



THURBER ENGINEERING LTD.

**FOUNDATION INVESTIGATION AND DESIGN REPORT
REHABILITATION OF STRUCTURAL CULVERT No. 29-155/C
HIGHWAY 60 DOUGLAS CREEK CULVERT
TOWNSHIP OF ADMASTON, ON
G.W.P. 4076-13-00
AGREEMENT NUMBER: 4016-E-0014**

GEOCRES NUMBER: 31F-203

**SUBMITTED TO
McINTOSH PERRY CONSULTING ENGINEERS**

**LOCATION:
LATITUDE: 45.52117°
LONGITUDE: -76.86081°**

**AUGUST 2018
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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) at the Douglas Creek Culvert located on Highway 60, within the Township of Admaston, Ontario. Thurber carried out the investigation as a subconsultant to McIntosh Perry Consulting Engineers (MPCE) under Agreement No. 4016-E-0014.

A base plan and a General Arrangement (GA) Drawing were provided by MPCE for the preparation of this report.

The purpose of this investigation was to explore the subsurface conditions at the Douglas Creek Culvert 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 BACKGROUND

At the time of Thurber's investigation, the northwest wingwall was noted as having rotated significantly from the culvert (see Photographs 3 and 4 provided in Appendix D) and some rotation of the northeast wingwall was also noted (see Photograph 5).

The results of the Ministry's 2011 and 2015 inspection reports indicate that both the northeast and northwest wingwalls at the Douglas Creek Culvert inlet have rotated away from the culvert and should be replaced.

The GEOCRES library includes a Memorandum (31F-172) for the Douglas Creek Culvert dated December 20, 1988, that indicates that the culvert had similar deficiencies as noted in the more recent inspection reports (i.e. rotation of inlet wingwalls). It also indicates that further investigation was required to determine the cause, however, no additional documentation was available to indicate whether further investigation or remedial work had been completed.

Based on historical construction drawings from 1963, bedrock along the culvert alignment varies from bedrock outcropping at approximate Elevation 115.2 m within the culvert near the south end (outlet), to below Elevation 113.0 m at the north end of the culvert (inlet); see Appendix A. The drawings indicate that the wingwall foundations at the inlet were designed to be founded on bedrock at elevations of 113.7 m and 114.1 m at the northwest and northeast sides of the culvert respectively.

It is noted that the culvert depicted in the 1963 construction drawings replaced an earlier structure which was located to the west of the new alignment.

3 SITE DESCRIPTION

Culvert 29-155/C is located at approximate Station 11+200 on Highway 60, approximately 200 m east of the Highway 60 / Barr Line intersection in Renfrew County, 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 60 within the project limits, will be assumed to run west-east. Flow through the culvert is from north to south.

At the project site Highway 60 is undivided with one through lane in each direction. Based on the drawings provided, the roadway cross-section at the culvert location consists of two, 3.7 m wide lanes and narrow gravel shoulders. Steel cable guiderails are present along both side of the highway in the vicinity of the culvert.

The existing 34.5 m long concrete, open footing culvert has an internal span of 6.1 m and an internal height of 3.0 m. The culvert includes concrete headwalls and wingwalls at both the inlet and outlet. The June 2018 GA Drawing, see Appendix A, indicates that the asphalt surface of the highway is at elevation 122.9 m and the cover over the culvert from shoulder to the top of the culvert is approximately 4.0 m. The creek bed is at approximate elevation 114.5 m upstream of the inlet based on the section drawings provided.

Based on the 1963 construction drawings the inlet wingwalls were to be constructed as outlined in Table 3-1.

Table 3-1: Design dimensions of Existing Inlet Wingwalls

Location	Exposed Height (m)		Length (m)	Underside of Footing Elevation (m)
	Top	Toe		
Northwest wingwall	4.5	2	8.3	113.7
Northeast wingwall	4.0	2	5.1	114.1

No settlement or stability or scour/erosion of the culvert foundations at the inlet were noted at the time of Thurber's field investigation. Scour/erosion was also not observed at the inlet wingwall foundations.

The culvert is located within a high fill section. The existing north embankment is sloped at approximately 2H:1V (Horizontal:Vertical) and is grass and brush covered; some trees are also present. No signs of settlement or erosion of the embankment slopes were noted at the time of the investigation. Boulders were present in front of the inlet and along the ditchline/toe of slope to the east of the culvert. A post and wire fence and overhead utility lines are present along the ditchline to the north of the culvert inlet.

Storm water drainage in the area is to ditches and the creek. It should be noted that the creek overtopped its banks and entered the investigation area to the north of the inlet during the field work.

Site photographs showing the general conditions at the site, and that of the inlet wingwalls are presented in Appendix D.

4 SITE INVESTIGATION

4.1 Field Investigation

The field investigation was carried out between February 6th and 18th, 2018, and included advancing four boreholes. The approximate MTM Zone 9 locations and ground surface elevations of the boreholes are shown on Drawing No. 1, provided in Appendix A and are summarized in Table 4-1. The structural inspection reports only noted rotational deformations at the north end (inlet) of the culvert hence, all boreholes were located at the culvert inlet.

Table 4-1: Borehole Summary

Borehole	Location	Northing (m)	Easting (m)	Ground Surface Elevation (m)	Borehole Termination Elevation (m)
18-1	Behind NW wingwall	5042427.0	276601.0	118.4	109.8
18-2	Toe of NW wingwall	5042431.9	276598.6	116.1	110.5
18-3	Behind NE wingwall	5042425.2	276615.1	118.3	110.5
18-4	Toe of NE wingwall	5042429.3	276614.6	116.0	112.4

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 portable drill rig with a half-weight hammer, equipped with NQ size coring equipment. 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. The SPT N values presented on the Record of Boreholes and summarised in the following sections have been corrected to provide an estimate of the SPT N value that would have been obtained with a standard weight hammer. The subsurface stratigraphy encountered in the boreholes was recorded in the field by Thurber personnel. 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 all boreholes 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.

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 top of footing elevation of the northeast footing line at the culvert inlet. The location of the TBM is indicated on Drawing No. 1 in Appendix A. The geodetic elevation of 115.210 m was used for the TBM as indicated on the 1963 construction drawings for the culvert.

4.2 Laboratory Testing

Geotechnical laboratory testing consisted of natural moisture content determination and visual identification of all soil samples. Grain size distribution analyses and Atterberg Limits testing were carried out on selected samples to MTO and ASTM standards. All recovered bedrock core was logged and core recoveries and Rock Quality Designation (RQD) values were determined.

Unconfined compressive 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.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

5.1 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 inlet wingwalls are presented on Drawing No. 1 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 granular embankment fill, over clay fill, overlying till overlying granite bedrock. A buried concrete layer was noted in Borehole 18-2. Cobbles and boulders were noted at the surface of Borehole 18-4 and cobbles were noted within the till stratum.

More detailed descriptions of the individual strata are presented below.

5.2 Boulders and Cobbles

A 600 mm thick layer of boulders and cobbles was present at the ground surface of Borehole 18-4. The base of this layer was at Elevation 115.4 m. Cobbles and boulders were present in front of the inlet and along the ditchline/toe of slope to the east of the culvert.

5.3 Fill

Sand with Silt some Gravel to Silty Sand some Gravel Fill

A fill layer consisting predominantly of sand with varying amounts of gravel and silt was encountered at the ground surface in all boreholes except Borehole 18-4. The top of this layer was encountered at elevations ranging from 116.1 m to 118.4 m. The thickness of the layer ranged from 0.7 m to 3.7 m. The SPT N values ranged from 5 to greater than 100; indicating a loose to very dense condition however, the ground was frozen at the time of the investigation which may have increased the values.

The moisture content of the samples tested ranged from 5% to 51%. The results of grain size analysis tests on three samples of the sand fill material indicated a gravel content ranging from 7% to 10%, a sand content ranging from 78% to 86%, and a fines content (combined silt and clay size particles) ranging from 5% to 15%. Grain size analysis results are illustrated on Figure 1 in Appendix C.

Clay with Sand and Silt to Clay with Silt some Gravel Fill

A fill layer consisting predominantly of clay and silt with varying amounts of sand and gravel was encountered below the sand fill layer in Boreholes 18-2 and 18-3, and below the cobble and boulder layer in Borehole 18-4. The top of the clay fill layer was encountered at elevations ranging from 115.4 m to 116.5 m. The thickness of the layer ranged from 1.2 m to 2.1 m. The SPT N values ranged from 4 to 48; indicating a firm to very stiff consistency but typically stiff. Cobbles were noted at the base of this layer and coring techniques were required to penetrate the layer.

The moisture content of the samples tested ranged from 21% to 39%. The results of grain size analysis tests on samples of the clay fill material indicated a gravel content ranging from 1% to 14%, a sand content ranging from 13% to 35%, a silt content ranging from 29% to 39% and a clay content ranging from 22% to 47%. Grain size analysis results are illustrated on Figure 2 in Appendix C.

The results of Atterberg Limits testing completed on samples of the clay fill material indicated a liquid limit ranging from 34 to 47, a plastic limit ranging from 19 to 22, and a plasticity index ranging from 15 to 25. Atterberg Limits analysis results are illustrated on Figure 3 in Appendix C, and indicate a clay with a low to intermediate plasticity.

5.4 Concrete

Buried concrete with a thickness of 700 mm was encountered in Borehole 18-2 underlying the clay fill layer at Elevation 113.3 m. Coring techniques were required to penetrate the concrete. A photograph of the recovered material is provided in Appendix B.

