



THURBER ENGINEERING LTD.

**FOUNDATION INVESTIGATION AND DESIGN REPORT
LARONDE CREEK BRIDGE REPLACEMENT
HIGHWAY 17, 20.3 KM WEST OF HIGHWAY 11
SITE NO. 43X-0065/B0**

G.W.P. NO. 5198-13-00

Geocres Number: 31L-224

Report

to

McIntosh Perry Consulting Engineers

**Latitude: 46.370246°
Longitude: -79.710742°**

August 2020

Thurber File No.: 23411



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PART 1: FACTUAL INFORMATION

1. INTRODUCTION

This section of the report presents the factual findings obtained from a preliminary desktop study and subsequent foundation investigation for detailed design completed at the Highway 17 bridge crossing over Laronde Creek, which is located about 20.3 km west of Highway 11 near North Bay, Ontario.

The purpose of this investigation was to supplement the subsurface conditions identified in the desktop study 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.

Thurber Engineering Ltd. (Thurber) was retained by McIntosh Perry Consulting Engineers (MPCE) to provide foundations engineering services for this project. This work is being carried out under assignment number 5018-E-0014.

2. SITE DESCRIPTION

The existing bridge was constructed in 1938 and is a two-lane single span structure. The overall open span and width of the structure are 13.7 and 10.2 m, respectively. The existing substructure consists of reinforced cast-in-place concrete abutments founded on timber piles. A sidewalk and gabion retaining wall were added to the south side of the structure in 2009.

The highway at this location has an east-west orientation and Laronde Creek has a north-south orientation, flowing to the south towards Lake Nipissing. Laronde Creek is about 12 m wide. The water level in the creek was reported to be about Elevation 195.7 m in October 2018. The road surface at the abutments is at about elevation 201.8 m based on the elevation of existing on-road boreholes (BH 4 and BH 6).

The embankment side slopes on the north side of the approaches are sloped at approximately 2H:1V. A perched gabion basket retaining wall, up to about 2.5 m in height, is present on the



south side of the approaches, with approximately 2H:1V sides slopes observed below the base of the retaining wall near the abutments.

A CN rail bridge crossing is located about 20 m south of the highway. To the north of the highway, the west creek bank has a shallow slope with an old asphalt paved access road to a boat launch and dock at the creek edge. On the east creek bank, the slope is steep and heavily vegetated with trees. An overhead power line is present about 15 m north of the highway. A ditch is also present on the north side of the highway at the toe of the approach embankments both on the east and west sides of the bridge.

Select photographs of the site are included in Appendix D.

3. DESKTOP STUDY

3.1 General

A desktop study was completed at the preliminary design phase based on a site reconnaissance and a review of geotechnical data gathered from available sources; no borehole drilling or sampling was carried out for this phase of the work. Information on the existing surface and subsurface conditions relevant to the foundations of the existing structures and embankments are summarized in the sections below.

3.2 MTO GEOCREs Files

Thurber has reviewed the following Foundation Investigation and Design Reports (FIDR) available from the Geocres library.

- Report by Thurber Engineering Ltd. to Stantec Consulting Group Ltd., titled “Preliminary Foundation Investigation and Design Report for Proposed Bridge over Laronde Creek – Site 43-65, Highway 17 from North Bay to Sturgeon Falls, W.P. 812-76-00 & 398-91-00, District 54, Sudbury”, dated January 19, 1999 (File No. 19-2487-0A) [Geocres No. 31L-72].
- Report by Golder Associates Ltd. to McCormick Rankin Corporation, titled “Foundation Investigation and Design, Laronde Creek Bridge Replacement, Highway 17, Site 43-65, W.P. 812-76-02, District 54, Sudbury, G.W.P 812-76-01”, dated June 2000 (Report No. 991-1164-1) [Geocres No. 31L-70].
- Report by Trow Associates Inc. to Ministry of Transportation, titled “Foundation Investigation and Design Report, Gabion Wall Construction near Laronde Creek, Hwy 17, District 54, G.W.P. 5274-08-00”, dated October 15, 2009 (File No. SD000360624d) [Geocres No. 31L-137].

The factual information from the GEOCREs reports (e.g., borehole plans, boreholes records and laboratory testing results) are provided in Appendix E.



It must be recognized that the service providers that produced the historical FIDR reports are solely responsible for the accuracy and quality of the subsurface information presented in their respective reports and that conditions, particularly near ground surface, may have changed subsequent to the investigations.

3.3 Archived Drawings

Archived drawings from 1937 prepared by the Department of Highways Ontario (Contract 37-128) were reviewed as part of this assessment; two of the drawings are provided in Appendix F.

3.4 Inspection Reports

A biannual inspection was completed by others in 2016 using the Ontario Bridge Management System (OBMS) to assess the condition of the existing structure through visual inspection. That report indicated deterioration of several elements of the Laronde Creek Bridge, including (but not limited to): light to severe scour at the base of the abutments, severe scaling and spalls of abutments and wing walls exposing corroded rebar, erosion of the northwest embankment, and settlement of the west approach wearing surface with corresponding ponding.

3.5 Site Reconnaissance Visit

A site reconnaissance visit was carried out by a Thurber Geotechnical Engineer on October 26, 2018 as part of the preliminary foundation design assessment. The project site was visited and visible geological/geotechnical features were documented and assessed with respect to structure foundation and embankment performance.

Based on site observations, it is evident that the existing approach embankments have experienced settlement, as evidenced by tension cracks in the asphalt at the edges of the abutment, surface ponding, and the presence of newer asphalt placed only at the approaches (paved in 2007 per the OBMS 2016 inspection report). The settlement that was visible at the time of the site visit was estimated to be in the range of 50 to 100 mm; it is considered that it may be indicative of long-term consolidation settlement of the underlying clay deposit. Higher magnitudes of settlement were evident on the west embankment versus the east embankment.

On the embankment side slopes, some settlement (approximately 50 to 75 mm) has occurred near the eastern end of the gabion basket retaining wall. There was no evidence of global slope instability noted during the site reconnaissance. Some erosional channels have cut through the embankment fill in some areas, most notably along the northwest wing wall and at the two sidewalk wooden crib walls, which have been undermined.

A previously installed piezometer was observed to have active artesian flow at the time of the site visit. This well was subsequently sealed during the fieldwork that was carried out for the detailed design foundation investigation in August 2019 (discussed in Section 4 below).



4. SITE INVESTIGATION AND FIELD TESTING

The investigation and field testing program was carried out between August 8th and 17th, 2019. The field investigation consisted of advancing five boreholes identified as 19-01, 19-02A, 19-02B, 19-03 and 19-04. Borehole 19-02A was advanced for the purpose of obtaining consecutive Shelby tube samples. One seismic cone penetration test (SCPT) identified as SCPT19-02 was pushed adjacent to Boreholes 19-02A and 19-02B.

Boreholes 19-01, 19-02A and 19-02B were advanced using a truck-mounted CME 55 drill rig off of the access roadway. Boreholes 19-03 and 19-04 were advanced using a portable Explo 220 drill rig. All boreholes were advancing using wash-boring techniques except for Borehole 19-02A which was advanced using mud-rotary techniques. SCPT19-02 was pushed from the truck-mounted CME 55 drill rig with portable cone penetration equipment. Prior to commencement of drilling, utility clearances were obtained in the vicinity of the borehole locations.

Table 4-1: Summary of Boreholes – Current Investigation

Borehole / SCPT No.	Foundation Element	Northing (m)	Easting (m)	Ground Surface Elevation (m)	Termination Depth Below Ground Surface (m)
19-01	West Approach	5136773.2	288543.8	199.8	11.9
19-02A	West Abutment	5136773.2	288554.2	198.4	16.6
19-02B		5136772.9	288556.8	198.3	5.2
SCPT19-02		5136772.5	288559.3	198.1	17.9
19-03	East Abutment	5136775.2	288596.7	197.8	20.0
19-04	East Approach	5136774.9	288616.4	202.8	11.9

In Boreholes 19-01, 19-02B, 19-03 and 19-04, soil samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT) following ASTM D1586. Field vane testing was carried out in the cohesive deposits at selected depths using an MTO N vane. Borehole 19-03 was advanced into bedrock using rotary diamond drilling techniques while collecting NQ sized bedrock core. SCPT19-02 was advanced to refusal on very dense till or bedrock. Borehole 19-02A was advanced to Shelby tube refusal. Boreholes 19-01, 19-02B and 19-04 were advanced to their required termination depth.

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



Vibrating wire piezometers (VWP) were installed in Boreholes 19-02A and 19-03 to allow for measurements of the porewater pressure after completion of drilling. The VWP installation details are illustrated on the Record of Borehole sheet provided in Appendix B. All other boreholes were backfilled with a low-permeability mixture of cuttings and bentonite pellets in accordance with Ontario MOE Regulation 903 as amended.

5. LABORATORY TESTING

Geotechnical laboratory testing consisted of natural moisture content determination and visual identification of all retained split-spoon soil samples. Grain size distribution and Atterberg limits testing was also carried out on selected samples to MTO and ASTM standards. All rock cores were photographed and their total core recovery (TCR), solid core recovery (SCR) and rock quality designation (RQD) were measured. Two samples of the bedrock core were submitted for unconfined compressive strength (UCS) testing. One-dimensional (incremental loading) consolidation testing was carried out on four samples. Two of those samples were subjected to long-term creep testing at a selected stress interval. Four samples were submitted for constant rate of strain (CRS) consolidation testing; a pair of samples were taken from two different elevations and subjected to different strain rates. One sample was submitted for triaxial testing (consolidated-undrained). Chemical analysis for determination of pH, conductivity, resistivity, sulphate, sulphide and chloride concentrations was carried out on four soil samples.

The results of the geotechnical tests are summarized on the Record of Borehole sheets included in Appendix B and all laboratory results are presented on the figures included in Appendix C.

6. SUBSURFACE CONDITIONS

Details of the soil stratigraphy encountered during the current investigation are presented on the Record of Borehole sheets and SCPT log included in Appendix B and the Borehole Location and Soil Strata drawings included in Appendix A. Detailed descriptions of the previous field investigation methodologies and results are presented in the individual reports listed in Section 3.2. Details of the soil stratigraphy encountered during the previous investigations are presented on the Record of Borehole sheets and the Borehole Locations and Soil Strata drawings provided in Appendix E. The laboratory test results from the previous investigations are also 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 governs any interpretation of the site conditions. It must be recognized that the soil and groundwater conditions may vary between and beyond borehole locations.

Soil descriptions are in accordance with the Unified Soil Classification System, ASTM D2487, as modified by current MTO standards for cohesionless soils.

In general terms, subsurface conditions at the site consist of fill and/or surficial sands/silts overlying an approximately 19 m thick deposit of varved clayey silt to silty clay, which is underlain by a thin layer of sand till over granitic bedrock. A summary of the subsurface information from the current investigation is presented in the following sections. Information from previous investigations that is considered pertinent to the foundation design is also included in the discussion below.



6.1 Topsoil

Topsoil was encountered at the surface in Boreholes 99-43, 99-45, 3, 5 and 19-04. The topsoil ranged from 15 mm to 600 mm thick. One SPT N value of 6 was recorded in this layer indicating a loose relative density. One moisture content on a tested sample was 25%.

6.2 Asphalt & Concrete

Asphalt was encountered at surface in Boreholes 1, 2, 4 and 6 advanced from Highway 17 and in Boreholes 99-44, 19-01 and 19-02B advanced from the boat ramp access road to the north of the highway. The thickness of the asphalt ranged from 50 mm to 300 mm. A 100 mm thick layer of concrete was encountered below the asphalt in Borehole 19-01.

6.3 Fill

Fill associated with the existing highway embankment and the old asphalt paved boat launch was encountered in Boreholes 98-3 to 98-6, 99-44, 1 to 7, 19-01 and 19-02B. Where present, the fill ranges in thickness from 0.6 to 3.0 m with the underside of the layer at elevations ranging from 194.4 m to 201.0 m. The fill generally consists of sandy gravel, to sand, to sand with silt, to silty sand, to sandy silt. SPT N values in this material ranged from 2 to 25 blows, indicating a very loose to compact relative density. The moisture content on tested samples typically ranged from 2% to 34%. One moisture content of 86% was recorded.

Gradation analyses were completed on five samples in the fill (one from the current investigation). The grain size distribution curve for the sample from the current investigation is included in Figure C1 in Appendix C. The results from all five tests are summarized in Table 4-1 below and the results are presented on the corresponding Record of Borehole sheets in Appendix B and Appendix E for the current and previous investigations, respectively.

Table 4-1: Summary of Gradation Test Results – Fill

Soil Particle	Percentage (%)
Gravel	0 to 64
Sand	3 to 70
Silt and Clay	6 to 97

6.4 Surficial Sand and Silt

A thin discontinuous surficial deposit of sand and silt was encountered in Boreholes 98-1, 98-2, 99-45, 3, 4, 6, 19-03 and 19-04. Where encountered, the surficial sand and silt is 0.8 to 4.4 m thick with the underside of the layer at elevations ranging from 195.7 m to 200.1 m. SPT N values in this material ranged from 2 to 12 blows, indicating a very loose to compact relative density. The moisture content on tested samples typically ranged from 11% to 31%. One moisture content of 61% was recorded.



Gradation analyses were completed on six samples of this deposit (three from the current investigation). The grain size distribution curves for the samples from the current investigation are included in Figure C2 in Appendix C. The results from all six tests are summarized in Table 4-2 below and the results are presented on the corresponding Record of Borehole sheets in Appendix B and Appendix E for the current and previous investigations, respectively.

Table 4-2: Summary of Gradation Test Results – Surficial Sand and Silt

Soil Particle	Percentage (%)
Gravel	0 to 10
Sand	0 to 56
Silt	31 to 93
Clay	3 to 13

Atterberg Limits tests were completed on four samples (three from the current investigation and one from the previous investigations). The results from the current investigation indicated that the sand and silt samples were non-plastic, and the test result from the previous investigation gave a liquid limit of 25 and plastic limit of 23, indicating a low plastic silt (ML). It is noted that this soil had a high susceptibility to erosion.

6.5 Clayey Silt (CL-ML to CL) to Clay (CI to CH)

A thick layer of compressible glaciolacustrine varved clay was encountered beneath the surficial deposits described above in all boreholes. The upper portion of the deposit is weathered and is generally described as clayey silt with trace sand. The upper clayey silt ranges in thickness from 0.8 to 3.8 m with bottom elevation ranging from 194.1 to 199.9 m. The lower clayey silt to clay is generally described as being varved with layers of silty clay and clayey silt. Occasional sand seams are also noted within the deposit. Where fully penetrated, the lower clay deposit ranges in thickness from 13.1 m to 19.1 m with bottom elevations ranging from 178.5 m to 181.1 m.

SPT 'N' values in the upper clayey silt deposit ranged from weight-of-hammer to 19 blows. One field vane test gave an undrained shear strength (s_u) of 39 kPa. These in situ test results indicate a firm to very stiff consistency for the upper clayey silt. Field vane testing conducted in the lower clayey clay deposit indicate that the s_u generally ranges from about 17 kPa (soft) at the top of the deposit to 100 kPa (stiff) at the bottom of the deposit, with a generally linear increase with depth. It is noted that there is variation in the vane results (± 20 kPa) between the three firms that carried out the testing. Remolded vane testing indicates that the sensitivity of the deposit generally ranges from 2 to 4 (medium sensitivity).

Eight dissipation tests were carried out during the advancement of the SCPT19-02. The results of that testing are summarized in Appendix B and indicate that the coefficient of consolidation in the horizontal direction, C_h , ranges from $1.4E-2$ to $8.6E-1$ cm^2/s . Shear wave velocity measurements were also taken during advancement of the SCPT, with the measured shear wave velocities in the clay deposit ranging from 81 to 206 m/s. Further details on the shear wave velocity testing results are provided in Appendix B.



Moisture contents in the upper clayey silt ranged from 18 to 41% and the moisture contents in the lower clay ranged from 21 to 68%.

Gradation analyses were completed on forty-two samples of this deposit (14 from the current investigation). The grain size distribution curves for the samples from the current investigation are included in Figures C3 to C6 in Appendix C. Previous lab test results are presented in Appendix E. The results from all of the tests are summarized in Table 4-3 below and the results are presented on the corresponding Record of Borehole sheets in Appendix B and Appendix E for the current and previous investigations, respectively.

Table 4-3: Summary of Gradation Test Results – Clay

Soil Particle	Percentage (%)		
	Upper Clayey Silt	Cohesive Lower Varved Clay	Non-Cohesive Silt Layer/Varve (19-02A, TW-15)
Gravel	0	0	0
Sand	0 to 1	0 to 2	4
Silt	63 to 80	25 to 71	82
Clay	20 to 37	27 to 73	14

Atterberg limits testing was completed on fifty-one samples (19 from the current investigation). All of the results are summarized in Table 4-4 below and the tests from the current investigation are summarized on Figures C8 to C11 in Appendix C.

Table 4-4: Summary of Atterberg Limits Test Results – Clay

Parameter	Value		
	Upper Clayey Silt	Cohesive Lower Varved Clay	Non-Cohesive Silt Layer/Varve (19-02A, TW-15)
Plastic Limit	19 to 24	14 to 25	Non-plastic
Liquid Limit	24 to 32	19 to 57	

The results of the testing indicates that the upper portion of the deposit is low plasticity (CL-ML) and the lower portion of the deposit ranges from low (CL) to high plasticity (CH).

One-dimensional (incremental loading) consolidation testing and constant rate of strain (CRS) testing was carried out on selected samples from Borehole 19-02A. One-dimensional consolidation testing was also completed on two samples from Borehole 99-45 during one of the previous investigations, although only one of those tests was considered reliable (SA9). The results of the testing are provided in Appendix C and Appendix E, and are summarized in Table 4-5 below.



Table 4-5: Consolidation Test Parameters

Borehole	19-02A							
Parameter	TW1	TW5	TW8	TW15	TW4 / SA1	TW4 / SA2	TW6 / SA3	TW6 / SA4
Sample Depth (m)	3.4	5.8	7.6	12.5	5.2	5.2	6.4	6.4
Sample Elevation (m)	195.0	192.6	190.8	185.9	193.2	193.2	192.0	192.0
Strain Rate (%/hr) for CRS	-	-	-	-	3.0%	0.10%	1.0%	0.30%
Natural Moisture Content, w_n (%)	46.0	51.2	47.2	26.5	64.3	57.4	46.5	42.9
Initial Void Ratio, e_o (-)	1.27	1.45	1.43	1.01	1.81	1.68	1.18	1.14
Unit Weight (kN/m^3)	17.1	16.5	16.2	16.9	15.9	16.0	18.3	18.2
In-situ Vertical Effective Stress, p_o' (kPa)	45	60	71	110	54	54	73	73
Preconsolidation Pressure, p_c' (kPa)	137	150	140	185	210	157	175	165
Overconsolidation Ratio, OCR (-)	3.0	2.5	2.0	1.7	3.9	2.9	2.4	2.3
Recompression Index, C_r	0.052	0.063	0.075	0.072	0.102	0.100	0.107	0.089
Coefficient of Consolidation, c_{vr} (m^2/yr)	28.0 - 53.2	17.5 - 32.2	21.1 - 56.6	26.4 - 44.6	-	-	-	-
Compression Index, C_c	0.67	1.01	1.06	0.61	0.96	1.48	0.53	0.48
Coefficient of Consolidation, c_v (m^2/yr)	2.0 - 5.7	0.7 - 3.5	1.5 - 5.0	1.5 - 4.4	-	-	-	-
Secondary Compression Index, $C_{\alpha}(-)$	0.015	0.015	-	-	-	-	-	-



Triaxial testing (consolidated-undrained) was also carried out on one sample (TW3 from Borehole 19-02A). The results of the triaxial testing are presented in Appendix C.

6.6 Till – Sand to Silty Sand to Sandy Silt with Gravel

A thin deposit of till consisting of sand, to silty sand, to sandy silt, and containing gravel, cobbles and boulders is was encountered below the clay in Boreholes 98-2, 98-3, 98-4, 99-43, 99-44, 99-45, 1, 2 and 19-03. Where encountered, the till is 0.2 to 4.2 m thick with an underside elevation ranging from 174.5 m to 180.7 m. SPT N values in this material were generally 27 to greater than 50 blows per 50 mm penetration, indicating a compact to very dense relative density. The moisture content on the tested samples ranged from 10% to 28%.

Gradation analyses were completed on two samples of this deposit (one from the current investigation). The grain size distribution curve for the sample from the current investigation is included in Figure C7 in Appendix C. Both of the test results are summarized in Table 4-6 below and the results are presented on the corresponding Record of Borehole sheets in Appendix B and Appendix E for the current investigation and previous investigation, respectively.

Table 4-6: Summary of Gradation Test Results – Till

Soil Particle	Percentage (%)
Gravel	6 to 16
Sand	18 to 84
Silt	8 to 54
Clay	2 to 12

Atterberg limits testing was completed on one sample from the current investigation. The plastic and liquid limits were 16 and 18, respectively, indicating a low plasticity silt (ML). The test result is summarized on Figure C12 in Appendix C.

6.7 Bedrock

Eight boreholes were cored into the bedrock and two others were terminated upon auger refusal on the assumed bedrock surface. Borehole 19-03 from the current investigation was advanced into bedrock by coring. The bedrock is generally described as fresh, very strong to extremely strong, undifferentiated Precambrian granite of the Canadian Shield. Two unconfined compressive strength (UCS) tests completed on the rock yielded strengths of 147 and 162 MPa, indicating very strong rock. The UCS test results are presented in Appendix C. Rock Quality Designation (RQD) values generally range between 50 and 80%, indicating fair to good quality rock; however, the upper 3 m of bedrock at Borehole 99-43 was weathered and contained significantly more fractures (RQD 0 to 5%), indicating very poor rock at that location. The upper bedrock was also fractured in Borehole 19-03 (RQD 0 to 25%). Photographs of the bedrock core are presented in Appendix C.



The bedrock surface depths and elevations encountered in the current and previous boreholes are summarized in the following table.

Table 4-7: Summary of Bedrock Surface Depths/Elevations

Borehole	Bedrock Surface	
	Depth (mbgs)	Elevation (m)
98-1	18.3*	178.5
98-2	20.9*	178.7
98-3	21.0*	176.5
98-4	26.9*	174.5
99-43	21.3*	179.5
99-44	17.1*	180.0
99-45	20.9*	180.7
1	22.3**	179.9
2	22.3**	179.6
19-03	17.9*	179.9

Notes: *Bedrock proven by coring

**Suspected bedrock surface based on auger refusal

Based on the bedrock surface elevations encountered in the boreholes, it appears that the bedrock surface gently slopes towards the river from both the east and west (i.e., in a trough shape), as well as to the south.

6.8 Groundwater

The groundwater levels measured in the VWP's installed during the current investigation and the wells/piezometers installed during the previous investigations are summarized in Table 4-8 below. A downward hydraulic gradient is evident in the multi-level installations of Boreholes 99-43 and 99-45. Artesian conditions were encountered during drilling in Boreholes 98-1 and 19-03, which are located close to the creek bank, and the VWP installed in Borehole 19-02A indicates a slight artesian condition.

These groundwater 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.

The water level in the creek was reported by others to be at 195.37 m in October 2018.

Table 4-8: Summary of Groundwater Level Measurements

Borehole	Strata and Elevation within Screened Interval	Ground Surface Elevation (m)	Groundwater Level		
			Depth (mbgs)	Elevation (m)	Date of Measurement
98-1	Clay (181.5 – 183.0)	196.8	-2.4*	199.2	October 21, 1998
			-2.4*	199.2	October 22, 1998
			-2.3*	199.1	October 23, 1998
			-2.3*	199.1	October 24, 1998
			-2.3*	199.1	October 25, 1998
			-2.3*	199.1	October 26, 1998
			-2.3*	199.1	October 27, 1998
98-4	Sand till (176.5 – 178.0)	201.4	4.1	197.3	October 24, 1998
			4.1	197.3	October 25, 1998
			4.3	197.1	October 26, 1998
			4.2	197.2	October 27, 1998
99-43 (shallow)	Clay (188.6 – 190.1)	200.8	2.0	198.8	October 3, 1999
99-43 (deep)	Granitic bedrock (175.0 – 176.5)		2.8	198.0	October 3, 1999
99-45 (shallow)	Clay (191.0 – 192.5)	201.6	1.0	200.6	October 3, 1999
99-45 (deep)	Granitic bedrock (177.6 – 179.1)		3.2	198.4	October 3, 1999
1	Clay (190.0 – 191.5)	202.2	0.9	201.3	September 11, 2009
2	Clay (189.7 – 190.2)	201.9	1.5	200.4	September 11, 2009
4	Clay (189.6 – 191.1)	201.8	3.1	198.7	September 11, 2009
5	Clay (185.6 – 187.1)	196.8	0.2	196.6	September 11, 2009
6	Clay (189.6 – 191.1)	201.8	0.9	200.9	September 11, 2009
19-02A	Clay (182.1 – 182.4) Tip Elev. 182.5	198.4	-0.3*	198.7	August 19, 2019
			-0.3*	198.7	August 20, 2019
			-0.5*	198.9	August 26, 2019
			-0.4*	198.8	August 27, 2019
19-03	Clay (192.9 – 194.1) Tip Elev. 193.0	197.8	1.7	196.1	August 27, 2019

*Note – Artesian conditions



6.9 Analytical Testing

Four samples of the native soils were submitted for analysis of pH, water soluble sulphate, sulphide, chloride, conductivity and resistivity. The analysis results are summarized in Table 4-9 below and a copy of the test results is provided in Appendix C.

Table 4-9: Results of Chemical Analysis

Borehole (Sample)	Depth (mbgs)	Sulphate (µg/g)	pH (-)	Resistivity (Ohm-cm)	Conductivity (uS/cm)	Chloride (µg/g)	Sulphide (%)
19-02B (SS2)	0.8 – 1.4	<5	7.68	15300	62	9	0.02
19-02B (SS4)	2.3 – 2.9	14	7.70	1570	637	231	0.02
19-03 (SS5)	2.4 – 3.0	28	7.77	4390	228	9	0.02
19-03 (SS8)	6.1 – 6.7	24	8.09	4390	228	5	<0.02



7. MISCELLANEOUS

Borehole locations were selected in consultation with the Ministry of Transportation and were located in the field by Thurber relative to existing site features. The as-drilled locations and ground surface elevation of the boreholes were surveyed by Thurber following completion of the field program. Survey elevation benchmarks were provided by MPCE. Marathon Drilling Ltd. of Greely, Ontario supplied and operated the drilling equipment to conduct the drilling, soil sampling, in-situ testing, VWP installation and borehole decommissioning. ConeTec Investigations Ltd. supplied and operated the CPT equipment. The field investigation was supervised on a full-time basis by Michel Johnston, E.I.T. and Sean O'Bryan, C.E.T. of Thurber. Overall supervision of the investigation program was provided by Stephen Dunlop, P.Eng.

Routine geotechnical laboratory testing was completed by Thurber's laboratory in Ottawa, Ontario. UCS, oedometer (incremental loading) consolidation testing, and triaxial testing were completed by Stantec's laboratory in Ottawa, Ontario. CRS consolidation testing was carried out at the Western University laboratory in London, Ontario. Analytical testing was completed by Paracel Laboratories in Ottawa, Ontario. Interpretation of the factual data and preparation of this report were carried out by Deanna Pizycki, P.Eng., Stephen Dunlop, P.Eng. and Dr. Fred Griffiths, P.Eng. The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundation Projects.



Deanna Pizycki, M.Eng., P.Eng.
Geotechnical Engineer



Stephen Dunlop, M.A.Sc., P.Eng.
Associate, Senior Geotechnical Engineer



Dr. Fred Griffiths, P.Eng.
Senior Associate
Senior Geotechnical Engineer



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



**FOUNDATION INVESTIGATION AND DESIGN REPORT
LARONDE CREEK BRIDGE REPLACEMENT
HIGHWAY 17, 20.3 KM WEST OF HIGHWAY 11
SITE NO. 43X-0065/B0
NORTH BAY, ONTARIO
G.W.P. No. 5198-13-00**

Geocres Number: 31L-224

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 geotechnical recommendations to assist the project team in designing a suitable foundation for the proposed replacement of the existing Laronde Creek bridge crossing Highway 17. The discussion and recommendations presented in this report are based on the information provided by MPCE and on the factual data obtained during the course of the investigation.

This foundation investigation and design report with the interpretation and recommendations are intended for the use of the Ministry of Transportation, 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.

8.1 Proposed Structure

It is understood that the new bridge will accommodate two through lanes, one right turn lane, and paved shoulders. At the preliminary stage, two options for the bridge replacement were reviewed, including a new bridge to the north of the existing structure with a new horizontal and vertical alignment and a new bridge on the existing horizontal alignment. Based on the review, it is understood that the technically preferred alternative is a new 34 m long single span bridge with a new horizontal and vertical alignment located to the north of the existing bridge. The proposed elevation of the road surface at the new west and east abutments is 203.262 and 203.433 m, respectively, which is about 1.5 m higher than the existing bridge.



8.2 Applicable Codes and Design Considerations

The geotechnical assessment presented within this report has been prepared based on the available data regarding the proposed bridge, the existing subsurface information and in accordance with the Canadian Highway Bridge Design Code (CHBDC), version CSA S6:19.

In accordance with the CHBDC it is anticipated that the replacement structure would be classified as *Major-Route Bridge* with a *Typical Consequence* resulting in a consequence factor (ψ) of 1.0. As per Section 6.5.3.2 of the CHBDC, the degree of site and prediction model understanding is considered to be *High Understanding* for the soils based on the extent and quality of the foundation data available. A typical understanding has been selected for design incorporating bedrock parameters.

9. EXISTING FOUNDATIONS

The available design drawings indicate that the existing abutments and wing walls are supported by piles. The elevation of the top of the piles, the pile layout, and the number of piles are provided on Drawing 620-2-2 (copy provided in Appendix F); however, no further foundation information is provided, such as the design loads, battering of piles, or the depth of the piles. The terms of reference for this assignment indicate that timber piles were utilized. The available information on the pile foundations is provided in Table 9-1 below.

Table 9-1: Summary of Available Existing Timber Pile Foundation Information

Foundation Element	Minimum Length of Pile based on Drawings* (ft / m)	Number of Piles	Assumed Pile Diameter (ft / m)
West Abutment	19.9 / 6.1	17	1.17 / 0.36
West Wing Walls	25.2 / 7.7	12	
East Abutment	19.7 / 6.0	17	
East Wing Walls	24.8 / 7.6	12	

*Note – Based on an assumed pile tip elevation of 187.5 m as discussed below.

Drawing 620-2-1 provides information on subsurface probe “soundings”; however, these probes refused in the clay and the drawing indicates that the piles were driven deeper. It is not clear if the timber piles were driven to the bedrock surface or reached refusal in the overburden. At a minimum, it can be assumed that the piles were driven to an elevation of 187.5 m based on the depth of the “soundings”. Based on this information, it can be conservatively assumed that each pile has a factored axial bearing resistance at SLS of 100 kN and at ULS of 190 kN. This assumes that the timber piles have not experienced any degradation. It is noted that the axial resistances may be higher than those provided in this assessment; however, this cannot be confirmed with the information available. If MPCE provides the loads on the existing abutments, the factored bearing resistances of the existing timber piles can be re-evaluated.



Given the compressible soils underlying the site, downdrag loads must be considered for the existing piles.

It is anticipated that the existing piles will be cut off at the river bed level during demolition of the existing structure and will not be reused to support the new structure. New piles to support the replacement bridge should be located at least 2 m from the existing timber piles to avoid damaging either set of piles during driving. The effect of vibrations due to pile driving activities will also need to be considered on the existing piles (see Section 16.3).

It is noted that Drawing 620-2 indicates a road surface elevation of 659.079 ft (200.89 m) and 658.92 ft (200.84 m) at the west and east abutments respectively, which differs from the surveyed elevations of the on-road boreholes advanced during the most recent investigation by Trow (El. 201.8 m). It is understood that this discrepancy is due to a change in the vertical datum.

10. SEISMIC CONSIDERATIONS

10.1 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). The seismic hazard for this site has been obtained from the GSC calculator. The data includes a peak ground acceleration (PGA), peak ground velocity (PGV) and the 5% spectral response acceleration values ($S_a(T)$) for the reference ground condition (Site Class C) for a range of periods (T) and for a range of return periods including 475-year, 975-year and 2475-year events. The GSC seismic hazard calculated data sheet for this site is included in Appendix G.

The site coefficients used to determine the design spectral acceleration and displacement values are a function of the Site Class and the peak ground acceleration (PGA). At this site, the PGA for a reference Site Class C with a 2% probability of exceedance in 50 years (2475-year event) is 0.131g. This value is to be scaled by the $F(PGA)$ based on the site-specific Site Class.

10.2 Seismic Liquefaction

The Boulanger & Idriss (2014) Simplified Method was used to assess the potential for liquefaction of the cohesionless deposits at this site. Based on the PGA and the subsurface conditions reported below the foundations at the existing and proposed alignments, the non-cohesive soils are not considered susceptible to liquefaction during a seismic event.

The susceptibility of the cohesive soils at the site to experience cyclic mobility or cyclic softening was initially assessed using the Bray et al. (2004) criteria and the results of index property testing. Soils that were considered potentially susceptible were subsequently assessed based on in-situ shear strength measurements using the simplified procedure outlined in Boulanger and Idriss (2007). Based on the results of both analyses, the cohesive material at this site is not considered susceptible to cyclic mobility or cyclic softening during a seismic event.



10.3 CHBDC Seismic Site Classification

In accordance with the CHBDC, the selection of the seismic site classification is based on the soil conditions encountered in the upper 30 m of the stratigraphy. This site has been classified as a Site Class E in accordance with Section 4.4.3.2 of the CHBDC (S6:19) based on shear wave velocity measurements from the SCPT.

An $F(PGA)$ of 1.79 has been interpreted from Table 4.8 of the CHBDC S6:19 resulting in a design PGA of 0.234g for this site.

11. DESIGN OPTIONS

The design of the bridge structure foundations and approach embankments will be governed by the presence of the soft to stiff compressible clay deposit that is present throughout the site. A summary of the engineering properties of the clayey silt to clay soils encountered at this site is provided graphically in Appendix I. Also indicated in the figure are the design parameters used to assess settlement. The geotechnical foundation design considerations include:

- The clay layer, which is soft to firm near the ground surface, does not have the capacity to support shallow foundations. Therefore, deep foundations will be required.
- The clay layer does not have the capacity to support the proposed earth fill embankments from a global slope stability perspective. The design will need to incorporate mitigation measures to achieve acceptable factors of safety against global slope instability.
- The clay layer is highly compressible. The loads applied from the approach embankments will result in new settlement. The design will need to incorporate mitigation measures to ensure that embankment settlement meets the MTO embankment settlement criteria (less than 25 mm of settlement within 20 years of construction within 20 m of the bridge abutment). Settlement of the clay soils will also introduce downdrag loads on piles.
- From a geotechnical perspective, the ground conditions at the site are generally suitable for integral abutments.

Further discussion regarding these design considerations, evaluation of design options and foundation recommendations are provided in the sections that follow.

11.1 Foundation Alternatives

Given the presence of the compressible clay deposit at this site, which is soft to firm near the ground surface, shallow foundations are not considered feasible for the proposed bridge replacement due to the very wide footings that would be required and the potential for unacceptable settlement. Therefore, deep foundations are recommended.

The following deep foundation alternatives were considered for the new bridge, with a comparison of the technical advantages and disadvantages presented in Appendix H:



F1 – Steel H-piles driven to refusal on the bedrock surface

F2 – Steel pipe piles driven to refusal on the bedrock surface

F3 – Caissons (drilled shaft piles), likely socketed into the bedrock

Driven steel piles will readily penetrate the clay soils at this site and there is a low potential for encountering obstructions. H-piles will provide higher resistances than pipe piles. Caissons will provide higher axial and lateral resistance than driven steel piles; however, caissons at this site will require depths up to about 20 m. Additionally, the artesian groundwater pressures at this site will present constructability challenges for drilled shafts.

11.2 Approach Embankment Alternatives

The design of the approach embankments will be governed by the presence of an approximately 15 m thick compressible varved clay deposit that underlies the site. The clay does not have the capacity to support the proposed earth fill embankments from a global slope stability perspective, see Section 14. In addition, the clay is highly compressible and the resulting settlements from an earth fill embankment would significantly exceed the MTO embankment settlement criteria. This settlement would be entirely differential to the abutments, which will be supported by relatively unyielding piles driven to the bedrock surface.

In order to provide an acceptable factor of safety against global instability while also achieving less than 25 mm of post-construction settlement near the abutment, the following design alternatives are presented for a new bridge to the north of the existing structure with a new horizontal and vertical alignment (i.e., the technically preferred alternative):

E1 – Preload/Surcharge with Additional Mitigations

The approach embankments would be constructed in advance (i.e., preloaded) and the majority of settlement would be allowed to take place before final paving. An additional height of soil (i.e., a surcharge) would need to be added above the design grade to accelerate settlement. Additional mitigations would also be required to maintain slope stability and meet the MTO settlement criteria, and are summarized as follows:

- A sheet pile wall would need to be installed to separate the existing embankment from the new embankment; otherwise, the existing embankment would settle an unacceptable amount during construction while it must remain operational. The sheet pile wall would need to be advanced from the top of the existing embankment to the base of the clay (approximately 22 m deep) and approximately 20 m long for each embankment.
- Wick drains at 1.5 m triangular spacing would be required to accelerate settlement such that the target settlement is achieved within a reasonable timeframe (e.g., 6 months); otherwise, several years could be required before the new highway can be completed. The wick drains would be installed to about elev. 181.5 m (approximately 17 m deep);



the wick drains should not be installed any deeper to avoid connecting to the artesian pressure that was encountered at the surface of the bedrock.

- Four layers of high-strength uniaxial geogrid would be required near the base of the embankment to provide additional strength to prevent slope stability failure. There would be two layers in the longitudinal direction and two layers in the transverse direction.
- Rockfill would be needed to construct the embankment as opposed to granular or earth fill. Rockfill has a higher internal friction angle and is needed to achieve acceptable factors of safety against slope instability. In addition, rockfill allows for steeper side slopes (1.25H:1V) and also has a lower unit weight than granular fill and therefore reduces the magnitude of embankment settlement.
- The embankment would need to be constructed with a 1.5 m high surcharge (i.e., 1.5 m higher than the final grade) to accelerate settlement. It is noted that a higher surcharge is not feasible due to unacceptable factors of safety against global slope instability.
- A preload/surcharge period of at least 6 months would be required. It is assumed that a longer preload/surcharge time would not be feasible from a scheduling perspective. The preference would be to utilize the winter months for the preload/surcharge period.
- A detailed monitoring program including settlement plates, inclinometers, and vibrating wire piezometers would be needed during embankment construction and the preload/surcharge period to monitor settlements, slope movement, and porewater pressure in the underlying clay.
- Following the 6-month preload/surcharge period, the surcharge would be removed and approximately 2.5 m of additional rockfill would need to be subexcavated below the final pavement level to allow for the placement of 1.5 m of lightweight expanded polystyrene (EPS), a concrete cover slab, and the pavement structure. This thickness of EPS is required to allow the construction to proceed in a timely manner. If EPS is not used, the surcharge could be allowed to stay for a period of 1 year; however, the estimated post-construction settlement would be in the range of 40 to 50 mm within the 20-year pavement design life, which does not meet the MTO embankment settlement criteria.

Following the placement of the EPS, the highway construction can be completed.

E2 – Construct the Embankments with Lightweight Fill

A portion of the embankment height can be constructed with lightweight fill, such as expanded polystyrene (EPS) blocks. This alternative will eliminate the risks associated with global slope instability and excessive settlement, will not greatly affect the construction schedule, and should result in fewer requirements for periodic maintenance/paving



compared to preloading. For this option the pavement structure fill over the EPS should be 2.0 m thick (the frost penetration depth) to prevent icing on the asphalt surface and the remainder of the embankment height should consist of EPS.

E3 – Ground Improvement

The stiffness of the underlying clay deposit could be increased by means of ground improvement (e.g., rammed aggregate piers or deep soil mixing) to limit settlements and improve slope stability. This option is technically feasible and would not greatly delay the construction schedule; however, these methods will likely not be practical or economically viable due to the thickness of the clay deposit.

E4 – Modify the Bridge Design to Reduce the Embankment Height

It is noted that the existing embankment height, which is about 1.5 m lower than the proposed embankments, allows for an acceptable factor of safety against global instability. It is also noted that the existing approach embankments have remained serviceable from a settlement perspective during the life-span of the bridge, recognizing that maintenance (asphalt padding) has been required on the approaches to correct for long-term post-construction settlement.

If the bridge design can be modified to reduce the embankment height, such as by lowering the vertical grade and/or lengthening the overall span between the abutments, it is anticipated that a more conventional granular fill embankment can be constructed with fewer mitigation measures than the option discussed above. It is noted that a shoring wall, wick drains and a surcharge may still be required to accelerate settlements to meet the MTO's embankment settlement criteria. A monitoring program may also be warranted depending on the magnitude of settlement that is estimated. To eliminate the requirement for these mitigations, the embankment fill height would likely need to be reduced to 2.5 m (or less), which may require a multiple span bridge.

A comparison of these alternatives, based on their respective advantages and disadvantages, is included in Appendix H. Additional discussion on the approach embankments is provided in Section 14.

11.3 Construction Staging Alternatives

The new bridge will have a new alignment to the north of the existing bridge, which will allow two lanes of traffic to remain open on the existing bridge throughout construction. Therefore, the requirement for staging is not anticipated.

11.4 Recommended Design Approach

It is recommended that the deep foundations for the new bridge consist of steel HP310x110 or HP360x132 piles end bearing on bedrock (F1). The use of caissons (F3) would be costlier due to the required founding depths and there would be constructability issues associated with the presence of artesian groundwater pressure; this option is not discussed further herein.



With respect to the approach embankments, the use of EPS lightweight fill (E2) to construct the embankment is recommended to satisfy global slope stability and settlement concerns. The EPS option will also be more cost effective than the other options that have been considered.

12. FOUNDATION RECOMMENDATIONS

12.1 Pile Axial Resistance

Steel HP310x110 or HP360x132 piles driven to refusal on the bedrock surface can be designed using the factored axial structural capacity of 2,000 kN, or 2,400 kN, respectively. The factored geotechnical capacities are estimated to be 3,500 kN and 4,100 kN, respectively for HP 310x110 and HP 310x132 piles which exceed the structural capacities given above. These geotechnical resistances incorporate a resistance factor of 0.4 in accordance with Table 6.2 of the CHBDC. The structural resistance of the pile under static and seismic conditions must be checked by a structural engineer. The factored geotechnical resistance at SLS will not govern steel piles bearing on bedrock. Likewise, H-piles founded on bedrock will not experience differential settlement.

It is anticipated that piles will encounter bedrock between elevation 178.5 m and 180.0 m at both the east and west abutments.

12.2 Downdrag

Downdrag (negative skin friction) will need to be considered for the piles in any instance where clay consolidation settlement is initiated after the piles have been installed. It is noted that downdrag will still apply for an EPS embankment due to the 2.0 m of granular fill that will be placed over the EPS. The settlement that will occur as a result of the placement of the new fill is estimated to be within acceptable limits but will induce downdrag loads. This effect could potentially be neglected provided that some of the existing ground (approximately 2.5 m thick) is subexcavated (equal to the weight of the pavement structure and earth fill on top of the EPS) to ensure zero net load increase on the underlying soil. However, this will require excavating below the creek level and would significantly increase dewatering efforts. The EPS is lighter than water and will heave due to buoyancy unless the groundwater pressure is controlled until the overlying pavement structure fill is placed over the EPS. If the groundwater management system fails even temporarily and heave occurs, complete reconstruction of the EPS would likely be required. Due to this risk, it is recommended that construction be maintained above the groundwater level and downdrag loads be included in the design.

The downdrag load is a function of the length of pile over which negative skin friction is developed and the weight of the overlying embankment soil. The unfactored downdrag load can be taken as 560 kN per pile. This downdrag load should be multiplied by a load factor of 1.25 (Table 3.3, CHBDC) to obtain a factored downdrag load. In accordance with Section 6.11.4.10 of the CHBDC, 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 downdrag calculations can be taken as the bottom of the clay layer which is at elevation 181 m (approximately).



It is noted that the existing pile foundations have already experienced downdrag loads during the settlement induced by the existing approach fills. Additional settlement will not induce loads greater than the piles have previously sustained.

12.3 Pile Lateral Resistance

The interaction between a pile and the surrounding soil may be analysed using horizontal subgrade reaction theory (k_h) and ultimate passive lateral resistance (P_{ult}). For a single driven H-pile, the k_h and p_{ult} values may be calculated using the following equations in conjunction with the parameter values provided in Table 12-1 below.

For cohesionless soils:

$$k_h = \frac{n_h z}{D}$$

$$P_{ult} = 3 \cdot \gamma' \cdot z \cdot K_p$$

For cohesive soils:

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

$$p_{ult} = 9 \cdot S_u$$

where:

k_h	=	coefficient of horizontal subgrade reaction (kN/m ³)
n_h	=	coefficient related to soil density (kPa/m)
p_{ult}	=	ultimate passive lateral resistance (kPa)
z	=	depth of pile embedment (m)
D	=	pile width perpendicular to load direction (m)
γ'	=	effective unit weight of soil (kN/m ³)
K_p	=	passive earth pressure coefficient
S_u	=	undrained shear strength (kPa)

The geotechnical parameters for use in assessment of the piles are presented in Table 12-1.

Table 12-1: Parameters for Lateral Pile Resistance

Location	Elevation (m)	Soil	γ' (kN/m ³)	n_h (kPa/m)	S_u (kPa)	K_p
West and East Abutments	197.0 to 195.6	Sandy fill / silt	20	2,200	-	2.7
	195.6 to 185.5	Firm clay	17	-	35	
	185.5 to 180.0	Stiff clay	17	-	60	



The elevations provided above are based on the borehole data and current grades. Where piles are installed within CSP filled with loose sand, properties of the loose sand will govern the soil-pile interaction and the n_h value of the loose sand should be used ($n_h = 1,300$ kPa/m, $\gamma' = 19.5$ kN/m³, and $K_p = 3.00$).

The spring constant, K , and ultimate lateral resistance, P_{ult} , for analysis may be obtained by the expressions:

$$K = k_h \cdot L \cdot D \quad (\text{kN/m})$$

$$P_{ult} = p_{ult} \cdot L \cdot D \quad (\text{kN})$$

where:

k_h	=	coefficient of horizontal subgrade reaction (kN/m ³)
p_{ult}	=	ultimate passive lateral resistance (kPa)
L	=	length of pile segment (m)
D	=	pile width (m)

Where lateral spacing between an adjacent pile or another structural element is less than four equivalent pile diameters, the lateral resistance will need to be reduced based on the center-to-center spacing. The reduction factors to be used can be obtained from Figures C6.11.3(r), C6.11.3(s) and C6.11.3(t) of the commentary of the 2014 version of the CHBDC.

The factored lateral resistance of piles at SLS determined based on the data and methods provided above should incorporate a resistance factor (ϕ_{gs}) of 0.9 as per Table 6.2 of the CHBDC (static analysis – high degree of understanding). The factored ultimate passive resistance should include a resistance factor of 0.55 per Table 6.2 of the CHBDC.

12.4 Abutment Type

The length of steel piles at this site (likely in the range of 15 to 20 m) will provide the required flexibility to allow for integral abutments provided that the piles can adequately resist lateral loads. To provide the required flexibility in the upper 3 m of the piles, a column of loose sand contained within a corrugated steel pipe (CSP) casing will be required, as specified by the integral abutment design requirements. A 600 mm diameter CSP is frequently used for this application. An NSSP has been provided in Appendix J outlining the gradation requirements for the sand backfill to be used in the CSP and other requirements.

12.5 Frost Protection

The frost penetration depth at this site is 2.0 m as per OPSD 3090.101. Accordingly, a minimum of 2.0 m of earth cover, or equivalent thermal cover, must be provided above the base of all pile caps to serve as frost protection. It is noted that EPS has insulating properties and can have a negative impact on the overlying pavement. From a foundations perspective, 2.0 m of granular fill can be placed over the EPS to construct the pavement structure to reduce the potential for icing



on the asphalt. Reference should also be given to the Pavement Design Report regarding frost tapers at the extents of the EPS.

13. RETAINING WALLS

If required, it is anticipated that any abutment or wing walls would be supported by the piled abutment foundations. Separate shallow foundations for retaining walls are not recommended due to the potential for settlement at this site. Lateral earth pressures required for design are provided in the following sections.

13.1 Backfill and Lateral Earth Pressure

Due to the settlement and stability concerns associated with the approach fill embankments, the backfill behind the abutments will consist primarily of EPS material. A mechanism for drainage behind the abutment should be provided by either:

- a) A column of granular backfill, fully supported on the abutment pile cap, or
- b) A geosynthetic sheet drain which would avoid the need for compaction of a narrow trench of granular fill between the abutment wall and EPS.

The backfill pressures acting on the back of the abutment should consider both:

- The gravity loads of the EPS backfill and overlying pavement structure pressing directly against the wall; and
- The active earth pressure from the soil behind the EPS backfill.

The methodology for assessing the pressures on the back of an abutment wall is described in Section 6 of NCHRP Report 529. The vertical load of EPS blocks will result in negligible active horizontal loading of the abutment wall. The horizontal pressure generated by the vertical stress imposed by the overlying pavement structure can be assumed to be equal to 0.1 times the vertical stress.

The earth pressure from the soil behind the EPS backfill may be calculated using the parameters provided in the following table for static conditions.

Table 13-1: Static Earth Pressure Coefficients

Condition	Earth Pressure Coefficient (K)		
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$	OPSS Granular B Type I and III $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$	OPSS SSM and Existing Fill/Silt $\phi = 30^\circ, \gamma = 21.0 \text{ kN/m}^3$
	Horizontal Surface Behind Wall	Horizontal Surface Behind Wall	Horizontal Surface Behind Wall
Active, K_A (Yielding Wall)	0.27	0.31	0.33
At Rest, K_O (Non-Yielding Wall)	0.43	0.47	0.50
Passive, K_P (Movement towards Soil Mass)	3.7	3.3	3.0

In accordance with Clause 6.14.7 of the CHBDC, retaining structures should be designed using earth pressure coefficients that include earthquake loading. The seismic component of the active earth pressure generated by the soil behind the EPS/soil interface can be calculated using the Mononobe-Okabe method with:

$kh = \frac{1}{2} * F(PGA) * PGA_{ref}$, for structures that are capable of 25 mm to 50 mm of movement (yielding walls), and

$kh = F(PGA) * PGA_{ref}$, for non-yielding walls

The coefficients of horizontal earth pressure for combined static and seismic loading presented in the Table 13-2 may be used for design. The earth pressure coefficients are provided for a **Seismic Site Class E**, a PGA_{ref} with a 2% probability of exceedance in 50 years (2475-year event) of 0.131 g (Geological Survey of Canada - Fifth Generation) and a $F(PGA)$ of 1.79 as per Table 4.8 of the CHBDC. The PGA scaled to a Seismic Site Class E at this site is 0.234 g.

Table 13-2: Combined Static and Seismic Earth Pressure Coefficients, Site Class E

Condition	Earth Pressure Coefficient (K)		
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$	OPSS Granular B Type I and III $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$	OPSS SSM and Existing Fill/Silt $\phi = 30^\circ, \gamma = 21.0 \text{ kN/m}^3$
	Horizontal Surface Behind Wall	Horizontal Surface Behind Wall	Horizontal Surface Behind Wall
Active, K_{AE} Yielding Wall	0.34	0.38	0.41
Active, K_{AE} Non-Yielding Wall	0.42	0.47	0.50



The horizontal coefficient of subgrade reaction of the EPS fill should be calculated based on the following equation:

$$K'_{\text{EPS}} = 0.14 * E_{\text{EPS}} / [H * (1 - \nu_{\text{EPS}}^2)] \text{ (units: kN/m}^3\text{)}$$

where:

K'_{EPS}	=	horizontal coefficient of subgrade reaction for EPS
E_{EPS}	=	Young's Modulus of EPS Blocks (refer to Table 8 on Page 42 of NCHRP's Report 529 for different types of EPS blocks)
H	=	Thickness (vertical) of EPS behind wall

The horizontal pressure applied by the wall to the EPS fill must be smaller than the Elastic Limit Stress of the EPS.

The K and K_{AE} values presented above have been calculated for a wall with a vertical back and horizontal backfill. If differing conditions are applicable, the values provided above will need to be re-evaluated.

14. APPROACH EMBANKMENTS

14.1 Global Slope Stability – Option E1

Stability analyses were carried out for the approach embankments under both static and seismic loading conditions. For seismic loading, the horizontal pseudo-static acceleration was taken as 50% of the peak ground acceleration (0.066g). The stability analyses were carried out utilizing the commercially available slope stability program Slope/W (Version 9) of the GeoStudio software package developed by Geo-Slope International with the option for Morgenstern-Price method of slices for limit equilibrium analyses. The target factors of safety from Table 6.2 of the CHBDC are as follows:

- Static-drained loading conditions (permanent condition) = 1.43
- Static-undrained loading conditions (temporary condition) = 1.25
- Seismic-undrained loading conditions = 1.1

The material properties used in the analyses and results of selected analysis are provided on the figures in Appendix I. A summary of selected analysis results is provided in the following table.

Table 14-1: Summary of Global Slope Stability Analysis Results

Location	Global Stability Factor of Safety		
	Static Conditions		Seismic Conditions
	Undrained	Drained	
West Approach (longitudinal), no mitigations	1.18	1.52	not tested
East Approach (longitudinal), no mitigations	1.15	1.27	not tested
West Approach (transverse), no mitigations	1.16	1.44	not tested
West Approach (longitudinal), rockfill and geogrid	1.47	1.73	1.20
- with 1.5 m temporary surcharge (staged)	1.30	-	-
East Approach (longitudinal), rockfill and geogrid	1.61	1.60	1.40
- with 1.5 m temporary surcharge (staged)	1.39	-	-
West Approach (transverse), rockfill and geogrid	1.40	1.43	1.11
- with 1.5 m temporary surcharge (staged)	1.30	-	-

Overall, the clay does not have the capacity to support the proposed earth fill embankments from a global slope stability perspective without mitigations. In this case, four layers of high-strength uniaxial geogrid would be required near the base of the embankment to provide additional strength to prevent slope stability failure. There would be two layers in the longitudinal direction and two layers in the transverse direction. It is noted that the geogrid in the transverse direction needs to extend to the temporary protection system (separating the new embankment from the existing embankment) in order to satisfy the global slope stability criteria. Rockfill would also be needed to construct the embankment as opposed to granular or earth fill.

The above analysis was carried out to assess the option of constructing the embankments with conventional earth or rock fill (i.e., Option E1). It is noted that if the embankments are constructed with EPS, there will be no issues with global slope stability.

14.2 Assessment of Settlement – Option E1

The proposed grade of the new bridge is about 1.5 m higher than that of the existing bridge and requires approach embankments up to about 7 m high. For this height of grade raise, constructed conventionally, settlement of the embankments will occur as a result of compression of the embankment fill itself, but more significantly due to consolidation of the underlying compressible clay deposit. This settlement would be entirely differential to the abutments, which will be supported by relatively unyielding piles driven to the bedrock surface.

An analysis was carried out to estimate the post-construction settlement of the foundation soils (both primary and secondary consolidation settlement) under the weight of the imposed embankment fill materials. 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 14-2 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	
Non-Freeway	25 mm	50 mm	100 mm	200 mm	20

An assessment of the estimated short-term and time-dependent settlement from construction of the proposed approach fills was carried out using the Rocscience Settle^{3D} modelling 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 on the Summary of Engineering Properties provided in Appendix I.

Settlement analyses with various combinations of surcharge height and EPS thickness were conducted and the results are summarized in the table below. All analysis cases assume wick drain installation at a 1.5 m triangular spacing prior to embankment construction and a waiting period of 6 months for the surcharge loading following the completion of fill placement. The settlement analyses also assume that the approach fills would be constructed using rock fill with 1.25H:1V side slopes and that the surcharge thickness is limited to 1.5 m due to slope stability concerns.

Table 14-3: Summary of Settlement Analysis Results

Case No.	Surcharge Thickness (m)	EPS Thickness (m)	Construction Settlement	Post-construction Settlement (20-Year)
1	-	-	350 mm	120 – 130 mm
2a	1 m	-	400 mm	90 -100 mm
2b	1 m	1 m	400 mm	50 – 60 mm
3a	1.5 m	-	460 mm	80 – 90 mm
3b	1.5 m	1.5 m	460 mm	20 – 25 mm

Additional discussion on the mitigations required to satisfy the settlement criteria (sheet pile wall, wick drains, surcharge, monitoring program, and partial construction with EPS) are discussed in Section 11.2. The above analysis was carried out to assess the option of constructing the embankments with conventional rock fill (i.e., Option E1). It is noted that if the embankments are constructed with EPS, the MTO embankment settlement criteria will be satisfied assuming that the fill over the EPS does not exceed 2.0 m in thickness.



14.3 EPS Construction Recommendations (Option E2)

It is recommended that the embankments be constructed using EPS lightweight fill (Option E2). This option addresses both settlement and stability concerns, avoids impacts to the existing structures due to settlement of the clay, and does not result in significant time delays to the project. It is noted that since the grade raise is generally limited to filling in the wedge between the bridge abutments, the valley slopes and the existing approach embankments, the volume of lightweight fill is relatively small. The EPS lightweight fill option is the preferred option from both a technical and risk management perspective. Considering the reduction in the loading that must be achieved (versus conventional earth fill construction), it is considered that expanded polystyrene (EPS) Geofoam will be the most practical lightweight fill type. Other lightweight fills could be considered (such as Cematrix or slag); however, the following recommendations are provided based on the use of EPS Geofoam.

The EPS fill treatment should be implemented in all areas where the proposed finished grade will be more than 2.0 m above current site grades. The proposed embankments will be keyed into the existing approach fill embankments. The limits of the EPS fill should be as follows:

- The bottom of the EPS should be stepped into the existing sloped ground surface by approximately 0.5 m.
- The top surface of the EPS within the embankment side slopes should be covered with a 10mil sheet of polyethylene and stepped such that the minimum soil cover includes 300 mm of modified Select Subgrade Material under 1.2 m of earth fill.
- The lateral extents of the EPS on the side slopes and forward slope should be a minimum of 4 m away from the edge of the creek (at the high-water level) to limit the potential for undermining due to erosion. The lower 2 m (vertical) of soil at the toes of the embankment can consist of granular/earth fill. The embankment slopes should be separated from the creek with rock protection.
- The top surface of the EPS beneath the highway platform should be covered with a concrete slab. A concrete slab thickness of 125 mm is typical. The top of the concrete slab should be located 2.0 m below the pavement surface.

A granular levelling pad consisting of a 300 mm of compacted OPSS Granular A should be provided beneath the EPS. Due to the firm to soft clay conditions, it is recommended that a non-woven geotextile be placed horizontally beneath the granular levelling pad as a separation layer.

Guidelines for the design of EPS embankments can be found in NCHRP Report 529.

To provide sufficient compressive strength (less than 1% deformation) to support the overlying granular fill and traffic loading, the upper 1.2 m of EPS should consist of Type 29. Below this level the EPS can consist of Type 29 or Type 22. The contract must include an NSSP for the EPS



embankment materials and construction. A draft version of suggested NSSP wording is provided in Appendix J.

The embankment design will need to take into consideration the potential for conflict between the EPS fill and foundations for signs, guiderails, utilities or other structures.

Based on the current General Arrangement drawings, it is anticipated that the base of the EPS will be no lower than elevation 196.5 m. This level is above the normal water level in Laronde Creek but approximately 0.6 m below the design high water level. Due to the weight of the concrete slab and granular fill that will be placed above the EPS, flotation is not a concern under long term conditions. During construction, the contractor must dewater the excavation to maintain the water level below the base of the EPS until the overlying pavement structure has been constructed to prevent a buoyant condition and use temporary ballast if/as required to prevent flotation of the EPS blocks.

15. 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. 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.

16. CONSTRUCTION CONSIDERATIONS

16.1 Excavations

All excavations must be conducted in accordance with the requirements of the Occupational Health & Safety Act & Regulations (OHSA) for Construction Projects. The soils at this site would be classified as Type 3 (1H:1V side slopes, or shallower) above the water level and Type 4 (3H:1V side slopes, or shallower) below the water level, in accordance with OHSA.

If there is not enough space to carry out the excavations using unsupported side slopes, the excavations will need to be carried out using a temporary protection system, as discussed in Section 0 below.



16.2 Temporary Protection Systems

It is anticipated that a temporary protection system may be required to separate the excavations for the pile caps from the adjacent creek and from the existing abutment(s). Steel sheet piles are considered a feasible and economical option for temporary protection systems at this site.

The protection system should be designed by a licenced Professional Engineer experienced in design of shoring with consideration of adjacent construction/traffic loads and any sloping retained surfaces. It should be designed in accordance with OPSS 539 as amended by SP105S09.

It is the Contractor's responsibility to design and build a suitable temporary protection system based on their evaluation of the data presented in Part 1 of this report. A braced excavation may be required to provide the necessary lateral support.

The close proximity between the existing and proposed structures presents additional considerations for both design and construction, including but not limited to the following:

- Roadway protection systems will be required to protect the existing abutments and approaches during excavation for construction of the new abutments.
- The design engineer will need to check to ensure that there is no conflict between the existing piles (taking into account the projection of battered piles) and the temporary protection system and its lateral support elements.
- Installation of sheet piles for the temporary protection system will result in vibrations that could impact the existing structure. Vibratory effort to install sheet piles should not be permitted.
- Due to the length, removal of the existing piles would likely be challenging and costly. It is recommended that the piles for the temporary protection system be left in place and cut-off in accordance with OPSS 539 within 5 m (laterally) of the existing abutment. Under no circumstances should the sheet piles be removed after placement of the EPS; otherwise the EPS could be damaged and the EPS could settle due to liquefaction of the subgrade soils.

16.3 Pile Installation and Vibrations

Driven piles must be installed in accordance with OPSS.PROV 903.

Piles are to be driven to refusal on bedrock. Pile tips should be protected from damage during driving. The tips of all piles must be protected with a driving shoe from an approved manufacturer such as Titus Steel (standard H-Point) or approved equivalent. High-Strain Dynamic load testing should be carried out to confirm pile capacity during construction. A sample special provision for the high-strain dynamic testing is provided in Appendix J. The high-strain dynamic testing should be carried out on a minimum of 10% of piles, but no fewer than two piles, for each abutment.



It is recommended that a settlement monitoring program be implemented for the existing structure during installation of sheet piles and H-piles.

Vibrations generated by piling operations have the potential to induce settlements of the existing timber piles which support the existing operational bridge. As a guideline, vibrations should be considered if piling operations will be required within a lateral distance of 15 m of the existing timber piles. If piling within this distance is required, a vibration and settlement monitoring program should be specified in the construction contract, with alert levels tied to settlement. The monitoring program should consist of survey targets at each foundation element with baseline readings taken prior to initiation of construction. Readings should be taken daily during the piling activities. An example NSSP for the monitoring program detailing the recommended monitoring requirements is provided in Appendix J.

Structural input is required; however, it does not appear feasible to shim the existing structure to mitigate against construction induced settlement. Vibrations could potentially be mitigated by means of using low-energy pile driving. Pre-drilling techniques should be avoided due to potential issues with artesian pressures at this site.

16.4 Dewatering and Artesian Pressure

All excavations for foundations must be dewatered (if required) prior to the placement of concrete, as per OPSS 902 and Special Provision (SP) No. FOUN0003, which is included in Appendix J for reference. The contractor should be prepared to control groundwater, if encountered, and surface water flow that may flow into the excavation.

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 system in accordance with FOUN0003 which amends OPSS 902. The design storm return period for designer Fill-In * in this SP should be equal to the storm period used to define the creek high-water level that is shown on the design drawings. Due to the potential for artesian groundwater flow, a preconstruction survey is recommended and Designer Fill-In ** in this SP should be "500 m". The groundwater level will fluctuate and the minimum groundwater elevation at the time of the proposed work should be taken as the creek water level of the design storm return period defined by the contract documents for the temporary dewatering system.

Dewatering and surface water diversion must remain operational and effective until the temporary excavation is backfilled.

Groundwater and/or surface water inflow to excavations is expected to be handled by sumps and pumps. Surface runoff should be diverted away from the excavations at all times and subgrade surfaces should be protected from precipitation.

Depending on the volume of water pumped during construction, either a Permit to Take Water (PTTW) or registration on the Environmental Activity and Sector Registry (EASR) with the Ministry of Environment, Conservation and Parks (MECP) could be required. In accordance with Table 1.0 of CDED-B517, a PTTW is required if groundwater pumping exceeds 400 m³/day and an EASR registration is required if groundwater pumping is between 50 and 400 m³/day.



Storm/surface water diversion is exempt from these permitting requirements. Estimating the volume of water that will be pumped during construction and the preparation of EASR/PTTW documents is not within the scope of this foundation investigation and should be carried out by others.

It is noted that artesian groundwater pressures have been reported for this site; however, the artesian pressure originates from deep in the soil deposit and is not anticipated to affect the excavations that are required at this site. The clay is also sufficiently cohesive to prevent upward seepage of water along the shafts of the newly installed driven piles. Upward seepage would not reduce the axial resistance of the piles resting on bedrock. Artesian pressure would be an issue for drilled shafts; therefore, drilled shafts should not be permitted on this project.

16.5 Erosion Protection

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. Slope protection, scour protection and drainage measures will be required to ensure the long-term surficial stability of the embankment slopes. The erosion and scour protection should be designed in accordance with Section 1.9.4 of the CHBDC and applied at the discretion of the Hydraulic Engineer.

Field observations and particle size analyses in conjunction with the Wischmeier Nomograph indicate that the surficial soils at this site have a high erosion potential. Rock protection should be provided over all embankment surfaces subjected to flowing water in accordance with OPSS 511. A vegetation cover should be established on all other exposed embankment surfaces to protect against surficial erosion in general accordance with OPSS.PROV 804.

16.6 Construction Concerns

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

- Cobbles and boulders may be encountered within the glacial till deposit and obstruct pile driving. Recommended wording for an NSSP alerting the Contractor to this condition is provided in Appendix J.
- Artesian groundwater pressure is present on this site. The contractor must be advised of this condition and prohibited from allowing the artesian pressure within the underlying till/bedrock to be connected to the ground surface by means of a vertical conduit (including shafts, boreholes, wick drains, etc.). Recommended wording for an NSSP addressing this issue is provided in Appendix J.
- The Contractor's selection of construction equipment and methodology should include assessment of the capability of the subgrade soils to support the proposed construction equipment and any temporary structures or fill (i.e. as a pad for crane support). Site conditions may limit the type of equipment suitable for use. The design and safety of any temporary works is the responsibility of the Contractor. Recommended wording for an NSSP addressing this issue is provided in Appendix J.



- The demolition of the existing bridge will induce vibrations that could impact the newly constructed bridge. The Structure Monitoring NSSP that is provided in Appendix J should also apply to the new bridge during demolition of the existing bridge.

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

17. CLOSURE

Deanna Pizycki, P.Eng., Stephen Dunlop, P.Eng. and Dr. Fred Griffiths, P.Eng. prepared this Foundation Investigation and Design Report. Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations projects, reviewed the report.

Thurber Engineering Ltd.



Deanna Pizycki, M.Eng., P.Eng.
Geotechnical Engineer



Stephen Dunlop, M.A.Sc., P.Eng.
Associate, Senior Geotechnical Engineer



Dr. Fred Griffiths, P.Eng.
Senior Associate
Senior Geotechnical Engineer

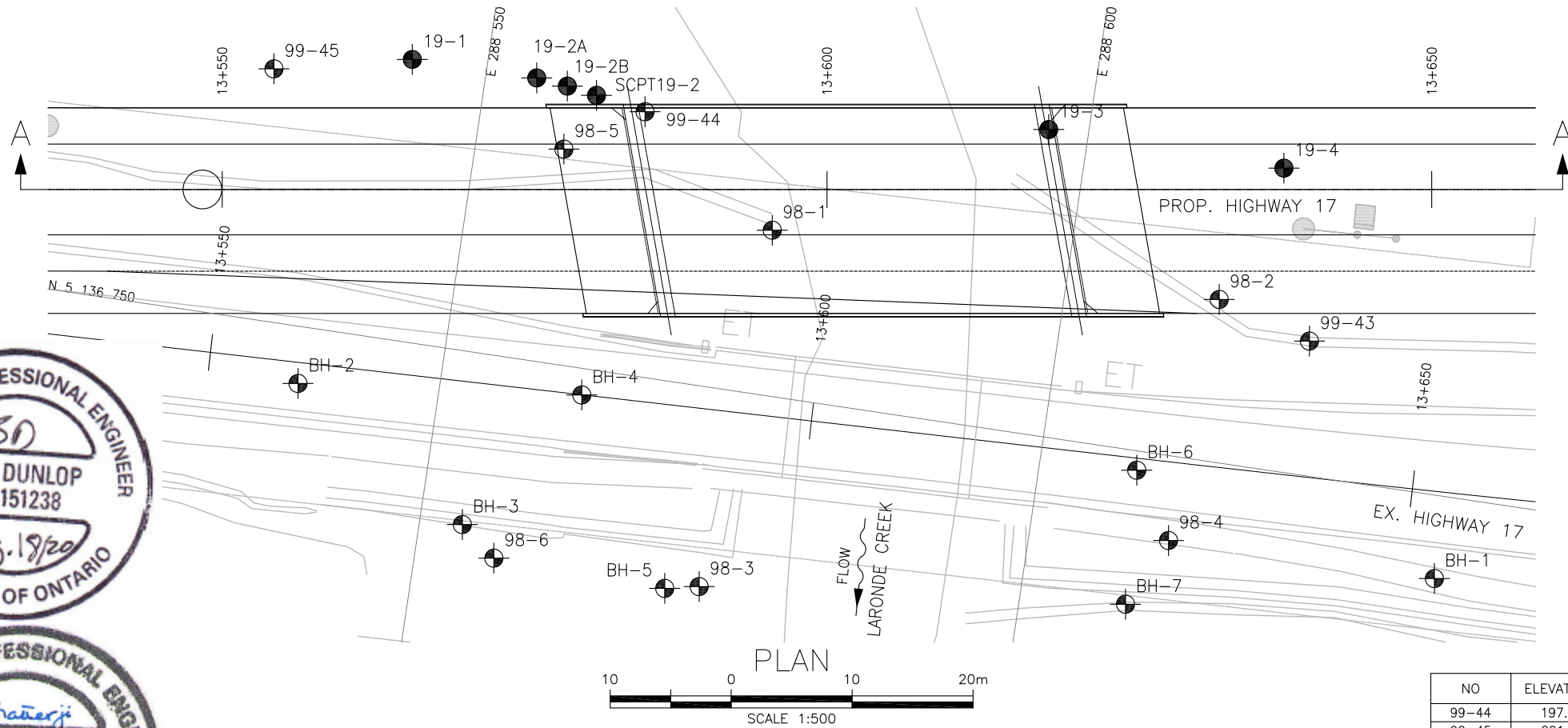


Dr. P.K. Chatterji, P.Eng.
MTO Review Principal
Senior Geotechnical Engineer



Appendix A.

Borehole Location Plan and Stratigraphic Drawings



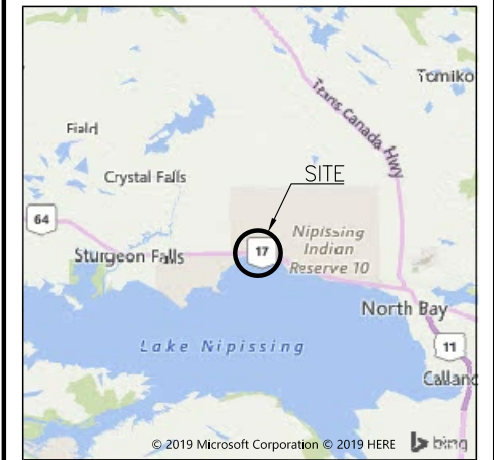
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DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT No
GWP No 5198-13-00



HIGHWAY 17
LARONDE CREEK
BRIDGE REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET



KEYPLAN

LEGEND

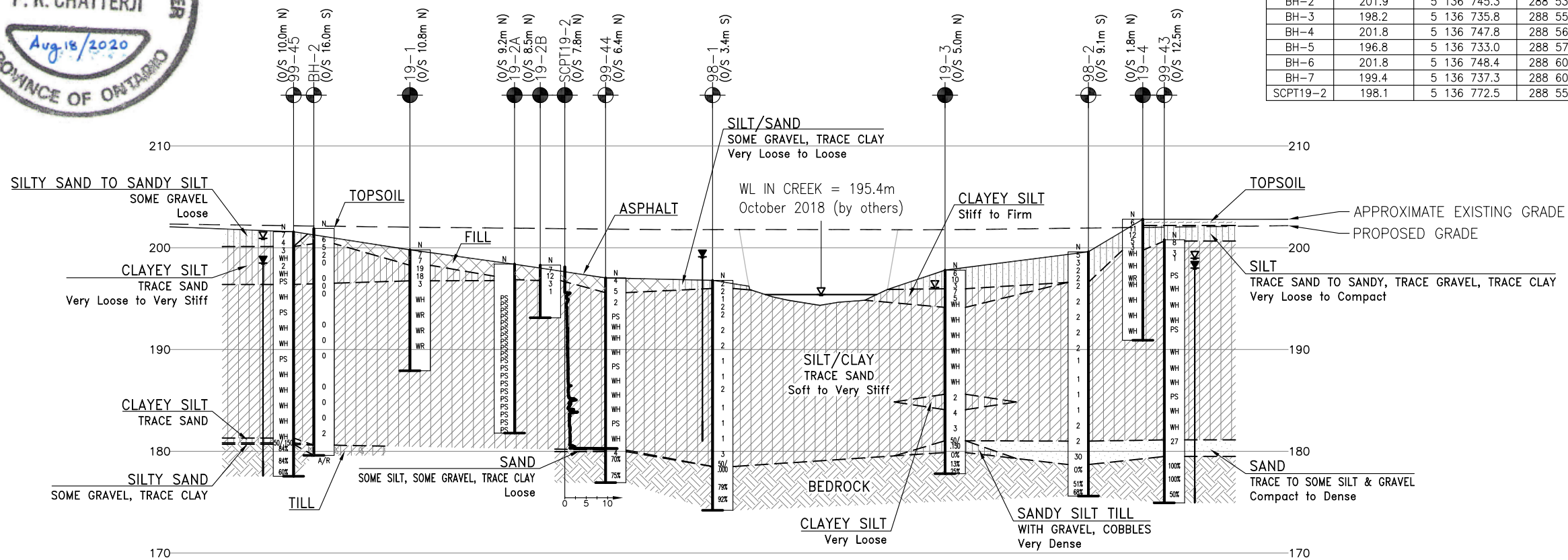
	Borehole (Current Investigation)
	Borehole (Previous Investigations)
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
	Water Level
	Head Artesian Water
	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
19-1	199.8	5 136 773.2	288 543.8
19-2A	198.4	5 136 773.2	288 554.2
19-2B	198.3	5 136 772.9	288 556.8
19-3	197.8	5 136 775.2	288 596.7
19-4	202.8	5 136 774.9	288 616.4
98-1	196.8	5 136 763.6	288 575.3
98-2	199.6	5 136 763.4	288 612.7
98-3	197.5	5 136 733.6	288 573.7
98-4	201.4	5 136 743.1	288 611.5
98-5	197.3	5 136 767.7	288 557.3
98-6	198.4	5 136 733.4	288 556.5
99-43	200.8	5 136 761.1	288 620.6

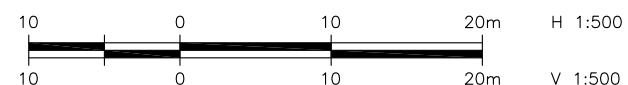
-NOTES-

- The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- This drawing is for subsurface information only. Surface details and features are for conceptual illustration.
- Coordinate system is MTM NAD 83 Zone 10.

GEOCRES No. 31L-224



PROFILE A-A ALONG Q PROPOSED HIGHWAY 17



REVISIONS	DATE	BY	DESCRIPTION
DESIGN	DP	CHK SD	CODE
DRAWN	MFA	CHK DP	SITE
LOAD	DATE	JUN 2020	
STRUCT	DWG	1	



Appendix B.

Record of Borehole Sheets SCPT Log



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 of silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
		OH	Organic clays of high plasticity, organic silts.
HIGHLY ORGANIC SOILS		Pt	Peat and other organic soils.

Note - W_L = Liquid Limit



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 19-01

1 OF 2

METRIC

GWP# 5198-13-00 LOCATION Lat: 46.370468°, Long: -79.711292°
 HWY 17 BOREHOLE TYPE CME 55 Trackmount, NW Casing ORIGINATED BY SOB
 DATUM Geodetic DATE 2019.08.09 - 2019.08.09 COMPILED BY MW
 CHECKED BY FG

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							WATER CONTENT (%)		
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE								
199.8	Pavement Surface							20	40	60	80	100					
0.0	ASPHALT (50mm)							20	40	60	80	100					
0.2	CONCRETE (100mm)																
	Silty SAND Loose Brown Moist (FILL)		1	SS	7		199										1 55 44 (SI+CL)
			2	SS	7												
198.3																	
1.5	Clayey SILT (CL-ML) Very stiff Grey-brown		3	SS	19		198										
			4	SS	18		197										0 0 71 29
196.8																	
3.0	Clayey SILT (CL) to CLAY (CI to CH) Varved Firm Grey		5	SS	3		196										
			6	SS	WH		195										
							194										
			7	SS	WR		193										
							192										0 0 38 62
			8	SS	WR		191										
			9	SS	WR		190										

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10


(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 19-01

2 OF 2

METRIC

GWP# 5198-13-00 LOCATION Lat: 46.370468°, Long: -79.711292°
Laronde Creek Bridge MTM Zone 10 N 5 136 773.2 E 288 543.8 ORIGINATED BY SOB
HWY 17 BOREHOLE TYPE CME 55 Trackmount, NW Casing COMPILED BY MW
DATUM Geodetic DATE 2019.08.09 - 2019.08.09 CHECKED BY FG

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								
	Continued From Previous Page							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE				WATER CONTENT (%) 20 40 60				
187.9	Clayey SILT (CL) to CLAY (CI to CH) Varved Firm Grey							6.0 +								
			10	TW	-		189		14.0 +							
									16.0 +							
11.9	End of Borehole						188	7.3 +								

+³, ×³: Numbers refer to Sensitivity 20 15 10 5 0 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 19-02A

1 OF 2

METRIC

GWP# 5198-13-00 LOCATION Lat: 46.370468°, Long: -79.711158°
Laronde Creek Bridge MTM Zone 10 N 5 136 773.2 E 288 554.2 ORIGINATED BY SOB/MJJ
HWY 17 BOREHOLE TYPE CME 55 Trackmount, NW Casing COMPILED BY MJJ
DATUM Geodetic DATE 2019.08.13 - 2019.08.13 CHECKED BY FG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	W _P W W _L	WATER CONTENT (%)	20 40 60		
198.4	Pavement Surface												
0.0	See 19-2B for stratigraphy												
195.4													
3.0	CLAY (CI to CH) Varved Firm Grey - TW1 : varve thickness ranged from 4 to 25 mm between 3.0 and 3.6 m depth; grey to light grey; varves are similar in composition; occ. SILT (ML) seams. - TW5 : varve thickness ranged from 5 to 20 mm between 5.5 and 6.1 m depth; grey to light grey; varves are similar in composition; occ. SILT (ML) seams. - TW8 : varve thickness ranged from 4 to 15 mm between 7.3 and 7.9 m depth; grey to light grey; varves are similar in composition; occ. SILT (ML) seams.		1	TW	-		195						0 0 48 52
			2	TW	-								
			3	TW	-		194						
			4	TW	-		193						0 0 33 67 0 0 31 69 0 0 42 58
			5	TW	-		192						
			6	TW	-		191						0 0 44 56
			7	TW	-		190						
			8	TW	-		189						
			9	TW	-								
			10	TW	-								
			11	TW	-								

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 19-02A

2 OF 2

METRIC

GWP# 5198-13-00 LOCATION Lat: 46.370468°, Long: -79.711158°
Laronde Creek Bridge MTM Zone 10 N 5 136 773.2 E 288 554.2 ORIGINATED BY SOB/MJJ
HWY 17 BOREHOLE TYPE CME 55 Trackmount, NW Casing COMPILED BY MJJ
DATUM Geodetic DATE 2019.08.13 - 2019.08.13 CHECKED BY FG

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	W _P W W _L						
								SHEAR STRENGTH kPa		WATER CONTENT (%)					
								○ UNCONFINED + FIELD VANE	● QUICK TRIAXIAL × LAB VANE						
	Continued From Previous Page		12	TW	-		188								
			13	TW	-										
			14	TW	-		187								
	- TW15: varve thickness ranged from 3 to 45 mm between 12.2 and 12.8 m depth; grey to light grey to red; varves alternating between clay and silt with occasional sand seams.		15	TW	-		186							0 0 46 54	
			16	TW	-									0 4 82 14 non-plastic	
			17	TW	-		185								
			18	TW	-		184								
			19	TW	-		183								
			20	TW	-		182								
181.8 16.6	End of Borehole Vibrating wire piezometer installed. Tip at elevation 182.5 m. Interpreted water level at 0.4 m above the ground surface (Elev. 198.8m) on August 27, 2019.														

