



THURBER ENGINEERING LTD.

**FOUNDATION INVESTIGATION AND DESIGN REPORT
HIGHWAY 401 NAPANEE RIVER BRIDGE
GREATER NAPANEE, ONTARIO
ASSIGNMENT NO. 4017-E-0021
GWP 4040-20-00**

SITE NO. 17X-0062/BO

GEOCRES NO.: 31C-302

Report to:

McIntosh Perry Consulting Engineers

Latitude: 44.267586°
Longitude: -76.932411°

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PART 1. FACTUAL INFORMATION

1 INTRODUCTION

This section of the report presents the factual findings obtained from a foundation investigation completed at the Highway 401 Napanee River Bridge in Greater Napanee, Ontario. Thurber Engineering Limited (Thurber) carried out the current field investigation as a sub-consultant to McIntosh Perry Consulting Engineers (MPCE) under Assignment No. 4017-E-0021.

The purpose of this investigation was to explore the subsurface conditions at the site and based on the data obtained, to provide a borehole location plan, records of boreholes, stratigraphic profile, laboratory test results, and a written description of the subsurface conditions. A model of the subsurface conditions influencing design and construction was developed in the course of the current investigation.

2 SITE DESCRIPTION

The existing bridge is understood to have been constructed in 1960 and is a 30.48 m long three-span concrete rigid frame structure. The bridge is 28.75 m wide with two lanes of highway traffic in each direction. Highway 401 has a centerline median barrier. The Palace Road interchange is located approximately 500 m east of the structure and the speed change lanes start/end approximately 100 m east of the east abutment. The bridge was rehabilitated in 1983 and 1987, and partially rehabilitated in 2017.

No visible signs of distress were noted at the foundation elements of the existing bridge. Evidence of recent settlement was also not observed. Steel guiderails are present east of the bridge adjacent to both sides of the highway and west of the bridge adjacent to the eastbound lane. The embankment slopes are grass covered and appeared to be stable. A ditch is present along the toe of the embankments. The edge of the right-of-way is lined with trees. Agricultural land is present west of the bridge to the north side of Highway 401. Residential buildings are located within approximately 200 m north and south of the bridge. Hydro transmission towers are located approximately 70 m north of the bridge and a buried fibre optic cable is present along the south edge of the right-of-way.

Traffic volumes on this section of Highway 401 are understood to be 37,700 AADT (2016).

The water level in Napanee River was measured at between elevation 85.2 and 85.6 m at the time of the field investigation. Flow was observed from north to south.

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

3 EXISTING GEOTECHNICAL INFORMATION

During preparation of this report, Thurber has reviewed the following historical foundation report that is available in the online Geocres library:

Report by the Engineering Division Hunting Technical and Exploration Services Limited to the Department of Highways Ontario, titled "Preliminary Site Investigation for the Proposed Crossing of Highway No. 401 at the Napanee River near Napanee, Ontario. #58-F-216-C", dated December 4, 1958. [Geocres No. 31C-065]

The investigation was carried out in 1958 before construction of Highway 401 and the existing bridge and included 3 boreholes at each abutment and 3 boreholes along the approaches. The overburden included surficial silt and sand with organic matter over interbedded silt and clay, overlying silt and sand over silty sand till. One borehole indicates bedrock at a depth of 71 feet (21.6 m) corresponding to elevation 211 feet (64.3 m). The remainder of the abutment boreholes were terminated within a glacial till deposit. The relevant drawings and borehole logs are presented in Appendix A and Appendix B, respectfully.

The 1959 construction drawings for the Napanee River bridge that were provided by MPCE include:

Province of Ontario Department of Highways, Planning and Design Branch Design Division, Fredericksburg N. Twp. Bridge #1 (Napanee River). D-8 WP138-58. Dated August 7, 1959.

The drawings show that the proposed structure foundations were supported on piles driven into dense to very dense till. Relevant historical drawings are presented in Appendix A.

4 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field-testing program was carried out between October 25th, 2019 and January 28th, 2020 which comprised of on-water, on-road and off-road drilling components. Two on-road boreholes identified as 19-2 and 19-7 were advanced with a truck-mounted CME-75 drill rig. Four off-road boreholes, identified as 19-1, 19-3, 19-6 and 19-8, were advanced using a portable Explo drill rig as a result of difficult access constraints for a larger drilling equipment. Two on-water boreholes, identified as 19-4 and 19-5, were advanced using a portable Hilti drill rig from a barge.

Two Seismic Cone Penetration Tests (SCPTu), identified as SCPT19-1 and SCPT19-2, were advanced through Highway 401 behind the abutments. Additional details regarding the SCPTu testing equipment and methodology are provided in Appendix E.

Prior to commencement of the field investigation, utility clearances were obtained in the vicinity of the borehole locations.

The approximate location of the boreholes and SCPTs are shown on the Borehole Locations and Soil Strata Drawing included in Appendix A. The coordinates and elevation of the boreholes are provided on this drawing and on the Record of Borehole sheets included in Appendix B. The borehole elevations were surveyed with a Nikon AP-8 level, with an accuracy of +/- 1.5 mm, relative to features on a base plan drawing provided by MPCE. Horizontal locations were measured relative to existing site features. The borehole locations, ground surface elevations and termination depths are summarized in Table 4-1. The site is within MTM Zone 9.

Table 4-1: Borehole Summary

Borehole No.	Foundation Element	Northing (m) (Latitude°)	Easting (m) (Longitude°)	Existing Ground Surface Elevation (m)	Termination Elevation (m)
19-1	West Abutment (off-road)	4,903,162.4 (44.267644°)	270,243.1 (-76.932844°)	87.7	67.7
19-2	West Abutment (on road)	4,903,141.0 (44.267452°)	270,248.3 (-76.932777°)	88.5	66.6
19-3	West Abutment (off-road)	4,903,114.3 (44.267212°)	270,256.3 (-76.932675°)	86.3	62.6
19-4	East Pier (barge)	4,903,170.8 (44.267721°)	270,271.7 (-76.932487°)	83.5	67.6
19-5	West Pier (barge)	4,903,137.1 (44.267419°)	270,275.6 (-76.932435°)	83.8	62.2
19-6	East Abutment (off-road)	4,903,184.6 (44.267847°)	270,293.0 (-76.93222°)	86.8	69.7
19-7	East Abutment (on road)	4,903,163.4 (44.267656°)	270,301.1 (-76.932117°)	88.8	59.6
19-8	East Abutment (off-road)	4,903,148.4 (44.267522°)	270,308.2 (-76.932028°)	86.5	61.7
SCPT19-1	West Abutment (on road)	4,903,138.0 (44.267425°)	270,242.4 (-76.932852°)	88.5	73.2
SCPT19-2	East Abutment (on road)	4,903,169.6 (44.267713°)	270,317.9 (-76.931908°)	88.9	64.9

Soil samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT) in accordance with ASTM D 1586. Thin walled (Shelby) tube samples of the cohesive materials were retrieved at various elevations to obtain relatively undisturbed soil samples for further laboratory testing. In-situ vane shear testing was conducted in the cohesive deposits with an MTO N-sized vane. The boreholes

where advanced to sampled depths ranging from 15.9 to 25.9 m (elev. 69.7 to 62.2 m). Bedrock was cored in Boreholes 19-7 and 19-8; the bedrock surface was encountered at depths of 25.6 and 21.6 m, respectively (elev. 63.2 and 64.9 m).

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

A standpipe piezometer, consisting of 25 mm diameter PVC pipe with a 1.5 m long slotted screen, was installed in Borehole 19-6 and 19-8 to allow for measurements of the groundwater levels. The standpipe piezometer details are illustrated on the Record of Borehole sheets, provided in Appendix B. Following completion of the field investigation, the boreholes and both standpipe piezometers were decommissioned in general accordance with MOE requirements (O.Reg. 903 as amended). The on-road boreholes were capped with granular and cold patch to reinstate the median pavement surface.

5 LABORATORY TESTING

The recovered soil samples were subjected to visual identification and to natural moisture content determination. Selected samples were subjected to Atterberg Limit testing and gradation analysis (hydrometer and/or sieve). One soil sample was submitted for organic content testing. The results of these tests are summarized on the Record of Borehole sheets included in Appendix B. Two relatively undisturbed soil samples obtained with Thin Walled (Shelby) Tubes underwent one-dimensional consolidation testing. Four samples of the bedrock were selected and submitted for Unconfined Compressive Strength (UCS) testing. Four samples were selected and submitted for analytical testing of corrosivity parameters and sulphate content. All laboratory test results from the field investigation are provided in Appendix C.

6 DESCRIPTION OF SUBSURFACE CONDITIONS

Details of the encountered soil stratigraphy are presented on the Record of Borehole sheets included in Appendix B and the Borehole Location and Soil Strata Drawing included in Appendix A. The SCPTu logs are provided in Appendix E. A general description of the stratigraphy, based on the conditions encountered in the boreholes, is given in the following paragraphs. However, the factual data presented on the Record of Borehole sheets takes precedence over this general description for interpretation of the site conditions. It must be recognized that the soil and groundwater conditions will vary between and beyond borehole locations.

In general terms, the encountered stratigraphy consisted of roadway embankment fill overlying surficial silt and sand with organics over interbedded silt and clay, overlying silt and sand over silty sand till. Bedrock was encountered in two boreholes at the east abutment. The boreholes not terminated in bedrock were terminated within the glacial till deposit upon SPT or machine refusal.

Soil classification is in accordance with ASTM D2487 with cohesive soils described as per current MTO terminology.

6.1 Topsoil

Topsoil was encountered in off-road Boreholes 19-1, 19-3, 19-6 and 19-8 and ranged in thickness from 50 mm to 175 mm. It is noted that topsoil thicknesses ranging from 300 mm to 600 mm were observed in the historic boreholes documented within Geocres Report No. 31C 065.

6.2 Fill

Asphalt

On-road Boreholes 19-2 and 19-7 were drilled in the eastbound median shoulder and encountered an asphalt thickness of 125 mm.

Fill: Sand / Sand with Gravel and Silt

Sand fill was encountered below the topsoil in Borehole 19-1 and sand fill with gravel and silt was encountered directly below the asphalt within Boreholes 19-2 and 19-7. The fill varied in thickness from 0.4 m to 3.2 m and extended to base depths ranging from 0.6 m to 3.3 m (base elevations 87.1 m to 85.2 m).

SPT N-Values in the sand fill generally ranged from 8 to 104 blows indicating a loose to very dense relative density. Recorded moisture contents ranged from 2 to 11%.

Gradation analyses completed on two samples of the sand fill are illustrated in Figure C1 of Appendix C. The results of the tests are summarized below and are presented on the Record of Borehole sheets in Appendix B.

Soil Particle	Percentage (%)
Gravel	20 to 35
Sand	52 to 68
Silt & Clay	12 to 13

6.3 Upper Sand to Silty Sand

A deposit of sand to silty sand was encountered below the fill in Boreholes 19-2 and 19-7, below the topsoil in Boreholes 19-6 and 19-8, below the clay in Borehole 19-3, and at the riverbed surface in Borehole 19-4. The sand deposit varied in thickness from 0.8 m to 3.8 m and extended to base depths ranging from 0.9 m to 6.1 m (elev. 83.2 m to 82.4 m). Organic matter was noted near the surface of this layer in several of the boreholes.

SPT N-Values in this deposit ranged from weight of hammer to 13 blows indicating a very loose to compact relative density. Recorded moisture contents typically ranged from 7 to 36% with some measurements near surface ranging from 57% to 72%. These higher moisture contents are likely due to the organic content.

The results of gradation analyses completed on six samples in this deposit are illustrated in Figure C2 of Appendix C. The results are summarized below and are presented on the Record of Borehole sheets in Appendix B.

Soil Particle	Percentage (%)	
Gravel	0 to 6	
Sand	51 to 89	
Silt	22 to 29	9 to 13
Clay	8 to 16	

Atterberg limits testing was conducted on four samples of the deposit and all were determined to be non-plastic.

6.4 Clayey Silt to Clay

A deposit of clayey silt to clay was encountered below the fill in Borehole 19-1, below the upper sand to silty sand in Boreholes 19-2, 19-4, 19-6, 19-7 and 19-8, below the topsoil in Borehole 19-3 and directly beneath the riverbed in Borehole 19-5.

6.4.1 Clayey Silt (CL) with Sand to Clay with Sand (CI) to Clayey Gravel (GC) with Sand

Clayey silt with sand was encountered below the topsoil in Borehole 19-3, clayey gravel with sand was encountered below the lakebed in Borehole 19-5 and clay with sand was encountered below the fill in Borehole 19-1. The deposit varied in thickness from 2.4 m to 5.5 m and extended to base depths ranging from 3.0 m to 6.1 m (base elevations 83.2 m to 81.4 m). An organic content of 3.9% was recorded in Borehole 19-3 at a depth of 2.6 m (elev. 83.7 m).

SPT N-Values in this deposit ranged from 1 to 20 blows. No vanes could be turned in this deposit inferring a stiff to very stiff consistency. Recorded moisture contents typically ranged from 23 to 59%.

The results of gradation analyses completed on four samples in this deposit are illustrated in Figure C3 in Appendix C. The results are summarized below and are presented on the Record of Borehole sheets in Appendix B.

Soil Particle	Percentage (%)
Gravel	0 to 35
Sand	18 to 27
Silt	18 to 58
Clay	15 to 51

Atterberg limit testing was completed on four samples of the deposit. The results are illustrated in Figure C9 in Appendix C. The results are summarized below and are presented on the Record of Borehole sheets in Appendix B. The laboratory results indicate the deposit exhibits low to intermediate plasticity (CL to CI).

Parameter	Value
Liquid Limit	29 to 48
Plastic Limit	20 to 23
Plasticity Index	7 to 27

6.4.2 Clayey Silt (CL-ML to CL) to Clay (CH)

Clayey silt to clay was encountered below the clay with sand in Borehole 19-1, below the clayey gravel in Boreholes 19-5 and below the sand with silt to silty sand in Boreholes 19-2, 19-3, 19-4, 19-6, 19-7 and 19-8. Occasional silt and sand seams were noted throughout this deposit. The deposit varied in thickness from 4.9 m to 9.2 m and extended to base depths ranging from 9.4 m to 15.2 m (base elevations 76.7 m to 73.2 m).

SPT N-Values recorded within this deposit ranged from weight of hammer to 13 blows. Field vane tests were performed within the deposit and recorded undrained shear strengths ranging from 21 to 98 kPa. It is noted that at some locations field vane tests were attempted but could not be sheared indicating shear strengths greater than 100kPa, or possibly indicating the presence of silt seams. Overall this deposit varied from soft to very stiff in consistency. The sensitivity typically ranged from 3 to 9 indicating medium to extra sensitivity. The recorded moisture contents ranged from 20% to 67%.

The results of gradation analyses completed on ten samples in this deposit are illustrated in Figures C4 and C5 of Appendix C. The results are summarized below and are presented on the Record of Borehole sheets in Appendix B.

Soil Particle	Value
Gravel	0 to 2
Sand	1 to 52(*)
Silt	22 to 75
Clay	16 to 77

Note: () gradation was completed on sample containing silt and sand seam*

Atterberg limit testing was completed on ten samples of the deposit. The results are illustrated in Figures C10 and C11 in Appendix C. The results are summarized below and are presented on the Record of Borehole sheets in Appendix B. The laboratory results indicate the deposit exhibits low to high plasticity (CL-ML to CH).

Parameter	Value
Liquid Limit	19 to 71
Plastic Limit	13 to 23
Plasticity Index	4 to 48

Oedometer (one-dimensional consolidation) tests were carried out on two relatively undisturbed samples obtained with Thin Walled (Shelby) tube taken from Boreholes 19-7 and 19-8. The results are presented in Appendix C and summarized in the following table. It should be expected that the compressibility characteristics will vary with depth in accordance with the soils index parameters and stress history.

Table 6-1. Summary of 1-D Consolidation Test Parameters

Parameter	Borehole	
	19-7 ST10	19-8 ST7
Sample Depth (m)	11.0	4.9
Sample Elevation (m)	77.8	81.6
Natural Moisture Content, w_n (%)	29	66
Liquidity Index	1.4	0.9
Initial Void Ratio, e_o (-)	0.779	1.806
Unit Weight, γ (kN/m ³)	19.5	15.9
In-situ Vertical Effective Stress, p_o' (kPa)	130	40
Preconsolidation Pressure, p_c' (kPa)	165	280
Overconsolidation Ratio, OCR	1.3	7.5
Recompression Index, C_r (-)	0.03	0.05
Coefficient of Reconsolidation, c_{vr} (cm ² /sec)	2×10^{-3}	5×10^{-3}
Compression Index, C_c (-)	0.3	1.2
Coefficient of Consolidation, c_v (cm ² /sec)	2×10^{-4}	4×10^{-4}

6.5 Lower Sandy Silt to Silty Sand

A deposit of sandy silt to silty sand to sand with trace gravel was encountered below the clayey silt to clay deposit in Boreholes 19-2 through 19-8. This deposit varied in thickness from 3.0 m to 9.4 m and extended to base depths ranging from 14.9 m to 22.9 m (elevations 63.8 m to 71.5 m).

SPT N-Values in this deposit ranged from 3 blows per 300 mm of penetration up to 100 blows per 250 mm of penetration indicating a very loose to very dense relative density. Recorded moisture contents typically ranged from 12 to 28%.

The results of gradation analyses completed on nine samples in this deposit are illustrated in Figures C6 and C7 of Appendix C. The results are summarized below and are presented on the Record of Borehole sheets in Appendix B.

Soil Particle	Percentage (%)	
Gravel	0 to 7	
Sand	34 to 95	
Silt	23 to 55	4 to 31
Clay	2 to 14	

Atterberg limits testing was conducted on five samples of the deposit and were determined to be non-plastic.

6.6 Silty Sand to Sandy Silt: Till

A deposit of glacial till was encountered below the clayey silt in Borehole 19-1 and below the lower sandy silt to silty sand deposit in Boreholes 19-2 through 19-8. The till consisted of a heterogeneous mixture of gravel, sand, silt and clay, most commonly a silty sand to sandy silt. Cobbles and boulders were also observed within this deposit. In some boreholes, coring was required to get through the boulders. Pieces up to 225 mm were obtained by coring. Where fully penetrated in Boreholes 19-7 and 19-8, the thickness of the deposit was 2.7 m and 1.8 m, respectively with a base depth of 25.6 m and 21.6 m (base elevations 63.2 m and 64.9 m). The remaining boreholes were terminated in this deposit at depths ranging from 17.1 m to 23.6 m (base elevations 69.7 m to 62.2 m).

SPT N-Values in this deposit ranged from 13 blows per 300 mm of penetration up to 100 blows per 0 mm of penetration indicating a compact to very dense relative density. Recorded moisture contents typically ranged from 10 to 21%.

The results of gradation analyses completed on six samples in this deposit are illustrated in Figure C8 of Appendix C. The results are summarized below and are presented on the Record of Borehole sheets in Appendix B.

Soil Particle	Percentage (%)	
Gravel	0 to 35	
Sand	32 to 91	
Silt	7 to 48	14 to 45
Clay	2 to 12	

Atterberg limits testing was conducted on two samples of the deposit and were determined to be non-plastic.

6.7 Bedrock and Refusal

Boreholes 19-1 through 19-6 were terminated at SPT refusal blow counts or drill rig refusal. Boreholes 19-7 and 19-8 were terminated in bedrock. A summary of the borehole terminations is summarized in Table 6-2 below.

Table 6-2. Summary of Refusal Depths/Bedrock Surface

Borehole ID	Termination Depth (m)	Elevation (m)	Comments
19-1	20.0	67.7	>3m past SPT Refusal
19-2	21.9	66.6	>3 m past SPT Refusal
19-3	23.6	62.6	SPT Refusal/ Drill Rig Refusal***
19-4	18.2*/15.9**	67.6	>3 m past SPT Refusal
19-5	23.3*/21.6**	62.2	SPT Refusal / Drill Rig Refusal***
19-6	17.1	69.7	3 m past SPT Refusal
19-7	25.6	63.2	Cored 3m Bedrock
19-8	21.6	64.9	Cored 3m Bedrock
SCPT19-1	15.3	73.2	SCPT Refusal
SCPT19-2	24.0	64.9	SCPT Refusal
<p>* Termination depth below top of barge ** Termination depth below riverbed *** Drill Rig Refusal = reached depth where drill rig had insufficient downforce/power to core further</p>			

The bedrock encountered in Boreholes 19-7 and 19-8 consisted of grey slightly weathered to fresh limestone. The Total Core Recovery (TCR) measured on the recovered bedrock cores was 100%, the Solid Core Recovery (SCR) ranged from 77 to 100% and the Rock Quality Designation (RQD) ranged from 76 to 100%. Based on these measured RQD values, the bedrock is classified as good to excellent quality in accordance with CFEM (2006).

Unconfined Compressive Strength (UCS) testing was carried out on four samples of the bedrock core with results ranging from about 44 to 177 MPa, indicating the bedrock to be medium strong to very strong.

Photographs of the bedrock cores are provided in Appendix C.

6.8 Water Level

A standpipe piezometer was installed in Boreholes 19-6 and 19-8 with screens installed to tip elevations of 70.3 and 78.8 m, respectively. The short term groundwater level measurements are summarized in the table below.

Table 6-3. Measured Water Levels

Location	Depth Below Ground Surface (m)	Elevation (m)	Date
Borehole 19-6 (screen elev. 70.3)	0.3	86.5	January 24, 2020
	0.4	86.4	January 29, 2020
Borehole 19-8 (screen elev. 78.8)	0.1	86.4	January 20, 2020
	0.3	86.2	January 23, 2020
	0.4	86.1	January 24, 2020
	0.4	86.1	January 29, 2020
Napanee River	-	85.2	October 25, 2019
	-	85.5	October 31, 2019
	-	85.6	January 19, 2020

Artesian pressures were noted while advancing some boreholes (19-3, 19-5) in the lower cohesionless deposits.

These observations are considered short term and it should be noted that the groundwater level at the time of construction may be different and seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater level may be at a higher elevation after periods of significant and/or prolonged precipitation.

6.9 Results of Analytical Tests

Four soil samples were submitted for analysis of pH, water soluble sulphate and chloride concentrations, resistivity and conductivity. The analysis results are included in Appendix C and are summarized in the following table.

Table 6-4. Summary of Analytical Test Results

Borehole	19-2	19-3	19-6	19-7
Sample	SS8	SS4	SS3	SS7
Depth (ft)	25' - 27'	7'6" - 9'6"	5' - 7'	20' - 22'
Depth (m)	7.6 - 8.2	2.3 - 2.9	1.5 - 2.1	6.1 - 10.1
Elevation (m)	282.8 - 282.2	280.7 - 280.1	283.2 - 282.6	285.2 - 281.2
Material	Clay	Clayey Silt	Silty Sand	Clayey Silt
Conductivity ($\mu\text{S}/\text{cm}$)	307	1,070	239	518
pH (-)	8.37	7.41	7.44	7.9
Resistivity (Ohm-m)	32.6	9.33	41.8	19.3
Chloride ($\mu\text{g}/\text{g}$)	48	500	50	271
Sulphate ($\mu\text{g}/\text{g}$)	74	72	12	92
Sulphide (%)	0.03	0.04	<0.02	0.09

7 MISCELLANEOUS

Borehole locations were selected relative to existing site features and anticipated foundation locations. Ground surface elevations at the investigated locations were recorded relative to benchmarks provided by MPCE, where possible. The remaining ground surface elevations were obtained off existing site features with elevations provided in the CAD drawing by MPCE.

Three different drill rigs were utilized to carry out the drilling, sampling, in-situ testing and borehole decommissioning; George Downing Estate Drilling Ltd. from Hawkesbury, Ontario for on-road work, Forage M3 Drilling also of Hawkesbury, Ontario for barge/on-water work and Marathon Underground Constructors Corporation of Greely, Ontario for off-road work. Traffic control was carried out by Beacon Lite of Kingston, Ontario. The field investigations were supervised on a full-time basis by Michael Wong, E.I.T. of Thurber. Overall supervision of the investigation program was conducted by Deanna Pizycki, P.Eng.

Routine geotechnical laboratory testing was carried out by Thurber's geotechnical laboratory in Ottawa, Ontario. One-dimensional consolidation testing, UCS testing and organic content testing was completed by Stantec's laboratory in Ottawa, Ontario. Analytical testing was carried out by Paracel Laboratories Ltd. in Ottawa, Ontario. Interpretation of the data and preparation of this report were carried out by Deanna Pizycki, P.Eng. The report was reviewed by Paul Carnaffan, P.Eng. and Dr. P.K. Chatterji, P.Eng., the Designated Principal Contact for MTO Foundation Projects.

Thurber Engineering Ltd.
Report Prepared By:



Deanna Pizycki, P.Eng.
Geotechnical Engineer



Stephen Peters, P.Eng.
Geotechnical Engineer



Paul Carnaffan, P.Eng.
Branch Manager, Principal
Senior Geotechnical Engineer



Dr. P.K. Chatterji, P.Eng.
Review Principal
Senior Geotechnical Engineer

**FOUNDATION INVESTIGATION AND DESIGN REPORT
HIGHWAY 401 NAPANEE RIVER BRIDGE
GREATER NAPANEE, ONTARIO
ASSIGNMENT NO. 4017-E-0021
GWP 4040-20-00**

GEOCRES NO.: 31C-302

PART 2. ENGINEERING DISCUSSION AND RECOMMENDATIONS

8 GENERAL

This section of the report provides an interpretation of the factual data from Part 1 of this report and presents foundation design recommendations to assist the project team in the *preliminary* design of the proposed Napanee River Bridge replacement in Greater Napanee, Ontario. The discussion and recommendations presented in this report are based on the information provided by McIntosh Perry Consulting Engineers (MPCE) and on the factual data obtained during the course of the investigation. Thurber Engineering Limited (Thurber) carried out the current field investigation as a sub-consultant to MPCE under Agreement No. 4017-E-0021.

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

In general terms, the encountered stratigraphy consisted of roadway embankment fill overlying surficial silt and sand with organics over interbedded silt and clay, overlying silt and sand over compact to very dense silty sand till. Bedrock was encountered in two boreholes at the east abutment. The boreholes not terminated in bedrock were terminated within the glacial till deposit upon SPT or drill rig refusal. The groundwater level was recorded between elev. 86.1 m and 86.5 m during the time of geotechnical field investigation.

The existing bridge is understood to have been constructed in 1960 and is a 30.48 m long three-span concrete rigid frame structure. The bridge is 28.75 m wide with two lanes of highway traffic in each direction. The existing bridge is understood to be supported on vertical and battered HP 310x79 steel piles. Highway 401 has a centerline median barrier. The Palace Road interchange is located approximately 500 m east of the structure and the speed change lanes start/end approximately 100 m east of the east abutment. The Napanee River bridge was rehabilitated in 1983 and 1987 including removal of the steel post barriers with concrete barriers, modification of the deck drains and waterproofing of

the deck. The bridge was partially rehabilitated in 2017 including reinforced normal concrete deck overlay, removal and replacement of the approach slabs, modifying and plugging the existing deck drains, waterproofing and paving and concrete patch repairs. The existing embankment slopes do not show signs of any recent instabilities.

8.1 Proposed Structures

Several proposed structure replacement options have been reviewed and presented by MPCE in their *Draft Long List Evaluation Report* dated January 2020. The options that MPCE has determined to be feasible consist of:

- 40 m long single span structure, or
- 48 m long three-span structure.

Both design options for the structure replacement will require permanent embankment widening and grade raise to accommodate an interim 6-lane and an ultimate 8-lane section. It is understood that the construction staging will consist of constructing the new portion of bridge outside the existing structure then shifting traffic to the new structure and completing the construction of middle portion of the bridge. It is noted that the field investigation was carried out based on the assumption that the replacement structure would be a two-span structure (as per the Terms of Reference).

8.2 Applicable Codes and Design Considerations

The geotechnical assessment presented below has been prepared based on the available data regarding the proposed foundations and existing ground conditions and in accordance with the most recent Canadian Highway Bridge Design Code (CHBDC), version CSA S6-19. In accordance with CHBDC, the analysis and design of structures takes into consideration the importance of the structure and the consequence associated with exceeding limit states. The importance category and consequence classification are defined by the Regulatory Authority, which in this case is the Ministry of Transportation, Ontario (MTO).

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

As per Section 6.5.3.2 of the CHBDC, the degree of site prediction model understanding is considered to be *Typical* based on the current information.

The frost penetration depth and associated recommendations are provided in Section 11.4.

9 SEISMIC CONSIDERATIONS

9.1 Spectral and Peak Acceleration Hazard Values

The seismic hazard data for the CHBDC is based on the fifth-generation seismic model developed by the Geological Survey of Canada (GSC). Seismic hazard data for this site has been obtained from the GSC's seismic hazard calculator. The data includes peak

ground acceleration (PGA), peak ground velocity (PGV), and the 5% damped spectral response acceleration values (Sa(T)) for the *reference* ground condition (Site Class C) for a range of periods (T) and for a range of return periods including the 475-year, 975-year and 2475-year events. The GSC seismic hazard calculation data sheet for this site is presented in Appendix F.

The site coefficients used to determine the design spectral acceleration values are a function of the Site Class, PGA and Sa(0.2). The PGA value at this site provided by GSC for a *reference* Site Class C with a 2% probability of exceedance in 50 years (2475-year event) is 0.095 g. This value is to be scaled by the F(PGA) based on the site-specific Site Class, as discussed below.

9.2 Liquefaction Potential

The susceptibility of the cohesive soils at this site to experience liquefaction/cyclic softening was assessed following the Boulanger and Idriss (2007)¹ criteria using measured undrained shear strengths. The results of the analysis indicate the cohesive material is not susceptible to cyclic mobility.

The susceptibility of the cohesionless soils at the site to experience liquefaction was assessed using the CPT data following the simplified method for cohesionless soil as outlined in Boulanger and Idriss (2014)². The cohesionless foundation soils are not considered to be susceptible to liquefaction.

9.3 CHBDC Seismic Site Classification and Performance Category

In accordance with the CHBDC, the selection of the seismic site classification is based on the soil and rock within the upper 30 m of the stratigraphy. As per Table 4.1 of the CHBDC, the foundations soils have been classified as a Seismic Site Class D based on the measured shear wave velocities from the SCPT's. The site adjusted Site Class D PGA is 0.123g.

It is anticipated that this bridge has an importance category of "*major*". As per Section 4.4.4 of the CHBDC, a seismic performance category 1 is applicable at this site for all structural periods (T).

10 STRUCTURE FOUNDATION ALTERNATIVES

Given the soil stratigraphy encountered, the following options have been considered from a geotechnical perspective for the support of the new bridge foundations:

- Spread footings,
- Caissons (drilled shafts), and
- Steel piles (H-piles, pipe piles)

The foundation alternatives are presented below and evaluated from a geotechnical perspective in terms of their respective advantages, disadvantages, risks and consequences. The evaluation is summarized in the table provided in Appendix G.

- Spread Footings

The foundation soils expected to be encountered at the founding elevation for spread footings (below the depth of frost) would consist of loose sand to silty sand and clayey silt which would offer low to moderate bearing resistance. Spread footings to support the abutments would be founded at an elevation below the Napanee River and would require excavations adjacent to the existing foundations and highway embankment. Scour and erosion protection must be considered for footings. Spread footings do not allow for construction of integral abutments, if required. The use of spread footings would also require large cofferdams to construct the in-water pier(s), if applicable.

It is not recommended to support the new bridge on spread footings. It may be possible to support short wingwalls, on spread footings, however, the preliminary design indicates wingwalls cantilevered from the abutment; this would need to be confirmed once the highway profile and wall geometry has been confirmed.

- Caissons

Caisson foundations, particularly when they are socketed into bedrock, offer high geotechnical resistance, however, bedrock was only encountered in two of the boreholes. Caissons at this site would be designed with the tips bearing a minimum of 4 m into the 100 blow glacial till deposit and would offer moderate geotechnical resistance. The high lateral stiffness of caissons is not compatible with integral abutments. A high water level was observed during the time of the field investigation; the drilling methods would need to take into account the potential for unbalanced pressure heads and caisson base boiling while drilling through the cohesionless soils. Founding caissons on dissimilar stratigraphy (e.g. some on bedrock and some in till) could result in differential settlements.

It is not recommended to support the new bridge on caissons.

- Steel piles

Based on the foundation soils encountered as part of the current investigation, driven steel piles are considered feasible for the support of the new abutments and pier. Driven steel piles would allow for the use of integral abutments. The existing bridge is understood to be supported on vertical and battered HP 310x79 piles. Driven piles will typically reduce the volumes of excavation required and will be less susceptible to scour and erosion when compared to shallow foundations.

It is recommended to support the new abutments and pier, where applicable, on driven steel piles.

11 FOUNDATION DESIGN RECOMMENDATIONS

11.1 Driven Steel Piles

11.1.1 Axial Geotechnical Resistance and Founding Elevation

Bedrock was proven by coring in Boreholes 19-7 and 19-8, which were advanced near the east abutment, with bedrock surface elevations ranging from 63.2 to 64.9 m, respectively. Borehole 19-6, also advanced near the east abutment, was terminated in a competent soil stratum at a termination elevation of 69.7 m. Boreholes 19-1 through 19-3 were advanced near the west abutment and were terminated within a competent soil stratum at termination elevations ranging from 67.7 to 62.6 m. Boreholes 19-4 and 19-5 were advanced near the center of the bridge span and were terminated within a competent soil stratum at termination elevations of 67.6 and 62.2 m. Therefore, variable refusal elevations should be anticipated for driven piles. A Notice to Contractor (NTC) indicating the potential for variable foundation tip depths has been provided in Appendix J.

There is a possibility of differential settlements across the structure where piles for an abutment are founded on both till and bedrock and consideration could be given to driving all pile to end bearing in the till. Pile resistances have been provided for bearing both in till and bedrock, where applicable.

The geotechnical axial resistances for HP310x110 piles founded within the till and on bedrock are provide in Table 11-1 and may be used in design. A heavier section pile could be considered to obtain higher resistances.

Table 11-1 Axial Geotechnical Resistances for HP310x110

Foundation	Location	Estimated Tip Elevation (m)	Factored Compression Resistance (Founding Layer)		Factored Tension Resistance (Founding Layer) ULS (kN)
			ULS (kN)	SLS (kN)	
West Abutment	19-1	74.6	1,400 (Till)	900 (Till)	100 (Till)
	19-2	69.7			
	19-3	61.7			
Pier	19-4	68.8			
	19-5	62.1			
East Abutment	19-6	71.0			2,050(*) (Bedrock)
	19-7	64.0			
	19-8	65.5			
	19-7	63.2			
	19.8	64.9			

Note: (*) structural capacity of the pile may govern the design

The geotechnical resistances provided in the table above are applicable for pile spacing greater than 3 pile diameters. The pile resistances include a resistance factor of 0.4 (ϕ_{gu}), 0.8 (ϕ_{gs}) and 0.3 (ϕ_{gu}) for ULS compression, SLS compression and ULS tension values, respectively, as per Table 6.2 of the CHBDC (static analysis – typical understanding).

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

11.1.2 Downdrag

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

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

The neutral plane for static downdrag calculations can be taken as the base of the cohesive deposits.

11.1.3 Lateral Geotechnical Resistance and Group Effects

Piles can be installed with a batter to resist lateral loads for a semi-integral abutment or at piers. During a seismic event, a battered pile foundation may become too rigid, therefore the structural engineer should review.

The lateral resistance of the soil adjacent to a vertical pile is developed on the face of the pile embedded in the foundation soils and estimated using p-y curves.

The p-y curves for static conditions are shown in Appendix H to allow for calculation of the *ultimate* lateral resistance of an individual pile. A geotechnical resistance factor of 0.5 (ϕ_{gu}) and 0.8 (ϕ_{gs}) as per Table 6.2 of the CHBDC (static analysis – typical understanding) should be applied to the *ultimate* ULS and SLS values, respectively.

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

11.1.4 Pile Tips

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

11.1.5 Pile Driving

Pile driving must be carried out in accordance with OPSS 903 and Special Provision 109F57 for piles driven to refusal in the till layer. Pile testing need not be used until piles are within 2 m of the design tip elevation. The appropriate pile driving note is "Piles to be driven to an ultimate resistance of "R" kN per pile". "R" must have a minimum value of twice the design load at ULS. The designer fill-in (*) in Section 903.07.02.07.03.01 shall be *High-Strain Dynamic Testing*.

Retapping of piles should be carried out as per OPSS.PROV 903.

Vibration monitoring of the existing bridge and newly constructed foundations should be considered.

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

11.1.6 Abutment Type

The subsurface conditions at this site are considered suitable for integral abutments. Semi-integral abutments may also be used.

11.2 Wingwalls

Based on preliminary general design alternative drawings, it is understood that the wingwalls will be cantilevered from the abutment stem. Alternatively, the wingwalls could be supported on driven steel piles similar to the abutments.

11.3 Backfill and Lateral Earth Pressures

Structural backfill material should consist of Granular A meeting the OPSS.PROV 1010 specifications. The backfill must be in accordance with OPSS 902 and placed to the extents shown on OPSD 3101.150. The backfill should be compacted and compaction equipment to be used adjacent to the structure must be restricted in accordance with OPSS.PROV 501.

The design of the abutments and wingwalls, where required, must incorporate a subdrain as shown in OPSD 3101.150. Lateral earth pressure provided in the equations in the sections below are based on the assumption that the backfill is fully drained so that there are no unbalanced hydrostatic pressures. If adequate drainage cannot be confirmed, the potential for buildup of hydrostatic pressures should be considered in design.

11.3.1 Static Lateral Earth Pressure

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

$$\sigma_h = K * (\gamma d + q)$$

where:

- σ_h = static lateral earth pressure on the wall at depth d (kPa)
- K = static earth pressure coefficient (see table below)
- γ = unit weight of retained soil (see table below)
adjusted below water level
- d = depth below top of fill where pressure is computed (m)
- q = value of any surcharge (kPa)

A lateral earth pressure due to backfill compaction should be added to the calculated lateral earth pressure in accordance with Clause 6.12 of the CHBDC. Typical earth pressure coefficients for vertical walls for backfill material are shown in Table 11-2.

Table 11-2. Static Earth Pressure Coefficients

Condition	Earth Pressure Coefficient (K)	
	OPSS Granular A and Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active, K_A (Yielding Wall)	0.27	0.40
At Rest, K_O (Non-Yielding Wall)	0.43	-

The parameters in the table correspond to full mobilization of active and passive earth pressures and require certain relative movements between the wall and adjacent soil to produce these conditions. Figure C6.27 and Table C6.12 of the Commentary to the CHBDC indicates the relative movement required to fully mobilize the active earth pressure. Where ground surfaces are sloped behind the walls, the corresponding coefficients provided in the Table 11-2 should be used.

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

Passive earth resistance in front of the structure should be ignored. A geotechnical resistance factor of 0.5 (ϕ_{gu}) should be applied in static design to the passive earth pressures behind the wall in accordance with Table 6.2 of the CHBDC (static analysis - *typical* understanding).

11.3.2 Combined Static and Seismic Lateral Earth Pressure

In accordance with Clause 6.14 of the CHBDC structures should be designed using dynamic earth pressure coefficients that incorporate the effects of earthquake loading. The following recommendations are per Section C6.14.7.2 of the Commentary of the CHBDC

which states that seismically induced lateral soil pressures may be calculated using Mononobe-Okabe Method with:

- $k_h = \frac{1}{2} * F(\text{PGA}) * \text{PGA}$, for structures that allow 25 to 50 mm of movement, and
- $k_h = F(\text{PGA}) * \text{PGA}$, for non-yielding walls

The coefficients of horizontal earth pressure for seismic loading presented in Table 11-3 may be used for vertical walls. The provided earth pressure coefficients are based on a Seismic Site Class D, *reference* PGA with a 2% probability of exceedance in 50 years of 0.095g (Geological Survey of Canada – Fifth Generation) and a F(PGA) of 1.29 as per Table 4.8 of the CHBDC.

Table 11-3. Combined Static and Seismic Earth Pressure Coefficients

Condition	Earth Pressure Coefficient (K)	
	OPSS Granular A and Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Slope Surface Behind Wall (2H:1V)
Active, K_{AE} Yielding Wall	0.31	0.46
Active, K_{AE} Non-Yielding Wall	0.34	0.54

The total pressure due to combined static and seismic loads acting at a specific depth below the top of the wall/soil may be determined using the following equation that includes consideration of material properties and the soils profile.

$$\sigma_{hAE} = K * \gamma * d + (K_{AE} - K_A) * \gamma * (H - d)$$

where:

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

11.4 Frost Depth

The frost penetration depth at this site is 1.5 m as per OPSD 3090.101. Therefore, a minimum of 1.5 m of earth cover, or thermal equivalent, must be provided above the base

of the footings and pile caps to serve as protection against frost. Thermally equivalent frost protection could be in the form of insulation, provided it is placed above the high water level.

Frost taper treatment should be as directed by the Pavement Engineer.

11.5 Cement Type and Corrosion Potential

Analytical tests were completed to determine the potential for degradation of concrete in the presence of soluble sulphates and the potential for corrosion of exposed steel used in buried infrastructure. The concentration of soluble sulphate provides an indication of the degree of sulphate attack that is expected for concrete in contact with soil and groundwater at the site. Soluble sulphate concentrations less than 1000 µg/g generally indicate that a low degree of sulphate attack is expected for concrete in contact with soil and groundwater. The class of concrete selected should consider the effects of road de-icing salts.

The pH, resistivity and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. The tests results provided in Section 6.9 may be used to aid in the selection of coatings and corrosion protection systems for buried steel objects. The corrosive effects of road de-icing salts should also be considered.

The suitability of steel, concrete, insulation and other construction materials in contact within the soils should be reviewed further by a National Association of Corrosion Engineers (NACE) certified corrosion expert.

11.6 Embankment Design and Reinstatement

11.6.1 Embankment Reconstruction and Widening

Based on the short list alternatives provided by MPCE, it is understood that grade raises up to 1.3 to 1.7 m at the east and west abutments respectively, are being considered at this site. Further, future widening of two additional lanes in each direction have been proposed.

Embankment reconstruction and widening should be carried out in accordance with OPSS.PROV 206. The embankment should be reinstated with side slopes of 2H:1V (or flatter) if constructed using Select Subgrade Material (SSM) or Granular B Type I. The fill should be placed and compacted in accordance with OPSS.PROV 501.

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

Topsoil thicknesses of 50 to 175 mm were encountered during the current field investigation and thicknesses up to 600 mm was documented in the historical Geocres report. It should be expected that the topsoil thicknesses will vary across the site and topsoil, organics and other deleterious materials should be removed prior to embankment construction in accordance with OPSS.PROV 206.

11.6.2 Embankment Stability

Preliminary stability analyses were carried out for the approach embankments under both static and seismic loading conditions. The stability analyses were carried out utilizing the

commercially available slope stability program SLOPE/W of the GeoStudio 2020 software package developed by Geo-Slope International. The Morgenstern-Price method of slices for limit equilibrium analyses was used. The following additional parameters and assumptions were used in the analysis:

- Estimated soil stratigraphy based on the nearest boreholes.
- Existing embankment side slope geometry.
- Grade raise of 1.3 and 1.7 m at the east and west abutments, respectively.
- Widening of the approach embankment to accommodate 4 new lanes (two in each direction) with 2H:1V side slopes.
- Seismic horizontal loading of 0.06g, equal to ½ of the site adjusted PGA value (Class D - 0.123g) for the seismic (pseudo-static) analysis.
- Target factors of safety as per the CHBDC were:
 - 1.5 – static long term (drained)
 - 1.3 – static short term (undrained)
 - 1.1 - seismic

Embankment widening and grade raises up to 1.7 m above the existing pavement elevation when constructed with sides slopes of 2H:1V (or flatter) and in accordance with the recommendations presented within this report should remain stable and meet the target factors of safety. Stability outputs have been included in Appendix I.

The stability of the embankment slopes with the proposed grade raise should be re-assessed when the detailed design geometries are determined. Ditch geometry should be determined such that the final design does not have an impact on embankment stability.

11.6.3 Embankment Settlement

An analysis was carried out to estimate the post-construction settlement of the foundation soils (both elastic and consolidation settlement) under the weight of the imposed embankment fill materials for the following cases:

- Proposed grade raise over existing highway embankment, and
- Embankment widening outside of existing highway embankment.

It is noted that the settlement calculations were carried out assuming a maximum grade raise of 1.7 m. The settlements are expected to be less for a lower grade raise. The settlement should be re-evaluated after the proposed detailed design GA has been provided.

In accordance with MTO's document "Embankment Settlement Criteria for Design" (March 2, 2010), the criteria adopted for embankment design at this site is as follows:

Table 11-4 Summary of MTO Settlement Criteria for Transitions

Distance from Abutment	Settlement Limits				Post Construction Settlement Period (years)
	0-20 m	20-50 m	50-75 m	>75 m	
Freeway	25 mm	50 mm	75 mm	100 mm	20

An assessment of the estimated short-term and time-dependent settlement was carried out using the multi-layer settlement analysis in Rocscience's Settle3 software. The design pre-consolidation pressure profile has been derived from the oedometer tests as well as correlations with the undrained shear strength and plasticity. Compression characteristics have been modelled using C_c , C_r , c_v and c_{vr} values from the current oedometer test results. The assumed engineering parameters used in the analyses are shown in Section 6.4.2.

A maximum settlement of approximately 110 mm is expected to occur at the site; this settlement is under the maximum point of fill near the toe of the existing embankment. Less than 25 mm settlement is expected to occur at the *existing* abutments under the proposed grade raise and widening. The MTO settlement criteria is satisfied at distances greater than 20 m from the abutment after a preload period of 4 months.

To satisfy the settlement criteria within 20 m of the *new* abutment in the areas of embankment widening, a preload period of 12 months would be required.

If the settlement durations presented are not considered acceptable, surcharge loading or lightweight backfill (LWF) could be considered. The use of LWF would require placement above the high-water level to avoid buoyancy concerns. Surcharging would have to be reviewed from a stability and lateral extent perspective with the proposed construction staging during the detailed design to avoid conflicts. The use of Prefabricated Vertical Drains (PVD) is not recommended due to the groundwater pressure within the deposits below the clay to clayey silt.

The magnitude of the compression for embankment constructed with granular materials is in the order of 0.5% of the newly placed embankment height and is expected to occur following fill placement.

See Section 11.1.2 for discussion on downdrag loading if the embankments are placed prior to construction of the deep foundations. The piles supporting the existing structure would have been subjected to downdrag loads due to the backfilling of the existing abutments.

11.6.4 Temporary Detour

A foundation investigation for a temporary detour embankment was not part of the current assignment. If temporary detour embankments are required, further assessment should be carried out and additional field investigations with recommendations should be provided.

12 CONSTRUCTION CONSIDERATIONS

12.1 Excavation

All excavation must be conducted in accordance with the requirements of the Occupational Health & Safety Act & Regulations (OHSA) for Construction Projects. The fill materials above the water table may be classified as Type 3 soil. The presence of alluvial deposits, cohesionless soils and fills below the groundwater level are classified as Type 4 soils. If an excavation penetrates more than one soil type, the entire excavation must be completed in accordance with the more stringent requirement. Excavations must not be planned and carried out in a manner that does not impact on the stability of existing adjacent structures.

Excavation for the structure must be carried out in accordance with OPSS 902 as amended by SP 109S12 and Nssp FOUN0003 and will be carried out through the existing embankment fill and into the underlying native deposits. Please refer to Section 12.3 for designer fill-in recommendations. Selection of the equipment and methodology to excavate and prepare the founding surface is the responsibility of the Contractor.

Material stockpiling is a temporary construction measure and the associated stability implications are the responsibility of the Contractor. It is recommended that stockpiling or surface surcharge should not be allowed on the embankment or side slopes. The selection and placement of construction equipment (such as cranes) are also the Contractor's responsibility.

At locations where there are space restrictions or where a slope has to be retained, the excavations will need to be carried out within a protection system. Further discussion on temporary protection systems (TPS) is presented in Section 12.2.

12.2 Temporary Protection Systems

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

The design of roadway protection is the responsibility of the Contractor. All protection systems should be designed by a licensed Professional Engineer experienced in such designs and retained by the Contractor. The design of the roadway protection system must incorporate traffic loading and surcharge loading due to construction equipment and operations. A suitable anchoring and/or bracing system may need to be incorporated into the temporary protection design to resist lateral earth pressure loadings.

Lateral earth pressure coefficients, under fully mobilized conditions, that can be used in design of the protection system installed through Granular A material are provided in Table 11-2. The lateral earth pressure coefficients for the existing soils are given below:

Existing sand fill:

$$\begin{aligned}\gamma &= 21 \text{ (kN/m}^3 \text{ bulk unit weight of soil, to be adjusted below water)} \\ K_A &= 0.33 \\ K_P &= 3.0\end{aligned}$$

Native sand to silty sand:

$$\begin{aligned}\gamma &= 20 \text{ (kN/m}^3 \text{ bulk unit weight of soil, to be adjusted below water)} \\ K_A &= 0.33 \\ K_P &= 3.0\end{aligned}$$

Native clay to clayey silt:

$$\begin{aligned}\gamma &= 18 \text{ (kN/m}^3 \text{ bulk unit weight of soil, to be adjusted below water)} \\ K_A &= 0.39 \\ K_P &= 2.5\end{aligned}$$

It is recommended that the protection systems near the structure foundations should be left in place and cut off in accordance with OPSS.PROV 539. The use of vibration during TPS installation and removal could result in settlement and must be carried out cautiously. An NTC has been provided in Appendix J.

12.3 Surface and Groundwater Control

The depth of excavation will extend below the groundwater level observed at the time of investigation. Water from surface flow and/or groundwater must be diverted away from excavation(s) at all times. Groundwater perched within the embankment and surface water will tend to seep into and accumulate in excavations. The Contractor must be prepared to control the groundwater and surface water at the site. The water level must be lowered below the base of the excavation to allow construction in the dry.

The design of dewatering systems is the responsibility of the Contractor. The Contract Documents must alert the Contractor to this responsibility and to design the dewatering systems in accordance with SP FOUN0003 which amends OPSS 902. A preconstruction survey is not required, thus Designer Fill-In ** in SP FOUN0003 should be "N/A".

