Foundation Investigation and Design Report – CNR Overhead – 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 42.814091
Longitude -81.235023

Geocres No. 40I14-224

Prepared for:

Ministry of Transportation, Ontario (MTO), West Region

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For
G.W.P. 3042-22-00
CNR Overhead

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 (Station between 13+100 and 11+100, south side of the existing Highway 3 & between Station12+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.

This Foundation Investigation Report has been prepared specifically and solely for the proposed CNR overhead structure. 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

Talbotville bypass is planned to cross CNR tracks at approximately Station 12+020, about 700 m west of the proposed Ron McNeil underpass in the Township of Southwold, Elgin County, Ontario. The site location is shown on the Key Plan inset to Drawing Nos. 1 to 4 included in Appendix A.

2.2 GENERAL SITE DESCRIPTION

At the proposed location of CNR Overhead, the proposed Highway 3 Talbotville Bypass is planned to be a divided freeway, with two lanes (with paved shoulders) in each direction, divided by a grass median. The orientation of Talbotville Bypass is approximately northwest-southeast and the orientation of the CNR track is approximately north-south. For the purposes of this report, the orientation of Talbotville Bypass and the CNR track are taken as east-west and north-south, respectively.

At the project site, CN ROW is approximately 25 m wide and contains one track.

The area immediately adjacent to the proposed overhead structure and within most of embankment areas consists of heavily wooded area. There are agricultural fields further north and south of the wood area. The ground surface at the site generally slopes towards south and west.

Photo 1. CNR Overhead North Embankment Site (looking North)

2.3 PROPOSED STRUCTURE AND EMBNAKMENTS

Based on the General Arrangement (GA) drawing, a wide single-span, rigid-frame structure (a 27 m long span and 145 m width) will carry both the EB and WB lanes of the proposed Highway 3 Talbotville Bypass over the CNR ROW. The proposed width of the Highway 3 median is about 22.5 m. The GA drawing indicates that the overhead structure will be constructed at 61.3º skew angle to the CNR track alignment.

Based on the plan, ± 300 m long bridge approach embankments are proposed on either side of the overhead structure. Both the EB and WB lane will be constructed on a single embankment with a top width of approximately 50 m. The centreline profile of EB and WB lanes at the proposed overhead location are planned to be at approximately elevation 251 m, approximately 10 m higher than the surrounding lands. Concrete retaining walls are also proposed the end of embankments.

The GA drawing is included in Appendix A for reference.

2.4 GEOLOGICAL INFORMATION

The site is located within the physiographic region of Mount Elgin Ridges, 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 at the proposed CNR overhead site is estimated to be about 85 m below the original ground surface (o.g.).

2.5 EXISTING UTILITIES

A review of available information indicated that there are no existing utilities in the immediate area of the proposed overhead structure and embankments (within the wood lot). There is a gas main and watermain running along Wonderland Road to the east and a gas main crossing the farm field northwest of the proposed overhead structure and embankments.

3.0 REVIEW OF PREVIOUS INVESTIGATIONS

A review of MTO GEOCRES database identified the following report at the CNR overhead site:

<u>GEOCRES Reference No. 40I14-070</u>

A foundation investigation report dated September 17, 1971, was available for the proposed crossing at CNR spur overhead and St. Thomas Expressway.

The report was referenced as follows:

> Foundation Investigation Report
> For Proposed Crossing at
> CNR Spur Overheads and St. Thomas Expressway
> Twps. Of Southwold; County of Elgin
> W.O. 71-11068 - W.P. 89-69-05 & 06

The investigation included a total of eight (8) sampled boreholes (BH No. 1 to 8), advanced to depths ranging from approximately 10.4 m to 30.2 m below grade (corresponding to approximately elevations 229.8 m to 210.1 m) and eight (8) dynamic cone penetration tests carried out adjacent to each borehole advanced in July 1971.

The boreholes encountered a deep stratum of stiff to hard clayey silt with some sand and trace gravel immediately below the topsoil. Except the top 2 m, the stratum had a moisture content that was at or below the Plastic Limit. The undrained shear strength of the stratum generally decreased with depth, being in excess of 240 kPa at approximate elevation 237.8 m and about 190 kPa at approximate elevation 213.4 m. The deposit appeared to be highly over-consolidated.

Groundwater levels were observed at elevations ranging from approximately 231 m to 218.1 m.

Following shifts in the alignment of the St. Thomas Expressway at the CNR overhead, five (5) additional borings (BH No.11 to 15) were advanced to a depth of approximately 5 m below grade at this site, which reported similar subsoil conditions as those indicated above.

For reference, copies of the Borehole Location Plan, stratigraphical 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 CNR overhead and embankments originally consisted of a total of 12 boreholes, designated as CNR-EMB01 to CNR-EMB12 for the proposed 600 m long embankments and four (4) boreholes, designated as CNR-OH1 to CNR-OH4 for the overhead structure abutments. As per discussions with the MTO, Boreholes CNR-EMB02, CNR-EMB08 and CNR-EMB10 were subsequently converted to seismic Cone Penetration Tests (sCPTs). The locations of the boreholes and sCPTs are shown on the Borehole Location Plan, Drawing Nos. 1 to 4 in Appendix A.

MTO cleared an alignment within the wooded area for the foundation investigation (along the bypass centreline). Prior to carrying out the investigation, Stantec contacted the public utility authorities, private locate and MTO to clear the borehole locations of private, public as well as and MTO-owned utilities.

The field drilling program was carried out from March 4, 2024, through to July 11, 2024. The approach embankment and overhead structure boreholes were advanced to depths of approximately 15.9 m and 44.8 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 CNR-OH1 to CNR-OH4. 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. 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. Shelby tube (thin-walled steel tube) samples were also obtained in several boreholes were lower N-values were obtained from the SPTs.

Three (3) CPT tests, designated as SCPT-CNREMB02, SCPT-CNREMB08 and SCPT-CNREMB10 were conducted by ConeTec at the site on May 9 and 10, 2024. The CPTs were advanced to depths of approximately 15.6 m, 20.1 m and 20.0 m below grade, respectively, where refusal to penetration of the cone was encountered. The CPTs included seismic measurements and pore dissipation tests. The ConeTec report dated May 24, 2024, is included in Appendix C.

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 CNR-OH1. The borehole annulus surrounding the slotted pipe section was backfilled with sand. The remaining annulus was backfilled with bentonite up to the ground surface. A groundwater level measurement in the monitoring well was taken out on August 29, 2024.

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

Multi-Channel Analysis of Surface Wave (MASW) measurements were carried out on both sides of CNR ROW to determine the seismic site class; the surveys consisted of a single a 69 m long array of geophones spaced at 3 m, at each site. The MASW report is included in Appendix F.

4.2 LOCATION AND ELEVATION SURVEY

The borehole locations and respective ground surface elevations were surveyed by Stantec Geomatics personnel using a 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 and CPT 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				
CNR-EMB1	4742387.6	408194.1	241.4	15.9	225.5	14
SCPT-CNREMB-02	4742437.0	408134.9	241.0	15.6	225.4	-
CNR-EMB3	4742473.2	408075.1	241.8	15.9	225.9	15
CNR-EMB4	4742503.0	408026.1	241.4	15.9	225.6	15

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				
CNR-EMB5	4742528.0	407978.3	240.8	15.9	224.9	15
CNR-EMB6	4742559.1	407913.2	239.9	15.9	224.0	15
CNR-EMB7	4742291.1	408305.7	240.1	15.9	224.3	14
SCPT-CNREMB-08	4742299.9	408360.7	240.0	20.1	219.9	-
CNR-EMB9	4742180.0	408425.8	239.7	15.9	223.9	14
SCPT-CNREMB-10	4742135.9	408477.0	239.0	20.0	219.0	-
CNR-EMB11	4742091.9	408523.1	239.2	15.9	223.4	15
CNR-EMB12	4742048.1	408572.4	239.1	15.9	223.2	15
CNR-OH1	4742326.5	408283.1	240.2	44.8	195.4	26
CNR-OH2	4742271.9	408298.2	240.4	44.8	195.6	25
CNR-OH3	4742374.5	408227.3	240.7	44.8	195.9	26
CNR-OH4	4742319.5	408248.9	240.5	44.8	195.7	26

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.

Table 4.2: Geotechnical Laboratory Testing Program

Laboratory Test Type	Number of Tests
Moisture Content	238
Gradation Analysis	54
Atterberg Limits	57
Consolidation (Oedometer)	4
Unconsolidated Undrained Triaxial Compression Test (UU)	1
Chemical Analysis	4

Four (4) soil samples from the boreholes advanced for the CNR overhead 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 and profile of the soils encountered in the boreholes along the bridge alignment and embankments are provided on Drawing Nos.1 and 2 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,
- Fill comprising silty clayey sand in CNR-EMB03; underlain by,
- Stiff to hard clayey silt till; underlain interbedded with,
- Stiff to hard clayey silt in CNR-EMB1, CNR-EMB6, CNR-EMB9, CNR-EMB11 and CNR-EMB12.

Detailed descriptions of the subsurface conditions are provided below.

5.2 OVERBURDEN

5.2.1 Topsoil

Topsoil was encountered at all boreholes. The thickness of the topsoil varied from approximately 100 mm to 300 mm.

Laboratory tests conducted on samples from the topsoil yielded moisture contents of approximately 25% and 50%.

5.2.2 Fill

A layer of fill material comprising brown silty clayey sand was encountered below the topsoil in borehole CNR-EMB03. Samples obtained from the fill layer contained trace gravel.

The fill layer was approximately 1.3 m thick and extended to a depth of approximately 1.5 m below grade corresponding to approximately elevation 240.3 m.

N-values of 2 and 10 blows per 0.3 m were obtained from the SPTs advanced in the fill layer, indicating a very loose to compact condition.

Laboratory tests conducted on the samples of the fill yielded natural moisture contents of approximately 22% and 15%.

Gradation analyses were carried out on a sample of the fill soils. 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:	6%
Sand:	46%
Silt:	30%
Clay:	18%

Atterberg Limits tests were conducted on the sample referenced above. The tests yielded a Liquid Limit of approximately 18%, a Plastic Limit of approximately 11%, and a corresponding Plasticity Index of approximately 7%. The test results are illustrated on the borehole record in Appendix C and on the gradation curve on Figure No. D2 in Appendix D.

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

5.2.3 Clayey Silt 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. Samples obtained from the top portion of the till deposit were noted to have a slightly higher moisture content and plasticity in some boreholes. Localized seams/layers of soils with higher silt content and lower plasticity (described in the proceeding section) were noted within this deposit. Presence of cobbles and/or boulders was inferred at depth in the till deposit in borehole CNR-OH2 due to difficulties in drilling. Presence of cobbles and/or boulders should be anticipated in the till soils at other locations and depths.

All boreholes except CNR-EMB1, CNR-EMB6 and CNR-EMB9 were terminated in this deposit after penetrating 14.8 m to 44.8 m into the layer.

The N-values obtained from the SPTs advanced in the top 1 metre of the clayey silt till deposit ranged from 2 to 8 blows per 0.3 m penetration, indicating a soft to stiff consistency. Below this surficial zone, N-values ranging from 10 to 65 were obtained from the SPTs. N-values of greater than 30, indicating a hard consistency, were obtained below a depth of approximately 27 m below grade (approximately elevation 214 m) in boreholes CNR-OH2 and CNR-OH3 and below a depth of approximately 32 m below grade (approximately elevation 208 m) in boreholes CNR-OH1 and CNR-OH4.

An in-situ shear vane test (MTO B-vane) attempted at a depth of approximately 1.4 m below grade in borehole CNR-EMB07 encountered refusal, indicating an undrained shear strength greater than 200 kPa.

An in-situ shear vane test (MTO B-vane) was conducted at a depth of approximately 19.2 m below grade, corresponding to approximate elevation 221 m in borehole CNR-OH1, which indicated undisturbed and remoulded undrained shear strengths of 160 kPa and 80 kPa, respectively corresponding to a sensitivity of 2.

Pocket penetrometer tests were also used to estimate the clayey silt till strength/consistency at the site. UCS, estimated by using the pocket penetrometer on recovered split-tube soil samples, were from 0.5 kgf/cm^2 to greater than 4.5 kgf/cm^2 and suggested an undrained shear strength of 50 kPa to greater than 220 kPa. Based on the results of these tests, the clayey silt till can be described as stiff to hard.

An Unconsolidated Undrained (UU) Triaxial test was conducted on a select Shelby tube sample retrieved at a depth of approximately 4.9 m below grade, corresponding to approximate elevation 235.8 m in borehole CNR-OH3. The test indicated a Compressive Strength of approximately 190 kPa corresponding to an undrained shear strength of approximately 95 kPa. The details of the test are included in the test sheets in Appendix D.

Laboratory tests conducted on samples of the clayey silt till deposit yielded natural moisture contents ranging from approximately 11% to 28%, averaging 16%. Higher moisture contents were obtained from the surficial samples of the clayey silt till deposit.

Gradation analyses were carried out on fifty 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 17%
Sand:	0 to 19%
Silt:	32 to 68%
Clay:	29 to 48%

Atterberg Limits tests were conducted on the samples referenced above. The tests yielded Liquid Limits ranging from approximately 23% to 37%, Plastic Limits ranging from approximately 11% to 19%, and corresponding Plasticity Indices ranging from approximately 9% to 21%. 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 and silty clay with a group symbol of CI based on the Unified Soil Classification System (USCS).

One-dimensional oedometer consolidation tests were carried out on portions of selected Shelby tube samples. The results are provided below in Table 5.1 and the details of the tests, including the data plots, are provided on the laboratory test sheets in Appendix D.

Table 5.1: One-Dimensional Oedometer Consolidation Test Results

Borehole/ Sample	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)
CNR-EMB1/ ST1	235.7	0.51	21.19	500	0.11/0.013	6.1	4.3e^{-3}
CNR-EMB7/ ST1	236.0	0.55	20.78	500	0.141/0.015	10	2.5e^{-3}
CNR-EMB9/ ST2	233.3	0.41	21.99	650	0.11/0.01	8.3	4.6e^{-3}
CNR-EMB11/ ST2	224.1	0.51	20.98	500	0.125/0.012	3.0	4.2e^{-3}

5.2.4 Clayey Silt

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 described in the preceding section in boreholes CNR-EMB01, CNR-EMB06, CNR-EMB09, CNR-EMB11 and CNR-EMB12.

The clayey silt layer was approximately 1.6 m and 1.5 m thick and extended from depths of approximately 11.7 m and 10.2 m below grade, corresponding to approximately elevations 227.5 m and 223.2 m to depths of approximately 13.3 m and 11.7 m below grade, corresponding to approximate elevation 225.9 m and 221.7 m, respectively in boreholes CNR-EMB11 and CNR-EMB12. Boreholes CNR-EMB1, CNR-EMB06 and CNR-EMB09 were terminated in the clayey silt layer after penetrating 1.1 m, 0.2 m and 1.1 into the layer, respectively.

N-values obtained from the SPTs advanced in the clayey silt layer ranged from 17 to 51, indicating very stiff to hard consistency.

Two (2) pocket penetrometer tests were also conducted on samples obtained from this layer to estimate the clayey silt strength/consistency at the site. UCS, estimated by using the pocket penetrometer on recovered split-tube soil samples, were 2 kgf/cm^2 and greater than 4.5 kgf/cm^2 and suggested an undrained shear strength of 100 kPa and greater than 220 kPa. Based on the results of these tests, the clayey silt till can be described as very stiff to hard.

Laboratory tests conducted on samples of the clayey silt deposit yielded natural moisture contents ranging from approximately 9% to 19%, 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 1%
Sand:	0 to 2%
Silt:	65 to 80%
Clay:	30 to 32%

Atterberg Limits tests were conducted on the samples referenced above. The tests yielded Liquid Limits ranging of approximately 19%, 20% and 23%, Plastic Limits of approximately 14%, 15% and 14%, and corresponding Plasticity Indices of approximately 5%, 5% and 9%. The test results are illustrated on the borehole records in Appendix C and on the gradation curve 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 and CL 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 CNR-OH1 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. Soil colour change from brown to grey was noted at approximate depths of 2.3 m to 3.8 m below grade, corresponding to approximately elevations 238.1 m to 236.1 m. Cave-in depths were also recorded. The groundwater level recorded in CNR-OH1 and inferred in the other boreholes are summarized in Table 5.1 below.

Table 5.2: Measured and Inferred Groundwater Levels

Borehole No	Date	Groundwater Level (m)		Remark
		Depth	Elevation	
CNR-EMB1	Upon Completion	Dry		Borehole Open
CNR-EMB3	Upon Completion	Dry		Borehole Open
CNR-EMB4	Upon Completion	Dry		Borehole Open
CNR-EMB5	Upon Completion	Dry		Borehole Open
CNR-EMB6	Upon Completion	Dry		Borehole Open
CNR-EMB7	Upon Completion	Dry		Borehole Open
CNR-EMB9	Upon Completion	Dry		Borehole Open
CNR-EMB11	Upon Completion	13.6	225.6	Cave-in at 14.5 m
CNR-EMB12	Upon Completion	Dry		Cave-in at 14.6 m

Borehole No	Date	Groundwater Level (m)		Remark
		Depth	Elevation	
CNR-OH1	Well installed on June 6, 2024 Measured on August 29, 2024	0.5	239.7	Borehole was dry upon start of mud drilling (from 3 m bgs)
CNR-OH2	Borehole was dry upon the start of mud drilling (from 3 m bgs). Further groundwater monitoring was not possible due to the introduction of water during drilling.			
CNR-OH3	Borehole was dry upon the start of mud drilling (from 3.6 m bgs). Further groundwater monitoring was not possible due to the introduction of water during drilling .			
CNR-OH4	Borehole was dry upon start of mud drilling (from 3.6 m bgs) Further groundwater monitoring was not possible due to the introduction of water during drilling.			

The fact that the boreholes were observed to be dry at the time of drilling reflects the low permeability of the soils at this site rather than the depth to the water table.

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

Four (4) representative samples from the soils at the site were tested for pH, water-soluble sulphate and chloride concentrations, and resistivity. The analysis results are provided in the following table.

Table 5.3: Results of Chemical Analysis

Borehole No	Sample No.	Depth (m)	pH	Chloride (µg/g)	Sulphate (µg/g)	Resistivity (Ohm-cm)
CNR-OH1	SS8	5.6	8.48	6	194	3040
CNR-OH2	SS5	3.4	8.30	15	206	3370
CNR-OH3	SS4	2.6	8.35	16	185	2920
CNR-OH4	SS9	6.4	8.68	10	277	2430

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.

The wood lot was cleared for the foundations investigation by the Ministry of Transportation Ontario (MTO).

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.

The CPT tests and MASW measurements 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.

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
CNR Overhead

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 the 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 single new bridge structure will carry the EB and WB lanes of Talbotville Bypass over the Canadian National Railway Right of Way (CNR ROW). Based on the Preliminary General Arrangement (GA) drawing, a single rigid-frame structure is proposed.

This section of the report provides foundation engineering recommendations for the detailed design and construction of the new CN overhead structures and associated approach embankments. The foundation design report is intended for the use of the Ministry of Transportation Ontario (MTO).

8.2 PROPOSED OVERHEAD STRUCTURE AND EMBANKMENTS

Based on the GA Drawing provided by the Stantec Structural team, the proposed rigid frame overhead structure will include 27 m long span, with a 61.3° skew angle to the CN track alignment. The proposed rigid frame structure will be about 145 m wide and the highway median between the EB and WB lanes will be 23 m wide. The overall bridge height, from the track level to the underside of the deck, will be at least 7.3 m to meet the required CN minimum vertical clearance. Concrete retaining walls are also proposed at both ends of the overhead structure.

A single wide embankment is planned for the Highway 3 Talbotville Bypass. The embankment will be up to approximately 10 m high and will have a crest width of approximately 50 m.

No staging is anticipated for this proposed bridge and embankment construction.

Key elevations associated with the proposed new bridge structures are as follows:

Existing CNR track grade	Approximately Elevation 241.5 m
Proposed overhead bridge deck bottom	Approximately Elevation 248.8 m
Proposed Talbotville Bypass grade	Approximately Elevation 250.5 m
Proposed Pile Cap base/bottom of spread footing	Approximately Elevation 239.3 m

Consideration is being given to supporting the CNR overhead structure on either deep or shallow foundations depending on the site constraints, settlement mitigation measure feasibility, settlement tolerance of the proposed rigid frame structure, etc.

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 considers 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. A “High degree of understanding” for the embankment stability analyses was used based on the 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 GEOCRES 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).
- 25% of the embankment boreholes were 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 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 top portion of the clayey silt till was noted to have a higher plasticity 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 overhead 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.

Table 8.1: Geotechnical Model for Talbotville Bypass – CNR Overhead Structure

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	239	Firm CLAYEY SILT to SILTY CLAY TILL (CL to CI)	20	28	50	• Cc= 0.15, Cr= 0.015, P'c=550 kPa, e0= 0.52, Cv= 0.004
Ground Surface	240	FILL: very loose to compact Silty Clayey SAND (SC-SM) – CNR-EMB03	20	28	N/A	E= 30
239	220	Stiff to hard CLAYEY SILT (CL) TILL	• 21 kN/m^3 to El. 235 m • 22 kN/m^3 to El. 225 m • 21 kN/m^3 to El. 220 m	30	• 170 kPa to El. 230 m • 220 kPa to El. 225 m • 150 kPa to El. 220 m	• Cc= 0.15, Cr= 0.015, P'c=550 kPa, e0= 0.52, Cv= 0.004 to El. 235 m • Cc= 0.125, Cr= 0.012, P'c=550 kPa, e0= 0.4, Cv= 0.004 to El. 225 m • Cc= 0.15, Cr= 0.015, P'c=550 kPa, e0= 0.5, Cv= 0.004 to El. 220 m

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)
220	<195	Hard CLAYEY SILT (CL) TILL	21 kN/m^3 at El. 220 to 22 m kN/m^3 at El. 195 m	30	150 kPa at EL. 220 m to 300 kPa at El. 195 m	• Cc= 0.1, Cr= 0.01, P'c=600 kPa, e0= 0.4, Cv= 0.004

Notes:

1 A groundwater level at elevation of 240 m is recommended for use in foundation design based on the groundwater level in the monitoring well and CPT sounding results. 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.

3 Compressibility Parameters: E = Soil Modulus, P'c = Estimated Pre-consolidation Pressure, Cr = Recompression Index, Cc = Compression Index, Cv = Coefficient of Consolidation

4 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.

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 14, 2024, by ConeTec and the results were provided in a report dated June 10, 2024. The survey consisted of two (2) Multichannel Analysis of Surface waves (MASW) tests (MASW24-01 and MASW24-02), one on each side of the CNR ROW. Based on the test results, shear wave velocity (V_{s30}) values of 300 m/s and 329 m/s for MASW24-01 and MASW24-02, respectively, are 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 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 Sa(0.2) and Sa(1.0) values based on NBCC2020 for the site using an average weighted shear wave velocity of 300 m/s are provided in Appendix F. As per the

8.6.3 Peak Ground Acceleration (PGA)

Seismic hazard values for the site were obtained from Natural Resources Canada (2020 National Building Code Seismic Hazard Tool) using an average shear wave velocity (V_{s30}) value of 300 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)*
0.102	0.192g	0.0805g

Note * based on the average Vs=300 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 include 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 is not be a significant issue for this project.

8.7 FOUNDATION OPTIONS

Both shallow and deep foundation options were assessed for the proposed overhead structure 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 through the stiff to hard clayey silt till deposit.

N-values of greater than 100 (MTO SPT N-value refusal criteria) were not obtained to the termination depth of the boreholes. In this respect, end bearing driven piles or end bearing drilled shafts are not considered feasible options for this site.

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:

Table 8.3: Comparison of Foundation Options for the CNR Overhead Structure

Option	Advantages	Disadvantages	Relative Cost	Risk/Consequences
Shallow foundation within very stiff to hard clayey silt till	• Feasible for the proposed rigid frame structure if larger settlement than typical foundation serviceability condition is acceptable • Lower construction costs than deep foundations • Less vibration than pile driving	• Lower geotechnical capacity compared to deep foundations which may necessitate large footing area • Larger excavation	Low to medium	• Potential excessive settlement under large structural loads • Increased potential for differential settlement • Potential embankment related settlements
Frictional Driven Steel H-Piles	• Feasible for the proposed rigid frame structure • Reduced soil setup time compared to frictional pipe piles (less displacement)	• Structural capacity cannot be fully utilized • Risk of soil plugging between flanges due to very stiff to hard clayey soils • Post construction foundation settlement and downdrag load should be considered if preloading is not carried out • Easement agreement with CNR may be required for battered piles.	Medium	• 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 • Potential embankment related settlements
Frictional Driven Steel Pipe Piles (Open-ended)	• Feasible for the proposed rigid frame structure • Soil plug-in issues are not anticipated • Minimize ground heaving and vibration	• Structural capacity cannot be fully utilized • More vibration during driving and possible heave and displacement of ground and adjacent piles are anticipated	Medium	• Long piles required for moderate capacities • Possible pile drivability issues (displacement piles) • Anticipated vibration and ground heave

Option	Advantages	Disadvantages	Relative Cost	Risk/Consequences
	potential (compare to close ended pipe piles)	• Compared to frictional H-Piles, longer set up time is required (displacement pile) • Post construction foundation settlement and downdrag load should be considered if preloading is not carried out • Easement agreement with CNR may be required for battered piles.		• Cobbles and boulders may be encountered in glacially derived soils that could impede pile penetration to required depths • Potential embankment related settlements
Drilled Shafts (Cast-in-place concrete piles)	• Feasible for the proposed rigid frame structure • Potentially reduced number of deep foundation elements compared to driven piles (dependent on drilled shaft diameter and depth) • Lower vibration than pile driving	• Structural capacity cannot be fully utilized • Temporary liner and/or drilling fluid may be required for relatively longer drilled shafts • Larger downdrag loads than driven steel piles are anticipated for larger diameter and deep drilled shafts • Post construction foundation settlement and downdrag load should be considered if preloading is not carried out • Shaft batter is not feasible	High	• Drill hole stability and groundwater control • More onerous Inspection and testing requirements than steel driven piles • Potential embankment related settlements
CFA Piles (Continuous Flight Auger Piles)	• Feasible for the proposed rigid frame structure • Lower vibration than pile driving • Relatively faster construction than drilled shafts • Cheaper unit construction cost than drilled shaft	• Reinforcement should be inserted into wet concrete and installation depth of reinforcement is limited • Pile batter is not feasible	Medium	• Relatively new technology for MTO bridges • Pile integrity and load tests requirements • Potential embankment related settlements
Drilled pipe Piles (with a ring bit and down the hole hammer)	• Feasible for the proposed rigid frame structure • Lower vibration than pile driving • Relatively faster construction than drilled shafts	• Structural capacity cannot be fully utilized • Pile batter is not feasible • Only limited number of specialist foundation contractors and unit construction cost can be high	High	• Relatively new technology for MTO bridges • Pile integrity and load tests requirements • Potential embankment related settlements

Based on the comparison presented in the above table, shallow foundations on improved ground are considered as a preferred option from the foundation's perspective. However, due to the site constraints with the CN ROW, ground improvement such as preloading and surcharging, shallow foundation construction within the CN ROW are considered not feasible.

Alternatively, consideration can also be given to the use of frictional steel H-Piles or open-ended steel pipe piles driven through the stiff to hard clayey silt till deposit. Driven steel pipe piles may be a preferred foundation option due to the possible soil plug issue of longer steel driven H piles. Due to the potential ground heaving, vibration and pile alignment issues, open ended pipe piles are considered more suitable than close ended pipe piles. Additional site grading and pile re-strike after the initial drive may be still required for open ended pipe piles. Based on the on-going MTO Highway 401 DB 2022-3004 project in London, Ontario, no specific pile driving related vibration monitoring was requested by the CN for the similar site setting for the existing CN Spur line. Stantec also made the work arrangement with the CN for this CNR overhead construction, and no specific vibration monitoring was requested by the CN for the proposed pile driving. Battered driven piles may encroach into the CN right-of-way (ROW), necessitating an easement agreement

All of the above foundation options would be subjected to excessive settlements imposed by the proposed high embankments if settlement control measures such as preloading and surcharging, the use of light weight fill (such as EPS, light weight foam or light weight foam concrete, etc.) and combination of both ground improvement and light weight fill are not carried out.

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 Canadian Highway Bridge Design Code (CHBDC, 2019).

8.7.1.1 Design Considerations

Driven pile foundations consisting of steel pipe piles, deriving their load-carrying capacity from both shaft friction and tip resistance (with consideration of soil plug), can be used to support the proposed rigid frame structure. Open-ended pipe piles (with tip reinforcement, e.g. APF cutting shoe) are recommended.

The driving of steel pipe piles for the new overhead structure is not expected to adversely affect the CN spur line track, which is not in service, in the vicinity. Therefore, vibration monitoring is not anticipated to be required.

Given the length of the pipe piles required to support the proposed structure, pile splicing may be required.

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 open-ended steel pipe piles were assessed using the API (American Petroleum Institute) design method with the program APILE (Ensoft, 2019) and Stantec's design build projects' foundation experience in London area. The geotechnical model outlined in Table 8.1 was used as input to the analysis. The factored geotechnical resistances at Ultimate Limit States (ULS) and Serviceability Limit States (SLS) are provided in the following table.

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_f[2,3] (kN)	Factored Geotechnical Resistance at SLS_f[3] (kN)
356 mm O.D. and. 9.53 mm thick pipe pile	35.0	204	1200	not governing

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 considered in calculating the factored geotechnical resistance at Serviceability Limit State (SLS).

The estimated geotechnical reaction at SLS_f for a 25 mm vertical settlement exceeds the geotechnical reaction at ULS_f 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.

Axial Resistance in Tension

The axial resistance in tension at Ultimate Limit State (ULS) for driven 356 mm O.D. and 9.53 mm thick wall steel pipe piles were assessed using the API (American petroleum institute) design method with the program APILE (Ensoft, 2019) and Stantec's design build projects' foundation experience in London area. 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_f (kN)
356 mm O.D and 9.53 mm thick pipe pile	35	850

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 CNR Overhead approach embankments will be about 10 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 may be close to the factored ULS loading) and unfactored downdrag loads acting on the 356 mm O.D. pipe piles may be about 600 kN if the site is not preloaded sufficiently or light weight fill is not utilized for the bridge abutment backfill zone as per section 8.9.3 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.

8.7.1.4 Pile/Soil Setup, Relaxation and Pile Capacity Validation

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_f 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 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.

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

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 pipe 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 is 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 2400 kN per pile (356 mm O.D. and 9.53 mm thick steel pipe pile) based on a geotechnical resistance factor of 0.5.

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 two times the factored geotechnical resistance at ULS for the selected pile type.

8.7.1.5 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 typical steel tube pile driving shoe such as Associated Pile & Fitting (APF) open end cutting shoe or equivalent.

8.7.1.6 Geotechnical Lateral Resistance

<u>P-Y Curves</u>

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 356 mm O.D. and 9.53 mm thick pipe piles. The geotechnical parameters provided in Table 8.1 and fixed pile head conditions were used in the analyses.

The p-y curve values versus depth at the abutments are presented in Figure E3 and Table E1 included in Appendix E. This table provides 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 pipe 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 resistances of the piles at ULS and SLS.

<u>Group Action</u>

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.

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

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

8.7.2.1 Design Considerations

Depending on the loading conditions and settlement tolerance of the rigid frame structure, the abutments of the proposed overhead can be supported on shallow foundations placed on the undisturbed native very stiff to hard clayey silt till below the local frost depth. The shallow foundations should be provided with a minimum of 1.2 m of earth cover to provide adequate protection against frost penetration.

The proposed retaining walls can also be supported on spread or strip footings placed on the undisturbed native very stiff to hard clayey silt till below the local frost depth. All foundations should be provided with a minimum of 1.2 m earth cover or equivalent thermal insulation for frost protection.

In preparation for construction of the new shallow 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 foundations.

Possible excavation within CN Right of Way (ROW) should be discussed with and approved by the CN.

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.

It should be noted that factored serviceability geotechnical resistance for the proposed overhead structure abutment and retaining wall foundations may not necessarily be applicable as the founding subgrade could settle as much as 150 mm in conjunction with the proposed 10 m high new embankment. The anticipated embankment load and settlement may govern the shallow foundations' serviceability and performance. A proper preloading, preloading plus surcharge or other settlement mitigation measures such as light weight fill should be implemented to accommodate the shallow foundation option for the overhead structure and retaining wall foundations.

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 sandy clayey silt till soils.

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

Footing Width (m)	Founding Elevation (m)	Factored Geotechnical Resistance at ULS_f (kPa)	Factored Geotechnical Reaction at SLS_f (kPa)*
3.5	238.5	425	125 (for 25 mm settlement) *
3.5	238.5	425	275 (for 50 mm settlement) *
7	238.5	450	75 (for 25 mm settlement) *
7	238.5	450	175 (for 50 mm settlement) *

Note: *actual shallow foundation settlement will be governed by the subgrade settlement under embankment loading.

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 settlements associated with the SLS_f values provided above are based on footing loads only and assume that either a preloading and surcharge program of has been carried out to eliminate the embankment fill settlements or that an EPS lightweight fill is used in the vicinity of the structure.

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 sandy 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 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).

8.8 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 overhead abutments should consist of free-draining granular fill placed and compacted using methods and 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.

Consideration can be given to the use of light weight embankment fill such as expanded polystyrene (EPS) to mitigate both embankment settlement and lateral earth pressure issues. More details about the EPS fill option are provided in Section 8.9.3.

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. 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:

$$P_A = \tfrac{1}{2} K_a \gamma H^2$$
$$P_O = \tfrac{1}{2} K_o \gamma H^2$$
$$P_P = \tfrac{1}{2} K_p \gamma H^2$$

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 8.8: 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	32º	35º
Coefficient of Earth Pressure at Rest (Ko)	0.47	0.43
Coefficient of Active Earth Pressure (Ka)	0.31	0.27
Coefficient of Passive Earth Pressure (Kp)	3.25	3.69

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

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:

$$P_{AE} = \tfrac{1}{2} K_{AE}\, \gamma\, H^2 (1 - k_V)$$

$$P_{PE} = \tfrac{1}{2} K_{PE}\, \gamma\, H^2 (1 - k_V)$$

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 8.9: Seismic Design Parameters to Estimate Lateral Earth Pressures

PGA	Horizontal Acceleration Coefficient, k_{ho}	Horizontal Acceleration Coefficient, k_h
	Non-Yielding	Yielding $(wall\ movements\ of\ 25\ mm\ to\ 50\ mm)$
0.102g	0.102	0.051
Note: k_{ho} is the seismic horizontal acceleration coefficient that corresponds to zero wall movement and is equal to the PGA 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 8.10 for horizontal backfill configuration. These values should be adjusted if sloped backfill is considered.

Table 8.10: 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
Height of Application of PPE from base as a ratio of wall height, (H)	0.325	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 EMBANKMENT DESIGN CONSIDERATIONS

As part of the project, new approach embankments will be constructed to carry the proposed Highway 3 Talbotville Bypass EB and WB lanes over CNR ROW. Based on the GA drawing, the approach embankments will be up to about 10 m higher than the surrounding grade.

Based on the GA and highway embankment cross-section drawings, the proposed embankment will be constructed with 2.5H:1V to 2H:1V side slopes. Given that the overall embankment height will be in excess of 8 m, a mid-slope bench should be provided 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).

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

Slope stability analyses were carried out at the critical locations of the approach embankments (i.e., section where the embankment is highest, right at the east and west abutments) using the commercially available slope stability analysis software, SLOPE/W (GeoStudio 2021). Slope stability analyses were also carried out for up to 4.5 m high embankment using cohesive embankment fill. The input geotechnical design parameters are summarized in Table 8.1 and Table 8.11 below. Given the consistency in subsurface conditions, a single model was developed for both east and west embankments. A 2 m wide mid-slope bench was not considered in the analyses since it is more for a slope maintenance purpose than slope stability enhancement. 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 8.11: Geotechnical Design Parameters – CNR Overhead 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 Ic**>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, Ic=(Liquid Limit - Natural Moisture Content)/(Liquid Limit – Plastic Limit)

A minimum factor of safety (FOS) of 1.33 to 1.43 (corresponding to resistance factor 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 a slope stability analysis for the approach embankment are presented on Figures E4 to E9 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, and FOS>1.1 for pseudo-static condition). Based on the stability analyses results, cohesive fills from the project deep cut area may be considered up to 4.5 m high embankment with 2H:1V side slope if cohesive fills have a consistency index higher than 0.85 and can be compacted to their 95% of SPMDD.

8.9.2 Embankment Settlements

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.

Table 8.12: 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
Freeways	25	50	75	100

The above criteria also have a differential settlement limit of 200:1 for Freeways.

A pseudo-three-dimensional settlement analysis using Rocscience Settle3D was carried out to check the settlement magnitude and timeline for the proposed embankment. Soil parameters summarized in Table 8.1 and CPT sounding results were used for the settlement analyses. Drainage conditions of the clay deposits were adjusted based on the CPT pore pressure profiles. 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 results are summarized below in Table 8.13.

Table 8.13: Estimated Settlements in Transition Zones

Approx. Station	Location	Approx. Embankment Height (m)	Settlement Criteria (mm)	Estimated Total Settlement (mm)	Estimated Immediate Settlement (mm)	Estimated Post Construction Settlement (mm)
North Abutment and Embankment						
12+000	North Abutment	10.0	25	150	50	100
19+980	20 m	10.0	50	150	50	100
19+950	50 m	9.5	75	143	48	95
19+925	75 m	9.0	100	135	45	90
South Abutment and Embankment						
12+030	South Abutment	10.0	25	150	50	100
12+050	20 m	10.0	50	150	50	100
12+080	50 m	9.5	75	143	48	95
12+130	75 m	9.5	100	135	45	90

As summarized above, the post construction settlements within the transition zones exceed the MTO Embankment Settlement Criteria. Immediate settlements will be completed during the embankment construction. Approximately 95% of the consolidation settlements are expected to be completed within four (4) months after the completion of embankment construction (under the full height embankment loading).

In addition to the immediate and consolidation settlements, self-weight settlements of the embankment materials should also be considered for the post construction settlement. If well compacted cohesive fills (compacted to minimum 95% of the materials' SPMDD as per OPSS 501) are used for the embankment construction, self-weight settlements of 0.5 to 1% of the new embankment height will occur within one to two years after the completion of embankment construction. Self-weight settlements of well compacted OPSS granular materials (compacted to minimum 98% of the materials' SPMDD as per OPSS 501) will be significantly lower than those of cohesive fills, with majority of the self-weight settlement of OPSS granular materials being completed during the embankment construction.

8.9.3 Embankment Settlement Mitigation Measures

Unless preloading or light weight fill is used in the area of the structure foundations, the structure foundations would undergo additional settlements to those previously discussed, due to the weight of the embankment fill, for both the shallow foundation and the deep foundation options.

Embankment settlement estimates presented in the preceding section indicate that about 100 mm of post construction settlement is anticipated under the maximum proposed embankment height of 10 m. Although the site's cohesive soils are very stiff to hard and over-consolidated, significant stresses induced by the proposed extremely wide embankment penetrate deeply due to its geometry (about 185 m wide and 10 m high embankment, with 2H:1V side slopes).

Even though all above settlement assessments are based on the site-specific subsurface information, geotechnical laboratory and field test results, some degree of uncertainty remains in geotechnical settlement assessment. Therefore, a settlement monitoring program should be developed and implemented to adjust the actual bridge foundation and embankment construction schedule as necessary. Based on the MTO GEOCRES information, up to 4 inches of post construction settlements were observed under approximately 6-10 m high embankments in the similar subsurface conditions. However, no settlement timeline is provided in the available MTO GEOCRES information. Potential post construction settlement mitigation measures have been assessed considering site-specific constraints and are summarized below:

PRELOADING (PLUS POSSIBLE SURCHARGING)

Based on the latest GA drawing, a single rigid frame structure is proposed over the existing single track CN ROW. The existing CN ROW is about 27 m wide and the proposed rigid frame structure abutment will be placed immediately behind the CN ROW boundaries. The full height preloading should extend at least 2 m beyond the foundation or pile cap footprints to minimize downdrag and differential settlement potentials.

Due to the limited space available between the proposed structure abutments and the CN track, regular 2H:1V or slightly steeper preloading fore-slopes to meet the CHBDC temporary embankment slope requirement cannot be accommodated. Considerations should be given to the use of gabion basket or precast concrete block slope facing with geosynthetic reinforcements to accommodate the required preloading. Surcharging in addition to the preloading could also be considered to expedite the subgrade settlements; but due to the above-mentioned space limitation, surcharging will be more challenging than preloading.

As mentioned in the preceding section, about 4-month waiting period will be required for the preloading to achieve about 95% consolidation settlement. A settlement monitoring program (including strategically placed settlement plates, vibrating wire piezometers and other relevant monitoring instrumentations) should be developed and carried out to verify the efficiency of the preloading and adjust the preloading schedule as necessary. After the 4-month waiting period, the preloading should be partially removed to allow for further structure and foundation construction. The approach embankment will be re-instated as per OPSSs 206 and 501 and the newly built rigid frame structure should be backfilled as per OPSD 3101.150.

Preloading, preloading with surcharging and all other constructions (excavation and backfill) within the CN ROW should be discussed with and approved by CN.

If preloading and/or preloading with surcharging is acceptable to the CN, a shallow foundation is the preferred foundation option based on the construction cost and schedule.

For this preloading (and possible surcharge) option, it is our understanding that EPS would also be required to reduce the lateral earth pressure acting on the rigid frame. A certain portion of structural backfill could be replaced with EPS in that perspective, but the vertical structural loading acting on the foundation would remain the same. If this option is selected, the thickness and extent of the EPS replacement within the fill would be designed in conjunction with the structural engineer to ensure that the horizontal thrust is appropriate reduced.

Given the project schedule and the constraints with the CN ROW discussed above, this option may not be feasible.

LIGHT WEIGHT FILL

If preloading and other construction activities are not allowed within the CN ROW, 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 may be also worthwhile to consider light weight foam (cellular) 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 rigid frame structure. Considerations can also be given to the use of relatively wide strip footing to support the overhead structure, but that foundation construction will likely be extended into the CN ROW.

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 pile cap arrangement, required foundation and overhead 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 or 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 (cellular) concrete
  - Flowable light-weight foam concrete will be poured in stages using form work to build the embankment between bridge abutment and 2H:1V earth embankment foreslope
  - The highway pavement structure and/or earth cover will be placed on the cured light weight foam concrete,

The use of light weight fill to restrict settlements at the foundations would also alleviate the horizontal earth pressure thrusts on the retaining walls, as well as the abutments.

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 using light-weight fill is recommended due to the project schedule and the physical site constraints.

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 significantly lower than regular soil backfill. At-rest earth pressure coefficient of backfill material can be estimated using the backfill material's Poisson's ratio, ν, by using following equation, $K_o = \nu / (1 - \nu)$. The typical Poisson's 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.

LONGER MULTIPLE SPAN BRIDGE STRUCTURE

Considerations can also be given to the longer multiple span structure over the CN ROW with typical 2H:1V embankment fore-slopes and side slopes. Higher approach embankments and additional foundation elements (i.e. piers) within the CN ROW (with a train crash protection as per AREMA) are anticipated. This option is beyond the typical foundation engineering work scope. It is our understanding that this option is not feasible due to the anticipated higher embankments and the additional foundation elements, which are not acceptable within the CN ROW.

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 the settlement (95% of the 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 CONCRETE RETAINING WALLS

Due to the aforementioned space limitation, concrete retaining walls are proposed at both ends of the rigid frame structure to mainly retain the embankment side slopes. No wall details were available at the time of preparing this report.

The proposed walls can be supported on driven steel pipe piles (Section 8.7.1) and/or shallow foundations (Section 8.7.2) depending on the wall loading conditions and the selected embankment settlement mitigation measure. If required, shorter piles can also be considered to support the proposed retaining walls. Sliding resistance between the wall base and founding subgrade can also be found in Section 8.7.2. as well. Wall base sliding can also be resisted by pile foundations or battered piles.

Earth pressures acting on the retaining walls can be assessed as per Section 8.8 of this report. The proposed walls should be backfilled with free draining granular backfill material and proper drainage system should also be provided to minimize possible hydro-static pressure build up behind the wall. Minimal earth pressures are expected if the embankment is fully built with EPS or light weight foam concrete.

Once the retaining wall details are available, a global stability of the wall can be assessed.

8.11 CEMENT TYPE AND CORROSION POTENTIAL

Three samples of the soil 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 5.3.

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 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 CNR-OH2 and CNR-OH4 (206 ppm and 277 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 1000μg/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

Given that the proposed overhead structure and embankment will be constructed within the wood lot (will be cleared for the proposed Highway 3 Talbotville Bypass construction) and farm fields, no construction staging and detour are anticipated.

9.2 EXCAVATION AND BACKFILLING (STRUCTURE AND EMBANKMENT)

Excavation and backfill for the new overhead 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:

Existing Fills (Silty Clayey Sand)	Type 3 Soil (above GWT) Type 4 Soil (below GWT)
Clayey Silt Till (Stiff to hard)	Type 3 Soil
Silty Clay (Firm to stiff)	Type 3 Soil

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, retaining wall and embankment footprints. 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, retaining walls and embankment.

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 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.

9.4 DEEP FOUNDATIONS

As pointed out in the preceding sections, cobbles and boulders should be expected in the soils at the site which may impede pile driving.

Deep foundation should be installed and monitored in accordance with OPSS.PROV 903 and pipe pile tip should be reinforced using APF open end cutting shoe 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.

9.5 UNWATERING/DEWATERING (GROUNDWATER CONTROL)

Based on the groundwater level recorded in the monitoring well in Borehole CNR-OH1, the groundwater level is expected at approximate elevation 240 m. The elevation of the anticipated bottom of excavation for pile caps or shallow foundations is approximately 238.8 m. In this respect, the excavations for the pile caps or shallow foundation are anticipated to be below the groundwater table.

Due to the anticipated low permeability of cohesive fill and native soils at the site, 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.

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 the subsurface deposits, it is expected that construction dewatering permitting will involve EASR registration, as dewatering volumes are unlikely to exceed 400,000 L/day. 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.

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 3001.100	Foundation, Piles, Steel Tube 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 902	Construction Specification for Excavation and Backfilling – Structures
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 Excavating and Backfilling-Structures
OPSS.PROV 903	Construction Specification for Deep Foundations
OPSS.PROV 1010	Material Specification for Aggregates

11.0 MISCELLANEOUS

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

The wood lot was cleared for the foundations investigation by the Ministry of Transportation Ontario (MTO).

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.

The CPT and MASW tests 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.

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.

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17713504
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Raymond Haché, M.Sc., P.Eng.
Designated Principal MTO Foundation Contact

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13.0 REFERENCES

ASTM. 1999. Standard Test Methods for Penetration Test and Split-Barrel Sampling of Soils (ASTM D1586). ASTM International, West Conshohocken, PA.

ASTM. 2000. Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System) (ASTM D2487). ASTM International, West Conshohocken, PA.

CFEM. 2006. Canadian Foundation Engineering Manual, Fourth Edition. Canadian Geotechnical Society, 888 p.

CFEM. 2023. Canadian Foundation Engineering Manual, Fifth Edition. Canadian Geotechnical Society

Chapman, L.J., and Putnam, D.F. 1984. The physiography of Southern Ontario, Ontario Geological Survey Special Volume 2. Ontario Research Foundation, Toronto, Ontario.

CHBDC. 2019. Canadian Highway Bridge Design Code. Canadian Standards Association, Mississauga, Ontario.

Dreimanis, A., 1963: Pleistocene Geology of the St. Thomas Area (East Half), Southern Ontario. Ontario Department of Mines, Preliminary Geological Map 238, scale 1:50,000.

Ensoft. 2019. A Program for the Analysis of the Axial Capacity of Driven Piles. APILE Plus Version 9.4. Ensoft, Inc., Austin, Texas.

Ensoft. 2019. User's Manual for Computer Program LPILE Plus Version 2019. Ensoft, Inc., Austin, Texas.

GEO-SLOPE International Ltd. 2021. Stability Modeling with SLOPE/W 2021. Calgary, AB.

Natural Resources Canada (NRCAN). 2020 National Building Code of Canada Seismic Hazard Calculator, 2020 National Building Code of Canada Seismic Hazard Tool (nrcan.gc.ca)

OHSA. 2015. Occupational Health and Safety Act Regulations for Construction Projects. Carswell, Toronto Ontario.

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). 1971. Foundation Investigation Report for Proposed Crossing at CNR Spur Overheads and St. Thomas Expressway City of St. Thomas, Co. of Elgin, District No. 2 (London), Site 5-212. Geocres No. 40I14-70.

Ontario Ministry of Transportation (MTO). 2022 Guideline for Foundation Engineering Services Version 3.0.

APPENDIX A

A.1 DRAWING NOS. 1 TO 4 – BOREHOLE LOCATION PLAN AND SOIL STRATA PLOTS

A.2 GENERAL ARRANGEMENT DRAWING

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

PLATE No
CONT
WP 3042-22-00

CNR TALBOTVILE OVERHEAD BRIDGE
BOREHOLE LOCATIONS & SOIL STRATA

SHEET
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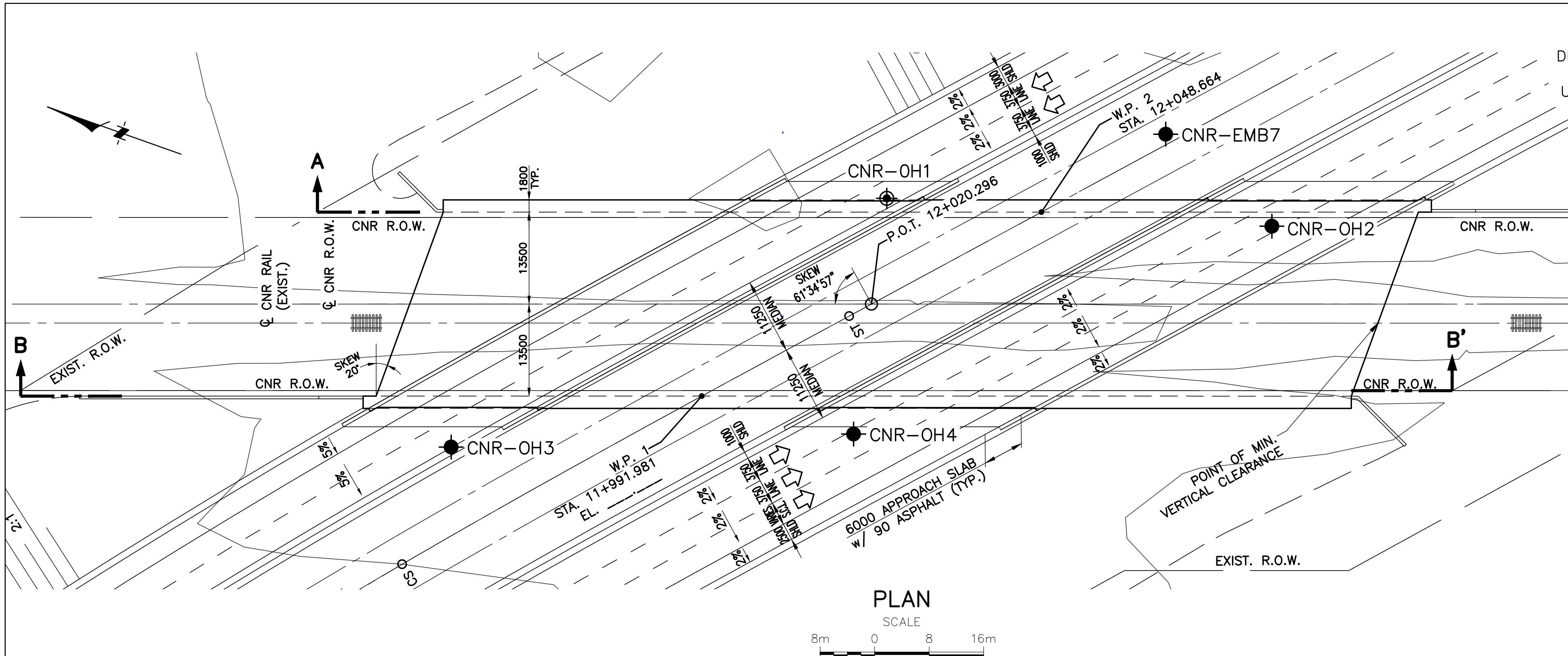
Stantec



PLAN

SCALE
8m 0 8 16m

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PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
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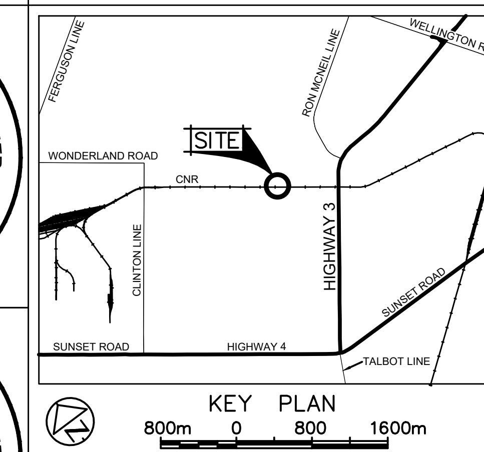
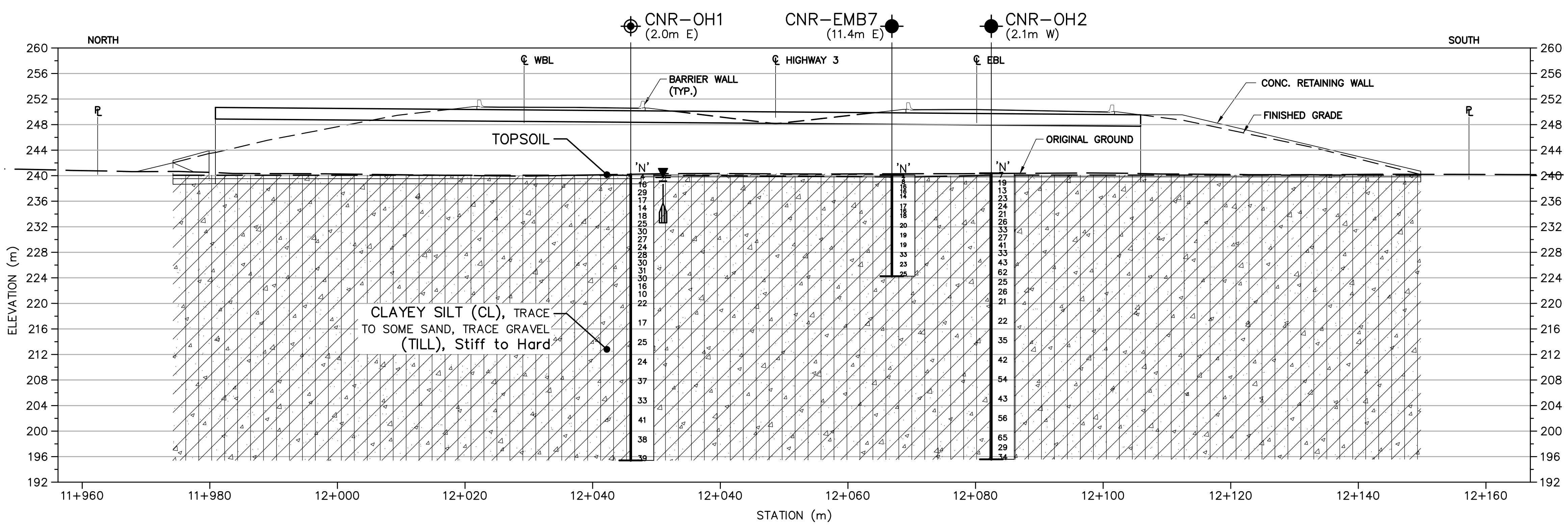
2025-03-31
PROVINCE OF ONTARIO



KEY PLAN
800m 0 800 1600m

LEGEND

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



CROSS SECTION A–A'

SCALE
8m 0 8 16m

No	ELEV	MTM ZONE 11 COORDINATES NORTH	EAST
CNR–EMB7	240.2	4 742 326.5	408 283.1
CNR–OH1	240.4	4 742 271.9	408 298.2
CNR–OH2	240.7	4 742 374.5	408 227.3
CNR–OH3	240.5	4 742 319.5	408 248.9
CNR–OH4	240.1	4 742 291.1	408 305.7

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

GEOCRES No 40I14–224

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METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

PLATE No

CONT
WP 3042–22–00

CNR TALBOTVILE
OVERHEAD BRIDGE
BORHOLE LOCATIONS & SOIL STRATA

SHEET
–

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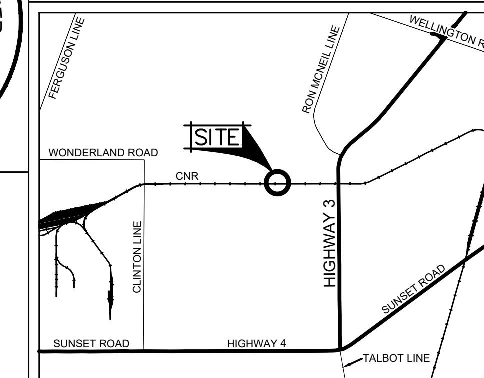
LICENSED PROFESSIONAL ENGINEER
G. H. ROH
100148471
2025-03-31
PROVINCE OF ONTARIO

Stantec



KEY PLAN

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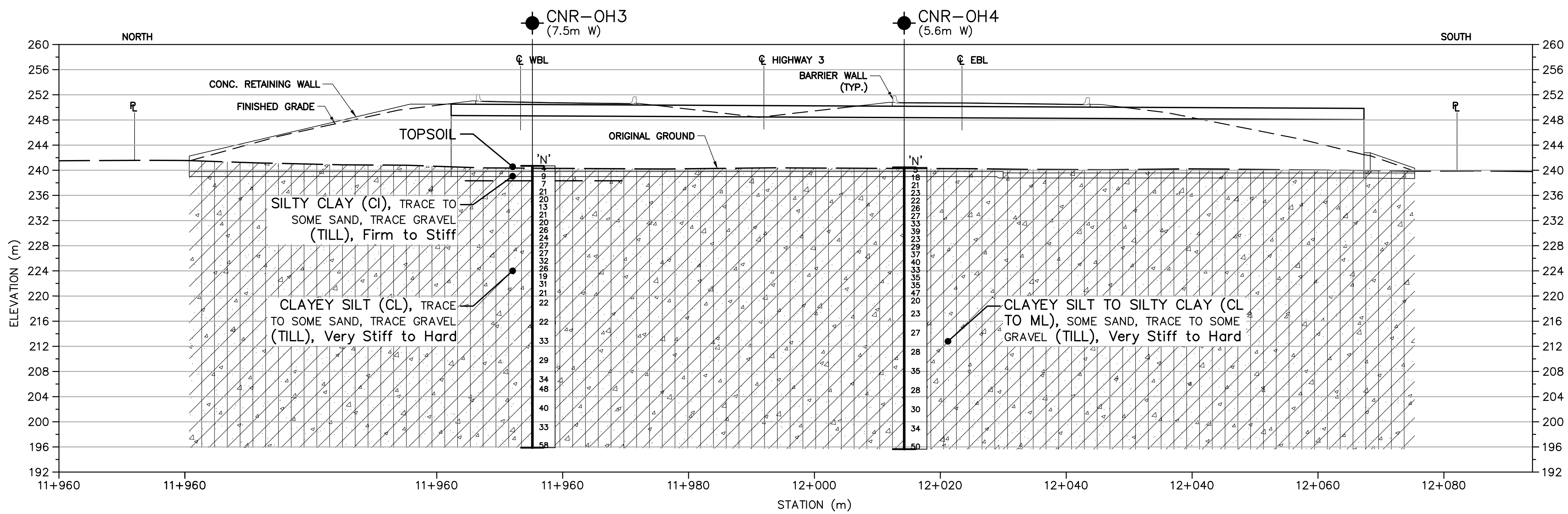

CROSS SECTION B–B'

SCALE

8m 0 8 16m

LEGEND

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

No	ELEV	MTM ZONE 11 COORDINATES NORTH	EAST
CNR–EMB7	240.2	4 742 326.5	408 283.1
CNR–OH1	240.4	4 742 271.9	408 298.2
CNR–OH2	240.7	4 742 374.5	408 227.3
CNR–OH3	240.5	4 742 319.5	408 248.9
CNR–OH4	240.1	4 742 291.1	408 305.7

NOTES

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REVISIONS DATE	BY	DESCRIPTION

GEOCRES No 40I14–224

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PLATE No

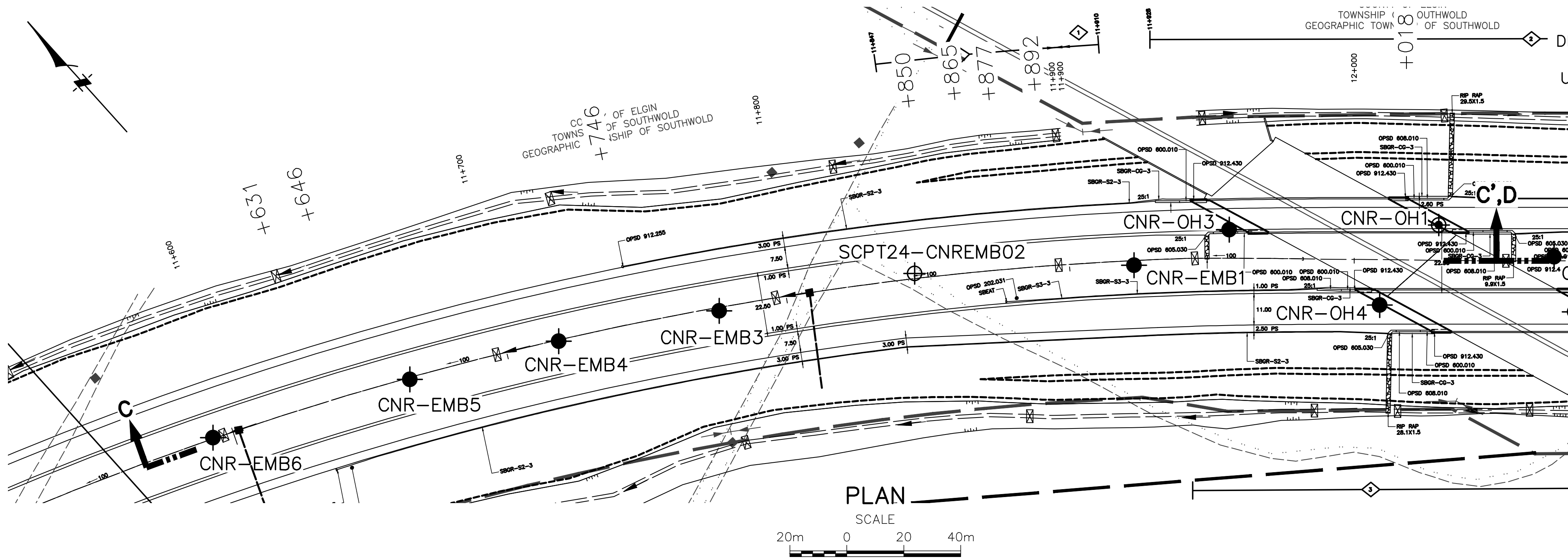
CONT WP 3042-22-00

CNR TALBOTVILE OVERHEAD BRIDGE
BOREHOLE LOCATIONS & SOIL STRATA

SHEET -

Stantec

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



PLAN
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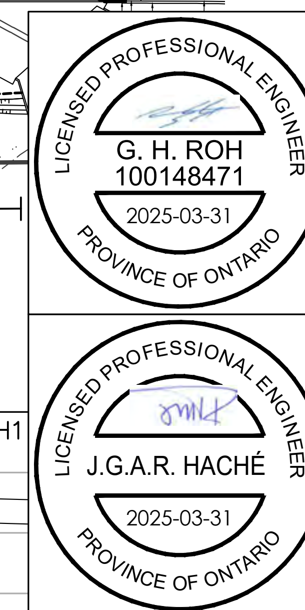
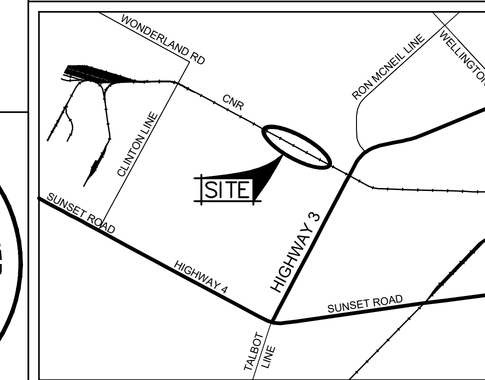
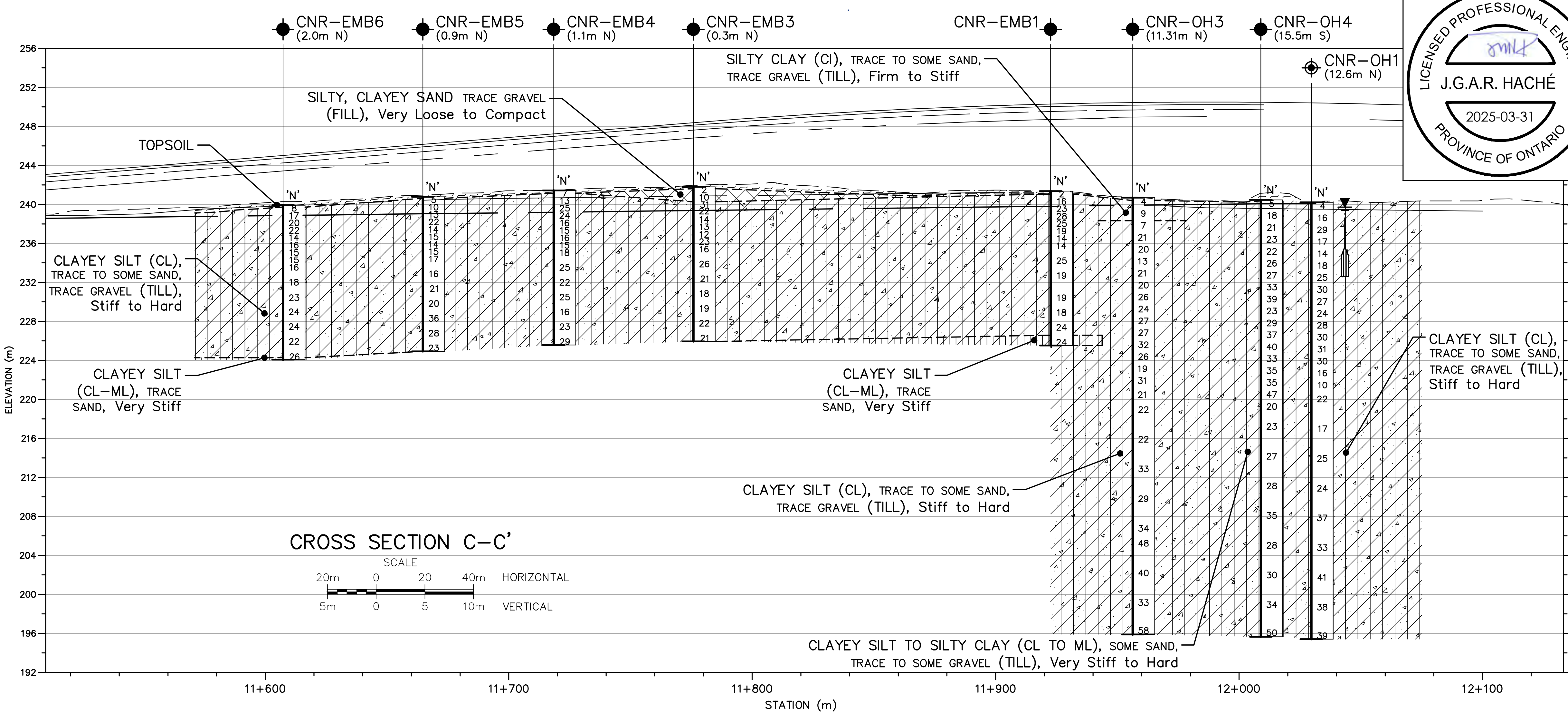




KEY PLAN
800m 0 800 1600m



CROSS SECTION C-C'
SCALE
20m 0 20 40m HORIZONTAL
5m 0 5 10m VERTICAL

LEGEND

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

No	ELEV	MTM ZONE 11 COORDINATES NORTH	EAST
CNR-OH1	240.2	4 742 326.5	408 283.1
CNR-OH2	240.4	4 742 271.9	408 298.2
CNR-OH3	240.7	4 742 374.5	408 227.3
CNR-OH4	240.5	4 742 319.5	408 248.9
CNR-EMB1	240.1	4 742 387.6	408 194.1
CNR-EMB3	241.8	4 742 473.2	408 075.1
CNR-EMB4	241.4	4 742 503.0	408 026.1
CNR-EMB5	240.8	4 742 528.0	407 978.3
CNR-EMB6	239.9	4 742 559.1	407 913.2
CNR-EMB7	240.1	4 742 291.1	408 305.7
CNR-EMB9	239.7	4 742 180.1	408 425.8
CNR-EMB11	239.2	4 742 091.9	408 523.1
CNR-EMB12	239.1	4 742 048.1	408 572.4
SCPT24-CNREMB02		4 742 437.0	408 134.9
SCPT24-CNREMB08		4 742 229.9	408 360.7
SCPT24-CNREMB10		4 742 135.9	408 477.0

NOTES

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

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REVISIONS DATE	BY	DESCRIPTION

GEOCRES No 40I14-224

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

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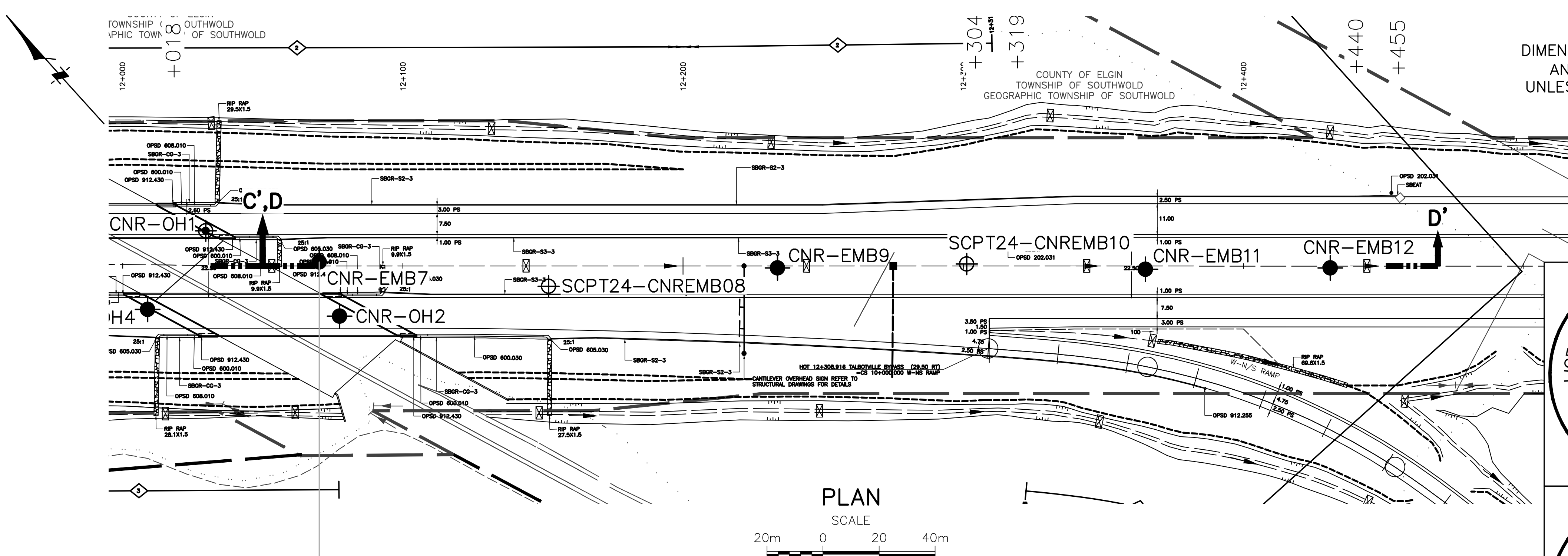
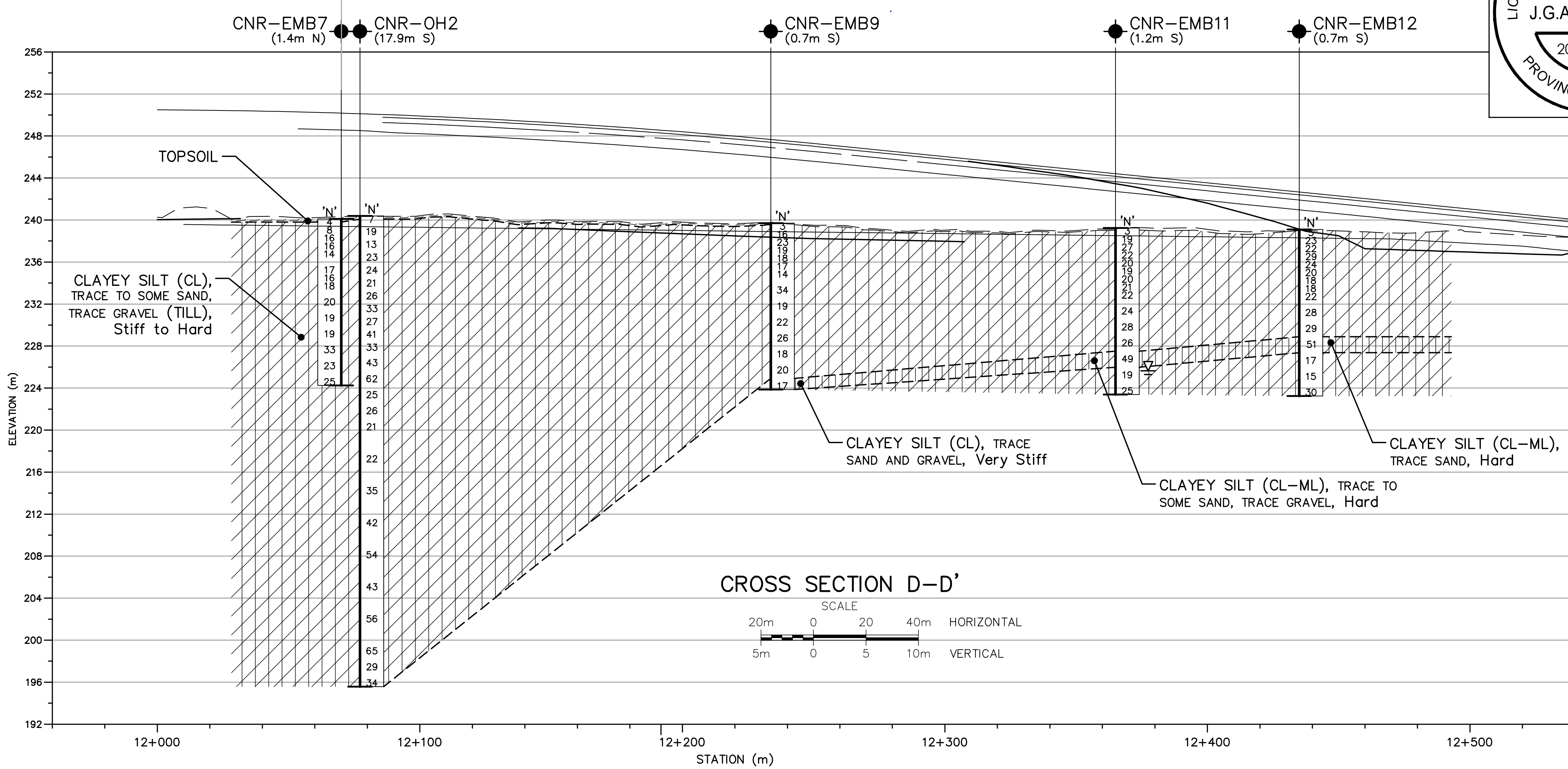


PLAN

SCALE 20m 0 20 40m



CROSS SECTION D-D'

SCALE 20m 0 20 40m HORIZONTAL

5m 0 5 10m VERTICAL

PLATE No

CONT WP 3042-22-00

CNR TALBOTVILE OVERHEAD BRIDGE
BOREHOLE LOCATIONS & SOIL STRATA

SHEET -

Stantec

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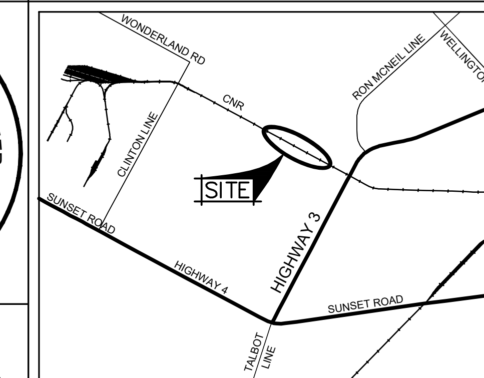
PROVINCE OF ONTARIO

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



KEY PLAN

800m 0 800 1600m

LEGEND

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

No	ELEV	MTM ZONE 11 COORDINATES NORTH	EAST
CNR-OH1	240.2	4 742 326.5	408 283.1
CNR-OH2	240.4	4 742 271.9	408 298.2
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CNR-EMB12	239.1	4 742 048.1	408 572.4
SCPT24-CNREMB02		4 742 437.0	408 134.9
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REVISIONS	DATE	BY	DESCRIPTION

GEOCRES No 40I14-224

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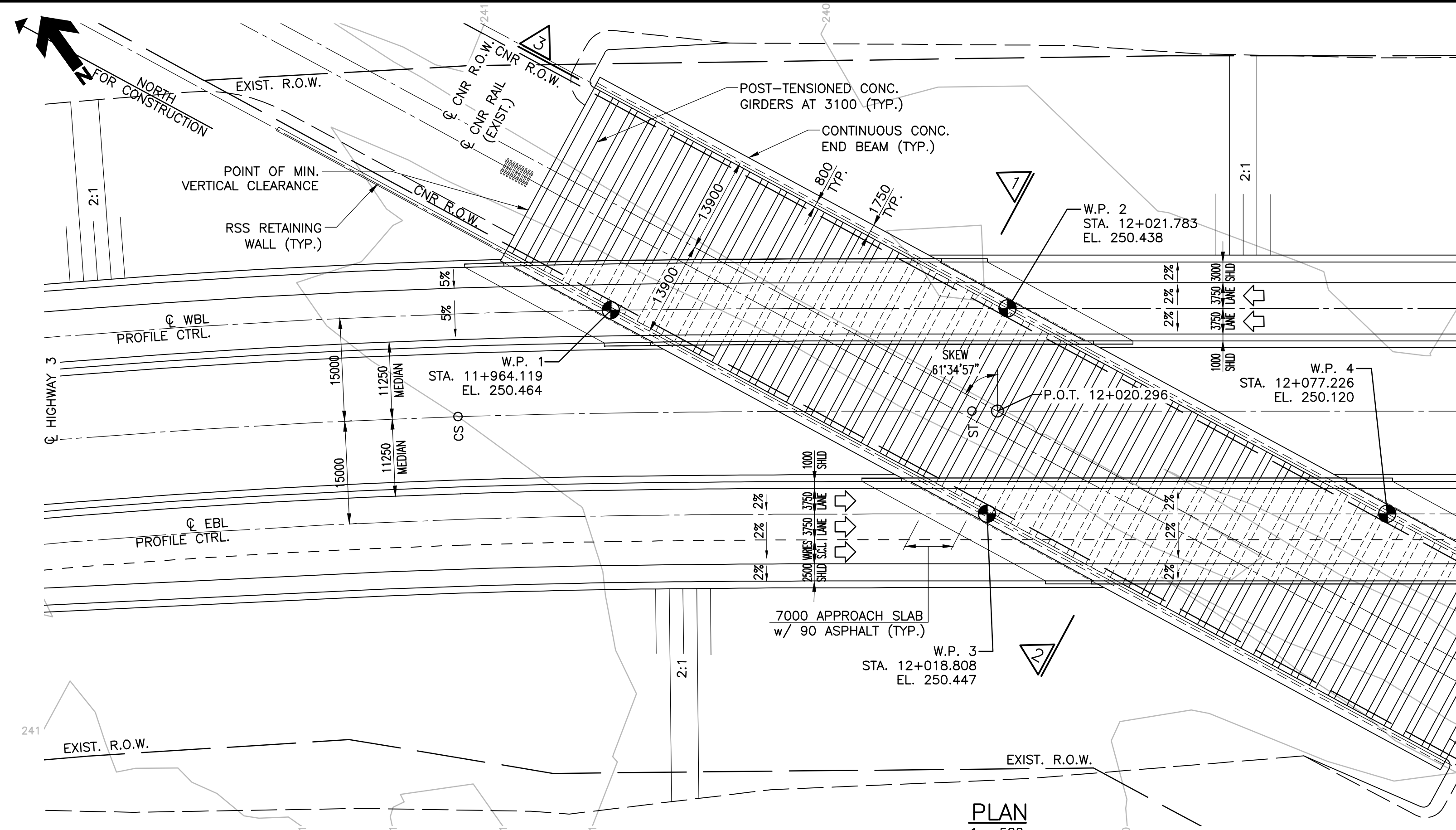
MINISTRY OF TRANSPORTATION, ONTARIO

PH-D-707 88-05

HWY 3	
CONT 2025-3007	
WP 3074-24-01	
CNR TALBOTVILLE OVERHEAD	SHEET
GENERAL ARRANGEMENT	395

Stantec

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



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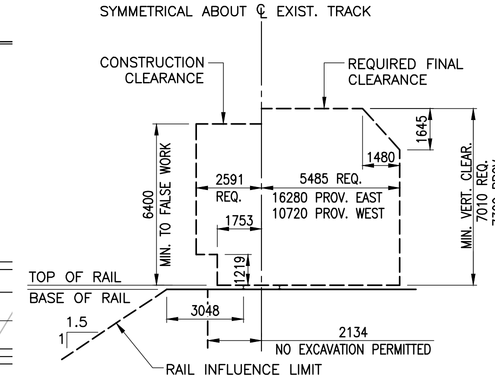


CNR CLEARANCE & ZONE OF INFLUENCE DIAGRAM
N.T.S.

