering Division, ASCE, Vol. 108, GT6, pp. 851-872.
18	Mayne, P.W., Robertson P.K. and Lunne T., 1998, “Clay stress history evaluated from seismic piezocone tests”, Proceedings of the First International Conference on Site Characterization – ISC ’98, Atlanta Georgia, Volume 2, 1113-1118.

No.	Reference
19	Mayne, P.W., 2014, "Generalized CPT Method for Evaluating Yield Stress in Soils", Geocharacterization for Modeling and Sustainability (GSP 235: Proc. GeoCongress 2014, Atlanta, GA), ASCE, Reston, Virginia: 1336-1346.
20	Mayne, P.W., 2015, "Geocharacterization by In-Situ Testing", Continuing Education Course, Vancouver, BC, January 6-8, 2015.
21	Robertson, P.K. and Fear, C.E., 1995, "Liquefaction of sands and its evaluation", Proceedings of the First International Conference on Earthquake Engineering, Keynote Lecture IS Tokyo '95, Tokyo Japan, 1995.
22	Mayne, P.W., Peuchen, J. and Boumeester, D., 2010, "Soil unit weight estimation from CPTs", Proceeding of the 2nd International Symposium on Cone Penetration Testing (CPT '10), Vol 2, Huntington Beach, California; Omnipress: 169-176.
23	Mayne, P.W., 2007, "NCHRP Synthesis 368 on Cone Penetration Test", Transportation Research Board, National Academies Press, Washington, D.C., 118 pages.
24	Mayne, P.W., 2014, "Interpretation of geotechnical parameters from seismic piezocone tests.", Key note address #2, proceedings, 3rd International Symposium on Cone Penetration Testing (CPT'14, Las Vegas), ISSMGE Technical Committee TC102.
25	Winckler, Christina, Davidson, Richard, Yenne, Lisa, Pilz, Jorgen, 2014, "CPTu-Based State Characterization of Tailings Liquefaction Susceptibility", Tailings and Mine Waste, 2014.
25a	Winckler, Christina, Davidson, Richard, Yenne, Lisa, Pilz, Jorgen, 2014, "CPTu-Based State Characterization of Tailings Liquefaction Susceptibility", Powerpoint presentation, Tailings and Mine Waste, 2014.
26	Schneider, J.A. and Moss, R.E.S., 2011, "Linking cyclic stress and cyclic strain based methods for assessment of cyclic liquefaction triggering in sands", Geotechnique Letters 1, 31-36.
27	Rice, A., 1984, "The Seismic Cone Penetrometer", M.A.Sc. thesis submitted to the University of British Columbia, Dept. of Civil Engineering, Vancouver, BC, Canada.
28	Gillespie, D.G., 1990, "Evaluating Shear Wave Velocity and Pore Pressure Data from the Seismic Cone Penetration Test", Ph.D. thesis submitted to the University of British Columbia, Dept. of Civil Engineering, Vancouver, BC, Canada.
29	Robertson, P.K and Cabal, K.L., 2010, "Estimating soil unit weight from CPT", Proceedings of the 2nd International Symposium on Cone Penetration Testing (CPT '10), Huntington Beach, California.
30	Robertson, P.K., 2016, "Cone penetration test (CPT)-based soil behaviour type (SBT) classification system – an update", Canadian Geotechnical Journal, July 2016.
31	Robertson, P.K., 2012, "Interpretation of in-situ tests – some insights", Mitchell Lecture, ISC'4, Recife, Brazil.
32	Robertson, P.K., Cabal, K.L. 2015, "Guide to Cone Penetration Testing for Geotechnical Engineering", 6th Edition.

APPENDIX B

Geotechnical Laboratory Test Results

Grain Size Distribution - Clayey Silt-Silt (CL-ML) to Sandy Clayey Silt-Silt (CL-ML) FILL



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

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	HRE-1	2A	0.8 - 1.2	218.7 to 218.3

CLIENT
AECOM / MTO

PROJECT
Bradford Bypass - Holland River East Branch

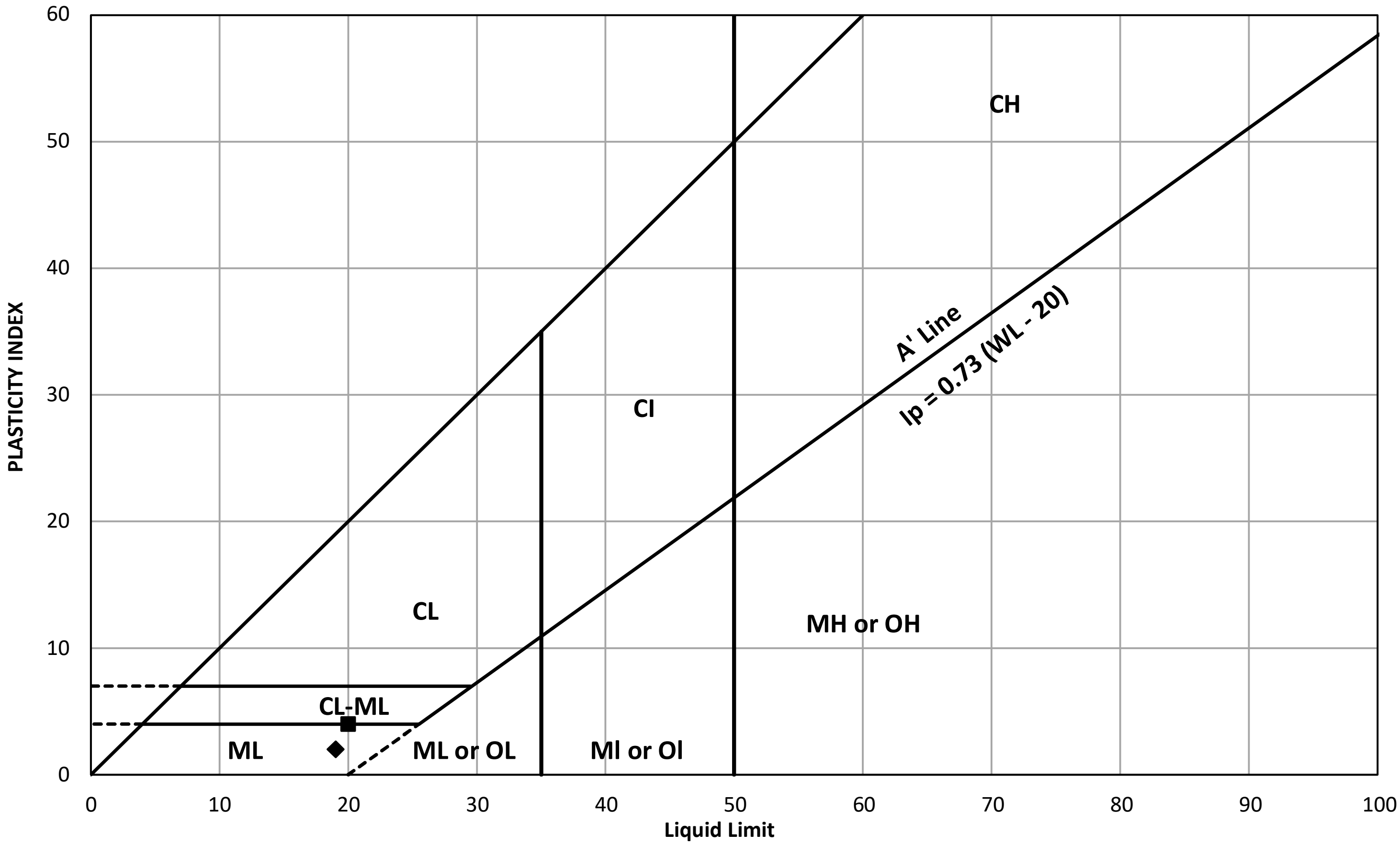
CONSULTANT
WSP GOLDER

YYYY-MM-DD	2022-09-12
DESIGNED	CC
PREPARED	CC
REVIEWED	KJB
APPROVED	KJB

TITLE
Grain Size Distribution
Clayey Silt-Silt (CL-ML) to Sandy Clayey Silt-Silt (CL-ML) FILL

PROJECT NO.	CONTROL	REV.	FIGURE
19136074	0	0	B1

Platicity Chart - Clayey Silt-Silt (CL-ML) to Sandy Clayey Silt-Silt (CL-ML) FILL



Symbol	Sample Location	Sample / Specimen Number	Elevation (m)	Natural Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
■	HRE-1	2A	218.7 to 218.3	24	20	16	4
◆	HRE-2	3	217.4 to 216.8	20.7	19	17	2

CLIENT

MTO

PROJECT

Bradford Bypass - Holland River East Branch

CONSULTANT

GOLDER MEMBER OF WSP

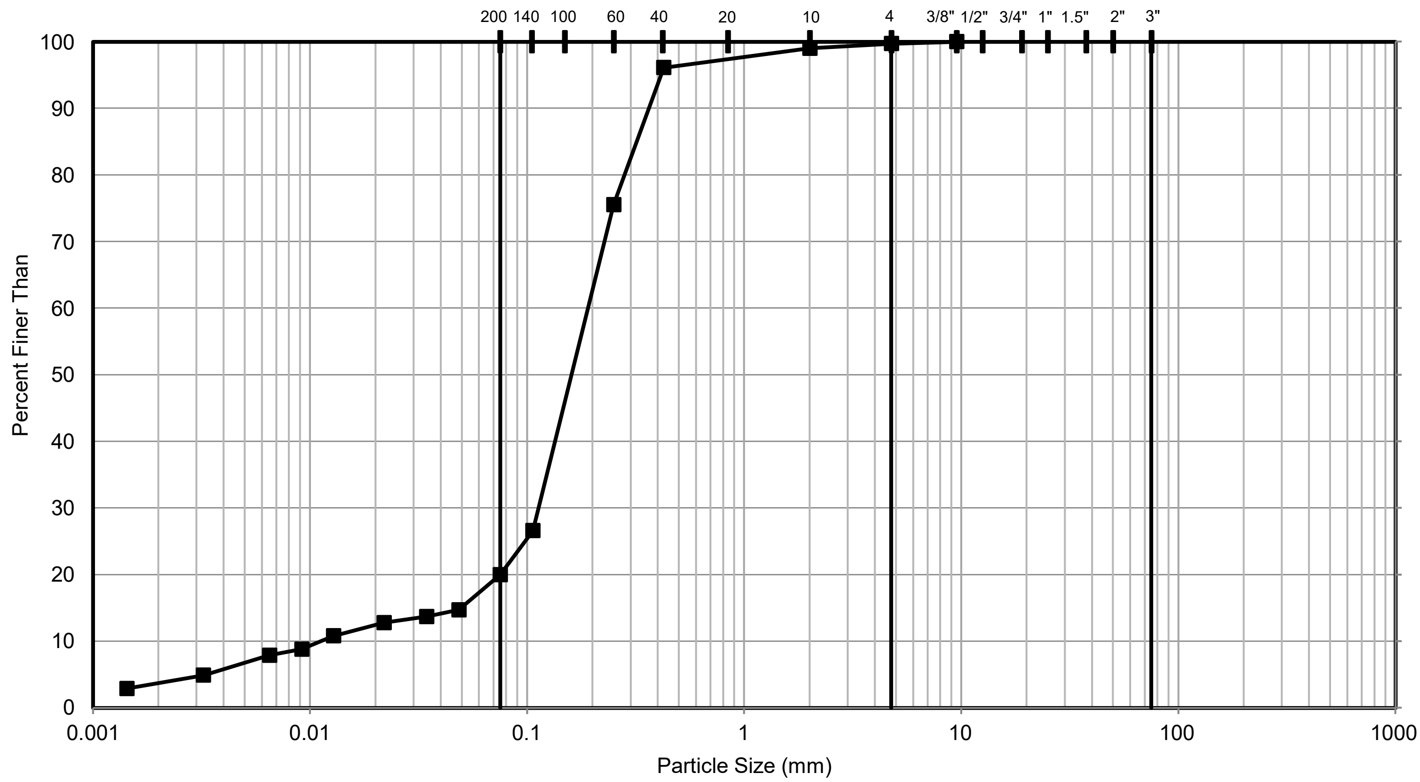
YYYY-MM-DD	2022-09-12
DESIGNED	CC
PREPARED	CC
REVIEWED	KJB
APPROVED	KJB

TITLE

Clayey Silt-Silt (CL-ML) to Sandy Clayey Silt-Silt (CL-ML) FILL

PROJECT NO.	CONTROL	REV.	FIGURE
19136074	0	0	B2

Grain Size Distribution - Silty Sand (SM) FILL



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

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	HRE-3	2	0.8 - 1.4	219.2 to 218.6

CLIENT

AECOM / MTO

PROJECT

Bradford Bypass - Holland River East Branch

CONSULTANT

WSP GOLDER

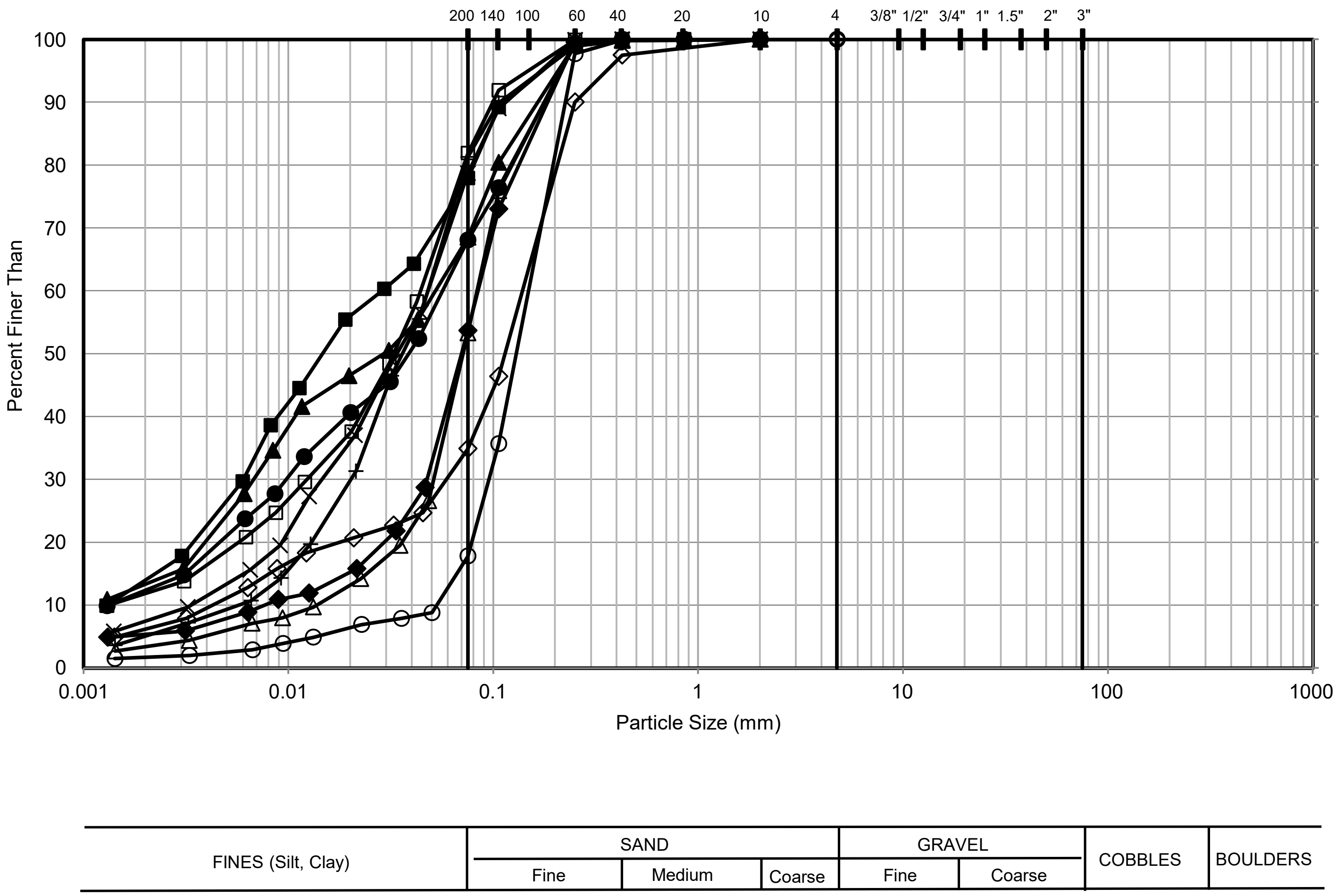
YYYY-MM-DD	2022-09-12
DESIGNED	CC
PREPARED	CC
REVIEWED	KJB
APPROVED	KJB

TITLE

Grain Size Distribution
Silty Sand (SM) FILL

PROJECT NO.	CONTROL	REV.	FIGURE
19136074	0	0	B3

Grain Size Distribution - Silty Sand (SM) to Sandy Silt (ML)



Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	HRE-1	4	2.4 - 3.1	217.0 to 216.4
◆	HRE-1	13B	13.9 - 14.3	205.6 to 205.1
▲	HRE-2	4	2.3 - 2.9	216.6 to 216.0
●	HRE-2	9	7.6 - 8.2	211.3 to 210.7
□	HRE-2	12	12.2 - 12.8	206.7 to 206.1
◇	HRE-3	4	2.3 - 2.9	217.7 to 217.1
△	HRE-3	6	3.8 - 4.4	216.1 to 215.5
○	HRE-3	7	4.6 - 5.2	215.4 to 214.8
×	HRE-3	12	12.2 - 12.8	207.8 to 207.2
+	HRE-3	14	15.2 - 15.9	204.7 to 204.1

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Bradford Bypass - Holland River East Branch

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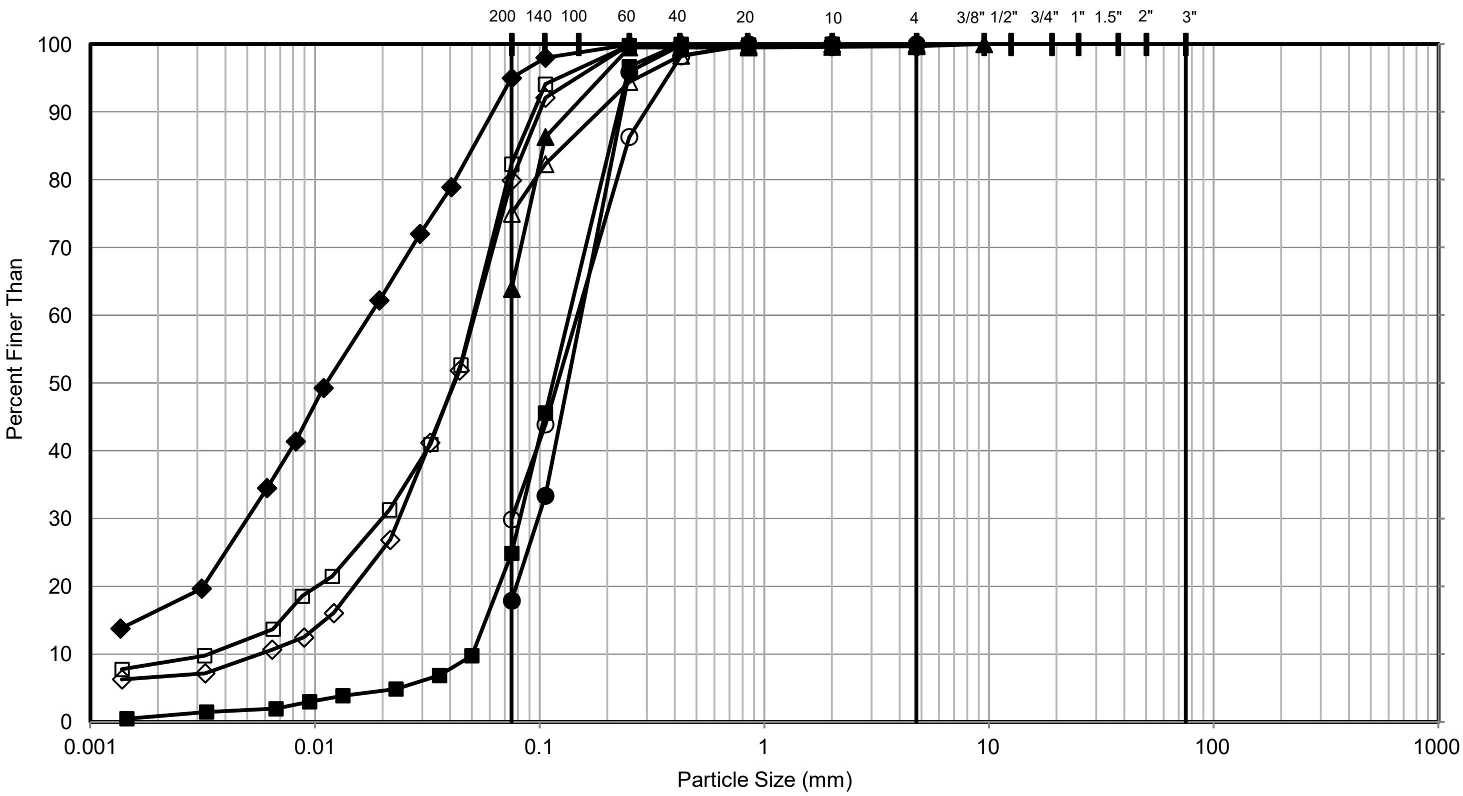
PREPARED	CC
REVIEWED	KJB
APPROVED	KJB

TITLE
Grain Size Distribution
Silty Sand (SM) to Sandy Silt (ML)

PROJECT NO.	CONTROL	REV.	FIGURE
19136074	0	0	B4A

Grain Size Distribution - Silty Sand (SM) to Sandy Silt (ML)



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

Symbol	Location ID	Sample Number	Depth (m)	Elevation (m)
■	HRE-3	17	21.3 - 22.0	198.6 to 198.0
◆	HRE-4	5	3.1 - 3.7	216.9 to 216.2
▲	HRE-4	7	4.6 - 5.2	215.3 to 214.7
●	HRE-4	11A	10.7 - 10.9	209.2 to 209.0
□	HRE-4	13	13.7 - 14.3	206.2 to 205.6
◇	HRE-4	15	16.8 - 17.4	203.1 to 202.5
△	HRE-4	17	21.3 - 22.0	198.6 to 198.0
○	HRE-4	19	27.4 - 28.0	192.5 to 191.9

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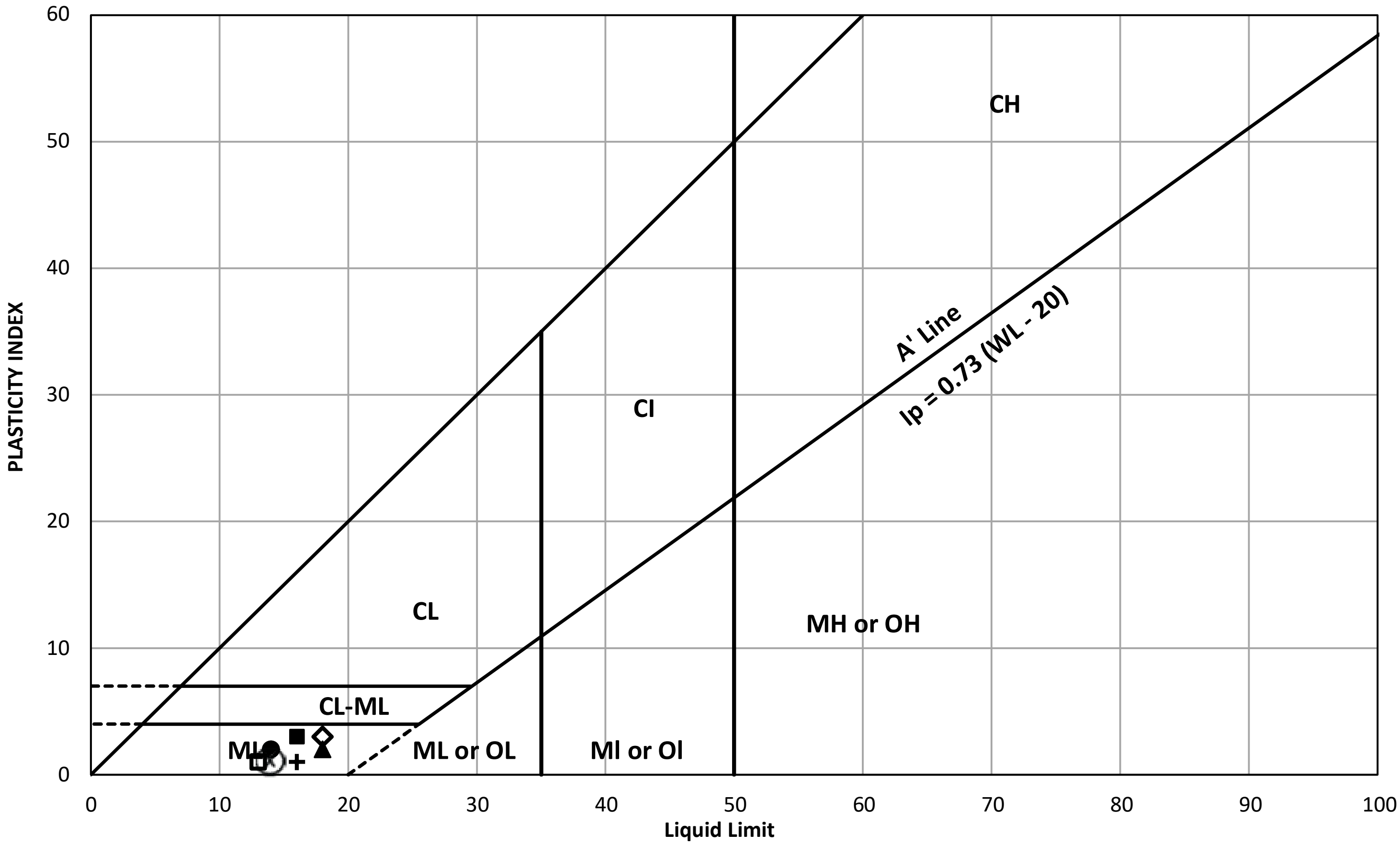
TITLE

Grain Size Distribution
Silty Sand (SM) to Sandy Silt (ML)

PROJECT NO.	CONTROL	REV.	FIGURE
19136074	0	0	B4B

Platicity Chart - Silty Sand (SM) to Sandy Silt (ML)



Symbol	Sample Location	Sample / Specimen Number	Elevation (m)	Natural Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
■	HRE-1	4	217.0 to 216.4	18.1	16	13	3
◆	HRE-2	4	216.6 to 216.0	18.4	14	12	2
▲	HRE-3	5	216.9 to 216.3	23.1	18	16	2
●	HRE-3	9	212.3 to 211.7	19.5	14	12	2
+	HRE-3	10	210.8 to 210.2	17.1	16	15	1
⊗	HRE-3	12	207.8 to 207.2	19.8	14	13	1
□	HRE-3	13	206.2 to 205.6	18.4	13	12	1
◇	HRE-4	5	216.9 to 216.2	17.9	18	15	3

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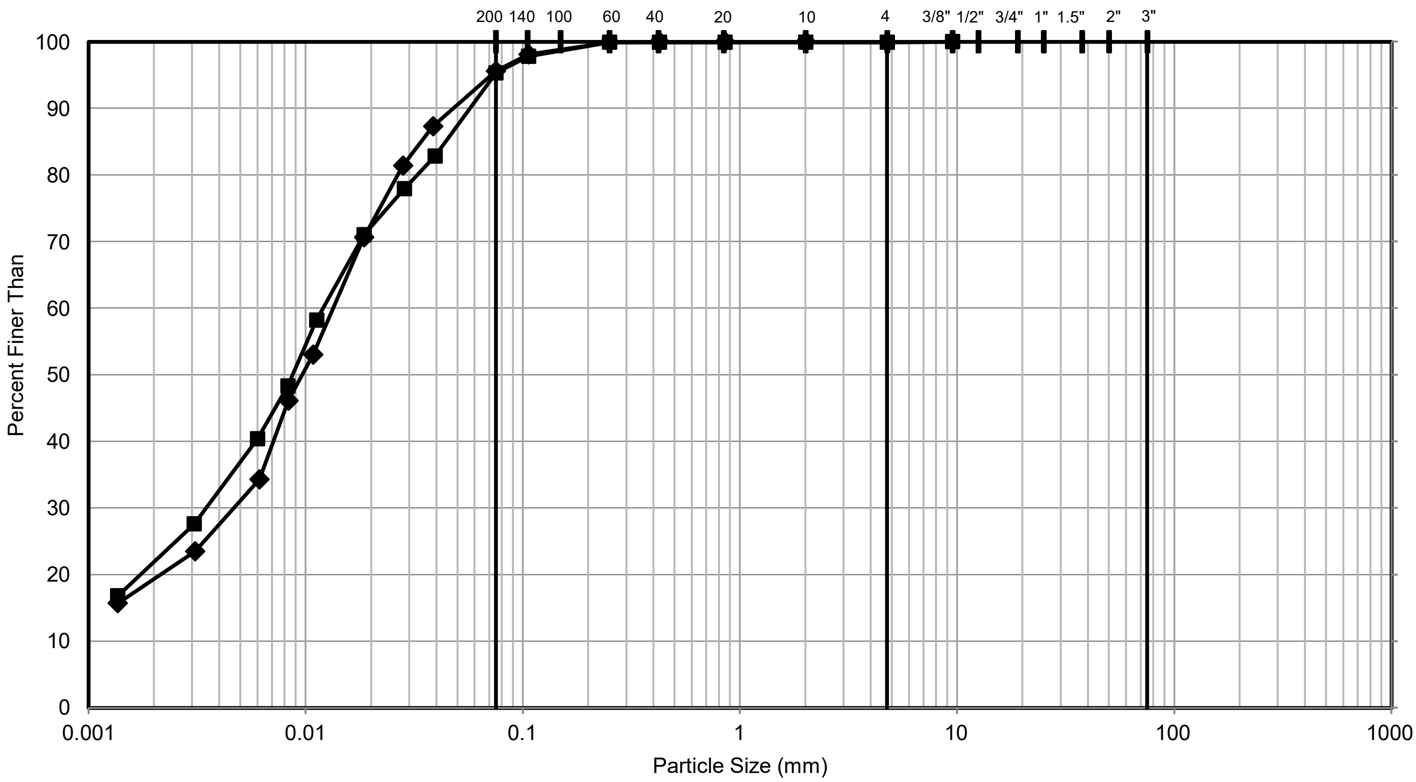
Bradford Bypass - Holland River East Branch

TITLE

Silty Sand (SM) to Sandy Silt (ML)

PROJECT NO.	CONTROL	REV.	FIGURE
19136074	0	0	B5

Grain Size Distribution - Clayey Silt-Silt (CL-ML) to Sandy Clayey Silt-Silt (CL-ML) - Interlayer



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

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	HRE-4	8A	6.1 - 6.5	213.8 to 213.5
◆	HRE-4	11B	10.9 - 11.3	209.0 to 208.6

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Bradford Bypass - Holland River East Branch

