



**Foundation Investigation and
Design Report – Ron McNeil Line
Interchange Underpass –
Highway 4 widening from Clinton
Line to New Talbotville Bypass
and New Talbotville Bypass from
Highway 4 to Highway 3 at Ron
McNeil Line**

Highway 3 Township of Southwold,
County of Elgin, ON
West Region

GWP 3042-22-00

Latitude 42.809693
Longitude -81.229026

Geocres No. 40114-223

Prepared for:

Ministry of Transportation, Ontario
(MTO), West Region

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UNDERPASS – HIGHWAY 4 WIDENING FROM CLINTON LINE TO NEW TALBOTVILLE BYPASS
AND NEW TALBOTVILLE BYPASS FROM HIGHWAY 4 TO HIGHWAY 3 AT RON MCNEIL LINE

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April 2025

FOUNDATION INVESTIGATION REPORT

For

G.W.P. 3042-22-00

Ron McNeil Line Interchange Underpass

Highway 4 widening from Clinton Line to New Talbotville Bypass and New Talbotville Bypass from Highway 4 to Highway 3 at Ron McNeil Line
West Region, Township of Southwold, County of Elgin, Ontario

1.0 INTRODUCTION

Stantec has been retained by the Ministry of Transportation Ontario (MTO) to provide preliminary and detailed design services for the Highway 4 widening from Clinton Line to the new Talbotville Bypass and for the new Talbotville Bypass from Highway 4 to Highway 3 at Ron McNeil Line (GWP 3042-22-00), and for the Highway 3 widening from Ron McNeil Line to Centennial Avenue (GWP 3041-22-00).

As part of the GWP 3042-22-00 new Talbotville Bypass from Highway 4 to Highway 3 at Ron McNeil Line, the following new structures are proposed:

- CNR Talbotville Overhead - Two (2) Single Span Bridges with about 300 m long approach embankment on both sides of bridges,
- Ron McNeil Line Interchange Overpass - Two Span Bridge with approach embankments, and
- Lindsay Creek Culvert (formerly Dodd's Creek Culvert).

As part of the GWP 3041-22-00 Highway 3 Twinning from Ron McNeil Line to Centennial Avenue, the following new structures, including two existing culverts replacement, are proposed:

- Wellington Road Interchange Underpass – New Two Span Bridge with approach embankments
- Kettle Creek WBL Bridge – New Three Span Bridge
- 05X-0266/C0 Underhill Drain Culvert – New Culvert Construction Under the proposed Highway Twinning
- 05X-0268/C0 – Existing CSP Culvert replacement & New Culvert Construction Under the proposed Highway Twinning
- Noise Walls (between Stations 13+100 and 11+100, south side of the existing Highway 3 & between Stations 12+400 and 13+600 on both sides of Highway 3)
- Deep Cuts (between Stations 13+650 and 15+050, north of the existing Highway 3)

Eighteen (18) Overhead Signs and three (3) Storm Water Management Ponds (SWMPs) were also planned at the early stage of the project. As per the preliminary design, three (3) Storm Water Management Ponds were eliminated, and four (4) structural culverts were added at the Ron McNeil Line interchange area.



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This Foundation Investigation Report has been prepared specifically and solely for the proposed Ron McNeil Line Overpass. Other project foundations engineering components are reported under separate cover.

The terms of reference for the foundation investigation work scope were provided in the MTO's RFP (Request for Proposal) and addenda. The MTO Guideline for Foundation Engineering Services V.3.0 is also considered for the borehole termination depth based on the clarifications provided during the bid phase.

2.0 SITE DESCRIPTION AND GEOLOGY

2.1 SITE LOCATION

Ron McNeil Line is planned to cross over the Talbotville Bypass at approximate Station 10+000, about 100 m northwest of its current intersection with Highway 3 in the City of St. Thomas, Ontario. The site location is shown on the Key Plan inset to Drawing No. 1 included in Appendix A.

2.2 GENERAL SITE DESCRIPTION

At the proposed location of the Ron McNeil Interchange Overpass, Highway 3 is planned to be a six-lane divided freeway, with three traffic lanes and shoulders in each direction.

At the site location, Ron McNeil Line is currently a two-lane roadway with shoulders on both sides. As part of the project, Ron McNeil Line is planned to be widened (two traffic lanes in each direction, with outer shoulders) and realigned to the west, and to connect to Highway 3 approximately 100 m northwest of its current intersection with Highway 3 and Ford Road.

The orientation of Highway 3 is approximately northwest-southeast, and the orientation of the Ron McNeil Line is approximately northeast-southwest. For the purposes of this report, the orientation of the Talbotville Bypass and the Ron McNeil Line are taken as east-west and north-south, respectively.

The area immediately adjacent to the proposed overpass consists of agricultural fields. The ground surface at the site generally is flat to gently undulating.



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Photo 1. Ron McNeil Line Underpass Site (looking north)

2.3 PROPOSED STRUCTURE

Based on the General Arrangement (GA) drawing, a two-span bridge structure with cast-in-place, post-tensioned concrete deck and integral abutments is planned for the underpass. The drawing also indicates that the bridge will be approximately 72 m long, and approximately 21.8 to 22.7 m wide, and will be located perpendicular to the planned alignment of Highway 3. The top of the highest sections of the proposed south and north approach embankments (at the locations of the abutments) are planned to be at approximately elevations 244 m and 246 m, approximately 7 m and 8 m higher than the surrounding lands, respectively. The embankments are planned to have 2 horizontal : 1 vertical side slopes and foreslopes.

The GA drawing is included in Appendix A for reference.

2.4 GEOLOGICAL INFORMATION

The site is located within the physiographic region of Ekfrid Clay Plain, as delineated in the Physiography of Southern Ontario (Chapman and Putnam, 1983). According to the Ontario Department of Mines Preliminary Geological Maps 238 (Pleistocene Geology of The St. Thomas Area, West Half) and P.606 (Pleistocene Geology of The St. Thomas Area, East Half), the site subsurface conditions are generally characterized by lacustrine deposits of silt, silty sand and clay, Port Stanley silty clay to clayey silt till and modern alluvium deposits of gravel, sand, and silt along watercourses. As per the Ontario Geological Survey Map 2441 (Geological Highway Map Southern Ontario), the bedrock within the project area is described as grey limestone of the Dundee Formation. Based on the Ontario Department of Mines Preliminary Geological Map P. 482 (St. Thomas Sheet), the bedrock depths with the bridge site is estimated to be about 85 m below the original ground surface (o.g.).



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2.5 EXISTING UTILITIES

A review of available information indicated that there is a water main and a gas main running parallel to Wonderland Road and extending to the immediate west of the planned Ron McNeil Line Overpass. There are also overhead cables running north-south to the immediate west of the planned Ron McNeil Line Overpass. No critical buried utilities were identified within the proposed structure foundations and embankments' footprint.

3.0 REVIEW OF PREVIOUS INVESTIGATIONS

A review of MTO GEOCREC database identified the following report at the Ron McNeil Line Overpass site:

GEOCREC Reference No. 40114-35

A foundation investigation report dated August 13, 1973, was available for the proposed crossing at St. Thomas Expressway and County Road #52.

The report was referenced as follows:

Foundation Investigation Report
For Proposed Crossing at
St. Thomas Expressway and County Road #52
Twps. Of Southwold; Co. of Elgin
District #2 (London)
W.O. 73-11021 - W.P. 89-69-07

The investigation included a total of three (3) sampled boreholes (BH No. 1 to 3), advanced to depths of approximately 18.8 m, 15.7 and 24.8 m below grade (corresponding to approximately elevations 218.4 m, 221.4 m and 212.7 m) and six (6) dynamic cone penetration tests advanced in May 1973.

The boreholes encountered a deep stratum of very stiff to hard clayey silt to silty clay with small amounts of sand and trace gravel. Occasional pockets and/or thin seams of silt were also noted, and sand partings were inferred to be present within this deposit. Except within the top 2 m, the stratum had a moisture content that was at or below the Plastic Limit. Based on the N-values obtained, the undrained shear strength of the stratum was inferred to be higher than approximately 100 kPa everywhere and as high as 240 kPa.

The boreholes were dry upon completion. However, it was noted in the report that due to the relatively impermeable nature of the soils encountered and short duration of the fieldwork, groundwater levels at the site could not be established conclusively but were inferred to be well below the elevation of the proposed structure footing at the time (i.e., approximately Elevation 234 m). It was noted that the randomly distributed silt seams and/or sand partings could be water bearing.



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For reference, copies of the Borehole Location Plan, stratigraphic profile, borehole records and laboratory test results are included in Appendix B.

4.0 INVESTIGATION PROCEDURES

4.1 FIELD INVESTIGATION

The geotechnical investigation for the detailed design of the proposed Ron McNeil Line consisted of a total of five (5) boreholes. Two (2) of the boreholes, designated as RMN-UP1 and RMN-UP3 were advanced for the abutments, borehole RMN-UP2 was advanced for the central pier, and two (2) boreholes, designated as RMN-A1 and RMN-A2 were advanced for the approach embankments. The investigation also included advancing one (1) Cone Penetration Test, designated as CPT24-RMNAPP01 (and additional shear wave velocity measurements in a separate sounding -sCPT24-RMNAPP01). The locations of the boreholes and CPT are shown on the Borehole Location Plan, Drawing No. 1 in Appendix A.

Prior to carrying out the investigation, Stantec contacted the public utility authorities to clear the borehole locations of private, public as well as and MTO-owned utilities.

The field drilling program was carried out between May 28, 2024, and July 4, 2024. The abutment boreholes were advanced to a depth of approximately 44.6 m below grade and boreholes RMN-A1 and RMN-A2 were advanced to the depths of approximately 14.5 m and 12.8 m below grade, respectively. All boreholes were advanced using hollow-stem continuous-flight augers. Wash boring technique was used below a depth of 3 m in boreholes RMN-UP1 to RMN-UP3. Drilling was carried out with CME55 and D50 track-mounted rigs equipped for soil sampling.

The subsurface stratigraphy encountered in each borehole was recorded in the field by an experienced Stantec field technician. Standard Penetration Tests (SPT) were carried out in the drilled holes and split spoon samples were collected at regular intervals (0.75 m interval for the shallow depth / critical zone, 1.5 m interval to 20 m below grade and 3 m interval to the termination depths of the boreholes to meet the typical MTO subsurface investigation sampling requirements) in accordance with ASTM D1586. Shelby tube (thin-walled steel tube) samples were also obtained in the boreholes at select depths. All recovered SPT and Shelby tube samples were returned to our Markham and Ottawa laboratories for detailed classification and testing. The undrained shear strength of cohesive soils was determined using an in-situ shear vane (MTO B-vane) in accordance with ASTM D2573 wherever applicable. A pocket penetrometer was also used to estimate the shear strength/consistency of clayey soil samples at the site.

One (1) CPT, designated as CPT24-RMNAPP01 (and additional shear wave velocity measurement - sCPT24-RMNAPP01) was conducted at the site on May 5, 2024. The CPT was advanced to a target depth of approximately 15 m, below grade. ConeTec CPT report dated May 24, 2024, is included in Appendix C.



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A single line of Multi-Channel Analysis of Surface Wave (MASW) was also carried out at the site to determine the seismic site classification. The MASW report is included in Appendix F.

Groundwater was observed in open boreholes during drilling. Following completion of drilling, a 50 mm diameter groundwater monitoring well, screened over a depth of 4.6 m to 7.6 m below ground surface, was installed in Borehole RMN-UP2. The borehole annulus surrounding the slotted pipe section was backfilled with sand. The remaining annulus was backfilled with bentonite up to the ground surface. Groundwater level measurement in the monitoring well was carried out on September 11, 2024.

After completion of drilling, the remaining boreholes were backfilled with a mix of bentonite and drill cuttings.

4.2 LOCATION AND ELEVATION SURVEY

The borehole locations and respective ground surface elevations were surveyed by Stantec Geomatics personnel using Trimble R12i GPS with an elevation and spatial accuracy of ± 0.02 m vertically and ± 0.01 m horizontally to meet the survey accuracy requirements (vertical accuracy of 0.1 m and horizontal accuracy of 0.5 m) of the Guideline for MTO Foundation Engineering Services V2.

Table 4.1 below summarizes the borehole survey information and includes the drilling depth, end of borehole elevation and number of samples recovered for each borehole.

Table 4.1: Borehole Information Summary

Test Hole	MTM Zone 11 Coordinates		Ground surface elevation (m)	End of borehole depth (m)	End of borehole elevation (m)	Number of soil samples
	Northing	Easting				
RMN-A1	4741937.4	408801.5	239.2	14.5	224.7	14
RMN-A2	4741843.9	408703.6	237.2	12.8	224.4	13
RMN-UP1	4741914.1	408760.6	237.6	44.6	193.0	26
RMN-UP2	4741885.4	408747.7	237.6	44.8	192.8	26
RMN-UP3	4741861.5	408721.2	237.0	44.7	192.3	26
CPT24-RMNAPP01	4741913.8	408759.6	237.6	15.0	222.6	-

4.3 LABORATORY TESTING

All samples were taken to Stantec's Markham and Ottawa laboratories where they were subjected to a detailed visual and tactile examination by a Geotechnical Engineer.

The geotechnical laboratory testing program for the boreholes samples is summarized in the following table.



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Table 4.2: Geotechnical Laboratory Testing Program

Laboratory Test Type	Number of Tests
Moisture Content	112
Gradation Analysis	25
Atterberg Limits	28
Consolidation (oedometer)	3
Unconsolidated Undrained Triaxial Compression Test (UU)	1
Chemical Analysis	3

Three soil samples from the boreholes advanced for the overpass structure abutments were forwarded to AGAT Laboratories. The samples were tested for pH, soluble sulphate content, chloride content, and resistivity.

Samples remaining after testing will be placed in storage for a period of one year after issuance of the final report. After the storage period, the samples will be discarded unless we are directed otherwise by MTO.

5.0 SUBSURFACE CONDITIONS

5.1 OVERVIEW

The detailed soil and groundwater conditions encountered in the boreholes and the results of the in-situ and laboratory testing are shown on the Borehole Records included in Appendix C. An explanation of the symbols and terms used to describe the Borehole Records is also provided in Appendix C. The results of the geotechnical laboratory testing are presented on Figures D1 to D6 included in Appendix D. It is noted that clay size particles include all particles smaller than 0.002 mm.

A borehole location plan and a stratigraphic section of the soils encountered in the boreholes along the bridge alignment are provided on Drawing No.1 in Appendix A.

The stratigraphic boundaries on the borehole records and the strata plot are inferred from non-continuous sampling and therefore represent transitions between soil types rather than exact boundaries between geological units. The subsurface conditions will vary between and beyond the borehole locations.

In general, the subsurface stratigraphy encountered in the boreholes generally consisted of:

- Topsoil; underlain by,
- Cohesionless fill comprising silty sand in RMN-A1; underlain by,
- Cohesive fill comprising silty clay and sandy clayey silt (except in RMN-UP1 and RMN-UP2); underlain by,
- Firm to hard clayey silt till; interbedded with,
- Hard clayey silt in RMN-A1, RMN-UP1 and RMN-UP2.



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Groundwater level was measured at a depth of approximately 5.8 m below grade corresponding to approximate elevation 231.8 m in the monitoring well installed in borehole RMN-UP2.

Detailed descriptions of the subsurface conditions are provided below.

5.2 OVERBURDEN

5.2.1 Topsoil

Topsoil was encountered at all borehole locations, except RMN-A1 which was advanced from the existing Ron McNeil Line gravel shoulder. The thickness of the topsoil varied from approximately 200 mm to 300 mm.

5.2.2 Fill Materials

5.2.2.1 Cohesionless Fill

A layer of fill material comprising brown silty sand was encountered below the gravel shoulder in borehole RMN-A1. Samples obtained from the fill layer contained trace gravel. The fill layer was approximately 0.3 m thick.

An N-value of 13 blows per 0.3 m was obtained from the SPT advanced in the cohesionless fill layer, indicating a compact condition.

Laboratory tests conducted on the sample of the cohesionless fill yielded a natural moisture content of approximately 16%.

5.2.2.2 Cohesive Fill

A layer of brown to black silty clay fill was encountered below the cohesionless fill layer described in the preceding section in borehole RMN-A1. Layers of brown to grey sandy clayey silt fill were encountered below the topsoil in boreholes RMN-A2 and RMN-UP3. Samples obtained from these cohesive fill layers typically contained trace gravel.

The cohesive fill layer was approximately 2 m, 1.2 m and 1.3 m thick in boreholes RMN-A1, RMN-A2 and RMN-UP3, respectively. The bottom of the cohesive fill layer was encountered at the depths of approximately 2.3 m, 1.5 m and 1.5 m corresponding to approximately elevations 236.9 m, 235.7 m and 235.5 m, respectively in boreholes RMN-A1, RMN-A2 and RMN-UP3.

The N-values obtained from the SPTs advanced in the cohesive fill layer ranged from 2 to 9 blows per 0.3 m penetration, indicating a soft to stiff consistency.

Laboratory tests conducted on samples of the cohesive fill yielded natural moisture contents ranging from approximately 19% to 27%, averaging 21%.



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Gradation analyses were carried out on a representative sample of the cohesive fill. The test results are illustrated on the borehole record in Appendix C and on the gradation curve on Figure No. D1 in Appendix D. The tests yielded the following results:

Gravel:	1%
Sand:	12%
Silt:	58%
Clay:	29%

Atterberg Limits tests were conducted on the sample referenced above. The tests yielded a Liquid Limit of approximately 38%, a Plastic Limit of approximately 22%, and a corresponding Plasticity Index of approximately 16%. The test results are illustrated on the borehole record in Appendix C and on the graph on Figure No. D2 in Appendix D.

Based on the results of the laboratory tests, the sample tested can be classified as silty clay with a group symbol of CI based on the Unified Soil Classification System (USCS).

5.2.3 Clayey Silt (CL) Till

An extensive deposit of brown to grey clayey silt till was encountered below the topsoil and/or fill materials in all boreholes. The deposit typically contained various but minor amounts of sand and gravel. Layers of soils with higher silt content and lower plasticity (described in the preceding section) were noted within this deposit in several boreholes. Presence of cobbles and/or boulders was inferred in the till deposit, based on auger grinding.

All boreholes were terminated in this deposit after penetrating approximately 11.3 m to 44.7 m into the layer.

The N-values obtained from the SPTs advanced in the clayey silt till deposit ranged from 7 to in excess of 100. The lower N-values were generally obtained in the surficial zone of this deposit (i.e., top 1 m) and the refusal blow counts were obtained at depth.

In-situ shear vane tests (MTO B-vane) were conducted in the clayey silt till deposit in boreholes RMN-A1 and RMN-UP2. The results of the tests are summarized in Table 5.1 below.

Table 5.1: In-situ Shear Vane Test Results – Clayey Silt Till

Borehole	Type	Depth (m)	Elevation (m)	In-situ Undrained Shear Strength (kPa)	Sensitivity
RMN-A1	B-vane	14.4	224.8	170	1.4
RMN-UP2	B-vane	17.8	219.8	180	2.7

An Unconsolidated Undrained (UU) Triaxial test was conducted on a select Shelby tube sample retrieved at a depth of approximately 5.6 m below grade, corresponding to approximate elevation 232 m in



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borehole RMN-UP2. The test indicated a Compressive Strength of approximately 210 kPa corresponding to an undrained shear strength of approximately 105 kPa. The details of test are included in the test sheets in Appendix D.

Based on the results of these tests, the clayey silt till can generally be described as stiff to hard, except for the top 1 m, which can be described as firm.

Laboratory tests conducted on samples of the clayey silt till deposit yielded natural moisture contents ranging from approximately 11% to 29%, averaging 17%.

Gradation analyses were carried out on 21 samples of the clayey silt till soils. The test results are illustrated on the borehole records in Appendix C and on the gradation curve on Figure No. D3 in Appendix D. The tests yielded the following results:

Gravel:	0 to 15%
Sand:	5 to 28%
Silt:	40 to 52%
Clay:	22 to 53%

Atterberg Limits tests were conducted on the samples referenced above as well as three Shelby Tube samples retrieved from boreholes RMN-A1, RMN-A2 and RMN-UP2. The tests yielded Liquid Limits ranging from approximately 18% to 35%, Plastic Limits ranging from approximately 10% to 17%, and corresponding Plasticity Indices ranging from approximately 8% to 18%. The test results are illustrated on the borehole records in Appendix C and on the gradation curve on Figure No. D4 in Appendix D.

Based on the results of the laboratory tests, the samples tested can be classified as clayey silt with a group symbol of CL based on the Unified Soil Classification System (USCS).

One-dimensional oedometer consolidation tests were carried out on portions of select Shelby tube samples. The results are provided below in Table 5.2 and the details of the tests, including the data plots, are provided on the laboratory test sheets in Appendix D.

Table 5.2: One-Dimensional Oedometer Consolidation Test Results

Borehole/ Sample	Depth/ Elevation (m)	Initial Void Ratio	Initial Unit Weight (kN/m ³)	Estimated Pre- consolidation Stress, Pc' (kPa)	Recompression Index Cr / Compression Index Cc	Over Consolidation Ratio OCR	Coefficient of Consolidation C _v (cm ² /s)
RMN-A1/ TW9	6.3/232.9	0.42	21.9	400	0.01/0.084	3	2.7x10 ⁻³



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Borehole/ Sample	Depth/ Elevation (m)	Initial Void Ratio	Initial Unit Weight (kN/m ³)	Estimated Pre- consolidation Stress, Pc' (kPa)	Recompression Index Cr / Compression Index Cc	Over Consolidation Ratio OCR	Coefficient of Consolidation C _v (cm ² /s)
RMN-A2 TW7	4.8/232.4	0.47	21.4	480	0.015/0.12	4.7	3x10 ⁻³
RMN-UP2 TW8	5.6/231.9	0.52	20.9	350	0.013/0.093	3	3x10 ⁻³

5.2.4 Clayey Silt (CL-ML)

Localized layers of clayey silt with higher silt content and lower plasticity than the clayey silt till described in the preceding section were noted interbedded in the clayey silt till deposit in boreholes RMN-A1, RMN-UP1 and RMN-UP2. Samples obtained from the clayey silt layers typically contained trace sand and gravel.

The clayey silt layer was approximately 1.5 m, 3.6 m and 1 m thick and extended from depths of approximately 10.2 m, 7.2 m and 8.7 m below grade, corresponding to approximately elevations 229 m, 230.4 and 228.9 m to depths of approximately 11.7 m, 10.8 m and 9.7 m below grade, corresponding to approximately elevations 227.5 m, 226.8 m and 227.9 m, respectively; in boreholes RMN-A1, RMN-UP1 and RMN-UP2.

N-values obtained from the SPTs advanced in the clayey silt layer ranged from 44 to 63, indicating hard consistency.

Laboratory tests conducted on samples of the clayey silt layer yielded natural moisture contents ranging from approximately 13% to 17%, averaging 15%.

Gradation analyses were carried out on three (3) samples of the clayey silt soils. The test results are illustrated on the borehole records in Appendix C and on the gradation curve on Figure No. D5 in Appendix D. The tests yielded the following results:

Gravel:	0 to 10%
Sand:	2 to 9%
Silt:	54 to 74%
Clay:	24 to 28%

Atterberg Limits tests were conducted on the samples referenced above. The tests yielded Liquid Limits of approximately 19%, 20% and 20%, Plastic Limits of approximately 14%, 14% and 13%, and



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corresponding Plasticity Indices of approximately 5%, 6% and 7%. The test results are illustrated on the borehole records in Appendix C and on the graph on Figure No. D6 in Appendix D.

Based on the results of the laboratory tests, the samples tested can be classified as clayey silt with a group symbol of CL-ML based on the Unified Soil Classification System (USCS).

5.2.5 Bedrock

No bedrock was encountered in any boreholes within the investigation depths.

5.2.6 Groundwater

A monitoring well was installed in borehole RMN-UP2 to observe the long-term groundwater levels. In other boreholes, groundwater level observations were made during drilling operations, and in the open boreholes upon completion of drilling. The groundwater level recorded in RMN-UP2 and inferred in the other boreholes are summarized in Table 5.3 below.

Table 5.3: Measured and Inferred Groundwater Levels

Borehole No	Date	Groundwater Level (m)	
		Depth	Elevation
RMN-A1	Upon Completion	Dry	
RMN-A2	Upon Completion	Dry	
RMN-UP1	Upon commencement of mud drilling at 3 m below grade	Dry	
RMN-UP2	September 11, 2024	5.8	231.8
RMN-UP3	Upon commencement of mud drilling at 3 m below grade	Dry	

Groundwater levels at the site will be subject to fluctuations due to seasonal changes, snowmelt and precipitation events. The water levels should be expected to be higher during the spring season and during and following periods of heavy precipitation or snow melt.

5.3 CHEMICAL TESTING

One representative sample from the soils at the site was tested for pH, water-soluble sulphate and chloride concentrations, and resistivity. The analysis results are provided in the following table.

Table 5.4: Results of Chemical Analysis

Borehole No	Sample No.	Depth (m)	pH	Chloride (µg/g)	Sulphate (µg/g)	Resistivity (Ohm-cm)
RMN-UP1	SS10	7.9	8.79	6	272	2730
RMN-UP2	SS7	4.9	8.46	5	318	2520
RMN-UP3	SS8	5.6	8.38	7	174	3560



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6.0 MISCELLANEOUS

The field work was carried out under the supervision of Mr. Muhammed Cuned and Mr. Harpreet Singh, under the direction of Mr. Gwangha Roh, P. Eng., Ph.D.

Both public and private utility locates were arranged by Stantec staff prior to initiation of drilling.

The drilling equipment was supplied and operated by London Soil Ltd. based in London, Ontario and DBW Drilling Ltd. based in North York, Ontario.

CPT, sCPT and MASW were carried out by ConeTec based in Richmond Hill, Ontario.

The borehole locations and elevations were surveyed by Stantec's Geomatics division based in London.

Geotechnical laboratory testing was carried out at Stantec's laboratories in Markham and Ottawa, Ontario.

This report was prepared by Roshan Rashed, M.Sc., P.Eng., and reviewed by Gwangha Roh, P. Eng., Ph.D., and Raymond Haché, M.Sc., P.Eng., Designated Principal MTO Foundation Contact.



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7.0 CLOSURE

A subsurface investigation is a limited sampling of a site. The subsurface conditions given herein are based on information gathered at the specific borehole locations. Should any conditions at the site be encountered which differ from those at the borehole locations, we request that we be notified immediately in order to assess the additional information.

Respectfully Submitted;

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FOUNDATION DESIGN REPORT For G.W.P. 3042-22-00 Ron McNeil Line Interchange Underpass

Highway 4 widening from Clinton Line to New Talbotville Bypass and New
Talbotville Bypass from Highway 4 to Highway 3 at Ron McNeil Line
West Region, Township of Southwold, County of Elgin, Ontario

8.0 DISCUSSIONS AND ENGINEERING RECOMMENDATIONS

8.1 PROJECT PURPOSE/DESCRIPTION

This project involves preliminary and detailed design of the Highway 4 widening from Clinton Line to the new Talbotville Bypass and new Talbotville Bypass from Highway 4 to Highway 3 at Ron McNeil Line (GWP 3042-22-00), and the Highway 3 widening from Ron McNeil Line to Centennial Avenue (GWP 3041-22-00). As part of the project, a new bridge structure (there is no existing bridge) will carry the realigned Ron McNeil Line over the future realigned Highway 3, about 80 northwest of the current intersection, in an area currently undeveloped. Based on the General Arrangement (GA) drawing, a two-span integral abutment bridge is proposed.

This section of the report provides foundation engineering recommendations for the detailed design and construction of the new Ron McNeil Line Overpass structure and associated approach embankments. The foundation design report is intended for the use of the Ministry of Transportation Ontario (MTO).

8.2 PROPOSED UNDERPASS STRUCTURES AND APPROACH EMBANKMENTS

Based on the GA Drawing provided by Stantec Structural team, a two-span structure with cast-in-place, post-tensioned concrete deck and integral abutments is proposed for the new underpass. The proposed bridge structure will be approximately 72 m long and approximately 21.8 m to 22.7 m wide and will be located almost perpendicular to the proposed alignment of Highway 3. The bridge abutments and central pier may be supported on driven steel H-piles. Wing walls at four bridge corners are also presented in the GA drawing.

The south and north approach embankment heights are planned to be approximately 7 m and 8 m, respectively. The embankments are planned to have 2 horizontal : 1 vertical side slopes and fore-slopes.

The new bridge and embankment construction will require staging and use of temporary roadway protection systems to keep the existing Highway 3 open.



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Key elevations associated with the proposed new overpass structure are as follows:

	South Side	North Side
Highway 3 grade (proposed)		Approximately Elev. 238 m
Proposed bridge abutment bottom	Approximately Elev. 239.3 m	Approximately Elev. 240.5 m
Proposed bottom of central pier pile cap		Approximately Elev. 235 m
Proposed top of approach embankment	Approximately Elev. 244 m	Approximately Elev. 246 m

8.3 DEGREE OF SITE UNDERSTANDING AND CONSEQUENCE CLASSIFICATION

The Canadian Highway Bridge Design Code (CHBDC S6-19) requires an assessment of the “degree of site and prediction model understanding” as a component of the geotechnical engineering investigation and/or services. The site and prediction model understanding consider the geotechnical properties of the soils underlying the site and the accuracy and degree of confidence regarding the numerical performance prediction models to be used to estimate the geotechnical serviceability limit states reactions and ultimate limit states resistances.

Based on the scope of subsurface investigations completed and available subsurface information related to this site, a “Typical degree of understanding” has been adopted for foundation design assessment purposes except that a “High degree of understanding” has been adopted for assessment of embankment stability. We considered “High degree of understanding” for the embankment stability analyses due to following reasons:

- Subsurface and groundwater conditions were investigated in accordance with the MTO guideline for foundation engineering services (typical degree of understanding). There is additional GEOCRESS borehole information available.
- Based on the subsurface conditions encountered along the bypass alignment, the stability of the embankment will likely be governed by the new embankment fills.
- The proposed embankment will be constructed using controlled & approved materials with a proper QC/QA program to meet OPSS. PROV 206 and 501 requirements.
- Advanced geotechnical laboratory tests were carried out (such as triaxial compression tests, standard proctor test and direct shear tests for the native founding soil and possible fill materials).
- One of two approach embankment boreholes was replaced with sCPT sounding to get continuous soil data.
- Observational approach (evidence of performance, as per the CFEM) - up to 13-meter-high cohesive fill embankments and 7-meter-high cut slopes (with a 2H:1V slope configuration) within the project limit have performed very well over the past five decades since their construction in the early 1970s.

The consequence classification has been selected as “Typical Consequence” in accordance with Section 6.5 of the Commentary on CHBDC S6-19. Should the consequence classification change, the



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foundation assessment and recommendations provided below should be reviewed and revised accordingly.

8.4 GEOTECHNICAL DESIGN PARAMETERS

The soil conditions encountered at the site generally consist of topsoil underlain by localized fill materials underlain by a thick deposit of stiff to hard clayey silt till. The surficial zone of the till was found to be firm and layers/zones of clayey silt with lower plasticity were also encountered interbedded within the clayey silt till deposit.

The results of the current and previous investigations indicate a consistency in site subsurface condition. A single geotechnical model (soil profile) has been prepared for the overpass structure foundation design, and the embankment stability and settlement evaluation.

The design soil profile is summarized in the following table and on Drawing Nos. E1 and E2 in Appendix E. The geotechnical parameters identified in the soil profile were developed based on a synthesis of the borehole data, the measured penetration resistance values, and laboratory index test results (including moisture contents) of soil samples obtained in the investigation.

The elevations provided on in the table reflect a synthesis of the borehole data; the Records of Borehole should be consulted to determine the conditions at specific locations.

Table 8.1: Geotechnical Model for the Ron McNeil Line Overpass

Elevation (m)		Soil Type	Design Soil Parameters			
From	To		Total Unit Weight ¹ γ (kN/m ³)	Drained Friction Angle $\phi'^{(2)}$ (°)	Undrained Shear Strength $S_u^{(2)}$ (kPa)	Compressibility Characteristics ³ E (MPa) or P' _c , C _r , C _c and C _v (cm ² /s)
Ground Surface	236	FILL: soft to stiff silty clay/ sandy clayey silt (RMN-A1, RMN-A2 and RMN-UP3) or Firm CLAYEY SILT (CL) TILL (RMN-UP1 and RMN-UP2)	21	28	60	P' _c =300 kPa C _r =0.02 C _c =0.15 C _v =3x10 ⁻³
236	225	Stiff to hard CLAYEY SILT (CL) TILL interbedded with hard CLAYEY SILT (CL-ML) (in RMN-A1, RMN-UP1 and RMN-UP2)	21.5	30	180 kPa	P' _c =400 kPa C _r =0.015 C _c =0.12 C _v =3x10 ⁻³
225	<190	Very stiff to hard CLAYEY SILT (CL) TILL	21	32	180 at EL. 225 m to 300 kPa at EL. 190 m	P' _c =600 kPa C _r =0.017 C _c =0.15 C _v =3x10 ⁻³

Notes:

¹ A groundwater level at an elevation of 232 m is recommended for use in foundation design. Submerged unit weight (γ') should be used below the groundwater level.

² The friction angles are applicable to drained conditions only and the shear strengths are applicable to undrained conditions only. These design parameters are estimated based on the SPT N values and CPT sounding results.

³ Compressibility Parameters: E = Soil Modulus, P'_c = Estimated Pre-consolidation Pressure, C_r = Recompression Index, C_c = Compression Index, C_v = Coefficient of Consolidation



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8.5 FROST PENETRATION

In accordance with OPSD 3090.101, the design frost penetration depth for foundations, f , at the site is 1.2 m. Therefore, all foundation elements such as footings and pile caps should be provided with a minimum of 1.2 m of soil cover or equivalent insulation for protection against frost heaving.

This depth of frost penetration should also be considered in the design of frost tapers adjacent to the bridge abutment and backfill zones.

8.6 SEISMIC DESIGN CONSIDERATIONS

8.6.1 Site Class

The seismic site class determination is based on the soil conditions in the upper 30 m of the stratigraphy as encountered in the boreholes for the Geotechnical Investigation.

A geophysical survey was conducted at the Site location on May 15, 2024, by ConeTec and the results were provided in a report dated June 10, 2024. The survey consisted of one Multichannel Analysis of Surface waves (MASW) test (MASW24-03). Based on the test results, a shear wave velocity (V_{s30}) value of 311 m/s was considered representative for the harmonic mean values over a 30 m depth. In this respect, this site is assessed to be Seismic Site Class D in accordance with Table 4.1 CHBDC S16-19. Shear wave velocities were also measured during the CPT sounding up to a depth of 15 m and comparable shear wave velocities to the MASWs (in respect to seismic site classification) were obtained.

The results of the geophysical survey (MASWs) are included in Appendix F and shear wave velocity measurements during CPT sounding are included in Appendix C for reference.

8.6.2 Seismic Performance Category

According to CHBDC S6-19 Section 4.4.4., a seismic performance category is assigned for each bridge based on the site-specific spectral acceleration, for a 2% in 50-year probability of exceedance, the fundamental period of the bridge, T , in the direction under consideration as well as the importance category. Spectral $S_a(0.2)$ and $S_a(1.0)$ values based on NBCC2020 for the site using an average weighted shear wave velocity of 311 m/s are provided in Appendix F.

As per the MTO Structural Manual (2024) Section 1 exceptions to the CHBDC S6-19, a seismic performance category of the bridge can be assigned based on $S_a(1.0)$ regardless of the bridge fundamental period (T) and seismic performance category 1 can be considered for all bridges (lifeline, major route and other bridges), when $0.05 \leq S_a(1.0) \leq 0.10$. For lifeline bridges in seismic performance category 1, detailing of structural elements should adopt requirement for seismic performance category 2 as a minimum. It is our understating that the proposed bridge is a major route bridge and above lifeline bridge detailing requirement is not appliable.



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8.6.3 Peak Ground Acceleration (PGA)

Seismic hazard values were obtained from Natural Resources Canada (2020 National Building Code), using a shear wave velocity (V_{s30}) value of 311 m/s. The table below summarizes the parameters obtained and recommended for use in the design based on a 2475-year return period.

Table 8.2: Peak Ground Acceleration Data

PGA^*	$S_a(0.2)^*$	$S_a(1.0)^*$
0.101	0.193	0.0785

Note * based on the average $V_s=311$ m/sec

The 2020 NBC Seismic Hazard calculation sheet is provided in Appendix F.

8.6.4 Liquefaction Potential

Seismic liquefaction is the sudden loss in stiffness and strength of soil due to cyclic loading effects of an earthquake. Liquefaction occurs due to increased pore water pressures that can arise from earthquake shaking (or other rapid loading). Under these conditions, the soil flows in a manner resembling a liquid until the shear stresses acting on the mass are as low as the reduced shear resistance.

The CHBDC describes saturated low-plastic silts exhibiting sand-like behaviour (e.g., $PI < 7$), sands, sand-silt mixtures, gravels confined by low permeability soil layers and gravel-sand mixtures, as having potential for liquefaction. The CHBDC references the use of the Bray et al. (2004) criteria for evaluation of liquefaction susceptibility in fine-grained soils. The Bray criteria includes consideration for a Plasticity Index < 12 and a ratio of the Natural Moisture Content to Liquid Limit > 0.85 as an indication of possible liquefaction.

Based on our local experience and site clayey soils' properties (plasticity, shear strength, sensitivity, OCR, natural moisture content close to or lower than plastic limit, etc.), shear strength degradation potential under anticipated earthquake condition is considered minimal (Idriss and Boulanger, 2008) and cyclic mobility not considered to be a significant issue for this project.

8.7 FOUNDATION OPTIONS

Both shallow and deep foundation options were evaluated for the proposed bridge in the following table. Shallow foundations would be placed within the stiff to hard clayey silt till below frost depth and deep foundations would extend deeper into the stiff to hard clayey silt till deposit.

The bridge structure is to be constructed on undeveloped land, about 80 m northwest of the existing interchange. Significant construction staging constraints are not anticipated.

The following table presents the advantages, disadvantages, relative assessment of cost and the risks/consequences for various foundation options for the abutment foundations of the proposed bridge structures from a foundations design and constructability perspective:



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Table 8.3: Comparison of Foundation Options for Ron McNeil Line Overpass

Option	Advantages	Disadvantages	Relative Cost	Risk/Consequences
Shallow Foundations - Central Pier Only	<ul style="list-style-type: none"> Ease of construction Lower foundation costs than deep foundations Feasible for the central pier 	<ul style="list-style-type: none"> Not suitable for integral abutments Potential for unacceptable settlement 	Low to medium	<ul style="list-style-type: none"> Potential excessive settlement Increased potential for differential settlement
Driven Steel H-Piles (Frictional Piles)	<ul style="list-style-type: none"> Reduced total and differential settlement compared to shallow foundations Reduced pile length compared to piles driven to 'refusal' feasible for an integral abutment 	<ul style="list-style-type: none"> Structural capacity of piles may not be fully utilized Risk of soil plugging between flanges due to very stiff to hard clayey soils Post construction foundation settlement and down drag load should be considered, if preloading is not carried out 	Medium	<ul style="list-style-type: none"> Long piles required for moderate capacities To avoid soil plugging (and related capacity reduction potential), piles should be driven without splicing or early pile splicing without time lag is required. Cobbles and boulders may be encountered in glacially derived soils that could impede pile penetration to required depths
Driven Steel Pipe Piles (Close-Ended & Frictional Piles)	<ul style="list-style-type: none"> Reduced total and differential settlement compared to shallow foundations Reduced pile length compared to piles driven to 'refusal' feasible for an integral abutment (based on the recent project experience) 	<ul style="list-style-type: none"> Structural capacity of piles may not be fully utilized Risk of soil plugging between flanges due to very stiff to hard clayey soils Post construction foundation settlement and down drag load should be considered if preloading is not carried out More vibration during driving and possible heave and displacement of ground and adjacent piles are anticipated 	Medium	<ul style="list-style-type: none"> Long piles required for moderate capacities Possible drivability and ground heaving issues Cobbles and boulders may be encountered in glacially derived soils that could impede pile penetration to required depths
Driven Steel H-Piles (End bearing on Refusal Materials)	<ul style="list-style-type: none"> Higher geotechnical resistances than frictional piles No or negligible settlement 	<ul style="list-style-type: none"> Excessive pile lengths Acceptable refusal materials (more than 3 m of SPT over 100 blows/0.3 m materials) were not encountered 	Medium to high	<ul style="list-style-type: none"> Cobbles and boulders may be encountered in glacially derived soils that could impede pile penetration to required depths



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Option	Advantages	Disadvantages	Relative Cost	Risk/Consequences
	<ul style="list-style-type: none"> End bearing piles avoids issues related to soil plugging between flanges 	<ul style="list-style-type: none"> during the foundation investigation 		<ul style="list-style-type: none"> Additional foundation investigation is required to confirm the pile end bearing soils and refusal.
Drilled Shafts (Frictional Cast-in-place Concrete Piles) - Central Pier Only	<ul style="list-style-type: none"> Can support/resist higher axial and lateral loads than driven steel piles Feasible for the central pier 	<ul style="list-style-type: none"> Not suitable for integral abutments Caissons would extend below the water table. Temporary liners (remove after concreting) or drilling mud may be required Separate caisson drill rig mobilization 	High	<ul style="list-style-type: none"> Liners and drilling mud likely required due to presence of groundwater. Use of “wet” installation methods precludes ability to review/confirm materials at the base of the caissons and assess the potential for reduced capacity
CFA Piles (Continuous Flight Auger Piles) - Central Pier Only	<ul style="list-style-type: none"> Feasible for the central pier only Lower vibration than pile driving Relatively faster construction than drilled shafts Cheaper unit construction cost than drilled shaft 	<ul style="list-style-type: none"> Not suitable for integral abutments Reinforcement should be inserted into wet concrete and installation depth of reinforcement is limited Pile batter is not feasible 	Medium	<ul style="list-style-type: none"> Relatively new technology for MTO bridges Pile integrity and load tests requirements

Based on the comparison presented in the above table, frictional steel H piles driven through the stiff to hard clayey silt till deposit are recommended for the abutments and central pier as the preferred option from the foundation’s perspective. Driven steel H-piles are also presented in the GA drawing (Appendix A).

Depending on loading conditions, drilled caissons and shallow foundations can also be considered for the central pier.

For the Ron McNeil Underpass structure, it has been assumed that the embankments will be constructed prior to pile driving to preload the site with sufficient waiting period, to avoid imposing embankment related settlements on the bridge structure and downdrag on the abutment piles. The Ron McNeil underpass will be constructed off the alignments of the existing Highway 3 and Ron McNeil Road, approximately 80 m northwest of the existing intersection on undeveloped land, and in this respect no constraints are anticipated for preloading. If preloading is not feasible due to construction schedule, consideration should be given to the use of light weight fill for the bridge approach embankments.

8.7.1 Driven Pile Foundations

The design recommendations presented in the following sections have been developed in accordance with the requirements and methods described in the CHBDC.



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8.7.1.1 Design Considerations

Driven pile foundations consisting of steel H-piles, deriving their load-carrying capacity from both shaft friction and tip resistance, can be used to support the abutments and pier of the proposed underpass structure. Closed-end pipe piles are not recommended as they would displace more soil than H-piles during installation which could lead to deformation/heave of adjacent piles and the adjacent ground during pile installation. Closed-end pipe pile driving will also generate significant higher vibration than steel H-piles.

The driving of steel H-piles for the new underpass is not expected to adversely affect the surrounding area. Therefore, vibration monitoring is not anticipated to be required.

Given the length of the H-piles required to support the proposed underpass structure, pile splicing may be required. If H-Piles cannot be driven without splicing, it is recommended to utilize **a short initial pile length (shorter than 12.5 m) and minimize the time lag for pile slicing (no longer than 24 hours)** to mitigate the soil plugging issues noted in the nearby MTO bridge site (Highway 401-Wonderland Road). Longer initial pile length and longer time lag for pile slicing could develop sufficient soil-plug between pile flanges and it could be further dragged down with piles to eventually reduce both short-term and long-term pile capacities significantly.

Piles should be supplied and installed/constructed in accordance with the requirements of OPSS.PROV 903– Construction Specification for Deep Foundations.

8.7.1.2 Geotechnical Axial Resistance

Geotechnical Axial Resistance in Compression

The axial resistance at Ultimate Limit State (ULS) for driven steel 310x110 and 360x132 H-piles were assessed using the API (American petroleum institute) design methods with the program APILE (Ensoft, 2019). The geotechnical model outlined in Table 8.1 and on Drawing No. E1 was used as input to these analyses.

The factored geotechnical resistances at Ultimate Limit States (ULS_r) outlined in Table 8.4 may be used in design.

Table 8.4: Factored Geotechnical Resistances at ULS and at SLS – Pile Foundations

Pile Type	Anticipated Pile Length (m)	Anticipated Pile tip Elevation ¹ (m)	Factored Geotechnical Resistance at $ULS_r^{2,3}$ (kN)	Factored Geotechnical Resistance at SLS_r^3 (kN)
South Abutment				
HP 310x110	35	204.3	1350	1025
HP 360x132	30	209.3	1350	1025



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Pile Type	Anticipated Pile Length (m)	Anticipated Pile tip Elevation ¹ (m)	Factored Geotechnical Resistance at ULS _r ^{2,3} (kN)	Factored Geotechnical Resistance at SLS _r ³ (kN)
North Abutment				
HP 310x110	35	205.5	1350	1025
HP 360x132	30	210.5	1350	1025
Pier				
HP 310x110	35	200	1350	1025
HP 360x132	30	205	1350	1025

Notes:

- ¹ Pile lengths and tip elevations are based on the underside of the abutment walls as provided above in Section 8.2.
- ² In accordance with Table 6.1 in the CHBDC, the ULS Geotechnical Resistances were determined based on a consequence level of "Typical" with a consequence factor equal to 1.
- ³ In accordance with Table 6.2 in the CHBDC and the site and prediction model understanding classification of "Typical", a resistance factor of 0.4 (static analysis, compression) has been used in calculating the factored geotechnical resistance at Ultimate Limit State (ULS) and a resistance factor of 0.8 (static analysis, settlement, and lateral deflection) has been used in calculating the factored geotechnical resistance at Serviceability Limit State (SLS).

The estimated geotechnical reaction at SLS_r for a 25 mm vertical settlement exceeds the geotechnical reaction at ULS_r given above. No ground settlement induced by the proposed embankment loading was considered for the above geotechnical pile resistance at serviceability condition. It should be noted that the actual pile settlement will be dependent on the ground settlement at the pile neutral plane and group pile arrangement. As discussed later, the embankments should be constructed prior to pile driving operations to preload the site with sufficient waiting period to mitigate pile foundation settlement and downdrag.

It should be noted that lighter or heavier pile sections in similar dimension H-piles may be considered based on the structural considerations. However, these heavier sections with similar dimension H-piles can be considered to be geotechnically equivalent and the axial and lateral capacities provided in this report can also be used for the similar dimension H-piles with heavier sections.

Axial Resistance in Tension

The axial resistance in tension at Ultimate Limit State (ULS) for driven steel HP 310x 110 and HP 360 x 132 piles were assessed using the API (American petroleum institute) design method with the program APILE (Ensoft, 2019). The geotechnical model outlined in Table 8.1 was used as input to the analysis. For design against uplift, the tensile resistance provided in the following table is recommended.

Table 8.5: Recommended Uplift Resistance – Pile Foundations

Pile Type	Assumed Pile Length (m)	Factored Geotechnical Resistance (Tension) at ULS _r (kN)
South and North Abutments and Pier		
HP 310x110	35	1000
HP 360x132	30	1000



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A resistance factor, ϕ_{gu} , of 0.3 has been applied to calculate the ULS resistance. The factored geotechnical resistance (tension) at ULS provided above does not include the self-weight of the pile.

Minimum Pile Spacing

A minimum pile spacing of 2.5 diameter (centre to centre) of pile, but not less than 0.75 m should be maintained as per the CHBDC.

8.7.1.3 Downdrag

The proposed Ron McNeil Line Underpass approach embankments will be about 7 to 8 m high and will induce long-term consolidation settlement of the underlying clayey silt to clayey silt till soils. The anticipated consolidation settlement is time-dependent and will not be completed during the embankment construction, unless the embankments are placed in advance (i.e. not less than four months in advance to reduce residual settlement to the tolerable range) of bridge construction (including pile driving) or other settlement mitigation measures are implemented (e.g. surcharge in conjunction with preloading, lightweight fill embankments). Post-construction settlement of site cohesive soil deposits relative to the piles of more than 0.4 inch (≈ 10 mm) will result in development of downdrag loads acting on the piles.

The neutral plane of piles is anticipated to be located approximately one third of pile length below the pile head (with consideration of permanent pile load which can be considered to be close to the factored SLS pile capacity). The unfactored down drag loads acting on the HP 310 x 110 piles and HP 360 x132 piles are anticipated to be about 600 kN, if the site is not preloaded sufficiently or light weight fill is not utilized as per the section 8.9.2 of this report. The downdrag loads have no impacts on the geotechnical axial pile capacity and should not be considered in geotechnical pile design. The pile structural capacity should be properly assessed with consideration of downdrag loads as per the CHBDC.

No down drag is expected for the piles at the central pier.

8.7.1.4 Relaxation of Piles

For H-piles deriving their capacity predominantly from friction within the clayey silt till, relaxation and reduction of pile capacity is not considered to be a concern.

8.7.1.5 Soil Setup and Pile Capacity Testing

Pile/Soil setup effect is a natural phenomenon where pile load capacity increases over time as the results of dissipation of pore-water pressure. The magnitude of pile/soil setup is governed by three main factors: pile slenderness ratio, elapsed time, and type of surrounding soil.

Piles will be driven through significant thickness of clay/clayey soils at the site. Piles driven in cohesive soils generally gain capacity after driving has been completed and excess pore pressures have dissipated (i.e., the capacity of friction piles in clayey soils increases with time). The ULS_r capacities identified in the previous sections represent the 'long-term' capacities of the piles. Capacities determined by static pile testing or restriking of piles (particularly piles that derive most of their capacity from skin friction) at the



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time of, or shortly following, driving would not be expected to equal the long-term capacities. To determine the actual, long-term pile capacities the following pilot pile test procedures are recommended to be carried out.

- At each abutment, two of the production piles should be driven to the targeted tip elevation while full-time Pile Driving Analysis (PDA) testing is carried out to obtain the initial drive resistances.
- These ‘test piles’ shall remain in place for two weeks to allow for 14 days of soil set-up to occur.
- PDA testing of the piles shall be carried out on day 14.
- The result of the day-zero and day-14 results will be used to project the capacities after one year using the following relationship.

$$Q_t = Q_o \left(A \log \left(\frac{t}{t_o} \right) + 1 \right) \quad \text{Skov and Denver, 1988}$$

The ‘A’ constant will be determined based on the setup determined at day-zero and day-14, followed by calculation of Q_{365} which will be considered the long-term capacity of the piles.

For H-piles deriving their capacity predominantly from friction within the very stiff to hard clayey silt and clayey silt till, relaxation and reduction of pile capacity may not be a concern.

The Hiley Formula as defined on Structural Drawing SS103-11 should be applied to each driven pile to provide a relative comparison between piles where PDA testing is carried out and the remaining piles. The “Hiley Formula Pile Resistance” for all piles shall be submitted to the geotechnical engineer for comparison with the PDA tested piles.

As per OPSS.PROV 903 (Section 903.07.02.07.06), 10% of the piles rounded up to the next whole number but no fewer than two piles in each pile group, shall be re-tapped to confirm that the ultimate axial resistance has been sustained.

Piles should be supplied and installed/constructed in accordance with the requirements of OPSS.PROV 903 – Construction Specification for Deep Foundations.

The following pile note should be included in the “Pile Data Table”:

- The pile driving equipment shall be appropriate to the driving conditions and capable of delivering a minimum specified hammer energy of 70 kJ.

The following “Pile Driving Note” should be included on the structural drawings:

- Piles to be driven in accordance with Standard SS 103-11 and PDA testing using an ultimate geotechnical resistance of 3000 kN per pile (HP 310x110) based on a geotechnical resistance factor of 0.5.
- If piles are planning to drive with a splicing, initial pile driving length prior to splicing shall be limited to 12.5 m and time lag for a pile splicing shall be no longer than 24 hours.



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The specified resistance load per pile in the note above is dependent on the pile size selected and the structural load planned to be supported on each pile and is equal to twice the factored geotechnical resistance at ULS for the selected pile type.

8.7.1.6 Drivability

The pile driving equipment shall be appropriate to the driving conditions and capable of achieving the design pile capacity. The pile termination or set criteria should be dependent on the pile driving hammer type, helmet, select pile size and length. The set criteria should be established at the time of pile driving once the equipment is decided.

The site soil generally consists of stiff to hard clayey silt clayey silt till. As such, the site is not expected to pose unusual resistance to pile driving although the presence of larger particles such as cobbles and boulders should be anticipated based on the type of soil deposits encountered and the conditions encountered in the boreholes. Pile tip should be reinforced using Titus point, APF hard bite or equivalent.

8.7.1.7 Geotechnical Lateral Resistance

P-Y Curves

The response of a pile to lateral loads is a non-linear relationship. Non-linear elastic-plastic springs (i.e., p-y curves representing the load intensity per unit length of pile (p) versus the lateral deflection of the pile) can be used in evaluating the structural response of the pile in response to lateral loads.

The program LPILE 2019 developed by Ensoft, Inc. (Ensoft, 2019) was used to develop p-y curves for HP 310x110 and HP 360x132 piles. Separate models were developed for the south and north abutments and pier. It is noted that the top 3 m of the abutment piles are anticipated to be placed in Corrugated Steel Pipes (CSPs) filled with loose sand and this has been considered in the analysis. The p-y curves for the pier have been provided for vertical piles and can be adjusted for battered piles when the pile batter is determined. The geotechnical parameters provided in Table 8.1 were used in the analyses.

The p-y curve values versus depth at each foundation unit are presented in Tables E1 to E3 and Figures E3 to E5 for HP 310x110 and Tables E4 to E6 and Figures E6 to E8 for HP 360x132, included in Appendix E. These tables provide a series of curves obtained from the LPILE program generated for selected depths below the pile head. The p-y curves can be used in the structural evaluation of the H-piles noting that the p-y curves provided are unfactored and that appropriate resistance factors (i.e., as outlined in Table 6.2 of the CHBDC, 2019) should be applied when assessing the geotechnical lateral resistance of the piles at ULS and SLS.

Group Action

The horizontal resistance of piles should consider the group action of piles (pile interaction) in accordance with Section 6.11.3.4 and the associated commentary of the CHBDC.



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Group action of piles (pile interaction) for lateral loading should be considered if centreline spacing of piles is less than 8 pile diameters (or least lateral dimension of pile) parallel to the direction of lateral load or less than 4 pile diameters perpendicular to the load.

The effect of interaction between piles can be considered by applying a reduction factor to the soil resistance (i.e. the p-multiplier) of a single pile to obtain p-y curves for the pile group. The reduction factors to be applied are dependent on the pile spacing/group geometry. The reduction factors (i.e. p-multipliers) outlined in Figures C6.22 to C6.24 of Section C6.11.3.4 of the CHBDC should be used. The following reduction factors may be used to account for pile group action:

Table 8.6: Recommended Reduction Factors for Pile Groups

Pile spacing / pile diameter	Reduction Factor	Pile spacing / pile diameter	Reduction Factor
Load Parallel to Pile Spacing		Load Perpendicular to Pile Spacing	
7	1.0	4	1.0
4	0.8	3	0.9
3	0.7	2	0.75
2	0.6	-	-

8.7.2 Shallow Foundations – Central Pier

The design recommendations presented in the following sections have been developed in accordance with the requirements and methods described in the Canadian Highway Bridge Design Code (CHBDC, 2019).

8.7.2.1 Design Considerations

Depending on the loading conditions, the pier for the proposed overpass can be supported on shallow foundations founded on the stiff to hard clayey silt till soils encountered below elevation 236 m. The footings should be provided with a minimum of 1.2 m of earth cover to provide adequate protection against frost penetration.

In preparation for construction of the new bridge pier foundations, all organic soil (including topsoil), existing fill materials and any loose, wet, and/or otherwise disturbed native material should be removed from within the footprint of the proposed pier foundations.

Following completion of the preparation of the founding surface, a milestone inspection should be conducted by a Foundation Engineering Specialist in accordance with OPSS 902 to confirm that the subgrade has been suitably prepared.



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8.7.2.2 Geotechnical Resistance and Reaction

The factored geotechnical resistances at Ultimate Limit States (ULS) and Serviceability Limit States (SLS) outlined in the following table may be used in design of shallow footings founded on the very stiff to hard clayey silt till soils.

Table 8.7: Factored Geotechnical Resistances at ULS_f and at SLS_f – Shallow Foundations (Piers)

Footing Width (m)	Founding Elevation (m)	Factored Geotechnical Resistance at ULS _f (kPa)	Factored Geotechnical Reaction at SLS _f for 25 mm Settlement (kPa)
3	235	400	200
4	235	440	170

In accordance with Table 6.2 in the CHBDC, a resistance factor of 0.5 has been applied in calculating the factored geotechnical resistance at Ultimate Limit State (ULS_f).

In accordance with Table 6.2 in the CHBDC, a resistance factor of 0.8 has been applied in calculating the geotechnical reactions at Serviceability Limit State (SLS_f).

The geotechnical resistances are provided for loads applied perpendicular to the surface of the footings. Where this is not the case, eccentricity and inclination of the loads must be considered.

8.7.2.3 Geotechnical Horizontal Resistance

The unfactored horizontal resistance to sliding of the shallow footings for the proposed bridge may be calculated using the following unfactored coefficient of friction:

- 0.4 between cast-in-place concrete and clayey silt till subgrade (friction)
- but not exceeding an adhesion value of 65 kPa (cohesion)

The double constraint reflects the clay and clayey-silt nature of the till anticipated at the potential pier footing elevations.

In accordance with Table 6.2 of the CHBDC, a resistance factor against sliding of 0.8 (frictional) should be applied to obtain the resistance at ULS_f, and of 0.6 for the resistance based on adhesion (cohesion).

8.7.3 Drilled Caissons – Central Pier

The proposed bridge piers (integral piers) may also be supported on drilled caissons. The drilled caissons can be installed within the stiff to hard clayey silt till using the following factored geotechnical resistances:



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Table 8.8: Factored Geotechnical Resistances at ULS and at SLS – Drilled Shafts (Piers)

Foundation Elements	Drilled Shaft Length (m)	Diameter (m)	Factored Geotechnical Resistance at ULS _r (kN)	Factored Geotechnical Reaction at SLS _r for 25 mm Settlement (kN)
Piers	25	0.76	2500	2000
Piers	25	0.90	3200	2600

The ULS_r geotechnical resistance includes a resistance factor of 0.4 and SLS_r includes a resistance factor of 0.8.

Higher geotechnical resistances are available for larger or longer caissons.

Drilled shaft foundations should be constructed in accordance with OPSS.PROV 903. If drilled shaft foundations are selected for the piers, temporary liners (retrieved after the concreting) and/or drilling mud may be required to support the overburden soils during construction to minimize disturbance to the side walls. The liner should be advanced while filled with drilling mud to minimize the potential to control base disturbance. In addition, placement of concrete by tremie methods is recommended. Consideration must also be given to the potential presence of cobbles and boulders within the native soil deposits. Appropriate construction equipment and techniques must be selected to handle the anticipated cobbles and boulders.

The performance of drilled shafts will depend upon the shaft side walls and base quality. Each drilled shaft excavation should be cleaned to remove all loosened debris. Following construction of the drilled shafts, in particular if drilling mud and tremie concrete are used, it is recommended that non-destructive testing of the shafts be carried out, which could include pile integrity testing, cross-hole sonic logging, or thermal integrity testing.

Downdrag loads are not expected for the drilled caissons for the central piers.

A minimum caisson spacing of 2.5 diameter (centre to centre) of caisson, but not less than 0.75 m should be maintained as per the CHBDC.

Should the use of caisson be the preferred option to support the proposed bridge central piers, a separate analysis will need to be carried out using proposed diameter of caisson and reinforcement details for the lateral pile capacity and deflection analyses.

8.8 LATERAL EARTH PRESSURES

8.8.1 Abutment Backfill

Ontario Provincial Standard Drawing (OPSD) 3101.150 outlines the required extent of the granular backfill zone at the bridge abutments. The materials used as backfill behind the proposed bridge abutments should consist of free-draining granular fill placed and compacted using methods and



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equipment appropriate to the type of structure. For the purpose of this report, it is assumed that backfill materials meeting the requirements of OPSS Granular B (Type I or Type II) or Granular A materials will be used.

Excavation and backfill for the new bridge structure should be carried out in accordance with OPSS.PROV 902- Construction Specification for Excavation and Backfilling – Structures. Backfill materials should meet the requirements of OPSS.PROV 1010 and be placed and compacted in accordance with the requirements of OPSS.PROV 206 and OPSS.PROV 501, respectively.

8.8.2 Static Lateral Earth Pressures

Static lateral earth pressures will need to be considered in the design of abutments and retaining walls (wingwalls). These structures should be backfilled using imported free-draining granular fill materials meeting the gradation requirements of OPSS Granular A or Granular B Type I materials.

Computation of earth pressures should be in accordance with Section 6.12 of the CHBDC. For retaining walls that are designed to allow rotation, active earth pressure may be used for design. For rigidly tied and unyielding structures, the at-rest earth pressure should be used for design. The effects of compaction should be accounted for by applying a compaction surcharge as outlined in Section 6.12.3 and as shown in Figure 6.8 of the CHBDC. Where applicable (i.e., where unbalanced water pressures may develop), the structures should also be designed to account for hydrostatic pressures.

The total at rest, (P_O) active (P_A) and passive (P_P) thrusts can be calculated using the following equations:

$$P_A = \frac{1}{2} K_a \gamma H^2$$

$$P_O = \frac{1}{2} K_o \gamma H^2$$

$$P_P = \frac{1}{2} K_p \gamma H^2$$

where H is the height of the wall and γ is the unit weight of the backfill soil. Values for K_a , K_p , K_o and γ are provided in Table 8.9 for horizontal backfill conditions. These values should be adjusted if sloped backfill is considered. The thrust acts at a point one third up the height of the wall.

Table 8.9: Recommended Non-Seismic Earth Pressure Parameters (Horizontal Backfill)

Parameter	OPSS Granular B Type I	OPSS Granular A and Granular B Type II
Bulk Unit Weight, γ (kN/m ³)	22	22
Effective Friction Angle	32°	35°
Coefficient of Earth Pressure at Rest (K_o)	0.47	0.43
Coefficient of Active Earth Pressure (K_a)	0.31	0.27
Coefficient of Passive Earth Pressure (K_p)	3.25	3.69

*This material should be tested to confirm the friction angle and compacted density as per relevant OPSSs



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8.8.3 Seismic Lateral Earth Pressures

The following design parameters are provided for use in assessing the earth pressures induced on the bridge abutment and wingwalls under seismic loading conditions.

The total active and passive thrusts under seismic loading conditions can be calculated using the following equations:

$$P_{AE} = \frac{1}{2} K_{AE} \gamma H^2 (1 - k_v)$$

$$P_{PE} = \frac{1}{2} K_{PE} \gamma H^2 (1 - k_v)$$

where:

K_{AE} = active earth pressure coefficient (combined static and seismic)

K_{PE} = passive earth pressure coefficient (combined static and seismic)

H = height of wall

k_h = horizontal acceleration coefficient

k_v = vertical acceleration coefficient

γ = total unit weight

For this site, the following design parameters were used to develop the recommended K_{AE} and K_{PE} values as per CHBDC 2019.

Table 8.10: Seismic Design Parameters to Estimate Lateral Earth Pressures

<i>PGA</i>	Horizontal Acceleration Coefficient, k_{ho}	Horizontal Acceleration Coefficient, k_h
	Non-Yielding	Yielding (<i>wall movements of 25 mm to 50 mm</i>)
0.101g	0.101	0.051
Note: k_{ho} is the seismic horizontal acceleration coefficient that corresponds to zero wall movement and is equal to the <i>PGA</i> estimated at ground surface. The vertical acceleration coefficient (k_v) should be ignored in the calculations as per CHBDC 2019, section C4.14.7.2.		

The angle of friction between the soil and the wall has been set at 0° to provide a conservative estimate.

The seismic earth pressures may be calculated using the parameters detailed in Table 8.9 for horizontal backfill configuration. These values should be adjusted if sloped backfill is considered.

Table 8.11: Recommended Seismic Earth Pressure Parameters (Horizontal Backfill)

Parameter	OPSS Gran B Type I	OPSS Gran A and Gran B Type II
Bulk Unit Weight, γ (kN/m ³)	22	22
Effective Friction Angle	32	35
Passive Earth Pressure, (KPE)	3.16	3.59



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Parameter	OPSS Gran B Type I	OPSS Gran A and Gran B Type II
Height of Application of PPE from base as a ratio of wall height, (H)	0.326	0.326
Yielding Wall		
Active Earth Pressure (K_{AE}) for Yielding Wall	0.34	0.30
Height of Application of P_{AE} from base as a ratio of wall height, (H) for Yielding Wall	0.357	0.358
Non-Yielding Wall		
Active Earth Pressure (K_{AE}) for Non-Yielding Wall	0.37	0.33
Height of Application of P_{AE} from base as a ratio of wall height, (H) for Non-Yielding Wall	0.378	0.380

8.9 APPROACH EMBANKMENT DESIGN CONSIDERATIONS

As part of the project, new approach embankments will be constructed to carry the Ron McNeil Line over Highway 3. Based on the GA drawing, the south and north approach embankments will be up to about 7 m and 8 m higher than the surrounding grade, respectively.

Based on the GA and embankment cross-section drawings, the proposed embankment will be constructed with typical 2H:1V side slopes. Given that the overall embankment height will be 8 m, a mid-slope bench is not required for maintenance as per OPSD 202.010. It is typical that embankments higher than 4.5 m be constructed using OPSS 1010 SSM or better materials. Based on the project specific cut material reusability assessment, MTO Embankment Settlement Criteria for Design (dated July 2010) and project specific slope stability assessment, the site deep cut materials may be utilized in up to 4.5 m high embankments. All embankment construction should be carried out in accordance with relevant MTO standards such as OPSS.PROV 206 (subgrade preparation embankment construction) and OPSS.PROV 501 (compaction, quality control). Due to the possible pile driving through the newly built embankment, larger particles shouldn't be used for the embankment construction.

To reduce surface water erosion on the embankment side slopes, topsoil and seeding as per OPSS.MUNI 802 (Topsoil) and OPSS.PROV 804 (Seed and Cover) should be carried out as soon as possible after embankment construction. It is also imperative that the designs include provisions for preventing surface water flow on the embankment side slope face. Consideration can be given to using a mountable curb and gutter arrangement to control and divert surface water away from the top of the slope. Surface water must be properly directed to armored outfalls/outlets designed to drain into highway ditches.

8.9.1 Embankment Stability

Given the consistency of subsurface conditions encountered throughout the site, slope stability analyses were carried out at the critical location of the north approach embankment (i.e., section where the embankment is highest, right at the abutment) using the commercially available slope stability analysis



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software, SLOPE/W (GeoStudio 2021). The input geotechnical design parameters are summarized in Table 8.1 and Table 8.12 below. A horizontal seismic load coefficient of 0.051g (equal to half the site Adjusted PGA) was used for the seismic/pseudo-static slope stability evaluation.

Table 8.12: Geotechnical Design Parameters – Ron McNeil Line Underpass Embankment

Soil Type	Design Soil Parameters			
	Total Unit Weight ¹ γ (kN/m ³)	Effective Friction Angle ϕ' (°)	Effective Cohesion C' (kPa)	Undrained Shear Strength S_u (kPa)
New Granular Embankment Fill	21.0	30	0	-
New Cohesive Embankment Fill * (Consistency Index $I_c^{**} > 0.85$ and compacted to at least 95% of the material's SMPDD)	20.5	26	2.5	50

Note:

* Based on the direct shear test and consolidated undrained triaxial compression test results on the reconstituted clay samples from the project deep cut area (compacted to 95% of its SMPDD)

** Consistency Index, $I_c = (\text{Liquid Limit} - \text{Natural Moisture Content}) / (\text{Liquid Limit} - \text{Plastic Limit})$

A minimum factor of safety (FOS) of 1.33 to 1.43 (corresponding to resistance factors 0.70 and 0.75 as per the MTO Provincial Engineering Memorandum # 2020-01 dated March 23, 2020) is considered acceptable against static, deep-seated embankment instability depending on where the majority of slip circle is located. For seismic analyses, a minimum FOS of 1.1 is considered acceptable against pseudo-static, deep-seated embankment instability.

The results of the slope stability analysis are presented on Figures E9 to E14 in Appendix E. The results of these stability analyses indicate that the proposed embankment with a 2H:1V side slope is acceptable (FOS > 1.33 for slip surfaces within the embankment in static condition, FOS > 1.43 for deep-seated slip surfaces in static condition and FOS > 1.1 for pseudo-static condition). The factors of safety for the south approach embankment are anticipated to be higher than the north approach embankment.

8.9.2 Embankment Settlements and Settlement Mitigation Measures

The settlement performance criteria for design of high fill embankments are in accordance with the MTO Embankment Settlement Criteria for Design dated July 2010. As per the Figure 2 of that MTO criteria, below longitudinal transitions should be achieved for the post construction settlement of 20 years for King's Highway and Freeways.



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Table 8.13: Longitudinal Transitions (MTO Embankment Settlement Criteria for Design)

	Settlement Limits (mm)			
Distance from Transition Point (Abutment)	0-20 m	20-50 m	50-75 m	>75 m
Non-Freeways	25	50	100	200

The above criteria also have a differential settlement limit of 100:1 for Non-freeways.

The proposed embankment will induce settlement of the native soils (immediate settlement for granular soils and recompression of cohesive soils). A pseudo-three-dimensional settlement analysis using Rocscience Settle3 was carried out for the most critical embankment cross section of the embankments to check the magnitude of settlements across the embankment crest.

The soil parameters provided in Table 8.1 and CPT sounding results were used in analyses; the results are presented in Figure E15 and E16, provided in Appendix E. The maximum settlement along the embankment crest within the proposed Ron McNeil Line pavement section is estimated to be less than 130 mm and 110 mm for the north and south approach embankments, respectively. Total settlement magnitudes obtained from Settle 3D analyses based on the consolidation test results were also compared with CPT based settlement assessment using constrained modulus (M). The analysis indicates that 95% of the settlement will occur within 4 months after the construction of the embankments.

In addition to the above settlement, the self-weight settlement of new fill should also be considered. Approximately 0.5% of the new fill height (40 mm and 35 mm for a 8 m and 7 m high embankment, respectively) is generally considered as a self-weight settlement for well-compacted inorganic granular earth fills, which can take one to two years to complete, depending on material type and degree of compaction. Self-weight settlement of well- compacted OPSS 1010 SSM, and Granular A and B materials are generally significantly less than that of inorganic granular and cohesive earth fill.

PRELOADING (PLUS POSSIBLE SURCHARGING)

The post-construction embankment settlement (subgrade settlement plus self-weight fill settlement) should meet the MTO Embankment Settlement Criteria for Design, dated July 2010 (total settlement of 100 mm and differential settlement of 100:1 for non-freeway & longitudinal transitions). To meet these criteria, it is recommended that the site should be properly preloaded (extend the full height embankment load at least 5 m beyond the abutments) for at least four (4) months. The preloading is to minimize development of downdrag forces on the abutment piles, and the overall settlement of foundations supported on piles as well as the future Ron McNeil Line maintenance. Additionally, consideration can be given to using a surcharge to expedite the embankment settlement process.

LIGHT WEIGHT FILL

If preloading and/or surcharging are not feasible due to construction schedule and other constraints, consideration should be given to the use of light weight fill such as expanded polystyrene (EPS) to build the proposed overhead approach embankments. Due to the anticipated volume of the light-weight fill, it



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may be also worthwhile to consider light weight foam concrete (cost effectiveness will be better for a larger volume of light weight foam concrete).

For EPS and light weight foam concrete embankment options, driven steel piles may be the preferred foundation option to support the proposed integral abutment structure.

Since the post construction settlements of the embankment in the transition zone exceed the MTO settlement criteria, preloading beyond the light weight fill embankment may still be required. The extents of light weight fill can be determined based on the deep foundation and integral abutment arrangement, required foundation and underpass construction workspace and the regular earth fill embankment fore-slope configuration beyond the light weight fill embankment. To minimize the pile downdrag potential, the following approximate light weight fill configuration would be anticipated:

- The normal weight embankment would be constructed with a 2H:1V foreslope towards the proposed pile cap and the toe of the foreslope would terminate 8 m from the edge pile cap.
- The bottom of EPS blocks and base of light weight foam concrete should extend laterally 8 m beyond the pile cap edge.
- For the EPS
 - Within the 8 m offset noted above, the EPS blocks would extend vertically from the existing ground surface to within 1.5 to 1.2 m from the finished grade, depending on the pavement design requirements.
 - Beyond the 8 m offset noted above, the EPS blocks would be stepped into the normal weight embankment fill such that the base of the EPS blocks would rise at an average slope of 2H:1V.
 - The top of the EPS blocks would be constructed with appropriate environmental protection such as sufficient granular fill cover, a wire mesh reinforced concrete slab and polyethylene sheeting.
- For the light weight foam concrete
 - Flowable light weight foam concrete will be poured in staged using foam works to build the embankment between bridge abutment and 2H:1V earth embankment foreslope
 - On top of the cured light weight foam concrete, highway pavement structure and/or earth cover will be placed.

The use of light weight fill to restrict settlements at the foundations would alleviate the horizontal earth pressure thrusts on the structure of retaining walls.

The proposed 2H:1V interface slope between the light weight fill and the normal weight backfill assumes that the portion of the embankment constructed with normal weight fill will be constructed at least a few months prior to construction of the rigid frame structure and subsequent placement of the light weight fill; this will minimize differential post-construction settlements at the highway pavement level.

This embankment settlement mitigation measure is considered viable (mainly due to the project schedule) at the time of this report's preparation.

Due to the lighter unit weight of EPS (approximately 0.5 kN/m^3) and light weight foam concrete (approximately 5 kN/m^3), the anticipated pressures acting on the bridge abutment walls will be



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significantly lower than regular soil backfill. At-rest earth pressure coefficient of backfill material can be estimated using backfill material's poisson ratio, ν by using following equation, $K_0 = \nu / (1 - \nu)$. Typical poisson ratio of EPS is in between 0.1 and 0.2, and light weight foam concrete is in between 0.2 and 0.3. Active and passive earth pressure coefficients of light weight fill could be estimated based on the at-rest coefficient corresponding backfill material's friction angle. In general, active earth pressure coefficient is less than at-rest earth pressure coefficient, and passive earth pressure coefficient is significantly larger than at-rest earth pressure coefficient. The actual earth pressure coefficient of light weight fill material should be discussed with the material supplier with consideration of material properties and fill placement scheme.

DO NOTHING

Because the anticipated post construction settlements are not too excessive and relatively uniform settlements throughout the overhead structure and embankments are expected based on the subsurface conditions and comparable embankment loading to the overhead structure's serviceability structural loading to the shallow foundation (when EPS or light weight foam concrete is used to reduce the lateral earth pressure to the acceptable level), a typical embankment constructed using OPSS granular materials with a shallow foundation to support the proposed overhead structure without any settlement mitigation may also be considered. The settlement tolerance of structure should be reviewed and approved by the project structural designer to evaluate this option. As discussed in the preceding section, the majority of settlement (95% of consolidation settlement) will occur within 4 months following the completion of the bridge and embankment construction.

This option may also be considered for the embankment beyond the light weight fill embankment sections.

For this option, settlement monitoring mentioned above will be essential and additional grading & maintenance will also be required.

8.10 ADDITIONAL CULVERT

The additional structural culvert foundation investigation and design report will be provided separately.

8.11 CEMENT TYPE AND CORROSION POTENTIAL

Three (3) samples of the soils from the site were submitted to AGAT Laboratories in Mississauga, Ontario for analysis of pH, water-soluble sulphate and chloride concentrations, and resistivity. The testing was completed to determine the potential for degradation of the concrete in the presence of soluble sulphates and the potential for corrosion of exposed steel used in foundations and buried infrastructure. The analysis results are summarized in the Table 5.4.

The analytical test results of the soils samples were compared to Table 7.2 of the U.S. Federal Highway Administration Publication No. FHWA-NHI-14-007 (2015) *Table 7.2 Criteria for Assessing Ground Corrosion Potential* for the attack on buried steel. The chloride concentrations measured in the soil



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samples are less than the threshold for non-aggressive soils (less than 100 ppm). However, the concentration of sulfates for two of the soil samples in boreholes RMN-UP1 and RMN-UP2 (272 ppm and 318 ppm, respectively) is indicative of an “aggressive” soil (Sulfide concentration of more than 200 ppm).

As per the MTO Structural Manual (2021) section 2.8.5, concrete is considered subject to sulphate attack when

- Water-soluble sulphate (SO_4) content of the adjacent soil is equal to or greater than 0.10%; or,
- Sulphate (SO_4) in groundwater is equal to or greater than 150 mg/L.

When concrete is identified as subject to sulphate attack, the concrete shall be resistant to sulphate attack as per the relevant standards. Based on the test results, concrete will not be subject to sulphate attack for this bridge site (water soluble sulphate in soil samples <0.10% which is equivalent to 1000 $\mu\text{g/g}$).

It should be noted that the final selection of corrosion mitigation measures should be a decision of the structural design engineer.

9.0 CONSTRUCTION CONSIDERATIONS

9.1 CONSTRUCTION STAGING AND DETOUR

It is understood that the southeast toe of the proposed embankment for the underpass may encroach close to the existing Highway 3, and in this respect, temporary embankment retention systems may be required to hold the embankment to keep the existing Highway 3 open and build the embankment in stages.

9.2 EXCAVATION AND BACKFILLING (STRUCTURE AND EMBANKMENTS)

Excavation and backfill for the new bridge structure should be carried out in accordance with OPSS 902 Construction Specification for Excavation and Backfilling – Structures. Since the new bridge structure and embankment are proposed in an open area, only minimal excavation such as surficial material stripping, shallow foundation excavation and/or pile cap excavation are anticipated. The soils encountered at the site may be classified in accordance with the OHSA as follows:

Clayey silt to silty clay fill (soft to stiff)	Type 4 Soil
Clayey Silt (CL) Till (firm)	
Clayey Silt (CL)Till and Clayey Silt (CL-ML) (stiff to hard)	Type 3 Soil



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OHSA indicates that temporary excavations made within Type 3 soils that are above the water table and/or dewatered prior to excavation should be developed with side slopes no steeper than 1H:1V.

Any vegetation, fill, organic soils, and other deleterious materials must be removed from beneath proposed pile caps, shallow foundations and embankment footprint. Where deleterious materials are encountered, the materials should be excavated, removed, and replaced. The lateral extent of such excavation should include all deleterious material within the influence zone of the foundation elements.

Grading work should be carried out in accordance with OPSS 206 Construction Specification for Grading. All embankment fill materials should be compacted in accordance with OPSS 501.

Any side slopes for open cut excavations should conform to Occupational Health and Safety Act regulations for Construction Projects (OHSa).

9.3 TEMPORARY EMBANKMENT RETENSION SYSTEM

Temporary protection systems may be required to hold back the proposed embankment fill for the underpass structure to keep the existing Highway 3 open during the construction.

Depending on the height of earth retentions system, typical sheet pile wall, soldier pile and lagging wall or near vertical concrete block wall (with additional reinforcements, as necessary) could be considered. All options are considered feasible based on the site subsurface condition.

The protection systems must be designed to resist embankment loading, taking into account the upslope above the retention system. Additional reinforcement may be required depending on the retention height and other wall details.

Depending on the retention height and offset from the existing Highway 3, consideration should be given to the performance requirements described in OPSS.PROV 539. Typically, Performance Level 2 is acceptable for highway temporary roadway protection system.

The contractor will ultimately be responsible for developing and implementing a temporary embankment retention system, including selecting appropriate geotechnical parameters and lateral earth pressure distributions (depending on the embankment fill material) for use in the design of the system.

From a geotechnical perspective, the temporary embankment retention system can either be removed or left in place. If the system is to be removed, the removal operations shall be in accordance with OPSS.PROV 539.

9.4 UNWATERING/DEWATERING (GROUNDWATER CONTROL)

Based on the groundwater level recorded in the monitoring well in borehole RMN-UP2, the groundwater level is expected at approximately elevation 232 m. The elevation of the anticipated bottom of excavation for pile caps or shallow foundations for the pier is approximately 236 m. In this respect, the excavations for the pile caps are not anticipated to penetrate the groundwater table.



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Some seepage should be anticipated from the perched water in the fill materials near ground surface and/or from the more permeable seams/zones within the till deposit, although the volume is expected to be limited. Temporary unwatering, using conventional sump and pump techniques, is anticipated to be required for excavations and should be satisfactory to handle seepage and infiltration into excavations. If deeper excavations into the groundwater table is anticipated, dewatering should also be considered.

All groundwater control systems required for the construction of the replacement bridge should be designed and implemented in accordance with OPSS.PROV 517.

Depending on the water taking/dewatering volumes and source(s) of water, the dewatering activities may require a Permit to Take Water (PTTW) from the Ministry of Environment, Conservation and Parks (MECP) or registration of the water taking activity in the Environmental Activity and Sector Registry (EASR). Given the low permeability of founding subgrade soils and anticipated excavation will not intercept the groundwater, a dewatering permit or registration may not be required for the proposed bridge foundation construction. Ultimately, the design of dewatering/unwatering systems is the responsibility of the contractor. The permit/registration requirements are outlined in Table 1.0 of CDED B517.

9.5 SHALLOW FOUNDATIONS

All founding subgrades should be inspected and approved by a qualified geotechnical engineer prior to footing construction. If founding subgrade is sensitive to disturbance and softening or loosening due to water accumulation and construction equipment is expected, consideration should be given to the use of mud mat.

9.6 DEEP FOUNDATIONS

As pointed out in the preceding sections, cobbles and boulders should be expected in the soils at the site which may impact pile driving and/or drilled caisson construction.

Deep foundation should be installed and monitored in accordance with OPSS.PROV 903 and steel H pile tip should be reinforced using Titus point, AFP hard bite or equivalent. As mentioned earlier in this report, pile driving should be controlled by SS103-11 Hiley Formula and pile capacity should be verified by Pile Driving Analyzer (PDA) as per OPSS 903.

All caissons should be inspected and tested as per OPSS 903.



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10.0 SPECIFICATIONS

The following specifications are referenced in this report:

Table 10.1: Specifications Referenced in the Report

Document	Title
OPSD 202.010	Slope Flattening Using Surplus Excavated Material on Earth or Rock Embankment
OPSD 3000.100	Foundation, Piles, Steel H-Pile Driving Shoe
OPSD 3090.101	Foundation Frost Depths for Southern Ontario
OPSD 3101.150	Walls, abutment, backfill – Minimum Granular Requirements
OPSS.PROV 206	Construction Specification for Grading
OPSS.PROV 212	Construction Specification for Earth Borrow
OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 517	Construction Specification for Dewatering and Temporary Flow Passage Systems
OPSS. PROV 539	Construction Specification for Temporary Protection System
OPSS.MUNI 802	Construction Specification for Topsoil
OPSS.MUNI 804	Construction Specification for Seed and Cover
OPSS.PROV 804	Construction Specification for Temporary Erosion Control
OPSS.MUNI 805	Construction Specification for Temporary Erosion and Sediment Control Measures
OPSS.PROV 805	Construction Specification for Temporary Sediment Control
OPSS.PROV 902	Construction Specification for Excavation and Backfilling – Structures
OPSS.PROV 903	Construction Specification for Deep Foundations
OPSS.PROV 1010	Material Specification for Aggregates



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11.0 MISCELLANEOUS

The field work was carried out under the supervision of Mr. Muhammed Cuned and Mr. Harpreet Singh, under the direction of Mr. Gwangha Roh, P. Eng., Ph.D.

Both public and private utility locates were arranged by Stantec staff prior to initiation of drilling.

The drilling equipment was supplied and operated by London Soil Ltd. based in London, Ontario and DBW Drilling Ltd. based in North York, Ontario.

CPT, sCPT and MASW were carried out by ConeTec based in Richmond Hill, Ontario.

The borehole locations and elevations were surveyed by Stantec's Geomatics division based in London.

Geotechnical laboratory testing was carried out at Stantec's laboratories in Markham and Ottawa, Ontario.

This report was prepared by Roshan Rashed, M.Sc., P.Eng., and reviewed by Gwangha Roh, P. Eng., Ph.D., and Raymond Haché, M.Sc., P.Eng., Designated Principal MTO Foundation Contact.



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12.0 CLOSURE

The recommendations made in this report are in accordance with our present understanding of the project. We request that we be permitted to review our recommendations when the drawings and specifications are complete.

A soil investigation is a limited sampling of a site. The conclusions given herein are based on information gathered at the specific borehole locations. Should any conditions at the site be encountered which differ from those at the borehole locations, we request that we be notified immediately in order to assess the additional information and its effects on the above recommendations.

We trust the information presented herein meets your present requirements. Should you have any questions or require additional information, please do not hesitate to contact us.

Respectfully submitted,

STANTEC CONSULTING LTD.



Roshan Rashed., M.Sc., P.Eng.
Geotechnical Engineer



Gwangha Roh, Ph.D., P.Eng.
Senior Geotechnical Engineer



Raymond Haché, M.Sc., P.Eng.
Designated Principal MTO Foundation Contact



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13.0 REFERENCES

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- Ontario Geological Survey. 2010. Surficial Geology of Southern Ontario GIS data set.
- Ontario Ministry of Transportation (MTO). 2010. MTO Embankment Settlement Criteria for Design.
- Ontario Ministry of Transportation (MTO). 2021. Structural Manual. Bridge Office, St. Catharines, Ontario.
- Ontario Ministry of Transportation (MTO). 1973. Foundation Investigation Report for Proposed Crossing at St. Thomas Expressway and County Road #52, Twp. Of Southwold, Co. of Elgin, District #2 (London), W.O. 73-11021, W.P 89-69-07. Geocres No. 40114-35.
- Ontario Ministry of Transportation (MTO). 2020 Guideline for Foundation Engineering Services Version 2.0.