LIST OF DRAWINGS:

LEGEND

W.P.	– DENOTES WORKING POINT
P.O.T	– DENOTES POINT ON TANGENT
B/ABUT.	– DENOTES BOTTOM OF ABUTMENT
T/A	– DENOTES TOP OF ASPHALT
CS	– DENOTES CURVE TO SPIRAL
ST	– DENOTES SPIRAL TO TANGENT
R.O.W.	– DENOTES RIGHT-OF-WAY
REQ.	– DENOTES REQUIRED
PROV.	– DENOTES PROVIDED
CLEAR.	– DENOTES CLEARANCE
P.T.	– DENOTES POST-TENSIONED
CTRL.	– DENOTES CONTROL

GENERAL NOTES

1. SPECIFIED 28-DAY CONCRETE COMPRESSIVE STRENGTH:

POST-TENSIONED GIRDERS	35MPa
DECK	35MPa
MASS CONCRETE	20MPa
REMAINDER	30MPa

2. CLEAR COVER TO REINFORCING STEEL:

FOOTING		100±25
DECK	TOP	70±20
	BOTTOM	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.

GLASS FIBRE REINFORCED POLYMER REINFORCING BARS SHALL BE GRADE III AS SPECIFIED IN THE CONTRACT DRAWINGS. THE NOMINAL DIAMETER, TENSILE MODULUS OF ELASTICITY AND GUARANTEED MINIMUM TENSILE STRENGTH SHALL BE AS SPECIFIED IN THE CONTRACT DOCUMENTS.

BAR MARKS WITH THE PREFIX 'G' DENOTE GRADE III GLASS FIBRE REINFORCED POLYMER 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. HOOKS AND BENDS FOR GFRP BARS SHALL HAVE A MINIMUM BEND-RADIUS-TO-BAR-DIAMETER RATIO (r/d) OF 4.0.

4. EVA FOAM

EVA FOAM SHALL BE GRADE 2A2, UNLESS OTHERWISE NOTED.

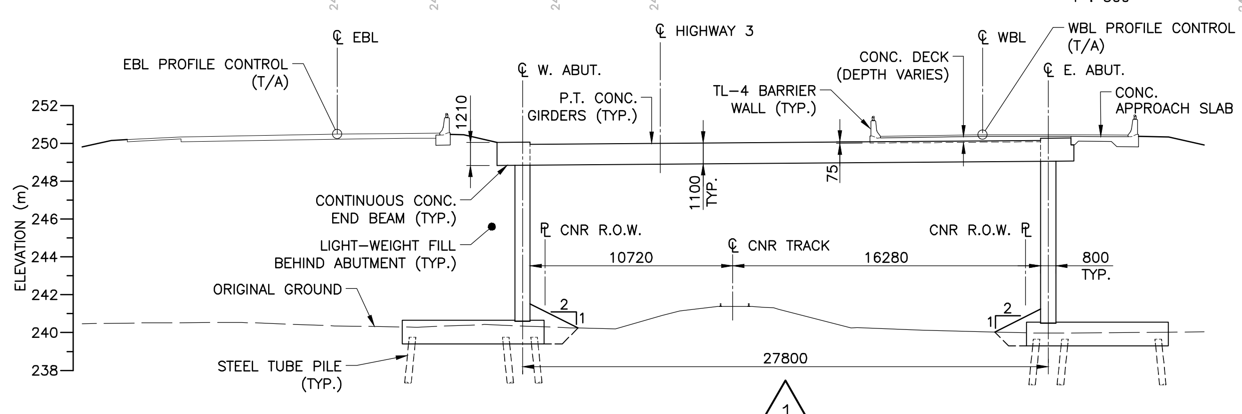
5. CONSTRUCTION

VERTICAL CONSTRUCTION JOINTS SHALL BE PROVIDED ALONG THE ENTIRE LENGTH OF THE ABUTMENT WALLS, SPACED AT A MAXIMUM OF 12m.

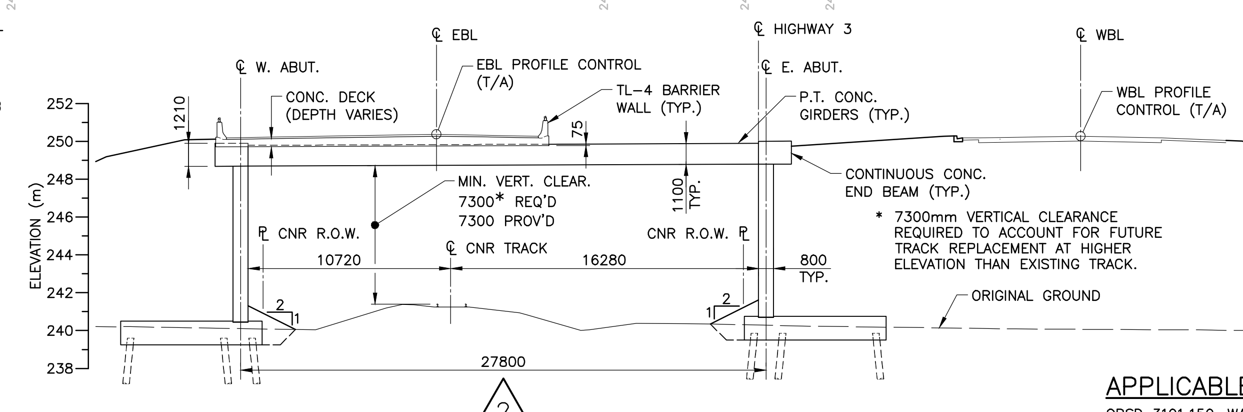
BACKFILL SHALL NOT BE PLACED BEHIND THE ABUTMENTS UNTIL ALL GIRDER POST-TENSIONING IS COMPLETE.

DECK FALSEWORK SHALL NOT BE REMOVED UNTIL ALL GIRDER POST-TENSIONING IS COMPLETE.

BACKFILL SHALL BE PLACED SIMULTANEOUSLY BEHIND BOTH CONCRETE ABUTMENTS KEEPING THE HEIGHT OF THE BACKFILL APPROXIMATELY THE SAME. AT NO TIME SHALL THE DIFFERENCE IN ELEVATION BE GREATER THAN 500mm.



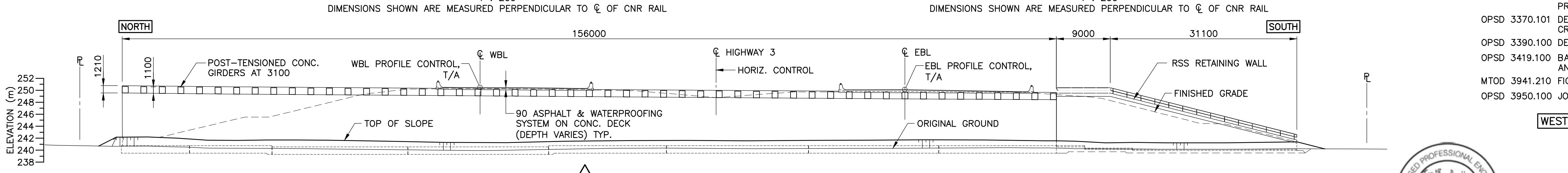
1
1 : 200
DIMENSIONS SHOWN ARE MEASURED PERPENDICULAR TO ℄ OF CNR RAIL



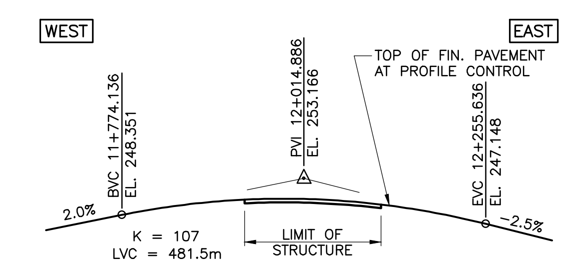
2
1 : 200
DIMENSIONS SHOWN ARE MEASURED PERPENDICULAR TO ℄ OF CNR RAIL

APPLICABLE STANDARD DRAWINGS

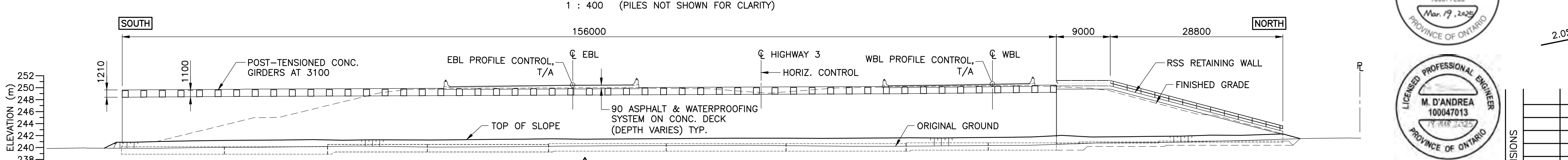
OPSD	3101.150	WALLS, ABUTMENT, BACKFILL, MINIMUM GRANULAR REQUIREMENT
OPSD	3370.100	DECK WATERPROOFING, HOT APPLIED ASPHALT MEMBRANE WITH PROTECTION BOARD
OPSD	3370.101	DECK WATERPROOFING, HOT APPLIED ASPHALT MEMBRANE AT ACTIVE CRACKS GREATER THAN 2mm WIDE AND CONSTRUCTION JOINTS
OPSD	3390.100	DECK, DRIP CHANNEL
OPSD	3419.100	BARRIERS AND RAILINGS, STEEL BEAM, GUIDE RAIL AND CHANNEL ANCHORAGE
MTOD	3941.210	FIGURES IN CONCRETE, SITE NUMBER AND DATE, LAYOUT
OPSD	3950.100	JOINTS, CONCRETE EXPANSION & CONSTRUCTION ON STRUCTURE



3 ELEVATION – EAST ABUTMENT
1 : 400 (PILES NOT SHOWN FOR CLARITY)



PROFILE OF HIGHWAY 3
N.T.S.



4 ELEVATION – WEST ABUTMENT
1 : 400 (PILES NOT SHOWN FOR CLARITY)

LICENSED PROFESSIONAL ENGINEER D. HUANG 100577288 Mar. 19, 2025 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER M. D'ANDREA 100047013 PROVINCE OF ONTARIO

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS

DATE	BY	DESCRIPTION

DESIGN D.H.	CHK M.D.	CODE CSA S6:19	LOAD CL-625-ONT	DATE MAR 2025
DRAWN B.L.H.	CHK D.H.	SITE 05X-0375/B0	STRUCT	SCHEME	DWG. 1

165001308-5-375-01.DWG Mar 19 2025

APPENDIX B

B.1 AVAILABLE GEOCRES INFORMATION

DOCUMENT MICROFILMING IDENTIFICATION

G.I.-30 SEPT. 1976

GEOCRES No. 40I14-70

DIST. 2 REGION

W.P. No. 89-69-05

CONT. No.

W. O. No.

STR. SITE No. 5-212

HWY. No. 3N

LOCATION PROPOSED CROSSING AT CNR

NO. OF PAGES -

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

FORM OD-MT - 126 (REV 1969) OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 1

FOUNDATION SECTION

JOB 71-11068 LOCATION Co-Ord's 557,826 N. 339,591 E. ORIGINATED BY P.P.
W.P. 89-69-05 & 06 BORING DATE July 22, 1971 COMPILED BY P.P.
DATUM Geodetic BOREHOLE TYPE Continuous Flight Auger CHECKED BY

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100
SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 2000 4000
LIQUID LIMIT w_L PLASTIC LIMIT w_P WATER CONTENT w — WATER CONTENT % 10 20 30

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS / FOOT	ELEV SCALE	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
788.1	Ground level.						
0.0	Clayey silt, some sand, trace of gravel.	1	SS	29			
		2	TW	PH	780	134	1 13 49 3
	Very stiff to hard.	3	SS	27			
		4	SS	70/6"	770		
		5	SS	34			
		6	TW	PH	760	140	
		7	SS	34			
		8	TW	PH	750	142	
		9	SS	40			
		10	TW	PH	740	136.5	
		11	SS	22			
					730		
		12	TW	PH		133	3 9 43 45
					720		
		13	SS	29			▼ 715.6'
					710		
		14	TW	PH		131	
					700		
		14A	SS	33			
689.1		15	TW	PH	690		
99.0	End of borehole.						

End of cone test.

20 / 15 5 / 10 % STRAIN AT FAILURE

DEPARTMENT OF HIGHWAYS– ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 2

FOUNDATION SECTION

JOB 71-11068 LOCATION Co-Ord's 557,929 N. 339,505 E. ORIGINATED BY P.P.
W.P. 89-69-05 & 06 BORING DATE July 23, 1971 COMPILED BY H.S.
DATUM Geodetic BOREHOLE TYPE Continuous Flight Auger CHECKED BY

SOIL PROFILE			SAMPLES							
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FOOT 20 40 60 80 100; SHEAR STRENGTH P.S.F. ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE 2000 4000	LIQUID LIMIT w_L PLASTIC LIMIT w_P WATER CONTENT w; WATER CONTENT % 10 20 30	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
787.0	Ground level.									
	Clayey silt, some sand, trace of gravel.		1	SS	25			○		
			2	TW	PH	780	>●	○——	134.5	
	Very stiff to hard.		3	SS	20		End of cone test.	○		
			4	TW	PH	770	>○	○	138	3 7 53 37
			5	SS	41			○		
			6	SS	65/6"	760		○		
			7	TW	PH		>●	○	140	▼755.5
748.0			8	SS	52	750		○		
39.0	End of borehole.									
						740				

20
15 ⊹ 5 % STRAIN AT FAILURE
10

FORM OD-MT-126 (REV 1969) OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 3

FOUNDATION SECTION

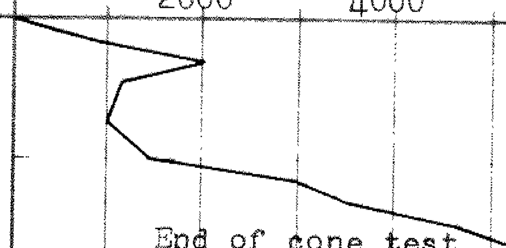
JOB 71-11068 LOCATION Co-Ord's 557,920 N. 339,456 E. ORIGINATED BY P.P.
W.P. 89-69-05 & 06 BORING DATE July 26, 1971 COMPILED BY H.S.
DATUM Geodetic BOREHOLE TYPE Continuous Flight Auger. CHECKED BY

SOIL PROFILE: ELEV. DEPTH / DESCRIPTION / STRAT PLOT — SAMPLES: NUMBER / TYPE / BLOWS/FOOT — ELEV SCALE — DYNAMIC PENETRATION RESISTANCE BLOWS/FOOT 20 40 60 80 100; SHEAR STRENGTH P.S.F. (○ UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE) 2000 4000 — LIQUID LIMIT w_L, PLASTIC LIMIT w_P, WATER CONTENT w; WATER CONTENT % 10 20 30 — BULK DENSITY γ P.C.F. — REMARKS GR SA SI CL

787.0 Ground level.

Clayey silt, some sand, trace of gravel.
Hard.

Number	Type	Blows/Foot	Shear strength	Bulk density γ P.C.F.	Remarks
1	SS	34			
2	TW	PH	>○	135.5	
3	SS	32			
4	TW	PH	>●	140	
5	SS	93/6"			
6	SS	67			
7	TW	PH	>○	138	
8	SS	65			5 12 51 32
9	SS	67			



Water level: 748.5

740.5
46.5 End of borehole.

Elev scale: 780, 770, 760, 750, 740

20
15 -○- 5 % STRAIN AT FAILURE
10

FORM OD-MT-126 (REV 1969)　　OFFICE REPORT ON SOIL EXPLORATION

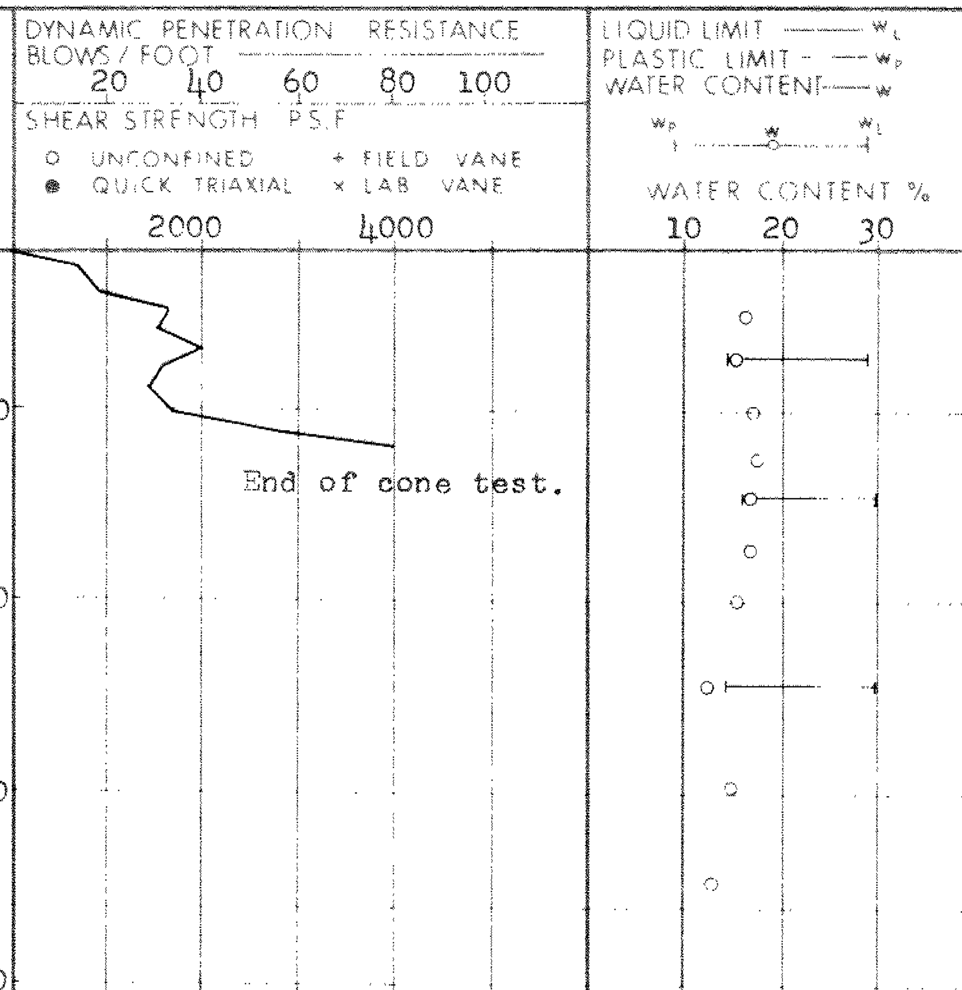
DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 4

FOUNDATION SECTION

JOB 71-11068　LOCATION Co-Ord's 558,019 N. 339,370 E.　ORIGINATED BY P.P.
W.P. 89-69-05 & 06　BORING DATE July 27, 1971　COMPILED BY H.S.
DATUM Geodetic　BOREHOLE TYPE Continuous Flight Auger.　CHECKED BY

ELEV. DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	REMARKS GR.SA.SI.CL.
787.9	Ground level.						
	Clayey silt, some sand, trace of gravel. Very stiff to hard.		1	SS	17		
			2	SS	27		
			3	SS	35	780	
			4	SS	27		
			5	SS	25		
			6	SS	25		
			7	SS	27	770	
			8	SS	33		
			9	SS	24	760	2 14 49 35
							▼ 757.9'
753.9			10	SS	40		
34.0	End of borehole.						
						750	



20
15 -◇- 5 % STRAIN AT FAILURE
10

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 5

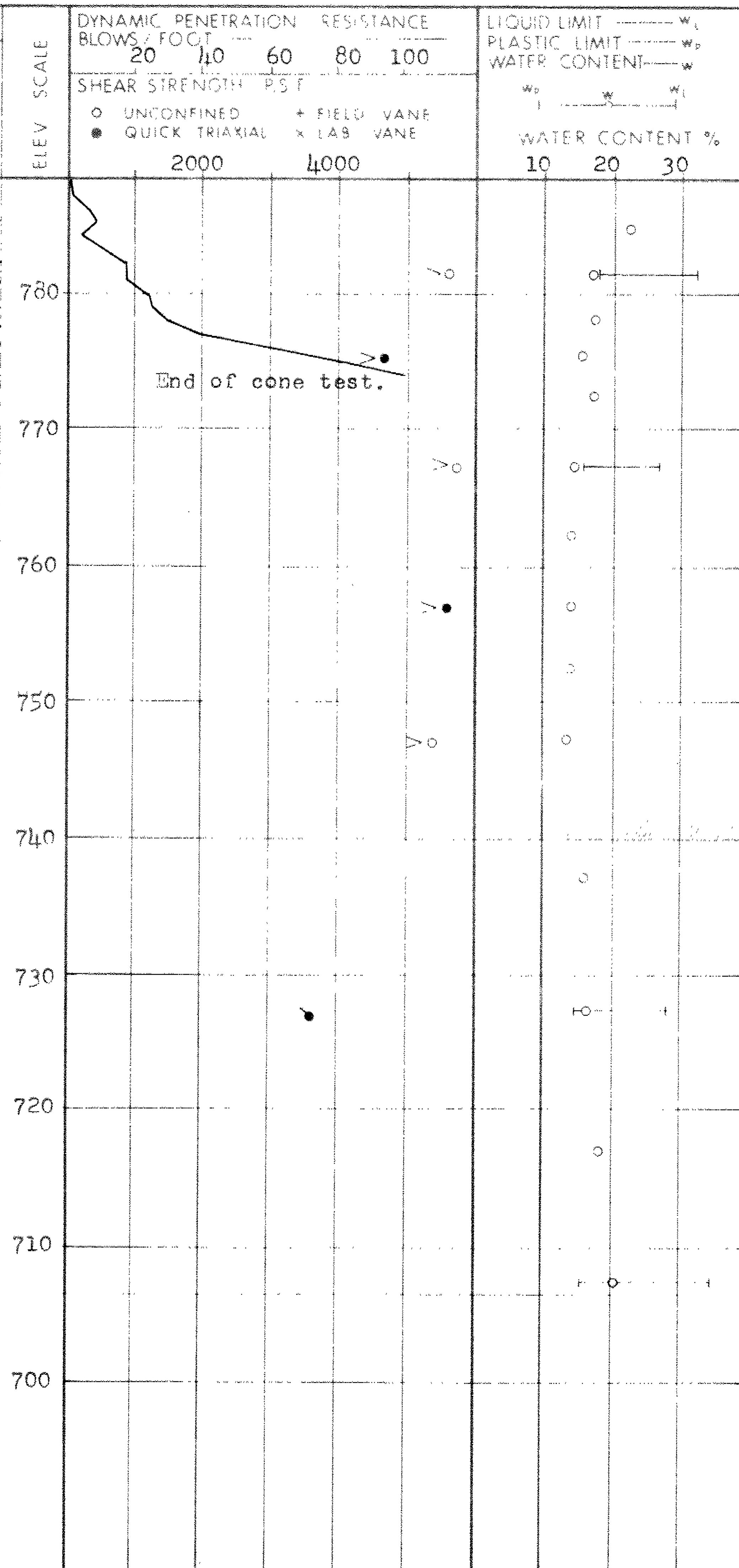
FOUNDATION SECTION

JOB 71-11068 LOCATION Co-Ord's 558,163 N. 339,320 E. ORIGINATED BY P.P.
W.P. 89-69-05 & 06 BORING DATE July 27, 1971 COMPILED BY H.S.
DATUM Geodetic BOREHOLE TYPE Continuous Flight Auger CHECKED BY

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT. PLOT
788.3	Ground level.	
0.0	Clayey silt, some sand, trace of gravel. Very stiff to hard.	
706.8 / 81.5	End of borehole.	

SAMPLE NUMBER	TYPE	BLOWS/FOOT	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
1	SS	16		
2	TW	PH	136	
3	SS	47		
4	TW	PH	135	
5	SS	55		
6	TW	PH	139	
7	SS	55		
8	TW	PH	140	2 13 51 34
9	SS	40		
10	TW	PH		
11	SS	69		
12	TW	PH	133	
13	SS	34		
14	SS	50		0 9 41 50

Water level: 723.3



20
15 ◇ 5 % STRAIN AT FAILURE
10

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 6

FOUNDATION SECTION

JOB 71-11068 LOCATION Co-Ord's 558,074 N. 339,402 E. ORIGINATED BY P.P.
W.P. 89-69-05 & 06 BORING DATE July 26, 1971 COMPILED BY H.S.
DATUM Geodetic BOREHOLE TYPE Continuous Flight Auger. CHECKED BY

ELEV. DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS / FOOT	ELEV SCALE	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
788.4	Ground level.						
0.0	Clayey silt, some sand, trace of gravel. Very stiff to hard.	1	SS	26			
		2	SS	48			
		3	SS	51	780		
		4	TW	PH		136	1 12 49 38
		5	SS	65			
					770		
		6	TW	PH		141	
		7	SS	55			
					760		
		8	TW	PH		140	
		9	SS	56			754.4'
					750		
746.9 / 41.5	End of borehole.						
					740		

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100
SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 2000 4000

End of cone test.

LIQUID LIMIT w_L PLASTIC LIMIT w_P WATER CONTENT w
WATER CONTENT % 10 20 30

20 15 5 10 % STRAIN AT FAILURE

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 7

FOUNDATION SECTION

JOB 71-11068 LOCATION Co-Ord's 558,080 N, 339,453 E. ORIGINATED BY P.P.
W.P. 89-69-05 & 06 BORING DATE July 26, 1971 COMPILED BY H.S.
DATUM Geodetic BOREHOLE TYPE Continuous Flight Auger. CHECKED BY

ELEV. DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	REMARKS (GR. SA. SI. CL.)
787.8	Ground level.					
0.0	Clayey silt, some sand, trace of gravel. Stiff to hard.	1	SS	12		
		2	SS	24		
		3	SS	31	780	
		4	SS	31		
		5	SS	31		
		6	SS	29		
		7	SS	35	770	2 14 51 33
		8	SS	28		
		9	SS	34	760	
		10	SS	25		752.8
748.8		11	SS	27	750	
39.0	End of borehole.					
					740	

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT: 20 40 60 80 100
SHEAR STRENGTH P.S.F.: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB. VANE — 2000 4000

End of cone test.

LIQUID LIMIT w_L, PLASTIC LIMIT w_P, WATER CONTENT w — WATER CONTENT %: 10 20 30

BULK DENSITY γ P.C.F.

20 / 15 5 / 10 % STRAIN AT FAILURE

FORM OD-MT-[illegible]26 (REV. 1969) OFFICE REPORT ON SOIL EXPLORATION

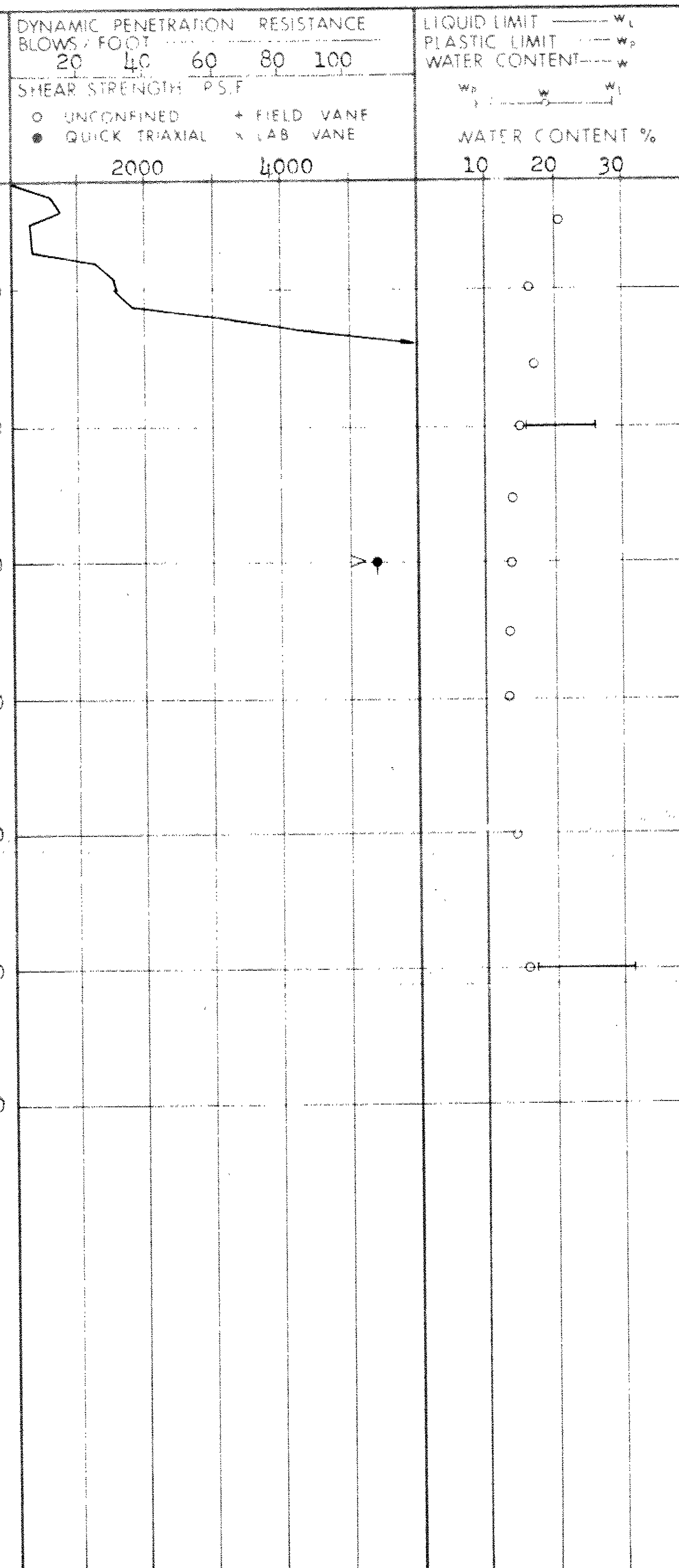
DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 8

FOUNDATION SECTION

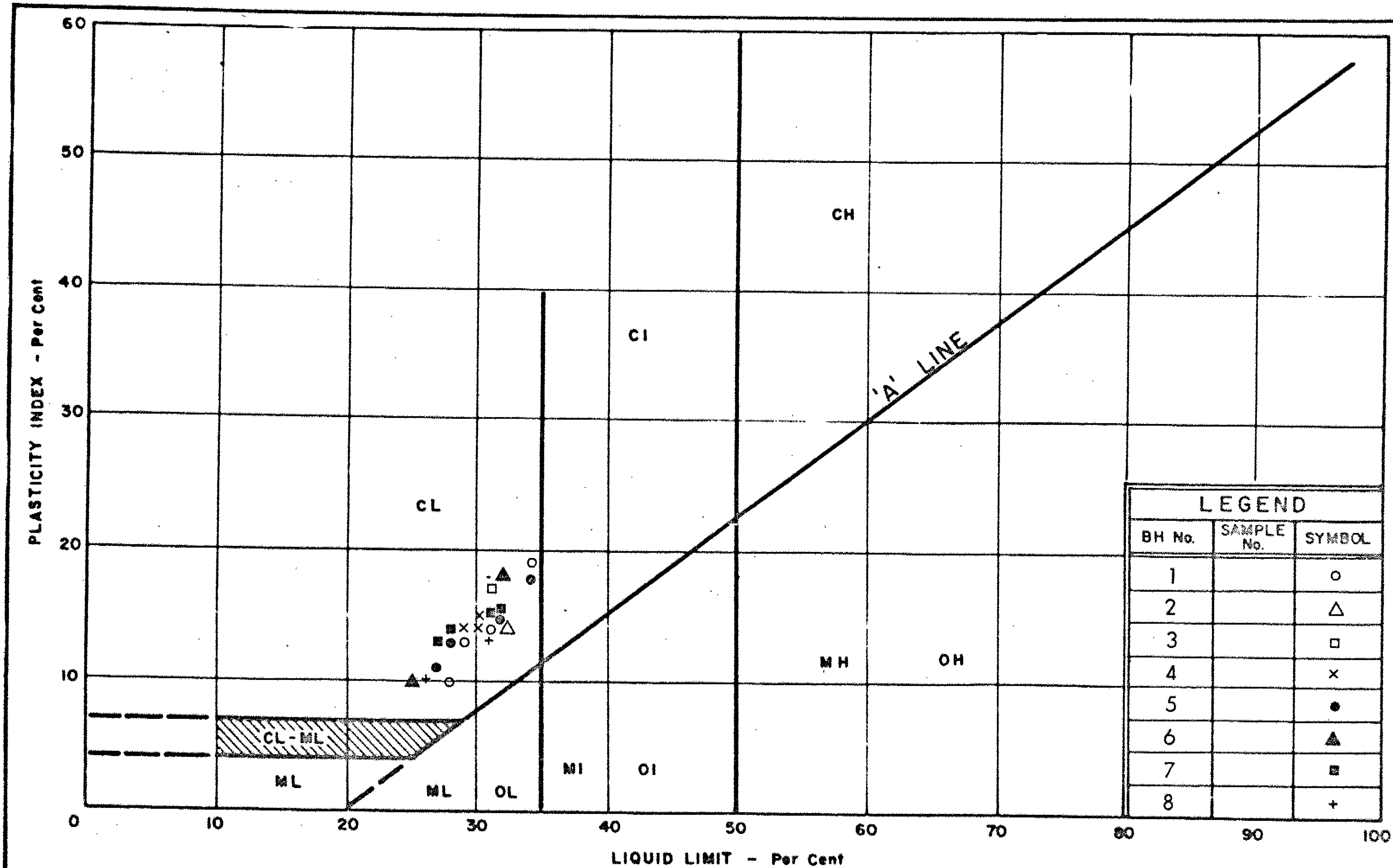
JOB 71-11068 LOCATION Co-ord's 557, 988 N. 339, 33. E. ORIGINATED BY P.P.
W.P. 89-69-05 & 06 BORING DATE July 26, 1971 COMPILED BY P.P.
DATUM Geodetic BOREHOLE TYPE Continuous Flight Auger. CHECKED BY

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS/FOOT	ELEV SCALE	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
788.0	Ground level.						
0.0	Clayey silt, some sand, trace of gravel. Stiff to hard.	1	SS	13			
		2	TW	PH	780		
		3	SS	26			
		4	TW	PH	770		
		5	SS	36			
		6	TW	PH	760	139	3 14 53 30
		7	SS	43			
		8	SS	52	750		
		9	SS	50	740		
729.0		10	SS	35	730		
59.0	End of borehole.						
					720		



20
15 5 % STRAIN AT FAILURE
10

FORM OB-MT-252
REVISED JUL 1966



DEPARTMENT OF HIGHWAYS
MATERIALS and TESTING DIVISION

PLASTICITY CHART

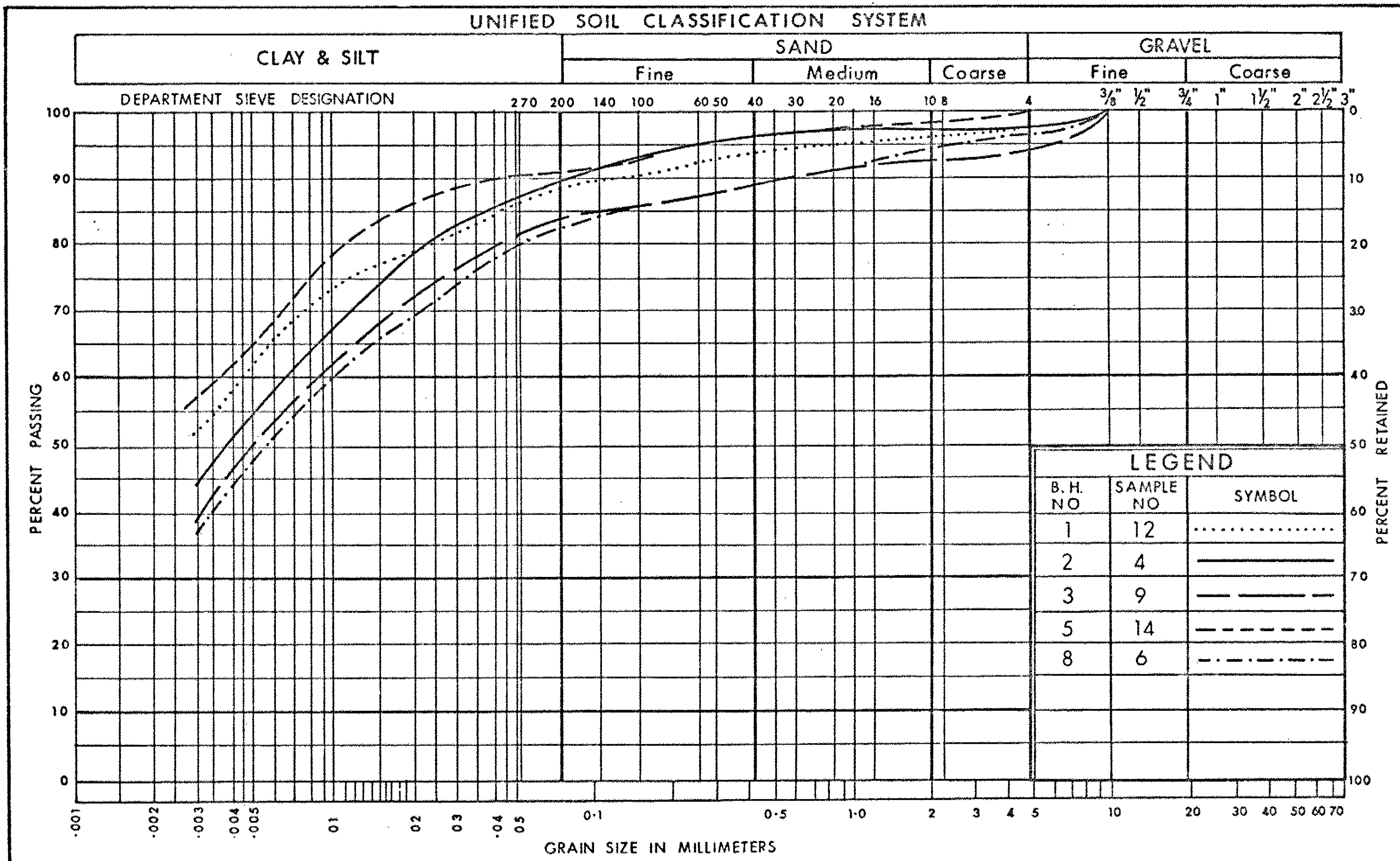
CLAYEY SILT, SOME SAND, TRACE OF GRAVEL

W.P. No. 89-69-05 & 06

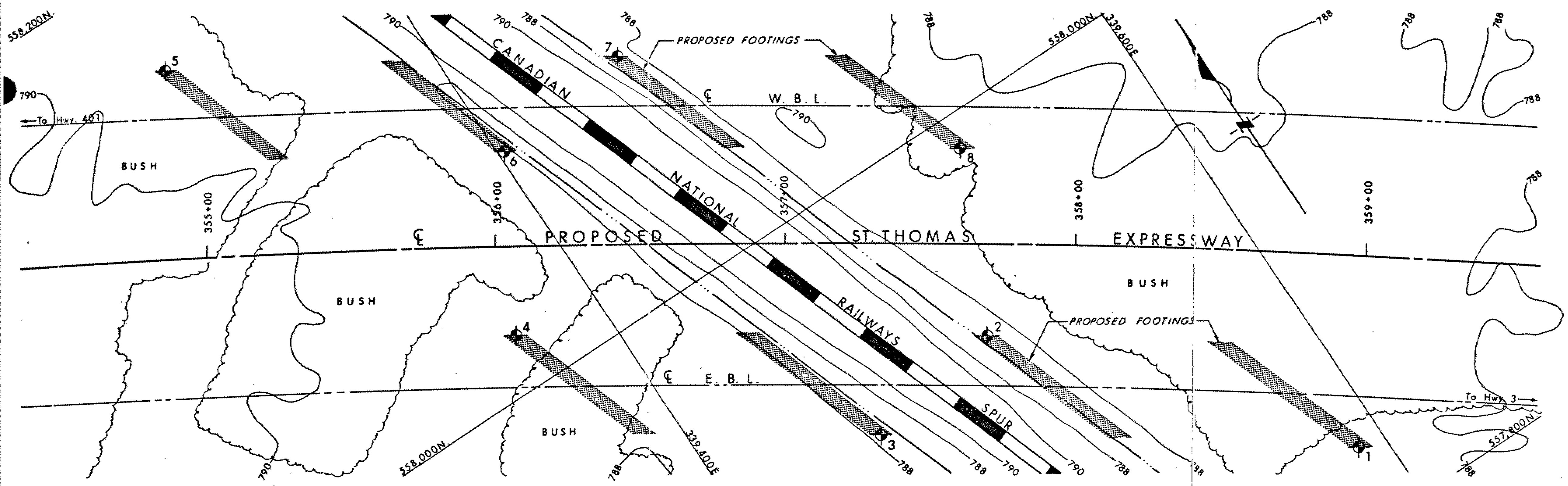
JOB No. 71-11068

FIG. No 1

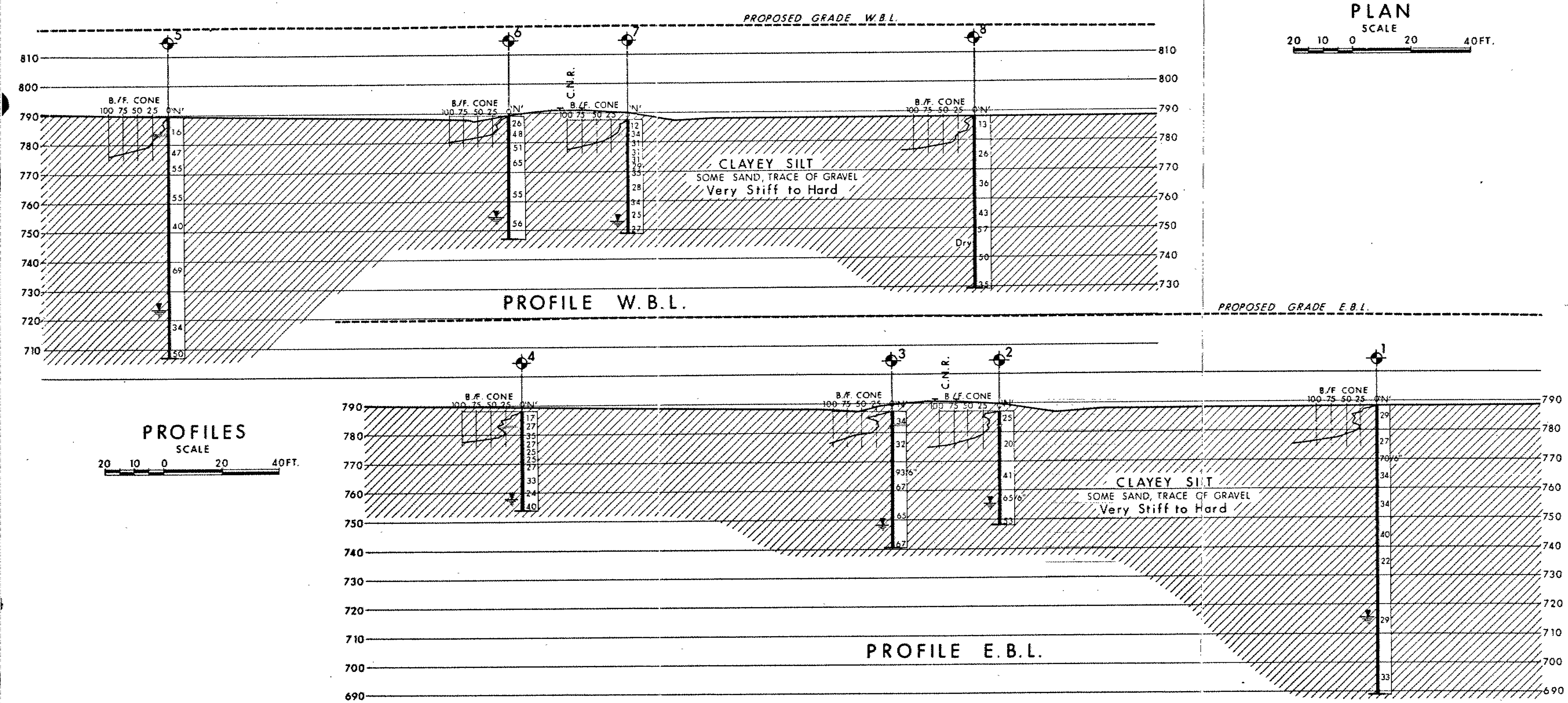
FORM OD-MT-1
REV. AUG. 1971



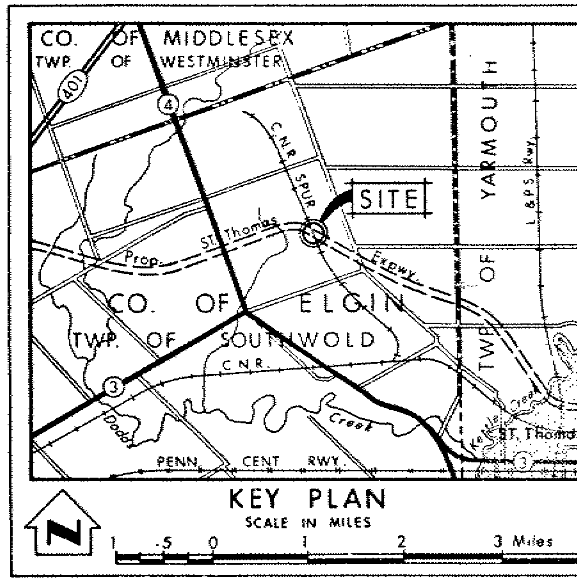
DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS ONTARIO DESIGN SERVICES BRANCH	**GRAIN SIZE DISTRIBUTION** CLAYEY SILT, SOME SAND, TRACE OF GRAVEL	W.P. No. 89-69-05 & 06 JOB No. 71-11068 FIG. № 2



PLAN
SCALE
20 10 0 20 40FT.



PROFILES
SCALE
20 10 0 20 40FT.



KEY PLAN
SCALE IN MILES
1 .5 0 1 2 3 Miles

LEGEND

- Bore Hole
- Cone Penetration Test
- Bore Hole & Cone Test
- Water Levels established at time of field investigation. July 1971

NO.	ELEVATION	CO-ORDINATES NORTH	CO-ORDINATES EAST
1	788·1	557,826	339,591
2	787·0	557,929	339,505
3	787·0	557,920	339,456
4	787·9	558,019	339,370
5	788·3	558,163	339,320
6	788·4	558,074	339,402
7	787·8	558,080	339,453
8	788·0	557,988	339,533

— NOTE —

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.

REVISIONS	DATE	BY	DESCRIPTION

DEPARTMENT OF TRANSPORTATION & COMMUNICATIONS
DESIGN SERVICES BRANCH — FOUNDATION OFFICE

C. N. R. SPUR LINE

HIGHWAY NO. Prop. ST. THOMAS EXPWY. DIST. NO. 2
CO. ELGIN
TWP. SOUTHWOLD LOT 42 & 43 CON. E.S.T.R.

BORE HOLE LOCATIONS & SOIL STRATA

SUBMD. P.P.	CHECKED	W.P. NO 89-69-05&06	DRAWING NO 71-11068A
DRAWN	CHECKED	JOB NO. 71-11068	
DATE Aug. 31, 1971		SITE NO.	BRIDGE DRAWING NO.
APPROVED PRINCIPAL FOUNDATION ENGINEER		CONT. NO.	

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE Nº 11

JOB 71-11068 LOCATION Co-ords. 558,021 N; 339,473 E. ORIGINATED BY LJH

W.P. 89-69-05/06 BORING DATE Nov. 7, 1973 COMPILED BY LJH

DATUM Geodetic BOREHOLE TYPE Cont. Flight Auger CHECKED BY

SOIL PROFILE			SAMPLES							
ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100; SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT —W_L PLASTIC LIMIT —W_P WATER CONTENT —W; W_P W W_L; WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
787.5	Ground Level									
0.0	Clayey silt, some sand, trace of gravel		1	SS	20					
	Very Stiff		2	SS	19	780				
	Hard		3	SS	26					774 ▽
771.0			4	SS	35					
16.5	End of Borehole					770				

20 / 15 ◇ 5 / 10 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

DESIGN SERVICES BRANCH | FOUNDATIONS OFFICE

RECORD OF BOREHOLE Nº 12

JOB 71-11068 | LOCATION Co-ords. 558,098 N; 339,398 E. | ORIGINATED BY LJH
W.P. 89-69-05/06 | BORING DATE Nov. 8, 1973 | COMPILED BY LJH
DATUM Geodetic | BOREHOLE TYPE Cont. Flight Auger | CHECKED BY

ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT; SHEAR STRENGTH P.S.F. (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE)	LIQUID LIMIT W_L, PLASTIC LIMIT W_P, WATER CONTENT W; WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
788.2	Ground Level									
0.0	Clayey silt, some sand traces of gravel. Very Stiff to Hard		1	SS	28					
			2	SS	54	780				
			3	SS	33					
771.7			4	SS	45					Hole Dry
16.5	End of Borehole					770				

OFFICE REPORT SOIL EXPLORATION

20
15 -◇- 5 % STRAIN AT FAILURE
10

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE № 13

JOB 71-11068
W.P. 89-69-05/06
DATUM Geodetic

LOCATION Co-ords. 558,208 N; 339,405 E.
BORING DATE Nov. 7, 1973
BOREHOLE TYPE Cont. Flight Auger

ORIGINATED BY LJH
COMPILED BY LJH
CHECKED BY [signature]

SOIL PROFILE: ELEV. / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT. PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS/FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT; SHEAR STRENGTH P.S.F. (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE)	LIQUID LIMIT W_L, PLASTIC LIMIT W_P, WATER CONTENT W; WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
789.0	Ground Level									
0.0	Clayey silt, some sand, traces of gravel.		1	SS	11					
			2	SS	10					
						780				
	Stiff to Hard		3	SS	30					776.5 (water level)
772.5			4	SS	38					
16.5	End of Borehole									
						770				

OFFICE REPORT ON SOIL EXPLORATION

20
15 ◇ 5 % STRAIN AT FAILURE
10

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE Nº 14

JOB 71-11068
W.P. 89-69-05/06
DATUM Geodetic

LOCATION Co-ords. 558,263 N; 339,337 E.
BORING DATE Nov. 8, 1973
BOREHOLE TYPE Cont. Flight Auger

ORIGINATED BY LJH
COMPILED BY LJH
CHECKED BY

ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT; SHEAR STRENGTH P.S.F. O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT — W_L; PLASTIC LIMIT — W_P; WATER CONTENT — W; WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
789.4	Ground Level									
0.0	Clayey silt, some sand, traces of gravel.		1	SS	9					
			2	SS	23					
	Very Stiff to Hard		3	SS	30	780				
772.9			4	SS	52					Hole Dry
16.5	End of Borehole					770				

15 20 10 5 % STRAIN AT FAILURE

OFFICE REPORT SOIL EXPLORATION

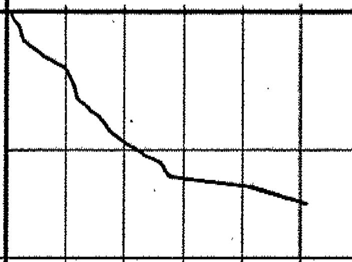
MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

DESIGN SERVICES BRANCH — FOUNDATIONS OFFICE

RECORD OF BOREHOLE Nº 15

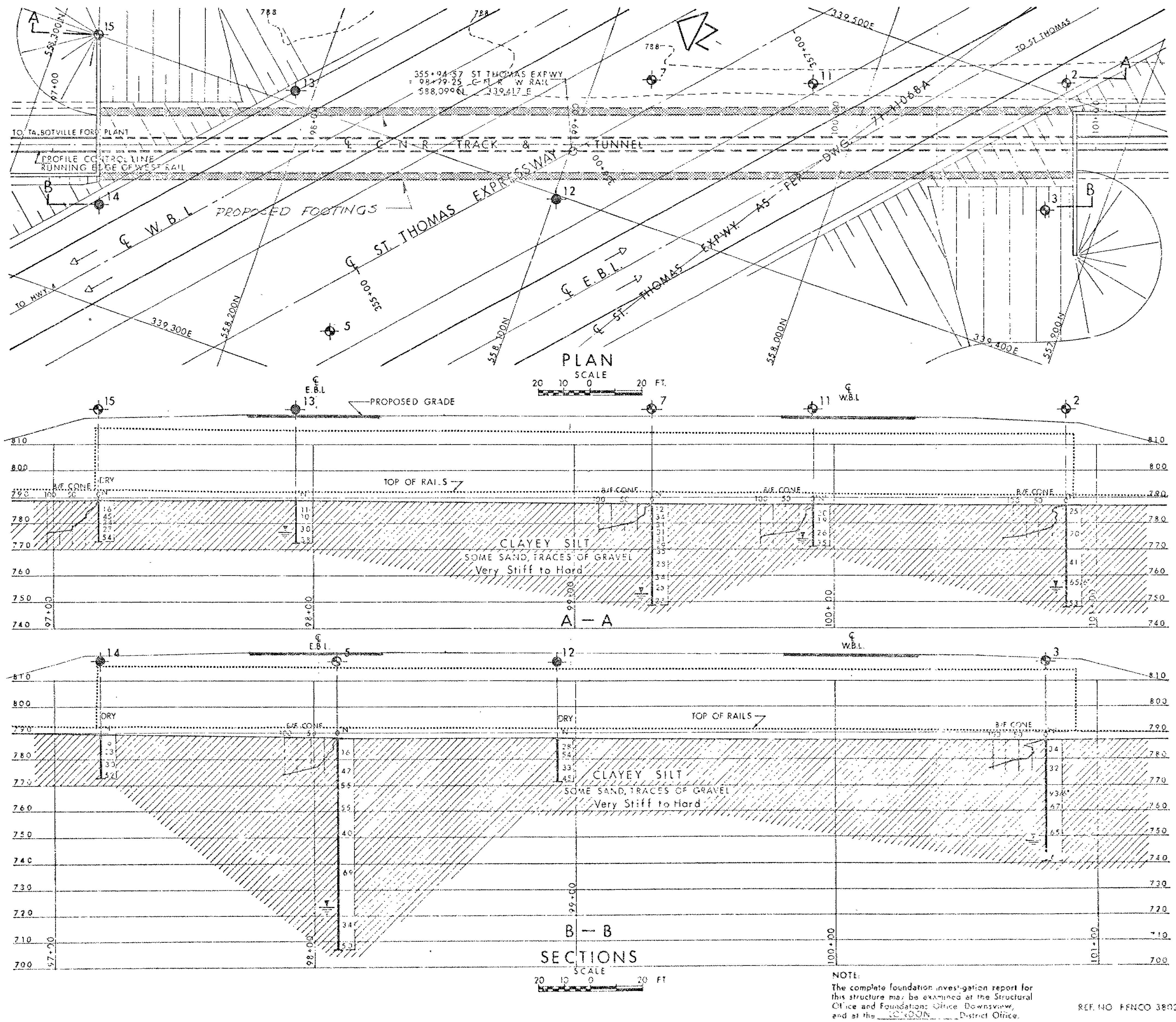
JOB 71-11068	LOCATION Co-ords. 558,286 N; 339,400 E.	ORIGINATED BY LJH
W.P. 89-69-05/06	BORING DATE November 7, 1973	COMPILED BY LJH.
DATUM Geodetic	BOREHOLE TYPE Cont. Flight Auger	CHECKED BY

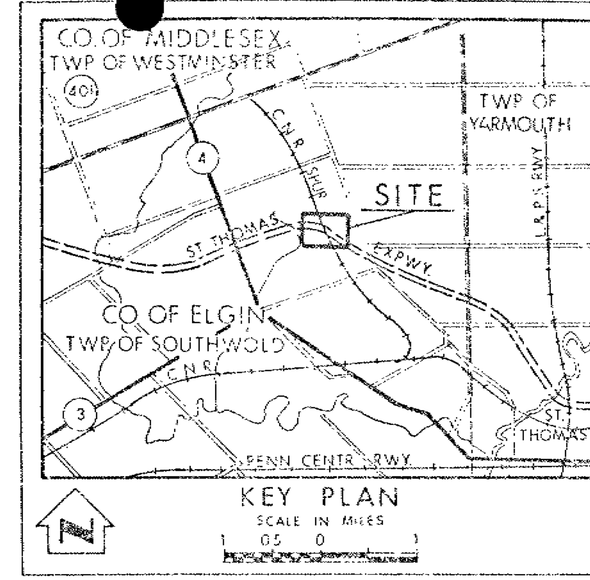
ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100; SHEAR STRENGTH P.S.F. O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT W_L; PLASTIC LIMIT W_P; WATER CONTENT W; WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
789.3	Ground Level									
0.0	Clayey silt, some sand, traces of gravel. Very Stiff to Hard		1	SS	16					
			2	SS	45					
			3	SS	24	780				
			4	SS	27					
772.8			5	SS	54					Hole Dry
16.5	End of Borehole					770				



20
15 ◇ 5 % STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION





LEGEND

- Bore Hole
- Cone Penetration Test
- Bore Hole & Cone Test
- Water levels established at time of field investigation July 1971 / July 1973

NO.	ELEVATION	CO-ORDINATES NORTH	CO-ORDINATES EAST	
2	787.0	557,929	339,505	July 71
3	787.0	557,920	339,456	
5	788.3	558,163	339,320	
7	787.8	558,080	339,453	
11	787.5	558,021	339,473	July 73
12	788.2	558,098	339,398	
13	789.0	558,208	339,405	
14	789.4	558,263	339,337	
15	789.3	558,286	339,400	

— 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

NOTE:
The complete foundation investigation report for this structure may be examined at the Structural Office and Foundations Office Downsview, and at the LONDON District Office.

REF. NO. FENCO 3802-97-109

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

C.N.R. SPUR LINE OVERPASS

HIGHWAY NO. PROP. ST. THOMAS EXPWY. DIST. NO. 2
CO. ELGIN
TWP. SOUTHWOLD LOT 42 & 43 CON. E.S.T.R.

BORE HOLE LOCATIONS & SOIL STRATA

SUBM'D J.H. | CHECKED | W.P. NO. 89-69-05/05 | DRAWING NO. 71-11068 B
DRAWN D.J. | CHECKED | W.O. NO. 71-11069
DATE 22 NOV 1973 | SITE NO. | BRIDGE DRAWING NO.
APPROVED | CONT. NO.

APPENDIX C

C.1 SYMBOLS AND TERMS USED ON BOREHOLE RECORDS

C.2 BOREHOLE RECORDS

C.3 CPT SOUNDING RECORDS

SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

SOIL DESCRIPTION

Terminology describing common soil genesis:

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

Terminology describing soil structure:

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

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:

Trace, or occasional	Less than 10%
Some	10-20%
Frequent	> 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
Very Loose	<4
Loose	4-10
Compact	10-30
Dense	30-50
Very Dense	>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	
Very Soft	<0.25	<12.5	<2
Soft	0.25 - 0.5	12.5 - 25	2-4
Firm	0.5 - 1.0	25 - 50	4-8
Stiff	1.0 - 2.0	50 – 100	8-15
Very Stiff	2.0 - 4.0	100 - 200	15-30
Hard	>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.

Boulders Cobbles Gravel	Sand	Silt	Clay	Organics	Asphalt	Concrete	Fill	Igneous Bedrock	Meta-morphic Bedrock	Sedi-mentary Bedrock

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

▼ measured in standpipe, piezometer, or well

▽ inferred

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
γ	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

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No CNR-EMB1

METRIC

W.P. 3041-22-00	LOCATION CNR Overhead, Southwold, Ontario N: 4742387.6 E: 408194.1		ORIGINATED BY HS
DIST West HWY Hwy 3	BOREHOLE TYPE Hollow Stem Auger		COMPILED BY KL
DATUM Geodetic	DATE 2024.05.16 - 2024.05.16	LATITUDE 42.81425905 LONGITUDE -81.23566536	CHECKED BY RR

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
241.4 / 0.0	300 mm TOPSOIL				
241.1 / 0.3	CLAYEY SILT (CL), trace to some sand, trace gravel (TILL) Stiff to very stiff (SS1 very soft) Brown to grey Moist	1	SS	2	
		2	SS	16	2 13 48 37
		3	SS	23	
		4	SS	28	
		5	SS	25	
	Grey below 3.8 m	6	SS	19	0 12 41 47 PP = 3.5 TSF
		7	SS	14	PP = 3.5 TSF
		1	TW		Consolidation Test
		8	SS	14	2 9 42 47 PP = 3.5 TSF
		9	SS	25	PP = 4.0 TSF
		10	SS	19	PP = 4.5 TSF
		11	SS	19	PP = 4.5 TSF
		12	SS	18	PP = 2.5 TSF
		13	SS	24	4 14 48 34 PP = 3.0 TSF
226.6 / 14.8	CLAYEY SILT (CL-ML), trace sand Very stiff Grey Moist	14	SS	24	
225.5 / 15.9	END OF BOREHOLE Borehole open and dry on completion of drilling.				

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (241 to 226); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m^3

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

RECORD OF BOREHOLE No CNR-EMB3

METRIC

W.P. 3041-22-00 LOCATION CNR Overhead, Southwold, Ontario N: 4742473.2 E: 408075.1 ORIGINATED BY HS

DIST West HWY Hwy 3 BOREHOLE TYPE Hollow Stem Auger COMPILED BY KL

DATUM Geodetic DATE 2024.03.04 - 2024.03.04 LATITUDE 42.81505 LONGITUDE -81.23712 CHECKED BY RR

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
241.8					
241.6 / 0.0	180 mm TOPSOIL				
0.2	SILTY, CLAYEY SAND (FILL), trace gravel Very loose to compact Brown Moist	1	SS	2	PP = 1.0 TSF
		2	SS	10	6 46 30 18
240.3 / 1.5	CLAYEY SILT (CL), trace to some sand, trace gravel (TILL) Stiff to hard Brown to grey Moist	3	SS	31	PP = 0.5 TSF
		4	SS	22	PP = 4.5 TSF
		5	SS	14	4 12 42 42 PP = 3.0 TSF
	Grey below 3.8 m	6	SS	13	PP = 3.5 TSF
		7	SS	12	PP = 3.5 TSF
		8	SS	23	PP = 3.5 TSF
		9	SS	16	PP = 3.5 TSF
		10	SS	26	PP = 4.5 TSF
		11	SS	21	PP = 4.5 TSF
		12	SS	18	1 13 48 38 PP = 4.5 TSF
		13	SS	19	PP = 4.5 TSF
		14	SS	22	PP = 4.5 TSF
225.9		15	SS	21	PP = 3.75 TSF
15.9	END OF BOREHOLE Borehole open and dry on completion of drilling.				

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (241–226); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE, 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³.

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

Ministry of Transportation Ontario

RECORD OF BOREHOLE No CNR-EMB4

METRIC

W.P. 3041-22-00	LOCATION CNR Overhead, Southwold, Ontario N: 4742503.0 E: 408026.1		ORIGINATED BY HS
DIST West HWY Hwy 3	BOREHOLE TYPE Hollow Stem Auger		COMPILED BY KL
DATUM Geodetic	DATE 2024.03.04 - 2024.03.04	LATITUDE 42.81533 LONGITUDE -81.23771	CHECKED BY RR

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
241.4						
241.1 / 0.0	280 mm TOPSOIL					
0.3	CLAYEY SILT (CL), trace to some sand, trace gravel (TILL) Stiff to very stiff (SS1 firm) Brown to grey Moist	1	SS	7	241	PP = 3.5 TSF
		2	SS	13	240	PP = 4.0 TSF
		3	SS	25		1 12 44 44
		4	SS	24	239	
	Grey below 3.0 m	5	SS	16	238	PP = 3.5 TSF
		6	SS	15	237	PP = 4.0 TSF
		7	SS	16		PP = 4.0 TSF
		8	SS	15	236	PP = 4.5 TSF
		9	SS	18	235	1 12 46 42 PP = 4.5 TSF
					234	
		10	SS	25	233	PP > 4.5 TSF
		11	SS	22	232	PP > 4.5 TSF
					231	
		12	SS	25	230	PP > 4.5 TSF
		13	SS	16	229	PP = 4.5 TSF
					228	
		14	SS	23	227	2 15 46 37 PP > 4.5 TSF
		15	SS	29	226	PP > 4.5 TSF
225.6 / 15.9	END OF BOREHOLE Borehole open and dry on completion of drilling.					

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

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

RECORD OF BOREHOLE No CNR-EMB5

METRIC

W.P.	3041-22-00	LOCATION	CNR Overhead, Southwold, Ontario N: 4742528.0 E: 407978.3	ORIGINATED BY	HS
DIST	West HWY Hwy 3	BOREHOLE TYPE	Hollow Stem Auger	COMPILED BY	KL
DATUM	Geodetic	DATE	2024.03.05 - 2024.03.05 LATITUDE 42.81556 LONGITUDE -81.23829	CHECKED BY	RR

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
240.8	
240.6 / 0.0 / 0.2	150 mm TOPSOIL
	CLAYEY SILT (CL), trace to some sand, trace gravel (TILL) Stiff to hard (SS1 firm) Brown to grey Moist
	Grey below 3.8 m
224.9 / 15.9	END OF BOREHOLE Borehole open and dry on completion of drilling.

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	5	PP = 3.0 TSF
2	SS	10	PP = 4.5 TSF
3	SS	13	PP = 4.5 TSF
4	SS	22	0 12 42 46 PP > 4.5 TSF
5	SS	14	PP > 4.5 TSF
6	SS	15	PP = 3.5 TSF
7	SS	14	PP > 4.5 TSF
8	SS	15	2 12 44 42 PP > 4.5 TSF
9	SS	17	PP > 4.5 TSF
10	SS	16	PP > 4.5 TSF
11	SS	21	PP > 4.5 TSF
12	SS	20	PP = 4.5 TSF
13	SS	36	3 16 46 35 PP > 4.5 TSF
14	SS	28	PP > 4.5 TSF
15	SS	23	PP > 4.5 TSF

ELEVATION SCALE: 240, 239, 238, 237, 236, 235, 234, 233, 232, 231, 230, 229, 228, 227, 226, 225

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

GROUND WATER CONDITIONS

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

RECORD OF BOREHOLE No CNR-EMB6

METRIC

W.P.	3041-22-00	LOCATION	CNR Overhead, Southwold, Ontario N: 4742559.1 E: 407913.2	ORIGINATED BY	HS
DIST	West HWY Hwy 3	BOREHOLE TYPE	Hollow Stem Auger	COMPILED BY	KL
DATUM	Geodetic	DATE	2024.03.06 - 2024.03.06 LATITUDE 42.81585 LONGITUDE -81.23908	CHECKED BY	RR

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
239.9	
239.9 / 0.0	200 mm TOPSOIL
239.7 / 0.2	CLAYEY SILT (CL), trace to some sand, trace gravel (TILL) Stiff to very stiff Brown to grey Moist Grey below 3.0 m
224.3	
224.1 / 15.7	CLAYEY SILT (CL-ML), trace sand Very stiff Grey Moist to wet
15.9	END OF BOREHOLE Borehole open and dry on completion of drilling.

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	8	PP > 4.5 TSF
2	SS	17	1 11 43 44 PP > 4.5 TSF
3	SS	20	PP > 4.5 TSF
4	SS	22	PP > 4.5 TSF
5	SS	14	PP > 4.5 TSF
6	SS	16	PP > 4.5 TSF
7	SS	15	PP > 4.5 TSF
8	SS	15	PP > 4.5 TSF
9	SS	16	PP > 4.5 TSF
10	SS	18	5 15 45 36 PP > 4.5 TSF
11	SS	23	PP > 4.5 TSF
12	SS	24	PP > 4.5 TSF
13	SS	24	PP = 4.5 TSF
14	SS	22	PP = 4.0 TSF
15	SS	26	1 11 49 39 PP > 4.5 TSF

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

Elevation scale: 239, 238, 237, 236, 235, 234, 233, 232, 231, 230, 229, 228, 227, 226, 225

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

RECORD OF BOREHOLE No CNR-EMB7

METRIC

W.P. 3041-22-00 LOCATION CNR Overhead, Southwold, Ontario N: 4742291.1 E: 408305.7 ORIGINATED BY HS

DIST West HWY Hwy 3 BOREHOLE TYPE Hollow Stem Auger COMPILED BY KL

DATUM Geodetic DATE 2024.05.15 - 2024.05.15 LATITUDE 42.81338129 LONGITUDE -81.23433375 CHECKED BY RR

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
240.1 / 0.0	300 mm TOPSOIL
239.8 / 0.3	CLAYEY SILT (CL), trace to some sand, trace gravel (TILL) Stiff to hard (SS1 soft) Brown to grey Moist Grey below 3.8 m
224.3 / 15.9	END OF BOREHOLE Borehole open and dry on completion of drilling.

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	4	
2	SS	8	PP = 2.5 TSF
	VANE		(B-Vane Refusal)
3	SS	16	2 13 46 39
4	SS	16	PP = 3.0 TSF
5	SS	14	PP = 3.0 TSF
1	TW		Consolidation Test
6	SS	17	0 10 45 44 PP = 3.5 TSF
7	SS	16	PP = 3.5 TSF
8	SS	18	PP = 4.0 TSF
9	SS	20	PP = 4.5 TSF
10	SS	19	2 11 50 37 PP = 4.0 TSF
11	SS	19	PP = 4.5 TSF
12	SS	33	PP = 4.5 TSF
13	SS	23	PP = 4.0 TSF
14	SS	25	PP = 4.0 TSF

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (240 to 225); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE, 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Field vane at sample 2/VANE: >>

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

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No CNR-EMB9

1 OF 1 **METRIC**

W.P. 3041-22-00 LOCATION CNR Overhead, Southwold, Ontario N: 4742180.1 E: 408425.8 ORIGINATED BY HS

DIST West HWY Hwy 3 BOREHOLE TYPE Hollow Stem Auger COMPILED BY KL

DATUM Geodetic DATE 2024.05.14 - 2024.05.15 LATITUDE 42.81236663 LONGITUDE -81.23288586 CHECKED BY RR

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
239.7	
239.6 / 0.0	150 mm TOPSOIL
0.2	CLAYEY SILT (CL), trace to some sand, trace gravel (TILL) Stiff to very stiff (SS1 soft) Brown to grey Moist Grey below 3.0 m
224.9 / 14.8	CLAYEY SILT (CL), trace sand, trace gravel Very stiff Grey Moist to wet
223.9 / 15.9	END OF BOREHOLE Borehole open and dry on completion of drilling.