DOUBLE LINE 23411 LARONDE.GPJ 2012TEMPLATE(MTO).GDT 15/6/20

RECORD OF BOREHOLE No 19-02B

1 OF 1

METRIC

GWP# 5198-13-00 LOCATION Lat: 46.370465°, Long: -79.711124°
Laronde Creek Bridge MTM Zone 10 N 5 136 772.9 E 288 556.8 ORIGINATED BY SOB/MJJ
HWY 17 BOREHOLE TYPE CME 55 Trackmount, NW Casing COMPILED BY MJJ
DATUM Geodetic DATE 2019.08.14 - 2019.08.14 CHECKED BY FG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				WATER CONTENT (%)				
198.3	Pavement Surface															
0.0	ASPHALT (150 mm)															
0.2	SAND Loose to Compact Brown Moist (FILL)		1	SS	7		198									
			2	SS	12											
196.8							197									
1.5	CLAY (CI), some fibrous organics Stiff Grey-Brown		3	SS	3										0 1 64 35	
196.0							196									
2.3	Clayey SILT (CL) to CLAY (CI to CH) Varved Firm Grey		4	SS	1											
							195									
							194									
193.1																
5.2	End of Borehole															

+³, ×³: Numbers refer to Sensitivity 20 15 10 5 0 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 19-03

1 OF 3

METRIC

GWP# 5198-13-00 LOCATION Lat: 46.370487°, Long: -79.710605°
Laronde Creek Bridge MTM Zone 10 N 5 136 775.2 E 288 596.7 ORIGINATED BY MJJ
HWY 17 BOREHOLE TYPE Explo Portable, NW Casing / NQ Coring COMPILED BY MW
DATUM Geodetic DATE 2019.08.16 - 2019.08.17 CHECKED BY FG

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	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
197.8	Ground Surface							20	40	60	80	100		
0.0	SILT, occ. organics near surface Loose to very loose Grey to brown Dry to moist -becoming clayey at 1.2 m		1	SS	6									
			2	SS	10									
			3	SS	3									
196.0														
1.8	Clayey SILT (CL-ML to CL) Stiff to firm Grey		4	SS	7									
			5	SS	5									
			6	SS	WH									
194.1														
3.7	Clayey SILT (CL) to CLAY (CI to CH) Varved Soft to firm Grey							5.0 +						
								5.8 +						
			7	SS	WH									
								7.0 +						
								7.5 +						
			8	SS	WH									
								6.5 +						
								13.6 +						
190			9	SS	WH									
							5.8 +							
189							6.0 +							
			10	SS	WH									
188														

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity 20
15 10 5 0
(%) STRAIN AT FAILURE

METRIC

[illegible]

+³, ×³: Numbers refer to Sensitivity

DOUBLE LINE 23411 LARONDE.GPJ 2012TEMPLATE(MTO).GDT 15/6/20

RECORD OF BOREHOLE No 19-03

3 OF 3

METRIC

GWP# 5198-13-00 LOCATION Lat: 46.370487°, Long: -79.710605°
Laronde Creek Bridge MTM Zone 10 N 5 136 775.2 E 288 596.7 ORIGINATED BY MJJ
HWY 17 BOREHOLE TYPE Explo Portable, NW Casing / NQ Coring COMPILED BY MW
DATUM Geodetic DATE 2019.08.16 - 2019.08.17 CHECKED BY FG

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	W P	W	W L	WATER CONTENT (%)		
20.0	Continued From Previous Page End of Borehole Vibrating wire piezometer installed. Tip at elevation 193.0 m. Water level at a depth of 1.7 m below the ground surface (Elev. 196.1 m) on August 27, 2019.													

DOUBLE LINE 23411 LARONDE.GPJ 2012TEMPLATE(MTO).GDT 15/6/20

RECORD OF BOREHOLE No 19-04

1 OF 2

METRIC

GWP# 5198-13-00 LOCATION Lat: 46.370484°, Long: -79.710349°
 HWY 17 BOREHOLE TYPE Explo Portable, NW Casing ORIGINATED BY MJJ
 DATUM Geodetic DATE 2019.08.15 - 2019.08.15 COMPILED BY MW
 CHECKED BY FG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																							
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W P W W L				GR SA SI CL																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																							
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Continued Next Page

+ 3, × 3: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 19-04

2 OF 2

METRIC

GWP# 5198-13-00 LOCATION Lat: 46.370484°, Long: -79.710349°
 HWY 17 BOREHOLE TYPE Explo Portable, NW Casing ORIGINATED BY MJJ
 DATUM Geodetic DATE 2019.08.15 - 2019.08.15 COMPILED BY MW
 CHECKED BY FG

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							
	Continued From Previous Page							20 40 60 80 100							
	Clayey SILT (CL) to CLAY (CI to CH) Varved Soft to Firm Grey							20 40 60 80 100							
190.9			12	SS	WH		192	20 40 60 80 100							
11.9	End of Borehole						191	20 40 60 80 100							

+³, ×³: Numbers refer to Sensitivity
 20
 15 10 5 0
 (%) STRAIN AT FAILURE



THURBER ENGINEERING LTD.

Thurber Engineering Ltd.

104, 2460 Lancaster Road

Ottawa, ON

www.thurber.ca

Project: Laronde Creek Bridge, MTM Zone 10

Location: Highway 17

CPT: SCPT19-02

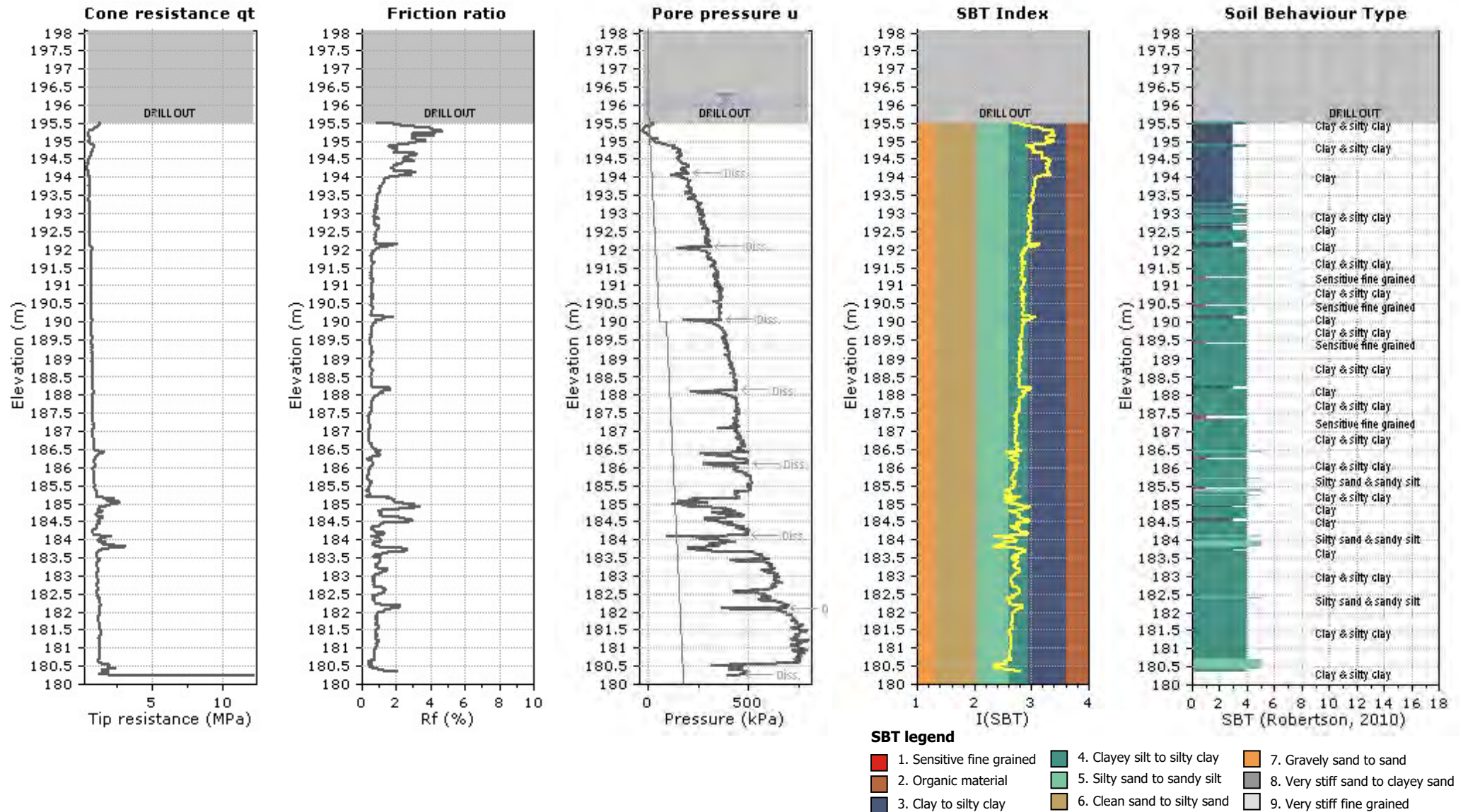
Total depth: 17.88 m, Date: 2019-08-08

Surface Elevation: 198.12 m

Coords: X:288559.30, Y:5136772.50

Cone Type: 15cm2

Cone Operator: ConeTec





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Location: Highway 17

CPT: SCPT19-02

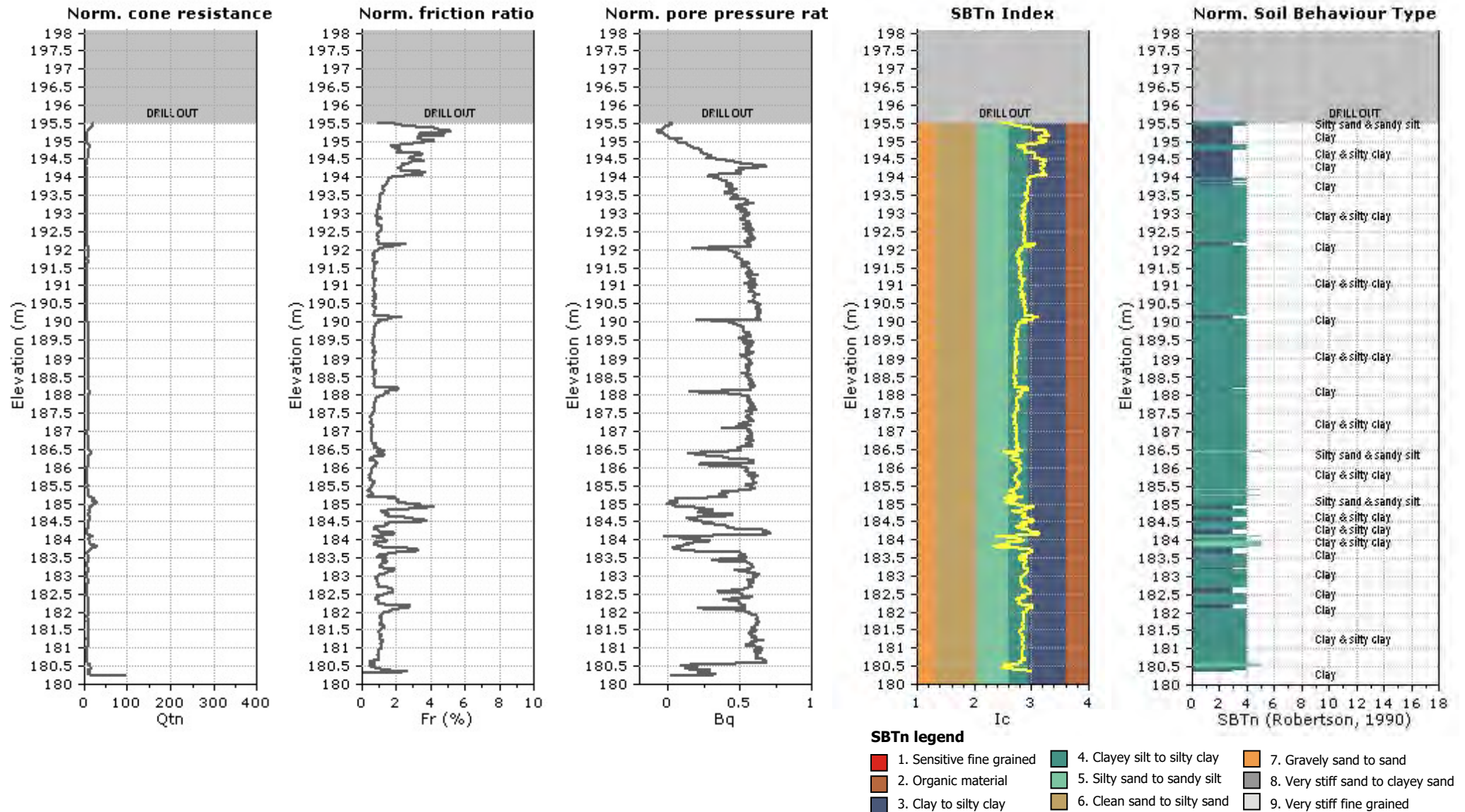
Total depth: 17.88 m, Date: 2019-08-08

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Location: Highway 17

CPT: SCPT19-02

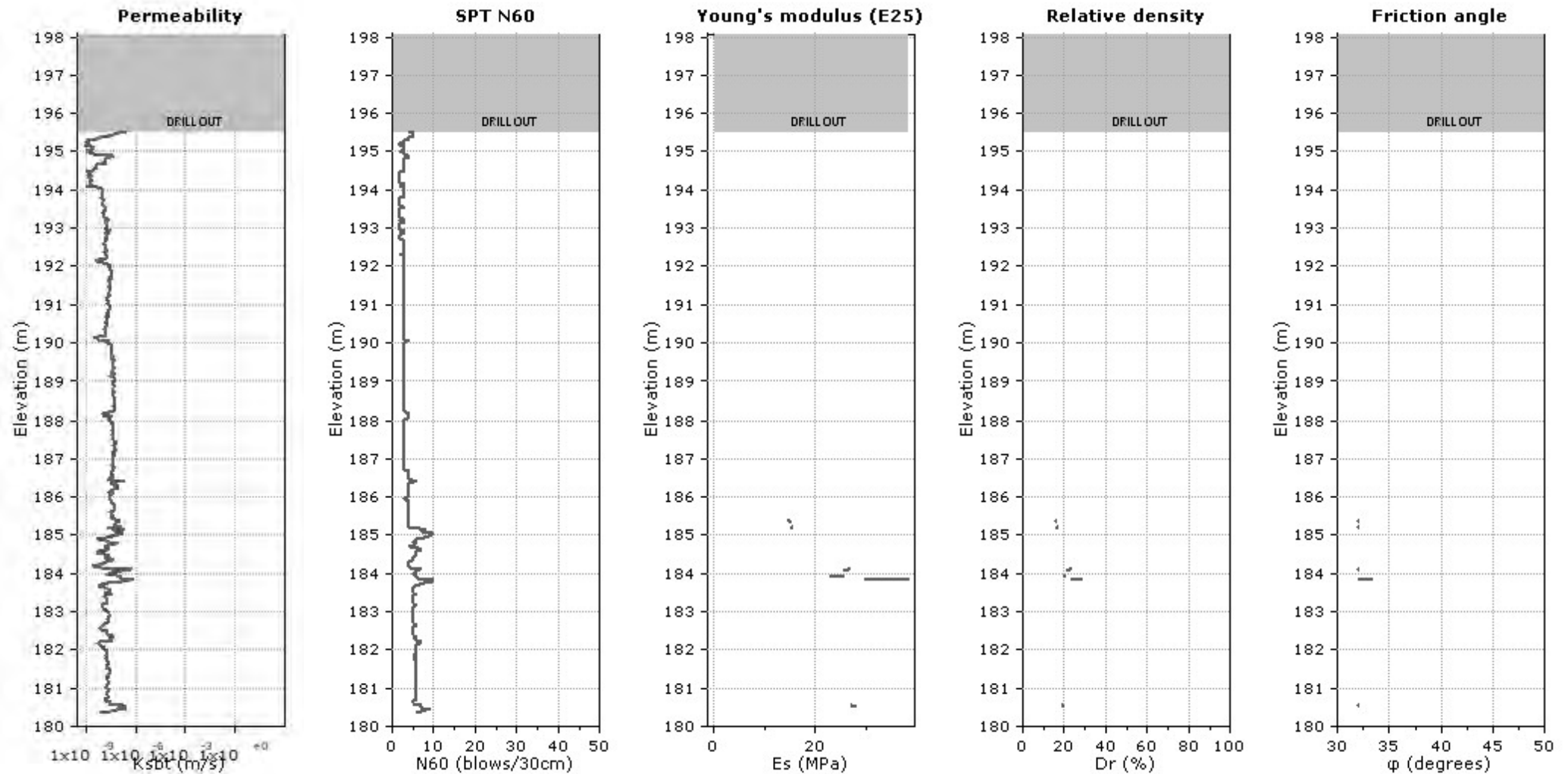
Total depth: 17.88 m, Date: 2019-08-08

Surface Elevation: 198.12 m

Coords: X:288559.30, Y:5136772.50

Cone Type: 15cm2

Cone Operator: ConeTec



Calculation parameters

Permeability: Based on SBT_n

SPT N_{60} : Based on I_c and q_t

Young's modulus: Based on variable alpha using I_c (Robertson, 2009)

Relative density constant, C_{Dr} : 350.0

Phi: Based on Kulhawy & Mayne (1990)

● User defined estimation data



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Location: Highway 17

CPT: SCPT19-02

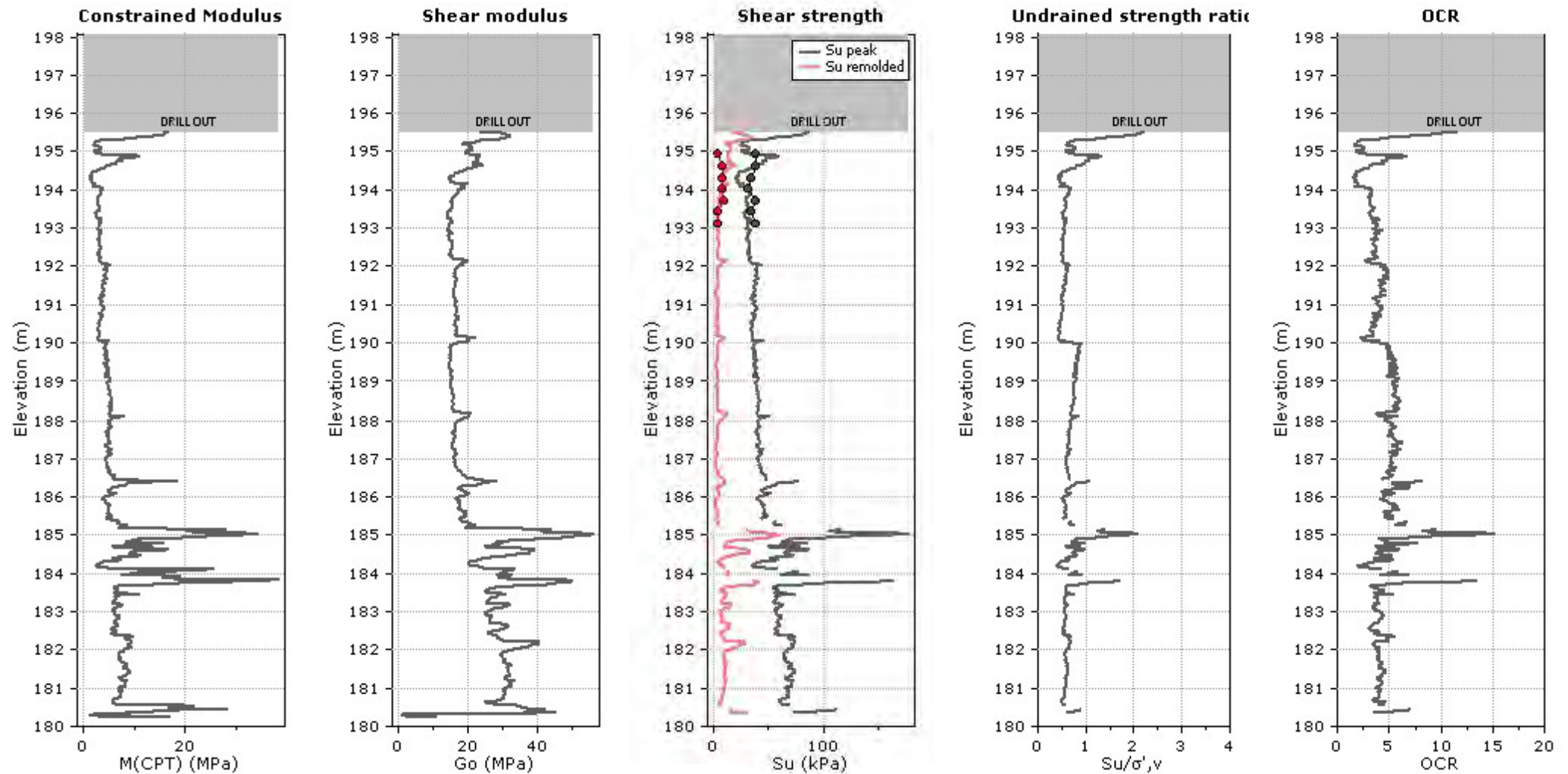
Total depth: 17.88 m, Date: 2019-08-08

Surface Elevation: 198.12 m

Coords: X:288559.30, Y:5136772.50

Cone Type: 15cm2

Cone Operator: ConeTec



Calculation parameters

Constrained modulus: Based on variable alpha using I_c and Q_{tn} (Robertson, 2009)

Go: Based on variable alpha using I_c (Robertson, 2009)

Undrained shear strength cone factor for clays, N_{kt} : 14

OCR factor for clays, N_{kt} : Auto

—●— User defined estimation data

—●— Flat Dilatometer Test data



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Location: Highway 17

CPT: SCPT19-02

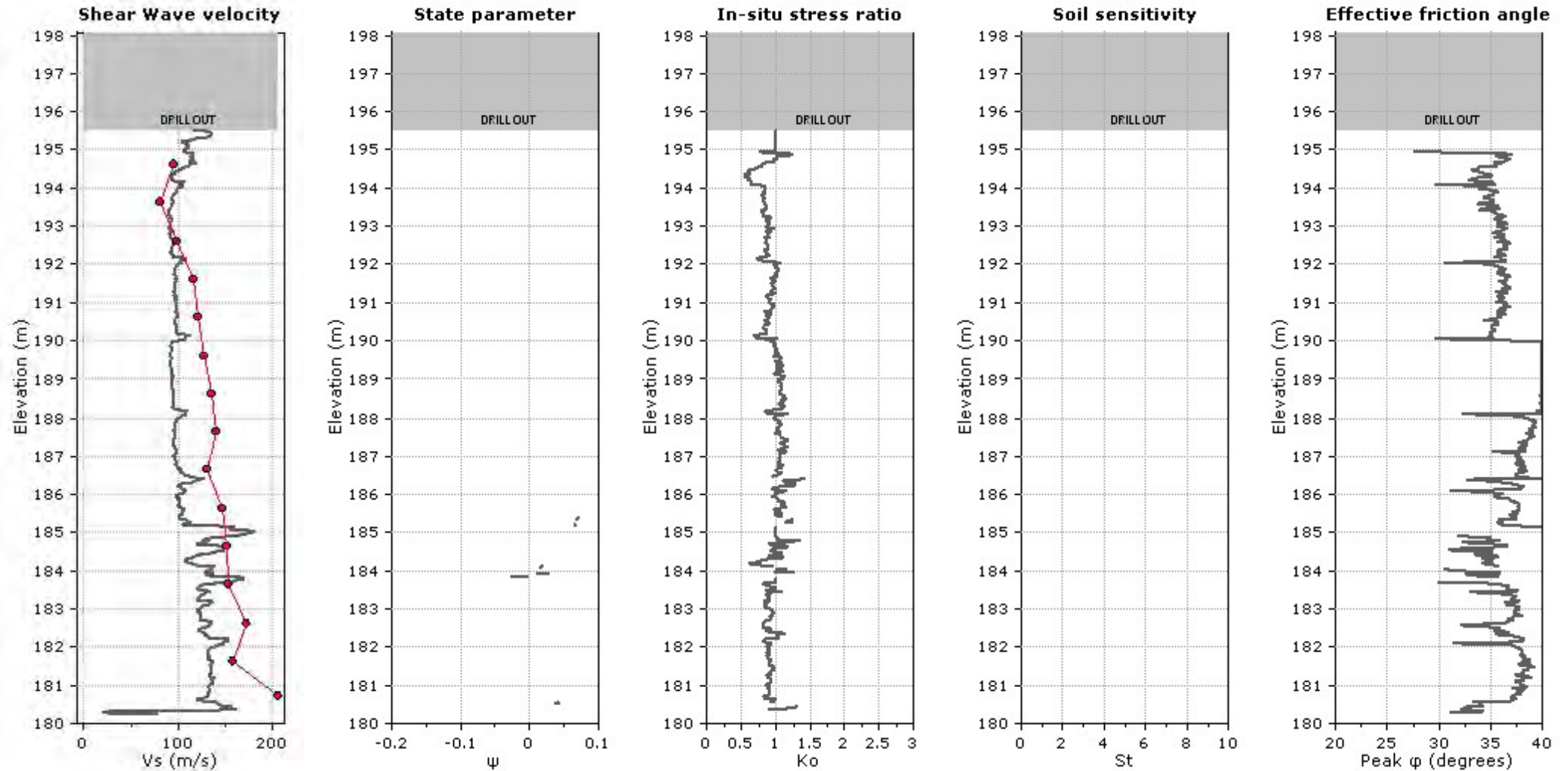
Total depth: 17.88 m, Date: 2019-08-08

Surface Elevation: 198.12 m

Coords: X:288559.30, Y:5136772.50

Cone Type: 15cm2

Cone Operator: ConeTec



Calculation parameters

Sol Sensitivity factor, N_s : 350.00

—●— User defined estimation data



Job No: 19-05054
Client: Thurber Engineering
Project: Laronde Creek Bridge
Sounding ID: SCPT19-01
Date: 08-Aug-2019

Seismic Source: Beam
Source Offset (m): 1.00
Source Depth (m): 0.00
Geophone Offset (m): 0.20

SCPT_u SHEAR WAVE VELOCITY TEST RESULTS - V_s

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
3.00	2.80	2.97			
4.00	3.80	3.93	0.96	9.93	96
5.00	4.80	4.90	0.97	12.05	81
6.03	5.83	5.92	1.01	10.20	99
7.00	6.80	6.87	0.96	8.21	117
8.03	7.83	7.89	1.02	8.42	121
9.00	8.80	8.86	0.96	7.56	127
10.00	9.80	9.85	0.99	7.34	135
11.00	10.80	10.85	1.00	7.13	140
12.00	11.80	11.84	1.00	7.63	131
13.03	12.83	12.87	1.03	6.96	147
14.00	13.80	13.84	0.97	6.35	152
15.00	14.80	14.83	1.00	6.48	154
16.00	15.80	15.83	1.00	5.77	173
17.00	16.80	16.83	1.00	6.31	158
17.88	17.68	17.71	0.88	4.26	206

Dissipation Tests Results

Dissipation tests

Dissipation tests consists of stopping the piezocone penetration and observing porepressures (u) with elapsed time (t). The data are automatic recorded by the field computer and should take place until a minimum of 50% dissipation.

The porepressures are plotted as a function of square root of (t). The graphical technique suggested by Robertson and Campanella (1989), yields a value for t_{50} , which corresponds to the time for 50% consolidation.

The value of the coefficient of consolidation in the radial or horizontal direction c_h was then calculated by Houlsby and Teh's (1988) theory using the following equation:

$$c_h = \frac{T \times r^2 \times I_r^{0.5}}{t_{50}}$$

where:

T: time factor given by Houlsby and Teh's (1988) theory corresponding to the porepressure position

r: piezocone radius

I_r : stiffness index, equal to shear modulus G divided by the undrained strength of clay (S_u).

t_{50} : time corresponding to 50% consolidation

Permeability estimates based on dissipation test

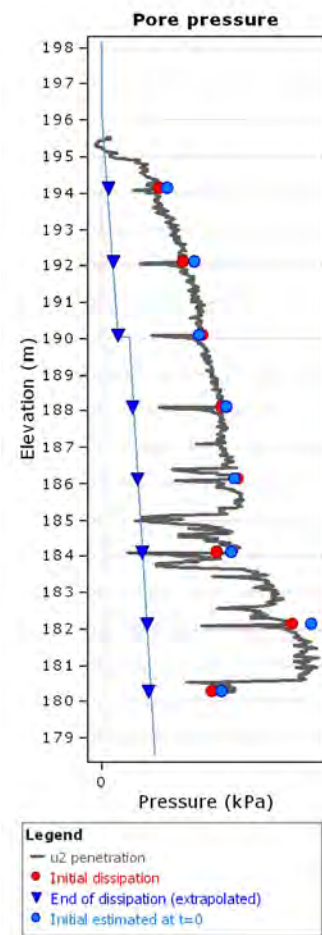
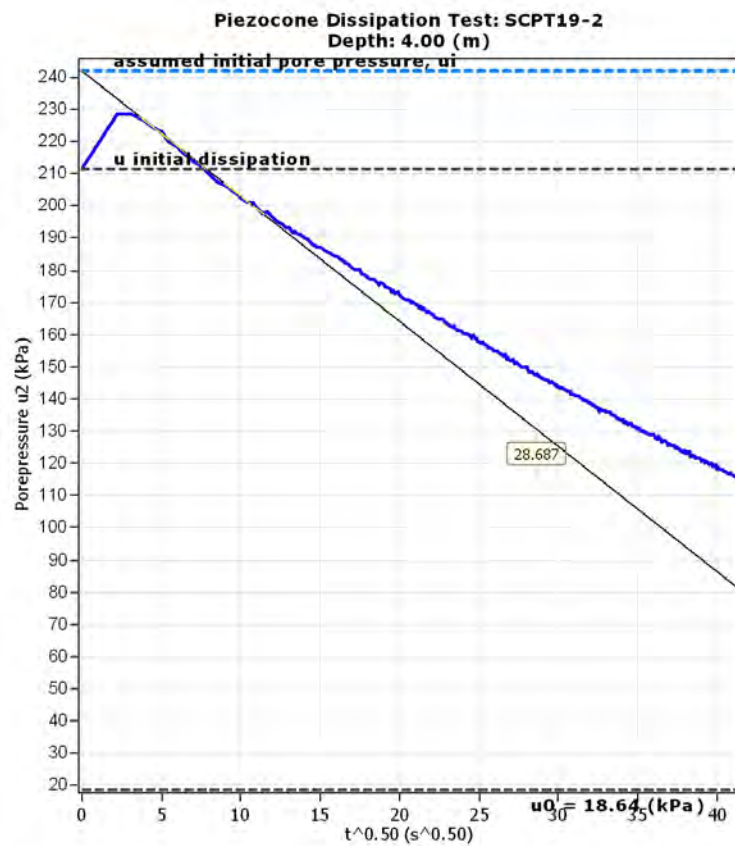
The dissipation of pore pressures during a CPTu dissipation test is controlled by the coefficient of consolidation in the horizontal direction (c_h) which is influenced by a combination of the soil permeability (k_h) and compressibility (M), as defined by the following:

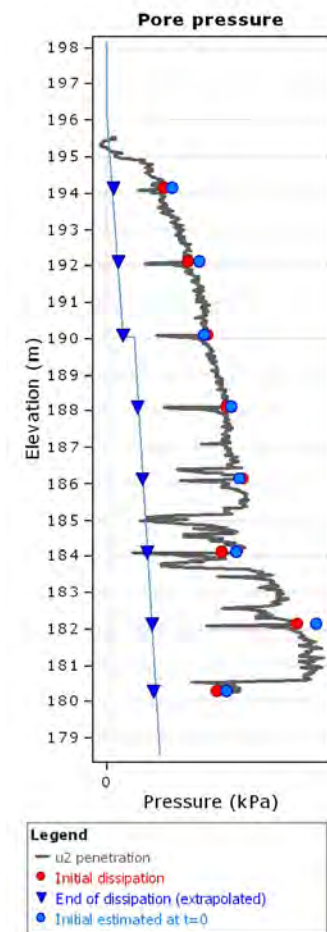
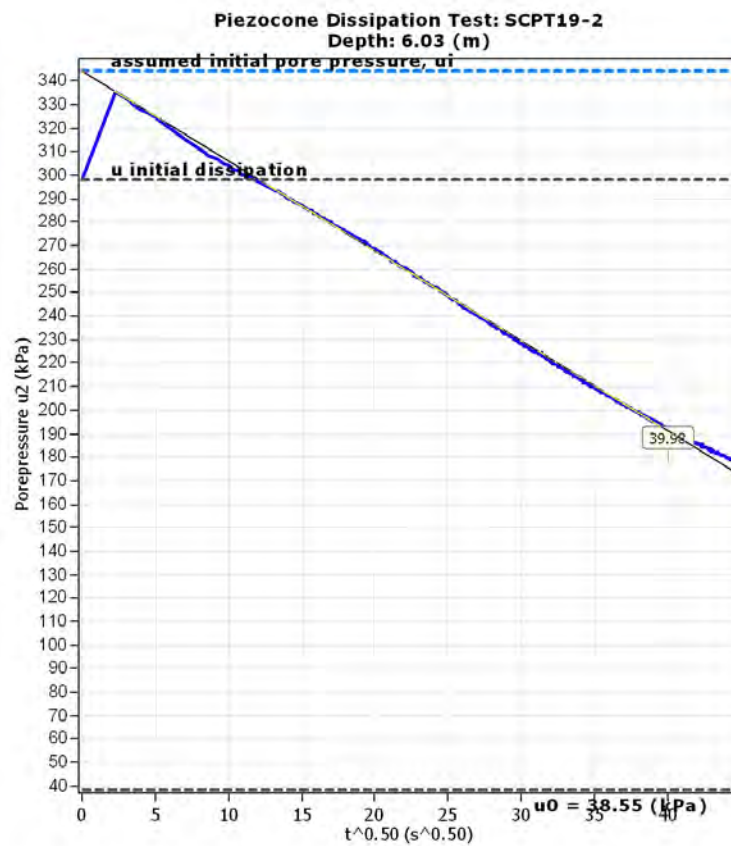
$$k_h = c_h \times \gamma_w / M$$

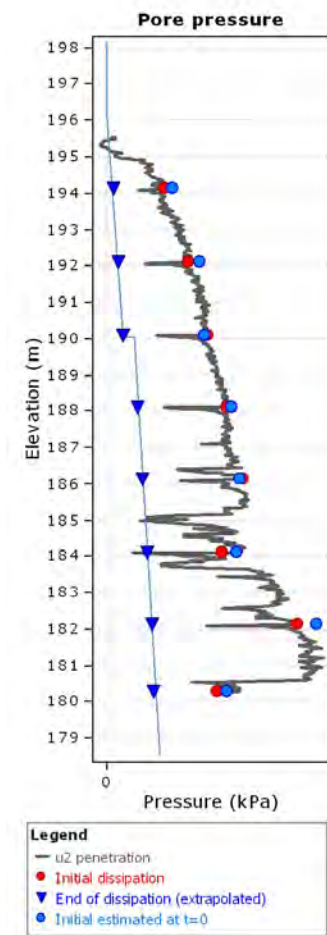
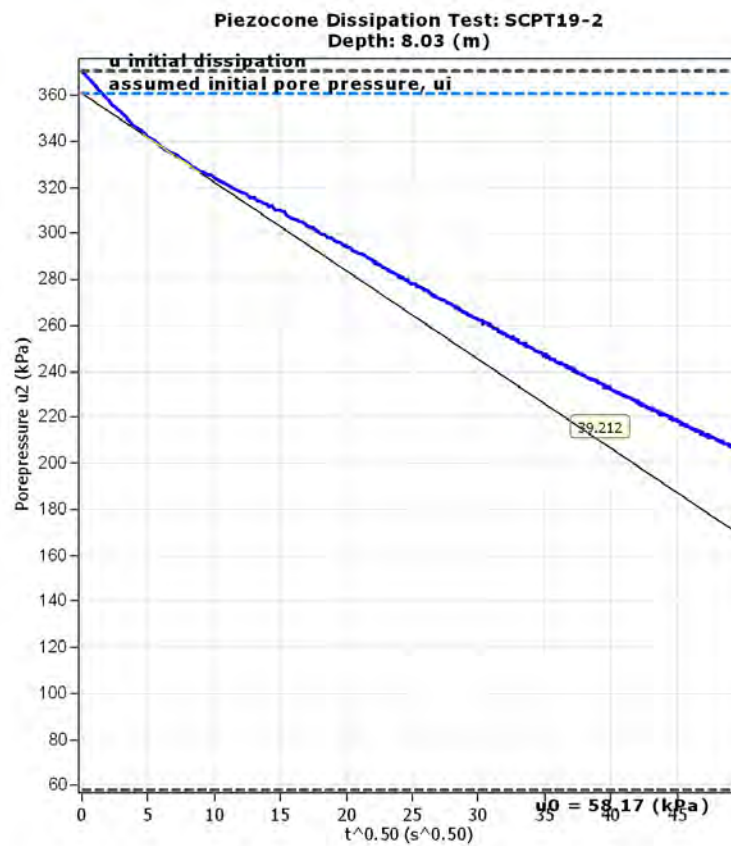
where: M is the 1-D constrained modulus and γ_w is the unit weight of water, in compatible units.

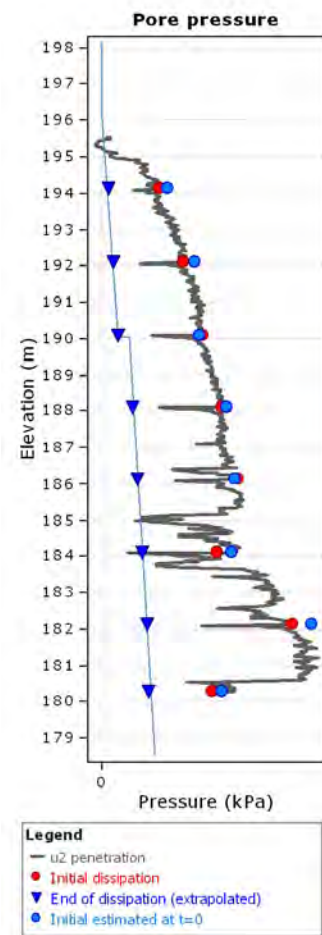
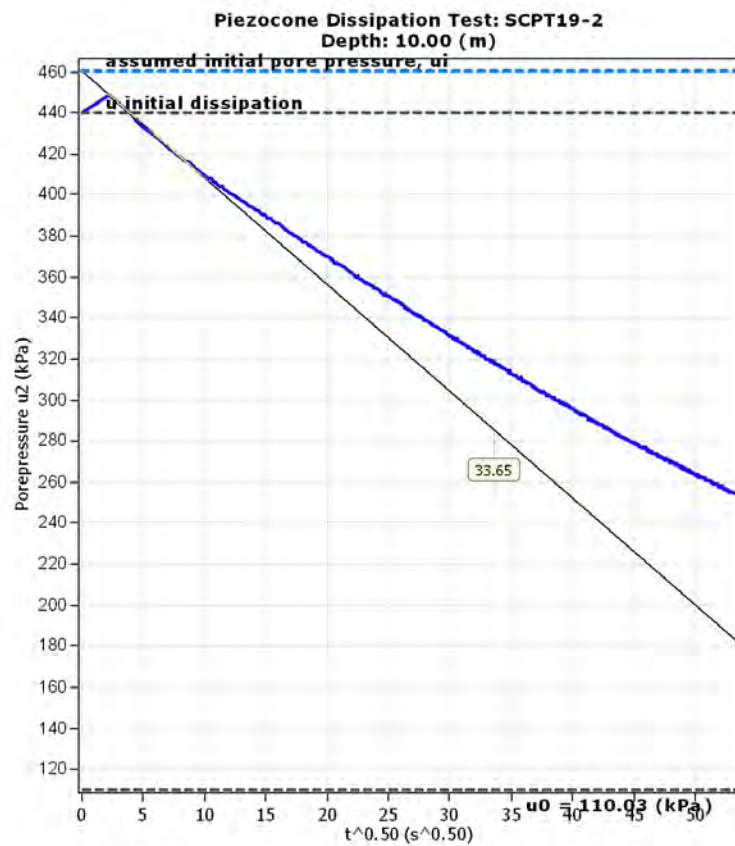
Tabular results

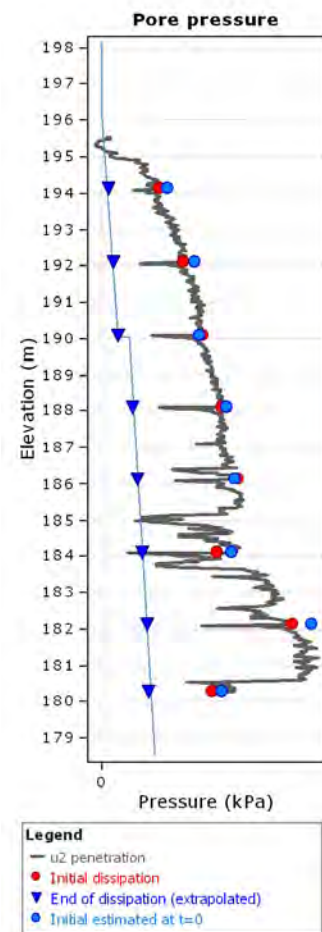
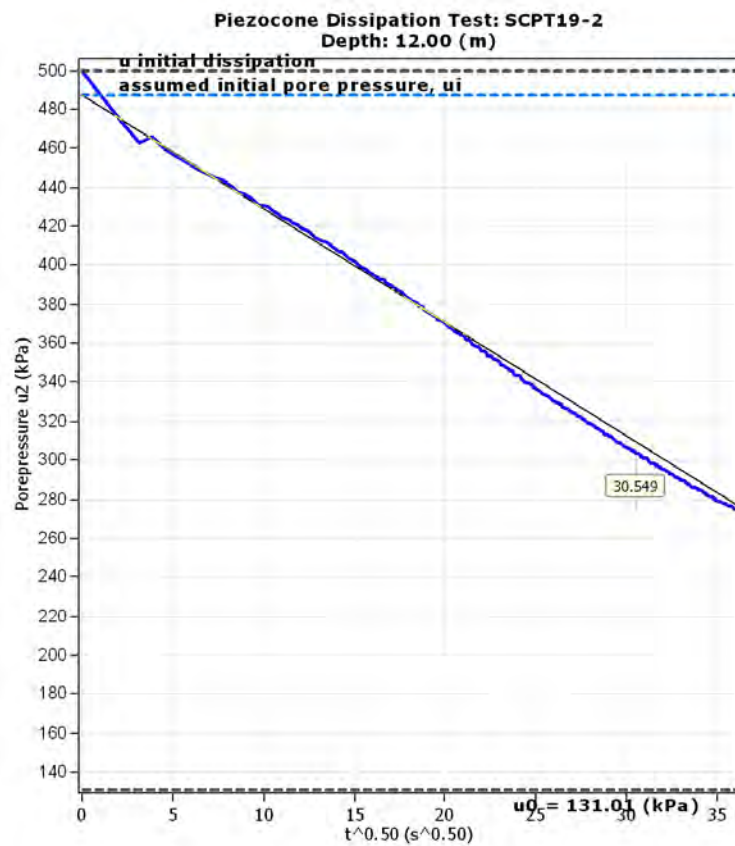
CPTU Borehole	Depth (m)	$(t_{50})^{0.50}$	t_{50} (s)	t_{50} (years)	G/ S_u	c_h (m ² /s)	c_h (m ² /year)	M (MPa)	k_h (m/s)
SCPT19-2	4.00	28.7	823	2.61E-005	212.30	2.08E-006	66	2.25	9.05E-009
SCPT19-2	6.03	40.0	1598	5.07E-005	381.96	1.44E-006	45	4.84	2.91E-009
SCPT19-2	8.03	39.2	1538	4.88E-005	357.57	1.45E-006	46	5.15	2.75E-009
SCPT19-2	10.00	33.7	1132	3.59E-005	390.44	2.05E-006	65	6.71	3.00E-009
SCPT19-2	12.00	30.5	933	2.96E-005	264.90	2.05E-006	65	7.31	2.75E-009
SCPT19-2	14.00	5.8	34	1.07E-006	100.00	3.49E-005	1099	15.63	2.19E-008
SCPT19-2	16.00	38.5	1483	4.70E-005	422.62	1.63E-006	51	14.02	1.14E-009
SCPT19-2	17.85	3.7	14	4.33E-007	100.00	8.61E-005	2715	6.70	1.26E-007

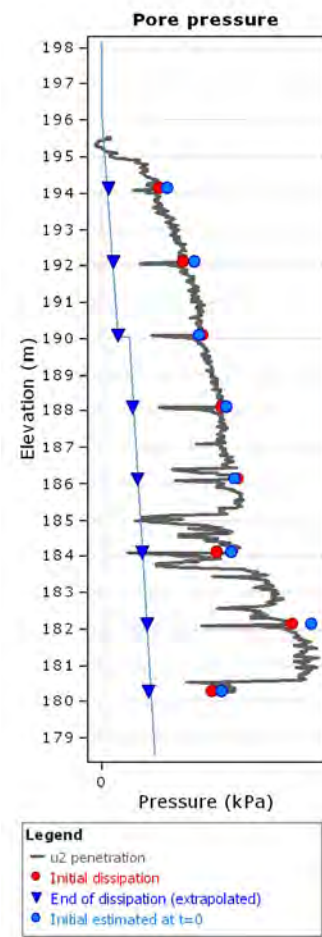
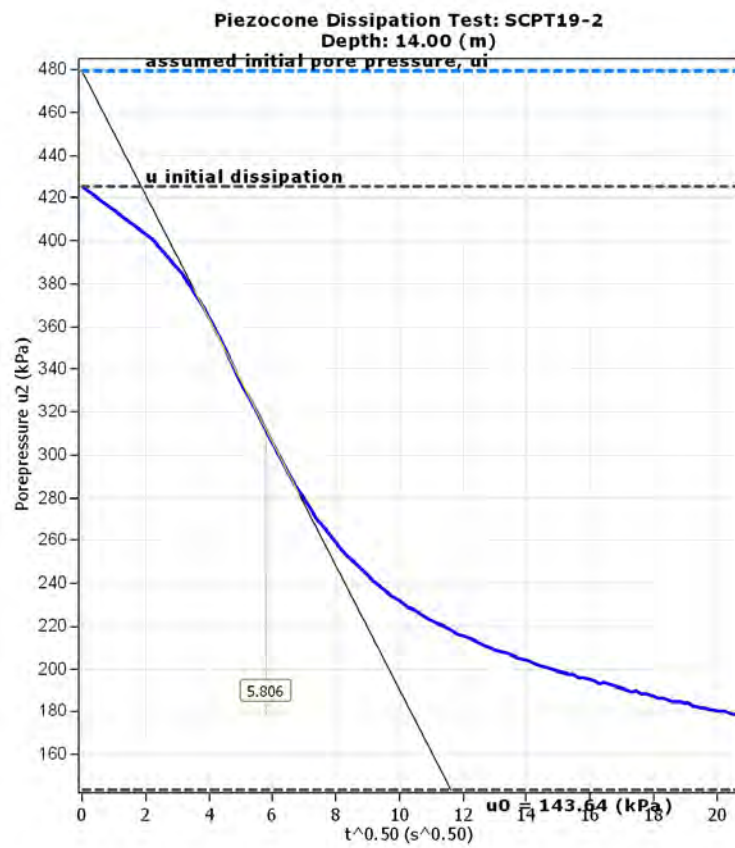


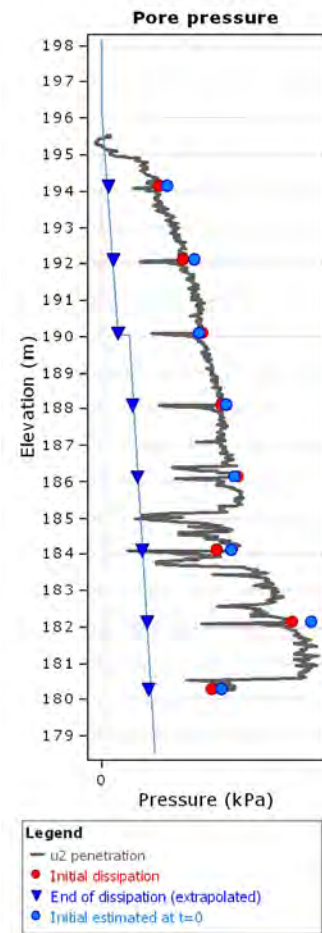
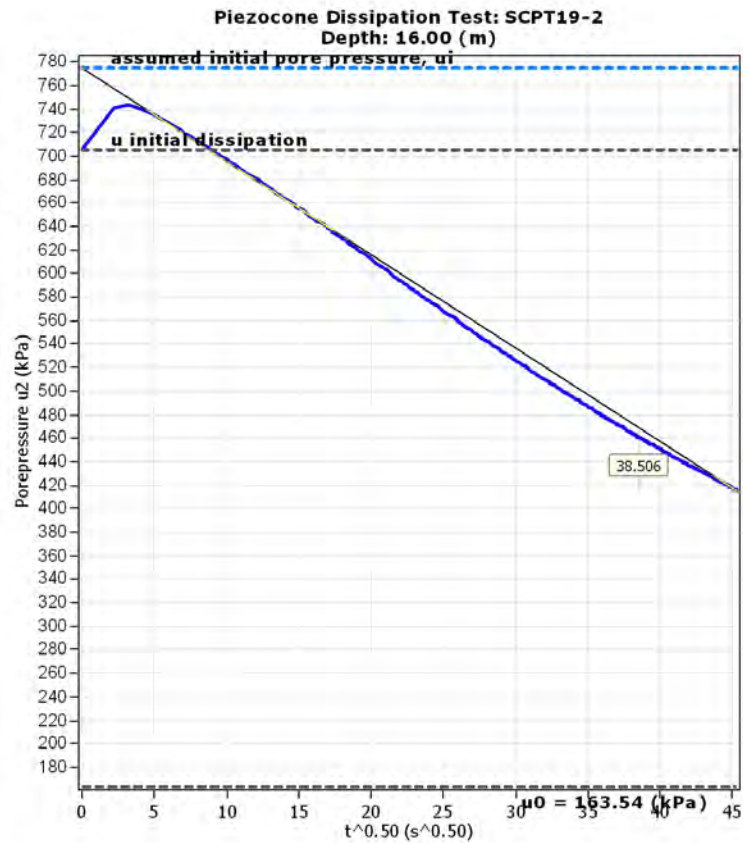


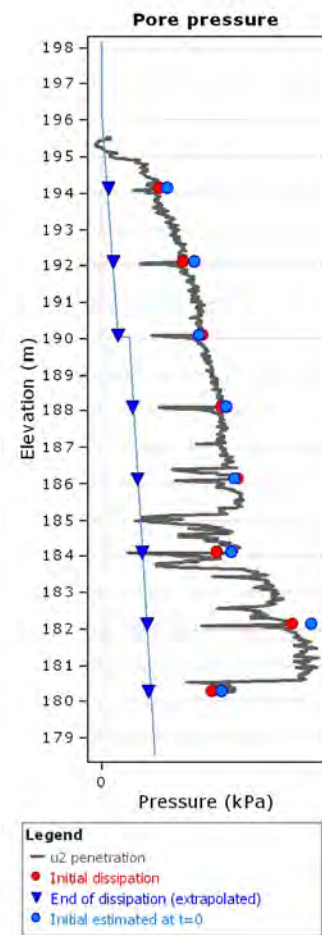
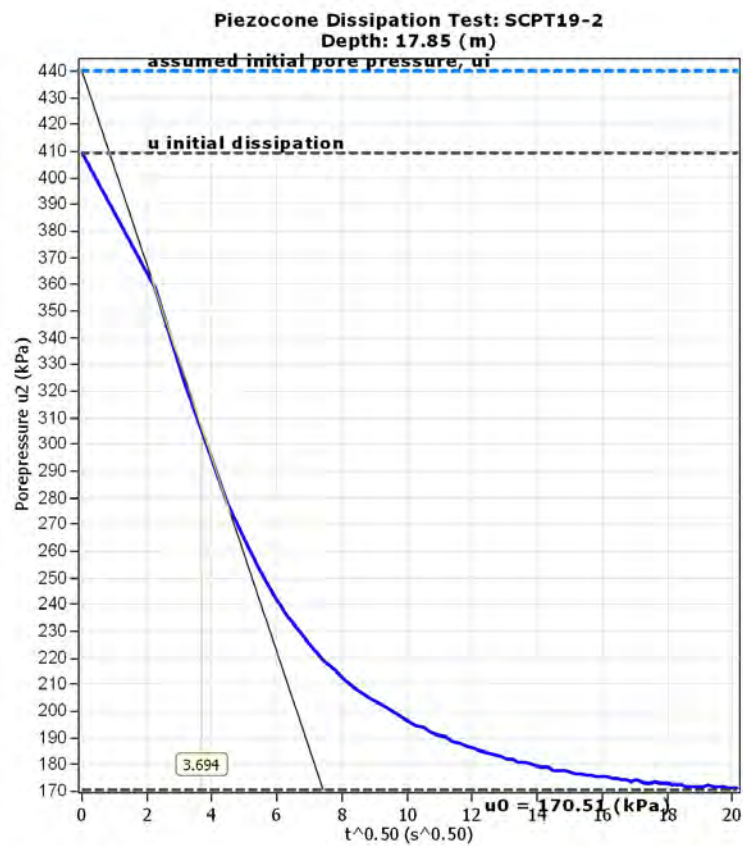














Appendix C.

Laboratory Testing



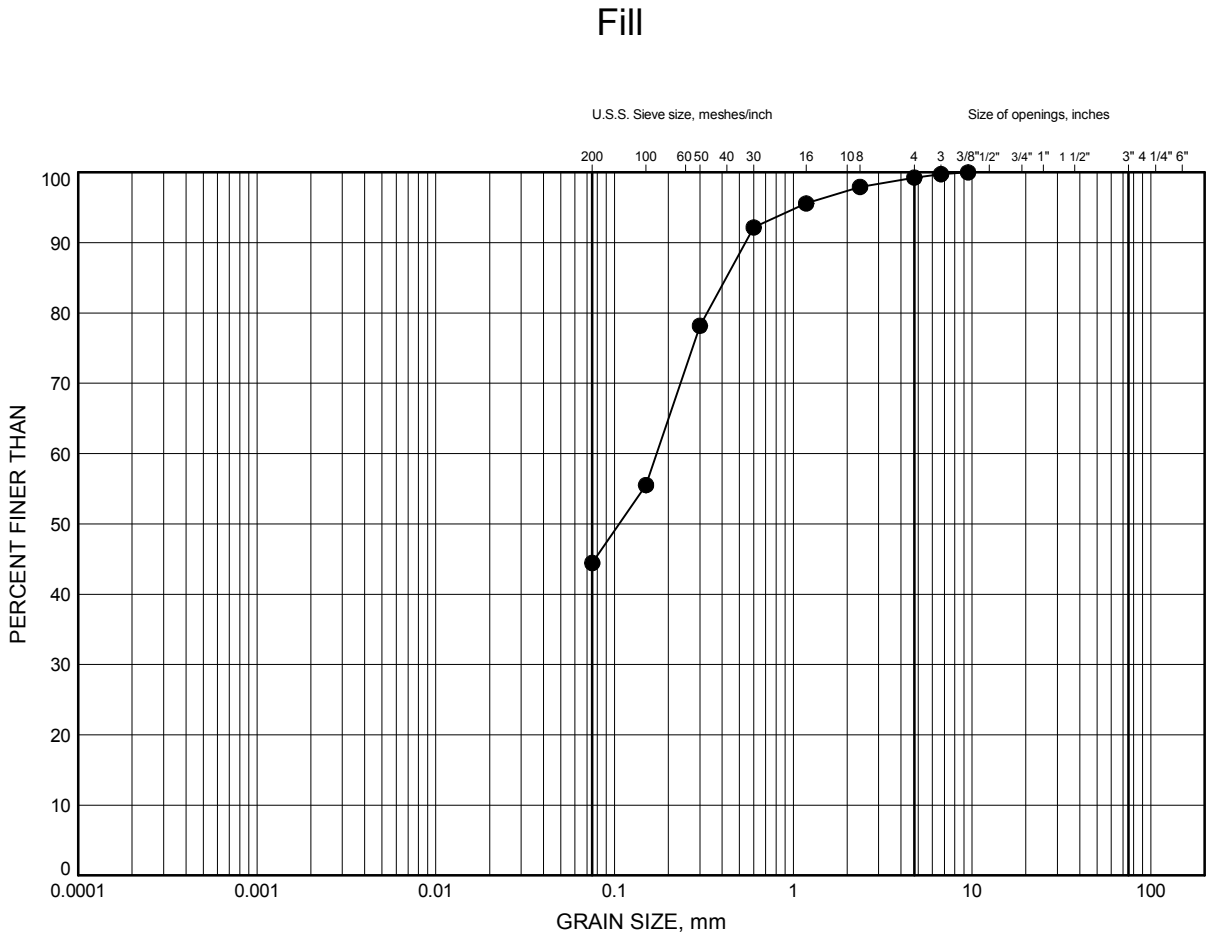
Appendix C.1

Particle Size Analysis and Atterberg Limits Figures

Highway 17 Laronde Creek Bridge

GRAIN SIZE DISTRIBUTION

FIGURE C1



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	19-01	0.3	199.5

Date 05-11-00
GWP# 5198-13-00

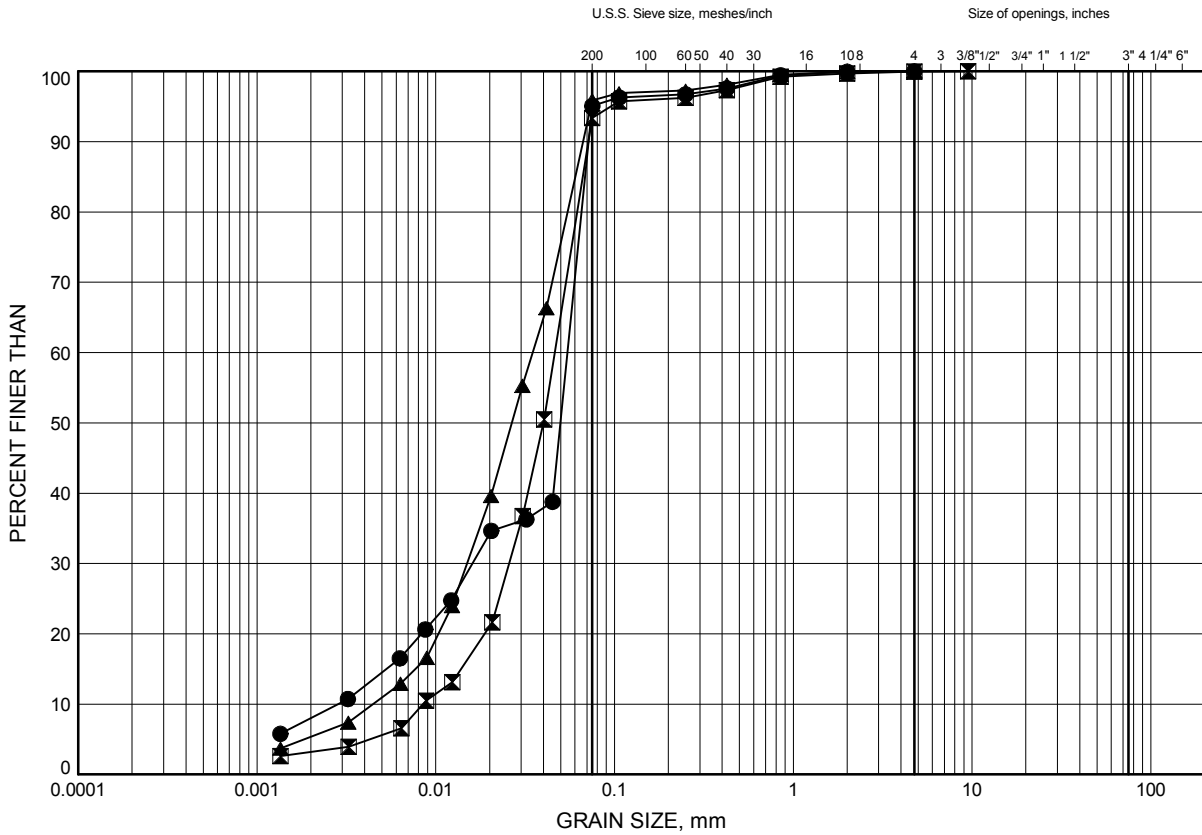


Prep'd DJP
Chkd. FG

Highway 17 Laronde Creek Bridge GRAIN SIZE DISTRIBUTION

FIGURE C2

Surficial Sand and Silt



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	19-3	0.9	196.9
⊠	19-4	0.9	201.9
▲	19-4	2.7	200.1

Date 05-11-00
GWP# 5198-13-00



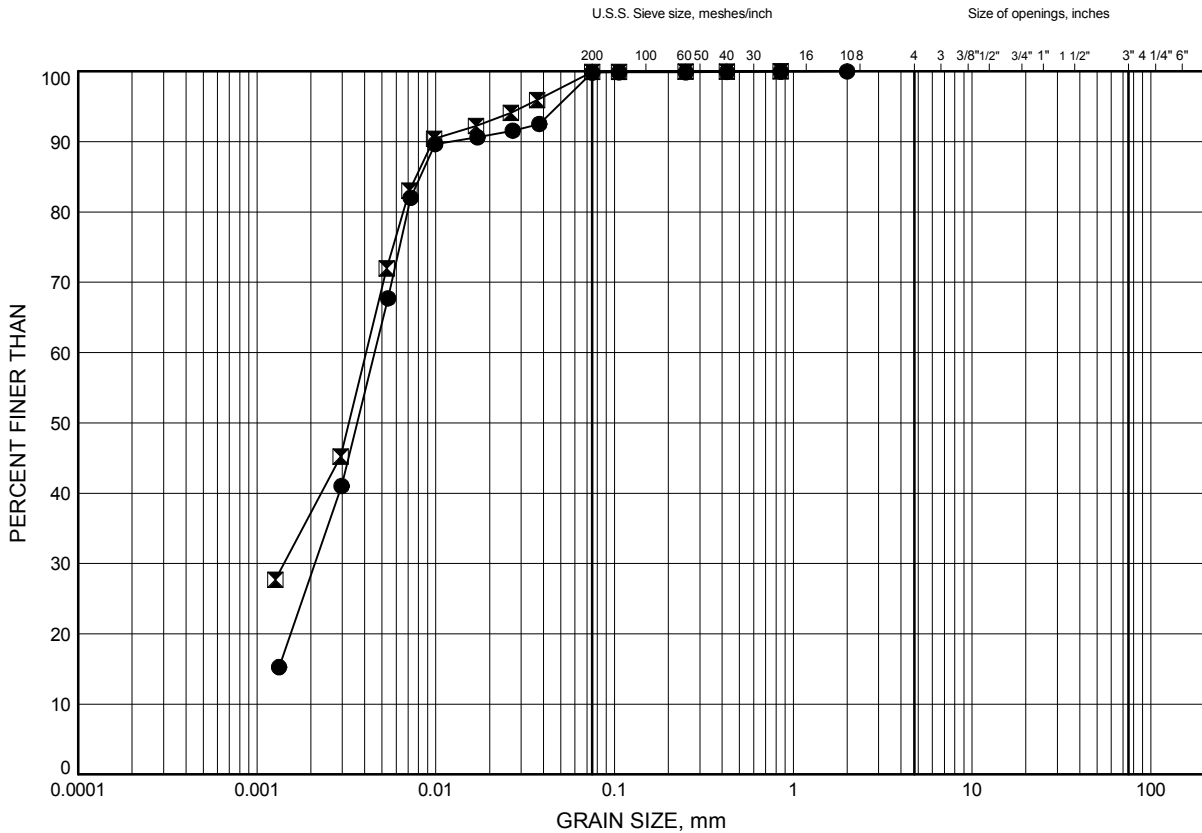
Prep'd DJP
Chkd. FG

Highway 17 Laronde Creek Bridge

GRAIN SIZE DISTRIBUTION

FIGURE C3

Upper Clayey Silt (CL-ML to CL)



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	19-01	2.6	197.2
⊠	19-3	3.4	194.4

Date 05-11-00
GWP# 5198-13-00



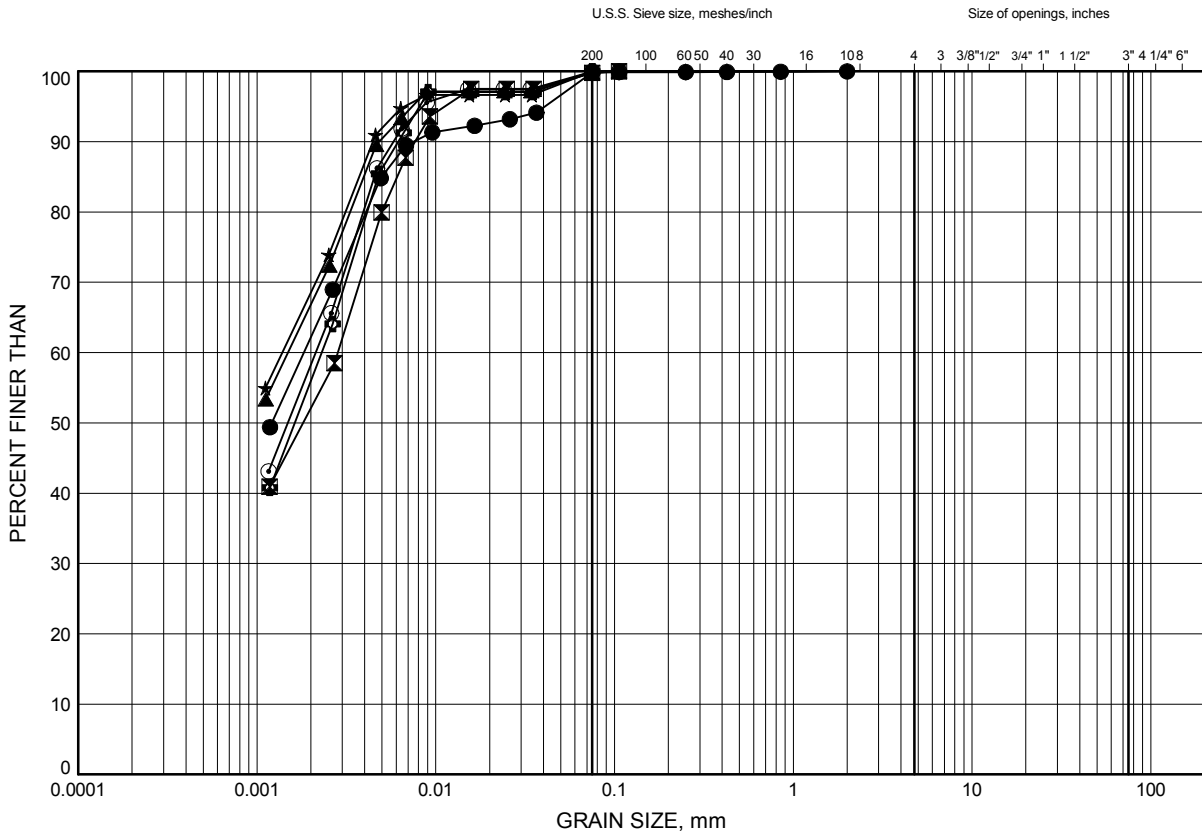
Prep'd DJP
Chkd. FG

Highway 17 Laronde Creek Bridge

GRAIN SIZE DISTRIBUTION

FIGURE C4

Lower Clayey Silt (CL to CL-ML) to Clay (CI to CH)



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	19-01	7.9	191.9
⊠	19-2A	3.4	195.0
▲	19-2A	5.6	192.8
★	19-2A	5.8	192.6
⊙	19-2A	5.9	192.5
⊕	19-2A	7.6	190.8

Date 06-11-00
GWP# 5198-13-00



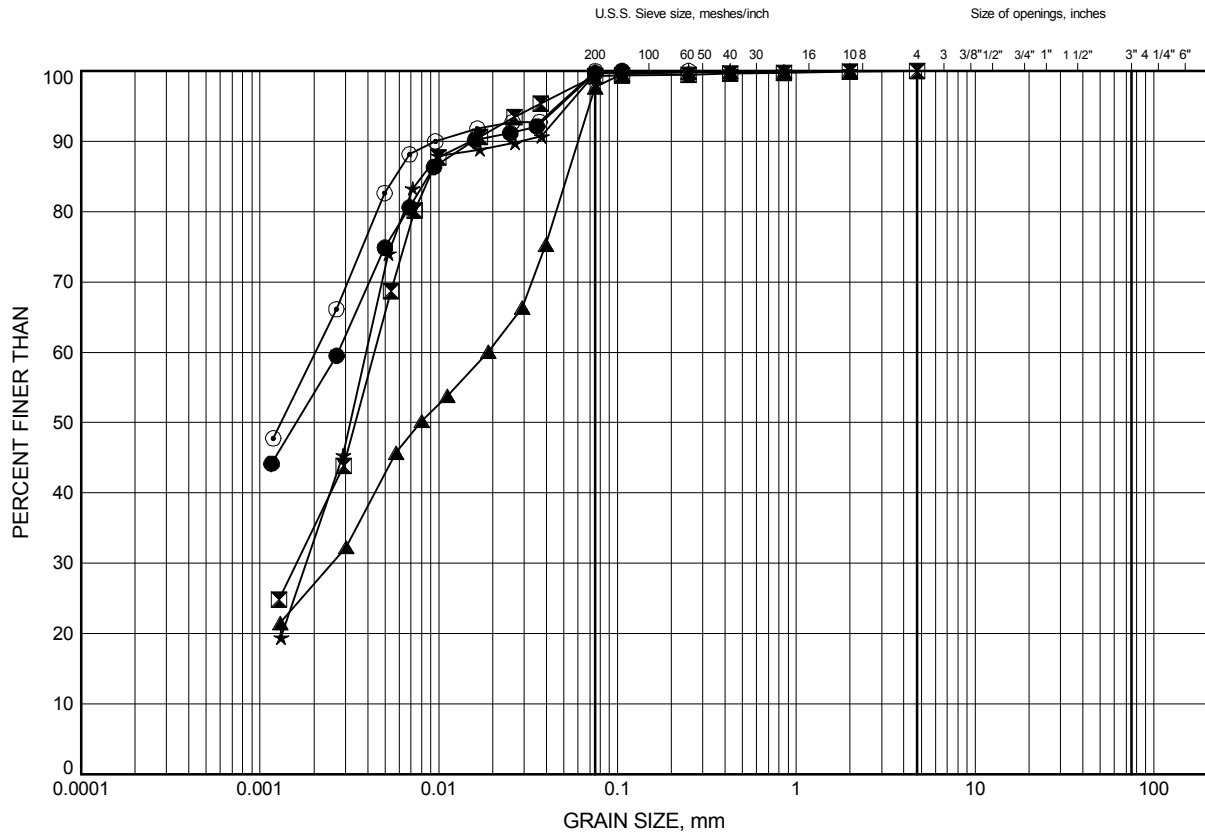
Prep'd DJP
Chkd. FG

Highway 17 Laronde Creek Bridge

GRAIN SIZE DISTRIBUTION

FIGURE C5

Lower Clayey Silt (CL to CL-ML) to Clay (CI to CH)



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	19-02A	12.4	186.0
⊠	19-02B	1.8	196.5
▲	19-03	12.5	185.3
★	19-04	4.6	198.2
⊙	19-04	9.4	193.4

Date 06-11-2006
GWP# 5198-13-00



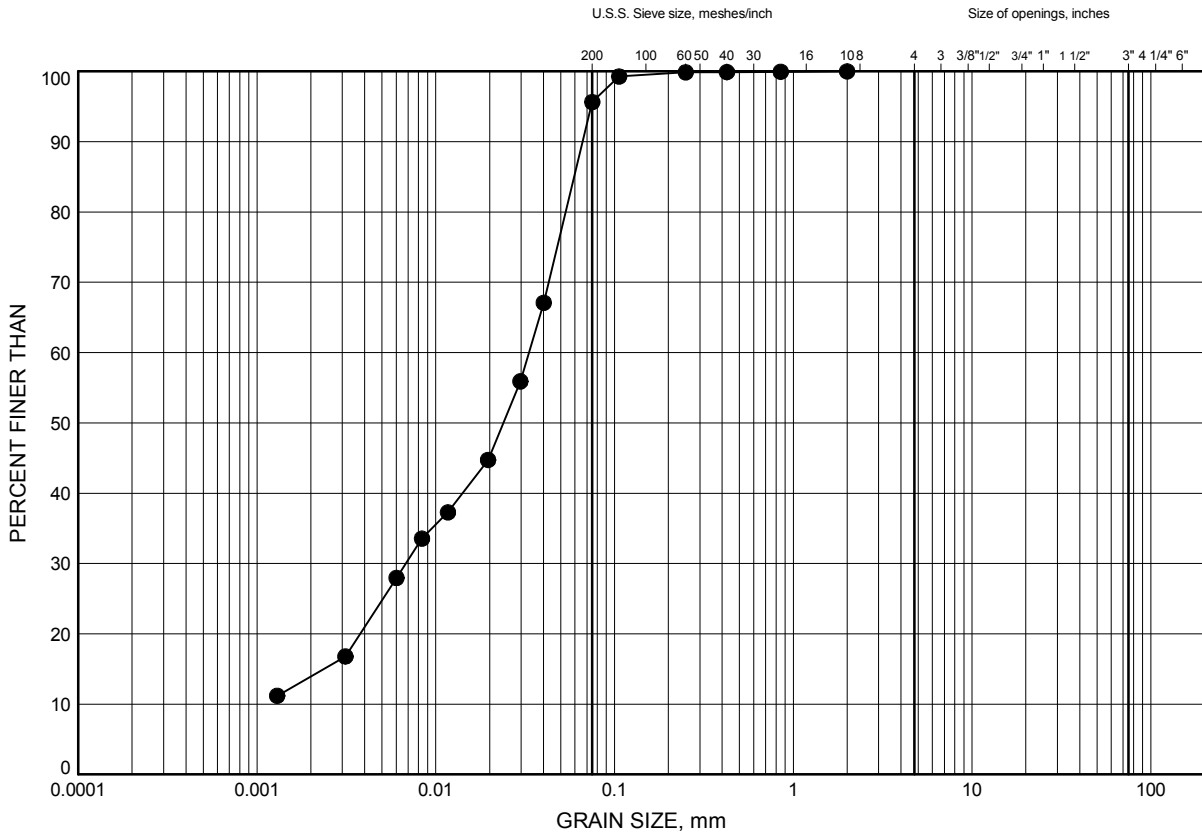
Prep'd DJP
Chkd. FG

Highway 17 Laronde Creek Bridge

GRAIN SIZE DISTRIBUTION

FIGURE C6

Lower Clayey Silt (CL) to Clay (CI to CH) - Silt Layer



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	19-2A	12.6	185.8

Date 05-11-2006
GWP# 5198-13-00



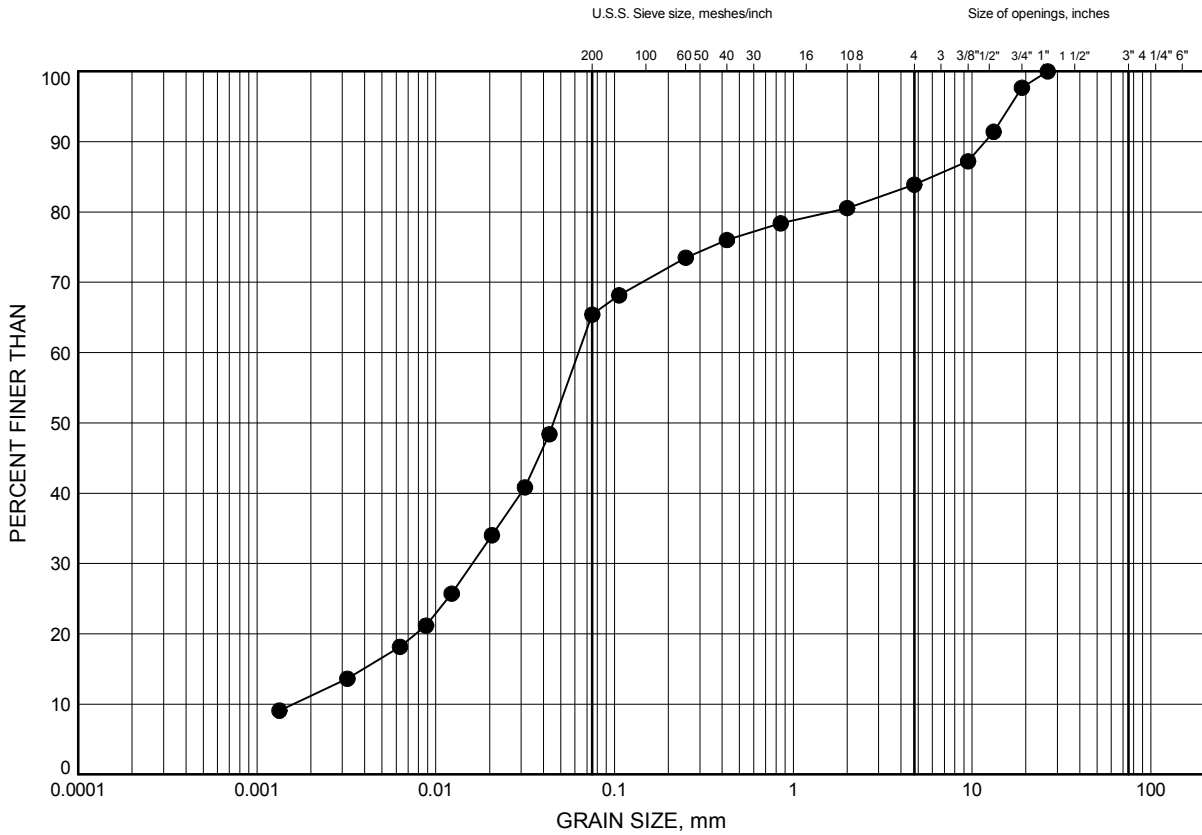
Prep'd DJP
Chkd. FG

Highway 17 Laronde Creek Bridge

GRAIN SIZE DISTRIBUTION

FIGURE C7

Sandy Silt with Gravel Till



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	19-3	16.9	180.9

Date 05-11-2006
GWP# 5198-13-00

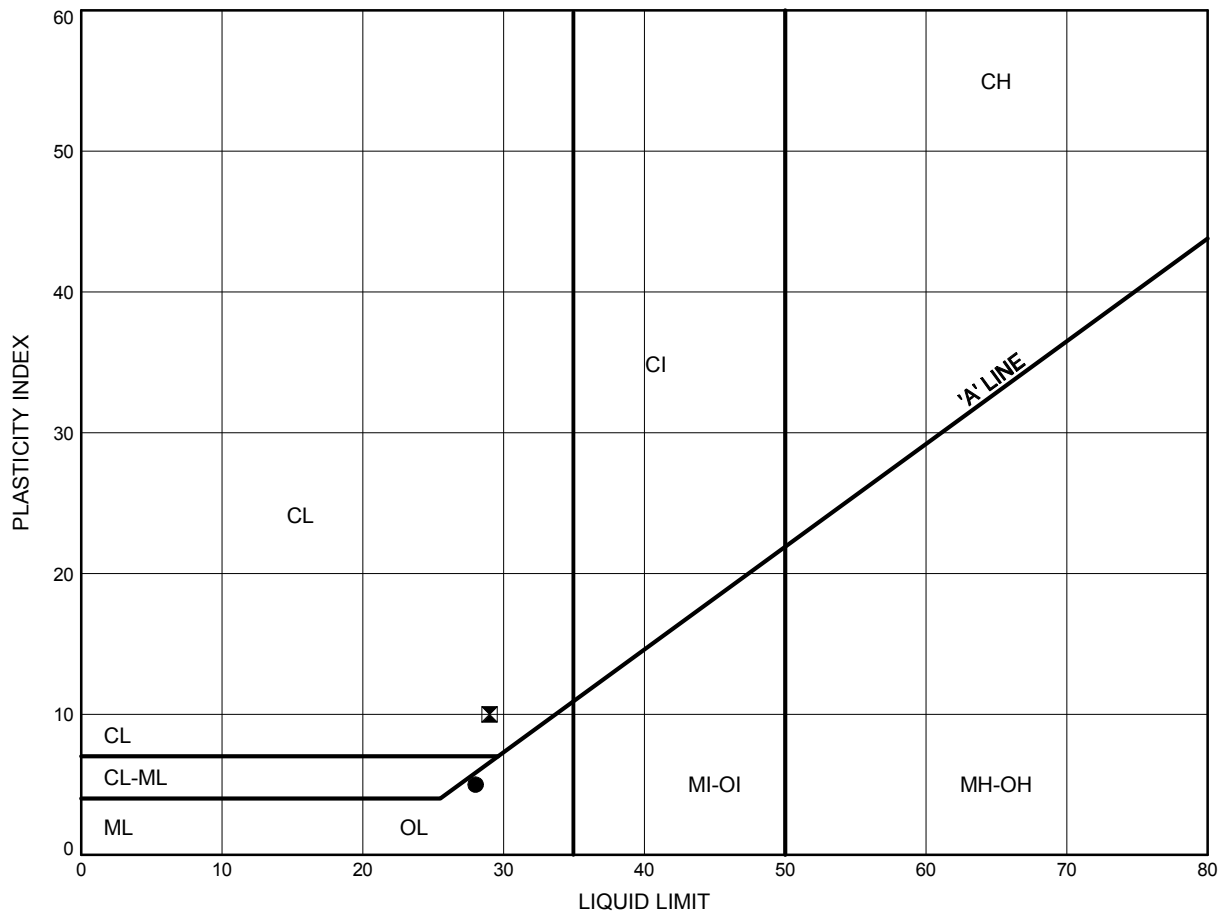


Prep'd DJP
Chkd. FG

Highway 17 Laronde Creek Bridge ATTERBERG LIMITS TEST RESULTS

FIGURE C8

Upper Clayey Silt (CL-ML to CL)



