



THURBER ENGINEERING LTD.

**FOUNDATION INVESTIGATION AND DESIGN REPORT
REPLACEMENT OF STRUCTURAL CULVERT NO. 38S-243
HIGHWAY 17 LAUZON CREEK CULVERT
TOWNSHIP OF LONG, ON
G.W.P. 5074-09-00
AGREEMENT NUMBER: 5016-E-0043**

GEOCRES NUMBER: 41J-114

**SUBMITTED TO
McINTOSH PERRY CONSULTING ENGINEERS**

**LOCATION:
LATITUDE: 46.18885°
LONGITUDE: -82.80792°**

**AUGUST 2018
19511**



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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual data obtained from a foundation investigation conducted by Thurber Engineering Ltd. (Thurber) for the replacement of the Lauzon Creek Culvert located on Highway 17, within the Township of Long, Ontario. Thurber carried out the investigation as a subconsultant to McIntosh Perry Consulting Engineers (MPCE) under Agreement No. 5016-E-0043.

No previous foundation investigation information for the subject culvert was available. A base plan and a Preliminary General Arrangement Drawing (GA) were provided by MPCE for the preparation of this report.

The purpose of this investigation was to explore the subsurface conditions at the site and, based on this data, provide a borehole location plan, record of boreholes, a stratigraphic profile, laboratory test results and a written description of the subsurface conditions.

2 SITE DESCRIPTION

Culvert 38S-243 is located at approximate Station 10+943 on Highway 17, approximately 0.7 km east of the west junction of Highway 538 in the District of Algoma Mills, Ontario. The location of the culvert is shown on the inset Key Plan on Drawing No. 1 in Appendix A.

It is noted that for project orientation purposes, Highway 17 within the project limits, will be assumed to run west-east. Flow through the culvert is from north to south.

The existing 26.8 m long reinforced concrete arch culvert has an internal span of 12.2 m and height of 4.4 m. There is also a retained soil system (RSS) located on the east and west sides of the culvert at both the culvert inlet and outlet. Archival drawings indicate that the arch culvert was designed to be founded on 1.2 m wide by 1.5 m deep footings with a top of footing elevation of approximately 179.2 m. The streambed elevation is at approximately 179.2 m. The drawings also indicate that east footing appears to be founded on bedrock while the west footing is founded on hard gravel.

At the culvert site, Highway 17 is undivided with one through lane in each direction. The August 2018 GA Drawing indicates that the roadway cross-section at the culvert location consists of two, 3.75 m wide lanes with 3.0 m wide paved shoulders. A steel beam guide rail and concrete curb and gutters are present along both sides of the highway. There is a private entrance on the south side of the highway, approximately 20 m east of the existing culvert.

The existing embankment slopes below the existing RSS system are inclined at approximately 2H:1V to 2.5H:1V (Horizontal:Vertical) and are grass and brush covered; some trees are also present. Cobbles and boulders were observed on the lower slopes and within the creek. The GA drawing indicates that the top of an asphalt surface of the highway is at elevation 184.5 m and that cover over the existing culvert from shoulder to the top of the culvert is approximately 0.8 m. The storm water drainage in the area is to existing culverts, ditchlines and Lauzon Creek.

Site photographs showing the general conditions at the site, along the highway embankment, and at the inlet and outlet are presented in Appendix D.

2.1 Regional Geology

Based on the 1:250 000 Scale Bedrock Geology Mapping of Ontario-Revision 1 (MRD126-REV1), the area is bounded by two main bedrock formations. The regional bedrock geology includes quartz-felspar, sandstone, argillite conglomerate and felsic intrusive rocks including granite, alkali granite, granodiorite, quartz feldspar; and minor related volcanic rocks.

The regional surficial geology of the area consists mainly of glaciofluvial deposits of sand and gravel, including shallow water, glaciolacustrine and glaciomarine deposits (Ministry of Northern Development and Mines, Map 2518 Surficial Geology of Northern Ontario)

3 SITE INVESTIGATION

3.1 Field Investigation

The field investigation was carried out between December 4th and December 20th, 2017, and included advancing six boreholes. Due to equipment breakdown Borehole 17-03 was terminated prior to reaching the required depth so an additional borehole, Borehole 17-03B was advanced approximately 1.0 m to the west of Borehole 17-03. The approximate MTM Zone 13 locations and ground surface elevations of the boreholes are shown on Drawing No. 1, provided in Appendix A and are summarized in Table 3-1.

Table 3-1: Borehole Summary

Borehole	Location	Northing (m)	Easting (m)	Ground Surface Elevation (m)	Borehole Termination Elevation (m)
17-01	Eastbound	5117243.4	396787.1	184.4	174.8
17-02	Westbound	5117253.8	396793.8	184.6	173.2
17-03	Eastbound	5117247.4	396796.4	184.4	173.7
17-03B	Eastbound	5117246.6	396795.7	184.4	172.0
17-04	Westbound	5117261.4	396812.8	184.7	174.5
17-05	Eastbound	5117255.2	396815.4	184.5	173.7
17-06	Westbound	5117265.8	396823.5	184.7	176.9

As a component of our standard procedures and due diligence, Thurber contacted Ontario One Call to provide utility locates/clearances for the intended borehole locations.

The boreholes were advanced with a CME750 truck mounted drill rig.

The subsurface stratigraphy encountered in the boreholes was recorded in the field by Thurber personnel. Split spoon samples were collected at regular depth intervals in all boreholes during the completion of Standard Penetration Tests (SPT), following the methods described in ASTM Standard D1586-11. In-situ shear vane testing was carried out within the cohesive strata. All soil samples recovered from the boreholes were placed in moisture-proof containers and the samples were transported to Thurber's Ottawa geotechnical laboratory for further examination and testing. Bedrock was cored following ASTM Standard D6032-08 in Boreholes 17-02 through 17-05, including 17-03B, with NQ size coring equipment. Bedrock core samples were stored in core boxes for transport.

The boreholes were backfilled with low-permeability bentonite pellets in accordance with Ontario MOE Regulation 903 as amended. All boreholes were capped with 150 mm of cold patch asphalt.

The as-drilled locations of the boreholes and ground surface elevations at the borehole locations were surveyed by Thurber. The vertical datum used was the Temporary Benchmark No. 301 (TBM) provided by MPCE and located in the southeast embankment near the culvert outlet. The TBM has a geodetic elevation of 185.366 m. The location of the TBM is indicated on Drawing No. 1 in Appendix A.

3.2 Laboratory Testing

Geotechnical laboratory testing consisted of natural moisture content determination and visual identification of all soil samples in accordance with the current MTO standards. Grain size distribution analyses, and Atterberg Limits testing were also carried out on selected samples to MTO and ASTM standards. All rock core was logged and core recoveries and RQD values were measured. Unconfined compression strength testing was carried out on select samples of the recovered bedrock.

The geotechnical laboratory test results are presented on the Record of Borehole sheets in Appendix B and are illustrated on the figures in Appendix C.

Chemical analysis for determination of pH, resistivity, soluble sulphate and chloride concentrations was carried out on two soil samples. A copy of the chemical analysis results is provided in Appendix C.

4 DESCRIPTION OF SUBSURFACE CONDITIONS

4.1 Overview / General

Reference is made to the Record of Borehole sheets in Appendix B for details of the soil stratigraphy encountered in the boreholes. Stratigraphic profiles for the culvert area are presented on Drawing Nos. 1 and 2 in Appendix A for illustrative purposes. An overall description of the stratigraphy is given in the following paragraphs; however, the factual data presented in the Record of Boreholes governs any interpretation of the site conditions.

In general, the stratigraphy in the area of the boreholes is characterized by embankment fill overlying sandy clay (east side of culvert), overlying silty sand with gravel till underlain by granite bedrock. Cobbles and boulders were noted in both embankment fill and the underlying glacial till layer.

More detailed descriptions of the individual strata are presented below.

4.2 Asphalt

All boreholes were advanced through the Highway 17 lane or shoulder pavement structure. The thickness of the asphalt at the borehole locations ranged from 100 mm to 150 mm.

4.3 Fill

A fill layer comprising the pavement structure and the underlying embankment fill and consisting predominantly of sand and varying amounts of gravel and silt was encountered in all boreholes. The top of this layer ranges from Elevation 184.3 m to 184.6 m. The thickness of the layer ranged from 1.9 m to 6.4 m. The SPT N values ranged from 3 to greater than 100, indicating a very loose to very dense condition, typically loose to compact. Occasional cobbles were noted in this layer. Trace organics were observed in Borehole 17-04 below a depth of 3.0 m. Portions of the fill in Borehole 17-05 were clayey.

The moisture content of the samples tested ranged from 1% to 33%. The results of five grain size analysis tests indicate a gravel content of 23% to 43%, a sand content of 29% to 57%, and a fines content (combined silt and clay size particles) of 16% to 32%. Grain size analysis results are illustrated on Figure 1 in Appendix C. An Atterberg Limits test on the fines from a fill sample in Borehole 17-05 indicated that the fines to be a silty clay (CL-ML). The Atterberg Limits test result is illustrated on Figure 2 in Appendix C.

4.4 Sandy Silty Clay (CL-ML) to Clay (CL to CI)

A reddish grey clay deposit with varying amounts of sand was encountered in Boreholes 17-04 to 17-06. The top of this layer ranges from Elevation 180.9 m to 182.8 m. In-situ shear vane test results indicated undrained shear strengths ranging from 75 kPa to greater than 100 kPa; indicating a stiff to very stiff consistency.

The moisture content of the samples tested ranged from 22% to 31%. The results of three grain size analysis tests indicated a gravel content ranging from 0% to 6%, sand content ranging from 2% to 40%, a silt content ranging from 40% to 63% and a clay content ranging from 14% to 38%. Grain size analysis results are illustrated on Figure 3 in Appendix C.

The results of three Atterberg Limits tests completed on this material indicated a liquid limit ranging from 18 to 40, a plastic limit ranging from 13 to 18, and a plasticity index ranging from 5 to 22. Atterberg Limits analysis results are illustrated on Figure 4 in Appendix C, and indicate a silty clay (CL-ML) to a clay with a low to intermediate plasticity (CL to CI).

4.5 Glacial Till – Silty Sand (SM) to Sandy Silt (ML)

A glacial till deposit consisting of a heterogeneous mixture of sand, silt, gravel, cobbles and boulders was encountered beneath the embankment fill materials in Boreholes 17-01 through 17-03B and below the clay layers in Boreholes 17-04 through 17-06. The till is classified as a silty sand with gravel to sandy silt with gravel.

The top of this layer ranges from Elevation 178.0 m to 180.9 m. The thickness of this layer ranged from 2.4 m to 4.0 m. The SPT N values ranged from 13 to greater than 100 indicating a compact to very dense condition, but typically dense. Frequent cobbles and boulders were noted in all boreholes. Coring techniques were required to penetrate through the cobbles and boulders at several locations; these locations are indicated on the Record of Borehole sheets.

The moisture content of the samples tested ranged from 1% to 17%. The results of five grain size analysis tests indicated a gravel content ranging from 5% to 38%, sand content ranging from 26 to 50%, and a fines content ranging from 12% to 69%. Grain size analysis results are illustrated on Figure 5 in Appendix C.

The results of Atterberg Limit testing carried out on three samples of this material indicate the glacial till be to be non-plastic.

4.6 Granite Bedrock

The overburden materials were underlain by red to grey granite bedrock. Boreholes 17-02 through 17-05 were advanced into the bedrock by coring with NQ-size coring equipment. The bedrock elevation near the west culvert footing line ranged from Elevation 175.1 m to 176.3 m and from Elevation 176.4 m to 177.7 m near the east footing line. Photographs of the bedrock core are provided in Appendix B.

A summary of the bedrock surface elevation is provided in Table 4-1.

Table 4-1: Bedrock Summary

Borehole	Location	Ground Surface Elevation (m)	Depth Below Existing Grade (m)	Top of Bedrock Elevation (m)
17-02	West Footing Line	184.6	8.3	176.3
17-03		184.4	8.8	175.6
17-03B		184.4	9.3	175.1
17-04	East Footing Line	184.7	7.0	177.7
17-05		184.5	8.2	176.3

The total core recovery was 100%, the solid core recovery ranged from 17% to 100% and the Rock Quality Designation (RQD) ranged from 50% to 91% except one value of 17% in Borehole 17-03. Unconfined compressive strength testing was carried out on four samples of the bedrock; (see results in Appendix C). The results ranged from 76 MPa to 242 MPa. Based on the RQD value the bedrock is classified as fair to excellent quality. Based on unconfined compression strength testing the bedrock is strong to very strong.

4.7 Groundwater Conditions

Groundwater levels measured in the open boreholes were not considered representative due to the introduction of water into the boreholes during coring operations.

The water level in the Lauzon Creek Culvert was measured at the time of Thurber's field investigation at depths ranging from 5.1 m and 5.5 m below the top of the roadway; corresponding to elevations ranging from 179.7 m to 179.4 m. The archival drawings indicate a highwater level of the creek at approximately Elevation 179.6 m.

These observations are considered short-term readings and seasonal fluctuations of the groundwater level and water level in the culvert are to be expected. In particular, the groundwater level may be at a higher elevation after the spring snowmelt or after periods of heavy rainfall.

4.8 Analytical Test Results

Two samples of the native soils encountered at the site were submitted to Paracel Laboratories in Ottawa, Ontario for analysis of pH, water soluble sulphate and chloride concentrations, and resistivity. The analysis results are summarized in the Table 4-2. A copy of the test results is provided in Appendix C.

Table 4-2: Results of Chemical Analysis

Borehole	Sample	Depth (m)	pH	Resistivity (Ohm-cm)	Chloride (µg/g)	Sulphate (µg/g)
17-02	SS6	4.1	5.5	1270	547	48
17-02	SS9	7.9	7.2	9340	30	12

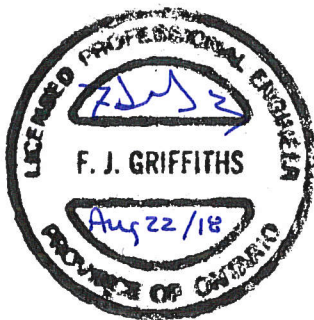
5 MISCELLANEOUS

Thurber staked and/or marked the borehole locations in the field and obtained utility clearances prior to drilling. Thurber surveyed the borehole locations and determined the ground surface elevations based on a temporary benchmark provided by McIntosh Perry Consulting Engineers. George Downing Estate Drilling Ltd. of Hawkesbury, Ontario supplied and operated the drilling equipment to carry out the drilling, sampling, and in-situ testing. Traffic control operations were provided by Leroy Construction of Blind River Ontario. The drilling, and sampling operations in the field were supervised on a full-time basis by Katya Edney, P.Eng. of Thurber. Laboratory testing was carried out by Thurber in its MTO-approved laboratory in Ottawa. Unconfined Compressive Strength Testing of the bedrock was carried out by Stantec Consulting Ltd. in its MTO-approved laboratory in Ottawa.

Overall project management and direction of the field program was provided by Fred Griffiths, P.Eng. Interpretation of the field data and preparation of this report was completed by Kenton Power, P.Eng. The report was reviewed by Dr. Fred Griffiths, P.Eng., and Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations.



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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

6 GENERAL

This report presents the interpretation of the factual data obtained from a foundation investigation conducted by Thurber for the replacement of the Lauzon Creek Culvert located on Highway 17, in the Township of Long, Ontario. Geotechnical assessment and recommendations are provided to assist the design team in designing a suitable foundation for the proposed replacement.

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 replacement of the existing Lauzon Creek Culvert. The discussions and recommendations presented in this report are based on the information provided by McIntosh Perry Consulting Engineers (MPCE) and on the factual data obtained during the course of this investigation.

6.1 Existing and Proposed Structure

The existing 26.8 m long reinforced concrete arch culvert has an internal span of 12.2 m and height of 4.4 m. There is also a retained soil system (RSS) located on the east and west sides of the culvert at both the culvert inlet and outlet. Archival drawings indicate that the arch culvert is designed to be founded on 1.2 m wide by 1.5 m deep footings with a top of footing elevation of approximately 179.2 m. The drawings also indicate that east footing appears to be founded on bedrock while the west footing is founded on hard gravel. Based on available information the footings were to be constructed in the dry within a cofferdam.

The August 2018 GA Drawing (see Appendix A), indicates that the existing arch culvert is to be replaced along the existing alignment with a new 20 m long, open bottom, pre-cast concrete arch culvert supported on cast-in-place concrete spread footings. The arch culvert is to have an approximate internal span of 16.4 m and an approximate rise of 4.6 m. Concrete headwalls are to be constructed at both ends of the culvert with concrete retaining walls set an approximate angle of 30° to the headwall on the north side (inlet) of the culvert and parallel to the roadway on the south side (outlet). The maximum exposed height of the walls is anticipated to be 5.2 m. The height of the walls at the north side of the culvert (inlet) is to taper to a height of 4.2 m and 3.6 m at the northeast and northwest wall respectively.