5.5 Till

Silty Sand with Gravel Till

A grey silty sand with gravel till deposit was encountered beneath the fill layer in Borehole 18-1. The top of this layer was encountered at Elevation 114.7 m and the layer had a thickness of 2.5 m. The SPT N values ranged from 13 to greater than 100; indicating a compact to very dense condition. Cobbles were noted in this layer.

The moisture content of the samples tested ranged from 14% to 23%. The results of a grain size analysis test indicated a gravel content of 18%, a sand content of 46%, a silt content of 28% and a clay content of 8%. Grain size analysis results are illustrated on Figure 4 in Appendix C.

The results of Atterberg Limits testing completed on a sample of this material indicated the fines were non-plastic.

Sandy Clay with Gravel to Gravely Clay with Sand Till

A clay till layer with sand and gravel was encountered below the fill and the buried concrete in Borehole 18-2 and beneath the fill layers in Boreholes 18-3 and 18-4. The top of this layer was encountered at elevations ranging from 112.5 m to 114.6 m. The thickness of the layer ranged from 0.4 m to 0.6 m. The SPT N values ranged from 4 to greater than 100; indicating a firm to very stiff consistency. Cobbles were noted at the base of this layer and coring techniques were required to penetrate the layer.

The moisture content of the samples tested ranged from 25% to 35%. The results of grain size analysis tests conducted on two samples of the clay till material indicated a gravel content ranging from 18% and 27%, a sand content ranging from 21 and 27%, a silt content ranging from 29% and 30% and a clay content ranging from 23% and 25%. Grain size analysis results are illustrated on Figure 5 in Appendix C.

The results of Atterberg Limits testing completed on two samples of the clay till material indicated a liquid limit of 31 and 35, a plastic limit of 17, and a plasticity index of 14 and 18. Atterberg Limits analysis results are illustrated on Figure 6 in Appendix C, and indicate a clay with a low to intermediate plasticity.

5.6 Granite Bedrock

The overburden materials were underlain by a grey granite bedrock. All four boreholes were advanced into bedrock by coring with NQ-size coring equipment. Photographs of the bedrock core are provided in Appendix B.

A summary of the bedrock surface elevation is provided in Table 5-1.

Table 5-1: Bedrock Summary

Borehole	Location	Ground Surface Elevation at Borehole Location (m)	Depth Below Existing Grade (m)	Top of Bedrock Elevation (m)
18-1	Behind NW wingwall	118.4	6.3	112.1
18-2	Toe of NW wingwall	116.1	4.2	111.9
18-3	Behind NE wingwall	118.3	4.3	114.0
18-4	Toe of NE wingwall	116.0	2.2	113.8

The total core recovery ranged from 65% to 100%, the solid core recovery ranged from 65% to 100% and the RQD ranged from 42% to 100%. Unconfined compressive strength testing was carried out on five samples of the bedrock; (see results in Appendix C). The results ranged from 85.7 MPa to 217.7 MPa.

Based on the RQD value the bedrock is classified as poor to excellent quality. Based on unconfined compressive strength testing the bedrock is strong to very strong.

5.7 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 Douglas Creek Culvert was surveyed on February 16, 2018, at Elevation 114.9 m.

Seasonal fluctuations of the water level in the culvert is to be expected. In particular, the water level may be at a higher elevation after the spring snowmelt or after periods of heavy rainfall. It should be noted that the creek overtopped its banks and entered the area to the north of the inlet during the investigation.

5.8 Analytic Test Results

Two samples of the 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 5-2. A copy of the test results is provided in Appendix C.

Table 5-2: Results of Chemical Analysis

Borehole	Sample	Depth (m)	pH	Resistivity (Ohm-cm)	Chloride (µg/g)	Sulphate (µg/g)
18-1	SS3	1.5	8.5	10500	19	<5
18-3	SS4	2.1	8.3	3170	61	14

6 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 construction drawings provided by McIntosh Perry Consulting Engineers. Forage M3 Drilling Services Inc. of Hawkesbury, Ontario supplied and operated the drilling equipment to carry out the drilling, sampling, and in-situ testing. Beacon Lite Ltd. of Ottawa provided traffic control services for lane closures during set-up and tear down of the drilling equipment. The drilling, and sampling operations in the field were supervised on a full-time basis by Nick Weil and 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 Paul Carnaffan, 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

7 GENERAL

This report presents the interpretation of the factual data obtained from a foundation investigation conducted by Thurber at the Douglas Creek Culvert located on Highway 60, in the Township of Admaston, Ontario. Geotechnical assessment and recommendations are provided to assist the design team in review of possible replacement alternatives and recommending an appropriate replacement for the inlet wingwalls from a geotechnical engineering perspective.

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 rehabilitation/replacement of the inlet wingwalls of the Douglas 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.

7.1 Geotechnical Conclusions

Based on available construction drawings and the results of the current investigation, the cause of the rotation of the existing wingwalls is likely due to the type of backfill placed behind the walls and the footings not being placed consistently on bedrock.

The 1963 construction drawings indicate that the northwest wingwall was designed to be founded on bedrock at an elevation of 113.7 m. Boreholes advanced in the vicinity of the northwest wingwall indicated that bedrock surface was at an elevation of 112.0 m. The observed rotation at the northwest wingwall may have been caused by the reduced bearing capacity of the till material versus the design value assumed for the bedrock.

As noted on the Record of Borehole sheet for Borehole 18-3, the fill behind the wingwall consists in part of clay with silt and sand which is not considered free draining and could allow for an accumulation of groundwater with an increase in hydrostatic pressures acting on the back of the walls. The geotechnical driving force may be higher than that contemplated in the original design. The accumulation of water in the moderately frost susceptible soils could also increase the

likelihood of frost action. The buildup of hydrostatic pressure and frost action would increase the pressure on the back of the wall, contributing to the observed rotation of the wingwalls.

7.2 Proposed Culvert Rehabilitation

The existing, open footed, concrete culvert is 34.5 m long, and has an internal span of 6.1 m and an internal height of 3.0 m. The existing wingwalls at the inlet have a maximum height of 4.5 m and 4.0 m at the west and east walls respectively that taper down to a height of 2.0 m. The west wingwall is 8.3 m long, while the east wingwall is 5.1 m long. The walls are set at obtuse angles to the culvert headwall.

The June 2018 GA drawing indicates that culvert rehabilitation includes the replacement of the inlet wingwalls with cast-in-place concrete walls, along the existing alignments. The footprint area for the west wingwall is to be approximately 2.4 m by 8.3 m while the east wingwall footprint area is to be approximately 2.2 m by 5.1 m. It is understood that the existing culvert will not be replaced as part of the current rehabilitation project and will remain operational during construction.

Temporary protection systems, cofferdams and temporary creek water diversion may be required to access the culvert/wingwall foundations for the rehabilitation/replacement of the wingwalls.

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

It is understood that the replacement of the inlet wingwalls is being designed to the Major Route importance category.

This project has been assigned Typical Consequence Classification, in accordance with Section 6.5.1 of the CHBDC. Accordingly, a consequence factor (Ψ) of 1.0, as per Table 6.1 of the CHBDC, has been used in assessing factored geotechnical resistances.

7.4 Geotechnical Assessment

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

- The depth to bedrock varies at the site with the bedrock surface encountered higher at the northeast wingwall than at the northwest wingwall. Bedrock was encountered at as low as Elevation 111.9 m at the northwest wingwall and at Elevation 113.8 m at the northeast wingwall.
- Construction drawings indicate that the existing inlet wingwalls were designed with spread footings founded at approximate elevations of 113.7 m and 114.1 m at the

northwest and northeast wingwalls respectively, hence the existing footings may not be founded on bedrock.

- The existing overburden soils at the site are not consistent beneath the proposed foundations and should be removed from within the footprint of the wingwall foundations during subgrade preparation.
- The bedrock encountered is considered suitable to support shallow foundations with moderate to high bearing resistance. Wingwalls should be founded on sound, level bedrock.
- A temporary protection system will be required in order to protect the existing embankment slope during construction.
- Excavation below the creek water level to construct the wingwall foundations will be required. An adequate and effective surface water management plan, construction of a cofferdam and dewatering plan must be implemented to enable excavation to the required founding elevation of the wingwall foundations.
- A temporary flow passage system may be required. It should be noted that the creek overtopped its banks during the investigation.
- The frost penetration depth at this site is 1.8 m as per OPSD 3090.101. Foundations placed directly on sound bedrock do not need full frost protection cover.

8 SEISMIC CONSIDERATIONS

8.1 Spectral and Peak Acceleration Hazard Values

The seismic hazard data for the CHBDC is based on the fifth-generation seismic model developed by the Geological Survey of Canada (GSC). Seismic hazard data for this site has been obtained from the GSC's seismic hazard calculator. The data includes peak ground acceleration (PGA), peak ground velocity (PGV), and the 5% damped spectral response acceleration values ($S_a(T)$) 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 E.

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) which is 0.218g for this location for a seismic event with a 2% probability of exceedance in 50 years.

8.2 CHBDC Seismic Site Classification

In accordance with the CHBDC, the selection of the seismic site classification is based on the soil conditions encountered in the upper 30 m of the stratigraphy.

Based on the soil and bedrock conditions encountered and the anticipated wingwall backfill elevations, the site is classified as a Seismic Site Class C in accordance with Table 4.1 of the CHBDC for lateral earth pressure design.