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UNDERPASS – HIGHWAY 4 WIDENING FROM CLINTON LINE TO NEW TALBOTVILLE BYPASS
AND NEW TALBOTVILLE BYPASS FROM HIGHWAY 4 TO HIGHWAY 3 AT RON MCNEIL LINE**

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APPENDIX A

A.1 DRAWING NO. 1 – BOREHOLE LOCATION PLAN AND SOIL STRATA PLOTS

A.2 GENERAL ARRANGEMENT DRAWING



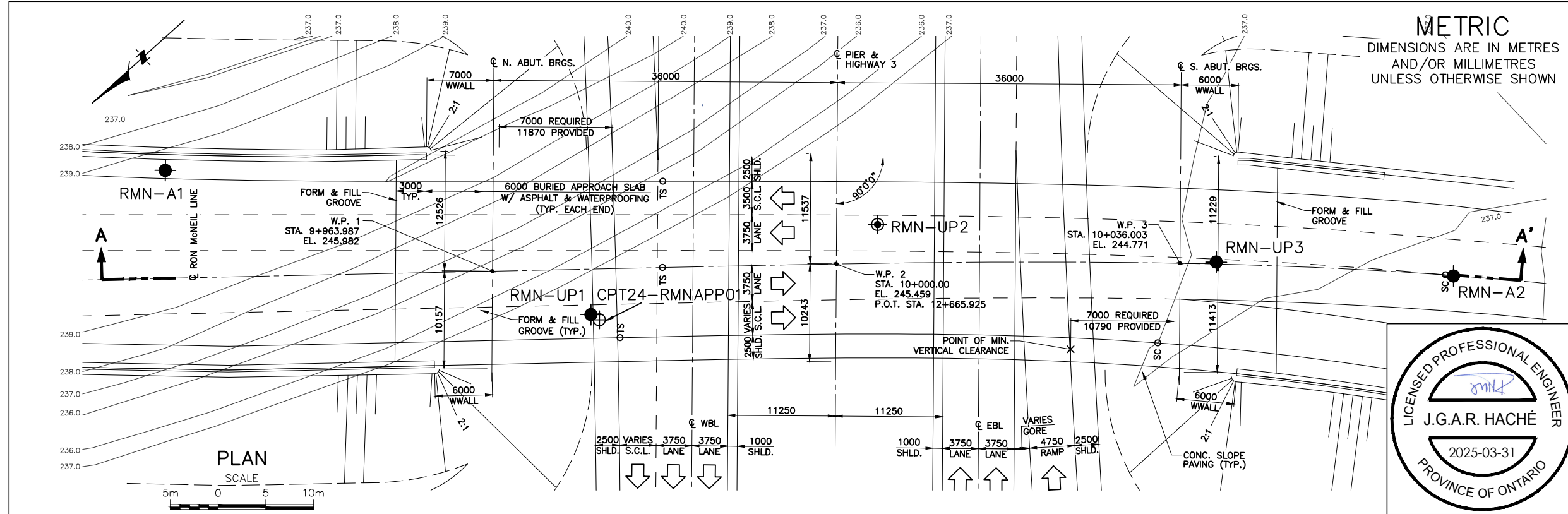


PLATE No
CONT
WP 3042-22-00

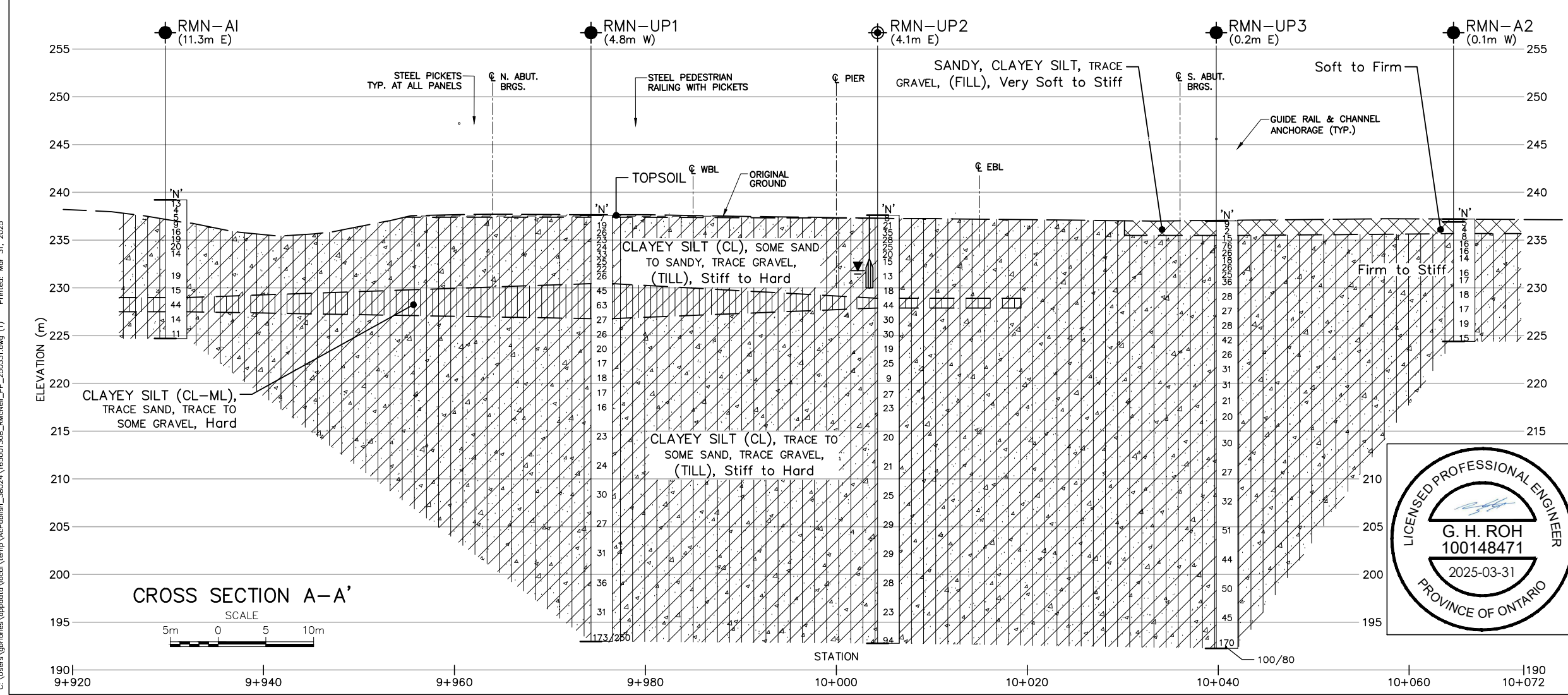
RON MCNEIL LINE
INTERCHANGE BRIDGE
BOREHOLE LOCATIONS & SOIL STRATA

SHEET
—

Stantec

KEY PLAN
800m 0 800 1600m

LICENSED PROFESSIONAL ENGINEER
J.G.A.R. HACHÉ
2025-03-31
PROVINCE OF ONTARIO



LEGEND

- Borehole (Stantec, 2024)
- Borehole with Monitoring Well (Stantec, 2024)
- Cone Penetration Test (Stantec, 2024)
- (x.x m) Offset from Cross Section Line
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- WL Measured on September 2024
- Piezometer

No	ELEV	MTM ZONE 11 NORTH	COORDINATES EAST
RMN-A1	239.2	4 741 937.4	408 801.5
RMN-A2	237.2	4 741 843.9	408 703.6
RMN-UP1	237.6	4 741 914.1	408 760.6
RMN-UP2	237.6	4 741 885.4	408 747.7
RMN-UP3	237.0	4 741 861.5	408 721.2
CPT24-RMNAPP01		4 741 913.8	408 759.6

NOTES

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

This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 102-2 of Form 100.

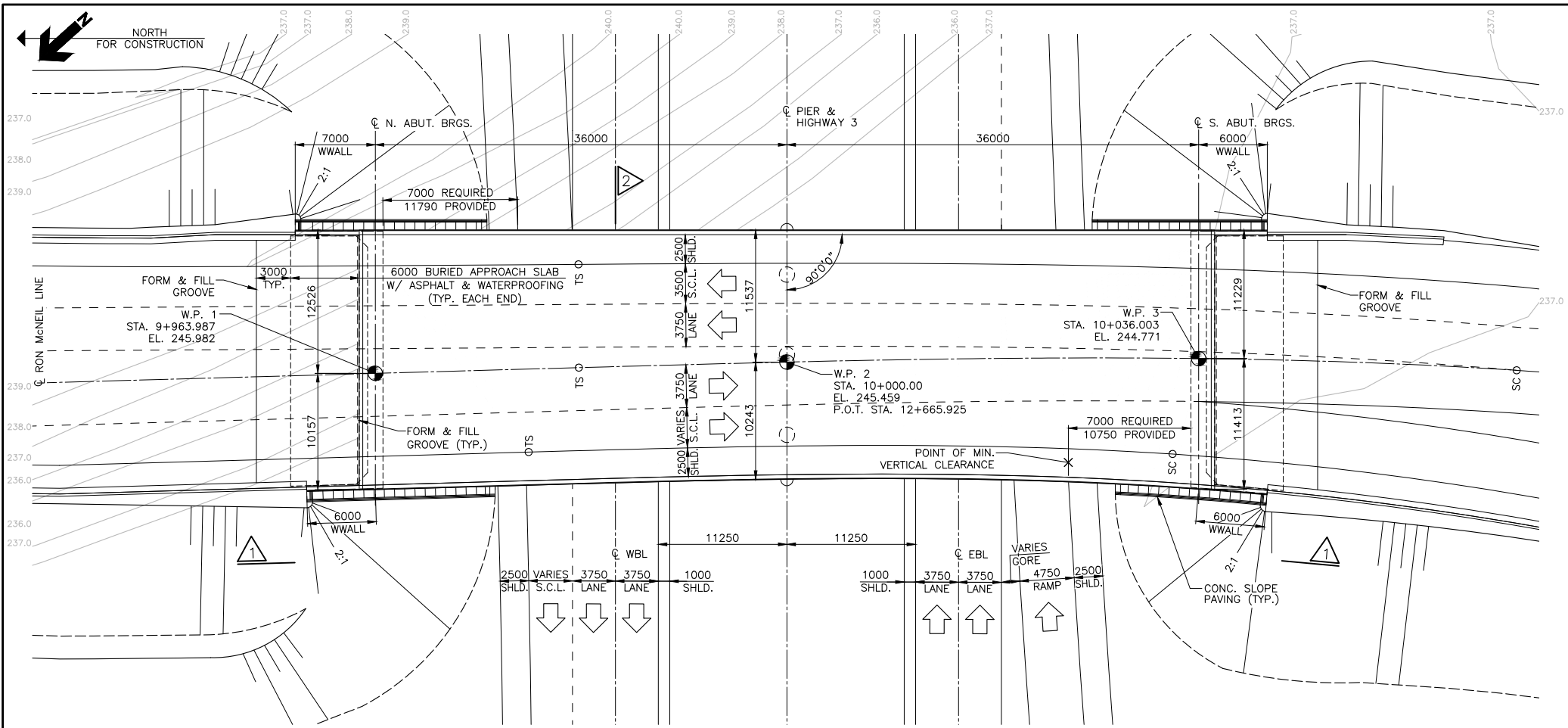
REVISIONS

DATE	BY	DESCRIPTION

GEORES No 40114-223

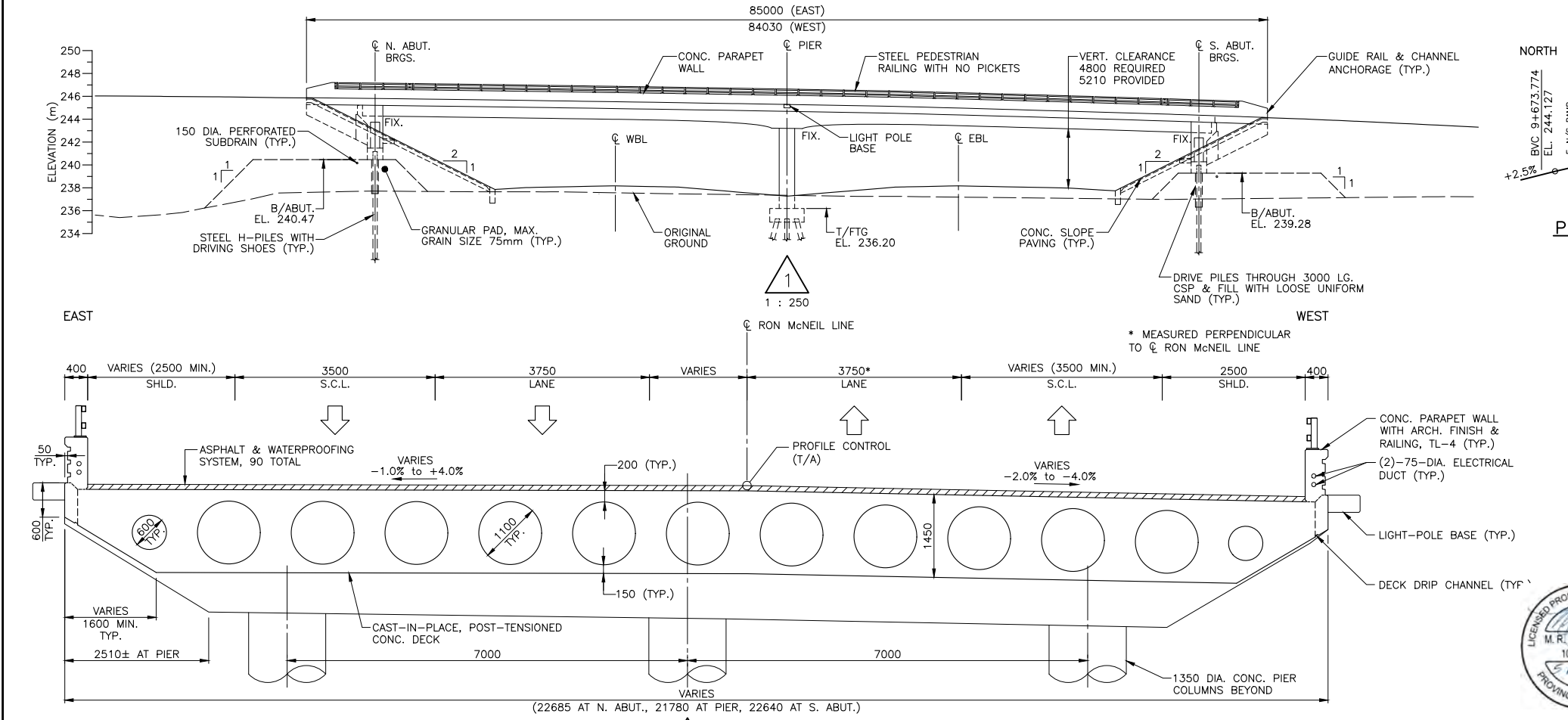
HWY No 3	SUBM'D RR	CHECKED	DATE 2025-03-31	SITE05X-0376/B0	DIST

LICENSED PROFESSIONAL ENGINEER
G. H. ROH
100148471
2025-03-31
PROVINCE OF ONTARIO



PLAN

1 : 250



PROFILE

1 : 250

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

HWY 3
CONT 2025-3007
WP 3077-24-01

RON McNEIL LINE UNDERPASS
GENERAL ARRANGEMENT

SHEET
415

LIST OF DRAWINGS

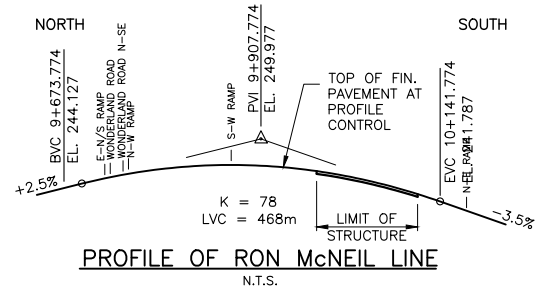
1. GENERAL ARRANGEMENT
2. BOREHOLE LOCATION & SOIL STRATA
3. FOUNDATION LAYOUT
4. PILE CAP, PIER FOOTING & COLUMN REINFORCING
5. SOUTH ABUTMENT
6. NORTH ABUTMENT
7. WINGWALLS
8. DECK DETAILS I
9. LONGITUDINAL TENDONS I
10. LONGITUDINAL TENDONS II
11. TRANSVERSE TENDONS I
12. DECK REINFORCING I
13. DECK REINFORCING II
14. PARAPET WALL
15. RAILING FOR PARAPET WALL
16. BURIED APPROACH SLAB
17. CONCRETE SLOPE PAVING
18. AS-CONSTRUCTED ELEVATIONS & DIMENSIONS
19. PILE DRIVING CONTROL
20. MISCELLANEOUS DETAILS
21. ELECTRICAL EMBEDDED WORK

LEGEND

- W.P. - DENOTES WORKING POINT
T/A - DENOTES TOP OF ASPHALT
P.O.T. - DENOTES POINT ON TANGENT (HWY 3)
TS - DENOTES TANGENT TO SPIRAL
SC - DENOTES SPIRAL TO CURVE
S.C.L. - DENOTES SPEED CHANGE LANE

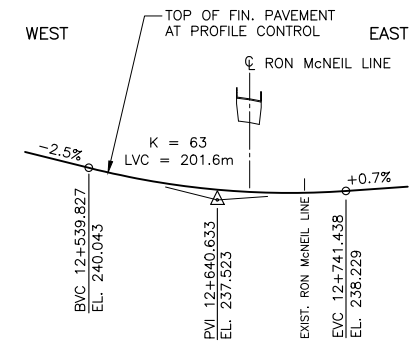
GENERAL NOTES

1. SPECIFIED 28-DAY CONCRETE COMPRESSIVE STRENGTH:
DECK 35MPa
REMAINDER 30MPa
UNLESS OTHERWISE NOTED.
2. CLEAR COVER TO REINFORCING STEEL:
FOOTINGS 100±25
PIER COLUMNS 70±10
DECK TOP 70±20
BOTTOM, SIDES, WEBS 50±10
REMAINDER 70±20
UNLESS OTHERWISE NOTED.
3. REINFORCING STEEL:
REINFORCING STEEL SHALL BE GRADE 500W UNLESS OTHERWISE SPECIFIED.
STAINLESS REINFORCING STEEL SHALL BE TYPE 316LN, OR DUPLEX 2205 AND HAVE A MINIMUM YIELD STRENGTH OF 500MPa.
BAR MARKS WITH PREFIX 'S' DENOTE STAINLESS STEEL BARS.
TENSION LAP LENGTHS NOT INDICATED ON THE CONTRACT DRAWINGS SHALL BE CLASS B.
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.
4. CONSTRUCTION
THE CONTRACTOR SHALL ESTABLISH THE BEARING SEAT ELEVATIONS BY DEDUCTING THE ACTUAL BEARING THICKNESSES FROM THE TOP OF BEARING ELEVATIONS. IF THE ACTUAL BEARING THICKNESSES ARE DIFFERENT FROM THOSE GIVEN WITH THE BEARING DESIGN DATA, THE CONTRACTOR SHALL ADJUST THE REINFORCING STEEL TO SUIT.
CONSTRUCT ABUTMENTS IN STAGES AS SHOWN ELSEWHERE.
THE CONTRACTOR SHALL DESIGN AND SUPPLY TWO TEMPORARY LATERAL BRACING SYSTEMS FOR THE ABUTMENT WALLS DURING CONSTRUCTION:
THE FIRST SYSTEM IS FOR THE LATERAL STABILITY OF THE FREE-STANDING ABUTMENTS CORE WALLS ON PILES SO THAT THEY REMAIN STABLE DURING THE CONSTRUCTION AND LOADING OF THE DECK SLAB UNTIL DECK AND ABUTMENT WALLS ARE MADE INTEGRAL.
THE SECOND SYSTEM IS FOR THE LATERAL STABILITY OF THE CORE WALLS TO PREVENT THEM FROM MOVING INWARD (TOWARD THE PIER) DURING THE DECK POST-TENSIONING OPERATIONS. DESIGN FORCES PER ABUTMENT AND OTHER REQUIREMENTS FOR THIS SYSTEM ARE STATED ON DWG. (5) AND ELSEWHERE IN THE CONTRACT DOCUMENTS
THE TWO LATERAL BRACING SYSTEMS CAN BE INTEGRATED INTO ONE SYSTEM IF PREFERRED BY THE CONTRACTOR. THE LATERAL BRACING SYSTEMS SHALL NOT BE REMOVED UNTIL THE DECK IS MADE INTEGRAL WITH THE ABUTMENT WALLS AND THE CONCRETE OF THE INTEGRAL CONNECTION HAS REACHED 70% OF IT'S 28 DAY STRENGTH.
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.



PROFILE OF RON McNEIL LINE

N.T.S.



PROFILE OF HIGHWAY 3

N.T.S.



DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS		DATE		BY		DESCRIPTION	
DESIGN	M.C. CHK M.D.	CODE	CSA S6:19	LOAD	CL-625-ONT	DATE	MAR 2025
DRAWN	B.L.H. CHK M.C.	SITE	05X-0376/B0	STRUCT	SCHEME	DWG.	1

165001308-5-376-01.DWG Mar 5 2025

BENCHMARK 237.859m:
BASED ON CGVD28, TOP OF RIB, STATION: 13+153.27 SOUTHWOLD OFFSET: -20.59m

**FOUNDATION INVESTIGATION AND DESIGN REPORT – RON MCNEIL LINE INTERCHANGE
UNDERPASS – HIGHWAY 4 WIDENING FROM CLINTON LINE TO NEW TALBOTVILLE BYPASS
AND NEW TALBOTVILLE BYPASS FROM HIGHWAY 4 TO HIGHWAY 3 AT RON MCNEIL LINE**

April 2025

APPENDIX B

B.1 AVAILABLE GEOCRETS INFORMATION



MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO

MEMORANDUM

401-183

TO: Mr. A. P. Watt, (2)
Regional Structural Planning Eng.,
Southwestern Region,
London, Ontario.

FROM: Foundations Office,
Design Services Branch,
West Bldg., Downsview.

ATTENTION:

DATE: August 13, 1973.

OUR FILE REF.

IN REPLY TO

AUG 28 1973

SUBJECT:

FOUNDATION INVESTIGATION REPORT
For
Proposed Crossing at St. Thomas
Expressway and County Road #52
Twp. of Southwold, Co. of Elgin
District #2 (London)
W.O. 73-11021 -- W.P. 89-69-07

40114-35
GEOCRE No.

Attached we are forwarding to you our detailed foundation investigation report on the subsoil conditions existing at the above-mentioned site.

We believe that the factual data and recommendations contained therein will prove adequate for your design requirements. Should additional information be required, please do not hesitate to contact our Office.

AGS/ao
Attch.

c.c. E. J. Orr
B. R. Davis
A. Rutka
A. Wittenberg
L. E. Walker
B. J. Giroux
J. R. Roy
G. A. Wrong
B. A. Singh

A. G. Stermac
A. G. Stermac,
PRINCIPAL FOUNDATIONS ENGINEER.

Foundations Files ✓
Documents

FOUNDATIONS OFFICE

JOB 73-11021

LOCATION Co-ords. 15,556,624 N; 1,340,915 E.

ORIGINATED BY LJH

W.P. 89-69-07

BORING DATE May 18, 1973

COMPILED BY L J H

DATUM Geodetic

BOREHOLE TYPE Hollow Stem Auger & Cone

CHECKED BY TK

20
15 ϕ 5 % STRAIN AT FAILURE
10

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 2

CHECKED BY MC

SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE			LIQUID LIMIT ——— w _L			BULK DENSITY γ P.C.F.	REMARKS	
ELEV. DEPTH	DESCRIPTION	STRAT. PLT	NUMBER	TYPE	BLOWS/FOOT	BLOWS / FOOT			PLASTIC LIMIT ——— w _p					WATER CONTENT — w
						20	40	60	80	100	w _p			
						SHEAR STRENGTH P.S.F.								
						○ UNCONFINED + FIELD VANE								
						● QUICK TRIAXIAL × LAB VANE								
									w _p ——— w ——— w _L					
									10 20 30					
777.8	Ground Level													
	Brown Grey Clayey silt to silty clay, some sand, traces of gravel. Occasional thin seams or pockets of silt. Very Stiff to Hard		1	SS	31									
			2	SS	26									
			3	SS	27									
			4	SS	25									
			5	SS	26									
			6	SS	25									
			7	SS	36									
			8	SS	37									
			9	SS	36									
			10	SS	45									
726.3														
51.5	End of Borehole													

20
15 5 % STRAIN AT FAILURE
10

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 3

JOB 73-11021

LOCATION Co-ords. 15,556,767 N; 1,341,045 E

ORIGINATED BY L.J.H.

W.P. 89-69-07

BORING DATE May 22, 1973

COMPILED BY L.J.H.

DATUM GEODETIC

BOREHOLE TYPE HOLLOW STEM AUGER AND CONE

CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT				LIQUID LIMIT w_L PLASTIC LIMIT w_p WATER CONTENT w			BULK DENSITY γ	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLT	NUMBER	TYPE	BLOWS/FOOT		20	40	60	80	100	w_p	w	w_L	
779.3	GROUND LEVEL														
0.0	Brown Grey		1	SS	45										2 31 53 32
			2	SS	48	770									
	Clayey silt to silty clay, some sand, traces of gravel		3	SS	31										1 18 47 34
			4	SS	25	760									
			5	SS	15										
			6	SS	26	750									
	occasional thin seams or pockets of silt.		7	SS	43										
			8	SS	26	740									
	Very Stiff to Hard		9	SS	26										3 9 56 32
			10	SS	19	730									
			11	SS	18										
			12	SS	27	720									
			13	SS	26	710									
697.8			14	SS	30	700									HOLE DRY.
81.5	END OF BOREHOLE														3 32 40 25
						690									

FOUNDATIONS OFFICE

JOB 73-11021

LOCATION _____ Co-ords. 15,556,642 N; 1,340,891 E

ORIGINATED BY L.J.H.

W.P. 89-69-07

BORING DATE May 22, 1973

COMPILED BY L.J.H.

DATUM GEODETIC

BOREHOLE TYPE CONE TEST

CHECKED BY

15 $\frac{20}{10}$ 5 % STRAIN AT FAILURE

FOUNDATIONS OFFICE

JOB 73-11021

LOCATION Co-ords.. 15556694.38N; 1340978.19E

ORIGINATED BY L.J.H.

W.P. 89-69-07

BORING DATE May 22, 1973

COMPILED BY L.J.H.

DATUM GEODETIC

BOREHOLE TYPE CONE TEST

CHECKED BY 17

[illegible]

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 6

JOB 73-11021

LOCATION Co-ords. 15556788.65N; 1341022.06E

ORIGINATED BY L.J.H.

W.P. 89-69-07

BORING DATE May 18, 1973

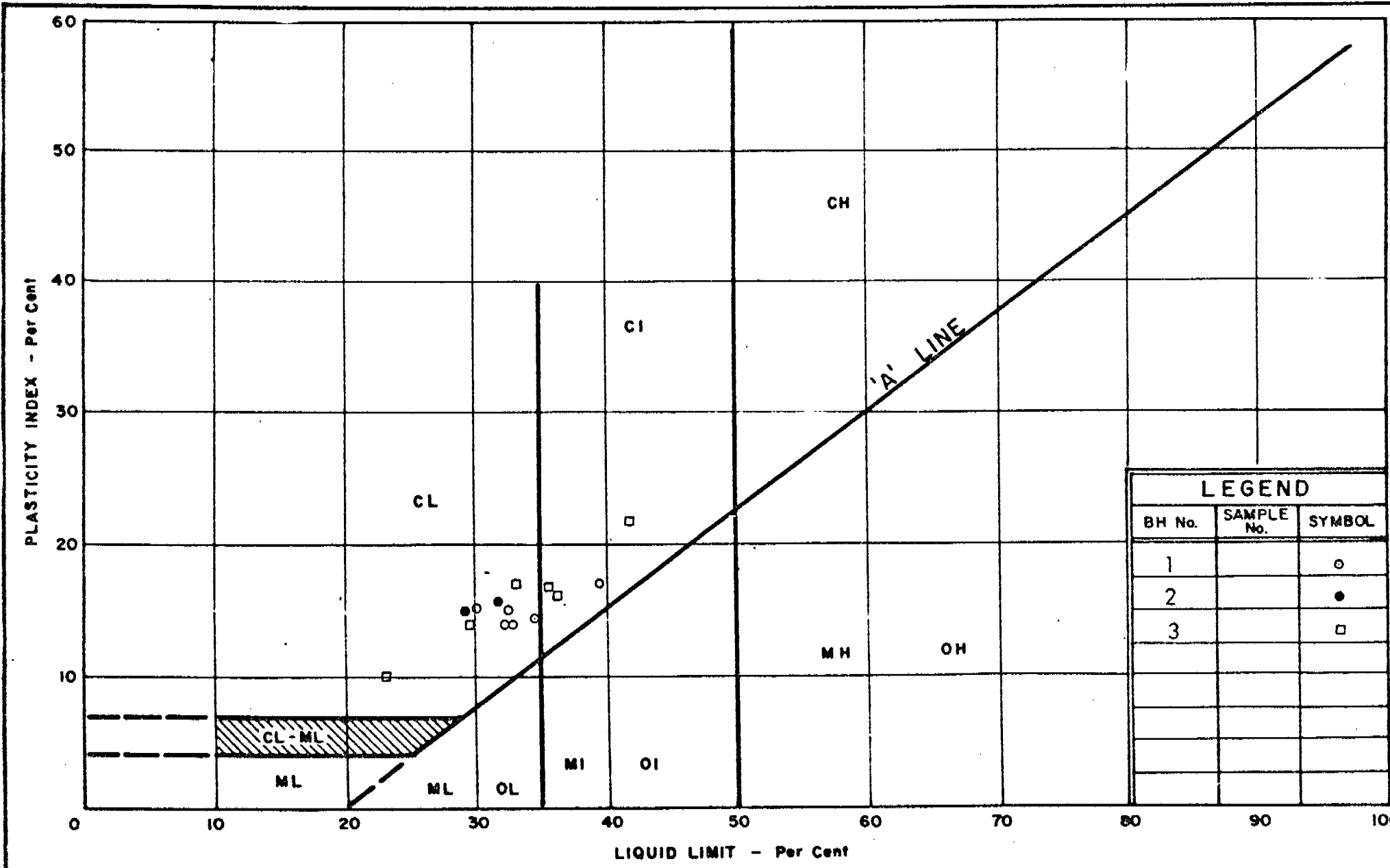
COMPILED BY L.J.H.

DATUM GEODETIC

BOREHOLE TYPE CONE TEST

CHECKED BY *HS*

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT — w_L			BULK DENSITY	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT		BLOWS / FOOT	20	40	60	80	100	PLASTIC LIMIT — w_p	WATER CONTENT — w		
779.4	GROUND LEVEL															
768.5						770										
10.9	END OF CONE TEST					760										

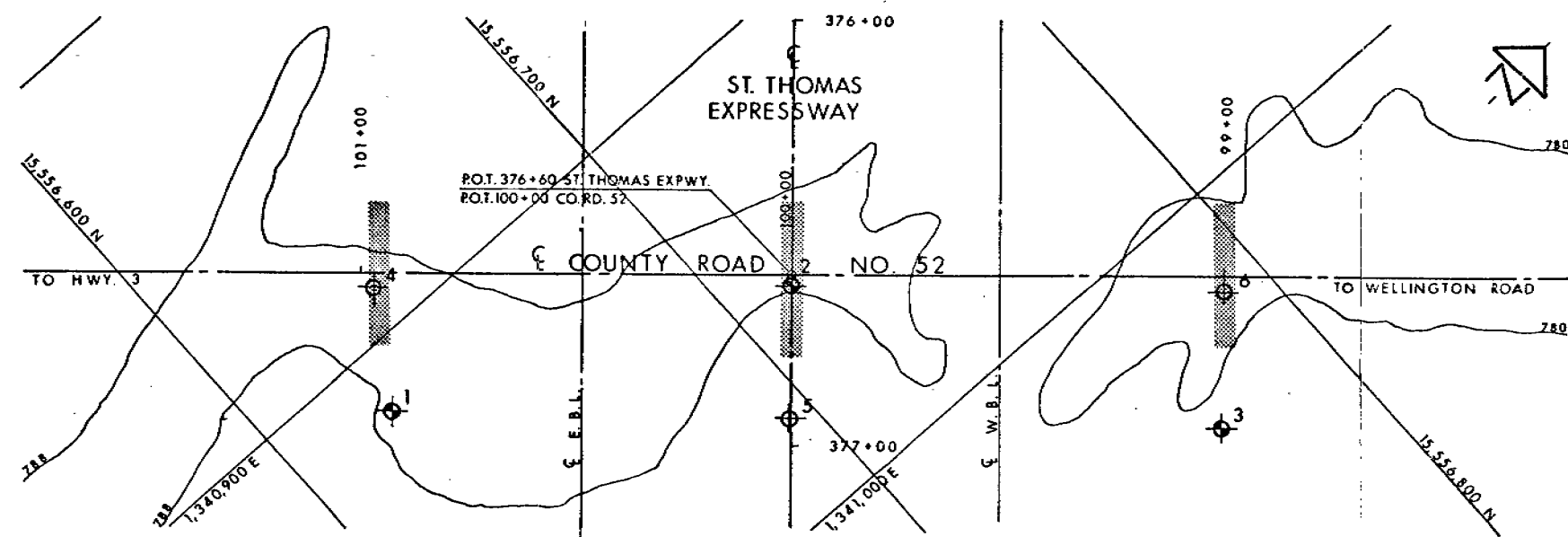


DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

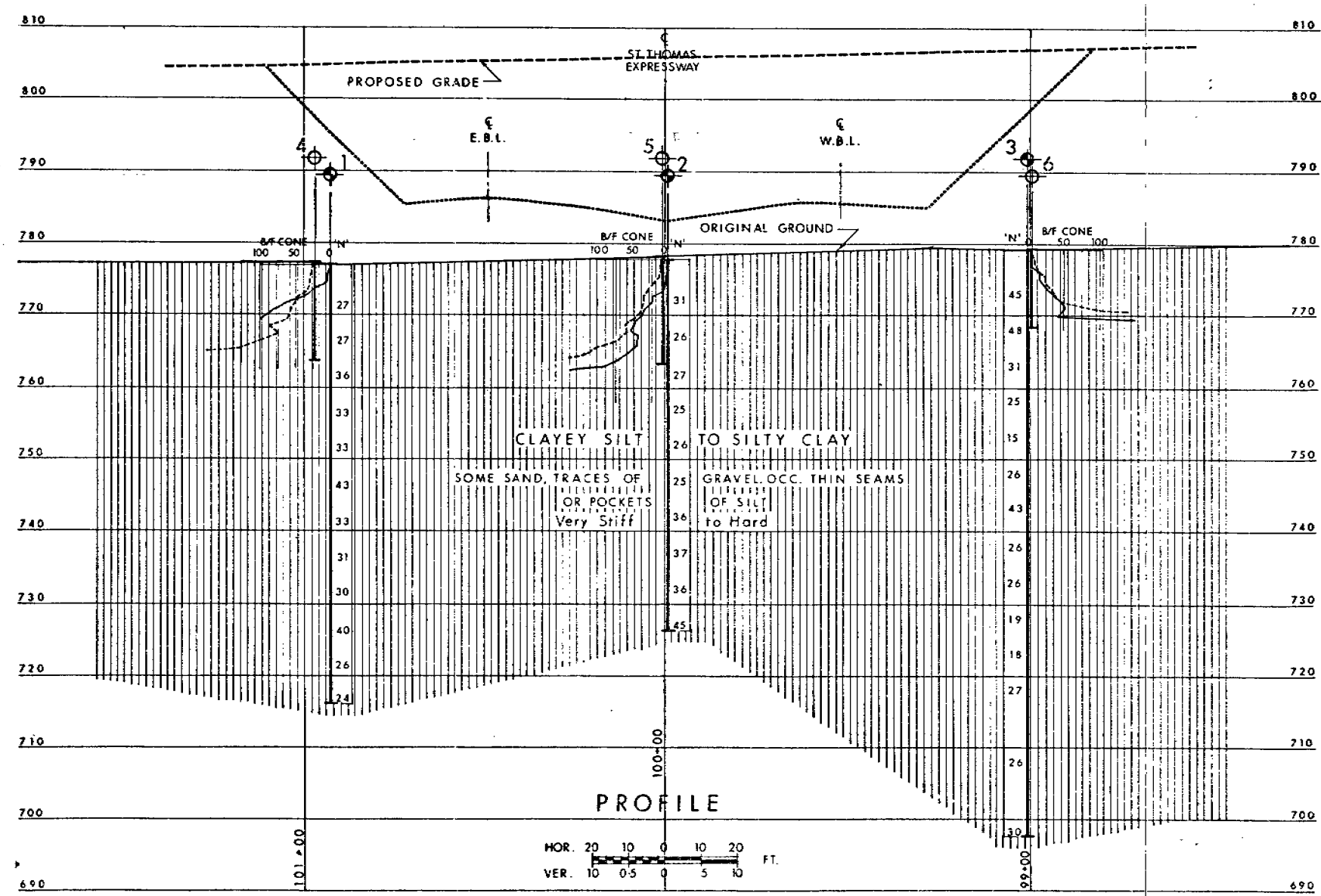
PLASTICITY CHART

CLAYEY SILT TO SILTY CLAY

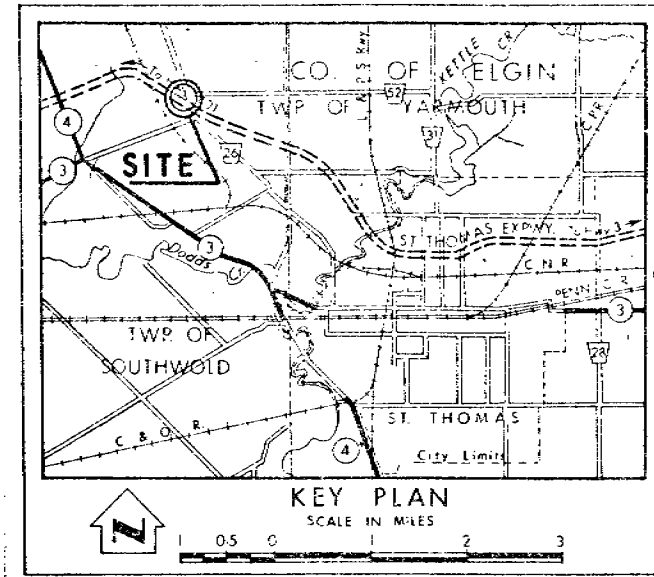
WP. No. 89-69-07
JOB No. 73-11021
FIG. 1



PLAN



PROFILE



LEGEND

- Bore Hole
- ⊕ Cone Penetration Test
- ⊕ Bore Hole & Cone Test
- ≡ Water Levels established at time of field investigation.
- Holes Dry May 1973

NO.	ELEVATION	CO-ORDINATES	
		NORTH	EAST
1	777.7	15,556,624	1,340,915
2	777.8	15,556,715	1,340,955
3	779.3	15,556,767	1,341,045
4	777.3	15,556,642	1,340,891
5	778.2	15,556,694	1,340,978
6	779.4	15,556,789	1,341,022

NOTE

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

REVISIONS	DATE	BY	DESCRIPTION



REF. No FENCO 3802-87-03

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS—ONTARIO
DESIGN SERVICES BRANCH—FOUNDATIONS OFFICE

COUNTY ROAD NO. 52

HIGHWAY NO PROP. ST. THOMAS EXPWY. DIST. NO. 2
CO. ELGIN
TWP. SOUTHWOLD LOT CON

BORE HOLE LOCATIONS & SOIL STRATA

SUBWD. A.P. CHECKED	WP. NO. 89-69-4	DRAWING NO.
DRAWN O.L. J. CHECKED	WO NO 73-11021	73-11021A
DATE 20 AUG 1973	SITE NO.	BRIDGE DRAWING NO.
APPROVED <i>[Signature]</i>	CONT. NO.	

PRINCIPAL FOUNDATION ENGINEER

**FOUNDATION INVESTIGATION AND DESIGN REPORT – RON MCNEIL LINE INTERCHANGE
UNDERPASS – HIGHWAY 4 WIDENING FROM CLINTON LINE TO NEW TALBOTVILLE BYPASS
AND NEW TALBOTVILLE BYPASS FROM HIGHWAY 4 TO HIGHWAY 3 AT RON MCNEIL LINE**

April 2025

APPENDIX C

C.1 SYMBOLS AND TERMS USED ON BOREHOLE RECORDS

C.2 BOREHOLE RECORDS

C.3 SITE INVESTIGATION RESULTS, HIGHWAY 3 ST. THOMAS CPT



SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

SOIL DESCRIPTION

Terminology describing common soil genesis:

<i>Rootmat</i>	- vegetation, roots and moss with organic matter and topsoil typically forming a mattress at the ground surface
<i>Topsoil</i>	- mixture of soil and humus capable of supporting vegetative growth
<i>Peat</i>	- mixture of visible and invisible fragments of decayed organic matter
<i>Till</i>	- unstratified glacial deposit which may range from clay to boulders
<i>Fill</i>	- material below the surface identified as placed by humans (excluding buried services)

Terminology describing soil structure:

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

Terminology describing soil types:

The classification of soil types are made on the basis of grain size and plasticity in accordance with the Unified Soil Classification System (USCS) (ASTM D 2487 or D 2488) which excludes particles larger than 75 mm. For particles larger than 75 mm, and for defining percent clay fraction in hydrometer results, definitions proposed by Canadian Foundation Engineering Manual, 4th Edition are used. The USCS provides a group symbol (e.g. SM) and group name (e.g. silty sand) for identification.

Terminology describing cobbles, boulders, and non-matrix materials (organic matter or debris):

Terminology describing materials outside the USCS, (e.g. particles larger than 75 mm, visible organic matter, and construction debris) is based upon the proportion of these materials present:

<i>Trace, or occasional</i>	Less than 10%
<i>Some</i>	10-20%
<i>Frequent</i>	> 20%

Terminology describing compactness of cohesionless soils:

The standard terminology to describe cohesionless soils includes compactness (formerly "relative density"), as determined by the Standard Penetration Test (SPT) N-Value - also known as N-Index. The SPT N-Value is described further on page 3. A relationship between compactness condition and N-Value is shown in the following table.

Compactness Condition	SPT N-Value
<i>Very Loose</i>	<4
<i>Loose</i>	4-10
<i>Compact</i>	10-30
<i>Dense</i>	30-50
<i>Very Dense</i>	>50

Terminology describing consistency of cohesive soils:

The standard terminology to describe cohesive soils includes the consistency, which is based on undrained shear strength as measured by *in situ* vane tests, penetrometer tests, or unconfined compression tests. Consistency may be crudely estimated from SPT N-Value based on the correlation shown in the following table (Terzaghi and Peck, 1967). The correlation to SPT N-Value is used with caution as it is only very approximate.

Consistency	Undrained Shear Strength		Approximate SPT N-Value
	kips/sq.ft.	kPa	
<i>Very Soft</i>	<0.25	<12.5	<2
<i>Soft</i>	0.25 - 0.5	12.5 - 25	2-4
<i>Firm</i>	0.5 - 1.0	25 - 50	4-8
<i>Stiff</i>	1.0 - 2.0	50 - 100	8-15
<i>Very Stiff</i>	2.0 - 4.0	100 - 200	15-30
<i>Hard</i>	>4.0	>200	>30

ROCK DESCRIPTION

Except where specified below, terminology for describing rock is as defined by the International Society for Rock Mechanics (ISRM) 2007 publication "The Complete ISRM Suggested Methods for Rock Characterization, Testing and Monitoring: 1974-2006"

Terminology describing rock quality:

RQD	Rock Mass Quality
0-25	Very Poor Quality
25-50	Poor Quality
50-75	Fair Quality
75-90	Good Quality
90-100	Excellent Quality

Alternate (Colloquial) Rock Mass Quality	
Very Severely Fractured	Crushed
Severely Fractured	Shattered or Very Blocky
Fractured	Blocky
Moderately Jointed	Sound
Intact	Very Sound

RQD (Rock Quality Designation) denotes the percentage of intact and sound rock retrieved from a borehole of any orientation. All pieces of intact and sound rock core equal to or greater than 100 mm (4 in.) long are summed and divided by the total length of the core run. RQD is determined in accordance with ASTM D6032.

SCR (Solid Core Recovery) denotes the percentage of solid core (cylindrical) retrieved from a borehole of any orientation. All pieces of solid (cylindrical) core are summed and divided by the total length of the core run (It excludes all portions of core pieces that are not fully cylindrical as well as crushed or rubble zones).

Fracture Index (FI) is defined as the number of naturally occurring fractures within a given length of core. The Fracture Index is reported as a simple count of natural occurring fractures.

Terminology describing rock with respect to discontinuity and bedding spacing:

Spacing (mm)	Discontinuities	Bedding
>6000	Extremely Wide	-
2000-6000	Very Wide	Very Thick
600-2000	Wide	Thick
200-600	Moderate	Medium
60-200	Close	Thin
20-60	Very Close	Very Thin
<20	Extremely Close	Laminated
<6	-	Thinly Laminated

Terminology describing rock strength:

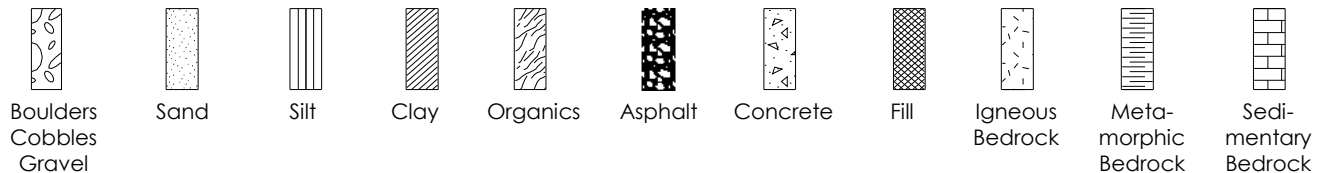
Strength Classification	Grade	Unconfined Compressive Strength (MPa)
Extremely Weak	R0	<1
Very Weak	R1	1 – 5
Weak	R2	5 – 25
Medium Strong	R3	25 – 50
Strong	R4	50 – 100
Very Strong	R5	100 – 250
Extremely Strong	R6	>250

Terminology describing rock weathering:

Term	Symbol	Description
Fresh	W1	No visible signs of rock weathering. Slight discoloration along major discontinuities
Slightly	W2	Discoloration indicates weathering of rock on discontinuity surfaces. All the rock material may be discolored.
Moderately	W3	Less than half the rock is decomposed and/or disintegrated into soil.
Highly	W4	More than half the rock is decomposed and/or disintegrated into soil.
Completely	W5	All the rock material is decomposed and/or disintegrated into soil. The original mass structure is still largely intact.
Residual Soil	W6	All the rock converted to soil. Structure and fabric destroyed.

STRATA PLOT

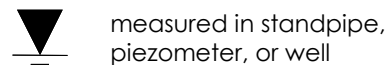
Strata plots symbolize the soil or bedrock description. They are combinations of the following basic symbols. The dimensions within the strata symbols are not indicative of the particle size, layer thickness, etc.



SAMPLE TYPE

SS	Split spoon sample (obtained by performing the Standard Penetration Test)
ST	Shelby tube or thin wall tube
DP	Direct-Push sample (small diameter tube sampler hydraulically advanced)
PS	Piston sample
BS	Bulk sample
HQ, NQ, BQ, etc.	Rock core samples obtained with the use of standard size diamond coring bits.

WATER LEVEL MEASUREMENT



RECOVERY

For soil samples, the recovery is recorded as the length of the soil sample recovered. For rock core, recovery is defined as the total cumulative length of all core recovered in the core barrel divided by the length drilled and is recorded as a percentage on a per run basis.

N-VALUE

Numbers in this column are the field results of the Standard Penetration Test: the number of blows of a 140 pound (63.5 kg) hammer falling 30 inches (760 mm), required to drive a 2 inch (50.8 mm) O.D. split spoon sampler one foot (300 mm) into the soil. In accordance with ASTM D1586, the N-Value equals the sum of the number of blows (N) required to drive the sampler over the interval of 6 to 18 in. (150 to 450 mm). However, when a 24 in. (610 mm) sampler is used, the number of blows (N) required to drive the sampler over the interval of 12 to 24 in. (300 to 610 mm) may be reported if this value is lower. For split spoon samples where insufficient penetration was achieved and N-Values cannot be presented, the number of blows are reported over sampler penetration in millimetres (e.g. 50/75). Some design methods make use of N-values corrected for various factors such as overburden pressure, energy ratio, borehole diameter, etc. No corrections have been applied to the N-values presented on the log.

DYNAMIC CONE PENETRATION TEST (DCPT)

Dynamic cone penetration tests are performed using a standard 60 degree apex cone connected to '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 one foot (300 mm) into the soil. The DCPT is used as a probe to assess soil variability.

OTHER TESTS

S	Sieve analysis
H	Hydrometer analysis
k	Laboratory permeability
y	Unit weight
G _s	Specific gravity of soil particles
CD	Consolidated drained triaxial
CU	Consolidated undrained triaxial with pore pressure measurements
UU	Unconsolidated undrained triaxial
DS	Direct Shear
C	Consolidation
Q _u	Unconfined compression
I _p	Point Load Index (I _p on Borehole Record equals I _p (50) in which the index is corrected to a reference diameter of 50 mm)

	Single packer permeability test; test interval from depth shown to bottom of borehole
	Double packer permeability test; test interval as indicated
	Falling head permeability test using casing
	Falling head permeability test using well point or piezometer

RECORD OF BOREHOLE No RMN-A1

1 OF 1

METRIC

W.P. 3041-22-00 LOCATION Ron McNeil Line Underpass, Southwold, Ontario N:4741937.4 E:408801.5 ORIGINATED BY MC
DIST West HWY Hwy 3 BOREHOLE TYPE Hollow Stem Auger COMPILED BY KL
DATUM Geodetic DATE 2024.06.25 - 2024.06.25 LATITUDE 42.810131 LONGITUDE -81.228339 CHECKED BY RR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa			W _p	W	W _L		
								20 40 60 80 100	20 40 60	20 40 60					
239.2	Gravel Shoulder														
238.9	SILTY SAND to SAND, trace gravel (FILL)		1	SS	13		239						○		
0.3	Compact Brown Moist														
	SILTY CLAY, trace to some sand, trace gravel (FILL)		2	SS	4		238						○		
	Soft to stiff Brown to black Moist		3	SS	5								○		1 12 59 29
236.9							237						○		PP > 4.5 TSF
2.3	CLAYEY SILT (CL), some sand to sandy, trace gravel (TILL)		4	SS	9								○		
	Stiff to very stiff Brown to grey Moist		5	SS	16		236						○		PP > 4.5 TSF
	Grey below 3.8 m		6	SS	19		235						○		PP > 4.5 TSF
			7	SS	20								○		9 21 41 29 PP = 4.25 TSF
			8	SS	14		234						○		
			9	TW	PH		233						○		Consolidation Test
							232						○		
			10	SS	19		231						○		1 10 40 48 PP = 4.5 TSF
							230						○		
229.0							229						○		
10.2	CLAYEY SILT (CL-ML), trace sand Hard Grey Moist		12	SS	44		228						○		0 2 72 26
227.5							227						○		PP = 3.0 TSF
11.7	CLAYEY SILT (CL), trace to some sand, trace gravel (TILL)		13	SS	14		226						○		
	Stiff Grey Moist						225						○		PP = 1.5 TSF Su=170 kPa (B-Vane)
224.7	END OF BOREHOLE												○		
14.5	Borehole open and dry upon completion of drilling.												○		

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

ONTARIO MTO 165001308_MTO_RMNBYPASS_20241008.GPJ ONTARIO MTO GDT 10/16/24

RECORD OF BOREHOLE No RMN-A2

1 OF 1

METRIC

W.P. 3041-22-00 LOCATION Ron McNeil Line Underpass, Southwold, Ontario N:4741843.9 E:408703.6 ORIGINATED BY MC
DIST West HWY Hwy 3 BOREHOLE TYPE Hollow Stem Auger COMPILED BY KL
DATUM Geodetic DATE 2024.06.24 - 2024.06.24 LATITUDE 42.809303 LONGITUDE -81.229552 CHECKED BY RR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					w _p	w	w _L		WATER CONTENT (%)	GR	SA	SI	CL
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× LAB VANE										
237.2	Grass																				
0.3	300 mm TOPSOIL		1	SS	5		237							○							
	SANDY, CLAYEY SILT, trace gravel (FILL) Soft to firm Brown to grey Moist		2	SS	4		236							○							
235.7																					
1.5	CLAYEY SILT (CL), some sand, trace gravel (TILL) Firm to stiff Brown to grey Moist Grey below 2.3 m		3	SS	8		235							○							
			4	SS	16		234							○							
			5	SS	16		234							○							
			6	SS	14		233							○							
			7	TW	PH		232							○							
			8	SS	16		232							○							
			9	SS	17		231							○							
							230														
			10	SS	18		229							○							
							228														
			11	SS	17		227							○							
							226														
			12	SS	19		226							○							
							225														
							</														

ONTARIO MTO 165001308_MTO_RMNBYPASS_20241008.GPJ ONTARIO MTO.GDT 10/16/24

RECORD OF BOREHOLE No RMN-UP1

1 OF 3

METRIC

W.P. 3041-22-00 LOCATION Ron McNeil Line Underpass, Southwold, Ontario N:4741914.1 E:408760.6 ORIGINATED BY HS
DIST West HWY Hwy 3 BOREHOLE TYPE Hollow Stem Auger/Wash Boring COMPILED BY KL
DATUM Geodetic DATE 2024.05.28 - 2024.07.04 LATITUDE 42.809927 LONGITUDE -81.228843 CHECKED BY RR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							W _p	W	W _L
								20 40 60 80 100									
237.6	Grass																
237.4	200 mm TOPSOIL																
0.2	CLAYEY SILT (CL), some sand, trace gravel (TILL) Very stiff to hard Brown to grey Moist		1	SS	7		237								2 12 42 43		
			2	SS	19		236										
			3	SS	26		235								PP = 4.5 TSF Wash Boring below 3.0m PP = 4.5 TSF		
	Grey below 3.0 m		4	SS	23		234								PP = 4.5 TSF		
			5	SS	24		233								PP = 4.5 TSF		
			6	SS	33		232								1 16 47 35 PP = 4.5 TSF		
			7	SS	25		231								PP = 4.0 TSF		
			8	SS	22		230								PP = 4.0 TSF		
			9	SS	26		229										
230.4							228										
7.2	CLAYEY SILT (CL-ML), trace sand Hard Grey Moist		10	SS	45		227								0 2 74 24		
			11	SS	63		226										
							225								PP = 1.0 TSF		
226.8							224										
10.8	CLAYEY SILT (CL), trace to some sand, trace gravel (TILL) Very stiff Grey Moist		12	SS	27		223								PP = 1.0 TSF		
			13	SS	26		222								PP = 1.0 TSF		
			14	SS	20		221										
			15	SS	17												
			15A	TW	PH												
220.6																	

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

ONTARIO MTO 165001308_MTO_RMNBYPASS_20241008.GPJ ONTARIO MTO.GDT 10/16/24

RECORD OF BOREHOLE No RMN-UP1

2 OF 3

METRIC

W.P. 3041-22-00 LOCATION Ron McNeil Line Underpass, Southwold, Ontario N:4741914.1 E:408760.6 ORIGINATED BY HS
DIST West HWY Hwy 3 BOREHOLE TYPE Hollow Stem Auger/Wash Boring COMPILED BY KL
DATUM Geodetic DATE 2024.05.28 - 2024.07.04 LATITUDE 42.809927 LONGITUDE -81.228843 CHECKED BY RR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE							w _p — w — w _L		
							20	40	60	80	100	20	40	60		GR SA SI CL	
17.0	CLAYEY SILT (CL), trace to some sand, trace gravel (TILL) Very stiff to hard Grey Moist		16	SS	18								○			PP = 1.0 TSF	
			17	SS	17								● —			1 13 42 43 PP = 1.0 TSF	
			18	SS	16								○			PP = 1.0 TSF	
			19	SS	23								○			PP = 1.0 TSF	

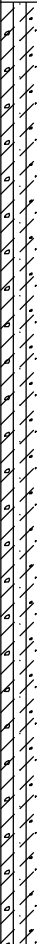
ONTARIO MTO 165001308_MTO_RMNBYPASS_20241008.GPJ ONTARIO MTO.GDT 10/16/24

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

RECORD OF BOREHOLE No RMN-UP1 3 OF 3 METRIC

W.P. 3041-22-00 LOCATION Ron McNeil Line Underpass, Southwold, Ontario N:4741914.1 E:408760.6 ORIGINATED BY HS
DIST West HWY Hwy 3 BOREHOLE TYPE Hollow Stem Auger/Wash Boring COMPILED BY KL
DATUM Geodetic DATE 2024.05.28 - 2024.07.04 LATITUDE 42.809927 LONGITUDE -81.228843 CHECKED BY RR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE												
34.0	CLAYEY SILT (CL), sandy to some sand, trace gravel (TILL) Hard Grey Moist																			
			23	SS	31															
					24	SS	36													

ONTARIO MTO 165001308_MTO_RMNBYPASS_20241008.GPJ ONTARIO MTO.GDT 10/16/24

RECORD OF BOREHOLE No RMN-UP2

1 OF 3

METRIC

W.P. 3041-22-00 LOCATION Ron McNeil Line Underpass, Southwold, Ontario N:4741885.4 E:408747.7 ORIGINATED BY MC/HS
DIST West HWY Hwy 3 BOREHOLE TYPE Hollow Stem Auger/Wash Boring COMPILED BY KL
DATUM Geodetic DATE 2024.06.14 - 2024.06.28 LATITUDE 42.80967 LONGITUDE -81.229006 CHECKED BY RR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W _p	W	W _L			WATER CONTENT (%)
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	20 40 60 80 100	20 40 60					
237.6	Grass														
237.4	200 mm TOPSOIL														
0.2	CLAYEY SILT (CL), trace to some sand, trace to some gravel (TILL) Stiff to hard Brown to grey Moist		1	SS	8							○			
			2	SS	21							○			
			3	SS	35							○			
			4	SS	28							○			
			5	SS	25							○			
	Grey below 3.8 m		6	SS	20							○			
			7	SS	15							○			
			8	TW	PH							○			
			9	SS	13							○			
			10	SS	18							○			
228.9															
8.7	CLAYEY SILT (CL-ML), trace sand, trace to some gravel Hard Grey Moist		11	SS	44							○			
227.9															
9.7	CLAYEY SILT (CL), trace to some sand, trace gravel (TILL) Very stiff to hard Grey Moist														
			12	SS	30							○			
			13	SS	30							○			
	SS14 contains a layer of fine sand		14	SS	19							○			
			15	SS	25							○			
220.6															

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

ONTARIO MTO 165001308_MTO_RMNBYPASS_20241008.GPJ ONTARIO MTO.GDT 10/16/24

RECORD OF BOREHOLE No RMN-UP2

2 OF 3

METRIC

W.P. 3041-22-00 LOCATION Ron McNeil Line Underpass, Southwold, Ontario N:4741885.4 E:408747.7 ORIGINATED BY MC/HS
DIST West HWY Hwy 3 BOREHOLE TYPE Hollow Stem Auger/Wash Boring COMPILED BY KL
DATUM Geodetic DATE 2024.06.14 - 2024.06.28 LATITUDE 42.80967 LONGITUDE -81.229006 CHECKED BY RR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							W _P W W _L	
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE								
17.0	CLAYEY SILT (CL), trace to some sand, trace gravel (TILL) Stiff to very stiff Grey Moist		16	SS	9										PP = 0.5 TSF	
																Su = 180 kPa (B-Vane)
				17	SS	27										1 5 42 53
				18	SS	23										PP = 0.5 TSF
				19	SS	20										PP = 0.5 TSF
				20	SS	21										PP = 1.0 TSF
			21	SS	25										PP = 1.0 TSF	

ONTARIO MTO 165001308_MTO_RMNBYPASS_20241008.GPJ ONTARIO MTO.GDT 10/16/24

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

RECORD OF BOREHOLE No RMN-UP2 3 OF 3 METRIC

W.P. 3041-22-00 LOCATION Ron McNeil Line Underpass, Southwold, Ontario N:4741885.4 E:408747.7 ORIGINATED BY MC/HS
DIST West HWY Hwy 3 BOREHOLE TYPE Hollow Stem Auger/Wash Boring COMPILED BY KL
DATUM Geodetic DATE 2024.06.14 - 2024.06.28 LATITUDE 42.80967 LONGITUDE -81.229006 CHECKED BY RR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× LAB VANE	20	40	60	80					
34.0	CLAYEY SILT (CL), sandy to some sand, trace gravel (TILL) Very stiff to hard Grey Moist 																			

ONTARIO MTO 165001308_MTO_RMNBYPASS_20241008.GPJ ONTARIO MTO.GDT 10/16/24

RECORD OF BOREHOLE No RMN-UP3

1 OF 3

METRIC

W.P. 3041-22-00 LOCATION Ron McNeil Line Underpass, Southwold, Ontario N:4741861.5 E:408721.2 ORIGINATED BY MC
DIST West HWY Hwy 3 BOREHOLE TYPE Hollow Stem Auger/Wash Boring COMPILED BY KL
DATUM Geodetic DATE 2024.06.11 - 2024.06.14 LATITUDE 42.809459 LONGITUDE -81.229334 CHECKED BY RR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT		LIQUID LIMIT	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
237.0	Grass													
236.8	230 mm TOPSOIL		1	SS	9									
0.2	SANDY, CLAYEY SILT, trace gravel (TILL) Very soft to stiff Brown to grey Moist		2	SS	2		236							
235.5														
1.5	CLAYEY SILT (CL), some sand, trace gravel (TILL) Stiff to hard Brown to grey Moist Inferred cobbles/boulder based on auger grinding between 1.8 m and 2.1 m		3	SS	15		235							1 9 50 40
			4	SS	76		234							Wash Boring below 3.0m
	Grey below 3.8 m		5	SS	26		233							
			6	SS	18		232							
			7	SS	26		231							
			8	SS	25		230							
			9	SS	36		229							2 11 52 35
	Inferred cobbles/boulder based on auger grinding at 6.9 m						228							
			10	SS	28		227							PP = 4.5 TSF
							226							2 15 45 38 PP = 4.0 TSF
							225							PP = 4.5 TSF
			12	SS	28		224							
							223							PP = 3.25 TSF
			13	SS	42		222							
							221							PP = 2.75 TSF
			14	SS	26									
			15	SS	31									
220.0														

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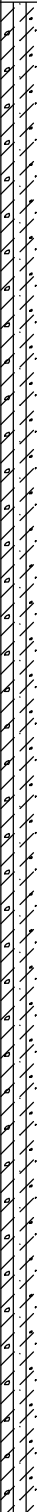
ONTARIO MTO 165001308_MTO_RMNBYPASS_20241008.GPJ ONTARIO MTO.GDT 10/16/24

RECORD OF BOREHOLE No RMN-UP3

2 OF 3

METRIC

W.P. 3041-22-00 LOCATION Ron McNeil Line Underpass, Southwold, Ontario N:4741861.5 E:408721.2 ORIGINATED BY MC
DIST West HWY Hwy 3 BOREHOLE TYPE Hollow Stem Auger/Wash Boring COMPILED BY KL
DATUM Geodetic DATE 2024.06.11 - 2024.06.14 LATITUDE 42.809459 LONGITUDE -81.229334 CHECKED BY RR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa			w _p	w	w _L			WATER CONTENT (%)
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL						
								20 40 60 80 100								
17.0	CLAYEY SILT (CL), trace to some sand, trace gravel (TILL) Very stiff to hard Grey Moist		16	SS	31		219								PP = 2.75 TSF	
				17	SS	21										1 12 40 47 PP = 1.75 TSF
				18	SS	20		217								PP = 1.75 TSF
				18A	TW	PH		216								
								215								
				19	SS	30		214								PP = 2.25 TSF
								213								
								212								
				20	SS	27		211								PP = 2.0 TSF
								210								
								209								
				21	SS	32		208								1 5 45 49 PP = 2.25 TSF
								207								
								206								
								205								
				22	SS	51		204								
203.0	Inferred cobbles/boulder based on auger grinding at 32.9 m															

ONTARIO MTO 165001308_MTO_RMNBYPASS_20241008.GPJ ONTARIO MTO.GDT 10/16/24

Continued Next Page

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

RECORD OF BOREHOLE No RMN-UP3 3 OF 3 METRIC

W.P. 3041-22-00 LOCATION Ron McNeil Line Underpass, Southwold, Ontario N:4741861.5 E:408721.2 ORIGINATED BY MC
DIST West HWY Hwy 3 BOREHOLE TYPE Hollow Stem Auger/Wash Boring COMPILED BY KL
DATUM Geodetic DATE 2024.06.11 - 2024.06.14 LATITUDE 42.809459 LONGITUDE -81.229334 CHECKED BY RR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)
								○ UNCONFINED	+	FIELD VANE						
						● QUICK TRIAXIAL	×	LAB VANE								
						20	40	60	80	100	20	40	60			
34.0	CLAYEY SILT (CL), some sand, trace gravel (TILL) Hard Grey Moist						202								PP = 1.0 TSF	
			23	SS	44		201									
							200									
							199									
			24	SS	50		198									
							197									
							196									
			25	SS	45		195									
							194									
	Inferred cobbles/boulder based on auger grinding at 39.0 m					193								PP > 4.5 TSF		
26			SS	170												
192.3 44.7	DCPT refusal at 44.7 m END OF BOREHOLE				100/ 80											
	Borehole dry upon commencement of mud drilling at 3 m below grade.															

ONTARIO MTO 165001308_MTO_RMNBYPASS_20241008.GPJ ONTARIO MTO.GDT 10/16/24

PRESENTATION OF SITE INVESTIGATION RESULTS

HWY 3 St Thomas CPT

Prepared for:

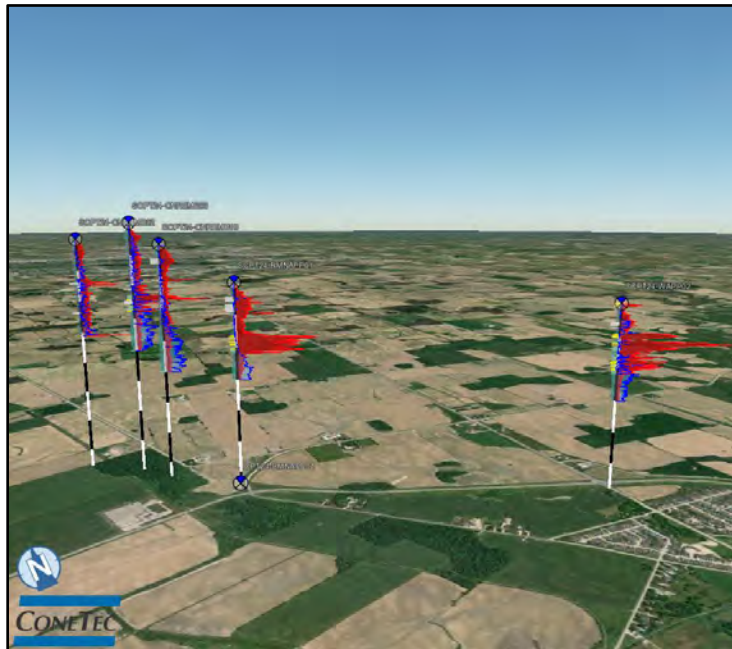
Stantec Consulting Ltd.

ConeTec Job No: 24-05-27609

Project Start Date: 2024-05-09

Project End Date: 2024-05-10

Report Date: 2024-05-24



Prepared by:

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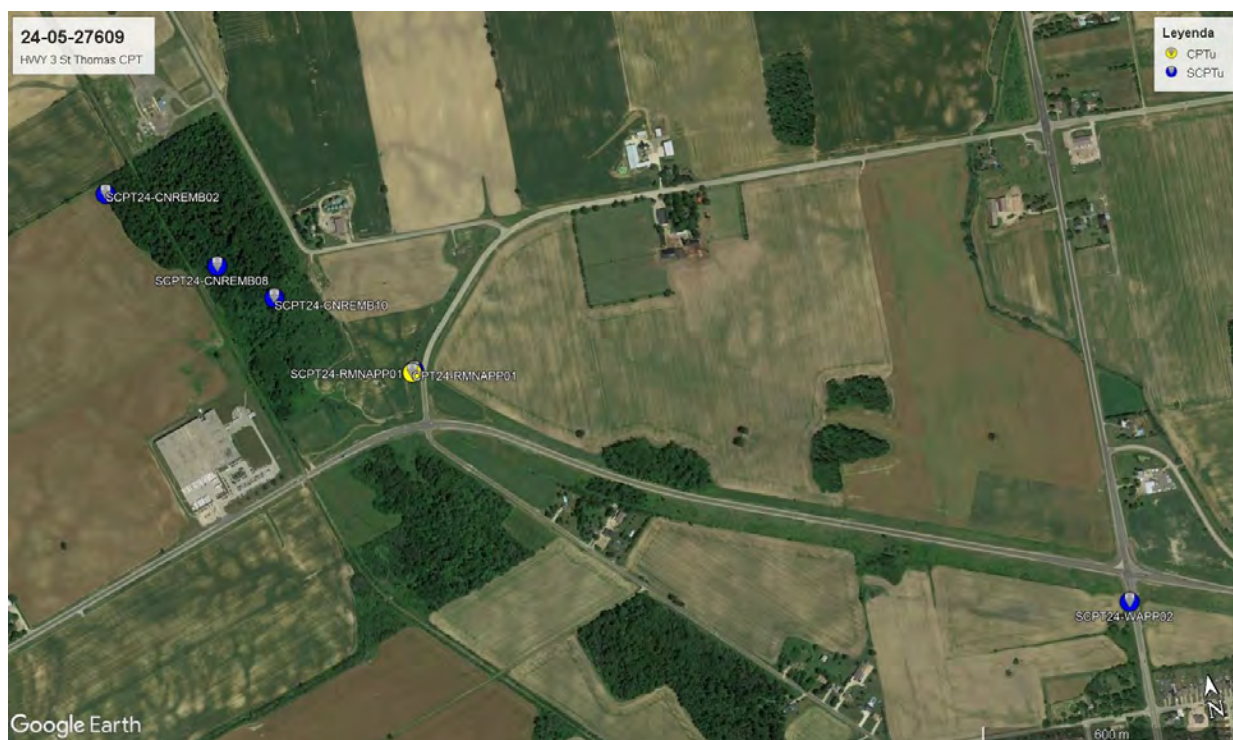
Introduction

The enclosed report presents the results of the site investigation program conducted by ConeTec Investigations Ltd. for Stantec Consulting Ltd. at HWY 3, St. Thomas, ON. The program consisted of 1 cone penetration test (CPTu) and 5 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	Stantec Consulting Ltd.
Project	HWY 3 St Thomas CPT
ConeTec project number	24-05-27609

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



Rig Description	Deployment System	Test Type
CPT track rig (TC23)	30 ton rig cylinder	CPTu, SCPTu

Coordinates		
Test Type	Collection Method	EPSG Number
CPTu, 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)
729:T1500F15U35	729	15	225	1500	15	35
Cone 729 was used for all CPTu soundings.						

Cone Penetration Test (CPTu)	
Depth reference	Depths are referenced to the existing ground surface at the time of each test.
Tip and sleeve data offset	0.1 meter This has been accounted for in the CPT data files.
Additional plots	<ul style="list-style-type: none"> Advanced plots with I_c, S_u, ϕ and $N1(60)I_c$ Seismic shear wave velocity plots Soil Behaviour Type (SBT) scatter plots

Calculated Geotechnical Parameter Tables	
Additional information	<p>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).</p> <p>Effective stresses are calculated based on unit weights that have been assigned to the individual soil behaviour type zones and hydrostatic conditions were assumed.</p> <p>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).</p>

Limitations

3rd Party Disclaimer

This report titled “HWY 3 St Thomas CPT”, referred to as the (“Report”), was prepared by ConeTec for Stantec Consulting 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 Stantec Consulting Ltd. to collect and provide the raw data (“Data”) which is included in this report titled “HWY 3 St Thomas CPT”, which is referred to as the (“Report”). ConeTec has collected and reported the Data in accordance with current industry standards. No other warranty, express or implied, with respect to the Data is made by ConeTec. In order to properly understand the Data included in the Report, reference must be made to the documents accompanying and other sources referenced in the Report in their entirety. Any analysis, interpretation, judgment, calculations and/or geotechnical parameters (collectively “Interpretations”) included in the Report, including those based on the Data, are outside the scope of ConeTec’s retainer and are included in the Report as a courtesy only. Other than the Data, the contents of the Report (including any Interpretations) should not be relied upon in any fashion without independent verification and ConeTec is in no way responsible for any loss, damage or expense resulting from the use of, and/or reliance on, such material by any party.

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

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

ConeTec penetrometers are manufactured with various tip, friction and pore pressure capacities in both 10 cm² and 15 cm² 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 cm² penetrometers do not require friction reducers as they have a diameter larger than the deployment rods. The 10 cm² piezocones use a friction reducer consisting of a rod adapter extension behind the main cone body with an enlarged cross sectional area (typically 44 millimeters diameter over a length of 32 millimeters with tapered leading and trailing edges) located at a distance of 585 millimeters above the cone tip.

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

All ConeTec piezocones can record pore pressure at various locations. Unless otherwise noted, the pore pressure filter is located directly behind the cone tip in the "u₂" position ([ASTM Type 2](#)). The filter is six millimeters thick, made of porous plastic (polyethylene) having an average pore size of 125 microns (90-160 microns). The function of the filter is to allow rapid movements of extremely small volumes of water needed to activate the pressure transducer while preventing soil ingress or blockage.

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

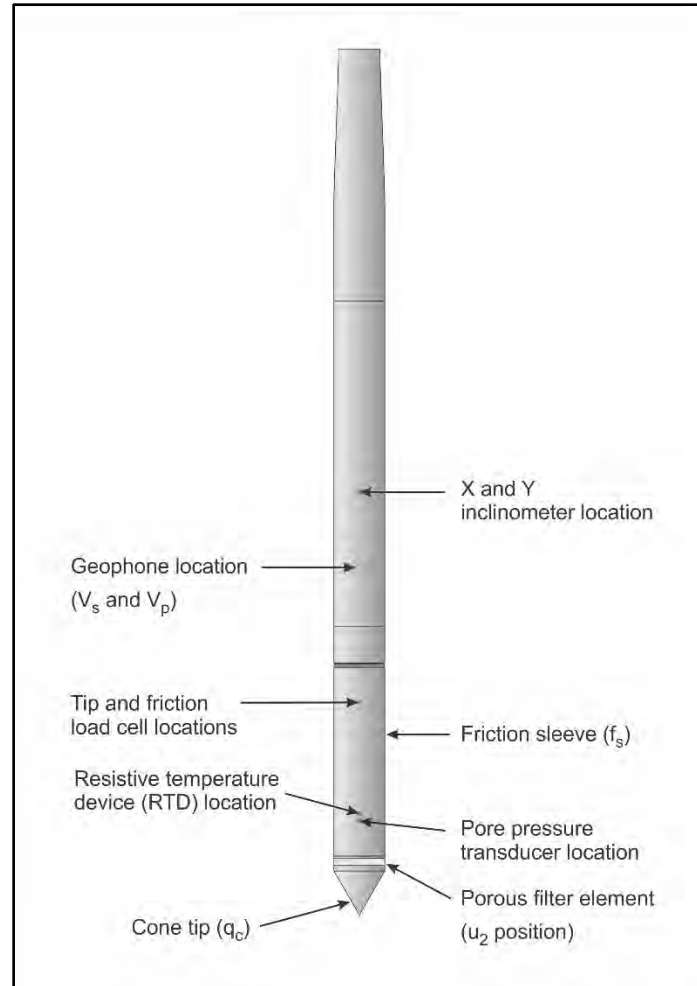


Figure CPTu. Piezocone Penetrometer (15 cm²)

The ConeTec data acquisition systems consist of a Windows based computer and a signal interface box and power supply. The signal interface combines depth increment signals, seismic trigger signals and the downhole digital data. This combined data is then sent to the Windows based computer for collection and presentation. The data is recorded at fixed depth increments using a depth wheel attached to the push cylinders or by using a spring loaded rubber depth wheel that is held against the cone rods. The typical recording interval is 2.5 centimeters; custom recording intervals are possible.

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

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

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

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

The CPTu is conducted at a steady rate of two centimeters per second, within acceptable tolerances. Typically one meter length rods with an outer diameter of 38.1 millimeters are added to advance the cone to the sounding termination depth. After cone retraction final baselines are recorded.

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

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

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

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

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

where: q_t is the corrected tip resistance

q_c is the recorded tip resistance

u_2 is the recorded dynamic pore pressure behind the tip (u_2 position)

a is the Net Area Ratio for the piezocone (0.8 for ConeTec probes)

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

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

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

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

For additional information on CPTu interpretations and calculated geotechnical parameters, refer to [Robertson et al. \(1986\)](#), [Lunne et al. \(1997\)](#), [Robertson \(2009\)](#), Mayne (2013, 2014), [Mayne and Peuchen \(2012\)](#) and [Mayne et al. \(2023\)](#).

References

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

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

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

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

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

Mayne, P.W., Cargill, E. and Greig, J. (2023). The Cone Penetration Test: Better Information, Better Decisions. produced by ConeTec Group, Burnaby, B.C. www.conetec.com

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

Robertson, P.K., 1990, "Soil Classification Using the Cone Penetration Test", Canadian Geotechnical Journal, Volume 27: 151-158. DOI: [10.1139/T90-014](#).

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

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

ConeTec's 15 cm² 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 may be used for both shear waves and compression waves. The hammer and beam act as a contact trigger that initiates the recording of the seismic wave traces. For impulsive devices an accelerometer trigger may be used. The traces are recorded in the memory of the cone using a fast analog to digital converter. The seismic trace is then transmitted digitally uphole to a Windows based computer through a signal interface box for recording and analysis. An illustration of the shear wave testing configuration is presented in [Figure SCPTu-1](#).

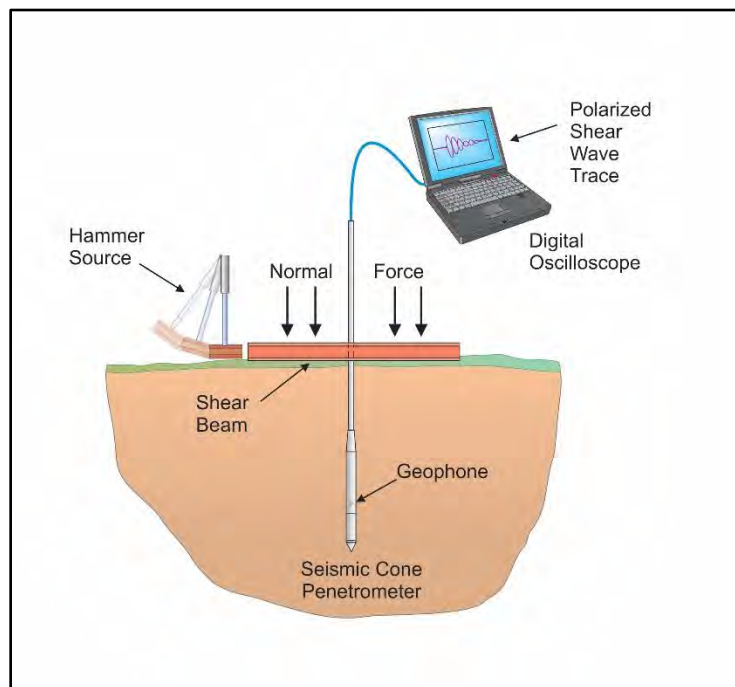


Figure SCPTu-1. Illustration of the SCPTu system

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

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

Prior to recording seismic waves at each test depth, cone penetration is stopped and the rods are decoupled from the rig to avoid transmission of rig energy down the rods. Typically, five wave traces for each orientation are recorded for quality control and uncertainty analysis purposes. After reviewing wave traces for consistency the cone is pushed to the next test depth (typically one meter intervals or as requested by the client). [Figure SCPTu-2](#) presents an illustration of a SCPTu test.

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

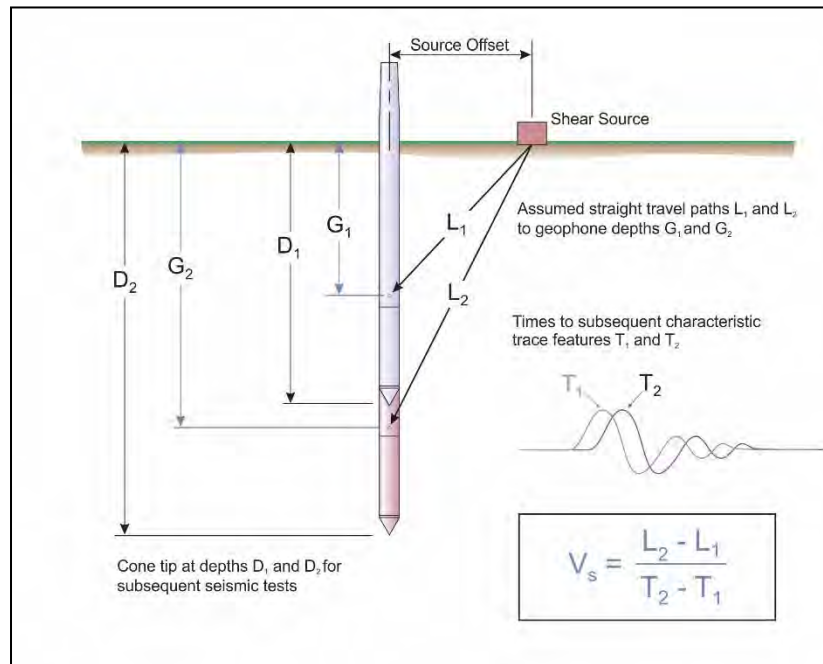


Figure SCPTu-2. Illustration of a seismic cone penetration test

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 thirty meters (V_{s30}) has been calculated and provided for all applicable soundings using an equation presented in [Crow et al. \(2012\)](#).

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

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

References

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

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

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

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

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

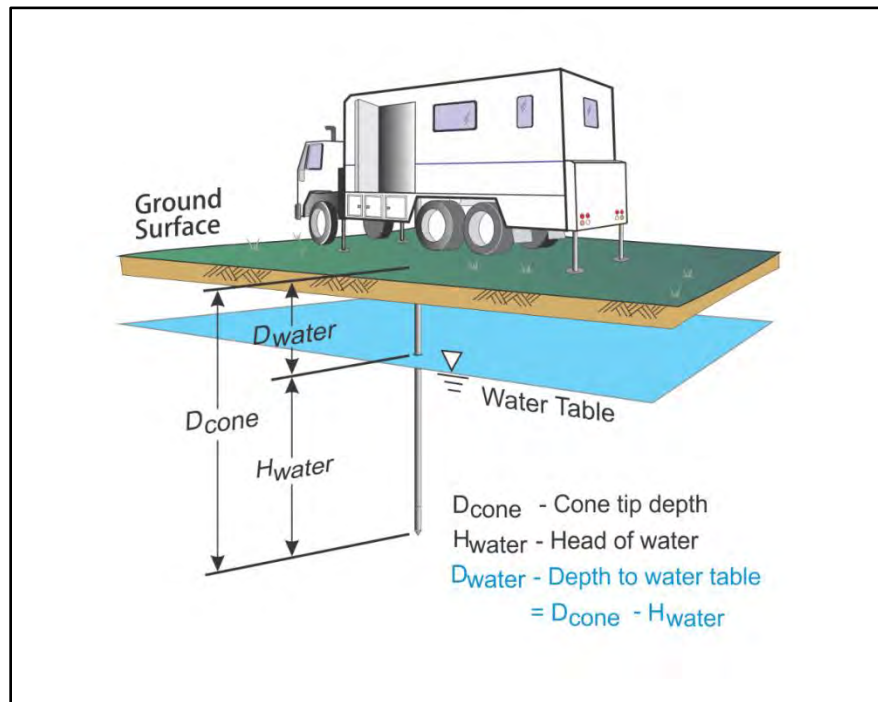


Figure PPD-1. Pore pressure dissipation test setup

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

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

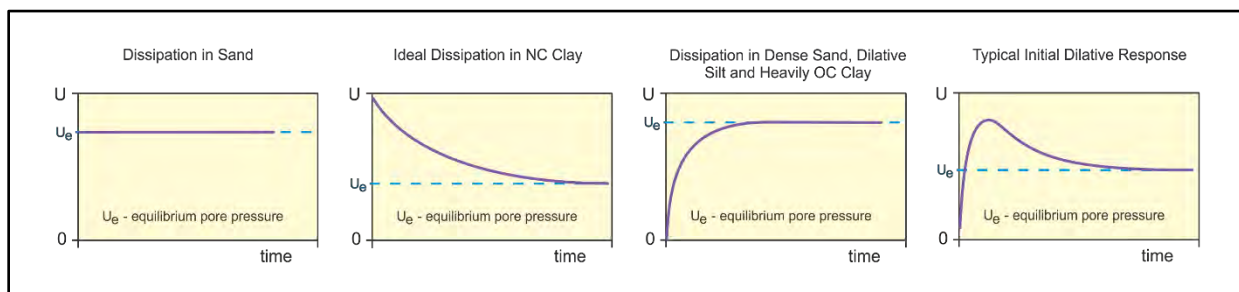


Figure PPD-2. Pore pressure dissipation curve examples

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

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

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

Where:

T^* is the dimensionless time factor ([Table Time Factor](#))

a is the radius of the cone

I_r is the rigidity index

t is the time at the degree of consolidation

Table Time Factor. T^* versus degree of dissipation ([Teh and Houlsby, 1991](#))

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

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

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

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

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

References

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

The appendices listed below are included in the report:

- Cone Penetration Test Summary and Standard Cone Penetration Test Plots
- Advanced Cone Penetration Test Plots
- Seismic Cone Penetration Test Plots
- Seismic Cone Penetration Test Shear Wave (V_s) Tabular Results
- Seismic Cone Penetration Test Shear Wave (V_s) Traces
- Soil Behaviour Type (SBT) Scatter Plots
- Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots
- Description of Methods for Calculated CPT Geotechnical Parameters

Cone Penetration Test Summary and Standard Cone Penetration Test Plots



Job No: 24-05-27609
Client: Stantec Consulting Ltd.
Project: HWY 3 St Thomas CPT
Start Date: 2024-05-09
End Date: 2024-05-10

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
CPT24-RMNAPP01	24-05-27609_CP-RM-01	2024-05-10	729:T1500F15U35	15	2.2	15.025	4739733	481289	
SCPT24-RMNAPP01	24-05-27609_SP-RM-01	2024-05-10	729:T1500F15U35	15	2.2	15.075	4739737	481294	
SCPT24-CNREMB02	24-05-27609_SP-CN-02	2024-05-09	729:T1500F15U35	15	2.0	15.575	4740267	480674	
SCPT24-WAPP02	24-05-27609_SP-WA-02	2024-05-10	729:T1500F15U35	15	1.8	15.000	4738905	482817	3
SCPT24-CNREMB08	24-05-27609_SP-CN-08	2024-05-09	729:T1500F15U35	15	1.5	20.100	4740056	480896	
SCPT24-CNREMB10	24-05-27609_SP-CN-10	2024-05-10	729:T1500F15U35	15	2.0	20.000	4739960	481010	

1. The assumed phreatic surface was based on the dynamic pore pressure response, unless otherwise noted. Hydrostatic conditions were assumed for the calculated parameters.
2. Coordinates were collected with a consumer grade GPS device with datum WGS84/UTM Zone 17 North.
3. The assumed phreatic surface was based on a pore pressure dissipation test.