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	3	
2	SS	16	1 13 46 40
1	TW		
3	SS	23	PP = 4.5 TSF
4	SS	19	0 12 43 44 PP = 4.5 TSF
5	SS	18	PP = 4.0 TSF
6	SS	17	PP = 4.5 TSF
7	SS	14	PP = 4.5 TSF
2	TW		Consolidation Test
8	SS	34	PP = 4.5 TSF
9	SS	19	2 17 47 34 PP = 3.5 TSF
10	SS	22	
11	SS	26	
12	SS	18	
13	SS	20	
14	SS	17	1 2 65 32 PP = 2.0 TSF

STRAT PLOT | GROUND WATER CONDITIONS | ELEVATION SCALE: 239, 238, 237, 236, 235, 234, 233, 232, 231, 230, 229, 228, 227, 226, 225, 224

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

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

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L

WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m^3

ONTARIO MTO 165001308_MTO_CNR-BYPASS_20240926.GPJ ONTARIO MTO.GDT 9/26/24

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

Ministry of Transportation Ontario

RECORD OF BOREHOLE No CNR-EMB11

METRIC

W.P. 3041-22-00 LOCATION CNR Overhead, Southwold, Ontario N: 4742091.9 E: 408523.1 ORIGINATED BY HS

DIST West HWY Hwy 3 BOREHOLE TYPE Hollow Stem Auger COMPILED BY KL

DATUM Geodetic DATE 2024.05.14 - 2024.05.14 LATITUDE 42.811559 LONGITUDE -81.231714 CHECKED BY RR

Column headings: SOIL PROFILE (ELEV / DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
239.2					
239.0 / 0.0	200 mm TOPSOIL				
0.2	CLAYEY SILT (CL), trace to some sand, trace gravel (TILL) Very stiff (SS1 soft) Brown to grey Moist	1	SS	3	
		2	SS	19	2 13 44 41 PP = 4.5 TSF
		3	SS	27	PP = 4.5 TSF
		4	SS	22	PP = 4.5 TSF
	Grey below 3.0 m	5	SS	20	PP = 4.5 TSF
		6	SS	19	3 14 48 35 PP = 4.5 TSF
		7	SS	20	PP = 4.0 TSF
		8	SS	21	PP = 4.0 TSF
		9	SS	22	PP = 4.5 TSF
		10	SS	24	6 11 46 36 PP = 4.5 TSF
		11	SS	28	PP = 4.5 TSF
		12	SS	26	PP = 4.5 TSF
227.5 / 11.7	CLAYEY SILT (CL-ML) Hard Grey Wet	13	SS	49	0 0 80 20
226.0 / 13.3	CLAYEY SILT (CL), trace to some sand, trace gravel (TILL) Very stiff Grey Moist	14	SS	19	PP = 2.5 TSF
		1	TW		Consolidation Test
		15	SS	25	PP = 2.5 TSF
223.4 / 15.9	END OF BOREHOLE Groundwater and cave-in measured at 13.6 m and 14.5 m below grade on completion of drilling, respectively.				

Elevation scale: 239, 238, 237, 236, 235, 234, 233, 232, 231, 230, 229, 228, 227, 226, 225, 224

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

ONTARIO MTO 165001308_MTO_CNR-BYPASS_20240926.GPJ ONTARIO MTO.GDT 9/26/24

Ministry of Transportation Ontario

RECORD OF BOREHOLE No CNR-EMB12

METRIC

W.P. 3041-22-00 LOCATION CNR Overhead, Southwold, Ontario N: 4742048.1 E: 408572.4 ORIGINATED BY HS

DIST West HWY Hwy 3 BOREHOLE TYPE Hollow Stem Auger COMPILED BY KL

DATUM Geodetic DATE 2024.05.13 - 2024.05.13 LATITUDE 42.811159 LONGITUDE -81.231118 CHECKED BY RR

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
239.1					
239.0 / 0.1	100 mm TOPSOIL				
	CLAYEY SILT (CL), trace to some sand, trace gravel (TILL) Very stiff (SS1 soft) Brown to grey Moist	1	SS	5	
		2	SS	23	
		3	SS	22	3 13 45 39 PP = 4.5 TSF
		4	SS	29	PP = 4.5 TSF
	Grey below 3.0 m	5	SS	24	PP = 4.5 TSF
		6	SS	20	PP = 3.5 TSF
		7	SS	18	6 14 46 34 PP = 4.5 TSF
		8	SS	18	PP = 4.5 TSF
		9	SS	22	PP = 4.5 TSF
		10	SS	28	PP = 4.5 TSF
		11	SS	29	PP = 4.5 TSF
228.9 / 10.2	CLAYEY SILT (CL-ML), trace sand Hard Grey Wet	12	SS	51	0 2 78 20
227.4 / 11.7	CLAYEY SILT (CL), trace to some sand (TILL) Very stiff to hard Grey Moist	13	SS	17	PP = 3.0 TSF
		14	SS	15	PP = 2.5 TSF
		1	TW		
		15	SS	30	
223.2 / 15.9	END OF BOREHOLE Borehole dry and cave-in measured at 14.6 m below grade on completion of drilling.				

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (239 to 224); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE, 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL.

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

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No CNR-OH1

METRIC

W.P. 3041-22-00 LOCATION CNR Overhead, Southwold, Ontario N: 4742326.5 E: 408283.1 ORIGINATED BY MC

DIST West HWY Hwy 3 BOREHOLE TYPE Hollow Stem Augers/ Wash Boring COMPILED BY KL

DATUM Geodetic DATE 2024.06.03 - 2024.06.06 LATITUDE 42.81370323 LONGITUDE -81.23460378 CHECKED BY RR

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
240.2	
240.0 / 0.2	150 mm TOPSOIL
	CLAYEY SILT (CL), some sand, trace gravel (TILL) Stiff to hard (SS1 soft) Brown to grey Moist Grey below 2.3 m
223.2	

Continued Next Page

SAMPLES AND REMARKS

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	4	
2	SS	16	
3	SS	29	1 12 42 45
4	SS	17	Wash Boring below 3.0m
5	SS	14	PP = 2.0 TSF
6	SS	18	1 12 43 44 PP = 2.5 TSF
7	TW		
8	SS	25	PP = 4.0 TSF
9	SS	30	PP = 3.5 TSF
10	SS	27	5 12 45 38 PP = 4.5 TSF
11	SS	24	PP = 4.0 TSF
12	SS	28	PP = 4.0 TSF
13	SS	30	PP = 2.5 TSF
14	SS	31	PP = 3.5 TSF
15	SS	30	PP = 3.0 TSF

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (240 to 224); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³

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

RECORD OF BOREHOLE No CNR-OH1

METRIC

W.P. 3041-22-00 — LOCATION CNR Overhead, Southwold, Ontario N: 4742326.5 E: 408283.1 — ORIGINATED BY MC

DIST West HWY Hwy 3 — BOREHOLE TYPE Hollow Stem Augers/ Wash Boring — COMPILED BY KL

DATUM Geodetic — DATE 2024.06.03 - 2024.06.06 — LATITUDE 42.81370323 — LONGITUDE -81.23460378 — CHECKED BY RR

SOIL PROFILE: ELEV / DEPTH; DESCRIPTION; STRAT PLOT

SAMPLES: NUMBER; TYPE; "N" VALUES

GROUND WATER CONDITIONS; ELEVATION SCALE

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT W_P; NATURAL MOISTURE CONTENT W; LIQUID LIMIT W_L; WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION	REMARKS	GR	SA	SI	CL
17.0	CLAYEY SILT (CL), some sand, trace to some gravel (TILL) Stiff to hard Grey Moist	16	SS	16	223	PP = 2.0 TSF				
		17	SS	10	222–221	PP = 1.0 TSF				
			VANE		221	Su= 160 kPa (B-vane)				
		18	SS	22	220	PP = 1.5 TSF	1	13	41	45
		19	SS	17	217	PP = 1.0 TSF				
		20	SS	25	214	PP = 1.5 TSF				
		21	SS	24	211	PP = 1.5 TSF				
		22	SS	37	208	PP = 1.5 TSF	14	10	38	37
206.2										

Elevation scale: 223, 222, 221, 220, 219, 218, 217, 216, 215, 214, 213, 212, 211, 210, 209, 208, 207

Field vane at 221: >>

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

RECORD OF BOREHOLE No CNR-OH1

METRIC

W.P. 3041-22-00 LOCATION CNR Overhead, Southwold, Ontario N: 4742326.5 E: 408283.1 ORIGINATED BY MC

DIST West HWY Hwy 3 BOREHOLE TYPE Hollow Stem Augers/ Wash Boring COMPILED BY KL

DATUM Geodetic DATE 2024.06.03 - 2024.06.06 LATITUDE 42.81370323 LONGITUDE -81.23460378 CHECKED BY RR

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
34.0	CLAYEY SILT (CL), some sand, trace gravel (TILL) Hard Grey Moist				
		23	SS	33	PP = 1.25 TSF
		24	SS	41	PP = 1.5 TSF
		25	SS	38	1 13 45 41 PP = 1.0 TSF
195.4		26	SS	39	PP = 1.5 TSF
44.8	END OF BOREHOLE Borehole dry upon start of mud drilling at approximately 3 m below grade. Monitoring well installed in borehole, screened from approximately 4.6 m to 7.6 m below grade. Groundwater level recorded in monitoring well at approximately 0.5 m below grade on August 29, 2024.				

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (206, 205, 204, 203, 202, 201, 200, 199, 198, 197, 196); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m^3.

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

RECORD OF BOREHOLE No CNR-OH2

METRIC

W.P. 3041-22-00 | LOCATION CNR Overhead, Southwold, Ontario N: 4742271.9 E: 408298.2 | ORIGINATED BY MC

DIST West HWY Hwy 3 | BOREHOLE TYPE Hollow Stem Augers/ Wash Boring | COMPILED BY KL

DATUM Geodetic | DATE 2024.05.28 - 2024.06.03 | LATITUDE 42.81321029 | LONGITUDE -81.23442911 | CHECKED BY RR

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL	REMARKS
240.4 / 0.0	300 mm TOPSOIL								
240.1 / 0.3	CLAYEY SILT (CL), some sand, trace gravel (TILL) Stiff to hard (SS1 firm) Brown to grey Moist	1	SS	7					
		2	SS	19	2	14	44	41	
		3	SS	13					PP = 4.5 TSF
		4	SS	23					PP = 4.5 TSF Wash Boring below 3.0m
	Grey below 3.0 m	5	SS	24	1	12	43	43	PP = 4.5 TSF
		6	SS	21					PP = 3.0 TSF
		7	SS	26					PP = 4.5 TSF
		8	SS	33	3	14	47	37	PP = 4.5 TSF
		9	SS	27					PP = 4.5 TSF
		10	SS	41					PP = 4.5 TSF
		11	SS	33					PP = 4.5 TSF
		12	SS	43					PP = 4.5 TSF
		13	SS	62					PP = 4.5 TSF
223.4									

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (240 to 224); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

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

RECORD OF BOREHOLE No CNR-OH2

METRIC

W.P. 3041-22-00	LOCATION CNR Overhead, Southwold, Ontario N: 4742271.9 E: 408298.2		ORIGINATED BY MC
DIST West HWY Hwy 3	BOREHOLE TYPE Hollow Stem Augers/ Wash Boring		COMPILED BY KL
DATUM Geodetic	DATE 2024.05.28 - 2024.06.03	LATITUDE 42.81321029 LONGITUDE -81.23442911	CHECKED BY RR

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
17.0	CLAYEY SILT (CL), some sand, trace to some gravel (TILL) Very stiff to hard Grey Moist	14	SS	25	223	PP = 2.25 TSF
		15	SS	26	222	PP = 2.0 TSF
		16	SS	21	221 / 220	1 11 42 47 PP = 1.5 TSF
		17	SS	22	219 / 218 / 217	PP = 1.5 TSF
		18	SS	35	216 / 215 / 214	PP = 1.5 TSF
		19	SS	42	213 / 212 / 211	PP = 1.5 TSF
		20	SS	54	210 / 209 / 208 / 207	
206.4						

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

Continued Next Page

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

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No CNR-OH2

METRIC

W.P. 3041-22-00 | LOCATION CNR Overhead, Southwold, Ontario N: 4742271.9 E: 408298.2 | ORIGINATED BY MC

DIST West HWY Hwy 3 | BOREHOLE TYPE Hollow Stem Augers/ Wash Boring | COMPILED BY KL

DATUM Geodetic | DATE 2024.05.28 - 2024.06.03 LATITUDE 42.81321029 LONGITUDE -81.23442911 | CHECKED BY RR

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
34.0	CLAYEY SILT (CL), some sand, trace gravel (TILL) Very stiff to hard Grey Moist				206	
		21	SS	43	205	3 13 46 38 PP = 1.5 TSF
					204	
					203	
		22	SS	56	202	PP = 0.5 TSF
					201	
	Inferred cobbles/boulder based on drilling difficulties between 39.9 m and 40.8 m				200	
		23	SS	65	199	
					198	
		24	SS	29	197	
195.6	Frequent seams/layers of SILTY CLAY to CLAYEY SILT	25	SS	34	196	0 2 49 48 PP = 1.5 TSF
44.8	END OF BOREHOLE Borehole dry upon start of mud drilling at approximately 3 m below grade.					

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

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

RECORD OF BOREHOLE No CNR-OH3

METRIC

W.P. 3041-22-00	LOCATION CNR Overhead, Southwold, Ontario N: 4742374.5 E: 408227.3		ORIGINATED BY HS/MC
DIST West HWY Hwy 3	BOREHOLE TYPE Hollow Stem Augers/ Wash Boring		COMPILED BY KL
DATUM Geodetic	DATE 2024.05.21 - 2024.06.11	LATITUDE 42.81414325 LONGITUDE -81.2352769	CHECKED BY RR

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
240.7						
240.6 0.0	150 mm TOPSOIL					
240.5 0.2	SILTY CLAY (CI), trace to some sand, trace gravel (TILL) Firm to stiff (SS1 soft) Brown Moist	1	SS	4		
		2	SS	9	240	
		3	SS	7	239	1 17 43 39
238.5 2.2	CLAYEY SILT (CL), some sand, trace gravel (TILL) Stiff to hard Brown to grey Moist	4	SS	21	238	Wash Boring below 3.0m
		5	SS	20	237	
	Grey below 3.8 m	6	SS	13		PP = 1.5 TSF
		1	TW		236	UU Test
		7	SS	21	235	PP = 3.0 TSF
		8	SS	20	234	2 13 46 39 PP = 2.5 TSF
		9	SS	26	233	
		10	SS	24	232 / 231	PP = 3.5 TSF
		11	SS	27	230	PP = 3.0 TSF
		12	SS	27	229 / 228	PP = 4.5 TSF
		13	SS	32	227	2 12 48 39 PP = 3.0 TSF
		14	SS	26	226 / 225	PP = 3.5 TSF
223.7					224	

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

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

RECORD OF BOREHOLE No CNR-OH3

METRIC

W.P. 3041-22-00 LOCATION CNR Overhead, Southwold, Ontario N: 4742374.5 E: 408227.3 ORIGINATED BY HS/MC

DIST West HWY Hwy 3 BOREHOLE TYPE Hollow Stem Augers/ Wash Boring COMPILED BY KL

DATUM Geodetic DATE 2024.05.21 - 2024.06.11 LATITUDE 42.81414325 LONGITUDE -81.2352769 CHECKED BY RR

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
17.0	CLAYEY SILT (CL), trace to some sand, trace gravel (TILL) Very stiff to hard Grey Moist	15	SS	19			PP = 1.5 TSF
						223	
		16	SS	31		222	0 0 68 32
						221	
		17	SS	21			PP = 1.5 TSF
						220	
		18	SS	22		219	PP = 1.5 TSF
						218	
						217	
		19	SS	22		216	PP = 1.5 TSF
						215	
						214	
		20	SS	33		213	PP = 1.5 TSF
						212	
						211	
		21	SS	29		210	3 13 42 42 PP = 1.5 TSF
						209	
						208	
206.7		22	SS	34		207	PP = 1.5 TSF

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

Continued Next Page

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

RECORD OF BOREHOLE No CNR-OH3

METRIC

W.P. 3041-22-00 LOCATION CNR Overhead, Southwold, Ontario N: 4742374.5 E: 408227.3 ORIGINATED BY HS/MC

DIST West HWY Hwy 3 BOREHOLE TYPE Hollow Stem Augers/ Wash Boring COMPILED BY KL

DATUM Geodetic DATE 2024.05.21 - 2024.06.11 LATITUDE 42.81414325 LONGITUDE -81.2352769 CHECKED BY RR

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
34.0	CLAYEY SILT (CL), some sand, trace gravel (TILL) Very stiff to hard Grey Moist					206	
		23	SS	48		205	PP = 1.5 TSF
						204	
						203	
		24	SS	40		202	PP = 1.5 TSF
						201	
						200	
		25	SS	33		199	3 13 43 41 PP = 1.75 TSF
						198	
						197	
195.9		26	SS	58		196	PP = 2.0 TSF
44.8	END OF BOREHOLE Borehole dry upon start of mud drilling at approximately 3.6 m below grade.						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

UNIT WEIGHT γ kN/m^3

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

RECORD OF BOREHOLE No CNR-OH4

METRIC

W.P. 3041-22-00	LOCATION CNR Overhead, Southwold, Ontario N: 4742319.5 E: 408248.9		ORIGINATED BY MC
DIST West HWY Hwy 3	BOREHOLE TYPE Hollow Stem Augers/ Wash Boring		COMPILED BY KL
DATUM Geodetic	DATE 2024.07.04 - 2024.07.11	LATITUDE 42.81364502 LONGITUDE -81.23502322	CHECKED BY RR

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
240.5	
240.4 0.1	100 mm TOPSOIL
	CLAYEY SILT to SILTY CLAY (CL to CI), some sand, trace to some gravel (TILL) Very stiff to hard (SS1 firm) Brown to grey Moist
	Grey below 3.8 m
	Frequent layers of SILT with clay to CLAYEY SILT in SS9
223.5	

SAMPLES / REMARKS & GRAIN SIZE DISTRIBUTION (%) (GR SA SI CL)

NUMBER	TYPE	"N" VALUES	REMARKS
1	SS	5	
2	SS	18	
3	SS	21	PP = 4.5 TSF
4	SS	23	0 13 44 42 PP = 4.5 TSF Wash Boring below 3.0m
5	SS	22	PP = 4.5 TSF
6	SS	26	PP = 3.0 TSF
7	SS	27	PP = 4.5 TSF
8	SS	33	3 16 44 37 PP > 4.5 TSF
9	SS	39	PP = 3.0 TSF
10	SS	23	PP = 4.25 TSF
11	SS	29	PP = 4.5 TSF
12	SS	37	14 19 37 30 PP = 4.0 TSF
13	SS	40	PP = 4.25 TSF
14	SS	33	
15	SS	35	17 19 35 29 PP = 4.0 TSF

STRAT PLOT | GROUND WATER CONDITIONS | ELEVATION SCALE: 240, 239, 238, 237, 236, 235, 234, 233, 232, 231, 230, 229, 228, 227, 226, 225, 224

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT W_L; WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

Continued Next Page

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

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No CNR-OH4

METRIC

W.P. 3041-22-00	LOCATION CNR Overhead, Southwold, Ontario N: 4742319.5 E: 408248.9		ORIGINATED BY MC
DIST West HWY Hwy 3	BOREHOLE TYPE Hollow Stem Augers/ Wash Boring		COMPILED BY KL
DATUM Geodetic	DATE 2024.07.04 - 2024.07.11	LATITUDE 42.81364502 LONGITUDE -81.23502322	CHECKED BY RR

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
17.0	CLAYEY SILT (CL), some sand, trace gravel (TILL) Very stiff to hard Grey Moist	16	SS	35	223		PP = 3.5 TSF
		17	SS	47	222		PP = 1.0 TSF
		18	SS	20	221 / 220		PP = 1.25 TSF
		19	SS	23	219 / 218 / 217		PP = 1.25 TSF
		20	SS	27	216 / 215 / 214		5 19 32 32 PP = 1.0 TSF
		21	SS	28	213 / 212 / 211		PP = 1.25 TSF
		22	SS	35	210 / 209 / 208 / 207		PP = 1.0 TSF
206.5							

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

Continued Next Page

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

RECORD OF BOREHOLE No CNR-OH4

METRIC

W.P. 3041-22-00 LOCATION CNR Overhead, Southwold, Ontario N: 4742319.5 E: 408248.9 ORIGINATED BY MC

DIST West HWY Hwy 3 BOREHOLE TYPE Hollow Stem Augers/ Wash Boring COMPILED BY KL

DATUM Geodetic DATE 2024.07.04 - 2024.07.11 LATITUDE 42.81364502 LONGITUDE -81.23502322 CHECKED BY RR

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
34.0	CLAYEY SILT (CL), some sand, trace gravel (TILL) Very stiff to hard Grey Moist					206	
		23	SS	28		205	PP = 1.75 TSF
						204	
						203	
		24	SS	30		202	2 13 45 40 PP = 1.25 TSF
						201	
						200	
		25	SS	34		199	
						198	
						197	
		26	SS	50		196	
195.7 44.8	END OF BOREHOLE Borehole dry upon start of mud drilling at approximately 3.6 m below grade.						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

UNIT WEIGHT γ kN/m³

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

PRESENTATION OF SITE INVESTIGATION RESULTS

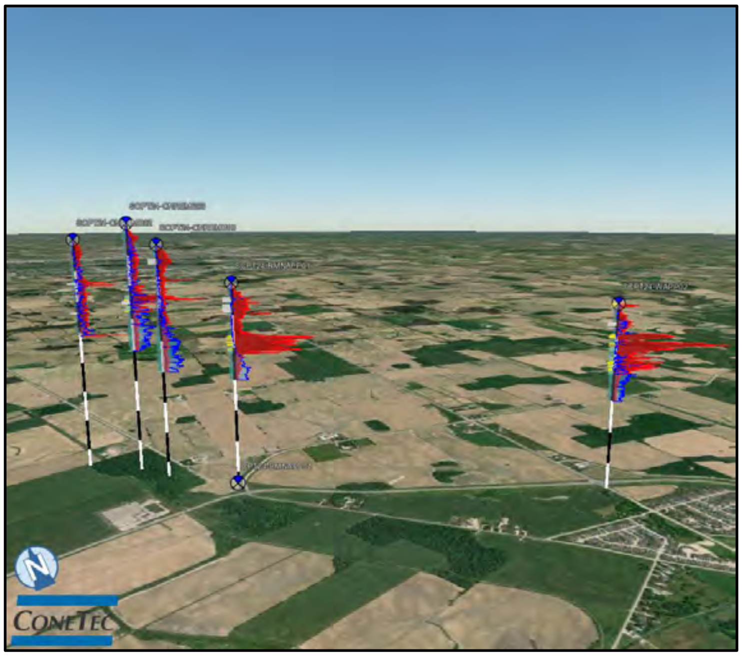
HWY 3 St Thomas CPT

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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www.conetecdataservices.com

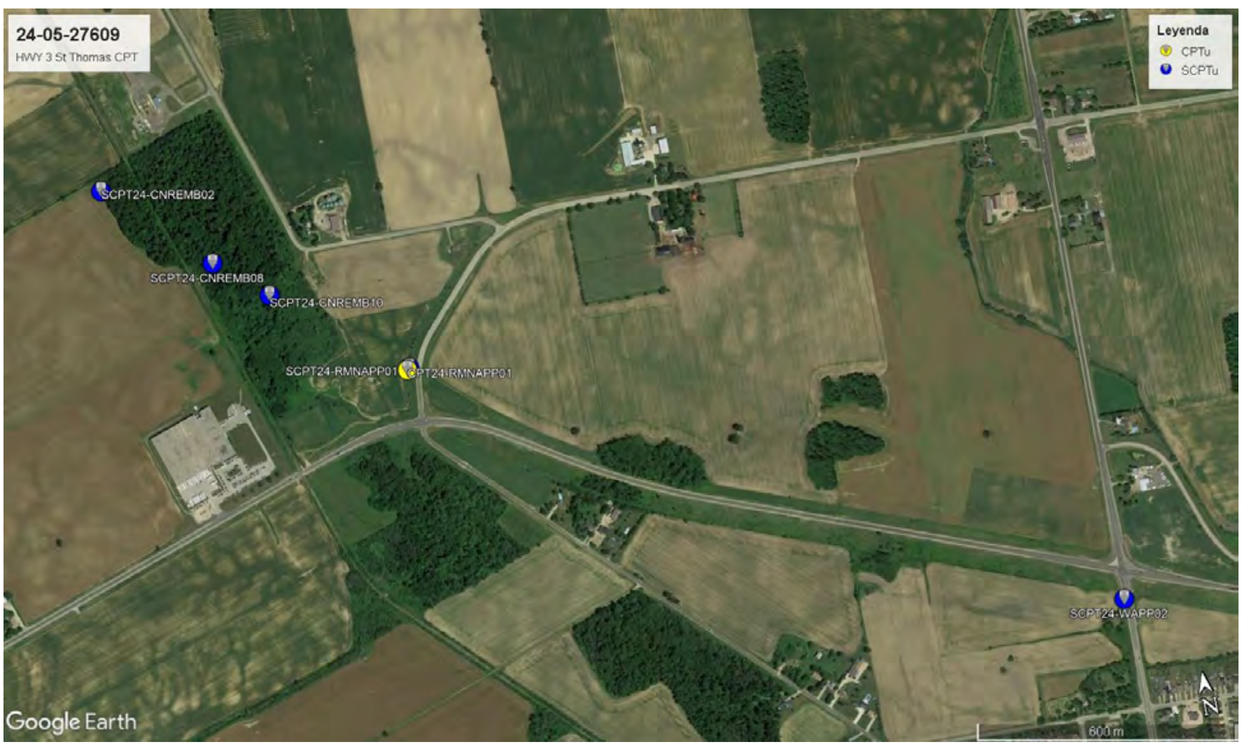
Introduction

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	• Advanced plots with Ic, Su, phi and N1(60)Ic • Seismic shear wave velocity plots • Soil Behaviour Type (SBT) scatter plots

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

Limitations

3rd Party Disclaimer

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

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

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

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

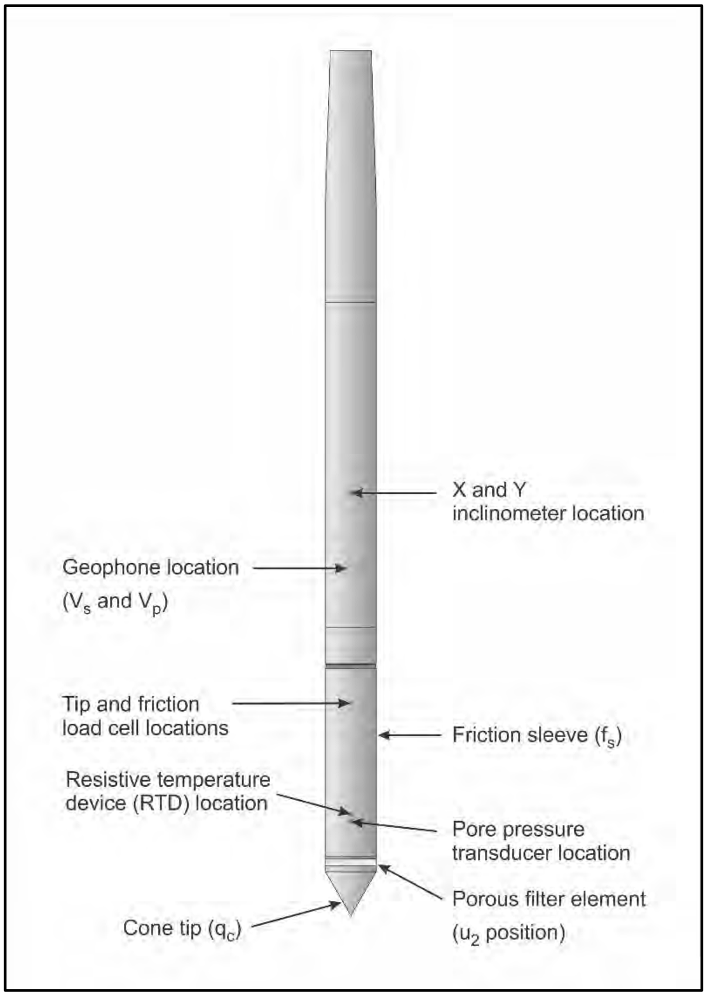


Figure CPTu. Piezocone Penetrometer (15 cm^2)

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

For additional information on CPTu interpretations and calculated geotechnical parameters, refer to Robertson et al. (1986), Lunne et al. (1997), Robertson (2009), Mayne (2013, 2014), 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 (Vs) testing is performed in conjunction with the piezocone penetration test (SCPTu) in order to collect interval velocities. For some projects seismic compression wave velocity (Vp) testing is also performed.

ConeTec's 15 cm^2 piezocone penetrometers are manufactured with one horizontally active geophone (28 hertz) and one vertically active geophone (28 hertz). Both geophones are rigidly mounted in the body of the cone penetrometer, 0.2 meters behind the cone tip. The vertically mounted geophone is more sensitive to compression waves; however, it is often affected by the compression wave travelling through the cone rods.

Shear waves are typically generated by using an impact hammer horizontally striking a beam that is held in place by a normal load. In some instances, an auger source or an imbedded impulsive source 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.

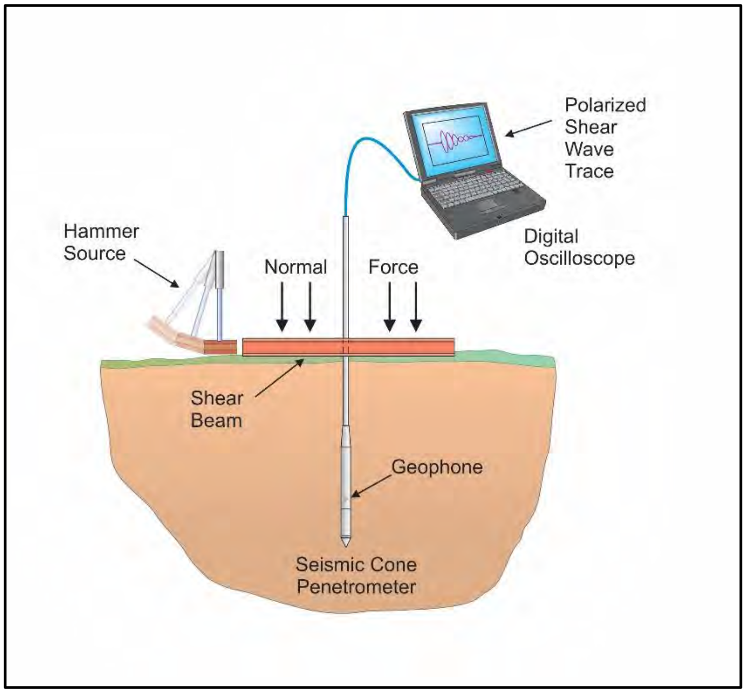


Figure SCPTu-1. Illustration of the SCPTu system

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).

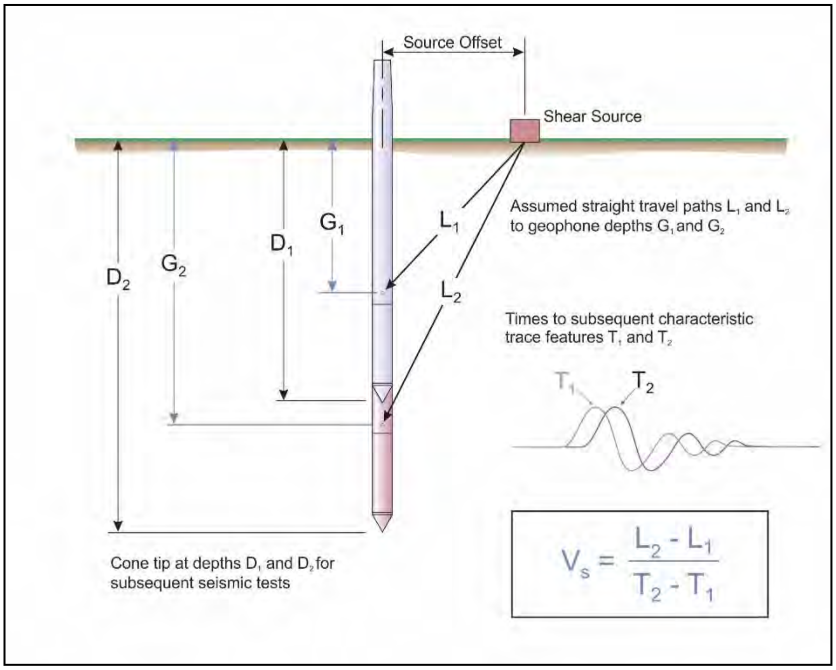


Figure SCPTu-2. Illustration of a seismic cone penetration test

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).

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

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).

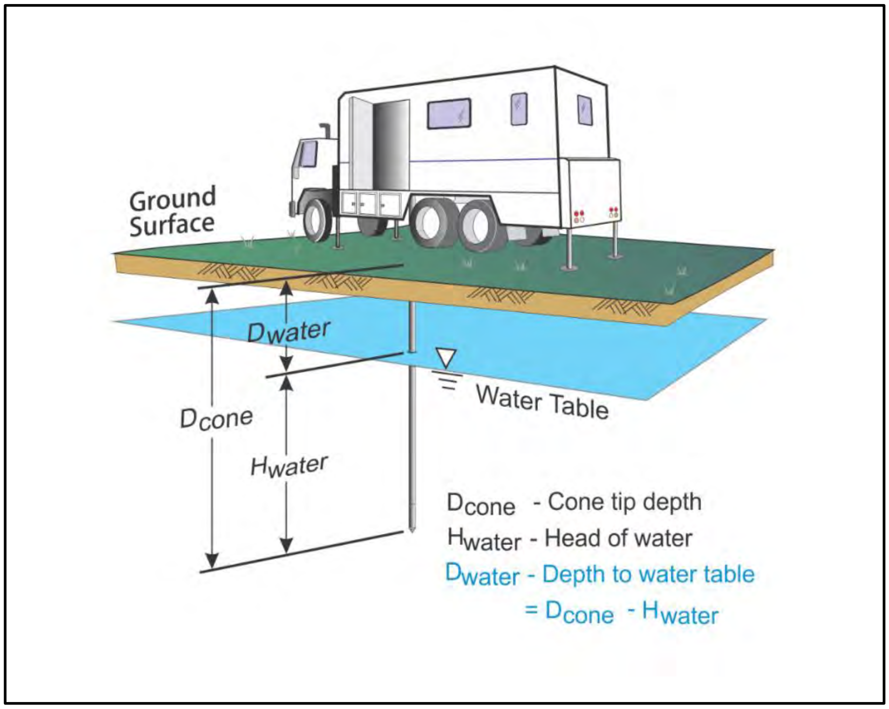


Figure PPD-1. Pore pressure dissipation test setup

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.

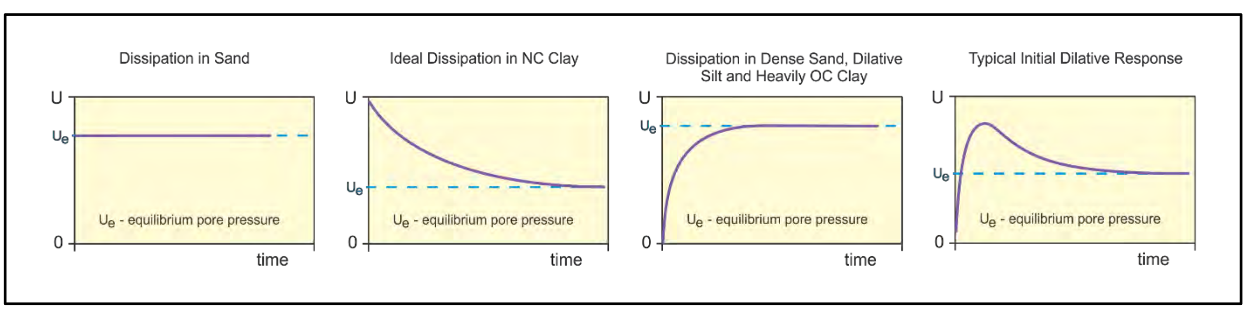


Figure PPD-2. Pore pressure dissipation curve examples

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

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

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.

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

Cone Penetration Test Summary and Standard Cone Penetration Test Plots

CONETEC

Job No:	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	

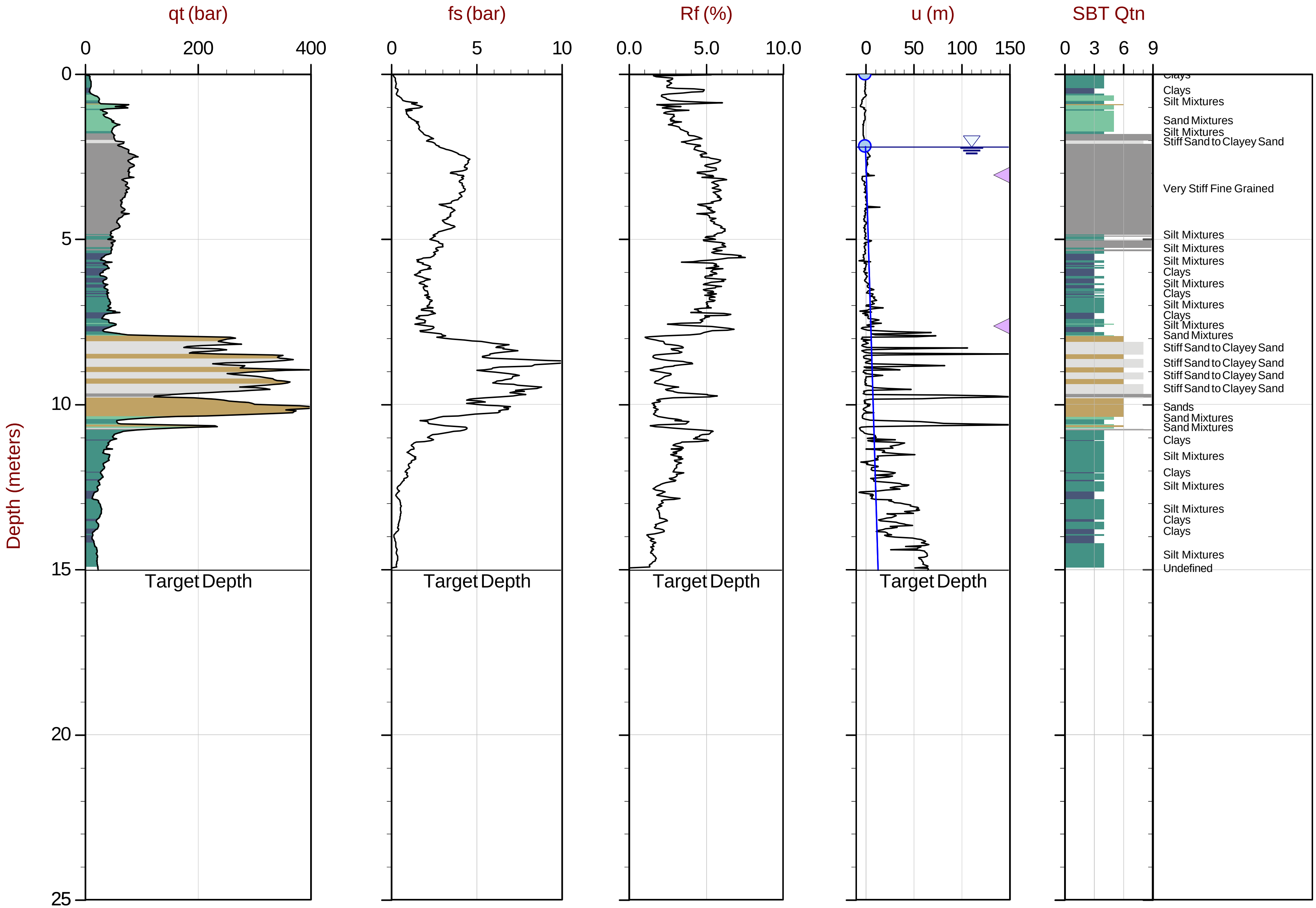
1. The assumed phreatic surface was based on the dynamic pore pressure response, unless otherwise noted. Hydrostatic conditions were assumed for the calculated parameters.
2. Coordinates were collected with a consumer grade GPS device with datum WGS84/UTM Zone 17 North.
3. The assumed phreatic surface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

CONETEC

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: UTM 17N N: 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

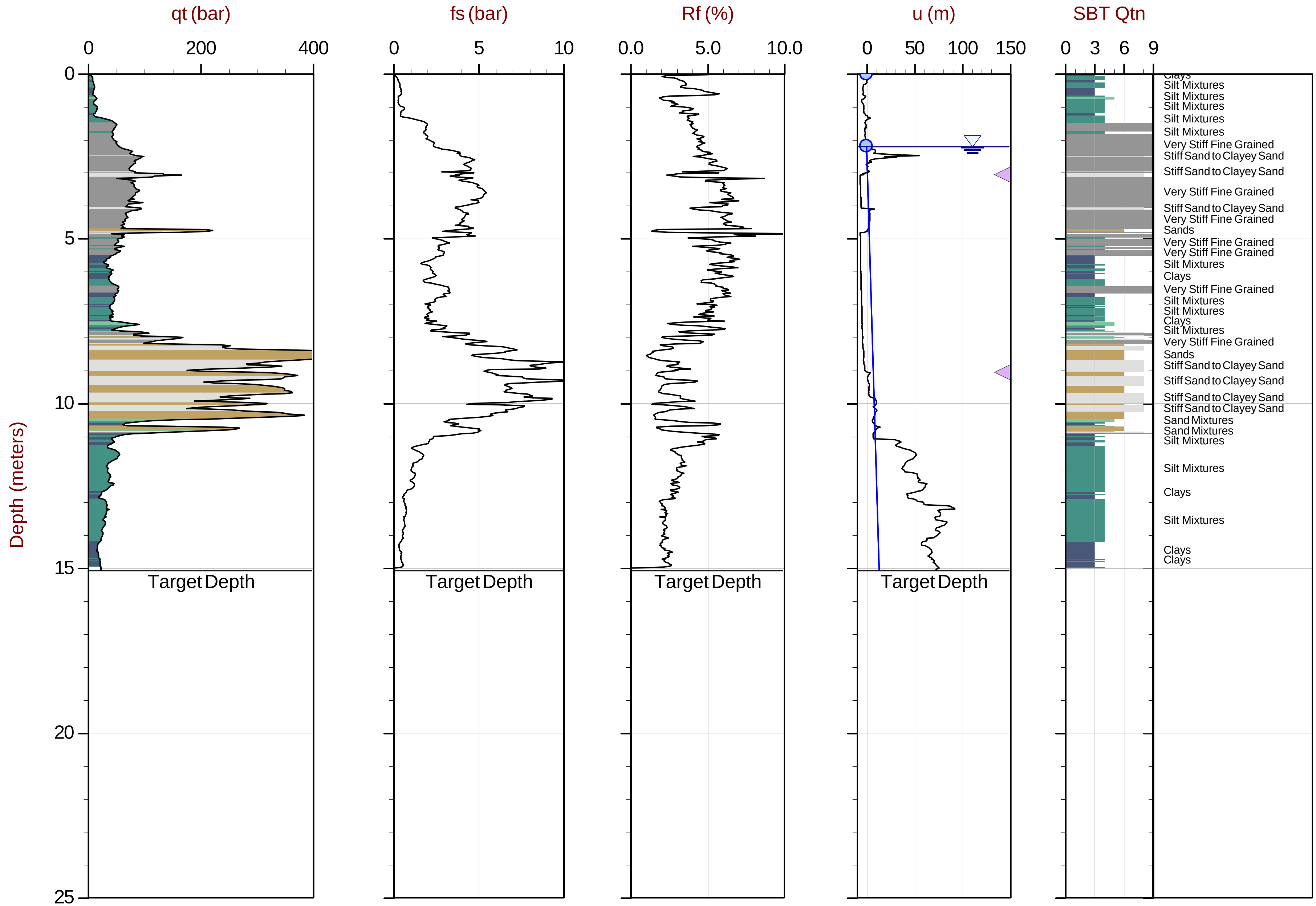
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

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: UTM 17N N: 4739737m E: 481294m
Sheet No: 1 of 1

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

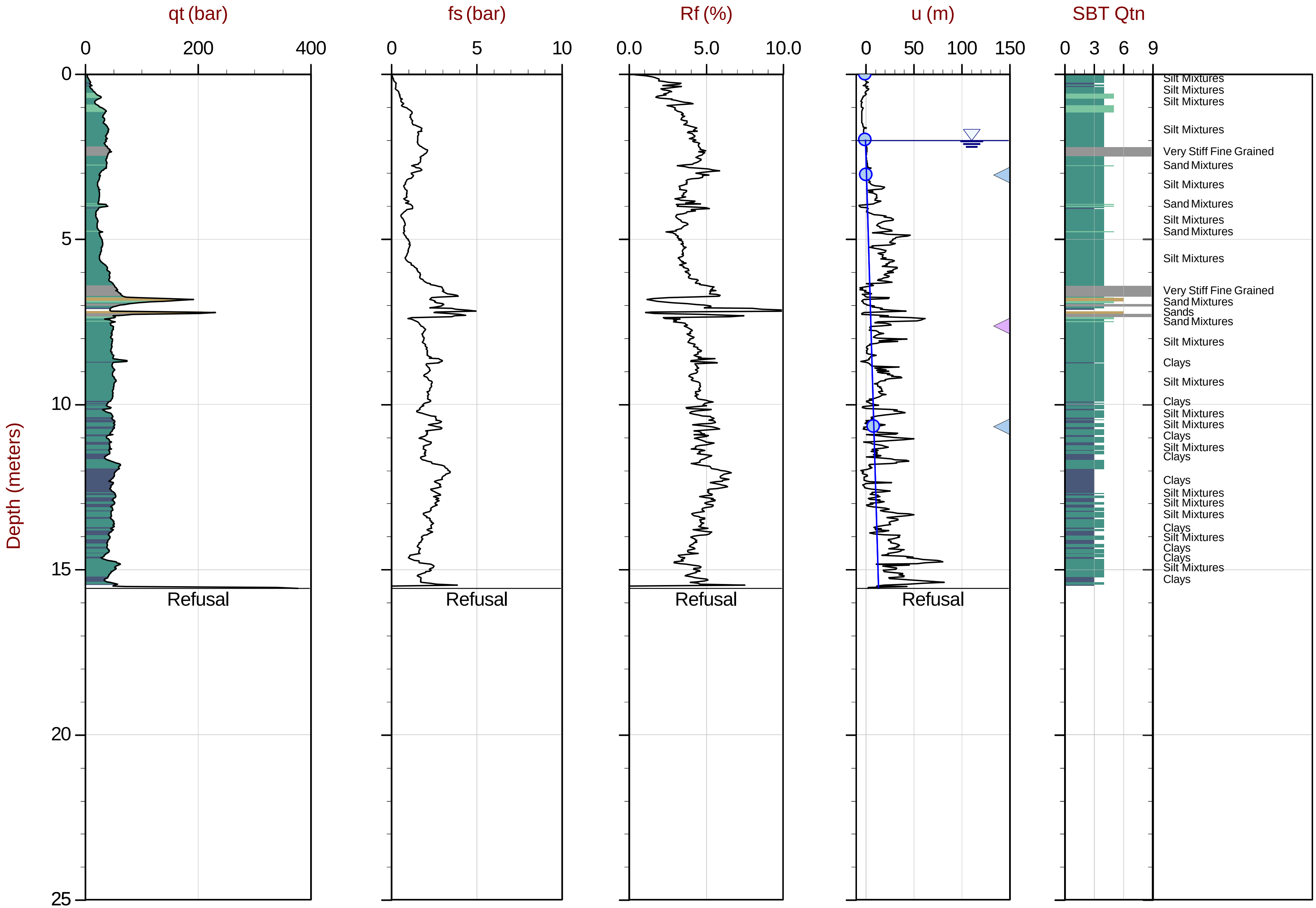
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

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: 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

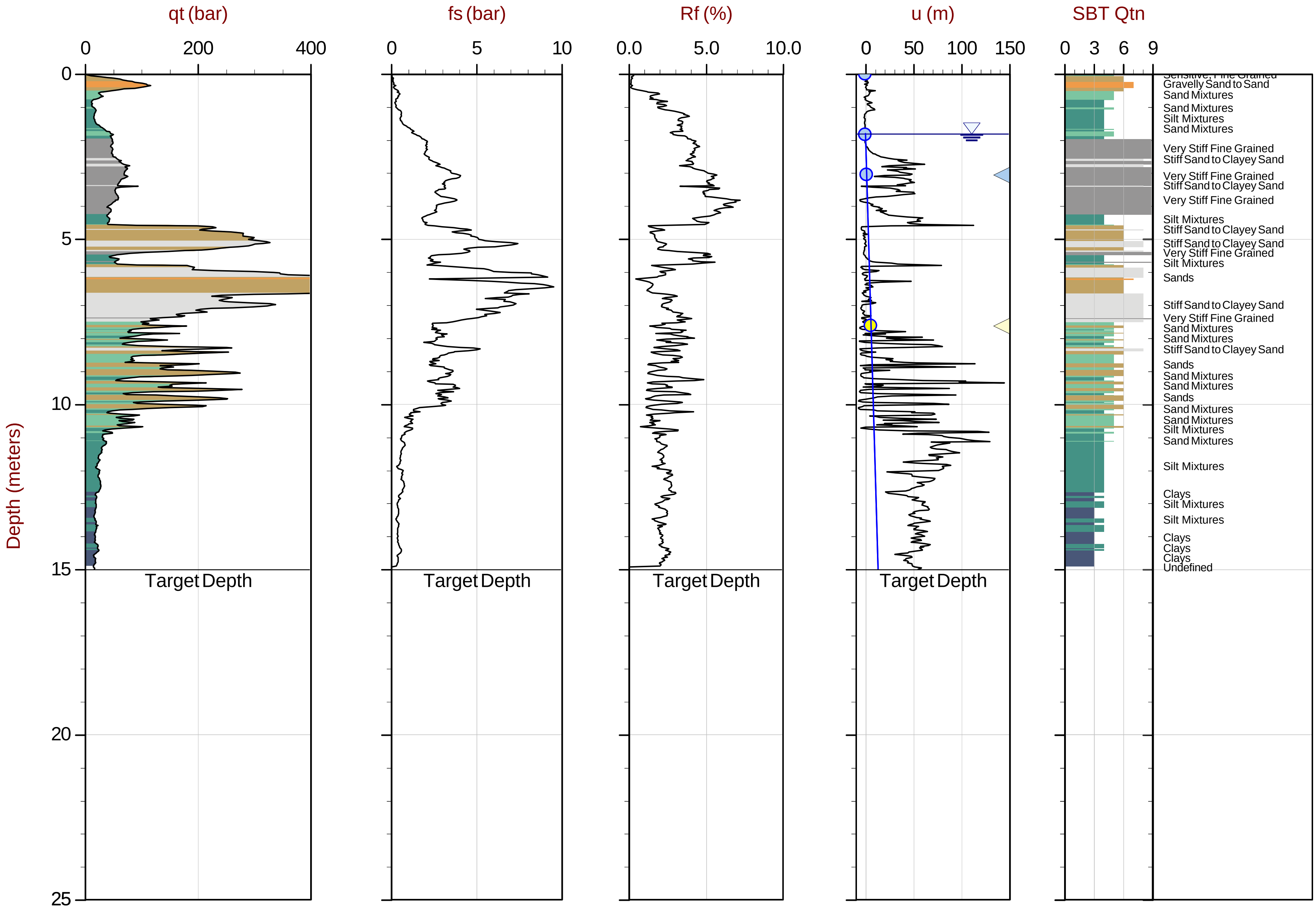
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

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: UTM 17N N: 4738905m E: 482817m
Sheet No: 1 of 1

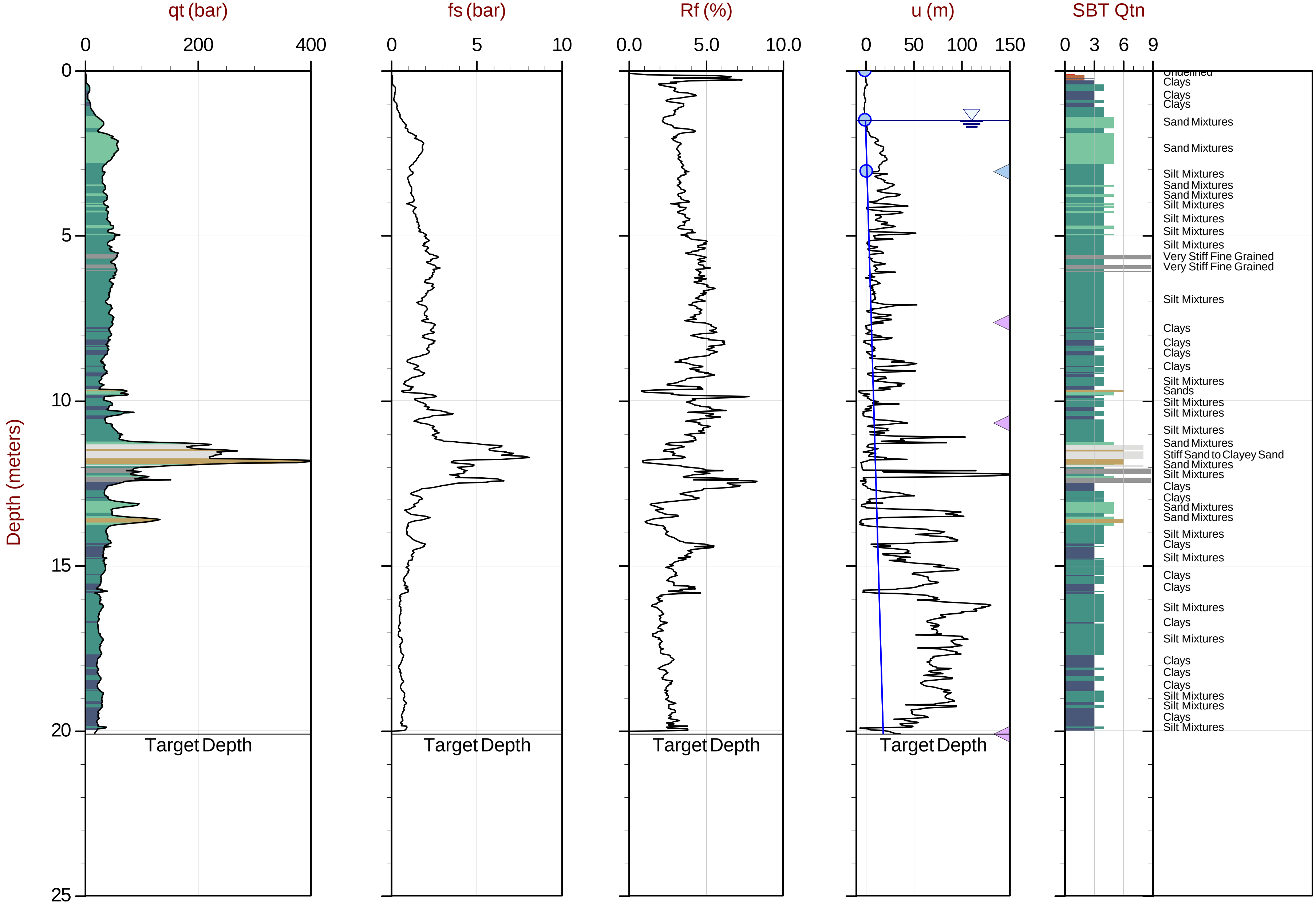
Overplot Item: Ueq   Assumed Ueq   Dissipation, Ueq achieved   Dissipation, Ueq not achieved   Dissipation, Ueq assumed   Hydrostatic Line

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: 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

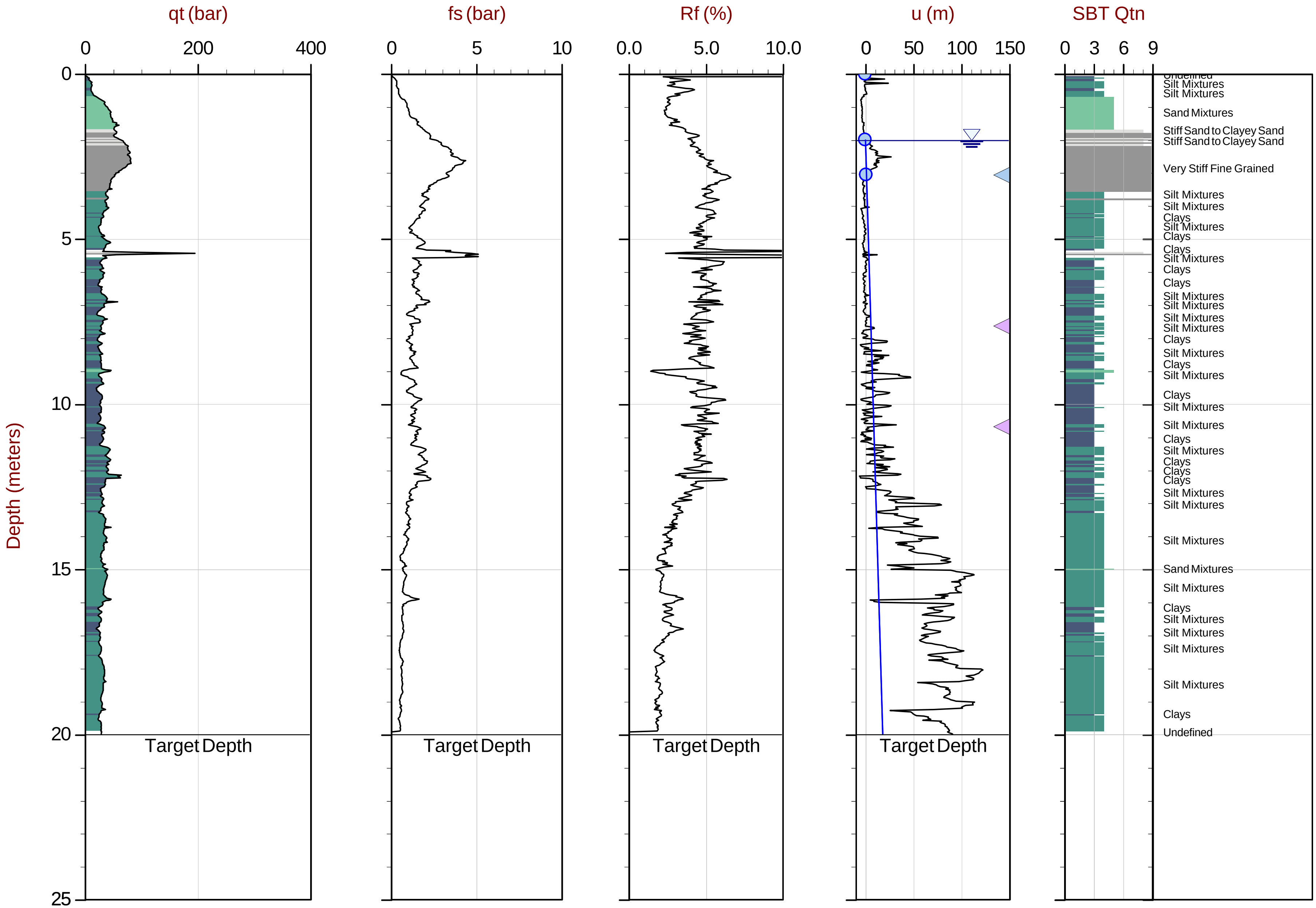
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

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: Every Point

File: 24-05-27609_SP-CN-10.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 4739960m E: 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.

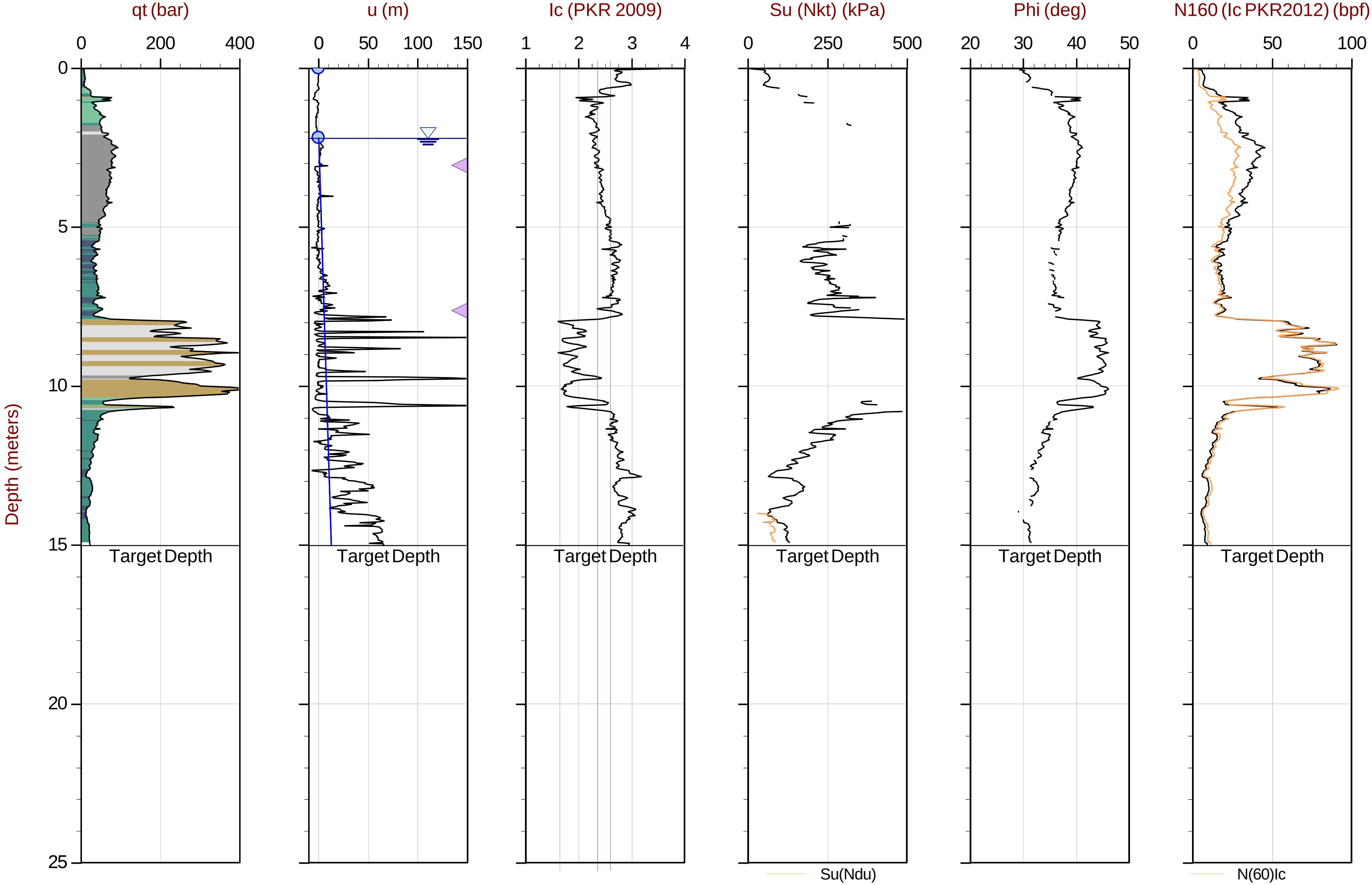
Advanced Cone Penetration Test Plots

CONETEC

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)
Su Nkt/Ndu: 15.0 / 6.0

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

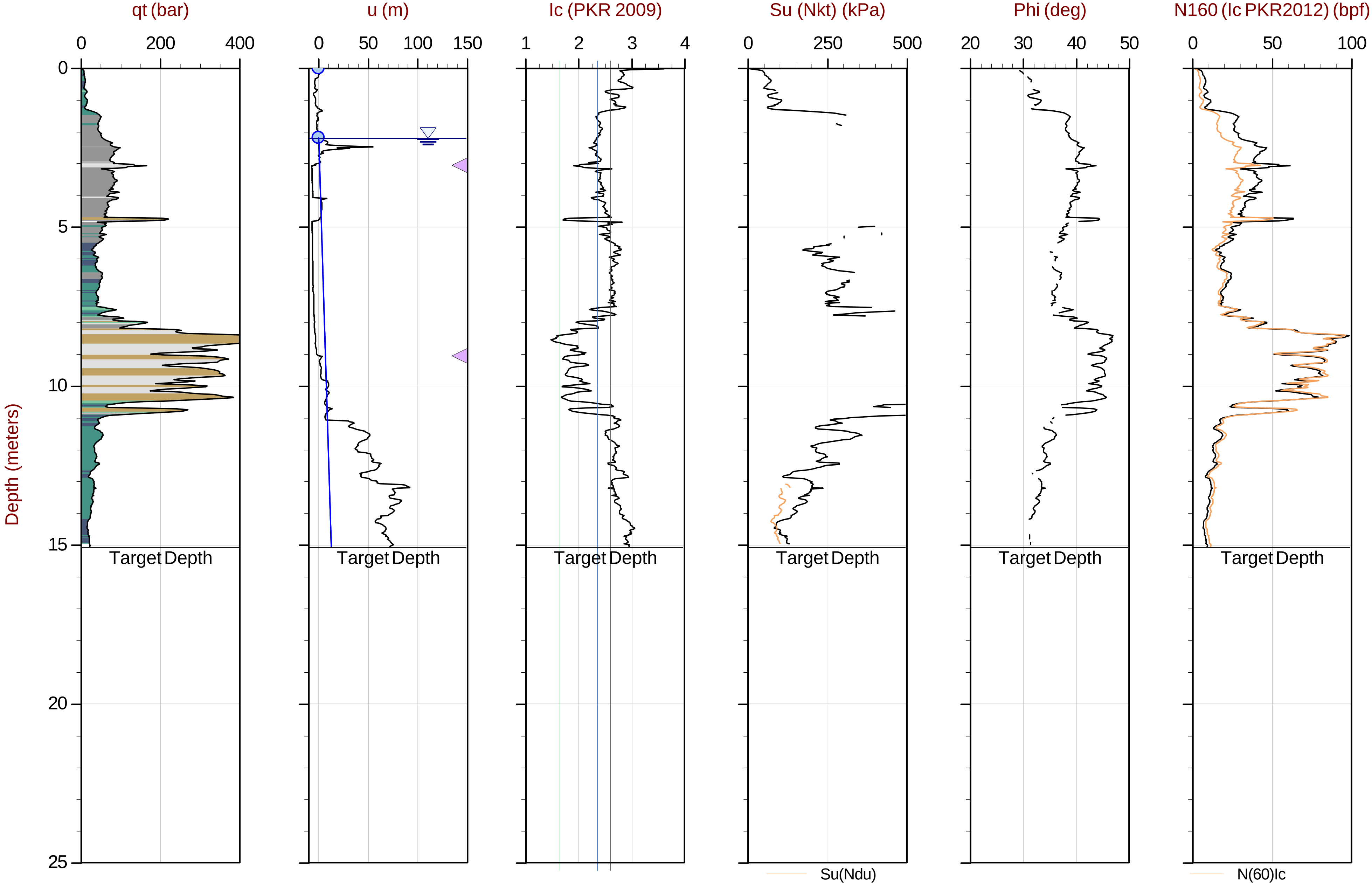
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

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: UTM 17N N: 4739737m E: 481294m
Sheet No: 1 of 1

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

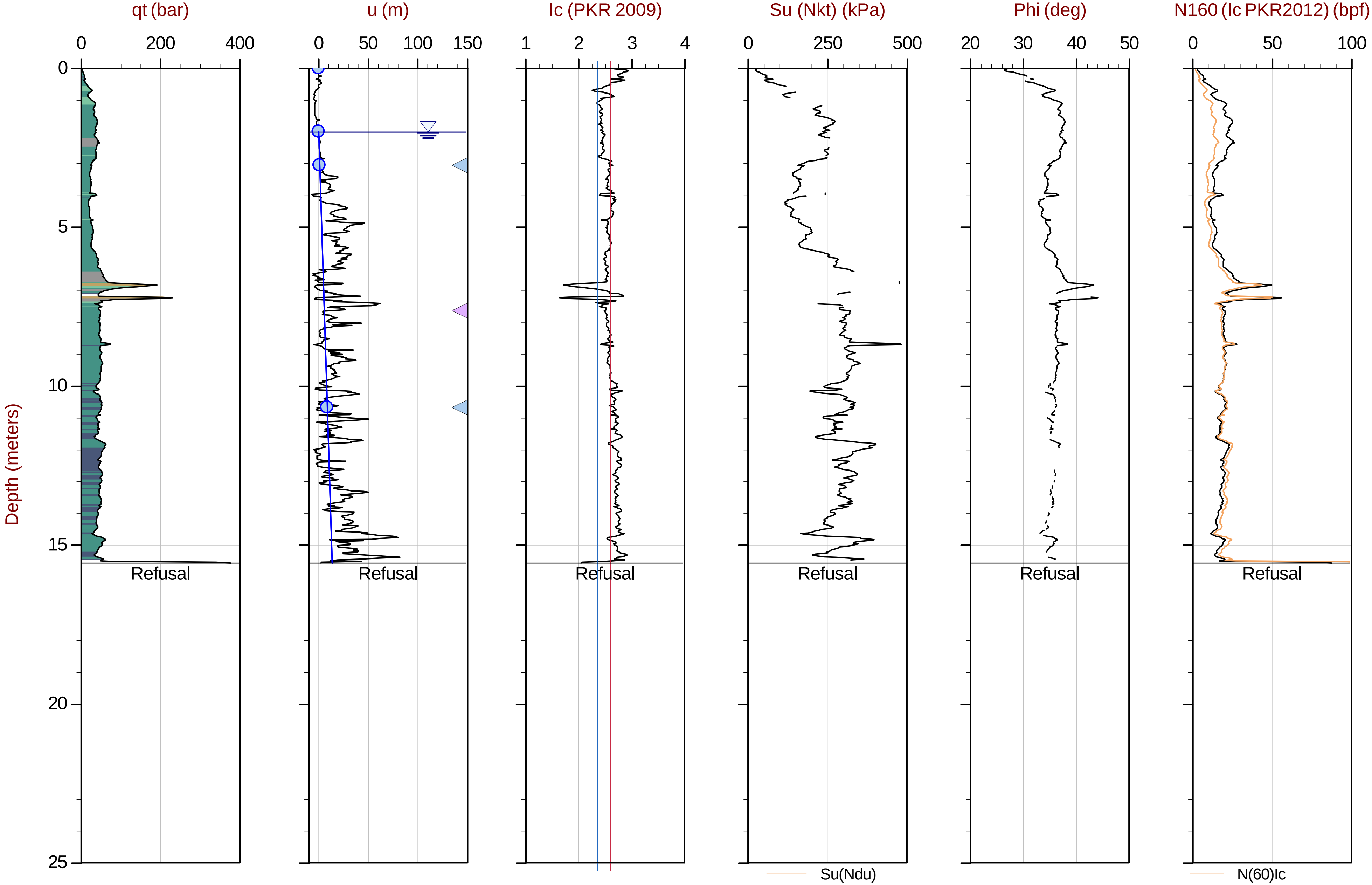
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

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)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 4740267m E: 480674m
Sheet No: 1 of 1

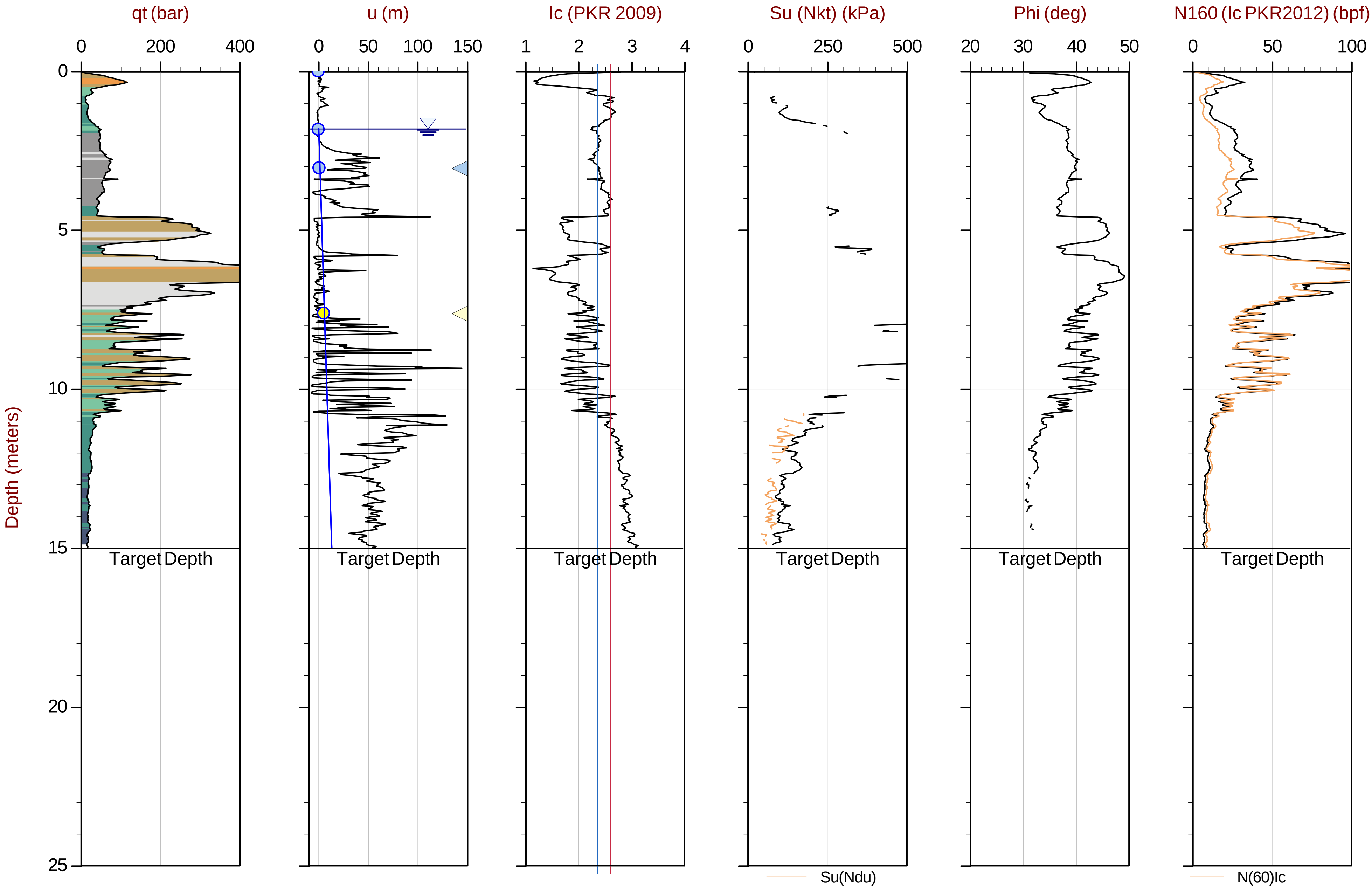
Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Hydrostatic Line

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

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)
Su Nkt/Ndu: 15.0 / 6.0

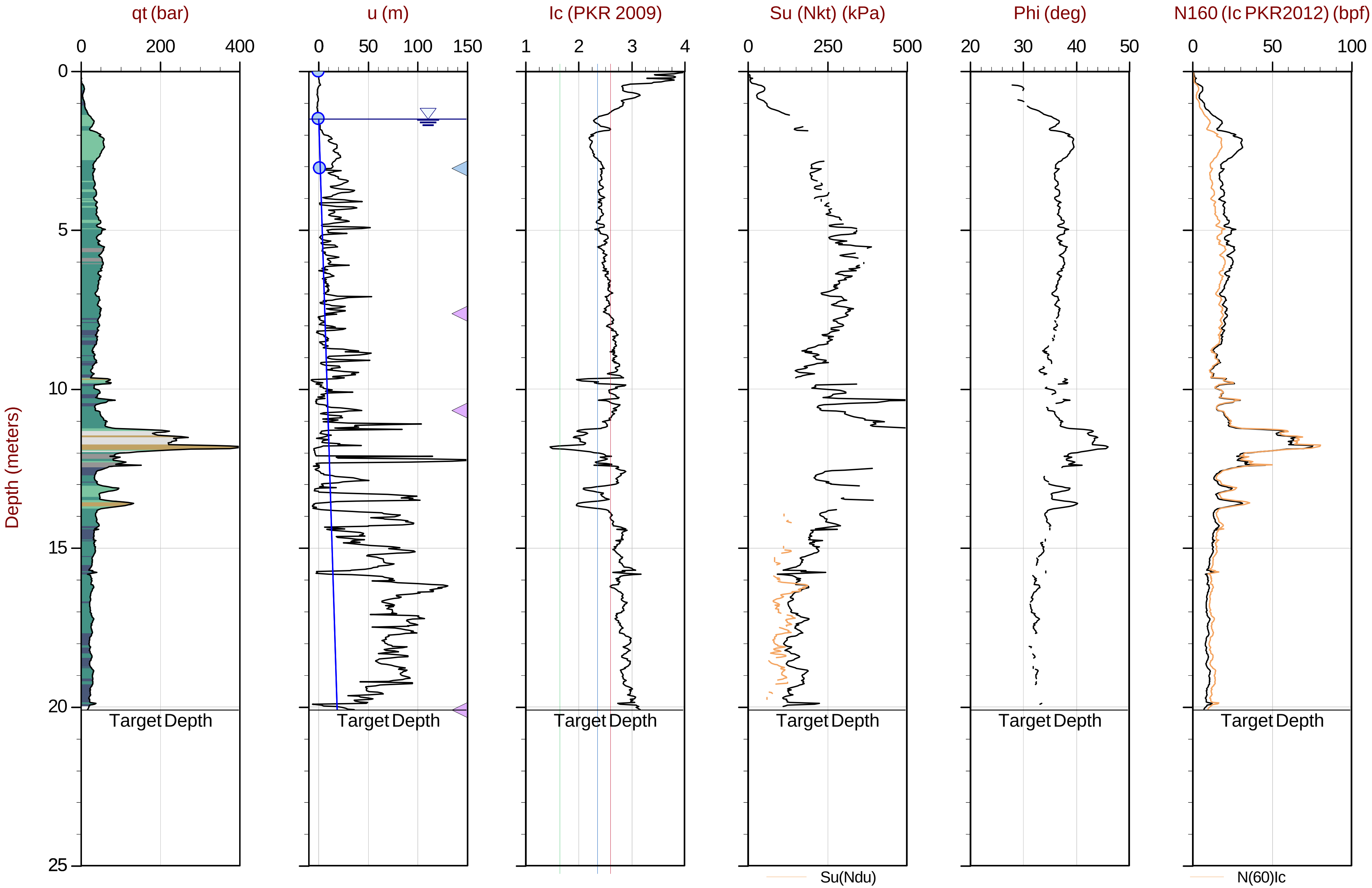
SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 4738905m E: 482817m
Sheet No: 1 of 1

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.

