THURBER ENGINEERING LTD.

FINAL

FOUNDATION INVESTIGATION AND DESIGN REPORT
HIGHWAY 17 TWINNING, RENFREW AREA
DOCHART CREEK CROSSINGS 21+340 EB AND 21+360 WB
WP 4068-09-00 / ASSIGNMENT NO. 4018-E-0009

Geocres No.: 31F-219

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104, 2460 Lancaster Road, Ottawa ON K1B 4S5 T. 613 247 2121 F. 613 247 2185
thurber.ca

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PART 1. FACTUAL INFORMATION

1 INTRODUCTION

Thurber Engineering Ltd. (Thurber) has been engaged by the Ministry of Transportation Ontario (MTO) under Assignment No. 4018-E-0009 to carry out Foundation Investigations to support the design of the Highway 17 Twinning Project which extends from Scheel Drive westerly to 3 km west of Bruce Street in the Renfrew area.

This report addresses the replacement of the three pipe culverts under the existing (proposed eastbound) embankment and the installation of a new structure for the new westbound embankment at Dochart Creek in the Township of McNab/Braeside within Renfrew County, Ontario.

This section of the report presents the factual findings obtained from historical foundation investigations available from the online Geocres Library and from the foundation investigation completed as part of the current study.

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

Previous foundation investigation information from boreholes completed in 2005 for the proposed westbound structure was available under Geocres 31F-153 while information from boreholes completed in 2018 for the rehabilitation of eastbound culverts was available under Geocres 31F-204.

2 BACKGROUND

2.1 Site Description

The site is located on Highway 17 approximately 600 m west of the Highway 17 Scheel Drive Overpasses. The existing Highway 17 in this area consists of a transition zone from the two-lane undivided Highway 17 to the west and the previously twinned Highway 417 to the east. At the site, there is a single lane in each direction with paved shoulders separated by a central gore area. The existing road surface (proposed eastbound) is at approximate elevation 120.1 m. Based on profiles provided by MTO the proposed westbound lanes are to be at a similar elevation.

The land adjacent to the site typically consists of forests and agricultural fields. The terrain is relatively flat with shallow and exposed bedrock approximately 500 m east and west of the site.

Creek flow beneath existing Highway 17 is from the south to the north through three 43.5 m long, 2.75 m diameter structural plate, corrugated steel pipe (CSP) culverts installed on a skew to the highway. The center-to-center distance between the three culverts is approximately 4.4 m. The existing culvert inverts are at approximate elevation 115.2 m and 115.3 m at the south and north ends, respectively. During a site visit on April 27, 2020, the main Dochart Creek channel at the new westbound lanes was 2 m to 3 m wide and about 0.7 m deep. Near the north end of the existing culverts (outlet), the creek was 12 m wide and about 0.3 m deep. See Photo 4 in Appendix D to see the orientation of the creek and highway. The cover above the existing culverts is approximately 2.0 m.

The existing embankment side slopes did not show any visible signs of distress at the time of the investigation and were sloped at approximately 2H:1V. Along the westbound alignment approximately 40 m east of the creek, a rockfill stockpile approximately 6 m high was placed as part of the previous twinning project (See Appendix A and Photo 4 in Appendix D).

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

2.2 Site Geology

Based on published geological information in *The Physiography of Southern Ontario* by Chapman and Putnam (1984), the culvert site lies within the physiographic region known as the Ottawa Valley Clay Plains. The Ottawa Valley Clay Plains are characterized primarily by clay plains deposited by the Champlain Sea (Leda Clay) interrupted by ridges of rock or sand.

A Physical Setting Report for the overall project prepared by ERIS and based on Ontario Geological Mapping Indicates that the underlying bedrock at the Dochart Creek site is typically carbonate meta-sedimentary rocks of the Grenville Province.

3 SITE INVESTIGATION AND FIELD TESTING

The current site investigation and field testing program was carried out between October 18th and 21st, 2019. The field investigation consisted of advancing four boreholes identified as DOC19-1 through DOC19-4. Prior to commencement of drilling, utility clearances were obtained in the vicinity of the borehole locations.

Historical Boreholes 18-1 and 18-2 (Geocres 31F-204) were drilled by Thurber in June 2018 for the rehabilitation of the existing culverts. It is planned to replace the inlet portion of the three culverts. To date this work has not yet been completed. Historical Boreholes 29-416/C1S, 29-

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416/C1M and 29-416/C1N (Geocres 31F-153) were drilled east of the creek in May 2005 as part of the preliminary investigation for the proposed westbound structure. The five historical boreholes have been fully incorporated into this report.

The northing, easting and elevation of the 2019 boreholes were surveyed by Thurber staff using a Trimble Catalyst DA1 antenna with centimeter accuracy and are shown on the Borehole Location and Soil Strata Drawing No. 1 in Appendix A, the individual Record of Borehole sheets in Appendix B, and in Table 3-1 below. The site is located within MTM Zone 9.

Table 3-1: Borehole Summary (Geocres 31F-153, 31F-204 and Current Investigation)

Borehole No.	Drilled Location	Northing (m)	Easting (m)	Ground Surface Elevation (m)	Termination Depth Below Ground Surface (m)	Comments
18-1	Highway 17 Eastbound Shoulder	5033329.1	307562.1	120.2	15.8	-
18-2	Inlet of Eastbound Culverts	5033313.8	307569.7	116.0	11.4	Piezometer
29-416/C1M	Mid-point of Westbound Embankment	5033368.0	307640.0	116.1	5.8	Piezometer
29-416/C1N	Downstream of Westbound Embankment	5033368.8	307663.9	116.1	4.3	-
29-416/C1S	Upstream of Westbound Embankment	5033353.8	307622.7	116.5	5.5	-
DOC19-1	Outlet of Eastbound Culvert	5033341.8	307603.9	115.7	8.5	Well
DOC19-2	Upstream of Westbound Embankment	5033362.9	307604.7	116.1	11.2	-
DOC19-3	Mid-point of Westbound Embankment	5033369.8	307638.7	116.2	9.5	-
DOC19-4	Downstream of Westbound Embankment	5033390.1	307640.6	115.5	7.8	Well

For the 2018 investigation, the drilling for Borehole 18-2 was carried out using a portable drill rig using wash boring while Borehole 18-1 was drilled using a truck mounted CME 55 drill rig equipped with hollow stem augers and rotary diamond drilling equipment. The boreholes in the 2005 investigation were advanced using a track-mounted CME drill rig equipped with hollow stem augers. The 2019 investigation utilized a track mounted CME 45 drill rig equipped with hollow stem augers and rotary diamond drilling equipment.

Soil samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). Relatively undisturbed, thin-walled tube samples were acquired by pushing hydraulically in Boreholes DOC19-2 and 29-416/CIM. Upon achieving auger refusal or SPT refusal, Boreholes 18-1 and DOC19-1 through DOC 19-4 were advanced into bedrock while collecting NQ core.

38 mm monitoring wells were installed in Boreholes DOC19-1 and DOC19-4. During the previous investigations 25 mm diameter piezometers were installed in Borehole 18-2 and 29-416/C1M. The installation details are illustrated on the Record of Borehole sheets provided in Appendix B. The boreholes were backfilled in accordance with MOE requirements (O.Reg 903, as amended). The monitoring wells installed in 2019 are to be utilized during an upcoming hydrogeological study. They will be subsequently decommissioned by Thurber upon completion of an associated Hydrogeological Study

The drilling and sampling operations were supervised on a full-time basis by a 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.

4 LABORATORY TESTING

Laboratory testing was selected in accordance with the current MTO Guideline for Foundation Engineering Services, Section 5. Geotechnical laboratory testing consisted of natural moisture content determination and visual identification of all retained soil samples. At least 25% of soil samples were subjected to grain size distribution analysis and Atterberg limits tests where appropriate. The testing was carried out to MTO and ASTM standards. One-dimensional consolidation testing was carried out on four thin-walled tube samples from DOC19-2 with incremental loading in accordance with ASTM D2435.

All rock cores were photographed and their total core recovery (TCR), solid core recovery (SCR) and rock quality designation (RQD) were measured. Chemical analysis for determination of pH, conductivity, resistivity, sulphide, sulphate and chloride concentrations were carried out on one soil sample from DOC19-1 and DOC19-4.

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.

5 GENERAL DESCRIPTION OF SUBSURFACE CONDITIONS

Details of the encountered soil stratigraphy are presented on the Record of Borehole sheets included in Appendix B and the Borehole Location and Soil Strata Drawing included in Appendix A. A general description of the stratigraphy, based on the conditions encountered in the boreholes, is given in the following sections. However, the factual data presented on the Borehole Records takes precedence over the Soil Strata Drawing and the general description. It must be recognized that the soil and groundwater conditions may vary between and beyond borehole locations. Soil classification is in accordance with ASTM D2487.

Generally, the site was underlain by embankment fill overlying a native deposit of sensitive clayey silt to clay, over a glacial till deposit. The overburden soils are underlain by limestone bedrock.

5.1 Asphalt

A 110 mm thick layer of asphalt was observed at ground surface in Borehole 18-1.

5.2 Fill

5.2.1 Sandy Gravel Some Silt

A fill layer consisting predominantly of sandy gravel some silt was encountered beneath the asphalt surface in Borehole 18-1. Occasional cobbles were observed in this layer. The top of this layer was encountered at Elevation 120.1 m and the layer had a thickness of 1.4 m. The SPT N-values were 34 and 44; indicating a dense condition.

The moisture content of the samples tested was 4%. Grain size analysis test results for a sample of this fill material are summarized below and are illustrated on Figure C1 in Appendix C.

Table 5-1: Summary of Grain Size Distribution Testing

Soil Particle	Percentage (%)
Gravel	47
Sand	45
Silt	8
Clay	

5.2.2 Rock Fill with Sand Infill

Rock fill with sand infill was encountered within the core of the existing Highway 17 embankment beneath the sandy gravel fill in Borehole 18-1. The top of this layer was encountered at Elevation 118.7 m and the layer had a thickness of 2.0 m. The SPT N-values ranged from weight of hammer (WH) to 17; indicating a very loose to compact condition but typically, compact. Augering and tri-cone drilling techniques were used to advance the borehole through this layer.

Sample recovery was poor as boulders and cobbles were noted throughout this layer and the sand infill was likely washed away during drilling.

The moisture content of the samples tested were 2% and 18%.

5.2.3 Silty Clay with Sand

A fill layer consisting predominantly of silty clay with sand was encountered beneath the topsoil layer in Borehole 18-2. Pieces of wood and rootlets were observed in this unit. The top of this layer was encountered at Elevation 115.8 m and the layer had a thickness of 0.4 m. The SPT N-value was 12; indicating a stiff consistency. The moisture content of the sample tested was 51%.

5.3 Topsoil

Topsoil with a thickness between 100 mm and 300 mm was noted at surface in all the off-road boreholes. The moisture content of two samples tested were 33 and 44%.

5.4 Clayey Silt to Clay (CL to CH)

A native deposit of clayey silt to clay was encountered in all boreholes. The top of the deposit was encountered at elevations ranging from 115.4 m to 116.7 m. The upper 0.5 m to 2.4 m of the deposit in Boreholes 18-1, 18-2, DOC19-1, DOC19-3 and DOC19-4 was noted to be a weathered crust. The total thickness of the deposit ranged from 2.9 m to 7.8 m with an underside elevation ranging from 107.6 m to 112.5 m.

SPT tests conducted in the layer gave N-values ranging from Weight of Hammer to 11. In situ shear vane test results indicated undrained shear strengths greater than 100 kPa in the crust and 25 kPa to 85 kPa below the crust indicating very stiff consistency in the crust and a firm to stiff consistency below. The measured sensitivity ranged from 3 to 21 indicating a medium sensitive to quick clay deposit (CFEM, 2006).

The moisture content of the samples tested ranged from 27 to 64%. The results of grain size analysis tests conducted on thirteen samples of this material are summarized below and are illustrated on Figures C2, C3 and C4 in Appendix C.

Table 5-2: Summary of Grain Size Distribution Testing

Soil Particle	Percentage (%)
Gravel	0 – 2
Sand	0 –16
Silt	36 – 50
Clay	38 – 62

The results of Atterberg Limits testing carried out on fourteen samples of this material are summarized below and are illustrated on Figures C6, C7 and C8 in Appendix C. They indicate the material ranges from a clayey silt (CL) to clay of high plasticity (CH) but generally of intermediate plasticity (CI).

Table 5-3: Summary of Atterberg Limit Testing

Parameter	Value
Liquid Limit	29 – 59
Plastic Limit	16 – 25
Plasticity Index	13 – 37

It should be noted in accordance with the MTO Guideline for Foundation Engineering Services (May 2019) that where Atterberg limits tests indicate a CL material, the deposit should be described as “clayey silt”. The historical boreholes were completed prior to this version of the guideline and refer to CL material as “clay”.

The results of laboratory oedometer (one-dimensional consolidation) tests carried out on four relatively undisturbed clay samples obtained with thin-walled tube samples are presented in Appendix C and summarized below.

Table 5-4: Consolidation Test Results

Parameter	Results			
Borehole	DOC19-2	DOC19-2	DOC19-2	DOC19-2
Sample	ST1	ST2	ST3	ST4
Sample Depth, (m)	1.2	1.8	2.4	3.0
Sample Elevation, (m)	114.9	114.3	113.7	113.1
Approx. Existing Effective Stress, P_0, (kPa)	8	12	17	20
Moisture Content, (%)	54	52	49	55
Liquid Limit, (%)	51	42	39	37
Plastic Limit, (%)	21	20	22	20
Liquidity Index	1.1	1.45	1.59	2.06
Unit Weight, γ (kN/m^3)	16.5	16.5	17.0	16.6
Specific Gravity, G_s	2.790	2.790	2.790	2.790
Initial Void Ratio e_o	1.562	1.527	1.400	1.561
Pre-consolidation Pressure, P_c’, (kPa)	200	140	230	200
Over Consolidation Ratio, OCR	25.0	11.7	13.5	10.0
Compression Index, C_c	1.01	0.85	0.80	1.00
Recompression Index, C_r	0.043	0.032	0.060	0.068
Coefficient of consolidation, c_v (mm^2/s)	0.09	0.16	0.26	0.14
Coefficient of re-consolidation, c_{vr} (mm^2/s)	1.0	2.8	1.8	2.0

5.5 Glacial Till

A glacial till deposit ranging from sandy silt some clay and gravel to silty gravel with sand was encountered beneath the clayey silt to clay in all boreholes except 26-416/C1N. The top of this layer ranges from elevation 107.6 m to 112.5 m. The thickness of the layer ranges from 0.6 m to 3.0 m.

SPT tests conducted in this layer gave N-values ranging from 6 to 33, indicating a loose to dense relative density. N-values greater than 100 blows were encountered in a number of boreholes at the till/bedrock contact.

The moisture content of the samples tested ranged from 3 to 33%. The results of grain size analyses on five samples of the till are summarized below and are illustrated on Figure C5 in Appendix C. The results of Atterberg Limits testing completed on the fines of three of the samples indicated that the fines were non-plastic (ML).

Table 5-5: Summary of Grain Size Distribution Testing

<table>
<tr><th>Soil Particle</th><th colspan="2">Percentage (%)</th></tr>
<tr><td>Gravel</td><td colspan="2">17 – 49</td></tr>
<tr><td>Sand</td><td colspan="2">37 – 50</td></tr>
<tr><td>Silt</td><td>28 - 30</td><td rowspan="2">14 - 29</td></tr>
<tr><td>Clay</td><td>4 - 6</td></tr>
</table>

5.6 Refusal and Bedrock

Split spoon refusal on inferred bedrock was encountered in the Borehole 18-2 at elevation 104.6 m. Auger refusal on inferred bedrock was encountered in Boreholes 29-416/C1S, 29-416/C1M and 29-416/C1N at elevations ranging from 110.3 m to 111.8 m. It is noted that refusal could also be due to cobbles and boulders in the till. Bedrock was proven by coring in boreholes 18-1, DOC19-1, DOC19-2, DOC19-3 and DOC19-4. The bedrock surface was encountered at varying elevations across the site indicating sloping bedrock conditions and a summary of the bedrock surface information is provided below:

Table 5-6: Summary of Bedrock Depth/Elevation

Borehole No.	Depth to Bedrock Surface (m)	Bedrock Surface Elevation (m)
18-1	12.2	108.0
18-2	11.4*	104.6*
29-416/C1M	5.8*	110.3*
29-416/C1N	4.3*	111.8*
29-416/C1S	5.5*	111.0*
DOC19-1	5.3	110.4
DOC19-2	7.1	109.0
DOC19-3	6.4	109.8
DOC19-4	3.6	111.9

Notes: * – Inferred, SPT or Auger refusal

The bedrock encountered within the cored boreholes consisted of slightly weathered to freshly weathered grey to black limestone. The bedrock in Borehole DOC19-3 was observed to be vuggy between elevation 107.9 m and 109.4 m and contain voids and a clay seam at approximate elevation 107.0 m.

The Total Core Recovery (TCR) measured on the recovered bedrock core ranged from 68 to 100%, the Solid Core Recovery (SCR) ranged from 62 to 100% and the Rock Quality Designation (RQD) ranged from 15 to 100%. Based on the measured RQD values, the bedrock is classified as very poor to excellent quality (CFEM, 2006). Photographs of the bedrock core are provided in Appendix C.

The bedrock generally slopes downwards from north to south and east to west along the creek alignment.

5.7 Groundwater Conditions

The water level in Dochart Creek was measured at an approximate elevation of 115.5 m on November 26th, 2019. Two 38 mm diameter monitoring wells (DOC19-1 and DOC19-4) were installed at the site during the current investigation. Two 25 mm diameter piezometers were installed in boreholes from the previous investigations. Artesian conditions were noted at different locations across the site during and upon completion of drilling in Borehole 18-2, DOC19-1 and DOC 19-4 originating from the glacial till layer. The non-stabilized artesian level in Borehole 18-2 was measured at least 0.6 m above the ground surface or elevation 116.6 m; the artesian flow was sealed at the source with bentonite pellets while decommissioning the borehole. Groundwater levels from wells and piezometers are presented below:

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Table 5-7: Summary of Groundwater Levels

Borehole No.	Depth (mbgs)	Groundwater Elevation (m)	Date of Measurement	Notes
18-2	-0.6	116.6	June 26, 2018	25mm Standpipe Piezometer
29-416/C1M	0.0	116.1	June 1, 2005	25mm Standpipe Piezometer
29-416/C1M	-0.8	116.9	April 21, 2020	25mm Standpipe Piezometer
29-416/C1M	-0.6	116.7	April 28, 2020	25mm Standpipe Piezometer
29-416/C1M	-1.0	117.1	May 4, 2020	25mm Standpipe Piezometer
DOC19-1	-0.9	116.6	November 26, 2019	38mm Monitoring Well
DOC19-1	> -1.3	>117.0	April 21, 2020	38mm Monitoring Well
DOC19-1	> -1.1	>116.8	April 28, 2020	38mm Monitoring Well
DOC19-1	-1.5	117.2	May 4, 2020	38mm Monitoring Well
DOC19-4	-0.2	115.7	November 26, 2019	38mm Monitoring Well
DOC19-4	-0.3	115.8	April 21, 2020	38mm Monitoring Well
DOC19-4	-0.1	115.6	April 28, 2020	38mm Monitoring Well
DOC19-4	-0.4	115.9	May 4, 2020	38mm Monitoring Well
Creek Water Level	N/A	115.5	November 26, 2019	Creek near DOC19-1
Creek Water Level	N/A	116.44	N/A	2-yr Flow (from GA Drawing)

Note: Negative depth indicates above ground/artesian conditions

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

5.8 Analytical Testing

Samples of the native soils were submitted to Paracel Laboratories in Ottawa, Ontario for analysis of pH, water soluble sulphate, sulphides, chloride concentrations, resistivity and electrical conductivity. The analysis results are summarized below and provided in Appendix C.

Table 5-8: Results of Chemical Analysis

Sample	Depth (m)	pH	Resistivity (Ohm-cm)	Chloride (µg/g)	Sulphate (µg/g)	Sulphide (%)	Conductivity µS/cm
18-2 SS3	1.5	7.8	1300	393	77	<0.02	770
DOC19-1 SS2	1.1	7.8	1170	439	53	0.10	852
DOC19-4 SS1	0.3	7.5	2410	52	42	0.04	415

6 MISCELLANEOUS

Borehole locations were selected by Thurber relative to the proposed westbound centerline and to the existing culverts. The as-drilled locations and ground surface elevation of the boreholes were surveyed by Thurber following completion of the field program. The elevation survey was carried out with reference to geodetic elevation benchmarks provided by the MTO.

For the 2019 investigation, Marathon Drilling of Greely, Ontario supplied and operated the drilling equipment and carried out the drilling, soil sampling, in-situ testing, well installation and borehole decommissioning. The field investigation was supervised on a full-time basis by Mr. Michel Johnston of Thurber. Overall supervision of the investigation program was provided by Mr. Justin Gray, P.Eng.

Routine geotechnical laboratory testing was completed by Thurber's laboratory in Ottawa, Ontario. Oedometer testing was carried out by Thurber's Oakville laboratory. Analytical testing was completed by Paracel Laboratories in Ottawa, Ontario.

Overall project management and direction of the field program was provided by Dr. Fred Griffiths, P.Eng. Interpretation of the factual data and preparation of this report were carried out by Mr. Justin Gray, P.Eng. and by Dr. Fred Griffiths, P.Eng. The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

LICENSED PROFESSIONAL ENGINEER
J. A. GRAY
100198584
July 15, 2021
PROVINCE OF ONTARIO

Justin Gray, P.Eng.
Geotechnical Engineer

LICENSED PROFESSIONAL ENGINEER
D. J. PIZYCKI
100211802
July 15, 2021
PROVINCE OF ONTARIO

Deanna Pizycki, P.Eng.
Geotechnical Engineer

LICENSED PROFESSIONAL ENGINEER
F. J. GRIFFITHS
July 15, 2021
PROVINCE OF ONTARIO

Dr. Fred Griffiths, P.Eng.
Senior Associate

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
July 15, 2021
PROVINCE OF ONTARIO

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

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PART 2. ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 INTRODUCTION

This report presents the interpretation of the factual data obtained from a foundation investigation conducted by Thurber for the replacement of the Dochart Creek pipe culverts under the existing Highway 17 (proposed eastbound) embankment and the installation of a new structure for the new Highway 17 westbound embankment. The site is located approximately 600 m west of the Highway 17- Scheel Drive Overpasses in the Township of McNab/Braeside, within Renfrew County, Ontario.

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. Contractors must make their own interpretation based on the factual data in Part 1 of the report. Where comments are made on construction, they are provided only in order to highlight those aspects which could affect the design of the project. Contractors must make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods and scheduling.

The following sections provide geotechnical recommendations for the new and replacement structures under the twinned Highway 17 configuration, as well as for the high fill resulting from the proposed westbound embankment in vicinity of Dochart Creek. The discussions and recommendations presented in this report are based on the information provided by the Ministry of Transportation of Ontario (MTO), Morrison Hershfield (MH) and on the factual data obtained from the online Geocres Library and during the course of this investigation as presented in Part 1.

7.1 Existing Structure

It is noted that the existing culverts under the existing highway were subject of a detailed design assignment with the purpose of extending the culverts at the inlet (south) end. The associated 2018 GA is provided in Appendix I.

Three existing structural plate, corrugated steel pipe (CSP) culverts currently convey Dochart Creek beneath the Highway 17 embankment from south to north. No wingwalls or headwalls are present. The culverts are approximately 43.5 m long, and have a diameter of 2.75 m. They were installed on a 29 degree skew to the highway alignment with a center-to-center distance of approximately 4.4 m. The existing culvert inverts are at approximate elevation 115.2 m and 115.3 m at the south and north ends respectively. During a site visit on April 27, 2020, the main Dochart Creek channel at the new westbound lanes was 2 m to 3 m wide and about 0.7 m deep. Near the north end of the existing culverts (outlet), the creek was 12 m wide and about 0.3 m deep. See Photo 4 in Appendix D to see the orientation of the creek.

The top of the existing (future eastbound) embankment above the culverts is at approximate elevation 120.1 m. The existing embankment supports one east bound lane and one west bound lane separated by a central gore area. Paved shoulders are present on both sides. A guiderail is present on the south side and concrete barriers mark the north edge of shoulder. The paved width is approximately 16 m wide perpendicular to centreline. The embankment is approximately 5 m high and the embankment slopes are graded at approximately 2H:1V.

Photographs in Appendix D show the existing condition of the culverts and road platform at the time of the field investigations.

No evidence of excessive settlement, erosion or embankment instability was observed during the site investigation.

It is noted that a stockpile of rockfill approximately 40 m east of the creek and extending easterly along the new westbound alignment has been in-place since 2015 as part of the previous twinning contract (See Appendix A and Photo 4 in Appendix D).

7.2 Proposed Structure

The proposed work is a component of the Highway 17 Twinning Project from Scheel Drive westerly to past Renfrew.

Based on proposed alignments provided by MTO, the centreline of the proposed westbound lanes will be approximately 65 m north of the centreline of the existing highway which will become the new eastbound lanes. The proposed top of the new westbound embankment is expected to be 13.0 m wide to allow for two 3.75 m wide lanes, a 2.5 m wide median shoulder, and a 3.0 m wide outside paved shoulder. The new westbound road surface is to be at elevation 120.2 m at the locations of the creek crossing and the existing/proposed stream bed is at elevation 114.9 m. Similarly, the road surface will be at 120.3 m elevation and the existing/proposed stream bed at 115.3 m elevation for the eastbound lanes.

The 2004 Preliminary Design Report indicated that the crossing would consist of new culverts for both the eastbound and west bound embankments. The culverts would be 7.0 m wide by 2.7 m high concrete box culverts with lengths of approximately 40 m.

It is understood that a 2019 culvert inventory concluded that the existing Dochart Creek Culverts beneath existing Highway 17 are candidates for replacement due to the condition of the culverts below the waterline. It is anticipated that replacement of the existing culverts with new culverts for the new eastbound lanes would not modify the geometry of the existing Highway 17 embankment.

Bridges are also being considered for this site. It is noted that the available design of the bridges is also preliminary, six alternatives have been presented for each bridge with three girder types on two skews in a package of preliminary General Arrangement (GA) drawings dated November 2020. We have presented only one alternative for each bridge in Appendix I, however, it is noted that the span length and abutment location will be affected by the bridge alternative selected. For the purposes of this foundation design report, Option 1A has been presented for the eastbound bridge (24 m span, zero skew) and Option 1A has been presented for the westbound bridge (35 m span, zero skew). The elevation of the underside of the abutments for the westbound bridge will be assumed to be 114.1m and the elevation of the underside of the pile caps for the eastbound bridge will be assumed to be 115.5m. Wing walls approximately 7 m in length are indicated for all bridge options in all four quadrants for both bridges.

The recommendations presented herein must be reassessed once the type, location and orientation of the foundation elements are established to ensure suitability given the variations in stratigraphy and bedrock elevation at the site.

7.3 Design Code Considerations

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

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

It is understood that the new/replacement structures are being designed to the “Major Route” importance category.

This project has been assigned Typical Consequence Classification, in accordance with Section 6.5.1 of the CHBDC (pending confirmation by MTO). Accordingly, a consequence factor (Ψ) of 1.0, as per Table 6.1 of the CHBDC, has been used in assessing the factored geotechnical resistances. If the consequence classification changes, the geotechnical assessment and recommendations provided within this report will need to be reviewed and revised.

The degree of site and prediction model understanding for this site has been assessed to be typical understanding (Section 6.5.3 of CHBDC).

8 SEISMIC CONSIDERATIONS

8.1 Spectral and Peak Acceleration Hazard Values

The seismic hazard data for the CHBDC is based on the fifth-generation seismic model developed by the Geological Survey of Canada (GSC). Seismic hazard data for this site has been obtained from the GSC's seismic hazard calculator. The data includes peak ground acceleration (PGA), peak ground velocity (PGV), and the 5% damped spectral response acceleration values (Sa(T)) for the reference ground condition (Site Class C) for a range of periods (T) and for a range of return periods including the 475-year, 975-year and 2475-year events. The GSC seismic hazard calculation data sheet for this site is presented in Appendix F.

The site coefficients used to determine the design spectral acceleration and displacement values are a function of the Site Class, the peak ground acceleration (PGA) and Sa(0.2). The PGA for this location for a *reference* Site Class C with a 2% probability of exceedance in 50 years is 0.233g (1 in 2475 year). This value is to be scaled by the F(PGA) based on the site-specific Site Class as per Section 4.4.3.3 (Table 4.8) of the CHBDC (see Section 8.2).

8.2 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.

Based on the average undrained shear strengths measured below the anticipated foundation elevations, the site-specific Site Class is classified as a Seismic Site Class D in accordance with Table 4.1 of the CHBDC. As per Table 4.8 of the CHBDC for a 1 in 2475 year event, Site Class D with a PGA_{ref} of 0.186 (0.8*PGA, as per 4.4.3.3 of S6-19) yields an F(PGA) of 1.13 for the site. These values give a design PGA of 0.263g.

8.3 Seismic Liquefaction

The soils beneath the anticipated founding elevation consist of firm to very stiff clay overlying a loose to dense glacial till deposit. The glacial till is not considered susceptible to liquefaction under earthquake loading associated with the seismic hazard data for this site. The cohesive soils are also not susceptible to cyclic mobility based on the results of moisture content and plasticity testing and the criteria presented by Bray et. al. (2004)[i]. This is confirmed by the results of consolidation testing on samples of clay which indicate an Over Consolidation Ratio of 10 or greater for all four samples.

9 GEOTECHNICAL ASSESSMENT / CONSIDERATIONS

In general terms, the site was found to include embankment fill comprising sand, gravel and rockfill overlying a native deposit of clayey silt to clay, which is underlain by a till deposit. A summary of the clayey silt to clay material properties is presented against elevation in Appendix G. The overburden soils are underlain by limestone bedrock which is generally increasing in elevation to the north and to the east. An artesian groundwater condition (0.1 to 1.5 m above the existing ground surface) was observed originating from within the till layer.

Based on the results of the field and laboratory investigation and the information provided by MTO with regards to the proposed project requirements, the geotechnical foundation design considerations include the following:

Bearing Resistance

- The near surface overburden soils at this site, comprising of native firm to stiff clayey silt to clay, will not provide sufficient geotechnical resistance for spread footings to support an open footed culvert or bridge foundation. These structure options would require excavation of the existing overburden and placement of engineered fill or tremie concrete.
- The clayey silt to clay deposit at this site would provide sufficient bearing resistance to allow for the installation of a closed bottom box culvert or multiple circular culverts.
- Deep foundations could be used to support an open bottom culvert or short single span bridge. The deep foundations should extend to bedrock. The depth to bedrock is variable and slopes at this site.

Settlement

The reinstated eastbound embankment will have a similar height and footprint to the existing embankment, no additional settlement is expected along the existing alignment. However, settlement should be expected if the embankments are reinstated to design grades greater than the existing grades.

The future westbound alignment will result in a fill height about 5 m higher than the existing grades. Settlement and differential settlement of the roadway embankment need to be considered not only in terms of pavement performance on the approaches but also in selection and design of the structure foundations.

An assessment of the time dependent settlement that would result from construction of the proposed westbound embankment using conventional granular fill with 2H:1V side slopes was carried out using Rocscience's Settle3 modelling software with a Boussinesq stress distribution. The design pre-consolidation pressure profile has been derived from the oedometer test carried out on the native clay material, supplemented by a correlation with undrained shear strength and index properties.

The following has been used for assessment of the embankment geometry:

- Proposed Grade = 120.2 m
- Original Grade = 115.0 m
- Groundwater Level = 116.2 m
- Platform Width = 13.0 m
- Side slopes = 2H:1V

The geotechnical parameters used in the settlement analysis were based on soil thicknesses encountered in Borehole DOC19-3 and the consolidation test results from Boreholes DOC19-2.

Table 9-1 presents the properties used in the Settle3 analyses for the various sub-layers.

Table 9-1: Settle3 Inputs

Layer	Elevation (m)	C_c	Cr	P_c' (kPa)	e_o	c_v (mm^2/s)	c_{vr} (mm^2/s)	E_s (kPa)
Clay	115 to 111	0.76	0.05	170 to 215	1.513	0.16	1.9	-
Till	111 to 110	-	-	-	-	-	-	30,000

The results of the settlement analyses for the proposed approach embankments are summarized as follows:

- The magnitude of *total* settlement beneath the proposed embankment centerline of the new westbound lane has been estimated to be about 110 mm.
- The magnitude of the embankment compression constructed with granular materials is in the order of 0.5% of the embankment height, i.e. approximately 25 mm and is expected to occur during and following fill placement.
- It is estimated there will be approximately 5 to 10 mm of immediate settlement, occurring largely in the till layer above the bedrock which will occur during construction.
- The effective stresses in the clay layer are expected to remain below the pre-consolidation pressure. Time dependent recompression of the clay layer is estimated to be approximately 80 mm.

The results of settlement analyses for the new westbound lane at the proposed culvert alignment or proposed bridge abutments are summarized as follows:

- The magnitude of *total* settlement beneath the proposed culvert or at the proposed bridge abutments has been estimated to be about 85 mm
- It is anticipated that it will take less than 2 to 3 months to achieve 90% of the settlement due to the proposed grade raise. The site is suitable for application of a preload. A temporary culvert may be required to be installed to accommodate creek flow during the preload period.

It should be noted that the estimated total settlements presented above are greater than the tolerable limit at a structure as presented in the MTO document "Embankment Settlement Criteria for Design, July 2010". Therefore, without preloading, additional maintenance measures would be required following construction.

If preloading time is not available, light weight fill such as Expanded Polystyrene (EPS) could be considered for use as embankment fill. The use of EPS would need to take the effects of buoyancy into consideration and EPS should not be placed below the high-water level.

THURBER

It is anticipated that since the new westbound lanes will be constructed along a new alignment, preloading should be achievable and is considered to be a suitable mitigation measure.

Construction

Excavations will extend below the water level in the creek. An adequate and effective dewatering plan including surface water management, cofferdams, creek diversion and excavation dewatering will be required to enable excavation to the required founding elevation and construction of the foundations in the dry (See Section 12.3).

Artesian conditions were encountered in the till below the clayey silt to clay. Deep foundations extending to or though this layer will need to include design details to stabilize the water during construction and mitigate loss of fines and lateral support of the foundation over time. In addition, dewatering plans for excavations into the overlying clayey silt to clay layer may need to incorporate special measures to prevent basal heave from hydrostatic pressures.

10 EVALUATION OF DESIGN OPTIONS

It is noted that the foundation investigations at this site completed to date were planned to provide recommendations for structural culverts. Design-build contractors should consider drilling additional boreholes to meet MTO requirements for bridge foundations.

Both culvert and bridge structures were considered as part of the current study. The following sections outline the foundation options evaluated.

10.1 Culvert Type/Foundation Alternatives

A detailed assessment of culvert types and foundation options was carried out. Options evaluated included open bottom culverts, closed concrete box culverts and multiple circular culverts. The key findings and conclusion of the assessment are summarized as follows:

- Open footed concrete or steel plate arch culverts on spread footings are not feasible at this site due to insufficient bearing resistance available from the underlying clay and the potential settlement in the clay due to footing loads.
- Open footed concrete or steel plate arch culverts on deep foundations were determined to be feasible but are not cost effective and hence not recommended.
- Multiple circular pipes installed with appropriate granular bedding over the clay subgrade in a similar manner to the existing eastbound culverts were determined to be feasible.
- Closed bottom box culverts supported on the clay were determined to be feasible and are recommended however mitigation of anticipated settlement under the new westbound embankment will be required.

An evaluation of the culvert/foundation alternatives including the advantages, disadvantages, risk/consequences and relative cost from a foundation perspective is provided in Appendix E.

10.2 Bridge Foundations

An assessment of the foundation options for the support of bridge structures was also carried out. The comments below are based on the available borehole data. Design-build contractors should consider drilling additional boreholes to meet MTO requirements for bridge foundations. The key findings and conclusions of the assessment are summarized as follows:

- Spread footings: the use of spread footings would require the abutments to be founded at or below the depth of frost. The soils expected to be encountered at the founding elevation would be native clayey silt to clay which would be prone to differential settlement and would provide a low bearing capacity relative to other foundation options. Scour and erosion protection must be considered for footings. Spread footings do not allow for construction of integral abutments.

- Spread footings on an engineered pad constructed on the underlying till was considered for the new westbound structure. However, it is not recommended from a foundations perspective as the till varies from loose to dense and additional construction concerns are present due to the artesian conditions encountered in the till and the difficulties that would be encountered with deep excavations and dewatering. As noted above, spread footings do not allow for construction of integral abutments.

  It is not recommended to support the new eastbound bridge on spread footings on an engineered fill as the top of the till layer was observed to be as deep as 7.7m below stream bed in Borehole 18-2.

- Spread footings on mass concrete have also been considered for the new westbound bridge. For this scenario, all of the overburden would be removed and mass concrete placed on the bedrock up to the underside of the abutment footings. Consideration could be given to placing the concrete with tremie techniques to minimize the impact of the artesian conditions. It is likely that this solution will be relatively expensive due to the depth of excavation and quantity of concrete required. As noted above, spread footings do not allow for construction of integral abutments.

  It is not recommended to support the new eastbound bridge on spread footings on mass concrete as the bedrock surface was observed to be more than 10.7m below stream bed in Borehole 18-2.