The design top of streambed is at an approximate elevation of 179.2 m and no permanent changes to the highway profile above the culvert are proposed.

The August 2018 GA also indicates that the span of the new culvert is approximately 4.2 m greater than the existing culvert but will have a similar rise. The new footings will be constructed to the outside of the existing footings, which have an outside to outside dimension of approximately 13.7 m. It is expected that the founding elevation of the new and existing footings will be similar.

The Contractor's excavation plan will need to take into consideration the potential for conflict between the existing and new structures.

One lane of traffic will operate through the construction zone during all stages of construction controlled by temporary signals. Temporary protection systems, cofferdams and temporary creek water diversion may be required for the removal of the existing culvert and to install the new culvert.

6.2 Applicable Codes and Design Considerations

The geotechnical assessment presented below has been prepared based on the available data regarding the proposed foundations and existing ground conditions and in accordance with the Canadian Highway Bridge Design Code, version CSA S6-14 (CHBDC).

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

It is understood that MTO has designated the new structure as follows:

Table 6-1: Structure Classification

Criteria	Classification	CHBDC Section
Importance Category	Major Route	4.4.2
Consequence Classification	Typical Consequence	6.5.1

Accordingly, a consequence factor (Ψ) of 1.0, as per Table 6.1 of the CHBDC, has been used in assessing factored geotechnical resistances.

The frost penetration depth at this site is 2.1 m as per OPSD 3090.101. Foundations placed directly on sound bedrock do not need full frost protection cover.

7 SEISMIC CONSIDERATIONS

7.1 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 ($S_a(T)$) for the reference ground condition (Site Class C) for a range of periods (T) and for a range of

return periods including 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 and the peak ground acceleration (PGA) for this location for a reference Site Class C with a with a 2% probability of exceedance in 50 years is 0.040g. This value is to be scaled by the $F(PGA)$ based on the site-specific Site Class.

7.2 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 soil and bedrock conditions encountered below the anticipated culvert foundation elevation, the site is classified as a Seismic Site Class C in accordance with Table 4.1 of the CHBDC.

7.3 Seismic Liquefaction

The soils beneath the anticipated founding elevation consist of compact to very dense glacial till deposits overlying bedrock. Neither the glacial till or bedrock are considered susceptible to liquefaction from the design seismic event.

8 DESIGN DISCUSSION

8.1 Geotechnical Assessment

Based on the results of the field and laboratory investigation and the information provided by MPCE with regards to the proposed culvert replacement, the geotechnical foundation design considerations include the following.

- The existing sand and silt fill and clay materials encountered in Boreholes 17-02 through 17-05 advanced near the proposed footing line locations are not considered suitable to support shallow foundations.
- The glacial till and bedrock materials encountered are considered suitable to support shallow foundations with moderate to high bearing resistance.
- The depth to bedrock varies at the site with the bedrock surface encountered higher at the east footing line than at the west footing line. Bedrock was encountered at as low as Elevation 175.1 m at the west footing line and as high as Elevation 177.7 m at the east footing line.
- 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 equipment must be developed to enable excavation to the required founding elevation and construction of the footings in the dry.
- Increased difficulties installing temporary protection systems and sheet pile cofferdams should be anticipated due to the presence of cobbles and boulders within the fill and till layers and the relatively shallow depth to the bedrock surface.

- It is understood that the existing culvert foundations may be left in place during construction of the new footings to help facilitate the temporary flow passage system. This is acceptable provided that the new structure is founded at the same elevation as the footings for the existing culvert or where the footings of the existing and proposed structures are founded on bedrock and where there is no conflict between existing and proposed footings.
- Depending on the excavation and dewatering procedures employed by the Contractor, unbalanced earth and hydrostatic pressure may develop on the existing culvert foundations while excavations for the new footings are carried out. It is expected that backfill above the existing culvert and the existing culvert structure will be removed prior to excavation for the new culvert foundations. If the proposed and existing culverts are founded at the same elevation, there should be minimal influence between/on the new and existing footings since neither is under significant load prior to backfilling. However, should the substrate and backfill within the existing culvert footprint remain while excavations for the new footing are carried then unbalanced earth pressures will develop and must be taken into consideration in the design of the excavation and dewatering procedures employed by the Contractor.
- The frost penetration depth at this site is 2.1 m as per OPSD 3090.101. Foundations placed directly on sound bedrock do not need full frost protection cover.
- Scour protection must be provided for the spread footing alternative.

8.2 Foundation Design Alternatives

The results of the field and laboratory investigation and historical data indicate that the embankment fill is underlain by glacial till deposits overlying granite bedrock.

Approximate key elevations are as follows:

- | | |
|-----------------------------------------------|--------------------|
| • Pavement surface | 184.4 m to 184.7 m |
| • Glacial till surface near east footing line | 179.8 m to 180.1 m |
| • Glacial till surface near west footing line | 179.2 m to 179.4 m |
| • Creek water on December 20, 2017 | 179.7 m to 179.4 m |
| • Bedrock surface near east footing line | 176.3 m to 177.7 m |
| • Bedrock surface near west footing line | 175.1 m to 176.3 m |

The glacial till deposit consists predominantly of dense silty sand with gravel that includes occasional to frequent cobbles and boulders. Based on the results of unconfined compressive strength testing the granite bedrock encountered is strong to very strong.

Since the glacial till and bedrock are relatively shallow at this site, spread footings are considered the most cost-effective foundations option. Accordingly, a deep foundation option has not been carried forward for this site.

9 FOUNDATION DESIGN RECOMMENDATIONS

9.1 Shallow Foundations

The existing fill materials encountered in Boreholes 17-02 through 17-05 advanced near the proposed footing lines are not considered suitable to support shallow foundations. Spread footings must be founded on the undisturbed native glacial till or on/in the existing bedrock.

Based on the design top of streambed elevation of 179.2 m and frost cover requirements of 2.1 m, the underside of footing elevation would be at or below approximately 177.1 m. At Elevation 177.1 m, footings would be founded on/in the existing bedrock at the east footing line (Boreholes 17-04 and 17-05) and within the dense glacial till at the west footing line (Boreholes 17-02 and 17-03).

Spread footings between 2 and 3 m in width and founded at various founding elevations may be designed based on the factored geotechnical resistances as outlined in Table 9-1.

The factored geotechnical resistances provided in Table 9-1 include the following factors:

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

The factored geotechnical resistance at SLS corresponds to total footing settlement of 25 mm.

For footings founded on/in the bedrock, settlement is considered negligible under the anticipated loadings and therefore SLS condition will not govern the design.

Table 9-1: Spread Footing Design Alternatives

Option	Approximate Underside of Footing Elevation (m)	Founding Material	Footing Width (m)	Geotechnical Resistance (kPa)		Issues
				ULS _f	SLS _f	
1	178	Compact to dense glacial till	2	320	300	<ul style="list-style-type: none"> • Similar depth as the existing footing • May be easier to construct with less extensive dewatering • Footing base is above the depth of frost penetration, and there is a risk of frost heave of the foundations. Mitigation option with EPS insulation equivalent to 2.1 m of soil cover for the foundations could be considered • Scour protection will be required and could conflict with the frost protection
			2.5	350	275	
			3	390	250	
2	177.1	West footing line on glacial till	2	600	500	<ul style="list-style-type: none"> • A deeper excavation and more extensive dewatering than Option 1 will be required • Driving sheet piles into dense glacial till to form a sealed cofferdam may not be possible • Scour protection may be required depending on depth of scour
		East footing line on glacial till or bedrock	2.5	650	500	
			3	700	500	
3	177.1	Bedrock or tremie concrete over bedrock	2	2000	N/A	<ul style="list-style-type: none"> • No scour or frost protection required as footings are founded on bedrock • Sheet pile cofferdam enclosure may be difficult to construct • Use of tremie concrete to reduce dewatering efforts
			2.5			
			3			

For Option 2, it is noted that the south portion of the east footing line would be supported on glacial till while the north portion would be on bedrock. It is recommended that the till be removed and replaced with mass concrete to provide a more consistent bearing surface for the east footing line. A similar treatment would be recommended for the west footing line if differential settlement of 25 mm between the east and west footings cannot be tolerated.

For footings founded on bedrock, the footings may have to be stepped to avoid rock excavation.

Option 3 is recommended for the culvert foundations and could include maintaining an underside of footing elevation of 177.1 m and replacing all of the soil from that level to bedrock with mass concrete. The concrete could be tremie to reduce dewatering efforts. The mass concrete should extend 0.5 m beyond the footing edge. Based on the bedrock surface elevations encountered during the field investigation, approximately 0.6 m of till material will need to be removed and replaced beneath the proposed east footing and between 0.7 m and 1.9 m beneath the west footing.

The geotechnical resistances are for vertical concentric loading and will need to be adjusted for the effects of inclined or eccentric loading, if applicable. The geotechnical resistance should be calculated as illustrated in the CHBDC Clause 6.10.3 and Clause 6.10.4.

Resistance to lateral forces and sliding resistance between concrete and underlying materials should be evaluated using an unfactored coefficients of friction provided in Table 9.2.

Table 9-2: Unfactored Coefficients of Friction between Concrete and Founding Material

Foundation Structure Material	Founding Material		
	Native Till	Granular A	Bedrock
Cast-in-place concrete	0.50	0.55	0.70

Additional lateral resistance, if needed, could be provided by dowelling into the bedrock.

9.2 Subgrade Preparation and Structural Backfilling

Excavation and backfilling for installation of the new culvert should be carried out in accordance OPSS 902 and MTO Special Provision (SP) No. 109S12, Amendment to OPSS 902, March 2018.

The water level in the Lauzon Creek Culvert was measured at elevations ranging from 179.7 m to 179.4 m as such, the base of a footing excavation would be below the measured creek level. Therefore, for the footing options creek diversion and dewatering will be required for the preparation of the subgrade, and the construction of the new foundations in a dry and stable excavation. Please refer to Section 10.3.

Subgrade preparation for the construction of the new foundations should include excavation and removal of the existing fill, culvert foundations, and native materials and backfill materials from beneath the proposed founding elevation to the bedrock surface.

The excavation beneath the underside of footings should be backfilled with tremie concrete, extending to a least 0.5 m beyond the edge of the footings.

It is understood that the existing culvert foundations may be left in place during construction of the new footings to help facilitate the temporary flow passage system. It is anticipated that the water course diversion will be carried out with a cofferdam directing creek water through a pipe

or series of pipes placed through the existing culvert during construction. Consideration for the unbalanced earth and hydrostatic pressures that may develop during excavations as outlined above in Section 8.1 will need to be considered.

The subgrade within the footprint of the culvert foundations should consist of either bedrock or tremie concrete on bedrock. The bedrock surface should be probed, and loose material removed prior to placing tremie concrete.

Backfill for the culvert should consist of compacted free-draining granular fill consisting of Granular B Type II material placed and compacted in accordance with OPSS.PROV 501. Compaction equipment to be used adjacent to the walls should be restricted in accordance with OPSS.PROV 501. Backfill requirements from the arch culvert supplier must also be confirmed.

9.3 Retaining Wall Foundations

The August 2018 GA drawing includes concrete retaining walls extending out from both sides of the concrete headwalls at each end of the culvert. The elevation of the base of the retaining walls is to be at the same elevation as the footings for the arch culvert. The design of the retaining walls is to include a 2H:1V backslope behind the walls. It is understood that the maximum exposed height of the walls is anticipated to be a maximum of 5.2 m at the headwall tapering down to a height of 4.2 m and 3.6 m at the northeast and northwest wall respectively. The walls on south side of the culvert are to be constructed with an exposed height of 5.5 m along the entire length of the wall.

The existing ground conditions beneath the retaining walls may include glacial till, or bedrock. As noted in Section 9.2, subgrade preparation for the construction of new foundations should include excavation and removal of the existing fill, culvert foundations, and backfill materials from beneath be removed from beneath the retaining walls. Any soft, organic or loose alluvial materials must be removed and replaced with Granular B Type II material, or tremie concrete.

The bearing resistance values provided in Section 9.1 and the lateral earth pressure comments provided in Section 9.5 may be used for design of the proposed retaining walls.

The global stability for the proposed embankment behind the concrete gravity retaining wall constructed using OPSS.PROV 1010 Granular B Type II, was evaluated using GeoStudio 2012 Slope/W software for limit equilibrium analysis. Input parameters for the analysis are based on the in-situ SPT N values.

The following additional parameters were used in the analysis.

- A traffic surcharge load as per Section 6.12.5 of the CHBDC was used for static analysis
- A seismic horizontal loading of 0.02g, equal to ½ of the site adjusted PGA value (0.04g) was used for seismic analysis.
- A backslope of 2H:1V.

The global stability analysis results indicate a factor of safety of 1.6 under static and seismic conditions respectively. Copies of the output from the global stability analyses for both static and seismic conditions are provided in Appendix E.

The retaining wall design must be checked against sliding, and overturning modes of failure.

Table 9-3: Static Lateral Earth Pressure Coefficient

Parameter	Existing Fill & SSM	OPSS Granular A & B Type II	Glacial Till
Soil Unit Weight, kN/m^3 , γ	20	22	21
Angle of Internal Friction, ϕ	30°	35°	35°
Horizontal Backfill			
Coefficient of at Rest Earth Pressure, K_o (Non-Yielding Wall)	0.50	0.43	0.43
Coefficient of Active Earth Pressure, K_a (Yielding Wall)	0.33	0.27	0.27
2H:1V Sloped Backfill			
Coefficient of at Rest Earth Pressure, K_o (Non-Yielding Wall)	0.72	0.62	0.62
Coefficient of Active Earth Pressure, K_a (Yielding Wall)	0.54	0.39	0.39

If lateral movement is not permissible and/or the wall is retained from lateral yielding, it is recommended that at-rest horizontal lateral earth pressures be used for design. Active pressures should 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.

A lateral pressure due to backfill compaction should be added to the calculated lateral earth pressure in accordance with Section 6.12.3 of the CHBDC.

9.5.2 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:

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

The ratio of wall movement to wall height required to mobilize the active condition would be approximately 0.002 for a yielding structure with respect to the assessment of seismically induced lateral earth pressures.

The recommended seismic lateral earth pressure parameters for use in the design that are provided in Table 9-4 assume the following:

- Seismic Site Class of C
- Site Coefficient $F(\text{PGA})$ of 1.0 as per Table 4.8 of the CHBDC,
- Site adjusted PGA with a 2% probability of exceedance in 50 years of 0.040g; as outlined in Section 7.0.

Table 9-4: Lateral Earth Pressure (Under Seismic Loads)

Parameter	OPSS Granular A & B Type II
Soil Unit Weight, kN/m ³ , γ	22
Angle of Internal Friction, ϕ	35°
Horizontal Backslope	
Non-Yielding Wall – Active Earth Pressure Coefficient, K_{AE}	0.29
Yielding Wall – Active Earth Pressure Coefficient, K_{AE}	0.28
2H:1V Backslope	
Non-Yielding Wall – Active Earth Pressure Coefficient, K_{AE}	0.45
Yielding Wall – Active Earth Pressure Coefficient, K_{AE}	0.42

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:

$$\sigma_h = K\gamma d + (K_{AE} - K_a) \gamma (H - d)$$

where:

- σ_h = lateral earth pressure 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/m³); 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)

9.6 Cement Type and Corrosion Potential

Two samples of the soils encountered were submitted to Paracel Laboratories in Ottawa, Ontario for analysis of pH, water soluble sulphate and chloride concentrations, and resistivity. The analysis results are summarized in Table 4-2 and a copy of the test results is provided in Appendix C.

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 4-2 were compared with Table 3 of Canadian Standards Association Standard A23.1-14 (CSA A23.1) and generally indicate a low degree of sulphate attack potential on concrete structures at this site.

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 4-2 were compared with Table 3.2 of the MTO Gravity Pipe Design Guideline and generally indicate a very low to severe corrosive environment. The test results provided in Table 4-2 may be used to aid in the selection of coatings and corrosion protection systems for buried steel objects.

10 CONSTRUCTION CONSIDERATIONS

10.1 Excavations

It is anticipated that temporary excavations in the order of 8.0 m below the existing highway surface will be required for the removal of the existing culvert, foundations and the construction of the new foundations and installation of the new culvert.

The potential for conflict with the existing culvert foundations must be reviewed. The theoretical locations of the existing and new foundations are shown on the General Arrangement Drawing provided in Appendix A. Suggested wording for an NSSP to alert the contractor to the potential conflict with existing foundations is provided in Appendix F.