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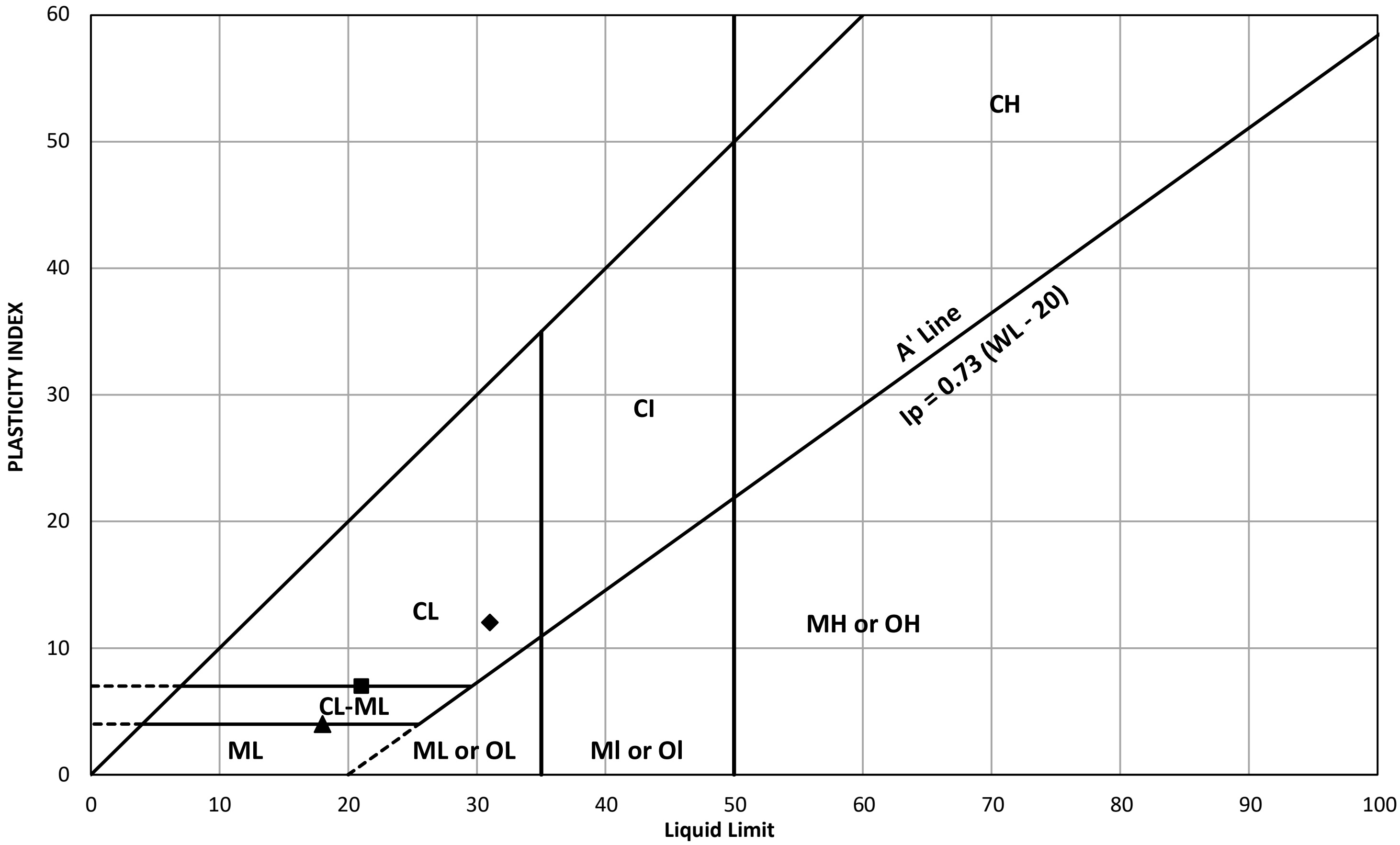
TITLE

Grain Size Distribution
Clayey Silt-Silt (CL-ML) to Sandy Clayey Silt-Silt (CL-ML) -

PROJECT NO.	CONTROL	REV.	FIGURE
19136074	0	0	B6

Platicity Chart - Clayey Silt-Silt (CL-ML) to Sandy Clayey Silt-Silt (CL-ML) - Interlayer



Symbol	Sample Location	Sample / Specimen Number	Elevation (m)	Natural Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
■	HRE-2	8B	212.5 to 212.2	19.8	21	14	7
◆	HRE-4	2	219.1 to 218.5	28	31	19	12
▲	HRE-4	11B	209.0 to 208.6	19.4	18	14	4

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YYYY-MM-DD	2022-09-12
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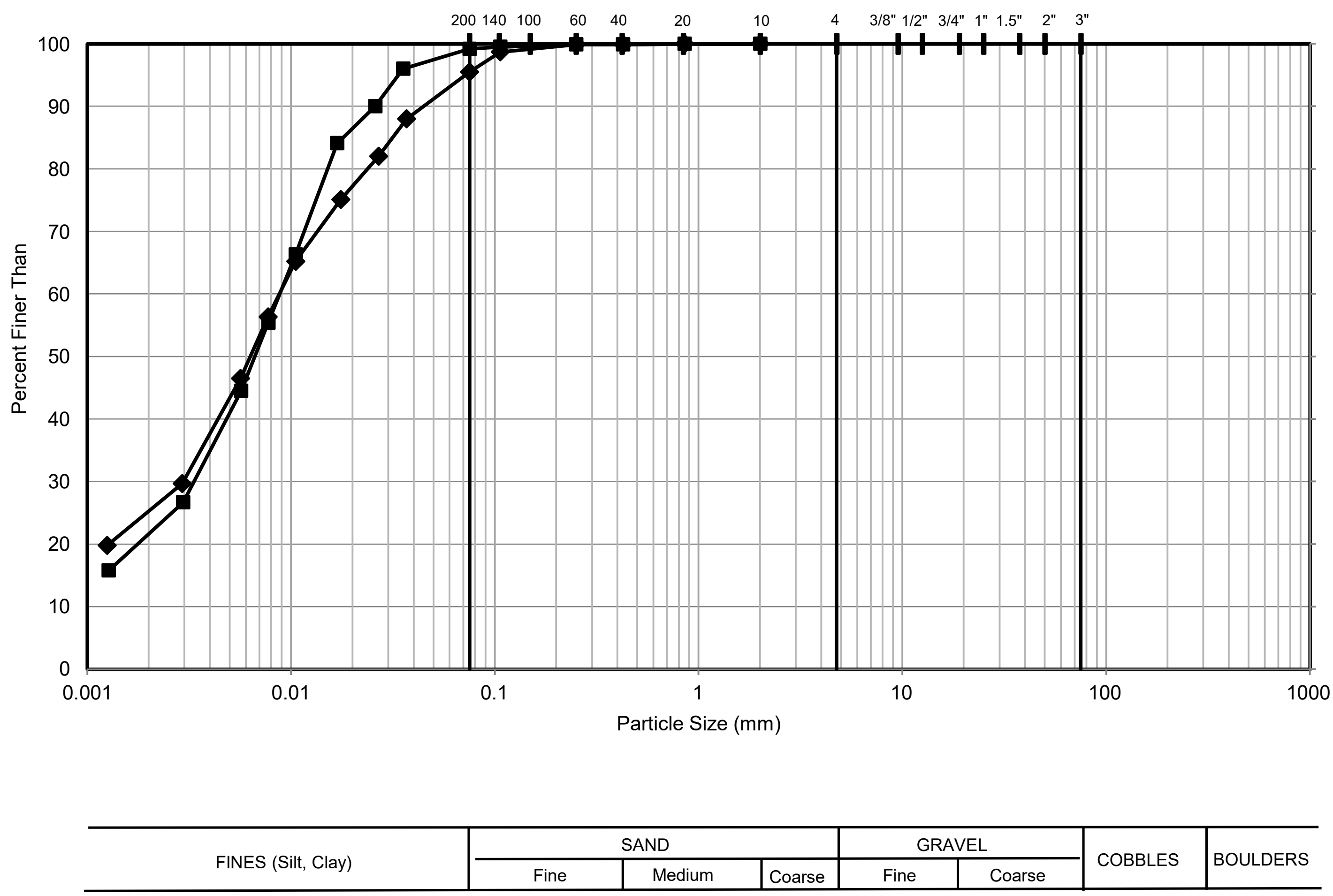
PREPARED	CC
REVIEWED	KJB
APPROVED	KJB

TITLE
Clayey Silt-Silt (CL-ML) to Sandy Clayey Silt-Silt (CL-ML) - Interlayer

PROJECT NO.	CONTROL	REV.	FIGURE
19136074	0	0	B7

Grain Size Distribution - Clayey Silt (CL) to Clayey Silt-Silt (CL-ML)



Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	HRE-1	7	4.6 - 5.2	214.9 to 214.3
◆	HRE-1	11	10.7 - 11.3	208.8 to 208.2

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Bradford Bypass - Holland River East Branch

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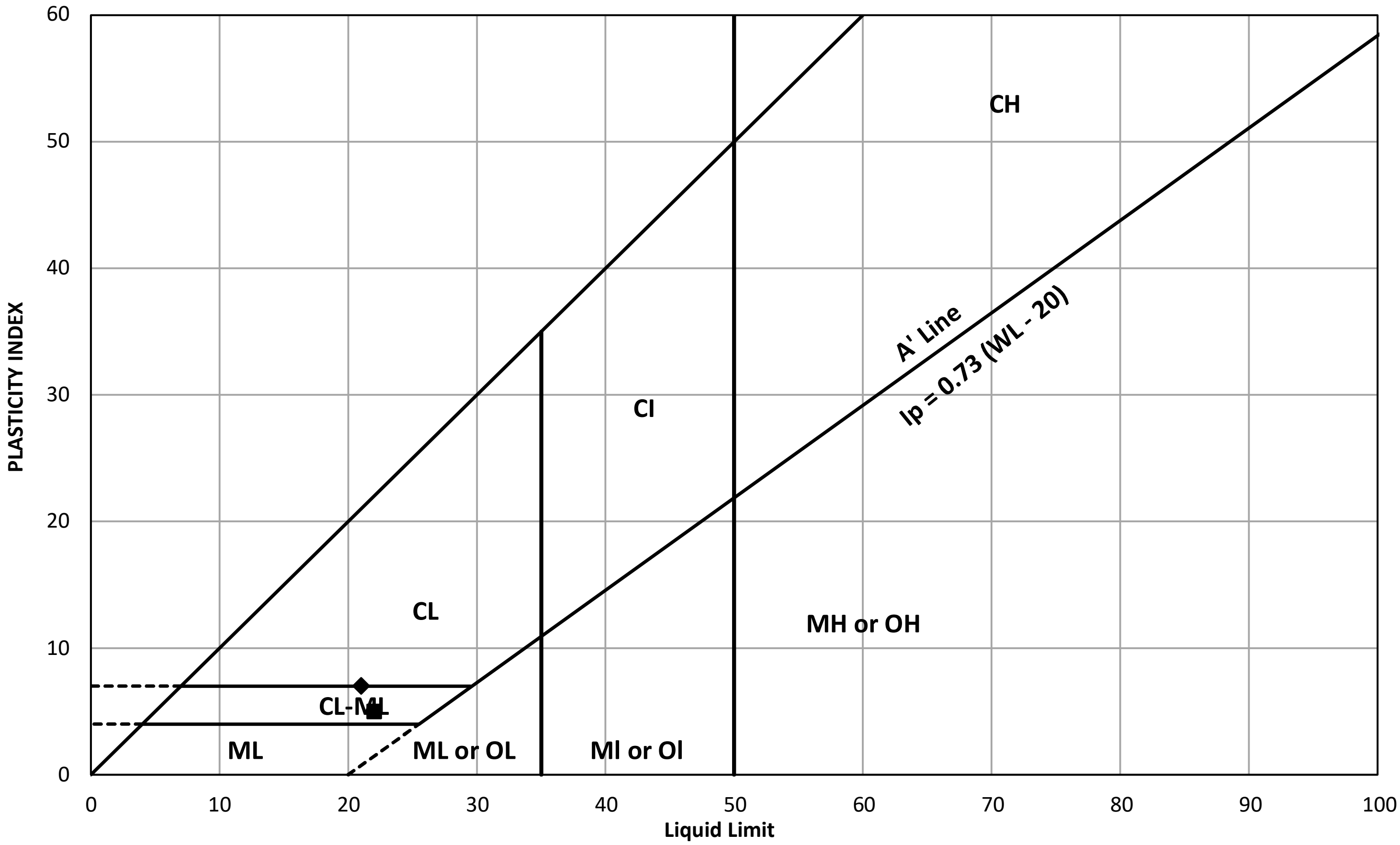
YYYY-MM-DD	2022-09-12
DESIGNED	CC
PREPARED	CC
REVIEWED	KJB
APPROVED	KJB

TITLE

Grain Size Distribution
Clayey Silt (CL) to Clayey Silt-Silt (CL-ML)

PROJECT NO.	CONTROL	REV.	FIGURE
19136074	0	0	B8

Platicity 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
■	HRE-1	7	214.9 to 214.3	24.3	22	17	5
◆	HRE-1	11	208.8 to 208.2	17.2	21	14	7

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APPROVED	KJB

TITLE

Clayey Silt (CL) to Clayey Silt-Silt (CL-ML)

PROJECT NO.	CONTROL	REV.	FIGURE
19136074	0	0	B9

CONSOLIDATION TEST SUMMARY

ASTM D2435/D2435M

FIGURE B10

SAMPLE IDENTIFICATION

Project Number	19136074	Sample Number	6
Borehole Number	HRE-2	Sample Depth, m	3.81-4.42

TEST CONDITIONS

Test Type	Laboratory Standard	Load Duration, hr	24
Oedometer Number	9		
Date Started	01/21/2022		
Date Completed	02/03/2022		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.91	Unit Weight, kN/m^3	20.17
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m^3	16.55
Area, cm^2	31.53	Specific Gravity, measured	2.73
Volume, cm^3	60.10	Solids Height, cm	1.178
Water Content, %	21.91	Volume of Solids, cm^3	37.14
Wet Mass, g	123.62	Volume of Voids, cm^3	22.95
Dry Mass, g	101.4	Degree of Saturation, %	96.8

TEST COMPUTATIONS

Stress kPa	Corr. Height cm	Void Ratio	Average Height cm	t_{90} sec	cv. cm^2/s	mv m^2/kN	k cm/s
0.00	1.906	0.618	1.906				
6.39	1.906	0.618	1.906				
11.21	1.905	0.617	1.906	145	5.31E-03	3.59E-05	1.87E-08
21.04	1.902	0.615	1.904	125	6.15E-03	1.78E-04	1.07E-07
30.53	1.899	0.612	1.900	113	6.78E-03	1.78E-04	1.18E-07
11.23	1.900	0.613	1.899				
30.53	1.899	0.612	1.899	83	9.21E-03	3.43E-05	3.09E-08
40.45	1.896	0.609	1.897	97	7.87E-03	1.62E-04	1.25E-07
79.49	1.886	0.601	1.891	98	7.73E-03	1.25E-04	9.49E-08
157.02	1.874	0.591	1.880	70	1.07E-02	8.18E-05	8.58E-08
312.20	1.858	0.577	1.866	231	3.20E-03	5.48E-05	1.72E-08
622.78	1.835	0.558	1.847	167	4.33E-03	3.84E-05	1.63E-08
1244.14	1.801	0.529	1.818	107	6.55E-03	2.87E-05	1.84E-08
2488.04	1.759	0.493	1.780	83	8.09E-03	1.77E-05	1.40E-08
622.78	1.764	0.498	1.762				
157.09	1.774	0.506	1.769				
40.51	1.781	0.512	1.778				
11.20	1.794	0.523	1.788				

Note:
Consolidation loading and unloading schedule assigned by the client.
cv and k are approximate only based on t_{90} estimated from Square Root of Time Method (ASTMD2435/2435M)
Specimen swelled under 6.39kPa

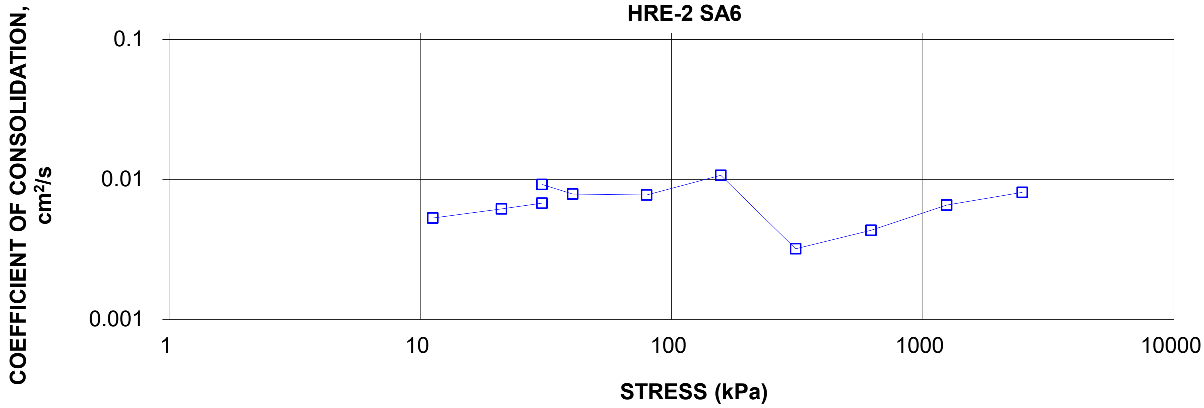
SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.79	Unit Weight, kN/m^3	20.87
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m^3	17.58
Area, cm^2	31.53	Specific Gravity, measured	2.73
Volume, cm^3	56.57	Solids Height, cm	1.178
Water Content, %	18.74	Volume of Solids, cm ^{3}	37.14
Wet Mass, g	120.40	Volume of Voids, cm ^{3}	19.43
Dry Mass, g	101.4		