- Steel H-Piles: bedrock was inferred or encountered at elevations ranging from approximately 104.6 (Borehole 18-2) to 111.9 m (Borehole DOC19-4) which could result in driven piles seated on bedrock as short as 2.2 m below the underside of the abutment of the westbound bridge. It should be expected that bedrock elevations could be higher at other locations. Cobbles and boulders may be encountered in the till and could also affect pile length. The depth to bedrock and presence of boulders in the till may not provide the required flexible length of piles needed and would therefore limit the feasibility of integral abutments. Therefore, *driven* steel piles are not considered appropriate for a majority of this site. Drilled-in steel piles socketed into bedrock could be considered feasible but would require liners when drilling through the artesian pressured till. Integral abutments could be included in a design with drilled-in steel piles socketed into bedrock.

- Micropiles offer lower lateral capacities compared to other deep foundation options and have a higher cost. Therefore, micropiles will not be discussed further within this report.

- Caissons: supporting the bridge abutments on caissons socketed into bedrock is considered a suitable foundation option. Socketed caissons generally provide a high geotechnical resistance, however, the voids and clay seams observed in the bedrock core will reduce the geotechnical capacity at this site. The high lateral stiffness of caissons is not compatible with integral abutments. The installation methodology would need to take into account the artesian pressures in the till and the potential for unbalanced pressure heads. A temporary liner would be required and should be extended above the ground surface during construction to stabilize the artesian pressures prior to placing concrete. The rockfill within the existing embankment would likely be removed as part of the culvert removal. Nonetheless, caisson installation equipment should be able to advance past cobble and boulder sized particles as they may be encountered in the till.

An evaluation of the bridge foundation alternatives including the advantages, disadvantages, risk/consequences and relative cost from a foundation perspective is provided in Appendix E.

10.3 Construction Staging Alternatives

Installation of a culvert beneath the new westbound alignment is expected to be straightforward with no major staging requirements, however, installation of a temporary CSP culvert(s) is anticipated to allow preloading for a two month period prior to installation of the permanent culvert (See Section 9). Should a suitable cambered or an over-sized culvert designed to accommodate the anticipated settlement be selected, there would likely be no need for a preload period.

Installation for a bridge for the new westbound alignment would require preloading the approach embankments. It is anticipated that sufficient space exists along the proposed westbound alignment to complete the preloading without encroaching into the existing creek and it is expected that preloading for the approach embankments could likely be completed without the need for the installation of a temporary culvert(s). The available space should be reviewed further as staging plans are developed.

It is anticipated that traffic will be detoured onto the new westbound lanes during removal of the existing culverts beneath the existing Highway 17 (future eastbound lanes). Structure replacement either with a culvert or bridge would be unimpeded and straightforward for the eastbound lanes with traffic on the new embankment to the north. The Highway 17 twinning project staging is not fully developed currently, additional recommendations can be provided for temporary embankment widening and roadway protection, if needed.

10.4 Recommended Approach

A closed bottom box culvert is recommended for both the eastbound and westbound lanes. It is anticipated that construction for the westbound lanes would be carried out while traffic remains on the existing alignment. A minimum two month preload period prior to carrying out the open cut excavation for culvert installation would be required. It is anticipated that all traffic will be detoured onto the new westbound lanes upon their completion. This would allow replacement of the eastbound culvert with an open cut under full road closure.

Alternatively, single span bridges supported on rock socketed caissons could be selected. Construction staging would be similar to that for the culvert option.

Based on the shallow variable bedrock, at the westbound structure in particular, the site in general is not considered suitable for driven piles and consequently not suitable for integral abutments. However, it is recognized that integral abutment bridges offer significant long-term advantages. Therefore recommendations are provided herein for drilled-in, rock socketed piles specifically for integral abutments.

11 FOUNDATION DESIGN RECCOMENDATIONS

The recommendations presented herein must be reassessed once the type, location and orientation of the foundation elements are established to ensure suitability given the variations in stratigraphy and bedrock elevation at the site.

11.1 Culvert Foundations

Closed bottom box culverts are recommended for both the eastbound and westbound lanes.

Approximate key elevations are as follows:

• Proposed top of pavement	120.2 m
• Proposed invert	115.1 m
• Dochart Creek water level on November 26, 2019	115.5 m
• Top of clay layer	115.4 to 116.7 m
• Glacial till surface	107.6 to 112.1 m
• Groundwater level on November 26, 2019	115.7 to 116.6 m

11.1.1 Bearing Resistances

The recommended geotechnical resistances for a 7.0 m wide (interior) pre-cast closed bottom box culvert with a 0.4 m thick lower slab installed on a 0.5 m thick bedding layer on an undisturbed clay subgrade at or below elevation 115.1 m are as follows:

- Factored Geotechnical Resistance at ULS of 155 kPa
- Factored Geotechnical Resistance at SLS of 75 kPa

The factored geotechnical resistances include the following factors:

- Consequence factor (Ψ) of 1.0 (as per CHBDC Table 6.1)
- Geotechnical resistance factors (as per CHBDC Table 6.2):
  - ϕ_{gu} = 0.5 (static analysis; typical degree of understanding)
  - ϕ_{gs} = 0.8 (static analysis; typical degree of understanding)

The bearing resistance values are for vertical, concentric loading. In the case of eccentric or inclined loading, the bearing resistance must be reduced in accordance with CHBDC Clause 6.10.2 and Clause 6.10.5. Foundation settlement, based on the above SLS resistance, is expected to be less than 25 mm.

Resistance to lateral forces/sliding resistance between the precast concrete and the underlying Granular 'A' bedding (Section 11.1.2) should be evaluated in accordance with the CHBDC assuming an unfactored coefficient of friction of 0.45. An unfactored coefficient of friction of 0.35 can be assumed for the interface between the Granular 'A' and the clay.

11.1.2 Subgrade Preparation, Bedding and Backfilling

After excavation and removal of the existing culvert and existing fill, all organics, peat, soft or loose deposits, disturbed soils, alluvial deposits and deleterious materials must be stripped from the footprint of the culvert foundations to expose competent native subgrade material at or below the desired founding elevations. Given the sensitive clayey silt to clay materials anticipated at the founding level of the new culverts it is critical not to disturb the subgrade, construction equipment should not be permitted to travel on the exposed subgrade.

The exposed subgrade must be inspected to confirm that the subgrade is suitable and uniformly competent. Any soft or organic materials at the subgrade level should be sub-excavated and replaced with granular fill consisting of OPSS.PROV 1010 Granular A material compacted as per OPSS.PROV 501 as soon as practical to protect the subgrade from disturbance during construction. In order to provide a more uniform foundation subgrade condition for the culvert, a minimum 0.5 m thick layer of bedding material conforming to OPSS.PROV 1010 Granular A requirements must be provided under the base of the culvert as per OPSS 422 and OPSD 803.010 (box culvert) unless loose/soft or organic deposits are encountered at the founding elevation where sub-excavation will then be required as recommended above.

The compaction of granular bedding directly above the subgrade may result in disturbance of the material with pumping of fines into the granular bedding and difficulty achieving the specified degree of compaction. Protection of the subgrade should include installation of a Class II non-woven geotextile with a maximum FOS of 150 μm (OPSS.PROV 1860) installed beneath the 0.5 m thick Granular A bedding layer. The geotextile should be placed as soon as possible after reaching the subgrade level and following receipt of written notice to proceed in accordance with

SP 109S12. An NSSP is provided in Appendix J to include in the contract documents to alert the Contractor of the sensitive nature of the foundation soils.

It is noted that construction will extend below the creek elevation. Water diversion and dewatering will be required to prepare the subgrade in the dry.

It is recommended that culvert cover be in accordance with OPSS 902 and consist of free-draining, non-frost susceptible granular materials such as Granular A or Granular B Type II material meeting the requirements of OPSS.PROV 1010.

Culvert backfill above the granular cover should be in accordance with OPSS 902 and consist of material meeting the requirements of OPSS Granular A or Granular B Type I, II or III compacted in regular lifts as per OPSS.PROV 501. Heavy compaction equipment, used adjacent to the culvert, must be restricted in accordance with OPSS.PROV 501. Care must be exercised when compacting the fill adjacent to and above the culvert in order not to damage the culvert.

11.2 Bridge Foundations

The geotechnical recommendations provided below are based on the subsurface data provided in Part 1 of this report. Design-build contractors should consider drilling additional boreholes to meet MTO requirements for bridge foundations.

Approximate key elevations are as follows:

• Proposed underside of abutment for westbound structure	114.1 m
• Proposed underside of abutment for eastbound structure	115.5 m
• Glacial till surface	107.6 to 112.1 m
• Bedrock surface at westbound structure	109.0 to 111.8* m
• Bedrock surface at eastbound structure	104.6* to 110.4 m

Note (*) – Inferred, SPT or Auger refusal

11.2.1 Spread Footings on an Engineered Granular Pad

As indicated in Section 10.2, spread footings supported on an engineered pad constructed on the underlying till was considered for the new westbound lane structure, only. However, in light of the excavation depth and dewatering requirement it is not recommended from a foundations perspective and foundation recommendations have not been provided.

11.2.2 Spread Footings on Mass Concrete Extended to Bedrock

Spread footings on mass concrete have also been considered for the new westbound bridge. All of the overburden should be removed and mass concrete placed on the bedrock up to the underside of the abutment footings. Consideration could be given to placing the concrete with tremie techniques to minimize the impact of the artesian conditions. The area of the mass concrete should extend at least 0.5 m beyond the perimeter of the footing. The mass concrete should be the same class and strength as the footing concrete.

The recommended geotechnical resistances for a 5.0 m wide footing installed on mass concrete placed on bedrock are as follows:

- Factored Geotechnical Resistance at ULS of 3,000 kPa
- Factored Geotechnical Resistance at SLS is not applicable.

The factored geotechnical resistances include the following factors:

- Consequence factor (Ψ) of 1.0 (as per CHBDC Table 6.1)
- Geotechnical resistance factors (as per CHBDC Table 6.2):
  - ϕ_{gu} = 0.5 (static analysis; typical degree of understanding)
  - ϕ_{gs} = 0.8 (static analysis; typical degree of understanding)

The bearing resistance values are for vertical, concentric loading. In the case of eccentric or inclined loading, the bearing resistance must be reduced in accordance with CHBDC Clause 6.10.2 and Clause 6.10.5.

Resistance to lateral forces/sliding resistance between the cast-in-place concrete and the underlying bedrock should be evaluated in accordance with the CHBDC assuming an unfactored coefficient of friction of 0.70. If sufficient lateral resistance is not available, rock dowels could be considered.

11.2.3 Caissons

Drilled in caissons socketed into sound bedrock are a feasible option to support the new bridge structures. The caissons should consist of temporary steel casing liners seated onto bedrock and filled with concrete. The steel liners must be continuous and form a tight seal at the bedrock surface to minimize the ingress of soils and artesian water pressures and to facilitate cleaning of the socket base. The caisson base should be inspected as per OPSS.PROV 903.

The installation methodology would need to consider the artesian pressures in the till and the potential for unbalanced pressure heads. The temporary liner should be extended above the ground surface during construction to stabilize the artesian pressures prior to placing concrete.

11.2.3.1 Axial Geotechnical Resistance and Founding Elevation

The axial geotechnical capacity at factored ULS for a caisson socketed a minimum of 2 caisson diameters into sound bedrock is provided in the table below. The caisson capacities include a resistance factor of 0.4 and 0.3 (ϕ_{gu}) for ULS compression and tension, respectively as per Table 6.2 of the CHBDC (static analysis – typical understanding). The SLS condition will not govern for a caisson socketed into sound bedrock.

Table 11-1 Axial Geotechnical Resistance for Caissons

Caisson Diameter (mm)	Factored ULS (Compression) (kN)	Factored ULS (Tension) (kN)	Factored SLS (Compression) (kN)
406	1,050	250	will not govern
610	2,400	550	will not govern
915	5,000	1250	will not govern
1200	9,000	2000	will not govern

The structural resistance of the caissons must be checked by the structural designer. The required depth of socket into sound bedrock should be lengthened, if required, based on the required lateral capacity requirements (recommendations provided in Section 11.2.3.3), moment capacity and seismic analysis to satisfy the structural assessment.

Construction of caissons will require temporary steel casing to support the sidewalls through the native soils and enable machine-cleaning of the socket base. The axial bearing resistances provided are based, in part, on end bearing and the base of the socket must be thoroughly cleaned. The caisson equipment supplied by the Contractor must be capable of advancing through the existing soils and penetrate or push aside potential obstructions in the till. Coring equipment must be able to seat the casing into sloping bedrock and also penetrate into the bedrock without fracturing the sidewalls. The tension/uplift resistances provided are based on full contact of the caisson concrete with the socket sidewalls.

11.2.3.2 Downdrag

Downdrag forces (negative skin friction) acting upon the caisson are expected to develop as a result of settlement of the cohesive overburden soils under the imposed loading from the newly placed fill. For this reason, it is recommended that the preloading period outlined in Sections 9 and 10.3, for construction of the new westbound lane embankments, be incorporated into the design.

If initiation of the construction of the caissons for the new westbound lane structure is delayed to occur after the end of the preload period, downdrag forces need not be considered. Likewise, downdrag forces for the new eastbound lane structure need not be applied if the reinstated grades and cross-section are not changed from the existing.

If construction staging does not permit a preloading period, downdrag loads will need to be designed to carry the additional static downdrag loads developed along the length of the caissons embedded in the cohesive layers and overlying materials. The *unfactored* static downdrag load for each caisson size is presented in Table 11-2.

Table 11-2 *Unfactored* Downdrag Load for Caissons

Caisson Diameter (mm)	*Unfactored* Static Downdrag Load (kN)
406	250
380	380
915	570
1200	750

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

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

11.2.3.3 Lateral Geotechnical Resistance and Group Effects

The lateral resistance of a caisson can be estimated using p-y curves. The p-y curves for static conditions are shown in Table H1 in Appendix H to allow for the calculation of the ultimate lateral capacity. A geotechnical resistance factor of 0.5 (ϕ_{gu}) and 0.8 (ϕ_{gs}) as per Table 6.2 of the CHBDC (static analysis – typical understanding) should be applied to the ultimate ULS and SLS values, respectively.

A minimum caisson embedment of two caisson diameter into sound bedrock should be used in design irrespective of the calculated lateral capacity.

Where the lateral spacing between an adjacent caisson embedded into the rock is less than 4 equivalent diameters, the subgrade modulus will need to be reduced based on the center-to-center spacing. The reduction factors to be used are provided in Figure C6.22, C6.23 and C6.24 of the CHBDC.

11.2.3.4 Abutment Type

The subsurface conditions at this site are considered suitable for caisson foundations and semi-integral abutments.

11.2.4 H-Piles / Integral Abutments

As discussed in Section 10.2, driven piles are not considered a feasible option at this site given the shallow bedrock.

There are two concerns with integral abutments at this site: shallow bedrock and artesian groundwater conditions.

<u>Shallow Bedrock</u>

The bedrock elevation in the boreholes at the westbound structure was observed or inferred to range from 109.0 to 111.9m; piles driven to bedrock would therefore range from 2.2 to 5.1 m in length. Similarly, the bedrock elevation in the boreholes at the eastbound structure was observed or inferred to range from 108.0 to 110.4m; piles driven to bedrock would therefore range from 5.1 to 7.5m in length. It is noted that bedrock elevation will vary between borehole locations. Piles for integral abutments typically exceed 5m in length below the abutment stem with the upper 3m in loose sand. Integral abutments could be utilized for these structures as follows:

1. Utilize standard installation procedure for H-Piles supporting integral abutments in locations where bedrock is encountered deeper than 5 m below the underside of the abutment. H-Piles should be driven to refusal on bedrock and the uppermost 3m of the pile should be placed in a 600mm diameter CSP. The CSP should be filled with loose sand.
2. A modified procedure should be used for H-Piles supporting integral abutments in locations where bedrock is encountered less than 5 m below the underside of the abutment. A minimum pile length of 5 m is required. The lower portion of the pile should be placed within a drilled rock socket 900 mm in diameter and at least 1 m in depth in sound bedrock. This is the minimum socket from a geotechnical engineering perspective. The structural designer should assess the required socket depth to satisfy toe fixity and lateral stability requirements. The lower portion of the socket should be filled with tremie concrete around the H-Pile to ensure fixity, minimum of 1m of concrete. The drilled hole should be 900 mm in diameter and lined when advanced through overburden. The liner should remain in place and the upper 4 m of pile length should be backfilled with loose sand (liner and upper portion of rock socket).

An NSSP has been provided in Appendix J outlining the gradation requirements for the sand backfill to be used in the CSP.

Given the variability of the bedrock surface at this site (5m$\pm$2m) and the likelihood of encountering cobbles and boulders in the till overlying the bedrock, it is recommended that all H-Piles be installed in pre-drilled holes extending a minimum of 1m into bedrock at both structures. Further, a single procedure will consolidate the construction equipment required to complete the work.

Artesian Groundwater Conditions

Artesian groundwater conditions were noted in several of the piezometers and wells with the highest observed level at 116.6 m elevation. The artesian conditions will require special consideration during rock socketing such as the use of liners, drilling mud and a tight seal at the bedrock surface. In addition, where long term flow of groundwater through the loose sand around the pile could occur, the upper portions of the liner should include a granular filter progressing upwards from fine to coarse material to prevent loss of materials. Where long term flow of groundwater from the till to ground surface is not permitted for environmental reasons or may have an impact on neighboring wells, a bentonite seal will be required within the liner.

11.2.4.1 Axial Geotechnical Resistance and Founding Elevation

The geotechnical axial resistances for HP310x110 piles socketed in bedrock are provided in Table 11-3 and may be used in design.

Table 11-3 Axial Geotechnical Resistance for HP310x110 Piles

Pile Size	Factored ULS (Compression) (kN)	Factored ULS (Tension) (kN)	Factored SLS (Compression) (kN)
HP310x110	2000	200	will not govern

The geotechnical resistances provided in the table above are applicable for pile spacing greater than 3 pile diameters as per Section 6.11.4.7 of the CHBDC. The pile capacities also include a resistance factor of 0.4 (ϕgu), 0.8 (ϕgs) and 0.3 (ϕgu) for ULS compression, SLS compression and ULS tension values, respectively, as per Table 6.2 of the CHBDC (static analysis – typical understanding).

The structural resistance of the pile must be checked by the structural engineer which may govern the design.

11.2.4.2 Downdrag

Downdrag loading will not need to be considered if the H-Piles are installed within permanent liners socketed into bedrock.

11.2.4.3 Lateral Geotechnical Resistance and Group Effects

The ultimate lateral resistance force that can be mobilized by the embedded portion of H-Piles concreted within 900 mm diameter pre-drilled rock sockets in sound bedrock is constant with depth and can be taken as 2,500 kN/m length of concrete encapsulated pile in sound rock. A suitable reduction factor should be applied to this ultimate value in accordance with Table 6.2 of the CHBDC. The socket depth should be designed based on lateral resistance of fixity requirements but should not be less than 1 m into sound bedrock. The structural capacity of the concrete within the bedrock socket should be verified.

Where the lateral spacing between an adjacent pile embedded into the rock is less than 4 equivalent diameters, the subgrade modulus will need to be reduced based on the center-to-center spacing. The reduction factors to be used are provided in Figure C6.22, C6.23 and C6.24 of the CHBDC.

11.2.4.4 Abutment Type

Integral abutments are considered suitable for this site provided bedrock is cored to allow for piles to be socketed into bedrock as described above.

11.2.5 Wingwalls

Based on the available GA drawings, it is understood that the wingwalls could be cantilevered from the abutment stem and in that case, foundation recommendations would not required.

If concrete wingwall foundations are required, footings for wingwalls should be founded at a depth, when measured perpendicular to the ground surface, that is greater than the depth of frost (see Section 11.3). The walls should be founded on a leveling pad with a minimum thickness of 0.5 m consisting of Granular A material. The top of the Granular A pad must extend to 0.5 m beyond the outside edge of all sides of the footing and sloped away from the footing at 1H:1V, or flatter.

The geotechnical resistance values and subgrade provided in Section 11.1.1 are recommended for an assumed 10m long wingwall with a 2.0 m base width on an engineered pad 0.5 m thick placed on native, undisturbed clayey silt/clay. Subgrade preparation recommendations are provided in Section 11.1.2.

11.2.6 Retained Soil Systems

A retained soil system (RSS) for a culvert wingwall or retaining wall is not recommended at this site as it is located within a watercourse and could be affected by fluctuating water levels.

11.3 Frost Protection

The frost penetration depth at this site is 1.9 m as per OPSD 3090.101. The underside of pile caps and concrete wing wall foundations should be provided with 1.9 m of frost cover. Typically, closed bottom box culverts are not provided with frost protection.

The Pavement Design Report should be consulted for the need for frost tapers for the culvert solution.

11.4 Lateral Earth Pressures and Structure Backfill

Backfilling and monitoring of backfilling operations for the installation of the structures should be carried out in accordance OPSS 902 and MTO Special Provision (SP) 109S12.

Structural backfill material should consist of Granular A or Granular B Type II meeting the OPSS.PROV 1010 and SP110S06 specifications. Large scale direct shear box testing on samples of Granular A and Granular B Type II from numerous nearby aggregate sources was completed for this project. The results indicate that for design of structural backfill for this project, an internal angle of friction of 40 degrees can be used for Granular B Type II and quarry- sourced Granular A in this area provided the vertical pressure on the material is less than 150 kPa (Geocres Memorandum 31F-213). An Operational Constraint will be required in the contract restricting the source of Granular A to quarries.

Backfill for the culverts or abutments should be placed and compacted in accordance with OPSS.PROV 501. The compaction equipment to be used adjacent to the structure must be restricted in accordance with OPSS.PROV 501. For culverts, the minimum granular backfill requirements shall be similar to those shown on OPSD 803.010, notwithstanding the culvert span.

It is recommended that abutment incorporate a subdrain as shown in OPSD 3101.150. The lateral earth pressure parameters provided in Table 11-4 and Table 11-5 are based on the assumption that the backfill is fully drained so that there are no unbalanced hydrostatic pressures. If adequate drainage cannot be confirmed, the potential for buildup of hydrostatic pressures should be considered in the design.

11.4.1 Static Lateral Earth Pressure Coefficients

Lateral earth pressures acting on structures shall be computed in accordance with the CHBDC but generally are given by the expression:

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

where:

- σ_h = static lateral earth pressure on the wall at depth d (kPa)
- K = static earth pressure coefficient
- γ = unit weight of retained soil (kN/m^3); use submerged unit weight for soils below the groundwater level
- d = depth below top of fill where pressure is computed (m)
- q = value of any surcharge (kPa)

The recommended lateral earth pressure parameters for use in the design of vertical walls with a horizontal backslope are provided in Table 11-1.

If lateral movement is not permissible and/or the wall is retained from lateral yielding, it is recommended that the at-rest horizontal lateral earth pressures be used for design. Active pressures shall be used for the design of unrestrained walls. For static analysis of permanent structures, passive earth resistance should be ignored, and therefore has not been provided.

Table 11-4: Static Lateral Earth Pressure Coefficient, K

Parameter	OPSS Granular B Type II and Quarry Source Granular A	Pit Sourced OPSS Granular A
Soil Unit Weight, kN/m^3, γ	22.8	22.8
Angle of Internal Friction, ϕ	40°	35°
Coefficient of at Rest Earth Pressure, K_o (Restrained Wall)	0.36	0.43
Coefficient of Active Earth Pressure, K_A (Unrestrained Wall)	0.22	0.27

The parameters in the table correspond to full mobilization of the earth pressures and require certain relative movements between the wall and adjacent soil to produce these conditions. Table C6.12 of the Commentary to the CHBDC indicates the relative movement required to fully mobilize the earth pressures. Where ground surfaces are sloped behind the walls, the coefficients will need to be revised.

A lateral pressure due to backfill compaction shall be added to the calculated lateral earth pressure in accordance with Section 6.12.3 of the CHBDC. A live load surcharge shall be considered as per Section 6.12.5 of the CHBDC.

11.4.2 Combined Static and Seismic Lateral Earth Pressure Parameters

The following recommendations are per Section C4.6.5 of the Commentary of the CHBDC which states that seismically induced lateral soil pressures may be calculated using the Mononobe-Okabe Method with:

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

The recommended combined static and seismic lateral earth pressure parameters for use in the design of vertical walls that are provided in Table 11-2 assume the following:

- Seismic Site Class of D,
- Site Coefficient F(PGA) of 1.13 as per Table 4.8 of the CHBDC, and
- Site adjusted PGA value with a 2% probability of exceedance in 50 years of 0.263g as outlined in Section 8.2.

Table 11-5: Lateral Earth Pressure (Under Combined Static and Seismic Loads)

Parameter	OPSS Granular B Type II and Quarry Source Granular A	Pit Sourced OPSS Granular A
Soil Unit Weight, kN/m^3, γ	22.8	22.8
Angle of Internal Friction, ϕ	40°	35°
Coefficient of Active Earth Pressure, K_{AE} (Restrained Wall)	0.37	0.45
Coefficient of Active Earth Pressure, K_{AE} (Unrestrained Wall)	0.29	0.35

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

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

where:

σ_{hE} = seismic lateral earth pressure on wall at depth d (kPa)
d = depth below the top of the wall (m)
K = static earth pressure coefficient
(K_o for non-yielding and K_A for yielding walls)
γ = unit weight of retained soil (kN/m^3); use submerged unit weight for soils below the groundwater level
K_{AE} = combined static and seismic earth pressure coefficient
H = total height of the wall (m)

11.5 Embankment Design and Reinstatement

11.5.1 Embankment Construction

The embankment material could consist of OPSS Granular A, Granular B Type I, II or III, or Select Subgrade Material (SSM). Backfill adjacent to and on top of structures should be as per Section 11.4. All backfill and embankment material shall be placed and compacted in accordance with OPSS.PROV 501

11.5.2 Reinstatement of the Eastbound Embankment

Reinstatement of the eastbound embankment is expected to result in minimal settlement of the underlying soils.

The global stability for the reinstated eastbound embankment is not an issue provided the embankment is constructed with 2H:1V or flatter side slopes.

11.5.3 Construction of the Westbound Embankment

The new westbound embankment is expected to settle approximately 110 mm at the new centreline, with approximately 90% of the settlement occurring over the first month (See Section 9).

Consideration could be given to preloading the area prior to constructing the structure. Preloading may require installation of a temporary culvert such as a CSP culvert. Monitoring of the embankment would be required to determine when the settlement was complete. With a preload scenario it is anticipated that that construction of the foundations of the new structure would commence approximately 2 months after placement of the embankment fill.

Alternatively, the structure could be installed without a preload provided:

- a segmental culvert designed to accommodate the settlement with specially designed joints was utilized and included an adequate camber or over-sizing. Overbuilding of the embankment height could be considered to minimize the subsequent maintenance which will be required to re-establish design grades of the embankment after settlement occurs.
- the foundation design for a bridge on deep foundations includes downdrag loading and it is acknowledged that the additional maintenance will be required to re-establish design grades of the embankment after settlement occurs.

11.5.4 Global Stability

Based on the general arrangement drawings, the proposed grade of the travelled lanes for both the eastbound and westbound alignment is to be at about Elevation 120.2 m, requiring embankment fill up to 4.2 m high above the existing ground surface, which is at about Elevation 116 m.

A slope stability assessment of the embankment was carried out for the westbound lanes only; it is assumed the eastbound embankment will reinstated to a stable condition using conventional granular fill with 2H:1V or flatter side slopes.

Slope stability assessments were also carried out for the forward slopes (towards the creek) if a bridge alternative is selected. The assessments were carried out for the west abutment at both of the westbound and eastbound embankments; the west abutment is the critical section with a lower bedrock elevation.

Embankment slope stability was evaluated using GeoStudio 2020 Slope/W software for limit equilibrium analysis. Input parameters for the analysis are based on the SPT N values, undrained shear strengths and the results of laboratory testing. The following additional parameters were used in the analysis:

- Estimated soil stratigraphy based on the nearest boreholes;
- For the embankment side slopes, the maximum fill height of 3.7m is based on a roadway elevation of 120.2 m and an original grade of 116.5 m at Station 21+330 (greatest height of SSM fill).
- Groundwater table at elevation 115.5 m;
- Side slopes of 2H:1V for SSM fill;
- A traffic surcharge load as per Section 6.12.5 of the CHBDC (static analysis only);
- Site adjusted PGA value of 0.132 g, equal to ½ of the site adjusted PGA value (0.263 g) was used for seismic analysis, as per Section 4.4.3.3, of the CHBDC and outlined in Section 8.2; and
- A traffic surcharge of 17 kPa has been applied as a temporary load.

Copies of the output from the stability analyses are provided in Appendix G, Figures G1 to G9. Each output figure shows the slope geometry, groundwater conditions, soil stratigraphy and soil strength parameters utilized in the analysis. The stability analyses generated the following factor of safety values:

Table 11-6 Slope Stability Anaysis Results

Condition	Case	Factor of Safety		
		Westbound Embankment (2H:1V) [Perpendicular to CL]	**Westbound Forward Slope (West Abutment)**	**Eastbound Forward Slope (West Abutment)**
Temporary (traffic loading)	Short Term (Undrained)	1.8 (Figure F1)	2.2 (Figure F4)	2.2 (Figure F7)
Permanent (no traffic loading)	Long Term (Drained)	1.5 (Figure F2)	1.7 (Figure F5)	1.6 (Figure F8)
Seismic	Pseudo-Static (Undrained)	1.4 (Figure F3)	1.7 (Figure F6)	1.8 (Figure F9)

Table 6.2 of the CHBDC for embankment fills with a typical degree of understanding and a Ψ of 1.0 generates minimum Factors of Safety of 1.5 and 1.3 for permanent and temporary conditions respectively**.** All of the static results presented in Table 11-6 meet or exceed the target Factors of Safety.

Table 6.3 in Section 6.14.4.1 of the CHBDC indicates a minimum seismic resistance factor of 0.95 for force-based design and 1.0 for performance-based design. Based on these values and Ψ of 1.0, a target Factor of Safety of 1.1 for this temporary condition with a typical degree of understanding is appropriate for the pseudo-static seismic analysis. The pseudo-static result presented in Table 11-6 above, exceeds the target Factor of Safety for seismic design. It is noted that some displacement of the embankment can occur where the pseudo-static Factor of Safety is less than 1.3. However, as noted in Table 11.7 above this criterion has also been satisfied for all cases.

An embankment fill slope of 2H:1V satisfies all of the static and pseudo-static slope stability requirements.

Slope protection and drainage measures will be required to ensure the long-term surficial stability of the embankment slopes, see Section 12.4.

11.6 Cement Type and Corrosion Potential

Chemical analysis for determination of pH, water soluble sulphate, sulphides, chloride concentrations, resistivity and electrical conductivity was carried out on samples of the native materials. The analysis results are summarized in Section 5.8 and a copy of the test results is provided in Appendix C.

The pH, resistivity and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. The test results provided in Table 5-8 were compared with Table 3.2 of the MTO Gravity Pipe Design Guideline and generally indicate a severe corrosive environment. The test results provided in Section 5.8 may be used to aid in the selection of coatings and corrosion protection systems for buried steel objects.

The concentration of soluble sulphate provides an indication of the degree of sulphate attack that is expected for concrete in contact with the soil and groundwater at the site. The sulphate results in Table 5-8 were compared with Table 3 of Canadian Standards Association Standards A23.1-14 (CSA A23.1) and generally indicate a low degree of sulphate attack potential on concrete structures at this site.

12 CONSTRUCTION CONSIDERATIONS

The recommendations presented herein must be reviewed once the type, location and orientation of the foundation elements are established to ensure suitability given the variations in stratigraphy and bedrock elevation at the site.

12.1 Excavations

It is anticipated that temporary excavations up to 6 m below the existing top of roadway will be required to allow the removal of the existing eastbound culverts and installation of the new structure. Excavations for the new westbound structure after preloading are expected to a similar depth.

Excavation should be carried out in accordance OPSS 902, SP No. 109S12 and NSSP FOUN0003.

All excavations must be conducted in accordance with the requirements of the Occupational Health & Safety Act & Regulations (OHSA) for Construction Projects. The fills and native soils at the site shall be classified as Type 3 in accordance with OHSA. Unsupported excavations made in Type 3 soils must have side slopes no steeper than 1H:1V from the base of the excavation.

The management and disposal of excess material shall be in accordance with OPSS.PROV 180.

Selection of the equipment and methodology to excavate and prepare the founding surface is the responsibility of the Contractor. In addition, the Contractor must plan the work appropriately to ensure stable work platforms for equipment.

At locations where there are space restriction or where a slope has to be retained, the excavation will need to be carried out within a protection system.

12.2 Temporary Protection System

Should Temporary Protection Systems (TPS) be required for excavation support or groundwater control, they must be implemented in accordance with OPSS.PROV 539 and designed for Performance Level 2 (maximum 25 mm horizontal deflection). The actual pressure distribution acting on the shoring system is a function of the construction sequence and the relative flexibility of the wall and these factors must be considered when designing the shoring system. The protection system should be installed at a suitable distance away from the new culvert or foundation element to limit the disturbance to subgrade associated with removal of the protection system following completing of construction. Alternatively, the protection system could be left in place and cut off as per OPSS 903. The use of vibratory equipment should not be precluded at this site for installation or removal of the temporary protection systems.

Lateral earth pressure coefficients, under fully mobilized conditions, that can be used in design of the protection system installed through embankment fill and culvert backfill are provided in Section 11.4. Suggested lateral earth pressure coefficients for the existing clay deposit are given below:

γ	=	17 kN/m^3	(must be adjusted for water table)
K_A	=	0.36	
K_P	=	2.77	
Su	=	50 kPa	

Suggested lateral earth pressure coefficients for the existing glacial till deposit are given below:

γ	=	21 kN/m^3	(must be adjusted for water table)
K_A	=	0.27	
K_P	=	3.69	

The suggested values provided are for a horizontal backslope behind, and a horizontal surface in front of the protection system. If the backslope behind or if the ground surface in front of the temporary protection systems are not horizontal, the lateral earth pressure parameters provided above do not apply and recalculation of the earth pressure parameters will be required.

The design of protection systems is the responsibility of the Contractor and should be designed by a licensed Professional Engineer experienced in such designs and retained by the Contractor. The designer of the temporary protection system must ensure the penetration depth is sufficient to provide base fixity and incorporate traffic loading and surcharge loading due to construction equipment and their operations and shall consider the slope of temporary embankments above the top of the protection system and location of existing utilities and trenches.

The use of sheet piles is not considered feasible due to potential obstructions such as cobbles and boulders in the till, artesian pressures noted in the till and the limited thickness of native soils that may not provide sufficient depth to achieve lateral stability. In addition, there will be high lateral earth pressures associated with the embankment (retained heights of up to 6 m); tie back anchors consisting of soil anchors installed within the till or rock may be required to maintain stability. The use of deadman anchor blocks or internal bracing could also be considered.

A soldier pile and lagging system is a feasible option. It may be necessary to predrill for the soldier piles. Lateral support may need to be enhanced by socketing the soldier piles into bedrock and/or by using bracing or rakers. Suggested wording for an NSSP for obstructions is included in Appendix J.

12.3 Surface and Groundwater Control

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

The design of dewatering systems is the responsibility of the Contractor. The Contract Documents must alert the Contractor to this responsibility and to design the dewatering systems in accordance with SP FOUN0003 which amends OPSS 902. As buildings are present near the new westbound structure, a preconstruction survey is recommended, thus Designer Fill-In ** in SP FOUN0003 should be “100m”.

The water level will fluctuate and the minimum groundwater elevation for the site at the time of the excavation should be taken as the expected high water level defined in SP517F01 and SP FOUN0003. Given the presence of artesian groundwater conditions it is recommended that the dewatering system design engineer requirement be invoked in SP517F01. Excavation base instability due to artesian conditions must be considered for the site. In addition, the potential for bottom heave due to the presence of cohesive soils needs to be assessed.

A sheetpile cofferdam enclosure might be difficult to install at this site. Alternative dewatering methods such as a sandbag cofferdam at the inlet and outlet with sump pumps to extract water from the excavation are likely sufficient. If required, a soldier piled enclosure can be designed following the recommendations provided in Section 12.2.

Excavations that extend below the groundwater level without prior dewatering are not recommended since the inflow of groundwater will make it difficult to maintain a dry, sound base on which to work. Disturbance of the subgrade soils is considered to be a risk without proper consideration of groundwater lowering. The groundwater level should be lowered to 0.5 m below the planned base of excavation for each stage of excavation.

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

It is noted that a Hydrogeological Investigation and Design Report is under preparation for the Highway 17 Twinning Project. Please refer to that document for additional discussion on dewatering with respect to this assignment.

12.4 Erosion Protection

The Contractor shall provide silt fences and erosion control blankets, as required, throughout the duration of the construction to prevent silt/sediments from running off the site as per OPSS 805.

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

Particle size analysis on samples of the existing embankment materials indicate that the soils have a low to moderate potential for soil erodibility (Wischmeier Nomograph Factor, K of less than 0.25).

Culverts

Erosion protection shall be provided at culvert inlet and outlet areas. Effective scour and erosion protection should be provided along the waterline, ditches and around footings. Design of the erosion protection measures must consider hydrologic and hydraulic factors and shall be carried out by specialists experienced in this field. Typically, rock protection should be provided over all surfaces with which creek water is likely to be in contact. Treatment at the inlet and outlet of culverts shall be in accordance with OPSD 810.010.

It is recommended that a clay seal be used to minimize the potential for erosion near culvert inlets. The clay seal shall extend a minimum of 0.3 m above the high-water level and laterally for the width of the granular material, and have a minimum thickness of 0.5 m. The material requirements shall be in accordance with OPSS.PROV 1205. A geosynthetic clay liner may also be considered.

