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FINAL
FOUNDATION INVESTIGATION AND DESIGN REPORT
HIGHWAY 11 POPLAR RAPIDS BRIDGE
HAGGART TOWNSHIP, ONTARIO
ASSIGNMENT NO.: 5021-E-0025
GWP 5278-19-00

GEOCRES No. 42H00-095

Location: Lat: 49.289217°, Long: -81.784584°

Client Name: LEA Consulting Ltd.
Date: April 5, 2024
File: 33443

Suite 125, 2611 Queensview Drive, Ottawa, Ontario K2B 8K2 (613) 247-2121

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PART 1. FACTUAL INFORMATION

1. INTRODUCTION

This section of the report presents the factual findings obtained from a foundation investigation conducted by Thurber Engineering Ltd. (Thurber) for the replacement of the Highway 11 Bridge (Site No. 39E-0001/B0) crossing Poplar Rapids River in Haggart Township within the District of Cochrane, Ontario. The propose bridge replacement is to be constructed under a Ministry of Transportation Design Build procurement. Thurber carried out the foundation investigation as a subconsultant to LEA Consulting Ltd. (LEA) under Agreement No. 5021-E-0025.

The purpose of the investigation was to explore the subsurface conditions at the site and based on the data obtained, provide a borehole location plan, record of boreholes, stratigraphic profile, laboratory test results and a written description of the subsurface conditions. The stratigraphic profile of the subsurface conditions was developed during the current investigation.

It is a condition of this report that Thurber's performance of its professional services is subject to the attached Statement of Limitations and Conditions.

2. SITE DESCRIPTION

2.1 General

The existing Highway 11 crosses Poplar Rapids River approximately 13.5 km west of the junction of Highway 11 and Highway 634. The bridge site is near Sta. 16+150 Haggart Township. For project purposes, Highway 11 at the bridge site is herein described as oriented east-west, and the river's flow is described as oriented south to north.

In the area of the bridge, Highway 11 is a two-lane highway and has a posted speed limit of 90 km/h. The bridge is a concrete rigid frame structure comprising four spans. The river has a

30 degree northeast to southwest skew to the bridge crossing. Galvanized W-beam guiderails supported on metal posts extend from both ends of the bridge. The CNR railway runs approximately 30 m south of the bridge site; the railway runs approximately parallel to the highway alignment. Traffic volumes for this section of Highway 11 is understood to have been 3,300 AADT in 2016.

The existing roadway embankment side slopes at the site did not show any visible signs of global instability at the time of the investigation. The embankment near the north and south approaches are sloped at approximately 2.5H:1V. The highway alignment is inferred to be in a cut section near the west abutment.

AutoCAD drawings provided by LEA indicate that the road surface is at approximate elevation 229.0 m near the highway centreline. The river's water level was measured by Thurber at elevation 221.2 m on July 5, 2023. The river flows in a meandering manner from south to north toward Mattagami River.

The site is in a rural setting and the area adjacent to the highway is undeveloped and densely vegetated with mixed forests of coniferous and some deciduous trees and shrubs. Overhead utility lines were present along the north side of the highway. A MTO traffic monitoring camera is mounted on the pole northwest of the bridge.

Photographs showing the existing conditions in the project area at the time of the field investigation are included in Appendix D for reference.

2.2 Existing Structure Information

The Terms of Reference (TOR) describe the bridge as a four-span, 63.8 m long reinforced concrete rigid frame constructed in 1944. The bridge spans are approximately 15.2 m long with an overall width of 11.5 m. Contract Drawing 2003-5135 indicates that the 15.2 m span measurement is from the inside face of the west pier to the centerline of the middle pier, whereas the same span measurement is shown from the centerline of the middle pier to the centerline of the east pier. The Poplar Rapids Bridge abutments and piers are on a 30-degree skew to align with the river alignment.

Contract Drawings 42-03 provided by MTO indicate that the creek was identified as Departure Creek (the community of Departure Lake is approximately 800 m west of the bridge site) and shows the structure as three-spans, similar to the three east most spans of the current structure (sheet number not legible). There is a handwritten note on the sheet that a 50' span was added. Sheet D2794-7 included with the same drawings is titled as "Additional 50' Span" which represents the current west span which would make up the current four-span structure. Similar to the 2003-5135 drawings, the span measurement for the “additional span” is shown from the face of the west abutment to the outside face of the west pier, not to the centerline of the pier. The piers and abutments are shown to be supported on shallow footing foundations. The piers are indicated to be founded near elev. 218.1 m (715.5 feet). The east abutment also has a transverse footing supporting the concrete wingwalls. Sheet 2781-1 shows a two-span structure with a handwritten note that the plan was revised for a three-span structure. Although the 42-03 drawings show two-span, three-span and three-span plus and additional span, it is assumed that the four-span structure was the originally built structure.

The bridge was indicated in the RFP to have been extensively rehabilitated in 2004 (Contract 2003-5135). The rehabilitation included concrete repairs to the various bridge components, reconstruction of the deck overhangs and railing system with deck widening, reconstruction of the approach slabs, and waterproofing and paving of the deck and approaches.

The 2019 OSIM report indicates that the existing structure is in fair condition with minor deterioration of several elements, and more significant deterioration of the deck soffit, abutment, and piers. The OSIM report recommends rehabilitation in 1-5 years.

2.3 Existing Subsurface Information

A historical foundation investigation report was not available for this site within the online Geocres Library. Soil stratigraphy and design bearing resistances for the shallow footings were not shown on the historical contract drawings. Geocres Report No. 42H-30, for a foundation investigation conducted 500 m east of the bridge, was reviewed for regional information only and the stratigraphy has not been used further in the report. However, the one-dimensional consolidation test results from the Geocres report were utilized herein.

General Arrangement (GA) drawing dated October 2023 and base plan mapping was provided by LEA for the preparation of this report.

2.4 Site Geology

According to Crins et al. 2009[1] the project area is described as Ecoregion 3E (Lake Abitibi Ecoregion) within the Ontario Shield Ecozone. According to Wester et al. 2018[2] the ecoregion is subdivided into Ecodistrict 3E-1 (Clay Belt Ecodistrict). The project area is in near center of the

[1] *https://files.ontario.ca/mnrf-ecosystemspart1-accessible-july2018-en-2020-01-16.pdf*
[2] *https://files.ontario.ca/ecosystems-ontario-part2-03262019.pdf*

ecodistrict, which is characterized by deep, fine texture morainal and glaciolacustrine deposits overlying Precambrian bedrock.

Map M.5036[3] indicates that the project area is composed of till and clay and is within a transition area from morainal to glaciofluvial landform. Map M.2555[4] indicates that the project area is composed of undifferentiated, fine grained, predominantly silty clay to silt matrix, commonly clast poor, high matrix carbonate content till.

Bedrock Geology Map (MRD126)[5] indicates the site is underlain by metasedimentary rocks: paragneiss and migmatites.

3. SITE INVESTIGATION AND FIELD TESTING

The foundation investigation and field-testing program was carried out between July 5 and July 15, 2023, and consisted of two on-road SCPTu 23-107 and 23-108, two on-road boreholes identified as 23-101 and 23-104 and four off-road boreholes identified as 23-102, 23-103, 23-105, and 23-106. The ConeTec report documenting two SCPTu tests (SCPT23-107 & SCPT23-108) is provided in Appendix E. Well slug testing was carried out in September 2023 in monitoring wells installed in Borehole 23-102 and 23-105.

The boreholes were advanced using a track mounted CME 55 LC drill rig equipped with Hollow Stem Augers, NW casing, and NQ coring equipment. The two on-road Testholes identified as 23-107 and 23-108 were advanced utilizing SCPTu equipment. Prior to commencement of drilling, utility clearances were obtained in the vicinity of the borehole locations.

A summary of the borehole coordinates, elevations, and termination depths is provided in Table 3-1. The as-drilled borehole elevations were surveyed by Thurber with a Trimble Catalyst DA2 receiver and were checked relative to BM HCP 108 (elev. 228.821 m) and BM 109 (elev. 230.128 m). Horizontal locations were measured by Thurber relative to existing site features. The elevations and borehole coordinates were reviewed and referenced to the survey data provided by LEA. The borehole coordinates and elevations are shown on the Borehole Location and Soil Strata drawing included in Appendix A and on the individual Record of Borehole sheets included in Appendix B. The borehole coordinates are referenced to MTM Zone 12.

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[3] *http://www.geologyontario.mndm.gov.on.ca/index.html*
[4] *http://www.geologyontario.mndm.gov.on.ca/index.html*
[5] *http://www.geologyontario.mndm.gov.on.ca/mines/data/google/mrd126/doc.kml*

Table 3-1 Borehole Summary

BOREHOLE NO.	LOCATION	NORTHING (m)	EASTING (m)	GROUND SURFACE ELEVATION (m)	TERMINATION DEPTH (m)
23-101	East Abutment	5 461 553.9	247 779.4	229.3	28.1
23-102	East Abutment	5 461 566.1	247 791.1	227.0	26.0
23-103	East Approach	5 461 567.1	247 804.1	227.3	15.8
23-104	West Abutment	5 461 545.7	247 695.6	229.8	30.8
23-105	West Abutment	5 461 553.1	247 698.1	228.5	31.1 (DCPT 31.7)
23-106	West Approach	5 461 550.1	247 669.1	229.1	14.3
23-107 (SCPTu)	East Abutment	5 461 554.2	247 782.4	229.3	22.2
23-108 (SCPTu)	West Abutment	5 461 545.5	247692.6	229.8	18.9

Soil samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT) in general accordance with ASTM D 1586. In-situ shear vane testing was carried out within the cohesive layers, where possible, using an MTO 'N' sized vane in general accordance with ASTM D 2573. Thin-Walled (Shelby) Tube samples were pushed and retrieved at various elevations in the boreholes to obtain relatively undisturbed cohesive soil samples for further laboratory testing. The boreholes were advanced to sampled depths ranging from 14.3 to 31.1 m below the existing ground surface (elev. 214.8 to 197.4 m). A Dynamic Cone Penetration (DCPT) was completed below the sampled depth in Borehole 23-105 to a tip elevation at 196.8 m (31.7 m below the ground surface). Pre-drilling was not required to advance the SCPTu equipment. Coring was required to advance the boreholes past the existing pavement structure, through cobbles and boulders, and into bedrock. Bedrock coring was carried out in Boreholes 23-101 and 23-102.

The drilling and sampling operations were supervised on a full-time basis by a member of Thurber's technical staff. The drilling supervisor logged the boreholes and processed the recovered soil and rock samples for transport to Thurber's Ottawa laboratory for further examination and testing.

A 32 mm diameter monitoring well was installed in Boreholes 23-102 and 23-105 to allow for measurements of the groundwater level after drilling. The details for the wells are illustrated on the respective Record of Borehole sheets provided in Appendix B.

Following completion of the field investigation, the boreholes were decommissioned in general accordance with O.Reg. 903, as amended. Boreholes 23-101 and 23-104 and Testholes 23-107 and 23-108 were capped with concrete followed by cold patch asphalt to reinstate the pavement surface.

4. LABORATORY TESTING

Laboratory testing was selected in general accordance with the April 2022 version of the MTO Guidelines for Foundation Engineering Services, Section 5. Geotechnical laboratory testing consisted of natural moisture content determination and visual identification of all retained soil samples. Recovered soil samples were selected for grain size distribution and, where appropriate, Atterberg Limit testing in accordance with MTO and ASTM standards. The rock cores were photographed, and the total core recovery (TCR), solid core recovery (SCR), and rock quality designation (RQD) were measured. Unconfined compressive strength (UCS) testing was carried out on select intact bedrock cores. The results of these tests are summarized on the Record of Borehole sheets included in Appendix B.

Four relatively undisturbed soil samples obtained in Thin Walled (Shelby) Tubes were extruded and underwent one-dimensional consolidation testing (ASTM D 2435). Two one-dimensional consolidation tests were also carried out as part the current assignment for the culvert site (Geocres *TBD*) located approximately 500 m east of the bridge site and the details can be found in that report.

Four soil samples were selected and submitted for analytical testing of corrosivity parameters and sulphate content.

All laboratory test results from the field investigation are provided in Appendix C.

5. DESCRIPTION OF SUBSURFACE CONDITIONS

Details of the encountered soil stratigraphy are presented on the Record of Borehole sheets included in Appendix B and on the Borehole Location and Soil Strata Drawing included in Appendix A. A general description of the stratigraphy, based on the conditions encountered in the boreholes, is given in the following sections. However, the factual data presented on the Record of Borehole sheets takes precedence over this general description for interpretation of the site conditions. It must be recognized that the soil and groundwater conditions will vary between and beyond borehole locations. Soil classification is in general accordance with ASTM D2487 with the description of secondary components as outlined in the MTO Guideline for Foundation Engineering Services Manual (April 2022).

In general, the encountered stratigraphy consists of sand and gravel fill overlaying clayey silt fill over native deposits of clay to silty clay followed by alternating layers of silt to silty sand over bedrock.

5.1 Surficial Materials

5.1.1 Asphalt

Asphalt was encountered at the ground surface in the on-road Boreholes 23-101 and 23-104 and Test Holes 23-107 and 23-108. The asphalt was measured to have a thickness of 75 mm and 175 mm in the boreholes.

5.1.2 Concrete

Concrete was encountered beneath the asphalt in Borehole 23-104. The concrete was measured to have a thickness of 230 mm.

5.2 Fill

5.2.1 Sand to Gravelly Sand Fill

A fill layer consisting of sand some gravel to gravelly sand was encountered below the asphalt in Borehole 23-101, at the ground surface in Borehole 23-102, and below the concrete in Borehole 23-104. Varying amounts of silt was noted within the layer, and the layer in Borehole 23-102 was noted to contain organics. A hydrogen sulfide odour was coming from the sample in Borehole 23-102. The fill layer was 0.6 to 0.8 m thick (base elev. 228.7 to 226.2 m). SPT N-values ranging from 18 to 32 blows were recorded, indicating a compact to dense relative density.

Moisture contents ranging from 3 to 9% were recorded. The results of gradation analyses completed on three samples of the layer are illustrated in Figure C1 of Appendix C. The results of the tests are summarized in the table below and on the Record of Borehole sheets in Appendix B.

<table>
<tr><th>SOIL PARTICLE</th><th colspan="2">PERCENTAGE (%)</th></tr>
<tr><td>Gravel</td><td colspan="2">10 – 21</td></tr>
<tr><td>Sand</td><td colspan="2">69 – 80</td></tr>
<tr><td>Silt</td><td rowspan="2">10</td><td>12</td></tr>
<tr><td>Clay</td><td>4</td></tr>
</table>

5.2.2 Silty Sand Fill

A fill layer consisting of silty sand some clay containing organics was encountered at the ground surface in the off-road Boreholes 23-103, 23-105, and 23-106. Hydrogen sulfide odour was coming from the samples from the layer in Boreholes 23-103 and 23-106. The fill layer was 0.1 to 0.8 m thick (base elev. 229.0 to 226.5 m). An SPT N-value of 2 blows was recorded, indicating a very loose relative density.

Moisture contents ranging from 17 to 51% was recorded. The results of a gradation analysis completed on a sample of the layer are illustrated in Figure C2 of Appendix C. The results of the tests are summarized in the table below and on the Record of Borehole sheets in Appendix B.

SOIL PARTICLE	PERCENTAGE (%)
Gravel	0
Sand	61
Silt	21
Clay	18

Atterberg Limit testing was completed on the fines portion of one sample of the fill. The results are illustrated in Figures C3 in Appendix C and summarized below and on the Record of Borehole sheets in Appendix B. The laboratory results indicate that the fines portion of the silty sand fill exhibits a low plastic behaviour (CL).

PARAMETER	VALUE
Liquid Limit	27
Plastic Limit	16
Plasticity Index	11

5.2.3 Silt Clay Fill

A layer of silt clay fill was encountered below the sand fill in Boreholes 23-101 and 23-104. The layer ranged in thickness from 1.9 to 2.1 m with an underside depth of 2.9 to 3.0 m (base elev. 226.8 to 226.4 m). SPT N-values in the fill ranged from 7 to 14 blows. The silty clay fill is described as very stiff in consistency based on N-values and tactile evaluations of strength.

The recorded moisture contents of the fill ranged from 19 to 24%. The results of a gradation analysis completed on a sample of the fill are illustrated on Figure C4 of Appendix C. The results of the tests are summarized in the table below and on the Record of Borehole sheets in Appendix B.

SOIL PARTICLE	PERCENTAGE (%)
Gravel	0
Sand	10
Silt	44
Clay	46

Atterberg Limit testing was completed on one sample of the fill. The results are illustrated in Figures C5 in Appendix C and summarized below and on the Record of Borehole sheets in Appendix B. The laboratory results indicate that the silty clay fill exhibits an intermediate plastic behaviour (CI).

PARAMETER	VALUE
Liquid Limit	40
Plastic Limit	18
Plasticity Index	22

5.3 Clay (CH) to Silt Clay (CI) to Clayey Silt (CL / CL-ML)

A native layer varying from clay to silty clay to clayey silt was found below the silty clay fill in Boreholes 23-101 and 23-104, below the sand fill in Borehole 23-102, and below the silty sand fill in Boreholes 23-103, 23-105, and 23-106. Varying amounts of sand were noted in the layer. The layer ranged in thickness from 6.1 to 9.0 m with an underside depth of 7.5 to 9.1 m (base elev. 220.9 to 219.4 m).

The layer was noted to contain light grey silt seams, see photographs of extruded samples in Appendix C. Sample ST1 from Borehole 23-105 (elev. 222.9 m) had approximately 5 mm thick silt seams at 25 mm separations. Gravel pieces were noted in sample ST1 from Borehole 23-102 (elev. 223.6).

Where SPT were conducted within the layer, the N-values ranged from 2 to 21 blows but were typically less than 15 blows. Field vane tests were attempted but indicated undrained shear strengths greater than 100 kPa. The layer is described as very stiff in consistency based on N-values and the in-situ undrained shear strength measurements.

The recorded moisture contents ranged from 11 to 46%. The results of gradation analyses completed on 16 samples of the layer are illustrated on Figures C6, C7, and C8 of Appendix C. The results of the tests are summarized in the table below and on the Record of Borehole sheets in Appendix B.

SOIL PARTICLE	PERCENTAGE (%)
Gravel	0 – 1
Sand	0 – 23
Silt	28 – 74
Clay	26 – 72

Atterberg Limit testing were completed on 16 samples of the material. The results are illustrated in Figures C9, C10, and C11 of Appendix C and summarized below and on the Record of Borehole sheets in Appendix B. The laboratory results indicate that the soil layer exhibits a low to high plastic behaviour but is predominantly intermediate plasticity (CH to CI to CL / CL-ML).

PARAMETER	VALUE
Liquid Limit	24 -51
Plastic Limit	16 – 24
Plasticity Index	6 – 28

One-dimensional consolidation testing (ASTM D 2435) was carried out on relatively undisturbed cohesive samples from Boreholes 23-102 and 23-105. Load increments were maintained for 24 hours. Photographs of the extruded samples are provided in Appendix C. The testing results are presented in Appendix C and are summarized in the Table 5-1. The preconsolidation stresses summarized in the table were obtained from the end-of-increment void ratio. It should be expected that compressibility characteristics will vary with depth in accordance with the soil index parameters and stress history.

Table 5-1 Advanced Laboratory Test Results from Silty Clay to Clay

Borehole	23-102	23-105
Sample	ST1	ST1
Sample Depth (m)	3.0 – 3.6	5.3 – 5.9
Sample Elevation (m)	223.7	222.9
Soil Layer	Silty Clay (CI) to Clayey Silt (CL)	Clay (CH)
Moisture Content (%)	22	41
Liquidity Index (-)	0.3	0.7
Initial Void Ratio (-)	0.64	1.15
Moist Unit Weight (kN/m^3)	20.0	17.7
In-situ Vertical Effective Stress (kPa)	60	94
Preconsolidation Stress (kPa)	337	404
Overconsolidation Ratio (-)	5.6	4.3
Recompression Index (-)	0.03	0.04
Compression Index (-)	0.14	0.47
Coefficient of Reconsolidation (cm^2/sec)	5×10^{-3}	4×10^{-3}
Coefficient of Consolidation (cm^2/sec)	6×10^{-4}	7×10^{-4}
Load Increment Duration (hrs.)	24	24

5.4 Silt to Sandy Silt (ML) with Clay Seams

A layer of silt to sandy silt with clay seams was encountered below the clay layers within Boreholes 23-101 through 23-106. The sand content was noticed to increase with depth. The SCPTu results (see Appendix E) indicate the presence of layering which could be representative of the seams that were observed in the soil samples retained from the spoons and Thin-Walled tube samples. Where fully penetrated in Boreholes 23-101, 23-102, 23-104 and 23-105, the layer ranged in thickness from 9.8 to 12.2 m with an underside depth of 17.4 to 20.4 m (base elev. 211.1 to 207.2 m). The layer was not fully penetrated in Boreholes 23-103 and 23-106 but was proven to extend to depths of 15.8 m and 14.3 m below the ground surface (elev. 211.5 m and 214.8 m), respectively. SPT N-values ranged from 2 to 35 blows but were typically between 7 and 16 blows, indicating the layer is typically a very loose to compact relative density.

The layer was noted to contain grey clay seams, see photographs of extruded samples in Appendix C showing variations in layering thicknesses. Sample ST1 from Borehole 23-101 (elev. 219.9 m) had approximately 25-30 mm thick clay seams at 300 mm separations. Sample ST1 from Borehole 23-104 (elev. 220.4 m) had approximately 25-30 mm thick clay seams at 200 mm separations.

The recorded moisture contents ranged from 20 to 52%. The results of gradation analyses completed on 15 samples of the silt to sandy silt are illustrated on Figures C12, C13, and C14 of Appendix C. The results of the tests are summarized below and on the Record of Borehole sheets in Appendix B.

SOIL PARTICLE	PERCENTAGE (%)
Gravel	0
Sand	0 – 37
Silt	60 – 93
Clay	3 – 25

Atterberg Limit testing were completed on 15 samples of the material. The results are illustrated in Figure C15 in Appendix C and summarized below and on the Record of Borehole sheets in Appendix B. Thirteen samples were found to be non-plastic. The laboratory results from the remaining two samples indicate that the silt to sandy silt exhibits low plastic behaviour (ML).

PARAMETER	VALUE
Liquid Limit	21 – 22
Plastic Limit	18 – 20
Plasticity Index	2 – 3

The values presented in the table above were tested as a mixed sample with no separation of the clay seams. Sample ST1 from Borehole 23-104 was tested for index properties after separating out a clay seam from the remaining constituents of the sample. Dual test results have been shown on the borehole logs to represent each test. The results of the tests carried out on the clay seam only indicate the seam was 22% silt and 78% clay with a liquid limit of 55% and a plastic limit of 25%. Results are illustrated in Figures C16 and C17 in Appendix C.

One-dimensional consolidation testing (ASTM D 2435) was carried out on relatively undisturbed cohesive samples from Boreholes 23-101 and 23-104. Load increments were maintained for 24 hours. Photographs of the extruded samples are provided in Appendix C. The results are presented in Appendix C and are summarized in the Table 5-2. The preconsolidation stresses summarized in the table were obtained from the end-of-increment void ratio. It should be expected that compressibility characteristics will vary with depth in accordance with the soil index parameters and stress history.

Table 5-2 Advanced Laboratory Test Results from Silt with Clay Seams

Borehole	23-101	23-104
Sample	ST1	ST1
Sample Depth (m)	9.1 – 9.7	9.1 – 9.7
Sample Elevation (m)	219.9	220.4
Soil Layer	Silt (ML)	Clay Seam
Moisture Content (%)	41	41
Liquidity Index (-)	-	0.5
Initial Void Ratio (-)	1.03	1.10
Moist Unit Weight (kN/m^3)	18.3	17.9
In-situ Vertical Effective Stress (kPa)	130	131
Preconsolidation Stress (kPa)	466	475
Overconsolidation Ratio (-)	3.6	3.6
Recompression Index (-)	0.05	0.05
Compression Index (-)	0.37	0.34
Coefficient of Reconsolidation (cm^2/sec)	3×10^{-3}	5×10^{-3}
Coefficient of Consolidation (cm^2/sec)	1×10^{-3}	2×10^{-3}
Load Increment Duration (hrs.)	24	24

5.5 Silty Sand (SM)

A layer of silty sand was encountered below the silt in Boreholes 23-101, 23-102, 23-104, and 23-105. Cobbles and boulders were encountered in the layer and coring was required to advance past the cobbles and boulders in boreholes 23-101, 23-102 and 23-105. The layer ranged in thickness from 2.9 to 5.9 m with an underside depth of 22.7 to 25.9 m (base elev. 205.6 to 203.9 m). SPT N-values ranged from 30 to 77 blows but were typically less than 45 blows, indicating a compact to dense relative density.

The recorded moisture contents ranged from 9 to 19%. The results of gradation analyses completed on four samples of the silty sand are illustrated on Figure C18 of Appendix C. The results of the tests are summarized in the table below and on the Record of Borehole sheets in Appendix B.

SOIL PARTICLE	PERCENTAGE (%)
Gravel	0 – 10
Sand	60 – 78
Silt	19 – 36
Clay	1 – 4

5.6 Sand and Silt

A native layer of sand and silt was encountered below the silty sand in Boreholes 23-104 and 23-105. The layer was not fully penetrated in the boreholes but was proven to have a thickness of at least 4.9 to 8.2 m and extend to depths of 30.8 m and 31.1 m below the ground surface (elev. 199.0 m and 197.4 m), respectively. SPT N-values ranged from 10 and 44 blows, indicating a compact to dense relative density. Lower N-values were obtained near the base of Borehole 23-105 but are inferred to be due to disturbance from hydraulic pressures.

The recorded moisture contents ranged from 10 to 17%. The results of gradation analyses completed on three samples of the deposit are illustrated on Figures C19 of Appendix C. The results of the tests are summarized below and on the Record of Borehole sheets in Appendix B.

SOIL PARTICLE	PERCENTAGE (%)
Gravel	0 – 2
Sand	37 – 41
Silt	55 – 56
Clay	3 – 6

The results of Atterberg Limit testing conducted on the fines portion of three samples of the deposit indicate a non-plastic material.

5.7 Bedrock and Refusal

The SCPTu testing carried out by ConeTec encountered a refusal at a depth of 22.2 and 18.9 m (elev. 229.3 and 229.8 m) in Test Holes 23-107 and 23-108, respectively.

A dynamic cone penetration test (DCPT) was carried out below the sampled depth in Borehole 23-105, and a refusal blow count was encountered at a depth of 31.7 m (base elev. 196.8 m).

Borehole 23-104 was terminated with an SPT achieving a refusal blow count at a depth of 30.8 m (elev. 199.0 m)

Bedrock was proven by coring in Boreholes 23-101 and 23-102 which were advanced near the east abutment, and the bedrock surface was encountered at a depth of 22.7 to 24.6 m (elev. 204.7 to 204.3 m). The bedrock encountered consisted of slightly weathered to fresh jointed, medium grained, whitish grey, strong to very strong Paragneiss containing quartz inclusions. Photographs of the bedrock core are provided in Appendix C. The rock core quality measurements are summarized in the following table.

Table 5-3 Bedrock Details

PARAMETER	RANGE
Total Core Recovery (TCR), %	100
Solid Core Recovery (SCR), %	74 – 100
Rock Quality Designation (RQD), %	72 – 100
Fracture Index (fractures per 0.3 m)(1)	0 – 8
Unconfined Compressive Strength Testing (Mpa)(2)	92 – 209

Notes: (1) Indicated as “FI” on Borehole Logs
(2) Sample tested from Boreholes 23-101 (Runs 1 and 3) and 23-102 (Runs 1 and 2).

Based on the RQD, the bedrock quality is described as fair to excellent (CFEM, 2006). The result of an unconfined compressive strength testing (UCS) indicates that the tested samples of the bedrock are strong to very strong (CFEM, 2006). The results of the UCS testing are included in Appendix C.

5.8 Groundwater Level

The river’s water level was measured by Thurber at elevation 221.2 m on July 5, 2023. The measured groundwater levels from the wells and open boreholes are summarized in Table 5-4.

Table 5-4 Measured Water Levels

Borehole	Bottom of Screen Depth /Elevation (m)	Soil in Zone of Screen	Groundwater Level		Date of Measurement	Comments
			Depth (mbgs)	Elevation (m)		
23-101	-	-	5.6	223.7	2023/07/06	Open Borehole
23-102	12.2 / 214.8	Silt to Sandy Silt	3.7 4.2 4.8 4.1 4.1	223.3 222.8 222.2 222.9 222.9	2023/07/13 2023/07/14 2023/07/15 2023/09/05 2023/09/10	-
23-104	-	-	6.0	223.8	2023/07/07	Open Borehole
23-105	11.7 / 216.8	Silt	6.4 6.4 6.4 5.7 5.5	222.1 222.1 222.1 222.8 223.0	2023/07/11 2023/07/13 2023/07/15 2023/09/05 2023/09/10	-

It should be noted that the values shown above are considered short-term readings and may not reflect groundwater levels at the time of construction. Seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater level may be at a higher elevation after periods of significant and/or prolonged precipitation events.

A Single Well Response Test (SWRT), or “slug test”, was carried out on September 10, 2023 in the monitoring wells installed in Boreholes 23-102 and 23-105 by lowering the water level within the monitoring well and recording the recovery of the water level over time with a data logger. The slug tests were completed and analyzed using the Hvorslev method and the plots of the slug test results are included in Appendix B. The hydraulic conductivity value calculated from the in-situ slug test is summarized in Table 5-5.

Table 5-5 Single Well Response Test Results

Borehole /Monitoring Well	Bottom of Screen Depth /Elevation (m)	Soil in Zone of Screen	Estimated Hydraulic Conductivity (m/s)
23-102	12.2 / 214.8	Silt to Sandy Silt	5×10^{-7}
23-105	12.2 / 216.3	Silt	7×10^{-7}

It should be expected that variations in hydraulic conductivity will exist within the various soils deposits that were encountered. It should also be expected that the thinner layering/seams within the thicker soil layers will exhibit their own dissimilarities.

Both wells were decommissioned following the completion of the testing on September 10, 2023.

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5.9 Analytical Testing

Four soil samples were submitted for analytical testing. The analysis results are included in Appendix C and are summarized in the following table.

Table 5-6 Analytical Test Results

BOREHOLE	23-101	23-102	23-104	23-105
SAMPLE	SS7	ST1	SS9	SS7
DEPTH (ft/m)	20'0" – 22'0" 7.6 – 8.2	10'0" – 12'0" 1.5 – 2.1	20'0" – 22'0" 6.1 – 6.7	15'0" – 17'0" 4.6 – 5.2
ELEVATION (m)	221.7	225.5	223.7	223.9
SOIL TYPE	Silty Clay (CI)	Silty Clay (CI)	Clayey Silt (CL)	Silty Clay (CI) to Clay (CH)
CONDUCTIVITY (µS/cm)	955	222	213	337
pH	7.58	7.36	7.53	7.77
RESISTIVITY (Ohm-cm)	1,050	4,510	4,690	2,960
CHLORIDE (µg/g)	506	18	10	112
SULPHATE (µg/g)	17	35	39	24
SULPHIDE (%)	< 0.04	< 0.04	0.04	< 0.04

6. MISCELLANEOUS

The borehole locations reflect existing site features and access constraints. The as-drilled locations and ground surface elevation were measured by Thurber following completion of the field program. George Downing Estate Drilling Ltd. of Hawkesbury, Ontario, supplied and operated the drill rig used to drill, test, sample, and decommission the boreholes and wells. Cone Penetration Testing (CPT) was performed by ConeTec Investigation Ltd. of Burnaby, B.C. Traffic control was performed in accordance with Ontario Book 7 and was provided by Demora Construction Services Inc. of North Bay, Ontario. The field investigation was supervised on a full-time basis by Mr. D. Amorim Pereira, Geotechnical Technician. Well slug testing was carried out by Mr. I. Khan, EIT. Overall supervision of the field investigation program was provided by Mr. A. de Oliveira, EIT.

Routine geotechnical laboratory testing and one-dimensional consolidation testing were completed by Thurber's laboratory in Ottawa. Specific gravity and UCS testing on bedrock cores were carried out by Stantec's geotechnical laboratory in Ottawa. Analytical testing was completed by Paracel Laboratories Ltd. in Ottawa.

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Interpretation of the factual data and preparation of this report was completed by Mr. D. Amorim Pereira, Geotechnical Technician and A. de Oliveira, EIT. The report was reviewed by Mr. S. Peters, P.Eng. and Dr. P.K. Chatterji, P.Eng., the Designated Principal Contacts for MTO Foundation Projects.

Thurber Engineering Ltd.
Report Prepared By:

Darlan Amorim Pereira, M.Sc.
Geotechnical Technician

Anderson de Oliveira, M.A.Sc., EIT
Engineering Intern

LICENSED PROFESSIONAL ENGINEER
S. PETERS
100194478
April 5, 2024
PROVINCE OF ONTARIO

Stephen Peters, M.A.Sc., P.Eng.
Associate | Geotechnical Engineer

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
April 5, 2024
PROVINCE OF ONTARIO

Dr. P.K. Chatterji, P.Eng.
Designated Principal Contact
Senior Geotechnical Engineer

**FINAL
FOUNDATION INVESTIGATION AND DESIGN REPORT
HIGHWAY 11 POPLAR RAPIDS BRIDGE RAPIDS BRIDGE
HAGGART TOWNSHIP, ONTARIO
ASSIGNMENT NO.: 5021-E-0025
GWP 5278-19-00**

PART 2. ENGINEERING DISCUSSION AND RECOMMENDATIONS

7. GENERAL

This section of the report provides an interpretation of the factual data from Part 1 of this report and presents foundation design recommendations to assist the project team in the preparation of a Design Build package for the replacement of the Poplar Rapids River bridge located on Highway 11 at Sta. 16+150 Haggart Township. Thurber Engineering Limited (Thurber) carried out the assignment as a sub-consultant to LEA Consulting Ltd. (LEA) under Agreement No. 5021-E-0025. The discussion and recommendations presented in this report are based on the information provided by LEA and the factual data obtained during the current field investigation.

This foundation investigation and design report with the interpretation and recommendations are intended for the use of the Ministry of Transportation Ontario and their designer, LEA Consulting Ltd., and shall not be used or relied upon for any other purposes or by any other parties including the construction or design-build contractor. Contractors must make their own interpretation based on the factual data in Part 1 of the report. Where comments are made on construction, they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction must make their own interpretation of the factual information provided as such interpretations may affect equipment selection, proposed construction methods, scheduling and the like.

It is a condition of this report that Thurber's performance of its professional services will be subject to the attached Statement of Limitations and Conditions.

7.1 Background

The bridge site (Site No. 39E-0001/B0) is located on Highway 11 approximately 13.5 km west of the junction of Highway 11 and Highway 634. Highway 11 is a two-lane highway with a width of 11.5 m across the bridge. The four span bridge is 63.8 m long and was constructed in 1944

(Contract 42-03) and rehabilitated in 2004 (Contract 2003-5135). The bridge abutments and piers are indicated to be founded on spread footings.

In general terms, the encountered stratigraphy consists of sand and gravel fill overlaying clayey silt fill over native deposits of clay to silty clay followed by alternating layers of silt to silty sand over bedrock. The river level was measured at elev. 221.2 m (July 2023) and the wells installed in Boreholes 23-102 and 23-105 recorded a groundwater level at elev. 223.0 m (Sept 2023).

7.2 Proposed Structure

It is indicated in the Terms of Reference that this assignment involves Preliminary Design and Design Build Ready services for the replacement of the existing bridge.

A General Arrangement (GA) drawing dated October 2023 was provided by LEA, copy included in Appendix A, and indicates the proposed replacement structure is to be a 61 m long single span bridge with 6 m long wing walls. The underside of the abutments is indicated to be at elevation 223.70 and 223.26 m for the west and east abutments, respectively. The GA drawing indicates that integral abutments are proposed to be founded on a single row of steel H-piles The Highway 11 centerline is to be shifted 7 m north of the existing highway alignment. The existing river is skewed to the northeast and the shift of the structure northward has the centerline of the west and east abutments shifted approximately 10.5 and 8.4 m east, respectively, from their current locations. The new abutments are also indicated to be perpendicular to the highway centerline.

It is understood that the work will require excavations into the existing approach fills and placement of fill around the existing west abutment. It is also understood that a portion of the new bridge will be constructed while two-lane traffic is maintained on the existing structure. Traffic would then be shifted to the new structure, maintaining a single lane of traffic controlled by temporary traffic signals. The following stage would then proceed, starting with demolition of the existing bridge in its entirety. The remaining portion of the new structure would then be built while maintaining the single lane of traffic on the previously completed portion. Temporary protection systems will most likely be needed during the staged construction.

7.3 Applicable Codes and Design Considerations

The geotechnical assessment presented below has been prepared based on the available data regarding the proposed foundations, existing ground surface conditions and in accordance with the Canadian Highway Bridge Design Code (CHBDC) version CSA S6-19.

In accordance with the CHBDC, the analysis and design of the structure takes into consideration the importance of the structure and the consequence associated with exceeding limit states. The importance category and consequence classification are defined by the Regulatory Authority, which, in this case, is the Ministry of Transportation, Ontario (MTO).

It is understood that the replacement structure is designated as a “Major-Route Bridge” importance category. As per Section 6.14.2.1.b of the CHBDC, a “major-route geotechnical system” is required to have 100% of the travelled lanes available with a return period of at least 475-years.

It is understood that the replacement bridge has a consequence classification of *Typical Consequence*, in accordance with Section 6.5.1 of the CHBDC. Accordingly, a consequence factor (Ψ) of 1.0, as per Table 6.1 of the CHBDC, has been used in assessing factored geotechnical resistances. If this consequence classification changes, the geotechnical assessment and recommendations provided within this report will need to be reviewed and revised.

As per Section 6.5.3 of the CHBDC, the degree of site prediction model understanding is considered to be *Typical* based on the current information.

The frost penetration depth and associated recommendations are provided in Section 9.5.

8. SEISMIC CONSIDERATIONS

8.1 Spectral and Peak Acceleration Hazard Values

The seismic hazard data for the CHBDC is based on the fifth-generation seismic model developed by the Geological Survey of Canada (GSC)[6]. The GSC seismic hazard calculation data sheet for this site for the *reference* ground condition (Site Class C) is presented in Appendix F. The site coefficients used to determine the design spectral acceleration values are a function of the Site Class, PGA and Sa(0.2). The PGA value at this site provided by GSC for a *reference* Site Class C with a 2% probability of exceedance in 50 years (2475-year event) is 0.117 g. This value is to be scaled by the F(PGA) based on the *site-specific* Site Class, as discussed in Section 8.3.

[6] *https://earthquakescanada.nrcan.gc.ca/hazard-alea/interpolat/calc-en.php*

8.2 Liquefaction Potential

The susceptibility of the cohesive soils at this site to experience liquefaction/cyclic softening was assessed following the Bray et al. (2004)[7] criteria using index properties and in-situ shear strengths measured in the boreholes. Based on this assessment, the cohesive foundation soils are not considered susceptible to cyclic mobility or cyclic softening under the design earthquake.

The susceptibility of the cohesionless soils at the site to experience liquefaction was assessed using the SPT data following the simplified method for cohesionless soil as outlined in Boulanger and Idriss (2014)[8]. Based on this assessment, the cohesionless foundation soils are not considered to be susceptible to liquefaction under the design earthquake.

8.3 CHBDC Seismic Site Classification and Performance Category

In accordance with Section 4.4.3.2 of the CHBDC, the selection of the seismic site classification is based on the nature of soil deposit within the upper 30 m of the stratigraphy. As per Table 4.1 within Section 4.4.3.2 of the CHBDC, the site has been assessed to be Seismic Site Class D based on the measured shear wave velocities from the SCPT results.

The F(PGA) as per Table 4.8 within Section 4.4.3.3 of the CHBDC is equal to 1.29 for a Site Class D yielding a scaled *site-specific* PGA of 0.151g.

As per Section 4.4.4 of the CHBDC, the Seismic Performance Category is assigned based on the fundamental period, the importance category and the spectral accelerations scaled to the site class. The F(0.2) and F(1.0), as per Tables 4.2 and 4.4 within Section 4.4.3.3 of the CHBDC, is equal to 1.24 and 1.55, respectively for this site yielding a scaled *site-specific* Sa(0.2) of 0.22 and Sa(1.0) of 0.068. A Seismic Performance Category of either 1 or 2 could be applicable for this site based on Table 4.10 of the CHBDC. The seismic performance category should be confirmed by the structural engineer.

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[7] *Bray, J. D. et al. (2004b). Liquefaction susceptibility of fine-grained soils. Proc., 11th Int. Conf. onSoil Dynamics and Earthquake Engineering and 3rd Int. Conf. on Earthquake Geotechnical Engineering, D. Doolin et al., eds., Stallion Press, Singapore, 655–662*

[8] *Boulanger, R. W., and Idriss, I. M. (2014). CPT and SPT based liquefaction triggering procedures, Report No. UCD/CGM-14/01, Center for Geotechnical Modeling, Department of Civil and Environmental Engineering, University of California, Davis, CA, 134 pp.*

9. FOUNDATION DESIGN RECOMMENDATIONS

9.1 Foundation Type

Given the proposed staged construction in proximity of the existing structure and the soil stratigraphy, depth to bedrock and groundwater conditions encountered during the course of the current field investigation, driven pile foundations are considered suitable to support the new abutments for the single span replacement bridge. The underside of the pile caps should meet the recommendations for frost depth, see Section 9.5.

Spread footings have been considered but would require excavations in close proximity to the existing shallow foundations and would be more sensitive to differential settlement as a result of staged construction and placement of new fill to raise the grade. Additionally, dewatering would be a larger concern for spread footings. Spread footings are therefore not recommended. The site is also not considered suitable for caisson foundations based on the proximity of the existing shallow foundations, cohesionless soils below the water table and the depth to bedrock.

9.2 Deep Foundations

9.2.1 Axial Capacity of Driven Piles

Bedrock was proven by coring in Boreholes 23-101 and 23-102 at the east abutment and refusal was encountered in Boreholes 23-104 and 23-105 at the west abutment. Differing refusal elevations were encountered between the abutments and a summary of refusal elevations is provided in the table below.

Table 9-1 Refusal Elevation

Borehole	Location	Refusal Elevation	Comments
23-101	East Abutment	204.7	Bedrock Cored
23-102	East Abutment	204.3	Bedrock Cored
23-104	West Abutment	199.0	SPT Refusal
23-105	West Abutment	196.8	DCPT Refusal

Driven steel piles are considered a feasible option to support the bridge abutments. The geotechnical resistances for HP310 x 110 piles are provided in the table below.

Table 9-2 Axial Geotechnical Resistance for HP310x110

Location	Factored Compression Resistance		Factored Tension Resistance
	ULS (kN)	SLS (kN)	ULS (kN)
East Abutment	2,100	Will not govern	450
West Abutment	1,200*	1,000*	500

Note: (*) bedrock was not proven, resistance value presented is based on encountering dense soil layer at pile tip

The geotechnical resistances provided in the table above include a resistance factor of 0.4 (ϕ_{gu}), 0.8 (ϕ_{gs}) and 0.3 (ϕ_{gu}) for ULS compression, SLS compression and ULS tension values, respectively, as per Table 6.2 of the CHBDC (static analysis – typical understanding).

To prevent differing founding conditions, it is recommended that both abutments have piles driven to bedrock. During detailed design, an additional investigation could be carried out at the west abutment to confirm the bedrock elevation(s). The axial capacity for piles driven to bedrock at the west abutment can use the factored compression resistance values presented for the east abutment as indicated in Table 9-2.

The structural resistance of the pile must be checked by the structural engineer.

9.2.2 Downdrag

Time dependent settlement of the cohesive foundation soils will occur due to the proposed grade raise and permanent embankment widening. As a result, deep foundations at the abutment locations will need to be designed to carry the additional static downdrag loads developed along the length of the piles embedded in the cohesive layers and overlying materials. The unfactored static downdrag load for HP 310x110 is estimated to be up to 400 kN.

This downdrag load should be factored in accordance with the CHBDC. In accordance with Section 6.11.4.10 of the CHBDC and Clause C6.11.4.10 of the Commentary, in the structural design of a pile, the factored downdrag load should be added to the factored permanent loads to assess the effects of downdrag. In geotechnical analysis of downdrag, live load effects should not be considered.

The neutral plane for static downdrag calculations can be taken as elev. 211 m.

9.2.3 Lateral Geotechnical Resistance and Group Effects

Piles can be installed with a batter to resist lateral loads.

Alternatively, the lateral resistance of the soil adjacent to a vertical pile is developed on the face of the pile embedded in the foundation soils and estimated using p-y curves. The p-y curves for static conditions are shown in Appendix G to allow for calculation of the ultimate lateral resistance of an individual pile. A geotechnical resistance factor of 0.5 (ϕ_{gu}) and 0.8 (ϕ_{gs}) as per Table 6.2 of the CHBDC (static analysis – typical understanding) *should be applied* to the ultimate ULS and SLS values, respectively. Note that the information provided in Appendix G is based on the pile cap elevation shown on the October 2023 GA Drawing and will need to be reviewed if the pile caps are modified.

Where lateral spacing between an adjacent pile or another structural element is less than four equivalent pile diameters, the lateral resistance will also need to be further reduced based on the center-to-center spacing. The reduction factors to be used can be obtained from Figures C6.22, C6.23 and C6.24 of the CHBDC.

9.2.4 Pile Tips

It is expected the pile installation will encounter cobbles and boulders. Care must be exercised while driving into dense soils and to bedrock. The tips of all piles must be protected from damage when driving and should be fitted with a Titus Steel (standard H-Point) or approved equal.

9.2.5 Pile Driving

Pile driving must be carried out in accordance with OPSS.PROV 903. The appropriate pile driving note is “Piles to be driven to bedrock“.

Retapping of piles as per OPSS.PROV 903 is not necessary for pile driven to bedrock.

Vibration and settlement monitoring of the existing bridge and newly constructed foundations should be carried out during installations.

Pile locations/layout may need to be selected to avoid the potential for interference with existing foundation elements.

9.2.6 Abutment Type

The subsurface conditions at this site are considered suitable semi-integral or conventional type abutment designs. The use of a single row of vertical piles would allow for the design of an integral abutment.

9.3 Wingwalls

Based on current GA drawing, it is understood that the wingwalls will be cantilevered from the abutment stem.

9.4 Backfill and Lateral earth Pressure

Structural backfill material should consist of Granular A meeting the OPSS.PROV 1010 specifications and SP110S06. The backfill must be in accordance with OPSS 902 and placed to the extents shown on OPSD 3121.150. The backfill should be compacted in accordance with OPSS.PROV 501 and compaction equipment to be used adjacent to the structure must be restricted in accordance with OPSS.PROV 501.07.02.

The design of the abutments and wingwall must incorporate a subdrain as shown in OPSD 3101.150. Lateral earth pressure provided in the equations in the sections below are based on the assumption that the backfill is fully drained so that there are no unbalanced hydrostatic pressures. If adequate drainage cannot be confirmed, the potential for buildup of hydrostatic pressures should be considered in wall design. A lateral earth pressure due to backfill compaction should be added to the calculated lateral earth pressure in accordance with Clause 6.12.3 of the CHBDC.

Lateral earth pressures acting on vertical structures should be computed in accordance with the Section 6.12 of the CHBDC but under fully drained conditions, the lateral pressures are generally given by the following expression:

$$\sigma_h = K * (\gamma * d + q) \qquad \text{[static]}$$

$$\sigma_{hAE} = K * \gamma * d + (K_{AE} - K_A) * \gamma * (H - d) \qquad \text{[combined static and seismic]}$$

where:

σ_h	=	static lateral earth pressure on the wall at depth d (kPa)
σ_{hAE}	=	combined static and seismic lateral earth pressure on wall at depth d (kPa)
K	=	static earth pressure coefficient (see table below)
		(K_A for yielding walls, K_o for non-yielding walls)
K_{AE}	=	combined static and seismic earth pressure coefficient (see table below)
γ	=	unit weight of retained soil (see table below)
		adjusted to submerged unit weights below water level
d	=	depth below top of fill where pressure is computed (m)
H	=	total height of the wall (m)
q	=	value of any surcharge (kPa)

Static Lateral Earth Pressure Coefficients

Typical earth pressure coefficients for vertical walls for backfill material are shown in Table 9-3.

Table 9-3. Static Earth Pressure Coefficients

Material	Unit Weight (kN/m^3)	K_A (yielding wall)	K_0 (non-yielding wall)	K_p (movement toward soil)	Ground Surface *Behind Wall*
OPSS Granular A	22.8	0.27	0.43	3.7	Horizontal
		0.40	-	-	2H:1V
OPSS Granular B Type II	22.0	0.27	0.43	3.7	Horizontal
		0.40	-	-	2H:1V

The parameters in the table correspond to full mobilization of active and passive earth pressures and require certain relative movements between the wall and adjacent soil to produce these conditions. Figure C6.27 and Table C6.12 of the Commentary to the CHBDC indicates the relative movement required to fully mobilize the active earth pressure. Where ground surfaces are sloped at 2H:1V behind the walls, the corresponding coefficients in Table 9-3 should be used.

If lateral movement is not permissible and/or the wall is restrained, the at-rest/non-yielding earth pressure coefficient should be used. If the wall design allows lateral movement, the active/yielding earth pressures should be used.

A geotechnical resistance factor of 0.5 (ϕ_{gu}) should be applied in static design to the *passive* earth pressures in accordance with Table 6.2 of the CHBDC (static analysis - *typical* understanding). The soils within the depth of frost should be ignored from providing passive lateral resistance, however the equivalent surcharge loading from the weight of the soils above the frost depth should be incorporated into the lower soil layers.

Combined Static and Seismic Lateral Earth Pressure Coefficients

In accordance with Clause 6.14 of the CHBDC, structures should be designed using dynamic earth pressure coefficients that incorporate the effects of earthquake loading. The following recommendations are per Section C6.14.7.2 of the Commentary of the CHBDC which states that seismically induced lateral soil pressures may be calculated using Mononobe Okabe Method with:

- k_h = ½ * F(PGA) * PGA, for structures that allow 25 to 50 mm of movement, and
- k_h = F(PGA) * PGA, for non-yielding walls

The coefficients of horizontal earth pressure for seismic loading presented in Table 9-4 may be used for vertical walls. The provided earth pressure coefficients are based on a 1 in 2475yr seismic event and a Seismic Site Class D.

Table 9-4. Combined Static and Seismic Earth Pressure Coefficients

Material	Unit Weight (kN/m^3)	K_{AE} (yielding wall)	K_{AE} (non-yielding wall)	Ground Surface *Behind Wall*
OPSS Granular A	22.8	0.31	0.36	Horizontal
		0.48	0.58	2H:1V
OPSS Granular B Type II	22.0	0.31	0.36	Horizontal
		0.48	0.58	2H:1V

9.5 Frost Depth

The frost penetration depth at this site is 2.55 m as per OPSD 3090.100.

For all footings a minimum of 2.55 m of earth cover, or thermal equivalent, must be provided above the base of the footings to serve as protection against frost. The earth cover should be measured perpendicular to the ground surface. Thermally equivalent frost protection could be in the form of insulation provided it is placed *above* the high-water level. It should be noted that porous materials, such as rock protection, does not have the same thermal protection as soil.

9.6 Cement Type and Corrosion Potential

Analytical tests were completed to determine the pH, water soluble sulphate, sulphide, chloride concentration, resistivity and electrical conductivity of samples of the soil. The analysis results are summarized in Section 5.9 and a copy of the test results is provided in Appendix C. The pH, resistivity and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. The tests results provided in Section 5.9 may be used to aid in the selection of coatings and corrosion protection systems for buried steel objects. The corrosive effects of road de-icing salts should also be considered.

The concentration of soluble sulphate provides an indication of the degree of sulphate attack that is expected for concrete in contact with soil and groundwater at the site. Soluble sulphate concentrations less than 1000 µg/g generally indicate that a low degree of sulphate attack is expected for concrete in contact with soil and groundwater. The sulphate content in the soils is low ranging from 17 to 35 µg/g, see Section 5.9. The selection for class of concrete should include consideration of the effects of road de-icing salts.

9.7 Embankment Design and Construction

It is understood that one or two lanes of traffic is required to be maintained during construction and that replacement of the existing bridge would be carried out in stages. The new bridge alignment is proposed to be constructed 7 m north of the existing alignment and the elevation of the highway approach embankment is to match the existing highway elevation.

9.7.1 Embankment Construction

The existing highway embankment side slopes are not sloped steeper than 2H:1V. The existing slopes did not show any visible signs of global instability at the time of the investigation, but localized surficial erosion was noticed. Embankment reinstatement beyond the limits of structural backfill should be carried out in accordance with OPSS.PROV 206. If constructed using Select Subgrade Material (SSM) or Granular B Type I, the embankment should be reconstructed with side slopes of 2H:1V, or flatter. The fill should be placed and compacted in accordance with OPSS.PROV 501.

Prior to placement of fill, topsoil organic or otherwise deleterious soils should be removed. Where newly placed embankment fill is placed against existing embankment slopes or on a sloping ground surface steeper than 3H:1V, benching of the existing slope should be carried out in accordance with OPSD 208.010.

9.7.2 Settlement

Based on the existing and proposed ground surfaces received from LEA, the new west abutment is shown to be in front of the existing abutment and the ground surface to backfill behind the new abutment is indicated to be approximately 3.2 m higher than the existing ground surface. The new east abutment is shown behind the existing abutment and the grade raise for the embankment widening is up to 2.9 m at the new north shoulder with no increase at the new centerline.

An analysis was carried out to estimate the settlement of the foundation soils under the weight of the proposed new embankment fill which is up to 3.2 m. In accordance with MTO's settlement criteria, up to 25 mm of post construction settlement is allowed within 20 m of the abutments.

The magnitude of the self-compression of an embankment constructed with SSM is approximately 25 mm.

The estimated settlement of the foundation soils utilized the one-dimensional consolidation properties from testing carried out as part of the current study as summarized in Table 5-1 and Table 5-2. It is estimated that approximately 55 mm of settlement will occur at the new centerline of the west abutment as a result of the 3.2 m of grade raise. It is recommended that a minimum delay of 1 month is carried prior to paving to allow for post construction settlement to meet MTO's criteria.

Based on the requirement to have centerline temporary protection systems to separate the new and existing structures during the staged construction, the settlement of the newly placed fill should have limited impact on the existing foundations.

9.7.3 Embankment Stability

Stability analyses were carried out for the proposed conditions utilizing the commercially available slope stability program Slope/W of the GeoStudio software package developed by Geo-Slope International with the option of Morgenstern-Price method of slices for limit equilibrium.

The input parameters and soil model used in the stability analyses, including soil stratigraphy, engineering properties, groundwater conditions and modeled geometry for the west abutment analyses are shown in Appendix G. The material properties used in the analyses were determined from in-situ and laboratory testing conducted during the current study, soil index correlations developed during current and past projects. The computed factor of safety meets the requirements of Table 6.2 of Section 6.9.1 of the CHBDC.

10. CONSTRUCTION CONSIDERATIONS

10.1 Excavation

All excavation must be conducted in accordance with the requirements of the Occupational Health & Safety Act & Regulations (OHSA) for Construction Projects. The fill materials and native soils above the water table are Type 3 soil. For excavations that extend below the water table, the soils are Type 4 soils *If an excavation penetrates more than one soil type, the entire excavation must be completed in accordance with the more stringent requirement as per O.Reg. 213/91, s. 227 (3)*.

Excavation should occur in a dewatered environment (see Section 10.3). Excavations must be planned and carried out in a manner to not impact the stability of the roadway, existing structure and new structure. The temporary cut slopes may have to be protected from precipitation and runoff to avoid erosion and surficial instabilities. The duration of temporary open excavations and cut slopes should be minimized to reduce the likelihood of causing instability concerns. Temporary embankment and cut slope stability is the responsibility of the Contractor.

Excavation for the structure must be carried out in accordance with OPSS 902 as amended by SP 109S12 and NSSP FOUN0003 and will be carried out through the existing fill and native soils. Please refer to Section 10.3 for designer fill-in recommendations. Selection of the equipment and methodology to excavate and prepare the founding surface is the responsibility of the Contractor.

Material stockpiling is a temporary construction measure and the associated stability implications are the responsibility of the Contractor. The selection and placement of construction equipment (such as cranes) and the construction of temporary construction access roads are also the Contractor's responsibility. Placement of the crane or temporary stockpiling must not destabilize the embankment.

10.2 Temporary Protection Systems

Temporary Protection Systems (TPS) will be required during various stages of construction and must be implemented in accordance with OPSS.PROV 539 as amended by SP 105S09. Performance Level 2 (maximum 25 mm horizontal deflection) is considered appropriate where the protection supports the existing highway. More stringent performance levels are required for the protection system to support existing structures or utilities. The actual pressure distribution acting on the shoring system is a function of the construction sequence and the relative flexibility of the wall and these factors must be considered when designing the shoring system.

The design of the roadway protection system is the responsibility of the Contractor. All protection systems should be designed by a licensed Professional Engineer experienced in such designs and retained by the Contractor. The design of the roadway protection system must incorporate traffic loading and surcharge loading due to construction equipment and operations.

Steel sheet piles are considered a suitable option, however. A suitable anchoring and/or bracing system may need to be incorporated into the temporary protection design to resist lateral earth pressure loadings.

Lateral earth pressure coefficients, under fully mobilized conditions, that can be used in design of the protection system installed through granular fill materials are provided in the Table 9-3 for static conditions. The lateral earth pressure coefficients for the existing soils are given below for a vertical wall and horizontal backslope. Unit weights provided herein are to be adjusted when applied to depths below the water level. Unbalanced hydrostatic pressures should also be considered in the design of the protection systems.

Table 10-1 Static Earth Pressure Coefficients for Existing Soil

Material	Unit Weight(*) (kN/m^3)	K_A	K_P	Ground Surface Behind Structure
Existing Fills	21	0.33	3.0	Horizontal
Native Clay & Silt Layers	18	0.36	2.8	Horizontal

Note: (*) to be adjusted when below water level

The use of vibration for installation and of sheet pile protection systems could result in settlement and must be carried out cautiously. It is recommended that the protection systems within 3 m from the edges of the wingwalls/structure should be left in place and cut off in accordance with OPSS.PROV 539.

10.3 Surface and Groundwater Control

Bridge construction, subgrade preparation and placement and compaction of granular materials must be carried out in the dry. Based on the groundwater elevation at the time of the investigation, it is anticipated that the underside of the pile caps will intercept the groundwater level. Dewatering will likely be required to lower the groundwater to below the excavation for the abutments. Excavations that extend below the groundwater level without prior dewatering are not recommended since the inflow of groundwater will make it difficult to maintain a dry, sound base on which to work. Surface runoff will tend to seep into and accumulate into the excavations and flow diversion and/or a dewatering system may be required. The Contractor must control groundwater, perched groundwater, and surface water flow at the site to permit construction in a dry and stable excavation. The dewatering system will be required to remain operational and effective until the temporary excavations are backfilled and then should be decommissioned and removed.

The design of dewatering systems is the responsibility of the Contractor. The Contract Documents must alert the Contractor to this responsibility and to design the dewatering systems in accordance with SP FOUN0003 which amends OPSS 902 and SP51701 which amends OPSS.PROV 517. Given the site conditions and anticipated works, the design Engineer and design-checking Engineer do not need a minimum of 5 years of experience in designing similar dewatering systems and the Designer Fill-In ***** in SP517F01 Table A should be a “No”. A preconstruction survey is not required, thus Designer Fill-In ** in SP FOUN0003 should be “N/A”.

The surface water level will fluctuate and the minimum water elevation for the site at the time of the excavation should be taken as the expected high-water level defined in the contract documents.

It is anticipated that sump pumps will likely be sufficient to extract surface water from the excavation for the bridge abutments. A sheet pile enclosure should be considered for excavations that extend below the proposed underside of abutments. Pumping should continue until control of inflow is achieved and the excavations are backfilled. Multiple pumps may be required.

Assessment of the dewatering requirements and the need for registration on the Environmental Activity and Sector Registry (EASR) or a Permit to take Water (PTTW) should be carried out by specialists experienced in this field.

10.4 Scour and Erosion Protection

The Contractor should provide silt fences and erosion control blankets as per OPSS 805 and OPSD 219.110 throughout the duration of construction to prevent transport of silt/sediment.

Slope protection and drainage measures will be required to ensure the long-term surficial stability of the embankment and cut slopes. A vegetation cover should be established on all exposed earth surfaces to protect against surficial erosion in general accordance with OPSS.PROV 803 and OPSS.PROV 804. Slope vegetation should be established as soon as possible after completion of construction in order to limit surficial erosion and water should be prevented from running down an unprotected slope. Existing erosion protection material at this location must be reinstated.

Scour and erosion protection must be provided along the waterline to protect the abutments. Design of the protection measure must consider hydrologic and hydraulic factors and shall be carried out by specialists experienced in this field. Typically, rock protection should be provided over all earth surfaces subjected to flowing water in accordance with OPSS.PROV 511.

The existing foundation elements should be cut off at the ground surface. Any excavations could disturb the existing riverbanks and riverbed.

11. CONSTRUCTION CONCERNS

Potential construction concerns include, but are not necessarily limited to:

- <u>Existing Structure</u>

  The new structure will be in close proximity to the existing structure. Temporary protection systems and construction of foundations to support the new structure will occur close to existing foundations. The construction planning and activities will need to be carried out in a manner that does not destabilize the existing structure.

- Equipment Selection

  The Contractor's selection of construction equipment and methodology must include assessment of the capability of the existing soils to support the proposed construction equipment and supplies.

The successful performance of the project will depend largely upon good workmanship and quality control during construction. Subgrade examination and field density testing should be carried out by qualified personnel during construction to confirm that foundation recommendations are correctly implemented, and material specifications are met.

12. CLOSURE

Engineering analysis and preparation of this report were carried out by Mr. Stephen Peters, P.Eng. The report was reviewed by Fred Griffiths, P.Eng., and P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundation Projects.

Thurber Engineering Ltd.
Report Prepared By:

Stephen Peters, P.Eng.
Associate | Geotechnical Engineer

P.K. Chatterji, P.Eng., Ph.D.
Designated Principal Contact
Senior Geotechnical Engineer

THURBER

STATEMENT OF LIMITATIONS AND CONDITIONS

1. STANDARD OF CARE

This Report has been prepared in accordance with generally accepted engineering or environmental consulting practices in the applicable jurisdiction. No other warranty, expressed or implied, is intended or made.

2. COMPLETE REPORT

All documents, records, data and files, whether electronic or otherwise, generated as part of this assignment are a part of the Report, which is of a summary nature and is not intended to stand alone without reference to the instructions given to Thurber by the Client, communications between Thurber and the Client, and any other reports, proposals or documents prepared by Thurber for the Client relative to the specific site described herein, all of which together constitute the Report.

IN ORDER TO PROPERLY UNDERSTAND THE SUGGESTIONS, RECOMMENDATIONS AND OPINIONS EXPRESSED HEREIN, REFERENCE MUST BE MADE TO THE WHOLE OF THE REPORT. THURBER IS NOT RESPONSIBLE FOR USE BY ANY PARTY OF PORTIONS OF THE REPORT WITHOUT REFERENCE TO THE WHOLE REPORT.

3. BASIS OF REPORT

The Report has been prepared for the specific site, development, design objectives and purposes that were described to Thurber by the Client. The applicability and reliability of any of the findings, recommendations, suggestions, or opinions expressed in the Report, subject to the limitations provided herein, are only valid to the extent that the Report expressly addresses proposed development, design objectives and purposes, and then only to the extent that there has been no material alteration to or variation from any of the said descriptions provided to Thurber, unless Thurber is specifically requested by the Client to review and revise the Report in light of such alteration or variation.

4. USE OF THE REPORT

The information and opinions expressed in the Report, or any document forming part of the Report, are for the sole benefit of the Client. NO OTHER PARTY MAY USE OR RELY UPON THE REPORT OR ANY PORTION THEREOF WITHOUT THURBER'S WRITTEN CONSENT AND SUCH USE SHALL BE ON SUCH TERMS AND CONDITIONS AS THURBER MAY EXPRESSLY APPROVE. Ownership in and copyright for the contents of the Report belong to Thurber. Any use which a third party makes of the Report, is the sole responsibility of such third party. Thurber accepts no responsibility whatsoever for damages suffered by any third party resulting from use of the Report without Thurber's express written permission.

5. INTERPRETATION OF THE REPORT

a) Nature and Exactness of Soil and Contaminant Description: Classification and identification of soils, rocks, geological units, contaminant materials and quantities have been based on investigations performed in accordance with the standards set out in Paragraph 1. Classification and identification of these factors are judgmental in nature. Comprehensive sampling and testing programs implemented with the appropriate equipment by experienced personnel may fail to locate some conditions. All investigations utilizing the standards of Paragraph 1 will involve an inherent risk that some conditions will not be detected and all documents or records summarizing such investigations will be based on assumptions of what exists between the actual points sampled. Actual conditions may vary significantly between the points investigated and the Client and all other persons making use of such documents or records with our express written consent should be aware of this risk and the Report is delivered subject to the express condition that such risk is accepted by the Client and such other persons. Some conditions are subject to change over time and those making use of the Report should be aware of this possibility and understand that the Report only presents the conditions at the sampled points at the time of sampling. If special concerns exist, or the Client has special considerations or requirements, the Client should disclose them so that additional or special investigations may be undertaken which would not otherwise be within the scope of investigations made for the purposes of the Report.

b) Reliance on Provided Information: The evaluation and conclusions contained in the Report have been prepared on the basis of conditions in evidence at the time of site inspections and on the basis of information provided to Thurber. Thurber has relied in good faith upon representations, information and instructions provided by the Client and others concerning the site. Accordingly, Thurber does not accept responsibility for any deficiency, misstatement or inaccuracy contained in the Report as a result of misstatements, omissions, misrepresentations, or fraudulent acts of the Client or other persons providing information relied on by Thurber. Thurber is entitled to rely on such representations, information and instructions and is not required to carry out investigations to determine the truth or accuracy of such representations, information and instructions.

c) Design Services: The Report may form part of design and construction documents for information purposes even though it may have been issued prior to final design being completed. Thurber should be retained to review final design, project plans and related documents prior to construction to confirm that they are consistent with the intent of the Report. Any differences that may exist between the Report's recommendations and the final design detailed in the contract documents should be reported to Thurber immediately so that Thurber can address potential conflicts.

d) Construction Services: During construction Thurber should be retained to provide field reviews. Field reviews consist of performing sufficient and timely observations of encountered conditions in order to confirm and document that the site conditions do not materially differ from those interpreted conditions considered in the preparation of the report. Adequate field reviews are necessary for Thurber to provide letters of assurance, in accordance with the requirements of many regulatory authorities.