8.3 Seismic Liquefaction

It is anticipated that the wingwalls will be founded on bedrock which is not considered susceptible to liquefaction from the design seismic event.

9 FOUNDATION DESIGN RECOMMENDATIONS

9.1 Foundation Design Alternative

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:

- | | |
|------------------------------------|-----------------------------|
| • Existing top of culvert (NE-NW) | 118.6 m – 118.7 m |
| • Top of east culvert footing line | 115.2 m |
| • Creek water at inlet | 114.9 m (February 16, 2018) |
| • Bedrock surface at NW wingwall | 111.9 m – 112.1 m |
| • Bedrock surface at NE wingwall | 113.8 m – 114.0 m |

Based on the results of unconfined compressive strength testing the granite bedrock encountered is strong to very strong.

Considering the relatively shallow depth to bedrock, the height and backslope of the retained material and the current deficiencies of the inlet wingwalls, it is recommended that the new wingwall foundations be placed on bedrock.

9.2 Wingwall Foundation Bearing Resistances

Wingwall footings between 2.4 and 3.5 m in width founded on or in the existing bedrock, prepared as outlined in Section 9.3, may be designed based on a factored geotechnical resistance at ULS of 2,000 kPa. The factored geotechnical resistance includes 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)

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

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.

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

Resistance to lateral forces and sliding resistance between cast-in-place concrete and bedrock should be evaluated using an unfactored coefficient of friction of 0.7. Consideration could be given to doweling the wingwall foundations into the bedrock for additional overturning and sliding resistance, if required.

9.3 Subgrade Preparation and Backfilling

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

Subgrade preparation for the wingwalls should include excavation and removal of the existing overburden material, walls and foundations to the bedrock surface. All loose rock should be removed from the sidewalls and base of the excavations. The bedrock surface should be cleaned and levelled with mass concrete prior to the placement of the foundations.

It is considered critical to the design of the replacement wingwalls that the backfill behind the walls consist of a free draining material and that proper drainage measures, such as subdrains and weep holes, be provided behind the wingwalls to prevent any buildup of hydrostatic groundwater pressure acting on the wingwalls.

Backfill behind the wingwalls should consist of compacted Granular A material meeting the specifications of OPSS.PROV 1010, placed and compacted in accordance with OPSS.PROV 501.

9.4 Lateral Earth Pressures

The lateral earth pressure parameters provided in Tables 9-1 and 9-2 in the sections below are based on the assumption that the backfill is fully drained so that there are no unbalanced hydrostatic pressures. If adequate drainage cannot be confirmed, the potential for buildup of hydrostatic pressures should be considered in the design.

If the sloped embankment or backfill above the wingwalls or temporary protection systems is different from those presented in Tables 9-1 and 9-2, the lateral earth pressure parameters provided do not apply and recalculation of the earth pressure parameters will be required.

9.4.1 Static Lateral Earth Pressure Coefficients

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

$$P_h = K^*(\gamma h + q)$$

where:

- P_h = horizontal pressure on the wall (kPa)
- K = earth pressure coefficient
- γ = unit weight of retained soil (kN/m³); use submerged unit weight for soils below the groundwater level
- h = depth below top of fill where pressure is computed (m)
- q = value of any surcharge (kPa)

The recommended lateral earth pressure parameters for use in the design of vertical walls for both horizontal and 2H:1V backslopes are provided in Table 9-1.

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.

Table 9-1: Static Lateral Earth Pressure Coefficient

Parameter	OPSS Granular A & B Type II	Existing Granular Fill
Soil Unit Weight, kN/m^3 , γ	22.8	20.0
Angle of Internal Friction, ϕ	35°	32°
Horizontal Backfill		
Coefficient of at Rest Earth Pressure, K_o (Restrained Wall)	0.43	0.47
Coefficient of Active Earth Pressure, K_a (Unrestrained Wall)	0.27	0.31
2H:1V Sloped Backfill		
Coefficient of at Rest Earth Pressure, K_o (Restrained Wall)	0.62	0.68
Coefficient of Active Earth Pressure, K_a (Unrestrained Wall)	0.39	0.47

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.

It should be noted that the fill above the retaining wall should be sloped no steeper than 2H:1V.

9.4.2 Combined Static and Seismic Lateral Earth Pressure Parameters

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

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

For 2H:1V backslopes, the parameters are beyond the limitations for the Mononobe-Okabe method and the general limit equilibrium method has been used to calculate the seismic lateral earth pressures.

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 for both horizontal and 2H:1V backslopes are provided in Table 9-2 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 0.218g; as outlined in Section 8.0.

Table 9-2: Lateral Earth Pressure (Under Seismic Loads)

Parameter	OPSS Granular A & B Type II
Soil Unit Weight, kN/m^3 , γ	22.8
Angle of Internal Friction, ϕ	35°
Horizontal Backslope	
Coefficient of Active Earth Pressure, K_{AE} (Restrained Wall)	0.41
Coefficient of Active Earth Pressure, K_{AE} (Unrestrained Wall)	0.33
2H:1V Backslope	
Coefficient of Active Earth Pressure, K_{AE} (Restrained Wall)	0.74
Coefficient of Active Earth Pressure, K_{AE} (Unrestrained Wall)	0.50

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/m^3); use submerged unit weight for soils below the groundwater level
- K_{AE} = combined static and seismic earth pressure coefficient
- H = total height of the wall (m)

9.5 Assessment of Global Stability

The global stability for the inlet wingwalls 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 0.109g, equal to ½ of the site adjusted PGA value (0.218g) was used for seismic analysis.
- A backslope of 2H:1V

The global stability analysis results indicate a factor of safety of 1.5 and 1.2 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. In all cases examined, the critical slope circle was within the embankment fill above the wingwalls.

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 5-2 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 5-2 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. Accordingly, GU could be specified for concrete in below grade applications.

The pH, resistivity and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. The test results provided in Table 5-2 were compared with Table 3.2 of the MTO Gravity Pipe Design Guideline and generally indicate a very low to moderately corrosive environment. The test results provided in Table 5-2 may be used to aid in the selection of coatings and corrosion protection systems for buried steel objects.

10 CONSTRUCTION CONSIDERATIONS

10.1 Excavations

The top of culvert at the inlet was surveyed at the time of Thurber's investigation at an approximate elevation of 118.6 m. It is anticipated that temporary excavations in the order of 7 m below the existing top of culvert will be required to allow access for removal of the existing inlet wingwall foundations and construction of the replacement wingwalls.

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

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

Care should be taken not to undermine the existing culvert foundations during excavation for the wingwall foundations. The contractor should be prepared (and have equipment ready) to underpin the culvert foundation with unshrinkable backfill material or flowable grout. Suggested wording for an NSSP to alert the Contractor to this requirement is provided in Appendix F.

10.2 Dewatering

The depth of excavations required to construct the wingwalls 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 wingwalls in a dry and stable excavation.

Excavation for and construction of the wingwalls 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 No. FOUN0003 which amends OPSS 902. A preconstruction survey is not recommended, thus Designer Fill-In ** in this SP should be "NA".

In accordance with SP FOUN0003, the dewatering system is to be designed in accordance with OPSS.PROV 517 and SP517F01. It is recommended that the design Engineer and design-checking Engineer have a minimum of 5 years of experience in designing systems of similar nature and scope to the required work, thus Designer Fill-In ***** in SP517F01 should be "Yes". A preconstruction survey is not recommended, thus Designer Fill-In ***** in SP517F01 should be "NA".

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.

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. Sump pumps will be required to extract water from the excavations. During periods when the creek water level is low, consideration should be given to using a sandbag cofferdam provided the cobble and boulder layer and the sand to silty sand fill material is removed from beneath the footprint of the sandbag cofferdam. During periods of higher creek levels, a watertight braced enclosure system should be considered. The comments on installation and extraction of temporary protection systems are also relevant for cofferdams.

10.3 Temporary Protection Systems

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.

Soil or rock anchors or a deadman anchor system may be required to maintain stability.

Table 10-1: Static Lateral Earth Pressure Coefficient

Parameter	Silty Sand with Gravel Till & Sandy Clay with Gravel Till	Existing Granular Fill	Existing Clay Fill
Soil Unit Weight, kN/m^3 , γ	21.0	20.0	20.0
Angle of Internal Friction, ϕ	35°	32°	30°
Horizontal Backfill			
Coefficient of at Rest Earth Pressure, K_o	0.43	0.47	0.50
Coefficient of Active Earth Pressure, K_a	0.27	0.31	0.33
Coefficient of Passive Earth Pressure, K_p	3.69	3.25	3.00
2H:1V Sloped Backfill			
Coefficient of at Rest Earth Pressure, K_o	0.62	0.68	0.72
Coefficient of Active Earth Pressure, K_a	0.39	0.47	0.54

The design of temporary protection systems is the responsibility of the Contractor. All shoring should be designed by a licensed professional engineer experienced in such designs. The designer of the protection systems must ensure that the penetration depth is sufficient to provide base fixity and incorporate traffic loading and surcharge loading due to construction equipment and operations and shall consider the slope and height of temporary embankments above the top of the protection system.

Increased difficulty with the installation of temporary protection systems should be anticipated due to the presence of shallow bedrock, cobbles, boulders and buried obstructions. Suggested wording for NSSPs to alert the Contractor to these conditions and the requirement to use appropriate equipment and installation techniques are provided in Appendix F.