Liaison between the Foundations Consultant, Structural Engineer and Hydraulic/Drainage Engineer will be required in design to ensure that scour protections is adequately addressed.

Bridges

Scour protection should be reviewed once bridge design is selected and foundation types determined. Where shallow foundations are employed they should be provided with scour protection appropriate to the hydraulic regime.

Liaison between the Foundations Consultant, Structural Engineer and Hydraulic/Drainage Engineer will be required in design to ensure that scour protections is adequately addressed.

13 CONSTRUCTION CONCERNS

The recommendations presented herein must be reassessed once the type, location and orientation of the foundation elements are established to ensure suitability given the variations in stratigraphy and bedrock elevation at the site.

The planned construction methodology includes open cut excavations for the installation of two new structures. Potential construction concerns include, but are not necessarily limited to the following:

- Construction will extend below the water level in the creek. An adequate and effective surface water management and dewatering plan must be implemented to construct the foundations in the dry.
- Artesian conditions were encountered in the glacial till layer beneath the clay unit. Excavation base instability due to artesian conditions must be considered for the site. While advancing drilled piles/caissons, the contractor will need to maintain proper head in the drilled shaft. This could be mitigated by increasing the casing height above the ground surface or by use of dewatering wells in the excavation area.
- The clay which will be exposed beneath a culvert bedding layer or wing wall spread footings is sensitive and readily disturbed. A Notice to Contractor is provided in Appendix J.
- The Contractor’s selection of construction equipment and methodology must include assessment of the capability of the existing soils to support the proposed construction equipment and supplies.
- Mitigation of the settlement induced by the new westbound embankment will require a preload or a structure designed to accommodate the movements. An instrumentation and monitoring program will need to be implemented to assess the progress of the preload and observe impacts on adjacent structures (i.e. existing culverts under the existing Highway 17 embankment). Given the limited project length, the monitoring program would include approximately six settlement rods located on the new alignment with a nominal spacing of 25 m. The base plates should be installed prior to fill placement and the rods will require extension as fill is placed around them. The top of the settlement rods should be surveyed every week during preload construction and for the anticipated two month preload period. The installation of the monitoring equipment and surveying would typically be carried out by the Contractor, with the results evaluated by the Contract Administration team.

THURBER

The geotechnical recommendations provided herein are based on the subsurface data provided in Part 1 of this report. Design-build Contractors or other Consultants should consider drilling additional boreholes to meet MTO requirements for bridge foundations.

The successful performance of this structure will depend largely upon good workmanship and quality control during construction. Observation of the excavation and backfilling operations will be required as per OPSS 902 during construction to confirm that the foundation recommendations are correctly implemented, and material specifications are met.

14 CLOSURE

Engineering analysis and preparation of this report was carried out by Justin Gray, P.Eng., Deanna Pizycki P.Eng. and by Dr. Fred Griffiths, P.Eng. The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

Thurber Engineering Ltd.
Report Prepared By:

Justin Gray, P.Eng.
Geotechnical Engineer

Deanna Pizycki, P.Eng.
Geotechnical Engineer

Dr. Fred Griffiths, P.Eng.
Senior Associate

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

THURBER

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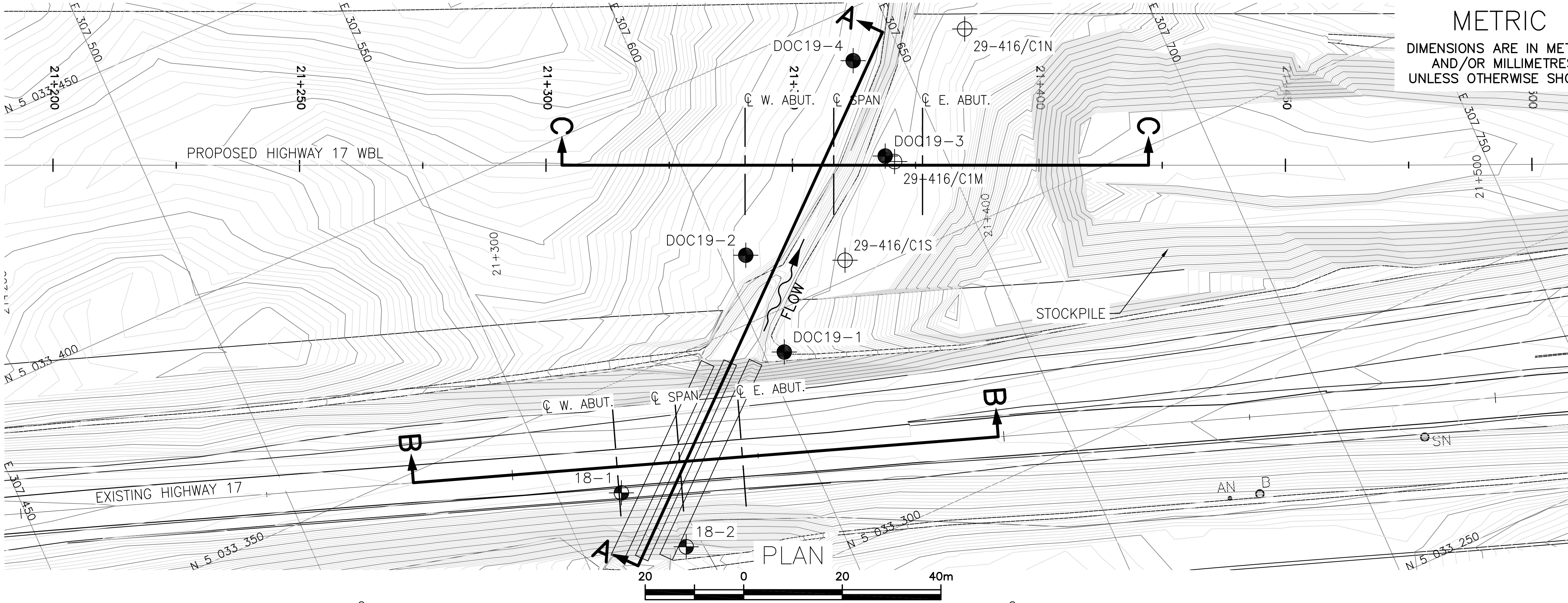
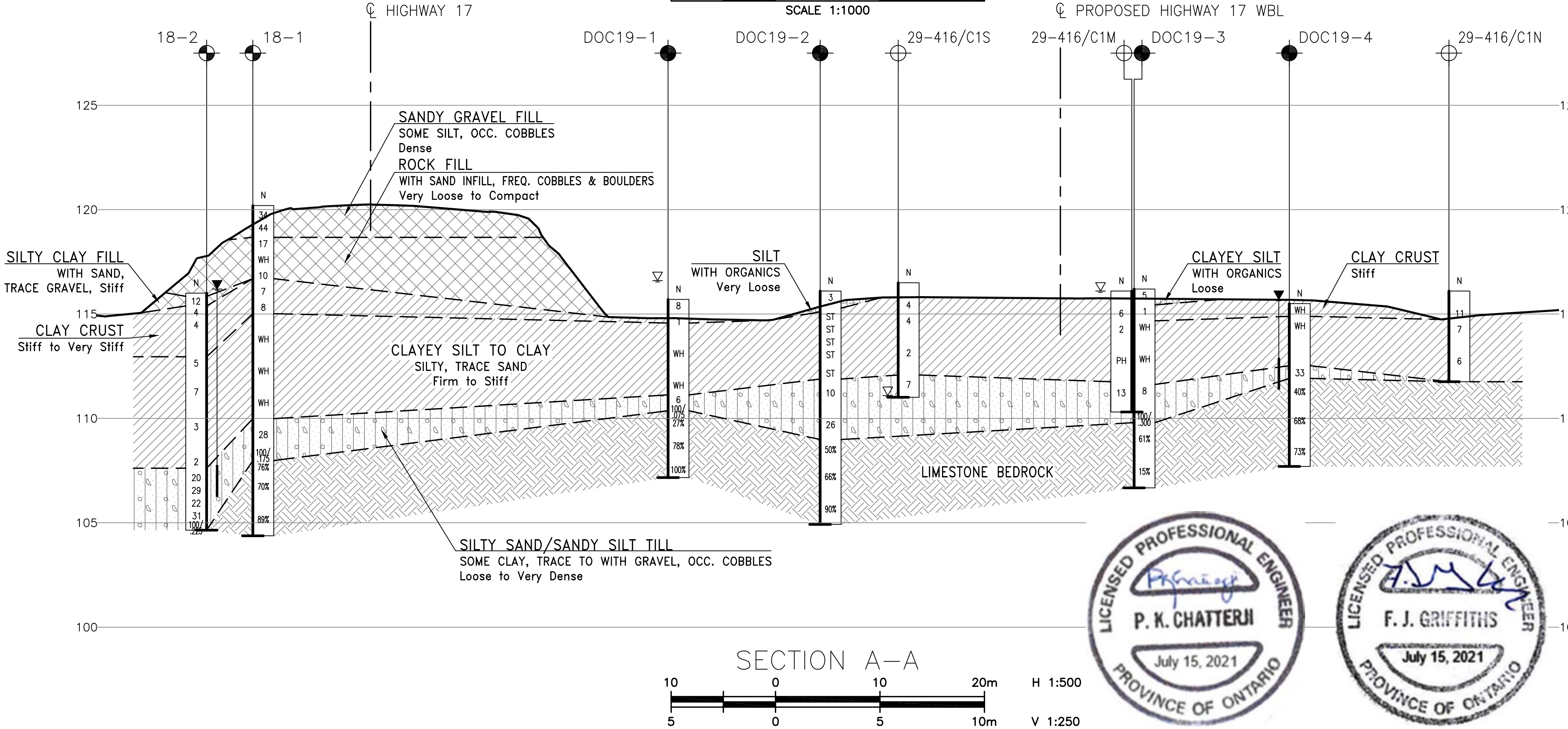
Appendix A.

Borehole Location Plan and Stratigraphic Drawings

MINISTRY OF TRANSPORTATION, ONTARIO





LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
July 15, 2021
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
F. J. GRIFFITHS
July 15, 2021
PROVINCE OF ONTARIO

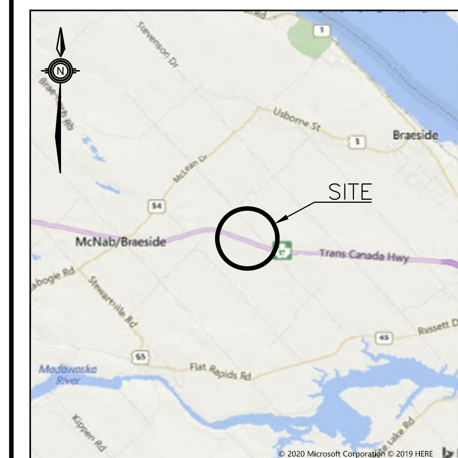
CONT No
WP No 4068–09–00

SHEET

HIGHWAY 17 TWINNING
DOCHART CREEK
CULVERTS
BOREHOLE LOCATIONS AND SOIL STRATA

Ontario

THURBER ENGINEERING LTD.



KEYPLAN

LEGEND

	Borehole (2019 Investigation)
	Borehole (2018 Investigation)
	Borehole (2005 Investigation)
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
18–1	120.2	5 033 329.1	307 562.1
18–2	116.0	5 033 313.8	307 569.7
29–416/C1M	116.1	5 033 368.0	307 640.0
29–416/C1N	116.1	5 033 386.8	307 663.9
29–416/C1S	116.5	5 033 353.8	307 622.7
DOC19–1	115.7	5 033 341.8	307 603.9
DOC19–2	116.1	5 033 362.9	307 604.7
DOC19–3	116.2	5 033 369.8	307 638.7
DOC19–4	115.5	5 033 390.1	307 640.6

-NOTES-

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

GEOCRES No. 31F-21

REVISIONS	DATE	BY	DESCRIPTION
DESIGN JG	CHK –	CODE	LOAD	DATE JUL 2021
DRAWN MFA	CHK JG	SITE	STRUCT	DWG 1

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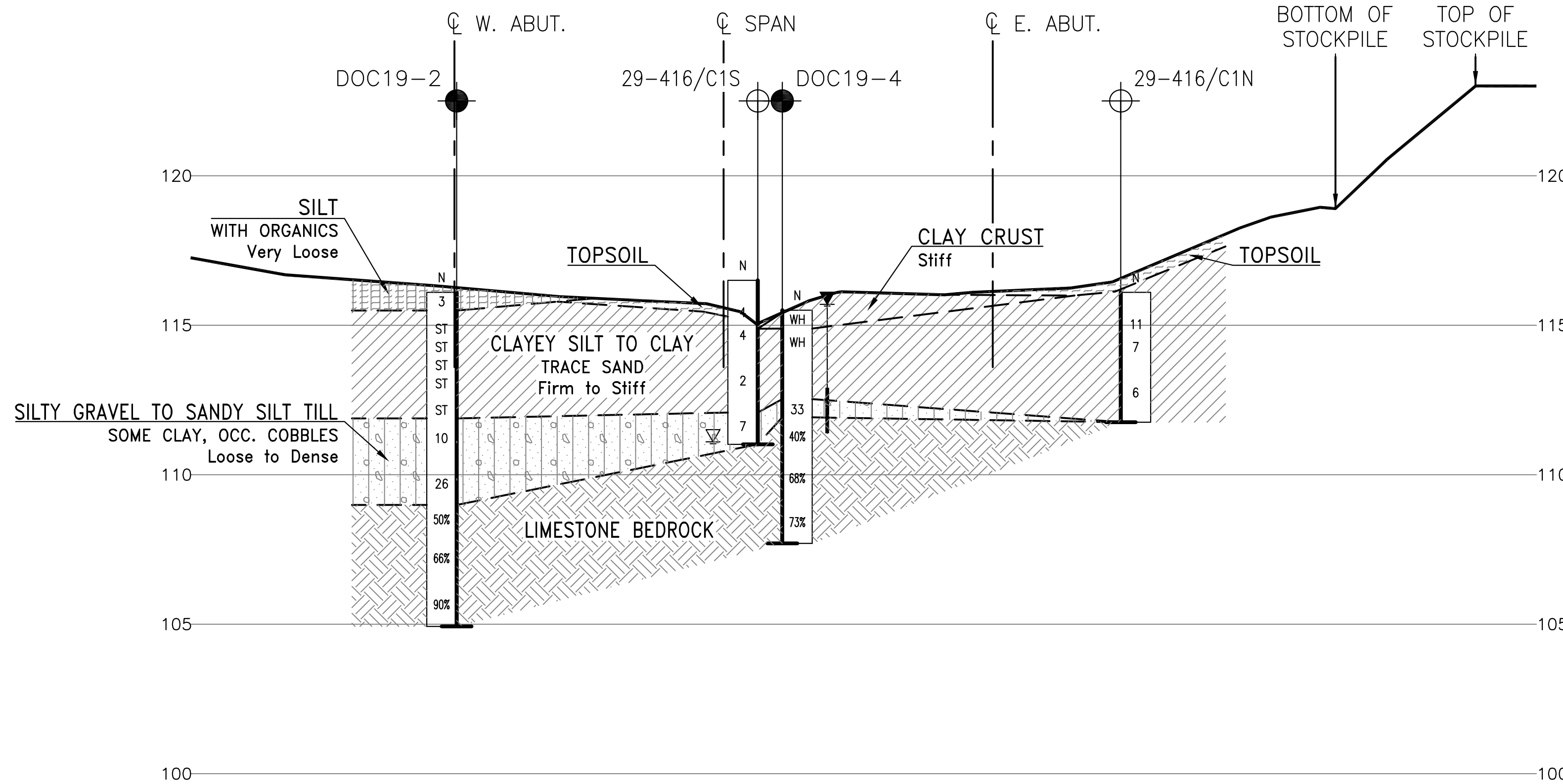
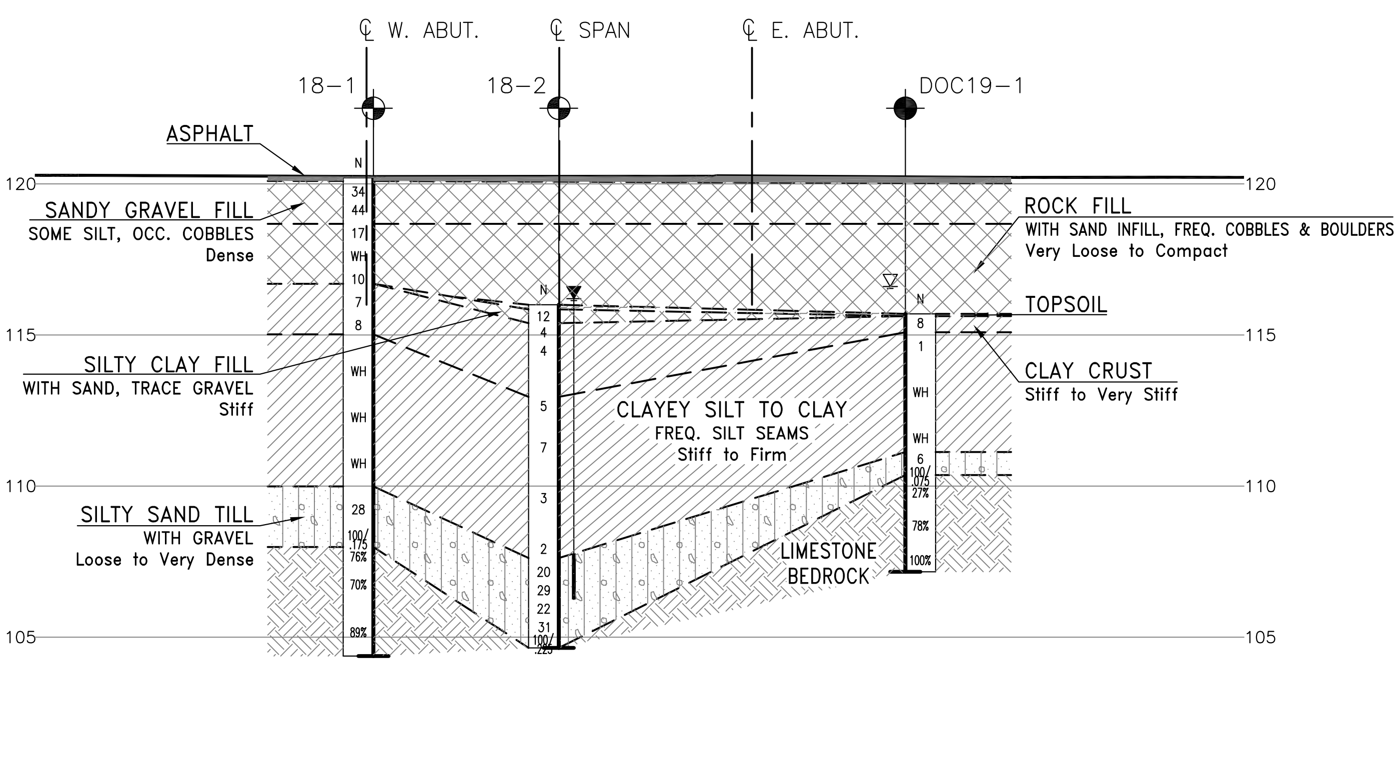
PLOTDATE: 7/15/2021 10:57 AM

MINISTRY OF TRANSPORTATION, ONTARIO



SECTION C–C (PROPOSED HIGHWAY 17 WBL)

H 1:500 V 1:250



SECTION B–B (EXISTING HIGHWAY 17 PROPOSED EBL)

H 1:500 V 1:250

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

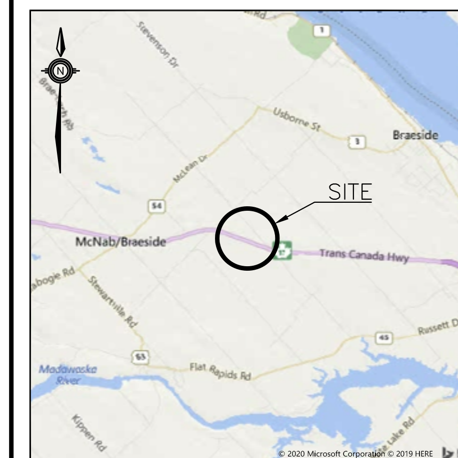
CONT No
WP No 4068–09–00

SHEET

HIGHWAY 17 TWINNING
DOCHART CREEK
CULVERTS
BOREHOLE LOCATIONS AND SOIL STRATA

Ontario

THURBER ENGINEERING LTD.



KEYPLAN

LEGEND

	Borehole (2019 Investigation)
	Borehole (2018 Investigation)
	Borehole (2005 Investigation)
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
18–1	120.2	5 033 329.1	307 562.1
18–2	116.0	5 033 313.8	307 569.7
29–416/C1M	116.1	5 033 368.0	307 640.0
29–416/C1N	116.1	5 033 362.9	307 663.9
29–416/C1S	116.5	5 033 353.8	307 622.7
DOC19–1	115.7	5 033 341.8	307 603.9
DOC19–2	116.1	5 033 362.9	307 604.7
DOC19–3	116.2	5 033 369.8	307 638.7
DOC19–4	115.5	5 033 390.1	307 640.6

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-NOTES-

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

GEOCRES No. 31F-21

REVISIONS	DATE	BY	DESCRIPTION

DESIGN JG	CHK –	CODE	LOAD	DATE JUL 2021
DRAWN MFA	CHK JG	SITE	STRUCT	DWG 2

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PLOTDATE: 7/15/2021 10:58 AM

Appendix B.

Record of Borehole Sheets

THURBER

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.

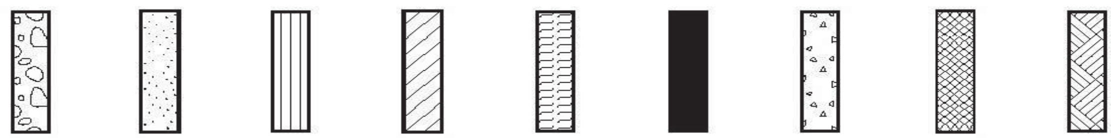
N-VALUE:

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

THURBER

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

THURBER

EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION

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

TERMS

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

DISCONTINUITY SPACING

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

STRENGTH CLASSIFICATION

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No DOC19-1

METRIC

WP#	4068-09-00	LOCATION	Lat: 45.440002°, Long: -76.46416° Dochart Creek, MTM zone 9: N 5 033 341.8 E 307 603.9	ORIGINATED BY	MJ
HWY	17	BOREHOLE TYPE	CME45 Track Mount, HSA, NQ Coring	COMPILED BY	JG
DATUM	Geodetic	DATE	2019.10.21 - 2019.10.21	CHECKED BY	FG

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	SHEAR STRENGTH kPa (Field Vane)	WATER CONTENT (%)	UNIT WEIGHT γ kN/m3 / FI	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
115.7	Ground Surface							
0.0 / 0.1	**100 mm TOPSOIL**							
	CLAY (CI) crust Stiff Brown-Grey	1	SS	8				
115.1 / 0.6	**CLAY (CI)** Stiff Grey	2	SS	1				
					6.5, 10.7			
		3	SS	WH				0 1 50 49
					15.0, 7.6			
		4	SS	WH				
111.4 / 4.3	Silty **SAND** with gravel Loose Grey **(TILL)**	5	SS	6				17 49 28 6 Non-Plastic
110.4 / 5.3	**LIMESTONE BEDROCK** Slightly Weathered Foliated Grey	6	SS	100/ 75mm			FI <10	
		1	RUN				4, 1, 1	RUN #1 TCR=100% SCR=92% RQD=27%
		2	RUN				2, 1, 3, 2	RUN #2 TCR=100% SCR=95% RQD=78%
107.2 / 8.5		3	RUN				1, 2, 1	RUN #3 TCR=100% SCR=100% RQD=100%

End of Borehole

1.0 m stickup on well

WATER LEVEL READINGS:

DATE	DEPTH (m)	ELEV. (m)
2019.11.26	0.9 above g.s.	116.6
2020.04.21	>1.3 above g.s.	>117.0
2020.04.28	>1.1 above g.s.	>116.8
2020.05.04	1.5 above g.s.	117.2

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; ELEVATION SCALE 115, 114, 113, 112, 111, 110, 109, 108.

+ 3, × 3: Numbers refer to Sensitivity

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No DOC19-2

METRIC

WP#	4068-09-00	LOCATION	Lat: 45.440193°, Long: -76.46415° Dochart Creek, MTM zone 9: N 5 033 362.9 E 307 604.7	ORIGINATED BY	MJ
HWY	17	BOREHOLE TYPE	CME45 Track Mount, HSA, NQ Coring	COMPILED BY	JG
DATUM	Geodetic	DATE	2019.10.18 - 2019.10.18	CHECKED BY	FG

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	UNIT WEIGHT / FI	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
116.1	Ground Surface									
0.0	**200 mm Topsoil**				116					
0.2	**CLAY (CI)** stiff brown to grey	1	SS	3						
		1	ST		115	e_o=1.562 γ=16.5 P'_c=200kPa	0	1	43	56
		2	ST			e_o=1.527 γ=16.5 P'_c=140kPa	0	1	46	53
		3	ST		114	e_o=1.400 γ=17.1 P'_c=230kPa	0	0	48	52
		4	ST		113	e_o=1.561 γ=16.6 P'_c=200kPa	0	2	52	46
		5	ST		112					
111.9 / 4.2	**GRAVEL**, silty with sand Compact Grey **(TILL)**	2	SS	10	111					
		3	SS	26	110		49	37	14 (SI+CL)	
109.0 / 7.1	**LIMESTONE BEDROCK** Slightly Weathered Foliated Grey	1	RUN		109	FI: <10, 4, 3	RUN #1 TCR=100% SCR=74% RQD=50%			
		2	RUN		108, 107	2, 3, 1, 2, 2, 2	RUN #2 TCR=100% SCR=94% RQD=66%			

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

Continued Next Page

+ ³, × ³: Numbers refer to Sensitivity

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

DOUBLE LINE 24726 DOCHART CREEK.GPJ 2012TEMPLATE(MTO).GDT 8/7/21

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No DOC19-2

METRIC

WP#	4068-09-00	LOCATION	Lat: 45.440193°, Long: -76.46415° Dochart Creek, MTM zone 9: N 5 033 362.9 E 307 604.7	ORIGINATED BY	MJ
HWY	17	BOREHOLE TYPE	CME45 Track Mount, HSA, NQ Coring	COMPILED BY	JG
DATUM	Geodetic	DATE	2019.10.18 - 2019.10.18	CHECKED BY	FG

ELEV DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100)	PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page										
	LIMESTONE BEDROCK Slightly Weathered Foliated Grey		3	RUN			106			2	RUN #3 TCR=100% SCR=100% RQD=90%
										3	
										3	
104.9							105			1	
11.2	End of Borehole										

DOUBLE LINE 24726 DOCHART CREEK.GPJ 2012TEMPLATE(MTO).GDT 8/7/21

+ 3, × 3: Numbers refer to Sensitivity

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No DOC19-3

METRIC

WP#	4068-09-00	ORIGINATED BY	MJ
LOCATION	Lat: 45.440254°, Long: -76.463715° Dochart Creek, MTM zone 9: N 5 033 369.8 E 307 638.7		
HWY	17	COMPILED BY	JG
BOREHOLE TYPE	CME45 Track Mount, HSA, NQ Coring		
DATUM	Geodetic	CHECKED BY	FG
DATE	2019.10.21 - 2019.10.21		

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	SHEAR STRENGTH kPa (FIELD VANE, sensitivity)	UNIT WEIGHT γ kN/m³ / FI	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
116.2	Ground Surface						
0.0 / 115.9	**300 mm Topsoil**	1	SS	5			
0.3	**CLAY (CI) crust** Stiff Brown	2	SS	1			0 3 47 50
114.7 / 1.5	**CLAY (CI)** Firm to Stiff Grey	3	SS	WH	13.0, 8.0		
		4	SS	WH	5.5, 6.0		
111.6 / 4.6	Silty **SAND** with gravel Loose Grey **(TILL)** -poor sample recovery	5	SS	8			
		6	SS	100/300mm		FI	
109.8 / 6.4	**LIMESTONE BEDROCK** Slightly Weathered Foliated Grey Fair to poor quality -vugs from 6.8 m to 7.2 m -vugs from 7.9 m to 8.3 m	1	RUN			3, 5, 3, 2, 5	RUN #1 TCR=95% SCR=90% RQD=61%
	-voids/broken rock from 9.1 m to 9.3 m - 50 mm clay seam at 9.3 m	2	RUN			5, 5, 5, 4, <10	RUN #2 TCR=79% SCR=65% RQD=15%
106.7 / 9.5	End of Borehole						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

ELEVATION SCALE: 116, 115, 114, 113, 112, 111, 110, 109, 108, 107

+³, ×³: Numbers refer to Sensitivity

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

DOUBLE LINE 24726 DOCHART CREEK.GPJ 2012TEMPLATE(MTO).GDT 8/7/21

RECORD OF BOREHOLE No DOC19-4

METRIC

WP#	4068-09-00	LOCATION	Lat: 45.440438°, Long: -76.46369° Dochart Creek, MTM zone 9: N 5 033 390.1 E 307 640.6	ORIGINATED BY	MJ
HWY	17	BOREHOLE TYPE	CME45 Track Mount, HSA, NQ Coring	COMPILED BY	JG
DATUM	Geodetic	DATE	2019.10.18 - 2019.10.18	CHECKED BY	FG

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa (FIELD VANE)	WATER CONTENT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
115.5	Ground Surface								
0.0 / 0.1	**100 mm TOPSOIL**								
114.9 / 0.6	**CLAY (CI) crust** Stiff Brown	1	SS	WH	115				
112.5 / 3.0	**CLAY (CI)** Firm to Stiff Grey	2	SS	WH	114, 113	9.3, 7.0, 8.0, 4.8			0 2 43 55
111.9 / 3.6	**GRAVEL**, silty with sand Dense Grey **(TILL)**	3	SS	33	112				
107.7 / 7.8	**LIMESTONE BEDROCK** Slightly Weathered to Fresh Grey to Grey-Black Poor to fair quality Broken from 3.6 m to 3.7 m	1	RUN		111			FI: <10, 6, 4, 4, 2	RUN #1 TCR=68% SCR=62% RQD=40%
		2	RUN		110			2, 1, 1, 2	RUN #2 TCR=100% SCR=97% RQD=68%
		3	RUN		109, 108			1, 2, 2, 4, 1	RUN #3 TCR=98% SCR=95% RQD=73%

End of Borehole

1.0 m stickup on well

WATER LEVEL READINGS:

DATE	DEPTH (m)	ELEV. (m)
2019.11.26	0.2 above g.s.	115.7
2020.04.21	0.3 above g.s.	115.8
2020.04.28	0.1 above g.s.	115.6
2020.05.04	0.4 above g.s.	115.9

+ 3, × 3: Numbers refer to Sensitivity

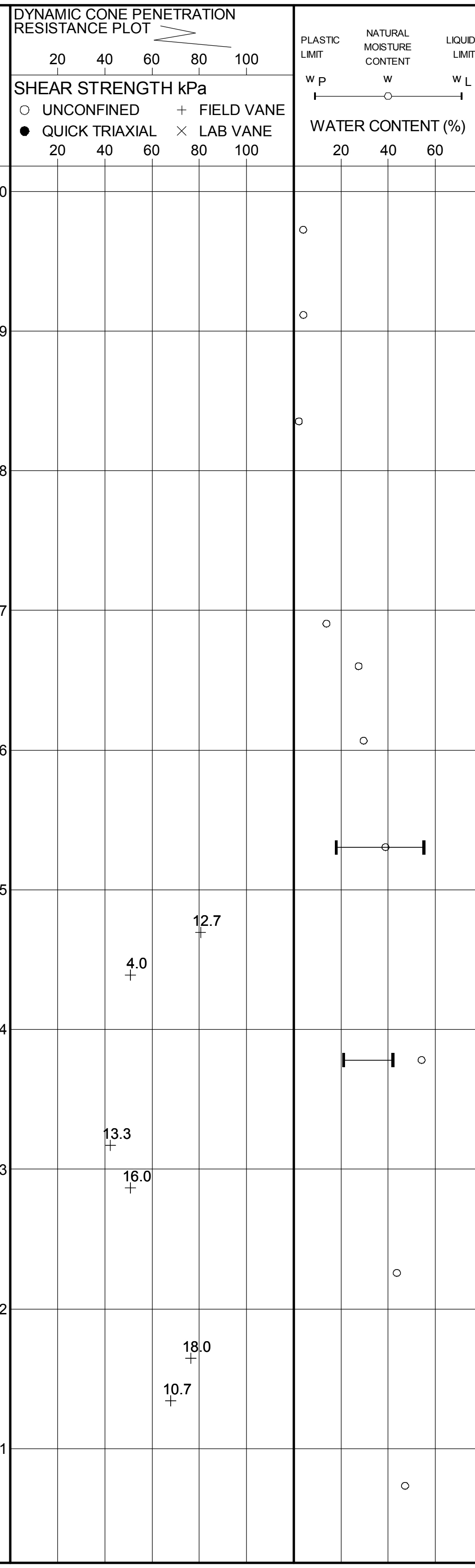
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 18-1

METRIC

GWP# 4076-13-00	LOCATION Lat: 45.4398887°, Long: -76.4646941° MTM Zone 9: N 5 033 329.1 E 307 562.1	ORIGINATED BY CM
HWY 17	BOREHOLE TYPE HSA / HW casing	COMPILED BY CM
DATUM Geodetic	DATE 2018.06.06 - 2018.06.07	CHECKED BY KP

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
120.2					
0.0	**110 mm ASPHALT**				
0.1	Sandy gravel some silt, occasional cobbles Dense Grey Dry **FILL**	1	SS	34	
		2	SS	44	47 45 8 (SI+CL)
118.7 1.5	Rock fill with sand infill - frequent cobbles and boulders Very loose to compact Brown **ROCK FILL** - borehole advanced by tri-cone from 1.5 m to 3.5 m	3	SS	17	
		4	SS	WH	
		5	SS	10	
116.7 3.5	**CLAY (CH) Crust** - Slight mottling Very Stiff Grey to brown	6	SS	7	
		7	SS	8	0 8 43 49
115.0 5.2	**CLAY (CI)** Stiff to firm Grey				
		8	SS	WH	0 0 48 52
	- frequent silt seams below 7.6 m	9	SS	WH	
		10	SS	WH	



Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 18-1

METRIC

GWP#	4076-13-00	LOCATION	Lat: 45.4398887°, Long: -76.4646941° MTM Zone 9: N 5 033 329.1 E 307 562.1	ORIGINATED BY	CM
HWY	17	BOREHOLE TYPE	HSA / HW casing	COMPILED BY	CM
DATUM	Geodetic	DATE	2018.06.06 - 2018.06.07	CHECKED BY	KP

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

Continued From Previous Page

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH	WATER CONTENT	REMARKS / GR SA SI CL
110.0					110	11.2 (+)		
10.2	**SILTY SAND (SM) with Gravel TILL** Compact to very dense Grey							
		11	SS	28	109		○	17 47 30 6 Non-plastic
108.0		12	SS	100/ 175mm	108		○	
12.2	**LIMESTONE BEDROCK** Slightly weathered to fresh Close joint spacing Fair to good quality Grey to black	1	RUN					RUN #1 TCR=100% SCR=100% RQD=76%
		2	RUN		107			RUN #2 TCR=91% SCR=81% RQD=70%
		3	RUN		106 / 105			RUN #3 TCR=100% SCR=98% RQD=89%
104.4 / 15.8	End of Borehole							

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

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 18-2

METRIC

GWP# 4076-13-00
LOCATION Lat: 45.4397509°, Long: -76.464597° MTM Zone 9: N 5 033 313.8 E 307 569.7
ORIGINATED BY SOB

HWY 17
BOREHOLE TYPE Portable - full weight hammer
COMPILED BY CM

DATUM Geodetic
DATE 2018.06.06 - 2018.06.06
CHECKED BY KP

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
116.0						
0.0	**150 mm TOPSOIL**					
0.2	Silty clay with sand, trace gravel and rootlets and wood pieces Stiff Grey **FILL**	1	SS	12		
115.4 / 0.6	**CLAY (CH) Crust** - Slight mottling Very Stiff Grey	2	SS	4	115	0 2 36 62
		3	SS	4	114	
113.0 / 3.0	**CLAY (CL)** Stiff to firm Grey	4	SS	5	113 / 112	
		5	SS	7	111 / 110	
		6	SS	3	109	
		7	SS	2	108	2 16 44 38
107.6 / 8.4	**SILTY SAND (SM) with Gravel TILL** Compact to very dense Grey	8	SS	20	107	
		9	SS	29	106	26 45 29 (SI+CL)

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

Field vane sensitivity values: 20.8, 10.1, 20.2, 15.1, 13.4

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m3

Continued Next Page

+3, ×3: Numbers refer to Sensitivity

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

DOUBLE_LINE DOCHART CREEK CULVERT.GPJ 2012TEMPLATE(MTO).GDT 6/7/18

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 18-2

METRIC

GWP# 4076-13-00

LOCATION Lat: 45.4397509°, Long: -76.464597° MTM Zone 9: N 5 033 313.8 E 307 569.7

ORIGINATED BY SOB

HWY 17

BOREHOLE TYPE Portable - full weight hammer

COMPILED BY CM

DATUM Geodetic

DATE 2018.06.06 - 2018.06.06

CHECKED BY KP

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE) 20 40 60 80 100	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page										
			10	SS	22				○		
			11	SS	31				○		18 50 28 4 Non-plastic
							105				
104.6			12	SS	100/ 225mm				○		
11.4	End of Borehole on Inferred Bedrock Groundwater measured at least 0.6 m above existing grade or elevation 116.6 m on 2018-06-26										

+ 3, × 3: Numbers refer to Sensitivity

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

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 29-416/C1N

METRIC

W.P.	647-92-01	LOCATION	N 5 033 386.8 E 307 663.9	ORIGINATED BY	SL
HWY	17/ 417	BOREHOLE TYPE	Hollow Stem Augers	COMPILED BY	JL/HS
DATUM	Geodetic	DATE	26.05.05 - 26.05.05	CHECKED BY	MRA

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
116.1							
0.0 / 115.8	**TOPSOIL** (250mm)					116	
0.3	Silty **CLAY**, trace sand Stiff to Firm Brown Moist						
		1	SS	11		115	
		2	SS	7		114	
	Becoming Grey	3	SS	6		113	
111.8 / 4.3						112	
	END OF BOREHOLE AT 4.34 m. AUGER REFUSAL AT 4.34 m ON PROBABLE BEDROCK OR BOULDERS. BOREHOLE OPEN AND DRY UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE.						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

Field vane at approx. elev. 113.8: 3.0

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

UNIT WEIGHT γ kN/m³

+ ³, × ³: Numbers refer to Sensitivity

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

ONTMT4S 5182CUVERTS.GPJ 04/10/05

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 29-416/C1M

METRIC

W.P.	647-92-01	LOCATION	N 5 033 368.0 E 307 640.0	ORIGINATED BY	SL
HWY	17/ 417	BOREHOLE TYPE	Hollow Stem Augers	COMPILED BY	JL/HS
DATUM	Geodetic	DATE	26.05.05 - 26.05.05	CHECKED BY	MRA

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	NATURAL MOISTURE CONTENT / LIMITS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
116.1						
0.0 / 115.8	**TOPSOIL** (250mm)					
0.3	Silty **CLAY**, trace sand Firm Brown Moist	1	SS	6		
	Becoming Grey	2	SS	2		0 2 47 51
		1	TW	PH		
111.8 / 4.3	Sandy **SILT**, some clay and gravel Compact Grey Wet (TILL)	3	SS	13		
110.3 / 5.8	END OF BOREHOLE AT 5.79 m. AUGER REFUSAL AT 5.79 m ON PROBABLE BEDROCK OR BOULDERS. Piezometer installation consists of 25 mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.					