Stantec

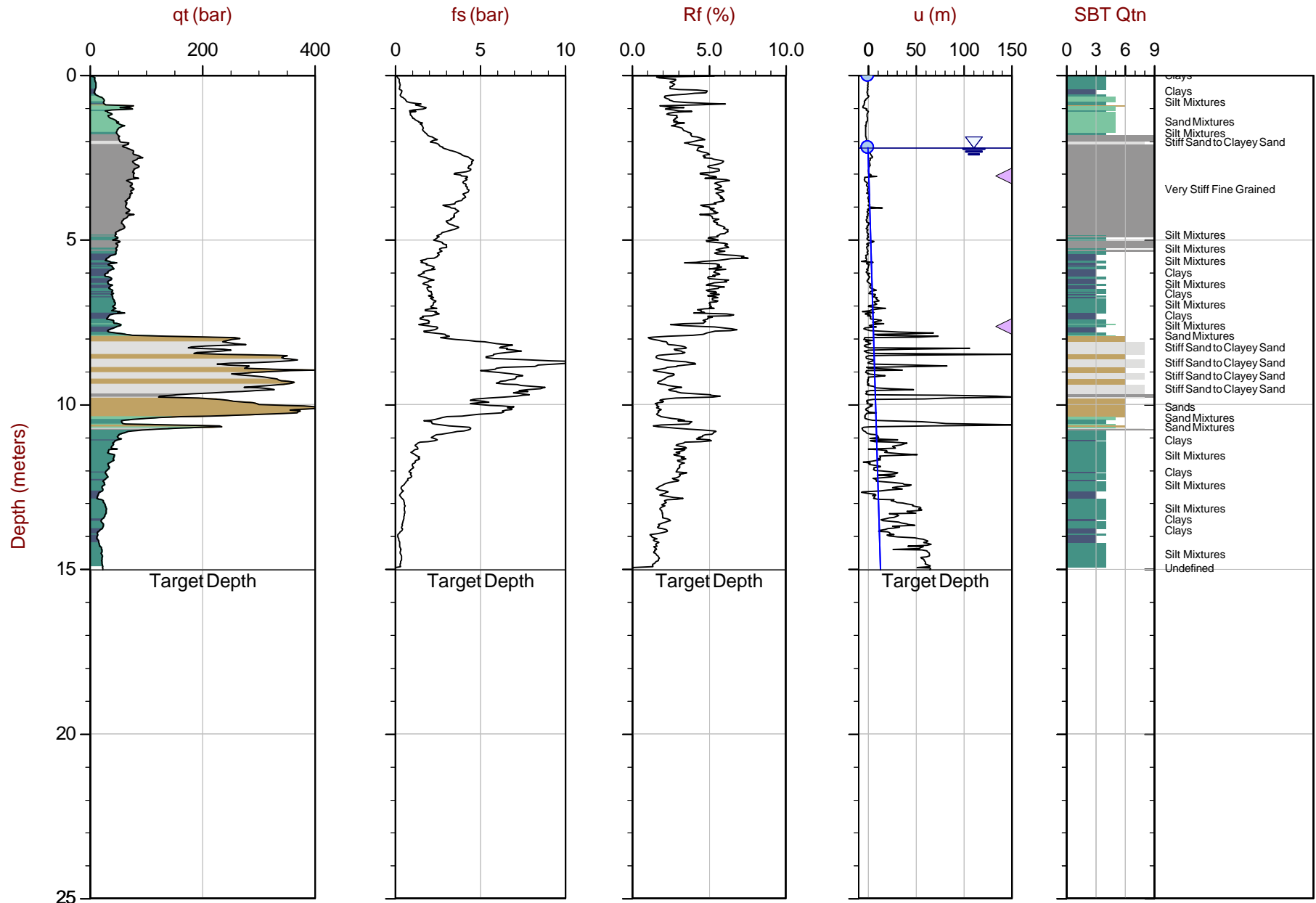
Job No: 24-05-27609

Date: 2024-05-10 12:15

Site: HWY 3, St.Thomas, ON

Sounding: CPT24-RMNAPP01

Cone: 729:T1500F15U35 Area=15 cm²



Max Depth: 15.025 m / 49.29 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: Every Point

File: 24-05-27609_CP-RM-01.COR

Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010

Coords: UTM17N: 4739733m E: 481289m

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



Stantec

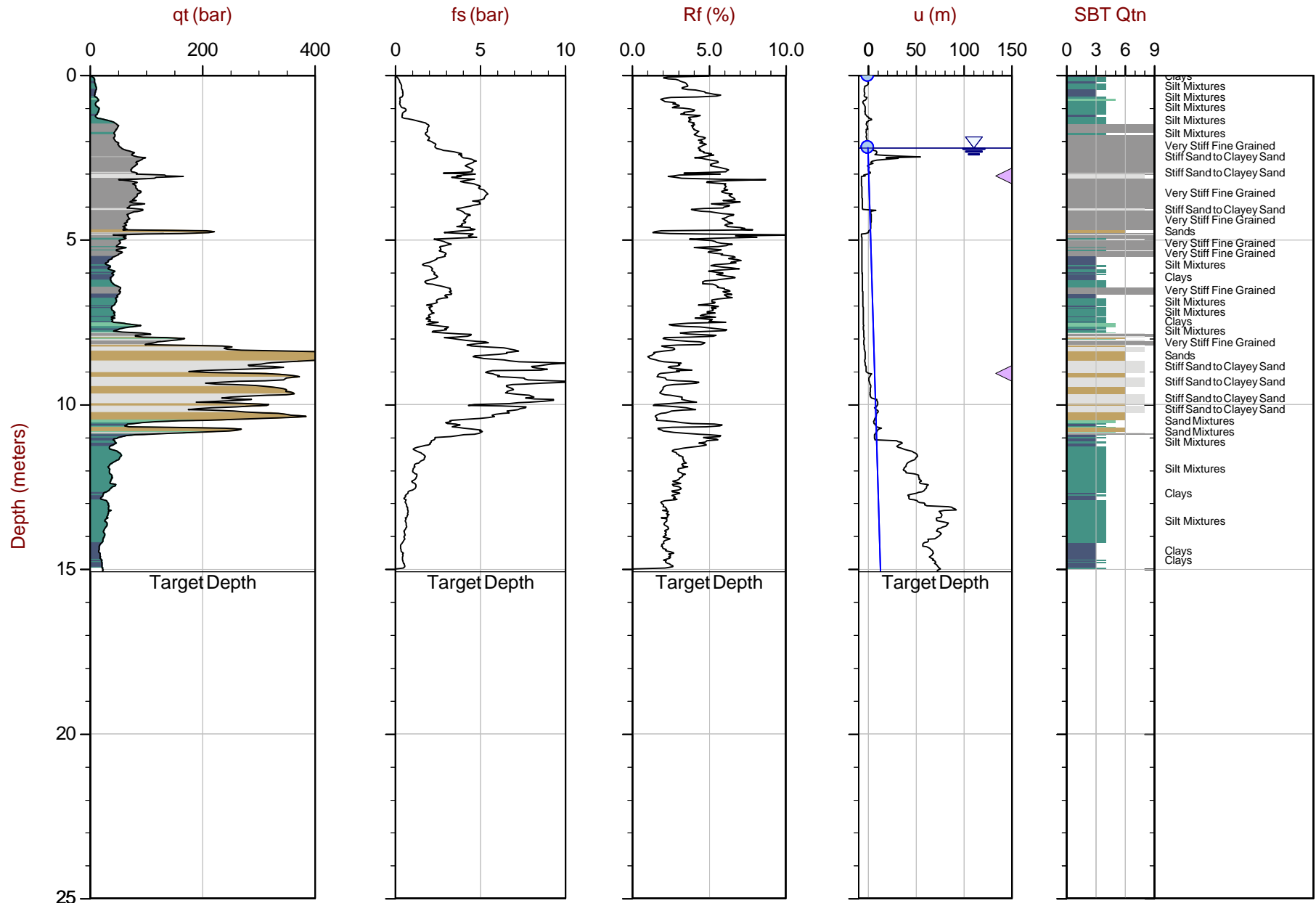
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Date: 2024-05-10 10:24

Site: HWY 3, St.Thomas, ON

Sounding: SCPT24-RMNAPP01

Cone: 729:T1500F15U35 Area=15 cm²



Max Depth: 15.075 m / 49.46 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: Every Point

File: 24-05-27609_SP-RM-01.COR

Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010

Coords: UTM17N:4739737mE:481294m

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



Stantec

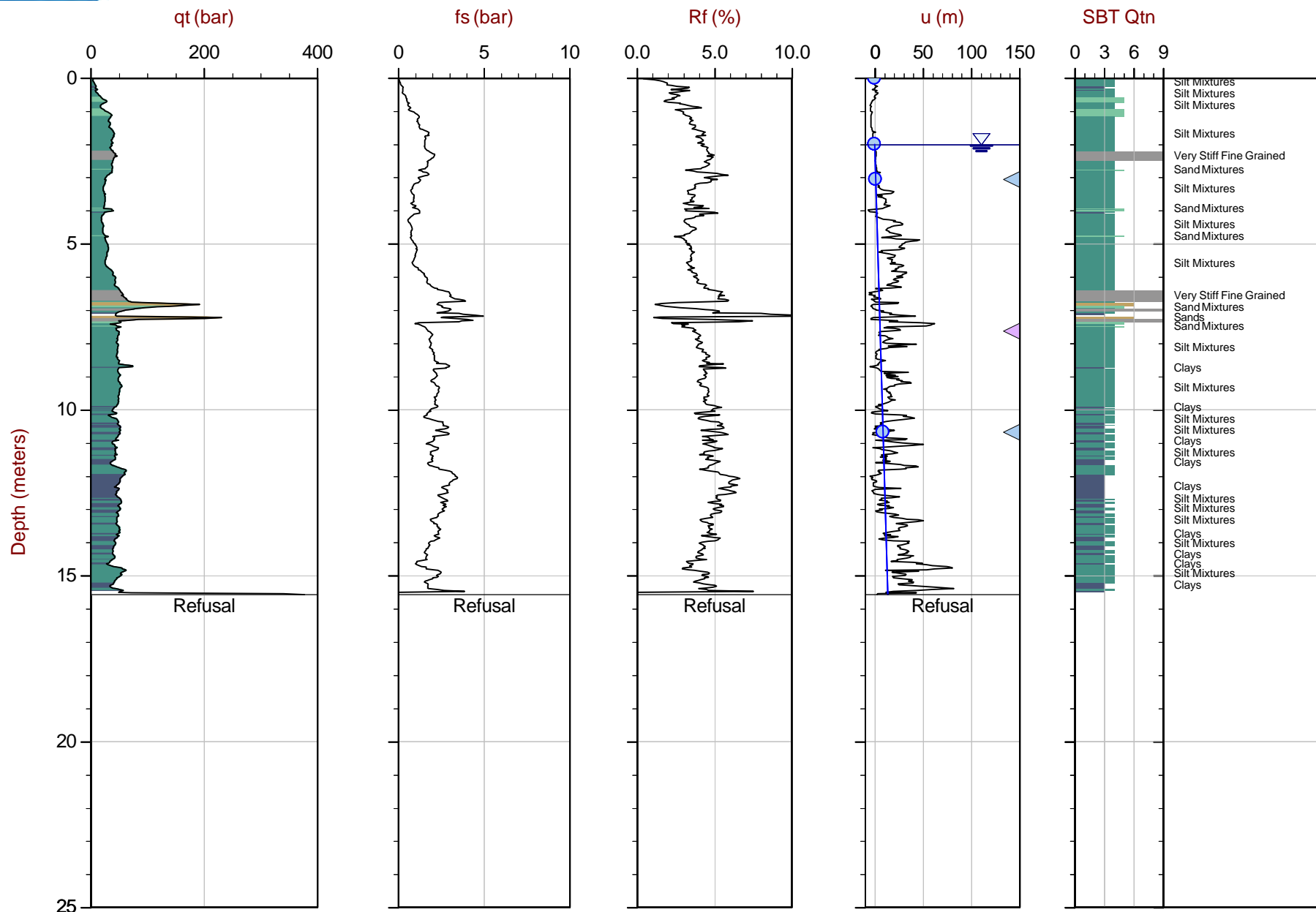
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Date: 2024-05-09 12:06

Site: HWY 3, St.Thomas, ON

Sounding: SCPT24-CNREMB02

Cone: 729:T1500F15U35 Area=15 cm²



Max Depth: 15.575 m / 51.10 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: Every Point

File: 24-05-27609_SP-CN-02.COR

Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010

Coords: UTM 17N N: 4740267m E: 480674m

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



Stantec

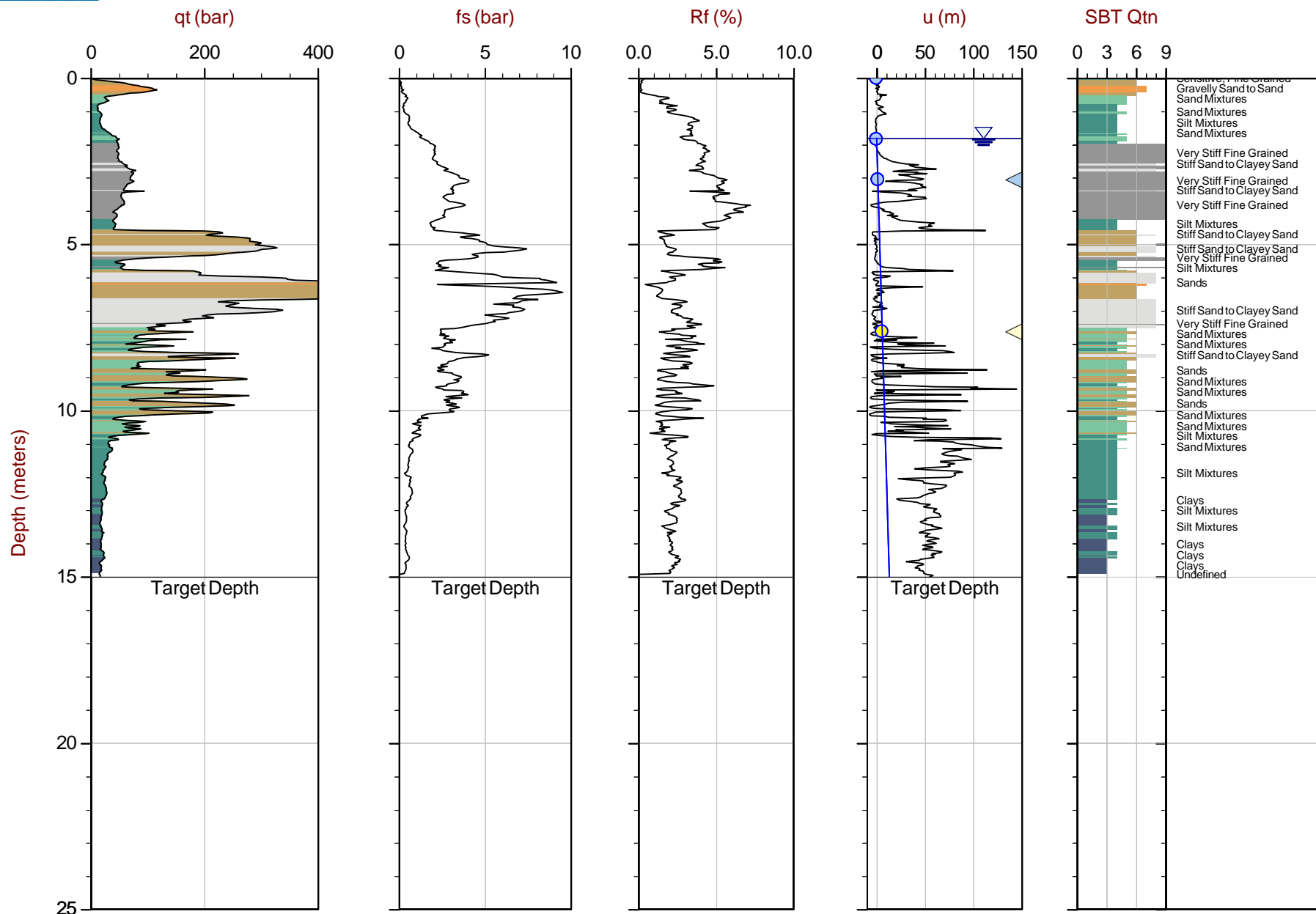
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Date: 2024-05-10 14:49

Site: HWY 3, St.Thomas, ON

Sounding: SCPT24-WAPP02

Cone: 729:T1500F15U35 Area=15 cm²



Max Depth: 15.000 m / 49.21 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: Every Point

File: 24-05-27609_SP-WA-02.COR

Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010

Coords: UTM17N: 4738905m E: 482817m

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



Stantec

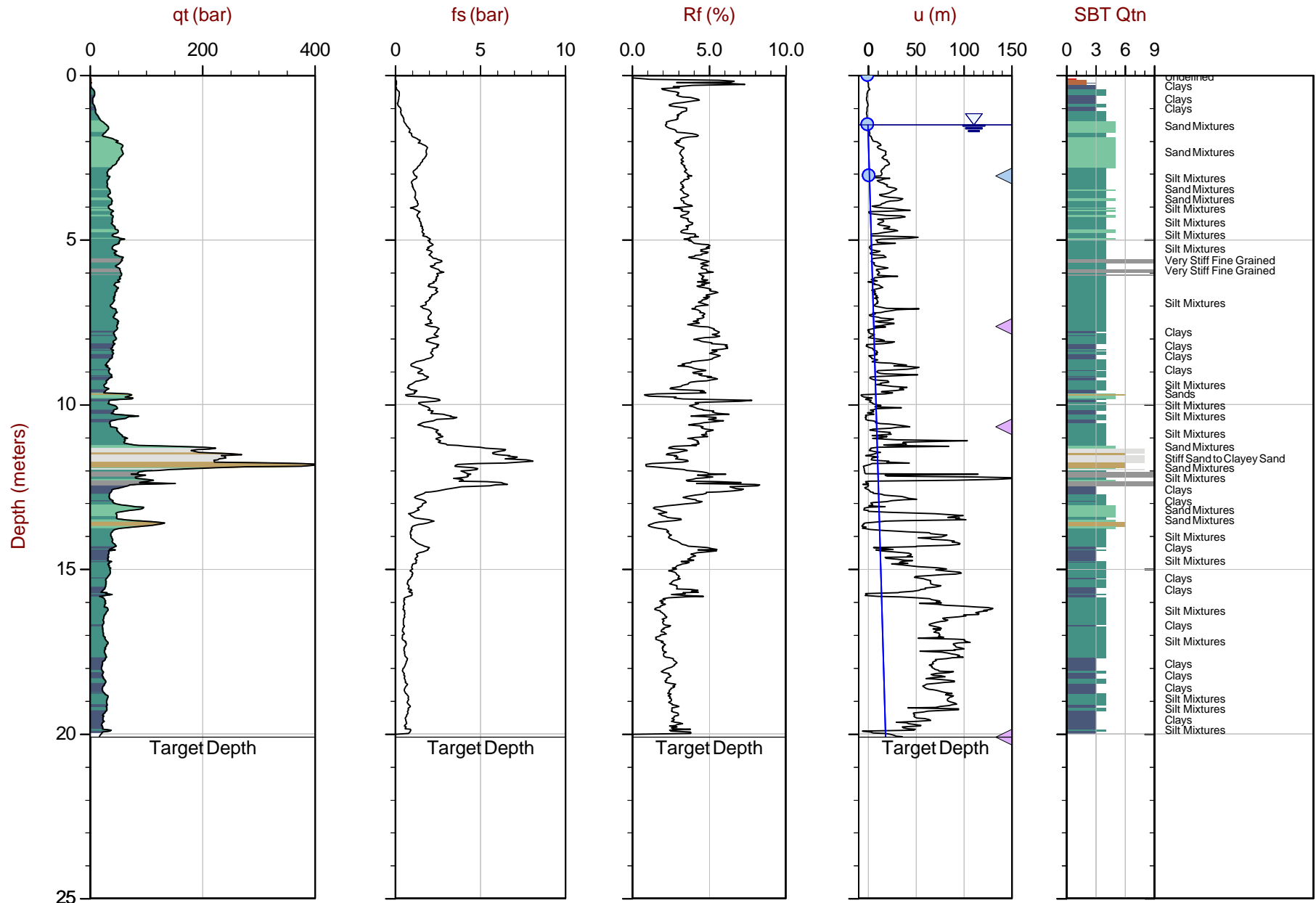
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Date: 2024-05-09 16:43

Site: HWY 3, St.Thomas, ON

Sounding: SCPT24-CNREMB08

Cone: 729:T1500F15U35 Area=15 cm²



Max Depth: 20.100 m / 65.94 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: Every Point

File: 24-05-27609_SP-CN-08.COR

Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010

Coords: UTM 17N N: 4740056m E: 480896m

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



Stantec

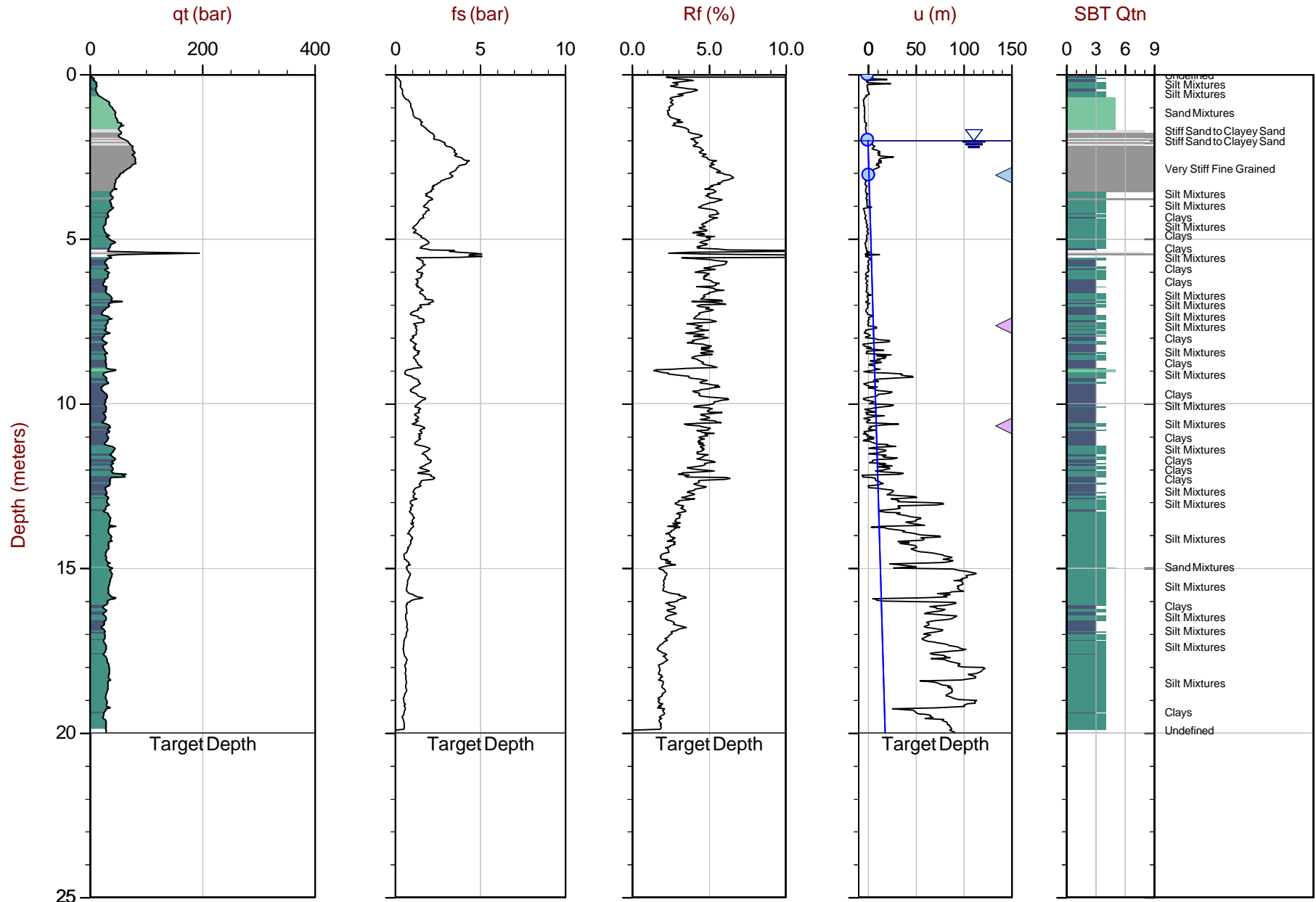
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Date: 2024-05-10 06:58

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Sounding: SCPT24-CNREMB10

Cone: 729:T1500F15U35 Area=15 cm²



Max Depth: 20.000 m / 65.62 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: Every Point

File: 24-05-27609_SP-CN-10.COR

Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010

Coords: UTM17N: 4739960mE: 481010m

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

Advanced Cone Penetration Test Plots



Stantec

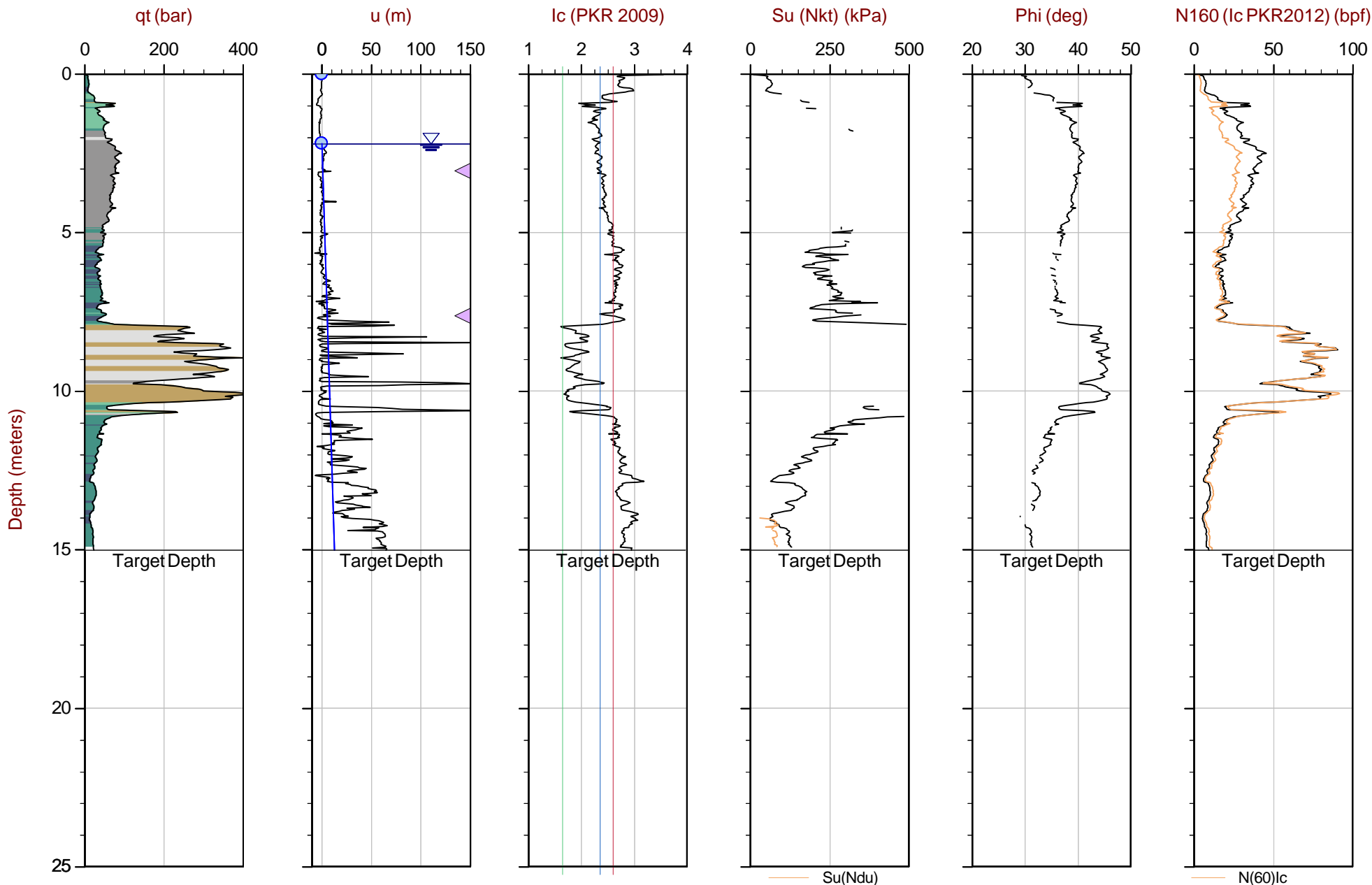
Job No: 24-05-27609

Date: 2024-05-10 12:15

Site: HWY 3, St.Thomas, ON

Sounding: CPT24-RMNAPP01

Cone: 729:T1500F15U35 Area=15 cm²



Max Depth: 15.025 m / 49.29 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: EveryPoint

File: 24-05-27609_CP-RM-01.COR

Unit Wt: SBTQtn(PKR2009)

SuNkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010

Coords: UTM17N: 4739733mE: 481289m

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



Stantec

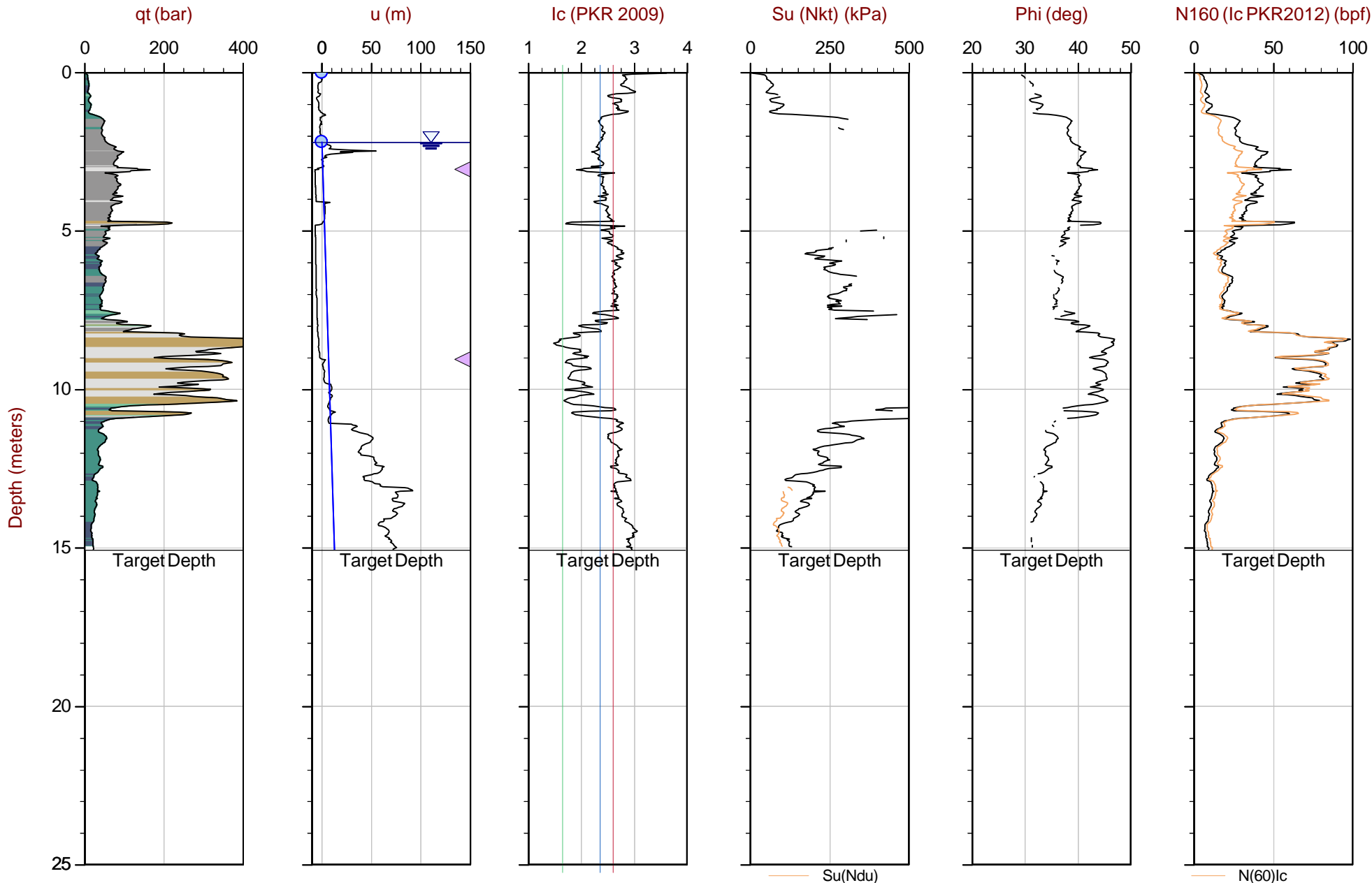
Job No: 24-05-27609

Date: 2024-05-10 10:24

Site: HWY 3, St.Thomas, ON

Sounding: SCPT24-RMNAPP01

Cone: 729:T1500F15U35 Area=15 cm²



Max Depth: 15.075 m / 49.46 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: Every Point

File: 24-05-27609_SP-RM-01.COR

Unit Wt: SBTQtn(PKR2009)

Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010

Coords: UTM17N: 4739737mE: 481294m

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



Stantec

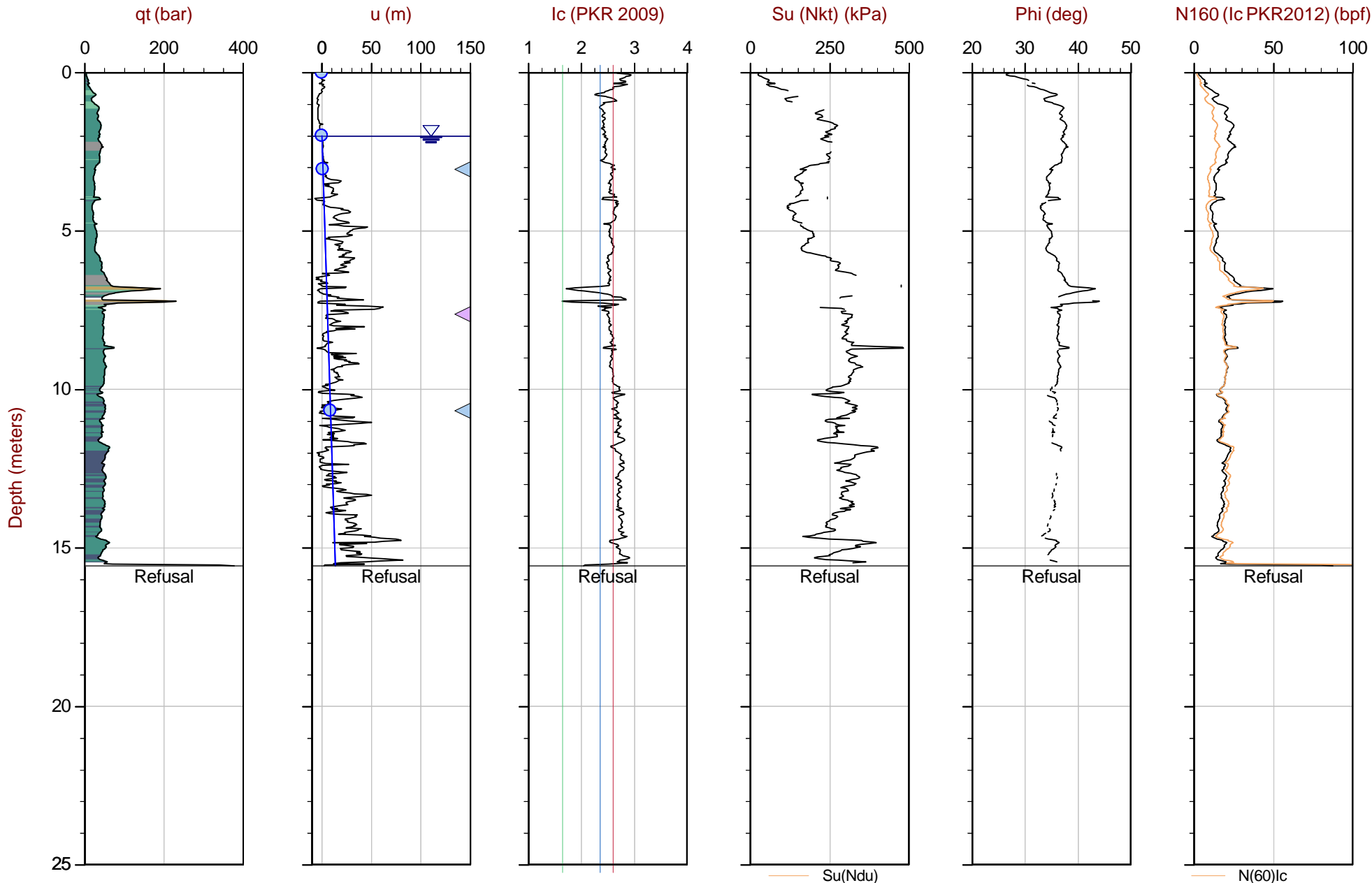
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Date: 2024-05-09 12:06

Site: HWY 3, St.Thomas, ON

Sounding: SCPT24-CNREMB02

Cone: 729:T1500F15U35 Area=15 cm²



Max Depth: 15.575 m / 51.10 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: EveryPoint

File: 24-05-27609_SP-CN-02.COR

Unit Wt: SBTQtn(PKR2009)

Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010

Coords: UTM17N: 4740267mE: 480674m

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



Stantec

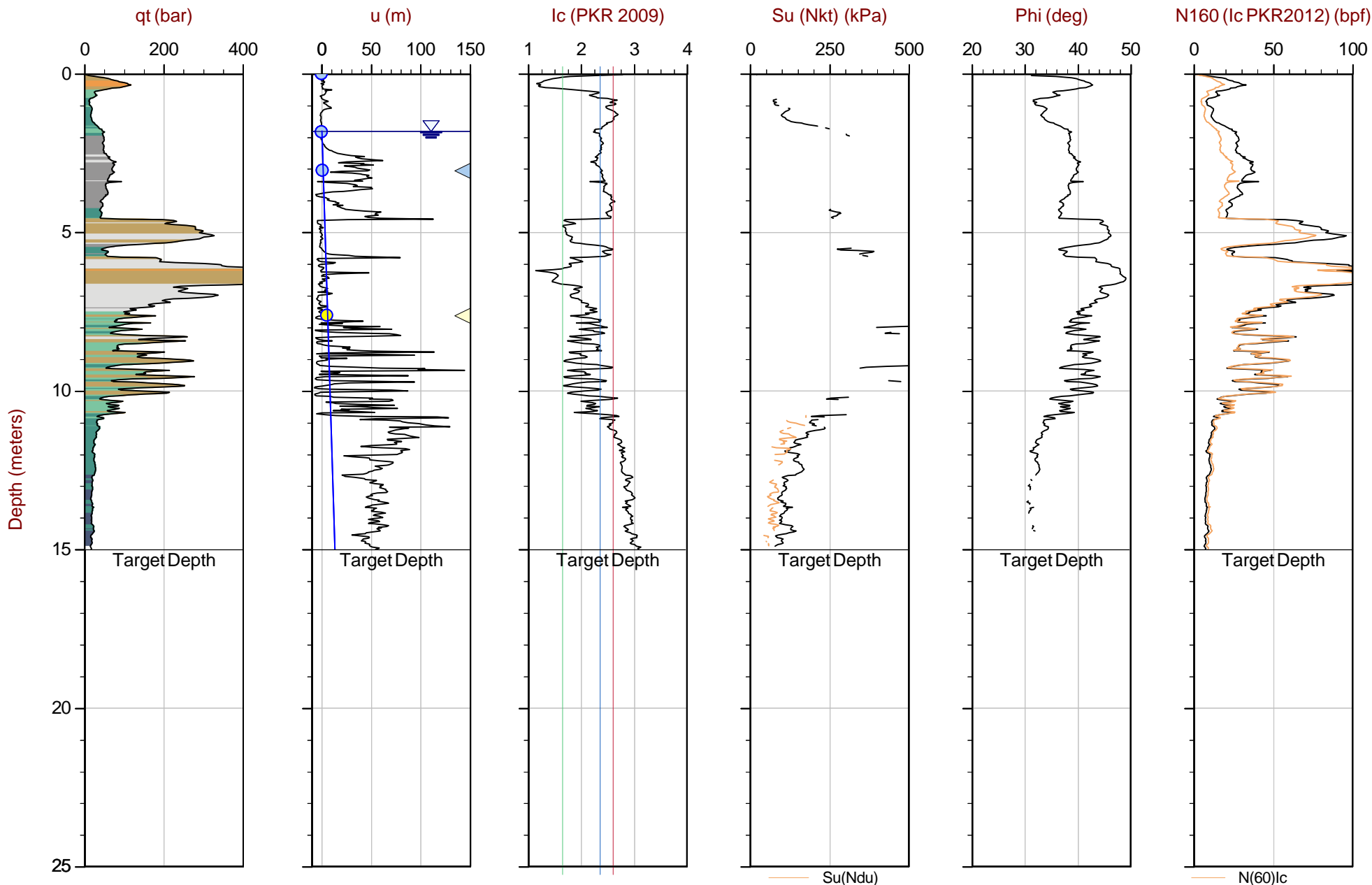
Job No: 24-05-27609

Date: 2024-05-10 14:49

Site: HWY 3, St.Thomas, ON

Sounding: SCPT24-WAPP02

Cone: 729:T1500F15U35 Area=15 cm²



Max Depth: 15.000 m / 49.21 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: EveryPoint

File: 24-05-27609_SP-WA-02.COR

Unit Wt: SBTQtn (PKR2009)

Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010

Coords: UTM17N: 4738905mE: 482817m

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



Stantec

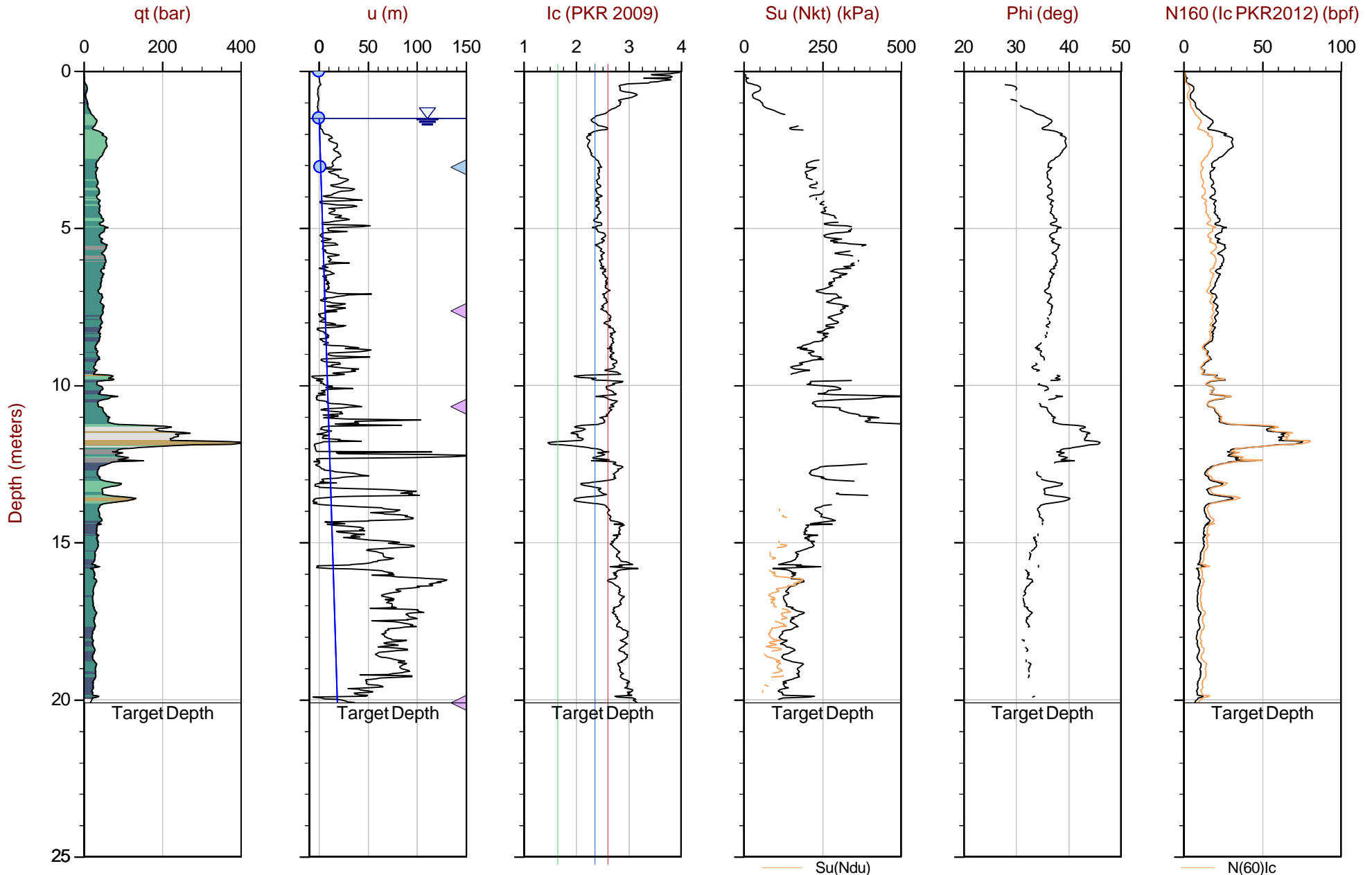
Job No: 24-05-27609

Date: 2024-05-09 16:43

Site: HWY 3, St.Thomas, ON

Sounding: SCPT24-CNREMB08

Cone: 729:T1500F15U35 Area=15 cm²



Max Depth: 20.100 m / 65.94 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: EveryPoint

File: 24-05-27609_SP-CN-08.COR

Unit Wt: SBTQtn(PKR2009)

Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010

Coords: UTM17N N: 4740056m E: 480896m

Sheet No: 1 of 1

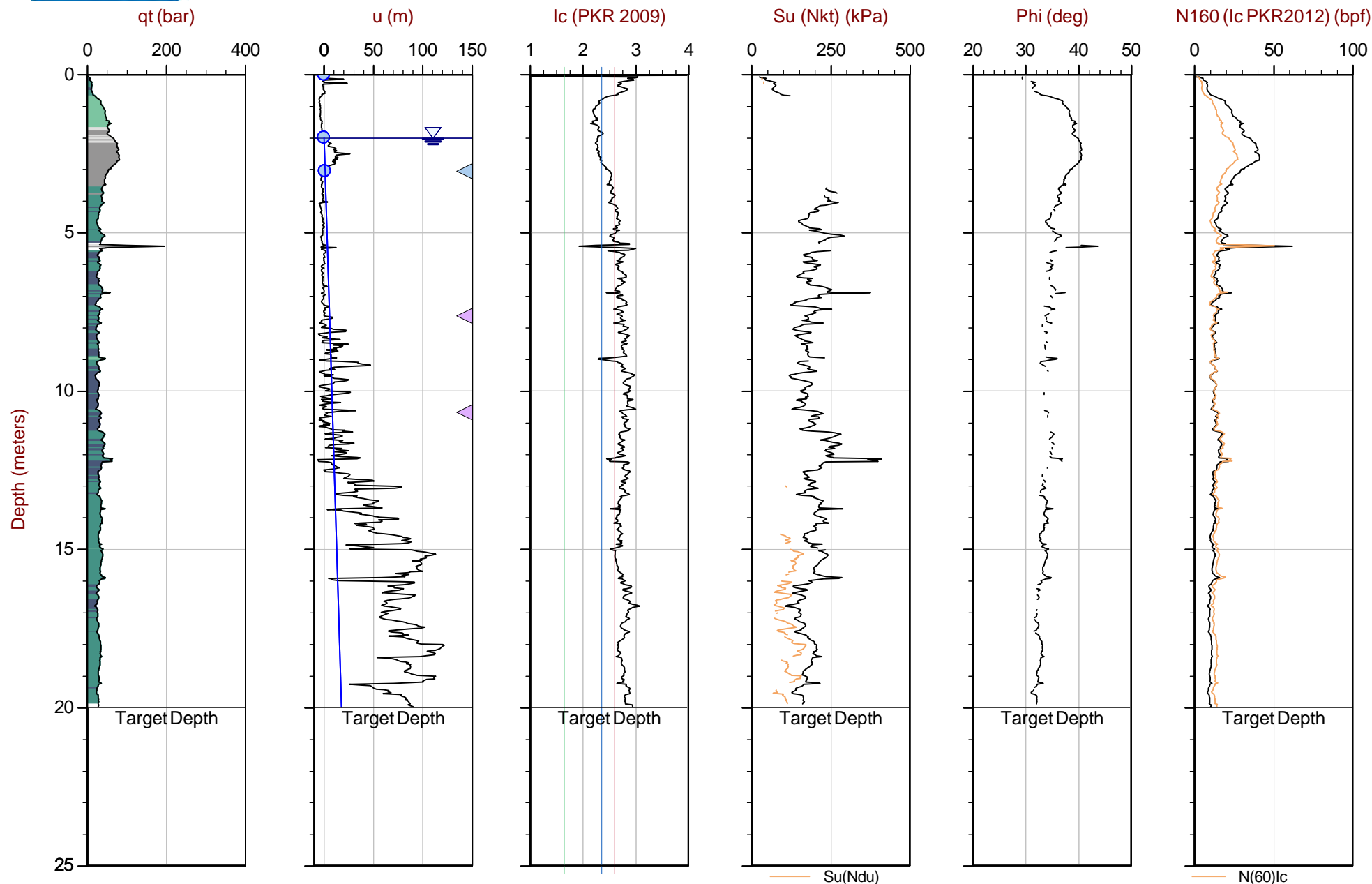
Overplot Item: ● Ueq ● Assumed Ueq ▲ Dissipation, Ueq achieved ▲ Dissipation, Ueq not achieved ▲ Dissipation, Ueq assumed — Hydrostatic Line

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



Site: HWY 3, St.Thomas, ON

Cone: 729:T1500F15U35 Area=15 cm²



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

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

Seismic Cone Penetration Test Plots



Stantec

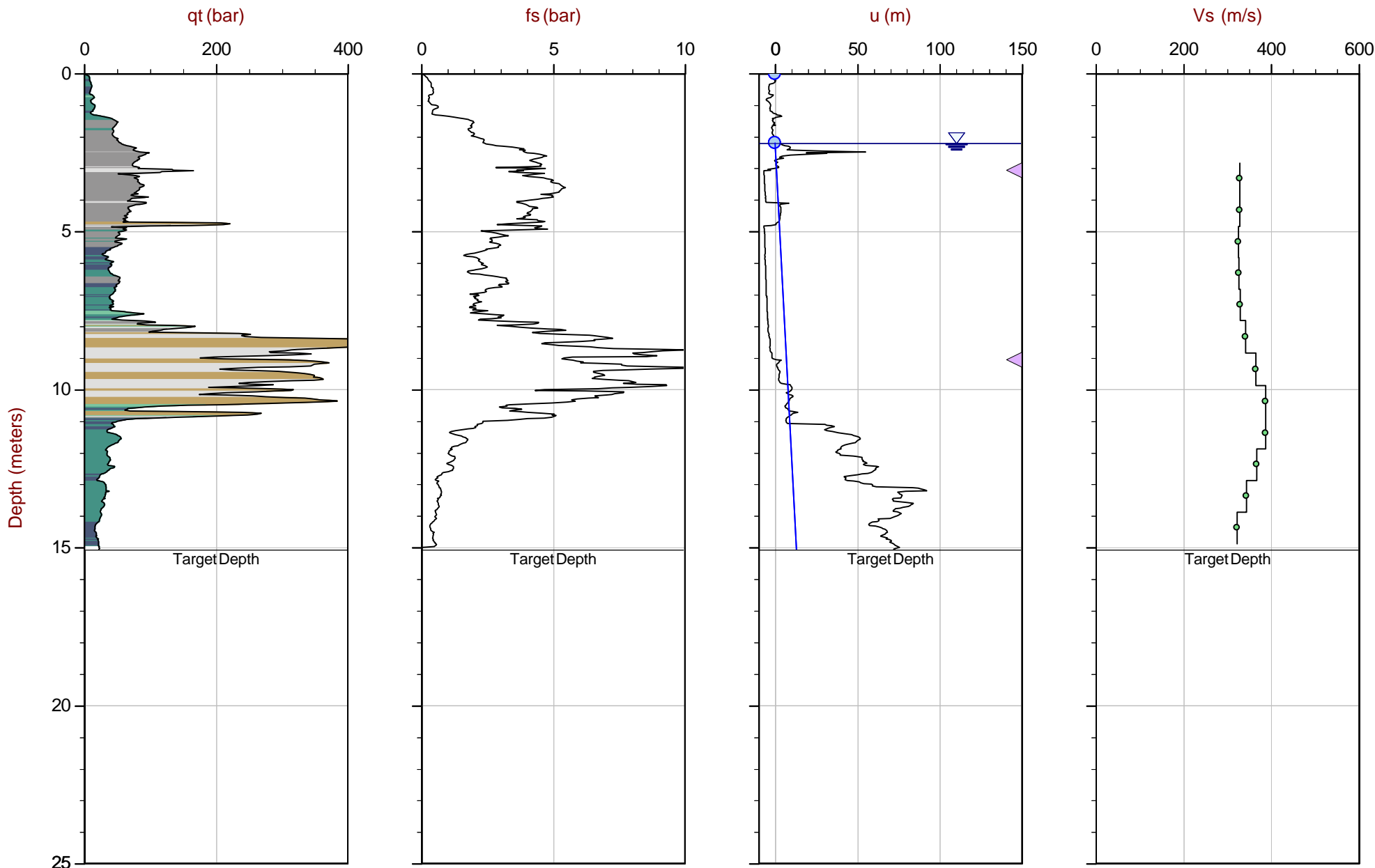
Job No: 24-05-27609

Date: 2024-05-10 10:24

Site: HWY 3, St.Thomas, ON

Sounding: SCPT24-RMNAPP01

Cone: 729:T1500F15U35 Area=15 cm²



Max Depth: 15.075 m / 49.46 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: Every Point

File: 24-05-27609_SP-RM-01.COR

Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010

Coords: UTM17N: 4739737mE: 481294m

Sheet No: 1 of 1

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

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



Stantec

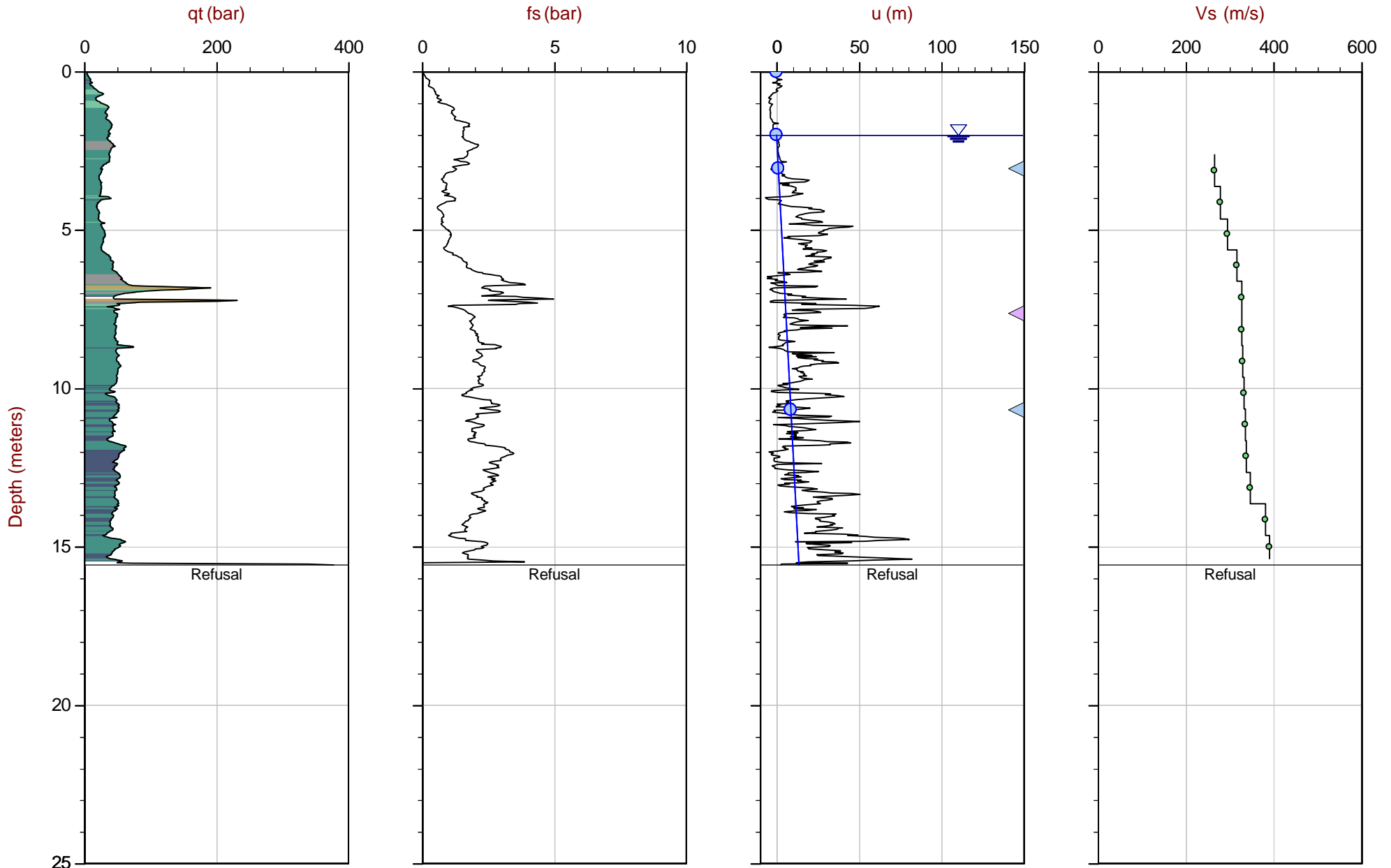
Job No: 24-05-27609

Date: 2024-05-09 12:06

Site: HWY 3, St.Thomas, ON

Sounding: SCPT24-CNREMB02

Cone: 729:T1500F15U35 Area=15 cm²



Max Depth: 15.575 m / 51.10 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: Every Point

File: 24-05-27609_SP-CN-02.COR

Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010

Coords: UTM17N N: 4740267m E: 480674m

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



Stantec

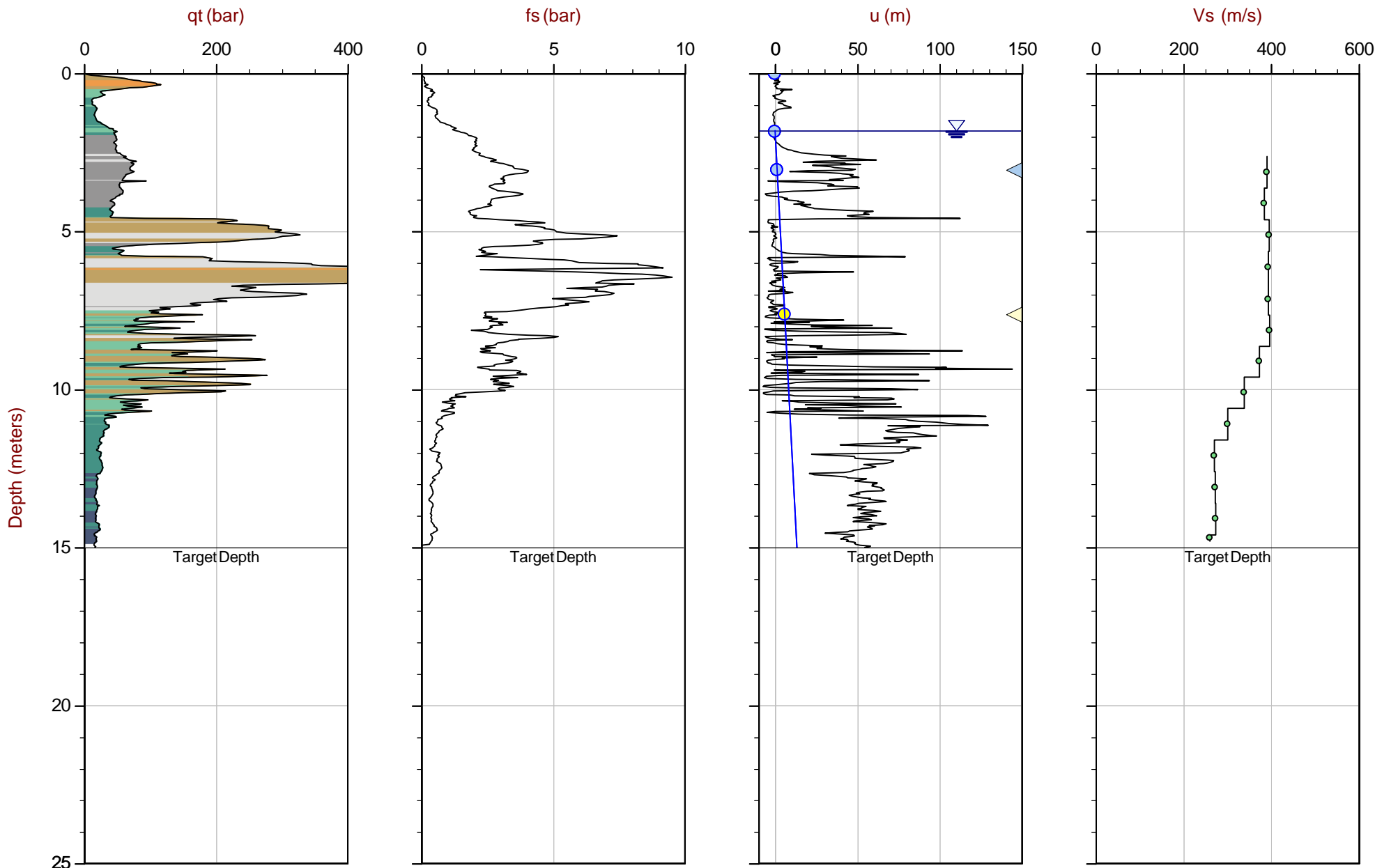
Job No: 24-05-27609

Date: 2024-05-10 14:49

Site: HWY 3, St.Thomas, ON

Sounding: SCPT24-WAPP02

Cone: 729:T1500F15U35 Area=15 cm²



Max Depth: 15.000 m / 49.21 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: Every Point

File: 24-05-27609_SP-WA-02.COR

Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010

Coords: UTM17N N: 4738905m E: 482817m

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



Stantec

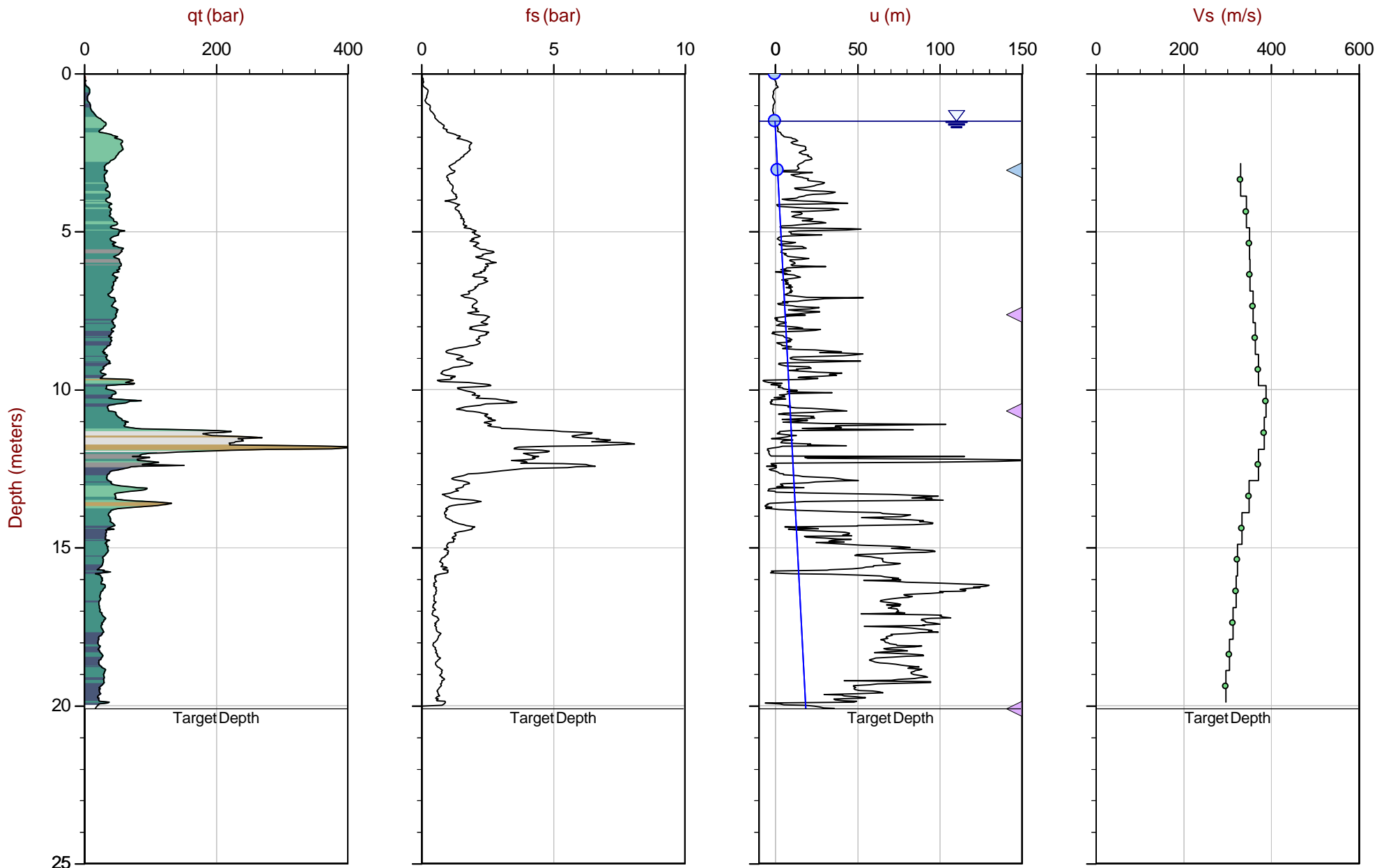
Job No: 24-05-27609

Date: 2024-05-09 16:43

Site: HWY 3, St.Thomas, ON

Sounding: SCPT24-CNREMB08

Cone: 729:T1500F15U35 Area=15 cm²



Max Depth: 20.100 m / 65.94 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: EveryPoint

File: 24-05-27609_SP-CN-08.COR

Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010

Coords: UTM17N: 4740056mE: 480896m

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



Stantec

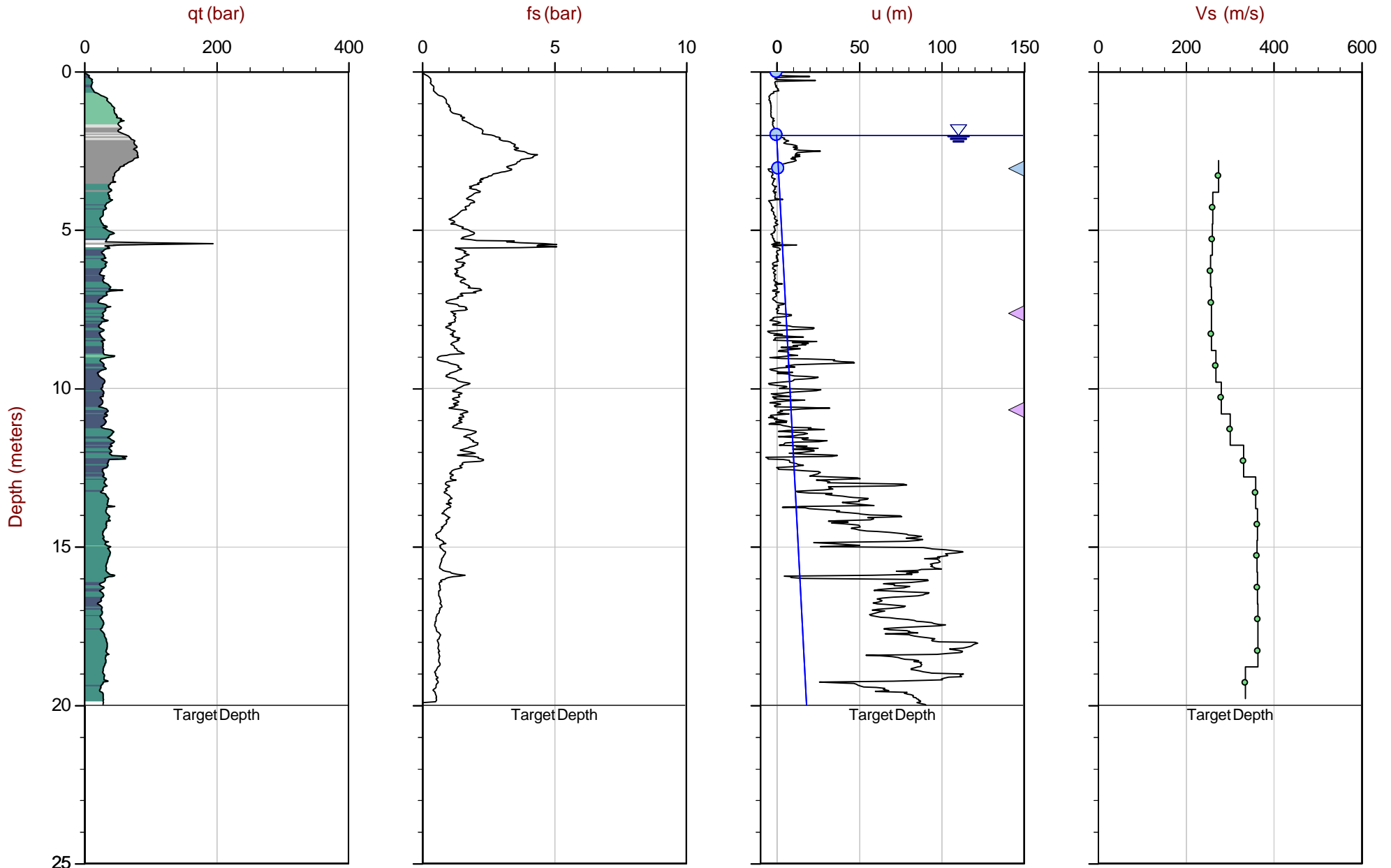
Job No: 24-05-27609

Date: 2024-05-10 06:58

Site: HWY 3, St.Thomas, ON

Sounding: SCPT24-CNREMB10

Cone: 729:T1500F15U35 Area=15 cm²



Max Depth: 20.000 m / 65.62 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: EveryPoint

File: 24-05-27609_SP-CN-10.COR

Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010

Coords: UTM17N: 4739960mE: 481010m

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

Seismic Cone Penetration Test Shear Wave (V_s) Tabular Results



Job No: 24-05-27609
Client: Stantec
Project: HWY 3 St Thomas CPT
Sounding ID: SCPT24-RMNAPP01
Date: 2024-05-10

Seismic Source: Beam
Seismic Offset (m): 3.20
Source Depth (m): 0.00
Geophone Offset (m): 0.20

SCPT_u SHEAR WAVE VELOCITY TEST RESULTS - V_s

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
3.03	2.83	4.27			
4.03	3.83	4.99	0.72	2.19	328
5.03	4.83	5.79	0.80	2.45	328
6.02	5.82	6.64	0.85	2.61	325
7.02	6.82	7.53	0.89	2.73	326
8.02	7.82	8.45	0.92	2.78	329
9.05	8.85	9.41	0.96	2.82	341
10.08	9.88	10.39	0.97	2.67	365
11.08	10.88	11.34	0.96	2.47	387
12.08	11.88	12.30	0.96	2.49	387
13.08	12.88	13.27	0.97	2.64	367
14.08	13.88	14.24	0.97	2.83	343
15.08	14.88	15.22	0.98	3.03	322



Job No: 24-05-27609
Client: Stantec
Project: HWY 3 St Thomas CPT
Sounding ID: SCPT24-CNREMB02
Date: 2024-05-09

Seismic Source: Beam
Seismic Offset (m): 3.20
Source Depth (m): 0.00
Geophone Offset (m): 0.20

SCPT_u 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)
2.82	2.62	4.14			
3.82	3.62	4.83	0.70	2.63	265
4.85	4.65	5.65	0.81	2.92	279
5.82	5.62	6.47	0.82	2.79	295
6.82	6.62	7.35	0.89	2.80	316
7.85	7.65	8.29	0.94	2.87	327
8.85	8.65	9.22	0.93	2.84	327
9.85	9.65	10.17	0.94	2.87	329
10.85	10.65	11.12	0.95	2.87	332
11.85	11.65	12.08	0.96	2.87	335
12.85	12.65	13.05	0.97	2.87	337
13.85	13.65	14.02	0.97	2.81	346
14.85	14.65	15.00	0.98	2.56	380
15.58	15.38	15.71	0.71	1.83	390



Job No: 24-05-27609
Client: Stantec
Project: HWY 3 St Thomas CPT
Sounding ID: SCPT24-WAPP02
Date: 2024-05-10

Seismic Source: Beam
Seismic Offset (m): 3.20
Source Depth (m): 0.00
Geophone Offset (m): 0.20

SCPT_u SHEAR WAVE VELOCITY TEST RESULTS - V_s

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
2.82	2.62	4.14			
3.82	3.62	4.83	0.70	1.79	390
4.82	4.62	5.62	0.79	2.05	384
5.82	5.62	6.47	0.85	2.14	396
6.85	6.65	7.38	0.91	2.32	393
7.85	7.65	8.29	0.91	2.32	393
8.83	8.63	9.20	0.91	2.30	396
9.80	9.60	10.12	0.92	2.46	373
10.80	10.60	11.07	0.95	2.82	338
11.80	11.60	12.03	0.96	3.20	300
12.80	12.60	13.00	0.97	3.58	270
13.80	13.60	13.97	0.97	3.57	272
14.80	14.60	14.95	0.98	3.58	273
15.00	14.80	15.14	0.20	0.75	260



Job No: 24-05-27609
Client: Stantec
Project: HWY 3 St Thomas CPT
Sounding ID: SCPT24-CNREMB08
Date: 2024-05-09

Seismic Source: Beam
Seismic Offset (m): 3.20
Source Depth (m): 0.00
Geophone Offset (m): 0.20

SCPT_u SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
3.05	2.85	4.29			
4.08	3.88	5.03	0.74	2.26	330
5.08	4.88	5.84	0.81	2.35	343
6.08	5.88	6.69	0.86	2.45	351
7.08	6.88	7.59	0.89	2.55	351
8.08	7.88	8.51	0.92	2.56	358
9.08	8.88	9.44	0.93	2.57	363
10.08	9.88	10.39	0.95	2.56	370
11.08	10.88	11.34	0.96	2.46	388
12.10	11.90	12.32	0.98	2.56	384
13.08	12.88	13.27	0.95	2.56	371
14.10	13.90	14.26	0.99	2.84	349
15.10	14.90	15.24	0.98	2.93	333
16.10	15.90	16.22	0.98	3.03	323
17.10	16.90	17.20	0.98	3.07	319
18.10	17.90	18.18	0.98	3.15	313
19.10	18.90	19.17	0.99	3.23	305
20.10	19.90	20.16	0.99	3.32	297



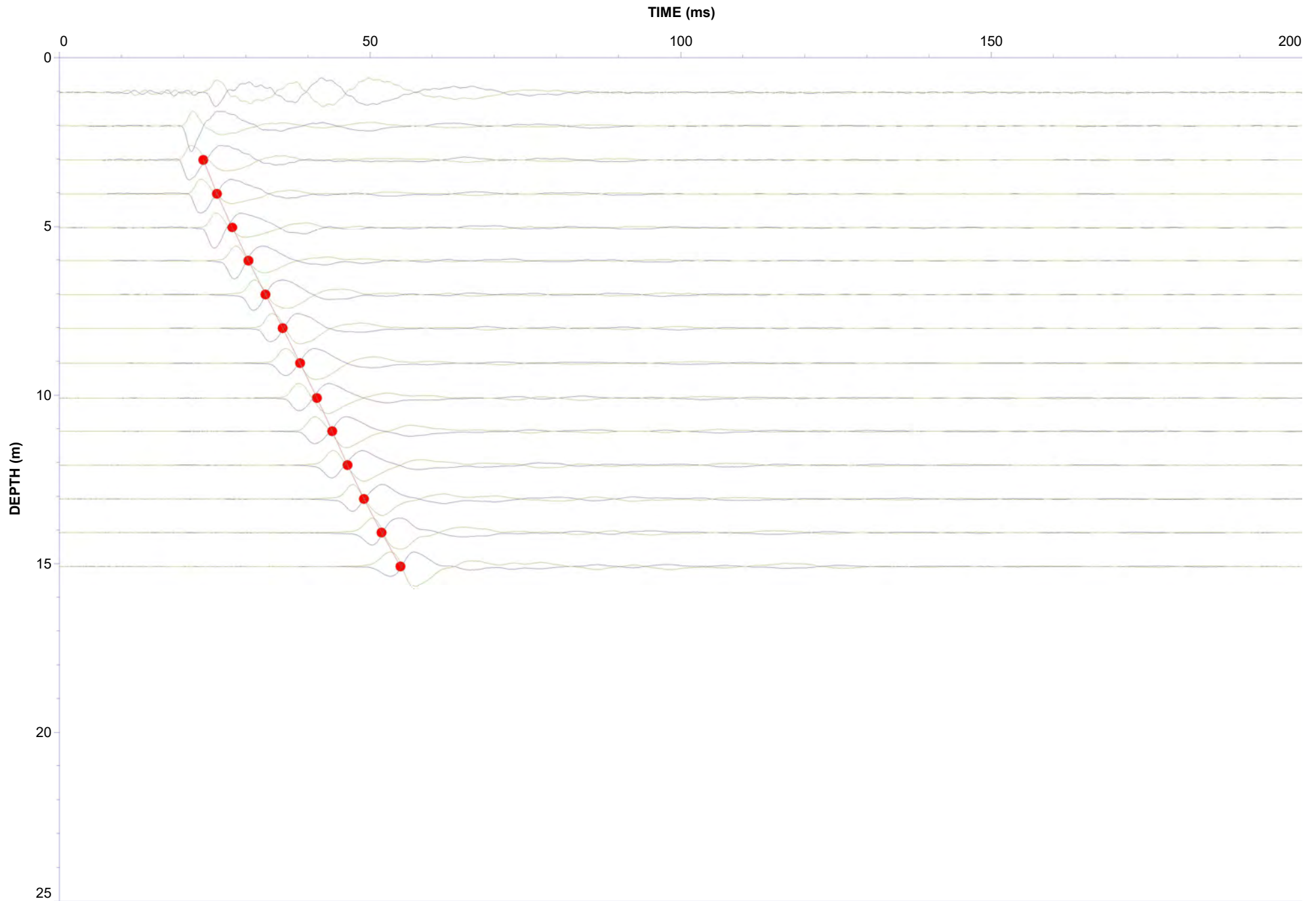
Job No: 24-05-27609
Client: Stantec
Project: HWY 3 St Thomas CPT
Sounding ID: SCPT24-CNREMB10
Date: 2024-05-10

Seismic Source: Beam
Seismic Offset (m): 3.20
Source Depth (m): 0.00
Geophone Offset (m): 0.20

SCPT_u SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
3.00	2.80	4.25			
4.00	3.80	4.97	0.72	2.61	275
5.00	4.80	5.77	0.80	3.07	261
6.00	5.80	6.62	0.86	3.29	260
7.00	6.80	7.52	0.89	3.48	256
8.00	7.80	8.43	0.92	3.55	258
9.00	8.80	9.36	0.93	3.61	258
10.00	9.80	10.31	0.95	3.53	268
11.00	10.80	11.26	0.96	3.41	280
12.00	11.80	12.23	0.96	3.20	301
13.00	12.80	13.19	0.97	2.92	332
14.00	13.80	14.17	0.97	2.71	359
15.00	14.80	15.14	0.98	2.70	362
16.00	15.80	16.12	0.98	2.71	362
17.00	16.80	17.10	0.98	2.71	363
18.00	17.80	18.09	0.98	2.71	363
19.00	18.80	19.07	0.99	2.71	364
20.00	19.80	20.06	0.99	2.95	335