For preliminary assessment purposes, the use of soldier piles and lagging is considered feasible in this situation. The soldier piles should be installed in holes pre-drilled into the bedrock and grouted into place. 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.

10.4 Erosion 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).

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 to protect against surficial erosion in general accordance with OPSS.PROV 804.

11 CONSTRUCTION CONCERNS

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

- Construction will extend below the water level in the creek. An adequate and effective surface water management and dewatering plan must be implemented to replace the wingwalls in the dry.
- Installation of the temporary protection systems will encounter obstructions. Suggested wording for an NSSP alerting the Contractor to the presence of obstructions has been provided in Appendix F.
- Care should be taken not to undermine the existing culvert foundations during excavation for the wingwall foundations. Suggested wording for an NSSP to alert the Contractor to this requirement is provided in Appendix F.

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 No. 109S12, amendment to OPSS 902 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 Paul Carnaffan, 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.



Kenton C. Power, P.Eng.
Geotechnical Engineer



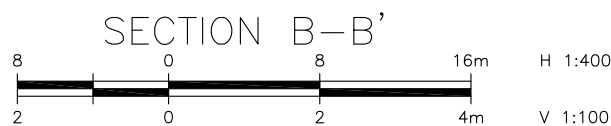
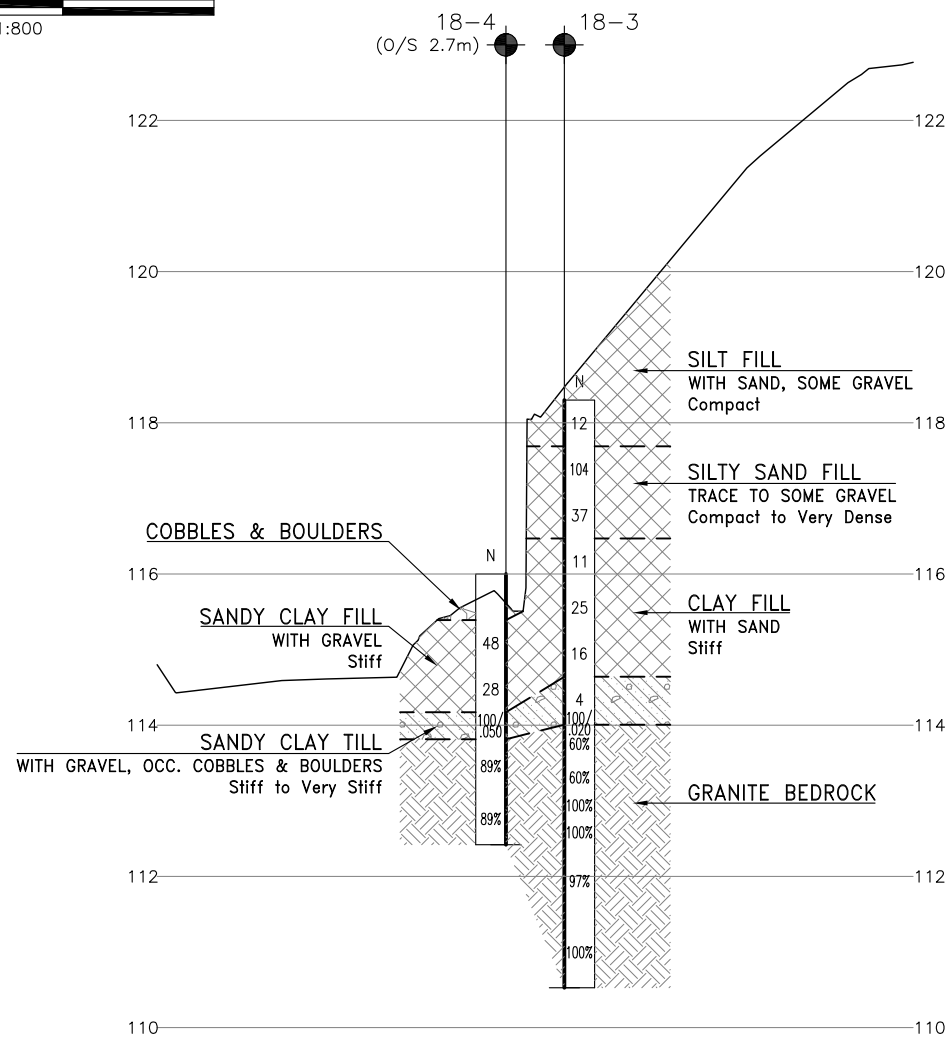
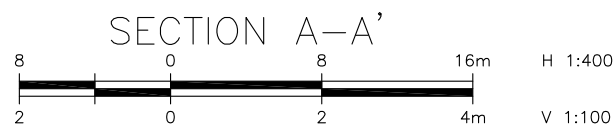
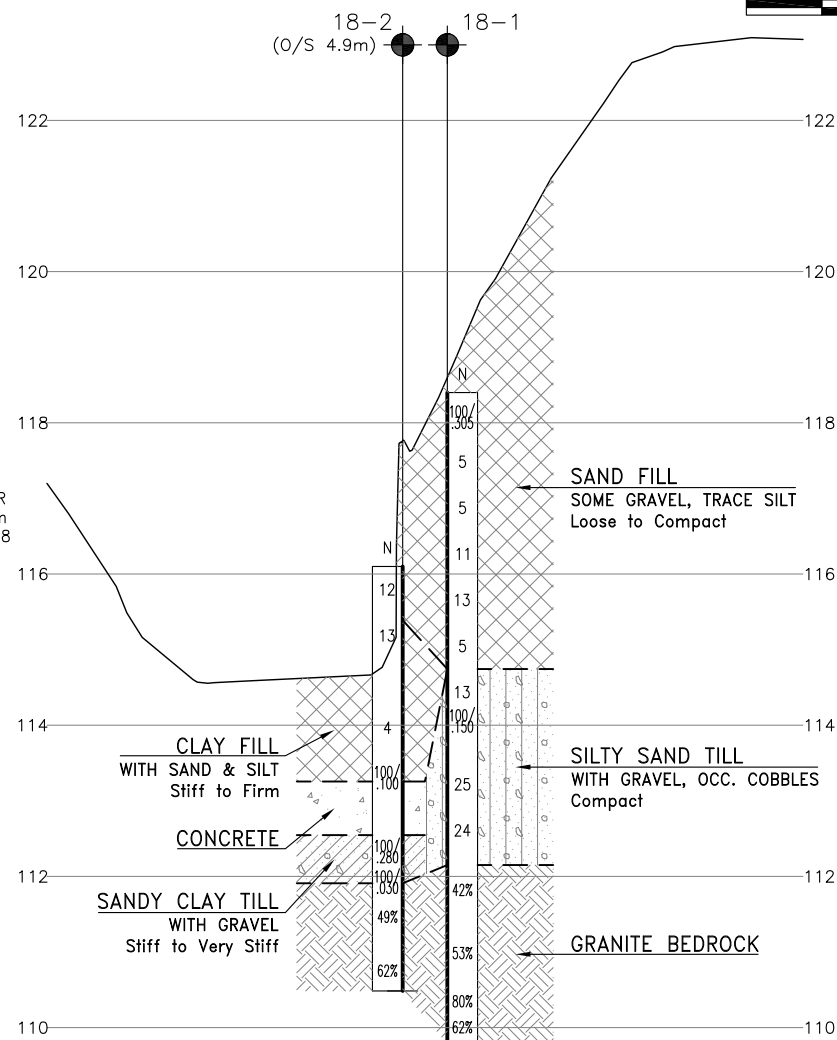
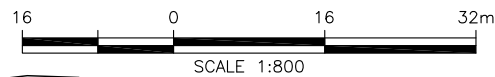
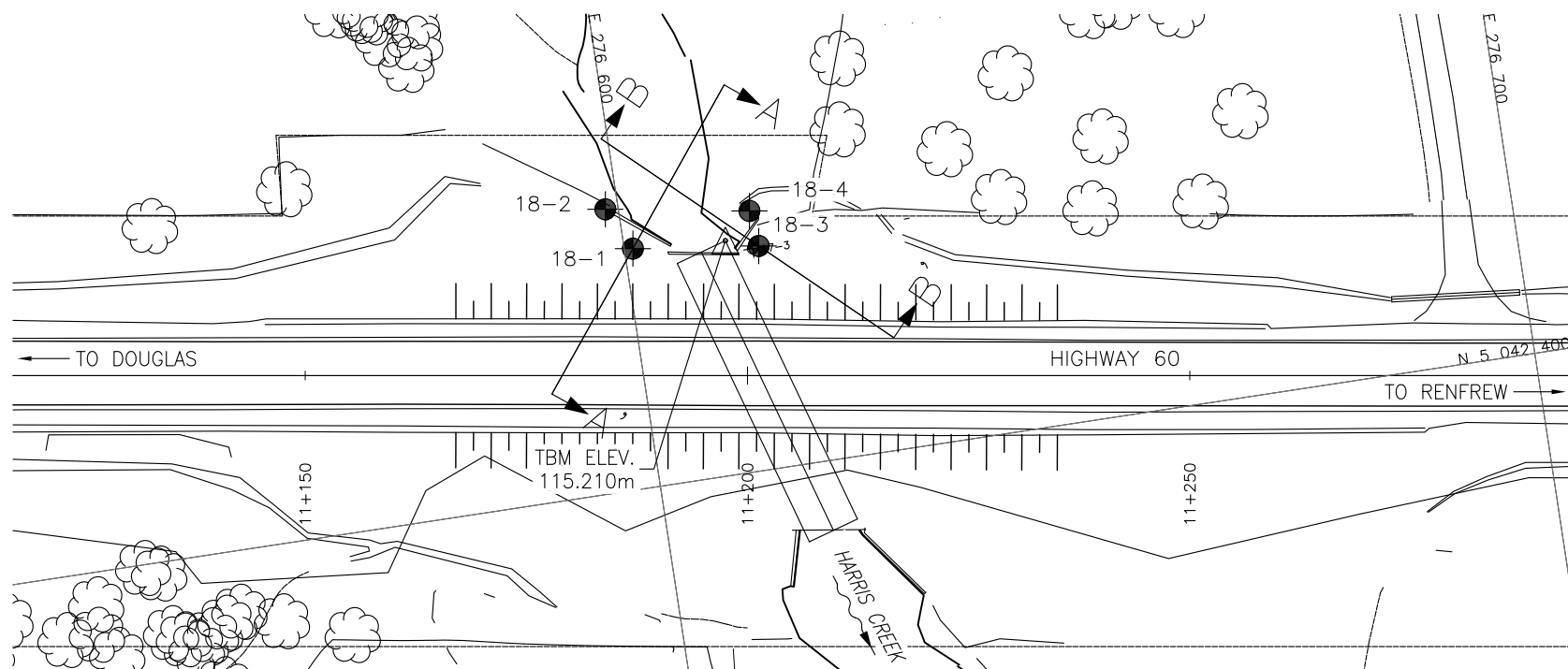
Fred J. Griffiths, Ph.D., P.Eng.
Senior Associate, Senior Geotechnical Engineer