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

Stantec



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)
Su Nkt/Ndu: 15.0 / 6.0

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

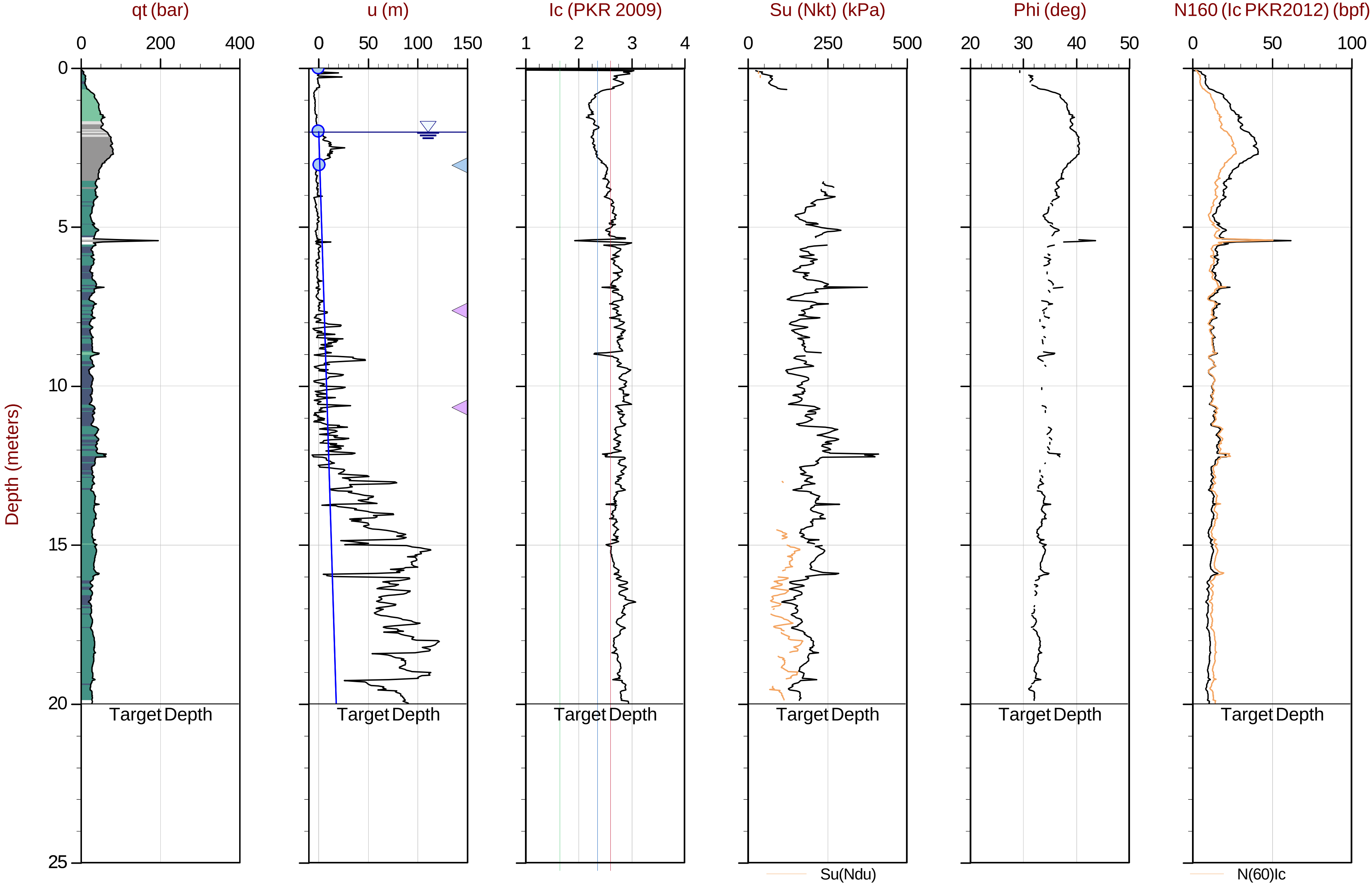
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

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: Every Point

File: 24-05-27609_SP-CN-10.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N:4739960m E: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 Plots

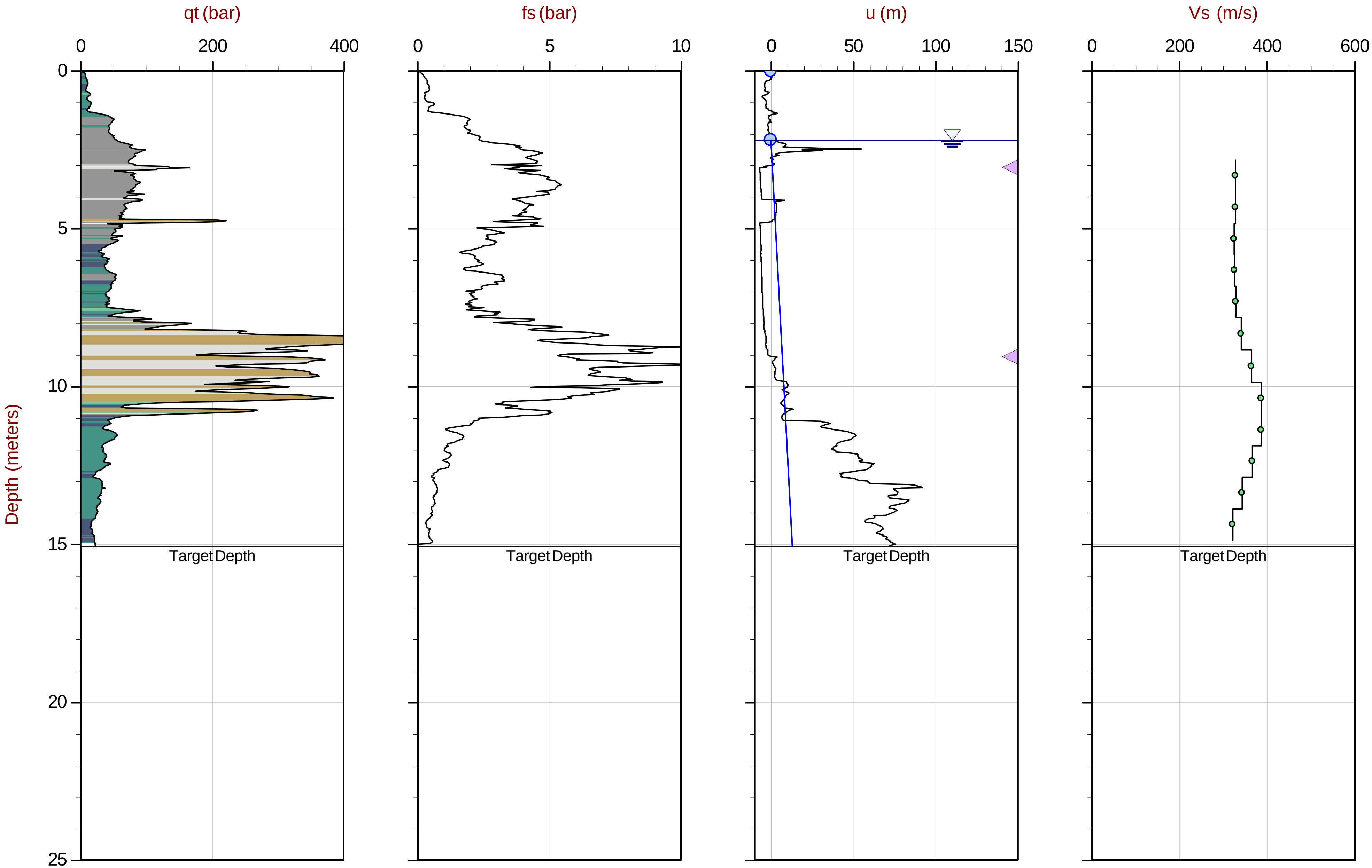
ConeTec

ConeTec

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: UTM 17N N: 4739737m E: 481294m
Sheet No: 1 of 1

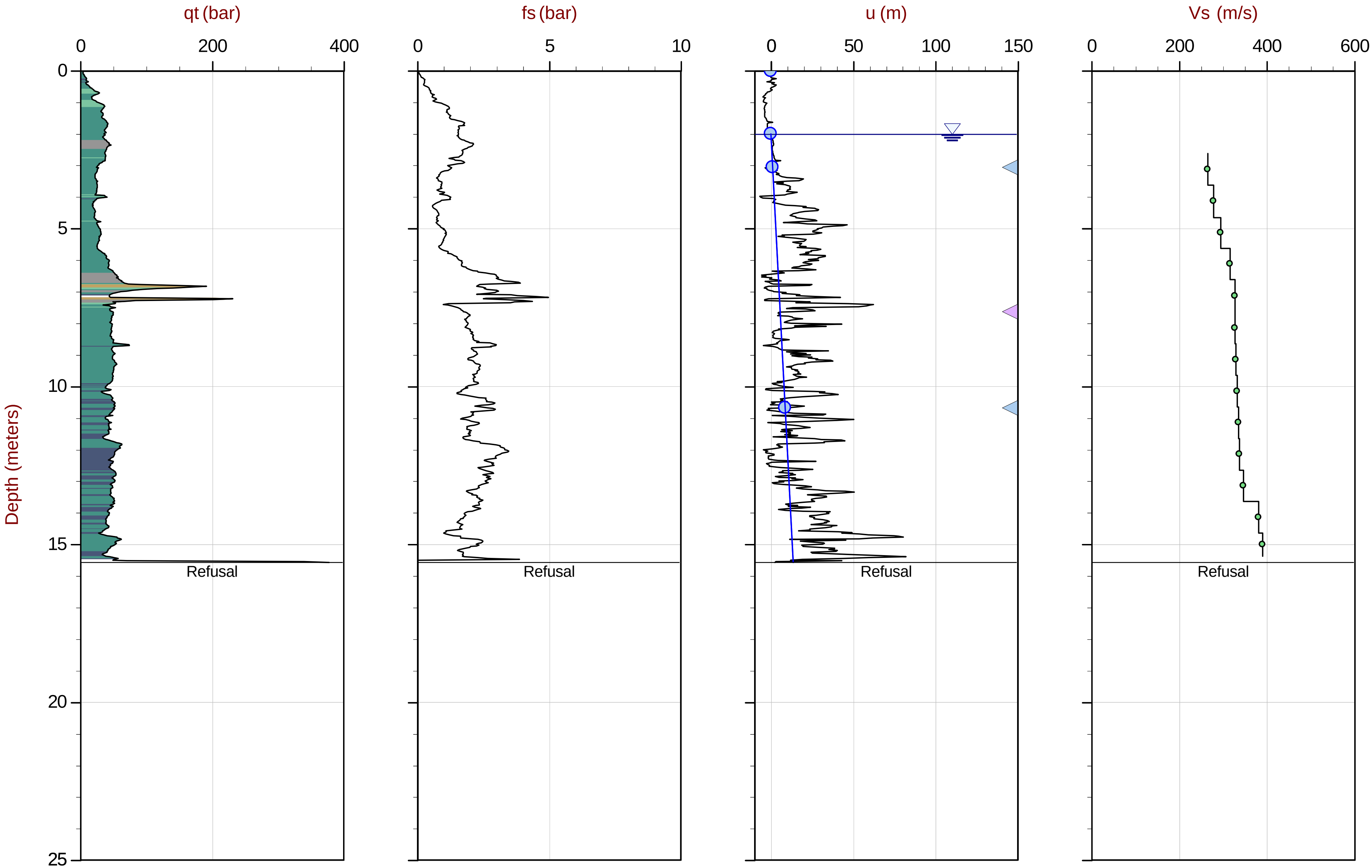
Overplot Item: Ueq   Assumed Ueq   Dissipation, Ueq achieved   Dissipation, Ueq not achieved   Dissipation, Ueq assumed   Hydrostatic Line

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

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: UTM 17N N: 4740267m E: 480674m
Sheet No: 1 of 1

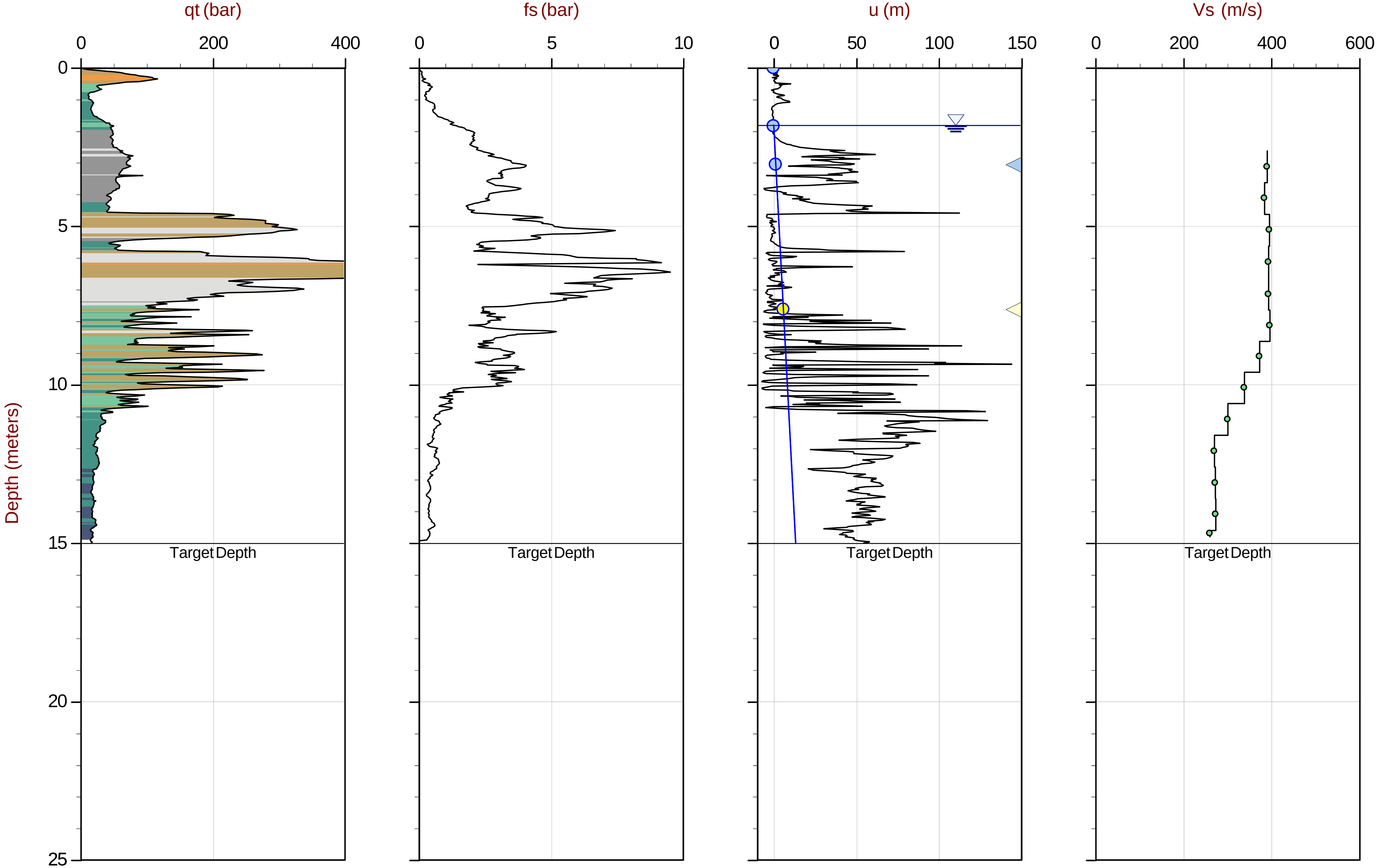
Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Hydrostatic Line

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

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: UTM 17N 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

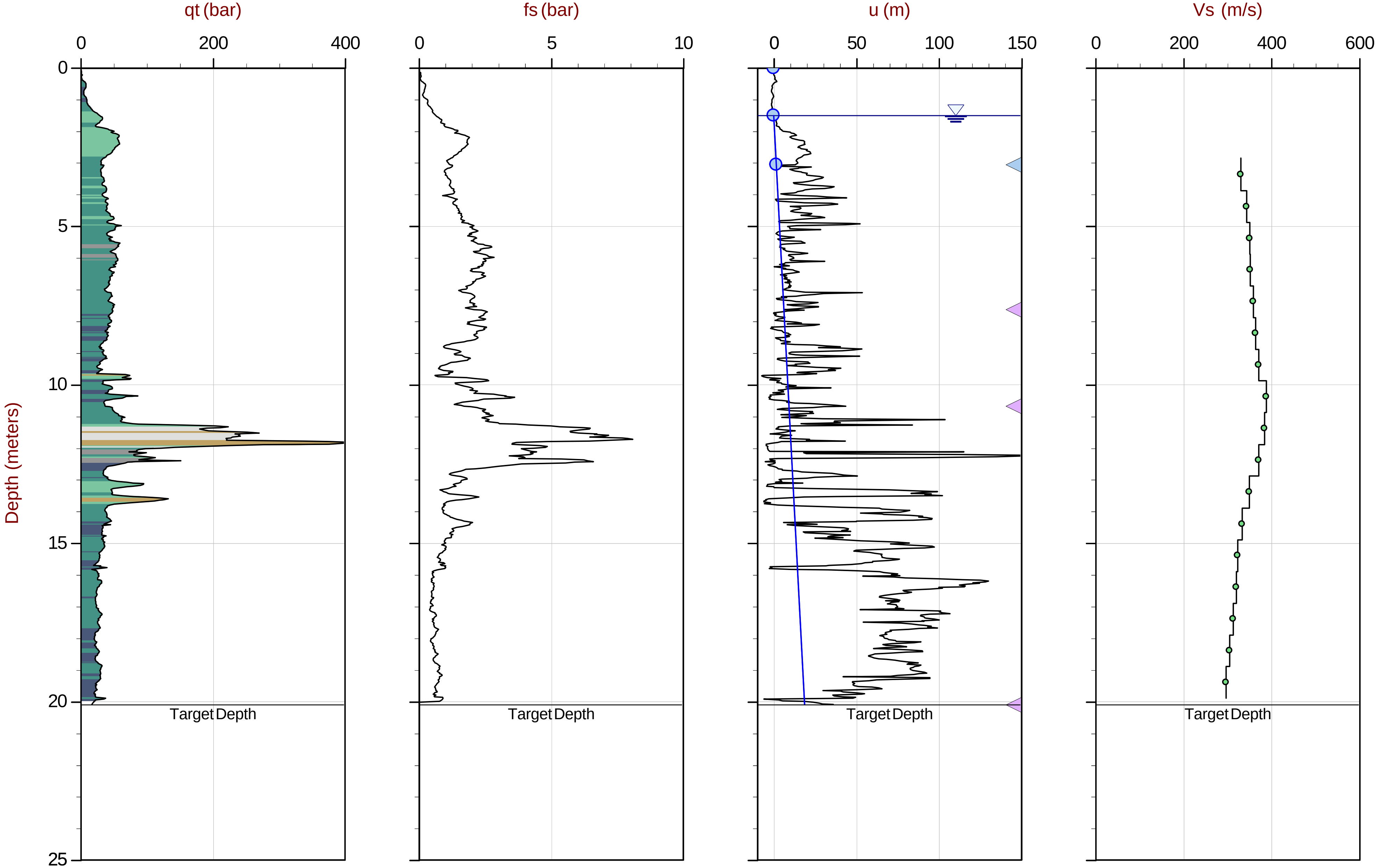
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

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: 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

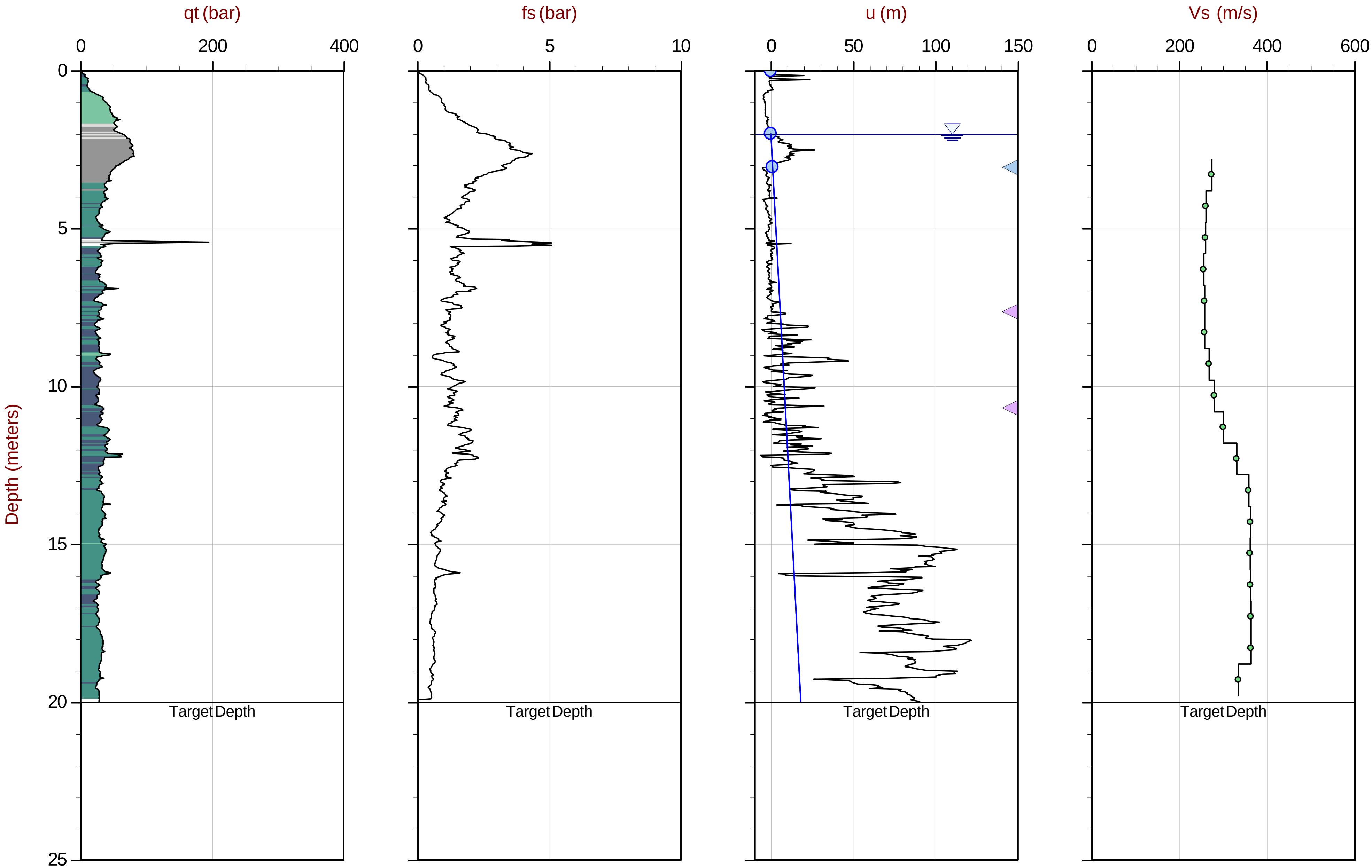
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

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: Every Point

File: 24-05-27609_SP-CN-10.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 4739960m E: 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 (Vs) 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

SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
3.03	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

SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
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

SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
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

SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
3.05	2.85	4.29			
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

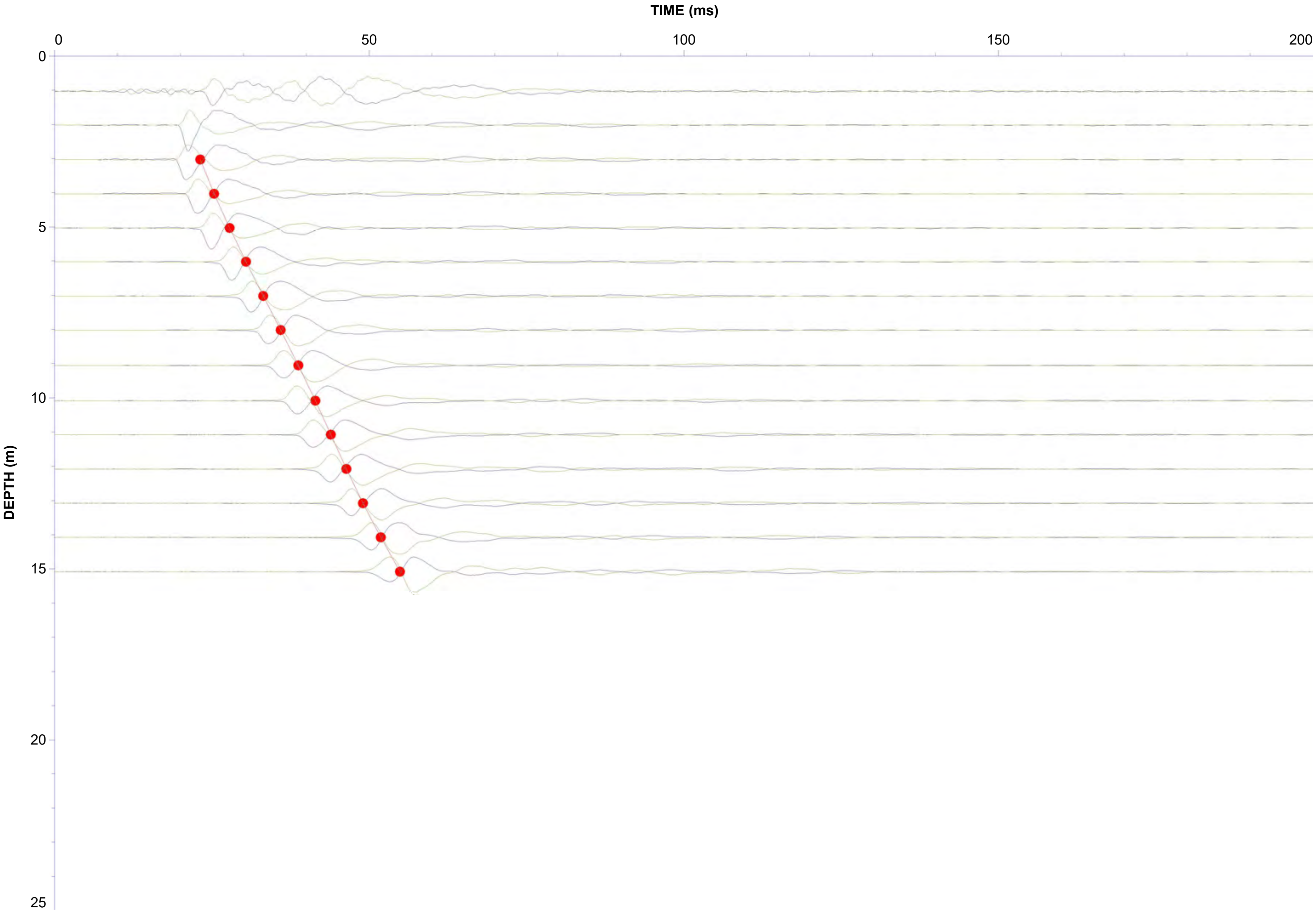
Seismic Source:	Beam
Seismic Offset (m):	3.20
Source Depth (m):	0.00
Geophone Offset (m):	0.20

SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

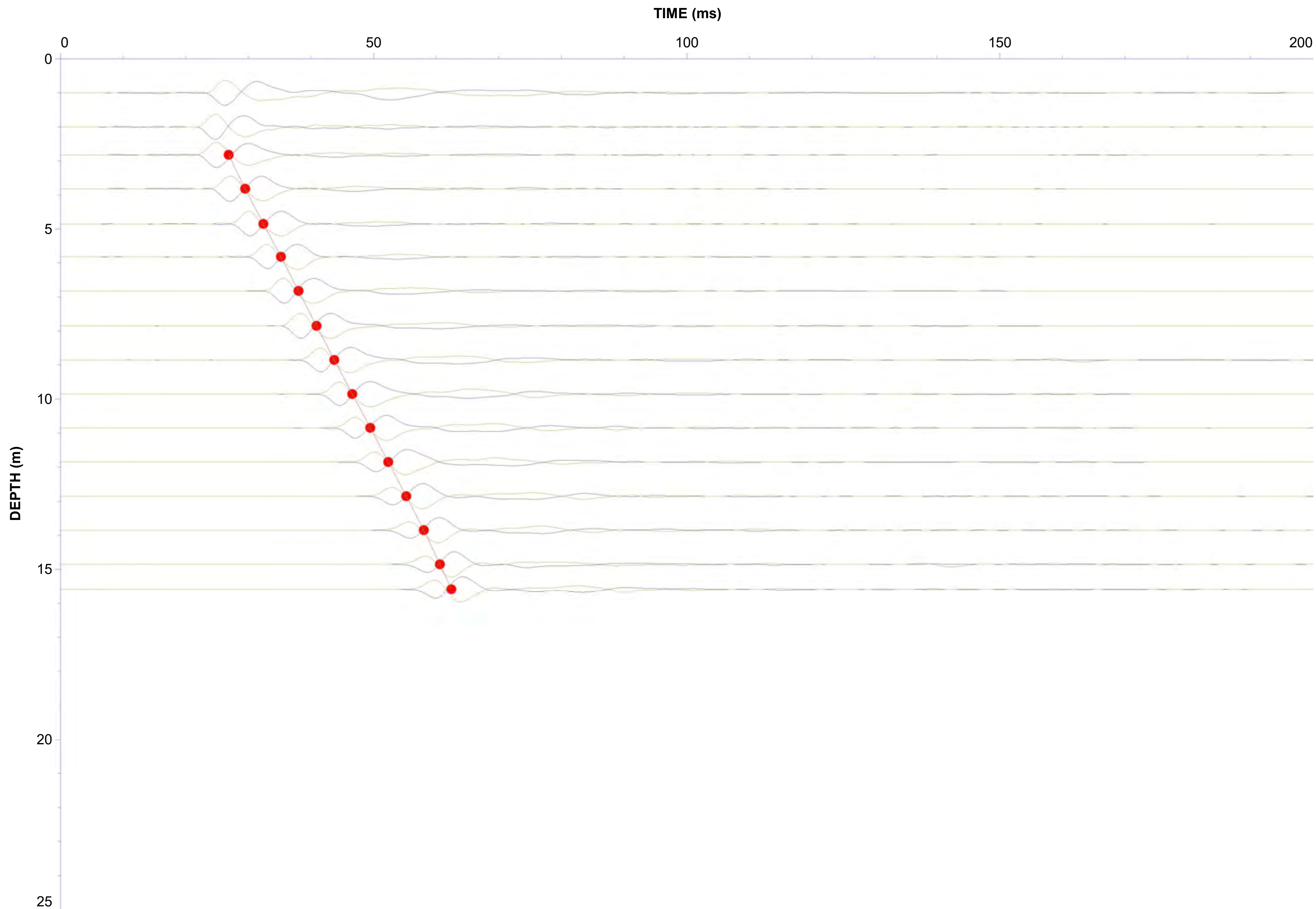
Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
3.00	2.80	4.25			
4.00	3.80	4.97	0.72	2.61	275
5.00	4.80	5.77	0.80	3.07	261
6.00	5.80	6.62	0.86	3.29	260
7.00	6.80	7.52	0.89	3.48	256
8.00	7.80	8.43	0.92	3.55	258
9.00	8.80	9.36	0.93	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 (Vs) Traces

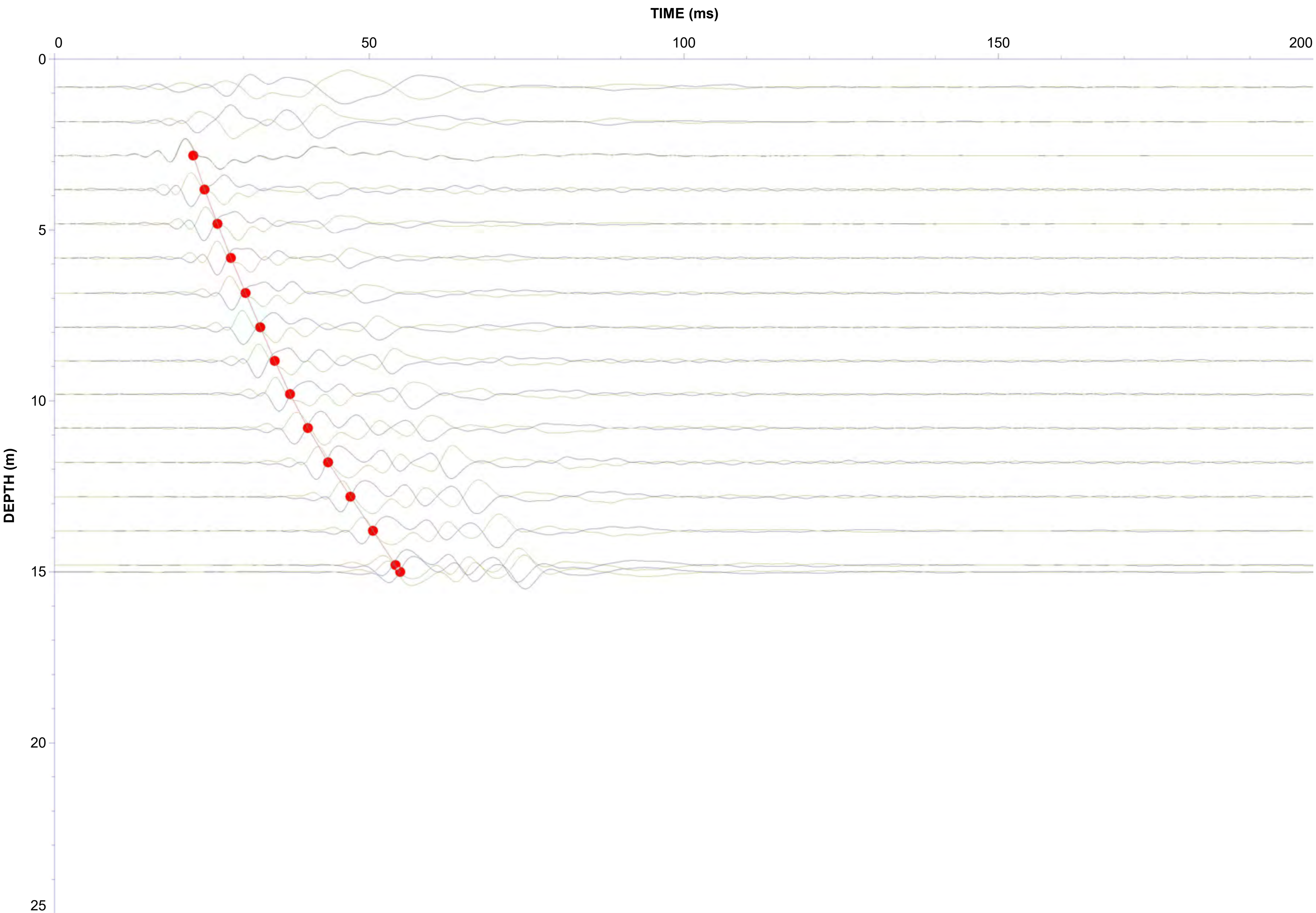
Job No: 24-05-27609 Client: Stantec Project: HWY 3 St Thomas CPT Sounding: SCPT24-RMNAPP01 Filter: None Date: 2024-05-10
Analysis: S Wave - Geo X



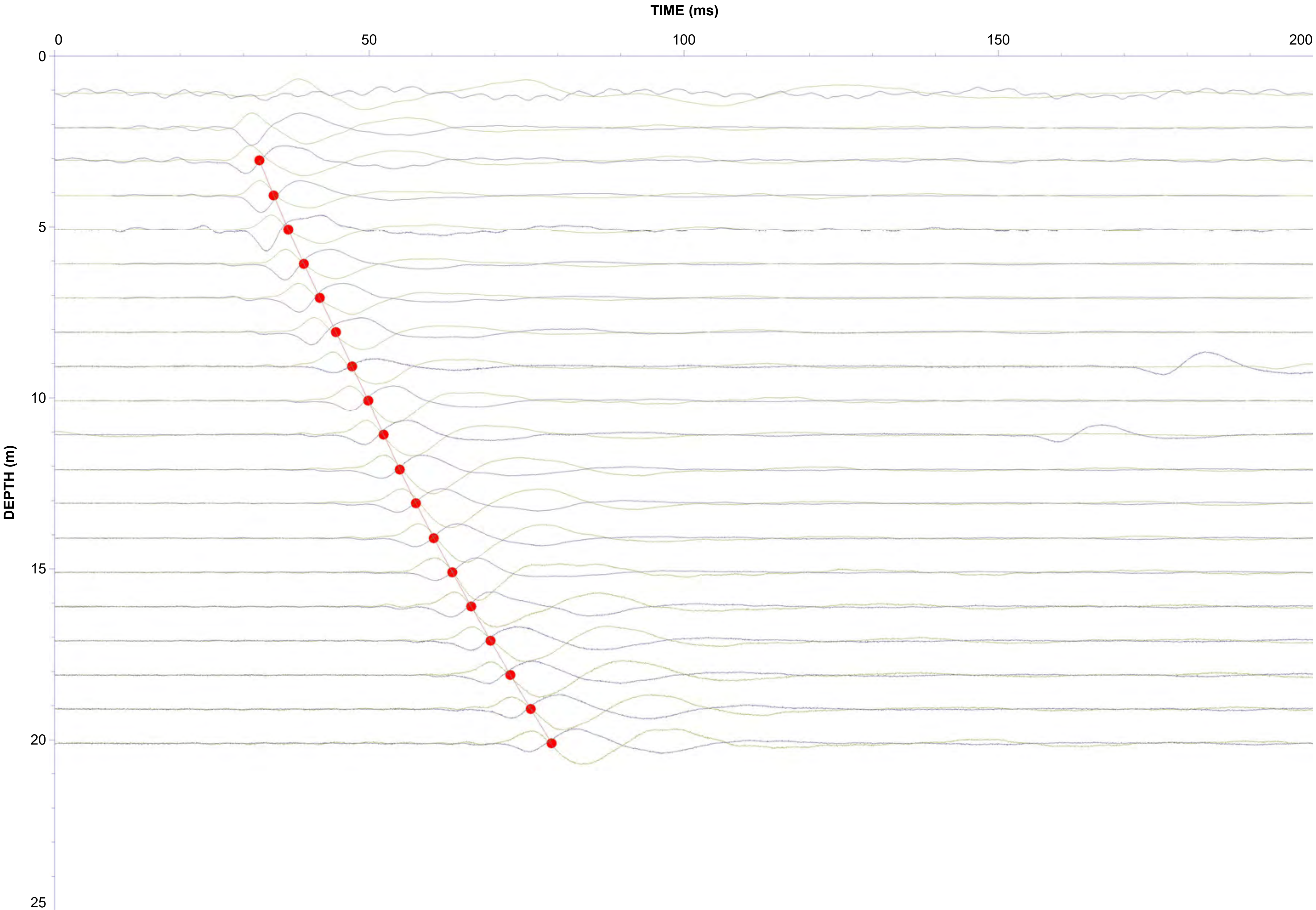
Job No: 24-05-27609 Client: Stantec Project: HWY 3 St Thomas CPT Sounding: SCPT24-CNREMB02 Filter: None Date: 2024-05-09
Analysis: S Wave - Geo X



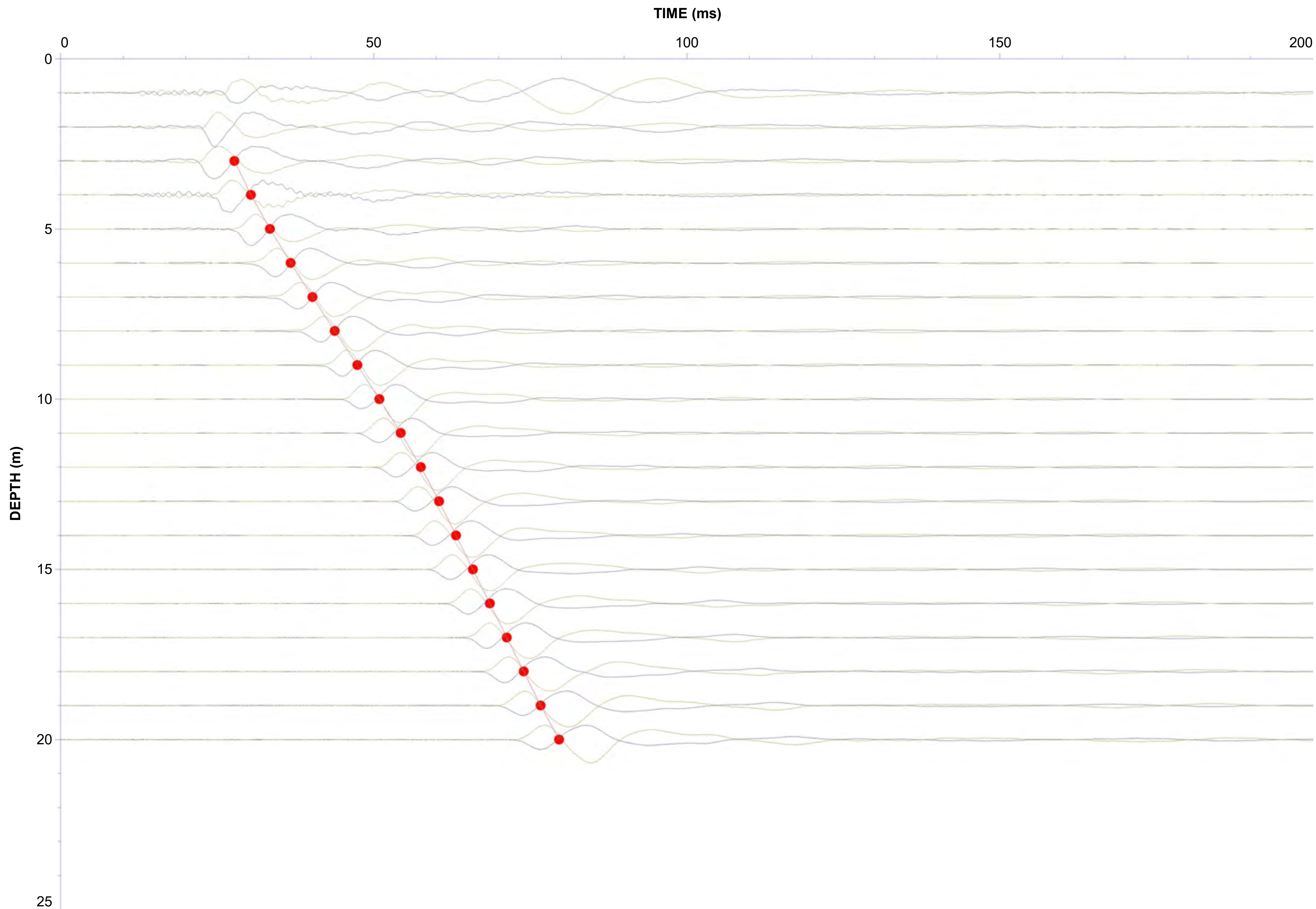
Job No: 24-05-27609 Client: Stantec Project: HWY 3 St Thomas CPT Sounding: SCPT24-WAPP02 Filter: 0-300Hz Date: 2024-05-10
Analysis: S Wave - Geo X



Job No: 24-05-27609 Client: Stantec Project: HWY 3 St Thomas CPT Sounding: SCPT24-CNREMB08 Filter: None Date: 2024-05-09
Analysis: S Wave - Geo X



Job No: 24-05-27609 Client: Stantec Project: HWY 3 St Thomas CPT Sounding: SCPT24-CNREMB10 Filter: None Date: 2024-05-10
Analysis: S Wave - Geo X



Soil Behaviour Type (SBT) Scatter Plots

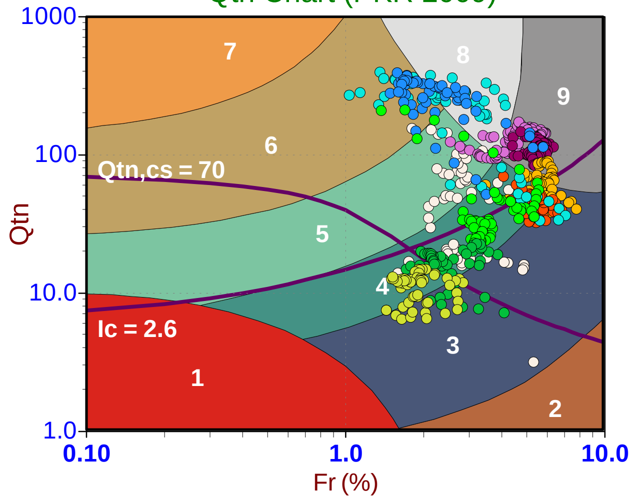
CONETEC

Stantec

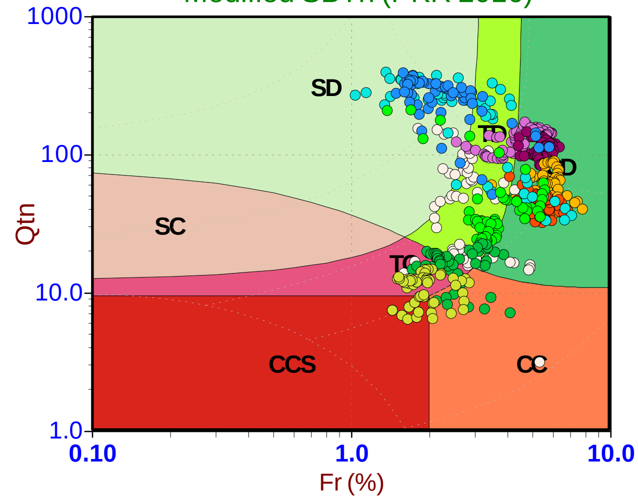
Job No: 24-05-27609
Date: 2024-05-10 12:15
Site: HWY 3, St.Thomas, ON

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

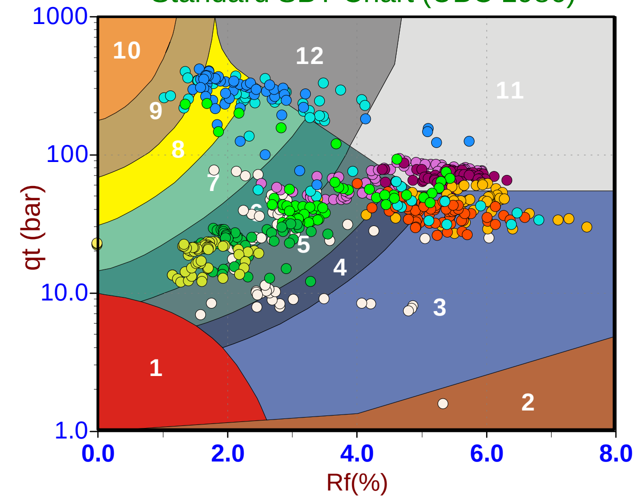
Qtn Chart (PKR 2009)



Modified SBTn (PKR 2016)



Standard SBT Chart (UBC 1986)



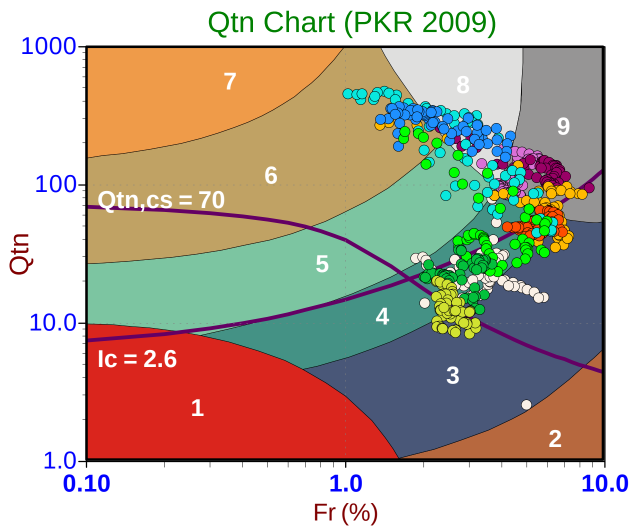
Depth Ranges
- >0.0 to 1.5 m
- >1.5 to 3.0 m
- >3.0 to 4.5 m
- >4.5 to 6.0 m
- >6.0 to 7.5 m
- >7.5 to 9.0 m
- >9.0 to 10.5 m
- >10.5 to 12.0 m
- >12.0 to 13.5 m
- >13.5 to 15.0 m
- >15.0 m

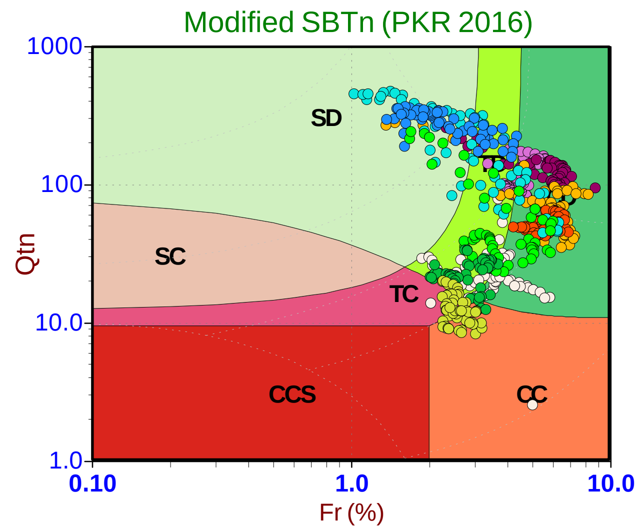
Legend
- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

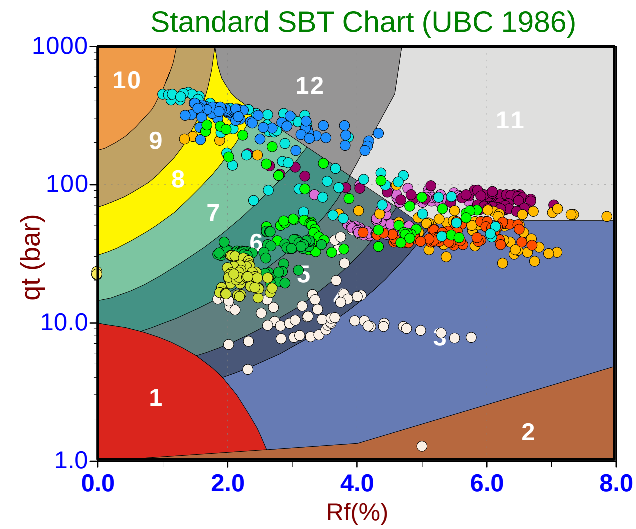
Legend
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

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

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

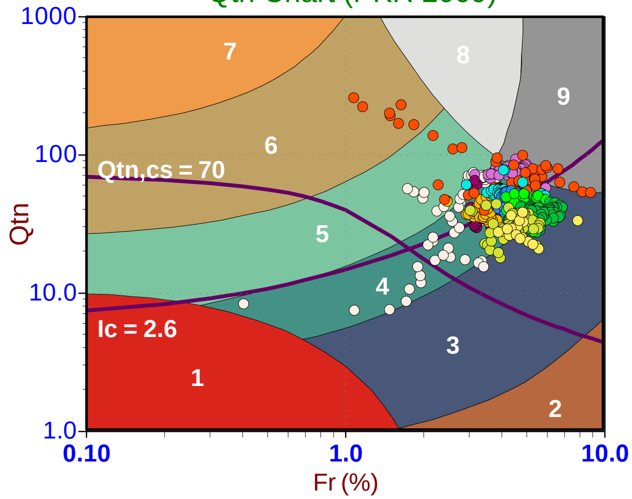
CONETEC

Stantec

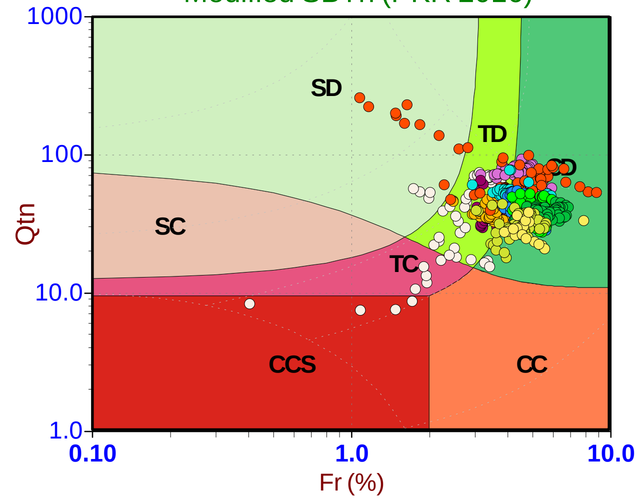
Job No: 24-05-27609
Date: 2024-05-09 12:06
Site: HWY 3, St.Thomas, ON

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

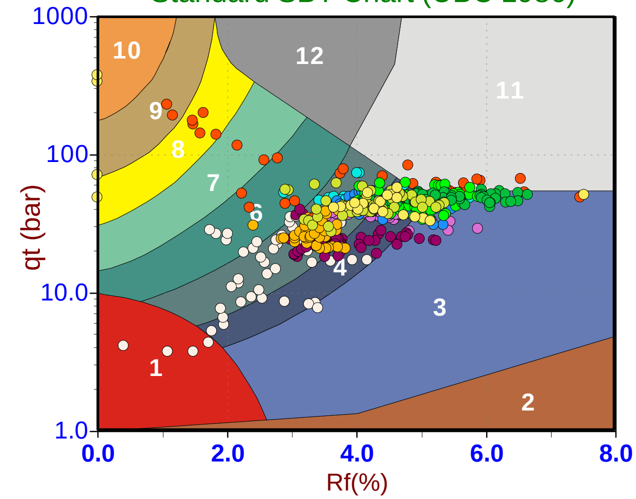
Qtn Chart (PKR 2009)



Modified SBTn (PKR 2016)



Standard SBT Chart (UBC 1986)



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

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

Legend
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
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- CD (Dil. clay like)
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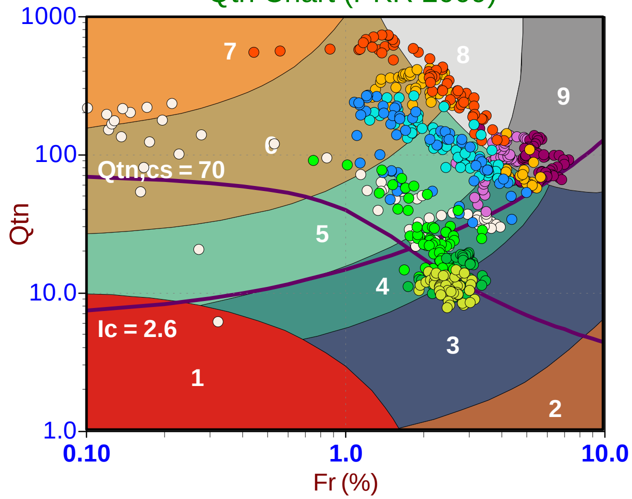
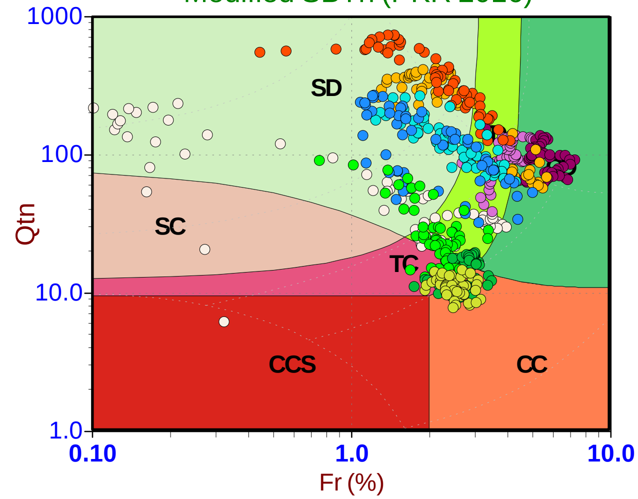
Legend
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

CONETEC	*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

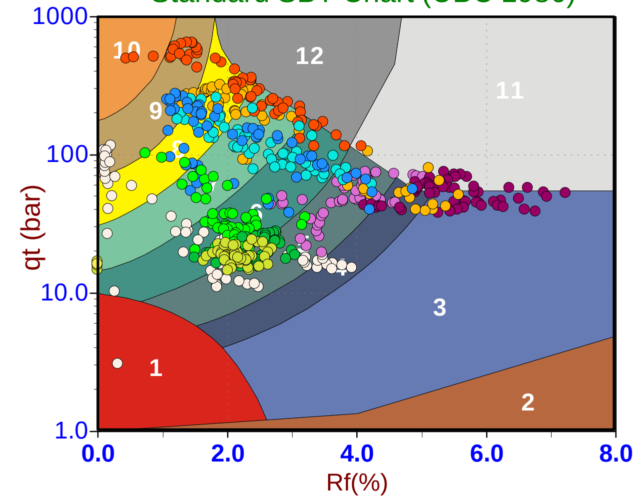
Qtn Chart (PKR 2009)



Modified SBTn (PKR 2016)



Standard SBT Chart (UBC 1986)



Depth Ranges

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

Legend

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

Legend

- CCS (Cont. sensitive clay like)
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Legend

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

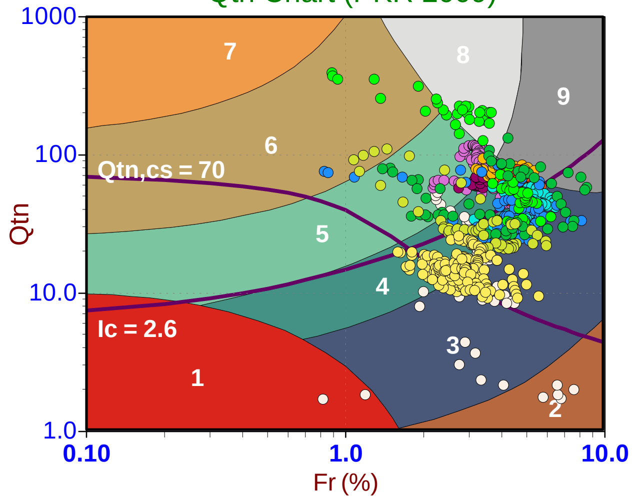
CONETEC

Stantec

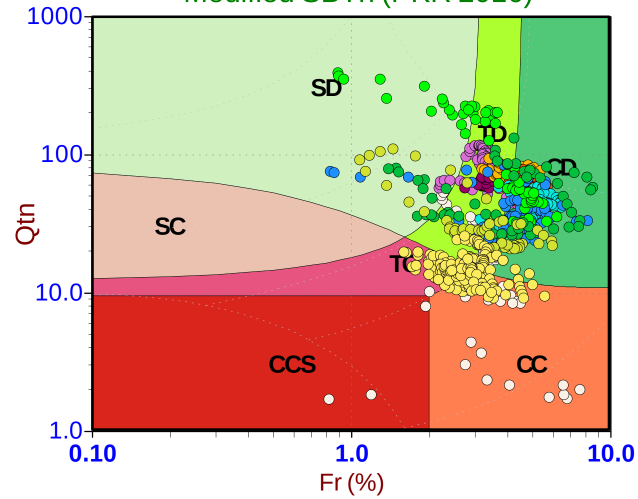
Job No: 24-05-27609
Date: 2024-05-09 16:43
Site: HWY 3, St.Thomas, ON

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

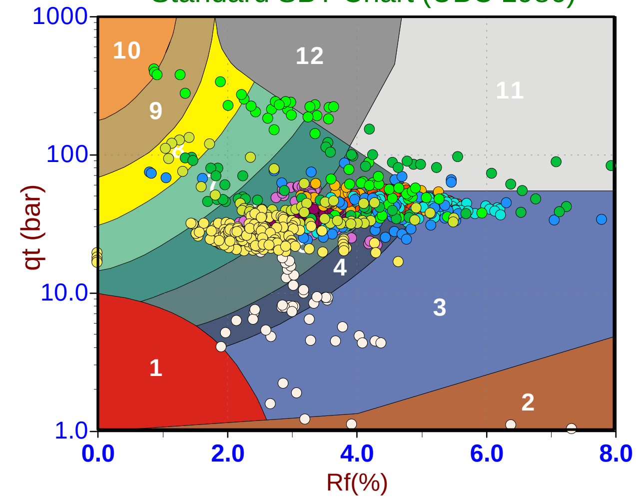
Qtn Chart (PKR 2009)



Modified SBTn (PKR 2016)



Standard SBT Chart (UBC 1986)



Depth Ranges

- >0.0 to 1.5 m
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- >4.5 to 6.0 m
- >6.0 to 7.5 m
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- >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
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Legend

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Legend

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- Clayey Silt
- Silt
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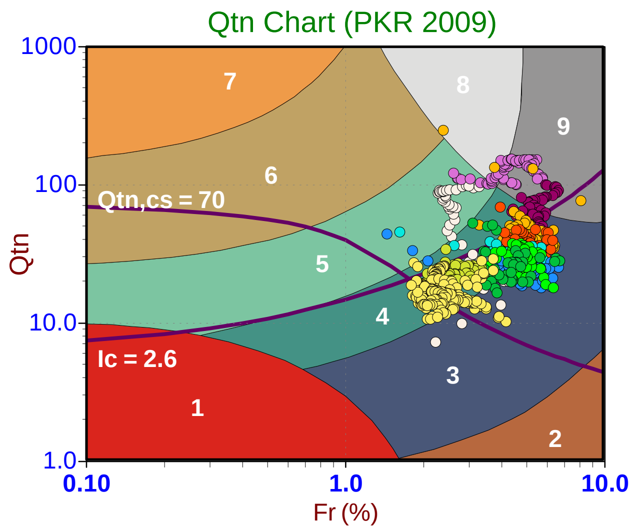
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

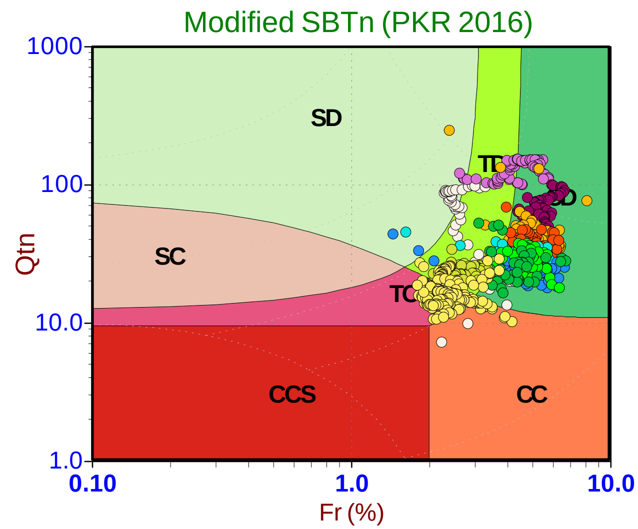
CONETEC

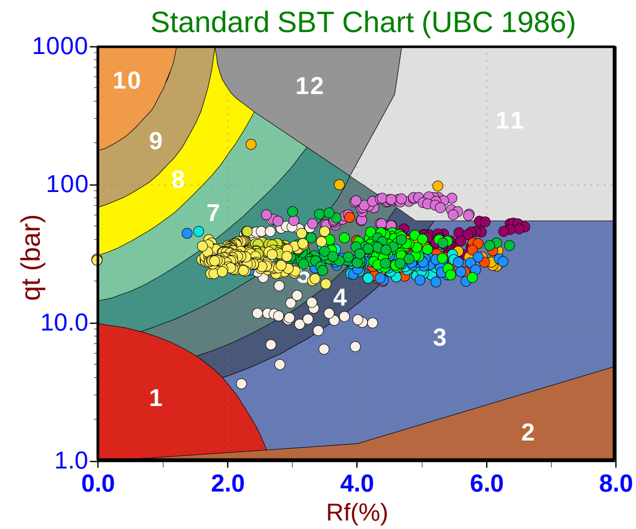
Stantec

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
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- >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
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Legend
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- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

CONETEC

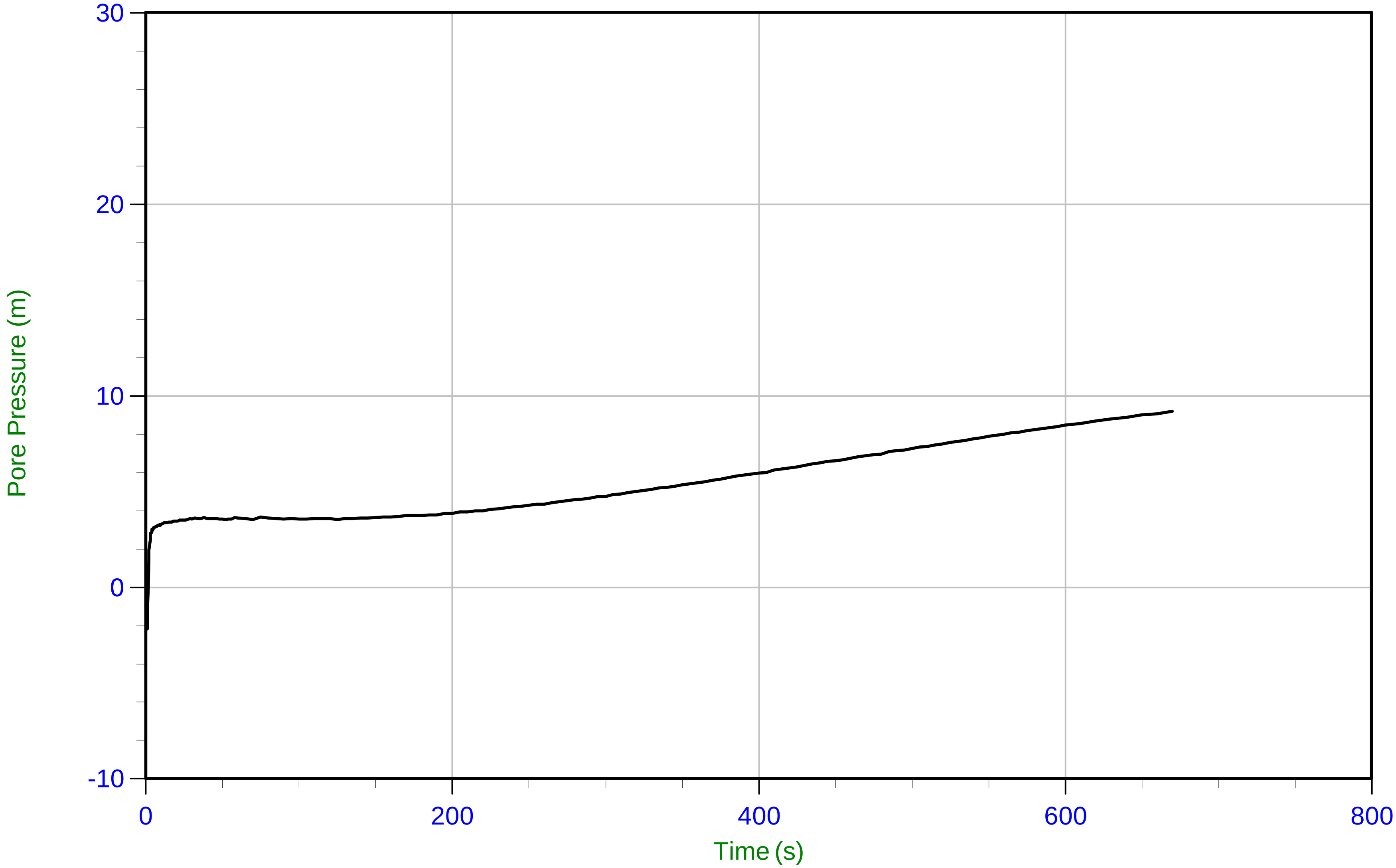
Job No: 24-05-27609
Client: Stantec Consulting Ltd.
Project: HWY 3 St Thomas CPT
Start Date: 2024-05-09
End Date: 2024-05-10

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

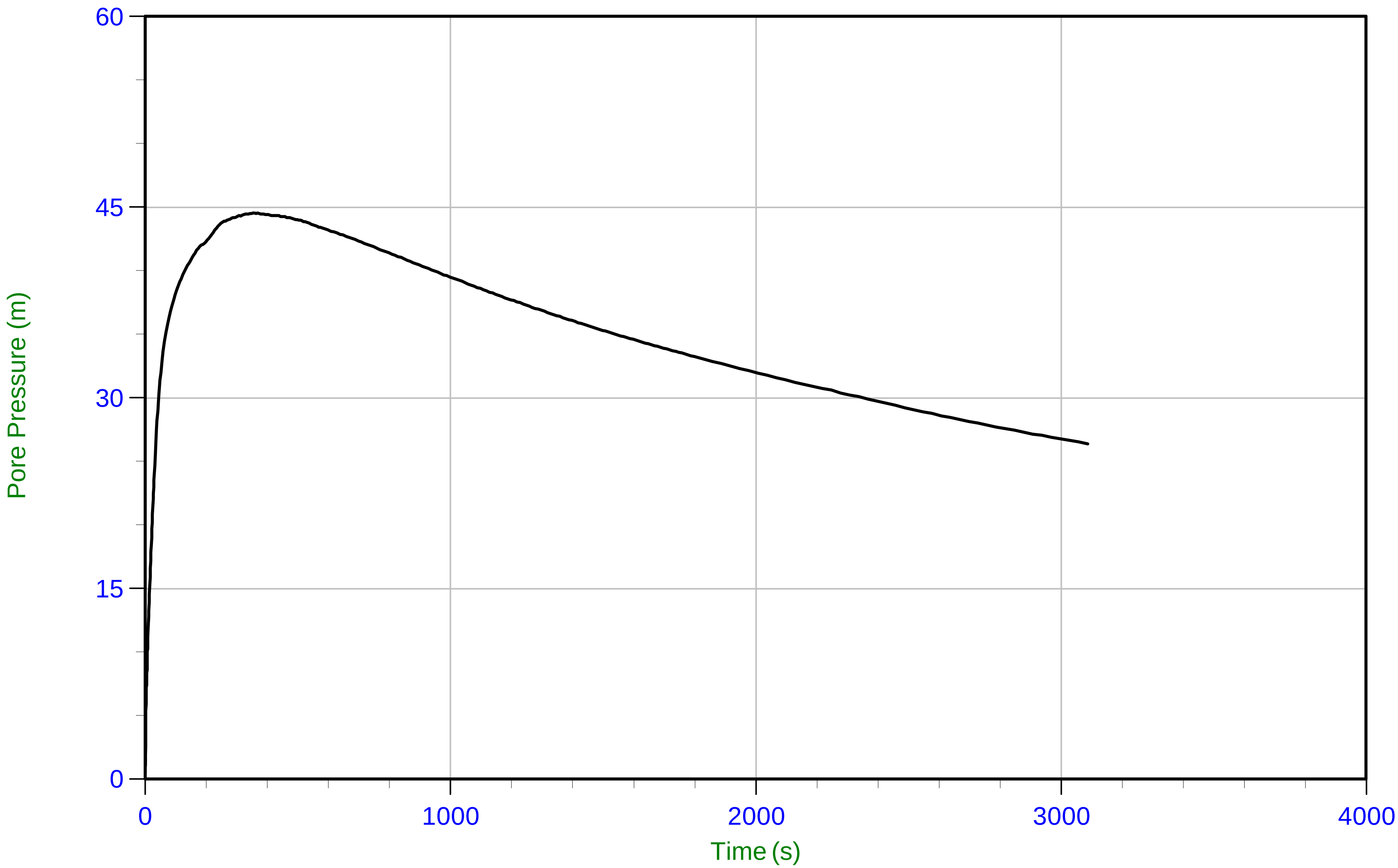


Trace Summary:

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	Sounding: CPT24-RMNAPP01	
	Date: 2024-05-10 12:15	Cone: 729:T1500F15U35	Area=15 cm²
	Site: HWY 3, St.Thomas, ON		