WATER LEVEL READINGS:

DATE	DEPTH (m)
01/06/05	0.00

Chart headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa — O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20 40 60); UNIT WEIGHT γ kN/m^3; STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (116, 115, 114, 113, 112, 111).

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

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 29-416/C1S

METRIC

W.P. 647-92-01 | LOCATION N 5 033 353.8 E 307 622.7 | ORIGINATED BY SL

HWY 17/ 417 | BOREHOLE TYPE Hollow Stem Augers | COMPILED BY JL/HS

DATUM Geodetic | DATE 26.05.05 - 26.05.05 | CHECKED BY MRA

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
116.5							
0.0	**TOPSOIL** (200mm)						
0.2	Silty **CLAY** Firm Brown Moist					116	
		1	SS	4			
		2	SS	4		115	
						114	
	Becoming Grey	3	SS	2		113	0 2 48 50
112.1 / 4.4	Sandy **SILT**, some clay, trace gravel and cobbles Loose Grey Wet (TILL)					112	
		4	SS	7	▽		
111.0 / 5.5	END OF BOREHOLE AT 5.49 m. AUGER REFUSAL AT 5.49m. BOREHOLE OPEN TO 5.49 m AND WATER LEVEL AT 5.40 m UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE.						

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

+ ^{3}, × ^{3}: Numbers refer to Sensitivity   15-⊕-5 (20, 10) (%) STRAIN AT FAILURE

Appendix C.

Laboratory Testing

Appendix C.1

Particle Size Analysis Figures

Atterberg Limit Test Results

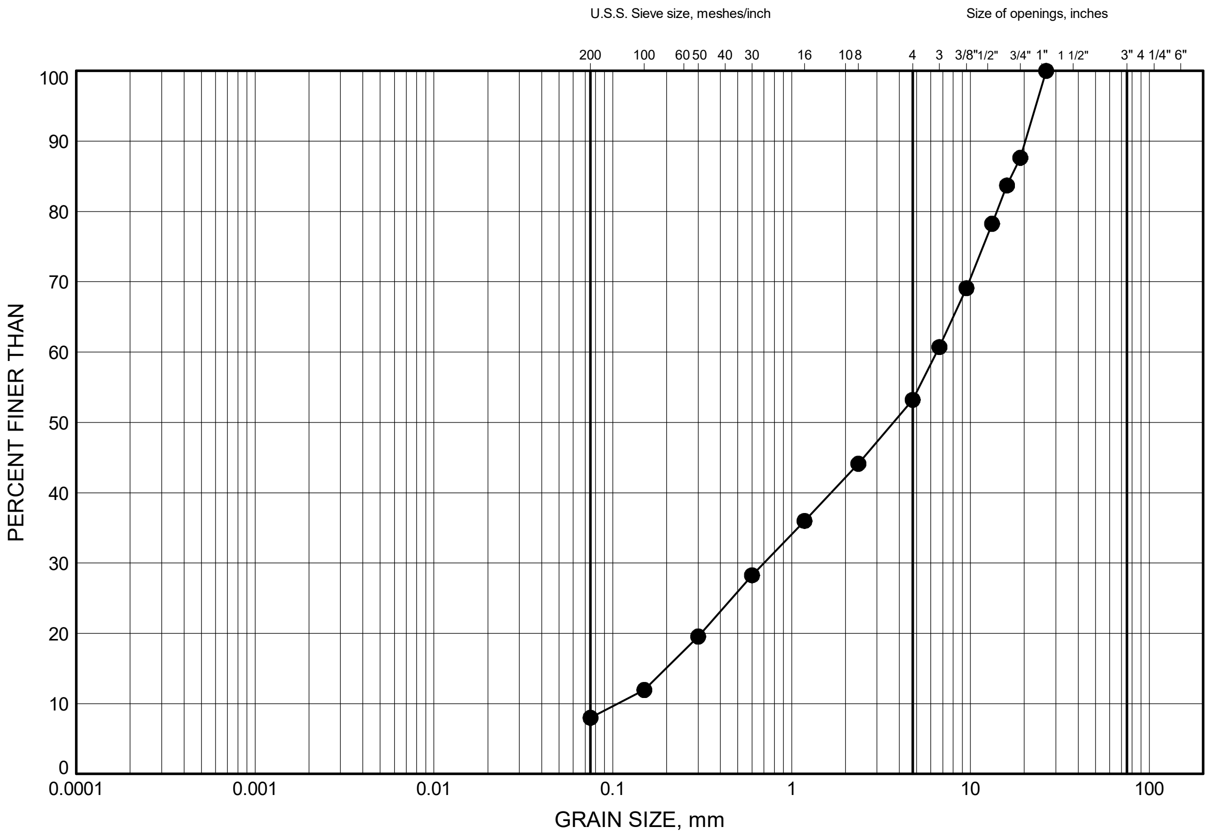
One-Dimensional Consolidation Test Results

Highway 17 Twinning

GRAIN SIZE DISTRIBUTION

FIGURE C1

Sandy Gravel Fill



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	18-1	1.1	119.1

GRAIN SIZE DISTRIBUTION - THURBER 24726 DOCHART CREEK.GPJ 9/7/21

Date July 2021

WP# 4068-09-00

THURBER

Prep'd DJP

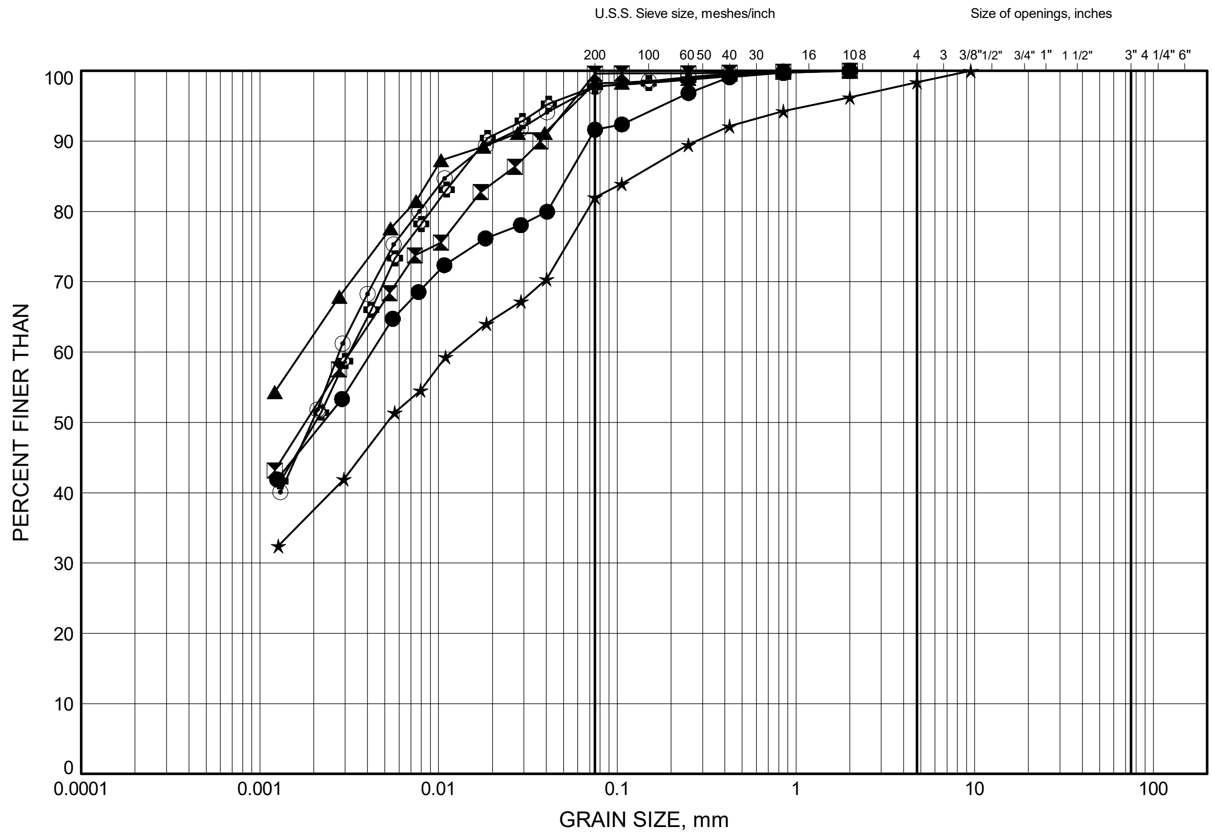
Chkd. FG

Highway 17 Twinning

GRAIN SIZE DISTRIBUTION

FIGURE C2

Clayey SILT to CLAY (CL to CH)



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	18-1	4.9	115.3
⧗	18-1	6.4	113.8
▲	18-2	0.9	115.1
★	18-2	8.1	107.9
⊙	29-416/C1M	1.8	114.3
✪	29-416/C1S	3.4	113.1

Date July 2021

WP# 4068-09-00

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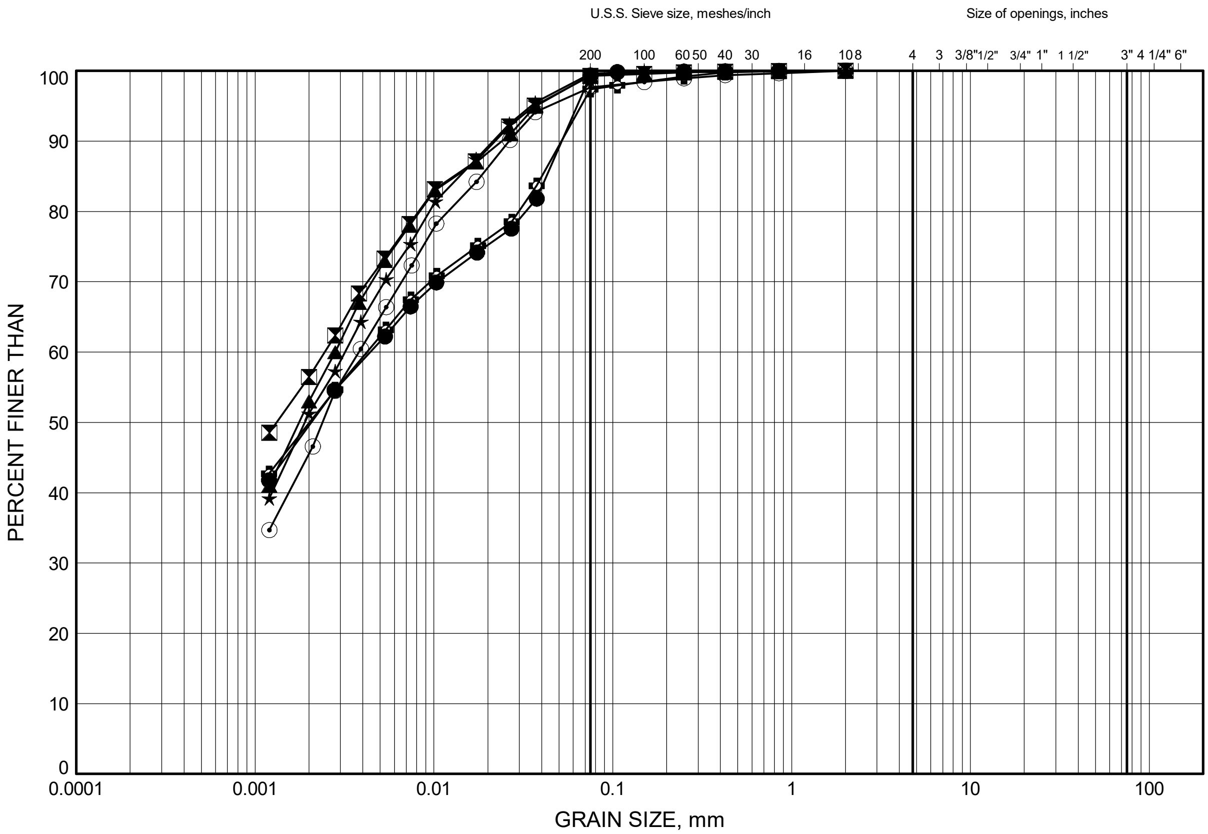
GRAIN SIZE DISTRIBUTION - THURBER 24726 DOCHART CREEK.GPJ 9/7/21

Highway 17 Twinning

GRAIN SIZE DISTRIBUTION

FIGURE C3

Clayey SILT to CLAY (CL to CH)



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	DOC19-1	2.6	113.1
⧗	DOC19-2	1.2	114.9
▲	DOC19-2	1.8	114.3
★	DOC19-2	2.4	113.7
⊙	DOC19-2	3.0	113.1
✿	DOC19-3	1.1	115.1

Date July 2021

WP# 4068-09-00

THURBER

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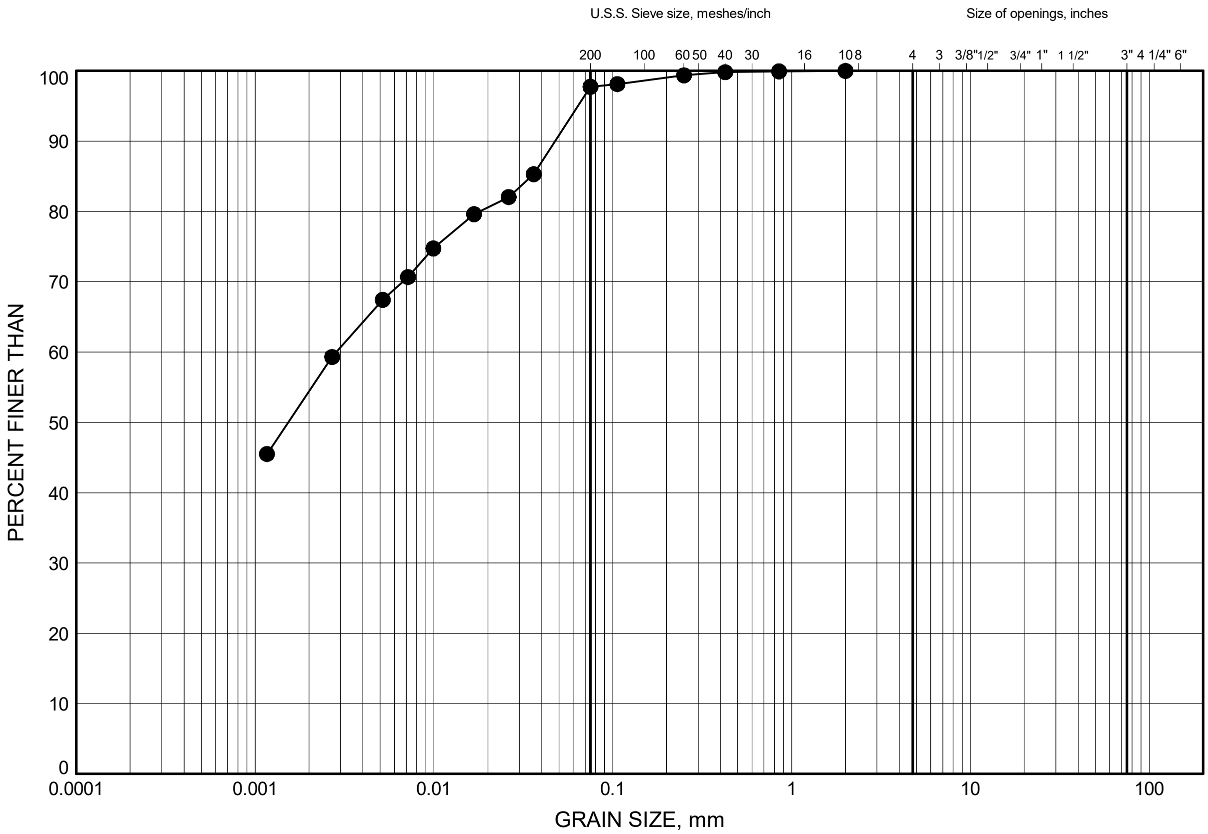
Highway 17 Twinning

GRAIN SIZE DISTRIBUTION

FIGURE C4

Clayey SILT to CLAY (CL to CH)



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	DOC19-4	1.1	114.4

Date July 2021

WP# 4068-09-00

THURBER

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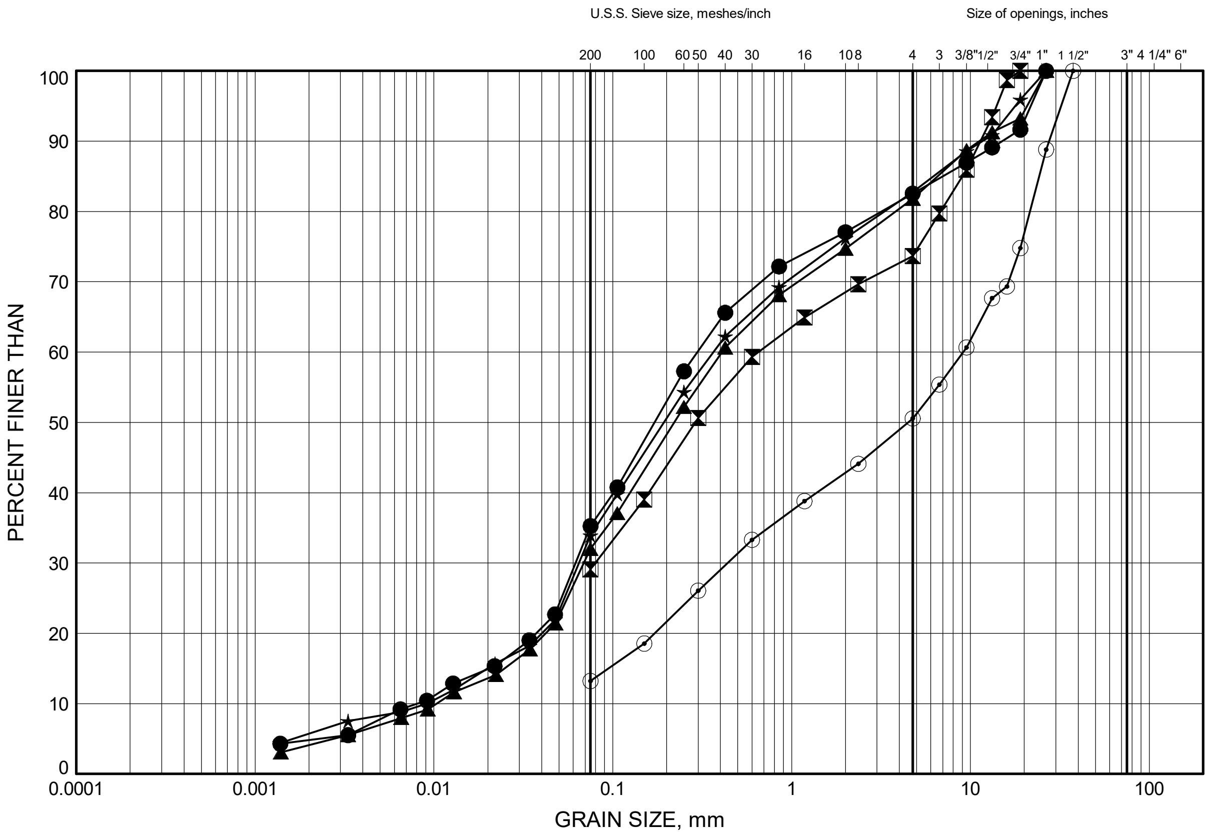
Highway 17 Twinning

GRAIN SIZE DISTRIBUTION

FIGURE C5

Glacial Till



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	18-1	11.0	109.2
⧗	18-2	9.4	106.6
▲	18-2	10.7	105.3
★	DOC19-1	4.9	110.8
⊙	DOC19-2	6.4	109.7

GRAIN SIZE DISTRIBUTION - THURBER 24726 DOCHART CREEK.GPJ 9/7/21

Date July 2021

WP# 4068-09-00

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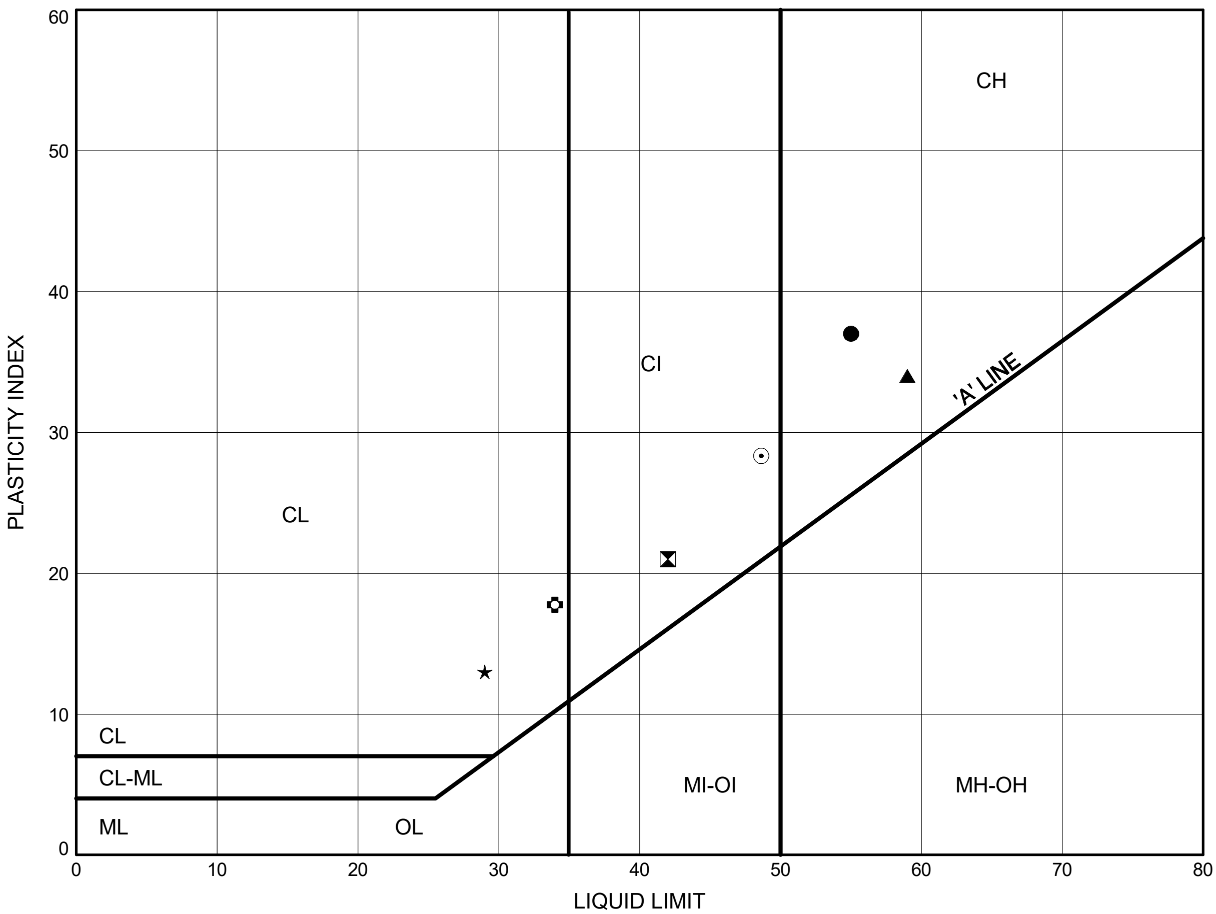
Highway 17 Twinning

ATTERBERG LIMITS TEST RESULTS

FIGURE C6

Clayey SILT to CLAY (CL to CH)



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	18-1	4.9	115.3
⧗	18-1	6.4	113.8
▲	18-2	0.9	115.1
★	18-2	8.1	107.9
⊙	29-416/C1M	1.8	114.3
✪	29-416/C1N	3.4	112.7

THURBALT 24726 DOCHART CREEK.GPJ 9/7/21

Date July 2021

WP# 4068-09-00

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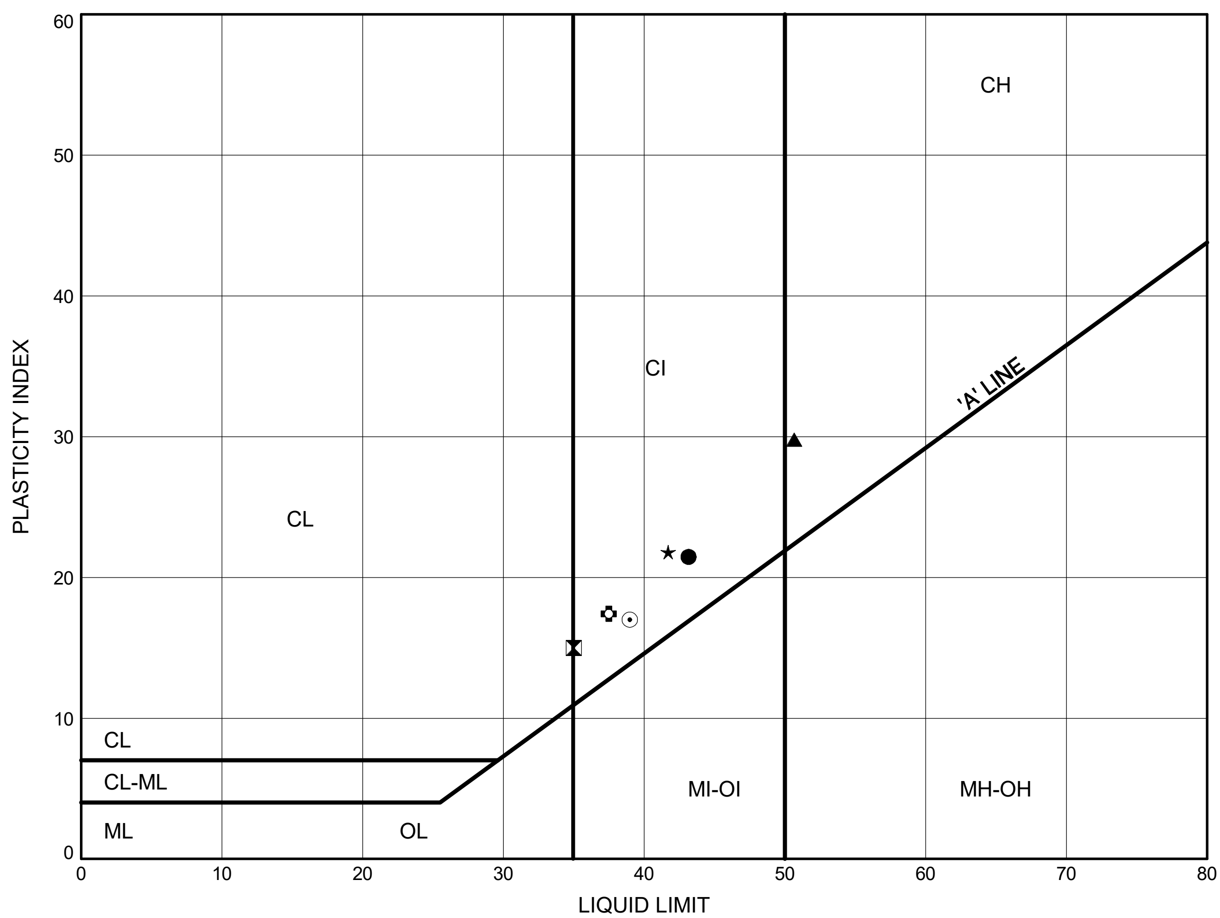
Highway 17 Twinning ATTERBERG LIMITS TEST RESULTS	FIGURE C7

Clayey SILT to CLAY (CL to CH)



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	29-416/C1S	3.4	113.1
⧗	DOC19-1	2.6	113.1
▲	DOC19-2	1.2	114.9
★	DOC19-2	1.8	114.3
⊙	DOC19-2	2.4	113.7
✪	DOC19-2	3.0	113.1

THURBALT 24726 DOCHART CREEK.GPJ 9/7/21

Date July 2021

WP# 4068-09-00

THURBER

Prep'd DJP

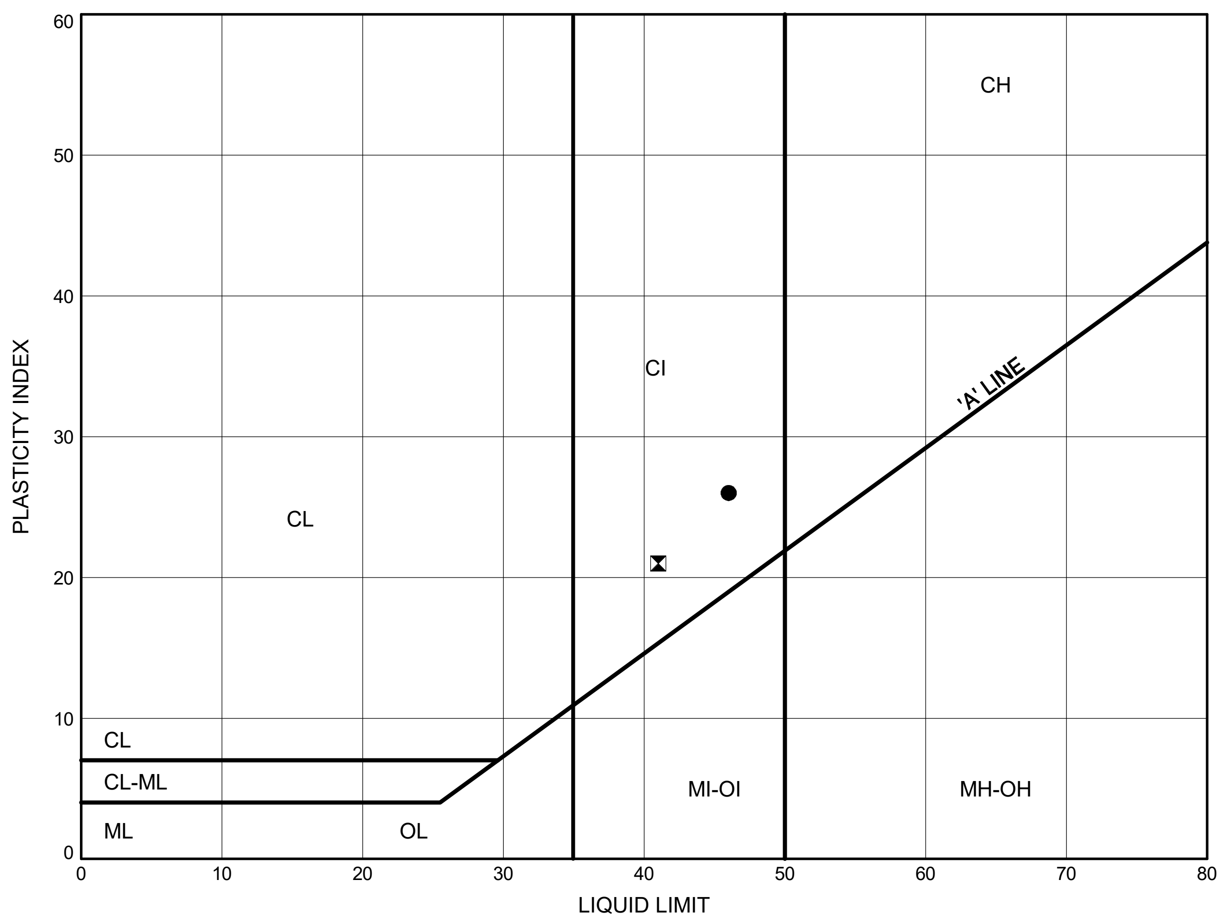
Chkd. FG

Highway 17 Twinning

ATTERBERG LIMITS TEST RESULTS

FIGURE C8

Clayey SILT to CLAY (CL to CH)



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	DOC19-3	1.1	115.1
⧗	DOC19-4	1.1	114.4

THURBALT 24726 DOCHART CREEK.GPJ 9/7/21

Date July 2021

WP# 4068-09-00

THURBER

Prep'd DJP

Chkd. FG

THURBER ENGINEERING LTD.

Consolidation Test Report

CLIENT: Thurber Engineering (Ottawa)

FILE NUMBER: 24726

PROJECT: Highway 17 Twinning - Renfrew

REPORT DATE: January 17, 2020

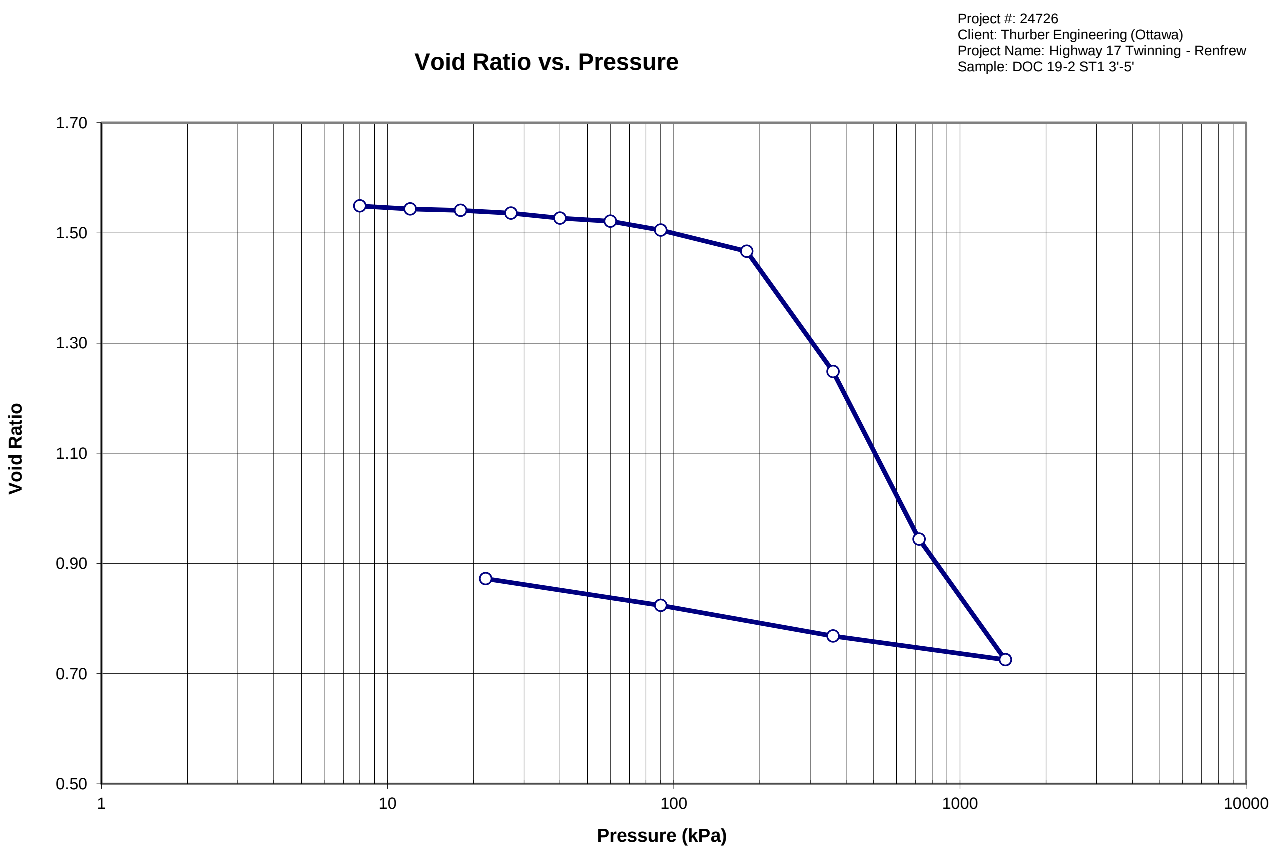
TEST DATES: November 20, 2019 - December 01, 2019

SAMPLE: DOC 19-2 ST1 3'-5'
Silty clay, trace sand, grey, moist.
LL=51, PL=21, I_p = 30.