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

APPENDIX A

**BOREHOLE LOCATIONS AND SOIL STRATA DRAWING
PRELIMINARY GENERAL ARRANGEMENT DRAWING
1963 CONSTRUCTION DRAWINGS FOR EXISTING CULVERT**



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



CONT No
GWP No 4076-13-00

HIGHWAY 60
DOUGLAS CREEK
CULVERT REHABILITATION
BOREHOLE LOCATIONS AND SOIL STRATA



KEYPLAN

LEGEND

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

NO	ELEVATION	NORTHING	EASTING
18-1	118.4	5 042 427.0	276 601.0
18-2	116.1	5 042 431.9	276 598.6
18-3	118.3	5 042 425.2	276 615.1
18-4	116.0	5 042 429.3	276 614.6

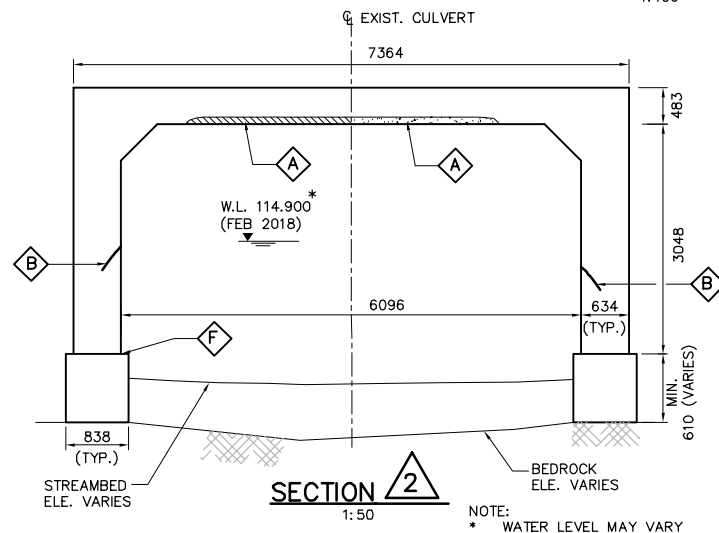
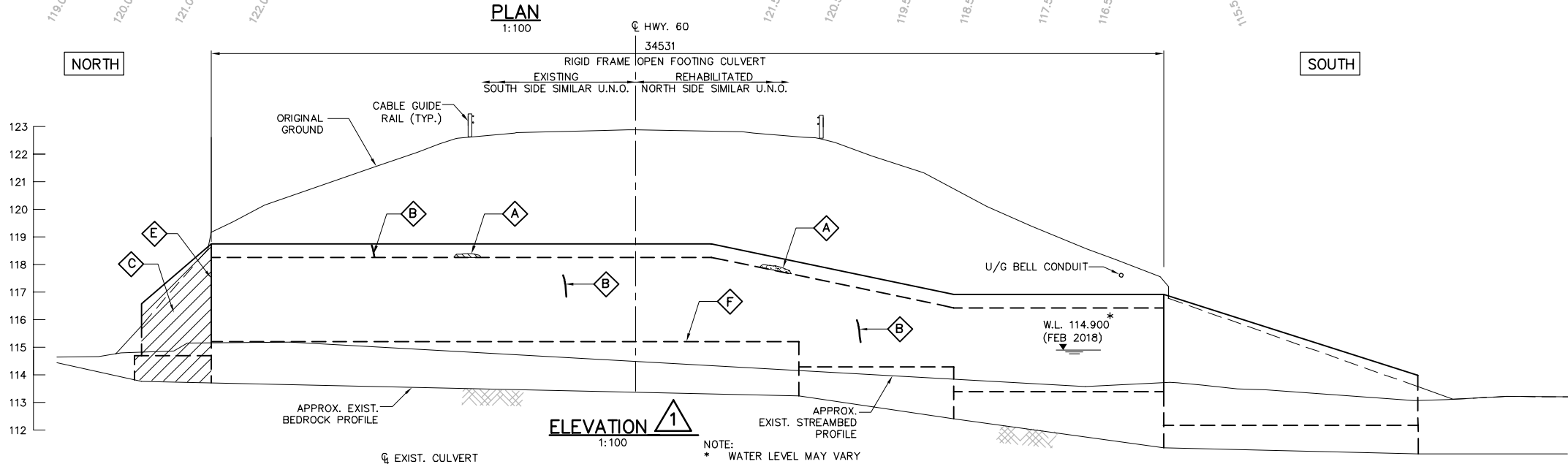
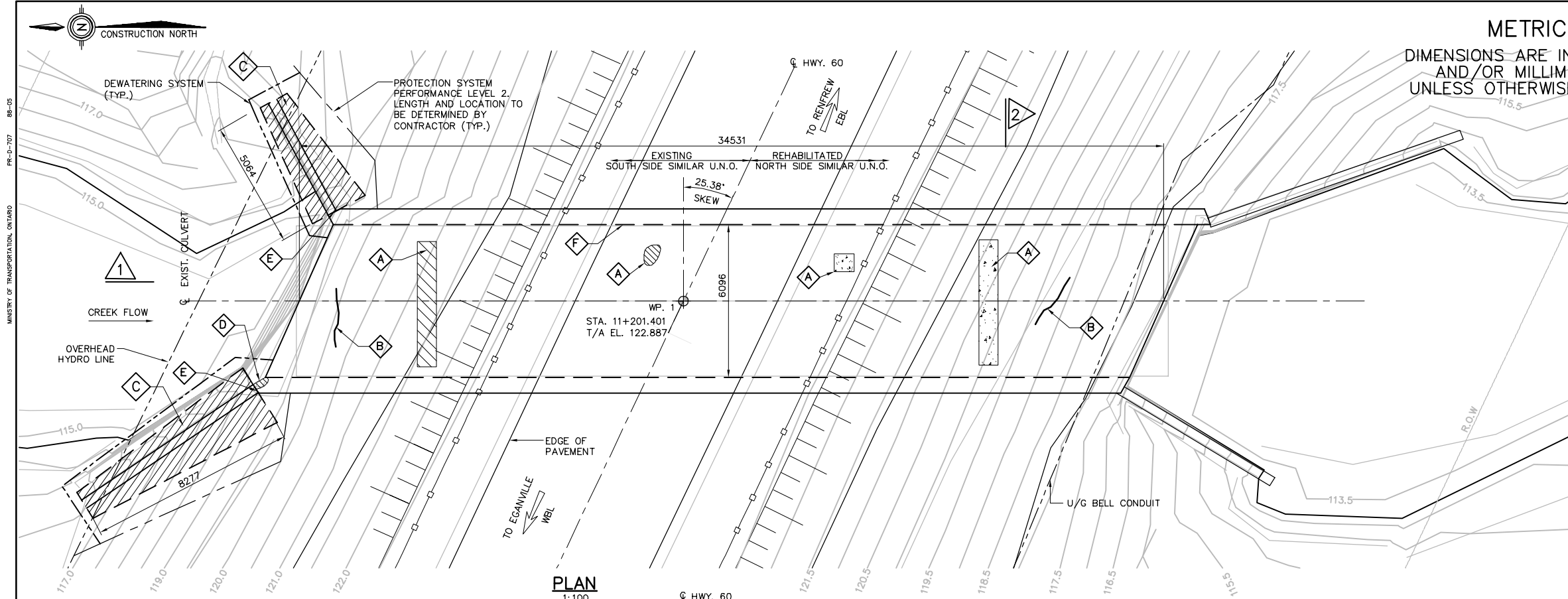
NOTES

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

GEOCRES No. 31F-203

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	KP	CHK -	CODE
DRAWN	MFA	CHK KP	SITE 29-155/C/STRUCT
DATE	JUL 2018	LOAD	DATE
DWG	1		