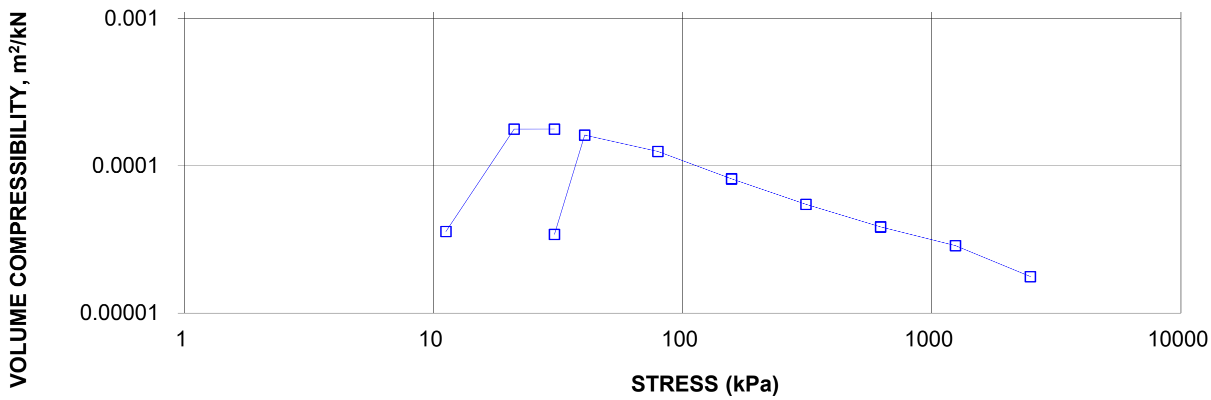
Prepared By: LH **Golder Associates** Checked By: MM

CONSOLIDATION TEST SUMMARY	FIGURE B11

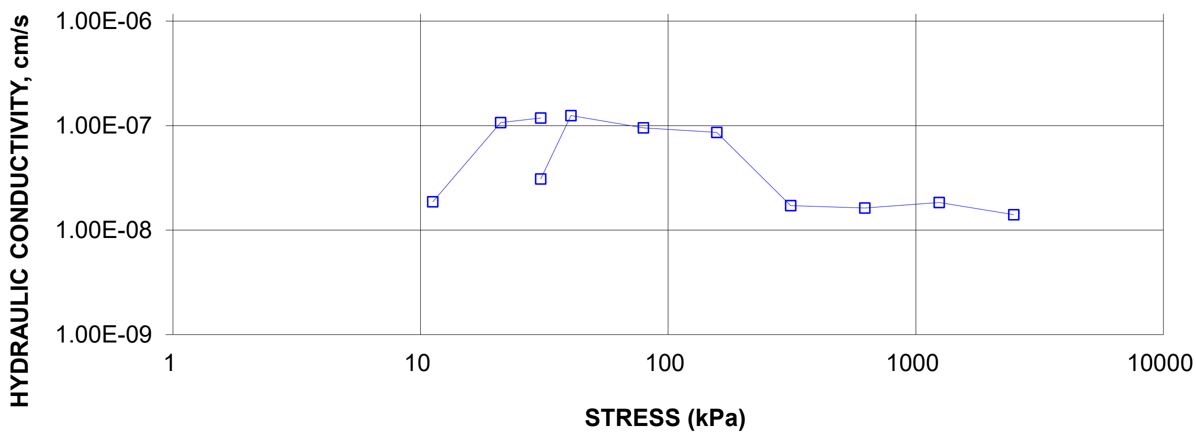
CONSOLIDATION TEST
CV cm²/s VS STRESS (kPa)
HRE-2 SA6



CONSOLIDATION TEST
MV m²/kN vs STRESS (kPa)
HRE-2 SA6



CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs STRESS
HRE-2 SA6



Project No. 19136074
Prepared By: LH

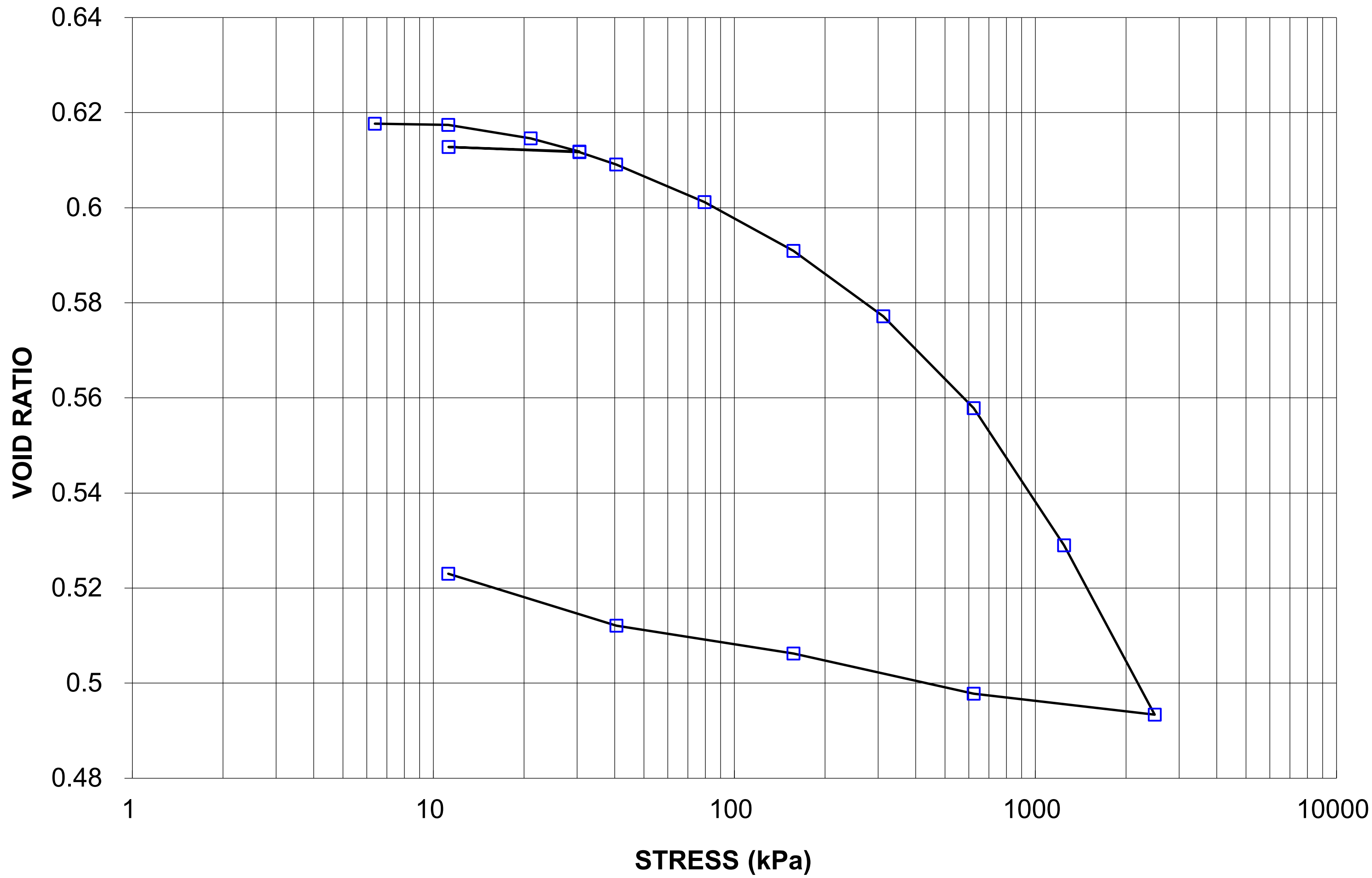
Golder Associates

Checked By: MM

Project No. 19136074

Prepared By: LH

CONSOLIDATION TEST
VOID RATIO vs STRESS
HRE-2 SA6



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CONSOLIDATION TEST
VOID RATIO VS LOG STRESS

FIGURE B12

Project No. 19136074
Prepared By: LH

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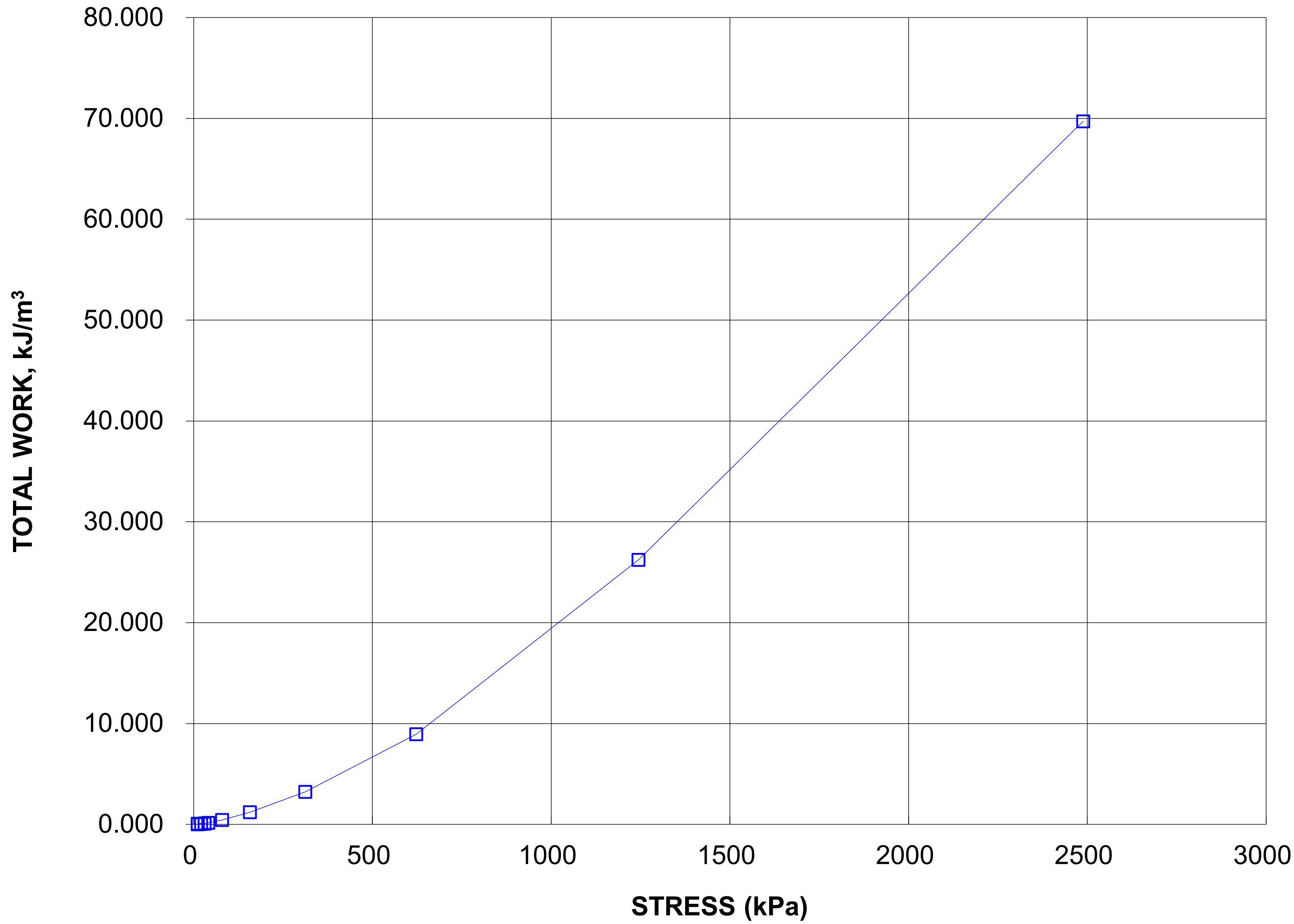
CONSOLIDATION TEST
TOTAL WORK, kJ/m³ vs STRESS
HRE-2 SA6



Checked By: MM

CONSOLIDATION TEST
TOTAL WORK VS STRESS

FIGURE B13

CONSOLIDATION TEST SUMMARY

FIGURE 14

ASTM D2435/D2435M

SAMPLE IDENTIFICATION

Project Number	19136074	Sample Number	4
Borehole Number	HRE1	Sample Depth, m	4.57-5.18

TEST CONDITIONS

Test Type	Laboratory Standard	Load Duration, hr	24
Oedometer Number	1		
Date Started	11/3/2021		
Date Completed	11/17/2021		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.53	Unit Weight, kN/m^3	20.04
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m^3	16.21
Area, cm^2	31.61	Specific Gravity, measured	2.71
Volume, cm^3	80.07	Solids Height, cm	1.545
Water Content, %	23.62	Volume of Solids, cm^3	48.83
Wet Mass, g	163.58	Volume of Voids, cm^3	31.24
Dry Mass, g	132.33	Degree of Saturation, %	100.0

TEST COMPUTATIONS

Stress kPa	Corr. Height cm	Void Ratio	Average Height cm	t_{90} sec	cv. cm^2/s	mv m^2/kN	k cm/s
0.00	2.533	0.640	2.533				
5.88	2.526	0.635	2.529	34	3.99E-02	4.97E-04	1.94E-06
10.68	2.518	0.630	2.522	60	2.25E-02	6.00E-04	1.32E-06
20.70	2.508	0.624	2.513	43	3.11E-02	4.06E-04	1.24E-06
40.13	2.492	0.613	2.500	57	2.32E-02	3.29E-04	7.50E-07
78.70	2.471	0.600	2.481	66	1.98E-02	2.13E-04	4.13E-07
39.93	2.471	0.599	2.471				
10.74	2.474	0.601	2.472				
39.93	2.471	0.599	2.472	49	2.64E-02	4.33E-05	1.12E-07
78.82	2.465	0.596	2.468	38	3.40E-02	5.58E-05	1.86E-07
156.15	2.440	0.579	2.452	73	1.75E-02	1.31E-04	2.25E-07
310.96	2.401	0.554	2.420	54	2.30E-02	9.79E-05	2.21E-07
620.72	2.360	0.528	2.381	60	2.00E-02	5.20E-05	1.02E-07
1240.27	2.316	0.499	2.338	58	2.00E-02	2.85E-05	5.59E-08
2479.94	2.266	0.467	2.291	66	1.69E-02	1.58E-05	2.60E-08
620.72	2.272	0.471	2.269				
156.22	2.283	0.478	2.277				
39.93	2.289	0.482	2.286				
10.74	2.304	0.492	2.297				

Note:
Consolidation loading and unloading schedule assigned by the client.
cv and k are approximate only based on t_{90} estimated from Square Root of Time Method (ASTMD2435/2435M)
Specimen taken 38-43cm from top of the tube.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

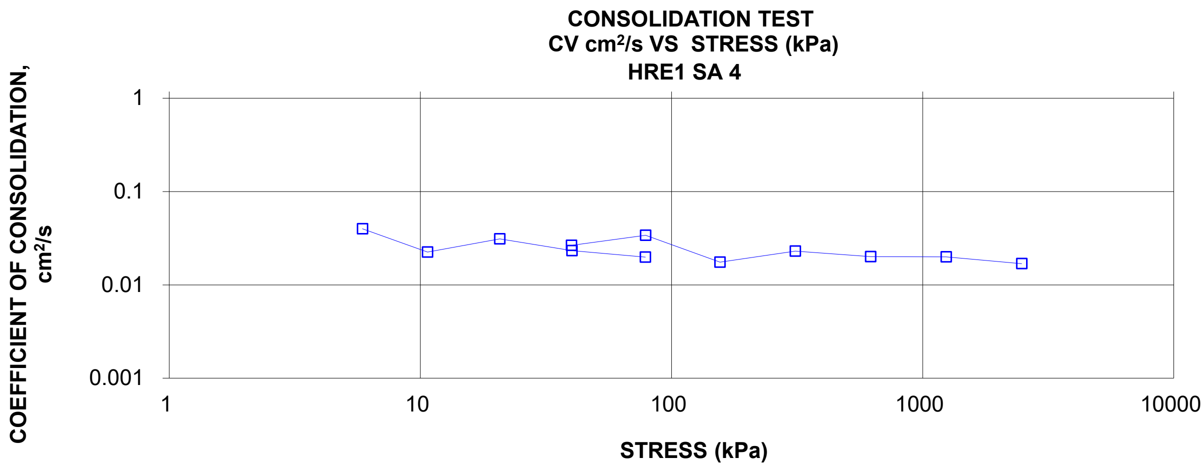
Sample Height, cm	2.30	Unit Weight, kN/m^3	21.27
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m^3	17.82
Area, cm^2	31.61	Specific Gravity, measured	2.71
Volume, cm^3	72.84	Solids Height, cm	1.545
Water Content, %	19.37	Volume of Solids, cm ^{3}	48.83
Wet Mass, g	157.96	Volume of Voids, cm ^{3}	24.01
Dry Mass, g	132.33		