Seismic Cone Penetration Test Shear Wave (V_s) Traces





Job No: 24-05-27609
Analysis: S Wave - Geo X

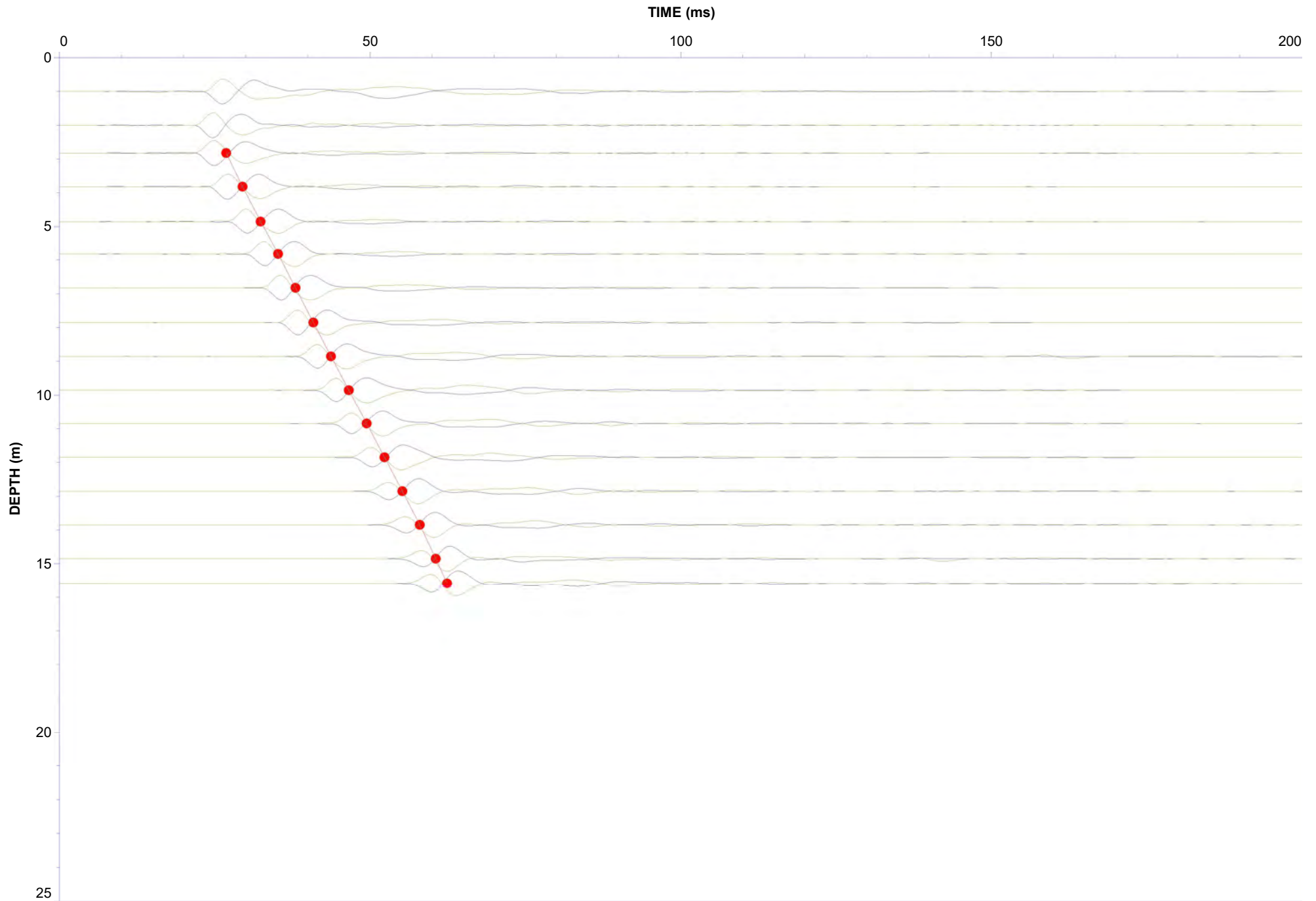
Client: Stantec

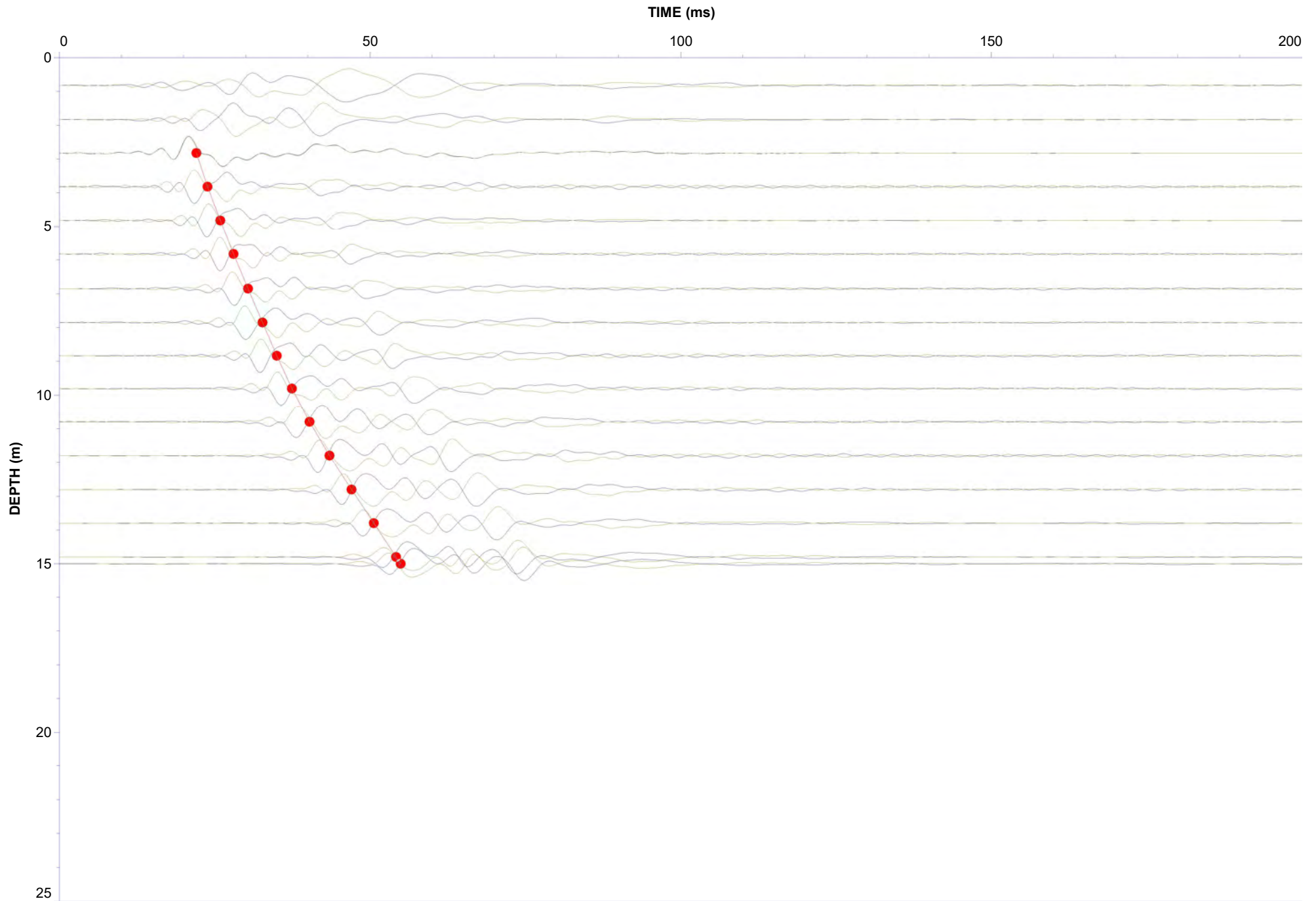
Project: HWY 3 St Thomas CPT

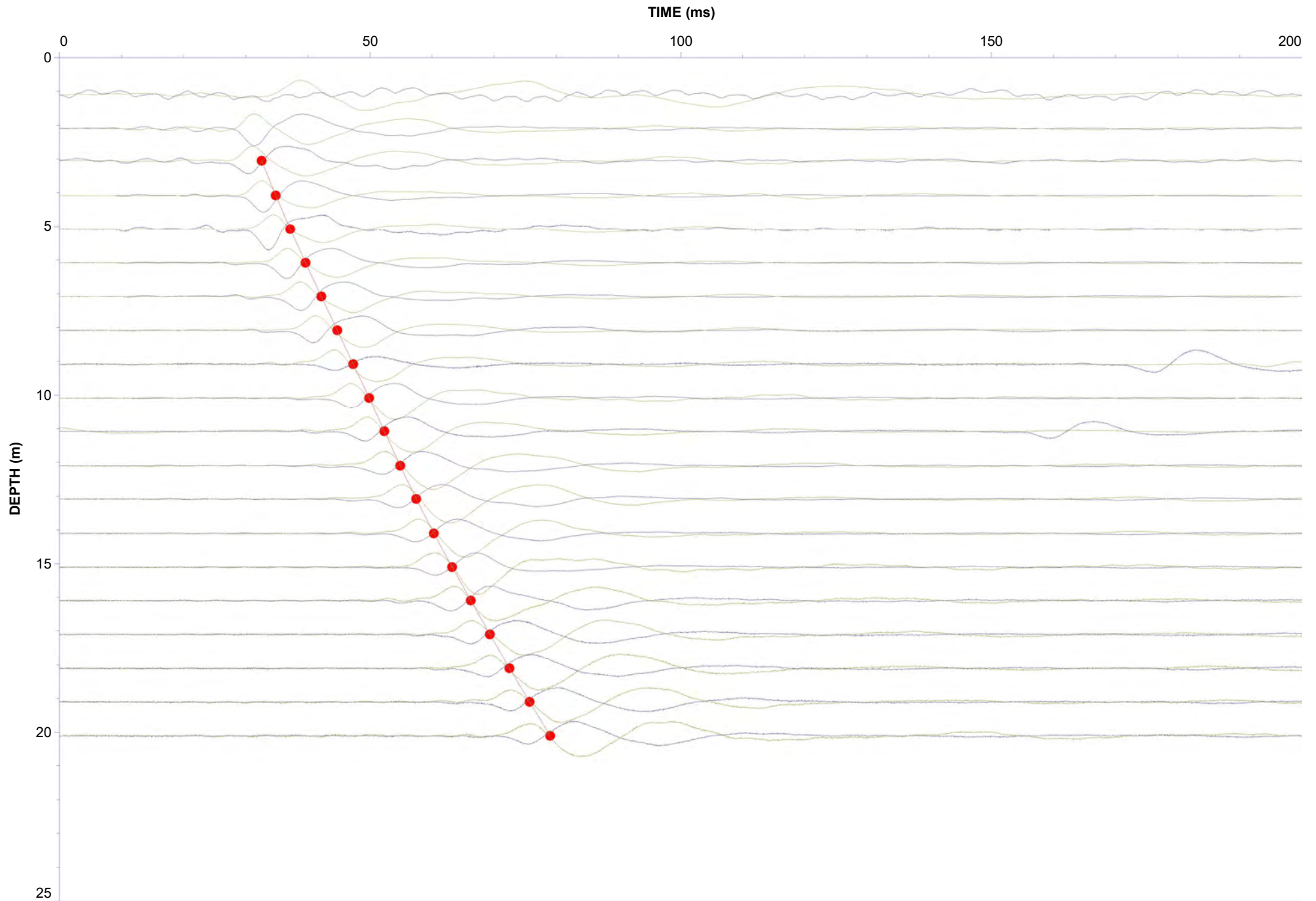
Sounding: SCPT24-CNREMB02

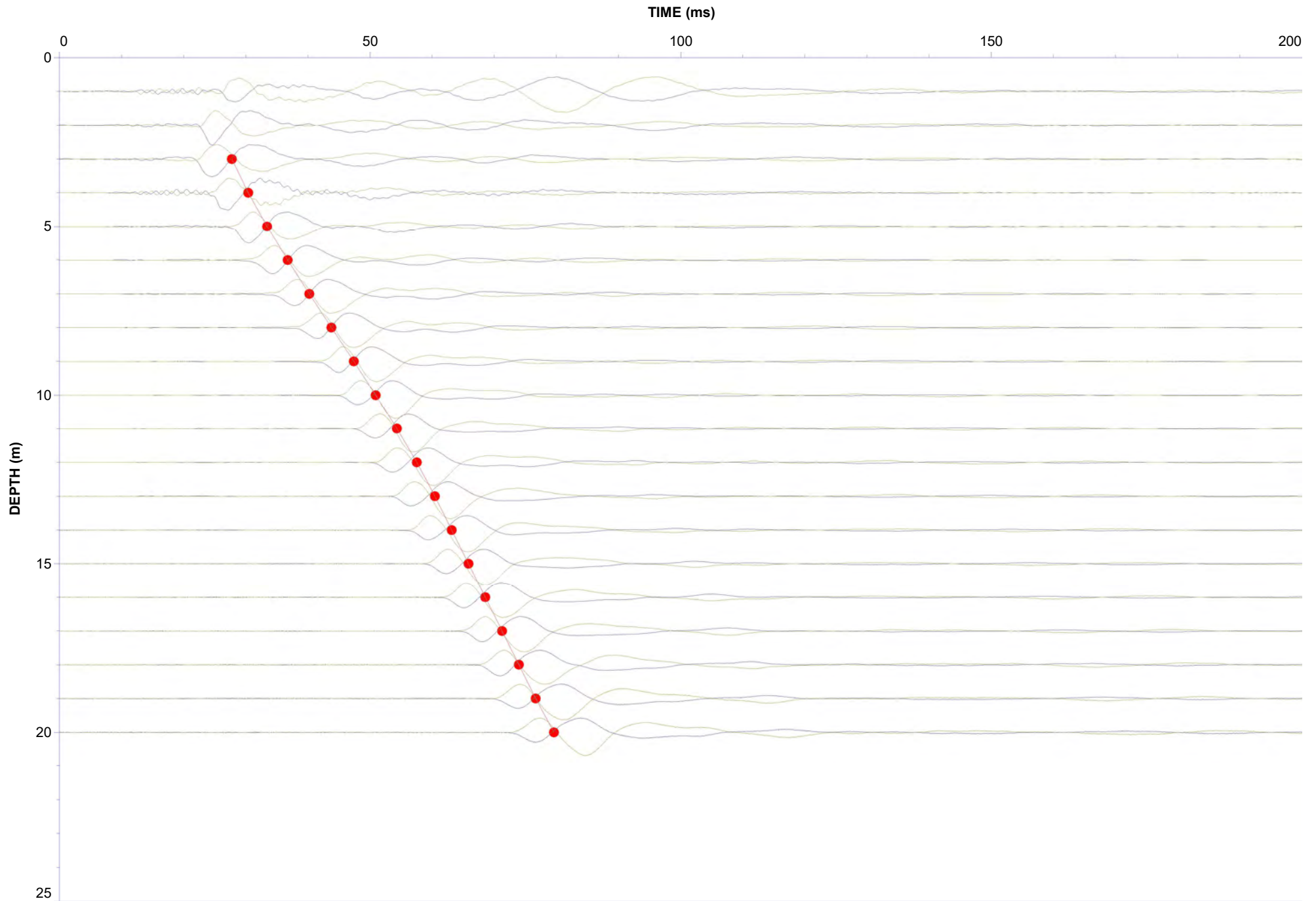
Filter: None

Date: 2024-05-09









Soil Behaviour Type (SBT) Scatter Plots



Stantec

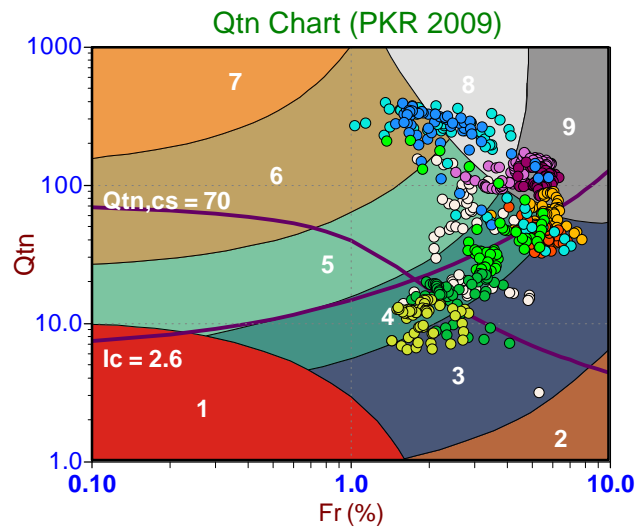
Job No: 24-05-27609

Date: 2024-05-10 12:15

Site: HWY 3, St.Thomas, ON

Sounding: CPT24-RMNAPP01

Cone: 729:T1500F15U35 Area=15 cm²

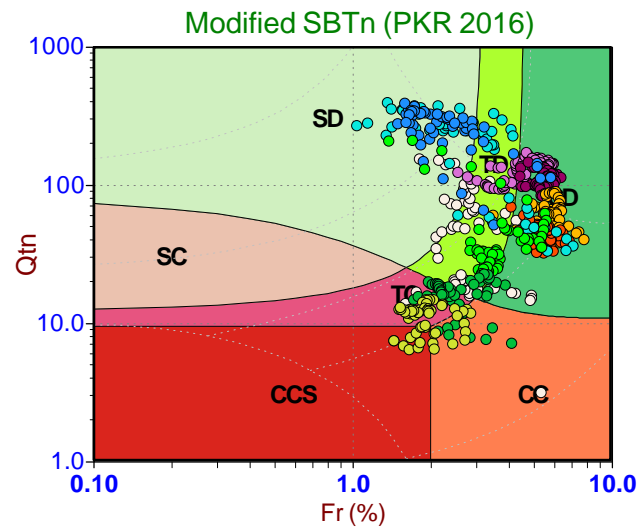


Depth Ranges

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

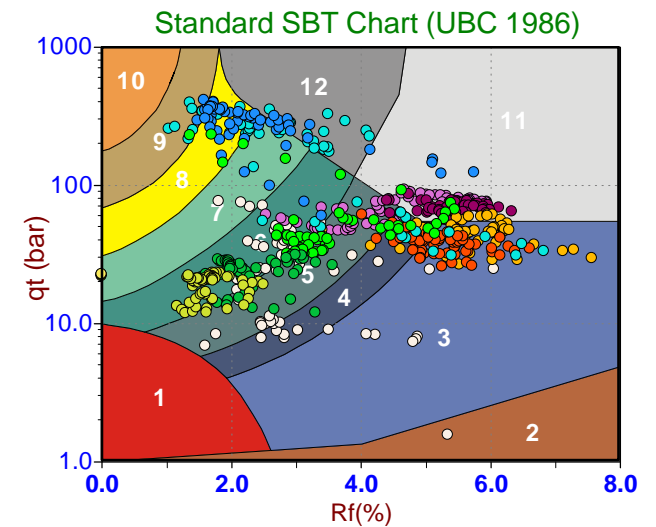
Legend

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



Legend

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



Legend

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



Stantec

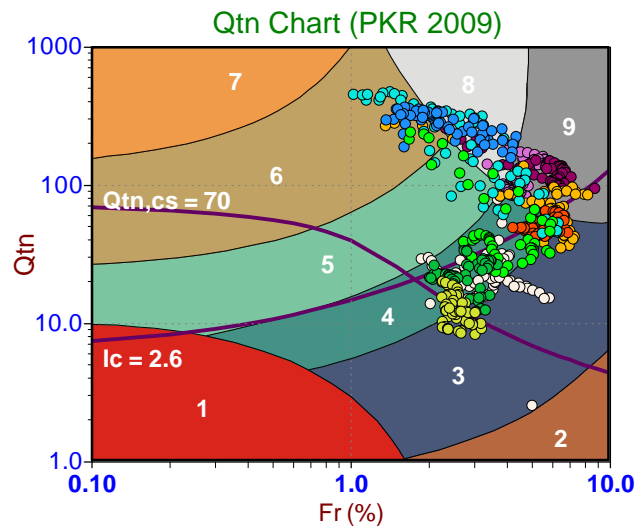
Job No: 24-05-27609

Date: 2024-05-10 10:24

Site: HWY 3, St.Thomas, ON

Sounding: SCPT24-RMNAPP01

Cone: 729:T1500F15U35 Area=15 cm²

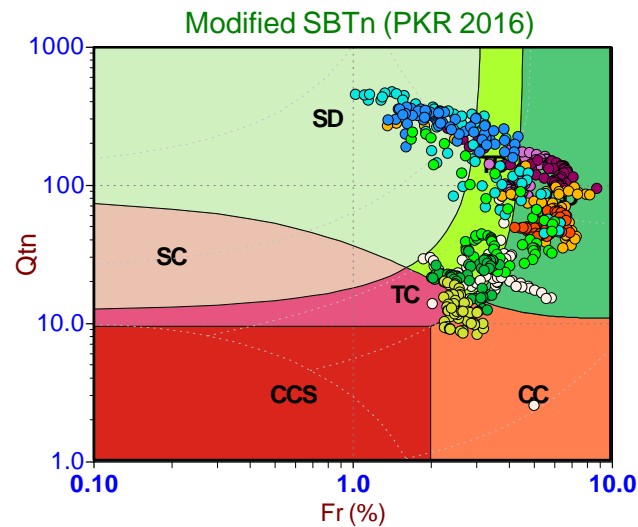


Depth Ranges

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

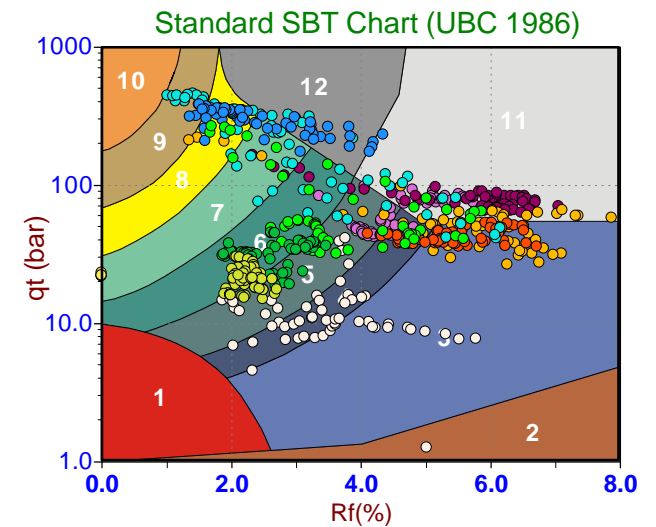
Legend

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



Legend

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



Legend

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



Stantec

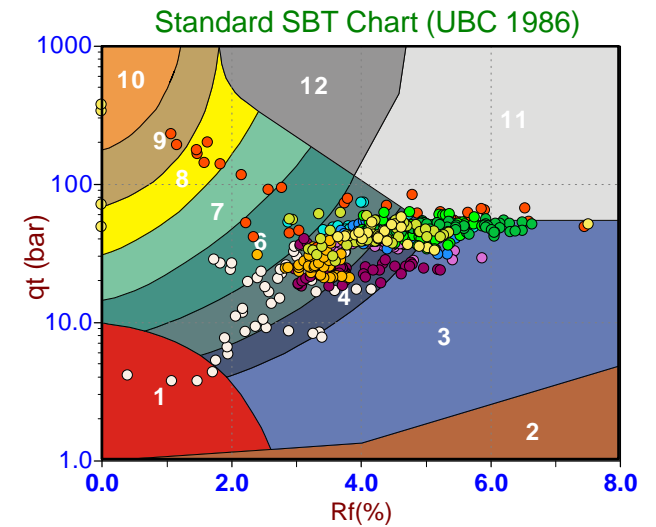
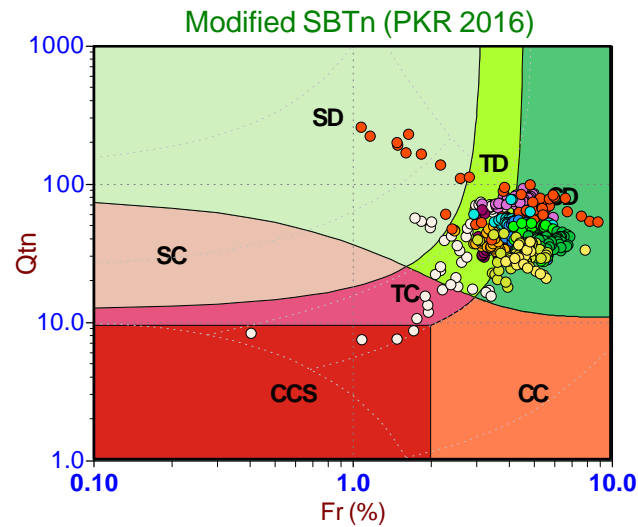
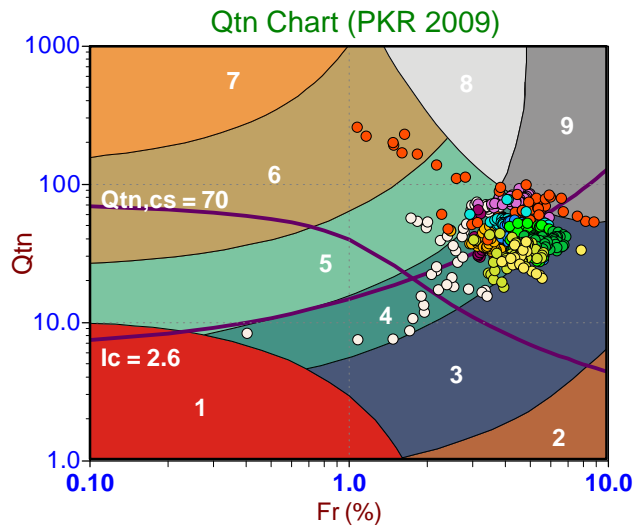
Job No: 24-05-27609

Date: 2024-05-09 12:06

Site: HWY 3, St.Thomas, ON

Sounding: SCPT24-CNREMB02

Cone: 729:T1500F15U35 Area=15 cm²



Depth Ranges

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

Legend

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

Legend

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

Legend

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



Stantec

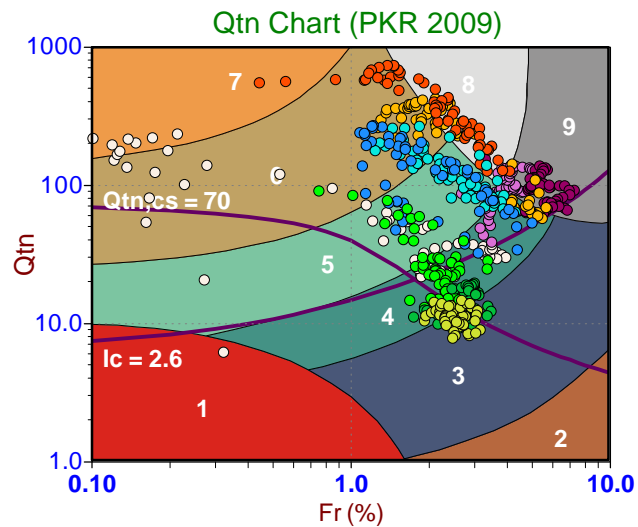
Job No: 24-05-27609

Date: 2024-05-10 14:49

Site: HWY 3, St.Thomas, ON

Sounding: SCPT24-WAPP02

Cone: 729:T1500F15U35 Area=15 cm²

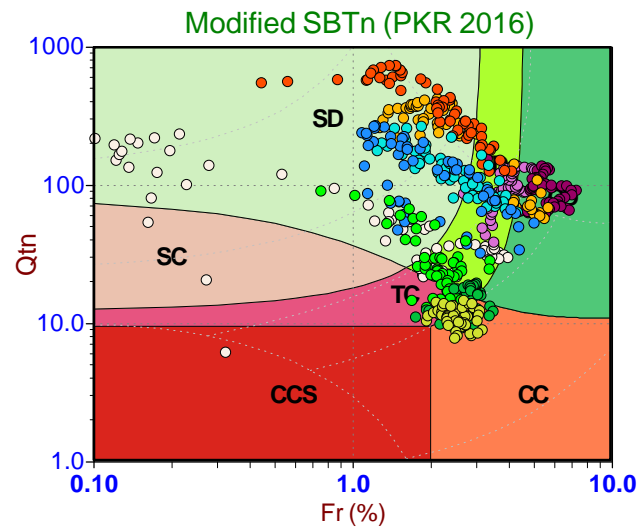


Depth Ranges

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

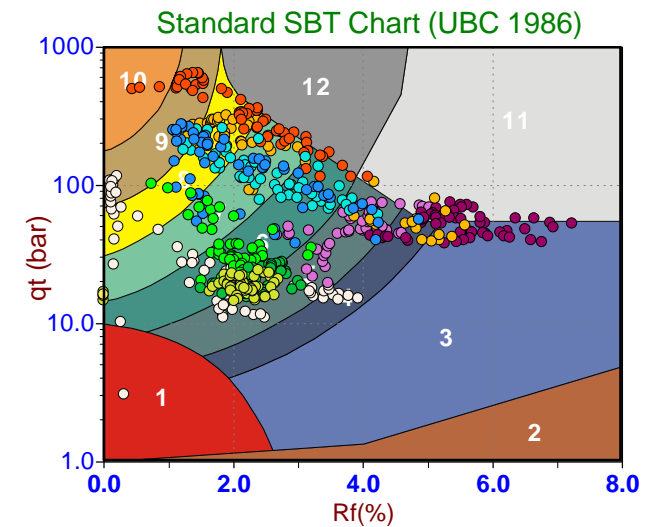
Legend

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



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- 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



Stantec

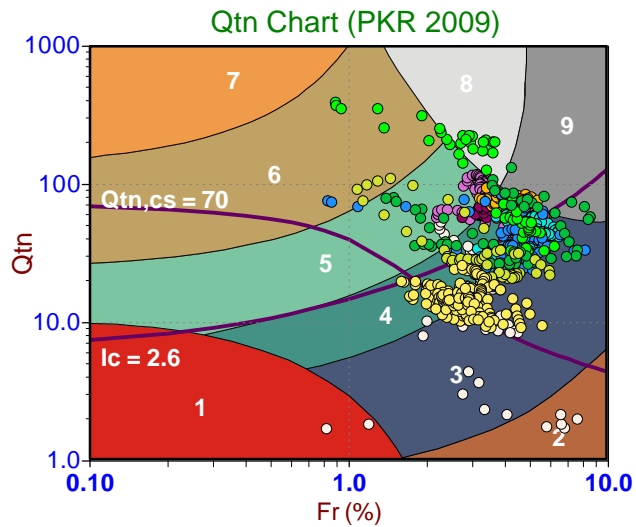
Job No: 24-05-27609

Date: 2024-05-09 16:43

Site: HWY 3, St.Thomas, ON

Sounding: SCPT24-CNREMB08

Cone: 729:T1500F15U35 Area=15 cm²

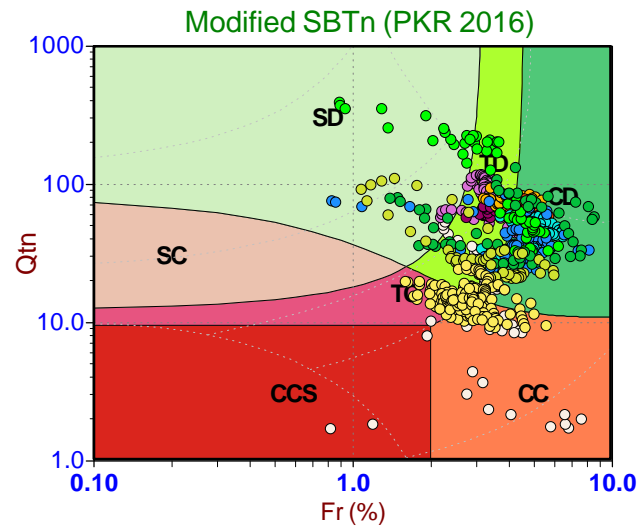


Depth Ranges

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

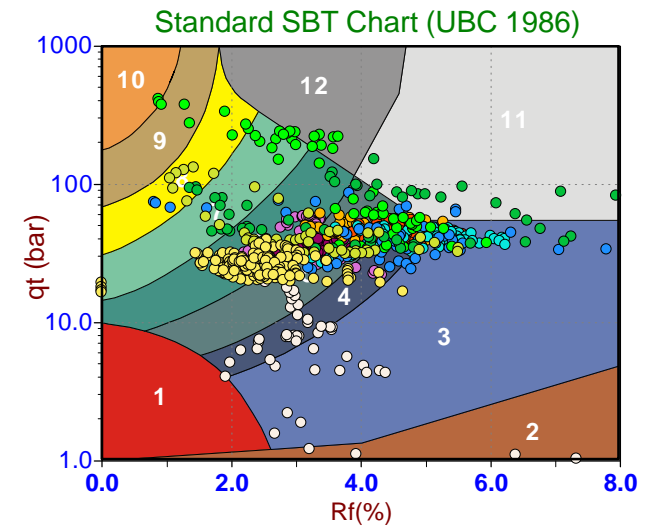
Legend

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



Legend

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



Legend

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



Stantec

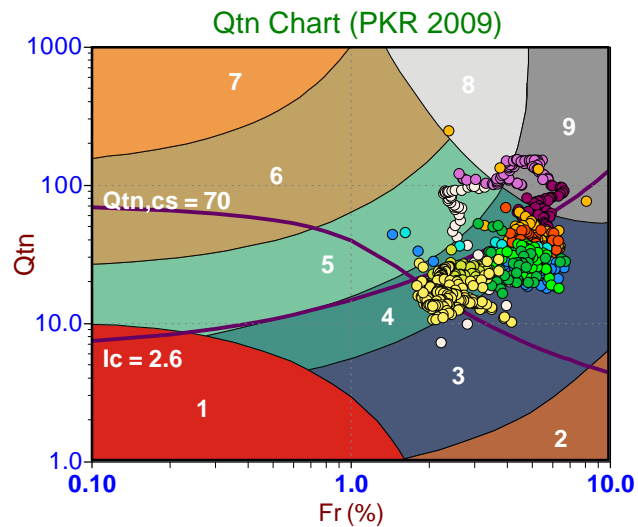
Job No: 24-05-27609

Date: 2024-05-10 06:58

Site: HWY 3, St.Thomas, ON

Sounding: SCPT24-CNREMB10

Cone: 729:T1500F15U35 Area=15 cm²

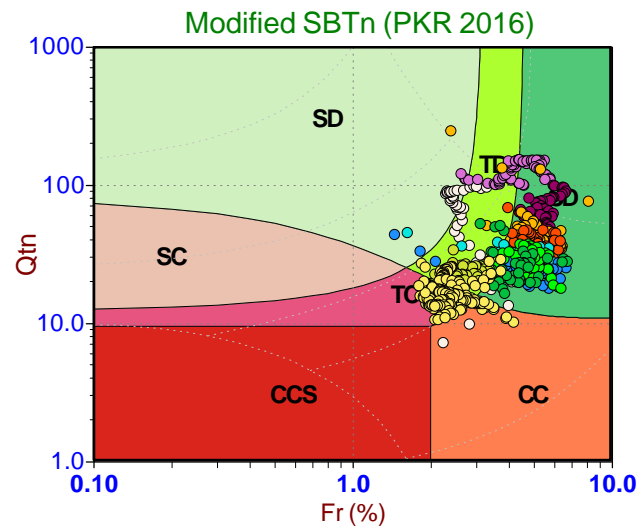


Depth Ranges

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

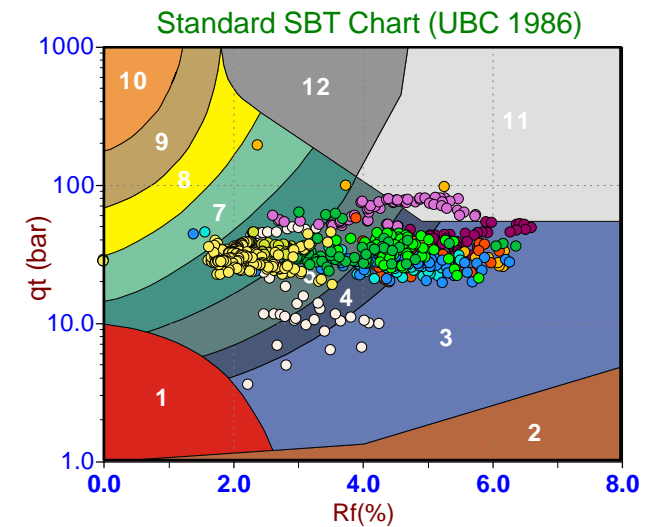
Legend

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



Legend

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



Legend

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

Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots



Job No: 24-05-27609
Client: Stantec Consulting Ltd.
Project: HWY 3 St Thomas CPT
Start Date: 2024-05-09
End Date: 2024-05-10

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)	Observed Equilibrium Pore Pressure U _{eq} (m)	Estimated Equilibrium Pore Pressure U _{eq} (m)	Assumed Phreatic Surface (m)	Percent Dissipation (%)	t ₅₀ (s) ₁	Assumed Rigidity Index (I _r)	c _h (cm ² /min) ₂	Refer to Notation Number
CPT24-RMNAPP01	24-05-27609_CP-RM-01	15	670	3.050	0.9	9.2	-2.2	9.2								
CPT24-RMNAPP01	24-05-27609_CP-RM-01	15	3090	7.625	0.4	44.6	0.4	26.4								
SCPT24-RMNAPP01	24-05-27609_SP-RM-01	15	790	3.050	1.4	1.4	-8.7	-7.3								
SCPT24-RMNAPP01	24-05-27609_SP-RM-01	15	1150	9.050	1.4	2.4	0.9	2.4								
SCPT24-CNREMB02	24-05-27609_SP-CN-02	15	3330	3.050	1.1	38.2	1.0	19.4		1.1	2.0	51	2923	100	0.2	3
SCPT24-CNREMB02	24-05-27609_SP-CN-02	15	1090	7.625	13.8	42.9	13.8	38.9								
SCPT24-CNREMB02	24-05-27609_SP-CN-02	15	2940	10.675	8.2	61.5	7.9	34.0		8.7	2.0	52	2423	100	0.3	3
SCPT24-WAPP02	24-05-27609_SP-WA-02	15	2010	3.050	31.5	99.6	31.5	49.9		1.2	1.8	51	1786	100	0.4	3
SCPT24-WAPP02	24-05-27609_SP-WA-02	15	1100	7.625	13.1	13.1	1.1	5.8	5.8		1.8	100				
SCPT24-CNREMB08	24-05-27609_SP-CN-08	15	3090	3.050	11.8	51.9	8.7	26.5		1.6	1.5	50	2773	100	0.3	3
SCPT24-CNREMB08	24-05-27609_SP-CN-08	15	1550	7.625	9.2	67.2	6.7	54.3								
SCPT24-CNREMB08	24-05-27609_SP-CN-08	15	2370	10.675	47.5	96.8	46.6	53.4								
SCPT24-CNREMB08	24-05-27609_SP-CN-08	15	470	20.100	40.1	70.1	40.1	66.8								
SCPT24-CNREMB10	24-05-27609_SP-CN-10	15	3030	3.050	-0.6	17.5	-0.6	9.2		1.1	2.0	50	2880	100	0.2	3
SCPT24-CNREMB10	24-05-27609_SP-CN-10	15	740	7.625	-0.6	30.1	-0.6	30.0								
SCPT24-CNREMB10	24-05-27609_SP-CN-10	15	3450	10.675	10.0	44.0	10.0	32.8								

1. Time for 50 percent dissipation was based on U_{max}, U_{min}, and the applied U_{eq}. Note the time is relative to where U_{max} occurred.

2. Teh and Houlsby, 1991.

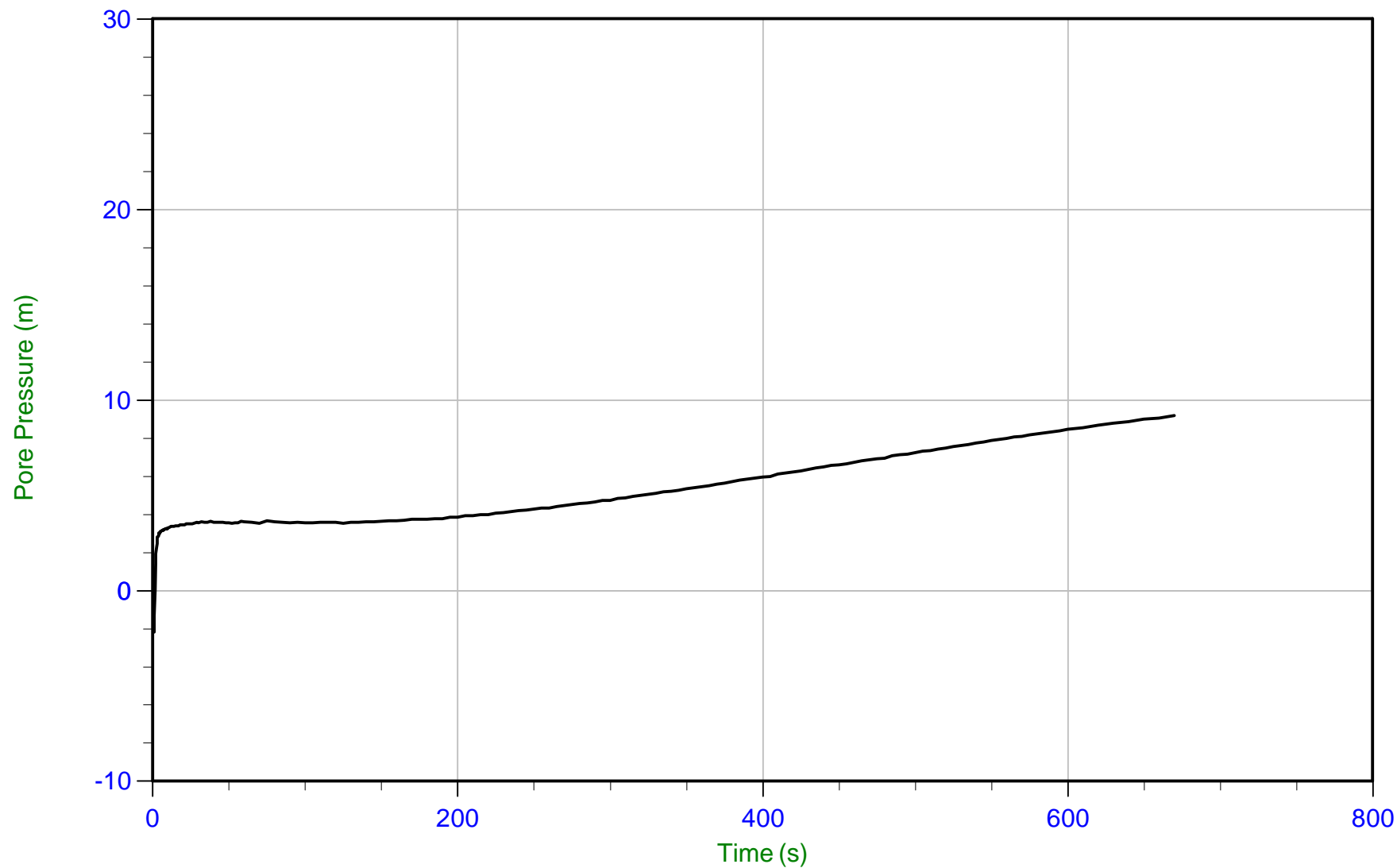
3. The estimated equilibrium pore pressure was based on a hydrostatic assumption from the assumed phreatic surface.



Stantec

Job No: 24-05-27609
Date: 2024-05-10 12:15
Site: HWY 3, St.Thomas, ON

Sounding: CPT24-RMNAPP01
Cone: 729:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-05-27609_CP-RM-01.PPF2
Depth: 3.050 m / 10.006 ft
Duration: 670.0 s

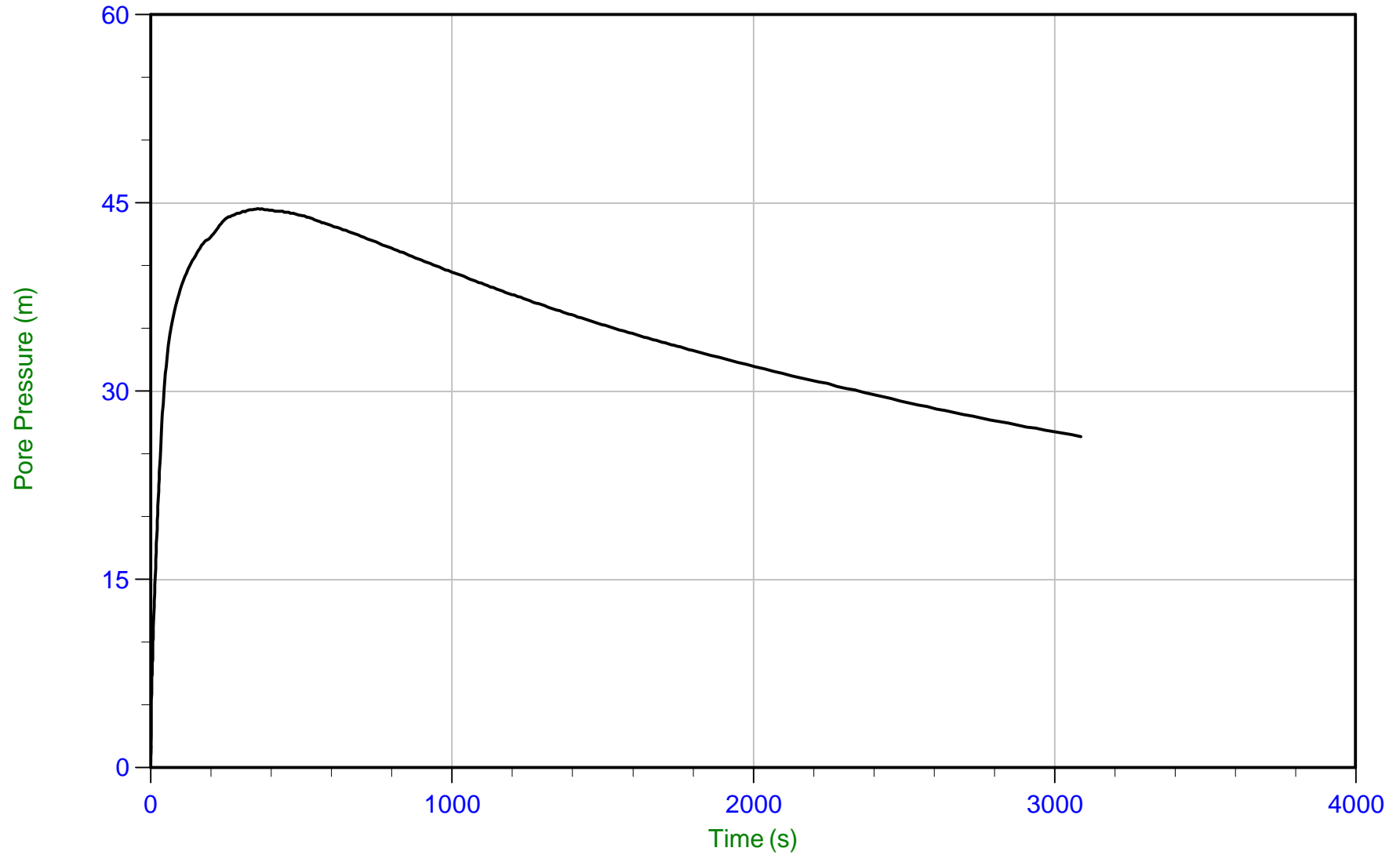
u Min: -2.2 m
u Max: 9.2 m
u Final: 9.2 m



Stantec

Job No: 24-05-27609
Date: 2024-05-10 12:15
Site: HWY 3, St.Thomas, ON

Sounding: CPT24-RMNAPP01
Cone: 729:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-05-27609_CP-RM-01.PPF2
Depth: 7.625 m / 25.016 ft
Duration: 3090.0 s

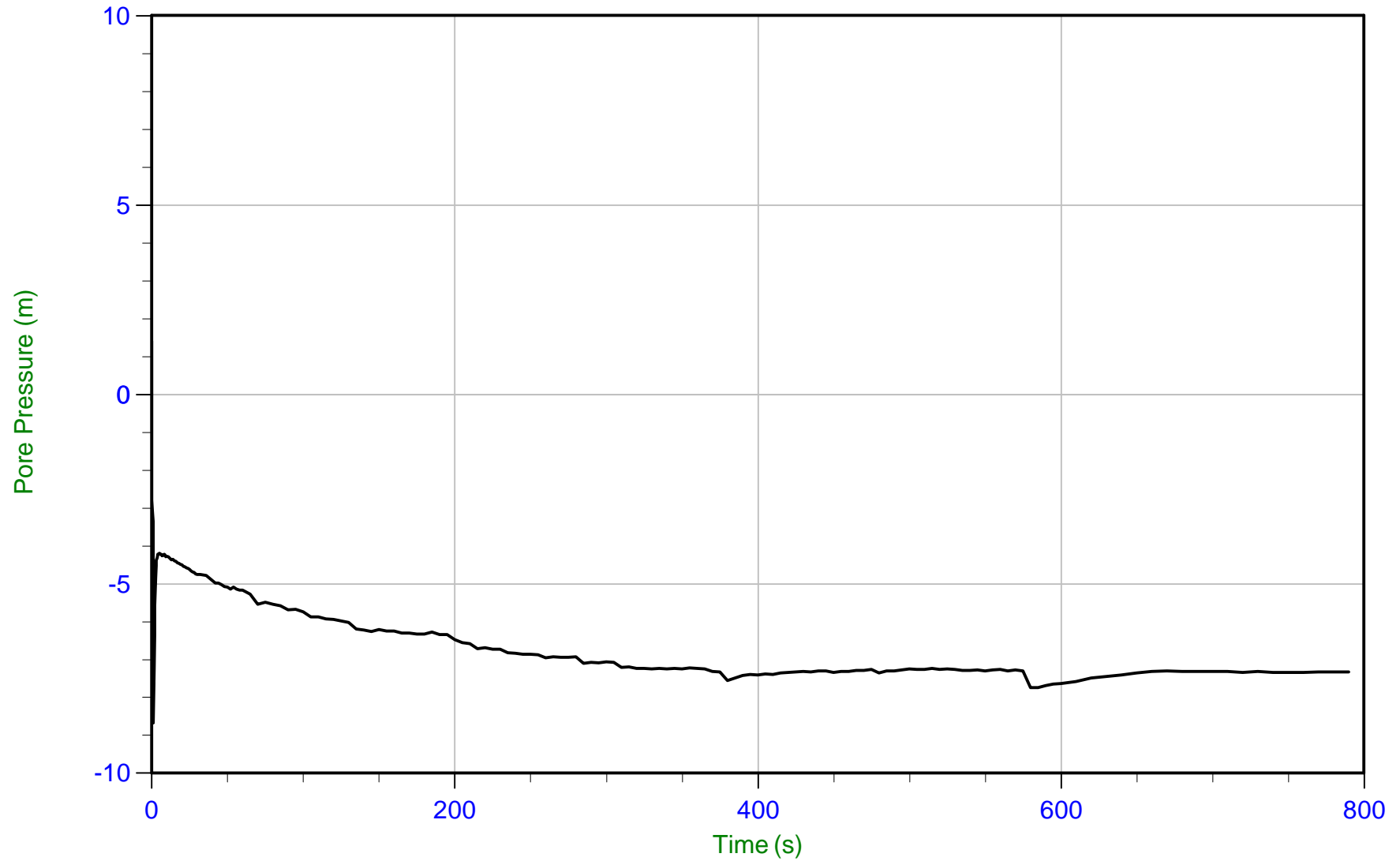
u Min: 0.4 m
u Max: 44.6 m
u Final: 26.4 m



Stantec

Job No: 24-05-27609
Date: 2024-05-10 10:24
Site: HWY 3, St.Thomas, ON

Sounding: SCPT24-RMNAPP01
Cone: 729:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-05-27609_SP-RM-01.PPF2
Depth: 3.050 m / 10.006 ft
Duration: 790.0 s

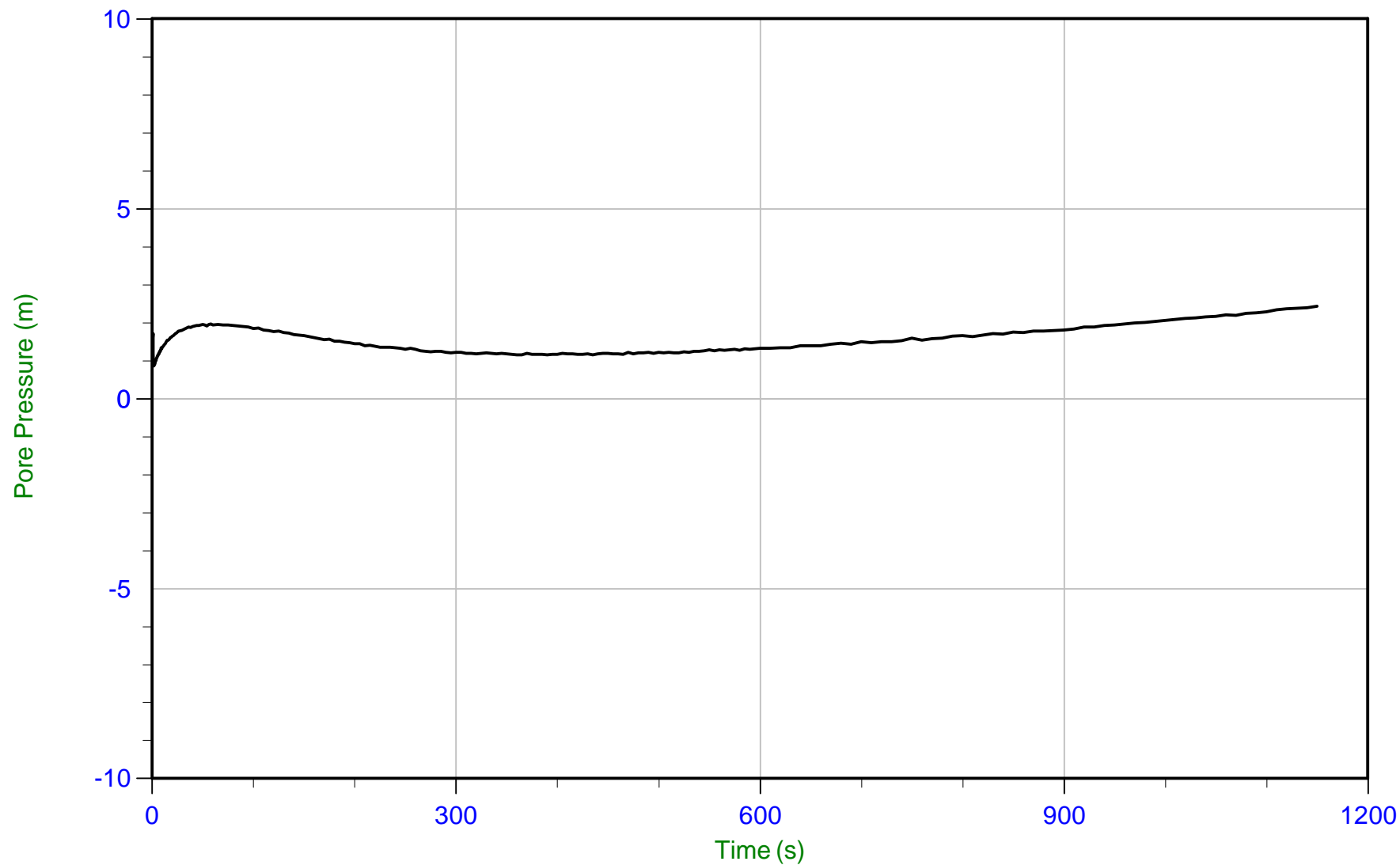
u Min: -8.7 m
u Max: 1.4 m
u Final: -7.3 m



Stantec

Job No: 24-05-27609
Date: 2024-05-10 10:24
Site: HWY 3, St.Thomas, ON

Sounding: SCPT24-RMNAPP01
Cone: 729:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-05-27609_SP-RM-01.PPF2
Depth: 9.050 m / 29.691 ft
Duration: 1150.0 s

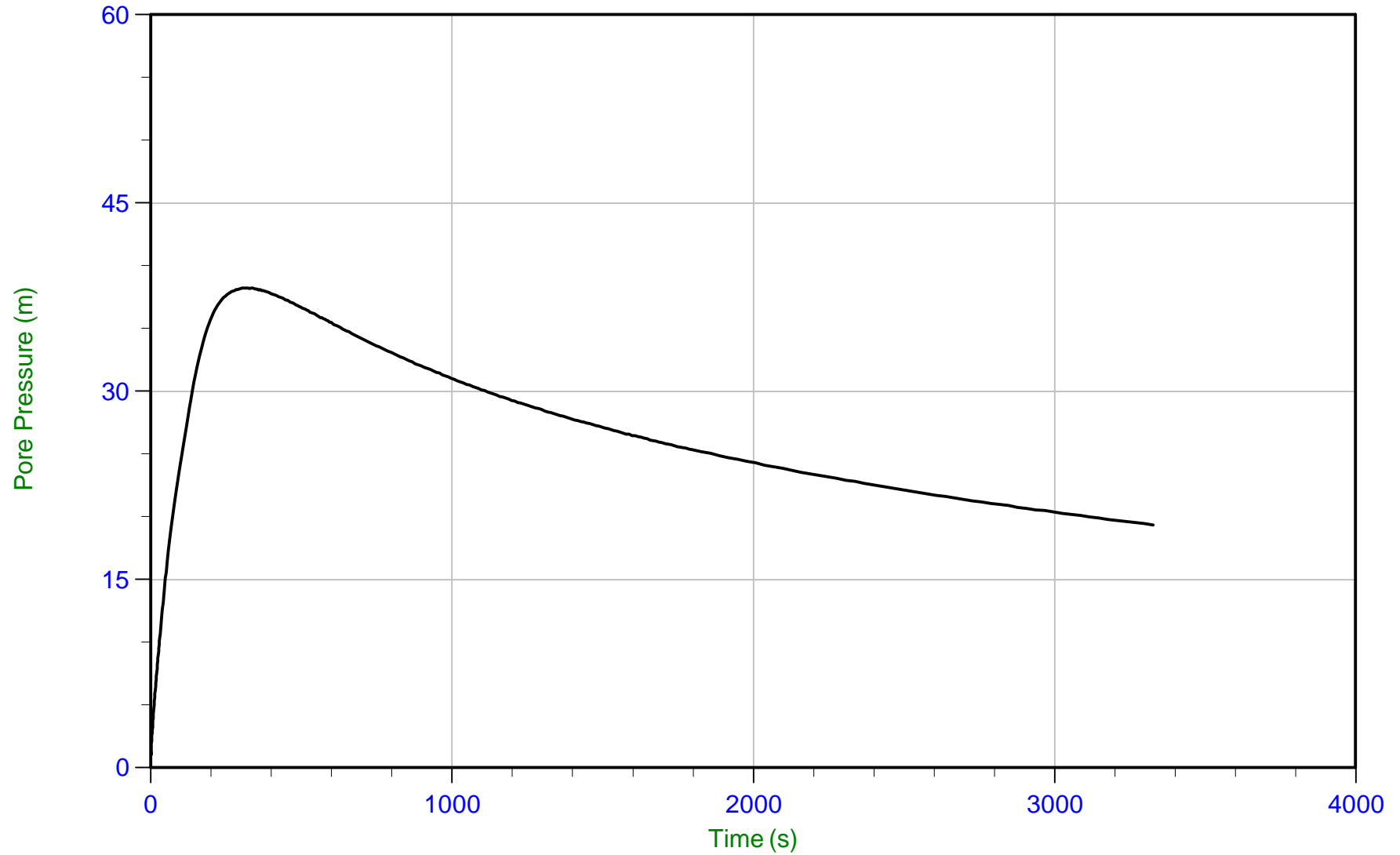
u Min: 0.9 m
u Max: 2.4 m
u Final: 2.4 m



Stantec

Job No: 24-05-27609
Date: 2024-05-09 12:06
Site: HWY 3, St.Thomas, ON

Sounding: SCPT24-CNREMB02
Cone: 729:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-05-27609_SP-CN-02.PPF2
Depth: 3.050 m / 10.006 ft
Duration: 3330.0 s

u Min: 1.0 m
u Max: 38.2 m
u Final: 19.4 m

WT: 2.0 m / 6.6 ft
Ueq: 1.1 m
U(50): 19.64 m

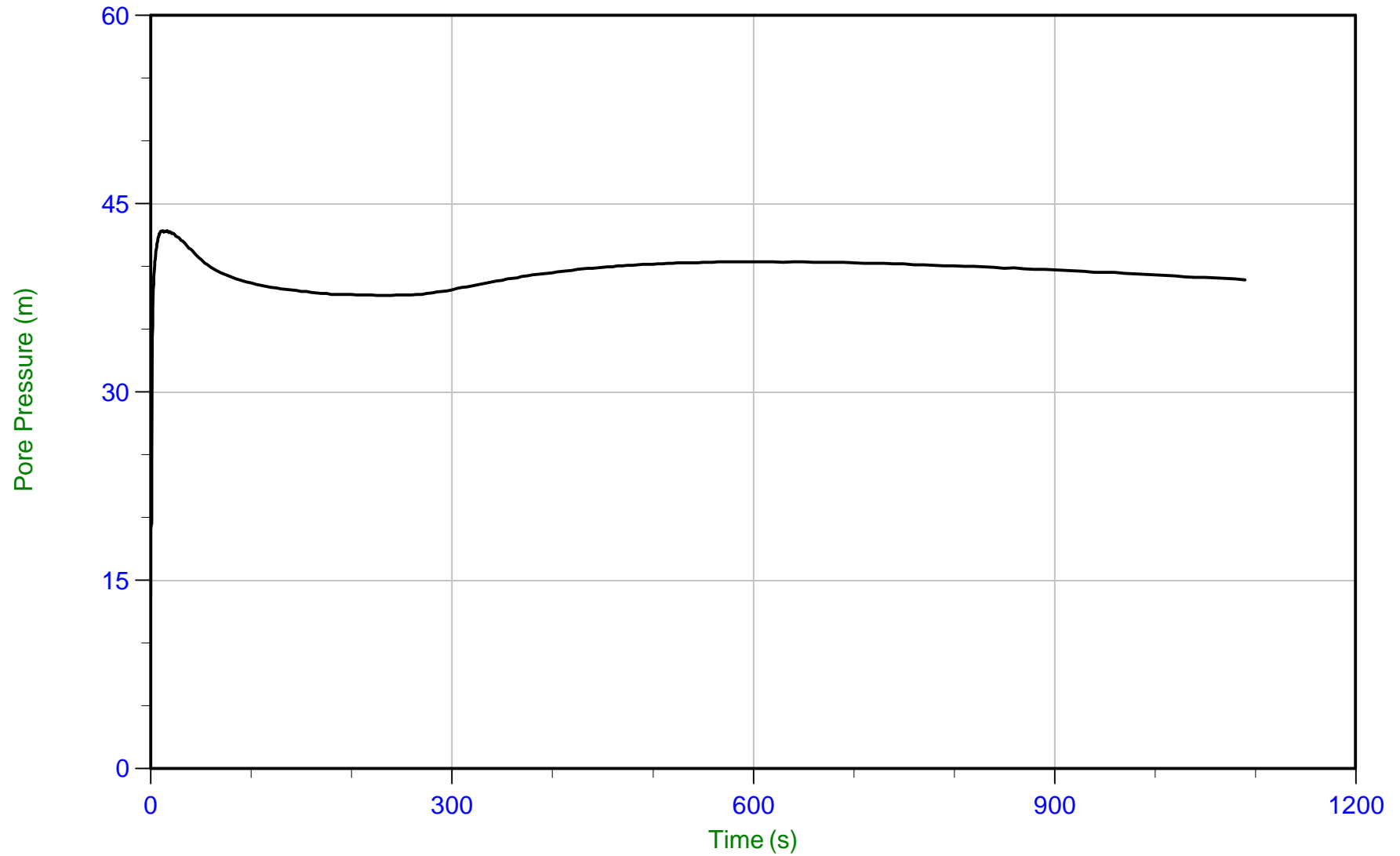
T(50): 2923.2 s
Ir: 100
Ch: 0.2 cm²/min



Stantec

Job No: 24-05-27609
Date: 2024-05-09 12:06
Site: HWY 3, St.Thomas, ON

Sounding: SCPT24-CNREMB02
Cone: 729:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-05-27609_SP-CN-02.PPF2
Depth: 7.625 m / 25.016 ft
Duration: 1090.0 s

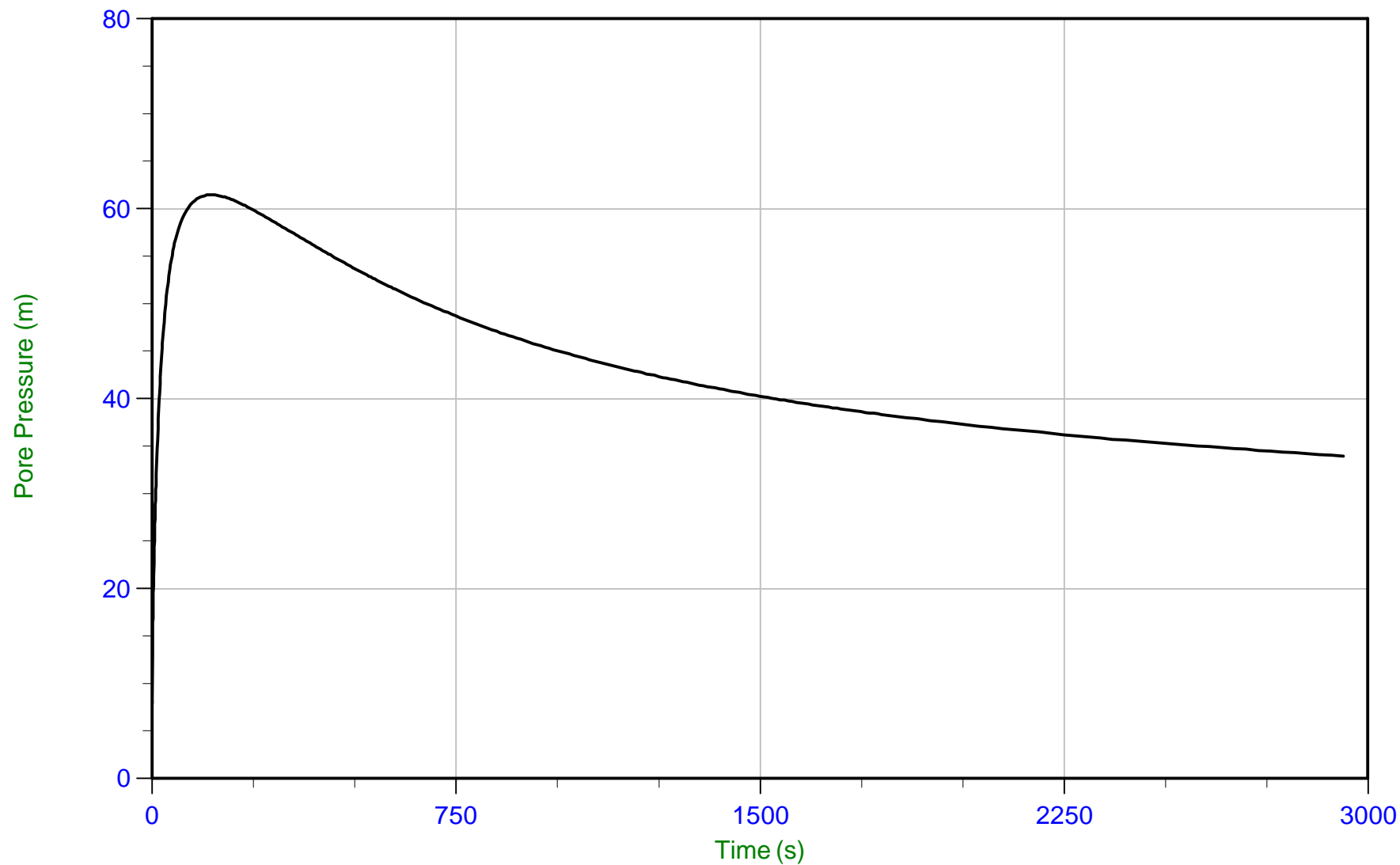
u Min: 13.8 m
u Max: 42.9 m
u Final: 38.9 m



Stantec

Job No: 24-05-27609
Date: 2024-05-09 12:06
Site: HWY 3, St.Thomas, ON

Sounding: SCPT24-CNREMB02
Cone: 729:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-05-27609_SP-CN-02.PPF2
Depth: 10.675 m / 35.023 ft
Duration: 2940.0 s

u Min: 7.9 m
u Max: 61.5 m
u Final: 34.0 m

WT: 2.0 m / 6.6 ft
Ueq: 8.7 m
U(50): 35.07 m

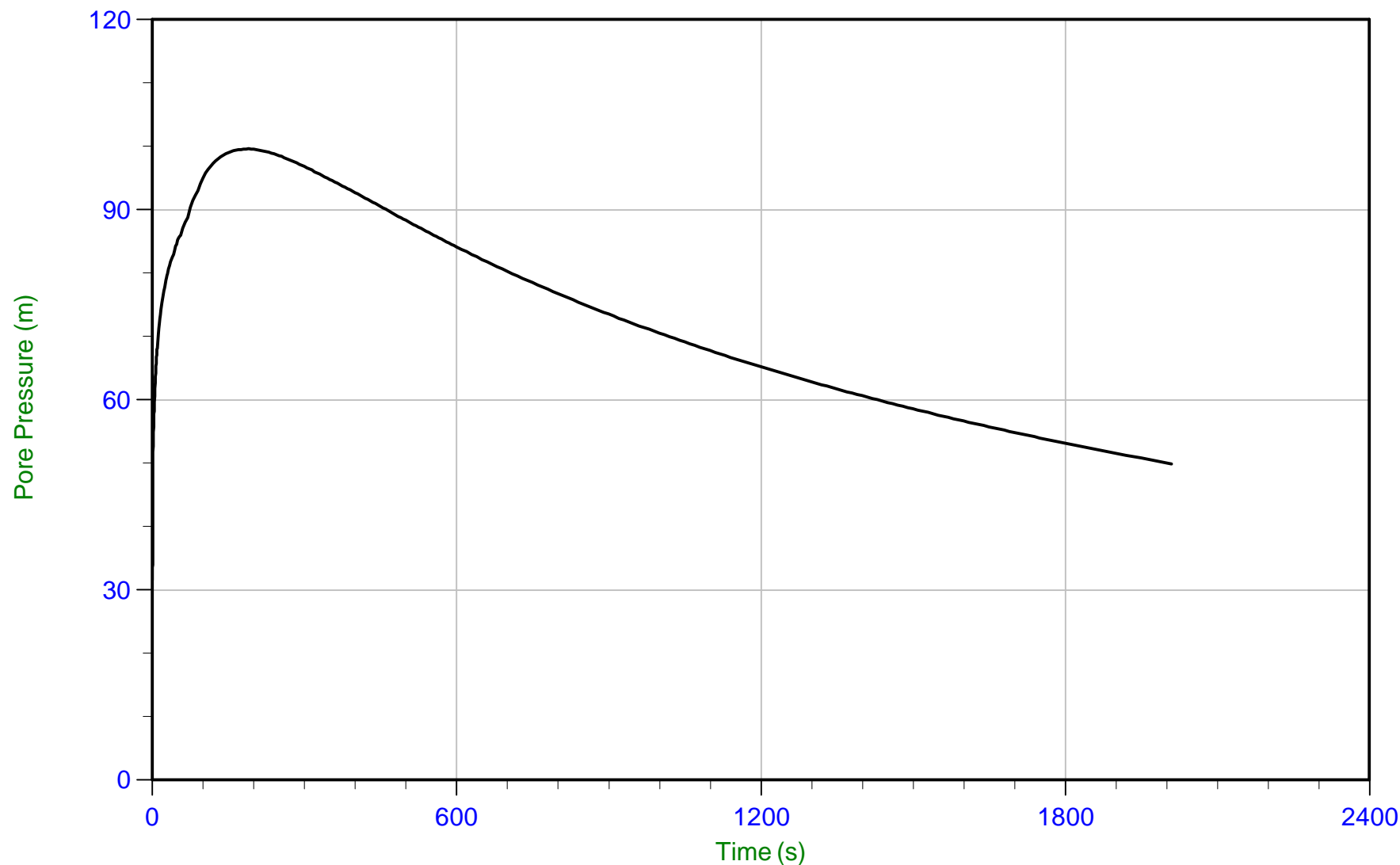
T(50): 2423.2 s
Ir: 100
Ch: 0.3 cm²/min



Stantec

Job No: 24-05-27609
Date: 2024-05-10 14:49
Site: HWY 3, St.Thomas, ON

Sounding: SCPT24-WAPP02
Cone: 729:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-05-27609_SP-WA-02.PPF2
Depth: 3.050 m / 10.006 ft
Duration: 2010.0 s

u Min: 31.5 m
u Max: 99.6 m
u Final: 49.9 m

WT: 1.8 m / 6.0 ft
Ueq: 1.2 m
U(50): 50.41 m

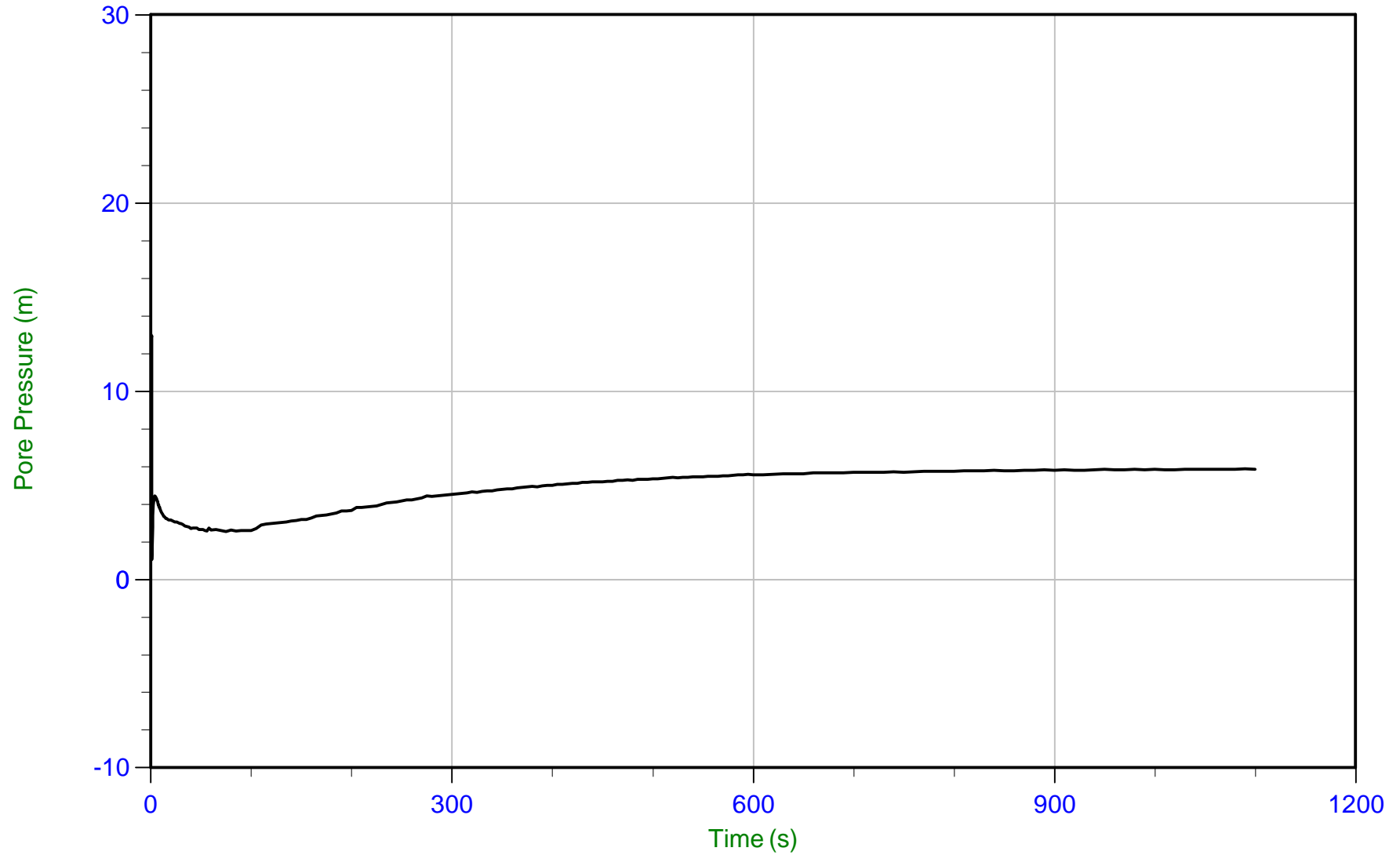
T(50): 1785.6 s
Ir: 100
Ch: 0.4 cm²/min



Stantec

Job No: 24-05-27609
Date: 2024-05-10 14:49
Site: HWY 3, St.Thomas, ON

Sounding: SCPT24-WAPP02
Cone: 729:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-05-27609_SP-WA-02.PPF2
Depth: 7.625 m / 25.016 ft
Duration: 1100.0 s

u Min: 1.1 m
u Max: 13.1 m
u Final: 5.8 m

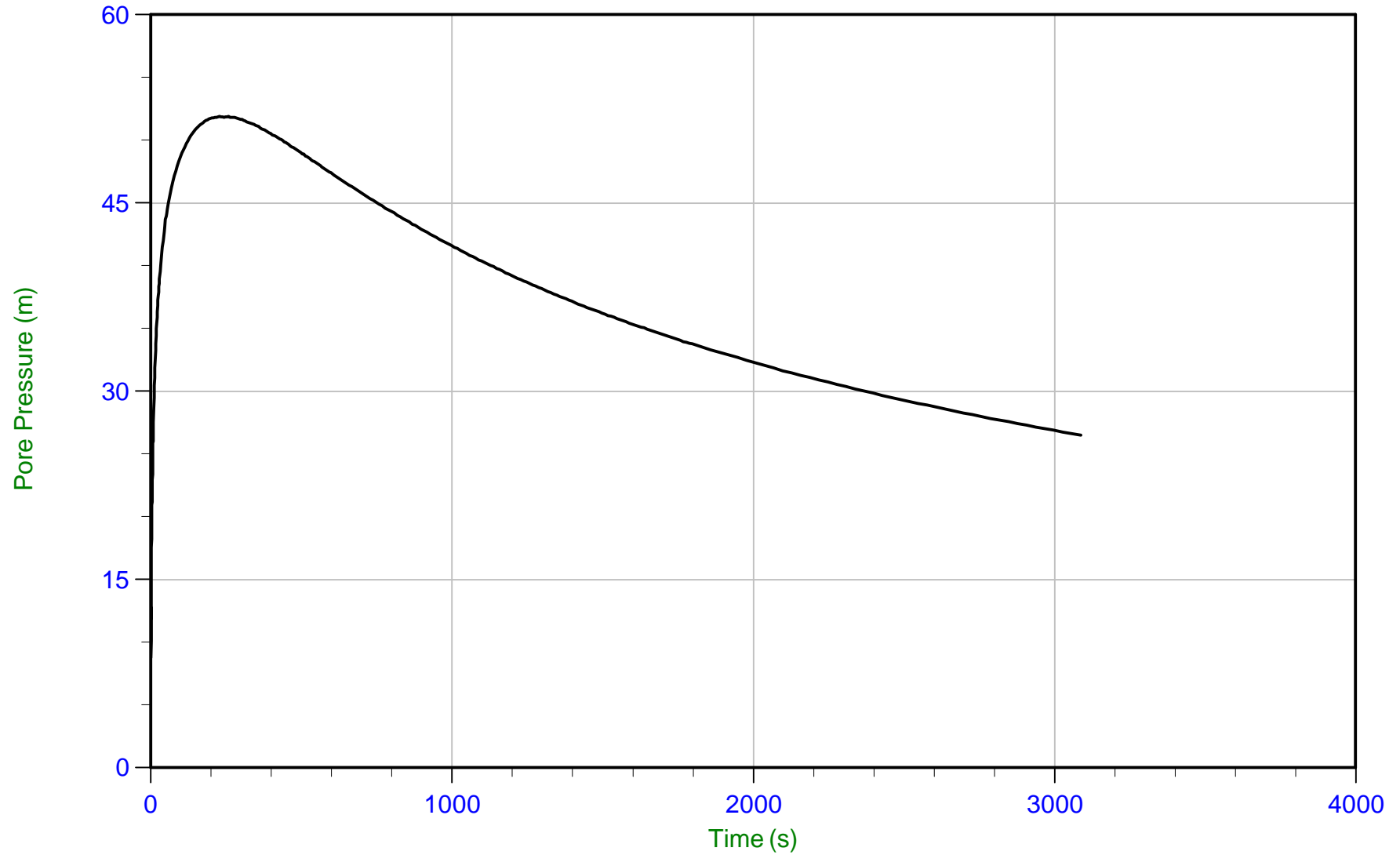
WT: 1.8 m / 6.0 ft
Ueq: 5.8 m



Stantec

Job No: 24-05-27609
Date: 2024-05-09 16:43
Site: HWY 3, St.Thomas, ON

Sounding: SCPT24-CNREMB08
Cone: 729:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-05-27609_SP-CN-08.PPF2
Depth: 3.050 m / 10.006 ft
Duration: 3090.0 s

u Min: 8.7 m
u Max: 51.9 m
u Final: 26.5 m

WT: 1.5 m / 4.9 ft
Ueq: 1.6 m
U(50): 26.72 m

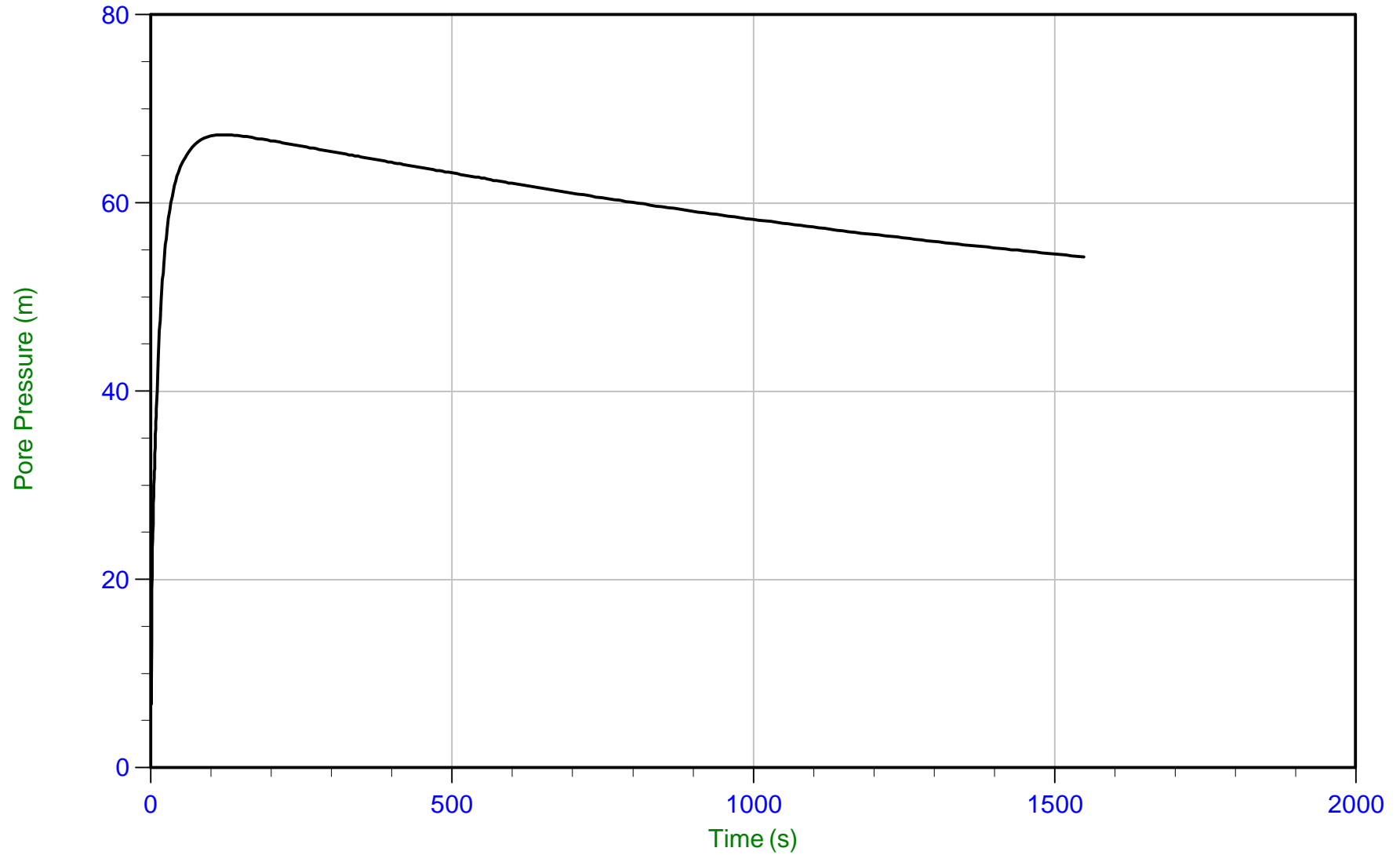
T(50): 2772.8 s
Ir: 100
Ch: 0.3 cm²/min



Stantec

Job No: 24-05-27609
Date: 2024-05-09 16:43
Site: HWY 3, St.Thomas, ON

Sounding: SCPT24-CNREMB08
Cone: 729:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-05-27609_SP-CN-08.PPF2
Depth: 7.625 m / 25.016 ft
Duration: 1550.0 s

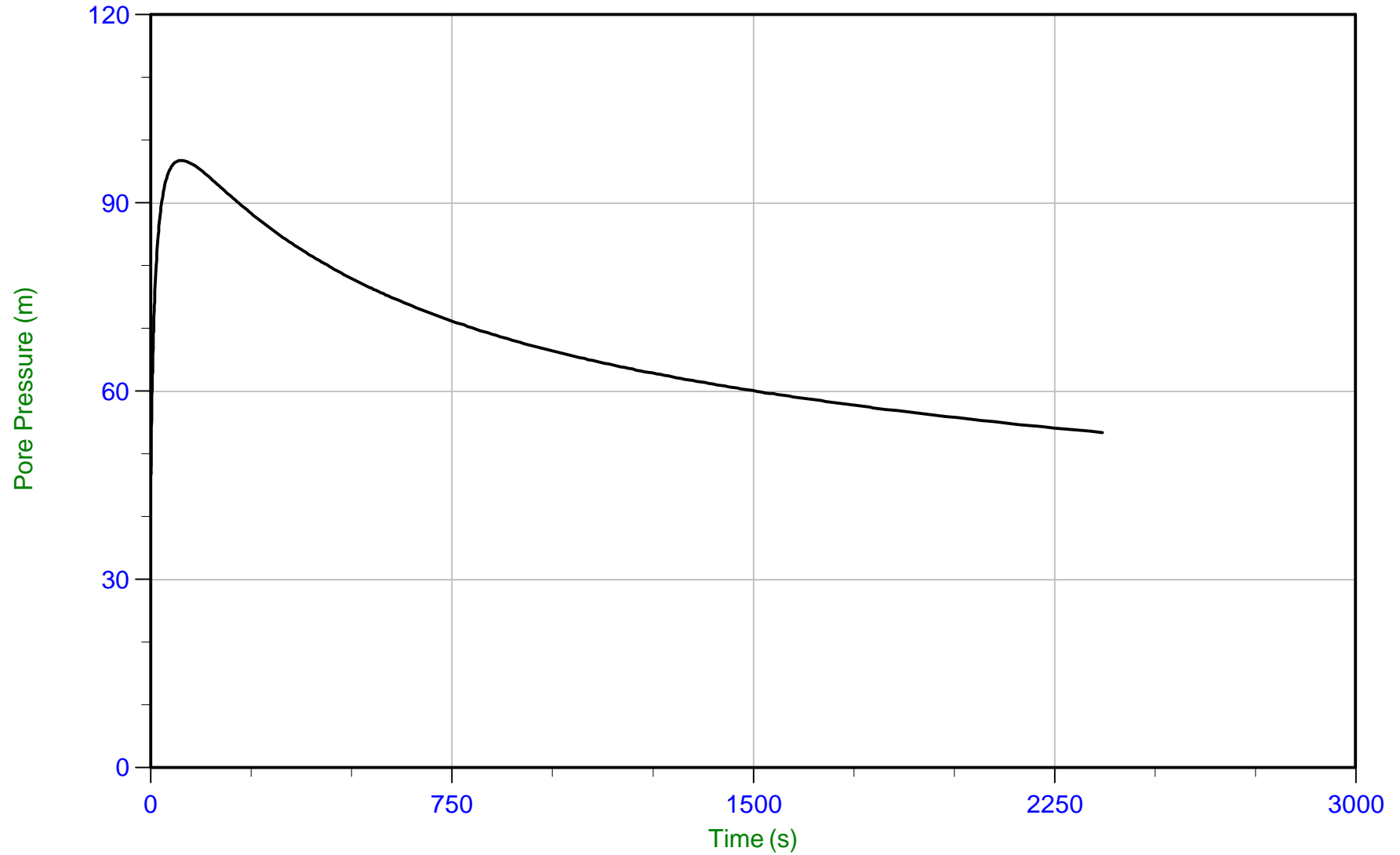
u Min: 6.7 m
u Max: 67.2 m
u Final: 54.3 m



Stantec

Job No: 24-05-27609
Date: 2024-05-09 16:43
Site: HWY 3, St.Thomas, ON

Sounding: SCPT24-CNREMB08
Cone: 729:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-05-27609_SP-CN-08.PPF2
Depth: 10.675 m / 35.023 ft
Duration: 2370.0 s

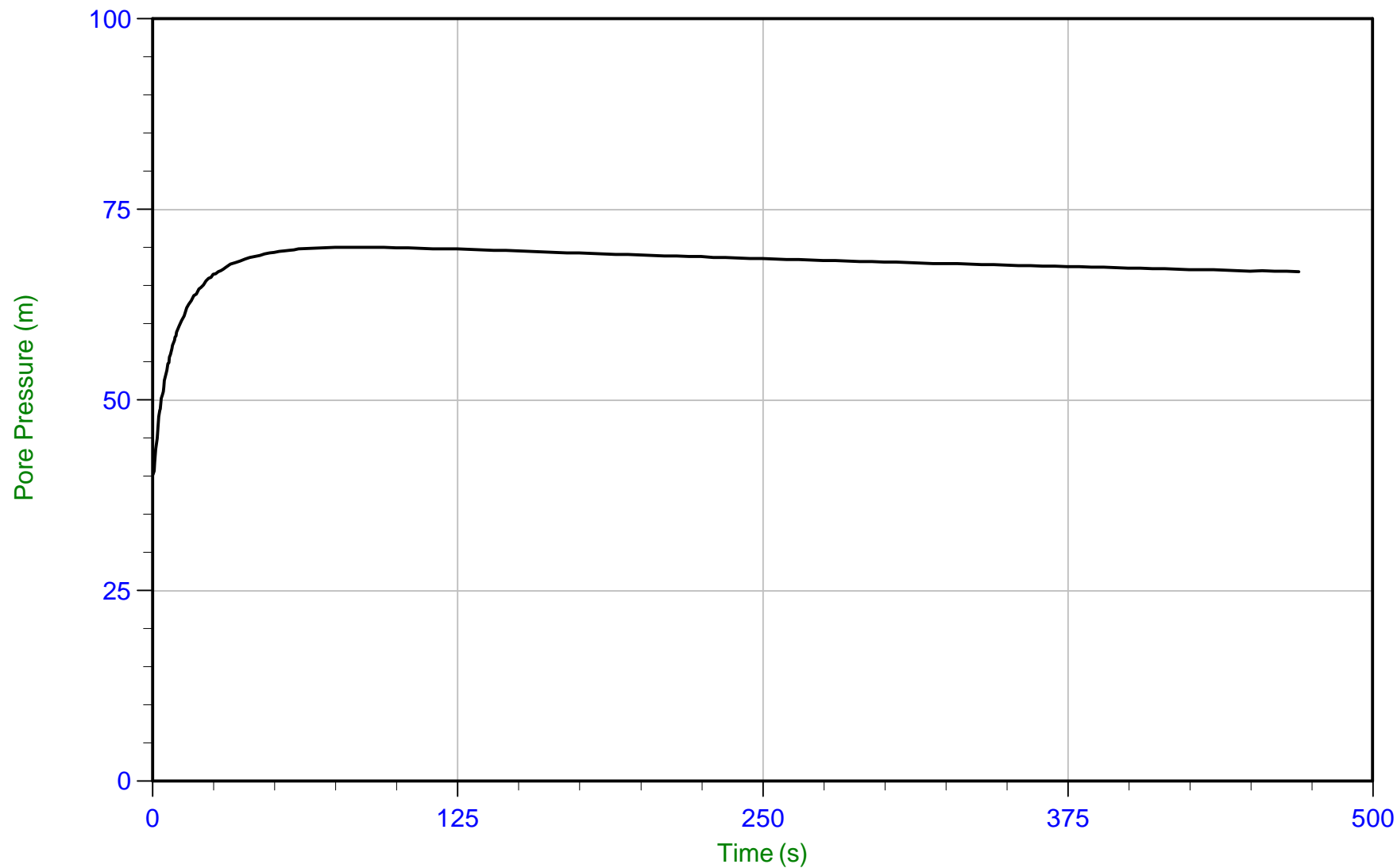
u Min: 46.6 m
u Max: 96.8 m
u Final: 53.4 m