PROCEDURE: Test carried out in accordance with Standard Test Method for One-Dimensional Consolidation Properties of Soils, ASTM D 2435-11, method B

	Start of Test	End of Test
Wet Dens. (kg/m^3)	1679.0	1974.2
Dry Dens. (kg/m^3)	1088.8	1489.9
Moisture Cont. (%)	54.2	32.5
Void Ratio	1.562	0.872
Saturation (%)	96.8	

Note: A Specific Gravity (Gs) of 2.79 was obtained for the void ratio and saturation calculations.



TEST DONE BY: BT
REVIEWED BY: KS

Consolidation Test Report

Highway 17 Twinning - Renfrew
24726 DOC 19-2 ST1 3'-5'

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

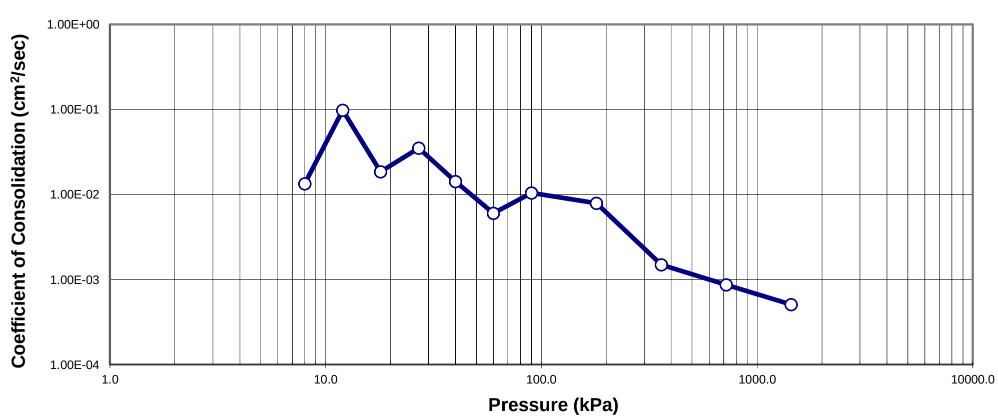
LOADING: A seating load of 8 kPa was applied and the consolidometer was flooded with distilled water. Sample was monitored to ensure no swelling effect occurred before the start of the test. Subsequent loads were applied after 100% primary consolidation was reached at each load increment.

CALCULATIONS: Coefficients of Consolidation were calculated by the square root time method.

Pressure (kPa)	Corr. H. (mm)	Avg. H. (mm)	D_{90} (mm)	t_{90} (min)	c_v (cm^2/s)	Void Ratio	m_v (m^2/kN)	k (cm/s)
0.0	25.400					1.562		
8.0	25.268	25.334	-0.033	1.69	1.34E-02	1.549	6.49E-04	8.55E-07
12.0	25.215	25.242	-0.015	0.23	9.77E-02	1.543	5.23E-04	5.01E-06
18.0	25.188	25.201	-0.012	1.21	1.85E-02	1.541	1.82E-04	3.32E-07
27.0	25.141	25.164	-0.029	0.64	3.50E-02	1.536	2.05E-04	7.04E-07
40.0	25.050	25.096	-0.041	1.56	1.42E-02	1.527	2.78E-04	3.88E-07
60.0	24.992	25.021	-0.047	3.69	6.00E-03	1.521	1.16E-04	6.83E-08
90.0	24.835	24.913	-0.065	2.10	1.04E-02	1.505	2.10E-04	2.15E-07
180.0	24.454	24.645	-0.230	2.72	7.88E-03	1.467	1.70E-04	1.31E-07
360.0	22.287	23.371	-1.340	12.96	1.49E-03	1.248	4.92E-04	7.19E-08
720.0	19.274	20.780	-2.065	17.64	8.65E-04	0.944	3.75E-04	3.18E-08
1440.0	17.103	18.188	-1.670	23.04	5.07E-04	0.725	1.56E-04	7.78E-09
360.0	17.527	17.315				0.768		
90.0	18.080	17.803				0.824		
22.0	18.562	18.321				0.872		

Project #: 24726
Client: Thurber Engineering (Ottawa)
Project Name: Highway 17 Twinning - Renfrew
Sample: DOC 19-2 ST1 3'-5'

Coefficient of Consoildation vs. Pressure



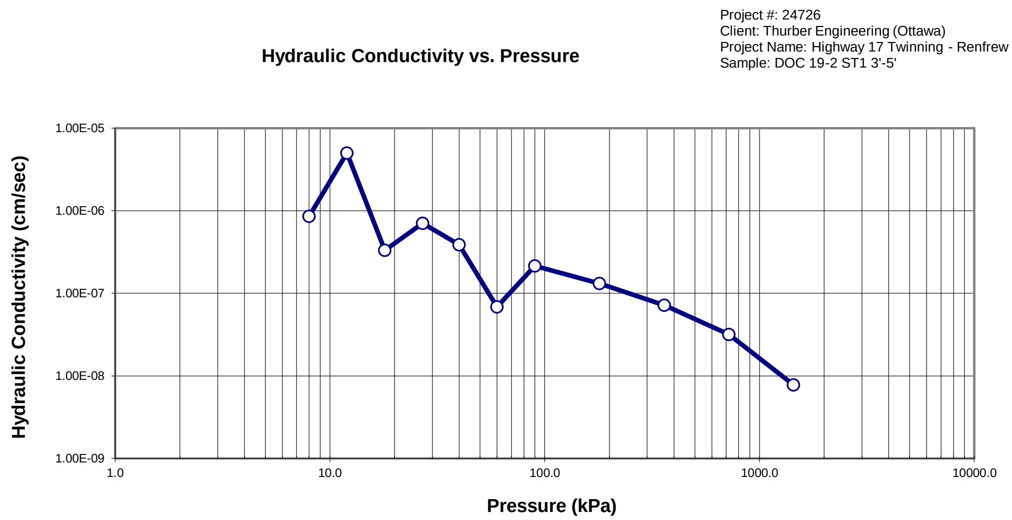
Notes: C_v and k calculated using t_{90} values

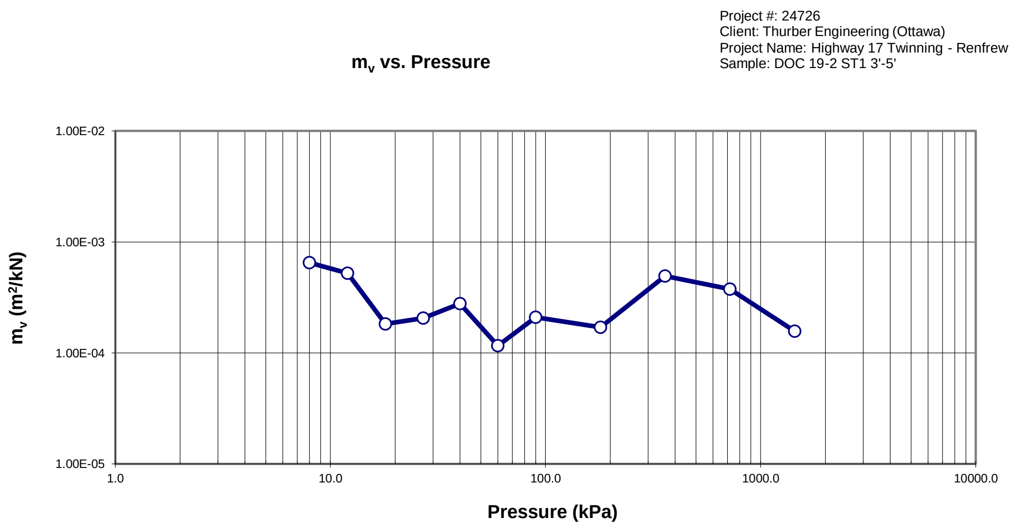
TEST DONE BY: BT
REVIEWED BY: KS

Consolidation Test Report

Highway 17 Twinning - Renfrew
24726

DOC 19-2 ST1 3'-5'





TEST DONE BY: BT
REVIEWED BY: KS

Consolidation Test Report

CLIENT: Thurber Engineering (Ottawa) FILE NUMBER: 24726

PROJECT: Highway 17 Twinning - Renfrew REPORT DATE: January 17, 2020

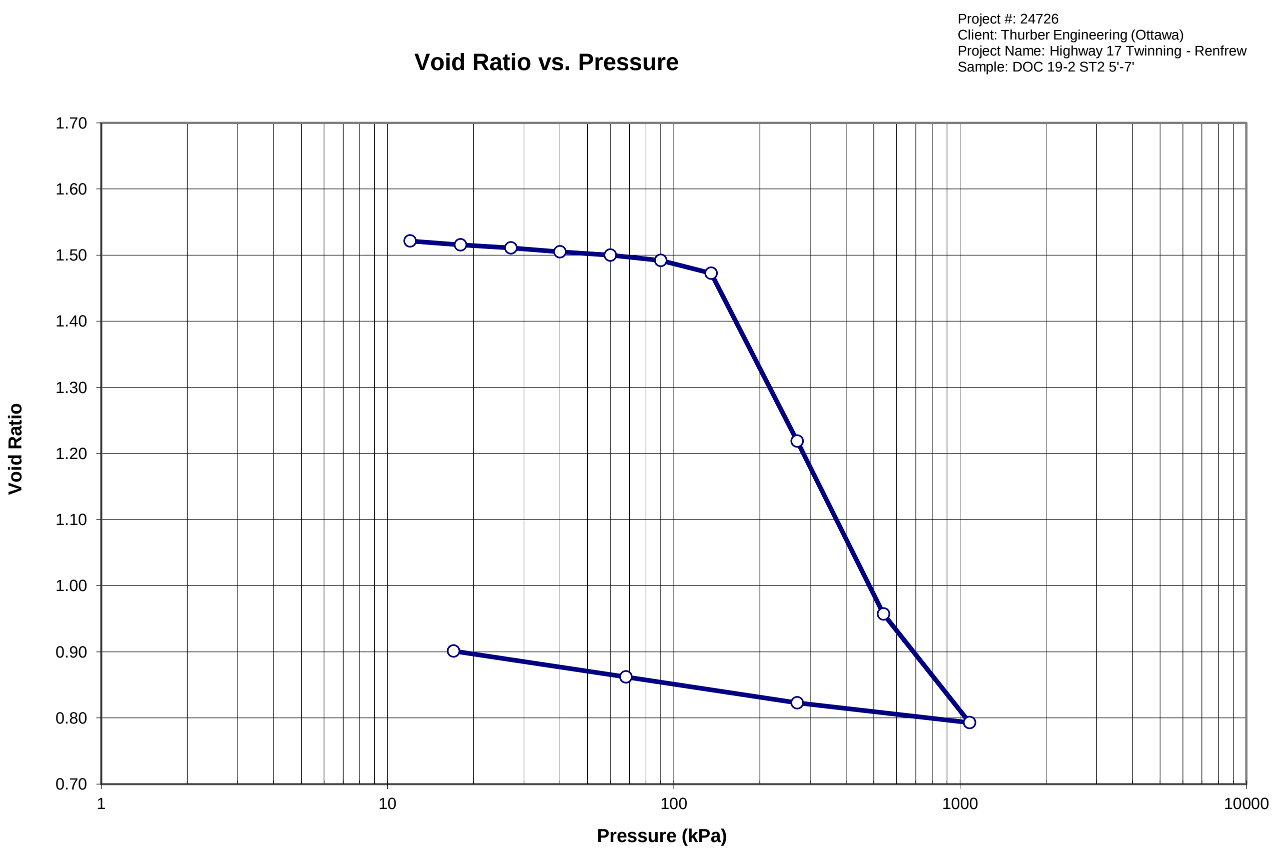
TEST DATES: November 20, 2019 - November 30, 2019

SAMPLE: DOC 19-2 ST2 5'-7'
Silty clay, trace sand, grey, moist.
LL=42, PL=20, I_p = 22.

PROCEDURE: Test carried out in accordance with Standard Test Method for One-Dimensional Consolidation Properties of Soils, ASTM D 2435-11, method B

	Start of Test	End of Test
Wet Dens. (kg/m^3)	1681.0	1944.8
Dry Dens. (kg/m^3)	1105.7	1469.5
Moisture Cont. (%)	52.0	32.3
Void Ratio	1.527	0.901
Saturation (%)	95.2	

Note: A Specific Gravity (Gs) of 2.79 was obtained for the void ratio and saturation calculations.



TEST DONE BY: BT
REVIEWED BY: KS

THURBER ENGINEERING LTD.

Consolidation Test Report

Highway 17 Twinning - Renfrew
24726 DOC 19-2 ST2 5'-7'

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

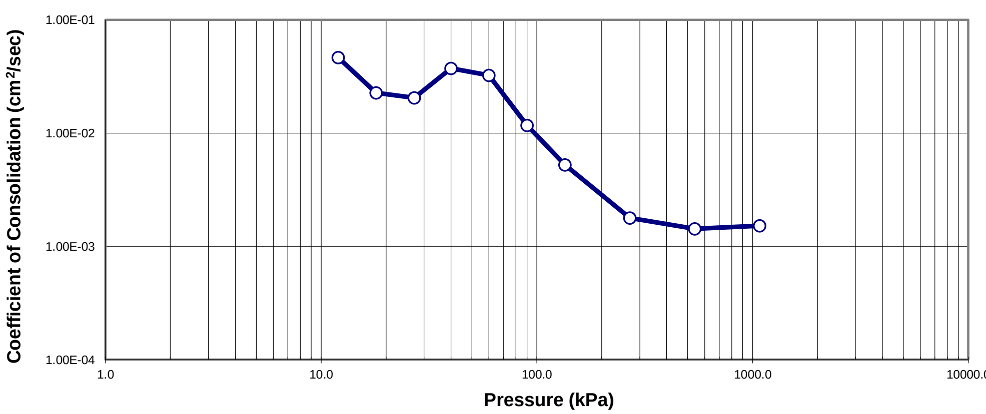
LOADING: A seating load of 12 kPa was applied and the consolidometer was flooded with distilled water. Sample was monitored to ensure no swelling effect occurred before the start of the test. Subsequent loads were applied after 100% primary consolidation was reached at each load increment.

CALCULATIONS: Coefficients of Consolidation were calculated by the square root time method.

Pressure (kPa)	Corr. H. (mm)	Avg. H. (mm)	D_{90} (mm)	t_{90} (min)	c_v (cm^2/s)	Void Ratio	m_v (m^2/kN)	k (cm/s)
0.0	25.400					1.527		
12.0	25.343	25.372	-0.023	0.49	4.64E-02	1.521	1.87E-04	8.50E-07
18.0	25.284	25.314	-0.032	1.00	2.26E-02	1.515	3.87E-04	8.60E-07
27.0	25.238	25.261	-0.035	1.10	2.05E-02	1.511	2.03E-04	4.07E-07
40.0	25.181	25.209	-0.023	0.60	3.74E-02	1.505	1.75E-04	6.40E-07
60.0	25.130	25.156	-0.038	0.69	3.25E-02	1.500	1.00E-04	3.19E-07
90.0	25.047	25.089	-0.052	1.90	1.17E-02	1.492	1.10E-04	1.26E-07
135.0	24.852	24.950	-0.103	4.20	5.23E-03	1.472	1.74E-04	8.91E-08
270.0	22.303	23.577	-1.080	11.02	1.78E-03	1.219	7.60E-04	1.33E-07
540.0	19.673	20.988	-1.800	10.89	1.43E-03	0.957	4.37E-04	6.12E-08
1080.0	18.021	18.847	-1.110	8.24	1.52E-03	0.793	1.56E-04	2.32E-08
270.0	18.324	18.173				0.823		
68.0	18.715	18.520				0.862		
17.0	19.112	18.913				0.901		

Project #: 24726
Client: Thurber Engineering (Ottawa)
Project Name: Highway 17 Twinning - Renfrew
Sample: DOC 19-2 ST2 5'-7'

Coefficient of Consoildation vs. Pressure



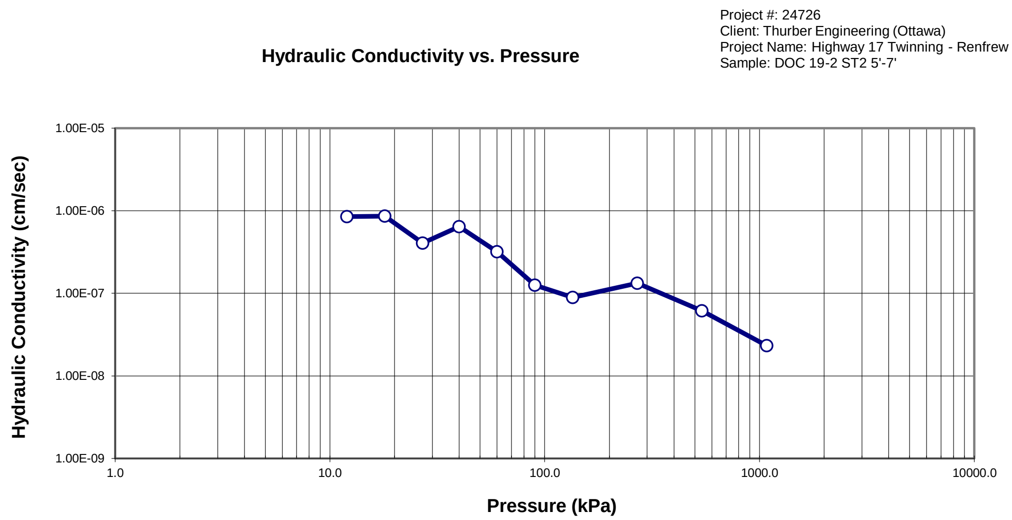
Notes: C_v and k calculated using t_{90} values

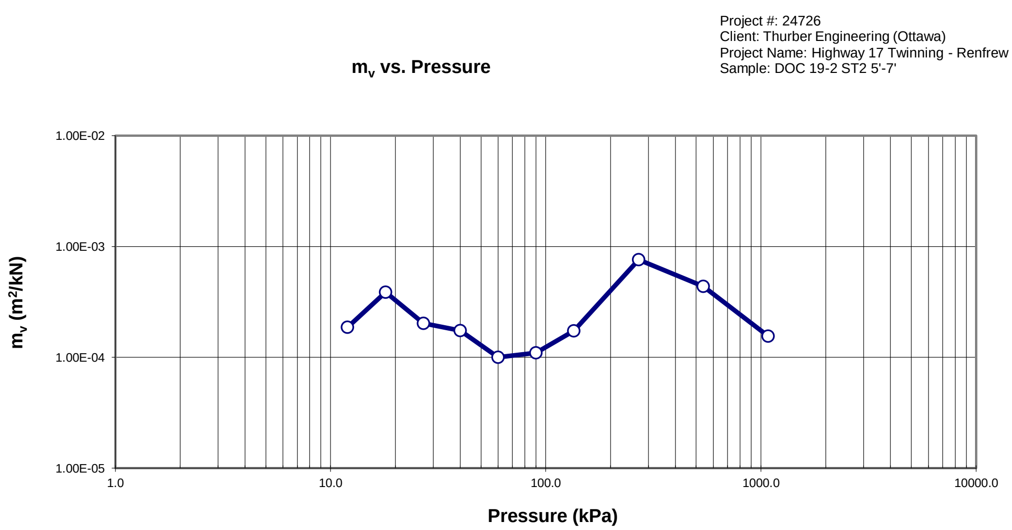
TEST DONE BY: BT
REVIEWED BY: KS

Consolidation Test Report

Highway 17 Twinning - Renfrew
24726

DOC 19-2 ST2 5'-7'





THURBER ENGINEERING LTD.

Consolidation Test Report

CLIENT: Thurber Engineering (Ottawa) FILE NUMBER: 24726

PROJECT: Highway 17 Twinning - Renfrew REPORT DATE: January 17, 2020

TEST DATES: December 18, 2020 - December 29, 2020

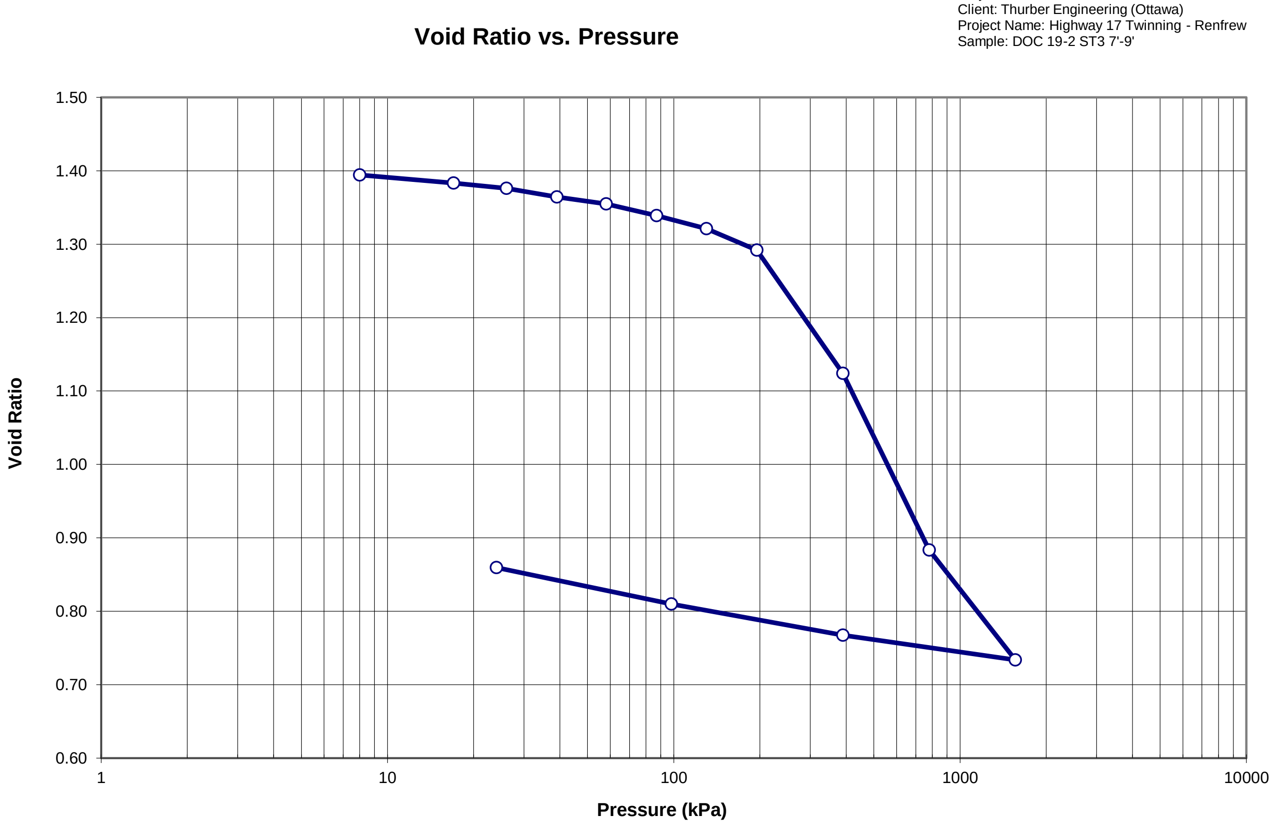
SAMPLE: DOC 19-2 ST3 7'-9'
Silty clay, trace sand, grey, moist.
LL=39, PL=22, I_p = 17.

PROCEDURE: Test carried out in accordance with Standard Test Method for One-Dimensional Consolidation Properties of Soils, ASTM D 2435-11, method B

	Start of Test	End of Test
Wet Dens. (kg/m^3)	1730.6	1979.3
Dry Dens. (kg/m^3)	1163.9	1502.1
Moisture Cont. (%)	48.7	31.8
Void Ratio	1.400	0.860
Saturation (%)	97.2	

Note: A Specific Gravity (Gs) of 2.79 was obtained for the void ratio and saturation calculations.

Project #: 24726
Client: Thurber Engineering (Ottawa)
Project Name: Highway 17 Twinning - Renfrew
Sample: DOC 19-2 ST3 7'-9'



TEST DONE BY: BT
REVIEWED BY: KS

THURBER ENGINEERING LTD.

Consolidation Test Report

Highway 17 Twinning - Renfrew
24726 DOC 19-2 ST3 7'-9'

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

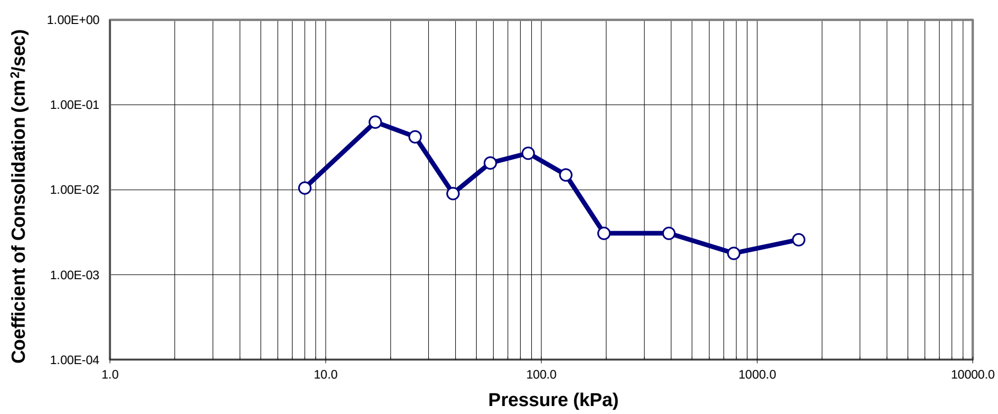
LOADING: A seating load of 8.0 kPa was applied and the consolidometer was flooded with distilled water. Sample was monitored to ensure no swelling effect occurred before the start of the test. Subsequent loads were applied after 100% primary consolidation was reached at each load increment.

CALCULATIONS: Coefficients of Consolidation were calculated by the square root time method.

Pressure (kPa)	Corr. H. (mm)	Avg. H. (mm)	D_{90} (mm)	t_{90} (min)	c_v (cm^2/s)	Void Ratio	m_v (m^2/kN)	k (cm/s)
0.0	25.400					1.400		
8.0	25.340	25.370	-0.025	2.16	1.05E-02	1.394	2.96E-04	3.06E-07
17.0	25.223	25.281	-0.068	0.36	6.27E-02	1.383	5.13E-04	3.16E-06
26.0	25.147	25.185	-0.038	0.53	4.21E-02	1.376	3.35E-04	1.38E-06
39.0	25.024	25.085	-0.079	2.46	9.02E-03	1.364	3.75E-04	3.31E-07
58.0	24.922	24.973	-0.073	1.06	2.08E-02	1.355	2.15E-04	4.37E-07
87.0	24.756	24.839	-0.092	0.81	2.69E-02	1.339	2.31E-04	6.08E-07
130.0	24.567	24.661	-0.090	1.44	1.49E-02	1.321	1.77E-04	2.60E-07
195.0	24.257	24.412	-0.138	6.86	3.07E-03	1.292	1.94E-04	5.83E-08
390.0	22.479	23.368	-0.652	6.30	3.06E-03	1.124	3.76E-04	1.13E-07
780.0	19.932	21.205	-1.735	8.88	1.79E-03	0.883	2.90E-04	5.10E-08
1560.0	18.348	19.140	-1.020	5.02	2.58E-03	0.734	1.02E-04	2.58E-08
390.0	18.704	18.526				0.767		
98.0	19.153	18.928				0.810		
24.0	19.680	19.416				0.860		

Project #: 24726
Client: Thurber Engineering (Ottawa)
Project Name: Highway 17 Twinning - Renfrew
Sample: DOC 19-2 ST3 7'-9'

Coefficient of Consoildation vs. Pressure



Notes: C_v and k calculated using t_{90} values

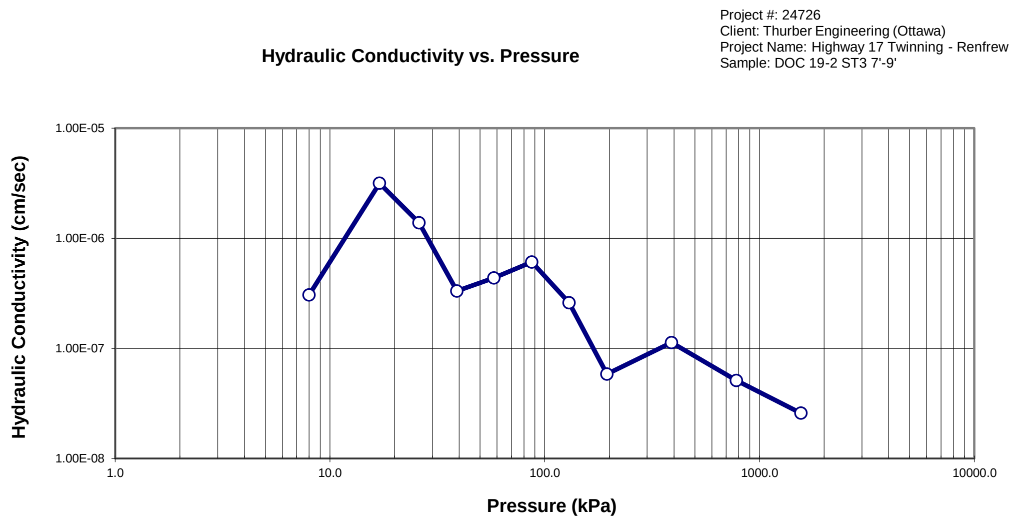
TEST DONE BY: BT
REVIEWED BY: KS

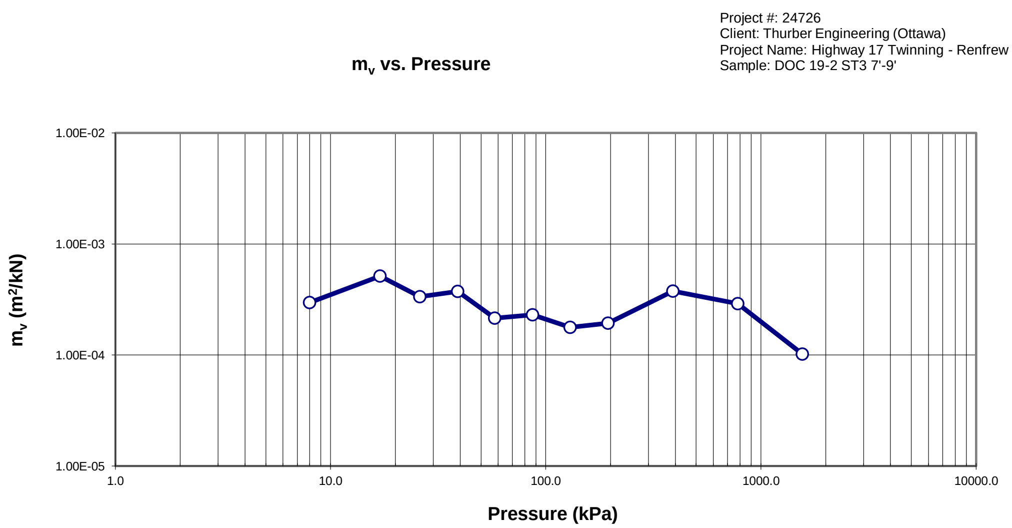
THURBER ENGINEERING LTD.

Consolidation Test Report

Highway 17 Twinning - Renfrew
24726

DOC 19-2 ST3 7'-9'





TEST DONE BY: BT
REVIEWED BY: KS

THURBER ENGINEERING LTD.

Consolidation Test Report

CLIENT: Thurber Engineering (Ottawa) FILE NUMBER: 24726

PROJECT: Highway 17 Twinning - Renfrew REPORT DATE: January 17, 2020

TEST DATES: December 18, 2020 - December 29, 2020

SAMPLE: DOC 19-2 ST4 9'-11'
Silty clay, trace sand, grey, moist.
LL=37, PL=20, I_p = 17.

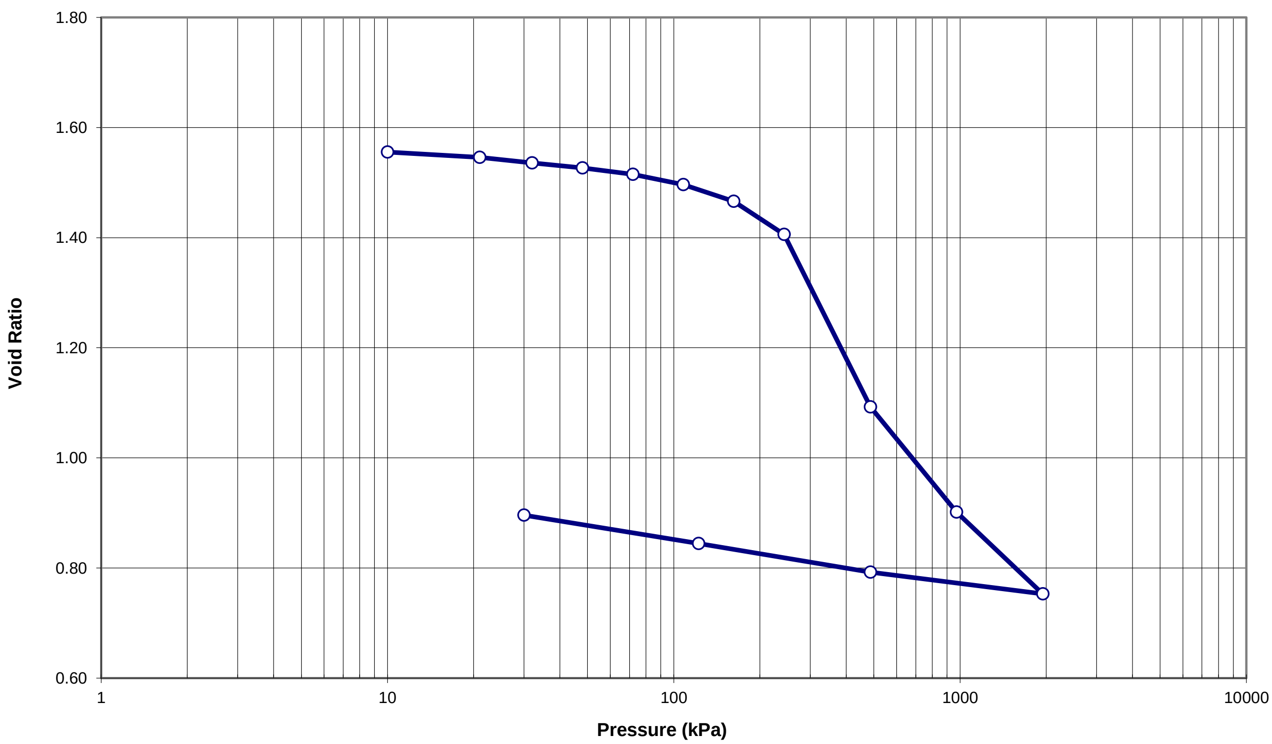
PROCEDURE: Test carried out in accordance with Standard Test Method for One-Dimensional Consolidation Properties of Soils, ASTM D 2435-11, method B

	Start of Test	End of Test
Wet Dens. (kg/m^3)	1694.8	1956.3
Dry Dens. (kg/m^3)	1090.9	1473.3
Moisture Cont. (%)	55.4	32.8
Void Ratio	1.561	0.896
Saturation (%)	99.1	

Note: A Specific Gravity (Gs) of 2.79 was obtained for the void ratio and saturation calculations.

Project #: 24726
Client: Thurber Engineering (Ottawa)
Project Name: Highway 17 Twinning - Renfrew
Sample: DOC 19-2 ST4 9'-11'

Void Ratio vs. Pressure



TEST DONE BY: BT
REVIEWED BY: KS

THURBER ENGINEERING LTD.

Consolidation Test Report

Highway 17 Twinning - Renfrew
24726 DOC 19-2 ST4 9'-11'

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

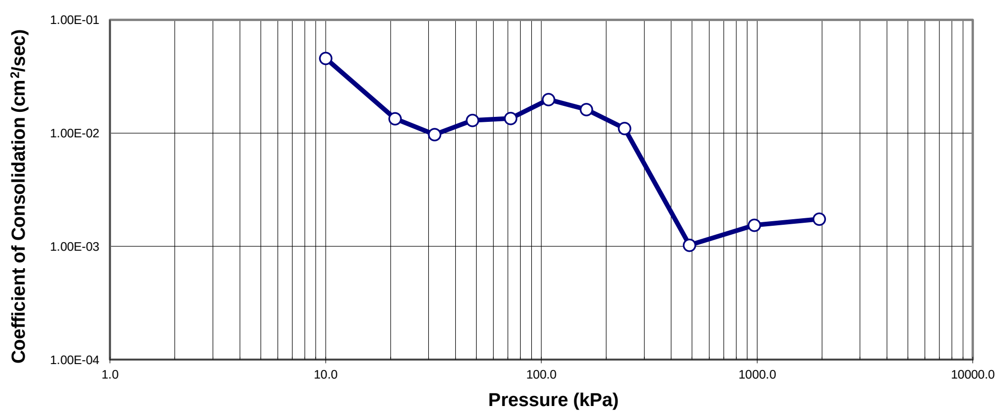
LOADING: A seating load of 10 kPa was applied and the consolidometer was flooded with distilled water. Sample was monitored to ensure no swelling effect occurred before the start of the test. Subsequent loads were applied after a constant load increment duration of 24 hours. Load increment durations were reduced when swelling was apparent.

CALCULATIONS: Coefficients of Consolidation were calculated by the square root time method.