All excavations must be conducted in accordance with the requirements of the Occupational Health & Safety Act & Regulations (OHSA) for Construction Projects. The fills and clays at the site above the groundwater level should be classified as Type 3 in accordance with OHSA. The native till material should be classified as Type 2 provided it is dewatered. As indicated in OHSA, if an excavation contains more than one type of soil, the soil type for the excavation shall be classified as the type with the highest number among the soil types present within the excavation. Unsupported excavations made in Type 3 soils must have side slopes no steeper than 1H:1V from the base of the excavation. Excavations within the bedrock may be carried out near vertical.

Subgrade preparation and construction of foundations must be carried out in the dry.

The management and disposal of excess material should 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. As cobbles and boulders were observed in the boreholes recommended wording for an NSSP alerting bidders to their presence has been provided in Appendix F.

10.2 Temporary Protection Systems

The proposed methodology is to replace the existing culvert in stages with a temporary protection system running east-west near the centerline of the highway. Temporary protection systems should be provided in accordance with OPSS.PROV 539 and designed for Performance Level 2. All protection systems should be designed by a Professional Engineer experienced in such designs.

Typical lateral earth pressure coefficients are provided in Table 10-1 for the design of vertical temporary protection systems. The values provided are for a horizontal and 2H:1V backslope behind, and a horizontal surface in front of the protection system.

If the sloped embankment or backslope above the temporary protection systems are not horizontal or 2H:1V and/or the ground surface in front of the walls is not horizontal, the lateral earth pressure parameters provided in Table 10-1 do not apply and recalculation of the earth pressure parameters will be required.

Table 10-1: Static Lateral Earth Pressure Coefficient

Parameter	Existing Fill & SSM	OPSS Granular A & B Type II	Native Clay	Glacial Till
Soil Unit Weight, kN/m^3 , γ	20	22	18	21
Angle of Internal Friction, ϕ	30°	35°	27°	35°
Horizontal Backfill				
Coefficient of at Rest Earth Pressure, K_o	0.50	0.43	0.55	0.43
Coefficient of Active Earth Pressure, K_a	0.33	0.27	0.38	0.27
Coefficient of Passive Earth Pressure, K_p	3.00	3.69	2.66	3.69
2H:1V Sloped Backfill				
Coefficient of at Rest Earth Pressure, K_o	0.72	0.62	0.79	0.62
Coefficient of Active Earth Pressure, K_a	0.54	0.39	0.71	0.39

The design of protection systems is the responsibility of 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.

Increased difficulty with the installation of protection systems should be anticipated due to the presence of cobbles and boulders within the fill and native glacial till. Sheet piles systems are not considered suitable within the glacial till. One option is to use soldier piles and timber lagging with the piles installed in holes predrilled through the embankment and socketed into bedrock. Alternatively, lateral support could be enhanced with rock anchors, dead man anchors or rakers.

To limit the disturbance of the culvert foundation soils, culvert backfill and existing roadway embankment it is recommended that the piles in close proximity to the culvert and embankment be cut off and left in place in accordance with OPSS.PROV 539.

10.3 Dewatering

The depth of excavations required to replace the existing culvert will extend below the creek level observed at the time of the investigation. Furthermore, groundwater and surface runoff will tend to seep into and accumulate into the excavations. The Contractor must control groundwater and creek/surface water flow at the site to permit the replacement of the culvert in a dry and stable excavation.

Excavation for and construction of the culvert must be carried out with a properly designed dewatering system to control groundwater and creek/surface water and may include coffer dams, creek diversion, pumping etc. The dewatering system will be required to remain operational and effective until the temporary excavations are backfilled and then should be decommissioned and removed.

The design of dewatering systems is the responsibility of the Contractor. The Contract Documents must alert the Contractor to this responsibility and to design the system in accordance with

SP FOUN0003 which amends OPSS 902. A preconstruction survey is not recommended, thus Designer Fill-In ** in this SP should be "NA". Recommended wording for an NSSP amending SP FOUN0003 to include the requirement that the design Engineer and design-checking Engineer of the dewatering system have a minimum of 5 years of experience in designing systems of similar nature and scope to the required work has been provided in Appendix F.

The groundwater level will fluctuate and the minimum groundwater elevation at the time of the proposed work should be taken as the creek water level of the design storm return period defined by the contract documents for the temporary dewatering system.

If the selected dewatering method is not effective in the granular till the base of the excavation is likely to result in boiling condition and resulting in post construction settlement of the retaining wall footings.

It is anticipated that the water course diversion will be carried out with a cofferdam directing creek water through a pipe or series of pipes placed through the existing culvert. During periods of higher creek levels, a watertight braced enclosure system should be considered. It should be noted that it may not be possible to drive interlocking sheet piles through the cobbles and boulder in the till. Sump pumps will be required to extract water from the excavations.

The comments on installation and extraction of temporary protection systems are also relevant for cofferdams.

10.4 Erosion and Scour Protection

Slope protection and drainage measures will be required to ensure the long-term surficial stability of the embankment slopes. The Contractor should provide silt fences and erosion control blankets, as required, throughout the duration of construction to prevent silt/sediments from running off the site as per OPSS 805.

Particle size analysis on samples of the embankment materials indicated that the soils have a low to moderate potential for soil erodibility (Wischmeier Nomograph factor, K of 0.05 and 0.22).

Erosion and scour protection should be provided at the spread footing foundations not founded on bedrock. Design of the protection measures must consider hydrologic and hydraulic factors and should 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 should be in accordance with OPSD 810.010.

A vegetation cover should be established on all other exposed earth surfaces as soon as practical to protect against surficial erosion in general accordance with OPSS.PROV 804.

11 CONSTRUCTION CONCERNS

The planned construction methodology includes an open cut excavation for the installation of a new culvert on spread footings.

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

- Although the proposed foundation layout on the structural drawings has been selected to minimize conflict with existing culvert foundations the potential for

conflict exists and will need to be checked prior to commencing excavations for the new foundations. Recommended wording for an NSSP alerting the Contractor to the potential for conflict is provided in Appendix F.

- 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 replacement culvert foundations in the dry.
- Excavation to the design elevations may encounter bedrock.
- Increased difficulties installing temporary protection systems and sheet piles for cofferdams should be anticipated due to the presence of cobbles and boulders within the fill and till layers and the shallow depth to the bedrock surface. Suggested wording for an NSSP alerting the Contractor to the presence of obstructions has been provided in Appendix F.
- Boulders may be encountered in the glacial till. The Contractor must be prepared to deal with obstruction during excavations, cofferdam and/or temporary protection system installation.

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 MTO SP 109S12 during construction to confirm that the foundation recommendations are correctly implemented, and material specifications are met.

12 CLOSURE

Overall project management and direction of the field program was provided by Fred Griffiths, P.Eng. Interpretation of the field data and preparation of this report was completed by Kenton Power, P.Eng. The report was reviewed by Dr. Fred Griffiths, P.Eng., and Dr. P.K. Chatterji, a Designated Principal Contact for MTO Foundations.



Kenton C. Power, M.A.Sc., P.Eng.
Geotechnical Engineer



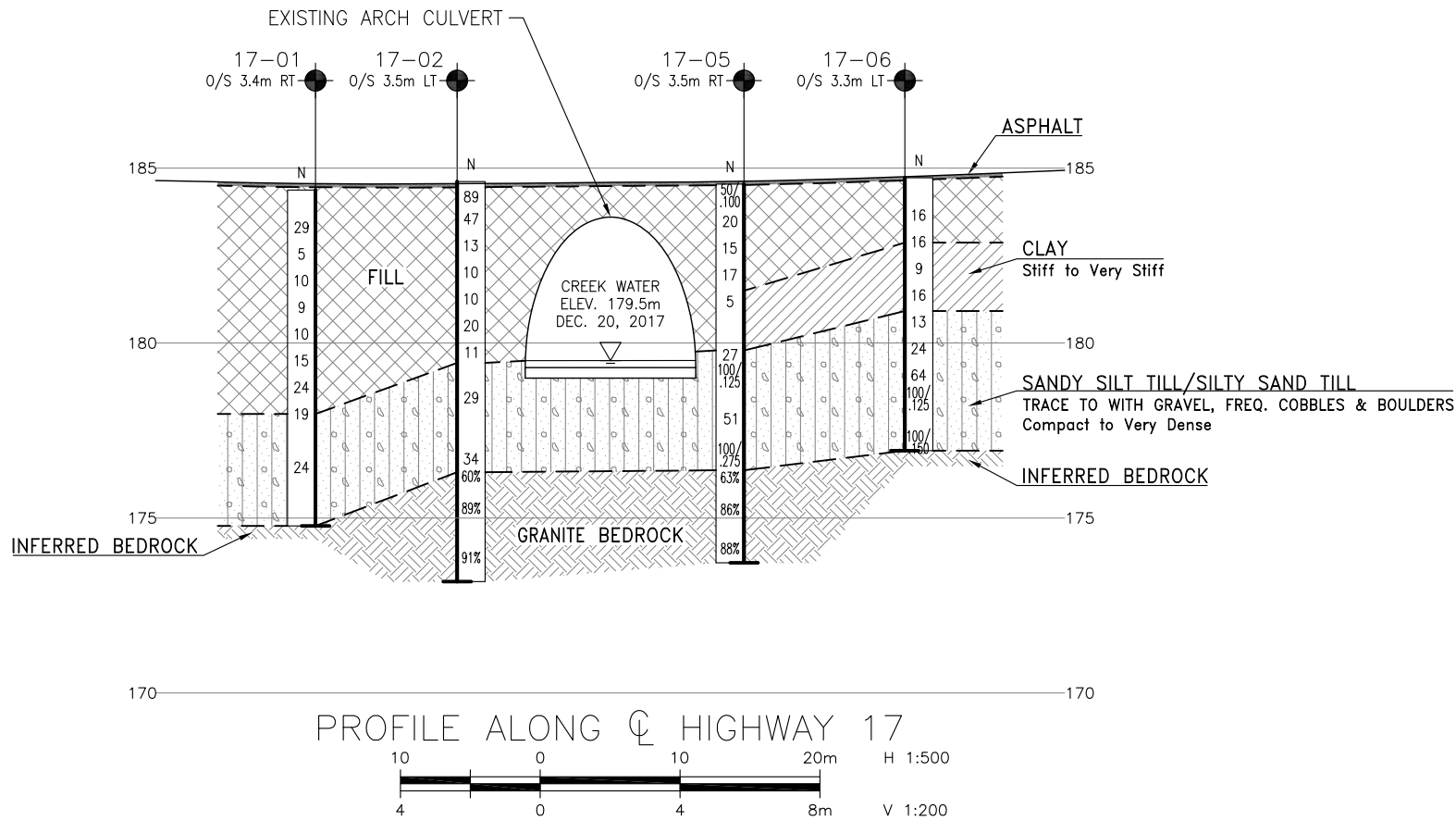
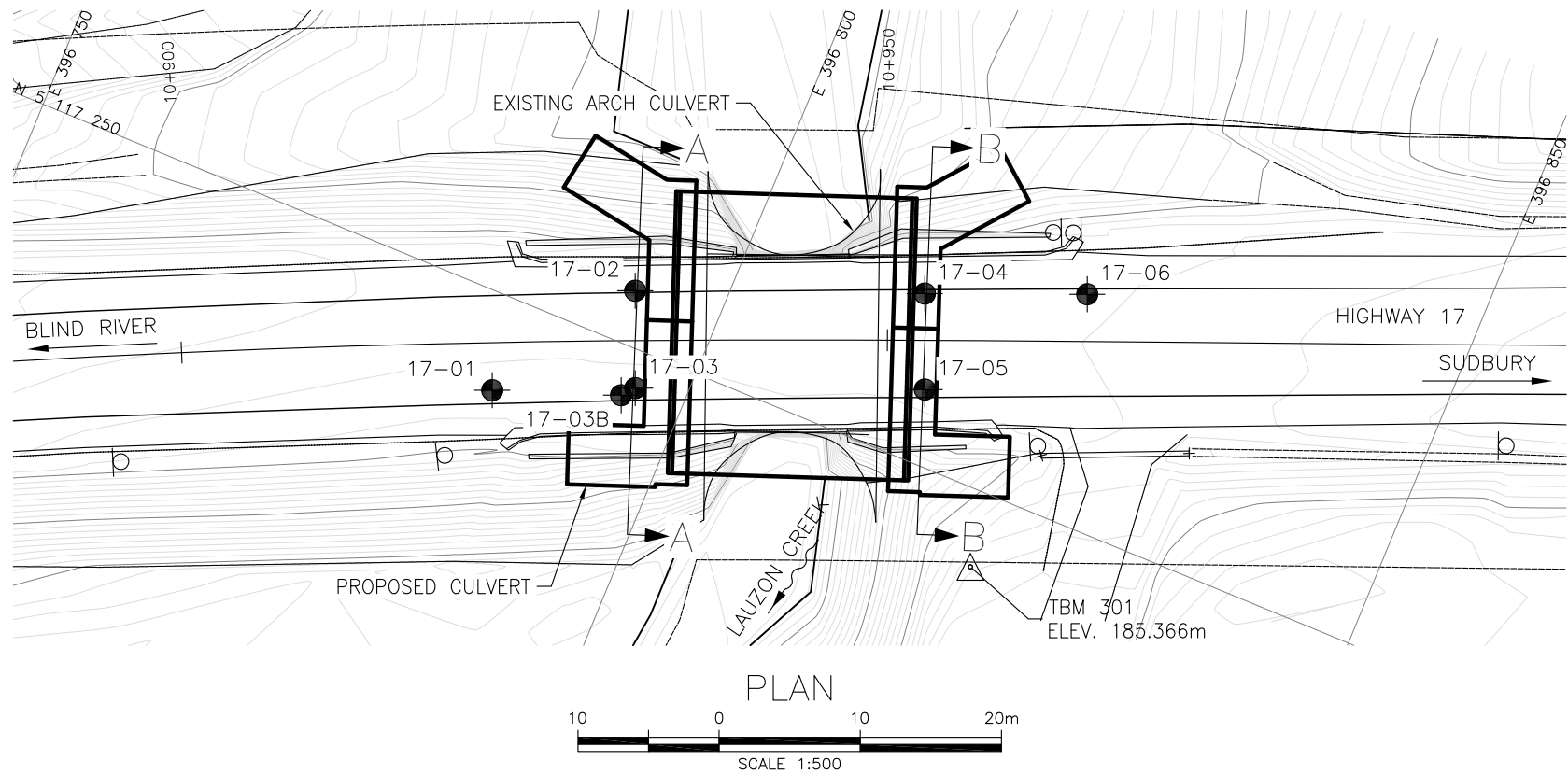
Fred J. Griffiths, Ph.D., P.Eng.
Senior Associate, Senior Geotechnical Engineer



P.K. Chatterji, Ph.D., P.Eng.
Review Principal, Designated MTO Contact

APPENDIX A

BOREHOLE LOCATIONS AND SOIL STRATA DRAWING PRELIMINARY GENERAL ARRANGEMENT DRAWING

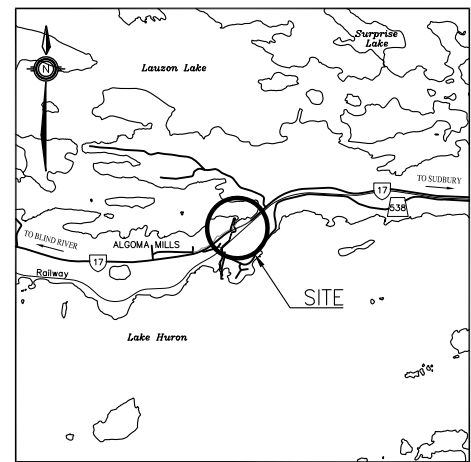


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DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN



CONT No
GWP No 5074-09-00

HIGHWAY 17
LAUZON CREEK
CULVERT REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA



KEYPLAN

LEGEND

●	Borehole
⊙	Borehole and Cone
N	Blows /0.3m (Std Pen Test, 475J/blow)
△	Temporary Benchmark
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