Stantec

Job No: 24-05-27609
Date: 2024-05-09 16:43
Site: HWY 3, St.Thomas, ON

Sounding: SCPT24-CNREMB08
Cone: 729:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-05-27609_SP-CN-08.PPF2
Depth: 20.100 m / 65.944 ft
Duration: 470.0 s

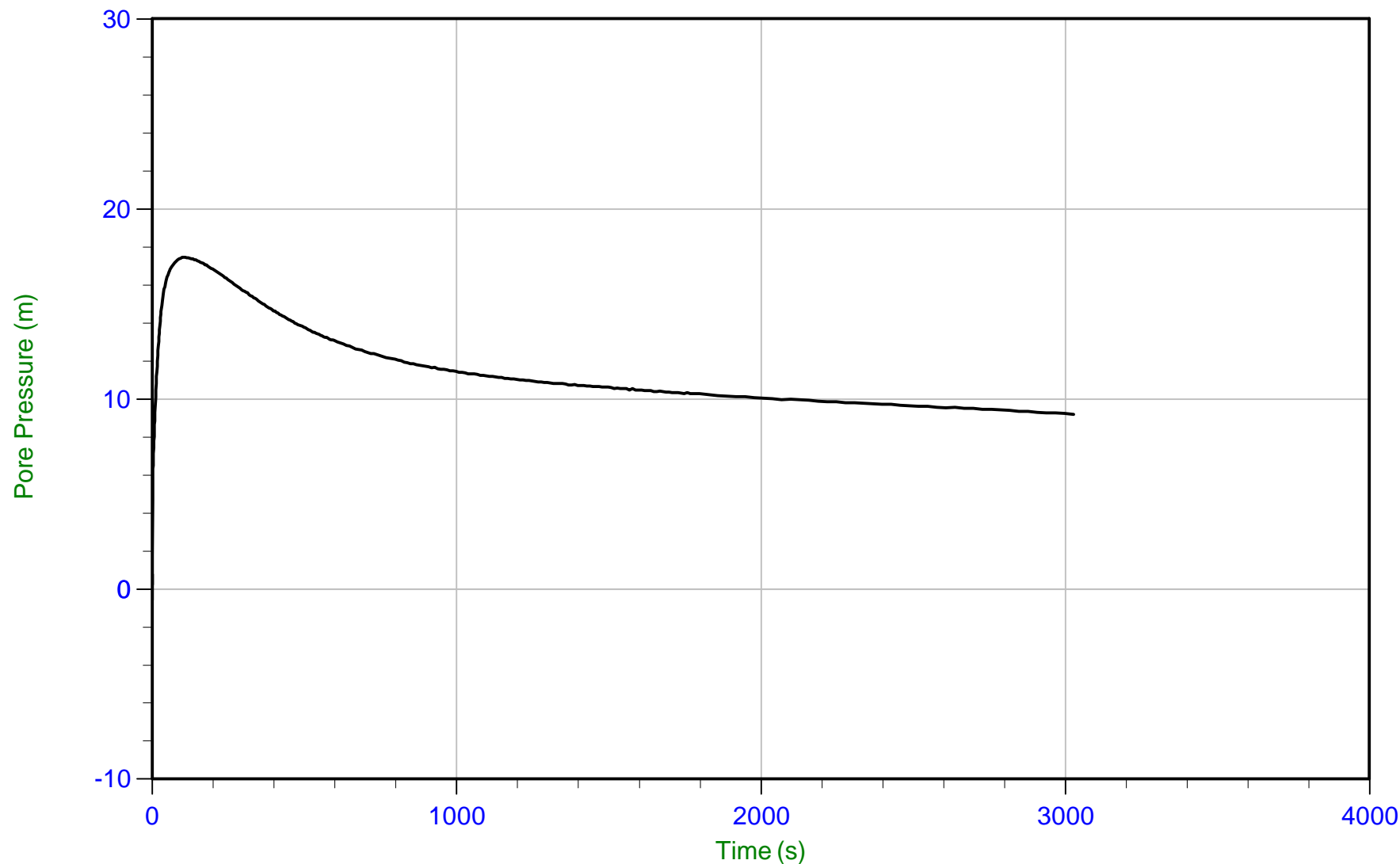
u Min: 40.1 m
u Max: 70.1 m
u Final: 66.8 m



Stantec

Job No: 24-05-27609
Date: 2024-05-10 06:58
Site: HWY 3, St.Thomas, ON

Sounding: SCPT24-CNREMB10
Cone: 729:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-05-27609_SP-CN-10.PPF2
Depth: 3.050 m / 10.006 ft
Duration: 3030.0 s

u Min: -0.6 m
u Max: 17.5 m
u Final: 9.2 m

WT: 2.0 m / 6.6 ft
Ueq: 1.1 m
U(50): 9.26 m

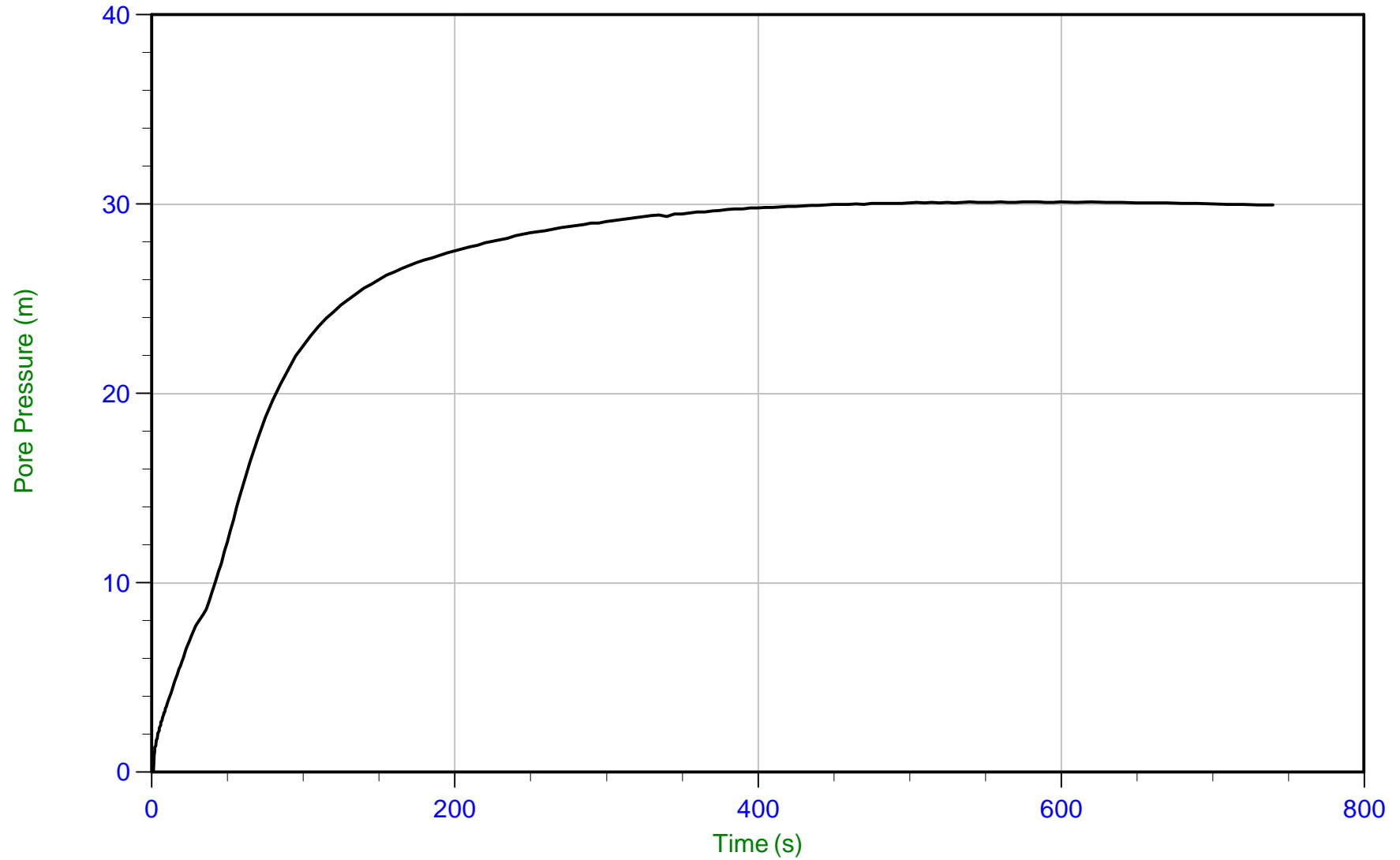
T(50): 2879.9 s
Ir: 100
Ch: 0.2 cm²/min



Stantec

Job No: 24-05-27609
Date: 2024-05-10 06:58
Site: HWY 3, St.Thomas, ON

Sounding: SCPT24-CNREMB10
Cone: 729:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-05-27609_SP-CN-10.PPF2
Depth: 7.625 m / 25.016 ft
Duration: 740.0 s

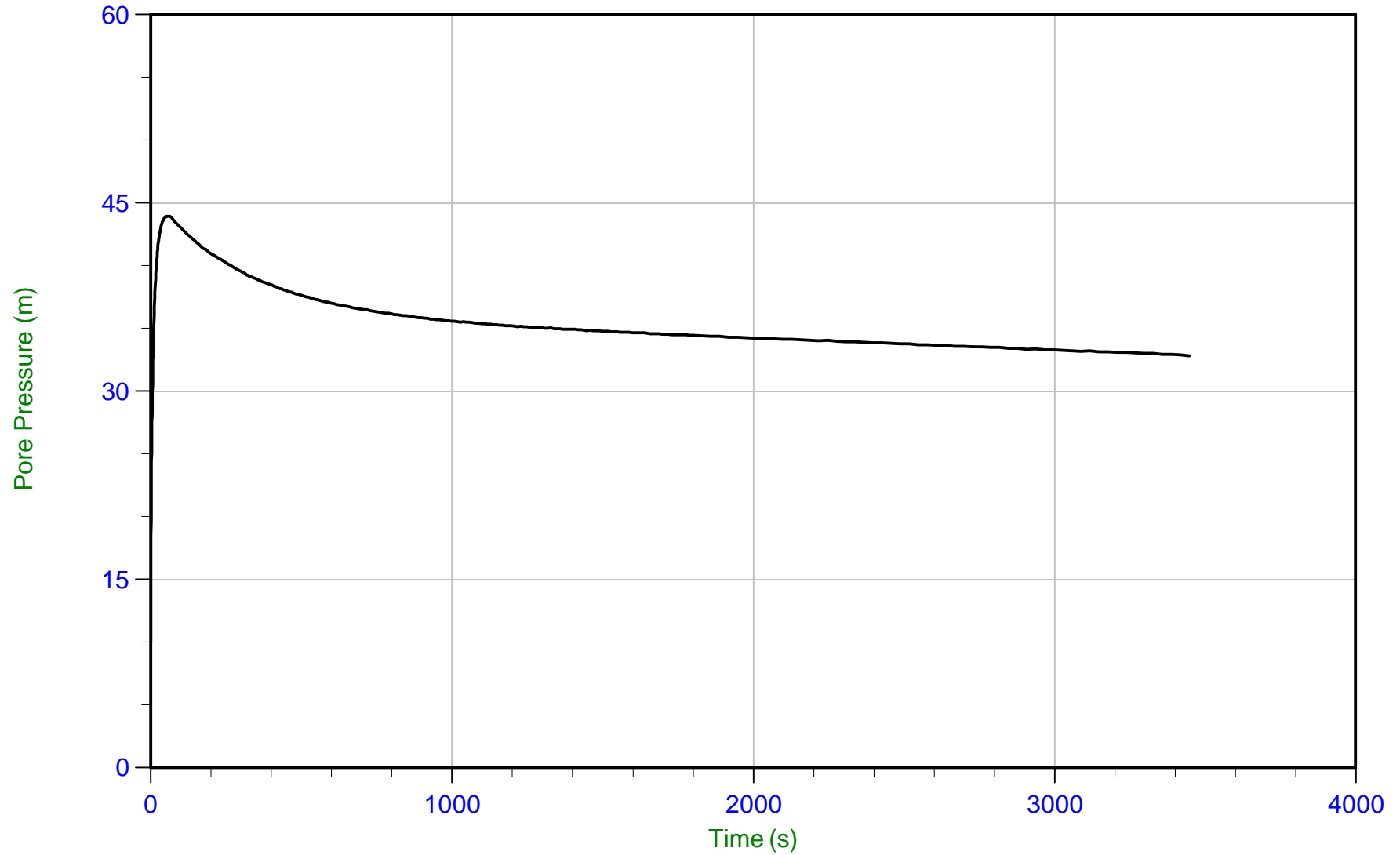
u Min: -0.6 m
u Max: 30.1 m
u Final: 30.0 m



Stantec

Job No: 24-05-27609
Date: 2024-05-10 06:58
Site: HWY 3, St.Thomas, ON

Sounding: SCPT24-CNREMB10
Cone: 729:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-05-27609_SP-CN-10.PPF2
Depth: 10.675 m / 35.023 ft
Duration: 3450.0 s

u Min: 10.0 m
u Max: 44.0 m
u Final: 32.8 m

Description of Methods for Calculated CPT Geotechnical Parameters

CALCULATED CPT GEOTECHNICAL PARAMETERS

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



Revision SZW-Rev 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: $q_t = q_c + (1-a) \cdot u_2$ (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 B_q parameter. The normalized Q_{tn} SBT chart shown in Figure 4 is based on the work by Robertson (2009) utilizing a variable stress ratio exponent, n , for normalization based on a slightly modified redefinition and iterative approach for I_c . The boundary curves drawn on the chart are based on the work described in Robertson (2010).

Figure 5 shows a revised 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.

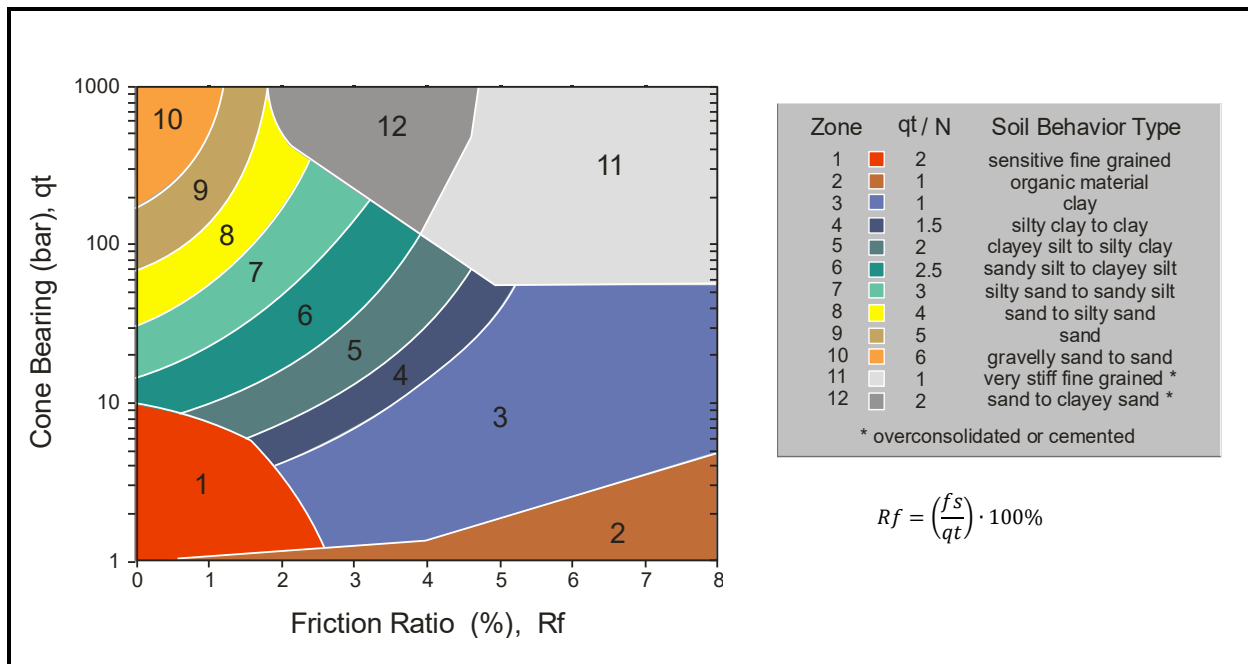


Figure 1. Non-normalized Soil Behavior Type Classification Chart (SBT)

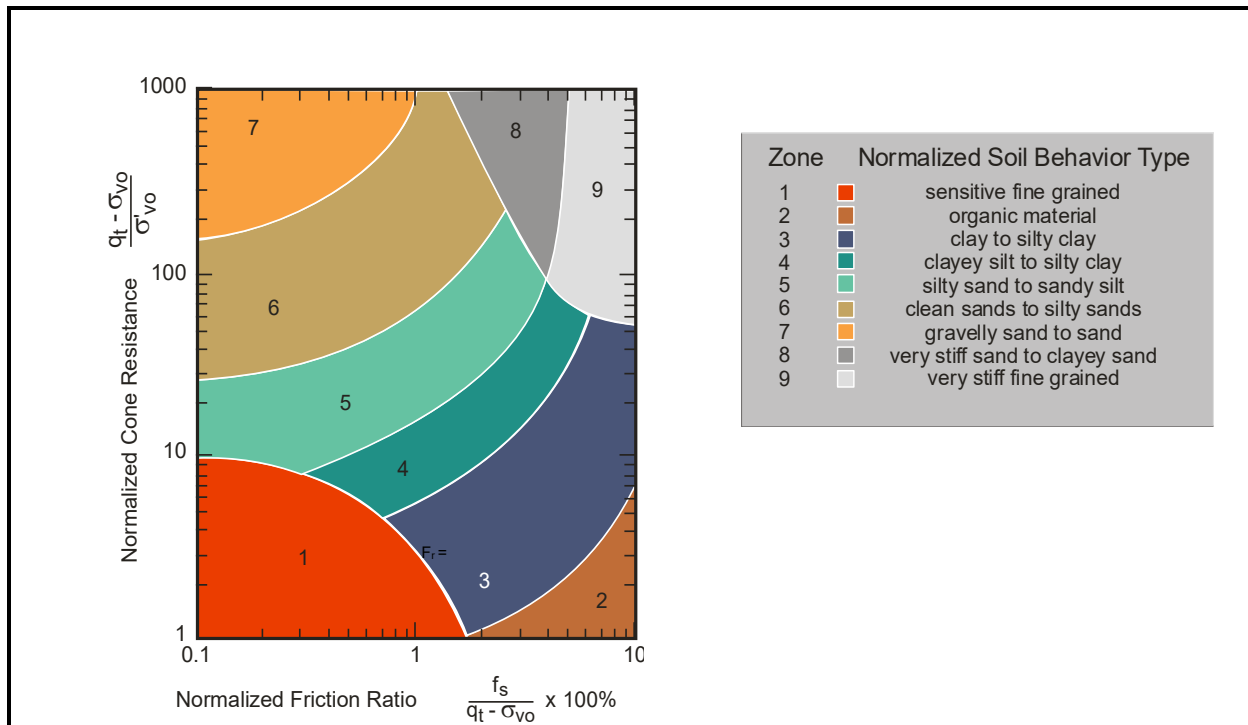
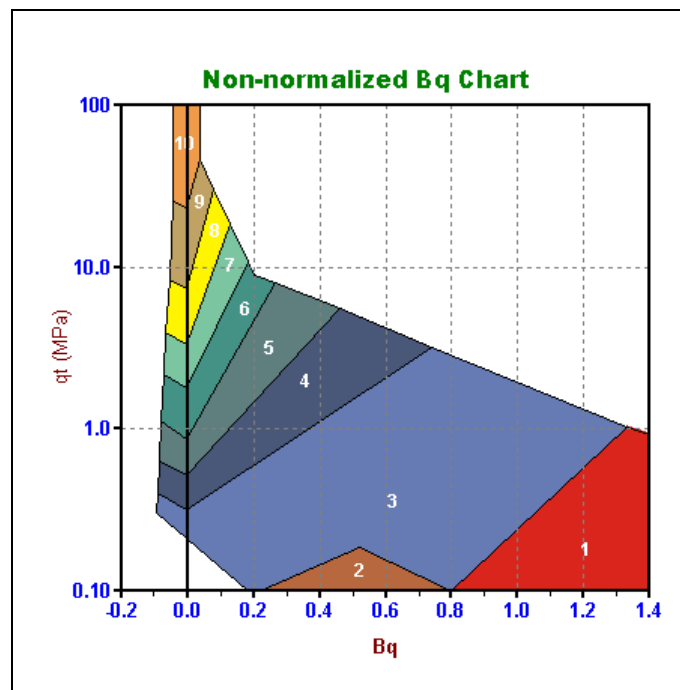
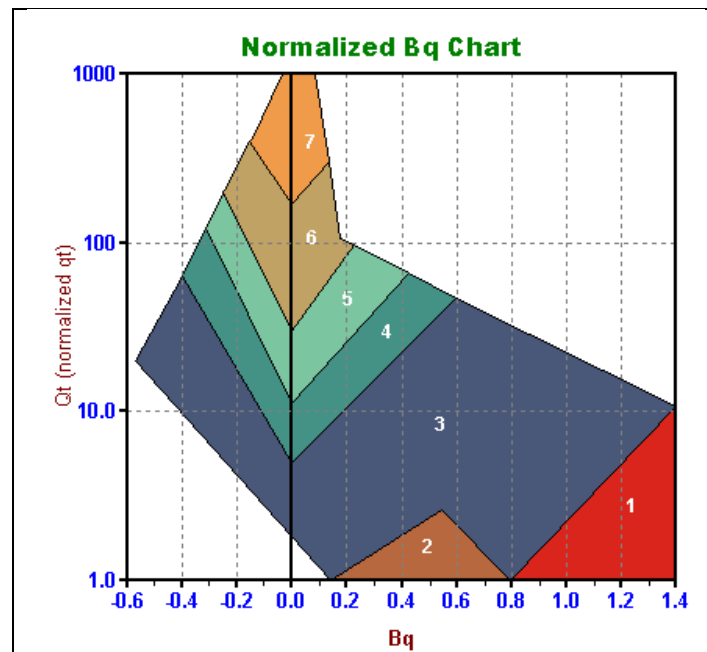
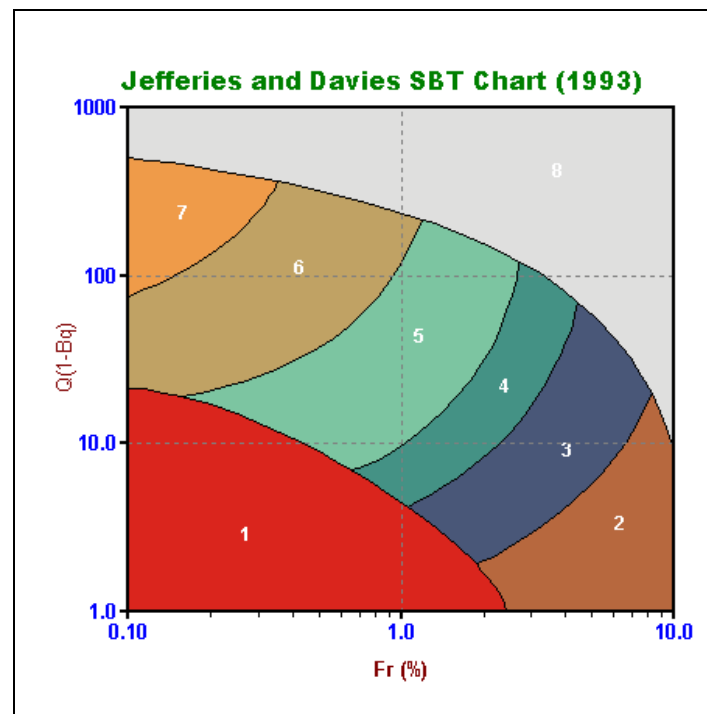


Figure 2. Normalized Soil Behavior Type Classification Chart (SBTn)

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: $Q(1-B_q)$ - F_r

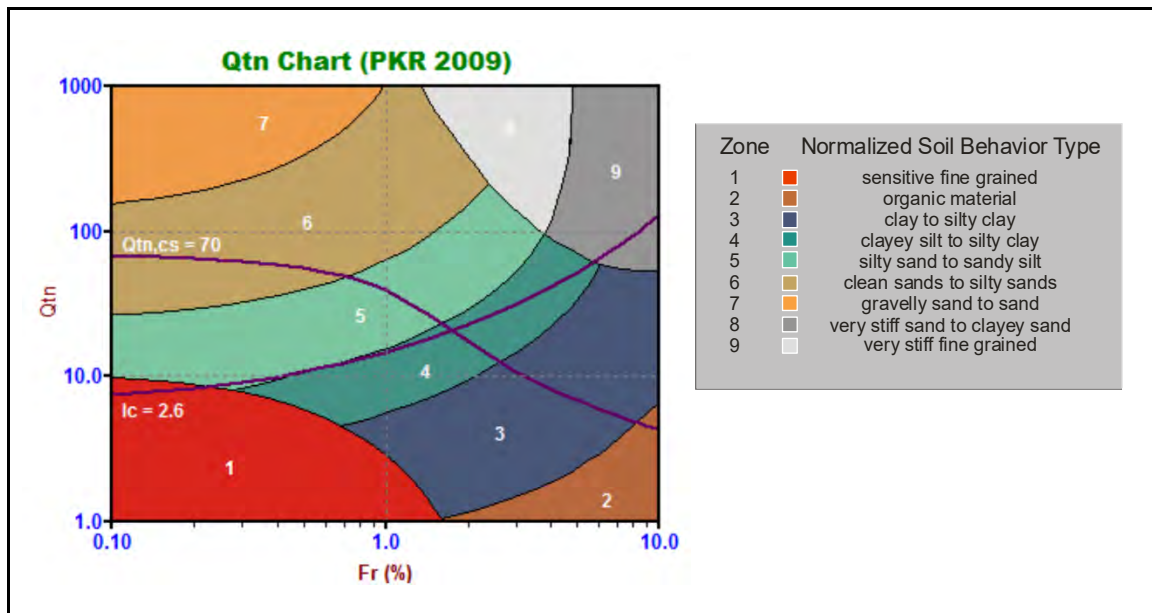
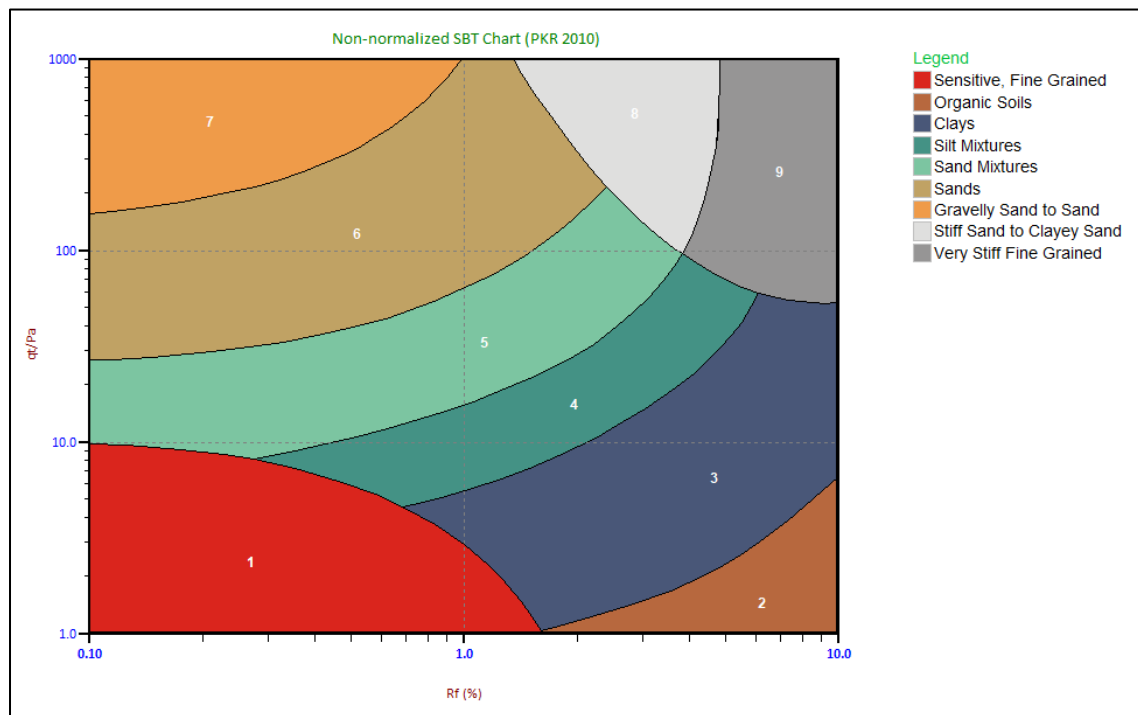
Figure 4. Normalized Soil Behavior Type Chart using Q_{tn} (SBT Q_{tn})

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

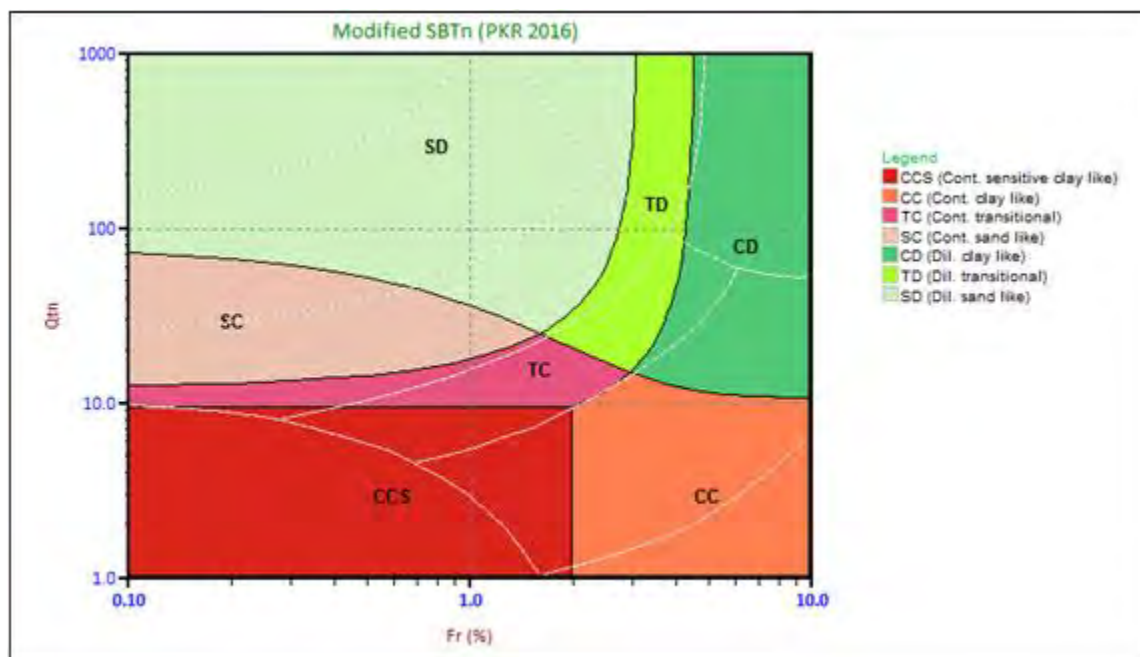


Figure 6. Modified SBTn Behavior Based Chart

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 1a and 1b 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

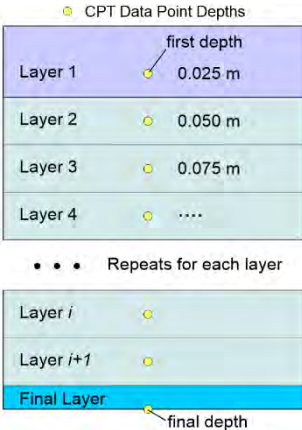
by a three or four character indicator of the output set selected (e.g. BSC, TBL, NLI, NL2, IFI, IFI2, IFI3) and possibly followed by an operator selected suffix identifying the characteristics of the particular calculation run.

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 <i>(where calculations are done at each point then Mid Layer Depth = Recorded Depth)</i>	$[Depth (Layer Top) + Depth (Layer Bottom)] / 2.0$	CK*
Elevation	Elevation of Mid Layer 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)	$Avgqc = \frac{1}{n} \sum_{i=1}^n q_c$ <i>n=1 when calculations are done at each point</i>	CK*
Avg qt	Averaged corrected tip (q_t) where: $q_t = q_c + (1 - \alpha) \cdot u_2$ 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.	$Avgqt = \frac{1}{n} \sum_{i=1}^n q_t$ <i>n=1 when calculations are done at each point</i>	1
Avg fs	Averaged sleeve friction (f_s) No pore pressure corrections are applied to f_s .	$Avgfs = \frac{1}{n} \sum_{i=1}^n f_s$ <i>n=1 when calculations are done at each point</i>	CK*
Avg Rf	Averaged friction ratio (R_f) where friction ratio is defined as: $R_f = 100\% \cdot \frac{f_s}{q_t}$	$AvgRf = 100\% \cdot \frac{Avgfs}{Avgqt}$ <i>not an average of individual R_f values</i>	CK*
Avg u	Averaged dynamic pore pressure (u)	$Avgu = \frac{1}{n} \sum_{i=1}^n u_i$ <i>n=1 when calculations are done at each point</i>	CK*
Avg Res	Averaged Resistivity (this data is not always available since it is a specialized test requiring an additional module)	$AvgRes = \frac{1}{n} \sum_{i=1}^n Resistivity_i$ <i>n=1 when calculations are done at each point</i>	CK*
Avg UVIF	Averaged UVIF ultra-violet induced fluorescence (this data is not always available since it is a specialized test requiring an additional module)	$AvgUVIF = \frac{1}{n} \sum_{i=1}^n UVIF_i$ <i>n=1 when calculations are done at each point</i>	CK*
Avg Temp	Averaged Temperature (this data is not always available)	$AvgTemp = \frac{1}{n} \sum_{i=1}^n Temperature_i$ <i>n=1 when calculations are done at each point</i>	CK*
Avg Gamma	Averaged Gamma Counts (this data is not always available since it is a specialized test requiring an additional module)	$AvgGamma = \frac{1}{n} \sum_{i=1}^n Gamma_i$ <i>n=1 when calculations are done at each point</i>	CK*
SBT	Soil Behavior Type as defined by Robertson et al 1986 (often referred to as Robertson and Campanella, 1986)	See Figure 1	1, 5
SBTn	Normalized Soil Behavior Type as defined by Robertson 1990 (linear normalization using Q_t , now referred to as Q_{t1})	See Figure 2	2, 5

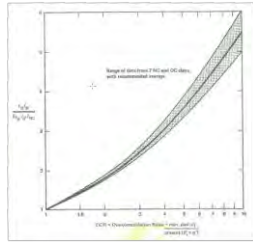
Calculated Parameter	Description	Equation	Ref
SBT-B _q	Non-normalized Soil Behavior type based on non-normalized tip resistance and the B _q parameter	See Figure 3a	1, 2, 5
SBT-B _{qn}	Normalized Soil Behavior type based on normalized tip resistance (Q _t , now called Q _{t1}) and the B _q parameter	See Figure 3b	2, 5
SBT-JandD	Soil Behavior Type as defined by Jeffries and Davies	See Figure 3c	7
SBT Q _{tn}	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.	<p>Unit Weight of soil determined from one of the following user selectable options:</p> <ol style="list-style-type: none"> 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 Q_{tn} 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 <p>The last option may co-exist with any of the other options.</p>	See references	3, 5, 15, 21, 24, 29, 33

Calculated Parameter	Description	Equation	Ref
<p>TStress</p> <p>σ_v</p>	<p>Total vertical overburden stress at Mid Layer Depth</p> <p><i>A layer is defined as the averaging interval specified by the user where depths are reported at their respective mid-layer depth.</i></p> <p>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.</p> <p>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.</p> <p>For over-water work the total stress due to the column of water above the mud line is taken into account where appropriate.</p>	<p>$TStress = \sum_{i=1}^n \gamma_i h_i$</p> <p>where γ_i is layer unit weight h_i is layer thickness</p> 	CK*
<p>EStress</p> <p>σ_v'</p>	<p>Effective vertical overburden stress at mid-layer depth.</p>	$\sigma_v' = \sigma_v - u_{eq}$	CK*
<p>Equil u</p> <p>u_{eq} or u_0</p>	<p>Equilibrium pore pressures are determined from one of the following user selectable options:</p> <ol style="list-style-type: none"> 1) hydrostatic below the water table 2) user supplied profile 3) combination of those above <p>When a user supplied profile is used/provided a linear interpolation is performed between equilibrium pore pressures defined at specific depths. If the profile values start below the water table then a linear transition from zero pressure at the water table to the first defined point is used.</p> <p>Equilibrium pore pressures may come from dissipation tests, adjacent piezometers or other sources. Occasionally, an extra equilibrium point ("assumed value") will be provided in the profile that does not come from a recorded value to smooth out any abrupt changes or to deal with material interfaces. These "assumed" values will be indicated on our plots and in tabular summaries.</p>	<p>For the hydrostatic option:</p> $u_{eq} = \gamma_w \cdot (D - D_{wt})$ <p>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</p>	CK*
K_0	Coefficient of earth pressure at rest, K_0 .	$K_0 = (1 - \sin \Phi') OCR^{\sin \Phi'}$	17
C_n	Overburden stress correction factor used for $(N_1)_{60}$ and older CPT parameters.	$C_n = (P_a / \sigma_v')^{0.5}$ <p>where $0.0 < C_n < 2.0$ (user adjustable, typically ranging from 1.7 to 2.0) P_a is atmospheric pressure (100 kPa)</p>	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) <i>Robertson and Wride define C_q to be the same as C_n. The Olson definition above is used in the program.</i>	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.	$(N_1)_{60} = C_n \cdot N_{60}$	4
N_{60I_c}	SPT N_{60} values based on the I_c parameter, as defined by Robertson and Wride 1998 (3), or by Robertson 2009 (15).	$(q_t/P_a)/N_{60} = 8.5 (1 - I_c/4.6)$ $(q_t/P_a)/N_{60} = 10^{(1.1268 - 0.2817I_c)}$ P_a being atmospheric pressure	3, 5 15, 31
$(N_1)_{60I_c}$	SPT N_{60} value corrected for overburden pressure (using N_{60I_c}). User has 3 options.	1) $(N_1)_{60I_c} = C_n \cdot (N_{60I_c})$ 2) $q_{c1n}/(N_1)_{60I_c} = 8.5 (1 - I_c/4.6)$ 3) $(Q_{tn})/(N_1)_{60I_c} = 10^{(1.1268 - 0.2817I_c)}$	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.	$S_u = \frac{q_t - \sigma_v}{N_{kt}}$	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.	$S_u = \frac{u_2 - u_{eq}}{N_{\Delta u}}$	1, 5
D_r	Relative Density determined from one of the following user selectable options: 1) Ticino Sand 2) Hoksund 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
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)	$= \frac{\Delta u}{q_t}$ 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	$B_q = \frac{\Delta u}{q_t - \sigma_v}$ <p>where: $\Delta u = u - u_{eq}$ and u = dynamic pore pressure u_{eq} = equilibrium pore pressure</p>	1, 2, 5
Net q_t or qt_{Net}	Net tip resistance (used in many subsequent correlations)	$q_t - \sigma_v$	36
q_e or qE or qE	Effective tip resistance (using the dynamic pore pressure u_2 and not equilibrium pore pressure)	$q_t - u_2$	36
qe_{Norm}	Normalized effective tip resistance	$\frac{q_t - u_2}{\sigma_v}$	36
Q_t or Norm: Q_t 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.	$Q_t = \frac{q_t - \sigma_v}{\sigma_v}$	2, 5, 15
F_r or Norm: F_r	Normalized Friction Ratio for Soil Behavior Type classification as defined by Robertson (1990)	$F_r = 100\% \cdot \frac{fs}{q_t - \sigma_v}$	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.	$Q \cdot (1 - B_q)$ $Q \cdot (1 - B_q) + 1$ where B_q 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)	$q_{c1} = q_t \cdot (P_a / \sigma_v')^{0.5}$ 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)	$q_{c1} (0.5) = (q_t / P_a) \cdot (P_a / \sigma_v')^{0.5}$ where: P_a = atmospheric pressure	5
$q_{c1} (C_n)$	Normalized tip resistance, q_{c1} , based on C_n (this method has stress units)	$q_{c1}(C_n) = C_n * q_t$	5, 12
$q_{c1} (C_q)$	Normalized tip resistance, q_{c1} , based on C_q (this method has stress units)	$q_{c1} (C_q) = C_q * q_t$ (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 I_c (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. I_c (RW1998) is different from that of Jefferies and Davies (7) and is different from I_c (PKR2009).	$I_c = [(3.47 - \log_{10} Q)^2 + (\log_{10} Fr + 1.22)^2]^{0.5}$ <p>Where: $Q = \left(\frac{qt - \sigma_v}{P_a} \right) \left(\frac{P_a}{\sigma_v} \right)^n$</p> <p>Or $Q = q_{c1n} = \left(\frac{qt}{P_a} \right) \left(\frac{P_a}{\sigma_v} \right)^n$</p> <p>depending on the iteration in determining I_c</p> <p>And Fr is in percent P_a = atmospheric pressure</p> <p>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</p> <p>Note that NCEER replaced 0.75 with 0.70</p>	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).	$I_c \text{ (PKR 2009)} = [(3.47 - \log_{10} Q_{tn})^2 + (1.22 + \log_{10} Fr)^2]^{0.5}$	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).	$n \text{ (PKR 2009)} = 0.381 (I_c) + 0.05 (\sigma_v'/P_a) - 0.15$	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).	$Q_{tn} = [(qt - \sigma_v)/P_a](P_a/\sigma_v')^n$ <p>where P_a = atmospheric pressure (100 kPa) n = stress ratio exponent described above</p>	15
FC	Apparent fines content (%)	$FC = 1.75(I_c^{3.25}) - 3.7$ $FC = 100 \text{ for } I_c > 3.5$ $FC = 0 \text{ for } I_c < 1.26$ $FC = 5\% \text{ if } 1.64 < I_c < 2.6 \text{ AND } F_r < 0.5$	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 Q_{tn} 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.	$CD = 70 = (Q_{tn} - 11) (1 + 0.06F_r)^{17}$ <p>lower bound of CD = 60: $CD = 60 = (Q_{tn} - 9.5) (1 + 0.06F_r)^{17}$ </p>	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) 1 st order approximation using q_t Net (clays) 3) 1 st order approximation using Δu_2 (clays) 4) 1 st order approximation using q_e (clays) 5) Based on V_s	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) $\sigma_p' = (V_s/4.59)^{1.47}$	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 of $S_u/\sigma_v' / (S_u/\sigma_v')_{NC}$ 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 Q_t	1) requires a user defined value for NC S_u/P_c' ratio 2 through 5) <i>based on yield stresses</i> 6) $YSR (Vs) = \sigma_p' (Vs) / \sigma_v'$ 7) $OCR = 0.25 \cdot (Q_t)^{1.25}$	9 19 20 20 20 18 32
E_s/q_t	Intermediate parameter for calculating Young's Modulus, E , in sands. It is the Y axis of the reference chart. Note that Figure 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
	<p>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.</p> <p>Figure 5.59 from LRP shows a dimensionless form of the equation, q_{c1}, displaying the same range of values.</p> <p>Figure 5.59's X axis uses $q_{c1} = \left(\frac{q_c}{p_a}\right) \left(\frac{p_a}{\sigma'_v}\right)^{0.5}$</p> <p>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.</p> <p>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.</p>		
Es or Es Young's Modulus E	<p>Young's Modulus based on the work done in Italy. There are three types of sands considered in this technique. The user selects the appropriate type for the site from:</p> <ul style="list-style-type: none"> a) OC Sands b) Aged NC Sands c) Recent NC Sands <p>Each sand type has a family of curves that depend on mean normal stress. The program calculates mean normal stress and linearly interpolates between the two extremes provided in the E_s/q_t chart. E_s is evaluated for an axial strain of 0.1%.</p>	<p>Mean normal stress is evaluated from:</p> $\sigma'_m = \frac{1}{3}(\sigma'_v + \sigma'_h + \sigma'_h)$ <p>where σ'_v = vertical effective stress σ'_h = horizontal effective stress</p> <p>and $\sigma'_h = K_o \cdot \sigma'_v$ with K_o assumed to be 0.5</p>	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	$= S_u (N_{kt}) / \sigma'_v$	9, 23
Vs or Vs	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 Vp	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.	$V_{s30} = \frac{\text{total thickness of all layers to 30 m}}{\sum \left(\frac{\text{layer thickness}}{\text{layer shear wave velocity}} \right)}$ $V_{s30} = \frac{\text{total thickness of all layers to 30 m}}{\sum (\text{layer travel times})}$	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)	$= (q_t - \sigma_v) / G_{max}$ 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	$G_o = 0.0188[10^{(0.55I_c + 1.68)}](q_t - \sigma_v)$	15
Estimated E ₂₅	Estimated value for Young’s Modulus, E, at a 25% working load	$E_{25} = \alpha_E (qtNet)$ where $\alpha_E = 0.015[10^{(0.55I_c + 1.68)}]$	15
k _{SBT}	Estimated soil permeability derived from Soil Behavior Type (SBT) Chart I _c values.	For $1.0 < I_c \leq 3.27$: $k = 10^{(0.952 - 3.04I_c)} \quad \text{in m/s}$ For $3.27 < I_c < 4.0$: $k = 10^{(-4.52 - 1.37I_c)} \quad \text{in m/s}$	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) $\alpha_M = Qt \quad \text{when } Qt < 14$ $\alpha_M = 14 \quad \text{when } Qt > 14$ $I_c < 2.2$ (coarse grained) $\alpha_M = 0.0188 [10^{(0.55I_c + 1.68)}]$ $D' = \alpha_D (q_t - \sigma_v)$ 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 $(N_1)_{60}$	$K_{SPT} = 1 + ((0.75/30) \cdot (FC - 5))$	10
K_{CPT} or K_c (RW1998)	Equivalent clean sand correction for q_{c1N}	$K_{cpt} = 1.0$ for $l_c \leq 1.64$ $K_{cpt} = f(l_c)$ for $l_c > 1.64$ (see reference) $K_c = -0.403 l_c^4 + 5.581 l_c^3 - 21.63 l_c^2 + 33.75 l_c - 17.88$	3, 10
K_c (PKR 2010)	Clean sand equivalent factor to be applied to Q_{tn}	$K_c = 1.0$ for $l_c \leq 1.64$ $K_c = -0.403 l_c^4 + 5.581 l_c^3 - 21.63 l_c^2 + 33.75 l_c - 17.88$ for $l_c > 1.64$	16
$(N_1)_{60cs} l_c$	Clean sand equivalent SPT $(N_1)_{60} l_c$. User has 3 options.	1) $(N_1)_{60cs} l_c = \alpha + \beta((N_1)_{60} l_c)$ 2) $(N_1)_{60cs} l_c = K_{SPT} * ((N_1)_{60} l_c)$ 3) $(q_{c1ncs}) / (N_1)_{60cs} l_c = 8.5 (1 - l_c/4.6)$ FC \leq 5%: $\alpha = 0, \quad \beta = 1.0$ FC \geq 35%: $\alpha = 5.0, \quad \beta = 1.2$ 5% < FC < 35%: $\alpha = \exp[1.76 - (190/FC^2)]$ $\beta = [0.99 + (FC^{1.5}/1000)]$	10 10 5
q_{c1ncs}	Clean sand equivalent q_{c1n}	$q_{c1ncs} = q_{c1n} \cdot K_{cpt}$	3
$Q_{tn,cs}$ (PKR 2010)	Clean sand equivalent for Q_{tn} described above - Q_{tn} being the normalized tip resistance based on a variable stress exponent as defined by Robertson (2009)	$Q_{tn,cs} = Q_{tn} \cdot K_c$ (PKR 2016)	16
$S_u(Liq)/ES_v$ 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
$S_u(Liq)/ES_v$ 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
K_g 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

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**FOUNDATION INVESTIGATION AND DESIGN REPORT – RON MCNEIL LINE INTERCHANGE
UNDERPASS – HIGHWAY 4 WIDENING FROM CLINTON LINE TO NEW TALBOTVILLE BYPASS
AND NEW TALBOTVILLE BYPASS FROM HIGHWAY 4 TO HIGHWAY 3 AT RON MCNEIL LINE**

April 2025

APPENDIX D

D.1 GRAIN SIZE DISTRIBUTION PLOTS AND PLASTICITY CHARTS (FIGURES D1-D6)

D.2 CONSOLIDATION TESTS

D.3 TRIAXIAL TEST RESULTS

D.4 CORROSIVITY TEST RESULTS

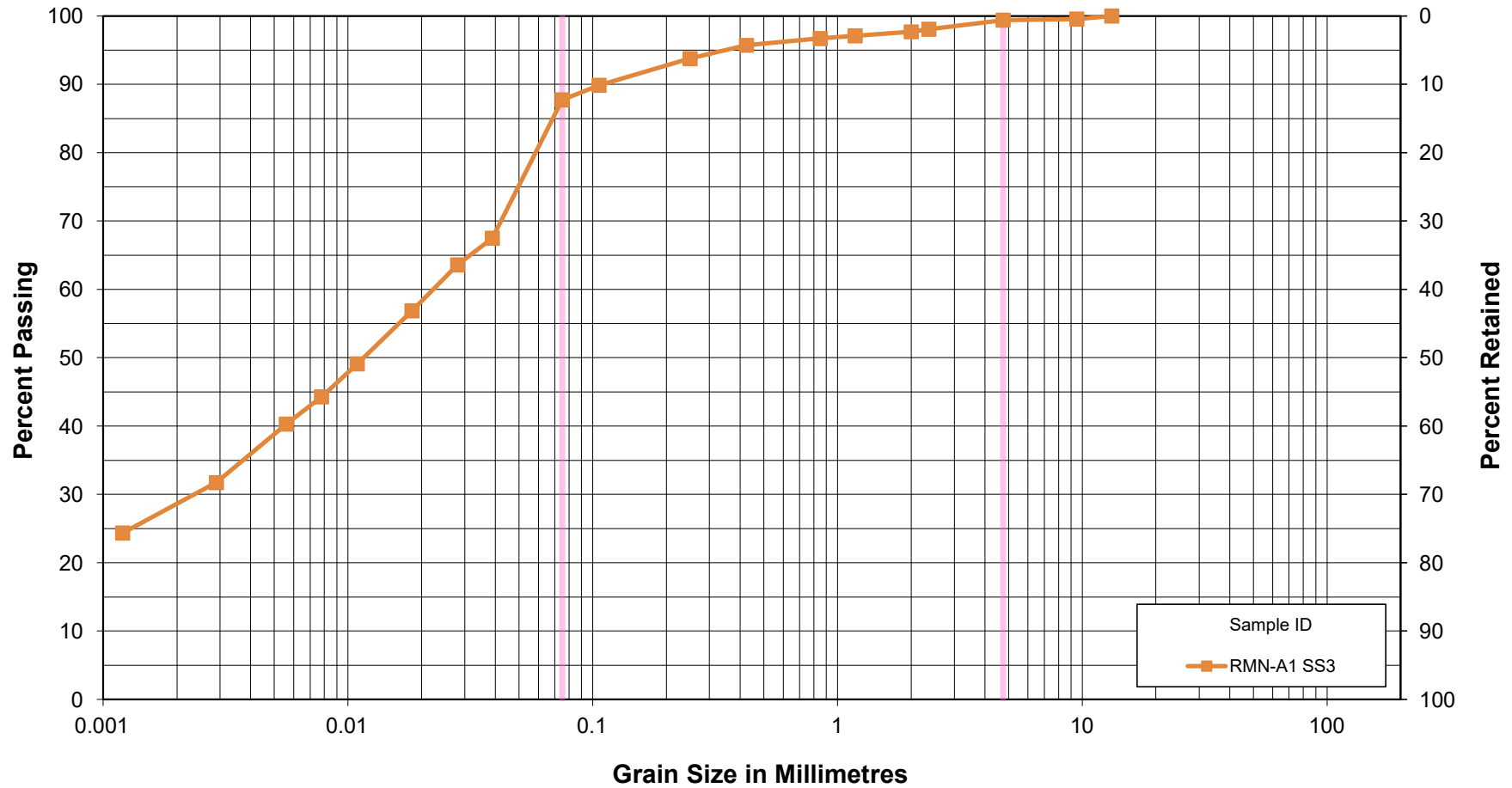


Unified Soil Classification System

CLAY & SILT	SAND			Gravel	
	Fine	Medium	Coarse	Fine	Coarse

U.S. Std. Sieve No.

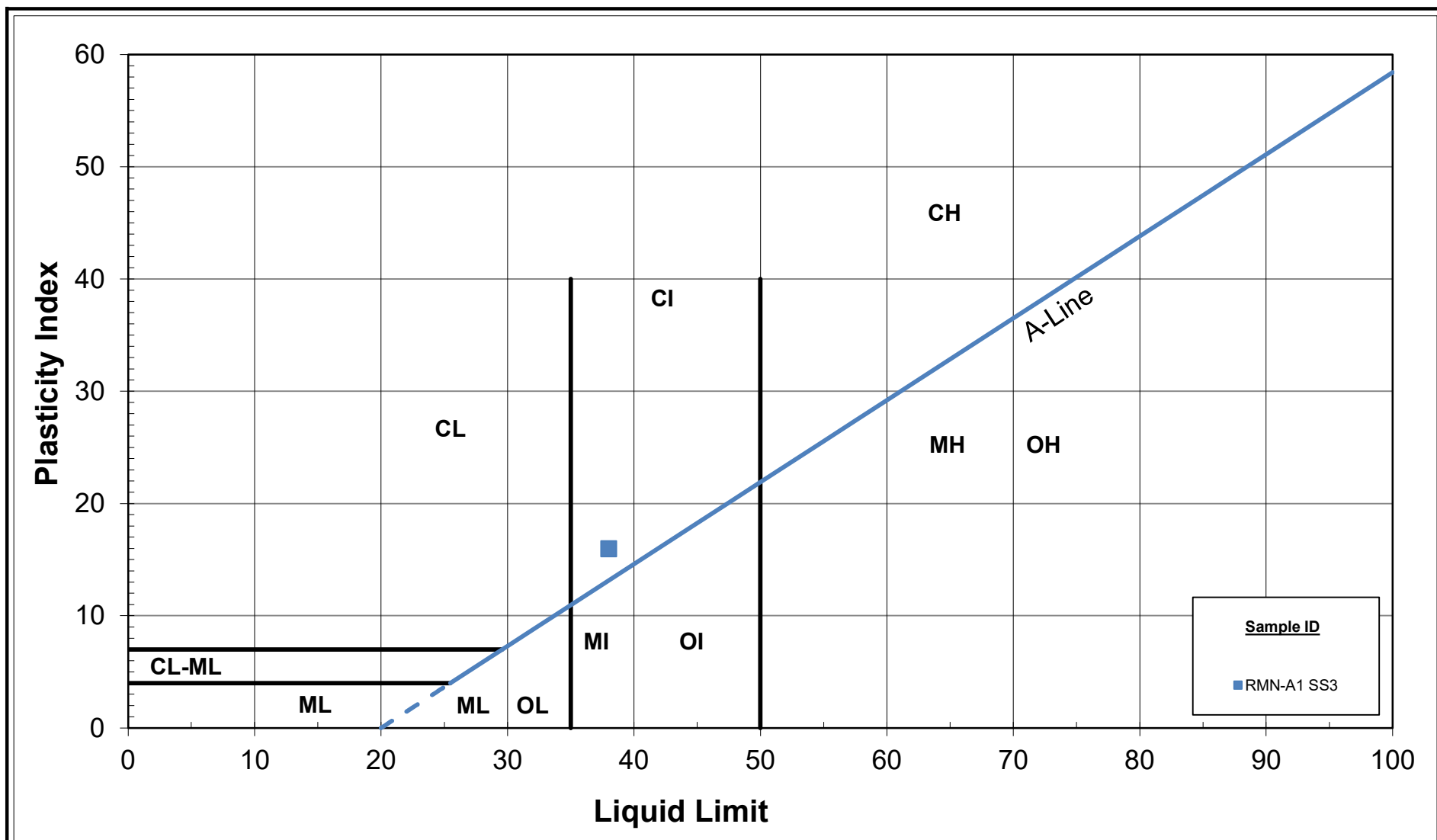
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FILL: SILTY CLAY (CI)
Ministry of Transportation (MTO)
Talbotville Bypass - Ron McNeil Line Overpass

Figure No. D1

Project No. 165001308



FILL: SILTY CLAY (CI)

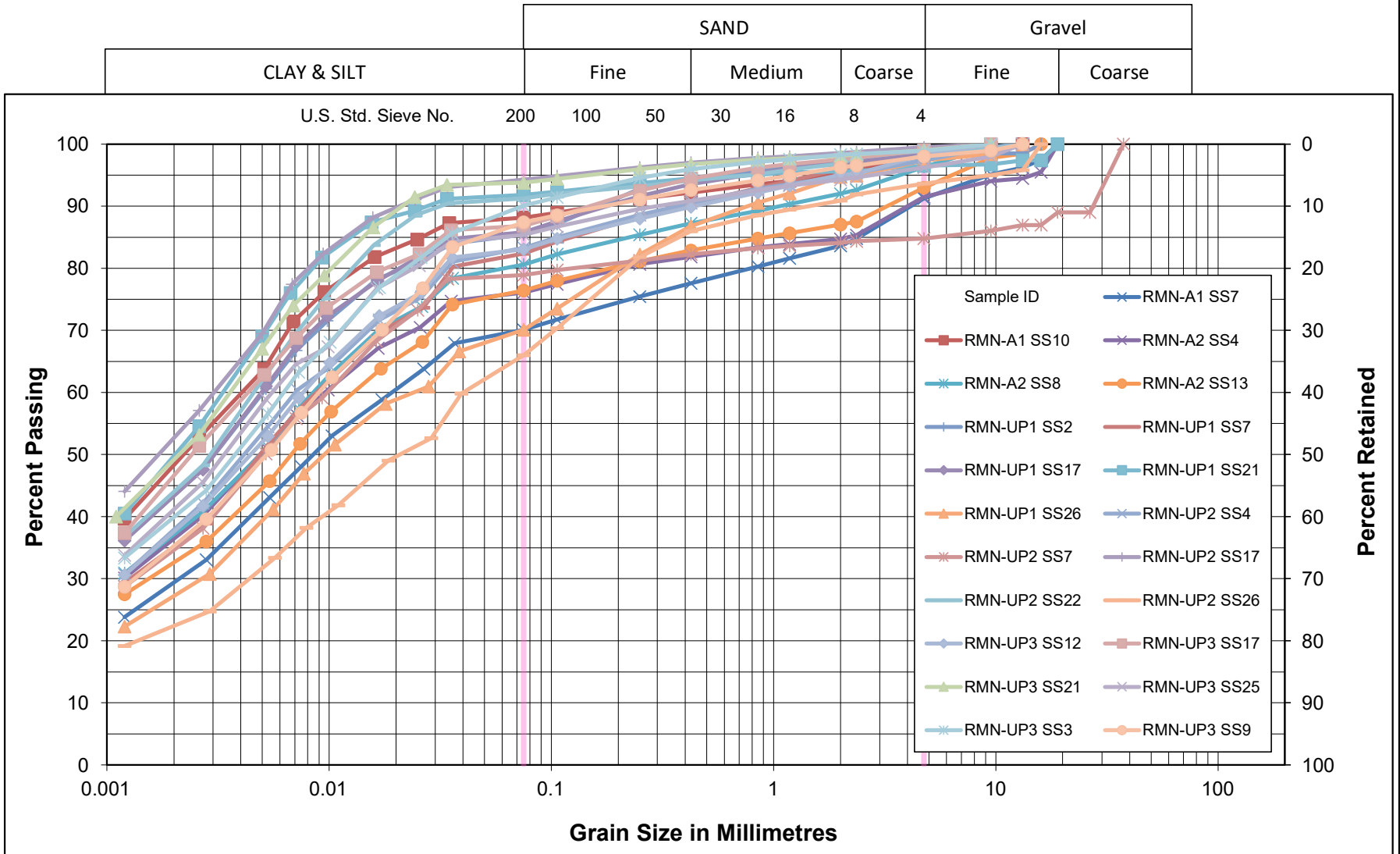
Ministry of Transportation (MTO)

Talbotville Bypass - Ron McNeil Overpass

Figure No. D2

Project No. 165001308

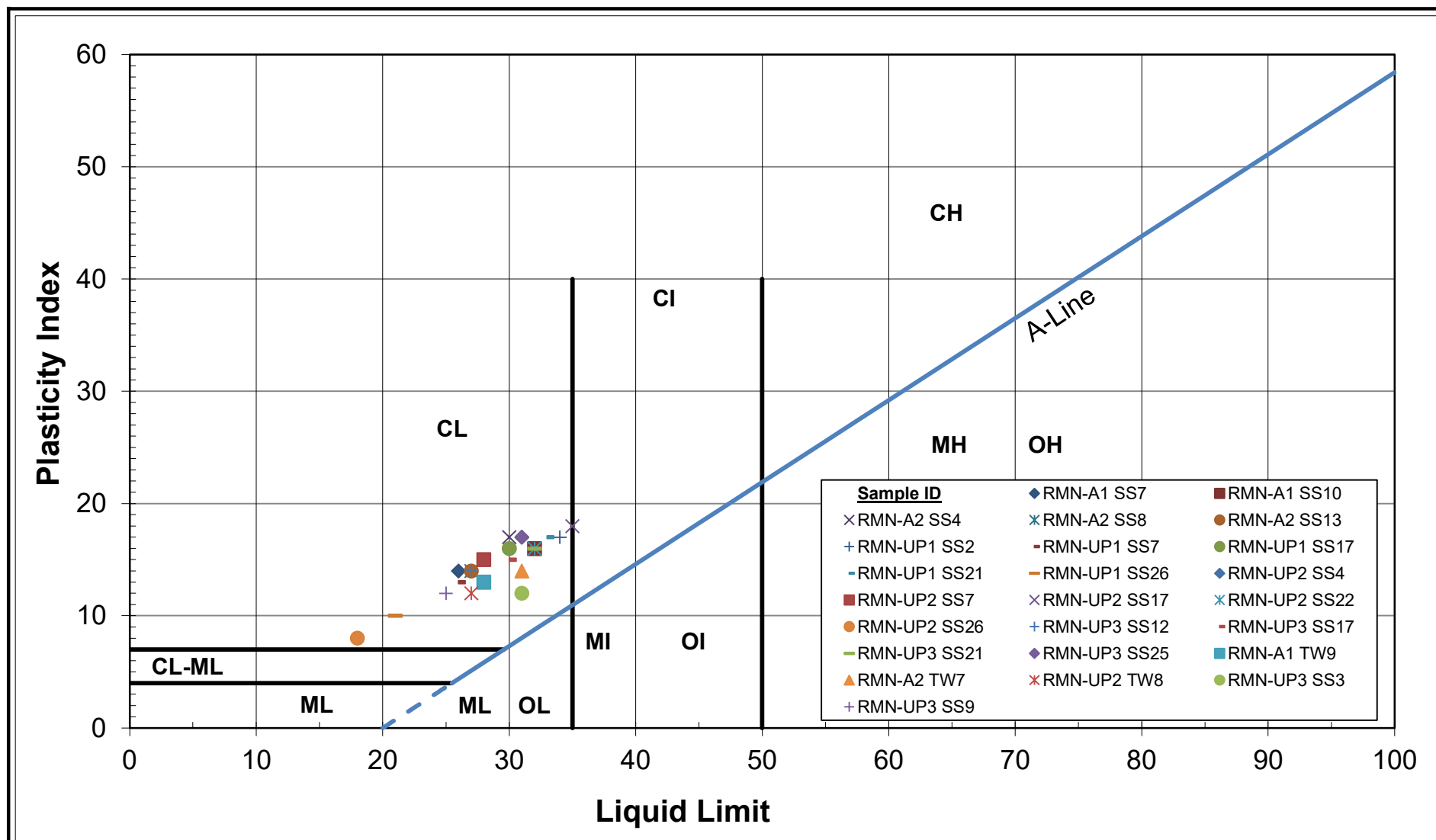
Unified Soil Classification System



TILL: Clayey SILT (CL)
 Ministry of Transportation (MTO)
 Talbotville Bypass - Ron McNeil Line Overpass

Figure No. D3

Project No. 165001308



TILL: Clayey SILT (CL)

Ministry of Transportation (MTO)

Talbotville Bypass - Ron McNeil Underpass

Figure No. D4

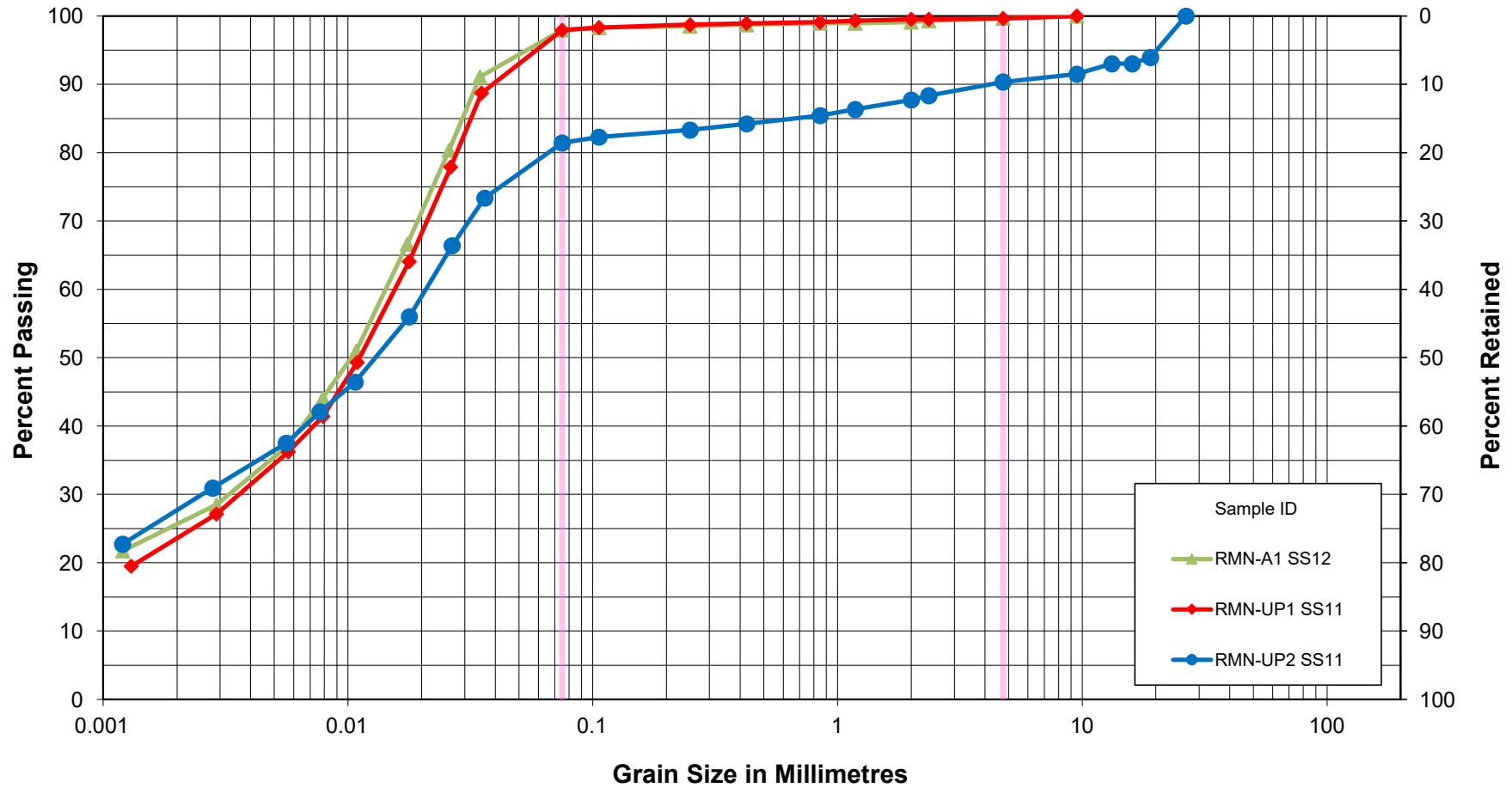
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Unified Soil Classification System

CLAY & SILT	SAND			Gravel	
	Fine	Medium	Coarse	Fine	Coarse

U.S. Std. Sieve No.

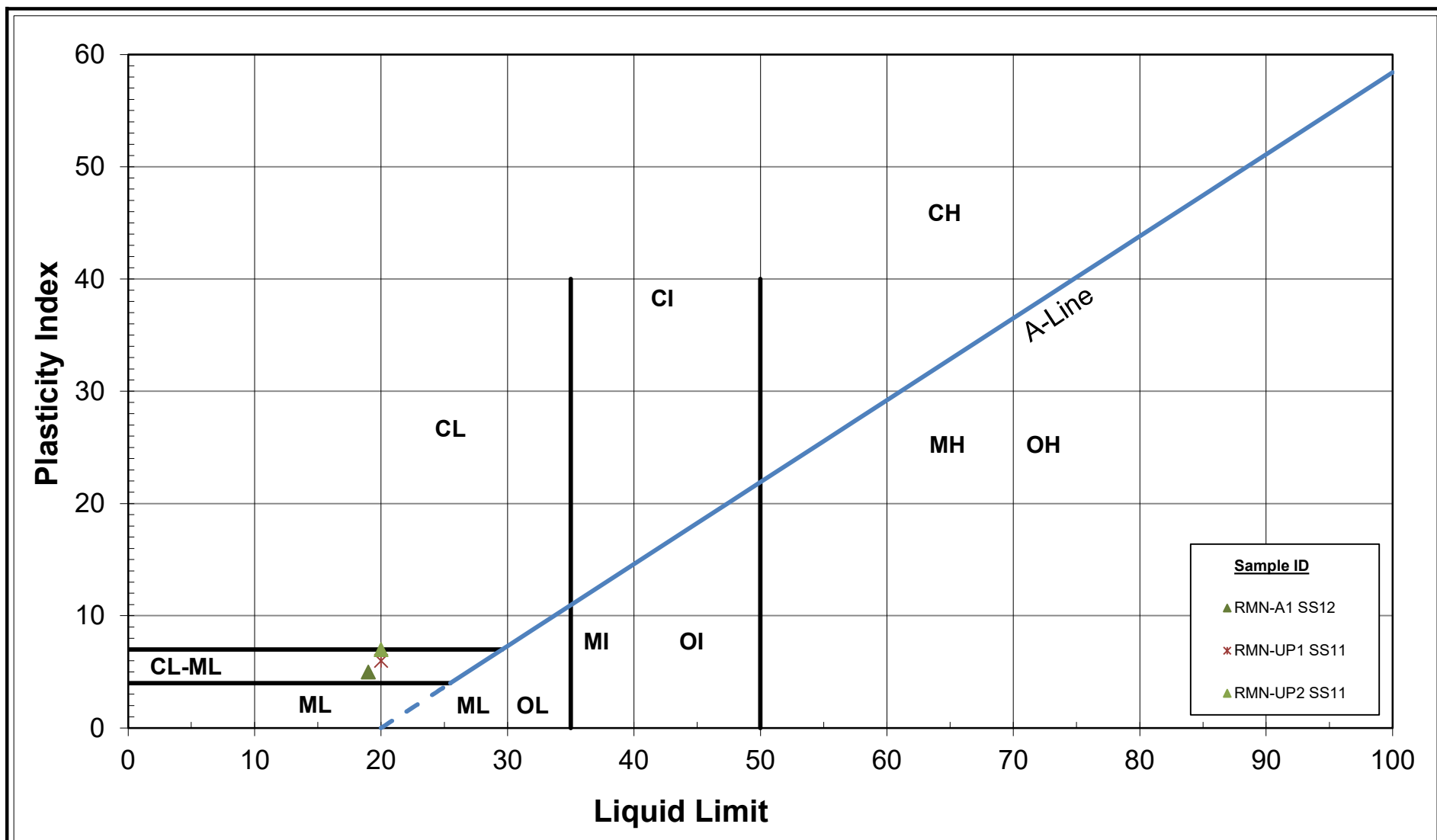
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Clayey SILT (CL-ML)
Ministry of Transportation (MTO)
Talbotville Bypass - Ron McNeil Line Overpass

Figure No. D5

Project No. 165001308



Clayey SILT (CL-ML)
Ministry of Transportation (MTO)
Talbotville Bypass - Ron McNeil Underpass

Figure No. D6

Project No. 165001308

**UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST
(ASTM D2850)**

Tables 1-4

MTO Hwy 3 Talbotville Bypass
SPECIMEN IDENTIFICATION

Borehole/Sample No.:	BH RMN UP2, TW8	Sample Type:	Intact
Sample Depth (ft):	17.7-19.5	Soil Classification:	CL
Liquid Limit:	27.4%	Specific Gravity:	2.764
Plastic limit:	14.8%		
Soil Description & Classification:	<i>Lean clay of low plasticity, stiff to very stiff, brown, moist, CL</i>		

INITIAL SPECIMEN DIMENSIONS AND PROPERTIES

Test No	3	4
Specimen Height, (mm)	152	152
Specimen Diameter, (mm)	70	70
Natural Water Content (Cuttings), (%)	11.4	17.9
Void Ratio	0.41	0.51
Degree of Saturation, (%)	76.3	96.4
Dry Unit Weight (kN/m ³)	19.12	17.85

SHEARING/FAILURE

Max. Deviator Stress, ($\sigma_1 - \sigma_3$), (kPa)	117.1	212.0
Axial Strain At Maximum ($\sigma_1 - \sigma_3$), (%)	15.00	15.01
Compressive Strength, Max, (kN)	0.5	1.0
Max Total Principal Stress Ratio, (σ_1 / σ_3)	2.2	1.6
Deviator Stress At (σ_1 / σ_3) Max, (kPa)	117.1	212.0
Total Major Principal Stress At Failure, σ_1 , (kPa)	217.1	551.1
Total Minor Principal Stress At Failure, σ_3 , (kPa)	100.0	339.1
Average Rate of Strain, (%/min)	1.00	1.00

Test Notes: **Top of tube (Tes# 3) is stiff, Bottom of tube (Tes# 4) is very stiff**

Specimen Saturation Method	N/A	N/A
Failure Criterion	15% A. Strain	15% A. Strain
Membrane Thickness Correction Applied, Y/N	Y	Y

Project No.: 165001308.451.200

Date: August 19, 2024



Prepared By : DB

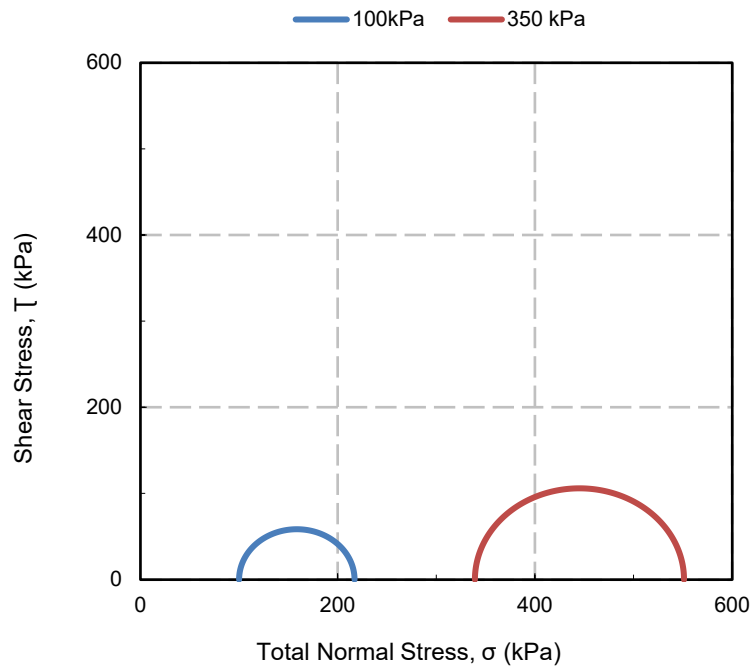
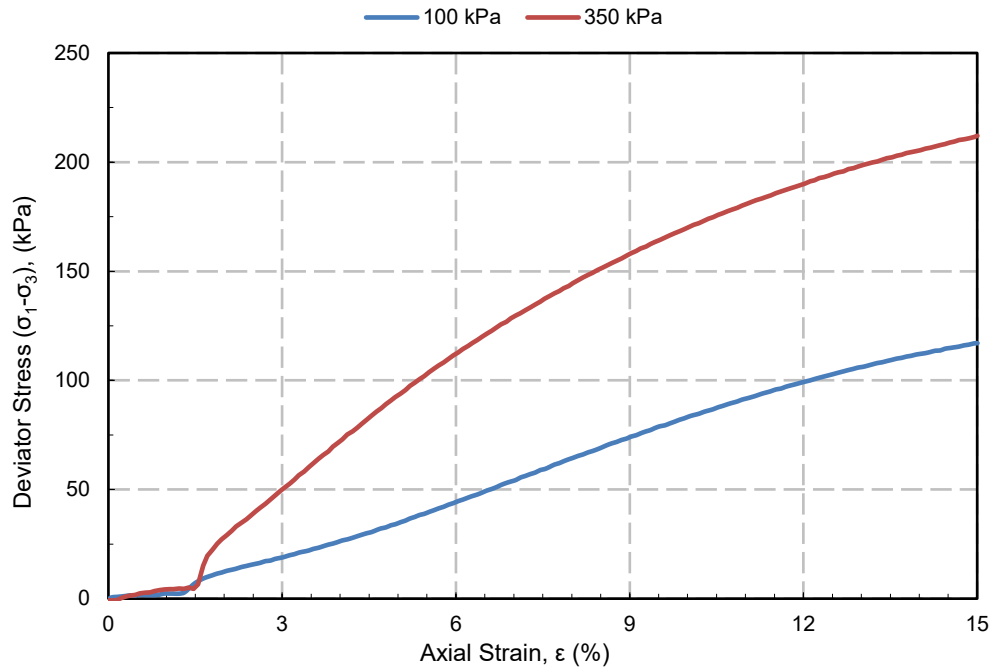
Checked By : RG

UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST (ASTM D2850)

Figures 1-2

MTO Hwy 3 Talbotville Bypass

BH RMN UP2, ST1



Project No.: 165001308.451.200
Date: August 19, 2024



Prepared By : DB
Checked By : RG

MTO Hwy 3 Talbotville Bypass

Lean clay of low plasticity, stiff to very stiff, brown, moist, CL

BH RMN UP2, ST1



Project No. : 165001308.451.200

Date : August 19, 2024



Prepared by : DB

Checked by : RG

MTO Hwy 3 Talbotville Bypass

Lean clay of low plasticity, stiff to very stiff, brown, moist, CL

BH RMN UP2, ST1



100 kPa Shearing



350 kPa Shearing


Project No. : 165001308.451.200

Date : August 19, 2024



Prepared by : DB

Checked by : RG

CONSOLIDATION TEST SUMMARY								
SAMPLE IDENTIFICATION								
Borehole No. :	BH RMN-A1	Sample No. :	TW9					
		Sample Depth (ft) :	20-21.5					
TEST CONDITIONS								
Test Type :	ASTM D2435/D2435M	Date Started :	21-Aug-24					
Load Duration (hr) :	Method B	Date Completed :	23-Aug-24					
SAMPLE DIMENSIONS AND PROPERTIES _ INITIAL								
Sample Height (mm) :	20.50	Unit Weight (kN/m ³) :	21.86					
Sample Diameter (mm) :	50.00	Dry Unit Weight (kN/m ³) :	19.01					
Area (cm ²) :	19.63	Specific Gravity (Assumed)	2.757					
Volume (cm ³) :	40.25	Solid Height (mm) :	14.41					
Water Content (%) :	14.98	Volume of Solids (cm ³) :	28.30					
Wet Mass (g) :	89.71	Volume of Voids (cm ³) :	11.95					
Dry Mass (g) :	78.02	Degree of Saturation (%) :	97.80					
TEST COMPUTATIONS								
Axial Stress (kPa)	Height (H) (mm)	Corrected Deformation (ΔH) (mm)	Axial Strain (ε _a) (%)	Void Ratio e	t ₉₀ (sec)	C _v (cm ² /s)	m _v (m ² /kN)	k (m/s)
0	20.5000			0.422				
10	20.3196	0.1804	1.09	0.407	105.83	8.29E-03	1.09E-03	8.90E-09
20	20.1592	0.3408	1.89	0.396	211.89	4.07E-03	7.93E-04	3.17E-09
40	19.9201	0.5799	3.08	0.379	280.29	3.01E-03	5.96E-04	1.76E-09
80	19.6822	0.8178	4.24	0.362	415.70	1.99E-03	2.91E-04	5.67E-10
160	19.4556	1.0444	5.37	0.346	377.65	2.14E-03	1.41E-04	2.96E-10
240	19.2844	1.2156	6.02	0.337	656.02	1.21E-03	8.16E-05	9.67E-11
160			5.99	0.337				
80			5.81	0.340				
160	19.2862	1.2138	5.94	0.338	169.20	4.66E-03	1.60E-05	7.34E-11
240	19.2513	1.2487	6.14	0.335	284.52	2.76E-03	2.51E-05	6.80E-11
480	19.0410	1.4590	7.35	0.318	301.02	2.57E-03	5.05E-05	1.27E-10
960	18.7427	1.7573	8.83	0.297	240.74	3.13E-03	3.08E-05	9.44E-11
1920	18.4125	2.0875	10.48	0.273	209.62	3.47E-03	1.71E-05	5.84E-11
3840	18.0269	2.4731	12.34	0.247	218.55	3.20E-03	9.71E-06	3.05E-11
4800	17.8897	2.6103	12.94	0.238	286.59	2.38E-03	6.21E-06	1.45E-11
1920			12.67	0.242				
480			11.82	0.254				
80			10.12	0.278				
10			8.46	0.302				
SAMPLE DIMENSIONS AND PROPERTIES _ FINAL								
Sample Height (mm) :	18.77	Unit Weight (kN/m ³) :	23.44					
Sample Diameter (mm) :	50.00	Dry Unit Weight (kN/m ³) :	20.77					
Area (cm ²) :	19.63	Specific Gravity (Assumed) :	2.757					
Volume (cm ³) :	36.85	Solid Height (mm) :	14.41					
Water Content (%) :	12.89	Volume of Solids (cm ³) :	28.30					
Wet Mass (g) :	88.08	Volume of Voids (cm ³) :	8.55					
Dry Mass (g) :	78.02							
Project No. :	165001308.451.102					Prepared By :	DB	
Date :	24-Aug-24					Checked By :	RG	

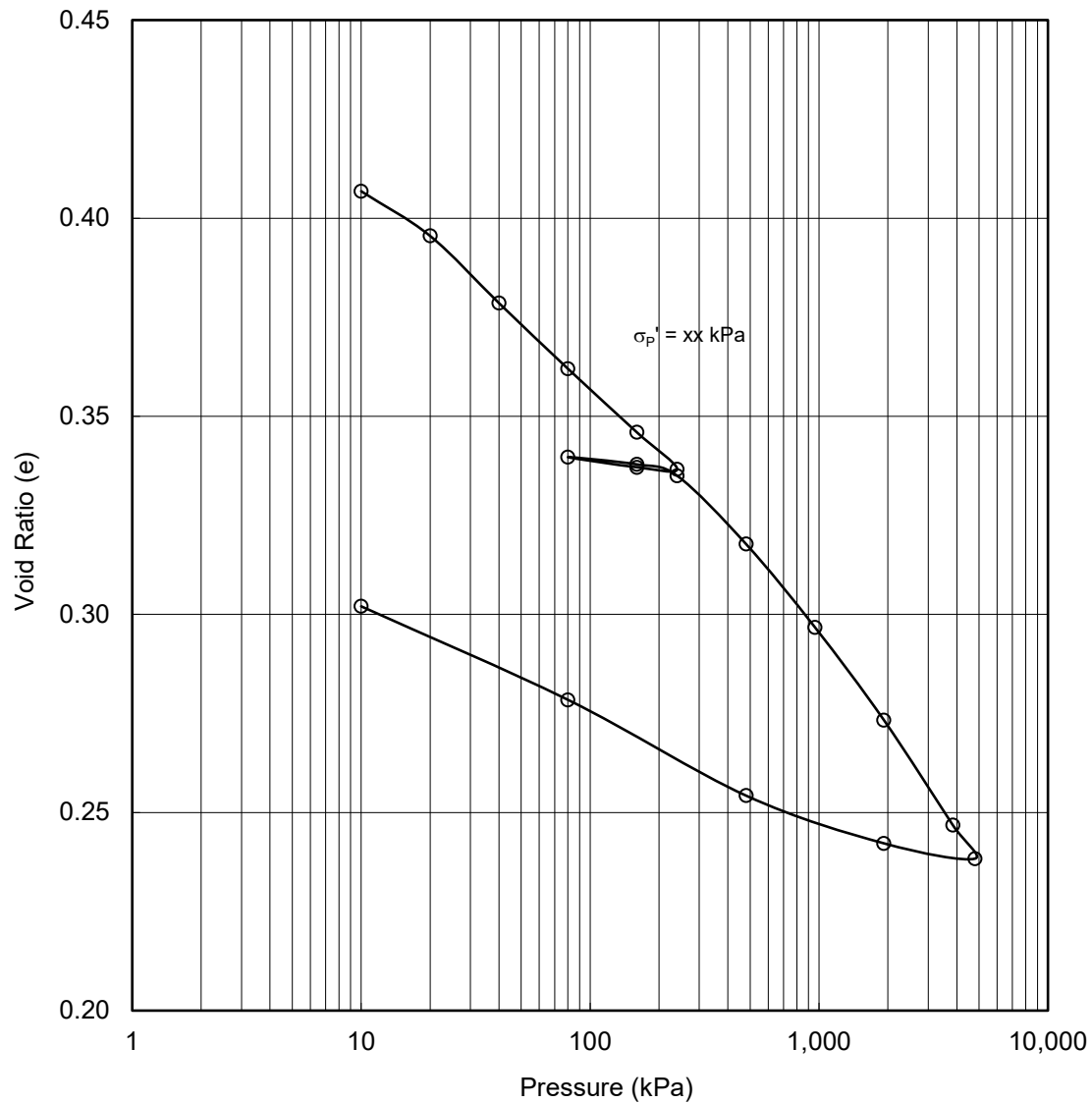
CONSOLIDATION TEST

FIGURE 1

MTO Hwy 3 Talbotville Bypass

BH RMN-A1, TW9

Void Ratio vs Pressure



Soil Type : *Overconsolidated Lean clay of low plasticity, hard, brown, moist, CL*