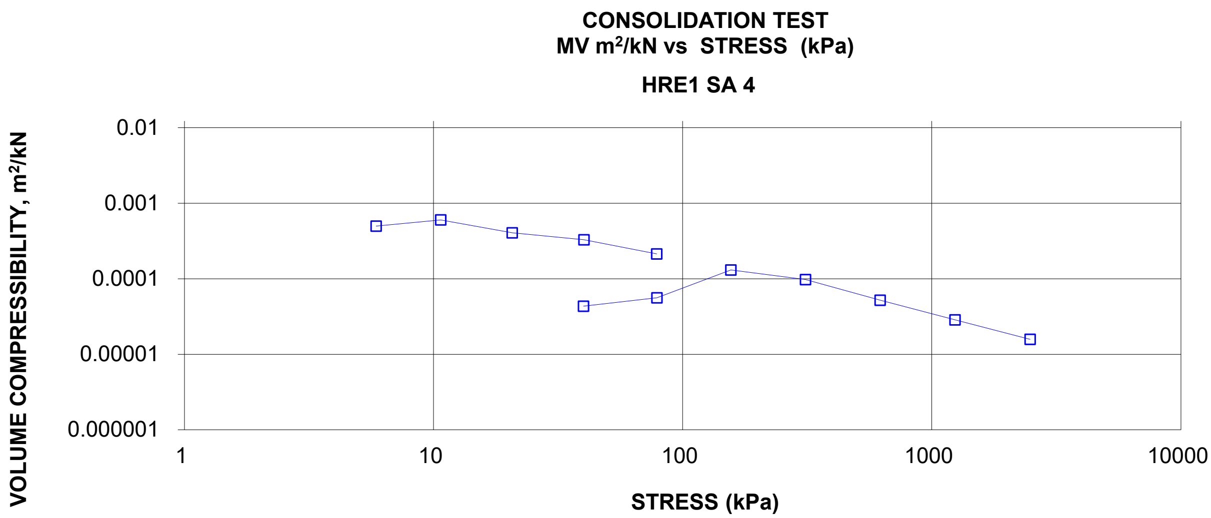
Prepared By: LH

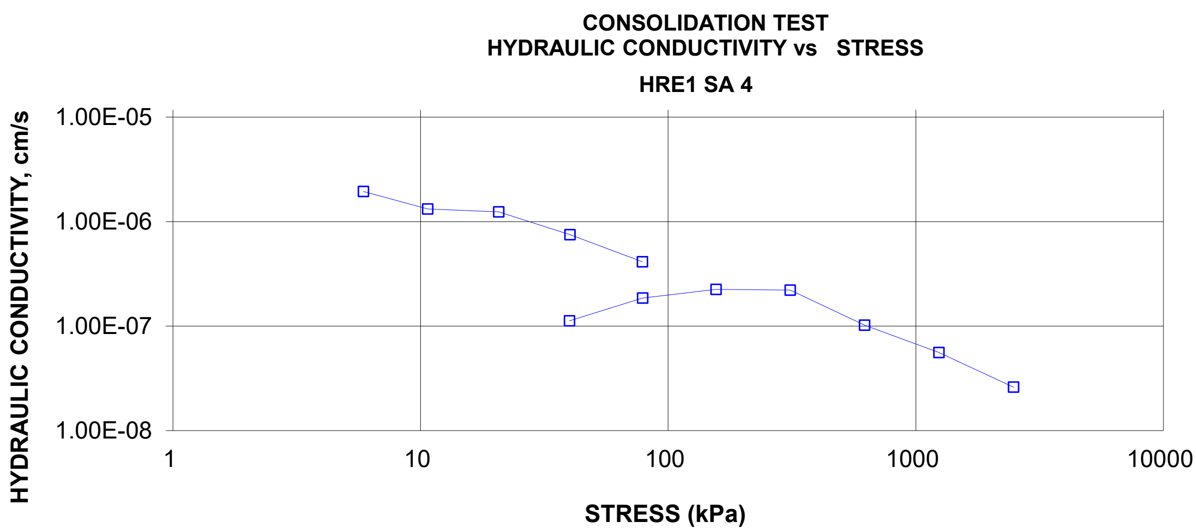
Golder Associates

Checked By: MM

CONSOLIDATION TEST SUMMARY	FIGURE 15







Project No. 19136074

Prepared By: LH

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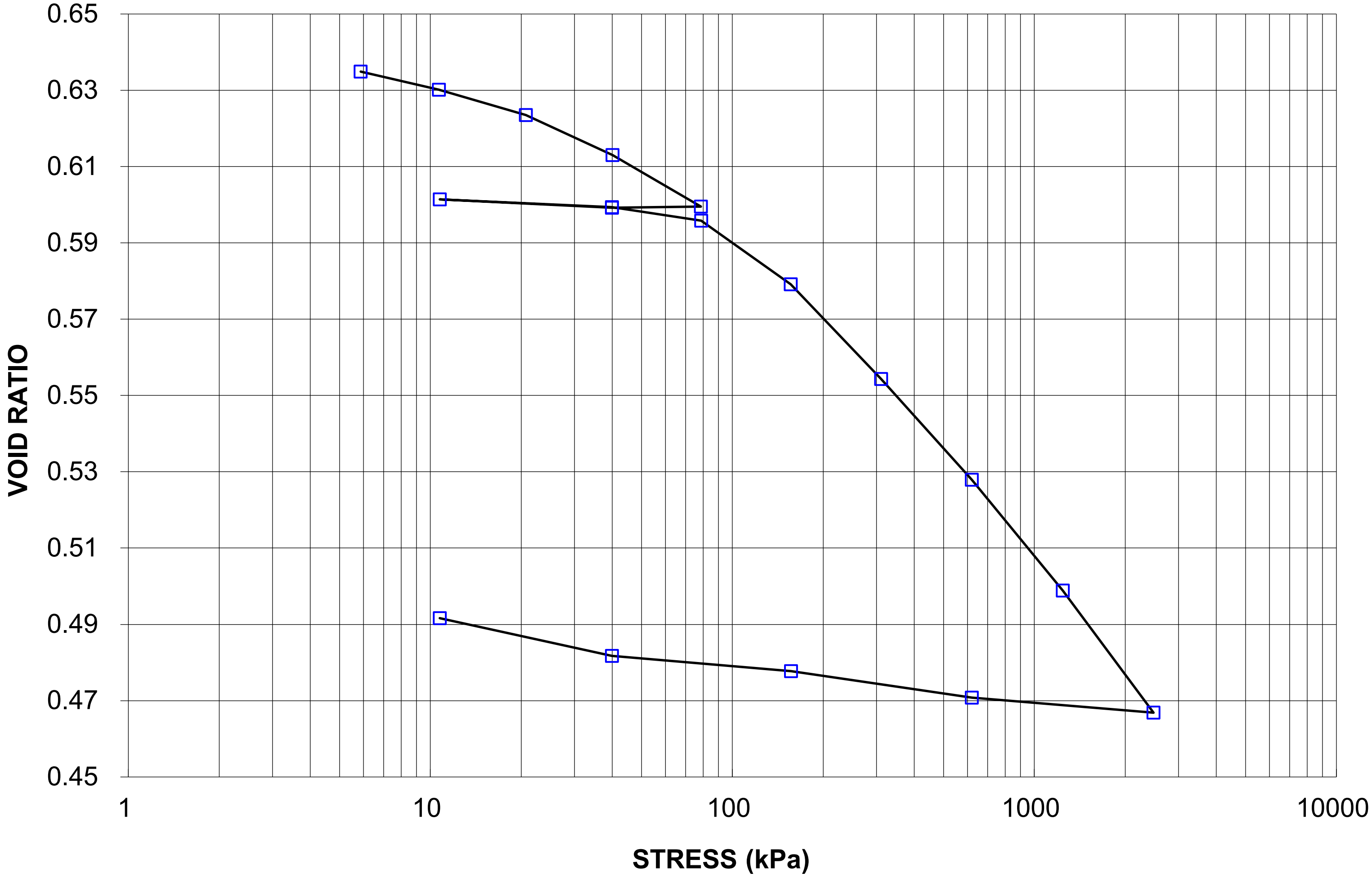
Project No. 19136074
Prepared By: LH

CONSOLIDATION TEST
VOID RATIO vs STRESS
HRE1 SA 4



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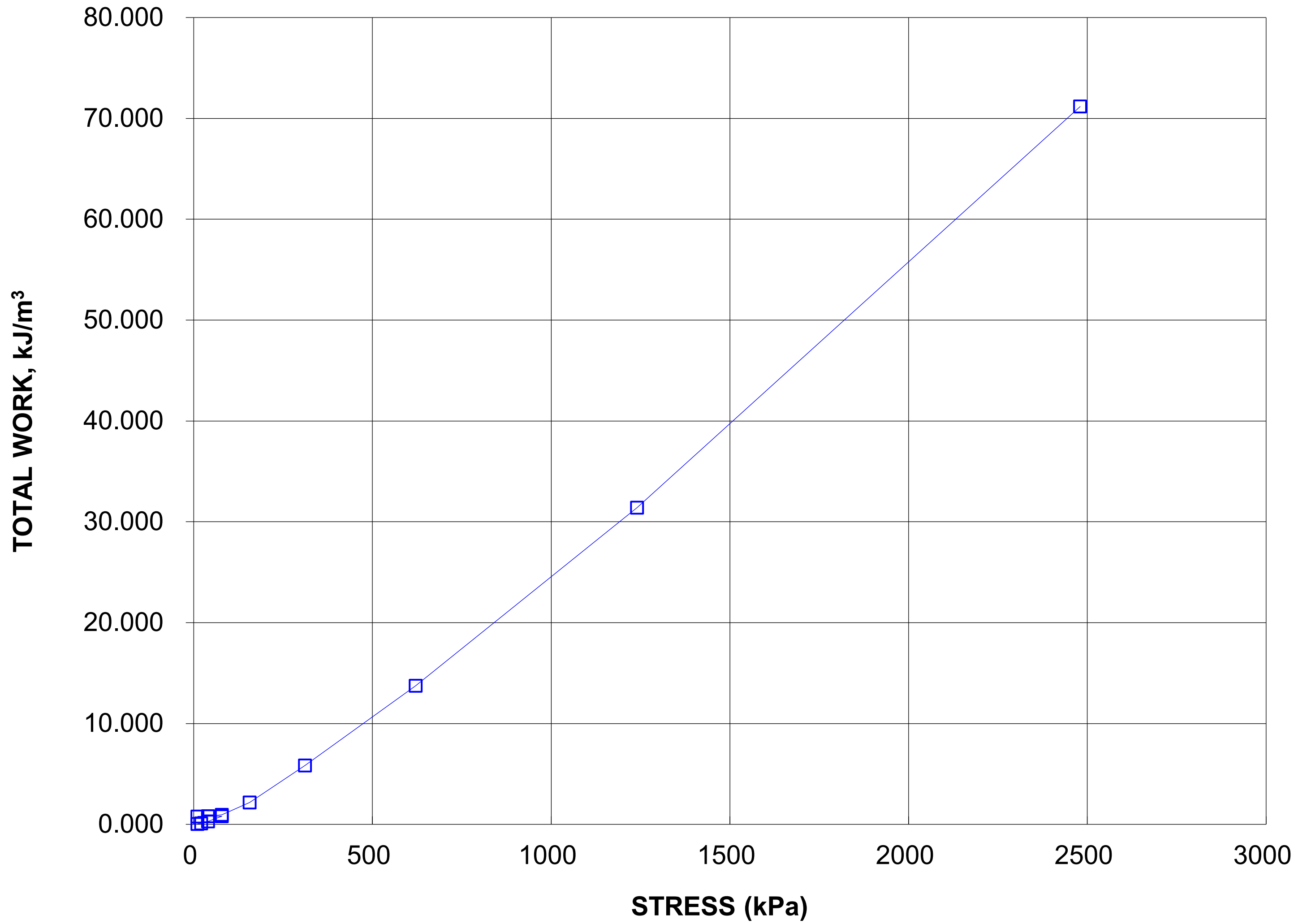
CONSOLIDATION TEST
VOID RATIO VS LOG STRESS

FIGURE 16

Project No.19136074
Prepared By: LH

CONSOLIDATION TEST
TOTAL WORK, kJ/m³ vs STRESS
HRE1 SA 4



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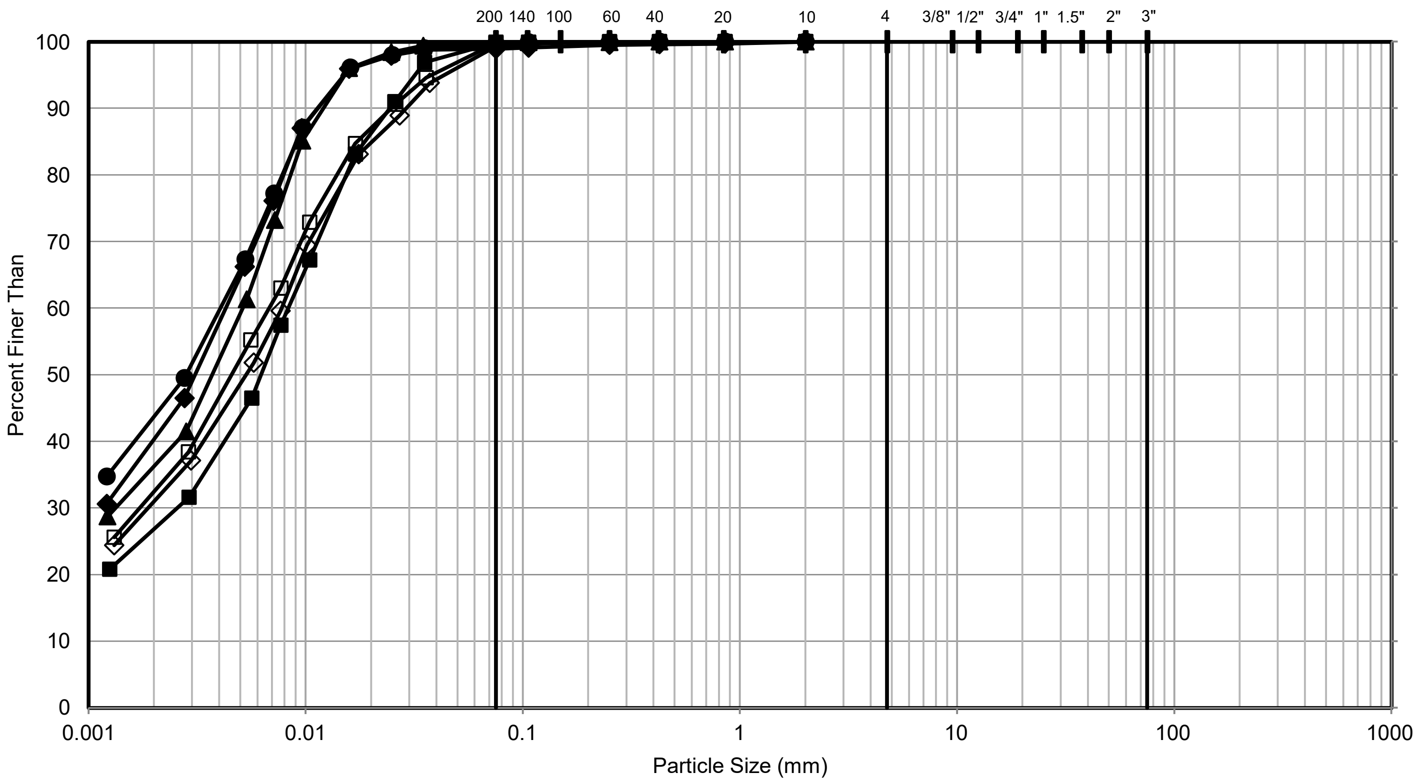
CONSOLIDATION TEST
TOTAL WORK VS STRESS

FIGURE 17

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)
■	HRE-1	19	27.4 - 28.0	192.0 to 191.4
◆	HRE-1	24	39.6 - 40.2	179.8 to 179.2
▲	HRE-1	26	45.7 - 46.3	173.7 to 173.1
●	HRE-2	19	27.4 - 28.0	191.5 to 190.9
□	HRE-3	22	36.6 - 37.2	183.4 to 182.8
◇	HRE-4	21	33.5 - 34.1	186.4 to 185.8