The dewatering system is to be designed in accordance with OPSS.PROV 517 and SP517F01. It is recommended that the design Engineer and design-checking Engineer have a minimum of 5 years of experience in designing systems of similar nature and scope to the required work. The Designer Fill-In ***** in SP517F01 Table A should be "Yes". A preconstruction survey is not required, thus Designer Fill-In ***** in Table A should be "N/A".

The water level will fluctuate and the minimum groundwater elevation for the site at the time of the proposed culvert installation should be taken as the expected high water level defined in SP517F01 and SP FOUN0003.

The construction of a cofferdam will be required to isolate the excavation if an in-water pier is to be constructed. Cofferdams may also be required for the abutment excavations. It is

recommended that excavation be carried out within a water tight enclosure. This will require installation of sheet piles to cut off the flow. A sheet piled cofferdam can be designed following the recommendations provided in Sections 12.1 and 12.2. The use of vibration during the removal of sheet pile cofferdams could result in settlement and must be carried out cautiously. It may be prudent to cut off the sheet piles in accordance with OPSS 539.

Excavation below the groundwater level without prior dewatering is not recommended since the inflow of groundwater will make it difficult to maintain a dry, sound base on which to work. Disturbance of the subgrade soils is considered to be a significant risk without proper consideration of groundwater lowering. The groundwater level should be lowered to 0.5 m below the planned base of excavation for each stage of excavation. A NTC has been provided in Appendix J.

Further assessment of the dewatering requirements and the need for a Permit to take Water (PTTW) should be carried out by specialists experienced in this field.

12.4 Scour Protection and Erosion Control

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

Scour and erosion protection should be provided along the banks of the river and ditches. Design of the scour and erosion protection measures must consider hydrologic and hydraulic concerns and should be carried out by specialists experienced in this field. Typically, rock protection should be provided over all earth surfaces subjected to flowing water in accordance with OPSS 511.

Slope protection and drainage measures will be required to ensure the long-term surficial stability of the embankment slopes. A vegetation cover should be established on all exposed earth surfaces to protect against surficial erosion in general accordance with OPSS.PROV 804.

13 FUTURE WORK

During detailed design, the quantity and location of existing borehole and CPT records should be reviewed and additional investigations should be carried out, as required, to meet the current MTO standards for the proposed detailed design.

Additionally, during detailed design an updated assessment of the settlement and stability of permanent and detour embankments and/or surcharged embankments should be carried out.

14 CONSTRUCTION CONCERNS

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

- Although not encountered during drilling, buried obstructions (e.g. debris from original bridge construction) may be encountered during construction and interfere with excavations and installation of temporary protection systems. A NTC has been provided in Appendix J.

- The existing bridge is supported on steel piles and includes battered piles. The potential for conflict between the existing piles and piles or tie-backs for temporary protection systems must be considered.
- Excavation below the groundwater level and the water level of the Napanee River is expected to be required. The stability of the excavation slopes and the base of the excavations due to seepage forces and upward gradients must be considered in the design and selection of excavation protection and dewatering systems.
- Consideration will have to be given to the proximity of the nearby utilities and special attention will need to be taken into account during construction to avoid any damage to the utilities.
- The proposed construction includes pile driving, excavations and placement of embankment fill material in close proximity to the existing bridge structure. The existing bridge should be monitored for movement throughout construction.
- Water levels will fluctuate. A suitable dewatering / unwatering system must be employed to enable control of the groundwater seepage. The Contractor should be prepared to take appropriate measures to construct in a dry and stable environment.
- The Contractor's selection of construction equipment and methodology must include assessment of the capability of the existing soils to support the proposed construction equipment and supplies.

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

15 CLOSURE

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

Thurber Engineering Ltd.
Report Prepared By:



Stephen Peters, P.Eng.
Geotechnical Engineer



Paul Carnaffan, P.Eng.
Branch Manager, Principal
Senior Geotechnical Engineer



Dr. P.K. Chatterji, P.Eng.
Review Principal
Senior Geotechnical Engineer

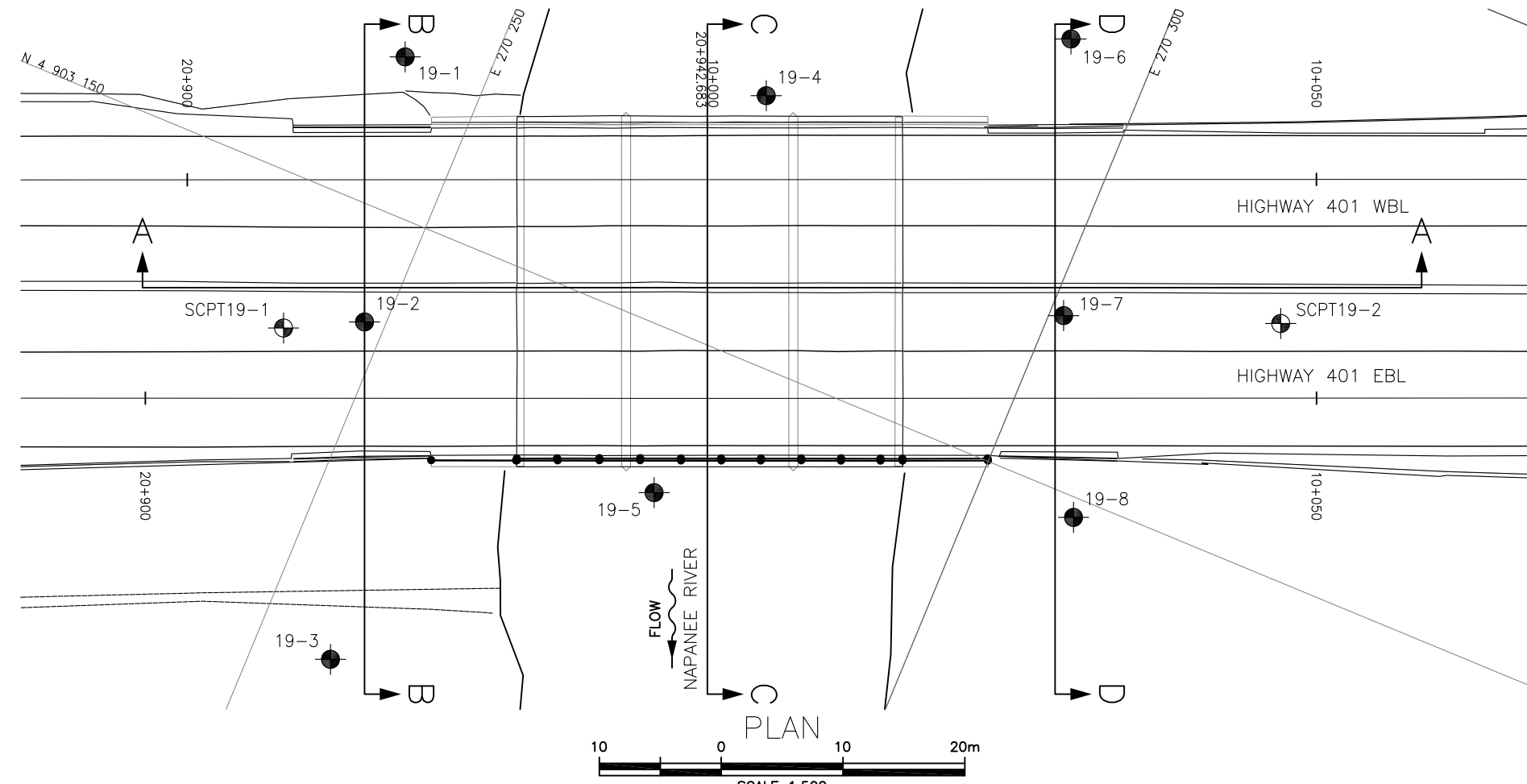
References

¹ Boulanger, R. W. and Idriss, I. M. (2007). Evaluation of cyclic softening in silts and clays, ASCE, Journal of Geotechnical and Geoenvironmental Engineering, 133(6), 641-652.

² Boulanger, R. W., and Idriss, I. M. (2014). CPT and SPT based liquefaction triggering procedures, Report No. UCD/CGM-14/01, Center for Geotechnical Modeling, Department of Civil and Environmental Engineering, University of California, Davis, CA, 134 pp.

Appendix A.

Drawings

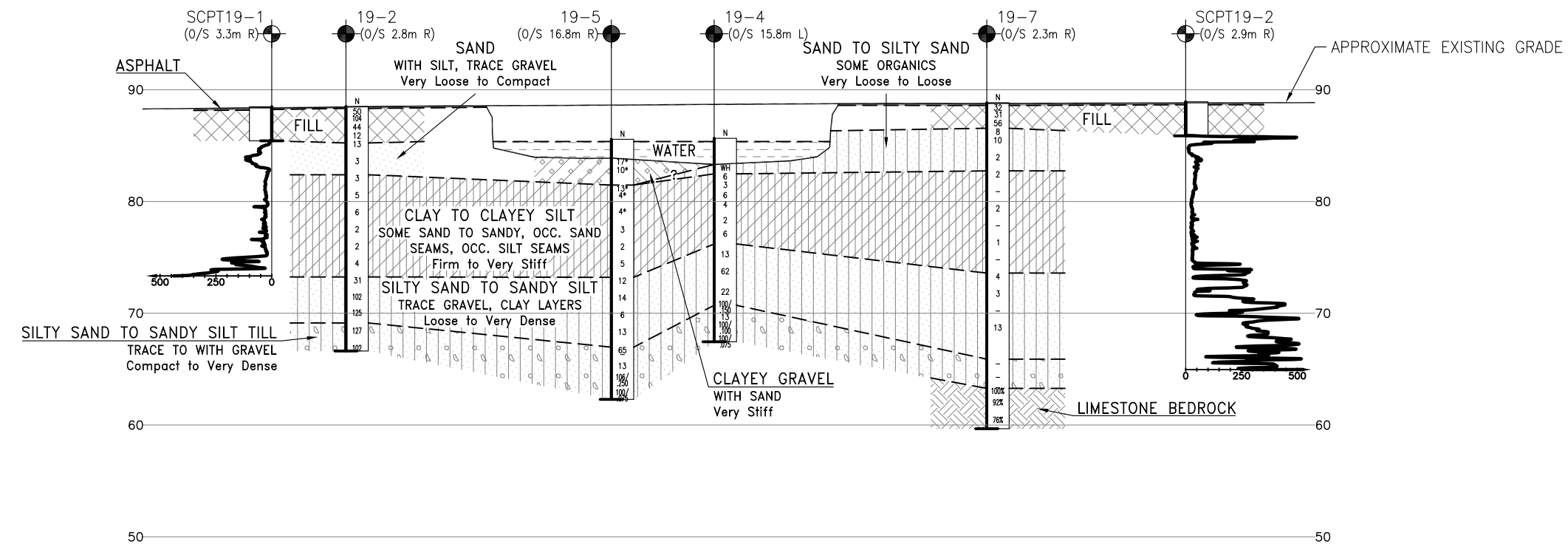
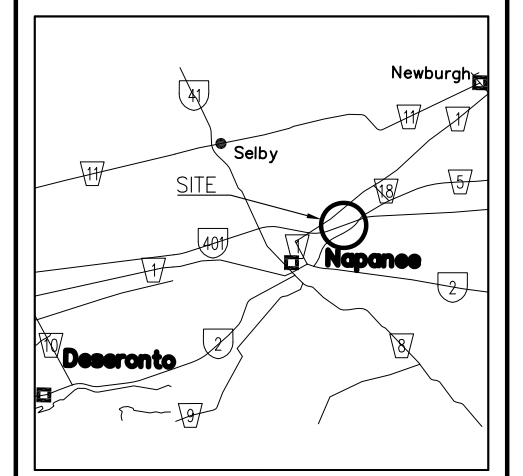


METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

LICENSED PROFESSIONAL ENGINEER
P. K. Chatterji
P. K. CHATTERJI
March 24/2021
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
S. Peters
S. PETERS
100194478
Mar 25/24
PROVINCE OF ONTARIO

CONT No GWP No 4040-20-00	
HIGHWAY 401 NAPANEE RIVER BRIDGE BOREHOLE LOCATIONS AND SOIL STRATA	SHEET



LEGEND

	Borehole
	Borehole and Cone
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60' Cone, 475J/blow)
PH	Pressure, Hydraulic
	Water Level
	Head Artesian Water
	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
19-1	87.7	4 903 162.4	270 243.1
19-2	88.5	4 903 141.0	270 248.3
19-3	86.3	4 903 114.3	270 256.3
19-4	85.6	4 903 170.8	270 271.7
19-5	85.6	4 903 137.1	270 275.6
19-7	88.8	4 903 163.4	270 301.1
19-8	86.5	4 903 148.4	270 308.2
SCPT19-1	88.5	4 903 138.0	270 242.4
SCPT19-2	88.9	4 903 169.6	270 317.9

- NOTES-**
- 1) The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
 - 2) This drawing is for subsurface information only. Surface details and features are for conceptual illustration.
 - 3) Coordinate system is MTM NAD 83 Zone 9.

GEOCREs No. 31C-302

REVISIONS	DATE	BY	DESCRIPTION

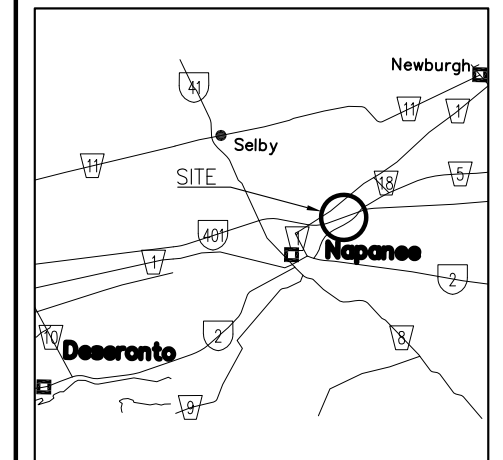
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METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT No
GWP No 4040-20-00

HIGHWAY 401
NAPANEE RIVER
BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET



KEYPLAN

LEGEND

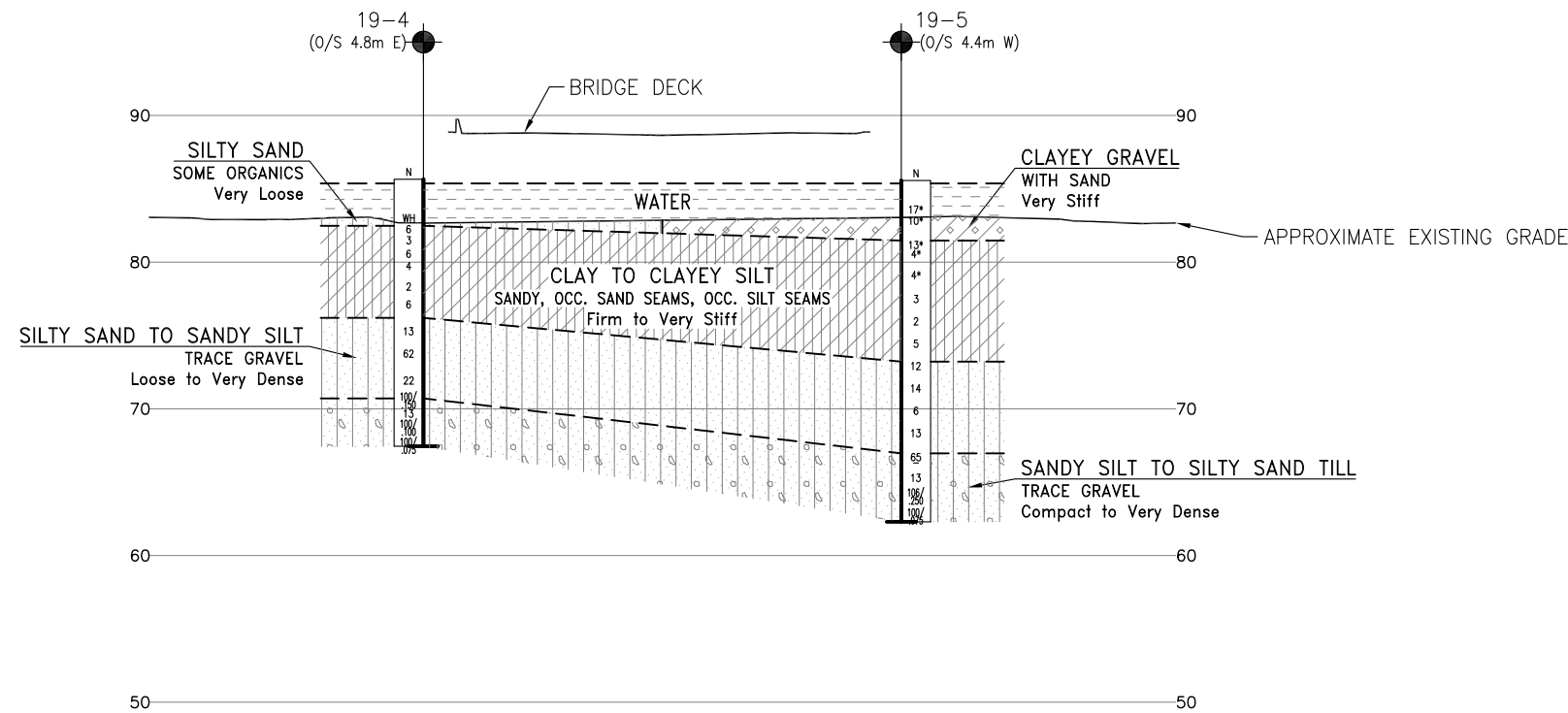
- Borehole
- ⊕ Borehole and Cone
- N Blows /0.3m (Std Pen Test, 475J/blow)
- CONE Blows /0.3m (60' Cone, 475J/blow)
- PH Pressure, Hydraulic
- W Water Level
- HA Head Artesian Water
- PZ Piezometer
- 90% Rock Quality Designation (RQD)
- A/R Auger Refusal

NO	ELEVATION	NORTHING	EASTING
19-1	87.7	4 903 162.4	270 243.1
19-2	88.5	4 903 141.0	270 248.3
19-3	86.3	4 903 114.3	270 256.3
19-4	85.6	4 903 170.8	270 271.7
19-5	85.6	4 903 137.1	270 275.6
19-7	88.8	4 903 163.4	270 301.1
19-8	86.5	4 903 148.4	270 308.2
SCPT19-1	88.5	4 903 138.0	270 242.4
SCPT19-2	88.9	4 903 169.6	270 317.9

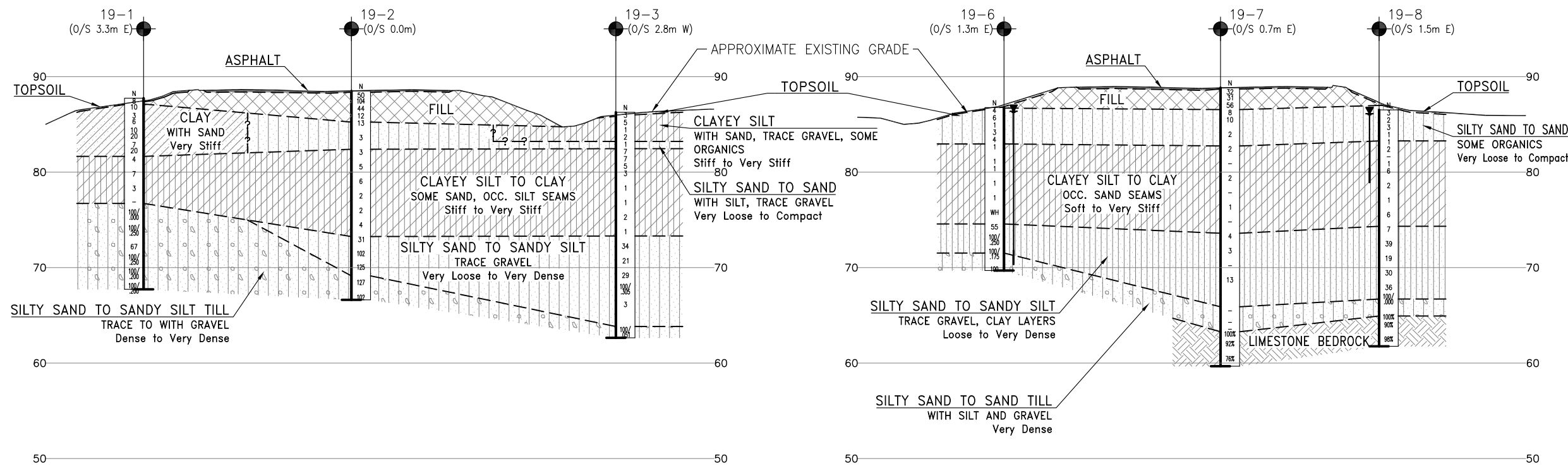
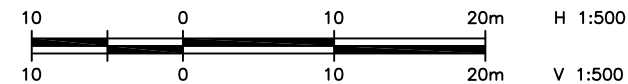
-NOTES-

- 1) The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- 2) This drawing is for subsurface information only. Surface details and features are for conceptual illustration.
- 3) Coordinate system is MTM NAD 83 Zone 9.

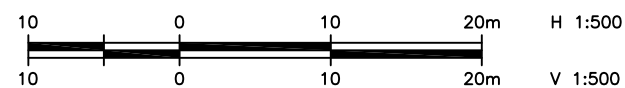
GEOCREs No. 31C-302



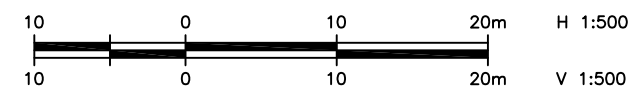
SECTION C-C AT PIER



SECTION B-B AT WEST ABUTMENT

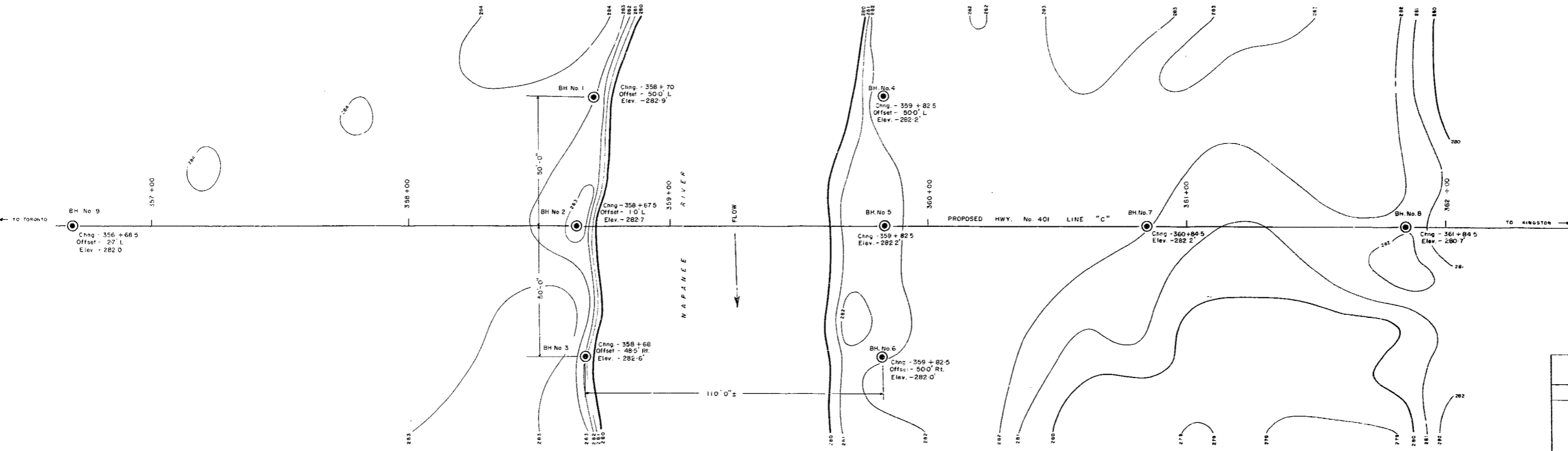
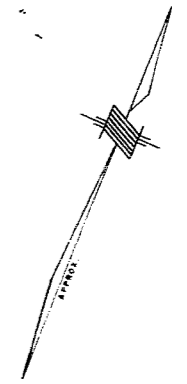


SECTION D-D AT EAST ABUTMENT

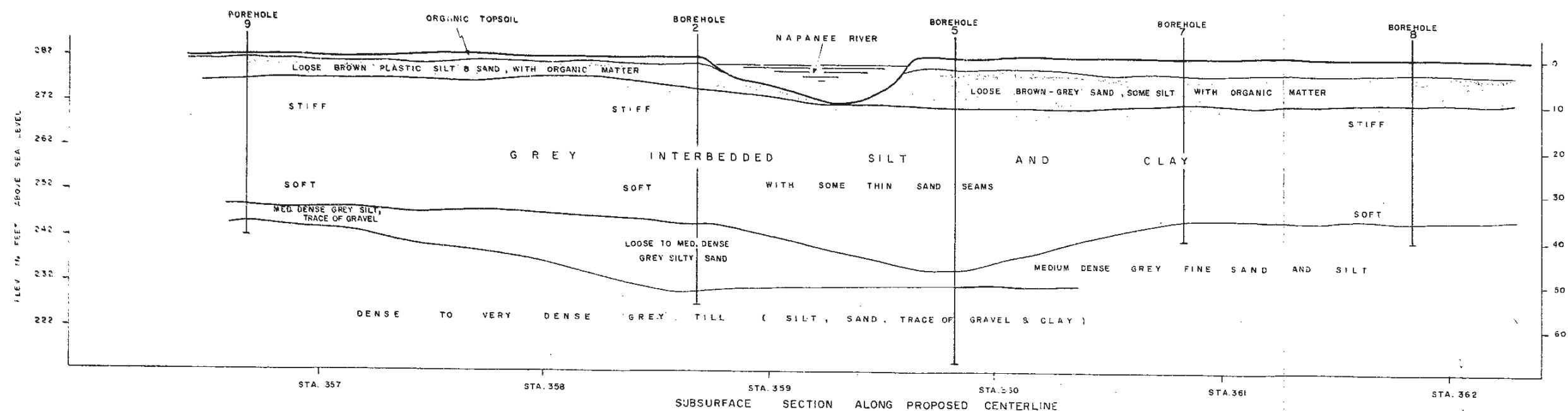


REVISIONS	DATE	BY	DESCRIPTION

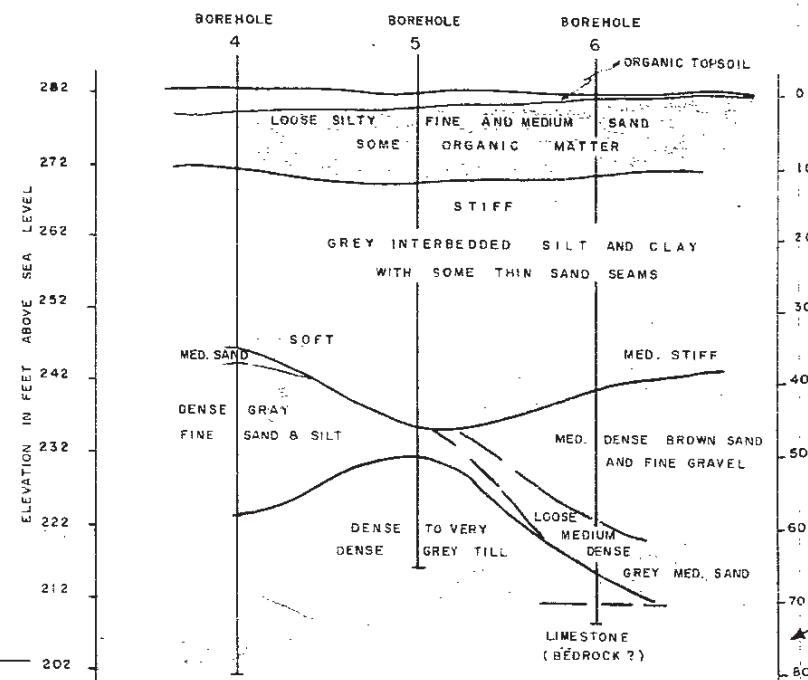
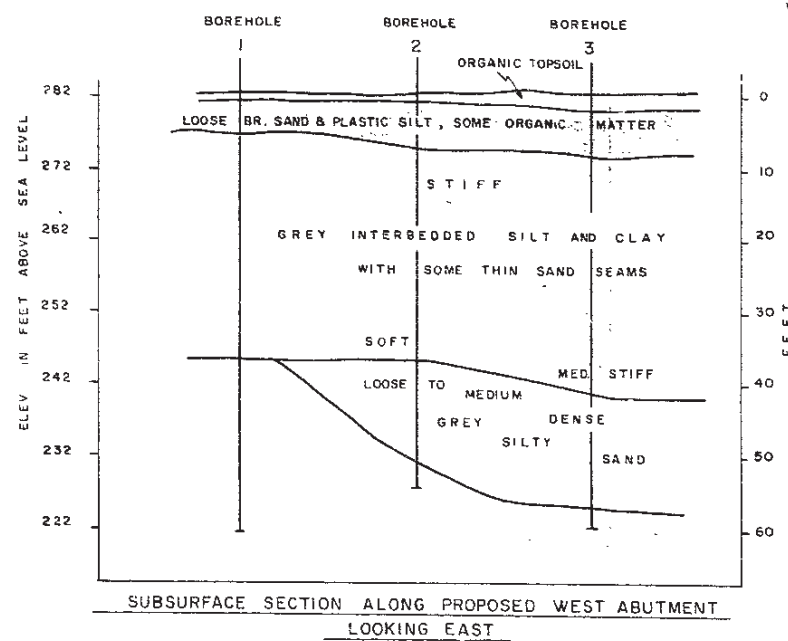
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HUNTING TECHNICAL & EXPLORATION SERVICES LTD TORONTO		
DEPARTMENT OF HIGHWAYS - ONTARIO		
PLAN SHOWING LOCATION OF BOREHOLES FOR PROPOSED CROSSING AT NAPANEE RIVER AND THE KING'S HIGHWAY No. 401 PROPOSED LINE "C" BRIDGE SITE		
SCALE - 1 inch = 20 ft.	Drawn by - C.F.B.	Date - May 1958
Reference - Plan - E 3374-1		

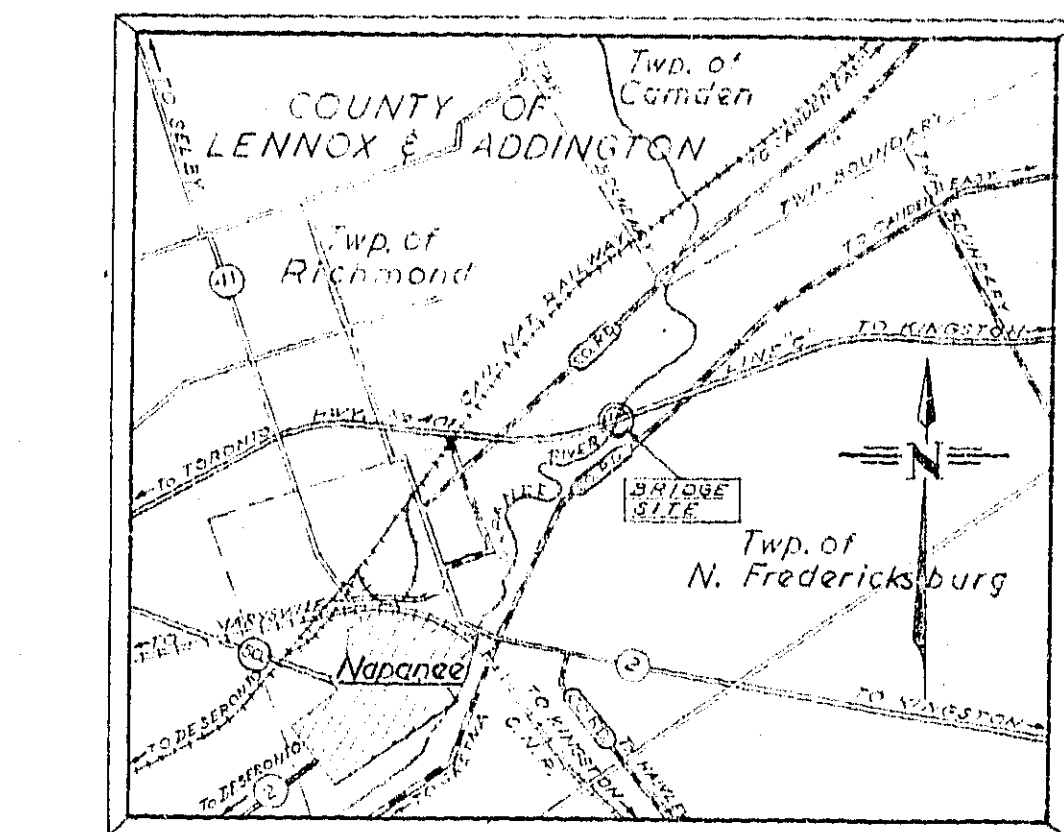
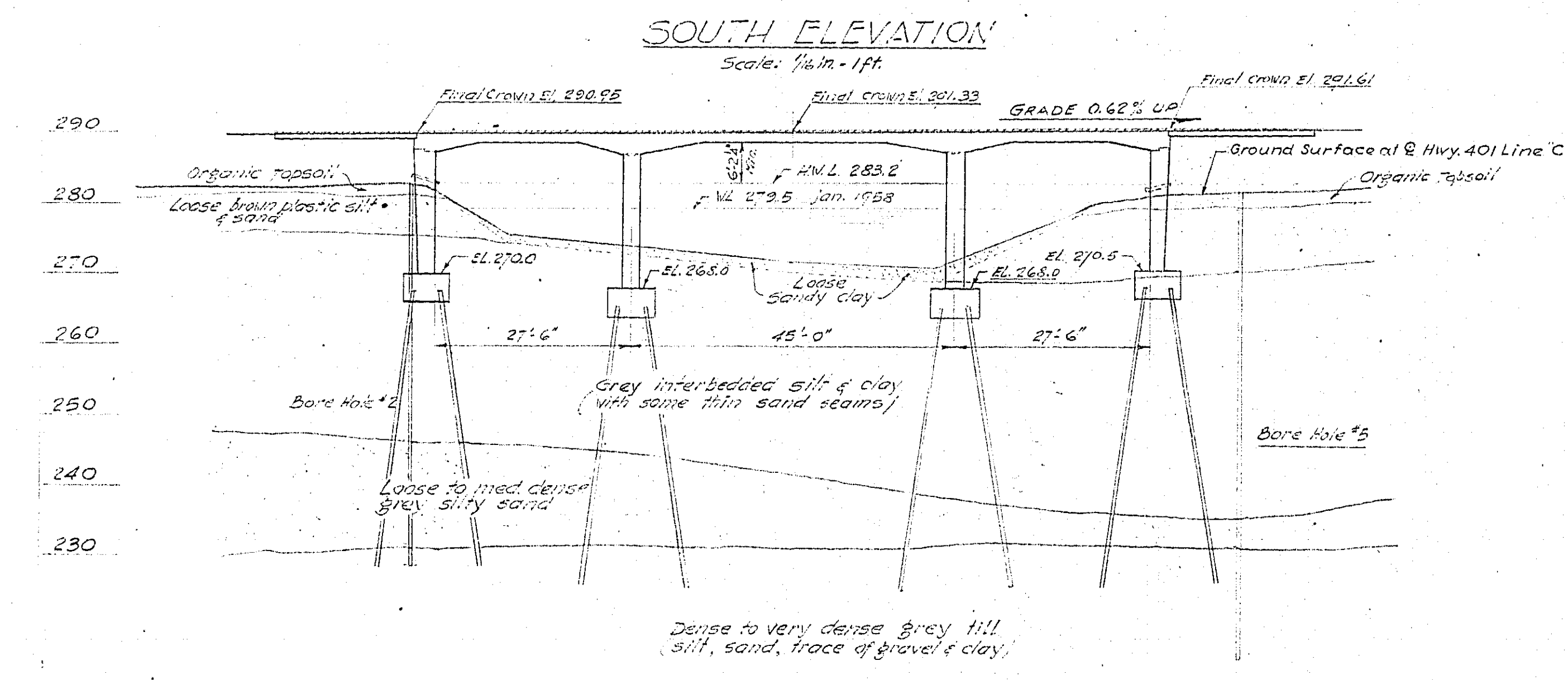
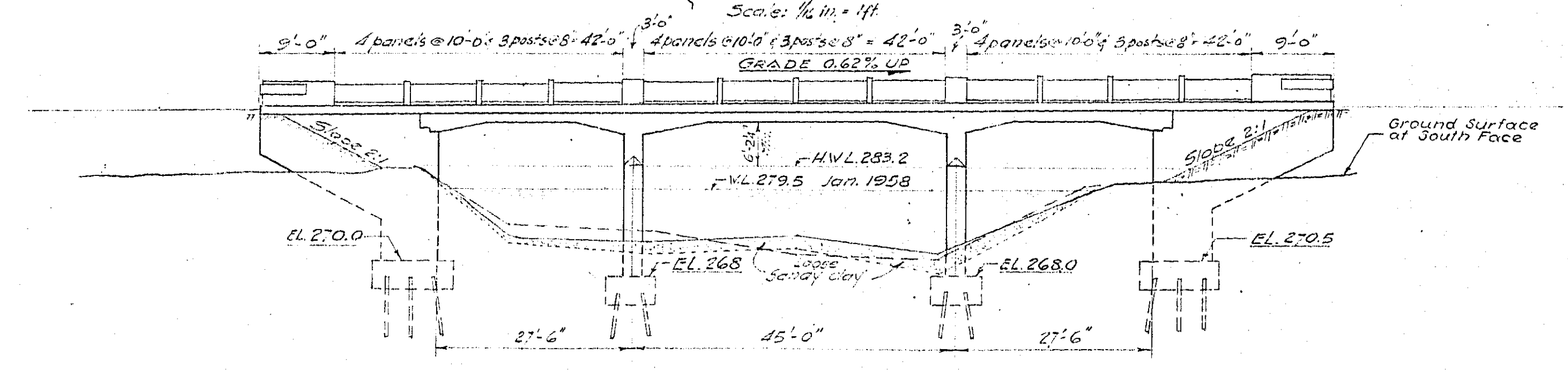
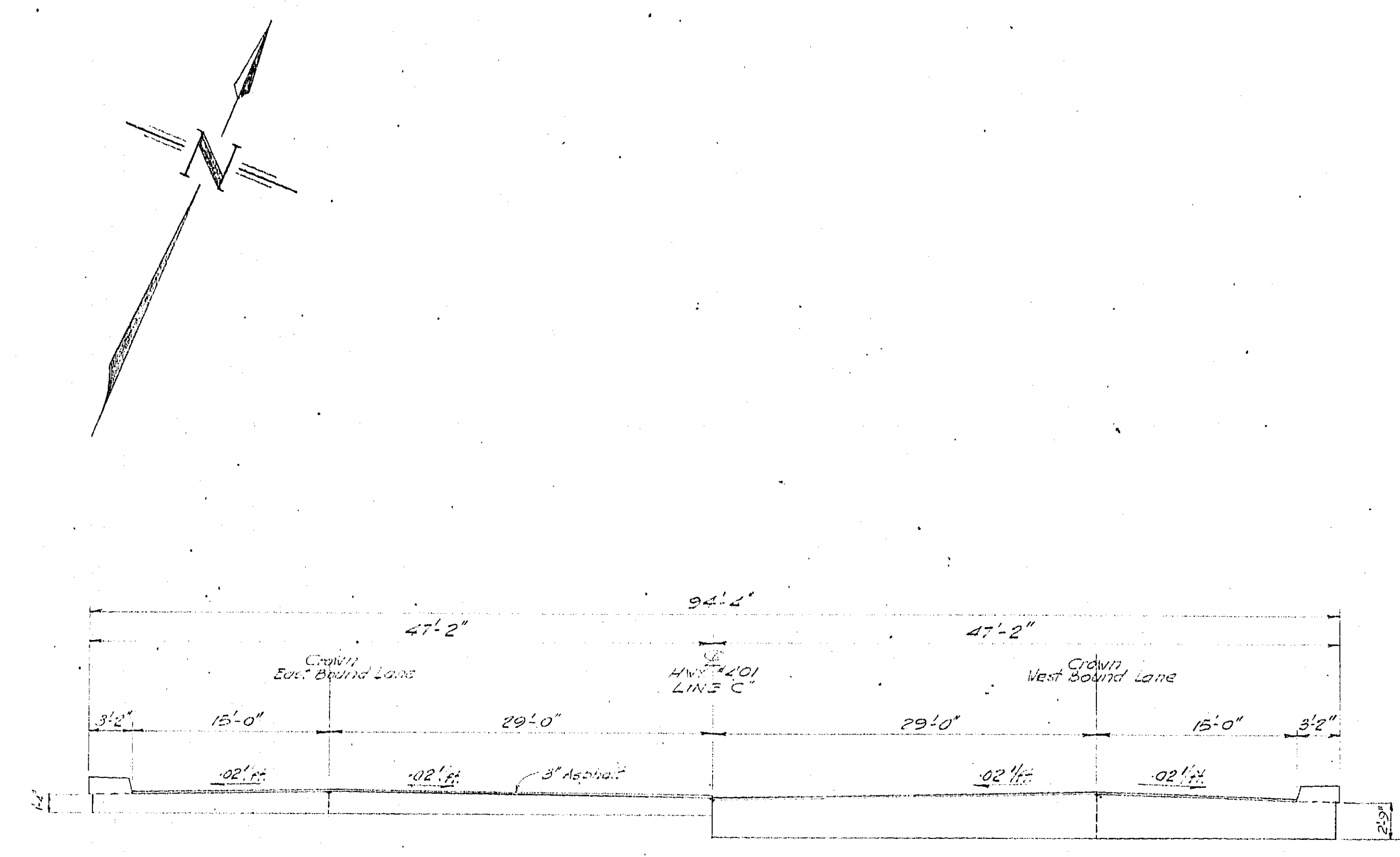
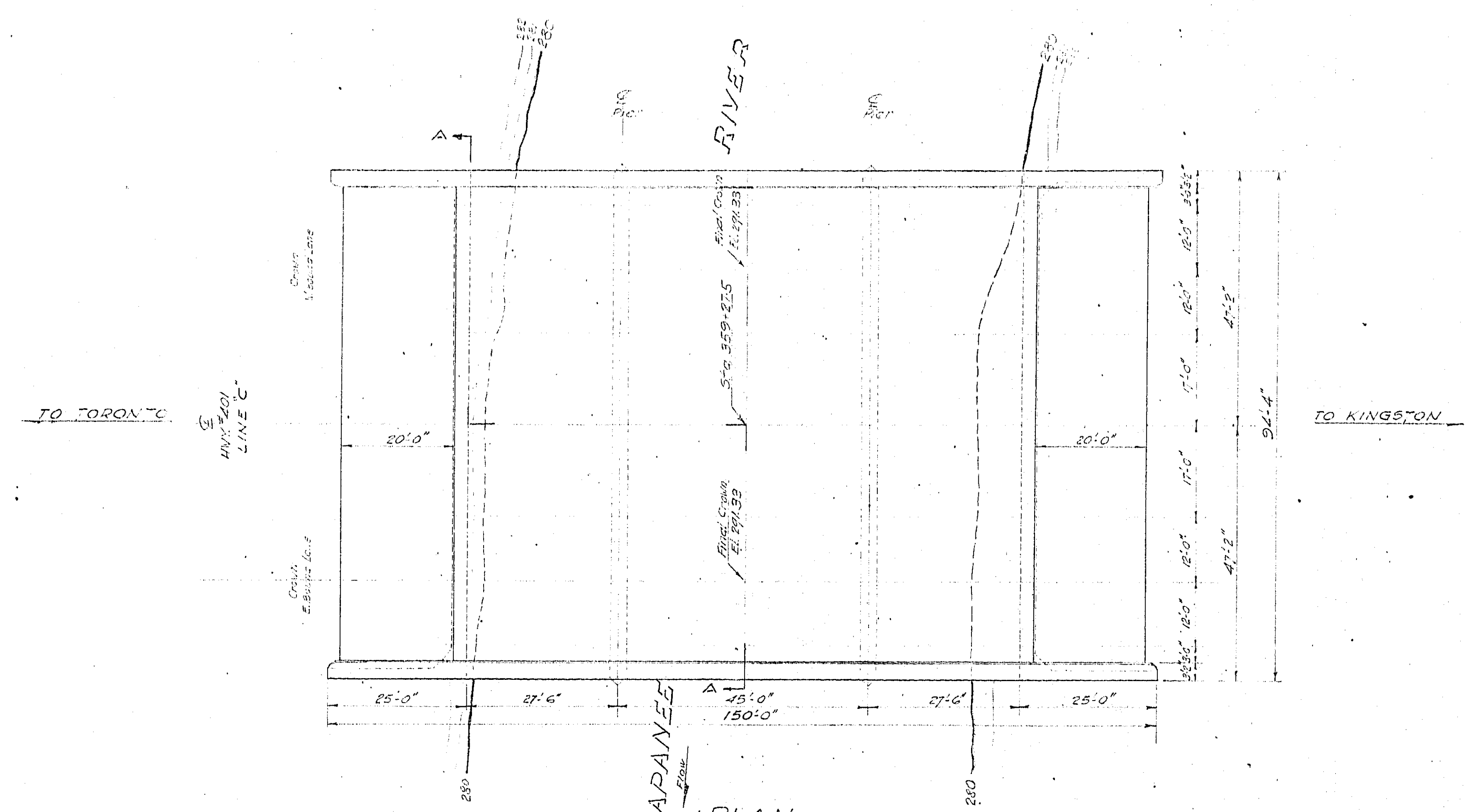


Scale -
 hor. - 1" = 40'
 vert. - 1" = 20'



PROPOSED CROSSING OF NAPANEE RIVER
 AND KING'S HIGHWAY No. 401

SUBSURFACE SECTION ALONG PROPOSED EAST ABUTMENT LOOKING EAST



NOTE: The Grade as per profile No. 3174-2 raised by .50 in order to get min 6.0 ft vertical clearance above H.W.L.