$e_o =$	0.422	$w_L =$	27.9%	$\sigma_{v0}' =$	kPa
$w =$	15.0%	$w_p =$	15.3%	$\sigma_p' =$	kPa
$\gamma =$	21.9 kN/m ³	PI =	12.6%		
$G_s =$	2.757				

Project No. : 165001308.451.102
Date : 24-Aug-24



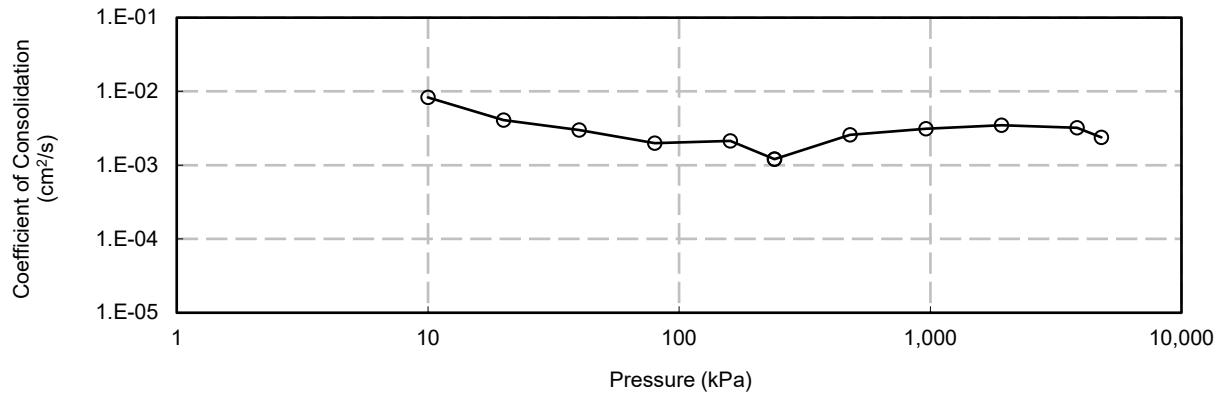
Prepared By : DB
Checked By : RG

CONSOLIDATION TEST

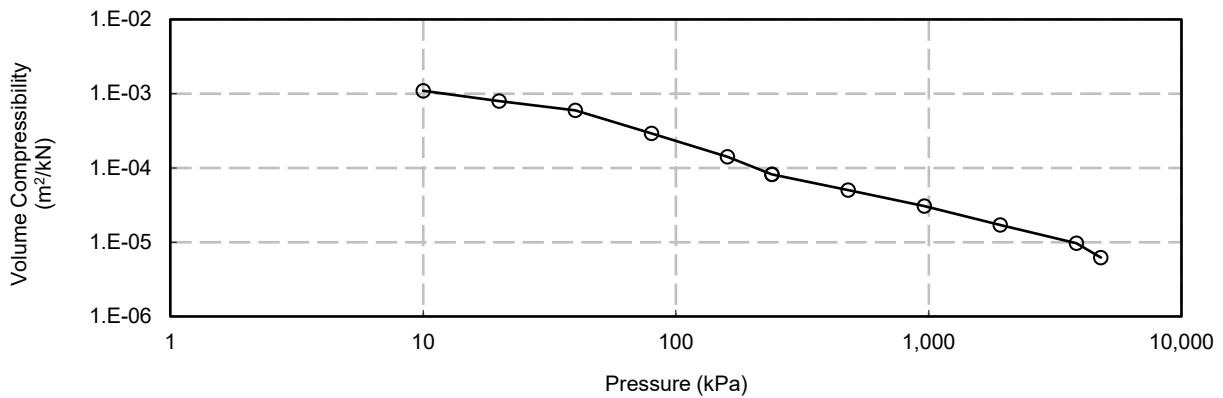
FIGURES 2, 3 & 4

*MTO Hwy 3 Talbotville Bypass
BH RMN-A1, ST1*

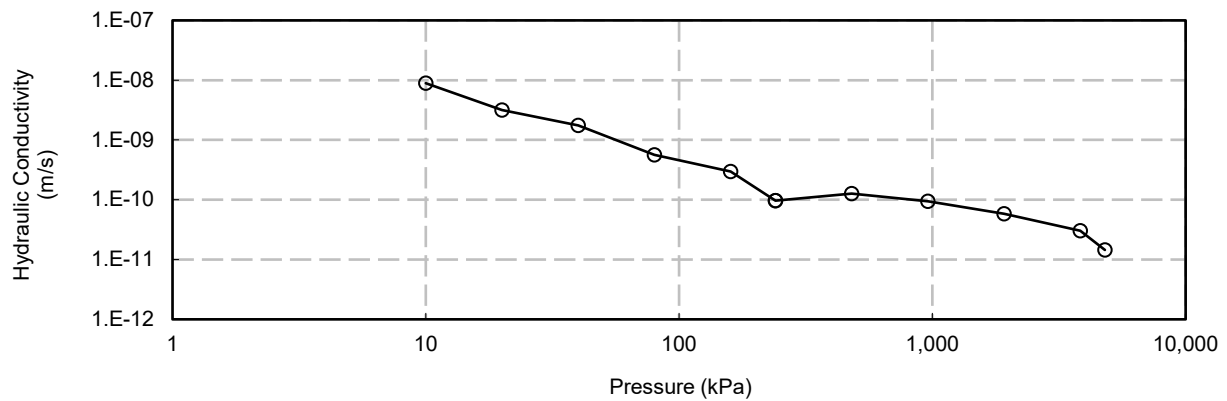
Cv vs Pressure



mv vs Pressure



k vs Pressure



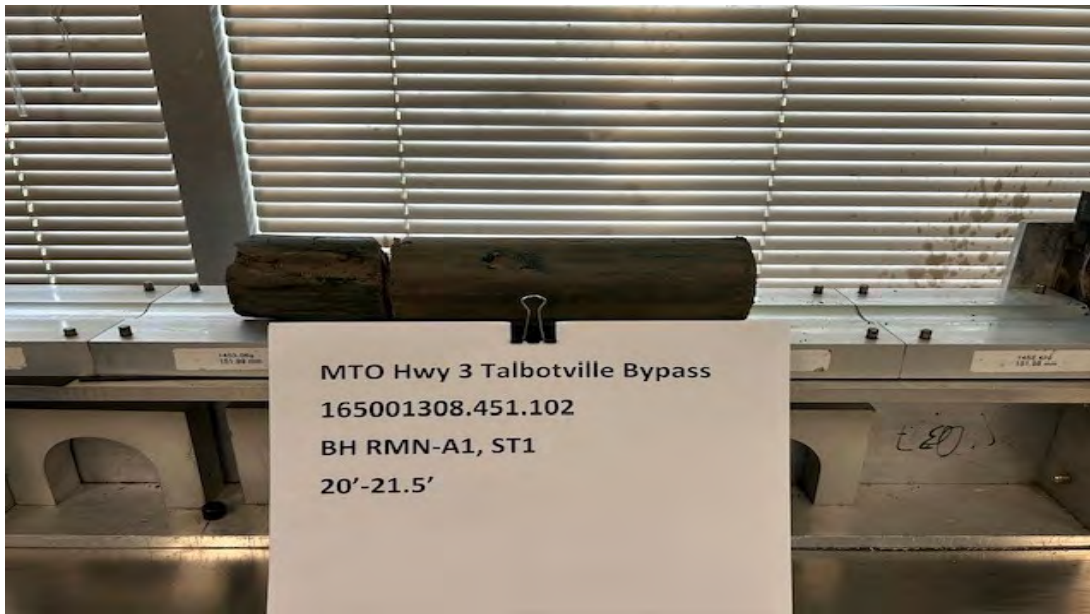
Project No. : 165001308.451.102
Date : 24-Aug-24



Prepared By : DB
Checked By : RG

MTO Hwy 3 Talbotville Bypass

Overconsolidated Lean clay of low plasticity, hard, brown, moist, CL



BH RMN-A1, ST1



BH RMN-A1, ST1


Project No. : 165001308.451.102

Date : 24-Aug-2024



Prepared by : DB

Checked by : RG

CONSOLIDATION TEST SUMMARY								
SAMPLE IDENTIFICATION								
Borehole No. :	BH RMN-A2	Sample No. :	TW7					
		Sample Depth (ft) :	15-16.5					
TEST CONDITIONS								
Test Type :	ASTM D2435/D2435M	Date Started :	21-Aug-24					
Load Duration (hr) :	Method B	Date Completed :	23-Aug-24					
SAMPLE DIMENSIONS AND PROPERTIES _ INITIAL								
Sample Height (mm) :	20.50	Unit Weight (kN/m ³) :	21.42					
Sample Diameter (mm) :	50.00	Dry Unit Weight (kN/m ³) :	18.35					
Area (cm ²) :	19.63	Specific Gravity (Assumed)	2.761					
Volume (cm ³) :	40.25	Solid Height (mm) :	13.89					
Water Content (%) :	16.76	Volume of Solids (cm ³) :	27.28					
Wet Mass (g) :	87.93	Volume of Voids (cm ³) :	12.98					
Dry Mass (g) :	75.31	Degree of Saturation (%) :	97.26					
TEST COMPUTATIONS								
		Corrected	Axial	Void Ratio	t ₉₀	C _v	m _v	k
Axial Stress	Height (H)	Deformation (ΔH)	Strain (ε _a)	e	(sec)	(cm ² /s)	(m ² /kN)	(m/s)
(kPa)	(mm)	(mm)	(%)					
0	20.5000			0.476				
10	20.3736	0.1264	0.65	0.466	106.79	8.25E-03	6.49E-04	5.26E-09
20	20.3343	0.1657	0.87	0.463	297.16	2.95E-03	2.18E-04	6.32E-10
40	20.2510	0.2490	1.33	0.456	338.70	2.57E-03	2.31E-04	5.84E-10
80	20.1172	0.3828	2.03	0.446	435.55	1.98E-03	1.75E-04	3.39E-10
160	19.9207	0.5793	2.98	0.432	566.56	1.49E-03	1.19E-04	1.74E-10
240	19.7944	0.7056	3.61	0.422	400.58	2.08E-03	7.91E-05	1.61E-10
160			3.51	0.424				
80			3.21	0.428				
160	19.8064	0.6936	3.42	0.425	179.93	4.63E-03	2.70E-05	1.23E-10
240	19.7556	0.7444	3.71	0.421	246.83	3.36E-03	3.53E-05	1.16E-10
480	19.5415	0.9585	4.94	0.403	294.62	2.77E-03	5.16E-05	1.40E-10
960	19.1957	1.3043	6.64	0.378	320.48	2.46E-03	3.53E-05	8.53E-11
1920	18.7652	1.7348	8.77	0.346	311.48	2.43E-03	2.22E-05	5.29E-11
3840	18.2609	2.2391	11.24	0.310	234.54	3.07E-03	1.29E-05	3.89E-11
4800	18.0867	2.4133	12.00	0.299	284.38	2.45E-03	7.96E-06	1.91E-11
1920			11.66	0.304				
480			9.93	0.329				
80			7.57	0.364				
10			5.31	0.397				
SAMPLE DIMENSIONS AND PROPERTIES _ FINAL								
Sample Height (mm) :	19.41	Unit Weight (kN/m ³) :	22.35					
Sample Diameter (mm) :	50.00	Dry Unit Weight (kN/m ³) :	19.38					
Area (cm ²) :	19.63	Specific Gravity (Assumed) :	2.761					
Volume (cm ³) :	38.11	Solid Height (mm) :	13.89					
Water Content (%) :	15.36	Volume of Solids (cm ³) :	27.28					
Wet Mass (g) :	86.88	Volume of Voids (cm ³) :	10.84					
Dry Mass (g) :	75.31							
<div style="display: flex; justify-content: space-between; align-items: flex-end;"> <div> Project No. : 165001308.451.102 Date : 24-Aug-24 </div> <div style="text-align: center;">  </div> <div style="text-align: right;"> Prepared By : DB Checked By : RG </div> </div>								

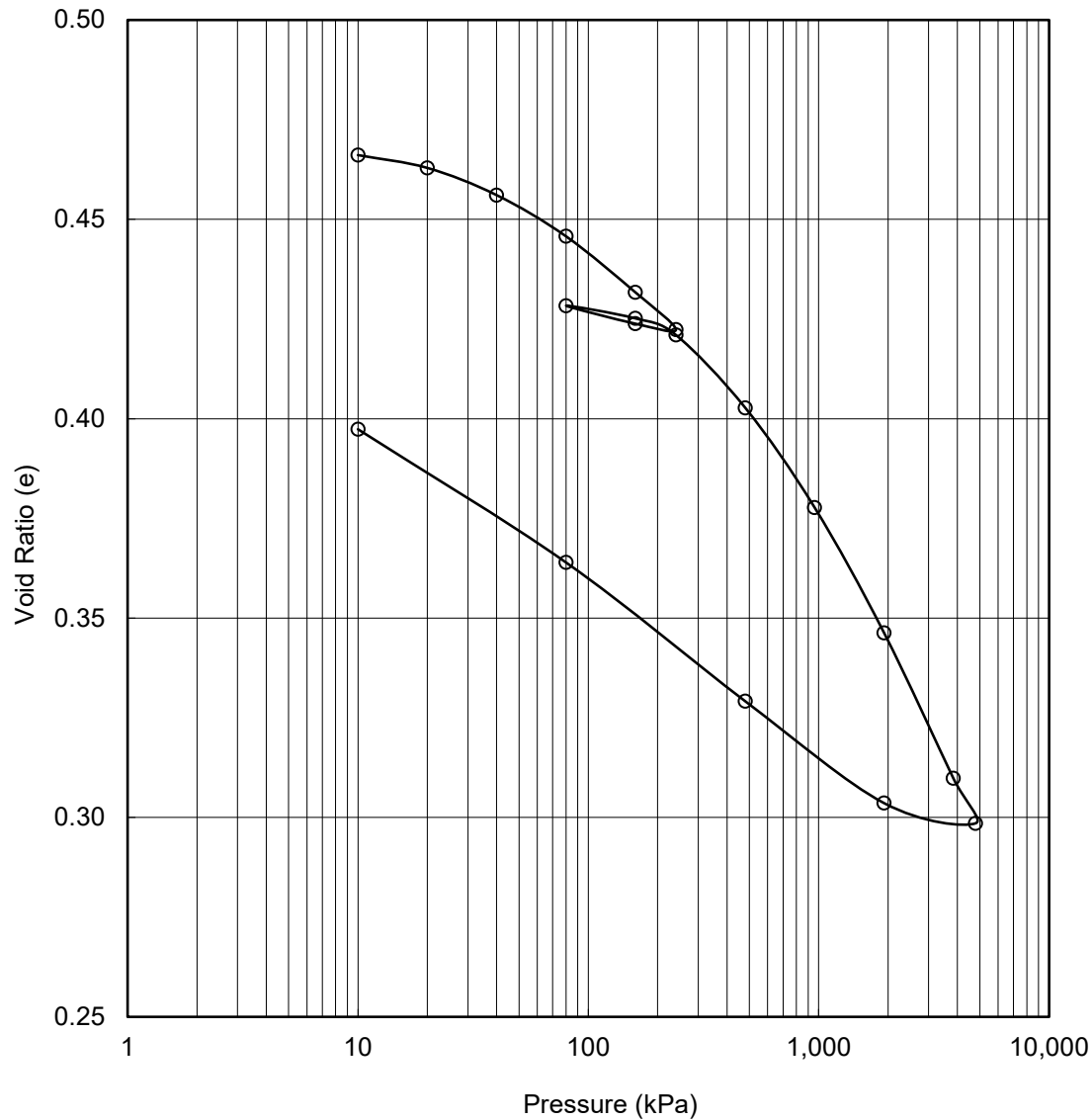
CONSOLIDATION TEST

FIGURE 1

MTO Hwy 3 Talbotville Bypass

BH RMN-A2, TW7

Void Ratio vs Pressure



Soil Type : *Overconsolidated Lean clay of low plasticity, hard, brown, moist, CL*

$e_o =$	0.476	$w_L =$	30.7%	$\sigma_{v0}' =$	kPa
$w =$	16.8%	$w_p =$	16.6%	$\sigma_p' =$	kPa
$\gamma =$	21.4 kN/m ³	$PI =$	14.1%		
$G_s =$	2.761				

Project No. : 165001308.451.102
Date : 24-Aug-24



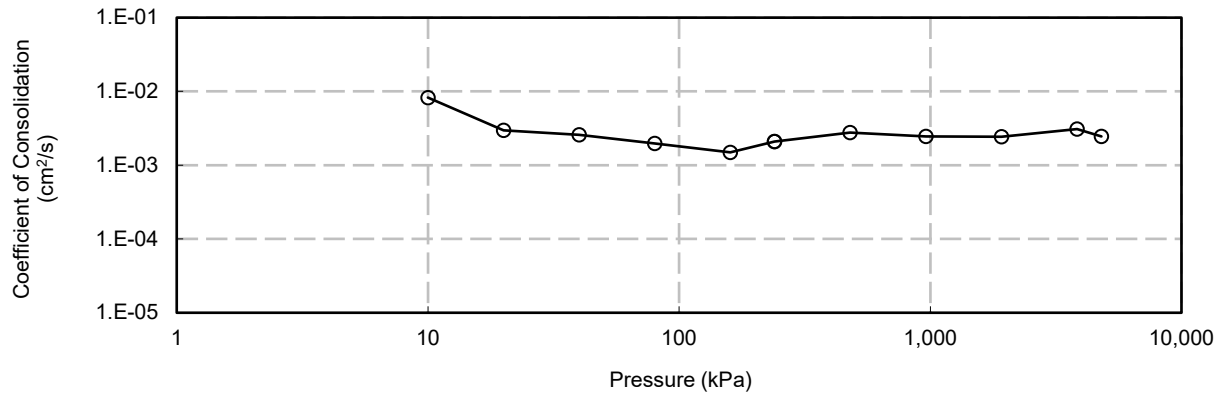
Prepared By : DB
Checked By : RG

CONSOLIDATION TEST

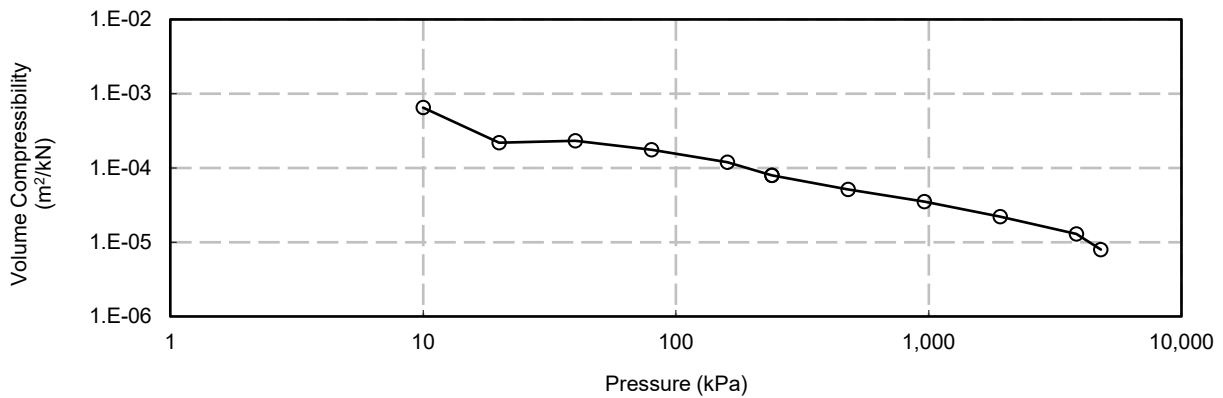
FIGURES 2, 3 & 4

*MTO Hwy 3 Talbotville Bypass
BH RMN-A2, ST1*

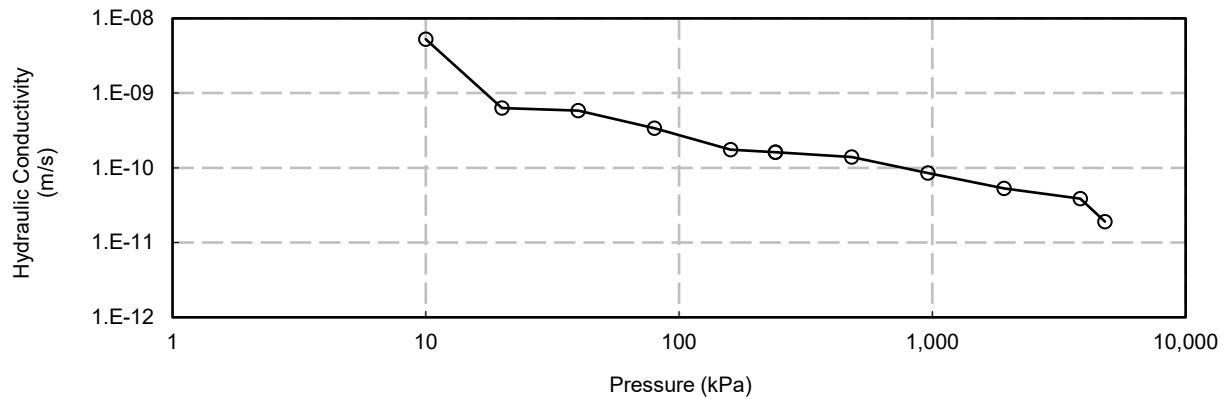
Cv vs Pressure



mv vs Pressure



k vs Pressure



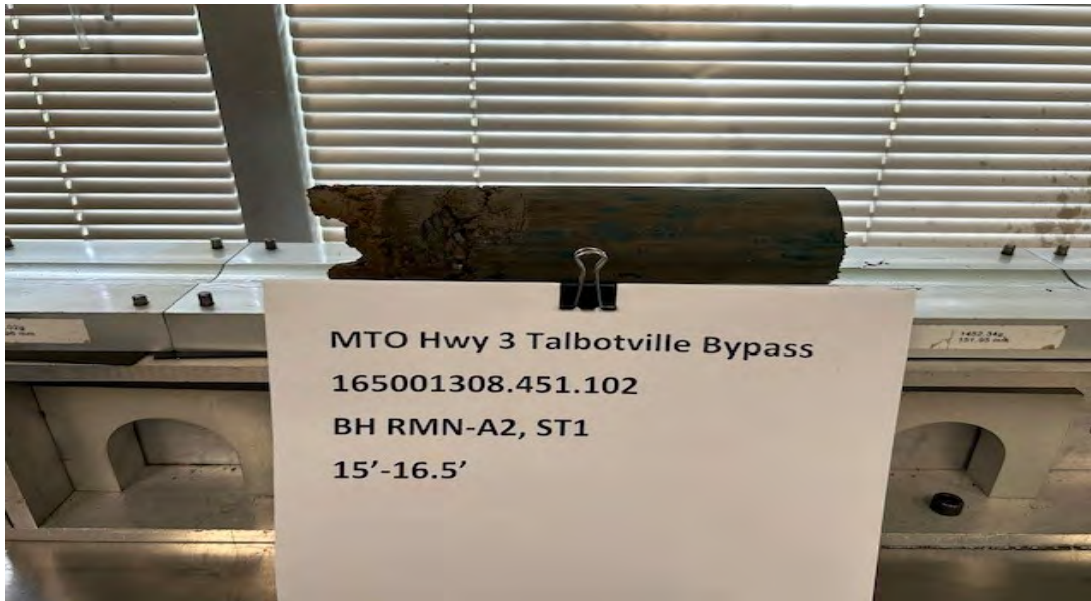
Project No. : 165001308.451.102
Date : 24-Aug-24



Prepared By : DB
Checked By : RG

MTO Hwy 3 Talbotville Bypass

Overconsolidated Lean clay of low plasticity, hard, brown, moist, CL



BH RMN-A2, ST1



BH RMN-A2, ST1

Project No. : 165001308.451.102

Date : 24-Aug-2024



Prepared by : DB

Checked by : RG

CONSOLIDATION TEST SUMMARY								
SAMPLE IDENTIFICATION								
Borehole No. :	BH RMN UP2				Sample No. :	TW8		
					Sample Depth (ft) :	17.5-19.5		
TEST CONDITIONS								
Test Type :	ASTM D2435/D2435M				Date Started :	12-Aug-24		
Load Duration (hr) :	Method B				Date Completed :	14-Aug-24		
SAMPLE DIMENSIONS AND PROPERTIES _ INITIAL								
Sample Height (mm) :	20.50				Unit Weight (kN/m ³) :	20.91		
Sample Diameter (mm) :	50.00				Dry Unit Weight (kN/m ³) :	17.76		
Area (cm ²) :	19.63				Specific Gravity :	2.764		
Volume (cm ³) :	40.25				Solid Height (mm) :	13.43		
Water Content (%) :	17.74				Volume of Solids (cm ³) :	26.37		
Wet Mass (g) :	85.83				Volume of Voids (cm ³) :	13.88		
Dry Mass (g) :	72.90				Degree of Saturation (%) :	93.18		
TEST COMPUTATIONS								
		Corrected	Axial	Void Ratio	t_{90}	C_v	m_v	k
Axial Stress	Height (H)	Deformation (ΔH)	Strain (ϵ_a)	e	(sec)	(cm ² /s)	(m ² /kN)	(m/s)
(kPa)	(mm)	(mm)	(%)					
0	20.5000	0.0000	0.00	0.526				
5	20.2699	0.2301	1.63	0.501	122.69	7.13E-03	3.26E-03	2.28E-08
10	20.1179	0.3821	2.27	0.491	155.31	5.53E-03	1.28E-03	6.97E-09
20	19.9500	0.5500	3.15	0.478	181.00	4.67E-03	8.72E-04	4.00E-09
40	19.7376	0.7624	4.22	0.462	378.22	2.19E-03	5.39E-04	1.16E-09
80	19.4821	1.0179	5.52	0.442	336.66	2.40E-03	3.24E-04	7.64E-10
160	19.1566	1.3434	6.96	0.420	476.74	1.65E-03	1.80E-04	2.91E-10
320	18.8677	1.6323	8.57	0.395	201.22	3.78E-03	1.00E-04	3.72E-10
160			8.46	0.397				
80			8.24	0.400				
160	18.7889	1.7111	8.38	0.398	134.55	5.57E-03	1.76E-05	9.62E-11
320	18.7222	1.7778	8.76	0.392	247.60	3.01E-03	2.38E-05	7.03E-11
640	18.4622	2.0378	10.26	0.370	345.62	2.11E-03	4.70E-05	9.73E-11
1280	18.0823	2.4177	12.07	0.342	332.50	2.11E-03	2.82E-05	5.85E-11
2560	17.7236	2.7764	13.92	0.314	198.73	3.40E-03	1.45E-05	4.82E-11
640			13.56	0.319				
160			12.74	0.332				
40			11.52	0.350				
10			10.48	0.366				
5			9.93	0.375				
SAMPLE DIMENSIONS AND PROPERTIES _ FINAL								
Sample Height (mm) :	18.46				Unit Weight (kN/m ³) :	22.36		
Sample Diameter (mm) :	50.00				Dry Unit Weight (kN/m ³) :	19.72		
Area (cm ²) :	19.63				Specific Gravity :	2.764		
Volume (cm ³) :	36.25				Solid Height (mm) :	13.43		
Water Content (%) :	13.40				Volume of Solids (cm ³) :	26.37		
Wet Mass (g) :	82.67				Volume of Voids (cm ³) :	9.88		
Dry Mass (g) :	72.90							
Project No. :	165001308.451.102				Prepared By :	DB		
Date :	19-Aug-24				Checked By :	RG		



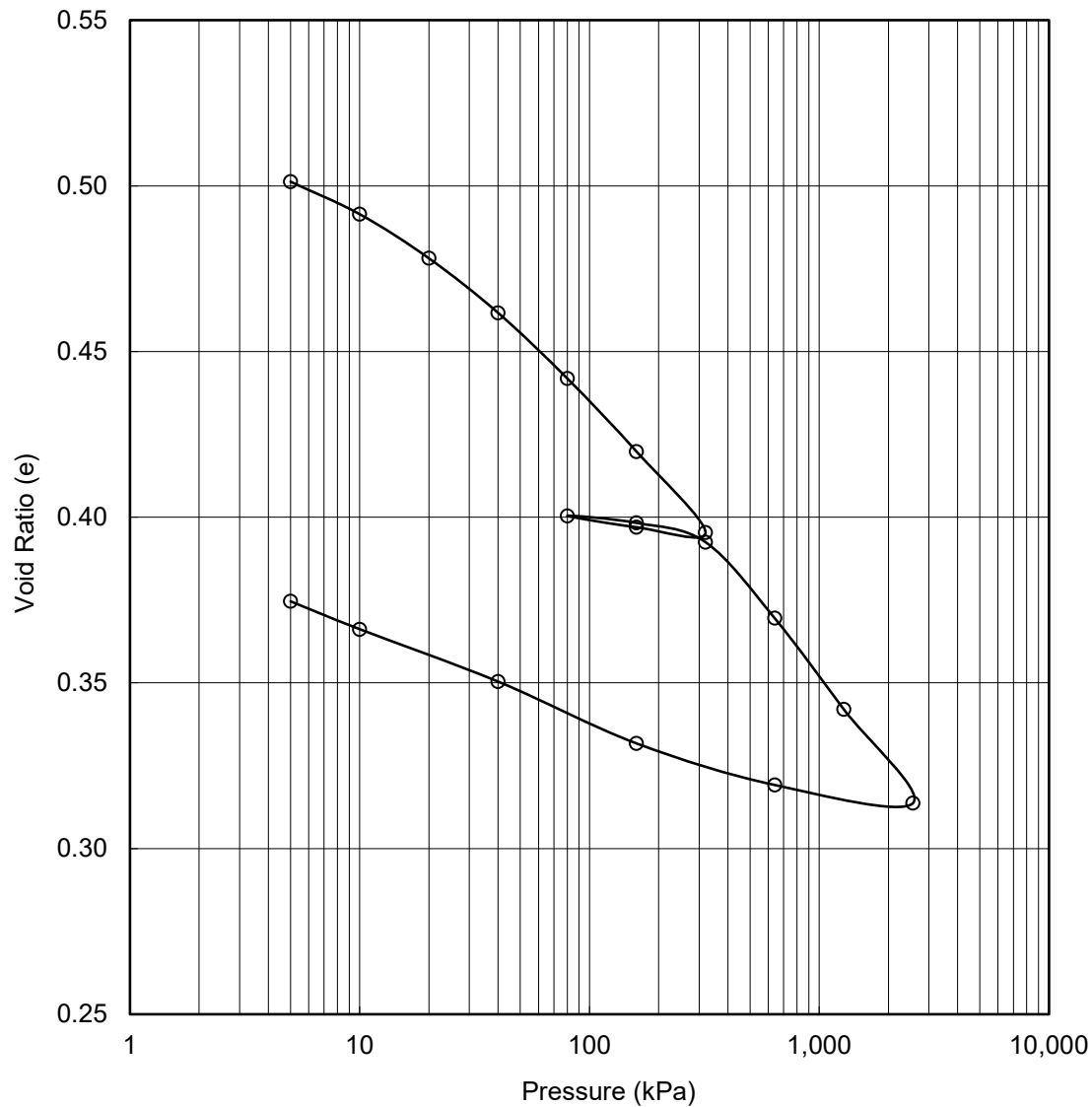
CONSOLIDATION TEST

FIGURE 1

MTO Hwy 3 Talbotville Bypass

BH RMN UP2, TW8

Void Ratio vs Pressure



Soil Type : *Overconsolidated Lean clay of low plasticity , v. stiff to hard, brown, moist, -CL*

$e_o =$	0.526	$w_L =$	27.4%	$\sigma_{v0}' =$	kPa
$w =$	17.7%	$w_p =$	14.8%	$\sigma_p' =$	kPa
$\gamma =$	20.9 kN/m ³	$PI =$	12.6%		
$G_s =$	2.764				

Project No. : 165001308.451.102
Date : 19-Aug-24



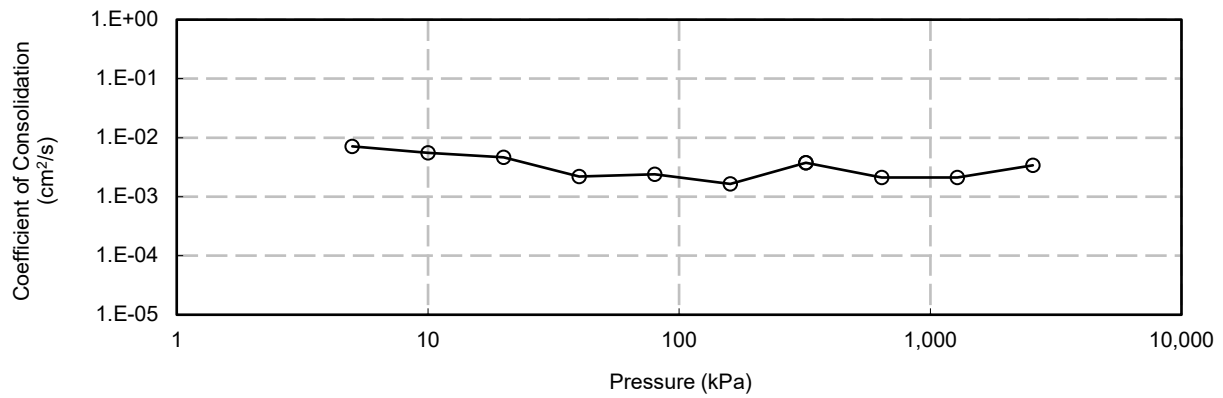
Prepared By : DB
Checked By : RG

CONSOLIDATION TEST

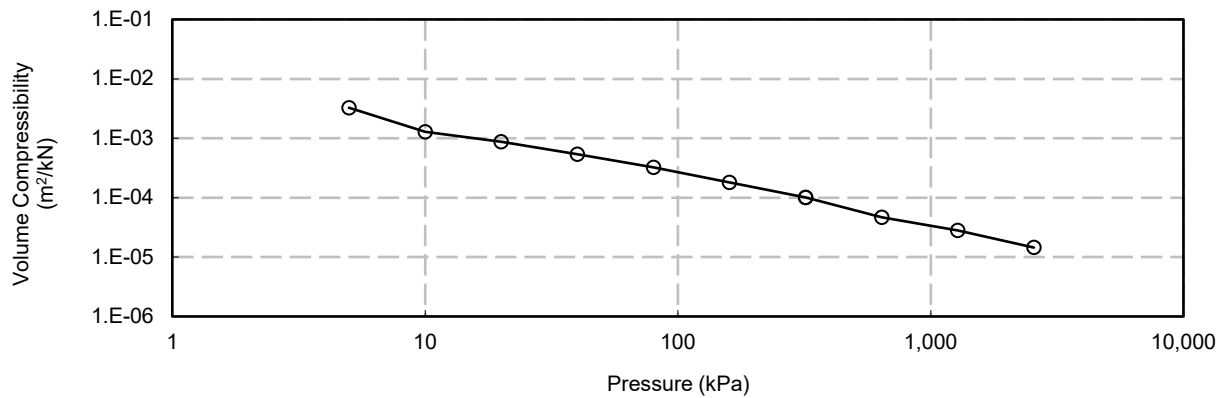
FIGURES 2, 3 & 4

*MTO Hwy 3 Talbotville Bypass
BH RMN UP2, ST1*

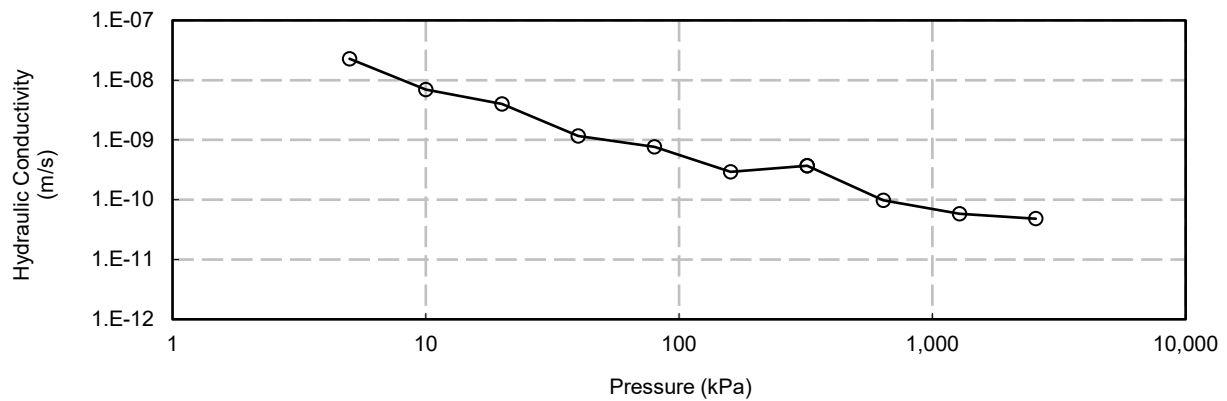
Cv vs Pressure



mv vs Pressure



k vs Pressure



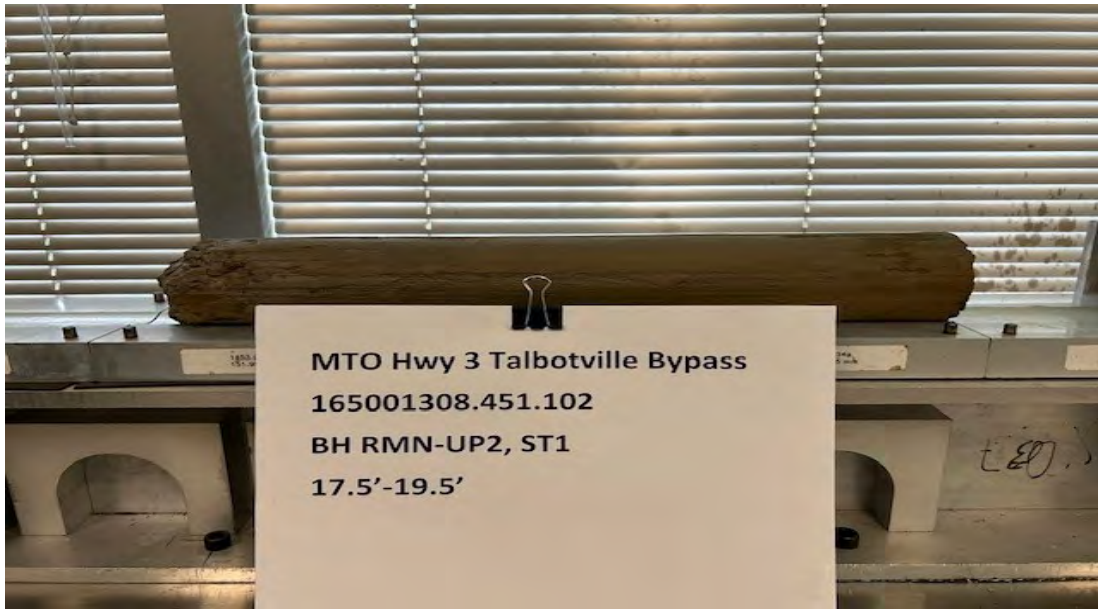
Project No. : 165001308.451.102
Date : 19-Aug-24



Prepared By : DB
Checked By : RG

MTO Hwy 3 Talbotville Bypass

Overconsolidated Lean clay of low plasticity , v. stiff to hard, brown, moist, -CL



BH RMN UP2, ST1



BH RMN UP2, ST1

Project No. : 165001308.451.102

Date : 19-Aug-2024



Prepared by : DB

Checked by : RG

CLIENT NAME: STANTEC CONSULTING LTD
300-675 Cochrane Drive
MARKHAM, ON L3R0B8
(905) 444-7777

ATTENTION TO: Bahram Siavash

PROJECT: 165001308.551.102

AGAT WORK ORDER: 24T167277

ROCK ANALYSIS REVIEWED BY: Jewel Shibu, Lab Supervisor

SOIL ANALYSIS REVIEWED BY: Sukhwinder Randhawa, Inorganic Team Lead

DATE REPORTED: Jul 05, 2024

PAGES (INCLUDING COVER): 7

VERSION*: 1

Should you require any information regarding this analysis please contact your client services representative at (403) 735-2005

*Notes

Disclaimer:

- All work conducted herein has been done using accepted standard protocols, and generally accepted practices and methods. AGAT test methods may incorporate modifications from the specified reference methods to improve performance.
- All samples will be disposed of within 30 days after receipt unless a Long Term Storage Agreement is signed and returned. Some specialty analysis may be exempt, please contact your Client Project Manager for details.
- AGAT's liability in connection with any delay, performance or non-performance of these services is only to the Client and does not extend to any other third party. Unless expressly agreed otherwise in writing, AGAT's liability is limited to the actual cost of the specific analysis or analyses included in the services.
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- The test results reported herewith relate only to the samples as received by the laboratory.
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- All reportable information is available on request from AGAT Laboratories, in accordance with ISO/IEC 17025:2017, ISO/IEC 17025:2005 (Quebec), DR-12-PALA and/or NELAP Standards.
- This document is signed by an authorized signatory who meets the requirements of the MELCCFP, CALA, CCN and NELAP.
- For environmental samples in the Province of Quebec: The analysis is performed on and results apply to samples as received. A temperature above 6°C upon receipt, as indicated in the Sample Reception Notification (SRN), could indicate the integrity of the samples has been compromised if the delay between sampling and submission to the laboratory could not be minimized.



Certificate of Analysis

AGAT WORK ORDER: 24T167277

PROJECT: 165001308.551.102

2910 12TH STREET NE
CALGARY, ALBERTA
CANADA T2E 7P7
TEL (403)735-2005
FAX (403)735-2771
<http://www.agatlabs.com>

CLIENT NAME: STANTEC CONSULTING LTD

ATTENTION TO: Bahram Siavash

SAMPLING SITE:

SAMPLED BY:

(284-137) Sulfide (CGY)

DATE RECEIVED: 2024-06-27

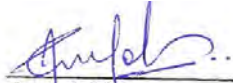
DATE REPORTED: 2024-07-05

		SAMPLE DESCRIPTION:		WR-UP3-SS7	RMN-UP3-SS8	CNR-OH1-SS8	CNR-OH2-SS5	CNR-OH3-SS4
		SAMPLE TYPE:		Soil	Soil	Soil	Soil	Soil
		DATE SAMPLED:		2024-06-26	2024-06-26	2024-06-26	2024-06-26	2024-06-26
Parameter	Unit	G / S	RDL	5964762	5964839	5964840	5964841	5964842
Sulfide	%	0.01	0.08	0.14	0.16	<0.01	<0.01	

Comments: RDL - Reported Detection Limit; G / S - Guideline / Standard

Analysis performed at AGAT Calgary (unless marked by *)

Certified By:


Jewel Shibu



Certificate of Analysis

AGAT WORK ORDER: 24T167277

PROJECT: 165001308.551.102

2910 12TH STREET NE
CALGARY, ALBERTA
CANADA T2E 7P7
TEL (403)735-2005
FAX (403)735-2771
<http://www.agatlabs.com>

CLIENT NAME: STANTEC CONSULTING LTD

ATTENTION TO: Bahram Siavash

SAMPLING SITE:

SAMPLED BY:

Corrosivity Package

DATE RECEIVED: 2024-06-27

DATE REPORTED: 2024-07-05

		SAMPLE DESCRIPTION:		WR-UP3-SS7	RMN-UP3-SS8	CNR-OH1-SS8	CNR-OH2-SS5	CNR-OH3-SS4
		SAMPLE TYPE:		Soil	Soil	Soil	Soil	Soil
		DATE SAMPLED:		2024-06-26	2024-06-26	2024-06-26	2024-06-26	2024-06-26
Parameter	Unit	G / S	RDL	5964762	5964839	5964840	5964841	5964842
Chloride (2:1)	µg/g	2	127	7	6	15	16	
Sulphate (2:1)	µg/g	2	154	174	194	206	185	
pH (2:1)	pH Units	NA	8.33	8.38	8.48	8.30	8.35	
Electrical Conductivity (2:1)	mS/cm	0.005	0.516	0.281	0.329	0.297	0.342	
Resistivity (2:1) (Calculated)	ohm.cm	1	1940	3560	3040	3370	2920	
Redox Potential 1	mV	NA	127	340	305	139	198	
Redox Potential 2	mV	NA	120	339	278	137	199	
Redox Potential 3	mV	NA	102	318	288	131	199	

Comments: RDL - Reported Detection Limit; G / S - Guideline / Standard

5964762-5964842 EC, pH, Chloride and Sulphate were determined on the extract obtained from the 2:1 leaching procedure (2 parts DI water: 1 part soil). Resistivity is a calculated parameter.

Redox potential measured on as received sample. Due to the potential for rapid change in sample equilibrium chemistry with exposure to oxidative/reduction conditions laboratory results may differ from field measured results.

Redox potential measurement in soil is quite variable and non reproducible due in part, to the general heterogeneity of a given soil. It is also related to the introduction of increased oxygen into the sample after extraction. The interpretation of soil redox potential should be considered in terms of its general range rather than as an absolute measurement.

Analysis performed at AGAT Toronto (unless marked by *)

Certified By:



Quality Assurance

CLIENT NAME: STANTEC CONSULTING LTD

PROJECT: 165001308.551.102

SAMPLING SITE:

AGAT WORK ORDER: 24T167277

ATTENTION TO: Bahram Siavash

SAMPLED BY:

Rock Analysis

RPT Date: Jul 05, 2024			DUPLICATE			Method Blank	REFERENCE MATERIAL		METHOD BLANK SPIKE		MATRIX SPIKE	
PARAMETER	Batch	Sample Id	Dup #1	Dup #2	RPD		Measured Value	Acceptable Limits		Recovery	Acceptable Limits	
								Lower	Upper		Lower	Upper

(284-137) Sulfide (CGY)

Total Sulfur 5964762 5964762 0.10 0.09 11.4% < 0.01 108% 80% 120%

Sulfate 5950778 5950778 0.04 0.04 0.6% < 0.01 87% 80% 120%

Comments: RPDs are calculated using raw analytical data and not the rounded duplicate values reported.

Duplicate/ Replicate NA: Results are less than 10X the RDL and RPD will not be calculated

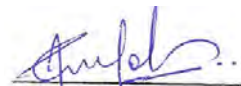
(284-137) Sulfide (CGY)

Sulfate 5964762 5964762 0.02 0.02 2% < 0.01 80% 120%

Comments: RPDs are calculated using raw analytical data and not the rounded duplicate values reported.

Duplicate/ Replicate NA: Results are less than 10X the RDL and RPD will not be calculated

Certified By:


Jewel Shibu

Quality Assurance

CLIENT NAME: STANTEC CONSULTING LTD

PROJECT: 165001308.551.102

SAMPLING SITE:

AGAT WORK ORDER: 24T167277

ATTENTION TO: Bahram Siavash

SAMPLED BY:

Soil Analysis

RPT Date: Jul 05, 2024			DUPLICATE			Method Blank	REFERENCE MATERIAL		METHOD BLANK SPIKE		MATRIX SPIKE	
PARAMETER	Batch	Sample Id	Dup #1	Dup #2	RPD		Measured Value	Acceptable Limits		Recovery	Acceptable Limits	
								Lower	Upper		Lower	Upper

Corrosivity Package

Chloride (2:1)	5961472		21	21	0.0%	< 2	101%	70%	130%	96%	80%	120%	95%	70%	130%
Sulphate (2:1)	5961472		77	77	0.0%	< 2	101%	70%	130%	96%	80%	120%	92%	70%	130%
pH (2:1)	5962742		7.86	7.56	3.9%	NA	97%	80%	120%						
Electrical Conductivity (2:1)	5962742		1.67	1.69	1.2%	< 0.005	103%	80%	120%						
Redox Potential 1	5964762					NA	100%	90%	110%						

Comments: NA signifies Not Applicable.

pH duplicates QA acceptance criteria was met relative as stated in Table 5-15 of Analytical Protocol document.

Corrosivity Package

pH (2:1)	5964762	5964762	8.33	8.00	4.0%	NA	98%	80%	120%
Electrical Conductivity (2:1)	5964762	5964762	0.516	0.509	1.4%	< 0.005	102%	80%	120%

Comments: NA signifies Not Applicable.

pH duplicates QA acceptance criteria was met relative as stated in Table 5-15 of Analytical Protocol document.

Certified By:



Method Summary

CLIENT NAME: STANTEC CONSULTING LTD

AGAT WORK ORDER: 24T167277

PROJECT: 165001308.551.102

ATTENTION TO: Bahram Siavash

SAMPLING SITE:

SAMPLED BY:

PARAMETER	AGAT S.O.P	LITERATURE REFERENCE	ANALYTICAL TECHNIQUE
Soil Analysis			
Chloride (2:1)	INOR-93-6004	modified from SM 4110 B	ION CHROMATOGRAPH
Sulphate (2:1)	INOR-93-6004	modified from SM 4110 B	ION CHROMATOGRAPH
pH (2:1)	INOR 93-6031	modified from EPA 9045D and MCKEAGUE 3.11	PH METER
Electrical Conductivity (2:1)	INOR-93-6075	modified from MSA PART 3, CH 14 and SM 2510 B	PC TITRATE
Resistivity (2:1) (Calculated)	INOR-93-6036	McKeague 4.12, SM 2510 B, SSA #5 Part 3	CALCULATION
Redox Potential 1	INOR-93-6066	G200-20, SM 2580 B	REDOX POTENTIAL ELECTRODE
Redox Potential 2	INOR-93-6066	ASTM G200-20, SM 2580 B	REDOX POTENTIAL ELECTRODE
Redox Potential 3	INOR-93-6066	ASTM G200-20, SM 2580 B	REDOX POTENTIAL ELECTRODE

CLIENT NAME: STANTEC CONSULTING LTD
300-675 Cochrane Drive
MARKHAM, ON L3R0B8
(905) 444-7777

ATTENTION TO: Bahram Siavash

PROJECT: 165001308.551.102

AGAT WORK ORDER: 24T187247

ROCK ANALYSIS REVIEWED BY: Jewel Shibu, Lab Supervisor

SOIL ANALYSIS REVIEWED BY: Sukhwinder Randhawa, Inorganic Team Lead

DATE REPORTED: Aug 28, 2024

PAGES (INCLUDING COVER): 7

VERSION*: 1

Should you require any information regarding this analysis please contact your client services representative at (403) 735-2005

*Notes

Disclaimer:

- All work conducted herein has been done using accepted standard protocols, and generally accepted practices and methods. AGAT test methods may incorporate modifications from the specified reference methods to improve performance.
- All samples will be disposed of within 30 days after receipt unless a Long Term Storage Agreement is signed and returned. Some specialty analysis may be exempt, please contact your Client Project Manager for details.
- AGAT's liability in connection with any delay, performance or non-performance of these services is only to the Client and does not extend to any other third party. Unless expressly agreed otherwise in writing, AGAT's liability is limited to the actual cost of the specific analysis or analyses included in the services.
- This Certificate shall not be reproduced except in full, without the written approval of the laboratory.
- The test results reported herewith relate only to the samples as received by the laboratory.
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- All reportable information is available on request from AGAT Laboratories, in accordance with ISO/IEC 17025:2017, ISO/IEC 17025:2005 (Quebec), DR-12-PALA and/or NELAP Standards.
- This document is signed by an authorized signatory who meets the requirements of the MELCCFP, CALA, CCN and NELAP.
- For environmental samples in the Province of Quebec: The analysis is performed on and results apply to samples as received. A temperature above 6°C upon receipt, as indicated in the Sample Reception Notification (SRN), could indicate the integrity of the samples has been compromised if the delay between sampling and submission to the laboratory could not be minimized.



Certificate of Analysis

AGAT WORK ORDER: 24T187247

PROJECT: 165001308.551.102

2910 12TH STREET NE
CALGARY, ALBERTA
CANADA T2E 7P7
TEL (403)735-2005
FAX (403)735-2771
<http://www.agatlabs.com>

CLIENT NAME: STANTEC CONSULTING LTD

SAMPLING SITE:

ATTENTION TO: Bahram Siavash

SAMPLED BY:

(284-137) Sulfide (CGY)

DATE RECEIVED: 2024-08-20

DATE REPORTED: 2024-08-28

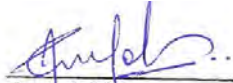
		SAMPLE DESCRIPTION: CNR-OH4 - SS9		RMN-UP2 - SS7		RMN-UP1 - SS10
		SAMPLE TYPE: Soil		Soil		Soil
		DATE SAMPLED: 2024-08-20		2024-08-20		2024-08-20
Parameter	Unit	G / S	RDL	6087931	6087963	6087966
Sulfide	%	0.01	0.09	0.14	0.13	

Comments: RDL - Reported Detection Limit; G / S - Guideline / Standard

6087931-6087966 Sulfide is a calculated parameter and is non-accredited. The parameters that are components of the calculation are accredited.

Analysis performed at AGAT Calgary (unless marked by *)

Certified By:


Jewel Shibu

Certificate of Analysis

AGAT WORK ORDER: 24T187247

PROJECT: 165001308.551.102

2910 12TH STREET NE
CALGARY, ALBERTA
CANADA T2E 7P7
TEL (403)735-2005
FAX (403)735-2771
<http://www.agatlabs.com>

CLIENT NAME: STANTEC CONSULTING LTD

ATTENTION TO: Bahram Siavash

SAMPLING SITE:

SAMPLED BY:

Corrosivity Package

DATE RECEIVED: 2024-08-20

DATE REPORTED: 2024-08-28

		SAMPLE DESCRIPTION: CNR-OH4 - SS9 RMN-UP2 - SS7 RMN-UP1 - SS10			
		SAMPLE TYPE: Soil Soil Soil			
		DATE SAMPLED: 2024-08-20 2024-08-20 2024-08-20			
Parameter	Unit	G / S	RDL	6087931	6087963 6087966
Chloride (2:1)	µg/g		2	10	5 6
Sulphate (2:1)	µg/g		2	277	318 272
pH (2:1)	pH Units		NA	8.68	8.46 8.79
Electrical Conductivity (2:1)	mS/cm		0.005	0.412	0.397 0.366
Resistivity (2:1) (Calculated)	ohm.cm		1	2430	2520 2730
Redox Potential 1	mV		NA	201	199 196
Redox Potential 2	mV		NA	186	205 216
Redox Potential 3	mV		NA	195	221 229

Comments: RDL - Reported Detection Limit; G / S - Guideline / Standard

6087931-6087966 EC, pH, Chloride and Sulphate were determined on the extract obtained from the 2:1 leaching procedure (2 parts DI water: 1 part soil). Resistivity is a calculated parameter.

Redox potential measured on as received sample. Due to the potential for rapid change in sample equilibrium chemistry with exposure to oxidative/reduction conditions laboratory results may differ from field measured results.

Redox potential measurement in soil is quite variable and non reproducible due in part, to the general heterogeneity of a given soil. It is also related to the introduction of increased oxygen into the sample after extraction. The interpretation of soil redox potential should be considered in terms of its general range rather than as an absolute measurement.

Analysis performed at AGAT Toronto (unless marked by *)

Certified By:



Quality Assurance

CLIENT NAME: STANTEC CONSULTING LTD

PROJECT: 165001308.551.102

SAMPLING SITE:

AGAT WORK ORDER: 24T187247

ATTENTION TO: Bahram Siavash

SAMPLED BY:

Rock Analysis

RPT Date: Aug 28, 2024			DUPLICATE			Method Blank	REFERENCE MATERIAL		METHOD BLANK SPIKE		MATRIX SPIKE	
PARAMETER	Batch	Sample Id	Dup #1	Dup #2	RPD		Measured Value	Acceptable Limits		Recovery	Acceptable Limits	
								Lower	Upper		Lower	Upper

(284-137) Sulfide (CGY)

Total Sulfur 6087931 6087931 0.11 0.13 16.8% < 0.01 105% 80% 120%

Sulfate 6074983 6074983 <0.01 <0.01 NA < 0.01 99% 80% 120%

Comments: RPDs are calculated using raw analytical data and not the rounded duplicate values reported.

Duplicate/ Replicate NA: Results are less than 10X the RDL and RPD will not be calculated

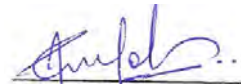
(284-137) Sulfide (CGY)

Sulfate 6087931 6087931 0.02 0.02 0.2% < 0.01 80% 120%

Comments: RPDs are calculated using raw analytical data and not the rounded duplicate values reported.

Duplicate/ Replicate NA: Results are less than 10X the RDL and RPD will not be calculated

Certified By:


Jewel Shibu

Quality Assurance

CLIENT NAME: STANTEC CONSULTING LTD

PROJECT: 165001308.551.102

SAMPLING SITE:

AGAT WORK ORDER: 24T187247

ATTENTION TO: Bahram Siavash

SAMPLED BY:

Soil Analysis

RPT Date: Aug 28, 2024			DUPLICATE			Method Blank	REFERENCE MATERIAL		METHOD BLANK SPIKE		MATRIX SPIKE	
PARAMETER	Batch	Sample Id	Dup #1	Dup #2	RPD		Measured Value	Acceptable Limits		Recovery	Acceptable Limits	
								Lower	Upper		Lower	Upper

Corrosivity Package

Chloride (2:1)	6089108		40	40	0.0%	< 2	94%	70%	130%	98%	80%	120%	95%	70%	130%
Sulphate (2:1)	6089108		171	172	0.6%	< 2	98%	70%	130%	101%	80%	120%	100%	70%	130%
pH (2:1)	6089108		8.52	8.34	2.1%	NA	96%	80%	120%						
Electrical Conductivity (2:1)	6089108		0.353	0.364	3.1%	< 0.005	102%	80%	120%						
Redox Potential 1	6087931					NA	100%	90%	110%						

Comments: NA signifies Not Applicable.

pH duplicates QA acceptance criteria was met relative as stated in Table 5-15 of Analytical Protocol document.

Certified By:



Method Summary

CLIENT NAME: STANTEC CONSULTING LTD

AGAT WORK ORDER: 24T187247

PROJECT: 165001308.551.102

ATTENTION TO: Bahram Siavash

SAMPLING SITE:

SAMPLED BY:

PARAMETER	AGAT S.O.P	LITERATURE REFERENCE	ANALYTICAL TECHNIQUE
Soil Analysis			
Chloride (2:1)	INOR-93-6004	modified from SM 4110 B	ION CHROMATOGRAPH
Sulphate (2:1)	INOR-93-6004	modified from SM 4110 B	ION CHROMATOGRAPH
pH (2:1)	INOR 93-6031	modified from EPA 9045D and MCKEAGUE 3.11	PH METER
Electrical Conductivity (2:1)	INOR-93-6075	modified from MSA PART 3, CH 14 and SM 2510 B	PC TITRATE
Resistivity (2:1) (Calculated)	INOR-93-6036	McKeague 4.12, SM 2510 B, SSA #5 Part 3	CALCULATION
Redox Potential 1	INOR-93-6066	G200-20, SM 2580 B	REDOX POTENTIAL ELECTRODE
Redox Potential 2	INOR-93-6066	ASTM G200-20, SM 2580 B	REDOX POTENTIAL ELECTRODE
Redox Potential 3	INOR-93-6066	ASTM G200-20, SM 2580 B	REDOX POTENTIAL ELECTRODE

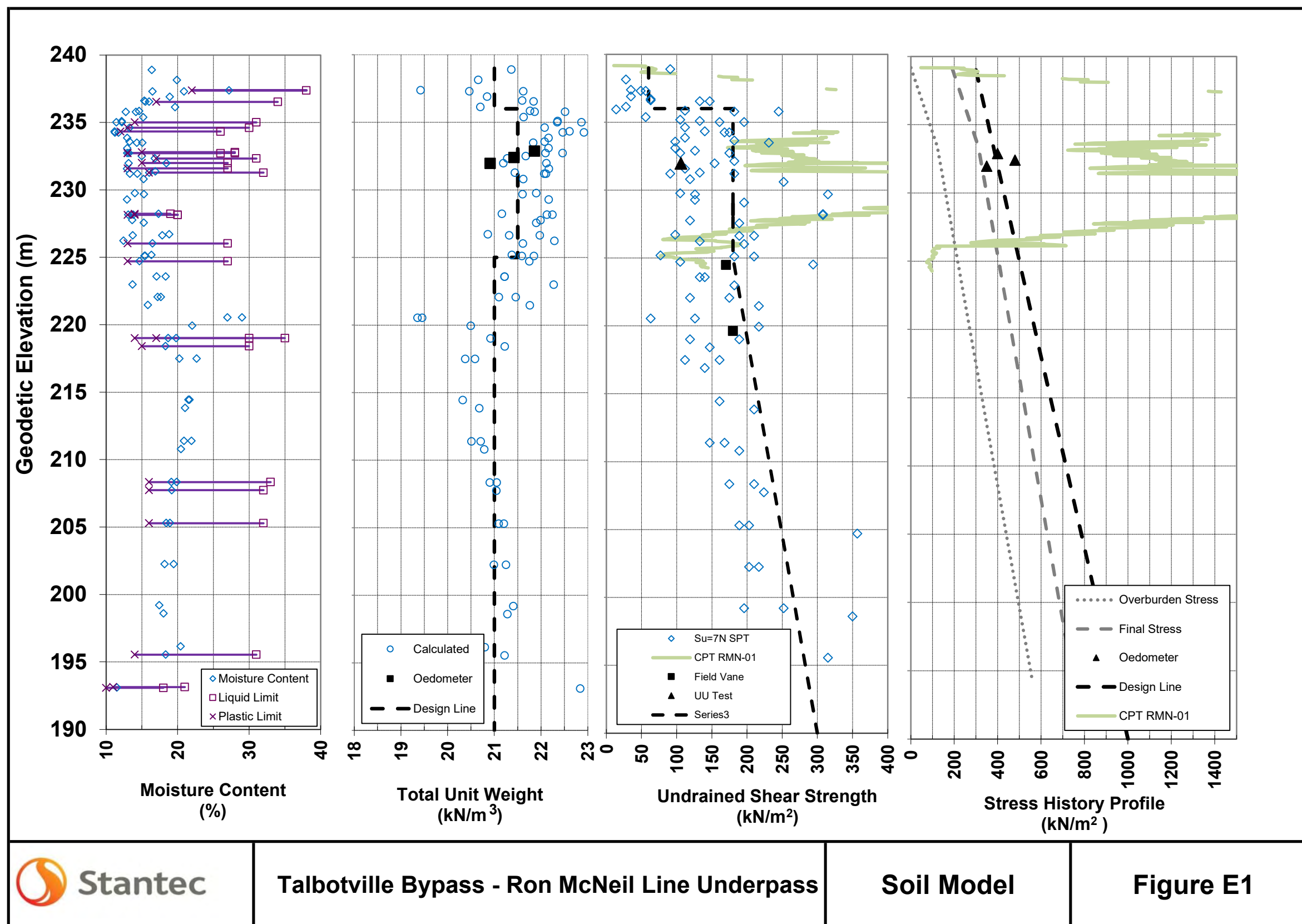
**FOUNDATION INVESTIGATION AND DESIGN REPORT – RON MCNEIL LINE INTERCHANGE
UNDERPASS – HIGHWAY 4 WIDENING FROM CLINTON LINE TO NEW TALBOTVILLE BYPASS
AND NEW TALBOTVILLE BYPASS FROM HIGHWAY 4 TO HIGHWAY 3 AT RON MCNEIL LINE**

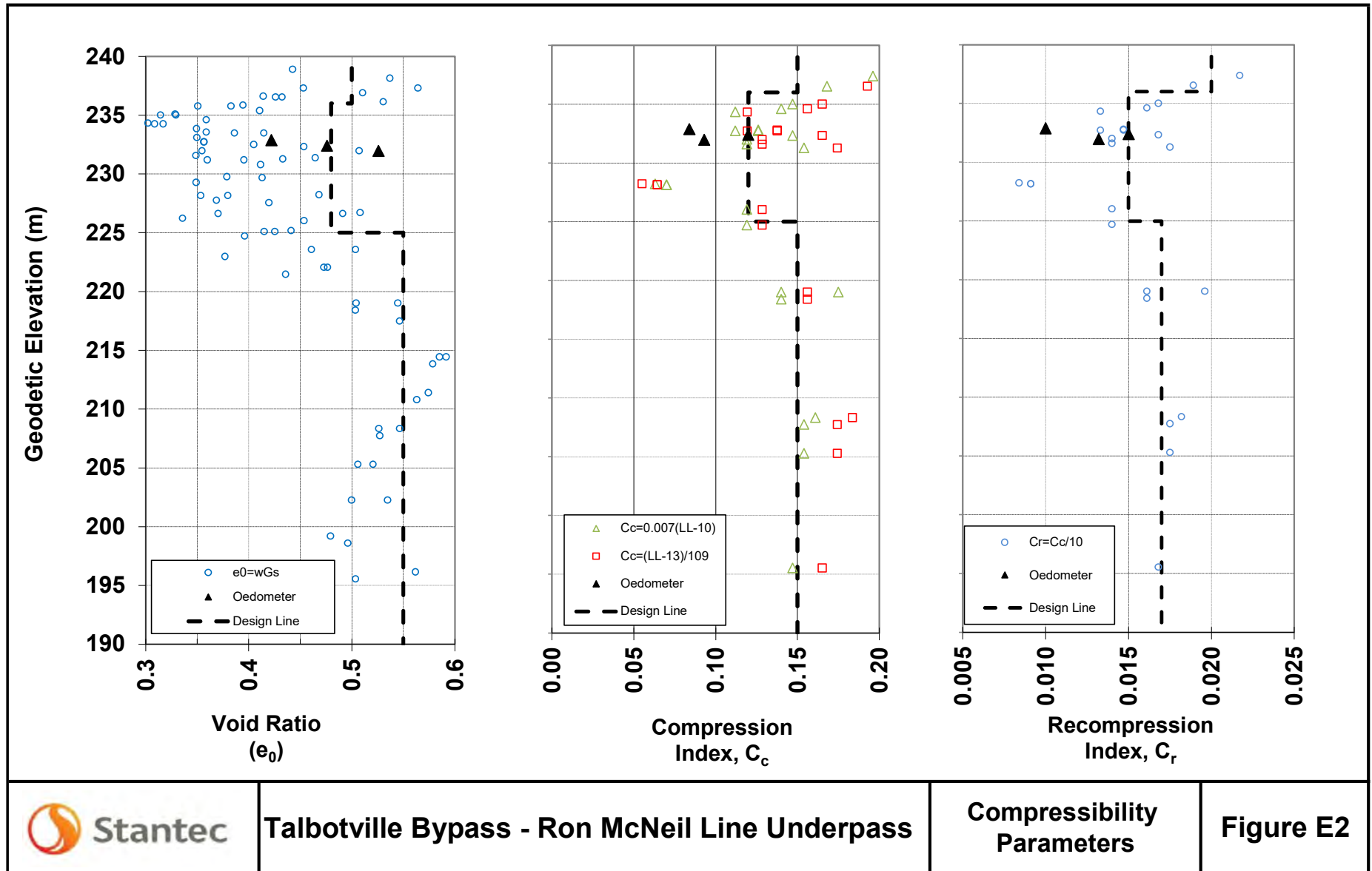
April 2025

APPENDIX E

- E.1 COHESIVE SOIL MODEL (FIGURE E1 AND E2)**
- E.2 P-Y CURVES (FIGURE E3 TO E8)**
- E.3 P-Y TABLES (TABLE E1 TO E6)**
- E.4 SLOPE STABILITY RESULTS (FIGURE E9 TO E14)**
- E.5 SETTLEMENT ANALYSIS RESULTS (FIGURE E15 AND E16)**







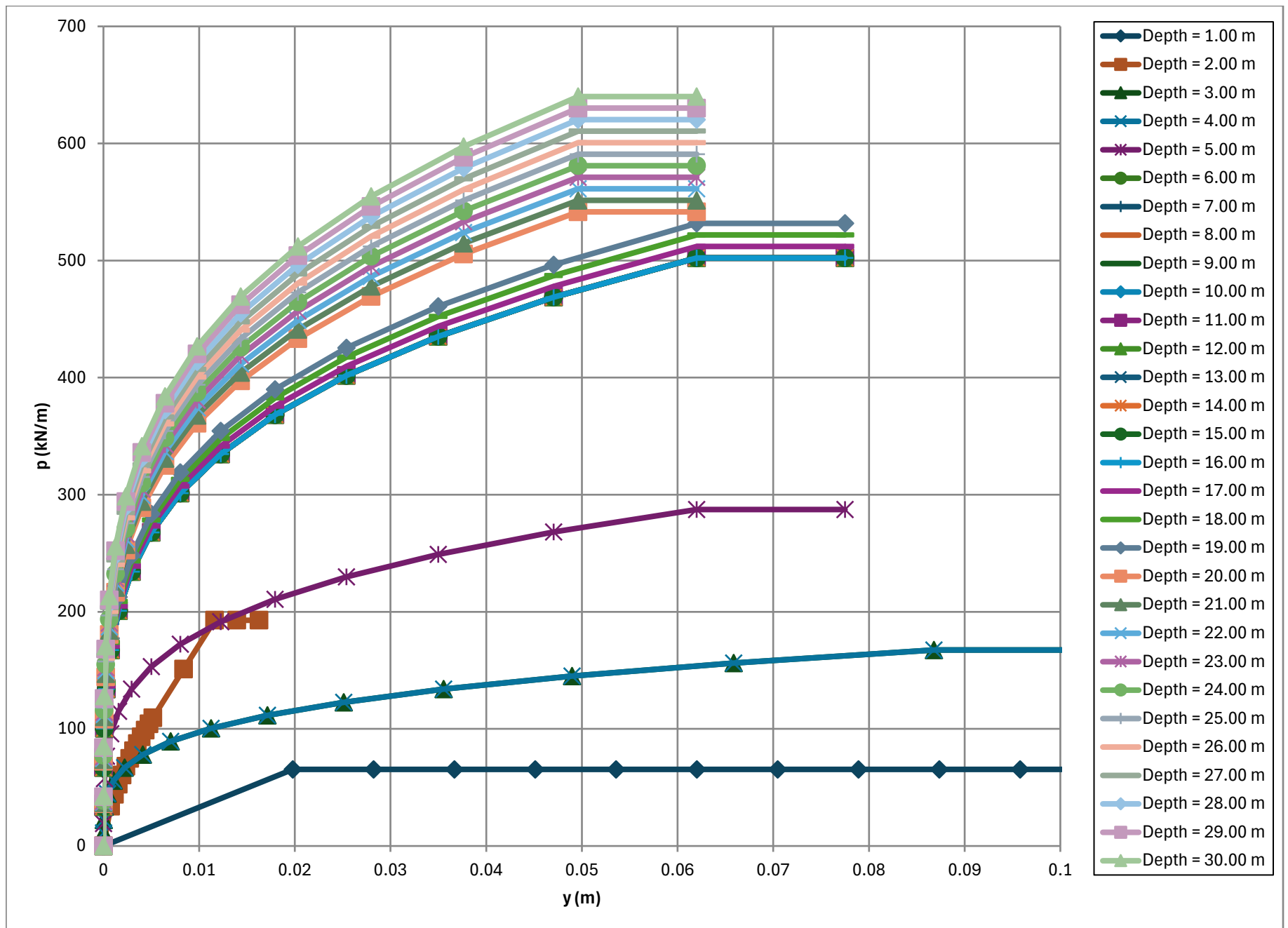


Figure E3: Load Intensity p (kN/m) vs Lateral Deflection y (m) Curves for South Abutment - HP 310 x 110

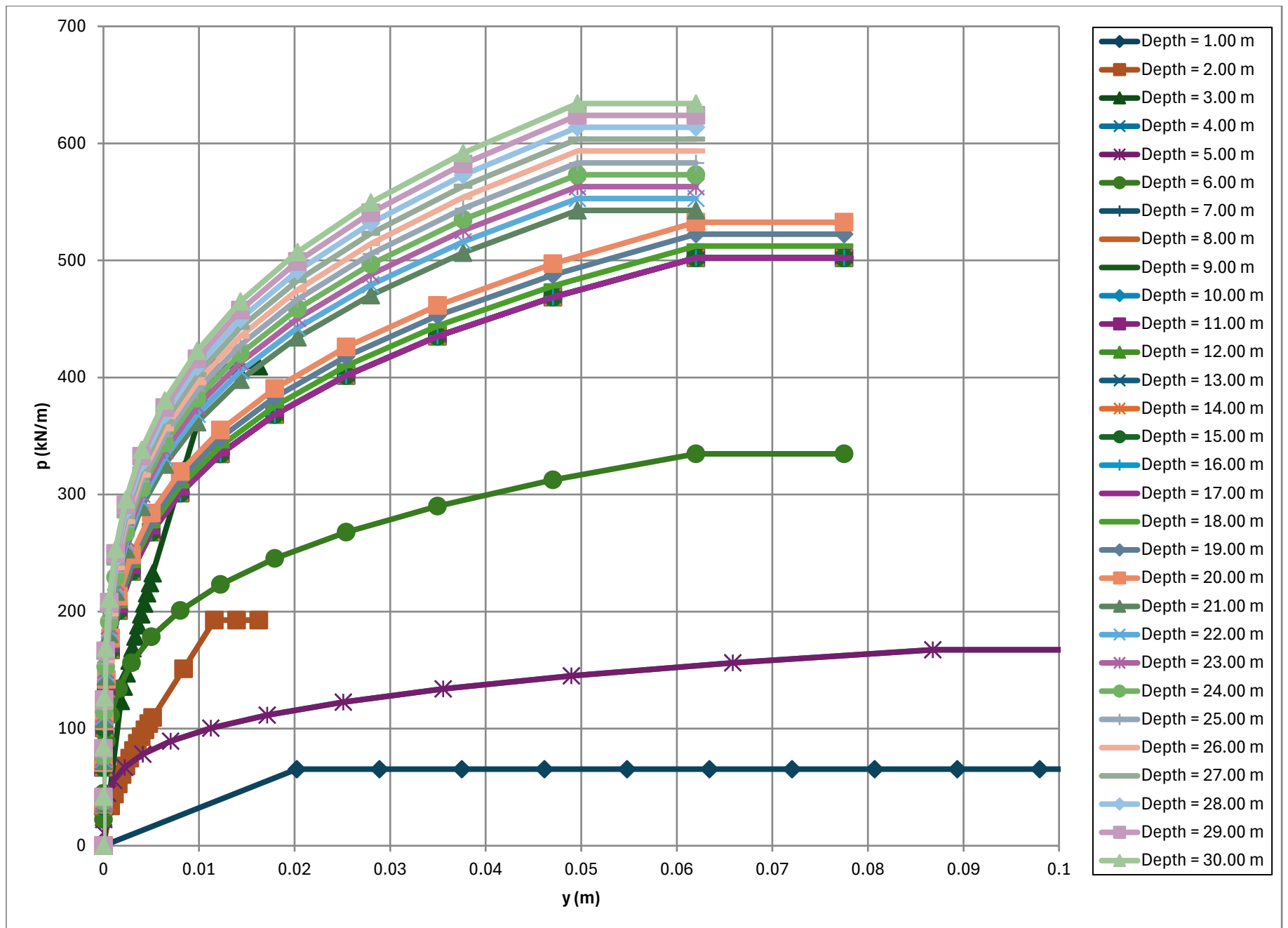


Figure E4: Load Intensity p (kN/m) vs Lateral Deflection y (m) Curves for North Abutment - HP 310 x 110

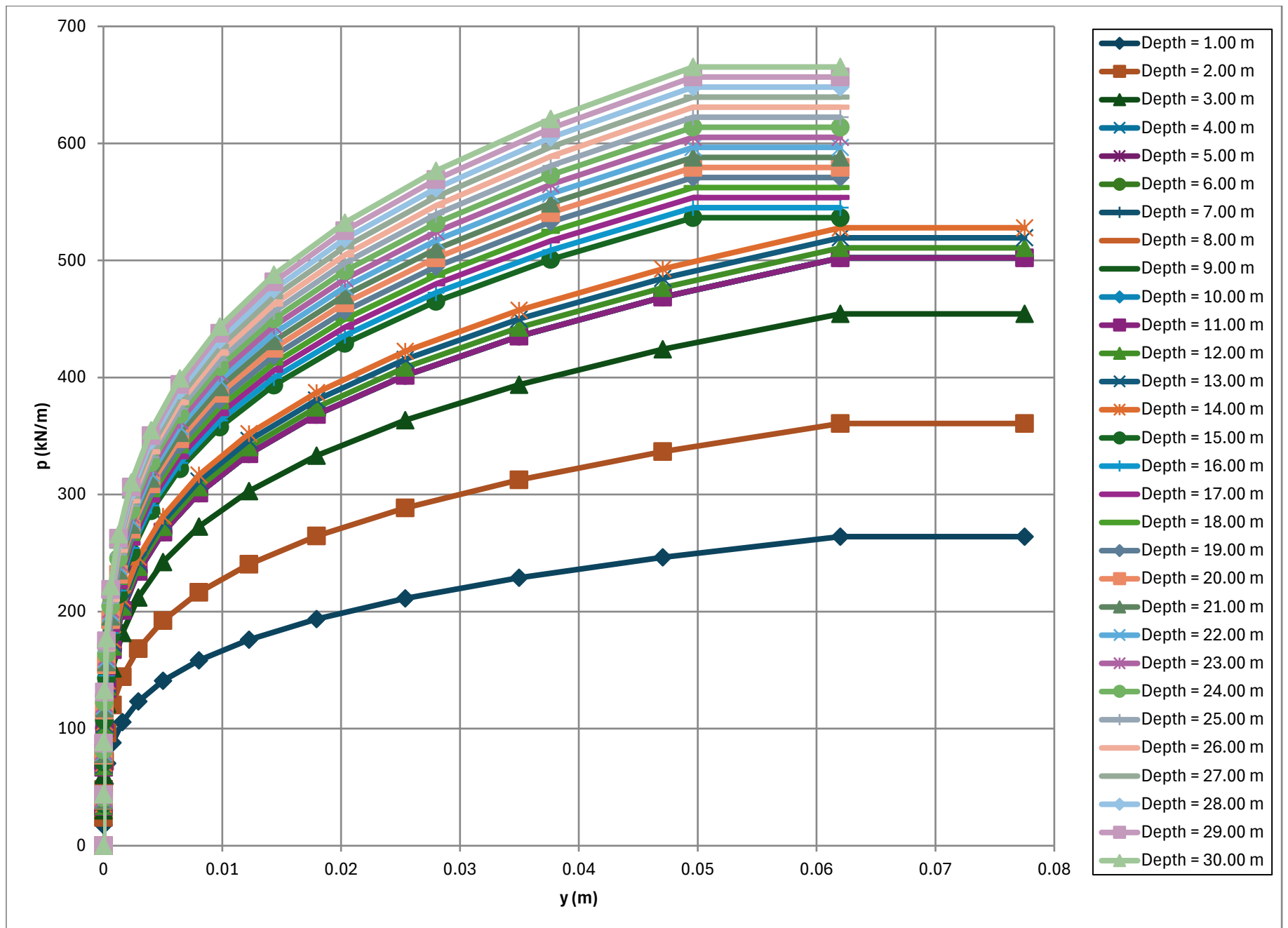


Figure E5: Load Intensity p (kN/m) vs Lateral Deflection y (m) Curves for Pier- HP 310 x 110

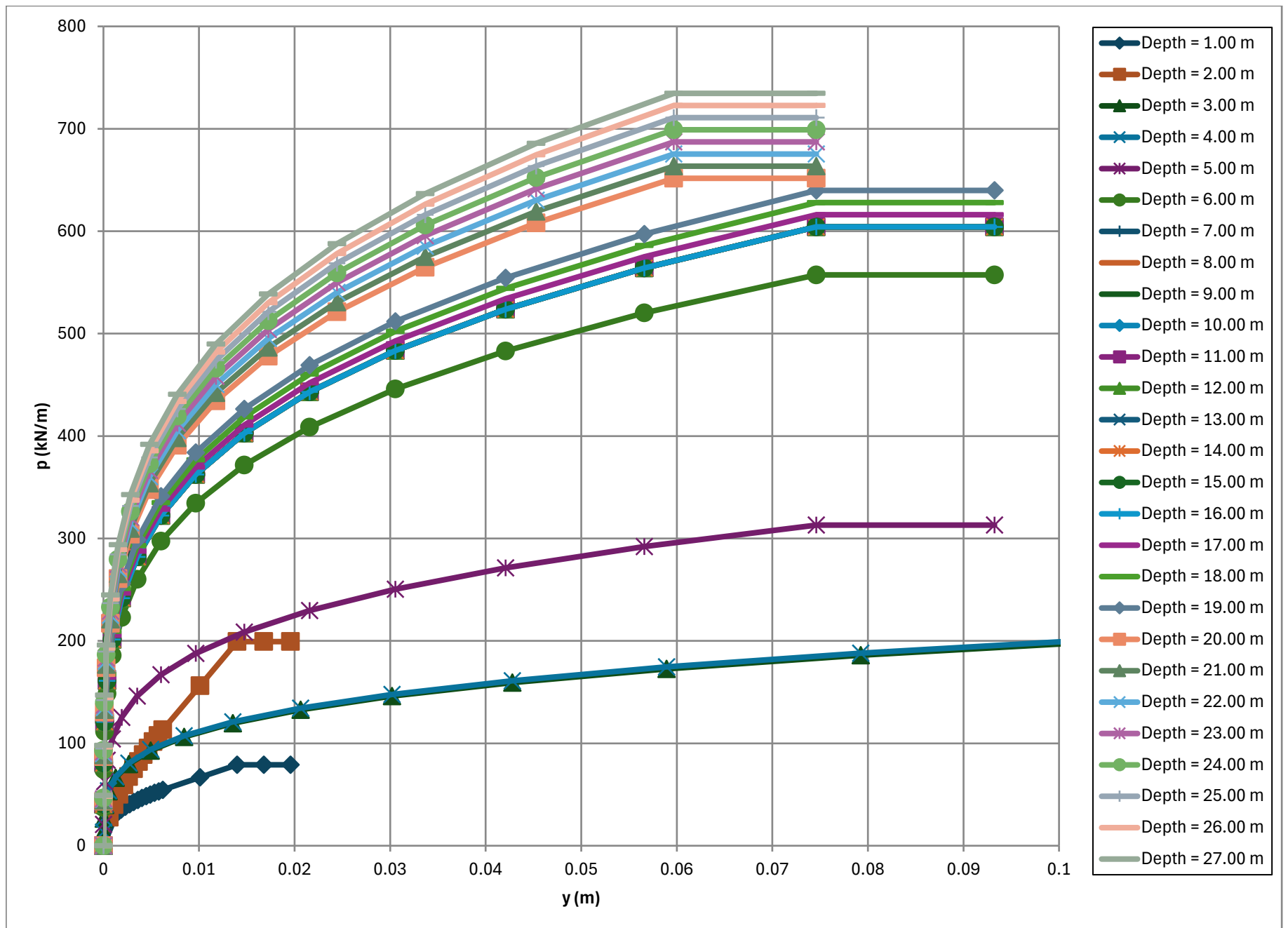


Figure E6: Load Intensity p (kN/m) vs Lateral Deflection y (m) Curves for South Abutment - HP 360 x 132

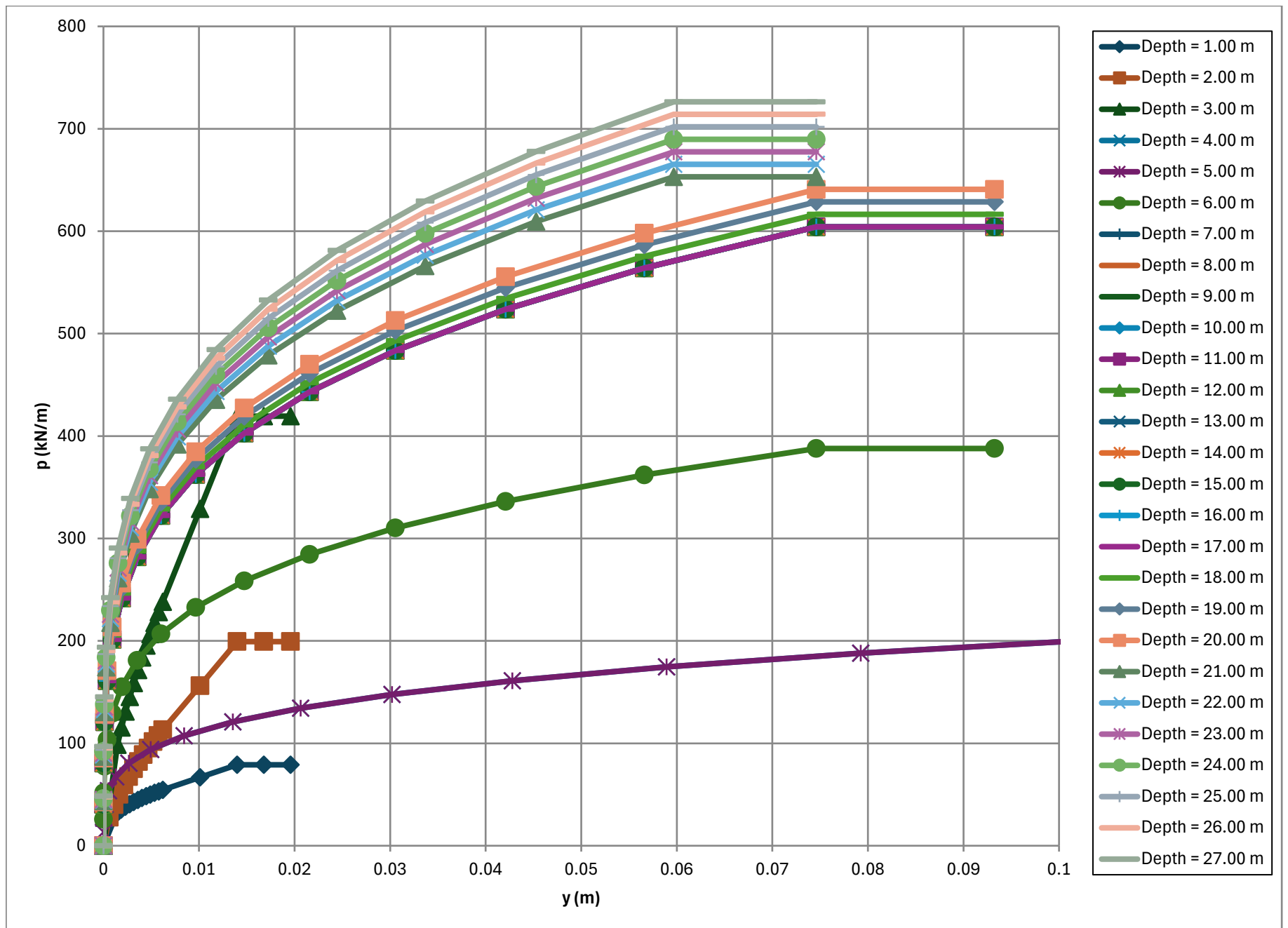


Figure E7: Load Intensity p (kN/m) vs Lateral Deflection y (m) Curves for North Abutment - HP 360 x 132