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AECOM / MTO

PROJECT
Bradford Bypass - Holland River East Branch

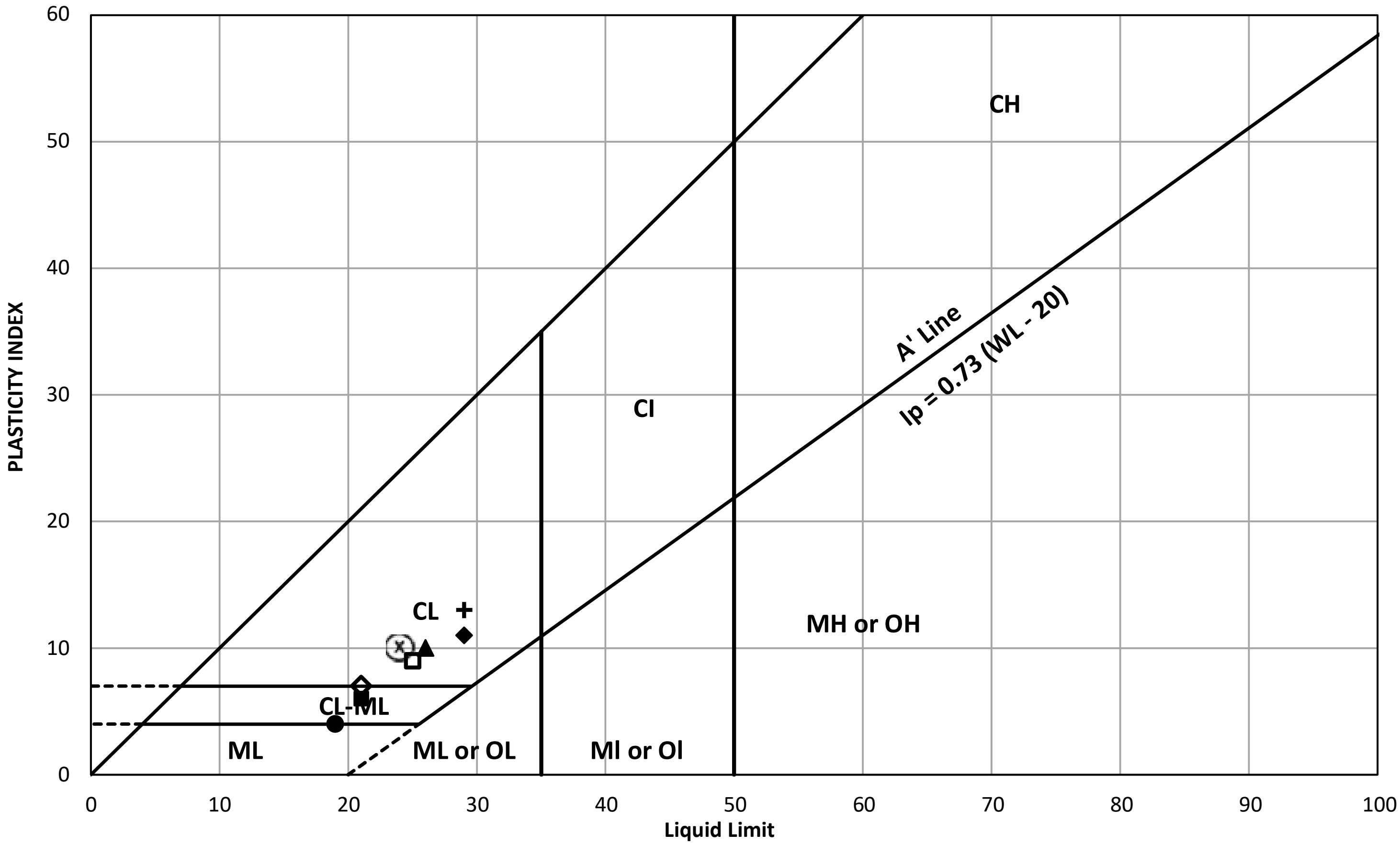
CONSULTANT
WSP GOLDER

YYYY-MM-DD	2022-09-12
DESIGNED	CC
PREPARED	CC
REVIEWED	KJB
APPROVED	KJB

TITLE
Grain Size Distribution
Clayey Silt (CL) to Clayey Silt-Silt (CL-ML)

PROJECT NO.	CONTROL	REV.	FIGURE
19136074	0	0	B18

Platicity 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
■	HRE-1	19	192.0 to 191.4	22.2	21	15	6
◆	HRE-1	24	179.8 to 179.2	23.8	29	18	11
▲	HRE-1	26	173.7 to 173.1	42	26	16	10
●	HRE-2	17	197.6 to 197.0	20.2	19	15	4
+	HRE-2	19	191.5 to 190.9	26.1	29	16	13
⊗	HRE-3	21	186.4 to 185.8	26.8	24	14	10
□	HRE-3	27	171.2 to 170.6	21.8	25	16	9
◇	HRE-4	21	186.4 to 185.8	21.6	21	14	7
△	HRE-4	27	168.1 to 167.5	21.1	21	15	6

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MTO

PROJECT
Bradford Bypass - Holland River East Branch

CONSULTANT
GOLDER
MEMBER OF WSP

YYYY-MM-DD	2022-09-12
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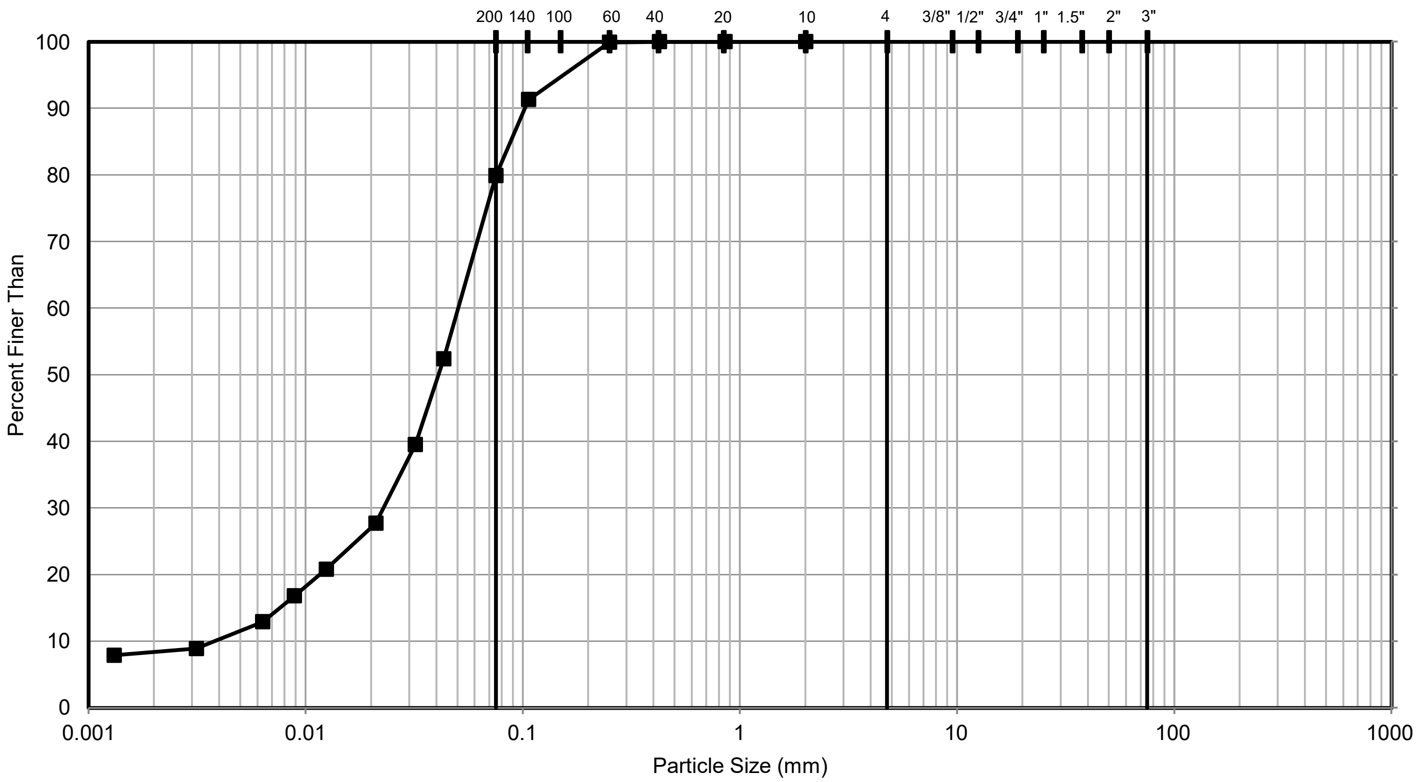
PREPARED	CC
REVIEWED	KJB
APPROVED	KJB

TITLE
Clayey Silt (CL) to Clayey Silt Silt (CL-ML)

PROJECT NO.	CONTROL	REV.	FIGURE
19136074	0	0	B19

Grain Size Distribution - Sandy Silt (ML) to Silt (ML) - Interlayer



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

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	HRE-2	15	16.8 - 17.4	202.2 to 201.6

CLIENT

AECOM / MTO

PROJECT

Bradford Bypass - Holland River East Branch

CONSULTANT

WSP GOLDER

YYYY-MM-DD	2022-09-12
DESIGNED	CC
PREPARED	CC
REVIEWED	KJB
APPROVED	KJB

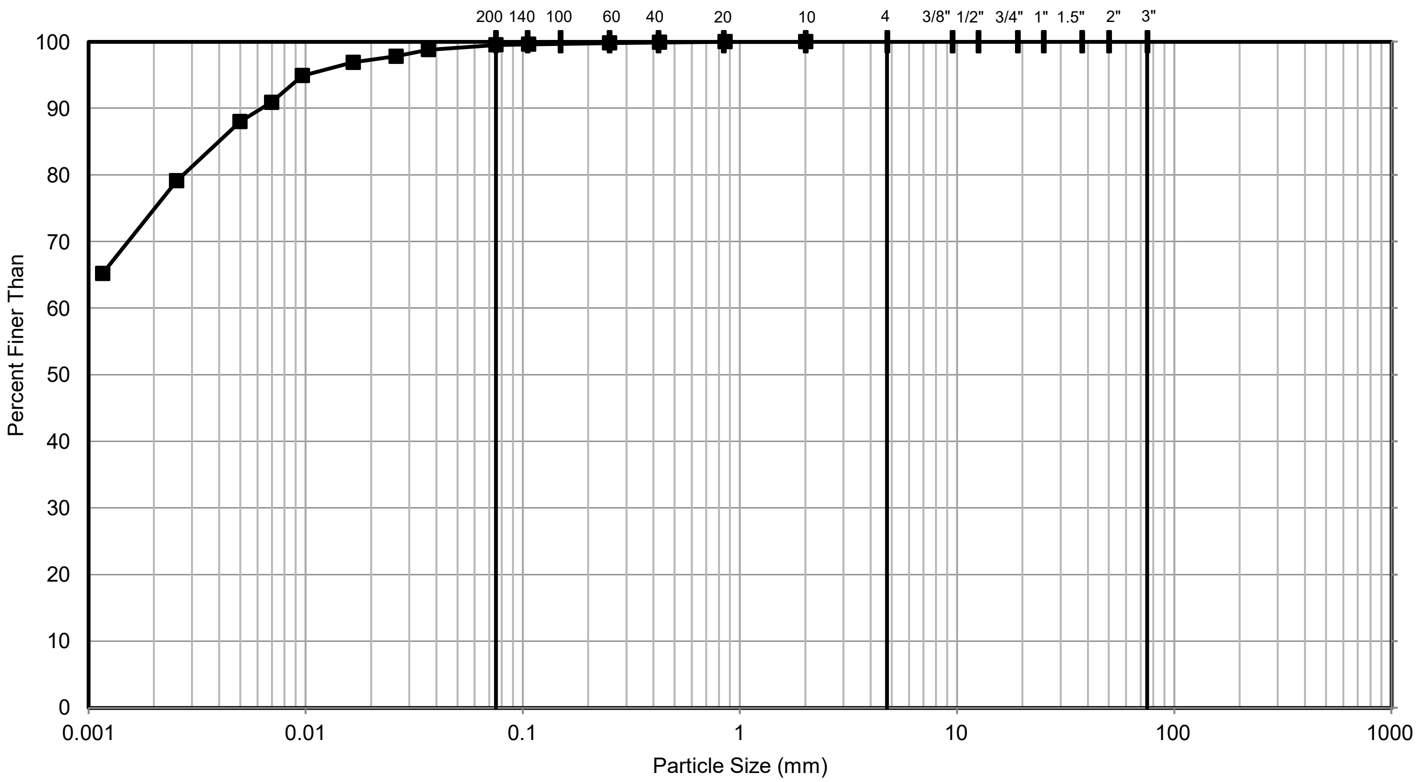
TITLE

Grain Size Distribution
Sandy Silt (ML) to Silt (ML) - Interlayer

PROJECT NO.	CONTROL	REV.	FIGURE
19136074	0	0	B20

PATH: https://golderassociates.sharepoint.com/sites/120387/Project Files/6 Deliverables/Foundations/Holland River East Branch | FILE NAME: Laboratory Particle Size Distribution MTO HRE.xlsm

Grain Size Distribution - Clay (CH) - Interlayer



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

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	HRE-4	23	39.6 - 40.2	180.3 to 179.7

CLIENT

AECOM / MTO

PROJECT

Bradford Bypass - Holland Rive East Branch

CONSULTANT

WSP GOLDER

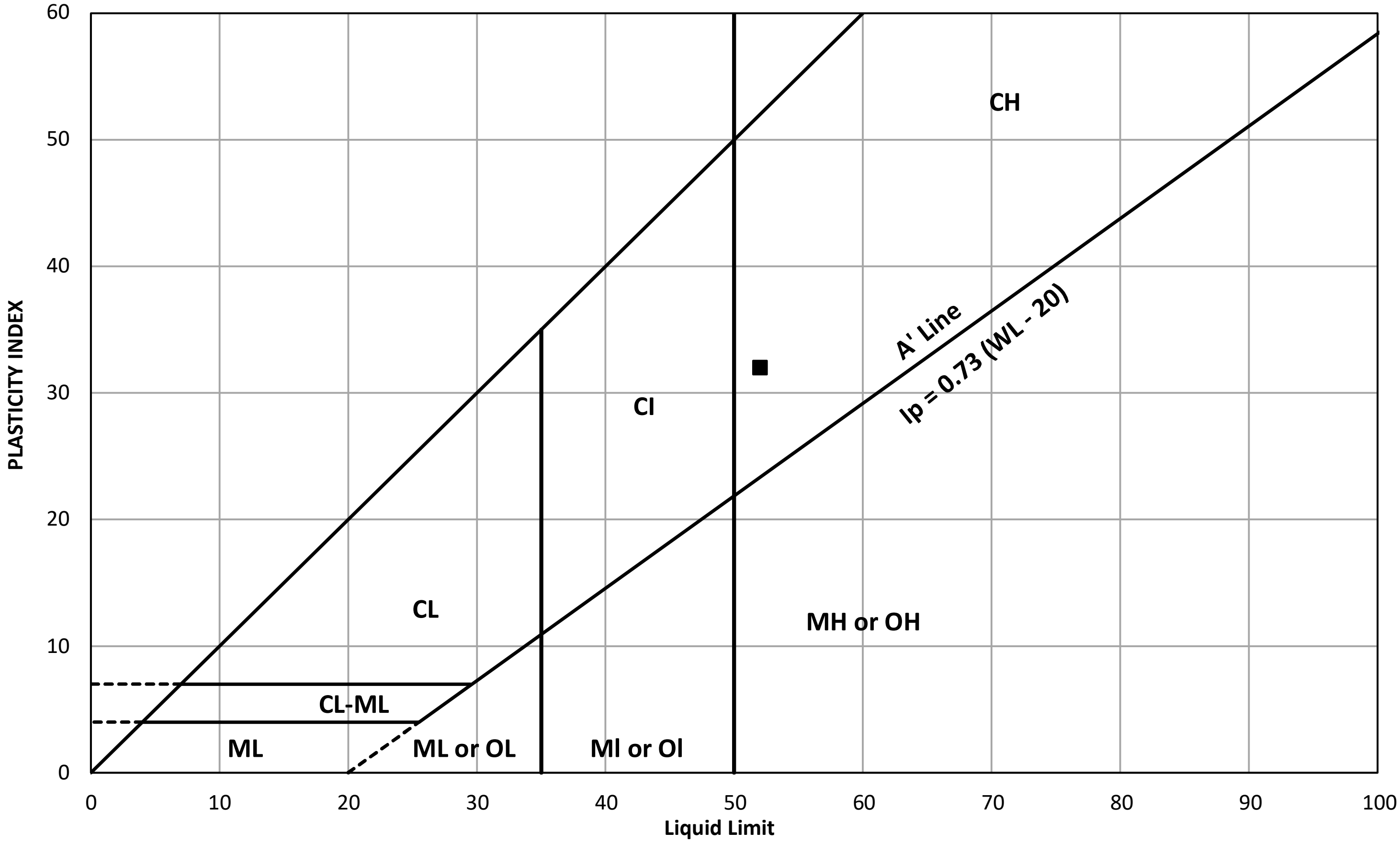
YYYY-MM-DD	2022-09-12
DESIGNED	CC
PREPARED	CC
REVIEWED	KJB
APPROVED	KJB

TITLE

Grain Size Distribution
Clay (CH) - Interlayer

PROJECT NO.	CONTROL	REV.	FIGURE
19136074	0	0	B21

Platicity Chart - Clay (CH) - Interlayer



Symbol	Sample Location	Sample / Specimen Number	Elevation (m)	Natural Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
■	HRE-4	23	180.3 to 179.7	39.7	52	20	32