DRAWING NAME: F:\7269\Assignment #9 - Dochart, Douglas, Delia\Drafting\Douglas Creek Culvert\7269-Douglas Culvert-01-General Arrangement.dwg
MODIFIED: Jul 22, 2018-12:10pm
MAY 2007
CREATED:



COORDINATES OF WORK POINTS			
WP #	STATION	NORTH COORDINATE	EAST COORDINATE
1	11+201.401	5042410.687	276613.008

LIST OF ABBREVIATIONS:	
C.J.	CONSTRUCTION JOINT
CL	CENTRE LINE
CONC.	CONCRETE
EL.	ELEVATION
STA.	STATION
T/	TOP OF
T/A	TOP OF ASPHALT (FINISHED ELEV.)
TYP.	TYPICAL
WP	WORKING POINT
W.L.	WATER LEVEL

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

HWY. 60
CONT No
WP No 4115-13-01

HIGHWAY 60
DOUGLAS CREEK CULVERT REHABILITATION
GENERAL ARRANGEMENT

LEA
Joint Venture

MINTOSH PERRY
Mp

SHEET

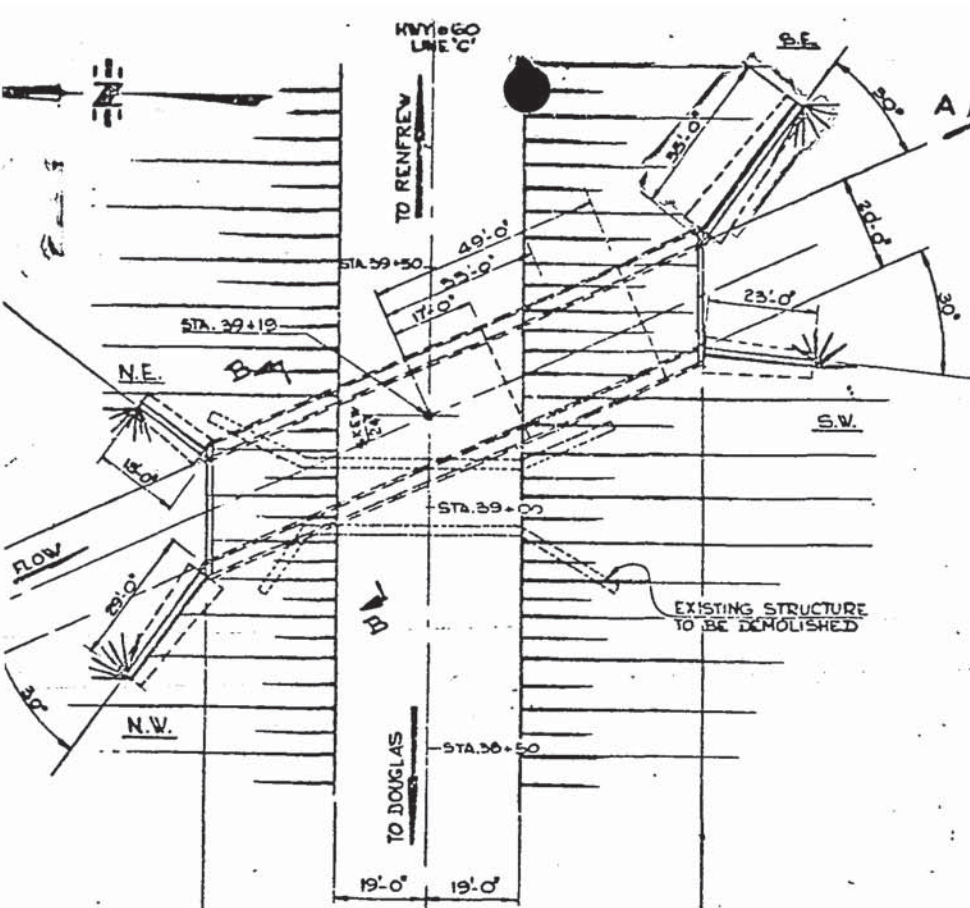
- GENERAL NOTES:**
- CLASS OF CONCRETE 30 MPa
 - CLEAR COVER TO REINFORCING STEEL
SOFFIT 40±10
REMAINDER (U.N.O.) 70±20
 - REINFORCING STEEL
REINFORCING STEEL SHALL BE DEFORMED BARS CONFORMING TO CSA STANDARD G30.18M GRADE 400.
- CONSTRUCTION NOTES:**
- CONTRACTOR SHALL VERIFY ALL DIMENSIONS OF THE EXISTING WORK AND ALL DETAILS ON SITE AND REPORT ANY DISCREPANCIES TO THE CONTRACT ADMINISTRATOR BEFORE PROCEEDING WITH THE WORK.
 - SAWCUTS IN CONC. WHERE DESIGNATED SHALL BE 25mm DEEP OR TO THE FIRST LEVEL OF REINFORCING STEEL, WHICHEVER IS LESS, UNLESS OTHERWISE NOTED.
 - DETERIORATED, SPALLED AND DELAMINATED AREAS SHOWN ON THE DRAWINGS ARE APPROXIMATE AND DO NOT REFLECT THE FULL EXTENT OR LOCATION OF THE PARTIAL DEPTH CONCRETE REMOVALS AND REPAIRS THAT SHALL BE DETERMINED ON SITE BY THE CONTRACT ADMINISTRATOR.
 - THE CONTRACTOR SHALL ENSURE THAT DUST & DEBRIS DOES NOT ENTER THE WATERCOURSE.
 - ALL SERVICES ARE TO ACCURATELY LOCATED PRIOR TO CONSTRUCTION AND ADEQUATE PROTECTION PROVIDED AT ALL THE TIME. ANY INTERFERENCE OF EXISTING SERVICES OR UTILITIES WITH PROPOSED STRUCTURE OR CONSTRUCTION OPERATIONS IS TO BE REPORTED TO THE CONTRACT ADMINISTRATOR PRIOR TO THE CONTINUATION OF CONSTRUCTION.
 - PROTECTION SYSTEM AROUND THE FOOTPRINT OF THE EXISTING RETAINING WALL SHALL BE INSTALLED BEFORE EXCAVATION TO PROTECT THE EXISTING CULVERT FOUNDATION.

- SCOPE OF WORK**
- REMOVE ALL LOOSE AND DELAMINATED CONCRETE FROM UNDERSIDE OF ROOF SLAB AND REPAIR WITH FORMED AND PUMP CONCRETE.
 - REPAIR CRACKS WITH POLYURETHANE CRACK INJECTION.
 - REMOVE AND REPLACE NORTHEAST AND NORTHWEST RETAINING WALLS.
 - RECONSTRUCT CULVERT INLET AT NORTHWEST RETAINING WALL.
 - REMOVE AND REPLACE JOINT SEAL.
 - REPAIR CONCRETE VOIDS WITH GROUT.

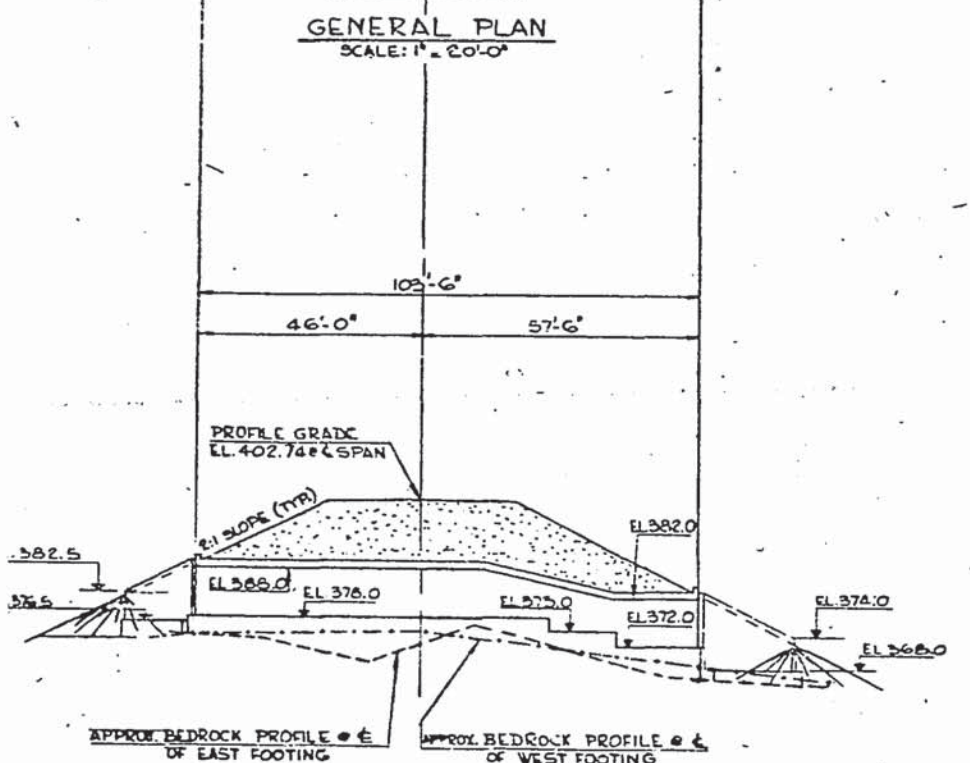
- LEGEND:**
- FULL DEPTH CONCRETE REMOVAL
 - PARTIAL DEPTH CONCRETE REMOVAL
 - NEW CONCRETE
 - MEDIUM CONCRETE CRACK