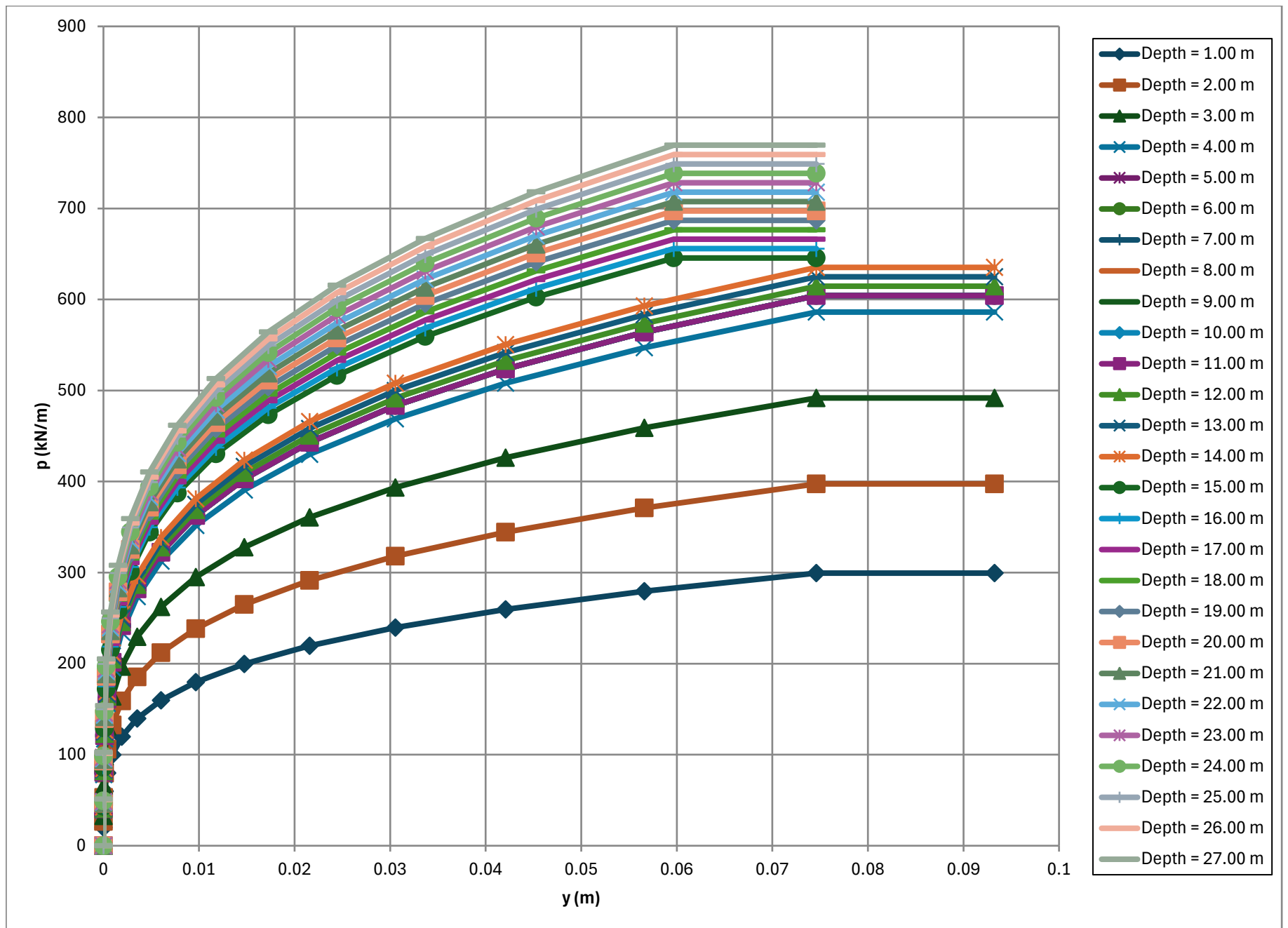


Figure E8: Load Intensity p (kN/m) vs Lateral Deflection y (m) Curves for Pier- HP 360 x 132

Table E1: Load Intensity p (kN/m) vs Lateral Deflection y (m) Data Points for South Abutment - HP 310 x 110

Depth Below Abutment Wall (m)		Curve Points																
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	
1.0	Y	0	0.01978	0.02823	0.03667	0.04512	0.05357	0.06201	0.07046	0.0789	0.08735	0.0958	0.10424	0.11269	0.18132	0.25355	0.30426	0.354975
	P	0	65.4256	65.4256	65.4256	65.4256	65.4256	65.4256	65.4256	65.4256	65.4256	65.4256	65.4256	65.4256	65.4256	65.4256	65.4256	65.4256
2.0	Y	0	0.00075	0.00115	0.00155	0.00196	0.00236	0.00276	0.00316	0.00356	0.00396	0.00436	0.00477	0.00517	0.0084	0.01163	0.01395	0.016275
	P	0	33.9285	44.0068	52.7691	60.6769	67.9686	74.7868	81.2255	87.3505	93.21	98.8411	104.272	109.527	151.147	192.768	192.768	192.7677
3.0	Y	0	1.7E-06	2.7E-05	0.00014	0.00044	0.00107	0.00222	0.00412	0.00702	0.01125	0.01715	0.0251	0.03555	0.04897	0.06587	0.0868	0.1085
	P	0	11.16	22.32	33.48	44.64	55.8	66.96	78.12	89.28	100.44	111.6	122.76	133.92	145.08	156.24	167.4	167.4
4.0	Y	0	1.7E-06	2.7E-05	0.00014	0.00044	0.00107	0.00222	0.00412	0.00702	0.01125	0.01715	0.0251	0.03555	0.04897	0.06587	0.0868	0.1085
	P	0	11.16	22.32	33.48	44.64	55.8	66.96	78.12	89.28	100.44	111.6	122.76	133.92	145.08	156.24	167.4	167.4
5.0	Y	0	1.2E-06	2E-05	9.9E-05	0.00031	0.00077	0.00159	0.00294	0.00502	0.00804	0.01225	0.01793	0.0254	0.03498	0.04705	0.062	0.0775
	P	0	19.1573	38.3146	57.472	76.6293	95.7866	114.944	134.101	153.259	172.416	191.573	210.731	229.888	249.045	268.203	287.36	287.3598
6.0	Y	0	1.2E-06	2E-05	9.9E-05	0.00031	0.00077	0.00159	0.00294	0.00502	0.00804	0.01225	0.01793	0.0254	0.03498	0.04705	0.062	0.0775
	P	0	33.48	66.96	100.44	133.92	167.4	200.88	234.36	267.84	301.32	334.8	368.28	401.76	435.24	468.72	502.2	502.2
7.0	Y	0	1.2E-06	2E-05	9.9E-05	0.00031	0.00077	0.00159	0.00294	0.00502	0.00804	0.01225	0.01793	0.0254	0.03498	0.04705	0.062	0.0775
	P	0	33.48	66.96	100.44	133.92	167.4	200.88	234.36	267.84	301.32	334.8	368.28	401.76	435.24	468.72	502.2	502.2
8.0	Y	0	1.2E-06	2E-05	9.9E-05	0.00031	0.00077	0.00159	0.00294	0.00502	0.00804	0.01225	0.01793	0.0254	0.03498	0.04705	0.062	0.0775
	P	0	33.48	66.96	100.44	133.92	167.4	200.88	234.36	267.84	301.32	334.8	368.28	401.76	435.24	468.72	502.2	502.2
9.0	Y	0	1.2E-06	2E-05	9.9E-05	0.00031	0.00077	0.00159	0.00294	0.00502	0.00804	0.01225	0.01793	0.0254	0.03498	0.04705	0.062	0.0775
	P	0	33.48	66.96	100.44	133.92	167.4	200.88	234.36	267.84	301.32	334.8	368.28	401.76	435.24	468.72	502.2	502.2
10.0	Y	0	1.2E-06	2E-05	9.9E-05	0.00031	0.00077	0.00159	0.00294	0.00502	0.00804	0.01225	0.01793	0.0254	0.03498	0.04705	0.062	0.0775
	P	0	33.48	66.96	100.44	133.92	167.4	200.88	234.36	267.84	301.32	334.8	368.28	401.76	435.24	468.72	502.2	502.2
11.0	Y	0	1.2E-06	2E-05	9.9E-05	0.00031	0.00077	0.00159	0.00294	0.00502	0.00804	0.01225	0.01793	0.0254	0.03498	0.04705	0.062	0.0775
	P	0	33.48	66.96	100.44	133.92	167.4	200.88	234.36	267.84	301.32	334.8	368.28	401.76	435.24	468.72	502.2	502.2
12.0	Y	0	1.2E-06	2E-05	9.9E-05	0.00031	0.00077	0.00159	0.00294	0.00502	0.00804	0.01225	0.01793	0.0254	0.03498	0.04705	0.062	0.0775
	P	0	33.48	66.96	100.44	133.92	167.4	200.88	234.36	267.84	301.32	334.8	368.28	401.76	435.24	468.72	502.2	502.2
13.0	Y	0	1.2E-06	2E-05	9.9E-05	0.00031	0.00077	0.00159	0.00294	0.00502	0.00804	0.01225	0.01793	0.0254	0.03498	0.04705	0.062	0.0775
	P	0	33.48	66.96	100.44	133.92	167.4	200.88	234.36	267.84	301.32	334.8	368.28	401.76	435.24	468.72	502.2	502.2
14.0	Y	0	1.2E-06	2E-05	9.9E-05	0.00031	0.00077	0.00159	0.00294	0.00502	0.00804	0.01225	0.01793	0.0254	0.03498	0.04705	0.062	0.0775
	P	0	33.48	66.96	100.44	133.92	167.4	200.88	234.36	267.84	301.32	334.8	368.28	401.76	435.24	468.72	502.2	502.2
15.0	Y	0	1.2E-06	2E-05	9.9E-05	0.00031	0.00077	0.00159	0.00294	0.00502	0.00804	0.01225	0.01793	0.0254	0.03498	0.04705	0.062	0.0775
	P	0	33.48	66.96	100.44	133.92	167.4	200.88	234.36	267.84	301.32	334.8	368.28	401.76	435.24	468.72	502.2	502.2
16.0	Y	0	1.2E-06	2E-05	9.9E-05	0.00031	0.00077	0.00159	0.00294	0.00502	0.00804	0.01225	0.01793	0.0254	0.03498	0.04705	0.062	0.0775
	P	0	33.48	66.96	100.44	133.92	167.4	200.88	234.36	267.84	301.32	334.8	368.28	401.76	435.24	468.72	502.2	502.2
17.0	Y	0	1.2E-06	2E-05	9.9E-05	0.00031	0.00077	0.00159	0.00294	0.00502	0.00804	0.01225	0.01793	0.0254	0.03498	0.04705	0.062	0.0775
	P	0	34.1365	68.2729	102.409	136.546	170.682	204.819	238.955	273.092	307.228	341.365	375.501	409.638	443.774	477.911	512.047	512.0471
18.0	Y	0	1.2E-06	2E-05	9.9E-05	0.00031	0.00077	0.00159	0.00294	0.00502	0.00804	0.01225	0.01793	0.0254	0.03498	0.04705	0.062	0.0775
	P	0	34.7929	69.5859	104.379	139.172	173.965	208.758	243.551	278.344	313.136	347.929	382.722	417.515	452.308	487.101	521.894	521.8941
19.0	Y	0	1.2E-06	2E-05	9.9E-05	0.00031	0.00077	0.00159	0.00294	0.00502	0.00804	0.01225	0.01793	0.0254	0.03498	0.04705	0.062	0.0775
	P	0	35.4494	70.8988	106.348	141.798	177.247	212.696	248.146	283.595	319.045	354.494	389.944	425.393	460.842	496.292	531.741	531.7412
20.0	Y	0	9.8E-07	1.6E-05	7.9E-05	0.00025	0.00061	0.00127	0.00235	0.00401	0.00643	0.0098	0.01434	0.02032	0.02798	0.03764	0.0496	0.062
	P	0	36.1059	72.2118	108.318	144.424	180.529	216.635	252.741	288.847	324.953	361.059	397.165	433.271	469.376	505.482	541.588	541.5882
21.0	Y	0	9.8E-07	1.6E-05	7.9E-05	0.00025	0.00061	0.00127	0.00235	0.00401	0.00643	0.0098	0.01434	0.02032	0.02798	0.03764	0.0496	0.062
	P	0	36.7624	73.5247	110.287	147.049	183.812	220.574	257.336	294.099	330.861	367.624	404.386	441.148	477.911	514.673	551.435	551.4353
22.0	Y	0	9.8E-07	1.6E-05	7.9E-05	0.00025	0.00061	0.00127	0.00235	0.00401	0.00643	0.0098	0.01434	0.02032	0.02798	0.03764	0.0496	0.062
	P	0	37.4188	74.8376	112.256	149.675	187.094	224.513	261.932	299.351	336.769	374.188	411.607	449.026	486.445	523.864	561.282	561.2824
23.0	Y	0	9.8E-07	1.6E-05	7.9E-05	0.00025	0.00061	0.00127	0.00235	0.00401	0.00643	0.0098	0.01434	0.02032	0.02798	0.03764	0.0496	0.062
	P	0	38.0753	76.1506	114.226	152.301	190.376	228.452	266.527	304.602	342.678	380.753	418.828	456.904	494.979	533.054	571.129	571.1294
24.0	Y	0	9.8E-07	1.6E-05	7.9E-05	0.00025	0.00061	0.00127	0.00235	0.00401	0.00643	0.0098	0.01434	0.02032	0.02798	0.03764	0.0496	0.062
	P	0	38.7318	77.4635	116.195	154.925	193.659	232.391	271.122	309.854	348.586	387.318	426.049	464.781	503.513	542.245	580.976	580.9765
25.0	Y	0	9.8E-07	1.6E-05	7.9E-05	0.00025	0.00061	0.00127	0.00235	0.00401	0.00643	0.0098	0.01434	0.02032	0.02798	0.03764	0.0496	0.062
	P	0	39.3882	78.7765	118.165	157.553	196.941	236.329	275.718	315.106	354.494	393.882	433.271	472.659	512.047	551.435	590.824	590.8235
26.0	Y	0	9.8E-07	1.6E-05	7.9E-05	0.00025	0.00061	0.00127	0.00235	0.00401	0.00643	0.0098	0.01434	0.02032	0.02798	0.03764	0.0496	0.062
	P	0	40.0447	80.0894	120.134	160.179	200.224	240.268	280.313	320.358	360.402	400.447	440.492	480.536	520.581	560.626	600.671	600.6706
27.0	Y	0	9.8E-07	1.6E-05	7.9E-05	0.00025	0.00061	0.00127	0.00235	0.00401	0.00643	0.0098	0.01434	0.02032	0.02798	0.03764	0.0496	0.062
	P	0	40.7012	81.4024	122.104	162.805	203.506	244.207	284.908	325.609	366.311	407.012	447.713	488.414	529.115	569.816	610.518	610.5177
28.0	Y	0	9.8E-07	1.6E-05	7.9E-05	0.00025	0.00061	0.00127	0.00235	0.00401	0.00643	0.0098	0.01434	0.02032	0.02798	0.03764	0.0496	0.062
	P	0	41.3576	82.7153	124.073	165.431	206.788	248.146	289.504	330.861	372.219	413.576	454.934	496.292	537.649	579.007	620.365	620.3647

Table E2: Load Intensity p (kN/m) vs Lateral Deflection y (m) Data Points for North Abutment - HP 310 x 110

Depth Below Abutment Wall (m)		Curve Points																
			1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
1.0	Y	0	0.02023	0.02887	0.03751	0.04615	0.05479	0.06343	0.07207	0.08071	0.08934	0.09798	0.10662	0.11526	0.1873	0.25934	0.31121	0.363073
	P	0	65.4256	65.4256	65.4256	65.4256	65.4256	65.4256	65.4256	65.4256	65.4256	65.4256	65.4256	65.4256	65.4256	65.4256	65.4256	65.4256
2.0	Y	0	0.00075	0.00115	0.00155	0.00196	0.00236	0.00276	0.00316	0.00356	0.00396	0.00436	0.00477	0.00517	0.0084	0.01163	0.01395	0.016275
	P	0	33.9285	44.0068	52.7691	60.6769	67.9686	74.7868	81.2255	87.3505	93.21	98.8411	104.272	109.527	151.147	192.768	192.768	192.7677
3.0	Y	0	0.00183	0.00213	0.00243	0.00274	0.00304	0.00335	0.00365	0.00395	0.00426	0.00456	0.00486	0.00517	0.0084	0.01163	0.01395	0.016275
	P	0	123.728	135.843	147.297	158.203	168.645	178.685	188.374	197.751	206.85	215.698	224.318	232.729	321.166	409.604	409.604	409.6036
4.0	Y	0	1.7E-06	2.7E-05	0.00014	0.00044	0.00107	0.00222	0.00412	0.00702	0.01125	0.01715	0.0251	0.03555	0.04897	0.06587	0.0868	0.1085
	P	0	11.16	22.32	33.48	44.64	55.8	66.96	78.12	89.28	100.44	111.6	122.76	133.92	145.08	156.24	167.4	167.4
5.0	Y	0	1.7E-06	2.7E-05	0.00014	0.00044	0.00107	0.00222	0.00412	0.00702	0.01125	0.01715	0.0251	0.03555	0.04897	0.06587	0.0868	0.1085
	P	0	11.16	22.32	33.48	44.64	55.8	66.96	78.12	89.28	100.44	111.6	122.76	133.92	145.08	156.24	167.4	167.4
6.0	Y	0	1.2E-06	2E-05	9.9E-05	0.00031	0.00077	0.00159	0.00294	0.00502	0.00804	0.01225	0.01793	0.0254	0.03498	0.04705	0.062	0.0775
	P	0	22.32	44.64	66.96	89.28	111.6	133.92	156.24	178.56	200.88	223.2	245.52	267.84	290.16	312.48	334.8	334.8
7.0	Y	0	1.2E-06	2E-05	9.9E-05	0.00031	0.00077	0.00159	0.00294	0.00502	0.00804	0.01225	0.01793	0.0254	0.03498	0.04705	0.062	0.0775
	P	0	33.48	66.96	100.44	133.92	167.4	200.88	234.36	267.84	301.32	334.8	368.28	401.76	435.24	468.72	502.2	502.2
8.0	Y	0	1.2E-06	2E-05	9.9E-05	0.00031	0.00077	0.00159	0.00294	0.00502	0.00804	0.01225	0.01793	0.0254	0.03498	0.04705	0.062	0.0775
	P	0	33.48	66.96	100.44	133.92	167.4	200.88	234.36	267.84	301.32	334.8	368.28	401.76	435.24	468.72	502.2	502.2
9.0	Y	0	1.2E-06	2E-05	9.9E-05	0.00031	0.00077	0.00159	0.00294	0.00502	0.00804	0.01225	0.01793	0.0254	0.03498	0.04705	0.062	0.0775
	P	0	33.48	66.96	100.44	133.92	167.4	200.88	234.36	267.84	301.32	334.8	368.28	401.76	435.24	468.72	502.2	502.2
10.0	Y	0	1.2E-06	2E-05	9.9E-05	0.00031	0.00077	0.00159	0.00294	0.00502	0.00804	0.01225	0.01793	0.0254	0.03498	0.04705	0.062	0.0775
	P	0	33.48	66.96	100.44	133.92	167.4	200.88	234.36	267.84	301.32	334.8	368.28	401.76	435.24	468.72	502.2	502.2
11.0	Y	0	1.2E-06	2E-05	9.9E-05	0.00031	0.00077	0.00159	0.00294	0.00502	0.00804	0.01225	0.01793	0.0254	0.03498	0.04705	0.062	0.0775
	P	0	33.48	66.96	100.44	133.92	167.4	200.88	234.36	267.84	301.32	334.8	368.28	401.76	435.24	468.72	502.2	502.2
12.0	Y	0	1.2E-06	2E-05	9.9E-05	0.00031	0.00077	0.00159	0.00294	0.00502	0.00804	0.01225	0.01793	0.0254	0.03498	0.04705	0.062	0.0775
	P	0	33.48	66.96	100.44	133.92	167.4	200.88	234.36	267.84	301.32	334.8	368.28	401.76	435.24	468.72	502.2	502.2
13.0	Y	0	1.2E-06	2E-05	9.9E-05	0.00031	0.00077	0.00159	0.00294	0.00502	0.00804	0.01225	0.01793	0.0254	0.03498	0.04705	0.062	0.0775
	P	0	33.48	66.96	100.44	133.92	167.4	200.88	234.36	267.84	301.32	334.8	368.28	401.76	435.24	468.72	502.2	502.2
14.0	Y	0	1.2E-06	2E-05	9.9E-05	0.00031	0.00077	0.00159	0.00294	0.00502	0.00804	0.01225	0.01793	0.0254	0.03498	0.04705	0.062	0.0775
	P	0	33.48	66.96	100.44	133.92	167.4	200.88	234.36	267.84	301.32	334.8	368.28	401.76	435.24	468.72	502.2	502.2
15.0	Y	0	1.2E-06	2E-05	9.9E-05	0.00031	0.00077	0.00159	0.00294	0.00502	0.00804	0.01225	0.01793	0.0254	0.03498	0.04705	0.062	0.0775
	P	0	33.48	66.96	100.44	133.92	167.4	200.88	234.36	267.84	301.32	334.8	368.28	401.76	435.24	468.72	502.2	502.2
16.0	Y	0	1.2E-06	2E-05	9.9E-05	0.00031	0.00077	0.00159	0.00294	0.00502	0.00804	0.01225	0.01793	0.0254	0.03498	0.04705	0.062	0.0775
	P	0	33.48	66.96	100.44	133.92	167.4	200.88	234.36	267.84	301.32	334.8	368.28	401.76	435.24	468.72	502.2	502.2
17.0	Y	0	1.2E-06	2E-05	9.9E-05	0.00031	0.00077	0.00159	0.00294	0.00502	0.00804	0.01225	0.01793	0.0254	0.03498	0.04705	0.062	0.0775
	P	0	33.48	66.96	100.44	133.92	167.4	200.88	234.36	267.84	301.32	334.8	368.28	401.76	435.24	468.72	502.2	502.2
18.0	Y	0	1.2E-06	2E-05	9.9E-05	0.00031	0.00077	0.00159	0.00294	0.00502	0.00804	0.01225	0.01793	0.0254	0.03498	0.04705	0.062	0.0775
	P	0	34.1564	68.3127	102.469	136.625	170.782	204.938	239.095	273.251	307.407	341.564	375.72	409.876	444.033	478.189	512.345	512.3455
19.0	Y	0	1.2E-06	2E-05	9.9E-05	0.00031	0.00077	0.00159	0.00294	0.00502	0.00804	0.01225	0.01793	0.0254	0.03498	0.04705	0.062	0.0775
	P	0	34.8327	69.6655	104.498	139.331	174.164	208.996	243.829	278.662	313.495	348.327	383.16	417.993	452.825	487.658	522.491	522.4909
20.0	Y	0	1.2E-06	2E-05	9.9E-05	0.00031	0.00077	0.00159	0.00294	0.00502	0.00804	0.01225	0.01793	0.0254	0.03498	0.04705	0.062	0.0775
	P	0	35.5091	71.0182	106.527	142.036	177.545	213.055	248.564	284.073	319.582	355.091	390.6	426.109	461.618	497.127	532.636	532.6364
21.0	Y	0	9.8E-07	1.6E-05	7.9E-05	0.00025	0.00061	0.00127	0.00235	0.00401	0.00643	0.0098	0.01434	0.02032	0.02798	0.03764	0.0496	0.062
	P	0	36.1855	72.3709	108.556	144.742	180.927	217.113	253.298	289.484	325.669	361.855	398.04	434.225	470.411	506.596	542.782	542.7818
22.0	Y	0	9.8E-07	1.6E-05	7.9E-05	0.00025	0.00061	0.00127	0.00235	0.00401	0.00643	0.0098	0.01434	0.02032	0.02798	0.03764	0.0496	0.062
	P	0	36.8618	73.7236	110.585	147.447	184.309	221.171	258.033	294.895	331.756	368.618	405.48	442.342	479.204	516.065	552.927	552.9273
23.0	Y	0	9.8E-07	1.6E-05	7.9E-05	0.00025	0.00061	0.00127	0.00235	0.00401	0.00643	0.0098	0.01434	0.02032	0.02798	0.03764	0.0496	0.062
	P	0	37.5382	75.0764	112.615	150.153	187.691	225.229	262.767	300.305	337.844	375.382	412.92	450.458	487.996	525.535	563.073	563.0727
24.0	Y	0	9.8E-07	1.6E-05	7.9E-05	0.00025	0.00061	0.00127	0.00235	0.00401	0.00643	0.0098	0.01434	0.02032	0.02798	0.03764	0.0496	0.062
	P	0	38.2145	76.4291	114.644	152.858	191.073	229.287	267.502	305.716	343.931	382.145	420.36	458.575	496.789	535.004	573.218	573.2182
25.0	Y	0	9.8E-07	1.6E-05	7.9E-05	0.00025	0.00061	0.00127	0.00235	0.00401	0.00643	0.0098	0.01434	0.02032	0.02798	0.03764	0.0496	0.062
	P	0	38.8909	77.7818	116.673	155.564	194.455	233.345	272.236	311.127	350.018	388.909	427.8	466.691	505.582	544.473	583.364	583.3636
26.0	Y	0	9.8E-07	1.6E-05	7.9E-05	0.00025	0.00061	0.00127	0.00235	0.00401	0.00643	0.0098	0.01434	0.02032	0.02798	0.03764	0.0496	0.062
	P	0	39.5673	79.1345	118.702	158.269	197.836	237.404	276.971	316.538	356.105	395.673	435.24	474.807	514.375	553.942	593.509	593.5091
27.0	Y	0	9.8E-07	1.6E-05	7.9E-05	0.00025	0.00061	0.00127	0.00235	0.00401	0.00643	0.0098	0.01434	0.02032	0.02798	0.03764	0.0496	0.062
	P	0	40.2436	80.4873	120.731	160.975	201.218	241.462	281.705	321.949	362.193	402.436	442.68	482.924	523.167	563.411	603.655	603.6546
28.0	Y	0	9.8E-07	1.6E-05	7.9E-05	0.00025	0.00061	0.00127	0.00235	0.00401	0.00643	0.0098	0.01434	0.02032	0.02798	0.03764	0.0496	0.062
	P	0	40.92	81.84	122.76	163.68	204.6	245.52	286.44	327.36	368.28	409.2	450.12	491.04	531.96	572.88	613.8	613.8
29.0	Y	0	9.8E-07	1.6E-05	7.9E-													

Table E3: Load Intensity p (kN/m) vs Lateral Deflection y (m) Data Points for Pier - HP 310 x 110

Depth Below Abutment Wall (m)		Curve Points																
			1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
1.0	Y	0	1.2E-06	2E-05	9.9E-05	0.00031	0.00077	0.00159	0.00294	0.00502	0.00804	0.01225	0.01793	0.0254	0.03498	0.04705	0.062	0.0775
	P	0	17.6043	35.2087	52.813	70.4173	88.0217	105.626	123.23	140.835	158.439	176.043	193.648	211.252	228.856	246.461	264.065	264.065
2.0	Y	0	1.2E-06	2E-05	9.9E-05	0.00031	0.00077	0.00159	0.00294	0.00502	0.00804	0.01225	0.01793	0.0254	0.03498	0.04705	0.062	0.0775
	P	0	24.0487	48.0973	72.146	96.1947	120.243	144.292	168.341	192.389	216.438	240.487	264.535	288.584	312.633	336.681	360.73	360.73
3.0	Y	0	1.2E-06	2E-05	9.9E-05	0.00031	0.00077	0.00159	0.00294	0.00502	0.00804	0.01225	0.01793	0.0254	0.03498	0.04705	0.062	0.0775
	P	0	30.2905	60.5809	90.8714	121.162	151.452	181.743	212.033	242.324	272.614	302.905	333.195	363.486	393.776	424.067	454.357	454.357
4.0	Y	0	1.2E-06	2E-05	9.9E-05	0.00031	0.00077	0.00159	0.00294	0.00502	0.00804	0.01225	0.01793	0.0254	0.03498	0.04705	0.062	0.0775
	P	0	33.48	66.96	100.44	133.92	167.4	200.88	234.36	267.84	301.32	334.8	368.28	401.76	435.24	468.72	502.2	502.2
5.0	Y	0	1.2E-06	2E-05	9.9E-05	0.00031	0.00077	0.00159	0.00294	0.00502	0.00804	0.01225	0.01793	0.0254	0.03498	0.04705	0.062	0.0775
	P	0	33.48	66.96	100.44	133.92	167.4	200.88	234.36	267.84	301.32	334.8	368.28	401.76	435.24	468.72	502.2	502.2
6.0	Y	0	1.2E-06	2E-05	9.9E-05	0.00031	0.00077	0.00159	0.00294	0.00502	0.00804	0.01225	0.01793	0.0254	0.03498	0.04705	0.062	0.0775
	P	0	33.48	66.96	100.44	133.92	167.4	200.88	234.36	267.84	301.32	334.8	368.28	401.76	435.24	468.72	502.2	502.2
7.0	Y	0	1.2E-06	2E-05	9.9E-05	0.00031	0.00077	0.00159	0.00294	0.00502	0.00804	0.01225	0.01793	0.0254	0.03498	0.04705	0.062	0.0775
	P	0	33.48	66.96	100.44	133.92	167.4	200.88	234.36	267.84	301.32	334.8	368.28	401.76	435.24	468.72	502.2	502.2
8.0	Y	0	1.2E-06	2E-05	9.9E-05	0.00031	0.00077	0.00159	0.00294	0.00502	0.00804	0.01225	0.01793	0.0254	0.03498	0.04705	0.062	0.0775
	P	0	33.48	66.96	100.44	133.92	167.4	200.88	234.36	267.84	301.32	334.8	368.28	401.76	435.24	468.72	502.2	502.2
9.0	Y	0	1.2E-06	2E-05	9.9E-05	0.00031	0.00077	0.00159	0.00294	0.00502	0.00804	0.01225	0.01793	0.0254	0.03498	0.04705	0.062	0.0775
	P	0	33.48	66.96	100.44	133.92	167.4	200.88	234.36	267.84	301.32	334.8	368.28	401.76	435.24	468.72	502.2	502.2
10.0	Y	0	1.2E-06	2E-05	9.9E-05	0.00031	0.00077	0.00159	0.00294	0.00502	0.00804	0.01225	0.01793	0.0254	0.03498	0.04705	0.062	0.0775
	P	0	33.48	66.96	100.44	133.92	167.4	200.88	234.36	267.84	301.32	334.8	368.28	401.76	435.24	468.72	502.2	502.2
11.0	Y	0	1.2E-06	2E-05	9.9E-05	0.00031	0.00077	0.00159	0.00294	0.00502	0.00804	0.01225	0.01793	0.0254	0.03498	0.04705	0.062	0.0775
	P	0	33.48	66.96	100.44	133.92	167.4	200.88	234.36	267.84	301.32	334.8	368.28	401.76	435.24	468.72	502.2	502.2
12.0	Y	0	1.2E-06	2E-05	9.9E-05	0.00031	0.00077	0.00159	0.00294	0.00502	0.00804	0.01225	0.01793	0.0254	0.03498	0.04705	0.062	0.0775
	P	0	34.0523	68.1046	102.157	136.209	170.262	204.314	238.366	272.418	306.471	340.523	374.575	408.628	442.68	476.732	510.785	510.7846
13.0	Y	0	1.2E-06	2E-05	9.9E-05	0.00031	0.00077	0.00159	0.00294	0.00502	0.00804	0.01225	0.01793	0.0254	0.03498	0.04705	0.062	0.0775
	P	0	34.6246	69.2492	103.874	138.498	173.123	207.748	242.372	276.997	311.622	346.246	380.871	415.495	450.12	484.745	519.369	519.3692
14.0	Y	0	1.2E-06	2E-05	9.9E-05	0.00031	0.00077	0.00159	0.00294	0.00502	0.00804	0.01225	0.01793	0.0254	0.03498	0.04705	0.062	0.0775
	P	0	35.1969	70.3938	105.591	140.788	175.985	211.182	246.378	281.575	316.772	351.969	387.166	422.363	457.56	492.757	527.954	527.9539
15.0	Y	0	9.8E-07	1.6E-05	7.9E-05	0.00025	0.00061	0.00127	0.00235	0.00401	0.00643	0.0098	0.01434	0.02032	0.02798	0.03764	0.0496	0.062
	P	0	35.7692	71.5385	107.308	143.077	178.846	214.615	250.385	286.154	321.923	357.692	393.462	429.231	465	500.769	536.538	536.5385
16.0	Y	0	9.8E-07	1.6E-05	7.9E-05	0.00025	0.00061	0.00127	0.00235	0.00401	0.00643	0.0098	0.01434	0.02032	0.02798	0.03764	0.0496	0.062
	P	0	36.3415	72.6831	109.025	145.366	181.708	218.049	254.391	290.732	327.074	363.415	399.757	436.098	472.44	508.782	545.123	545.1231
17.0	Y	0	9.8E-07	1.6E-05	7.9E-05	0.00025	0.00061	0.00127	0.00235	0.00401	0.00643	0.0098	0.01434	0.02032	0.02798	0.03764	0.0496	0.062
	P	0	36.9138	73.8277	110.742	147.655	184.569	221.483	258.397	295.311	332.225	369.138	406.052	442.966	479.88	516.794	553.708	553.7077
18.0	Y	0	9.8E-07	1.6E-05	7.9E-05	0.00025	0.00061	0.00127	0.00235	0.00401	0.00643	0.0098	0.01434	0.02032	0.02798	0.03764	0.0496	0.062
	P	0	37.4862	74.9723	112.458	149.945	187.431	224.917	262.403	299.889	337.375	374.862	412.348	449.834	487.32	524.806	562.292	562.2923
19.0	Y	0	9.8E-07	1.6E-05	7.9E-05	0.00025	0.00061	0.00127	0.00235	0.00401	0.00643	0.0098	0.01434	0.02032	0.02798	0.03764	0.0496	0.062
	P	0	38.0585	76.1169	114.175	152.234	190.292	228.351	266.409	304.468	342.526	380.585	418.643	456.702	494.76	532.818	570.877	570.8769
20.0	Y	0	9.8E-07	1.6E-05	7.9E-05	0.00025	0.00061	0.00127	0.00235	0.00401	0.00643	0.0098	0.01434	0.02032	0.02798	0.03764	0.0496	0.062
	P	0	38.6308	77.2615	115.892	154.523	193.154	231.785	270.415	309.046	347.677	386.308	424.938	463.569	502.2	540.831	579.462	579.4615
21.0	Y	0	9.8E-07	1.6E-05	7.9E-05	0.00025	0.00061	0.00127	0.00235	0.00401	0.00643	0.0098	0.01434	0.02032	0.02798	0.03764	0.0496	0.062
	P	0	39.2031	78.4062	117.609	156.812	196.015	235.218	274.422	313.625	352.828	392.031	431.234	470.437	509.64	548.843	588.046	588.0462
22.0	Y	0	9.8E-07	1.6E-05	7.9E-05	0.00025	0.00061	0.00127	0.00235	0.00401	0.00643	0.0098	0.01434	0.02032	0.02798	0.03764	0.0496	0.062
	P	0	39.7754	79.5508	119.326	159.102	198.877	238.652	278.428	318.203	357.978	397.754	437.529	477.305	517.08	556.855	596.631	596.6308
23.0	Y	0	9.8E-07	1.6E-05	7.9E-05	0.00025	0.00061	0.00127	0.00235	0.00401	0.00643	0.0098	0.01434	0.02032	0.02798	0.03764	0.0496	0.062
	P	0	40.3477	80.6954	121.043	161.391	201.738	242.086	282.434	322.782	363.129	403.477	443.825	484.172	524.52	564.868	605.215	605.2154
24.0	Y	0	9.8E-07	1.6E-05	7.9E-05	0.00025	0.00061	0.00127	0.00235	0.00401	0.00643	0.0098	0.01434	0.02032	0.02798	0.03764	0.0496	0.062
	P	0	40.92	81.84	122.76	163.68	204.6	245.52	286.44	327.36	368.28	409.2	450.12	491.04	531.96	572.88	613.8	613.8
25.0	Y	0	9.8E-07	1.6E-05	7.9E-05	0.00025	0.00061	0.00127	0.00235	0.00401	0.00643	0.0098	0.01434	0.02032	0.02798	0.03764	0.0496	0.062
	P	0	41.4923	82.9846	124.477	165.969	207.462	248.954	290.446	331.938	373.431	414.923	456.415	497.908	539.4	580.892	622.385	622.3846
26.0	Y	0	9.8E-07	1.6E-05	7.9E-05	0.00025	0.00061	0.00127	0.00235	0.00401	0.00643	0.0098	0.01434	0.02032	0.02798	0.03764	0.0496	0.062
	P	0	42.0646	84.1292	126.194	168.258	210.323	252.388	294.452	336.517	378.582	420.646	462.711	504.775	546.84	588.905	630.969	630.9692
27.0	Y	0	9.8E-07	1.6E-05	7.9E-05	0.00025	0.00061	0.00127	0.00235	0.00401	0.00643	0.0098	0.01434	0.02032	0.02798	0.03764	0.0496	0.062
	P	0	42.6369	85.2738	127.911	170.548	213.185	255.822	298.458	341.095	383.732	426.369	469.006	511.643	554.28	596.917	639.554	639.5539
28.0	Y	0	9.8E-07	1.6E-05	7.9E-05	0.00025	0.00061	0.00127	0.00235	0.00401	0.00643	0.0098	0.01434	0.02032	0.02798	0.03764	0.0496	0.062
	P	0	43.2092	86.4185	129.628	172.837	216.046	259.255	302.465	345.674	388.883	432.092						

Table E4: Load Intensity p (kN/m) vs Lateral Deflection y (m) Data Points for South Abutment - HP 360 x 132

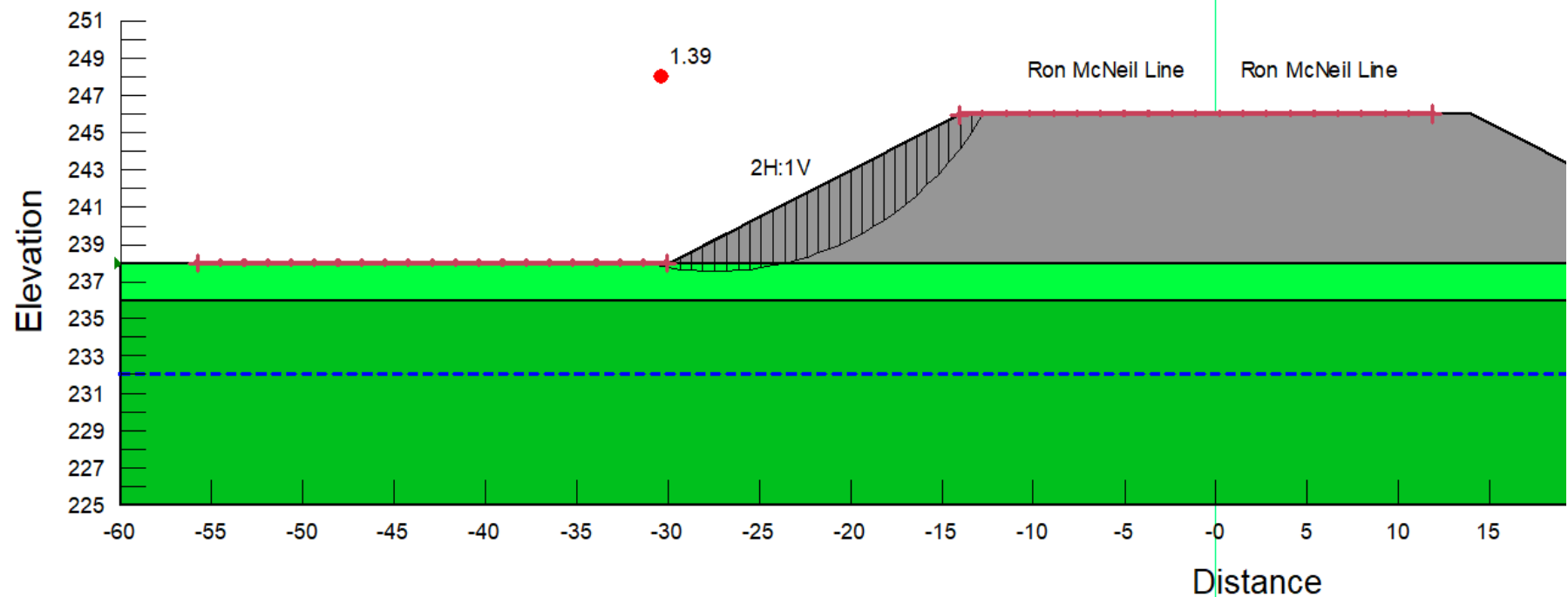
Depth Below Abutment Wall (m)	Curve Points																	
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16		
1.0	Y	0	0.00142	0.00185	0.00229	0.00273	0.00316	0.0036	0.00404	0.00447	0.00491	0.00534	0.00578	0.00622	0.0101	0.01399	0.01679	0.019583
	P	0	31.9866	35.2323	38.0191	40.4836	42.707	44.7418	46.6243	48.3806	50.0305	51.589	53.068	54.4772	66.7411	79.005	79.005	79.00495
2.0	Y	0	0.00061	0.00112	0.00163	0.00214	0.00265	0.00316	0.00367	0.00418	0.00469	0.0052	0.00571	0.00622	0.0101	0.01399	0.01679	0.019583
	P	0	27.726	40.0229	50.2355	59.2504	67.4543	75.0589	82.1957	88.9532	95.3945	101.567	107.506	113.241	156.272	199.304	199.304	199.3037
3.0	Y	0	2.1E-06	3.3E-05	0.00017	0.00053	0.00129	0.00267	0.00495	0.00845	0.01354	0.02063	0.0302	0.04278	0.05892	0.07925	0.10444	0.13055
	P	0	13.2825	26.565	39.8475	53.13	66.4125	79.695	92.9775	106.26	119.542	132.825	146.107	159.39	172.672	185.955	199.237	199.2374
4.0	Y	0	2.1E-06	3.3E-05	0.00017	0.00053	0.00129	0.00267	0.00495	0.00845	0.01354	0.02063	0.0302	0.04278	0.05892	0.07925	0.10444	0.13055
	P	0	13.428	26.856	40.284	53.712	67.14	80.568	93.996	107.424	120.852	134.28	147.708	161.136	174.564	187.992	201.42	201.42
5.0	Y	0	1.5E-06	2.4E-05	0.00012	0.00038	0.00092	0.00191	0.00354	0.00604	0.00967	0.01474	0.02157	0.03056	0.04209	0.05661	0.0746	0.09325
	P	0	20.8698	41.7396	62.6094	83.4792	104.349	125.219	146.089	166.958	187.828	208.698	229.568	250.438	271.308	292.177	313.047	313.0471
6.0	Y	0	1.5E-06	2.4E-05	0.00012	0.00038	0.00092	0.00191	0.00354	0.00604	0.00967	0.01474	0.02157	0.03056	0.04209	0.05661	0.0746	0.09325
	P	0	37.158	74.3159	111.474	148.632	185.79	222.948	260.106	297.264	334.422	371.58	408.737	445.895	483.053	520.211	557.369	557.3693
7.0	Y	0	1.5E-06	2.4E-05	0.00012	0.00038	0.00092	0.00191	0.00354	0.00604	0.00967	0.01474	0.02157	0.03056	0.04209	0.05661	0.0746	0.09325
	P	0	40.284	80.568	120.852	161.136	201.42	241.704	281.988	322.272	362.556	402.84	443.124	483.408	523.692	563.976	604.26	604.26
8.0	Y	0	1.5E-06	2.4E-05	0.00012	0.00038	0.00092	0.00191	0.00354	0.00604	0.00967	0.01474	0.02157	0.03056	0.04209	0.05661	0.0746	0.09325
	P	0	40.284	80.568	120.852	161.136	201.42	241.704	281.988	322.272	362.556	402.84	443.124	483.408	523.692	563.976	604.26	604.26
9.0	Y	0	1.5E-06	2.4E-05	0.00012	0.00038	0.00092	0.00191	0.00354	0.00604	0.00967	0.01474	0.02157	0.03056	0.04209	0.05661	0.0746	0.09325
	P	0	40.284	80.568	120.852	161.136	201.42	241.704	281.988	322.272	362.556	402.84	443.124	483.408	523.692	563.976	604.26	604.26
10.0	Y	0	1.5E-06	2.4E-05	0.00012	0.00038	0.00092	0.00191	0.00354	0.00604	0.00967	0.01474	0.02157	0.03056	0.04209	0.05661	0.0746	0.09325
	P	0	40.284	80.568	120.852	161.136	201.42	241.704	281.988	322.272	362.556	402.84	443.124	483.408	523.692	563.976	604.26	604.26
11.0	Y	0	1.5E-06	2.4E-05	0.00012	0.00038	0.00092	0.00191	0.00354	0.00604	0.00967	0.01474	0.02157	0.03056	0.04209	0.05661	0.0746	0.09325
	P	0	40.284	80.568	120.852	161.136	201.42	241.704	281.988	322.272	362.556	402.84	443.124	483.408	523.692	563.976	604.26	604.26
12.0	Y	0	1.5E-06	2.4E-05	0.00012	0.00038	0.00092	0.00191	0.00354	0.00604	0.00967	0.01474	0.02157	0.03056	0.04209	0.05661	0.0746	0.09325
	P	0	40.284	80.568	120.852	161.136	201.42	241.704	281.988	322.272	362.556	402.84	443.124	483.408	523.692	563.976	604.26	604.26
13.0	Y	0	1.5E-06	2.4E-05	0.00012	0.00038	0.00092	0.00191	0.00354	0.00604	0.00967	0.01474	0.02157	0.03056	0.04209	0.05661	0.0746	0.09325
	P	0	40.284	80.568	120.852	161.136	201.42	241.704	281.988	322.272	362.556	402.84	443.124	483.408	523.692	563.976	604.26	604.26
14.0	Y	0	1.5E-06	2.4E-05	0.00012	0.00038	0.00092	0.00191	0.00354	0.00604	0.00967	0.01474	0.02157	0.03056	0.04209	0.05661	0.0746	0.09325
	P	0	40.284	80.568	120.852	161.136	201.42	241.704	281.988	322.272	362.556	402.84	443.124	483.408	523.692	563.976	604.26	604.26
15.0	Y	0	1.5E-06	2.4E-05	0.00012	0.00038	0.00092	0.00191	0.00354	0.00604	0.00967	0.01474	0.02157	0.03056	0.04209	0.05661	0.0746	0.09325
	P	0	40.284	80.568	120.852	161.136	201.42	241.704	281.988	322.272	362.556	402.84	443.124	483.408	523.692	563.976	604.26	604.26
16.0	Y	0	1.5E-06	2.4E-05	0.00012	0.00038	0.00092	0.00191	0.00354	0.00604	0.00967	0.01474	0.02157	0.03056	0.04209	0.05661	0.0746	0.09325
	P	0	40.284	80.568	120.852	161.136	201.42	241.704	281.988	322.272	362.556	402.84	443.124	483.408	523.692	563.976	604.26	604.26
17.0	Y	0	1.5E-06	2.4E-05	0.00012	0.00038	0.00092	0.00191	0.00354	0.00604	0.00967	0.01474	0.02157	0.03056	0.04209	0.05661	0.0746	0.09325
	P	0	41.0739	82.1478	123.222	164.296	205.369	246.443	287.517	328.591	369.665	410.739	451.813	492.887	533.96	575.034	616.108	616.1082
18.0	Y	0	1.5E-06	2.4E-05	0.00012	0.00038	0.00092	0.00191	0.00354	0.00604	0.00967	0.01474	0.02157	0.03056	0.04209	0.05661	0.0746	0.09325
	P	0	41.8638	83.7275	125.591	167.455	209.319	251.183	293.046	334.91	376.774	418.638	460.501	502.365	544.229	586.093	627.956	627.9565
19.0	Y	0	1.5E-06	2.4E-05	0.00012	0.00038	0.00092	0.00191	0.00354	0.00604	0.00967	0.01474	0.02157	0.03056	0.04209	0.05661	0.0746	0.09325
	P	0	42.6536	85.3073	127.961	170.615	213.268	255.922	298.576	341.229	383.883	426.536	469.19	511.844	554.497	597.151	639.805	639.8047
20.0	Y	0	1.2E-06	1.9E-05	9.5E-05	0.0003	0.00074	0.00153	0.00283	0.00483	0.00773	0.01179	0.01726	0.02444	0.03367	0.04529	0.05968	0.0746
	P	0	43.4435	86.8871	130.331	173.774	217.218	260.661	304.105	347.548	390.992	434.435	477.879	521.322	564.766	608.209	651.653	651.6529
21.0	Y	0	1.2E-06	1.9E-05	9.5E-05	0.0003	0.00074	0.00153	0.00283	0.00483	0.00773	0.01179	0.01726	0.02444	0.03367	0.04529	0.05968	0.0746
	P	0	44.2334	88.4668	132.7	176.934	221.167	265.4	309.634	353.867	398.101	442.334	486.568	530.801	575.034	619.268	663.501	663.5012
22.0	Y	0	1.2E-06	1.9E-05	9.5E-05	0.0003	0.00074	0.00153	0.00283	0.00483	0.00773	0.01179	0.01726	0.02444	0.03367	0.04529	0.05968	0.0746
	P	0	45.0233	90.0466	135.07	180.093	225.116	270.14	315.163	360.186	405.211	450.233	495.256	540.28	585.303	630.326	675.349	675.3494
23.0	Y	0	1.2E-06	1.9E-05	9.5E-05	0.0003	0.00074	0.00153	0.00283	0.00483	0.00773	0.01179	0.01726	0.02444	0.03367	0.04529	0.05968	0.0746
	P	0	45.8132	91.6264	137.44	183.253	229.066	274.879	320.692	366.505	412.319	458.132	503.945	549.758	595.571	641.384	687.198	687.1977
24.0	Y	0	1.2E-06	1.9E-05	9.5E-05	0.0003	0.00074	0.00153	0.00283	0.00483	0.00773	0.01179	0.01726	0.02444	0.03367	0.04529	0.05968	0.0746
	P	0	46.6031	93.2061	139.809	186.412	233.015	279.618	326.221	372.824	419.428	466.031	512.634	559.237	605.84	652.443	699.046	699.0459
25.0	Y	0	1.2E-06	1.9E-05	9.5E-05	0.0003	0.00074	0.00153	0.00283	0.00483	0.00773	0.01179	0.01726	0.02444	0.03367	0.04529	0.05968	0.0746
	P	0	47.3929	94.7859	142.179	189.572	236.965	284.358	331.751	379.144	426.536	473.929	521.322	568.715	616.108	663.501	710.894	710.8941
26.0	Y	0	1.2E-06	1.9E-05	9.5E-05	0.0003	0.00074	0.00153	0.00283	0.00483	0.00773	0.01179	0.01726	0.02444	0.03367	0.04529	0.05968	0.0746
	P	0	48.1828	96.3656	144.548	192.731	240.914	289.097	337.28	385.463	433.645	481.828	530.011	578.194	626.377	674.56	722.742	722.7424
27.0	Y	0	1.2E-06	1.9E-05	9.5E-05	0.0003	0.00074	0.00153	0.00283	0.00483	0.00773	0.01179	0.01726	0.02444	0.03367	0.04529	0.05968	0.0746
	P	0	48.9727	97.9454	146.918	195.891	244.864	293.836	342.809	391.782	440.754	489.727	538.7	587.662	636.645	685.618	734.591	734.5906

Table E5: Load Intensity p (kN/m) vs Lateral Deflection y (m) Data Points for North Abutment - HP 360 x 132

Depth Below Abutment Wall (m)		Curve Points																
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	
1.0	Y	0	0.00142	0.00185	0.00229	0.00273	0.00316	0.0036	0.00404	0.00447	0.00491	0.00534	0.00578	0.00622	0.0101	0.01399	0.01679	0.019583
	P	0	31.9866	35.2323	38.0191	40.4836	42.707	44.7418	46.6243	48.3806	50.0305	51.589	53.068	54.4772	66.7411	79.005	79.0069	79.00495
2.0	Y	0	0.00061	0.00112	0.00163	0.00214	0.00265	0.00316	0.00367	0.00418	0.00469	0.0052	0.00571	0.00622	0.0101	0.01399	0.01679	0.019583
	P	0	27.726	40.0229	50.2355	59.2504	67.4543	75.0589	82.1957	88.9532	95.3945	101.567	107.506	113.241	156.272	199.304	199.3034	199.30307
3.0	Y	0	0.00146	0.00189	0.00232	0.00276	0.00319	0.00362	0.00405	0.00449	0.00492	0.00535	0.00578	0.00622	0.0101	0.01399	0.01679	0.019583
	P	0	98.6399	115.533	130.962	145.296	158.771	171.545	183.732	195.419	206.672	217.542	228.073	238.3	328.853	419.407	419.4073	419.40707
4.0	Y	0	2.1E-06	3.3E-05	0.00017	0.00053	0.00129	0.00267	0.00495	0.00845	0.01354	0.02063	0.0302	0.04278	0.05892	0.07925	0.10444	0.13055
	P	0	13.428	26.856	40.284	53.712	67.14	80.568	93.996	107.424	120.852	134.28	147.708	161.136	174.564	187.992	201.42	201.42
5.0	Y	0	2.1E-06	3.3E-05	0.00017	0.00053	0.00129	0.00267	0.00495	0.00845	0.01354	0.02063	0.0302	0.04278	0.05892	0.07925	0.10444	0.13055
	P	0	13.428	26.856	40.284	53.712	67.14	80.568	93.996	107.424	120.852	134.28	147.708	161.136	174.564	187.992	201.42	201.42
6.0	Y	0	1.5E-06	2.4E-05	0.00012	0.00038	0.00092	0.00191	0.00354	0.00604	0.00967	0.01474	0.02157	0.03056	0.04209	0.05661	0.0746	0.09325
	P	0	25.856	51.7119	77.5679	103.424	129.28	155.136	180.992	206.848	232.704	258.56	284.416	310.272	336.128	361.983	387.839	387.8394
7.0	Y	0	1.5E-06	2.4E-05	0.00012	0.00038	0.00092	0.00191	0.00354	0.00604	0.00967	0.01474	0.02157	0.03056	0.04209	0.05661	0.0746	0.09325
	P	0	40.284	80.568	120.852	161.136	201.42	241.704	281.988	322.272	362.556	402.84	443.124	483.408	523.692	563.976	604.26	604.26
8.0	Y	0	1.5E-06	2.4E-05	0.00012	0.00038	0.00092	0.00191	0.00354	0.00604	0.00967	0.01474	0.02157	0.03056	0.04209	0.05661	0.0746	0.09325
	P	0	40.284	80.568	120.852	161.136	201.42	241.704	281.988	322.272	362.556	402.84	443.124	483.408	523.692	563.976	604.26	604.26
9.0	Y	0	1.5E-06	2.4E-05	0.00012	0.00038	0.00092	0.00191	0.00354	0.00604	0.00967	0.01474	0.02157	0.03056	0.04209	0.05661	0.0746	0.09325
	P	0	40.284	80.568	120.852	161.136	201.42	241.704	281.988	322.272	362.556	402.84	443.124	483.408	523.692	563.976	604.26	604.26
10.0	Y	0	1.5E-06	2.4E-05	0.00012	0.00038	0.00092	0.00191	0.00354	0.00604	0.00967	0.01474	0.02157	0.03056	0.04209	0.05661	0.0746	0.09325
	P	0	40.284	80.568	120.852	161.136	201.42	241.704	281.988	322.272	362.556	402.84	443.124	483.408	523.692	563.976	604.26	604.26
11.0	Y	0	1.5E-06	2.4E-05	0.00012	0.00038	0.00092	0.00191	0.00354	0.00604	0.00967	0.01474	0.02157	0.03056	0.04209	0.05661	0.0746	0.09325
	P	0	40.284	80.568	120.852	161.136	201.42	241.704	281.988	322.272	362.556	402.84	443.124	483.408	523.692	563.976	604.26	604.26
12.0	Y	0	1.5E-06	2.4E-05	0.00012	0.00038	0.00092	0.00191	0.00354	0.00604	0.00967	0.01474	0.02157	0.03056	0.04209	0.05661	0.0746	0.09325
	P	0	40.284	80.568	120.852	161.136	201.42	241.704	281.988	322.272	362.556	402.84	443.124	483.408	523.692	563.976	604.26	604.26
13.0	Y	0	1.5E-06	2.4E-05	0.00012	0.00038	0.00092	0.00191	0.00354	0.00604	0.00967	0.01474	0.02157	0.03056	0.04209	0.05661	0.0746	0.09325
	P	0	40.284	80.568	120.852	161.136	201.42	241.704	281.988	322.272	362.556	402.84	443.124	483.408	523.692	563.976	604.26	604.26
14.0	Y	0	1.5E-06	2.4E-05	0.00012	0.00038	0.00092	0.00191	0.00354	0.00604	0.00967	0.01474	0.02157	0.03056	0.04209	0.05661	0.0746	0.09325
	P	0	40.284	80.568	120.852	161.136	201.42	241.704	281.988	322.272	362.556	402.84	443.124	483.408	523.692	563.976	604.26	604.26
15.0	Y	0	1.5E-06	2.4E-05	0.00012	0.00038	0.00092	0.00191	0.00354	0.00604	0.00967	0.01474	0.02157	0.03056	0.04209	0.05661	0.0746	0.09325
	P	0	40.284	80.568	120.852	161.136	201.42	241.704	281.988	322.272	362.556	402.84	443.124	483.408	523.692	563.976	604.26	604.26
16.0	Y	0	1.5E-06	2.4E-05	0.00012	0.00038	0.00092	0.00191	0.00354	0.00604	0.00967	0.01474	0.02157	0.03056	0.04209	0.05661	0.0746	0.09325
	P	0	40.284	80.568	120.852	161.136	201.42	241.704	281.988	322.272	362.556	402.84	443.124	483.408	523.692	563.976	604.26	604.26
17.0	Y	0	1.5E-06	2.4E-05	0.00012	0.00038	0.00092	0.00191	0.00354	0.00604	0.00967	0.01474	0.02157	0.03056	0.04209	0.05661	0.0746	0.09325
	P	0	40.284	80.568	120.852	161.136	201.42	241.704	281.988	322.272	362.556	402.84	443.124	483.408	523.692	563.976	604.26	604.26
18.0	Y	0	1.5E-06	2.4E-05	0.00012	0.00038	0.00092	0.00191	0.00354	0.00604	0.00967	0.01474	0.02157	0.03056	0.04209	0.05661	0.0746	0.09325
	P	0	41.0978	82.1956	123.293	164.391	205.489	246.587	287.685	328.783	369.88	410.978	452.076	493.174	534.272	575.369	616.467	616.4673
19.0	Y	0	1.5E-06	2.4E-05	0.00012	0.00038	0.00092	0.00191	0.00354	0.00604	0.00967	0.01474	0.02157	0.03056	0.04209	0.05661	0.0746	0.09325
	P	0	41.9116	83.8233	125.735	167.647	209.558	251.47	293.381	335.293	377.205	419.116	461.028	502.94	544.851	586.763	628.675	628.6746
20.0	Y	0	1.5E-06	2.4E-05	0.00012	0.00038	0.00092	0.00191	0.00354	0.00604	0.00967	0.01474	0.02157	0.03056	0.04209	0.05661	0.0746	0.09325
	P	0	42.7255	85.4509	128.176	170.902	213.627	256.353	299.078	341.804	384.529	427.255	469.98	512.705	555.431	598.156	640.882	640.8818
21.0	Y	0	1.2E-06	1.9E-05	9.5E-05	0.0003	0.00074	0.00153	0.00283	0.00483	0.00773	0.01179	0.01726	0.02444	0.03367	0.04529	0.05968	0.0746
	P	0	43.5393	87.0785	130.618	174.157	217.696	261.236	304.775	348.314	391.853	435.393	478.932	522.471	566.011	609.55	653.089	653.0891
22.0	Y	0	1.2E-06	1.9E-05	9.5E-05	0.0003	0.00074	0.00153	0.00283	0.00483	0.00773	0.01179	0.01726	0.02444	0.03367	0.04529	0.05968	0.0746
	P	0	44.3531	88.7062	133.059	177.412	221.765	266.119	310.472	354.825	399.178	443.531	487.884	532.237	576.59	620.943	665.296	665.2964
23.0	Y	0	1.2E-06	1.9E-05	9.5E-05	0.0003	0.00074	0.00153	0.00283	0.00483	0.00773	0.01179	0.01726	0.02444	0.03367	0.04529	0.05968	0.0746
	P	0	45.1669	90.3338	135.501	180.668	225.835	271.001	316.168	361.335	406.502	451.669	496.836	542.003	587.17	632.337	677.504	677.5036
24.0	Y	0	1.2E-06	1.9E-05	9.5E-05	0.0003	0.00074	0.00153	0.00283	0.00483	0.00773	0.01179	0.01726	0.02444	0.03367	0.04529	0.05968	0.0746
	P	0	45.9807	91.9615	137.942	183.923	229.904	275.884	321.865	367.846	413.827	459.807	505.788	551.769	597.749	643.73	689.711	689.7105
25.0	Y	0	1.2E-06	1.9E-05	9.5E-05	0.0003	0.00074	0.00153	0.00283	0.00483	0.00773	0.01179	0.01726	0.02444	0.03367	0.04529	0.05968	0.0746
	P	0	46.7945	93.5891	140.384	187.178	233.973	280.767	327.562	374.356	421.151	467.945	514.74	561.535	608.329	655.124	701.918	701.9182
26.0	Y	0	1.2E-06	1.9E-05	9.5E-05	0.0003	0.00074	0.00153	0.00283	0.00483	0.00773	0.01179	0.01726	0.02444	0.03367	0.04529	0.05968	0.0746
	P	0	47.6084	95.2167	142.825	190.433	238.042	285.65	333.259	380.867	428.475	476.084	523.692	571.3	618.909	666.517	714.125	714.1255
27.0	Y	0	1.2E-06	1.9E-05	9.5E-05	0.0003	0.00074	0.00153	0.00283	0.00483	0.00773	0.01179	0.01726	0.02444	0.03367	0.04529	0.05968	0.0746
	P	0	48.4222	96.8444	145.267	193.689	242.111	290.533	338.955	387.377	435.8	484.222	532.644	581.066	629.488	677.911	726.333	726.3327

Table E6: Load Intensity p (kN/m) vs Lateral Deflection y (m) Data Points for Pier - HP 360 x 132

Depth Below Abutment Wall (m)		Curve Points																
			1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
1.0	Y	0	1.5E-06	2.4E-05	0.00012	0.00038	0.00092	0.00191	0.00354	0.00604	0.00967	0.01474	0.02157	0.03056	0.04209	0.05661	0.0746	0.09325
	P	0	19.9626	39.9253	59.8879	79.8505	99.8132	119.776	139.738	159.701	179.664	199.626	219.589	239.552	259.514	279.477	299.44	299.4395
2.0	Y	0	1.5E-06	2.4E-05	0.00012	0.00038	0.00092	0.00191	0.00354	0.00604	0.00967	0.01474	0.02157	0.03056	0.04209	0.05661	0.0746	0.09325
	P	0	26.4973	52.9945	79.4918	105.989	132.486	158.984	185.481	211.978	238.475	264.973	291.47	317.967	344.464	370.962	397.459	397.459
3.0	Y	0	1.5E-06	2.4E-05	0.00012	0.00038	0.00092	0.00191	0.00354	0.00604	0.00967	0.01474	0.02157	0.03056	0.04209	0.05661	0.0746	0.09325
	P	0	32.7882	65.5764	98.3646	131.153	163.941	196.729	229.517	262.306	295.094	327.882	360.67	393.458	426.247	459.035	491.823	491.8231
4.0	Y	0	1.5E-06	2.4E-05	0.00012	0.00038	0.00092	0.00191	0.00354	0.00604	0.00967	0.01474	0.02157	0.03056	0.04209	0.05661	0.0746	0.09325
	P	0	39.0791	78.1583	117.237	156.317	195.396	234.475	273.554	312.633	351.712	390.791	429.871	468.95	508.029	547.108	586.187	586.1872
5.0	Y	0	1.5E-06	2.4E-05	0.00012	0.00038	0.00092	0.00191	0.00354	0.00604	0.00967	0.01474	0.02157	0.03056	0.04209	0.05661	0.0746	0.09325
	P	0	40.284	80.568	120.852	161.136	201.42	241.704	281.988	322.272	362.556	402.84	443.124	483.408	523.692	563.976	604.26	604.26
6.0	Y	0	1.5E-06	2.4E-05	0.00012	0.00038	0.00092	0.00191	0.00354	0.00604	0.00967	0.01474	0.02157	0.03056	0.04209	0.05661	0.0746	0.09325
	P	0	40.284	80.568	120.852	161.136	201.42	241.704	281.988	322.272	362.556	402.84	443.124	483.408	523.692	563.976	604.26	604.26
7.0	Y	0	1.5E-06	2.4E-05	0.00012	0.00038	0.00092	0.00191	0.00354	0.00604	0.00967	0.01474	0.02157	0.03056	0.04209	0.05661	0.0746	0.09325
	P	0	40.284	80.568	120.852	161.136	201.42	241.704	281.988	322.272	362.556	402.84	443.124	483.408	523.692	563.976	604.26	604.26
8.0	Y	0	1.5E-06	2.4E-05	0.00012	0.00038	0.00092	0.00191	0.00354	0.00604	0.00967	0.01474	0.02157	0.03056	0.04209	0.05661	0.0746	0.09325
	P	0	40.284	80.568	120.852	161.136	201.42	241.704	281.988	322.272	362.556	402.84	443.124	483.408	523.692	563.976	604.26	604.26
9.0	Y	0	1.5E-06	2.4E-05	0.00012	0.00038	0.00092	0.00191	0.00354	0.00604	0.00967	0.01474	0.02157	0.03056	0.04209	0.05661	0.0746	0.09325
	P	0	40.284	80.568	120.852	161.136	201.42	241.704	281.988	322.272	362.556	402.84	443.124	483.408	523.692	563.976	604.26	604.26
10.0	Y	0	1.5E-06	2.4E-05	0.00012	0.00038	0.00092	0.00191	0.00354	0.00604	0.00967	0.01474	0.02157	0.03056	0.04209	0.05661	0.0746	0.09325
	P	0	40.284	80.568	120.852	161.136	201.42	241.704	281.988	322.272	362.556	402.84	443.124	483.408	523.692	563.976	604.26	604.26
11.0	Y	0	1.5E-06	2.4E-05	0.00012	0.00038	0.00092	0.00191	0.00354	0.00604	0.00967	0.01474	0.02157	0.03056	0.04209	0.05661	0.0746	0.09325
	P	0	40.284	80.568	120.852	161.136	201.42	241.704	281.988	322.272	362.556	402.84	443.124	483.408	523.692	563.976	604.26	604.26
12.0	Y	0	1.5E-06	2.4E-05	0.00012	0.00038	0.00092	0.00191	0.00354	0.00604	0.00967	0.01474	0.02157	0.03056	0.04209	0.05661	0.0746	0.09325
	P	0	40.9726	81.9452	122.918	163.89	204.863	245.836	286.808	327.781	368.754	409.726	450.699	491.671	532.644	573.617	614.589	614.5892
13.0	Y	0	1.5E-06	2.4E-05	0.00012	0.00038	0.00092	0.00191	0.00354	0.00604	0.00967	0.01474	0.02157	0.03056	0.04209	0.05661	0.0746	0.09325
	P	0	41.6612	83.3225	124.984	166.645	208.306	249.967	291.629	333.29	374.951	416.612	458.274	499.935	541.596	583.257	624.918	624.9185
14.0	Y	0	1.5E-06	2.4E-05	0.00012	0.00038	0.00092	0.00191	0.00354	0.00604	0.00967	0.01474	0.02157	0.03056	0.04209	0.05661	0.0746	0.09325
	P	0	42.3498	84.6997	127.05	169.399	211.749	254.099	296.449	338.799	381.149	423.498	465.848	508.198	550.548	592.898	635.248	635.2477
15.0	Y	0	1.2E-06	1.9E-05	9.5E-05	0.0003	0.00074	0.00153	0.00283	0.00483	0.00773	0.01179	0.01726	0.02444	0.03367	0.04529	0.05968	0.0746
	P	0	43.0385	86.0769	129.115	172.154	215.192	258.231	301.269	344.308	387.346	430.385	473.423	516.462	559.5	602.538	645.577	645.5769
16.0	Y	0	1.2E-06	1.9E-05	9.5E-05	0.0003	0.00074	0.00153	0.00283	0.00483	0.00773	0.01179	0.01726	0.02444	0.03367	0.04529	0.05968	0.0746
	P	0	43.7271	87.4542	131.181	174.908	218.635	262.362	306.09	349.817	393.544	437.271	480.998	524.725	568.452	612.179	655.906	655.9062
17.0	Y	0	1.2E-06	1.9E-05	9.5E-05	0.0003	0.00074	0.00153	0.00283	0.00483	0.00773	0.01179	0.01726	0.02444	0.03367	0.04529	0.05968	0.0746
	P	0	44.4157	88.8314	133.247	177.663	222.078	266.494	310.91	355.326	399.741	444.157	488.573	532.988	577.404	621.82	666.235	666.2354
18.0	Y	0	1.2E-06	1.9E-05	9.5E-05	0.0003	0.00074	0.00153	0.00283	0.00483	0.00773	0.01179	0.01726	0.02444	0.03367	0.04529	0.05968	0.0746
	P	0	45.1043	90.2086	135.313	180.417	225.522	270.626	315.73	360.834	405.939	451.043	496.147	541.252	586.356	631.46	676.565	676.5646
19.0	Y	0	1.2E-06	1.9E-05	9.5E-05	0.0003	0.00074	0.00153	0.00283	0.00483	0.00773	0.01179	0.01726	0.02444	0.03367	0.04529	0.05968	0.0746
	P	0	45.7929	91.5858	137.379	183.172	228.965	274.758	320.55	366.343	412.136	457.929	503.722	549.515	595.308	641.101	686.894	686.8939
20.0	Y	0	1.2E-06	1.9E-05	9.5E-05	0.0003	0.00074	0.00153	0.00283	0.00483	0.00773	0.01179	0.01726	0.02444	0.03367	0.04529	0.05968	0.0746
	P	0	46.4815	92.9631	139.445	185.926	232.408	278.889	325.371	371.852	418.334	464.815	511.297	557.778	604.26	650.742	697.223	697.2231
21.0	Y	0	1.2E-06	1.9E-05	9.5E-05	0.0003	0.00074	0.00153	0.00283	0.00483	0.00773	0.01179	0.01726	0.02444	0.03367	0.04529	0.05968	0.0746
	P	0	47.1702	94.3403	141.51	188.681	235.851	283.021	330.191	377.361	424.531	471.702	518.872	566.042	613.212	660.382	707.552	707.5523
22.0	Y	0	1.2E-06	1.9E-05	9.5E-05	0.0003	0.00074	0.00153	0.00283	0.00483	0.00773	0.01179	0.01726	0.02444	0.03367	0.04529	0.05968	0.0746
	P	0	47.8588	95.7175	143.576	191.435	239.294	287.153	335.011	382.87	430.729	478.588	526.446	574.305	622.164	670.023	717.882	717.8815
23.0	Y	0	1.2E-06	1.9E-05	9.5E-05	0.0003	0.00074	0.00153	0.00283	0.00483	0.00773	0.01179	0.01726	0.02444	0.03367	0.04529	0.05968	0.0746
	P	0	48.5474	97.0948	145.642	194.19	242.737	291.284	339.832	388.379	436.926	485.474	534.021	582.569	631.116	679.663	728.211	728.2108
24.0	Y	0	1.2E-06	1.9E-05	9.5E-05	0.0003	0.00074	0.00153	0.00283	0.00483	0.00773	0.01179	0.01726	0.02444	0.03367	0.04529	0.05968	0.0746
	P	0	49.236	98.472	147.708	196.944	246.18	295.416	344.652	393.888	443.124	492.36	541.596	590.832	640.068	689.304	738.54	738.54
25.0	Y	0	1.2E-06	1.9E-05	9.5E-05	0.0003	0.00074	0.00153	0.00283	0.00483	0.00773	0.01179	0.01726	0.02444	0.03367	0.04529	0.05968	0.0746
	P	0	49.9246	99.8492	149.774	199.698	249.623	299.548	349.472	399.397	449.322	499.246	549.171	599.095	649.02	698.945	748.869	748.8692
26.0	Y	0	1.2E-06	1.9E-05	9.5E-05	0.0003	0.00074	0.00153	0.00283	0.00483	0.00773	0.01179	0.01726	0.02444	0.03367	0.04529	0.05968	0.0746
	P	0	50.6132	101.226	151.84	202.453	253.066	303.679	354.293	404.906	455.519	506.132	556.746	607.359	657.972	708.585	759.198	759.1985
27.0	Y	0	1.2E-06	1.9E-05	9.5E-05	0.0003	0.00074	0.00153	0.00283	0.00483	0.00773	0.01179	0.01726	0.02444	0.03367	0.04529	0.05968	0.0746
	P	0	51.3018	102.604	153.906	205.207	256.509	307.811	359.113	410.415	461.717	513.018	564.32	615.622	666.924	718.226	769.528	769.5277



Color	Name	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
■	Firm CLAYEY SILT (CL) TILL (Drained)	21	0	28
■	New Granular Embankment Fill	21	0	30
■	Stiff to hard CLAYEY SILT (CL) TILL (Drained)	21.5	0	30

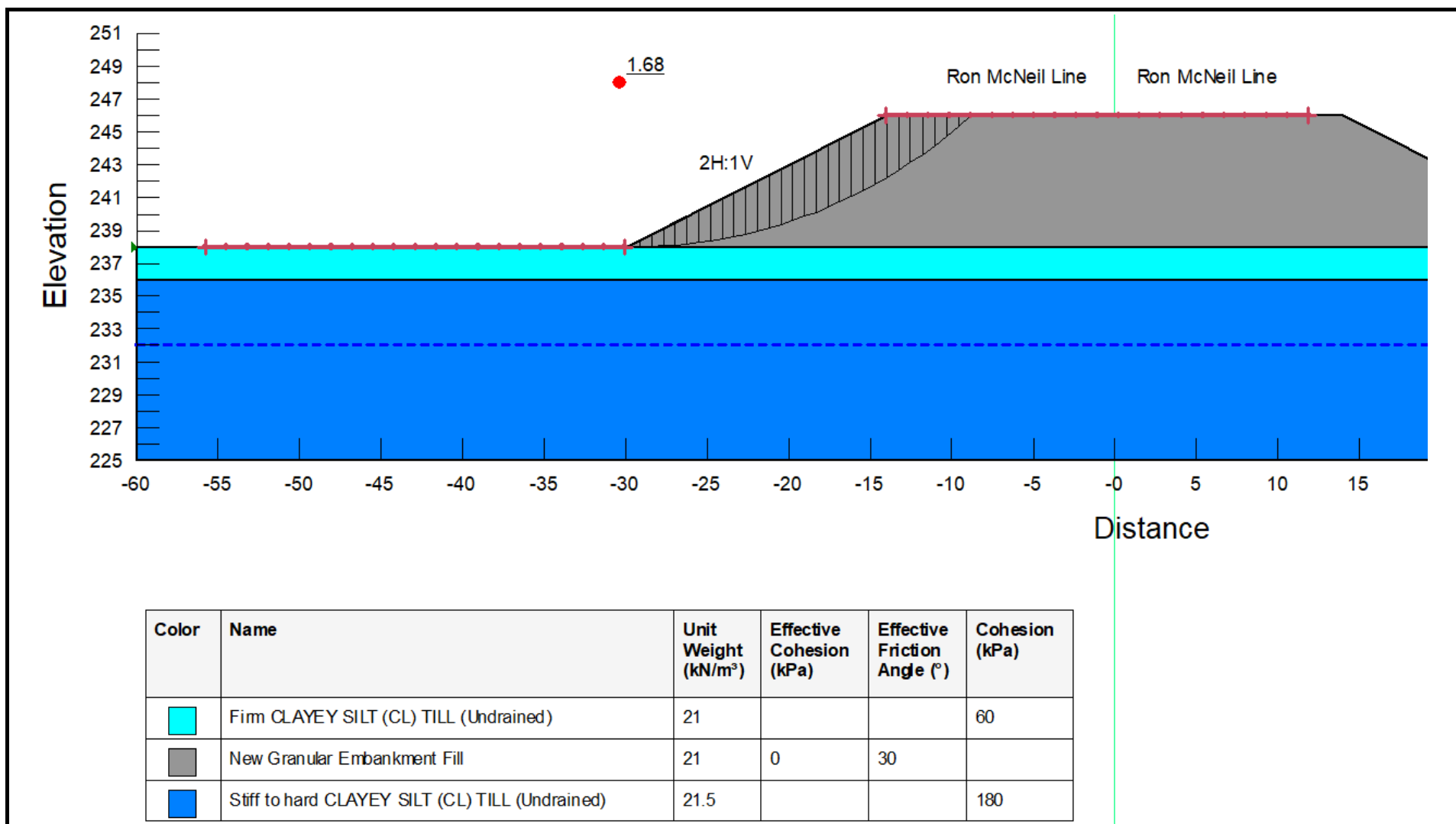


Slope Stability Analysis (Static)
 (Drained Conditions - Granular Embankment Fill)
 North Approach Embankment
 Talbotville Bypass - Ron McNeil Line Underpass

Figure E9

Project No. 165001308

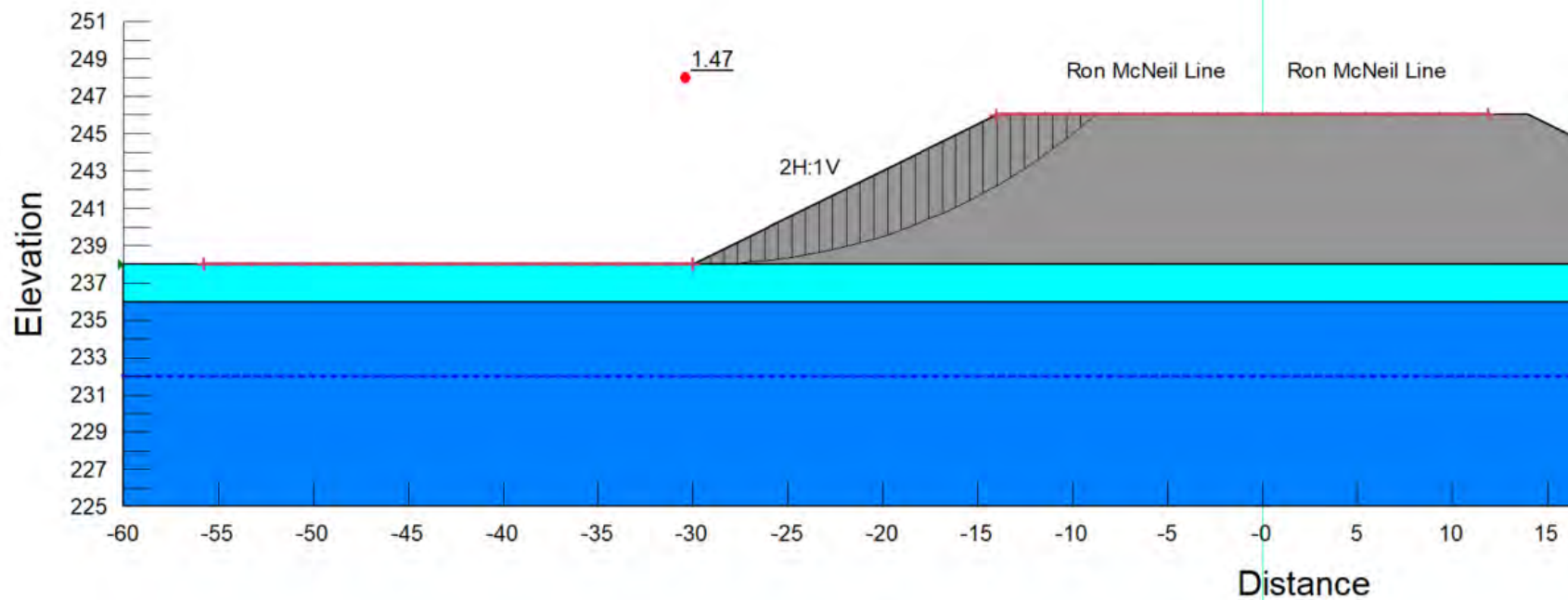
GWP No. 3042-22-00






Slope Stability Analysis (Static)
 (Undrained Conditions - Granular Embankment Fill)
 North Approach Embankment
 Talbotville Bypass - Ron McNeil Line Underpass

Figure E10

Project No. 165001308
GWP No. 3042-22-00



Color	Name	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Cohesion (kPa)
	Firm CLAYEY SILT (CL) TILL (Undrained)	21			60
	New Granular Embankment Fill	21	0	30	
	Stiff to hard CLAYEY SILT (CL) TILL (Undrained)	21.5			180

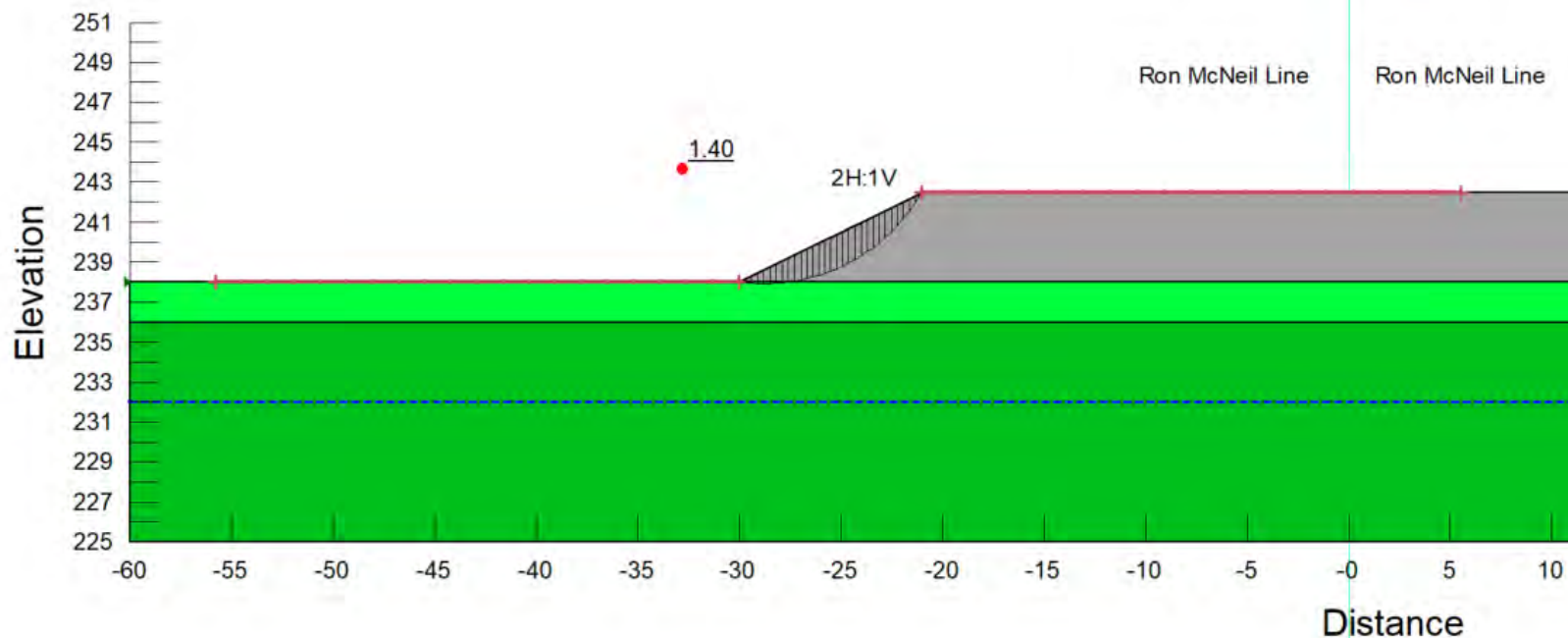


Slope Stability Analysis (Pseudo-static)
 (Undrained Conditions - Granular Embankment Fill)
 North Approach Embankment
 Talbotville Bypass - Ron McNeil Line Underpass