6. RELEASE OF POLLUTANTS OR HAZARDOUS SUBSTANCES

Geotechnical engineering and environmental consulting projects often have the potential to encounter pollutants or hazardous substances and the potential to cause the escape, release or dispersal of those substances. Thurber shall have no liability to the Client under any circumstances, for the escape, release or dispersal of pollutants or hazardous substances, unless such pollutants or hazardous substances have been specifically and accurately identified to Thurber by the Client prior to the commencement of Thurber's professional services.

7. INDEPENDENT JUDGEMENTS OF CLIENT

The information, interpretations and conclusions in the Report are based on Thurber's interpretation of conditions revealed through limited investigation conducted within a defined scope of services. Thurber does not accept responsibility for independent conclusions, interpretations, interpolations and/or decisions of the Client, or others who may come into possession of the Report, or any part thereof, which may be based on information contained in the Report. This restriction of liability includes but is not limited to decisions made to develop, purchase or sell land.

THURBER ENGINEERING LTD.

APPENDIX A

Borehole Locations and Strata Drawing

General Arrangement Drawing (October 2023)

METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

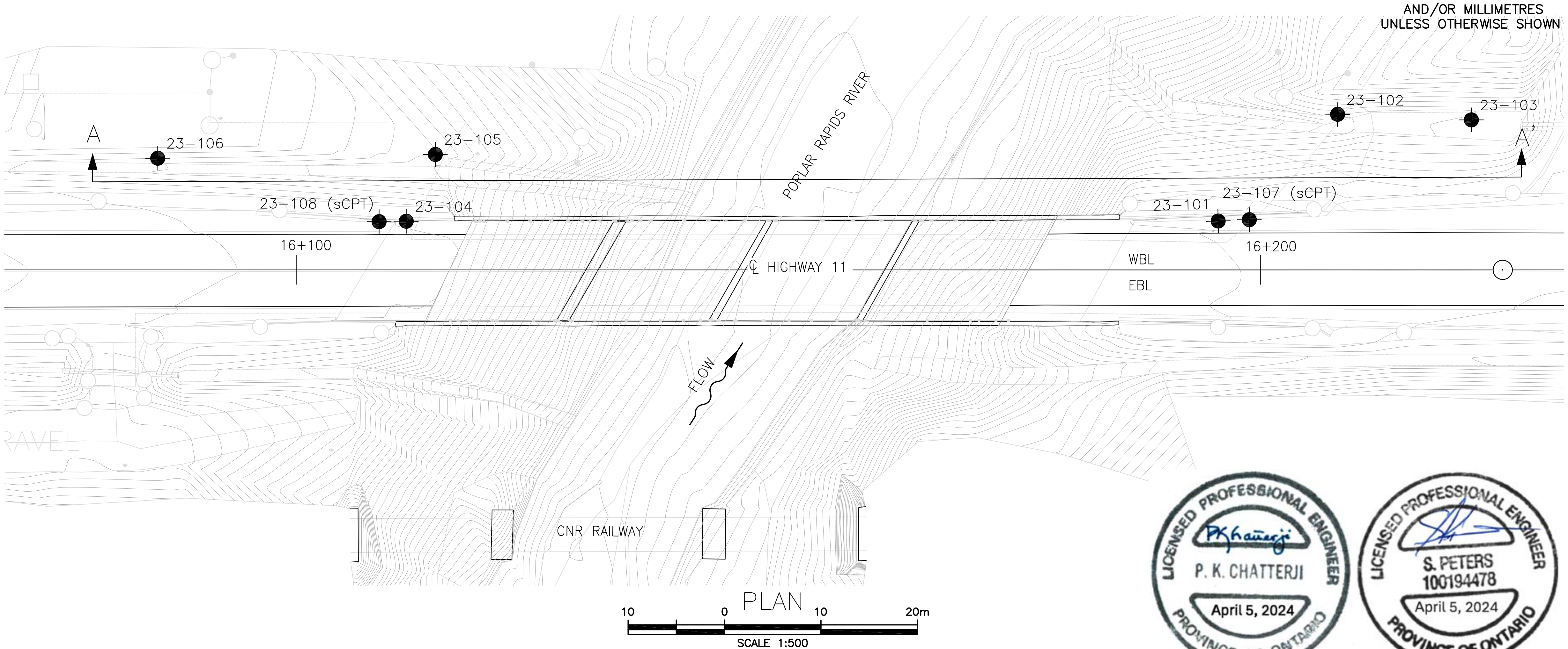
CONT No
GWP No 5021–E–0025

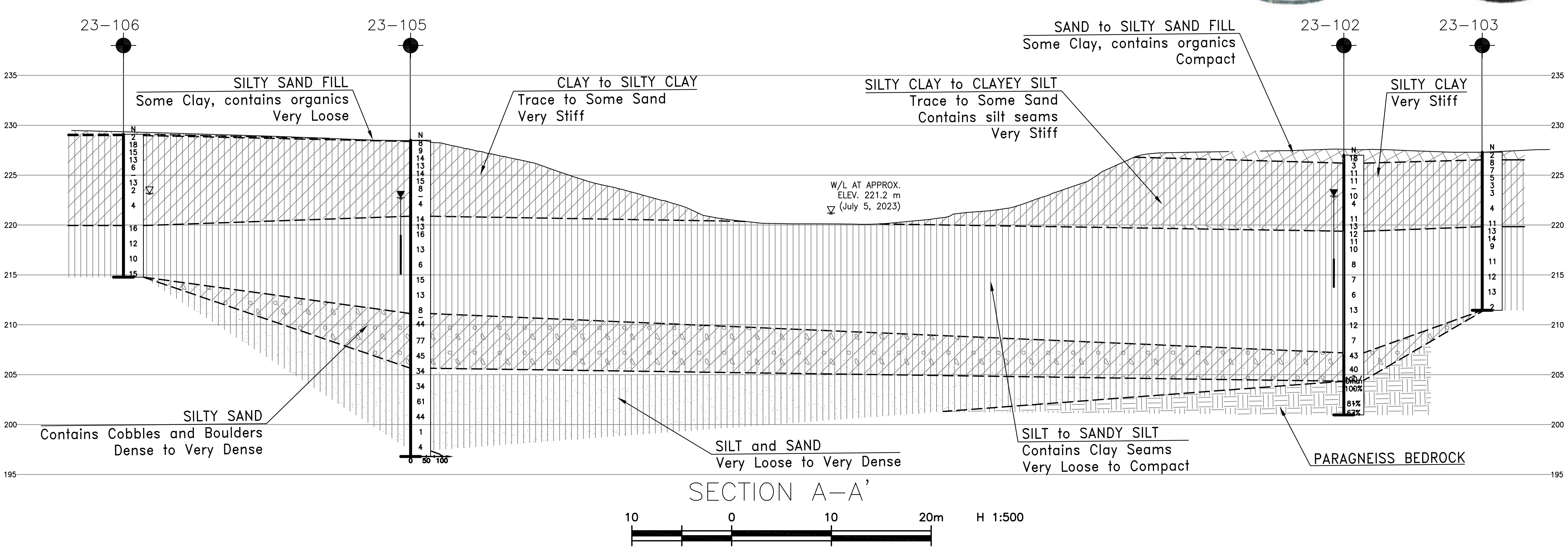
HIGHWAY 11
POPLAR RAPIDS BRIDGE
HAGGART TOWNSHIP
BOREHOLE LOCATIONS AND SOIL STRATA

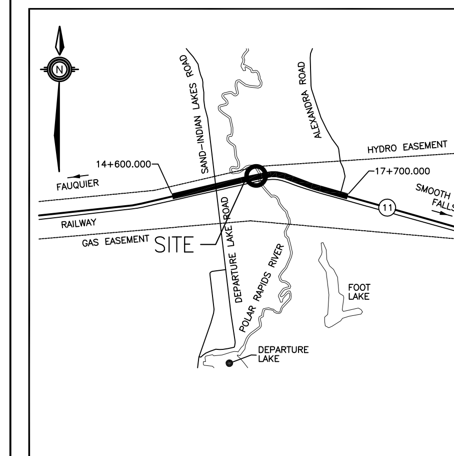
SHEET 1

Ontario

THURBER ENGINEERING LTD.







KEYPLAN

LEGEND

Symbol	Description
●	Borehole
⊕	Borehole (Previous Investigation)
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
▽	Water Level Upon Completion of Drilling
▼	Water Level in Monitoring Well/Piezometer
▮	Monitoring Well/Piezometer Screen
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
23–101	229.3	5 461 553.9	247 779.4
23–102	227.0	5 461 566.1	247 791.1
23–103	227.3	5 461 567.1	247 804.1
23–104	229.8	5 461 545.7	247 695.6
23–105	228.5	5 461 553.1	247 698.1
23–106	229.1	5 461 550.1	247 669.1
23–107	229.5	5 461 554.2	247 782.4
23–108	230.0	5 461 545.5	247 692.6

-NOTES-

1) The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
2) This drawing is for subsurface information only. Surface details and features are for conceptual illustration.
3) Coordinate system is MTM NAD 83 Zone 12.

GEOCRES No. 42H00-095

REVISIONS

DATE	BY	DESCRIPTION

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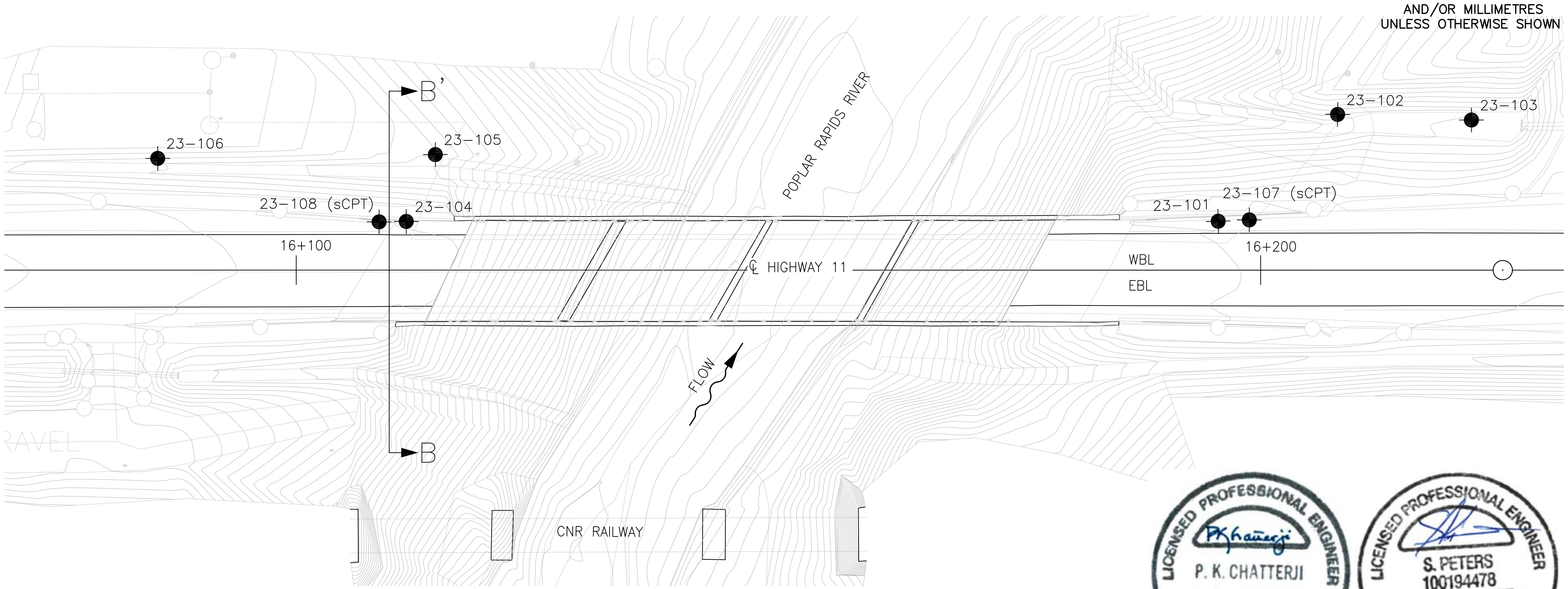
CONT No
GWP No 5021–E–0025

HIGHWAY 11
POPLAR RAPIDS BRIDGE
HAGGART TOWNSHIP
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET 2

Ontario

THURBER ENGINEERING LTD.



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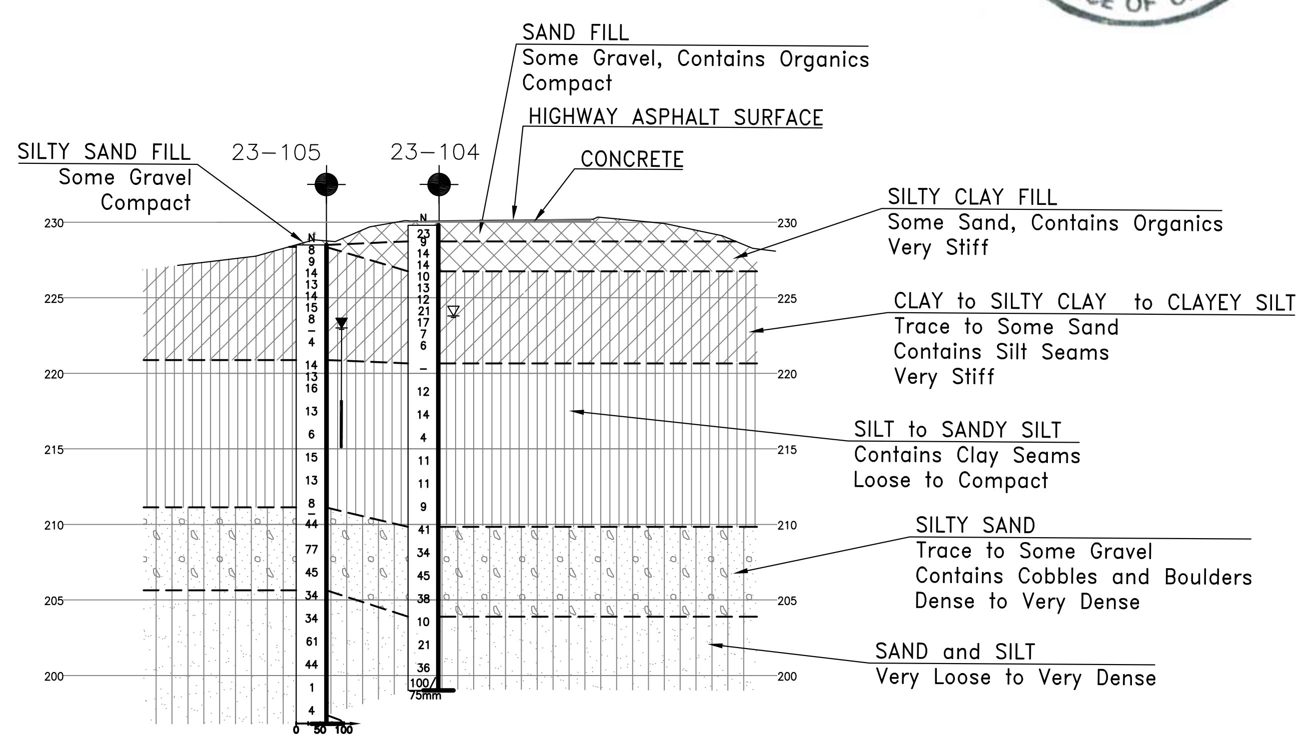


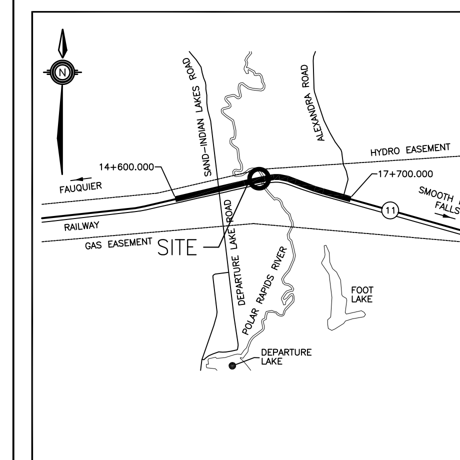
SECTION B–B'

H 1:500

V 1:250

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
April 5, 2024
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
S. PETERS
100194478
April 5, 2024
PROVINCE OF ONTARIO



KEYPLAN

LEGEND

Symbol	Description
●	Borehole
⊕	Borehole (Previous Investigation)
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
▽	Water Level Upon Completion of Drilling
▼	Water Level in Monitoring Well/Piezometer
ǀ	Monitoring Well/Piezometer Screen
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
23–101	229.3	5 461 553.9	247 779.4
23–102	227.0	5 461 566.1	247 791.1
23–103	227.3	5 461 567.1	247 804.1
23–104	229.8	5 461 545.7	247 695.6
23–105	228.5	5 461 553.1	247 698.1
23–106	229.1	5 461 550.1	247 669.1
23–107	229.5	5 461 554.2	247 782.4
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GEOCRES No. 42H00-095

REVISIONS	DATE	BY	DESCRIPTION

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MINISTRY OF TRANSPORTATION, ONTARIO

METRIC
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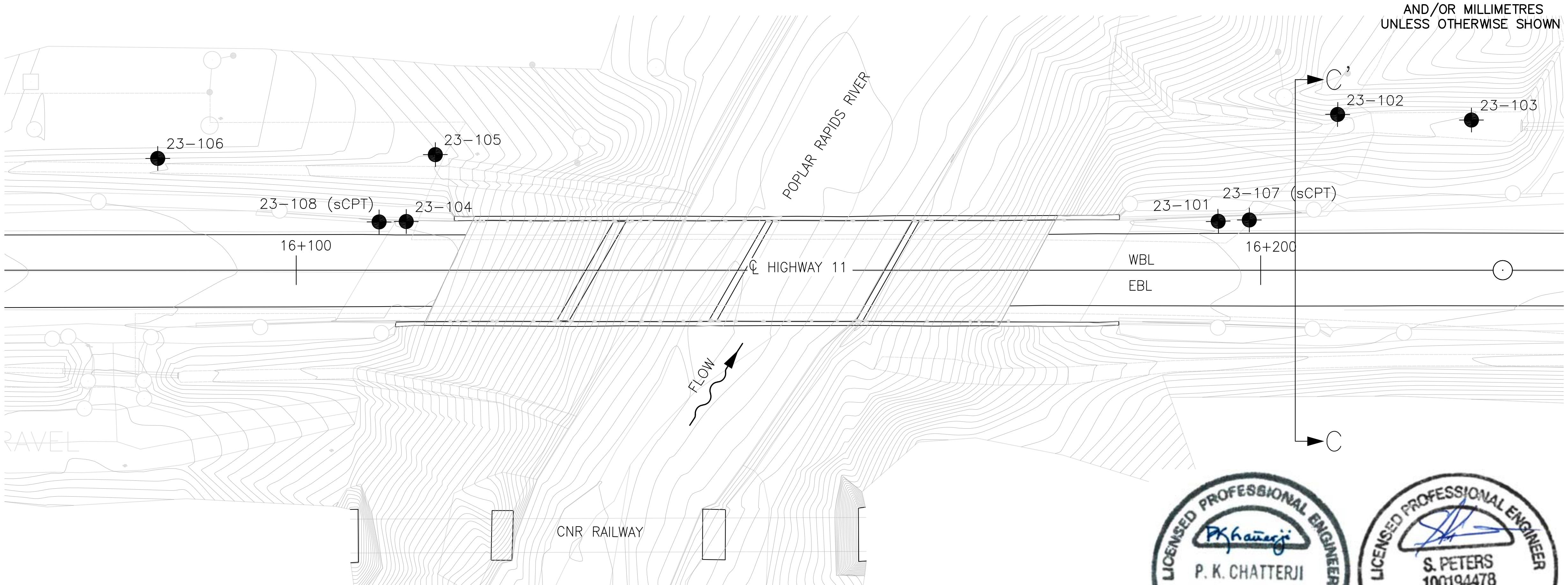
CONT No
GWP No 5021–E–0025

HIGHWAY 11
POPLAR RAPIDS BRIDGE
HAGGART TOWNSHIP
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET 3

Ontario

THURBER ENGINEERING LTD.



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10 0 10 20m
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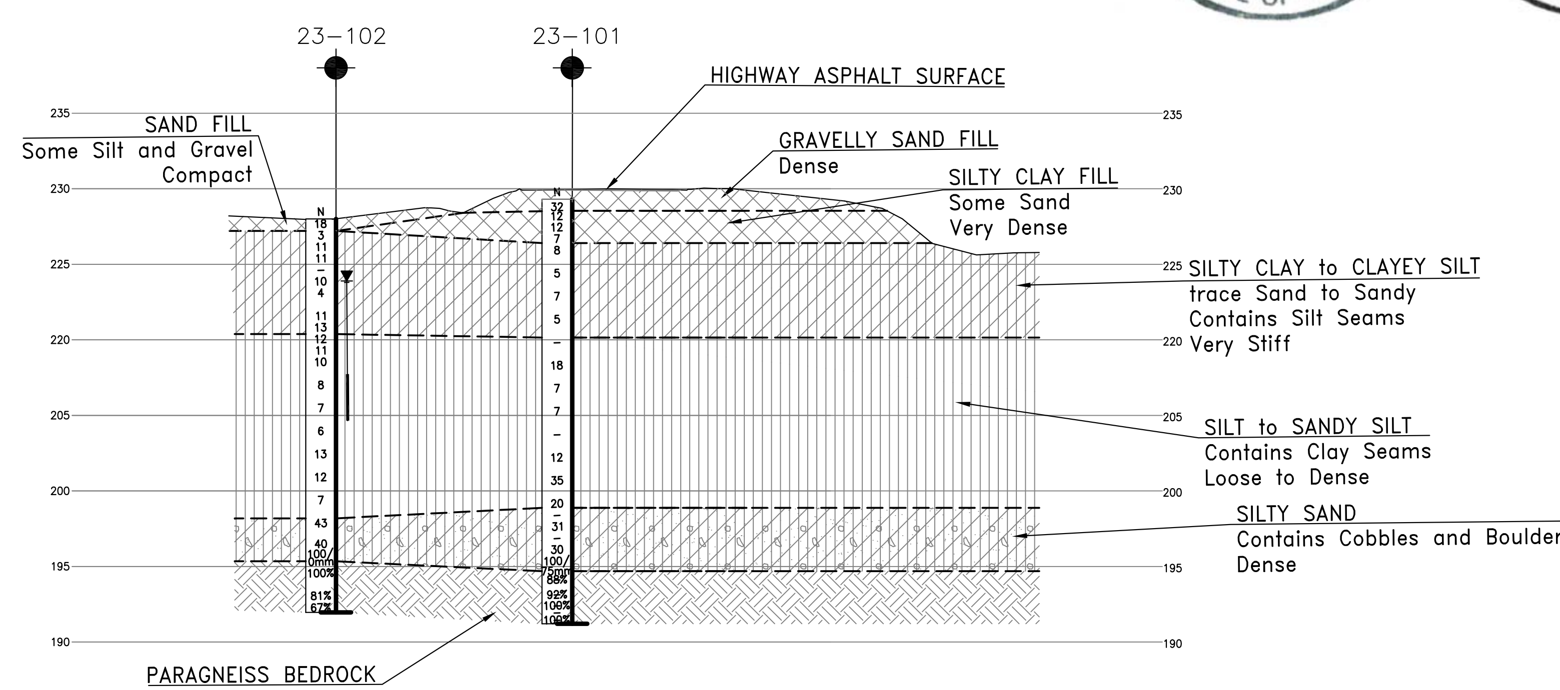

SECTION C–C'

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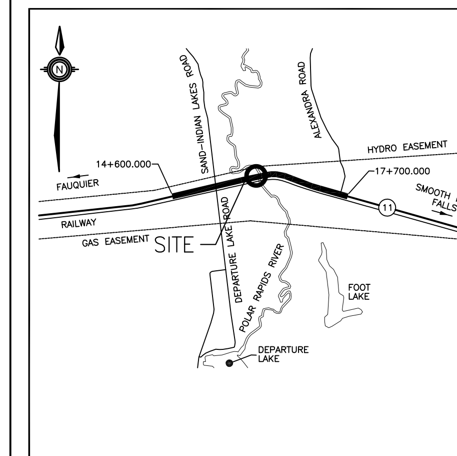
5 0 5 10m V 1:250



KEYPLAN

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
April 5, 2024
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
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100194478
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LEGEND

(symbol)	Borehole
(symbol)	Borehole (Previous Investigation)
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(symbol)	Water Level Upon Completion of Drilling
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GEOCRES No. 42H00-095

REVISIONS

DATE	BY	DESCRIPTION

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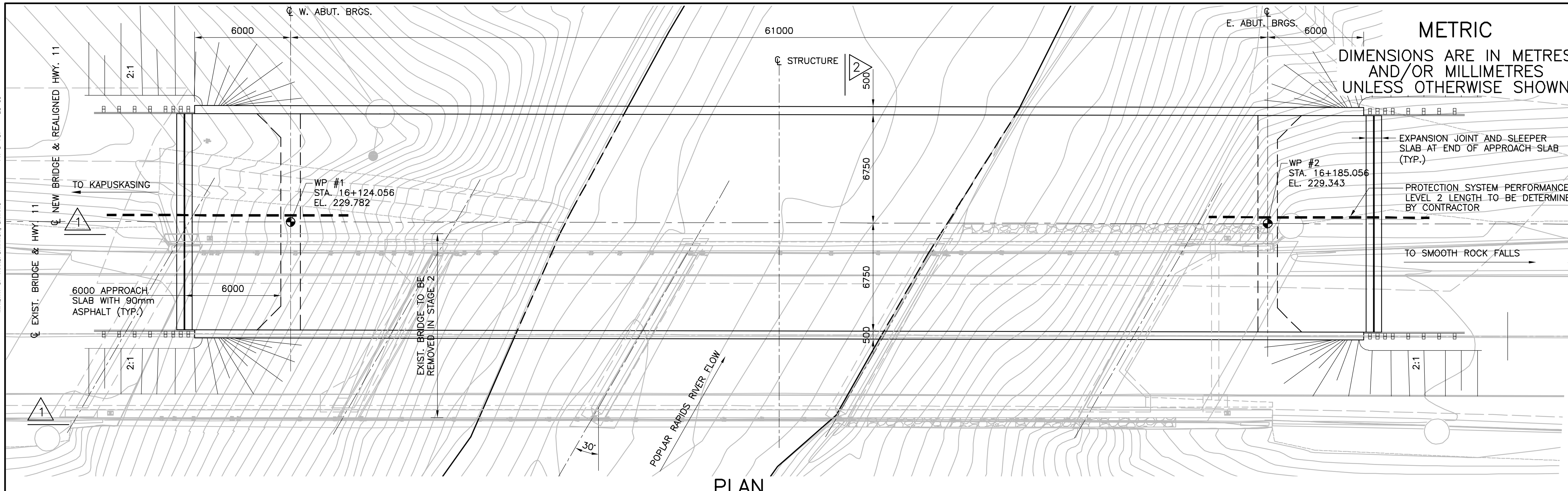
HWY. 11
CONT No
WP No 5345-19-01

POPLAR RAPIDS RIVER
BRIDGE REPLACEMENT

GENERAL ARRANGEMENT

SHEET
—

METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN



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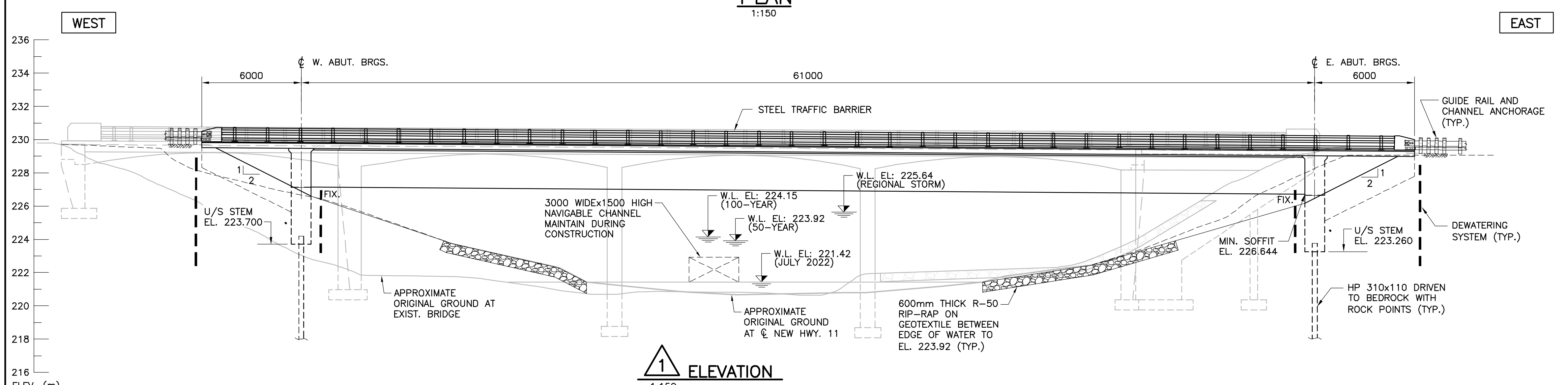

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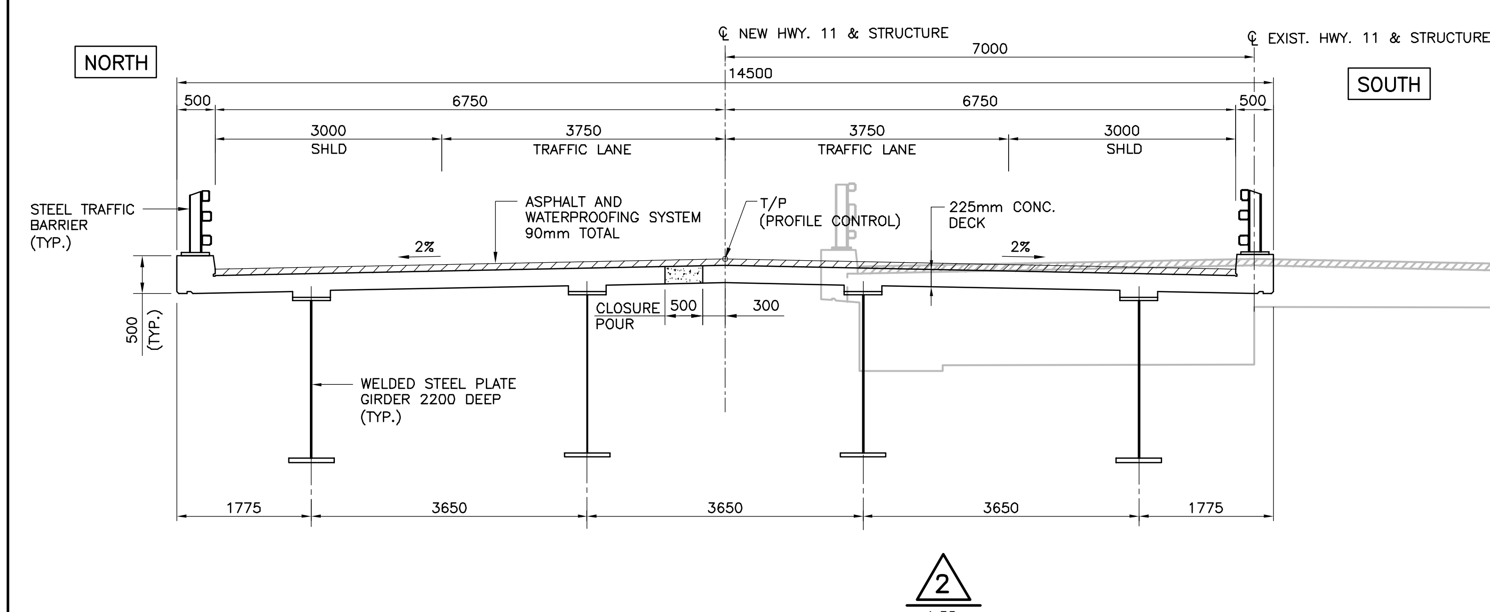
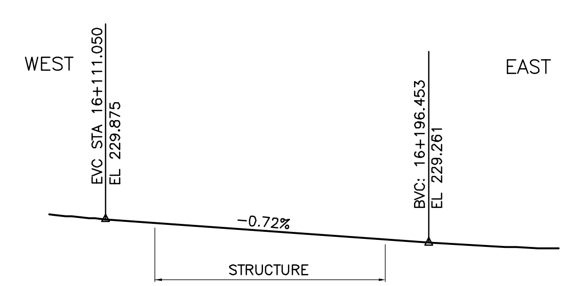


1:50

WORKING POINTS

W.P. No.	LOCATION	STATION	COORDINATES	
1	WEST ABUT.	16+124.056	N 5461548.942	E 247707.747
2	EAST ABUT.	16+185.056	N 5461554.684	E 247768.476



NEW HWY. 11 PROFILE
N.T.S.

GENERAL NOTES:

CLASS OF CONCRETE

ALL CONCRETE (UNLESS OTHERWISE NOTED) .... 30 MPa
CLOSURE POURS HIGH EARLY STRENGTH (HE) .... 60 MPa

CLEAR COVER TO REINFORCING STEEL

DECK TOP .... 70 ± 20
BOT. .... 40 ± 10
REMAINDER (UNLESS NOTED OTHERWISE) ....70 ± 20

REINFORCING STEEL

1. REINFORCING STEEL SHALL BE GRADE 500W.
2. UNLESS SHOWN OTHERWISE, TENSION LAP SPLICES FOR REINFORCING STEEL BARS SHALL BE CLASS B.
3. BAR MARKS WITH PREFIX 'S' DENOTE STAINLESS STEEL BARS. STAINLESS REINFORCING STEEL SHALL BE TYPE 316LN OR DUPLEX 2205 AND HAVE A MINIMUM YIELD STRENGTH OF 500 MPA, UNLESS OTHERWISE SPECIFIED.
4. BAR HOOKS SHALL HAVE STANDARD HOOK DIMENSIONS USING MINIMUM BEND DIAMETERS, WHILE STIRRUPS AND TIES SHALL HAVE MINIMUM HOOK DIMENSIONS. ALL HOOKS SHALL BE IN ACCORDANCE WITH THE STRUCTURAL STANDARD DRAWING SS112-1, UNLESS INDICATED OTHERWISE.

CONSTRUCTION NOTES

1. THE CONTRACTOR SHALL ESTABLISH THE BEARING SEAT ELEVATIONS BY DEDUCTING THE ACTUAL BEARING THICKNESS FROM THE TOP OF BEARING ELEVATIONS. IF THE ACTUAL BEARING THICKNESS ARE DIFFERENT FROM THOSE GIVEN WITH THE BEARING DESIGN DATA, THE CONTRACTOR SHALL ADJUST THE REINFORCING STEEL TO SUIT.
2. THE CONTRACTOR SHALL VERIFY ALL DIMENSIONS, DETAILS AND ELEVATIONS OF THE EXISTING STRUCTURE THAT ARE RELEVANT TO THE WORK SHOWN ON THE DRAWINGS PRIOR TO COMMENCEMENT OF THE ANY DISCREPANCIES SHALL BE REPORTED TO THE CONTRACT ADMINISTRATOR AND THE PROPOSED ADJUSTMENT OF THE WORK REQUIRED TO MATCH THE EXISTING STRUCTURE SHALL BE SUBMITTED FOR APPROVAL.
3. BACKFILL SHALL NOT BE PLACED BEHIND THE ABUTMENTS UNTIL THE DECK SLAB IS IN PLACE AND HAS REACHED 70% OF ITS DESIGN STRENGTH.
4. BACKFILL SHALL BE PLACED SIMULTANEOUSLY BEHIND BOTH ABUTMENTS KEEPING THE HEIGHT OF THE BACKFILL APPROXIMATELY THE SAME. AT NO TIME SHALL THE DIFFERENCE IN ELEVATION BE GREATER THAN 500mm.
5. CONSTRUCT ABUTMENTS AND WINGWALLS TO THE BEARING SEAT ELEVATIONS. THE CONTRACTOR SHALL SUPPLY TEMPORARY LATERAL BRACING FOR THE ABUTMENTS. FORMWORK AND LATERAL BRACING SHALL NOT BE REMOVED UNTIL CONCRETE HAS REACHED 70% OF ITS SPECIFIED 28-DAY STRENGTH.

LIST OF ABBREVIATIONS:

- T/A – TOP OF ASPHALT (FINISHED ELEV.)
- WP – WORKING POINT
- U/S – UNDERSIDE
- BRGS. – BEARINGS
- CONC. – CONCRETE
- TYP. – TYPICAL
- EL. – ELEVATION
- ABUT. – ABUTMENT
- W.L. – WATER LEVEL
- ℄ – CENTRE LINE
- STA. – STATION
- EXIST. – EXISTING

APPLICABLE STANDARD DRAWINGS:

- OPSD– 912.430 GUIDE RAIL SYSTEM, STEEL BEAM STRUCTURE CONNECTION
- OPSD–3101.150 WALLS ABUTMENT, BACKFILL MINIMUM GRANULAR REQUIREMENT
- OPSD–3102.100 WALLS ABUTMENT BACKFILL DRAIN
- OPSD–3370.100 DECK, WATERPROOFING HOT APPLIED ASPHALT MEMBRANE WITH PROTECTION BOARD
- OPSD–3419.100 DECK, WATERPROOFING HOT APPLIED ASPHALT MEMBRANE AT ACTIVE CRACKS GREATER THAN 2mm WIDE AND CONSTRUCTION JOINT

LIST OF DRAWINGS

1. GENERAL ARRANGEMENT
2. CONSTRUCTION STAGING

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS	DATE	BY	DESCRIPTION		
DESIGN PC	CHK NB	CODE CHBDC S6-19	LOAD ONT CL-625	DATE OCTOBER 2023	
DRAWN DT	CHK PC	SITE 39E-0001/B0	STRUCT –	SCHEME –	DWG 1

MINISTRY OF TRANSPORTATION, ONTARIO PR-D-707 88-05

DRAWING NAME: F:\23092\Structures\Drafting\23092–Poplar Bridge–S01–GA.dwg
CREATED: JUNE 2023 MODIFIED: Oct 18, 2023–1:49pm

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APPENDIX B

Symbols and Terms

Record of Boreholes Sheets

Single Well Response Test

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SYMBOLS, ABBREVIATIONS AND TERMS USED ON TEST HOLE RECORDS

TERMINOLOGY DESCRIBING COMMON SOIL GENESIS

Topsoil	mixture of soil and humus capable of supporting vegetative growth
Peat	mixture of fragments of decayed organic matter
Till	unstratified glacial deposit which may include particles ranging in sizes from clay to boulder
Fill	material below the surface identified as placed by humans (excluding buried services)

TERMINOLOGY DESCRIBING SOIL STRUCTURE:

Desiccated	having visible signs of weathering by oxidization of clay materials, shrinkage cracks, etc.
Fissured	having cracks, and hence a blocky structure
Varved	composed of alternating layers of silt and clay
Stratified	composed of alternating successions of different soil types, e.g. silt and sand
Layer	> 75 mm in thickness
Seam	2 mm to 75 mm in thickness
Parting	< 2 mm in thickness

RECOVERY:

For soil samples, the recovery is recorded as the length of the soil sample recovered.

N-VALUE:

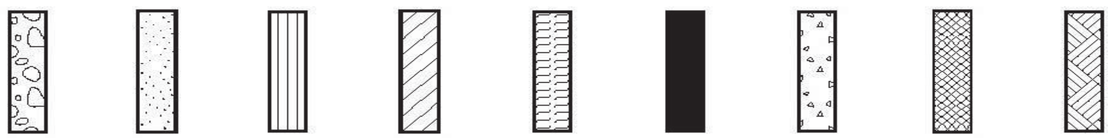
Numbers in this column are the field results of the Standard Penetration Test: the number of blows of a 63.5 kg hammer falling 0.76 m, required to drive a 50 mm O.D. split spoon sampler 0.3 m into undisturbed soil. For samples where insufficient penetration was achieved and N-value cannot be presented, the number of blows are reported over the sampler penetration in millimetres (e.g. 50/75).

DYNAMIC CONE PENETRATION TEST (DCPT):

Dynamic cone penetration tests are performed using a standard 60 degree apex cone connected to an “A” size drill rods with the same standard fall height and weight as the Standard Penetration Test. The DCPT value is the number of blows of the hammer required to drive the cone 0.3 m into the soil. The DCPT is used as a probe to assess soil variability.

THURBER

STRATA PLOT:
Strata plots symbolize the soil and bedrock description. They are combinations of the following basic symbols. The dimensions within the strata symbols are not indicative of the particle size, layer thickness, etc.



Boulders Cobbles Gravel	Sand	Silt	Clay	Organics	Asphalt	Concrete	Fill	Bedrock

TEXTURING CLASSIFICATION OF SOILS

Classification	**Particle Size**
Boulders	Greater than 200 mm
Cobbles	75 – 200 mm
Gravel	4.75 – 75 mm
Sand	0.075 – 4.75 mm
Silt	0.002 – 0.075 mm
Clay	Less than 0.002 mm

TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

Descriptive Term	**Undrained Shear Strength (kPa)**
Very Soft	12 or less
Soft	12 – 25
Firm	25 – 50
Stiff	50 – 100
Very Stiff	100 – 200
Hard	Greater than 200

NOTE: Clay sensitivity is defined as the ratio of the undisturbed strength over the remolded strength.

SAMPLE TYPES

SS	Split spoon samples
ST	Shelby tube or thin wall tube
DP	Direct push sample
PS	Piston sample
BS	Bulk sample
WS	Wash sample
HQ, NQ, BQ etc.	Rock core sample obtained with the use of standard size diamond coring equipment

TERMS DESCRIBING CONSISTENCY (COHESIONLESS SOILS ONLY)

Descriptive Term	**SPT “N” Value**
Very Loose	Less than 4
Loose	4 – 10
Compact	10 – 30
Dense	30 – 50
Very Dense	Greater than 50

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MODIFIED UNIFIED SOIL CLASSIFICATION

Major Divisions		Group Symbol	Typical Description
COARSE GRAINED SOIL	GRAVEL AND GRAVELLY SOILS	GW	Well-graded gravels or gravel-sand mixtures, little or no fines.
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines.
		GM	Silty gravels, gravel-sand-silt mixtures.
		GC	Clayey gravels, gravel-sand-clay mixtures.
	SAND AND SANDY SOILS	SW	Well-graded sands or gravelly sands, little or no fines.
		SP	Poorly-graded sands or gravelly sands, little or no fines.
		SM	Silty sands, sand-silt mixtures.
		SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS	SILT AND CLAY SOILS $W_L < 35\%$	ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
		OL	Organic silts and organic silty-clays of low plasticity.
	SILT AND CLAY SOILS $35\% < W_L < 50\%$	MI	Inorganic compressible fine sandy silt with clay of medium plasticity, clayey silts.
		CI	Inorganic clays of medium plasticity, silty clays.
		OI	Organic silty clays of medium plasticity.
	SILT AND CLAY SOILS $W_L > 50\%$	MH	Inorganic silts, micaceous or diatomaceous fine sandy of silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
		OH	Organic clays of high plasticity, organic silts.
HIGHLY ORGANIC SOILS		Pt	Peat and other organic soils.

Note - W_L= Liquid Limit

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EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION

Fresh (FR)	No visible signs of weathering.
Fresh Jointed (FJ)	Weathering limited to surface of major discontinuities.
Slightly Weathered (SW)	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock materials.
Moderately Weathered (MW)	Weathering extends throughout the rock mass, but the rock material is not friable.
Highly Weathered (HW)	Weathering extends throughout the rock mass and the rock is partly friable.
Completely Weathered (CW)	Rock is wholly decomposed and in a friable condition, but the rock texture and structures are preserved.

TERMS

Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.
Solid Core Recovery: (SCR)	Percent ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.
Rock Quality Designation: (RQD)	Total length of sound core recovered in pieces 0.1 m in length or larger, as a percentage of total core length
Unconfined Compressive Strength: (UCS)	Axial stress required to break the specimen.
Fracture Index: (FI)	Frequency of natural fractures per 0.3 m of core run.

DISCONTINUITY SPACING

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

STRENGTH CLASSIFICATION

Rock Strength	Approximate Uniaxial Compressive Strength (MPa)
Extremely Strong	Greater than 250
Very Strong	100 – 250
Strong	50 – 100
Medium Strong	25 – 50
Weak	5 – 25
Very Weak	1 – 5
Extremely Weak	0.25 – 1

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 23-101

METRIC

GWP# 5278-19-00

LOCATION Lat: 49.289327°, Long: -81.783905° Poplar Rapids Bridge, Haggart Township, MTM z12: N 5 461 553.9 E 247 779.4

ORIGINATED BY DAP

HWY 11

BOREHOLE TYPE CME 55 LC Track Mounted / HSA / NW Casing / NQ Coring

COMPILED BY RH

DATUM Geodetic

DATE 2023.07.05 - 2023.07.06

CHECKED BY AO

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
229.3	Ground Surface				
0.0	**ASPHALT (175 mm)**				
0.2	GRAVELLY SAND, dense, light brown, **FILL**	1	SS	32	21 69 10 (SI+CL)
228.5 / 0.8	SILTY CLAY, some sand, very stiff, brownish grey with dark brown mottles, **FILL**	2	SS	12	
		3	SS	12	
		4	SS	7	
226.4 / 2.9	**SILTY CLAY (CI)**, trace sand to sandy, very stiff, brownish grey with brown mottles, **WEATHERED CRUST**	5	SS	8	0 21 45 34
		6	SS	5	0 23 42 35
		7	SS	7	
221.7 / 7.6	**SILTY CLAY (CI)**, very stiff, grey	8	SS	5	0 4 33 63
220.2 / 9.1	**SILT to SANDY SILT (ML)**, contains grey clay seams, loose to dense, light grey, stratified structure	1	ST	-	0 0 82 18 Non-plastic OED: e_0 = 1.03 C_c = 0.37 C_r = 0.05

Field vane shear strength readings: > 100 kPa (at approx. elev. 225.4, 225.1, 223.8, 223.5, 222.3, 222.0, 220.8, 220.5)

Ground water level symbol at approx. elev. 223.7

Elevation scale: 229, 228, 227, 226, 225, 224, 223, 222, 221, 220

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

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

DOUBLE LINE 33443 - 100 BHS- POPLAR RIVER BRIDGE.GPJ 2012TEMPLATE(MTO).GDT 4-4-24

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 23-101

METRIC

GWP# 5278-19-00

LOCATION Lat: 49.289327°, Long: -81.783905° Poplar Rapids Bridge, Haggart Township, MTM z12: N 5 461 553.9 E 247 779.4

ORIGINATED BY DAP

HWY 11

BOREHOLE TYPE CME 55 LC Track Mounted / HSA / NW Casing / NQ Coring

COMPILED BY RH

DATUM Geodetic

DATE 2023.07.05 - 2023.07.06

CHECKED BY AO

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH / WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILT to SANDY SILT (ML) contains grey clay seams loose to dense light grey stratified structure				219	Field vane > 100 kPa; > 100 kPa	
		9	SS	18	218		
		10	SS	7	217		0 1 84 15 Non-plastic
		11	SS	7	216 / 215	Field vane > 100 kPa; > 100 kPa	
		12	GS	-	214		0 37 60 3 Non-plastic
		13	SS	12	213 / 212		
		14	SS	35	211 / 210		

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; STRAT PLOT; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE, 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m^3

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

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

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 23-101

METRIC

GWP# 5278-19-00
LOCATION Lat: 49.289327°, Long: -81.783905° Poplar Rapids Bridge, Haggart Township, MTM z12: N 5 461 553.9 E 247 779.4
ORIGINATED BY DAP

HWY 11
BOREHOLE TYPE CME 55 LC Track Mounted / HSA / NW Casing / NQ Coring
COMPILED BY RH

DATUM Geodetic
DATE 2023.07.05 - 2023.07.06
CHECKED BY AO

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT kN/m3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILT to SANDY SILT (ML) contains grey clay seams loose to dense, light grey stratified structure	15	SS	20	209	~21		
208.9 / 20.4	**SILTY SAND (SM)** contains cobbles and boulders light grey dense - 205 mm boulder at a depth of 20.7 m	1	NQ	-	208			
		16	SS	31		~12		0 70 27 3
	- 125 mm cobble at a depth of 22.3 m	2	NQ	-	207			
		17	SS	30	206	~14		
204.7		18	SS	100/ 75mm	205	~9	FI	
24.6	**PARAGNEISS BEDROCK** contains quartz inclusions slightly weathered to fresh jointed whitish grey medium grained very strong	1	RUN	-	204		2, 2, 2, 8	RUN #1 TCR=100% SCR=80% RQD=88% UCS=204MPa
		2	RUN	-	203		1, 2	RUN #2 TCR=100% SCR=86% RQD=92%
		3	RUN	-			3, 0	RUN #3 TCR=100% SCR=100% RQD=100% UCS=155MPa
		4	RUN	-	202		0, 0	RUN #4 TCR=100% SCR=100% RQD=100%
201.2		5	RUN	-			1, 1	RUN #5 TCR=100% SCR=100% RQD=100%
28.1	**End of Borehole** Unstabilized water level in cased borehole at a depth of 5.6 m (elev. 223.7 m) upon completion of drilling.							

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

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

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

DOUBLE LINE 33443 - 100 BHS- POPLAR RIVER BRIDGE.GPJ 2012TEMPLATE(MTO).GDT 4-4-24

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 23-102

METRIC

GWP#	5278-19-00	LOCATION	Lat: 49.289435°, Long: -81.783751° Poplar Rapids Bridge, Haggart Township, MTM z12: N 5 461 566.1 E 247 791.1	ORIGINATED BY	DAP
HWY	11	BOREHOLE TYPE	CME 55 LC Track Mounted / NW Casing / NQ Coring	COMPILED BY	RH
DATUM	Geodetic	DATE	2023.07.12 - 2023.07.12	CHECKED BY	AO

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
227.0	Ground Surface
0.0	SAND, some silt and gravel, contains organics, hydrogen sulfide odour, compact, dark grey **FILL**
226.2 / 0.8	**SILTY CLAY (CI)** to **CLAYEY SILT (CL)**, trace to some sand, contains light grey silt seams, very stiff, greyish brown to brownish grey **WEATHERED CRUST**
222.4 / 4.6	**SILTY CLAY (CI)**, contains light grey silt seams, very stiff, grey
219.4 / 7.6	**SILT to SANDY SILT (ML)**, contains grey clay seams, loose to compact, light grey, stratified structure

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	18	13 71 12 4
2	SS	3	
3	SS	11	0 9 45 46
4	SS	11	
1	ST	-	0 13 40 47 OED: e_0 = 0.64, C_c = 0.14, C_r = 0.03
5	SS	10	
6	SS	4	0 0 31 69
7	SS	11	
8	SS	13	
9	SS	12	
10	SS	11	0 1 84 15 Non-plastic
11	SS	10	

Field vane: > 100 kPa; > 100 kPa

ELEVATION SCALE: 226, 225, 224, 223, 222, 221, 220, 219, 218

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

Continued Next Page

+³, ×³: Numbers refer to Sensitivity; 20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 23-102

METRIC

GWP#	5278-19-00	LOCATION	Lat: 49.289435°, Long: -81.783751° Poplar Rapids Bridge, Haggart Township, MTM z12: N 5 461 566.1 E 247 791.1	ORIGINATED BY	DAP
HWY	11	BOREHOLE TYPE	CME 55 LC Track Mounted / NW Casing / NQ Coring	COMPILED BY	RH
DATUM	Geodetic	DATE	2023.07.12 - 2023.07.12	CHECKED BY	AO

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL	REMARKS
	SILT to SANDY SILT (ML) contains grey clay seams loose to compact light grey stratified structure										
		12	SS	8	216	~22	0	2	93	5	Non-plastic
					215						
		13	SS	7		~24	0	20	72	8	Non-plastic
					214						
		14	SS	6	213	~20					
					212						
		15	SS	13		~20					
					211						
		16	SS	12	210	~25					
					209						
		17	SS	7		~24					
					208						
207.2 / 19.8	**SILTY SAND (SM)**										

Continued Next Page

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

+³, ×³: Numbers refer to Sensitivity

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

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 23-102

METRIC

GWP#	5278-19-00	LOCATION	Lat: 49.289435°, Long: -81.783751° Poplar Rapids Bridge, Haggart Township, MTM z12: N 5 461 566.1 E 247 791.1	ORIGINATED BY	DAP
HWY	11	BOREHOLE TYPE	CME 55 LC Track Mounted / NW Casing / NQ Coring	COMPILED BY	RH
DATUM	Geodetic	DATE	2023.07.12 - 2023.07.12	CHECKED BY	AO

Continued From Previous Page

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT	UNIT WEIGHT / FI	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILTY SAND (SM) contains cobbles and boulders, dense, light grey	18	SS	43		o		0 78 19 3
					206			
		19	SS	40		o		
					205			
		1	NQ	-				
204.3 / 22.7	- 300 mm boulder at a depth of 22.2 m	20	SS	100/ 0mm		o	FI	
	PARAGNEISS BEDROCK contains quartz inclusions, slightly weathered to fresh jointed, whitish grey, medium grained, strong to very strong						0	
					204		0	
	- pink quartz vein from a depth of 23.6 to 23.8 m	1	RUN	-			1	RUN #1 TCR=100% SCR=96% RQD=100% UCS=209MPa
	- whitish pink quartz vein from a depth of 24.0 to 25.0 m						0	
					203		1	
							4	
							1	
		2	RUN	-	202		2	RUN #2 TCR=100% SCR=74% RQD=81% UCS=92MPa
							2	
							3	
		3	RUN	-			1	RUN #3 TCR=100% SCR=80% RQD=67%
201.0 / 26.0					201		2	

End of Borehole

Monitoring well installed:
Scredule 40 PVC standpipe with 32-mm diameter and 3.0-m slotted screen.

Water Level Readings:

DATE	DEPTH (m)	ELEV. (m)
2023/07/13	3.7	223.3
2023/07/14	4.2	222.8
2023/07/15	4.8	222.2
2023/09/05	4.1	222.9
2023/09/10	4.1	222.9

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m^3; STRAT PLOT

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

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

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 23-103

METRIC

GWP# 5278-19-00

LOCATION Lat: 49.289444°, Long: -81.783561° Poplar Rapids Bridge, Haggart Township, MTM z12: N 5 461 567.1 E 247 804.1

ORIGINATED BY DAP

HWY 11

BOREHOLE TYPE CME 55 LC Track Mounted / NW Casing / NQ Coring

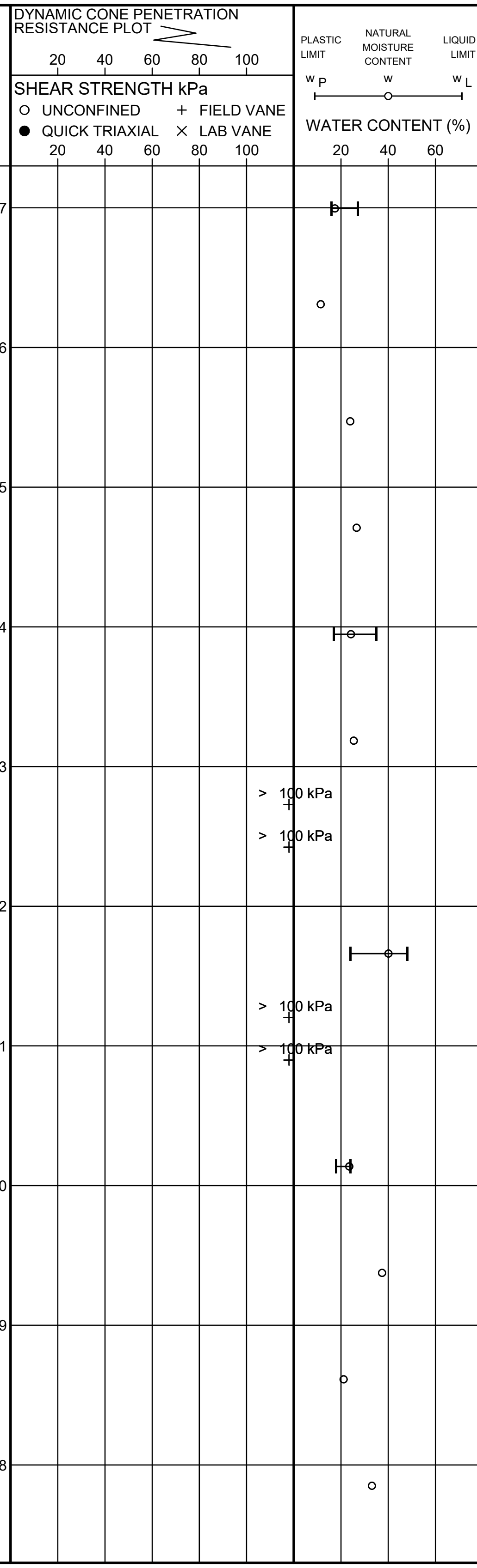
COMPILED BY RH

DATUM Geodetic

DATE 2023.07.13 - 2023.07.13

CHECKED BY AO

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
227.3	Ground Surface								
0.0	SILTY SAND, some clay contains organics hydrogen sulfide odour very loose grey **FILL**	1	SS	2	227	0	61	21	18
226.5 0.8	**SILTY CLAY (CI)**, some sand contains sand partings very stiff brownish grey **WEATHERED CRUST**	2	SS	8	226				
		3	SS	7	225				
		4	SS	5					
		5	SS	3	224	0	10	45	45
		6	SS	3	223				
	Field vane: > 100 kPa; > 100 kPa								
222.0 5.3	**SILTY CLAY (CI)** to **CLAYEY SILT (CL-ML)** contains light grey silt seams very stiff grey	7	SS	4	222	0	0	31	69
	Field vane: > 100 kPa; > 100 kPa				221				
		8	SS	11	220	0	0	74	26
219.8 7.5	**SILT (ML)** contains grey clay seams very loose to compact light grey stratified structure	9	SS	13	219				
		10	SS	14					
		11	SS	9	218				



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+ 3, × 3 : Numbers refer to Sensitivity

15 20 5 10 (%) STRAIN AT FAILURE

DOUBLE LINE 33443 - 100 BHS- POPLAR RIVER BRIDGE.GPJ 2012TEMPLATE(MTO).GDT 4-4-24

RECORD OF BOREHOLE No 23-103

METRIC

GWP# 5278-19-00

LOCATION Lat: 49.289444°, Long: -81.783561° Poplar Rapids Bridge, Haggart Township, MTM z12: N 5 461 567.1 E 247 804.1

ORIGINATED BY DAP

HWY 11

BOREHOLE TYPE CME 55 LC Track Mounted / NW Casing / NQ Coring

COMPILED BY RH

DATUM Geodetic

DATE 2023.07.13 - 2023.07.13

CHECKED BY AO

ELEV DEPTH	DESCRIPTION (Continued From Previous Page)	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILT (ML) contains grey clay seams, very loose to compact, light grey, stratified structure				217		
		12	SS	11		~20	0 2 84 14 Non-plastic
					216		
					215		
		13	SS	12		~31	
					214		
		14	SS	13		~22	
					213		
					212		
211.5		15	SS	2		~22	0 4 90 6 Non-plastic
15.8	**End of Borehole** A representative open-hole groundwater level measurement was not obtained due to the introduction of water during drilling.						

Column headings also printed: STRAT PLOT; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m^3

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

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

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 23-104

METRIC

GWP# 5278-19-00 | LOCATION Lat: 49.289246°, Long: -81.785056° Poplar Rapids Bridge, Haggart Township, MTM z12: N 5 461 545.7 E 247 695.6 | ORIGINATED BY DAP

HWY 11 | BOREHOLE TYPE CME 55 LC Track Mounted / NW Casing / NQ Coring | COMPILED BY RH

DATUM Geodetic | DATE 2023.07.07 - 2023.07.07 | CHECKED BY AO

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH / WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
229.8	Ground Surface											
0.0	**ASPHALT (75 mm)**											
229.5 0.1	**CONCRETE (230 mm)**											
0.3	SAND, some gravel, compact, light brown **FILL**	1	SS	23		229			10	80	10 (SI+CL)	
228.7		2	SS	9								
1.1	SILTY CLAY, some sand, contains organics, very stiff, greyish brown with yellow mottles **FILL**	3	SS	14		228			0	10	44	46
		4	SS	14		227						
226.8 3.0	**CLAYEY SILT (CL)**, trace sand, very stiff, greyish brown with yellow mottles **WEATHERED CRUST**	5	SS	10		226						
	- blackish grey sand lens at a depth of 4.1 m	6	SS	13		225						
		7	SS	12								
		8	SS	21	▽	224			0	12	49	39
		9	SS	17		223						
222.9 6.9	**SILTY CLAY (CI)**, contains light grey silt seams, very stiff, grey	10	SS	7		222						
		11	SS	6					0	0	44	56
						221	> 100 kPa (field vane); > 100 kPa (field vane)		OED: e_0 = 1.10, C_c = 0.34, C_r = 0.05			
220.7 9.1	**SILT to SANDY SILT (ML)**, contains grey clay seams, loose to compact, light grey, stratified structure	1	ST	-		220			0	0	22	78
									0	0	87	13
									Non-plastic			

Continued Next Page

+ ³, × ³: Numbers refer to Sensitivity

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

Dynamic cone penetration resistance plot: 20 40 60 80 100. Shear strength kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100. Plastic limit w_P, natural moisture content w, liquid limit w_L; water content (%) 20 40 60.

DOUBLE LINE 33443 - 100 BHS- POPLAR RIVER BRIDGE.GPJ 2012TEMPLATE(MTO).GDT 4-4-24

RECORD OF BOREHOLE No 23-104

METRIC

GWP# 5278-19-00
LOCATION Lat: 49.289246°, Long: -81.785056°
Poplar Rapids Bridge, Haggart Township, MTM z12: N 5 461 545.7 E 247 695.6
ORIGINATED BY DAP
HWY 11
BOREHOLE TYPE CME 55 LC Track Mounted / NW Casing / NQ Coring
COMPILED BY RH
DATUM Geodetic
DATE 2023.07.07 - 2023.07.07
CHECKED BY AO

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

Continued From Previous Page

SILT to SANDY SILT (ML)
contains grey clay seams
loose to compact
light grey
stratified structure

Field vane: > 100 kPa
Field vane: > 100 kPa

NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL	REMARKS
12	SS	12	0	0	85	15	Non-plastic
13	SS	14	0	3	92	5	Non-plastic
14	SS	4					
15	SS	11					
16	SS	11					
17	SS	9					

Elevation scale: 219, 218, 217, 216, 215, 214, 213, 212, 211, 210

ELEV 209.8

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity
20 / 15 / 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 23-104

METRIC

GWP# 5278-19-00

LOCATION Lat: 49.289246°, Long: -81.785056° Poplar Rapids Bridge, Haggart Township, MTM z12: N 5 461 545.7 E 247 695.6

ORIGINATED BY DAP

HWY 11

BOREHOLE TYPE CME 55 LC Track Mounted / NW Casing / NQ Coring

COMPILED BY RH

DATUM Geodetic

DATE 2023.07.07 - 2023.07.07

CHECKED BY AO

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
20.0	**SILTY SAND (SM)** dense light grey	18	SS	41	
		19	SS	34	0 60 36 4
		20	SS	45	
		21	SS	38	
203.9 25.9	**SAND and SILT** compact to dense light grey	22	SS	10	0 40 55 5 Non-plastic
		23	SS	21	
		24	SS	36	

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³

Elevation scale: 209, 208, 207, 206, 205, 204, 203, 202, 201, 200

Continued Next Page

+ ³, × ³: Numbers refer to Sensitivity

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 23-104

METRIC

GWP# 5278-19-00
LOCATION Lat: 49.289246°, Long: -81.785056° Poplar Rapids Bridge, Haggart Township, MTM z12: N 5 461 545.7 E 247 695.6
ORIGINATED BY DAP

HWY 11
BOREHOLE TYPE CME 55 LC Track Mounted / NW Casing / NQ Coring
COMPILED BY RH

DATUM Geodetic
DATE 2023.07.07 - 2023.07.07
CHECKED BY AO

Continued From Previous Page

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE (20 40 60 80 100)	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SAND and SILT compact to dense light grey		25	SS	100/ 75mm				○		
199.0 / 30.8	**End of Borehole**										
	Unstabilized water level in cased borehole at a depth of 6.0 m (elev. 223.8 m) upon completion of drilling.										