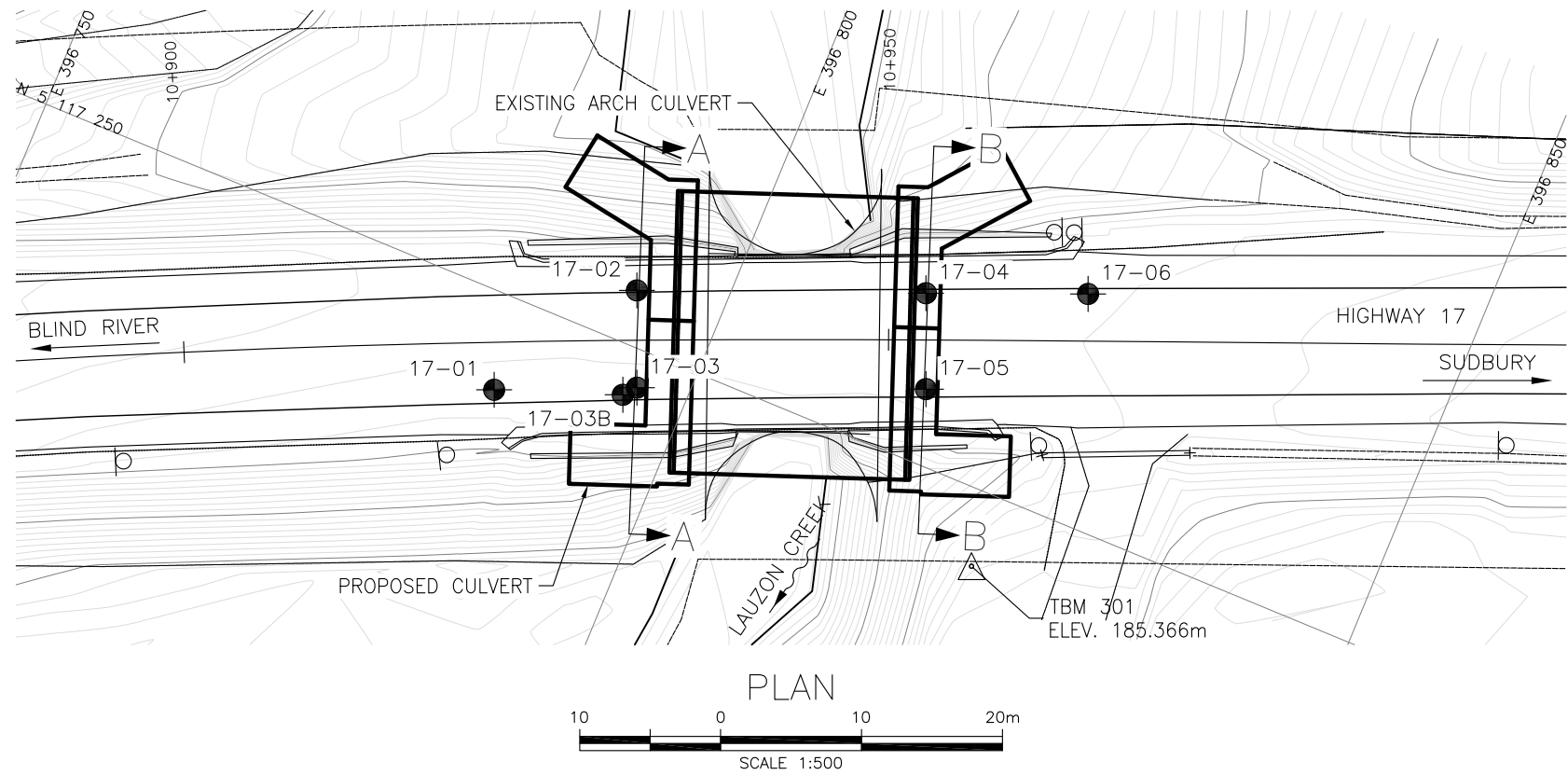
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17-02	184.6	5 117 253.8	396 793.8
17-03	184.4	5 117 247.4	396 796.4
17-03B	184.4	5 117 246.6	396 795.7
17-04	184.7	5 117 261.4	396 812.8
17-05	184.5	5 117 255.2	396 815.4
17-06	184.7	5 117 265.8	396 823.5

-NOTES-

- The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- This drawing is for subsurface information only. Surface details and features are for conceptual illustration.
- Boreholes are shown in MTM Zone 13 coordinates.

GEOCRES No. 41J-114

REVISIONS	DATE	BY	DESCRIPTION
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LOAD			DATE AUG 2018
STRUCT			DWG 1

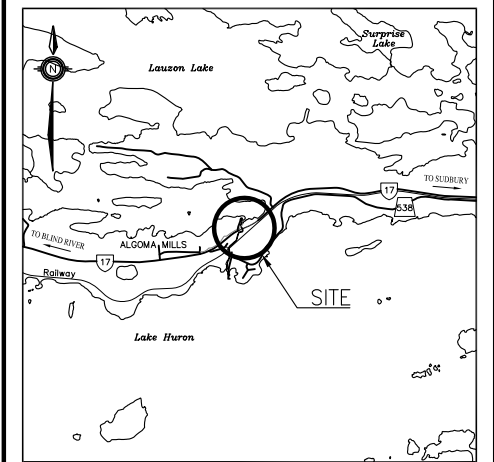


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DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN



CONT No
GWP No 5074-09-00

HIGHWAY 17
LAUZON CREEK
CULVERT REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA



KEYPLAN

LEGEND

●	Borehole
⊙	Borehole and Cone
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△	Temporary Benchmark
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
W	Water Level
↑	Head Artesian Water
⊥	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

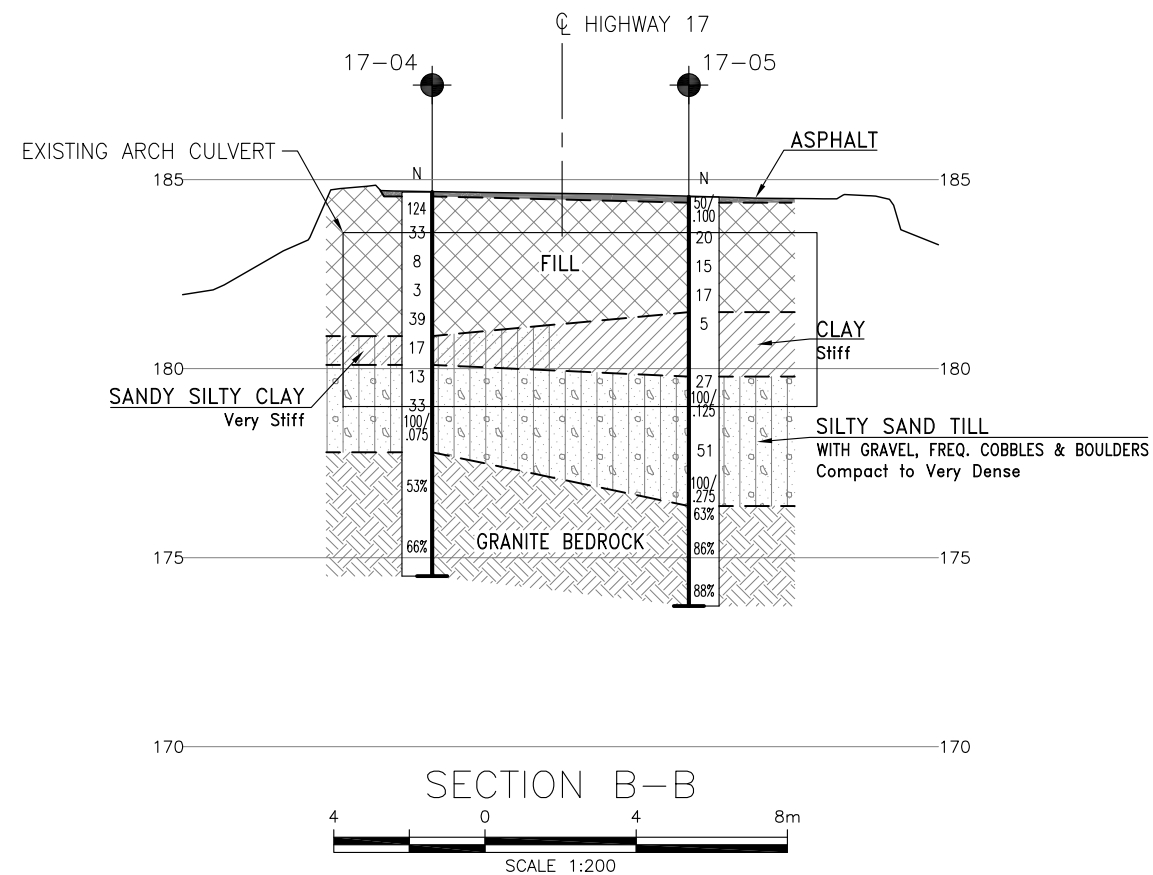
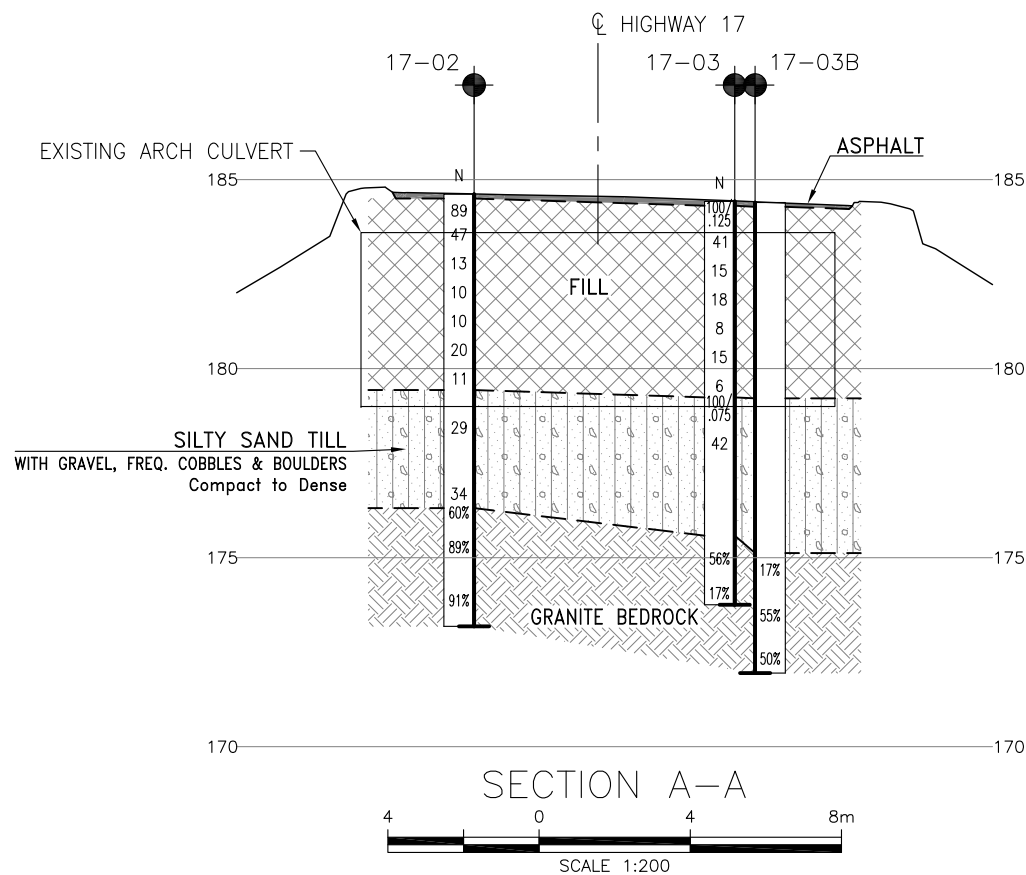
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GEOCRES No. 41J-114

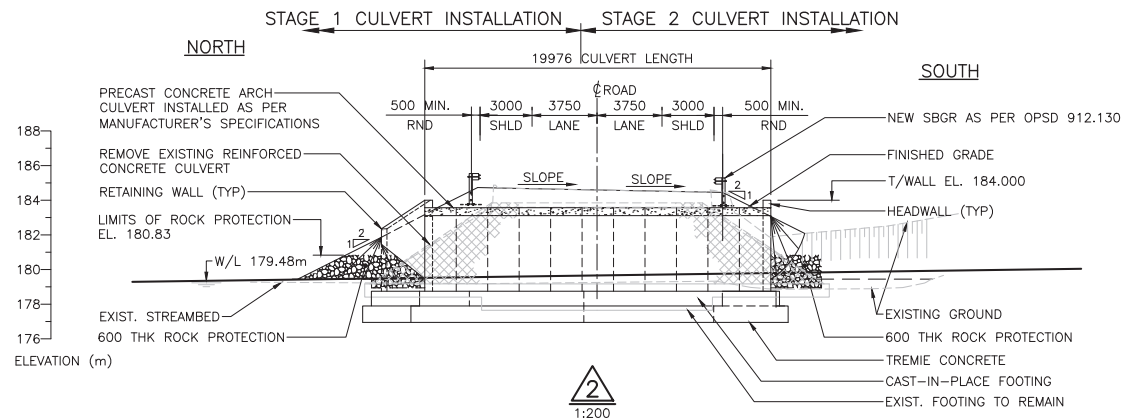
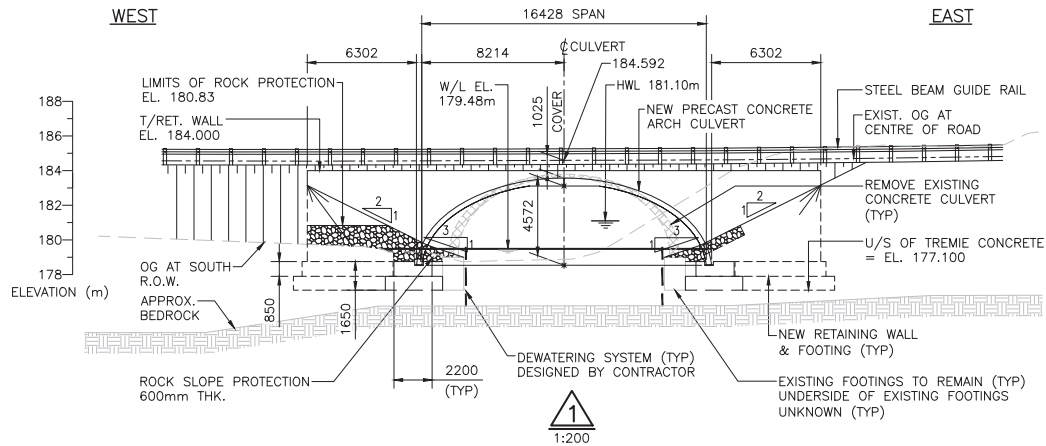
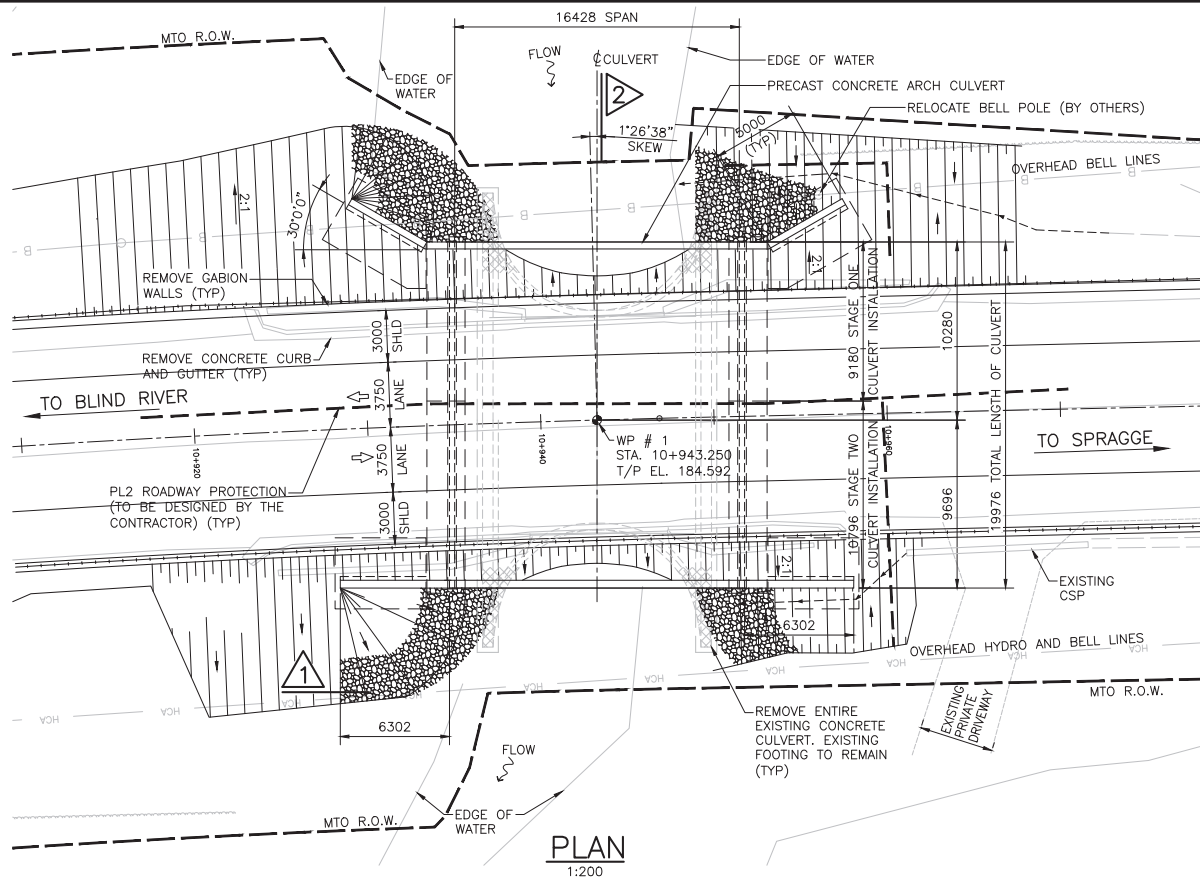
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DESIGN	KP	CHK -	CODE
DRAWN	MFA	CHK KP	SITE 38S-243
LOAD			DATE AUG 2018
STRUCT			DWG 2