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	19-01	2.6	197.2
⊠	19-3	3.4	194.4

Date August 2020
GWP# 5198-13-00

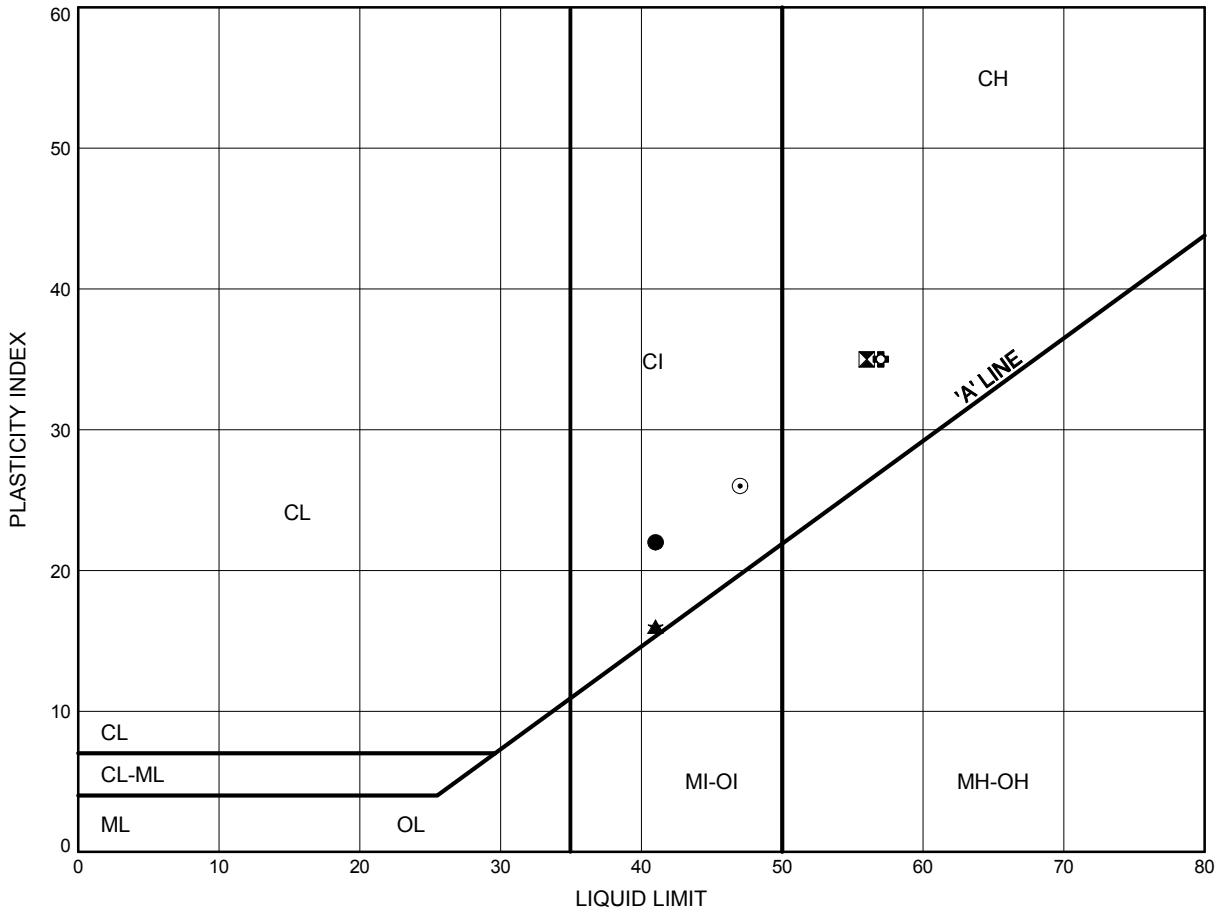


Prep'd DJP
Chkd. FG

Highway 17 Laronde Creek Bridge
ATTERBERG LIMITS TEST RESULTS

FIGURE C9

Lower Clayey Silt (CL to CL-ML) to Clay (CI to CH)



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	19-01	7.9	191.9
⊠	19-2A	3.4	195.0
▲	19-2A	5.1	193.3
★	19-2A	5.3	193.1
⊙	19-2A	5.6	192.8
⊕	19-2A	5.8	192.6

Date August 2020
 GWP# 5198-13-00

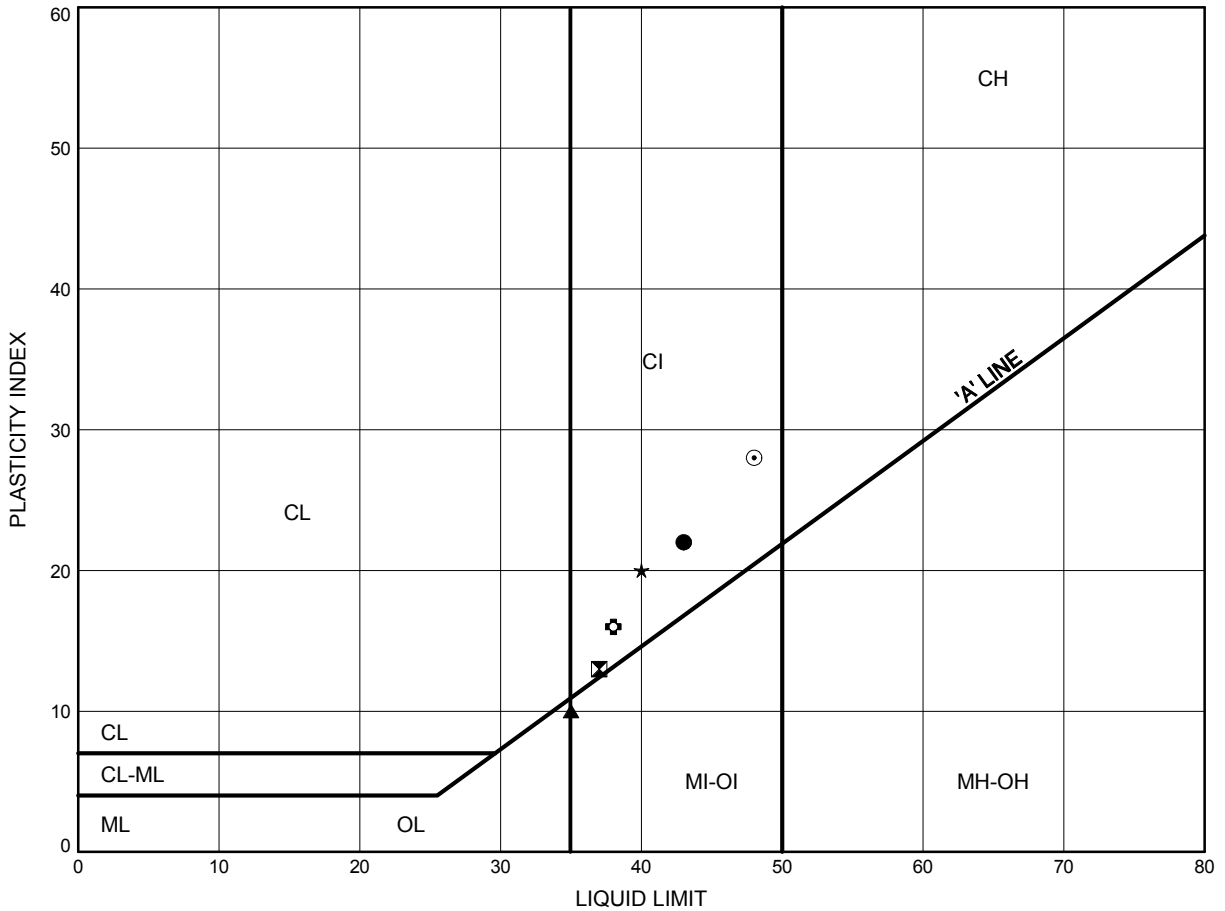


Prep'd DJP
 Chkd. FG

Highway 17 Laronde Creek Bridge
ATTERBERG LIMITS TEST RESULTS

FIGURE C10

Lower Clayey Silt (CL to CL-ML) to Clay (CI to CH)



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	19-2A	5.9	192.5
⊠	19-2A	6.3	192.1
▲	19-2A	6.5	191.9
★	19-2A	7.6	190.8
⊙	19-2A	12.4	186.0
⊕	19-2B	1.8	196.5

Date August 2020
 GWP# 5198-13-00

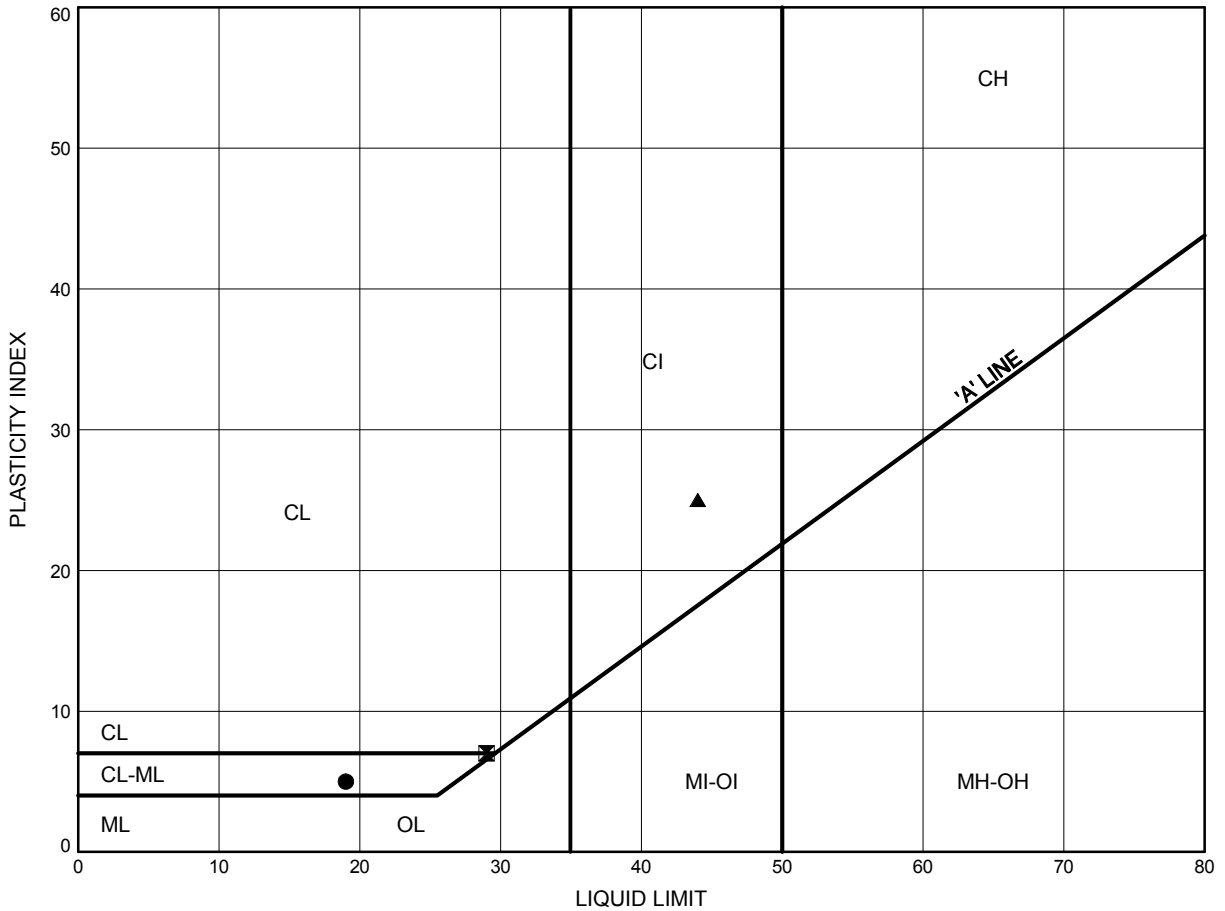


Prep'd DJP
 Chkd. FG

Highway 17 Laronde Creek Bridge
ATTERBERG LIMITS TEST RESULTS

FIGURE C11

Lower Clayey Silt (CL to CL-ML) to Clay (CI to CH)



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	19-3	12.5	185.3
⊠	19-4	4.6	198.2
▲	19-4	9.4	193.4

Date August 2020
 GWP# 5198-13-00

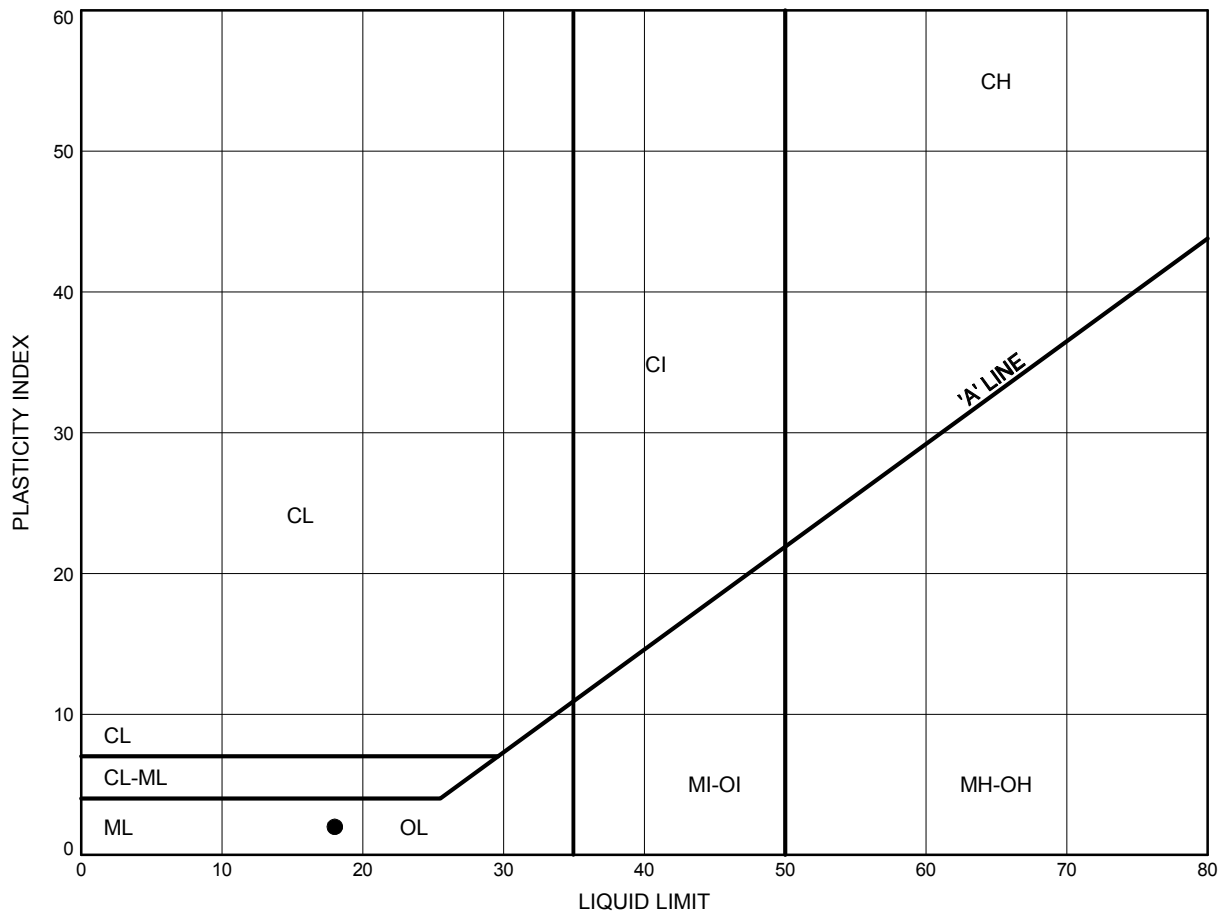


Prep'd DJP
 Chkd. FG

Highway 17 Laronde Creek Bridge
ATTERBERG LIMITS TEST RESULTS

FIGURE C12

Sandy Silt with Gravel Till



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	19-3	17.0	180.8

Date August 2020
 GWP# 5198-13-00



Prep'd DJP
 Chkd. FG



Appendix C.2

Consolidation Test Results



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

November 15, 2019
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 17 (Laronde) project, Thurber Consulting Ltd.,
File #23411: BH 19-2A, TW 1 & TW 5, sampled on August 13, 2019**

This letter presents the results of one-dimensional consolidation tests carried out on the above referenced samples 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.

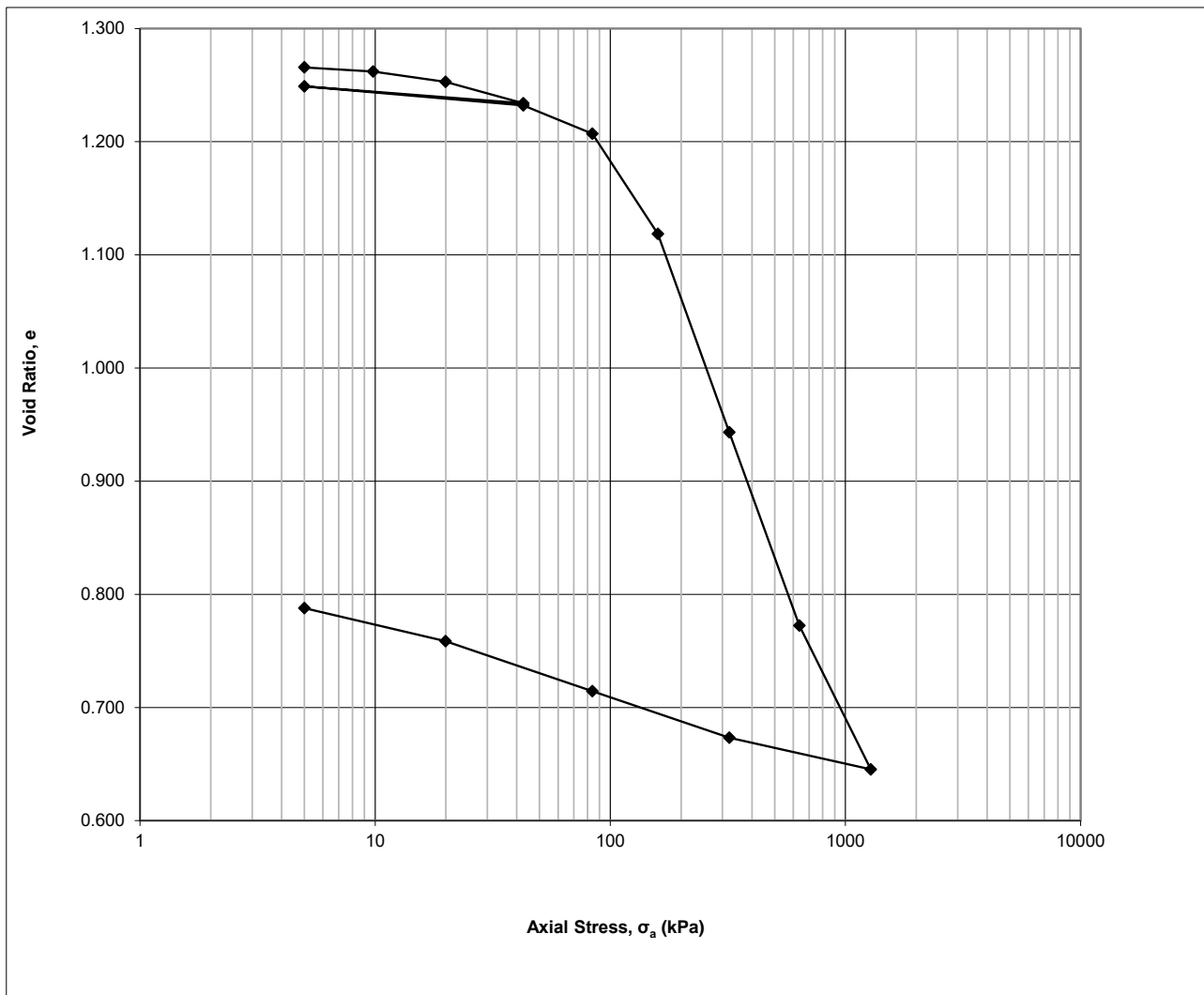
A handwritten signature in blue ink, appearing to read "Rajib Dey".

Rajib Dey Ph.D., P.Eng.
Geotechnical Engineer
Rajib.dey@stantec.com

v:\01216\active\laboratory_standing_offers\2019 laboratory standing offers\122410864 thurber engineering\sept 4, four consols, hydros, lims, sgs, file#23411\consolidation\122410864_let_consolidation_bh19-2a tw 1& 5.docx

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

Thurber Engineering, File# 23411
122410864
BH19-2A
TW 1
10-12 ft.





Stantec Consulting Ltd.

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

Specimen Details

Project Name	Thurber Engineering, File# 23411
Project Location	Hwy 17, Laronde
Borehole	BH19-2A
Sample No.	TW 1
Depth	10-12 ft.
Sample Date	August 13, 2019
Test Number	Three
Technician Name	Daniel Boateng

Soil Description & Classification

Clay and silt, grey, varved, moist - CH	
Specific Gravity of Solids	2.715
Liquid Limit %	56
Plastic Limit %	21
Plasticity Index %	35
Average water content of trimmings %	46
Additional Notes (information source, occurrence and size of large isolated particles etc.)	

Initial Specimen Conditions

Height	mm	19.01
Diameter	mm	50.02
Area	mm ²	1965
Volume	mm ³	37356
Mass	g	65.30
Dry Mass	g	44.72
Density	Mg/m ³	1.748
Dry Density	Mg/m ³	1.197
Water Content	%	46.02
Degree of Saturation	%	98.5
Height of Solids	mm	8.38
Initial Void Ratio		1.268

Final Specimen Conditions

Water Content	%	29.72
Final Void Ratio		0.788
Differential Height	mm	14.99

One-Dimensional Consolidation Test using Incremental Loading

ASTM D2435/D2435M - 11

Specimen Details

Project Name	Thurber Engineering, File# 23411
Project Location	Hwy 17, Laronde
Borehole	BH19-2A
Sample No.	TW 1
Depth	10-12 ft.
Sample Date	August 13, 2019
Test Number	Three
Technician Name	Daniel Boateng

Test Procedure

Date Started	September 9, 2019
Date Finished	November 8, 2019
Machine Number	Frame A
Cell Number	A
Ring Number	A
Trimming Procedure	Turntable/Cutting shoe
Moisture Condition	Inundated
Axial Stress at Inundation kPa	5
Water Used	Deaired 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	19.0100	0.00	1.268
1	1440.0	5	0.0190	18.9910	0.10	1.266
2	1440.0	10	0.0500	18.9600	0.26	1.262
3	1440.0	20	0.1250	18.8850	0.66	1.253
4	1440.0	43	0.2840	18.7260	1.49	1.234
5	4320.0	5	0.1590	18.8510	0.84	1.249
6	1440.0	43	0.3000	18.7100	1.58	1.232
7	1440.0	84	0.5090	18.5010	2.68	1.207
8	60480.0	160	1.2530	17.7570	6.59	1.118
9	1440.0	321	2.7220	16.2880	14.32	0.943
10	1440.0	636	4.1530	14.8570	21.85	0.772
11	4320.0	1281	5.2190	13.7910	27.45	0.645
12	1440.0	321	4.9850	14.0250	26.22	0.673
13	1440.0	84	4.6390	14.3710	24.40	0.714
14	1440.0	20	4.2690	14.7410	22.46	0.759
15	1440.0	5	4.0240	14.9860	21.17	0.788



Stantec Consulting Ltd.

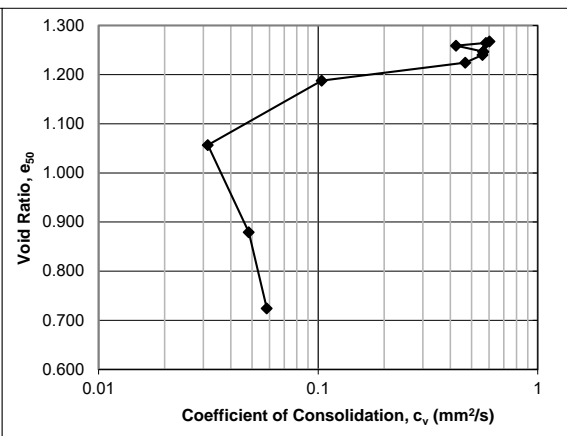
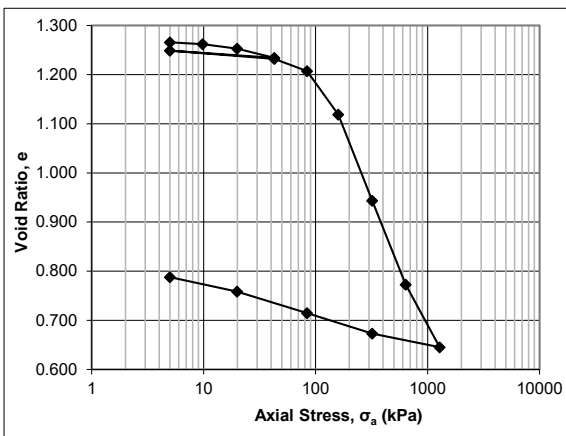
One-Dimensional Consolidation Test using Incremental Loading ASTM D2435/D2435M - 11

Specimen Details

Job Ref.	Thurber Engineering, File# 23411
Job Location	Hwy 17, Laronde
Borehole	BH19-2A
Sample No.	TW 1
Depth	10-12 ft.
Sample Date	August 13, 2019
Test Number	Three
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	0.0039	19.0061	0.02	1.267			127	6.03E-01
2	7	0.0291	18.9809	0.15	1.264			131	5.82E-01
3	15	0.0750	18.9350	0.39	1.259			179	4.25E-01
4	31	0.1784	18.8316	0.94	1.247			133	5.66E-01
5	24	0.2328	18.7772	1.22	1.240				
6	24	0.2331	18.7769	1.23	1.240			133	5.61E-01
7	63	0.3648	18.6452	1.92	1.224			157	4.68E-01
8	122	0.6702	18.3398	3.53	1.188			687	1.04E-01
9	240	1.7696	17.2404	9.31	1.057			2003	3.15E-02
10	479	3.2566	15.7534	17.13	0.879			1090	4.83E-02
11	959	4.5548	14.4552	23.96	0.725			760	5.83E-02
12	801	5.0965	13.9135	26.81	0.660				
13	202	4.8383	14.1717	25.45	0.691				
14	52	4.5156	14.4944	23.75	0.729				
15	12	4.2267	14.7833	22.23	0.764				





Project No.: 122410864

Project Name: Thurber Engineering, File# 23411

Photo Log

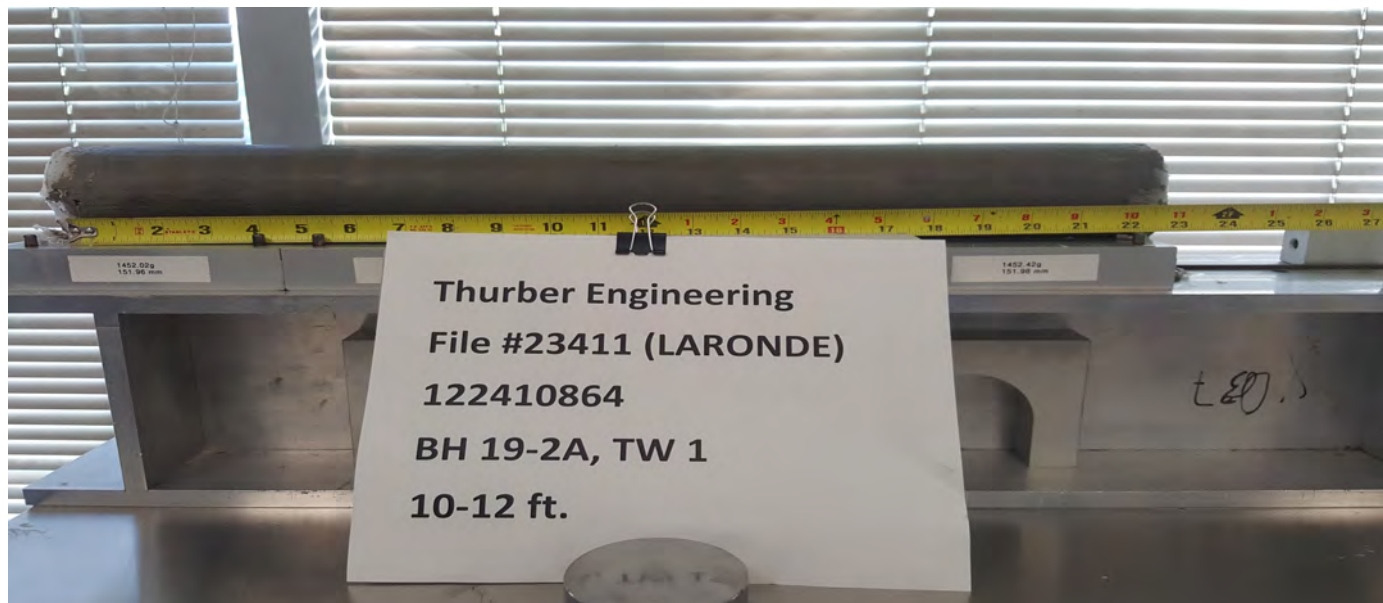


Photo No.:

1

Borehole: BH19-2A TW-1

Depth: 10 – 12 ft



Photo No.:

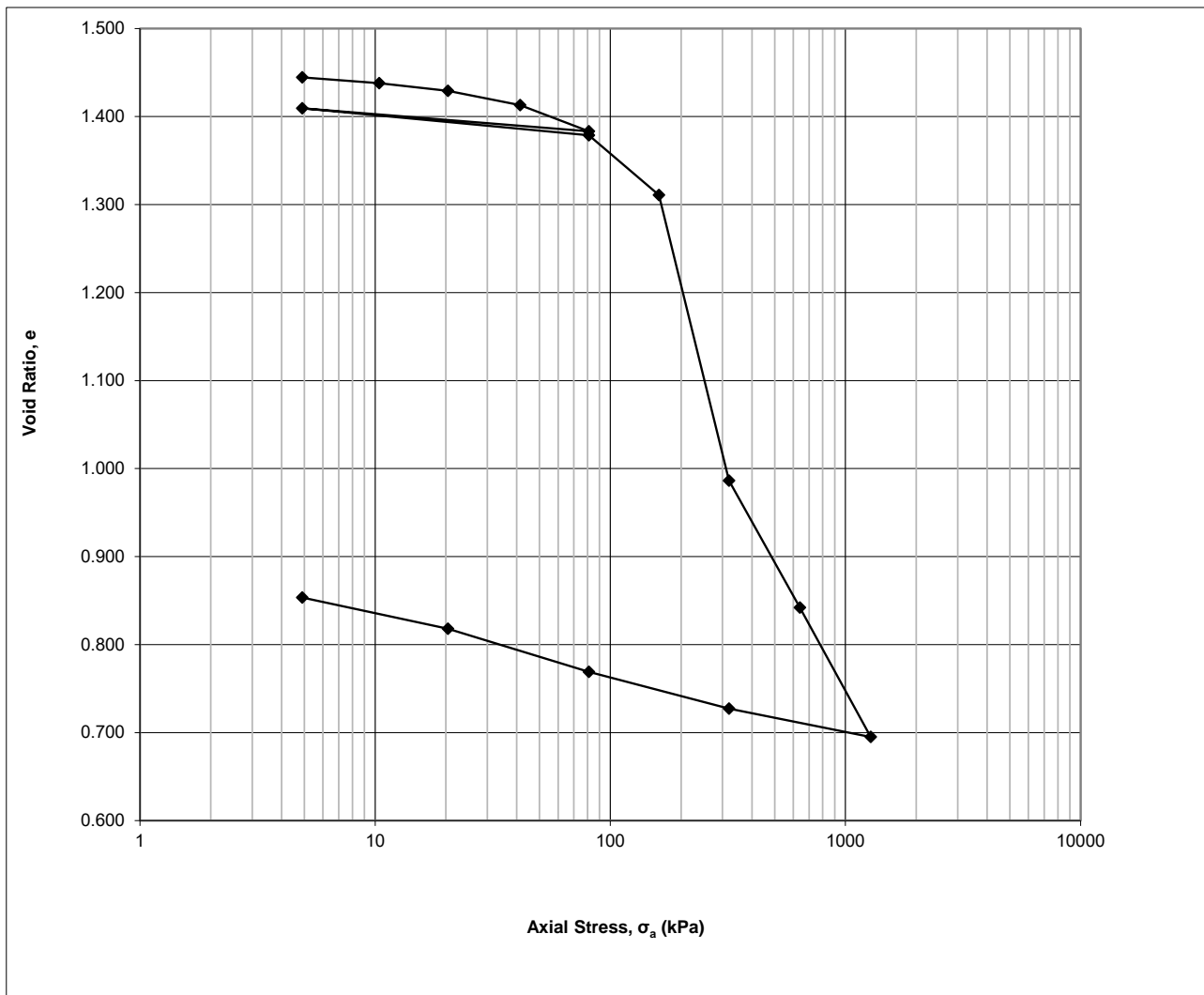
2

Borehole: BH19-2A TW-1

Depth: 10 – 12 ft

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

Thurber Engineering, File# 23411
122410864
BH19-2A
TW 5
18-20 ft.





Stantec Consulting Ltd.

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

Specimen Details

Project Name	Thurber Engineering, File# 23411
Project Location	Hwy 17, Laronde
Borehole	BH19-2A
Sample No.	TW 5
Depth	18-20 ft.
Sample Date	August 13, 2019
Test Number	Four
Technician Name	Daniel Boateng

Soil Description & Classification

Clay and silt, grey, varved, moist - CI	
Specific Gravity of Solids	2.732
Liquid Limit %	47
Plastic Limit %	21
Plasticity Index %	26
Average water content of trimmings %	51
Additional Notes (information source, occurrence and size of large isolated particles etc.)	

Initial Specimen Conditions

Height	mm	19.03
Diameter	mm	50.86
Area	mm ²	2032
Volume	mm ³	38662
Mass	g	65.21
Dry Mass	g	43.12
Density	Mg/m ³	1.687
Dry Density	Mg/m ³	1.115
Water Content	%	51.23
Degree of Saturation	%	96.6
Height of Solids	mm	7.77
Initial Void Ratio		1.449

Final Specimen Conditions

Water Content	%	30.66
Final Void Ratio		0.853
Differential Height	mm	14.40

One-Dimensional Consolidation Test using Incremental Loading

ASTM D2435/D2435M - 11

Specimen Details

Project Name	Thurber Engineering, File# 23411
Project Location	Hwy 17, Laronde
Borehole	BH19-2A
Sample No.	TW 5
Depth	18-20 ft.
Sample Date	August 13, 2019
Test Number	Four
Technician Name	Daniel Boateng

Test Procedure

Date Started	September 9, 2019
Date Finished	November 7, 2019
Machine Number	Frame B
Cell Number	B
Ring Number	B
Trimming Procedure	Turntable/Cutting shoe
Moisture Condition	Inundated
Axial Stress at Inundation kPa	5
Water Used	Deaired 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	19.0300	0.00	1.449
1	1440.0	5	0.0390	18.9910	0.20	1.445
2	1440.0	10	0.0890	18.9410	0.47	1.438
3	1440.0	20	0.1570	18.8730	0.83	1.429
4	1440.0	41	0.2840	18.7460	1.49	1.413
5	4320.0	81	0.5160	18.5140	2.71	1.383
6	1440.0	5	0.3130	18.7170	1.64	1.409
7	1440.0	81	0.5510	18.4790	2.90	1.379
8	1440.0	161	1.0770	17.9530	5.66	1.311
9	59040.0	320	3.6000	15.4300	18.92	0.986
10	1440.0	640	4.7190	14.3110	24.80	0.842
11	1440.0	1280	5.8620	13.1680	30.80	0.695
12	4320.0	320	5.6110	13.4190	29.49	0.727
13	1440.0	81	5.2860	13.7440	27.78	0.769
14	1440.0	20	4.9070	14.1230	25.79	0.818
15	1440.0	5	4.6320	14.3980	24.34	0.853



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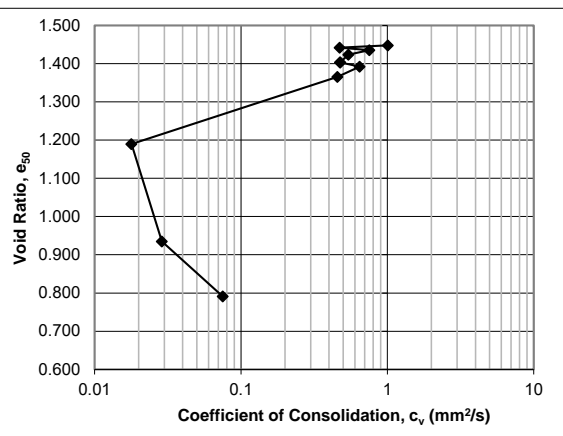
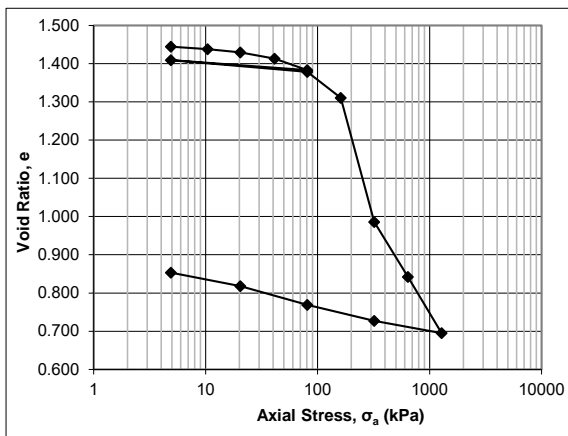
One-Dimensional Consolidation Test using Incremental Loading ASTM D2435/D2435M - 11

Specimen Details

Job Ref.	Thurber Engineering, File# 23411
Job Location	Hwy 17, Laronde
Borehole	BH19-2A
Sample No.	TW 5
Depth	18-20 ft.
Sample Date	August 13, 2019
Test Number	Four
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	2	0.0120	19.0180	0.06	1.448			76	1.01E+00
2	8	0.0572	18.9728	0.30	1.442			161	4.73E-01
3	15	0.1070	18.9230	0.56	1.436			101	7.54E-01
4	31	0.2012	18.8288	1.06	1.424			139	5.42E-01
5	61	0.3574	18.6726	1.88	1.404			155	4.76E-01
6	43	0.4136	18.6164	2.17	1.396				
7	43	0.4439	18.5861	2.33	1.392			113	6.48E-01
8	121	0.6548	18.3752	3.44	1.365			157	4.56E-01
9	240	2.0198	17.0102	10.61	1.190			3430	1.79E-02
10	480	3.9972	15.0328	21.00	0.935			1661	2.88E-02
11	960	5.1168	13.9132	26.89	0.791			545	7.53E-02
12	800	5.7221	13.3079	30.07	0.713				
13	201	5.5036	13.5264	28.92	0.741				
14	51	5.1500	13.8800	27.06	0.787				
15	13	4.8489	14.1811	25.48	0.825				





Project No.: 122410864

Project Name: Thurber Engineering, File# 23411

Photo Log

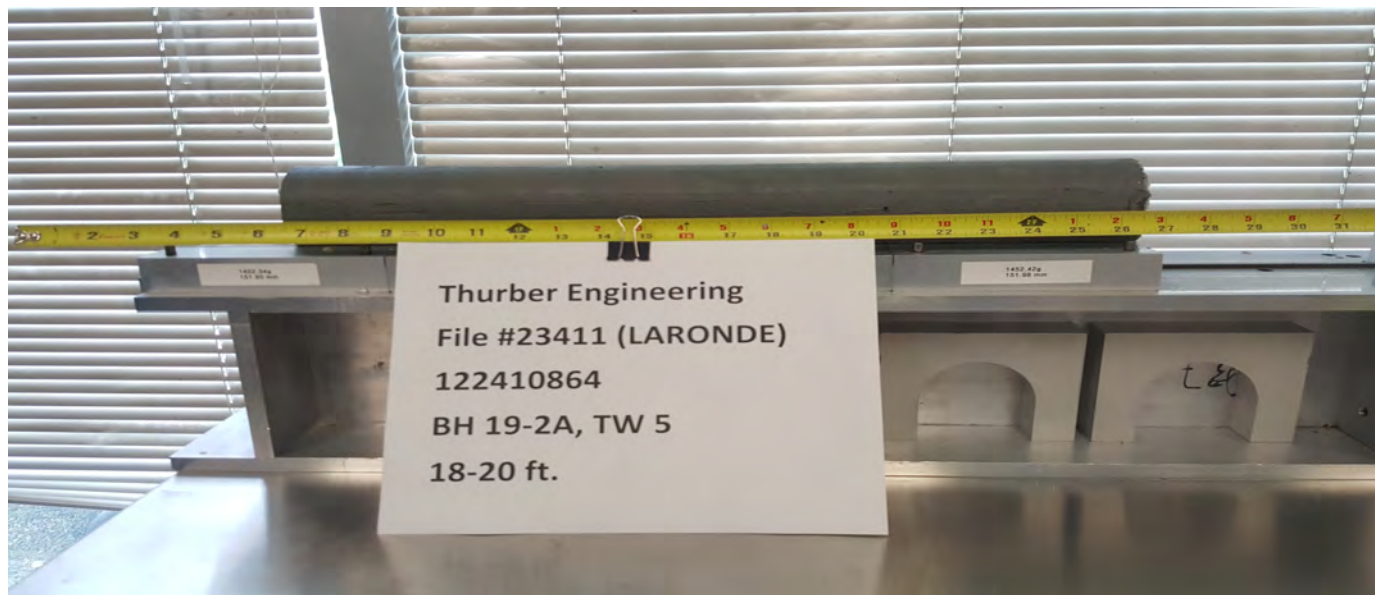


Photo No.:

1

Borehole: BH19-2A TW-5

Depth: 18 – 20 ft



Photo No.:

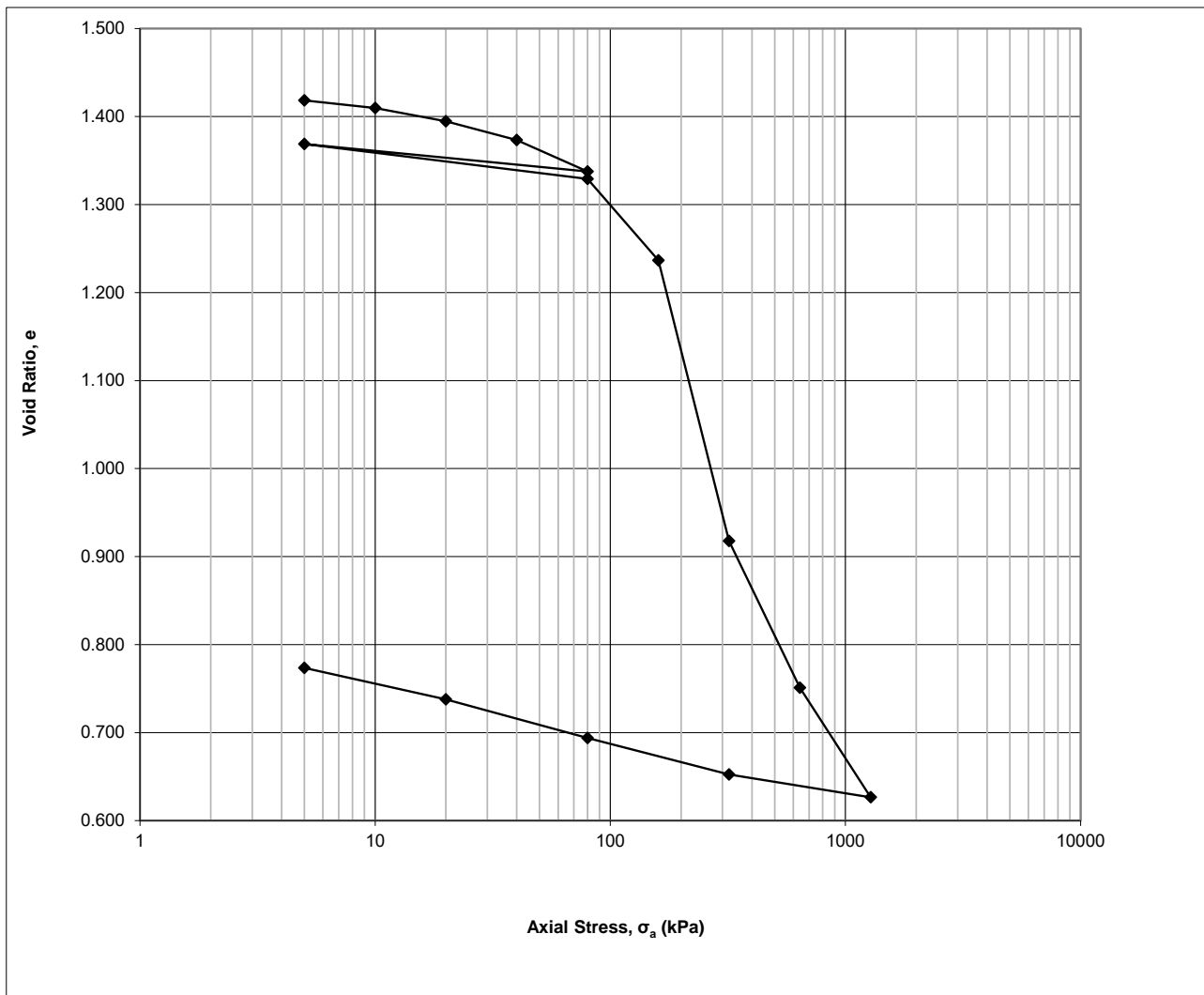
2

Borehole: BH19-2A TW-5

Depth: 18 – 20 ft

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

Thurber Engineering, File# 23411
122410864
BH19-2A
TW 8
24-26 ft.





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

Specimen Details

Project Name	Thurber Engineering, File# 23411
Project Location	Hwy 17, Laronde
Borehole	BH19-2A
Sample No.	TW 8
Depth	24-26 ft.
Sample Date	August 13, 2019
Test Number	Frame D
Technician Name	Daniel Boateng

Soil Description & Classification

Clay and silt, grey, varved, moist - CI	
Specific Gravity of Solids	2.725
Liquid Limit %	40
Plastic Limit %	20
Plasticity Index %	20
Average water content of trimmings %	47
Additional Notes (information source, occurrence and size of large isolated particles etc.)	
Loading Schedule Provided by Client	

Initial Specimen Conditions

Height	mm	20.00
Diameter	mm	50.00
Area	mm ²	1963
Volume	mm ³	39270
Mass	g	64.84
Dry Mass	g	44.06
Density	Mg/m ³	1.651
Dry Density	Mg/m ³	1.122
Water Content	%	47.16
Degree of Saturation	%	90.0
Height of Solids	mm	8.23
Initial Void Ratio		1.429

Final Specimen Conditions

Water Content	%	25.08
Final Void Ratio		0.774
Differential Height	mm	14.60

One-Dimensional Consolidation Test using Incremental Loading

ASTM D2435/D2435M - 11

Specimen Details

Project Name	Thurber Engineering, File# 23411
Project Location	Hwy 17, Laronde
Borehole	BH19-2A
Sample No.	TW 8
Depth	24-26 ft.
Sample Date	August 13, 2019
Test Number	Three
Technician Name	Daniel Boateng

Test Procedure

Date Started	September 6, 2019
Date Finished	September 23, 2019
Machine Number	Frame D
Cell Number	D
Ring Number	D
Trimming Procedure	Turntable/Cutting Ring
Moisture Condition	Inundated
Axial Stress at Inundation kPa	5
Water Used	Deaired 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	1.429
1	1440.0	5	0.0853	19.9147	0.43	1.418
2	1440.0	10	0.1566	19.8434	0.78	1.410
3	1440.0	20	0.2813	19.7187	1.41	1.395
4	1440.0	40	0.4571	19.5429	2.29	1.373
5	1440.0	80	0.7499	19.2501	3.75	1.338
6	1440.0	5	0.4940	19.5060	2.47	1.369
7	1440.0	80	0.8198	19.1802	4.10	1.329
8	1440.0	160	1.5824	18.4176	7.91	1.237
9	1440.0	320	4.2080	15.7920	21.04	0.918
10	1440.0	640	5.5824	14.4176	27.91	0.751
11	1440.0	1280	6.6069	13.3931	33.03	0.626
12	1440.0	320	6.3919	13.6081	31.96	0.653
13	1440.0	80	6.0524	13.9476	30.26	0.694
14	1440.0	20	5.6907	14.3093	28.45	0.738
15	1440.0	5	5.3954	14.6046	26.98	0.774

One-Dimensional Consolidation Test using Incremental Loading

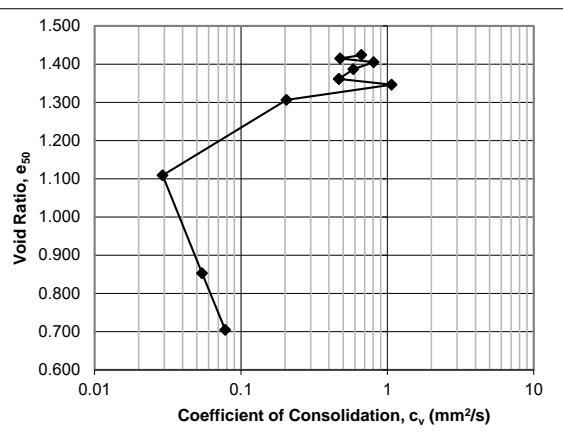
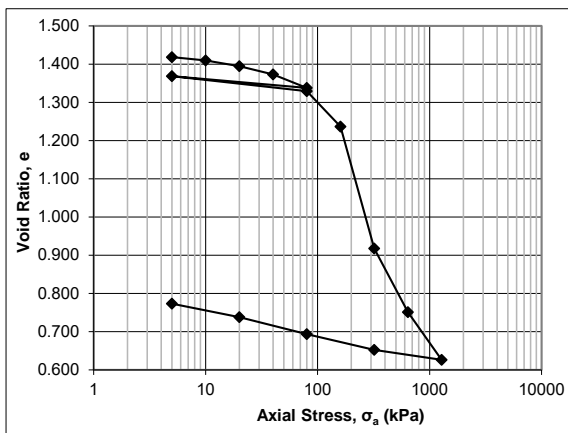
ASTM D2435/D2435M - 11

Specimen Details

Job Ref.	Thurber Engineering, File# 23411
Job Location	Hwy 17, Laronde
Borehole	BH19-2A
Sample No.	TW 8
Depth	24-26 ft.
Sample Date	August 13, 2019
Test Number	Three
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	0.0414	19.9586	0.21	1.424			127	6.64E-01
2	8	0.1117	19.8883	0.56	1.415			176	4.76E-01
3	15	0.1969	19.8031	0.98	1.405			103	8.05E-01
4	30	0.3439	19.6561	1.72	1.387			139	5.88E-01
5	60	0.5533	19.4467	2.77	1.362			171	4.68E-01
6	43	0.5970	19.4030	2.99	1.356				
7	43	0.6776	19.3224	3.39	1.346			74	1.07E+00
8	120	1.0067	18.9933	5.03	1.306			374	2.04E-01
9	240	2.6329	17.3671	13.16	1.109			2192	2.92E-02
10	480	4.7393	15.2607	23.70	0.853			911	5.42E-02
11	960	5.9649	14.0351	29.82	0.704			534	7.82E-02
12	800	6.4835	13.5165	32.42	0.641				
13	200	6.2338	13.7662	31.17	0.672				
14	50	5.9331	14.0669	29.67	0.708				
15	13	5.6028	14.3972	28.01	0.748				





Project No.: 122410864

Project Name: Thurber Engineering, File# 23411

Photo Log

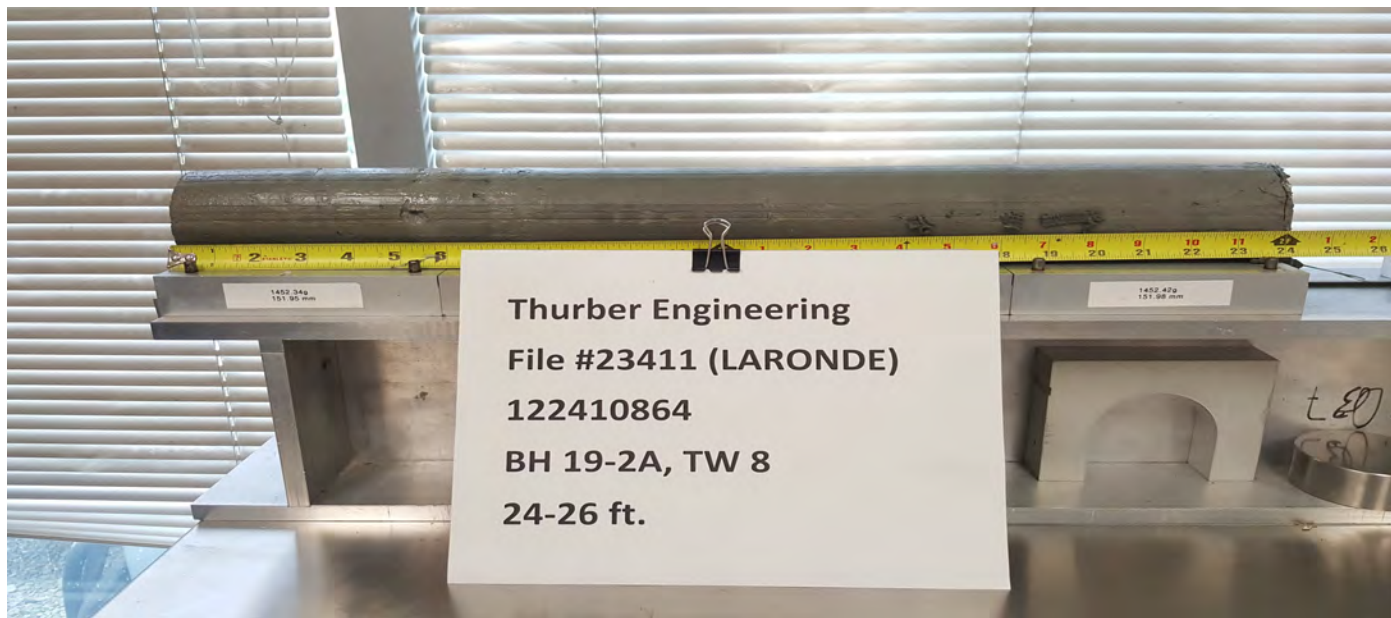


Photo No.:

1

Borehole: BH19-2A TW-8

Depth: 24 – 26 ft



Photo No.:

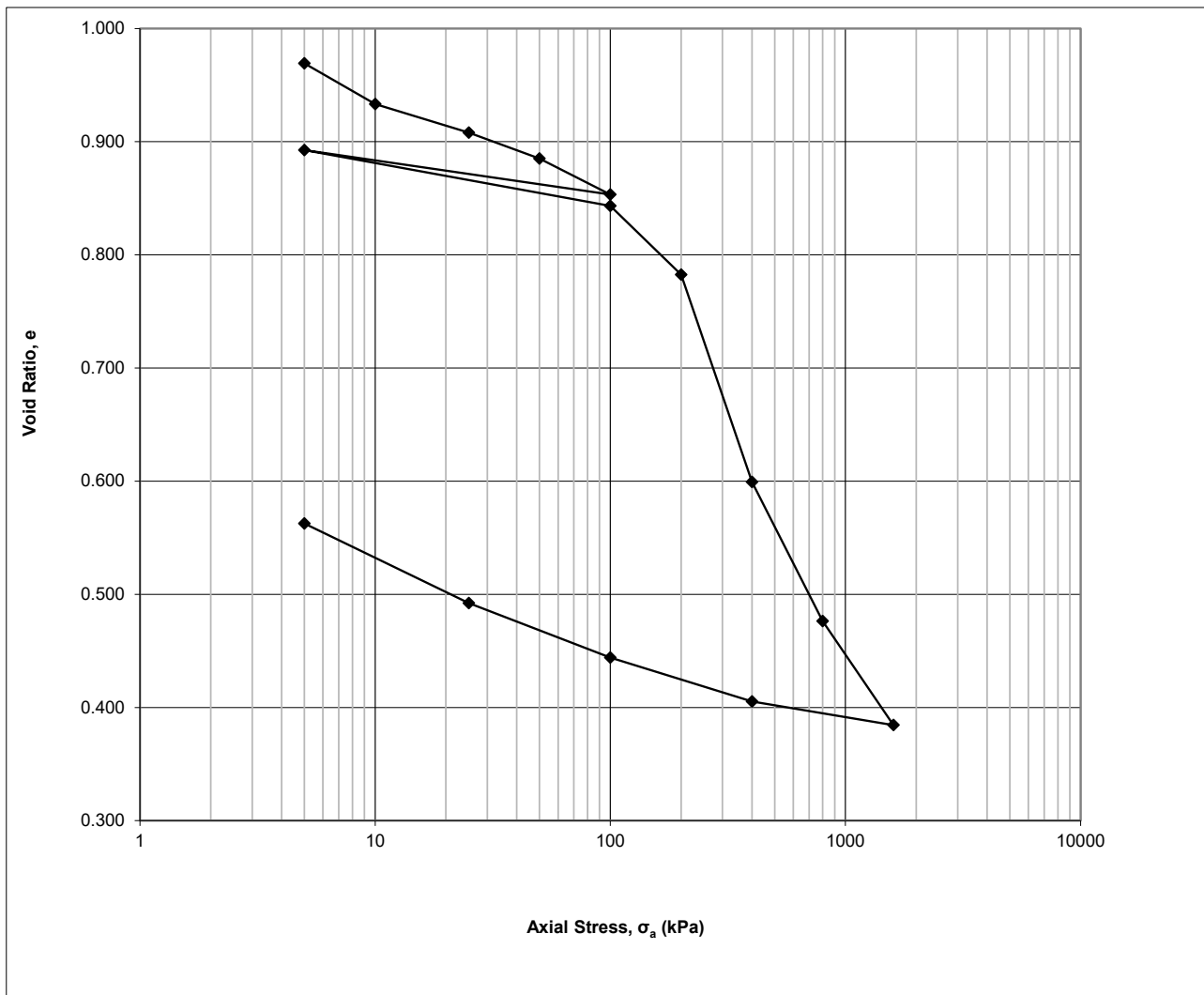
2

Borehole: BH19-2A TW-8

Depth: 24 – 26 ft

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

Thurber Engineering, File# 23411
122410864
BH19-2A
TW 15
40-42 ft.





Stantec Consulting Ltd.

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

Specimen Details

Project Name	Thurber Engineering, File# 23411
Project Location	Hwy 17, Laronde
Borehole	BH19-2A
Sample No.	TW 15
Depth	40-42 ft.
Sample Date	August 13, 2019
Test Number	Frame C
Technician Name	Daniel Boateng

Soil Description & Classification

Clay and silt, sand seams, grey, varved, moist - CI	
Specific Gravity of Solids	2.735
Liquid Limit %	48
Plastic Limit %	20
Plasticity Index %	28
Average water content of trimmings %	26
Additional Notes (information source, occurrence and size of large isolated particles etc.)	
Loading Schedule Provided by Client	

Initial Specimen Conditions

Height	mm	20.00
Diameter	mm	50.00
Area	mm ²	1963
Volume	mm ³	39270
Mass	g	67.73
Dry Mass	g	53.55
Density	Mg/m ³	1.725
Dry Density	Mg/m ³	1.364
Water Content	%	26.48
Degree of Saturation	%	72.0
Height of Solids	mm	9.97
Initial Void Ratio		1.006

Final Specimen Conditions

Water Content	%	14.64
Final Void Ratio		0.563
Differential Height	mm	15.58

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

Specimen Details

Project Name	Thurber Engineering, File# 23411
Project Location	Hwy 17, Laronde
Borehole	BH19-2A
Sample No.	TW 15
Depth	40-42 ft.
Sample Date	August 13, 2019
Test Number	Frame C
Technician Name	Daniel Boateng

Test Procedure

Date Started	September 6, 2019
Date Finished	September 23, 2019
Machine Number	Frame C
Cell Number	C
Ring Number	C
Trimming Procedure	Turntable/Cutting Ring
Moisture Condition	Inundated
Axial Stress at Inundation	5 kPa
Water Used	Deaired 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	1.006
1	1440.0	5	0.3631	19.6369	1.82	0.969
2	1440.0	10	0.7225	19.2775	3.61	0.933
3	1440.0	25	0.9725	19.0275	4.86	0.908
4	1440.0	50	1.2030	18.7970	6.02	0.885
5	1440.0	100	1.5197	18.4803	7.60	0.853
6	1440.0	5	1.1274	18.8726	5.64	0.893
7	1440.0	100	1.6184	18.3816	8.09	0.843
8	1440.0	200	2.2258	17.7742	11.13	0.782
9	1440.0	400	4.0529	15.9471	20.26	0.599
10	1440.0	800	5.2789	14.7211	26.39	0.476
11	1440.0	1600	6.1955	13.8045	30.98	0.384
12	1440.0	400	5.9859	14.0141	29.93	0.405
13	1440.0	100	5.5999	14.4001	28.00	0.444
14	1440.0	25	5.1195	14.8805	25.60	0.492
15	1440.0	5	4.4185	15.5815	22.09	0.563

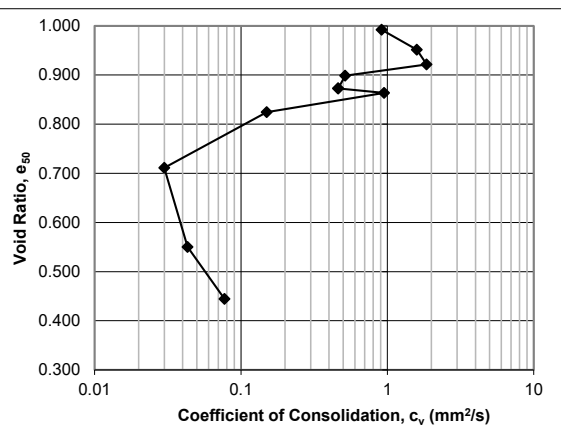
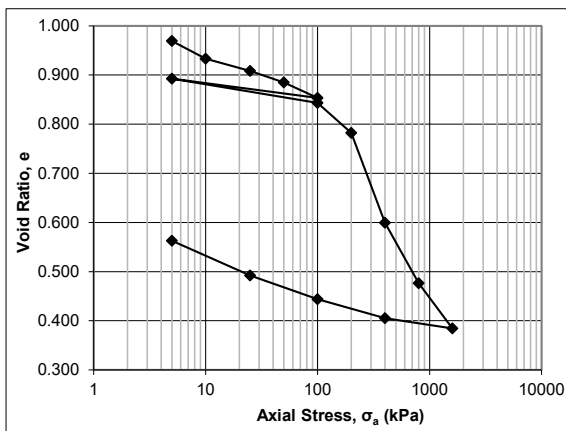
One-Dimensional Consolidation Test using Incremental Loading ASTM D2435/D2435M - 11

Specimen Details

Job Ref.	Thurber Engineering, File# 23411
Job Location	Hwy 17, Laronde
Borehole	BH19-2A
Sample No.	TW 15
Depth	40-42 ft.
Sample Date	August 13, 2019
Test Number	Frame C
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	0.1342	19.8658	0.67	0.992			91	9.15E-01
2	8	0.5412	19.4588	2.71	0.951			50	1.59E+00
3	18	0.8381	19.1619	4.19	0.922			42	1.86E+00
4	38	1.0667	18.9333	5.33	0.899			147	5.15E-01
5	75	1.3229	18.6771	6.61	0.873			160	4.62E-01
6	53	1.2907	18.7093	6.45	0.876				
7	53	1.4156	18.5844	7.08	0.864			77	9.52E-01
8	150	1.8076	18.1924	9.04	0.824			467	1.50E-01
9	300	2.9370	17.0630	14.69	0.711			2063	2.99E-02
10	600	4.5417	15.4583	22.71	0.550			1174	4.31E-02
11	1200	5.5977	14.4023	27.99	0.444			570	7.72E-02
12	1000	6.0715	13.9285	30.36	0.397				
13	250	5.8037	14.1963	29.02	0.424				
14	63	5.4190	14.5810	27.10	0.462				
15	15	4.7947	15.2053	23.97	0.525				





Project No.: 122410864

Project Name: Thurber Engineering, File# 25728

Photo Log

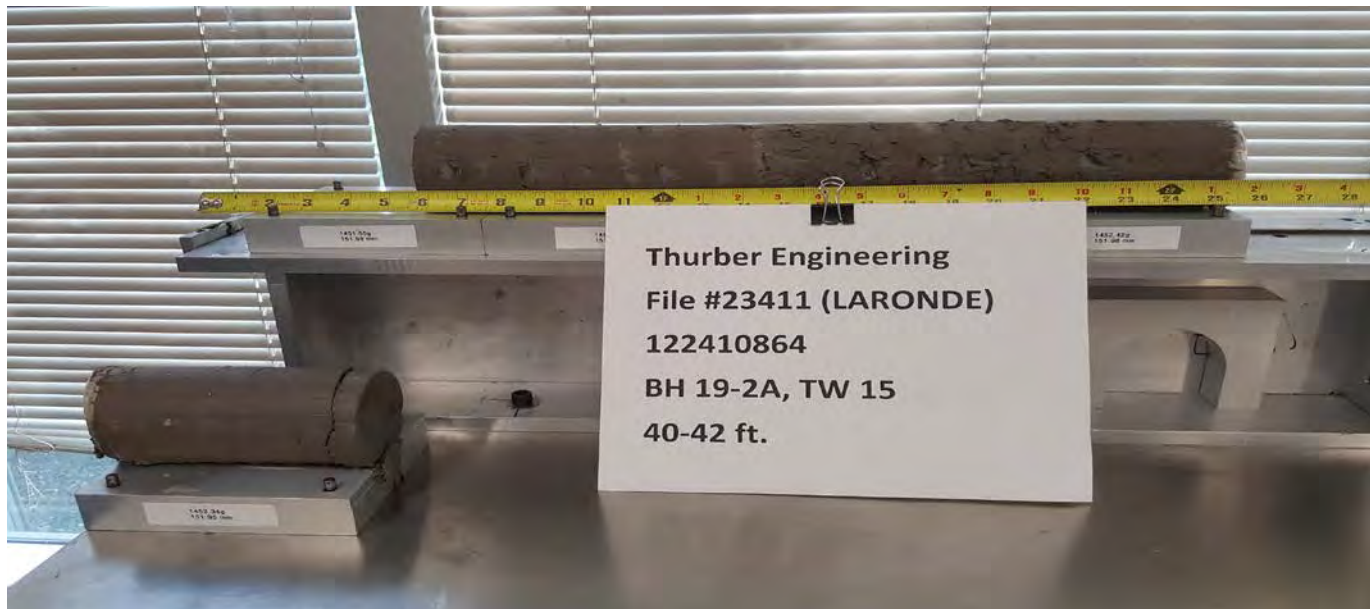


Photo No.:

1

Borehole: BH 19-2A TW-15

Depth: 40 – 42 ft

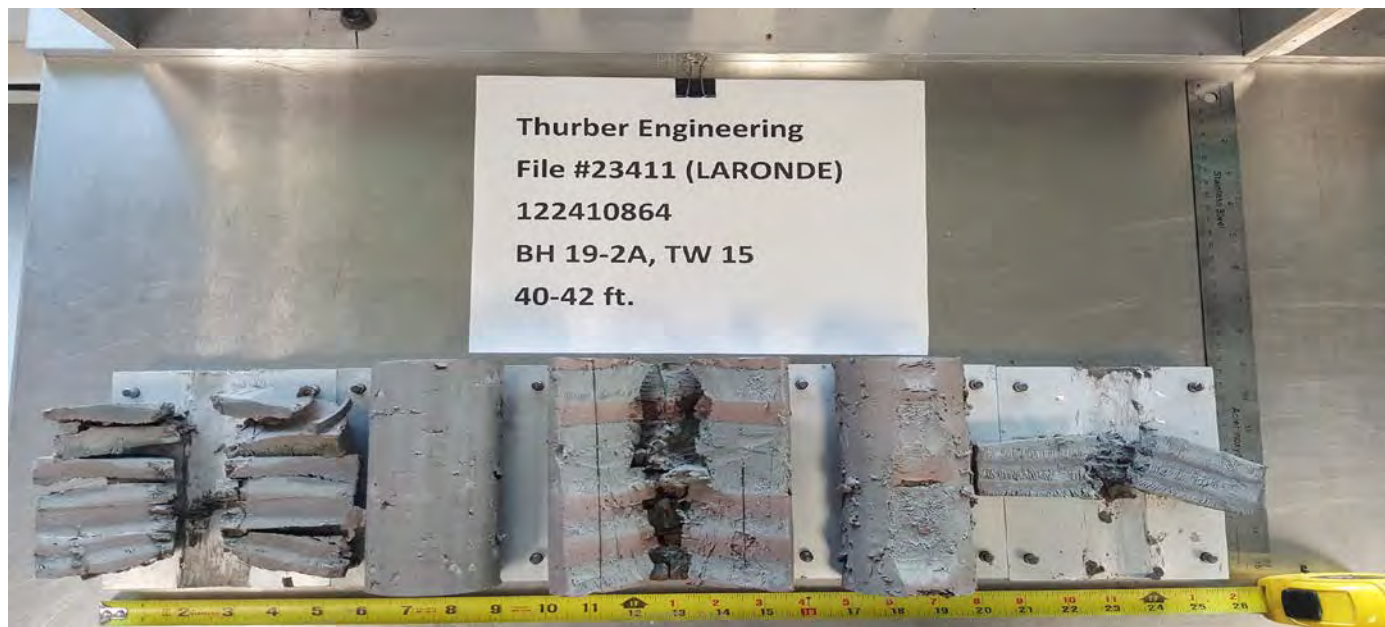


Photo No.:

2

Borehole: BH 19-2A TW-15

Depth: 40 – 42 ft



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 Undrained Triaxial Test Results for Laronde Creek Bridge Project,
Thurber Engineering Ltd., File #23411: BH 19-02A, TW 3, sampled on August 13, 2019**

This letter presents the results of consolidated undrained triaxial compression test carried out on the above referenced samples in accordance with ASTM D4767 - 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.

A handwritten signature in blue ink, appearing to read "Rajib Dey".