Pressure (kPa)	Corr. H. (mm)	Avg. H. (mm)	D_{90} (mm)	t_{90} (min)	c_v (cm^2/s)	Void Ratio	m_v (m^2/kN)	k (cm/s)
0.0	25.400					1.561		
10.0	25.345	25.372	-0.025	0.50	4.58E-02	1.555	2.17E-04	9.73E-07
21.0	25.252	25.298	-0.056	1.69	1.34E-02	1.546	3.35E-04	4.40E-07
32.0	25.147	25.199	-0.076	2.31	9.71E-03	1.535	3.77E-04	3.59E-07
48.0	25.058	25.103	-0.058	1.72	1.30E-02	1.527	2.20E-04	2.80E-07
72.0	24.943	25.001	-0.056	1.64	1.35E-02	1.515	1.91E-04	2.53E-07
108.0	24.761	24.852	-0.079	1.10	1.98E-02	1.497	2.03E-04	3.95E-07
162.0	24.456	24.608	-0.103	1.32	1.62E-02	1.466	2.28E-04	3.62E-07
243.0	23.860	24.158	-0.153	1.88	1.10E-02	1.406	3.01E-04	3.24E-07
486.0	20.756	22.308	-1.920	17.14	1.03E-03	1.093	5.35E-04	5.39E-08
972.0	18.860	19.808	-1.250	9.00	1.54E-03	0.902	1.88E-04	2.84E-08
1944.0	17.387	18.124	-0.600	6.66	1.74E-03	0.753	8.03E-05	1.37E-08
486.0	17.776	17.582				0.792		
122.0	18.294	18.035				0.844		
30.0	18.808	18.551				0.896		

Project #: 24726
Client: Thurber Engineering (Ottawa)
Project Name: Highway 17 Twinning - Renfrew
Sample: DOC 19-2 ST4 9'-11'

Coefficient of Consoildation vs. Pressure



Notes: C_v and k calculated using t_{90} values

TEST DONE BY: BT
REVIEWED BY: KS

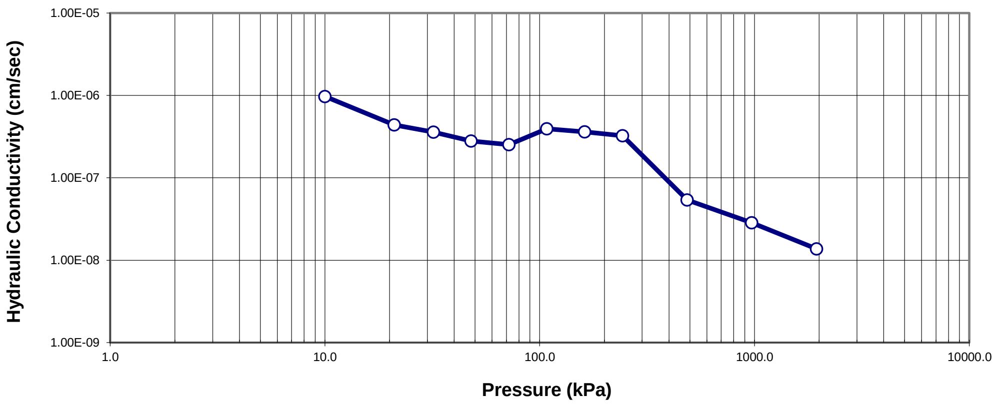
Consolidation Test Report

Highway 17 Twinning - Renfrew
24726

DOC 19-2 ST4 9'-11'

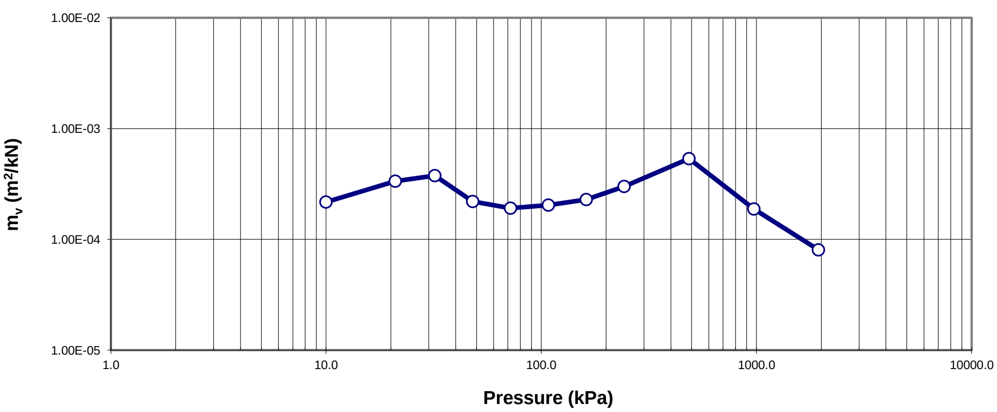
Hydraulic Conductivity vs. Pressure

Project #: 24726
Client: Thurber Engineering (Ottawa)
Project Name: Highway 17 Twinning - Renfrew
Sample: DOC 19-2 ST4 9'-11'



m_v vs. Pressure

Project #: 24726
Client: Thurber Engineering (Ottawa)
Project Name: Highway 17 Twinning - Renfrew
Sample: DOC 19-2 ST4 9'-11'



TEST DONE BY: BT
REVIEWED BY: KS

Appendix C.2

Analytical Testing Results

PARACEL LABORATORIES LTD.
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Certificate of Analysis

Thurber Engineering Ltd.

2460 Lancaster Rd, Suite 104
Ottawa, ON K1B 4S5
Attn: Paul Carnaffan

Client PO:
Project: 24726 Hwy 17 Twinning, Dochart
Custody: 49178

Report Date: 30-Oct-2019
Order Date: 24-Oct-2019

Order #: 1943587

This Certificate of Analysis contains analytical data applicable to the following samples as submitted:

Paracel ID	Client ID
1943587-01	SS2/DOC 19-1, 2'6"-4'6"
1943587-02	SS1/DOC 19-4, 0'-2'

Approved By: Mark Foto

Mark Foto, M.Sc.
Lab Supervisor

Certificate of Analysis

Client: Thurber Engineering Ltd.

Client PO:

Report Date: 30-Oct-2019

Order Date: 24-Oct-2019

Project Description: 24726 Hwy 17 Twinning, Dochart

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	29-Oct-19	30-Oct-19
Conductivity	MOE E3138 - probe @25 °C, water ext	28-Oct-19	28-Oct-19
pH, soil	EPA 150.1 - pH probe @ 25 °C, CaCl buffered ext.	28-Oct-19	28-Oct-19
Resistivity	EPA 120.1 - probe, water extraction	28-Oct-19	28-Oct-19
Solids, %	Gravimetric, calculation	25-Oct-19	25-Oct-19

Certificate of Analysis

Client: Thurber Engineering Ltd.

Client PO:

Report Date: 30-Oct-2019

Order Date: 24-Oct-2019

Project Description: 24726 Hwy 17 Twinning, Dochart

	Client ID:	SS2/DOC 19-1, 2'6"-4'6"	SS1/DOC 19-4, 0'-2'	-	-
	Sample Date:	21-Oct-19 09:00	18-Oct-19 09:00	-	-
	Sample ID:	1943587-01	1943587-02	-	-
	MDL/Units	Soil	Soil	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	73.9	56.7	-	-
General Inorganics					
Conductivity	5 uS/cm	852	415	-	-
pH	0.05 pH Units	7.76	7.48	-	-
Resistivity	0.10 Ohm.m	11.7	24.1	-	-
Anions					
Chloride	5 ug/g dry	439	52	-	-
Sulphate	5 ug/g dry	53	42	-	-

Method Quality Control: Blank

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	ND	5	ug/g						
Sulphate	ND	5	ug/g						
General Inorganics									
Conductivity	ND	5	uS/cm						
Resistivity	ND	0.10	Ohm.m						

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	417	5	ug/g dry	439			5.2	20	
Sulphate	51.1	5	ug/g dry	52.8			3.3	20	
General Inorganics									
Conductivity	200	5	uS/cm	203			1.4	5	
pH	7.52	0.05	pH Units	7.62			1.3	2.3	
Resistivity	50.0	0.10	Ohm.m	49.3			1.4	20	
Physical Characteristics									
% Solids	77.2	0.1	% by Wt.	77.5			0.4	25	

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	528	5	ug/g	439	88.6	82-118			
Sulphate	163	5	ug/g	52.8	110	80-120			

Qualifier Notes:

Login Qualifiers :

Container(s) - Bottle and COC sample ID don't match -
Applies to samples: SS1/DOC 19-4, 0'-2'

Sample Data Revisions

None

Work Order Revisions / Comments:

None

Other Report Notes:

n/a: not applicable
ND: Not Detected
MDL: Method Detection Limit
Source Result: Data used as source for matrix and duplicate samples
%REC: Percent recovery.
RPD: Relative percent difference.

Soil results are reported on a dry weight basis when the units are denoted with 'dry'.
Where %Solids is reported, moisture loss includes the loss of volatile hydrocarbons.

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Subcontracted Analysis

Thurber Engineering Ltd.
2460 Lancaster Rd, Suite 104
Ottawa, ON K1B 4S5
Attn: Paul Carnaffan

Tel: (613) 247-2121
Fax: (613) 247-2185

Paracel Report No	**1943587**
Client Project(s):	**24726 Hwy 17 Twinning, Dochart**
Client PO:	
Reference:	**Standing Offer**
CoC Number:	**49178**

Order Date:	24-Oct-19
Report Date:	23-Dec-19

Sample(s) from this project were subcontracted for the listed parameters. A copy of the subcontractor's report is attached

Paracel ID	Client ID	Analysis
1943587-01	SS2/DOC 19-1, 2'6"-4'6"	Sulphide, solid
1943587-02	SS1/DOC 19-4, 0'-2'	Sulphide, solid

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

23-December-2019

Paracel Laboratories
Attn : Dale Robertson

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

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

Date Rec. : 19 December 2019
LR Report: **CA12696-DEC19**
Reference: Project#:1943587

Copy: #1

CERTIFICATE OF ANALYSIS

Final Report

Sample ID	Sample Date & Time	Sulphide %
1: Analysis Start Date		23-Dec-19
2: Analysis Start Time		12:07
3: Analysis Completed Date		23-Dec-19
4: Analysis Completed Time		13:28
5: QC - Blank		< 0.02
6: QC - STD % Recovery		115%
7: QC - DUP % RPD		48%
8: RL		0.02
9: SS2/DOC 19-1, 2'6"-4'6"	21-Oct-19	0.10
10: SS1/DOC 19-4. 0'-2'	18-Oct-19	0.04

RL - SGS Reporting Limit

Note: Samples were received past the 14 day holding time; results may be unreliable.

Kimberley Didsbury,
Project Specialist,
Environment, Health & Safety

OnLine LIMS

0001995894

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Certificate of Analysis

Thurber Engineering Ltd.

2460 Lancaster Rd, Unit 107
Ottawa, ON K1B4S5
Attn: Kenton Power

Client PO:
Project: 22912 Dochart Creek
Custody: 39855

Report Date: 26-Jun-2018
Order Date: 21-Jun-2018

Order #: 1825667

This Certificate of Analysis contains analytical data applicable to the following samples as submitted:

Paracel ID	Client ID
1825667-01	18-2 SS3 (4'-6')

Approved By:

Dale Robertson, BSc
Laboratory Director

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	26-Jun-18	26-Jun-18
Conductivity	MOE E3138 - probe @25 °C, water ext	26-Jun-18	26-Jun-18
pH, soil	EPA 150.1 - pH probe @ 25 °C, CaCl buffered ext.	26-Jun-18	26-Jun-18
Resistivity	EPA 120.1 - probe, water extraction	26-Jun-18	26-Jun-18
Solids, %	Gravimetric, calculation	26-Jun-18	26-Jun-18

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

Report Date: 26-Jun-2018
Order Date: 21-Jun-2018
Project Description: 22912 Dochart Creek

	Client ID:	18-2 SS3 (4'-6')	-	-	-
	Sample Date:	06/06/2018 12:38	-	-	-
	Sample ID:	1825667-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	65.0	-	-	-
General Inorganics					
Conductivity	5 uS/cm	770	-	-	-
pH	0.05 pH Units	7.83	-	-	-
Resistivity	0.10 Ohm.m	13.0	-	-	-
Anions					
Chloride	5 ug/g dry	393	-	-	-
Sulphate	5 ug/g dry	77	-	-	-

Method Quality Control: Blank

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	ND	5	ug/g						
Sulphate	ND	5	ug/g						
General Inorganics									
Conductivity	ND	5	uS/cm						
Resistivity	16000	0.10	Ohm.m						

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	42.2	5	ug/g dry	42.0			0.6	20	
Sulphate	33.5	5	ug/g dry	33.7			0.4	20	
General Inorganics									
Conductivity	381	5	uS/cm	397			4.3	6.2	
pH	7.72	0.05	pH Units	7.64			1.0	10	
Resistivity	26.3	0.10	Ohm.m	25.2			4.3	20	
Physical Characteristics									
% Solids	71.0	0.1	% by Wt.	63.9			10.5	25	

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

Report Date: 26-Jun-2018
Order Date: 21-Jun-2018
Project Description: 22912 Dochart Creek

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	138	5	ug/g	42.0	96.0	78-113			
Sulphate	141	5	ug/g	33.7	108	78-111			

Qualifier Notes:

Login Qualifiers :

Container(s) - Bottle and COC sample ID don't match -
Applies to samples: 18-2 SS3 (4'-6')

Sample Data Revisions

None

Work Order Revisions / Comments:

None

Other Report Notes:

n/a: not applicable
ND: Not Detected
MDL: Method Detection Limit
Source Result: Data used as source for matrix and duplicate samples
%REC: Percent recovery.
RPD: Relative percent difference.

Soil results are reported on a dry weight basis when the units are denoted with 'dry'.
Where %Solids is reported, moisture loss includes the loss of volatile hydrocarbons.

Subcontracted Analysis

Thurber Engineering Ltd.
2460 Lancaster Rd, Unit 107
Ottawa, ON K1B4S5
Attn: Kenton Power

Tel: (613) 247-2121
Fax: (613) 247-2185

Paracel Report No **1825667**
Client Project(s): **22912 Dochart Creek**
Client PO:
Reference: **Standing Offer**
CoC Number: **39855**

Order Date: 21-Jun-18
Report Date: 05-Jul-18

Sample(s) from this project were subcontracted for the listed parameters. A copy of the subcontractor's report is attached

Paracel ID	Client ID	Analysis
1825667-01	18-2 SS3 (4'-6')	Sulphide, solid

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

05-July-2018

Paracel Laboratories
Attn : Dale Robertson

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

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

Date Rec. : 26 June 2018
LR Report: **CA12858-JUN18**
Reference: Project#:1825667

Copy: #1

CERTIFICATE OF ANALYSIS

Final Report

Sample ID	Sample Date & Time	Sulphide %
1: Analysis Start Date		05-Jul-18
2: Analysis Start Time		10:43
3: Analysis Completed Date		05-Jul-18
4: Analysis Completed Time		13:06
5: QC - Blank		<0.02
6: QC - STD % Recovery		85%
7: QC - DUP % RPD		11%
8: RL		0.02
9: 18-2 SS3 (4'-6')	21-Jun-18	< 0.02

RL - SGS Reporting Limit

Kimberley Didsbury
Project Specialist
Environmental Services, Analytical

OnLine LIMS

0001429755

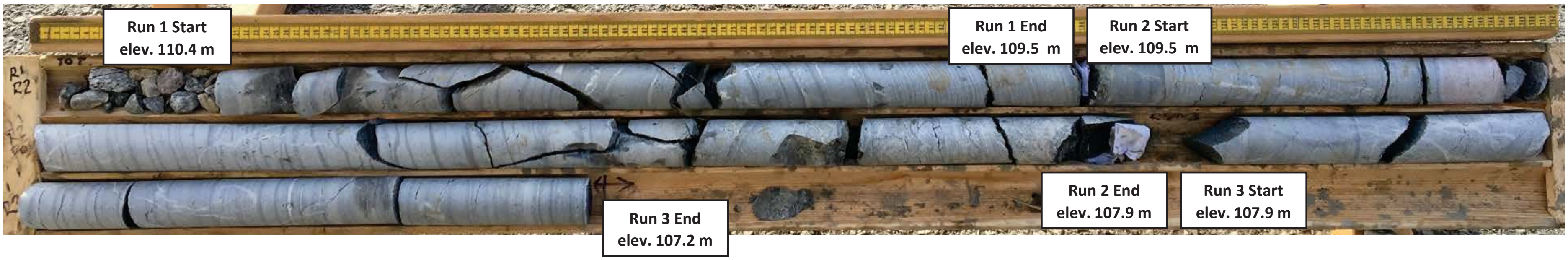
Appendix C.3

Bedrock Core Photographs

Borehole DOC19-1

Run 1 to 3 (of 3)

Elevation 110.4 m to 107.2 m



THURBER ENGINEERING LTD.

Geotechnical Investigation
HWY 17 Twinning (Dochart Creek)
Renfrew, Ontario

WP: 4068-09-00

Project No.: 24726

Borehole DOC19-2

Run 1 to 3 (of 3)

Elevation 109.0 m to 104.9 m

THURBER ENGINEERING LTD.

Geotechnical Investigation
HWY 17 Twinning (Dochart Creek)
Renfrew, Ontario

WP: 4068-09-00

Project No.: 24726

Borehole DOC19-3

Run 1 to 2 (of 2)

Elevation 109.0 m to 106.7 m

THURBER ENGINEERING LTD.

THURBER ENGINEERING LTD.	Geotechnical Investigation HWY 17 Twinning (Dochart Creek) Renfrew, Ontario	WP: 4068-09-00 Project No.: 24726

Borehole DOC19-4

Run 1 to 3 (of 3)

Elevation 109.0 m to 104.9 m

THURBER ENGINEERING LTD.	Geotechnical Investigation HWY 17 Twinning (Dochart Creek) Renfrew, Ontario	WP: 4068-09-00 Project No.: 24726

Appendix D.

Site Photographs

Photo 1. Looking South towards outlets of existing culverts (2019/11/26)

Photo 2. Looking North from existing embankment towards proposed westbound (2019/06/24)

Photo 3. Looking East along existing roadway
(2018/05/22)

Photo 4. Google Earth image showing variable creek width at North end of culvert and stockpile North East of the existing culverts, along the proposed Highway 17 westbound alignment.
(Imagery Date 2020/04/27)

Appendix E.

Foundation Comparison

COMPARISON OF CULVERT ALTERNATIVES

	Circular Pipes	Open Footing Culvert	Closed Bottom Box Culvert	Open Bottom Box Culvert Deep Foundation
Advantages	• Readily available materials and simple installation methods • Can accommodate more settlement	• More flexibility for installation of temporary flow passage system	• Relatively expedient installation if precast units are used	• Less prone to effects of scour and erosion
Disadvantages	• Numerous parallel pipes required to provide hydraulic opening equivalent to existing culvert • Less durable	• Existing clay subgrade is not suitable for the use of open footed culverted supported on shallow foundations • Founding elevation is deeper than with closed bottom box, requiring deeper excavation	• Requires brief preload period or cambered installation • Would likely require a temporary flow passage system	• Settlement will induce Downdrag loads on piles • Deeper excavation for pile caps • Short piles
Risks/ Constructability	• Potential for damage due to settlement	• Potential for base disturbance if groundwater not controlled / added cost and schedule delays • Potential for damage due to settlement	• Potential for damage due to settlement	• Potential for base disturbance if groundwater not controlled / added cost and schedule delays • Piles may hit refusal in the glacial deposit / reduced bearing resistance
Relative Cost	Low	Moderate	Moderate	High
Recommendation	**Feasible**	**Not Feasible**	**Recommended**	**Not Recommended**

COMPARISON OF BRIDGE FOUNDATION ALTERNATIVES

	Spread Footings on an Engineered Granular A Pad	Spread Footings on Mass Concrete Extended to Bedrock	Steel Pile	Caissons
Advantages	• Requires less specialized construction equipment	• Requires less specialized construction equipment • Higher geotechnical capacity than spread footings on Granular A pad	• Higher geotechnical capacity than spread footings • Construction could continue in winter weather conditions • Likely requires less concrete than spread footings • Less dewatering efforts • Shorter construction period • Could allow for integral abutment	• Higher geotechnical capacity than piled foundations • Construction could continue in winter weather conditions
Disadvantages	• Lower geotechnical capacity • Requires deeper excavations to construct granular pads • Less effective resistance to uplift or overturning • Granular pad to be protected from erosion/scour	• Requires deeper excavation than spread footings on Granular A pad • High cost due to large quality of concrete	• Higher unit costs than spread footings • Requires specialized construction equipment • Lower geotechnical resistance than caissons • If integral abutment is selected, bedrock coring will be required to achieve sufficient pile length	• Higher unit costs than spread footings • Requires specialized installation measures such as equipment, liners and drilling mud will be required • Difficulty in cleaning and inspecting the base • May be difficult to dewater
Risks/ Constructability	• Large excavations • Requires dewatering an excavation beside the creek	• Large excavations • Requires dewatering an excavation beside the creek and into artesian conditions in the till overlying the bedrock	• Shallow, variable, sloping bedrock • Risk of encountering obstructions	• Risk of encountering obstructions • Encountering artesian conditions in the till
Relative Cost	Moderate	High	Moderate to High	Moderate to High
Recommendation	**Feasible – Not Recommended** (for new westbound lane bridge)	**Feasible – Not Recommended** (for new westbound lane bridge)	**Feasible**	**Feasible**

Appendix F.

GSC Seismic Hazard Calculation
Slope Stability Analysis Results

2015 National Building Code Seismic Hazard Calculation

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

Site: 45.440N 76.464W **User File Reference:** Highway 17 Dochart Creek 2019-12-17 15:16 UT

Probability of exceedance per annum	0.000404	0.001	0.0021	0.01
Probability of exceedance in 50 years	2 %	5 %	10 %	40 %
Sa (0.05)	0.367	0.189	0.109	0.033
Sa (0.1)	0.434	0.235	0.142	0.047
Sa (0.2)	0.362	0.203	0.127	0.044
Sa (0.3)	0.276	0.158	0.101	0.036
Sa (0.5)	0.197	0.115	0.074	0.027
Sa (1.0)	0.100	0.060	0.039	0.013
Sa (2.0)	0.048	0.029	0.018	0.005
Sa (5.0)	0.013	0.007	0.004	0.001
Sa (10.0)	0.005	0.003	0.002	0.001
PGA (g)	0.233	0.129	0.078	0.025
PGV (m/s)	0.164	0.093	0.057	0.018

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

References

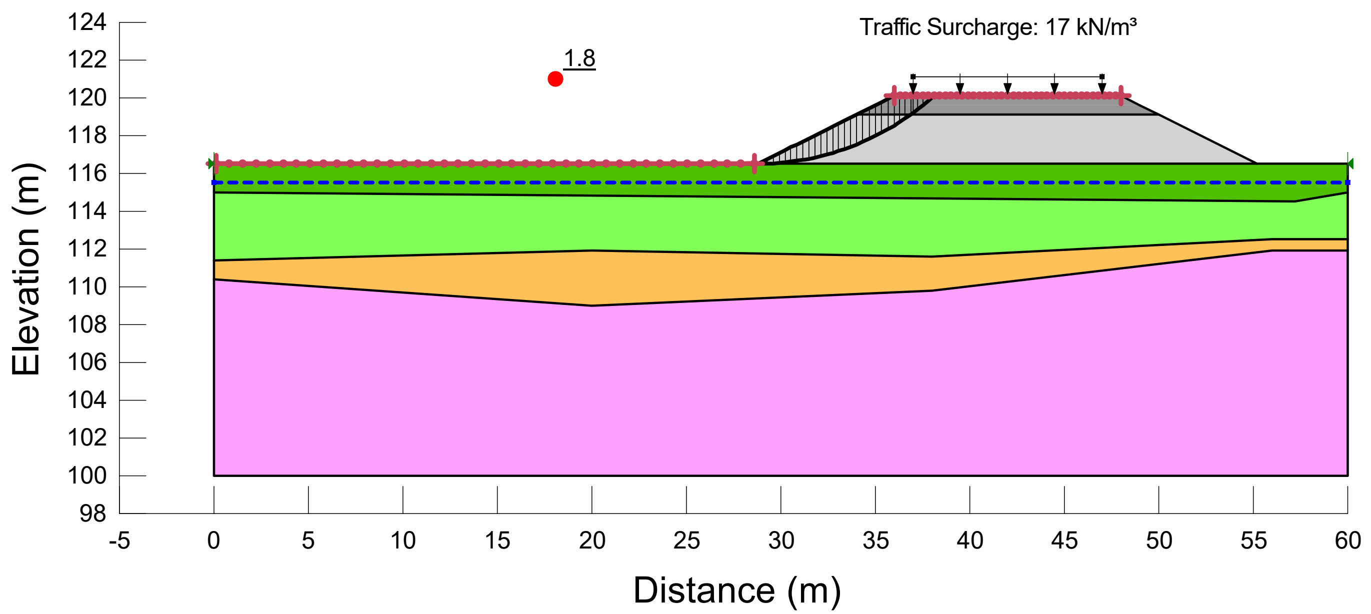
National Building Code of Canada 2015 NRCC no. 56190; Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada

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

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

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

Color	Name	Material Model	Unit Weight (kN/m³)	Cohesion (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	1. GBI or SSM	Mohr-Coulomb	21		0	30
	1. GBII / GA	Mohr-Coulomb	22		0	40
	2. CLAY (crust) TSA	Undrained (Phi=0)	17	75		
	3. CLAY TSA	Undrained (Phi=0)	17	50		
	4. TILL	Mohr-Coulomb	21		0	35
	5. BEDROCK	Bedrock (Impenetrable)				



THURBER

Project
24726 Highway 17 Dochart Creek

Analysis
F1. Short Term (TSA) - Static

Seismic Coefficient
H: 0g, V: 0g

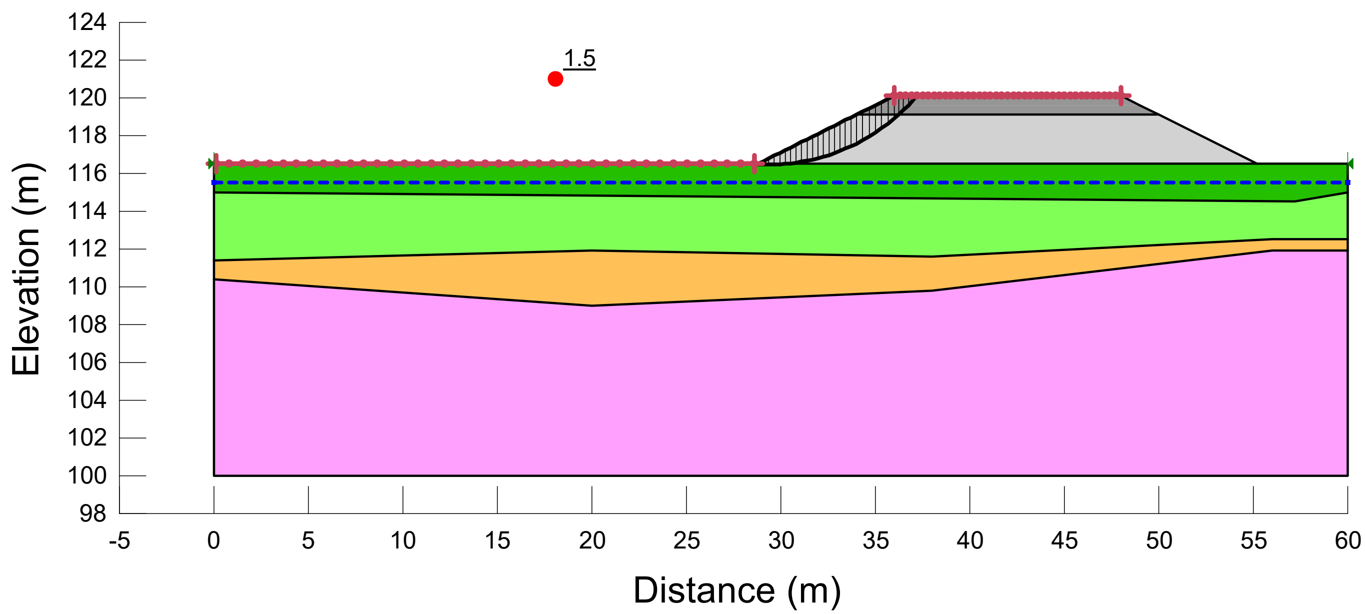
Last Run
07/14/2021, 06:14:26 PM

Scale
1:400

Additional Details
Name: 1 Westbound Embankment
Comments: Slope Stability Assessment
Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m
Entry: (38.1, 120.1) m, Exit: (28.6, 116.5) m
Center: (28.780863, 130.35745) m, Radius: 13.858626 m

Figure F1

Color	Name	Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	1. GBI or SSM	Mohr-Coulomb	21	0	30
	1. GBII / GA	Mohr-Coulomb	22	0	40
	2. CLAY (crust) ESA	Mohr-Coulomb	17	3	28
	3. CLAY ESA	Mohr-Coulomb	17	0	28
	4. TILL	Mohr-Coulomb	21	0	35
	5. BEDROCK	Bedrock (Impenetrable)			



THURBER

Project	Analysis	Seismic Coefficient	Last Run	Scale
24726 Highway 17 Dochart Creek	F2. Long Term (ESA) - Static	H: 0g, V: 0g	07/14/2021, 06:14:39 PM	1:400

Additional Details

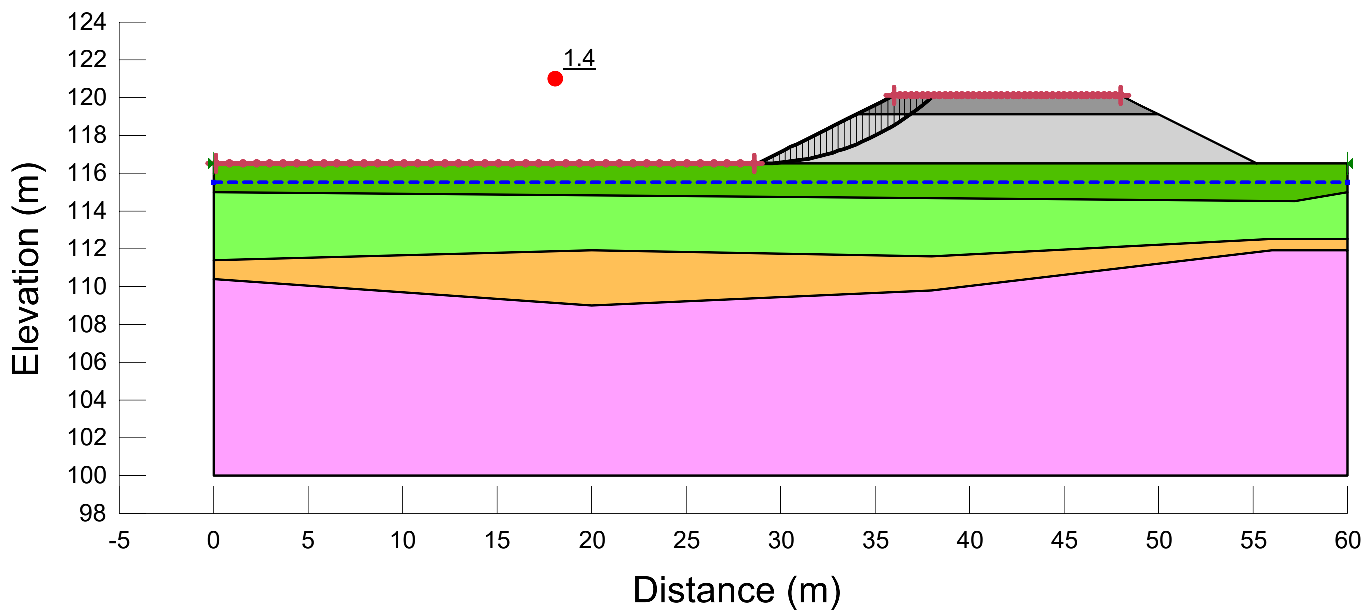
Name: 1 Westbound Embankment
Comments: Slope Stability Assessment
Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m
Entry: (37.2, 120.1) m, Exit: (28.6, 116.5) m
Center: (29.413358, 126.6292) m, Radius: 10.161803 m

Figure F2

H:\Projects\20001 to 30000\24726 - Hwy 17 Arnprior to Haley Station\PART 1A\Foundations\Dochart Creek Culverts\Analysis\Stability\Hwy 17 Dochart_003.gsz

Tool Version: 11.1.2.22321

Color	Name	Material Model	Unit Weight (kN/m³)	Cohesion (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	1. GBI or SSM	Mohr-Coulomb	21		0	30
	1. GBII / GA	Mohr-Coulomb	22		0	40
	2. CLAY (crust) TSA	Undrained (Phi=0)	17	75		
	3. CLAY TSA	Undrained (Phi=0)	17	50		
	4. TILL	Mohr-Coulomb	21		0	35
	5. BEDROCK	Bedrock (Impenetrable)				



Project	Additional Details
24726 Highway 17 Dochart Creek	Name: 1 Westbound Embankment
Analysis F3. Seismic (TSA) - Pseudo-Static (2475)	Comments: Slope Stability Assessment Method: Morgenstern-Price, Half-Sine Minimum Slip Surface Depth: 1.52 m
Seismic Coefficient H: 0.132g, V: 0g; **Last Run** 07/14/2021, 06:14:14 PM; **Scale** 1:400	Entry: (38.1, 120.1) m, Exit: (28.6, 116.5) m Center: (28.780863, 130.35745) m, Radius: 13.858626 m

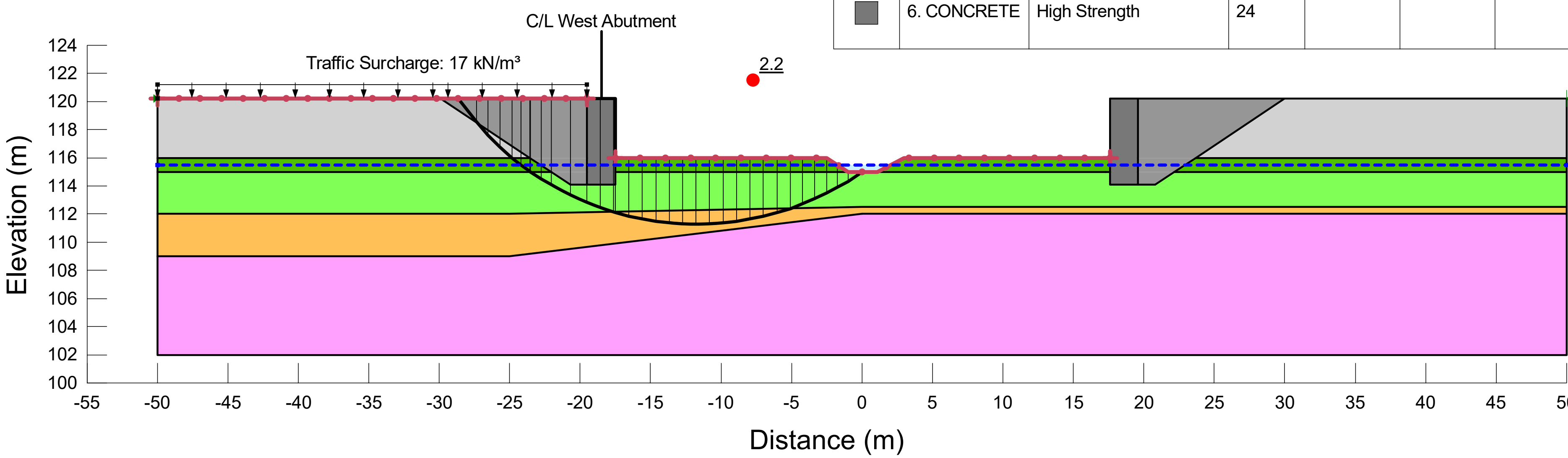
THURBER

Figure F3

H:\Projects\20001 to 30000\24726 - Hwy 17 Arnprior to Haley Station\PART 1A\Foundations\Dochart Creek Culverts\Analysis\Stability\Hwy 17 Dochart_003.gsz

Tool Version: 11.1.2.22321

Color	Name	Material Model	Unit Weight (kN/m³)	Cohesion (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	1. GBI or SSM	Mohr-Coulomb	21		0	30
	1. GBII / GA	Mohr-Coulomb	22		0	40
	2. CLAY (crust) TSA	Undrained (Phi=0)	17	75		
	3. CLAY TSA	Undrained (Phi=0)	17	50		
	4. TILL	Mohr-Coulomb	21		0	35
	5. BEDROCK	Bedrock (Impenetrable)				
	6. CONCRETE	High Strength	24			