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MODIFIED: 8/2/2018 11:07:57 AM BY: G.PENNY
DATE PLOTTED: 8/2/2018 11:13:57 AM BY: GREG PENNY

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

CONSTRUCTION
NORTH



LEGEND:

- ROCK PROTECTION
- RIGHT-OF-WAY
- REMOVALS

METRIC DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN		CONT. No. 2018-XXXX GWP No. 5074-09-00	
		HIGHWAY 17 LAUZON CREEK CULVERT REPLACEMENT	SHEET
		GENERAL ARRANGEMENT	14
		McINTOSH PERRY	

GENERAL NOTES:

- CLASS OF CONCRETE 35 MPa TREMIE CONCRETE
PRECAST CONCRETE 45 MPa
REMAINDER 30 MPa UNLESS OTHERWISE NOTED
- CLEAR COVER TO REINFORCING STEEL:
FOOTING (CAST-IN-PLACE) 100 ± 25
PRECAST CONCRETE 40 ± 10
ALL EXPOSED CORNERS TO BE CHAMFERED 20mm
- REINFORCING STEEL:
-REINFORCING STEEL SHALL BE GRADE 400W UNLESS OTHERWISE SPECIFIED
-BAR MARKS WITH PREFIX 'S' DENOTE STAINLESS STEEL BARS.
-STAINLESS REINFORCING STEEL SHALL BE TYPE 316LN OR DUPLEX 2205 AND HAVE A MINIMUM YIELD STRENGTH OF 500 MPa.
-UNLESS SHOWN OTHERWISE, TENSION LAP SPLICES SHALL BE CLASS 'B'
-BAR HOOKS SHALL HAVE STANDARD HOOK DIMENSIONS USING MINIMUM BEND DIAMETER WHILE STIRRUPS AND TIES SHALL HAVE A MINIMUM HOOK DIMENSIONS. ALL HOOKS SHALL BE IN ACCORDANCE WITH THE STRUCTURAL STANDARD DRAWINGS SS12-1 UNLESS INDICATED OTHERWISE.
- SEE HIGHWAY STAGING DRAWING FOR CONSTRUCTION STAGING AND TRAFFIC CONTROL DETAILS.
- PRECAST CONCRETE ARCH CULVERT TO BE DESIGNED BY CULVERT MANUFACTURER.
- CULVERT SPAN, RISE AND LENGTH MAY VARY BASED ON ACTUAL DIMENSIONS OF CULVERT MANUFACTURER. DIMENSIONS SHOWN ARE MINIMUM VALUES REQUIRED.

CONSTRUCTION NOTES:

- THE CONTRACTOR IS ADVISED NOT TO RELY ON THE WATER LEVEL SHOWN ON THE DRAWINGS. THE WATER LEVEL IS SUBJECT TO VARIATIONS.
- THE CONTRACTOR SHALL DESIGN AND CONSTRUCT A TEMPORARY FLOW PASSAGE SYSTEM TO ISOLATE THE WORK AREA FROM THE AQUATIC ENVIRONMENT AND ENSURE ALL WORK IS CARRIED OUT UNDER DRY CONDITIONS. THE CONTRACTOR SHALL SUBMIT DETAILS OF THE SYSTEM TO THE CONTRACT ADMINISTRATOR FOR INFORMATION PURPOSES ONLY.
- CONTRACTOR TO PROTECT ALL EXISTING UTILITIES WITH A METHOD APPROVED BY THE GOVERNING UTILITY.
- THE CONTRACTOR SHALL VERIFY STREAMBED ELEVATIONS ON SITE BEFORE CONSTRUCTION. THE CONTRACTOR SHALL REPORT THE STREAMBED ELEVATIONS TO THE CONTRACT ADMINISTRATOR AND, IF NECESSARY, SHALL MODIFY THE STREAMBED AS DIRECTED BY THE CONTRACT ADMINISTRATOR TO ENSURE THAT THE NEW STREAMBED ELEVATIONS MATCH THE EXISTING STREAMBED AND THERE ARE NO SUDDEN RISE OR FALL OBSTRUCTING THE FLOW.
- BEDROCK LEVEL IS APPROXIMATE. ACTUAL FIELD CONDITIONS MAY VARY.
- THE CONTRACTOR IS FULLY RESPONSIBLE FOR THE DESIGN, CONSTRUCTION METHODS AND PERFORMANCE OF THE TEMPORARY SLOPE PROTECTION SYSTEM AND ASSOCIATED WORKS.
- CONTRACTOR IS RESPONSIBLE FOR MAINTAINING THE STABILITY OF THE EXISTING STRUCTURES AT ALL TIMES, INCLUDING EXCAVATION, BACKFILL, REMOVALS, INSTALLATIONS, ETC.. CONTRACTOR TO DESIGN AND PROVIDE ANY TEMPORARY SUPPORT SYSTEMS FOR EXISTING AND NEW STRUCTURES AT VARIOUS STAGES OF CONSTRUCTION AS REQUIRED TO SUIT THEIR METHOD OF CONSTRUCTION.
- THE CONTRACTOR SHALL DESIGN AN ADEQUATE ROADWAY PROTECTION SYSTEM TO MEET HIS METHOD OF OPERATION. ROADWAY PROTECTION TO BE PERFORMANCE LEVEL 2.
- BACKFILL SHALL BE PLACED SIMULTANEOUSLY BEHIND BOTH SIDES OF CULVERT, KEEPING THE HEIGHT OF THE BACKFILL APPROXIMATELY THE SAME. AT NO TIMES SHALL THE DIFFERENCE IN ELEVATION BE GREATER THAN 500mm OR AS SPECIFIED BY THE MANUFACTURER.
- CULVERT SUBGRADE TO BE INSPECTED FOLLOWING SUB-EXCAVATION TO ENSURE THAT ALL ORGANICS AND OTHER UNSUITABLE MATERIALS HAVE BEEN REMOVED.

LIST OF ABBREVIATIONS:

T/O	TOP OF	N	NORTH
EL.	ELEVATION	E	EAST
NTS	NOT TO SCALE	W.L.	WATER LEVEL
TYP	TYPICAL	P.G.	PROFILE GRADE
STA.	STATION	SBGR	STEEL BEAM GUIDE RAIL
WP	WORKING POINT	MAX	MAXIMUM
HWY	HIGHWAY		

LIST OF DRAWINGS:

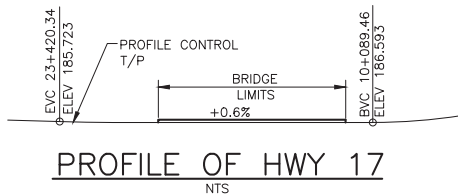
- GENERAL ARRANGEMENT
- BOREHOLE LOCATIONS AND SOIL STRATA - 1
- BOREHOLE LOCATIONS AND SOIL STRATA - 2
- FOUNDATION LAYOUT
- FOUNDATION DETAILS
- NORTH RETAINING WALLS DETAILS
- SOUTH RETAINING WALLS DETAILS

APPLICABLE STANDARDS DRAWINGS:

MTOD 912.109 GUIDE RAIL SYSTEM, STEEL BEAM TYPE M20 - STEEL POST WITH OFFSET BLOCK AND BASE PLATE COMPONENT

OPSD 912.186 GUIDE RAIL SYSTEM, STEEL BEAM, TYPE M20 - ADJACENT TO 2H:1V SLOPE INSTALLATION - RAIL AT SHOULDER

REVISIONS	DATE	BY	REV	DESCRIPTION
DESIGN	AS	CHK	LD	CODECHBDC S6-14/LOAD CL-625-ONT
DRAWN	KGP	CHK	AS	SITE 385-243/STRUCT SCHEME DWG 1



FLOW DATA: CULVERT		
DURATION	PEAK FLOW "Q" (m³/s)	WATER LEVEL (m)
2 YR.	15	180.14
5 YR.	28	180.43

WP	NORTHING	EASTING	ELEVATION
1	5117254.810	396805.399	184.592

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

APPENDIX B

RECORD OF BOREHOLE SHEETS BEDROCK CORE PHOTOGRAPHS



SYMBOLS, ABBREVIATIONS AND TERMS USED ON TEST HOLE RECORDS

TERMINOLOGY DESCRIBING COMMON SOIL GENESIS

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

TERMINOLOGY DESCRIBING SOIL STRUCTURE:

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

RECOVERY:

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

N-VALUE:

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

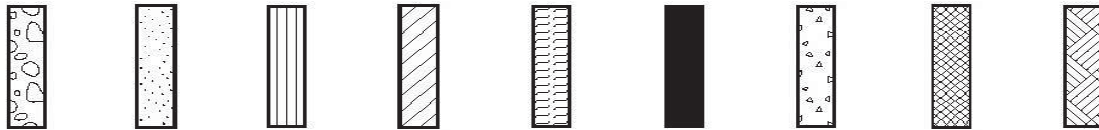
DYNAMIC CONE PENETRATION TEST (DCPT):

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



STRATA PLOT:

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



Boulders Cobbles Gravel	Sand	Silt	Clay	Organics	Asphalt	Concrete	Fill	Bedrock
-------------------------------	------	------	------	----------	---------	----------	------	---------

TEXTURING CLASSIFICATION OF SOILS

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

TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

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

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

SAMPLE TYPES

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

TERMS DESCRIBING CONSISTENCY (COHESIONLESS SOILS ONLY)

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

MODIFIED UNIFIED SOIL CLASSIFICATION

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

Note - W_L = Liquid Limit



EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION

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

TERMS

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

DISCONTINUITY SPACING

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

STRENGTH CLASSIFICATION

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

METRIC

Lat: 46.1889346°, Long: -82.807851°
MTM Zone 13: N 5 117 243.4 E 396 787.1

DOUBLE LINE LAUZON CREEK.GPJ 2012TEMPLATE(MTO).GDT 3/8/18

+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 17-02

1 OF 2

METRIC

GWP# 5074-09-00 LOCATION Lat: 46.1888403°, Long: -82.807958° MTM Zone 13: N 5 117 253.8 E 396 793.8 ORIGINATED BY KE
HWY 17 BOREHOLE TYPE NW Casing / NQ Coring COMPILED BY KCP
DATUM Geodetic DATE 2017.12.07 - 2017.12.07 CHECKED BY FG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								20 40 60 80 100						
184.6														
0.0	120 mm ASPHALT													
0.1	Sand with gravel Grey to brown Very dense FILL - frost to 1.1 m		1	SS	89		184							
183.5			2	SS	47									
1.1	Silty sand with gravel - occasional cobbles Loose to compact Brown FILL		3	SS	13		183							
			4	SS	10		182							
			5	SS	10		181							23 57 20 (SI+CL)
			6	SS	20		180							
			7	SS	11									
179.4							179							
5.2	SILTY SAND (SM) with gravel TILL - frequent cobbles and boulders Compact to dense Grey Boulders from 5.2 m to 5.8 m		8	SS	29		178							38 50 12 (SI+CL)
			9	SS	34		177							
176.3			1	NQ			176							RUN #1 TCR=100% SCR=67% RQD=60%
8.3	GRANITE BEDROCK Slightly weathered to fresh Strong to very strong Fair to excellent quality Close to wide joint spacing Red to grey		2	NQ			175							RUN #2 TCR=100% SCR=100% RQD=89%

Continued Next Page


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

RECORD OF BOREHOLE No 17-02

2 OF 2

METRIC

GWP# 5074-09-00 LOCATION Lat: 46.1888403°, Long: -82.807958° MTM Zone 13: N 5 117 253.8 E 396 793.8 ORIGINATED BY KE
 HWY 17 BOREHOLE TYPE NW Casing / NQ Coring COMPILED BY KCP
 DATUM Geodetic DATE 2017.12.07 - 2017.12.07 CHECKED BY FG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
	Continued From Previous Page																
173.2	GRANITE BEDROCK Slightly weathered to fresh Strong to very strong Fair to excellent quality Close to wide joint spacing Red to grey		3	NQ			174									RUN #3 TCR=100% SCR=98% RQD=91%	
11.4	End of Borehole																

RECORD OF BOREHOLE No 17-03

1 OF 2

METRIC

GWP# 5074-09-00 LOCATION Lat: 46.1888964° Long: -82.8079905° MTM Zone 13: N 5 117 247.4 E 396 796.4 ORIGINATED BY KE
HWY 17 BOREHOLE TYPE NW Casing / NQ Coring COMPILED BY KCP
DATUM Geodetic DATE 2017.12.11 - 2017.12.13 CHECKED BY FG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								20 40 60 80 100						
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
							WATER CONTENT (%)							
							PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT w _p w w _L							
184.4														
0.0	120 mm ASPHALT													
0.1	Sand with gravel Grey to brown Very dense FILL		1	SS	100/ 125mm		184							
183.6	- frost to 1.2 m													
0.8	Silty gravel with sand - occasional cobbles Loose to dense Brown FILL		2	SS	41		183							43 41 16 (SI+CL)
			3	SS	15		182							
			4	SS	18		181							
			5	SS	8		180							
			6	SS	15		179							
			7	SS	6		178							33 49 18 (SI+CL)
179.2														
5.2	SILTY SAND (SM) with gravel TILL - frequent cobbles and boulders Compact to dense Grey		8	SS	100/ 75mm		177							
			9	SS	42		176							
	- boulders from 6.5 m to 8.8 m													
			1	NQ			175							
			2	NQ										
175.6														
8.8	GRANITE BEDROCK Slightly weathered to fresh Strong to very strong Very poor to fair quality Close to wide joint spacing Red to grey		3	NQ										RUN #3 TCR=100% SCR=96% RQD=56% UCS=97MPa

DOUBLE LINE LAUZON CREEK.GPJ 2012TEMPLATE(MTO).GDT 3/8/18

Continued Next Page

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

RECORD OF BOREHOLE No 17-03

2 OF 2

METRIC

GWP# 5074-09-00 LOCATION Lat: 46.1888964° Long: -82.8079905° MTM Zone 13: N 5 117 247.4 E 396 796.4 ORIGINATED BY KE
 HWY 17 BOREHOLE TYPE NW Casing / NQ Coring COMPILED BY KCP
 DATUM Geodetic DATE 2017.12.11 - 2017.12.13 CHECKED BY FG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE												
	Continued From Previous Page							20	40	60	80	100								
173.7	GRANITE BEDROCK Slightly weathered to fresh Strong to very strong		4	NQ			174										RUN #4 TCR=100% SCR=46% RQD=17%			
10.7	End of Borehole																			

RECORD OF BOREHOLE No 17-03B

1 OF 2

METRIC

GWP# 5074-09-00 LOCATION Lat: 46.1887656°; Long: -82.8082148° MTM Zone 13: N 5 117 246.6 E 396 795.7 ORIGINATED BY KE
 HWY 17 BOREHOLE TYPE NW Casing / NQ Coring COMPILED BY KCP
 DATUM Geodetic DATE 2017.12.14 - 2017.12.18 CHECKED BY FG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa			W _P	W	W _L		
184.4								20 40 60 80 100							
0.0	120 mm ASPHALT							20 40 60 80 100							
0.1	Inferred FILL Borehole advanced to the bedrock surface without sampling. Refer to Record of Borehole 17-03 for overburden stratigraphy							20 40 60 80 100							
							184								
							183								
							182								
							181								
							180								
179.2	Inferred TILL						179								
							178								
							177								
							176								
175.1	GRANITE BEDROCK Slightly weathered to fresh Strong to very strong Very poor to fair quality Close to wide joint spacing		1	NQ			175							RUN #1 TCR=100% SCR=17% RQD=17% Vertical fracture	
9.3															