CLIENT

MTO

PROJECT

Bradford Bypass - Holland River East Branch

CONSULTANT

GOLDER
MEMBER OF WSP

YYYY-MM-DD	2022-09-12
DESIGNED	CC
PREPARED	CC
REVIEWED	KJB
APPROVED	KJB

TITLE

Clay (CH) - Interlayer

PROJECT NO.	CONTROL	REV.	FIGURE
19136074	0	0	B22

CONSOLIDATION TEST SUMMARY

FIGURE B23

ASTM D2435/D2435M

SAMPLE IDENTIFICATION

Project Number	19136074	Sample Number	TO11
Borehole Number	HRE3	Sample Depth, m	38.11-38.72

TEST CONDITIONS

Test Type	Laboratory Standard	Load Duration, hr	24
Oedometer Number	11		
Date Started	01/28/2022		
Date Completed	02/17/2022		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.54	Unit Weight, kN/m^3	17.50
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m^3	12.20
Area, cm^2	31.56	Specific Gravity, measured	2.75
Volume, cm^3	80.22	Solids Height, cm	1.150
Water Content, %	43.43	Volume of Solids, cm^3	36.29
Wet Mass, g	143.14	Volume of Voids, cm^3	43.93
Dry Mass, g	99.8	Degree of Saturation, %	98.6

TEST COMPUTATIONS

Stress kPa	Corr. Height cm	Void Ratio	Average Height cm	t_{90} sec	cv. cm^2/s	mv m^2/kN	k cm/s
0.00	2.542	1.211	2.542				
6.05	2.530	1.200	2.536	2323	5.87E-04	8.00E-04	4.60E-08
10.79	2.517	1.189	2.523	3557	3.80E-04	1.04E-03	3.86E-08
20.67	2.491	1.166	2.504	7935	1.68E-04	1.05E-03	1.72E-08
40.02	2.445	1.127	2.468	11426	1.13E-04	9.27E-04	1.03E-08
78.89	2.385	1.074	2.415	6998	1.77E-04	6.12E-04	1.06E-08
156.43	2.313	1.012	2.349	4335	2.70E-04	3.63E-04	9.59E-09
299.22	2.239	0.947	2.276	3241	3.39E-04	2.06E-04	6.84E-09
78.97	2.264	0.969	2.251				
20.55	2.300	1.000	2.282				
78.97	2.282	0.984	2.291	1270	8.76E-04	1.20E-04	1.03E-08
156.43	2.260	0.965	2.271	1750	6.25E-04	1.10E-04	6.75E-09
299.22	2.227	0.937	2.244	1162	9.18E-04	9.01E-05	8.11E-09
628.84	2.137	0.858	2.182	2053	4.92E-04	1.08E-04	5.20E-09
1249.82	2.034	0.768	2.085	1717	5.37E-04	6.54E-05	3.44E-09
2492.15	1.924	0.673	1.979	1215	6.83E-04	3.48E-05	2.33E-09
628.84	1.957	0.702	1.941				
299.22	1.986	0.727	1.971				
78.89	2.051	0.784	2.018				
10.74	2.126	0.849	2.089				

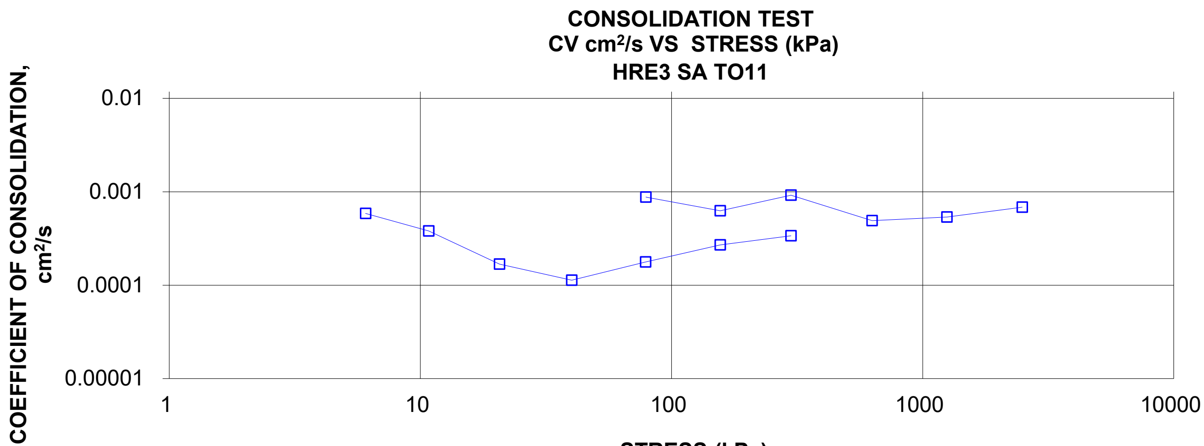
Note:
Consolidation loading and unloading schedule assigned by the client.
cv and k are approximate only based on t_{90} estimated from Square Root of Time Method (ASTMD2435/2435M)
Specimen taken 49-55cm from top of the tube.

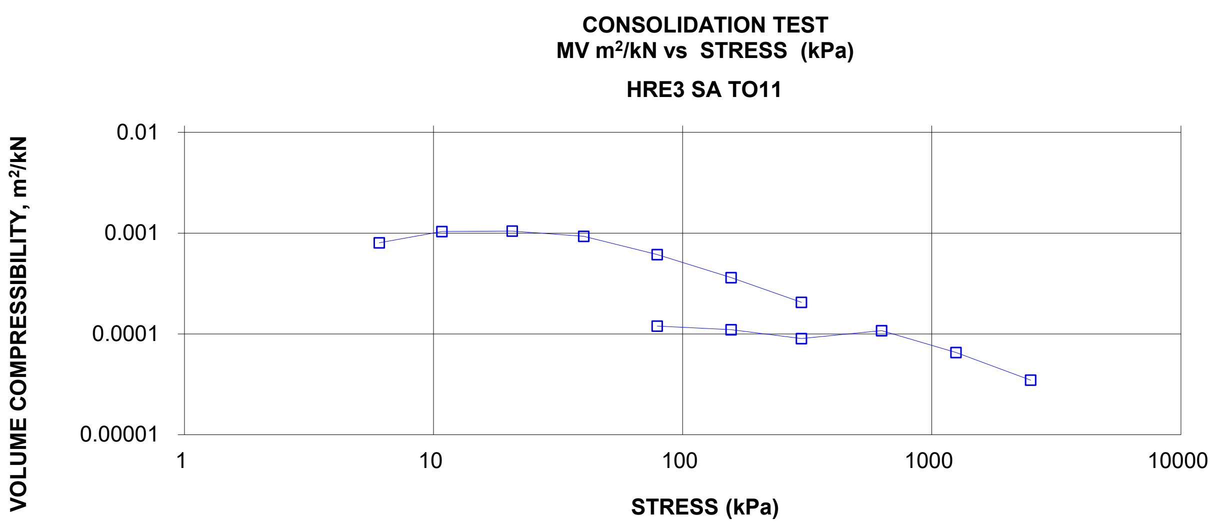
SAMPLE DIMENSIONS AND PROPERTIES - FINAL

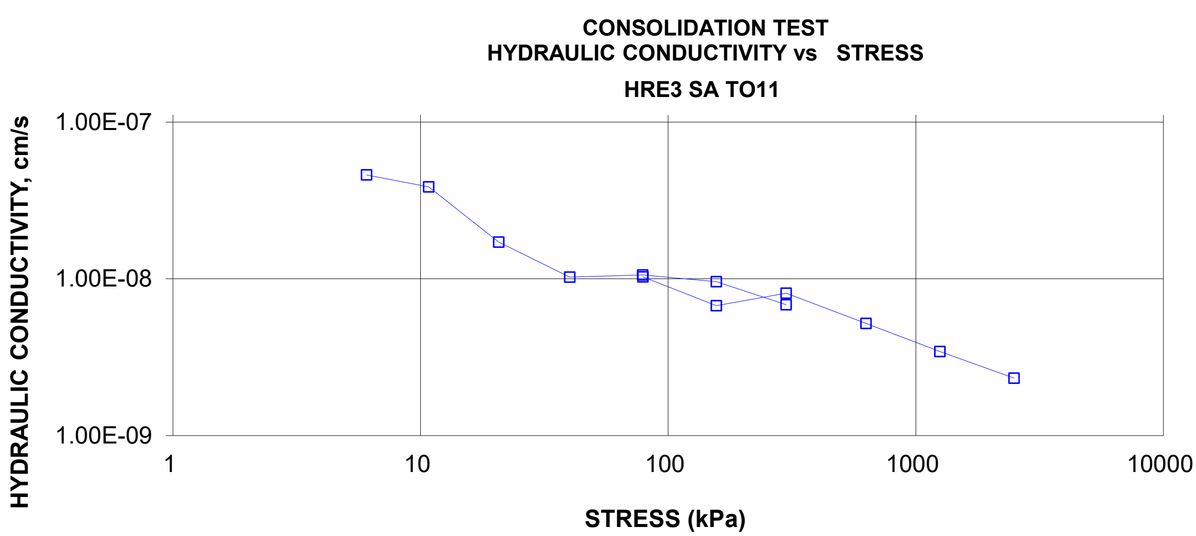
Sample Height, cm	2.13	Unit Weight, kN/m^3	19.27
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m^3	14.58
Area, cm^2	31.56	Specific Gravity, measured	2.75
Volume, cm^3	67.11	Solids Height, cm	1.150
Water Content, %	32.14	Volume of Solids, cm ^{3}	36.29
Wet Mass, g	131.88	Volume of Voids, cm ^{3}	30.82
Dry Mass, g	99.8		