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

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Consolidated Undrained Triaxial Compression Test for Cohesive Soils
ASTM D4767 - 11

January 9, 2020
January 9, 2020

Date:
Date:

D. Boateng
R. Dey

Checked by:
Approved by:

C:\Users\dboateng\Desktop\Reports for Deanna\Lai
January 9, 2020

Filename:
Date:

Sample Details	Specimen 1	Specimen 2	Specimen 3
Project Name	Thurber Engineering, File# 23411		
Project Location	Laronde Creek Bridge		
Borehole	19-02A	19-02A	19-02A
Sample Number	TW 3	TW 3	TW 3
Depth	14-16 ft	14-16 ft	14-16 ft
Sample Date	August 13, 2019	August 13, 2019	August 13, 2019
Test Number	One	Two	Three
Technician Name	Daniel Boateng	Daniel Boateng	Daniel Boateng

Soil Description & Classification

Clay and silt, grey, varved, moist - CH			
Specific Gravity of Solids	2.731	2.731	2.731
Liquid Limit %	59.2	59.2	59.2
Plastic Limit %	22.3	22.3	22.3
Plasticity Index %	36.8	36.8	36.8
Additional Notes (unusual conditions or other information necessary to interpret the test results):			
Effective consolidation stress assigned by client			
Specimen is varved with variable moisture contents across length of shelly tube			
Specimen 2 and 3 consolidated in stages as per ASTM			
Departures from the test procedure outlined in ASTM D4767-11:			

Initial Specimen Conditions

Height	mm	151.4	142.2	142.2
Diameter	mm	70.0	70.0	70.0
Dry Unit Weight	Mg/m ³	0.94	1.00	0.93
Void Ratio		1.90	1.72	1.94
Water Content	%	70.56	65.84	76.88
Degree of Saturation	%	101.6	104.7	108.3
Method used for obtaining water content		Cuttings	Cuttings	Cuttings

Membrane Properties

Young's Modulus	kPa	1400	1400	1400
Thickness	mm	0.3	0.3	0.3

Filter-Paper Strip Properties

Load carried per unit length	kN/mm	0.00019	0.00019	0.00019
Specimen perimeter covered by strips	mm	220	220	220

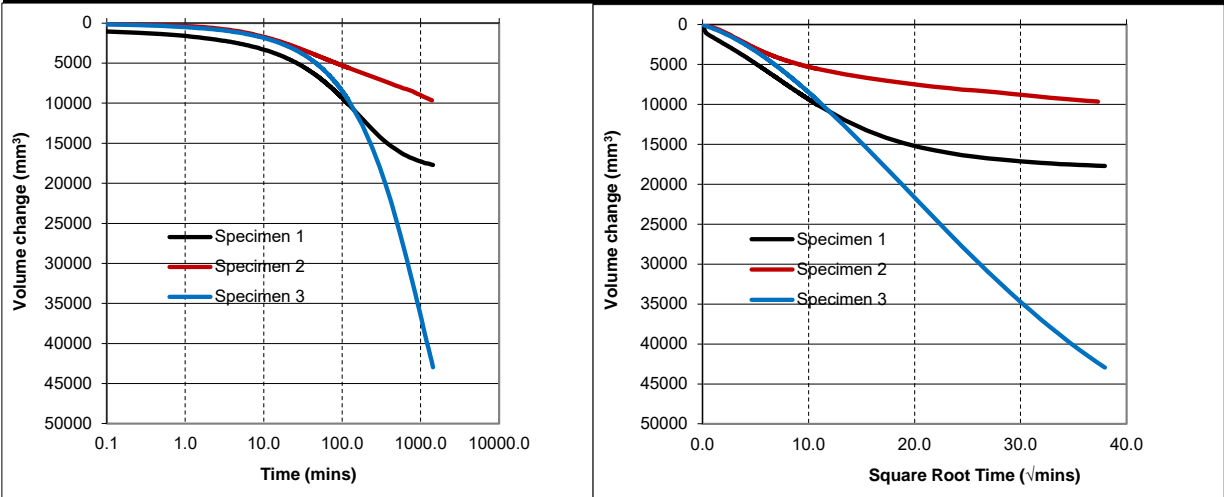
Consolidated Undrained Triaxial Compression Test for Cohesive Soils
ASTM D4767 - 11

Sample Details	Specimen 1	Specimen 2	Specimen 3
Project Name	Thurber Engineering, File# 23411		
Project Location	Laronde Creek Bridge		
Borehole	19-02A	19-02A	19-02A
Sample Number	TW 3	TW 3	TW 3
Depth	14-16 ft	14-16 ft	14-16 ft
Sample Date	August 13, 2019	August 13, 2019	August 13, 2019
Test Number	One	Two	Three
Technician Name	Daniel Boateng	Daniel Boateng	Daniel Boateng

Test Setup			
Date Started	December 24, 2019	December 28, 2019	January 3, 2020
Date Finished	December 28, 2019	January 3, 2020	January 8, 2020
Top Drain Used	Yes	Yes	Yes
Base Drain Used	Yes	Yes	Yes
Side Drains Used (Filter-Paper Strips)	Yes	Yes	Yes
Pressure System Number	21705	21705	21705
Cell Number	21969	21969	21969

Measurement of Pore Pressure Parameter			
Cell Pressure Increment kPa	10.0	10.0	10.0
Cell Pressure at B determination kPa	210.0	210.0	210.0
Back Pressure at B determination kPa	190.0	190.0	190.0
Pore Pressure at B determination kPa	201.7	201.9	201.6
Pore Pressure Parameter B at 2 min	1.0	1.0	1.0
Method used for specimen saturation	Wet	Wet	Wet

End of Consolidation Stage			
Consolidation Stress kPa	50.0	150.0	250.0
Effective Consolidation Stress kPa	48.7	148.2	213.6
Total Back Pressure kPa	200.0	200.0	200.0
Time to 50 % primary consolidation min	112	104	467
Interpretation method used for t ₅₀	1	1	1
Dry Unit Weight Mg/m ³	0.95	1.04	1.00
Void Ratio	1.87	1.62	1.73
Water Content %	68.44	57.28	63.68
Degree of Saturation %	100.0	96.7	100.3
Cross-sectional Area, A _c mm ²	3783.0	3705.5	3571.2
Method used to determine Area, A _c	A	A	A

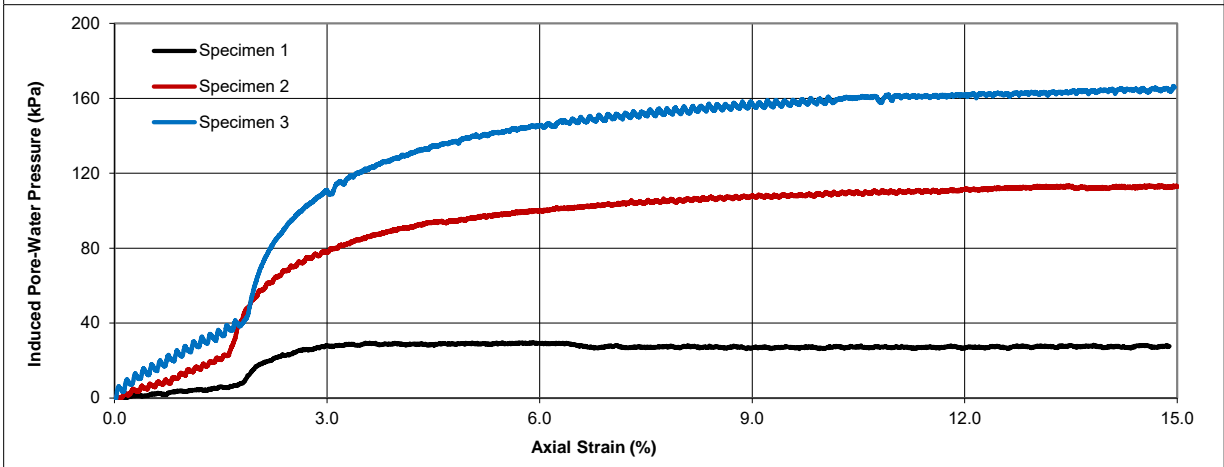
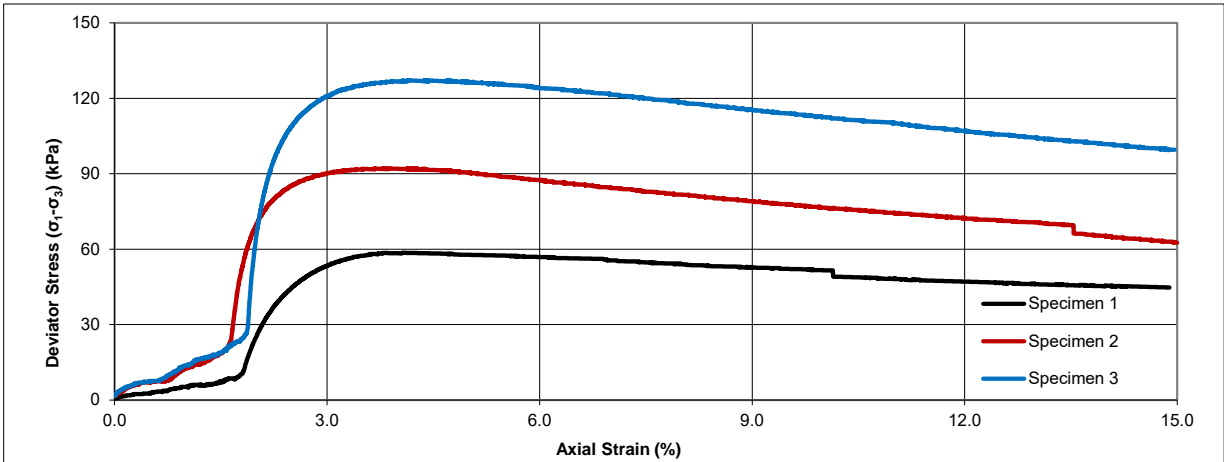


Consolidated Undrained Triaxial Compression Test for Cohesive Soils
ASTM D4767 - 11

Sample Details	Specimen 1	Specimen 2	Specimen 3
Project Name	Thurber Engineering, File# 23411		
Project Location	Laronde Creek Bridge		
Borehole	19-02A	19-02A	19-02A
Sample Number	TW 3	TW 3	TW 3
Depth	14-16 ft	14-16 ft	14-16 ft
Sample Date	August 13, 2019	August 13, 2019	August 13, 2019
Test Number	One	Two	Three
Technician Name	Daniel Boateng	Daniel Boateng	Daniel Boateng

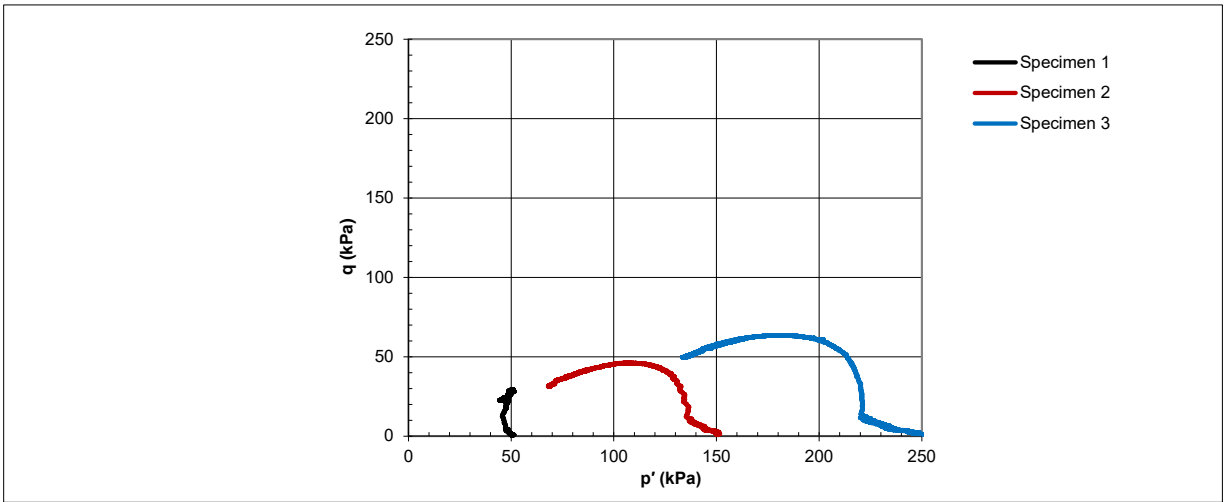
Shearing Stage			
Failure Criterion	15% Axial Strain	15% Axial Strain	15% Axial Strain
Rate of axial strain %/min	0.017	0.006	0.006

Response at Failure			
Deviator Stress ($\sigma_1 - \sigma_3$) kPa	58.6	92.2	127.2
Axial Strain %	4.06	3.78	4.13
Max Effective Principal Stress Ratio (σ'_1 / σ'_3)	3.8	2.5	2.1
Effective Major Principal Stress kPa	79.6	154.2	247.2
Effective Minor Principal Stress kPa	21.0	62.0	120.0
Values corrected for membrane?	No	No	No
Values corrected for filter-paper strips?	No	No	No

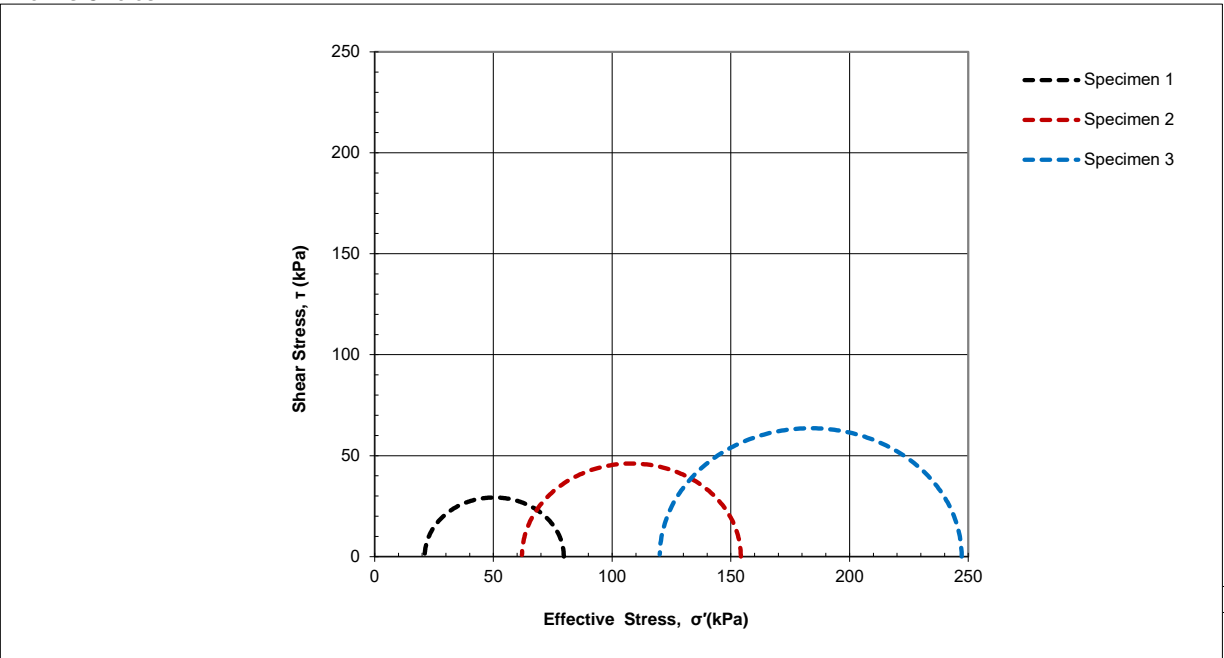


Consolidated Undrained Triaxial Compression Test for Cohesive Soils
ASTM D4767 - 11

Sample Details	Specimen 1	Specimen 2	Specimen 3
Project Name	Thurber Engineering, File# 23411		
Project Location	Laronde Creek Bridge		
Borehole	19-02A	19-02A	19-02A
Sample Number	TW 3	TW 3	TW 3
Depth	14-16 ft	14-16 ft	14-16 ft
Sample Date	August 13, 2019	August 13, 2019	August 13, 2019
Test Number	One	Two	Three
Technician Name	Daniel Boateng	Daniel Boateng	Daniel Boateng



Mohr's Circles



Consolidated Undrained Triaxial Compression Test for Cohesive Soils
ASTM D4767 - 11

Sample Details	Specimen 1	Specimen 2	Specimen 3
Project Name	Thurber Engineering, File# 23411		
Project Location	Laronde Creek Bridge		
Borehole	19-02A	19-02A	19-02A
Sample Number	TW 3	TW 3	TW 3
Depth	14-16 ft	14-16 ft	14-16 ft
Sample Date	August 13, 2019	August 13, 2019	August 13, 2019
Test Number	One	Two	Three
Technician Name	Daniel Boateng	Daniel Boateng	Daniel Boateng

Failure Photographs of the Specimens



Specimen 1



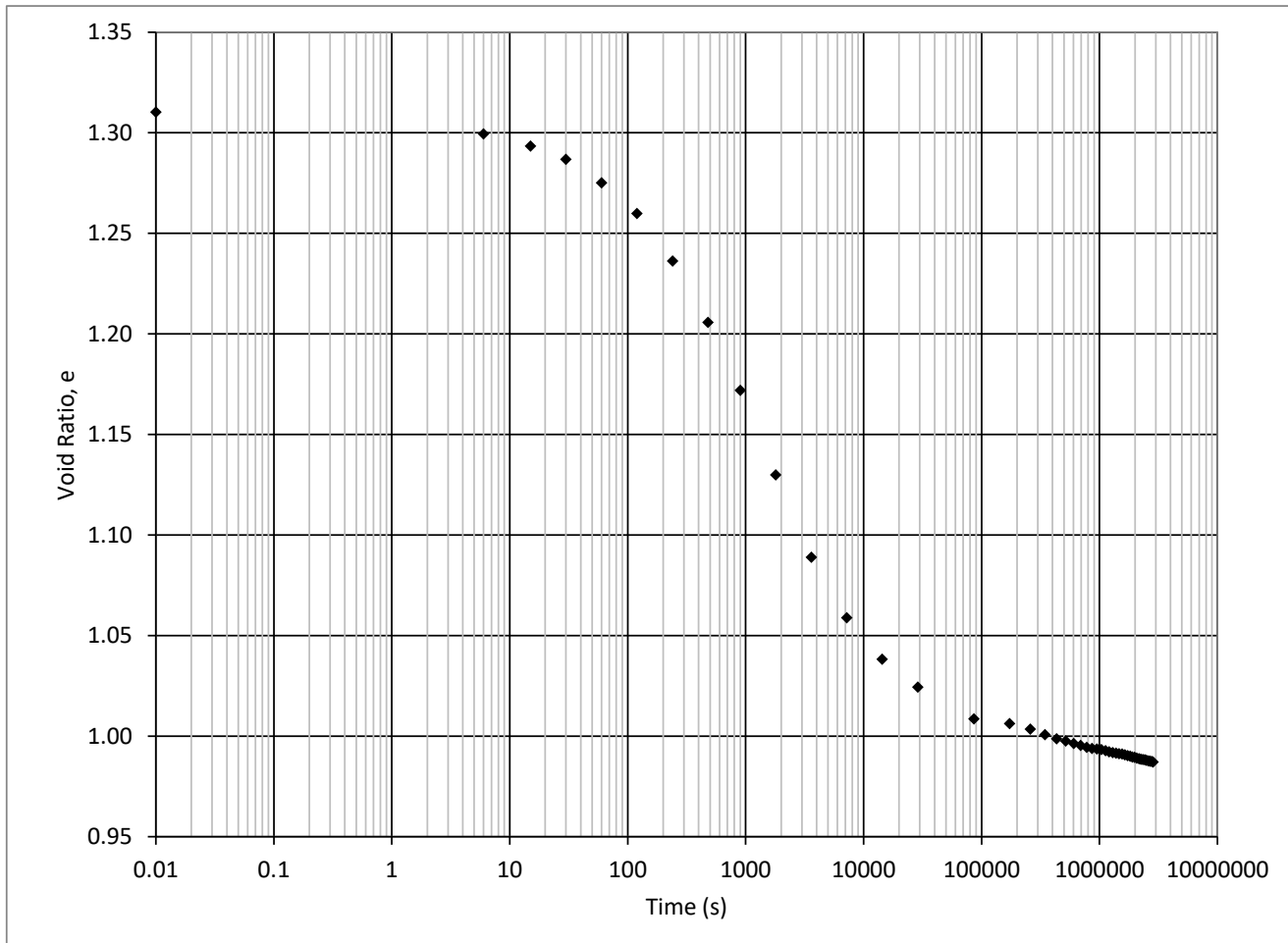
Specimen 2



Specimen 3

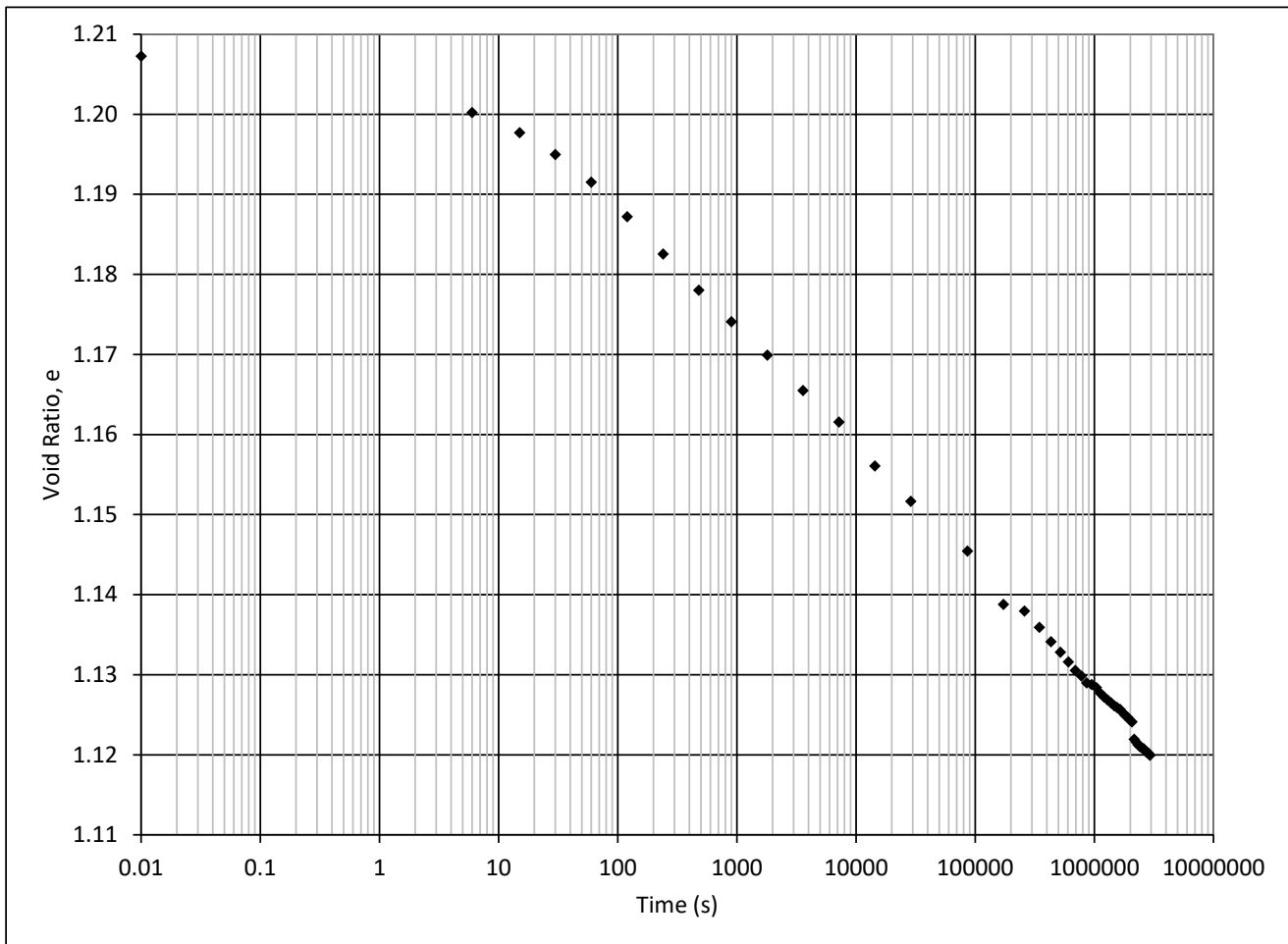
Project
Project No.
Borehole No.
Sample No.
Sample Depth
Axial Stress

Thurber Engineering, File# 23411
122410864
BH19-2A
TW 5
18-20 ft.
320.1 kPa



Project
Project No.
Borehole No.
Sample No.
Sample Depth
Axial Stress

Thurber Engineering, File# 23411
122410864
BH19-2A
TW 1
10-12 ft.
159.5 kPa



September 11, 2019

CRS Test

BH 19-2A

Sample #1

Depth: 16-18'

Strain rate started at 3%/hr

Saturation pressure 500kPa Saturate over 120min

Consolidation pressure 500kPa

Maximum Stress 640kPa

Creep 1440 min

Height of sample = 25.38mm

Sample diameter = 49.7mm

Sample weight = 77.79g

Initial moisture content = 64.29%

Final moisture content = 43.21%

Tested by Melodie Richards

October 2, 2019

CRS Test

BH 19-2A

Sample #2

Depth: 16-18'

Strain rate started at 0.1%/hr

Saturation pressure 500kPa Saturate over 120min

Consolidation pressure 500kPa

Maximum Stress 400kPa

Creep 1440 min

Height of sample = 25.38mm

Sample diameter = 49.7mm

Sample weight = 78.2g

Initial moisture content = 57.44%

Final moisture content = 44.60%

Tested by Melodie Richards

September 23, 2019

CRS Test

BH 19-2A

Sample #3

Depth: 20-22'

Strain rate started at 1%/hr

Saturation pressure 500kPa Saturate over 120min

Consolidation pressure 500kPa

Maximum Stress 640kPa

Creep 1440 min

Height of sample = 25.38mm

Sample diameter = 49.7mm

Sample weight = 89.23g

Initial moisture content = 46.46%

Final moisture content = 31.41%

Tested by Melodie Richards

September 23, 2019

CRS Test

BH 19-2A

Sample #4

Depth: 20-22'

Strain rate started at 0.3%/hr

Saturation pressure 500kPa Saturate over 120min

Consolidation pressure 500kPa

Maximum Stress 640kPa

Creep 1440 min

Height of sample = 25.38mm

Sample diameter = 49.7mm

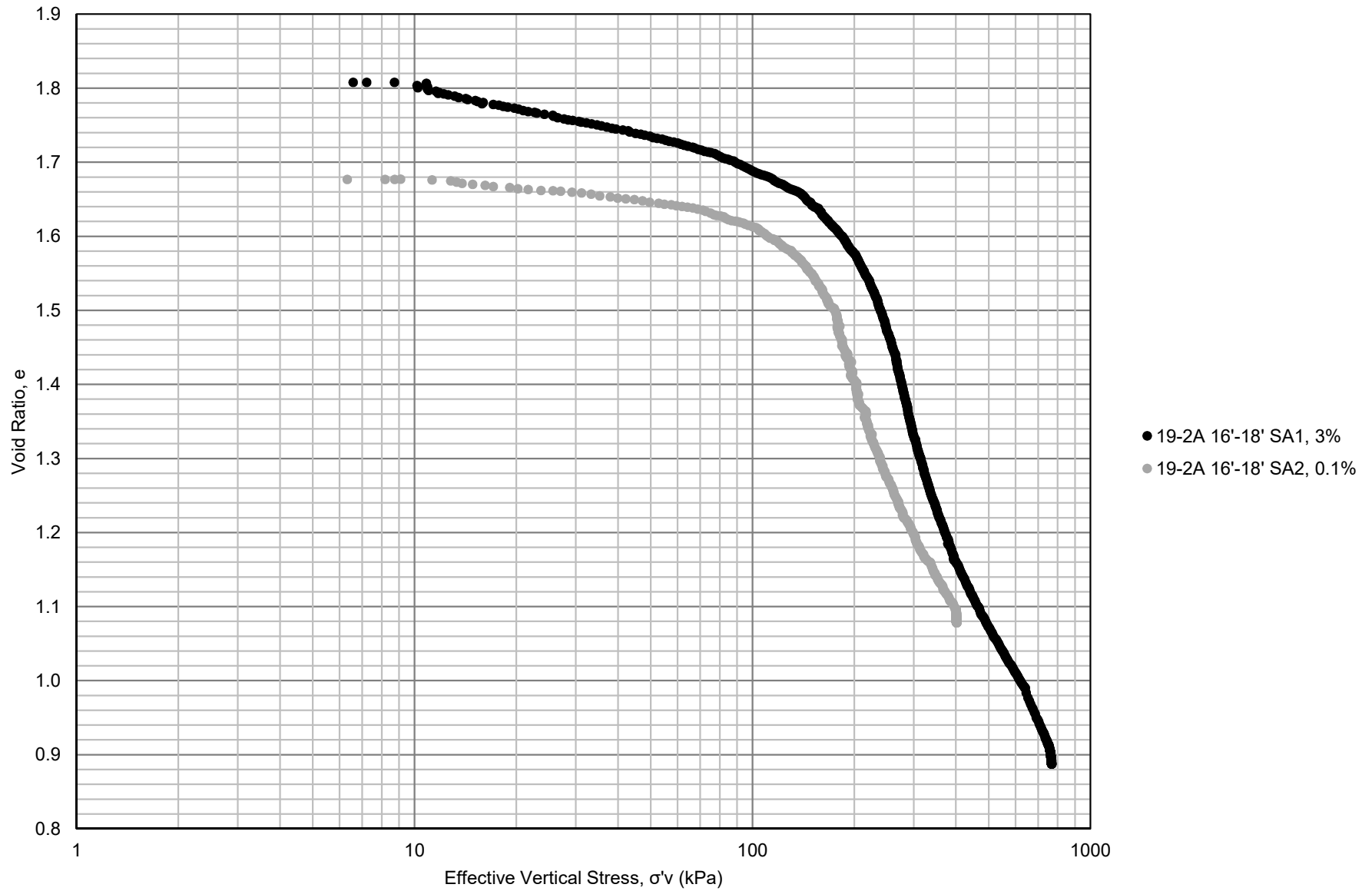
Sample weight = 88.94g

Initial moisture content = 42.87%

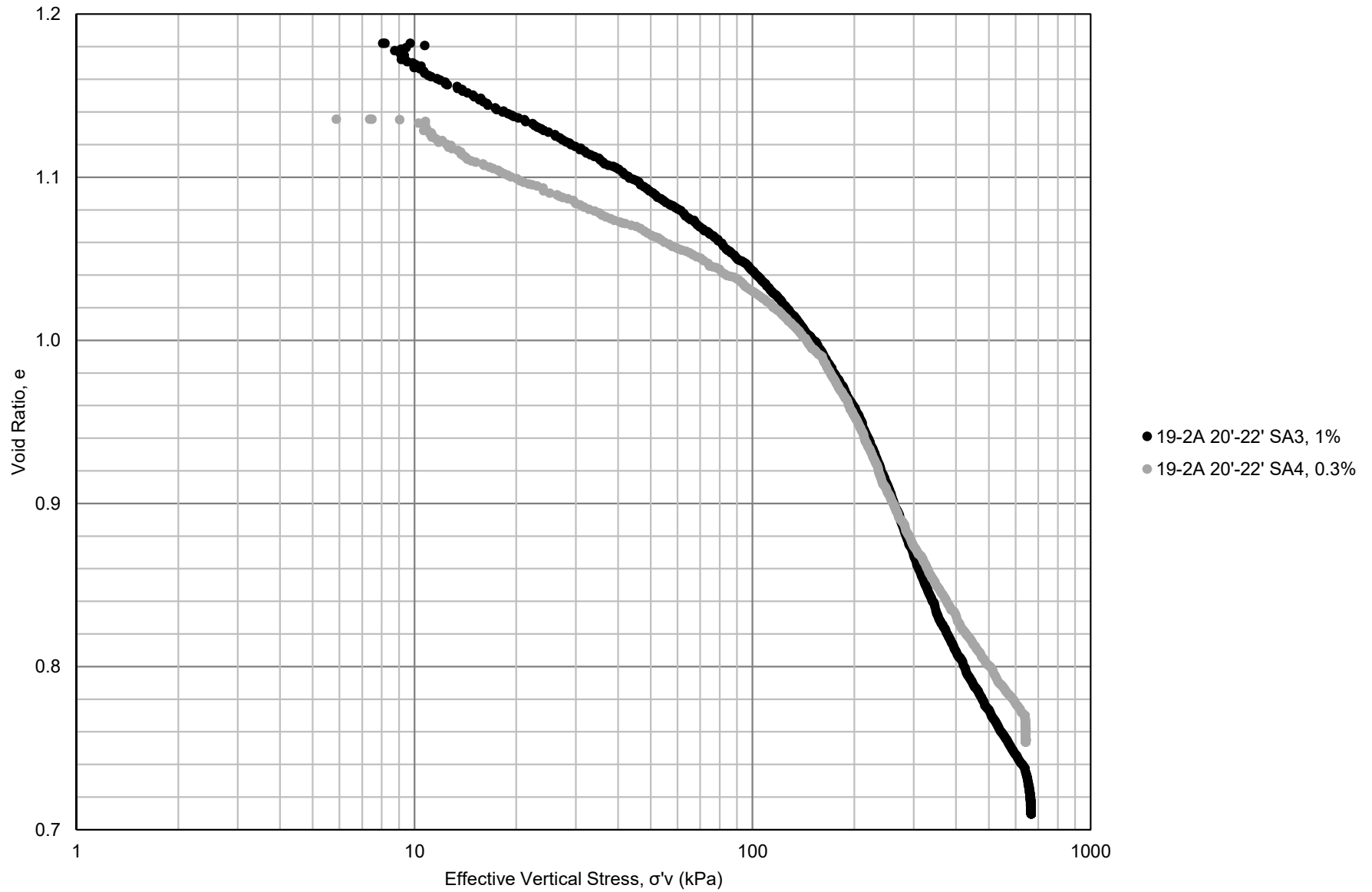
Final moisture content = 31.82%

Tested by Melodie Richards

Laronde Creek Bridge
Constant Rate of Strain (CRS) Consolidation Test Results
19-2A 16'-18' SA1 and 2



Laronde Creek Bridge
Constant Rate of Strain (CRS) Consolidation Test Results
19-2A 20'-22' SA3 and 4





Appendix C.3

Rock Core Photos Rock Core Testing Results

Borehole 19-03
Run 1 to 3
Elevation 179.9 m to 177.8 m





Stantec

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

September 10, 2019
File: 122410864

Attention: Thurber Engineering, File #23411

Reference: ASTM D7012, Method C, Unconfined Compressive Strength of Intact Rock Core
Highway 17 Laronde

The following table summarizes two rock core compressive strength results.

Location	Sample Depth	Compressive Strength (MPa)	Description of Break
19-3 UCS-3	62'3"-62'7"	146.6	Well-formed cones at both ends
19-3 UCS-2	62'8"-63'2"	162.1	Well-formed cones at both ends

Sincerely,

Stantec Consulting Ltd

Brian Prevost

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



Appendix C.4
Analytical Testing Results

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

Report Date: 06-Sep-2019

Order Date: 3-Sep-2019

Project Description: 23411(Laronde)

Client ID:	19-2B SS2 2'6"-4'-6"	19-2B SS4 7'6"-9'-6"	19-3 SS5 8'-10'	19-3 SS8 20'-22'
Sample Date:	14-Aug-19 09:00	14-Aug-19 09:00	16-Aug-19 09:00	17-Aug-19 09:00
Sample ID:	1936023-01	1936023-02	1936023-03	1936023-04
MDL/Units	Soil	Soil	Soil	Soil

Physical Characteristics

% Solids	0.1 % by Wt.	87.7	65.7	74.8	69.9
----------	--------------	------	------	------	------

General Inorganics

Conductivity	5 uS/cm	62	637	228	228
pH	0.05 pH Units	7.68	7.70	7.77	8.09
Resistivity	0.10 Ohm.m	153	15.7	43.9	43.9

Anions

Chloride	5 ug/g dry	9	231	9	5
Sulphate	5 ug/g dry	<5	14	28	24

**SGS Canada Inc.**

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

Paracel Laboratories

Attn : Dale Robertson

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

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

13-September-2019

Date Rec. : 10 September 2019

LR Report: CA12210-SEP19

Reference: Project#: 1936023

Copy: #1

CERTIFICATE OF ANALYSIS

Final Report

Sample ID	Sample Date & Time	Sulphide %
1: Analysis Start Date		13-Sep-19
2: Analysis Start Time		13:06
3: Analysis Completed Date		13-Sep-19
4: Analysis Completed Time		13:21
5: QC - Blank		< 0.02
6: QC - STD % Recovery		115%
7: QC - DUP % RPD		ND
8: RL		0.02
9: 19-2B SS2 2'6"-4'6"	14-Aug-19	0.02
10: 19-2B SS4 7'6"-9'6"	14-Aug-19	0.02
11: 19-2B SS5 8'-10'	16-Aug-19	0.02
12: 19-3 SS8 20'-22'	17-Aug-19	< 0.02

RL - SGS Reporting Limit
ND - Not Detected

Kimberley Didsbury
Project Specialist,
Environment, Health & Safety



Appendix D.
Site Photographs



Photo 1: General view of bridge, looking west [October 26, 2018]



Photo 2: General view of bridge, looking east [October 26, 2018]



Photo 3: View of southwest gabion wall and west abutment [October 26, 2018]



Photo 4: View of southeast gabion wall and east abutment [October 26, 2018]



Photo 5: Erosion undermining sidewalk crib footing [October 26, 2018]



Photo 6: Southeast gabion retaining wall [October 26, 2018]



Photo 7: Tension crack above east abutment, note ponding on approach [October 26, 2018]



Photo 8: Tension crack above west abutment, note ponding on approach [October 26, 2018]



Photo 9: Newer asphalt padding on east approach [October 26, 2018]



Photo 10: Newer asphalt padding on west approach [October 26, 2018]



Photo 11: Erosion along northwest wing wall [October 26, 2018]



Photo 12: Piezometer installed in 1998 with active artesian flow [October 26, 2018]



Photo 13: View of east bank on north side of the bridge [October 26, 2018]

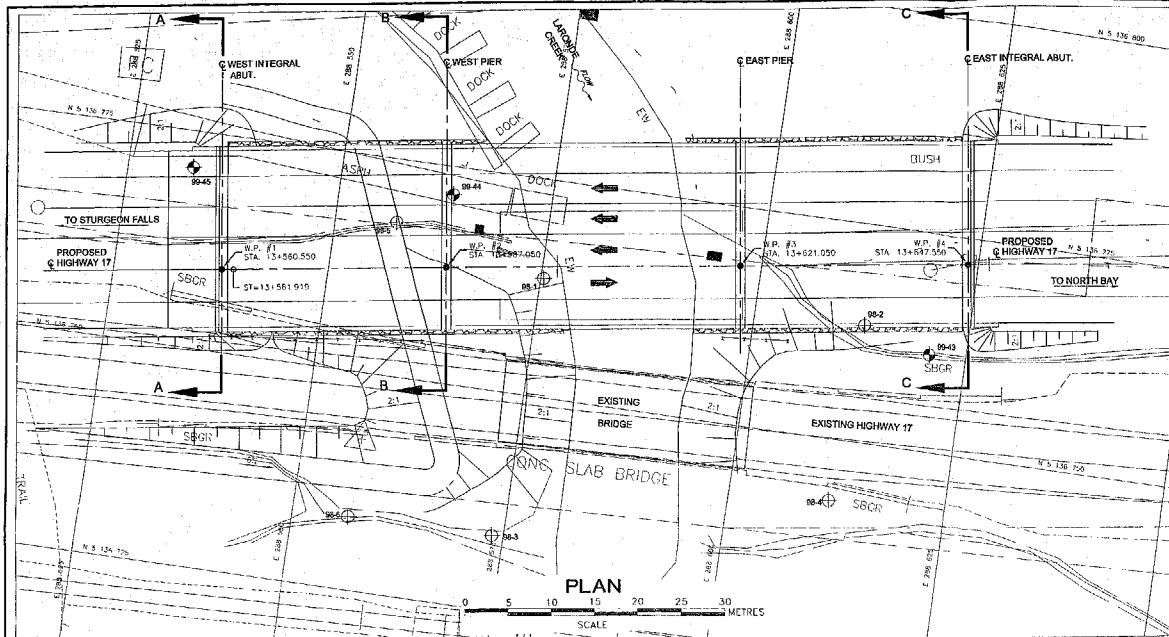


Photo 14: View of west bank on north side of the bridge [October 26, 2018]



Appendix E.

Factual Subsurface Information from GEOCREES Reports



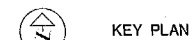
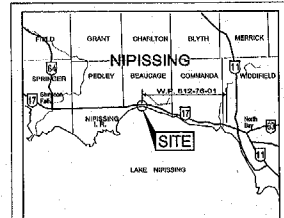
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT. No.
WP No. 812-76-02

LARONDE CREEK BRIDGE
BOREHOLE LOCATIONS & SOIL STRATA



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



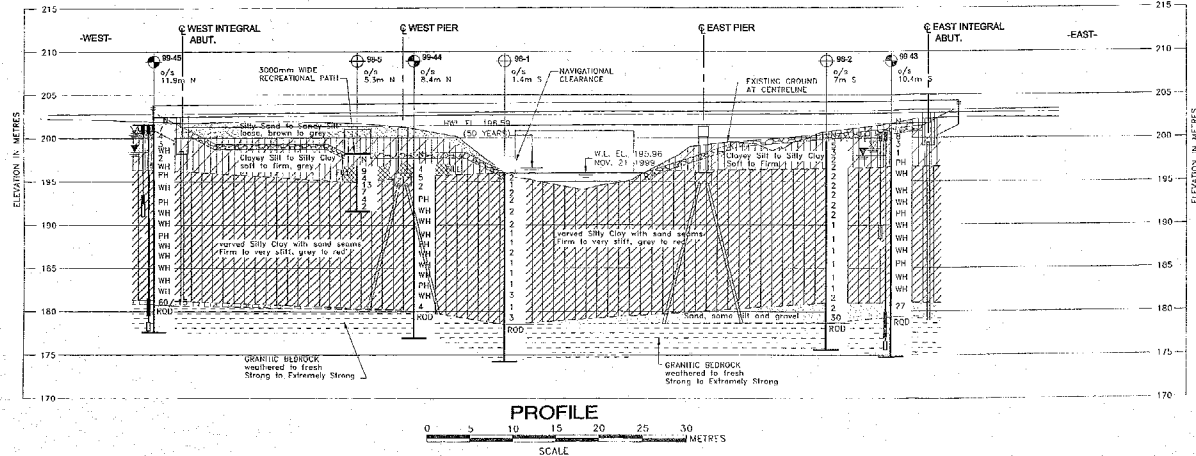
- LEGEND**
- 99-43 Borehole by Golder Associates (current investigation)
 - 99-51 Borehole by Thurber Engineering Ltd. (Report Dated March 1988)
 - Seal
 - Piezometer
 - N Standard Penetration Test value
 - 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 l/blow)
 - 100% Rock Quality Designation (RQD)
 - WL in deep piezometer on Oct. 3, 1989
 - WL in shallow piezometer on Oct. 3, 1989

No.	ELEVATION	NORTHING	EASTING
99-43	200.78	5156761	288620.6
99-44	187.04	5136772	288563.5
99-45	201.57	5136771	288532.6
98-1	196.80	5136764	288575.3
98-2	199.80	5136763	288612.7
98-3	197.50	5136754	288575.7
98-4	201.40	5136754	288611.5
98-5	N/A	5136768	288557.3
98-6	N/A	5136733	288556.5

REFERENCE
This drawing was created from digital files provided by McCannick Barker Co.

NOTES

- the boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- For detailed Stratigraphy of Borehole locations refer to Record of Borehole Sheets.



NO.	DATE	BY	REVISION
1	1991-11-04	RY	1
2	1991-11-04	RY	2
3	1991-11-04	RY	3
4	1991-11-04	RY	4
5	1991-11-04	RY	5
6	1991-11-04	RY	6
7	1991-11-04	RY	7
8	1991-11-04	RY	8
9	1991-11-04	RY	9
10	1991-11-04	RY	10
11	1991-11-04	RY	11
12	1991-11-04	RY	12
13	1991-11-04	RY	13
14	1991-11-04	RY	14
15	1991-11-04	RY	15
16	1991-11-04	RY	16
17	1991-11-04	RY	17
18	1991-11-04	RY	18
19	1991-11-04	RY	19
20	1991-11-04	RY	20
21	1991-11-04	RY	21
22	1991-11-04	RY	22
23	1991-11-04	RY	23
24	1991-11-04	RY	24
25	1991-11-04	RY	25
26	1991-11-04	RY	26
27	1991-11-04	RY	27
28	1991-11-04	RY	28
29	1991-11-04	RY	29
30	1991-11-04	RY	30

PROJECT 991-1164

RECORD OF BOREHOLE No 99-43

1 OF 2

METRIC

W.P. 812-76-01

LOCATION N 5136761.07; E 288620.59 (Laronde Creek, Site 43-65)

ORIGINATED BY DRS

DIST 54 HWY 17

BOREHOLE TYPE Bombardier CME-55

COMPILED BY BVB

DATUM Geodetic

DATE 10/2/1999

CHECKED BY DEB

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						
200.78	GROUND SURFACE							20 40 60 80 100						
0.00	Topsoil							○ UNCONFINED + FIELD VANE						
0.12	Clayey Silt/Silty Clay		1	50 DO	8			● QUICK TRIAXIAL × REMOULDED						
	Firm		2	50 DO	3									
	Grey		3	50 DO	1									0 1 79 20
			4	75 TO	PH									
196.82	Varved Silty Clay, sand seams													
3.96	Firm to stiff													
	Grey to red		5	50 DO	WH									
			6	50 DO	WH									
			7	50 DO	WH									
			8	75 TO	PH									
			9	50 DO	WH									
			10	50 DO	WH									
			11	50 DO	WH									

Continued Next Page

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

ON_MOT 991-1164.GPJ ON_MOT.GDT 27/6/00

PROJECT 991-1164			RECORD OF BOREHOLE No 99-43			2 OF 2		METRIC					
W.P. 812-76-01			LOCATION N 5136761.07, E 288620.59 (Laronde Creek, Site 43-65)			ORIGINATED BY DRS							
DIST 54 HWY 17			BOREHOLE TYPE Bombardier CME-55			COMPILED BY BVB							
DATUM Geodetic			DATE 10/2/1999			CHECKED BY DEB							
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID		UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	W _p W W _L				
— CONTINUED FROM PREVIOUS PAGE —													
181.12	Varved Silty Clay, sand seams Firm to stiff Grey to red		12	75 TO	PH			185					
			13	50 DO	WH			184					
			14	50 DO	WH			183	X				
								182					
181	Sand, some silt and gravel Compact Grey to red		15	50 DO	27			181					
180													
179.50	Granitic Bedrock Weathered Strong to very strong becoming extremely strong with depth							180					
179													
178													
177													
176													
175													
174.93	For bedrock coring details refer to Record of Drillhole 99-43.												
25.85	END OF BOREHOLE												
	Notes: Water levels in shallow and deep piezometers at Elev. 198.8m and Elev. 198.0m, respectively on Oct. 3/99.												

ON_MOT 991-1164.GPJ ON_MOT.GDT 27/8/00

PROJECT 991-1164			RECORD OF BOREHOLE No 99-44			1 OF 2			METRIC								
W.P. 812-76-01			LOCATION N 5136771.75; E 288563.47 (Laronde Creek, Site 43-65)			ORIGINATED BY DRS											
DIST 54 HWY 17			BOREHOLE TYPE Bombardier CME-55			COMPILED BY BVB											
DATUM Geodetic			DATE 9/29/1999			CHECKED BY DEB											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT WATER CONTENT (%)			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED			W _p W W _L			γ	GR SA SI CL		
197.04	GROUND SURFACE							20 40 60 80 100									
0.00	Asphalt		1	50 DO	4												
0.13	Gravelly Sand, some silt, wood fragments and asphalt		2	50 DO	5		196										
	Loose Brown (Fill)																
195.59	Varved Silty Clay, sand seams		3	50 DO	2		195										
1.45	Firm to stiff Grey to red		4	75 TO	PH		194										
			5	50 DO	WH		193										
			6	50 DO	WH		192										
			7	50 DO	WH		191										
			8	75 TO	PH		190										
			9	50 DO	WH		189										
			10	50 DO	WH		188										
			11	50 DO	WH		187										
			12	75 TO	PH		186										
							185										
							184										
							183										

ON_MOT_991-1164.GPJ ON_MOT.GDT 27/6/00

Continued Next Page

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

PROJECT 991-1164			RECORD OF BOREHOLE No 99-44			2 OF 2			METRIC							
W.P. 812-76-01			LOCATION N 5136771.75; E 288563.47 (Laronde Creek, Site 43-65)			ORIGINATED BY DRS										
DIST 54 HWY 17			BOREHOLE TYPE Bombardier CME-55			COMPILED BY BVB										
DATUM Geodetic			DATE 9/29/1999			CHECKED BY DEB										
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 × REMOULDED								WATER CONTENT (%)
— CONTINUED FROM PREVIOUS PAGE —																
	Varved Silty Clay, sand seams Firm to stiff Grey to red		13	50 DO	WH		181									
180.12			14	50 DO	4		180									
17.07	Sand, some silt, some gravel, trace clay Loose Grey Granitic Bedrock Fresh Very to extremely strong						179									
								178								
176.92	For bedrock coring details refer to Record of Drillhole 99-44.						177									
20.12	END OF BOREHOLE															

ON_MOT 991-1164 GPJ ON_MOT.GDT 27/6/00

PROJECT: 991-1164

RECORD OF DRILLHOLE: 99-44

SHEET 1 OF 1

LOCATION: N 5136771.75; E 288563.47

DRILLING DATE: 10/30/1999

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: Bombardier CME-55

DRILLING CONTRACTOR:

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH	COLOUR % RETURN	FR-FRACTURE	F-FAULT	SM-SMOOTH	FL-FLEXURED	BC-BROKEN CORE	DIAMETRAL INDEX (IPR)	NOTES WATER LEVELS INSTRUMENTATION
									CL-CLEAVAGE	J-JOINT	R-ROUGH	UE-UNEVEN	MB-MECH. BREAK		
									SH-SHEAR	P-POLISHED	ST-STEPPED	W-WAVY	B-BEDDING		
									VN-VEIN	S-SLICKENSIDED	PL-PLANAR	C-CURVED			
		CONTINUED FROM PREVIOUS PAGE		182.04 15.00											
15															
16															
17		Granitic Bedrock, fresh Very to Extremely Strong		179.97 17.07											
18															
19															
20				176.92 20.12											
21		END OF BOREHOLE													
22															
23															
24															
25															

DRILLHOLE 1164 ROCK GPJ GLDR CAN GDT 8/2/00

DEPTH SCALE

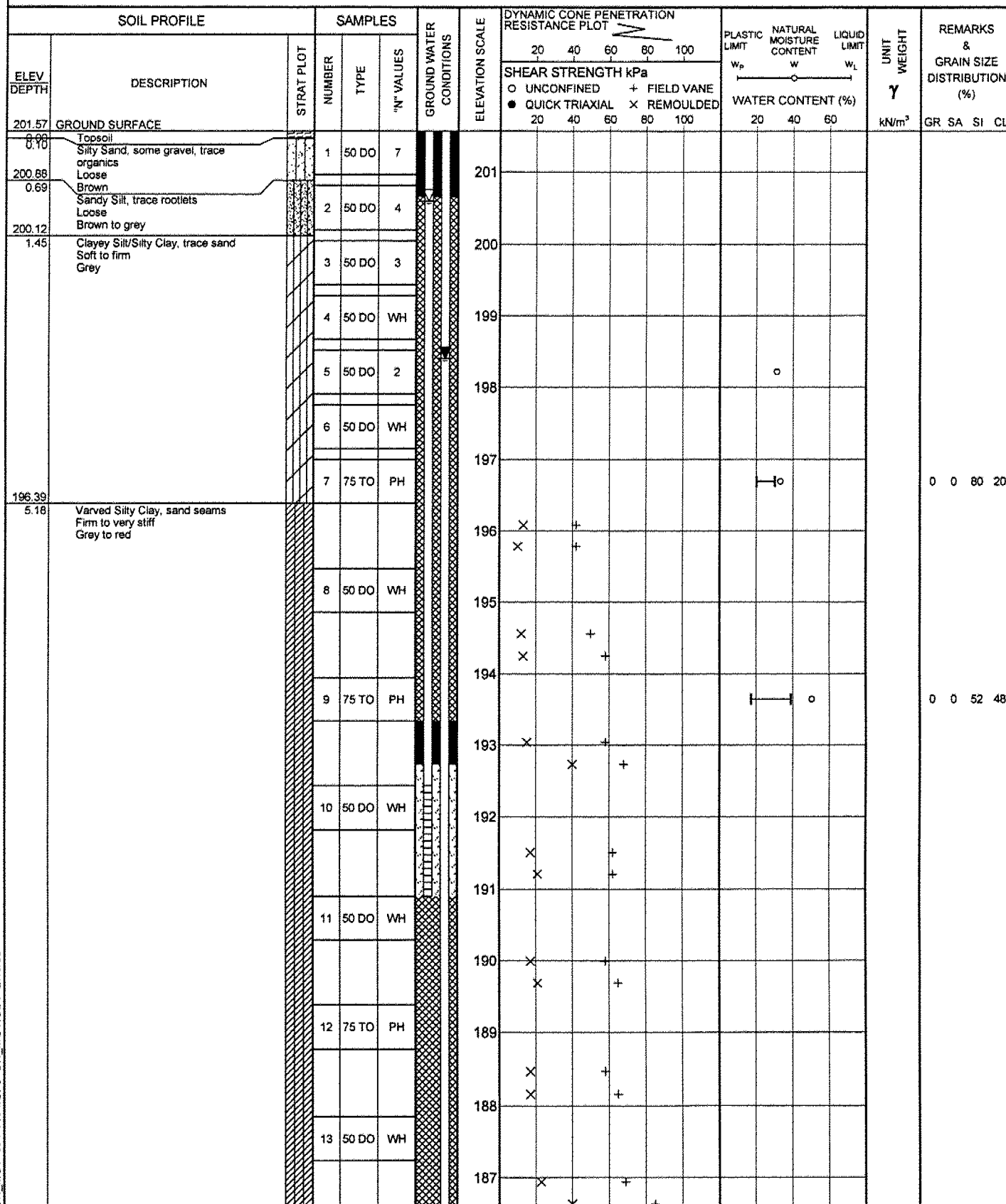
1 : 50



LOGGED: DRS

CHECKED: BVB

PROJECT <u>991-1164</u>		RECORD OF BOREHOLE No 99-45		1 OF 2	METRIC
W.P. <u>812-76-01</u>		LOCATION <u>N 5136770.75; E 288532.60 (Laronde Creek, Site 43-65)</u>		ORIGINATED BY <u>DRS</u>	
DIST <u>54</u> HWY <u>17</u>		BOREHOLE TYPE <u>Bombardier CME-55</u>		COMPILED BY <u>BVB</u>	
DATUM <u>Geodetic</u>		DATE <u>9/27/1999</u>		CHECKED BY <u>DEB</u>	



ON MOT 991-1164.GPJ ON MOT.GDT 27/6/00

Continued Next Page

+³, X³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

PROJECT 991-1164		RECORD OF BOREHOLE No 99-45		2 OF 2		METRIC							
W.P. 812-76-01		LOCATION N 5136770.75; E 288532.60 (Laronde Creek, Site 43-65)		ORIGINATED BY DRS									
DIST 54 HWY 17		BOREHOLE TYPE Bombardier CME-55		COMPILED BY BVB									
DATUM Geodetic		DATE 9/27/1999		CHECKED BY DEB									
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED		WATER CONTENT (%) W _p W W _L		γ kN/m ³	GR SA SI CL
--- CONTINUED FROM PREVIOUS PAGE ---													
181.30	Varved Silty Clay, sand seams Firm to very stiff Grey to red		14	50 DO	WH		186						
							185	×		+			
			15	50 DO	WH		184	×		+			
							183						
			16	50 DO	WH		182	×		+			
							181						
20.27	Clayey Silt, trace sand Grey		17	50 DO	WH		180						
180.84	Silty Sand, some gravel, trace clay Grey		18	50 DO	60/15		179						
20.88	Granitic Bedrock Fresh Very to extremely strong						178						
177.55	For bedrock coring details refer to Record of Drillhole 99-45.												
24.02	END OF BOREHOLE Note: Water levels in shallow and deep piezometers at Elev. 200.6m and Elev. 198.4m, respectively on Oct. 3/99.												

ON_MOT_991-1164.GPJ ON_MOT.GDT 27/6/00

PROJECT: 991-1164

RECORD OF DRILLHOLE: 99-45

SHEET 1 OF 1

LOCATION: N 5136770.75; E 288532.60

DRILLING DATE: 9/28/1999

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: Bombardier CME-55

DRILLING CONTRACTOR:

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV.	RUN No.	PENETRATION RATE (mm/min)	FLUSH COLOUR % RETURN	FR-FRACTURE		F-FAULT		SM-SMOOTH		FL-FLEXURED		BC-BROKEN CORE		DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
				DEPTH (m)				CL-CLEAVAGE		J-JOINT		R-ROUGH		UE-UNEVEN		MB-MECH. BREAK			
								SH-SHEAR		P-POLISHED		ST-STEPPED		W-WAVY		B-BEDDING			
								VN-VEIN		S-SLICKENSIDED		PL-PLANAR		C-CURVED					
								RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY k, cm/sec					
TOTAL CORE %	SOLID CORE %	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	10 ⁴	10 ³	10 ²	10 ¹												
20		CONTINUED FROM PREVIOUS PAGE		181.57 20.00															
21	NQ Core	Granitic Bedrock, fresh Very to Extremely Strong		180.89 20.88	1		100												
22				2		100													
23				3		100													
24		END OF BOREHOLE		177.55 24.02															
25																			
26																			
27																			
28																			
29																			
30																			

DRILLHOLE 1164ROCK GPJ GLDR. CAN.GDT 8/2/00

DEPTH SCALE

1 : 50



LOGGED: DRS

CHECKED: BVB

TABLE B1

SUMMARY OF WATER CONTENT, ATTERBERG LIMITS AND SPECIFIC GRAVITY DETERMINATIONS

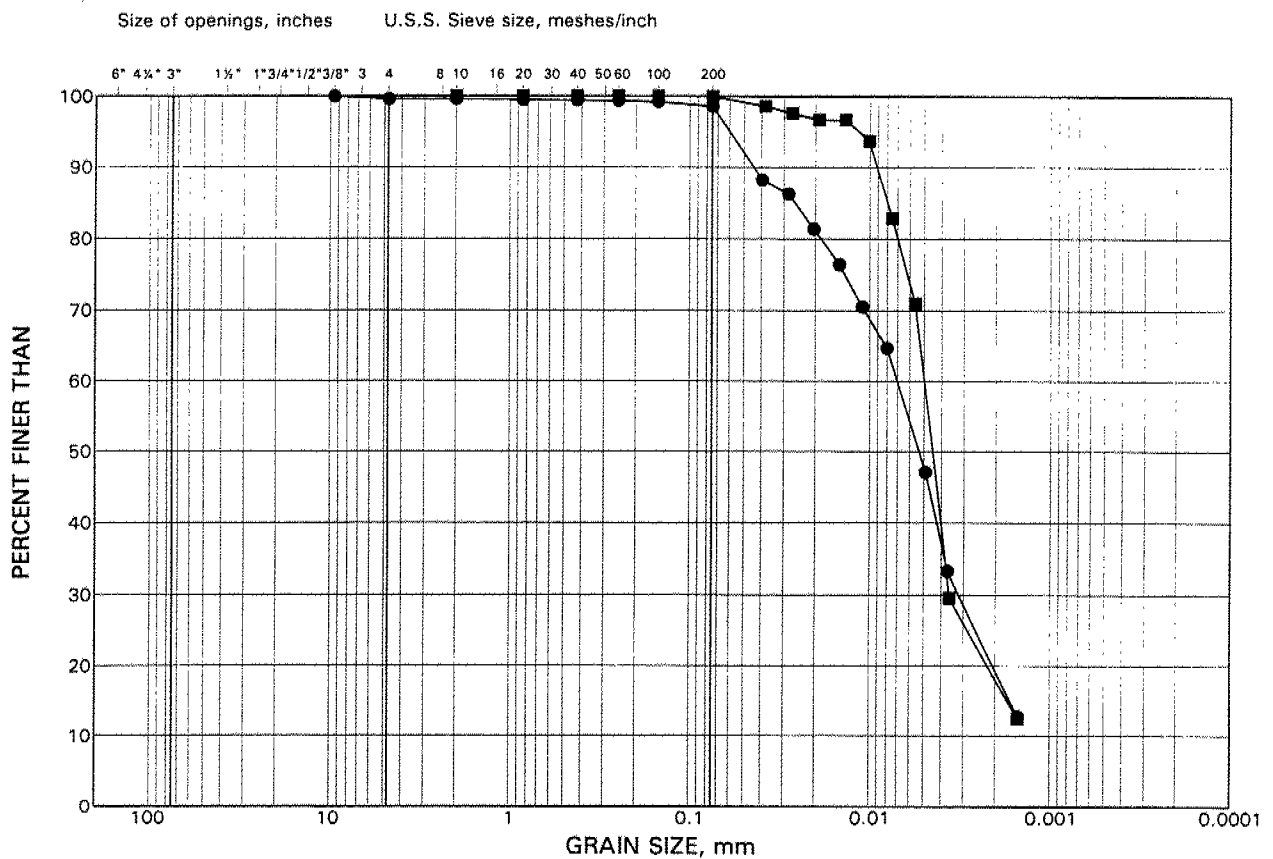
PROJECT NUMBER		991-1164			
PROJECT NAME		McCormick Rankin / Highway 17 / North Bay			
DATE TESTED		October/November, 1999			
Borehole No.	Sample No.	Depth (m)	Water Content (%)	Atterberg Limits Wl,Wp,Ip	Specific Gravity
99-43	3	1.52-2.13	10.0		
99-43	5	3.05-3.66	30.2		
99-43	8	6.10-6.71	22.6		
99-44	6	4.57-5.18	55.4		
99-44	11	12.19-12.80	41.2	Wl=28.6, Wp=16.0, Ip=12.6	
99-45	5	3.05-3.51	31.1		
99-45	7	4.57-5.18	33.0	Wl=30.1, Wp=20.1, Ip=10.0	2.70
99-45	9	7.62-8.08	50.3	Wl=38.8, Wp=17.2, Ip=21.6	2.66
99-46	3	1.52-2.13	34.3	Wl=30.6, Wp=19.5, Ip=11.1	
99-46	7	7.62-8.23	56.9	Wl=47.7, Wp=21.4, Ip=26.3	
99-46	14	18.29-18.90	41.4		

Notes: Specific gravity test carried out using distilled water.
 Wl = Liquid Limit
 Wp = Plastic Limit
 Ip = Plasticity Index

GRAIN SIZE DISTRIBUTION

Clayey Silt / Silty Clay trace sand

FIGURE B1



COBBLE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
SIZE	GRAVEL SIZE		SAND SIZE			FINE GRAINED

LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
●	99-43	3	2.1
■	99-45	7	-

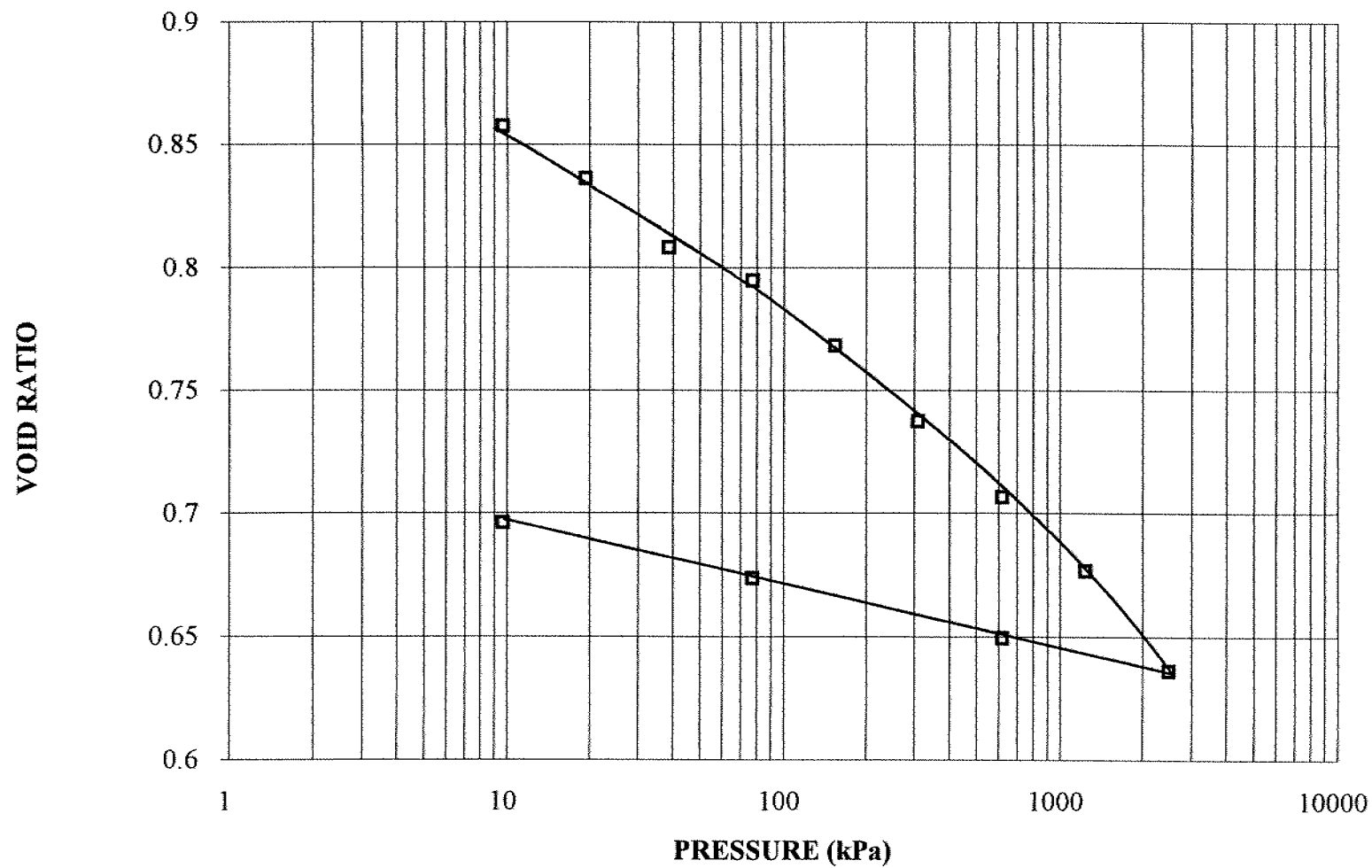
FIGURE B2



SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
--------	----------	--------	----------

9

CONSOLIDATION TEST
VOID RATIO vs LOG. PRESSURE
BH 99-45 SA 7



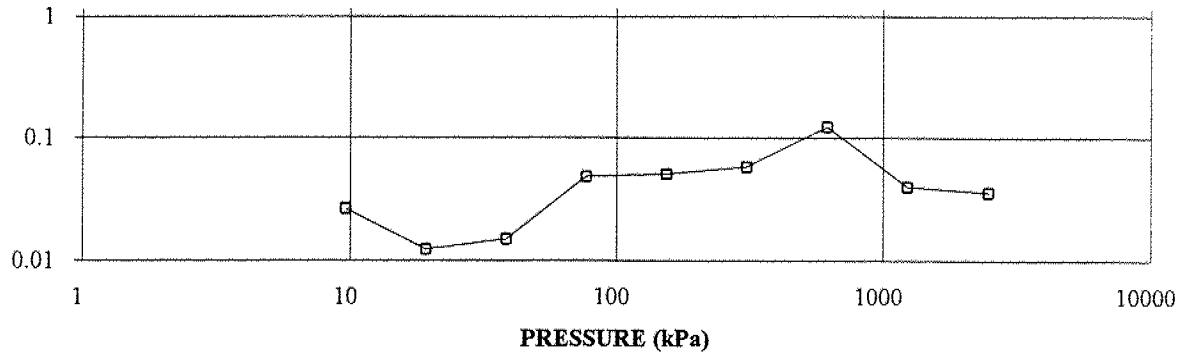
CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

FIGURE B3

OEDOMETER CONSOLIDATION SUMMARY

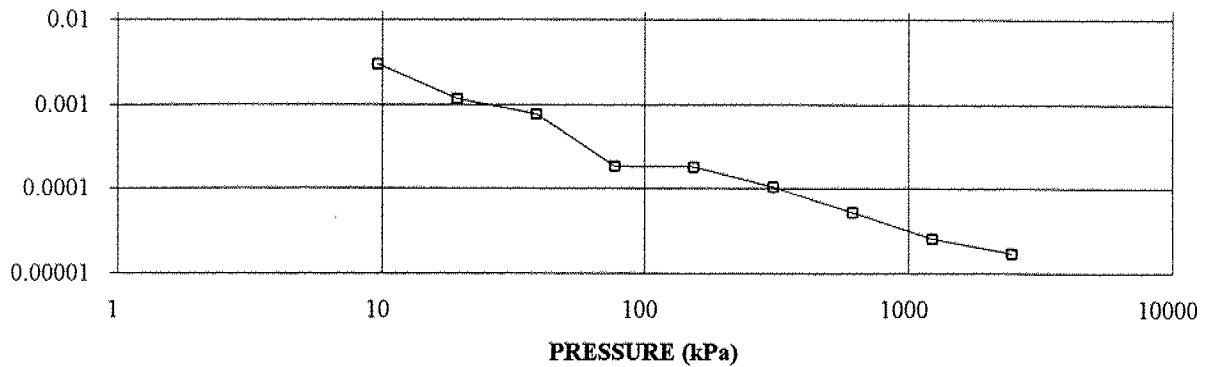
COEFFICIENT OF CONSOLIDATION, cm^2/s

CONSOLIDATION TEST
LOG. $\text{cv cm}^2/\text{s}$ vs LOG. PRESSURE (kPa)
BH 99-45 SA 7



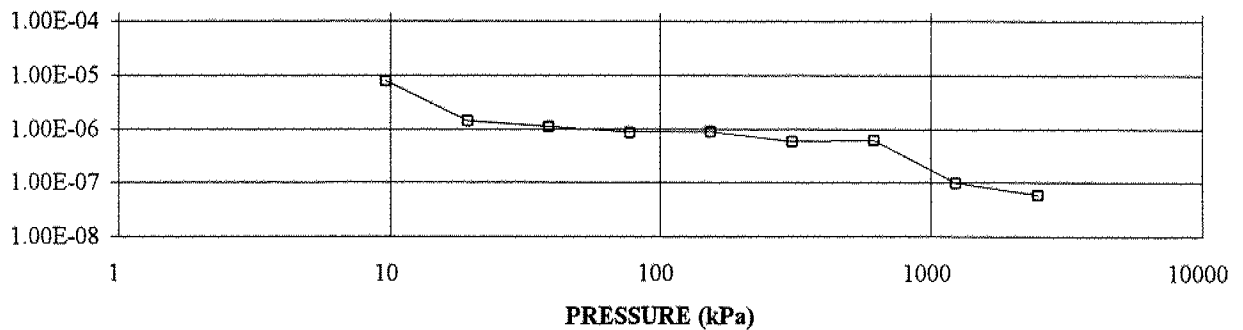
VOLUME
COMPRESSIBILITY,
 m^2/kN

CONSOLIDATION TEST
LOG. $\text{mv, m}^2/\text{kN}$ vs LOG. PRESSURE (kPa)
BH 99-45 SA 7



HYDRAULIC
CONDUCTIVITY, cm/s

CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs LOG. PRESSURE
BH 99-45 SA 7



OEDOMETER CONSOLIDATION SUMMARY

SAMPLE IDENTIFICATION

Project Number	991-1164	Sample Number	7
Borehole Number	99-45	Sample Depth, m	4.6-5.2

TEST CONDITIONS

Test Type	Quick /Standard	Load Duration, hr	(0.10 - 22)
Oedometer Number	7		
Date Started	99-10-08		
Date Completed	99-10-09		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.88	Unit Weight, kN/m ³	18.64
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	13.84
Area, cm ²	31.67	Specific Gravity, measured	2.70
Volume, cm ³	59.63	Solids Height, cm	0.984
Water Content, %	34.72	Volume of Solids, cm ³	31.17
Wet Mass, g	113.37	Volume of Voids, cm ³	28.47
Dry Mass, g	84.15	Degree of Saturation, %	102.6

TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t ₉₀ sec	cv. cm ² /s	mv m ² /kN	k cm/s
0.00	1.883	0.913	1.883				
9.66	1.828	0.858	1.856	28	2.61E-02	3.02E-03	7.71E-06
19.31	1.807	0.836	1.818	57	1.23E-02	1.15E-03	1.39E-06
38.63	1.779	0.808	1.793	46	1.48E-02	7.62E-04	1.11E-06
77.25	1.766	0.795	1.773	14	4.76E-02	1.83E-04	8.53E-07
154.50	1.740	0.768	1.753	13	5.01E-02	1.79E-04	8.81E-07
309.00	1.710	0.737	1.725	11	5.73E-02	1.04E-04	5.85E-07
618.00	1.679	0.706	1.694	5	1.22E-01	5.24E-05	6.25E-07
1236.00	1.650	0.676	1.664	15	3.92E-02	2.54E-05	9.73E-08
2471.99	1.610	0.636	1.630	16	3.52E-02	1.70E-05	5.87E-08
618.00	1.623	0.649	1.617				
77.25	1.647	0.673	1.635				
9.66	1.669	0.696	1.658				

Notes:

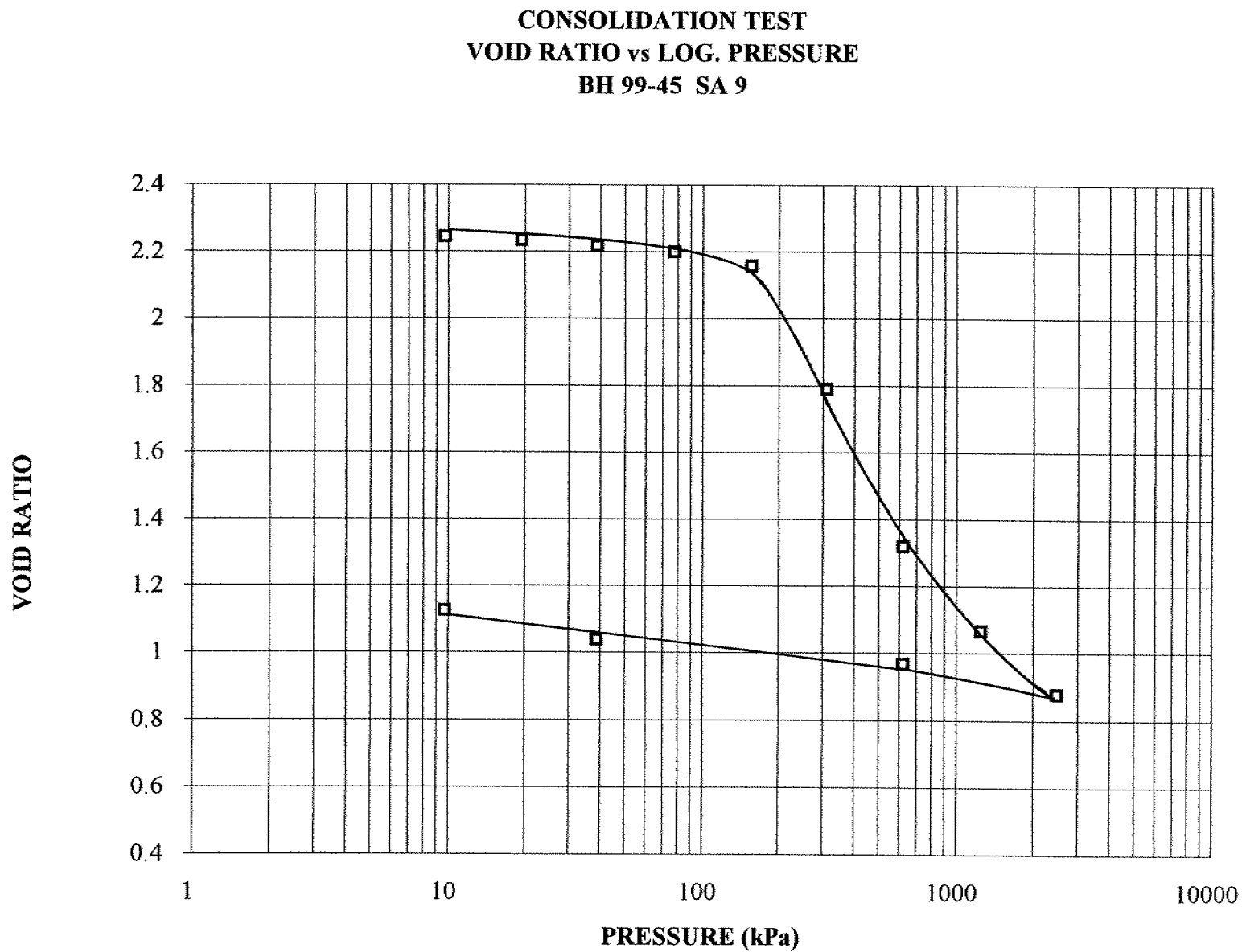
k calculated using cv based on t₉₀ values.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.67	Unit Weight, kN/m ³	19.84
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	15.61
Area, cm ²	31.67	Specific Gravity, measured	2.70
Volume, cm ³	52.86	Solids Height, cm	0.984
Water Content, %	27.11	Volume of Solids, cm ³	31.17
Wet Mass, g	106.96	Volume of Voids, cm ³	21.69
Dry Mass, g	84.15		

CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

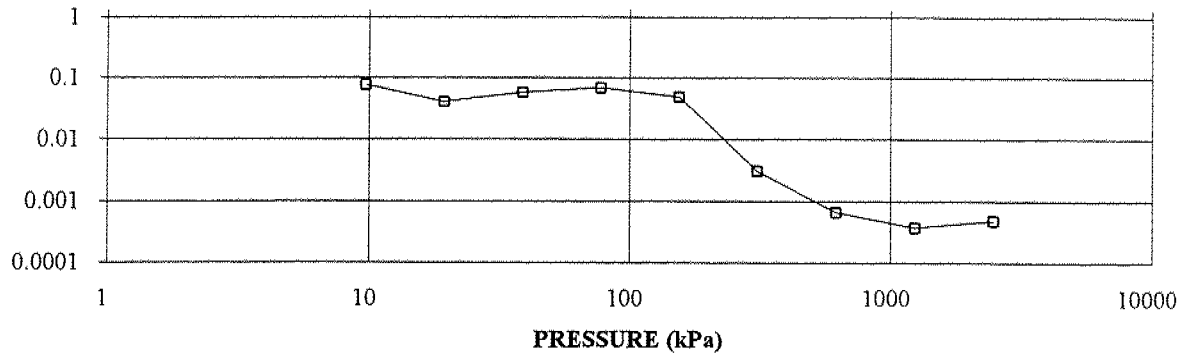
FIGURE B4



OEDOMETER CONSOLIDATION SUMMARY

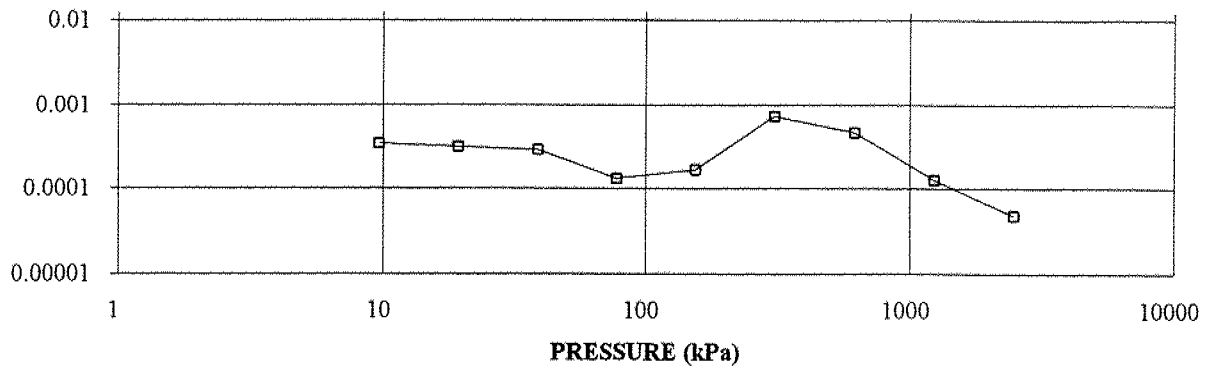
COEFFICIENT OF CONSOLIDATION, cm^2/s

CONSOLIDATION TEST
LOG. c_v cm^2/s vs LOG. PRESSURE (kPa)
BH 99-45 SA 9



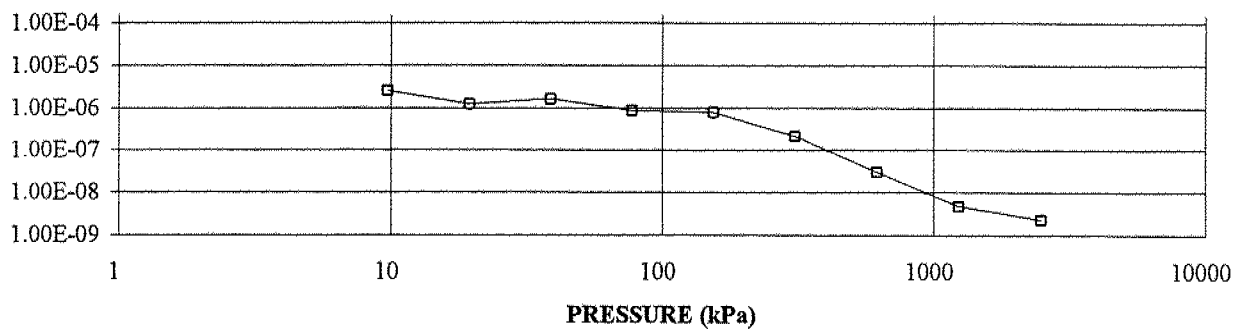
VOLUME
COMPRESSIBILITY,
 m^2/kN

CONSOLIDATION TEST
LOG. m_v , m^2/kN vs LOG. PRESSURE (kPa)
BH 99-45 SA 9



HYDRAULIC
CONDUCTIVITY, cm/s

CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs LOG. PRESSURE
BH 99-45 SA 9



OEDOMETER CONSOLIDATION SUMMARY

SAMPLE IDENTIFICATION

Project Number	991-1164	Sample Number	9
Borehole Number	99-45	Sample Depth, m	7.9

TEST CONDITIONS

Test Type	Quick /Standard	Load Duration, hr	(0.13 - 22)
Oedometer Number	6		
Date Started	99-10-07		
Date Completed	99-10-08		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.90	Unit Weight, kN/m ³	14.88
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m ³	8.02
Area, cm ²	31.52	Specific Gravity, measured	2.66
Volume, cm ³	59.92	Solids Height, cm	0.584
Water Content, %	85.63	Volume of Solids, cm ³	18.41
Wet Mass, g	90.92	Volume of Voids, cm ³	41.51
Dry Mass, g	48.98	Degree of Saturation, %	101.0

TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t ₉₀ sec	cv, cm ² /s	mv m ² /kN	k cm/s
0.00	1.901	2.254	1.901				
9.70	1.895	2.243	1.898	10	7.64E-02	3.42E-04	2.56E-06
19.40	1.889	2.233	1.892	19	3.99E-02	3.14E-04	1.23E-06
38.81	1.878	2.215	1.884	13	5.79E-02	2.90E-04	1.64E-06
77.62	1.869	2.199	1.873	11	6.76E-02	1.30E-04	8.63E-07
155.23	1.844	2.157	1.856	15	4.87E-02	1.67E-04	7.96E-07
310.46	1.629	1.788	1.736	211	3.03E-03	7.29E-04	2.17E-07
620.93	1.354	1.318	1.492	708	6.66E-04	4.65E-04	3.04E-08
1241.86	1.207	1.066	1.281	916	3.80E-04	1.25E-04	4.65E-09
2475.89	1.096	0.876	1.152	576	4.88E-04	4.72E-05	2.26E-09
620.93	1.149	0.966	1.122				
38.81	1.191	1.038	1.170				
9.70	1.241	1.124	1.216				

Notes:

k calculated using cv based on t₉₀ values.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.24	Unit Weight, kN/m ³	17.92
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m ³	12.28
Area, cm ²	31.52	Specific Gravity, measured	2.66
Volume, cm ³	39.11	Solids Height, cm	0.584
Water Content, %	45.92	Volume of Solids, cm ³	18.41
Wet Mass, g	71.47	Volume of Voids, cm ³	20.70
Dry Mass, g	48.98		

FIGURE 3
Vane Results vs. Depth

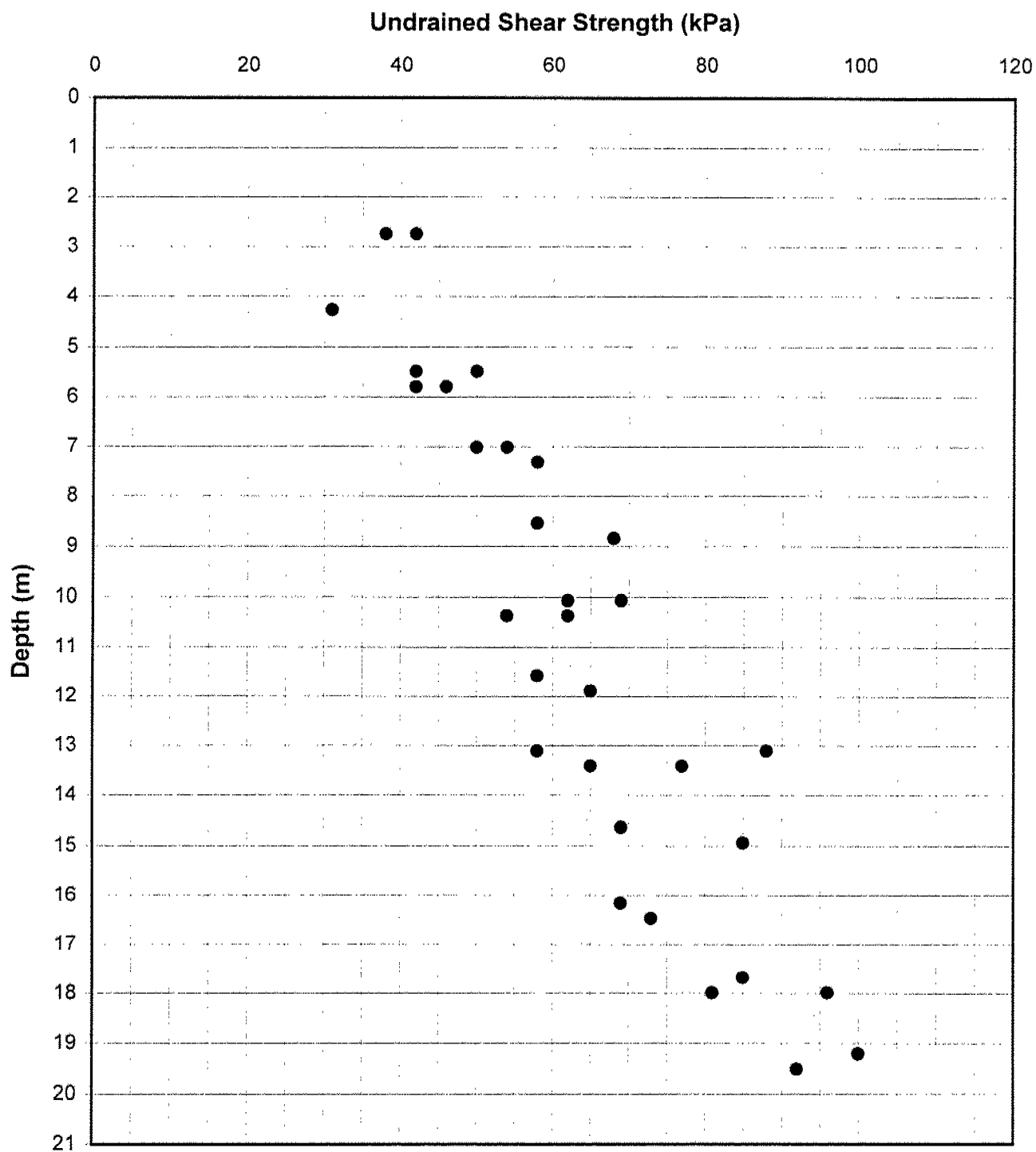
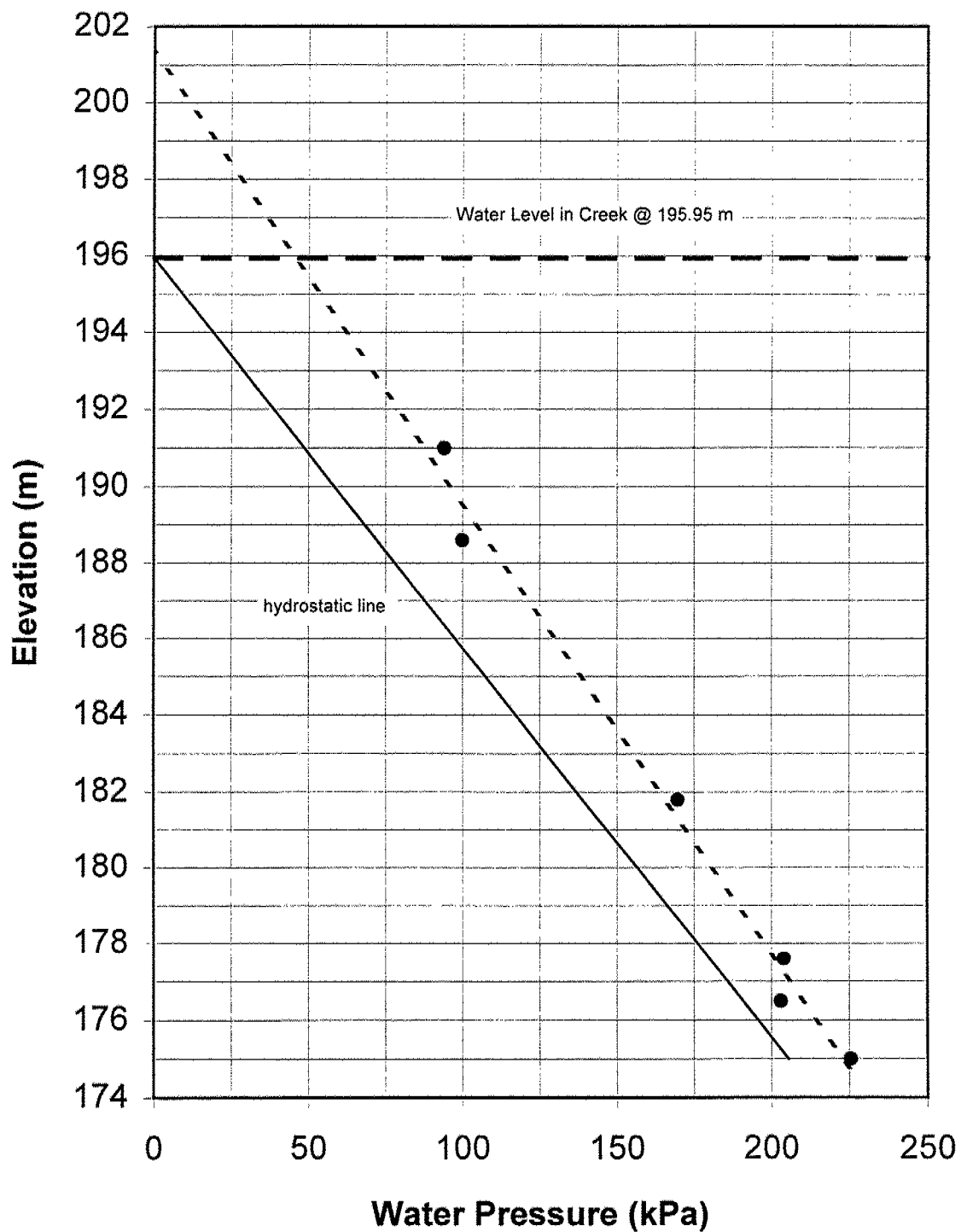



FIGURE 4
Ground Water Pressure vs. Elevation






DIST 54

CONT No.