+ ^{3}, × ^{3}: Numbers refer to Sensitivity

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

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 23-105

METRIC

GWP# 5278-19-00

LOCATION Lat: 49.289311°, Long: -81.785027° Poplar Rapids Bridge, Haggart Township, MTM z12: N 5 461 553.1 E 247 698.1

ORIGINATED BY DAP

HWY 11

BOREHOLE TYPE CME 55 LC Track Mounted / NW Casing / NQ Coring

COMPILED BY RH

DATUM Geodetic

DATE 2023.07.10 - 2023.07.10

CHECKED BY AO

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
228.5	Ground Surface					
0.0 / 0.1	SILTY SAND, some gravel and clay, contains organics, very loose, brownish grey **FILL**	1	SS	8	228	
	SILTY CLAY (CI), some sand to **CLAY (CH)**, very stiff, brownish grey with brown mottles **WEATHERED CRUST**	2	SS	9		
		3	SS	14	227	0 10 42 48
		4	SS	13	226	
		5	SS	14	225	
		6	SS	15	224	
		7	SS	8		
223.2 / 5.3	**CLAY (CH)**, contains light grey silt seams, very stiff, grey	1	ST	-	223	0 0 28 72; OED: e_0 = 1.15, C_c = 0.47, C_r = 0.04
		8	SS	4	222	0 0 28 72
					221	Field vane: > 100 kPa; > 100 kPa
220.9 / 7.6	**SILT to SANDY SILT (ML)**, contains grey clay seams, loose to compact, light grey, stratified structure	9	SS	14	220	
		10	SS	13		0 0 78 22
		11	SS	16	219	

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

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

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 23-105

METRIC

GWP#	5278-19-00	LOCATION	Lat: 49.289311°, Long: -81.785027° Poplar Rapids Bridge, Haggart Township, MTM z12: N 5 461 553.1 E 247 698.1	ORIGINATED BY	DAP
HWY	11	BOREHOLE TYPE	CME 55 LC Track Mounted / NW Casing / NQ Coring	COMPILED BY	RH
DATUM	Geodetic	DATE	2023.07.10 - 2023.07.10	CHECKED BY	AO

Continued From Previous Page

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILT to SANDY SILT (ML) contains grey clay seams, loose to compact, light grey, stratified structure				218		
		12	SS	13			
					217		
		13	SS	6	216		
					215		
		14	SS	15			
					214		
		15	SS	13	213		0 6 91 3 Non-plastic
					212		
211.1		16	SS	8			
17.4	**SILTY SAND (SM)**, some gravel, contains cobbles and boulders, dense to very dense, light grey - 370 mm boulder at a depth of 17.8 m				211		
		1	NQ	-			
		17	SS	44	210		10 66 23 1
					209		

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m^3

Continued Next Page

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

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

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 23-105

METRIC

GWP# 5278-19-00
LOCATION Lat: 49.289311°, Long: -81.785027° Poplar Rapids Bridge, Haggart Township, MTM z12: N 5 461 553.1 E 247 698.1
ORIGINATED BY DAP
HWY 11
BOREHOLE TYPE CME 55 LC Track Mounted / NW Casing / NQ Coring
COMPILED BY RH
DATUM Geodetic
DATE 2023.07.10 - 2023.07.10
CHECKED BY AO

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILTY SAND (SM), some gravel contains cobbles and boulders dense to very dense light grey	18	SS	77	208	
		19	SS	45	207	
					206	
205.6 22.9	**SAND and SILT** very loose to very dense light grey	20	SS	34	205	
		21	SS	34	204	
					203	
		22	SS	61	202	
		23	SS	44	201	0 41 56 3 Non-plastic
					200	
		24	SS	1	199	

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

Continued Next Page

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

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

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 23-105

METRIC

GWP# 5278-19-00

LOCATION Lat: 49.289311°, Long: -81.785027° Poplar Rapids Bridge, Haggart Township, MTM z12: N 5 461 553.1 E 247 698.1

ORIGINATED BY DAP

HWY 11

BOREHOLE TYPE CME 55 LC Track Mounted / NW Casing / NQ Coring

COMPILED BY RH

DATUM Geodetic

DATE 2023.07.10 - 2023.07.10

CHECKED BY AO

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT kN/m3	GR	SA	SI	CL
	Continued From Previous Page													
	SAND and SILT very loose to very dense light grey						198							
			25	SS	4						2	37	55	6
											Non-plastic			
197.4 / 31.1	End of sampled borehole Borehole advanced with DCPT						197							
196.8 / 31.7	**End of Borehole on DCPT refusal**													

Monitoring well installed:
Scredule 40 PVC standpipe with 32-mm diameter and 3.0-m slotted screen.

Water Level Readings:

DATE	DEPTH (m)	ELEV. (m)
2023/07/11	6.4	222.1
2023/07/13	6.4	222.1
2023/07/15	6.4	222.1
2023/09/05	5.7	222.8
2023/09/10	5.5	223.0

DOUBLE LINE 33443 - 100 BHS- POPLAR RIVER BRIDGE.GPJ 2012TEMPLATE(MTO).GDT 4-4-24

+ 3, × 3: Numbers refer to Sensitivity

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

RECORD OF BOREHOLE No 23-106

METRIC

GWP# 5278-19-00 | LOCATION Lat: 49.289281°, Long: -81.785418° Poplar Rapids Bridge, Haggart Township, MTM z12: N 5 461 550.1 E 247 669.1 | ORIGINATED BY DAP

HWY 11 | BOREHOLE TYPE CME 55 LC Track Mounted / NW Casing / NQ Coring | COMPILED BY RH

DATUM Geodetic | DATE 2023.07.11 - 2023.07.11 | CHECKED BY AO

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%) / SHEAR STRENGTH	GR	SA	SI	CL
229.1	Ground Surface									
0.0 / 0.1	SILTY SAND, some clay, contains organics, hydrogen sulfide odour, very loose, grey **FILL**	1	SS	2	229					
	SILTY CLAY (CI), trace sand, very stiff, brownish grey with brown mottles **WEATHERED CRUST**	2	SS	18	228		0	9	44	47
		3	SS	15						
		4	SS	13	227					
226.1 / 3.0	**SILTY CLAY (CI)**, contains light grey silt seams, very stiff, grey	5	SS	6	226		1	11	42	46
		1	ST	-	225					
		6	SS	13						
		7	SS	2	224					
					223	> 100 kPa				
						> 100 kPa				
		8	SS	4	222					
						> 100 kPa				
					221	> 100 kPa				
220.0 / 9.1	**SILT (ML)**, trace to some sand, contains grey clay seams, compact, light grey, stratified structure	9	SS	16	220					

Continued Next Page

Dynamic Cone Penetration Resistance Plot: 20 40 60 80 100. Shear Strength kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100. Plastic Limit w_P, Natural Moisture Content w, Liquid Limit w_L; Water Content (%) 20 40 60. Unit Weight γ kN/m³. Remarks & Grain Size Distribution (%). Ground Water Conditions. Strat Plot.

+ ³, × ³: Numbers refer to Sensitivity; 20 / 15 / 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 23-106

METRIC

GWP#	5278-19-00	LOCATION	Lat: 49.289281°, Long: -81.785418° Poplar Rapids Bridge, Haggart Township, MTM z12: N 5 461 550.1 E 247 669.1	ORIGINATED BY	DAP
HWY	11	BOREHOLE TYPE	CME 55 LC Track Mounted / NW Casing / NQ Coring	COMPILED BY	RH
DATUM	Geodetic	DATE	2023.07.11 - 2023.07.11	CHECKED BY	AO

Continued From Previous Page

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILT (ML) trace to some sand, contains grey clay seams, compact, light grey, stratified structure				219	
		10	SS	12	218	0 0 75 25
		11	SS	10	217	
					216	
214.8		12	SS	15	215	0 11 84 5 Non-plastic
14.3	**End of Borehole**					

A representative open-hole groundwater level measurement was not obtained due to the introduction of water during drilling.

+ 3, × 3: Numbers refer to Sensitivity

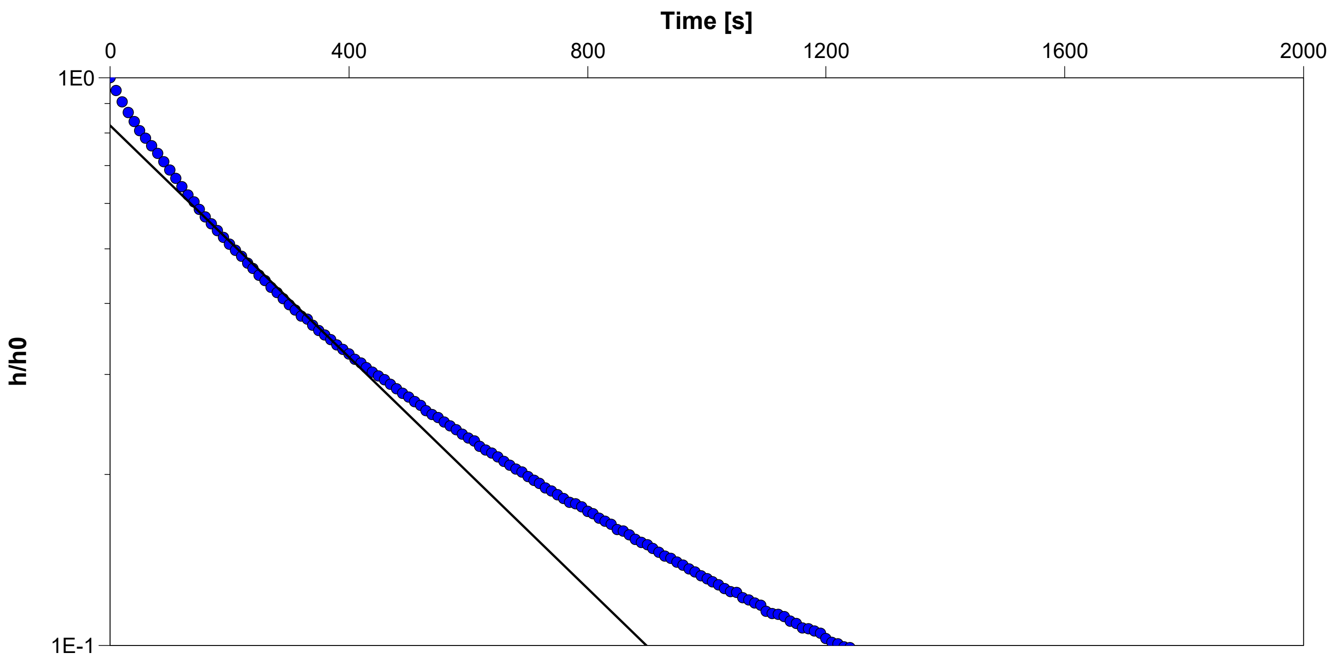
20 / 15 5 / 10 (%) STRAIN AT FAILURE

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

THURBER ENGINEERING LTD.

Slug Test Analysis Report	
Project:	Hwy 11 Poplar Rapids Bridge
Number:	33443
Client:	LEA

Location: Haggart Township, Ontario	Slug Test: 23-102	Test Well: 23-102
Test Conducted by: IK		Test Date: 2023-09-06
Analysis Performed by: SM	SWRT Analysis	Analysis Date: 2023-09-27
Aquifer Thickness:		
Reviewed by: AH		

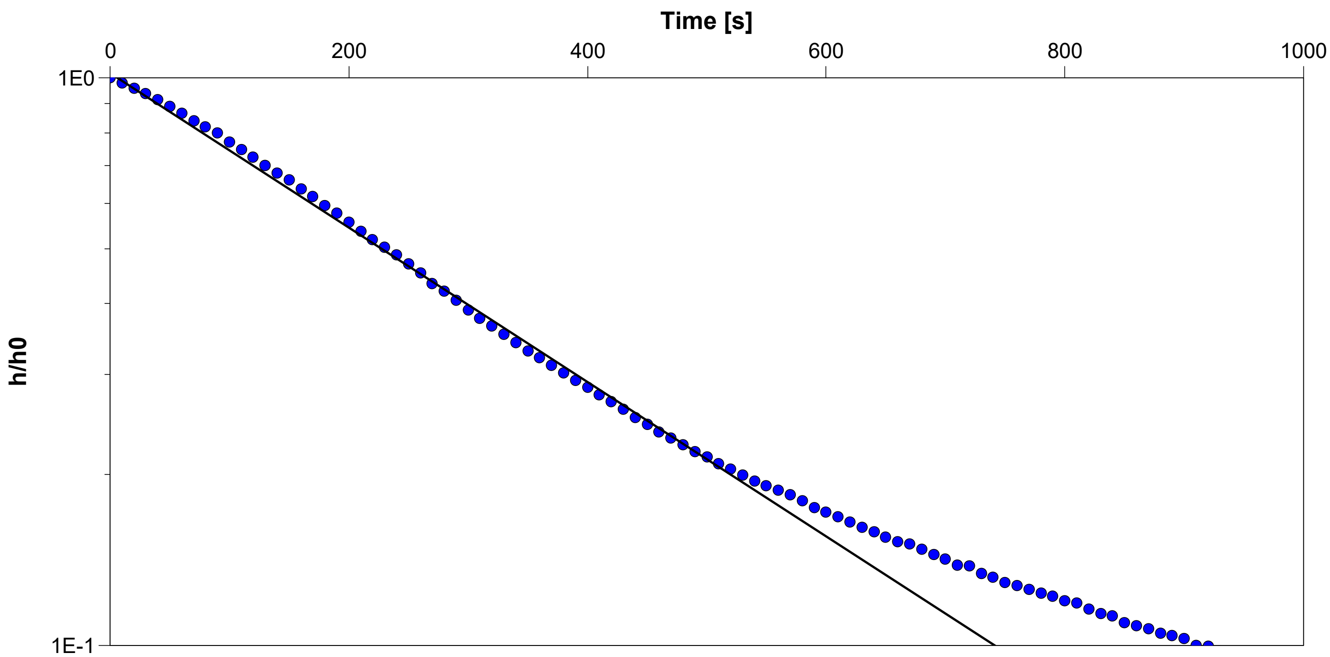


Calculation using Hvorslev

Observation Well	Hydraulic Conductivity [m/s]	
23-102	5.2×10^{-7}	

THURBER ENGINEERING LTD.	**Slug Test Analysis Report**
	Project: Hwy 11 Poplar Rapids Bridge
	Number: 33443
	Client: LEA

Location: Haggart Township, Ontario	Slug Test: 23-105	Test Well: 23-105
Test Conducted by: IK		Test Date: 2023-09-06
Analysis Performed by: SM	SWRT Analysis	Analysis Date: 2023-09-27
Aquifer Thickness:		
Reviewed by: AH		



Calculation using Hvorslev

Observation Well	Hydraulic Conductivity [m/s]	
23-105	7.0×10^{-7}	

THURBER ENGINEERING LTD.

APPENDIX C

Particle Size Analysis Figures

Atterberg Limits Figures

Consolidation Testing Results

Unconfined Compressive Strength Testing Results

Bedrock Core Photographs

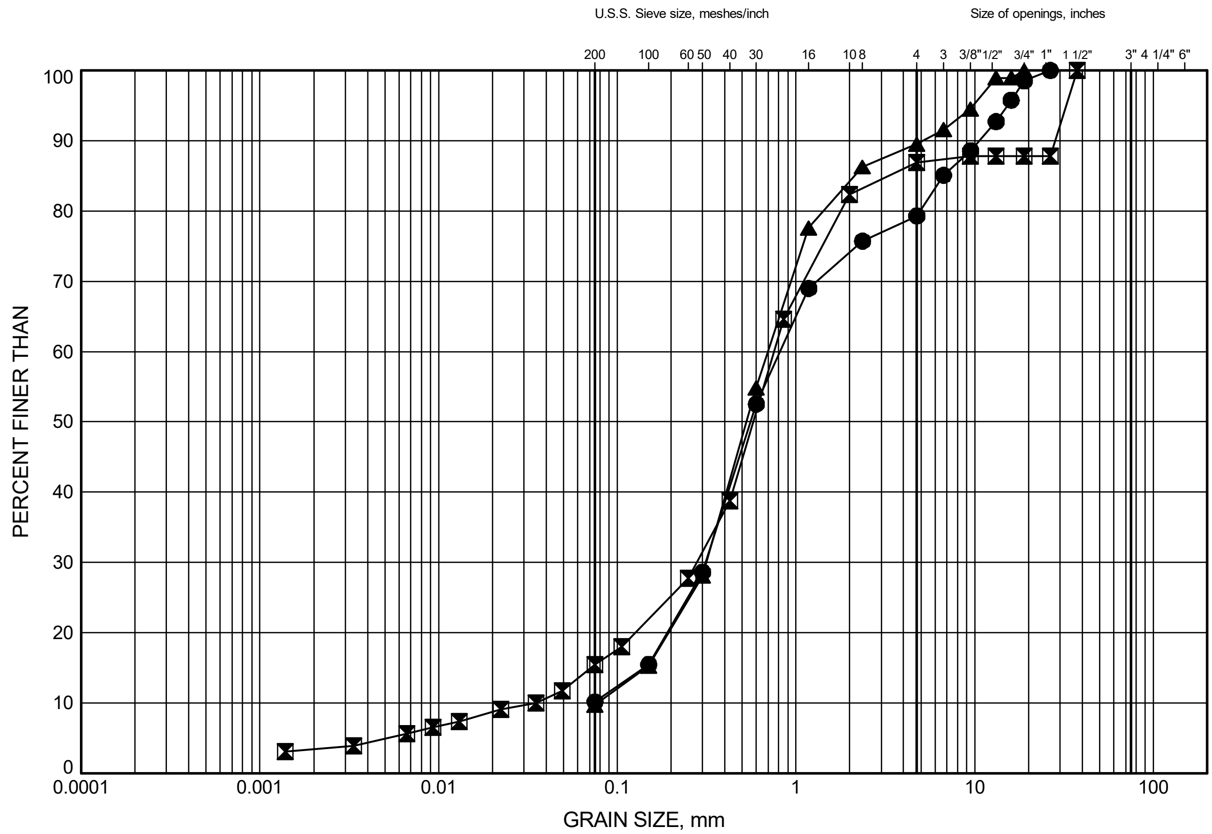
Analytical Testing Results

Highway 11 - Poplar Rapids Bridge

GRAIN SIZE DISTRIBUTION

FIGURE C1

FILL: Sand to Gravelly Sand



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	23-101	0.6	228.7
⧗	23-102	0.3	226.7
▲	23-104	0.5	229.3

Date September 2023

GWP# 5278-19-00

THURBER

Prep'd RH

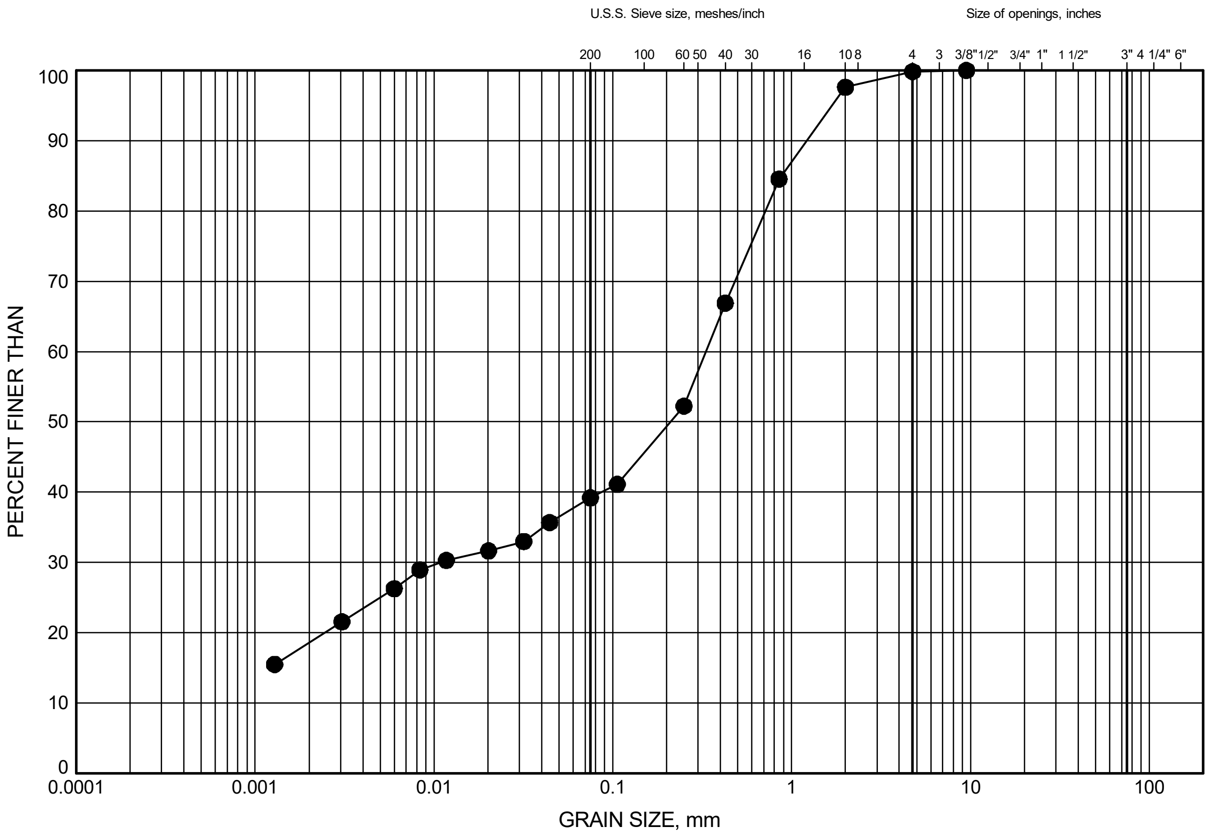
Chkd. AO

Highway 11 - Poplar Rapids Bridge

GRAIN SIZE DISTRIBUTION

FIGURE C2

FILL: Silty Sand



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	23-103	0.3	227.0

Date September 2023

GWP# 5278-19-00

THURBER

Prep'd RH

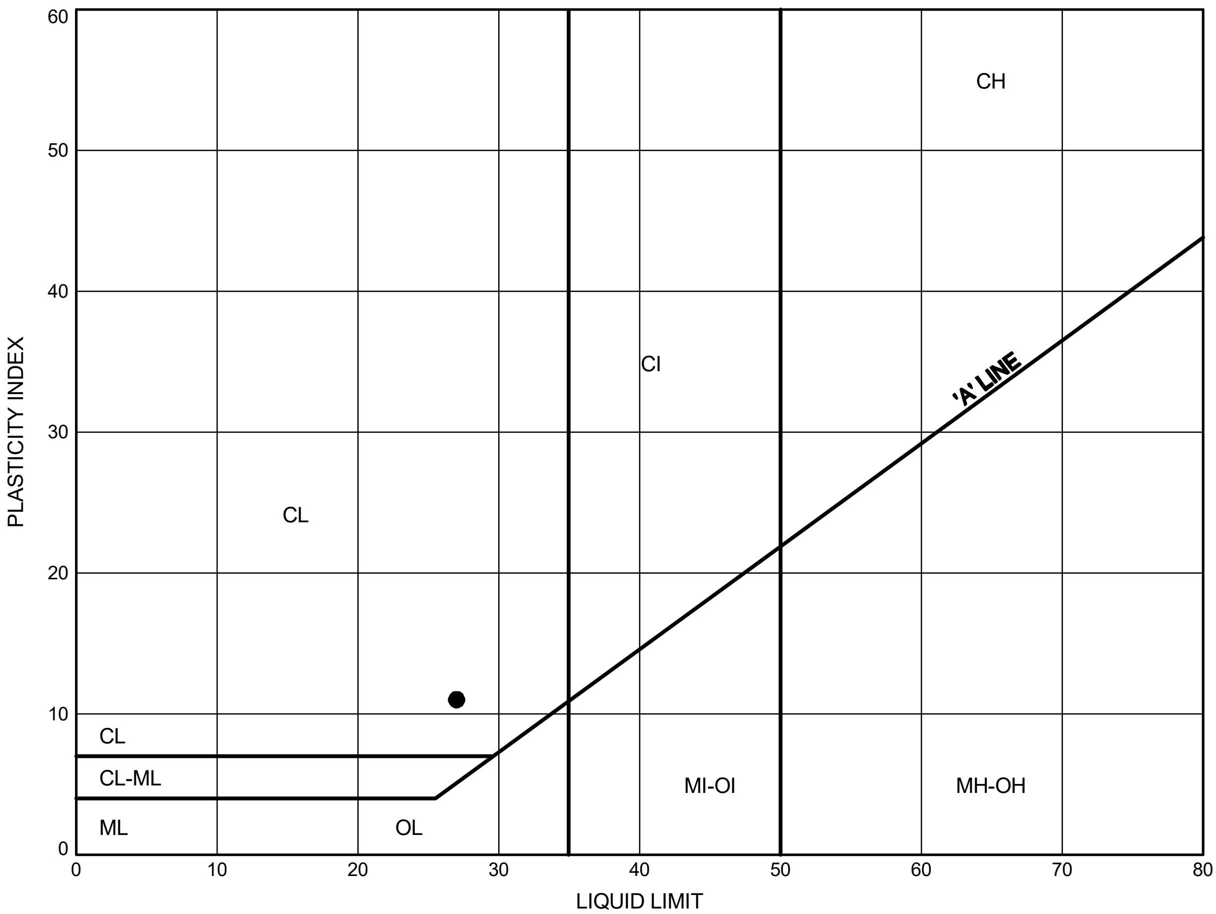
Chkd. AO

Highway 11 - Poplar Rapids Bridge

ATTERBERG LIMITS TEST RESULTS

FIGURE C3

FILL: Silty Sand



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	23-103	0.3	227.0

Date September 2023

GWP# 5278-19-00

THURBER

Prep'd RH

Chkd. AO

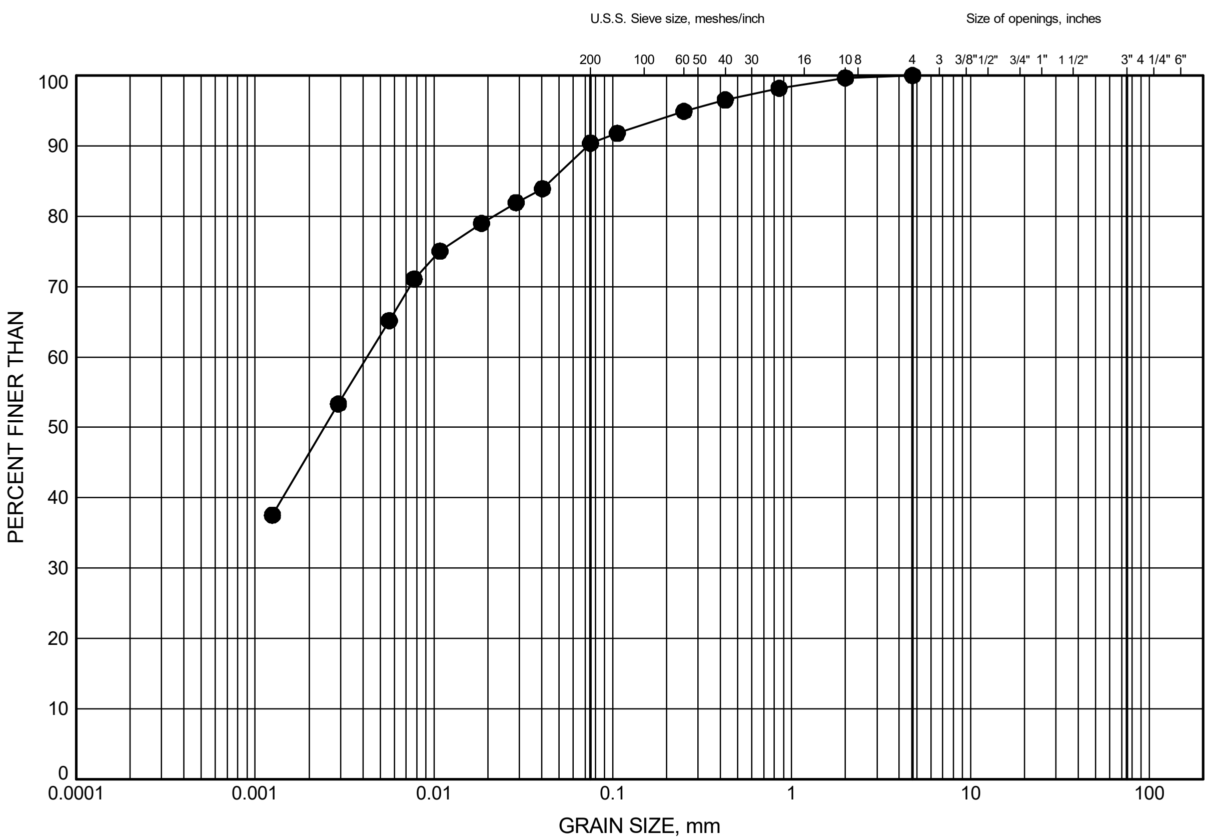
THURBALT 33443 - 100 BHS- POPLAR RIVER BRIDGE.GPJ 9-20-23

Highway 11 - Poplar Rapids Bridge

GRAIN SIZE DISTRIBUTION

FIGURE C4

FILL: Silty Clay



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	23-104	1.8	228.0

Date September 2023

GWP# 5278-19-00

THURBER

Prep'd RH

Chkd. AO

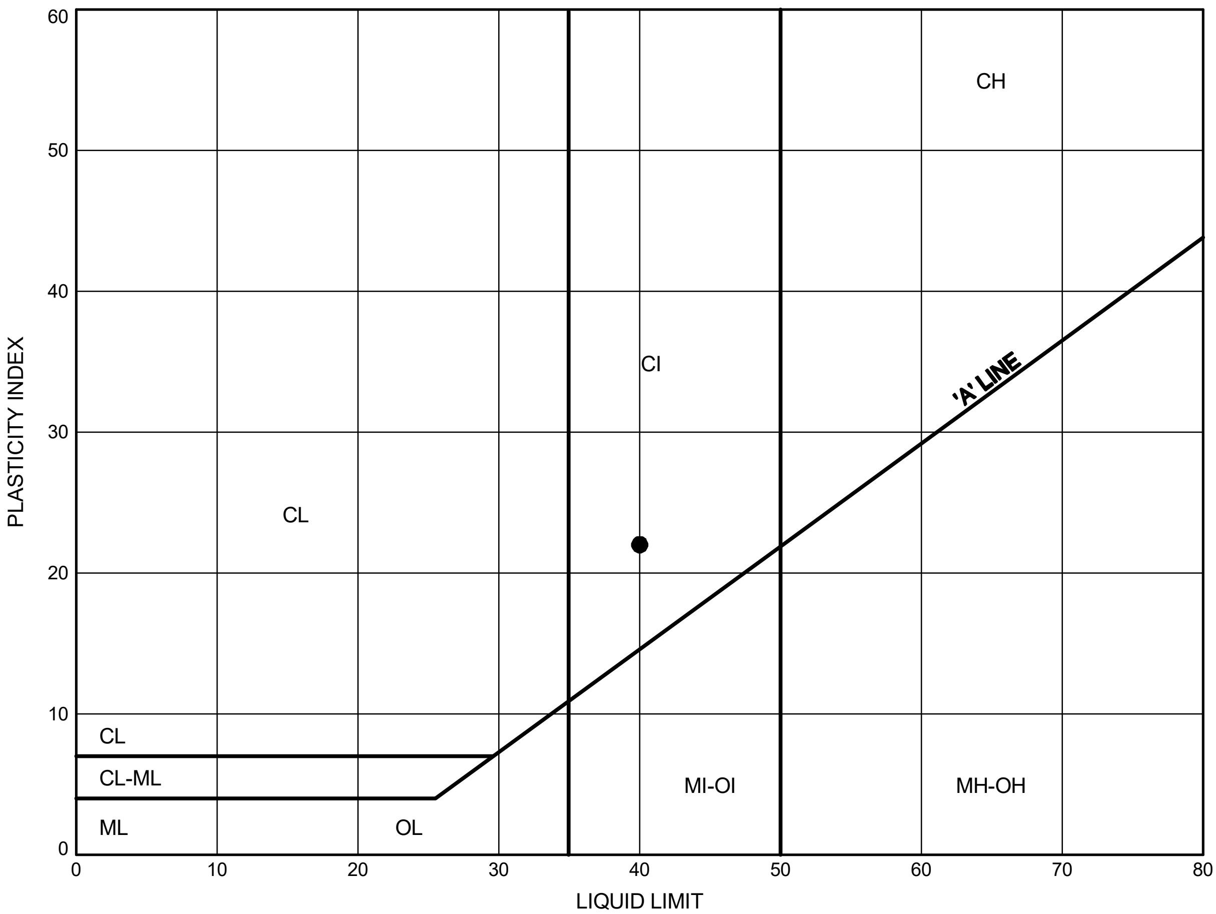
GRAIN SIZE DISTRIBUTION - THURBER 33443 - 100 BHS- POPLAR RIVER BRIDGE.GPJ 9-20-23

Highway 11 - Poplar Rapids Bridge

ATTERBERG LIMITS TEST RESULTS

FIGURE C5

FILL: Silty Clay



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	23-104	1.8	228.0

Date September 2023
GWP# 5278-19-00

THURBER

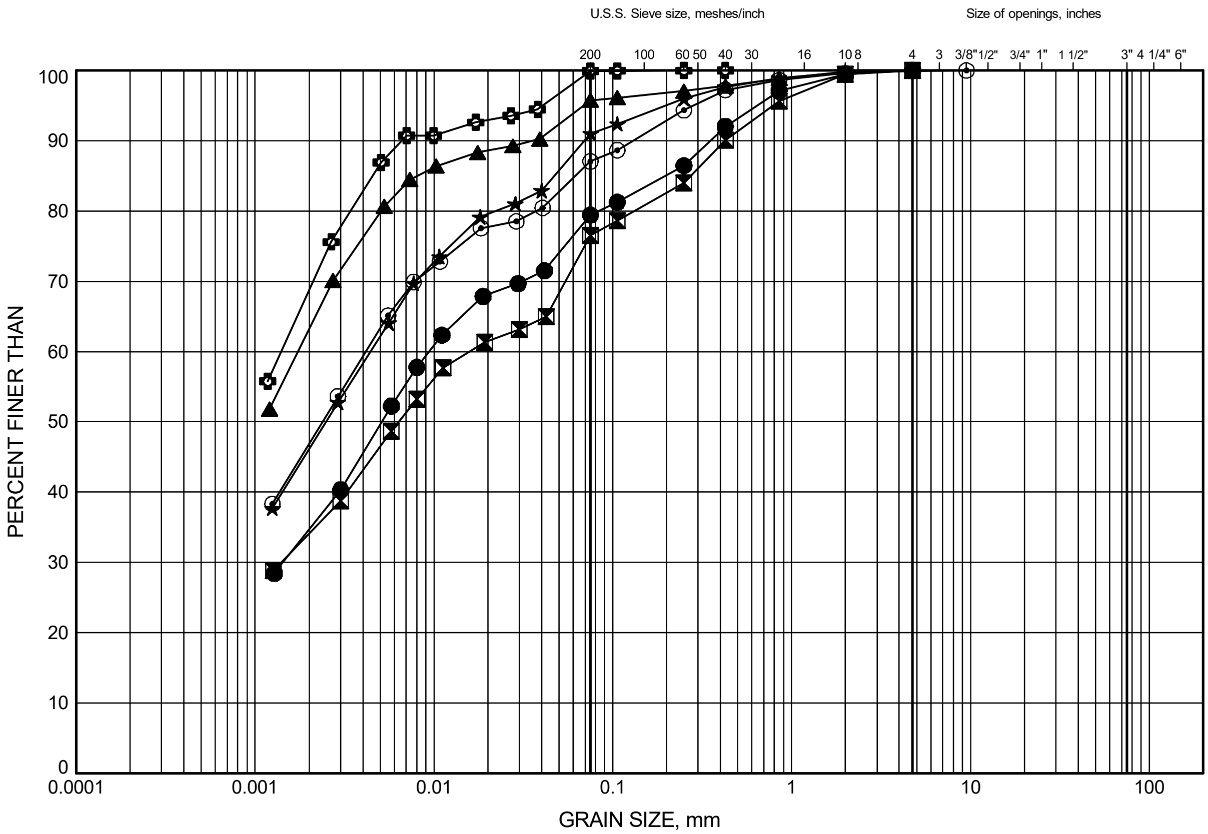
Prep'd RH
Chkd. AO

Highway 11 - Poplar Rapids Bridge

GRAIN SIZE DISTRIBUTION

FIGURE C6

Clay (CH) to Silty Clay (CI) to Clayey Silt (CL / CL-ML)



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	23-101	3.4	225.9
⧗	23-101	4.9	224.4
▲	23-101	7.9	221.4
★	23-102	1.8	225.2
⊙	23-102	3.4	223.7
✿	23-102	4.9	222.1

Date September 2023

GWP# 5278-19-00

THURBER

Prep'd RH

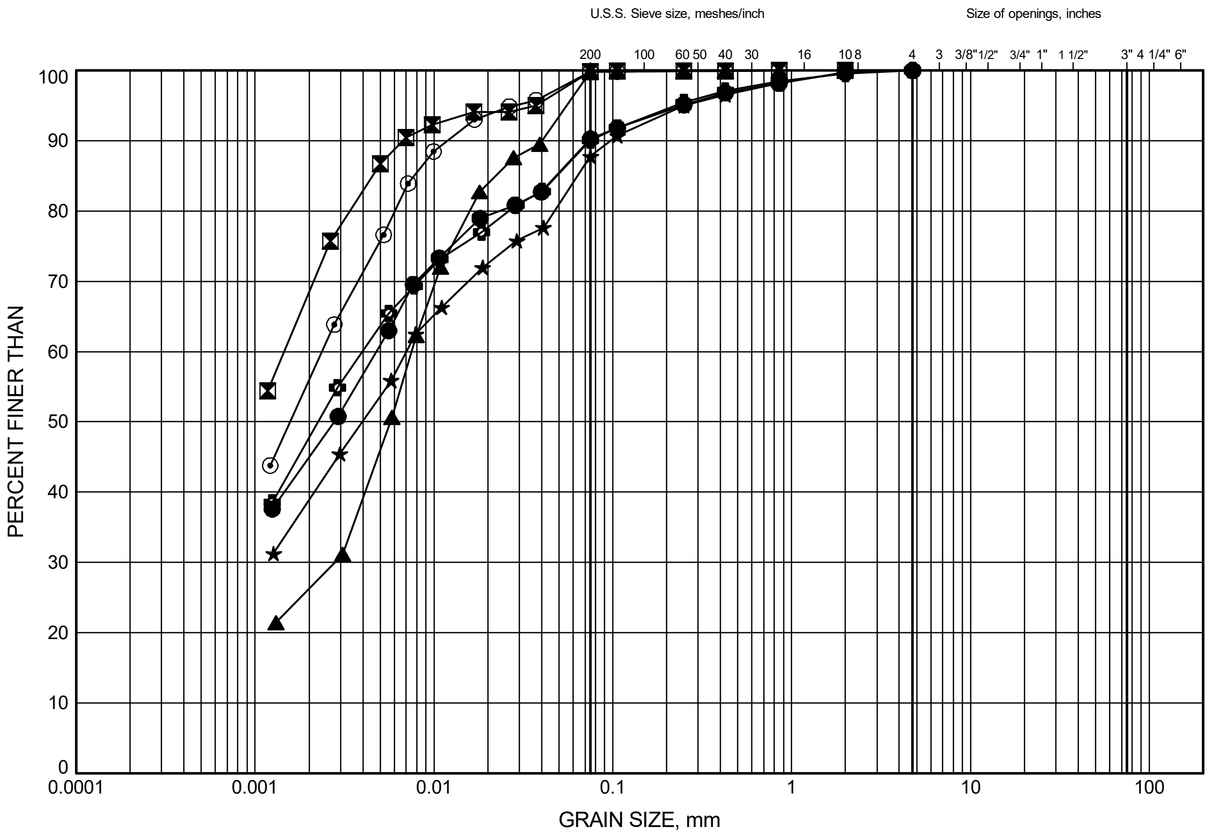
Chkd. AO

Highway 11 - Poplar Rapids Bridge

GRAIN SIZE DISTRIBUTION

FIGURE C7

Clay (CH) to Silty Clay (CI) to Clayey Silt (CL / CL-ML)



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	23-103	3.4	223.9
⧗	23-103	5.6	221.7
▲	23-103	7.2	220.1
★	23-104	5.6	224.1
⊙	23-104	7.9	221.9
✿	23-105	1.8	226.7

Date September 2023

GWP# 5278-19-00

THURBER

Prep'd RH

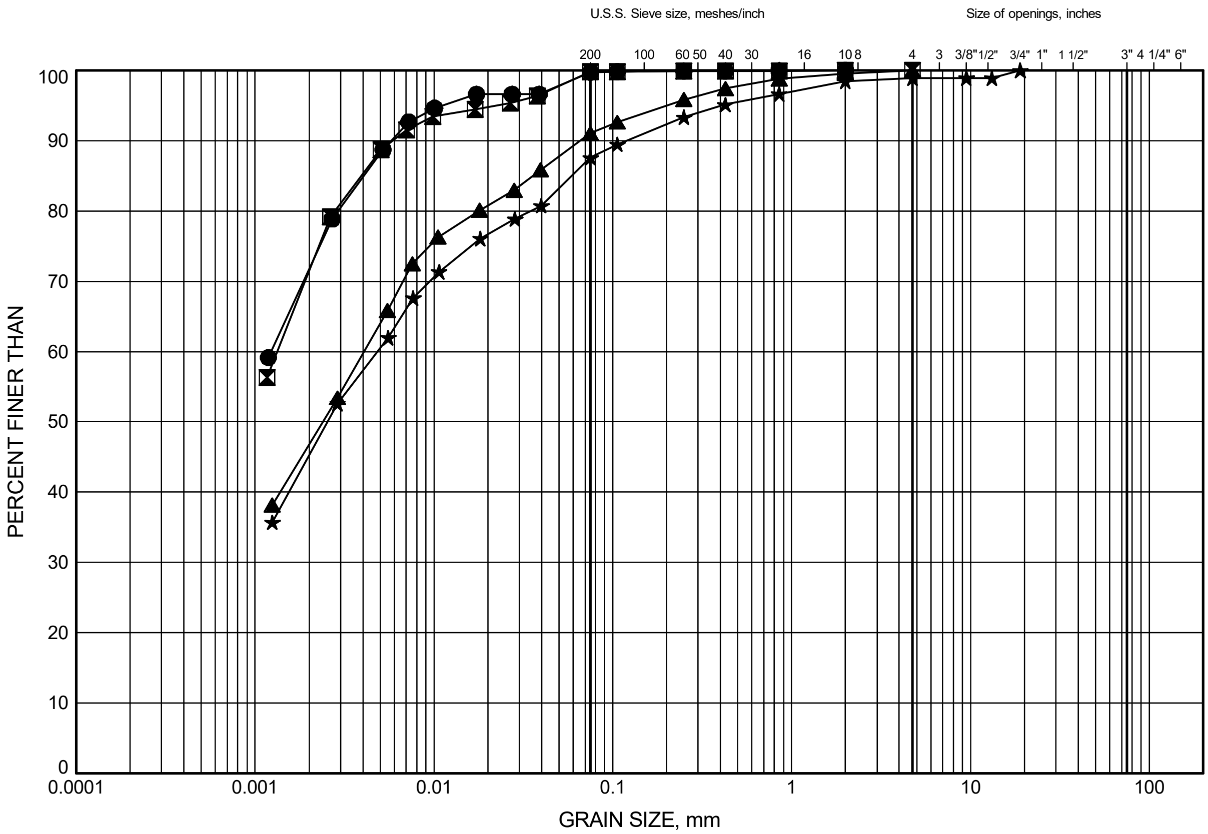
Chkd. AO

Highway 11 - Poplar Rapids Bridge

GRAIN SIZE DISTRIBUTION

FIGURE C8

Clay (CH) to Silty Clay (CI) to Clayey Silt (CL / CL-ML)



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	23-105	5.6	222.9
⊠	23-105	6.4	222.1
▲	23-106	1.1	228.0
★	23-106	3.4	225.7

Date September 2023

GWP# 5278-19-00

THURBER

Prep'd RH

Chkd. AO

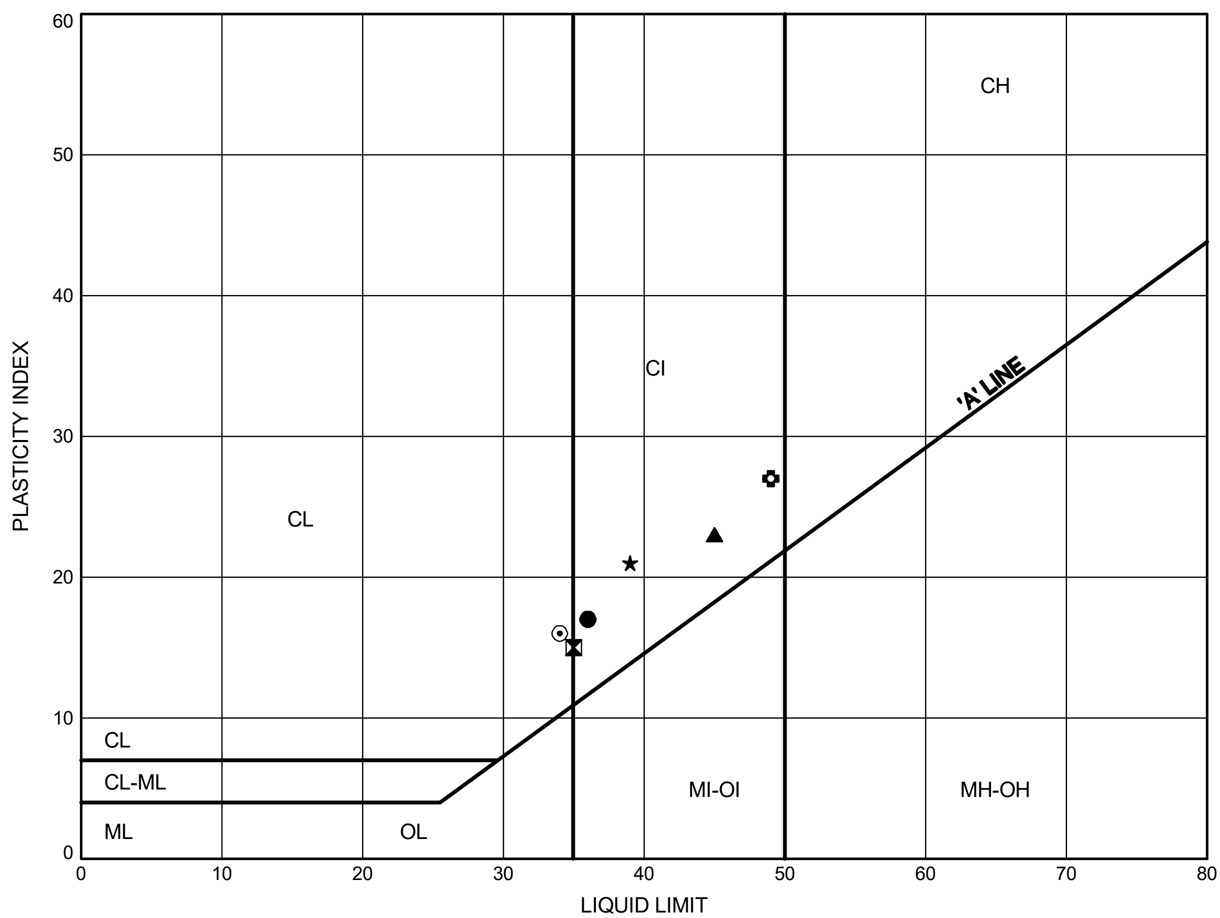
GRAIN SIZE DISTRIBUTION - THURBER 33443 - 100 BHS- POPLAR RIVER BRIDGE.GPJ 9-20-23

Highway 11 - Poplar Rapids Bridge

ATTERBERG LIMITS TEST RESULTS

FIGURE C9

Clay (CH) to Silty Clay (CI) to Clayey Silt (CL / CL-ML)



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	23-101	3.4	225.9
⊠	23-101	4.9	224.4
▲	23-101	7.9	221.4
★	23-102	1.8	225.2
⊙	23-102	3.4	223.7
✿	23-102	4.9	222.1

THURBALT 33443 - 100 BHS- POPLAR RIVER BRIDGE.GPJ 9-20-23

Date September 2023

GWP# 5278-19-00

THURBER

Prep'd RH

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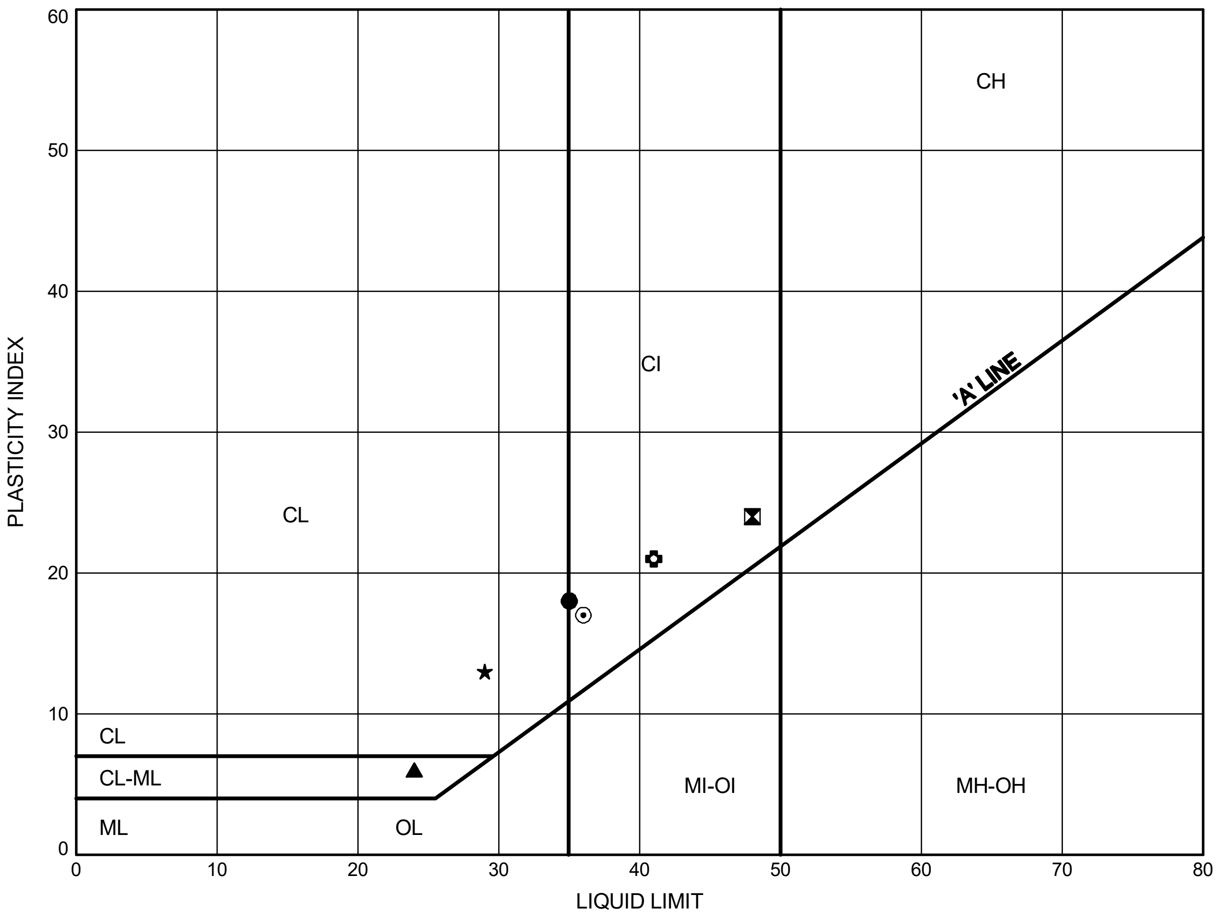
Highway 11 - Poplar Rapids Bridge

ATTERBERG LIMITS TEST RESULTS

FIGURE C10

Clay (CH) to Silty Clay (CI) to Clayey Silt (CL / CL-ML)



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	23-103	3.4	223.9
⧗	23-103	5.6	221.7
▲	23-103	7.2	220.1
★	23-104	5.6	224.1
⊙	23-104	7.9	221.9
✿	23-105	1.8	226.7

Date September 2023

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THURBER

Prep'd RH

Chkd. AO

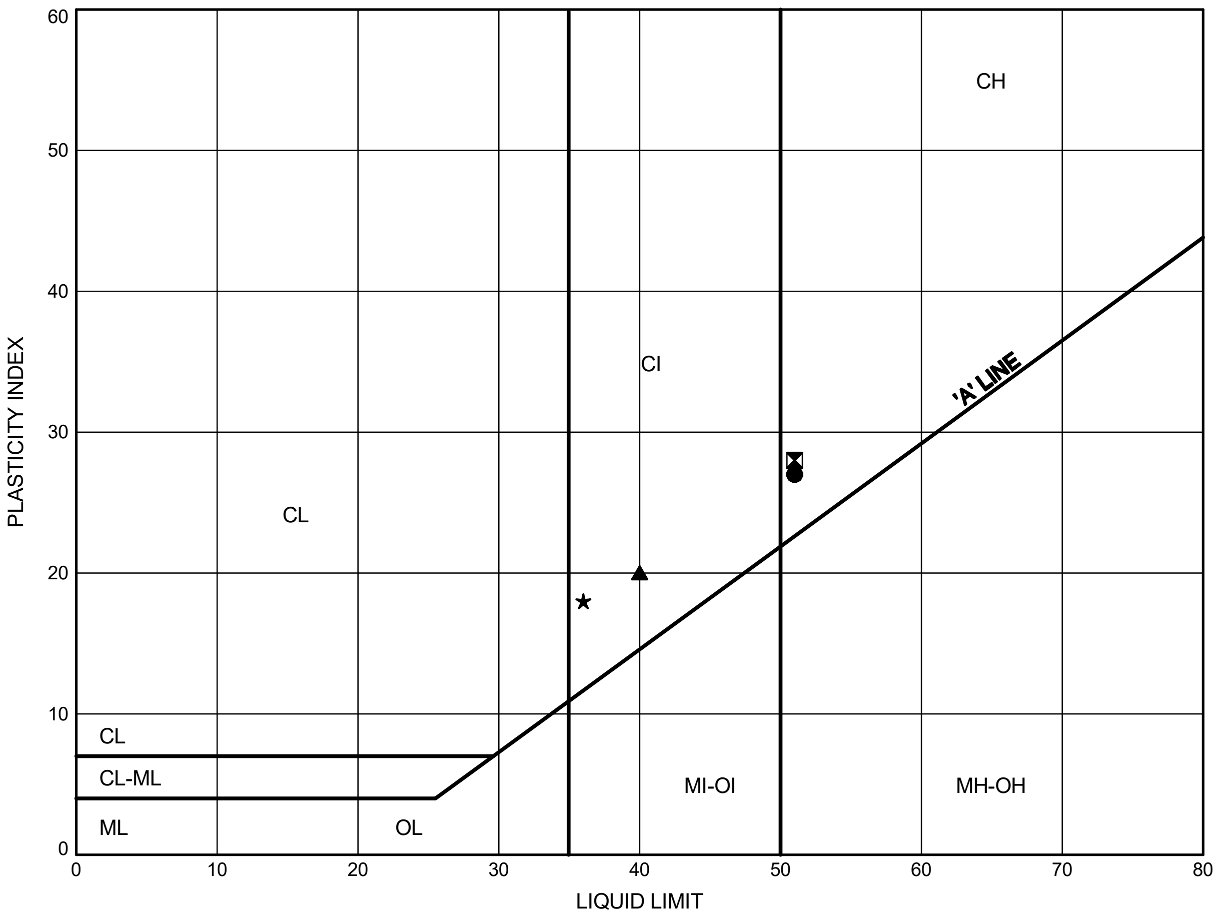
THURBALT 33443 - 100 BHS- POPLAR RIVER BRIDGE.GPJ 9-20-23

Highway 11 - Poplar Rapids Bridge

ATTERBERG LIMITS TEST RESULTS

FIGURE C11

Clay (CH) to Silty Clay (CI) to Clayey Silt (CL / CL-ML)



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	23-105	5.6	222.9
⧗	23-105	6.4	222.1
▲	23-106	1.1	228.0
★	23-106	3.4	225.7

THURBALT 33443 - 100 BHS- POPLAR RIVER BRIDGE.GPJ 9-20-23

Date September 2023

GWP# 5278-19-00

THURBER

Prep'd RH

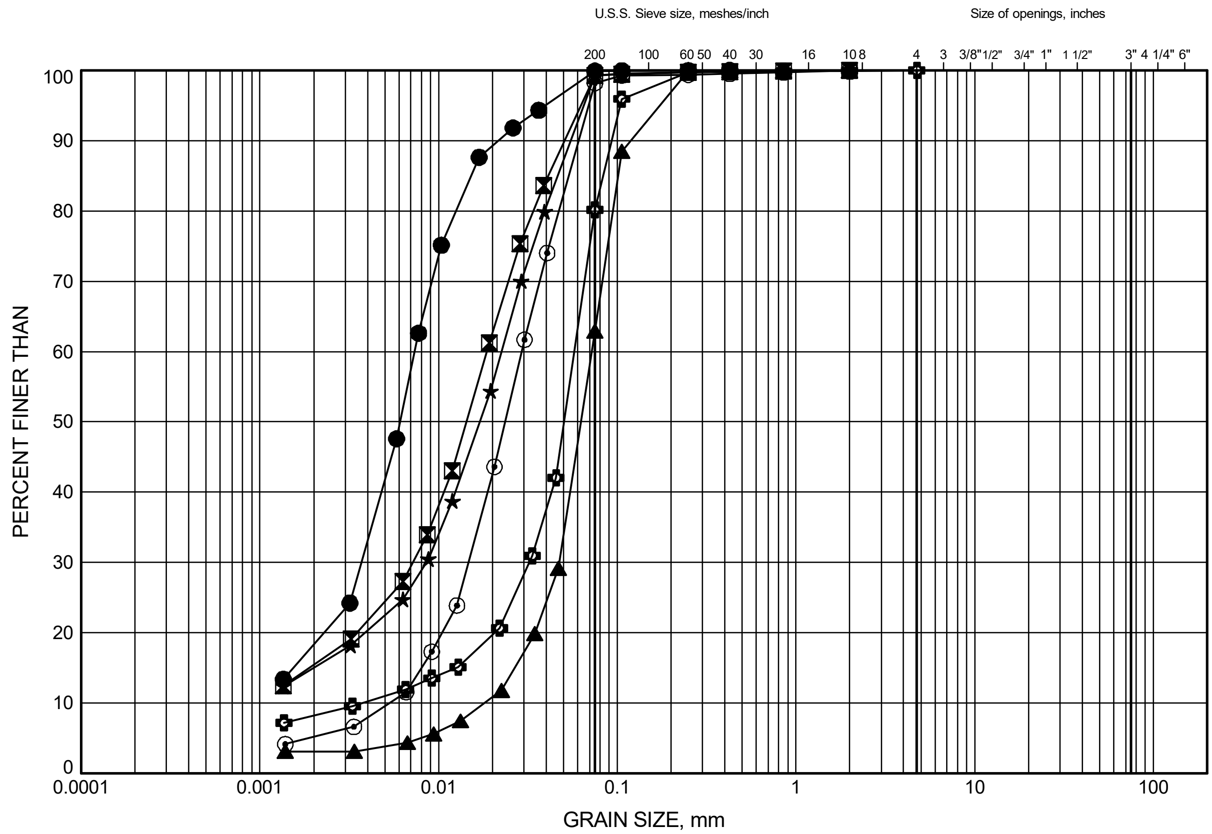
Chkd. AO

Highway 11 - Poplar Rapids Bridge

GRAIN SIZE DISTRIBUTION

FIGURE C12

Silt to Sandy Silt (ML)



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	23-101	9.4	219.9
⊠	23-101	12.5	216.8
▲	23-101	15.5	213.8
★	23-102	8.7	218.3
⊙	23-102	11.0	216.0
✿	23-102	12.5	214.5

Date September 2023

GWP# 5278-19-00

THURBER

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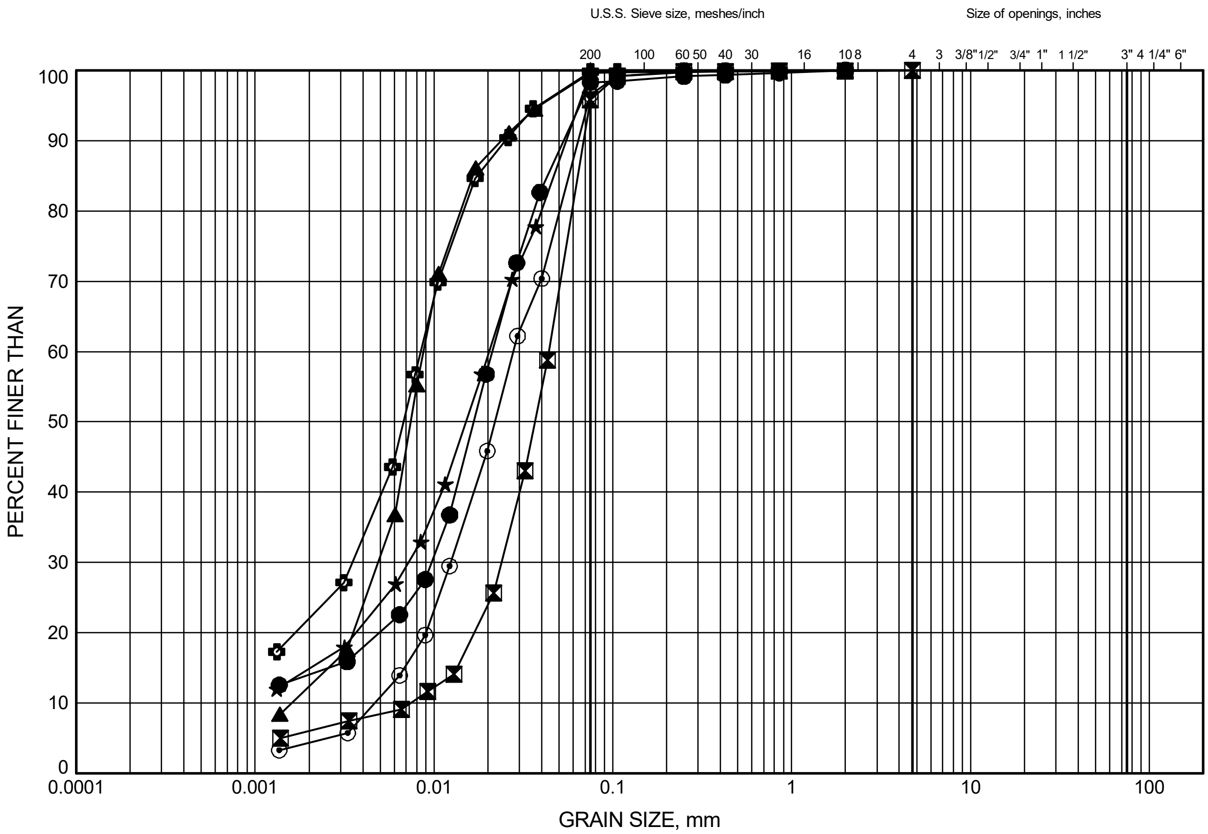
Highway 11 - Poplar Rapids Bridge

GRAIN SIZE DISTRIBUTION

FIGURE C13

Silt to Sandy Silt (ML)



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	23-103	11.0	216.3
⧗	23-103	15.5	211.8
▲	23-104	9.6	220.2
★	23-104	11.0	218.8
⊙	23-104	12.5	217.3
✪	23-105	8.7	219.8

GRAIN SIZE DISTRIBUTION - THURBER 33443 - 100 BHS- POPLAR RIVER BRIDGE.GPJ 9-25-23

Date September 2023

GWP# 5278-19-00

THURBER

Prep'd RH

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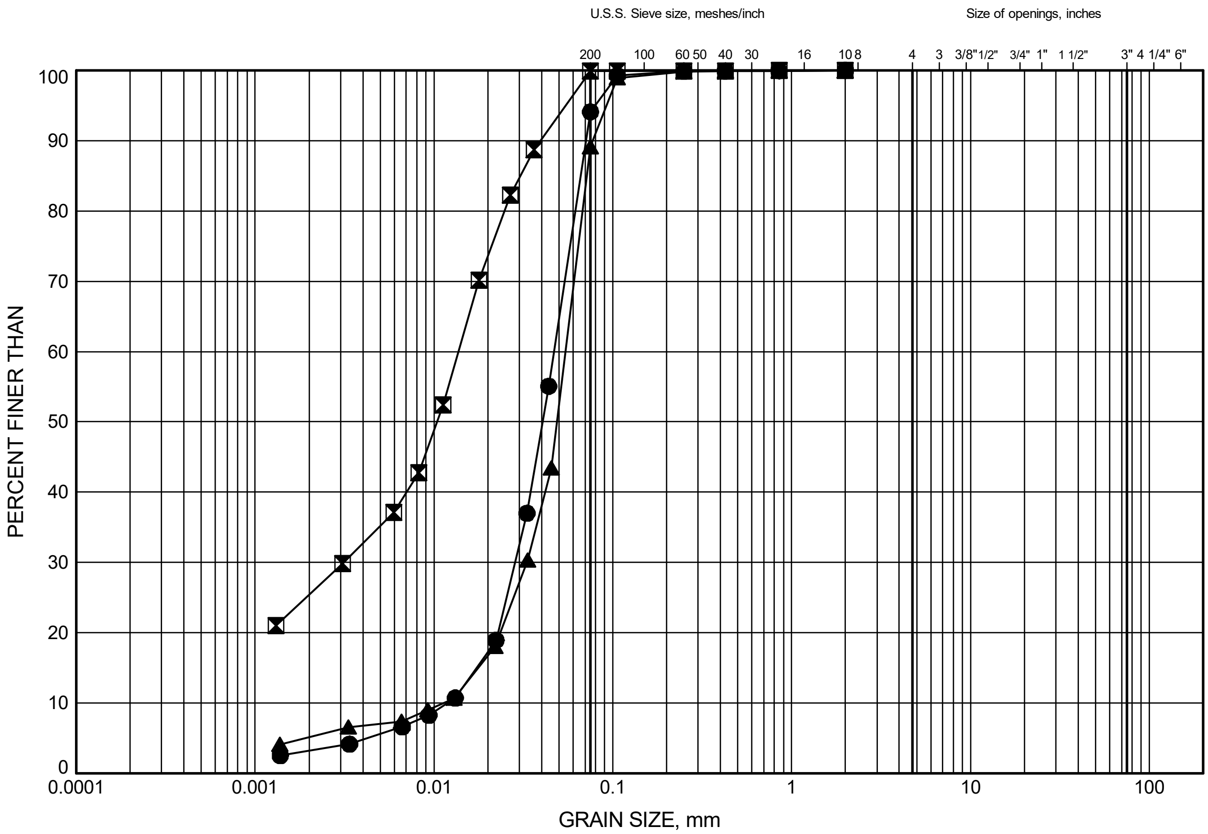
Highway 11 - Poplar Rapids Bridge

GRAIN SIZE DISTRIBUTION

FIGURE C14

Silt to Sandy Silt (ML)



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	23-105	15.5	213.0
⧗	23-106	11.0	218.1
▲	23-106	14.0	215.1