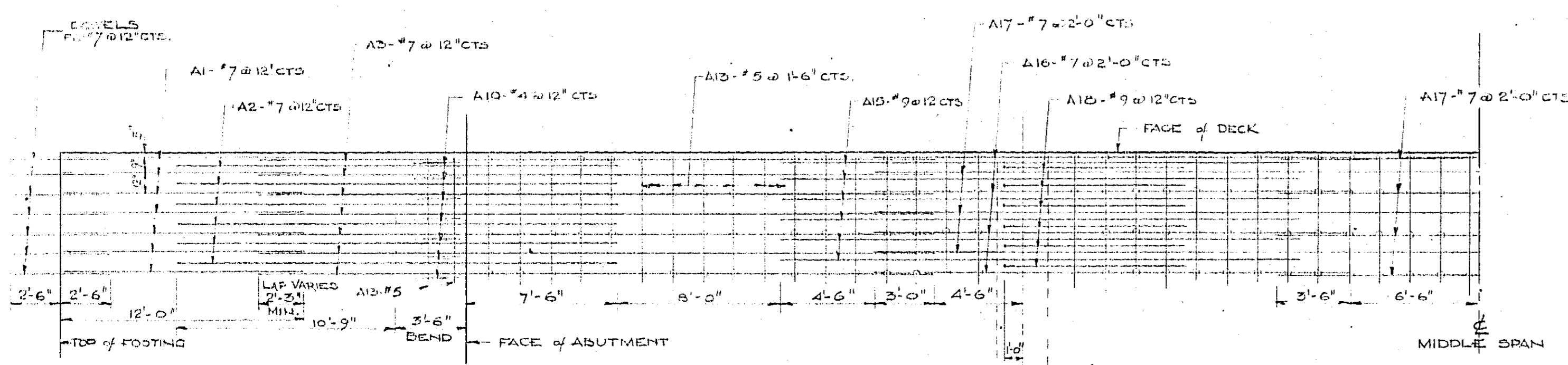
WP 138-58

DEPARTMENT OF HIGHWAYS-ONTARIO		BRIDGE OFFICE-TORONTO	
FREDERICKSBURG N TWP BRIDGE #1			
(NAPANEE RIVER)			
THE KING'S HIGHWAY No. 401		DIST. No. 5	
CO. LENNOX & ADDINGTON			
TWP. RICHMOND & N. FREDERICKSBURG LOT 28 (cont'd) of 125 (cont'd) CON. II & V			
PRELIMINARY PLAN			
APPROVED			
BRIDGE ENGINEER		DESIGN ENGINEER	
DESIGN	A.R.	CHECK	B.F.
DRAWING	A.R.	CHECK	
TRACING		CHECK	
DATE	NOVEMBER 12, 1958	LOADING	H20-S16
		DRAWING NUMBER	D4242-P1

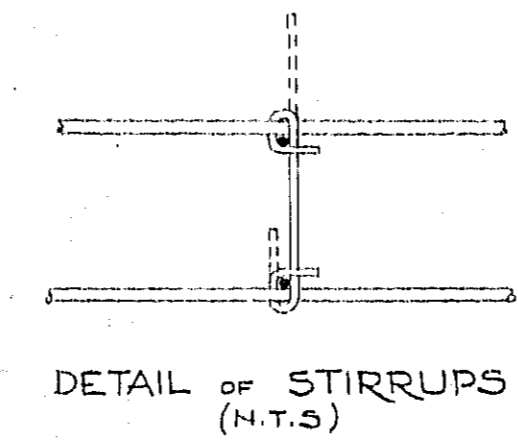
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NO.	FOR	DATE

REVISIONS	DATE	BY	DESCRIPTION

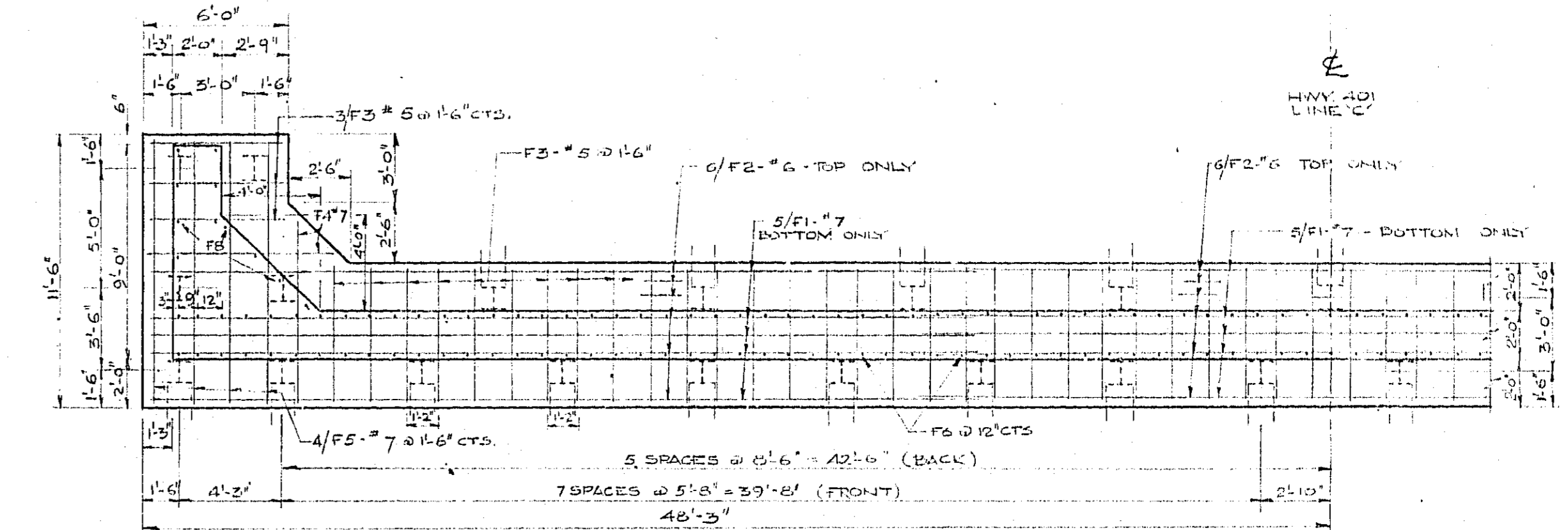
TWP #17-62-P1-A



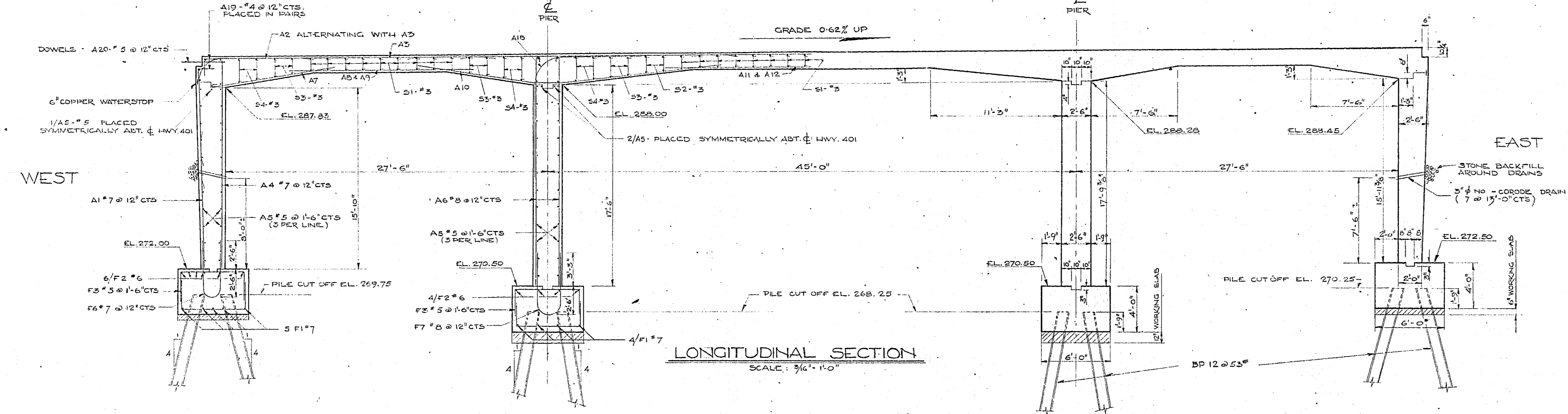
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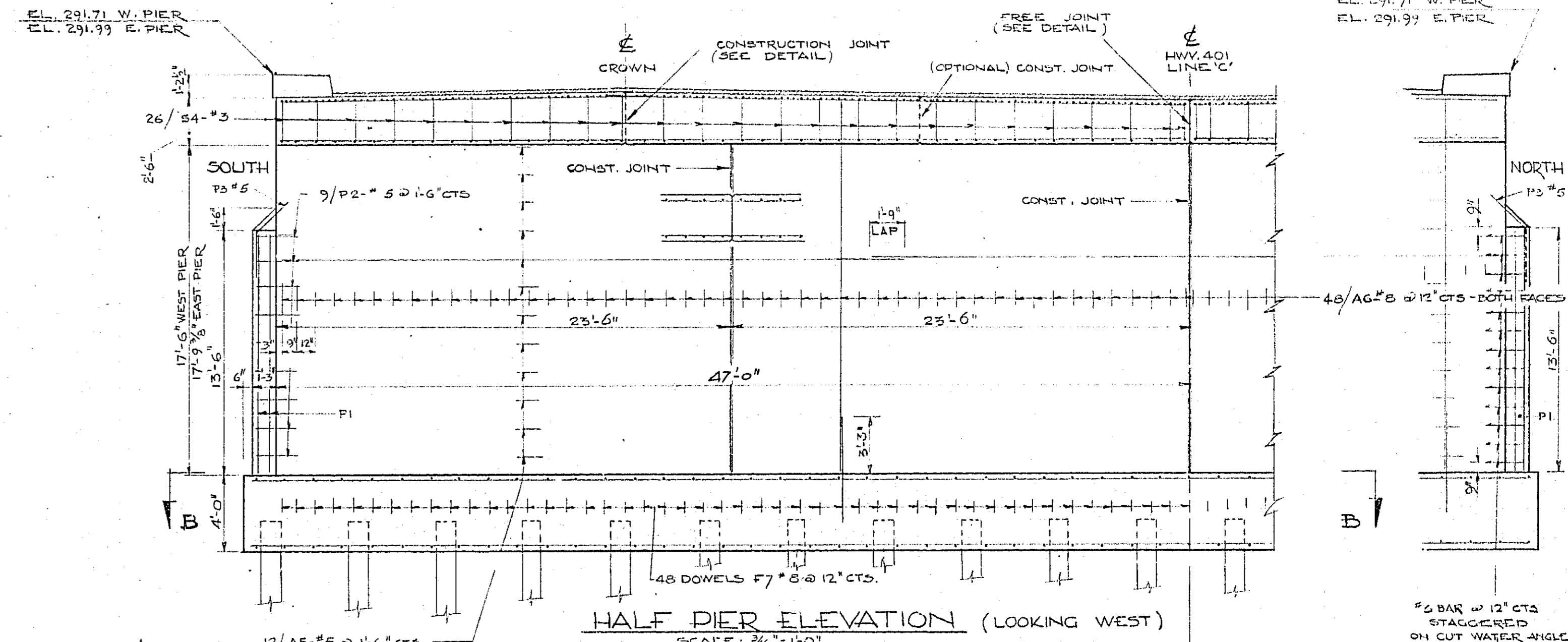
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(N.T.S.)



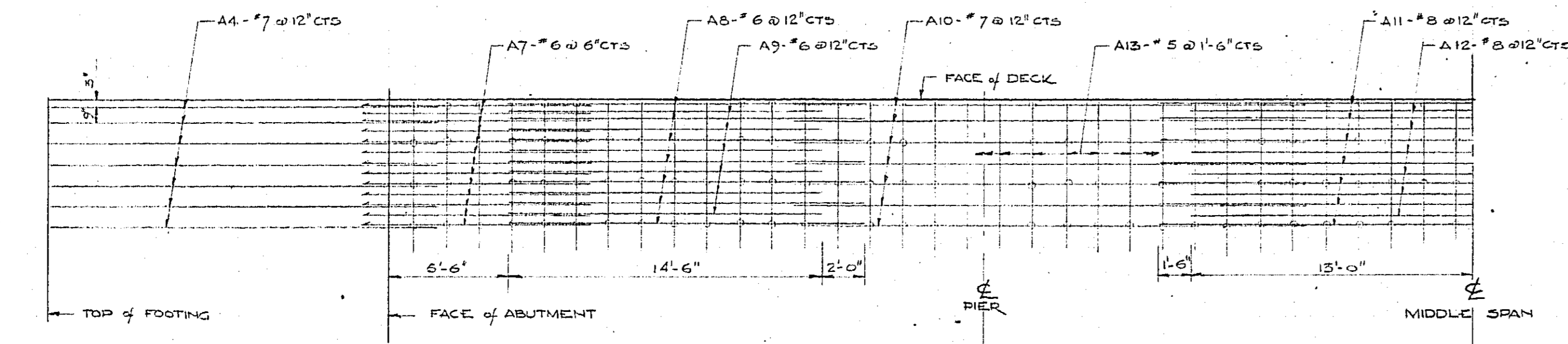
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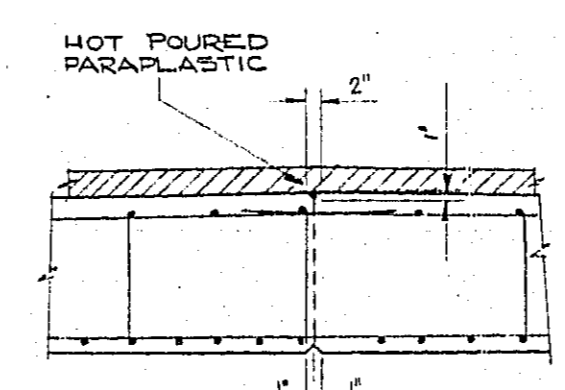
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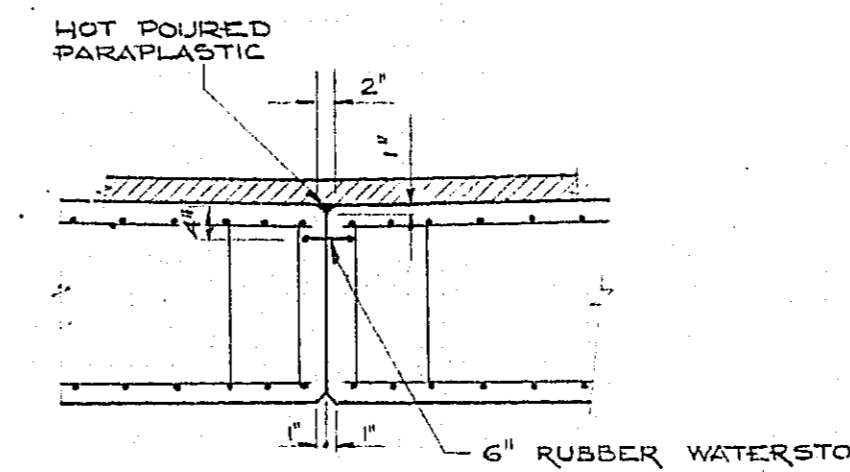
HALF PIER ELEVATION (LOOKING WEST)
SCALE: 3/8" = 1'-0"



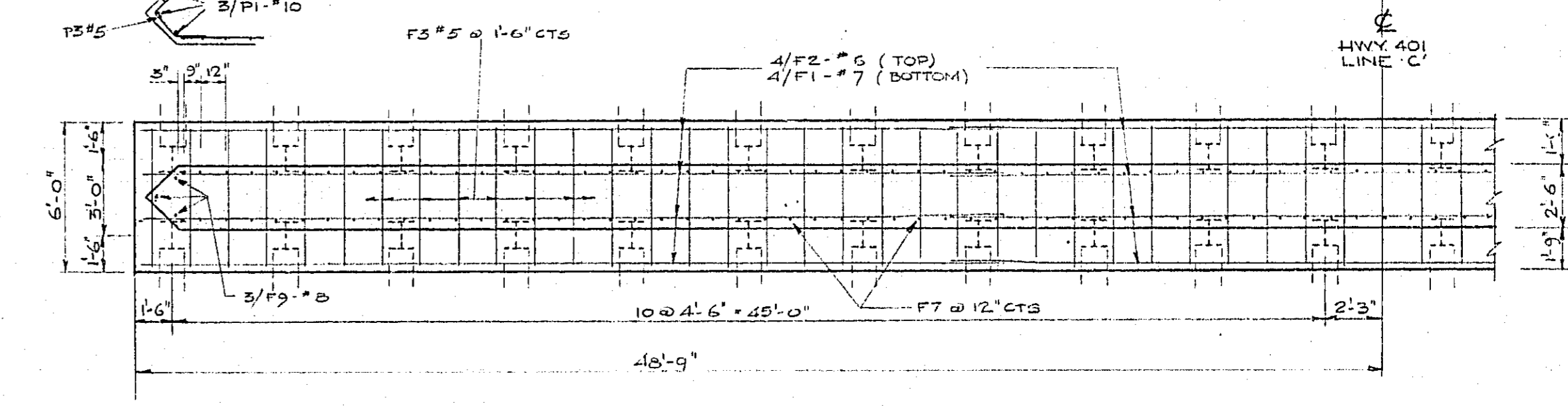
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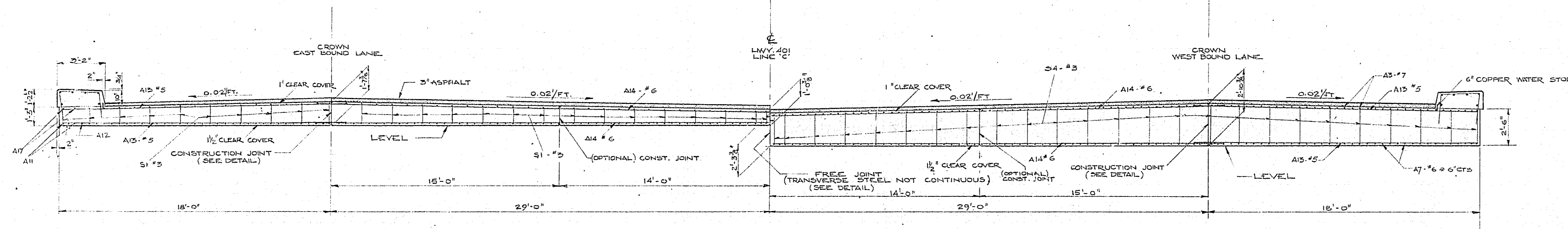
DETAIL OF CONSTRUCTION JOINT
N.T.S.



DETAIL OF FREE JOINT
N.T.S.

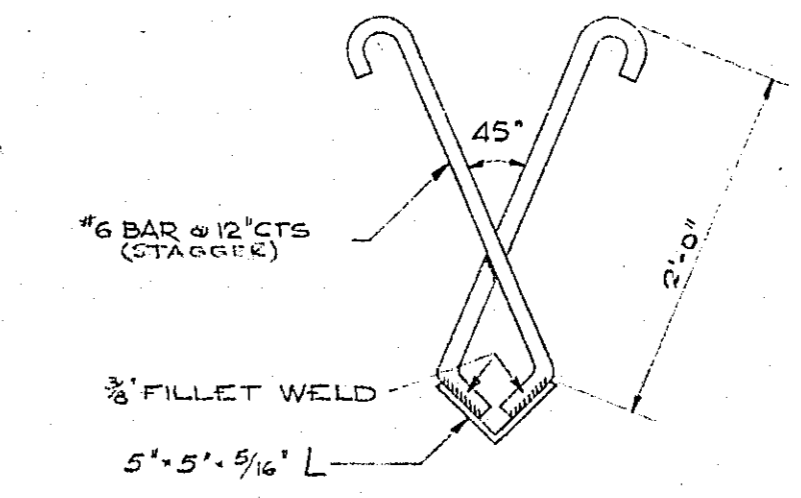


SECTION B-B
SCALE: 3/8" = 1'-0"



SECTION @ MIDDLE SPAN
SCALE: 1/2" = 1'-0"

SECTION @ ABUTMENT
SCALE: 1/2" = 1'-0"



CUT WATER DETAIL
SCALE: 1" = 1'-0"

REQUIRED: 2'-15'-6" LONG
SUPPLIED BY GENERAL CONTRACTOR

NO.	DATE	DESCRIPTION
1	7.2.59	REVISED
2	7.14.59	REVISED

REVISIONS	DATE	BY	DESCRIPTION
2	8.6.59	ALC	REV. AS CONST.

WP 138-58

DEPARTMENT OF HIGHWAYS - ONTARIO
BRIDGE OFFICE - TORONTO

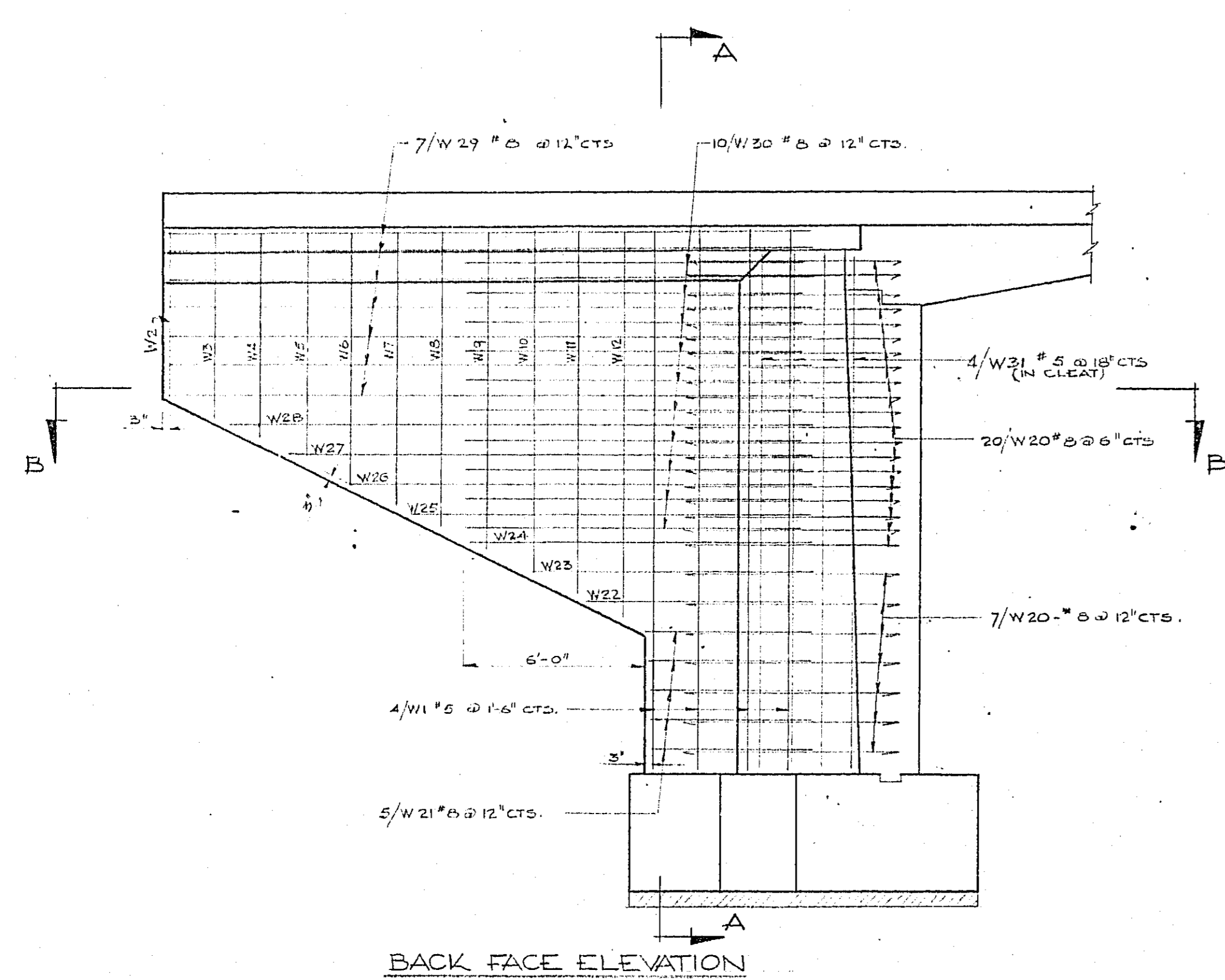
FREDERICKSBURG N. TWP. BRIDGE #1
(NAPANEE RIVER)

THE KING'S HIGHWAY No. 401 DIST. No. 05
CO. LENNOX & ADDINGTON
TWP. RICHMOND & FREDERICKSBURG LOT 25 (CON. 1) & 25 (CON. 2) & 25 (CON. 3) & 25 (CON. 4)

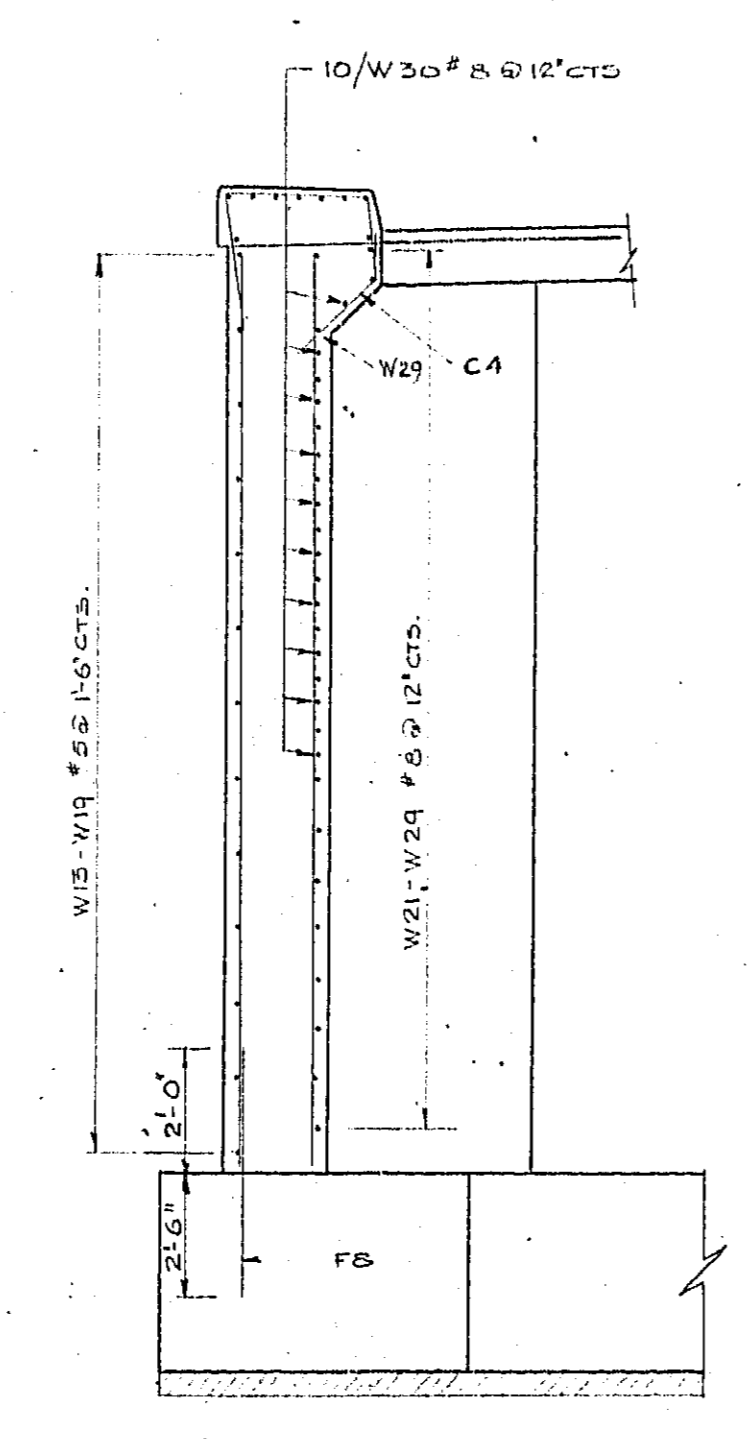
PIER, DECK, LONGITUDINAL SECTION & FTNG. PLAN

APPROVED: *[Signature]*
BRIDGE ENGINEER

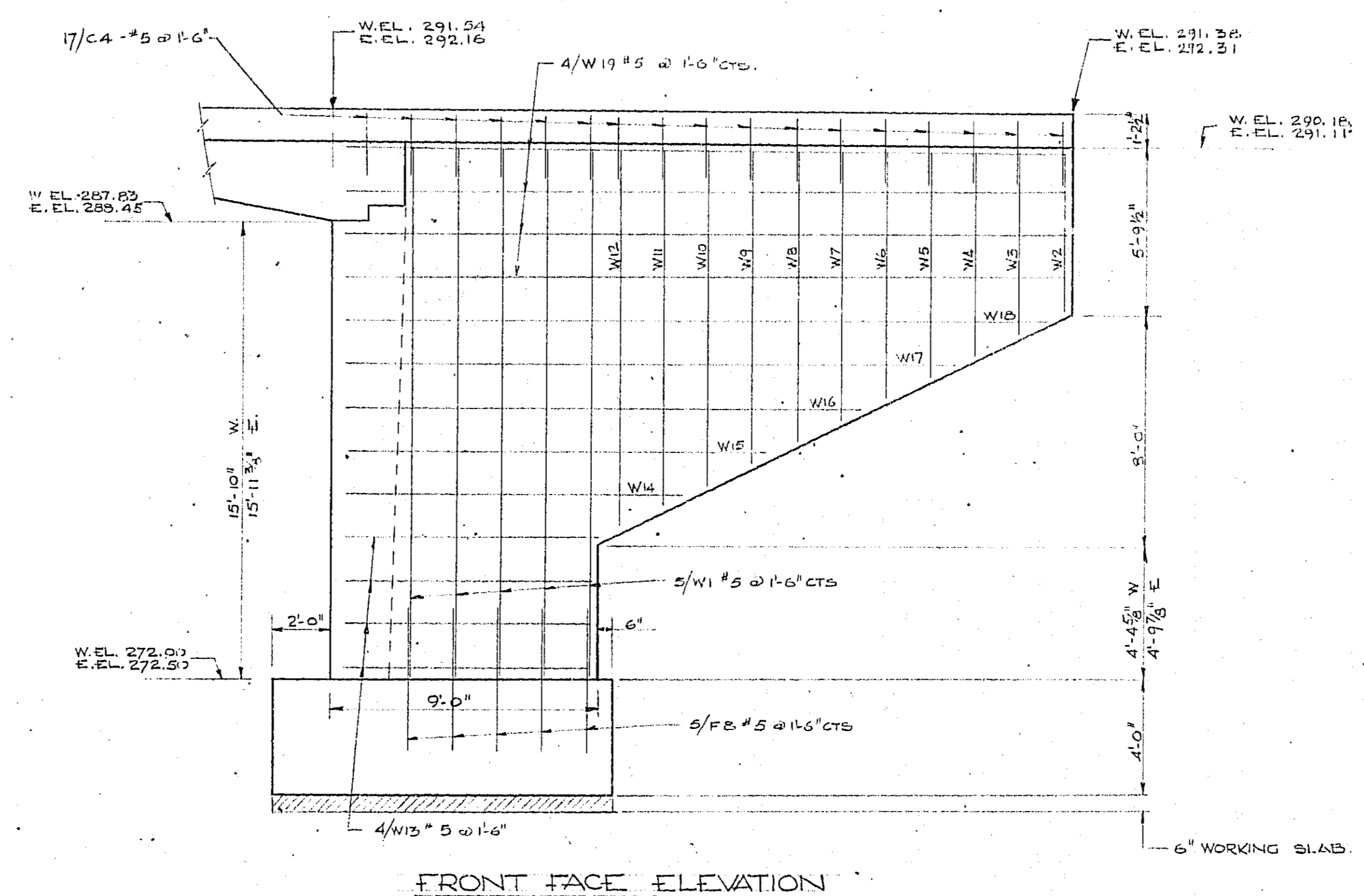
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DRAWING: W.V. CHECK: W.B.
TRACING: H.Z. CHECK: W.B. LOADING: H20
DATE: JULY 1959 NUMBER: D-4222



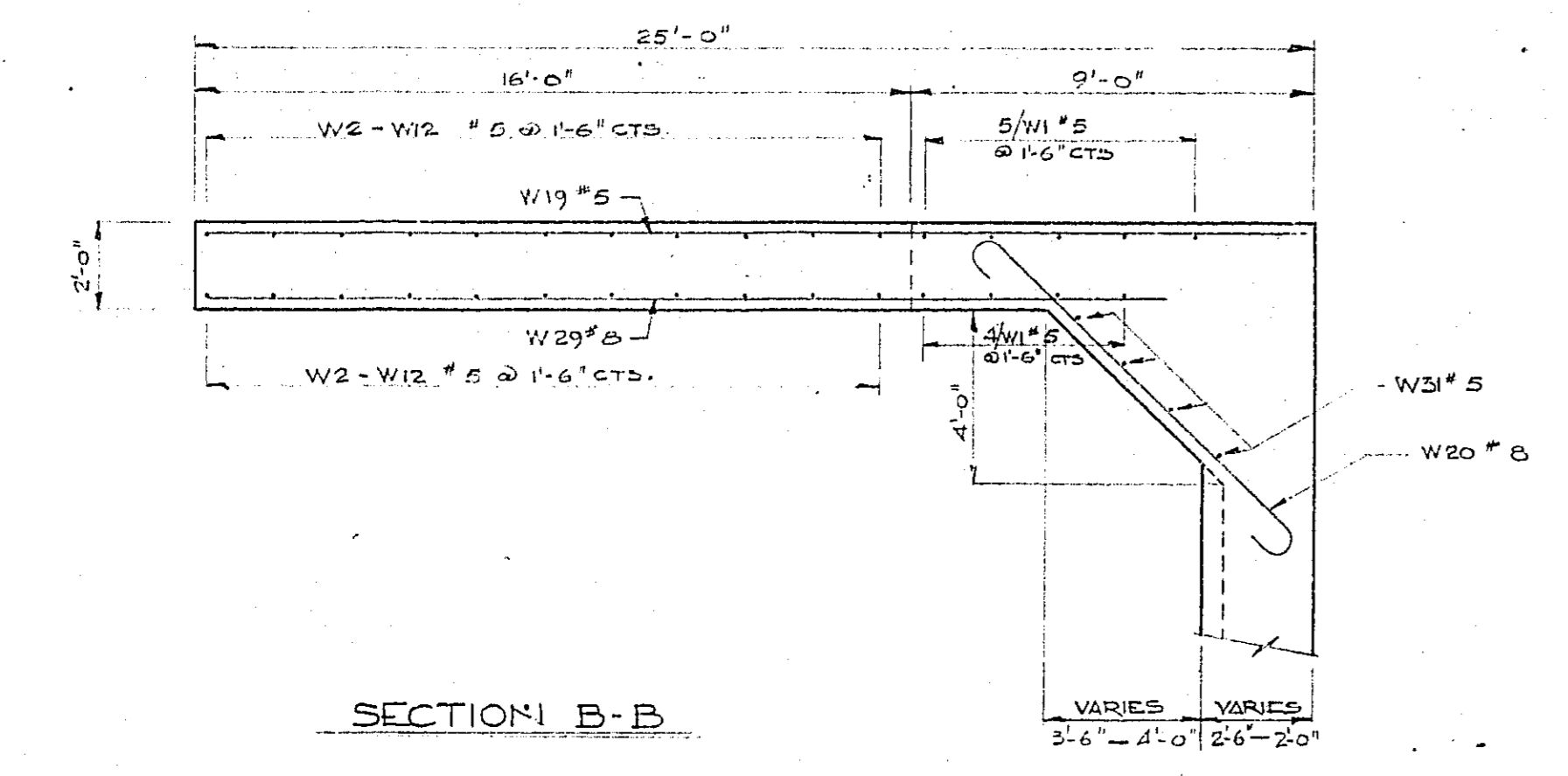
BACK FACE ELEVATION



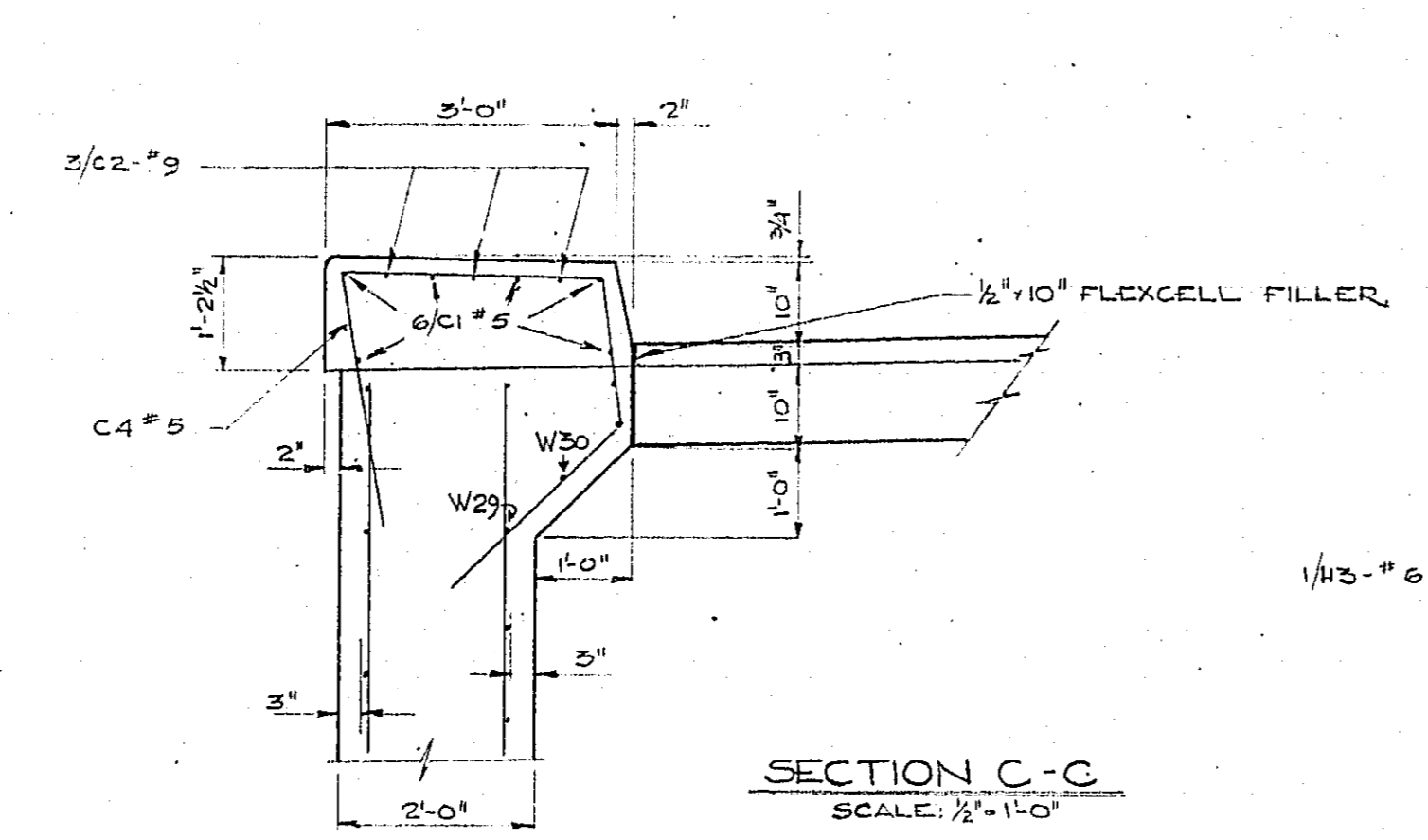
SECTION A-A
WING WALLS
SCALE: 1/2\"/>



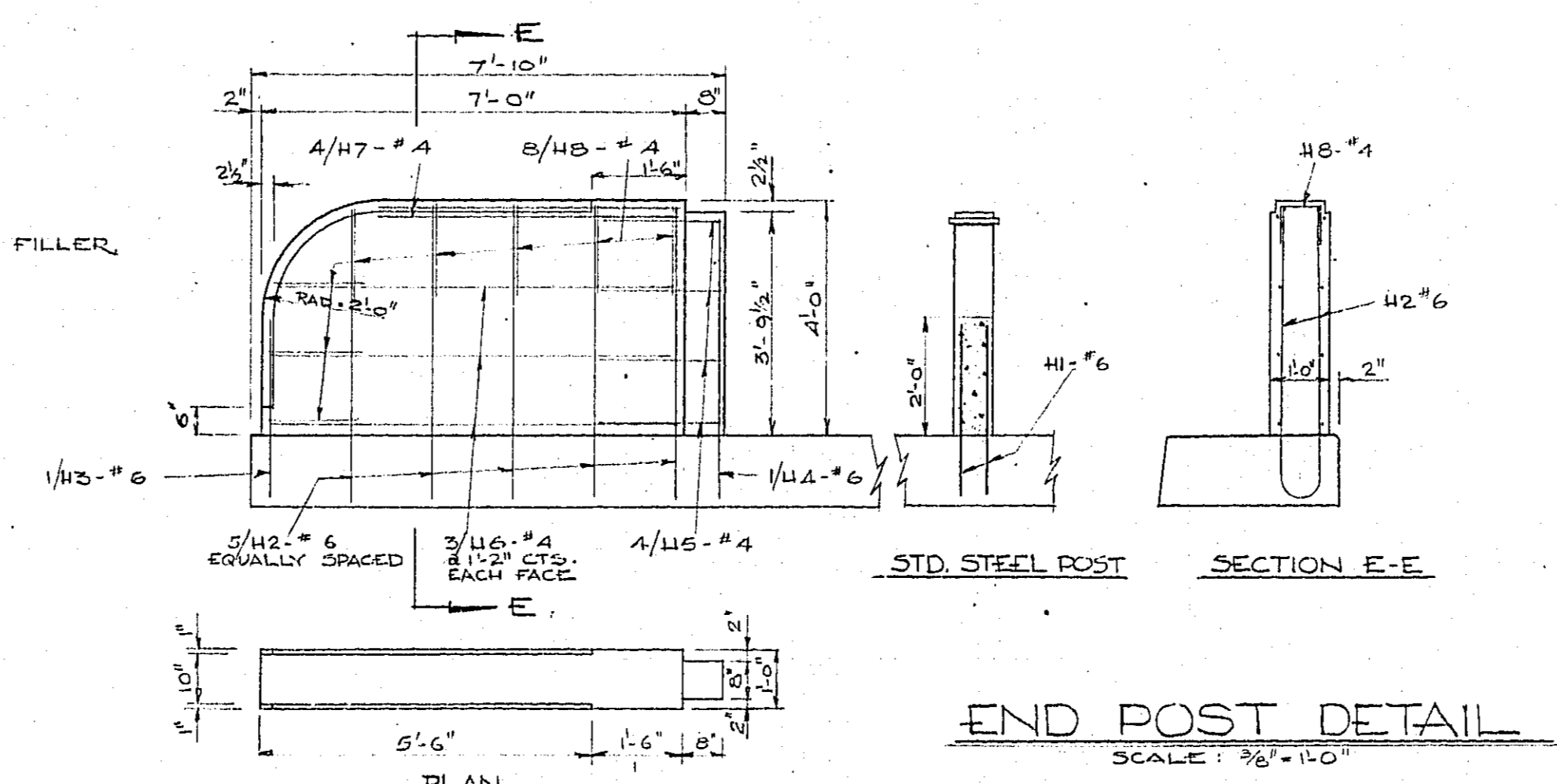
FRONT FACE ELEVATION



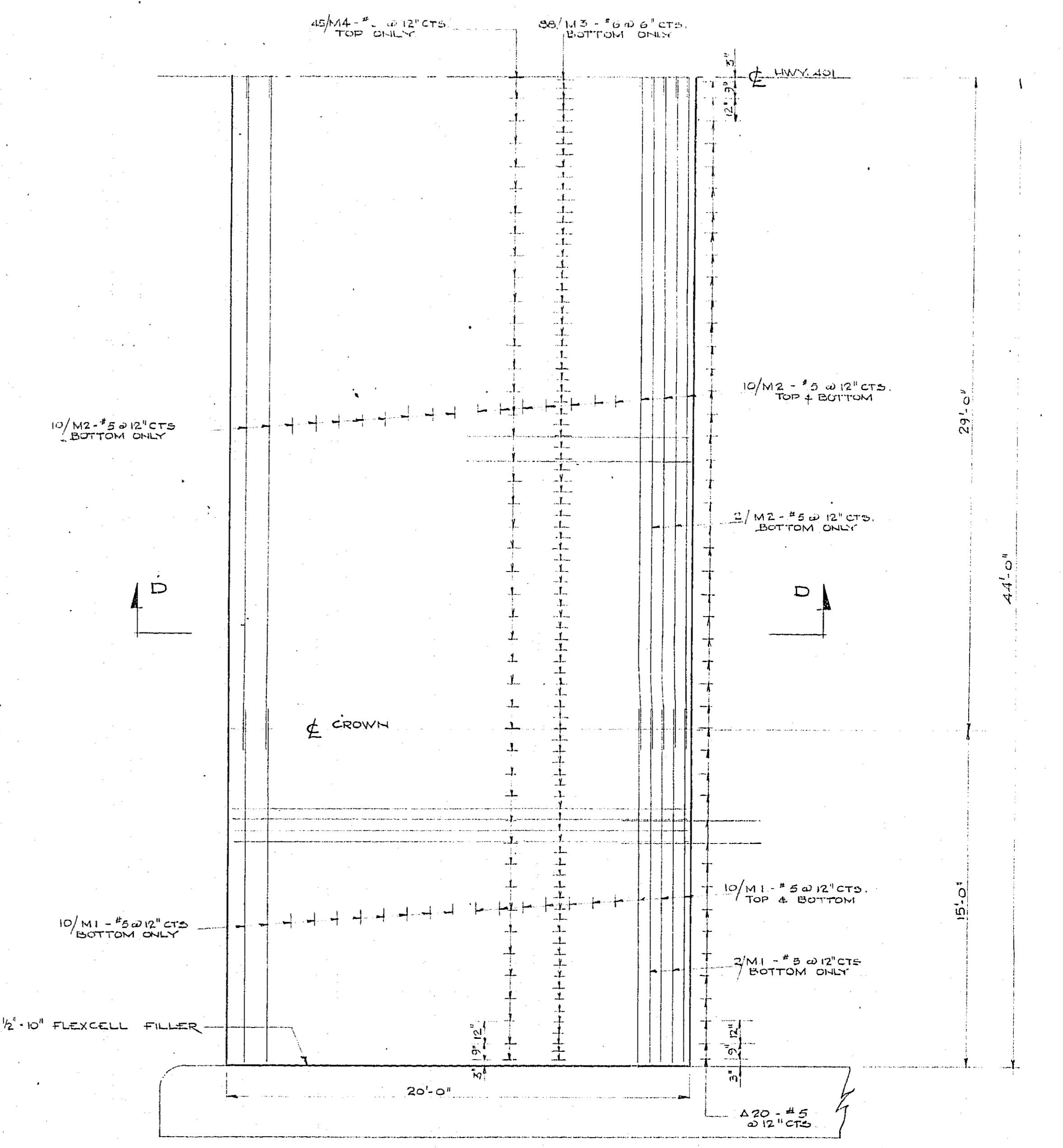
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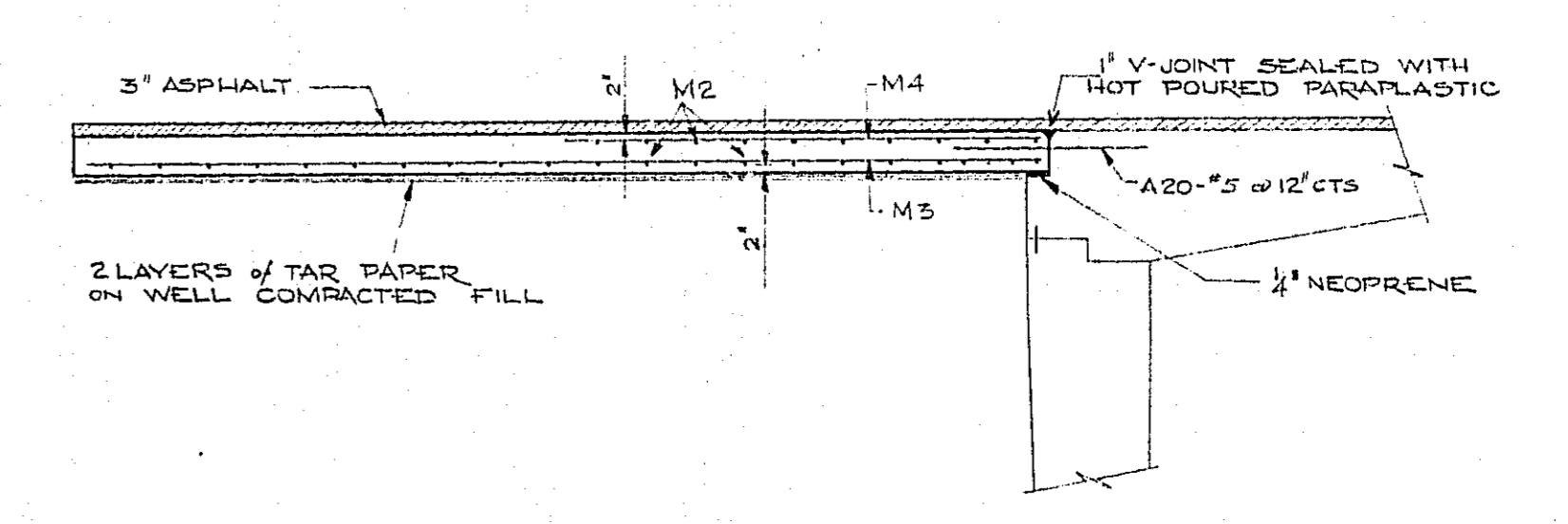
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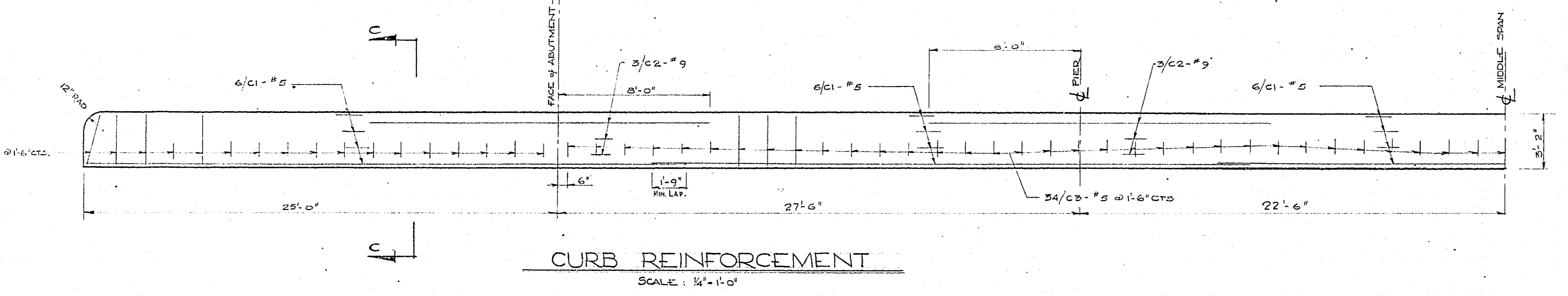
END POST DETAIL
SCALE: 3/8\"/>



APPROACH SLAB - HALF PLAN
SCALE: 1/2\"/>



SECTION D-D



CURB REINFORCEMENT
SCALE: 3/4\"/>

PRINT RECORD		
NO.	FOR	DATE
30	REVISION	5.6.60
31	REVISION	11.8.60
32	REVISION	3.7.61
33	REVISION	7.11.61

W.P. 158-58

DEPARTMENT OF HIGHWAYS-ONTARIO BRIDGE OFFICE-TORONTO			
FREDERICKSBURG N. TWP. BRIDGE #1 (NAPANEE RIVER)			
THE KING'S HIGHWAY No. 401		DIST. No. 2	
CO. LENNDY & ADDINGTON		LOT 20 (CON. I) & 23 (CON. IV) CON. II & IV	
WING WALLS, CURB, APPROACH SLAB & END POSTS			
APPROVED <i>[Signature]</i>			
BRIDGE ENGINEER		DESIGN ENGINEER	
DESIGN	A.R.	CHECK	W.B.
DRAWING	W.V.	CHECK	W.B.
TRACING		CHECK	
DATE	JULY 1959	LOADING NUMBER	H 20 S 15
		DRAWING NUMBER	D-4242-3

REVISIONS		REFERENCE PLANS	
NO.	DATE	NO.	DESCRIPTION
1	7.24.59	1	REV. AS CON'T.