WP No. 812-76-00 & 398-91-00



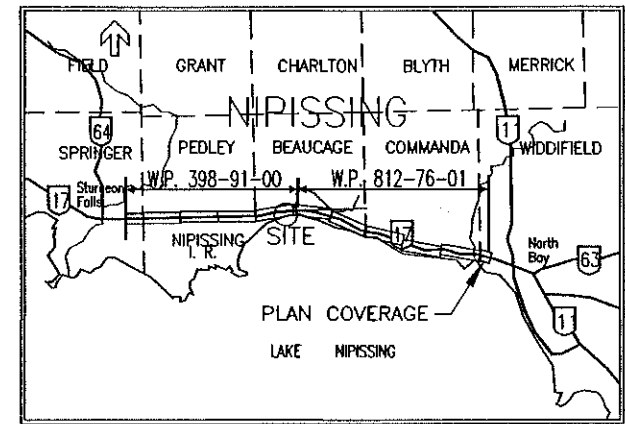
LARONDE CREEK BRIDGE

HIGHWAY 17


SHEET

THURBER ENGINEERING LTD.

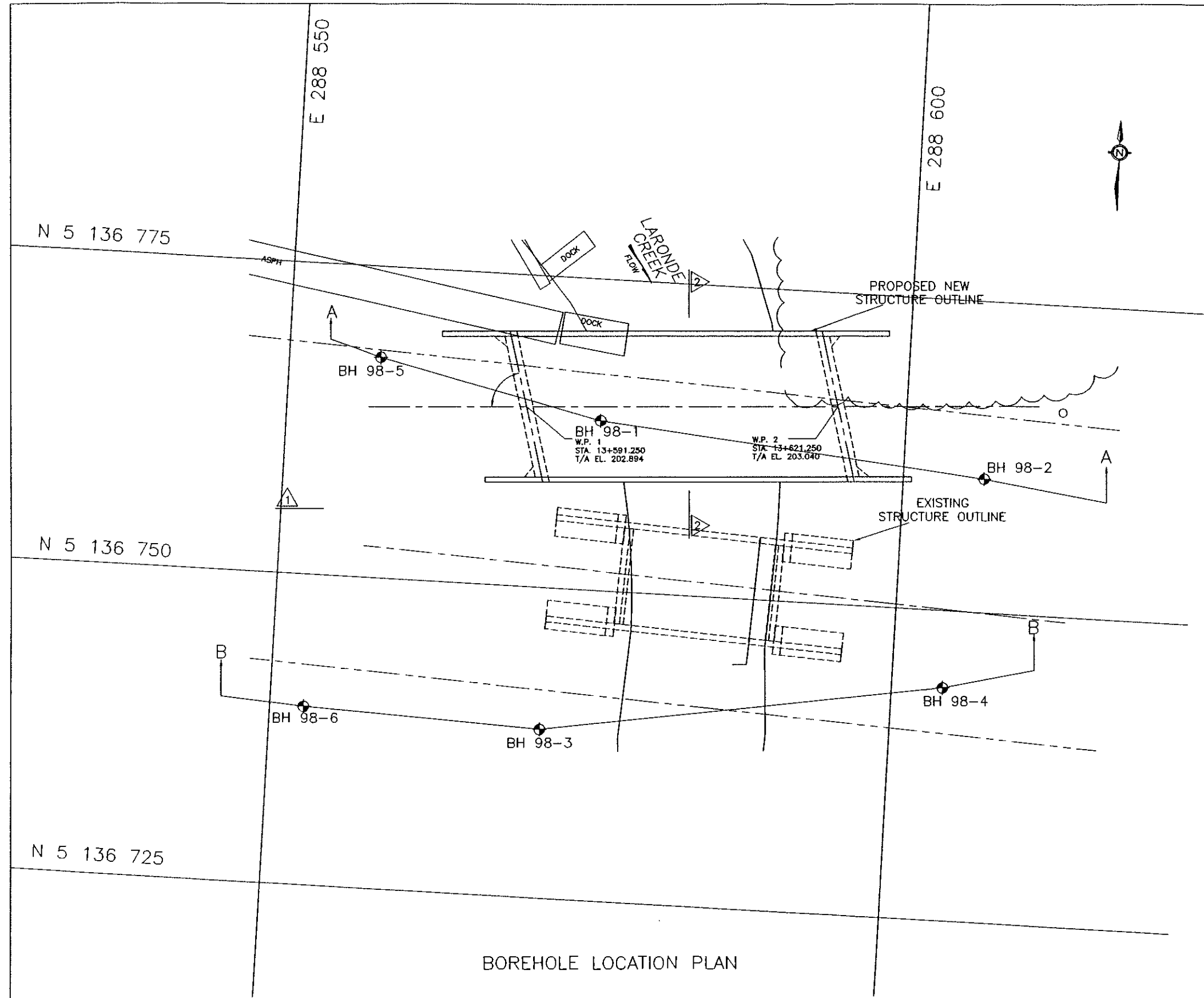
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN



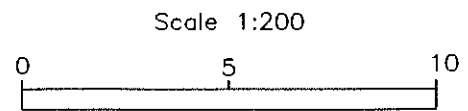
KEY PLAN

LEGEND			
<div> <div>BH1</div> <div>  </div> <div>Borehole</div> </div>			
No	ELEV.	LOCATION	
		NORTHING	EASTING
98-1	196.8	5136763.609	288575.314
98-2	199.6	5136763.377	288612.703
98-3	197.5	5136733.585	288573.659
98-4	201.4	5136743.057	288611.487
98-5	197.3	5136767.711	288557.288
98-6	198.4	5136733.414	288556.528

REVISIONS		DESCRIPTION			
DESIGN	AEG	CHK	PKG	CODE	LOAD
DRAWN	WM	CHK	SITE	812-76-00	STRUCT
			398-91-00		SCHEME
					DATE MAR 1999
					DWG 2



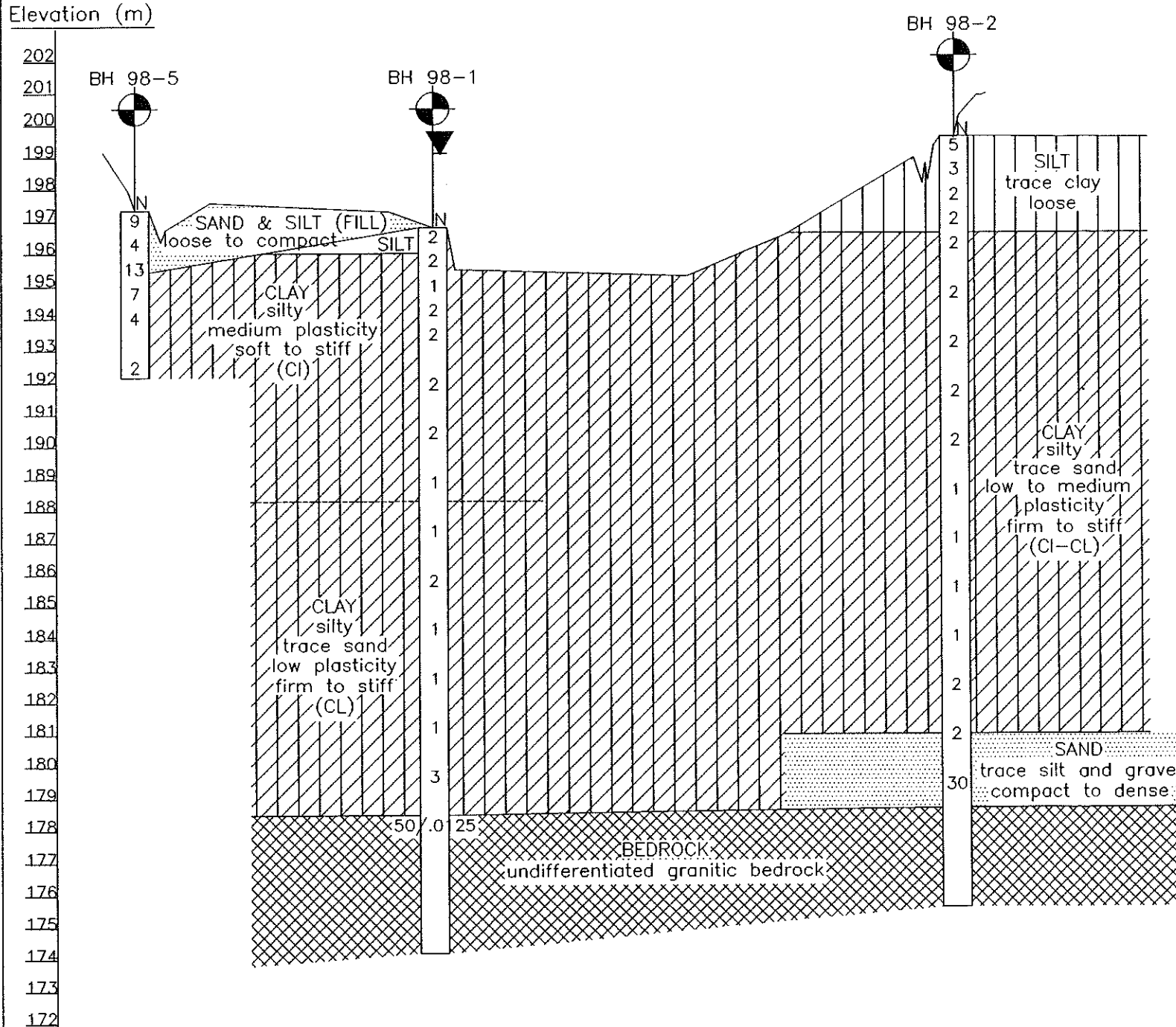
BOREHOLE LOCATION PLAN



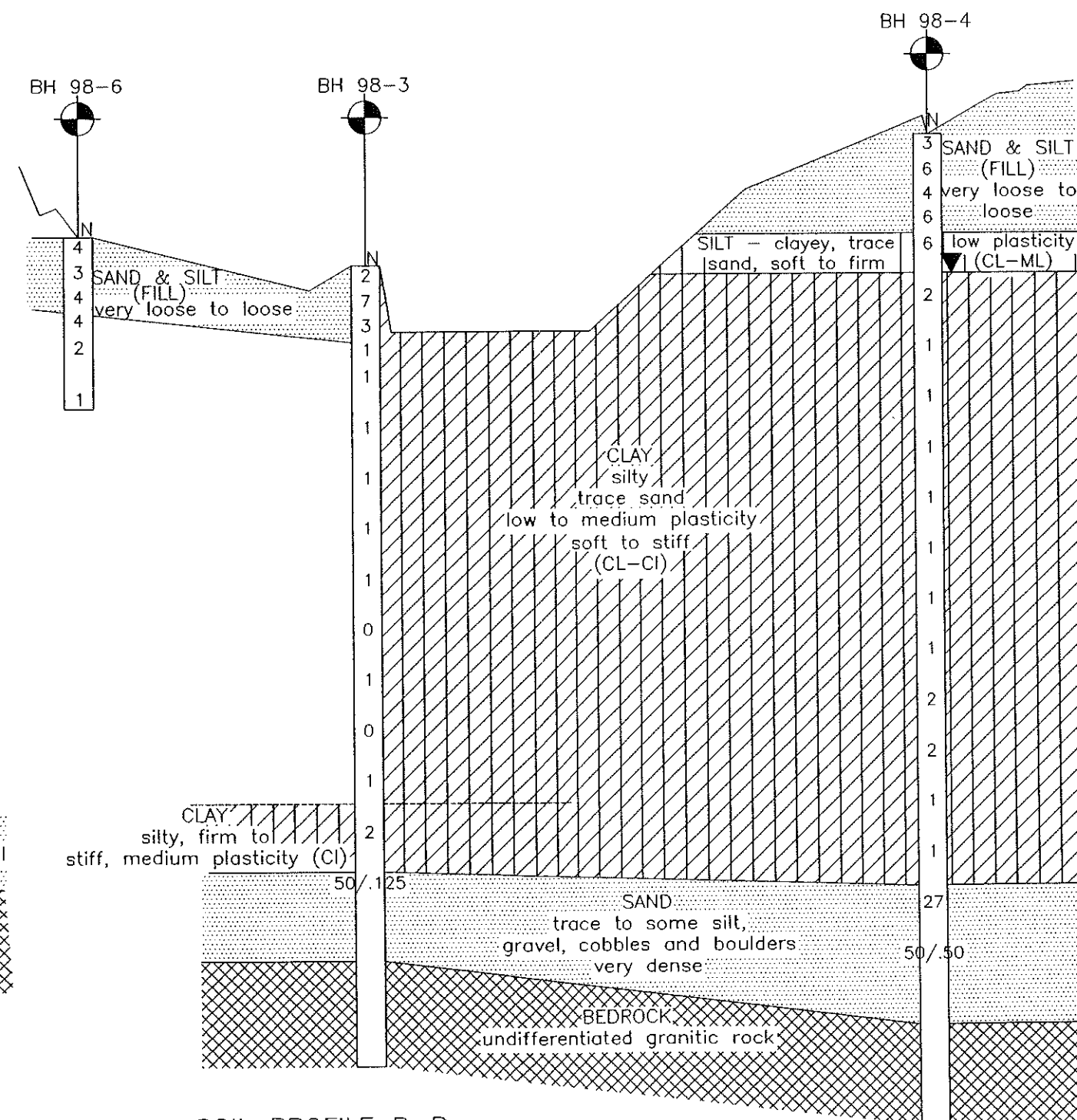
TABLET IN FACE OF W. ABUTMENT
OF CPR RAILWAY BRIDGE 24.6 RT.
AT STA. 13+596.2
(EXISTING HIGHWAY 17 L. ALIGNMENT)

PLOT-DATE

Elevation (m)





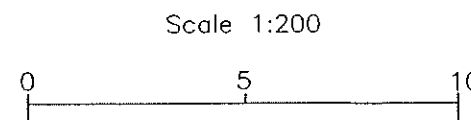
SOIL PROFILE A-A



SOIL PROFILE B-B

LEGEND

 BH 98-3
 WL October 27, 1998
 'N' Blows/0.3m (Std. Pen Test)



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

REVISIONS		DESCRIPTION			
DESIGN	AEG	CHK	PKC	CODE	LOAD
DRAWN	WM	CHK	SITE 812-76-00 & 398-91-00	STRUCT	SCHEME
					DATE MAR 1999
					DWG 2A

RECORD OF BOREHOLE No 98-1

1 OF 2

METRIC

W.P. 812-76-01,398-91-00

LOCATION Laronde Creek, N 5 136 763.6 E 288 575.3

ORIGINATED BY GA

DIST 54

HWY 17

BOREHOLE TYPE Hollow Stem Augers, N Core

COMPILED BY WM

DATUM Geodetic

DATE 98.10.15 - 98.10.15

CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)			
								○ UNCONFINED + FIELD VANE										
								● QUICK TRIAXIAL × LAB VANE										
							20	40	60	80	100	10	20	30				
196.8 0.0	SILT, sandy, very loose, grey, wet		1	SS	2									60.820				
196.1 0.8	CLAY, silty, medium plasticity, soft to firm, grey, wet (CI) some varves evident		2	SS	2		196											
			3	SS	1		195							41.7	0 0 41 59			
			4	SS	2		194							45.0	0 0 42 58			
			5	SS	2		193											
			6	SS	2		192							41.680				
			7	SS	2		191							48.480				
			8	SS	1		189		+					42.450	0 0 43 57			
188.3 8.5		CLAY, silty, trace sand, low plasticity, firm to stiff, grey, wet (CL) some varves evident		9	SS	1		188										
			10	SS	2		186		+						0 1 61 38			
			11	SS	1		185											
			12	SS	1		184											
							183							50.390				
							182											

Continued Next Page

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

RECORD OF BOREHOLE No 98-1

2 OF 2

METRIC

W.P. 812-76-01,398-91-00 LOCATION Laronde Creek, N 5 136 763.6 E 288 575.3 ORIGINATED BY GA
 DIST 54 HWY 17 BOREHOLE TYPE Hollow Stem Augers, N Core COMPILED BY WM
 DATUM Geodetic DATE 98.10.15 - 98.10.15 CHECKED BY AEG

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	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	
								SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE				WATER CONTENT (%) 10 20 30	
178.5			13	SS	1		181					52.930	
			14	SS	3		180					42.940	0 1 63 36
18.3	BEDROCK undifferentiated granitic rock, very dense		15	SS	50/ .0		178						
	Core #1 REC = 100% RQD = 79%		1	CORE			177						
	Core #2 REC = 100% RQD = 92%		2	CORE			176						
174.2							175						
22.6	END OF BOREHOLE AT 22.6m. Piezometer installation consist of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) 21/10/98 2.44(above surface) 22/10/98 2.44(above surface) 23/10/98 2.29(above surface) 24/10/98 2.29(above surface) 25/10/98 2.29(above surface) 26/10/98 2.29(above surface) 27/10/98 2.31(above surface)												

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

RECORD OF BOREHOLE No 98-2

1 OF 2

METRIC

W.P. 812-76-01,398-91-00 LOCATION Laronde Creek, N 5 136 763.4 E 288 612.7 ORIGINATED BY GA
DIST 54 HWY 17 BOREHOLE TYPE Hollow Stem Augers, N Core COMPILED BY WM
DATUM Geodetic DATE 98.10.20 - 98.10.20 CHECKED BY AEG

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	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	
199.6 0.0	SILT, trace clay, very loose to loose, grey to brown, wet (ML)		1	SS	5		199					
			2	SS	3		198					0 0 93 7
			3	SS	2		197					
196.5			4	SS	2		196					
3.0	CLAY, silty, trace sand, low to medium plasticity, firm to stiff, grey, wet (CL-CI)		5	SS	2		195					0 2 54 44
			6	SS	2		194					
			7	SS	2		193					
			8	SS	2		192					
			9	SS	2		191					
	some varves evident		10	SS	1		190					0 0 40 60
	(possibly CL between 10 & 12m)		11	SS	1		189					
			12	SS	1		188					
	(possibly CL between 13 & 15m)						187					
							186					
							185					

Continued Next Page

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

RECORD OF BOREHOLE No 98-2

2 OF 2

METRIC

W.P. 812-76-01,398-91-00 LOCATION Laronde Creek, N 5 136 763.4 E 288 612.7 ORIGINATED BY GA
DIST 54 HWY 17 BOREHOLE TYPE Hollow Stem Augers, N Core COMPILED BY WM
OATUM Geodetic DATE 98.10.20 - 98.10.20 CHECKED BY AEG

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			SHEAR STRENGTH kPa					
								20 40 60 80 100					
								20 40 60 80 100					
			13	SS	1		184					40.00	0 1 44 55
							183						
			14	SS	2							47.310	
							182						
								+					
181.0			15	SS	2		181					○	
18.6	SAND, trace silt, trace gravel, compact to dense, grey, wet						180					○	
			16	SS	30							○	6 84 8 2
178.7							179						
20.9	BEDROCK undifferentiated granitic rock, very dense Core #1 REC = 100% RQD = 0%		1	CORE			178						
	Core #2 REC = 100% RQD = 51%		2	CORE			177						
175.5	Core #3 REC = 100% RQD = 68%		3	CORE			176						
24.0	END OF BOREHOLE AT 24.0m. BOREHOLE GROUTED TO SURFACE.												

+ 3, × 3: Numbers refer to Sensitivity 20 15 10 5 0 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 98-3

1 OF 2

METRIC

W.P. 812-76-01,398-91-00 LOCATION Laronde Creek, N 5 136 733.6 E 288 573.7 ORIGINATED BY GA
 DIST 54 HWY 17 BOREHOLE TYPE Hollow Stem Augers, N Core COMPILED BY WM
 DATUM Geodetic DATE 98.10.24 - 98.10.24 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)			
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	20 40 60 80 100						10 20 30			
197.5 0.0	SILT and SAND, some clay, very loose to loose, dark brown: (FILL)		1	SS	2		197						86.030	0 36 53 11				
			2	SS	7		196											
			3	SS	3													
195.2 2.3	CLAY, silty, trace sand, low to medium plasticity, firm to stiff, grey, wet: (CL-CI) some varves evident (CL between 11 & 13m)		4	SS	1		195						46.890	0 0 46 54				
			5	SS	1		194								61.350			
							193	+								48.090		
							192											
							191									42.420	0 1 38 61	
							190	+								42.430		
							189											
							188										42.550	
							187											0 0 64 36
							186										42.340	
							185										47.70	
							184											
				183														
			12	SS	0								40.550					

Continued Next Page

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

RECORD OF BOREHOLE No 98-4

1 OF 3

METRIC

W.P. 812-76-01,398-91-00 LOCATION Laronde Creek, N 5 136 743.1 E 288 611.5 ORIGINATED BY GA
DIST 54 HWY 17 BOREHOLE TYPE Hollow Stem Augers, N Core COMPILED BY WM
DATUM Geodetic DATE 98.10.22 - 98.10.22 CHECKED BY AEG

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	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
201.4 0.0	SAND and SILT, very loose to loose, brown: (FILL)		1	SS	3		201						
			2	SS	6		200						
			3	SS	4		199						
198.4			4	SS	6		198						
3.0	SILT, clayey, trace sand, low plasticity, soft to firm, grey, wet (CL-ML)		5	SS	6		197						
197.6			6	SS	2		196						
3.8	CLAY, silty, medium plasticity, soft to firm, grey, wet (CL-CI)		7	SS	1		195						
	some varves evident		8	SS	1		194						
			9	SS	1		193						
			10	SS	1		192						
			11	SS	1		191						
			12	SS	1		190						
							189						
							188						
							187						

Continued Next Page

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

METRIC

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE ELEVATION (m)	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT	Liquid Limit	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI C	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES		SHEAR STRENGTH kPa	W			W L
							UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE		WATER CONTENT (%)				
							20 40 60 80 100	10 20 30					
178.7	some varves evident		13	SS	1								
			14	SS	2								
			15	SS	2							43.2%	
			16	SS	1							46.5%	
178.7	SAND, trace gravel, cobbles and boulders, compact to very dense, grey, wet		17	SS	1								
22.7													
			18	SS	27								
			19	SS	50/ .050								
174.5	BEDROCK undifferentiated granitic rock Core #1 REC = 100% RQD = 61% Core #2 REC = 100% RQD = 79% Core #3 REC = 100% RQD = 61%		1	CORE									
26.9													
			2	CORE									
171.5			3	CORE									

Continued Next Page

+ 3, x 3: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 98-4

3 OF 3

METRIC

W.P. 812-76-01,398-91-00 LOCATION Laronde Creek, N 5 136 743.1 E 288 611.5 ORIGINATED BY GA
 DIST 54 HWY 17 BOREHOLE TYPE Hollow Stem Augers, N Core COMPILED BY WM
 DATUM Geodetic DATE 98.10.22 - 98.10.22 CHECKED BY AEG



SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)
								20	40	60	80	100						
30.0	END OF BOREHOLE AT 29.97m. Piezometer installation consist of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) 24/10/98 4.1 25/10/98 4.1 26/10/98 4.3 27/10/98 4.2																	

RECORD OF BOREHOLE No 98-5

1 OF 1

METRIC

W.P. 812-76-01,398-91-00 LOCATION Laronde Creek, N 5 136 767.7 E 288 557.3 ORIGINATED BY GA
 DIST 54 HWY 17 BOREHOLE TYPE Hollow Stem Augers, N Coring COMPILED BY WM
 DATUM Geodetic DATE 98.10.19 - 98.10.19 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								20 40 60 80 100										10 20 30		
						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE														
0.0	SAND and SILT, loose to compact, brown, moist: (FILL)		1	SS	9															
			2	SS	4															
			3	SS	13															
1.9	CLAY, silty, low to medium plasticity, firm to stiff, grey, wet (CL)		4	SS	7												0 1 71 29			
			5	SS	4															
			6	SS	2													0 0 56 44		
5.2	END OF BOREHOLE AT 5.18m. BOREHOLE BACKFILLED WITH DRILL CUTTINGS.																			

RECORD OF BOREHOLE No 98-6

1 OF 1

METRIC

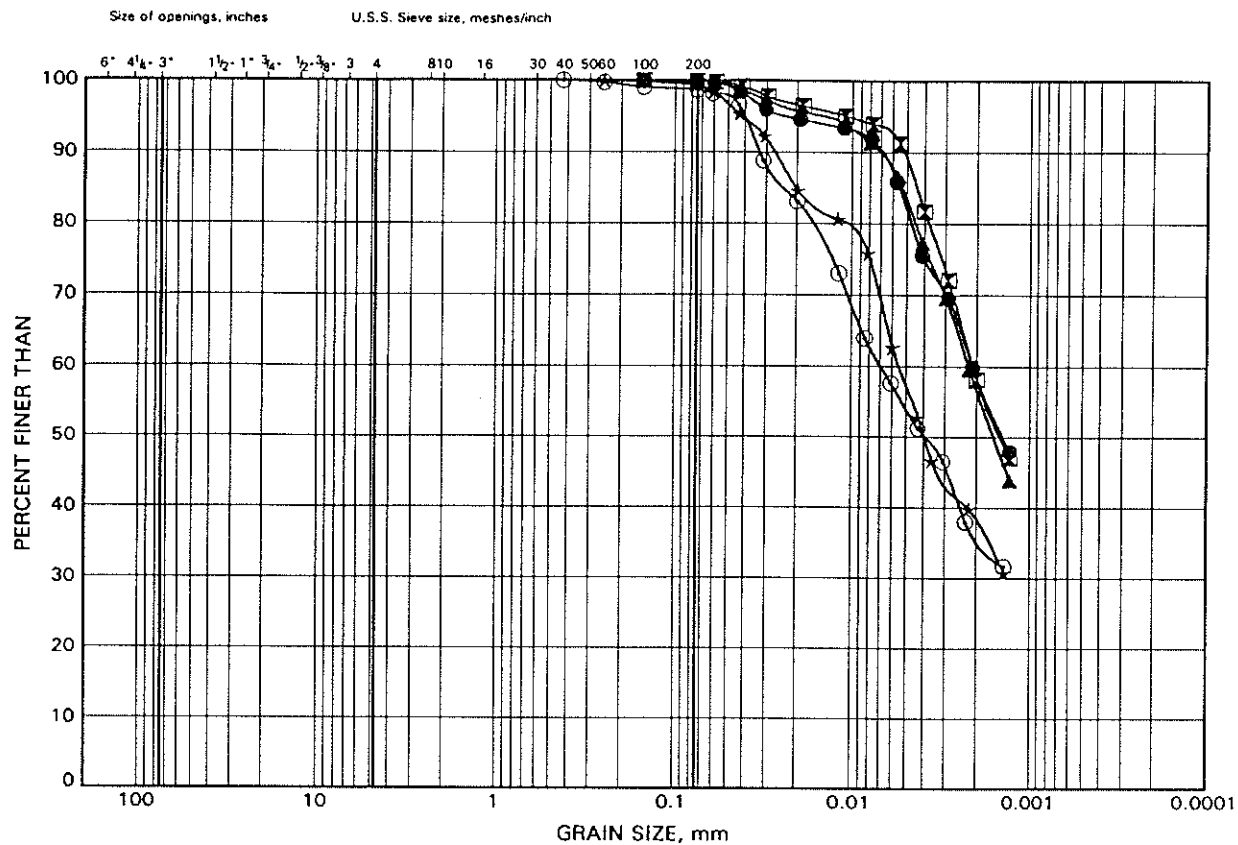
W.P. 812-76-01,398-91-00 LOCATION Laronde Creek, N 5 136 733.4 E 288 556.5 ORIGINATED BY GA
DIST 54 HWY 17 BOREHOLE TYPE Hollow Stem Augers, N Coring COMPILED BY WM
DATUM Geodetic DATE 98.10.26 - 98.10.26 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE												
								● QUICK TRIAXIAL × LAB VANE												
						20	40	60	80	100	10	20	30							
0.0	SILT, sandy, trace gravel, very loose to loose, brown to black, moist to wet: (POSSIBLE FILL) (ML)		1	SS	4															
			2	SS	3															
			3	SS	4															
2.3	SILT, clayey, soft, grey, moist to wet: (ML)		4	SS	4															
			5	SS	2															
4.0	CLAY, silty, medium plasticity, grey, wet: (CI)																			
			6	SS	1															
5.2	END OF BOREHOLE AT 5.18m. BOREHOLE BACKFILLED WITH DRILL CUTTINGS.																			

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

HWY 17, LARONDE CREEK GRAIN SIZE DISTRIBUTION

FIGURE B1



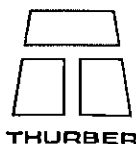
COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL BOREHOLE DEPTH (m) ELEVATION (m)

●	98-1	1.83	194.98
⊠	98-1	2.59	194.22
▲	98-1	7.92	188.89
★	98-1	10.97	185.84
⊙	98-1	17.07	179.74

Date December 1998

Project 812-76-01,398-91-00

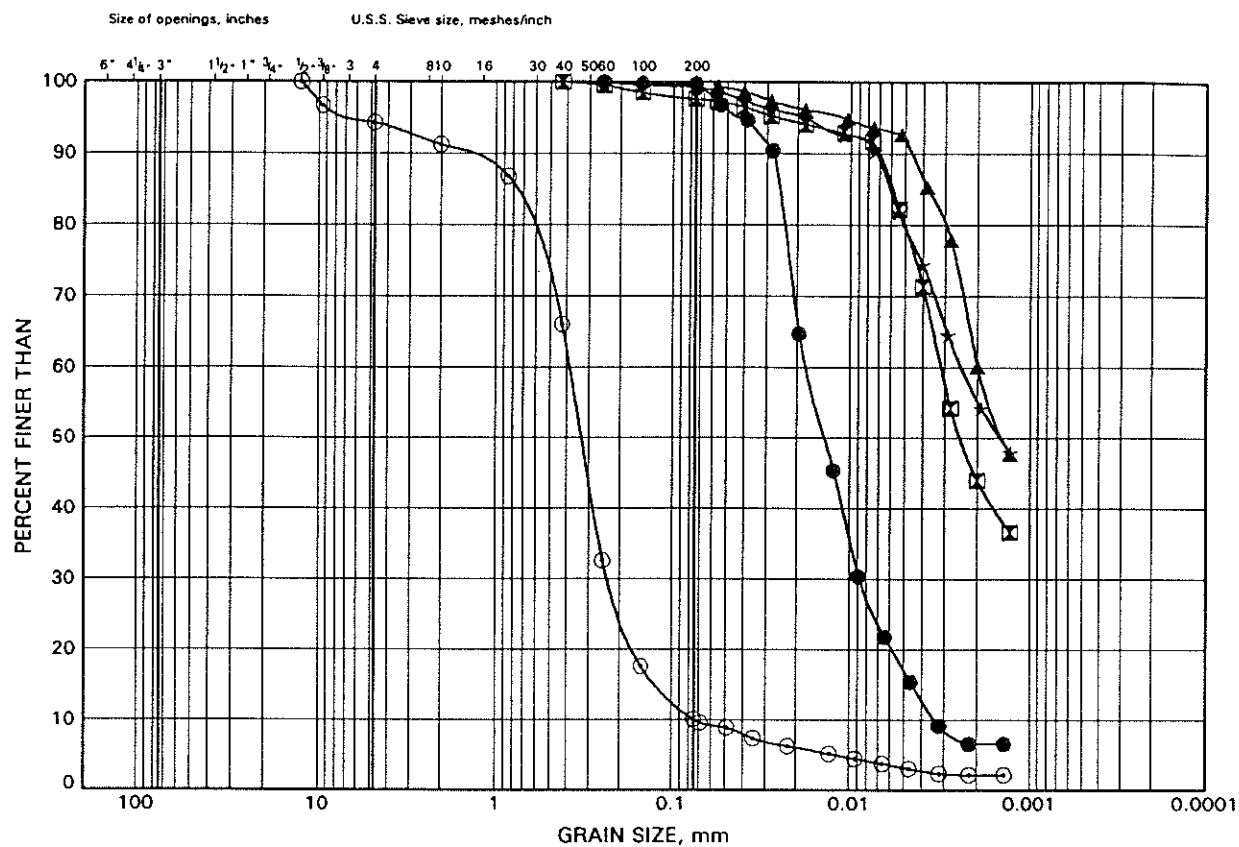


Prep'd WM

Chkd. AEG

HWY 17, LARONDE CREEK GRAIN SIZE DISTRIBUTION

FIGURE B2



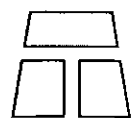
COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL BOREHOLE DEPTH (m) ELEVATION (m)

●	98-2	1.07	198.48
⊠	98-2	3.35	196.20
▲	98-2	9.45	190.10
★	98-2	15.54	184.01
⊙	98-2	20.12	179.43

Date December 1998

Project 812-76-01, 398-91-00



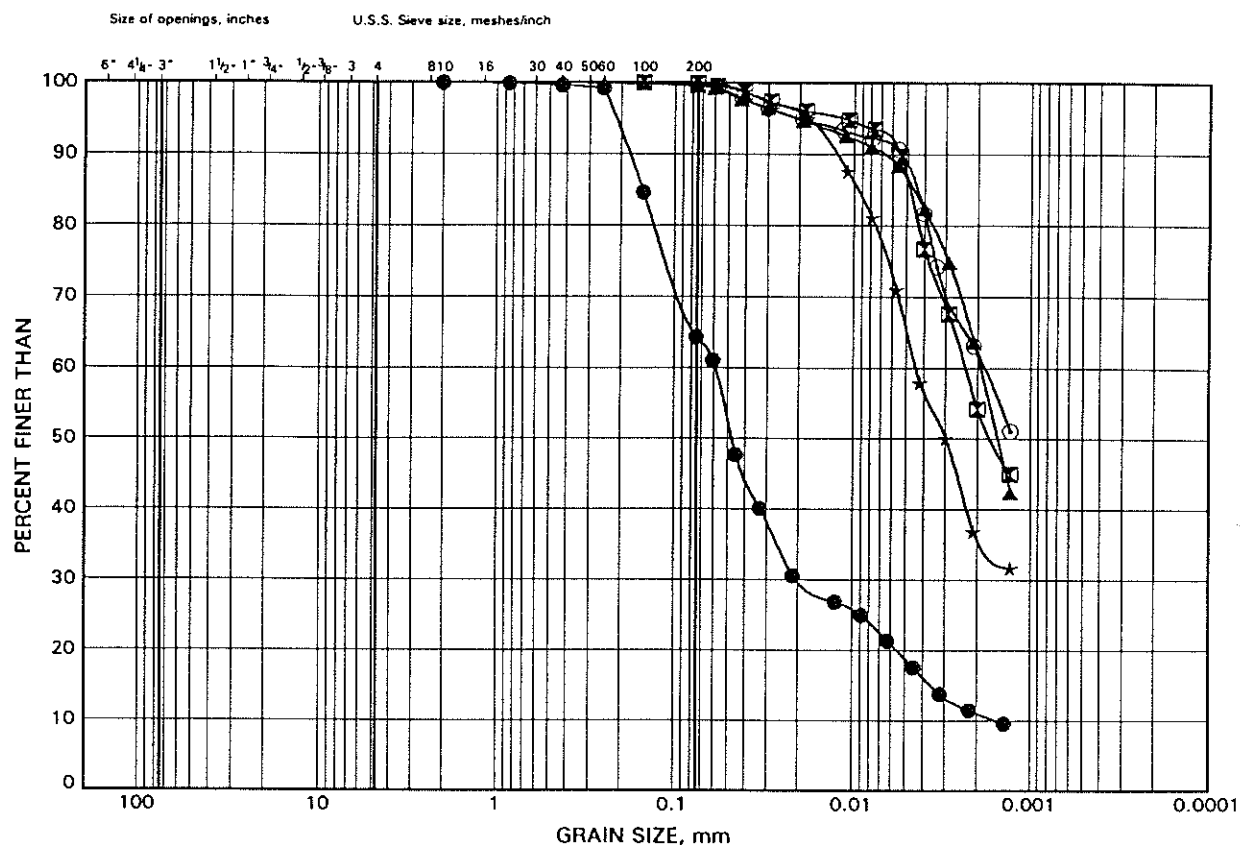
THURBER

Prep'd WM

Chkd. AEG

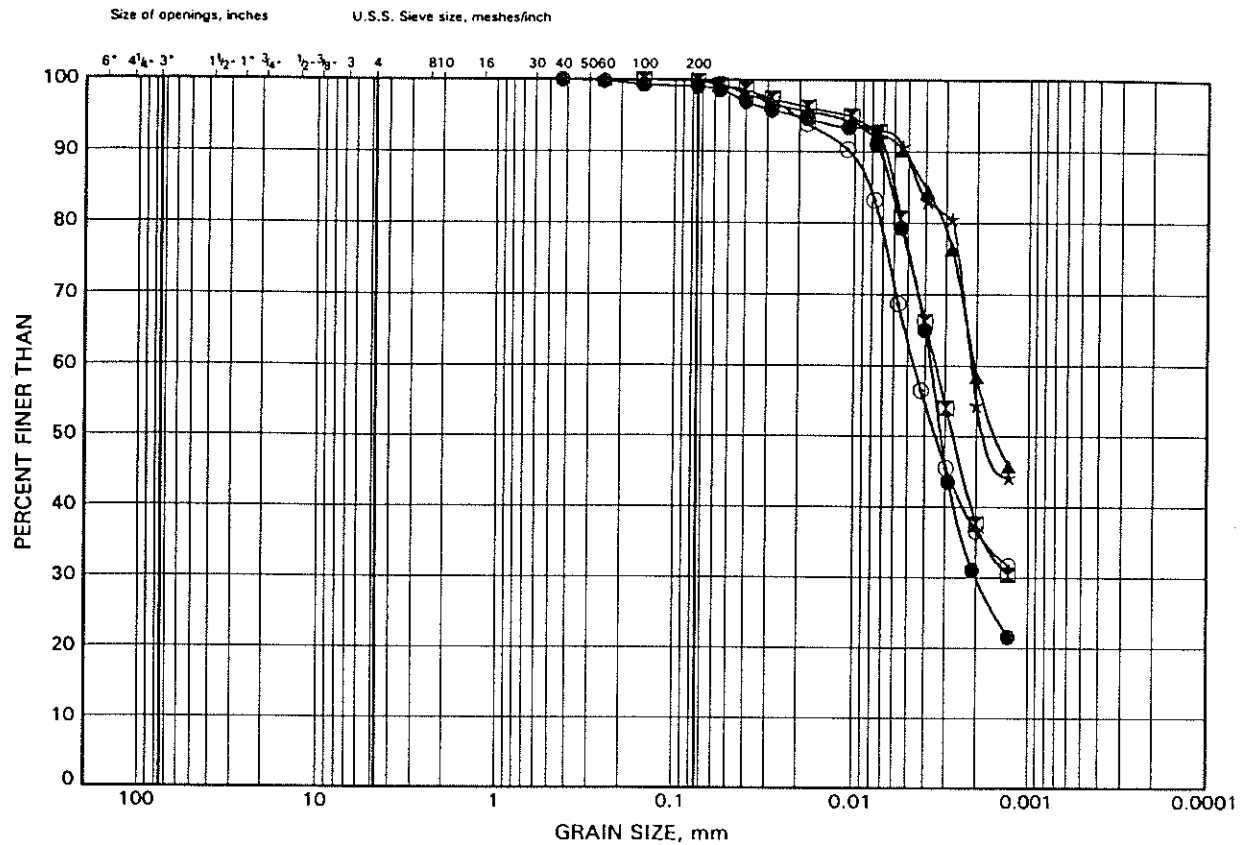
HWY 17, LARONDE CREEK GRAIN SIZE DISTRIBUTION

FIGURE B3



HWY 17, LARONDE CREEK GRAIN SIZE DISTRIBUTION

FIGURE B4

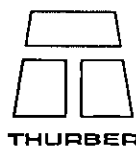


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	98-4	3.35	198.08
⊠	98-4	4.88	196.55
▲	98-4	10.97	190.46
★	98-4	14.02	187.41
⊙	98-4	17.07	184.36

Date December 1998

Project 812-76-01,398-91-00

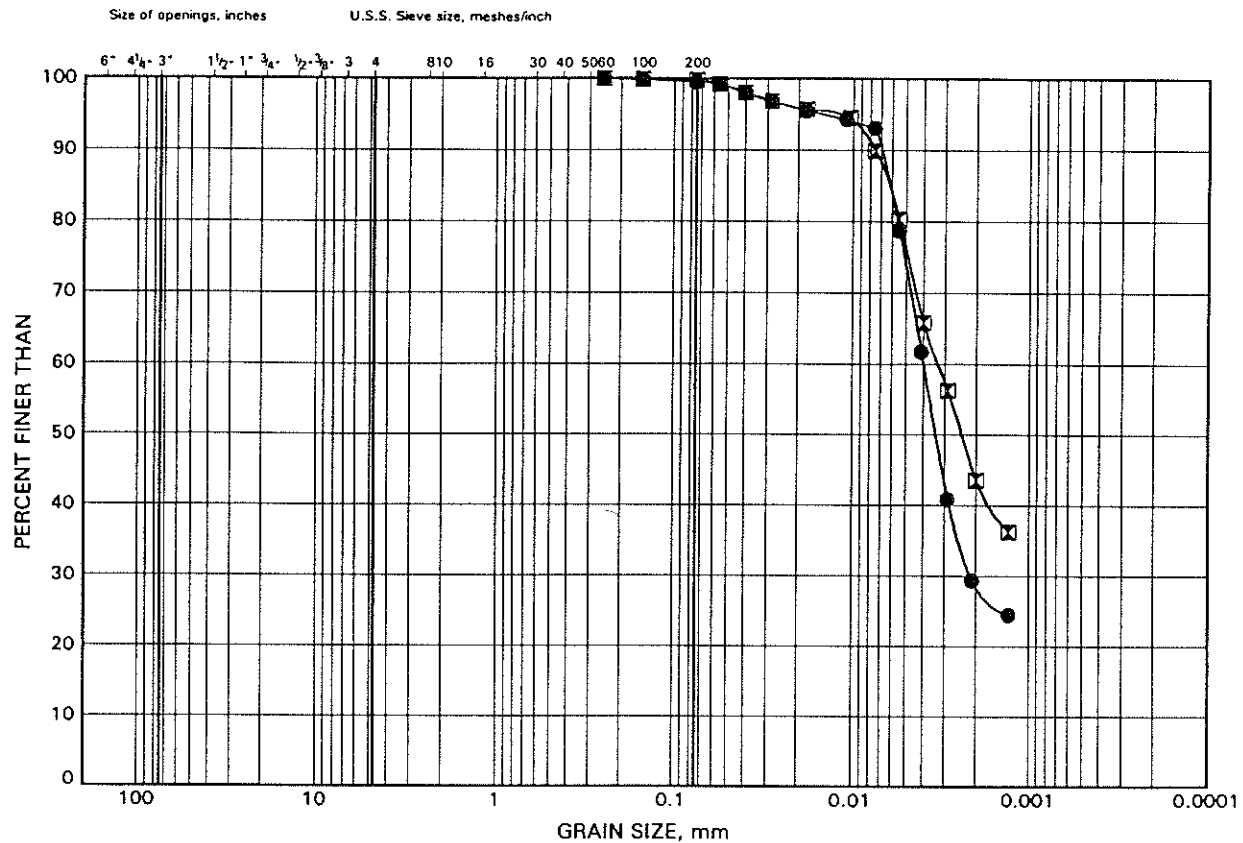


Prep'd WM

Chkd. AEG

HWY 17, LARONDE CREEK GRAIN SIZE DISTRIBUTION

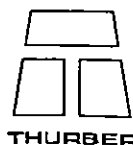
FIGURE B5



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

SYMBOL BOREHOLE DEPTH (m) ELEVATION (m)

● 98-5 2.59
 ☒ 98-5 4.88

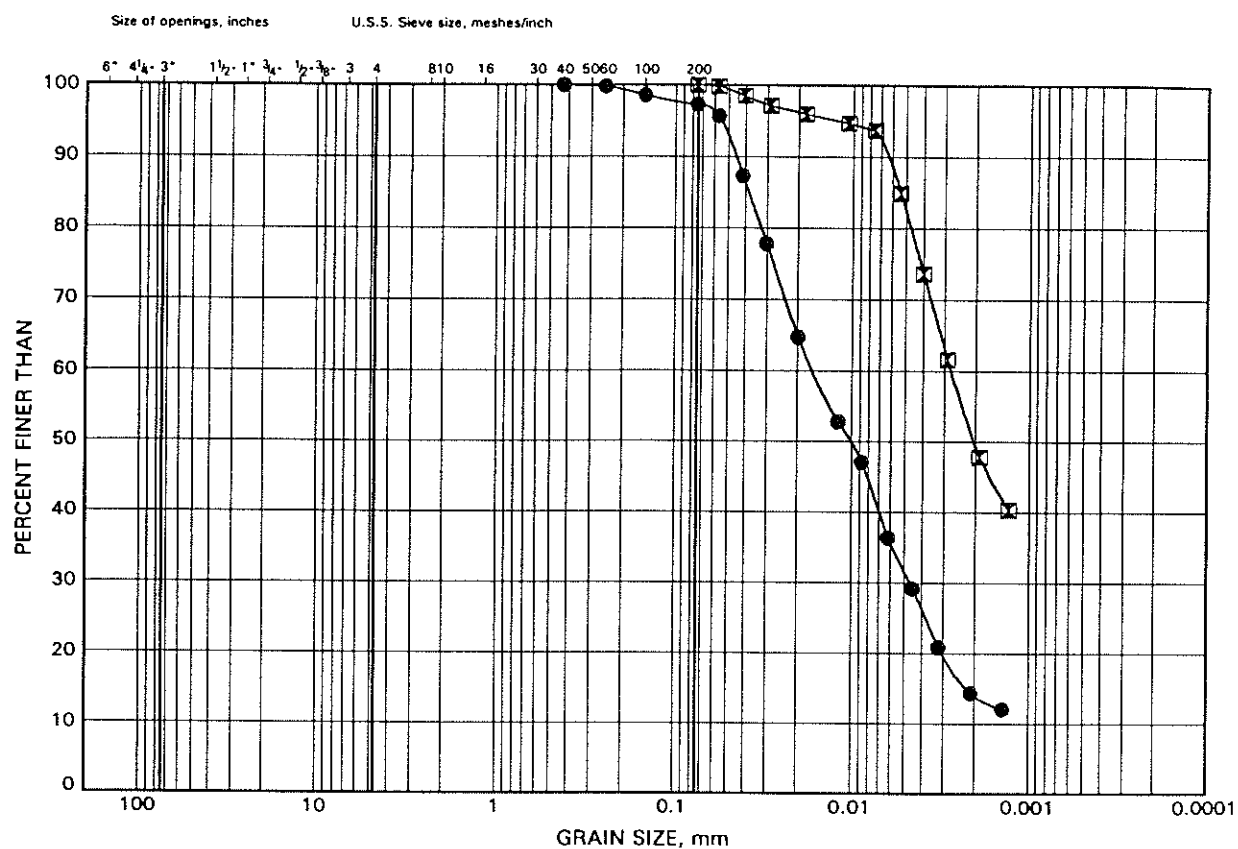


Date December 1998
 Project 812-76-01, 398-91-00

Prep'd WM
 Chkd. AEG

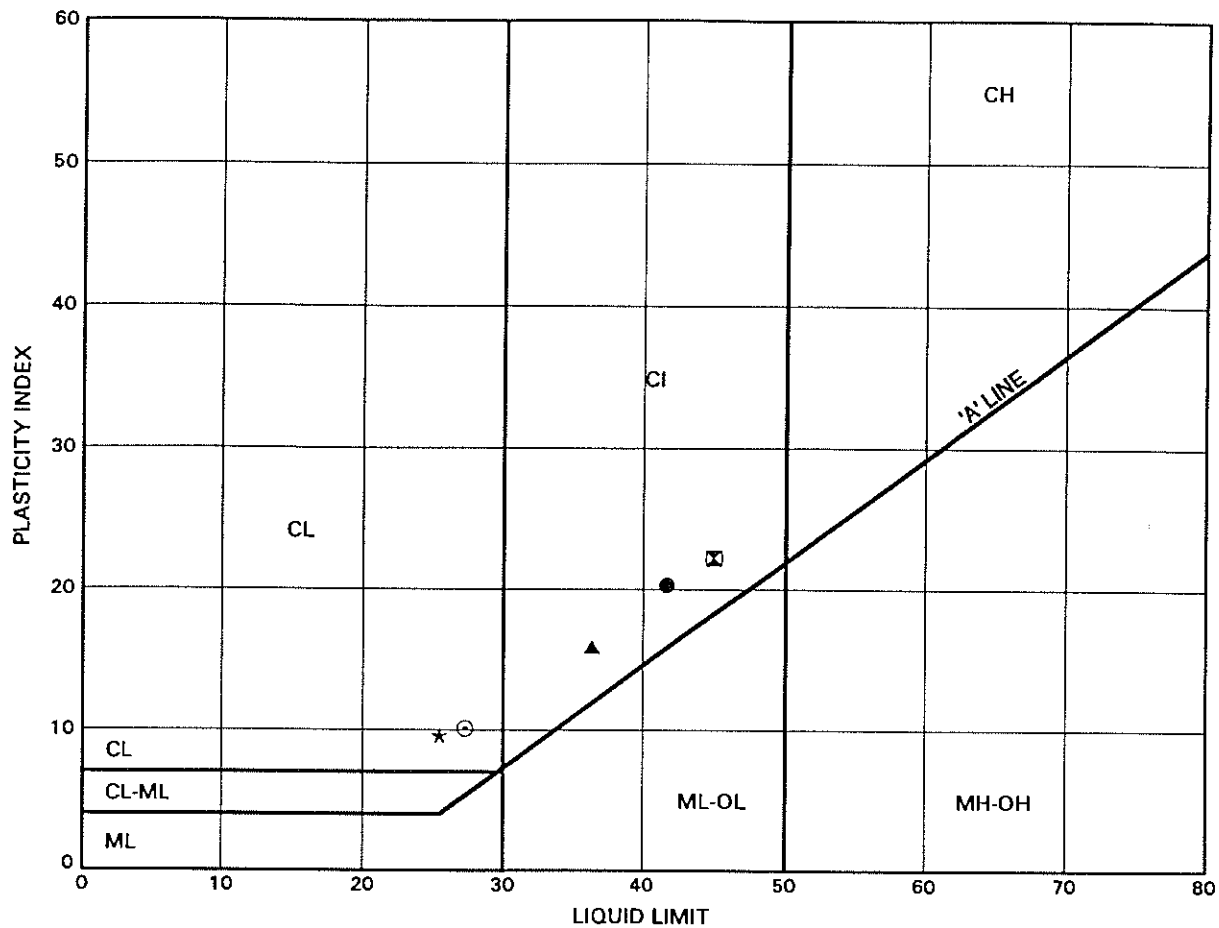
HWY 17, LARONDE CREEK GRAIN SIZE DISTRIBUTION

FIGURE B6



HWY 17, LARONDE CREEK ATTERBERG LIMITS TEST RESULTS

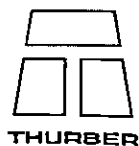
FIGURE B7



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	98-1	1.83	194.98
⊠	98-1	2.59	194.22
▲	98-1	7.92	188.89
★	98-1	10.97	185.84
⊙	98-1	17.06	179.75

Date December 1998

Project 812-76-01,398-91-00

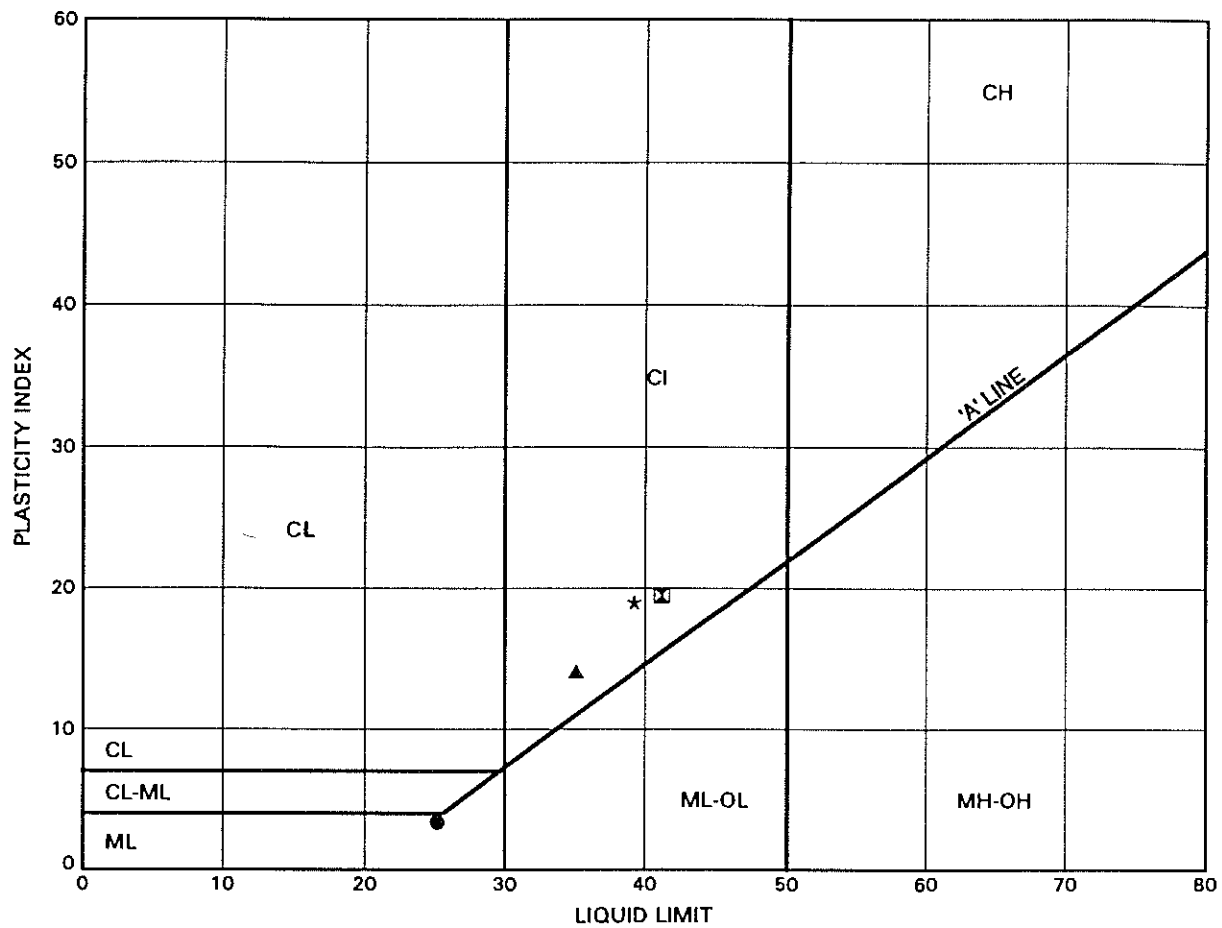


Prep'd WM

Chkd. AEG

HWY 17, LARONDE CREEK ATTERBERG LIMITS TEST RESULTS

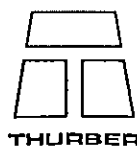
FIGURE B8



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	98-2	1.07	198.48
⊠	98-2	3.35	196.20
▲	98-2	9.45	190.10
★	98-2	15.54	184.01

Date December 1998

Project 812-76-01, 398-91-00

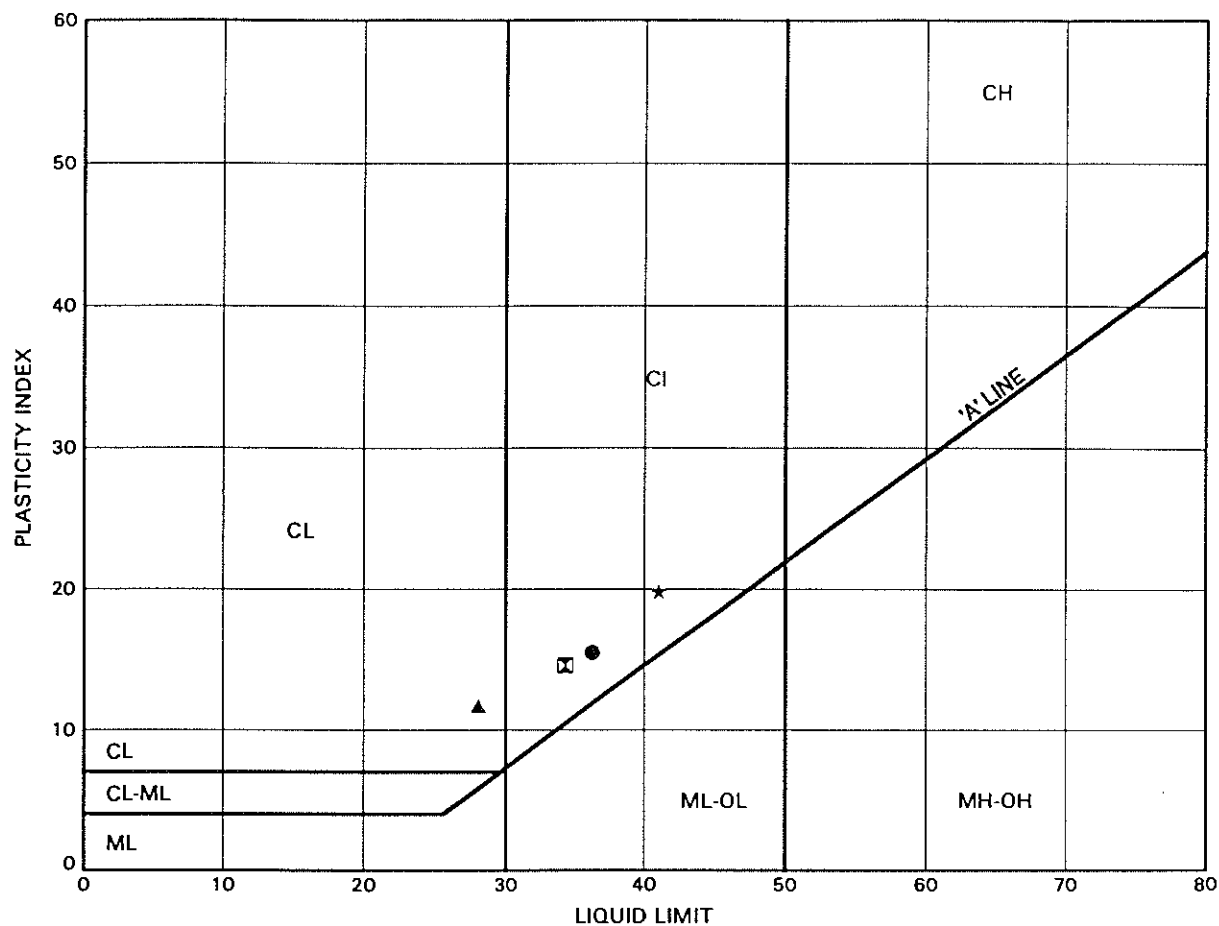


Prep'd WM

Chkd. AEG

HWY 17, LARONDE CREEK ATTERBERG LIMITS TEST RESULTS

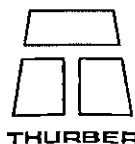
FIGURE B9



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	98-3	2.59	194.86
⊠	98-3	6.40	191.05
▲	98-3	12.50	184.95
★	98-3	17.07	180.38

Date December 1998

Project 812-76-01, 398-91-00

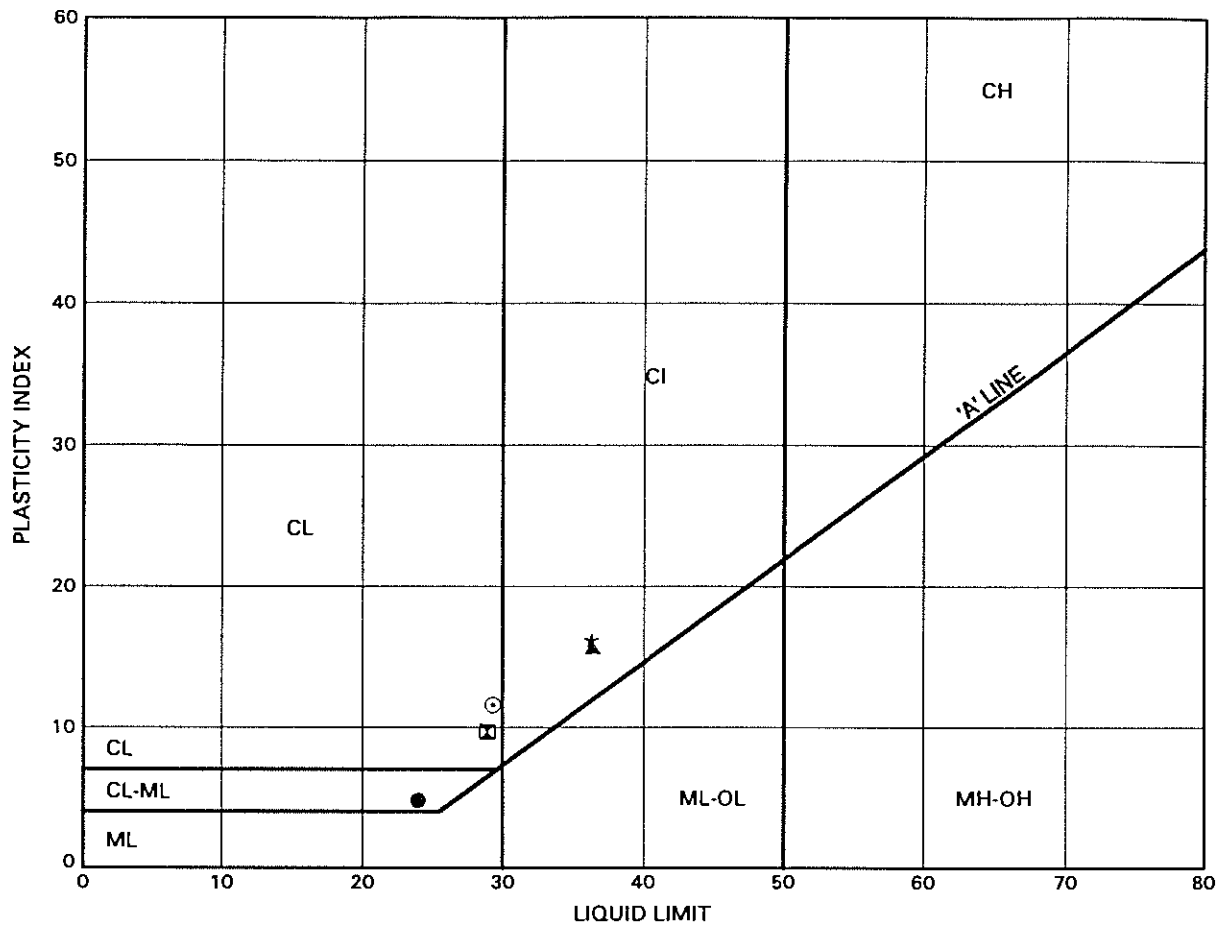


Prep'd WM

Chkd. AEG

HWY 17, LARONDE CREEK ATTERBERG LIMITS TEST RESULTS

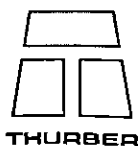
FIGURE B10



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	98-4	3.35	198.08
⊠	98-4	4.88	196.55
▲	98-4	10.97	190.46
★	98-4	14.02	187.41
⊙	98-4	17.07	184.36

Date December 1998

Project 812-76-01,398-91-00

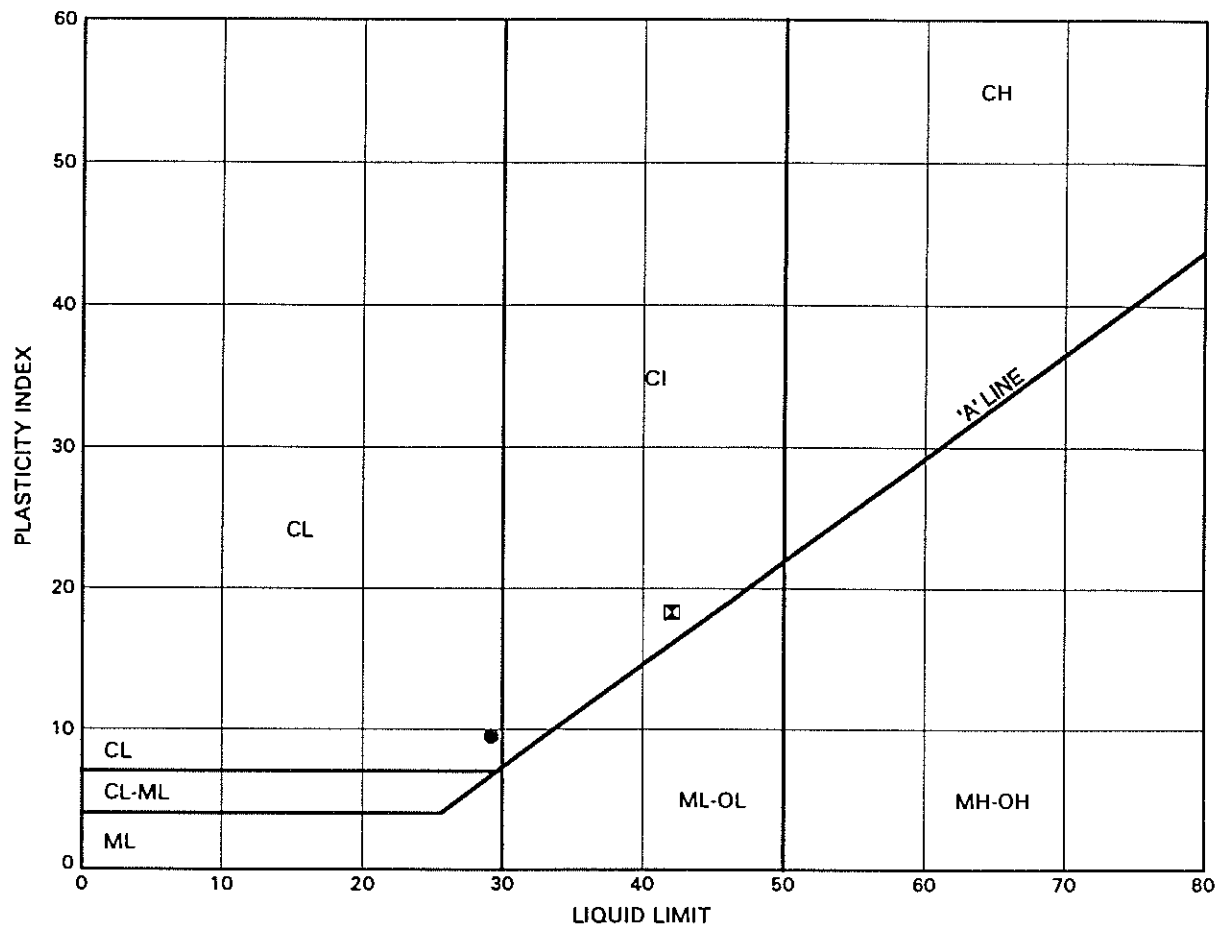


Prep'd WM

Chkd. AEG

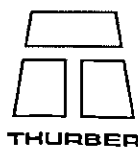
HWY 17, LARONDE CREEK ATTERBERG LIMITS TEST RESULTS

FIGURE B11



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	98-5	2.60	
⊠	98-5	4.88	

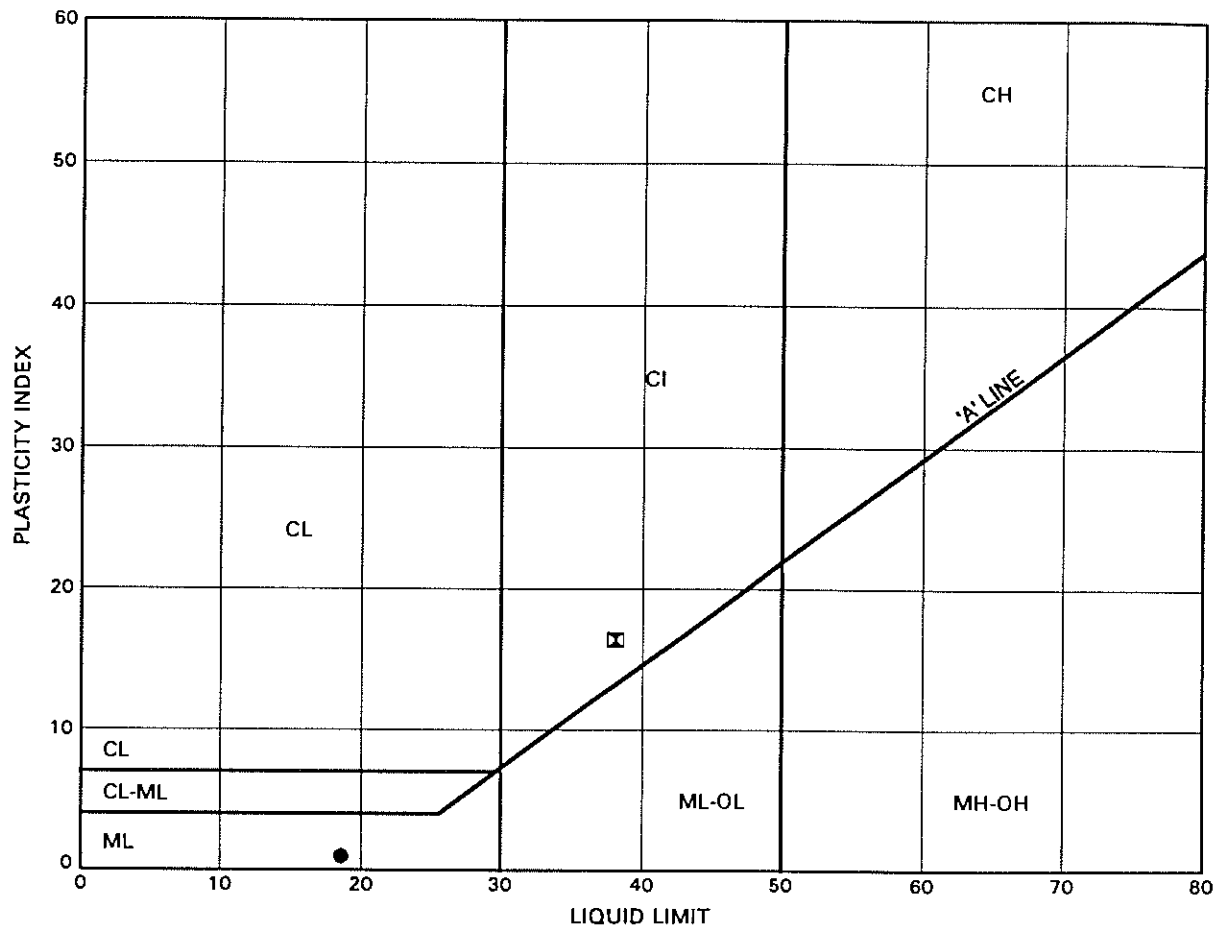
Date December 1998
 Project 812-76-01,398-91-00



Prep'd WM
 Chkd. AEG

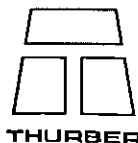
HWY 17, LARONDE CREEK ATTERBERG LIMITS TEST RESULTS

FIGURE B12



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	98-6	1.83	
⊠	98-6	4.88	

Date December 1998
Project 812-76-01, 398-91-00



Prep'd WM
Chkd. AEG

Table 1

Results of pH and Sulphate Testing

Sample	Depth (m)	pH	Sulphates (ppm)
98-2, Sa 3	1.8	8.6	36
98-3, Sa 3	2.0	8.0	7.2

DIMENSIONS ARE IN METERS
AND/OR MILLIMETERS
UNLESS OTHERWISE SHOWN

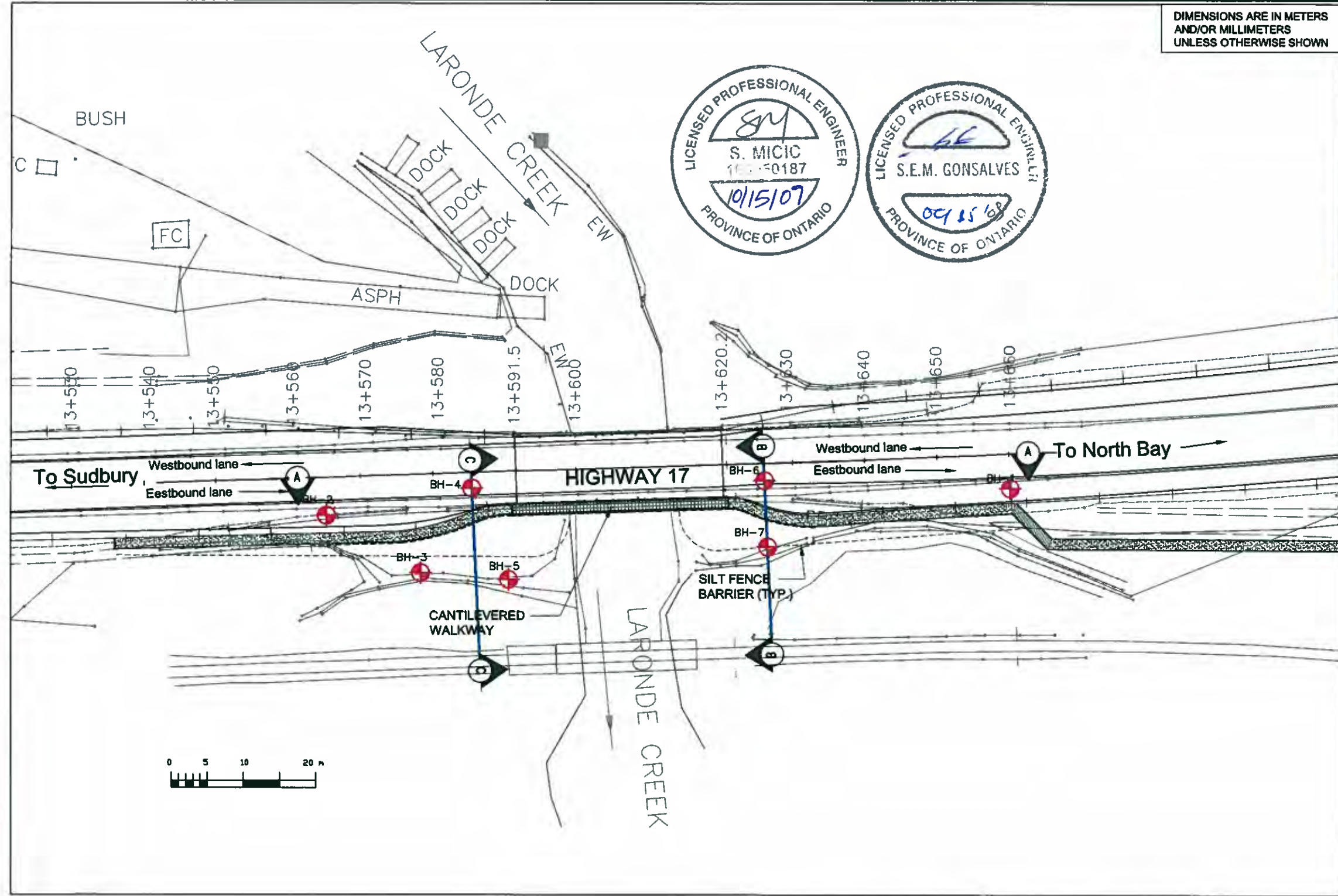
GWP

No. 5274-08-00

SITE PLAN AND
BOREHOLE LOCATIONS

N

SHEET
1



KEY MAP
Not to Scale

LEGEND

- LEGEND
- BOREHOLE
 - Water Level (Piezometer)
 - Water Level (Open hole)

No.	ELEVATION	STATION	OFFSET
BH-1	202.183	13+659.9	4.9
BH-2	201.872	13+565.0	5.3
BH-3	198.224	13+577.7	13.8
BH-4	201.799	13+585.3	2.2
BH-5	196.842	13+590.0	15.2
BH-6	201.824	13+625.9	2.2
BH-7	199.429	13+625.9	11.7

REVISIONS	DATE	BY	DESCRIPTION

Trow Associates Inc.
56 QUEEN STREET EAST, SUIT 301
BRAMPTON, ONTARIO, L6V 4M8
(905) 796-3200

PROJECT TITLE AND LOCATION:

**Gabion Wall Construction
near Laronde Creek
Hwy 17, Sudbury**

DRAWING TITLE:

**SITE PLAN AND
BOREHOLE LOCATIONS**

PROJECT NO.