Date September 2023

GWP# 5278-19-00

THURBER

Prep'd RH

Chkd. AO

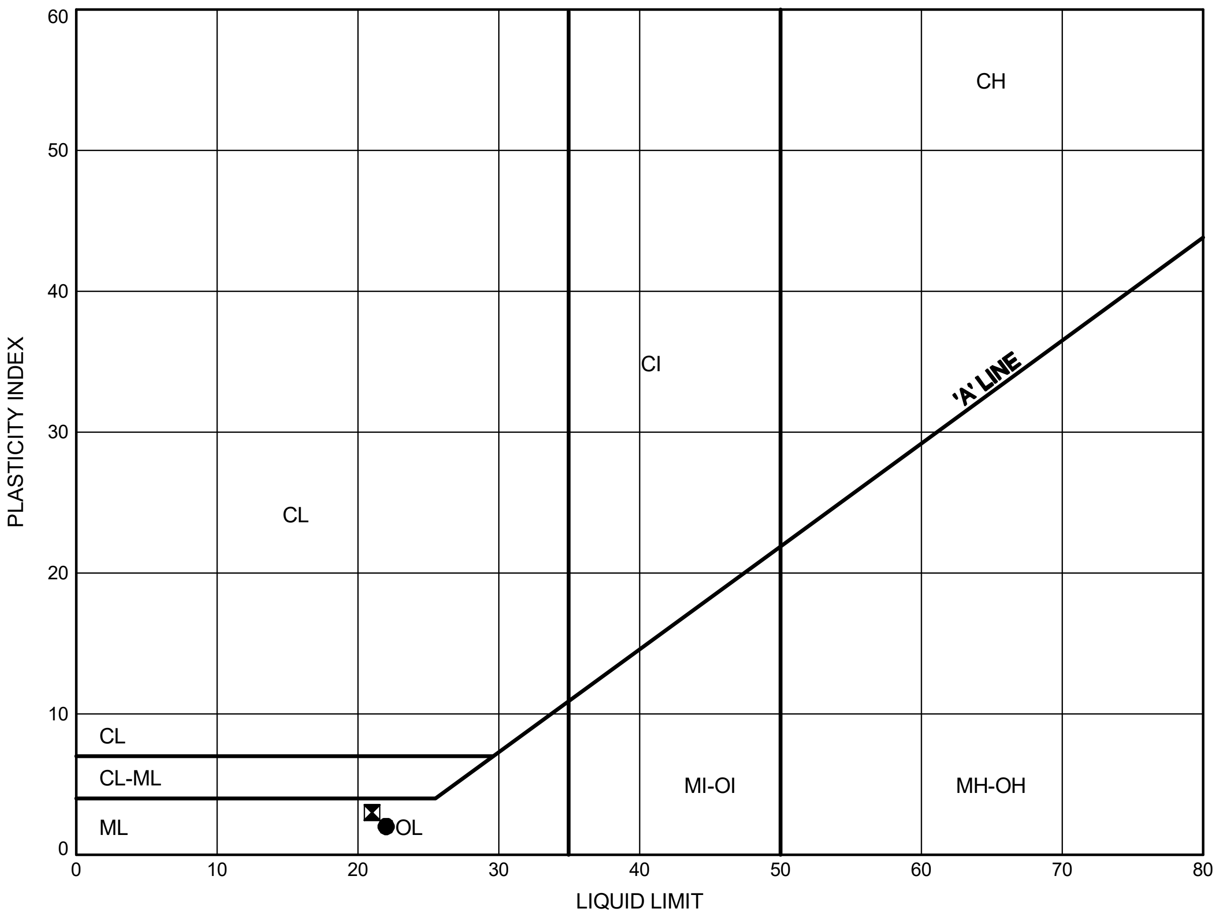
GRAIN SIZE DISTRIBUTION - THURBER 33443 - 100 BHS- POPLAR RIVER BRIDGE.GPJ 9-25-23

Highway 11 - Poplar Rapids Bridge

ATTERBERG LIMITS TEST RESULTS

FIGURE C15

Silt to Sandy Silt (ML)



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	23-105	8.7	219.8
⧗	23-106	11.0	218.1

Date September 2023

GWP# 5278-19-00

THURBER

Prep'd RH

Chkd. AO

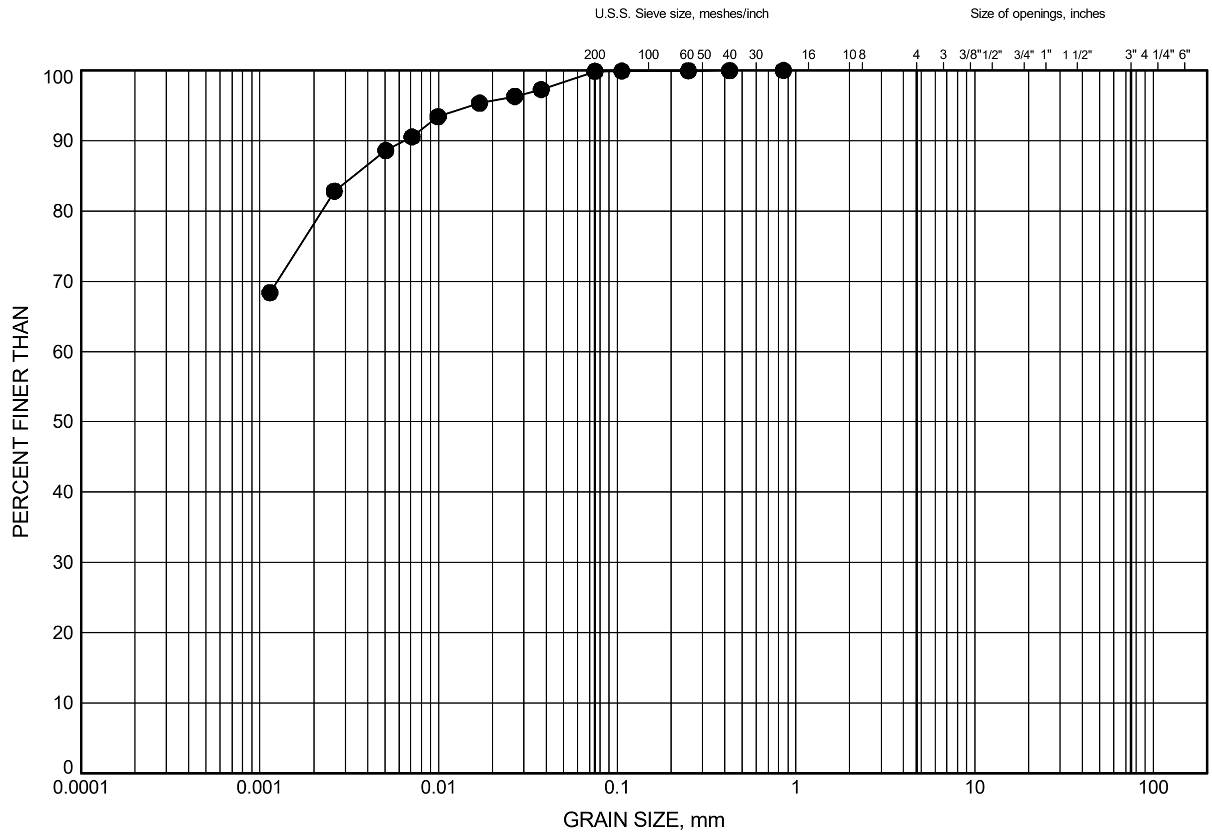
THURBALT 33443 - 100 BHS- POPLAR RIVER BRIDGE.GPJ 9-25-23

Highway 11 - Poplar Rapids Bridge

GRAIN SIZE DISTRIBUTION

FIGURE C16

Clay Seam



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	23-104	9.3	220.5

Date September 2023

GWP# 5278-19-00

THURBER

Prep'd RH

Chkd. AO

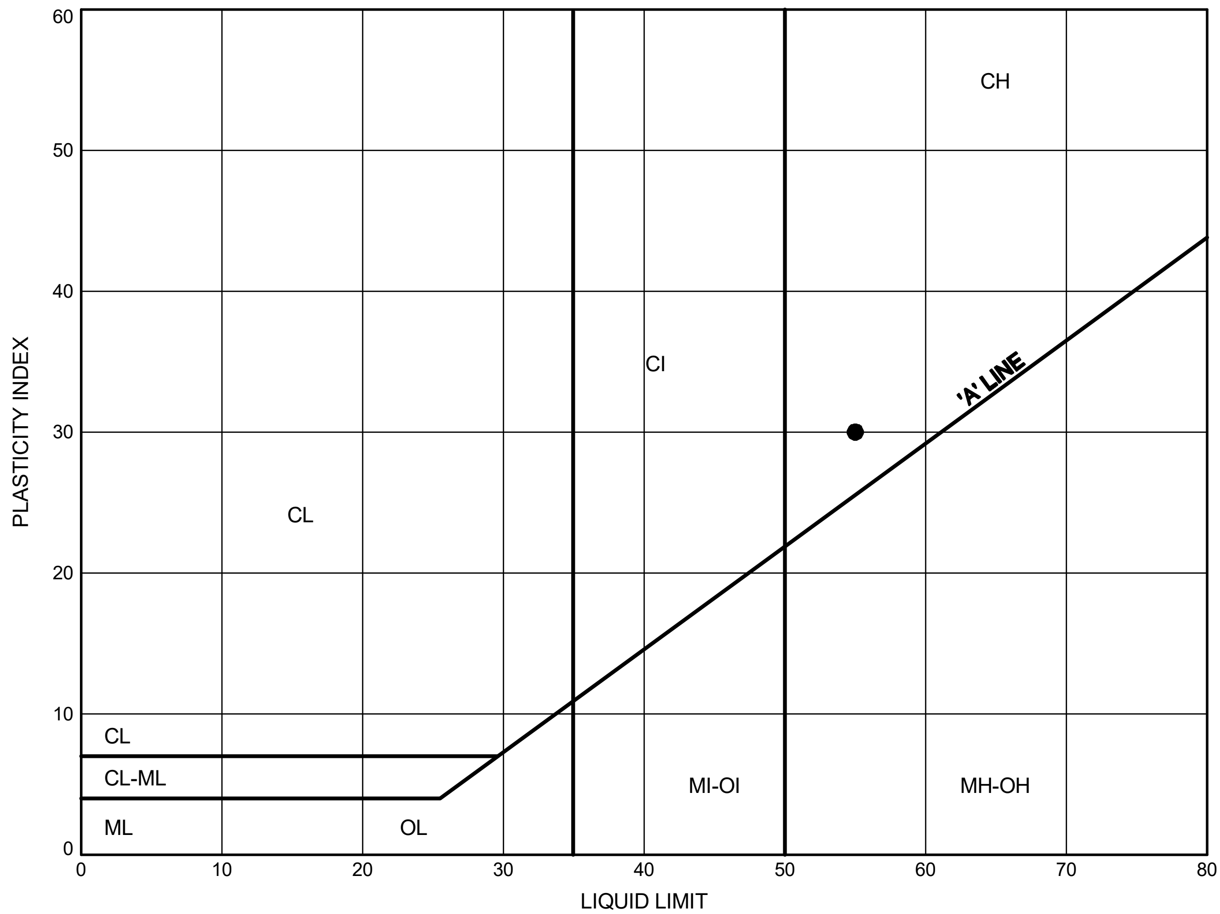
GRAIN SIZE DISTRIBUTION - THURBER 33443 - 100 BHS- POPLAR RIVER BRIDGE.GPJ 9-25-23

Highway 11 - Poplar Rapids Bridge

ATTERBERG LIMITS TEST RESULTS

FIGURE C17

Clay Seam



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	23-104	9.3	220.5

THURBALT 33443 - 100 BHS- POPLAR RIVER BRIDGE.GPJ 9-25-23

Date September 2023

GWP# 5278-19-00

THURBER

Prep'd RH

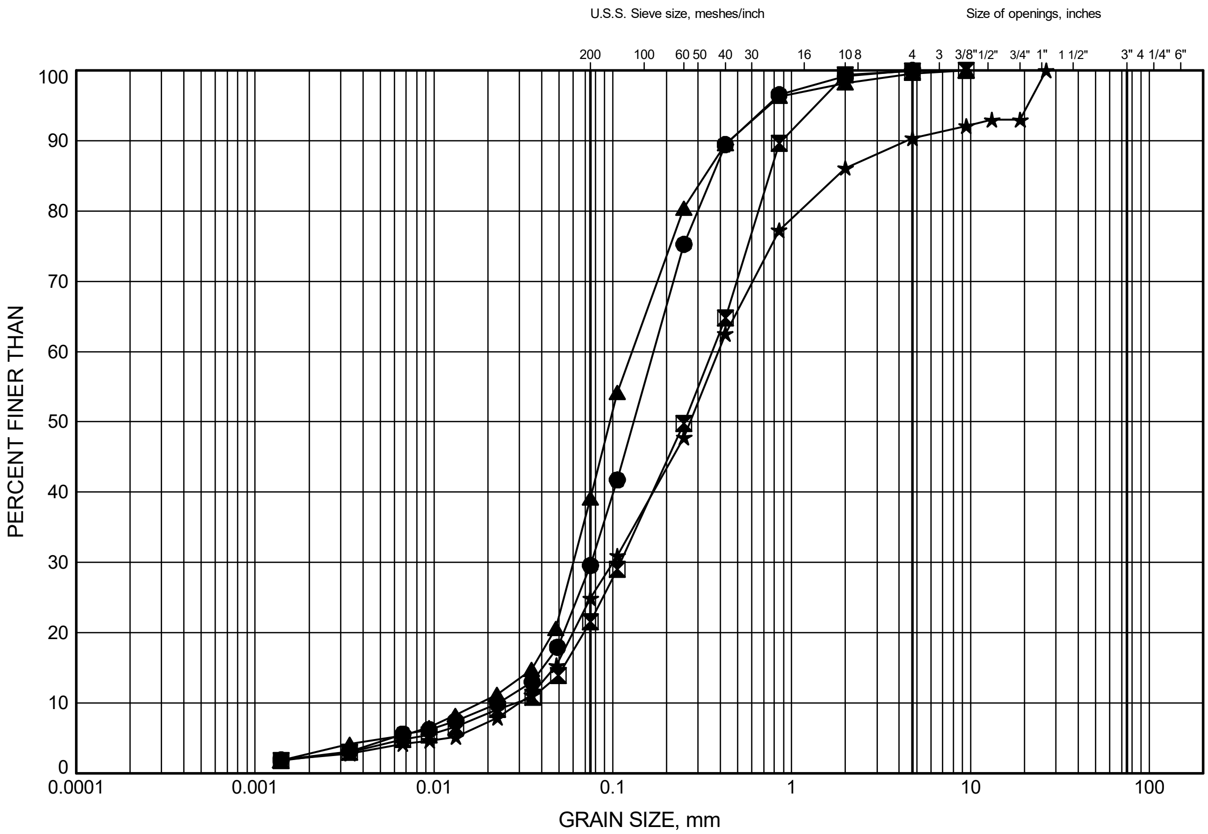
Chkd. AO

Highway 11 - Poplar Rapids Bridge

GRAIN SIZE DISTRIBUTION

FIGURE C18

Silty Sand (SM)



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	23-101	21.6	207.7
⧗	23-102	20.1	206.9
▲	23-104	21.6	208.1
★	23-105	18.4	210.1

Date September 2023

GWP# 5278-19-00

THURBER

Prep'd RH

Chkd. AO

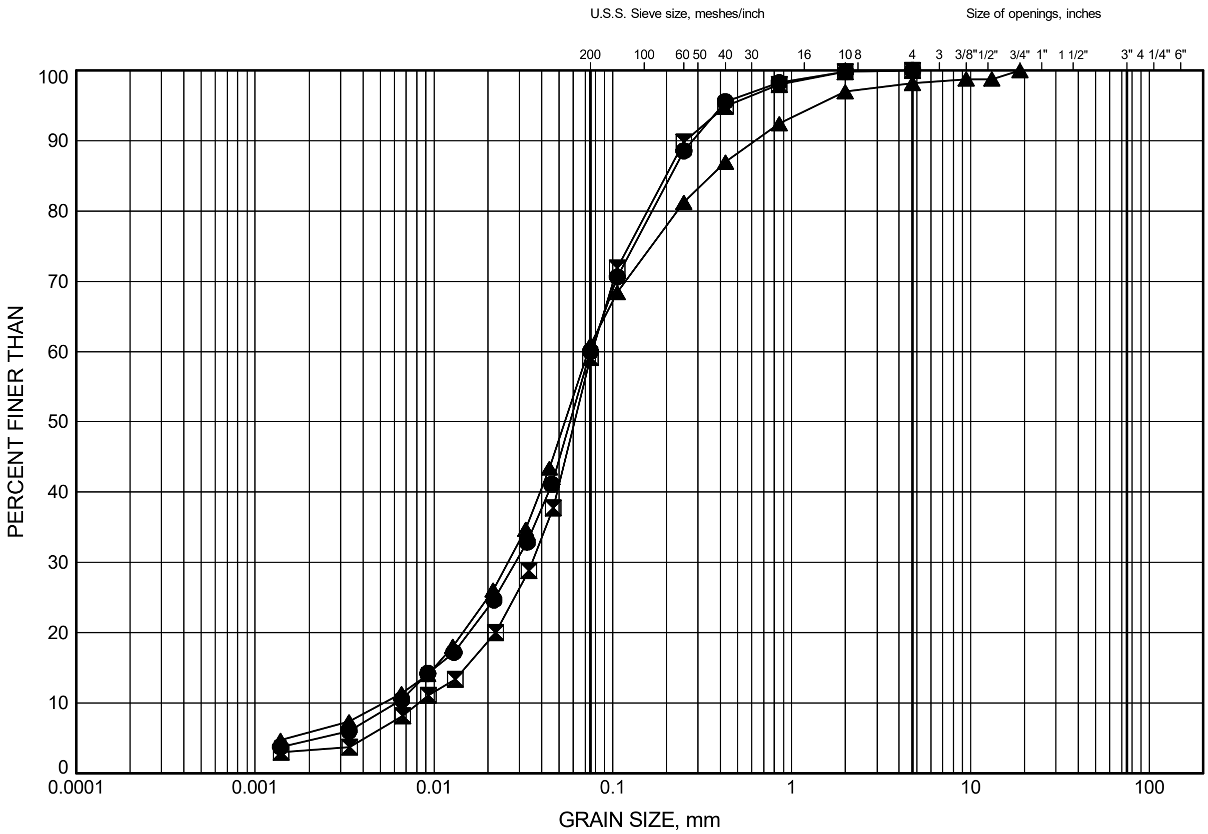
GRAIN SIZE DISTRIBUTION - THURBER 33443 - 100 BHS- POPLAR RIVER BRIDGE.GPJ 9-20-23

Highway 11 - Poplar Rapids Bridge

GRAIN SIZE DISTRIBUTION

FIGURE C19

Sand and Silt



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	23-104	26.2	203.6
⊠	23-105	27.7	200.8
▲	23-105	30.8	197.7

Date September 2023

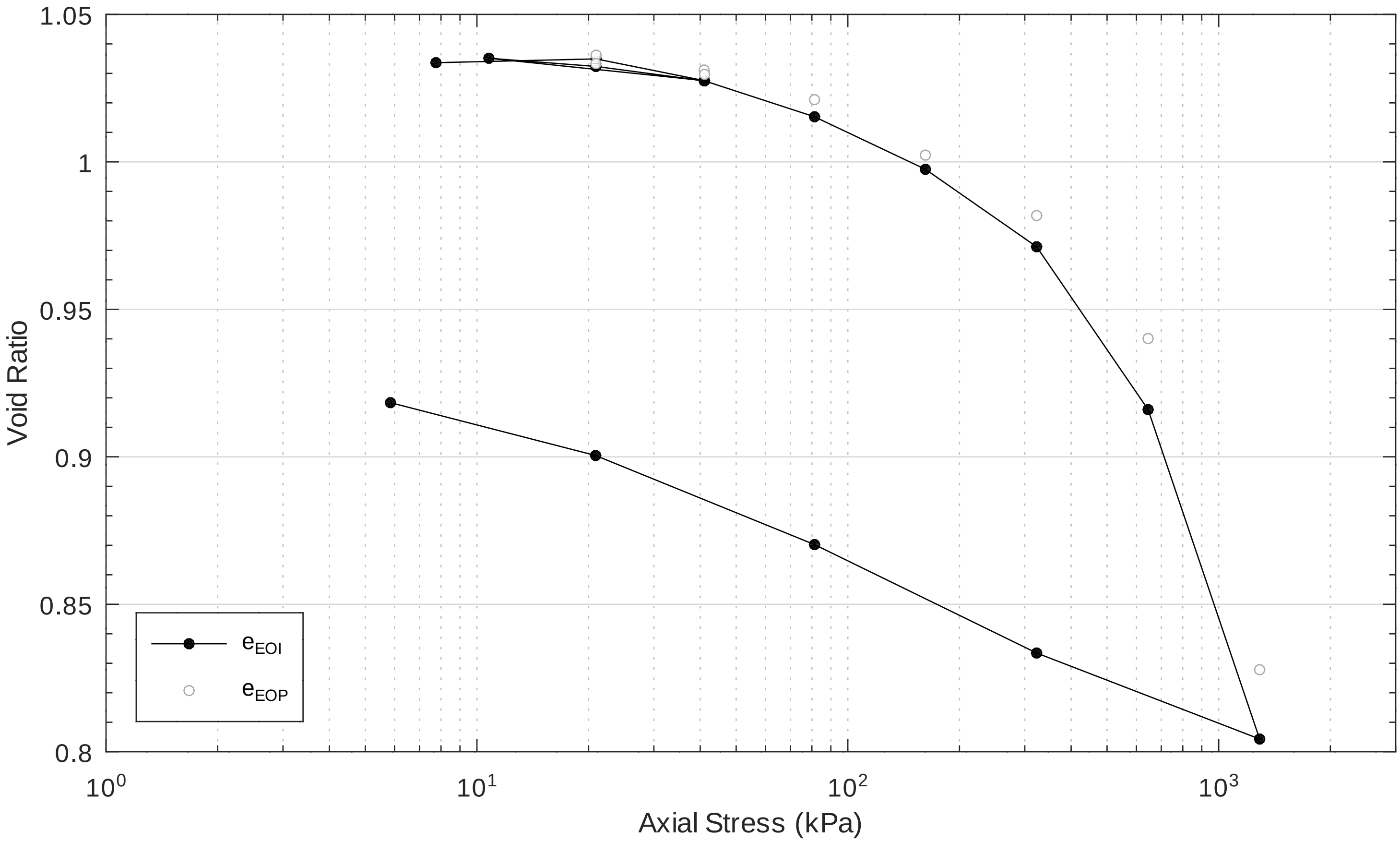
GWP# 5278-19-00

THURBER

Prep'd RH

Chkd. AO

Project: 33443
Hwy 11 Poplar Rapids Bridge
Borehole: 23-101
Sample: ST1
Depth:
Client: LEA/MTO



Start of Test			2023-09-01
Diameter of Sample	cm	D	6.329
Height of Sample	cm	H_o	2.529
Height of Solids	cm	H_s	1.244
Water Content	%	w_o	39.09
Dry Density	g/cm^3	ρ_d	1.35
Moist Unit Weight	kN/m^3	γ	18.3
Void Ratio	-	e_o	1.033
Degree of Saturation	-	S_{ro}	1.04
Specific Gravity	-	G_s	2.736

End of Test			2023-09-17
Height of Sample	cm	H_f	2.387
Water Content	%	w_f	35.39
Void Ratio	-	e_f	0.918

TRIMMING: the specimen was manually trimmed to the size of the consolidation ring, then mounted in a fixed ring consolidometer

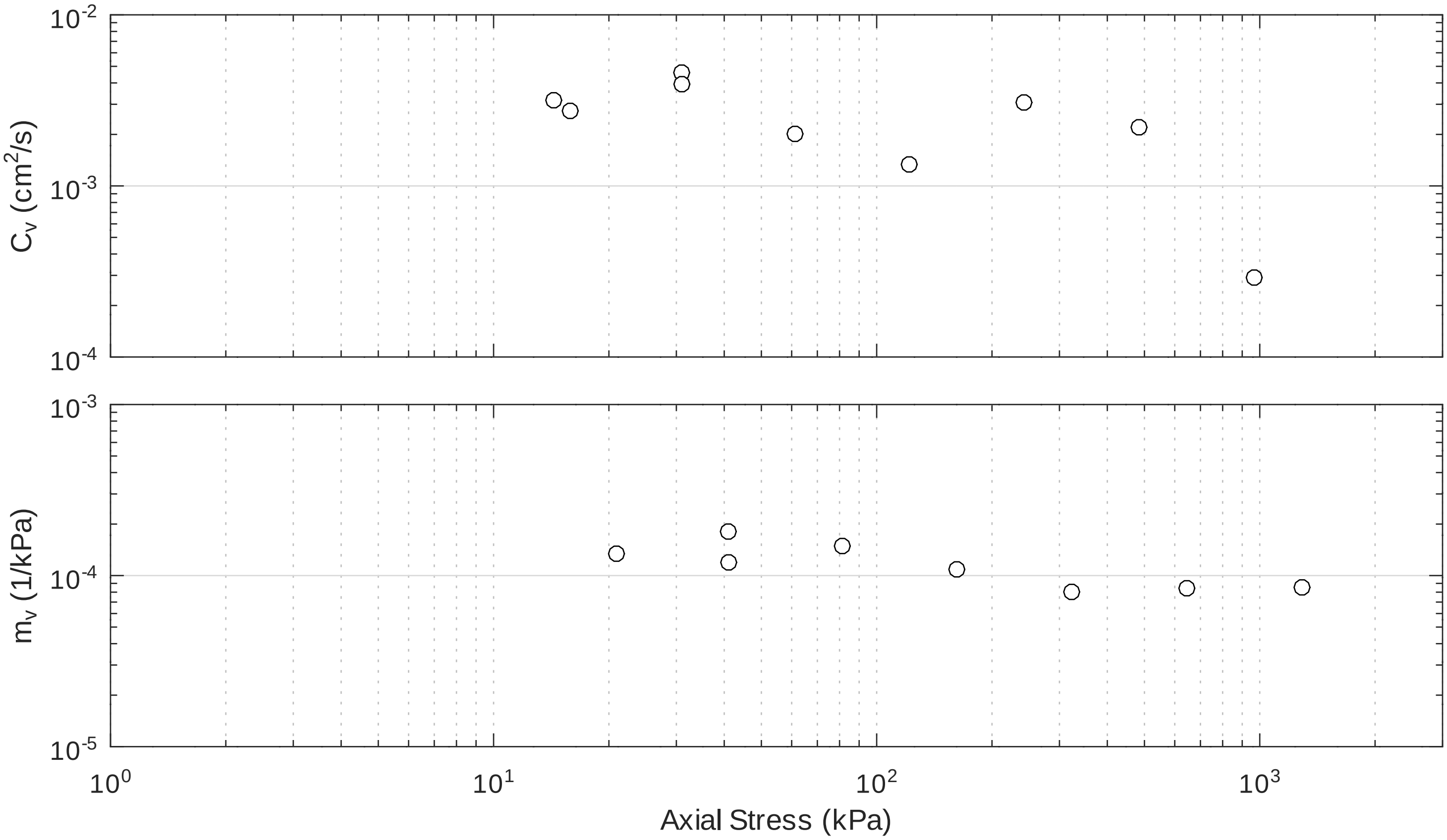
LOADING: the consolidometer was flooded with water with the seating load adjusted to limit swelling

CALCULATIONS: coefficients of consolidation were calculated by the square root time method, secondary consolidation was calculated based on the available duration of the time step

Interpreted Results			
Recompression Index (reloading)	-	C_r	0.050
Compression Index	-	C_c	0.371
Recompression Index (unloading)	-	C_r	0.056
Probable Preconsolidation Pressure	kPa	p'_c	466

Check: AO Review: __________

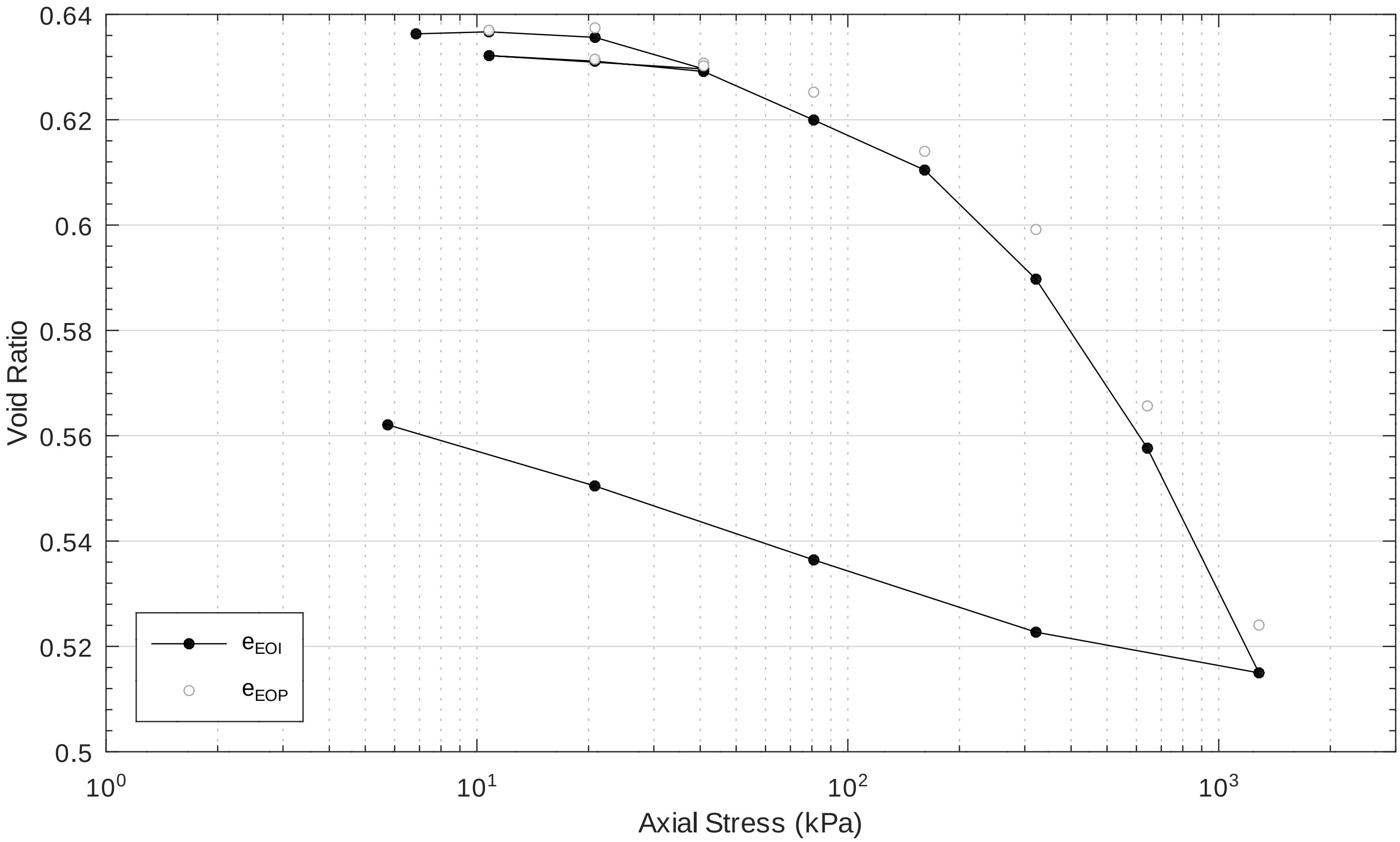
Project: 33443
Hwy 11 Poplar Rapids Bridge
Borehole: 23-101
Sample: ST1
Depth:
Client: LEA/MTO



Load No.	Axial Stress	Load Duration	System Deflec.	Dial	Sample Height	Axial Strain	Void Ratio	Void Ratio	Time U(0.99)	C_v	k_v	$C_{a\varepsilon}$
	kPa	min	mm	mm	cm	%	(EOI)	(EOP)	min	cm^2/s	cm/s	-
0				10.000	2.529	0.00	1.033					
1	7.8	1440.2	0.008	10.005	2.531	-0.05	1.034					
2	21.0	1440.2	0.102	9.926	2.532	-0.11	1.035	1.036	14.7	3.17e-03	-1.49e-08	0.0003
3	41.0	1440.1	0.141	9.795	2.523	0.25	1.028	1.031	10.0	4.60e-03	8.16e-08	0.0008
4	10.8	1440.3	0.115	9.915	2.532	-0.12	1.035					
5	20.9	1440.4	0.122	9.874	2.529	0.01	1.032	1.033	17.1	2.75e-03	3.62e-08	0.0002
6	41.1	1440.0	0.141	9.794	2.523	0.25	1.027	1.030	11.9	3.93e-03	4.61e-08	0.0005
7	81.3	1440.3	0.213	9.571	2.508	0.85	1.015	1.021	22.5	2.01e-03	2.94e-08	0.0015
8	161.9	1440.3	0.295	9.268	2.486	1.73	0.997	1.002	32.3	1.34e-03	1.42e-08	0.0014
9	322.9	1440.4	0.382	8.854	2.453	3.02	0.971	0.982	13.5	3.07e-03	2.42e-08	0.0023
10	645.1	1440.2	0.484	8.065	2.384	5.74	0.916	0.940	16.8	2.20e-03	1.82e-08	0.0060
11	1289.2	1440.0	0.601	6.558	2.245	11.23	0.804	0.828	85.0	2.91e-04	2.44e-09	0.0100
12	322.8	1440.0	0.466	7.055	2.282	9.80	0.833					
13	81.3	1440.3	0.347	7.632	2.327	7.99	0.870					
14	20.9	1440.3	0.280	8.075	2.365	6.50	0.900					
15	5.8	2880.3	0.223	8.354	2.387	5.62	0.918					

One-Dimensional Consolidation, ASTM D2435-11

Project: 33443
Hwy 11 Poplar Rapids Bridge
Borehole: 23-102
Sample: ST1
Depth: 3.0m
Client: LEA/MTO



Start of Test			2023-08-03
Diameter of Sample	cm	D	6.342
Height of Sample	cm	H_o	2.556
Height of Solids	cm	H_s	1.563
Water Content	%	w_o	21.96
Dry Density	g/cm^3	ρ_d	1.67
Moist Unit Weight	kN/m^3	γ	20.0
Void Ratio	-	e_o	0.635
Degree of Saturation	-	S_{ro}	0.94
Specific Gravity	-	G_s	2.730

End of Test			2023-08-20
Height of Sample	cm	H_f	2.441
Water Content	%	w_f	20.53
Void Ratio	-	e_f	0.562

TRIMMING: the specimen was manually trimmed to the size of the consolidation ring, then mounted in a fixed ring consolidometer

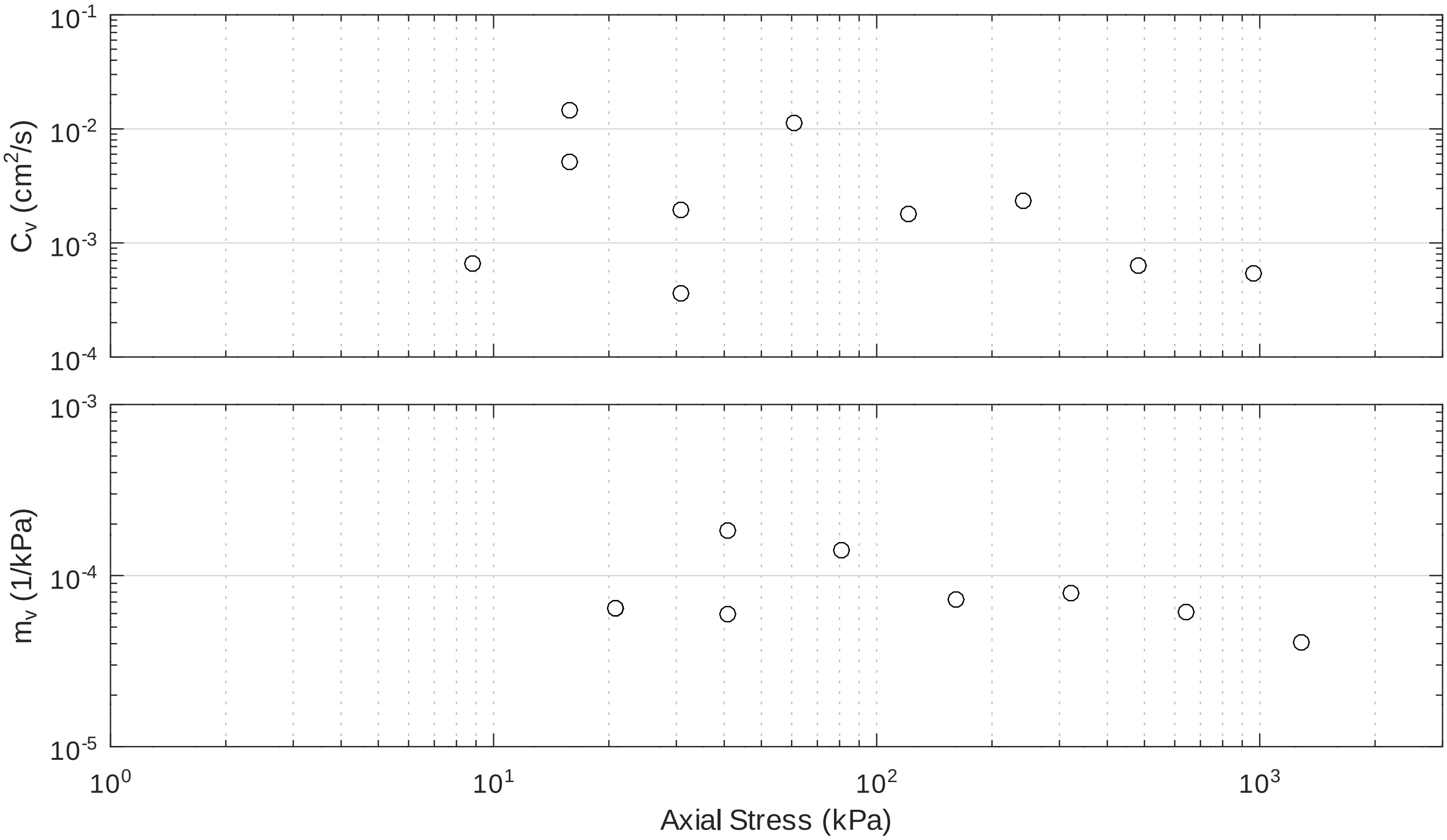
LOADING: the consolidometer was flooded with water with the seating load adjusted to limit swelling

CALCULATIONS: coefficients of consolidation were calculated by the square root time method, secondary consolidation was calculated based on the available duration of the time step

Interpreted Results			
Recompression Index (reloading)	-	C_r	0.031
Compression Index	-	C_c	0.142
Recompression Index (unloading)	-	C_r	0.023
Probable Preconsolidation Pressure	kPa	p'_c	337

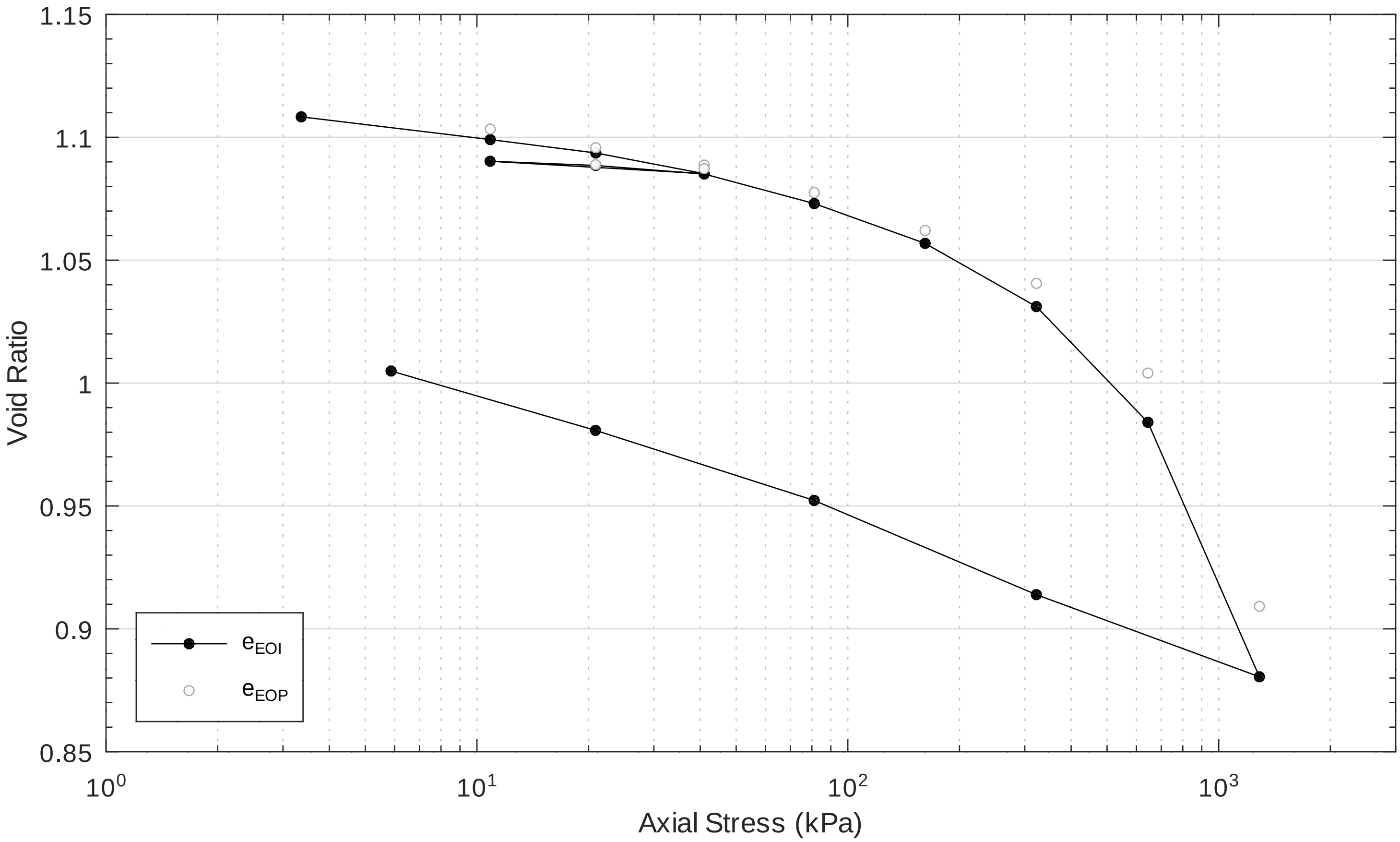
Check: AO Review: __________

Project: 33443
Hwy 11 Poplar Rapids Bridge
Borehole: 23-102
Sample: ST1
Depth: 3.0m
Client: LEA/MTO



Load No.	Axial Stress	Load Duration	System Deflec.	Dial	Sample Height	Axial Strain	Void Ratio	Void Ratio	Time U(0.99)	C_v	k_v	$C_{a\varepsilon}$
	kPa	min	mm	mm	cm	%	(EOI)	(EOP)	min	cm²/s	cm/s	-
0				10.000	2.556	0.00	0.635					
1	6.9	1440.1	0.021	9.997	2.557	-0.07	0.636					
2	10.8	1440.4	0.041	9.983	2.558	-0.09	0.637	0.637	73.2	6.60e-04	-3.91e-09	0.0001
3	20.8	1440.2	0.076	9.931	2.556	-0.03	0.636	0.637	3.3	1.46e-02	9.20e-08	0.0005
4	40.8	1440.4	0.122	9.791	2.547	0.34	0.630	0.631	130.3	3.61e-04	6.50e-09	0.0007
5	10.8	1440.4	0.088	9.864	2.551	0.18	0.632					
6	20.8	1440.4	0.102	9.835	2.549	0.25	0.631	0.631	9.4	5.13e-03	3.24e-08	0.0000
7	40.8	1440.3	0.128	9.779	2.546	0.37	0.629	0.630	24.6	1.95e-03	1.14e-08	0.0004
8	80.9	1440.1	0.186	9.576	2.532	0.93	0.620	0.625	4.2	1.13e-02	1.56e-07	0.0012
9	161.1	1440.3	0.266	9.347	2.517	1.51	0.610	0.614	25.3	1.79e-03	1.28e-08	0.0013
10	321.5	1440.3	0.362	8.927	2.485	2.78	0.590	0.599	18.5	2.34e-03	1.81e-08	0.0030
11	642.3	1440.4	0.474	8.313	2.435	4.74	0.558	0.566	61.4	6.33e-04	3.80e-09	0.0032
12	1283.8	1440.2	0.622	7.498	2.368	7.35	0.515	0.524	63.4	5.41e-04	2.16e-09	0.0042
13	321.5	1440.2	0.470	7.770	2.380	6.88	0.523					
14	81.0	1440.3	0.367	8.088	2.401	6.04	0.536					
15	20.8	1440.4	0.298	8.377	2.423	5.18	0.550					
16	5.7	2880.5	0.244	8.613	2.441	4.47	0.562					

Project: 33443
Hwy 11 Poplar Rapids Bridge
Borehole: 23-104
Sample: ST1
Depth: 9.1m
Client: LEA/MTO



Start of Test			2023-09-01
Diameter of Sample	cm	D	6.334
Height of Sample	cm	H_o	2.534
Height of Solids	cm	H_s	1.203
Water Content	%	w_o	41.25
Dry Density	g/cm^3	ρ_d	1.29
Moist Unit Weight	kN/m^3	γ	17.9
Void Ratio	-	e_o	1.106
Degree of Saturation	-	S_{ro}	1.02
Specific Gravity	-	G_s	2.722

End of Test			2023-09-18
Height of Sample	cm	H_f	2.412
Water Content	%	w_f	39.49
Void Ratio	-	e_f	1.005

TRIMMING: the specimen was manually trimmed to the size of the consolidation ring, then mounted in a fixed ring consolidometer

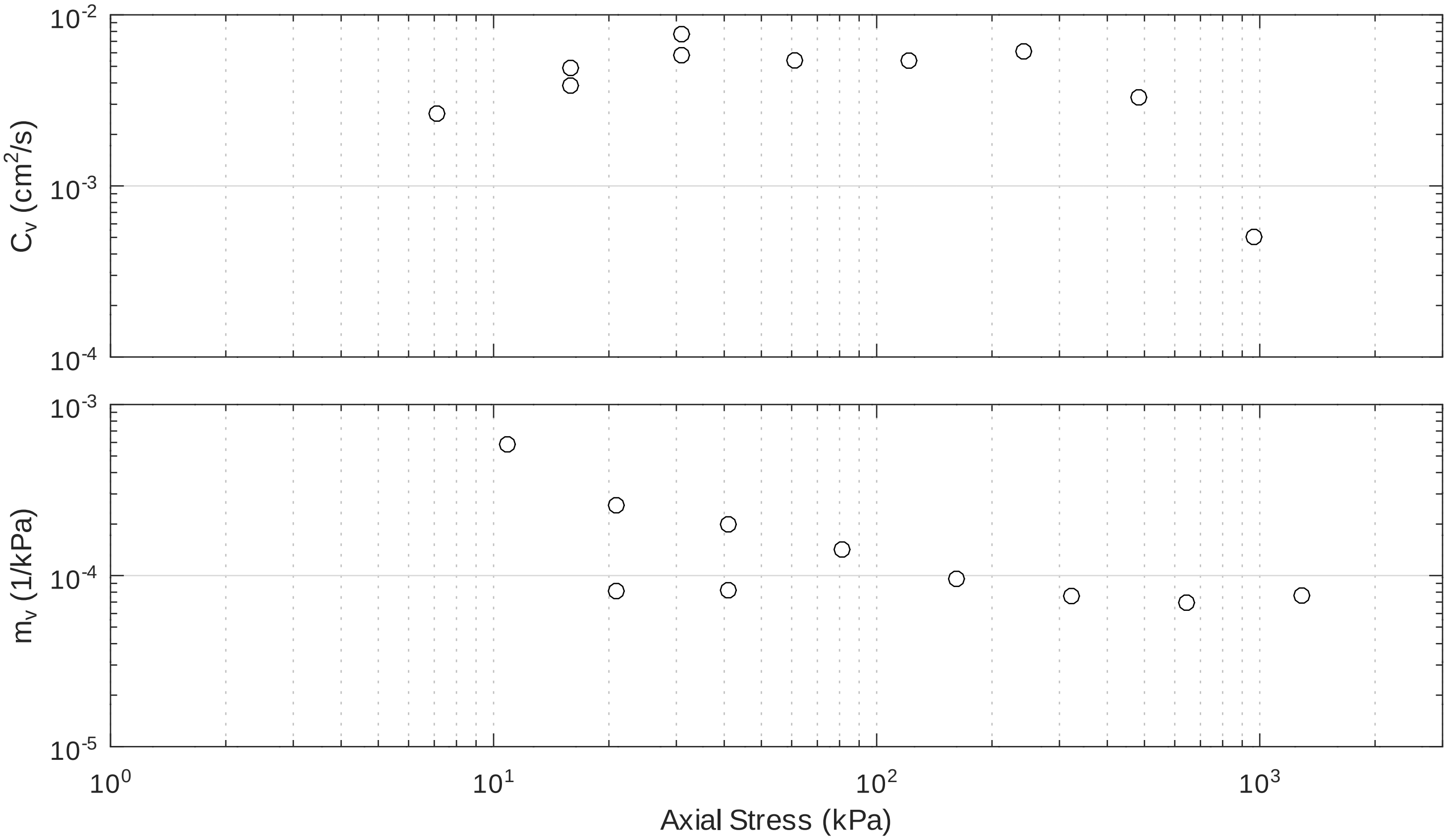
LOADING: the consolidometer was flooded with water with the seating load adjusted to limit swelling

CALCULATIONS: coefficients of consolidation were calculated by the square root time method, secondary consolidation was calculated based on the available duration of the time step

Interpreted Results			
Recompression Index (reloading)	-	C_r	0.047
Compression Index	-	C_c	0.344
Recompression Index (unloading)	-	C_r	0.056
Probable Preconsolidation Pressure	kPa	p'_c	475

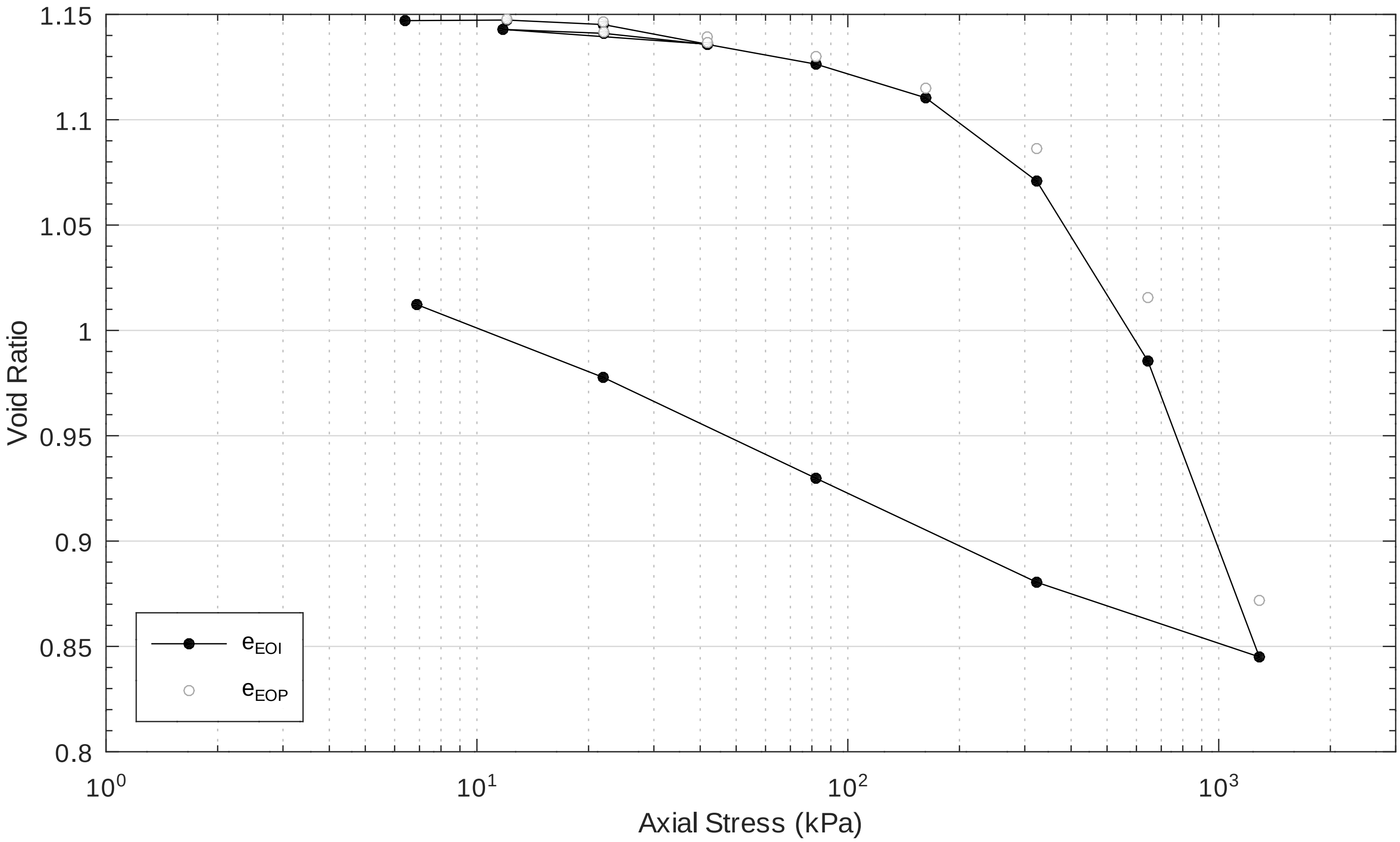
Check: AO Review: ___________

Project: 33443
Hwy 11 Poplar Rapids Bridge
Borehole: 23-104
Sample: ST1
Depth: 9.1m
Client: LEA/MTO



Load No.	Axial Stress	Load Duration	System Deflec.	Dial	Sample Height	Axial Strain	Void Ratio	Void Ratio	Time U(0.99)	C_v	k_v	$C_{a\varepsilon}$
	kPa	min	mm	mm	cm	%	(EOI)	(EOP)	min	cm^2/s	cm/s	-
0				10.000	2.534	0.00	1.106					
1	3.4	1440.3	0.021	10.005	2.537	-0.10	1.108					
2	10.9	1440.3	0.041	9.874	2.525	0.34	1.099	1.103	17.8	2.65e-03	1.52e-07	0.0012
3	20.9	1440.1	0.076	9.772	2.519	0.59	1.094	1.096	9.5	4.89e-03	1.24e-07	0.0005
4	41.0	1440.5	0.122	9.625	2.509	1.00	1.085	1.089	6.0	7.71e-03	1.51e-07	0.0007
5	10.9	1440.4	0.088	9.720	2.515	0.75	1.090					
6	20.9	1440.1	0.102	9.686	2.513	0.84	1.089	1.089	12.1	3.86e-03	3.07e-08	0.0000
7	41.0	1440.2	0.128	9.618	2.509	1.00	1.085	1.087	8.0	5.81e-03	4.68e-08	0.0004
8	81.2	1440.1	0.186	9.416	2.494	1.57	1.073	1.078	8.3	5.41e-03	7.55e-08	0.0009
9	161.6	1440.2	0.266	9.141	2.475	2.34	1.057	1.062	8.1	5.40e-03	5.07e-08	0.0012
10	322.5	1440.3	0.362	8.735	2.444	3.56	1.031	1.041	6.8	6.11e-03	4.55e-08	0.0018
11	644.1	1440.4	0.474	8.056	2.387	5.80	0.984	1.004	11.7	3.29e-03	2.24e-08	0.0045
12	1287.4	1440.5	0.622	6.661	2.262	10.71	0.880	0.909	54.9	5.03e-04	3.77e-09	0.0100
13	322.4	1440.4	0.470	7.215	2.303	9.13	0.914					
14	81.2	1440.4	0.367	7.779	2.349	7.31	0.952					
15	20.9	1440.0	0.298	8.191	2.383	5.95	0.981					
16	5.9	2880.0	0.244	8.536	2.412	4.81	1.005					

Project:	33443
	Hwy 11 Poplar Rapids Bridge
Borehole:	23-105
Sample:	ST1
Depth:	
Client:	LEA/MTO



Start of Test			2023-08-04
Diameter of Sample	cm	D	6.338
Height of Sample	cm	H_o	2.530
Height of Solids	cm	H_s	1.179
Water Content	%	w_o	41.42
Dry Density	g/cm^3	ρ_d	1.28
Moist Unit Weight	kN/m^3	γ	17.7
Void Ratio	-	e_o	1.146
Degree of Saturation	-	S_{ro}	0.99
Specific Gravity	-	G_s	2.740

End of Test			2023-08-21
Height of Sample	cm	H_f	2.373
Water Content	%	w_f	38.03
Void Ratio	-	e_f	1.012

TRIMMING: the specimen was manually trimmed to the size of the consolidation ring, then mounted in a fixed ring consolidometer

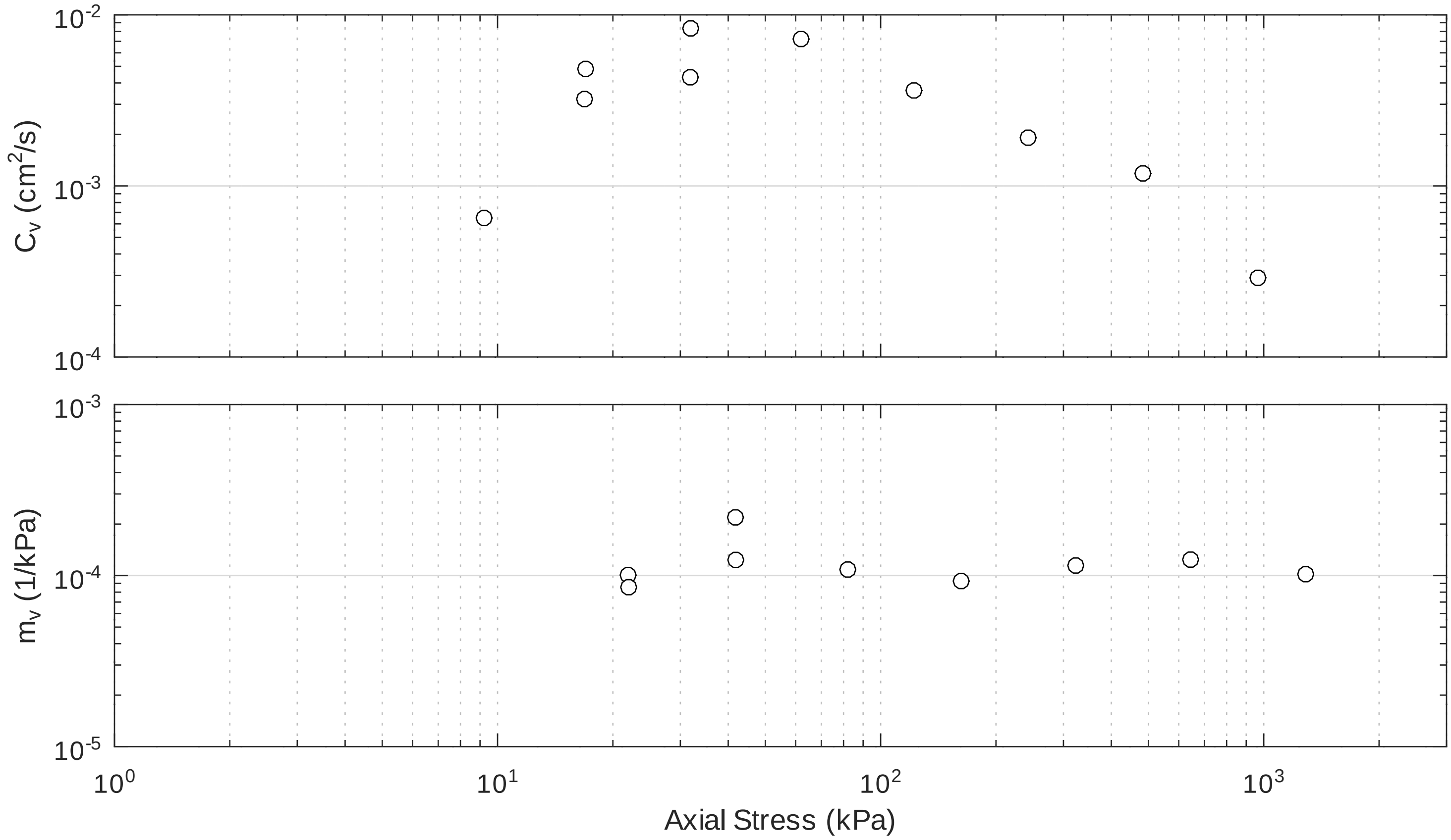
LOADING: the consolidometer was flooded with water with the seating load adjusted to limit swelling

CALCULATIONS: coefficients of consolidation were calculated by the square root time method, secondary consolidation was calculated based on the available duration of the time step

Interpreted Results			
Recompression Index (reloading)	-	C_r	0.043
Compression Index	-	C_c	0.467
Recompression Index (unloading)	-	C_r	0.083
Probable Preconsolidation Pressure	kPa	p'_c	404

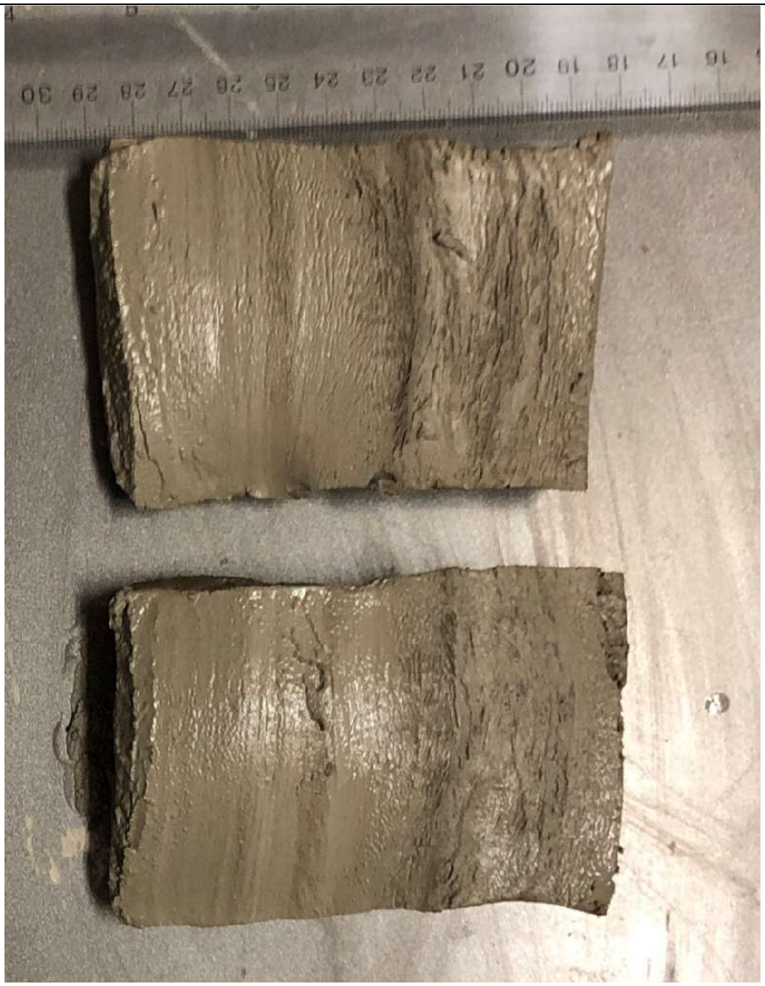
Check: AO Review: __________

Project: 33443
Hwy 11 Poplar Rapids Bridge
Borehole: 23-105
Sample: ST1
Depth:
Client: LEA/MTO



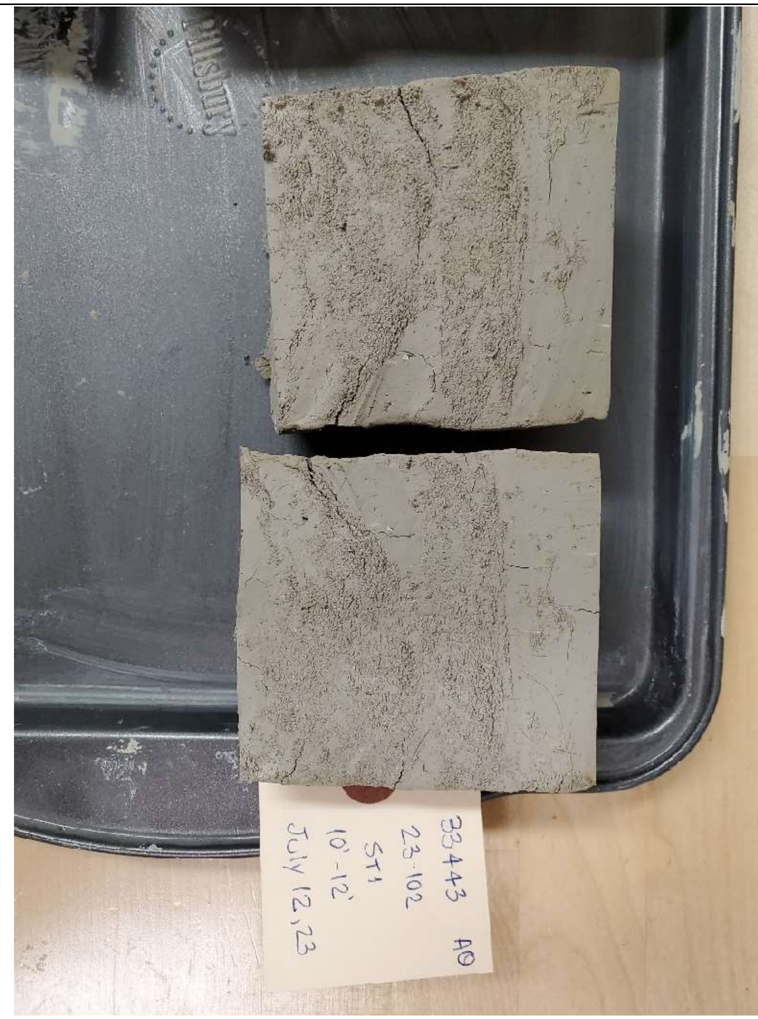
Load No.	Axial Stress	Load Duration	System Deflec.	Dial	Sample Height	Axial Strain	Void Ratio	Void Ratio	Time U(0.99)	C_v	k_v	$C_{a\varepsilon}$
	kPa	min	mm	mm	cm	%	(EOI)	(EOP)	min	cm^2/s	cm/s	-
0				10.000	2.530	0.00	1.146					
1	6.4	1440.2	0.008	10.010	2.532	-0.07	1.147					
2	12.0	1440.2	0.041	9.980	2.532	-0.08	1.147	1.148	72.9	6.50e-04	-1.42e-09	0.0002
3	21.9	1440.1	0.102	9.894	2.530	0.02	1.145	1.146	9.7	4.83e-03	4.77e-08	0.0003
4	41.8	1440.2	0.141	9.744	2.519	0.45	1.136	1.139	10.7	4.32e-03	9.26e-08	0.0007
5	11.7	1440.5	0.115	9.852	2.527	0.12	1.143					
6	22.0	1440.1	0.122	9.825	2.525	0.21	1.141	1.142	14.7	3.22e-03	2.70e-08	0.0001
7	41.9	1440.1	0.141	9.744	2.518	0.46	1.136	1.137	5.6	8.33e-03	1.01e-07	0.0002
8	82.1	1440.2	0.213	9.561	2.507	0.89	1.126	1.130	6.2	7.23e-03	7.69e-08	0.0006
9	162.2	1440.2	0.295	9.291	2.489	1.64	1.110	1.115	12.3	3.61e-03	3.29e-08	0.0010
10	323.0	1440.4	0.382	8.737	2.442	3.48	1.071	1.086	21.6	1.91e-03	2.15e-08	0.0039
11	644.2	1440.2	0.484	7.628	2.341	7.46	0.985	1.016	28.2	1.18e-03	1.44e-08	0.0081
12	1286.7	1440.1	0.601	5.854	2.176	14.01	0.845	0.872	70.0	2.90e-04	2.90e-09	0.0091
13	323.0	1440.3	0.466	6.407	2.217	12.35	0.880					
14	82.0	1440.0	0.347	7.108	2.276	10.05	0.930					
15	21.9	1440.5	0.280	7.741	2.332	7.82	0.978					
16	6.9	2820.1	0.223	8.205	2.373	6.21	1.012					

THURBER ENGINEERING LTD.