DOUBLE LINE LAUZON CREEK.GPJ 2012TEMPLATE(MTO).GDT 3/8/18

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10


(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 17-03B

2 OF 2

METRIC

GWP# 5074-09-00 LOCATION Lat: 46.1887656°, Long: -82.8082148° MTM Zone 13: N 5 117 246.6 E 396 795.7 ORIGINATED BY KE
 HWY 17 BOREHOLE TYPE NW Casing / NQ Coring COMPILED BY KCP
 DATUM Geodetic DATE 2017.12.14 - 2017.12.18 CHECKED BY FG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE LIQUID CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				WATER CONTENT (%)				
	Continued From Previous Page							20 40 60 80 100								
	GRANITE BEDROCK Slightly weathered to fresh Strong to very strong Poor to fair quality Close to wide spacing Red to grey															
			2	NQ			174									RUN #2 TCR=100% SCR=87% RQD=55% UCS=76MPa
							173									
172.0			3	NQ											RUN #3 TCR=100% SCR=94% RQD=50%	
12.4	End of Borehole															

RECORD OF BOREHOLE No 17-04

1 OF 2

METRIC

GWP# 5074-09-00 LOCATION Lat: 46.1887727° Long: -82.8082056° MTM Zone 13: N 5 117 261.4 E 396 812.8 ORIGINATED BY KE
HWY 17 BOREHOLE TYPE NW Casing / NQ Coring COMPILED BY KCP
DATUM Geodetic DATE 2017.12.04 - 2017.12.05 CHECKED BY FG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					
								20 40 60 80 100		20 40 60			
184.7													
0.0	120 mm ASPHALT												
0.1	Sand with gravel Grey to brown Very dense FILL		1	SS	124								
183.6			2	SS	33								24 55 21 (SH+CL)
1.1	Silty sand with gravel - occasional cobbles Very loose to loose Brown FILL		3	SS	8								
			4	SS	3								
181.7													
3.0	Silty sand trace gravel - trace organics Dense Grey FILL		5	SS	39								
180.9													
3.8	SANDY SILTY CLAY (CL-ML) Very stiff Grey		6	SS	17								6 40 40 14
180.1													
4.6	SILTY SAND with gravel TILL - frequent cobbles and boulders Compact to very dense Grey - boulders from 4.5 m to 6.9 m		7	SS	13								
			8	SS	33								
			9	SS	100/								
			1	NQ	75mm								
177.7			2	NQ									
7.0	GRANITE BEDROCK Slightly weathered to fresh Strong to very strong Fair quality Close to wide joint spacing Red to grey		3	NQ									RUN #3 TCR=100% SCR=94% RQD=53% UCS=242MPa
			4	NQ									RUN #4 TCR=100% SCR=77% RQD=66% UCS=220MPa

Continued Next Page

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

METRIC

[illegible]

RECORD OF BOREHOLE No 17-05

1 OF 2

METRIC

GWP# 5074-09-00 LOCATION Lat: 46.1888306°, Long: -82.8082381°
MTM Zone 13: N 5 117 255.2 E 396 815.4 ORIGINATED BY KE
HWY 17 BOREHOLE TYPE NW Casing / NQ Coring COMPILED BY KCP
DATUM Geodetic DATE 2017.12.18 - 2017.12.19 CHECKED BY FG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										
184.5							20	40	60	80	100							
0.0	150 mm ASPHALT		1	SS	50mm													
0.2	Sand with gravel Grey to brown Very dense FILL				100													
183.7																		
0.8	Silty clayey gravel with sand - occasional cobbles Compact Brown FILL		2	SS	20													
			3	SS	15													
			4	SS	17													
181.5																		
3.0	CLAY (CL) Stiff Grey to red		5	SS	5													
179.7	SILTY SAND (SM) with gravel TILL - frequent cobbles and boulders Compact to very dense Grey - boulders from 5.5 m to 7.5 m		6	SS	27													
			7	SS	100/ 125mm													
			8	SS	51													
			9	SS	100/ 275mm													
176.3			1	NQ														
8.2	GRANITE BEDROCK Slightly weathered to fresh Strong to very strong Fair to good quality Close to wide joint spacing Red to grey		2	NQ														
			3	NQ														
																	</	

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 17-05

2 OF 2

METRIC

GWP# 5074-09-00 LOCATION Lat: 46.1888306°, Long: -82.8082381°
MTM Zone 13: N 5 117 255.2 E 396 815.4 ORIGINATED BY KE
HWY 17 BOREHOLE TYPE NW Casing / NQ Coring COMPILED BY KCP
DATUM Geodetic DATE 2017.12.18 - 2017.12.19 CHECKED BY FG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
	Continued From Previous Page																
173.7	GRANITE BEDROCK Slightly weathered to fresh Strong to very strong		4	NQ			174										RUN #4 TCR=100% SCR=88% RQD=88%
10.8	End of Borehole																

RECORD OF BOREHOLE No 17-06

1 OF 1

METRIC

GWP# 5074-09-00 LOCATION Lat: 46.188738°, Long: -82.8083269° MTM Zone 13: N 5 117 265.8 E 396 823.5 ORIGINATED BY KE
HWY 17 BOREHOLE TYPE NW Casing / NQ Coring COMPILED BY KCP
DATUM Geodetic DATE 2017.12.20 - 2017.12.20 CHECKED BY FG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					
184.7													
0.0	120 mm ASPHALT												
0.1	Sand with gravel Grey to brown Very dense FILL		1	SS	16								
182.8			2	SS	16								
1.9	CLAY (Cl) Stiff to very stiff Grey to red - Vane attempted: S _u > 106 kPa		3	SS	9								1 2 59 38
			4	SS	16								
180.9													
3.8	SANDY SILT (ML) trace gravel TILL - occasional cobbles and boulders Compact to very dense Grey Boulders from 4.5 m to 7.2 m		5	SS	13								
			6	SS	24								
			7	SS	64								5 33 58 4 Non-plastic
			8	SS	100/ 125mm								
			9	SS	100/ 150mm								
176.9													
7.8	End of Borehole Inferred Bedrock												

DOUBLE LINE LAUZON CREEK.GPJ 2012TEMPLATE(MTO).GDT 3/8/18

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

Borehole 17-02

Runs 1 to 3

Elevation 173.2 to 176.4 m

Run 1 Start
elev.176.4 m

Run 1 End
elev.176.0 m

Run 2 Start
elev.176.0 m

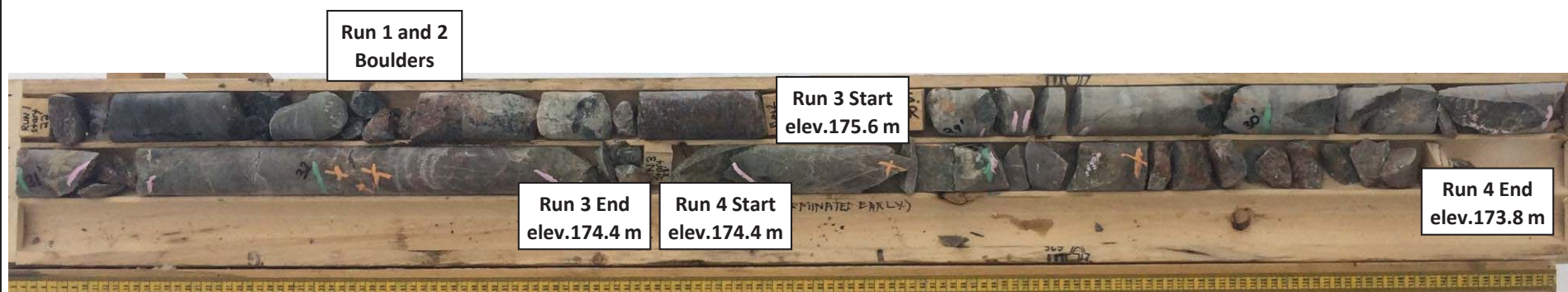
Run 2 End
elev.174.6 m

Run 3 Start
elev.174.6 m

Run 3 End
elev.173.2 m



Borehole 17-03
Runs 3 to 4
Elevation 173.8 to 175.6 m



Borehole 17-03B

Runs 1 to 3

Elevation 171.9 to 175.1 m



Run 3 Start
elev. 172.7 m



THURBER ENGINEERING LTD.

Foundation Investigation
Highway 17 Lauzon Creek Culvert
Algoma Mills, Ontario

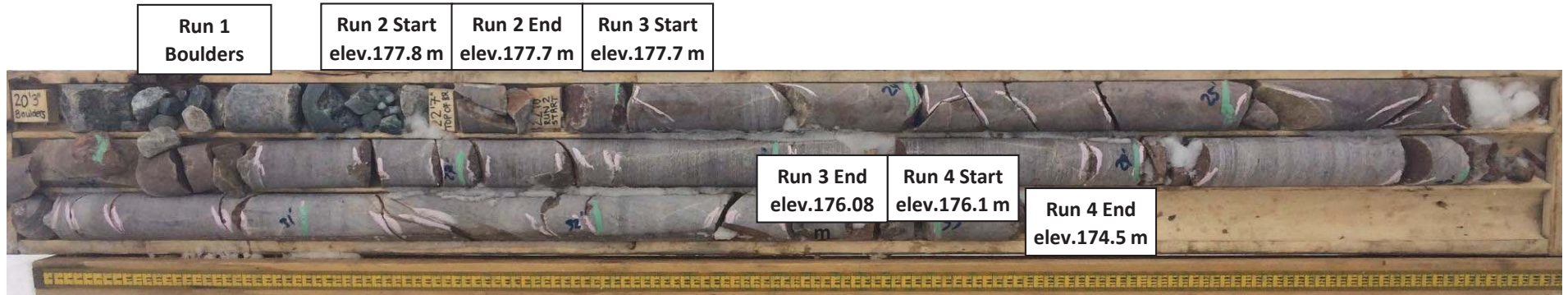
GWP: 5074-09-00

Project No.: 19511

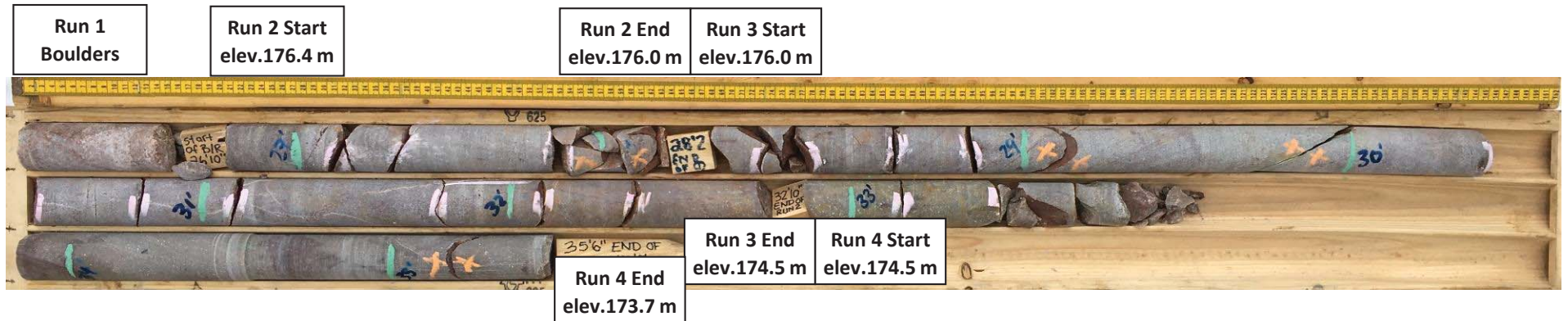
Borehole 17-04

Runs 2 to 4

Elevation 174.5 to 177.8 m



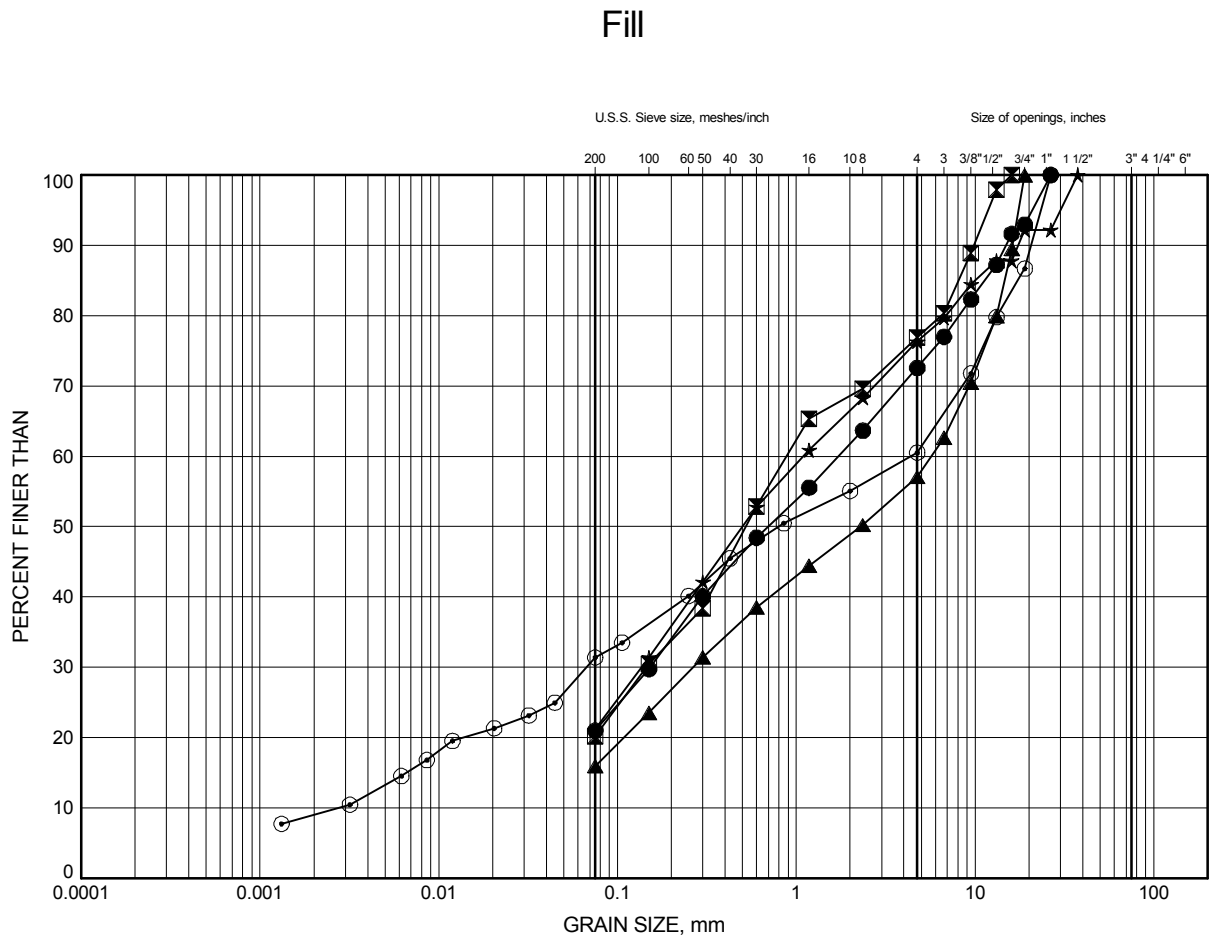
Borehole 17-05
Runs 2 to 4
Elevation 173.7 to 176.4 m



APPENDIX C
LABORATORY TEST RESULTS

Site No. 38S-243 Lauzon Creek Culvert
GRAIN SIZE DISTRIBUTION

FIGURE 1



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	17-01	5.64	178.76
⊠	17-02	3.35	181.25
▲	17-03	1.07	183.33
★	17-04	1.07	183.63
⊙	17-05	1.83	182.67