Prepared By: LH **Golder Associates** Checked By: MM

CONSOLIDATION TEST SUMMARY	FIGURE B24







Project No. 19136074

Prepared By: LH

Golder Associates

Checked By: MM

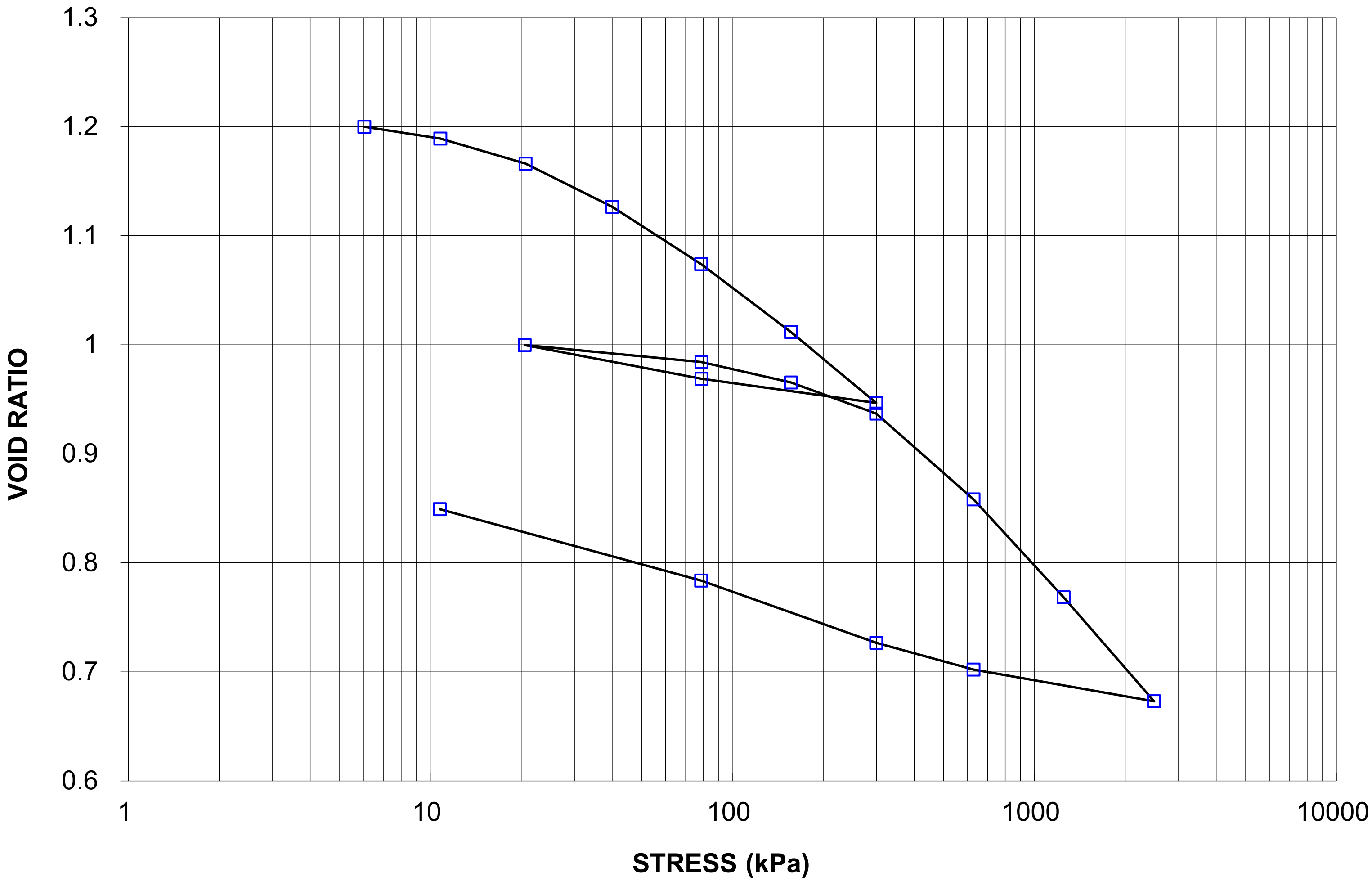
Project No. 19136074

Prepared By: LH

Golder Associates

Checked By: MM

CONSOLIDATION TEST
VOID RATIO vs STRESS
HRE3 SA TO11

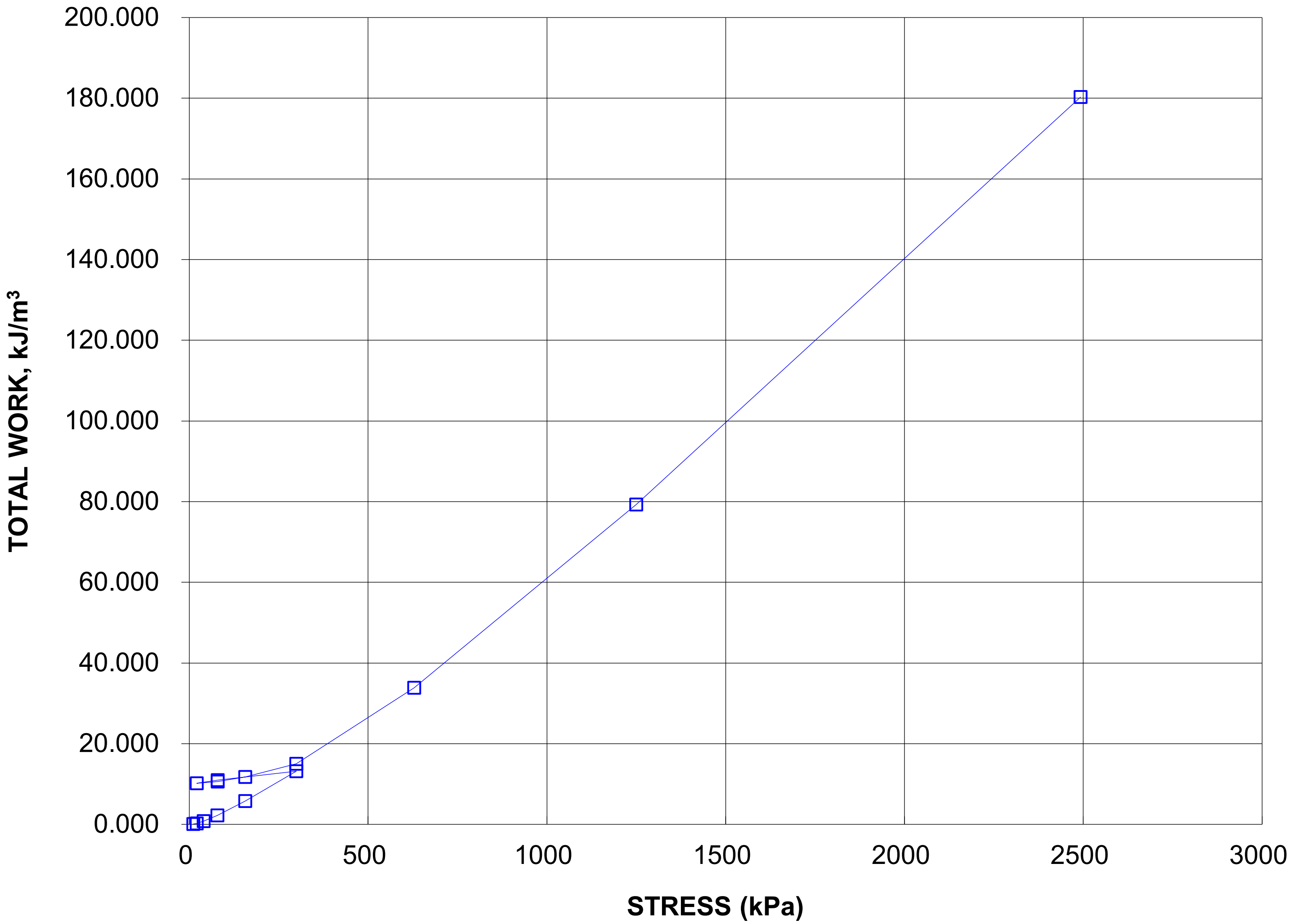


CONSOLIDATION TEST
VOID RATIO VS LOG STRESS

FIGURE B25

Project No.19136074
Prepared By: LH

CONSOLIDATION TEST
TOTAL WORK, kJ/m³ vs STRESS
HRE3 SA TO11



Golder Associates

Checked By: MM

CONSOLIDATION TEST
TOTAL WORK VS STRESS

FIGURE B26

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:

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.

Bureau Veritas Laboratories 6740 Campobello Road, Mississauga, Ontario, L5N 2L8 Tel: (905) 817-5700 Toll-Free: 800-563-6266 Fax: (905) 817-5777 www.bvlabs.com

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 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)

Bureau Veritas ID		QZC417			QZC417			QZC418		
Sampling Date		2021/09/29			2021/09/29			2021/10/06		
COC Number		n/a			n/a			n/a		
	UNITS	**HRE-1 SS #01**	**RDL**	**QC Batch**	**HRE-1 SS #01 Lab-Dup**	**RDL**	**QC Batch**	**HRW-4 SA-01**	**RDL**	**QC Batch**
Calculated Parameters										
Resistivity	ohm-cm	4800		7648263				4700		7648263
Inorganics										
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
Physical Testing										
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

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Microbiology testing is conducted at 6660 Campobello Rd. Chemistry testing is conducted at 6740 Campobello Rd.

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	Taslima 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	Taslima 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	Taslima 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

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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.

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QUALITY ASSURANCE REPORT

			Matrix Spike		SPIKED BLANK		Method Blank		RPD	
QC Batch	**Parameter**	**Date**	**% Recovery**	**QC Limits**	**% Recovery**	**QC Limits**	**Value**	**UNITS**	**Value (%)**	**QC Limits**
7653694	Available (CaCl2) pH	2021/10/22			100	97 - 103			0.52	N/A
7657473	Available (CaCl2) pH	2021/10/25			100	97 - 103			1.4	N/A
7657911	Soluble (20:1) Chloride (Cl-)	2021/10/26	113	70 - 130	101	70 - 130	<20	ug/g	NC	35
7657917	Soluble (20:1) Sulphate (SO4)	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 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

THE CHEMICAL PROFESSION OF ONTARIO
CHARTERED
Eva Pranjic
CHEMIST

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

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

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Microbiology testing is conducted at 6660 Campobello Rd. Chemistry testing is conducted at 6740 Campobello Rd.

BUREAU VERITAS

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)	
Company Name:	Golder Associates Ltd.	Company Name:	same	Quotation #:	Golder rates
Contact Name:	Manisha Ahuja	Contact Name:		P.O. #/ AFE#:	19136074
Address:	6925 Century Ave., Suite 100	Address:		Project #:	19136074
	Mississauga, ON			Site Location:	Bradford
Phone:	647-239-0174 Fax:	Phone:	Fax:	Site #:	
Email:	canadaaccountspayableinvoices@golder.com; Manisha_Ahuja@golder.com	Email:	Manisha_Ahuja@golder.com; 120387@golder.com	Site Location Province:	Ontario
				Sampled By:	

Turnaround Time (TAT) Required

X Regular TAT (5-7 days) Most analyses

PLEASE PROVIDE ADVANCE NOTICE FOR RUSH PROJECTS

Rush TAT (Surcharges will be applied)

☐ 1 Day ☐ 2 Days ☐ 3-4 Days

Date Required:

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

Regulation 153

☐ Table 1 ☐ Res/Park ☐ Med/ Fine
☐ Table 2 ☐ Ind/Comm ☐ Coarse
☐ Table 3 ☐ Agri/ Other
☐ Table ____

FOR RSC (PLEASE CIRCLE) Y / N

Other Regulations

☐ CCME ☐ Sanitary Sewer Bylaw
☐ MISA ☐ Storm Sewer Bylaw
☐ PWQO Region ____
☐ Other (Specify) ____
☐ REG 558 (MIN. 3 DAY TAT REQUIRED)

Include Criteria on Certificate of Analysis: Y / N

SAMPLES MUST BE KEPT COOL (< 10 °C) FROM TIME OF SAMPLING UNTIL DELIVERY TO BUREAU VERITAS

Analysis Requested

	SAMPLE IDENTIFICATION	DATE SAMPLED (YYYY/MM/DD)	TIME SAMPLED (HH:MM)	MATRIX	# OF CONTAINERS SUBMITTED	FIELD FILTERED (CIRCLE) Metals / Hg / CrVI	BTEX/ PHC F1	PHCs F2 - F4	VOCs	REG 153 METALS & INORGANICS	REG 153 ICPMS METALS	REG 153 METALS (Hg, Cr VI, ICPMS Metals, HWS - B)	CORROSIVITY PACKAGE (+ SULPHIDE)	HOLD- DO NOT ANALYZE	COMMENTS
1	HRE-1 SS #01	2021-09-29	PM	Soil	2								X		2 JARS. NO REDOX POTENTIAL.
2	HRW-4 SA-01	2021-10-06	PM	Soil	2								X		2 JARS. NO REDOX POTENTIAL.
3															
4															
5															
6															
7															
8															
9															
10															

LABORATORY USE ONLY

CUSTODY SEAL Y / N		COOLER TEMPERATURES
Present	Intact	
N	N	11/14/13

COOLING MEDIA PRESENT: Y / N

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.

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
Talha					

19-Oct-21 17:12
Ema Gitej
C1U4495
SYV ENV-870

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 #: C2O5551
Received: 2022/08/26, 15:58

Sample Matrix: Soil
Samples Received: 9

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Analytical Method
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 #: C2O5551
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
==
This report has been generated and distributed using a secure automated process.
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.

Bureau Veritas 6740 Campobello Road, Mississauga, Ontario, L5N 2L8 Tel: (905) 817-5700 Toll-Free: 800-563-6266 Fax: (905) 817-5777 www.bvlabs.com

Microbiology testing is conducted at 6660 Campobello Rd. Chemistry testing is conducted at 6740 Campobello Rd.

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)

Bureau Veritas ID		TOL232			TOL232			TOL233		
Sampling Date		2021/06/15			2021/06/15			2021/06/17		
COC Number		n/a			n/a			n/a		
	UNITS	**404-2 SA#4**	**RDL**	**QC Batch**	**404-2 SA#4 Lab-Dup**	**RDL**	**QC Batch**	**404-3 SA#3**	**RDL**	**QC Batch**
Calculated Parameters										
Resistivity	ohm-cm	3200		8193773				810		8193773
Inorganics										
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
Physical Testing										
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 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)

Bureau Veritas ID		TOL233			TOL234	TOL235	TOL236	TOL237		
Sampling Date		2021/06/17			2021/06/10	2021/12/21	2021/07/07	2022/01/13		
COC Number		n/a			n/a	n/a	n/a	n/a		
	UNITS	**404-3 SA#3 Lab-Dup**	**RDL**	**QC Batch**	**404-4 SA#4**	**2-1 SA#4**	**L-4B SA#4**	**HRE-3 SA#3B**	**RDL**	**QC Batch**
Calculated Parameters										
Resistivity	ohm-cm				700	2800	990	9000		8193773
Inorganics										
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
Physical Testing										
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

Bureau Veritas ID		TOL238			TOL238			TOL239		
Sampling Date		2022/02/22			2022/02/22			2021/11/11		
COC Number		n/a			n/a			n/a		
	UNITS	**HRE-4 SA#4**	**RDL**	**QC Batch**	**HRE-4 SA#4 Lab-Dup**	**RDL**	**QC Batch**	**HRW-1B SA#5**	**RDL**	**QC Batch**
Calculated Parameters										
Resistivity	ohm-cm	7200		8193773				6000		8193773
Inorganics										
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
Physical Testing										
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

Bureau Veritas 6740 Campobello Road, Mississauga, Ontario, L5N 2L8 Tel: (905) 817-5700 Toll-Free: 800-563-6266 Fax: (905) 817-5777 www.bvlabs.com

SOIL CORROSIVITY PACKAGE (SOIL)

Bureau Veritas ID		TOL239			TOL240		
Sampling Date		2021/11/11			2022/05/20		
COC Number		n/a			n/a		
	UNITS	**HRW-1B SA#5 Lab-Dup**	**RDL**	**QC Batch**	**FD-02 SA#3**	**RDL**	**QC Batch**
Calculated Parameters							
Resistivity	ohm-cm				5800		8193773
Inorganics							
Soluble (20:1) Chloride (Cl-)	ug/g				<20	20	8201208
Conductivity	umho/cm				173	2	8198559
Available (CaCl2) pH	pH				7.71		8199338
Soluble (20:1) Sulphate (SO4)	ug/g				<20	20	8201217
Sulphide	mg/kg				2.0 (1)	0.5	8205122
Physical Testing							
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

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	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 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	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 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

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	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 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	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 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	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 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	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

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	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 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	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 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 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 6740 Campobello Road, Mississauga, Ontario, L5N 2L8 Tel: (905) 817-5700 Toll-Free: 800-563-6266 Fax: (905) 817-5777 www.bvlabs.com

Microbiology testing is conducted at 6660 Campobello Rd. Chemistry testing is conducted at 6740 Campobello Rd.

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.

QUALITY ASSURANCE REPORT

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

			Matrix Spike		SPIKED BLANK		Method Blank		RPD	
QC Batch	**Parameter**	**Date**	**% 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 (CaCl2) pH	2022/08/31			100	97 - 103			1.2	N/A
8201208	Soluble (20:1) Chloride (Cl-)	2022/09/02	NC	70 - 130	103	70 - 130	<20	ug/g	0.69	35
8201217	Soluble (20:1) Sulphate (SO4)	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)

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

26-Aug-22 15:58
Ankita Bhalla
C205551
RJM ENV-1390

BUREAU VERITAS

L8
Free: 800-563-6266

WORK ORDER

CHAIN OF CUSTODY RECORD

Page __ of 1

Invoice Information		Report Information (if differs from invoice)		Project Information (where applicable)	
Company Name:	Golder Associates Ltd.	Company Name:	Golder Associates Ltd.	Quotation #:	Golder rates
Contact Name:	Kevin Bentley	Contact Name:	Muhammad Talha Irshad	P.O. #/ AFE#:	19136074
Address:	6925 Century Ave., Suite 100	Address:	6925 Century Ave., Suite 100	Project #:	19136074
	Mississauga, ON		Mississauga, ON	Site Location:	Bradford
Phone:	905 567-6100 Fax:	Phone:	778 228-5756 Fax:	Site #:	
Email:	canadaaccountspayableinvoices@golder.com; Kevin_Bentley@golder.com	Email:	mirshad@golder.com; 120387@golder.com	Site Location Province:	Ontario
				Sampled By:	

Turnaround Time (TAT) Required

- [X] Regular TAT (5-7 days) Most analyses

PLEASE PROVIDE ADVANCE NOTICE FOR RUSH PROJECTS

Rush TAT (Surcharges will be applied)

- [ ] 1 Day
- [ ] 2 Days
- [ ] 3-4 Days

Date Required:

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

Regulation 153

- [ ] Table 1
- [ ] Res/Park
- [ ] Med/ Fine
- [ ] Table 2
- [ ] Ind/Comm
- [ ] Coarse
- [ ] Table 3
- [ ] Agri/ Other
- [ ] Table ____

FOR RSC (PLEASE CIRCLE) Y / N

Other Regulations

- [ ] CCME
- [ ] Sanitary Sewer Bylaw
- [ ] MISA
- [ ] Storm Sewer Bylaw
- [ ] PWQO
- Region ____
- [ ] Other (Specify) ____
- [ ] REG 558 (MIN. 3 DAY TAT REQUIRED)

Include Criteria on Certificate of Analysis: Y / N

SAMPLES MUST BE KEPT COOL (< 10 °C) FROM TIME OF SAMPLING UNTIL DELIVERY TO BUREAU VERITAS

LABORATORY USE ONLY

CUSTODY SEAL Y / N Present	Intact	COOLER TEMPERATURES
N	N	5/4/5

COOLING MEDIA PRESENT: Y / N

Analysis Requested

	SAMPLE IDENTIFICATION	DATE SAMPLED (YYYY/MM/DD)	TIME SAMPLED (HH:MM)	MATRIX	# OF CONTAINERS SUBMITTED	FIELD FILTERED (CIRCLE) Metals / Hg / CrVI	BTEX/ PHC F1	PHCs F2 - F4	VOCs	REG 153 METALS & INORGANICS	REG 153 ICPMS METALS	REG 153 METALS (Hg, Cr VI, ICPMS Metals, HWS - B)		CORROSIVITY PACKAGE (+ SULPHIDE)							HOLD- DO NOT ANALYZE	COMMENTS
1	404-2 SA#4	2021-06-15	AM	Soil	2									X								2 JARS. NO REDOX POTENTIAL.
2	404-3 SA#3	2021-06-17	AM	Soil	2									X								2 JARS. NO REDOX POTENTIAL.
3	404-4 SA#4	2021-06-10	AM	Soil	2									X								2 JARS. NO REDOX POTENTIAL.
4	2-1 SA#4	2021-12-21	AM	Soil	2									X								2 JARS. NO REDOX POTENTIAL.
5	L-4B SA#4	2021-07-07	AM	Soil	2									X								2 JARS. NO REDOX POTENTIAL.
6	HRE-3 SA#3B	2022-01-13	AM	Soil	2									X								POTENTIAL.
7	HRE-4 SA#4	2022-02-22	AM	Soil	2									X								POTENTIAL.
8	HRW-1B SA#5	2021-11-11	AM	Soil	2									X								POTENTIAL.
9	FD-02 SA#3	2022-05-20	AM	Soil	2									X								POTENTIAL.
10																						

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.

RELINQUISHED BY: (Signature/Print)	DATE: (YYYY/MM/DD)	TIME: (HH:MM)	RECEIVED BY: (Signature/Print)	DATE: (YYYY/MM/DD)	TIME: (HH:MM)	BV JOB #
Muhammad Talha Irshad	2022-08-26		Alam Alam	2022/08/26	15:58	

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