<table>
<tr><td colspan="2">Borehole 23-101, Sample ST1, Depth 9.1m
(sample width approximately equal to diameter of Thin-Walled sample tube, ~70 mm)</td></tr>
<tr><td>“Wet”</td><td>“Dry”</td></tr>
<tr><td></td><td></td></tr>
</table>

Borehole 23-102, Sample ST1, Depth 3.0m

(sample width approximately equal to diameter of Thin-Walled sample tube, ~70 mm)

"Wet"	"Dry"
	

Client: LEA Consulting Ltd.

File No.: 33443

April 5, 2024

THURBER ENGINEERING LTD.

Borehole 23-102, Sample ST5, Depth 5.3m *(sample width approximately equal to diameter of Thin-Walled sample tube, ~70 mm)*	
"Wet"	"Dry"
	

Client: LEA Consulting Ltd.
File No.: 33443

April 5, 2024

Borehole 23-104, Sample ST1, Depth 9.1m *(sample width approximately equal to diameter of Thin-Walled sample tube, ~70 mm)*	
"Wet"	"Dry"
	N/A Sample was used for additonal testing.

Client: LEA Consulting Ltd.
File No.: 33443

April 5, 2024

Stantec

Stantec Consulting Ltd.
2781 Lancaster Rd, Suite 100 A&B, Ottawa ON K1B 1A7

August 8, 2023
File: 122410864

Client: Thurber Engineering, File #33443.10

Reference: ASTM D7012, Method C, Unconfined Compressive Strength of Intact Rock Core
Hwy 11 Poplar River Bridge

The following table summarizes unconfined compressive strength results for four intact rock cores.

Location	Sample Depth	Compressive Strength (MPa)	Description of Break
23-101 Run-1	82'5''-83'	203.8	Well-formed cones at both ends.
23-101 Run-3	88'4''-89'	155.2	Diagonal fracture through end
23-102 Run-3	76'5''-77 '2''	208.5	Diagonal fracture through end
23-102 Run-2	82'8''-83'5''	92.2	Diagonal fracture

Run 1

Sincerely,

Stantec Consulting Ltd.

Brian Prevost

Brian Prevost
Laboratory Supervisor
Tel: 613-738-6075
Fax: 613-722-2799
brian.prevost@stantec.com

Borehole 23-101

Runs 1 to 5
Depth 24.6 m to 28.1 m
Elevation 204.7 m to 201.2 m
Dry Sample

NQ 1 and 2 Cobbles and boulders

Run 1 Start
elev. 204.7 m

Run 1 End
elev. 203.6 m

Run 2 Start
elev. 203.6 m

Run 2 End
elev. 202.7 m

Run 3 Start
elev. 202.7 m

Run 3 End
elev. 202.2 m

Run 4 Start
elev. 202.2 m

Run 4 End
elev. 201.7 m

Run 5 Start
elev. 201.7 m

Run 5 End
elev. 201.2 m

THURBER ENGINEERING LTD.

Geotechnical Investigation
Highway 11
Poplar Rapids Bridge

BH 23-101
Project No.: 33443

Borehole 23-101

Runs 1 to 5
Depth 24.6 m to 28.1 m
Elevation 204.7 m to 201.2 m
Wet Sample

NQ 1 and 2 Cobbles and boulders

Run 1 Start
elev. 204.7 m

Run 1 End
elev. 203.6 m

Run 2 Start
elev. 203.6 m

Run 2 End
elev. 202.7 m

Run 3 Start
elev. 202.7 m

Run 3 End
elev. 202.2 m

Run 4 Start
elev. 202.2 m

Run 4 End
elev. 201.7 m

Run 5 Start
elev. 201.7 m

Run 5 End
elev. 201.2 m

THURBER ENGINEERING LTD.	Geotechnical Investigation Highway 11 Poplar Rapids Bridge	BH 23-101 Project No.: 33443

Borehole 23-102

Runs 1 to 3
Depth 22.7 m to 26.0 m
Elevation 204.3 m to 201.0 m
Dry Sample

NQ 1 Cobbles and boulders

Run 1 Start
elev. 204.3 m

Run 1 End
elev. 202.8 m

Run 2 Start
elev. 202.8 m

Run 2 End
elev. 201.4 m

Run 3 Start
elev. 201.4 m

Run 3 End
elev. 201.0 m

THURBER ENGINEERING LTD.

Geotechnical Investigation
Highway 11
Poplar Rapids Bridge

BH 23-102
Project No.: 33443

Borehole 23-102

Runs 1 to 3
Depth 22.7 m to 26.0 m
Elevation 204.3 m to 201.0 m
Wet Sample

NQ 1 Cobbles and boulders

Run 1 Start
elev. 204.3 m

Run 1 End
elev. 202.8 m

Run 2 Start
elev. 202.8 m

Run 2 End
elev. 201.4 m

Run 3 Start
elev. 201.4 m

Run 3 End
elev. 201.0 m

THURBER ENGINEERING LTD.

Geotechnical Investigation
Highway 11
Poplar Rapids Bridge

BH 23-102
Project No.: 33443

Borehole 23-105

NQ 1

Dry Sample

NQ 1, Cobbles and boulders

THURBER ENGINEERING LTD.

Geotechnical Investigation
Highway 11
Poplar Rapids Bridge

BH 23-105
Project No.: 33443

Borehole 23-105

NQ 1

Wet Sample

NQ 1, Cobbles and boulders

THURBER ENGINEERING LTD.

Geotechnical Investigation
Highway 11
Poplar Rapids Bridge

BH 23-105
Project No.: 33443

Certificate of Analysis

Client: Thurber Engineering Ltd.

Client PO:

Report Date: 04-Aug-2023

Order Date: 26-Jul-2023

Project Description: 33443 - Hwy 11 Poplar Rapids Bridge

Client ID:		23-101 SS7 (20'-22')	23-102 ST1 (10'-12')	23-104 SS9 (20'-22')	23-105 SS7 (15'-17')
Sample Date:		05-Jul-23 09:00	12-Jul-23 09:00	07-Jul-23 09:00	12-Jul-23 09:00
Sample ID:		2330225-01	2330225-02	2330225-03	2330225-04
	MDL/Units	Soil	Soil	Soil	Soil
Physical Characteristics					
% Solids	0.1 % by Wt.	86.8	82.3	82.7	74.6
General Inorganics					
Conductivity	5 uS/cm	955	222	213	337
pH	0.05 pH Units	7.58	7.36	7.53	7.77
Resistivity	0.1 Ohm.m	10.5	45.1	46.9	29.6
Anions					
Chloride	10 ug/g dry	506	18	10	112
Sulphate	10 ug/g dry	17	35	39	24

SGS Canada Inc.
P.O. Box 4300 - 185 Concession St.
Lakefield - Ontario - KOL 2HO
Phone: 705-652-2000 FAX: 705-652-6365

23-August-2023

Paracel Laboratories
Attn : Dale Robertson

300-2319 St.Laurent Blvd.
Ottawa, ON
K1G 4K6, Canada

Phone: 613-731-9577
Fax:613-731-9064

Date Rec. : 27 July 2023
LR Report: **CA15810-JUL23**
Reference: Project#: 2330225

Copy: #1

CERTIFICATE OF ANALYSIS

Final Report

Sample ID	Sample Date & Time	Sulphide (Na2CO3) %
1: Analysis Start Date		21-Aug-23
2: Analysis Start Time		10:50
3: Analysis Completed Date		23-Aug-23
4: Analysis Completed Time		08:40
5: QC - Blank		< 0.04
6: QC - STD % Recovery		114%
7: QC - DUP % RPD		70%
8: RL		0.0
9: 23-101 SS7 (20'-22')	05-Jul-23	< 0.04
10: 23-102 ST1 (10'-12')	12-Jul-23	< 0.04
11: 23-104 SS9 (20'-22')	07-Jul-23	0.04
12: 23-105 SS7 (15'-17')	10-Jul-23	< 0.04

RL - SGS Reporting Limit

Kimberley Didsbury,
Project Specialist,
Environment, Health & Safety

OnLine LIMS

0003442010

APPENDIX D

Site Photograph

Photo 1: Looking east at Poplar Rapids Bridge *(taken on July 15, 2023).* Rail bridge located to the right (south) of the highway alignment.

Photo 2: Looking southeast at Poplar Rapids Bridge *(taken on July 15, 2023).*

Photo 3: Looking east at the northwest embankment slope *(taken on July 15, 2023).*

Photo 4: Looking west at Poplar Rapids Bridge *(taken on July 12, 2023).* Rail bridge located to the left (south) of the highway alignment.

Photo 5: Looking southwest at Poplar Rapids Bridge *(taken on July 15, 2023).* Overhead utility lines to right (north) of ditch.

Photo 6: Looking west at the northeast embankment slope *(taken on July 15, 2023).* Overhead utility lines to right (north) of ditch. Photograph taken from location of entrance access with CSP culvert.

Photo 7: Looking east at Poplar Rapids Bridge and CNR railway *(taken on June 27, 2023).*

Photo 8: Looking northeast at Poplar Rapids Bridge piers and abutments *(taken on June 27, 2023).*

APPENDIX E

ConeTec Report

Client: LEA Consulting Ltd.
File No.: 33443

April 5, 2024

PRESENTATION OF SITE INVESTIGATION RESULTS

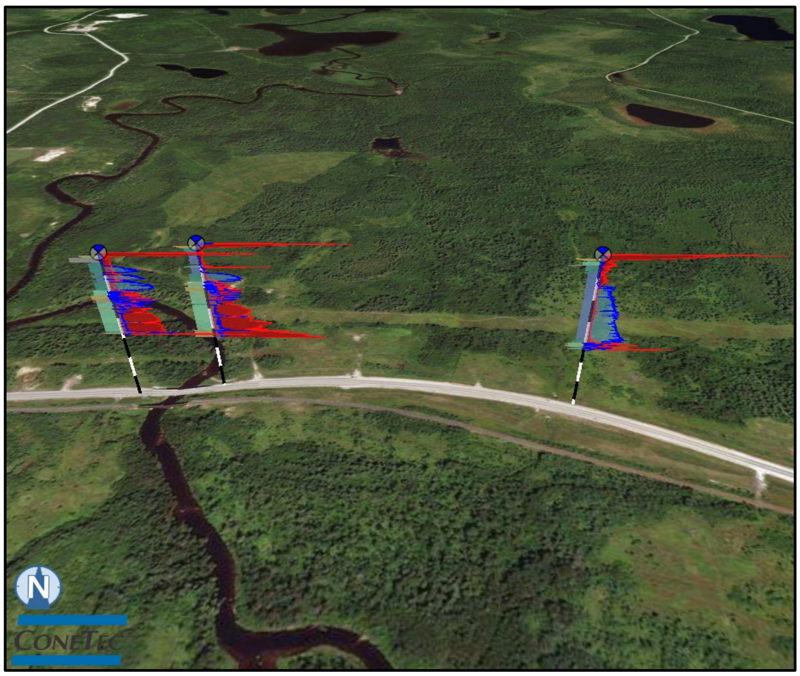
Poplar Rapids Bridge – Highway 11

Prepared for:

Thurber Engineering Ltd.

ConeTec Job No: 23-05-26042

Project Start Date: 2023-07-10
Project End Date: 2023-07-11
Report Date: 2023-07-21



Prepared by:

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

Tel: (905) 886-2663
Fax: (905) 886-2664

ConeTecON@conetec.com
www.conetec.com
www.conetecdataservices.com

Introduction

The enclosed report presents the results of the site investigation program conducted by ConeTec Investigations Ltd. for Thurber Engineering Ltd. at Highway 11 Poplar Rapids River Bridge, ON. The program consisted of 3 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	Thurber Engineering Ltd.
Project	Poplar Rapids Bridge - Highway 11
ConeTec project number	23-05-26042

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

Rig Description	Deployment System	Test Type
CPT truck rig (C-3)	30 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	765	15	225	1500	15	35
958:T1500F15U35	958	15	225	1500	15	35
The CPTu summary indicates which cone was used for each sounding.						

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.
Seismic calculations	Poisson's ratio (ν) was calculated from the shear wave (V_s) and compression wave (V_p) velocities using the following equation: $$\nu = \frac{(V_p/V_s)^2 - 2}{2\left((V_p/V_s)^2 - 1\right)}$$
Additional plots	• Standard plots with expanded range • Advanced plots with Ic, Su, phi and N1(60) • Seismic plots with Vs, Vp, and Poisson's ratio • 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). Effective stresses are calculated based on unit weights that have been assigned to the individual soil behaviour type zones and the assumed equilibrium pore pressure profile. 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 “Poplar Rapids Bridge - Highway 11”, referred to as the (“Report”), was prepared by ConeTec for Thurber Engineering Ltd.. 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 Thurber Engineering Ltd. to collect and provide the raw data (“Data”) which is included in this report titled “Poplar Rapids Bridge - Highway 11”, 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 16 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 mm diameter over a length of 32 mm with tapered leading and trailing edges) located at a distance of 585 mm above the cone tip.

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

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

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

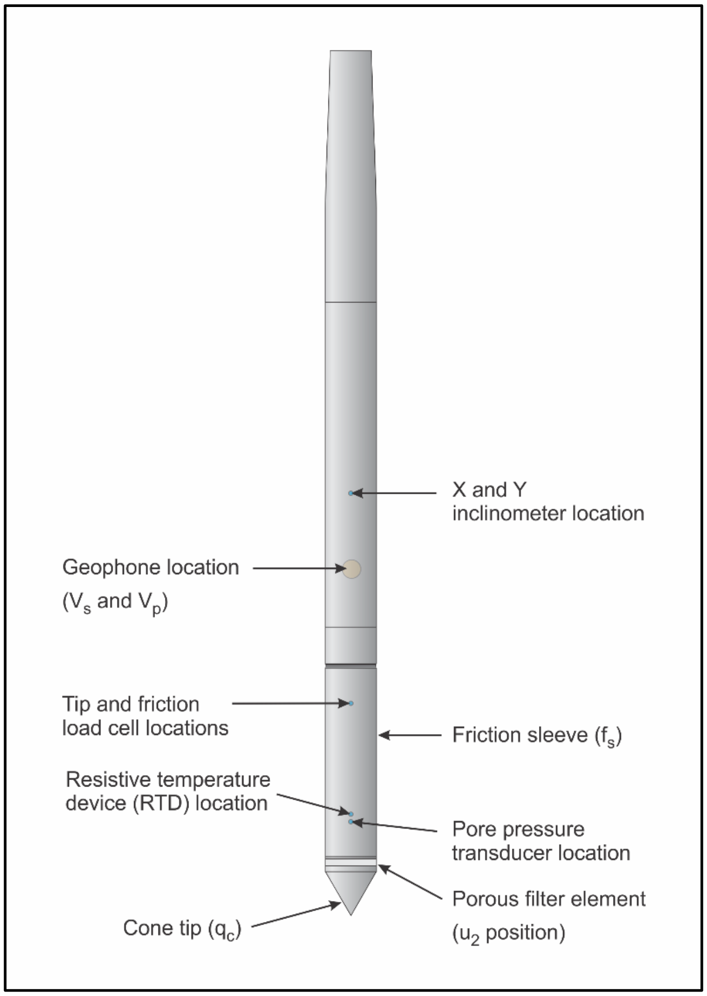


Figure CPTu. Piezocone Penetrometer (15 cm^2)

The ConeTec data acquisition systems consist of a Windows based computer and a signal 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 cm; custom recording intervals are possible.

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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 15 cm^2 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; however, it is often affected by the compression wave travelling through the cone rods.

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 maybe 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.

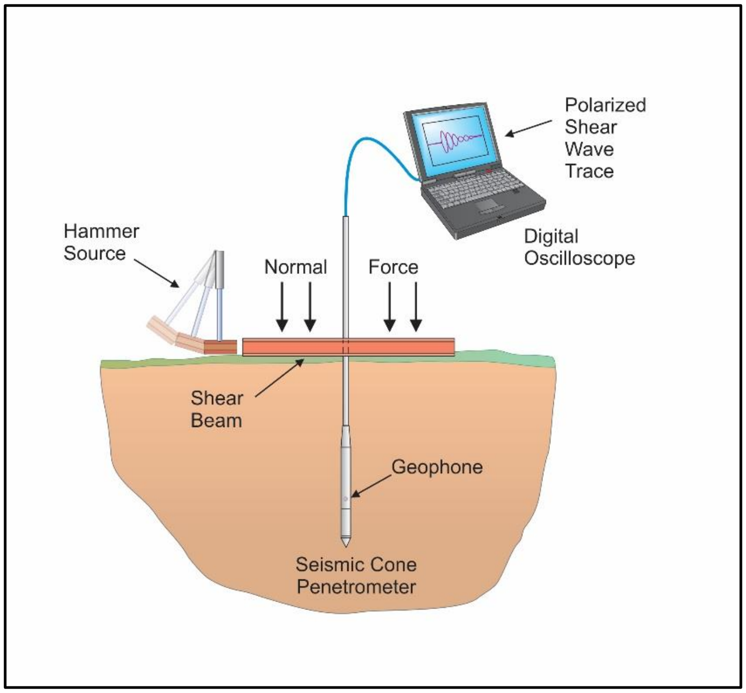


Figure SCPTu-1. Illustration of the SCPTu system

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).

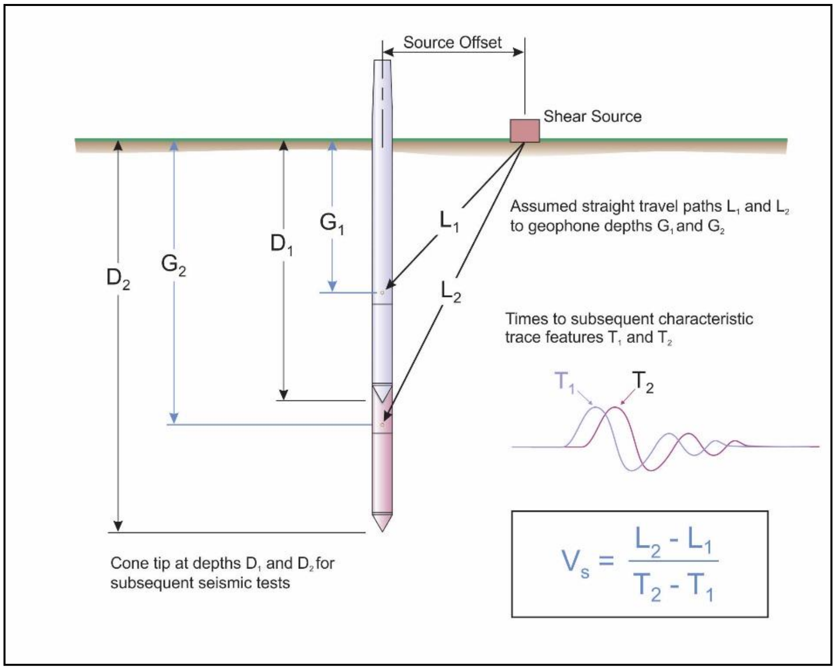


Figure SCPTu-2. Illustration of a seismic cone penetration test

For the determination of interval travel times the wave traces from all depths are displayed in analysis software. The results of the interval picks are supplied in the relevant appendix of this report. Standard practice for ConeTec is to record five wave traces for each source direction at each test depth. Outlier impacts are identified in the field and the impacts are repeated. For the final wave trace profile, the traces are stacked in the time domain to display a single average trace.

Determination of the shear wave interval velocities are performed by visually picking a common feature (e.g. the first characteristic peak, trough, or crossover) on all of the trace depths and taking the difference in ray path divided by the time difference between features at subsequent depths. The same process is used for compression waves, however the first break is most commonly used for selecting an arrival time. For velocity calculation, the 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.

In some cases, usually for shear wave velocity testing, more than one characteristic marker may be used. If there is an overlap between different sets of characteristic markers, then the average time value for those sets of interval times is applied to the determination of velocity.

Ideally, all depths are used for the determination of the velocity profile. However, an interval may be skipped if there is some ambiguity or quality concern with a particular depth, resulting in a larger interval.

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

The average shear wave velocity to a depth of 30 meters (V_{s30}) has been calculated and provided for all applicable soundings using an equation presented in Crow et al. (2012).

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

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

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).

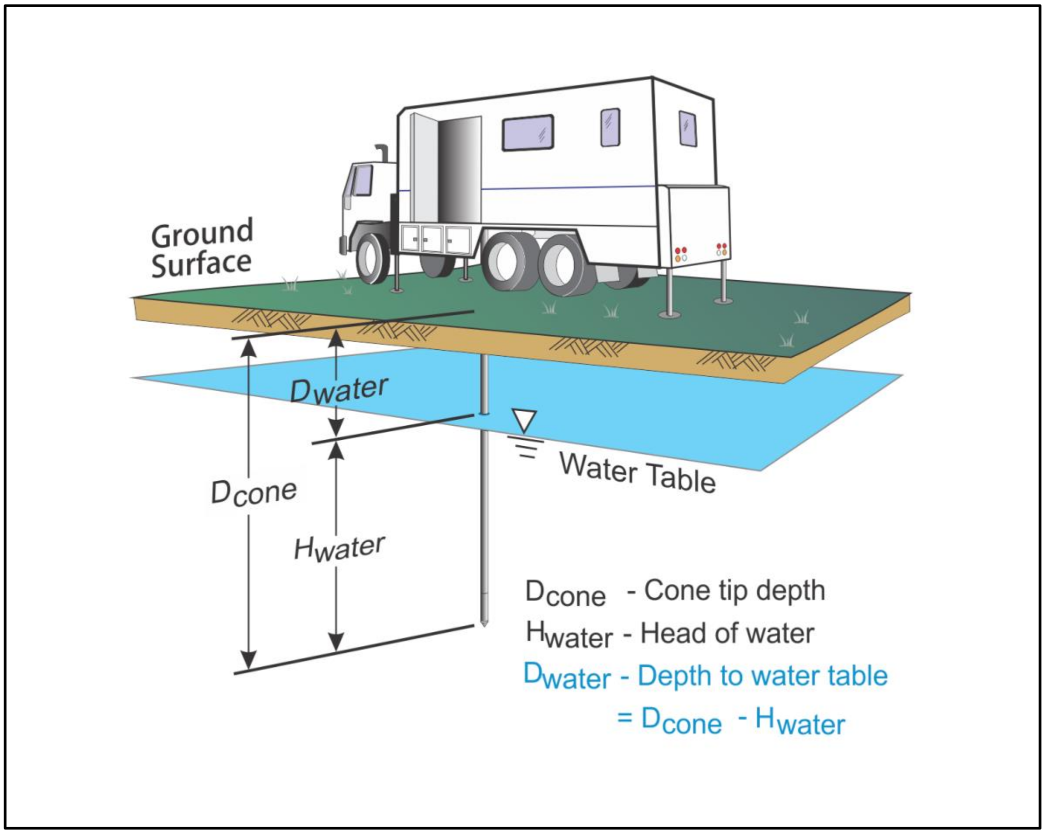


Figure PPD-1. Pore pressure dissipation test setup

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.

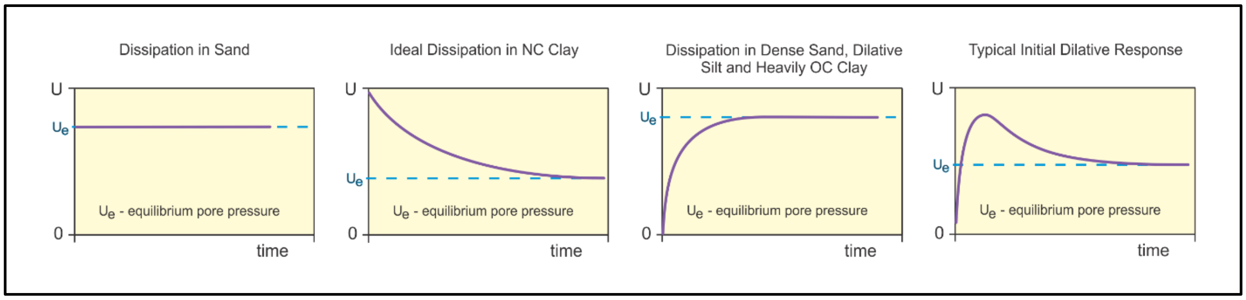


Figure PPD-2. Pore pressure dissipation curve examples

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.

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

ASTM D5778-20, 2020, "Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils", ASTM, West Conshohocken, US.

ASTM D7400/D7400M-19, 2019, "Standard Test Methods for Downhole Seismic Testing", ASTM, West Conshohocken, US.

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.

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.

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

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.

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

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

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

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.

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

The appendices listed below are included in the report:

- Cone Penetration Test Summary and Standard Cone Penetration Test Plots
- 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
- Seismic Cone Penetration Test Compression Wave (Vp) Tabular Results
- Seismic Cone Penetration Test Compression Wave (Vp) Traces
- Seismic Cone Penetration Test Poisson's Ratio Tabular Results
- 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:	23-05-26042
Client:	Thurber Engineering Ltd.
Project:	Poplar Rapids Bridge - Highway 11
Start Date:	2023-07-10
End Date:	2023-07-11

CONE PENETRATION TEST SUMMARY

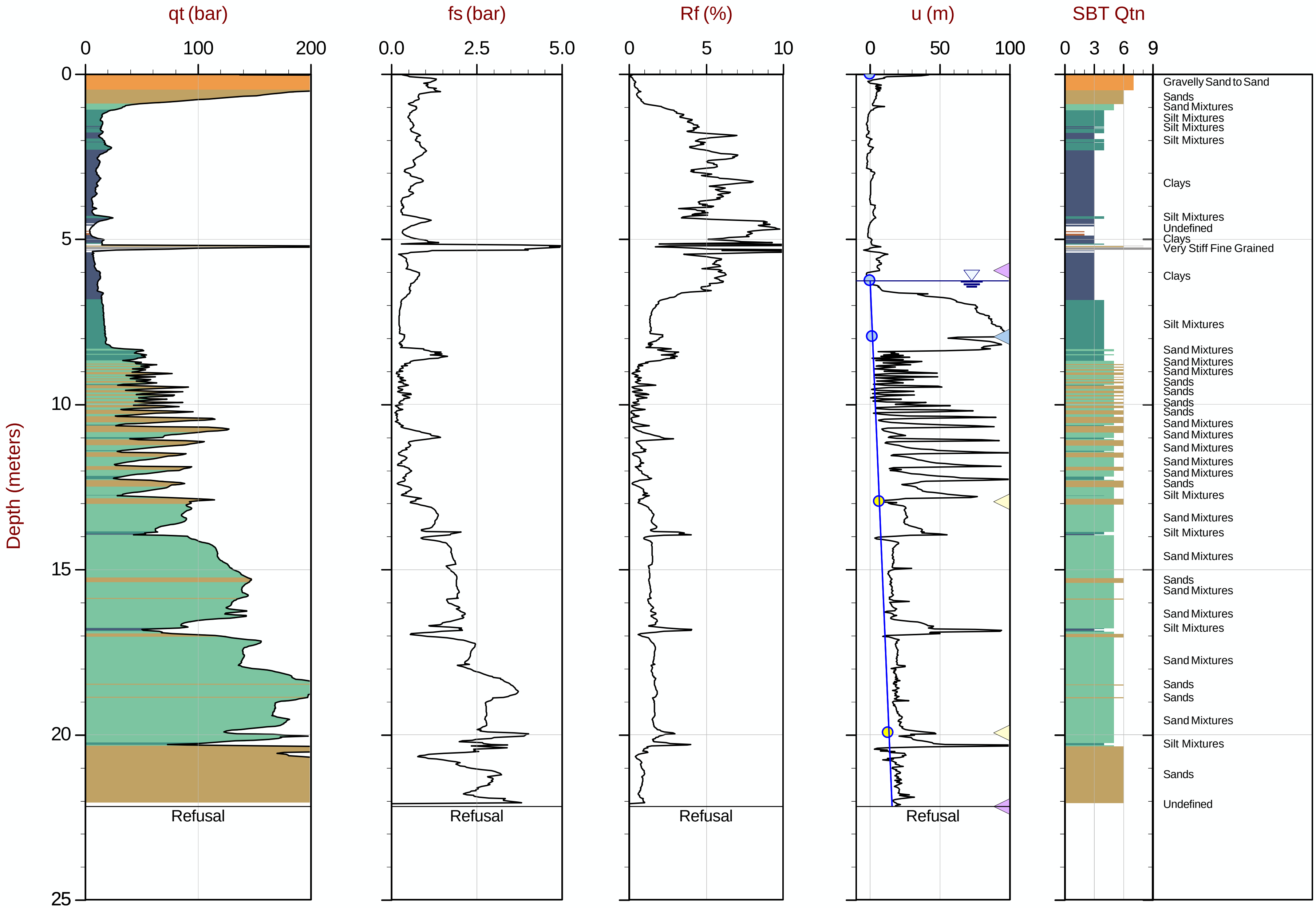
Sounding ID	File Name	Date	Cone	Cone Area (cm^2)	Assumed Phreatic Surface[1] (m)	Final Depth (m)	Northing[2] (m)	Easting[2] (m)	Refer to Notation Number
SCPT23-107	23-05-26042_SP23-107	2023-07-10	958:T1500F15U35	15	6.3	22.175	5459917	442998	
SCPT23-108	23-05-26042_SP23-108	2023-07-10	958:T1500F15U35	15	7.1	18.925	5459902	442905	
SCPT23-205	23-05-26042_SP23-205	2023-07-11	765:T1500F15U35	15	7.8	21.725	5459882	443394	

1. The assumed phreatic surface was based on pore pressure dissipation tests. Hydrostatic conditions were assumed for the calculated parameters.
2. Coordinates were acquired using consumer grade GPS equipment, Datum: NAD 83 / UTM Zone 17N.

Job No: 23-05-26042
Date: 2023-07-10 14:57
Site: Poplar Rapids Bridge, ON

Sounding: SCPT23-107
Cone: 958:T1500F15U35 Area= 15cm2

Thurber



Max Depth: 22.175 m / 72.75 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-26042_SP23-107.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 5459917m E: 442998m
Sheet No: 1 of 1

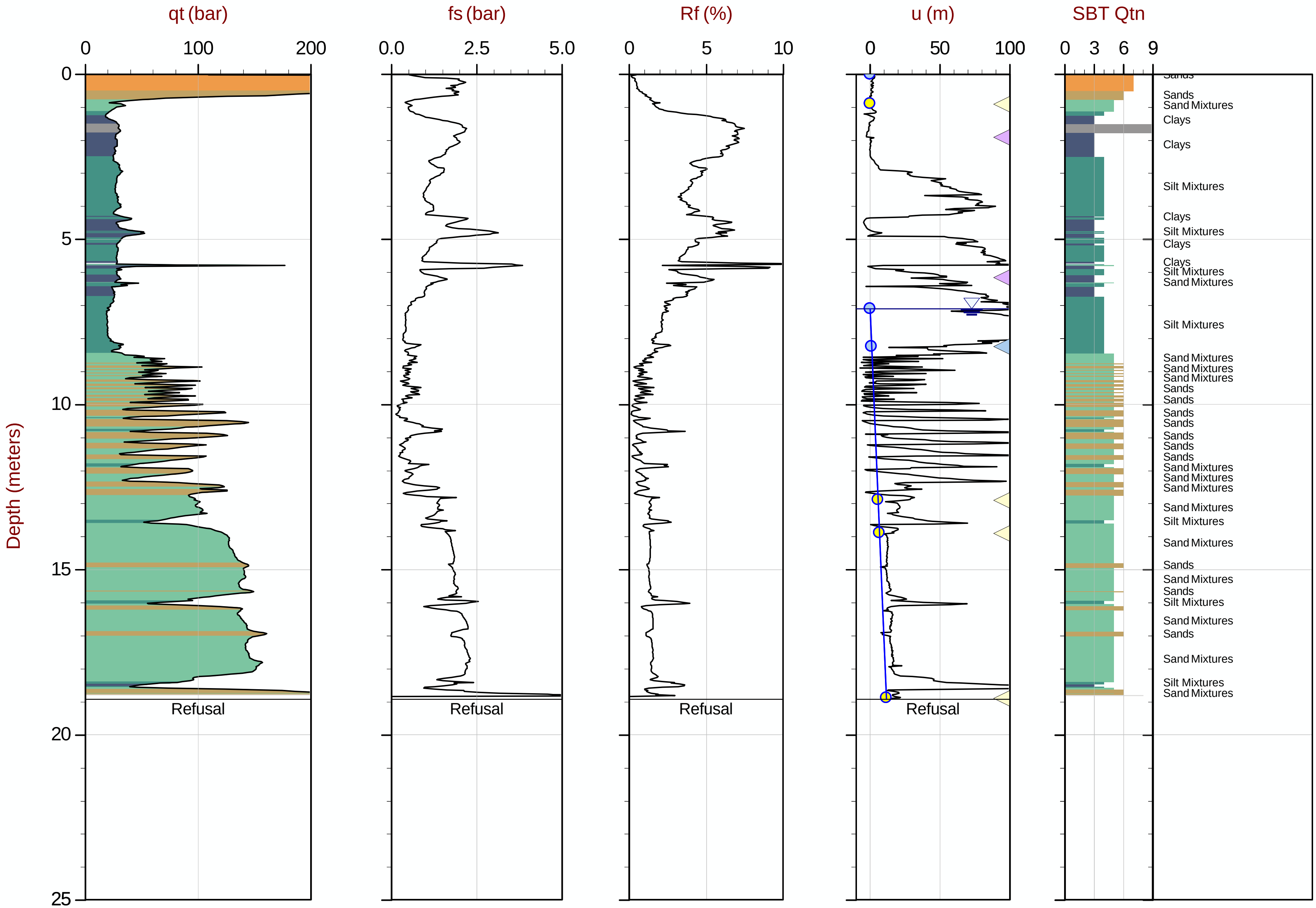
Overplot Item: Ueq, Assumed Ueq, Dissipation, Ueq achieved, Dissipation, Ueq not achieved, Dissipation, Ueq assumed, Hydrostatic Line

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

Thurber

Job No: 23-05-26042
Date: 2023-07-10 08:39
Site: Poplar Rapids Bridge, ON

Sounding: SCPT23-108
Cone: 958:T1500F15U35 Area= 15cm2



Max Depth: 18.925 m / 62.09 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-26042_SP23-108.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 5459902m E: 442905m
Sheet No: 1 of 1

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

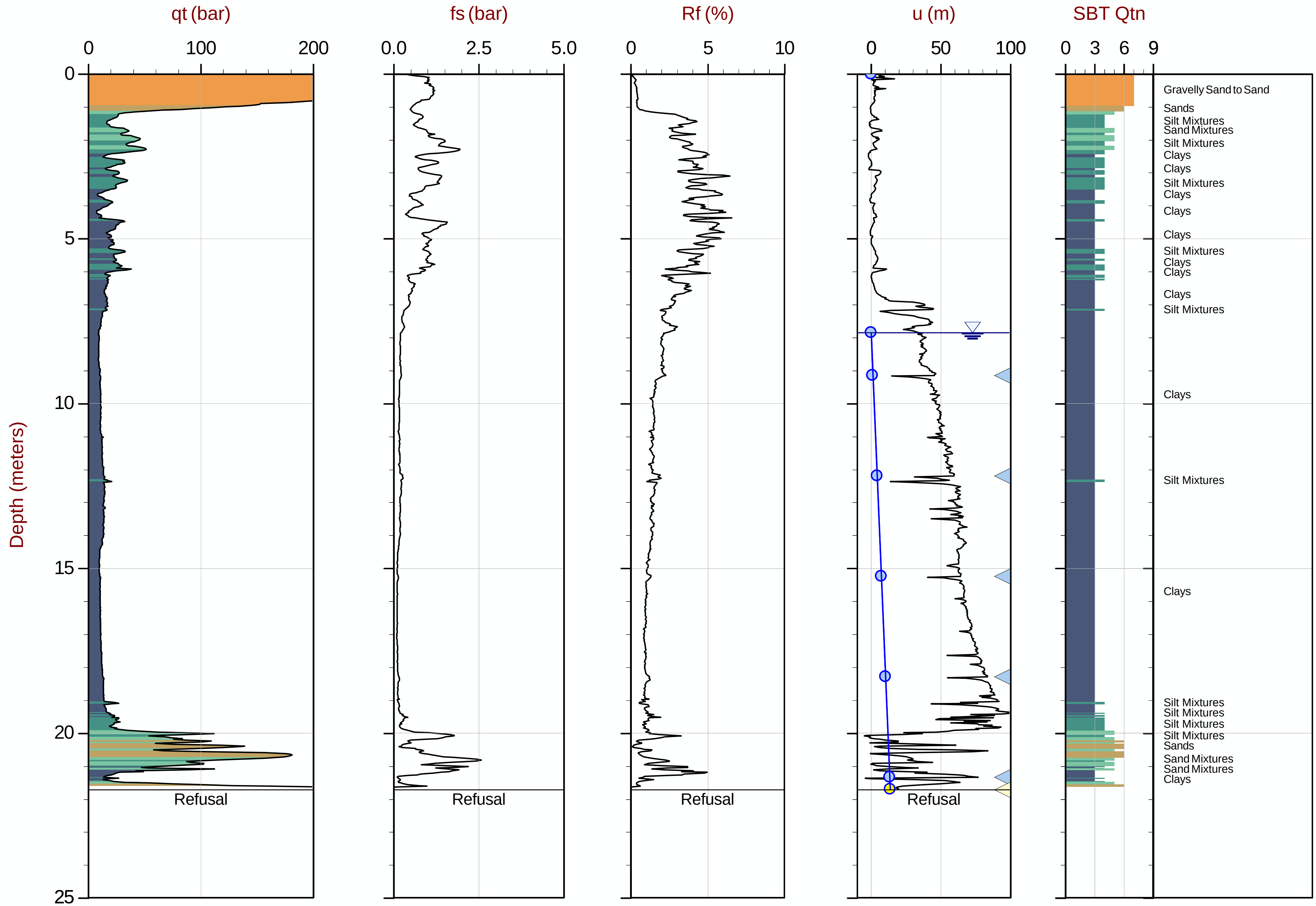
The reported coordinates were acquired from Hand Held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Thurber

Job No: 23-05-26042
Date: 2023-07-11 07:32
Site: Poplar Rapids Bridge, ON

Sounding: SCPT23-205
Cone: 765:T1500F15U35 Area= 15cm2



Max Depth: 21.725 m / 71.28 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-26042_SP23-205.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 5459882m E: 443394m
Sheet No: 1 of 1

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

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

Standard Cone Penetration Test Plots with Expanded Range

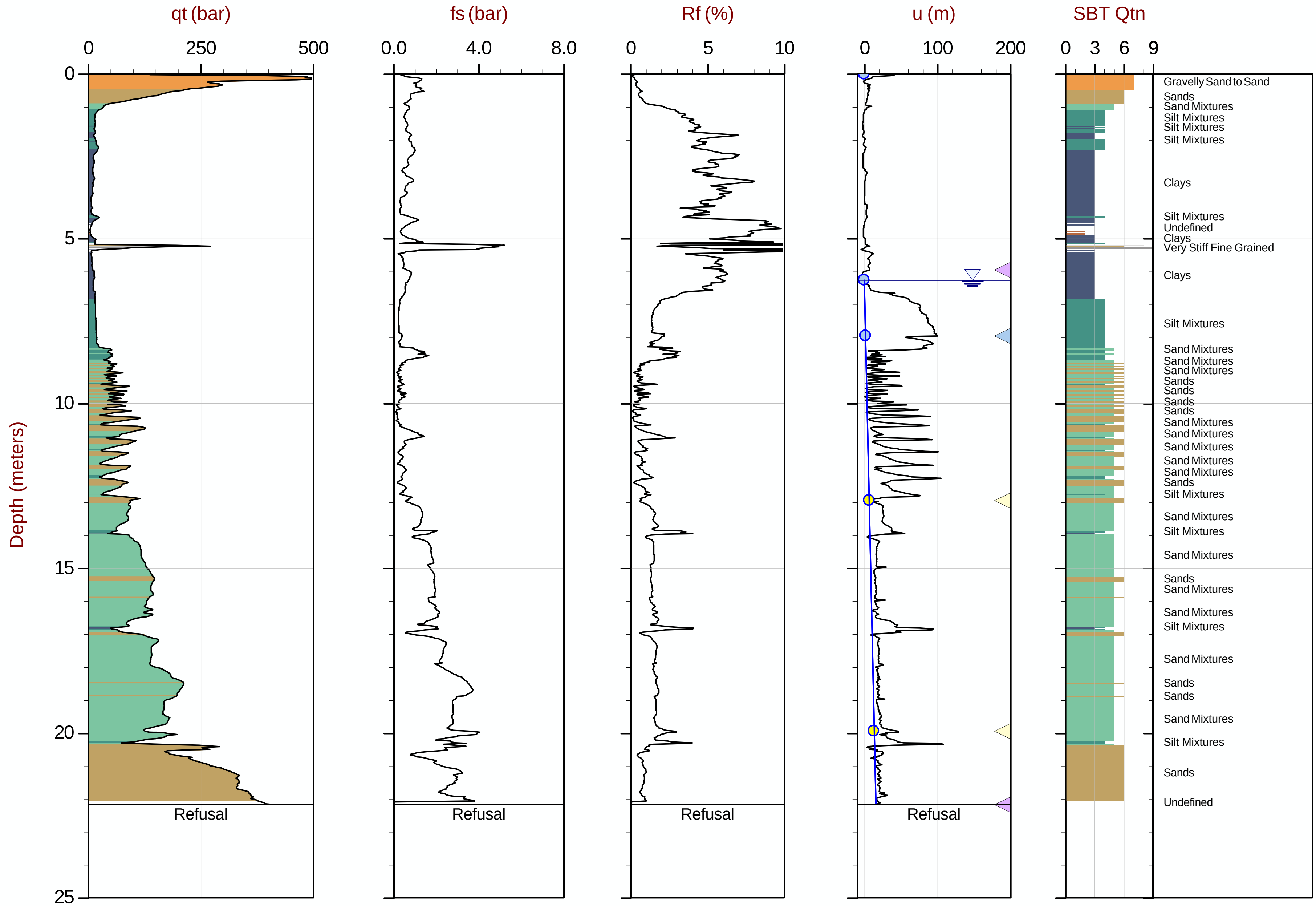
ConeTec

ConeTec

Thurber

Job No: 23-05-26042
Date: 2023-07-10 14:57
Site: Poplar Rapids Bridge, ON

Sounding: SCPT23-107
Cone: 958:T1500F15U35 Area= 15cm2



Max Depth: 22.175 m / 72.75 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-26042_SP23-107.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 5459917m E: 442998m
Sheet No: 1 of 1

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

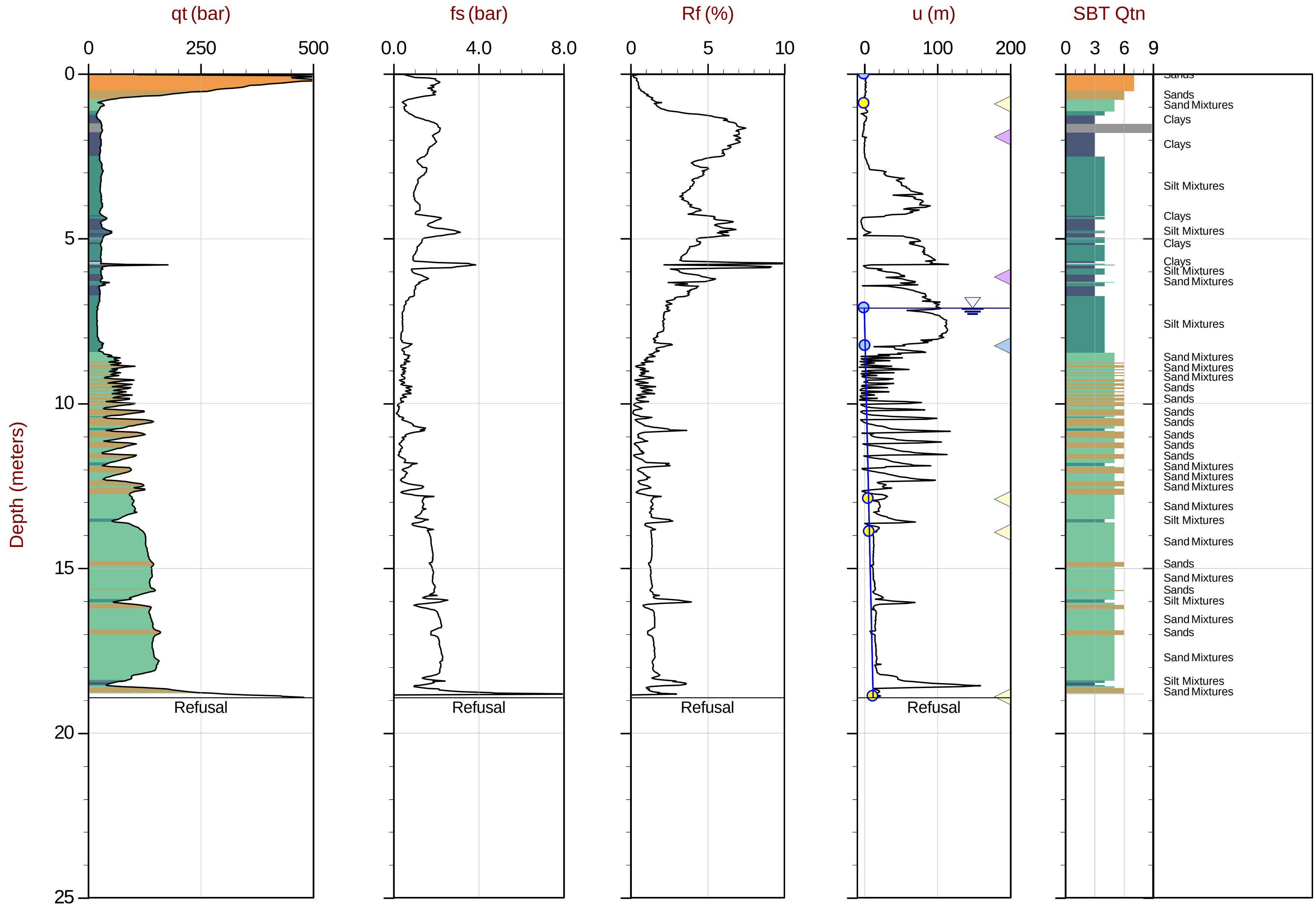
The reported coordinates were acquired from Hand Held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Thurber

Job No: 23-05-26042
Date: 2023-07-10 08:39
Site: Poplar Rapids Bridge, ON

Sounding: SCPT23-108
Cone: 958:T1500F15U35 Area= 15cm2



Max Depth: 18.925 m / 62.09 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-26042_SP23-108.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 5459902m E: 442905m
Sheet No: 1 of 1

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

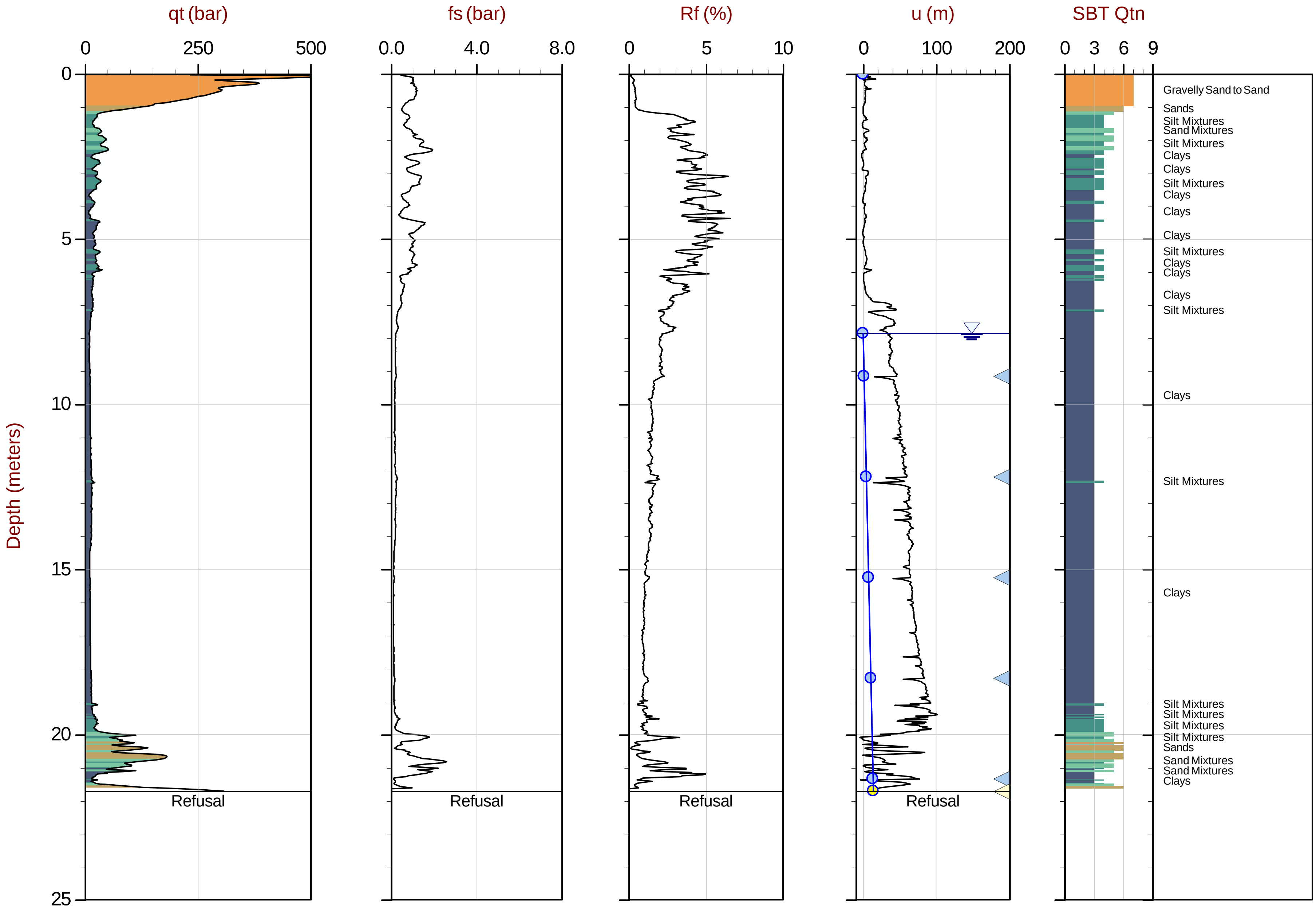
The reported coordinates were acquired from Hand Held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Thurber

Job No: 23-05-26042
Date: 2023-07-11 07:32
Site: Poplar Rapids Bridge, ON

Sounding: SCPT23-205
Cone: 765:T1500F15U35 Area= 15cm2



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

File: 23-05-26042_SP23-205.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 5459882m E: 443394m
Sheet No: 1 of 1

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

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

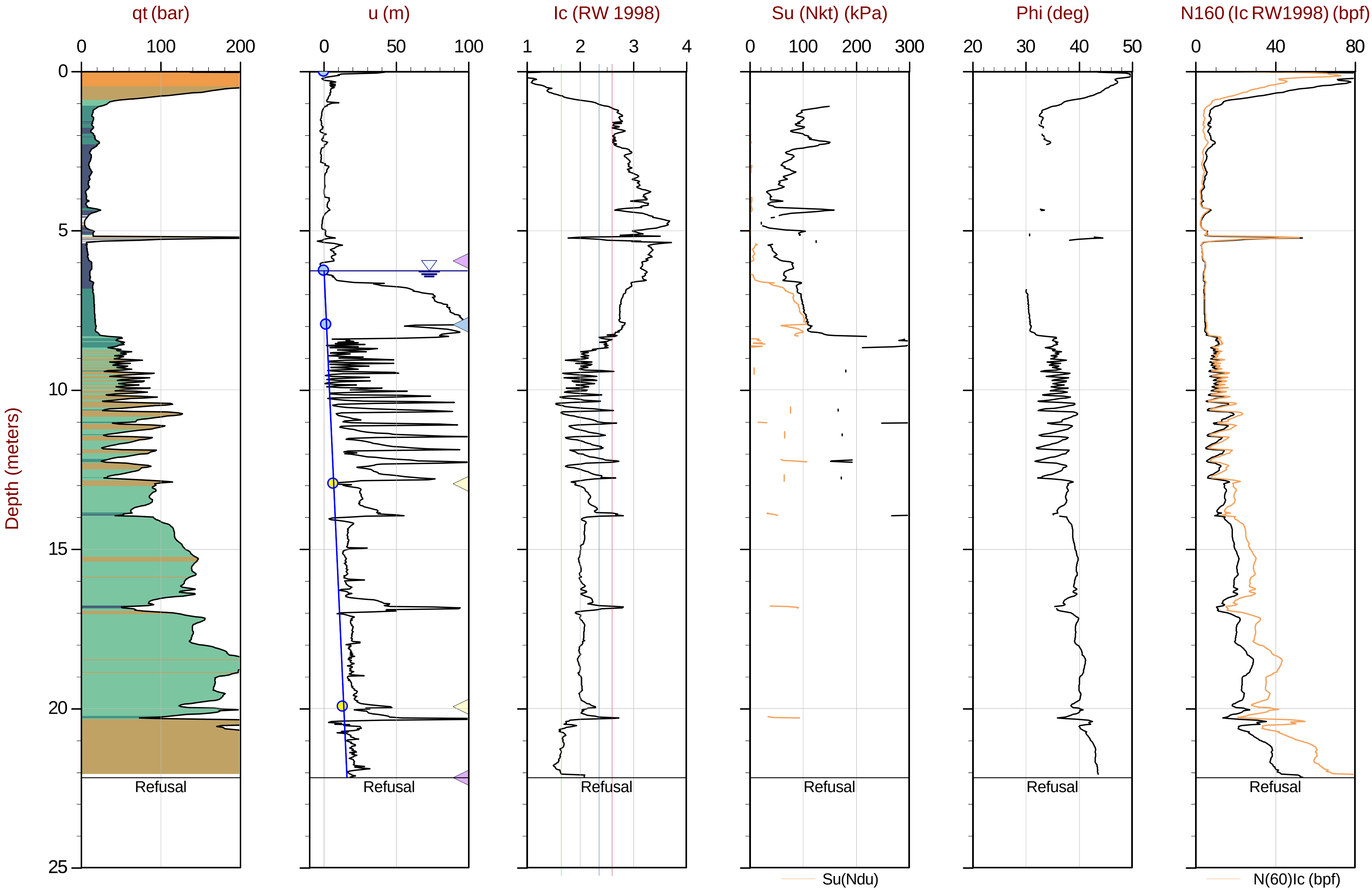
Advanced Cone Penetration Plots with Ic, Su(Nkt), Phi and N1(60)Ic

ConeTec

Thurber

Job No: 23-05-26042
Date: 2023-07-10 14:57
Site: Poplar Rapids Bridge, ON

Sounding: SCPT23-107
Cone: 958:T1500F15U35 Area= 15cm2



Max Depth: 22.175 m / 72.75 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-26042_SP23-107.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 9.0

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 5459917m E: 442998m
Sheet No: 1 of 1

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

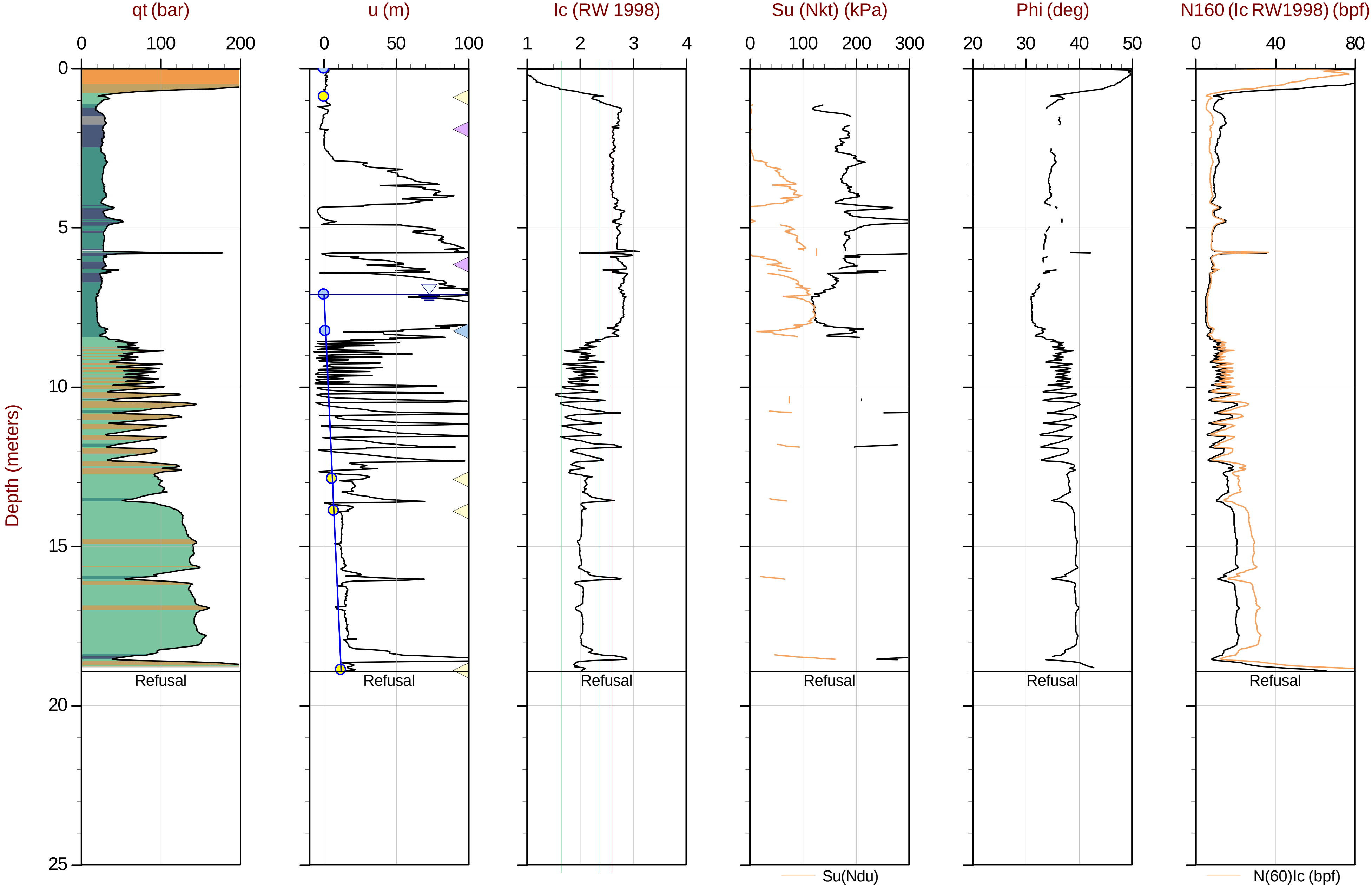
The reported coordinates were acquired from Hand Held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Thurber

Job No: 23-05-26042
Date: 2023-07-10 08:39
Site: Poplar Rapids Bridge, ON

Sounding: SCPT23-108
Cone: 958:T1500F15U35 Area= 15cm2



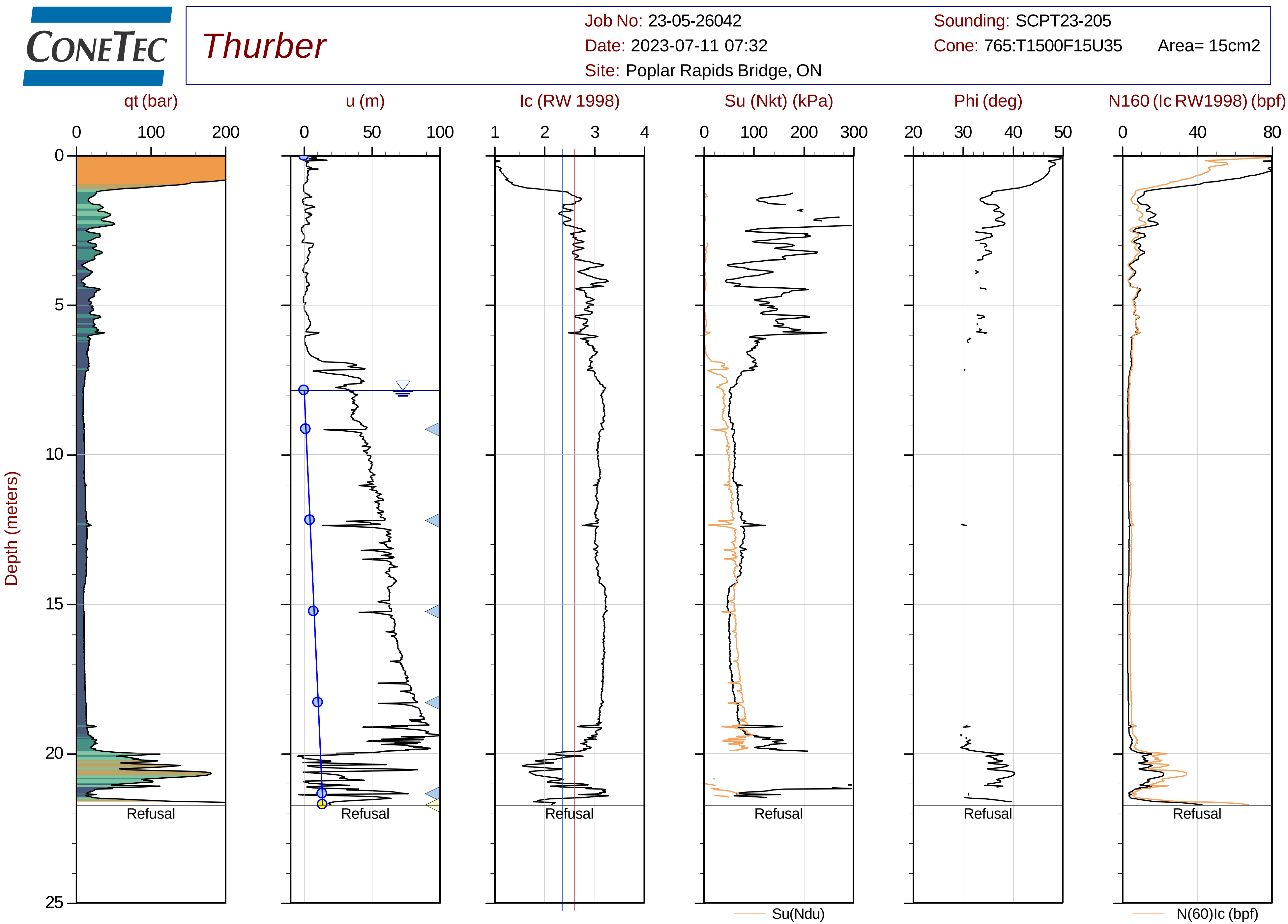
Max Depth: 18.925 m / 62.09 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-26042_SP23-108.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 9.0

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 5459902m E: 442905m
Sheet No: 1 of 1

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

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



Max Depth: 21.725 m / 71.28 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-26042_SP23-205.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 9.0

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 5459882m E: 443394m
Sheet No: 1 of 1

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

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

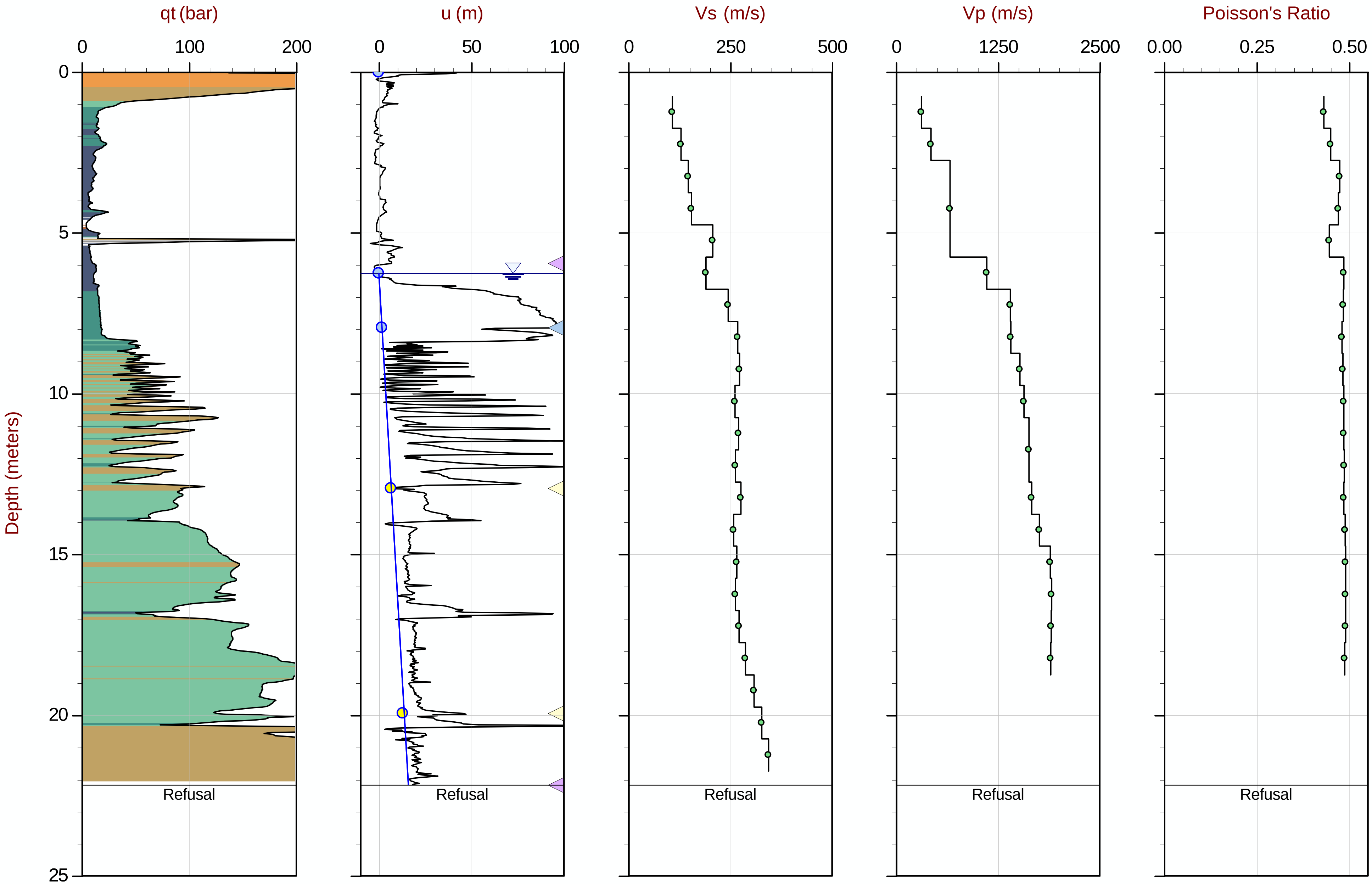
Seismic Cone Penetration Test Plots

CONETEC

Thurber

Job No: 23-05-26042
Date: 2023-07-10 14:57
Site: Poplar Rapids Bridge, ON

Sounding: SCPT23-107
Cone: 958:T1500F15U35 Area= 15cm2



Max Depth: 22.175 m / 72.75 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-26042_SP23-107.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 5459917m E: 442998m
Sheet No: 1 of 1