Date July 2018
 GWP# 5074-09-00

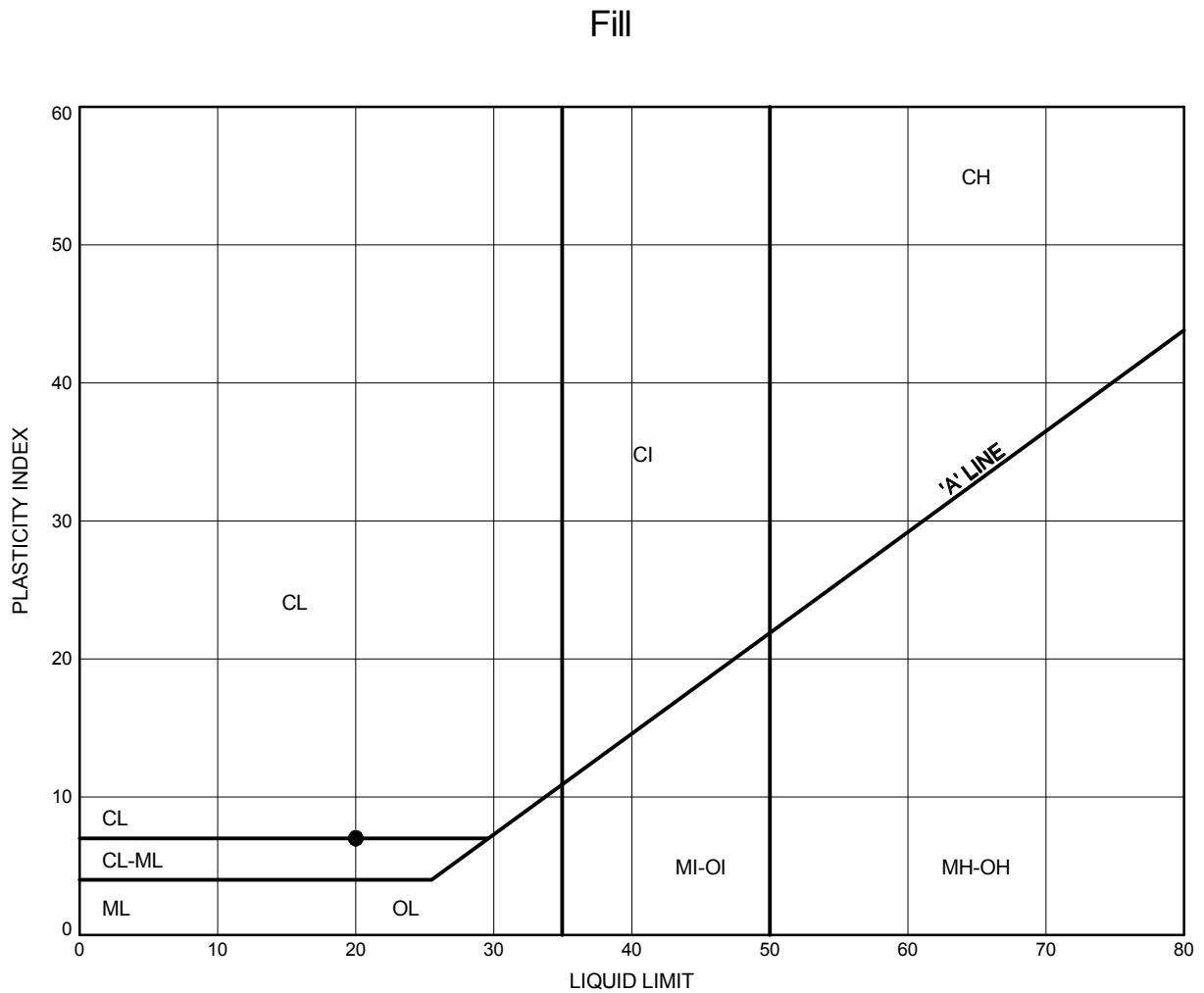


Prep'd KCP
 Chkd. FG

Site No. 38S-243 Lauzon Creek Culvert

ATTERBERG LIMITS TEST RESULTS

FIGURE 2



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	17-05	1.83	182.67

Date July 2018
GWP# 5074-09-00

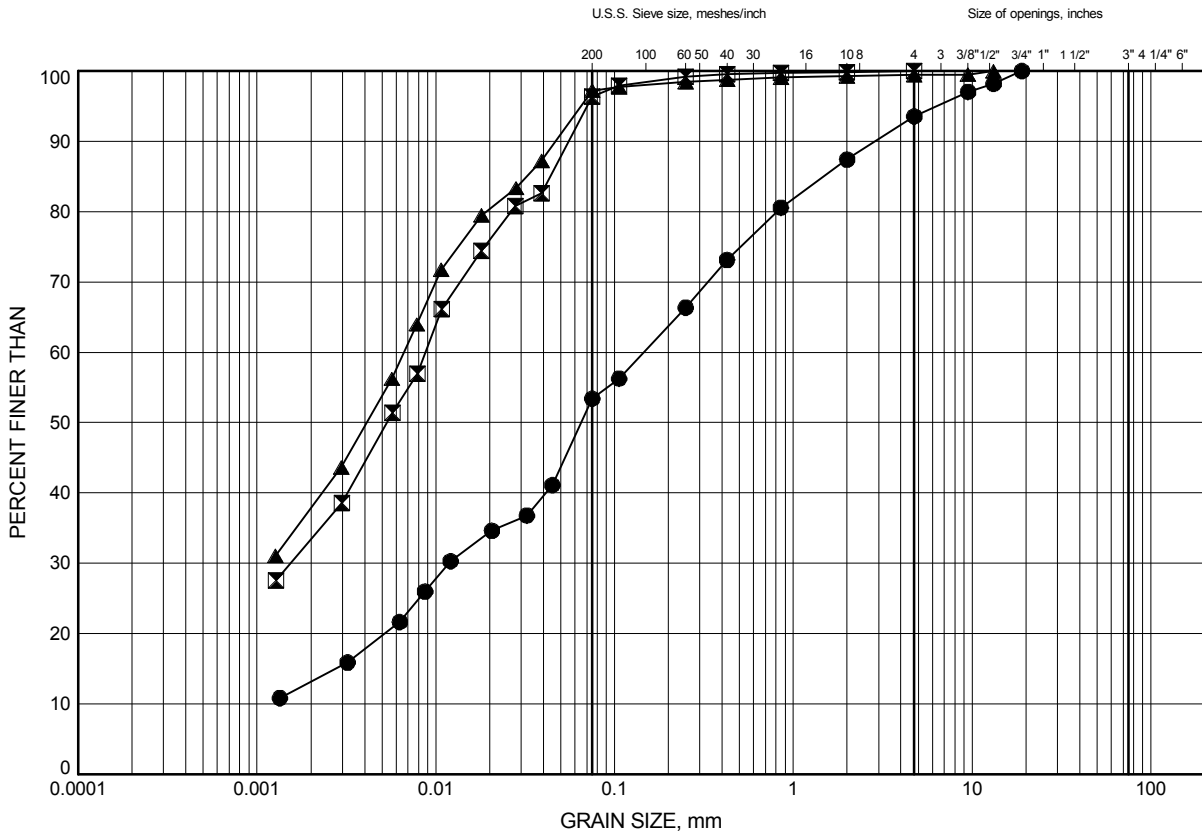


Prep'd KCP
Chkd. FG

Site No. 38S-243 Lauzon Creek Culvert
GRAIN SIZE DISTRIBUTION

FIGURE 3

Sandy Silty Clay to Clay



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	17-04	4.11	180.58
⊠	17-05	3.35	181.15
▲	17-06	2.59	182.11

Date July 2018

GWP# 5074-09-00



Prep'd KCP

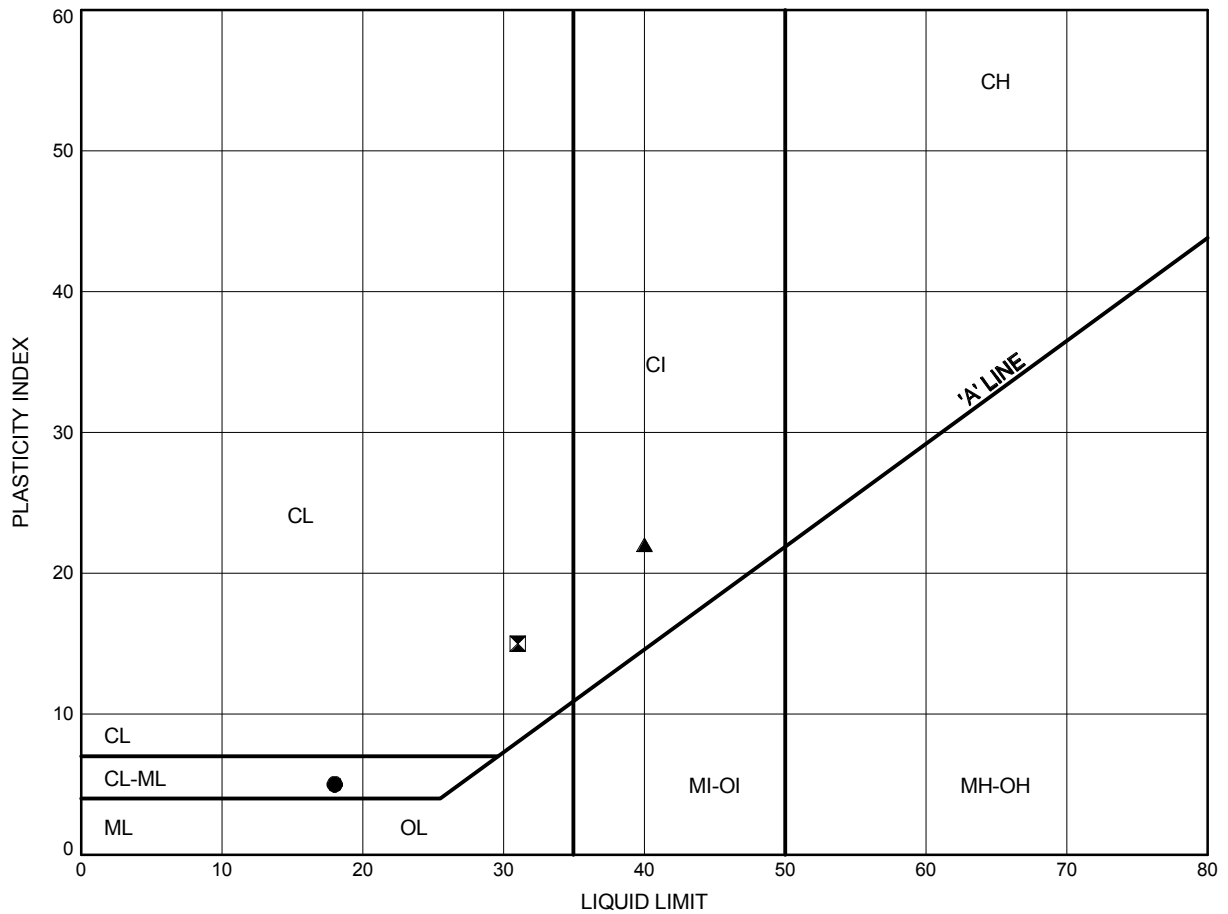
Chkd. FG

Site No. 38S-243 Lauzon Creek Culvert

ATTERBERG LIMITS TEST RESULTS

FIGURE 4

Sandy Silty Clay to Clay



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	17-04	4.11	180.58
⊠	17-05	3.35	181.15
▲	17-06	2.59	182.11

Date July 2018
GWP# 5074-09-00

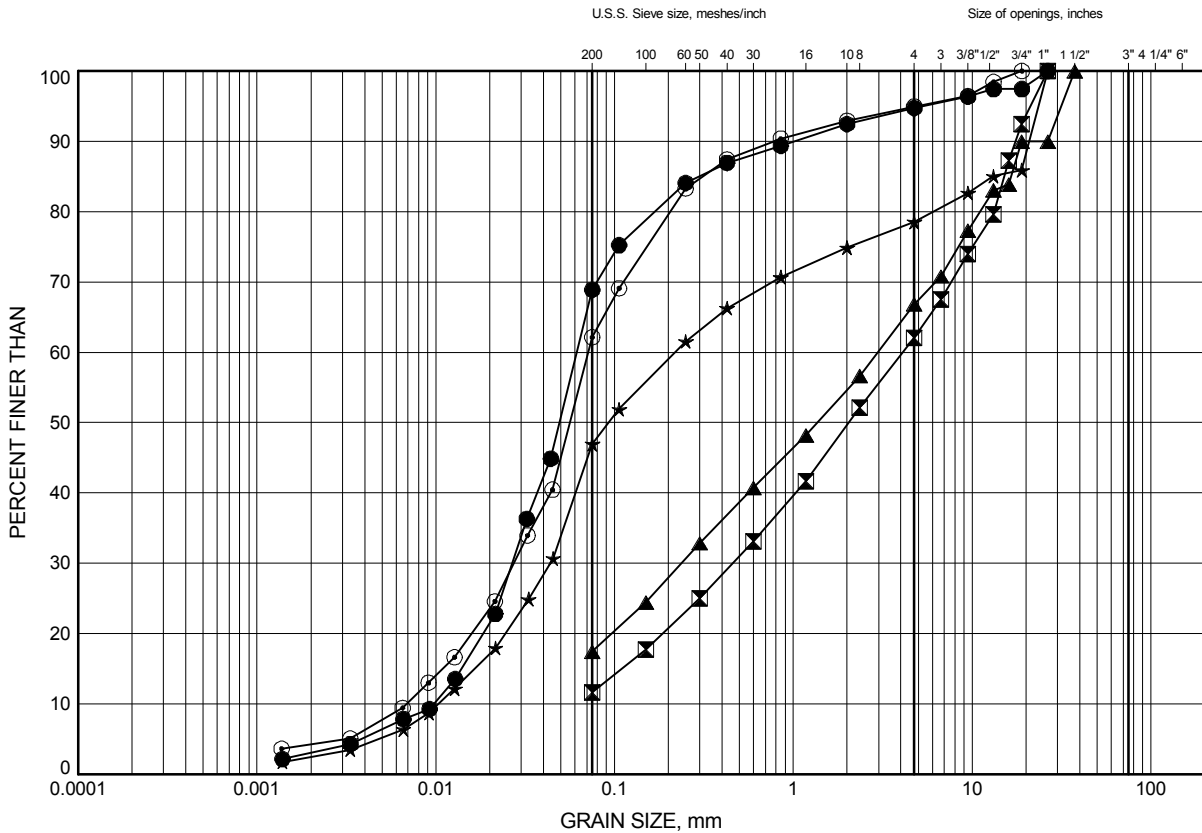


Prep'd KCP
Chkd. FG

Site No. 38S-243 Lauzon Creek Culvert
GRAIN SIZE DISTRIBUTION

FIGURE 5

Glacial Till - Silty Sand to Sandy Silt



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	17-01	7.92	176.48
⊠	17-02	6.17	178.43
▲	17-03	6.40	178.00
★	17-05	6.71	177.79
⊙	17-06	5.64	179.06

Date July 2018
 GWP# 5074-09-00



Prep'd KCP
 Chkd. FG



Stantec

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

January 4, 2018
File: 122410864

Attention: Thurber Engineering Ltd., File #19511

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

The table below summarizes four (4) rock core unconfined compressive strength results.

Location	Sample Depth	Compressive Strength (MPa)	Description of Break
BH 17-3	29'4"-30'	96.7	No well formed cones, diagonal cracks throughout
BH 17-3B	32'9"-33'4"	75.5	Well formed cone on bottom, 2 large cracks through top
BH 17-4	27'2"-28'	242.2	No well-formed cones, core broke into many small pieces
BH 17-4	29'-29'8"	219.8	Well-formed cone on bottom, cracks through rest of core

Sincerely,

Stantec Consulting Ltd

Brian Prevost

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

Certificate of Analysis

Thurber Engineering Ltd.

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

Client PO:
Project: 19511 Lauzon Creek Culvert
Custody: 38416

Report Date: 5-Jan-2018
Order Date: 2-Jan-2018

Order #: 1801062

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

Paracel ID	Client ID
1801062-01	BH17-2 SS9 25'-27'
1801062-02	BH17-2 SS6 12.5'-14.5'

Approved By:



Dale Robertson, BSc
Laboratory Director

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

Report Date: 05-Jan-2018

Order Date: 2-Jan-2018

Project Description: 19511 Lauzon Creek Culvert

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	3-Jan-18	3-Jan-18
pH, soil	EPA 150.1 - pH probe @ 25 °C, CaCl buffered ext.	2-Jan-18	3-Jan-18
Resistivity	EPA 120.1 - probe, water extraction	3-Jan-18	3-Jan-18
Solids, %	Gravimetric, calculation	3-Jan-18	3-Jan-18

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

Report Date: 05-Jan-2018

Order Date: 2-Jan-2018

Project Description: 19511 Lauzon Creek Culvert

Client ID:	BH17-2 SS9 25'-27'	BH17-2 SS6 12.5'-14.5'	-	-
Sample Date:	07-Dec-17	07-Dec-17	-	-
Sample ID:	1801062-01	1801062-02	-	-
MDL/Units	Soil	Soil	-	-

Physical Characteristics

% Solids	0.1 % by Wt.	86.4	90.2	-	-
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General Inorganics

pH	0.05 pH Units	7.18	5.52	-	-
Resistivity	0.10 Ohm.m	93.4	12.7	-	-

Anions

Chloride	5 ug/g dry	30	547	-	-
Sulphate	5 ug/g dry	12	48	-	-

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

Report Date: 05-Jan-2018

Order Date: 2-Jan-2018

Project Description: 19511 Lauzon Creek Culvert

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									
Resistivity	ND	0.10	Ohm.m						

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

Report Date: 05-Jan-2018

Order Date: 2-Jan-2018

Project Description: 19511 Lauzon Creek Culvert

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	29.9	5	ug/g dry	30.1			0.6	20	
Sulphate	11.2	5	ug/g dry	12.2			8.8	20	
General Inorganics									
pH	7.19	0.05	pH Units	7.18			0.1	10	
Resistivity	12.8	0.10	Ohm.m	12.7			0.6	20	
Physical Characteristics									
% Solids	83.7	0.1	% by Wt.	86.4			3.1	25	

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

Report Date: 05-Jan-2018

Order Date: 2-Jan-2018

Project Description: 19511 Lauzon Creek Culvert

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	132	5	ug/g	30.1	102	78-113			
Sulphate	117	5	ug/g	12.2	105	78-111			

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

Report Date: 05-Jan-2018

Order Date: 2-Jan-2018

Project Description: 19511 Lauzon Creek Culvert

Qualifier Notes:

None

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.