TWP# 17-62-3-A

Appendix B.
Record of Borehole Sheets



SYMBOLS, ABBREVIATIONS AND TERMS USED ON TEST HOLE RECORDS

TERMINOLOGY DESCRIBING COMMON SOIL GENESIS

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

TERMINOLOGY DESCRIBING SOIL STRUCTURE:

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

RECOVERY:

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

N-VALUE:

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

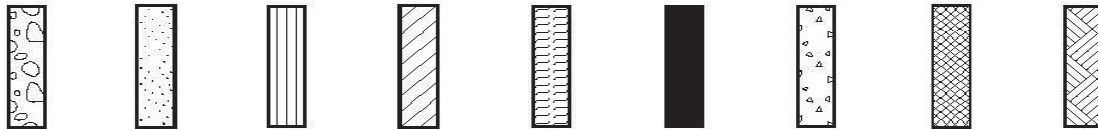
DYNAMIC CONE PENETRATION TEST (DCPT):

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



STRATA PLOT:

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



Boulders
Cobbles
Gravel Sand Silt Clay Organics Asphalt Concrete Fill Bedrock

TEXTURING CLASSIFICATION OF SOILS

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

TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

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

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

SAMPLE TYPES

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

TERMS DESCRIBING CONSISTENCY (COHESIONLESS SOILS ONLY)

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



MODIFIED UNIFIED SOIL CLASSIFICATION

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

Note - W_L = Liquid Limit



EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION

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

TERMS

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

DISCONTINUITY SPACING

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

STRENGTH CLASSIFICATION

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

RECORD OF BOREHOLE No 19-1

1 OF 3

METRIC

GWP# 4040-20-00 LOCATION Lat: 44.267644°, Long: -76.932844°
 Napanee River Bridge MTM Zone 9: N 4 903 162.4 E 270 243.1 ORIGINATED BY MW
 HWY 401 BOREHOLE TYPE Explo. NW Casing, NQ Coring COMPILED BY AC
 DATUM Geodetic DATE 2019.12.16 - 2020.01.09 CHECKED BY SBP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
						20	40	60	80	100	20	40	60		
87.7	Ground Surface														
0.0	TOPSOIL (175 mm)														
0.2	SAND, trace silt Loose Brown		1	SS	8										
87.1	Brown FILL														
0.6	CLAY (Cl) with sand Very stiff Brown to grey		2	SS	10										
			3	SS	3									0 18 57 25	
	-oxidized seams at 2.4 m		4	SS	6										
			5	SS	10										
			6	SS	20										
			7	SS	7										
			8	SS	20									1 18 30 51	
81.6	CLAYEY SILT (CL), occasional silt seams Stiff to very stiff Grey		9	SS	4									S _u >100 kPa	
6.1														S _u >100 kPa	
			10	SS	7									S _u >100 kPa	
														S _u >100 kPa	
			11	SS	3									0 1 66 33	

DOUBLE LINE 27253 HWY 401 NAPANEE RIVER BRIDGE.GPJ_2012TEMPLATE(MTO).GDT 24/3/21

Continued Next Page

+³, ×³: Numbers refer to Sensitivity
 20
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 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 19-1

2 OF 3

METRIC

GWP# 4040-20-00 LOCATION Lat: 44.267644°, Long: -76.932844°
Napane River Bridge MTM Zone 9: N 4 903 162.4 E 270 243.1 ORIGINATED BY MW
HWY 401 BOREHOLE TYPE Explo. NW Casing, NQ Coring COMPILED BY AC
DATUM Geodetic DATE 2019.12.16 - 2020.01.09 CHECKED BY SBP

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa						
76.7	Continued From Previous Page CLAYEY SILT (CL) , occasional silt seams Stiff to very stiff Grey		12	ST	-									
11.0	SILTY SAND (SM) to Sandy SILT (ML) with gravel Dense to very dense Grey TILL -tricone grinding from 11.0 m to 13.7 m on inferred gravel, cobbles, and boulders		13	SS	100/ 0 mm									
			14	SS	100/ 250 mm									
			15	SS	67								4 63 33 (SI+CL)	
			16	SS	100/ 250 mm									
			17	SS	100/ 200 mm								28 32 40 (SI+CL)	
67.7			18	SS	100/									

DOUBLE LINE 27253 HWY 401 NAPANEE RIVER BRIDGE.GPJ 2012TEMPLATE(MTO).GDT 24/3/21

Continued Next Page

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

RECORD OF BOREHOLE No 19-1

3 OF 3

METRIC

GWP# 4040-20-00 LOCATION Lat: 44.267644°, Long: -76.932844°
Napane River Bridge MTM Zone 9: N 4 903 162.4 E 270 243.1 ORIGINATED BY MW
 HWY 401 BOREHOLE TYPE Explo. NW Casing, NQ Coring COMPILED BY AC
 DATUM Geodetic DATE 2019.12.16 - 2020.01.09 CHECKED BY SBP

SOIL PROFILE		SAMPLES				GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
20.0	End of Borehole				200 mm												

DOUBLE LINE 27253 HWY 401 NAPANEE RIVER BRIDGE.GPJ_2012TEMPLATE(MTO).GDT 24/3/21

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

RECORD OF BOREHOLE No 19-2

1 OF 3

METRIC

GWP# 4040-20-00 LOCATION Lat: 44.267452°, Long: -76.932777° Napanee River Bridge MTM Zone 9: N 4 903 141.0 E 270 248.3 ORIGINATED BY MW
 HWY 401 BOREHOLE TYPE CME 75, HSA, NW Casing, NQ Coring COMPILED BY MW
 DATUM Geodetic DATE 2019.11.17 - 2019.11.17 CHECKED BY SBP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE							
88.5	Pavement Surface														
0.0	ASPHALT (125 mm)														
0.1	SAND, with silt and gravel Very dense to compact Brown to grey Dry FILL		1	SS	50		88							35 52 13 (SI+CL)	
			2	SS	104										
			3	SS	44		87								
			4	SS	12		86								
85.2	SAND with silt, trace gravel Compact to very loose Grey brown to dark brown Dry to wet		5	SS	13		85								
3.3			6	SS	3		84							2 85 13 (SI+CL)	
							83								
82.4	CLAY (CI) to CLAYEY SILT (CL), some sand Stiff to very stiff Grey brown		7	SS	3		82							0 19 27 54	
6.1							81								
			8	SS	5										
							80								
			9	SS	6		79								

DOUBLE LINE 27253 HWY 401 NAPANEE RIVER BRIDGE.GPJ_2012TEMPLATE(MTO).GDT_24/3/21

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

RECORD OF BOREHOLE No 19-2

2 OF 3

METRIC

GWP# 4040-20-00 LOCATION Lat: 44.267452°, Long: -76.932777° Napanee River Bridge MTM Zone 9: N 4 903 141.0 E 270 248.3 ORIGINATED BY MW
 HWY 401 BOREHOLE TYPE CME 75, HSA, NW Casing, NQ Coring COMPILED BY MW
 DATUM Geodetic DATE 2019.11.17 - 2019.11.17 CHECKED BY SBP

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa						
	Continued From Previous Page													
	CLAY (CI) to CLAYEY SILT (CL), some sand Stiff to very stiff Grey brown -becoming grey		10	SS	2								S _u >100 kPa S _u >100 kPa	
			11	SS	2								S _u >100 kPa S _u >100 kPa	
			12	SS	4									
73.3														
15.2	SILTY SAND (SM), trace gravel Dense to very dense Grey		13	SS	31									
			14	SS	102								2 74 24 (SI+CL)	
			15	SS	125									
69.1														
19.4	SILTY SAND (SM), trace gravel TILL Very dense Grey -inferred boulder at 19.4 m													

DOUBLE LINE 27253 HWY 401 NAPANEE RIVER BRIDGE.GPJ 2012TEMPLATE(MTO).GDT 24/3/21

Continued Next Page

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

RECORD OF BOREHOLE No 19-2

3 OF 3

METRIC

GWP# 4040-20-00 LOCATION Lat: 44.267452°, Long: -76.932777°
 Napanee River Bridge MTM Zone 9: N 4 903 141.0 E 270 248.3 ORIGINATED BY MW
 HWY 401 BOREHOLE TYPE CME 75, HSA, NW Casing, NQ Coring COMPILED BY MW
 DATUM Geodetic DATE 2019.11.17 - 2019.11.17 CHECKED BY SBP

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa								
						20	40	60	80	100						
66.6	Continued From Previous Page SILTY SAND (SM) , trace gravel Very dense Grey TILL		16	SS	127											
						68										
			17	SS	102											
21.9	End of Borehole					67										1 54 45 (SI+CL)

DOUBLE LINE 27253 HWY 401 NAPANEE RIVER BRIDGE.GPJ 2012TEMPLATE(MTO).GDT 24/3/21

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

RECORD OF BOREHOLE No 19-3

2 OF 3

METRIC

GWP# 4040-20-00 LOCATION Lat: 44.267212°, Long: -76.932675° Napanee River Bridge MTM Zone 9: N 4 903 114.3 E 270 256.3 ORIGINATED BY MW
 HWY 401 BOREHOLE TYPE Explo. NW Casing, NQ Coring COMPILED BY MW
 DATUM Geodetic DATE 2020.01.23 - 2020.01.28 CHECKED BY SBP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60					
	Continued From Previous Page														
76	CLAYEY SILT (CL to CL-ML), occasional silt seams Stiff to very stiff Grey		12	SS	2										1 10 67 22
75															
74			13	SS	1										
73.3	-could not push vane at 13.0 m														
73	SILTY SAND (SM) to Sandy SILT (ML) Very loose to very dense Grey to brown		14	SS	34										
72															
71			15	SS	21										0 55 37 8 -non-plastic
70															
69			16	SS	29										
68			17	SS	100/ 305 mm										
67															

DOUBLE LINE 27253 HWY 401 NAPANEE RIVER BRIDGE.GPJ_2012TEMPLATE(MTO).GDT_24/3/21

Continued Next Page

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

RECORD OF BOREHOLE No 19-3

3 OF 3

METRIC

GWP# 4040-20-00 LOCATION Lat: 44.267212°, Long: -76.932675°
Napane River Bridge MTM Zone 9: N 4 903 114.3 E 270 256.3 ORIGINATED BY MW
 HWY 401 BOREHOLE TYPE Explo. NW Casing, NQ Coring COMPILED BY MW
 DATUM Geodetic DATE 2020.01.23 - 2020.01.28 CHECKED BY SBP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
	Continued From Previous Page					○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE				WATER CONTENT (%)						
63.8	SILTY SAND (SM) to Sandy SILT (ML) Dense to compact Grey to brown		18	SS	3											0 46 45 9 -non-plastic
22.4	SILTY SAND (SM) with gravel Very dense Grey TILL -125mm cobble at 22.4 m		19	NQ												
			20	NQ												
			21	SS	100/ 51 mm											
62.6																
23.6	End of Borehole upon core barrel refusal on inferred boulder or bedrock. Pressurized sand was noted in the casing.															

DOUBLE LINE 27253 HWY 401 NAPANEE RIVER BRIDGE.GPJ_2012TEMPLATE(MTO).GDT 24/3/21

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

RECORD OF BOREHOLE No 19-4

1 OF 2

METRIC

GWP# 4040-20-00 LOCATION Lat: 44.267721°, Long: -76.932487° Napanee River Bridge MTM Zone 9: N 4 903 170.8 E 270 271.7 ORIGINATED BY MW
 HWY 401 BOREHOLE TYPE Portable, NW Casing, NQ Coring COMPILED BY DJP
 DATUM Geodetic DATE 2019.10.30 - 2019.11.04 CHECKED BY SBP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE							
85.8	Barge Surface														
0.0	BARGE														
85.5															
0.3	WATER														
83.5															
2.3	SILTY SAND, some organics (wood pieces) Very loose Black		1	SS	WH										
82.6															
3.2	CLAY (CI) to CLAYEY SILT (CL), sandy, occasional sand seams Very stiff to firm Grey		2	SS	6										
			3	SS	3									2 13 36 49	
			4	SS	6										
			5	SS	4										
			6	SS	2										
			7	SS	6									0 52 31 17	
76.4															
9.4	SILTY SAND (SM), trace gravel Compact to very dense Grey														

DOUBLE LINE 27253 HWY 401 NAPANEE RIVER BRIDGE.GPJ_2012TEMPLATE(MTO).GDT_24/3/21

Continued Next Page

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

RECORD OF BOREHOLE No 19-4

2 OF 2

METRIC

GWP# 4040-20-00 LOCATION Lat: 44.267721°, Long: -76.932487° Napanee River Bridge MTM Zone 9: N 4 903 170.8 E 270 271.7 ORIGINATED BY MW
 HWY 401 BOREHOLE TYPE Portable, NW Casing, NQ Coring COMPILED BY DJP
 DATUM Geodetic DATE 2019.10.30 - 2019.11.04 CHECKED BY SBP

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
					20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					WATER CONTENT (%)			GR SA SI CL
Continued From Previous Page													
70.9	SILTY SAND (SM) , trace gravel Compact to very dense Grey	8	SS	13									
		9	SS	62									
		10	SS	22									7 77 16 (SI+CL)
14.9	Sandy SILT (ML) , trace gravel Very dense Grey TILL - 125 mm cobble at 15.8 m	11	SS	100/ 150mm									1 39 48 12 -non-plastic
		12	NQ	13									
		13	SS	100/ 100mm									
67.6		14	SS	100/ 75mm									
18.2	End of Borehole Note: A half-weight hammer was used to advance the split-spoon sampler. The "N" Values presented above have been adjusted to provide an estimate of the "N" Value that would have been obtained with a standard hammer.												

DOUBLE LINE 27253 HWY 401 NAPANEE RIVER BRIDGE.GPJ 2012TEMPLATE(MTO).GDT 24/3/21

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

RECORD OF BOREHOLE No 19-5

1 OF 3

METRIC

GWP# 4040-20-00 LOCATION Lat: 44.267419°, Long: -76.932435°
 Napanee River Bridge MTM Zone 9: N 4 903 137.1 E 270 275.6 ORIGINATED BY MW
 HWY 401 BOREHOLE TYPE Portable, NW Casing, NQ Coring COMPILED BY DJP
 DATUM Geodetic DATE 2019.10.25 - 2019.10.30 CHECKED BY SBP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60						80	100	WATER CONTENT (%)
85.5	Barge Surface																	
0.0	BARGE																	
85.2																		
0.3	WATER																	
83.8																		
1.7	GRAVEL (GC), clayey, with sand Very stiff Grey brown		1	SS	17*									(CI)	35	20	18	27
			2	SS	10*													
	-material was washed away while advancing casing																	
81.4																		
4.1	CLAYEY SILT (CL), occasional silt seams Stiff to very stiff Grey		3	SS	13*													
			4	SS	4*													
			5	SS	4*									(CI)	0	2	54	44
			6	SS	3													
			7	SS	2													

DOUBLE LINE 27253 HWY 401 NAPANEE RIVER BRIDGE.GPJ 2012TEMPLATE(MTO).GDT 24/3/21

Continued Next Page

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

RECORD OF BOREHOLE No 19-5

2 OF 3

METRIC

GWP# 4040-20-00 LOCATION Lat: 44.267419°, Long: -76.932435°
 Napanee River Bridge MTM Zone 9: N 4 903 137.1 E 270 275.6 ORIGINATED BY MW
 HWY 401 BOREHOLE TYPE Portable, NW Casing, NQ Coring COMPILED BY DJP
 DATUM Geodetic DATE 2019.10.25 - 2019.10.30 CHECKED BY SBP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	W P W W L		
73.2	Continued From Previous Page CLAYEY SILT (CL) , occasional silt seams Stiff to very stiff Grey		8	SS	5									
12.3	Sandy SILT (ML) Loose to compact Grey		9	SS	12									
			10	SS	14									
			11	SS	6									
			12	SS	13									
66.9	SILTY SAND (SM) to SAND with silt Very dense to compact Grey TILL -casing refusal at 18.9m -100 mm, 50 mm cobbles at 19.1 m		13	SS	65									
18.6			14	NQ	-									

DOUBLE LINE 27253 HWY 401 NAPANEE RIVER BRIDGE.GPJ_2012TEMPLATE(MTO).GDT_24/3/21

Continued Next Page

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

RECORD OF BOREHOLE No 19-5

3 OF 3

METRIC

GWP# 4040-20-00 LOCATION Lat: 44.267419°, Long: -76.932435°
Napaneé River Bridge MTM Zone 9: N 4 903 137.1 E 270 275.6 ORIGINATED BY MW
HWY 401 BOREHOLE TYPE Portable, NW Casing, NQ Coring COMPILED BY DJP
DATUM Geodetic DATE 2019.10.25 - 2019.10.30 CHECKED BY SBP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					20 40 60				GR SA SI CL		
62.2	Continued From Previous Page SILTY SAND (SM) to SAND with silt Very dense to compact Grey TILL - 225 mm boulder at 21.7 m - minimum 225mm boulder or bedrock		15	SS	13											0 91 7 2 -non-plastic	
23.3	End of Borehole due to core barrel refusal in boulder or inferred bedrock. Note: A half-weight hammer was used to advance the split-spoon sampler. The "N" Values presented above have been adjusted to provide an estimate of the "N" Value that would have been obtained with a standard hammer. Note: (*) Denoted that sample was taken with a size "N" split spoon sampler.		16	SS	106/ 250mm												
			17	NQ	-												
			18	SS	100/												
			19	NQ	75mm												

DOUBLE LINE 27253 HWY 401 NAPANEÉ RIVER BRIDGE.GPJ_2012TEMPLATE(MTO).GDT_24/3/21

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

RECORD OF BOREHOLE No 19-6

2 OF 2

METRIC

GWP# 4040-20-00 LOCATION Lat: 44.267847°, Long: -76.93222° Napanee River Bridge MTM Zone 9: N 4 903 184.6 E 270 293.0 ORIGINATED BY MW
 HWY 401 BOREHOLE TYPE Explo. NW Casing, NQ Coring COMPILED BY MW
 DATUM Geodetic DATE 2020.01.21 - 2020.01.22 CHECKED BY SBP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60					
74.6	CLAYEY SILT (CL) Stiff to soft Grey	[Hatched pattern]	11	SS	WH		2.0								
12.2	SILTY SAND (SM), trace gravel Very dense Grey	[Dotted pattern]	12	SS	55		3.0							6 63 31 (SI+CL)	
71.5	SILTY SAND (SM) with gravel Very dense Grey TILL -inferred cobbles at 15.2 m	[Dotted pattern]	13	SS	100/ 250 mm		0.7								
69.7	SILTY SAND (SM) with gravel Very dense Grey TILL -inferred cobbles at 15.2 m	[Dotted pattern]	14	SS	100/ 175 mm										
17.1	End of Borehole		15	SS	100										
Standpipe Piezometer Readings: Date Depth (m) Elev. (m) 01/24/2020 0.3 86.5 01/29/2020 0.4 86.4															

DOUBLE LINE 27253 HWY 401 NAPANEE RIVER BRIDGE.GPJ_2012TEMPLATE(MTO).GDT_24/3/21

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

RECORD OF BOREHOLE No 19-7

3 OF 3

METRIC

GWP# 4040-20-00 LOCATION Lat: 44.267656°, Long: -76.932117°
 Napanee River Bridge MTM Zone 9: N 4 903 163.4 E 270 301.1 ORIGINATED BY MW
 HWY 401 BOREHOLE TYPE CME 75, HSA, NW Casing, NQ Coring COMPILED BY MW
 DATUM Geodetic DATE 2019.11.18 - 2019.11.19 CHECKED BY SBP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
						20	40	60	80	100	W _p	W	W _L				
Continued From Previous Page																	
65.9	Sandy SILT (ML) with occasional to frequent clay layers Loose to compact Grey		16	SS	13												
22.9	SAND with silt and gravel Very dense Grey TILL -200 mm boulder, 200 mm boulder, 130 mm cobble, 90 mm cobble, 60 mm cobble between 22.9 m and 23.7 m -75 mm cobble and 75 mm cobble between 23.7 m and 25.3 m		17	NQ	-												
			18	NQ	-												
63.2			19	NQ	-												
25.6	LIMESTONE BEDROCK Slightly weathered Fine grained Thinly bedded Grey		1	Run													
			2	Run													
			3	Run													
	-highly fractured from 28.6 m to 28.8 m																
59.6																	
29.2	End of Borehole																

DOUBLE LINE 27253 HWY 401 NAPANEE RIVER BRIDGE.GPJ_2012TEMPLATE(MTO).GDT_24/3/21

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

RECORD OF BOREHOLE No 19-8

2 OF 3

METRIC

GWP# 4040-20-00 LOCATION Lat: 44.267522°, Long: -76.932028° Napanee River Bridge MTM Zone 9: N 4 903 148.4 E 270 308.2 ORIGINATED BY MW
 HWY 401 BOREHOLE TYPE Explo. NW Casing, NQ Coring COMPILED BY AC
 DATUM Geodetic DATE 2020.01.14 - 2020.01.17 CHECKED BY SBP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60					
74.3	Continued From Previous Page CLAY (CH) to CLAYEY SILT (CL) , with occasional sand seams Firm to stiff Grey - silt, some sand seam		12	SS	6									1 12 71 16 -non-plastic	
12.2	SILTY SAND (SM) to SAND Loose to dense Brown to grey		13	SS	7									0 75 23 2 -non-plastic	
			14	SS	39										
			15	SS	19										
			16	SS	30										
			17	SS	36									1 95 4 (SI+CL)	
66.7			18	SS	100/										
19.8	Inferred TILL														

DOUBLE LINE 27253 HWY 401 NAPANEE RIVER BRIDGE.GPJ 2012TEMPLATE(MTO).GDT 24/3/21

Continued Next Page

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

RECORD OF BOREHOLE No 19-8

3 OF 3

METRIC

GWP# 4040-20-00 LOCATION Lat: 44.267522°, Long: -76.932028° Napanee River Bridge MTM Zone 9: N 4 903 148.4 E 270 308.2 ORIGINATED BY MW
 HWY 401 BOREHOLE TYPE Explo. NW Casing, NQ Coring COMPILED BY AC
 DATUM Geodetic DATE 2020.01.14 - 2020.01.17 CHECKED BY SBP

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa							
Continued From Previous Page															
64.9	Inferred TILL -tricone from 19.8 m to 21.6 m				0 mm										
21.6	LIMESTONE BEDROCK Slightly weathered Fine grained Freshly jointed Grey		1	RUN	-										RUN #1 TCR=100% SCR=100% RQD=100% RUN #2 TCR=100% SCR=98% RQD=90%
			2	RUN	-										
			3	RUN	-										RUN #3 TCR=100% SCR=100% RQD=98%
61.7	End of Borehole														
24.8	Standpipe Piezometer Readings: Date Depth (m) Elev. (m) 01/20/2020 0.1 86.4 01/23/2020 0.3 86.2 01/24/2020 0.4 86.1 01/29/2020 0.4 86.1														

DOUBLE LINE 27253 HWY 401 NAPANEE RIVER BRIDGE.GPJ 2012TEMPLATE(MTO).GDT 24/3/21

+³, ×³: Numbers refer to Sensitivity $\frac{20}{15 \pm 5}$ (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No SCPT 19-1

1 OF 1

METRIC

GWP# 4040-20-00 LOCATION Lat: 44.267425°, Long: -76.932852° Napanee River Bridge MTM Zone 9: N 4 903 138.0 E 270 242.4 ORIGINATED BY MW
 HWY 401 BOREHOLE TYPE CME 75, HSA COMPILED BY MW
 DATUM Geodetic DATE 2019.11.18 - 2019.11.18 CHECKED BY SBP

SOIL PROFILE			SAMPLES				GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	20			40	60	80	100	PLASTIC LIMIT W _p		
88.5	Pavement Surface														
0.0	ASPHALT														
0.1	Visual descriptions from cuttings SAND and GRAVEL Grey Dry Borehole pre-drilled to 10' for Conetec														
88															
87															
86															
85.5															
3.0															

DOUBLE LINE 27253 HWY 401 NAPANEE RIVER BRIDGE.GPJ 2012TEMPLATE(MTO).GDT 24/3/21

RECORD OF BOREHOLE No SCPT 19-2

1 OF 1

METRIC

GWP# 4040-20-00 LOCATION Lat: 44.267713°, Long: -76.931908° Napanee River Bridge MTM Zone 9: N 4 903 169.6 E 270 317.9 ORIGINATED BY MW
 HWY 401 BOREHOLE TYPE CME 75, HSA COMPILED BY MW
 DATUM Geodetic DATE 2019.11.18 - 2019.11.18 CHECKED BY SBP

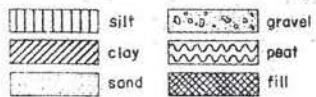
SOIL PROFILE			SAMPLES				GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	20			40	60	80						100	20
88.9	Pavement Surface																	
0.0	ASPHALT																	
0.1	Visual descriptions from cuttings SAND and GRAVEL Grey Dry Borehole pre-drilled to 10' for Conetec																	
88																		
87																		
86																		
85.9																		
3.0																		

DOUBLE LINE 27253 HWY 401 NAPANEE RIVER BRIDGE.GPJ 2012TEMPLATE(MTO).GDT 24/3/21

JOB No. H415/58 LOCATION HWY. 401 NAIJEE RIVER
 CLIENT DEPARTMENT OF HIGHWAYS ONTARIO
 COORDINATES STA. 258 + 70 50' L. OF CL.
 ELEV. (surface) 282.9 (collar) Datum GEOD.
 BOREHOLE NUMBER 1
 DATE (started) MAY 8, 1958 (finished) MAY 12, 1958
 RIG No. 1 TYPE JUN. LONGYEAR

HUNTING TECHNICAL AND EXPLORATION SERVICES

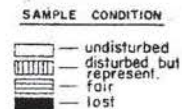
BOREHOLE No. 1



x — standard penetr. 2 s.s.
 Δ — vane shear
 o — pocket penetrometer

C — consolidation test
 M — mechanical analysis
 T — triaxial shear
 K — permeability
 U — unconfined compression

SS — split spoon
 ST — Shelby tube
 T.W.P. — thin walled piston
 D.B. — diamond bit

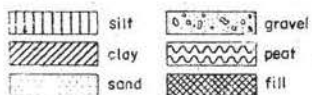


BORING LOG				FIELD TESTS						LABORATORY TESTS			
SCALE	DEPTH	ELEV.	LOG	SHEAR STRENGTH (TONS PER SQUARE FOOT)		SAMPLES				ATTERBERG LIMITS		REMARKS	
FT.	FT.	FT.	DESCRIPTION	1/2	1 1/2	NO.	COND.	DEPTH FROM TO	TYPE	RECOVERY LENGTH REC. DIST. DRIV.	W P X — O W I		
				STANDARD PENETRATION TEST (BLOWS PER FOOT)									
				20	40								
	1.0	282.9	BROWN TOPSOIL										
	5.0	276.9	LOOSE BROWN PLASTIC SILT AND SAND SOME ORGANIC MATTER	x		1	SS	2.0 3.5	SS	9/18	6		
	8.5	274.4	STIFF GREY BROWN SILT AND CLAY	x		2	SS	5.0 6.5	SS	15/18	5		
	10.0		STIFF GREY INTERBEDDED SILT AND CLAY WITH THIN SAND SEAMS			3	SS	8.0 9.5	SS	14/18	12		
	15.0			Δ		4	ST	10.0 11.5	ST	18/18	-	3" OSTERBERG	
	20.0				5	ST	15.0 16.5	ST	15/18	5			
	25.0				6	ST	20.0 21.5	ST	18/18	2			
	30.0				7	ST	25.0 26.5	ST	18/18	-	3" OSTERBERG		
	35.0		SOFT GREY INTERBEDDED SILT AND CLAY WITH THIN SAND SEAMS			8	ST	30.0 31.5	ST	18/18	1		
	40.0			Δ		9	ST	35.0 36.5	ST	18/18	-	35' END OF H CASING 3" OSTERBERG STONES INFLUENCING VANE READING	
	45.0		VERY DENSE GREY TILL CONSISTING OF SILT AND FINE SAND WITH TRACE OF COARSE SAND, GRAVEL AND CLAY			10	SS	40.0 41.5	SS	12/18	55		
	50.0			x		11	SS	44.0 44.5	SS	5/6	74/6"		
	60.0							55.0 60.0	DB				
	61.3	221.6	END OF BOREHOLE			12	SS	60.0 61.3	SS	12/16	130		

JOB No. H415/58 LOCATION HWY 401 NAPANEE RIVER
 CLIENT DEPARTMENT OF HIGHWAYS - ONTARIO
 COORDINATES STA. 358 + 67 1' L. OF CL.
 ELEV. (surface) 282.7 (collar) — Datum GEOD.
 BOREHOLE NUMBER 2
 DATE (started) MAY 3, 1958 (finished) MAY 6, 1958
 RIG No. 1 TYPE JUN. LONGYEAR

HUNTING TECHNICAL AND EXPLORATION SERVICES

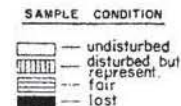
BOREHOLE No. 2



x — standard penetr. 2 S.S.
 Δ — vane shear
 ○ — pocket penetrometer

C — consolidation test
 M — mechanical analysis
 T — tri-axial shear
 K — permeability
 U — unconfined compression

SS — split spoon
 ST — Shelby tube
 T.W.P. — thin walled piston
 D.B. — diamond bit

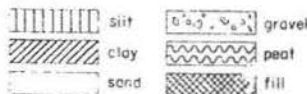


BORING LOG				FIELD TESTS						LABORATORY TESTS				
SCALE	DEPTH	ELEV	LOG	DESCRIPTION	SHEAR STRENGTH (TONS PER SQUARE FOOT)		SAMPLES						ATTERBERG LIMITS (wp X - O w)	REMARKS
					1/2	1 1/2	No.	COND.	DEPTH FROM TO	TYPE	RECOVERY LENGTH REC. DIST. DRIV.	PENETRATION RESISTANCE (BLOWS PER FOOT)		
FT.	FT.	FT.			STANDARD PENETRATION TEST (BLOWS PER FOOT)									
					20	40								
	1-2	282.7		BROWN ORGANIC TOPSOIL			1		0-1.5	SS	9/18	1		
		281.5		LOOSE BROWN SAND AND PLASTIC SILT, SOME ORGANIC MATTER			2		3.0-4.5	SS	18/18	3		
	5						3		6.0-7.5	SS	18/18	5		
	7.0	275.7					4		7.5-8.0	ST	6/6			
							5		9.0-9.0	ST	12/12			
	10			STIFF GREY INTERBEDDED SILT AND CLAY WITH THIN SAND SEAMS			6		14.0-15.0	ST	17/18	8		
	15						7		19.0-20.5	ST	18/18	3		
	20						8		24.0-25.5	ST	18/18			
	25						9		29.0-31.0	ST	0/24	1		
	30			SOFT GREY INTERBEDDED SILT AND CLAY WITH THIN SAND SEAMS			10		34.0-35.5	ST	14/18			
	37.0	245.7					11		39.0-40.5	ST	18/18	1		
	40			LOOSE TO MEDIUM DENSE GREY SILTY SAND			12		44.0-45.5	SS	8/18	11		
	45						13		49.0-50.5	SS	18/18			
	50						14		54.0-54.7	SS	8/8	108/8"		
	52.0	230.7		VERY DENSE GREY TILL (SEE REMARKS)										
	55	54.7		END OF BOREHOLE										
	60													

JOB No. H419/58 LOCATION HWY 401 NAPANEE RIVER
 CLIENT DEPARTMENT OF HIGHWAYS - ONTARIO
 COORDINATES STA. 358+60 WD. R. OF CL
 ELEV. (surface) 282.7 (coll.) — Datum GEOD.
 BOREHOLE NUMBER 3
 DATE (started) APR. 30, 1958 (finished) MAY 2, 1958
 RIG No. 1 TYPE JUN. A LONGYEAR

HUNTING TECHNICAL AND EXPLORATION SERVICES

BOREHOLE No. 3



x — standard penetr. 2 S.S.
 Δ — vane shear
 o — pocket penetrometer

C — consolidation test
 M — mechanical analysis
 T — triaxial shear
 K — permeability
 U — unconfined compression

SS — split spoon
 ST — Shelby tube
 TWP — thin walled piston
 D.B. — diamond bit







SAMPLE CONDITION
 — undisturbed
 — disturbed but represent.
 — fair
 — lost

BORING LOG				FIELD TESTS						LABORATORY TESTS		REMARKS		
SCALE	DEPTH	ELEV.	LOG	DESCRIPTION	SHEAR STRENGTH (TONS PER SQUARE FOOT)			SAMPLES					ATTERBERG LIMITS (W _L X — G _w)	
					1/2	1/2	1/2	No.	COND.	DEPTH FROM TO	TYPE			RECOVERY LENGTH REC. DIST. DRIV.
FT	FT	FT			STANDARD PENETRATION TEST (BLOWS PER FOOT)									
					20	40	60							
	2.0	282.7		BROWN TOPSOIL				1	SS	0-1.5	SS	18/18	1	
	5.0			LOOSE BROWN SAND AND SILT SOME ORGANIC MATTER				2	SS	3.0-4.5	SS	12/18	5	
	6.3	276.4	G.W.L.											
	8.0	274.7		LOOSE GREY GRAVEL				3	SS	6.0-7.5	SS	15/18	1	
	10.0			STIFF GREY INTERBEDDED SILT AND CLAY WITH THIN SAND SEAMS				4	ST	9.0-10.5	ST	10/18	9	
	15.0							5	ST	14.0-15.5	ST	18/18	5	
	20.0							6	ST	19.0-20.5	ST	15/18	4	
	25.0							7	ST	24.0-25.5	ST	18/18	1	END N X CASING 25'
	30.0			MEDIUM STIFF GREY INTERBEDDED SILT AND CLAY WITH THIN SAND SEAMS				8	ST	29.0-30.5	ST	18/18	1	
	35.0							9	ST	34.0-35.5	ST		0	PENETRATION BY ROD WEIGHT
	37.8	244.9		GRAVEL LAYER										
	38.5	243.2												
	40.0			MEDIUM STIFF GREY VARVED SILT AND CLAY				10	ST	39.0-40.5	ST		1	4 BLOWS ON SS 18" RECOVERY
	42.0	240.7												
	45.0							11	ST	44.0-45.5	ST	0/18	4	RECOVERED 16" WITH SS
	50.0			LOOSE TO MEDIUM DENSE GREY SILTY FINE SAND (SOME GRAVEL FRAGMENTS)				12	SS	49.0-50.5	SS	16/18	65*	* SAMPLER STUCK IN CASING
	55.0							13	SS	54.0-55.5	SS	16/18	24	
	57.5	225.2												
	60.0	222.7		VERY DENSE GREY TILL				14	SS	59.0-60.0	SS	5/12	150	END B X CASING 59' - PROBABLE BOULDER AT 60'
				END OF BOREHOLE										TILL CONSISTS OF SILT AND SAND WITH SOME FINE GRAVEL AND TRACE OF CLAY

JOB No. H 415/58 LOCATION HWY. 401 - NAPANEE RIVER
 CLIENT DEPARTMENT OF HIGHWAYS - ONTARIO
 COORDINATES STA. 359 + 82 50' L. OF CL.
 ELEV. (surface) 282.0 (collar) -- Datum: GEOD.
 BOREHOLE NUMBER 4
 DATE (started) APR. 10, 1958 (finished) APR. 15, 1958
 RIG No. 1 TYPE JUN. LONGYEAR

HUNTING TECHNICAL AND EXPLORATION SERVICES

BOREHOLE No. 4




 silt
 clay
 sand
 gravel
 peat
 fill

x — standard penetr. 2 s.s.
 Δ — vane shear
 o — pocket penetrometer

C — consolidation test
 M — mechanical analysis
 T — triaxial shear
 K — permeability
 U — unconfined compression

SS — split spoon
 ST — shelly tube
 T.W.P. — thin walled piston
 D.B. — diamond bit

SAMPLE CONDITION

 — undisturbed
 — disturbed but represent. for
 — lost

BORING LOG				FIELD TESTS						LABORATORY TESTS				
SCALE	DEPTH	ELEV.	LOG	SHEAR STRENGTH (TONS PER SQUARE FOOT)		SAMPLES				ATTERBERG LIMITS wp x — o w l			REMARKS	
FT.	FT.	FT.	DESCRIPTION	1/2	1/2	No.	COND.	DEPTH FROM TO	RECOVERY LENGTH REC. DIST. DRIV.	PENETRATION RESISTANCE (BLOWS PER FOOT)	25	50		75
				STANDARD PENETRATION TEST (BLOWS PER FOOT)							WATER CONTENT			
		282.0												
	3.0	279.0	SOFT BROWN ORGANIC SILT & SAND			1		0 2.0	ST 17/24					
	5		LOOSE GREY SAND, SOME SILT SOME ORGANIC MATTER			2		5.0 7.0	ST 16/24					
	10	270.7				3		10.0 11.9	ST 14/18					
	15		STIFF GREY CLAY WITH INTERBEDDED SILT SEAMS, AND THIN SAND SEAMS			4		15.3 16.8	ST 15/18				2.45	RATE 8 SEC. FOR 12"
	20					5		20.5 22.0	ST 10/18					NX CASING HARD TO DRIVE DUE TO FRICTION
	25					6		25.5 27.5	ST 22/24					
	30		SOFT			7		30.5 32.2	SS 19/20	12				
	35	246.5				8		35.5 37.0	ST & SS	19				SAMPLE RECOVERED WITH SIDE SLIT SAMPLER
	38.0	244.0	MEDIUM DENSE GREY MEDIUM SAND			9		40.3 41.8	SS 15/18	65				
	40		VERY DENSE GREY FINE SAND & SILT (WITH GRAVELLY LAYERS)			10		45.0 46.5	SS 8/18	178				
	45					11		50.3 51.8	SS 18/18	148				
	50					12		55.0 56.5	SS 16/18	58				REFUSAL ON SS WITH 350 LBS. HAMMER
	55					13		59.0 66.0	DB 12/84					
	59.0	223.0	VERY DENSE GREY TILL			14		70.0 73.0	DB 36/36					COARSE TILL & BOULDER CORE
	65					15		74.3 77.3	DB 24/36					
	70					16		77.5 81.2	DB 42/47					
	75													
	81.0	201.0	END OF BOREHOLE											

JOB No. H415/58 LOCATION HWY 401 NAPANEE RIVER

CLIENT DEPARTMENT OF HIGHWAYS - ONTARIO

COORDINATES STA. 359 + 74 CL

ELEV. (SURFACE) 282.0 (collar) Datum O.E.O.D.

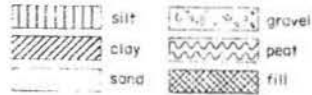
BOREHOLE NUMBER 5

DATE (started) APR. 16, 1958 (finished) APR. 18, 1958

RIG No. 1 TYPE JUN. LONGYEAR

HUNTING TECHNICAL AND EXPLORATION SERVICES

BOREHOLE No. 5

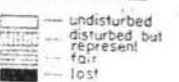


x — standard penetr. 2 S.S.
 Δ — vane shear
 ○ — pocket penetrometer

C — consolidation test
 M — mechanical analysis
 T — triaxial shear
 K — permeability
 U — unconfined compression

SS — split spoon
 ST — Shelby tube
 TWP — thin walled piston
 D.B. — diamond bit

SAMPLE CONDITION



BORING LOG

FIELD TESTS

LABORATORY TESTS

SCALE	DEPTH	ELEV.	WATER (SEPARATION)	LOG	DESCRIPTION	SHEAR STRENGTH (TONS PER SQUARE FOOT)			STANDARD PENETRATION TEST (BLOWS PER FOOT)			SAMPLES				ATTERBERG LIMITS			REMARKS			
						1/2	1/2	1/2	No.	COND.	DEPTH FROM TO	TYPE	RECOVERY LENGTH REC. DIST. DRIV.	PENETRATION RESISTANCE (BLOWS PER FOOT)	WD	X	—O W	U undist 1/ft ²		U remolded	M	
	2.5	279.5			SOFT BROWN ORGANIC SILT & SAND																	
	5				(SOME SILT) LOOSE GREY SAND SOME ORGANIC MATTER																	
	10	270.5																				
	15																					
	20				STIFF GREY INTERBEDDED SILT & CLAY WITH THIN SAND SEAMS																	
	25																					
	30																					
	35				SOFT GREY INTERBEDDED SILT & CLAY WITH THIN SAND SEAMS																	
	40																					
	45																					
	50	235.0			LOOSE GREY FINE SAND & SILT																	
	55	231.0																				
	60				DENSE TO VERY DENSE GREY TILL CONSISTING OF SAND, SILT SOME GRAVEL. TRACE OF SILT																	
	65																					
	70	215.0			END OF BOREHOLE																	

JOB No. H 415/58 LOCATION HWY 401 NAPANEE RIVER
 CLIENT DEPARTMENT OF HIGHWAYS - ONTARIO
 COORDINATES STA 359 + 82 50' RT. OF CL.
 ELEV. (surface) 282.0 (CL. 101) Datum GEOD.
 BOREHOLE NUMBER 6
 DATE (started) APR. 19, 1958 (finished) APR. 22, 1958
 FIG. No. 1 TYPE JUN. LONGYEAR

HUNTING TECHNICAL AND EXPLORATION SERVICES

BOREHOLE No. 6



x - standard penetr. 2 S.S.
 Δ - vane shear
 o - pocket penetrometer

C - consolidation test
 M - mechanical analysis
 T - triaxial shear
 K - permeability
 U - unconfined compression

SS - split spoon
 S.T. - Shelby tube
 T.W.P. - thin walled piston
 D.B. - diamond bit







SAMPLE CONDITION
 — undisturbed
 — disturbed but represent.
 — fair
 — lost

BORING LOG				FIELD TESTS				LABORATORY TESTS				REMARKS			
SCALE	DEPTH	ELEV.	LOG	SHEAR STRENGTH (TONS PER SQUARE FOOT)		STANDARD PENETRATION TEST (BLOWS PER FOOT)		SAMPLES		ATTERBERG LIMITS wp x - o wl			U undist. r /ft ²	U remolded "	
FT	FT	FT	DESCRIPTION	1/2	1 1/2	No.	COND.	DEPTH FROM TO	TYPE	RECOVERY LENGTH REC. D:ST. DRIV.	PENETRATION RESISTANCE (BLOWS PER FOOT)	25			50
		282.0	ORGANIC TOPSOIL	X		1		0 - 1.5	SS	13/18	2				
5			VERY LOOSE SILTY FINE AND MEDIUM SAND WITH ORGANIC MATTER	X		2		5.0 - 6.5	SS	13/18	2				
10		270.7		X		3		10.0 - 11.5	SS	18/18	3				
15			STIFF GREY INTERBEDDED SILT AND CLAY WITH SOME THIN SAND SEAMS		Δ	4		15.0 - 16.5	ST	13/18	14			2-25	1-50
20					Δ	5		20.0 - 21.5	ST	18/18	10				
25					Δ	6		25.5 - 27.0	ST	13/18	4				
30					Δ	7		30.0 - 31.5	ST	18/18	4				
35			MEDIUM STIFF GREY INTERBEDDED SILT AND CLAY WITH SOME THIN SAND SEAMS		Δ	8		35.0 - 36.5	ST	14/18	1	X	X	0.70	0.21
40		241.5			Δ	9		40.0 - 41.5	ST	18/18	1				
45				X	Δ	10		45.0 - 46.5	SS	18/18	10				
50			MEDIUM DENSE BROWN SAND AND FINE GRAVEL	X	Δ	11		50.0 - 51.5	SS	0/18	20				RECOVERED WITH SIDE SLIT SAMPLER
55				X		12		55.0 - 56.5	SS	0/18	16				" " " "
60		222.0		X		13		60.5 - 62.0	SS	18/18	8				
65			LOOSE TO MEDIUM DENSE GREY MEDIUM SAND TRACE OF SILT	X		14		65.0 - 66.5	SS	16/18	22				
70		214.0	MEDIUM DENSE GREY CLAY AND SILT	X		15		70.0 - 71.0	S.T.	5/5	3/5"				REFUSAL WITH 25 BLOWS
73		209.0	LIMESTONE (?BEDROCK)	X		16		70.0 - 73.0	S.S.	6/7	10/7"				
75			END OF BOREHOLE			17			D.B.	24/24					

JOB No. H 415/58 LOCATION HWY 401 NAPANEE RIVER
 CLIENT DEPARTMENT OF HIGHWAYS - ONTARIO
 COORDINATES STA. 360 + 84 CL
 ELEV. (surface) 282.0 (collar) Datum GEOD.
 BOREHOLE NUMBER 7
 DATE (started) APR 24, 1958 (finished) APR 25, 1958
 RIG No. 1 TYPE JUN. LONGYEAR

HUNTING TECHNICAL AND EXPLORATION SERVICES

BOREHOLE No. 7





 silt
 clay
 sand
 gravel
 peat
 fill





x — standard penetr. 2 S.S.
 Δ — vane shear
 ○ — pocket penetrometer

C — consolidation test
 M — mechanical analysis
 T — triaxial shear
 K — permeability
 U — unconfined compression

S.S. — split spoon
 S.T. — Shelby tube
 T.W.P. — thin walled piston
 D.B. — diamond bit

SAMPLE CONDITION

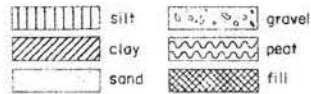
 — undisturbed
 — disturbed but representative
 — fair
 — lost

BORING LOG				FIELD TESTS										LABORATORY TESTS											
SCALE	DEPTH	ELEV.	LOG	SHEAR STRENGTH (TONS PER SQUARE FOOT)		STANDARD PENETRATION TEST (BLOWS PER FOOT)					SAMPLES					ATTERBERG LIMITS		REMARKS							
FT.	FT.	FT.		1/2	1 1/2	20		40		60		No.	COND.	DEPTH FROM TO	TYPE	RECOVERY LENGTH REC. DIST. DRIV.	PENETRATION RESISTANCE (BLOWS PER FOOT)	W.P.	X — O W.I.	U undist. t/ft?	U remolded "	M			
		282.0																							
	4.0	278.0										1		0 - 1.6	SS	9 / 18	3								
	5											2		3.0 - 4.5	SS	15 / 18	1								
	10	272.2										3		6.0 - 7.5	SS	7 / 18	1						✓		
	15											4		9.0 - 10.5	SS	13 / 18	3								
	20											5		14.0 - 15.5	ST	13 / 18	9								
	25											6		19.0 - 20.5	ST	17 / 18	3								
	30											7		24.0 - 25.5	ST	18 / 18	3								
	35	246.0										8		29.0 - 30.5	ST	15 / 18	3					0.75	0.25		
	40	241.5										9		34.0 - 35.5	ST	0 / 18	0								PUSHED BY WEIGHT OF HAMMER
	45											10		39.0 - 40.5	SS	16 / 18	4								SAND ROSE IN CASING
	50																								
	55																								
	60																								

JOB No. H 415/58 LOCATION HWY 401 NAPANEE RIVER
 CLIENT: DEPARTMENT OF HIGHWAYS - ONTARIO
 COORDINATES STA 361 + 84 CL
 ELEV. (Surface) 281.0 (collar) Datum GEOD.
 BOREHOLE NUMBER B
 DATE (started) APR 26, 1958 (finished) APR. 26, 1958
 R-G No. 1 TYPE JUN. LONGYEAR

HUNTING TECHNICAL AND EXPLORATION SERVICES

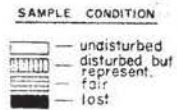
BOREHOLE No. 8



x — standard penetr. 2 S.S.
 Δ — vane shear
 o — pocket penetrometer

C — consolidation test
 M — mechanical analysis
 T — triaxial shear
 K — permeability
 U — unconfined compression

SS — split spoon
 S.T. — Shelby tube
 T.W.P. — thin walled piston
 D.B. — diamond bit

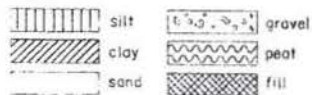


BOHING LOG				FIELD TESTS						LABORATORY TESTS							
SCALE	DEPTH	ELEV.	WATER OBSERVATION	DESCRIPTION	SHEAR STRENGTH (TONS PER SQUARE FOOT)			S A M P L E S				ATTERBERG LIMITS		REMARKS			
					1/2	1	1 1/2	No.	COND.	DEPTH FROM TO	RECOVERY LENGTH REC. DIST. DRIV.	PENETRATION RESISTANCE (BLOWS PER FOOT)	wp		wl		
FT	FT	FT			STANDARD PENETRATION TEST (BLOWS PER FOOT)												
					20	40	60										
		281.0															
	3.5	277.5		SOFT BROWN ORGANIC SILT	X			1		0	1.5	SS	13/18	1			
	5				X			2		3.0	4.5	SS	18/18	1			
	10.0	271.0	G.W.L. MAY 15	VERY LOOSE GREY BROWN SAND, SOME SILT WITH ORGANIC MATTER	X			3		6.0	7.5	SS	14/18	1		✓	
	15				X			4		9.0	10.5	SS	14/18	5			
	20			STIFF GREY INTERBEDDED SILT AND CLAY WITH SOME THIN SAND SEAMS				5	_____	14.0	15.5	ST	16/18	9			
	25							6	_____	19.0	20.5	ST	17/18	8			
	30							7	_____	24.0	25.5	ST	15/18	7			
	35			MEDIUM STIFF GREY INTERBEDDED SILT AND CLAY WITH SOME THIN SAND SEAMS				8	_____	29.0	30.5	ST	15/18	4			
	40	244.0						9	_____	34.0	35.5	ST	16/18	8			
	40.5	240.5		MEDIUM DENSE GREY BROWN FINE SAND AND SILT	X			10		39.0	40.5	SS	16/18	8			SAND ROSE IN CASING
				END OF BOREHOLE													

JOB No. H415/50 LOCATION HWY 401 - NAPANEE RIVER
 CLIENT DEPARTMENT OF HIGHWAYS - ONTARIO
 COORDINATES STA. 356 + 68 2.5 L. OF CL.
 ELEV. (surface) 282.0 (collar) Datum GEOD.
 BOREHOLE NUMBER 9
 DATE (started) MAY 15, 1958 (finished) MAY 16, 1958
 RIG No. 1 TYPE JUN. LONGYEAR

HUNTING TECHNICAL AND EXPLORATION SERVICES

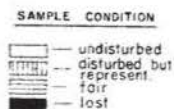
BOREHOLE No. 9



X — standard penetr. 2 S.S.
 Δ — vane shear
 ○ — pocket penetrometer

C — consolidation test
 M — mechanical analysis
 T — triaxial shear
 X — permeability
 U — unconfined compression

SS — split spoon
 ST — Shelby tube
 T.W.P. — thin walled piston
 D.B. — diamond bit



BORING LOG				FIELD TESTS				LABORATORY TESTS				
SCALE FT	DEPTH FT	ELEV. FT	LOG DESCRIPTION	SHEAR STRENGTH (TONS PER SQUARE FOOT)		SAMPLES				REMARKS		
				1/2	1/2	No.	COND	DEPTH FROM TO	TYPE		RECOVERY LENGTH REC. DIST. DRIV.	PENETRATION RESISTANCE (BLOWS PER FOOT)
		282.0	ORGANIC TOPSOIL									
		281.2	LOOSE DARK BROWN PLASTIC SILT AND SAND WITH ORGANIC MATTER			1	SS	2.0 - 3.5	SS	9/18	5	
	5.5	276.5	STIFF GREY BROWN SANDY CLAY			2	SS	5.0 - 6.5	SS	11/18	12	
	7.0	275.0	STIFF — VERY STIFF GREY INTERBEDDED SILT AND CLAY WITH SOME THIN SAND SEAMS			3	ST	8.0 - 9.5	ST	8/18	15	
	10					4	ST	14.0 - 15.5	ST	10/18	20	
	15					5	ST	19.0 - 20.5	ST	15/18	9	
	20					6	ST	24.0 - 25.5	ST	15/18	10	
	25		MEDIUM STIFF			7	ST	29.0 - 30.5	ST	LOST	7	
	30					8	SS	34.0 - 35.5	SS	18/18	11	
	33.0	249.0	MEDIUM DENSE GREY SILT TRACE GRAVEL FRAGMENTS			9	SS	39.0 - 40.0	SS	12/12	53	
	37.0	245.0	DENSE GREY TILL									
	40	242.0	(SAND, SILT WITH FINE GRAVEL)									
	45		END OF BOREHOLE									