5274-08-00

DWN.: GQ

SCALE:

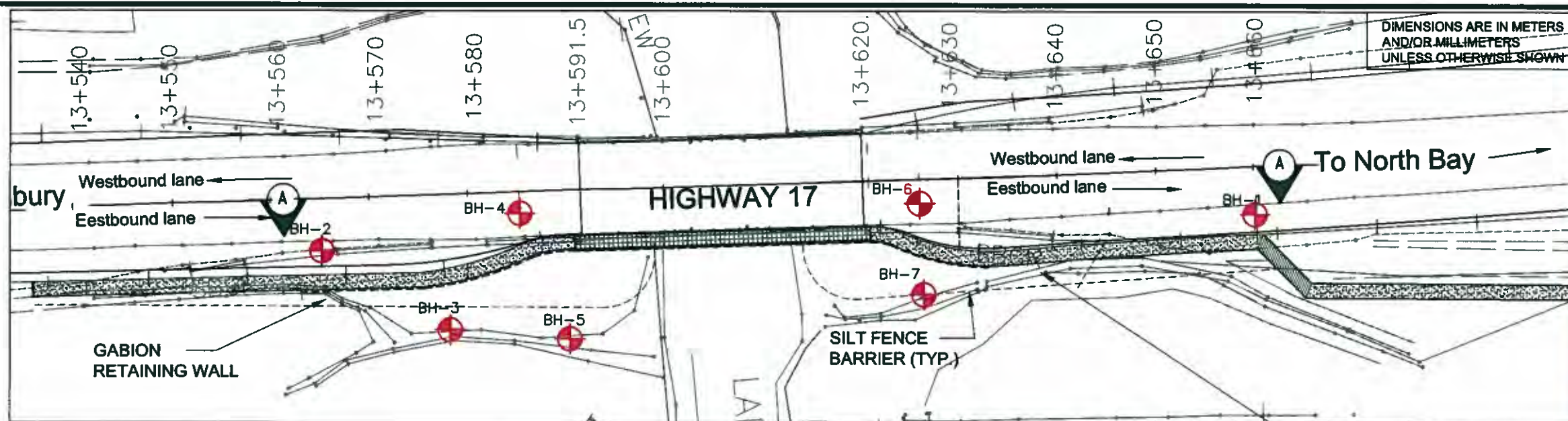
AS NOTED

CHKD.: SM

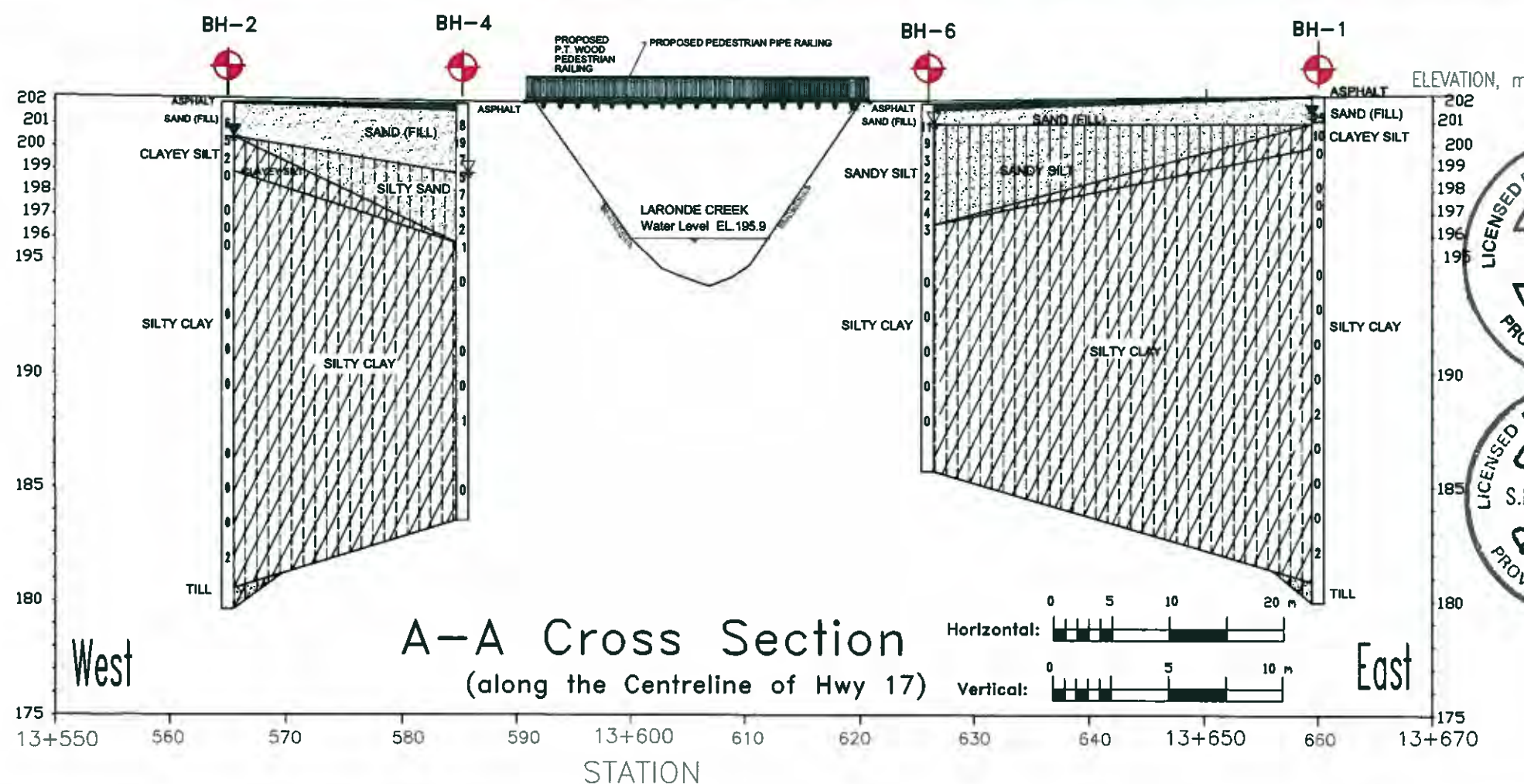
DATE:

Sept. 2009

DWG. No.: 1



KEY MAP
Not to Scale



A-A Cross Section
(along the Centreline of Hwy 17)



LEGEND

- BOREHOLE
- Water Level (Piezometer)
- Water Level (Open hole)

No.	ELEVATION	STATION	OFFSET
BH-1	202.183	13+659.9	4.9
BH-2	201.872	13+565.0	5.3
BH-3	198.224	13+577.7	13.8
BH-4	201.799	13+585.3	2.2
BH-5	196.842	13+590.0	15.2
BH-6	201.824	13+625.9	2.2
BH-7	199.429	13+625.9	11.7

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 to be read with subject report.
3. This drawing is for subsurface information only. Surface details and features are for conceptual illustration only.
4. Borehole locations are approximate.
5. Borehole elevations should not be used to design building(s), or floor slab(s), or parking lot(s) grades.
6. The elevation of the water level in the creek was measured by TROW on 11/Sept./2009.

REVISIONS	DATE	BY	DESCRIPTION

SOIL STRATA SYMBOLS:

ASPHALT	SAND	SILTY CLAY
SILTY SAND	CLAYEY SILT	TILL

TROW Associates Inc.
56 QUEEN STREET EAST, SUIT 301
BRAMPTON, ONTARIO, L6V 4M8
(905) 796-3200

PROJECT TITLE AND LOCATION:
**Gabion Wall Construction
near Laronde Creek
Hwy 17, Sudbury**

DRAWING TITLE:
A-A CROSS-SECTION

PROJECT NO. 5274-08-00	DWN.: GQ
SCALE: AS NOTED	CHKD.: SM
DATE: Sept. 2009	DWG. No.: 2



DIMENSIONS ARE IN METERS
AND/OR MILLIMETERS
UNLESS OTHERWISE SHOWN

GWP No. 5274-08-00

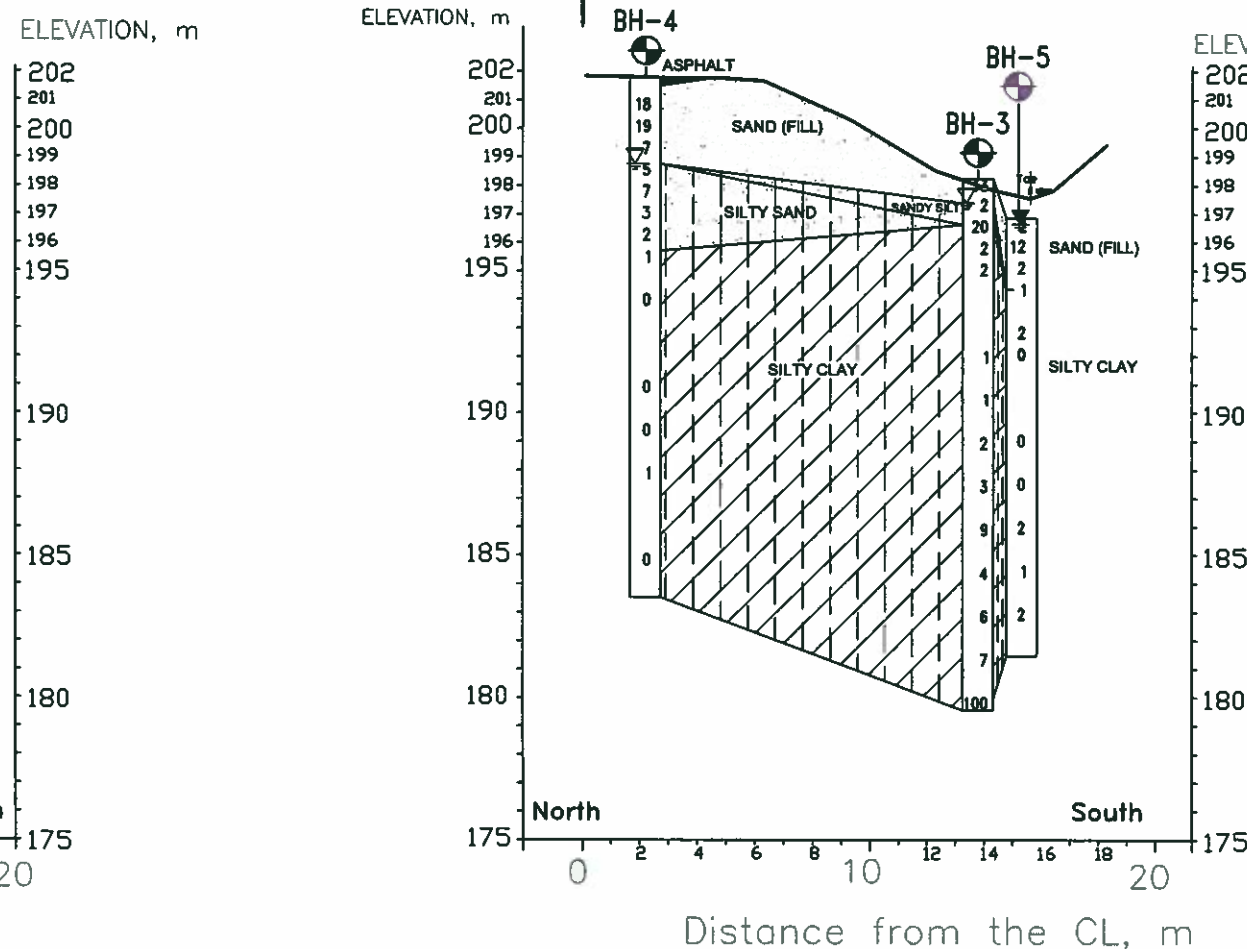
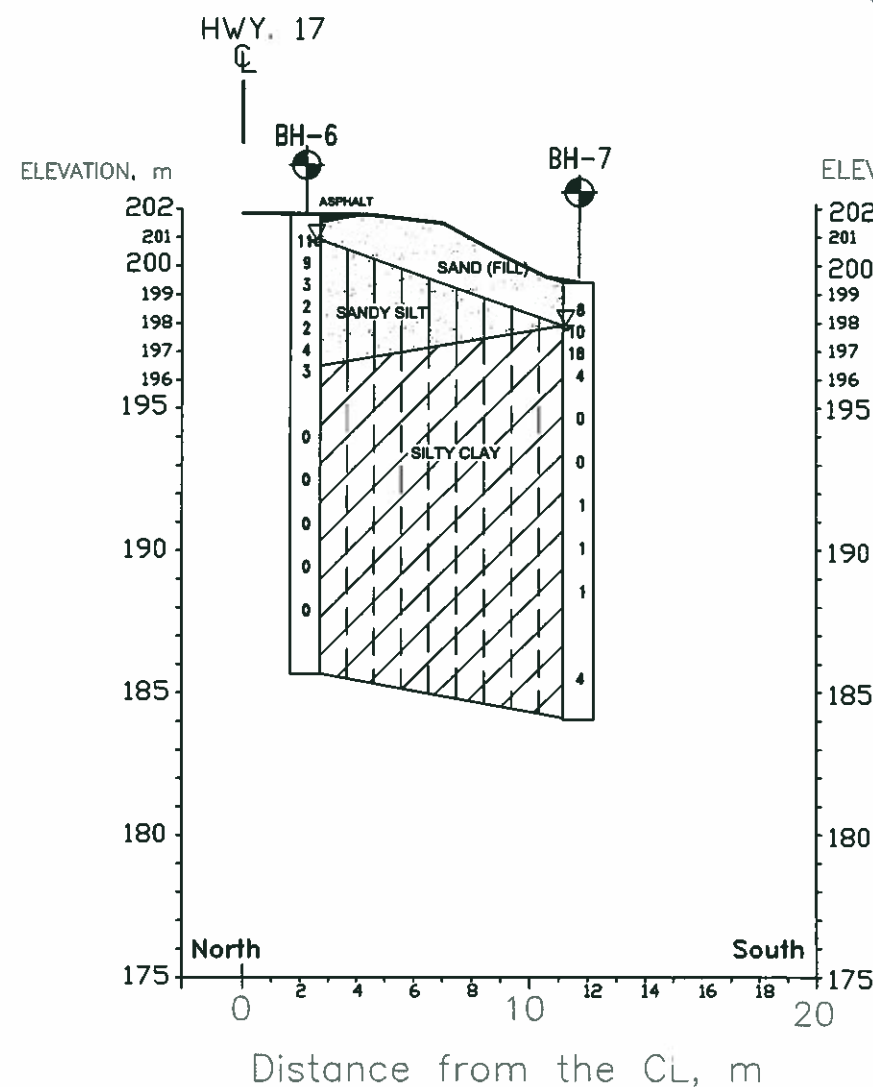


SHEET
3

Cross Section B-B
(at Station ~13+626)



Cross Section C-C
(at Station ~13+585)



KEY MAP
Not to Scale

LEGEND

- BOREHOLE
- Water Level (Piezometer)
- Water Level (Open hole)

No.	ELEVATION	STATION	OFFSET
BH-1	202.183	13+659.9	4.9
BH-2	201.872	13+565.0	5.3
BH-3	198.224	13+577.7	13.8
BH-4	201.799	13+585.3	2.2
BH-5	196.842	13+590.0	15.2
BH-6	201.824	13+625.9	2.2
BH-7	199.429	13+625.9	11.7

-NOTES-

- The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- This drawing to be read with subject report.
- This drawing is for subsurface information only. Surface details and features are for conceptual illustration only.
- Borehole locations are approximate.
- Borehole elevations should not be used to design building(s), or floor slab(s), or parking lot(s) grades.
- The elevation of the water level in the creek was measured by TROW on 11/Sept./2009

REVISIONS	DATE	BY	DESCRIPTION

SOIL STRATA SYMBOLS:

ASPHALT	SAND	SILTY CLAY
SILTY SAND	CLAYEY SILT	TILL

TROW Associates Inc.
56 QUEEN STREET EAST, SUIT 301
BRAMPTON, ONTARIO, L6V 4M8
(905) 796-3200

PROJECT TITLE AND LOCATION:
**Gabion Wall Construction
near Laronde Creek
Hwy 17, Sudbury**

DRAWING TITLE:
**CROSS-SECTIONS
B-B and C-C**

PROJECT NO. 5274-08-00	DWN.: GQ
SCALE: AS NOTED	CHKD.: SM
DATE: Sept. 2009	DWG. No.: 3

RECORD OF BOREHOLE No BH-1

1 OF 2

METRIC

W.P. 5274-08-00 LOCATION Laronde Creek, Nipissing Indian Reserve No. 10 ORIGINATED BY CS
DIST 54 HWY 17 BOREHOLE TYPE CME 200mm OI Hollow Stem Auger COMPILED BY KR
DATUM Geodetic DATE 09.9.3 CHECKED BY IM

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								WATER CONTENT (%)		
								○ UNCONFINED	+	FIELD VANE								
								● QUICK TRIAXIAL	×	LAB VANE								
202.2							20	40	60	80	100	10	20	30	GR	SA	SI	CL
200.9	ASPHALT, (~ 50 mm)		1	AS											64	30	(6)	
201.0	SAND (FILL) (SW), brown, damp, well graded, compact, fine to coarse grained, some gravel, trace to some silt.		2	SS	25													
1.2	Clayey SILT (ML), grey, damp to wet, compact, poorly graded, some fine grained sand.		3	SS	10													
199.9	SILTY CLAY (CL), grey, saturated, low plasticity, soft to stiff.		4	SS	0													
			5	TW														
			6	SS	0													
			7	SS	0													
			8	SS	0													
			9	TW														
			10	SS	0													
	varved below ~ 7.62 m depth.		11	SS	0													
			12	SS	0													
			13	SS	0													
			14	SS	2													
	brown/grey below ~ 13.72 m depth.		15	SS	0													
	with silt seems below ~ 15.24 m depth.		16	SS	0													
			17	SS	0													
	no silt seems below ~ 16.76 m depth.																	

Continued Next Page

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

ON_MOT_SO11878G - LARONDE CREEK BRIDGE BY GREG & GPJ ON_MOT_GDT 09/10/21

METRIC

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/m³	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	WATER CONTENT (%)			
	trace to some fine grained sand below ~ 19.81 m depth.		18	SS	2		182							
180.8								181						
21.3	SILTY SAND TILL , grey, wet, compact, poorly graded, trace to some gravel.			19	BAG									
179.9 22.3	BOREHOLE TERMINATED AT ~ 22.25 m DEPTH DUE TO AUGER REFUSAL ON SUSPECTED BEDROCK NOTES: 1. This drawing is to be read with the subject report and project number as presented above. 2. Interpretation assistance by Trow is required before use by others. 3. Date of W.L.=Sept. 11, 2009. 4. Installed monitoring well to 12.2 m depth.						180							

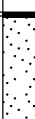



+³, ×³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No BH-2

1 OF 2

METRIC

W.P. 5274-08-00 LOCATION Laronde Creek, Nipissing Indian Reserve No. 10 ORIGINATED BY CS
 DIST 54 HWY 17 BOREHOLE TYPE CME 200mm OI Hollow Stem Auger COMPILED BY KR
 DATUM Geodetic DATE 09.9.4 CHECKED BY IM

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							WATER CONTENT (%)	
201.9	ASPHALT, (~ 50 mm) SAND (FILL) (SW), brown, damp, loose, well graded, fine to coarse grained, some fine to coarse gravel, trace silt. - some silt below 0.8 m		1	AS			201								(Gs=2.726) 0 1 52 47	
200.8			2	SS	6		200									
200.4	CLAYEY SILT (ML), grey, wet, very loose to loose, trace sand, trace to some clay. very loose below ~ 2.29 m depth.		3	SS	5		199									(Gs=2.721) 0 2 25 73
1.5			4	SS	2		198									
198.8	SILTY CLAY (CI), grey, saturated, medium plasticity, firm to stiff.		5	SS	0		197									
			6	TW			196									
			7	SS	0		195									
			8	SS	0		194									
			9	SS	0		193									
			10	TW			192									
			11	SS	0		191									
			12	SS	0		190									
			13	SS	0		189									
			14	TW			188									
			15	SS	0		187									
			16	SS	0		186									
			17	SS	0		185									
									184							
									183							
						182										

Continued Next Page

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


ON_MOT_SO11878G - LARONDE CREEK BRIDGE BY GREG & GPJ ON_MOT_GDT 09/10/21

RECORD OF BOREHOLE No BH-2

2 OF 2

METRIC

W.P. 5274-08-00 LOCATION Laronde Creek, Nipissing Indian Reserve No. 10 ORIGINATED BY CS
 DIST 54 HWY 17 BOREHOLE TYPE CME 200mm OI Hollow Stem Auger COMPILED BY KR
 DATUM Geodetic DATE 09.9.4 CHECKED BY IM

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																	
			18	SS	2		20	40	60	80	100						
180.5																	
21.3	HARD AUGERING, suspected sand and gravel till.		19	BAG													
179.6																	
22.3	BOREHOLE TERMINATED AT ~ 22.25 m DEPTH DUE TO AUGER REFUSAL ON SUSPECTED BEDROCK																
	NOTES: 1. This drawing is to be read with the subject report and project number as presented above. 2. Interpretation assistance by Trow is required before use by others. 3. Date of W.L.=Sept. 11, 2009. 4. Installed monitoring well to 12.2 m depth.																

RECORD OF BOREHOLE No BH-3

1 OF 1

METRIC

W.P. 5274-08-00 LOCATION Laronde Creek, Nipissing Indian Reserve No. 10 ORIGINATED BY CS
 DIST 54 HWY 17 BOREHOLE TYPE Hollow Stem Auger (Wash Boring) COMPILED BY KR
 DATUM Geodetic DATE 09.9.8 CHECKED BY IM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)	
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE							
198.2	TOPSOIL (~76mm) over SAND (FILL) (SW) , brown, damp, compact, poorly graded, fine to coarse grained, some silt, trace to some gravel. SANDY SILT (SM) , grey, wet, very loose, some gravel. SILTY CLAY (CI-MI) , brown, saturated, medium plasticity, soft to stiff. grey below ~ 3.05 m depth.		1	SS	13		198									
197.4			2	SS	2		197									
0.8							196									
196.6		3	SS	20	196											
1.6		4	SS	2	195											
		5	SS	2	194			5.3								
		6	TW		193			5.3								
		7	SS	1	192			5.3								
		8	SS	1	191			5.3								
		9	SS	2	190			2.7								
		10	SS	3	189			2.8								
		11	SS	9	188			4.0								
		12	SS	4	187											
		13	SS	6	186											
		14	SS	7	185											
179.6	BOREHOLE TERMINATED AT ~ 18.67 m DEPTH DUE TO SPT REFUSAL ON SUSPECTED BEDROCK		15	SS	100		180									
18.7																

+³, X³: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

ON_MOT_SO11878G - LARONDE CREEK BRIDGE BY GREG & GPJ ON_MOT.GDT 09/10/21

RECORD OF BOREHOLE No BH-4

1 OF 2

METRIC

W.P. 5274-08-00 LOCATION Laronde Creek, Nipissing Indian Reserve No. 10 ORIGINATED BY CS
 DIST 54 HWY 17 BOREHOLE TYPE CME 200mm OI Hollow Stem Auger COMPILED BY KR
 DATUM Geodetic DATE 09.9.9 CHECKED BY IM

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			SHEAR STRENGTH kPa							WATER CONTENT (%)
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE						
201.8								20 40 60 80 100							
200.9	ASPHALT, (~ 300 mm)														
0.3	SAND (FILL) (SW), brown, damp, loose to compact, poorly graded, fine grained, trace to some silt, trace gravel, with silt below ~ 0.76 m depth. HARD AUGERING		1	AS	18		201								
			2	SS											
			3	SS	19		200								
	No Sample Recovery		4	SS	7		199								
198.7	SILTY SAND(SM) , brown, damp to wet, loose, trace clay.		5	SS	5		198								
3.1			6	SS	7										
	brown to grey, very loose below ~ 4.57 m depth.		7	SS	3		197								
	brown, wet, fine to medium grained, trace organics below ~ 5.33 m depth.		8	SS	2		196								
195.7	SILTY CLAY (CL), grey, saturated, firm to stiff, low plasticity		9	SS	1		195								
6.1			10	SS	0		194								
			11	TW			193								
			12	SS	0		192								
			13	SS	0		191								
			14	SS	1		190								
	varved below ~ 12.19 m depth.		15	TW			189								
			16	SS	0		188								
							187								
							186								
	brown/grey, with silt seems below ~ 16.76 m depth.						185								
183.5	BOREHOLE TERMINATED AT ~ 18.28 m DEPTH						184								
18.3	NOTES: 1. This drawing is to be read with the subject report and project number as presented above.														

Continued Next Page

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

ON_MOT_S011878G - LARONDE CREEK BRIDGE BY GREG & GPJ ON_MOT_GDT 09/10/21

RECORD OF BOREHOLE No BH-4

2 OF 2

METRIC

W.P. 5274-08-00 LOCATION Laronde Creek, Nipissing Indian Reserve No. 10 ORIGINATED BY CS
 DIST 54 HWY 17 BOREHOLE TYPE CME 200mm OI Hollow Stem Auger COMPILED BY KR
 DATUM Geodetic DATE 09.9.9 CHECKED BY IM

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					W _p	W			
	2. Interpretation assistance by Trow is required before use by others. 3. Date of W.L.=Sept. 11, 2009. 4. Installed PVC standpipes to 12.2 m depth.																

RECORD OF BOREHOLE No BH-5

1 OF 1

METRIC

W.P. 5274-08-00 LOCATION Laronde Creek, Nipissing Indian Reserve No. 10 ORIGINATED BY GQ
 DIST 54 HWY 17 BOREHOLE TYPE Hollow Stem Auger (Wash Boring) COMPILED BY GQ
 DATUM Geodetic DATE 09.9.9 CHECKED BY VD

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE						
196.8							20 40 60 80 100								
0.0	TOPSOIL, (~ 15 mm) over SAND (FILL) (SW) , some silt, trace rootlets and wood deris. brown, damp to wet, very loose to compact, fine grained. - a thin (0.15 m) layer of silty clay at a depth of about 0.9 m - become wet below 1.05 m		1	SS	2										
			2	SS	12										
			3	SS	2										
194.4															
2.5	SILTY CLAY (CL) , varved, grey, saturated, soft to stiff, low plasticity		4	SS	1										
			5	SS	2			5.3							
			6	SS	0										
								2.7							
			7	TW											
								4.5							
			8	SS	0										
								2.4							
			9	SS	0										
								3.1							
			10	SS	2										
								3.1							
			11	SS	1										
								2.1							
			12	SS	2										
								2.2							
								4.0							
181.5															
15.4	BOREHOLE TERMINATED AT ~ 15.4 m DEPTH														
	NOTES: 1. This drawing is to be read with the subject report and project number as presented above. 2. Interpretation assistance by Trow is required before use by others. 3. Date of W.L.=Sept. 11, 2009. 4. Installed monitoring well to 11.2 m depth.														

ON_MOT_SO11878G - LARONDE CREEK BRIDGE BY GREG & GPJ ON_MOT.GDT 09/10/21

RECORD OF BOREHOLE No BH-6

1 OF 1

METRIC

W.P. 5274-08-00 LOCATION Laronde Creek, Nipissing Indian Reserve No. 10 ORIGINATED BY CS
 DIST 54 HWY 17 BOREHOLE TYPE CME 200mm OI Hollow Stem Auger COMPILED BY KR
 DATUM Geodetic DATE 09.9.10 CHECKED BY IM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)	
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE							
201.8	ASPHALT, (~ 300 mm)					▽	20	40	60	80	100	10	20	30	(Gs=2.681) 10 33 50 7	
200.9	SAND (FILL) (SW), brown, damp, fine to coarse grained, trace fine grained gravel, some silt.		1	AS			20	40	60	80	100	10	20	30		
0.3	SANDY SILT(SM), brown, wet, very loose to compact, trace fine to coarse grained gravel. very loose below ~ 2.57 m depth. trace clay below ~ 3.05 m depth. clayey below ~ 4.57 m depth.		2	SS	11		20	40	60	80	100	10	20	30		
200.9			3	SS	9		20	40	60	80	100	10	20	30		
0.9			4	SS	3		20	40	60	80	100	10	20	30		
			5	SS	2		20	40	60	80	100	10	20	30		
			6	SS	2		20	40	60	80	100	10	20	30		
			7	SS	4		20	40	60	80	100	10	20	30		
			8	SS	3		20	40	60	80	100	10	20	30		
196.5	SILTY CLAY (CL), grey, saturated, low plasticity, firm to stiff		9	TW			20	40	60	80	100	10	20	30		(Gs=2.716) 0 2 48 50
5.3			10	SS	0		20	40	60	80	100	10	20	30		
			11	SS	0		20	40	60	80	100	10	20	30		
			12	SS	0		20	40	60	80	100	10	20	30		
			13	SS	0		20	40	60	80	100	10	20	30		
			14	SS	0		20	40	60	80	100	10	20	30		
			15	TW		20	40	60	80	100	10	20	30			
185.7		BOREHOLE TERMINATED AT ~ 16.15 m DEPTH					20	40	60	80	100	10	20	30		
16.2							20	40	60	80	100	10	20	30		
NOTES: 1. This drawing is to be read with the subject report and project number as presented above. 2. Interpretation assistance by Trow is required before use by others. 3. Date of W.L.=Sept. 11, 2009. 4. Installed PVC standpipes to 12.2 m depth.																

ON_MOT_SO11878G - LARONDE CREEK BRIDGE BY GREG & GPJ ON_MOT.GDT 09/10/21

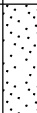







+³, ×³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No BH-7

1 OF 1

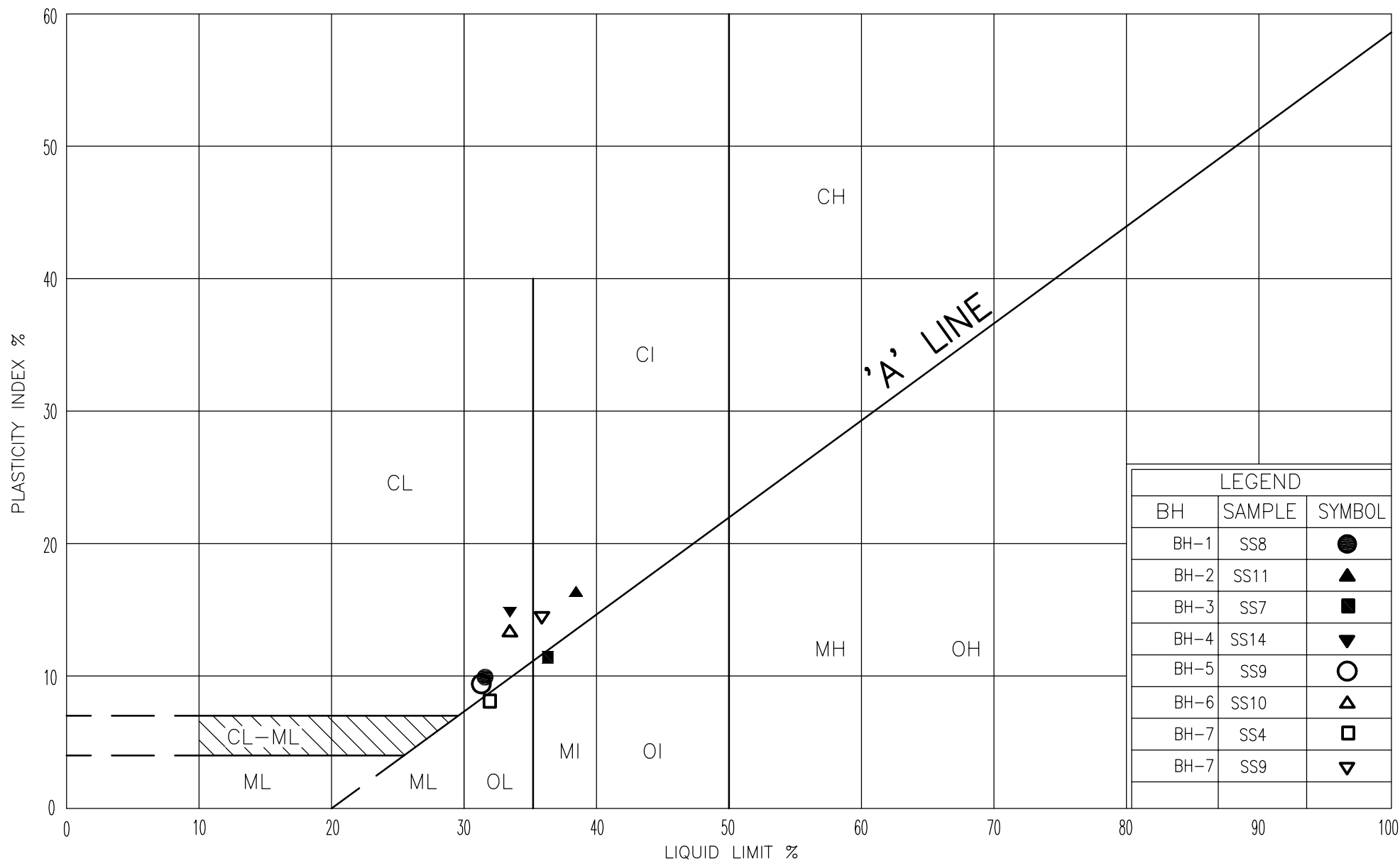
METRIC

W.P. 5274-08-00 LOCATION Laronde Creek, Nipissing Indian Reserve No. 10 ORIGINATED BY GQ
 DIST 54 HWY 17 BOREHOLE TYPE Hollow Stem Auger (Wash Boring) COMPILED BY GQ
 DATUM Geodetic DATE 09.9.10 CHECKED BY VD

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						
198.4 0.0	SAND (FILL) (SW), brown, damp, loose, fine to coarse grained, some silt, trace gravel.		1	AS			198							GR SA SI CL 24 70 (6)
			2	SS	8		197							
196.8 1.5	SILTY CLAY (ML-CI), grey, saturated, soft to stiff, low to medium plasticity		3	SS	10		196							
			4	SS	18		195							
			5	SS	4		194							
							193							
							192							
							191							
							190							
							189							
							188							
							187							
				186										
				185										
				184										
183.0 15.4	BOREHOLE TERMINATED AT ~ 15.4 m DEPTH						183							

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

ON_MOT_SO11878G - LARONDE CREEK BRIDGE BY GREG & GPJ ON_MOT_GDT 09/10/21

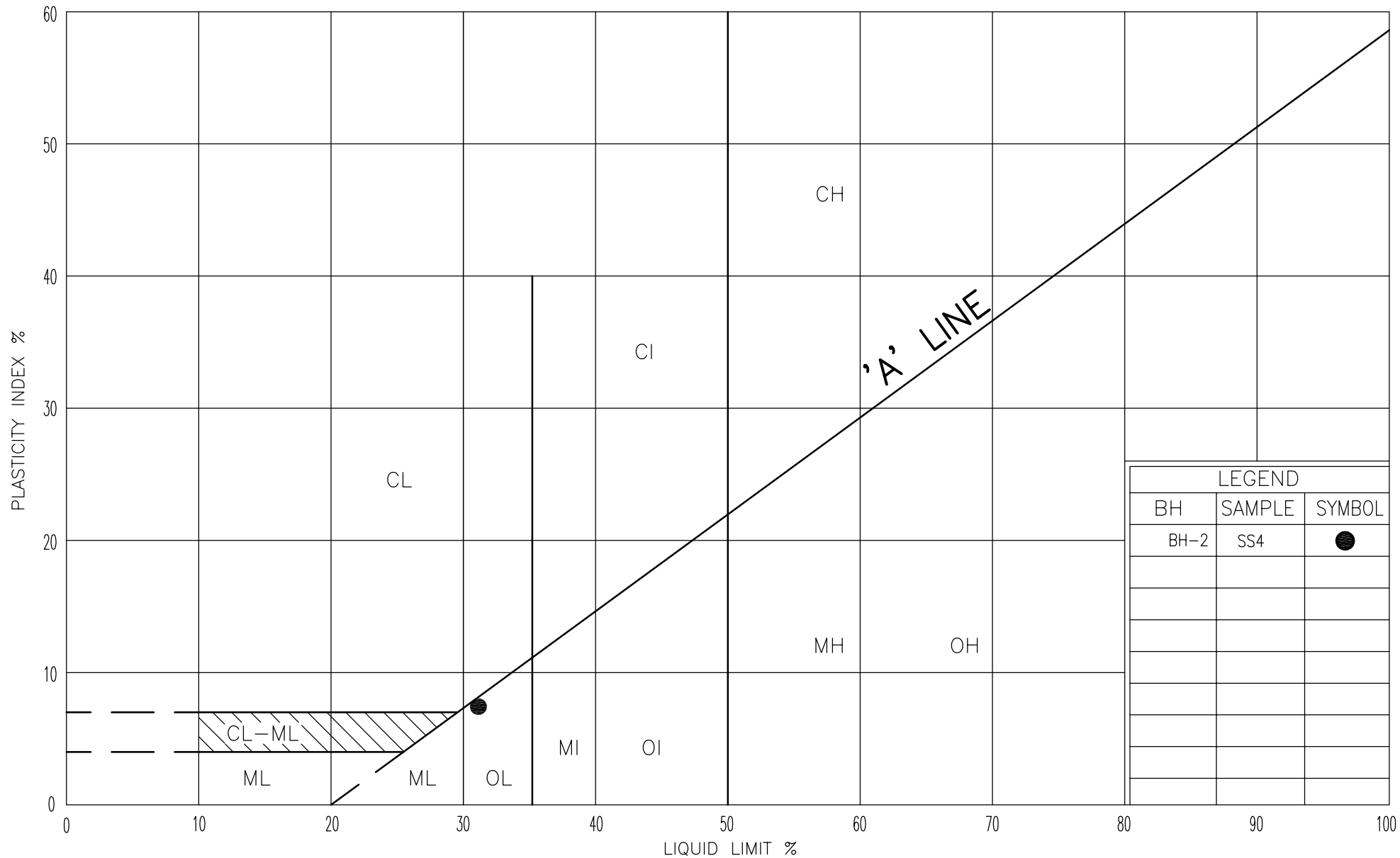


PLASTICITY CHART
SILTY CLAY, (CL, CI, ML, MI)

FIGURE No. 1

WO: 5274-08-00

Gabion Wall Construction, Hwy 17 Sudbury

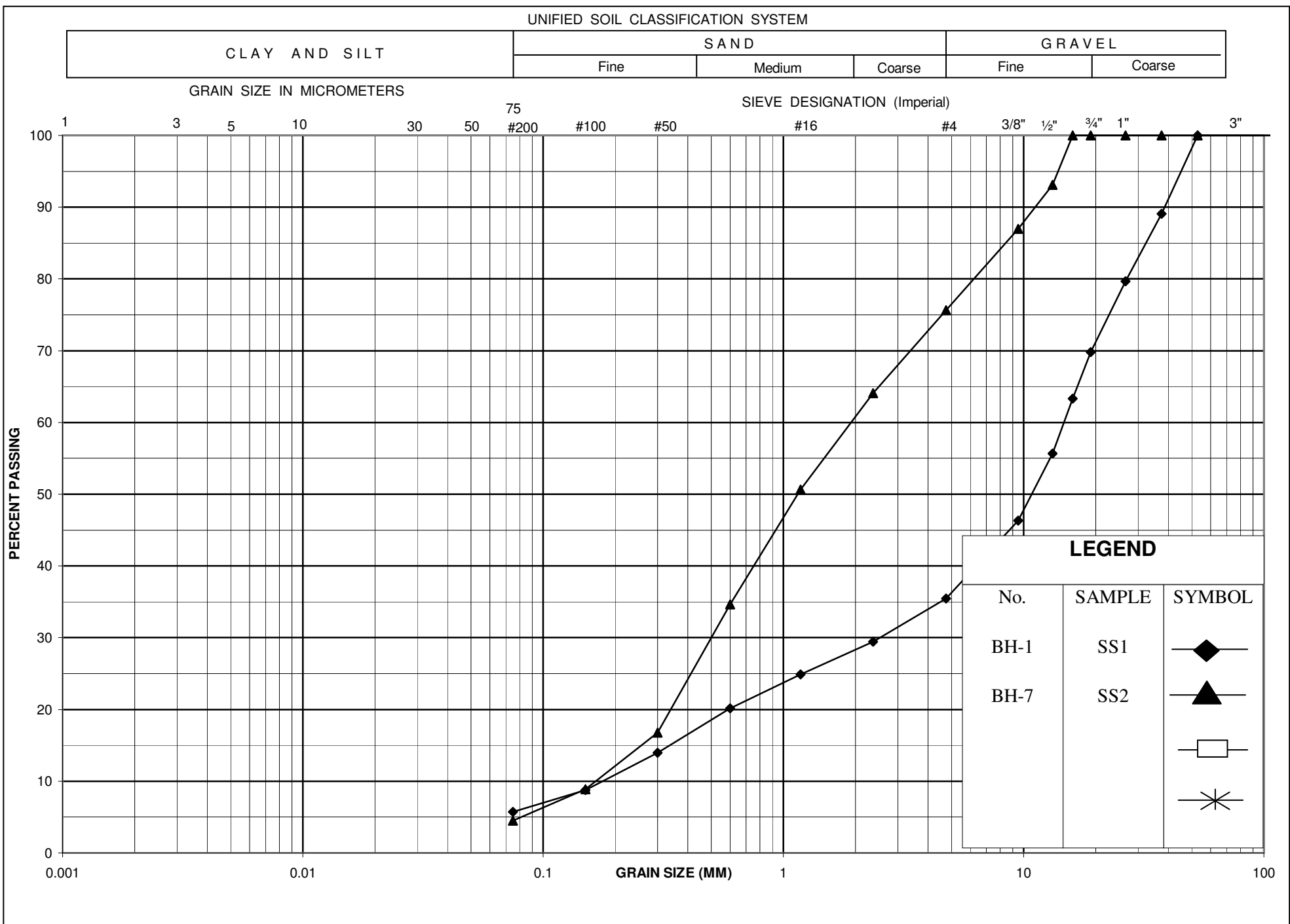


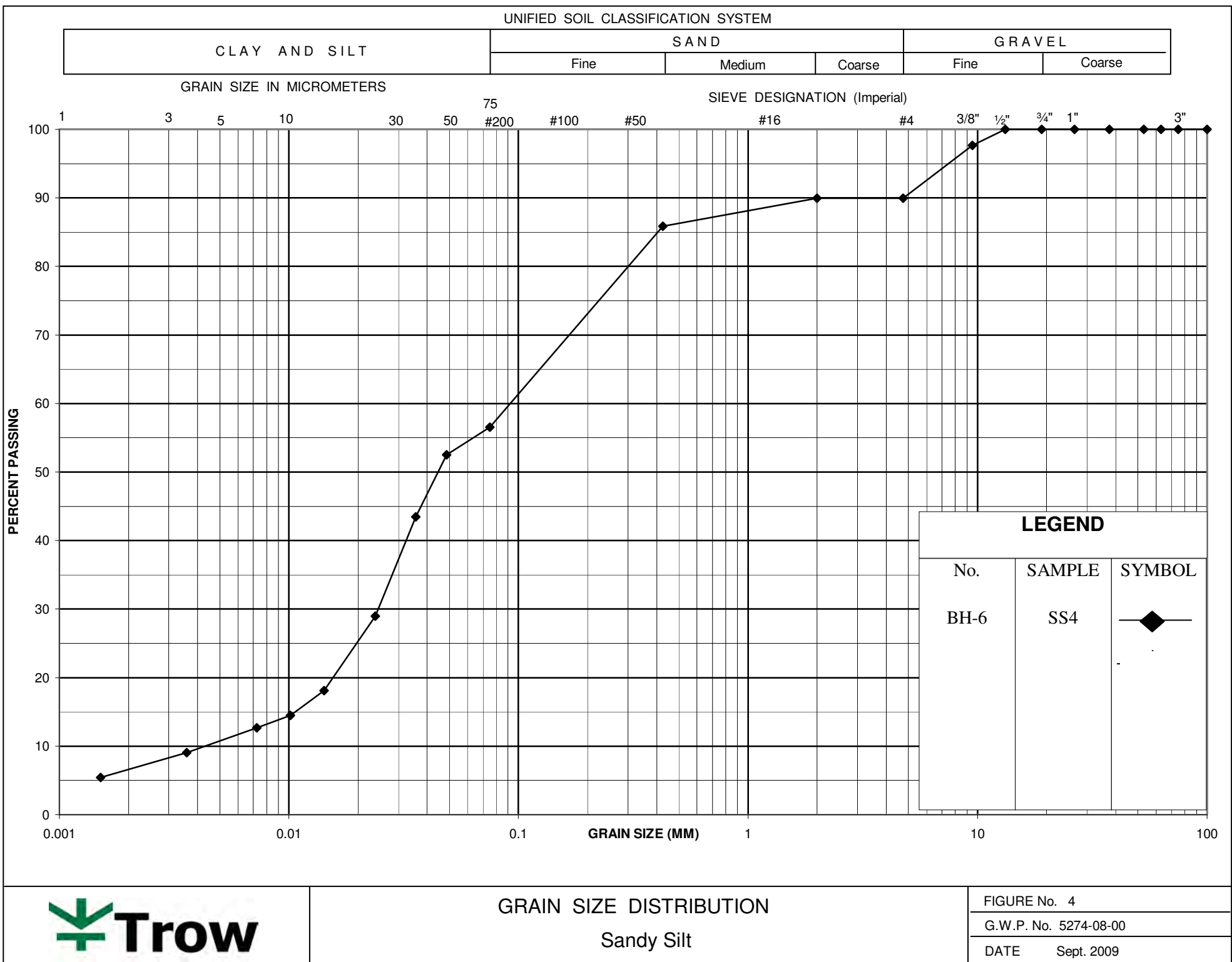
PLASTICITY CHART SILT, (ML)

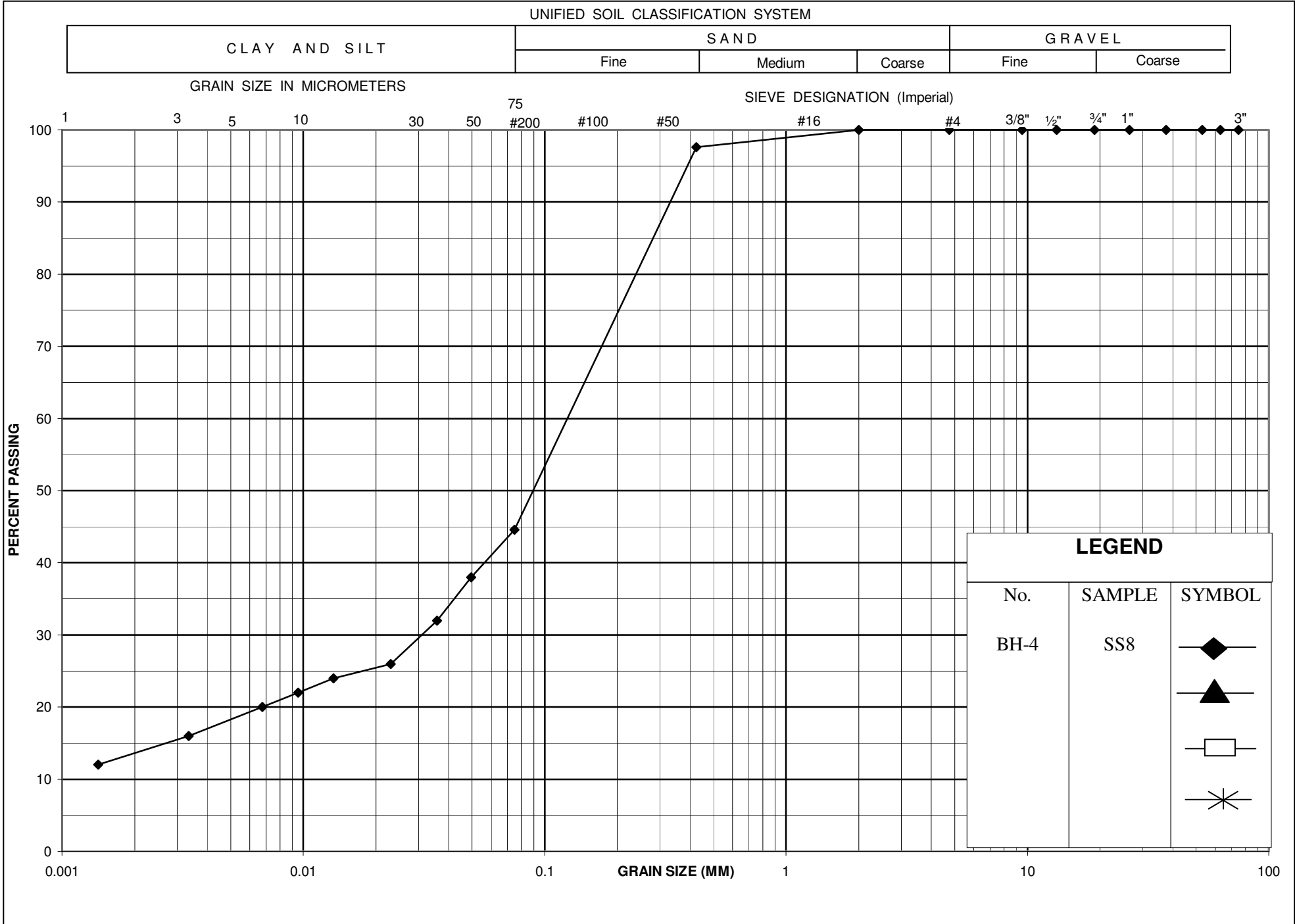
FIGURE No. 2

WO: 5274-08-00

Gabion Wall Construction, Hwy 17 Sudbury





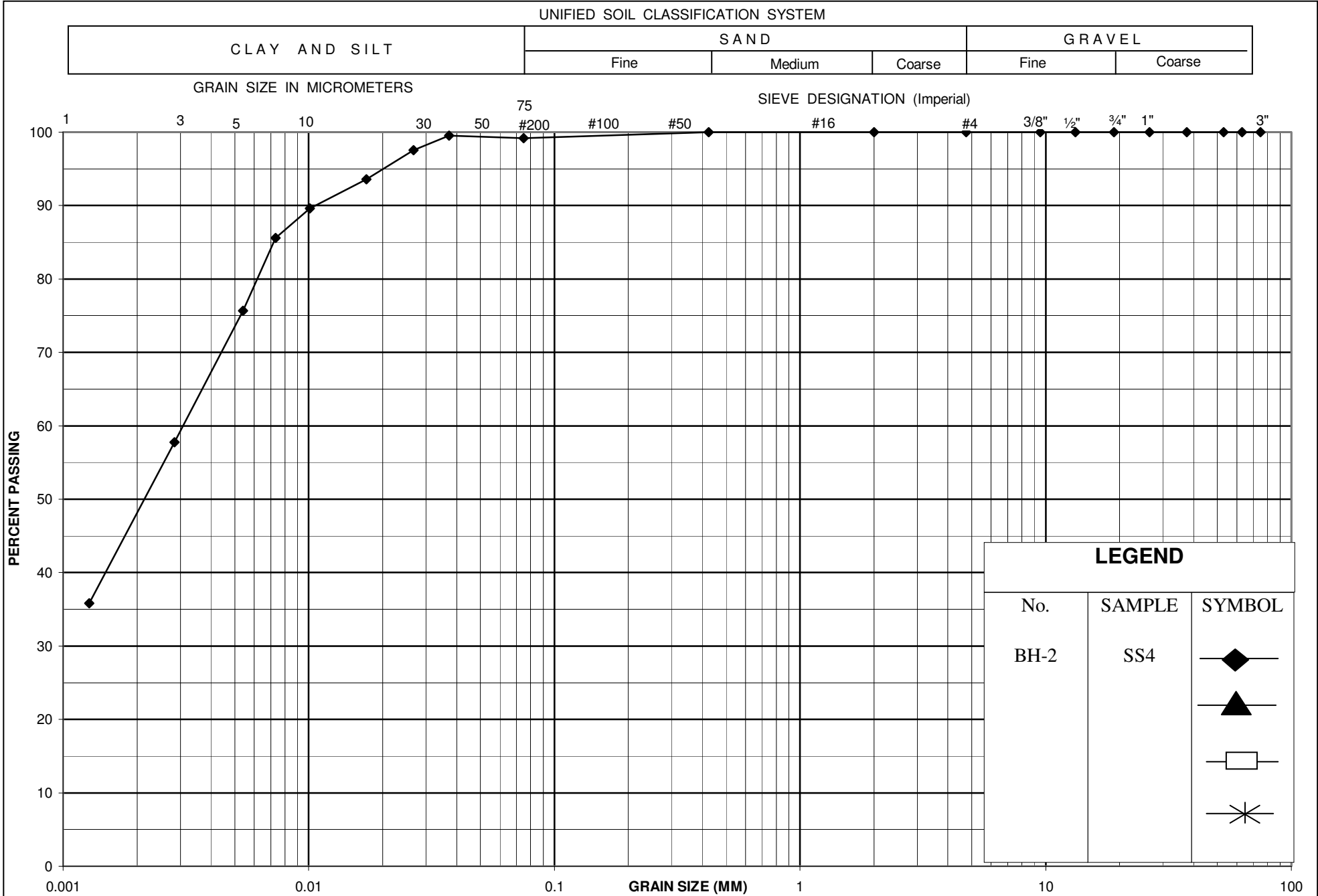


GRAIN SIZE DISTRIBUTION
Silty Sand

FIGURE No. 5

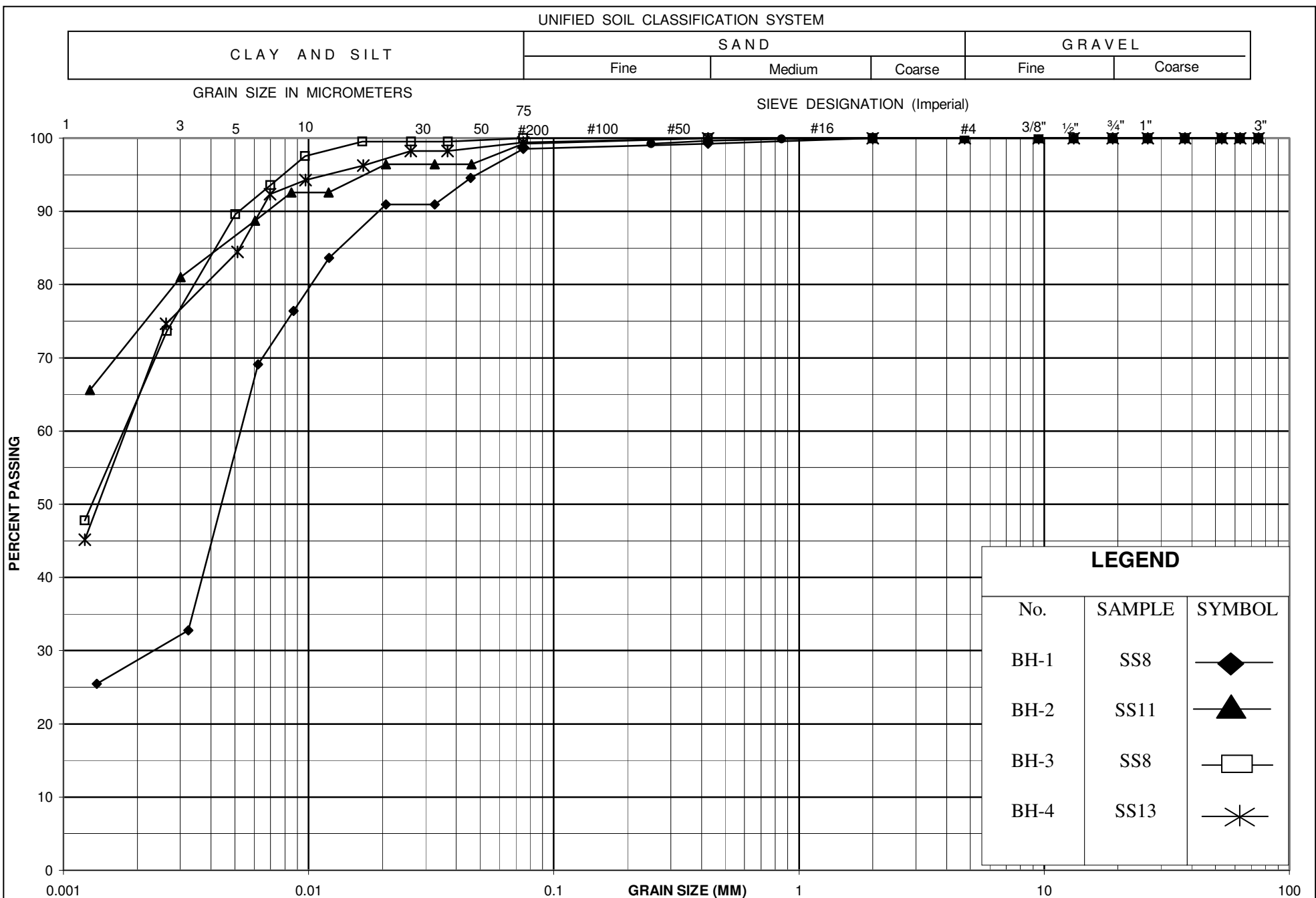
G.W.P. No. 5274-08-00

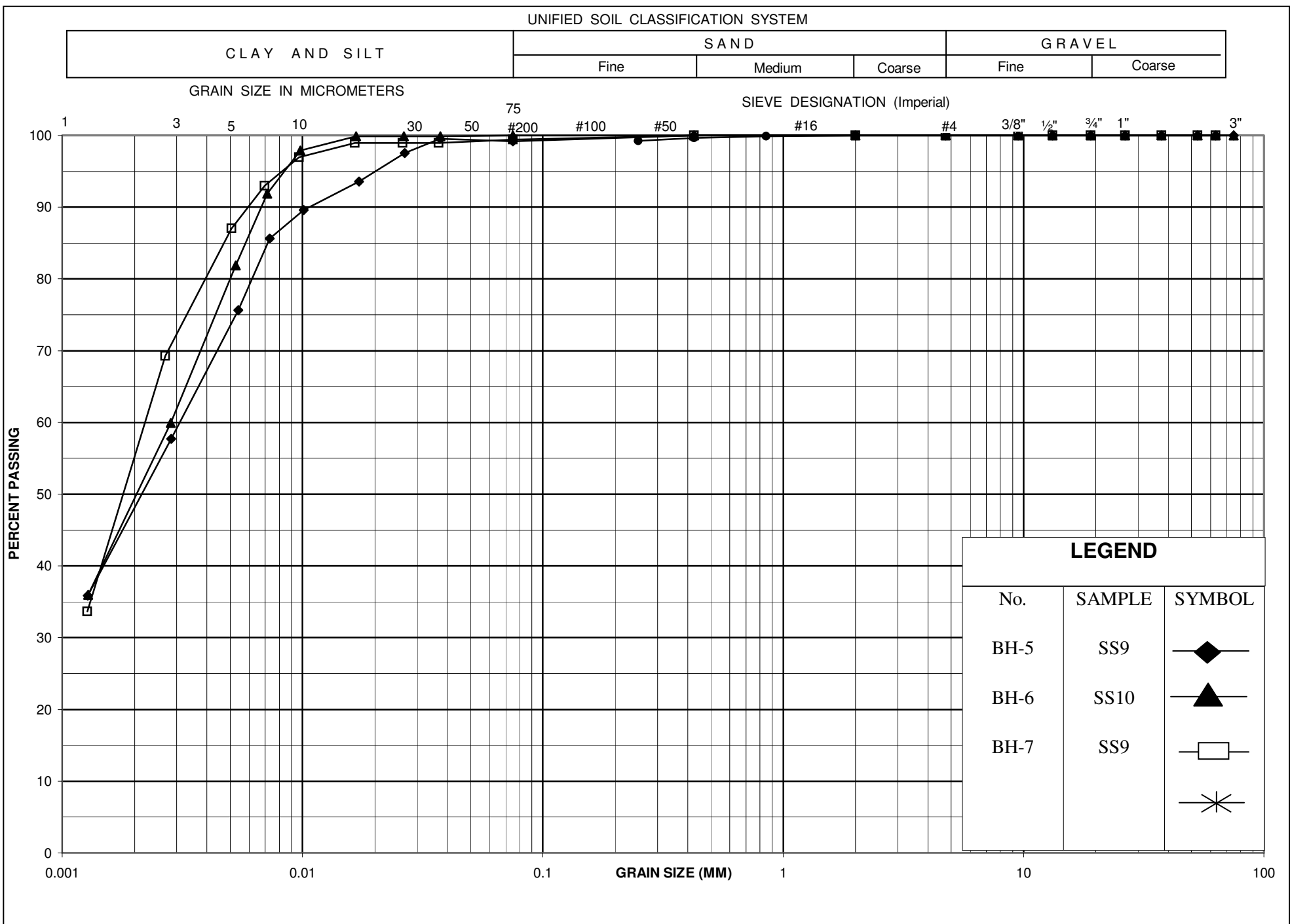
DATE Sept. 2009



GRAIN SIZE DISTRIBUTION
Silt

FIGURE No. 6
G.W.P. No. 5274-08-00
DATE Sept. 2009







Appendix F.
Archived Drawings



Appendix G.

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

June 28, 2018

Site: 46.3701 N, 79.7107 W User File Reference: Laronde Creek Bridge, Hwy 17, Beaucage Township
Requested by: , Thurber Engineering Ltd.

National Building Code ground motions: 2% probability of exceedance in 50 years (0.000404 per annum)

Sa(0.05)	Sa(0.1)	Sa(0.2)	Sa(0.3)	Sa(0.5)	Sa(1.0)	Sa(2.0)	Sa(5.0)	Sa(10.0)	PGA (g)	PGV (m/s)
0.182	0.232	0.210	0.168	0.126	0.068	0.034	0.0087	0.0035	0.131	0.102

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" (NBCC 2015 Site Class C, average shear wave velocity 450 m/s). NBCC2015 and CSAS6-14 values are specified in **bold** font. 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.*

Ground motions for other probabilities:

Probability of exceedance per annum	0.010	0.0021	0.001
Probability of exceedance in 50 years	40%	10%	5%
Sa(0.05)	0.018	0.068	0.110
Sa(0.1)	0.027	0.092	0.144
Sa(0.2)	0.027	0.083	0.130
Sa(0.3)	0.022	0.067	0.104
Sa(0.5)	0.017	0.050	0.077
Sa(1.0)	0.0082	0.027	0.042
Sa(2.0)	0.0034	0.013	0.020
Sa(5.0)	0.0007	0.0029	0.0048
Sa(10.0)	0.0004	0.0012	0.0020
PGA	0.015	0.050	0.080
PGV	0.011	0.036	0.059

References

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

User's Guide - NBC 2015, Structural Commentaries NRCC no. xxxxxx (in preparation)
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

Aussi disponible en français



Natural Resources
Canada

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Canada

Canada



Appendix H.
Comparison of Design Alternatives

Table H-1: Comparison of Foundation Alternatives

	F1 – Steel H-Piles Driven to Bedrock	F2 – Closed End Steel Pipe Piles Driven to Bedrock and Filled with Concrete	F3 – Caissons Socketed into Bedrock	Shallow Footings on Native Soil
Advantages	<ul style="list-style-type: none"> • Quick installation procedure • High bearing resistance • Negligible settlement • Compatible with integral abutment design 	<ul style="list-style-type: none"> • Quick installation procedure • High bearing resistance, but lower than H-piles • Negligible settlement • Lower cost than H-piles 	<ul style="list-style-type: none"> • Higher bearing and lateral resistance than driven steel piles, particularly if socketed into rock • Negligible settlement • Higher resistances allow for fewer foundation elements 	<p align="center">N/A, insufficient bearing capacity</p>
Disadvantages	<ul style="list-style-type: none"> • Piles can refuse on boulders above the bedrock surface • Lateral resistance will need to be provided by soil only; battered piles are not compatible with integral abutments and socketing into the bedrock would introduce risks associated with artesian groundwater pressure 	<ul style="list-style-type: none"> • Not readily accepted for integral abutments • Piles can refuse on boulders above the bedrock surface • Generally lower resistance than H-piles • Less robust than H-piles, higher possibility of damage due to obstructions • Lateral resistance will need to be provided by soil only; battered piles are not compatible with integral abutments and socketing into the bedrock would introduce risks associated with artesian groundwater pressure 	<ul style="list-style-type: none"> • Not compatible with integral abutments • Temporary liners, a heavy slurry, and/or a casing above ground surface would likely be required to maintain shaft stability due to artesian groundwater pressure • Requires specialized contractor • The base cannot be inspected for cleaning and inspection • Very high cost for the length required 	
Risks/ Constructability	<ul style="list-style-type: none"> • Minor potential for pile damage / deflection if obstructions are encountered during driving • Potential conflict with existing timber piles if the same alignment is used 	<ul style="list-style-type: none"> • Higher risk of damage / deflection than H-piles if obstructions are encountered • Potential conflict with existing timber piles if the same alignment is used 	<ul style="list-style-type: none"> • Artesian pressure could impede progress significantly. In the worst case, a different method could be necessitated 	
Relative Cost	Marginally higher than pipe piles but much less than caissons	Lower than H-piles	Very High	
Recommendation	Feasible Recommended	Feasible but not Recommended	Not Feasible	

Table H-2: Comparison of Approach Embankment Alternatives

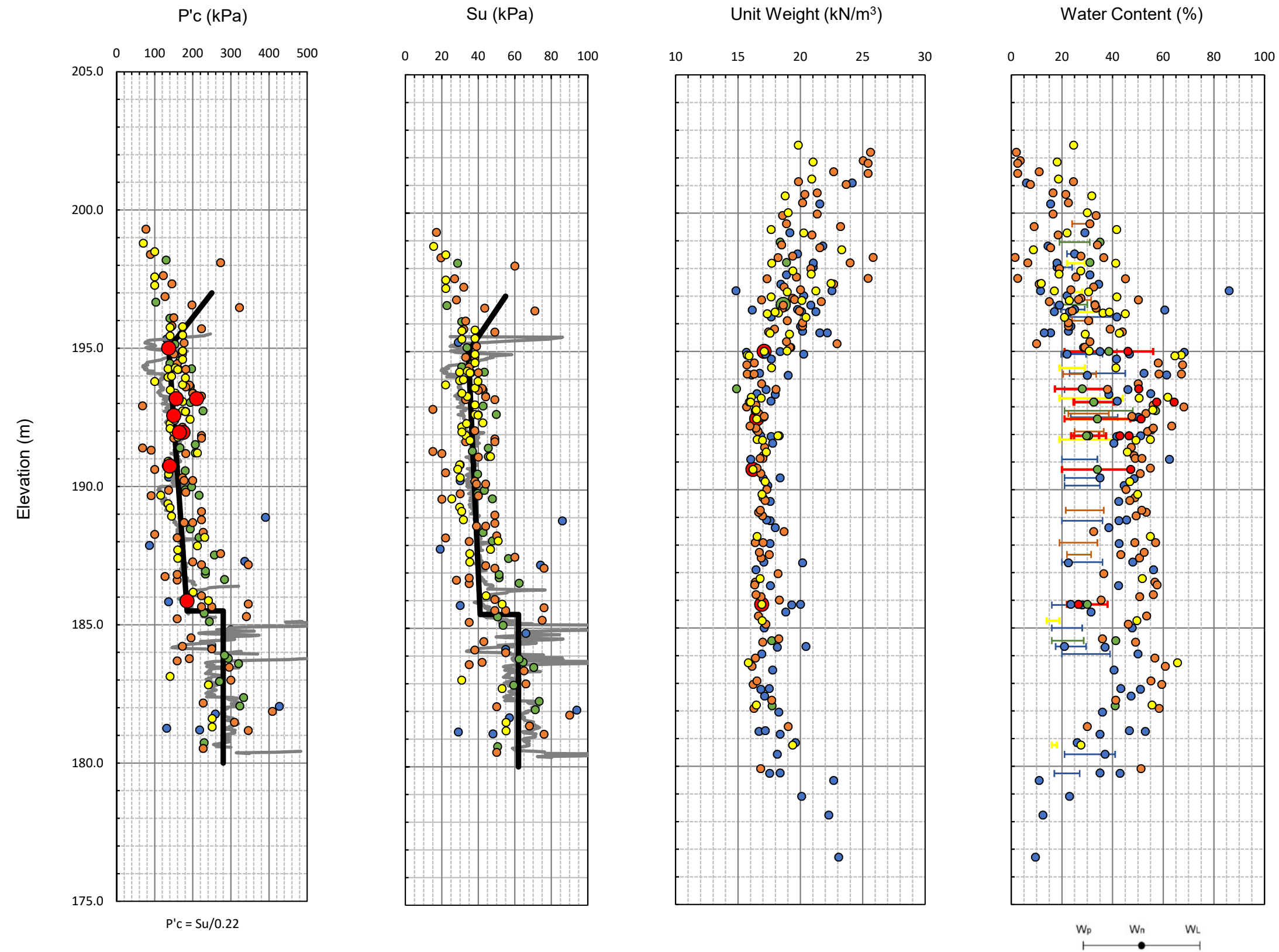
	E1 – Preload and Surcharge Granular Embankment	E2 – Lightweight Fill (EPS)	E3 – Ground Improvement	E4 – Modify the Bridge Design to Reduce Embankment Height
Advantages	<ul style="list-style-type: none"> Conventional construction 	<ul style="list-style-type: none"> Relatively fast construction Addresses both settlement and slope stability concerns without additional mitigations 	<ul style="list-style-type: none"> Addresses both settlement and slope stability concerns without additional mitigations 	<ul style="list-style-type: none"> Addresses both settlement and slope stability concerns without additional mitigations
Disadvantages	<ul style="list-style-type: none"> Requires sheet pile wall to separate the existing embankment from new settlement Requires wick drains Requires high-strength geogrid Requires settlement monitoring Would require years to meet MTO settlement criteria; a partial thickness of EPS is required for a more compressed schedule Long-term secondary settlements may occur, which can be difficult to predict 	<ul style="list-style-type: none"> Construction techniques require several non-standard details; the contractor needs experience with EPS construction and the CA will need to monitor construction carefully 	<ul style="list-style-type: none"> Specialized contractor required The thickness of the clay deposit limits the types of methods that are feasible and increases the costs substantially Technically feasible, but will require multiple mobilizations of specialized contractor 	<ul style="list-style-type: none"> Additional structural requirements High Cost
Risks/ Constructability	<ul style="list-style-type: none"> Construction schedule could be delayed if settlements are larger than estimated Secondary settlements could require pavement levelling more often than the 15-year design life 	<ul style="list-style-type: none"> Post-construction maintenance could be required if the EPS is improperly installed by an inexperienced contractor 	<ul style="list-style-type: none"> Environmental impacts to the creek could be difficult to avoid 	<ul style="list-style-type: none"> None
Relative Cost	High ~\$2.6M	Moderate ~\$1.4M	Very High	Very High ~ \$5M
Recommendation	Feasible but not recommended	Feasible Recommended	Not economically feasible	Not economically feasible

Note: Estimated costs provided by MPCE



Appendix I.