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

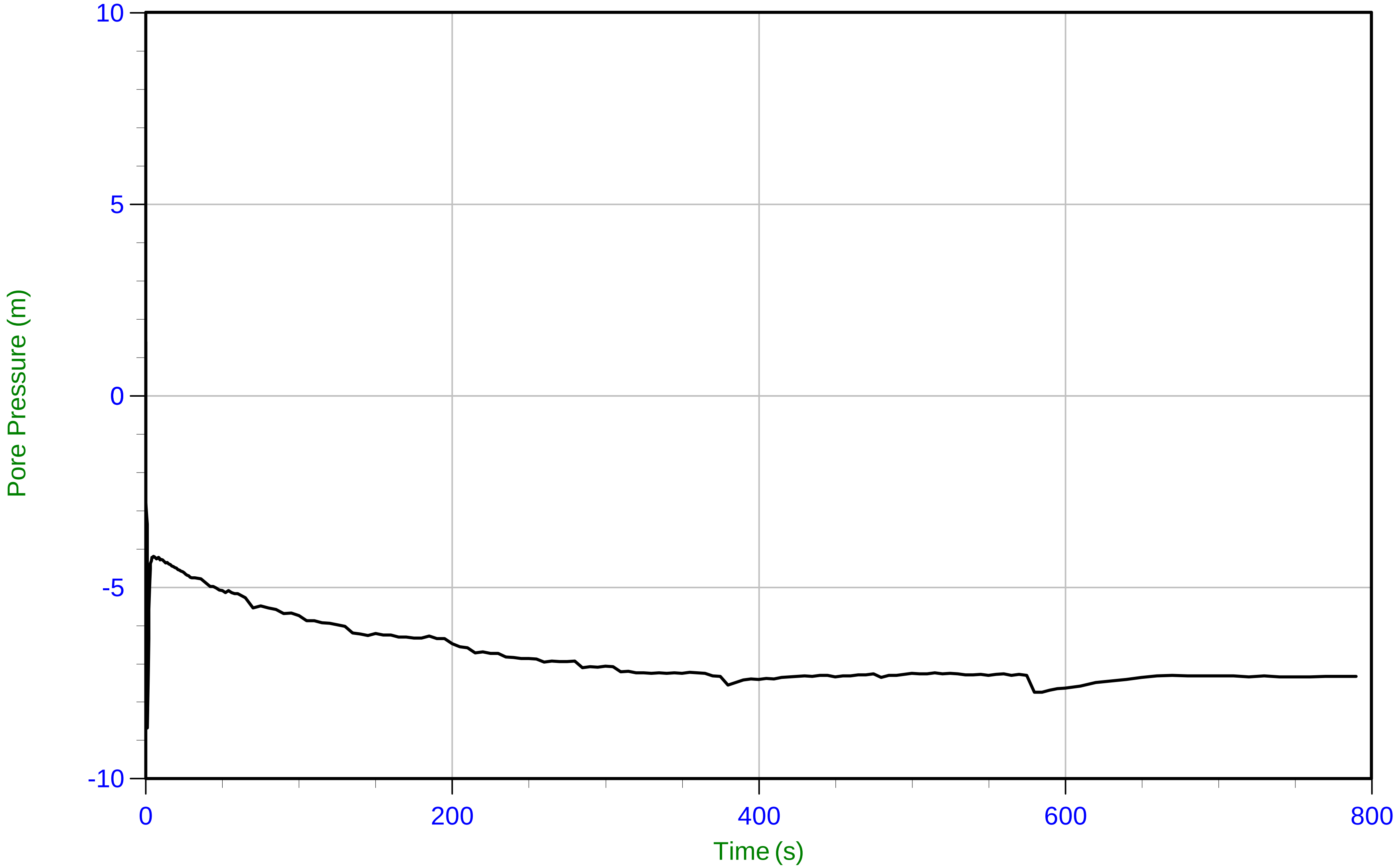
u Max: 44.6 m

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

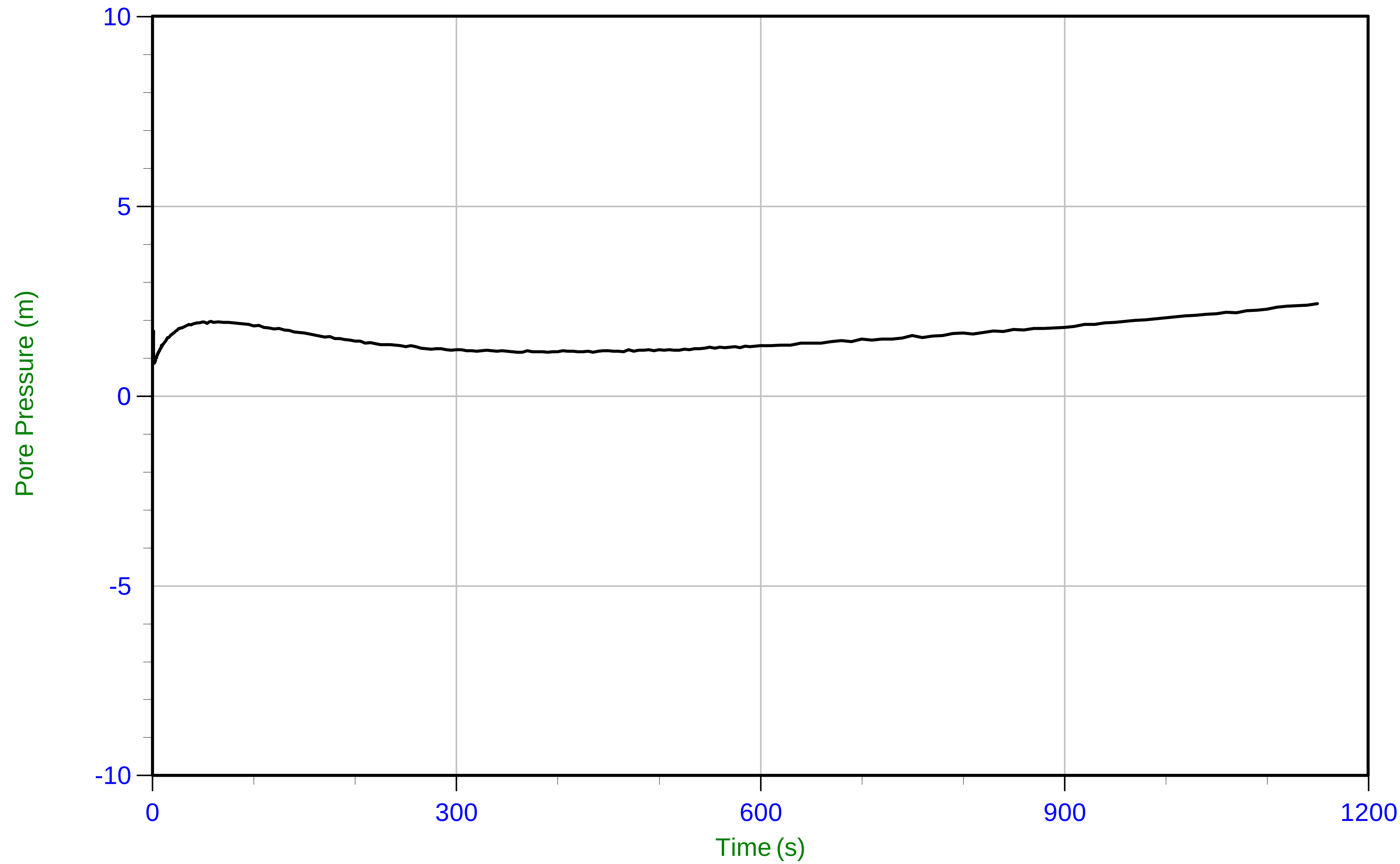


Trace Summary:

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:

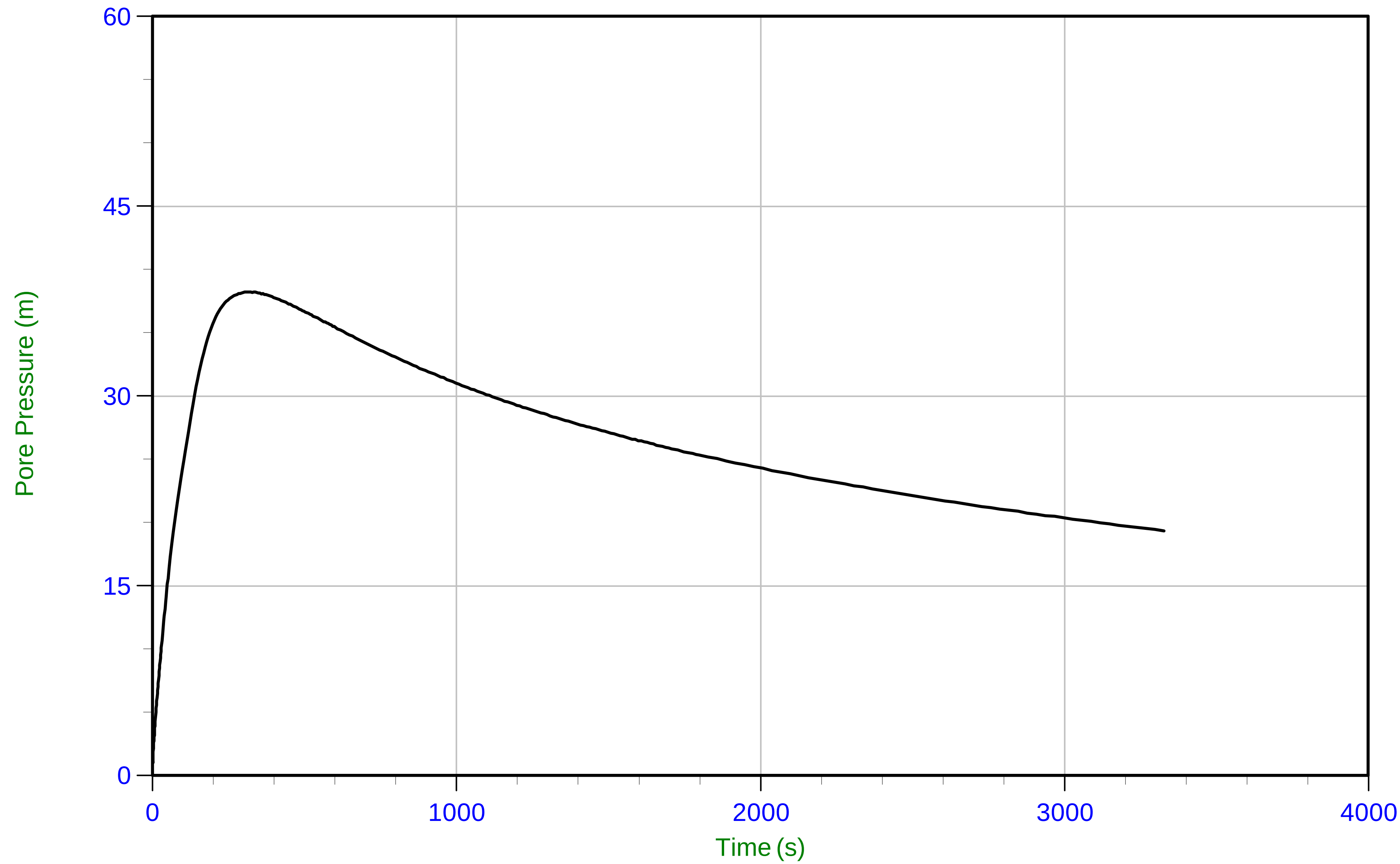
Filename: 24-05-27609_SP-RM-01.PPF2
Depth: 9.050 m / 29.691 ft
Duration: 1150.0 s

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

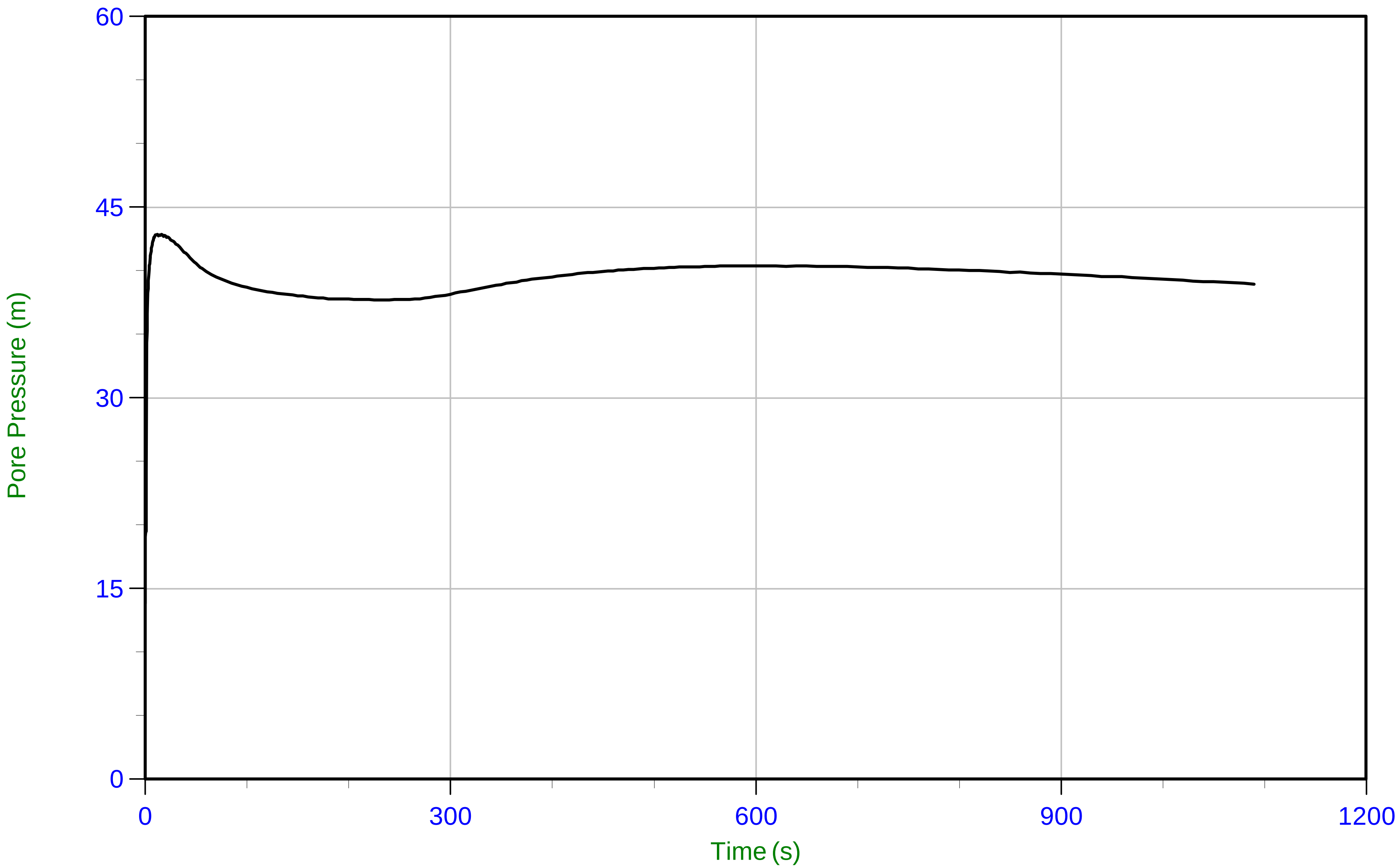


Trace Summary:	Filename:	24-05-27609_SP-CN-02.PPF2	u Min:	1.0 m	WT:	2.0 m / 6.6 ft	T(50):	2923.2 s
	Depth:	3.050 m / 10.006 ft	u Max:	38.2 m	Ueq:	1.1 m	Ir:	100
	Duration:	3330.0 s	u Final:	19.4 m	U(50):	19.64 m	Ch:	0.2 cm²/min

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:

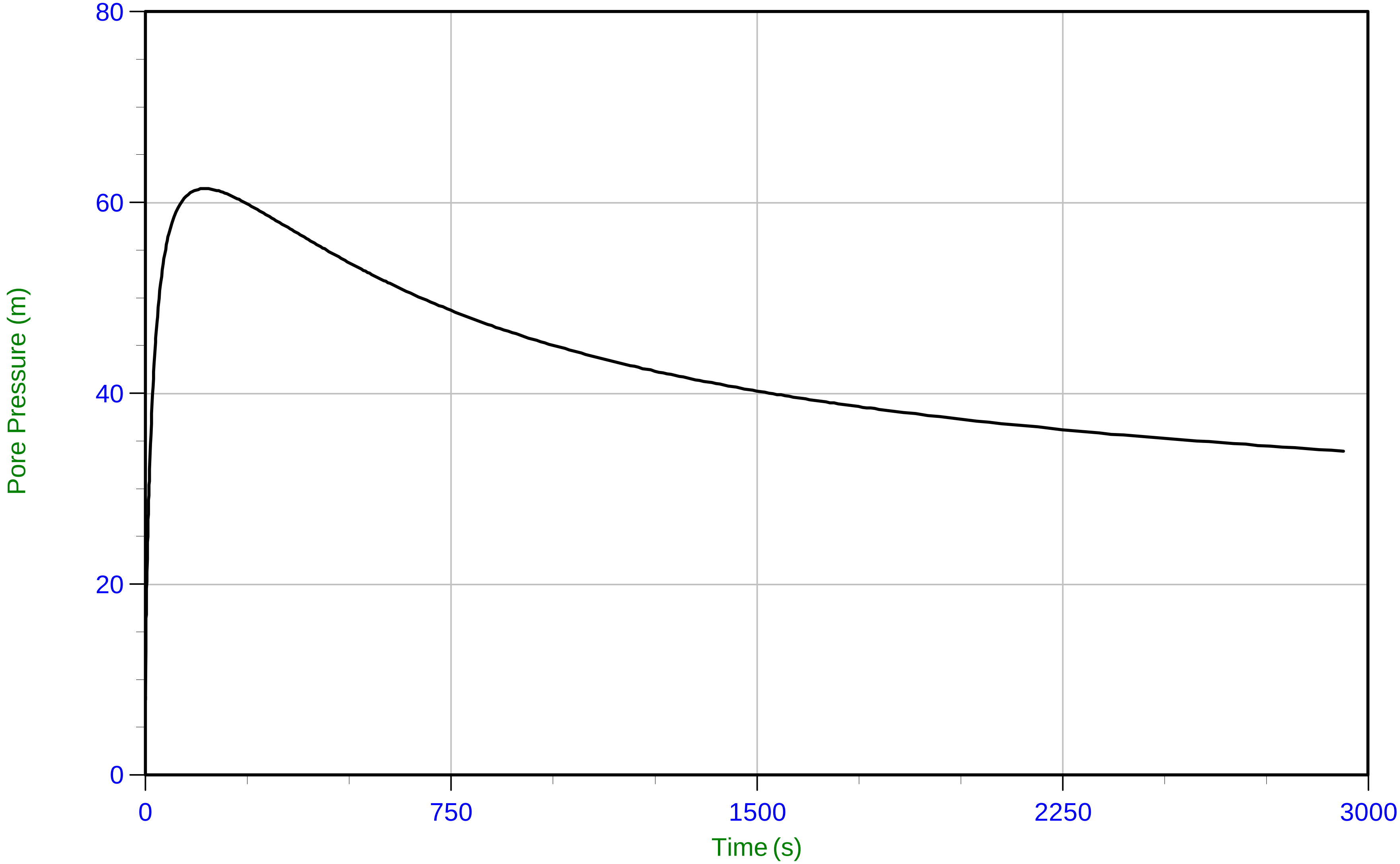
Filename: 24-05-27609_SP-CN-02.PPF2
Depth: 7.625 m / 25.016 ft
Duration: 1090.0 s

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

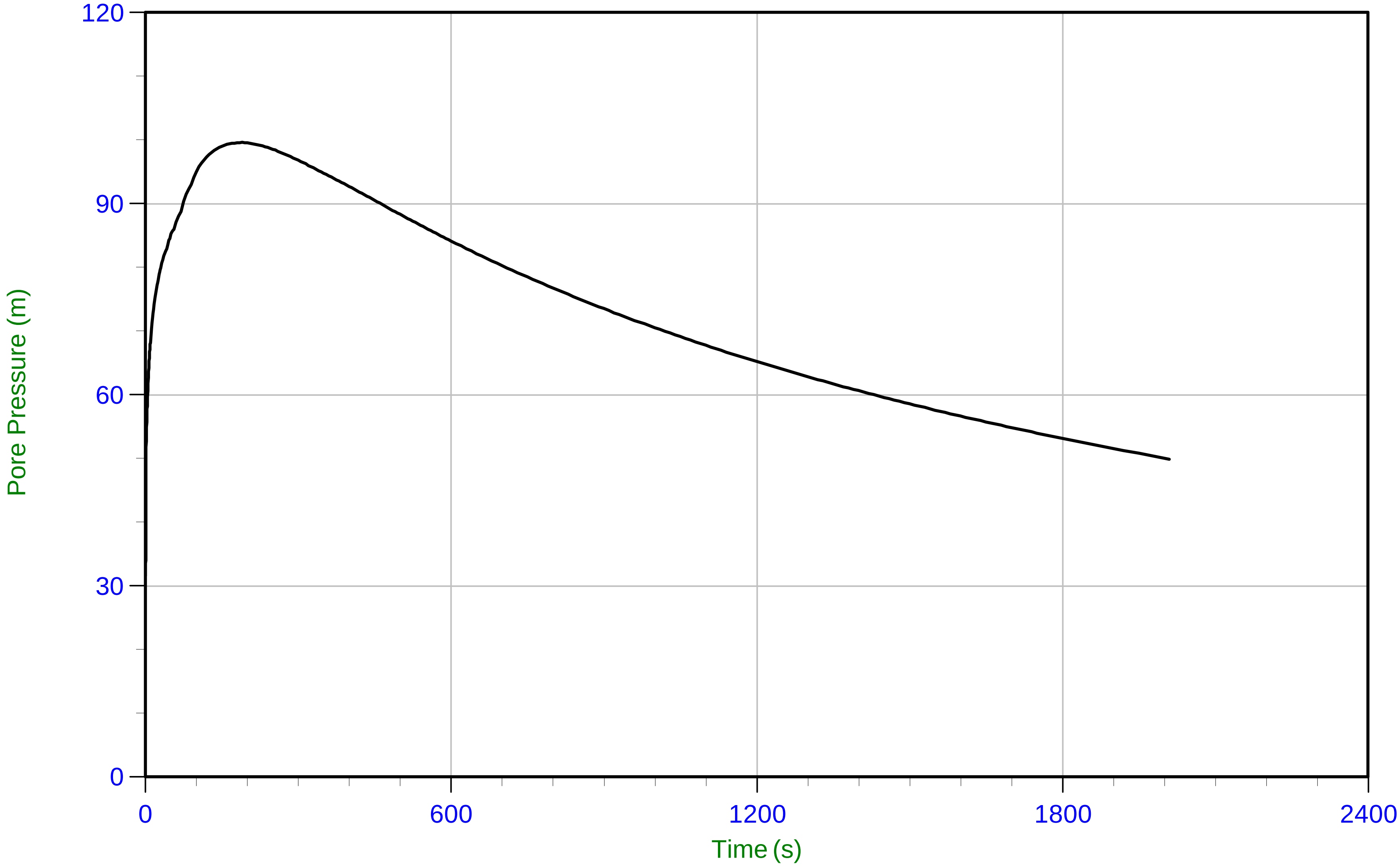
T(50): 2423.2 s
Ir: 100
Ch: 0.3 cm²/min

CONETEC

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

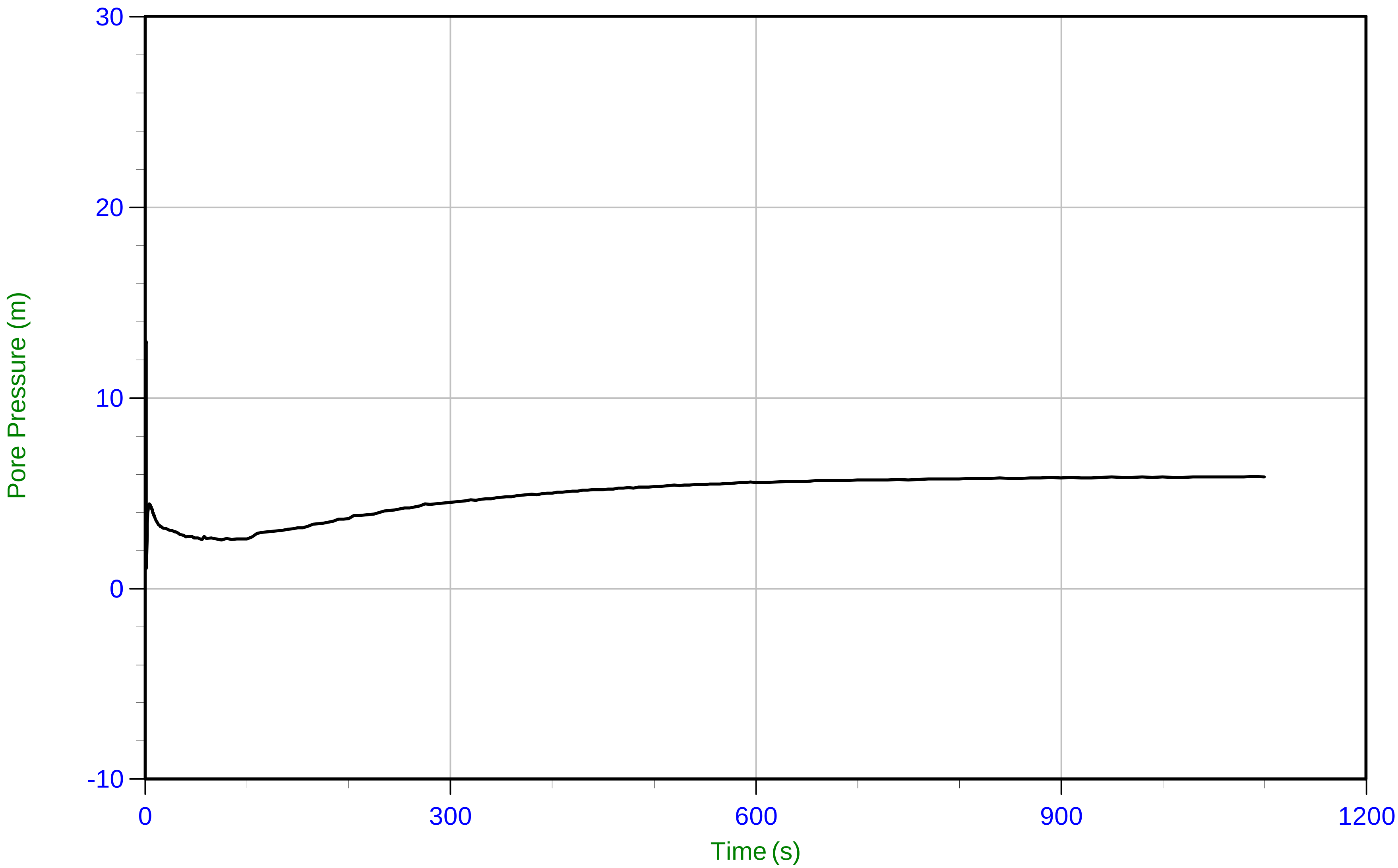


Trace Summary:	Filename: 24-05-27609_SP-WA-02.PPF2	u Min: 31.5 m	WT: 1.8 m / 6.0 ft	T(50): 1785.6 s
	Depth: 3.050 m / 10.006 ft	u Max: 99.6 m	Ueq: 1.2 m	Ir: 100
	Duration: 2010.0 s	u Final: 49.9 m	U(50): 50.41 m	Ch: 0.4 cm²/min

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

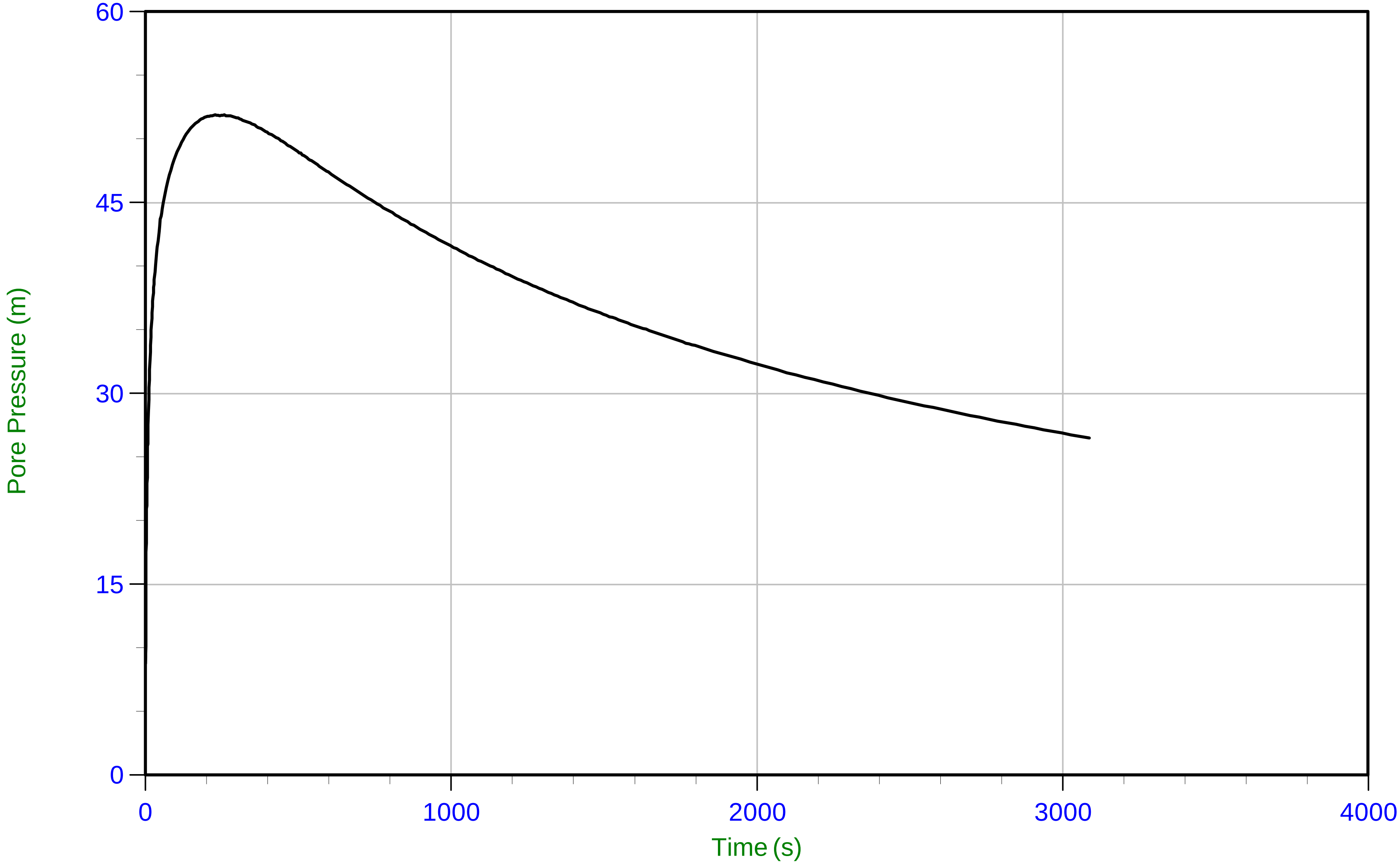
WT: 1.8 m / 6.0 ft
Ueq: 5.8 m

CONETEC

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

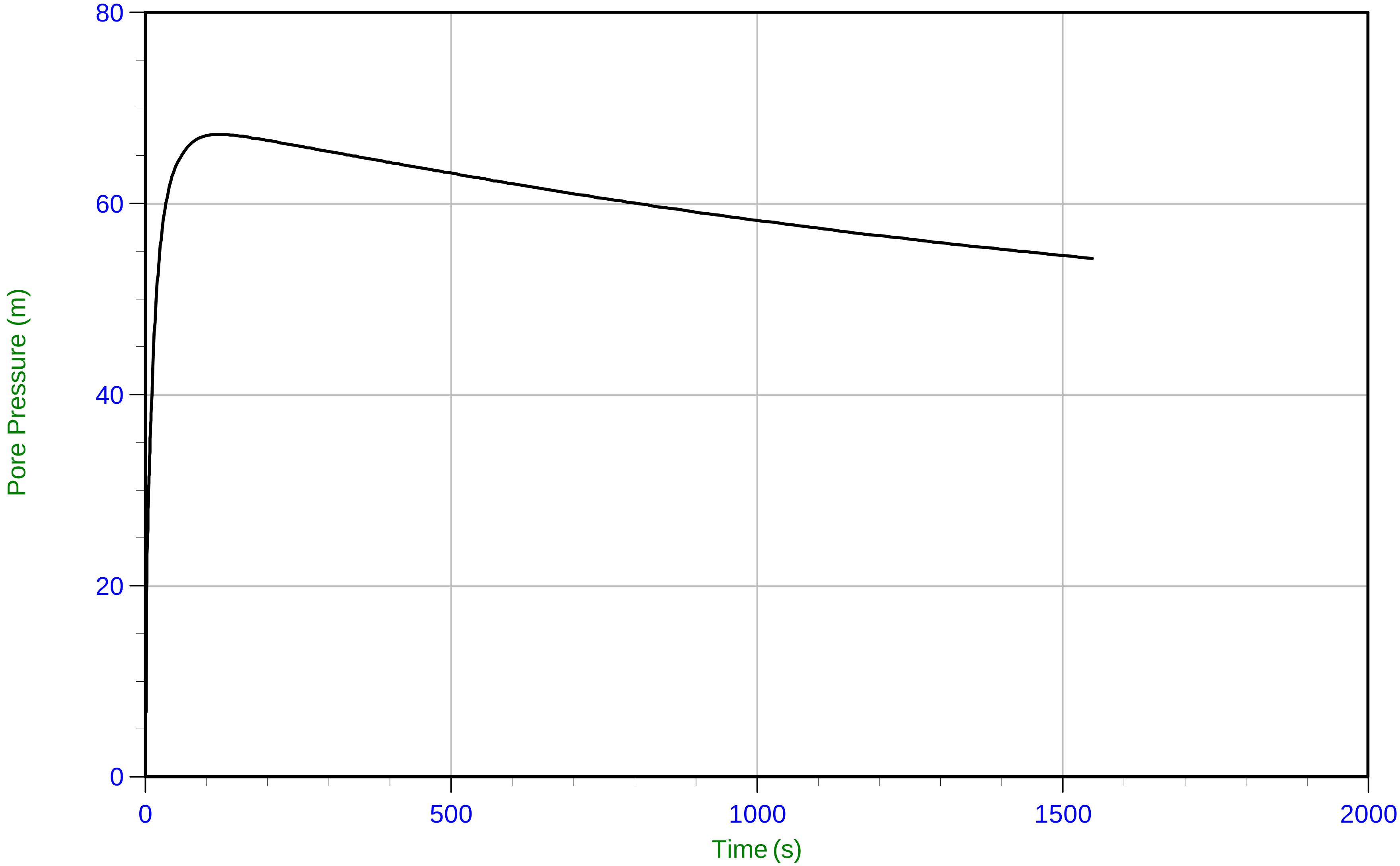
WT: 1.5 m / 4.9 ft
Ueq: 1.6 m
U(50): 26.72 m

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

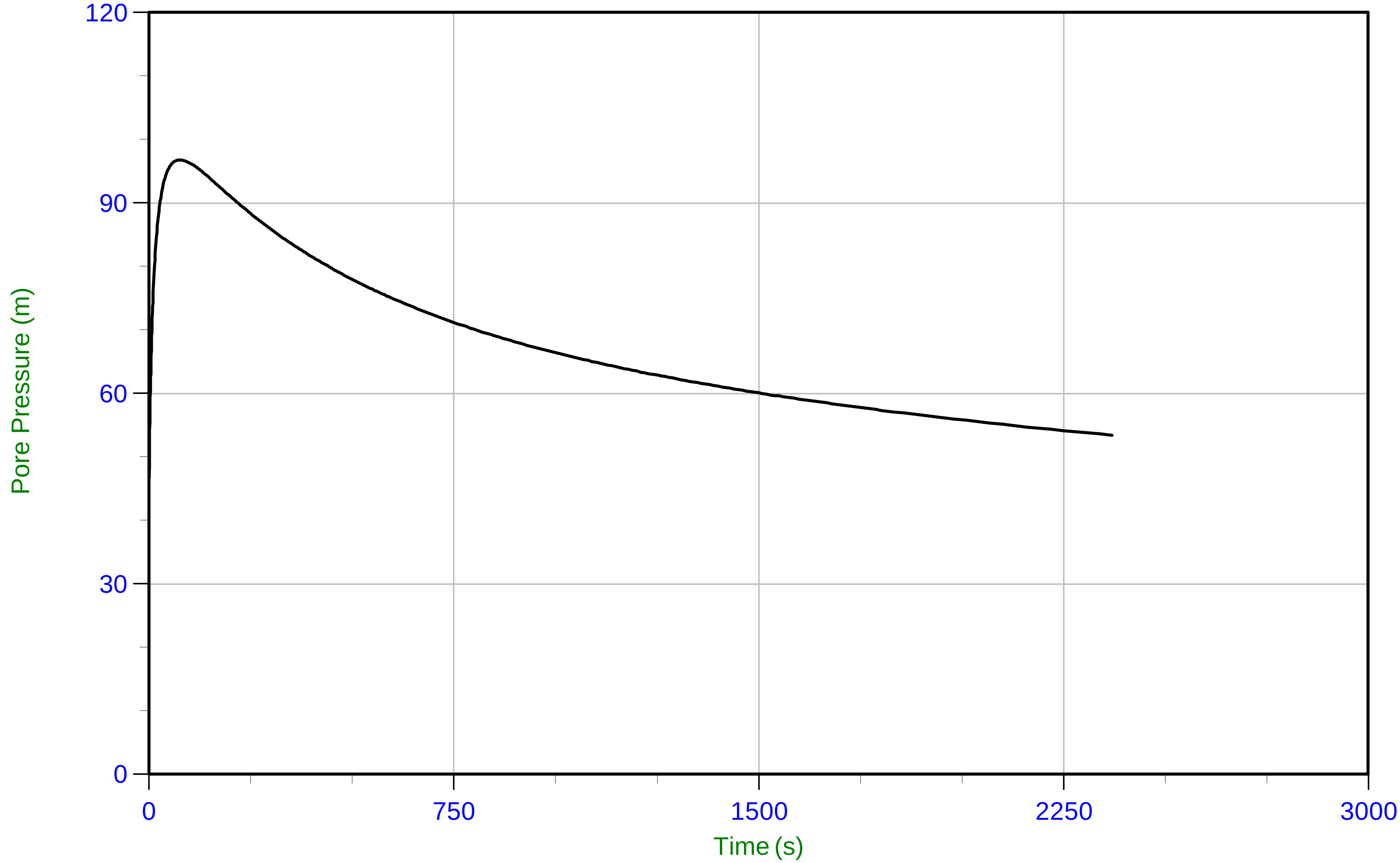


Trace Summary:

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:

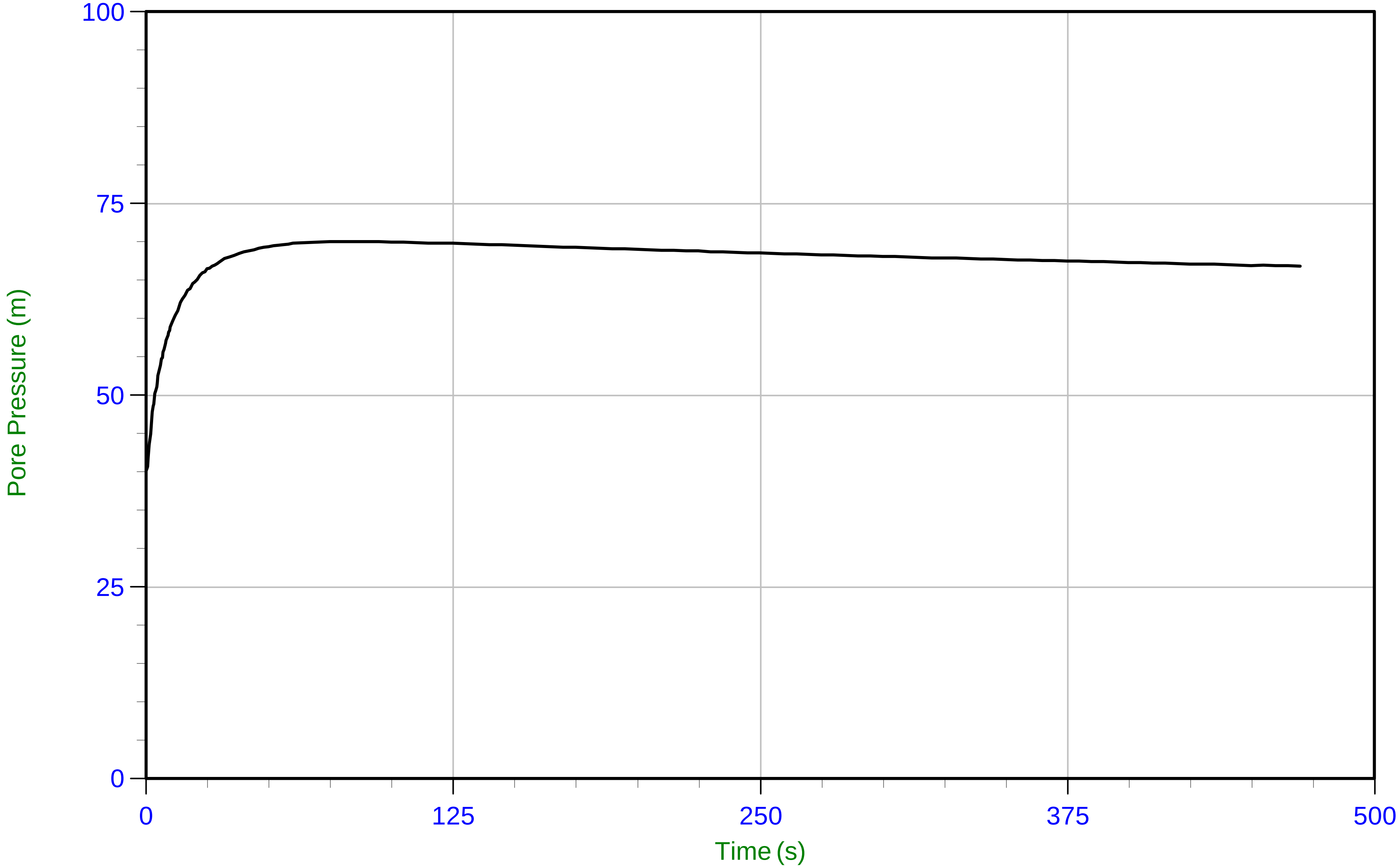
Filename: 24-05-27609_SP-CN-08.PPF2
Depth: 10.675 m / 35.023 ft
Duration: 2370.0 s

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:

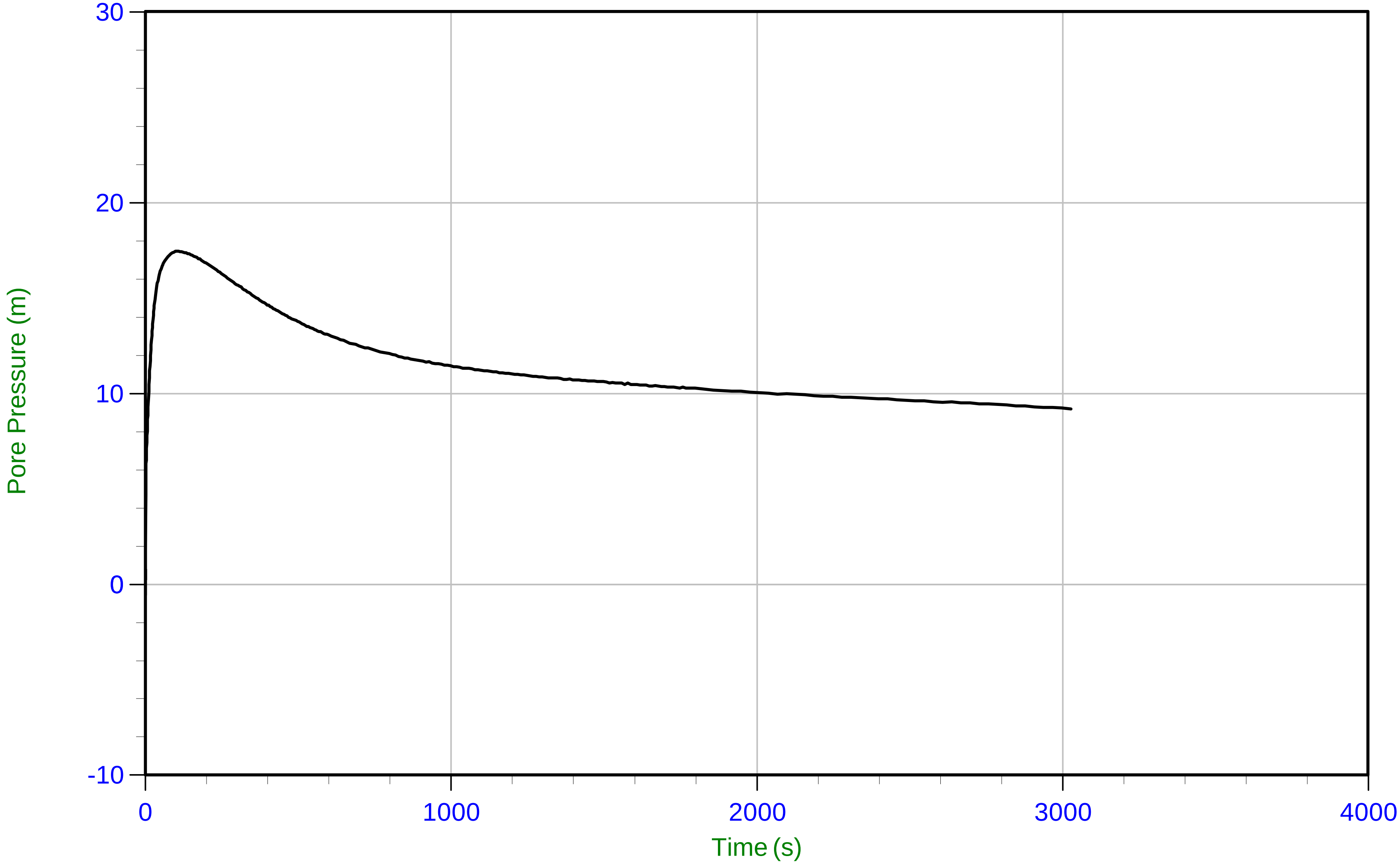
Filename: 24-05-27609_SP-CN-08.PPF2
Depth: 20.100 m / 65.944 ft
Duration: 470.0 s

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

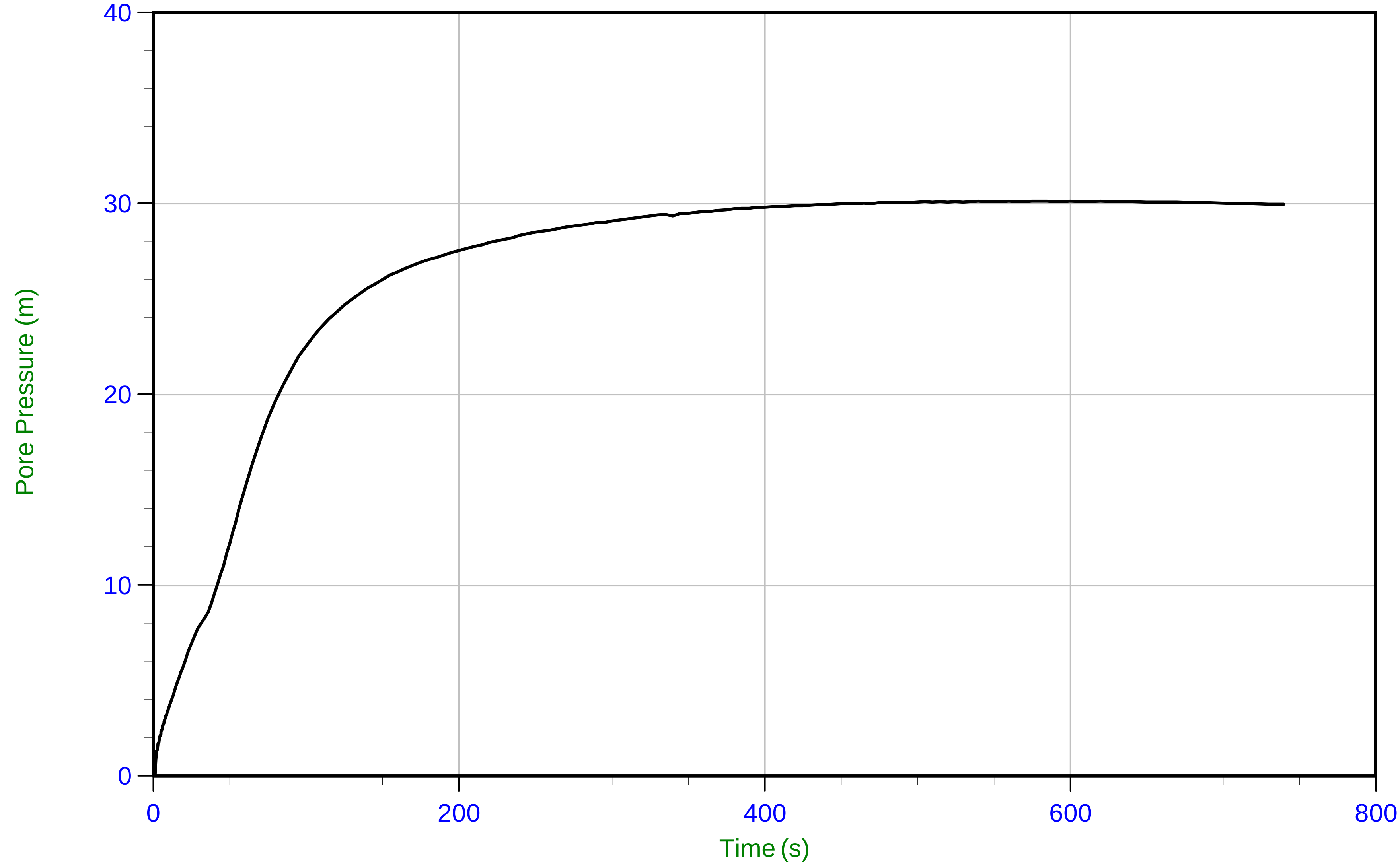
T(50): 2879.9 s
Ir: 100
Ch: 0.2 cm²/min

ConeTec

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

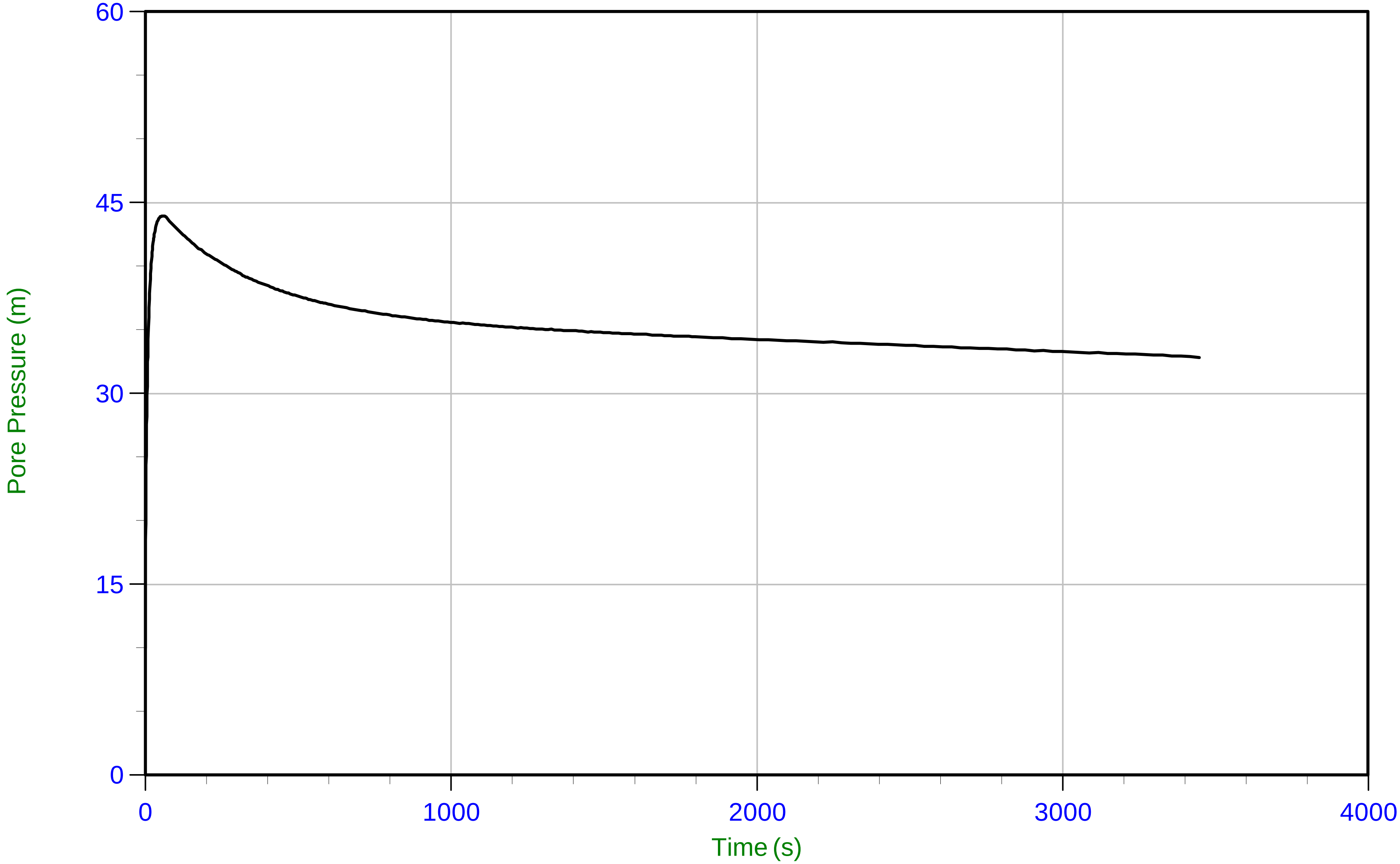


Trace Summary:

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

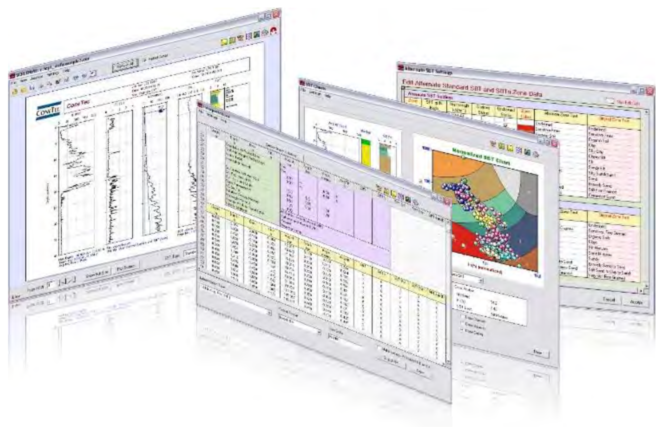
u Min: 10.0 m
u Max: 44.0 m
u Final: 32.8 m

Description of Methods for Calculated CPT Geotechnical Parameters

CONETEC

CALCULATED CPT GEOTECHNICAL PARAMETERS

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



Revision SZW-Rev 18

Revised February 10, 2023

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

ConeTec

Limitations

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

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

ConeTec's Calculated CPT Geotechnical Parameters as of February 10, 2023.

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

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

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

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

where: q_t is the corrected tip resistance
q_c is the recorded tip resistance
u_2 is the recorded dynamic pore pressure from behind the tip (u_2 position)
a is the Net Area Ratio for the cone (typically 0.80 for ConeTec cones)

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

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

A majority of parameter calculations are derived from or driven by results based on material types as determined by the various soil behavior type charts depicted in Figures 1 through 6. The parameter output files indicate the method(s) used.

The Soil Behavior Type classification chart shown in Figure 1 is the classic non-normalized SBT Chart developed at the University of British Columbia and reported in Robertson, Campanella, Gillespie and Greig (1986). Figure 2 shows the original normalized (linear method) SBTn chart developed by Robertson (1990). The Bq classification charts shown in Figures 3a and 3b incorporate pore pressures into the SBT classification and are based on the methods described in Robertson (1990). Many of these charts have been summarized in Lunne, Robertson and Powell (1997). The Jefferies and Davies SBT chart shown in Figure 3c is based on the techniques discussed in Jefferies and Davies (1993) which introduced the concept of the Soil Behavior Type Index parameter, I_c. Take note that the I_c parameter developed by Robertson and Fear (1995) and Robertson and Wride (1998) is similar in concept but uses a slightly different calculation method than that defined by Jefferies and Davies (1993) as the latter incorporates pore pressure in their technique through the use of the Bq parameter. The normalized Qtn SBT chart shown in Figure 4 is based on the work by Robertson (2009) utilizing a variable stress ratio exponent, n, for normalization based on a slightly modified redefinition and iterative approach for I_c. The boundary curves drawn on the chart are based on the work described in Robertson (2010).

Figure 5 shows a revised 1986 SBT Chart presented to CPT'10 by Robertson (2010b). It is known as the Updated non-normalized Soil Behavior Chart (also referred to as the Rev SBT Chart (PKR2010) in our output files). This chart was produced to be more in line with all post-1986 Robertson charts having the same 9 soil type zones, a $\log_{10}$ axis for friction ratio, R_f in this case, and a unitless tip resistance axis.

Figure 6 shows a revised behavior based chart by Robertson (2016) depicting contractive-dilative zones. As the zones represent material behavior rather than soil gradation ConeTec has chosen a set of zone colors that are less likely to be confused with material type colors from previous SBT charts. These colors differ from those used by Dr. Robertson. A green palette was selected for the dilative (desirable) side of the chart and a red palette for the contractive side of the chart.

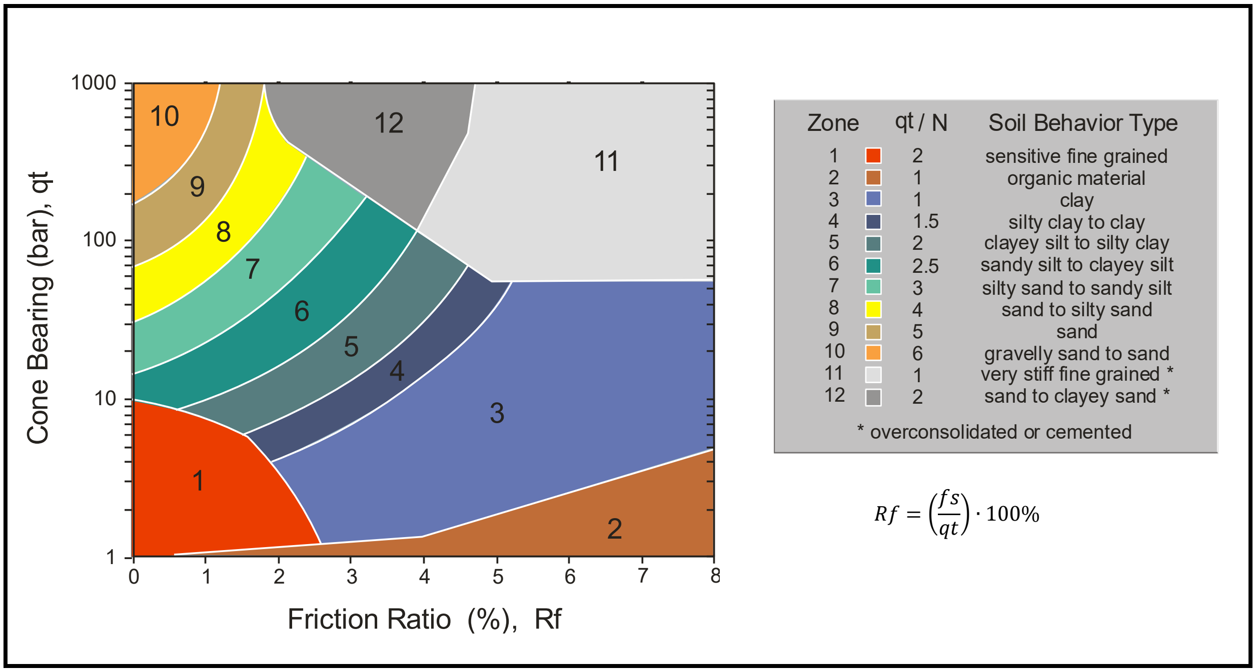


Figure 1. Non-normalized Soil Behavior Type Classification Chart (SBT)

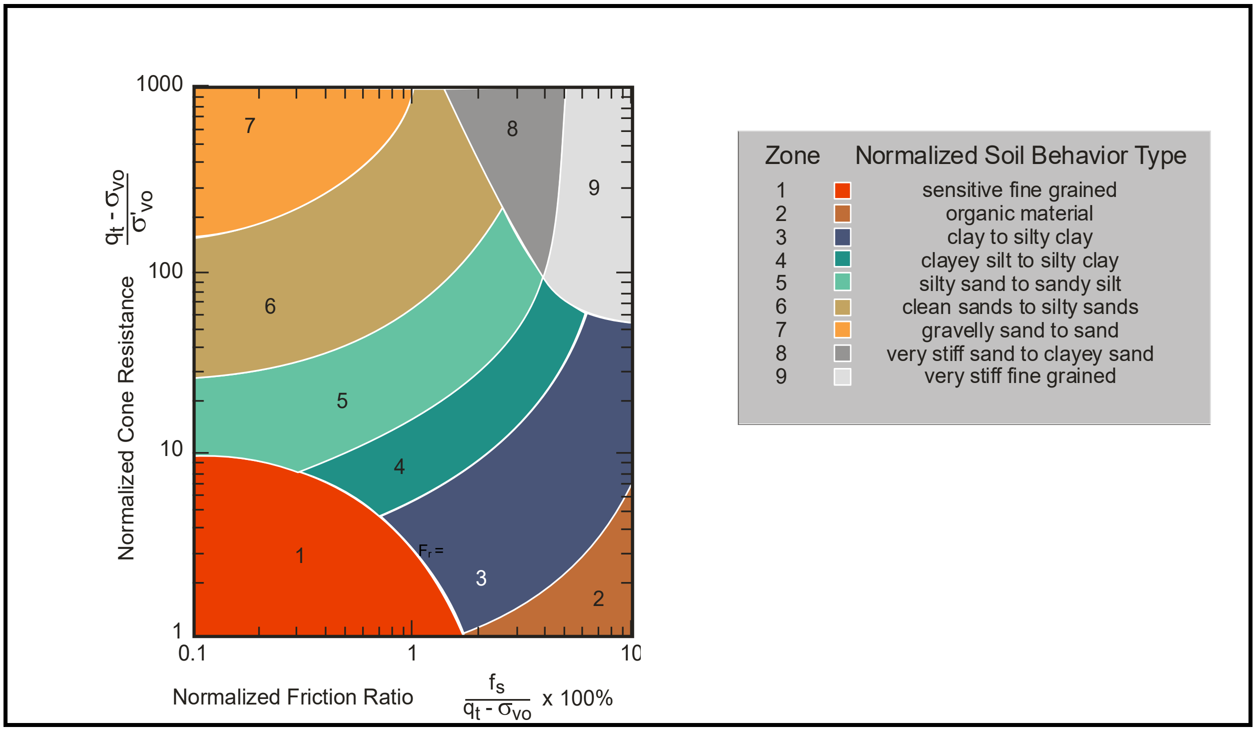


Figure 2. Normalized Soil Behavior Type Classification Chart (SBTn)

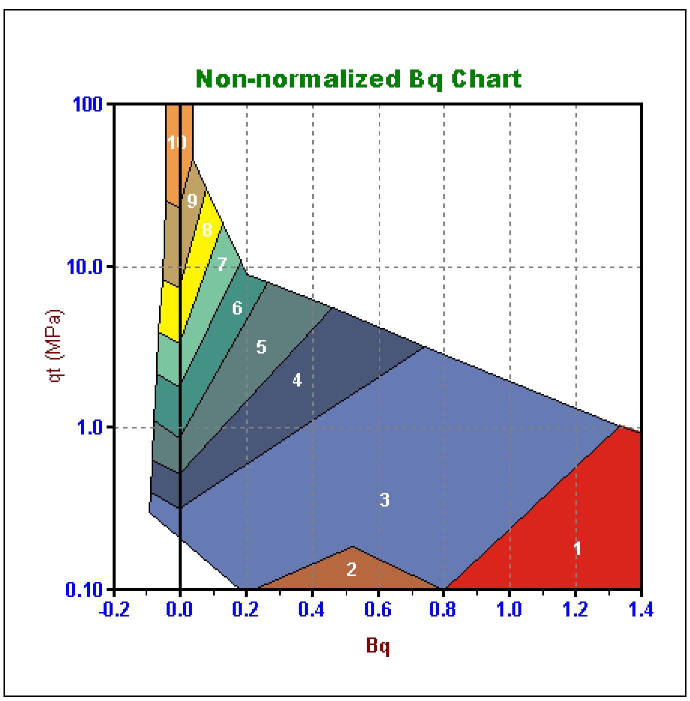


Figure 3a. Alternate Soil Behavior Type Chart (SBT Bq): q_t - B_q

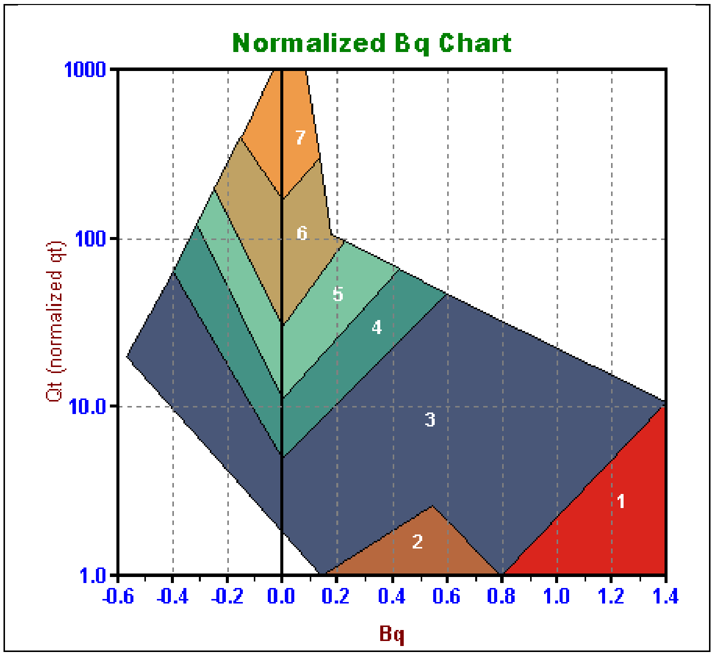


Figure 3b. Alternate Soil Behavior Type Charts (SBT Bqn): Q_t-B_q

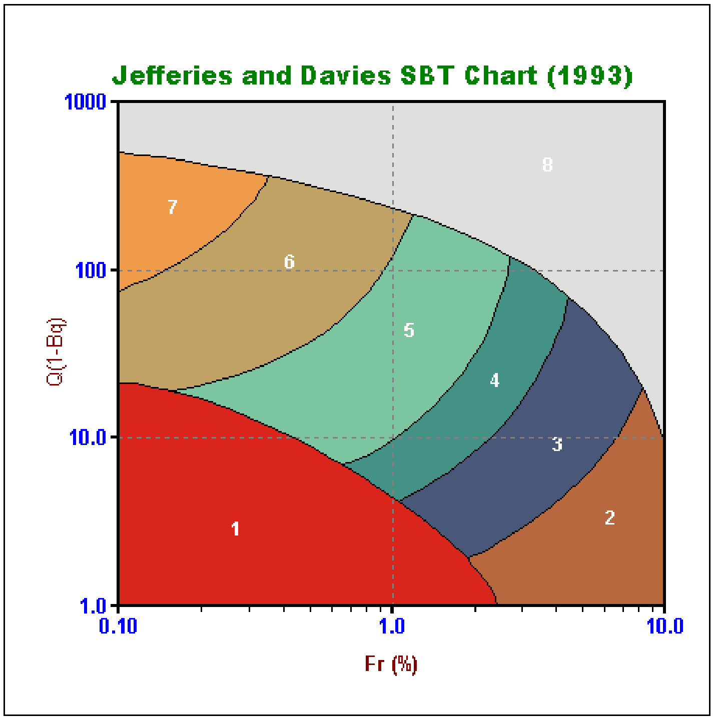


Figure 3c. Alternate Soil Behavior Type Charts: $Q(1-B_q)$ - F_r

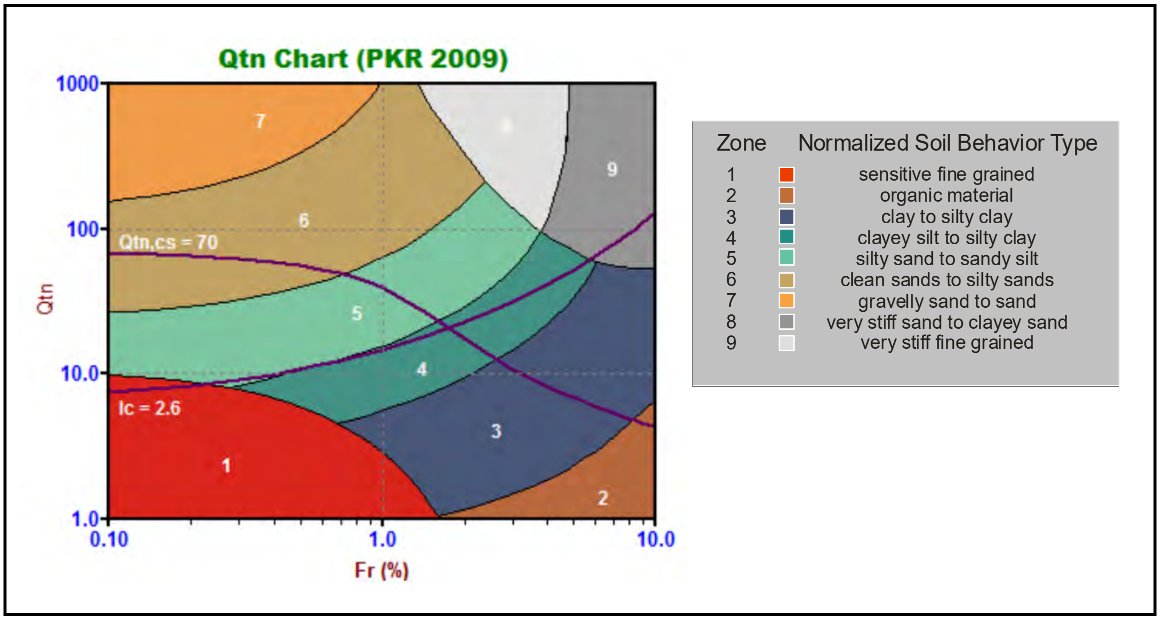


Figure 4. Normalized Soil Behavior Type Chart using Q_{tn} (SBT Qtn)

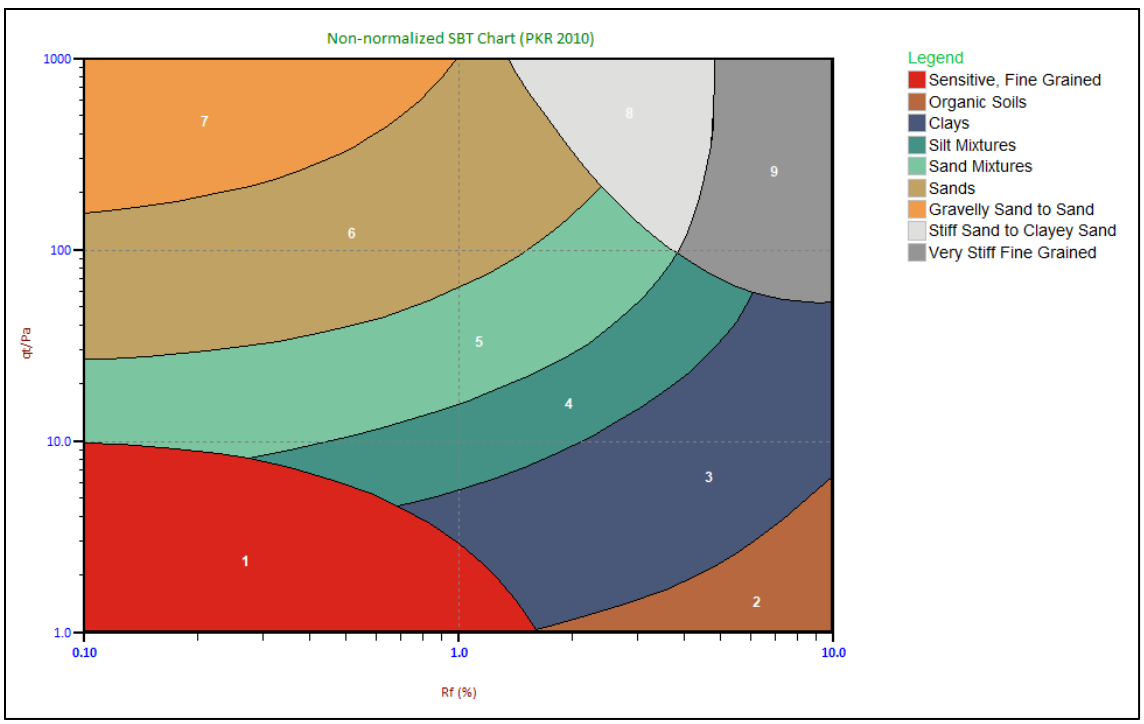


Figure 5. Non-normalized Soil Behavior Type Chart (2010)

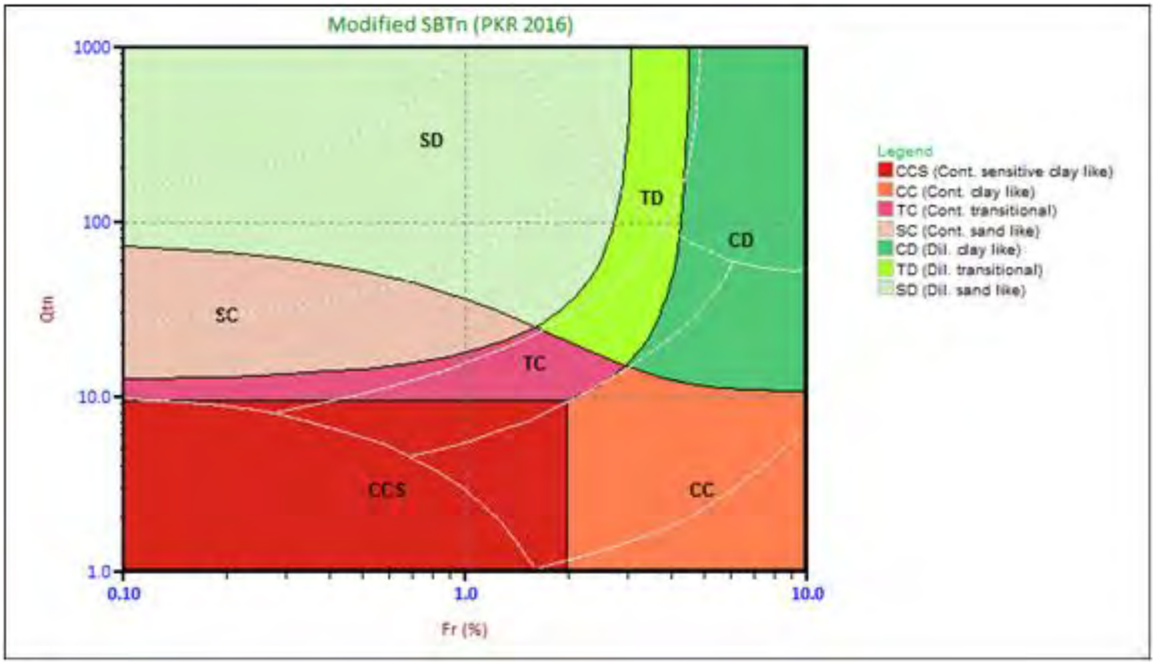


Figure 6. Modified SBTn Behavior Based Chart

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

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

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

1. Invalid or undefined CPT data (e.g., drilled out section or data gap).

2. Where the calculation method is inappropriate, for example, drained parameters in a material behaving in an undrained manner (and vice versa).