8 X CASING 0 - 39'

Appendix C.
Laboratory Testing

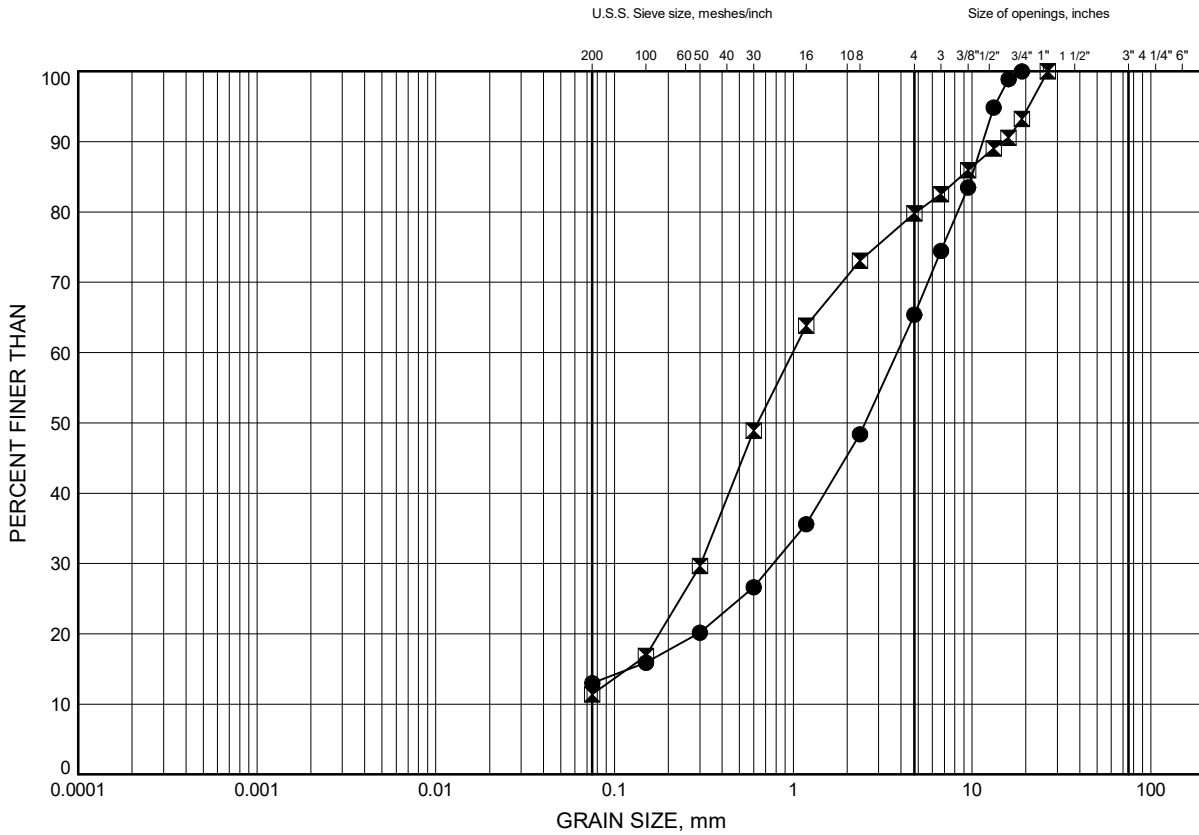
Appendix C.1

Particle Size Analysis, Atterberg Limits Test and Consolidation Test Figures

Hwy 401 Napanee River Bridge
GRAIN SIZE DISTRIBUTION

FIGURE C1

Sand Fill



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	19-2	0.4	88.1
⊠	19-7	1.8	87.0

Date March 2021
 GWP# 4040-20-00



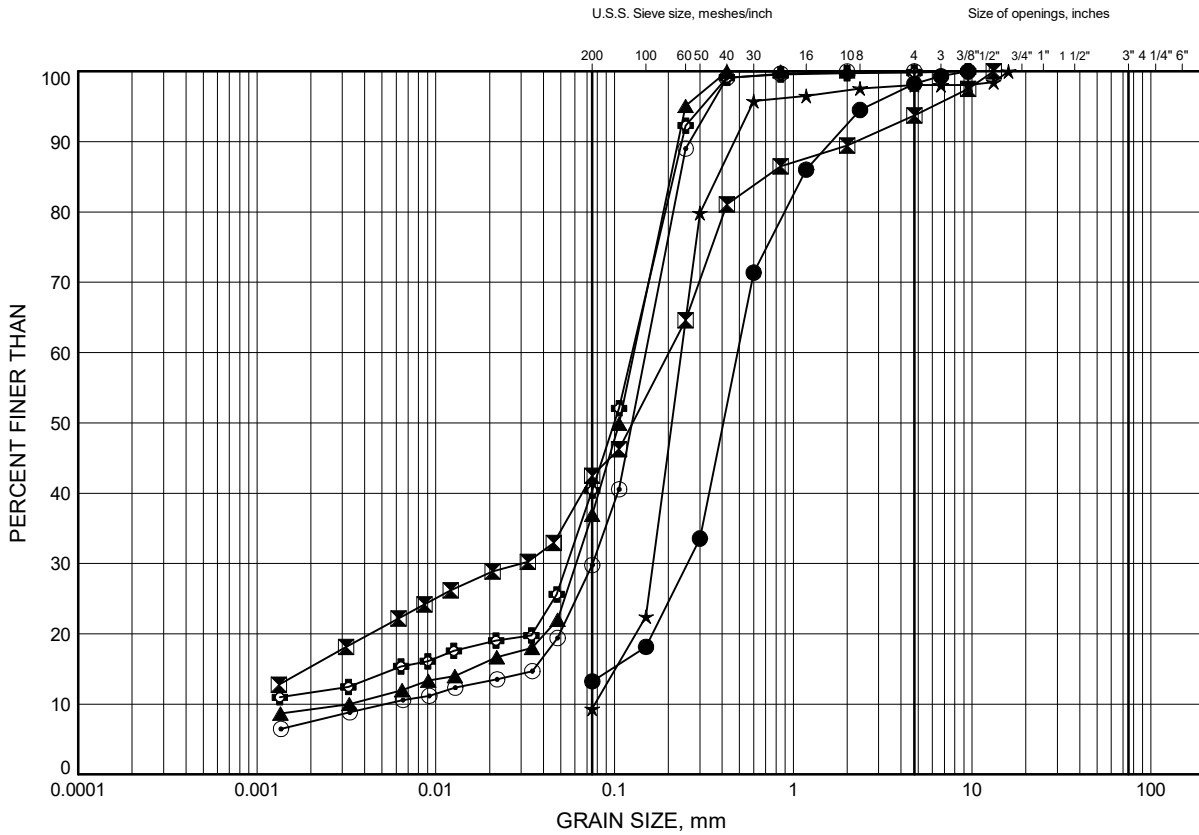
Prep'd SP
 Chkd. PC

GRAIN SIZE DISTRIBUTION - THURBER - 27253 HWY 401 NAPANEE RIVER BRIDGE.GPJ 24/3/21

Hwy 401 Napanee River Bridge GRAIN SIZE DISTRIBUTION

FIGURE C2

Upper Sand to Silty Sand



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	19-2	4.9	83.6
⊠	19-3	3.4	82.9
▲	19-6	1.8	84.9
★	19-6	3.4	83.4
⊙	19-7	4.9	83.9
⊕	19-8	1.8	84.7

Date March 2021
GWP# 4040-20-00

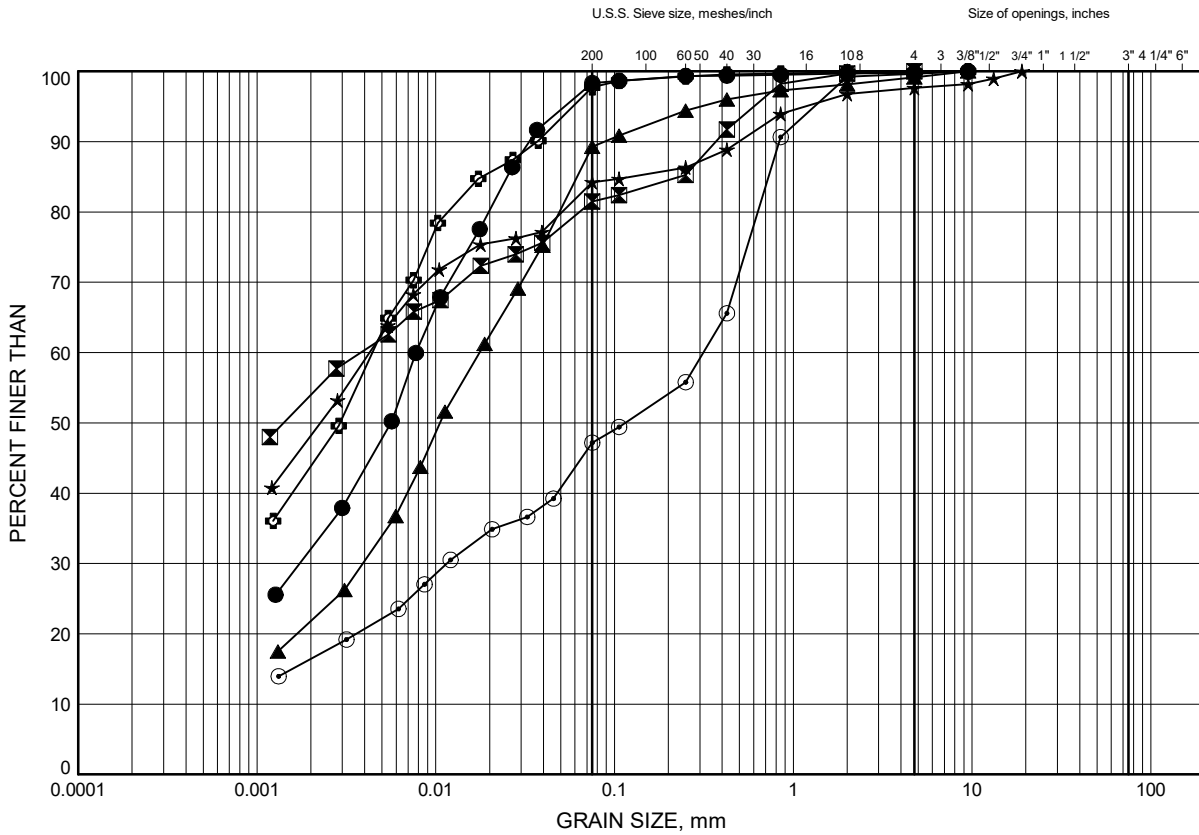


Prep'd SP
Chkd. PC

Hwy 401 Napanee River Bridge GRAIN SIZE DISTRIBUTION

FIGURE C4

Clayey Silt to Clay



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	19-1	9.4	78.3
⊠	19-2	6.4	82.1
▲	19-3	11.0	75.3
★	19-4	4.2	81.6
⊙	19-4	8.5	77.3
⊕	19-5	6.5	79.0

Date March 2021
GWP# 4040-20-00

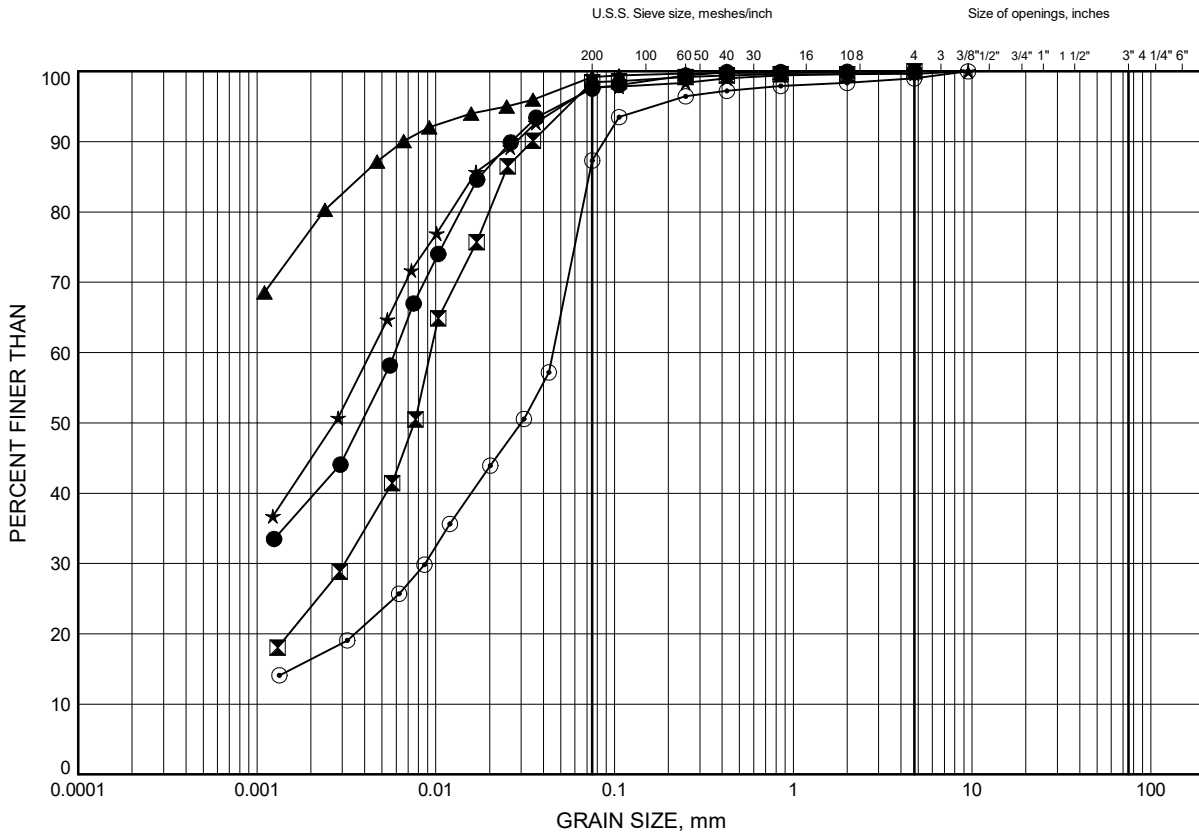


Prep'd SP
Chkd. PC

Hwy 401 Napanee River Bridge GRAIN SIZE DISTRIBUTION

FIGURE C5

Clayey Silt to Clay



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	19-6	7.9	78.8
⊠	19-7	11.0	77.8
▲	19-8	4.9	81.6
★	19-8	5.6	80.9
⊙	19-8	11.0	75.5

Date March 2021
GWP# 4040-20-00

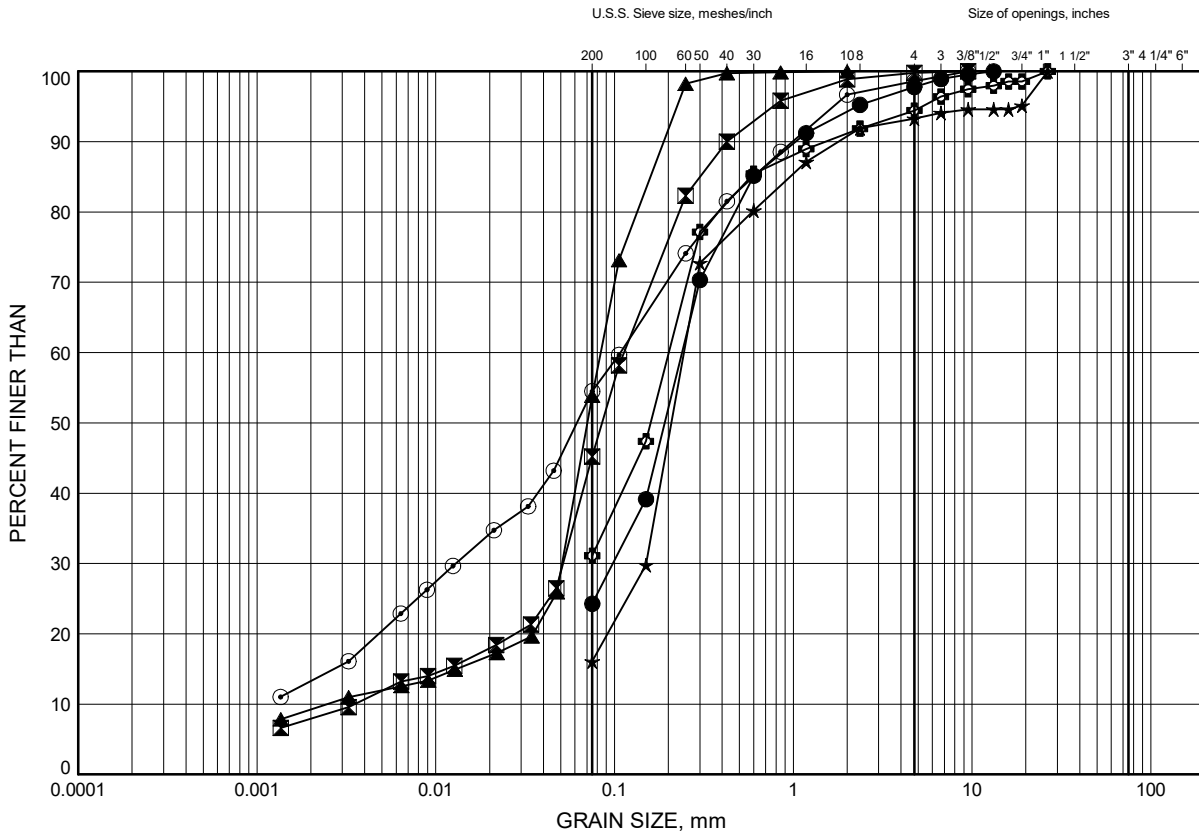


Prep'd SP
Chkd. PC

Hwy 401 Napanee River Bridge
GRAIN SIZE DISTRIBUTION

FIGURE C6

Lower Sandy Silt to Silty Sand



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	19-2	17.0	71.5
⊠	19-3	15.5	70.7
▲	19-3	20.1	66.1
★	19-4	13.7	72.1
⊙	19-5	17.2	68.3
⊕	19-6	12.5	74.3

Date March 2021
 GWP# 4040-20-00



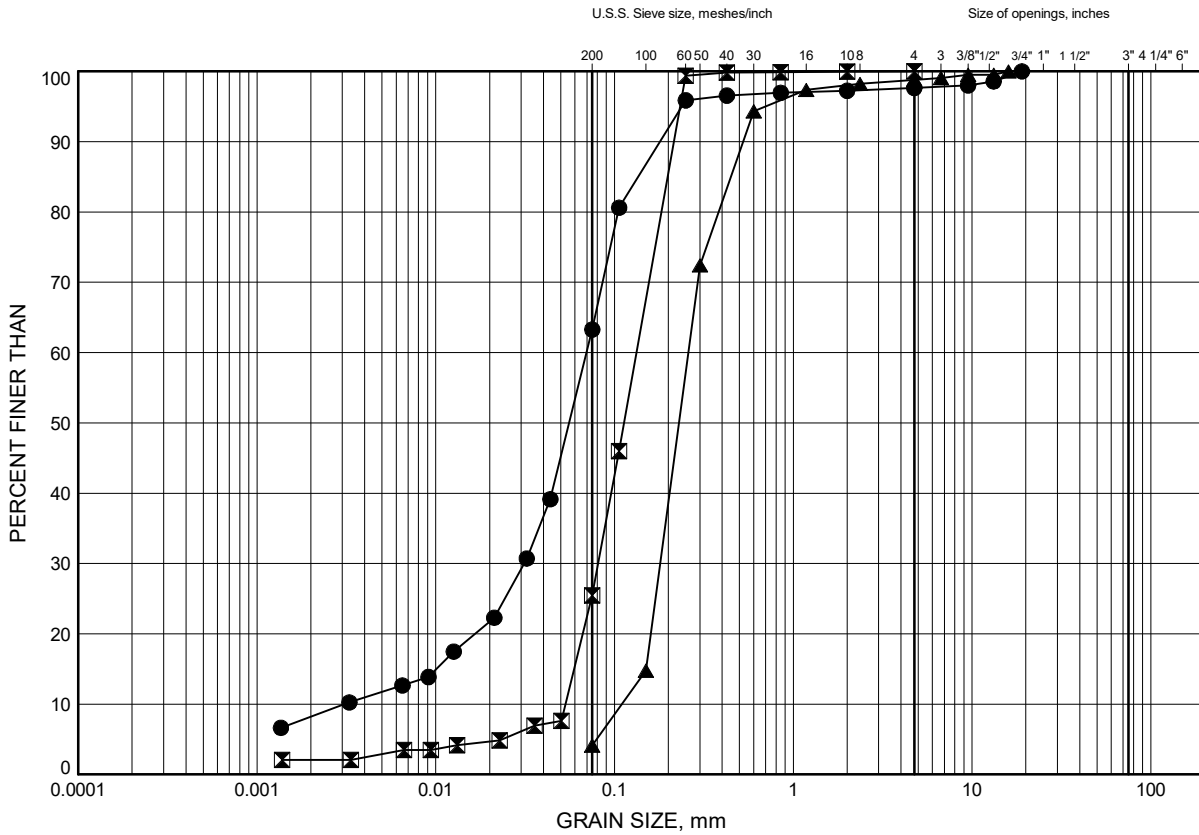
Prep'd SP
 Chkd. PC

GRAIN SIZE DISTRIBUTION - THURBER - 27253 HWY 401 NAPANEE RIVER BRIDGE.GPJ 24/3/21

Hwy 401 Napanee River Bridge
GRAIN SIZE DISTRIBUTION

FIGURE C7

Lower Sandy Silt to Silty Sand



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	19-7	15.5	73.3
⊠	19-8	12.5	74.0
▲	19-8	18.6	67.9

Date March 2021
 GWP# 4040-20-00

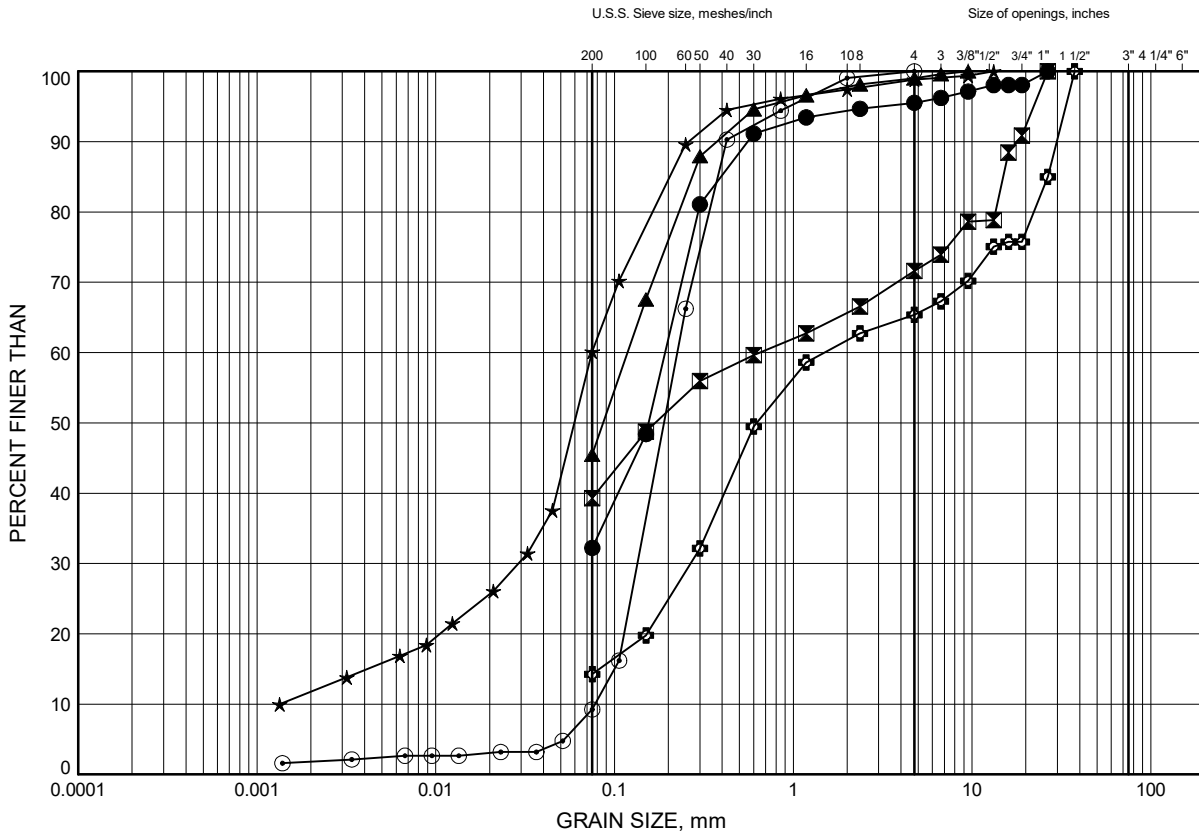


Prep'd SP
 Chkd. PC

Hwy 401 Napanee River Bridge
GRAIN SIZE DISTRIBUTION

FIGURE C8

Silty Sand to Silty Sand Till



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	19-1	15.5	72.2
⊠	19-1	18.4	69.3
▲	19-2	21.5	67.0
★	19-4	15.1	70.7
⊙	19-5	20.3	65.2
⊕	19-7	25.5	63.3

Date March 2021
 GWP# 4040-20-00



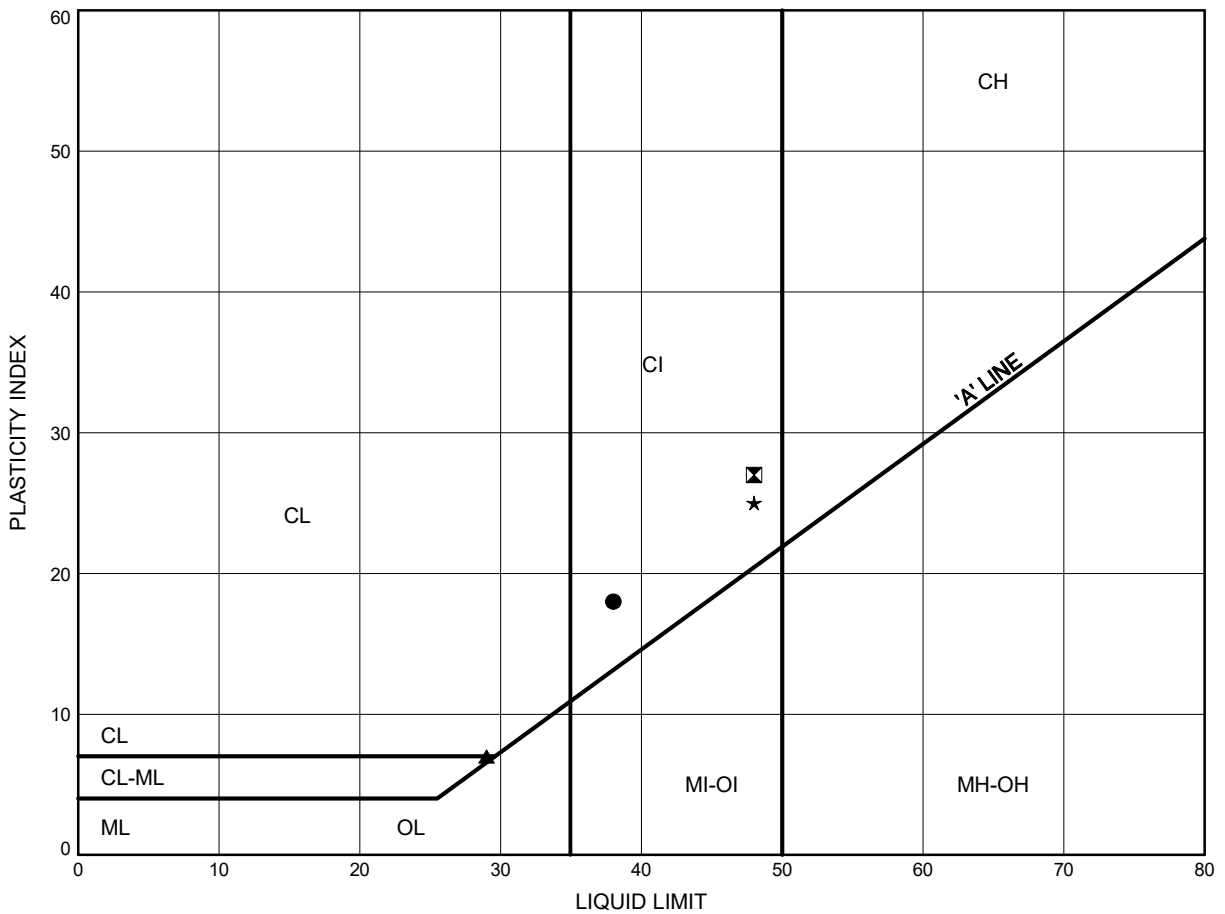
Prep'd SP
 Chkd. PC

GRAIN SIZE DISTRIBUTION - THURBER - 27253 HWY 401 NAPANEE RIVER BRIDGE.GPJ 24/3/21

Hwy 401 Napanee River Bridge
ATTERBERG LIMITS TEST RESULTS

FIGURE C9

Clayey Silt with Sand to Clay with Sand to Clayey Gravel with Sand



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	19-1	1.8	85.9
⊠	19-1	5.5	82.2
▲	19-3	1.8	84.4
★	19-5	2.0	83.5

THURBALT 27253 HWY 401 NAPANEE RIVER BRIDGE.GPJ 24/3/21

Date March 2021
 GWP# 4040-20-00

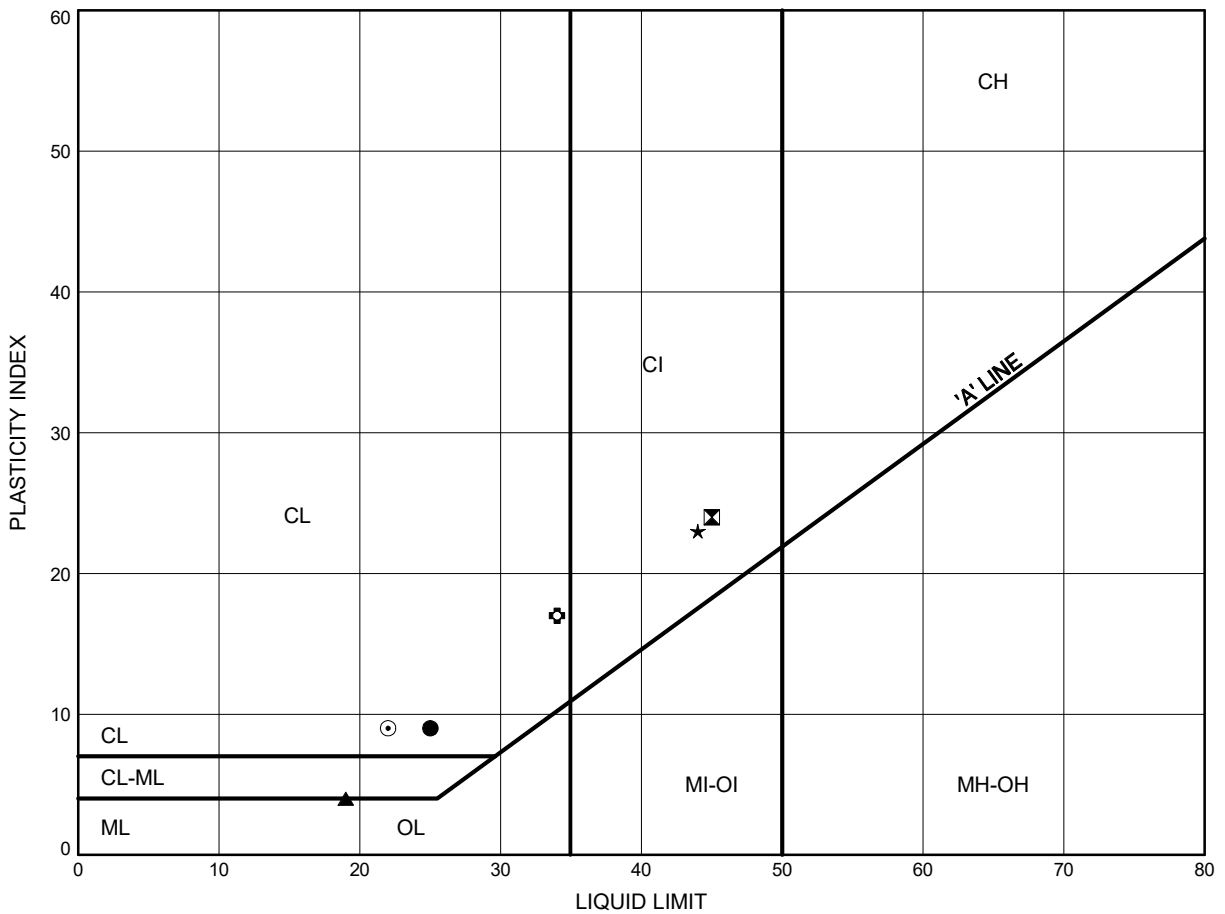


Prep'd SP
 Chkd. PC

Hwy 401 Napanee River Bridge
ATTERBERG LIMITS TEST RESULTS

FIGURE C10

Clayey Silt to Clay



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	19-1	9.4	78.3
⊠	19-2	6.4	82.1
▲	19-3	11.0	75.3
★	19-4	4.2	81.6
⊙	19-4	8.5	77.3
⊕	19-5	6.5	79.0

Date March 2021
 GWP# 4040-20-00



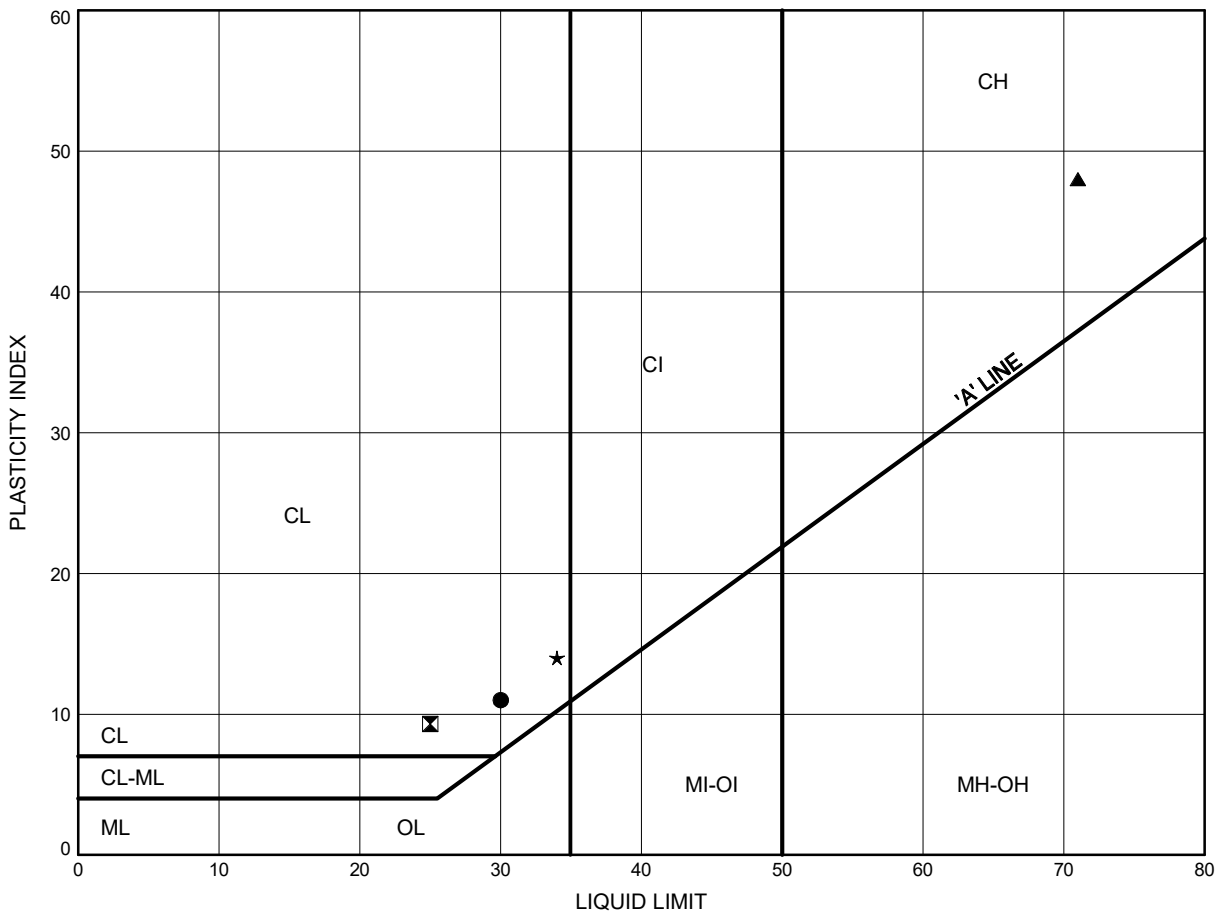
Prep'd SP
 Chkd. PC

THURBALT 27253 HWY 401 NAPANEE RIVER BRIDGE.GPJ 24/3/21

Hwy 401 Napanee River Bridge
ATTERBERG LIMITS TEST RESULTS

FIGURE C11

Clayey Silt to Clay



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	19-6	7.9	78.8
⊠	19-7	11.0	77.8
▲	19-8	4.9	81.6
★	19-8	5.6	80.9

THURBALT 27253 HWY 401 NAPANEE RIVER BRIDGE.GPJ 24/3/21

Date March 2021
 GWP# 4040-20-00



Prep'd SP
 Chkd. PC



Stantec Consulting Ltd
100 A&B – 2781 Lancaster Rd
Ottawa, ON K1B 1A7
Tel: (613) 738-6075
Fax: (613) 738-6067

Stantec

February 10, 2020
File: 122410864

Attention: Thurber Engineering, File #27253

Reference: ASTM D2974 Organic Content

The following table summarizes one Organic Content result.

Location	Source	Depth	Organic Content (%)
Highway 401, Napanee	19-3 SS4	7'6''-9'6''	3.9

Sincerely,

Stantec Consulting Ltd.

Brian Prevost

Brian Prevost
Laboratory Supervisor
Tel: 613-738-6075
Fax: 613-738-6067
brian.prevost@stantec.com



Stantec Consulting Ltd.
400 - 1331 Clyde Avenue, Ottawa ON K2C 3G4

January 9, 2020
File: 122410864

Attention: Deanna Pizycki, M. Eng., P.Eng.

Thurber Engineering Ltd.
104 – 2460 Lancaster Road
Ottawa, Ontario, Canada, K1B 4S5
Tel: 613-274-2121 ext. 7106
E-mail: dpizycki@thurber.ca

Dear Ms. Pizycki,

**Reference: Consolidation Test Results for HWY 401-Napanee River Bridge Project,
Thurber Engineering Ltd., File #27253: BH 19-7, ST10, sampled on November 18, 2019**

This letter presents the results of one-dimensional consolidation tests carried out on the above referenced sample in accordance with ASTM D2435/D2435M - 11. The test results are provided in the attached tables and figures.

This letter provides test results only and does not constitute any interpretation or engineering recommendations with respect to material suitability or specification compliance.

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.

Regards,

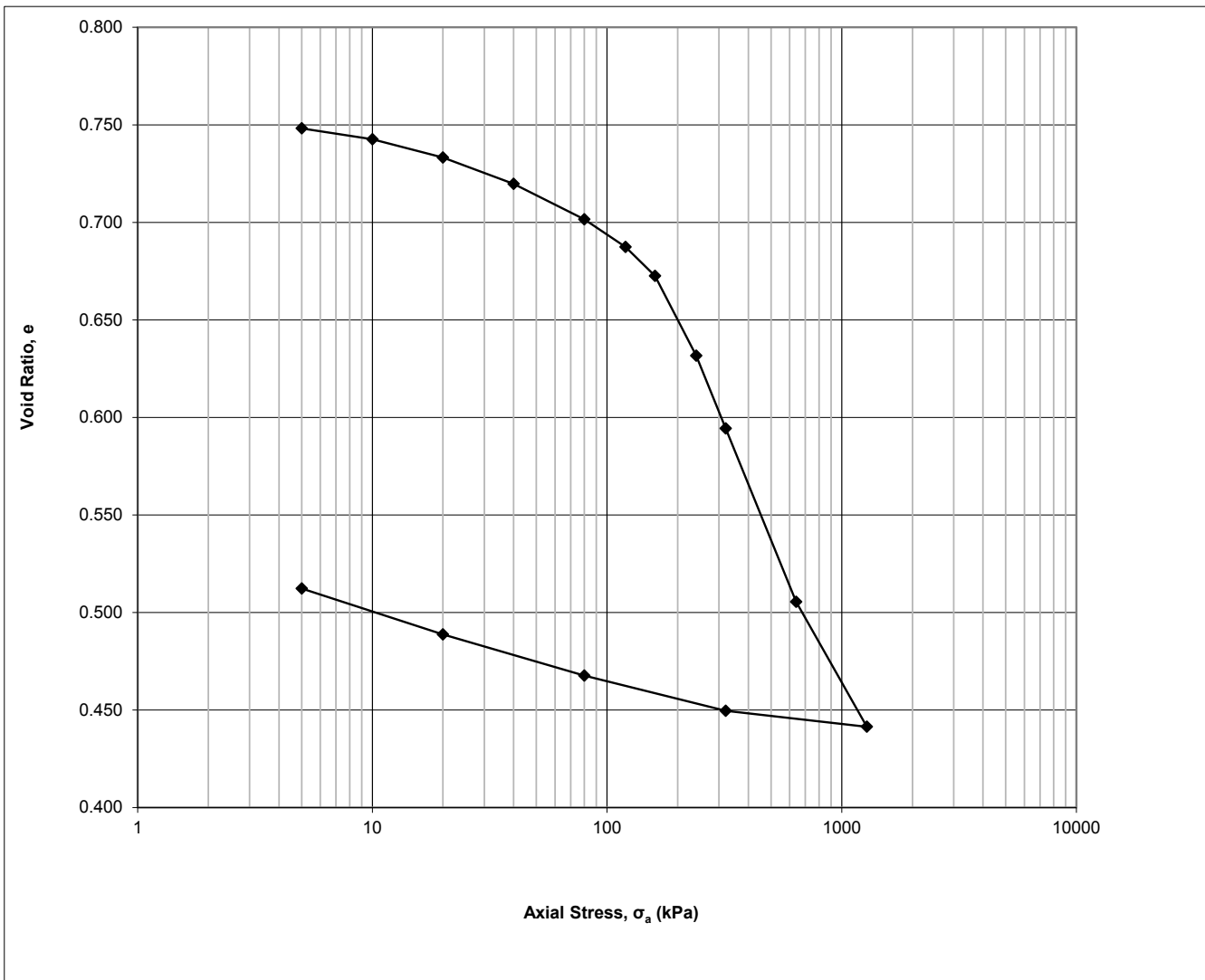
STANTEC CONSULTING LTD.

Rajib Dey
Geotechnical Engineer
Phone: 905 944 6190
Fax: 905 474 9889
Rajib.Dey@stantec.com

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Project
Project No.
Borehole No.
Sample No.
Sample Depth

Thurber Engineering, File#27253
122410864
BH 19-7
ST 10
35-37 ft



One-Dimensional Consolidation Test using Incremental Loading
ASTM D2435/D2435M - 11

January 9, 2020
 January 9, 2020

Date: Date:
 D. Boateng R. Dey

Checked by:
 Approved by:

Filename: C:\Users\dboateng\Desktop\Reports for Deanna\F
 Date: January 9, 2020

Specimen Details

Project Name	Thurber Engineering, File#27253
Project Location	HWY 401 Napanee River Bridge, ON
Borehole	BH 19-7
Sample No.	ST 10
Depth	35-37 ft
Sample Date	November 18, 2019
Test Number	One
Technician Name	Daniel Boateng

Soil Description & Classification

Clayey silt, trace sand, grey, varved, moist - CL	
Specific Gravity of Solids	2.758
Liquid Limit %	25.0
Plastic Limit %	15.7
Plasticity Index %	9.3
Average water content of trimmings %	29.37
Additional Notes (information source, occurrence and size of large isolated particles etc.)	
Piece of rock retained on the 37.5mm sieve observed at the top part of Shelby tube	

Initial Specimen Conditions

Height	mm	20.00
Diameter	mm	50.00
Area	mm ²	1963
Volume	mm ³	39270
Mass	g	78.76
Dry Mass	g	60.88
Density	Mg/m ³	2.006
Dry Density	Mg/m ³	1.550
Water Content	%	29.37
Degree of Saturation	%	104.0
Height of Solids	mm	11.24
Initial Void Ratio		0.779

Final Specimen Conditions

Water Content	%	21.52
Final Void Ratio		0.512
Height	mm	17.00

One-Dimensional Consolidation Test using Incremental Loading
ASTM D2435/D2435M - 11

Specimen Details

Project Name	Thurber Engineering, File#27253
Project Location	HWY 401 Napanee River Bridge, ON
Borehole	BH 19-7
Sample No.	ST 10
Depth	35-37 ft
Sample Date	November 18, 2019
Test Number	One
Technician Name	Daniel Boateng

Test Procedure

Date Started	December 2, 2019
Date Finished	December 19, 2019
Machine Number	Frame D
Cell Number	D
Ring Number	D
Trimming Procedure	Turntable/ Cutting Ring
Moisture Condition	Inundated
Axial Stress at Inundation	5 kPa
Water Used	De-aired Tap Water
Test Method	A
Interpretation Procedure for c_v	2

All Departures from Outlined ASTM D2435/D2435M-11 Procedure

--

Calculations

Load Increment	Increment Duration min	Axial Stress σ_a kPa	Corrected Deformation ΔH mm	Specimen Height H mm	Axial Strain ϵ_a %	Void Ratio e
Seating	0.0	0	0.0000	20.0000	0.00	0.779
1	1440.0	5	0.3470	19.6530	1.74	0.748
2	1440.0	10	0.4100	19.5900	2.05	0.743
3	1440.0	20	0.5145	19.4855	2.57	0.733
4	1440.0	40	0.6665	19.3335	3.33	0.720
5	1440.0	80	0.8702	19.1298	4.35	0.702
6	1440.0	120	1.0298	18.9702	5.15	0.687
7	1440.0	160	1.1976	18.8024	5.99	0.672
8	1440.0	240	1.6561	18.3439	8.28	0.632
9	1440.0	320	2.0756	17.9244	10.38	0.594
10	1440.0	640	3.0752	16.9248	15.38	0.505
11	1440.0	1280	3.7955	16.2045	18.98	0.441
12	1440.0	320	3.7032	16.2968	18.52	0.450
13	1440.0	80	3.5002	16.4998	17.50	0.468
14	1440.0	20	3.2631	16.7369	16.32	0.489
15	1440.0	5	2.9986	17.0014	14.99	0.512

January 9, 2020
January 9, 2020

Date: Date:
D. Boateng R. Dey

Checked by: Approved by:

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Filename: Date:
January 9, 2020

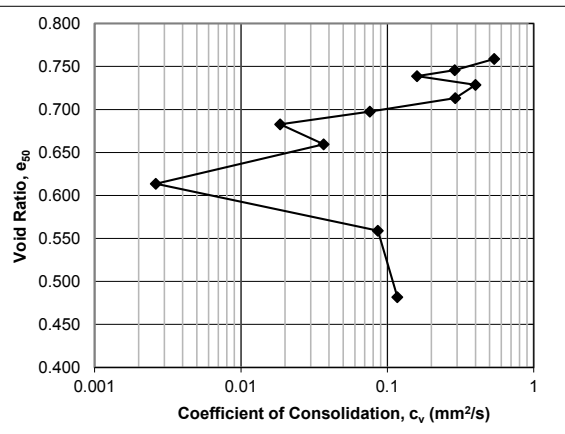
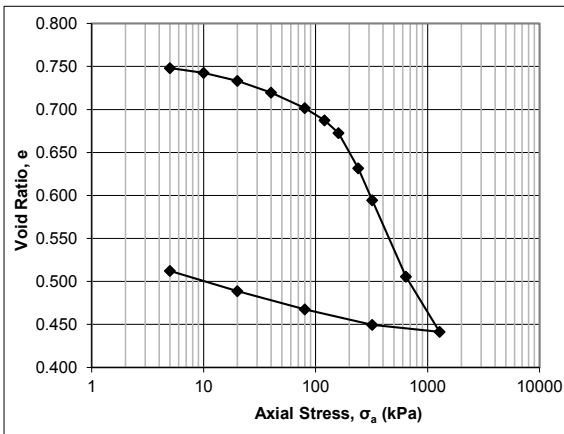
One-Dimensional Consolidation Test using Incremental Loading
ASTM D2435/D2435M - 11

Specimen Details

Job Ref.	Thurber Engineering, File#27253
Job Location	HWY 401 Napanee River Bridge, ON
Borehole	BH 19-7
Sample No.	ST 10
Depth	35-37 ft
Sample Date	November 18, 2019
Test Number	One
Technician Name	Daniel Boateng

Calculations

Load Increment	Axial Stress σ_a , average kPa	Calculated using Interpretation Procedure 2				Interpretation Procedure 1		Interpretation Procedure 2	
		Corrected Deformation ΔH_{50} mm	Specimen Height H_{50} mm	Axial Strain $\epsilon_{a,50}$ %	Void Ratio e_{50}	Time t_{50} sec	Coeff. Consol. c_v mm ² /s	Time t_{90} sec	Coeff. Consol. c_v mm ² /s
Seating	0								
1	3	0.2275	19.7725	1.14	0.759			154	5.39E-01
2	8	0.3735	19.6265	1.87	0.746			283	2.89E-01
3	15	0.4531	19.5469	2.27	0.739			508	1.60E-01
4	30	0.5676	19.4324	2.84	0.729			200	4.00E-01
5	60	0.7387	19.2613	3.69	0.713			270	2.91E-01
6	100	0.9167	19.0833	4.58	0.697			1014	7.61E-02
7	140	1.0820	18.9180	5.41	0.683			4096	1.85E-02
8	200	1.3433	18.6567	6.72	0.660			2009	3.67E-02
9	280	1.8604	18.1396	9.30	0.614			26573	2.63E-03
10	480	2.4727	17.5273	12.36	0.559			754	8.63E-02
11	960	3.3408	16.6592	16.70	0.482			503	1.17E-01
12	800	3.7347	16.2653	18.67	0.447				
13	200	3.5886	16.4114	17.94	0.460				
14	50	3.3960	16.6040	16.98	0.477				
15	13	3.1584	16.8416	15.79	0.498				



January 9, 2020
January 9, 2020

Date:
Date:

D. Boateng
R. Dey

Checked by:
Approved by:

C:\Users\dboateng\Desktop\Reports for Deanna\BH19-7 Napanee
January 9, 2020

Filename:
Date:



Project No.: 122410864

Project Name: Thurber Engineering, File# 27253

Photo Log

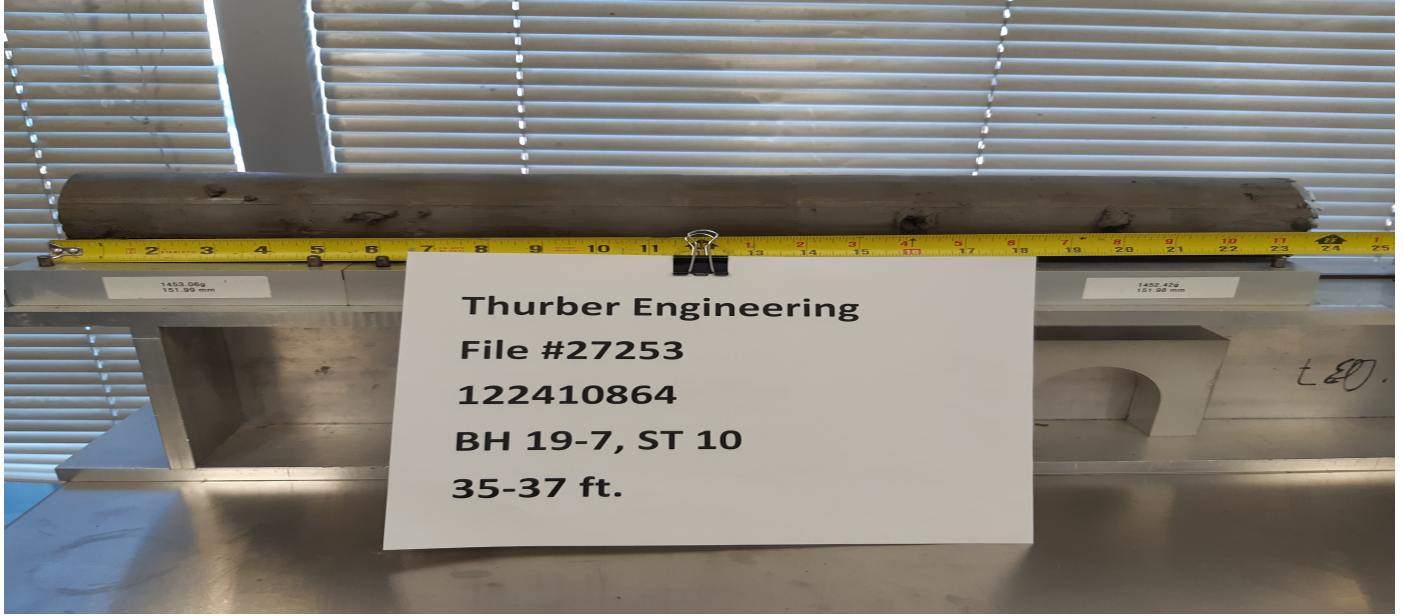


Photo No.:

1

Borehole: BH 19-7 ST 10

Depth: 35 – 37 ft

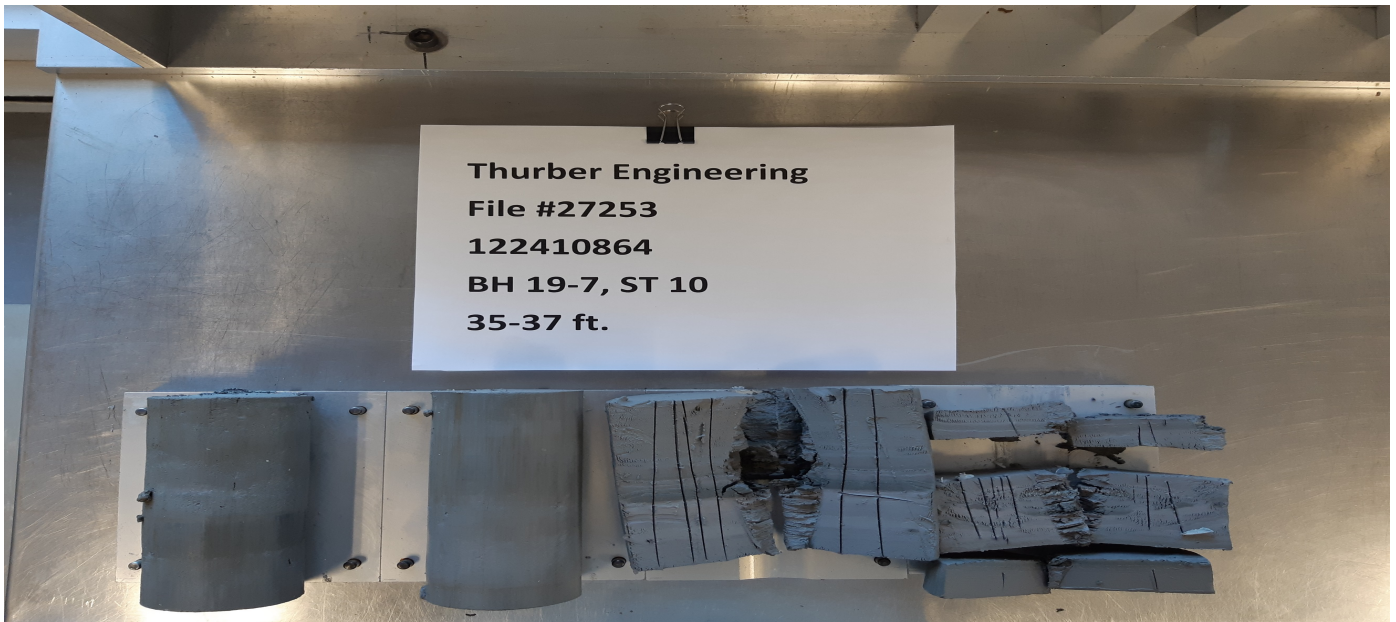


Photo No.:

2

Borehole: BH 19-7 ST 10

Depth: 35 – 37 ft



Stantec Consulting Ltd.
400 - 1331 Clyde Avenue, Ottawa ON K2C 3G4

February 26, 2020
File: 122410864

Attention: Deanna Pizycki, M. Eng., P.Eng.