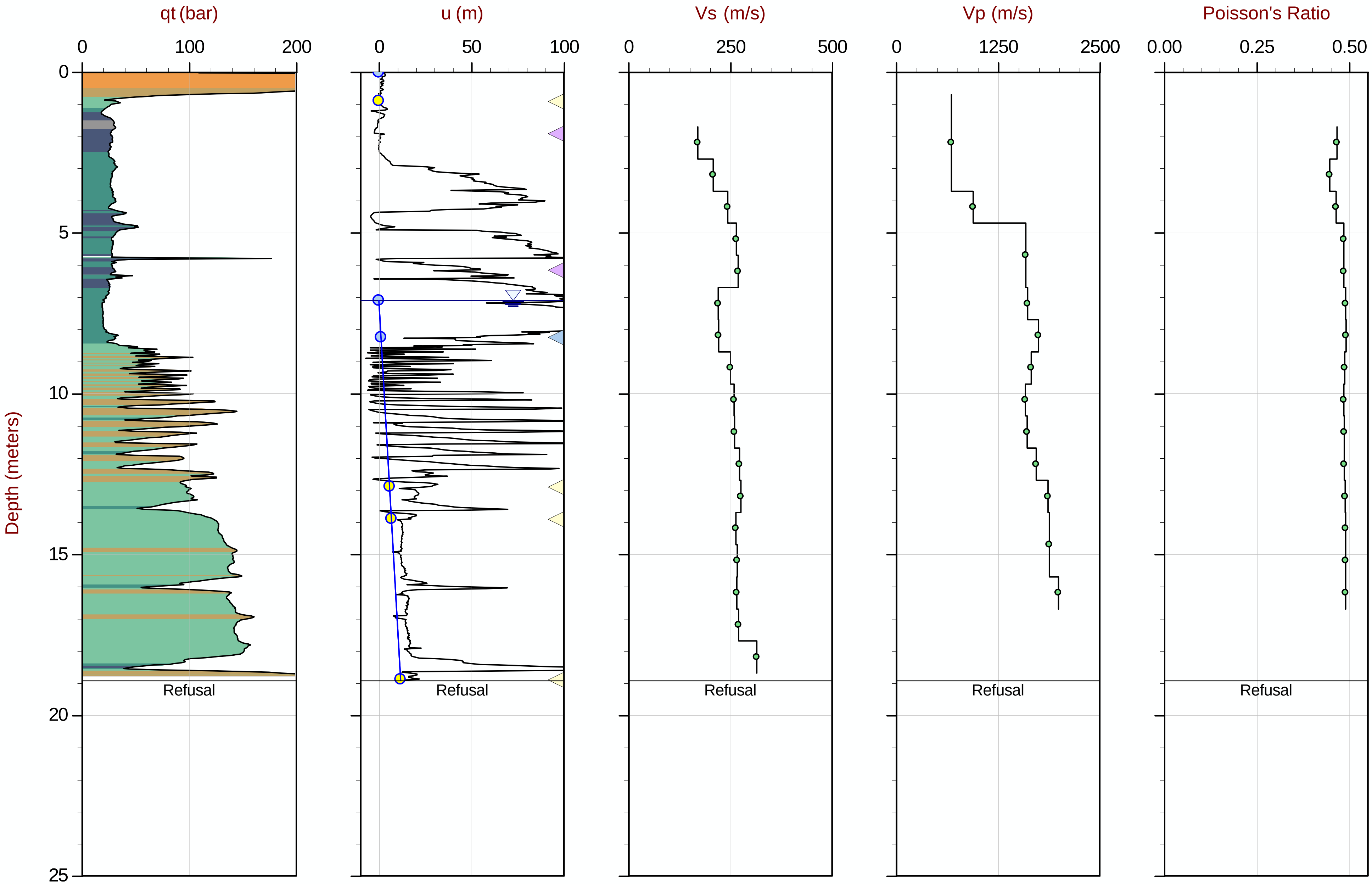
Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Hydrostatic Line

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

Thurber

Job No: 23-05-26042
Date: 2023-07-10 08:39
Site: Poplar Rapids Bridge, ON

Sounding: SCPT23-108
Cone: 958:T1500F15U35 Area= 15cm2



Max Depth: 18.925 m / 62.09 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-26042_SP23-108.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 5459902m E: 442905m
Sheet No: 1 of 1

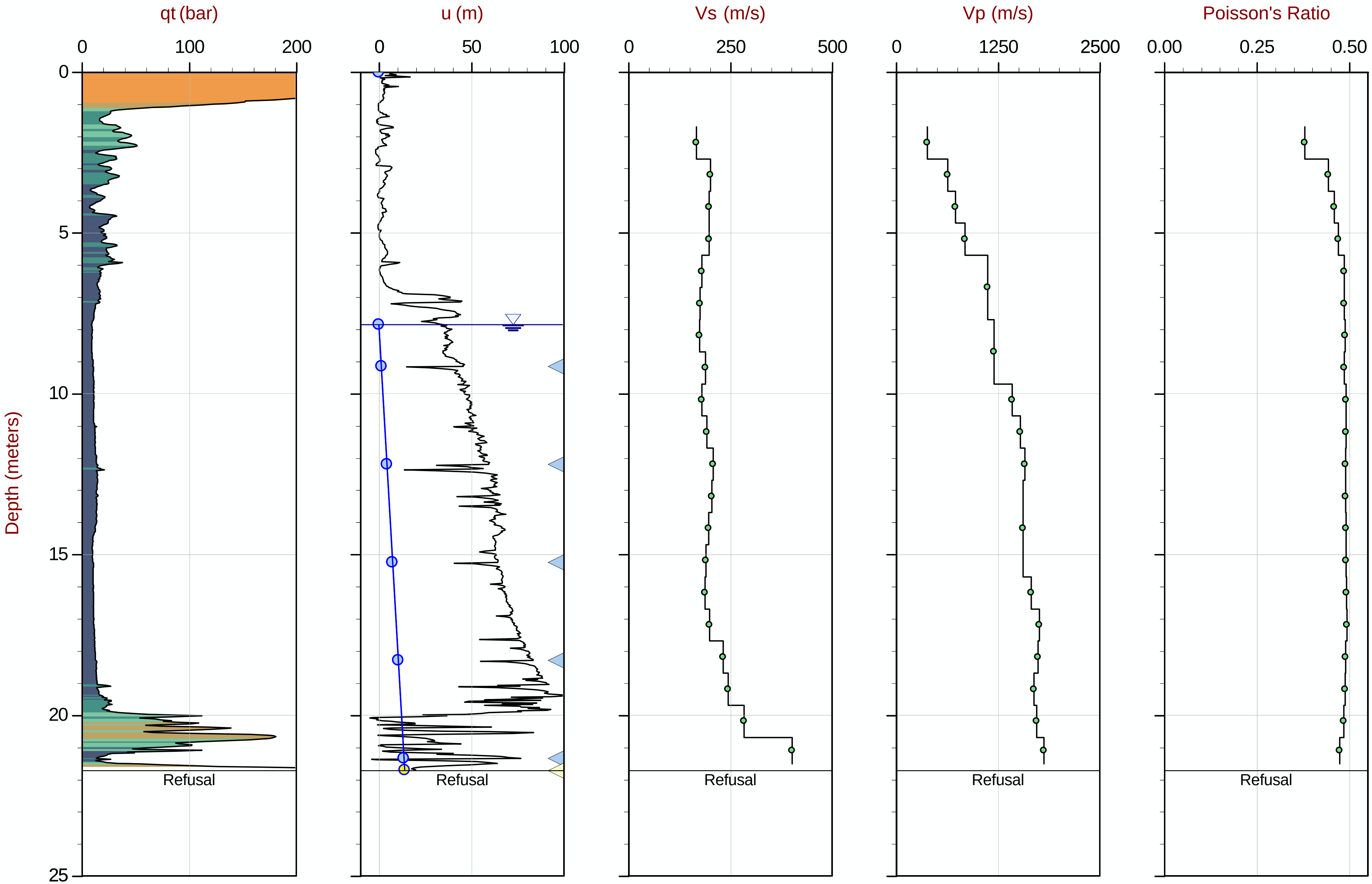
Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Hydrostatic Line

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

ConeTec

Thurber

Job No: 23-05-26042
Date: 2023-07-11 07:32
Site: Poplar Rapids Bridge, ON
Sounding: SCPT23-205
Cone: 765:T1500F15U35 Area= 15cm2



Max Depth: 21.725 m / 71.28 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-26042_SP23-205.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 5459882m E: 443394m
Sheet No: 1 of 1

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

The reported coordinates were acquired from Hand Held 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:	23-05-26042
Client:	Thurber Engineering Ltd.
Project:	Poplar Rapids Bridge - Highway 11
Sounding ID:	SCPT23-107
Date:	10-Jul-2023
Seismic Source:	Beam
Seismic Offset (m):	0.55
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)
0.95	0.75	0.93			
1.95	1.75	1.83	0.90	8.39	108
2.95	2.75	2.80	0.97	7.53	129
3.95	3.75	3.79	0.99	6.73	147
4.95	4.75	4.78	0.99	6.43	154
5.95	5.75	5.78	0.99	4.79	207
6.95	6.75	6.77	1.00	5.24	190
7.95	7.75	7.77	1.00	4.07	245
8.95	8.75	8.77	1.00	3.71	269
9.95	9.75	9.77	1.00	3.67	272
10.95	10.75	10.76	1.00	3.81	262
11.95	11.75	11.76	1.00	3.69	271
12.95	12.75	12.76	1.00	3.80	263
13.95	13.75	13.76	1.00	3.62	276
14.95	14.75	14.76	1.00	3.87	258
15.95	15.75	15.76	1.00	3.75	267
16.95	16.75	16.76	1.00	3.80	263
17.95	17.75	17.76	1.00	3.68	272
18.95	18.75	18.76	1.00	3.48	287
19.95	19.75	19.76	1.00	3.24	309
20.95	20.75	20.76	1.00	3.05	327
21.95	21.75	21.76	1.00	2.90	344

Job No:	23-05-26042
Client:	Thurber Engineering Ltd.
Project:	Poplar Rapids Bridge - Highway 11
Sounding ID:	SCPT23-108
Date:	10-Jul-2023
Seismic Source:	Beam
Seismic Offset (m):	0.55
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)
1.90	1.70	1.79			
2.90	2.70	2.76	0.97	5.69	170
3.90	3.70	3.74	0.99	4.73	208
4.90	4.70	4.73	0.99	4.07	243
5.90	5.70	5.73	0.99	3.76	265
6.90	6.70	6.72	1.00	3.70	269
7.90	7.70	7.72	1.00	4.53	220
8.90	8.70	8.72	1.00	4.49	222
9.90	9.70	9.72	1.00	3.98	251
10.90	10.70	10.71	1.00	3.84	260
11.90	11.70	11.71	1.00	3.84	260
12.90	12.70	12.71	1.00	3.66	273
13.90	13.70	13.71	1.00	3.62	276
14.90	14.70	14.71	1.00	3.78	264
15.90	15.70	15.71	1.00	3.74	267
16.90	16.70	16.71	1.00	3.75	266
17.90	17.70	17.71	1.00	3.70	271
18.90	18.70	18.71	1.00	3.17	315

Job No:	23-05-26042
Client:	Thurber Engineering Ltd.
Project:	Poplar Rapids Bridge - Highway 11
Sounding ID:	SCPT23-205
Date:	11-Jul-2023
Seismic Source:	Beam
Seismic Offset (m):	0.55
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)
1.90	1.70	1.79			
2.90	2.70	2.76	0.97	5.79	167
3.90	3.70	3.74	0.99	4.90	201
4.90	4.70	4.73	0.99	5.01	198
5.90	5.70	5.73	0.99	5.01	198
6.90	6.70	6.72	1.00	5.51	181
7.90	7.70	7.72	1.00	5.66	176
8.90	8.70	8.72	1.00	5.70	175
9.90	9.70	9.72	1.00	5.28	189
10.90	10.70	10.71	1.00	5.53	181
11.90	11.70	11.71	1.00	5.18	193
12.90	12.70	12.71	1.00	4.81	208
13.90	13.70	13.71	1.00	4.87	205
14.90	14.70	14.71	1.00	5.07	197
15.90	15.70	15.71	1.00	5.25	190
16.90	16.70	16.71	1.00	5.30	189
17.90	17.70	17.71	1.00	5.02	199
18.90	18.70	18.71	1.00	4.29	233
19.90	19.70	19.71	1.00	4.08	245
20.90	20.70	20.71	1.00	3.51	284
21.72	21.52	21.53	0.82	2.04	402

Seismic Cone Penetration Test Shear Wave (Vs) Traces

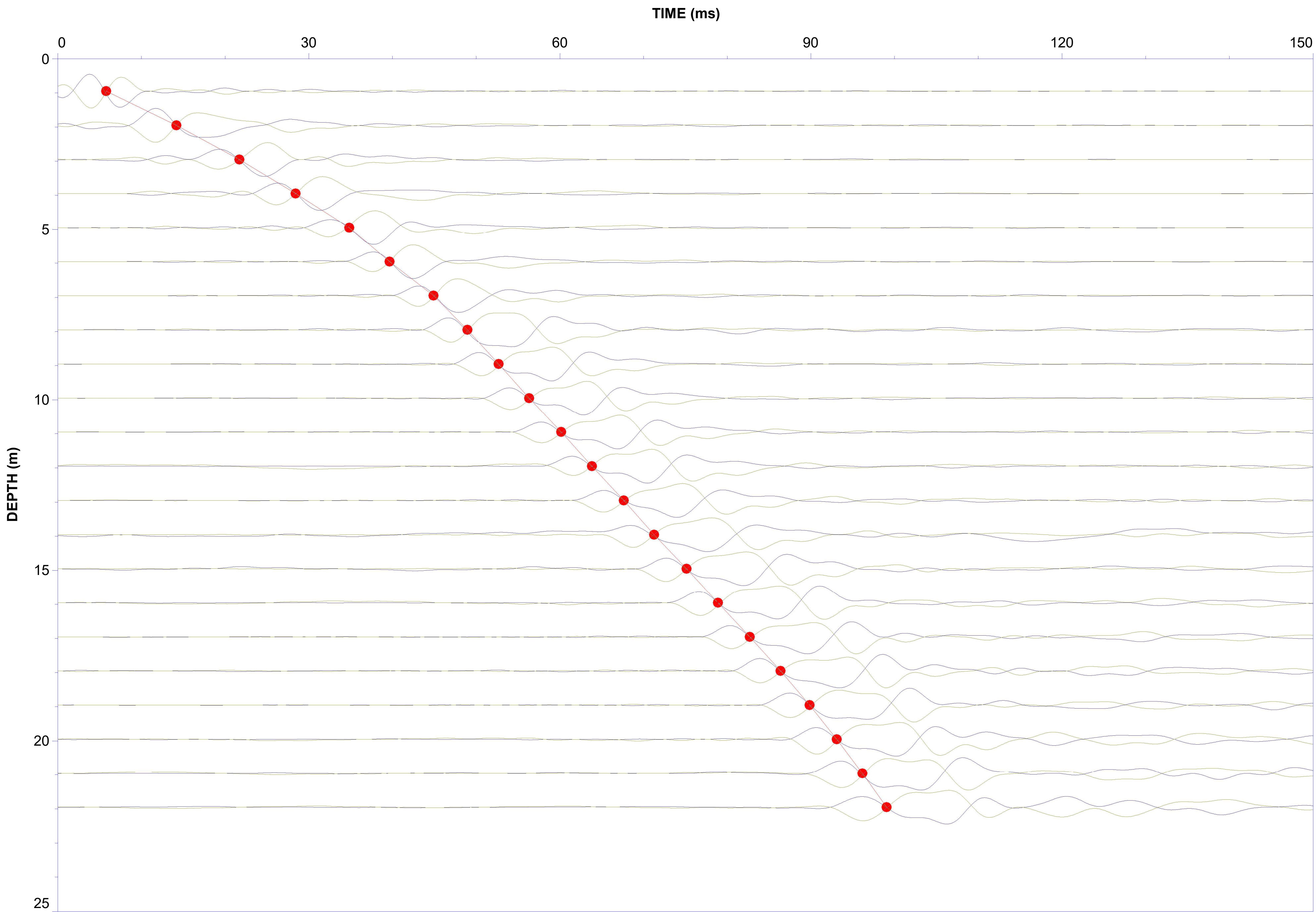
Job No: 23-05-26042
Analysis: S-Wave_Geo-X

Client: Thurber
Date: 10-Jul-2023

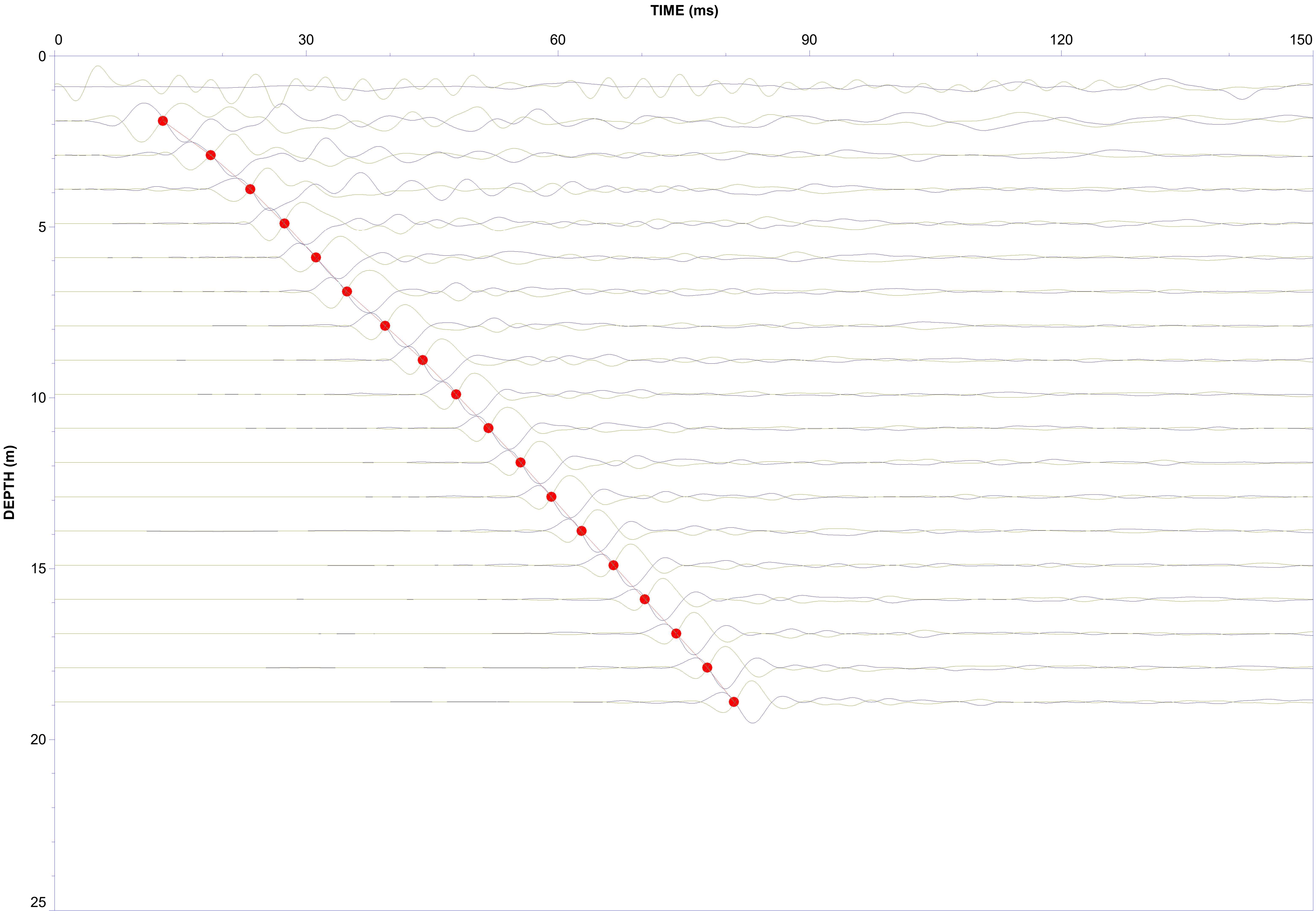
Project: Poplar Rapids Bridge - Highway 11

Filter: 0-200Hz

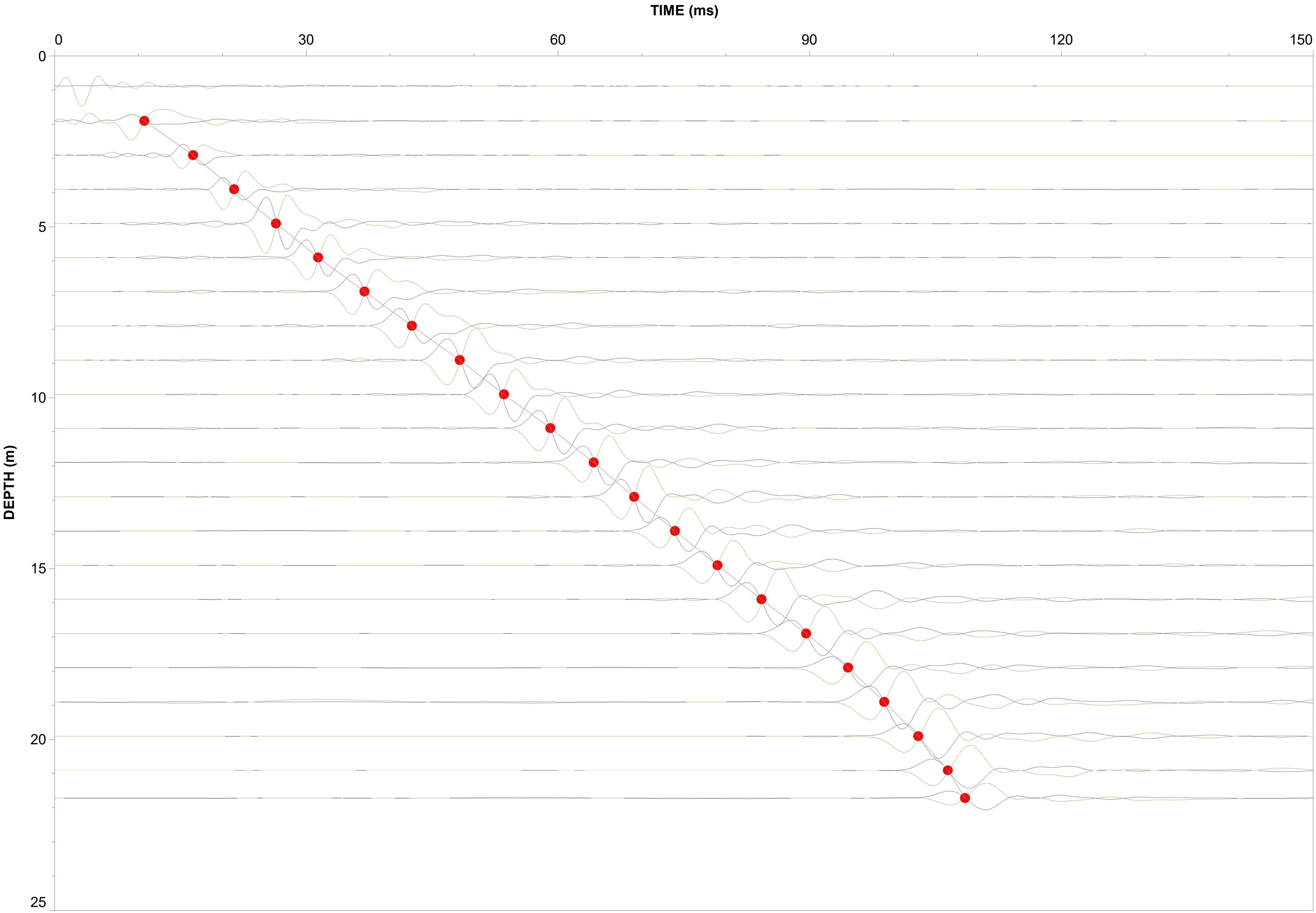
Sounding: SCPT23-107



Job No: 23-05-26042	Client: Thurber	Project: Poplar Rapids Bridge - Highway 11	Filter: 0-300Hz	Sounding: SCPT23-108
Analysis: S-Wave_Geo-X	Date: 10-Jul-2023			



Job No: 23-05-26042	Client: Thurber	Project: Poplar Rapids Bridge - Highway 11	Filter: 0-350Hz	Sounding: SCPT23-205
Analysis: S-Wave_Geo-X	Date: 11-Jul-2023			



Seismic Cone Penetration Test Compression Wave (Vp) Tabular Results

ConeTec

Job No:	23-05-26042
Client:	Thurber
Project:	Poplar Rapids Bridge - Highway 11
Sounding ID:	SCPT23-107
Date:	10-Jul-2023
Seismic Source:	Plate
Seismic Offset (m):	1.40
Source Depth (m):	0.00
Geophone Offset (m):	0.20

SCPTu COMPRESSION WAVE VELOCITY TEST RESULTS - Vp

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
0.95	0.75	1.59			
1.95	1.75	2.24	0.65	2.10	311
2.95	2.75	3.09	0.85	1.98	428
5.95	5.75	5.92	2.83	4.28	663
6.95	6.75	6.89	0.98	0.88	1115
7.95	7.75	7.88	0.98	0.70	1403
8.95	8.75	8.86	0.99	0.70	1409
9.95	9.75	9.85	0.99	0.65	1521
10.95	10.75	10.84	0.99	0.63	1574
12.95	12.75	12.83	1.99	1.22	1631
13.95	13.75	13.82	0.99	0.60	1664
14.95	14.75	14.82	1.00	0.57	1759
15.95	15.75	15.81	1.00	0.53	1895
16.95	16.75	16.81	1.00	0.52	1914
17.95	17.75	17.81	1.00	0.52	1906
18.95	18.75	18.80	1.00	0.53	1897

Job No: 23-05-26042
Client: Thurber
Project: Poplar Rapids Bridge - Highway 11
Sounding ID: SCPT23-108
Date: 10-Jul-2023

Seismic Source: Plate
Seismic Offset (m): 1.40
Source Depth (m): 0.00
Geophone Offset (m): 0.20

SCPTu COMPRESSION WAVE VELOCITY TEST RESULTS - Vp

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
0.90	0.70	1.57			
3.90	3.70	3.96	2.39	3.52	679
4.90	4.70	4.90	0.95	1.00	948
6.90	6.70	6.85	1.94	1.22	1594
7.90	7.70	7.83	0.98	0.61	1615
8.90	8.70	8.81	0.99	0.56	1747
9.90	9.70	9.80	0.99	0.60	1658
10.90	10.70	10.79	0.99	0.62	1587
11.90	11.70	11.78	0.99	0.62	1609
12.90	12.70	12.78	0.99	0.58	1724
13.90	13.70	13.77	0.99	0.53	1869
15.90	15.70	15.76	1.99	1.06	1886
16.90	16.70	16.76	1.00	0.50	1993

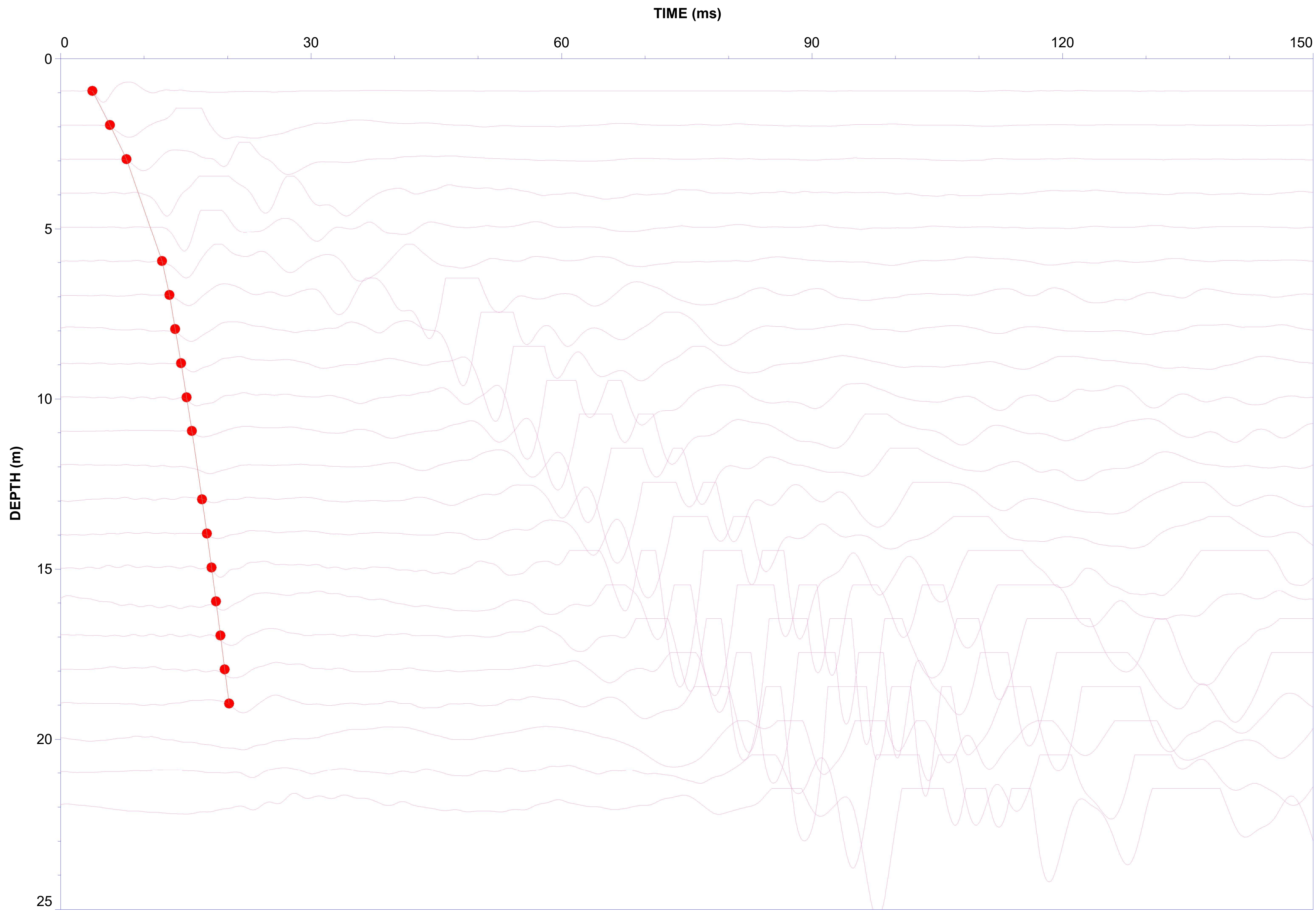
Job No:	23-05-26042
Client:	Thurber
Project:	Poplar Rapids Bridge - Highway 11
Sounding ID:	SCPT23-205
Date:	11-Jul-2023
Seismic Source:	Plate
Seismic Offset (m):	1.40
Source Depth (m):	0.00
Geophone Offset (m):	0.20

SCPTu COMPRESSION WAVE VELOCITY TEST RESULTS - Vp

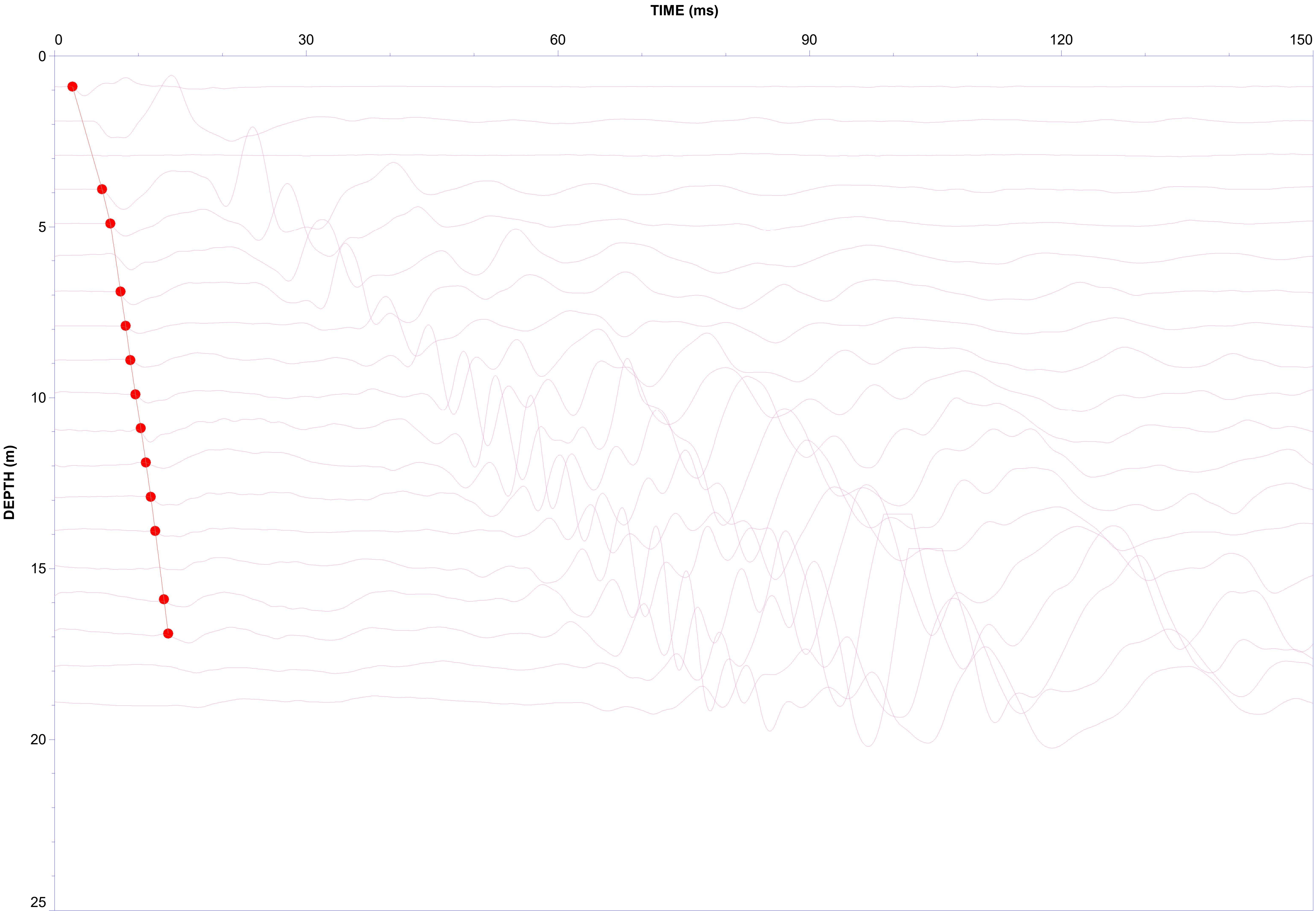
Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
1.90	1.70	2.20			
2.90	2.70	3.04	0.84	2.21	380
3.90	3.70	3.96	0.92	1.44	634
4.90	4.70	4.90	0.95	1.30	730
5.90	5.70	5.87	0.97	1.14	844
7.90	7.70	7.83	1.96	1.74	1122
9.90	9.70	9.80	1.98	1.64	1205
10.90	10.70	10.79	0.99	0.69	1428
11.90	11.70	11.78	0.99	0.65	1527
12.90	12.70	12.78	0.99	0.63	1583
15.90	15.70	15.76	2.99	1.92	1559
16.90	16.70	16.76	1.00	0.60	1661
17.90	17.70	17.76	1.00	0.57	1760
18.90	18.70	18.75	1.00	0.57	1744
19.90	19.70	19.75	1.00	0.59	1694
20.90	20.70	20.75	1.00	0.58	1728
21.72	21.52	21.57	0.82	0.45	1818

Seismic Cone Penetration Test Compression Wave (Vp) Traces

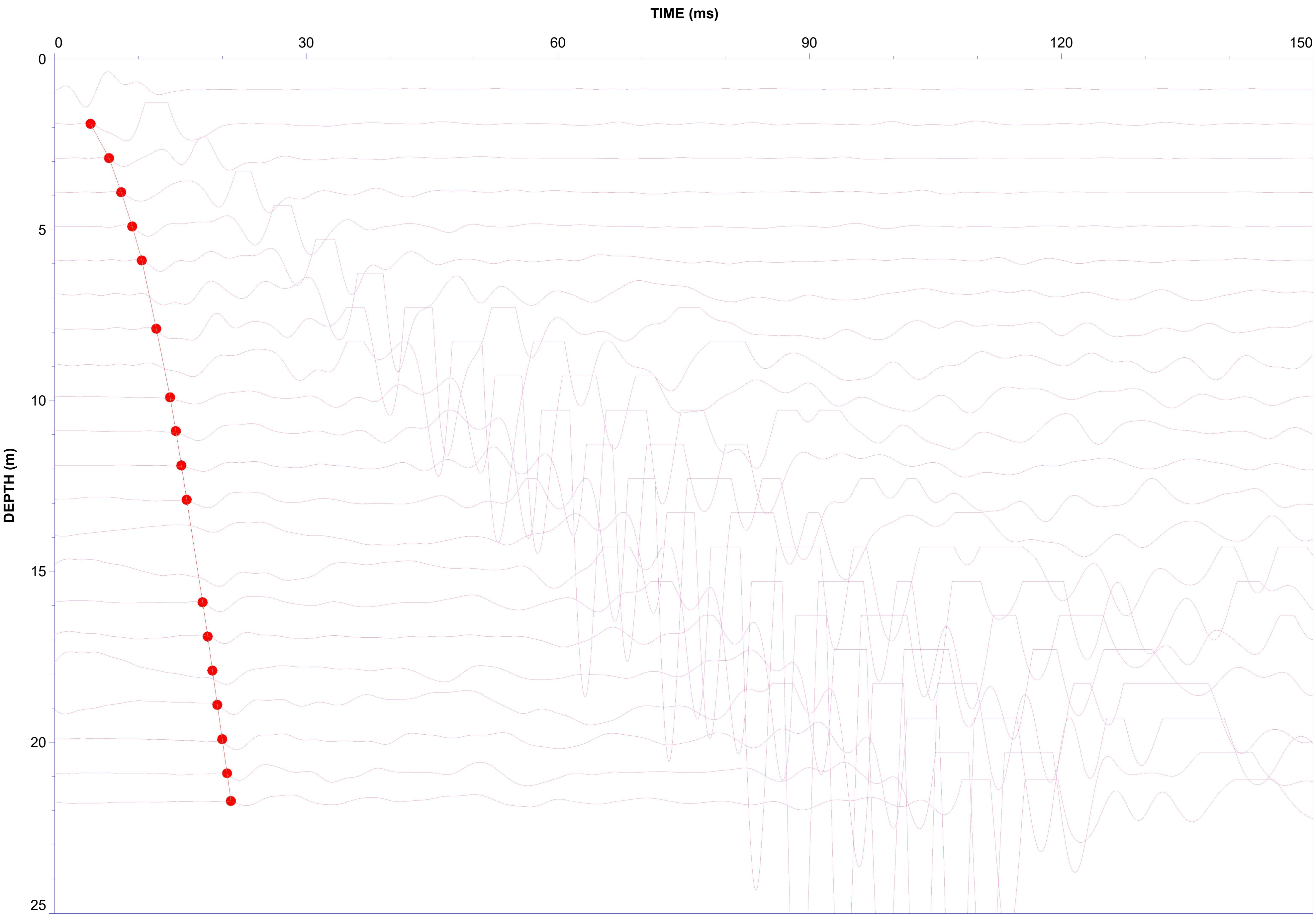
Job No: 23-05-26042	Client: Thurber	Project: Poplar Rapids Bridge - Highway 11	Filter: 0-500Hz	Sounding: SCPT23-107
Analysis: P-Wave_Geo-X	Date: 10-Jul-2023			



Job No: 23-05-26042 Client: Thurber Project: Poplar Rapids Bridge - Highway 11 Filter: 0-500Hz Sounding: SCPT23-108
Analysis: P-Wave_Geo-X Date: 10-Jul-2023



Job No: 23-05-26042 | Client: Thurber | Project: Poplar Rapids Bridge - Highway 11 | Filter: 0-300Hz | Sounding: SCPT23-205
Analysis: P-Wave_Geo-X | Date: 11-Jul-2023



Seismic Cone Penetration Test Poisson's Ratio Tabular Results

Job No: 23-05-26042
Client: Thurber
Project: Poplar Rapids Bridge - Highway 11
Sounding ID: SCPT23-107
Date: 10-Jul-2023

SCPTu POISSON'S RATIO RESULTS

Depth From (m)	Depth To (m)	Vs Interval Velocity (m/s)	Vp Interval Velocity (m/s)	Poisson's Ratio
0.75	1.75	108	311	0.43
1.75	2.75	129	428	0.45
2.75	3.75	147	663	0.47
3.75	4.75	154	663	0.47
4.75	5.75	207	663	0.45
5.75	6.75	190	1115	0.49
6.75	7.75	245	1403	0.48
7.75	8.75	269	1409	0.48
8.75	9.75	272	1521	0.48
9.75	10.75	262	1574	0.49
10.75	11.75	271	1631	0.49
11.75	12.75	263	1631	0.49
12.75	13.75	276	1664	0.49
13.75	14.75	258	1759	0.49
14.75	15.75	267	1895	0.49
15.75	16.75	263	1914	0.49
16.75	17.75	272	1906	0.49
17.75	18.75	287	1897	0.49

Job No:	23-05-26042
Client:	Thurber
Project:	Poplar Rapids Bridge - Highway 11
Sounding ID:	SCPT23-108
Date:	10-Jul-2023

SCPTu POISSON'S RATIO RESULTS

Depth From (m)	Depth To (m)	Vs Interval Velocity (m/s)	Vp Interval Velocity (m/s)	Poisson's Ratio
1.70	2.70	170	679	0.47
2.70	3.70	208	679	0.45
3.70	4.70	243	948	0.47
4.70	5.70	265	1594	0.49
5.70	6.70	269	1594	0.49
6.70	7.70	220	1615	0.49
7.70	8.70	222	1747	0.49
8.70	9.70	251	1658	0.49
9.70	10.70	260	1587	0.49
10.70	11.70	260	1609	0.49
11.70	12.70	273	1724	0.49
12.70	13.70	276	1869	0.49
13.70	14.70	264	1886	0.49
14.70	15.70	267	1886	0.49
15.70	16.70	266	1993	0.49

	Job No:	23-05-26042
	Client:	Thurber
	Project:	Poplar Rapids Bridge - Highway 11
	Sounding ID:	SCPT23-205
	Date:	11-Jul-2023

SCPTu POISSON'S RATIO RESULTS

Depth From (m)	Depth To (m)	Vs Interval Velocity (m/s)	Vp Interval Velocity (m/s)	Poisson's Ratio
1.70	2.70	167	380	0.38
2.70	3.70	201	634	0.44
3.70	4.70	198	730	0.46
4.70	5.70	198	844	0.47
5.70	6.70	181	1122	0.49
6.70	7.70	176	1122	0.49
7.70	8.70	175	1205	0.49
8.70	9.70	189	1205	0.49
9.70	10.70	181	1428	0.49
10.70	11.70	193	1527	0.49
11.70	12.70	208	1583	0.49
12.70	13.70	205	1559	0.49
13.70	14.70	197	1559	0.49
14.70	15.70	190	1559	0.49
15.70	16.70	189	1661	0.49
16.70	17.70	199	1760	0.49
17.70	18.70	233	1744	0.49
18.70	19.70	245	1694	0.49
19.70	20.70	284	1728	0.49
20.70	21.52	402	1818	0.47

Soil Behaviour Type (SBT) Scatter Plots

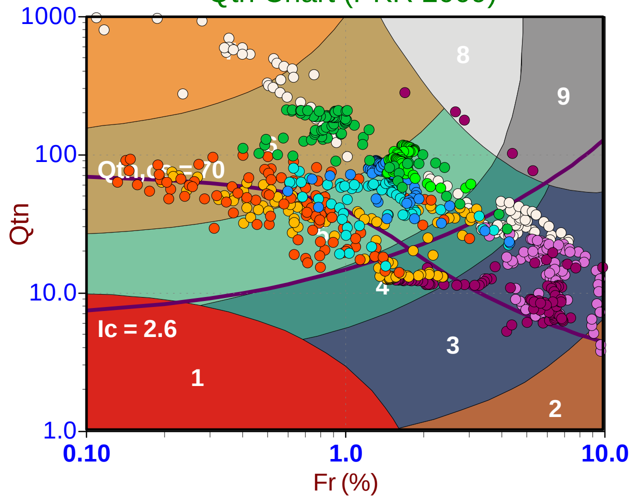
ConeTec

Thurber

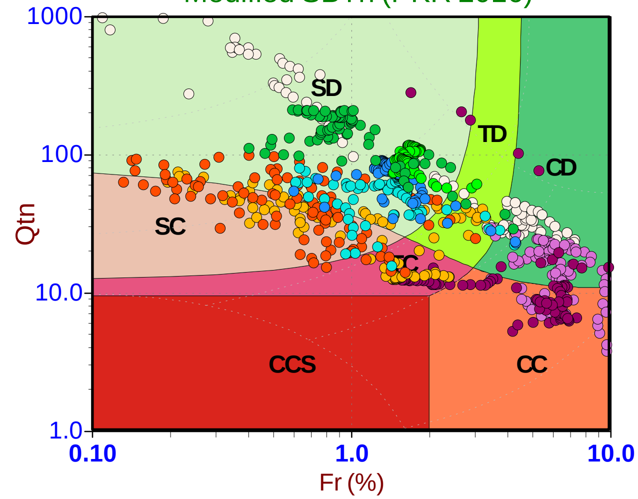
Job No: 23-05-26042
Date: 2023-07-10 14:57
Site: Poplar Rapids Bridge, ON

Sounding: SCPT23-107
Cone: 958:T1500F15U35

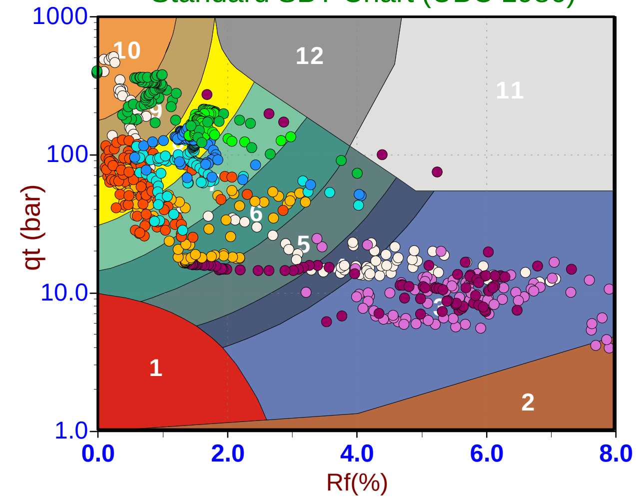
Qtn Chart (PKR 2009)



Modified SBTn (PKR 2016)



Standard SBT Chart (UBC 1986)



Depth Ranges

- >0.0 to 2.5 m
- >2.5 to 5.0 m
- >5.0 to 7.5 m
- >7.5 to 10.0 m
- >10.0 to 12.5 m
- >12.5 to 15.0 m
- >15.0 to 17.5 m
- >17.5 to 20.0 m
- >20.0 to 22.5 m
- >22.5 to 25.0 m
- >25.0 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

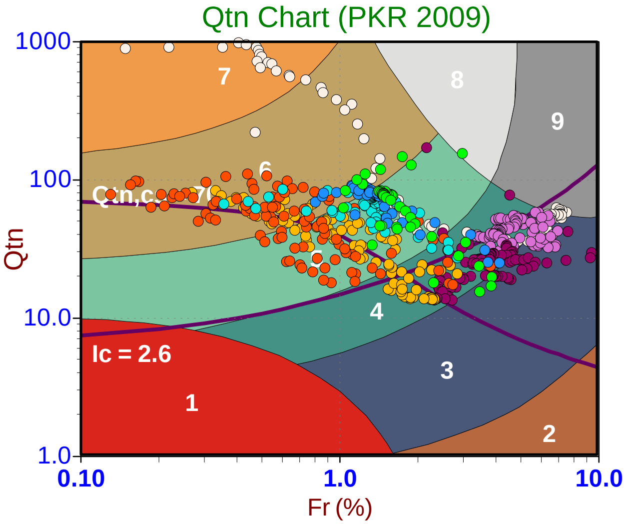
Legend

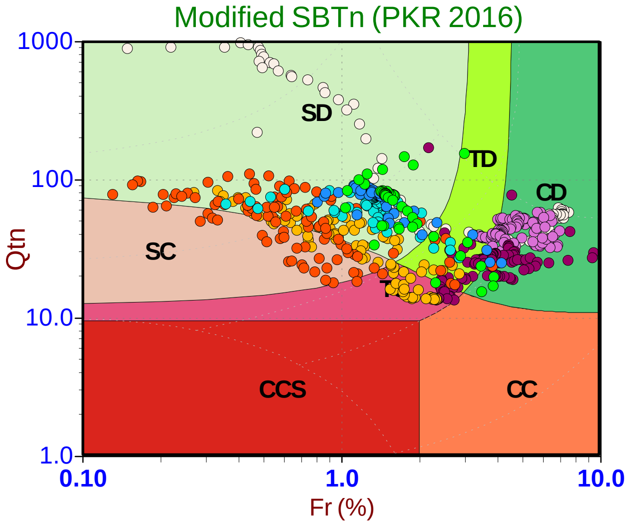
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

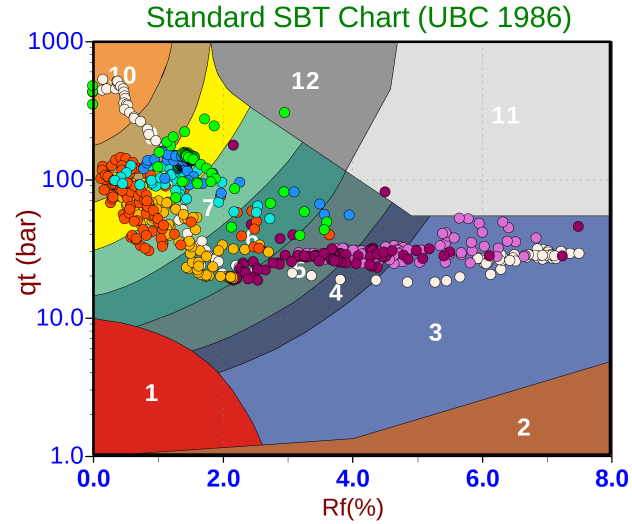
Legend

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

Thurber	Job No: 23-05-26042	Sounding: SCPT23-108
	Date: 2023-07-10 08:39	Cone: 958:T1500F15U35
	Site: Poplar Rapids Bridge, ON	







Depth Ranges
- >0.0 to 2.5 m
- >2.5 to 5.0 m
- >5.0 to 7.5 m
- >7.5 to 10.0 m
- >10.0 to 12.5 m
- >12.5 to 15.0 m
- >15.0 to 17.5 m
- >17.5 to 20.0 m
- >20.0 to 22.5 m
- >22.5 to 25.0 m
- >25.0 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

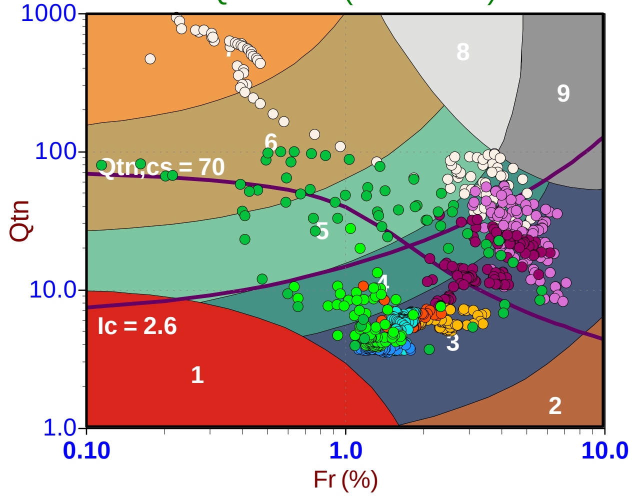
ConeTec

Thurber

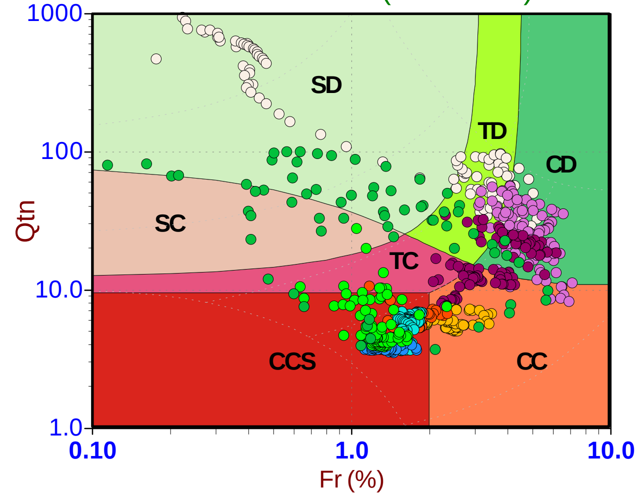
Job No: 23-05-26042
Date: 2023-07-11 07:32
Site: Poplar Rapids Bridge, ON

Sounding: SCPT23-205
Cone: 765:T1500F15U35

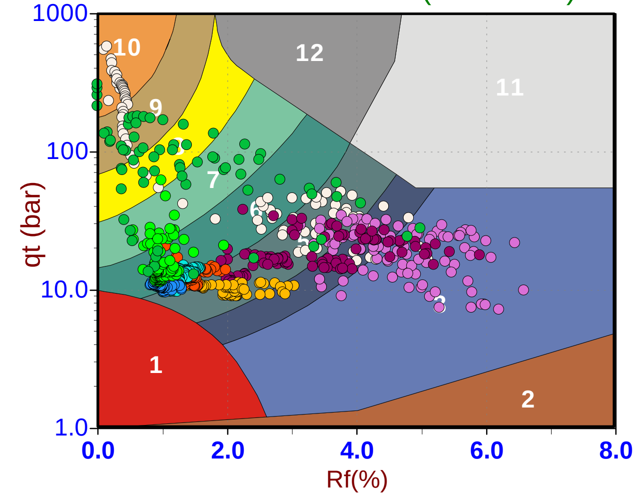
Qtn Chart (PKR 2009)



Modified SBTn (PKR 2016)



Standard SBT Chart (UBC 1986)



Depth Ranges

- >0.0 to 2.5 m
- >2.5 to 5.0 m
- >5.0 to 7.5 m
- >7.5 to 10.0 m
- >10.0 to 12.5 m
- >12.5 to 15.0 m
- >15.0 to 17.5 m
- >17.5 to 20.0 m
- >20.0 to 22.5 m
- >22.5 to 25.0 m
- >25.0 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

CONETEC

Job No: 23-05-26042
Client: Thurber Engineering Ltd.
Project: Poplar Rapids Bridge - Highway 11
Start Date: 2023-07-10
End Date: 2023-07-11

CPTu PORE PRESSURE DISSIPATION SUMMARY

Sounding ID	File Name	Cone Area (cm^2)	Duration (s)	Test Depth (m)	$U_{Initial}$ (m)	U_{max} (m)	U_{min} (m)	U_{final} (m)	Equilibrium Pore Pressure U_{eq} (m)	Estimated Equilibrium Pore Pressure U_{eq} (m)	Estimated Phreatic Surface (m)	Percent Dissipation (%)	t_{50} $(s)_1$	Assumed Rigidity Index (I_r)	c_h $(cm^2/min)_2$	Refer to Notation Number
SCPT23-107	23-05-26042_SP23-107	15	3390	5.950	6.1	11.4	5.5	5.5								
SCPT23-107	23-05-26042_SP23-107	15	1860	7.950	96.6	96.6	47.4	47.4		1.7	6.3	51.9	1705	100	0.4	3
SCPT23-107	23-05-26042_SP23-107	15	300	12.950	6.5	23.3	4.7	6.7	6.7		6.3	100.0				
SCPT23-107	23-05-26042_SP23-107	15	175	19.950	49.7	52.4	12.7	13.0	13.0		6.9	100.0				
SCPT23-107	23-05-26042_SP23-107	15	305	22.175	18.2	19.6	-1.0	15.6								
SCPT23-108	23-05-26042_SP23-108	15	640	0.900	1.3	1.3	-0.5	0.0	0.0							
SCPT23-108	23-05-26042_SP23-108	15	265	1.900	-0.1	2.1	-0.1	1.5								
SCPT23-108	23-05-26042_SP23-108	15	16320	6.150	32.5	85.7	32.5	42.6								
SCPT23-108	23-05-26042_SP23-108	15	610	8.250	46.4	75.9	31.5	31.5		1.1	7.1	33.0	363	100	1.9	3
SCPT23-108	23-05-26042_SP23-108	15	305	12.900	26.4	27.4	5.9	5.9	5.8		7.1	100.0				
SCPT23-108	23-05-26042_SP23-108	15	125	13.900	13.8	13.8	5.3	6.8	6.8		7.1	100.0				
SCPT23-108	23-05-26042_SP23-108	15	100	18.900	21.1	21.1	-8.9	11.7	11.8		7.1	100.0				
SCPT23-205	23-05-26042_SP23-205	15	5040	9.150	45.5	46.7	23.0	23.0		1.3	7.9	50.8	4499	100	0.2	3
SCPT23-205	23-05-26042_SP23-205	15	3720	12.200	59.4	62.1	40.7	40.7		4.3	7.9	34.0				3
SCPT23-205	23-05-26042_SP23-205	15	3600	15.250	63.9	63.9	41.6	41.6		7.4	7.9	39.5				3
SCPT23-205	23-05-26042_SP23-205	15	3780	18.300	83.5	83.5	51.9	51.9		10.4	7.9	43.3				3
SCPT23-205	23-05-26042_SP23-205	15	600	21.350	78.5	78.5	13.9	13.9		13.5	7.9	99.4	36	100	19.3	3
SCPT23-205	23-05-26042_SP23-205	15	310	21.725	19.0	20.3	6.0	13.9	13.9		7.9	100.0	8	100	93.5	

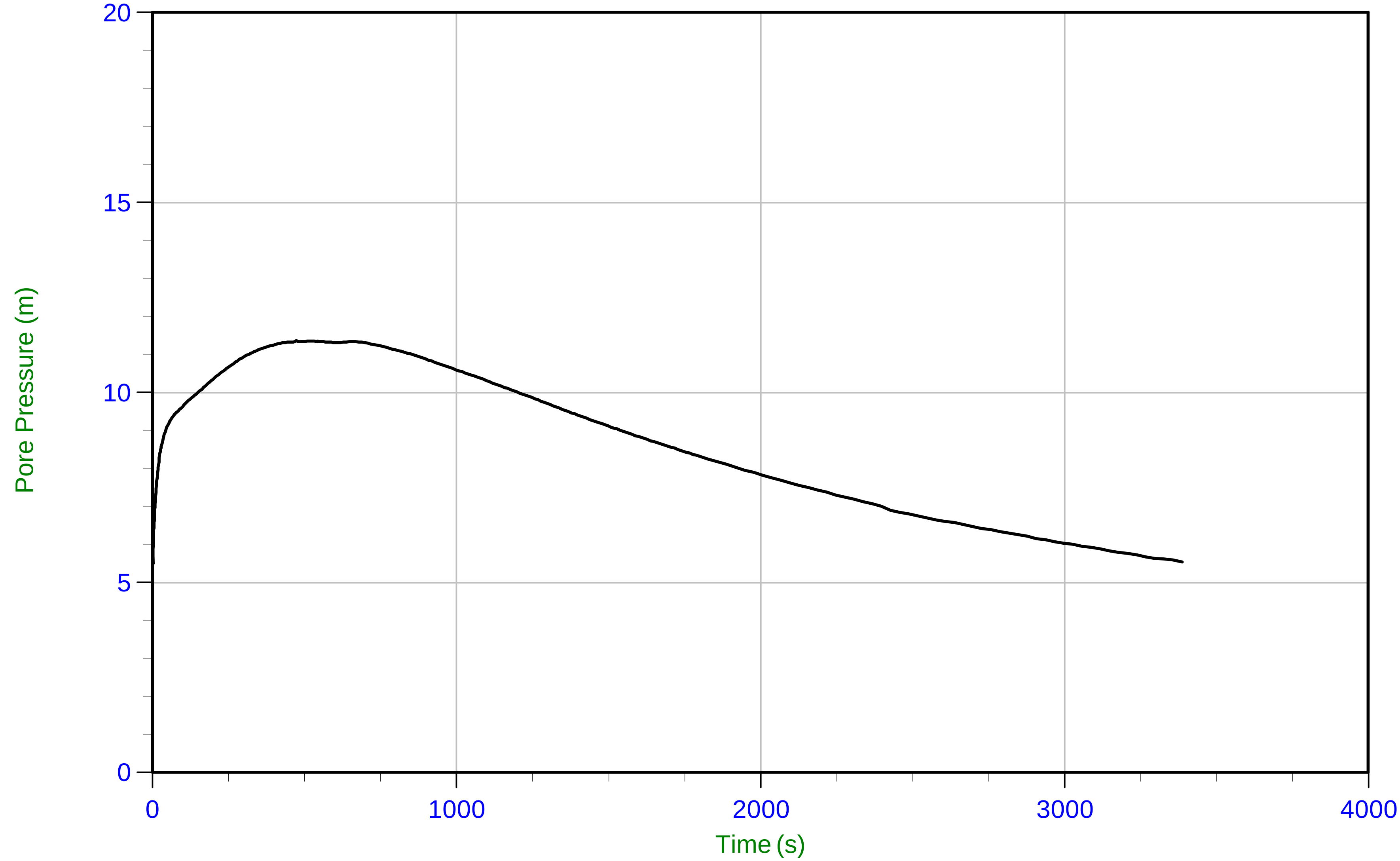
1. Time for 50 percent dissipation based in U_{max}, U_{min}, and the applied U_{eq}. Note the time is relative to where U_{max} occurred.
2. Houlsby and Teh, 1991.
3. The estimated equilibrium pore pressure was based on a hydrostatic assumption from the assumed phreatic surface.