THURBER

Project
24726 Highway 17 Dochart Creek

Analysis
F4. Short Term (TSA) - Static

Seismic Coefficient
H: g, V: g

Last Run
07/14/2021, 06:14:40 PM

Scale
1:450

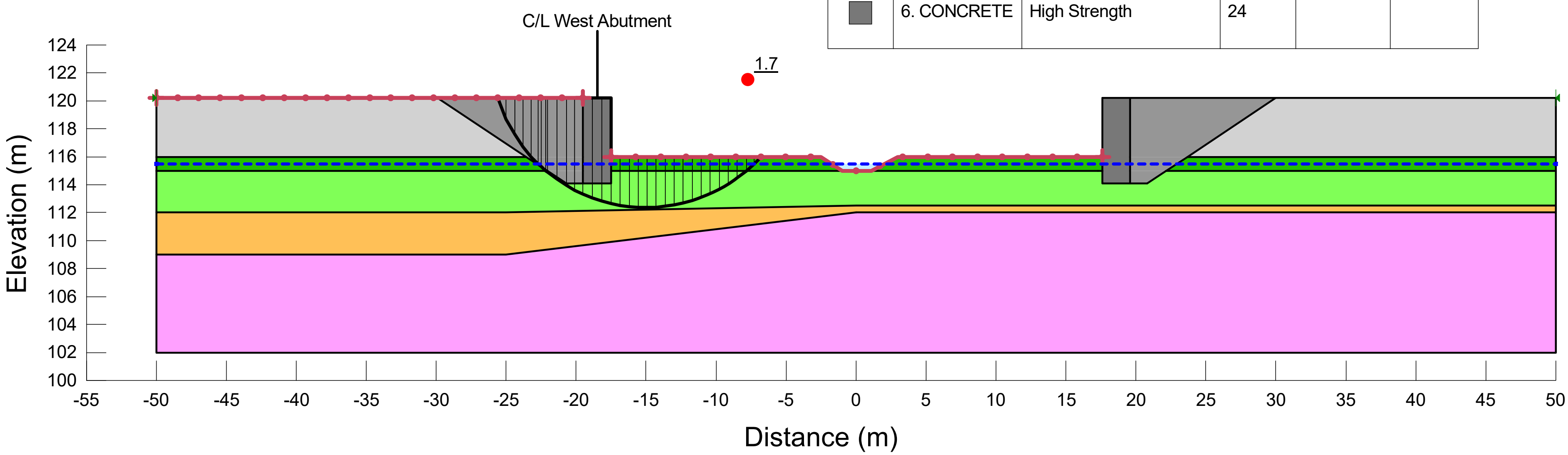
Additional Details
Name: 2 Westbound - Forward Slope
Comments: Slope Stability Assessment
Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 0.1 m
Entry: (-28.65, 120.2) m, Exit: (0.022157709, 115) m
Center: (-11.742689, 131.77746) m, Radius: 20.49133 m

Figure F4

H:\Projects\20001 to 30000\24726 - Hwy 17 Arnprior to Haley Station\PART 1A\Foundations\Dochart Creek Culverts\Analysis\Stability\Hwy 17 Dochart_003.gsz

Tool Version: 11.1.2.22321

Color	Name	Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	1. GBI or SSM	Mohr-Coulomb	21	0	30
	1. GBII / GA	Mohr-Coulomb	22	0	40
	2. CLAY (crust) ESA	Mohr-Coulomb	17	3	28
	3. CLAY ESA	Mohr-Coulomb	17	0	28
	4. TILL	Mohr-Coulomb	21	0	35
	5. BEDROCK	Bedrock (Impenetrable)			
	6. CONCRETE	High Strength	24		



Project
24726 Highway 17 Dochart Creek

Analysis
F5. Long Term (ESA) - Static

Seismic Coefficient
H: g, V: g

Last Run
07/14/2021, 06:14:41 PM

Scale
1:450

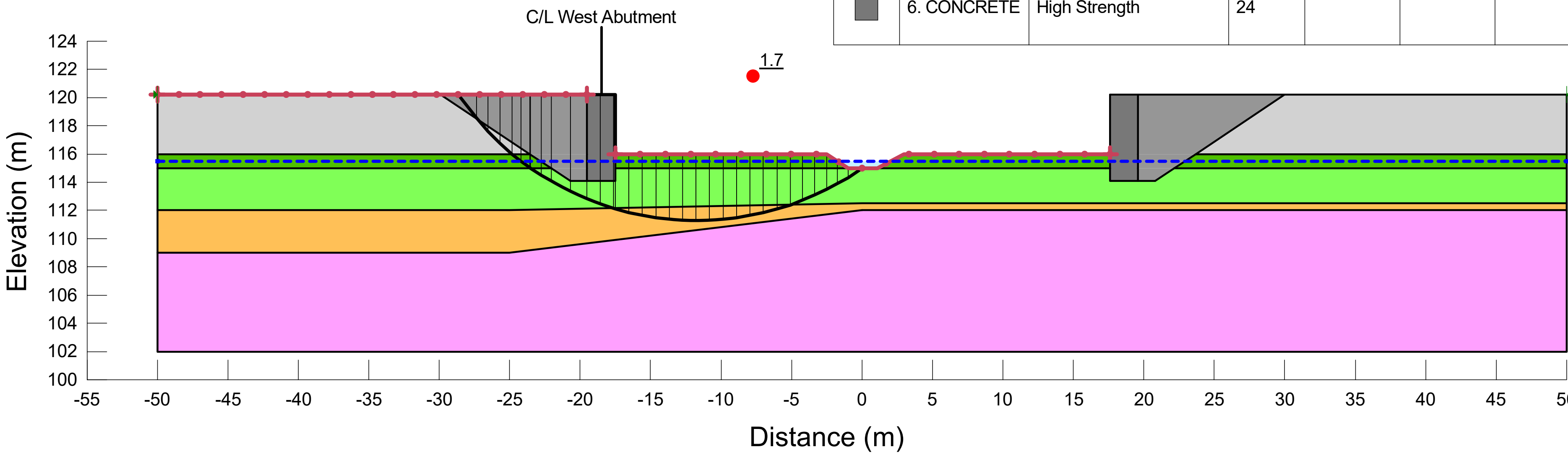
Additional Details
Name: 2 Westbound - Forward Slope
Comments: Slope Stability Assessment
Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 0.1 m
Entry: (-25.6, 120.2) m, Exit: (-6.80504, 116) m
Center: (-15.0048, 123.45979) m, Radius: 11.085328 m

Figure F5

H:\Projects\20001 to 30000\24726 - Hwy 17 Arnprior to Haley Station\PART 1A\Foundations\Dochart Creek Culverts\Analysis\Stability\Hwy 17 Dochart_003.gsz

Tool Version: 11.1.2.22321

Color	Name	Material Model	Unit Weight (kN/m³)	Cohesion (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	1. GBI or SSM	Mohr-Coulomb	21		0	30
	1. GBII / GA	Mohr-Coulomb	22		0	40
	2. CLAY (crust) TSA	Undrained (Phi=0)	17	75		
	3. CLAY TSA	Undrained (Phi=0)	17	50		
	4. TILL	Mohr-Coulomb	21		0	35
	5. BEDROCK	Bedrock (Impenetrable)				
	6. CONCRETE	High Strength	24			



THURBER

Project
24726 Highway 17 Dochart Creek

Analysis
F6. Seismic (TSA) - Pseudo-Static (2475)

Seismic Coefficient
H: 0.132g, V: 0g

Last Run
07/14/2021, 06:14:42 PM

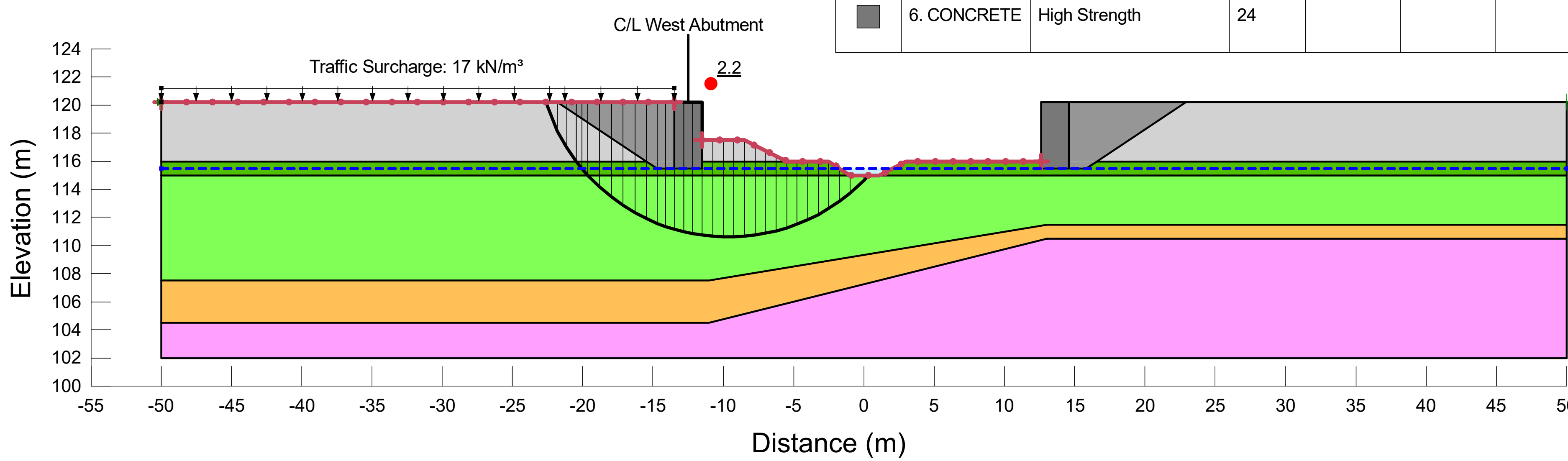
Scale
1:450

Additional Details
Name: 2 Westbound - Forward Slope
Comments: Slope Stability Assessment
Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 0.1 m
Entry: (-28.65, 120.2) m, Exit: (0.022157709, 115) m
Center: (-11.742689, 131.77746) m, Radius: 20.49133 m

Figure F6

H:\Projects\20001 to 30000\24726 - Hwy 17 Arnprior to Haley Station\PART 1A\Foundations\Dochart Creek Culverts\Analysis\Stability\Hwy 17 Dochart_003.gsz

Tool Version: 11.1.2.22321



Color	Name	Material Model	Unit Weight (kN/m³)	Cohesion (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	1. GBI or SSM	Mohr-Coulomb	21		0	30
	1. GBII / GA	Mohr-Coulomb	22		0	40
	2. CLAY (crust) TSA	Undrained (Phi=0)	17	75		
	3. CLAY TSA	Undrained (Phi=0)	17	50		
	4. TILL	Mohr-Coulomb	21		0	35
	5. BEDROCK	Bedrock (Impenetrable)				
	6. CONCRETE	High Strength	24			

THURBER

Project
24726 Highway 17 Dochart Creek

Analysis
F7. Short Term (TSA) - Static

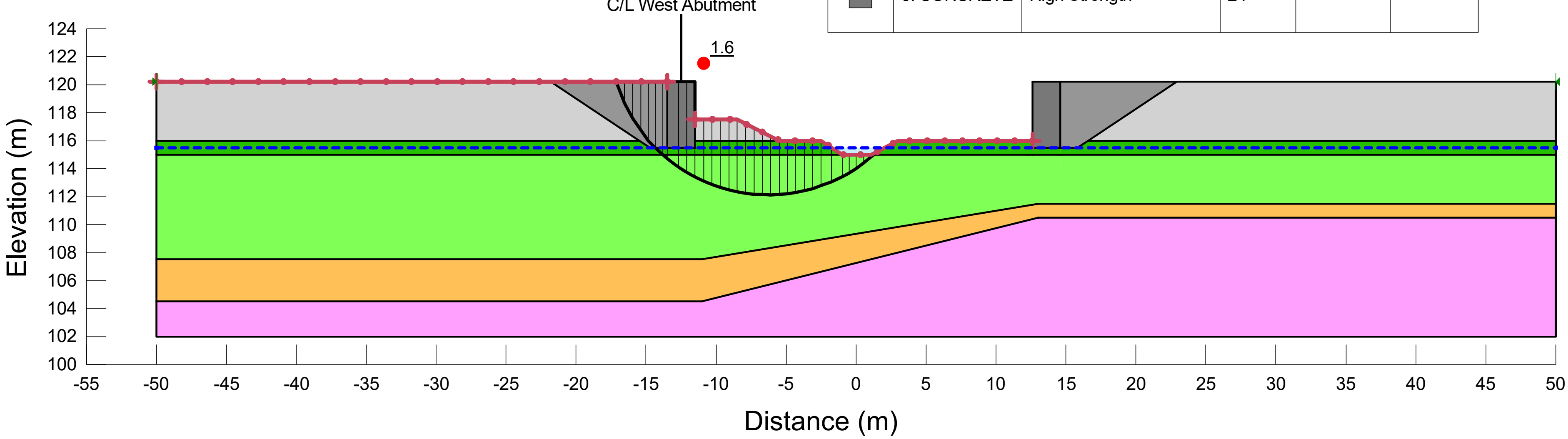
Seismic Coefficient
H: g, V: g

Last Run
07/14/2021, 06:14:42 PM

Scale
1:450

Additional Details
Name: 3 Eastbound - Forward Slope
Comments: Slope Stability Assessment
Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 0.1 m
Entry: (-22.625, 120.2) m, Exit: (0.34510673, 115) m
Center: (-9.6485303, 124.18808) m, Radius: 13.575475 m

Figure F7



Color	Name	Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	1. GBI or SSM	Mohr-Coulomb	21	0	30
	1. GBII / GA	Mohr-Coulomb	22	0	40
	2. CLAY (crust) ESA	Mohr-Coulomb	17	3	28
	3. CLAY ESA	Mohr-Coulomb	17	0	28
	4. TILL	Mohr-Coulomb	21	0	35
	5. BEDROCK	Bedrock (Impenetrable)			
	6. CONCRETE	High Strength	24		

THURBER

Project
24726 Highway 17 Dochart Creek

Analysis
F8. Long Term (ESA) - Static

Seismic Coefficient
H: g, V: g

Last Run
07/14/2021, 06:14:43 PM

Scale
1:450

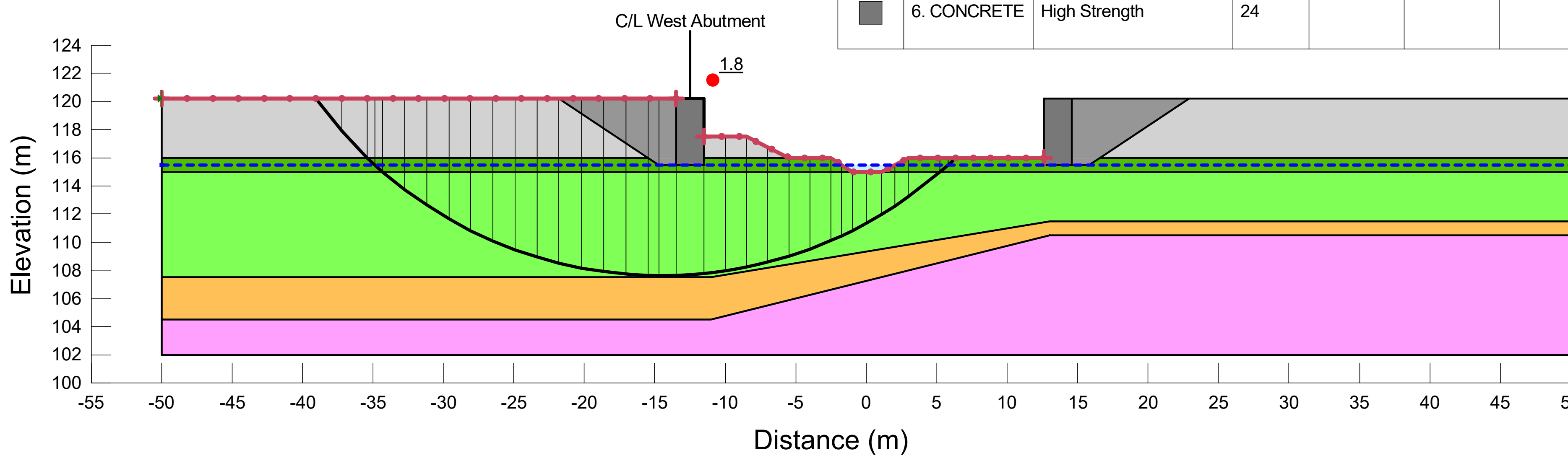
Additional Details
Name: 3 Eastbound - Forward Slope
Comments: Slope Stability Assessment
Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 0.1 m
Entry: (-17.15, 120.2) m, Exit: (1.5383046, 115.23069) m
Center: (-6.2695925, 123.4928) m, Radius: 11.367753 m

Figure F8

H:\Projects\20001 to 30000\24726 - Hwy 17 Arnprior to Haley Station\PART 1A\Foundations\Dochart Creek Culverts\Analysis\Stability\Hwy 17 Dochart_003.gsz

Tool Version: 11.1.2.22321

Color	Name	Material Model	Unit Weight (kN/m³)	Cohesion (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	1. GBI or SSM	Mohr-Coulomb	21		0	30
	1. GBII / GA	Mohr-Coulomb	22		0	40
	2. CLAY (crust) TSA	Undrained (Phi=0)	17	75		
	3. CLAY TSA	Undrained (Phi=0)	17	50		
	4. TILL	Mohr-Coulomb	21		0	35
	5. BEDROCK	Bedrock (Impenetrable)				
	6. CONCRETE	High Strength	24			



Project	24726 Highway 17 Dochart Creek
Analysis	F9. Seismic (TSA) - Pseudo-Static (2475)
Seismic Coefficient	H: 0.132g, V: 0g
Last Run	07/14/2021, 06:14:44 PM
Scale	1:450

THURBER

Additional Details
Name: 3 Eastbound - Forward Slope
Comments: Slope Stability Assessment
Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 0.1 m
Entry: (-39.05, 120.2) m, Exit: (6.3490078, 116) m
Center: (-14.525924, 137.82233) m, Radius: 30.198952 m

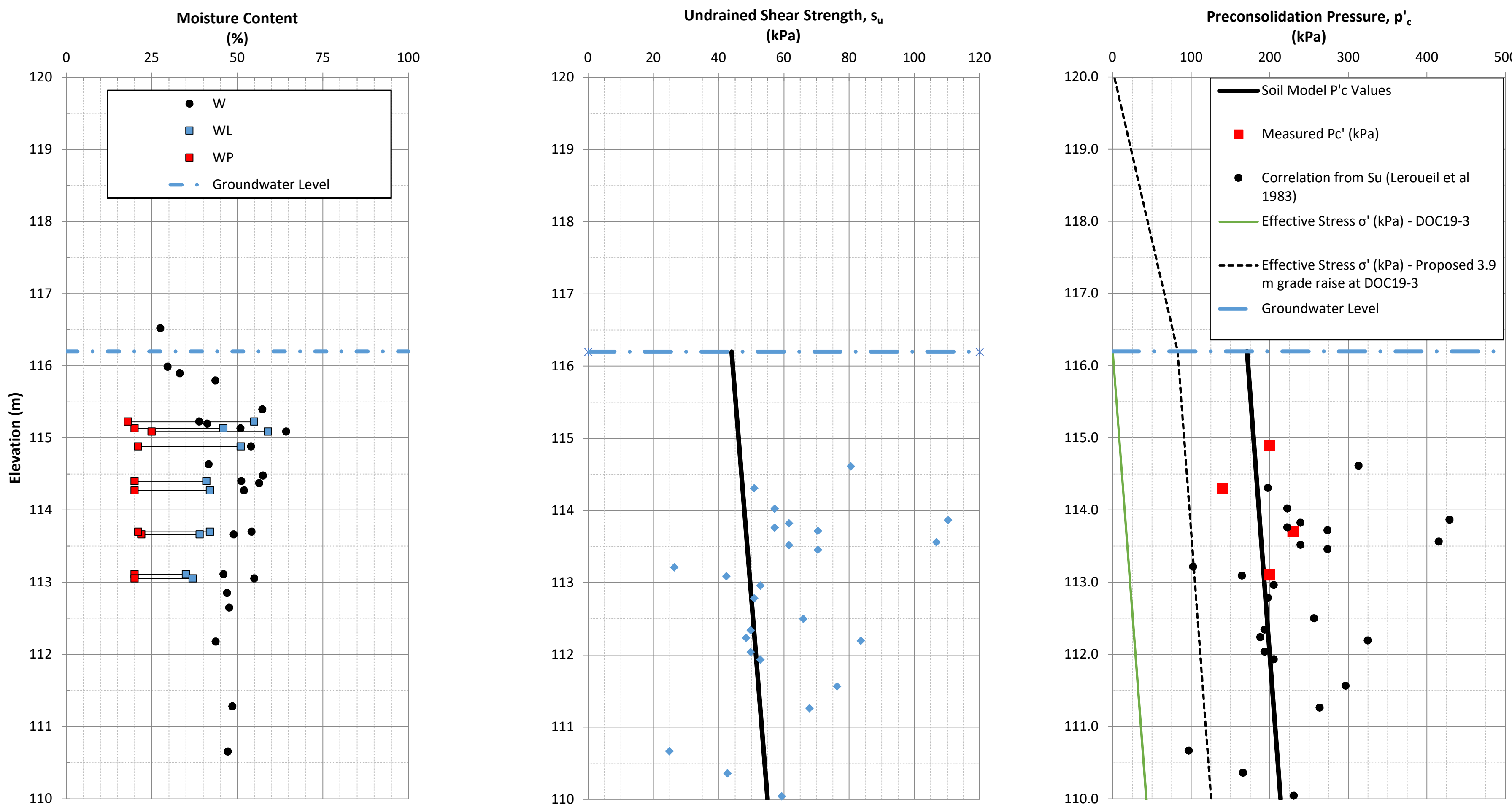
Figure F9

H:\Projects\20001 to 30000\24726 - Hwy 17 Arnprior to Haley Station\PART 1A\Foundations\Dochart Creek Culverts\Analysis\Stability\Hwy 17 Dochart_003.gsz

Tool Version: 11.1.2.22321

Appendix G.

Clay Property Summary Figures



THURBER

Clay Properties
Dochart Creek Culverts
Highway 17 Twinning
Renfrew County

W.P. 4068-09-00
Project No.: 24726

Appendix H.

P-Y Curves

SOIL P-Y CURVES **Dochart Creek**	**Table H1**

0.61m Diameter Caisson

Soil Type	**Clay**		**Till**		**Bedrock**	
P-y Curves**	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
Static	0.000	0.0	0.000	0.0	0.000	0.0
	0.000	10.0	0.002	62.8	0.000	1708.0
	0.000	19.9	0.003	76.8	0.000	1736.5
	0.001	29.9	0.003	89.3	0.000	1764.9
	0.002	39.8	0.004	100.8	0.000	1793.4
	0.005	49.8	0.005	111.5	0.001	1821.9
	0.008	59.7	0.006	121.5	0.001	1850.3
	0.012	69.7	0.006	131.1	0.001	1878.8
	0.019	79.6	0.007	140.2	0.001	1907.3
	0.026	89.6	0.008	148.9	0.001	1935.7
	0.036	99.5	0.009	157.3	0.001	1964.2
	0.048	109.5	0.009	165.5	0.001	1992.7
	0.062	119.4	0.010	173.4	0.001	2021.1
	0.079	129.4	0.017	239.3	0.001	2049.6
	0.099	139.3	0.023	305.2	0.001	2078.1
	0.122	149.3	0.027	305.2	0.001	2106.5
	0.130	149.3	0.032	305.2	0.001	2135.0

0.91m Diameter Caisson

Soil Type	**Clay**		**Till**		**Bedrock**	
P-y Curves**	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
Static	0.00E+00	0.000	0.00E+00	0.000	0.00E+00	0.000
	5.42E-05	13.053	1.37E-03	44.838	3.66E-04	2562.000
	4.34E-04	26.105	2.63E-03	66.746	4.88E-04	2604.700
	1.46E-03	39.158	3.89E-03	84.719	6.10E-04	2647.400
	3.47E-03	52.210	5.15E-03	100.501	7.32E-04	2690.100
	6.78E-03	65.263	6.41E-03	114.822	8.54E-04	2732.800
	1.17E-02	78.315	7.68E-03	128.072	9.76E-04	2775.500
	1.86E-02	91.368	8.94E-03	140.490	1.10E-03	2818.200
	2.78E-02	104.420	1.02E-02	152.237	1.22E-03	2860.900
	3.95E-02	117.473	1.15E-02	163.426	1.34E-03	2903.600
	5.42E-02	130.525	1.27E-02	174.141	1.46E-03	2946.300
	7.22E-02	143.578	1.40E-02	184.447	1.59E-03	2989.000
	9.37E-02	156.630	1.53E-02	194.395	1.71E-03	3031.700
	1.19E-01	169.683	2.48E-02	268.264	1.83E-03	3074.400
	1.49E-01	182.735	3.43E-02	342.134	1.95E-03	3117.100
	1.83E-01	195.788	4.12E-02	342.134	2.07E-03	3159.800
	1.94E-01	195.788	4.80E-02	342.134	2.20E-03	3202.500

The following assumptions were made in the analysis:
1- The analysis was completed for a vertical element (i.e. no inclination) and flat ground
2- These curves are for static loading. Seismic effects have not been included.
3- The effects of construction disturbance is not considered.
4- Depth above frost should be ignored.

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

Appendix I.

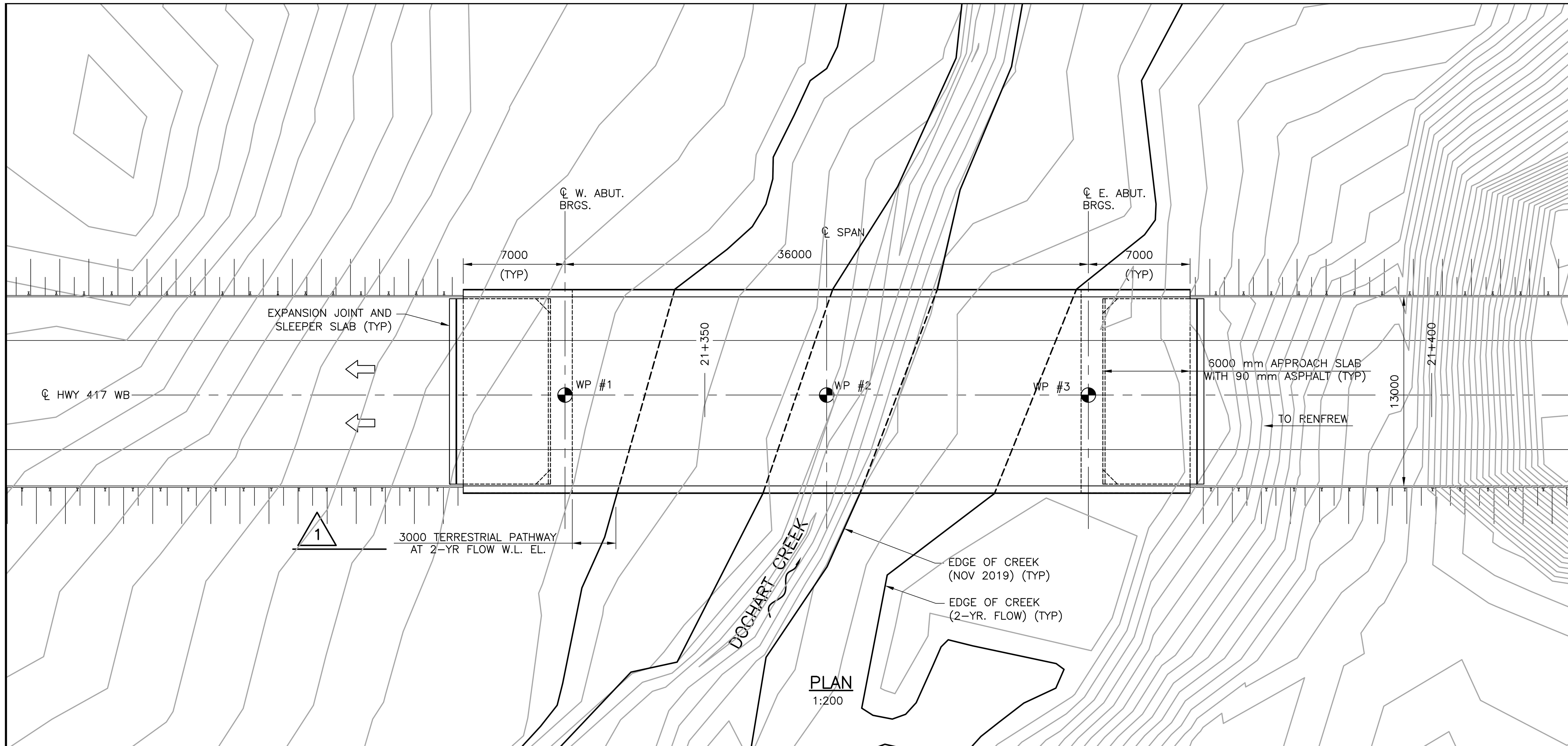
Preliminary General Arrangement (GA) Drawings
Contract Documents 2018-4018

METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

DIST. No.	
CONT. No.XXXX–XXXX	
WP. No.XXX–XXX–XX	
HWY 417 WB DOCHART CREEK BRIDGE OPTION 1A – PRECAST BOX GIRDERS GENERAL ARRANGEMENT	SHEET 01

MORRISON HERSHFIELD



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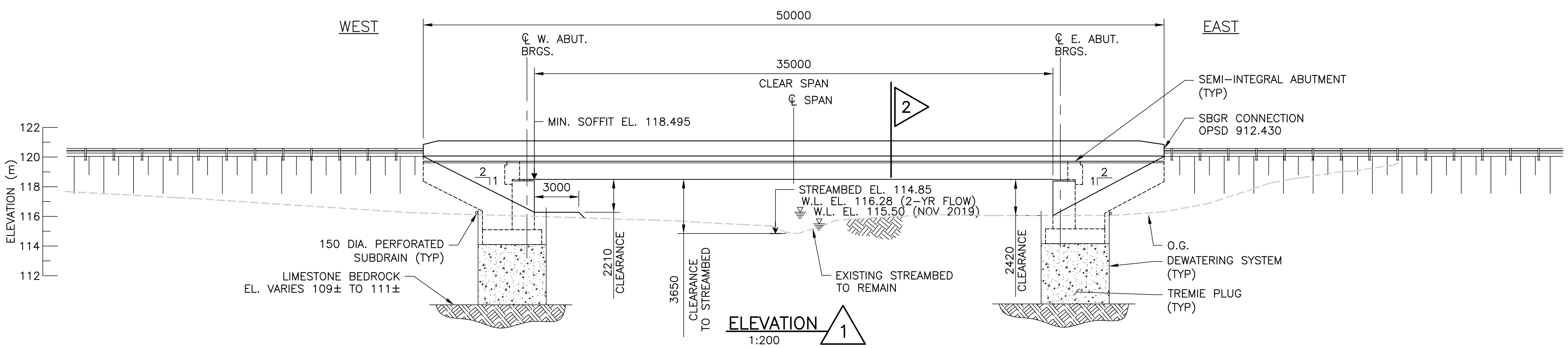
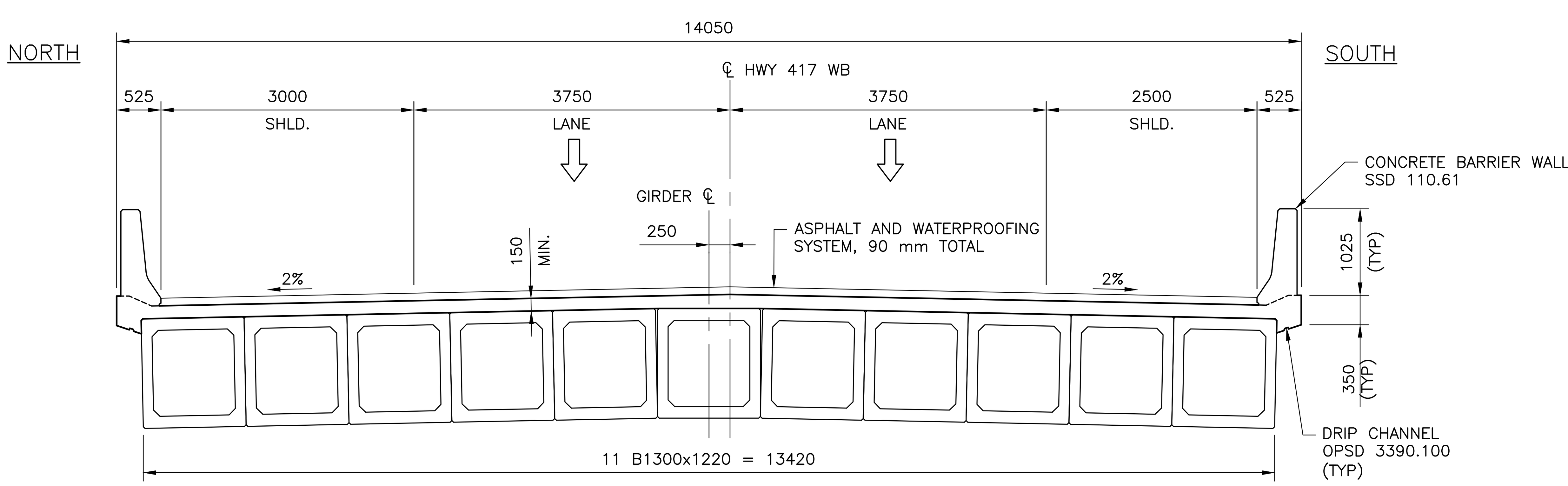


ELEVATION 1
1:200



TYPICAL SECTION 2
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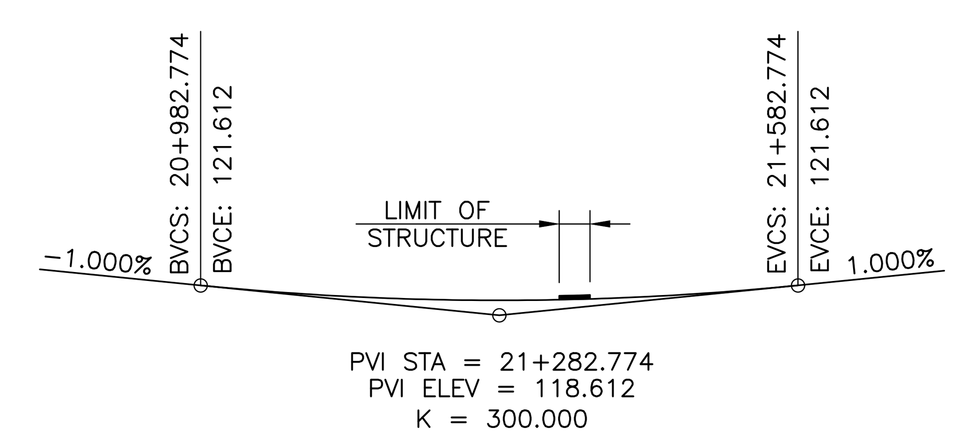

PROPOSED DESIGN PROFILE OF HWY 417 WB
N.T.S.

GENERAL NOTES:

1. CLASS OF CONCRETE: CAST–IN–PLACE 30 MPa MIN.
   PRECAST GIRDERS 50 MPa
2. CLEAR COVER TO REINFORCING STEEL:
   DECK TOPPING TOP ———— 70 ± 20
   BOTTOM ———— 40 ± 10
   FOOTINGS ———— 100 ± 25
   PRECAST GIRDERS ———— REFER TO SS107–14
   REMAINDER ———— 70 ± 20
   UNLESS NOTED OTHERWISE
3. REINFORCING STEEL:
   - REINFORCING STEEL SHALL BE GRADE 400W UNLESS SHOWN OTHERWISE.
   - STAINLESS REINFORCING STEEL SHALL BE TYPE 316LN OR DUPLEX 2205 AND HAVE A MINIMUM YIELD STRENGTH OF 500 MPa UNLESS OTHERWISE SPECIFIED.
   - BAR MARKS WITH PREFIX 'S' DENOTE STAINLESS STEEL BARS.
   - UNLESS SHOWN OTHERWISE, TENSION LAP SPLICES FOR REINFORCING STEEL BAR SHALL BE CLASS B.
   - BAR HOOKS SHALL HAVE STANDARD HOOK DIMENSIONS USING MINIMUM BEND DIAMETERS, WHILE STIRRUPS AND TIES SHALL HAVE MINIMUM HOOK DIMENSIONS. ALL HOOKS SHALL BE IN ACCORDANCE WITH THE STRUCTURAL STANDARD DRAWINGS SS12–1, UNLESS INDICATED OTHERWISE.

CONSTRUCTION NOTES:

1. THE CONTRACTOR SHALL VERIFY ALL DIMENSIONS, DETAILS AND ELEVATIONS OF THE EXISTING STRUCTURE THAT ARE RELEVANT TO THE WORK SHOWN ON THE DRAWINGS PRIOR TO COMMENCEMENT OF THE WORK. ANY DISCREPANCIES SHALL BE REPORTED TO THE CONTRACT ADMINISTRATOR.
2. THE CONTRACTOR SHALL CHECK AND IDENTIFY ALL EXISTING UTILITIES WITHIN THE WORK AREA. PRIOR TO THE CONSTRUCTION WORK, THE CONTRACTOR SHALL CARRY OUT ALL NECESSARY PROTECTION AND PRECAUTIONARY MEASURES FOR OR ARRANGE TO DIVERT EXISTING UTILITIES AS MAY BE REQUIRED BY RELEVANT AUTHORITIES.