Thurber Engineering Ltd.
104 – 2460 Lancaster Road
Ottawa, Ontario, Canada, K1B 4S5
Tel: 613-274-2121 ext. 7106
E-mail: dpizycki@thurber.ca

Dear Ms. Pizycki,

**Reference: Consolidation Test Results for HWY 401-Napanee River Bridge Project,
Thurber Engineering Ltd., File #27253: BH 19-8, ST 7, sampled on January 14, 2020**

This letter presents the results of one-dimensional consolidation tests carried out on the above referenced sample in accordance with ASTM D2435/D2435M - 11. The test results are provided in the attached tables and figures.

This letter provides test results only and does not constitute any interpretation or engineering recommendations with respect to material suitability or specification compliance.

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.

Regards,

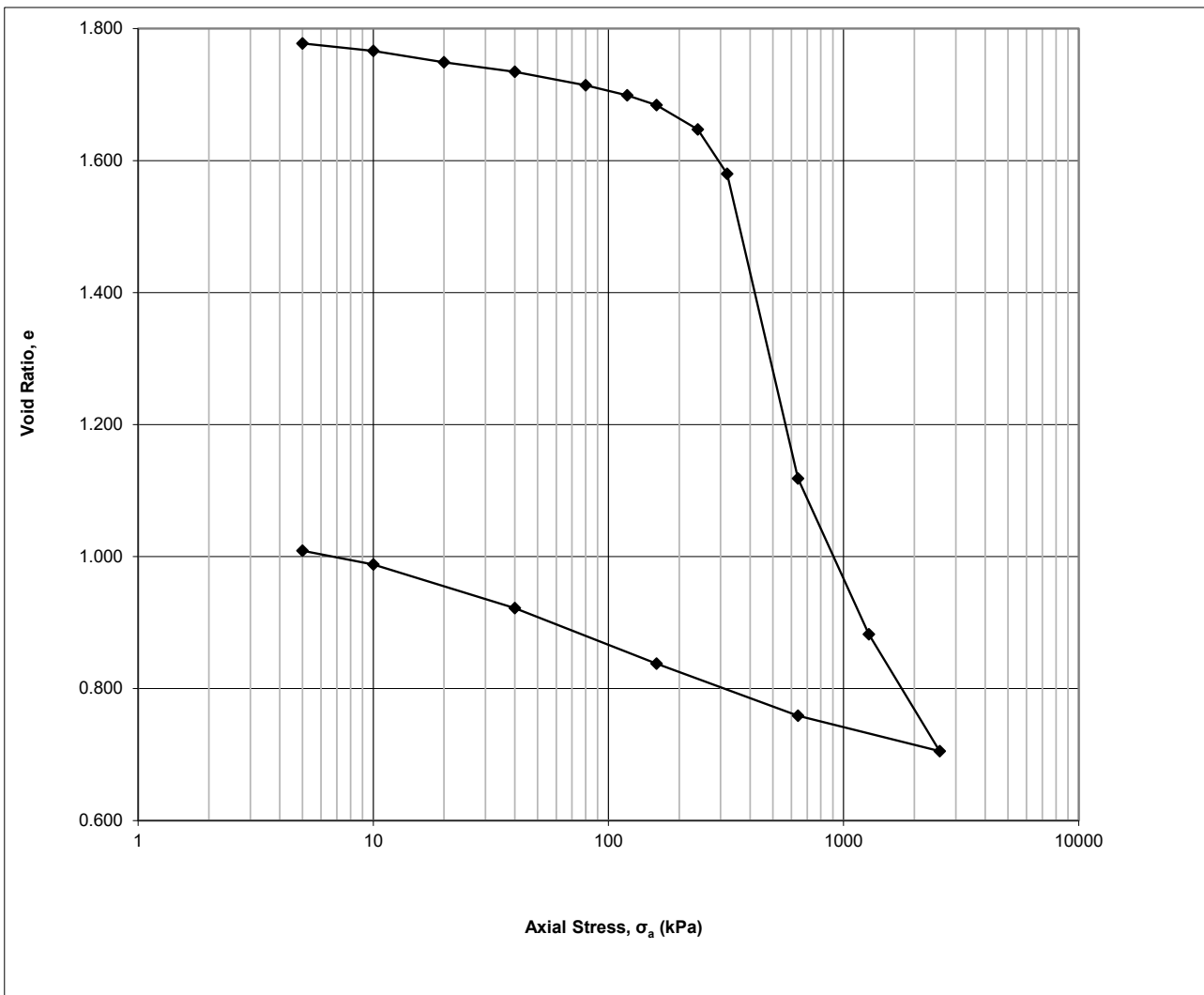
STANTEC CONSULTING LTD.

Rajib Dey Ph.D., P.Eng.
Geotechnical Engineer
Phone: 905 944 6190
Fax: 905 474 9889
Rajib.Dey@stantec.com

v:\01216\active\laboratory_standing_offers\2020 laboratory standing offers\122410864 thurber engineering\jan 29, con, lim,hyd,sg, file# 27253\122410864_let_consolidation_bh19-8_st 7.docx

Project
Project No.
Borehole No.
Sample No.
Sample Depth

Thurber Engineering, File#27253
122410864
BH 19-8
ST 7
15-17 ft





One-Dimensional Consolidation Test using Incremental Loading
ASTM D2435/D2435M - 11

February 26, 2020
February 26, 2020

Date: Date:
D. Boateng
Rajib Dey

Checked by:
Approved by:

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February 26, 2020

Filename:
Date:

Specimen Details

Project Name	Thurber Engineering, File#27253
Project Location	HWY 401 Napanee River Bridge, ON
Borehole	BH 19-8
Sample No.	ST 7
Depth	15-17 ft
Sample Date	January 14, 2020
Test Number	One
Technician Name	Daniel Boateng

Soil Description & Classification

Silty clay, brown/grey, dessicated/varved, moist - CH	
Specific Gravity of Solids	2.748
Liquid Limit %	71.5
Plastic Limit %	22.6
Plasticity Index %	48.9
Average water content of trimmings %	65.55
Additional Notes (information source, occurrence and size of large isolated particles etc.)	

Initial Specimen Conditions

Height	mm	20.00
Diameter	mm	50.00
Area	mm ²	1963
Volume	mm ³	39270
Mass	g	63.67
Dry Mass	g	38.46
Density	Mg/m ³	1.621
Dry Density	Mg/m ³	0.979
Water Content	%	65.55
Degree of Saturation	%	99.7
Height of Solids	mm	7.13
Initial Void Ratio		1.806

Final Specimen Conditions

Water Content	%	39.55
Final Void Ratio		1.009
Height	mm	14.32

One-Dimensional Consolidation Test using Incremental Loading
ASTM D2435/D2435M - 11

Specimen Details

Project Name	Thurber Engineering, File#27253
Project Location	HWY 401 Napanee River Bridge, ON
Borehole	BH 19-8
Sample No.	ST 7
Depth	15-17 ft
Sample Date	January 14, 2020
Test Number	One
Technician Name	Daniel Boateng

Test Procedure

Date Started	February 3, 2020
Date Finished	February 21, 2020
Machine Number	Frame A
Cell Number	A
Ring Number	A
Trimming Procedure	Turntable/ Cutting Ring
Moisture Condition	Inundated
Axial Stress at Inundation	5 kPa
Water Used	De-aired Tap Water
Test Method	A
Interpretation Procedure for c_v	2

All Departures from Outlined ASTM D2435/D2435M-11 Procedure

--

Calculations

Load Increment	Increment Duration min	Axial Stress σ_a kPa	Corrected Deformation ΔH mm	Specimen Height H mm	Axial Strain ϵ_a %	Void Ratio e
Seating	0.0	0	0.0000	20.0000	0.00	1.806
1	1440.0	5	0.2022	19.7978	1.01	1.778
2	1440.0	10	0.2819	19.7181	1.41	1.766
3	1440.0	20	0.4049	19.5951	2.02	1.749
4	1440.0	40	0.5077	19.4923	2.54	1.735
5	1440.0	80	0.6539	19.3461	3.27	1.714
6	1440.0	120	0.7631	19.2369	3.82	1.699
7	1440.0	160	0.8678	19.1322	4.34	1.684
8	1440.0	240	1.1311	18.8689	5.66	1.647
9	1440.0	320	1.6121	18.3879	8.06	1.580
10	1440.0	640	4.9025	15.0975	24.51	1.118
11	1440.0	1280	6.5846	13.4154	32.92	0.882
12	1440.0	2560	7.8450	12.1550	39.23	0.705
13	1440.0	640	7.4638	12.5362	37.32	0.759
14	1440.0	160	6.9010	13.0990	34.51	0.838
15	1440.0	40	6.2996	13.7004	31.50	0.922
16	1440.0	10	5.8291	14.1709	29.15	0.988
17	1440.0	5	5.6819	14.3181	28.41	1.009

February 26, 2020
February 26, 2020

Date: Date:
D. Boateng
Rajib Dey

Checked by:
Approved by:

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Filename:
Date:

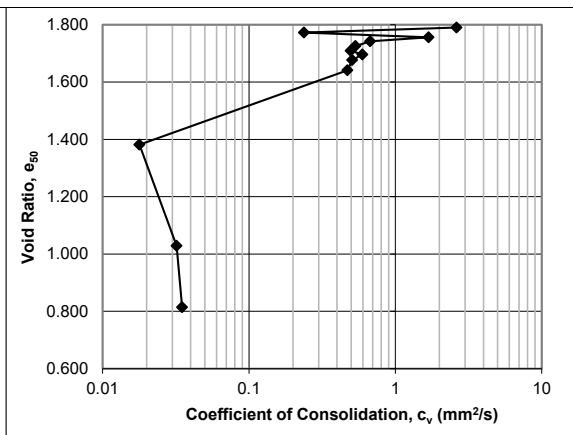
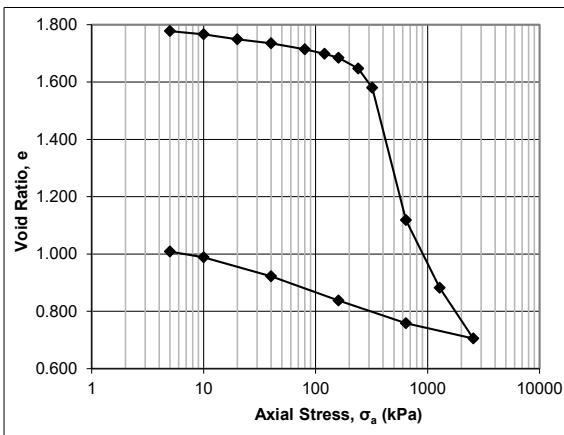
One-Dimensional Consolidation Test using Incremental Loading
ASTM D2435/D2435M - 11

Specimen Details

Job Ref.	Thurber Engineering, File#27253
Job Location	HWY 401 Napanee River Bridge, ON
Borehole	BH 19-8
Sample No.	ST 7
Depth	15-17 ft
Sample Date	January 14, 2020
Test Number	One
Technician Name	Daniel Boateng

Calculations

Load Increment	Axial Stress $\sigma_{a, average}$ kPa	Calculated using Interpretation Procedure 2				Interpretation Procedure 1		Interpretation Procedure 2	
		Corrected Deformation ΔH_{50} mm	Specimen Height H_{50} mm	Axial Strain $\epsilon_{a,50}$ %	Void Ratio e_{50}	Time t_{50} sec	Coeff. Consol. c_v mm ² /s	Time t_{90} sec	Coeff. Consol. c_v mm ² /s
Seating	0								
1	3	0.1124	19.8876	0.56	1.790			32	2.62E+00
2	8	0.2375	19.7625	1.19	1.773			348	2.38E-01
3	15	0.3557	19.6443	1.78	1.756			49	1.69E+00
4	30	0.4541	19.5459	2.27	1.742			120	6.72E-01
5	60	0.5760	19.4240	2.88	1.725			150	5.33E-01
6	100	0.6895	19.3105	3.45	1.709			160	4.95E-01
7	140	0.7867	19.2133	3.93	1.696			131	5.97E-01
8	200	0.9201	19.0799	4.60	1.677			152	5.08E-01
9	280	1.1730	18.8270	5.87	1.641			160	4.71E-01
10	480	3.0272	16.9728	15.14	1.381			3417	1.79E-02
11	960	5.5414	14.4586	27.71	1.028			1382	3.21E-02
12	1920	7.0676	12.9324	35.34	0.814			1017	3.49E-02
13	1600	7.6844	12.3156	38.42	0.728				
14	400	7.1878	12.8122	35.94	0.797				
15	100	6.5861	13.4139	32.93	0.882				
16	25	6.2891	13.7109	31.45	0.924				
17	8	5.8266	14.1734	29.13	0.988				



February 26, 2020
February 26, 2020

Date:
Date:

D. Boateng
Rajib Dey

Checked by:
Approved by:

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February 26, 2020

Filename:
Date:



Project No.: 122410864

Project Name: Thurber Engineering, File# 27253

Photo Log

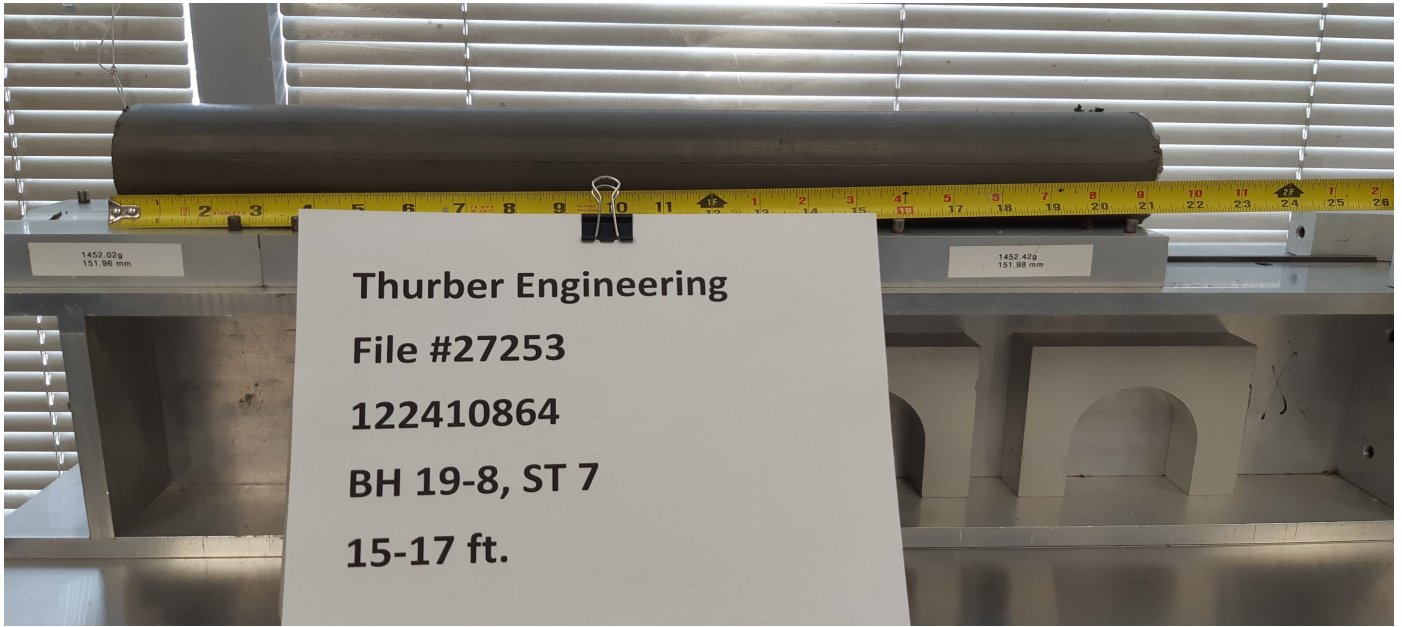


Photo No.: 1

Borehole: 19-8 ST 7

Depth: 15 – 17 ft



Photo No.: 2

Borehole: 19-8 ST 7

Depth: 15 – 17 ft



Project No.: 122410864

Project Name: Thurber Engineering, File# 27253

Photo Log



Photo No.: 3

Borehole: 19-8 ST 7

Depth: 15 – 17 ft



Photo No.: 4

Borehole: 19-8 ST 7

Depth: 15 – 17 ft

Appendix C.2
Analytical Testing Results

Certificate of Analysis
 Client: Thurber Engineering Ltd.
 Client PO:

Report Date: 05-Dec-2019

Order Date: 2-Dec-2019

Project Description: 27253

Client ID:	19-2 SS8-25'-27'	19-7 SS7 20'-22'	-	-
Sample Date:	17-Nov-19 09:00	18-Nov-19 09:00	-	-
Sample ID:	1949048-01	1949048-02	-	-
MDL/Units	Soil	Soil	-	-

Physical Characteristics

% Solids	0.1 % by Wt.	72.2	63.9	-	-
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General Inorganics

Conductivity	5 uS/cm	307	518	-	-
pH	0.05 pH Units	8.37	7.90	-	-
Resistivity	0.10 Ohm.m	32.6	19.3	-	-

Anions

Chloride	5 ug/g dry	48	271	-	-
Sulphate	5 ug/g dry	74	92	-	-



SGS Canada Inc.

P.O. Box 4300 - 185 Concession St.
Lakefield - Ontario - K0L 2H0
Phone: 705-652-2000 FAX: 705-652-6365

06-December-2019

Paracel Laboratories

Attn : Dale Robertson

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

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

Date Rec. : 03 December 2019
LR Report: CA14091-DEC19
Reference: Project#: 1949048

Copy: #1

CERTIFICATE OF ANALYSIS

Final Report

Sample ID	Sample Date & Time	Sulphide %
1: Analysis Start Date		05-Dec-19
2: Analysis Start Time		15:22
3: Analysis Completed Date		05-Dec-19
4: Analysis Completed Time		15:40
5: QC - Blank		< 0.02
6: QC - STD % Recovery		121%
7: QC - DUP % RPD		19%
8: RL		0.02
9: 19-2 SS8 25'-27'	17-Nov-19	0.03
10: 19-7 SS7 20'-22'	18-Nov-19	0.09

RL - SGS Reporting Limit

Kimberley Didsbury
Project Specialist,
Environment, Health & Safety

Certificate of Analysis
 Client: Thurber Engineering Ltd.
 Client PO: 27253

Report Date: 05-Feb-2020

Order Date: 30-Jan-2020

Project Description: Hwy 401, Napanee River Bridge

Client ID:	19-3 SS4 7'6"-9'6"	19-6 SS3 5'-7'	-	-
Sample Date:	23-Jan-20 09:00	21-Jan-20 09:00	-	-
Sample ID:	2005368-01	2005368-02	-	-
MDL/Units	Soil	Soil	-	-

Physical Characteristics

% Solids	0.1 % by Wt.	68.3	78.6	-	-
----------	--------------	------	------	---	---

General Inorganics

Conductivity	5 uS/cm	1070	239	-	-
pH	0.05 pH Units	7.41	7.44	-	-
Resistivity	0.10 Ohm.m	9.33	41.8	-	-

Anions

Chloride	5 ug/g dry	500	50	-	-
Sulphate	5 ug/g dry	72	12	-	-



SGS Canada Inc.

P.O. Box 4300 - 185 Concession St.
Lakefield - Ontario - K0L 2H0
Phone: 705-652-2000 FAX: 705-652-6365

06-February-2020

Paracel Laboratories

Attn : Dale Robertson

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

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

Date Rec. : 31 January 2020
LR Report: CA15593-JAN20
Reference: Project#: 2005368

Copy: #1

CERTIFICATE OF ANALYSIS

Final Report

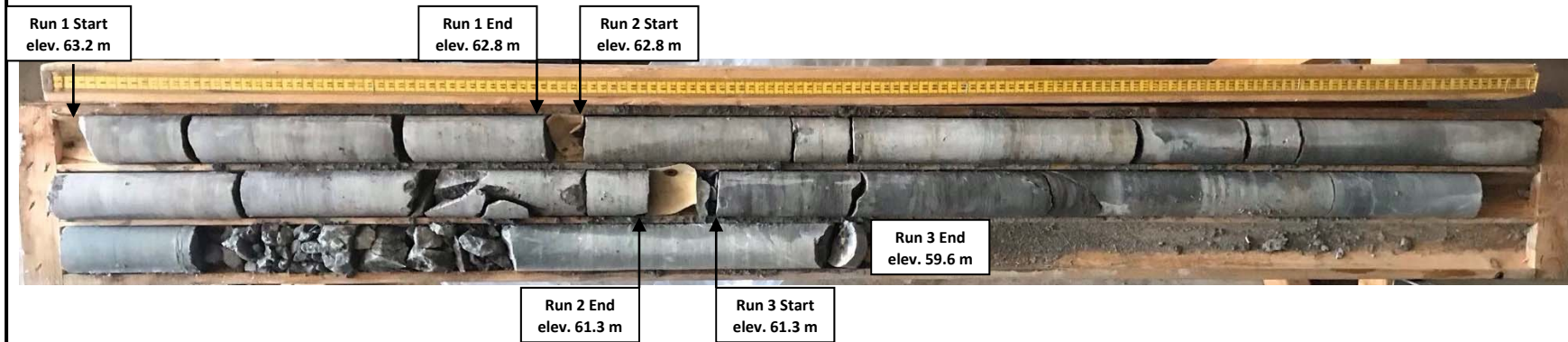
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1: Analysis Start Date		05-Feb-20
2: Analysis Start Time		16:33
3: Analysis Completed Date		05-Feb-20
4: Analysis Completed Time		16:58
5: QC - Blank		< 0.02
6: QC - STD % Recovery		110%
7: QC - DUP % RPD		NV
8: RL		0.02
9: 19-3 SS4 7'6"-9'6"	23-Jan-20	0.04
10: 19-6 SS3 5'-7'	21-Jan-20	< 0.02

RL - SGS Reporting Limit
NV - No Value

Kimberley Didsbury
Project Specialist,
Environment, Health & Safety

Appendix C.3
Rock Core UCS and Photographs

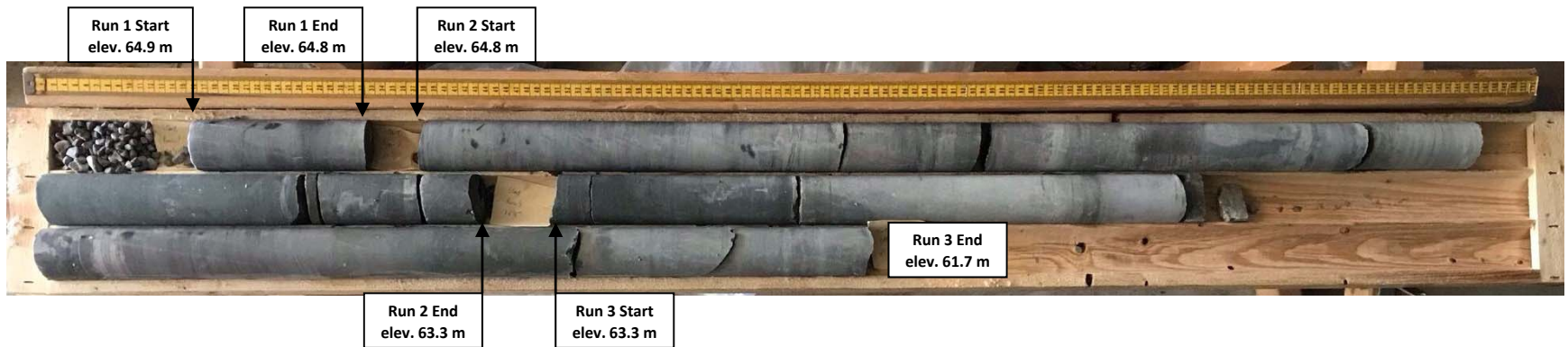
Borehole 19-7
Run 1 to 3 (of 3)
Elevation 63.2m to 59.6m



Geotechnical Investigation
HWY 401 Napanee River Bridge
Napanee, Ontario

BH 19-7
Project No.: 27253

Borehole 19-8
Run 1 to 3 (of 3)
Elevation 64.9m to 61.7m



Geotechnical Investigation
HWY 401 Napanee River Bridge
Napanee, Ontario

BH 19-8
Project No.: 27253



Stantec

Stantec Consulting Ltd
2781 Lancaster Rd, Suite 100 A&B
Ottawa, ON K1B 1A7
Tel: (613) 738-6075
Fax: (613) 722-2799

February 10, 2020
File: 122410864

Attention: Thurber Engineering, File #27253

Reference: ASTM D7012, Method C, Unconfined Compressive Strength of Intact Rock Core
Hwy 401 Napanee, ON

The following table summarizes four rock core compressive strength results.

Location	Sample Depth	Compressive Strength (MPa)	Description of Break
19-7 Run-2	85'4"-86'	80.4	Well-formed cones at both ends
19-7 Run-2	86'2"-87'1"	66.8	Vertical cracking through both ends
19-8 Run-2	71'5"-72'3"	177.0	Vertical cracking through both ends
19-8 Run-3	76'5"-77'1"	43.5	Well-formed cones at both ends

Sincerely,

Stantec Consulting Ltd

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

Appendix D.
Site Photographs



Photo 1. Looking south-east at north bridge profile [December 2019]



Photo 2. Looking south-west at north bridge profile [January 2020]



Photo 3. Looking south-east along Highway 401 [December 2019]



Photo 4. Looking north-east at the south bridge profile [January 2020]

Appendix E.
ConeTEC Report

PRESENTATION OF SITE INVESTIGATION RESULTS

Highway 401 Napanee River Bridge

Prepared for:

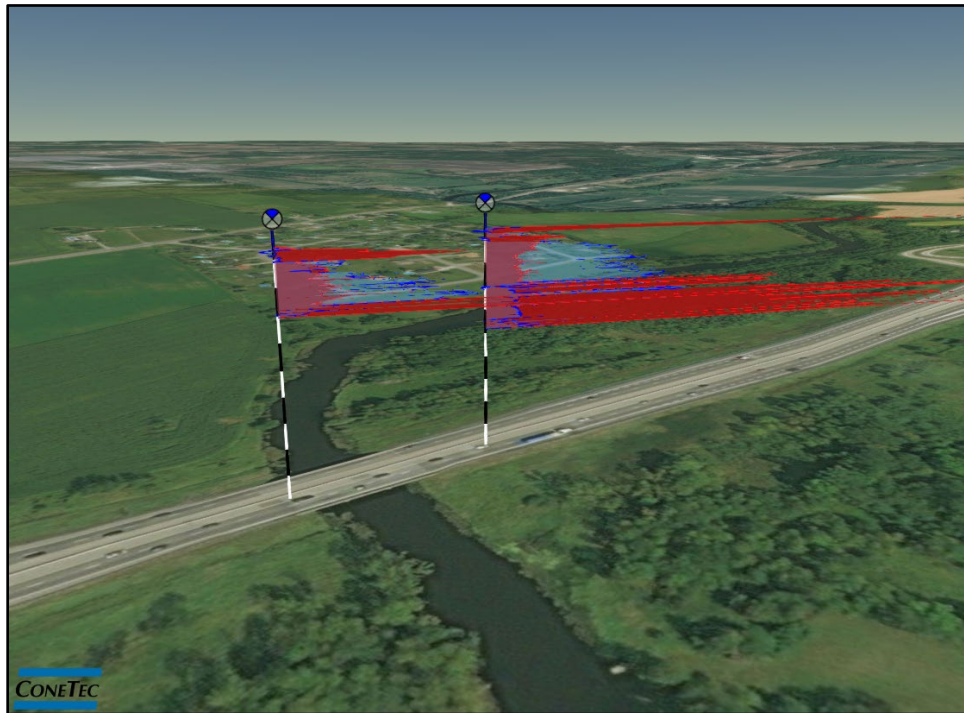
Thurber Engineering Ltd.

ConeTec Job No: 19-05077

Project Start Date: 18-Nov-2019

Project End Date: 18-Nov-2019

Report Date: 19-Nov-2019



Prepared by:

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

Tel: (905) 886-2663
Fax: (905) 886-2664
Toll Free: (800) 504-1116

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



Highway 401 Napanee River Bridge

Introduction

The enclosed report presents the results of the site investigation program conducted by ConeTec Investigations Ltd. for Thurber Engineering Ltd. at Napanee, ON. The program consisted of two seismic cone penetration tests (SCPTu).

Project Information

Project	
Client	Thurber Engineering Ltd.
Project	Highway 401 Napanee River Bridge
ConeTec project number	19-05077

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



Rig Description	Deployment System	Test Type
CPT truck rig (C3)	30 ton rig cylinder	SCPTu

Coordinates		
Test Type	Collection Method	EPSG Number
SCPTu	Consumer grade GPS	32618

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 (psi)
408:T1500F15U500	408	15	225	1500	15	500
Cone 408 was used for all CPT soundings.						

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

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

Limitations

This report has been prepared for the exclusive use of Thurber Engineering Ltd. (Client) for the project titled "Highway 401 Napanee River Bridge". The report's contents may not be relied upon by any other party without the express written permission of ConeTec Investigations Ltd. (ConeTec). ConeTec has provided site investigation services, prepared the factual data reporting and provided geotechnical parameter calculations consistent with current best practices. No other warranty, expressed or implied, is made.

The information presented in the report document and the accompanying data set pertain to the specific project, site conditions and objectives described to ConeTec by the Client. In order to properly understand the factual data, assumptions and calculations, reference must be made to the documents provided and their accompanying data sets, in their entirety.

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 a geophone sensor for recording seismic signals. All signals are amplified down hole within the cone body and the analog signals are sent to the surface through a shielded cable.

ConeTec penetrometers are manufactured with various tip, friction and pore pressure capacities in 5 cm², 10 cm² and 15 cm² tip base area configurations in order to maximize signal resolution for various soil conditions. The specific piezocone used for each test is described in the CPT summary table presented in the first appendix. The 15 cm² penetrometers do not require friction reducers as they have a diameter larger than the deployment rods. The 10 cm² piezocones use a friction reducer consisting of a rod adapter extension behind the main cone body with an enlarged cross sectional area (typically 44 mm diameter over a length of 32 mm with tapered leading and trailing edges) located at a distance of 585 mm above the cone tip.

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

All ConeTec piezocones can record pore pressure at various locations. Unless otherwise noted, the pore pressure filter is located directly behind the cone tip in the "u₂" position (ASTM Type 2). The filter is 6 mm thick, made of porous plastic (polyethylene) having an average pore size of 125 microns (90-160 microns). The function of the filter is to allow rapid movements of extremely small volumes of water needed to activate the pressure transducer while preventing soil ingress or blockage.

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

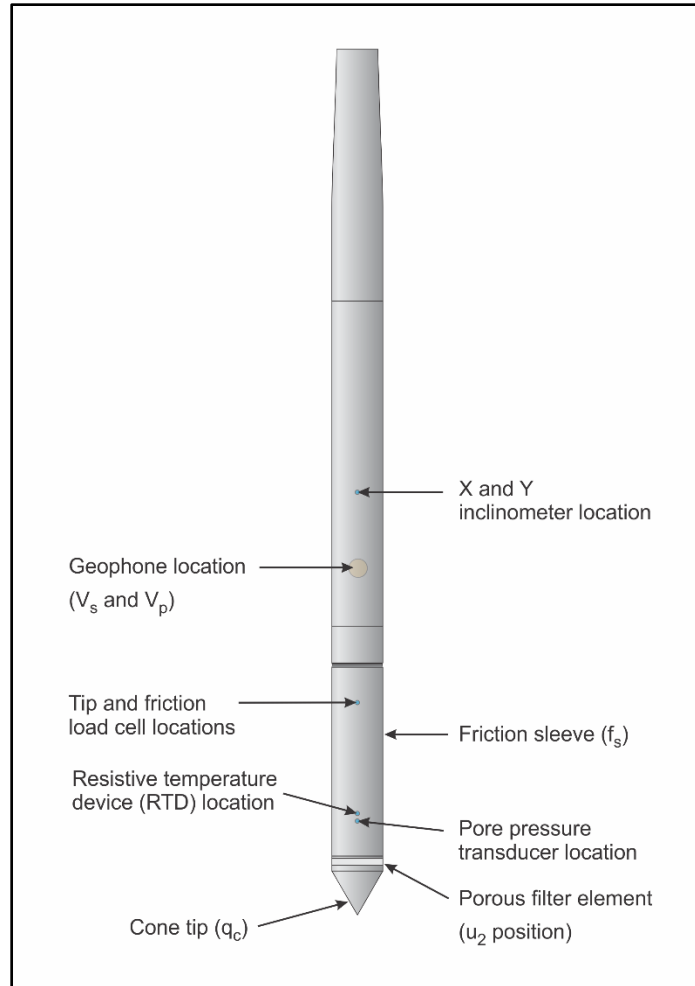


Figure CPTu. Piezocone Penetrometer (15 cm²)

The ConeTec data acquisition systems consist of a Windows based computer and a signal conditioner and power supply interface box with a 16 bit (or greater) analog to digital (A/D) converter. The data is recorded at fixed depth increments using a depth wheel attached to the push cylinders or by using a spring loaded rubber depth wheel that is held against the cone rods. The typical recording interval is 2.5 cm; custom recording intervals are possible.

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

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

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

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

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

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

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

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

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

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

where: q_t is the corrected tip resistance

q_c is the recorded tip resistance

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

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

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

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

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

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

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

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

ConeTec's piezocone penetrometers are manufactured with a horizontally active geophone (28 hertz) that is rigidly mounted in the body of the cone penetrometer, 0.2 meters behind the cone tip.

Shear waves are typically generated by using an impact hammer horizontally striking a beam that is held in place by a normal load. In some instances an auger source or an imbedded impulsive source maybe used for both shear waves and compression waves. The hammer and beam act as a contact trigger that initiates the recording of the seismic wave traces. For impulsive devices an accelerometer trigger may be used. The traces are recorded using an up-hole integrated digital oscilloscope which is part of the SCPTu data acquisition system. An illustration of the shear wave testing configuration is presented in Figure SCPTu-1.

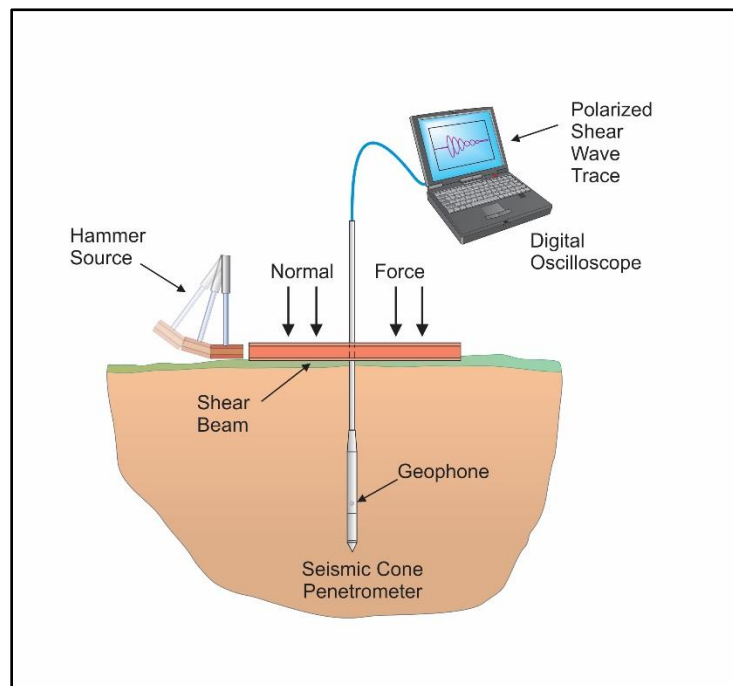


Figure SCPTu-1. Illustration of the SCPTu system

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

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

Prior to recording seismic waves at each test depth, cone penetration is stopped and the rods are decoupled from the rig to avoid transmission of rig energy down the rods. Typically, five wave traces for each orientation are recorded for quality control and uncertainty analysis purposes. After reviewing wave

traces for consistency the cone is pushed to the next test depth (typically one meter intervals or as requested by the client). Figure SCPTu-2 presents an illustration of a SCPTu test.

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

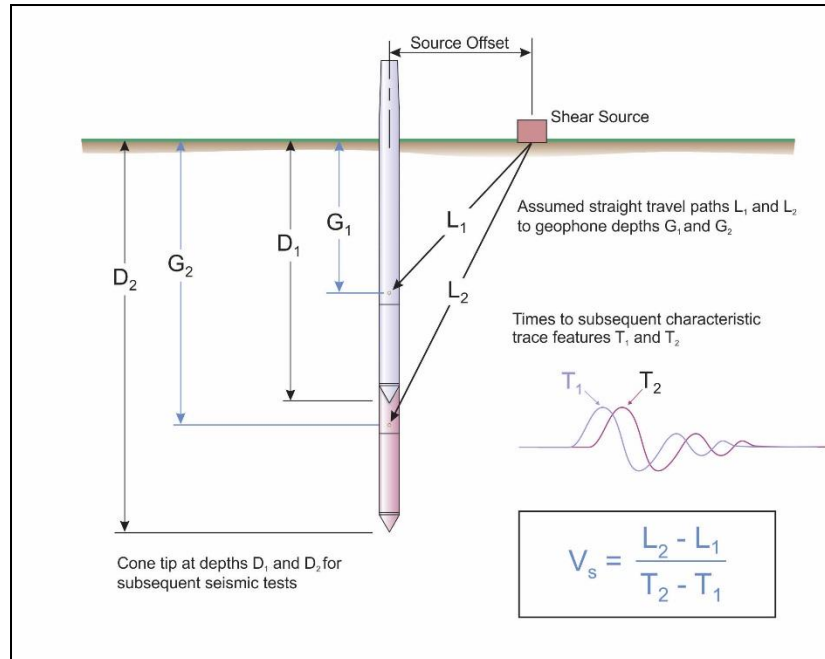


Figure SCPTu-2. Illustration of a seismic cone penetration test

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

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

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

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

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

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

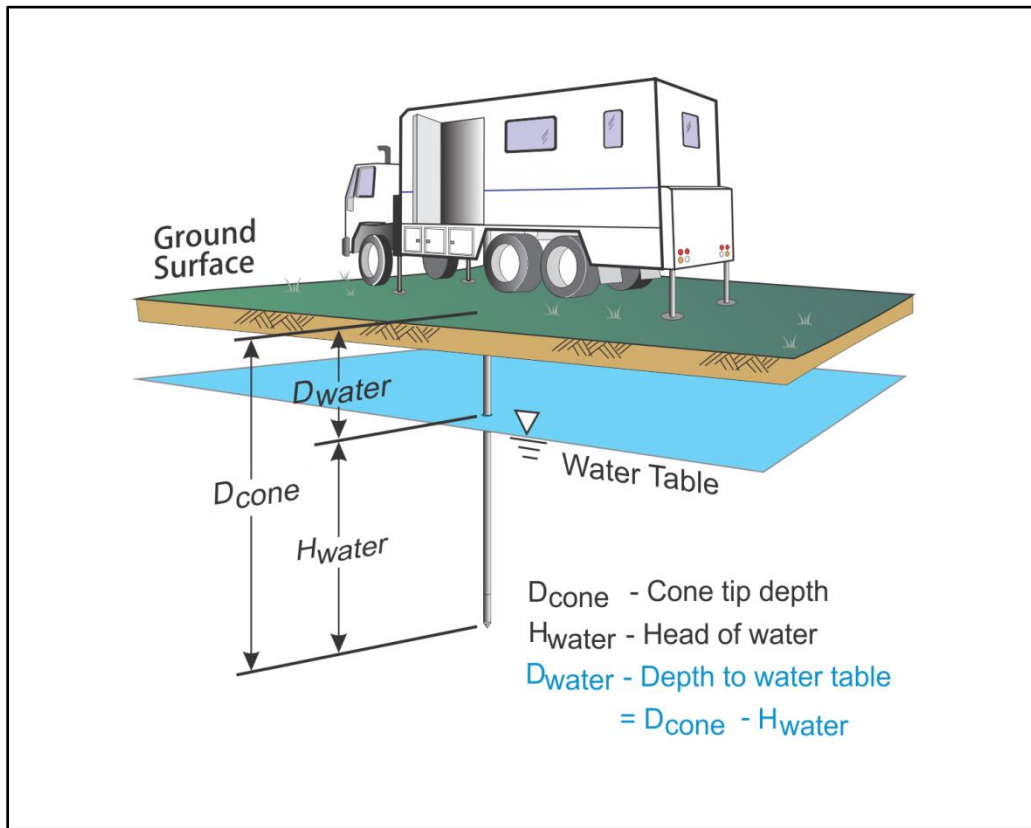


Figure PPD-1. Pore pressure dissipation test setup

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

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

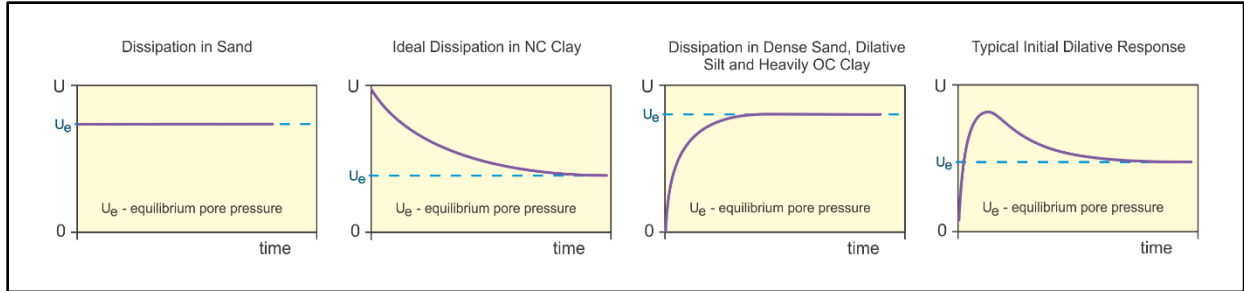


Figure PPD-2. Pore pressure dissipation curve examples

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

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

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

Where:

- T^* is the dimensionless time factor (Table Time Factor)
- a is the radius of the cone
- I_r is the rigidity index
- t is the time at the degree of consolidation

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

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

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

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

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

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

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

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

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REFERENCES

Sully, J.P., Robertson, P.K., Campanella, R.G. and Woeller, D.J., 1999, "An approach to evaluation of field CPTU dissipation data in overconsolidated fine-grained soils", *Canadian Geotechnical Journal*, 36(2): 369-381.