CONETEC

Thurber

Job No: 23-05-26042
Date: 07/10/2023 14:57
Site: Poplar Rapids Bridge, ON

Sounding: SCPT23-107
Cone: 958:T1500F15U35 Area=15 cm²



Trace Summary:

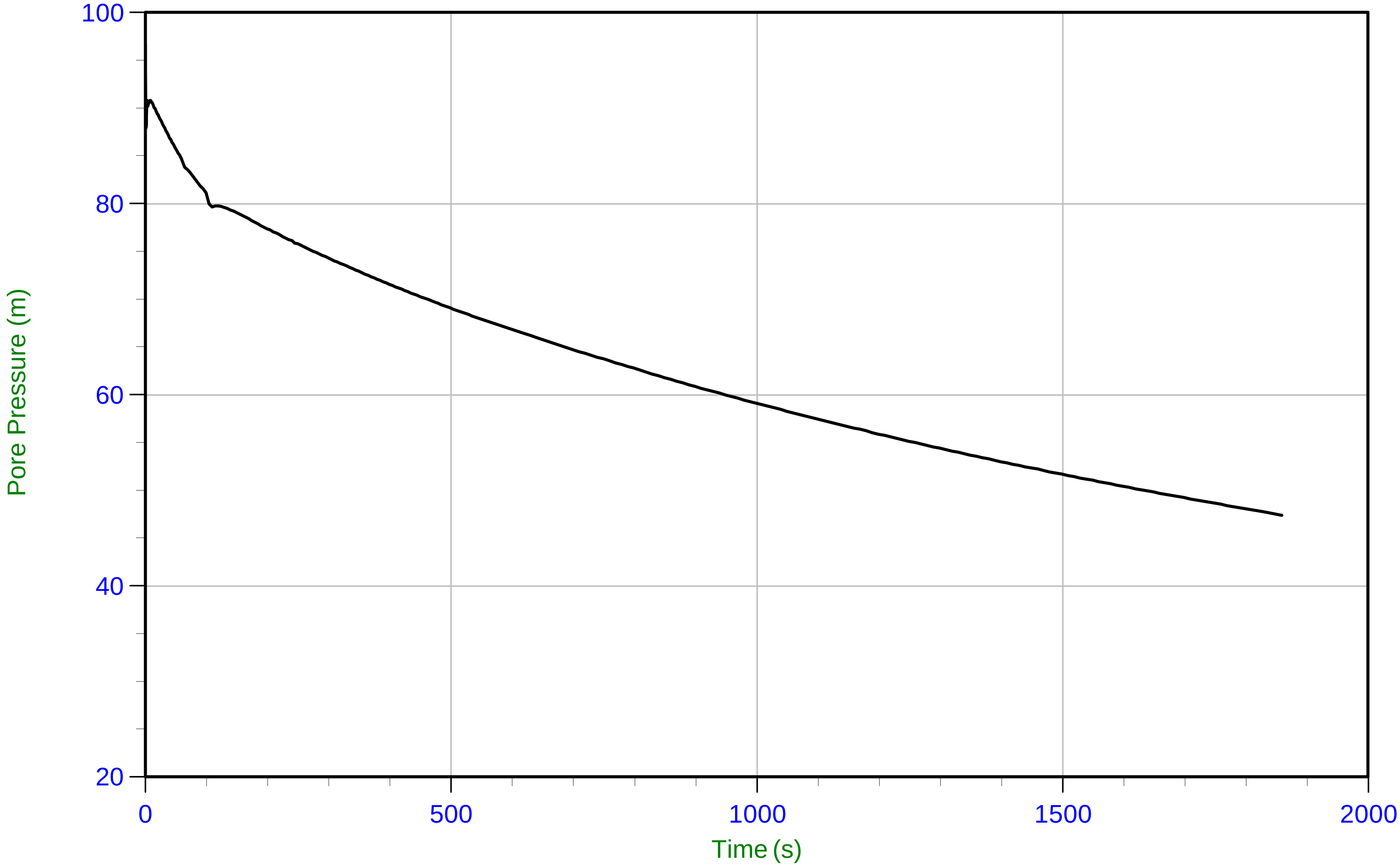
Filename: 23-05-26042_SP23-107.PPF2
Depth: 5.950 m / 19.521 ft
Duration: 3390.0 s

u Min: 5.5 m
u Max: 11.4 m
u Final: 5.5 m

Thurber

Job No: 23-05-26042
Date: 07/10/2023 14:57
Site: Poplar Rapids Bridge, ON

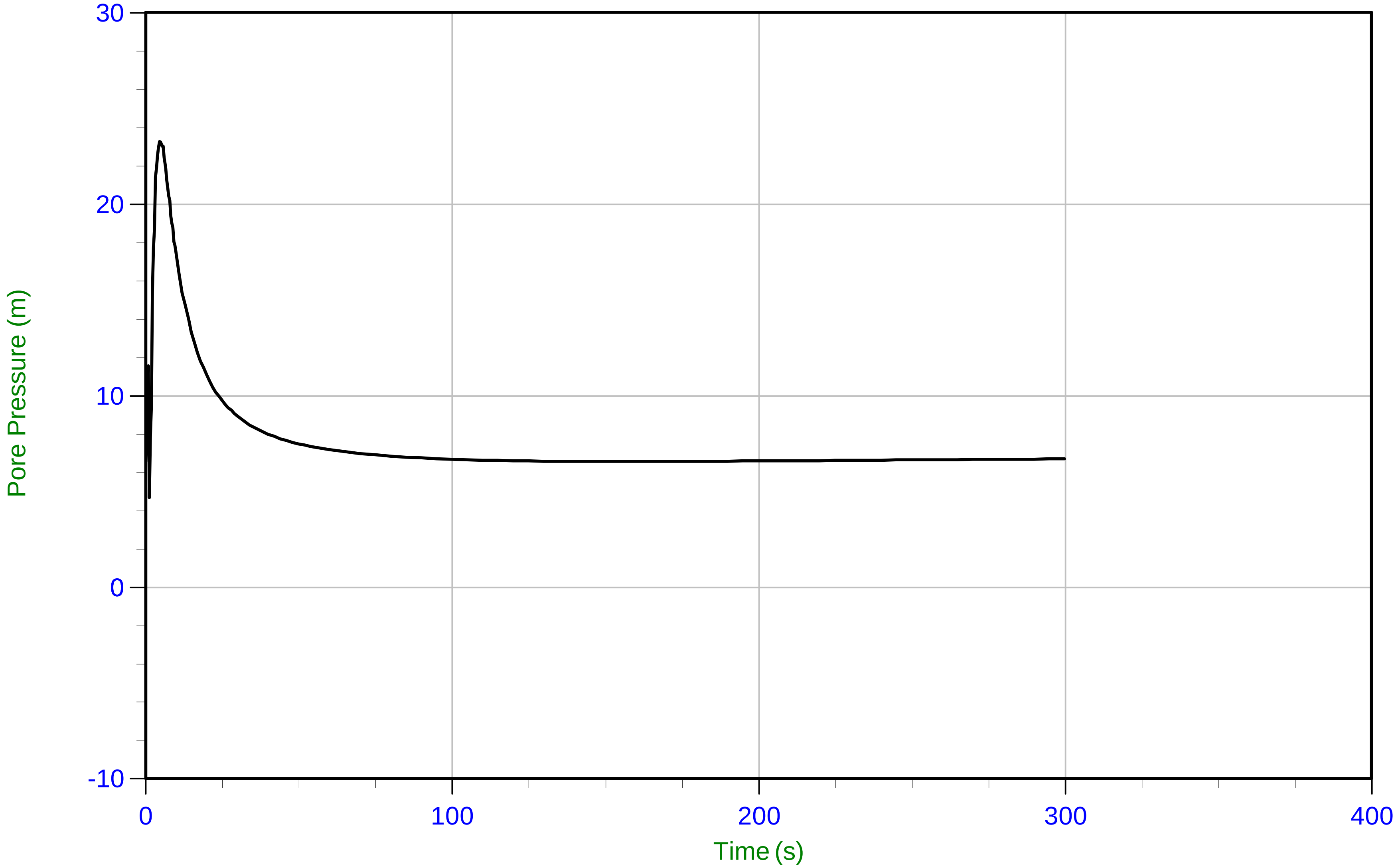
Sounding: SCPT23-107
Cone: 958:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 23-05-26042_SP23-107.PPF2	u Min: 47.4 m	WT: 6.255 m / 20.521 ft	T(50): 1705.0 s
Depth: 7.950 m / 26.082 ft	u Max: 96.6 m	Ueq: 1.7 m	Ir: 100
Duration: 1860.0 s	u Final: 47.4 m	U(50): 49.17 m	Ch: 0.4 cm²/min

Thurber	Job No: 23-05-26042 Date: 07/10/2023 14:57 Site: Poplar Rapids Bridge, ON	Sounding: SCPT23-107 Cone: 958:T1500F15U35 Area=15 cm²



Trace Summary:

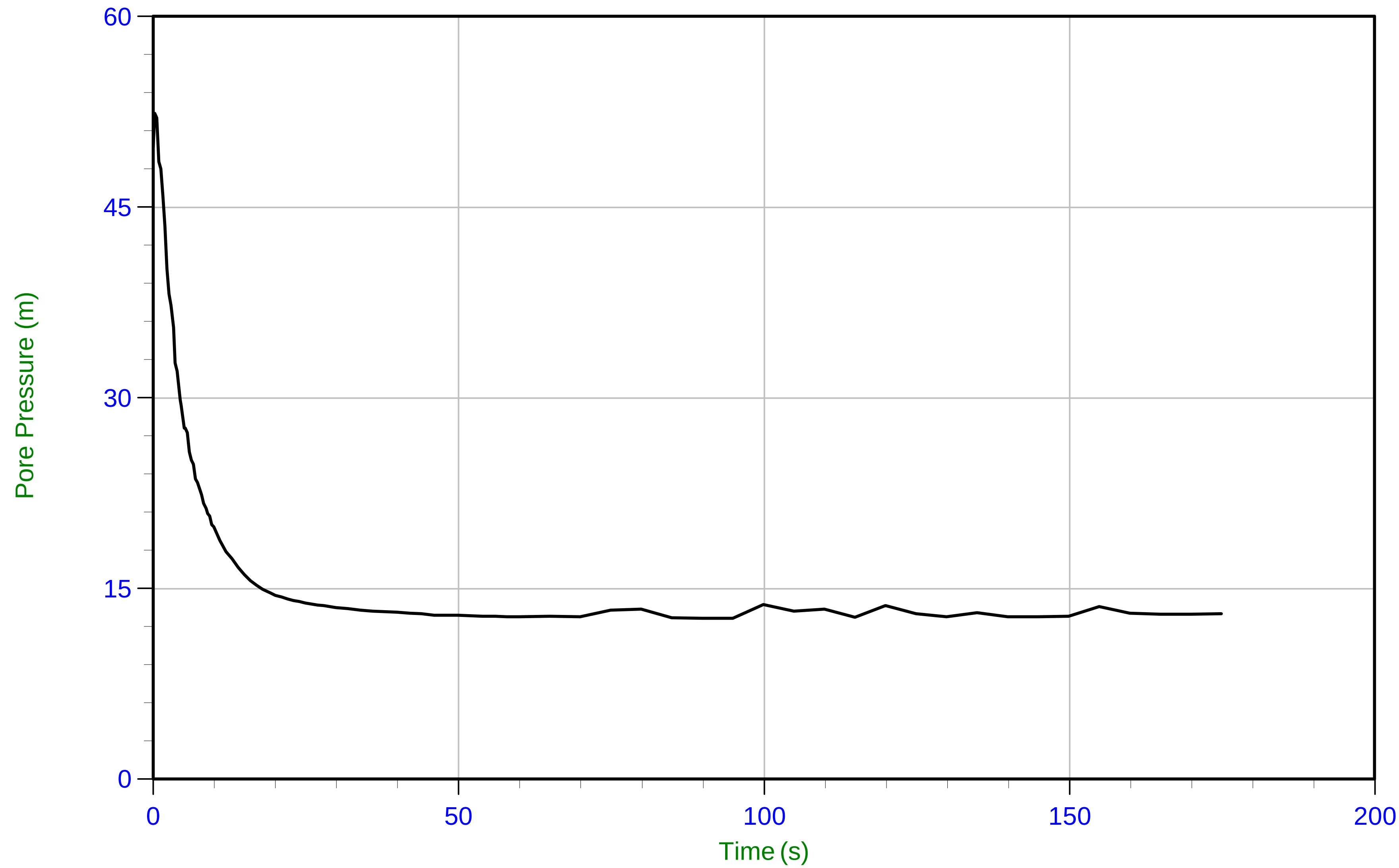
Filename: 23-05-26042_SP23-107.PPF2	u Min: 4.7 m	WT: 6.255 m / 20.521 ft
Depth: 12.950 m / 42.486 ft	u Max: 23.3 m	Ueq: 6.7 m
Duration: 300.0 s	u Final: 6.7 m	

CONETEC

Thurber

Job No: 23-05-26042
Date: 07/10/2023 14:57
Site: Poplar Rapids Bridge, ON

Sounding: SCPT23-107
Cone: 958:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 23-05-26042_SP23-107.PPF2
Depth: 19.950 m / 65.452 ft
Duration: 175.0 s

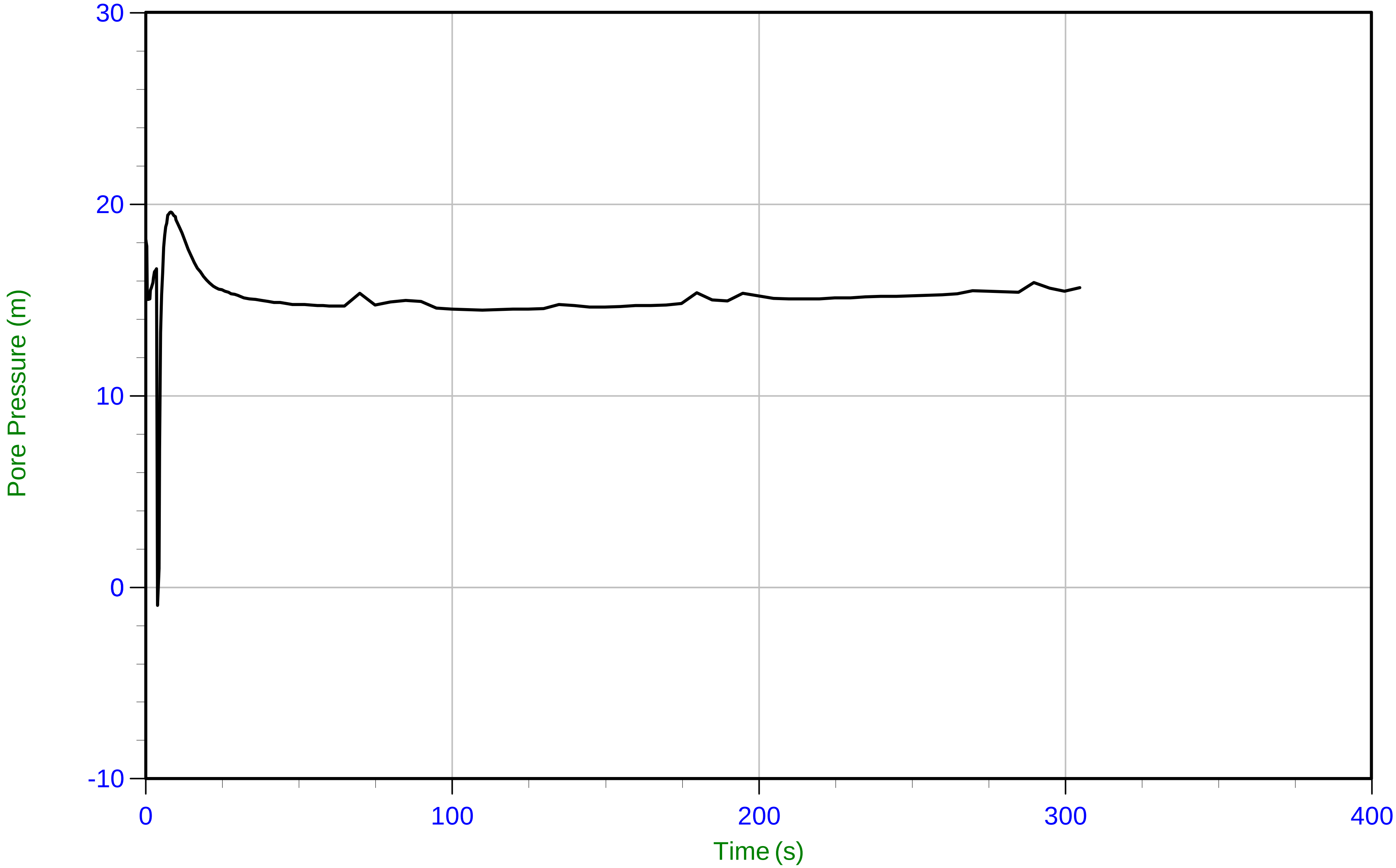
u Min: 12.7 m
u Max: 52.4 m
u Final: 13.0 m

WT: 6.946 m / 22.788 ft
Ueq: 13.0 m

Thurber

Job No: 23-05-26042
Date: 07/10/2023 14:57
Site: Poplar Rapids Bridge, ON

Sounding: SCPT23-107
Cone: 958:T1500F15U35 Area=15 cm²



Trace Summary:

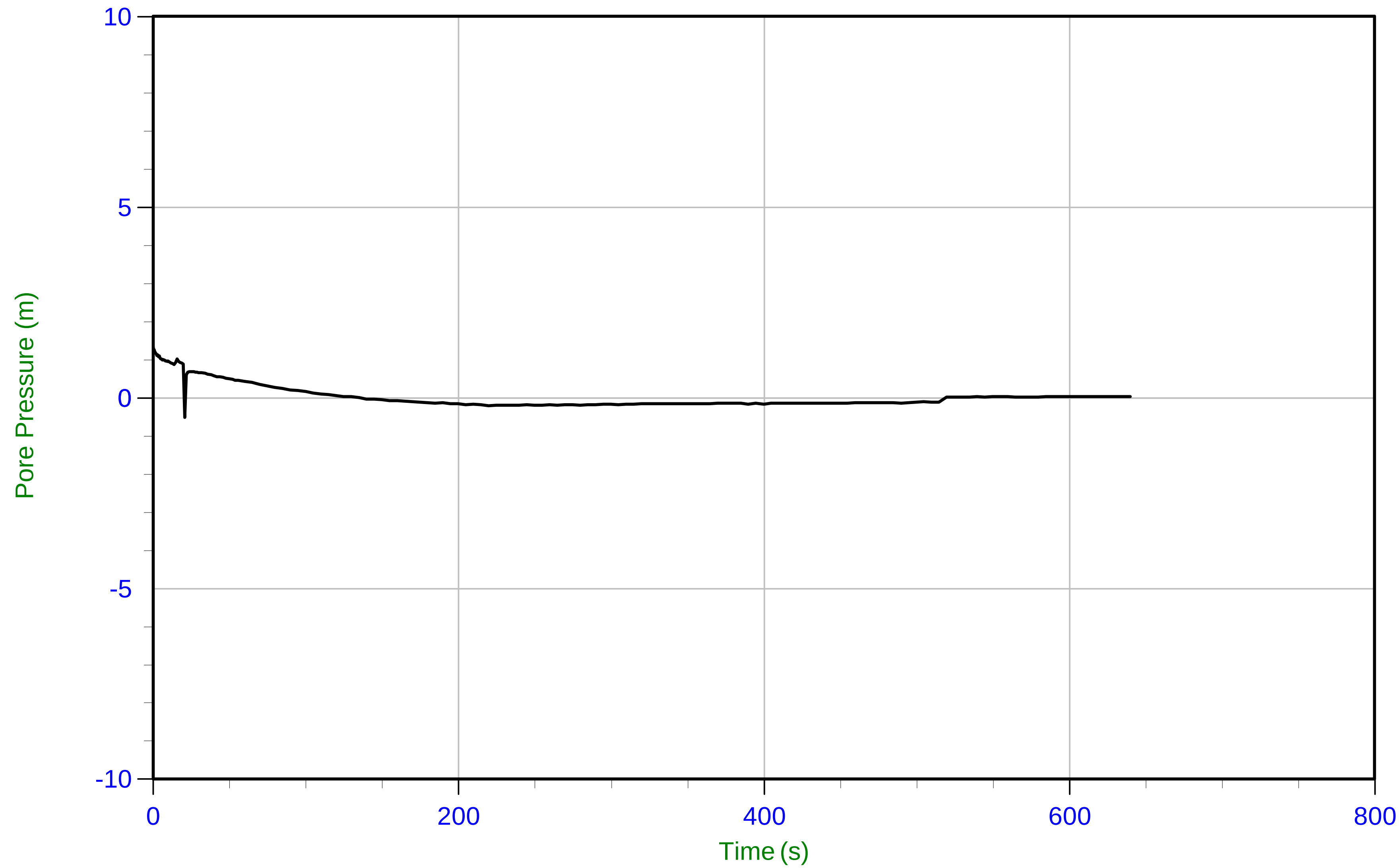
Filename: 23-05-26042_SP23-107.PPF2
Depth: 22.175 m / 72.752 ft
Duration: 305.0 s

u Min: -1.0 m
u Max: 19.6 m
u Final: 15.6 m

Thurber

Job No: 23-05-26042
Date: 07/10/2023 08:39
Site: Poplar Rapids Bridge, ON

Sounding: SCPT23-108
Cone: 958:T1500F15U35 Area=15 cm²



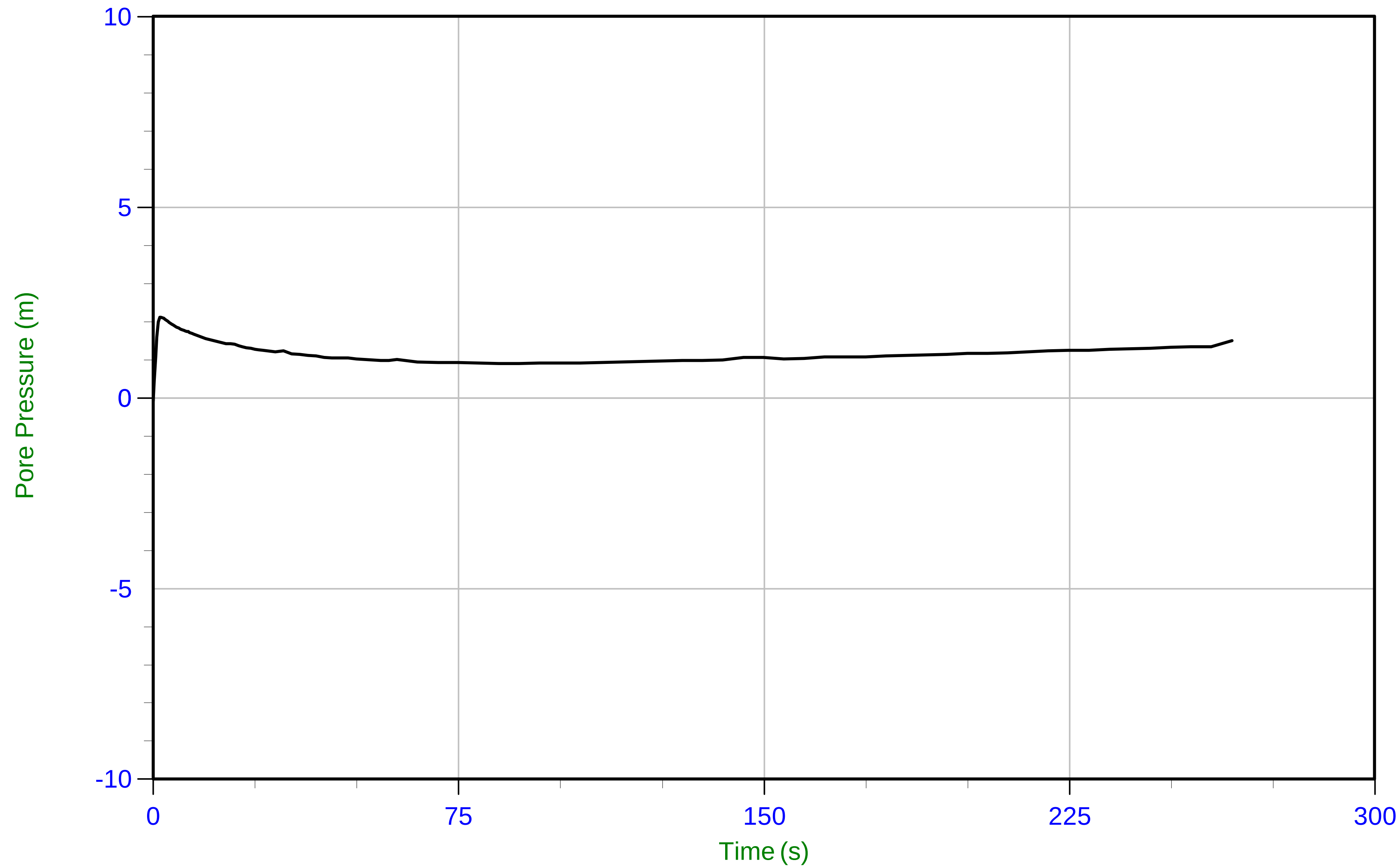
Trace Summary:

Filename: 23-05-26042_SP23-108.PPF2
Depth: 0.900 m / 2.953 ft
Duration: 640.1 s

u Min: -0.5 m
u Max: 1.3 m
u Final: 0.0 m

WT: 0.900 m / 2.953 ft
Ueq: 0.0 m

Thurber	Job No: 23-05-26042 Date: 07/10/2023 08:39 Site: Poplar Rapids Bridge, ON	Sounding: SCPT23-108 Cone: 958:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 23-05-26042_SP23-108.PPF2
Depth: 1.900 m / 6.234 ft
Duration: 265.0 s

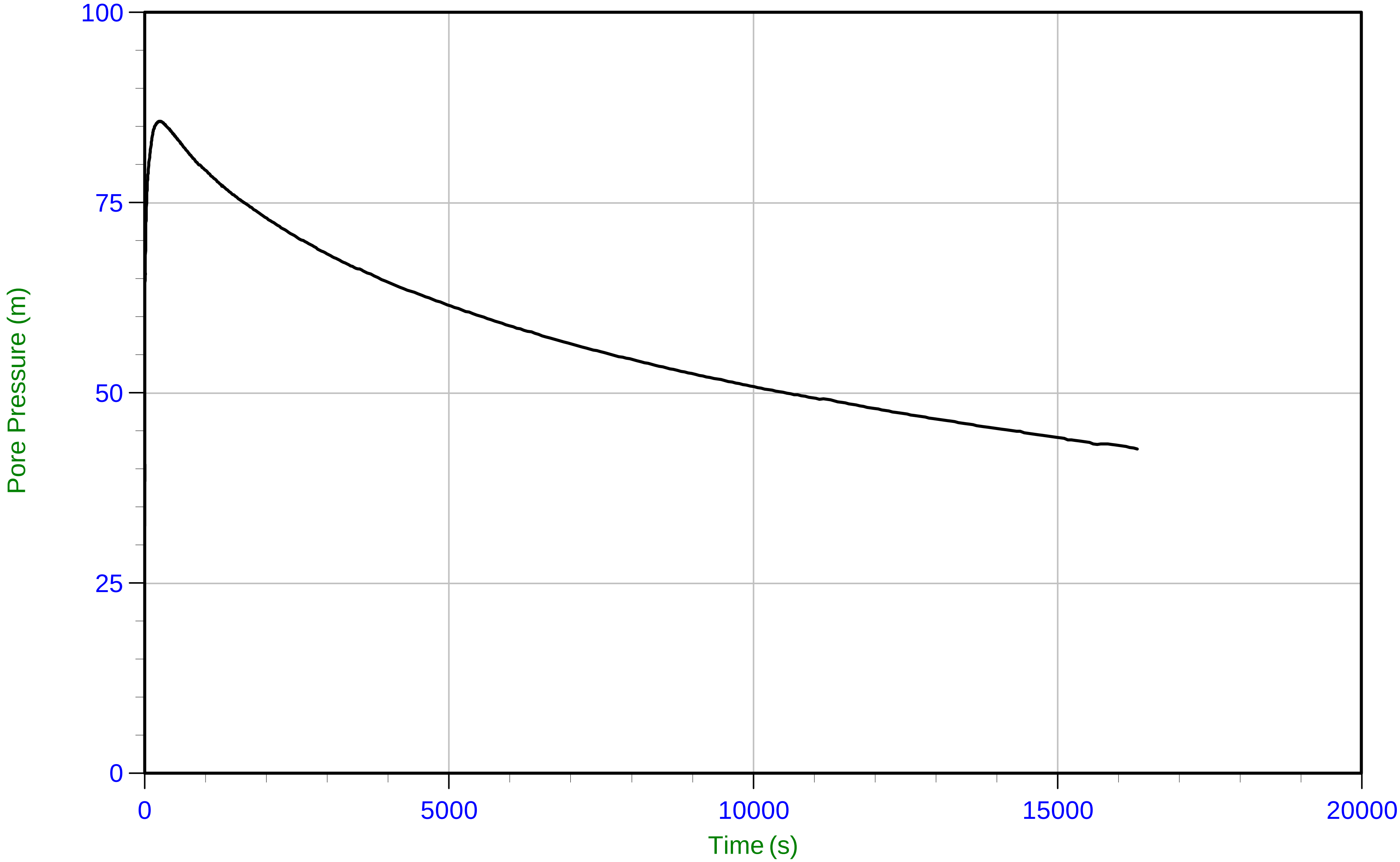
u Min: -0.1 m
u Max: 2.1 m
u Final: 1.5 m

ConeTec

Thurber

Job No: 23-05-26042
Date: 07/10/2023 08:39
Site: Poplar Rapids Bridge, ON

Sounding: SCPT23-108
Cone: 958:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 23-05-26042_SP23-108.PPF2
Depth: 6.150 m / 20.177 ft
Duration: 16320.0 s

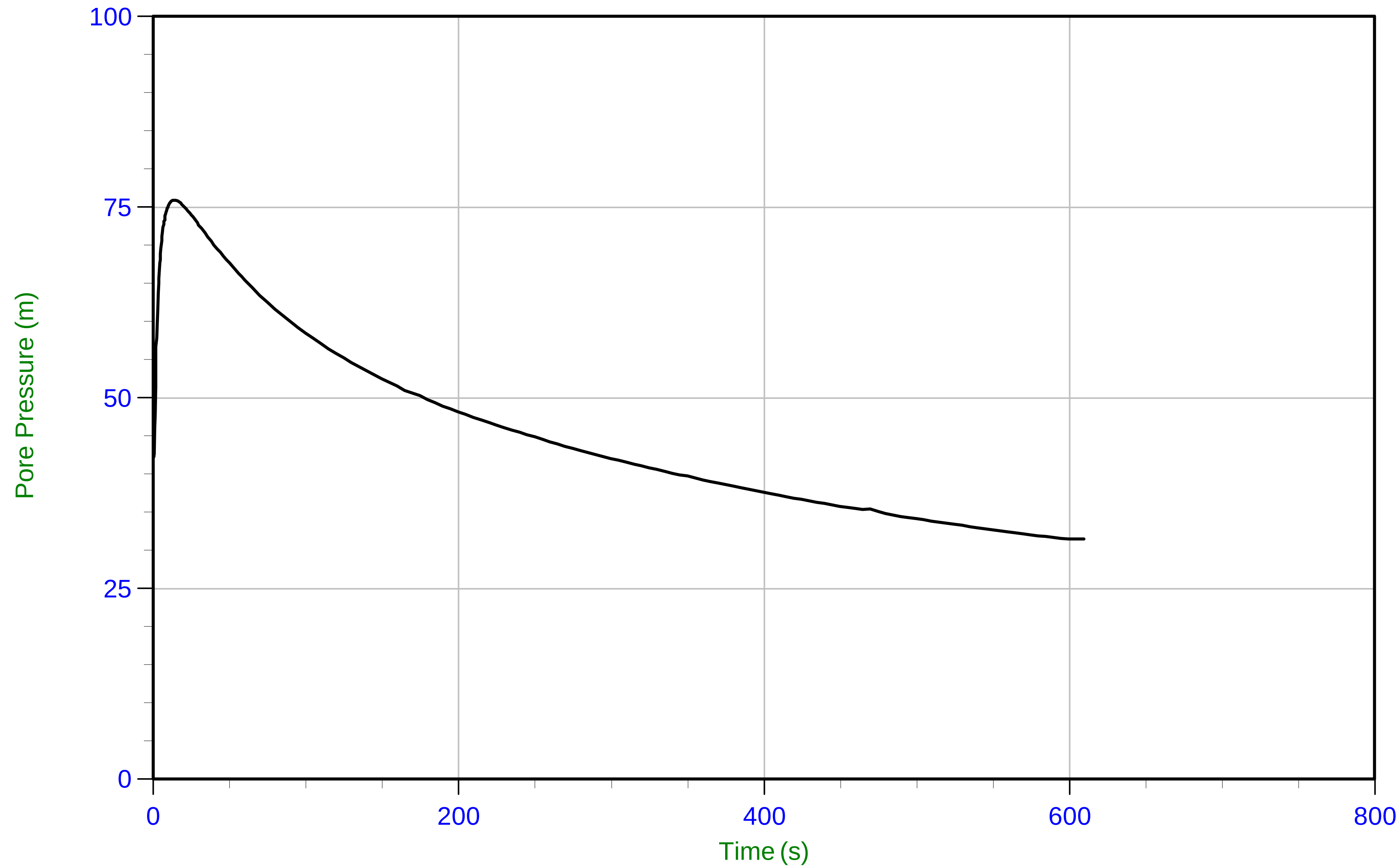
u Min: 32.5 m
u Max: 85.7 m
u Final: 42.6 m

ConeTec

Thurber

Job No: 23-05-26042
Date: 07/10/2023 08:39
Site: Poplar Rapids Bridge, ON

Sounding: SCPT23-108
Cone: 958:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 23-05-26042_SP23-108.PPF2
Depth: 8.250 m / 27.067 ft
Duration: 610.0 s

u Min: 31.5 m
u Max: 75.9 m
u Final: 31.5 m

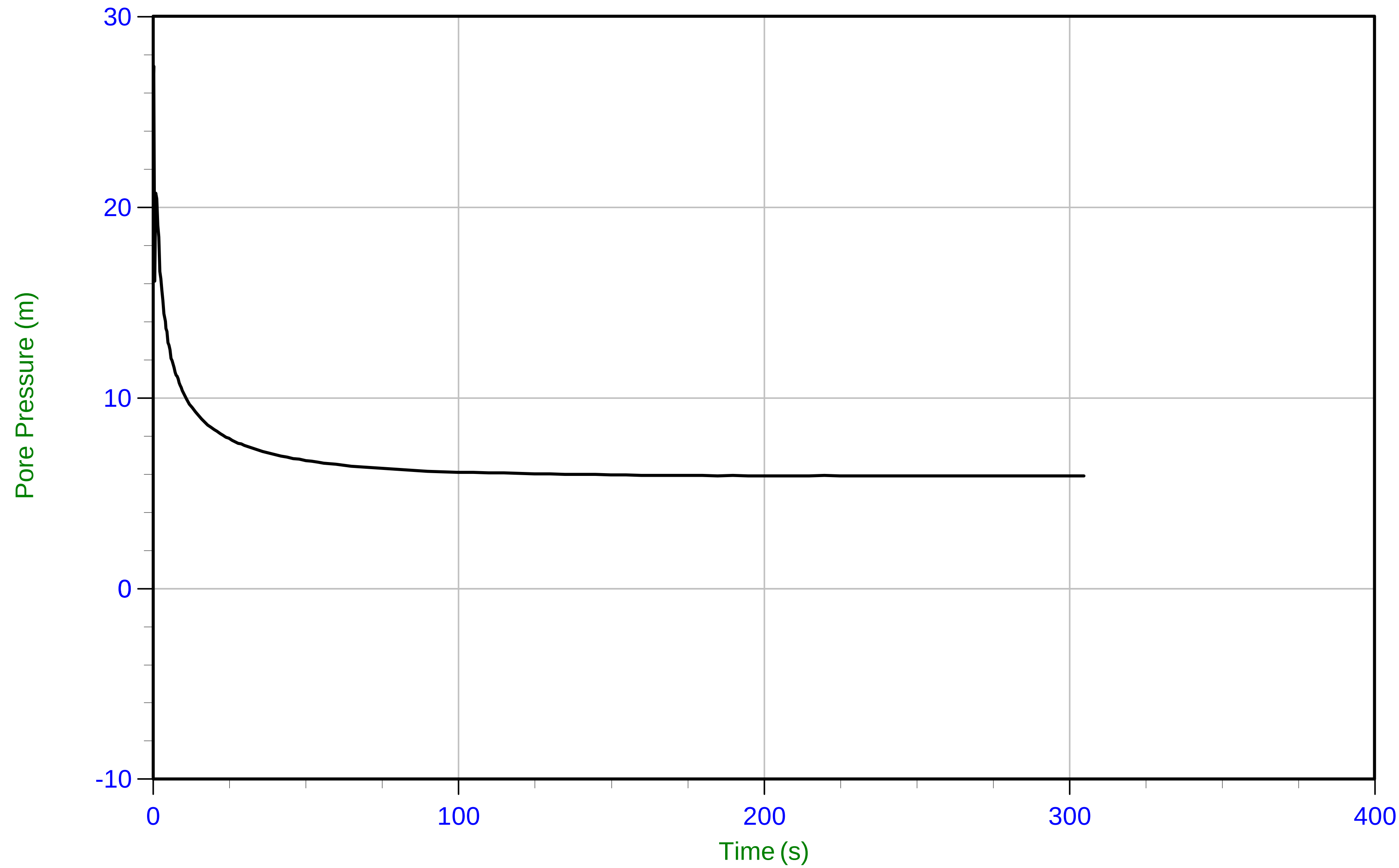
WT: 7.106 m / 23.313 ft
Ueq: 1.1 m
U(50): 38.53 m

T(50): 363.1 s
Ir: 100
Ch: 1.9 cm²/min

Thurber

Job No: 23-05-26042
Date: 07/10/2023 08:39
Site: Poplar Rapids Bridge, ON

Sounding: SCPT23-108
Cone: 958:T1500F15U35 Area=15 cm²



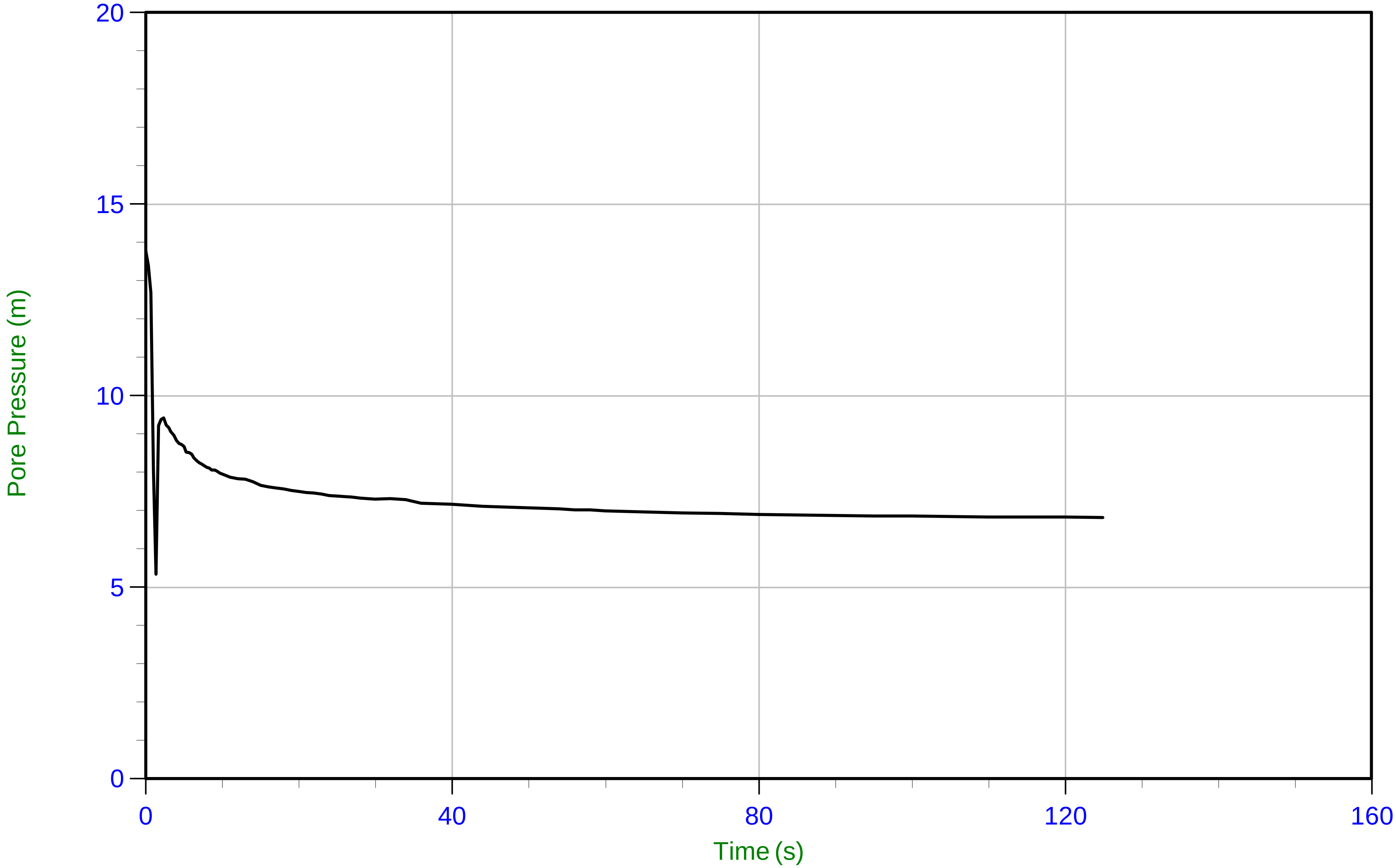
Trace Summary:

Filename: 23-05-26042_SP23-108.PPF2
Depth: 12.900 m / 42.322 ft
Duration: 305.0 s

u Min: 5.9 m
u Max: 27.4 m
u Final: 5.9 m

WT: 7.106 m / 23.313 ft
Ueq: 5.8 m

Thurber	Job No: 23-05-26042 Date: 07/10/2023 08:39 Site: Poplar Rapids Bridge, ON	Sounding: SCPT23-108 Cone: 958:T1500F15U35 Area=15 cm²



Trace Summary:

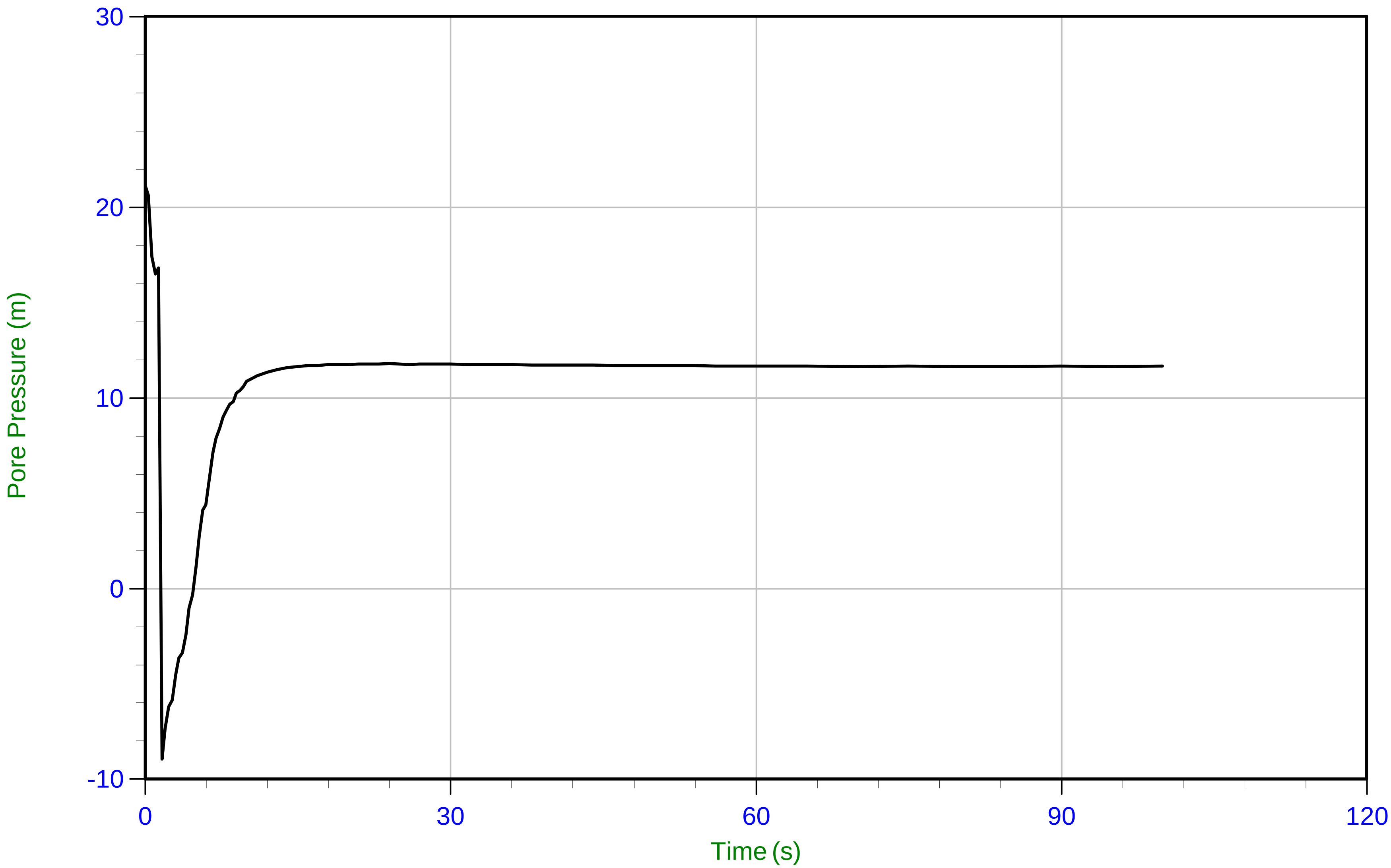
Filename: 23-05-26042_SP23-108.PPF2	u Min: 5.3 m	WT: 7.076 m / 23.215 ft
Depth: 13.900 m / 45.603 ft	u Max: 13.8 m	Ueq: 6.8 m
Duration: 125.0 s	u Final: 6.8 m	

CONETEC

Thurber

Job No: 23-05-26042
Date: 07/10/2023 08:39
Site: Poplar Rapids Bridge, ON

Sounding: SCPT23-108
Cone: 958:T1500F15U35 Area=15 cm²



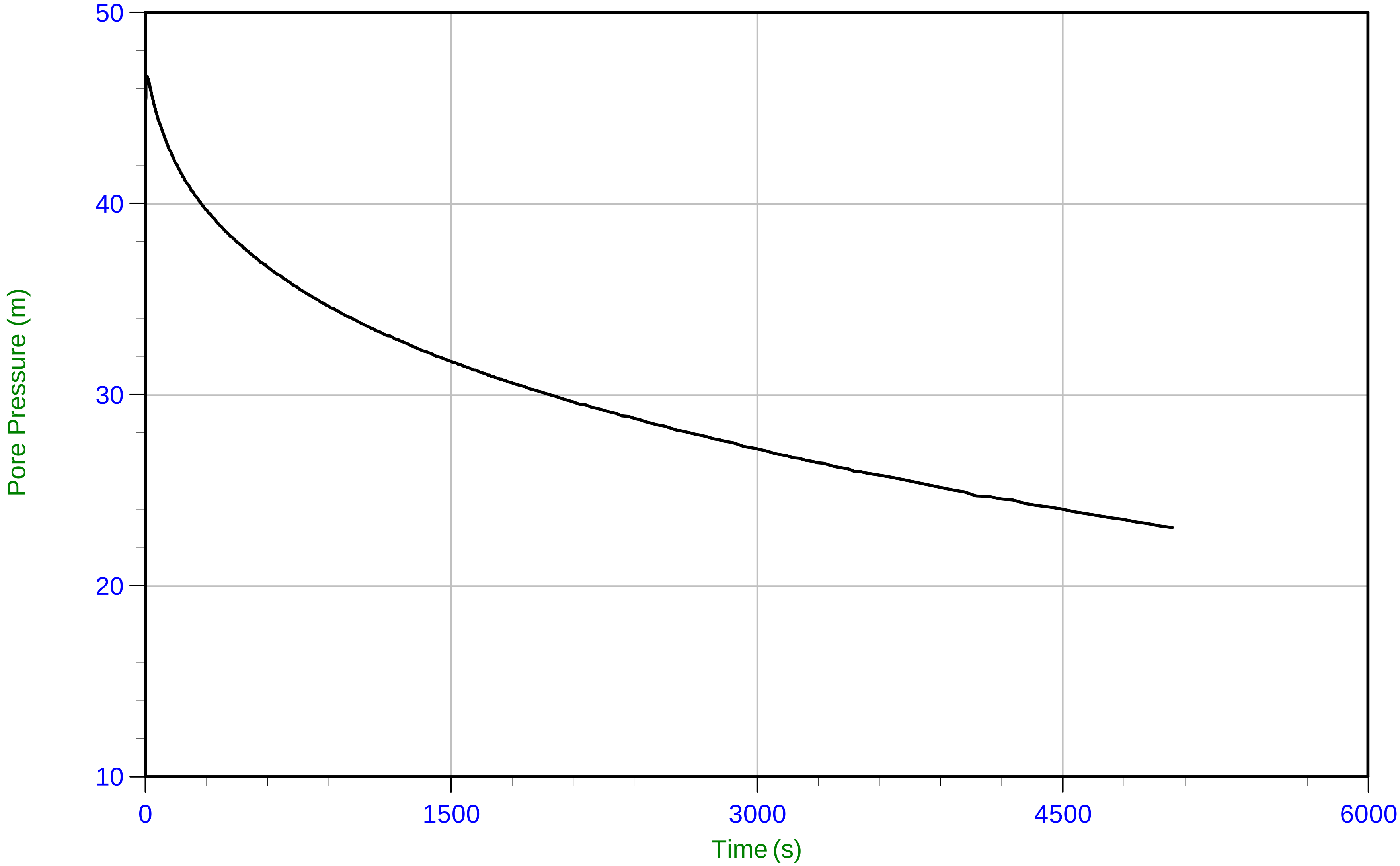
Trace Summary:

Filename: 23-05-26042_SP23-108.PPF2
Depth: 18.900 m / 62.007 ft
Duration: 100.0 s

u Min: -8.9 m
u Max: 21.1 m
u Final: 11.7 m

WT: 7.097 m / 23.284 ft
Ueq: 11.8 m

Thurber	Job No: 23-05-26042 Date: 07/11/2023 07:32 Site: Poplar Rapids Bridge, ON	Sounding: SCPT23-205 Cone: 765:T1500F15U35 Area=15 cm²



Trace Summary:

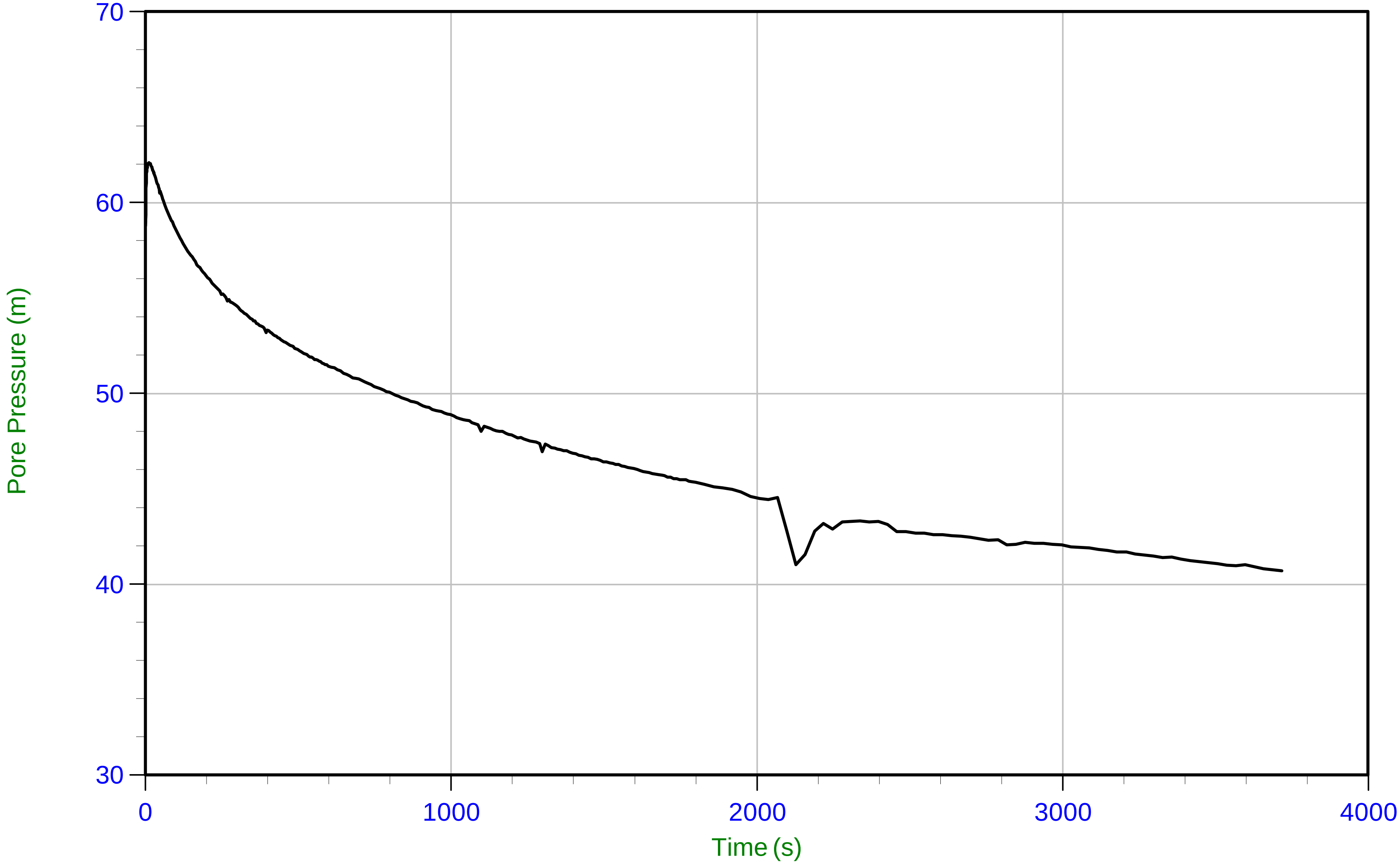
Filename: 23-05-26042_SP23-205.PPF2	u Min: 23.0 m	WT: 7.850 m / 25.754 ft	T(50): 4499.3 s
Depth: 9.150 m / 30.019 ft	u Max: 46.7 m	Ueq: 1.3 m	Ir: 100
Duration: 5040.0 s	u Final: 23.0 m	U(50): 23.98 m	Ch: 0.2 cm²/min

ConeTec

Thurber

Job No: 23-05-26042
Date: 07/11/2023 07:32
Site: Poplar Rapids Bridge, ON

Sounding: SCPT23-205
Cone: 765:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 23-05-26042_SP23-205.PPF2
Depth: 12.200 m / 40.026 ft
Duration: 3720.0 s

u Min: 40.7 m
u Max: 62.1 m
u Final: 40.7 m

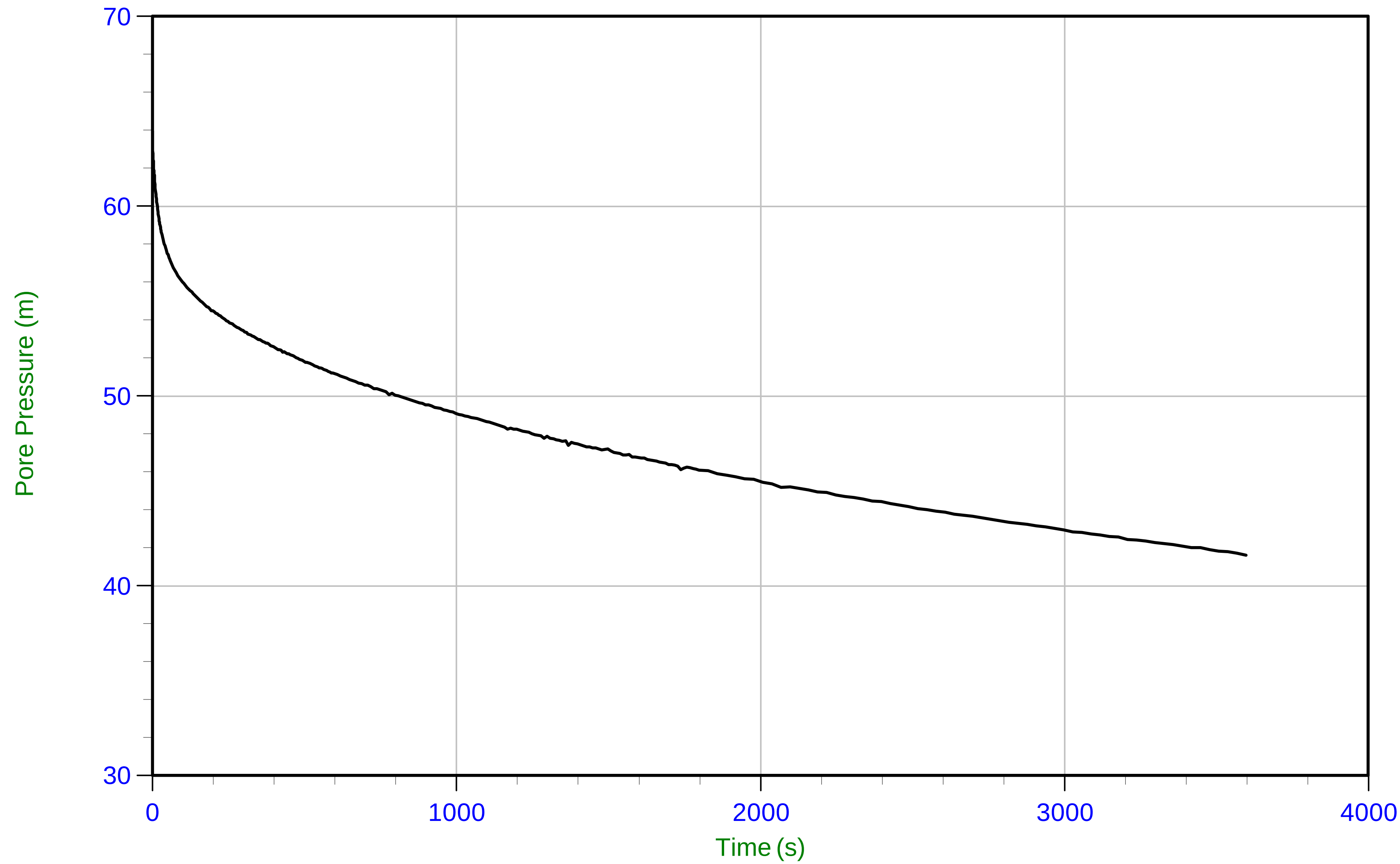
WT: 7.862 m / 25.794 ft
Ueq: 4.3 m

ConeTec

Thurber

Job No: 23-05-26042
Date: 07/11/2023 07:32
Site: Poplar Rapids Bridge, ON

Sounding: SCPT23-205
Cone: 765:T1500F15U35 Area=15 cm²



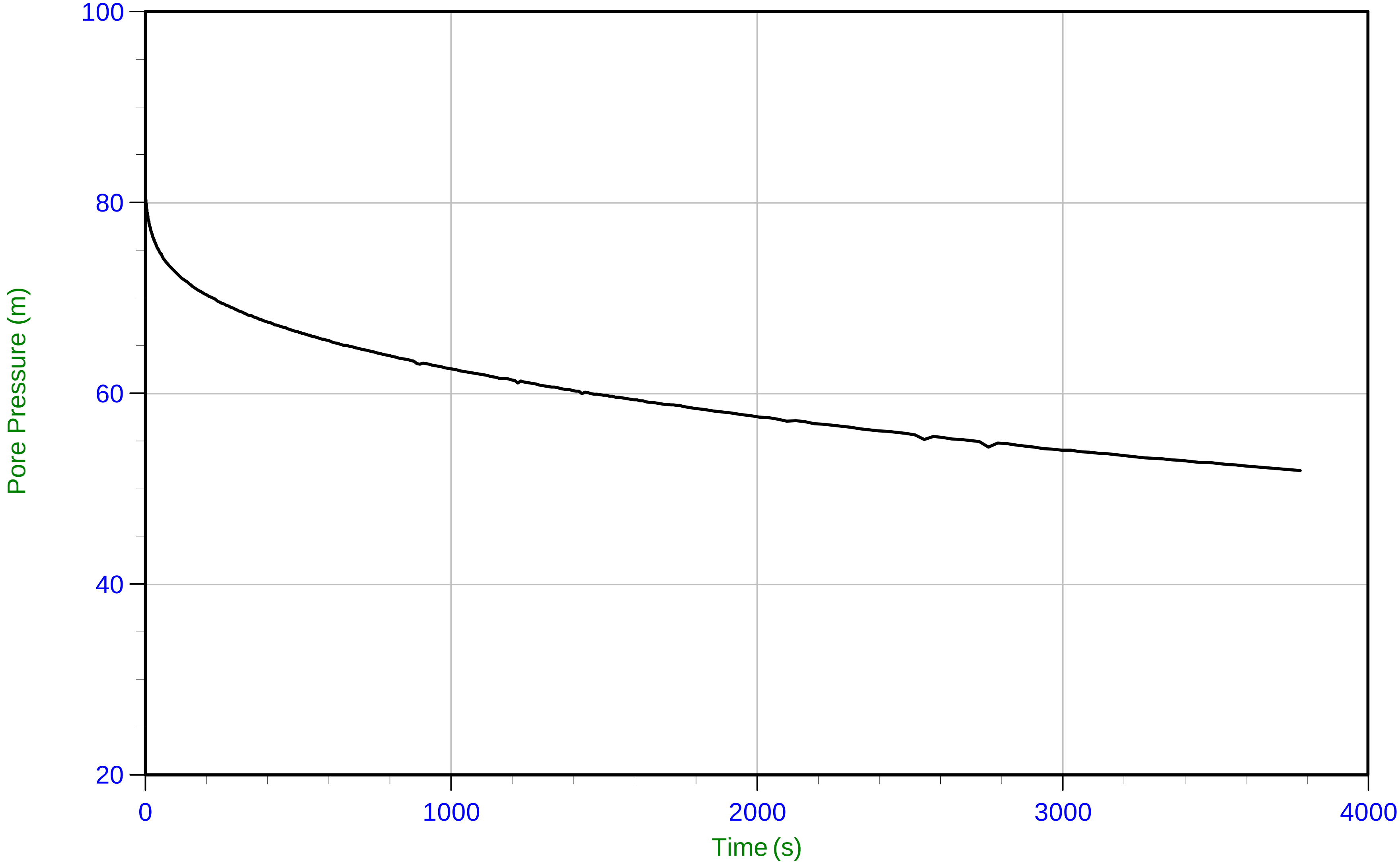
Trace Summary:

Filename: 23-05-26042_SP23-205.PPF2
Depth: 15.250 m / 50.032 ft
Duration: 3600.0 s

u Min: 41.6 m
u Max: 63.9 m
u Final: 41.6 m

WT: 7.862 m / 25.794 ft
Ueq: 7.4 m

Thurber	Job No: 23-05-26042 Date: 07/11/2023 07:32 Site: Poplar Rapids Bridge, ON	Sounding: SCPT23-205 Cone: 765:T1500F15U35 Area=15 cm²



Trace Summary:

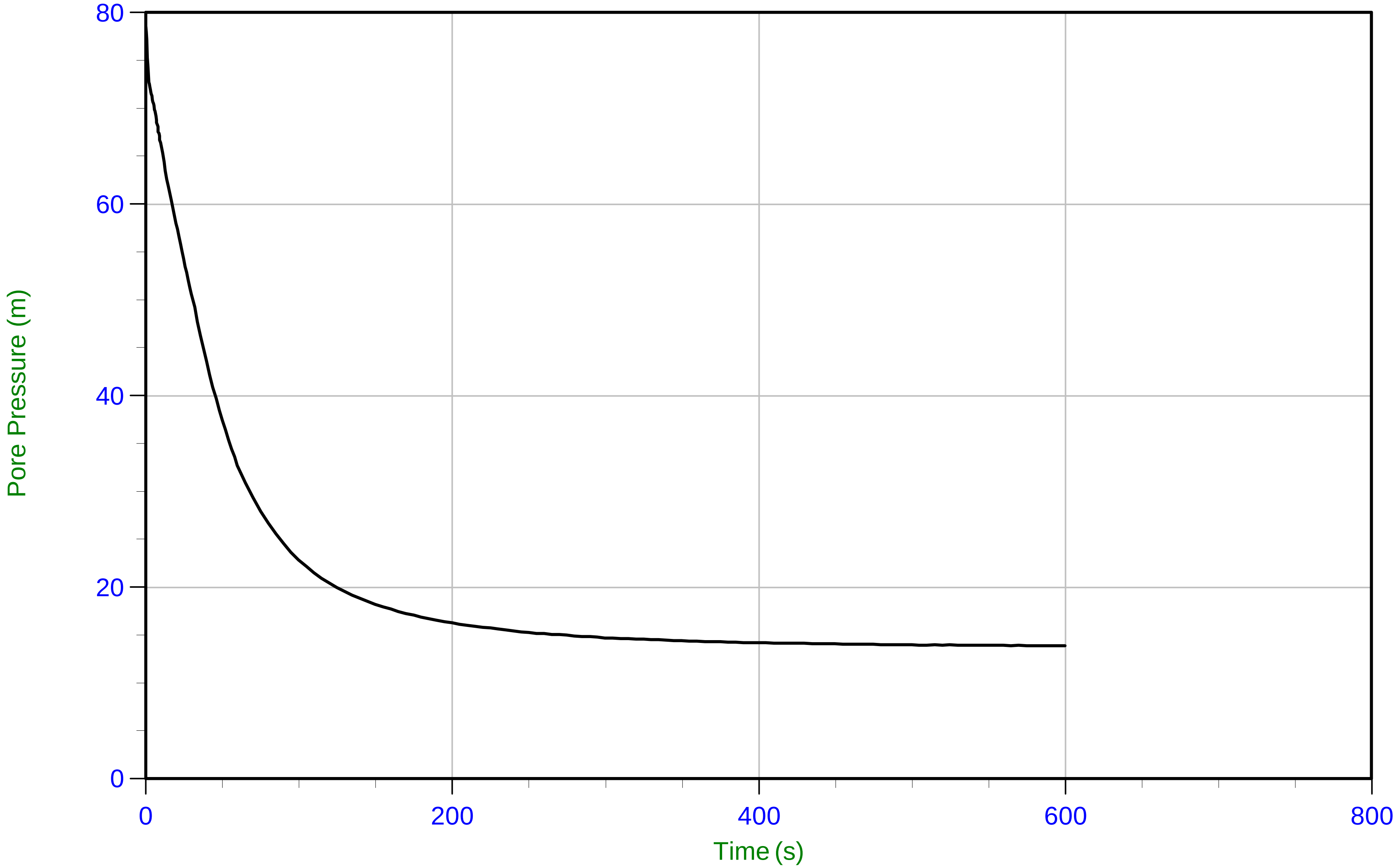
Filename: 23-05-26042_SP23-205.PPF2	u Min: 51.9 m	WT: 7.862 m / 25.794 ft
Depth: 18.300 m / 60.039 ft	u Max: 83.5 m	Ueq: 10.4 m
Duration: 3780.0 s	u Final: 51.9 m	

CONETEC

Thurber

Job No: 23-05-26042
Date: 07/11/2023 07:32
Site: Poplar Rapids Bridge, ON

Sounding: SCPT23-205
Cone: 765:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 23-05-26042_SP23-205.PPF2
Depth: 21.350 m / 70.045 ft
Duration: 600.0 s

u Min: 13.9 m
u Max: 78.5 m
u Final: 13.9 m

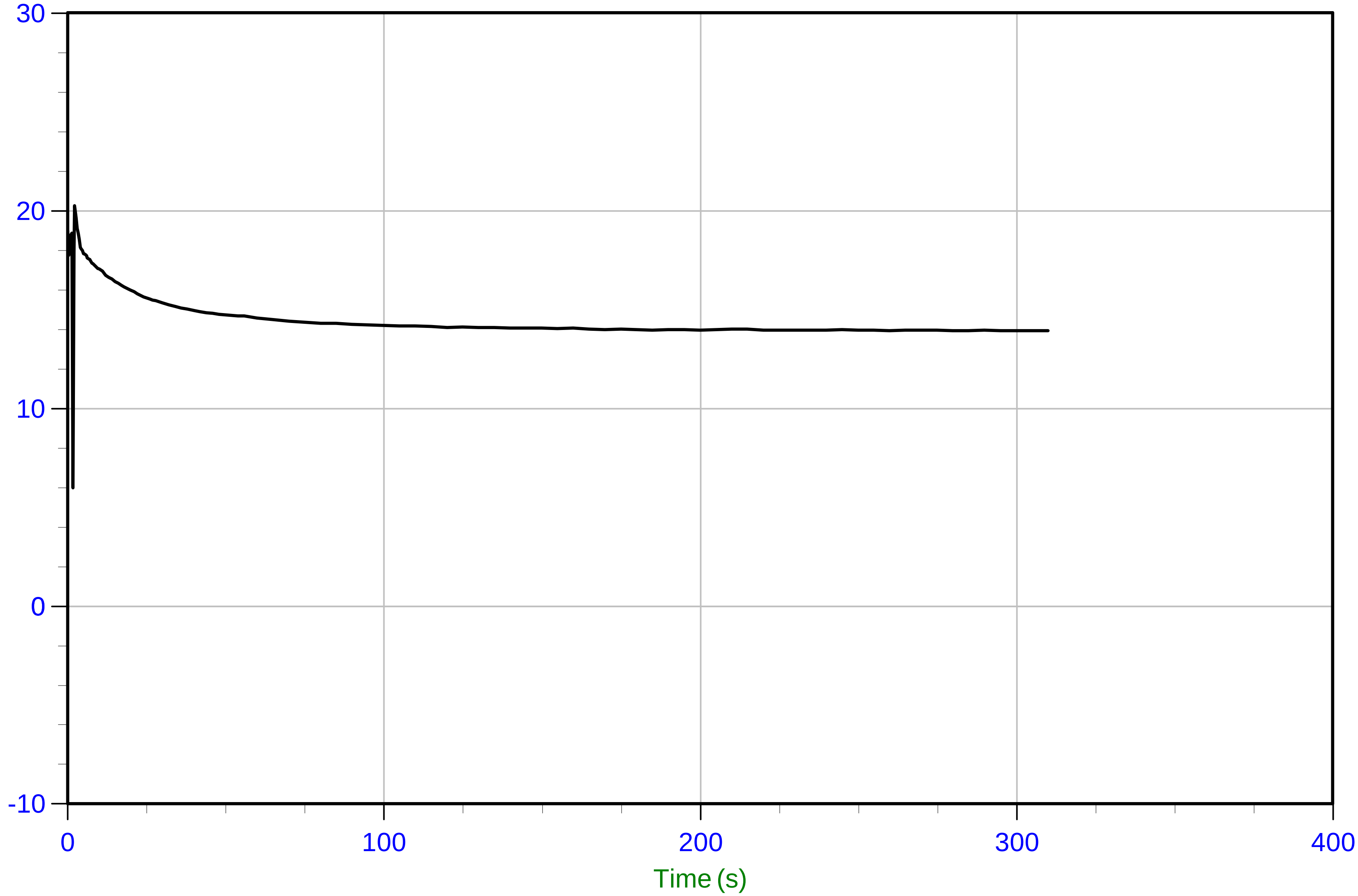
WT: 7.862 m / 25.794 ft
Ueq: 13.5 m
U(50): 46.01 m

T(50): 36.3 s
Ir: 100
Ch: 19.3 cm²/min

Thurber

Job No: 23-05-26042
Date: 07/11/2023 07:32
Site: Poplar Rapids Bridge, ON

Sounding: SCPT23-205
Cone: 765:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 23-05-26042_SP23-205.PPF2
Depth: 21.725 m / 71.275 ft
Duration: 310.0 s

u Min: 6.0 m
u Max: 20.3 m
u Final: 13.9 m

WT: 7.862 m / 25.794 ft
Ueq: 13.9 m
U(50): 17.06 m

T(50): 7.5 s
Ir: 100
Ch: 93.5 cm²/min

Description of Methods for Calculated CPT Geotechnical Parameters

CONETEC

CALCULATED CPT GEOTECHNICAL PARAMETERS

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

Revision SZW-Rev 18

Revised February 10, 2023
Prepared by Jim Greig, M.A.Sc, P.Eng (BC, AB, ON)

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 February 10, 2023.