ABBREVIATIONS:

ABUT. – ABUTMENT
B – BOTTOM
BRGS. – BEARINGS
℄ – CENTERLINE
C.J. – CONSTRUCTION JOINT
CSP – CORRUGATED STEEL PIPE
E – EAST
EB – EAST BOUND
EBL – EAST BOUND LANE
EL. – ELEVATION
EQ. – EQUAL
HWL – HIGH WATER LEVEL
HWY – HIGHWAY
LOC. – LOCATION
N.T.S. – NOT TO SCALE
O.G. – ORIGINAL GROUND
RD. – ROAD
SB – SOUTH BOUND
SBGR – STEEL BEAM GUIDE RAIL
SHLD. – SHOULDER
STA. – STATION
T – TOP
TYP – TYPICAL
T/C – TOP OF CONCRETE
T/F – TOP OF FOOTING
T/P – TOP OF PAVEMENT
U.N.O. – UNLESS NOTED OTHERWISE
W – WEST
WB – WEST BOUND
WBL – WEST BOUND LANE
WL – WATER LEVEL
WP – WORKING POINT
YR – YEAR

APPLICABLE STANDARD DRAWINGS:

- MTOD 3941.210 — FIGURES IN CONCRETE, SITE NUMBERS AND DATE
- OPSD 0912.430 — GUIDE RAIL SYSTEM, STEEL BEAM STRUCTURE CONNECTION
- OPSD 3190.100 — WALL, RETAINING WALL AND ABUTMENT, WALL DRAIN
- OPSD 3370.100 — DECK, WATERPROOFING, HOT APPLIED ASPHALT MEMBRANE WITH PROTECTION BOARD
- OPSD 3390.100 — DECK DRIP CHANNEL
- OPSD 3419.100 — BARRIERS AND RAILINGS, STEEL BEAM GUIDERAIL AND CHANNEL ANCHORAGE

LEGEND:

EARTH

BEDROCK

PRELIMINARY

WORKING POINT COORDINATES

WP #	STATION	ELEVATION	NORTHING	EASTING
1	21+340.882	120.169	5033379.630	307611.965
2	21+358.382	120.208	5033372.330	307628.418
3	21+375.882	120.257	5033365.030	307644.871

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS		
DATE	BY	DESCRIPTION

DESIGN PMA	CHK. JO	CODE CHBDC 2019	LOAD CL–625–ONT	DATE NOV 2020
DRAWN CTB	CHK. PMA	SITE 29X–0413/B2		DWG. 1

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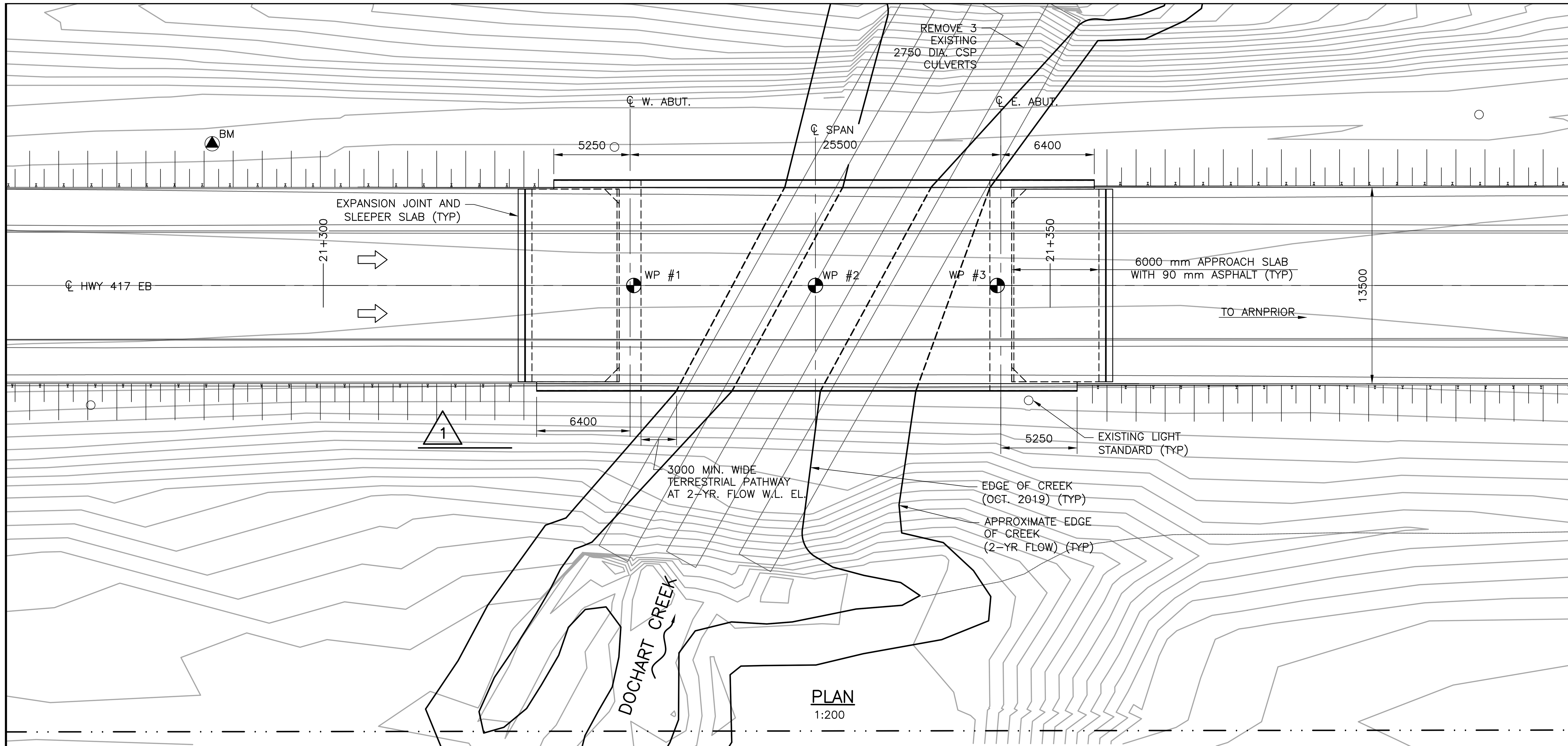
DRAWING LAYOUT: Option 1A LAST UPDATED: Dec 17, 2020–15:00
PRINTED: Dec 21, 2020–12:10

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

DIST. No.		
CONT. No.XXXX–XXXX		
WP. No.XXX–XXX–XX		
HWY 417 EB DOCHART CREEK BRIDGE GENERAL ARRANGEMENT OPTION 1A: PRECAST BOX GIRDERS	SHEET 01	

MORRISON HERSHFIELD



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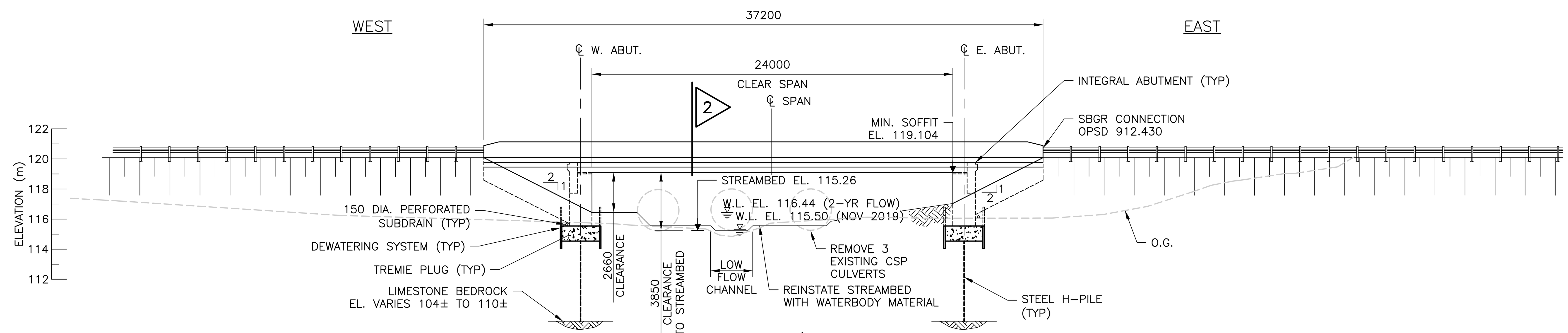
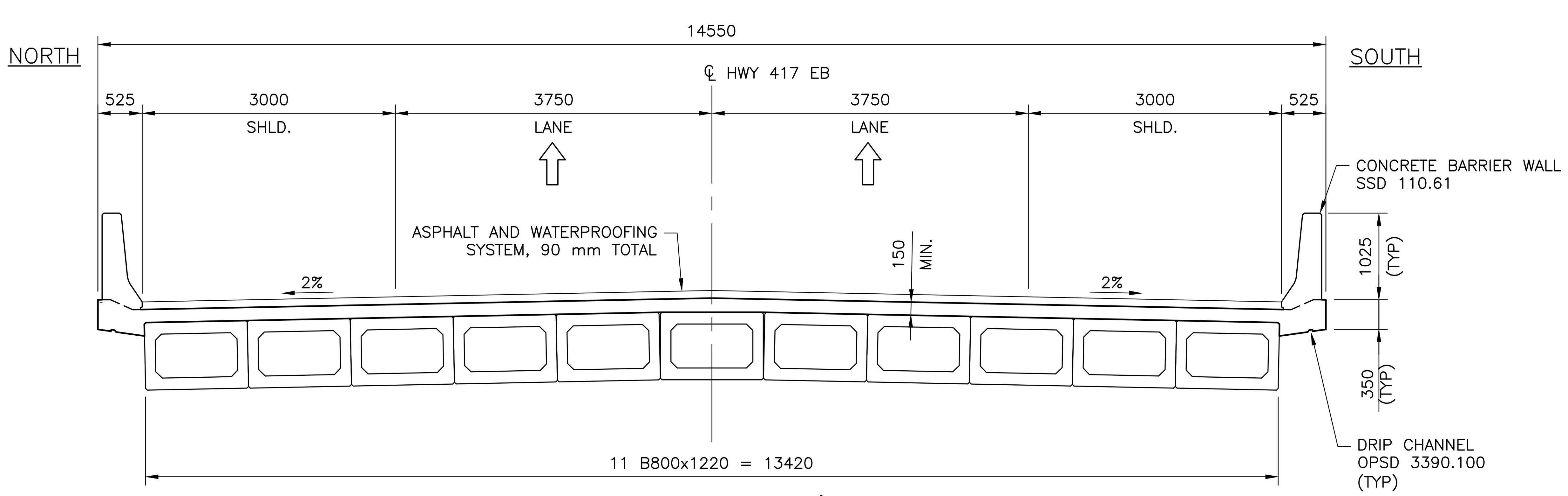


ELEVATION 1
1:200



TYPICAL SECTION 2
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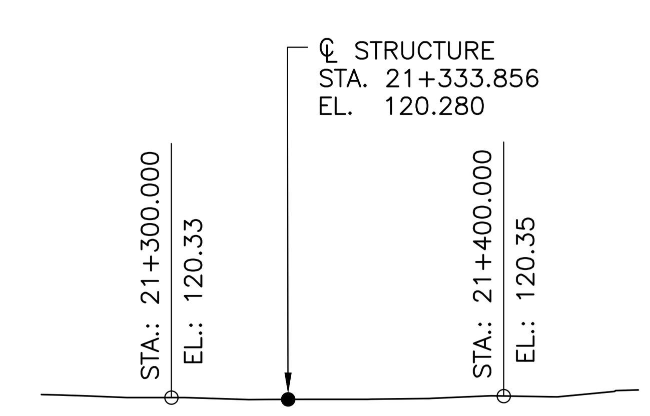

EXISTING PROFILE OF HWY 17
N.T.S.

GENERAL NOTES:

1. CLASS OF CONCRETE: CAST–IN–PLACE 30 MPa MIN. PRECAST GIRDERS 50 MPa
2. CLEAR COVER TO REINFORCING STEEL:
   DECK TOPPING TOP ———— 70 ± 20
   BOTTOM ———— 40 ± 10
   PILE CAPS ———— 100 ± 25
   PRECAST GIRDERS ———— REFER TO SS107–12
   REMAINDER ———— 70 ± 20
   UNLESS NOTED OTHERWISE
3. REINFORCING STEEL:
   - REINFORCING STEEL SHALL BE GRADE 400W UNLESS SHOWN OTHERWISE.
   - STAINLESS REINFORCING STEEL SHALL BE TYPE 316LN OR DUPLEX 2205 AND HAVE A MINIMUM YIELD STRENGTH OF 500 MPa UNLESS OTHERWISE SPECIFIED.
   - BAR MARKS WITH PREFIX 'S' DENOTE STAINLESS STEEL BARS.
   - UNLESS SHOWN OTHERWISE, TENSION LAP SPLICES FOR REINFORCING STEEL BAR SHALL BE CLASS B.
   - BAR HOOKS SHALL HAVE STANDARD HOOK DIMENSIONS USING MINIMUM BEND DIAMETERS, WHILE STIRRUPS AND TIES SHALL HAVE MINIMUM HOOK DIMENSIONS. ALL HOOKS SHALL BE IN ACCORDANCE WITH THE STRUCTURAL STANDARD DRAWINGS SS12–1, UNLESS INDICATED OTHERWISE.

CONSTRUCTION NOTES:

1. THE CONTRACTOR SHALL VERIFY ALL DIMENSIONS, DETAILS AND ELEVATIONS OF THE EXISTING STRUCTURE THAT ARE RELEVANT TO THE WORK SHOWN ON THE DRAWINGS PRIOR TO COMMENCEMENT OF THE WORK. ANY DISCREPANCIES SHALL BE REPORTED TO THE CONTRACT ADMINISTRATOR.
2. THE CONTRACTOR SHALL REFER TO THE REFERENCE DRAWINGS LISTED IN THE CONTRACT DOCUMENTS FOR DETAILS OF THE EXISTING STRUCTURE.
3. THE CONTRACTOR SHALL CHECK AND IDENTIFY ALL EXISTING UTILITIES WITHIN THE WORK AREA. PRIOR TO THE CONSTRUCTION WORK, THE CONTRACTOR SHALL CARRY OUT ALL NECESSARY PROTECTION AND PRECAUTIONARY MEASURES FOR OR ARRANGE TO DIVERT EXISTING UTILITIES AS MAY BE REQUIRED BY RELEVANT AUTHORITIES.
4. SHEETPILE COFFERDAM WILL BE LEFT IN PLACE. CONTRACTOR SHALL CUT SHEETPILE FLUSH WITH TOP OF TREMIE PLUG CONCRETE.
5. LOW FLOW CHANNEL TO BE FINALIZED DURING DETAILED DESIGN BY FLUVIAL GEOMORPHOLOGIST.

LEGEND:

EARTH

BEDROCK

PROPERTY LINE

ABBREVIATIONS:

- ABUT. – ABUTMENT
- B – BOTTOM
- BM – BENCHMARK
- BRGS. – BEARINGS
- ℄ – CENTERLINE
- C.J. – CONSTRUCTION JOINT
- CSP – CORRUGATED STEEL PIPE
- E – EAST
- EB – EAST BOUND
- EBL – EAST BOUND LANE
- EL. – ELEVATION
- EQ. – EQUAL
- HWL – HIGH WATER LEVEL
- HWY – HIGHWAY
- LOC. – LOCATION
- N.T.S. – NOT TO SCALE
- O.G. – ORIGINAL GROUND
- RD. – ROAD
- SB – SOUTH BOUND
- SBGR – STEEL BEAM GUIDE RAIL
- SHLD. – SHOULDER
- STA. – STATION
- T – TOP
- TYP – TYPICAL
- T/C – TOP OF CONCRETE
- T/F – TOP OF FOOTING
- T/P – TOP OF PAVEMENT
- U.N.O. – UNLESS NOTED OTHERWISE
- W – WEST
- WB – WEST BOUND
- WBL – WEST BOUND LANE
- WL – WATER LEVEL
- WP – WORKING POINT
- YR – YEAR

APPLICABLE STANDARD DRAWINGS:

- MTOD 3941.210 — FIGURES IN CONCRETE, SITE NUMBERS AND DATE
- OPSD 0912.430 — GUIDE RAIL SYSTEM, STEEL BEAM STRUCTURE CONNECTION
- OPSD 3190.100 — WALL, RETAINING WALL AND ABUTMENT, WALL DRAIN
- OPSD 3370.100 — DECK, WATERPROOFING, HOT APPLIED ASPHALT MEMBRANE WITH PROTECTION BOARD
- OPSD 3390.100 — DECK DRIP CHANNEL
- OPSD 3419.100 — BARRIERS AND RAILINGS, STEEL BEAM GUIDERAIL AND CHANNEL ANCHORAGE

PRELIMINARY

WORKING POINT COORDINATES

WP #	STATION	ELEVATION	NORTHING	EASTING
1	21+321.106	120.282	5033334.189	307563.675
2	21+333.856	120.280	5033330.028	307575.462
3	21+346.606	120.278	5033325.868	307587.250

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS	DATE	BY	DESCRIPTION	
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PRINTED: Dec 21, 2020–11:08 LAST UPDATED: Dec 21, 2020–10:36

BM EL. 119.879

METRIC

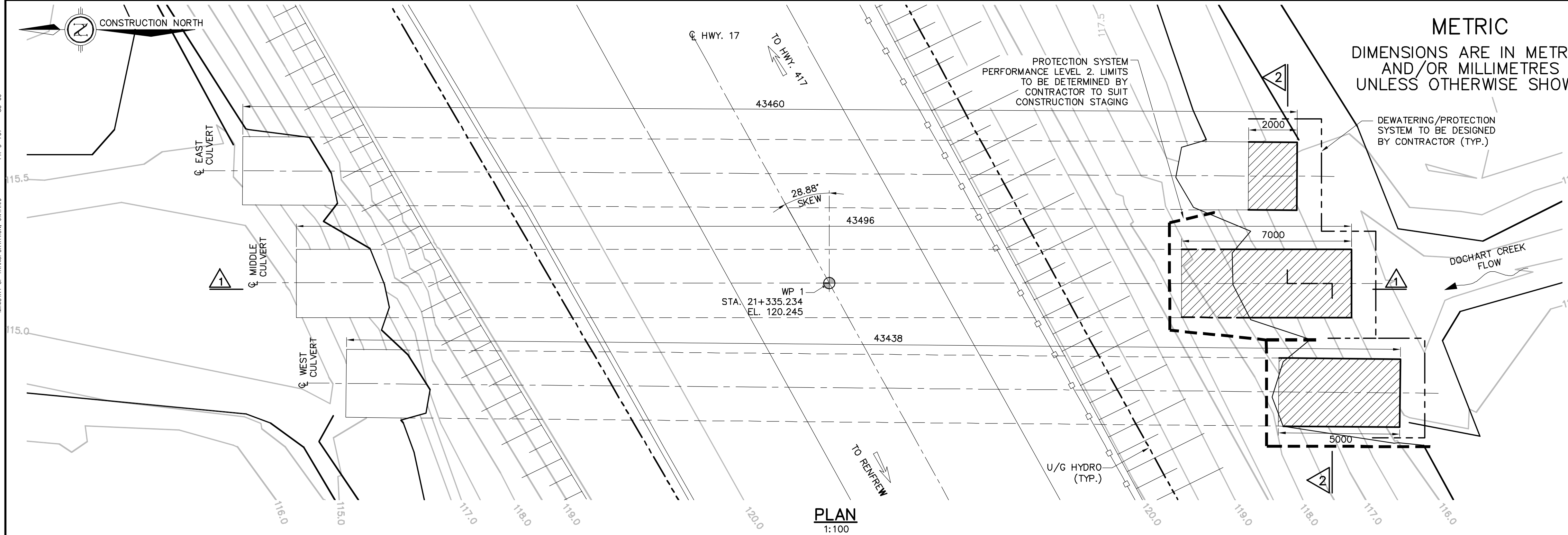
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

HWY. 17
CONT No
WP No

HWY. 17
DOCHART CREEK CULVERT REHABILITATION
GENERAL ARRANGEMENT

SHEET

LEA McINTOSH PERRY Joint Venture



PLAN
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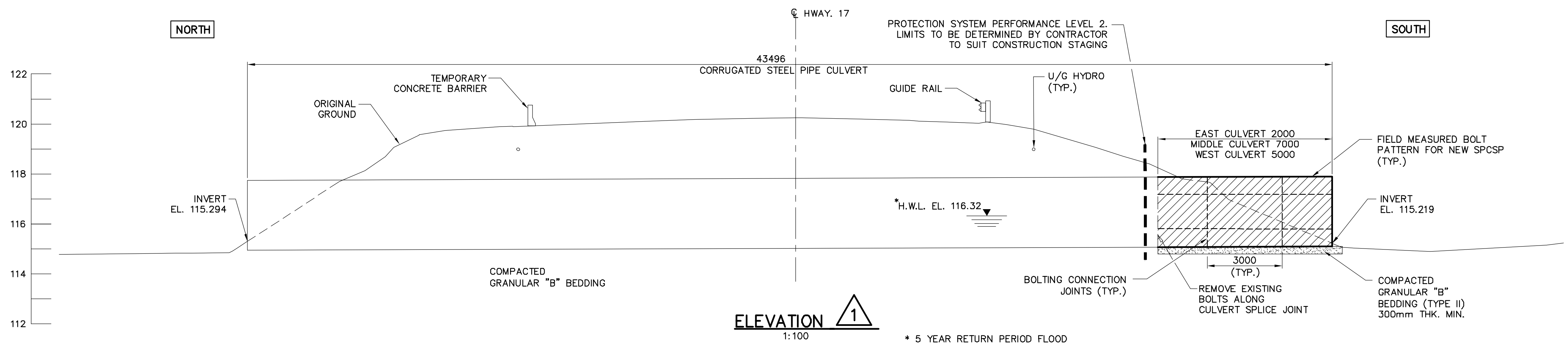
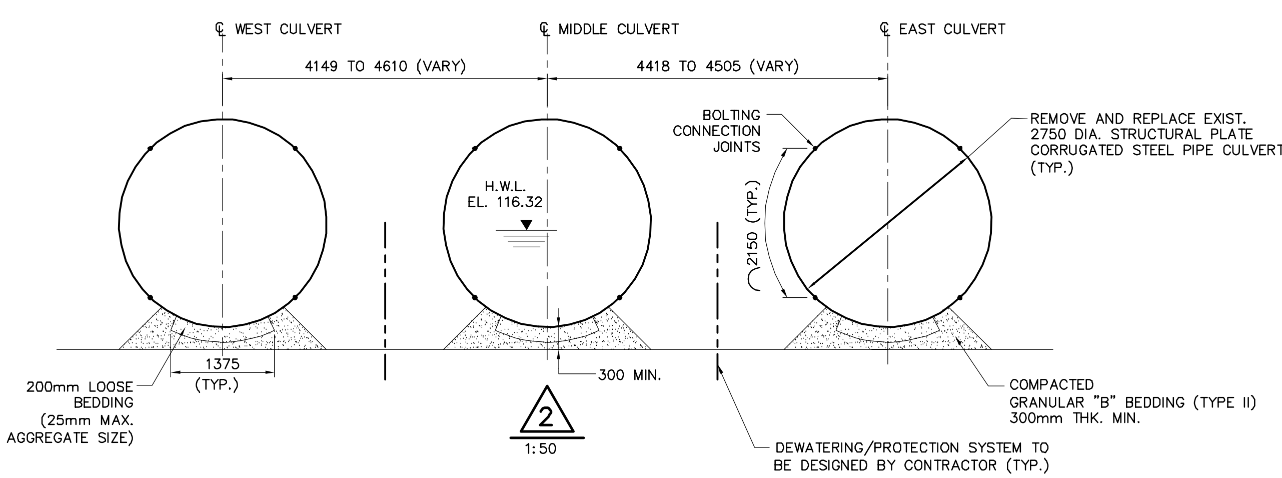


ELEVATION 1
1:100

* 5 YEAR RETURN PERIOD FLOOD



2
1:50

COORDINATES OF WORK POINTS

WP #	STATION	NORTH COORDINATE	EAST COORDINATE
1	21+335.234	5033329.658	307576.593

GENERAL NOTES:

CORRUGATED STEEL PIPES:

1. ALL STRUCTURAL CORRUGATED STEEL PIPES SHALL CONFORM TO CSA G401.
2. A325 BOLTS, TYPE 3, SHALL BE USED FOR ALL CONNECTIONS.
3. ALL CORRUGATED METAL CULVERTS MUST BE DESIGN FOR 3.0 FILL AND OVER ACCORDING TO CHBDC.

CONSTRUCTION NOTES:

1. CONTRACTOR SHALL VERIFY ALL DIMENSIONS OF EXISTING WORK AND ALL DETAILS ON SITE AND REPORT ANY DISCREPANCIES TO THE CONTRACT ADMINISTRATOR BEFORE PROCEEDING WITH THE WORK.
2. ALL STEEL PLATE REMOVAL SHALL BE DONE IN SEGMENTS. NEW CORRUGATED STEEL PLATES SHALL BE ONLY BOLTED TO SOUND STEEL PLATE WITHOUT ANY CORROSION OR PERFORATIONS.
3. THE CONTRACTOR SHALL BE RESPONSIBLE FOR PROTECTING THE EXISTING CULVERTS DURING THE CONSTRUCTION PROCESS.
4. THE COFFERDAMS INDICATED ARE SCHEMATIC ONLY. THE COMPLETE DETAILED DESIGN SHALL BE THE CONTRACTOR'S RESPONSIBILITY.
5. THE CONTRACTOR SHALL BE RESPONSIBLE FOR THE PROTECTION OF ALL UTILITIES TO SUIT THE WORK ON SITE.
6. THE FINAL LENGTH FOR REMOVAL OF DETERIORATED CORRUGATED STEEL PLATES OF EACH BARREL WILL BE CONFIRMED DURING THE CONSTRUCTION.
7. BACKFILL MATERIAL SHALL BE PLACED IN 200mm THICK LOOSE LAYERS AT CONSTANT GRADE.

LEGEND:

REMOVE AND REPLACE CORRUGATED STEEL PIPE

APPLICABLE STANDARD DRAWINGS:

OPSD 801.010 CUT END FINISH CIRCULAR PIPE AND PIPE-ARCH CORRUGATED STEEL PIPE

LIST OF DRAWINGS:

1. GENERAL ARRANGEMENT
2. CONSTRUCTION STAGING

LIST OF ABBREVIATIONS:

℄	CENTRE LINE
EL.	ELEVATION
STA.	STATION
T/	TOP OF
T/A	TOP OF ASPHALT (FINISHED ELEV.)
TYP.	TYPICAL
WP	WORKING POINT
W.L.	WATER LEVEL

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS | DATE | BY | DESCRIPTION

DESIGN FS	CHK RTK	CODE CHBDC 14	LOAD ONT CL-625	DATE APRIL 2018
DRAWN RM	CHK FS	SITE 29-413/C1	STRUCT –	SCHEME –	DWG 1

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

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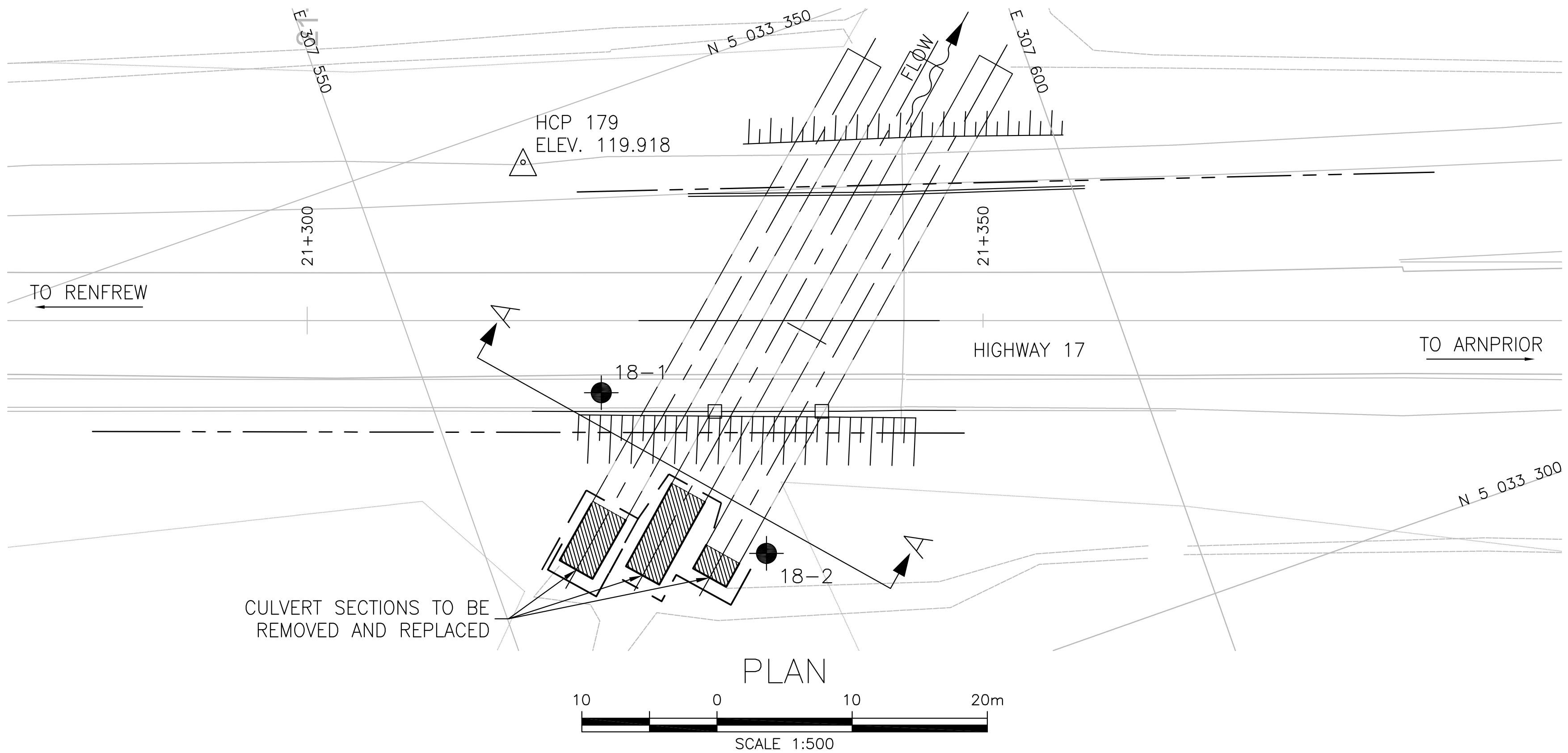
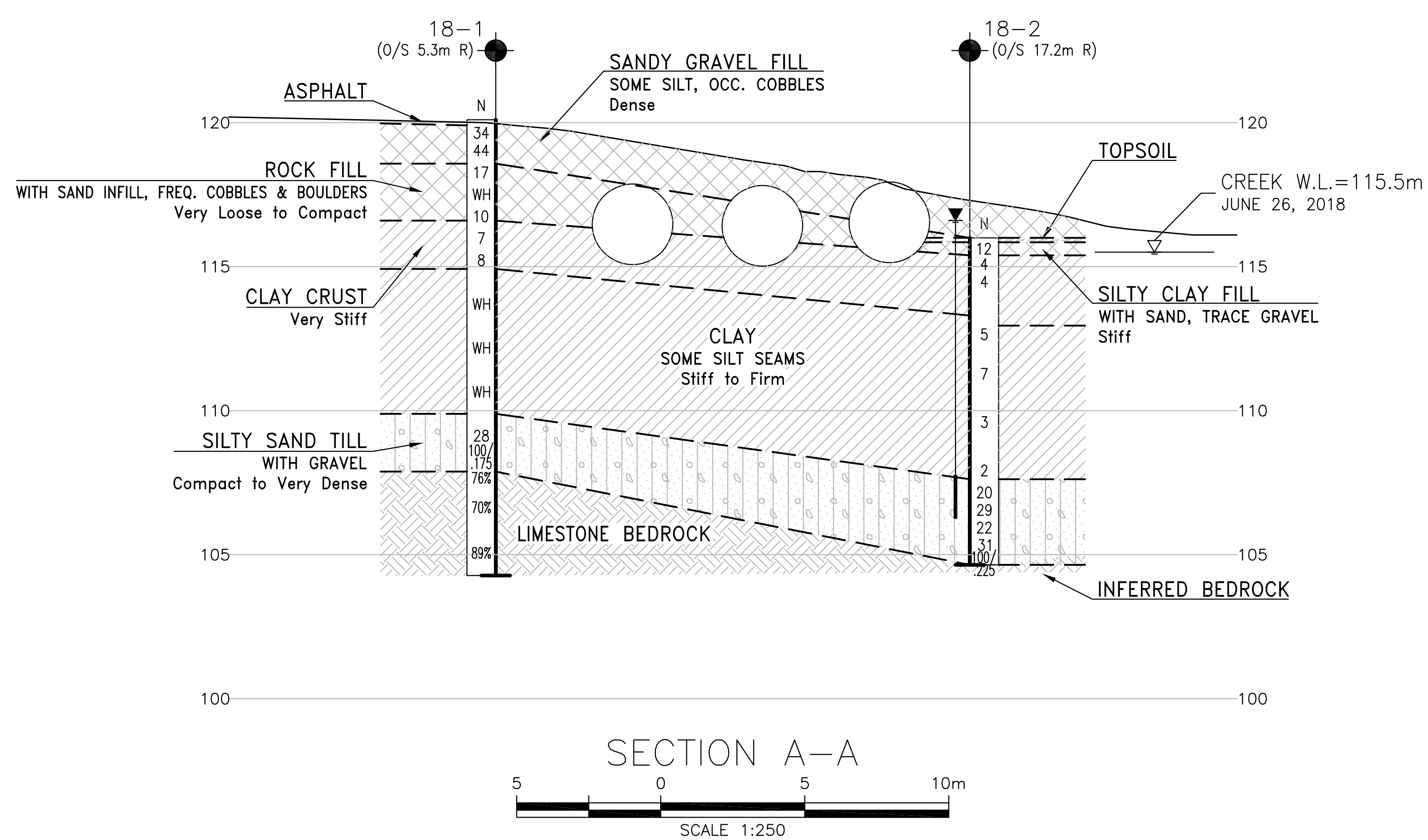




METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

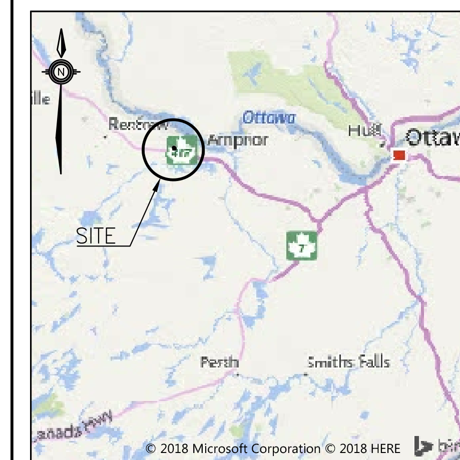
CONT No 2018-4018
WP No 4005-17-01

HIGHWAY 17
DOCHART CREEK
CULVERT REHABILITATION
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET 9

McINTOSH PERRY

THURBER ENGINEERING LTD.



KEYPLAN

LEGEND

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

NO	ELEVATION	NORTHING	EASTING
18-1	120.2	5 033 329.1	307 562.1
18-2	116.0	5 033 313.8	307 569.7

LICENSED PROFESSIONAL ENGINEER
F. J. GRIFFITHS
Sept 20/18
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
Sep 20/18
PROVINCE OF ONTARIO

-NOTES-

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

GEOCRES No. 31F-204

REVISIONS

DATE	BY	DESCRIPTION

DESIGN KP	CHK -	CODE	LOAD	DATE SEP 2018
DRAWN MFA	CHK KP	SITE29x-0413/C	STRUCT	DWG R2

FILENAME: H:\Projects\20001 to 30000\22912 - Dochart Creek Culvert\CAD\From Oakville 2018-09-20\TED-22912-BHPP-DC.dwg
PLOTDATE: 9/20/2018 10:35 AM

Appendix J.

List of Special Provisions and
OPSS Documents Referenced in this Report
Non-Standard Special Provisions

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

OPSD 803.010	Backfill and Cover for Concrete Culverts with Spans Less than or Equal to 3.0m
OPSD 810.010	General Rip-Rap Layout for Sewer and Culvert Outlets
OPSD 3090.101	Foundation Frost Depths for Southern Ontario
OPSD 3101.150	Abutment Backfill Minimum Granular Requirement
OPSS.PROV 180	Construction Specification for the Management of Excess Materials
OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS.PROV 804	Construction Specification for Seed and Cover
OPSS 422	Construction Specification for Precast Reinforced Concrete Box Culverts in Open Cut
OPSS 805	Construction Specification for Temporary Erosion and Sediment Control Measures
OPSS 902	Construction Specification for Excavating and Backfilling Structures
OPSS.PROV 1010	Material Specification for Aggregates Base, Subbase, Select Subgrade, and Backfill Material
OPSS.PROV 1205	Material Specification for Clay Seal
OPSS.PROV 1860	Material Specification for Geotextiles
SP 109S12	Amendment to OPSS 902 - QVE, Backfilling Compaction, and Certificate of Conformance
SP FOUN0003	Amendment to OPSS 902 – Dewatering Structure Excavations

2. Suggested wording for NSSPs

“Notice to Contractor: Protection of Sensitive Foundation Soils ”

The Contractor is advised that the native clayey silt and clay that will be exposed at the subgrade is moisture sensitive and may become disturbed or otherwise negatively impacted when subjected to construction or personnel traffic, freeze-thaw actions, ingress or ponding water. The Contractor shall be responsible for implementing adequate groundwater control measures and for protecting the subgrade.

“Notice to Contractor: Obstructions”

The Contractor is hereby notified that the native discontinuous tills at the site and as inferred from available information should be expected to contain cobbles and boulders. Considerations of these obstructions must be made in the selection of appropriate equipment and procedures for excavations, installations of deep foundations and temporary protection systems.

“Integral Abutment CSP and Sand Backfill”

The sand backfill used within the CSP to provide the required flexibility for the piles in the integral abutment design shall meet the following gradation envelope. Note piles should be driven first before placing the sand backfill in the CSP.

Integral Abutment Sand Backfill Grading

MTO Sieve Designation	Percent Passing (%)
#10	100
#30	80 – 100
#40	40 – 80
#60	5 – 25
#100	0 – 6