- LIST OF DRAWINGS:**
- GENERAL ARRANGEMENT
 - BOREHOLE SOIL STRATA
 - TYPICAL REPAIR DETAILS AND DEWATERING SYSTEM
 - RETAINING WALL DETAIL I
 - RETAINING WALL DETAIL II

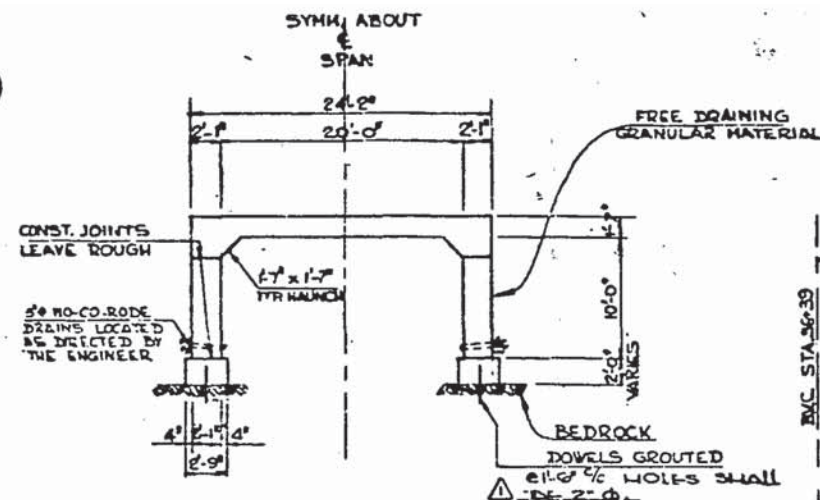
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DESIGN	FS	CHK	RTK	CODE CHBDC 14
DRAWN	RM	CHK	FS	SITE 29-155/C
		LOAD ONT CL-625	DATE	JUNE 2018
		STRUCT -	SCHEME -	DWG 1



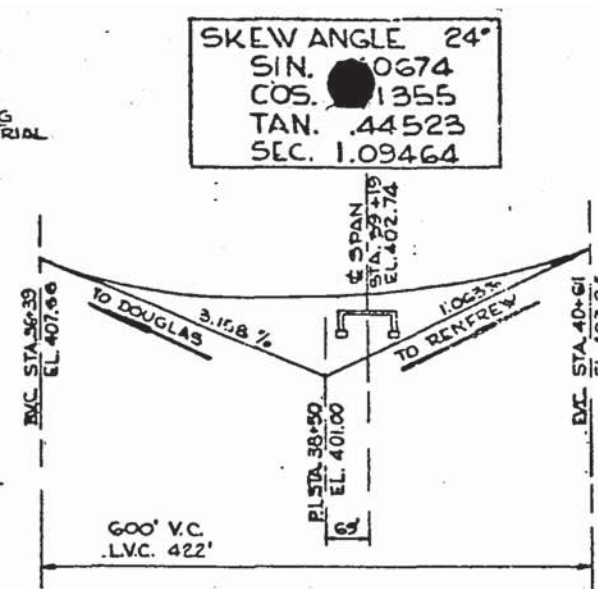
GENERAL PLAN
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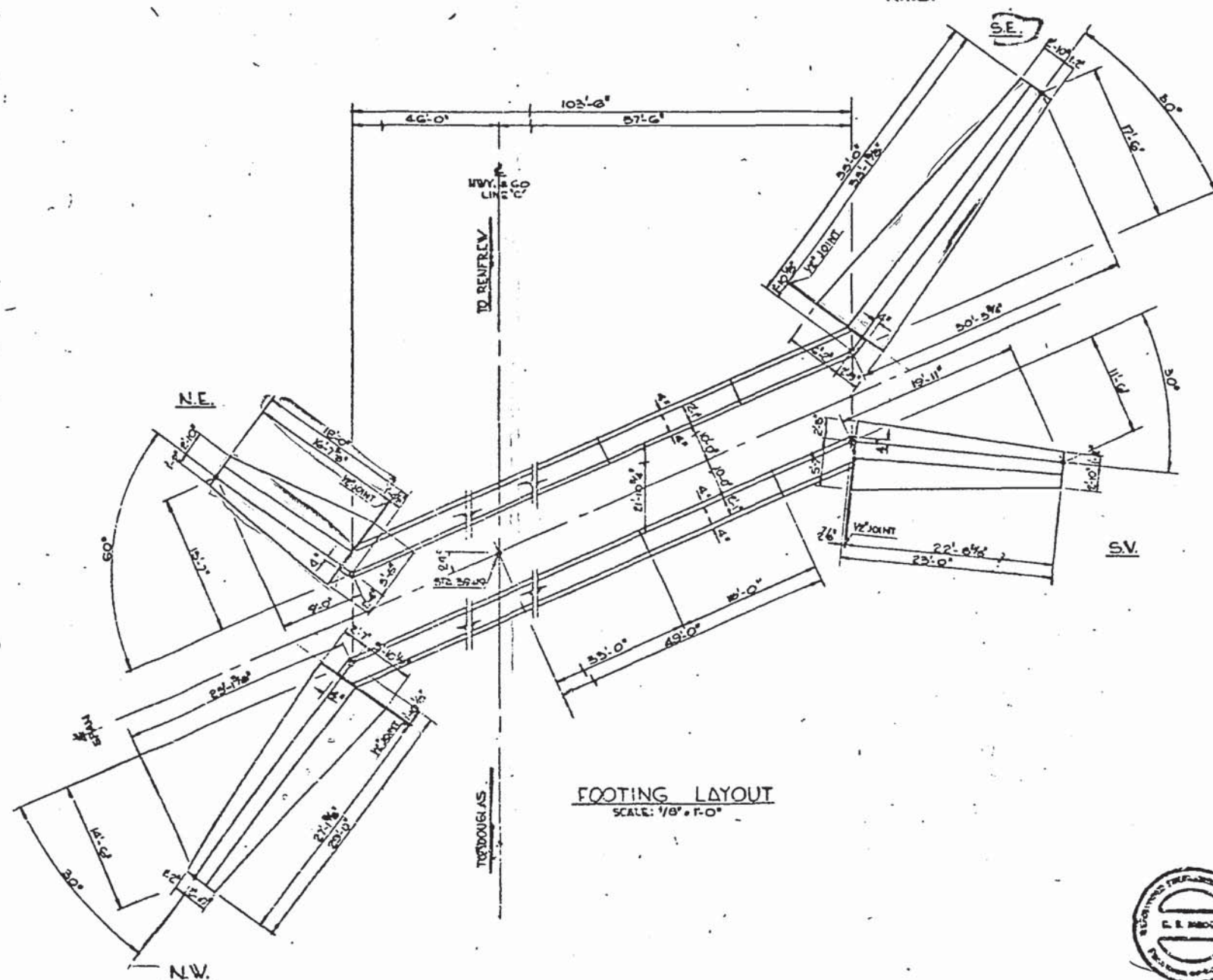
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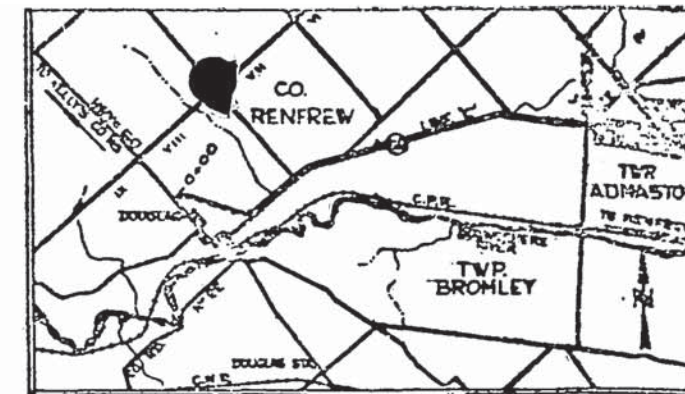
SECTION B-B
SCALE: 1/8" = 1'-0"



PROFILE GRADE
e & ROADWAY
N.T.S.



FOOTING LAYOUT
SCALE: 1/8" = 1'-0"



KEY PLAN
SCALE: 1/4" = 1 MILE

LIST OF DRAWINGS

D-5305-1	GENERAL PLAN & FOOTING LAYOUT
2	DETAILS OF CULVERT
3	RETAINING WALLS
4	REINFORCING STEEL SCHEDULE
5	DC

NOTES

- TO THE ENGINEER**
CONCRETE WORK ON THIS STRUCTURE MUST NOT BE COMMENCED UNTIL MONUMENTS TO FIX CONTROL POINTS HAVE BEEN ERECTED AND CHECKED BY THE ENGINEER.
- TO CONTRACTOR**
STRUCTURE TO BE BUILT IN ACCORDANCE WITH POWN NPS AND THE SPECIAL PROVISIONS EXTRA COPIES OF WHICH MAY BE OBTAINED FROM THE ENGINEER.
- CONCRETE MIX:**
MIN. STRENGTH OF CONCRETE @ 28 DAYS: 3000 P.S.I.
APPROVED ADMIXTURES SUPPLIED BY THE CONTRACTOR WILL BE ADDED TO ALL CONCRETE, AS SPECIFIED BY THE ENGINEER.
- CLEAR COVER ON REINFORCING STEEL:**
1 1/2" BOTTOM OF SLAB 3" ELSEWHERE
- CONSTRUCTION NOTES**
ALL EXPOSED EDGES TO BE CHAMFERED 1" X 1", EXCEPT AS NOTED.
ALL CONSTRUCTION JOINTS MUST BE APPROVED BY THE ENGINEER.
- BORING DATA**
NO SOIL INVESTIGATION REPORT FOR THIS STRUCTURE EXISTS. BEDROCK PROFILES SHOWN ON THESE PLANS ARE BASED ON FIELD INVESTIGATIONS THE ACCURACY OF WHICH IS NOT GUARANTEED BY THE D.H.O.