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

The appendices listed below are included in the report:

- Cone Penetration Test Summary and Standard Cone Penetration Test Plots
- Standard Cone Penetration Test Plots with Expanded Range
- Advanced Cone Penetration Test Plots with I_c , $S_u(N_{kt})$, Φ and $N1(60)I_c$
- Seismic Cone Penetration Test Plots
- Seismic Cone Penetration Test Tabular Results
- Seismic Cone Penetration Test Shear Wave (V_s) Traces
- Soil Behaviour Type (SBT) Scatter Plots
- Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

Cone Penetration Test Summary and Standard Cone Penetration Test Plots

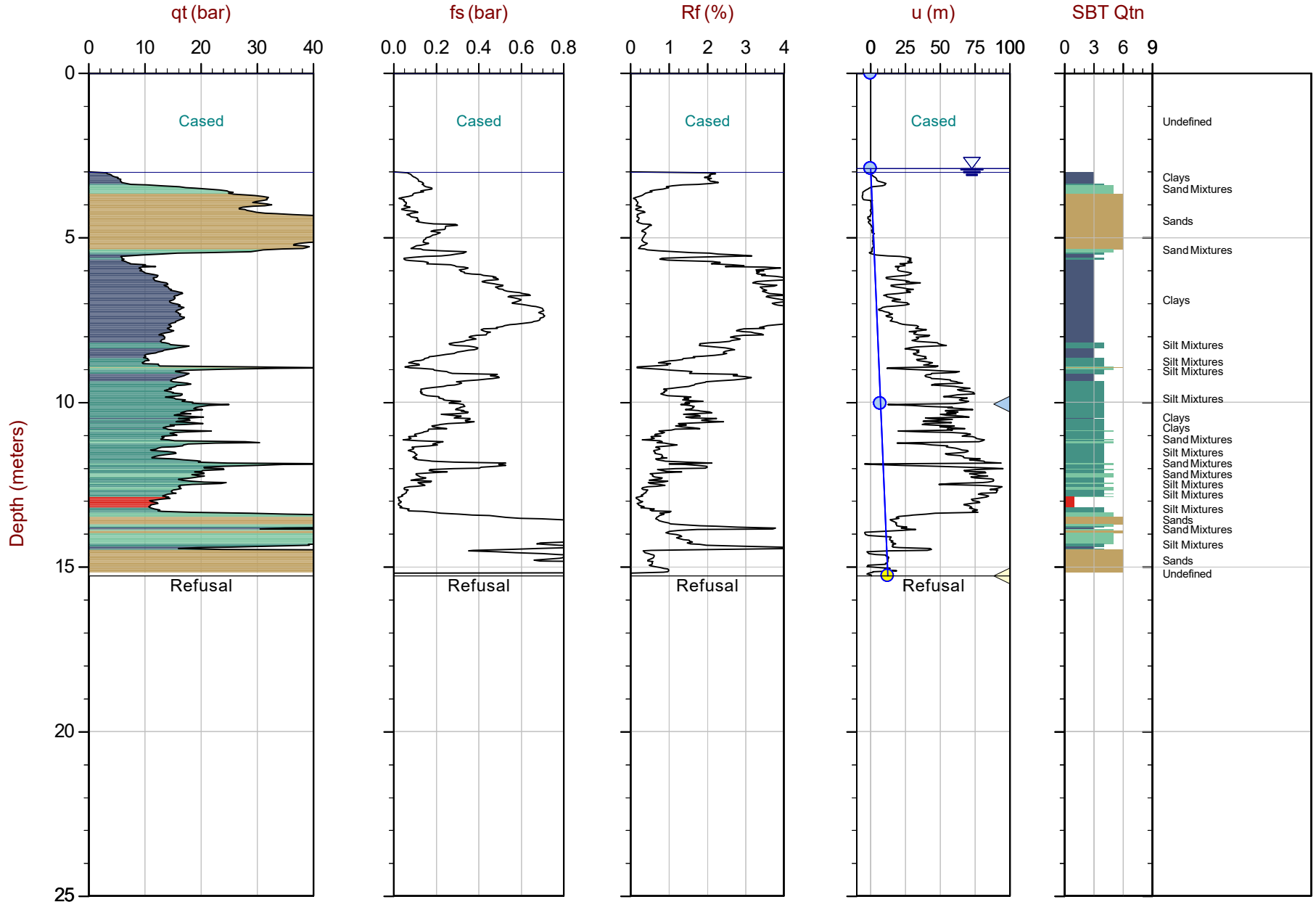


Job No: 19-05077
Client: Thurber Engineering Ltd.
Project: Highway 401 Napanee River Bridge
Start Date: 18-Nov-2019
End Date: 18-Nov-2019

CONE PENETRATION TEST SUMMARY

Sounding ID	File Name	Date	Cone	Assumed Phreatic Surface ¹ (m)	Final Depth (m)	Northing ² (m)	Easting ² (m)	Refer to Notation Number
SCPT19-01	19-05077_SP01	18-Nov-2019	408:T1500F15U500	2.9	15.275	4903399	345747	
SCPT19-02	19-05077_SP02	18-Nov-2019	408:T1500F15U500	2.9	23.975	4903423	345811	3

1. The assumed phreatic surface was based on pore pressure dissipation tests, unless otherwise noted. Hydrostatic conditions were assumed for calculated parameters.
2. Coordinates were collected with a consumer grade GPS in datum WGS84 / UTM Zone 18 North.
3. Phreatic surface based on SCPT19-01.

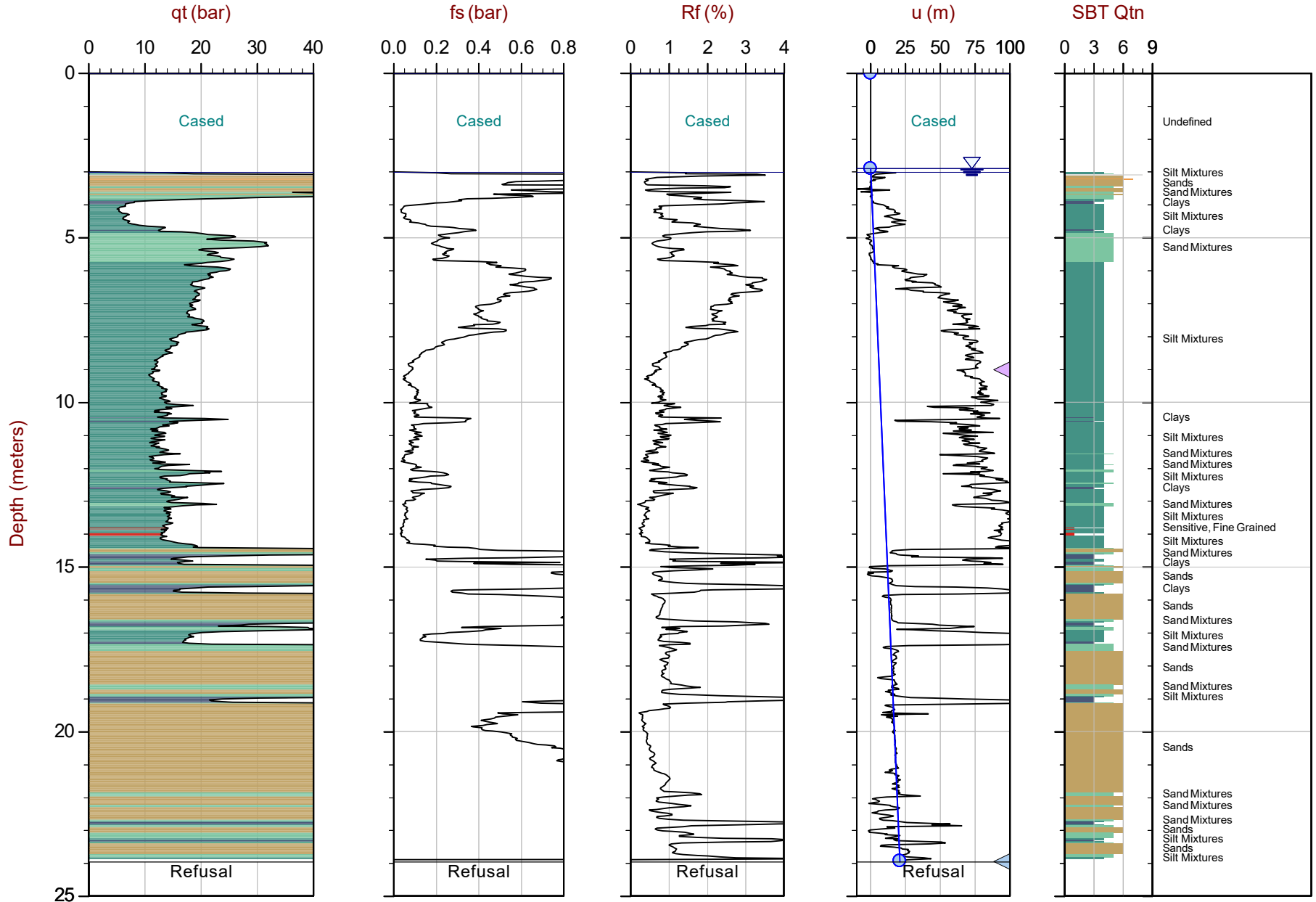


Max Depth: 15.275 m / 50.11 ft
 Depth Inc: 0.025 m / 0.082 ft
 Avg Int: EveryPoint

File: 19-05077_SP01.COR
 Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010
 Coords: UTM 18N: 4903399mE: 345747m
 PageNo: 1 of 1

● Ueq ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved ◁ Dissipation, Ueq assumed — Ueq Line — Hydrostatic Line



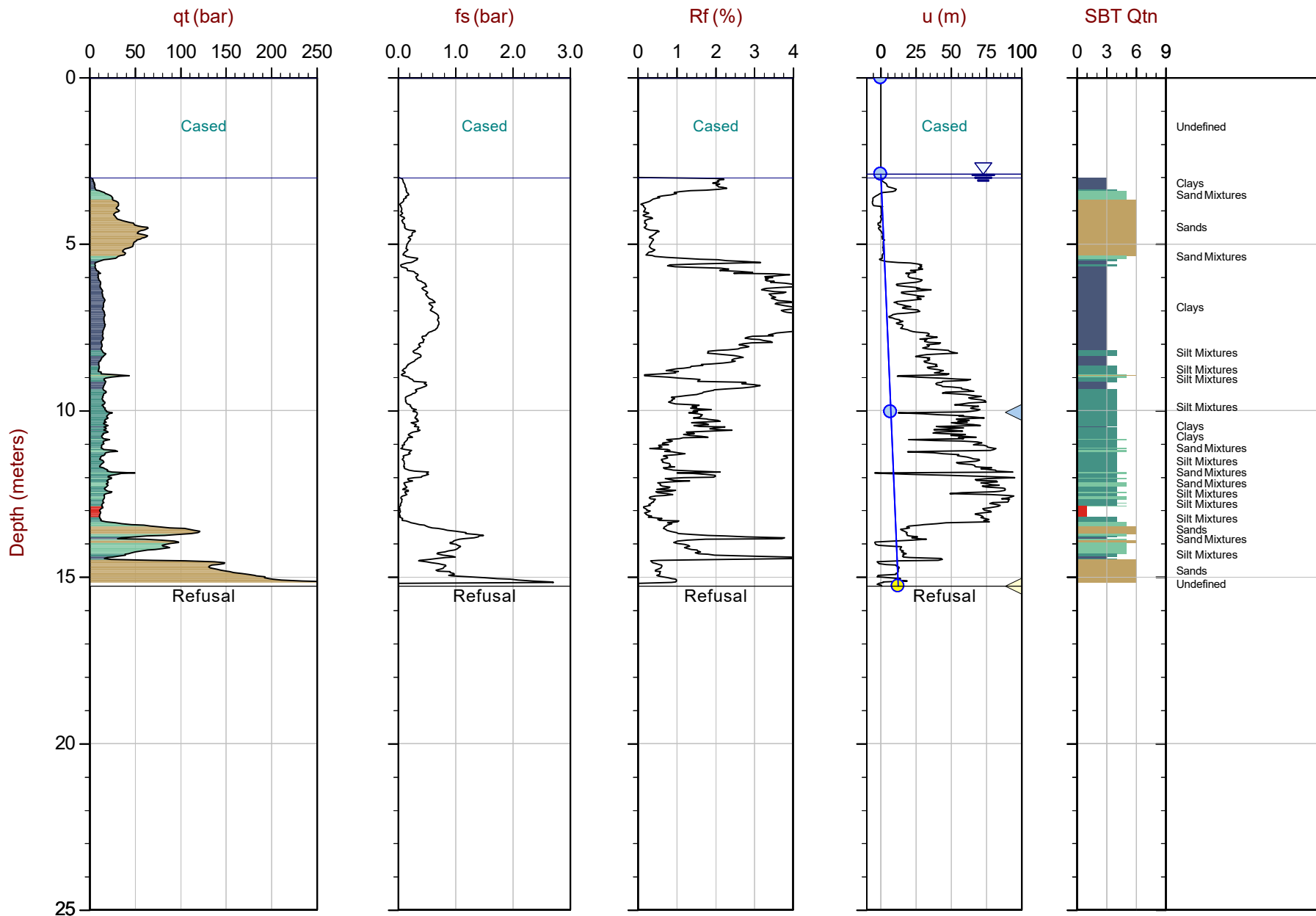
Max Depth: 23.975 m / 78.66 ft
 Depth Inc: 0.025 m / 0.082 ft
 Avg Int: EveryPoint

File: 19-05077_SP02.COR
 Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010
 Coords: UTM 18N N: 4903423m E: 345811m
 Page No: 1 of 1

● Ueq
 ● Assumed Ueq
 ◁ Dissipation, Ueq achieved
 ◁ Dissipation, Ueq not achieved
 ◁ Dissipation, Ueq assumed
 — Ueq Line
 — Hydrostatic Line

Standard Cone Penetration Test Plots with Expanded Range

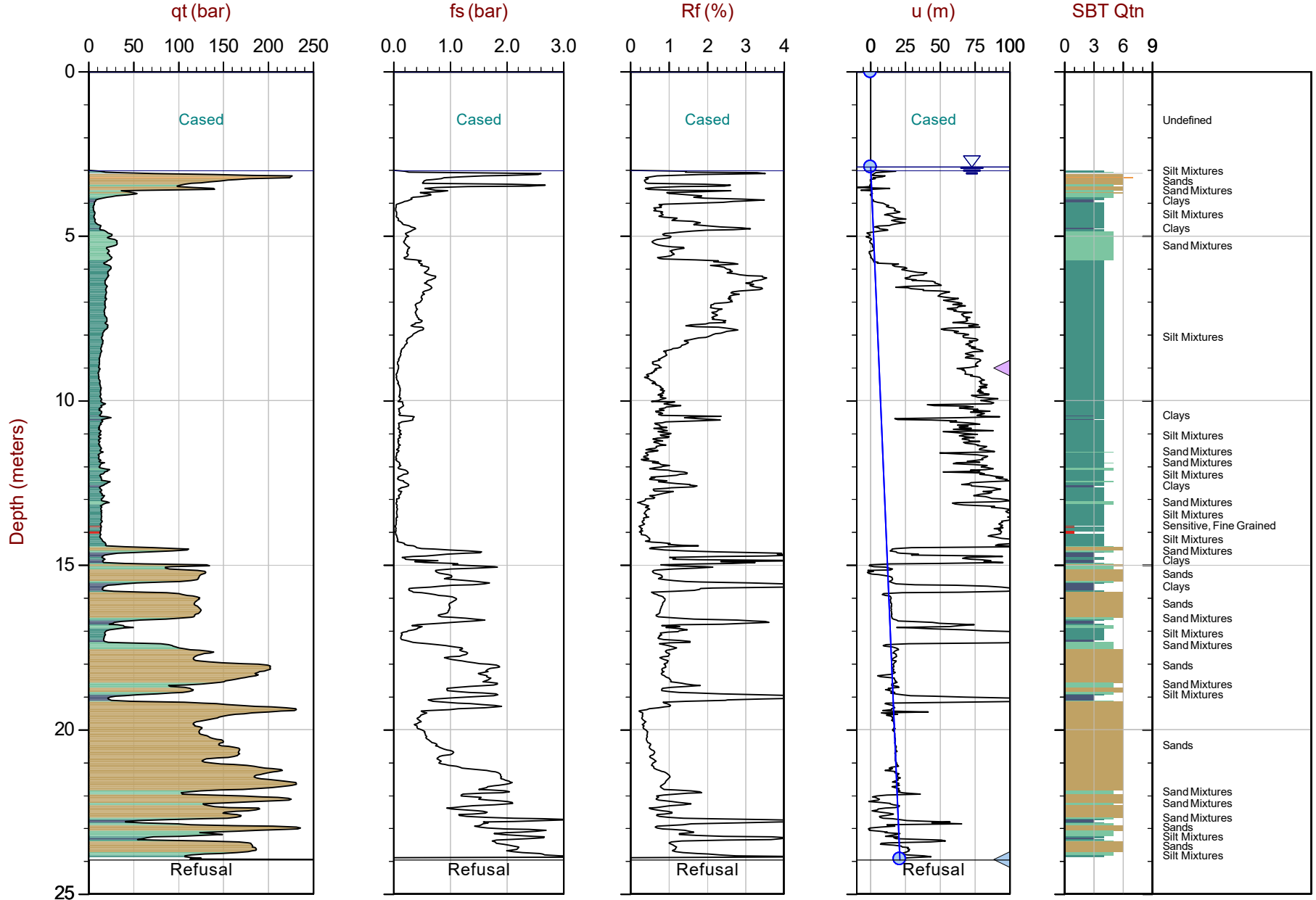


Max Depth: 15.275 m / 50.11 ft
 Depth Inc: 0.025 m / 0.082 ft
 Avg Int: EveryPoint

File: 19-05077_SP01.COR
 Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010
 Coords: UTM 18N N: 4903399m E: 345747m
 Page No: 1 of 1

● Ueq
 ● Assumed Ueq
 ◁ Dissipation, Ueq achieved
 ◁ Dissipation, Ueq not achieved
 ◁ Dissipation, Ueq assumed
 — Ueq Line
 — Hydrostatic Line



Max Depth: 23.975 m / 78.66 ft
 Depth Inc: 0.025 m / 0.082 ft
 Avg Int: EveryPoint

File: 19-05077_SP02.COR
 Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010
 Coords: UTM 18N N: 4903423m E: 345811m
 PageNo: 1 of 1

● Ueq ● Assumed Ueq ◁ Dissipation, Ueq achieved ◃ Dissipation, Ueq not achieved ◄ Dissipation, Ueq assumed — Ueq Line — Hydrostatic Line

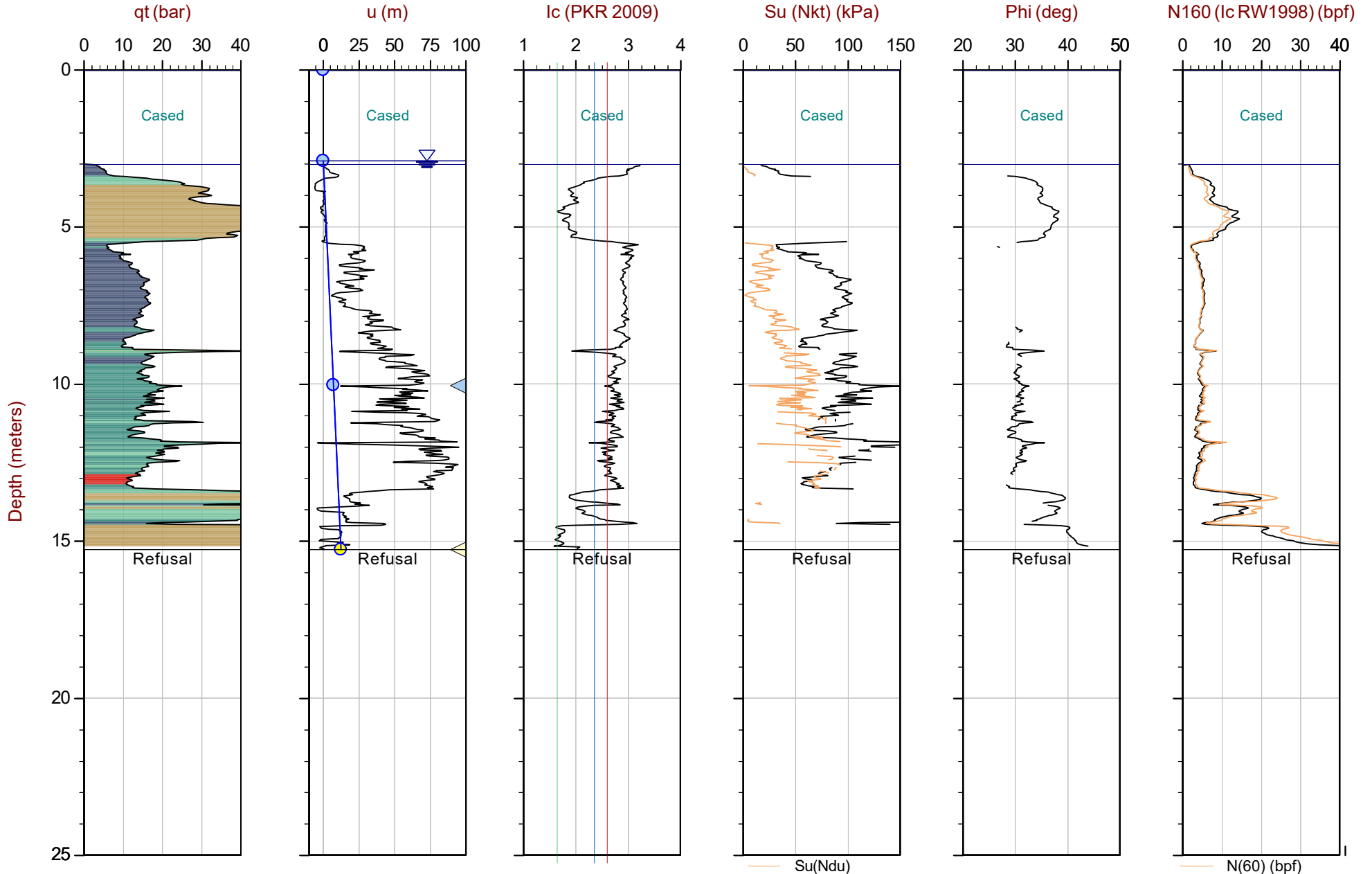
Advanced Cone Penetration Test Plots with I_c , $S_u(N_{kt})$, Φ and $N_{1(60)I_c}$



Thurber

Job No: 19-05077
Date: 2019-11-18 21:33
Site: Napanee, ON

Sounding: SCPT19-01
Cone: 408:T1500F15U500



Max Depth: 15.275 m / 50.11 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: EveryPoint

File: 19-05077_SP01.COR
Unit Wt: SBTQtn(PKR2009)
Su Nkt/Ndu: 15.0 / 9.0

SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 4903399m E: 345747m
Page No: 1 of 1

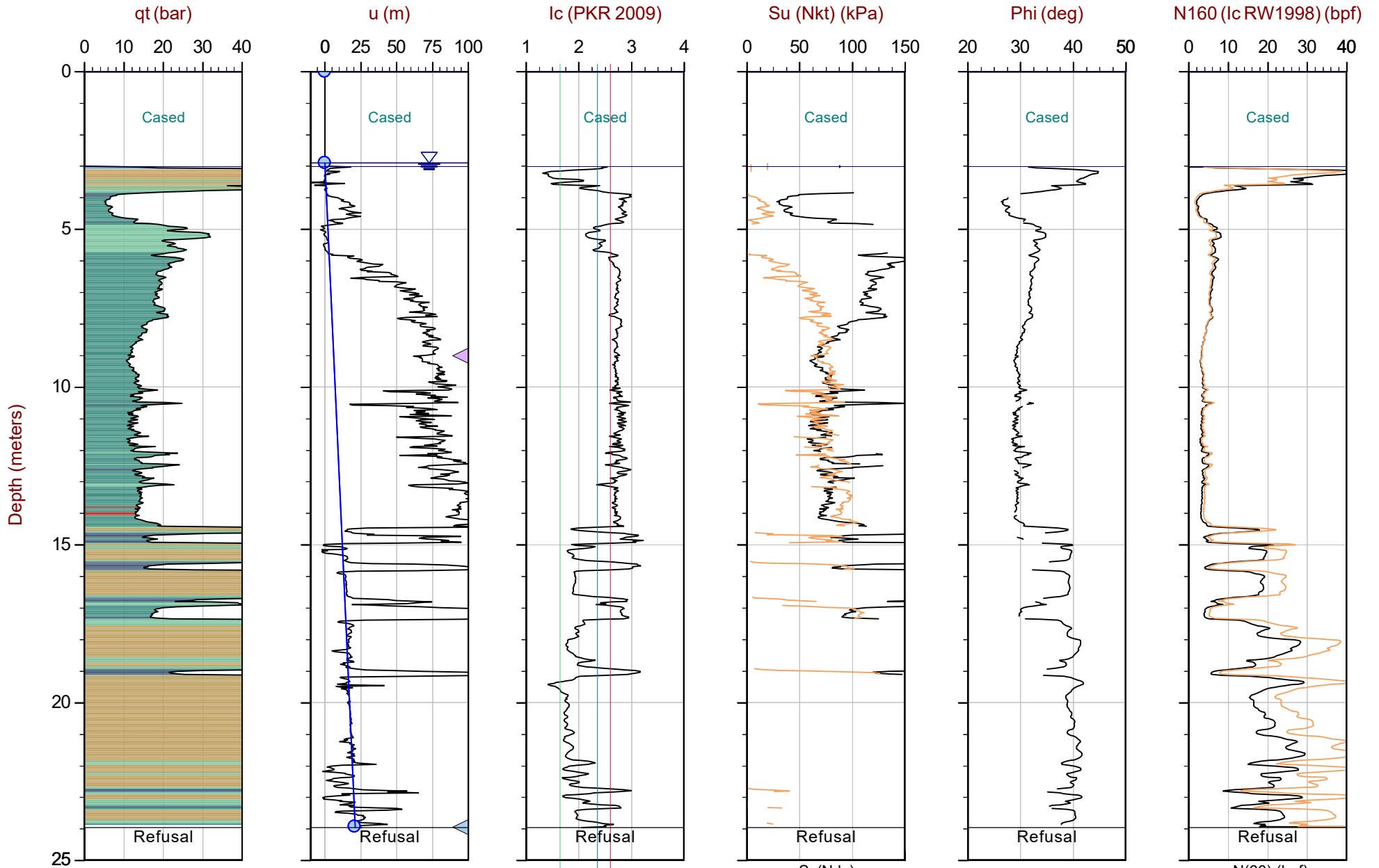
● Ueq ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved ◁ Dissipation, Ueq assumed — Ueq Line — Hydrostatic Line



Thurber

Job No: 19-05077
 Date: 2019-11-18 22:46
 Site: Napanee, ON

Sounding: SCPT19-02
 Cone: 408:T1500F15U500



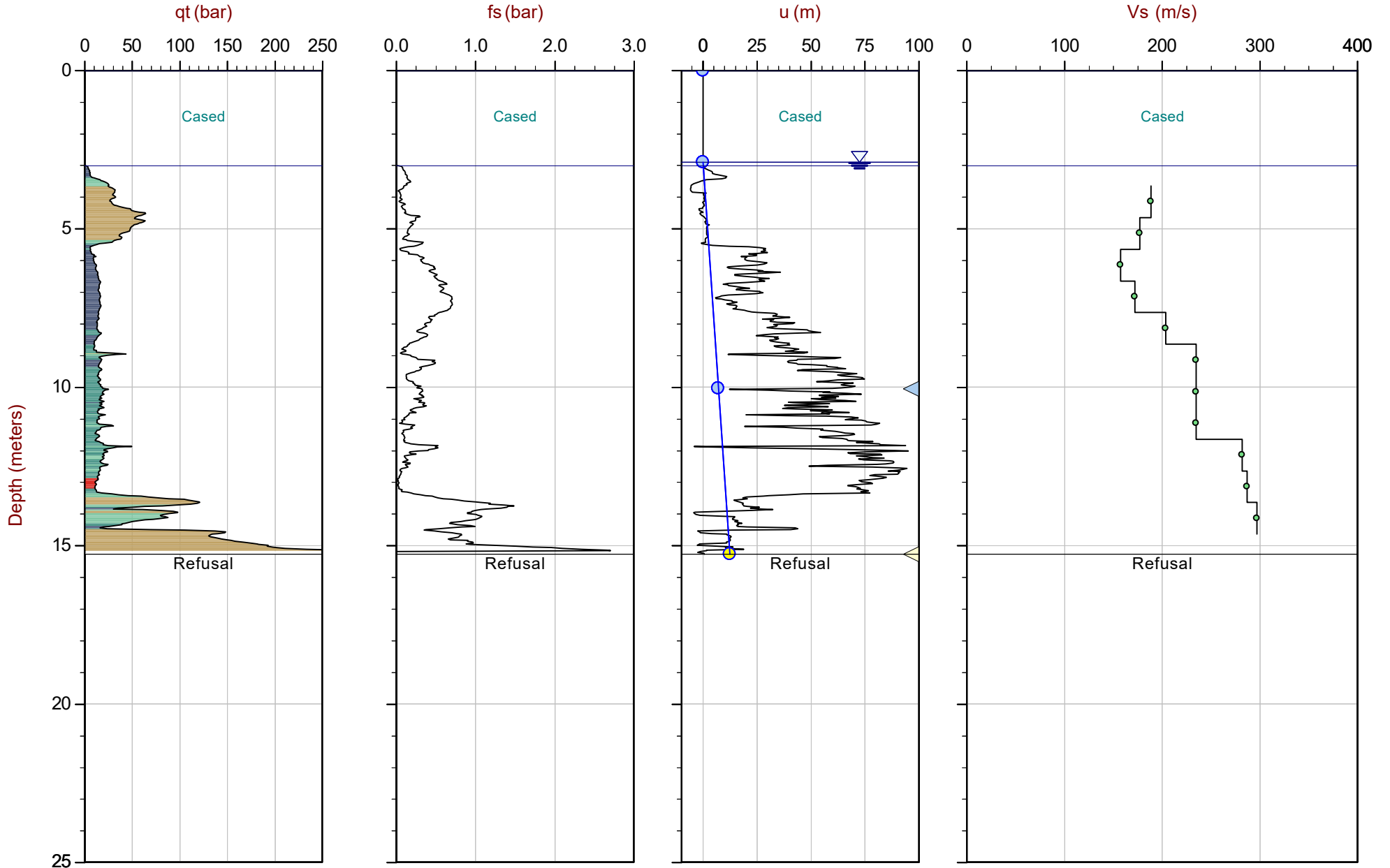
Max Depth: 23.975 m / 78.66 ft
 Depth Inc: 0.025 m / 0.082 ft
 Avg Int: EveryPoint

File: 19-05077_SP02.COR
 Unit Wt: SBTQtn(PKR2009)
 Su Nkt/Ndu: 15.0 / 9.0

SBT: Robertson, 2009 and 2010
 Coords: UTM 18N N: 4903423m E: 345811m
 Page No: 1 of 1

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

Seismic Cone Penetration Test Plots



Max Depth: 15.275 m / 50.11 ft
 Depth Inc: 0.025 m / 0.082 ft
 Avg Int: EveryPoint

File: 19-05077_SP01.COR
 Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010
 Coords: UTM 18N N: 4903399m E: 345747m
 Page No: 1 of 1

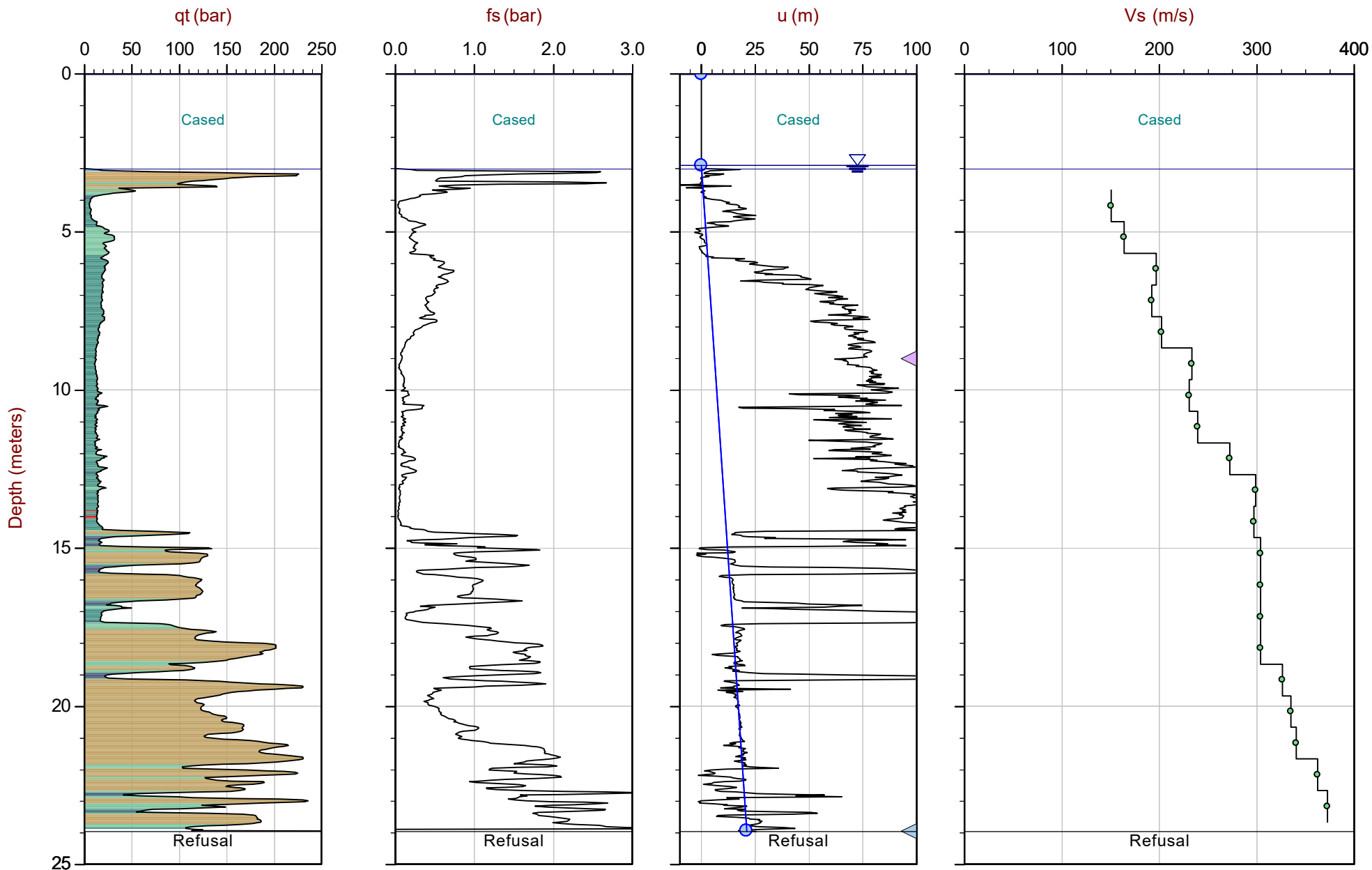
● Ueq
 ● Assumed Ueq
 ◁ Dissipation, Ueq achieved
 ◁ Dissipation, Ueq not achieved
 ◁ Dissipation, Ueq assumed
 — Ueq Line
 — Hydrostatic Line



Thurber

Job No: 19-05077
Date: 2019-11-18 22:46
Site: Napanee, ON

Sounding: SCPT19-02
Cone: 408:T1500F15U500



Max Depth: 23.975 m / 78.66 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: EveryPoint

File: 19-05077_SP02.COR
Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 4903423m E: 345811m
Page No: 1 of 1

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

Seismic Cone Penetration Test Tabular Results



Job No: 19-05077
Client: Thurber Engineering Ltd.
Project: Highway 401 Napanee River Bridge
Sounding ID: SCPT19-01
Date: 18-Nov-2019

Seismic Source: Beam
Seismic Offset (m): 0.55
Source Depth (m): 0.00
Geophone Offset (m): 0.20

SCPT_u SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
3.85	3.65	3.69			
4.85	4.65	4.68	0.99	5.25	189
5.85	5.65	5.68	1.00	5.61	178
6.85	6.65	6.67	1.00	6.31	158
7.85	7.65	7.67	1.00	5.78	173
8.85	8.65	8.67	1.00	4.89	204
9.85	9.65	9.67	1.00	4.25	235
10.85	10.65	10.66	1.00	4.25	235
11.85	11.65	11.66	1.00	4.25	235
12.85	12.65	12.66	1.00	3.54	282
13.85	13.65	13.66	1.00	3.48	287
14.85	14.65	14.66	1.00	3.36	297



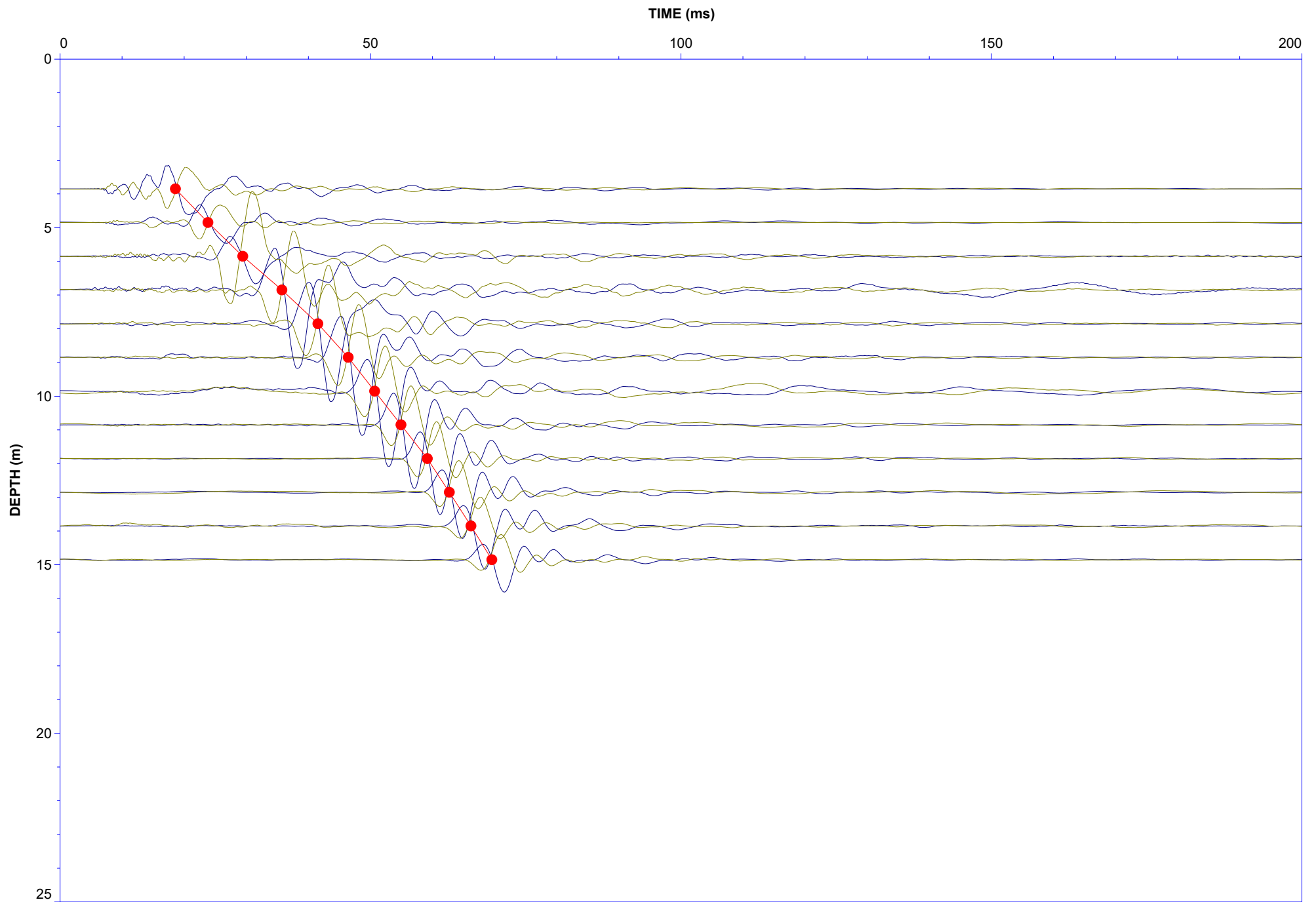
Job No: 19-05077
Client: Thurber Engineering Ltd.
Project: Highway 401 Napanee River Bridge
Sounding ID: SCPT19-02
Date: 18-Nov-2019

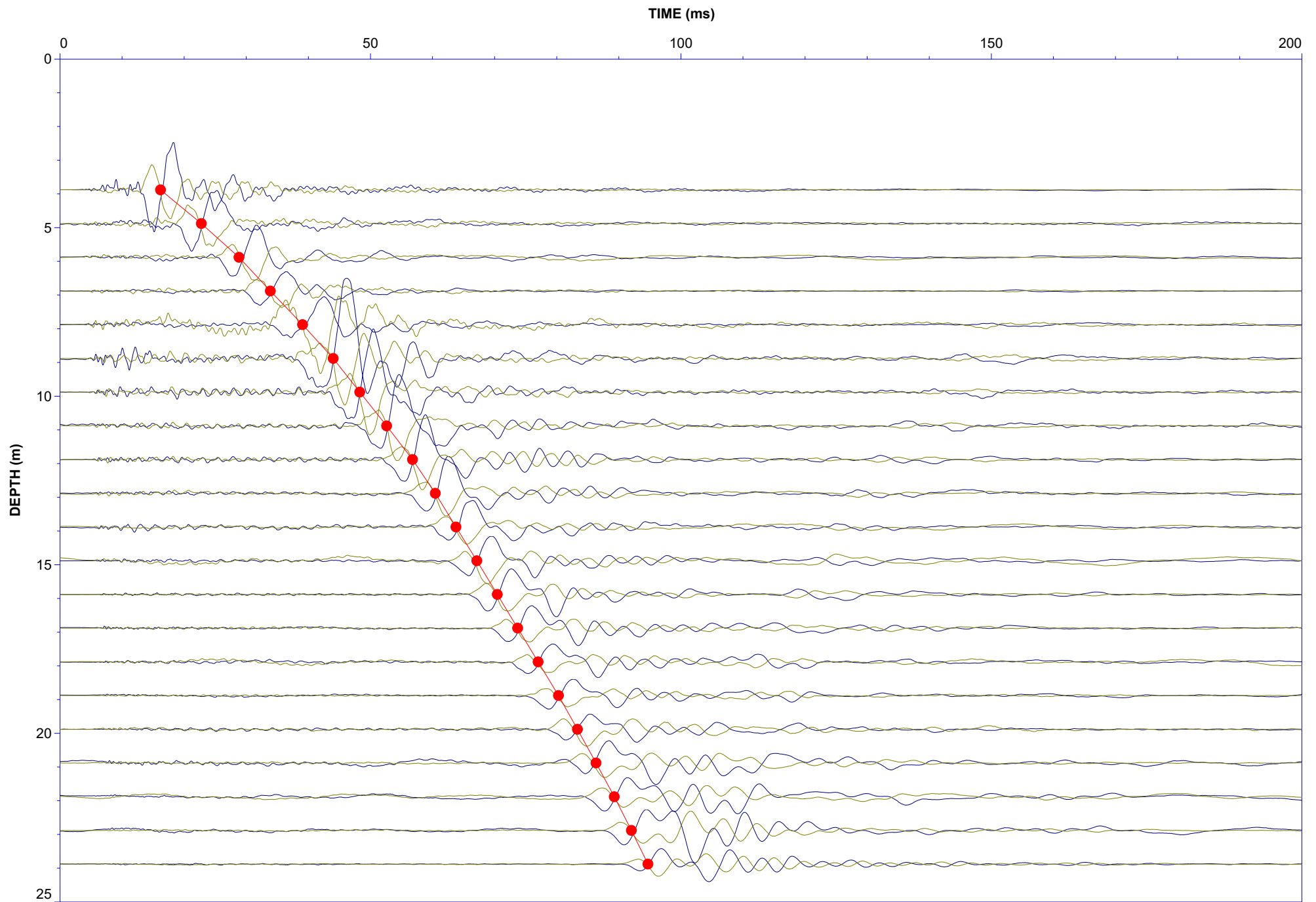
Seismic Source: Beam
Seismic Offset (m): 0.55
Source Depth (m): 0.00
Geophone Offset (m): 0.20

SCPT_u SHEAR WAVE VELOCITY TEST RESULTS - Vs

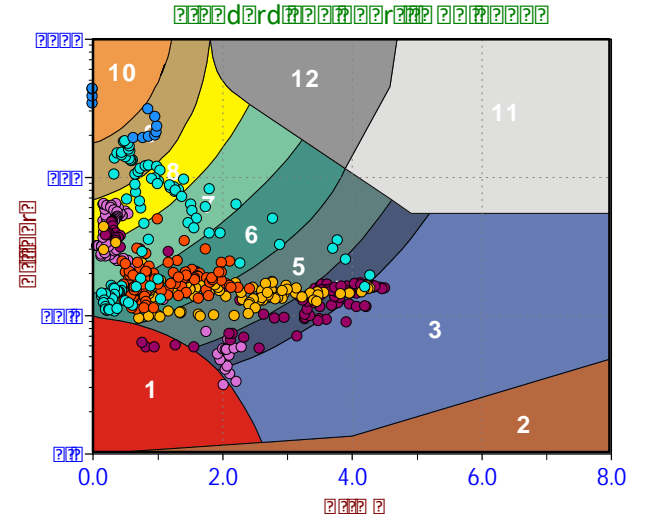
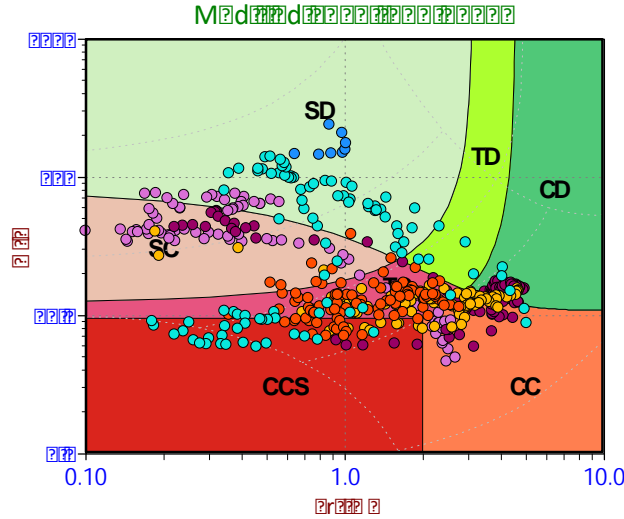
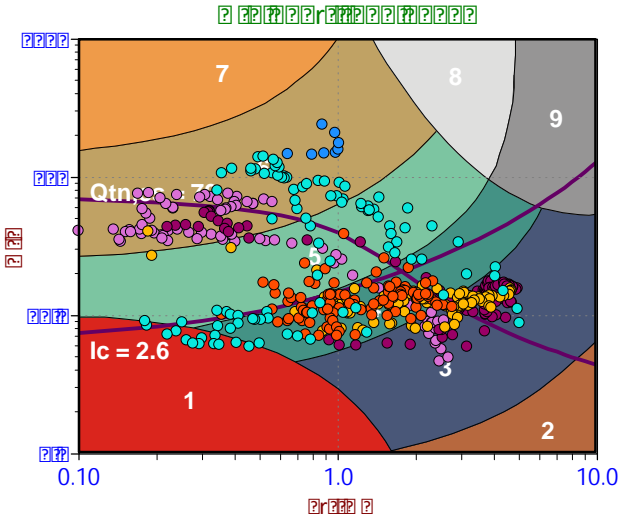
Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
3.88	3.68	3.72			
4.88	4.68	4.71	0.99	6.57	151
5.88	5.68	5.71	1.00	6.07	164
6.88	6.68	6.70	1.00	5.06	197
7.88	7.68	7.70	1.00	5.18	192
8.88	8.68	8.70	1.00	4.93	202
9.88	9.68	9.70	1.00	4.27	234
10.88	10.68	10.69	1.00	4.33	231
11.88	11.68	11.69	1.00	4.17	239
12.88	12.68	12.69	1.00	3.67	273
13.88	13.68	13.69	1.00	3.34	299
14.88	14.68	14.69	1.00	3.36	297
15.88	15.68	15.69	1.00	3.29	304
16.88	16.68	16.69	1.00	3.29	304
17.88	17.68	17.69	1.00	3.29	304
18.88	18.68	18.69	1.00	3.29	304
19.88	19.68	19.69	1.00	3.06	327
20.88	20.68	20.69	1.00	2.98	335
21.88	21.68	21.69	1.00	2.94	341
22.88	22.68	22.69	1.00	2.76	363
23.88	23.68	23.69	1.00	2.68	373

Seismic Cone Penetration Test Shear Wave (V_s) Traces





Soil Behaviour Type (SBT) Scatter Plots



Depth Ranges

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

Legend

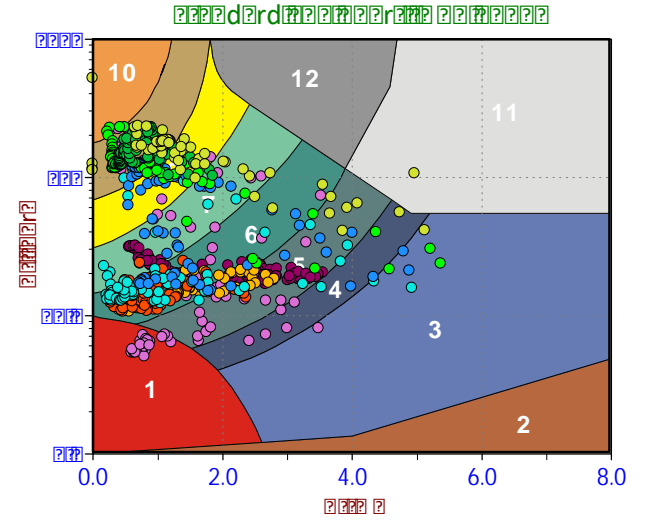
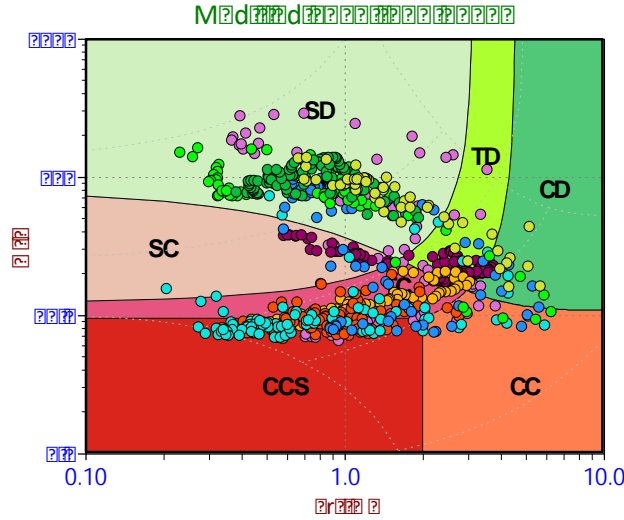
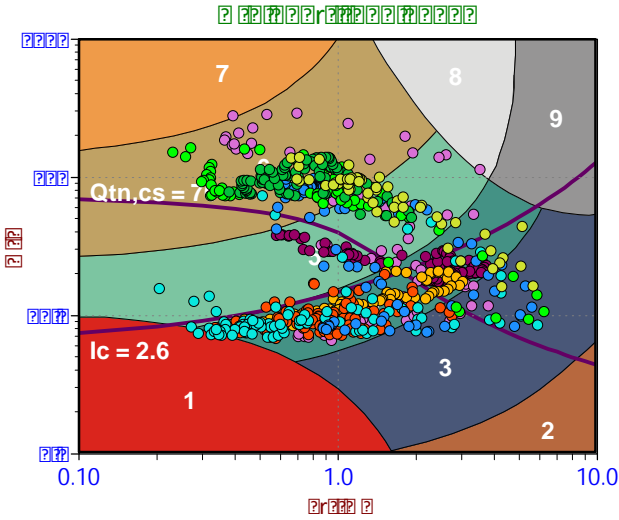
- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

Legend

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

Legend

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



Depth Ranges

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

Legend

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

Legend

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

Legend

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

Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots



Job No: 19-05077
Client: Thurber Engineering Ltd.
Project: Highway 401 Napanee River Bridge
Start Date: 18-Nov-2019
End Date: 18-Nov-2019

CPT_u PORE PRESSURE DISSIPATION SUMMARY

Sounding ID	File Name	Cone Area (cm ²)	Duration (s)	Test Depth (m)	Estimated Equilibrium Pore Pressure U _{eq} (m)	Calculated Phreatic Surface (m)	Estimated Phreatic Surface (m)	t ₅₀ ^a (s)	Assumed Rigidity Index (I _r)	c _h ^b (cm ² /min)
SCPT19-01	19-05077_SP01	15	500	10.050	Not Achieved		2.9	82	100	5.7
SCPT19-01	19-05077_SP01	15	300	15.275	12.4	2.9				
SCPT19-02	19-05077_SP02	15	1005	9.000	Not Achieved					
SCPT19-02	19-05077_SP02	15	555	23.950	Not Achieved		2.9	167	100	2.8

a. Time is relative to where u_{max} occurred.