3. Where input values are beyond the range of the referenced charts or specified limitations of the correlation method.

4. Where pre-requisite or intermediate parameter calculations are invalid.

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

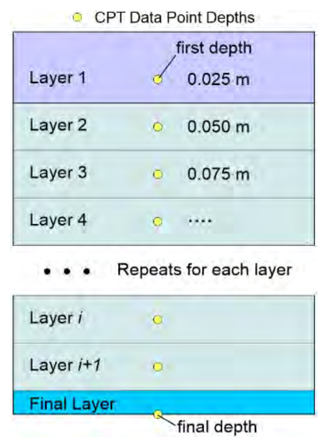
The output files are typically provided in Microsoft Excel XLS, XLSX or CSV format. The ConeTec software has several options for output depending on the number or types of calculated parameters desired or those specifically contracted for by the client. Each output file is named using the original file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

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

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

Calculated Parameter	Description	Equation	Ref
Depth	Mid Layer Depth *(where calculations are done at each point then Mid Layer Depth = Recorded Depth)*	*[Depth (Layer Top) + Depth (Layer Bottom)]/ 2.0*	CK*
Elevation	Elevation of Mid Layer is based on the sounding collar elevation supplied by the client or through a site survey In Sweden a variation of elevation is used where the elevation increases with depth. We refer to this as inverse elevation.	Elevation = Collar Elevation – Depth InverseElevation = Collar Elevation + Depth	CK* N/A
Avg qc	Averaged recorded tip value (q_c)	$Avgqc = \frac{1}{n}\sum_{i=1}^{n} q_c$ *n=1 when calculations are done at each point*	CK*
Avg qt	Averaged corrected tip (q_t) where: $q_t = q_c + (1 - a) \cdot u_2$ Averaged q_t is not calculated using the average q_c and averaged u values. Averaged q_t is based on the average of the q_t values calculated at each data point.	$Avgqt = \frac{1}{n}\sum_{i=1}^{n} q_t$ *n=1 when calculations are done at each point*	1
Avg fs	Averaged sleeve friction (f_s) No pore pressure corrections are applied to f_s.	$Avgfs = \frac{1}{n}\sum_{i=1}^{n} fs$ *n=1 when calculations are done at each point*	CK*
Avg Rf	Averaged friction ratio (R_f) where friction ratio is defined as: $R_f = 100\% \cdot \frac{fs}{q_t}$	$AvgRf = 100\% \cdot \frac{Avgfs}{Avgqt}$ *not an average of individual R_f values*	CK*
Avg u	Averaged dynamic pore pressure *(u)*	$Avgu = \frac{1}{n}\sum_{i=1}^{n} u_i$ *n=1 when calculations are done at each point*	CK*
Avg Res	Averaged Resistivity (this data is not always available since it is a specialized test requiring an additional module)	$AvgRes = \frac{1}{n}\sum_{i=1}^{n} Resistivity_i$ *n=1 when calculations are done at each point*	CK*
Avg UVIF	Averaged UVIF ultra-violet induced fluorescence (this data is not always available since it is a specialized test requiring an additional module)	$AvgUVIF = \frac{1}{n}\sum_{i=1}^{n} UVIF_i$ *n=1 when calculations are done at each point*	CK*
Avg Temp	Averaged Temperature (this data is not always available)	$AvgTemp = \frac{1}{n}\sum_{i=1}^{n} Temperature_i$ *n=1 when calculations are done at each point*	CK*
Avg Gamma	Averaged Gamma Counts (this data is not always available since it is a specialized test requiring an additional module)	$AvgGamma = \frac{1}{n}\sum_{i=1}^{n} Gamma_i$ *n=1 when calculations are done at each point*	CK*
SBT	Soil Behavior Type as defined by Robertson et al 1986 (often referred to as Robertson and Campanella, 1986)	See Figure 1	1, 5
SBTn	Normalized Soil Behavior Type as defined by Robertson 1990 (linear normalization using Q_t, now referred to as Q_{t1})	See Figure 2	2, 5

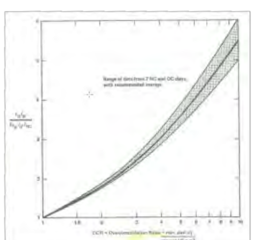
Calculated Parameter	Description	Equation	Ref
SBT-Bq	Non-normalized Soil Behavior type based on non-normalized tip resistance and the B_q parameter	See Figure 3a	1, 2, 5
SBT-Bqn	Normalized Soil Behavior type based on normalized tip resistance (Q_t, now called Q_{t1}) and the B_q parameter	See Figure 3b	2, 5
SBT-JandD	Soil Behavior Type as defined by Jeffries and Davies	See Figure 3c	7
SBT Qtn	Soil Behavior Type as defined by Robertson (2009) using a variable stress ratio exponent for normalization based on $I_{c\ (PKR\ 2009)}$	See Figure 4	15
Modified Non-normalized SBT Chart SBT (PKR2010)	This is a revised version of the simple 1986 non-normalized SBT chart (presented at CPT '10). The revised version has been reduced from 12 zones to 9 zones to be similar to the normalized Robertson charts. Other updates include a dimensionless tip resistance normalized to atmospheric pressure, q_t/P_a, on the vertical axis and a log scale for non-normalized friction ratio, R_f, along the horizontal axis.	See Figure 5	33
Modified SBTn (contractive /dilative)	Modified SBTn chart as defined by Robertson (2016) indicating zones of contractive/dilative behavior. Note that ConeTec displays the chart with colors different from Robertson. ConeTec's colors were chosen to avoid confusion with soil type descriptions.	See Figure 6	30
Unit Wt.	Unit Weight of soil determined from one of the following user selectable options: 1) uniform value 2) value assigned to each SBT zone 3) value assigned to each SBTn zone 4) value assigned to SBTn zone as determined from Robertson and Wride (1998) based on q_{c1n} 5) values assigned to SBT Qtn zones 6) values based on Robertson updated non-normalized Soil Behavior Type Chart (2010b) 6) Mayne f_s (sleeve friction) method 7) Robertson and Cabal 2010 method 8) user supplied unit weight profile The last option may co-exist with any of the other options.	See references	3, 5, 15, 21, 24, 29, 33

Calculated Parameter	Description	Equation	Ref
TStress σ_v	Total vertical overburden stress at Mid Layer Depth *A layer is defined as the averaging interval specified by the user where depths are reported at their respective mid-layer depth.* For data calculated at each point layers are defined using the recorded depth as the mid-point of the layer. Thus, a layer starts half-way between the previous depth and the current depth unless this is the first point in which case the layer start is at zero depth. The layer bottom is half-way from the current depth to the next depth unless it is the last data point. Defining layers affects how stresses are calculated since the unit weight attributed to a data point is used throughout the entire layer. This means that to calculate the stresses the total stress at the top and bottom of a layer are required. The stress at mid layer is determined by adding the incremental stress from the layer top to the mid-layer depth. The stress at the layer bottom becomes the stress at the top of the subsequent layer. Stresses are NOT calculated from mid-point to mid-point. For over-water work the total stress due to the column of water above the mud line is taken into account where appropriate.	$TStress = \sum_{i=1}^{n} \gamma_i h_i$ *where* γ_i is layer unit weight h_i is layer thickness 	CK*
EStress σ_v'	Effective vertical overburden stress at mid-layer depth.	$\sigma_v' = \sigma_v - u_{eq}$	CK*
Equil u u_{eq} or u_0	Equilibrium pore pressures are determined from one of the following user selectable options: 1) hydrostatic below the water table 2) user supplied profile 3) combination of those above When a user supplied profile is used/provided a linear interpolation is performed between equilibrium pore pressures defined at specific depths. If the profile values start below the water table then a linear transition from zero pressure at the water table to the first defined pointed is used. Equilibrium pore pressures may come from dissipation tests, adjacent piezometers or other sources. Occasionally, an extra equilibrium point ("assumed value") will be provided in the profile that does not come from a recorded value to smooth out any abrupt changes or to deal with material interfaces. These "assumed" values will be indicated on our plots and in tabular summaries.	For the hydrostatic option: $u_{eq} = \gamma_w \cdot (D - D_{wt})$ where u_{eq} is equilibrium pore pressure γ_w is the unit weight of water D is the current depth D_{wt} is the depth to the water table	CK*
K_0	Coefficient of earth pressure at rest, K_0.	$K_o = (1 - \sin\Phi')\ OCR^{\sin\Phi'}$	17
C_n	Overburden stress correction factor used for $(N_1)_{60}$ and older CPT parameters.	$C_n = (P_a/\sigma_v')^{0.5}$ where *$0.0 < C_n < 2.0$ (user adjustable, typically ranging from 1.7 to 2.0)* *P_a is atmospheric pressure (100 kPa)*	4, 12

Calculated Parameter	Description	Equation	Ref
C_q	Overburden stress normalizing factor.	$C_q = 1.8 / [0.8 + (\sigma_v'/P_a)]$ where *$0.0 < C_q < 2.0$ (user adjustable)* *P_a is atmospheric pressure (100 kPa)* *Robertson and Wride define C_q to be the same as C_n. The Olson definition above is used in the program.*	3, 12
N_{60}	SPT N value at 60% energy calculated from q_t/N ratios assigned to each SBT zone. This method has abrupt N value changes at zone boundaries.	*See Figure 1*	5
$(N_1)_{60}$	SPT N_{60} value corrected for overburden pressure.	$(N_1)_{60} = C_n \bullet N_{60}$	4
$N_{60}I_c$	SPT N_{60} values based on the I_c parameter, as defined by Robertson and Wride 1998 (3), or by Robertson 2009 (15).	$(q_t/P_a)/ N_{60} = 8.5\ (1 - I_c/4.6)$ $(q_t/P_a)/ N_{60} = 10^{(1.1268 - 0.2817Ic)}$ P_a being atmospheric pressure	3, 5 15, 31
$(N_1)_{60}I_c$	SPT N_{60} value corrected for overburden pressure (using N_{60} I_c). User has 3 options.	1) $(N_1)_{60}Ic = C_n \bullet (N_{60}\, I_c)$ 2) $q_{c1n}/ (N_1)_{60}I_c = 8.5\ (1 - I_c/4.6)$ 3) $(Q_{tn})/ (N_1)_{60}I_c = 10^{(1.1268 - 0.2817Ic)}$	4 5 15, 31
S_u or S_u (N_{kt})	Undrained shear strength based on q_t S_u factor N_{kt} is user selectable.	$Su = \dfrac{qt - \sigma_v}{N_{kt}}$	1, 5
S_u or S_u (N_{du}) or S_u ($N_{\Delta u}$)	Undrained shear strength based on pore pressure S_u factor $N_{\Delta u}$ is user selectable.	$Su = \dfrac{u_2 - u_{eq}}{N_{\Delta u}}$	1, 5
D_r	Relative Density determined from one of the following user selectable options: 1) Ticino Sand 2) Hokksund Sand 3) Schmertmann (1978) 4) Jamiolkowski (1985) - All Sands 5) Jamiolkowski et al (2003) (various compressibilities, K_o)	See reference (methods 1 through 4) Jamiolkowski et al (2003) reference	5 14
PHI ϕ	Friction Angle determined from one of the following user selectable options (methods 1 through 4 are for sands and method 5 is for silts and clays): 1) Campanella and Robertson 2) Durgunoglu and Mitchel 3) Janbu 4) Kulhawy and Mayne 5) NTH method (clays and silts)	See appropriate reference	5 5 5 11 23
Delta U/q_t $\Delta u/q_t$ du/q_t	Differential pore pressure ratio (older parameter used before B_q was established)	$= \dfrac{\Delta u}{qt}$ *where:* $\Delta u = u - u_{eq}$ *and u = dynamic pore pressure* *u_{eq} = equilibrium pore pressure*	39

Calculated Parameter	Description	Equation	Ref
B_q	Pore pressure parameter	$Bq = \frac{\Delta u}{qt - \sigma_v}$ where: $\Delta u = u - u_{eq}$ *and u = dynamic pore pressure* *u_{eq} = equilibrium pore pressure*	1, 2, 5
Net q_t or qtNet	Net tip resistance (used in many subsequent correlations)	$qt - \sigma_v$	36
q_e or qE or q_E	Effective tip resistance (using the dynamic pore pressure u_2 and not equilibrium pore pressure)	$q_t - u_2$	36
qeNorm	Normalized effective tip resistance	$\frac{qt - u_2}{\sigma_v'}$	36
Q_t or Norm: Qt or Q_{t1}	Normalized q_t for Soil Behavior Type classification as defined by Robertson (1990) using a linear stress normalization. Note this is different from Q_{tn}. This parameter was renamed to Q_{t1} in Robertson, 2009. Without normalization limits this parameter calculates to very high unrealistic values at low stresses.	$Qt = \frac{qt - \sigma_v}{\sigma_v'}$	2, 5, 15
F_r or Norm: Fr	Normalized Friction Ratio for Soil Behavior Type classification as defined by Robertson (1990)	$Fr = 100\% \cdot \frac{fs}{qt - \sigma_v}$	2, 5
$Q(1-B_q)$ $Q(1-B_q) + 1$	$Q(1-B_q)$ grouping as suggested by Jefferies and Davies for their classification chart and the establishment of their I_c parameter. Later papers added the +1 term to the equation.	$Q \cdot (1 - Bq)$ $Q \cdot (1 - Bq) + 1$ *where Bq is defined as above and Q is the same as the normalized tip resistance, Q_{t1}, defined above*	6, 7, 34
q_{c1}	Normalized tip resistance, q_{c1}, using a fixed stress ratio exponent, n (this method has stress units)	$q_{c1} = q_t \cdot (Pa/\sigma_v')^{0.5}$ where: P_a = atmospheric pressure	21
q_{c1} (0.5)	Normalized tip resistance, q_{c1}, using a fixed stress ratio exponent, n (this method is unit-less)	$q_{c1}\,(0.5) = (q_t/P_a) \cdot (Pa/\sigma_v')^{0.5}$ where: P_a = atmospheric pressure	5
q_{c1} (C_n)	Normalized tip resistance, q_{c1}, based on C_n (this method has stress units)	$q_{c1}(Cn) = C_n * q_t$	5, 12
q_{c1} (C_q)	Normalized tip resistance, q_{c1}, based on C_q (this method has stress units)	$q_{c1}\,(Cq) = C_q * q_t$ (some papers use q_c)	5, 12
q_{c1n}	normalized tip resistance, q_{c1n}, using a variable stress ratio exponent, n (where n=0.0, 0.70, or 1.0) (this method is unit-less)	$q_{c1n} = (q_t / P_a)(P_a/\sigma_v')^n$ where: P_a = atm. Pressure and n varies as described below	3

Calculated Parameter	Description	Equation	Ref
I_c or Ic (RW1998)	Soil Behavior Type Index as defined by Robertson and Wride (1997, 1998) for estimating grain size characteristics and providing smooth gradational changes across the SBTn chart. Ic(RW1998) is different from that of Jefferies and Davies (7) and is different from Ic(PKR2009).	$I_c = [(3.47 - \log_{10}Q)^2 + (\log_{10} Fr + 1.22)^2]^{0.5}$ *Where:* $Q = \left(\frac{qt - \sigma_v}{P_a}\right)\left(\frac{P_a}{\sigma_v'}\right)^n$ *Or* $Q = q_{c1n} = \left(\frac{qt}{P_a}\right)\left(\frac{P_a}{\sigma_v'}\right)^n$ *depending on the iteration in determining* I_c *And* *Fr is in percent* P_a *= atmospheric pressure* *n has the following distinct values:* *0.5, 0.75 and 1.0* *and is determined in an iterative manner based on the resulting* I_c *in each iteration* *Note that NCEER replaced 0.75 with 0.70*	3, 4, 5 10
I_c (PKR 2009)	Soil Behavior Type Index, I_c (PKR 2009) is based on a variable stress ratio exponent n, which itself is based on I_c (PKR 2009). An iterative calculation is required to determine I_c (PKR 2009) and its corresponding n (PKR 2009).	I_c *(PKR 2009) =* $[(3.47 - \log_{10}Q_{tn})^2 + (1.22 + \log_{10}F_r)^2]^{0.5}$	15
n (PKR 2009)	Stress ratio exponent n, based on I_c (PKR 2009). An iterative calculation is required to determine n (PKR 2009) and its corresponding I_c (PKR 2009).	$n \text{ (PKR 2009)} = 0.381 (I_c) + 0.05 (\sigma_v'/P_a) - 0.15$	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).	$Q_{tn} = [(qt - \sigma_v)/P_a](P_a/\sigma_v')^n$ *where* P_a *= atmospheric pressure (100 kPa)* *n = stress ratio exponent described above*	15
FC	Apparent fines content (%)	$FC=1.75(Ic^{3.25}) - 3.7$ $FC=100$ for $I_c > 3.5$ $FC=0$ for $I_c < 1.26$ $FC = 5\%$ if $1.64 < I_c < 2.6$ AND $F_r<0.5$	3
I_c Zone	This parameter is the Soil Behavior Type zone based on the I_c parameter (valid for zones 2 through 7 on SBTn or SBT Qtn charts)	$I_c < 1.31$ Zone = 7 $1.31 < I_c < 2.05$ Zone = 6 $2.05 < I_c < 2.60$ Zone = 5 $2.60 < I_c < 2.95$ Zone = 4 $2.95 < I_c < 3.60$ Zone = 3 $I_c > 3.60$ Zone = 2	3
CD	The contractive / dilative boundary on Robertson's Modified SBTn (contractive/dilative) Chart shown in Figure 6 above. The boundary is marked as CD = 70 on the chart in the relevant paper. Similar to the $Q_{tn,cs}$ = 70 line in Figure 4.	$CD = 70 = (Q_{tn} - 11) (1 + 0.06F_r)^{17}$ lower bound of CD = 60: $CD = 60 = (Q_{tn} - 9.5) (1 + 0.06F_r)^{17}$	30

Calculated Parameter	Description	Equation	Ref
I_B	Hyberbolic fit defining the boundary between SBT soil types proposed by Schneider as a better fit than the I_c circles. I_B = 32 represents the boundary for most sand like soils. I_B = 22 represents the upper boundary for most clay like soils. The region between I_B=22 and I_B=32 is the "transitional soil" zone.	$I_B = 100\,(Q_{tn} + 10) / (70 + Q_{tn}\,F_r)$	30
State Param or State Parameter or ψ	The state parameter index, ψ, is defined as the difference between the current void ratio, e, and the critical void ratio, e_c. Positive ψ - contractive soil Negative ψ - dilative soil This is based on the work by Been and Jefferies (1985) and Plewes, Davies and Jefferies (1992) This method uses mean normal stresses based on a uniform value of K_0 or a calculated K_0 using methods described elsewhere in this document	See reference	6, 8
Yield Stress σ_p'	Yield stress is calculated using the following methods 1) General method 2) 1st order approximation using q_tNet (clays) 3) 1st order approximation using Δu_2 (clays) 4) 1st order approximation using q_e (clays) 5) Based on Vs	All stresses in kPa 1) $\sigma_p' = 0.33\cdot(q_t - \sigma_v)^{m'}\,(\sigma_{atm}/100)^{1-m'}$ where $m' = 1 - \frac{0.28}{1+(I_c/2.65)^{25}}$ 2) $\sigma_p' = 0.33\cdot(q_t - \sigma_v)$ 3) $\sigma_p' = 0.54\cdot(\Delta u_2)$ $\Delta u_2 = u_2 - u_0$ 4) $\sigma_p' = 0.60\cdot(q_t - u_2)$ 5) $\sigma_p' = (Vs/4.59)^{1.47}$	19 20 20 20 18
OCR OCR(JS1978) YSR(Mayne2014) YSR (qtNet) YSR (deltaU) YSR (qe) YSR (Vs) OCR (PKR2015)	Over Consolidation Ratio based on 1) Schmertmann (1978) method involving a plot plot of S_u/σ_v' /($S_u/\sigma_v')_{NC}$ and OCR  2) based on Yield stresses described above 3) approximate version based on qtNet 4) approximate version based on Δu 5) approximate version based on effective tip, q_e 6) approximate version based on shear wave velocity, V_s and σ_v' 7) based on Qt	1) requires a user defined value for NC Su/P_c' ratio 2 through 5) *based on yield stresses* 6) YSR (Vs) = $\sigma_p'(Vs) / \sigma_v'$ 7) OCR = $0.25\cdot(Qt)^{1.25}$	9 19 20 20 20 18 32
Es/qt	Intermediate parameter for calculating Young's Modulus, E, in sands. It is the Y axis of the reference chart. Note that Figured 5.59 from reference 5, Lunne, Robertson and Powell, (LRP) has an error. The X axis values are too high by a factor of 10. The plot is based on Baldi's (not Bellotti as cited in	Based on Figure 5.59 in the reference	5, 37

Calculated Parameter	Description	Equation	Ref
	LRP) original Figure 3 where the X axis is: $\frac{q_c}{\sqrt{\sigma_v'}}$ (both in kPa) with a range of 200 to 3000. Figure 5.59 from LRP shows a dimensionless form of the equation, q_{c1}, displaying the same range of values. Figure 5.59's X axis uses $q_{c1} = \left(\frac{q_c}{P_a}\right)\left(\frac{P_a}{\sigma_v'}\right)^{0.5}$ The two expressions are not the same: they differ by a factor of $\frac{\sqrt{P_a}}{P_a}$. With P_a taken to be 100 kPa the factor is 1/10. Substituting typical values of 200 bar (20000 kPa) for q_c and 225 kPa for σ_v' one gets: 20000 / 15 = 1333.33 for Bellotti's axis and $(200/1)(100/225)^{0.5}$ = 200 * (10/15) = 133.3 for LRP's axis (noting that P_a = 1 bar) showing a factor of 10 difference.		
Es or E_s Young's Modulus E	Young's Modulus based on the work done in Italy. There are three types of sands considered in this technique. The user selects the appropriate type for the site from: a) OC Sands b) Aged NC Sands c) Recent NC Sands Each sand type has a family of curves that depend on mean normal stress. The program calculates mean normal stress and linearly interpolates between the two extremes provided in the E_s/q_t chart. E_s is evaluated for an axial strain of 0.1%.	Mean normal stress is evaluated from: $$\sigma_m' = \frac{1}{3}(\sigma_v' + \sigma_h' + \sigma_h')$$ where σ_v'= vertical effective stress σ_h'= horizontal effective stress *and* $\sigma_h = K_o \cdot \sigma_v'$ with K_o assumed to be 0.5	5
Delta U/TStress Δu / σ_v	Differential pore pressure ratio with respect to total stress	$= \frac{\Delta u}{\sigma_v}$ *where:* $\Delta u = u - u_{eq}$	39
Delta U/EStress, P Value, Excess Pore Pressure Ratio $\Delta u/\sigma_v'$	Differential pore pressure ratio with respect to effective stress. Key parameter (P, Normalized Pore Pressure Parameter, Excess Pore Pressure Ratio) in the Winckler et. al. static liquefaction method.	$= \frac{\Delta u}{\sigma_v'}$ *where:* $\Delta u = u - u_{eq}$	25, 25a
Su/EStress S_u/σ_v'	Undrained shear strength ratio with respect to vertical effective overburden stress using the S_u (N_{kt}) method	$= Su(N_{kt}) / \sigma_v'$	9, 23
Vs or V_s	Recorded shear wave velocities (not estimated). The shear wave velocities are typically collected over 1 m depth intervals. Each data point over the relevant depth range is assigned the same V_s value.	recorded data	27
Vp or V_p	Recorded compression wave (or P wave) velocities (not estimated). The P wave velocities are typically collected over 1 m depth intervals. Each data point over the relevant depth range is assigned the same V_p value.	recorded data	27

Calculated Parameter	Description	Equation	Ref
V_{s30} V_{s100}	The average shear wave velocity of the near surface materials to a depth of 30 m (100 ft). It is based on the sum of all travel times through all layers in the top 30m (100 ft). V_{s100} is the same calculation as V_{s30} except down to a depth of 100 feet.	$V_{s30} = \frac{total\ thickness\ of\ all\ layers\ to\ 30\ m}{\Sigma\left(\frac{layer\ thickness}{layer\ shear\ wave\ velocity}\right)}$ $V_{s30} = \frac{total\ thickness\ of\ all\ layers\ to\ 30\ m}{\Sigma\ (layer\ travel\ times)}$	38
G_{max}	G_{max} determined from SCPT shear wave velocities (not estimated values). Note that seismic data (V_s) is collected over set depth intervals (typically 1 meter). Each data point over the test segment is assigned the same V_s value. Since soil density changes with depth, slightly different G_{max} values may be calculated over the test depth interval.	$G_{max} = \rho V_s^2$ where ρ is the mass density of the soil determined from the estimated unit weights at each test depth	27
$qtNet/G_{max}$	Net tip resistance ratio with respect to the small strain modulus G_{max} determined from SCPT shear wave velocities (not estimated values)	$= (qt - \sigma_v) / G_{max}$ where $G_{max} = \rho V_s^2$ and ρ is the mass density of the soil determined from the estimated unit weights at each test depth	15, 28, 30
qUlt	A site specific and client specific parameter for estimating the limiting stress for "crane walk" accessibility	$q_{ult} = CraneWalkFactor \cdot S_u$ Where: *CraneWalkFactor* is client provided	U*
Estimated G_o	Estimated value for small strain shear modulus	$G_o = 0.0188[10^{(0.55Ic + 1,68)}](q_t - \sigma_v)$	15
Estimated E_{25}	Estimated value for Young's Modulus, E, at a 25% working load	$E_{25} = \alpha_E\ (qtNet)$ where $\alpha_E = 0.015[10^{(0.55Ic + 1,68)}]$	15
k_{SBT}	Estimated soil permeability derived from Soil Behavior Type (SBT) Chart I_c values.	For $1.0 < I_c \leq 3.27$: $k = 10^{(0.952 - 3.04Ic)}$ in m/s For $3.27 < Ic < 4.0$: $k = 10^{(-4.52 - 1.37Ic)}$ in m/s	35
M or D' Constrained Modulus	Constrained Modulus based on 1) Robertson, M 2) Mayne, D'	1) Robertson $M = \alpha_M\ (q_t - \sigma_v)$ $I_c > 2.2$ (fine grained) $\alpha_M = Qt$ when $Qt < 14$ $\alpha_M = 14$ when $Qt > 14$ $Ic < 2.2$ (coarse grained) $\alpha_M = 0.0188\ [10^{(0.55Ic + 1.68)}]$ $D' = \alpha_D\ (qt - \sigma_v)$ where $\alpha_D = 5$	32 23

Table 1b. CPT Parameter Calculation Methods – Liquefaction Parameters

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

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

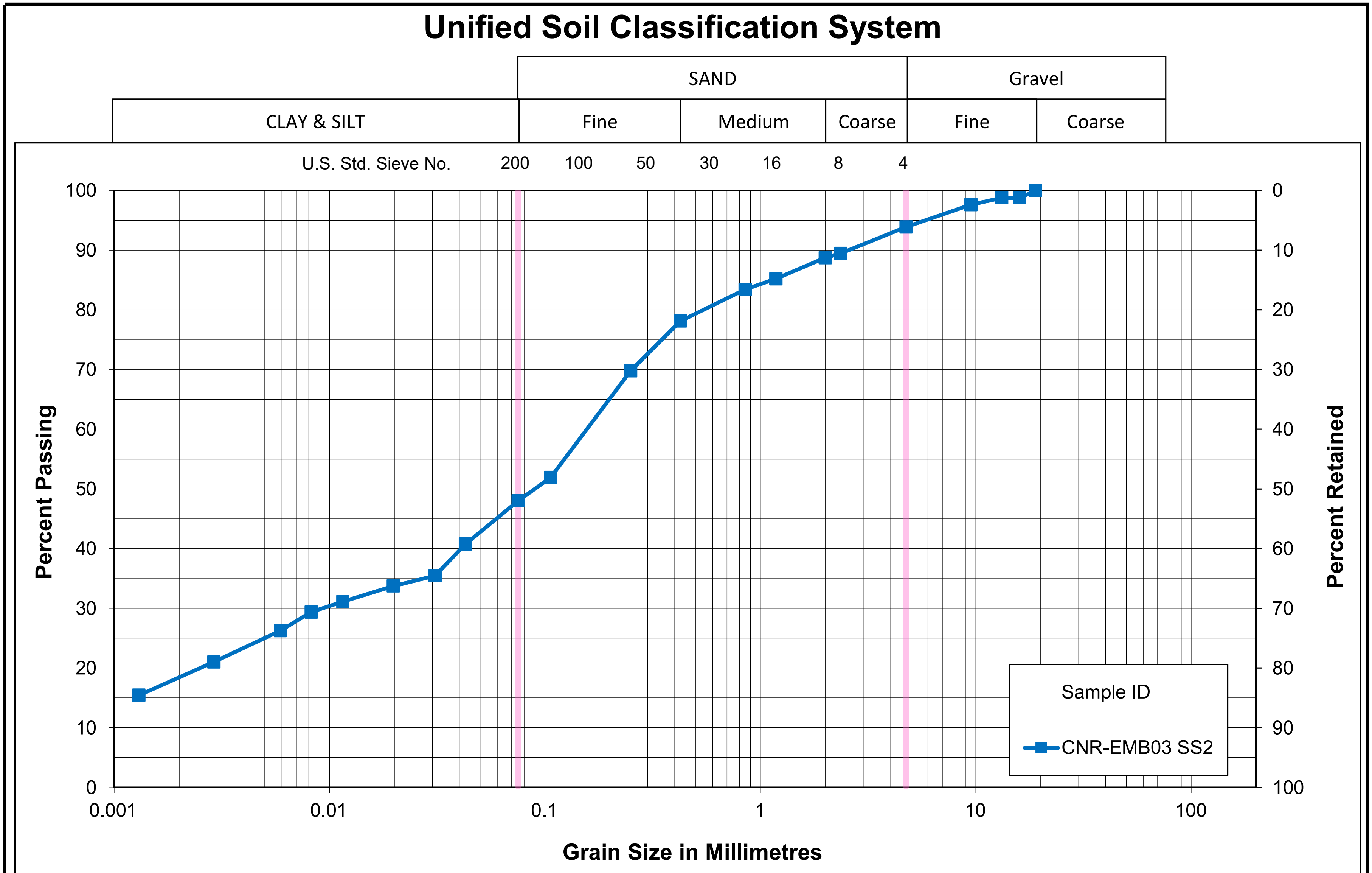
Table 2. References

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

No.	Reference
19	Mayne, P.W., 2014, "Generalized CPT Method for Evaluating Yield Stress in Soils", Geocharacterization for Modeling and Sustainability (GSP 235: Proc. GeoCongress 2014, Atlanta, GA), ASCE, Reston, Virginia: 1336-1346.
20	Mayne, P.W., 2015, "Geocharacterization by In-Situ Testing", Continuing Education Course, Vancouver, BC, January 6-8, 2015.
21	Robertson, P.K. and Fear, C.E., 1995, "Liquefaction of sands and its evaluation", Proceedings of the First International Conference on Earthquake Engineering, Keynote Lecture IS Tokyo '95, Tokyo Japan, 1995.
22	Mayne, P.W., Peuchen, J. and Boumeester, D., 2010, "Soil unit weight estimation from CPTs", Proceeding of the 2nd International Symposium on Cone Penetration Testing (CPT '10), Vol 2, Huntington Beach, California; Omnipress: 169-176.
23	Mayne, P.W., 2007, "NCHRP Synthesis 368 on Cone Penetration Test", Transportation Research Board, National Academies Press, Washington, D.C., 118 pages.
24	Mayne, P.W., 2014, "Interpretation of geotechnical parameters from seismic piezocone tests.", Key note address #2, proceedings, 3rd International Symposium on Cone Penetration Testing (CPT'14, Las Vegas), ISSMGE Technical Committee TC102.
25	Winckler, Christina, Davidson, Richard, Yenne, Lisa, Pilz, Jorgen, 2014, "CPTu-Based State Characterization of Tailings Liquefaction Susceptibility", Tailings and Mine Waste, 2014.
25a	Winckler, Christina, Davidson, Richard, Yenne, Lisa, Pilz, Jorgen, 2014, "CPTu-Based State Characterization of Tailings Liquefaction Susceptibility", Powerpoint presentation, Tailings and Mine Waste, 2014.
26	Schneider, J.A. and Moss, R.E.S., 2011, "Linking cyclic stress and cyclic strain based methods for assessment of cyclic liquefaction triggering in sands", Geotechnique Letters 1, 31-36.
27	Rice, A., 1984, "The Seismic Cone Penetrometer", M.A.Sc. thesis submitted to the University of British Columbia, Dept. of Civil Engineering, Vancouver, BC, Canada.
28	Gillespie, D.G., 1990, "Evaluating Shear Wave Velocity and Pore Pressure Data from the Seismic Cone Penetration Test", Ph.D. thesis submitted to the University of British Columbia, Dept. of Civil Engineering, Vancouver, BC, Canada.
29	Robertson, P.K and Cabal, K.L., 2010, "Estimating soil unit weight from CPT", Proceedings of the 2nd International Symposium on Cone Penetration Testing (CPT '10), Huntington Beach, California.
30	Robertson, P.K., 2016, "Cone penetration test (CPT)-based soil behaviour type (SBT) classification system – an update", Canadian Geotechnical Journal, July 2016.
31	Robertson, P.K., 2012, "Interpretation of in-situ tests – some insights", Mitchell Lecture, ISC'4, Recife, Brazil.
32	Robertson, P.K., Cabal, K.L. 2015, "Guide to Cone Penetration Testing for Geotechnical Engineering", 6th Edition.
33	Robertson, P.K., 2010b, "Soil behaviour type from CPT: an update", Proceedings of the 2nd International Symposium on Cone Penetration Testing (CPT '10), Huntington Beach, California.
34	Been, K., Romero, S., Obermeyer, J. and Hebeler, G., 2012, "Determining in situ state of sand and silt tailings from the CPT", Tailings and Mine Waster 2012, 325-333.
35	Robertson, P.K., 2010, "Estimating in-situ soil permeability from CPT & CPTu", Proceedings of the 2nd International Symposium on Cone Penetration Testing (CPT '10), Huntington Beach, California.
36	Mayne, P.W., Cargill, E. and Greig, J., 2023, "The Cone Penetration Test: A CPT Design Parameter Manual", ConeTec Group
37	Baldi, G., Bellotti, R., Ghionna,V., Jamiolkowski, M. and Lo Presti, D. 1989. Modulus of sands from CPTs and DMTs. *Proc. Intl. Conf. on Soil Mechanics & Foundation Engineering*, Vol. 1 (ICSMFE, Rio de Janeiro), Balkema, Rotterdam: 165–170. www.issmge.org
38	Crow, H.L, Hunter, J.A. and Bobrowsky, P.T., 2012, "National shear wave measurement guidelines for Canadian seismic site assessment", Proceedings of GeoManitoba 2012, the 65th Canadian Geotechnical Conference.
39	Campanella, R.G., Robertson, P.K., Gillespie, D., 1982, "Cone penetration testing in deltaic soils", Canadian Geotechnical Journal, 20: 23-35.

APPENDIX D

D.1 SOIL LABORATORY TEST RESULTS



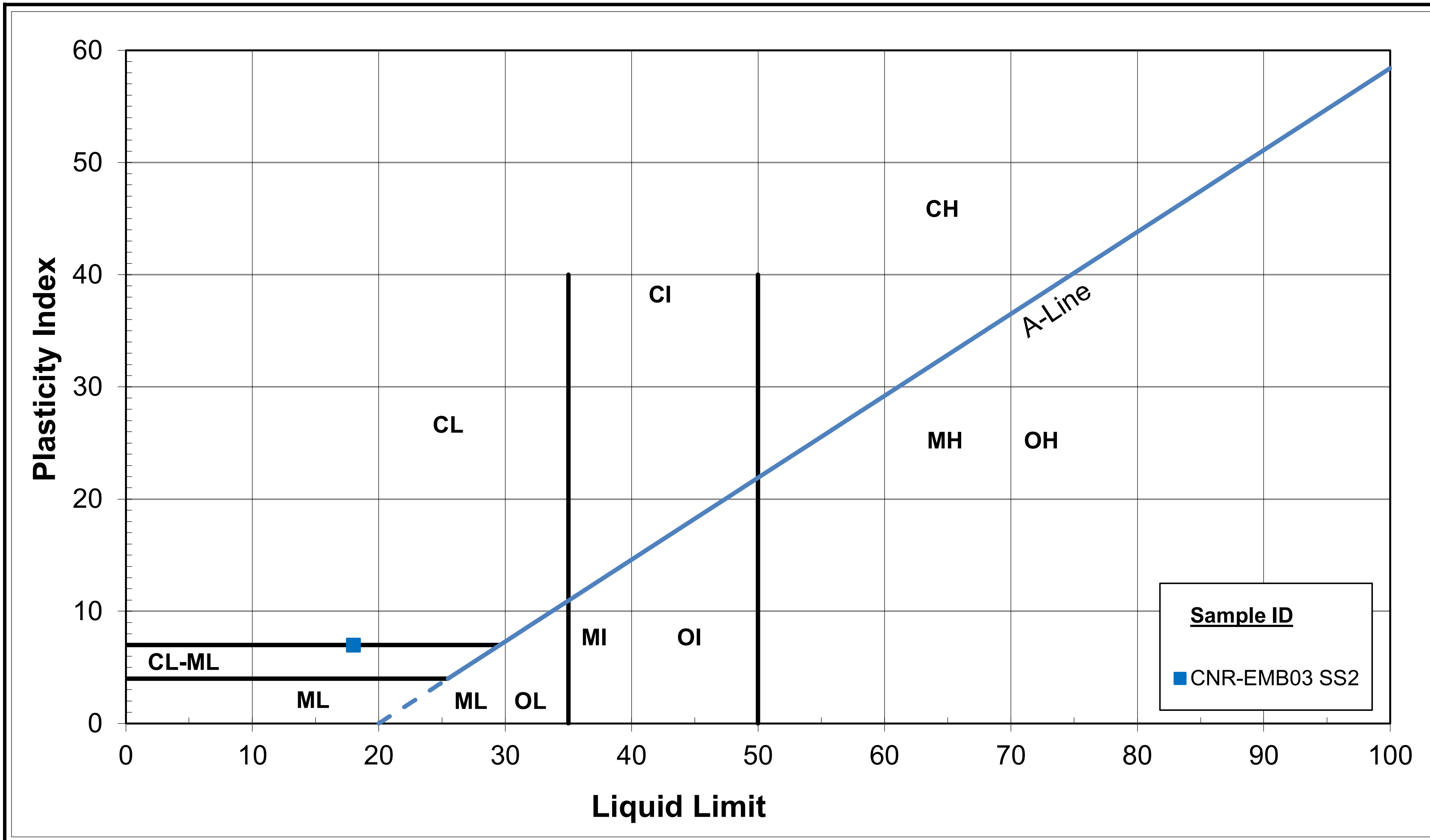
FILL: Silty Clayey SAND (SC-SM)

Ministry of Transportation (MTO)

HWY 3 Twinning - CNR Overhead Bridge

Figure No. D1

Project No. 165001308



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FILL: Silty Clayey SAND (SC-SM)

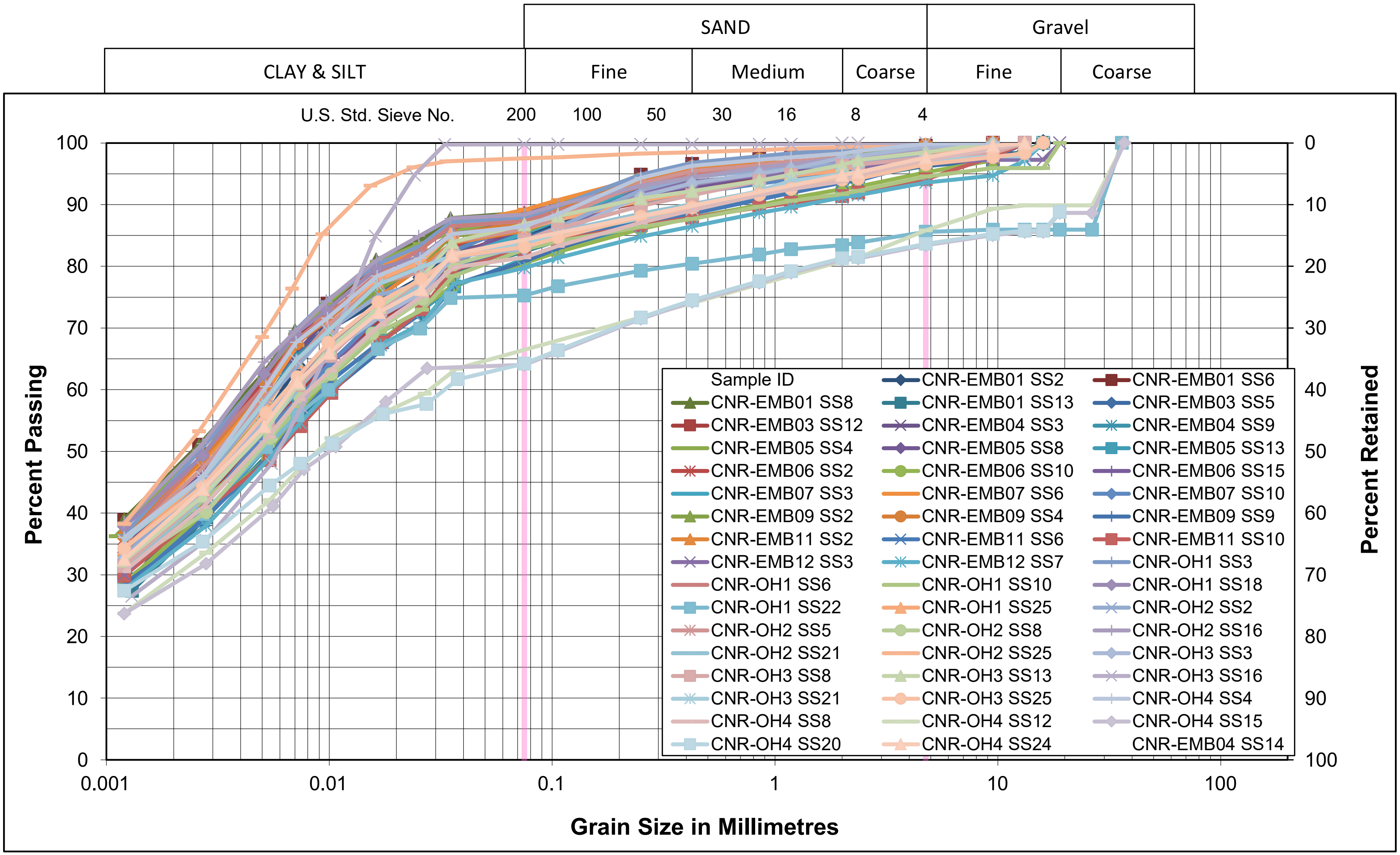
Ministry of Transportation (MTO)

HWY 3 Twinning - CNR Bridge

Figure No. D2

Project No. 165001308

Unified Soil Classification System



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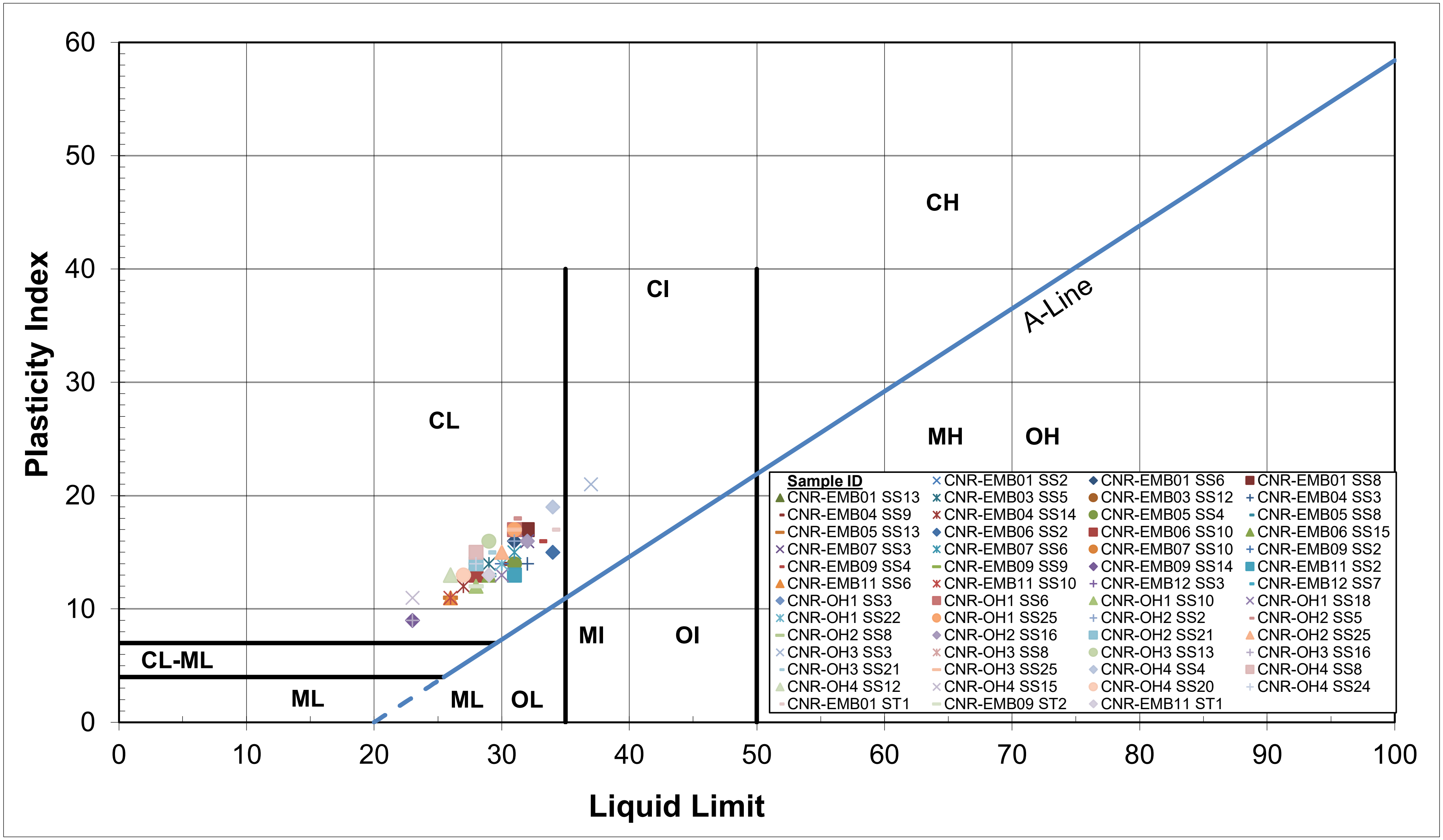
TILL: CLAYEY SILT (CL) to SILTY CLAY (CI)

Ministry of Transportation (MTO)

HWY 3 Twinning - CNR Overhead Bridge

Figure No. D3

Project No. 165001308



Stantec

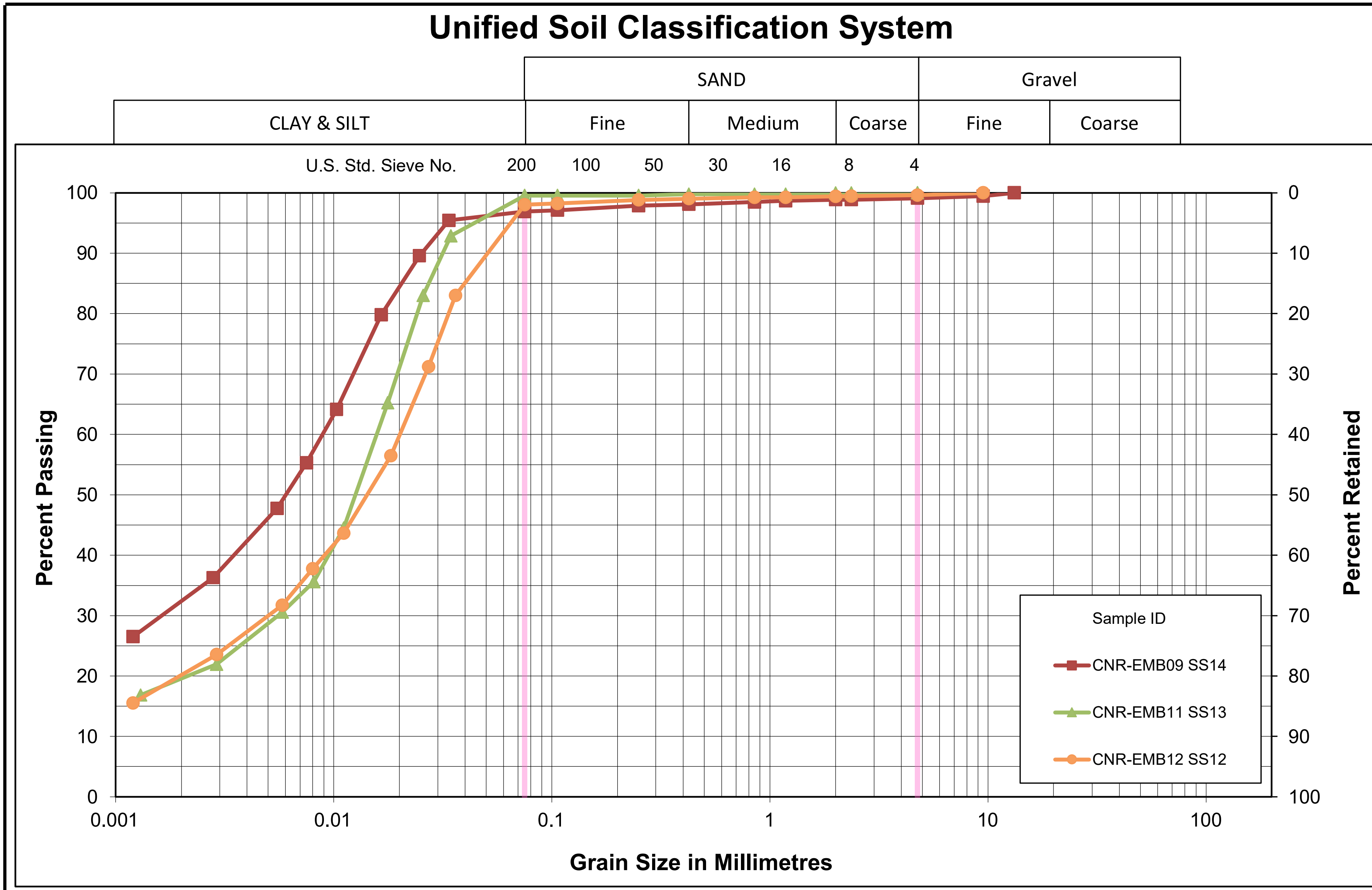
TILL: CLAYEY SILT (CL) to SILTY CLAY (CI)

Ministry of Transportation (MTO)

HWY 3 Twinning - CNR Bridge

Figure No. D4

Project No. 165001308



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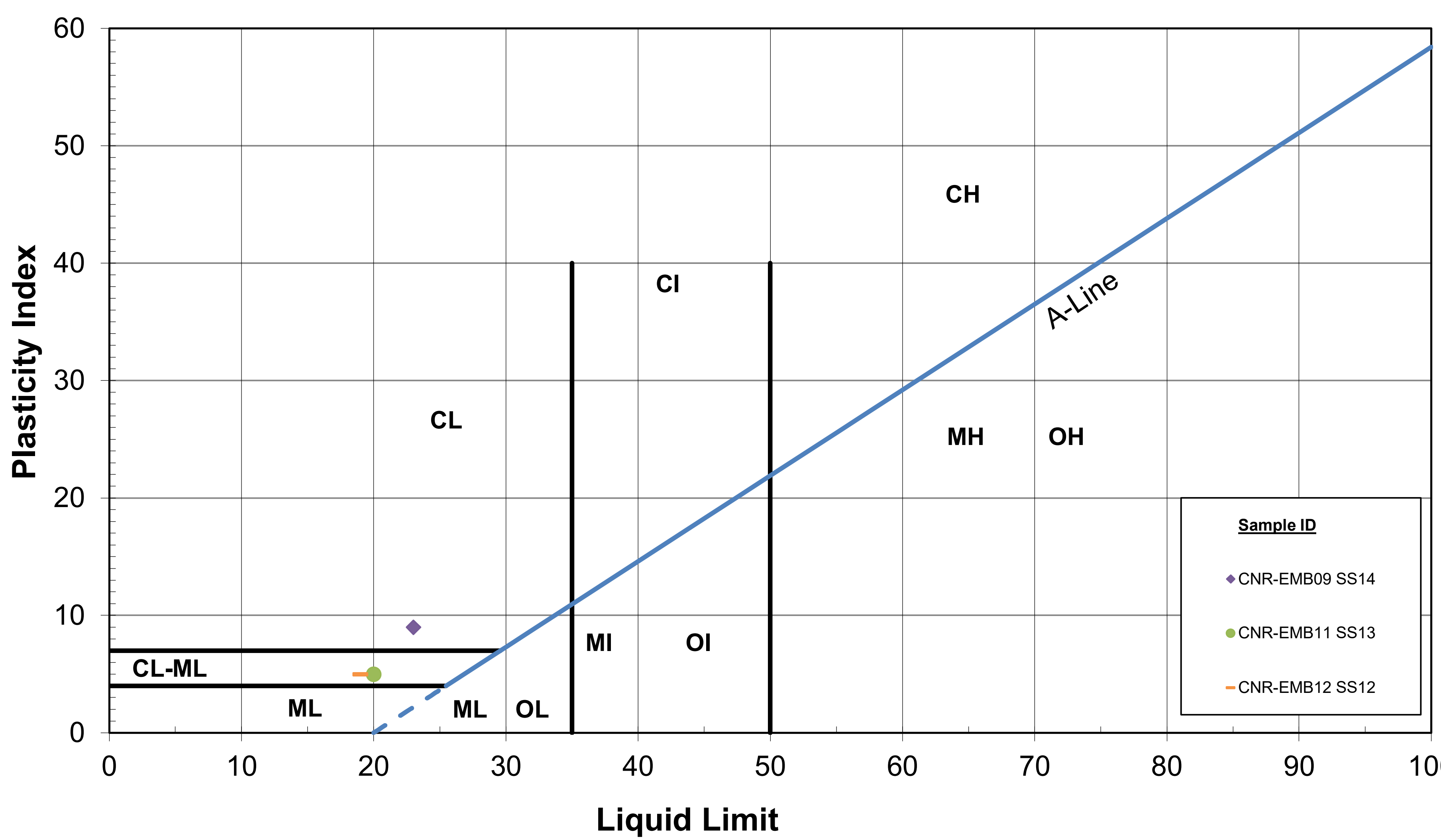
CLAYEY SILT (CL-ML to CL)

Ministry of Transportation (MTO)

HWY 3 Twinning - CNR Overhead Bridge

Figure No. D5

Project No. 165001308



Stantec	**CLAYEY SILT (CL-ML to CL)** **Ministry of Transportation (MTO)** **HWY 3 Twinning - CNR Bridge**	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 CNR OH3, ST1	Sample Type:	Intact
Sample Depth (ft):	13-15	Soil Classification:	CL
Liquid Limit:	31.0%	Specific Gravity:	2.761
Plastic limit:	16.8%		

Soil Description & Classification: *Lean clay of low plasticity, stiff, brown, moist, CL*

INITIAL SPECIMEN DIMENSIONS AND PROPERTIES

Test No	1	2
Specimen Height, (mm)	152	152
Specimen Diameter, (mm)	70	70
Natural Water Content (Cuttings), (%)	19.2	16.9
Void Ratio	0.57	0.52
Degree of Saturation, (%)	93.1	89.9
Dry Unit Weight (kN/m^3)	17.26	17.75

SHEARING/FAILURE

Max. Deviator Stress, (σ_1-σ_3), (kPa)	179.7	189.6
Axial Strain At Maximum (σ_1-σ_3), (%)	12.23	11.79
Compressive Strength, Max, (kN)	0.8	0.8
Max Total Principal Stress Ratio,(σ_1/σ_3)	2.8	1.9
Deviator Stress At (σ_1/σ_3) Max, (kPa)	179.7	189.6
Total Major Principal Stress At Failure, σ_1, (kPa)	278.3	389.5
Total Minor Principal Stress At Failure, σ_3, (kPa)	98.6	200.0
Average Rate of Strain, (%/min)	1.00	1.00

Test Notes:

Specimen Saturation Method	N/A	N/A
Failure Criterion	Max. σ_1-σ_3	Max. σ_1-σ_3
Membrane Thickness Correction Applied, Y/N	Y	Y

Project No.: 165001308.451.200

Date: August 19, 2024

Stantec

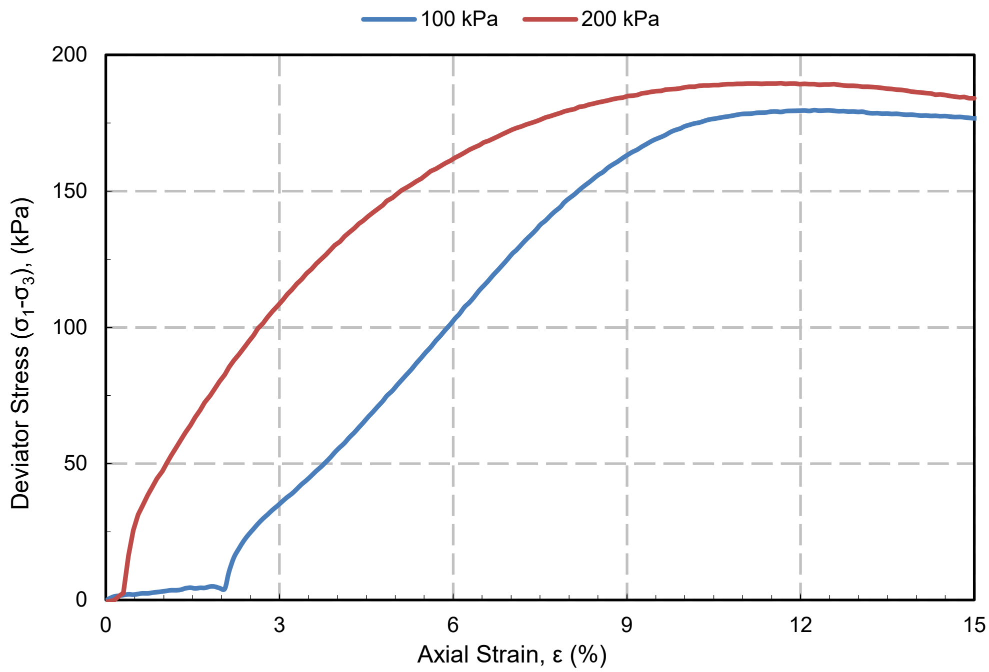
Prepared By : DB

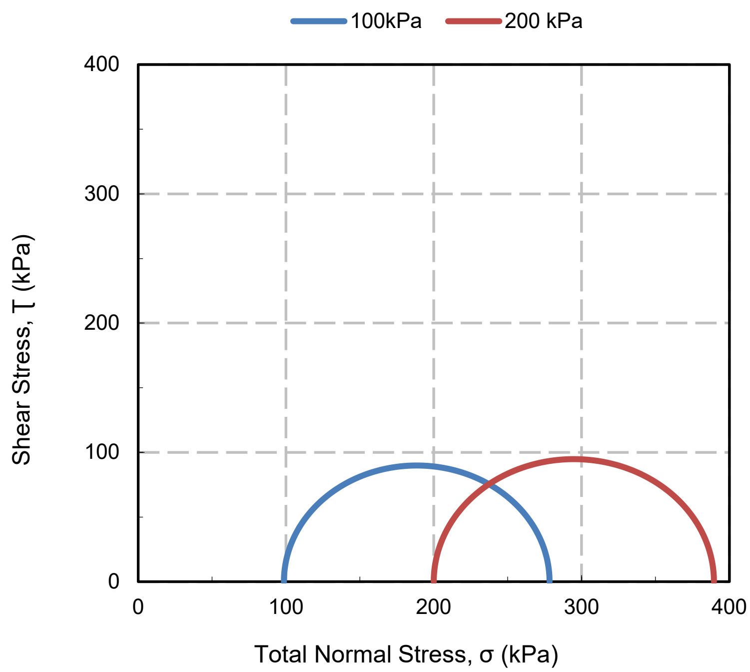
Checked By : RG

UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST (ASTM D2850)	Figures 1-2

MTO Hwy 3 Talbotville Bypass

BH CNR OH3, ST1





Project No.: 165001308.451.200
Date: August 19, 2024

Stantec

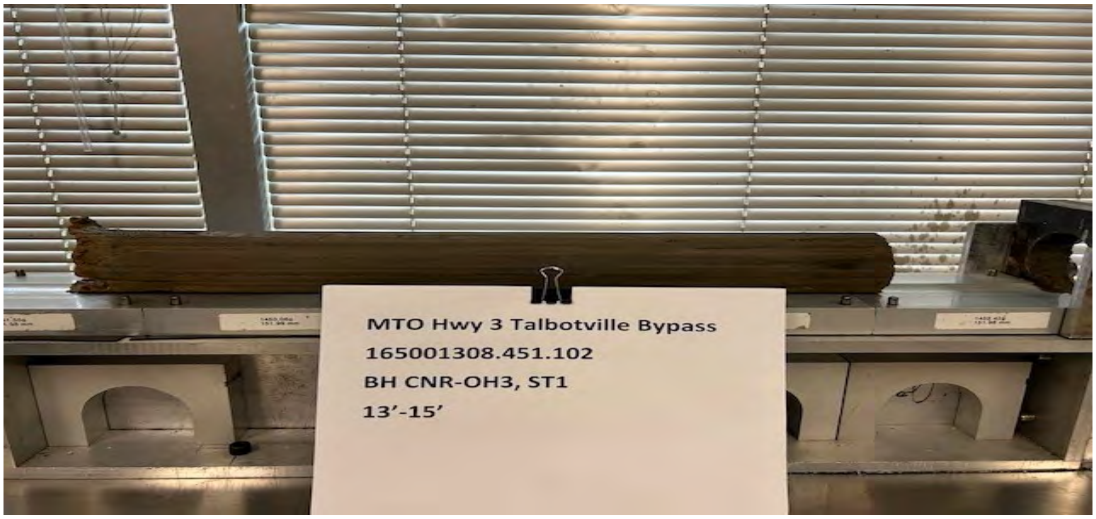
Prepared By : DB
Checked By : RG

PHOTOGRAPHS OF EXTRUDED/SLICED SPECIMENS	PHOTOS 1-2

MTO Hwy 3 Talbotville Bypass

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

BH CNR OH3, ST1



Project No. :	165001308.451.200	Stantec	Prepared by :	DB
Date :	August 19, 2024		Checked by :	RG

MTO Hwy 3 Talbotville Bypass

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

BH CNR OH3, ST1

100 kPa Shearing

200 kPa Shearing

CONSOLIDATION TEST SUMMARY

SAMPLE IDENTIFICATION

Borehole No. :	BH CNR EMB1	Sample No. :	ST1
		Sample Depth (ft) :	17.5-19.5

TEST CONDITIONS

Test Type :	ASTM D2435/D2435M	Date Started :	19-Aug-24
Load Duration (hr) :	Method B	Date Completed :	21-Aug-24

SAMPLE DIMENSIONS AND PROPERTIES _ INITIAL

Sample Height (mm) :	20.50	Unit Weight (kN/m^3) :	21.19
Sample Diameter (mm) :	50.00	Dry Unit Weight (kN/m^3) :	17.95
Area (cm^2) :	19.63	Specific Gravity : (Assumed)	2.761
Volume (cm^3) :	40.25	Solid Height (mm) :	13.59
Water Content (%) :	18.08	Volume of Solids (cm^3) :	26.68
Wet Mass (g) :	86.98	Volume of Voids (cm^3) :	13.57
Dry Mass (g) :	73.66	Degree of Saturation (%) :	98.14

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			0.509				
10	20.4213	0.0787	0.51	0.501	129.14	6.85E-03	5.06E-04	3.40E-09
20	20.3335	0.1665	0.87	0.496	1117.35	7.86E-04	3.68E-04	2.84E-10
40	20.2334	0.2666	1.49	0.486	585.65	1.49E-03	3.07E-04	4.47E-10
80	20.0674	0.4326	2.36	0.473	603.68	1.42E-03	2.17E-04	3.02E-10
160	19.8192	0.6808	3.51	0.456	807.31	1.04E-03	1.45E-04	1.47E-10
320	19.5457	0.9543	4.88	0.435	613.86	1.33E-03	8.55E-05	1.12E-10
160			4.68	0.438				
80			4.38	0.443				
160	19.5716	0.9284	4.56	0.440	229.99	3.53E-03	2.24E-05	7.78E-11
320	19.4896	1.0104	5.03	0.433	287.02	2.81E-03	2.94E-05	8.11E-11
480	19.3550	1.1450	5.73	0.422	563.37	1.42E-03	4.41E-05	6.13E-11
640	19.2231	1.2769	6.36	0.413	652.87	1.21E-03	3.95E-05	4.67E-11
1280	18.8632	1.6368	8.22	0.385	445.05	1.72E-03	2.90E-05	4.89E-11
2560	18.4444	2.0556	10.30	0.353	389.94	1.88E-03	1.62E-05	3.00E-11
4800	18.0496	2.4504	12.32	0.323	291.57	2.41E-03	9.01E-06	2.13E-11
2560			12.12	0.326				
640			11.15	0.341				
160			9.60	0.364				
40			8.15	0.386				
10			6.74	0.407				

SAMPLE DIMENSIONS AND PROPERTIES _ FINAL

Sample Height (mm) :	19.12	Unit Weight (kN/m^3) :	22.19
Sample Diameter (mm) :	50.00	Dry Unit Weight (kN/m^3) :	19.24
Area (cm^2) :	19.63	Specific Gravity (Assumed) :	2.761
Volume (cm^3) :	37.54	Solid Height (mm) :	13.59
Water Content (%) :	15.30	Volume of Solids (cm^3) :	26.68
Wet Mass (g) :	84.93	Volume of Voids (cm^3) :	10.86
Dry Mass (g) :	73.66		

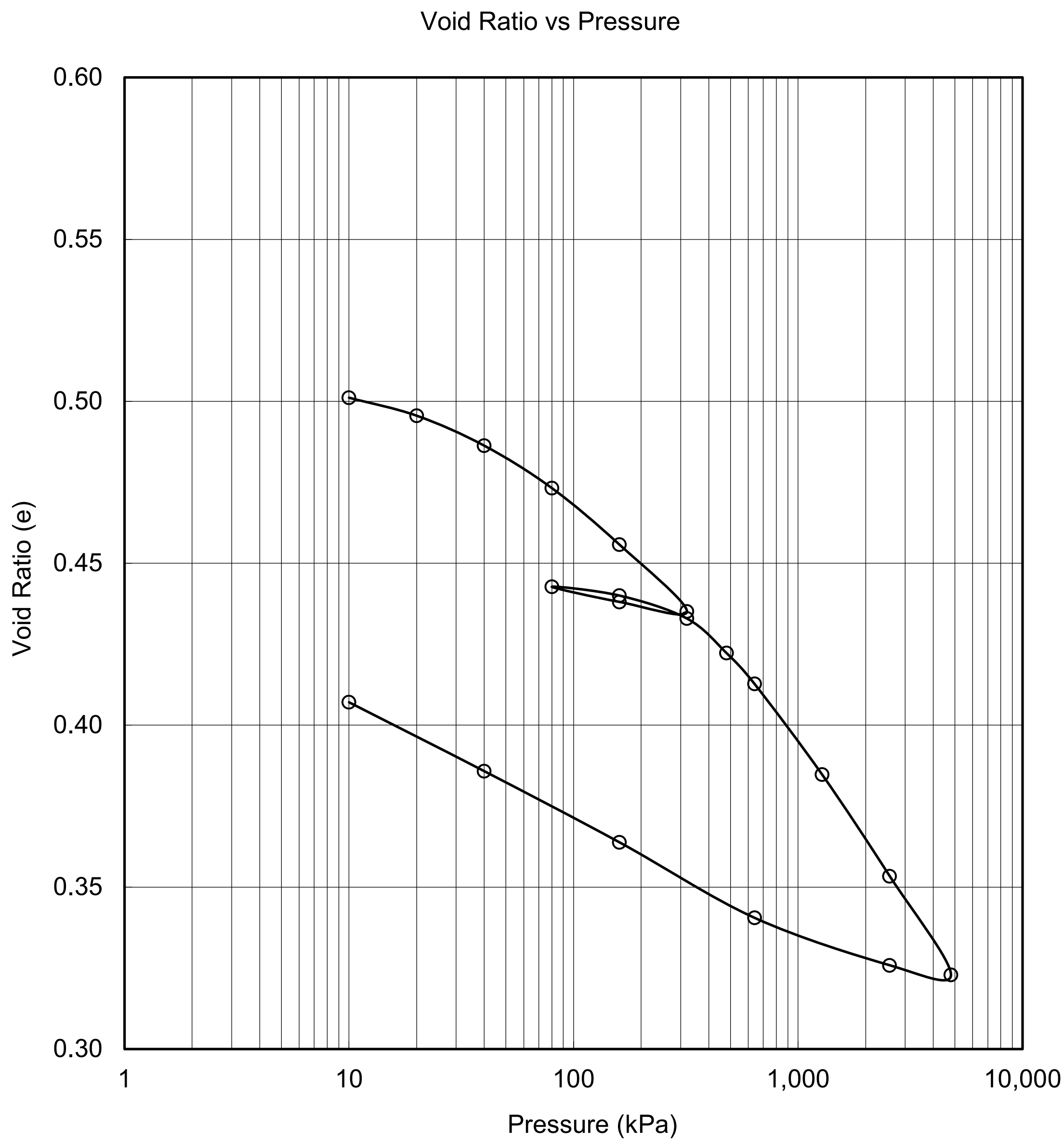
Project No. :	165001308.451.102	Prepared By :	DB
Date :	24-Aug-24	Checked By :	RG

Stantec

CONSOLIDATION TEST	FIGURE 1

MTO Hwy 3 Talbotville Bypass

BH CNR EMB1, ST1



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

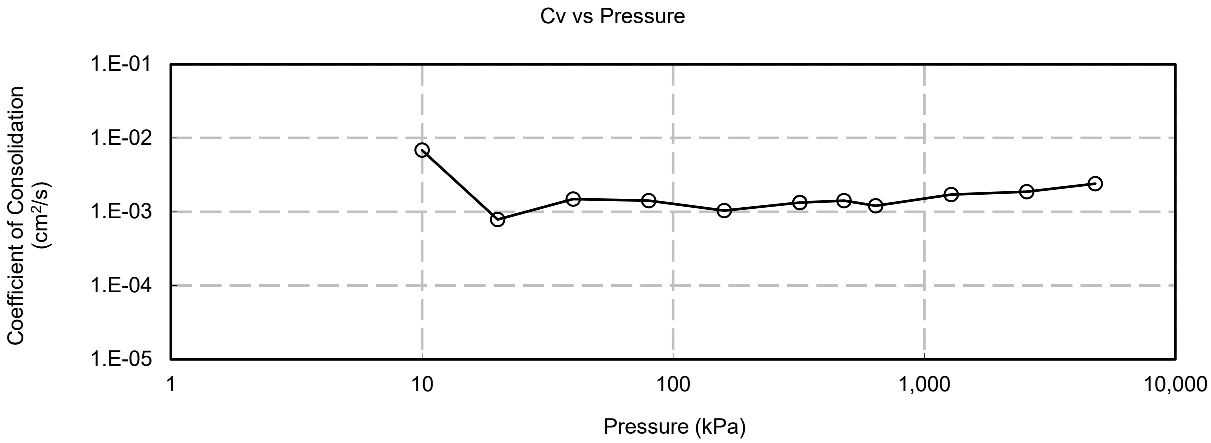
e_o =	0.509	w_L =	33.7%	σ_{v0}' =	kPa
w =	18.1%	w_P =	17.3%	σ_P' =	kPa
γ =	21.2 kN/m^3	PI =	16.4%		
Gs =	2.761				

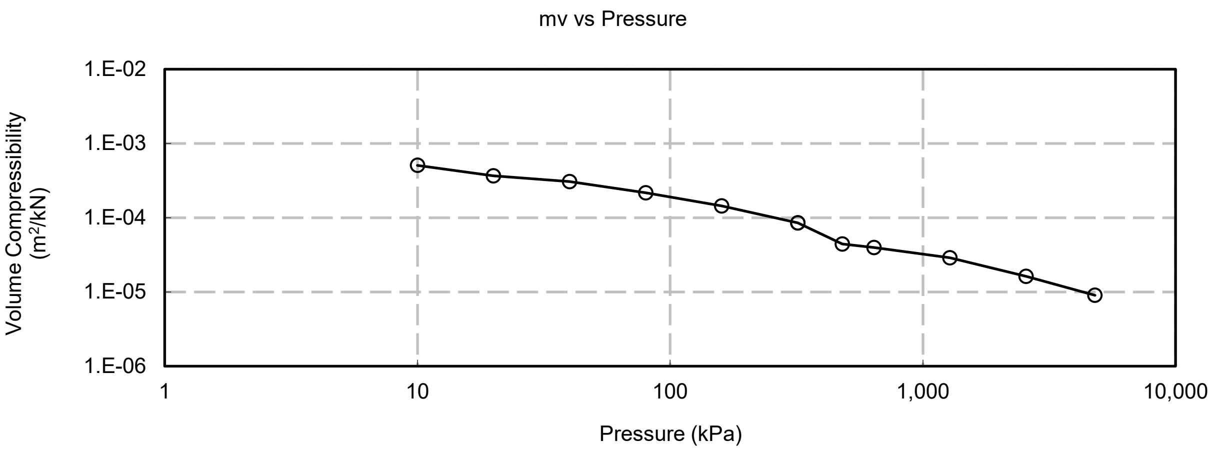
Project No. :	165001308.451.102	Prepared By :	DB
Date :	24-Aug-24	Checked By :	RG

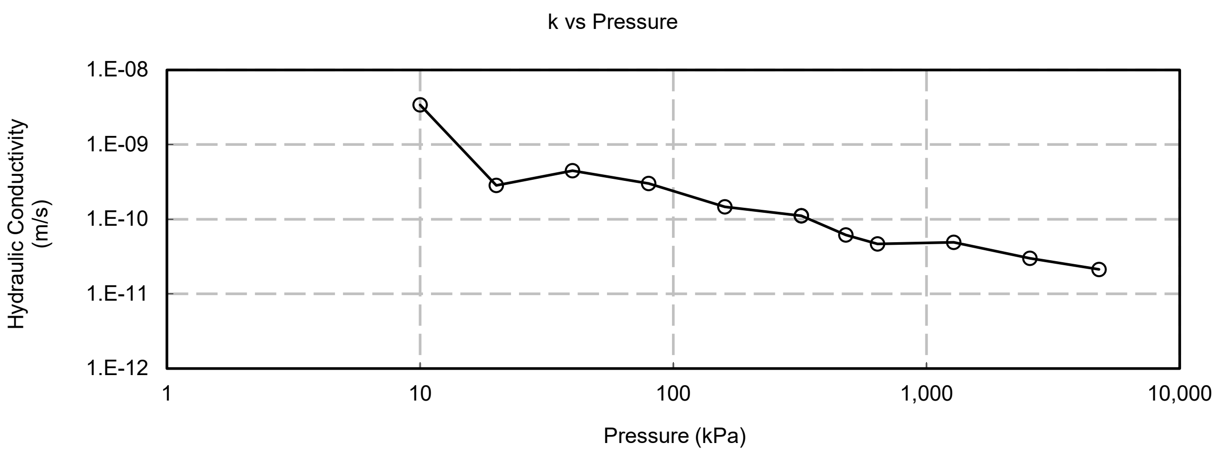
CONSOLIDATION TEST	**FIGURES 2, 3 & 4**

MTO Hwy 3 Talbotville Bypass

BH CNR EMB1, ST1



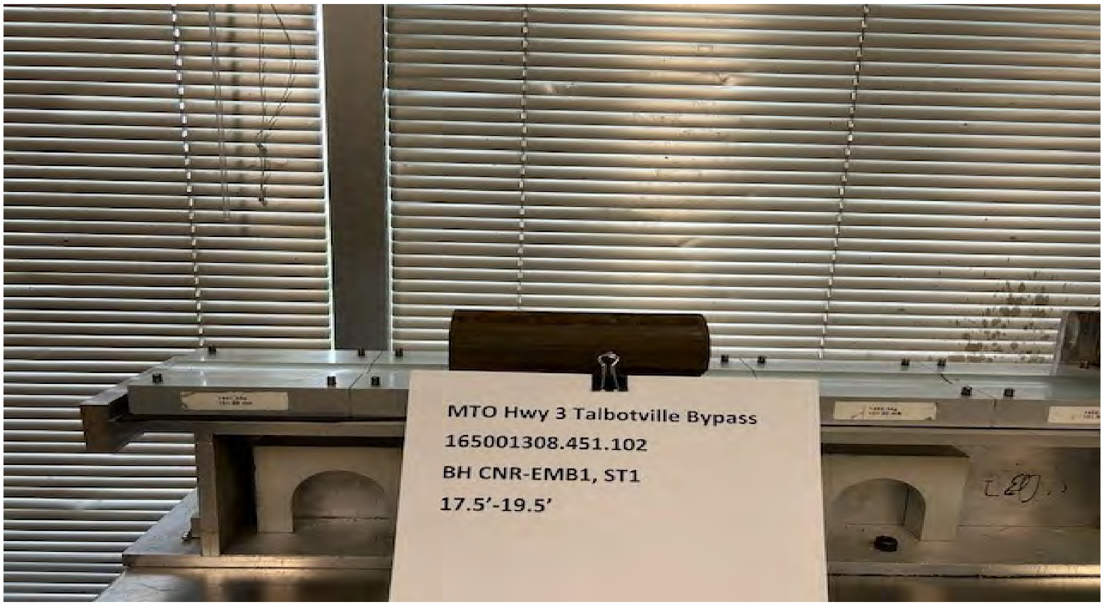




Project No. :	165001308.451.102	Stantec	Prepared By :	DB
Date :	24-Aug-24		Checked By :	RG

MTO Hwy 3 Talbotville Bypass

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



BH CNR EMB1, ST1

BH CNR EMB1, ST1

Project No. :	165001308.451.102	Prepared by :	DB
Date :	24-Aug-2024	Checked by :	RG

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CONSOLIDATION TEST SUMMARY

SAMPLE IDENTIFICATION

Borehole No. :	BH CNR EMB7	Sample No. :	ST1
		Sample Depth (ft) :	12.5-14.5

TEST CONDITIONS

Test Type :	ASTM D2435/D2435M	Date Started :	13-Aug-24
Load Duration (hr) :	Method B	Date Completed :	15-Aug-24

SAMPLE DIMENSIONS AND PROPERTIES _ INITIAL

Sample Height (mm) :	20.50	Unit Weight (kN/m^3) :	20.78
Sample Diameter (mm) :	50.00	Dry Unit Weight (kN/m^3) :	17.43
Area (cm^2) :	19.63	Specific Gravity : (Assumed)	2.757
Volume (cm^3) :	40.25	Solid Height (mm) :	13.21
Water Content (%) :	19.24	Volume of Solids (cm^3) :	25.94
Wet Mass (g) :	85.29	Volume of Voids (cm^3) :	14.31
Dry Mass (g) :	71.53	Degree of Saturation (%) :	96.18

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	0.0000	0.00	0.551				
10	20.2257	0.2743	1.51	0.528	94.84	9.16E-03	1.51E-03	1.35E-08
20	20.1224	0.3776	2.02	0.520	192.49	4.47E-03	5.19E-04	2.27E-09
40	19.9786	0.5214	2.82	0.508	233.92	3.63E-03	3.97E-04	1.41E-09
80	19.7848	0.7152	3.84	0.492	258.97	3.22E-03	2.56E-04	8.08E-10
160	19.5429	0.9571	5.17	0.471	213.09	3.82E-03	1.66E-04	6.21E-10
320	19.2069	1.2931	6.78	0.446	341.23	2.31E-03	1.01E-04	2.28E-10
160			6.58	0.449				
80			6.23	0.455				
160	19.1877	1.3123	6.44	0.452	314.04	2.49E-03	2.70E-05	6.58E-11
320	19.0989	1.4011	6.97	0.443	300.85	2.58E-03	3.33E-05	8.42E-11
480	18.9467	1.5533	7.81	0.430	508.69	1.50E-03	5.20E-05	7.68E-11
640	18.7787	1.7213	8.55	0.419	805.35	9.33E-04	4.67E-05	4.28E-11
1280	18.3603	2.1397	10.85	0.383	458.48	1.58E-03	3.59E-05	5.57E-11
2560	17.8597	2.6403	13.44	0.343	347.12	1.98E-03	2.03E-05	3.94E-11
4800	17.3326	3.1674	16.05	0.302	338.94	1.92E-03	1.16E-05	2.19E-11
2560			15.74	0.307				
640			14.46	0.327				
160			12.81	0.353				
40			10.92	0.382				
10			9.53	0.404				