Summary of Engineering Properties Slope Stability Figures



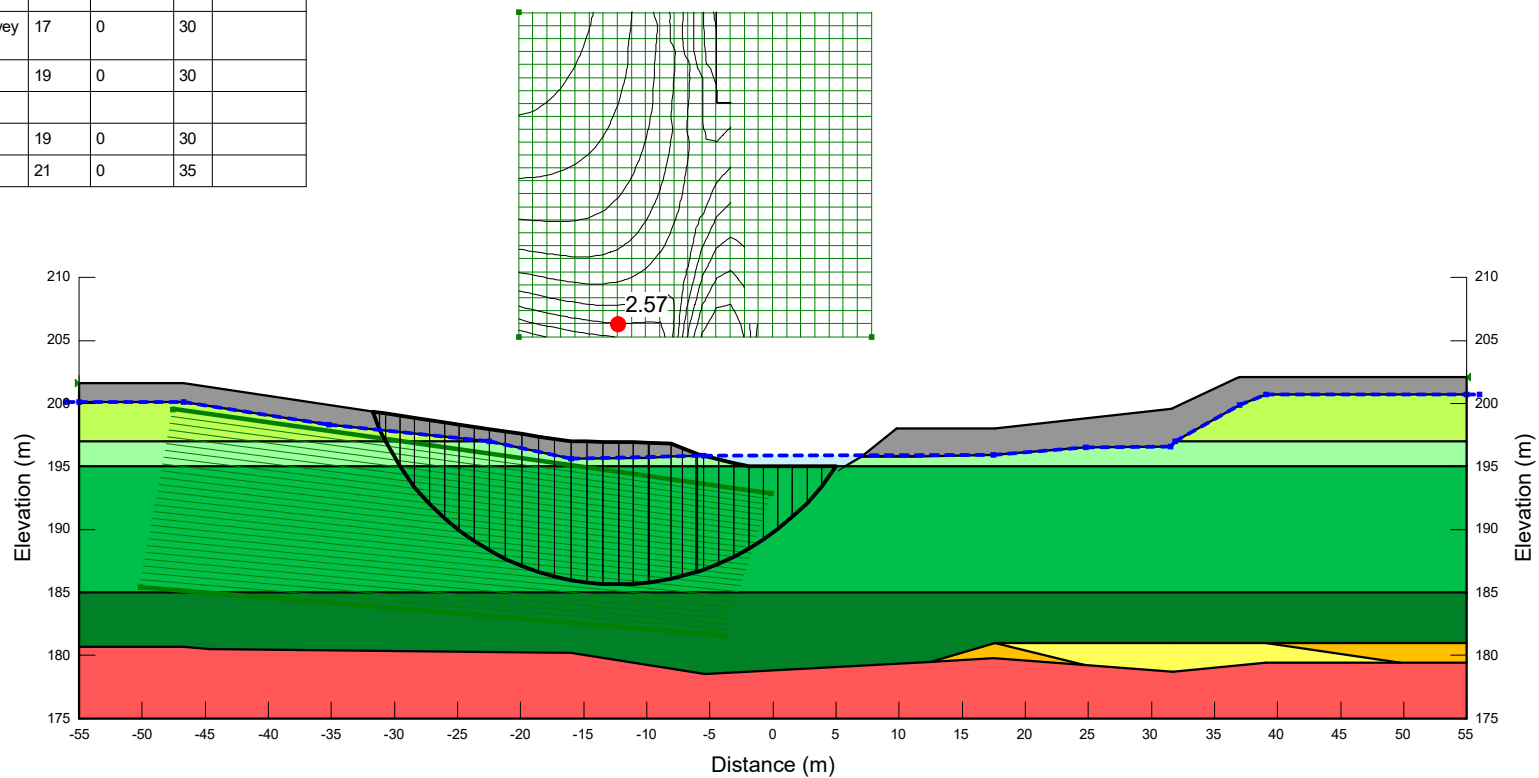
● TEL (1998) ● Golder ● TROW ● TEL (2019) — SCPT19-2 ● Consol. — Design Line

Summary of Engineering Properties Laronde Creek Bridge

File: 23411
Prepared by: DJP
Reviewed by: SD



Color	Name	Unit Weight (kN/m³)	Cohesion (kPa)	Phi (°)	Anisotropic Strength Fn
	C0 - Clayey SILT (ESA)	18.5	0	30	
	C1 - Upper Clayey SILT (ESA)	18.5	0	27	
	C2 - CLAY 1 (ESA)	17	0	27	Varved Clay - ESA
	C3 - Lower Clayey SILT (ESA)	17	0	30	
	F2 - SILT/FILL	19	0	30	
	R1 - Bedrock				
	S1 - SAND	19	0	30	
	T1 - TILL	21	0	35	



Project Name:
Laronde Creek Bridge

Analysis Title:
West Approach -Forward Existing

Project No.:
23411

Date:
01/24/2020

Seismic Coeff.:
H: 0g, V: 0g

Scale:
1:600

Prepared by:
Deanna Pizyck

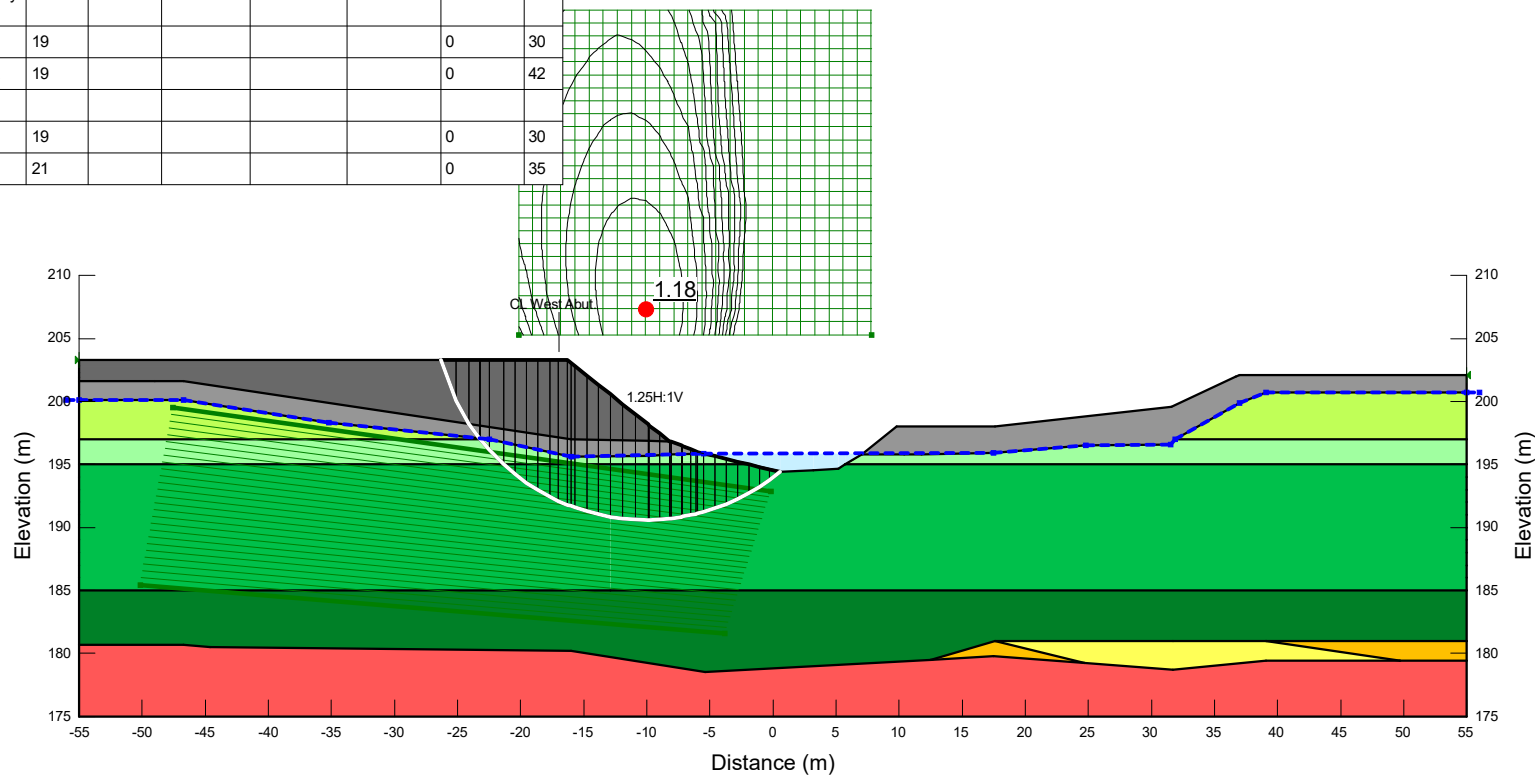
Reviewed by:
SD

Analysis Details:

Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1 m
Center: (-12.300981, 206.29902) m w/ Radius: 20.642123 m
PWP Conditions from: Piezometric Line
Horz Seismic Coef.: 0

Figure 1

Color	Name	Unit Weight (kN/m³)	C-Datum (kPa)	C-Rate of Change ((kN/m²)/m)	C-Maximum (kPa)	Anisotropic Strength F _n	Cohesion' (kPa)	Phi' (°)
	C0 - Clayey SILT (TSA)	19					75	0
	C1 - Upper Clayey SILT (TSA)	18.5	55	-12.5	30			
	C2 - CLAY 1 (TSA)	17	34.5	0.68	41	Varved Clay (TSA)		
	C3 - Lower Clayey SILT (TSA)	17					62	0
	F2 - SILT/FILL	19					0	30
	F4 - ROCKFILL	19					0	42
	R1 - Bedrock							
	S1 - SAND	19					0	30
	T1 - TILL	21					0	35



Project Name:
Laronde Creek Bridge

Analysis Title:
West Approach -Forward Undrained

Project No.:
23411

Date:
01/24/2020

Seismic Coeff.:
H: 0g, V: 0g

Scale:
1:600

Prepared by:
Deanna Pizyck

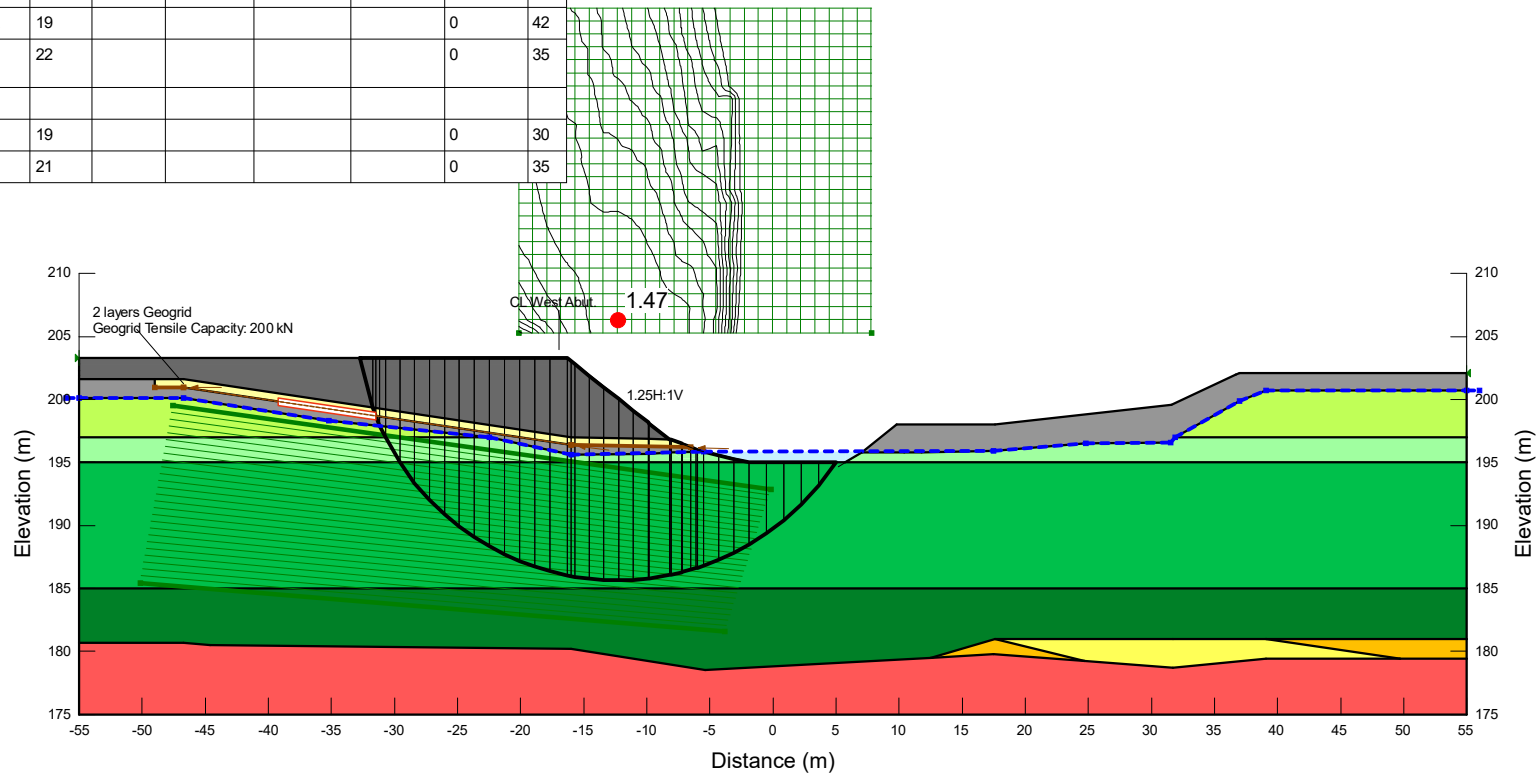
Reviewed by:
SD

Analysis Details:

Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1 m
Center: (-10.064099, 207.33269) m w/ Radius: 16.722372 m
PWP Conditions from: Piezometric Line
Horz Seismic Coef.: 0

Figure 2

Color	Name	Unit Weight (kN/m³)	C-Datum (kPa)	C-Rate of Change ((kN/m³)/m)	C-Maximum (kPa)	Anisotropic Strength F _n	Cohesion (kPa)	Phi' (°)
	C0 - Clayey SILT (TSA)	19					75	0
	C1 - Upper Clayey SILT (TSA)	18.5	55	-12.5	30			
	C2 - CLAY 1 (TSA) (Check)	17	34.5	0.68	41	Varved Clay - TSA		
	C3 - Lower Clayey SILT (TSA)	17					62	0
	F2 - SILT/FILL	19					0	30
	F4 - ROCKFILL	19					0	42
	Granular A or B Type II	22					0	35
	R1 - Bedrock							
	S1 - SAND	19					0	30
	T1 - TILL	21					0	35



Project Name:
Laronde Creek Bridge

Analysis Title:
West Approach -Forward Undrained -Geogrid

Project No.:
23411

Date:
01/24/2020

Seismic Coeff.:
H: 0g, V: 0g

Scale:
1:600

Prepared by:
Deanna Pizyck

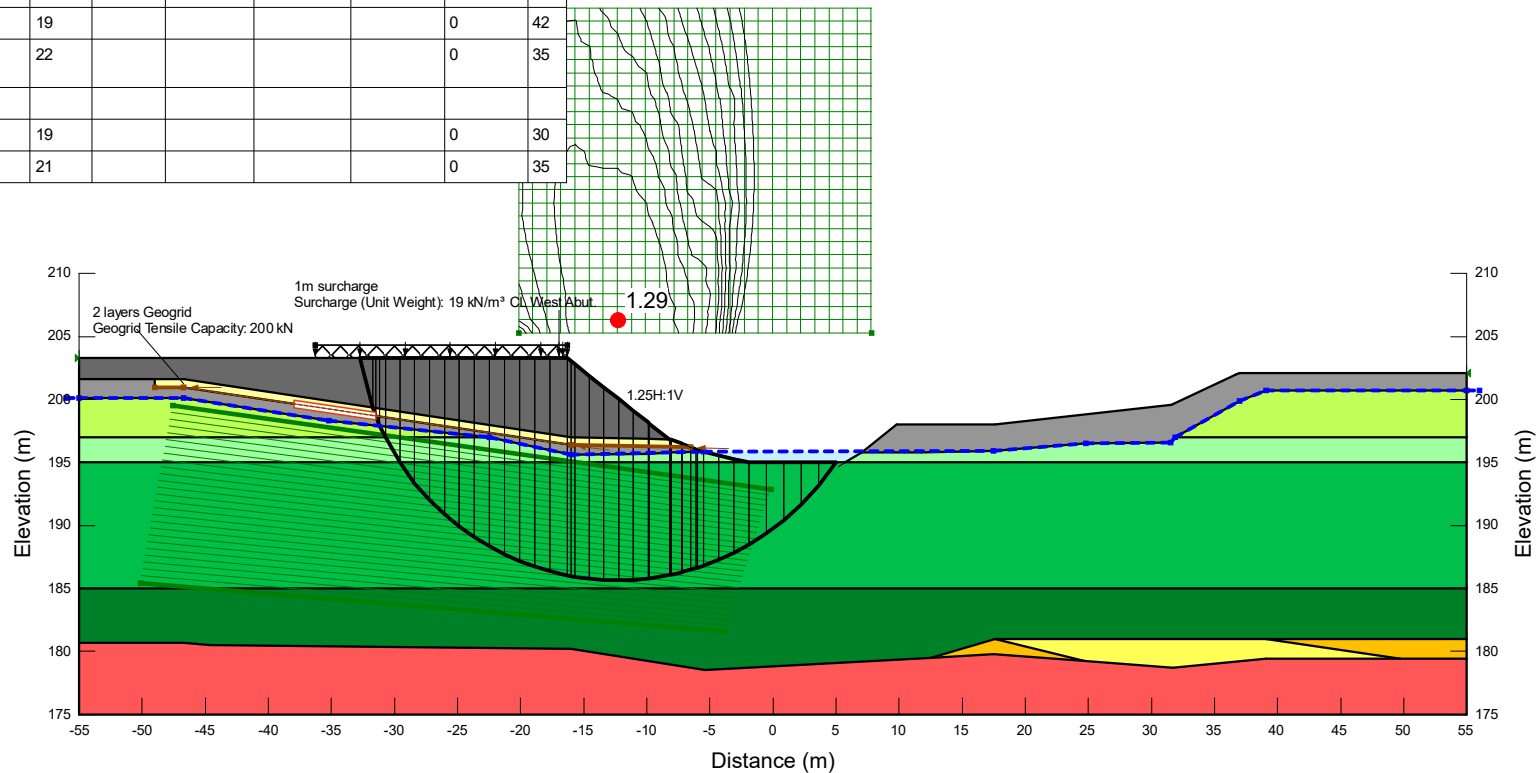
Reviewed by:
SD

Analysis Details:

Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1 m
Center: (-12.300981, 206.29902) m w/ Radius: 20.642123 m
PWP Conditions from: Piezometric Line with B-bar
Horz Seismic Coef.: 0

Figure 3

Color	Name	Unit Weight (kN/m³)	C-Datum (kPa)	C-Rate of Change ((kN/m²)/m)	C-Maximum (kPa)	Anisotropic Strength F _n	Cohesion (kPa)	Phi' (°)
	C0 - Clayey SILT (TSA)	19					75	0
	C1 - Upper Clayey SILT (TSA)	18.5	55	-12.5	30			
	C2 - CLAY 1 (TSA) (Check)	17	34.5	0.68	41	Varved Clay - TSA		
	C3 - Lower Clayey SILT (TSA)	17					62	0
	F2 - SILT/FILL	19					0	30
	F4 - ROCKFILL	19					0	42
	Granular A or B Type II	22					0	35
	R1 - Bedrock							
	S1 - SAND	19					0	30
	T1 - TILL	21					0	35



Project Name:
Laronde Creek Bridge

Analysis Title:
West Approach -Forward Undrained -Geogrid -1m srcg

Project No.:
23411

Date:
01/24/2020

Seismic Coeff.:
H: 0g, V: 0g

Scale:
1:600

Prepared by:
Deanna Pizyck

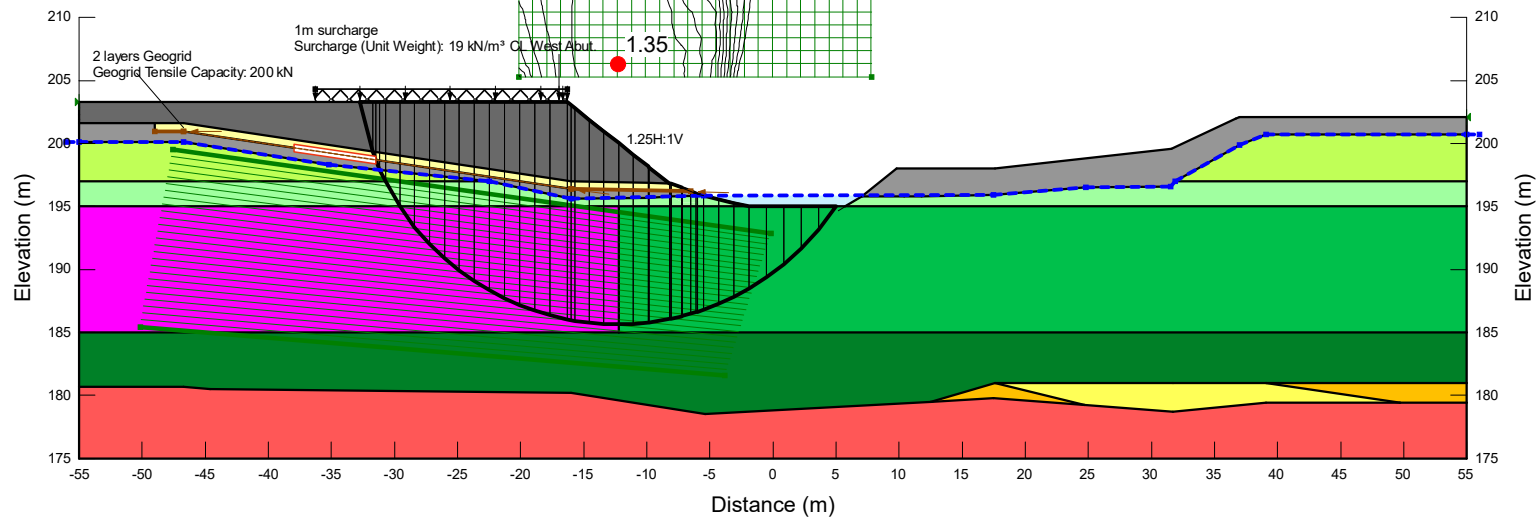
Reviewed by:
SD

Analysis Details:

Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1 m
Center: (-12.300981, 206.29902) m w/ Radius: 20.642123 m
PWP Conditions from: Piezometric Line with B-bar
Horz Seismic Coef.: 0
Surcharge (Unit Weight): 19 kN/m³

Figure 4

C0 - Clayey SILT (TSA)	19							75	0
C1 - Upper Clayey SILT (TSA)	18.5				55	-12.5	30		
C2 - CLAY 1 (TSA) (2)	17	35	0.22	Varved Clay - TSA					
C2 - CLAY 1 (TSA) (Check)	17			Varved Clay - TSA	34.5	0.68	41		
C3 - Lower Clayey SILT (TSA)	17							62	0
F2 - SILT/FILL	19							0	30
F4 - ROCKFILL	19							0	42
Granular A or B Type II	22							0	35
R1 - Bedrock									
S1 - SAND	19							0	30
T1 - TILL	21							0	35



Project Name:
Laronde Creek Bridge

Analysis Title:
West App -Forward Undrained -Geogrid -1m Srcg -Stgh Gn

Project No.:
23411

Date:
01/24/2020

Seismic Coeff.:
H: 0g, V: 0g

Scale:
1:600

Prepared by:
Deanna Pizyck

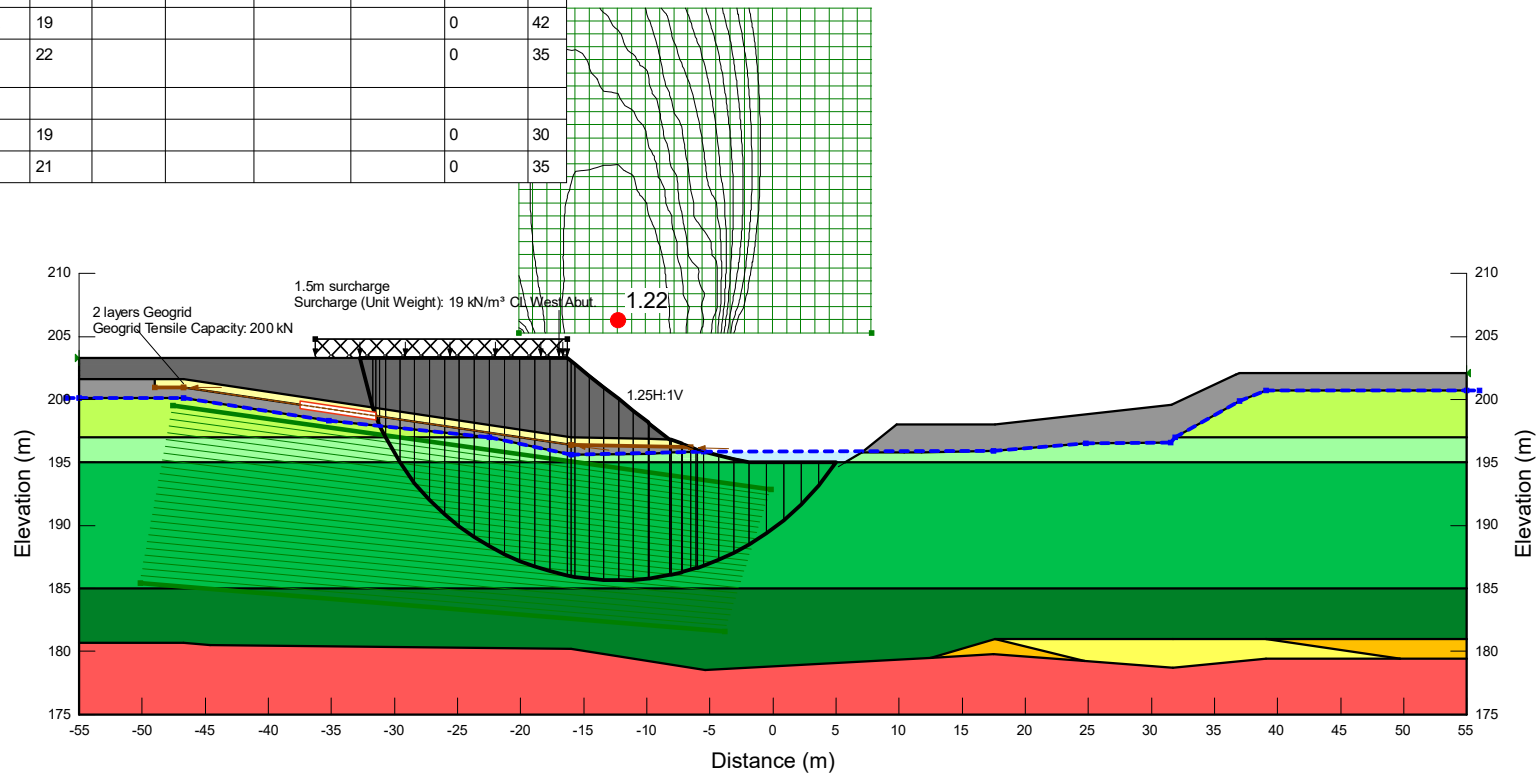
Reviewed by:
SD

Analysis Details:

Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1 m
Center: (-12.300981, 206.29902) m w/ Radius: 20.642123 m
PWP Conditions from: Piezometric Line with B-bar
Horz Seismic Coef.: 0
Surcharge (Unit Weight): 19 kN/m³

Figure 5

Color	Name	Unit Weight (kN/m³)	C-Datum (kPa)	C-Rate of Change ((kN/m³)/m)	C-Maximum (kPa)	Anisotropic Strength F _n	Cohesion' (kPa)	Phi' (°)
	C0 - Clayey SILT (TSA)	19					75	0
	C1 - Upper Clayey SILT (TSA)	18.5	55	-12.5	30			
	C2 - CLAY 1 (TSA) (Check)	17	34.5	0.68	41	Varved Clay - TSA		
	C3 - Lower Clayey SILT (TSA)	17					62	0
	F2 - SILT/FILL	19					0	30
	F4 - ROCKFILL	19					0	42
	Granular A or B Type II	22					0	35
	R1 - Bedrock							
	S1 - SAND	19					0	30
	T1 - TILL	21					0	35



Project Name:
Laronde Creek Bridge

Analysis Title:
West App -Forward Undrained -Geogrid -1.5m Srcg

Project No.:
23411

Date:
01/27/2020

Seismic Coeff.:
H: 0g, V: 0g

Scale:
1:600

Prepared by:
Deanna Pizyck

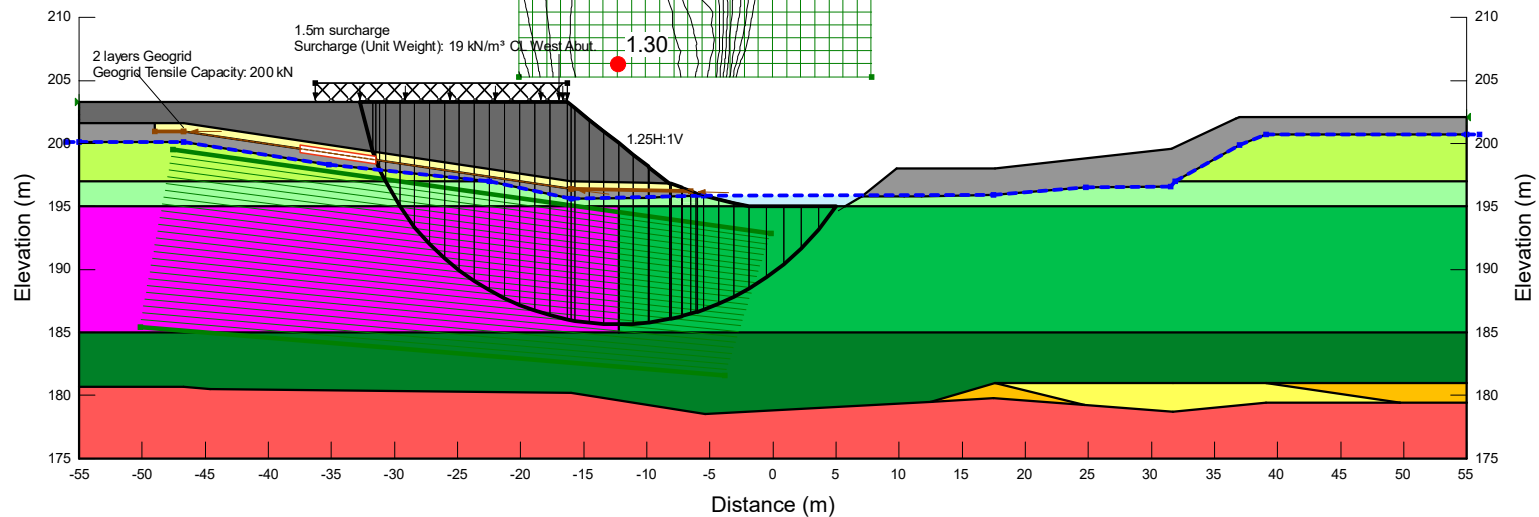
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SD

Analysis Details:

Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1 m
Center: (-12.300981, 206.29902) m w/ Radius: 20.642123 m
PWP Conditions from: Piezometric Line with B-bar
Horz Seismic Coef.: 0
Surcharge (Unit Weight): 19 kN/m³

Figure 6

C0 - Clayey SILT (TSA)	19							75	0
C1 - Upper Clayey SILT (TSA)	18.5				55	-12.5	30		
C2 - CLAY 1 (TSA) (2)	17	35	0.22	Varved Clay - TSA					
C2 - CLAY 1 (TSA) (Check)	17			Varved Clay - TSA	34.5	0.68	41		
C3 - Lower Clayey SILT (TSA)	17							62	0
F2 - SILT/FILL	19							0	30
F4 - ROCKFILL	19							0	42
Granular A or B Type II	22							0	35
R1 - Bedrock									
S1 - SAND	19							0	30
T1 - TILL	21							0	35

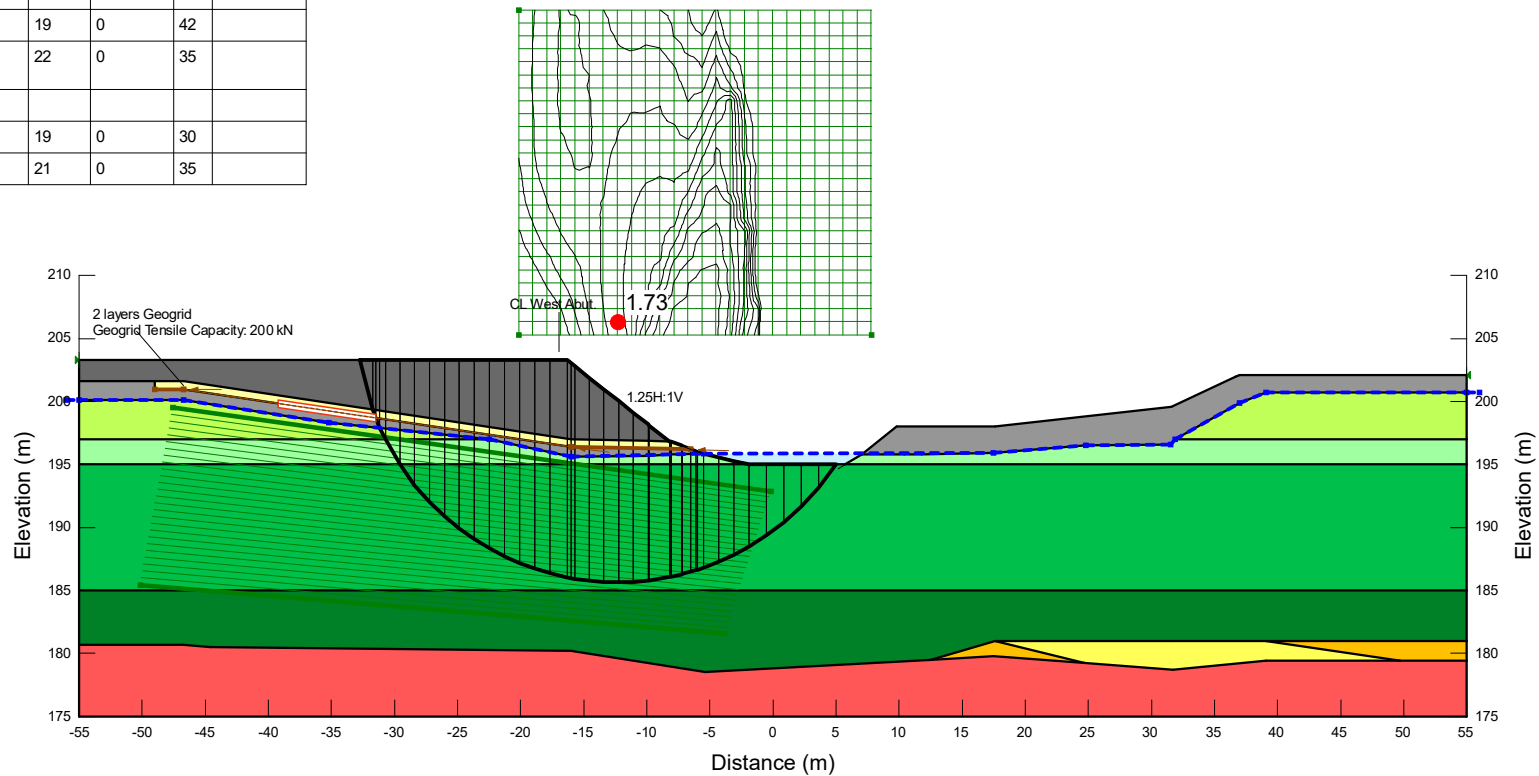


Project Name: Laronde Creek Bridge		
Analysis Title: West App -Forward Undrained -Geogrid -1.5m Srcg -Stgh Gn		
Project No.: 23411	Seismic Coeff.: H: 0g, V: 0g	Prepared by: Deanna Pizyck
Date: 01/27/2020	Scale: 1:600	Reviewed by: SD

Analysis Details:
Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1 m
Center: (-12.300981, 206.29902) m w/ Radius: 20.642123 m
PWP Conditions from: Piezometric Line with B-bar
Horz Seismic Coef.: 0
Surcharge (Unit Weight): 19 kN/m ³

Figure 7

Color	Name	Unit Weight (kN/m³)	Cohesion (kPa)	Phi (°)	Anisotropic Strength Fn
	C0 - Clayey SILT (ESA)	18.5	0	30	
	C1 - Upper Clayey SILT (ESA)	18.5	0	27	
	C2 - CLAY 1 (ESA)	17	0	27	Varved Clay - ESA
	C3 - Lower Clayey SILT (ESA)	17	0	30	
	F2 - SILT/FILL	19	0	30	
	F4 - ROCKFILL	19	0	42	
	Granular A or B Type II	22	0	35	
	R1 - Bedrock				
	S1 - SAND	19	0	30	
	T1 - TILL	21	0	35	



Project Name:
Laronde Creek Bridge

Analysis Title:
West Approach -Forward Drained -Geogrid

Project No.:
23411

Date:
01/24/2020

Seismic Coeff.:
H: 0g, V: 0g

Scale:
1:600

Prepared by:
Deanna Pizyck

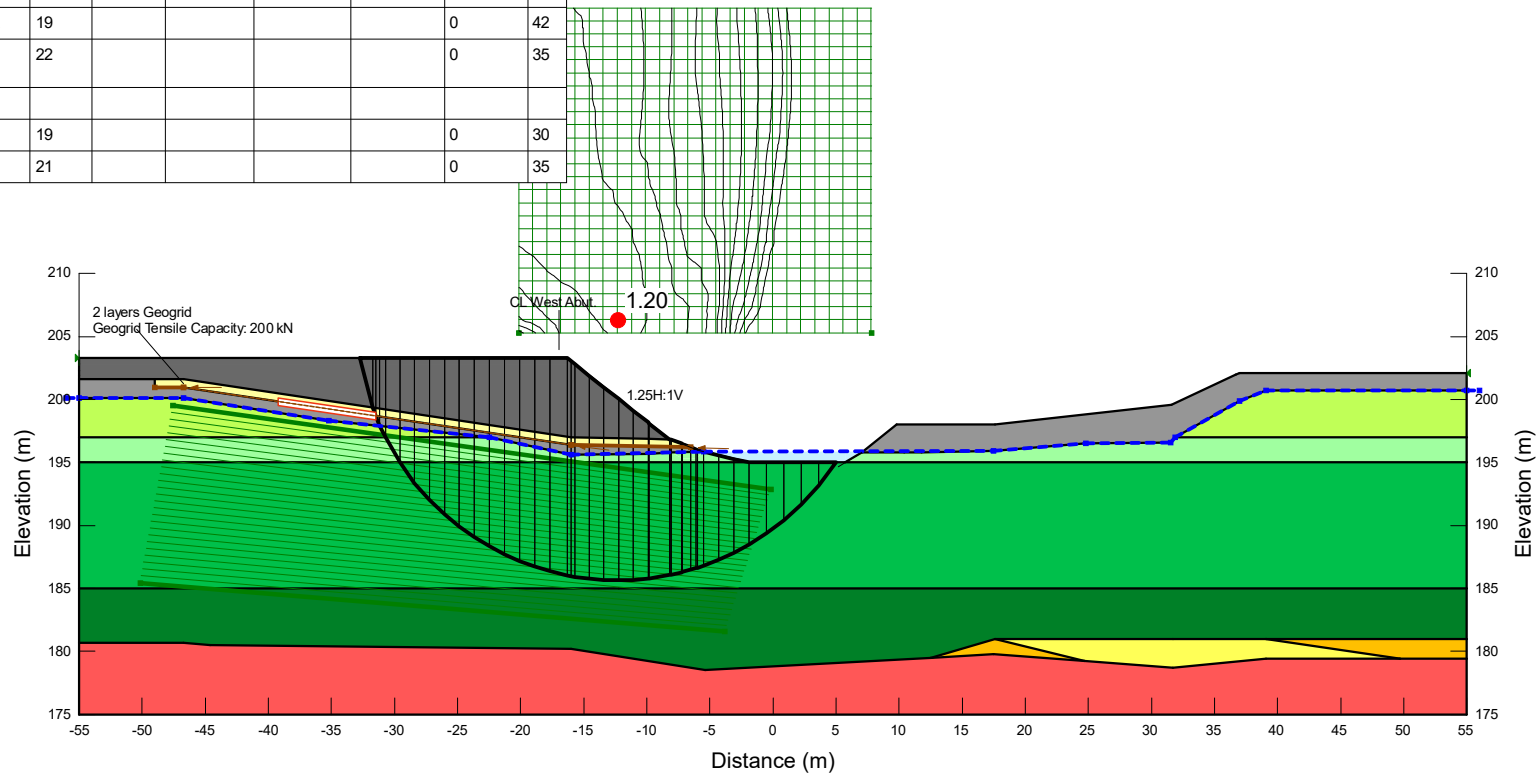
Reviewed by:
SD

Analysis Details:

Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1 m
Center: (-12.300981, 206.29902) m w/ Radius: 20.642123 m
PWP Conditions from: Piezometric Line with B-bar
Horz Seismic Coef.: 0

Figure 8

Color	Name	Unit Weight (kN/m³)	C-Datum (kPa)	C-Rate of Change ((kN/m³)/m)	C-Maximum (kPa)	Anisotropic Strength F _n	Cohesion (kPa)	Phi' (°)
	C0 - Clayey SILT (TSA)	19					75	0
	C1 - Upper Clayey SILT (TSA)	18.5	55	-12.5	30			
	C2 - CLAY 1 (TSA) (Check)	17	34.5	0.68	41	Varved Clay - TSA		
	C3 - Lower Clayey SILT (TSA)	17					62	0
	F2 - SILT/FILL	19					0	30
	F4 - ROCKFILL	19					0	42
	Granular A or B Type II	22					0	35
	R1 - Bedrock							
	S1 - SAND	19					0	30
	T1 - TILL	21					0	35



Project Name:
Laronde Creek Bridge

Analysis Title:
West Approach -Forward Seismic -Geogrid

Project No.:
23411

Date:
01/24/2020

Seismic Coeff.:
H: 0.066g, V: 0g

Scale:
1:600

Prepared by:
Deanna Pizyck

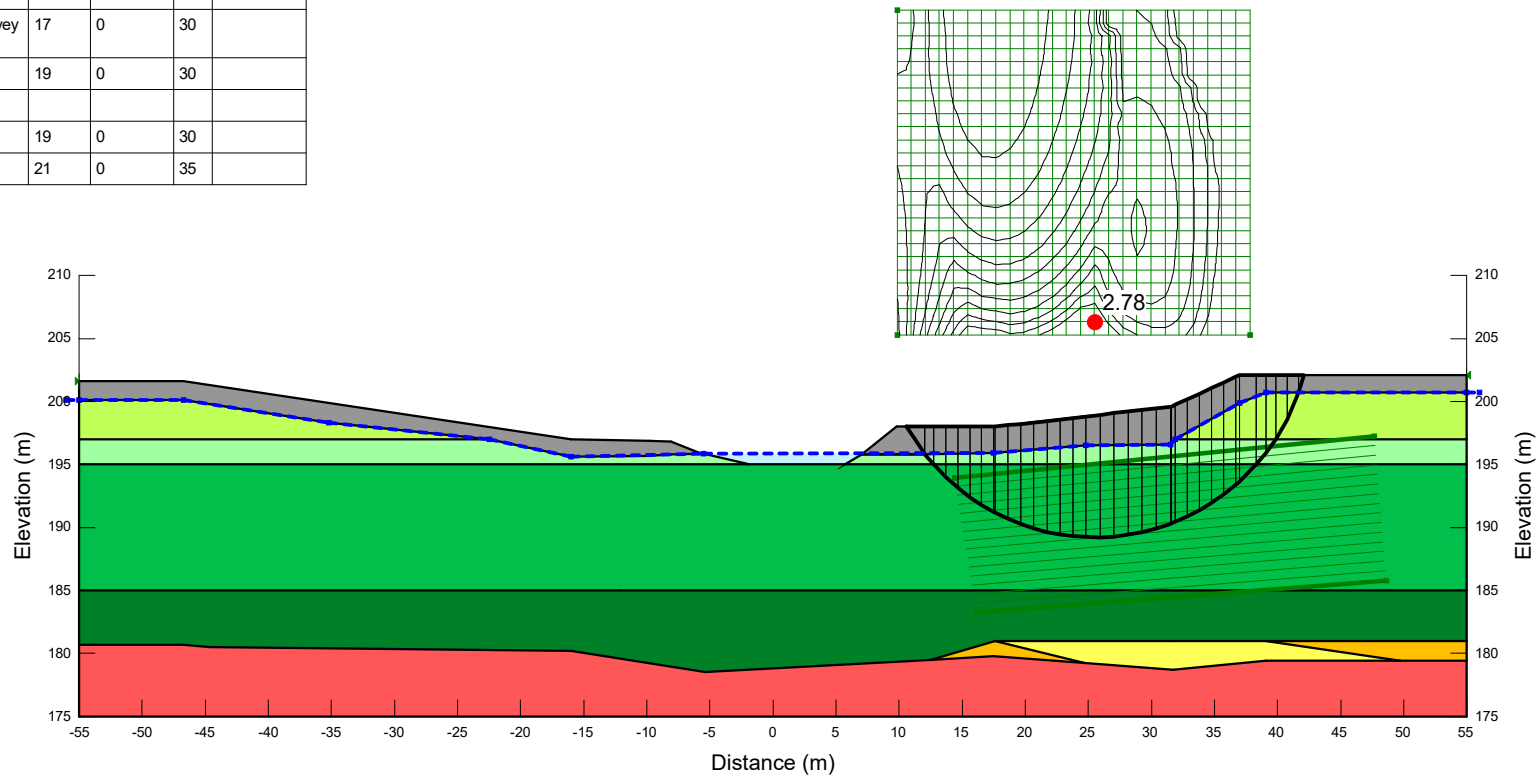
Reviewed by:
SD

Analysis Details:

Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1 m
Center: (-12.300981, 206.29902) m w/ Radius: 20.642123 m
PWP Conditions from: Piezometric Line with B-bar
Horz Seismic Coef.: 0.066

Figure 9

Color	Name	Unit Weight (kN/m³)	Cohesion (kPa)	Phi (°)	Anisotropic Strength Fn
	C0 - Clayey SILT (ESA)	18.5	0	30	
	C1 - Upper Clayey SILT (ESA)	18.5	0	27	
	C2 - CLAY 1 (ESA)	17	0	27	Varved Clay - ESA
	C3 - Lower Clayey SILT (ESA)	17	0	30	
	F2 - SILT/FILL	19	0	30	
	R1 - Bedrock				
	S1 - SAND	19	0	30	
	T1 - TILL	21	0	35	



Project Name:
Laronde Creek Bridge

Analysis Title:
East Approach -Forward Existing

Project No.:
23411

Date:
01/24/2020

Seismic Coeff.:
H: 0g, V: 0g

Scale:
1:600

Prepared by:
Deanna Pizyck

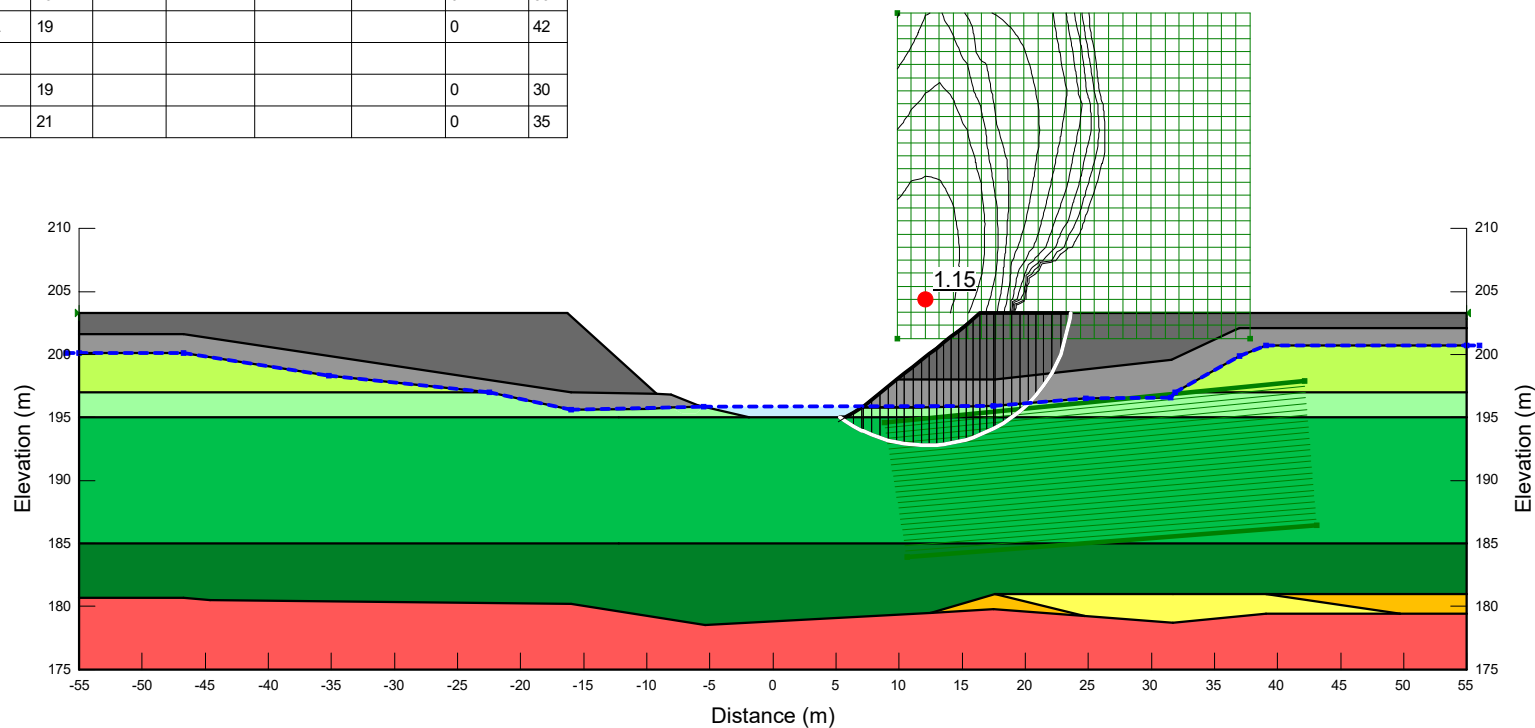
Reviewed by:
SD

Analysis Details:

Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1 m
Center: (25.528107, 206.29902) m w/ Radius: 17.07328 m
PWP Conditions from: Piezometric Line
Horz Seismic Coef.: 0

Figure 10

Color	Name	Unit Weight (kN/m³)	C-Datum (kPa)	C-Rate of Change ((kN/m³)/m)	C-Maximum (kPa)	Anisotropic Strength F _n	Cohesion* (kPa)	Phi* (°)
	C0 - Clayey SILT (TSA)	19					75	0
	C1 - Upper Clayey SILT (TSA)	18.5	55	-12.5	30			
	C2 - CLAY 1 (TSA) (Check)	17	34.5	0.68	41	Varved Clay - TSA		
	C3 - Lower Clayey SILT (TSA)	17					62	0
	F2 - SILT/FILL	19					0	30
	F4 - ROCKFILL	19					0	42
	R1 - Bedrock							
	S1 - SAND	19					0	30
	T1 - TILL	21					0	35



Project Name:
Laronde Creek Bridge

Analysis Title:
East Approach -Forward Undrained

Project No.:
23411

Date:
01/27/2020

Seismic Coeff.:
H: 0g, V: 0g

Scale:
1:600

Prepared by:
Deanna Pizyck

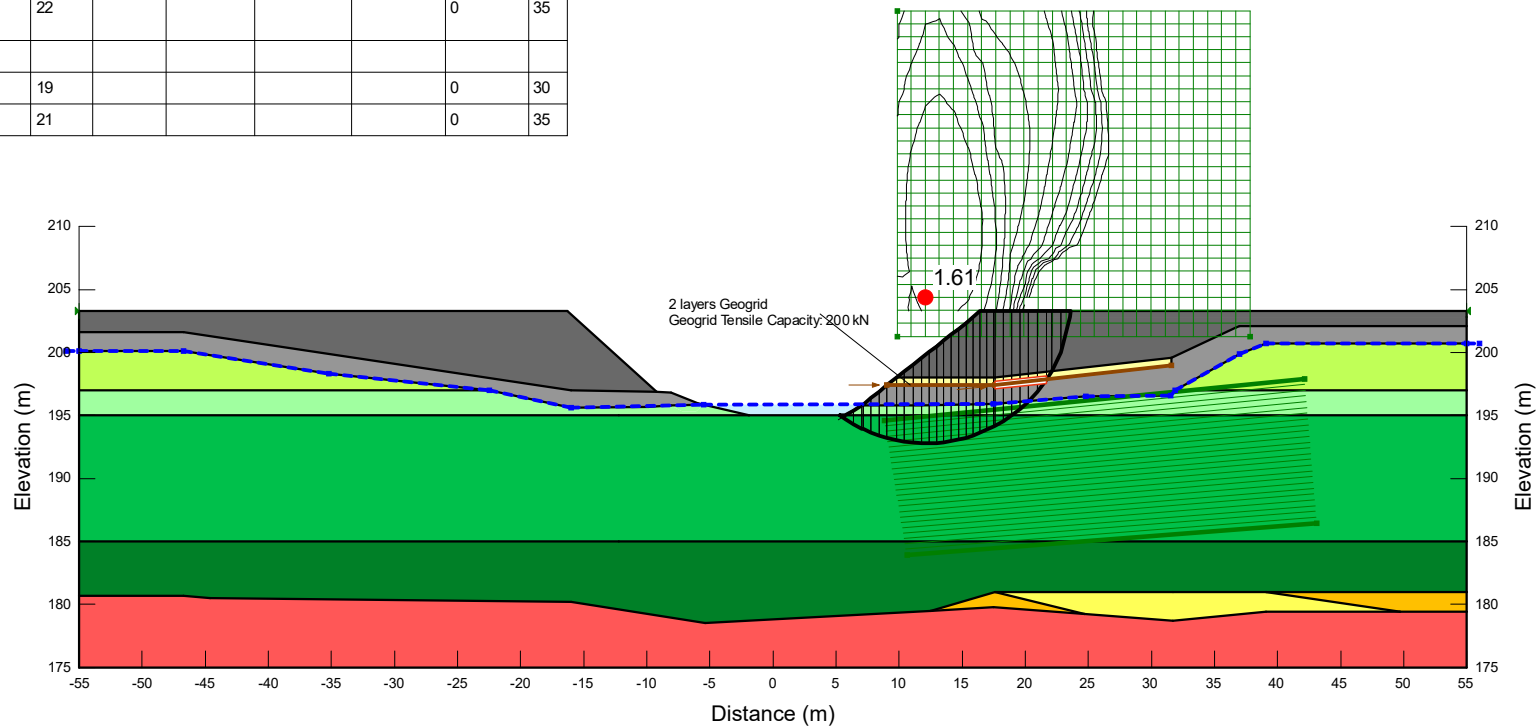
Reviewed by:
SD

Analysis Details:

Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1 m
Center: (12.106812, 204.36636) m w/ Radius: 11.567333 m
PWP Conditions from: Piezometric Line
Horz Seismic Coef.: 0

Figure 11

Color	Name	Unit Weight (kN/m³)	C-Datum (kPa)	C-Rate of Change ((kN/m²)/m)	C-Maximum (kPa)	Anisotropic Strength F _n	Cohesion* (kPa)	Phi* (°)
	C0 - Clayey SILT (TSA)	19					75	0
	C1 - Upper Clayey SILT (TSA)	18.5	55	-12.5	30			
	C2 - CLAY 1 (TSA) (Check)	17	34.5	0.68	41	Varved Clay - TSA		
	C3 - Lower Clayey SILT (TSA)	17					62	0
	F2 - SILT/FILL	19					0	30
	F4 - ROCKFILL	19					0	42
	Granular Aor B Type II	22					0	35
	R1 - Bedrock							
	S1 - SAND	19					0	30
	T1 - TILL	21					0	35



Project Name:
Laronde Creek Bridge

Analysis Title:
East Approach -Forward Undrained - Geogrid

Project No.:
23411

Date:
01/28/2020

Seismic Coeff.:
H: 0g, V: 0g

Scale:
1:600

Prepared by:
Deanna Pizyck

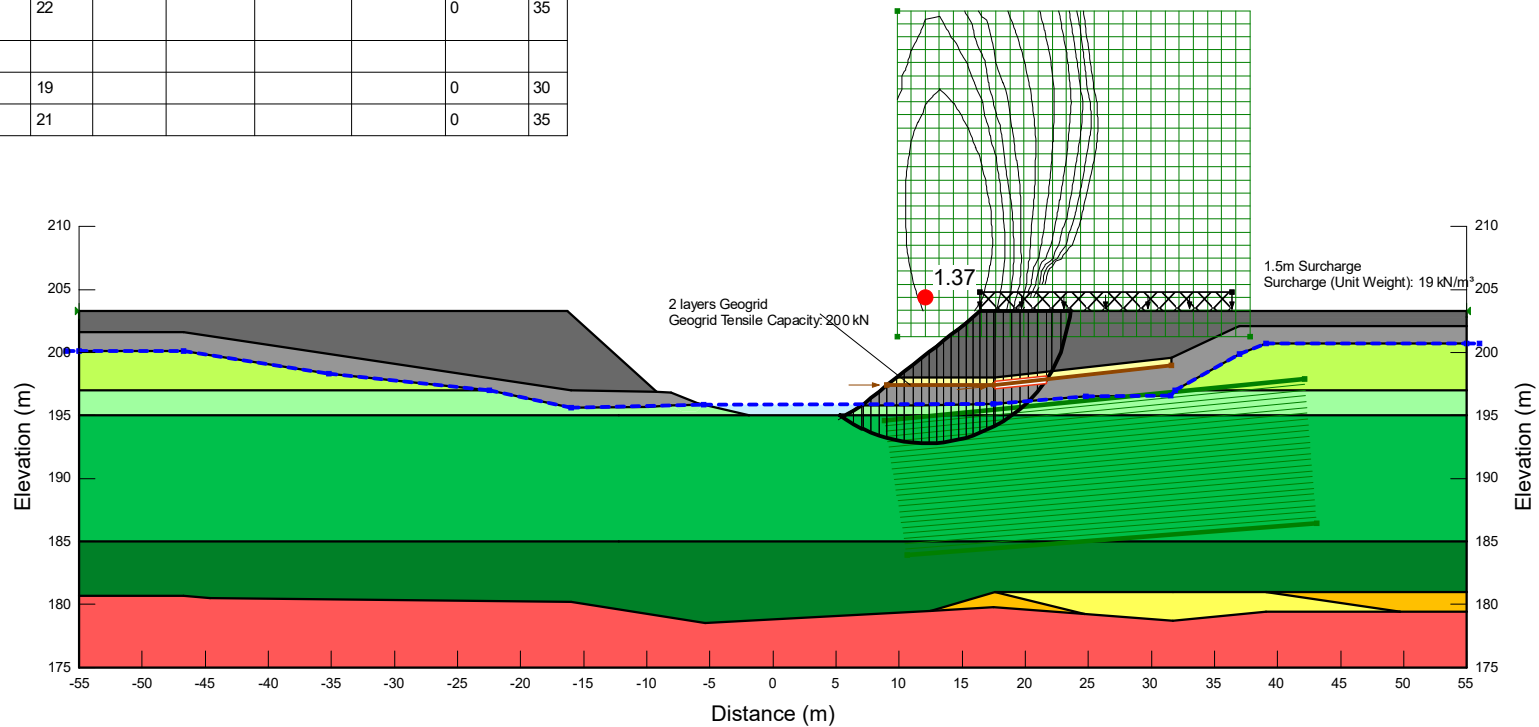
Reviewed by:
SD

Analysis Details:

Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1 m
Center: (12.106812, 204.36636) m w/ Radius: 11.567333 m
PWP Conditions from: Piezometric Line
Horz Seismic Coef.: 0

Figure 12

Color	Name	Unit Weight (kN/m³)	C-Datum (kPa)	C-Rate of Change ((kN/m²)/m)	C-Maximum (kPa)	Anisotropic Strength F _n	Cohesion* (kPa)	Phi* (°)
	C0 - Clayey SILT (TSA)	19					75	0
	C1 - Upper Clayey SILT (TSA)	18.5	55	-12.5	30			
	C2 - CLAY 1 (TSA) (Check)	17	34.5	0.68	41	Varved Clay - TSA		
	C3 - Lower Clayey SILT (TSA)	17					62	0
	F2 - SILT/FILL	19					0	30
	F4 - ROCKFILL	19					0	42
	Granular Aor B Type II	22					0	35
	R1 - Bedrock							
	S1 - SAND	19					0	30
	T1 - TILL	21					0	35



Project Name:
Laronde Creek Bridge

Analysis Title:
East Approach -Forward Undrained - Geogrid -1.5m Srcg

Project No.:
23411

Date:
01/28/2020

Seismic Coeff.:
H: 0g, V: 0g

Scale:
1:600

Prepared by:
Deanna Pizyck

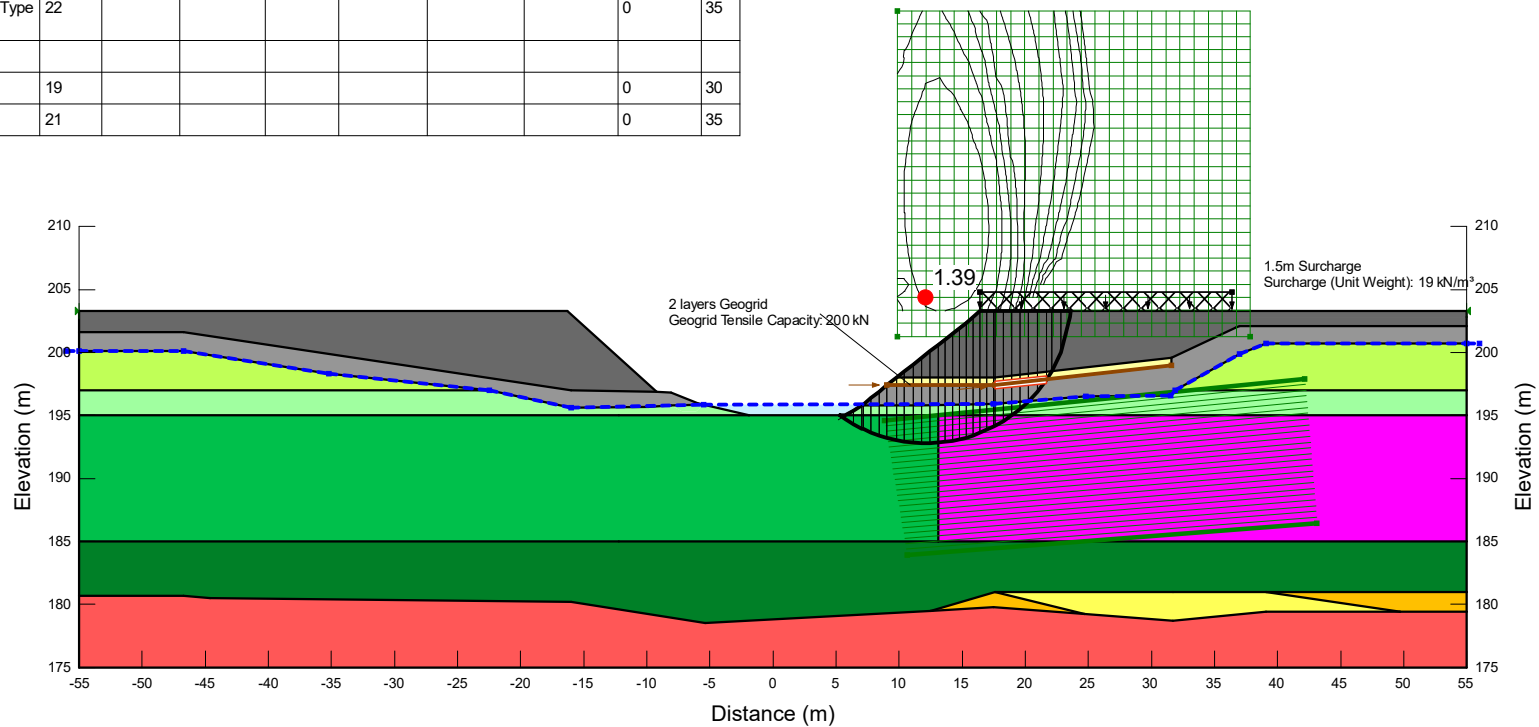
Reviewed by:
SD

Analysis Details:

Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1 m
Center: (12.106812, 204.36636) m w/ Radius: 11.567333 m
PWP Conditions from: Piezometric Line
Horz Seismic Coef.: 0
Surcharge (Unit Weight): 19 kN/m³

Figure 13

C0 - Clayey SILT (TSA)	19							75	0
C1 - Upper Clayey SILT (TSA)	18.5			55	-12.5	30			
C2 - CLAY 1 (TSA) (Check)	17			34.5	0.68	41	Varved Clay - TSA		
C2 - CLAY 1 (TSA) (Check) (2)	17	35	0.22				Varved Clay - TSA		
C3 - Lower Clayey SILT (TSA)	17							62	0
F2 - SILT/FILL	19							0	30
F4 - ROCKFILL	19							0	42
Granular A or B Type II	22							0	35
R1 - Bedrock									
S1 - SAND	19							0	30
T1 - TILL	21							0	35



Project Name:
Laronde Creek Bridge

Analysis Title:
East Approach -Forward Undrained - Geogrid -1.5m Srcg -Stgh Gn

Project No.:
23411

Date:
01/28/2020

Seismic Coeff.:
H: 0g, V: 0g

Scale:
1:600

Prepared by:
Deanna Pizyck

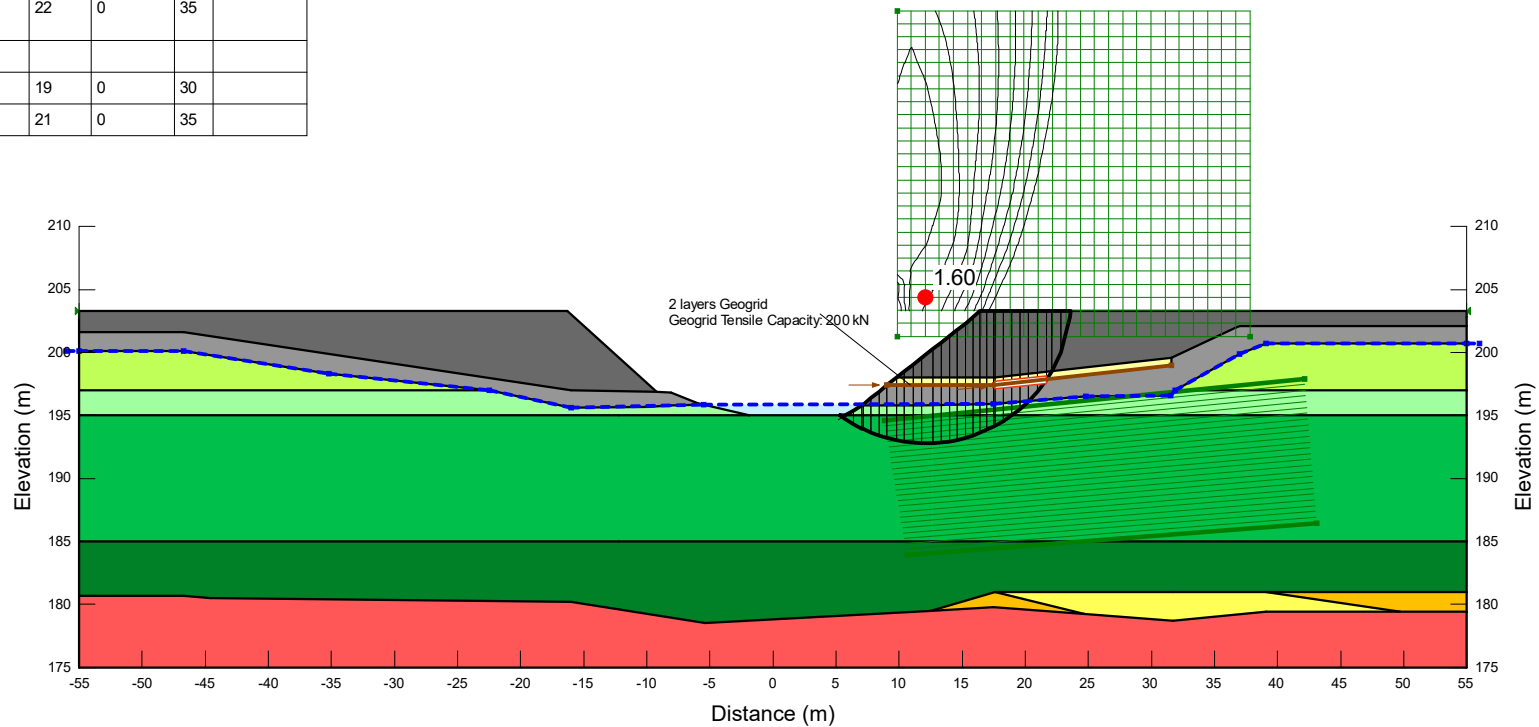
Reviewed by:
SD

Analysis Details:

Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1 m
Center: (12.106812, 204.36636) m w/ Radius: 11.567333 m
PWP Conditions from: Piezometric Line
Horz Seismic Coef.: 0
Surcharge (Unit Weight): 19 kN/m³

Figure 14

Color	Name	Unit Weight (kN/m³)	Cohesion (kPa)	Phi (°)	Anisotropic Strength Fn
	C0 - Clayey SILT (ESA)	18.5	0	30	
	C1 - Upper Clayey SILT (ESA)	18.5	0	27	
	C2 - CLAY 1 (ESA)	17	0	27	Varved Clay - ESA
	C3 - Lower Clayey SILT (ESA)	17	0	30	
	F2 - SILT/FILL	19	0	30	
	F4 - ROCKFILL	19	0	42	
	Granular A or B Type II	22	0	35	
	R1 - Bedrock				
	S1 - SAND	19	0	30	
	T1 - TILL	21	0	35	



Project Name:
Laronde Creek Bridge

Analysis Title:
East App -Forward Drained - Geogrid

Project No.:
23411

Date:
01/28/2020

Seismic Coeff.:
H: 0g, V: 0g

Scale:
1:600

Prepared by:
Deanna Pizyck

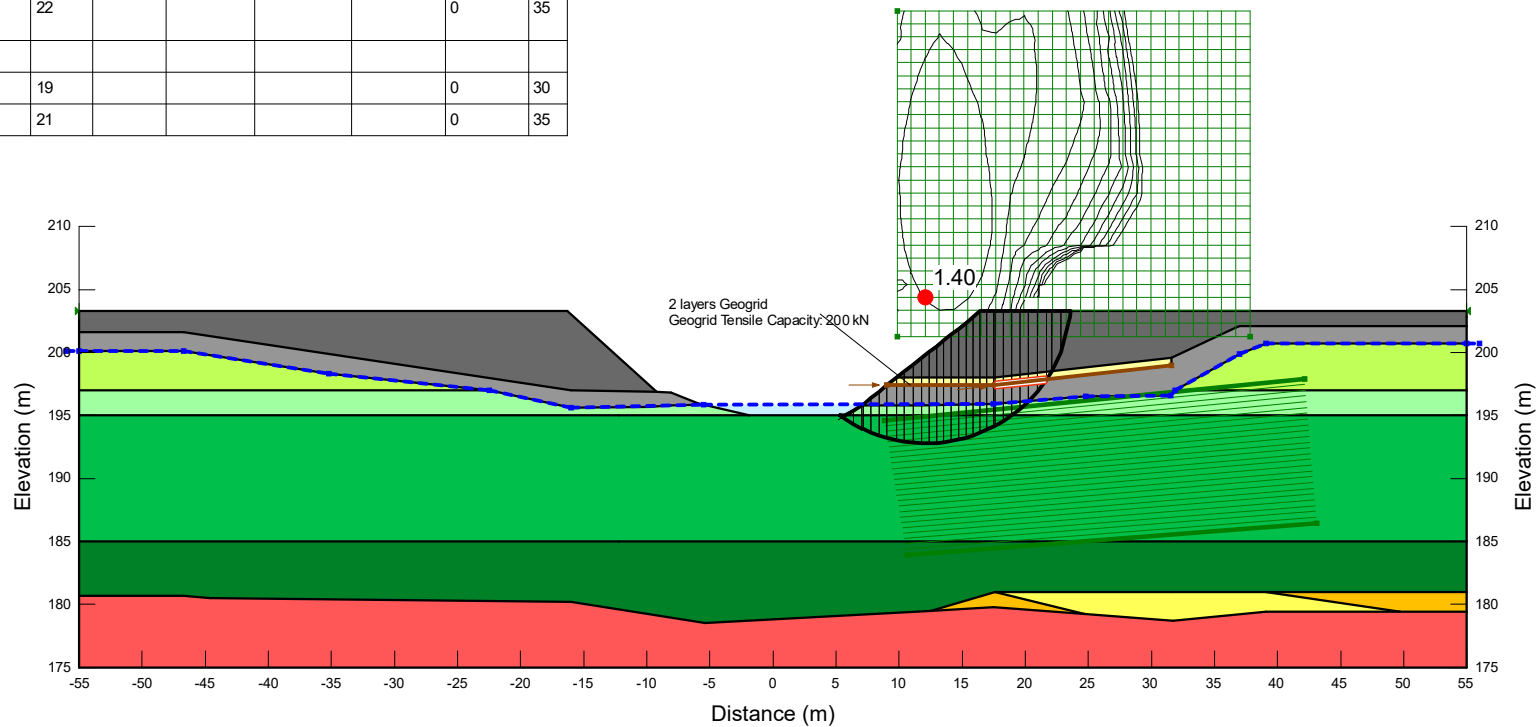
Reviewed by:
SD

Analysis Details:

Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1 m
Center: (12.106812, 204.36636) m w/ Radius: 11.567333 m
PWP Conditions from: Piezometric Line
Horz Seismic Coef.: 0

Figure 15

Color	Name	Unit Weight (kN/m³)	C-Datum (kPa)	C-Rate of Change ((kN/m²)/m)	C-Maximum (kPa)	Anisotropic Strength F _n	Cohesion* (kPa)	Phi* (°)
	C0 - Clayey SILT (TSA)	19					75	0
	C1 - Upper Clayey SILT (TSA)	18.5	55	-12.5	30			
	C2 - CLAY 1 (TSA) (Check)	17	34.5	0.68	41	Varved Clay - TSA		
	C3 - Lower Clayey SILT (TSA)	17					62	0
	F2 - SILT/FILL	19					0	30
	F4 - ROCKFILL	19					0	42
	Granular Aor B Type II	22					0	35
	R1 - Bedrock							
	S1 - SAND	19					0	30
	T1 - TILL	21					0	35



Project Name:
Laronde Creek Bridge

Analysis Title:
East Approach -Forward Siesmic - Geogrid

Project No.:
23411

Date:
01/28/2020

Seismic Coeff.:
H: 0.066g, V: 0g

Scale:
1:600

Prepared by:
Deanna Pizyck

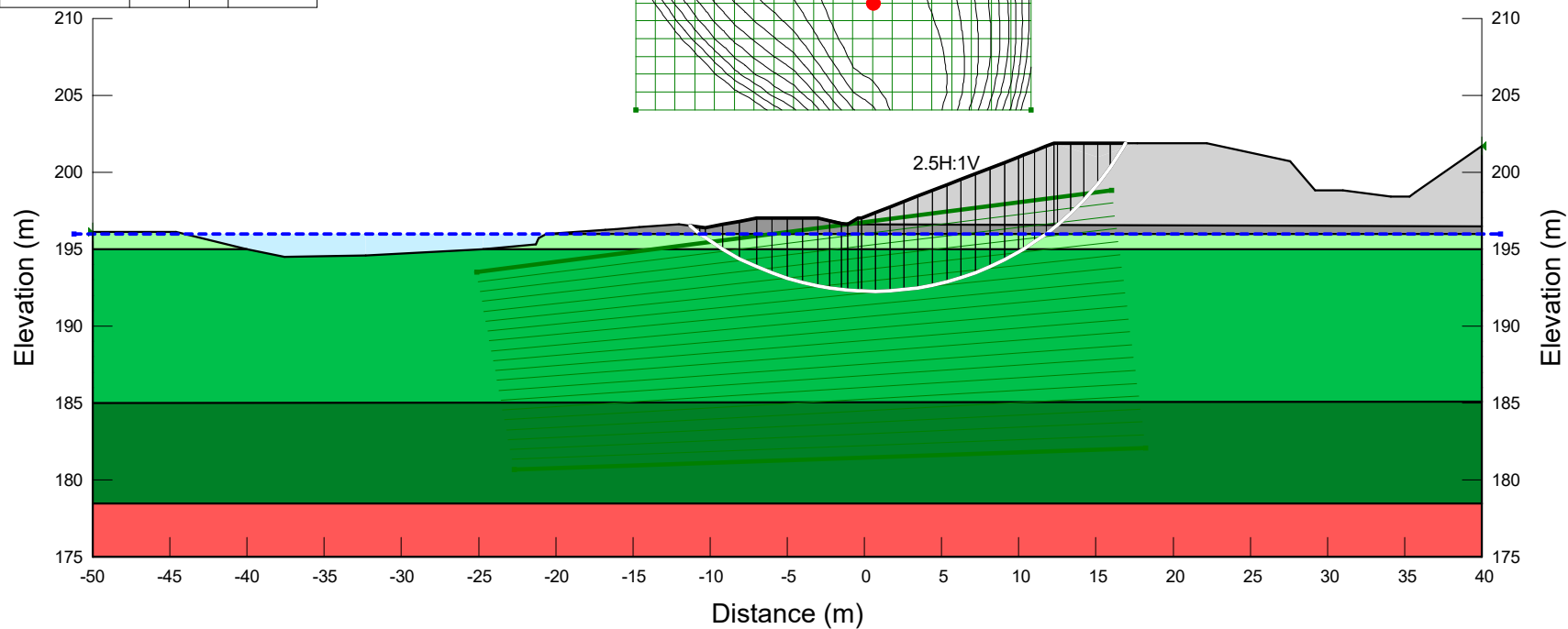
Reviewed by:
SD

Analysis Details:

Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1 m
Center: (12.106812, 204.36636) m w/ Radius: 11.567333 m
PWP Conditions from: Piezometric Line
Horz Seismic Coef.: 0.066

Figure 16

Color	Name	Unit Weight (kN/m³)	Phi° (°)	Anisotropic Strength Fn
■	C1 - Upper Clayey SILT (ESA)	18.5	27	
■	C2 - CLAY 1 (ESA)	17	27	Varved Clay - ESA
■	C3 - Lower Clayey SILT (ESA)	17	30	
■	F2 - SILT/FILL	19	30	
■	F8 - Existing FILL	22	32	
■	R1 - Bedrock			



Project Name:
Laronde Creek Bridge

Analysis Title:
West Approach - Side Existing

Project No.:
23411

Date:
01/27/2020

Seismic Coeff.:
H: g, V: g

Scale:
1:450

Prepared by:
Deanna Pizyck

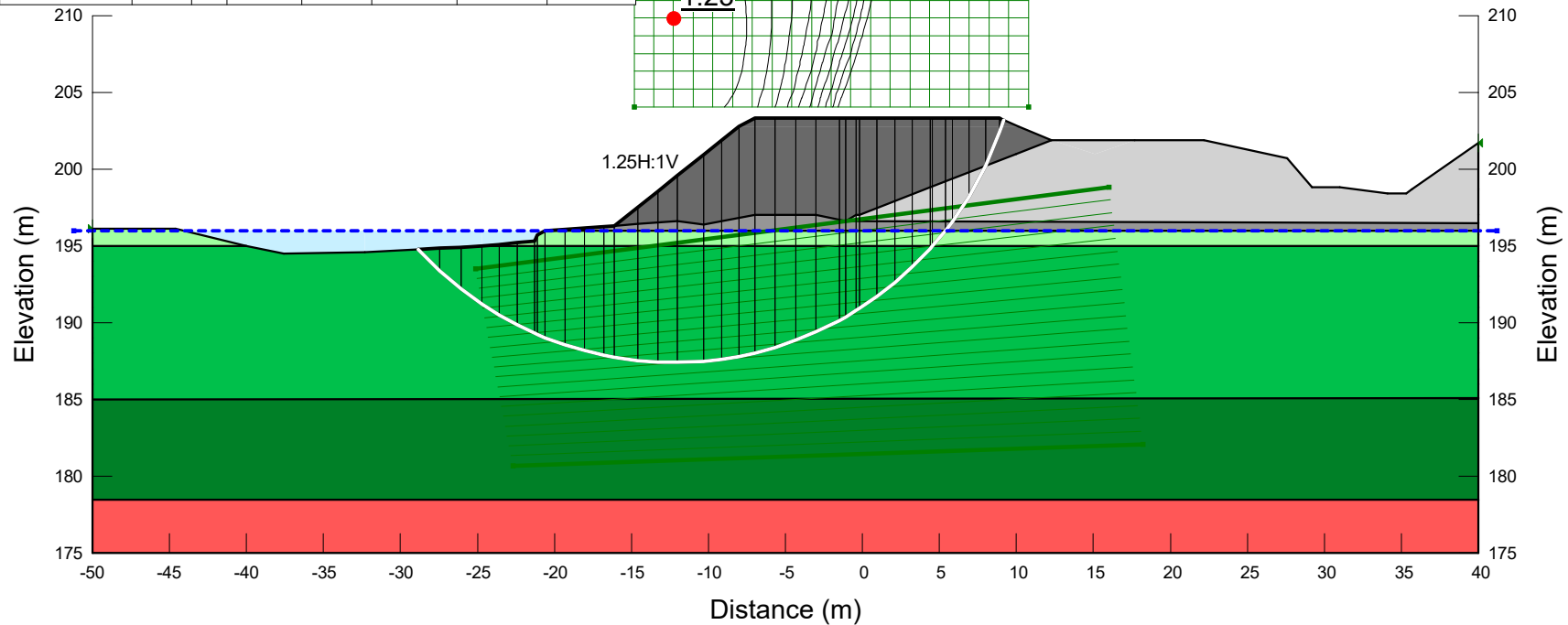
Reviewed by:
SD

Analysis Details:

Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1 m
Center: (0.56, 210.95834) m w/ Radius: 18.69412 m
PWP Conditions from: Piezometric Line
Horz Seismic Coef.:

Figure 17

Color	Name	Unit Weight (kN/m³)	Phi° (°)	Cohesion (kPa)	C-Datum (kPa)	C-Rate of Change ((kN/m²)/m)	C-Maximum (kPa)	Anisotropic Strength F _n
■	C1 - Upper Clayey SILT (TSA)	18.5			55	-12.5	30	
■	C2 - CLAY 1 (TSA) (Check)	17			34.5	0.68	41	Varved Clay (TSA)
■	C3 - Lower Clayey SILT (TSA)	17		62				
■	F2 - SILT/FILL	19	30					
■	F4 - ROCKFILL	19	42					
■	F8 - Existing FILL	22	32					
■	R1 - Bedrock							



Project Name:
Laronde Creek Bridge

Analysis Title:
West Approach - Side Undrained

Project No.:
23411

Date:
01/27/2020

Seismic Coeff.:
H: g, V: g

Scale:
1:450

Prepared by:
Deanna Pizyck

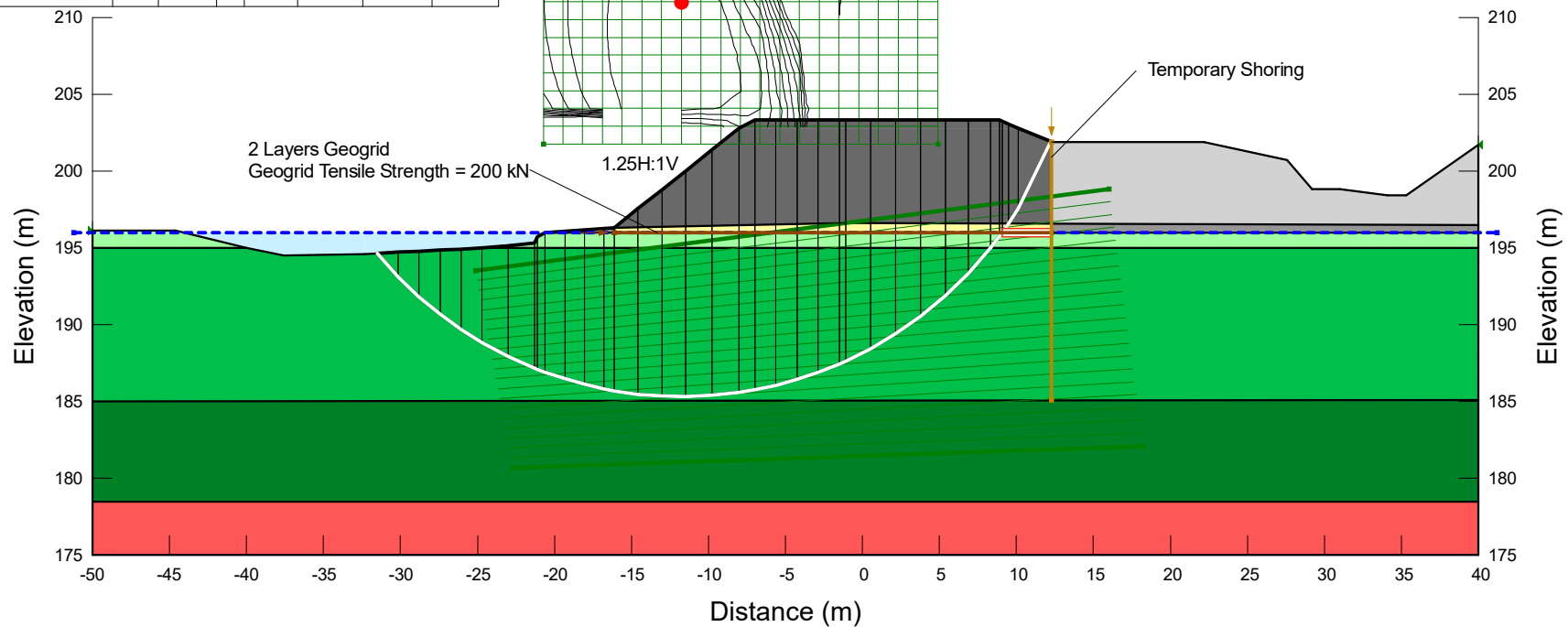
Reviewed by:
SD

Analysis Details:

Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1 m
Center: (-12.24, 209.80334) m w/ Radius: 22.388884 m
PWP Conditions from: Piezometric Line
Horz Seismic Coef.:

Figure 18

Color	Name	Unit Weight (kN/m³)	Cohesion (kPa)	Phi (°)	C-Datum (kPa)	C-Rate of Change ((kN/m²)/m)	C-Maximum (kPa)	Anisotropic Strength F _n
	C1 - Upper Clayey SILT (TSA)	18.5		55	-12.5	30		
	C2 - CLAY 1 (TSA) (Check)	17		34.5	0.68	41		Varved Clay (TSA)
	C3 - Lower Clayey SILT (TSA)	17	62	0				
	F2 - SILT/FILL	19	0	30				
	F4 - ROCKFILL	19	0	42				
	F6 - Existing FILL	22	0	32				
	Granular A or B Type II	22	0	35				
	R1 - Bedrock							



Project Name:
Laronde Creek Bridge

Analysis Title:
West Approach - Side Undrained -Geogrid -Shoring

Project No.:
23411

Date:
01/28/2020

Seismic Coeff.:
H: g, V: g

Scale:
1:450

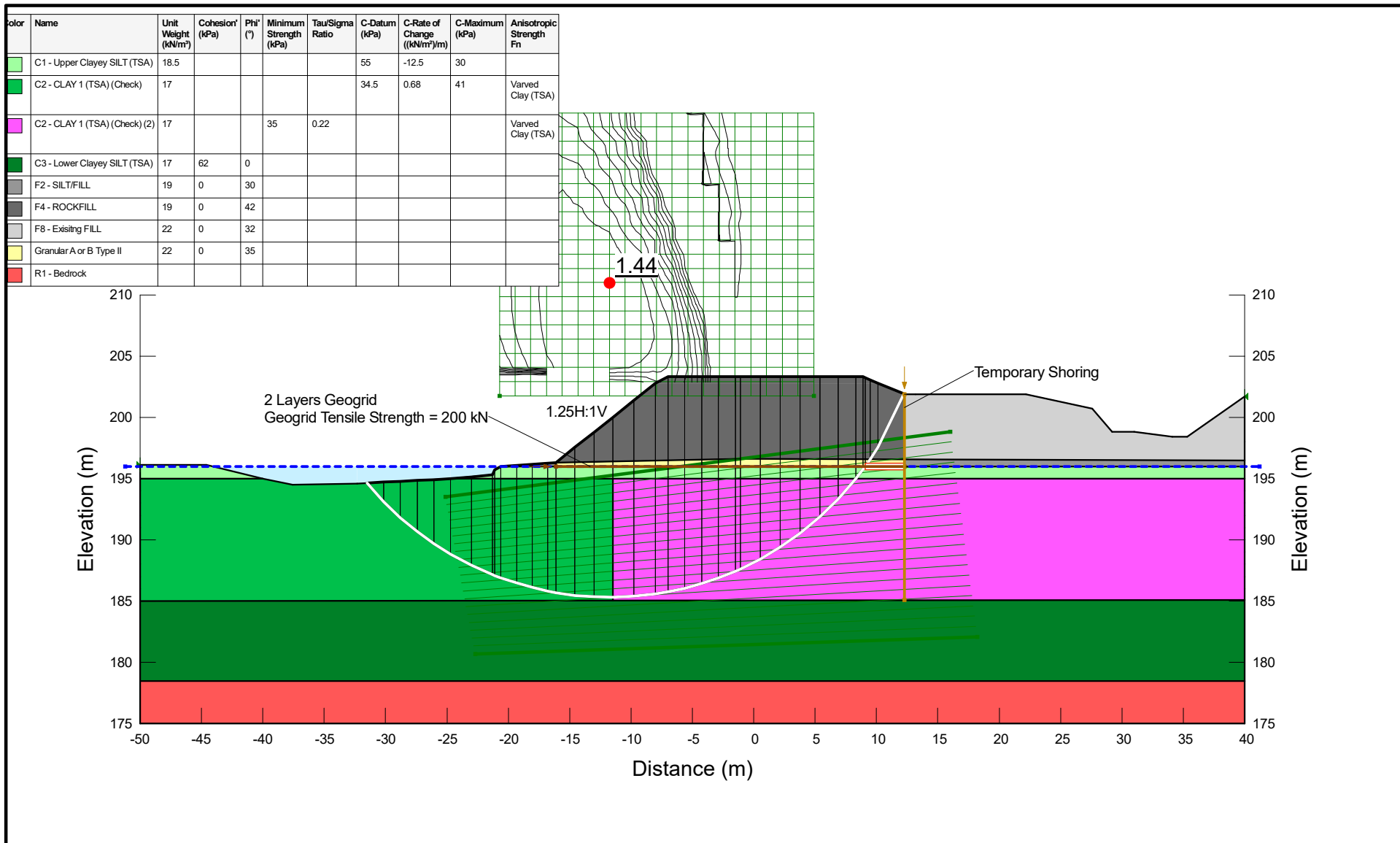
Prepared by:
Deanna Pizyck

Reviewed by:
SD

Analysis Details:

Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1 m
Center: (-11.74, 210.96834) m w/ Radius: 25.645058 m
PWP Conditions from: Piezometric Line
Horz Seismic Coef.:

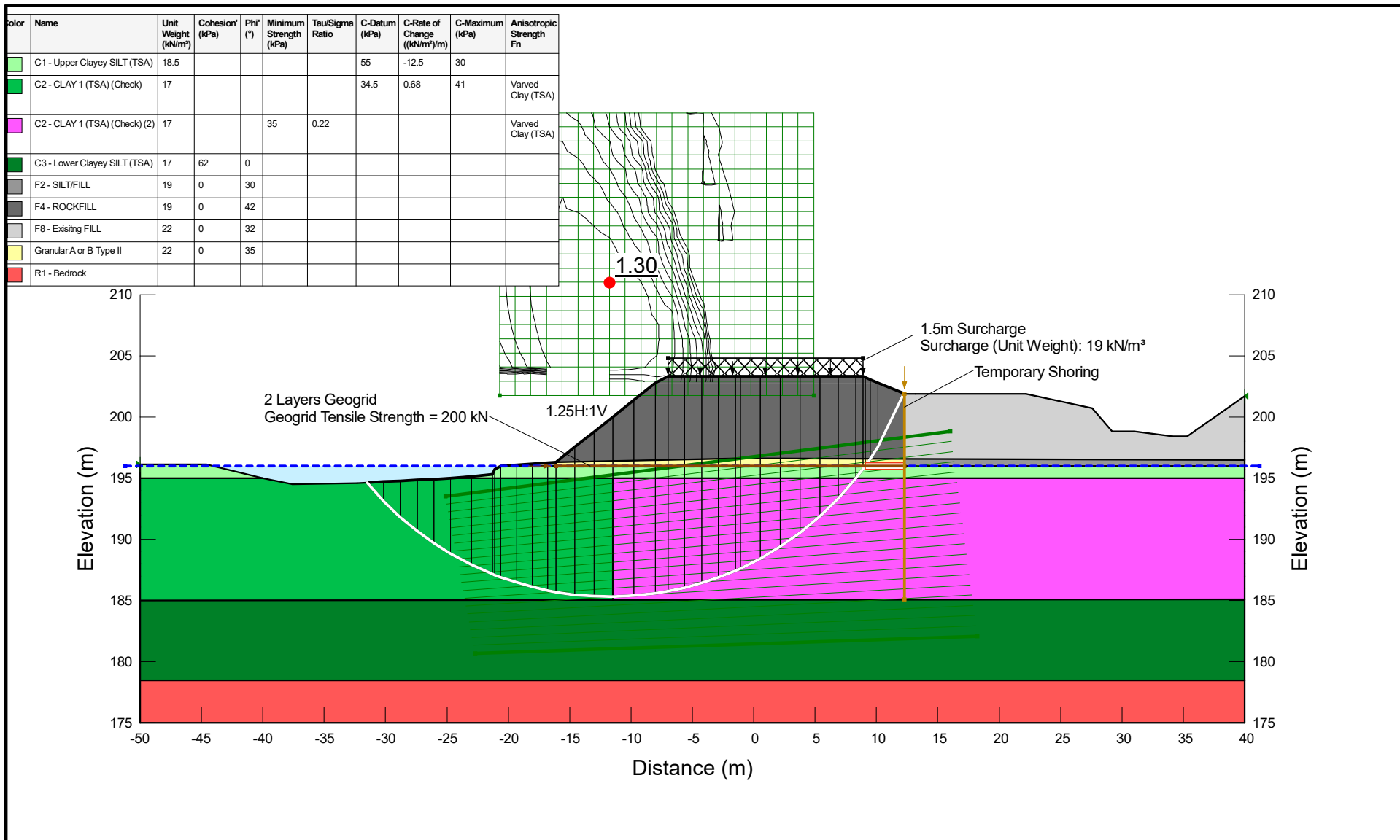
Figure 19



Project Name: Laronde Creek Bridge		
Analysis Title: West App - Side Undr -Geogrid -Shoring -Stgh Gn		
Project No.: 23411	Seismic Coeff.: H: g, V: g	Prepared by: Deanna Pizyck
Date: 01/28/2020	Scale: 1:450	Reviewed by: SD

Analysis Details:
 Method: Morgenstern-Price, Half-Sine
 Minimum Slip Surface Depth: 1 m
 Center: (-11.74, 210.96834) m w/ Radius: 25.645058 m
 PWP Conditions from: Piezometric Line
 Horz Seismic Coef.:

Figure 20



Project Name:

Laronde Creek Bridge

Analysis Title:

West App - Side Undr -Geogrid -Shoring -Stgh Gn -1.5m Srcg

Project No.:

23411

Seismic Coeff.:

H: g, V: g

Prepared by:

Deanna Pizyck

Date:

01/28/2020

Scale:

1:450

Reviewed by:

SD

Analysis Details:

Method: Morgenstern-Price, Half-Sine

Minimum Slip Surface Depth: 1 m

Center: (-11.74, 210.96834) m w/ Radius: 25.645058 m

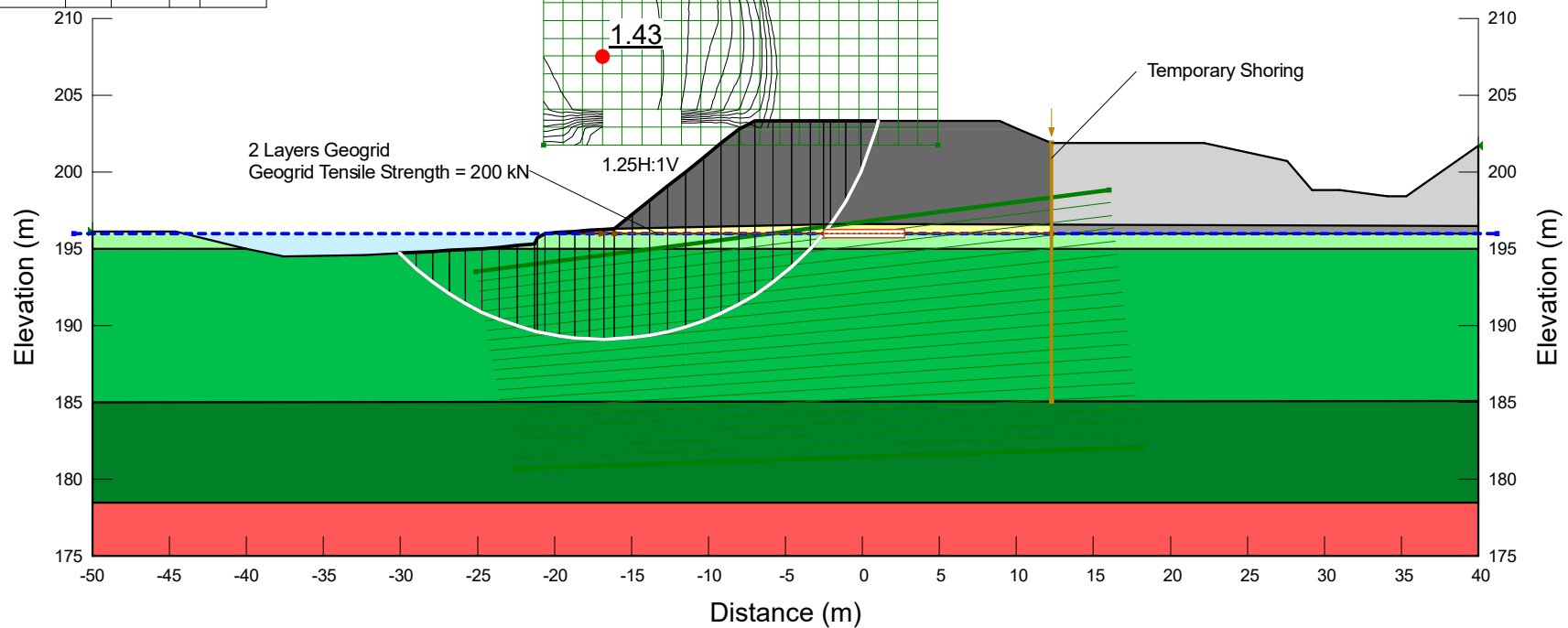
PWP Conditions from: Piezometric Line

Horz Seismic Coef.:

Surcharge (Unit Weight): 19 kN/m³

Figure 21

Color	Name	Unit Weight (kN/m ³)	Cohesion* (kPa)	Phi* (°)	Anisotropic Strength F _n
	C1 - Upper Clayey SILT (ESA)	18.5	0	27	
	C2 - CLAY 1 (ESA)	17	0	27	Varved Clay - ESA
	C3 - Lower Clayey SILT (ESA)	17	0	30	
	F2 - SILT/FILL	19	0	30	
	F4 - ROCKFILL	19	0	42	
	F8 - Existing FILL	22	0	32	
	Granular A or B Type II	22	0	35	
	R1 - Bedrock				



Project Name:
Laronde Creek Bridge

Analysis Title:
West Approach - Side Drained -Geogrid -Shoring

Project No.:
23411

Date:
01/28/2020

Seismic Coeff.:
H: g, V: g

Scale:
1:450

Prepared by:
Deanna Pizyck

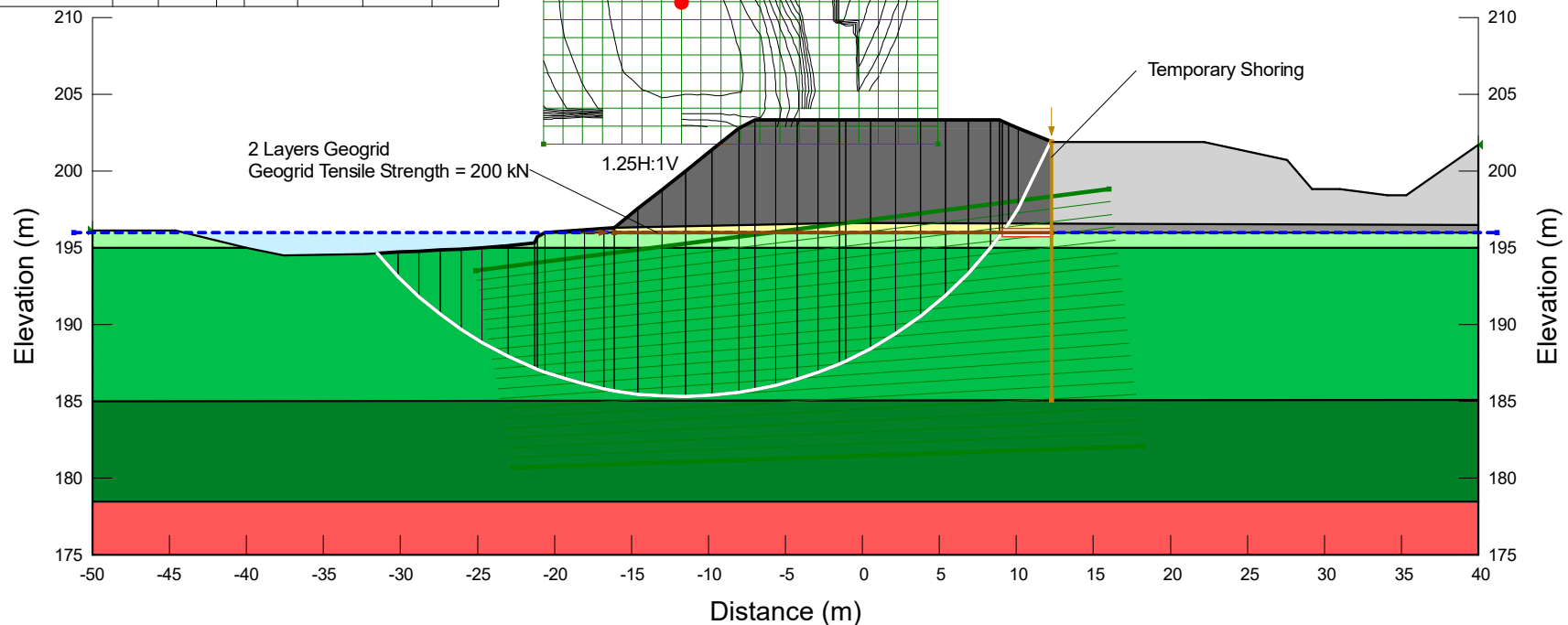
Reviewed by:
SD

Analysis Details:

Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1 m
Center: (-16.86, 207.50334) m w/ Radius: 18.383335 m
PWP Conditions from: Piezometric Line
Horz Seismic Coef.:

Figure 22

Color	Name	Unit Weight (kN/m ³)	Cohesion (kPa)	Phi (°)	C-Datum (kPa)	C-Rate of Change ((kN/m ²)/m)	C-Maximum (kPa)	Anisotropic Strength F _n
	C1 - Upper Clayey SILT (TSA)	18.5		55	-12.5	30		
	C2 - CLAY 1 (TSA) (Check)	17		34.5	0.68	41		Varved Clay (TSA)
	C3 - Lower Clayey SILT (TSA)	17	62	0				
	F2 - SILT/FILL	19	0	30				
	F4 - ROCKFILL	19	0	42				
	F6 - Existing FILL	22	0	32				
	Granular A or B Type II	22	0	35				
	R1 - Bedrock							



Project Name:
Laronde Creek Bridge

Analysis Title:
West Approach - Side Seismic -Geogrid -Shoring

Project No.:
23411

Date:
01/28/2020

Seismic Coeff.:
H: 0.066g, V: g

Scale:
1:450

Prepared by:
Deanna Pizyck

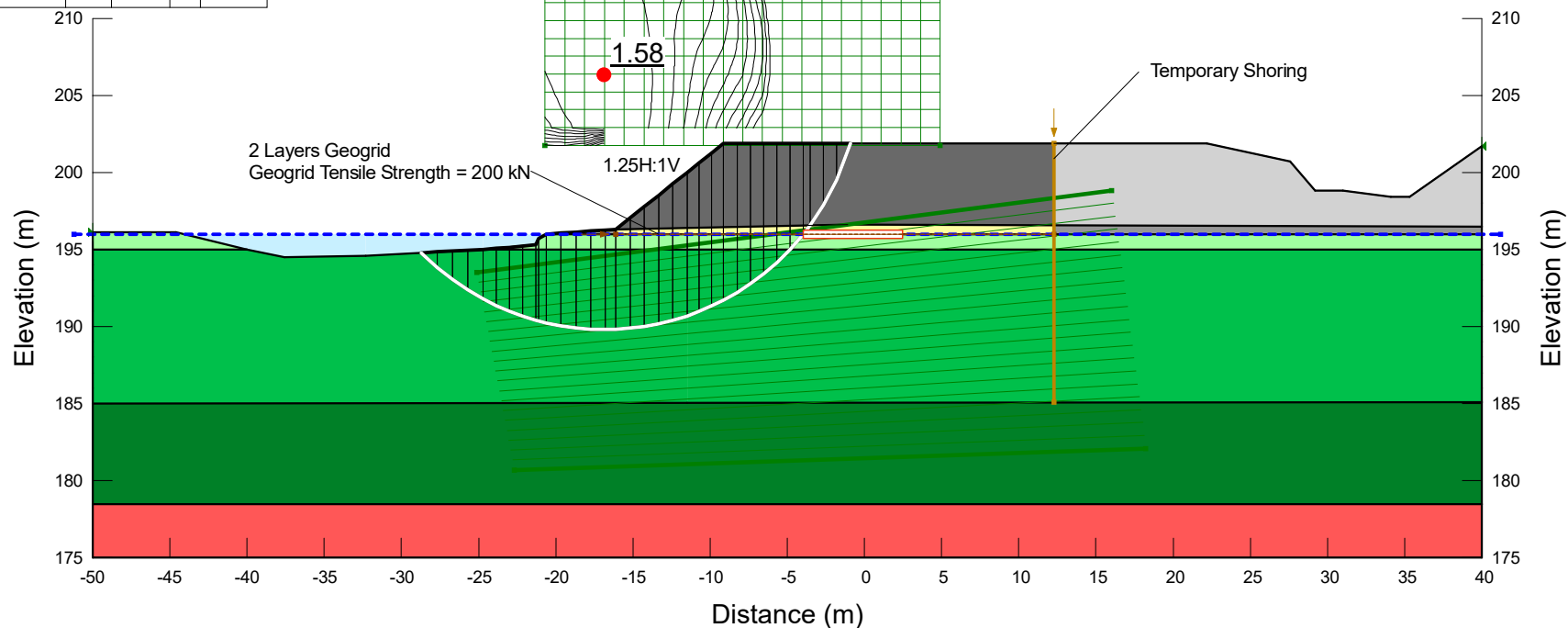
Reviewed by:
SD

Analysis Details:

Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1 m
Center: (-11.74, 210.96834) m w/ Radius: 25.645058 m
PWP Conditions from: Piezometric Line
Horz Seismic Coef.: 0.066

Figure 23

Color	Name	Unit Weight (kN/m ³)	Cohesion* (kPa)	Phi* (°)	Anisotropic Strength Factor
	C1 - Upper Clayey SILT (ESA)	18.5	0	27	
	C2 - CLAY 1 (ESA)	17	0	27	Varved Clay - ESA
	C3 - Lower Clayey SILT (ESA)	17	0	30	
	F2 - SILT/FILL	19	0	30	
	F4 - ROCKFILL	19	0	42	
	F8 - Existing FILL	22	0	32	
	Granular A or B Type II	22	0	35	
	R1 - Bedrock				



Project Name:
Laronde Creek Bridge

Analysis Title:
West App - Side Drained -Geogrid -Shoring -Lower Grd

Project No.:
23411

Date:
01/28/2020

Seismic Coeff.:
H: g, V: g

Scale:
1:450

Prepared by:
Deanna Pizyck

Reviewed by:
SD

Analysis Details:

Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1 m
Center: (-16.86, 206.34834) m w/ Radius: 16.542735 m
PWP Conditions from: Piezometric Line
Horz Seismic Coef.:

Figure 24



Appendix J.

List of Special Provisions and OPSS Documents Referenced in this Report Suggested Text for NSSPs and SP for EPS



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

OPSS.PROV 206	Construction Specification for Grading
OPSD 3090.101	Foundation, Frost Penetration Depths for Southern Ontario
OPSS.PROV 501	Construction Specification for Compacting
OPSS 511	Construction Specification for Rip-Rap, Rock Protection, and Granular Sheetting
OPSS.PROV 804	Construction Specification for Seed and Cover
OPSS 805	Construction Specification for Temporary erosion and Sediment Control Measures
OPSS 902	Construction Specification for Excavating and Backfilling Structures
OPSS 903	Construction Specification for Deep Foundations
OPSS.PROV 1010	Material Specification for Aggregates Base, Subbase, Select Subgrade, and Backfill Material
OPSS 1860	Material Specification for Geotextile
OPSD 208.010	Benching of Earth Slopes
SP109S12	Amendment to OPSS 902 – QVE, Backfilling Compaction, and Certificate of Conformance
SPFOUN0003	Amendment to OPSS 902 – Dewatering Structure Excavations

2. Suggested text for a NSSP on “Pile Obstructions”

Pile driving at the site may be impeded by obstructions within the glacial till (i.e. cobbles, boulders).

3. Suggested text for a NSSP on “Artesian Groundwater Pressure”

Artesian groundwater pressure is present on this site from the deep till deposit and bedrock. Vertical conduits (e.g., shafts, boreholes, wick drains) must not extend below elevation 185 m without prior written approval from the Contract Administrator. If artesian groundwater flow is encountered, the Contract Administrator must be informed immediately and the artesian flow must be sealed at its source as soon as possible.

4. Suggested text for a NSSP on “Use of Heavy Construction Equipment”

The use of heavy construction equipment and in particular heavy lift cranes may be required during removal of the existing and erection of the new bridge. The impact of the heavy equipment loads on the existing embankment and the native soft to firm clay soils underlying the



embankment must be considered during selection of the methodology and equipment employed for construction.

Prior to commencement of construction, the Contractor shall retain a Geotechnical Consultant to assess the impact of the proposed equipment loads and methodology, and determine requirements and/or restrictions necessary to safely support the loads. All Foundation Engineering services required for this project shall be performed by consultant(s) listed as accepted under the MTO's RAQS for providing services under the specialty of Geotechnical (Structures and Embankments) - High Complexity.

The assessment shall include, but not be limited to, the following:

- Determining appropriate setbacks for heavy equipment from the bridge abutments and existing foundations;
- Determining the permissible ground pressure that may be applied to the foundation soils by the equipment; and
- Providing recommendations for crane pad design to distribute the crane loads without causing foundation failure.

The Contractor shall submit the findings of the geotechnical assessment and details of the proposed equipment and construction methodology to the Contract Administrator for information purposes a minimum of two weeks prior to the start of construction.

5. Suggested text for the following SPs/NSSPs provided on subsequent pages.

- CSP for Integral Abutment
- Steel Pile High Strain Dynamic Testing
- Expanded Polystyrene Embankment
- Structure Monitoring
- Dewatering Structure Excavations (FOUN0003)

TABLE 1
Elastomeric Bearing Test Samples for Destructive Testing

Number of Samples to be Tested	Dimensions	Plain or Laminated	Number of Bearings Represented by Sample	Structure Identification	Bearings Location Represented by Test Sample
1	550x550x150	Laminated	6	Nicholas Street UP	Pier

922.10 BASIS OF PAYMENT

922.10.02.01 Elastomeric Bearings with Steel Laminates

Clause 922.10.02.01 of OPSS 922 is amended by the addition of the following to the third paragraph.

For the purpose of administering this payment adjustment, where rotational bearings are included in this tender item Bearings, the total value of all elastomeric laminated bearings shall be considered to be one half of the total value of this tender item. Where only elastomeric bearings are included in this tender item, the total value of all elastomeric laminated bearings shall be considered the total value of this tender item.

CSP FOR INTEGRAL ABUTMENT - Item No. 184

Special Provision

1.0 SCOPE

This specification covers the requirements for the installation of the Corrugated Steel Pipe (CSP) at the abutments, including augering, sand fill, and unshrinkable fill.

2.0 REFERENCES

This specification refers to the following standards, specification or publications:

Ontario Provincial Standard Specifications, General

OPSS 180 Management and Disposal of Excess Materials

Ontario Provincial Standard Specifications, Material

OPSS 1359 Unshrinkable Backfill

OPSS 1801 Corrugated Steel Pipe Products

Canadian Standards Association Standards

CSA G164-M Galvanizing of Irregularly-Shaped Articles

Ministry of Transportation Publications

MTO Manual of Designated Sources of Materials

3.0 DEFINITIONS

For the purposed of this specification, the following definitions apply:

Abutment Stem Wall: means the cast-in-place concrete component of the abutment placed over the top of the piles and forming the bearing seat for the girders.

CSP: means helical corrugated steel pipe.

Engineer: means an Engineer licensed to practice in the Province of Ontario who has a minimum of five (5) years experience in undertaking design and/or construction of bridges of similar nature and scope to the required work.

4.0 DESIGN AND SUBMISSION REQUIREMENTS**4.01 Submissions**

The Contractor shall submit the following to the Contract Administrator at least 14 Days prior to construction, for information purposes only:

- a) Detailed procedures for augering and installation of CSP.
- b) Source of the sand fill, and description of placing method and equipment.

All submissions shall bear the seal and signature of an Engineer.

5.0 MATERIALS**5.01 Corrugated Steel Pipe**

CSP shall be accordance with OPSS 1801. The CSP shall be of the diameter specified on the Contract Drawings with wall thickness of minimum 1.6 mm and shall be galvanized in accordance with CSA G164-M.

5.02 Sand Fill

The sand fill for backfilling in the CSP shall meet the gradation requirements of Table A below:

Table A – Sand Fill Gradation Requirements		
MTO Sieve Designation		Percentage Passing by Mass
2 mm	# 10	100 %
600 µm	# 30	80 % to 100 %
425 µm	# 40	40 % to 80 %

250 μ m	# 60	5 % to 25 %
150 μ m	# 100	0 % to 6 %

5.03 Unshrinkable Fill

Unshrinkable fill shall be according to OPSS 1359.

6.0 EQUIPMENT - Not Used

7.0 CONSTRUCTION

7.01 General

The sequence of construction for installing the CSP's, H-Piles, sand fill and substructure components shall be as specified in the Contract Documents and Working Drawings developed for the pile installation.

Augering shall not be completed until excavation work to expose the existing piles has been completed to the limits shown on the Contract Drawings.

In the event of a conflict with the existing piles during installation of the CSP or H-Pile, the Contractor shall immediately notify the Contract Administrator. A revised Working Drawing detailing the proposed relocation shall be submitted to the Contract Administrator as specified elsewhere in the Contract Documents.

Should the required adjustment to avoid conflict with the existing piles be outside of the allowable tolerances, extraction or overdriving of the existing pile shall be considered for approval by the Contract Administrator, as specified elsewhere in the Contract Documents.

7.02 Corrugated Steel Pipe

7.02.01 General

CSP's shall be supplied in the lengths as specified on the Contract Drawings. Cut ends shall be neat and free of burrs.

CSP's shall be handled and stored in such a manner to avoid damage or distortion to them. Damaged or distorted CSP's shall be rejected as determined by the Contract Administrator. Localized areas of damaged galvanizing or cut ends shall be repaired with two coats of zinc-rich paint.

The Contractor shall ensure the full perimeter of the CSP at the base of the abutment stem wall is at the elevation shown on the Contract Drawings.

When placement of the CSP is completed, the annular voids between the augered hole and CSP shall be filled with unshrinkable fill. Following the installation of the CSP, the Contractor shall take all measures necessary to prevent the ingress of water, backfill and debris into the CSP.

7.02.02 Tolerances

The CSP's at each pile shall be constructed to the following tolerances:

Criteria	Tolerance
----------	-----------

Maximum deviation of CSP from pile centroid.	± 50 mm
Maximum deviation of any point on the top perimeter of the CSP's from the specified elevation	± 10 mm

7.03 Sand Fill

The sand fill shall be placed dry of optimum and free-flowing, completely filling the volume between the CSP and the pile. No additional compaction effort other than the action of placing the sand fill itself shall be applied to the sand fill.

The placing of the sand fill shall be carried out in a manner such as to not damage and/or displace the CSP's.

After the sand fill has been placed to the top of each CSP, the Contractor shall take all measures necessary to prevent the ingress of water and other liquids into the sand fill until after the concrete in the abutment stem has been placed and cured.

8.0 QUALITY CONTROL

Prior to augering the holes for installation of CSP's, the Contractor shall ensure that the centroid of each pile in the abutment and associated reference points have been established, as specified elsewhere in the Contract Documents. The Contractor shall maintain the reference points until written permission to proceed with the installation of the abutments stem walls has been given by the Contract Administrator.

9.0 MEASUREMENT FOR PAYMENT - Not Used

10.0 BASIS OF PAYMENT

Payment at the Contract price for the above tender item shall be full compensation for all labour, equipment and material required to do the work.

Any additional labour, materials, or equipment that is required due to conflicts with the existing piles during installation of the CSP shall be deemed to be included as part of the work. In the event of a conflict with the existing piles following the installation of the CSP liner, payment for the extraction and relocation of the CSP(s) shall be administered in accordance with the provisions for extra work.

STEEL PILE HIGH STRAIN DYNAMIC TESTING – Item No.

Special Provision

Amendment to OPSS 903, April 2016

903.02 REFERENCES

Section 903.02 of OPSS 903 is amended by the addition of the following under ASTM International:

D 4945-12 Standard Test Method for High-Strain Dynamic Testing of Deep Foundations

903.03 DEFINITIONS

Section 903.03 of OPSS 903 is amended by the addition of the following:

High Strain Dynamic Testing means a method of evaluating the quality of deep foundations and/or performance of the drive system. It is a form of load testing and involves the instrumenting and application of dynamic loads to a tested pile.

903.04 DESIGN AND SUBMISSION REQUIREMENTS

903.04.02 Submission Requirements

Subsection 903.04.02 of OPSS 903 is amended by the addition of the following clause:

903.04.02.07 High-Strain Dynamic Testing

Prior to commencing high-strain dynamic testing, calibration certificates of all equipment used shall be submitted to the Contract Administrator. All equipment used shall be in good working condition, and shall have been calibrated within the last 2 years according to ASTM D 4945. Equipment set-up may be completed by trained Contractor personnel, however, testing shall be performed under the direction of an Engineer with at least 5 years of experience in high-strain dynamic testing and holding a proficiency rating at the Intermediate level or better for Dynamic Measurement and Analysis Proficiency Test as administered by the Pile Driving Contractors Association (PDCA). After December 31, 2020, the Engineer shall be required to hold a proficiency rating level of Advanced or better.

A preliminary report on the test results and its analysis shall be submitted to the Contract Administrator on the same day of the testing. The analysis shall be based on a closed-form solution (Case Method or approved equivalent) or signal-matching analyses (Case Pile Wave Analysis Program - CAPWAP or approved equivalent). As a minimum, the preliminary report shall include:

- a) Pile ultimate resistance and integrity.
- b) Calculated driving stresses.
- c) Transferred energy and hammer efficiency at the time of the test.

A final report shall be submitted to the Contract Administrator within 10 Days of the field testing. The final report shall include the following:

- a) Results of pile ultimate resistance and pile integrity based on signal-matching analyses (CAPWAP or approved equivalent), hammer performance and comparisons with any applicable static load test.
- b) Discussion and recommendations for soil setup/relaxation, and/or revised pile installation criteria.
- c) An appendix shall be included containing the following documents:
 - i. Pile installation record
 - ii. Reference subsurface information (borehole record)
 - iii. Pile location drawing
 - iv. Initial calibration check by the test computer unit
 - v. Test set up geometry

The report shall be signed and sealed by two Engineers of the testing company, one of whom shall be identified as MTO's designated contact and one of whom shall have the required experience in high-strain dynamic testing and hold the required certificate of PDCA Proficiency Test.

903.07 CONSTRUCTION

903.07.02.07 Monitoring Driven Piles

903.07.02.07.03 Driving to a Specified Ultimate Resistance

903.07.02.07.03.01 General

Clause 903.07.02.07.03.01 of OPSS 903 is deleted in its entirety and replaced with the following:

When piles are specified to be driven to a specified ultimate resistance, the specified ultimate resistance shall be validated using high-strain dynamic testing at end of drive (EOD). If the specified ultimate resistance is not achieved, retap/restrike should be conducted after sufficient time has passed to allow soil setup (minimum of 1 week).

The results of the high-strain dynamic tests shall be submitted to the Contract Administrator who shall, in collaboration with the independent testing company, verify that the specified ultimate resistance has been achieved.

903.07.02.07.04 Wave Equation Analysis

Clause 903.07.02.07.04 is deleted in its entirety and replaced with the following:

903.07.02.07.04 Wave Equation Analysis and High-Strain Dynamic Testing

903.07.02.07.04 .01 Wave Equation Analysis

Prior to mobilizing piling equipment to the site, a Wave Equation Analysis of Piles (WEAP) analysis shall be performed by the Contractor to demonstrate the potential for the proposed piling equipment to activate the specified ultimate resistance specified in the Contract Documents.

When requested by the Contract Administrator, all equipment, material, and personnel shall be supplied to conduct the wave equation analysis procedure.

903.07.02.07.04 .02 High-Strain Dynamic Testing

An independent testing company with no corporate affiliation with the Contractor shall be employed to perform the high-strain dynamic testing. The independent testing company shall be RAQs qualified (Specialty: Geotechnical (Structures and Embankments – Medium or High Complexity)).

High-strain dynamic tests shall be performed by an Engineer employed by the independent testing company. The Engineer shall have documented evidence of training and experience in foundation engineering and wave equation analyses, and a certificate of proficiency (intermediate level or better) in the PDCA Dynamic Measurement and Analysis Proficiency Test.

High-strain dynamic testing shall be performed using the Pile Driving Analyzer, or approved equivalent, for the determination of pile ultimate resistance, establishment of pile installation criteria, assessment of pile integrity, monitoring of hammer/drive system performance and driving stresses, as specified in the Contract Documents. The method and equipment for testing and its reporting shall be according to ASTM D 4945.

The location, sequencing and scheduling of the individual pile testing shall be proposed by the Contractor based on the purpose of the testing, and shall be submitted to the Contract Administrator for information purposes.

High-strain dynamic testing shall be carried out at the end of initial driving on a minimum of 2 piles per abutment; or as specified in the Contract Documents.

Additional high strain dynamic testing (i.e. restrike testing) shall be carried out during the retapping of piles, as specified in the Retapping Tests on Piles clause. Restrike testing shall be performed on a minimum of 2 piles in each pile group; or as specified in the Contract Documents.

Restrike testing shall be carried out no sooner than 24 hours after installation of the individual pile and at a time specified in the Contract Documents. If the hammer needs to be warmed up prior to performing a restrike, it shall not be warmed up by striking the intended test pile.

903.10 BASIS OF PAYMENT

Section 903.10 of OPSS 903 is amended by the addition of the following subsection:

903.10.04 High-Strain Dynamic Testing, Deep Foundations - Item

Payment for the above tender item shall be full compensation for all labour, Equipment and Material to do the work.

EXPANDED POLYSTYRENE EMBANKMENT – Item No.

Special Provision

1.0 SCOPE

This Special Provision covers the requirements for the supply and installation of Rigid Expanded Polystyrene (EPS) embankment fill and associated works as shown on the Contract Drawings.

2.0 REFERENCES

This Special Provision refers to the following standards, specifications or publications.

Ontario Provincial Standard Specifications, Construction

OPSS 212	Construction Specification for Earth Borrow
OPSS 501	Construction Specification for Compacting
OPSS 517	Construction Specification for Dewatering
OPSS 904	Construction Specification for Concrete Structures

Ontario Provincial Standard Specifications, Materials

OPSS 1010	Aggregates – Base, Subbase, Select Subgrade, and Backfill Material
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National Standards of Canada

CAN/ULC-S102-10	Standard Method of Test for Surface Burning Characteristics of Building Materials and Assemblies
CAN/ULC-S701-97	Thermal Insulation, Polystyrene, Boards and Pipe Covering

ASTM International

ASTM C177	Standard Test Method for Steady-State Heat Flux Measurements and Thermal Transmission Properties by Means of Guarded-Hot-Plate Apparatus
ASTM C203	Standard Test Method for Breaking Load and Flexural Properties of Block-Type Thermal Insulation
ASTM C518	Standard Test Method for Steady-State Thermal Transmission Properties by Means of the Heat Flow Meter Apparatus
ASTM D1621	Standard Test Method for Compressive Properties of Rigid Cellular Plastics
ASTM D2842	Standard Test Method for Water Absorption by Rigid Cellular Plastics
ASTM D2863	Standard Test Method for Measuring the Minimum Oxygen Content
ASTM D6817	Standard Specification for Rigid Cellular Polystyrene Geofoam

3.0 DEFINITIONS

For the purpose of this special provision, the following definitions apply:

Production Lot: means the quantity of rigid polystyrene blocks produced in a continuous period of manufacturing the same grade and thickness of product within the same production day.

Manufacturer: means the firm who supplies the Rigid Expanded Polystyrene

Rigid Expanded Polystyrene (EPS): means moulded rigid blocks produced by a process of pre-expansion, aging and forming of petroleum based raw material.

4.0 DESIGN AND SUBMISSION REQUIREMENTS

4.01 Design

4.01.01 Foundation Investigation Report

The subsurface conditions at the site are described in the Foundation Investigation Report for this Contract.

The Owner warrants the data in the Foundation Investigation Report, except that interpretations of the data and opinions expressed in the Foundation Investigation Report are not warranted.

4.02 Submissions

4.02.01 Working Drawings

At least three (3) weeks before the commencement of work, the Contractor shall submit to the Contract Administrator six copies of the Working Drawings and method statement that provides full details of materials and construction procedure. The submissions shall be signed and sealed by the Contractor's Engineer.

The Contractor shall submit full details of the following.

- a) The method of foundation excavation and preparation.
- b) The method of levelling pad construction.
- c) The method of placement of Rigid Expanded Polystyrene blocks including temporary ballasting and protection of blocks during installation. The shop drawings shall indicate laying pattern and block dimensions on a layer-by-layer basis.
- d) The method and limits of polyethylene sheeting placement.
- e) The method of placement of the reinforced concrete top slab.
- f) The method of placement of subbase material.
- g) The method of placement of side slope cover.

4.02.02 Delivery, Storage, Handling, and Protection Procedure

At least three (3) weeks before the commencement of work, the Contractor shall submit to the Contract Administrator the method of delivery, storage, handling and protection from damage by weather, traffic, construction staging and other causes as per the Rigid Expanded Polystyrene manufacturer's requirement.

4.02.03 Rigid Expanded Polystyrene

At least two (2) weeks prior to commencement of the installation of the Rigid Expanded Polystyrene blocks, the following details shall be submitted in writing to the Contract Administrator:

1. A general statement as to the type, composition, and method of production of the material.
2. The manufacturer's name, address, phone number, identification of a contact person and description of experience background in the manufacturing of the Rigid Expanded Polystyrene.
3. An identification of the laboratory accredited by the Standards Council of Canada to conduct the testing of the physical and mechanical properties of the Rigid Expanded Polystyrene.
4. The physical and mechanical properties of the Rigid Expanded Polystyrene including:
 - a) Geometry
 - b) Nominal Density
 - c) Compressive Strength
 - d) Flexural Strength
 - e) Thermal Resistance
 - f) Flammability
 - g) Water Absorption
5. Aging and durability characteristics of the Rigid Expanded Polystyrene including the chemical, biological and ultra-violet degradation resistance of the rigid polystyrene.
6. A sample of the Rigid Expanded Polystyrene material.

4.02.04 Quality Test Certificates

Prior to installation of the Rigid Expanded Polystyrene, the Contractor shall submit Quality test certification for each production lot supplied from a laboratory accredited by the Standards Council. The Quality test certificates shall demonstrate compliance with all requirements of this special provision.

4.02.05 Rigid Expanded Polystyrene embankment

For each Rigid Expanded Polystyrene embankment, a Request to Proceed shall be submitted to the Contract Administrator at each of the following milestones:

- a) Following submission of the Quality Test Certificate and prior to construction.
- b) Following foundation excavation and preparation and prior to installation of the leveling pad;
- c) Following placement of Rigid Expanded Polystyrene blocks and polyethylene sheeting and prior to construction of the concrete top slab;

The next operation shall not proceed until a Notice to Proceed has been received from the Contract Administrator.

4.02.06 As-Built Drawings

As-built drawings shall be submitted to the Contract Administrator in a reproducible format prior to final acceptance of work.

The as-built drawings shall be signed and sealed by the Contractor's Engineer.

5.0 MATERIALS

5.01 Granular Levelling Pad

The levelling pad shall be as specified elsewhere in the contract documents and shall consist of Granular A material with gradation and physical requirements as specified in OPSS 1010.

5.02 Rigid Expanded Polystyrene

5.02.01 Production Lots

Each block of the same production lot shall be stamped with the same production code showing plant identification, type and date of production. The Rigid Expanded Polystyrene shall be free from defects affecting serviceability.

5.02.02 Detail Requirements

The Rigid Expanded Polystyrene shall meet the physical and mechanical properties requirements shown in Table 1 and as described below.

Table 1 – Material Properties

PROPERTY	UNIT	REQUIREMENTS	TEST PROCEDURE
Geometry - Linear Dimensions - Flatness - Squareness	mm (min)	1200 × 600 × 300 with tolerances ± 1% 10mm in 3m ± 0.5%	--
Compressive Strength at 5% Deformation	kPa (min)	115 (EPS Type 22) 170 (EPS Type 29)	ASTM D1621 (Procedure A)
Flexural Strength	kPa (min)	240 (EPS Type 22) 345 (EPS Type 29)	ASTM C203 (Method 1, Procedure B.2.7.4)
Thermal Resistance	m ² .°C/W (min for 25 mm thickness)	0.7	ASTM C177 or C518
Flammability	Limiting Oxygen Index (min)	24	ASTM D2863
Water Absorption	% by Volume (max)	3 (EPS Type 22) 2 (EPS Type 29)	ASTM D2842

5.03 Polyethylene Sheeting

The protective sheeting shall be at a minimum 6 mil thick polyethylene sheeting or better if specified elsewhere in the Contract Package and shall be free from defects.

5.04 Concrete Top Slab

The reinforced concrete top slab shall be as specified elsewhere in the contract documents.

6.0 EQUIPMENT

All cutting of Rigid Expanded Polystyrene materials shall be by electric equipment or by hand.

Heavy equipment shall be limited in weight and size and restricted in operation to avoid damaging the Rigid Expanded Polystyrene as per the manufacturer's requirement.

7.0 CONSTRUCTION

7.01 General

7.01.01 Rigid Expanded Polystyrene Installation

The installation of the Rigid Expanded Polystyrene shall be undertaken under the supervision of the Contractor's Engineer.

The Contractor inspection of the Rigid Expanded Polystyrene shall be carried out full-time.

The Contractor's manufacturer representative shall be on site to oversee installation of the Rigid Expanded Polystyrene blocks at the commencement of the installation.

7.02 Delivery, Storage and Handling

The product shall be suitably marked to identify its type, number and the manufacturer's name or trademark.

The Contractor shall protect the Rigid Expanded Polystyrene from exposure to sunlight to avoid ultraviolet degradation as per manufacturer's recommendation.

Protection of materials and works from damage by weather, traffic, construction staging, fire or vandalism and other causes shall be the responsibility of the Contractor.

Rigid Expanded Polystyrene shall not be exposed to open flame or other ignition source. The contractor shall protect the Rigid Expanded Polystyrene blocks from petroleum based products such as gasoline and diesel fuel and organic solvents such as acetone, benzene and paint thinner.

7.03 Foundation Excavation

Foundation excavation shall be carried out to the design elevations shown on the drawings. Any softened, loosened or deleterious materials at the foundation footing elevation shall be subexcavated and replaced with OPSS 1010 Granular A material.

7.04

Leveling Pad

The Contractor shall place, level and compact a layer of OPSS 1010 Granular A material in accordance with OPSS 501 to within ± 30 mm of the design elevation. The leveling pad shall not deviate by more than 10 mm at any place on a 3 m straight edge over the limits of the bottom course of blocks and shall be constructed parallel to the final pavement centre line design profile. The levelling pad shall not be placed on standing water, accumulated snow or ice or frozen ground. The levelling pad must be placed in-the-dry.

7.05

Installation of Blocks

The Contractor shall have on site at the commencement of the work, a representative of the supplier of the Rigid Expanded Polystyrene to advise on recommended construction procedure.

The Contractor shall maintain liaison with the supplier throughout the construction of the embankment for advice and guidance as required. Periodic site visits by the supplier should be coordinated as required.

The Rigid Expanded Polystyrene embankment shall be installed to ensure that:

- 1) The individually marked blocks shall be placed on the prepared leveling pad. The top surface of the first layer of blocks is to be set plane and level. Local trimming of the blocks may be necessary.
- 2) Subsequent successive layers shall be oriented with the long axis of blocks positioned at 90° to the previous layer in order to avoid continuous joints. Block joints shall be offset and staggered between layers.
- 3) A continuous check shall be kept to ensure the evenness of the blocks is satisfactory in each layer. Blocks shall be laid with joints with maximum opening of 10 mm between blocks. Differences in heights between adjacent blocks in the same layer shall not exceed 5 mm.
- 4) Sloping end adjustments at the abutments shall be accomplished by leveling terraces in the subsoil in accordance with the block thickness.
- 5) Due to windy conditions temporary ballast shall be provided as necessary to prevent movement of Rigid Expanded Polystyrene both in storage and during placement. Timber fasteners or equivalent shall be used as necessary.
- 6) Buoyancy/flotation of the EPS blocks shall be prevented by means of dewatering to maintain the groundwater level below the base of the EPS and using temporary ballast as necessary. These means of preventing buoyancy/flotation are required until the overlying granular pavement structure has been constructed.
- 7) The Rigid Expanded Polystyrene embankment shall be protected from accidental ignition due to welding, smoking, grinding or cutting tools, etc. The Contractor shall take all necessary precautions to prevent ignition of the Rigid Expanded Polystyrene.
- 8) The Rigid Expanded Polystyrene shall be protected from organic solvents and other aggressive, harmful chemicals during construction.

- 9) Exposed blocks shall be covered immediately to avoid possible burrowing by animals.
- 10) Individually marked blocks shall be fabricated and placed to ensure the top surface matches the elevation and crossfall shown on the drawings.
- 11) The top surface and side surfaces of the Rigid Expanded Polystyrene shall be covered with 6 mil polyethylene sheeting extending onto adjacent work at the longitudinal ends of the embankment. All joints shall be lapped a minimum of 300 mm to provide a fully sealed enclosure.

7.06 Side Slope Cover

The side slopes of the Rigid Expanded Polystyrene embankment shall be covered with granular fill as detailed elsewhere in the Contract Drawings.

8.0 QUALITY ASSURANCE – NOT USED

9.0 MEASUREMENT FOR PAYMENT

9.01 Actual Measurement

Measurement will be by volume in cubic metres measured in its original position and based on cross-sections.

10.0 BASIS OF PAYMENT

The concrete base pad and granular leveling pad shall be paid for with the appropriate tender items as detailed elsewhere in the contract.

Payment at the contract price for the above tender item shall be full compensation for all labour, materials and equipment to do the work as described above and no extra payments will be made.

Structure Monitoring - Item No.

Special Provision

1.0 SCOPE

This special provision contains the requirements for the supply and installation of the following geotechnical instruments:

- Survey Benchmarks (BM)
- Settlement Pins (SP)
- Vibrating Wire Piezometers (VWP)

The purpose of these instruments is to monitor settlements of the existing bridge and pore water pressures in the foundation soils during pile installation for the proposed Laronde Creek Replacement Bridge on Highway 17.

The rate of pile driving, and hammer energy applied during pile driving shall be controlled by the instrumentation readings.

1.01 General Procedure

The benchmarks shall be installed prior to the bridge construction. The benchmarks consist of a steel rod anchored to the bottom of a borehole. Existing verified non-yielding benchmark monuments can be considered as a replacement for new benchmarks following approval by the Contract Administrator.

The settlement pins shall be installed on the pavement of the travelled portion of the existing bridge and edge of the existing bridge deck. The settlement pins shall be masonry nail heads recessed below pavement surface and installed along the edge of the existing concrete deck.

The VW piezometers shall be installed in boreholes prior to any pile driving. The VW signal cables shall be extended underneath the work zone through a metal or plastic conduit buried in trenches to a data logger located out of the work zone.

2.0 REFERENCES – Not Used

3.0 DEFINITIONS

3.01 Personnel

The Contractor shall retain a Geotechnical Consultant who is approved for MTO RAQS category of “**Geotechnical** (Structures and Embankments) – **Medium Complexity**”, to undertake the supply and installation of geotechnical instruments. Monitoring of the instruments shall be carried out by others.

The *Contractor* shall be understood to refer to the Contractor and his Geotechnical Consultant.

3.02 Or Equal

The term, “*or equal*”, shall be understood to indicate that the equal product is the same or better than the specified product in function, performance, reliability, quality and general configuration. Only one supplier should be selected for the supply of all vibrating wire instruments.

4.0 DESIGN AND SUBMISSION REQUIREMENTS

The Contractor shall submit details of proposed installation methods, including location and types of data-acquisition system, survey benchmarks, and installation schedule to the Contract Administrator, a minimum of 15 days before the start of instrument installation.

5.0 MATERIALS

5.01 Survey Benchmarks (BM)

5.01.01 General

The Contractor shall supply all materials and equipment required for the installation of the benchmarks.

5.01.02 Rod

The Contractor shall supply a steel pipe Schedule 40 with an outside diameter not less than 25.4 mm (1"), supplied in lengths as required to complete the installation.

The top end of each length of rod shall be threaded to receive a cap. A rounded cap shall be installed at the top of the rod in such a way that a single survey point can be clearly identified and returned to.

5.01.03 Sand

The Contractor shall supply clean washed sand with the following gradation:

MTO Sieve Designation	Percentage Passing
4.5 mm - #4	100%
2 mm - #10	80% - 100%
850 µm - #20	20% - 100%
425 µm - #40	5% - 40%
150 µm - #100	0% - 5%

5.01.04 Grout

The Contractor shall supply cement-bentonite grout. A suitable grout mix design consists of 17.7 kg of bentonite (OPSS 1205), 284 litres of water and 42.6 kg of cement (Type 10 - OPSS 1301).

5.01.05 Rod Anchor Grout

The Contractor shall supply cement-bentonite grout. A suitable grout mix design consists of 14 kg of

bentonite (OPSS 1205), 49 litres of water and 40 kg of cement (Type 10 - OPSS 1301).

5.02 Settlement Pins (SP)

5.02.01 General

The Contractor shall supply all materials and equipment required for the installation of the settlement pins.

5.02.02 Nail

The Contractor shall supply a masonry nail with a minimum length of 25 mm. The head of the nail shall be rounded in such a way that a single survey point can be clearly identified and repeated.

5.02.03 Metal Identification Tag

The Contractor shall supply a metal identification tag to be installed with surveying nail. The diameter of tag shall be 35 mm.

5.03 Vibrating Wire Piezometers (VWP)

5.03.01 VW Piezometers

The Contractor shall supply VW borehole piezometers (e.g. Geokon model 4500S rated at -5 to 50 psi, or equal); compatible with a data logger (e.g. Geokon model 8002-1, 8002-4, 8002-16 data logger, or equal). All VW piezometers and data loggers shall be of the same make.

All VWPs shall be calibrated prior to installation and the calibration data for each VWP shall be provided to the Contract Administrator.

5.03.02 Signal Cable

The Contractor shall supply signal cable compatible with the VW equipment. The VWPs shall withstand all the temperature variations. The length of cable for each piezometer shall be carefully estimated from the construction drawings to ensure that there is enough signal cable for each piezometer to provide enough slack in the borehole and along the trenches.

5.03.03 VW Data Recorder

The Contractor shall supply a VW Data Recorder (e.g. Geokon model GK-404, or equal); compatible with the above VW piezometers. All VW equipment shall be of the same make.

5.03.04 VW Data Loggers

The VWP signal cables shall be connected to the nearest data logger. Data loggers such as Geokon LC-2 Series Model 8002-1 (single channel), 8002-4 (four channel) and/or 8002-16 (16 channel) or equal shall be used. The data logger shall include, but not be limited to, interface modules, interface cables, data logger retrieval computer software, and continuous power supply that will allow for regular monitoring over the duration of the bridge construction. All data loggers shall be of the same make and shall be compatible with the VWP instruments.

The VW data shall be retrieved on site by direct wire (e.g. RS232 or USB Cable) with a portable laptop computer specified in Section 6.02.

5.03.05 Bentonite

The Contractor shall supply bentonite (OPSS 1205) in pellet form in sufficient quantity to form borehole plugs as required.

5.03.06 Grout

The annular space between the VWP cables and the borehole shall be filled with cement-bentonite grout prepared as follows: 17.7 kg of bentonite (OPSS 1205), 284 liters of water and 42.6 kg of cement (Type 10 - OPSS 1301).

5.03.07 Filter Sand

The Contractor shall supply clean sand for filter around VW sensors. The sand shall be Sakrete washed general purpose sand or equal.

5.03.08 Trench Burial and Conduit

The signal cable for each piezometer shall be buried in a shallow trench as shown in the contract drawings and taken out of the work zone. The Contractor shall supply suitable conduits (e.g. Schedule 40 - 75mm - 3" rigid PVC pipe) to protect the signal cables in the trenches and above ground surface. If appropriate, several signal cables may be housed in a single conduit and laid in a common trench. Before trenches are backfilled, the VW piezometers shall be tested.

5.03.09 Monitoring Enclosure

The Contractor shall supply all materials required to build weatherproof and lockable monitoring enclosures to house VW data loggers.

6.0 EQUIPMENT

6.01 Equipment Operation and Weather Conditions

All installation and monitoring equipment and associated materials shall be capable of withstanding the range of temperatures possible for their locations within the ground or on the surface. The instruments shall be capable of operating within the manufacturer's stated accuracy throughout the temperature range. The Contractor shall replace/repair non-functioning monitoring instruments as required at no cost to the Ministry.

6.02 Portable Laptop Computer

The Contractor shall supply a laptop computer, Lenovo X140e or equal, equipped with at least a 128 gigabyte (gb) solid state drive, 4gb of RAM, two batteries, Microsoft Windows 7 Professional (OS), Microsoft Office Home and Business 2017, Adobe Acrobat XI Standard and data logger software compatible with the selected data logger system.

The portable laptop computer shall be handed to the Contract Administrator after the installation of

instruments and before the commencement of the Monitoring Program.

The calibration factors for all vibrating wire instruments shall be entered in the portable laptop computer by the Contractor for initialization of the instruments.

7.0 CONSTRUCTION

7.01 Installation

7.01.01 Drawings

Reference shall be made to the following drawings in the Contract Drawings:

- Monitoring Section Location Plan and Profile
- Monitoring Instrument Details

7.01.02 Subsurface Conditions

The subsurface conditions at the sites are described in the following report:

FOUNDATION INVESTIGATION REPORT, LARONDE CREEK BRIDGE REPLACEMENT, HIGHWAY 17, 20.3 KM WEST OF HIGHWAY 11, SITE NO. 43X-0065/B0, G.W.P. NO. 5198-13-00 (GEOCRENS NO. 31L-224), prepared by Thurber Engineering.

7.01.03 Instrument Locations and Quantities

Prior to the installation of instruments, the Contractor shall accurately survey and stake the location of each instrument and obtain a ground elevation and northing/easting coordinates at each instrument location.

The quantities and locations of instruments are shown in Table 1.

Table 1 – Instrument Quantities

Location	NO. OF INSTRUMENTS	
	SP	VWP
West Abutment	2	
7 m west of West Abutment	2	2
17 m west of West Abutment	2	
East Abutment	2	
7 m east of East Abutment	2	2
17 m east of East Abutment	2	
Total	12	4

7.01.04 Survey Benchmarks (BM)

The Contractor shall provide a minimum of two non-yielding temporary survey benchmarks (BM), one on each side of the Laronde Creek River. Suggested locations of BMs are shown on the Contract

Drawings; however, the BM locations can be moved to suit site conditions provided that direct sighting is possible from all settlement pins (SP) to at least one benchmark.

The locations of the temporary benchmarks are to be approved by the Contract Administrator prior to installation of the monitoring instruments.

7.01.04.01 Number and Location

A minimum of two (2) benchmarks shall be provided for the purposes of monitoring bridge settlements. The number and locations of benchmarks shall be adjusted in the field such that the benchmarks are located at sufficient distances from the bridge to remain non-yielding, not affected by the bridge construction or pile driving and direct sighting is possible from all settlement pins (SP) to at least one benchmark.

7.01.04.02 Borehole Installation

The borehole shall be advanced to refusal on the bedrock surface using suitable drilling techniques. The diameter of the borehole shall be sufficient to fit the rod and rod anchor. The borehole shall be stable and free of drilling mud and debris.

7.01.04.03 Rod Couplings

The coupling of the rods shall be such that all sections have the same axis and no separation or contraction will occur at the couplings.

7.01.04.04 Rod Anchor

The rod shall be installed vertically in the borehole with its bottom end resting at the bottom of the borehole. The rod shall be grouted in place.

7.01.04.05 Installation Details

The elevation, easting and northing of the top of the benchmark rod shall be surveyed.

7.01.05 Settlement Pins (SP)

The settlement pin shall be installed on paved surface. The settlement pin installations in areas of vehicular traffic that could dislodge monitoring points (e.g. by snowplow) must be recessed below surrounding surface by a minimum of 5 mm.

7.01.05.01 Quantities

The locations of the settlement pins are shown on the contract drawings and are given in Table 1A.

Table 1A – SP Locations and Quantities

Location	Approx. Offset from Centreline	No. of SP
West Abutment	3.5 m north	1
	3.5 m south	1
7 m west of West Abutment	3.5 m north	1
	3.5 m south	1
17 m west of West Abutment	3.5 m north	1
	3.5 m south	1
East Abutment	3.5 m north	1
	3.5 m south	1
7 m east of East Abutment	3.5 m north	1
	3.5 m south	1
17 m east of East Abutment	3.5 m north	1
	3.5 m south	1
Total		12

7.01.05.02 Pre-drilled Hole

Prior to installing the masonry nail, a small size hole shall be drilled into the pavement or concrete. The inside of the hole must be clear of any drill powder or loose overbreak materials prior to installation.

7.01.05.03 Masonry Nail

The nail shall be hammered into the pre-drilled hole while maintaining contact with the inside wall. When fully inserted, the base of the head of the nail shall be flush with the pavement or concrete surface.

7.01.05.03 Installation Details

The elevation, easting and northing of the top of the nail shall be surveyed.

The Contractor shall install settlement pins as per the contract drawings provided.

7.01.06 VW Piezometers

7.01.06.01 Quantities

The Contractor shall install VW sensors at the locations and depths given in Table 1B.

Table 1B – VWP Locations and Quantities

Location	Approx. Offset from Centreline	No. of VWP	VWP TIP ELEVATION (m)
7 m west of West Abutment	4.0 m North	2	193.5
			187.5
7 m east of East Abutment	4.0 m North	2	193.5
			187.5

7.01.06.02**Borehole Installation**

The borehole shall be advanced to 500 mm below the lowest VWP tip elevation using suitable drilling techniques. The sides of the borehole shall be stable, and the borehole shall be free of drilling mud and debris.

The Contractor shall make a basic stratigraphic log of boreholes as they are being drilled. Occasional Standard Penetration Test (SPT) sampling are required to confirm soil type. Other in-situ testing and laboratory testing are not required.

7.01.06.03**Data Logger Boxes and Monitoring Enclosure**

The data-logger and all associated accessories shall be installed in a lockable enclosure to prevent vandalism and wear-out of the data-loggers against extreme weather.

The monitoring enclosure shall be lockable and weatherproof. The monitoring enclosure shall be attached on a wooden post (a minimum 100 mm x 100 mm) and secured in the ground. The monitoring enclosure shall be located approximately 1.5 m above the surrounding ground for easy access to data loggers. All data loggers and associated accessories shall be properly grounded and protected from lightning strike. The Contractor shall submit a detailed proposal of the monitoring enclosure (i.e. materials, location(s), etc.) to the Contract Administrator for review and approval, prior to construction. Suggested locations of enclosures are shown on the Contract Drawings; however, the enclosure locations can be moved to suit site conditions provided that they are easily accessible, located outside the work zone, and are readily cleared of snow during the winter.

The data loggers shall be securely attached inside the Monitoring Enclosure. The data logger accessories shall be properly grounded.

The Contractor shall ensure safe access to the Monitoring Enclosure at all times including, but not limited to, snow clearing in the winter.

7.01.06.04**Completion of Installation**

It is known that the process of installing VW piezometers can temporarily alter the pore water pressure acting on the piezometer tip. The installation of a VW piezometer shall not be considered to be complete until the pore pressure acting on the piezometer has returned to and stabilized at the value prevailing in the surrounding, unaffected soil mass. The Contractor shall take daily reading of the pore pressures until the value has stabilized. Stabilization shall be deemed to have occurred:

- a) When no change in the measured value has occurred over a period of 5 days and the measured value is within 10% of the anticipated hydrostatic value;
- b) When the daily rate of change is less than (3) kPa per day for three consecutive days and the measured value is within 5% of the anticipated hydrostatic value;
- c) Failing either of the two above conditions, as determined by the Contract Administrator.

The Contractor should be prepared to wait for a period of 15 days after completion of installation of piezometers for the baseline readings to stabilize.

7.01.07 Accuracy of Surveying for Elevations

Elevations shall be surveyed to an accuracy of ± 2 mm or better.

7.01.08 Underground Utilities

The Contractor shall be responsible for locating and protecting all underground utilities prior to drilling boreholes for installing instruments. Any damage to underground utilities caused by the Contractor's work shall be repaired by the Contractor, at no cost to the Ministry.

7.01.09 Marking and Labelling

The location of any above ground monitoring fixture shall be made clearly visible to nearby traffic before, during and after bridge construction. Marking shall be of sufficient size to be visible from a reversing vehicle and after a heavy snow fall.

Instruments or their data cables shall be clearly labelled in the field, each instrument having a unique identifier. The labelling shall remain legible during the monitoring period.

7.01.10 Protection of Instruments

All instruments shall be adequately protected by the Contractor such that they are not damaged during construction. Any instrument damaged by the Contractor's work shall be immediately replaced at the Contractor's cost.

Instruments should also be adequately protected by the Contractor from construction activities and natural effects. This can be achieved by using appropriate grounding and transient protection systems; such as Geokon Lab 3 Surge Module, lightning diversion systems and/or equipotential grounding systems, or equal.

7.01.11 Installation Program

Instrument installation shall commence prior to pile driving and construction of any work. No material stockpiling shall be allowed within the bridge construction area during instrument installation. Table 2 gives a summary of the installation schedule requirements.

Table 2 – Installation Program

TYPE	START INSTALLATION	FINISH INSTALLATION
BM	-	Before installation of other monitoring instruments
SP	After completion of benchmark installation	At least 15 days prior to any pile driving for new bridge
VWP		

7.02 Coordination with Monitoring

7.02.01 Survey Benchmarks (BM)

7.02.01.01 Notification

The Contractor shall notify the Contract Administrator no later than 3 days after installing a benchmark. At this time the Contractor shall also supply the following information to the Contract Administrator:

- Elevations of the bottom of the rod anchor and the top of rod;
- Dates of installation;
- Stratigraphic log of subsurface conditions at the benchmark, including drilling method notes;
- Installation notes, sketches and photographs;
- Description of benchmarks.

7.02.01.02 Monitoring

Monitoring of settlement with reference to the benchmarks shall be done by others. Monitoring shall be conducted before, during and after pile driving for the new bridge. The Contractor shall provide installation information as specified above and provide access to the benchmarks for monitoring including, but not limited to, snow clearing in the winter. The contractor shall provide electric power and general area lighting as needed.

7.02.02 Settlement Pins (SP)

7.02.02.01 Notification

The Contractor shall notify the Contract Administrator no later than 3 days after installing a settlement pin. At this time the Contractor shall also supply the following information to the Contract Administrator:

- Settlement pin location, easting and northing;
- Elevation of top of pin;
- Dates of installation and datum readings;
- Installation notes, sketches and photographs;

7.02.02.02 Monitoring

Monitoring of the settlement pins shall be done by others. Monitoring shall be conducted before, during and after the pile driving. The Contractor shall provide installation information as specified above and provide access to the settlement pins for settlement monitoring.

7.02.03 VW Piezometers (VWP)

7.02.03.01 Notification

The Contractor shall notify the Contract Administrator no later than 3 days after installing a VW piezometer. At this time, the Contractor shall also supply the following information to the Contract Administrator.

- VW piezometer location, easting, northing;
- Elevations of VWP sensor tips;
- Stratigraphic log of subsurface conditions, including drilling method notes;
- Dates of installation;
- Installation notes, grounding method, sketches and photographs;
- Model, make and serial number of VW sensors, data logger and signal cable;
- Calibration details of VW sensors.

7.02.03.02**Monitoring**

Monitoring of the VW piezometers, including establishment of baseline data, shall be done by others. Monitoring shall be conducted before, during and after the pile driving. The Contractor shall provide installation information as specified above and provide access to the monitoring shed for data retrieval.

The Contractor shall transfer the Portable Laptop Computer and VW Data Recorder to the Contract Administrator, including all data logging software and hardware, operating instructions and calibration constants. The contractor shall also transfer the keys for the locks of the Enclosure(s). The contractor shall be available for one site meeting with the Contract Administrator to transfer the items and answer any questions the Contract Administrator may have regarding the data-logging system.

7.03**Decommissioning of Instruments**

The Contractor shall decommission the VWPs at the end of the monitoring program following the end of construction unless advised otherwise by the Contract Administrator. The Benchmarks (BM) shall not be decommissioned unless advised otherwise by the Contract Administrator. Decommissioning of instrumentation shall be carried out as per the Ontario Water Resources Act, R.R.O. 1990, Regulation 903.

8.0**QUALITY ASSURANCE – Not Used****9.0****MEASUREMENT FOR PAYMENT**

Measurement for Payment for the Supply and Installation of Bridge Monitoring Equipment shall be Lump Sum.

10.0**BASIS OF PAYMENT**

Payment at the contract price for the above tender item shall be full compensation for all labour, equipment, and materials required to do the work herein described.

DEWATERING STRUCTURE EXCAVATIONS - Item No.

Special Provision

Amendment to OPSS 902, November 2010

902.02 REFERENCES

Section 902.02 of OPSS 902 is amended by the addition of the following:

Ontario Provincial Standard Specifications, Construction

OPSS 805 Temporary Erosion and Sediment Control Measures

902.03 DEFINITIONS

Section 903.03 of OPSS 902 is amended by the addition of the following:

Automatic Transfer Switch means an electrical device that transfers power supply to a backup power source when there is an outage of the primary power source.

Cofferdam means as defined in OPSS 539.

Cut-Off Wall means a below grade wall that restricts groundwater flow and/or supports excavations, typically using soil-bentonite or cement-bentonite.

Design Storm Return Period means the average number of years based upon probability, between the occurrences of a storm event of a certain severity or greater.

Dewatering System means the components required to control water to permit construction work to proceed under specified conditions, and may include a groundwater control system, impermeable barriers, pumps, and/or equipment to carry out unwatering.

Groundwater Control System means sump pumps, oversized excavations with perimeter ditches, deep wells or well points or other systems used to lower the groundwater table.

Plug means an impervious, natural, or constructed drainage work that blocks water.

Sediment means soil particles detached from an earth surface by erosion.

Sediment Control Measure means a measure to remove sediment from water prior to discharge to the natural environment and sewer systems.

Temporary Flow Control means temporary flow control devices, channels, pipes, and other materials used to convey or divert water past an area under construction.

Unwatering means the removal of ponded or flowing surface water.

Vegetated Discharge Area means a sloped, open area of land with existing vegetation suitable to prevent erosion.

Waterbody means as any permanent or intermittent, natural or constructed body of water including lakes, ponds, wetlands and watercourses, but does not include sewage works as defined in the Ontario Water Resources Act.

Watercourse means a stream, creek, river, or channel including ditches, in which the flow of water is permanent, intermittent, or temporary.

902.04 DESIGN AND SUBMISSION REQUIREMENTS

Subsections 902.04.01 and 902.04.02 of OPSS 902 are deleted in their entirety and replaced with the following:

902.04.01 Design Requirements

902.04.01.01 Dewatering

A dewatering system shall be designed to control water and the flow of water into the excavation, prevent disturbance of the foundation, permit the placing of concrete in the dry, and complete the excavating and backfilling for structures work. The design of the system shall be sufficient to permit the work to be carried out as specified in the Contract Documents.

The design shall meet the requirements of the Contract Documents, and where a waterbody is present, shall include channel and inlet and outlet protection measures as required to protect the environment in the event of system failure or the design flow rate being exceeded.

The design shall not include the use of embankments and/or structures in public use, either existing or to be constructed as part of the Work, to control or stop water flow, unless approved by the Contract Administrator.

The design shall not result in displacement or damage to property, buildings, structures, utilities and other facilities adjacent to the Working Area, including from drawdown related settlement or other groundwater related effects.

The system shall be designed to prevent soil loss or erosion where water is removed, pumped, or discharged. The system shall be designed to prevent basal heave or instability.

Where the system involves the taking of water from a waterbody, the design shall maintain the flow of water and the natural functions of the waterbody upstream and downstream of the work area, and shall not interfere with other uses of the water.

When the system includes temporary flow control, the temporary flow control shall be designed, as a minimum, for a [* Designer Fill-In, See Notes to Designer] year design storm return period, and groundwater discharge. A longer return period shall be used when determined appropriate for the work.

Temporary flow control shall include provision for fish passage during low flows.

902.04.02

Submission Requirements

902.04.02.01

Working Drawings

Three (3) sets of Working Drawings for the dewatering system shall be submitted to the Contract Administrator at least 7 Days prior to commencement of the dewatering system installation, for information purposes only. Prior to submission of Working Drawings, the seals and signatures of a design Engineer and a design-checking Engineer shall be affixed on the Working Drawings verifying that the drawings are consistent with the Contract Documents.

One person shall not perform both the design Engineer and design-checking Engineer roles for a system.

Where multi-discipline engineering work is depicted on the same Working Drawing and the design or design-checking Engineer or both are unable to seal and sign the Working Drawing for all aspects of the work, the drawing shall be sealed and signed by as many additional design and design-checking Engineers as necessary.

The following information and details shall be shown on the Working Drawings, where applicable:

a) Plans, Elevations, and Details

- i. Type of system(s).
- ii. Design calculations demonstrating adequacy of the system and equipment.
- iii. Design flow rate(s).
- iv. Plan location, description, and dimensions of system components, including dams, cofferdams, cut-off walls, temporary channels, pipes, culverts, sewers, groundwater control systems employing wells and/or well points, sedimentation basins, tanks, pumps, power supply, and standby equipment.
- v. Method of management of pumped water and plan location of all dewatering discharge points.
- vi. Profile drawings shall extend through and immediately beyond the limits of the system.
- vii. Water elevations upstream and downstream of the system at design flow rate.
- viii. Dam height or crest elevation, cofferdam depth and tip elevation, cutoff wall depth or base elevation, pipe invert elevations, depths of wells and wellpoints, pump intake elevation, and sedimentation basin depth or base elevation.
- ix. Plan location, elevation, and dimensions of environmental protection measures.
- x. Pipe type, size, and length, pump capacity, and tank capacity.
- xi. Material and construction standards to be used for the work.
- xii. Method for establishing and monitoring construction site groundwater levels.
- xiii. Criteria and method of removal of the system.

b) Procedures for the system construction, operation, and maintenance, including daily start-up sequence where applicable, and operation shut down.

c) Procedures for the removal of the system, including the removal sequence, and well decommissioning.

d) Stand-by power or pumping system requirements and the use of automatic transfer switching, when required to protect the environment and the Work.

e) A copy of the Permit to Take Water issued by the Ministry of the Environment and Climate Change or confirmation of registration of water taking for construction dewatering, if a permit or registration is required by provincial regulation.

f) When applicable, a copy of the water taking report and discharge plan required by provincial regulation.

- g) A copy of any necessary permits for the discharge of water to a sanitary sewer, or stormwater sewer system, stormwater pond, or other facility.

902.04.02.02 Preconstruction Survey

When a groundwater control system by wells or a well point system will be used, a condition survey of property and structures that may be affected by the work shall be carried out. The condition survey shall include the location and condition of adjacent properties, buildings, underground structures, water wells, Utilities, and structures, within a distance of [** Designer Fill-In, See Notes to Designer] metres from the groundwater control system. In addition, all water wells used as a supply of drinking water and located within this distance shall be tested for compliance with Ontario Drinking Water Quality Standards.

Water wells within the preconstruction survey distance can be located using the website <https://www.ontario.ca/environment-and-energy/map-well-records> or its successor site.

Copies of the condition survey and water quality test results shall be submitted to the Contract Administrator prior to the operation of the groundwater control system.

902.04.02.03 Milestone Inspections

The Quality Verification Engineer shall witness the following Interim Inspections of the work:

- a) Dewatering of excavation for structure.
- b) Completion of excavation for foundation.
- c) Excavation for backfill and frost tapers.
- d) Backfilling.

A copy of the written permission to proceed shall be submitted to the Contract Administrator prior to commencement of the successive operation.

902.07 CONSTRUCTION

Subsection 902.07.04 of OPSS 902 is deleted in its entirety and replaced with the following:

902.07.04 Dewatering Structure Excavation

902.07.04.01 General

The dewatering systems shall be constructed and operated according to the Working Drawings.

Activation of temporary flow control, if applicable, shall be as specified in the Contract Documents.

The dewatering system shall be continuously operational to control buoyancy forces until such forces can be resisted by backfill and structure self-weight, to keep excavations stable, to avoid erosion impacts from the release of accumulated water, and to keep the work area in the condition required to complete the associated work as specified in the Contract Documents.

When temporary flow control is to remain operational through a seasonal shutdown period, the Contractor shall be responsible for any maintenance or repair costs due to the temporary flow control during the seasonal shutdown period.

Temporary erosion and sediment control measures, including to control the discharge of water, shall be according to OPSS 805. Measures not specified in OPSS 805 shall be according to the Working Drawings. Temporary erosion and sediment control measures and cover material to protect exposed soils, as required by the Working Drawings, shall be installed as soon as is practical.

Stranded fish shall be managed as specified in the Contract Documents.

Unwatering shall be carried out as necessary.

Water suspected of being contaminated as indicated by visual or olfactory observations shall be reported to the Contract Administrator.

Dewatering and temporary flow control shall be discontinued in a manner that does not disturb any structure, pipeline, or flow channel. Operation of the dewatering system shall be shut down according to the procedures specified in the Working Drawings, where applicable.

902.07.04.02 Discharge of Water

Water from dewatering and unwatering operations shall be directed to a sediment control measure and/or a vegetated discharge area 30 m away from waterbodies or as far away as practicable from the top of the bank of any waterbody, prior to discharge to the natural environment.

Equipment and materials shall not be used or stored in vegetated discharge areas.

The discharge of water to the natural environment shall not be directed across pavements, sidewalks, curb and gutter or similar hard surfaces except through appurtenances as specified in the Contract Documents.

902.07.04.03 Monitoring

The Contract Administrator shall be notified of any complaints and any action taken or proposed to be taken in response to complaints.

Daily external visual monitoring of the surrounding area and property and structures on the preconstruction survey, if applicable, for impacts such as settlement and erosion shall be completed. Any observed impacts shall be immediately reported to the Contract Administrator. When public safety, the environment, or property is impacted or potentially impacted, the design Engineer shall, without delay, make a full assessment and direct changes to the system to eliminate impacts or potential impacts. Any changes shall be documented according to the System Amendments subsection.

When a groundwater control system is observed to negatively impact water supplies obtained from any adequate sources that were in use prior to groundwater control system operation, then water shall be supplied to the affected water users. The water shall be equivalent in quantity and quality to the normal water takings of the users. Supply shall continue until the negative impacts on the water supplies are removed, or until Contract Completion, whichever occurs first.

902.07.04.04 System Amendments

When displacement or damage to embankments and/or structures, or property adjacent to the Working Area, occurs due to the operation of the system, or soil loss or erosion occurs where water is removed, pumped, or discharged, the dewatering system or temporary flow control shall be amended to stop the displacement, damage, soil loss, or erosion.

Amendments shall be submitted to the Contract Administrator within two Business Days of the system being amended, on revised Working Drawings bearing the seal and signature of the design Engineer and design-checking Engineer.

902.07.04.05 Removal

Dewatering system and temporary flow control components shall be removed when no longer required. Removal of system components shall be according to the procedures specified on the Working Drawings, where applicable, and as specified in the Contract Documents.

Deactivation of temporary flow control shall be as specified in the Contract Documents.

Removal of temporary drainage work shall be according to OPSS 510.

Environmental protection measures and cut-off walls shall be removed, unless approved otherwise by the Contract Administrator.

Sedimentation basins and other excavations shall be backfilled with the original soil excavated, unless approved otherwise by the Contract Administrator. All disturbed areas shall be restored to an equivalent or better condition than existed prior to the commencement of construction.

NOTES TO DESIGNER:

Designer Fill-Ins

* Fill in the design storm return period according to MTO Drainage Design Standard TW-1.

** Fill in the preconstruction survey distance as recommended by the foundation engineer.

WARRANT: Include with this item **only** on the recommendation of a foundation engineer.

CUSTODIAN: Tony Sangiuliano, MERO - Foundation Group.