Figure E11

Project No. 165001308

GWP No. 3042-22-00



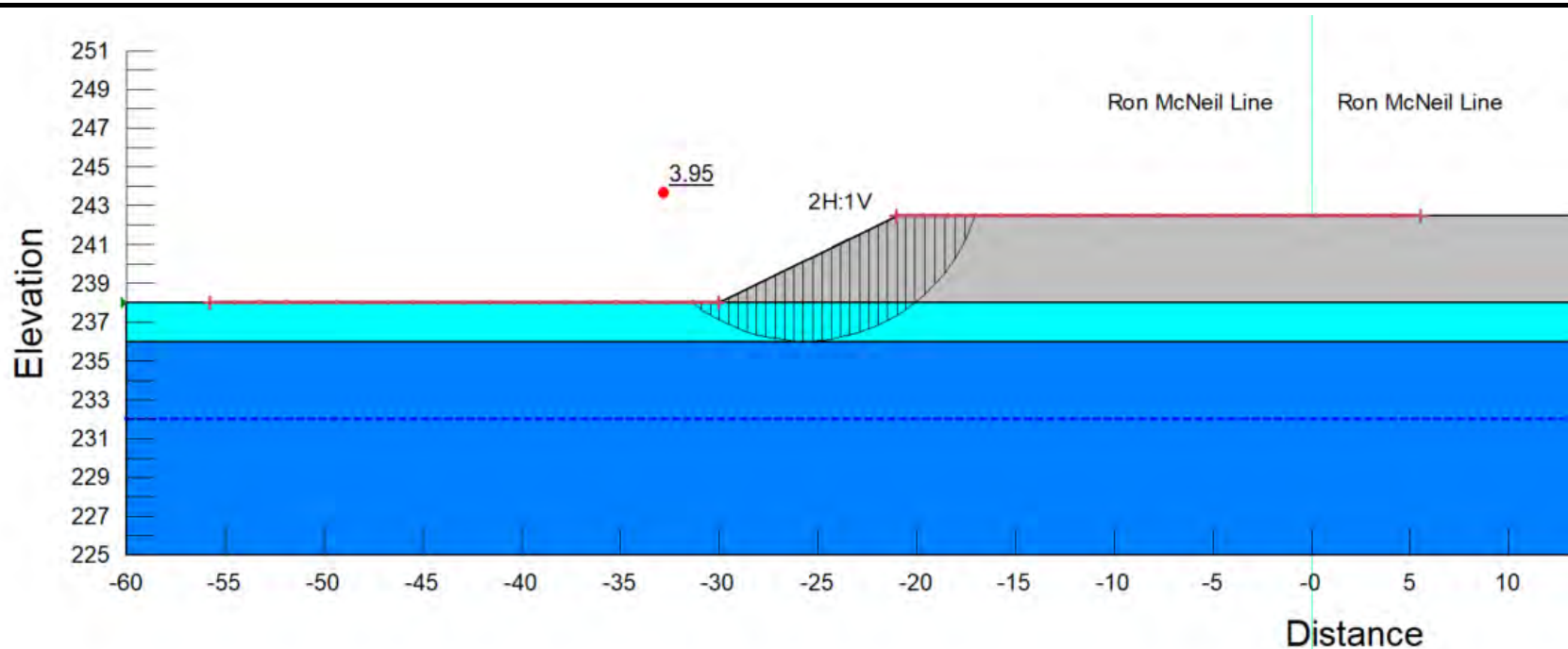
Color	Name	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
■	Firm CLAYEY SILT (CL) TILL (Drained)	21	0	28
■	New Cohesive Embankment Fill (Drained)	20.5	2.5	26
■	Stiff to hard CLAYEY SILT (CL) TILL (Drained)	21.5	0	30



Slope Stability Analysis (Static)
(Drained Conditions - Cohesive Embankment Fill)
Talbotville Bypass - Ron McNeil Line Underpass

Figure E12

Project No. 165001308
GWP No. 3042-22-00



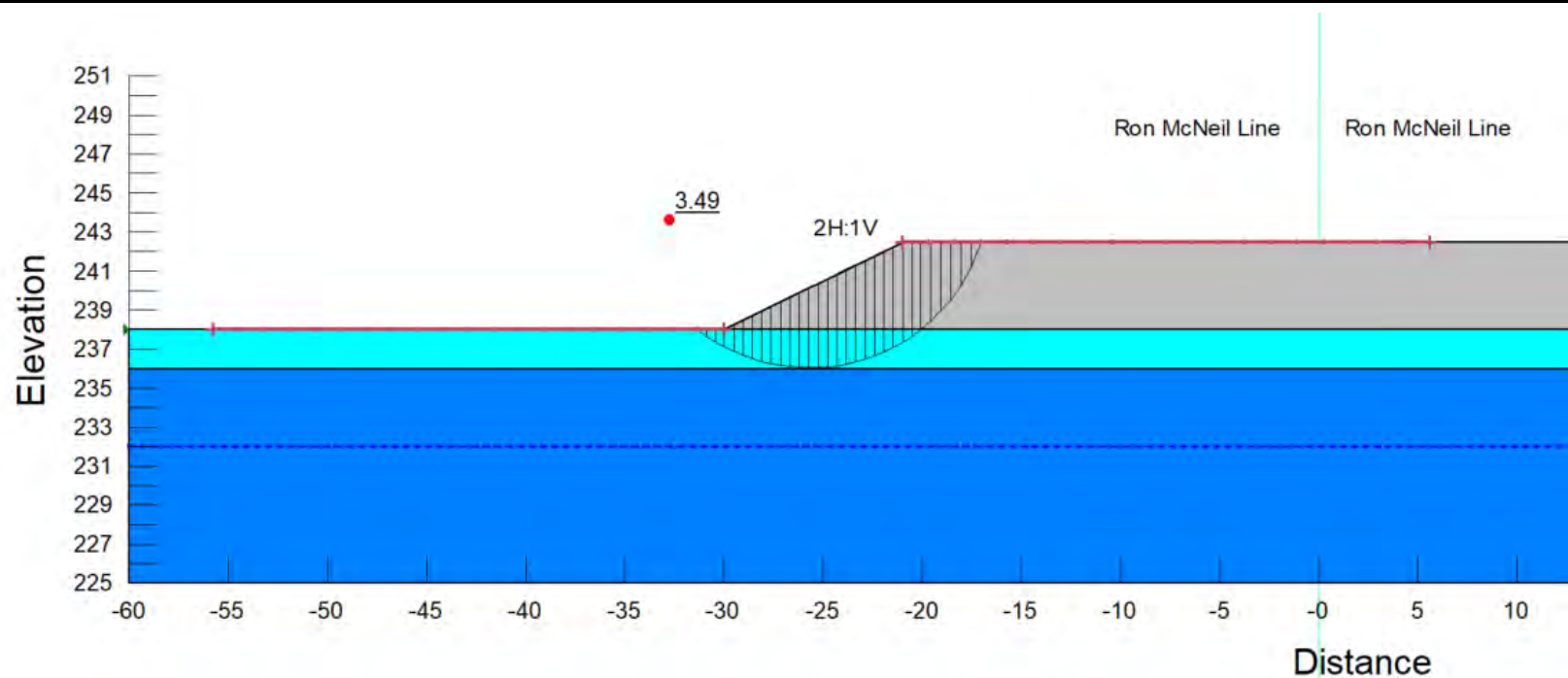
Color	Name	Unit Weight (kN/m³)	Cohesion (kPa)
■	Firm CLAYEY SILT (CL) TILL (Undrained)	21	60
■	New Cohesive Embankment Fill (Undrained)	20.5	50
■	Stiff to hard CLAYEY SILT (CL) TILL (Undrained)	21.5	180



Slope Stability Analysis (Static)
Undrained Conditions - Cohesive Embankment Fill
Talbotville Bypass - Ron McNeil Line Underpass

Figure E13

Project No. 165001308
GWP No. 3042-22-00



Color	Name	Unit Weight (kN/m³)	Cohesion (kPa)
■	Firm CLAYEY SILT (CL) TILL (Undrained)	21	60
■	New Cohesive Embankment Fill (Undrained)	20.5	50
■	Stiff to hard CLAYEY SILT (CL) TILL (Undrained)	21.5	180

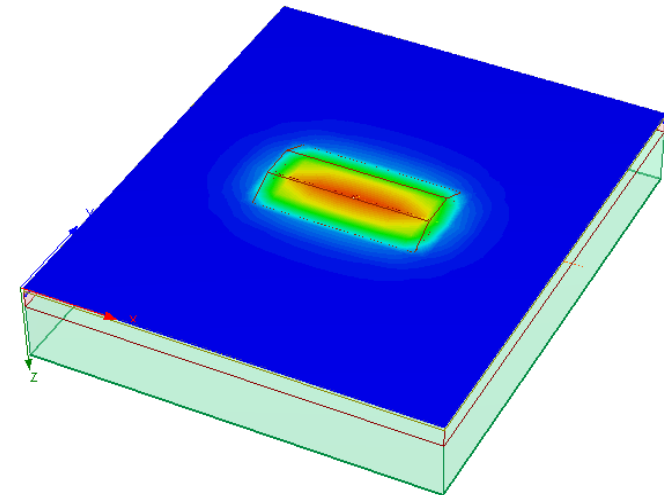
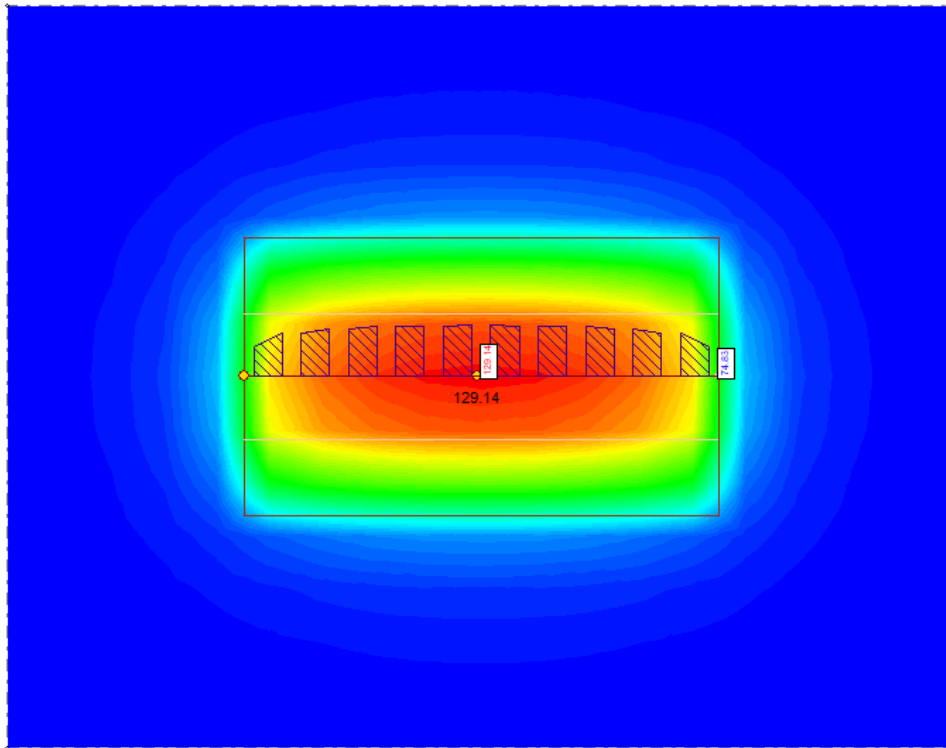





Slope Stability Analysis (Pseudo-static)
 Undrained Conditions - Cohesive Embankment Fill
 Talbotville Bypass - Ron McNeil Line Underpass

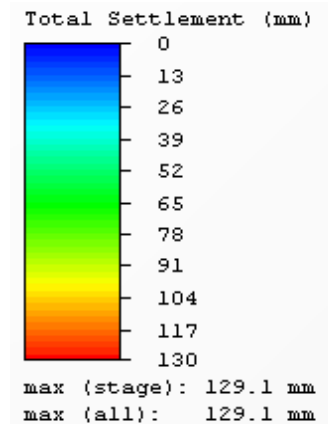
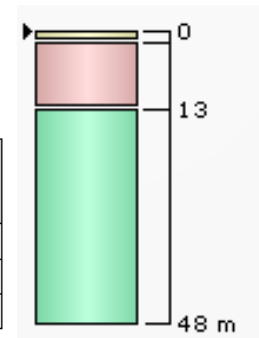
Figure E14

Project No. 165001308

GWP No. 3042-22-00



Material Name	Color	Unit Weight (kN/m ³)	Es (kPa)	Cc	Cr	Pc (kPa)	e0	Cv (cm ² /s)
Firm CLAYEY SILT (CL) TILL		20	-	0.15	0.02	300	0.5	0.003
Stiff to hard CLAYEY SILT (CL) TILL		21	-	0.12	0.015	400	0.48	0.003
Very stiff to hard CLAYEY SILT (CL) TILL		20	-	0.15	0.017	600	0.55	0.003



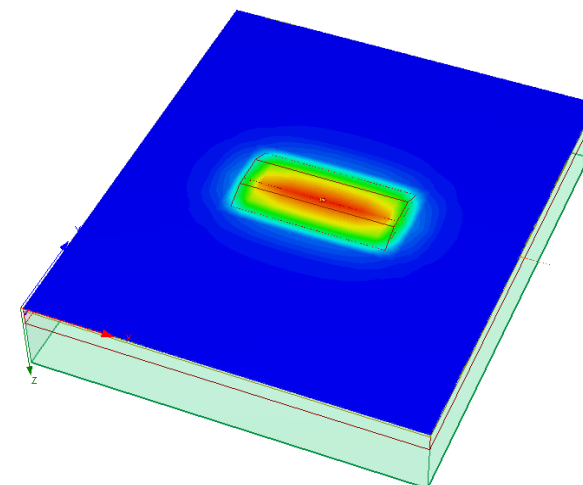
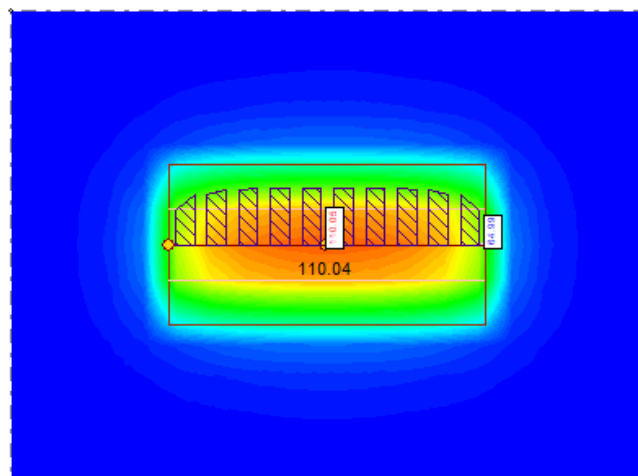
Settlement Analysis




North Approach Embankment
Talbotville Bypass - Ron McNeil Line Underpass

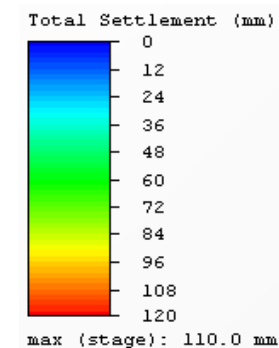
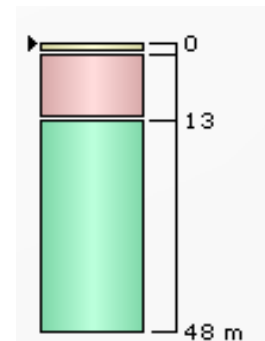
Figure E15

Project No. 165001308

GWP No. 3042-22-00



Material Name	Color	Unit Weight (kN/m ³)	Es (kPa)	Cc	Cr	Pc (kPa)	e0	Cv (cm ² /s)
Firm CLAYEY SILT (CL) TILL		20	-	0.15	0.02	300	0.5	0.003
Stiff to hard CLAYEY SILT (CL) TILL		21	-	0.12	0.015	400	0.48	0.003
Very stiff to hard CLAYEY SILT (CL) TILL		20	-	0.15	0.017	600	0.55	0.003



Settlement Analysis

South Approach Embankment
Talbotville Bypass - Ron McNeil Line Underpass

Figure E16

Project No. 165001308
GWP No. 3042-22-00

**FOUNDATION INVESTIGATION AND DESIGN REPORT – RON MCNEIL LINE INTERCHANGE
UNDERPASS – HIGHWAY 4 WIDENING FROM CLINTON LINE TO NEW TALBOTVILLE BYPASS
AND NEW TALBOTVILLE BYPASS FROM HIGHWAY 4 TO HIGHWAY 3 AT RON MCNEIL LINE**

April 2025

APPENDIX F

F.1 GEOPHYSICS REPORT

F.2 2020 NATIONAL BUILDING CODE OF CANADA SEISMIC HAZARD CALCULATION SHEET



PRESENTATION OF SITE INVESTIGATION RESULTS

HWY 3 St Thomas CPT

Prepared for:

Stantec Consulting Ltd.

ConeTec Job No: 24-05-27609.02

Project Start Date: 14-May-2024

Project End Date: 16-May-2024

Report Date: 10-June-2024



Prepared by:

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Introduction

The enclosed report presents the results of the geophysical site investigation program conducted by ConeTec Investigations Ltd. for RAM Geotechnical Engineering Ltd. at the HWY 3 St Thomas CPT project near St. Thomas, Ontario. The program consisted of seven one-dimensional (1D) Multichannel Analysis of Surface Waves (MASW) tests to provide shear wave velocity (V_s) soundings and calculate a time weighted average V_s of the upper 30 meters (V_{s30}) below grade. This report is in addition to 24-05-27609 which includes all other geotechnical testing completed by ConeTec Investigations Ltd. at this project. 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	Stantec Consulting Ltd.
Project	HWY 3 St Thomas CPT
ConeTec project number	24-05-27609.02

Coordinates		
Test Type	Collection Method	Coordinate Reference System
MASW	Handheld GPS	NAD83, UTM Zone 17 North

MASW Equipment Used for this Project				
Seismograph(s)	Geophones	Coupling Mechanism	Trigger Style	Seismic Sources
1x Geometrics Geode 24	Up to 24 x 4.5 Hz vertical	PVC pucks or spikes	Piezoelectric	Sledgehammer and plate, Passive sources

MASW Data and Results

The data quality for this project was fair. This most significant source of noise was caused by traffic passing on Highway 3. The noise was mitigated by timing shots to avoid passing vehicles and by taking multiple stacks to help improve data signal to noise ratio. In addition, passive seismic readings were collected to take advantage of low frequency seismic signals to increase the depth of investigation. Coherent surface wave energy in the 4 – 35 Hz band allowed the determination of 1D V_s models to over 30 meters deep on most readings. The shear wave velocity results and V_{s30} calculation tables are included in the appendices and digital release of this report. Examples of the raw time domain traces and overtone images are also included in the appendices of this report.

Closure

Thank you for the opportunity to work on this project. The equipment used and the field procedures followed complied with current accepted practice standards.

ConeTec Investigations Ltd.



Matvei Kootchin, P. Geo.

Limitations

3rd Party Disclaimer

This report titled “HWY 3 St Thomas CPT”, referred to as the (“Report”), was prepared by ConeTec for Stantec Consulting 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 Stantec Consulting Ltd. to collect MASW readings (“Data”) for the purpose of measuring 1D Vs soundings and to calculate Vs30. The Data is included in this report titled “HWY 3 St Thomas CPT” 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. Other than the Data, the contents of the Report 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.



Multichannel analysis of surface waves (MASW) is a non-intrusive in-situ test that uses the principles of elasticity and surface wave dispersion to determine the variation of shear wave velocity with depth at a site. The observation that surface waves (Rayleigh waves) of different wavelengths propagate at different phase velocities in non-ideal media, is called dispersion. This is a direct result of the fact that surface waves of different wavelengths propagate along the surface to varying depths, and hence, if material stiffness changes with depth (as is the case with most non-ideal materials), then an appropriately selected wavelength band will reflect such changes in the velocity of propagation.

The field methods for surface wave testing are very similar to other surface seismic data collection methods. Surface geophones are placed in a linear array along a survey line at a known separation (typically one metre). A series of recordings (shots) are collected with a known in-line source offset from the array. Each shot gather is represented in the time-offset domain and shows the amplitude of wave propagation through the array (refer to [Figure MASW-1](#)). For detailed frequency analysis, multiple records with different shot offset distances are collected to help better define the broad spectrum frequency-phase velocity response of the medium. Two-dimensional cross sections can be collected by moving the geophone array a small distance (typically two meters) along the line and repeating the shots at set offsets.

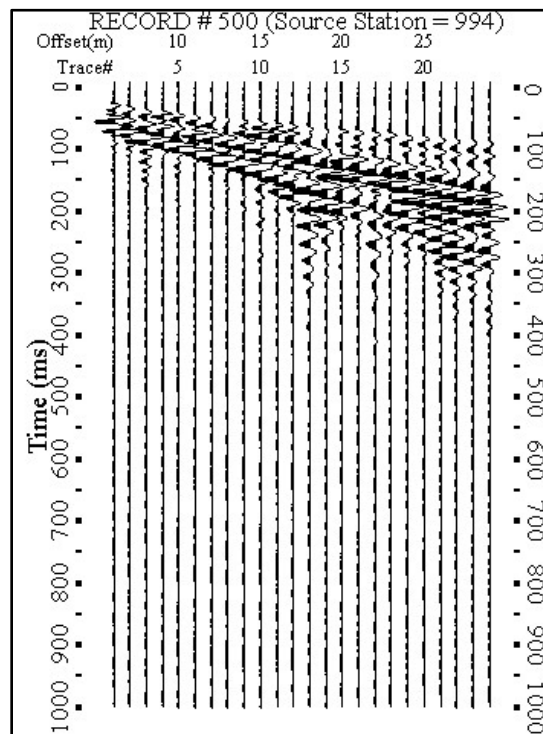


Figure MASW-1. Typical MASW time domain record (shot gather)

Given that surface wave velocity is closely related to the shear wave velocity and the wavelength related to depth, the surface wave results can be used to develop a profile of shear wave velocity versus depth through a process referred to as inversion. The program used to perform the inversion is SurfSeis 6.6, developed by the Kansas Geological Survey. In SurfSeis, the raw time domain traces are transformed to the frequency domain to create what is referred to as an overtone image as shown in [Figure MASW-2](#). The overtone image displays the amplitude of the primary surface wave mode and any potential higher modes. A dispersion curve is fitted to the overtone image, and the inversion process is then used to

determine the most appropriate shear wave velocity profile. The parameters used for the inversion of the dispersion data are provided in the data release folder in an Excel table.

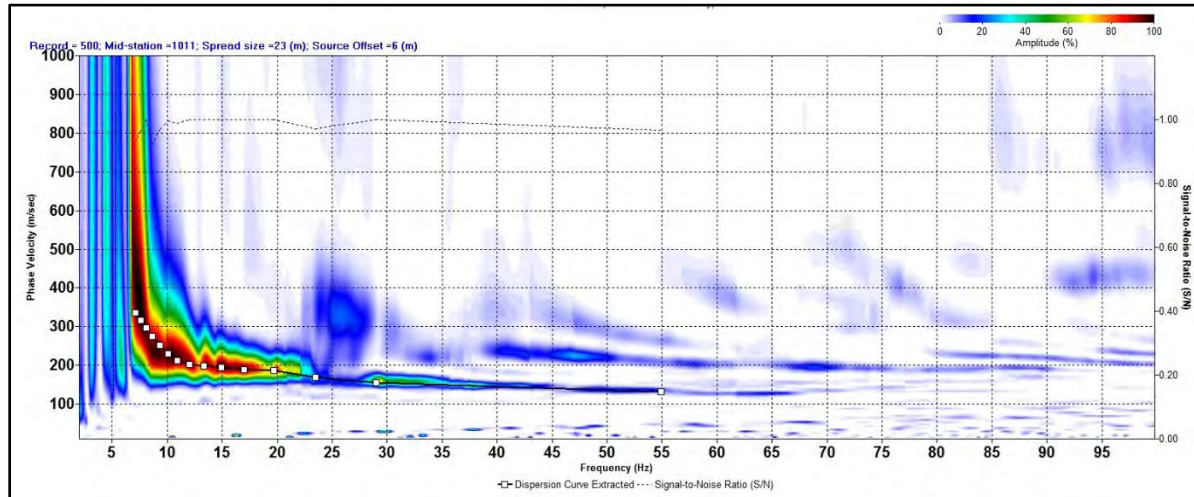


Figure MASW-2. Overtone image and a picked dispersion curve

For each test location, a 1D shear wave velocity profile comprising of a number of velocity layers of variable thickness (refer to [Figure MASW-3](#)) is provided. For 2D testing a series of 1D tests are combined to produce a shear wave velocity cross section.

The depth of investigation is related to the ground conditions and the amount of energy delivered by the surface wave source. The surface wave method uses Rayleigh waves that travel horizontally along the ground surface to a depth of about one wavelength. The actual depth of sampling of the ground is considered to be one-half to one-third of the Rayleigh (surface) wave wavelength. The wavelengths measured by the equipment will be a function of the frequency of the source and the velocity of the surface waves through the ground. As the depth of investigation increases, there will be less certainty in terms of layer boundaries and velocity values.

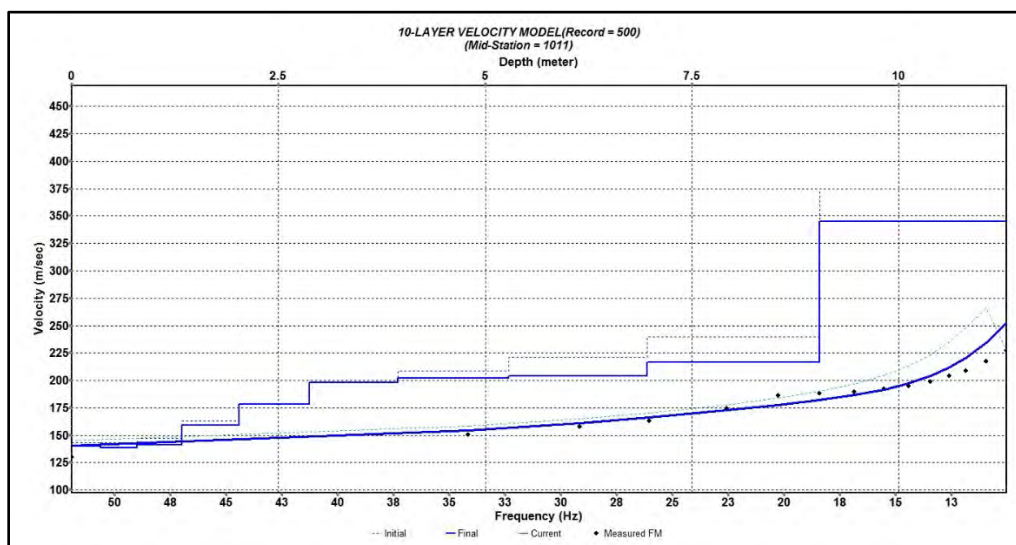


Figure MASW-3. 1D inversion result with fitted dispersion curve

The equipment, field procedures, and analysis software used by ConeTec all conform to the currently accepted best practices for MASW testing. The results of geophysical testing are always interpretative to a certain extent and should be confirmed by drilling or other intrusive testing.

References

Miller, R.D., Xia, J., Park, C.B., and Ivanov, J.M., 1999, Multichannel analysis of surface waves to map bedrock, Kansas Geological Survey, The Leading Edge, December, p. 1392-1396.

Park, C.B., Miller, R.D., and Xia, J., 1998b, Ground roll as a tool to image near-surface anomaly: 68th Ann. Internat. Mtg. Soc. Expl. Geophys., Expanded Abstracts, p. 874-877.

Park, C.B., Miller, R.D., and Xia, J., 1999, Multichannel analysis of surface waves: Geophysics, v. 64, n. 3, pp. 800-808.

Park, C.B., Miller, R.D., Xia, J., and Ivanov, J., 2007, Multichannel analysis of surface waves (MASW)-active and passive methods: The Leading Edge, January.

SurfSeis website: <http://www.kgs.ku.edu/software/surfseis/index.html>

Xia, J., R.D. Miller, and C.B. Park, 2000a, Advantages of calculating shear-wave velocity from surface waves with higher modes: [Exp. Abs.]: Soc. Expl. Geophys., p. 1295-1298.

Xia, J., Miller, R.D., Park, C.B., and Ivanov, J., 2000b, Construction of 2-D vertical shear-wave velocity field by the Multichannel Analysis of Surface Wave technique, Proceedings of the Symposium on the Application of Geophysics to Engineering and Environmental Problems (SAGEEP 2000), Washington D.C, February 20-24, p. 1197-1206.

The following appendices listed below are included in the report:

- MASW Summary and Map
- 1D MASW Results
- Vs30 Calculation Tables
- MASW Time Domain Traces and Overtone Images

MASW Summary and Map



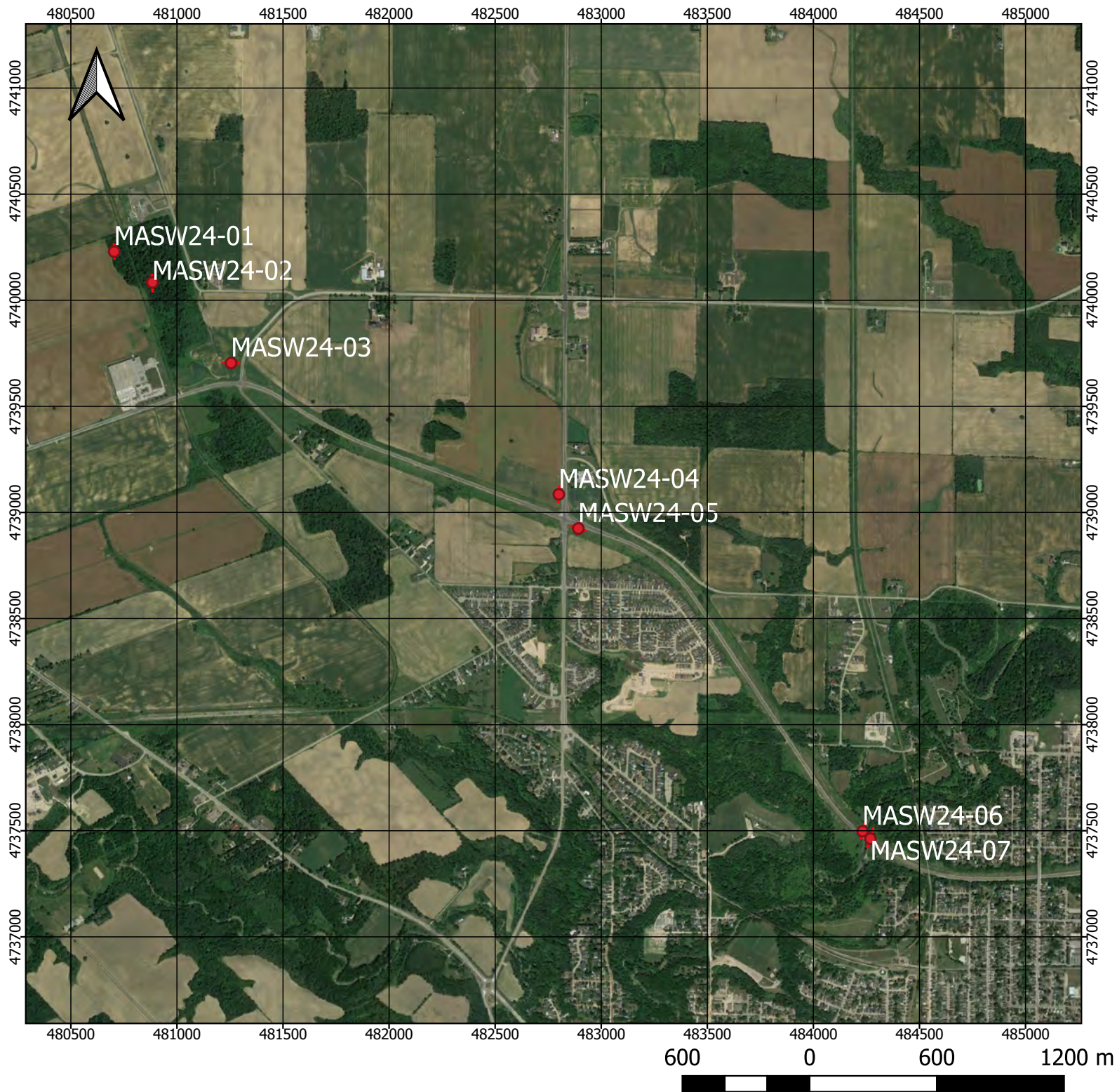
Job No: 24-05-27609
Client: Stantec Consulting Ltd.
Project: HWY 3 St Thomas CPT
Start Date: 14-May-2024
End Date: 16-May-2024

1D MASW TEST SUMMARY

Sounding ID	Date	Geophone Spacing (m)	Array Length (m)	Center Point Northing ¹ (m)	Center Point Easting ¹ (m)	Refer to Notation Number
MASW24-01	14-May-2024	3	69	4740229	480704	
MASW24-02	14-May-2024	3	69	4740084	480884	
MASW24-03	15-May-2024	3	69	4739704	481255	
MASW24-04	15-May-2024	3	69	4739085	482800	
MASW24-05	15-May-2024	3	69	4738924	482892	
MASW24-06	15-May-2024	3	69	4737497	484232	
MASW24-07	16-May-2024	3	69	4737462	484269	

1. Coordinates are presented in NAD83 (CSRS) UTM Zone 17 North.

HWY 3 St Thomas CPT - MASW Survey



Legend

- 1D MASW Location
- MASW Array

ConeTec Job Number: 24-05-27609
Survey Date(s): 14-May-2024 - 16-May-2024
Coordinate Reference System: NAD83 UTM Zone 17 North
Map Scale: 1:25000
Units: meters
Imagery Source: Google Earth

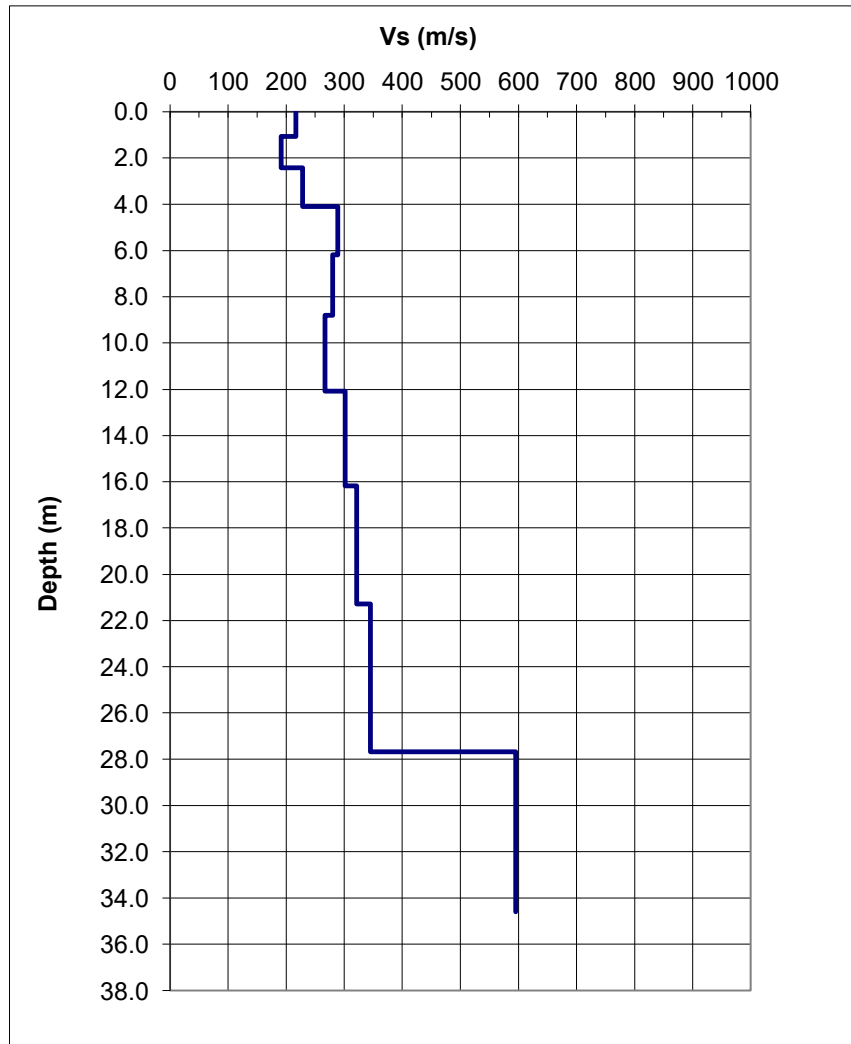
1D MASW Results



Job No: 24-05-27609
Client: Stantec Consulting Ltd.
Project: HWY 3 St Thomas
Sounding ID: MASW24-01
Date: 14-May-2024

1D MASW SHEAR WAVE VELOCITY TEST RESULTS

Layer	Layer Thickness (m)	Depth of Bottom of Layer (m)	Vs (m/s)
1	1.07	1.07	217
2	1.34	2.41	191
3	1.68	4.09	228
4	2.10	6.19	289
5	2.62	8.80	280
6	3.27	12.08	267
7	4.09	16.17	302
8	5.12	21.29	322
9	6.39	27.68	345
10	6.92	34.60	595

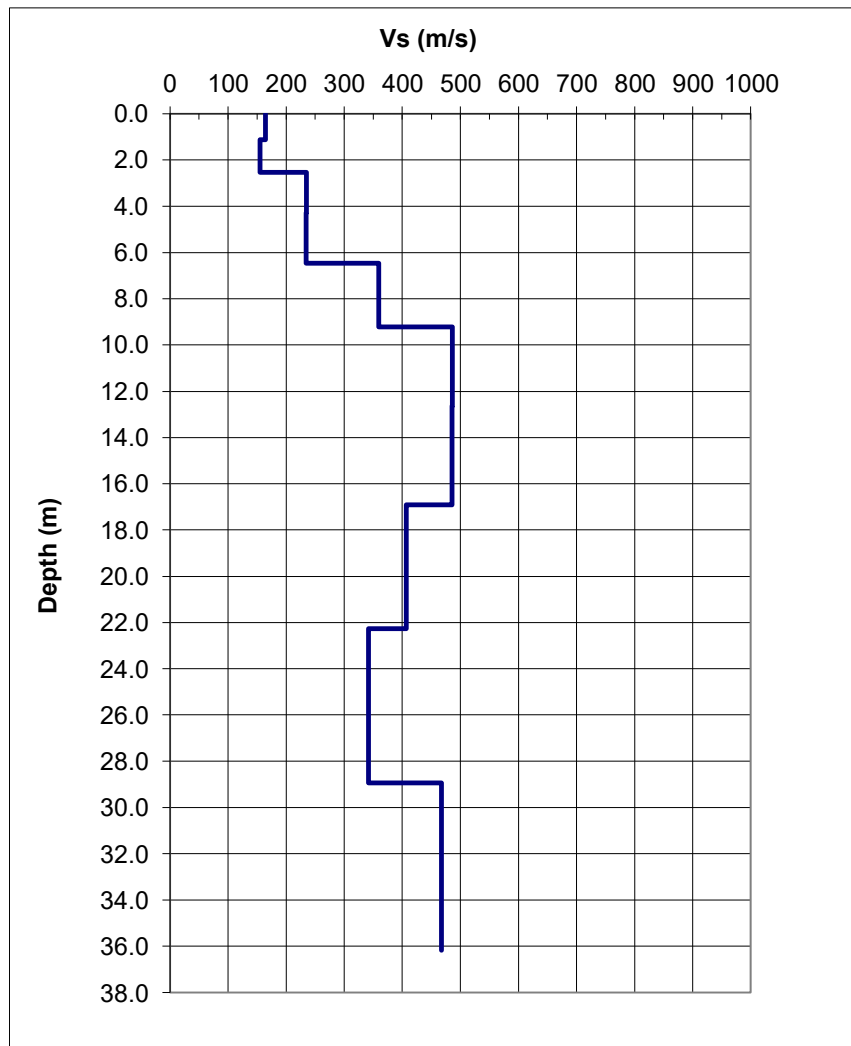




Job No: 24-05-27609
Client: Stantec Consulting Ltd.
Project: HWY 3 St Thomas
Sounding ID: MASW24-02
Date: 14-May-2024

1D MASW SHEAR WAVE VELOCITY TEST RESULTS

Layer	Layer Thickness (m)	Depth of Bottom of Layer (m)	Vs (m/s)
1	1.12	1.12	164
2	1.40	2.52	155
3	1.75	4.28	235
4	2.19	6.47	234
5	2.74	9.21	360
6	3.42	12.63	486
7	4.28	16.91	485
8	5.35	22.26	407
9	6.69	28.94	342
10	7.24	36.18	467

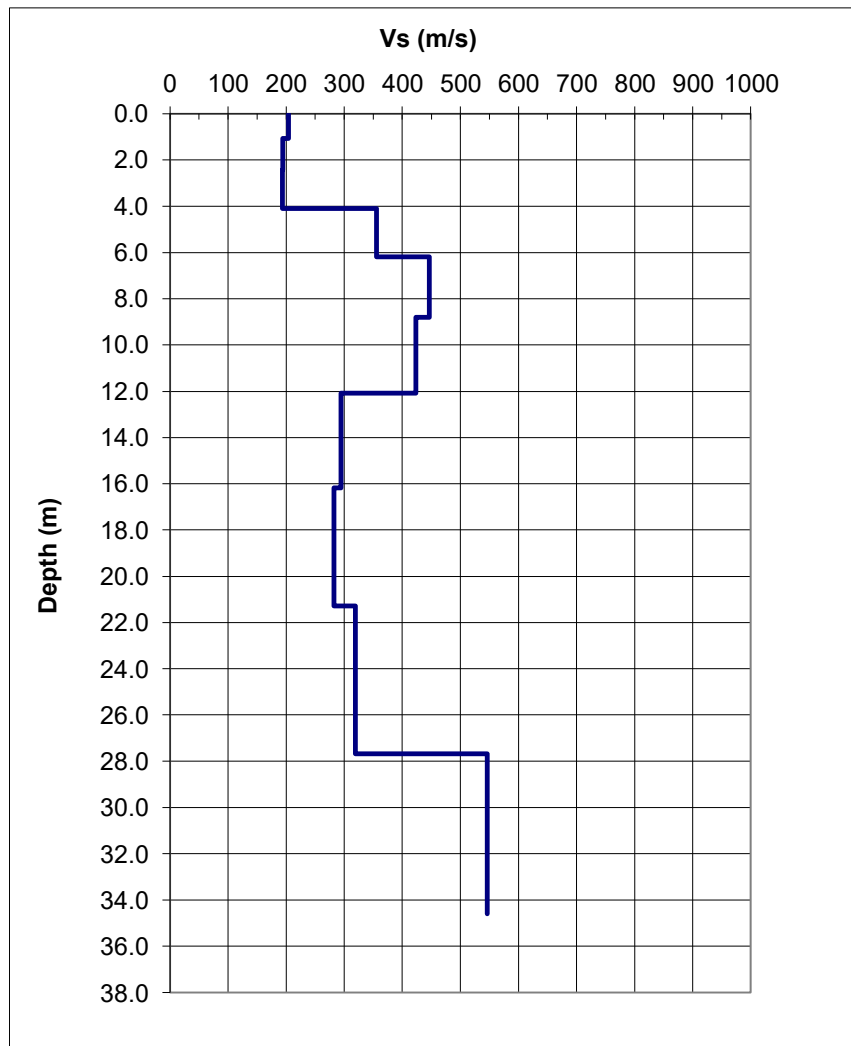




Job No: 24-05-27609
Client: Stantec Consulting Ltd.
Project: HWY 3 St Thomas
Sounding ID: MASW24-03
Date: 15-May-2024

1D MASW SHEAR WAVE VELOCITY TEST RESULTS

Layer	Layer Thickness (m)	Depth of Bottom of Layer (m)	Vs (m/s)
1	1.07	1.07	204
2	1.34	2.41	194
3	1.68	4.09	194
4	2.10	6.19	356
5	2.62	8.80	446
6	3.27	12.08	423
7	4.09	16.17	294
8	5.12	21.29	283
9	6.39	27.68	319
10	6.92	34.60	546

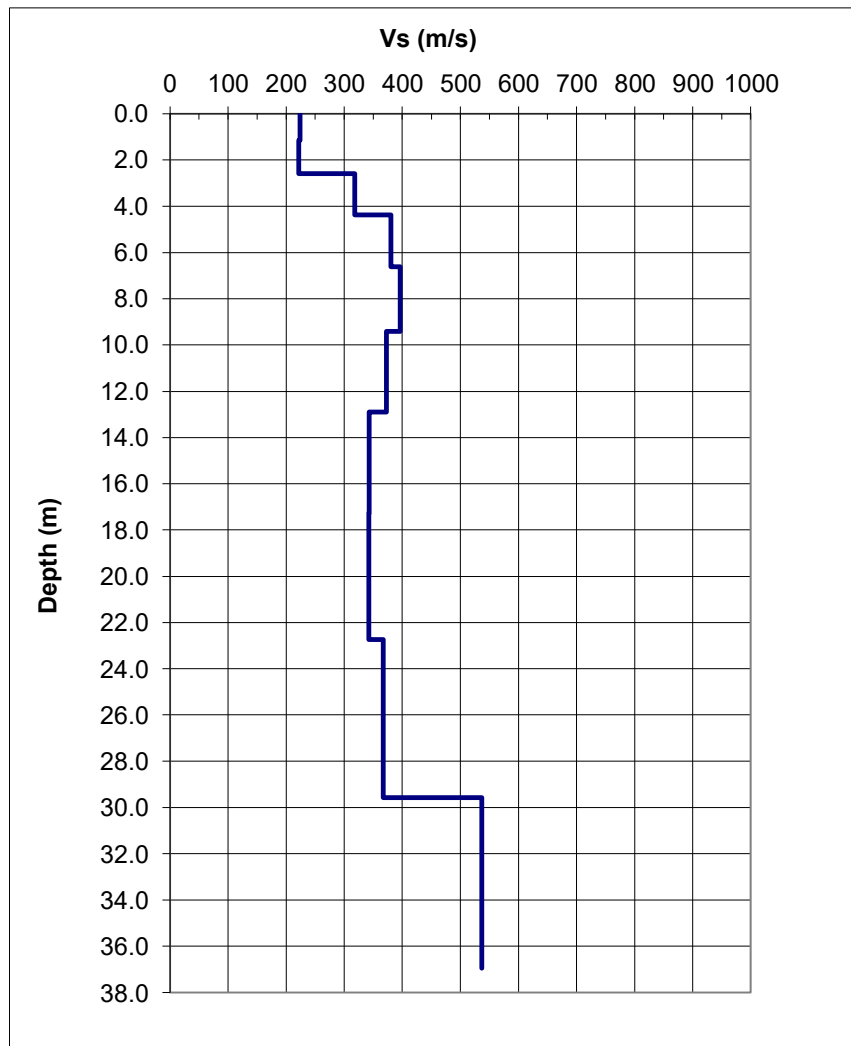




Job No: 24-05-27609
Client: Stantec Consulting Ltd.
Project: HWY 3 St Thomas
Sounding ID: MASW24-04
Date: 15-May-2024

1D MASW SHEAR WAVE VELOCITY TEST RESULTS

Layer	Layer Thickness (m)	Depth of Bottom of Layer (m)	Vs (m/s)
1	1.15	1.15	224
2	1.43	2.58	222
3	1.79	4.37	318
4	2.24	6.61	380
5	2.80	9.40	396
6	3.50	12.90	373
7	4.37	17.27	343
8	5.46	22.74	342
9	6.83	29.57	367
10	7.39	36.96	537

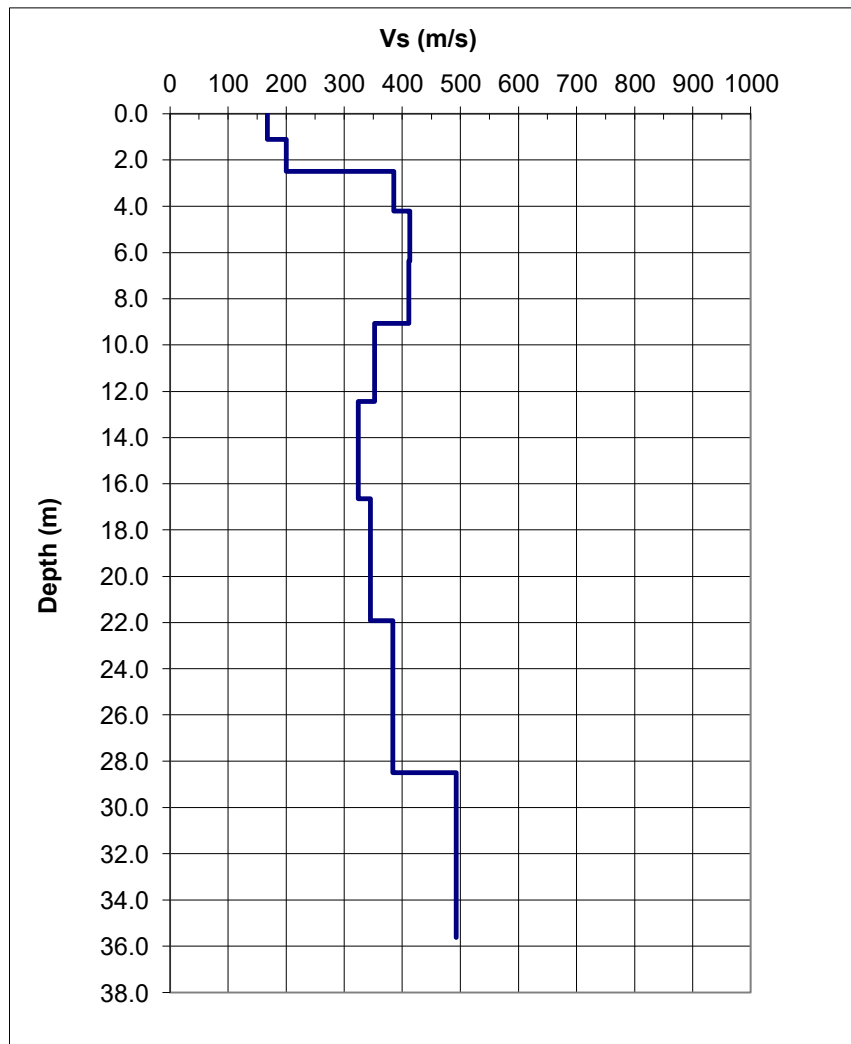




Job No: 24-05-27609
Client: Stantec Consulting Ltd.
Project: HWY 3 St Thomas
Sounding ID: MASW24-05
Date: 15-May-2024

1D MASW SHEAR WAVE VELOCITY TEST RESULTS

Layer	Layer Thickness (m)	Depth of Bottom of Layer (m)	Vs (m/s)
1	1.10	1.10	168
2	1.38	2.49	201
3	1.73	4.21	385
4	2.16	6.37	413
5	2.70	9.07	411
6	3.37	12.44	352
7	4.21	16.65	324
8	5.27	21.92	345
9	6.58	28.50	383
10	7.13	35.62	493

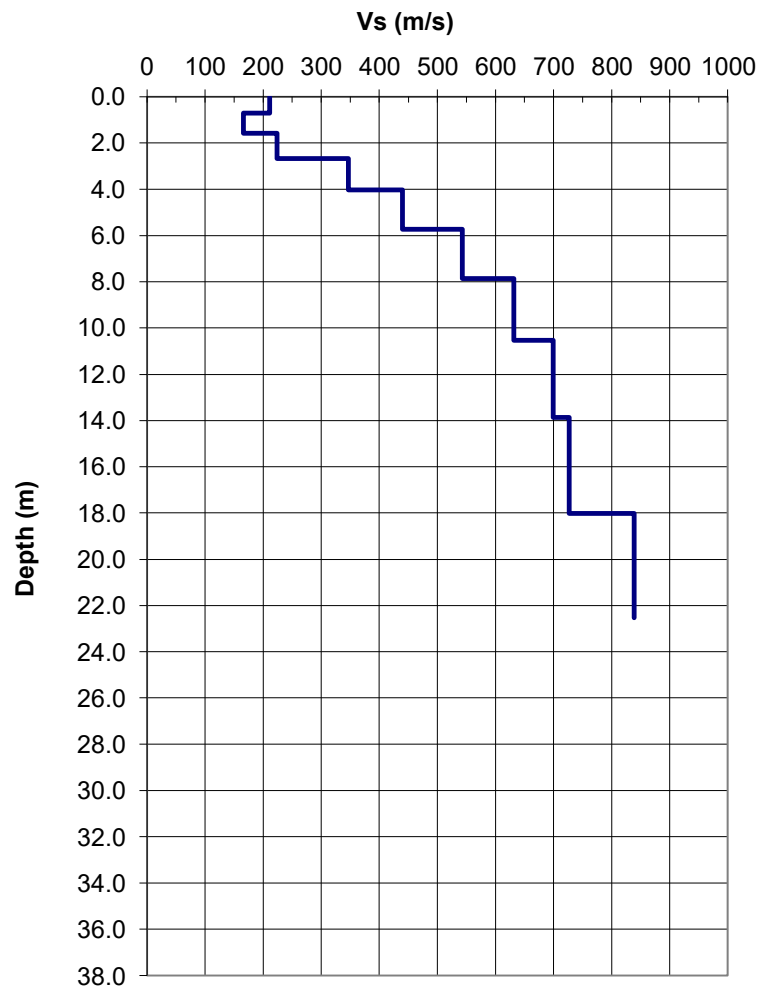




Job No: 24-05-27609
Client: Stantec Consulting Ltd.
Project: HWY 3 St Thomas
Sounding ID: MASW24-06
Date: 15-May-2024

1D MASW SHEAR WAVE VELOCITY TEST RESULTS

Layer	Layer Thickness (m)	Depth of Bottom of Layer (m)	Vs (m/s)
1	0.70	0.70	211
2	0.87	1.57	166
3	1.09	2.66	224
4	1.36	4.03	346
5	1.71	5.73	440
6	2.13	7.86	543
7	2.67	10.53	631
8	3.33	13.86	699
9	4.16	18.02	727
10	4.51	22.53	839

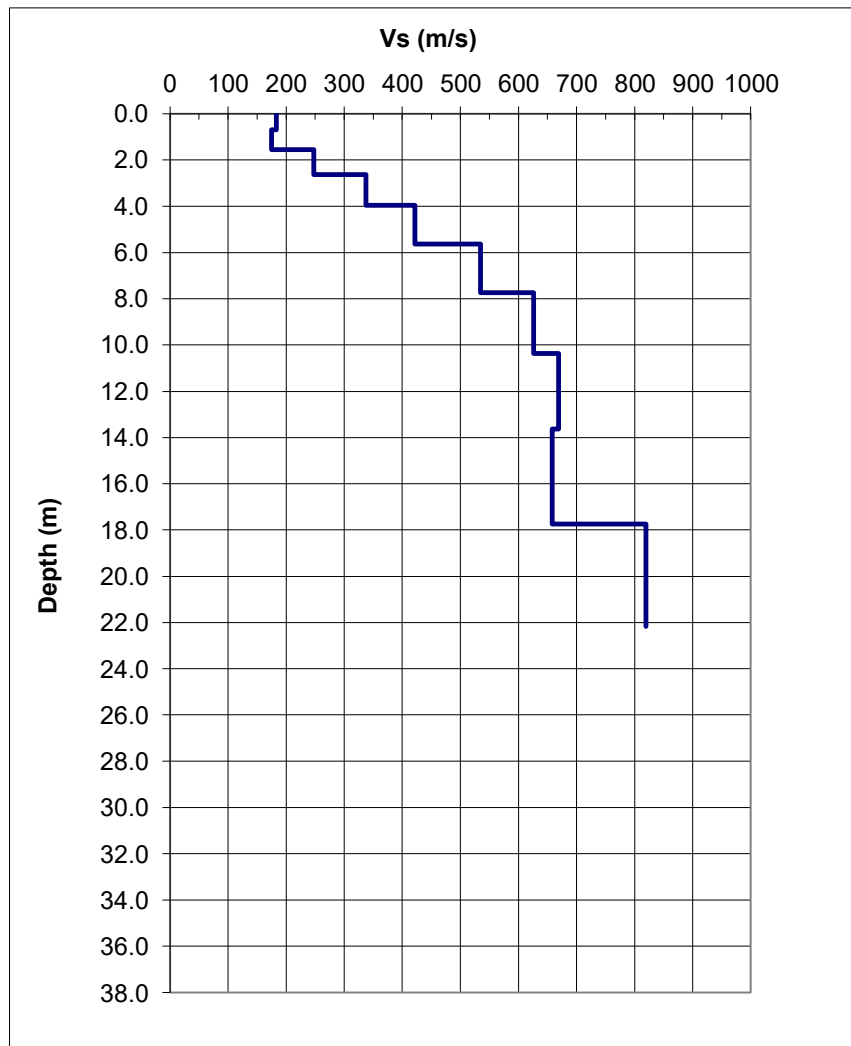




Job No: 24-05-27609
Client: Stantec Consulting Ltd.
Project: HWY 3 St Thomas
Sounding ID: MASW24-07
Date: 16-May-2024

1D MASW SHEAR WAVE VELOCITY TEST RESULTS

Layer	Layer Thickness (m)	Depth of Bottom of Layer (m)	Vs (m/s)
1	0.69	0.69	183
2	0.86	1.55	175
3	1.07	2.62	248
4	1.34	3.96	338
5	1.68	5.64	422
6	2.10	7.74	534
7	2.62	10.36	626
8	3.28	13.64	669
9	4.10	17.74	658
10	4.43	22.17	820



VS30 Calculation Tables



Job No: 24-05-27609
Client: Stantec Consulting Ltd.
Project: HWY 3 St Thomas CPT
Sounding: MASW24-01
Date: 14-May-2024

VS30 CALCULATION				
Layer Number	Layer Thickness (m)	Layer Bottom (m)	Vs (m/s)	Equivalent Vertical Travel Time (s)
1	1.07	1.07	217	0.00495
2	1.34	2.41	191	0.00701
3	1.68	4.09	228	0.00734
4	2.10	6.19	289	0.00724
5	2.62	8.80	280	0.00936
6	3.27	12.08	267	0.01225
7	4.09	16.17	302	0.01357
8	5.12	21.29	322	0.01590
9	6.39	27.68	345	0.01853
10	2.32	30.00	595	0.00390
Total Vertical Travel Time for 30m (s)				0.10005
Average Travel Time Weighted Shear Wave Velocity (m/s)				300

Notes: Yellow Highlighted Cells Indicate Projected Shear Wave Velocity



Job No: 24-05-27609
Client: Stantec Consulting Ltd.
Project: HWY 3 St Thomas CPT
Sounding: MASW24-02
Date: 14-May-2024

VS30 CALCULATION				
Layer Number	Layer Thickness (m)	Layer Bottom (m)	Vs (m/s)	Equivalent Vertical Travel Time (s)
1	1.12	1.12	164	0.00683
2	1.40	2.52	155	0.00903
3	1.75	4.28	235	0.00746
4	2.19	6.47	234	0.00935
5	2.74	9.21	360	0.00761
6	3.42	12.63	486	0.00704
7	4.28	16.91	485	0.00882
8	5.35	22.26	407	0.01314
9	6.69	28.94	342	0.01956
10	1.06	30.00	467	0.00227
Total Vertical Travel Time for 30m (s)				0.09111
Average Travel Time Weighted Shear Wave Velocity (m/s)				329

Notes: Yellow Highlighted Cells Indicate Projected Shear Wave Velocity



Job No: 24-05-27609
Client: Stantec Consulting Ltd.
Project: HWY 3 St Thomas CPT
Sounding: MASW24-03
Date: 15-May-2024

VS30 CALCULATION				
Layer Number	Layer Thickness (m)	Layer Bottom (m)	Vs (m/s)	Equivalent Vertical Travel Time (s)
1	1.07	1.07	204	0.00526
2	1.34	2.41	194	0.00690
3	1.68	4.09	194	0.00865
4	2.10	6.19	356	0.00589
5	2.62	8.80	446	0.00587
6	3.27	12.08	423	0.00774
7	4.09	16.17	294	0.01390
8	5.12	21.29	283	0.01809
9	6.39	27.68	319	0.02004
10	2.32	30.00	546	0.00425
Total Vertical Travel Time for 30m (s)				0.09661
Average Travel Time Weighted Shear Wave Velocity (m/s)				311

Notes: Yellow Highlighted Cells Indicate Projected Shear Wave Velocity



Job No: 24-05-27609
Client: Stantec Consulting Ltd.
Project: HWY 3 St Thomas CPT
Sounding: MASW24-04
Date: 15-May-2024

VS30 CALCULATION				
Layer Number	Layer Thickness (m)	Layer Bottom (m)	Vs (m/s)	Equivalent Vertical Travel Time (s)
1	1.15	1.15	224	0.00511
2	1.43	2.58	222	0.00646
3	1.79	4.37	318	0.00563
4	2.24	6.61	380	0.00588
5	2.80	9.40	396	0.00706
6	3.50	12.90	373	0.00938
7	4.37	17.27	343	0.01276
8	5.46	22.74	342	0.01597
9	6.83	29.57	367	0.01861
10	0.43	30.00	537	0.00081
Total Vertical Travel Time for 30m (s)				0.08766
Average Travel Time Weighted Shear Wave Velocity (m/s)				342

Notes: Yellow Highlighted Cells Indicate Projected Shear Wave Velocity



Job No: 24-05-27609
Client: Stantec Consulting Ltd.
Project: HWY 3 St Thomas CPT
Sounding: MASW24-05
Date: 15-May-2024

VS30 CALCULATION				
Layer Number	Layer Thickness (m)	Layer Bottom (m)	Vs (m/s)	Equivalent Vertical Travel Time (s)
1	1.10	1.10	168	0.00657
2	1.38	2.49	201	0.00689
3	1.73	4.21	385	0.00448
4	2.16	6.37	413	0.00522
5	2.70	9.07	411	0.00656
6	3.37	12.44	352	0.00957
7	4.21	16.65	324	0.01300
8	5.27	21.92	345	0.01527
9	6.58	28.50	383	0.01717
10	1.50	30.00	493	0.00304
Total Vertical Travel Time for 30m (s)				0.08779
Average Travel Time Weighted Shear Wave Velocity (m/s)				342

Notes: Yellow Highlighted Cells Indicate Projected Shear Wave Velocity



Job No: 24-05-27609
Client: Stantec Consulting Ltd.
Project: HWY 3 St Thomas CPT
Sounding: MASW24-06
Date: 15-May-2024

VS30 CALCULATION				
Layer Number	Layer Thickness (m)	Layer Bottom (m)	Vs (m/s)	Equivalent Vertical Travel Time (s)
1	0.70	0.70	211	0.00331
2	0.87	1.57	166	0.00526
3	1.09	2.66	224	0.00487
4	1.36	4.03	346	0.00394
5	1.71	5.73	440	0.00388
6	2.13	7.86	543	0.00393
7	2.67	10.53	631	0.00422
8	3.33	13.86	699	0.00476
9	4.16	18.02	727	0.00573
10	4.51	22.53	839	0.00537
11	7.47	30.00	839	0.00890
Total Vertical Travel Time for 30m (s)				0.05417
Average Travel Time Weighted Shear Wave Velocity (m/s)				554

Notes: Yellow Highlighted Cells Indicate Projected Shear Wave Velocity

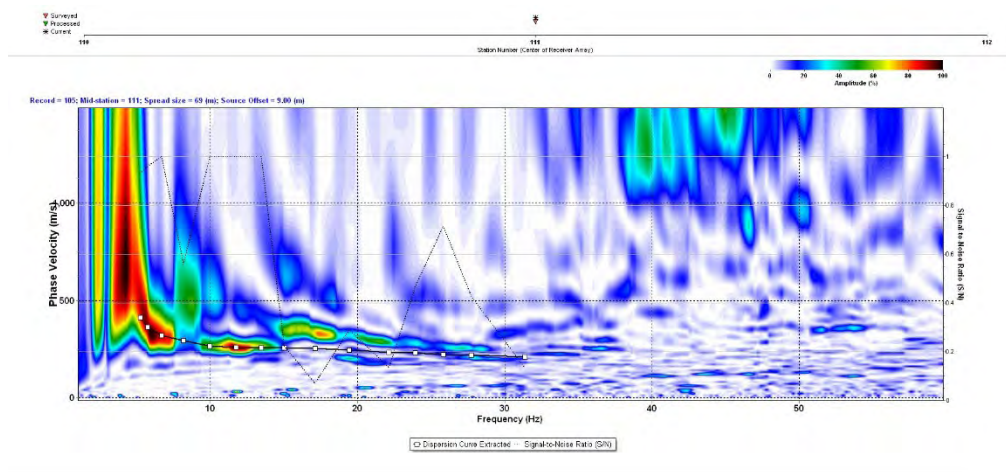
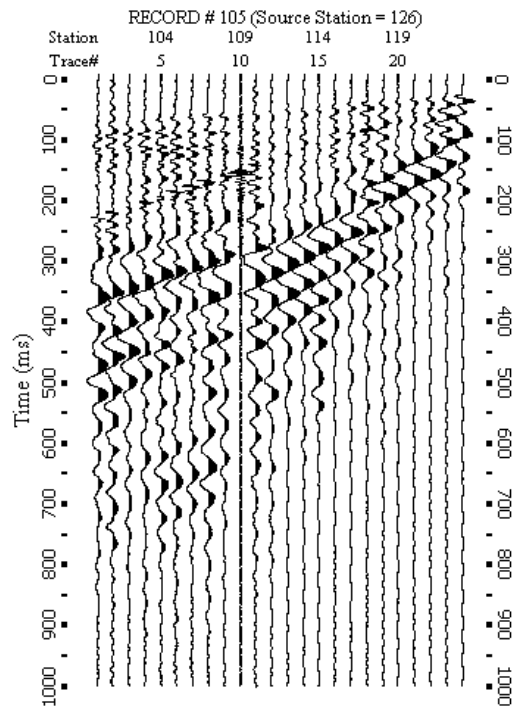


Job No: 24-05-27609
Client: Stantec Consulting Ltd.
Project: HWY 3 St Thomas CPT
Sounding: MASW24-07
Date: 16-May-2024

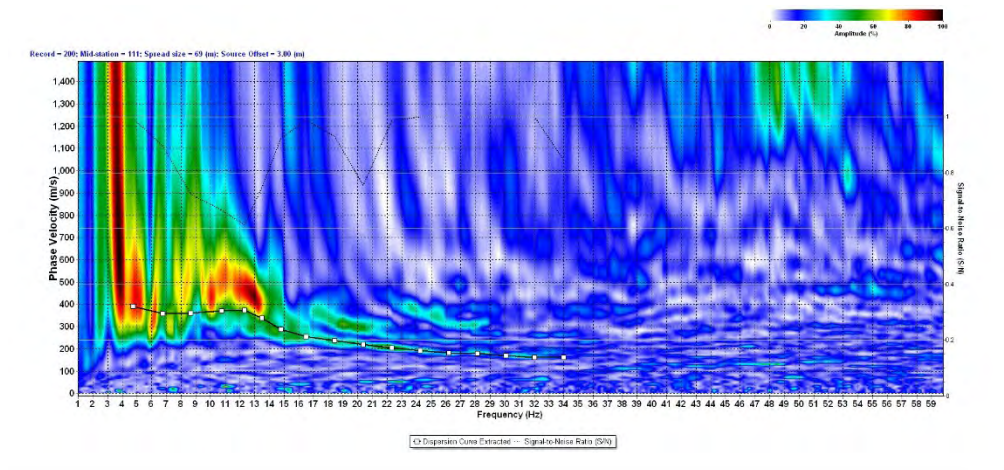
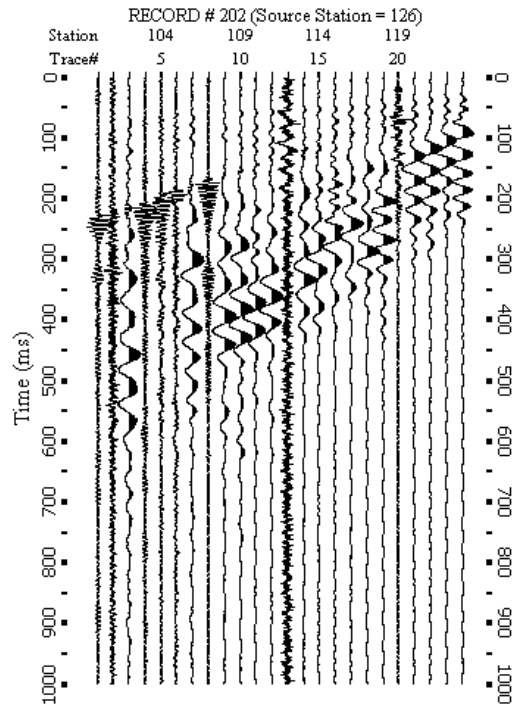
VS30 CALCULATION				
Layer Number	Layer Thickness (m)	Layer Bottom (m)	Vs (m/s)	Equivalent Vertical Travel Time (s)
1	0.69	0.69	183	0.00375
2	0.86	1.55	175	0.00492
3	1.07	2.62	248	0.00433
4	1.34	3.96	338	0.00398
5	1.68	5.64	422	0.00398
6	2.10	7.74	534	0.00393
7	2.62	10.36	626	0.00419
8	3.28	13.64	669	0.00490
9	4.10	17.74	658	0.00623
10	4.43	22.17	820	0.00541
11	7.83	30.00	820	0.00955
Total Vertical Travel Time for 30m (s)				0.05516
Average Travel Time Weighted Shear Wave Velocity (m/s)				544

Notes: Yellow Highlighted Cells Indicate Projected Shear Wave Velocity

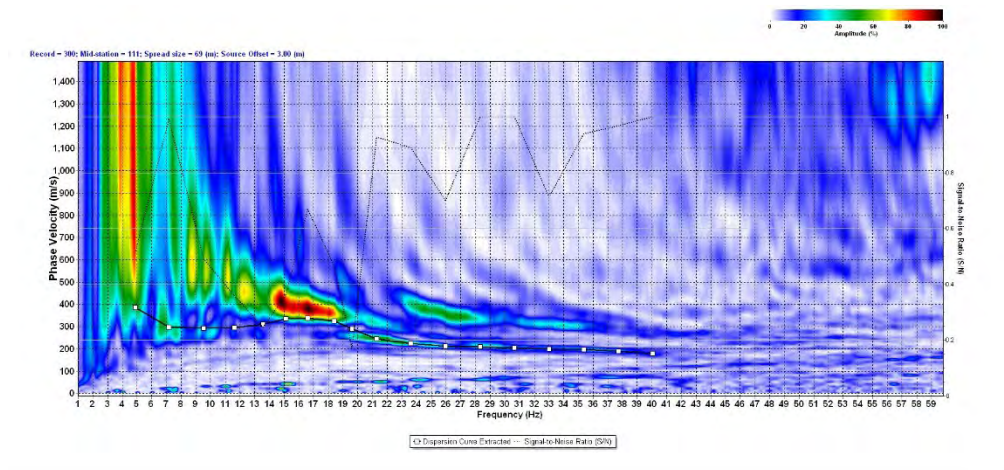
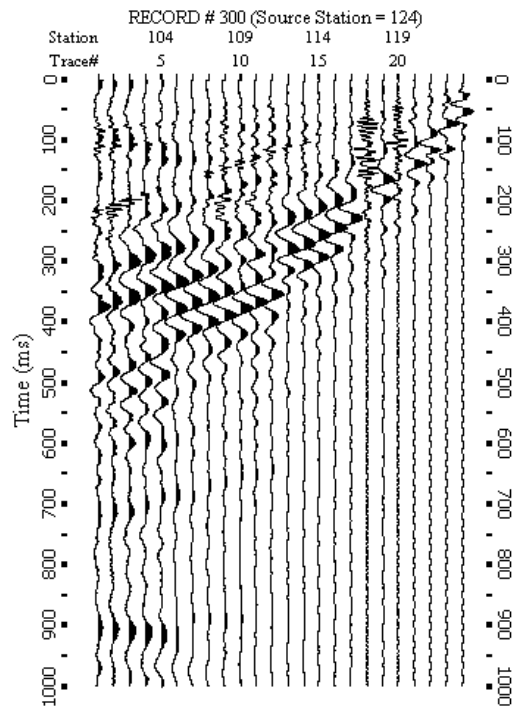
MASW Time Domain Traces and Overtone Images



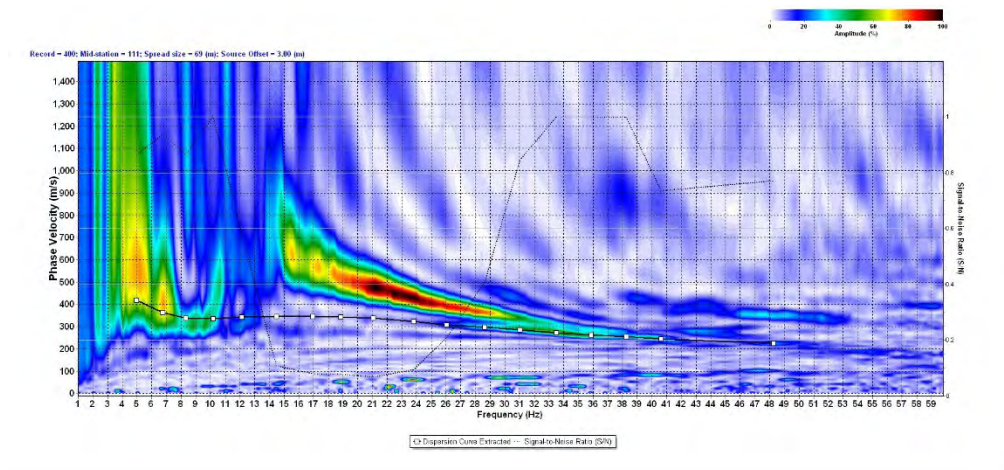
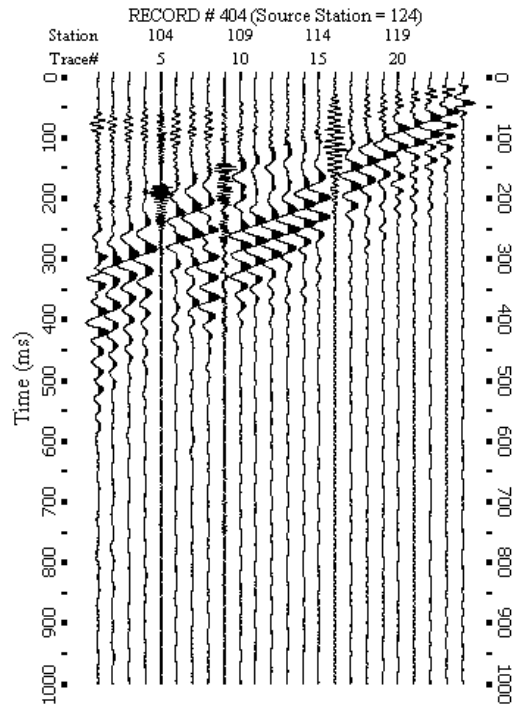
MASW24-01: Example time domain trace for active source (top) with resulting overtone image with picked dispersion curve (bottom).



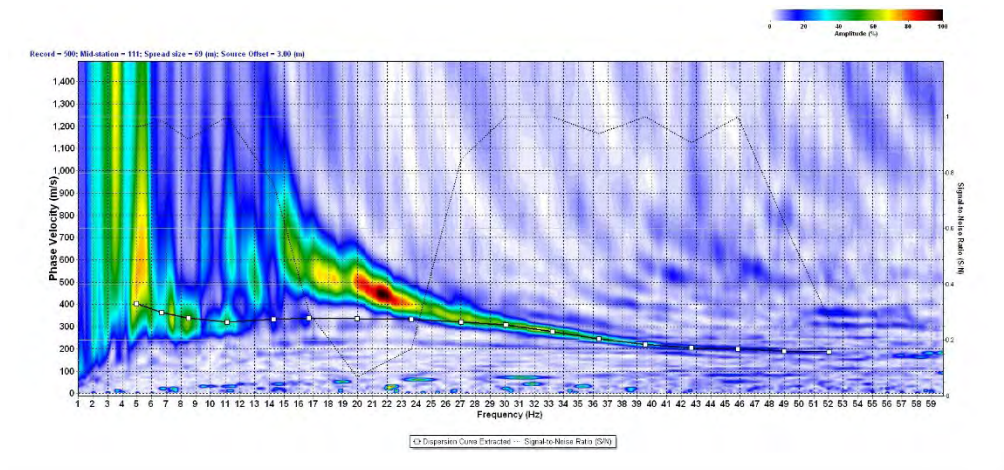
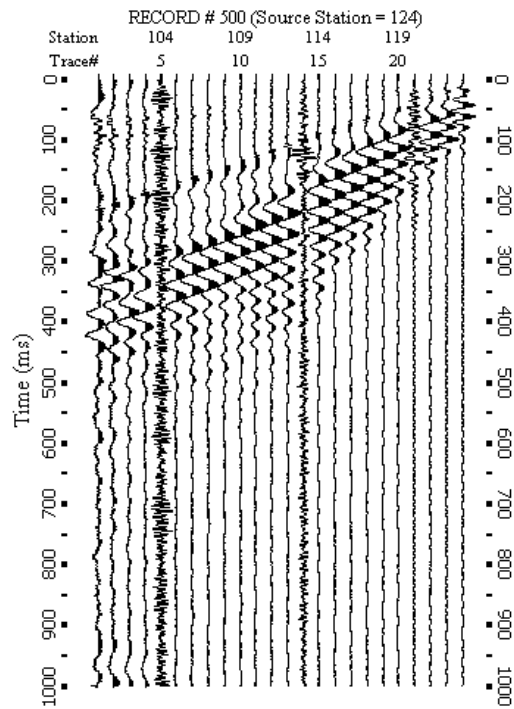
MASW24-02: Example time domain trace for active source (top) with resulting overtone image with picked dispersion curve (bottom).



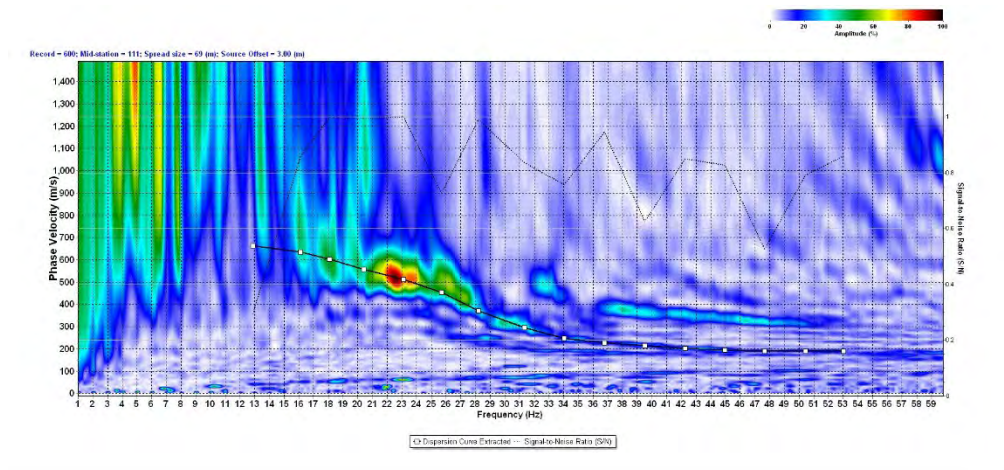
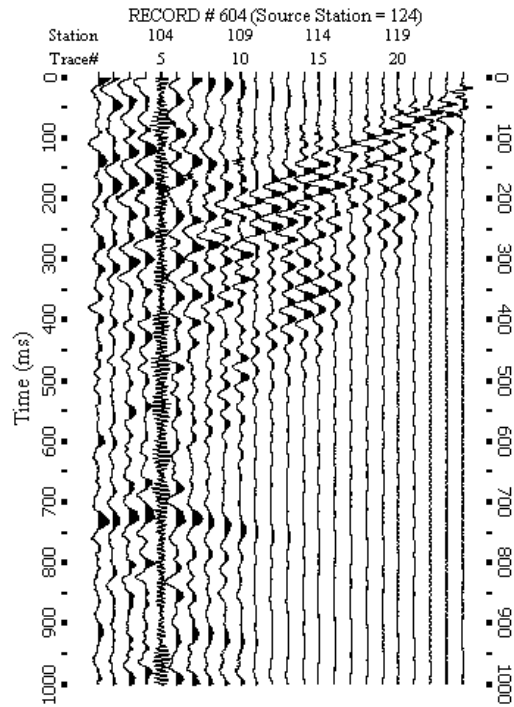
MASW24-03: Example time domain trace for active source (top) with resulting overtone image with picked dispersion curve (bottom).



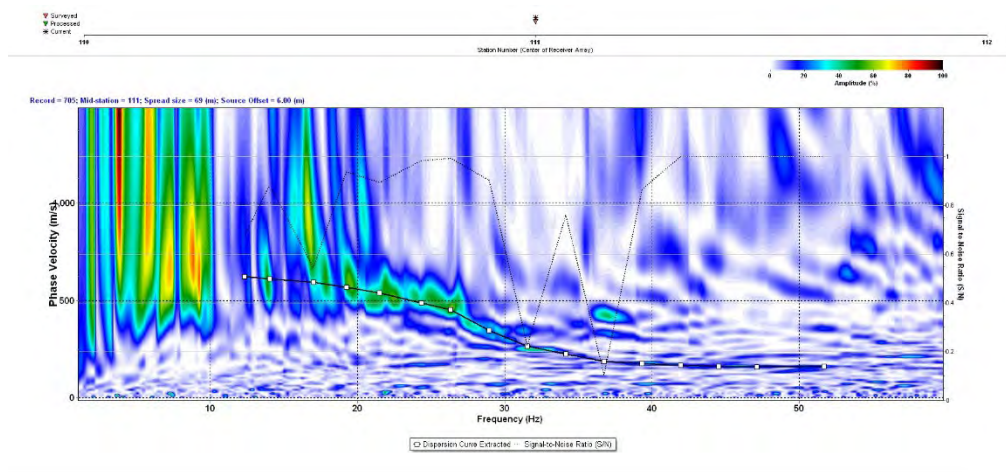
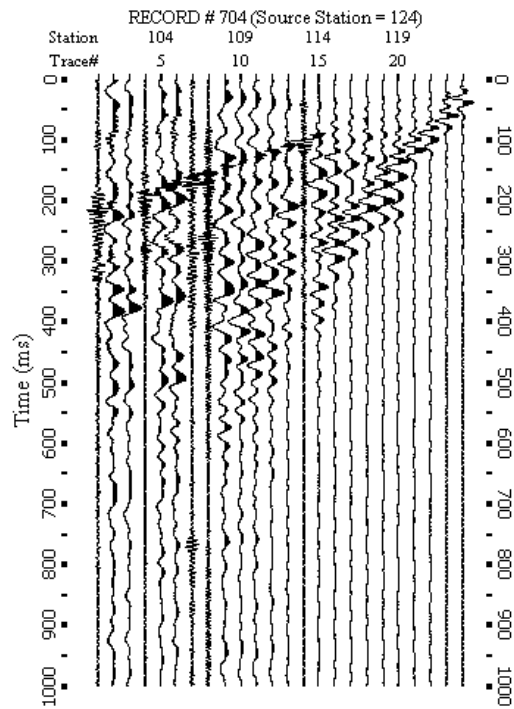
MASW24-04: Example time domain trace for active source (top) with resulting overtone image with picked dispersion curve (bottom).



MASW24-05: Example time domain trace for active source (top) with resulting overtone image with picked dispersion curve (bottom).



MASW24-06: Example time domain trace for active source (top) with resulting overtone image with picked dispersion curve (bottom).



MASW24-07: Example time domain trace for active source (top) with resulting overtone image with picked dispersion curve (bottom).

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2020 National Building Code of Canada Seismic Hazard Tool

i This application provides seismic values for the design of buildings in Canada under Part 4 of the National Building Code of Canada (NBC) 2020 as prescribed in Article 1.1.3.1. of Division B of the NBC 2020.

Seismic Hazard Values

User requested values

Code edition	NBC 2020
Site designation X_v	X_{311}
Latitude (°)	42.81
Longitude (°)	-81.229

Please select one of the tabs below.

NBC 2020

Additional Values

Plots

API

Background Information

The NBC 5% damped spectral acceleration values can be viewed in the NBC tab.

Additional hazard values for your site can be found below.

The 5%-damped spectral acceleration ($S_a(T)$, where T is the period, in s) and peak ground acceleration (PGA) values are given in units of acceleration due to gravity (g , 9.81 m/s^2). Peak ground velocity (PGV) is given in m/s. Probability is expressed in terms of percent (%) exceedance in 50 years.

By default, all probabilities for the user-specified site designation are shown. Other site designations can be selected from the respective drop-down menu in the table. In low hazard regions, a minimum value of $0.001g$ for $T \leq 2.0s$ and of $0.0001g$ for $T > 2.0s$ is

assigned. Further information on the calculation of seismic hazard is provided in the *Background Information* tab.

Site Designation	Probability	S _a (0.05)	S _a (0.1)	S _a (0.2)	S _a (0.3)	S _a (0.5)	S _a (1.0)	S _a (2.0)	S _a (5.0)	S _a (10.0)	PGA	PGV
X311	All											
X ₃₁₁	2	0.156	0.194	0.193	0.179	0.143	0.0785	0.0369	0.00953	0.00325	0.101	0.0886
X ₃₁₁	2.5	0.135	0.17	0.171	0.158	0.126	0.069	0.0322	0.00819	0.0028	0.0885	0.0769
X ₃₁₁	3.5	0.108	0.138	0.14	0.131	0.103	0.0563	0.026	0.00644	0.00222	0.0719	0.0618
X ₃₁₁	5	0.085	0.11	0.113	0.105	0.083	0.0447	0.0203	0.0049	0.00169	0.0572	0.0483
X ₃₁₁	7	0.0667	0.0877	0.0905	0.0846	0.0665	0.0355	0.0159	0.00369	0.00128	0.0455	0.0376
X ₃₁₁	10	0.0507	0.0678	0.0706	0.066	0.0518	0.0272	0.0119	0.00265	0.000914	0.0352	0.0283
X ₃₁₁	14	0.0384	0.0522	0.0549	0.0513	0.04	0.0206	0.00872	0.00186	0.000639	0.0271	0.0211
X ₃₁₁	20	0.0276	0.0384	0.0409	0.0383	0.0297	0.0148	0.00602	0.00121	0.000411	0.02	0.015
X ₃₁₁	30	0.0178	0.0256	0.0279	0.0262	0.02	0.00954	0.00365	0.000683	0.000225	0.0134	0.00953
X ₃₁₁	40	0.0121	0.018	0.0202	0.019	0.0144	0.00655	0.00237	0.000419	0.000134	0.00947	0.00651

Download CSV

← Go back to the [seismic hazard calculator form](#)

Date modified: 2021-04-06