SAMPLE DIMENSIONS AND PROPERTIES _ FINAL

Sample Height (mm) :	18.55	Unit Weight (kN/m^3) :	22.34
Sample Diameter (mm) :	50.00	Dry Unit Weight (kN/m^3) :	19.26
Area (cm^2) :	19.63	Specific Gravity (Assumed) :	2.757
Volume (cm^3) :	36.42	Solid Height (mm) :	13.21
Water Content (%) :	15.95	Volume of Solids (cm^3) :	25.94
Wet Mass (g) :	82.94	Volume of Voids (cm^3) :	10.47
Dry Mass (g) :	71.53		

Project No. :	165001308.451.102	Prepared By :	DB
Date :	16-Aug-24	Checked By :	RG

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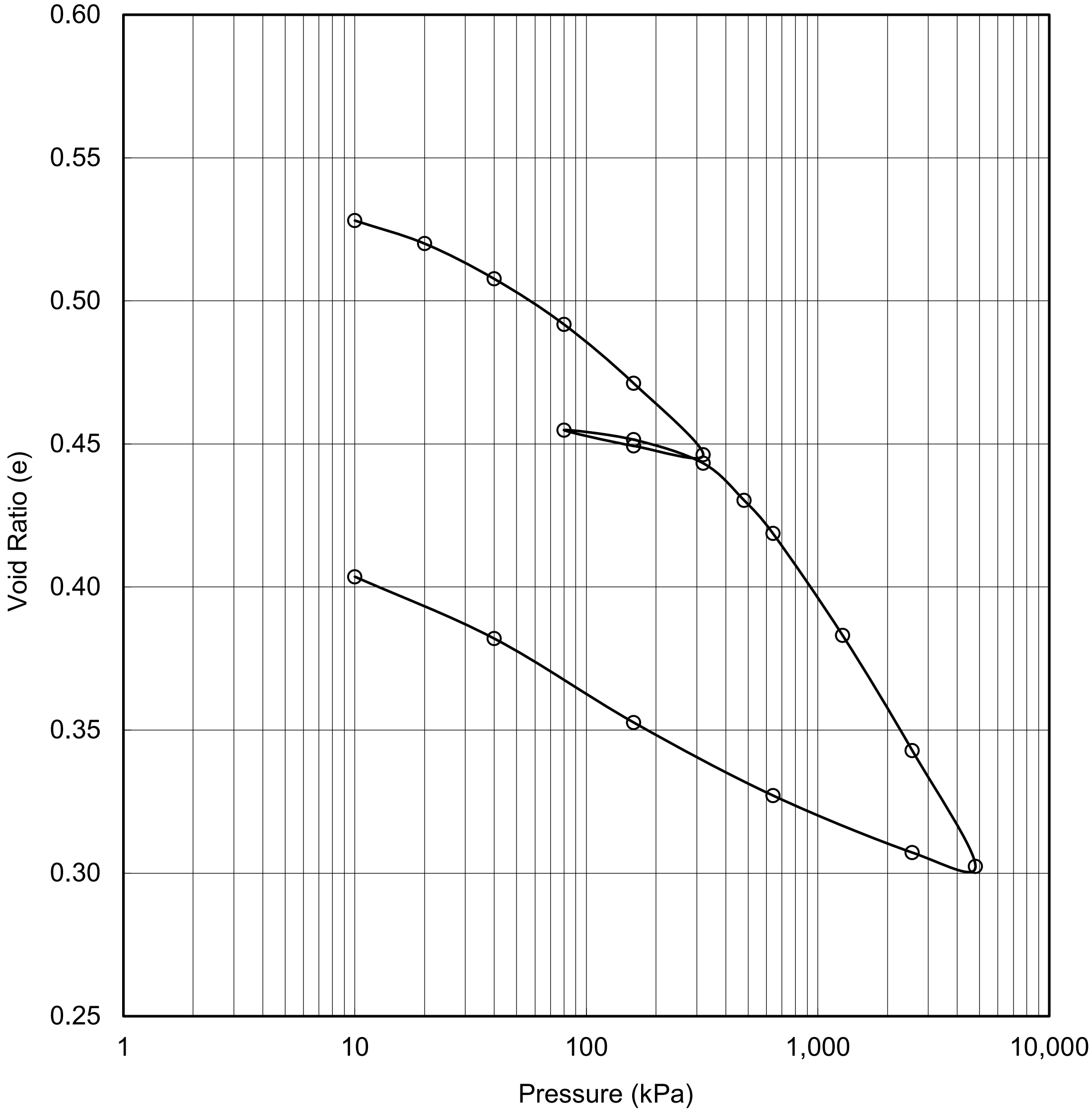
CONSOLIDATION TEST	FIGURE 1

MTO Hwy 3 Talbotville Bypass

BH CNR EMB7, ST1

Void Ratio vs Pressure



NB: *Not enough sample available for Atterberg Limits Testing*

Soil Type : *Overconsolidated silty clay , hard, brown, moist*

e_o =	0.551	w_L =	N/A	σ_{v0}' =	kPa
w =	19.2%	w_P =	N/A	σ_P' =	kPa
γ =	20.8 kN/m^3	PI =	N/A		
Gs =	2.757				

Project No. : 165001308.451.102
Date : 16-Aug-24

Stantec

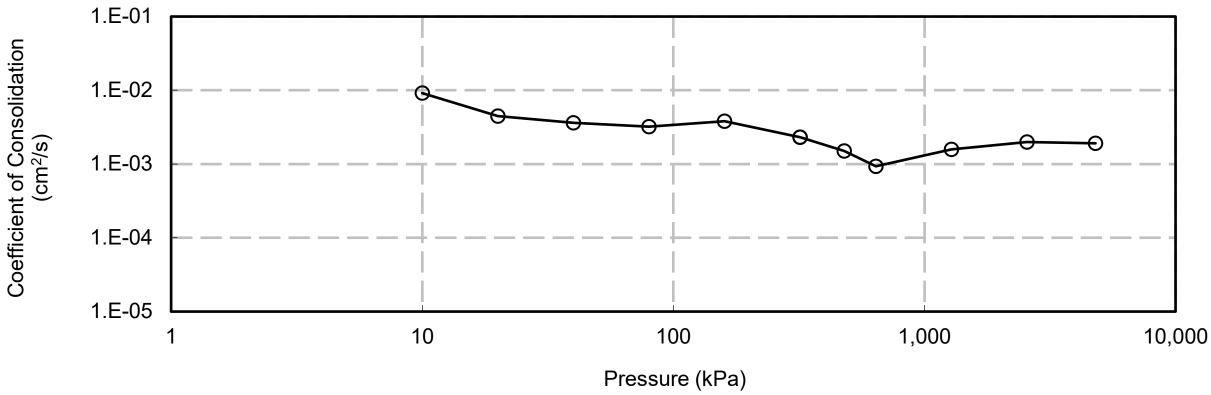
Prepared By : DB
Checked By : RG

CONSOLIDATION TEST	**FIGURES 2, 3 & 4**

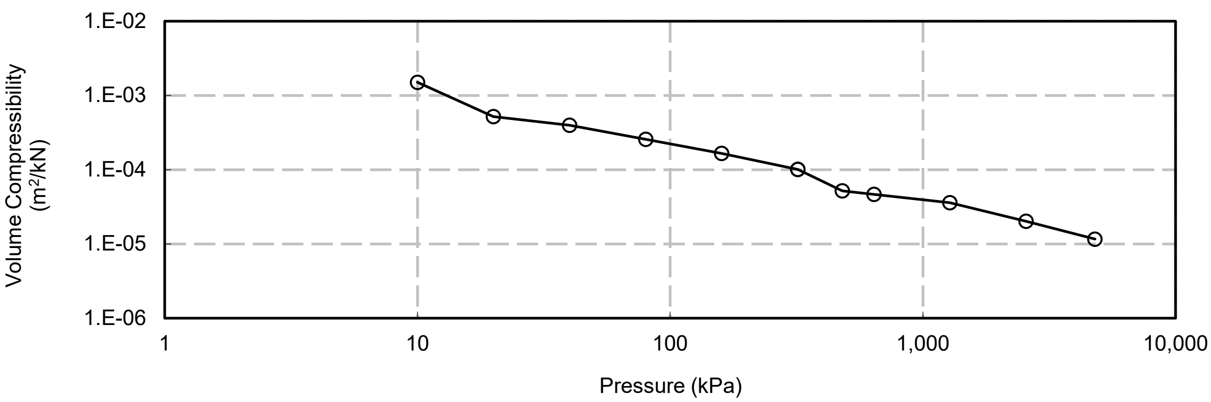
MTO Hwy 3 Talbotville Bypass

BH CNR EMB7, ST1

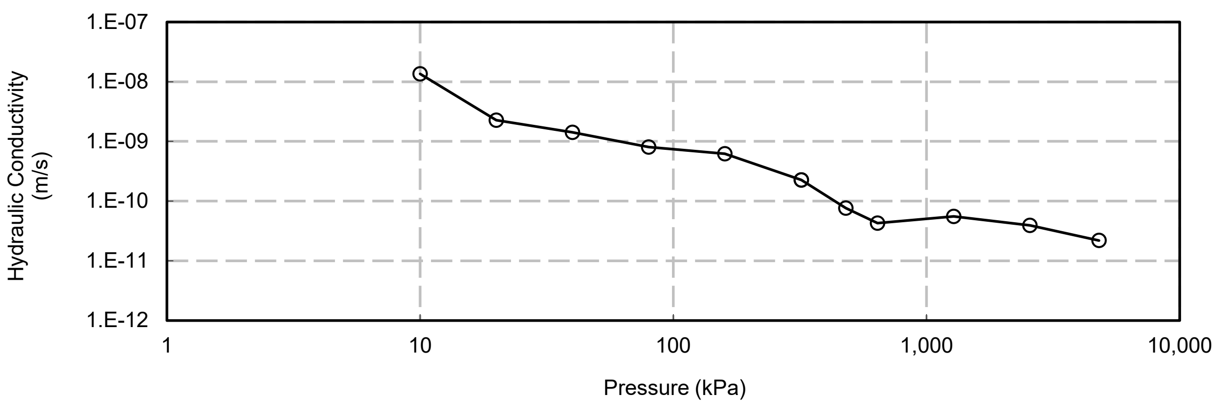
Cv vs Pressure



mv vs Pressure



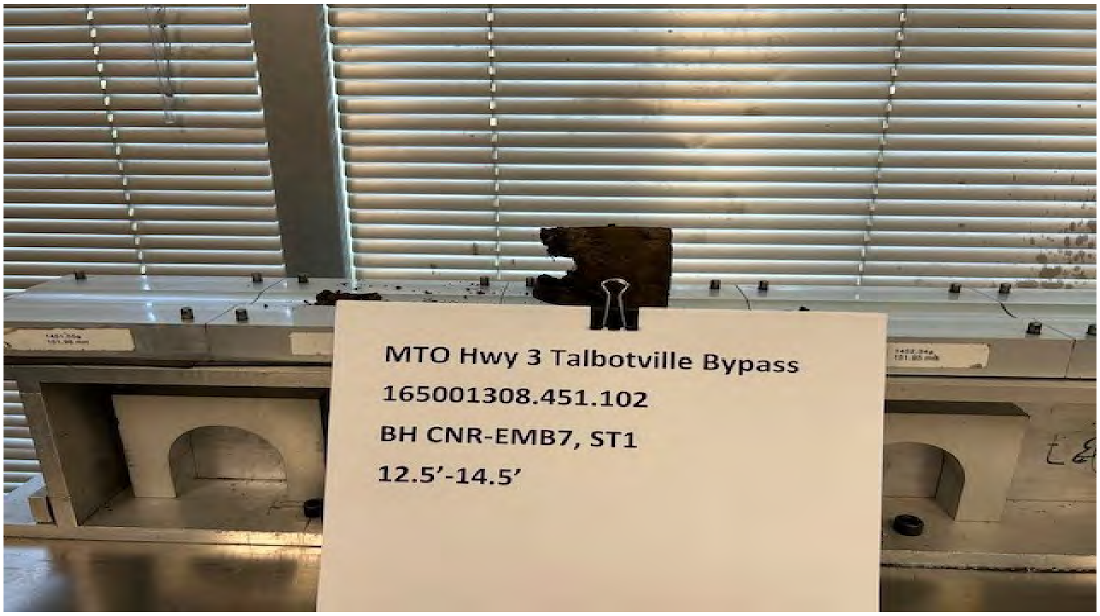
k vs Pressure



Project No. :	165001308.451.102	Stantec	Prepared By :	DB
Date :	16-Aug-24		Checked By :	RG

MTO Hwy 3 Talbotville Bypass

Overconsolidated silty clay , hard, brown, moist



BH CNR EMB7, ST1

BH CNR EMB7, ST1

CONSOLIDATION TEST SUMMARY

SAMPLE IDENTIFICATION

Borehole No. :	BH CNR EMB9	Sample No. :	ST2
		Sample Depth (ft) :	20-22

TEST CONDITIONS

Test Type :	ASTM D2435/D2435M	Date Started :	19-Aug-24
Load Duration (hr) :	Method B	Date Completed :	21-Aug-24

SAMPLE DIMENSIONS AND PROPERTIES _ INITIAL

Sample Height (mm) :	20.50	Unit Weight (kN/m^3) :	21.99
Sample Diameter (mm) :	50.00	Dry Unit Weight (kN/m^3) :	19.24
Area (cm^2) :	19.63	Specific Gravity : (Assumed)	2.764
Volume (cm^3) :	40.25	Solid Height (mm) :	14.55
Water Content (%) :	14.32	Volume of Solids (cm^3) :	28.57
Wet Mass (g) :	90.27	Volume of Voids (cm^3) :	11.68
Dry Mass (g) :	78.96	Degree of Saturation (%) :	96.80

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			0.409				
10	20.3219	0.1781	0.94	0.396	122.98	7.13E-03	9.44E-04	6.61E-09
20	20.2445	0.2555	1.37	0.390	211.15	4.12E-03	4.24E-04	1.72E-09
40	20.1241	0.3759	2.00	0.381	279.16	3.08E-03	3.18E-04	9.62E-10
80	19.9689	0.5311	2.79	0.370	311.81	2.72E-03	1.97E-04	5.27E-10
160	19.7773	0.7227	3.73	0.356	368.09	2.26E-03	1.17E-04	2.59E-10
320	19.5546	0.9454	4.80	0.341	353.68	2.31E-03	6.73E-05	1.52E-10
160	0.0000	0.0000	4.64	0.344	0.00	0.00E+00	1.04E-05	0.00E+00
80	0.0000	0.0000	4.37	0.347	0.00	0.00E+00	3.40E-05	0.00E+00
160	19.5767	0.9233	4.53	0.345	196.60	4.14E-03	2.04E-05	8.26E-11
320	19.5086	0.9914	4.93	0.340	162.42	4.98E-03	2.52E-05	1.23E-10
480	19.4042	1.0958	5.50	0.332	278.89	2.87E-03	3.52E-05	9.91E-11
640	19.3047	1.1953	5.99	0.325	281.65	2.81E-03	3.09E-05	8.52E-11
1280	19.0038	1.4962	7.56	0.303	220.48	3.50E-03	2.45E-05	8.41E-11
2560	18.6238	1.8762	9.48	0.275	181.68	4.10E-03	1.50E-05	6.04E-11
4800	18.2038	2.2962	11.64	0.245	127.16	5.61E-03	9.64E-06	5.30E-11
2560			11.38	0.249				
640			10.22	0.265				
160			8.68	0.287				
40			7.21	0.307				
10			6.05	0.324				

SAMPLE DIMENSIONS AND PROPERTIES _ FINAL

Sample Height (mm) :	19.26	Unit Weight (kN/m^3) :	23.07
Sample Diameter (mm) :	50.00	Dry Unit Weight (kN/m^3) :	20.48
Area (cm^2) :	19.63	Specific Gravity (Assumed) :	2.764
Volume (cm^3) :	37.82	Solid Height (mm) :	14.55
Water Content (%) :	12.68	Volume of Solids (cm^3) :	28.57
Wet Mass (g) :	88.97	Volume of Voids (cm^3) :	9.25
Dry Mass (g) :	78.96		

Project No. :	165001308.451.102	Prepared By :	DB
Date :	24-Aug-24	Checked By :	RG

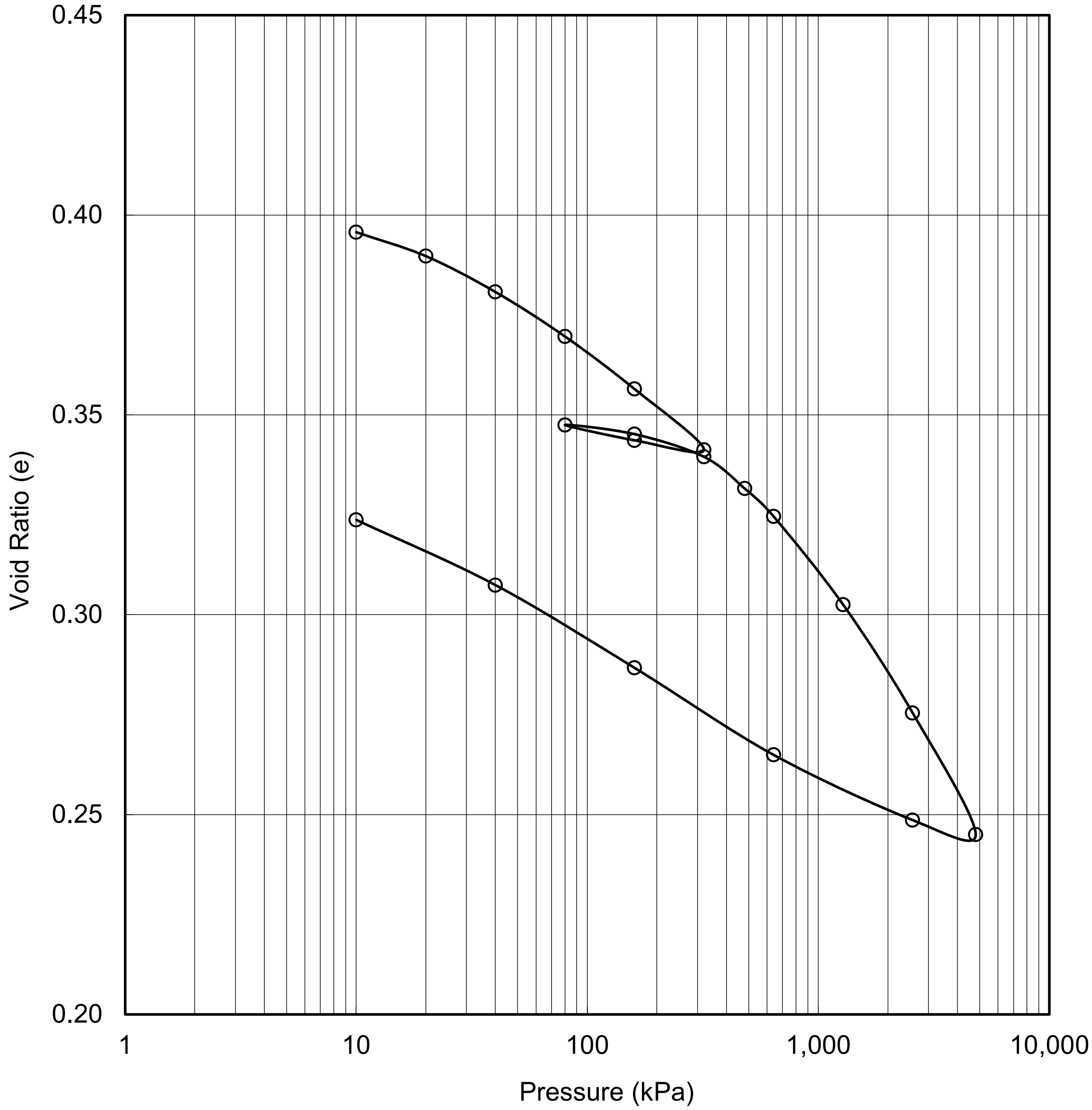
Stantec

CONSOLIDATION TEST	**FIGURE 1**

MTO Hwy 3 Talbotville Bypass

BH CNR EMB9, ST2

Void Ratio vs Pressure



Soil Type : *Overconsolidated Lean clay till of low plasticity, very stiff, brown, moist, CL*

e_o =	0.409	w_L =	28.4%	σ_{v0}' =	kPa
w =	14.3%	w_P =	15.6%	σ_P' =	kPa
γ =	22.0 kN/m^3	PI =	12.8%		
Gs =	2.764				

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

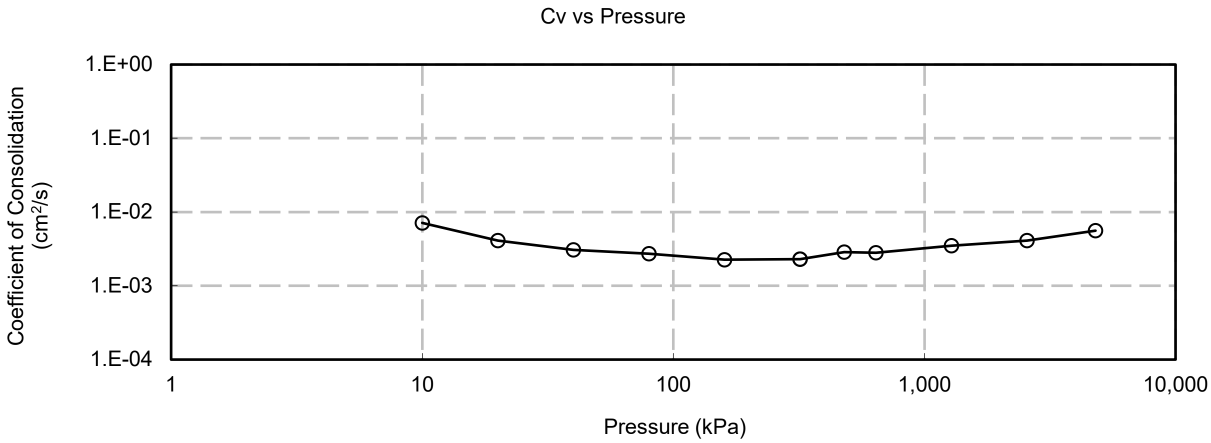
Stantec

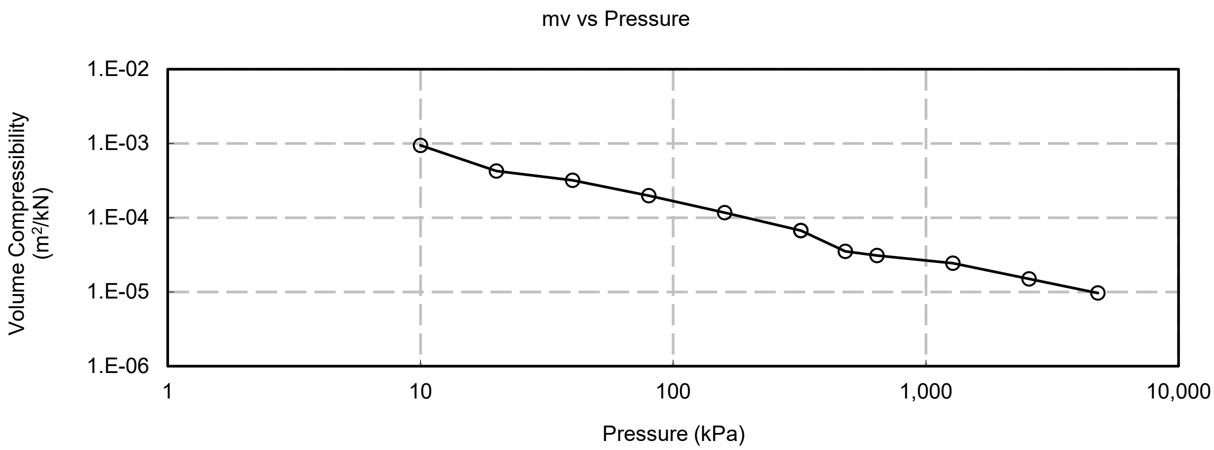
Prepared By : DB
Checked By : RG

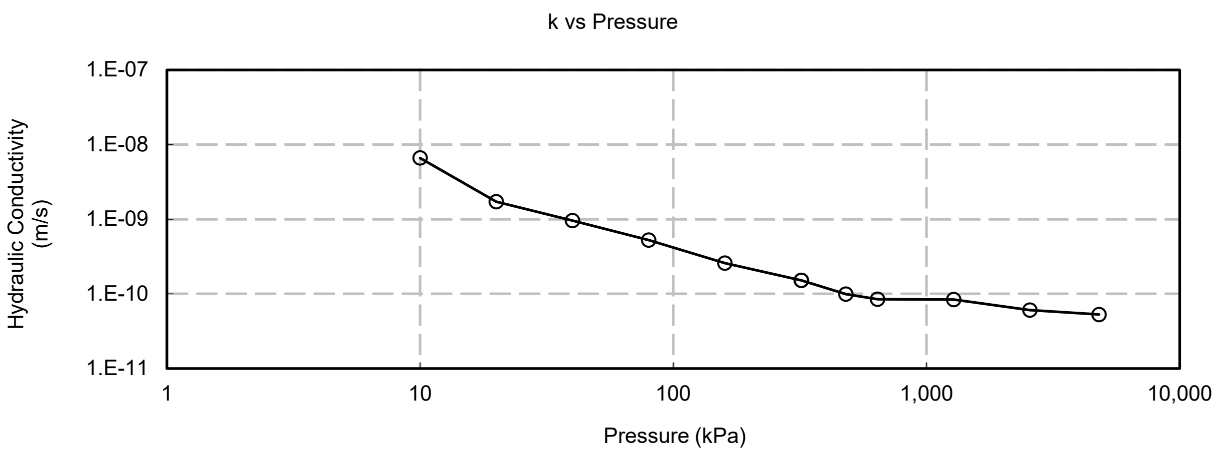
CONSOLIDATION TEST	FIGURES 2, 3 & 4

MTO Hwy 3 Talbotville Bypass

BH CNR EMB9, ST2







Project No. :	165001308.451.102	Prepared By :	DB
Date :	24-Aug-24	Checked By :	RG

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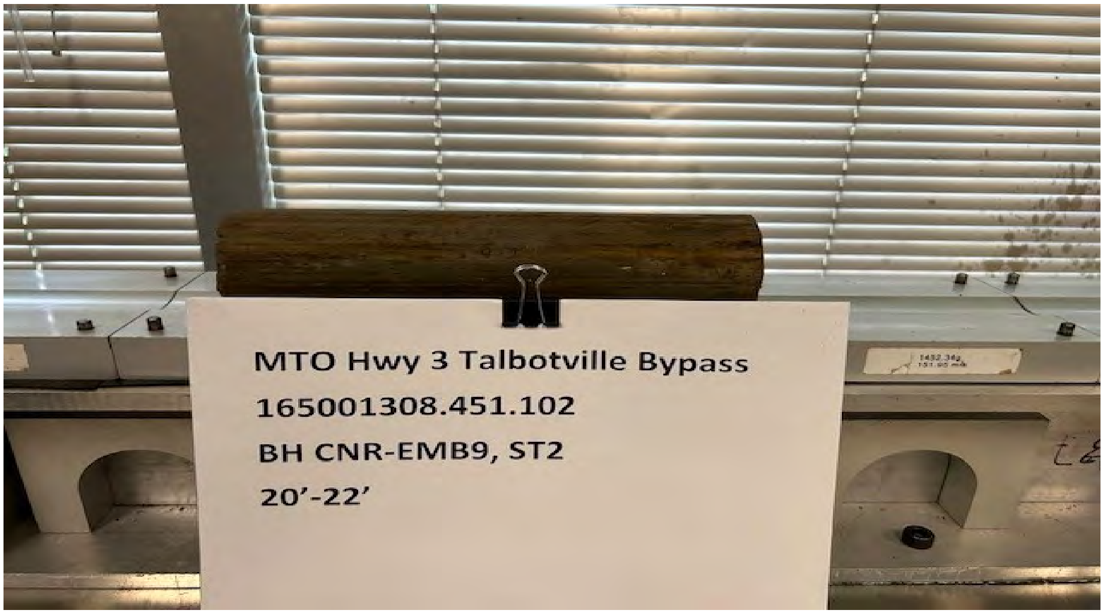
MTO Hwy 3 Talbotville Bypass

Overconsolidated Lean clay till of low plasticity, very stiff, brown, moist, CL



BH CNR EMB9, ST2

BH CNR EMB9, ST2

CONSOLIDATION TEST SUMMARY

SAMPLE IDENTIFICATION

Borehole No. :	BH CNR EMB11	Sample No. :	ST1
		Sample Depth (ft) :	48-50

TEST CONDITIONS

Test Type :	ASTM D2435/D2435M	Date Started :	9-Aug-24
Load Duration (hr) :	Method B	Date Completed :	11-Aug-24

SAMPLE DIMENSIONS AND PROPERTIES _ INITIAL

Sample Height (mm) :	20.50	Unit Weight (kN/m^3) :	20.98
Sample Diameter (mm) :	50.00	Dry Unit Weight (kN/m^3) :	17.91
Area (cm^2) :	19.63	Specific Gravity :	2.757
Volume (cm^3) :	40.25	Solid Height (mm) :	13.58
Water Content (%) :	17.12	Volume of Solids (cm^3) :	26.67
Wet Mass (g) :	86.11	Volume of Voids (cm^3) :	13.58
Dry Mass (g) :	73.52	Degree of Saturation (%) :	92.68

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	0.0000	0.00	0.509				
10	20.1934	0.3066	1.72	0.483	95.62	9.07E-03	1.72E-03	1.53E-08
20	20.0857	0.4143	2.18	0.476	304.66	2.81E-03	4.64E-04	1.28E-09
40	19.9713	0.5287	2.82	0.467	195.22	4.34E-03	3.17E-04	1.35E-09
80	19.8170	0.6830	3.61	0.455	188.88	4.42E-03	1.99E-04	8.62E-10
160	19.6054	0.8946	4.60	0.440	287.64	2.85E-03	1.23E-04	3.44E-10
320	19.3592	1.1408	5.84	0.421	266.12	3.01E-03	7.73E-05	2.28E-10
160			5.66	0.424				
80			5.37	0.428				
160	19.3683	1.1317	5.55	0.426	184.13	4.32E-03	2.16E-05	9.18E-11
320	19.2941	1.2059	5.97	0.419	189.33	4.18E-03	2.68E-05	1.10E-10
480	19.1928	1.3072	6.64	0.409	149.89	5.23E-03	4.17E-05	2.14E-10
640	19.0478	1.4522	7.28	0.399	312.30	2.47E-03	4.02E-05	9.74E-11
1280	18.6872	1.8128	9.27	0.370	201.36	3.72E-03	3.10E-05	1.13E-10
2560	18.2258	2.2742	11.58	0.335	175.57	4.08E-03	1.81E-05	7.24E-11
4800	17.7164	2.7836	13.98	0.298	191.37	3.54E-03	1.07E-05	3.71E-11
2560			13.74	0.302				
640			12.65	0.319				
160			11.15	0.341				
40			9.55	0.365				
10			7.99	0.389				

SAMPLE DIMENSIONS AND PROPERTIES _ FINAL

Sample Height (mm) :	18.86	Unit Weight (kN/m^3) :	22.27
Sample Diameter (mm) :	50.00	Dry Unit Weight (kN/m^3) :	19.47
Area (cm^2) :	19.63	Specific Gravity :	2.757
Volume (cm^3) :	37.03	Solid Height (mm) :	13.58
Water Content (%) :	14.42	Volume of Solids (cm^3) :	26.67
Wet Mass (g) :	84.12	Volume of Voids (cm^3) :	10.37
Dry Mass (g) :	73.52		

Project No. :	165001308.451.102	Prepared By :	DB
Date :	16-Aug-24	Checked By :	RG

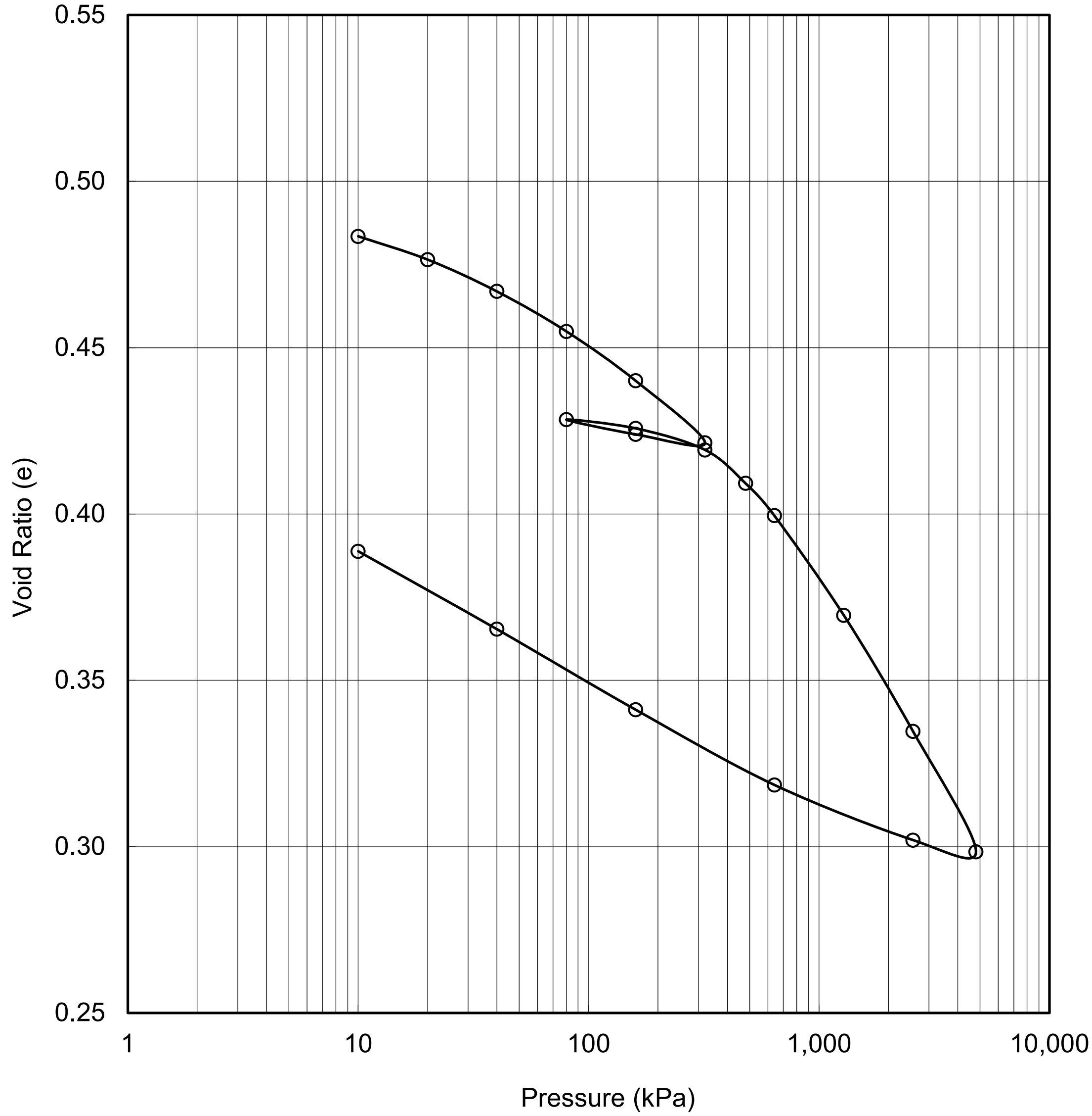
Stantec

CONSOLIDATION TEST	FIGURE 1

MTO Hwy 3 Talbotville Bypass

BH CNR EMB11, ST1

Void Ratio vs Pressure



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

e_o =	0.509	w_L =	28.9%	σ_{v0}' =	kPa
w =	17.1%	w_P =	16.2%	σ_P' =	kPa
γ =	21.0 kN/m^3	PI =	12.7%		
Gs =	2.757				

Project No. : 165001308.451.102
Date : 16-Aug-24

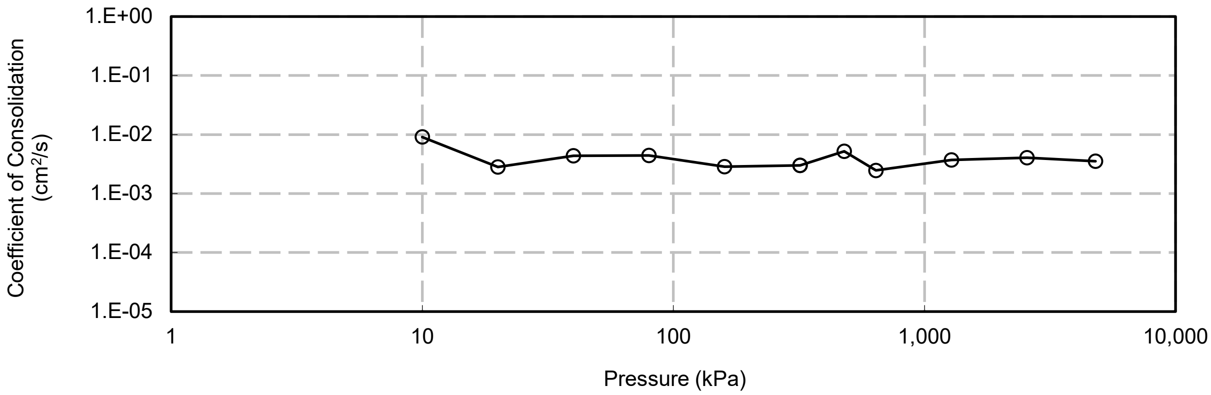
Prepared By : DB
Checked By : RG

CONSOLIDATION TEST	FIGURES 2, 3 & 4

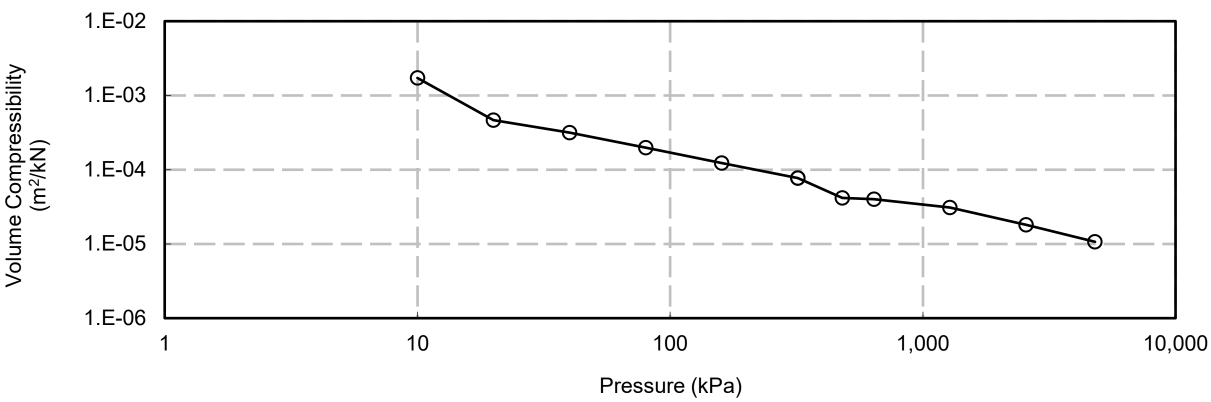
MTO Hwy 3 Talbotville Bypass

BH CNR EMB11, ST1

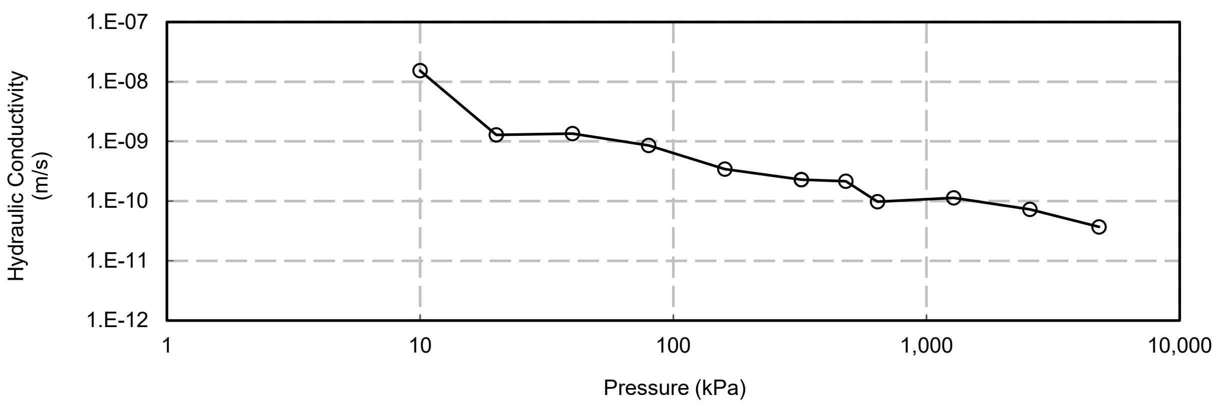
Cv vs Pressure



mv vs Pressure



k vs Pressure



Project No. :	165001308.451.102	Stantec	Prepared By :	DB
Date :	16-Aug-24		Checked By :	RG

MTO Hwy 3 Talbotville Bypass	**PHOTOS 1 & 2**

MTO Hwy 3 Talbotville Bypass

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

BH CNR EMB11, ST1

BH CNR EMB11, ST1

Project No. :	165001308.451.102	Stantec	Prepared by :	DB
Date :	16-Aug-2024		Checked by :	RG

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
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

Certificate of Analysis

AGAT WORK ORDER: 24T167277
PROJECT: 165001308.551.102

AGAT Laboratories
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
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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 *)

CHARTERED CHEMIST

Certified By:

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

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				REFERENCE MATERIAL			METHOD BLANK SPIKE			MATRIX SPIKE		
PARAMETER	Batch	Sample Id	Dup #1	Dup #2	RPD	Method Blank	Measured Value	Acceptable Limits Lower	Acceptable Limits Upper	Recovery	Acceptable Limits Lower	Acceptable Limits Upper	Recovery	Acceptable Limits Lower	Acceptable Limits 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

PARAMETER	Batch	Sample Id	Dup #1	Dup #2	RPD	Method Blank	Measured Value	Acceptable Limits Lower	Acceptable Limits Upper	Recovery	Acceptable Limits Lower	Acceptable Limits Upper	Recovery	Acceptable Limits Lower	Acceptable Limits Upper
(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

2910 12TH STREET NE
CALGARY, ALBERTA
CANADA T2E 7P7
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http://www.agatlabs.com

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 Lower	Acceptable Limits Upper	Recovery	Acceptable Limits Lower	Acceptable Limits Upper	Recovery	Acceptable Limits Lower	Acceptable Limits 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.

PARAMETER	Batch	Sample Id	Dup #1	Dup #2	RPD	Method Blank	Measured Value	Lower	Upper	Recovery	Lower	Upper	Recovery	Lower	Upper
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:

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

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

AGAT Laboratories

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Mississauga, Ontario
L4Z 1Y2
www.agatlabs.com · webearth.agatlabs.com
Ph.: 905.712.5100 · Fax: 905.712.5122 · Toll Free: 800.856.6261

Chain of Custody Record

Laboratory Use Only

Arrival Temperature: 20.4, 20.5, 21.0
AGAT WO #: 24T167277
Lab Temperature:
Notes: no ice, 1 box

Client Information:

Company: Stantec Consulting Ltd.
Contact: Bahram Siavash
Address: 300W-675 Cochran Drive West Tower
Markham, ON L3R 0B8
Phone: 905-479-9345 Fax: 905-474-9889
Project: 165001308.551.102 PO:
AGAT Quotation #:

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

Regulatory Requirements:

☐ Regulation 153/09 (reg. 511 Amend)
Table ____ Indicate one
☐ Ind/Com
☐ Res/Park
☐ Agriculture
Soil Texture *(check one)*
☐ Coarse ☐ Fine

☐ Sewer Use
Region ____ Indicate one
☐ Sanitary
☐ Storm

☐ Regulation 558
☐ CCME
☐ Other *(specify)*
☐ Prov. Water Quality Objectives (PWQO)
☐ None

Turnaround Time Required (TAT) Required*

Regular TAT
☑ 5 to 7 Working Days

Rush TAT (please provide prior notification)
Rush Surcharges Apply
☐ 3 Working Days
☐ 2 Working Days
☐ 1 Working Day

OR

Date Required (Rush surcharges may apply):

**TAT is exclusive of weekends and statutory holidays*

Invoice To:

Same: Yes ☑ No ☐
Company:
Contact:
Address:

Is this a drinking water sample?
(potable water intended for human consumption)
☐ Yes ☐ No
If "Yes", please use the *Drinking Water Chain of Custody Form*

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

Legend Matrix

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

Report Information – reports to be sent to:

1. Name: Bahram Siavash
Email: Bahram.Siavash@stantec.com
2. Name: Kirby Lales
Email: kirby.lales@stantec.com

Sample Identification	Date Sampled	Time Sampled	Sample Matrix	# of Containers	Comments Site/Sample Information
WR-UP3 - SS7	26-6-2024			1	15'-17'
RMN-UP3 - SS8	26-6-2024			1	20'-22'
CNR-OH1 - SS8	26-6-2024			1	17.5'-19.5'
CNR-OH2 - SS5	26-6-2024			1	10'-12'
CNR - OH3 - SS4	26-6-2024			1	7.5'-9.5'

Sample Identification	Metals and Inorganics	Metal Scan	Hydride Forming Metals	Client Custom Metals	ORPs: ☐ B-HWS ☐ Cl- ☐ CN- ☐ EC ☐ FOC ☐ Cr+6 ☐ SAR ☐ NO_2/NO_3 ☐ N-Total ☐ Hg ☐ pH	Nutrients: ☐ TP ☐ NH_3 ☐ TKN ☐ NO_3 ☐ NO_2 ☐ NO_3/NO_2	VOC: ☐ VOC ☐ THM ☐ BTEX	CCME Fractions 1 to 4	ABNs	PAHs	Chlorophenols	PCBs	Organochlorine Pesticides	TCLP Metals/Inorganics	TCLP:	Sewer Use	Corrosivity Pckg (pH, Redox Potential	sulphates and sulphides contents,	chlorides contents and resistivity)
WR-UP3 - SS7																	X	X	X
RMN-UP3 - SS8																	X	X	X
CNR-OH1 - SS8																	X	X	X
CNR-OH2 - SS5																	X	X	X
CNR - OH3 - SS4																	X	X	X

Samples Relinquished by (print name & sign): Date/Time:
Samples Received by (Print name & sign): [signature] Date/Time: Jun 27 11:15 AM
Samples Relinquished by (print name & sign): Date/Time:
Samples Received by (Print name & sign): Date/Time:

Pink Copy – Client
Yellow + Golden Copy – AGAT
White Copy – AGAT

Page ____ of ____
NO:

Document ID: DIV-78-1511.006
Date Issued: July 20, 2011

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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.*
- *Application of guidelines is provided "as is" without warranty of any kind, either expressed or implied, including, but not limited to, warranties of merchantability, fitness for a particular purpose, or non-infringement. AGAT assumes no responsibility for any errors or omissions in the guidelines contained in this document.*
- *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.*

AGAT Laboratories
2910 12TH STREET NE
CALGARY, ALBERTA
CANADA T2E 7P7
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http://www.agatlabs.com

Certificate of Analysis

AGAT WORK ORDER: 24T187247
PROJECT: 165001308.551.102

CLIENT NAME: STANTEC CONSULTING LTD
ATTENTION TO: Bahram Siavash
SAMPLING SITE:
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:

2910 12TH STREET NE
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CANADA T2E 7P7
TEL (403)735-2005
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Quality Assurance

CLIENT NAME: STANTEC CONSULTING LTD
AGAT WORK ORDER: 24T187247
PROJECT: 165001308.551.102
ATTENTION TO: Bahram Siavash
SAMPLING SITE:
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		Recovery	Acceptable Limits	
								Lower	Upper		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

AGAT Laboratories

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CALGARY, ALBERTA
CANADA T2E 7P7
TEL (403)735-2005
FAX (403)735-2771
http://www.agatlabs.com

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 Lower	Acceptable Limits Upper	Recovery	Acceptable Limits Lower	Acceptable Limits Upper	Recovery	Acceptable Limits Lower	Acceptable Limits 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:

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CALGARY, ALBERTA
CANADA T2E 7P7
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FAX (403)735-2771
http://www.agatlabs.com

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

AGAT Laboratories

5835 Coopers Avenue
Mississauga, Ontario
L4Z 1Y2
www.agatlabs.com · webearth.agatlabs.com

Chain of Custody Record

Ph.: 905.712.5100 · Fax: 905.712.5122 · Toll Free: 800.856.6261

Laboratory Use Only

Arrival Temperature: 21-2, 21-4, 21-7
AGAT WO #: 24T187247
Lab Temperature:
Notes: 1 box, no ice

Client Information:

Company: Stantec Consulting Ltd.
Contact: Bahram Siavash
Address: 300W-675 Cochran Drive West Tower
Markham, ON L3R 0B8
Phone: 905-479-9345 Fax: 905-474-9889
Project: 165001308.551.102 PO:
AGAT Quotation #:

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

Regulatory Requirements:

☐ Regulation 153/09 (reg. 511 Amend.)
Table ____ Indicate one
☐ Ind/Com
☐ Res/Park
☐ Agriculture
Soil Texture (check one)
☐ Coarse ☐ Fine

☐ Sewer Use
Region ____ Indicate one
☐ Sanitary
☐ Storm

☐ Regulation 558
☐ CCME
☐ Other (specify)
☐ Prov. Water Quality Objectives (PWQO)
☐ None

Turnaround Time Required (TAT) Required*

Regular TAT
☑ 5 to 7 Working Days

Rush TAT (please provide prior notification)
Rush Surcharges Apply
☐ 3 Working Days
☐ 2 Working Days
☐ 1 Working Day

OR

Date Required (Rush surcharges may apply):

*TAT is exclusive of weekends and statutory holidays

Invoice To: Same: Yes ☑ No ☐
Company:
Contact:
Address:

Is this a drinking water sample?
(potable water intended for human consumption)
☐ Yes ☐ No
If "Yes", please use the *Drinking Water Chain of Custody Form*

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

Legend Matrix
GW Ground Water **O** Oil
SW Surface Water **P** Paint
SD Sediment **S** Soil

Report Information – reports to be sent to:
1. Name: Kirby Lales
Email: Kirby.Lales@Stantec.com
2. Name: Maged Abdel Mesih
Email: Maged.Abdel-Mesih@stantec.com

Sample Identification	Date Sampled	Time Sampled	Sample Matrix	# of Containers	Comments Site/Sample Information	Metals and Inorganics	Metal Scan	Hydride Forming Metals	Client Custom Metals	ORPs: ☐ B-HWS ☐ Cl- ☐ CN- ☐ EC ☐ FOC ☐ Cr+6- ☐ SAR ☐ NO3/NO2 ☐ N-Total ☐ Hg ☐ pH	Nutrients: ☐ TP ☐ NH3 ☐ TKN ☐ NO3 ☐ NO2 ☐ NO3/NO3	VOC: ☐ VOC ☐ THM ☐ BTEX	CCME Fractions 1 to 4	ABNs	PAHs	Chlorophenols	PCBs	Organochlorine Pesticides	TCLP Metals/Inorganics	TCLP:	Sewer Use	Corrosivity Pckg (pH, Redox Potential	sulphates and sulphides contents,	chlorides contents and resistivity)					
CNR-OH4 - SS9	2024-08-20			1	20'-22'																	X	X	X					
RMN-UP2 - SS7	2024-08-20			1	15'-17'																	X	X	X					
RMN-UP1 - SS10	2024-08-20			1	25'-27'																	X	X	X					

Samples Relinquished by (print name & sign): Date/Time
Samples Received by (Print name & sign): [signature] Aug 20 9:45 PM Date/Time
Samples Relinquished by (print name & sign): Date/Time
Samples Received by (Print name & sign): Date/Time

~~Pink Copy~~ – Client
Yellow + Golden Copy – AGAT
White Copy – AGAT

Page ____ of ____
NO:

Document ID: DIV-78-1511.006
Date Issued: July 20, 2011

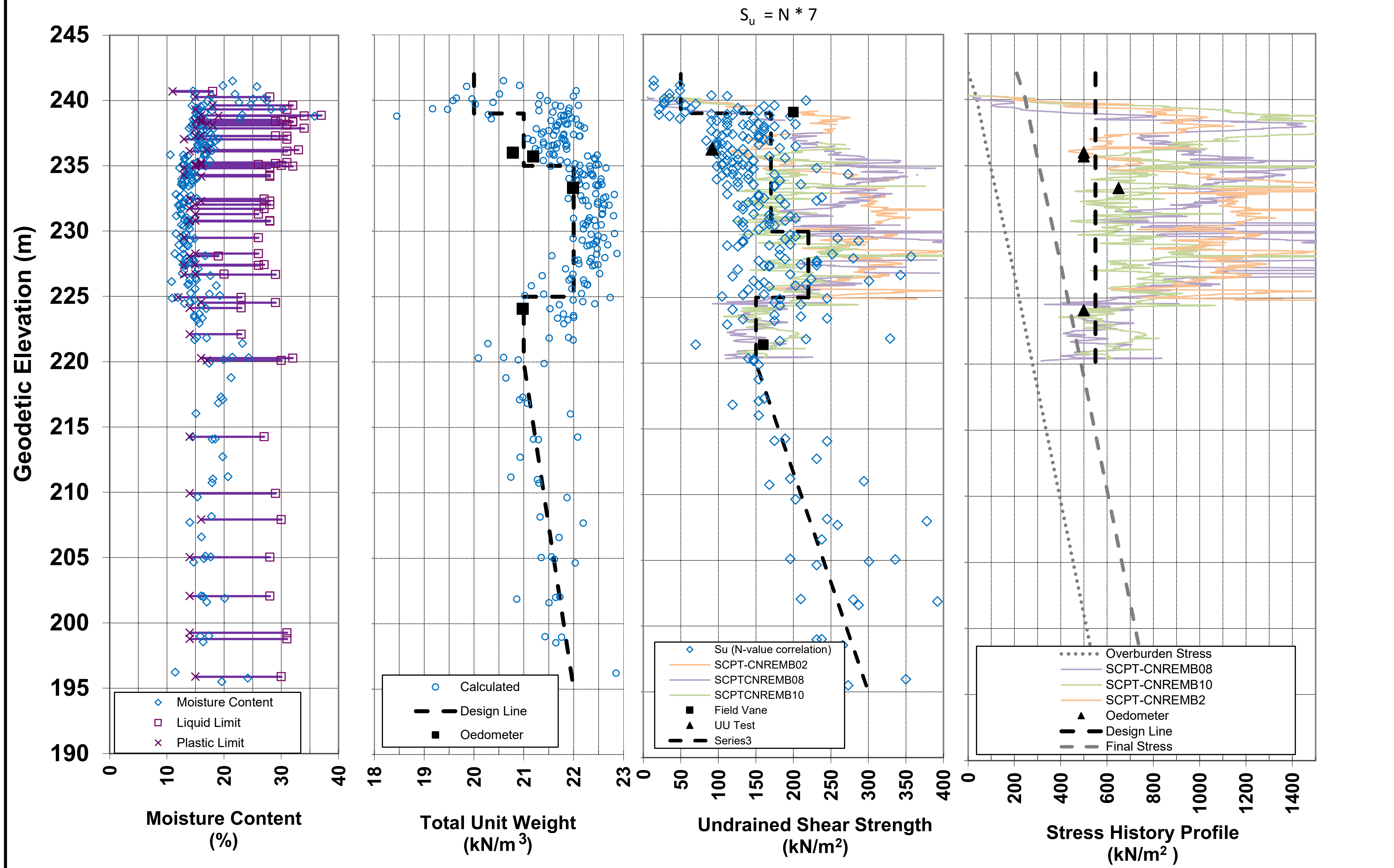
APPENDIX E

E.1 SOIL MODEL (FIGURE E1 AND E2)

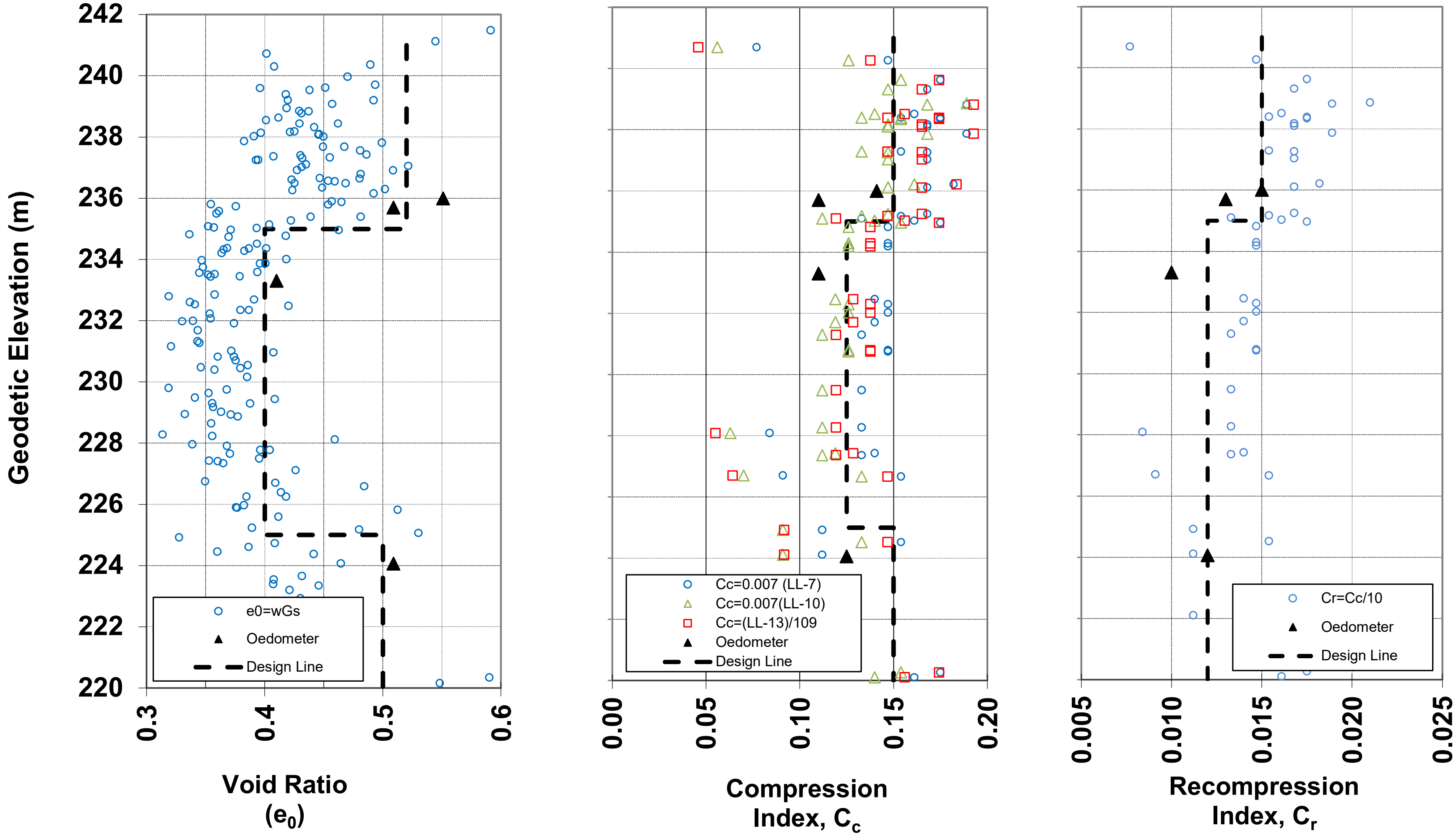
E.2 P-Y CHARTS (FIGURE 3)

E.3 P-Y TABLE (TABLE E1)

E.4 SLOPE STABILITY ANALYSIS RESULTS (FIGURES E4 TO E9)



Stantec	Talbotville Bypass - CNR Overhead	Soil Model	Figure E1



Stantec	Talbotville Bypass - CNR Overhead	Compressibility Parameters	Figure E2

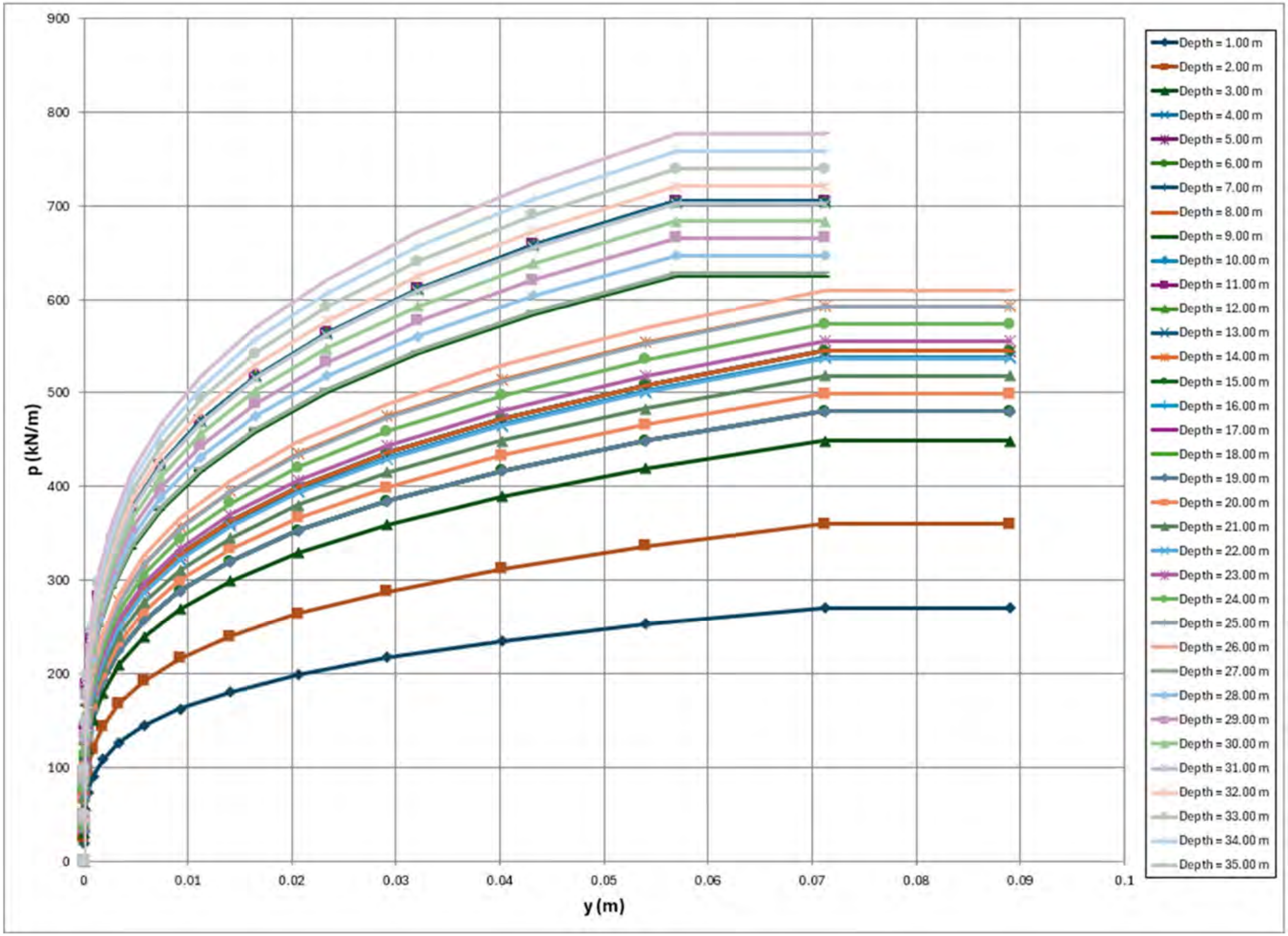
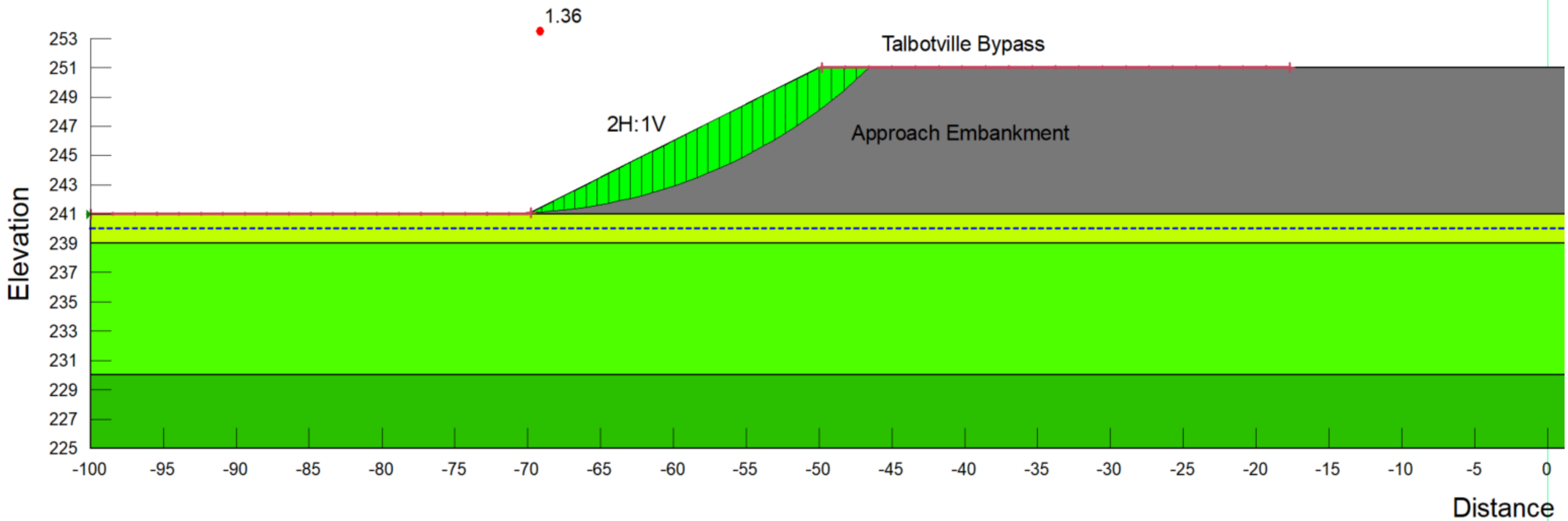


Figure E3: Load Intensity p (kN/m) vs Lateral Deflection y (m) Data Points for CNR Overhead Abutments - 356 mm OD 9.53 mm Thick Pipe Piles

Table E1: Load Intensity p (kN/m) vs Lateral Deflection y (m) Data Points for CNR Overhead Abutments - 356 mm OD 9.53 mm Thick Pipe Piles

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	1.4E-06	2.3E-05	0.00011	0.00036	0.00088	0.00182	0.00338	0.00576	0.00923	0.01406	0.02059	0.02916	0.04017	0.05403	0.0712	0.089
	P	0	18.0365	36.073	54.1094	72.1459	90.1824	108.219	126.255	144.292	162.328	180.365	198.401	216.438	234.474	252.511	270.547	270.5472
2.0	Y	0	1.4E-06	2.3E-05	0.00011	0.00036	0.00088	0.00182	0.00338	0.00576	0.00923	0.01406	0.02059	0.02916	0.04017	0.05403	0.0712	0.089
	P	0	23.969	47.9379	71.9069	95.8758	119.845	143.814	167.783	191.752	215.721	239.69	263.659	287.628	311.596	335.565	359.534	359.5344
3.0	Y	0	1.4E-06	2.3E-05	0.00011	0.00036	0.00088	0.00182	0.00338	0.00576	0.00923	0.01406	0.02059	0.02916	0.04017	0.05403	0.0712	0.089
	P	0	29.9014	59.8029	89.7043	119.606	149.507	179.409	209.31	239.212	269.113	299.014	328.916	358.817	388.719	418.62	448.522	448.5216
4.0	Y	0	1.4E-06	2.3E-05	0.00011	0.00036	0.00088	0.00182	0.00338	0.00576	0.00923	0.01406	0.02059	0.02916	0.04017	0.05403	0.0712	0.089
	P	0	35.8339	71.6678	107.502	143.336	179.17	215.004	250.837	286.671	322.505	358.339	394.173	430.007	465.841	501.675	537.509	537.5088
5.0	Y	0	1.4E-06	2.3E-05	0.00011	0.00036	0.00088	0.00182	0.00338	0.00576	0.00923	0.01406	0.02059	0.02916	0.04017	0.05403	0.0712	0.089
	P	0	36.312	72.624	108.936	145.248	181.56	217.872	254.184	290.496	326.808	363.12	399.432	435.744	472.056	508.368	544.68	544.68
6.0	Y	0	1.4E-06	2.3E-05	0.00011	0.00036	0.00088	0.00182	0.00338	0.00576	0.00923	0.01406	0.02059	0.02916	0.04017	0.05403	0.0712	0.089
	P	0	36.312	72.624	108.936	145.248	181.56	217.872	254.184	290.496	326.808	363.12	399.432	435.744	472.056	508.368	544.68	544.68
7.0	Y	0	1.4E-06	2.3E-05	0.00011	0.00036	0.00088	0.00182	0.00338	0.00576	0.00923	0.01406	0.02059	0.02916	0.04017	0.05403	0.0712	0.089
	P	0	36.312	72.624	108.936	145.248	181.56	217.872	254.184	290.496	326.808	363.12	399.432	435.744	472.056	508.368	544.68	544.68
8.0	Y	0	1.4E-06	2.3E-05	0.00011	0.00036	0.00088	0.00182	0.00338	0.00576	0.00923	0.01406	0.02059	0.02916	0.04017	0.05403	0.0712	0.089
	P	0	36.312	72.624	108.936	145.248	181.56	217.872	254.184	290.496	326.808	363.12	399.432	435.744	472.056	508.368	544.68	544.68
9.0	Y	0	1.1E-06	1.8E-05	9.1E-05	0.00029	0.0007	0.00146	0.0027	0.00461	0.00738	0.01125	0.01647	0.02333	0.03214	0.04322	0.05696	0.0712
	P	0	41.652	83.304	124.956	166.608	208.26	249.912	291.564	333.216	374.868	416.52	458.172	499.824	541.476	583.128	624.78	624.78
10.0	Y	0	1.1E-06	1.8E-05	9.1E-05	0.00029	0.0007	0.00146	0.0027	0.00461	0.00738	0.01125	0.01647	0.02333	0.03214	0.04322	0.05696	0.0712
	P	0	46.992	93.984	140.976	187.968	234.96	281.952	328.944	375.936	422.928	469.92	516.912	563.904	610.896	657.888	704.88	704.88
11.0	Y	0	1.1E-06	1.8E-05	9.1E-05	0.00029	0.0007	0.00146	0.0027	0.00461	0.00738	0.01125	0.01647	0.02333	0.03214	0.04322	0.05696	0.0712
	P	0	46.992	93.984	140.976	187.968	234.96	281.952	328.944	375.936	422.928	469.92	516.912	563.904	610.896	657.888	704.88	704.88
12.0	Y	0	1.1E-06	1.8E-05	9.1E-05	0.00029	0.0007	0.00146	0.0027	0.00461	0.00738	0.01125	0.01647	0.02333	0.03214	0.04322	0.05696	0.0712
	P	0	46.992	93.984	140.976	187.968	234.96	281.952	328.944	375.936	422.928	469.92	516.912	563.904	610.896	657.888	704.88	704.88
13.0	Y	0	1.1E-06	1.8E-05	9.1E-05	0.00029	0.0007	0.00146	0.0027	0.00461	0.00738	0.01125	0.01647	0.02333	0.03214	0.04322	0.05696	0.0712
	P	0	46.992	93.984	140.976	187.968	234.96	281.952	328.944	375.936	422.928	469.92	516.912	563.904	610.896	657.888	704.88	704.88
14.0	Y	0	1.4E-06	2.3E-05	0.00011	0.00036	0.00088	0.00182	0.00338	0.00576	0.00923	0.01406	0.02059	0.02916	0.04017	0.05403	0.0712	0.089
	P	0	39.516	79.032	118.548	158.064	197.58	237.096	276.612	316.128	355.644	395.16	434.676	474.192	513.708	553.224	592.74	592.74
15.0	Y	0	1.4E-06	2.3E-05	0.00011	0.00036	0.00088	0.00182	0.00338	0.00576	0.00923	0.01406	0.02059	0.02916	0.04017	0.05403	0.0712	0.089
	P	0	32.04	64.08	96.12	128.16	160.2	192.24	224.28	256.32	288.36	320.4	352.44	384.48	416.52	448.56	480.6	480.6
16.0	Y	0	1.4E-06	2.3E-05	0.00011	0.00036	0.00088	0.00182	0.00338	0.00576	0.00923	0.01406	0.02059	0.02916	0.04017	0.05403	0.0712	0.089
	P	0	32.04	64.08	96.12	128.16	160.2	192.24	224.28	256.32	288.36	320.4	352.44	384.48	416.52	448.56	480.6	480.6
17.0	Y	0	1.4E-06	2.3E-05	0.00011	0.00036	0.00088	0.00182	0.00338	0.00576	0.00923	0.01406	0.02059	0.02916	0.04017	0.05403	0.0712	0.089
	P	0	32.04	64.08	96.12	128.16	160.2	192.24	224.28	256.32	288.36	320.4	352.44	384.48	416.52	448.56	480.6	480.6
18.0	Y	0	1.4E-06	2.3E-05	0.00011	0.00036	0.00088	0.00182	0.00338	0.00576	0.00923	0.01406	0.02059	0.02916	0.04017	0.05403	0.0712	0.089
	P	0	32.04	64.08	96.12	128.16	160.2	192.24	224.28	256.32	288.36	320.4	352.44	384.48	416.52	448.56	480.6	480.6
19.0	Y	0	1.4E-06	2.3E-05	0.00011	0.00036	0.00088	0.00182	0.00338	0.00576	0.00923	0.01406	0.02059	0.02916	0.04017	0.05403	0.0712	0.089
	P	0	32.04	64.08	96.12	128.16	160.2	192.24	224.28	256.32	288.36	320.4	352.44	384.48	416.52	448.56	480.6	480.6
20.0	Y	0	1.4E-06	2.3E-05	0.00011	0.00036	0.00088	0.00182	0.00338	0.00576	0.00923	0.01406	0.02059	0.02916	0.04017	0.05403	0.0712	0.089
	P	0	33.2723	66.5446	99.8169	133.089	166.362	199.634	232.906	266.178	299.451	332.723	365.995	399.268	432.54	465.812	499.085	499.0846
21.0	Y	0	1.4E-06	2.3E-05	0.00011	0.00036	0.00088	0.00182	0.00338	0.00576	0.00923	0.01406	0.02059	0.02916	0.04017	0.05403	0.0712	0.089
	P	0	34.5046	69.0092	103.514	138.018	172.523	207.028	241.532	276.037	310.542	345.046	379.551	414.055	448.56	483.065	517.569	517.5692
22.0	Y	0	1.4E-06	2.3E-05	0.00011	0.00036	0.00088	0.00182	0.00338	0.00576	0.00923	0.01406	0.02059	0.02916	0.04017	0.05403	0.0712	0.089
	P	0	35.7369	71.4738	107.211	142.948	178.685	214.422	250.158	285.895	321.632	357.369	393.106	428.843	464.58	500.317	536.054	536.0539
23.0	Y	0	1.4E-06	2.3E-05	0.00011	0.00036	0.00088	0.00182	0.00338	0.00576	0.00923	0.01406	0.02059	0.02916	0.04017	0.05403	0.0712	0.089
	P	0	36.9692	73.9385	110.908	147.877	184.846	221.815	258.785	295.754	332.723	369.692	406.662	443.631	480.6	517.569	554.538	554.5385
24.0	Y	0	1.4E-06	2.3E-05	0.00011	0.00036	0.00088	0.00182	0.00338	0.00576	0.00923	0.01406	0.02059	0.02916	0.04017	0.05403	0.0712	0.089
	P	0	38.2015	76.4031	114.605	152.806	191.008	229.209	267.411	305.612	343.814	382.015	420.217	458.418	496.62	534.822	573.023	573.0231
25.0	Y	0	1.4E-06	2.3E-05	0.00011	0.00036	0.00088	0.00182	0.00338	0.00576	0.00923	0.01406	0.02059	0.02916	0.04017	0.05403	0.0712	0.089
	P	0	39.4338	78.8677	118.302	157.735	197.169	236.603	276.037	315.471	354.905	394.338	433.772	473.206	512.64	552.074	591.508	591.5077
26.0	Y	0	1.4E-06	2.3E-05	0.00011	0.00036	0.00088	0.00182	0.00338	0.00576	0.00923	0.01406	0.02059	0.02916	0.04017	0.05403	0.0712	0.089
	P	0	40.6662	81.3323	121.998	162.665	203.331	243.997	284.663	325.329	365.995	406.662	447.328	487.994	528.66	569.326	609.992	609.9923
27.0	Y	0	1.1E-06	1.8E-05	9.1E-05	0.00029	0.0007	0.00146	0.0027	0.00461	0.00738	0.01125	0.01647	0.02333	0.03214	0.04322	0.05696	0.0712
	P	0	41.8985	83.7969	125.695	167.594	209.492	251.391	293.289	335.188	377.086	418.985	460.883	502.782	544.68	586.578	628.477	628.4769
28.0	Y	0	1.1E-06	1.8E-05	9.1E-05	0.00029	0.0007	0.00146	0.0027	0.00461	0.00738	0.01125	0.01647	0.02333	0.03214	0.04322	0.05696	0.0712
	P	0	43.1308	86.2615	129.392	172.523	215.654	258.785	301.915	345.046	388.177	431.308	474.438	517.569	560.7	603.831	646.962	646.9615
29.0	Y	0	1.1E-06	1.8E-05	9.1E-05	0.00029	0.0007	0.00146	0.0027	0.00461	0.00738	0.01125	0.01647	0.02333	0.03214	0.04322	0.05696	0.0712
	P	0	44.3631	88.7262	133.089	177.452	221.815	266.178	310.542	354.905	399.268	443.631	487.994	532.357	576.72	621.083	665.446	665.4462
30.0	Y	0	1.1E-06	1.8E-05	9.1E-05	0.00029	0.0007	0.00146	0.0027	0.00461	0.00738	0.01125	0.01647	0.02333	0.03214	0.04322	0.05696	0.0712
	P	0	45.5954	91.1908	136.786	182.382	227.977	273.572	319.168	364.763	410.358	455.954	501.549	547.145	592.74	638.335	683.931	683.9308
31.0	Y	0	1.1E-06	1.8E-05	9.1E-05	0.00029	0.0007	0.00146	0.0027	0.00461	0.00738	0.01125	0.01647	0.02333	0.03214	0.04322	0.05696	0.0712
	P	0	46.8277	93.6554	140.483	187.311	234.138	280.966	327.794	374.622	421.449	468.277	515.105	561.932	608.76	655.588	702.415	702.4154
32.0	Y	0	1.1E-06	1.8E-05	9.1E-05	0.00029	0.0007	0.00146	0.0027	0.00461	0.00738	0.01125	0.01647	0.02333	0.03214	0.04322	0.05696	0.0712
	P	0	48.06	96.12	144.18	192.24	240.3	288.36	336.42	384.48	432.54	480.6	528.66	576.72	624.78	672.84	720.9	720.9
33.0	Y	0	1.1E-06	1.8E-05	9.1E-05	0.00029	0.0007	0.00146	0.0027	0.00461	0.00738	0.01125	0.01647	0.02333	0.03214	0.04322	0.05696	0.0712
	P	0	49.2923	98.5846	147.877	197.169	246.462	295.754	345.046	394.338	443.631	492.923	542.215	591.508	640.8	690.092	739.385	739.3846
34.0	Y	0	1.1E-06	1.8E-05	9.1E-05	0.00029	0.0007	0.00146	0.0027	0.00461	0.00738	0.01125	0.01647	0.02333	0.03214	0.04322	0.05696	0.0712
	P	0	50.5246	101.049	151.574	202.098	252.623	303.148	353.672	404.197	454.722	505.246	555.771	606.295	656.82	707.345	757.869	757.8692
35.0	Y	0	1.1E-06	1.8E-05	9.1E-05	0.00029	0.0007	0.00146	0.0027	0.00461	0.00738	0.01125	0.01647	0.02333	0.03214	0.04322	0.05696	0.0712
	P	0	51.7569	103.514	155.271	207.028	258.785	310.542	362.298	414.055	465.812	517.569	569.326	621.083	672.84	724.597	776.354	776.3539

The response of a pile to lateral loads is a nonlinear relationship. The p-y geotechnical approach was used to estimate the anticipated deformation of a pile within the soil medium. The p-y curves represent the load-deformation characteristics of elastic-plastic springs with a non-linear response within the elastic range. These non-linear elastic-plastic springs provide a more realistic representation or modeling of the soil pressure response against the face of the pile. The table presents the Load Intensity per unit length of pile p (kN/m) vs Lateral Deflection y (m). The p-y points can be used for the structural design of the pile in response to lateral loads. Where spring spacings of less than 1.0 m are proposed, the tabulated “p” values are to be multiplied by the actual spring spacing; i.e. by 0.25 for 0.25 m spacings.