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 calculations. Since all ConeTec cones have equal end area friction sleeves pore pressure corrections to sleeve friction, f_s, are not performed.

Corrected tip resistance: $q_t = q_c + (1\text{-}a) \cdot u_2$ (consistent units are required)

where: q_t is the corrected tip resistance
q_c is the recorded tip resistance
u_2 is the recorded dynamic pore pressure from 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.

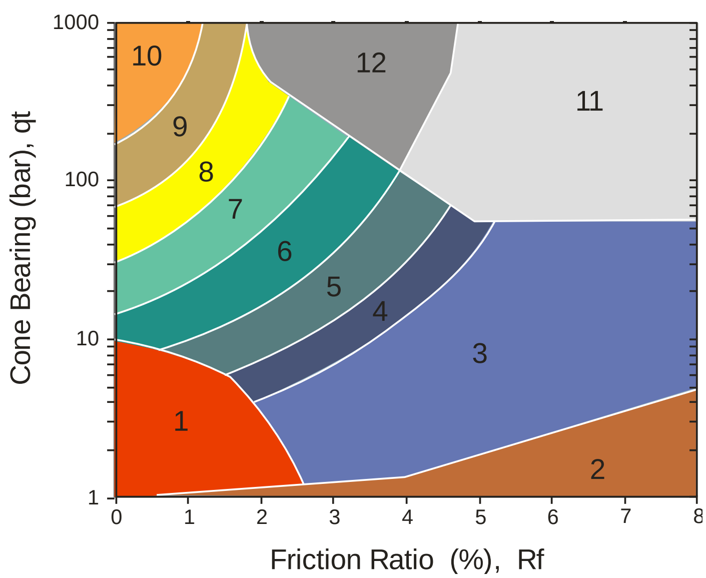
Effective vertical overburden stresses are calculated using the total stress and equilibrium pore pressure (u_{eq} or u_o) values derived from an assumed hydrostatic distribution of pore pressures below the water table or from a user defined equilibrium pore pressure profile (typically obtained from CPT dissipation tests) or a combination of the two. For over water projects the stress effects of the column of water above the mudline are taken into account as is the appropriate unit weight of water. How this is done depends on where the instruments are 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 from or driven by results based on material types as determined by the various soil behavior type charts depicted in Figures 1 through 6. 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) SBTn 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. Take 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 defined 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 1986 SBT Chart presented to CPT'10 by Robertson (2010b). It is known as the Updated non-normalized Soil Behavior Chart (also referred to as the Rev SBT Chart (PKR2010) in our output files). This chart was produced to be more in line with all post-1986 Robertson charts having the same 9 soil type zones, a $\log_{10}$ axis for friction ratio, R_f in this case, and a unitless tip resistance axis.

Figure 6 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. A green palette was selected for the dilative (desirable) side of the chart and a red palette for the contractive side of the chart.



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)

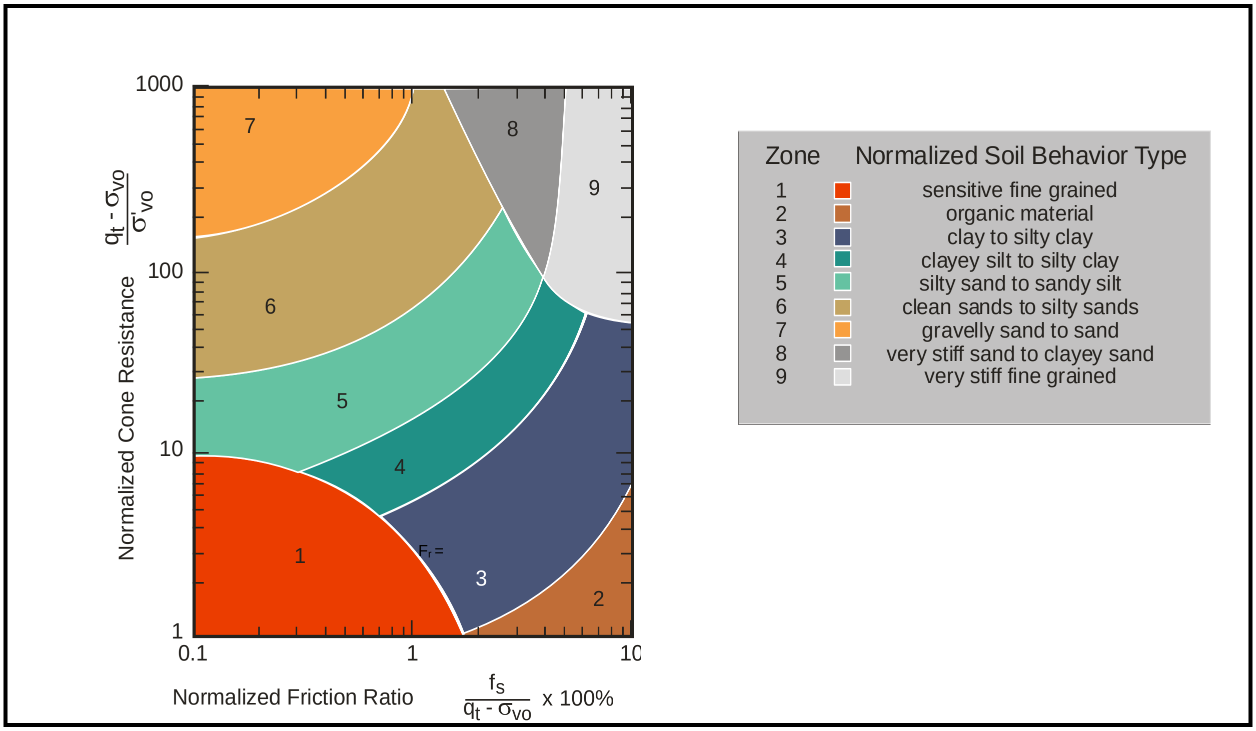


Figure 2. Normalized Soil Behavior Type Classification Chart (SBTn)

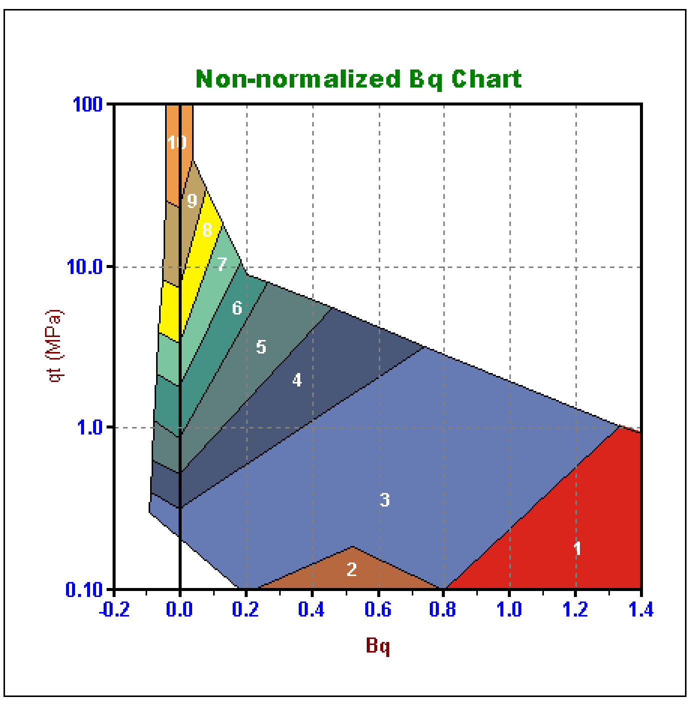


Figure 3a. Alternate Soil Behavior Type Chart (SBT Bq): q_t - B_q

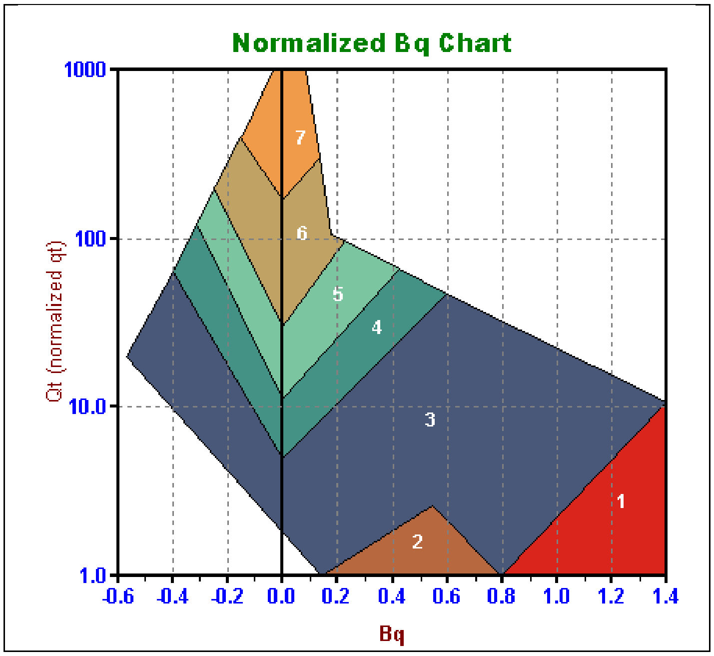


Figure 3b. Alternate Soil Behavior Type Charts (SBT Bqn): Q_t-B_q

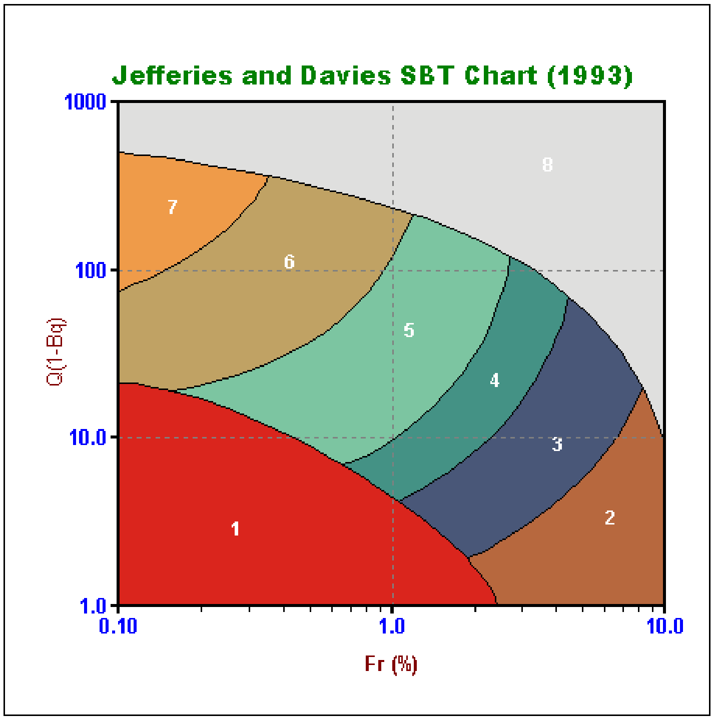


Figure 3c. Alternate Soil Behavior Type Charts: $Q(1\text{-}B_q)$ - F_r

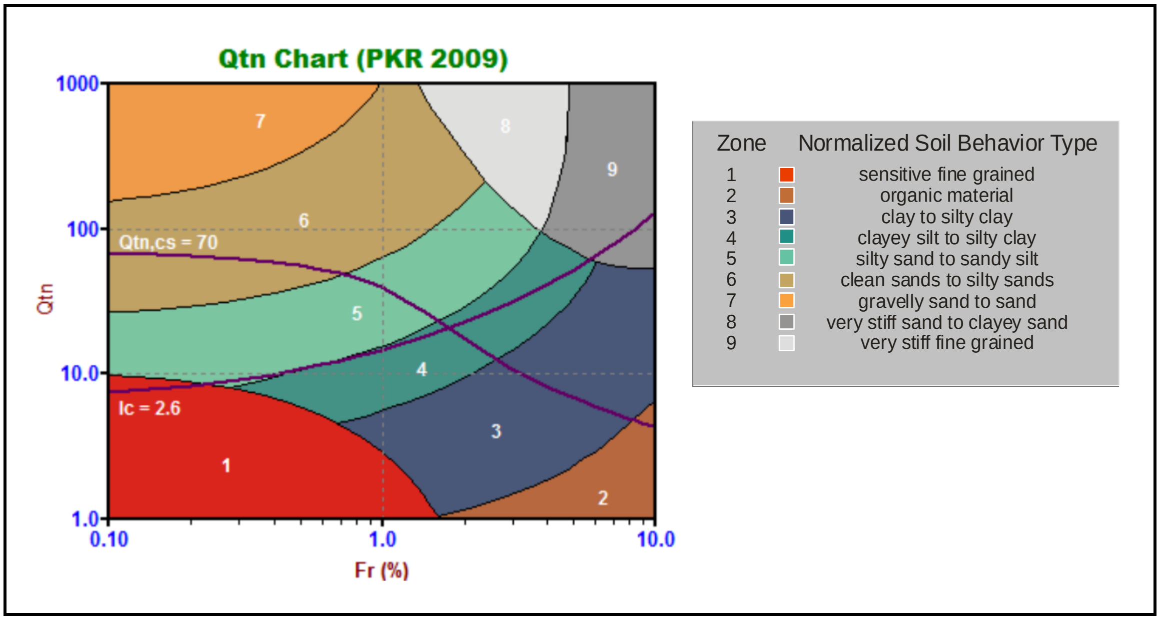


Figure 4. Normalized Soil Behavior Type Chart using Q_{tn} (SBT Qtn)

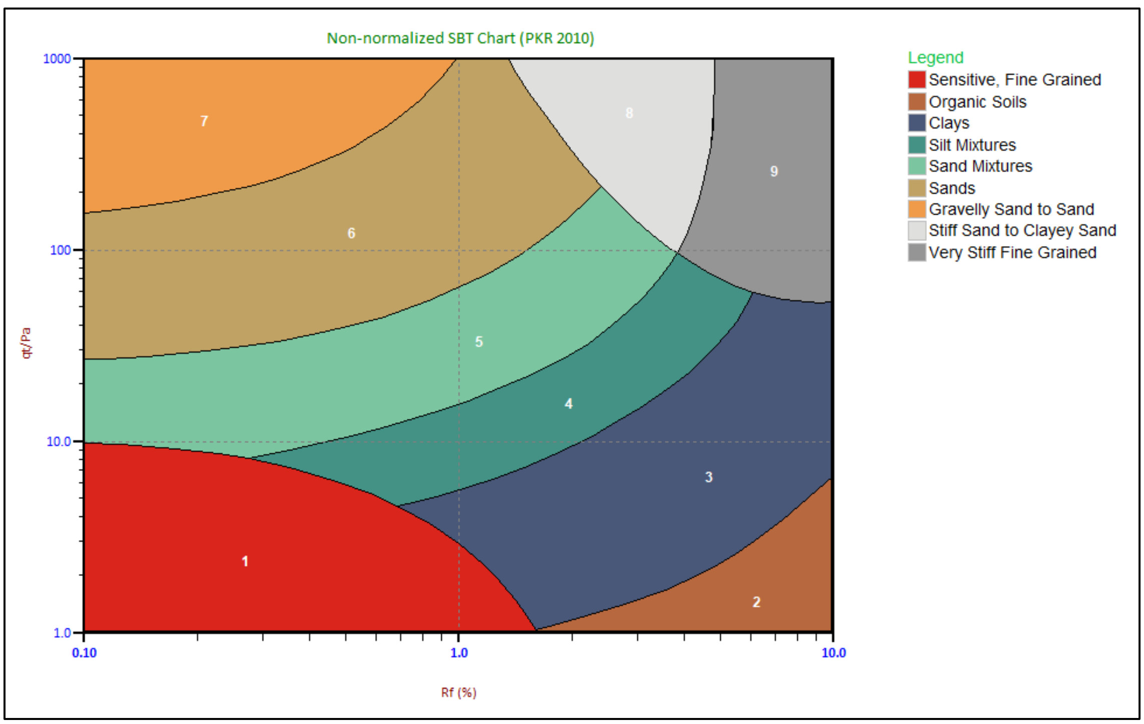


Figure 5. Non-normalized Soil Behavior Type Chart (2010)

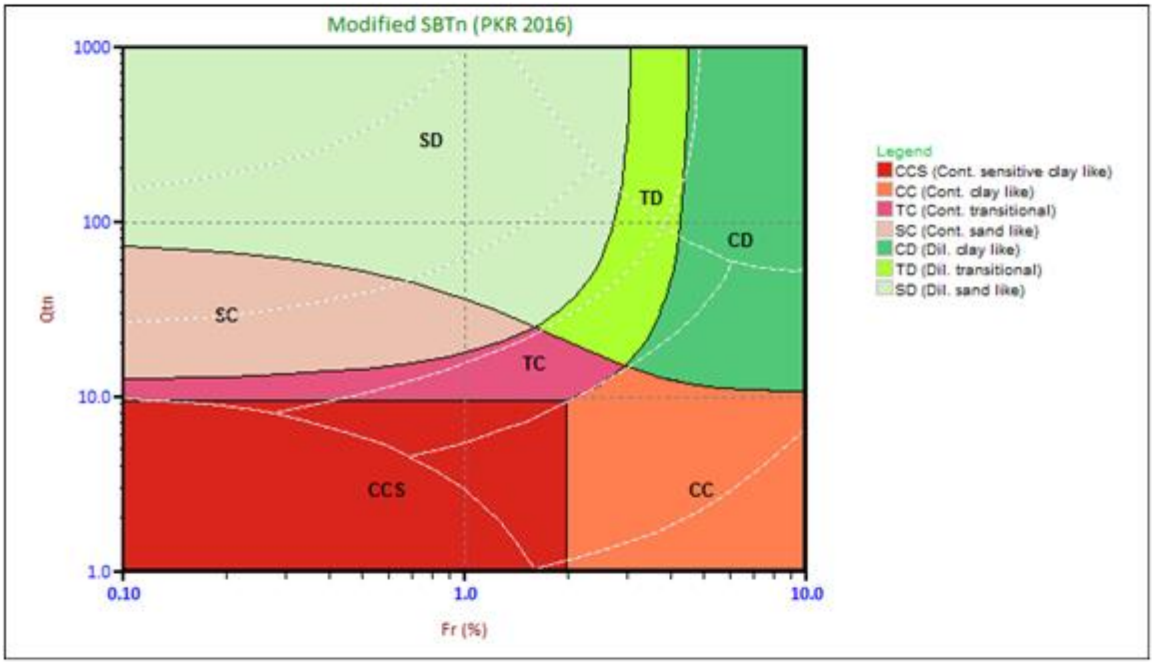


Figure 6. Modified SBTn Behavior Based Chart

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, we recommend that the user 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 in an undrained manner (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 Tables 1and 1a may be included in the output files delivered with this report.

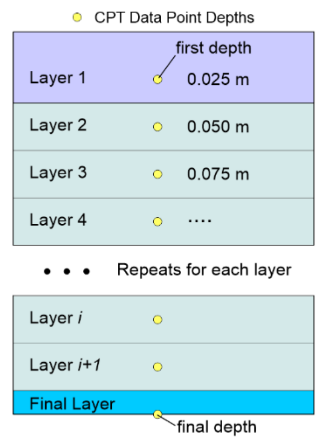
The output files are typically provided in Microsoft Excel XLS, XLSX or CSV format. The ConeTec software has several options for output depending on the number or types of calculated parameters desired or those specifically contracted for by the client. Each output file is named using the original file base name (from the .COR file) followed by a three or four character indicator of the output set selected (e.g. BSC, TBL, NLI, NL2, IFI, IFI2, IFI3) 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

Reference Notes: CK* - Common Knowledge, U* - Unpublished

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 is based on the sounding collar elevation supplied by the client or through a site survey In Sweden a variation of elevation is used where the elevation increases with depth. We refer to this as inverse elevation.	Elevation = Collar Elevation – Depth InverseElevation = Collar Elevation + Depth	CK* N/A
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: $q_t = q_c + (1 - a) \cdot u_2$ Averaged q_t is not calculated using the average q_c and averaged u values. Averaged q_t is based on the average of the q_t values calculated at each data point.	$Avgqt = \frac{1}{n}\sum_{i=1}^{n} q_t$ *n=1 when calculations are done at each point*	1
Avg fs	Averaged sleeve friction (f_s) No pore pressure corrections are applied to 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: $R_f = 100\% \cdot \frac{fs}{q_t}$	$AvgRf = 100\% \cdot \frac{Avgfs}{Avgqt}$ *not an average of individual R_f values*	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*
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)	$AvgTemp = \frac{1}{n}\sum_{i=1}^{n} Temperature_i$ *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 using Q_t, now referred to as Q_{t1})	See Figure 2	2, 5

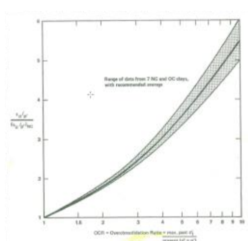
Calculated Parameter	Description	Equation	Ref
SBT-Bq	Non-normalized Soil Behavior type based on non-normalized tip resistance and the B_q parameter	See Figure 3a	1, 2, 5
SBT-Bqn	Normalized Soil Behavior type based on normalized tip resistance (Q_t, now called Q_{t1}) and the B_q parameter	See Figure 3b	2, 5
SBT-JandD	Soil Behavior Type as defined by Jeffries and Davies	See Figure 3c	7
SBT Qtn	Soil Behavior Type as defined by Robertson (2009) using a variable stress ratio exponent for normalization based on $I_{c\ (PKR\ 2009)}$	See Figure 4	15
Modified Non-normalized SBT Chart SBT (PKR2010)	This is a revised version of the simple 1986 non-normalized SBT chart (presented at CPT '10). The revised version has been reduced from 12 zones to 9 zones to be similar to the normalized Robertson charts. Other updates include a dimensionless tip resistance normalized to atmospheric pressure, q_t/P_a, on the vertical axis and a log scale for non-normalized friction ratio, R_f, along the horizontal axis.	See Figure 5	33
Modified SBTn (contractive /dilative)	Modified SBTn chart as defined by Robertson (2016) indicating zones of contractive/dilative behavior. Note that ConeTec displays the chart with colors different from Robertson. ConeTec's colors were chosen to avoid confusion with soil type descriptions.	See Figure 6	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) values based on Robertson updated non-normalized Soil Behavior Type Chart (2010b) 6) Mayne f_s (sleeve friction) method 7) Robertson and Cabal 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, 33

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 pressures are determined from one of the following user selectable options: 1) hydrostatic below the 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 the hydrostatic option: $u_{eq} = \gamma_w \cdot (D - D_{wt})$ where u_{eq} is equilibrium pore pressure γ_w is the 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 ranging from 1.7 to 2.0)* P_a *is atmospheric pressure (100 kPa)*	4, 12

Calculated Parameter	Description	Equation	Ref
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)* *Robertson and Wride define C_q to be the same as C_n. The Olson definition above is used in the program.*	3, 12
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
$(N_1)_{60}$	SPT N_{60} value corrected for overburden pressure.	$(N_1)_{60} = C_n \bullet N_{60}$	4
$N_{60}I_c$	SPT N_{60} values based on the I_c parameter, as defined by Robertson and Wride 1998 (3), or by Robertson 2009 (15).	$(q_t/P_a)/ N_{60} = 8.5\ (1 - I_c/4.6)$ $(q_t/P_a)/ N_{60} = 10^{(1.1268 - 0.2817Ic)}$ P_a being atmospheric pressure	3, 5 15, 31
$(N_1)_{60}I_c$	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
S_u or S_u (N_{kt})	Undrained shear strength based on q_t S_u factor N_{kt} is user selectable.	$Su = \dfrac{qt - \sigma_v}{N_{kt}}$	1, 5
S_u or S_u (N_{du}) or S_u ($N_{\Delta u}$)	Undrained shear strength based on pore pressure S_u factor $N_{\Delta u}$ is user selectable.	$Su = \dfrac{u_2 - u_{eq}}{N_{\Delta u}}$	1, 5
D_r	Relative Density determined from one of the following user selectable options: 1) Ticino Sand 2) Hokksund Sand 3) Schmertmann (1978) 4) Jamiolkowski (1985) - All Sands 5) Jamiolkowski et al (2003) (various compressibilities, K_o)	See reference (methods 1 through 4) Jamiolkowski et al (2003) reference	5 14
PHI ϕ	Friction Angle determined from one of the following user selectable options (methods 1 through 4 are for sands and method 5 is for silts and clays): 1) Campanella and Robertson 2) Durgunoglu and Mitchel 3) Janbu 4) Kulhawy and Mayne 5) NTH method (clays and silts)	See appropriate reference	5 5 5 11 23
Delta U/q_t $\Delta u/q_t$ du/q_t	Differential pore pressure ratio (older parameter used before B_q was established)	$= \dfrac{\Delta u}{qt}$ *where:* $\Delta u = u - u_{eq}$ *and* *u = dynamic pore pressure* *u_{eq} = equilibrium pore pressure*	39

Calculated Parameter	Description	Equation	Ref
B_q	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 q_t or qtNet	Net tip resistance (used in many subsequent correlations)	$qt - \sigma_v$	36
q_e or qE or q_E	Effective tip resistance (using the dynamic pore pressure u_2 and not equilibrium pore pressure)	$q_t - u_2$	36
qeNorm	Normalized effective tip resistance	$\frac{qt - u_2}{\sigma'_v}$	36
Q_t or Norm: Qt or Q_{t1}	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}. This parameter was renamed to Q_{t1} in Robertson, 2009. Without normalization limits this parameter calculates to very high unrealistic values at low stresses.	$Qt = \frac{qt - \sigma_v}{\sigma'_v}$	2, 5, 15
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
$Q(1-B_q)$ $Q(1-B_q) + 1$	$Q(1-B_q)$ grouping as suggested by Jefferies and Davies for their classification chart and the establishment of their I_c parameter. Later papers added the +1 term to the equation.	$Q \cdot (1 - Bq)$ $Q \cdot (1 - Bq) + 1$ *where Bq is defined as above and Q is the same as the normalized tip resistance,* Q_{t1}*, defined above*	6, 7, 34
q_{c1}	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: P_a = atmospheric pressure	21
q_{c1} (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: P_a = atmospheric pressure	5
q_{c1} (C_n)	Normalized tip resistance, q_{c1}, based on C_n (this method has stress units)	$q_{c1}(Cn) = C_n * q_t$	5, 12
q_{c1} (C_q)	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
q_{c1n}	normalized tip resistance, q_{c1n}, using a variable stress ratio exponent, n (where n=0.0, 0.70, or 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

Calculated Parameter	Description	Equation	Ref
I_c or Ic (RW1998)	Soil Behavior Type Index as defined by Robertson and Wride (1997, 1998) for estimating grain size characteristics and providing smooth gradational changes across the SBTn chart. Ic(RW1998) is different from that of Jefferies and Davies (7) and is different from Ic(PKR2009).	$I_c = [(3.47 - \log_{10}Q)^2 + (\log_{10} Fr + 1.22)^2]^{0.5}$ *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 has the following distinct values:* *0.5, 0.75 and 1.0* *and is determined in an iterative manner based on the resulting* I_c *in each iteration* *Note that NCEER replaced 0.75 with 0.70*	3, 4, 5 10
I_c (PKR 2009)	Soil Behavior Type Index, I_c (PKR 2009) is based on a variable stress ratio exponent n, which itself is based on I_c (PKR 2009). An iterative calculation is required to determine I_c (PKR 2009) and its corresponding n (PKR 2009).	I_c *(PKR 2009) =* $[(3.47 - \log_{10}Q_{tn})^2 + (1.22 + \log_{10}F_r)^2]^{0.5}$	15
n (PKR 2009)	Stress ratio exponent n, based on I_c (PKR 2009). An iterative calculation is required to determine n (PKR 2009) and its corresponding I_c (PKR 2009).	*n (PKR 2009) =* $0.381 (I_c) + 0.05 (\sigma_v'/P_a) - 0.15$	15
Q_{tn} (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 Q_{tn} (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
CD	The contractive / dilative boundary on Robertson's Modified SBTn (contractive/dilative) Chart shown in Figure 6 above. The boundary is marked as CD = 70 on the chart in the relevant paper. Similar to the $Q_{tn,cs}$ = 70 line in Figure 4.	$CD = 70 = (Q_{tn} - 11) (1 + 0.06F_r)^{17}$ lower bound of CD = 60: $CD = 60 = (Q_{tn} - 9.5) (1 + 0.06F_r)^{17}$	30

Calculated Parameter	Description	Equation	Ref
I_B	Hyperbolic fit defining the boundary between SBT soil types proposed by Schneider as a better fit than the I_c circles. I_B = 32 represents the boundary for most sand like soils. I_B = 22 represents the upper boundary for most clay like soils. The region between I_B=22 and I_B=32 is the "transitional soil" zone.	$I_B = 100\ (Q_{tn} + 10) / (70 + Q_{tn}\ F_r)$	30
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) This method uses mean normal stresses based on a uniform value of K_0 or a calculated K_0 using methods described elsewhere in this document	See reference	6, 8
Yield Stress σ_p'	Yield stress is calculated using the following methods 1) General method 2) 1st order approximation using q_tNet (clays) 3) 1st order approximation using Δu_2 (clays) 4) 1st order approximation using q_e (clays) 5) Based on Vs	All stresses in kPa 1) $\sigma_p' = 0.33\cdot(q_t - \sigma_v)^{m'}\ (\sigma_{atm}/100)^{1-m'}$ where $m' = 1 - \frac{0.28}{1+(I_c/2.65)^{25}}$ 2) $\sigma_p' = 0.33\cdot(q_t - \sigma_v)$ 3) $\sigma_p' = 0.54\cdot(\Delta u_2)$ $\Delta u_2 = u_2 - u_0$ 4) $\sigma_p' = 0.60\cdot(q_t - u_2)$ 5) $\sigma_p' = (Vs/4.59)^{1.47}$	19 20 20 20 18
OCR OCR(JS1978) YSR(Mayne2014) YSR (qtNet) YSR (deltaU) YSR (qe) YSR (Vs) OCR (PKR2015)	Over Consolidation Ratio based on 1) Schmertmann (1978) method involving a plot plot of S_u/σ_v' /($S_u/\sigma_v')_{NC}$ and OCR  2) based on Yield stresses described above 3) approximate version based on qtNet 4) approximate version based on Δu 5) approximate version based on effective tip, q_e 6) approximate version based on shear wave velocity, V_s and σ_v' 7) based on Qt	1) requires a user defined value for NC Su/P_c' ratio 2 through 5) *based on yield stresses* 6) YSR (Vs) = $\sigma_p'(Vs) / \sigma_v'$ 7) OCR = $0.25\cdot(Qt)^{1.25}$	9 19 20 20 20 18 32
Es/qt	Intermediate parameter for calculating Young's Modulus, E, in sands. It is the Y axis of the reference chart. Note that Figured 5.59 from reference 5, Lunne, Robertson and Powell, (LRP) has an error. The X axis values are too high by a factor of 10. The plot is based on Baldi's (not Bellotti as cited in	Based on Figure 5.59 in the reference	5, 37

Calculated Parameter	Description	Equation	Ref
	LRP) original Figure 3 where the X axis is: $\frac{q_c}{\sqrt{\sigma_v'}}$ (both in kPa) with a range of 200 to 3000. Figure 5.59 from LRP shows a dimensionless form of the equation, q_{c1}, displaying the same range of values. Figure 5.59's X axis uses $q_{c1} = \left(\frac{q_c}{P_a}\right)\left(\frac{P_a}{\sigma_v'}\right)^{0.5}$ The two expressions are not the same: they differ by a factor of $\frac{\sqrt{P_a}}{P_a}$. With P_a taken to be 100 kPa the factor is 1/10. Substituting typical values of 200 bar (20000 kPa) for q_c and 225 kPa for σ_v' one gets: 20000 / 15 = 1333.33 for Bellotti's axis and $(200/1)(100/225)^{0.5}$ = 200 * (10/15) = 133.3 for LRP's axis (noting that P_a = 1 bar) showing a factor of 10 difference.		
Es or E_s 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. E_s is evaluated for an axial strain of 0.1%.	Mean normal stress is evaluated from: $\sigma_m' = \frac{1}{3}(\sigma_v' + \sigma_h' + \sigma_h')$ where σ_v'= vertical effective stress σ_h'= horizontal effective stress *and* $\sigma_h = K_o \cdot \sigma_v'$ with K_o assumed to be 0.5	5
Delta U/TStress $\Delta u / \sigma_v$	Differential pore pressure ratio with respect to total stress	$= \frac{\Delta u}{\sigma_v}$ *where:* $\Delta u = u - u_{eq}$	39
Delta U/EStress, P Value, Excess Pore Pressure Ratio $\Delta u/\sigma_v'$	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
Su/EStress S_u/σ_v'	Undrained shear strength ratio with respect to vertical effective overburden stress using the S_u (N_{kt}) method	$= Su(N_{kt}) / \sigma_v'$	9, 23
Vs or V_s	Recorded shear wave velocities (not estimated). The shear wave velocities are typically collected over 1 m depth intervals. Each data point over the relevant depth range is assigned the same V_s value.	recorded data	27
Vp or V_p	Recorded compression wave (or P wave) velocities (not estimated). The P wave velocities are typically collected over 1 m depth intervals. Each data point over the relevant depth range is assigned the same V_p value.	recorded data	27

Calculated Parameter	Description	Equation	Ref
V_{s30} V_{s100}	The average shear wave velocity of the near surface materials to a depth of 30 m (100 ft). It is based on the sum of all travel times through all layers in the top 30m (100 ft). V_{s100} is the same calculation as V_{s30} except down to a depth of 100 feet.	$V_{s30} = \frac{total\ thickness\ of\ all\ layers\ to\ 30\ m}{\Sigma\left(\frac{layer\ thickness}{layer\ shear\ wave\ velocity}\right)}$ $V_{s30} = \frac{total\ thickness\ of\ all\ layers\ to\ 30\ m}{\Sigma\ (layer\ travel\ times)}$	38
G_{max}	G_{max} determined from SCPT shear wave velocities (not estimated values). Note that seismic data (V_s) is collected over set depth intervals (typically 1 meter). Each data point over the test segment is assigned the same V_s value. Since soil density changes with depth, slightly different G_{max} values may be calculated over the test depth interval.	$G_{max} = \rho V_s^2$ where ρ is the mass density of the soil determined from the estimated unit weights at each test depth	27
$qtNet/G_{max}$	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
qUlt	A site specific and client specific parameter for estimating the limiting stress for "crane walk" accessibility	$q_{ult} = CraneWalkFactor \cdot S_u$ Where: *CraneWalkFactor* is client provided	U*
Estimated G_o	Estimated value for small strain shear modulus	$G_o = 0.0188[10^{(0.55Ic + 1{,}68)}](q_t - \sigma_v)$	15
Estimated E_{25}	Estimated value for Young's Modulus, E, at a 25% working load	$E_{25} = \alpha_E\ (qtNet)$ where $\alpha_E = 0.015[10^{(0.55Ic + 1{,}68)}]$	15
k_{SBT}	Estimated soil permeability derived from Soil Behavior Type (SBT) Chart I_c values.	For $1.0 < I_c \leq 3.27$: $k = 10^{(0.952 - 3.04Ic)}$ in m/s For $3.27 < Ic < 4.0$: $k = 10^{(-4.52 - 1.37Ic)}$ in m/s	35
M or D' Constrained Modulus	Constrained Modulus based on 1) Robertson, M 2) Mayne, D'	1) Robertson $M = \alpha_M\ (q_t - \sigma_v)$ $I_c > 2.2$ (fine grained) $\alpha_M = Qt$ when $Qt < 14$ $\alpha_M = 14$ when $Qt > 14$ $Ic < 2.2$ (coarse grained) $\alpha_M = 0.0188\ [10^{(0.55Ic + 1.68)}]$ $D' = \alpha_D\ (qt - \sigma_v)$ where $\alpha_D = 5$	32 23

Table 1b. CPT Parameter Calculation Methods – Liquefaction Parameters

Calculated Parameter	Description	Equation	Ref
K_{SPT} or K_s	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
K_c (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
$(N_1)_{60cs}I_c$	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 ≤ 5%: $\alpha = 0$, $\beta=1.0$ FC ≥ 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
$Q_{tn,cs}$ (PKR 2010)	Clean sand equivalent for Q_{tn} described above - Q_{tn} being the normalized tip resistance based on a variable stress exponent as defined by Robertson (2009)	$Q_{tn,cs} = Q_{tn} \cdot K_c$ (PKR 2016)	16
Su(Liq)/ESv or $S_u(Liq)/\sigma_v'$	Liquefied shear strength ratio as defined by Olson and Stark	$\frac{S_u(Liq)}{\sigma_v'} = 0.03 + 0.0143(q_{c1})$ Note: σ_v' and s_v' are synonymous	13
Su(Liq)/ESv or $S_u(Liq)/\sigma_v'$ (PKR 2010)	Liquefied shear strength ratio as defined by Robertson (2010)	$\frac{S_u(Liq)}{\sigma_v'}$ Based on a function involving $Q_{tn,cs}$	16
S_u (Liq) (PKR 2010)	Liquefied shear strength derived from the liquefied shear strength ratio and effective overburden stress	$S_u(Liq) = \sigma_v' \cdot \left(\frac{S_u(Liq)}{\sigma_v'}\right)$	16
Cont/Dilat Tip	Contractive / Dilative q_{c1} Boundary based on $(N_1)_{60}$	$(\sigma_v')_{boundary} = 9.58 \times 10^{-4}\ [(N_1)_{60}]^{4.79}$ q_{c1} is calculated from specified q_t(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 or K_g	Small strain Stiffness Ratio Factor, K_g	$[G_{max}/q_t]/[q_{c1n}^{-m}]$ m = empirical exponent, typically 0.75	26

Calculated Parameter	Description	Equation	Ref
K_g*	Revised K_g factor extended to fine grained soils (Robertson).	$K_g^* = (G_o / q_n)(Q_{tn})^{0.75}$ where q_n is the net tip resistance = $q_t - \sigma_v$	30
SP Distance	State Parameter Distance, Winckler static liquefaction method	Perpendicular distance on Q_{tn} chart from plotted point to state parameter $\Psi = -0.05$ curve	25
URS NP Fr	Normalized friction ratio point on $\Psi = -0.05$ curve used in SP distance calculation		25
URS NP Q_{tn}	Normalized tip resistance (Q_{tn}) point on $\Psi = -0.05$ curve used in SP Distance calculation		25

Table 2. References

No.	Reference
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No.	Reference
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APPENDIX F

GSC Seismic Hazard Calculation

Client: LEA Consulting Ltd.

File No.: 33443

April 5, 2024

2015 National Building Code Seismic Hazard Calculation

INFORMATION: **Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836**
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

Site: 49.289N 81.785W **User File Reference:** Poplar Rapids 2023-09-21 12:31 UT

Probability of exceedance per annum	0.000404	0.001	0.0021	0.01
Probability of exceedance in 50 years	2 %	5 %	10 %	40 %
Sa (0.05)	0.181	0.085	0.042	0.007
Sa (0.1)	0.220	0.110	0.057	0.011
Sa (0.2)	0.181	0.093	0.051	0.013
Sa (0.3)	0.134	0.070	0.041	0.011
Sa (0.5)	0.091	0.050	0.030	0.009
Sa (1.0)	0.044	0.026	0.016	0.004
Sa (2.0)	0.021	0.012	0.007	0.002
Sa (5.0)	0.005	0.003	0.001	0.000
Sa (10.0)	0.002	0.001	0.001	0.000
PGA (g)	0.117	0.058	0.030	0.006
PGV (m/s)	0.069	0.037	0.021	0.005

Notes: Spectral (Sa(T), where T is the period in seconds) and peak ground acceleration (PGA) values are given in units of g (9.81 m/s^2). Peak ground velocity is given in m/s. Values are for "firm ground" (NBCC2015 Site Class C, average shear wave velocity 450 m/s). NBCC2015 and CSAS6-14 values are highlighted in yellow. Three additional periods are provided - their use is discussed in the NBCC2015 Commentary. Only 2 significant figures are to be used. **These values have been interpolated from a 10-km-spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the directly calculated values.**

References

National Building Code of Canada 2015 NRCC no. 56190; Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada

Structural Commentaries (User's Guide - NBC 2015: Part 4 of Division B)
Commentary J: Design for Seismic Effects

Geological Survey of Canada Open File 7893 Fifth Generation Seismic Hazard Model for Canada: Grid values of mean hazard to be used with the 2015 National Building Code of Canada

See the websites www.EarthquakesCanada.ca and www.nationalcodes.ca for more information

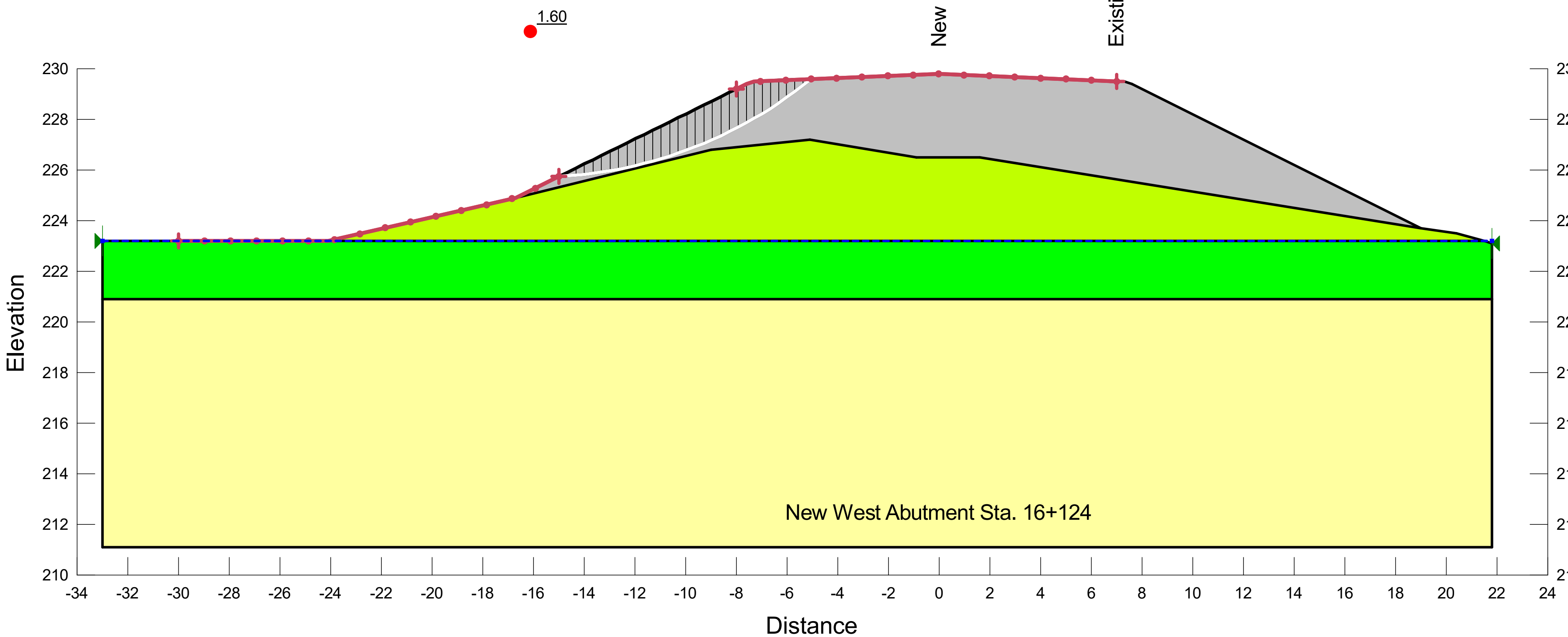
Natural Resources Canada Ressources naturelles Canada

Canada

APPENDIX G

Foundation Analyses

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Surface
	0_New FILL	Mohr-Coulomb	21	0	32	1
	1_CLAY to Silty CLAY (TSA)	Mohr-Coulomb	18	60	0	1
	2_CLAY to Silty CLAY (TSA)	Mohr-Coulomb	18	60	0	1
	3_SILT to Sandy SILT	Mohr-Coulomb	18	0	28	1



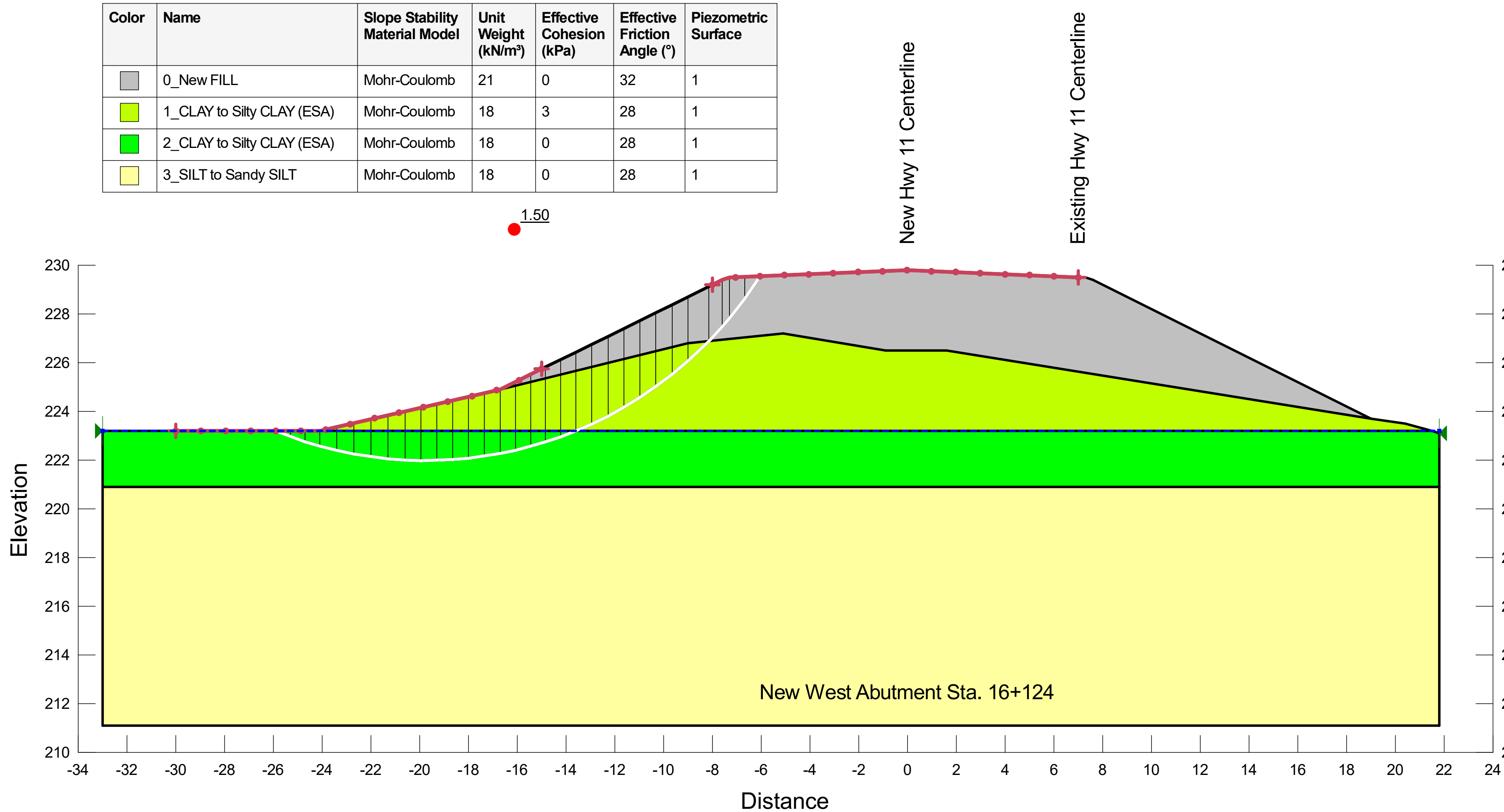
THURBER	Project: Poplar Rapids River Bridge		
	Analysis: Undrained		
	Seismic Coefficient: H: 0g, V: 0g	Last Run: 04/04/2024, 09:42:42 AM	Scale: 1:250

Additional Details
Name: West Abutment
Comments: Stability Assessment
Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m
Entry: (-15, 225.74066) m, Exit: (-5.0494536, 229.59249) m
Center: (-15.98316, 243.05918) m, Radius: 17.346401 m

Figure 1

O:\Projects\30001 to 40000\33443 - Hwy 11 Poplar Rapids Bridge\Analysis\Bridge\Poplar_001.gsz

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Surface
	0_New FILL	Mohr-Coulomb	21	0	32	1
	1_CLAY to Silty CLAY (ESA)	Mohr-Coulomb	18	3	28	1
	2_CLAY to Silty CLAY (ESA)	Mohr-Coulomb	18	0	28	1
	3_SILT to Sandy SILT	Mohr-Coulomb	18	0	28	1



Project
Poplar Rapids River Bridge

Analysis
Drained

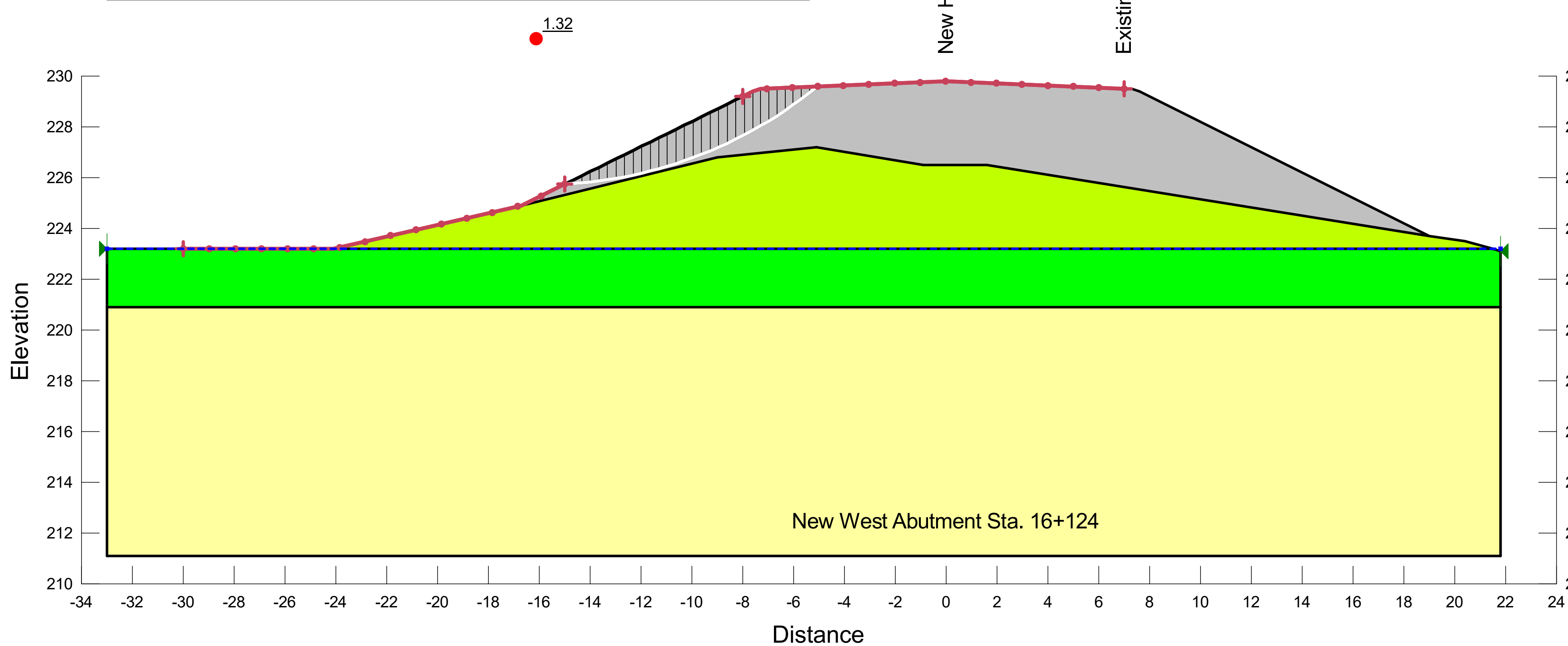
Seismic Coefficient	Last Run	Scale
H: 0g, V: 0g	04/04/2024, 09:42:40 AM	1:250

Additional Details
Name: West Abutment
Comments: Stability Assessment
Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m
Entry: (-25.896197, 223.2) m, Exit: (-6.0535747, 229.55122) m
Center: (-19.746186, 238.15798) m, Radius: 16.172939 m

Figure 2

O:\Projects\30001 to 40000\33443 - Hwy 11 Poplar Rapids Bridge\Analysis\Bridge\Poplar_001.gsz

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Surface
	0_New FILL	Mohr-Coulomb	21	0	32	1
	1_CLAY to Silty CLAY (TSA)	Mohr-Coulomb	18	60	0	1
	2_CLAY to Silty CLAY (TSA)	Mohr-Coulomb	18	60	0	1
	3_SILT to Sandy SILT	Mohr-Coulomb	18	0	28	1



THURBER

Project	Additional Details
Poplar Rapids River Bridge	Name: West Abutment
Analysis: Seismic	Comments: Stability Assessment
Seismic Coefficient: H: 0.075g, V: 0g	Method: Morgenstern-Price, Half-Sine
Last Run: 04/04/2024, 09:42:42 AM	Minimum Slip Surface Depth: 1.52 m
Scale: 1:250	Entry: (-15, 225.74066) m, Exit: (-5.0494536, 229.59249) m
	Center: (-15.98316, 243.05918) m, Radius: 17.346401 m

Figure 3

O:\Projects\30001 to 40000\33443 - Hwy 11 Poplar Rapids Bridge\Analysis\Bridge\Poplar_001.gsz

SOIL P-Y CURVES

Highway 11 Poplar River Bridge

HP310x110, Depth Below West Abutment (elev. 223.7 m)

Depth (m)	0.5		1.5		2.5		3.5		4.5		5.5		6.5		7.5		8.5		9.5		10.5		11.5		12.5		13.5	
P-y Curves	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
Static	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.000	0.000	0.000	0.000	0.000	0.000	0.0000	0.0
	0.0006	26.74	0.0007	46.69	0.0007	46.69	0.0027	36.24	0.0076	130.68	0.0010	21.18	0.0012	28.52	0.0012	32.91	0.0012	37.30	0.0012	41.69	0.001	46.077	0.001	50.465	0.001	54.854	0.0042	528.2
	0.0012	37.82	0.0013	66.03	0.0013	66.03	0.0029	38.00	0.0080	135.92	0.0020	42.36	0.0023	57.05	0.0023	65.82	0.0023	74.60	0.0023	83.38	0.002	92.154	0.002	100.931	0.002	109.707	0.0043	535.1
	0.0017	43.93	0.0020	76.93	0.0020	76.93	0.0032	39.72	0.0083	140.44	0.0030	63.54	0.0035	85.57	0.0035	98.74	0.0035	111.90	0.0035	125.07	0.003	138.231	0.003	151.396	0.003	164.561	0.0044	542.0
	0.0023	47.80	0.0026	84.01	0.0026	84.01	0.0034	41.39	0.0086	144.14	0.0041	84.73	0.0046	114.10	0.0046	131.65	0.0046	149.20	0.0046	166.76	0.005	184.309	0.005	201.862	0.005	219.415	0.0044	548.8
	0.0029	50.37	0.0033	88.84	0.0033	88.84	0.0036	43.01	0.0089	147.85	0.0051	105.91	0.0058	142.62	0.0058	164.56	0.0058	186.50	0.0058	208.44	0.006	230.386	0.006	252.327	0.006	274.269	0.0045	555.6
	0.0035	51.99	0.0039	92.07	0.0039	92.07	0.0038	44.60	0.0092	151.56	0.0061	127.09	0.0069	171.14	0.0069	197.47	0.0069	223.80	0.0069	250.13	0.007	276.463	0.007	302.793	0.007	329.122	0.0046	562.3
	0.0041	52.89	0.0046	94.06	0.0046	94.06	0.0041	46.15	0.0095	155.27	0.0071	148.27	0.0081	199.67	0.0081	230.39	0.0081	261.10	0.0081	291.82	0.008	322.540	0.008	353.258	0.008	383.976	0.0047	568.9
	0.0047	53.21	0.0052	95.04	0.0052	95.04	0.0043	47.67	0.0098	158.98	0.0081	169.45	0.0092	228.19	0.0092	263.30	0.0092	298.40	0.0092	333.51	0.009	368.617	0.009	403.724	0.009	438.830	0.0048	575.5
	0.0052	53.03	0.0059	95.19	0.0059	95.19	0.0045	49.16	0.0101	162.69	0.0091	190.63	0.0104	256.72	0.0104	296.21	0.0104	335.70	0.0104	375.20	0.010	414.694	0.010	454.189	0.010	493.684	0.0049	582.1
	0.0058	52.43	0.0065	94.61	0.0065	94.61	0.0047	50.62	0.0104	166.40	0.0101	211.82	0.0116	285.24	0.0116	329.12	0.0116	373.01	0.0116	416.89	0.012	460.771	0.012	504.654	0.012	548.537	0.0050	588.6
	0.0064	51.46	0.0072	93.40	0.0072	93.40	0.0049	52.05	0.0107	170.11	0.0112	233.00	0.0127	313.76	0.0127	362.03	0.0127	410.31	0.0127	458.58	0.013	506.849	0.013	555.120	0.013	603.391	0.0051	595.1
	0.0070	50.16	0.0078	91.64	0.0078	91.64	0.0052	53.46	0.0110	173.82	0.0122	254.18	0.0139	342.29	0.0139	394.95	0.0139	447.61	0.0139	500.27	0.014	552.926	0.014	605.585	0.014	658.245	0.0052	601.5
	0.0116	36.34	0.0130	66.10	0.0130	66.10	0.0084	73.77	0.0113	177.53	0.0132	275.36	0.0150	370.81	0.0150	427.86	0.0150	484.91	0.0150	541.95	0.015	599.003	0.015	656.051	0.015	713.099	0.0084	830.1
	0.0163	22.49	0.0182	40.53	0.0182	40.53	0.0116	94.09	0.0116	181.24	0.0142	296.54	0.0162	399.34	0.0162	460.77	0.0162	522.21	0.0162	583.64	0.016	645.080	0.016	706.516	0.016	767.952	0.0116	1058.6
	0.0209	8.64	0.0234	14.95	0.0234	14.95	0.0140	94.09	0.0140	181.24	0.0170	296.54	0.0194	399.34	0.0194	460.77	0.0194	522.21	0.0194	583.64	0.019	645.080	0.019	706.516	0.019	767.952	0.0140	1058.6
	0.0221	8.64	0.0247	14.95	0.0247	14.95	0.0163	94.09	0.0163	181.24	0.0199	296.54	0.0226	399.34	0.0226	460.77	0.0226	522.21	0.0226	583.64	0.023	645.080	0.023	706.516	0.023	767.952	0.0163	1058.6

Depth (m)	14.5		15.5		16.5		17.5		18.5		19.5		20.5		21.5		22.5		23.5		24.5		25.5		26.5		27.5	
P-y Curves	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
Static	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00														
	0.0042	567.32	0.0042	606.45	0.0042	645.57	0.0042	684.70	0.0042	723.83	0.0042	762.95	0.0042	802.08														
	0.0043	574.76	0.0043	614.40	0.0043	654.04	0.0043	693.68	0.0043	733.32	0.0043	772.96	0.0043	812.60														
	0.0044	582.14	0.0044	622.29	0.0044	662.44	0.0044	702.59	0.0044	742.73	0.0044	782.88	0.0044	823.03														
	0.0044	589.46	0.0044	630.11	0.0044	670.77	0.0044	711.42	0.0044	752.07	0.0044	792.72	0.0044	833.38														
	0.0045	596.72	0.0045	637.88	0.0045	679.03	0.0045	720.18	0.0045	761.34	0.0045	802.49	0.0045	843.64														
	0.0046	603.93	0.0046	645.58	0.0046	687.23	0.0046	728.88	0.0046	770.53	0.0046	812.18	0.0046	853.83														
	0.0047	611.08	0.0047	653.22	0.0047	695.36	0.0047	737.51	0.0047	779.65	0.0047	821.79	0.0047	863.94														
	0.0048	618.17	0.0048	660.80	0.0048	703.44	0.0048	746.07	0.0048	788.70	0.0048	831.33	0.0048	873.97														
	0.0049	625.22	0.0049	668.33	0.0049	711.45	0.0049	754.57	0.0049	797.69	0.0049	840.81	0.0049	883.93														
	0.0050	632.21	0.0050	675.81	0.0050	719.41	0.0050	763.01	0.0050	806.61	0.0050	850.21	0.0050	893.81														
	0.0051	639.15	0.0051	683.23	0.0051	727.31	0.0051	771.39	0.0051	815.47	0.0051	859.55	0.0051	903.63														
	0.0052	646.05	0.0052	690.60	0.0052	735.16	0.0052	779.71	0.0052	824.27	0.0052	868.82	0.0052	913.38														
	0.0084	891.54	0.0084	953.03	0.0084	1014.52	0.0084	1076.00	0.0084	1137.49	0.0084	1198.97	0.0084	1260.46														
	0.0116	1137.04	0.0116	1215.46	0.0116	1293.88	0.0116	1372.29	0.0116	1450.71	0.0116	1529.13	0.0116	1607.54														
	0.0140	1137.04	0.0140	1215.46	0.0140	1293.88	0.0140	1372.29	0.0140	1450.71	0.0140	1529.13	0.0140	1607.54														
	0.0163	1137.04	0.0163	1215.46	0.0163	1293.88	0.0163	1372.29	0.0163	1450.71	0.0163	1529.13	0.0163	1607.54														

The following assumptions were made in the analysis:

1. The analysis was completed for a vertical element (i.e. no inclination)
2. These curves are for static loading. Seismic effects have not been included.
3. The effects of construction disturbance is not considered.

NOTES:

1. **The p-y data provided is unfactored**. Lateral resistance or deflection calculated based on these parameters should be factored using the geotechnical resistance factors (Øgu and Øgs) provided in the CHBDC
2. If lateral spacing between an adjacent structural element is less than four equivalent diameters, suitable reduction factors based on center to center spacing should be applied based on tables of the CHBDC

APPENDIX H

List of Referenced Specifications and Contract Provisions

1. The following Special Provisions and OPSS Documents referenced in this report:

   - OPSS.PROV 206
   - OPSS.PROV 501
   - OPSS.PROV 511
   - OPSS.PROV 517
   - OPSS.PROV 539
   - OPSS.PROV 803
   - OPSS.PROV 804
   - OPSS.PROV 805
   - OPSS.PROV 902
   - OPSS.PROV 903
   - OPSS.PROV 1010
   - SP 105S09
   - SP 109S12
   - SP 110S06
   - SP 517F01
   - SP FOUN0003
   - OPSD 208.010
   - OPSD 219.110
   - OPSD 3090.100
   - OPSD 3101.150
   - OPSD 3121.150