APPENDIX D
SITE PHOTOGRAPHS



Photograph 1: Roadway platform at Culvert 38S-243 looking west along Highway 17 (2017/12/21)



Photograph 2: Looking south towards the existing culvert inlet (2017/12/11)



Photograph 3: Looking north towards the existing culvert outlet (2017/12/11)



Photograph 4: North embankment looking east towards culvert inlet (2017/12/04)



Photograph 5: South embankment looking east towards culvert outlet (2017/12/11)

APPENDIX E

2015 NBC SEISMIC HAZARD CALCULATION SLOPE STABILITY ANALYSIS RESULTS LIST OF REFERENCED SPECIFICATIONS

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

January 30, 2018

Site: 46.1888 N, 82.8079 W User File Reference: Lauzon Creek Culvert

Requested by: , Thurber Engineering Ltd.

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

Sa(0.05)	Sa(0.1)	Sa(0.2)	Sa(0.3)	Sa(0.5)	Sa(1.0)	Sa(2.0)	Sa(5.0)	Sa(10.0)	PGA (g)	PGV (m/s)
0.050	0.070	0.070	0.061	0.052	0.034	0.018	0.0042	0.0018	0.040	0.042

Notes. Spectral ($S_a(T)$, where T is the period in seconds) and peak ground acceleration (PGA) values are given in units of g (9.81 m/s^2). Peak ground velocity is given in m/s . Values are for "firm ground" (NBCC 2015 Site Class C, average shear wave velocity 450 m/s). NBCC2015 and CSAS6-14 values are specified in **bold** font. Three additional periods are provided - their use is discussed in the NBCC2015 Commentary. Only 2 significant figures are to be used. *These values have been interpolated from a 10-km-spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the directly calculated values.*

Ground motions for other probabilities:

Probability of exceedance per annum	0.010	0.0021	0.001
Probability of exceedance in 50 years	40%	10%	5%
Sa(0.05)	0.0054	0.017	0.028
Sa(0.1)	0.0092	0.027	0.042
Sa(0.2)	0.011	0.030	0.044
Sa(0.3)	0.010	0.028	0.040
Sa(0.5)	0.0083	0.024	0.035
Sa(1.0)	0.0042	0.015	0.022
Sa(2.0)	0.0017	0.0067	0.011
Sa(5.0)	0.0005	0.0015	0.0025
Sa(10.0)	0.0004	0.0008	0.0012
PGA	0.0051	0.016	0.024
PGV	0.0046	0.017	0.026

References

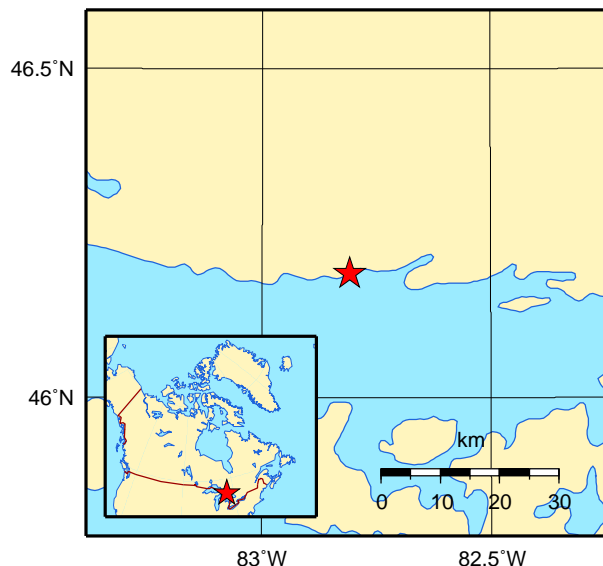
National Building Code of Canada 2015 NRCC no. 56190;
Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada

User's Guide - NBC 2015, Structural Commentaries NRCC no. xxxxxx (in preparation)
Commentary J: Design for Seismic Effects

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

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

Aussi disponible en français



Natural Resources
Canada

Ressources naturelles
Canada



Title: Lauzon Creek Culvert Replacement
Comments: Concrete Gravity Retaining Walls
Name: Seismic Condition

Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m
PWP Conditions Source: Piezometric Line
Seismic, H: 0.02 V: 0
Slip Surface Center: (2.1166667, 188.13333) w/ Radius: 11.633333 m
FoS Contours: 1.6 to 2.6, ++0.1

Embankment Fill	20 kN/m ³	0 kPa	30 °
Till	21 kN/m ³	0 kPa	35 °
Bedrock			
Concrete	24 kN/m ³		
Clay	18 kN/m ³	80 kPa	0 °

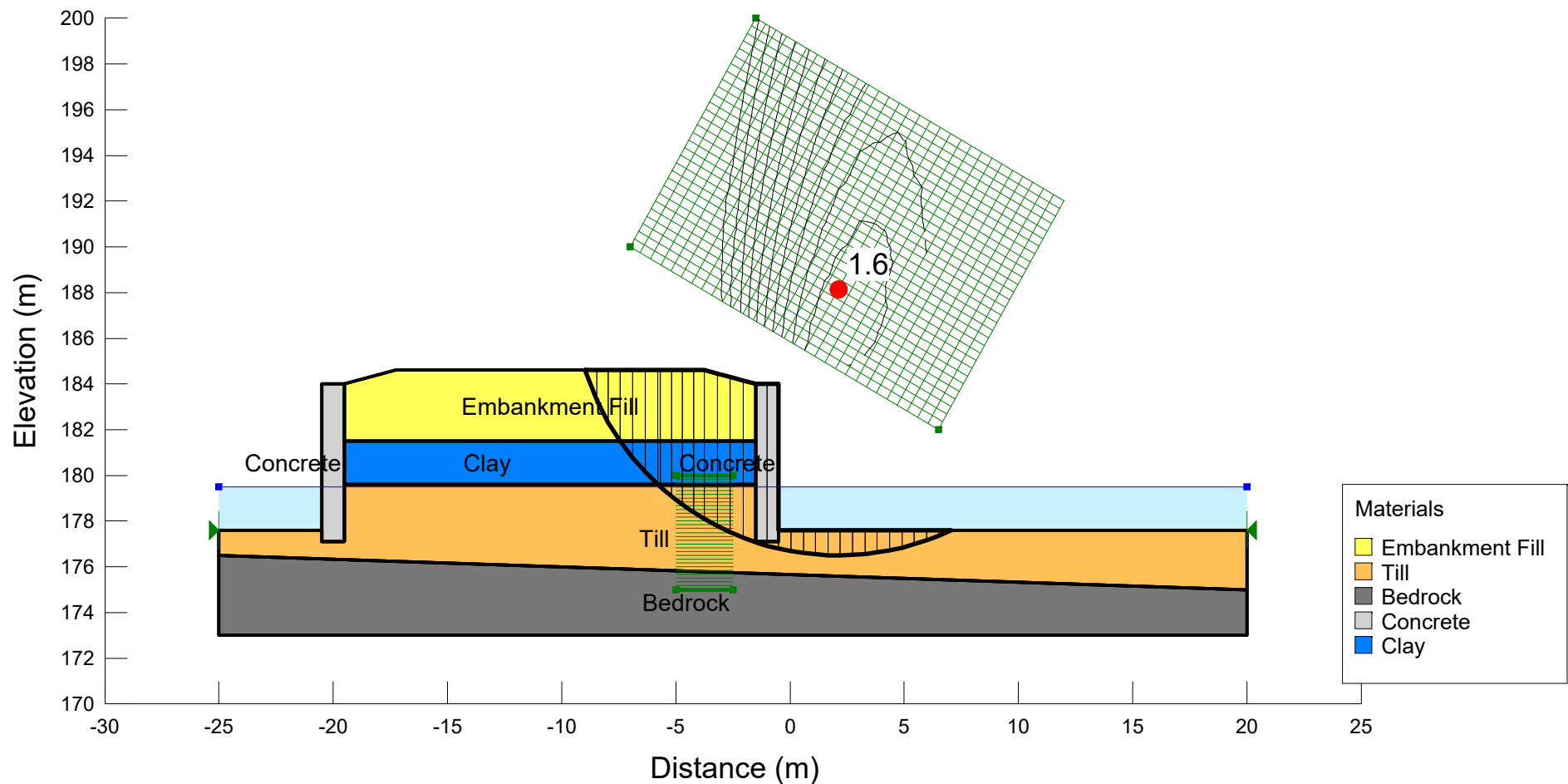


Figure 2

Title: Lauzon Creek Culvert Replacement
Comments: Concrete Gravity Retaining Walls
Name: Static Condition

Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m
PWP Conditions Source: Piezometric Line
Seismic, H: 0 V: 0
Slip Surface Center: (2.1166667, 188.13333) w/ Radius: 11.633333 m
FoS Contours: 1.6 to 2.6, ++0.1

Embankment Fill	20 kN/m ³	0 kPa	30 °
Till	21 kN/m ³	0 kPa	35 °
Bedrock			
Concrete	24 kN/m ³		
Clay	18 kN/m ³	80 kPa	0 °

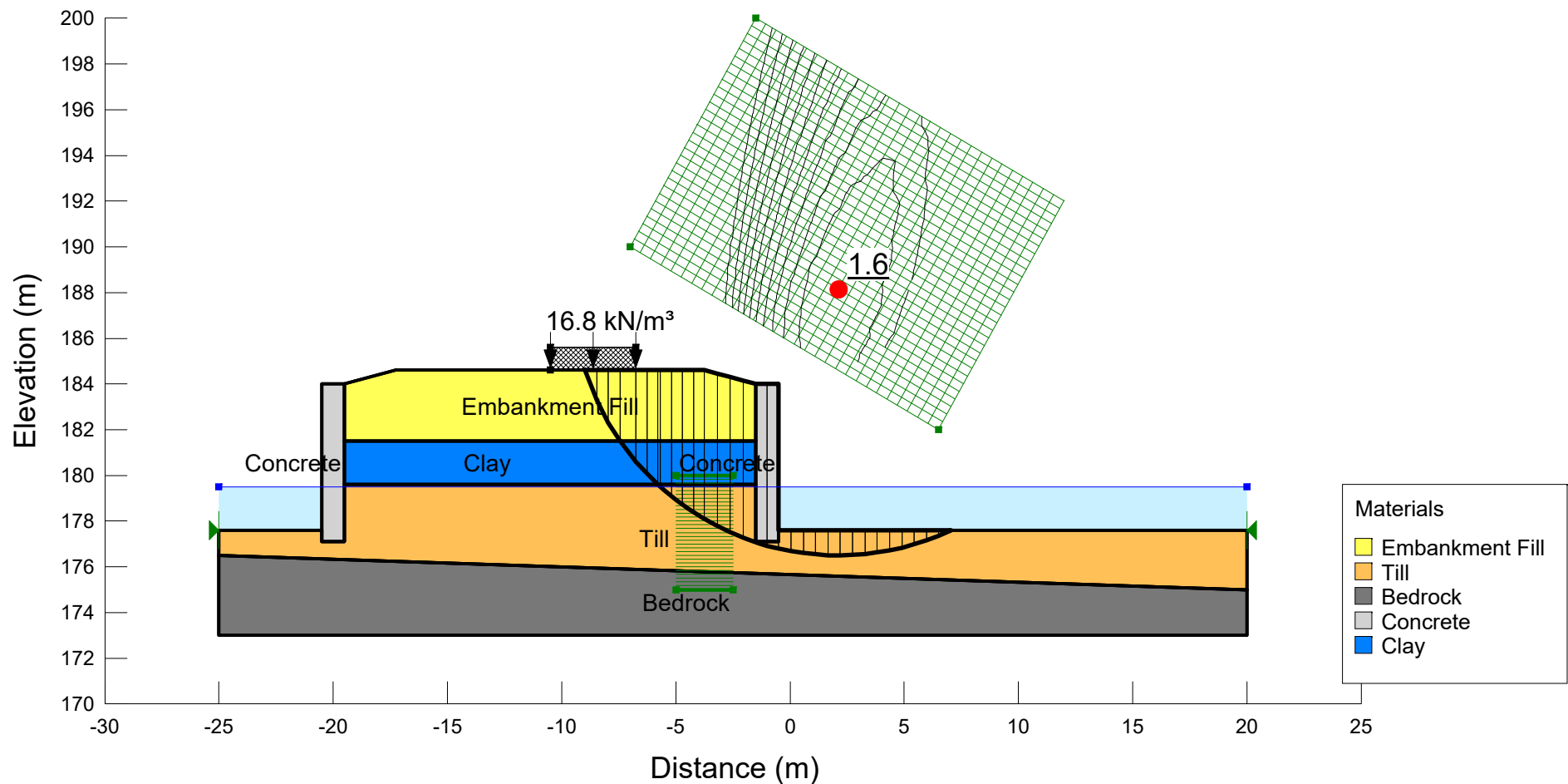


Figure 1

LIST OF REFERENCED SPECIFICATIONS

OPSD 208.010	Benching of Earth Slopes
OPSD 3090.101	Foundation, Frost Penetration Depths for Southern Ontario
OPSS.PROV 180	General Specification for the Management of Excess Materials
OPSS.PROV 206	Construction Specification for Grading
OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 517	Construction Specification for Dewatering
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS.PROV 804	Construction Specification for Seed and Cover
OPSS 805	Construction Specification for Temporary Erosion and Sediment Control Measures
OPSS 902	Construction Specification for Excavating and Backfilling-Structures
OPSS.PROV 903	Construction Specification for Deep Foundations
OPSS.PROV 1010	Material Specification for Aggregates-Base, Subbase, Select Subgrade, and Backfill Material
Special Provision 109S12	Amendment to OPSS 902, March 2018
Special Provision FOUN0003	Dewatering Structure Excavations, March 2018

APPENDIX F
NON-STANDARD SPECIAL PROVISIONS

SUGGESTED TEXT FOR “NSSP – 902.07.05 EXCAVATION”

Subsection 902.07.05 of OPSS 902 is amended by the addition of the following:

Excavations at the site may be impeded by obstructions within the existing fill and glacial till. The contractor shall be prepared to dislodge and remove these obstructions and extend the excavations to the design depths.

SUGGESTED TEXT FOR “NSSP ON INSTALLATION OF STEEL SHEET PILES”

Cobbles and boulders are present within the existing embankment fill and the underlying glacial till. Rock protection and boulders are present in the embankment slopes and toes. These cobbles and boulders may impede the driving of sheet piles and at some locations the sheet piles may not be able to penetrate the cobbles and boulders and reach the design depth of installation.

The Contractor shall be prepared to remove, drill through and/or penetrate these obstructions and extend the piles to the design depth.

RECOMMENDED WORDING FOR “NSSP – PRESENCE OF EXISTING FOUNDATIONS”

The Contractor shall note that the proposed foundations are to be installed adjacent to the existing culvert foundations and that existing foundations are to remain in place. Although the foundation layout on the structural drawings has been selected to minimize conflict with existing foundations the potential for conflict still exists. The Contractor shall review the installation program with respect to the theoretical locations of the existing and proposed foundations as shown on the General Arrangement drawings provided in the contract package. Prior to commencing excavations for the new foundations, the Contractor must expose the tops of the existing foundations and check for possible conflicts. Should the excavations for the new foundations conflict with the existing foundations the Contractor shall report the conflict to Contract Administrator to determine if adjustment to the foundations layout is required. The Contractor shall be prepared to remove existing foundations should it be deemed necessary by the Contract Administrator.

Preparation of the spread footing foundations for the culvert includes removal of existing material to bedrock and could extend to an elevation below the underside of the existing foundations. Where there is a risk of undermining the existing foundations, the Contractor shall complete the subgrade preparation work for the new foundations in 2 m increments such that the existing foundations are not compromised.

RECOMMENDED WORDING FOR “NSSP – SPECIAL PROVISION FOUN0003 – DEWATERING STRUCTURE EXCAVATIONS”

Subsection 902.04.01 Design Requirements of SP FOUN0003 is amended by the addition of the following:

The design Engineer and design-checking Engineer of the dewatering system shall have a minimum of 5 years of experience in designing systems of similar nature and scope to the required work.