Color	Name	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Cohesion (kPa)
	Firm CLAYEY SILT to SILTY CLAY TILL (Undrained)	20			50
	New Granular Embankment Fill	21	0	30	
	Stiff to hard CLAYEY SILT TILL 1 (Undrained)	21			170
	Stiff to hard CLAYEY SILT TILL 2 (Undrained)	22			220

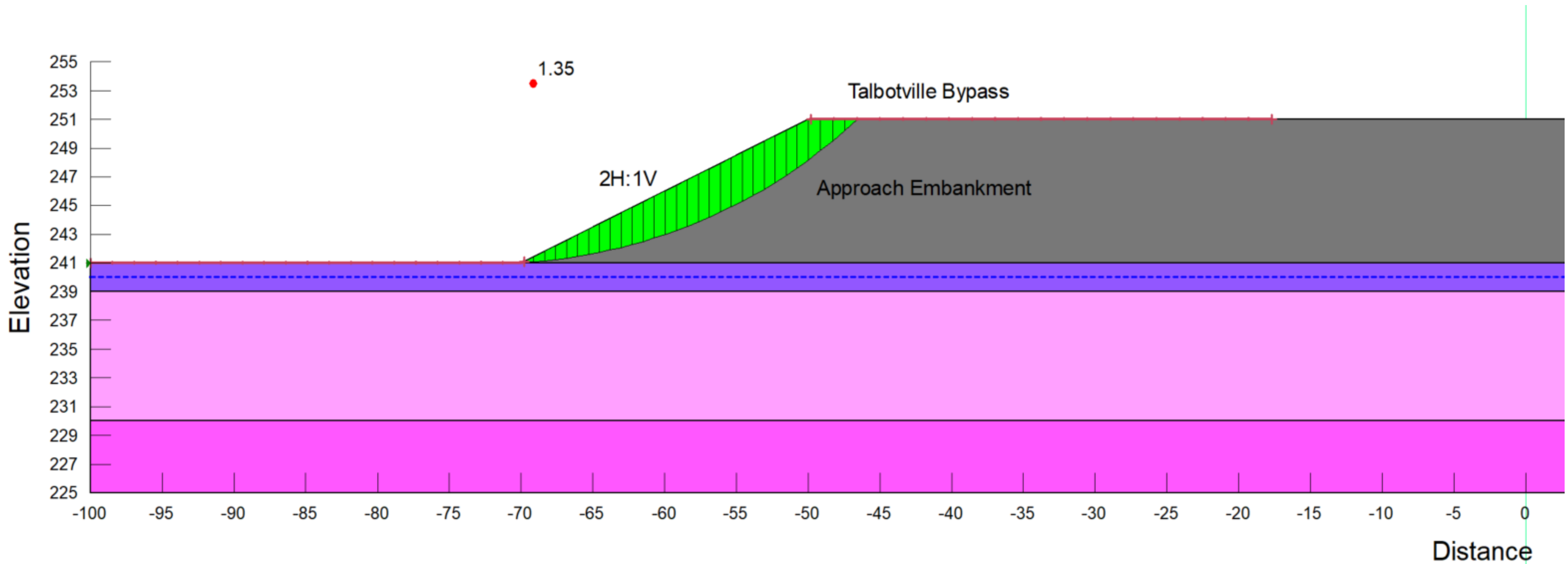
Stantec

Slope Stability Analysis (Static)
(Undrained Conditions)
New Granular Embankment
Talbotville Bypass at CNR Overhead

Figure E4

Project No. 165001308

GWP No. 3042-22-00



Color	Name	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Firm CLAYEY SILT to SILTY CLAY TILL (Drained)	20	0	28
	New Granular Embankment Fill	21	0	30
	Stiff to hard CLAYEY SILT TILL 1 (Drained)	21	0	30
	Stiff to hard CLAYEY SILT TILL 2 (Drained)	22	0	30

Stantec

Slope Stability Analysis (Static)

(Drained Conditions)

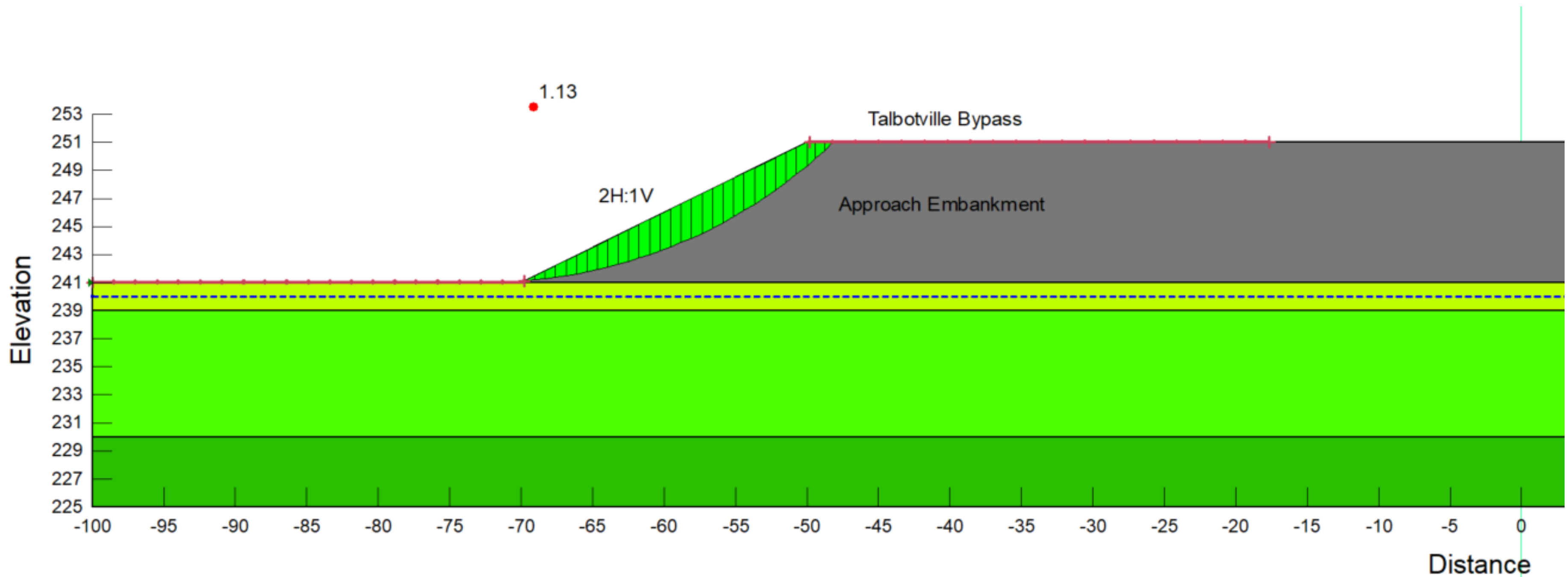
New Granular Embankment

Talbotville Bypass at CNR Overhead

Figure E5

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 to SILTY CLAY TILL (Undrained)	20			50
	New Granular Embankment Fill	21	0	30	
	Stiff to hard CLAYEY SILT TILL 1 (Undrained)	21			170
	Stiff to hard CLAYEY SILT TILL 2 (Undrained)	22			220

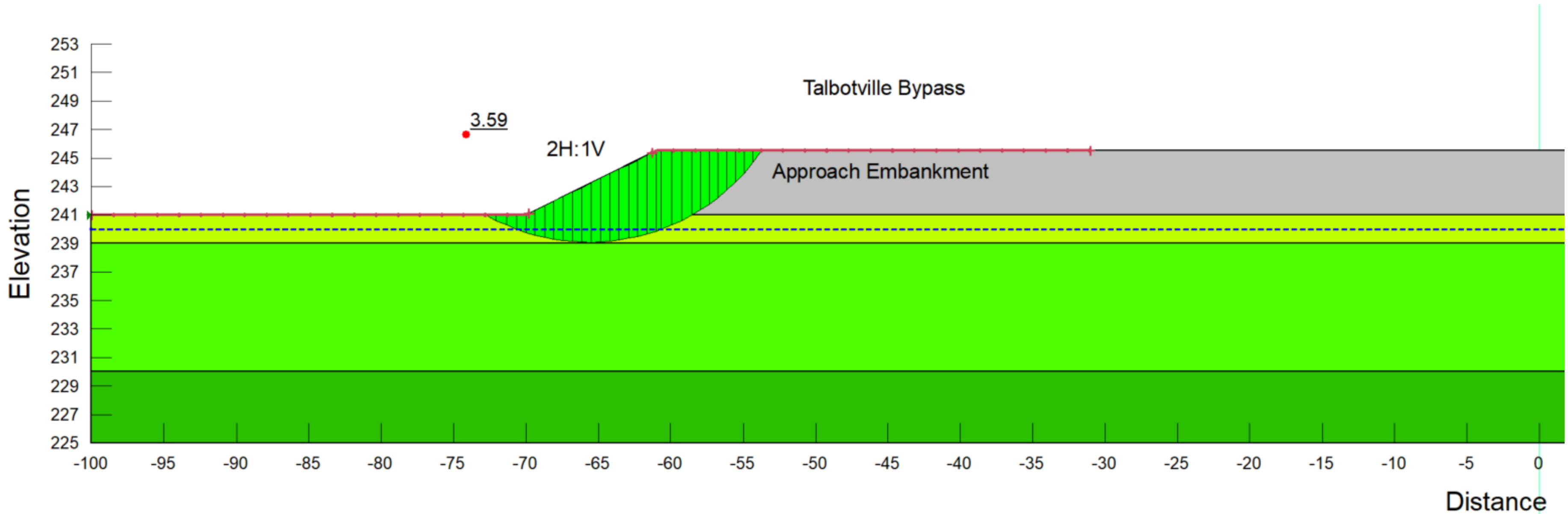
Stantec

Slope Stability Analysis (Pseudo-static)
(Undrained Conditions)
New Granular Embankment
Talbotville Bypass at CNR Overhead

Figure E6

Project No. 165001308

GWP No. 3042-22-00



Color	Name	Unit Weight (kN/m³)	Cohesion (kPa)
	Firm CLAYEY SILT to SILTY CLAY TILL (Undrained)	20	50
	New Cohesive Embankment Fill (Undrained)	20.5	50
	Stiff to hard CLAYEY SILT TILL 1 (Undrained)	21	170
	Stiff to hard CLAYEY SILT TILL 2 (Undrained)	22	220

Stantec

Slope Stability Analysis (Static)

(Undrained Conditions)

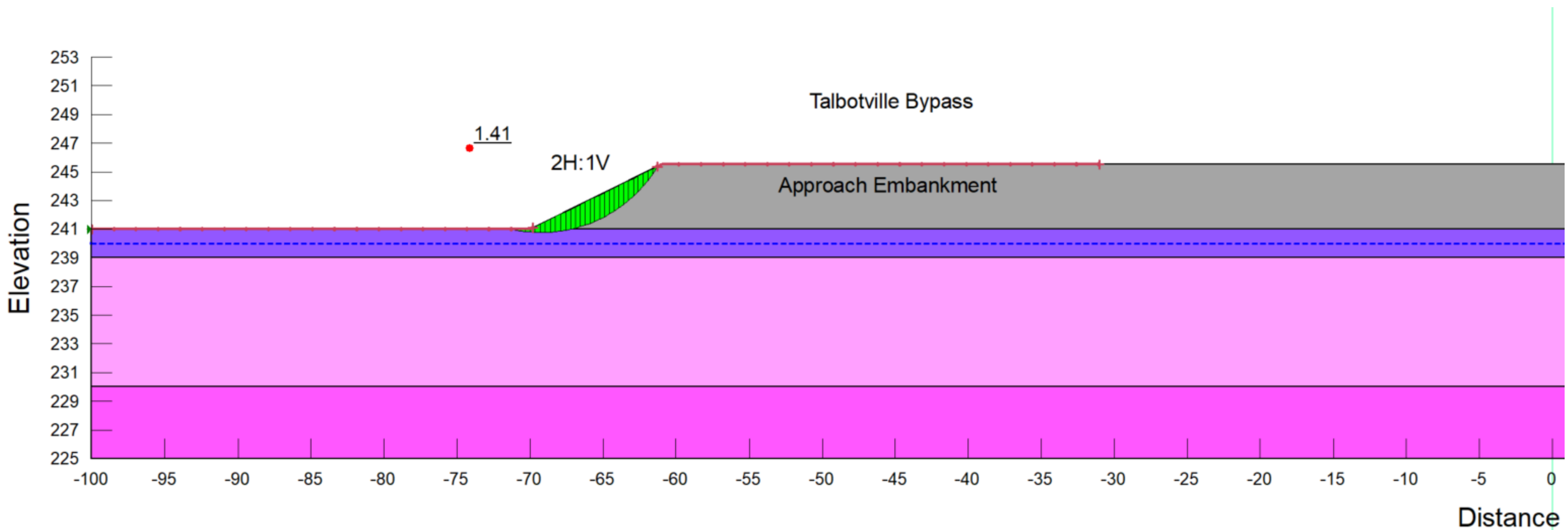
New Cohesive Embankment

Talbotville Bypass at CNR Overhead

Figure E7

Project No. 165001308

GWP No. 3042-22-00



Color	Name	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Firm CLAYEY SILT to SILTY CLAY TILL (Drained)	20	0	28
	New Cohesive Embankment Fill (Drained)	20.5	2.5	26
	Stiff to hard CLAYEY SILT TILL 1 (Drained)	21	0	30
	Stiff to hard CLAYEY SILT TILL 2 (Drained)	22	0	30

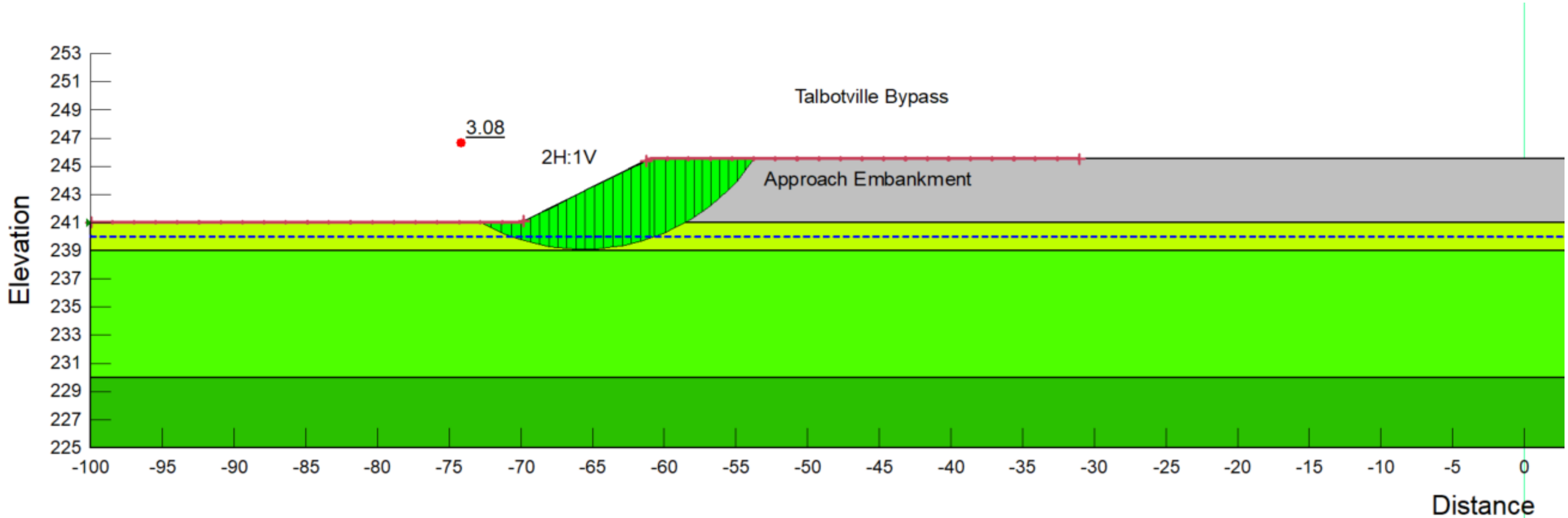
Stantec

Slope Stability Analysis (Static)
(Drained Conditions)
New Cohesive Embankment
Talbotville Bypass at CNR Overhead

Figure E8

Project No. 165001308

GWP No. 3042-22-00



Color	Name	Unit Weight (kN/m³)	Cohesion (kPa)
	Firm CLAYEY SILT to SILTY CLAY TILL (Undrained)	20	50
	New Cohesive Embankment Fill (Undrained)	20.5	50
	Stiff to hard CLAYEY SILT TILL 1 (Undrained)	21	170
	Stiff to hard CLAYEY SILT TILL 2 (Undrained)	22	220

Stantec

Slope Stability Analysis (Pseudo-static)
(Undrained Conditions)
New Cohesive Embankment
Talbotville Bypass at CNR Overhead

Figure E9

Project No. 165001308

GWP No. 3042-22-00

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 (Vs) soundings and calculate a time weighted average Vs of the upper 30 meters (Vs30) 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 Vs models to over 30 meters deep on most readings. The shear wave velocity results and Vs30 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.

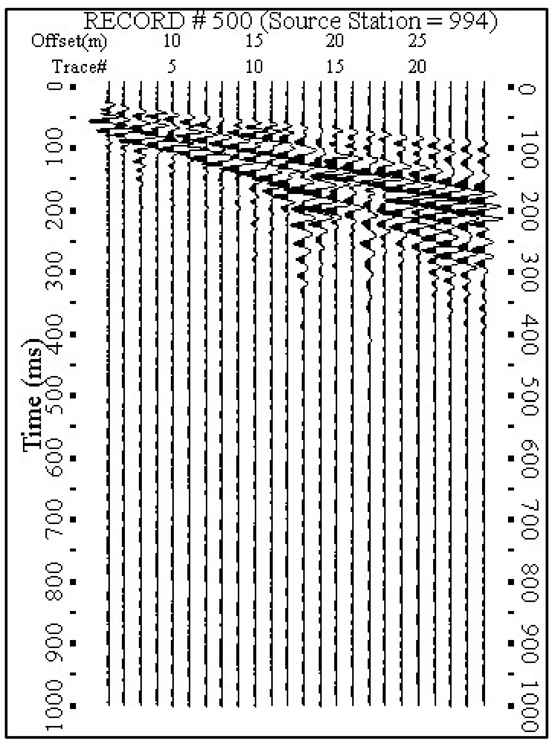


Figure MASW-1. Typical MASW time domain record (shot gather)

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.

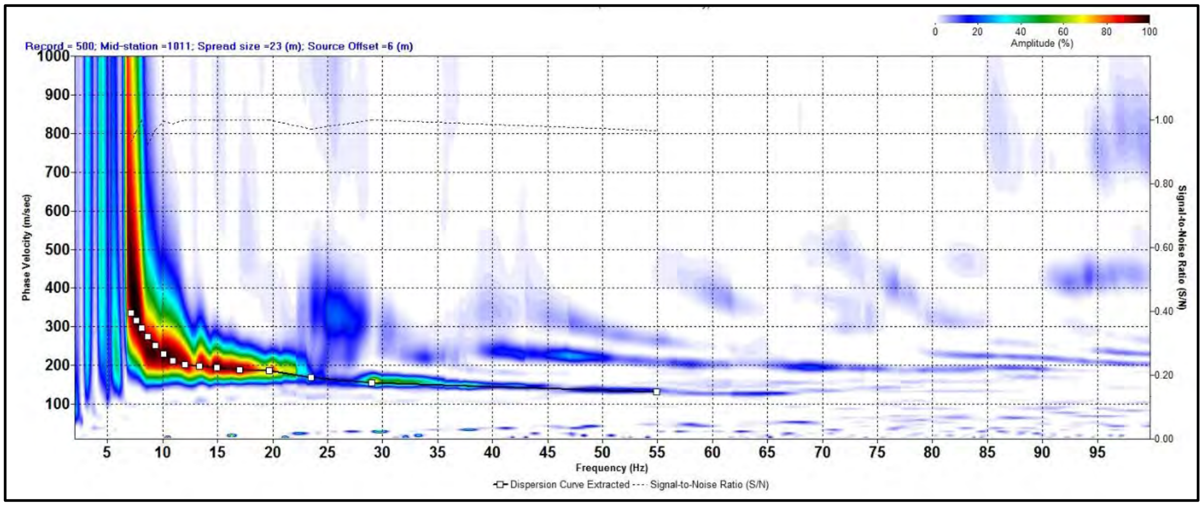


Figure MASW-2. Overtone image and a picked dispersion curve

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.

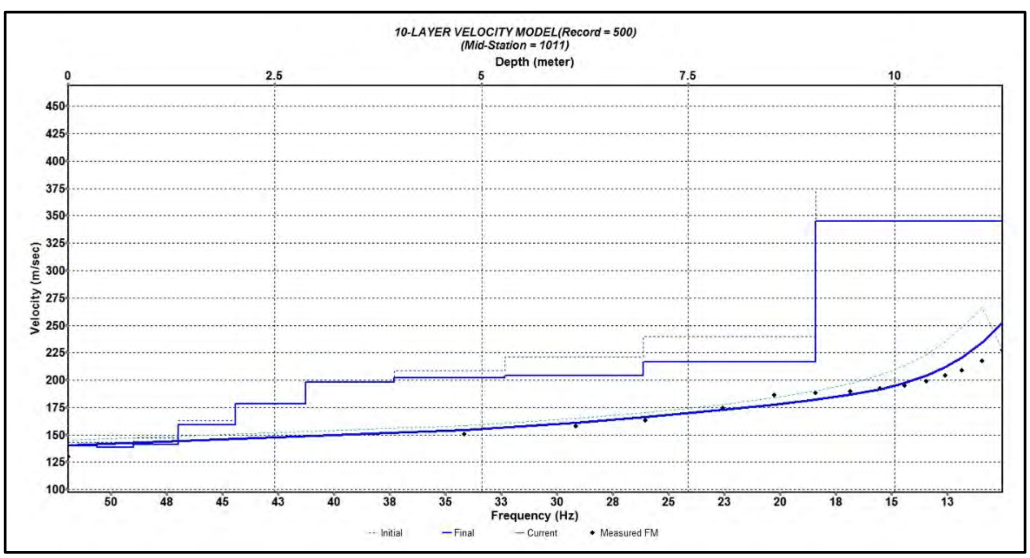


Figure MASW-3. 1D inversion result with fitted dispersion curve

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

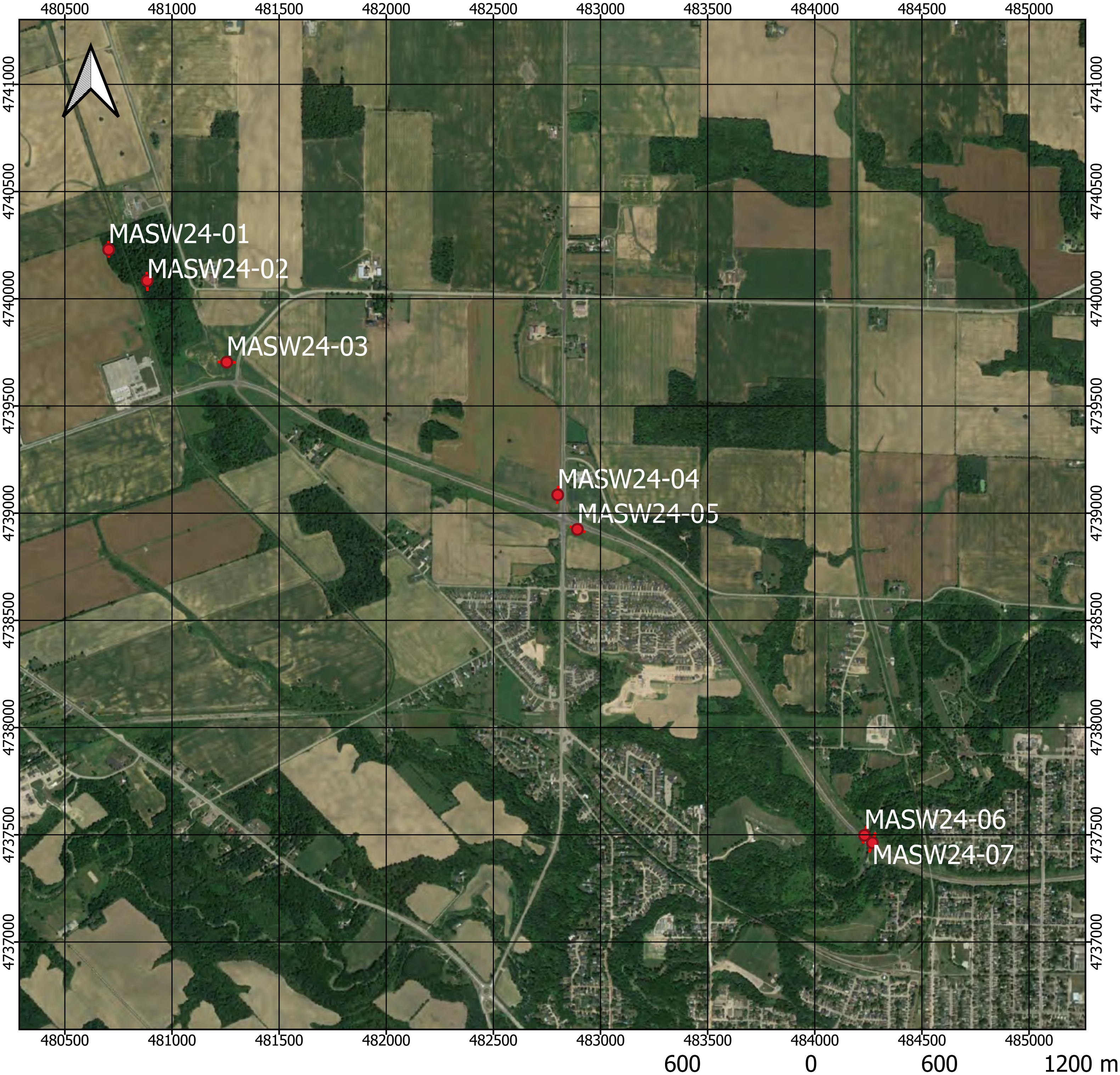
Job No: 24-05-27609
Client: Stantec Consulting Ltd.
Project: HWY 3 St Thomas CPT
Start Date: 14-May-2024
End Date: 16-May-2024

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

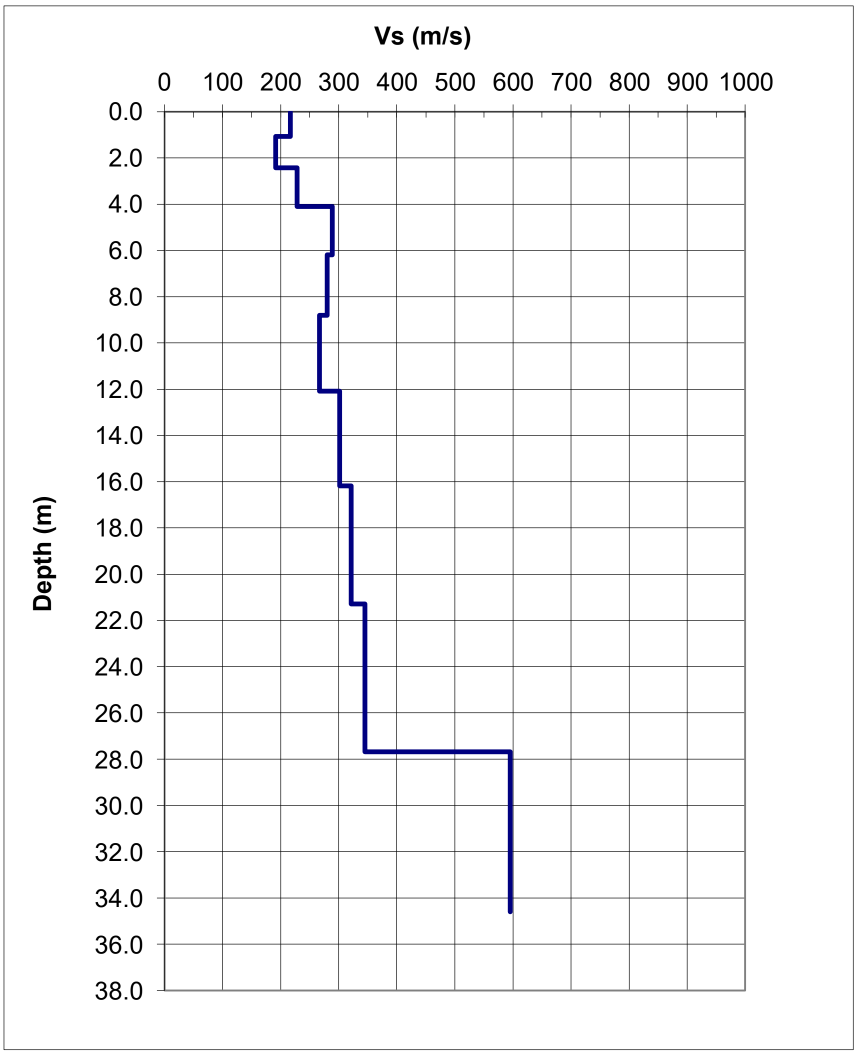
CONETEC

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

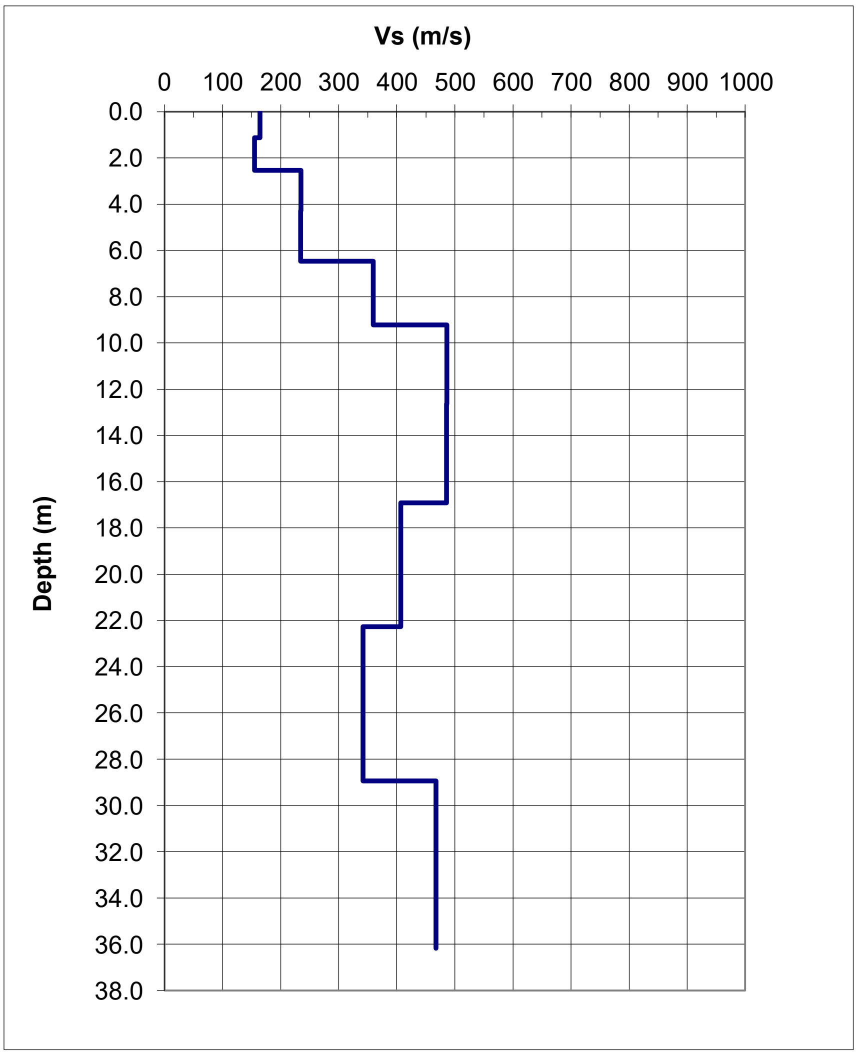
Layer	Layer Thickness (m)	Depth of Bottom of Layer (m)	Vs (m/s)
1	1.07	1.07	217
2	1.34	2.41	191
3	1.68	4.09	228
4	2.10	6.19	289
5	2.62	8.80	280
6	3.27	12.08	267
7	4.09	16.17	302
8	5.12	21.29	322
9	6.39	27.68	345
10	6.92	34.60	595



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

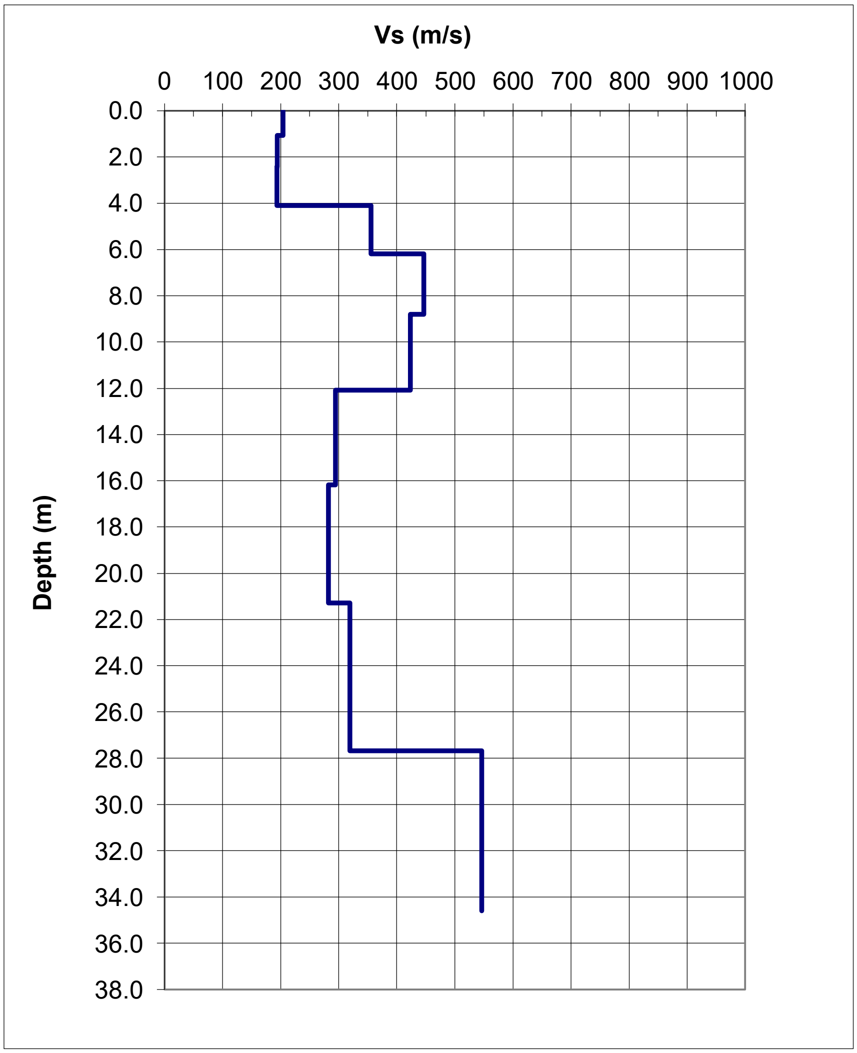
Layer	Layer Thickness (m)	Depth of Bottom of Layer (m)	Vs (m/s)
1	1.12	1.12	164
2	1.40	2.52	155
3	1.75	4.28	235
4	2.19	6.47	234
5	2.74	9.21	360
6	3.42	12.63	486
7	4.28	16.91	485
8	5.35	22.26	407
9	6.69	28.94	342
10	7.24	36.18	467



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

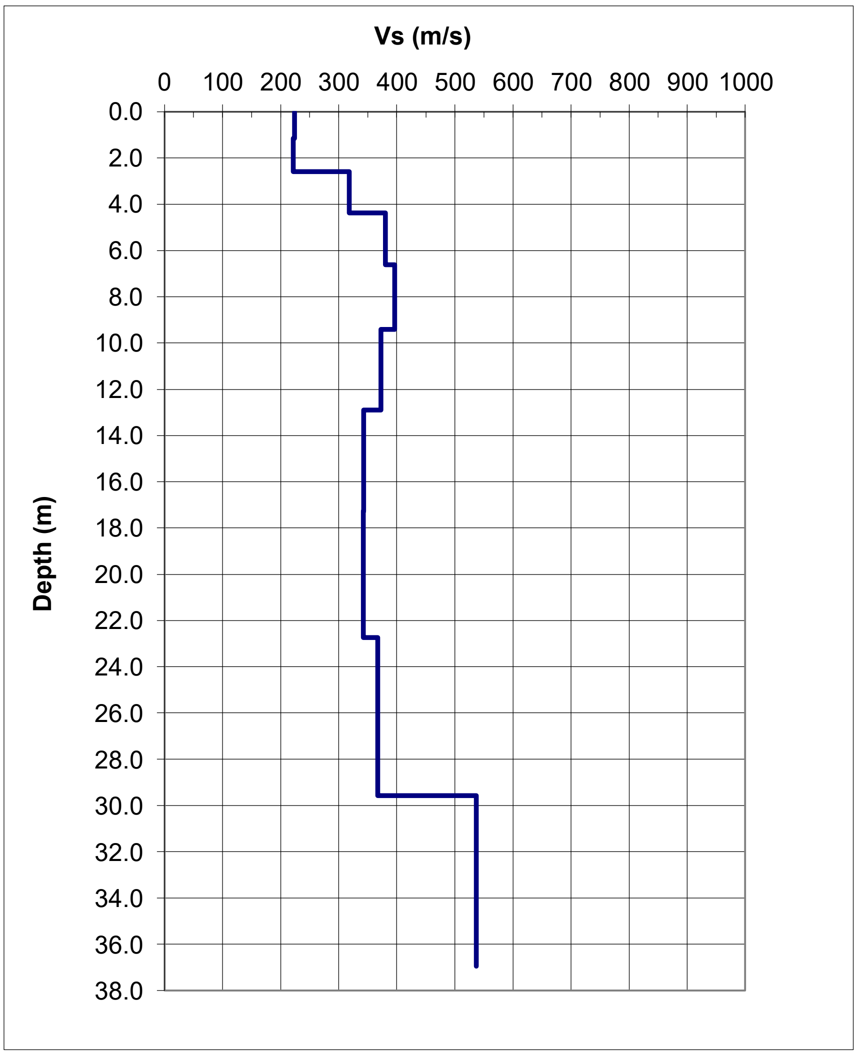
Layer	Layer Thickness (m)	Depth of Bottom of Layer (m)	Vs (m/s)
1	1.07	1.07	204
2	1.34	2.41	194
3	1.68	4.09	194
4	2.10	6.19	356
5	2.62	8.80	446
6	3.27	12.08	423
7	4.09	16.17	294
8	5.12	21.29	283
9	6.39	27.68	319
10	6.92	34.60	546



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

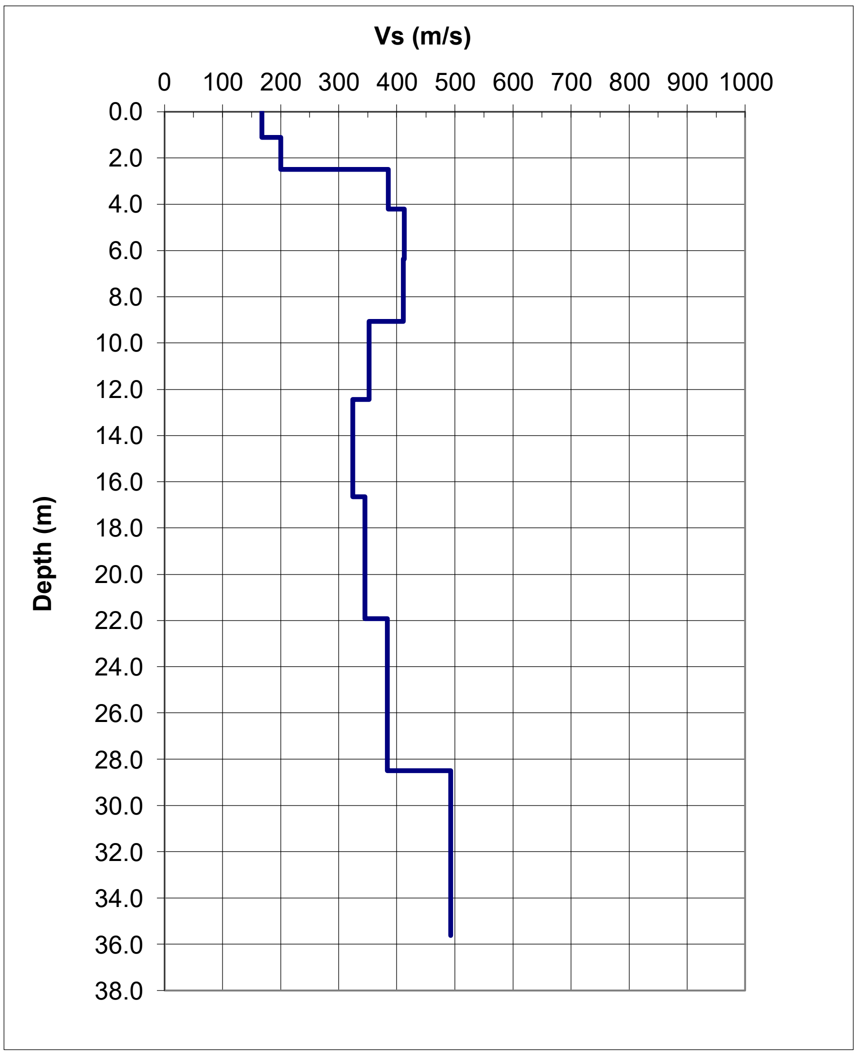
Layer	Layer Thickness (m)	Depth of Bottom of Layer (m)	Vs (m/s)
1	1.15	1.15	224
2	1.43	2.58	222
3	1.79	4.37	318
4	2.24	6.61	380
5	2.80	9.40	396
6	3.50	12.90	373
7	4.37	17.27	343
8	5.46	22.74	342
9	6.83	29.57	367
10	7.39	36.96	537



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

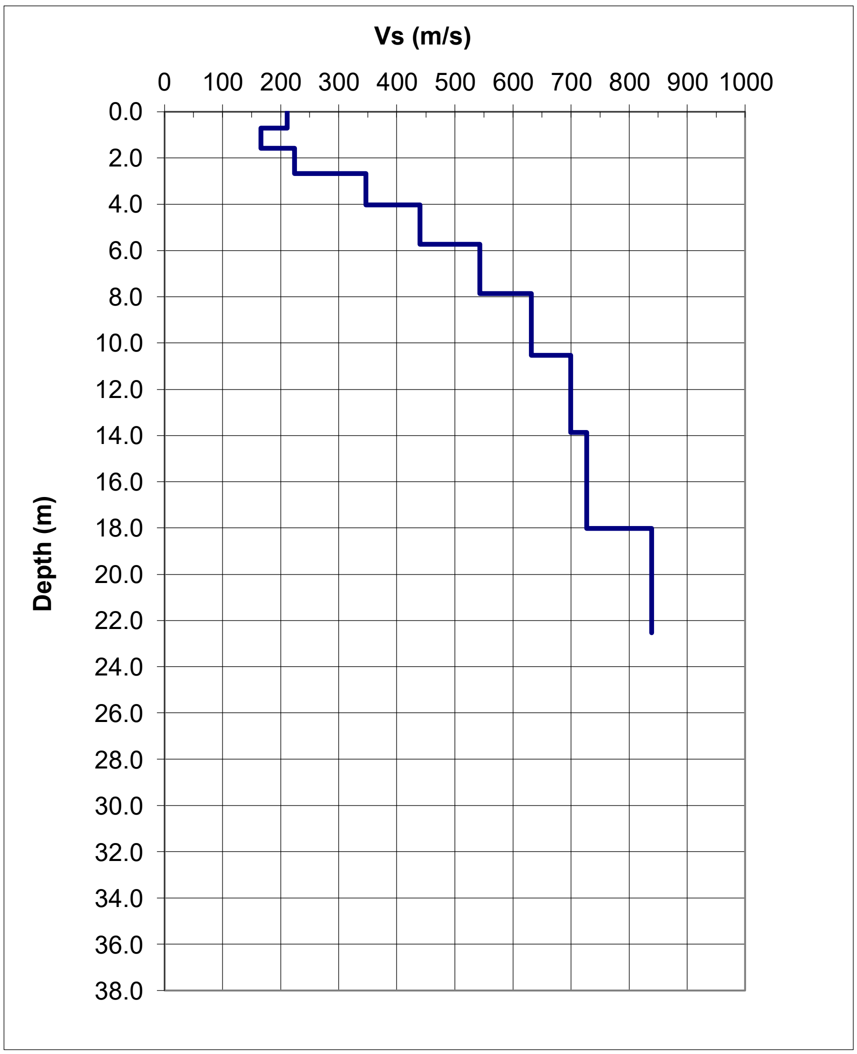
Layer	Layer Thickness (m)	Depth of Bottom of Layer (m)	Vs (m/s)
1	1.10	1.10	168
2	1.38	2.49	201
3	1.73	4.21	385
4	2.16	6.37	413
5	2.70	9.07	411
6	3.37	12.44	352
7	4.21	16.65	324
8	5.27	21.92	345
9	6.58	28.50	383
10	7.13	35.62	493



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

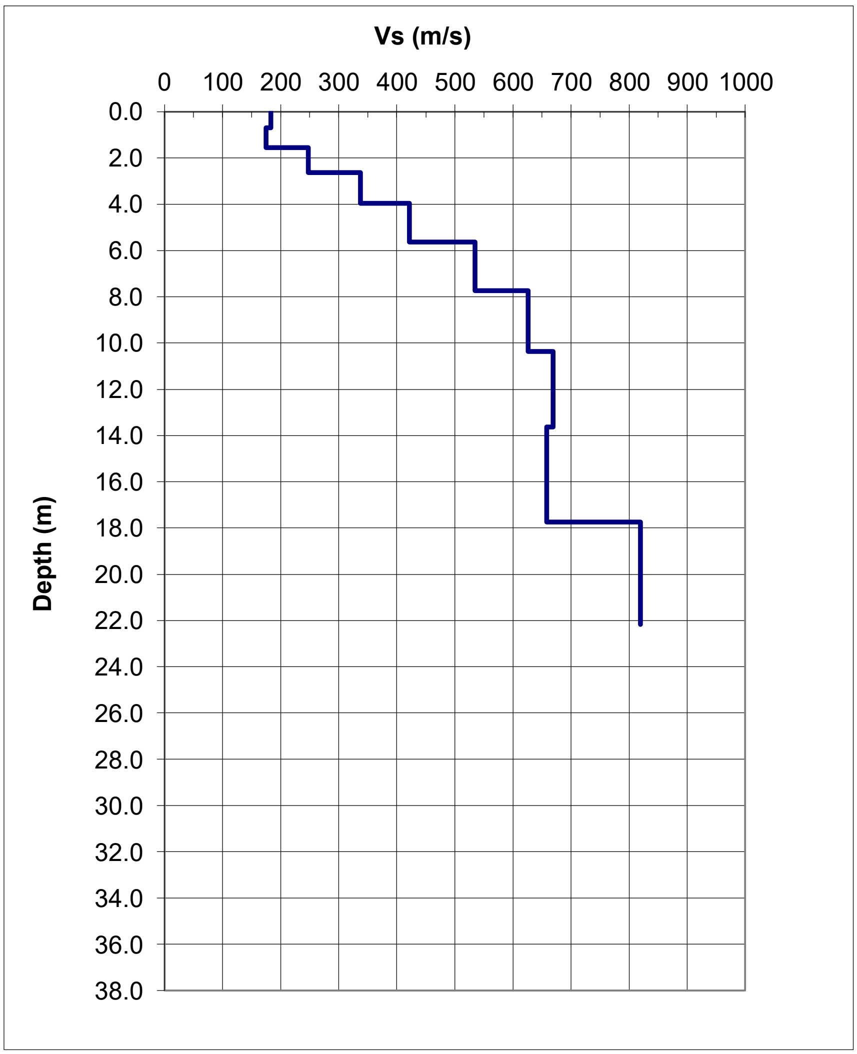
Layer	Layer Thickness (m)	Depth of Bottom of Layer (m)	Vs (m/s)
1	0.70	0.70	211
2	0.87	1.57	166
3	1.09	2.66	224
4	1.36	4.03	346
5	1.71	5.73	440
6	2.13	7.86	543
7	2.67	10.53	631
8	3.33	13.86	699
9	4.16	18.02	727
10	4.51	22.53	839



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

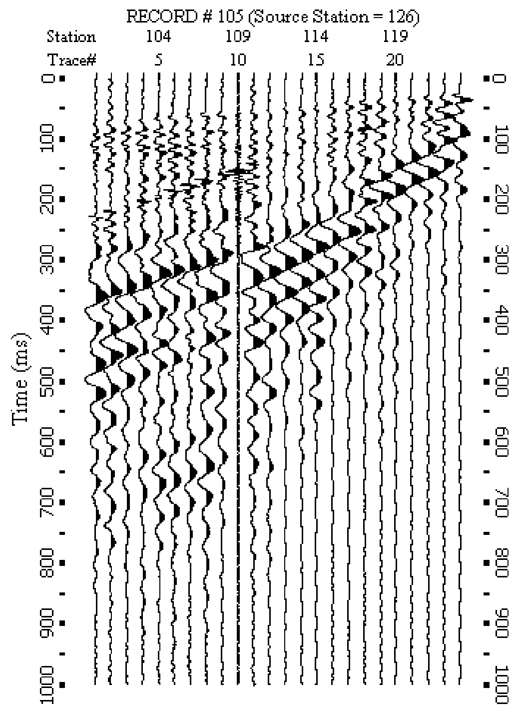
	Job No:	24-05-27609
	Client:	Stantec Consulting Ltd.
	Project:	HWY 3 St Thomas CPT
	Sounding:	MASW24-07
	Date:	16-May-2024

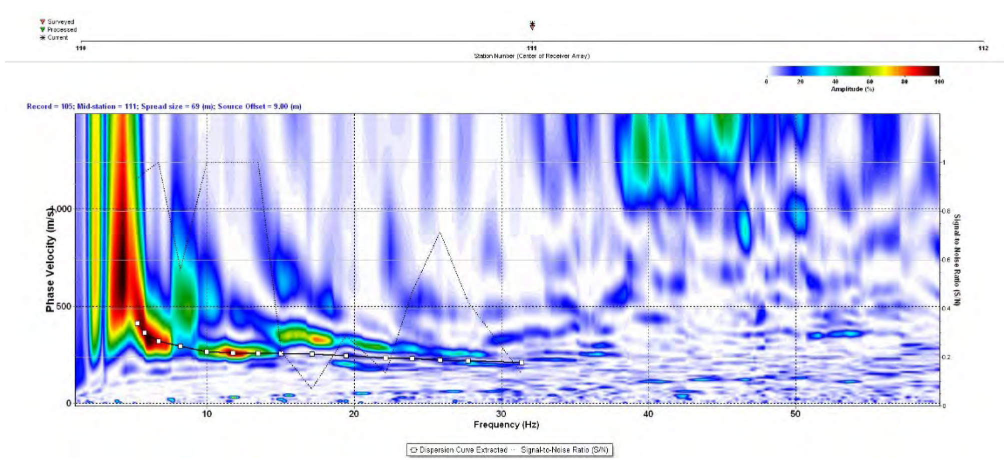
VS30 CALCULATION

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

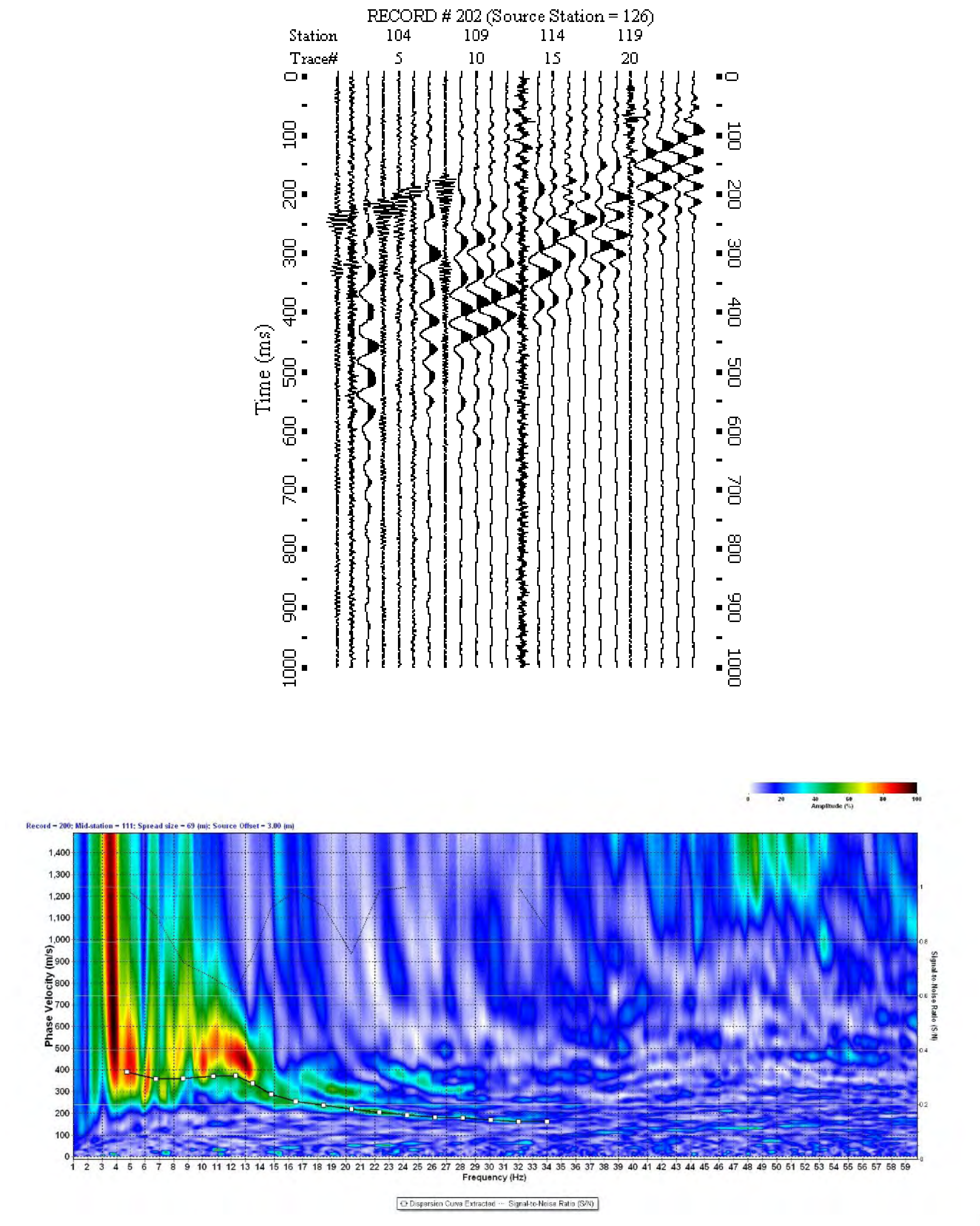
Notes: Yellow Highlighted Cells Indicate Projected Shear Wave Velocity

MASW Time Domain Traces and Overtone Images

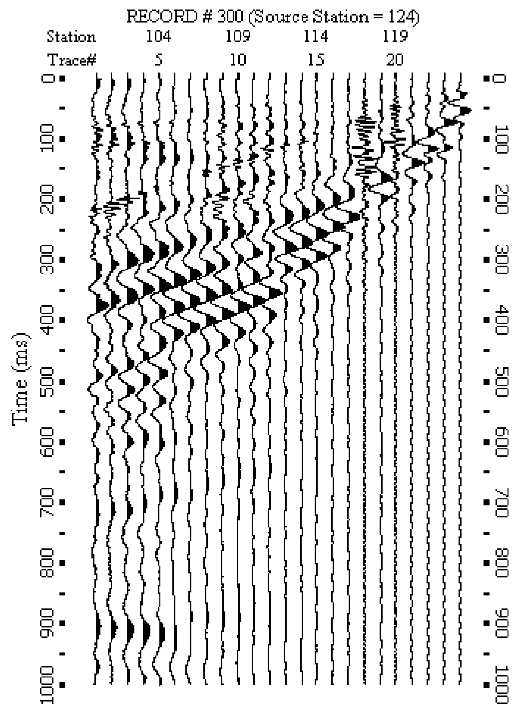


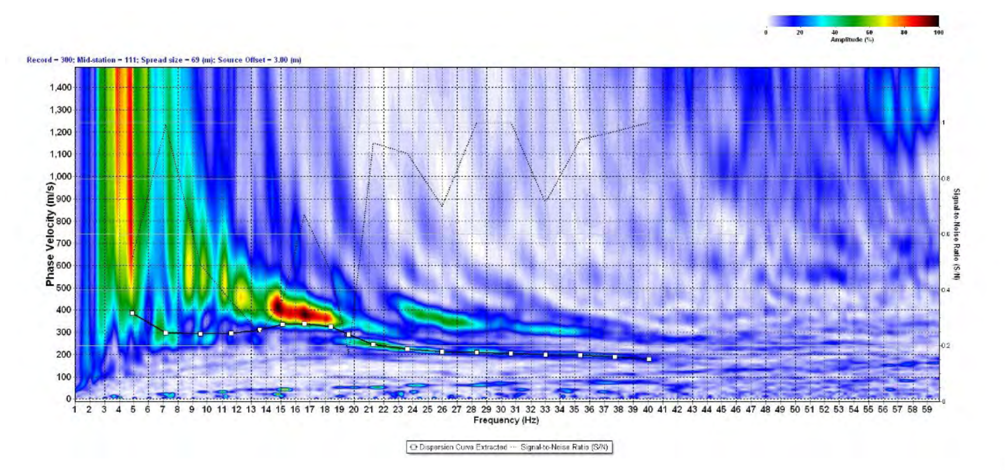


MASW24-01: Example time domain trace for active source (top) with resulting overtone image with picked dispersion curve (bottom).

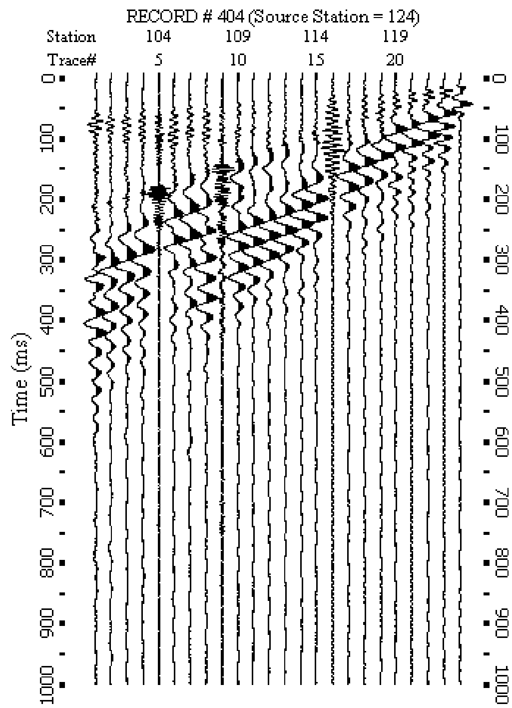


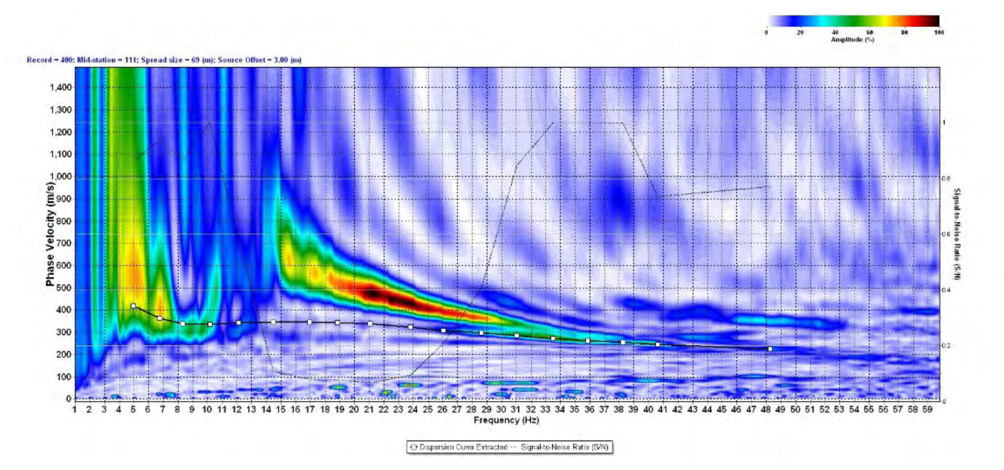
MASW24-02: Example time domain trace for active source (top) with resulting overtone image with picked dispersion curve (bottom).



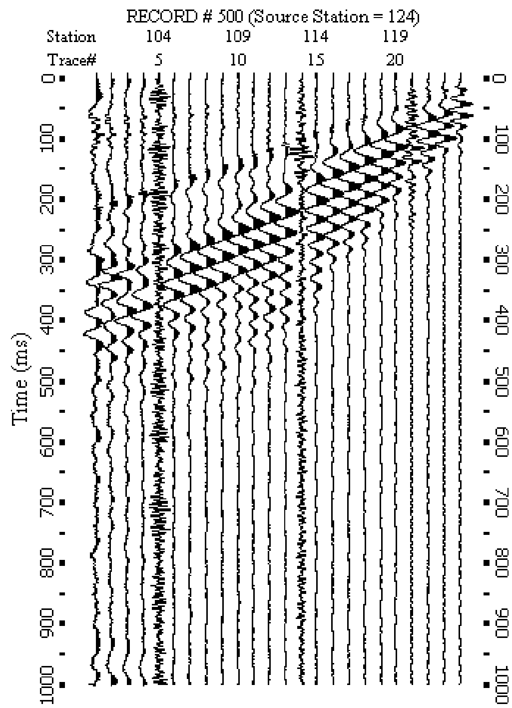


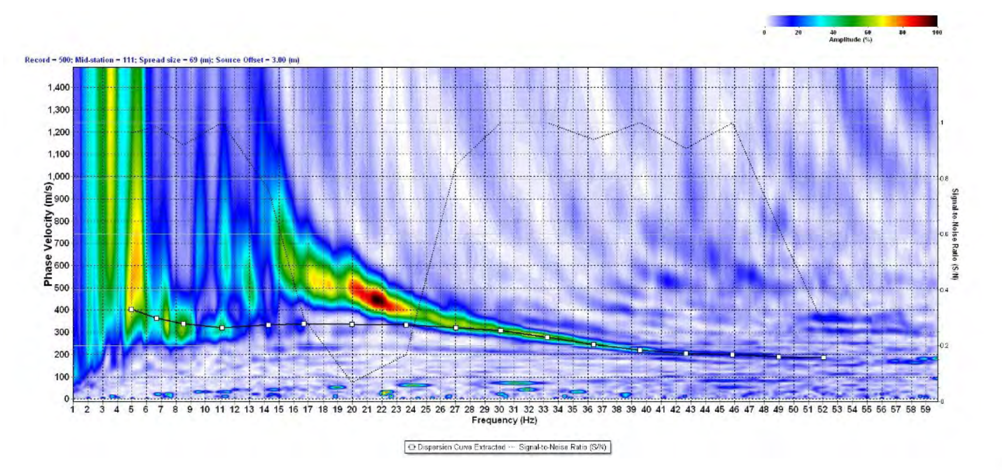
MASW24-03: Example time domain trace for active source (top) with resulting overtone image with picked dispersion curve (bottom).



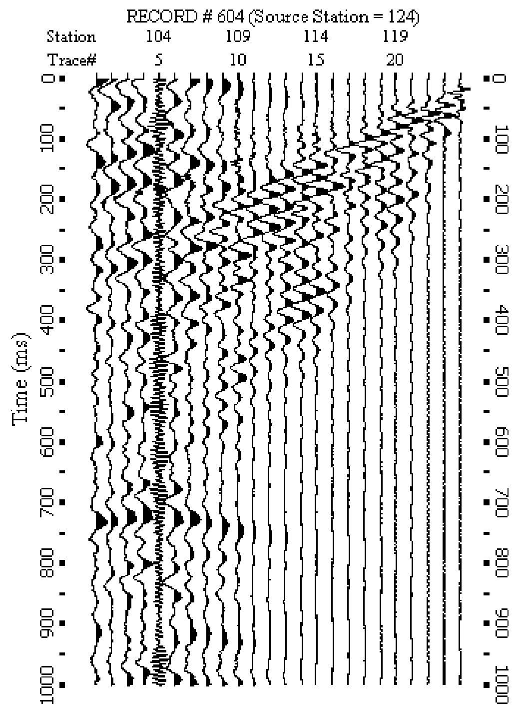


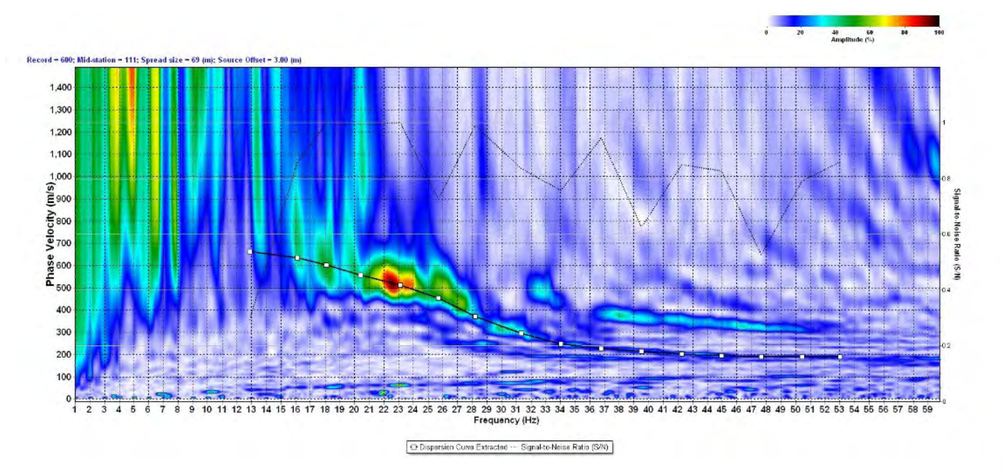
MASW24-04: Example time domain trace for active source (top) with resulting overtone image with picked dispersion curve (bottom).



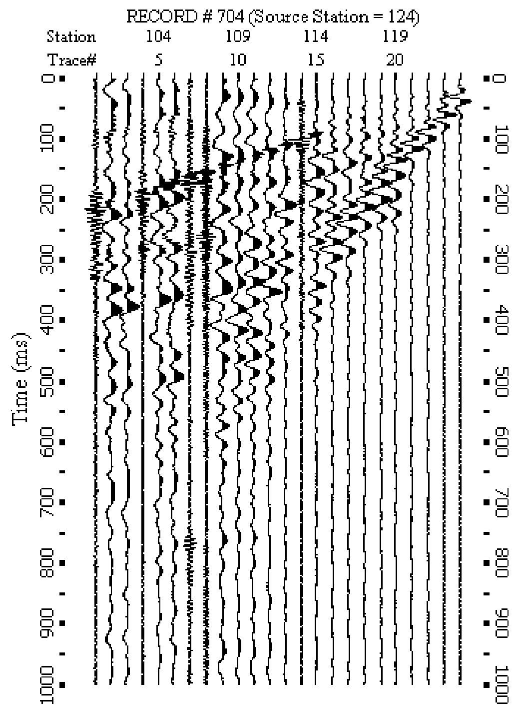


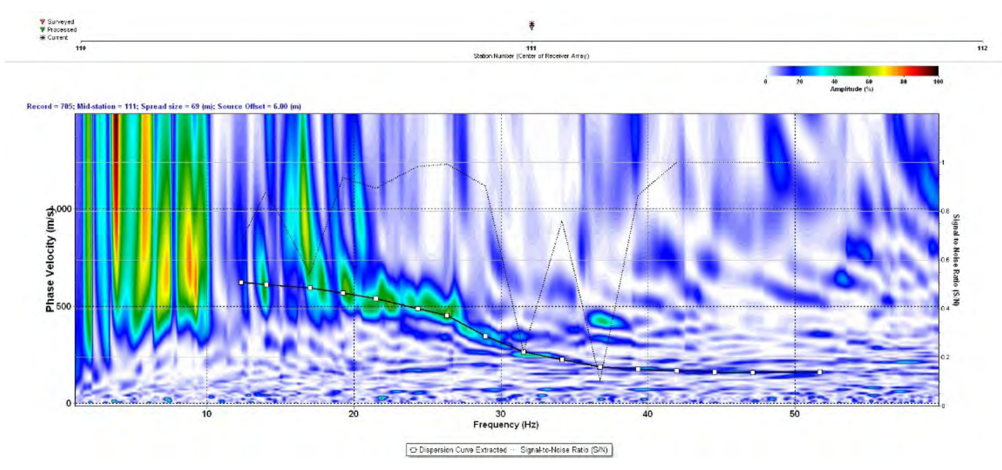
MASW24-05: Example time domain trace for active source (top) with resulting overtone image with picked dispersion curve (bottom).





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).

2020 National Building Code of Canada Seismic Hazard Tool

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_{300}
Latitude (°)	42.814
Longitude (°)	-81.235

Please select one of the tabs below.

NBC 2020 Additional Values Plots API

Background Information

The 5%-damped spectral acceleration ($S_a(T,X)$, where T is the period, in s, and X is the site designation) and peak ground acceleration (PGA(X)) values are given in units of acceleration due to gravity (g, 9.81 m/s^2). Peak ground velocity (PGV(X)) values are given in m/s. Probability is expressed in terms of percent exceedance in 50 years. Further information on the calculation of seismic hazard is provided under the *Background Information* tab.

The 2%-in-50-year seismic hazard values are provided in accordance with Article 4.1.8.4. of the NBC 2020. The 5%- and 10%-in-50-year values are provided for additional performance checks in accordance with Article 4.1.8.23. of the NBC 2020.

See the *Additional Values* tab for additional seismic hazard values, including values for other site designations, periods, and probabilities not defined in the NBC 2020.

NBC 2020 - 2%/50 years (0.000404 per annum) probability

$S_a(0.2, X_{300})$	$S_a(0.5, X_{300})$	$S_a(1.0, X_{300})$	$S_a(2.0, X_{300})$	$S_a(5.0, X_{300})$	$S_a(10.0, X_{300})$	$PGA(X_{300})$	$PGV(X_{300})$
0.192	0.146	0.0805	0.0378	0.00978	0.00332	0.102	0.0905

The log-log interpolated 2%/50 year $S_a(4.0, X_{300})$ value is : **0.0136**

▼ Tables for 5% and 10% in 50 year values

NBC 2020 - 5%/50 years (0.001 per annum) probability

$S_a(0.2, X_{300})$	$S_a(0.5, X_{300})$	$S_a(1.0, X_{300})$	$S_a(2.0, X_{300})$	$S_a(5.0, X_{300})$	$S_a(10.0, X_{300})$	$PGA(X_{300})$	$PGV(X_{300})$
0.112	0.0848	0.0458	0.0209	0.00502	0.00173	0.0579	0.0494

The log-log interpolated 5%/50 year $S_a(4.0, X_{300})$ value is : **0.0071**

NBC 2020 - 10%/50 years (0.0021 per annum) probability

$S_a(0.2, X_{300})$	$S_a(0.5, X_{300})$	$S_a(1.0, X_{300})$	$S_a(2.0, X_{300})$	$S_a(5.0, X_{300})$	$S_a(10.0, X_{300})$	$PGA(X_{300})$	$PGV(X_{300})$

$S_a(0.2, X_{300})$	$S_a(0.5, X_{300})$	$S_a(1.0, X_{300})$	$S_a(2.0, X_{300})$	$S_a(5.0, X_{300})$	$S_a(10.0, X_{300})$	$PGA(X_{300})$	$PGV(X_{300})$
0.0705	0.0529	0.0278	0.0122	0.00272	0.000935	0.0356	0.0289

The log-log interpolated 10%/50 year $S_a(4.0, X_{300})$ value is : **0.0039**

Download CSV

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Date modified: 2021-04-06