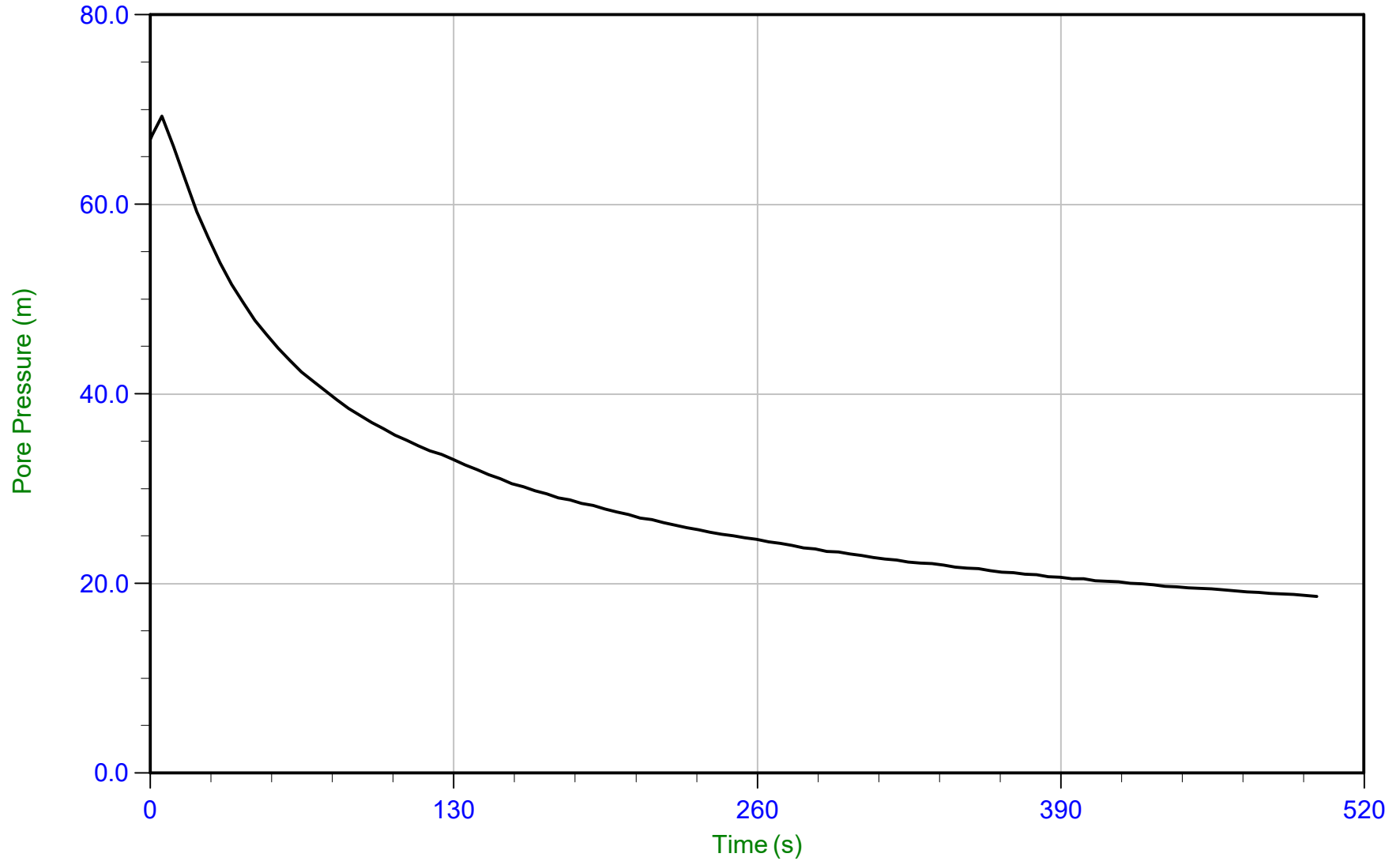
b. Hously and Teh, 1991.



Thurber

Job No: 19-05077
Date: 11/18/2019 21:33
Site: Napanee, ON

Sounding: SCPT19-01
Cone: 408:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 19-05077_SP01.PPF
Depth: 10.050 m / 32.972 ft
Duration: 500.0 s

u Min: 18.7 m
u Max: 69.3 m
u Final: 18.7 m

WT: 2.896 m / 9.501 ft
Ueq: 7.2 m
U(50): 38.23 m

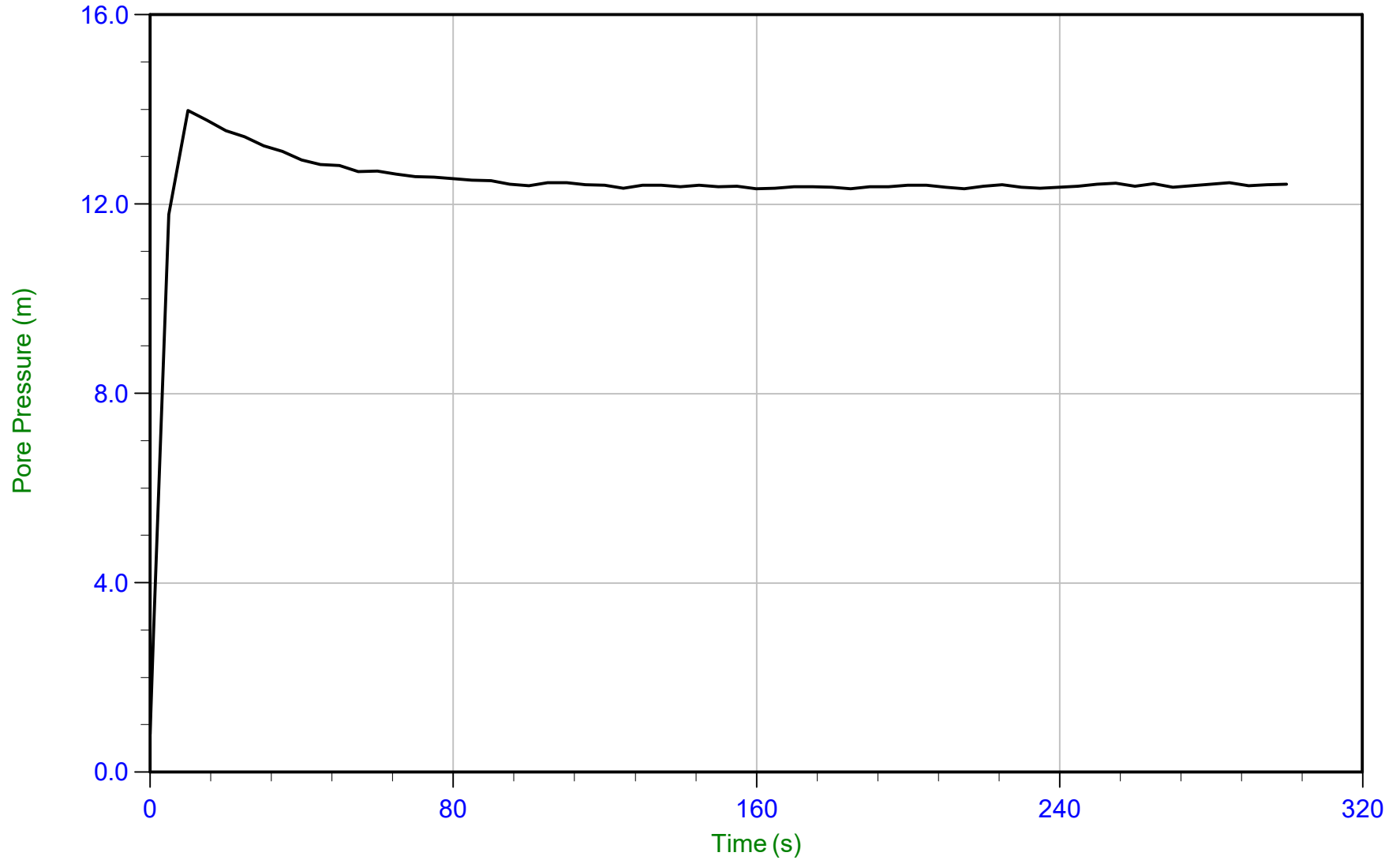
T(50): 81.5 s
lr: 100
Ch: 5.7 cm²/min



Thurber

Job No: 19-05077
Date: 11/18/2019 21:33
Site: Napanee, ON

Sounding: SCPT19-01
Cone: 408:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 19-05077_SP01.PPF
Depth: 15.275 m / 50.114 ft
Duration: 300.0 s

u Min: 0.8 m
u Max: 14.0 m
u Final: 12.4 m

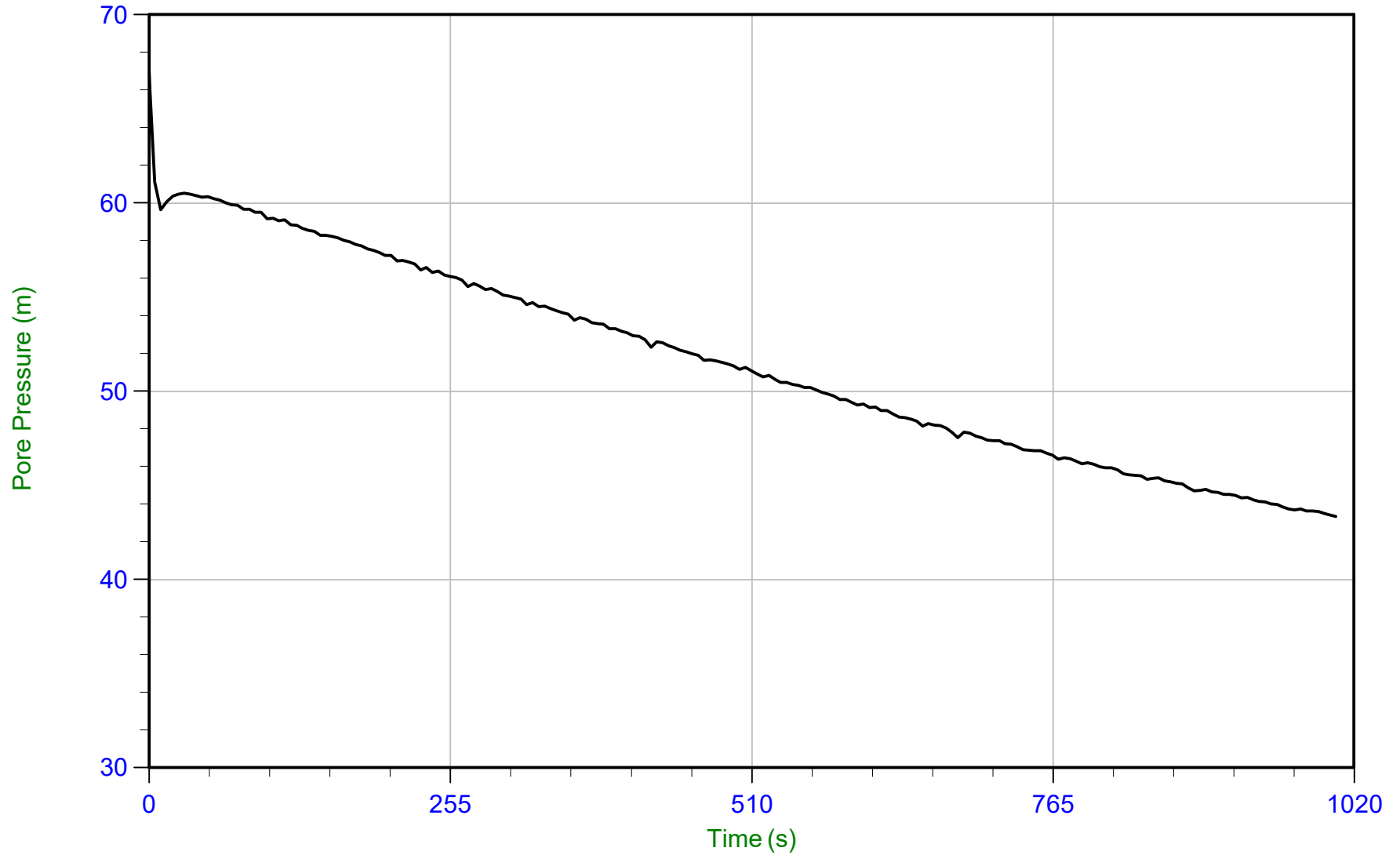
WT: 2.896 m / 9.501 ft
Ueq: 12.4 m



Thurber

Job No: 19-05077
Date: 11/18/2019 22:46
Site: Napanee, ON

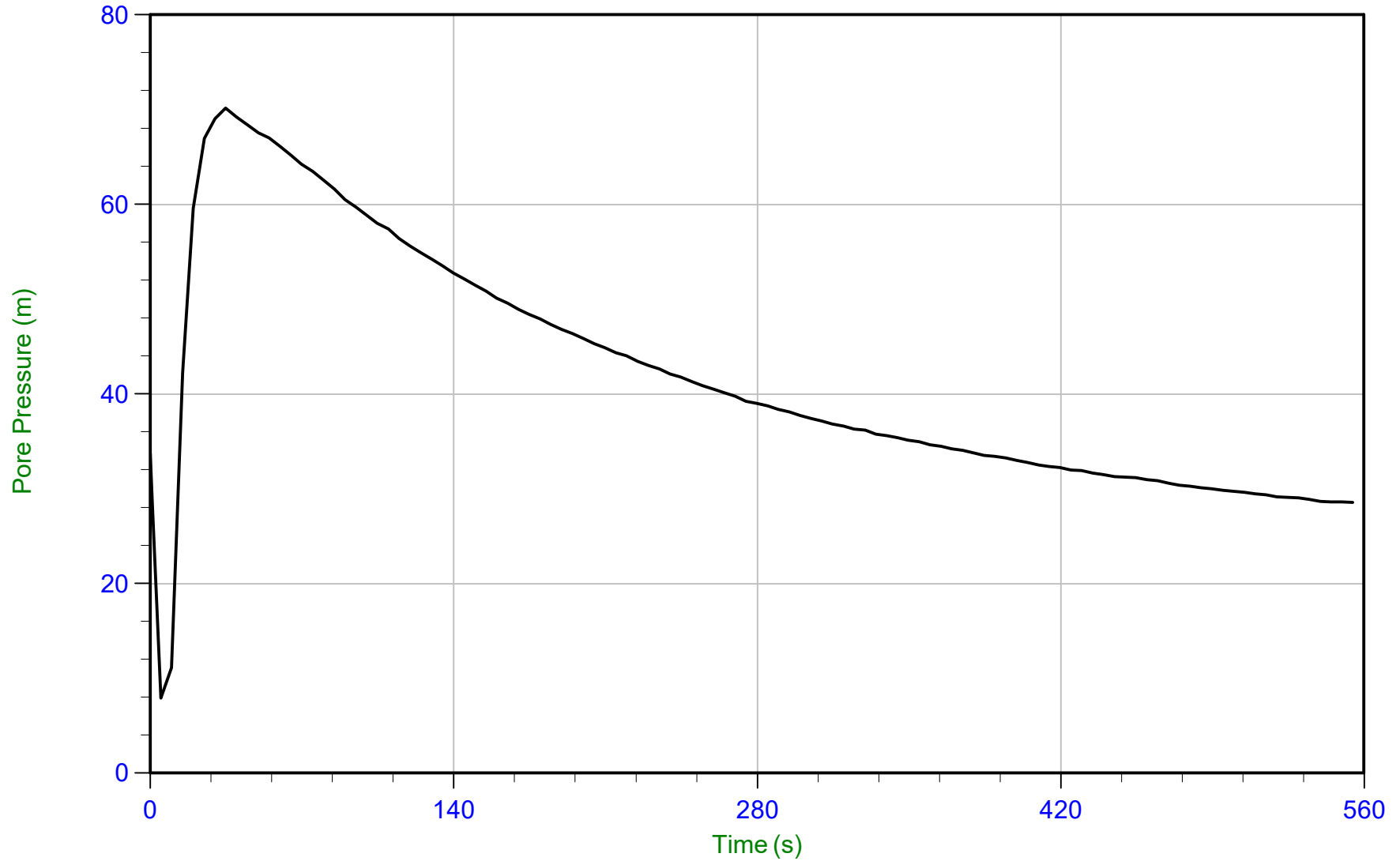
Sounding: SCPT19-02
Cone: 408:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 19-05077_SP02.PPF
Depth: 9.000 m / 29.527 ft
Duration: 1005.0 s

u Min: 43.3 m
u Max: 66.9 m
u Final: 43.3 m



Trace Summary:

Filename: 19-05077_SP02.PPF
 Depth: 23.950 m / 78.575 ft
 Duration: 555.0 s

u Min: 7.9 m
 u Max: 70.2 m
 u Final: 28.6 m

WT: 2.900 m / 9.514 ft
 Ueq: 21.1 m
 U(50): 45.61 m

T(50): 167.1 s
 Ir: 100
 Ch: 2.8 cm²/min

Appendix F.

GSC Seismic Hazard Calculation

2015 National Building Code Seismic Hazard Calculation

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

Site: 44.268N 76.932W

User File Reference: Highway 401 Napanee River Bridge

2020-01-30 18:19 UT

Requested by: Thurber Engineering Ltd.

Probability of exceedance per annum	0.000404	0.001	0.0021	0.01
Probability of exceedance in 50 years	2 %	5 %	10 %	40 %
Sa (0.05)	0.124	0.075	0.048	0.018
Sa (0.1)	0.165	0.104	0.070	0.027
Sa (0.2)	0.156	0.102	0.071	0.028
Sa (0.3)	0.131	0.087	0.061	0.025
Sa (0.5)	0.106	0.071	0.050	0.020
Sa (1.0)	0.063	0.042	0.029	0.011
Sa (2.0)	0.033	0.021	0.014	0.004
Sa (5.0)	0.009	0.005	0.003	0.001
Sa (10.0)	0.004	0.002	0.001	0.001
PGA (g)	0.095	0.060	0.040	0.015
PGV (m/s)	0.091	0.057	0.037	0.013

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

References

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

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

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

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

Appendix G.
Foundation Comparison

Comparison of Alternative Foundation Types

Shallow Foundation	Caisson (Drilled Shafts)	Driven Steel Piles
Advantages		
Generally less costly construction than deep foundations Requires less specialized construction equipment	Higher geotechnical resistance than spread footings and steel piles Construction can continue in winter weather conditions	Higher geotechnical capacity than spread footings Construction can continue in winter weather conditions Requires less concrete than spread footings and caissons Likely requires less dewatering effort Allows for integral abutments
Disadvantages		
Requires large excavations Lower geotechnical resistance than deep foundations Less efficient for resistance to uplift or overturning Granular pad must be protected from erosion/scour Low geotechnical resistance Does not allow for integral abutments	Specialized installation measures such as equipment, liners and drilling mud will be required Difficulty in cleaning and inspecting base Excavated fill to dispose of Does not allow for integral abutments	Lower geotechnical resistance than caissons Has potential to encounter obstructions in the till
Risk/Consequences		
Large Excavation Groundwater control will require enclosed excavation	Difficulty in advancing through till with high water table	Difficult advancing through obstructions Specified resistance not achieved at estimated pile tip elevation/longer piles required
Relative Cost		
Moderate	High	Moderate to High
Recommendation		
Not Recommended for support of Abutments and Pier	Not Recommended for support of Abutments and Pier	Recommended for support of Abutments and Pier

Appendix H.

P-y Curves

SOIL P-Y CURVES
Highway 401 Napanee River Bridge
West Abutment - 310x110 H-Piles

Figure H1

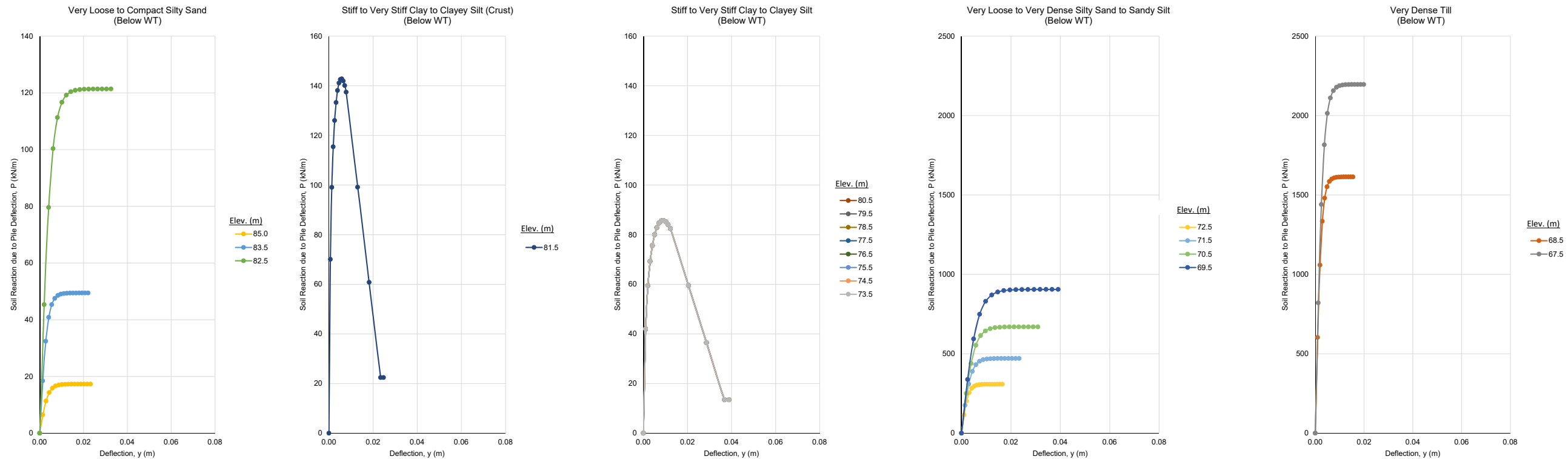
Soil Type	Very Loose to Compact Silty Sand (Below WT)						Stiff Clay (Crust)		Stiff to Very Stiff Clay to Clayey Silt (Below WT)												Very Loose to Very Dense Silty Sand to Sandy Silt (Below WT)						Dense to Very Dense Till									
	0.5		1.5		2.5		3.5		4.5		5.5		6.5		7.5		8.5		9.5		10.5		11.5		12.5		13.5		14.5		15.5		16.5		17.5	
Depth* (m)	85.0		83.5		82.5		81.5		80.5		79.5		78.5		77.5		76.5		75.5		74.5		73.5		72.5		71.5		70.5		69.5		68.5		67.5	
Elev* (m)	85.0		83.5		82.5		81.5		80.5		79.5		78.5		77.5		76.5		75.5		74.5		73.5		72.5		71.5		70.5		69.5		68.5		67.5	
P-y Curves**	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)		
Static	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	0.001	6.464	0.001	18.470	0.002	45.354	0.001	70.040	0.001	42.0	0.0010	42.0	0.0010	42.0	0.0010	42.0	0.0010	42.0	0.0010	42.0	0.0010	42.0	0.0010	115.0	0.0015	175.9	0.0019	250.2	0.0024	338.3	0.0010	603.0	0.0012	820.6		
	0.003	11.343	0.003	32.413	0.004	79.593	0.001	99.052	0.002	59.4	0.0020	59.4	0.0020	59.4	0.0020	59.4	0.0020	59.4	0.0020	59.4	0.0020	59.4	0.0020	201.9	0.0029	308.7	0.0039	439.1	0.0049	593.6	0.0019	1058.3	0.0025	1440.1		
	0.004	14.302	0.004	40.867	0.006	100.354	0.002	115.399	0.003	69.2	0.0031	69.2	0.0031	69.2	0.0031	69.2	0.0031	69.2	0.0031	69.2	0.0031	69.2	0.0031	254.6	0.0044	389.2	0.0058	553.7	0.0073	748.5	0.0029	1334.3	0.0037	1815.7		
	0.006	15.864	0.006	45.331	0.008	111.314	0.003	126.014	0.004	75.6	0.0041	75.6	0.0041	75.6	0.0041	75.6	0.0041	75.6	0.0041	75.6	0.0041	75.6	0.0041	282.4	0.0058	431.7	0.0077	614.1	0.0098	830.2	0.0039	1480.0	0.0050	2014.0		
	0.007	16.629	0.007	47.515	0.010	116.679	0.003	133.264	0.005	80.0	0.0051	80.0	0.0051	80.0	0.0051	80.0	0.0051	80.0	0.0051	80.0	0.0051	80.0	0.0051	296.0	0.0073	452.5	0.0097	643.7	0.0122	870.2	0.0048	1551.3	0.0062	2111.1		
	0.009	16.989	0.008	48.545	0.012	119.208	0.004	138.107	0.006	82.9	0.0061	82.9	0.0061	82.9	0.0061	82.9	0.0061	82.9	0.0061	82.9	0.0061	82.9	0.0061	302.4	0.0088	462.3	0.0116	657.7	0.0147	889.1	0.0058	1585.0	0.0075	2156.8		
	0.010	17.156	0.010	49.022	0.014	120.379	0.005	141.090	0.007	84.7	0.0072	84.7	0.0072	84.7	0.0072	84.7	0.0072	84.7	0.0072	84.7	0.0072	84.7	0.0072	305.4	0.0102	466.8	0.0135	664.1	0.0171	897.8	0.0068	1600.5	0.0087	2178.0		
	0.012	17.233	0.011	49.241	0.016	120.917	0.005	142.566	0.008	85.5	0.0082	85.5	0.0082	85.5	0.0082	85.5	0.0082	85.5	0.0082	85.5	0.0082	85.5	0.0082	306.7	0.0117	468.9	0.0155	667.1	0.0196	901.8	0.0077	1607.7	0.0099	2187.7		
	0.013	17.268	0.012	49.341	0.018	121.163	0.006	142.782	0.009	85.7	0.0092	85.7	0.0092	85.7	0.0092	85.7	0.0092	85.7	0.0092	85.7	0.0092	85.7	0.0092	307.3	0.0131	469.9	0.0174	668.5	0.0220	903.6	0.0087	1611.0	0.0112	2192.2		
	0.014	17.284	0.014	49.387	0.020	121.275	0.007	141.915	0.010	85.1	0.0102	85.1	0.0102	85.1	0.0102	85.1	0.0102	85.1	0.0102	85.1	0.0102	85.1	0.0102	307.6	0.0146	470.3	0.0193	669.1	0.0245	904.5	0.0097	1612.4	0.0124	2194.2		
	0.016	17.291	0.015	49.408	0.022	121.326	0.007	140.104	0.011	84.1	0.0113	84.1	0.0113	84.1	0.0113	84.1	0.0113	84.1	0.0113	84.1	0.0113	84.1	0.0113	307.8	0.0161	470.5	0.0213	669.4	0.0269	904.9	0.0106	1613.1	0.0137	2195.1		
	0.017	17.295	0.017	49.417	0.024	121.349	0.008	137.456	0.012	82.5	0.0123	82.5	0.0123	82.5	0.0123	82.5	0.0123	82.5	0.0123	82.5	0.0123	82.5	0.0123	307.8	0.0175	470.6	0.0232	669.5	0.0293	905.0	0.0116	1613.4	0.0149	2195.6		
	0.019	17.296	0.018	49.422	0.026	121.360	0.013	99.150	0.020	59.5	0.0205	59.5	0.0205	59.5	0.0205	59.5	0.0205	59.5	0.0205	59.5	0.0205	59.5	0.0205	307.8	0.0190	470.6	0.0251	669.6	0.0318	905.1	0.0126	1613.6	0.0161	2195.8		
	0.020	17.297	0.019	49.424	0.028	121.365	0.018	60.788	0.029	36.5	0.0286	36.5	0.0286	36.5	0.0286	36.5	0.0286	36.5	0.0286	36.5	0.0286	36.5	0.0286	307.9	0.0204	470.6	0.0271	669.6	0.0342	905.2	0.0135	1613.6	0.0174	2195.9		
	0.022	17.297	0.021	49.425	0.030	121.367	0.023	22.425	0.037	13.5	0.0368	13.5	0.0368	13.5	0.0368	13.5	0.0368	13.5	0.0368	13.5	0.0368	13.5	0.0368	307.9	0.0219	470.6	0.0290	669.6	0.0367	905.2	0.0145	1613.7	0.0186	2195.9		
	0.023	17.297	0.022	49.425	0.033	121.368	0.025	22.425	0.039	13.5	0.0389	13.5	0.0389	13.5	0.0389	13.5	0.0389	13.5	0.0389	13.5	0.0389	13.5	0.0389	307.9	0.0234	470.7	0.0309	669.6	0.0391	905.2	0.0155	1613.7	0.0199	2195.9		

* Depth is measured below the existing native ground surface (elevation 85 m)
 ** The values P(kN/m) represent soil reaction per metre of pile length; The values y(m) represent soil/pile deflection
 The following assumptions were made in the analysis:

1. The analysis was completed for a vertical pile (i.e. no inclination) and flat ground
2. The effects of construction disturbance or dredging is not considered. not included, the static p-y curves may be used under seismic loading.
3. Not applicable for bedrock.

NOTES:

1. The p-y data provided is unfactored. Lateral resistance or deflection calculated based on these parameters should be factored using the geotechnical resistance factors (ϕ_{gu} and ϕ_{gs}) provided in Table 6.2 of the CHBDC (S6-14)
2. If lateral spacing between an adjacent pile or another structural element is less than four equivalent pile diameters, suitable reduction factors based on center to center spacing should be applied based on Figures C6.11.3(r), C.6.11.3(s) and C6.11.3(t) of the CHBDC (S6-14)

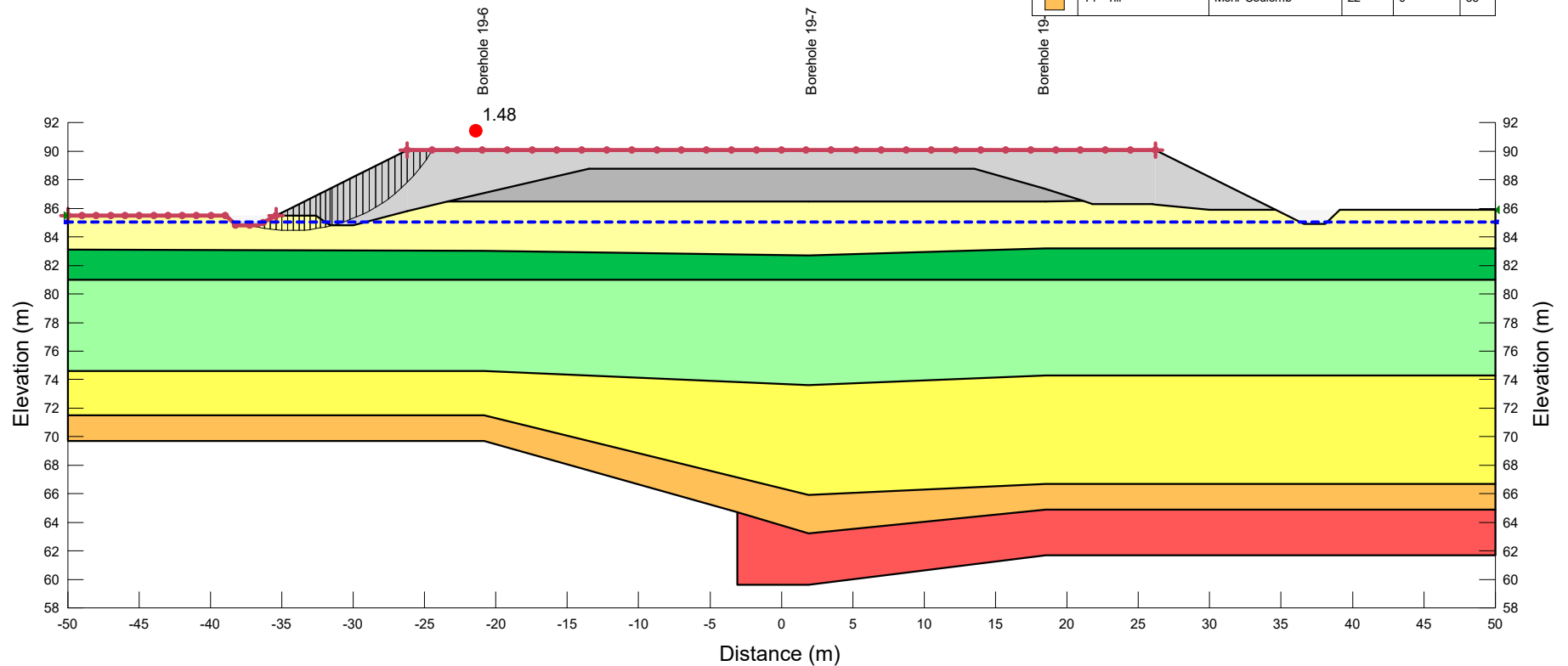


Date: 2020-03-01
 Project No.: 27253

Prepared By: DJP
 Checked By: PC

Appendix I.
Slope Stability

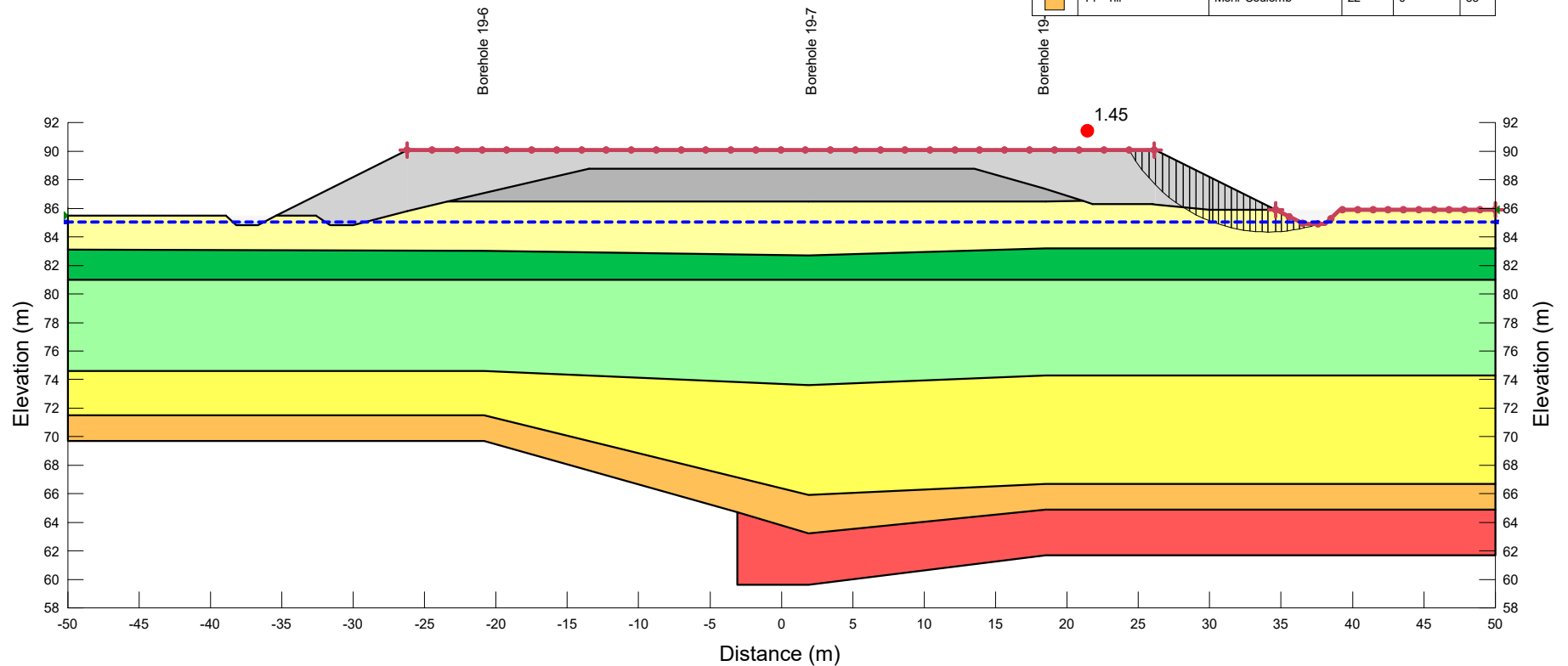
Color	Name	Model	Unit Weight (kN/m ³)	Cohesion* (kPa)	Phi* (°)
Red	B1 - Bedrock	Bedrock (Impenetrable)			
Green	C1 - Clayey SILT to CLAY 1 (ESA)	Mohr-Coulomb	17	0	30
Light Green	C2 - Clayey SILT to CLAY 2 (ESA)	Mohr-Coulomb	19	0	28
Grey	F1 - Existing FILL	Mohr-Coulomb	20	0	30
Light Grey	F1 - New FILL	Mohr-Coulomb	22	0	32
Yellow	S1 - Upper Silty SAND to SAND	Mohr-Coulomb	19	0	30
Light Yellow	S2 - Lower Silty Sand to SAND	Mohr-Coulomb	21	0	30
Orange	T1 - Till	Mohr-Coulomb	22	0	35



Project		Napanee River Bridge	
Analysis		EA-N-1.0 - Long Term (Drained)	
Seismic Coefficient	Last Run	Scale	
H: g, V: g	03/25/2021, 01:30:03 PM	1:450	

Additional Details
Name: East Abutment - North
Comments:
Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.5 m
Entry: (-24.453333, 90.1) m, Exit: (-37.2295, 84.8) m
Center: (-34.367285, 95.949449) m, Radius: 11.510972 m

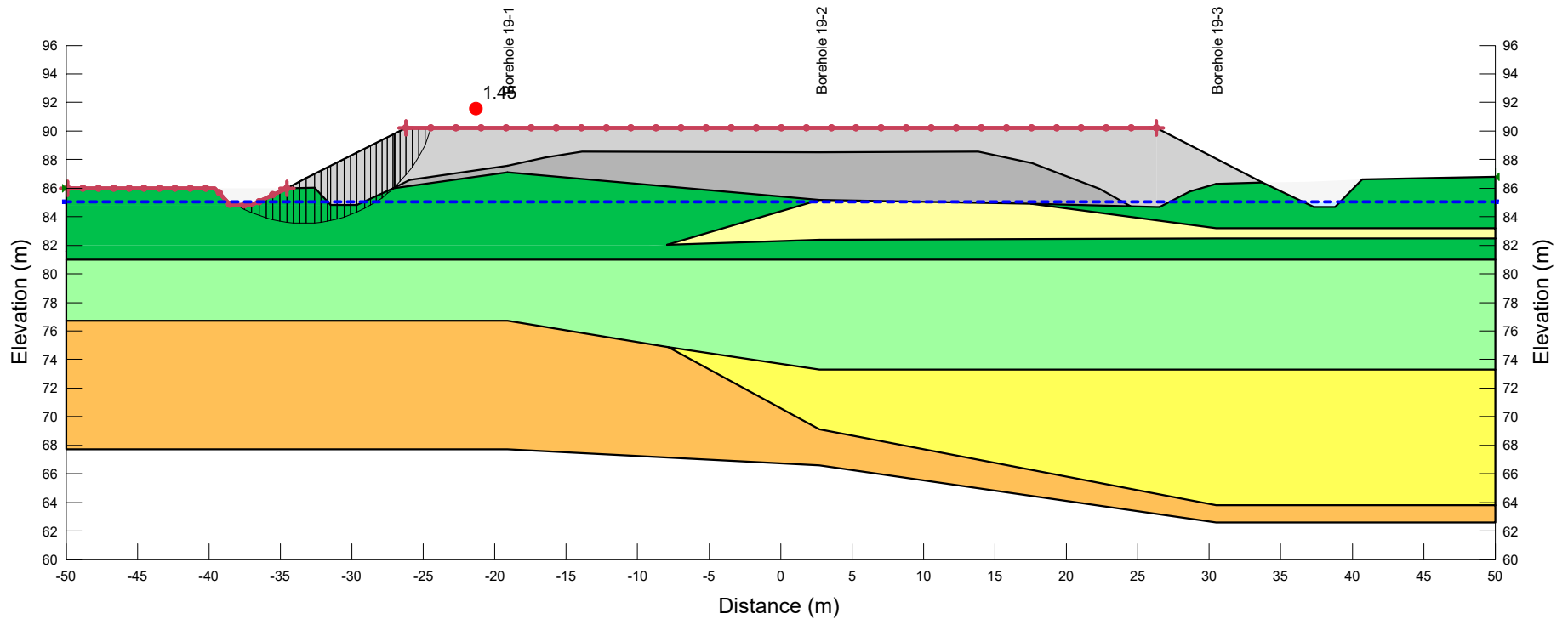
Color	Name	Model	Unit Weight (kN/m ³)	Cohesion* (kPa)	Phi* (°)
Red	B1 - Bedrock	Bedrock (Impenetrable)			
Green	C1 - Clayey SILT to CLAY 1 (ESA)	Mohr-Coulomb	17	0	30
Light Green	C2 - Clayey SILT to CLAY 2 (ESA)	Mohr-Coulomb	19	0	28
Grey	F1 - Existing FILL	Mohr-Coulomb	20	0	30
Light Grey	F1 - New FILL	Mohr-Coulomb	22	0	32
Yellow	S1 - Upper Silty SAND to SAND	Mohr-Coulomb	19	0	30
Light Yellow	S2 - Lower Silty Sand to SAND	Mohr-Coulomb	21	0	30
Orange	T1 - Till	Mohr-Coulomb	22	0	35



Project Napanee River Bridge		
Analysis EA-S-1.0 - Long Term (Drained)		
Seismic Coefficient H: g, V: g	Last Run 03/25/2021, 01:30:36 PM	Scale 1:450

Additional Details
Name: East Abutment - South
Comments:
Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.5 m
Entry: (24.356667, 90.1) m, Exit: (37.573988, 84.9) m
Center: (34.096822, 95.459609) m, Radius: 11.117375 m

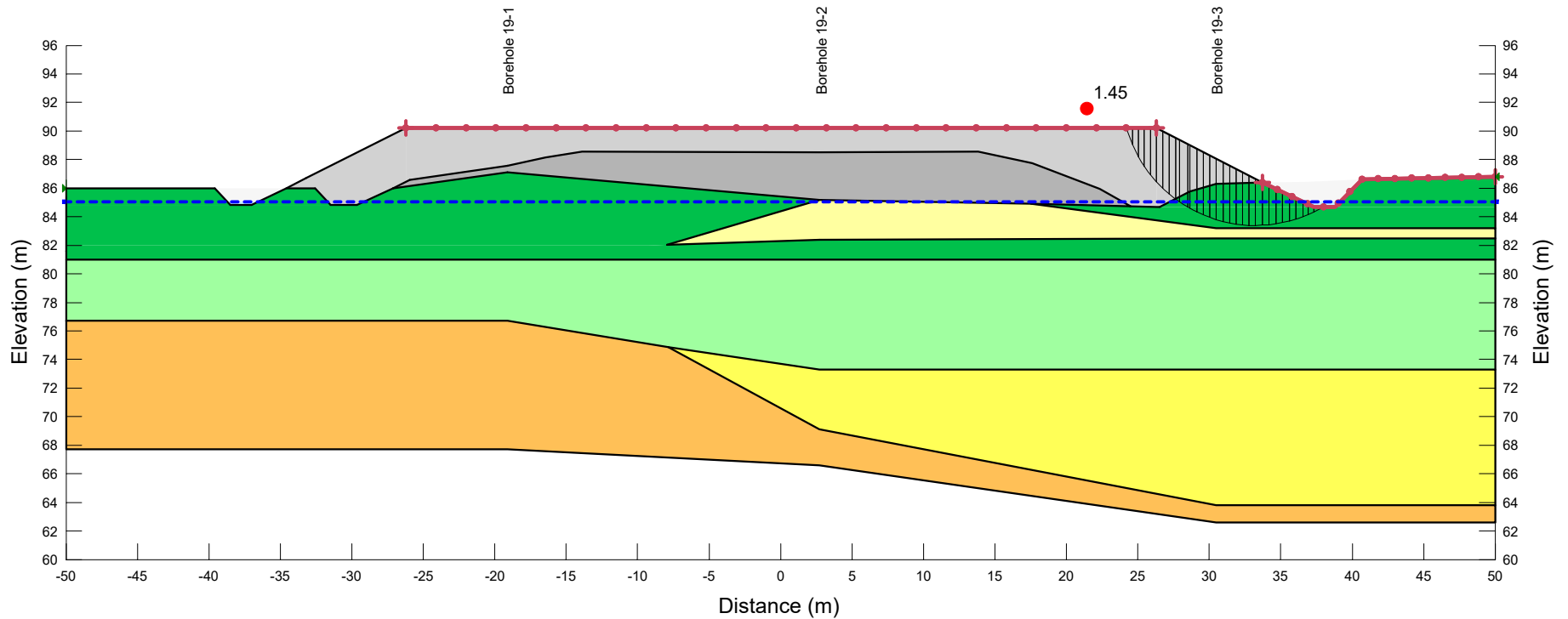
Color	Name	Model	Unit Weight (kN/m ³)	Cohesion (kPa)	Phi (°)
Green	C1 - Clayey SILT to CLAY 1 (ESA)	Mohr-Coulomb	17	0	30
Light Green	C2 - Clayey SILT to CLAY 2 (ESA)	Mohr-Coulomb	19	0	28
Grey	F1 - Existing FILL	Mohr-Coulomb	20	0	30
Light Grey	F1 - New FILL	Mohr-Coulomb	22	0	32
Yellow	S1 - Upper Silty SAND to SAND	Mohr-Coulomb	19	0	30
Light Yellow	S2 - Lower Silty Sand to SAND	Mohr-Coulomb	21	0	30
Orange	T1 - Till	Mohr-Coulomb	22	0	35



Project		Napanee River Bridge	
Analysis		WA-N-1.0 - Long Term (Drained)	
Seismic Coefficient	Last Run	Scale	
H: g, V: g	03/25/2021, 01:30:46 PM	1:450	

Additional Details	
Name: West Abutment - North	
Comments:	
Method: Morgenstern-Price, Half-Sine	
Minimum Slip Surface Depth: 1.5 m	
Entry: (-24.45, 90.2) m, Exit: (-38.041736, 84.8) m	
Center: (-33.373518, 92.855269) m, Radius: 9.310189 m	

Color	Name	Model	Unit Weight (kN/m ³)	Cohesion (kPa)	Phi (°)
Green	C1 - Clayey SILT to CLAY 1 (ESA)	Mohr-Coulomb	17	0	30
Light Green	C2 - Clayey SILT to CLAY 2 (ESA)	Mohr-Coulomb	19	0	28
Grey	F1 - Existing FILL	Mohr-Coulomb	20	0	30
Light Grey	F1 - New FILL	Mohr-Coulomb	22	0	32
Yellow	S1 - Upper Silty SAND to SAND	Mohr-Coulomb	19	0	30
Light Yellow	S2 - Lower Silty Sand to SAND	Mohr-Coulomb	21	0	30
Orange	T1 - Till	Mohr-Coulomb	22	0	35



Project		Napanee River Bridge	
Analysis		WA-S-1.0 - Long Term (Drained)	
Seismic Coefficient	Last Run	Scale	
H: g, V: g	03/25/2021, 01:30:09 PM	1:450	

Additional Details
Name: West Abutment - South
Comments:
Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.5 m
Entry: (24.2, 90.2) m, Exit: (37.985142, 84.689561) m
Center: (33.22211, 92.772124) m, Radius: 9.3815936 m

Appendix J.

List of Referenced Specifications and NTCs

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

- OPSS.PROV 206
- OPSS.PROV 501
- OPSS 511
- OPSS.PROV 517
- OPSS.PROV 539
- OPSS.PROV 804
- OPSS 805
- OPSS 902
- OPSS 903
- OPSS.PROV 1010
- OPSD 802.010
- OPSD 3090.101
- OPSD 3101.150
- SP105S09
- SP109S12
- SP517F01
- FOUN0003

2. Notice to Contractor – Variable Refusal Conditions

“The depths to refusal at this site are expected to be variable. It should be noted that different pile lengths will be required across the site.”

3. Notice to Contractor – Buried Obstructions

“The contractor is advised that obstructions (e.g. debris from original bridge construction) may be encountered during excavation, the installation of temporary protection systems and pile driving operations. Such obstructions may impede the work from reaching design depth of installation. The constructor shall be prepared to remove, drill through and/or penetrate these obstructions and extend the work to the design depths.”

4. Notice to Contractor – Vibratory Hammer

“The use of vibration during TPS installation and removal could result in settlement and must be carried out cautiously.”

5. Notice to Contractor – Dewatering Excavations

“Dewatering will be required to excavate below the groundwater level to maintain a dry, sound base on which to work. Disturbance of the subgrade soils is considered to be a significant risk without proper consideration of groundwater lowering. The groundwater level should be lowered to 0.5 m below the planned base of excavation for each stage of excavation.”