REVISIONS	DATE	BY	REVISIONS
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DEPARTMENT OF HIGHWAYS ONTARIO

DOUGLAS CREEK CULVERT 4 MILES WEST OF HIGHWAY 17

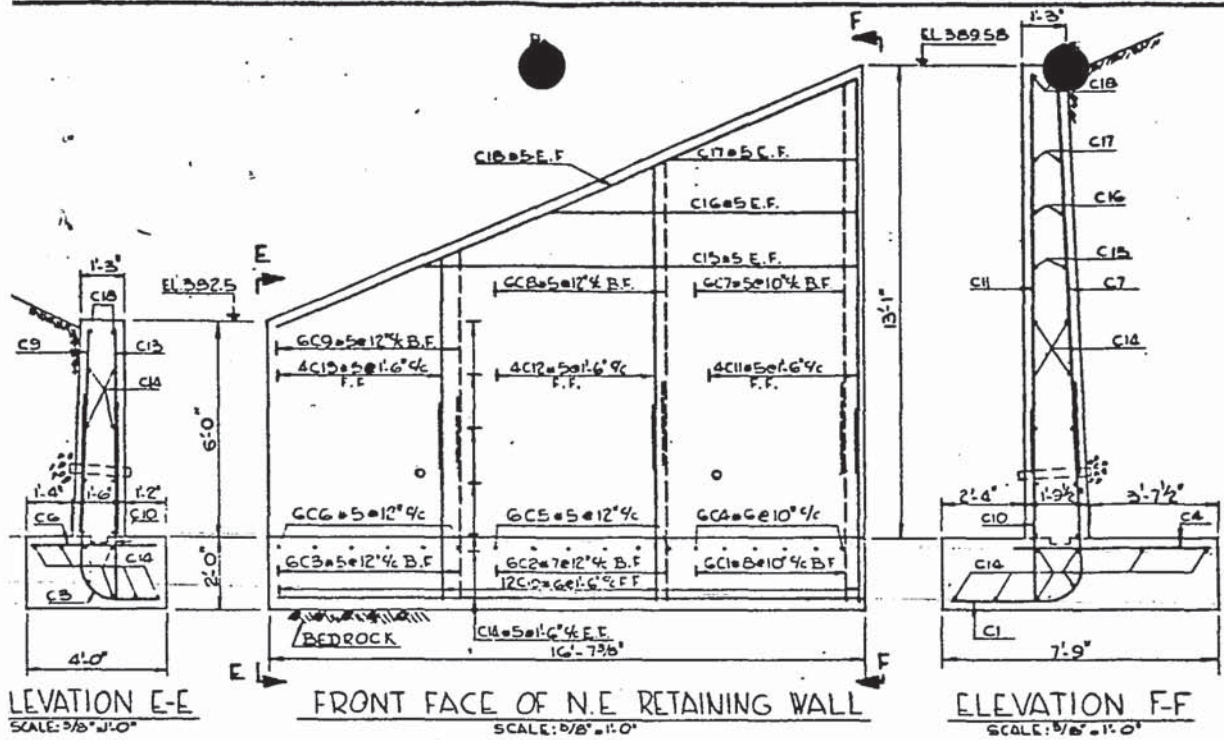
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CO. RENFREW STA. 39+19
TWP. ADMASTON LOT 29 & 36 CON. IV

GENERAL PLAN & FOOTING LAYOUT

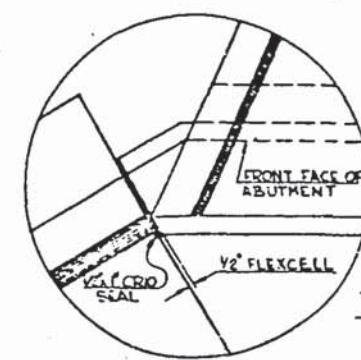
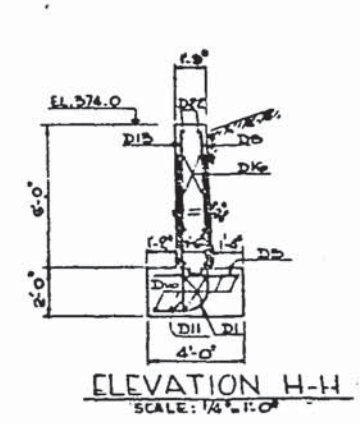
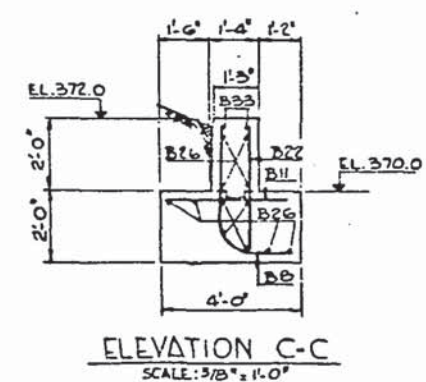
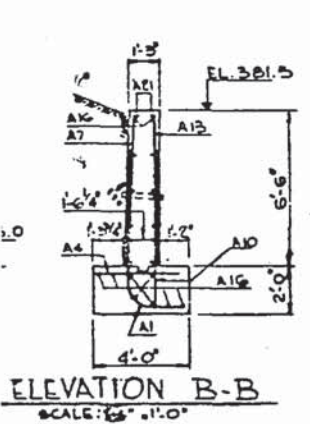
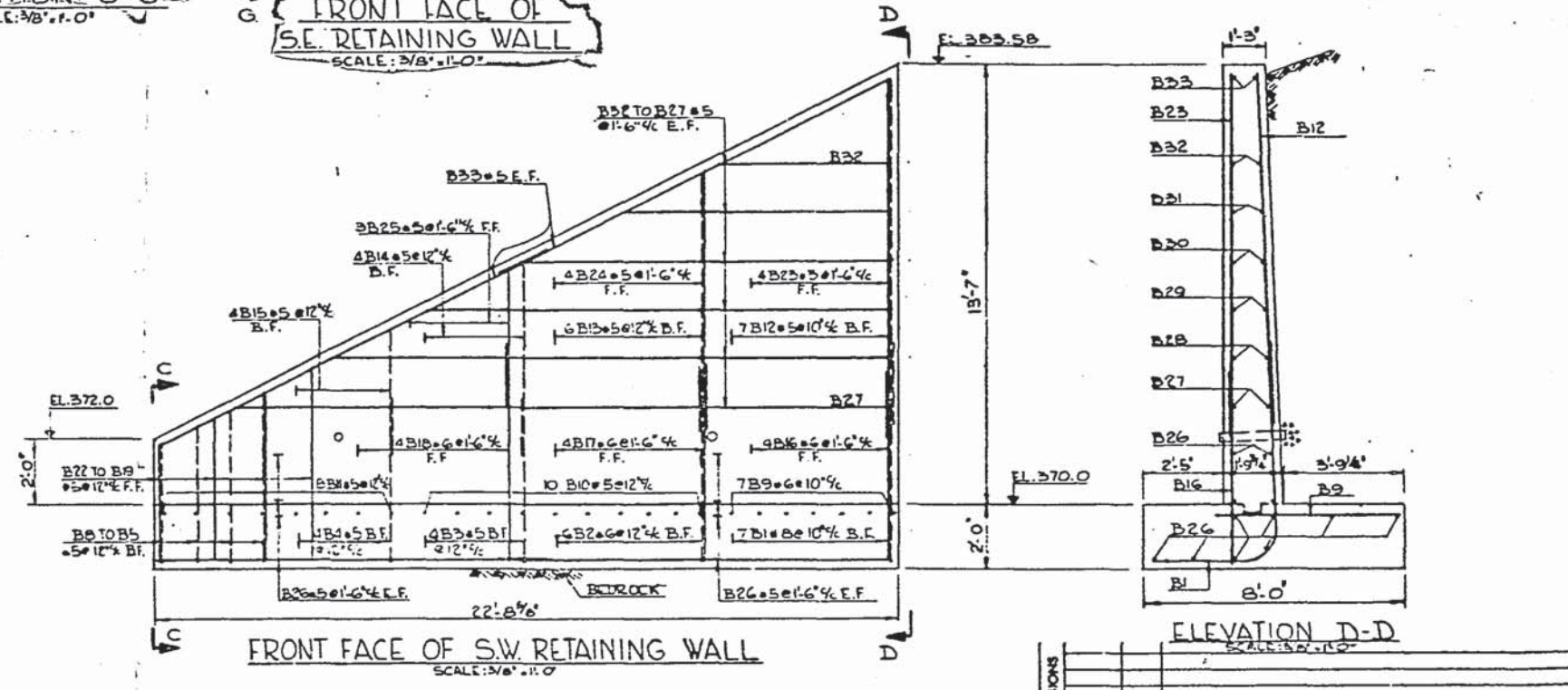
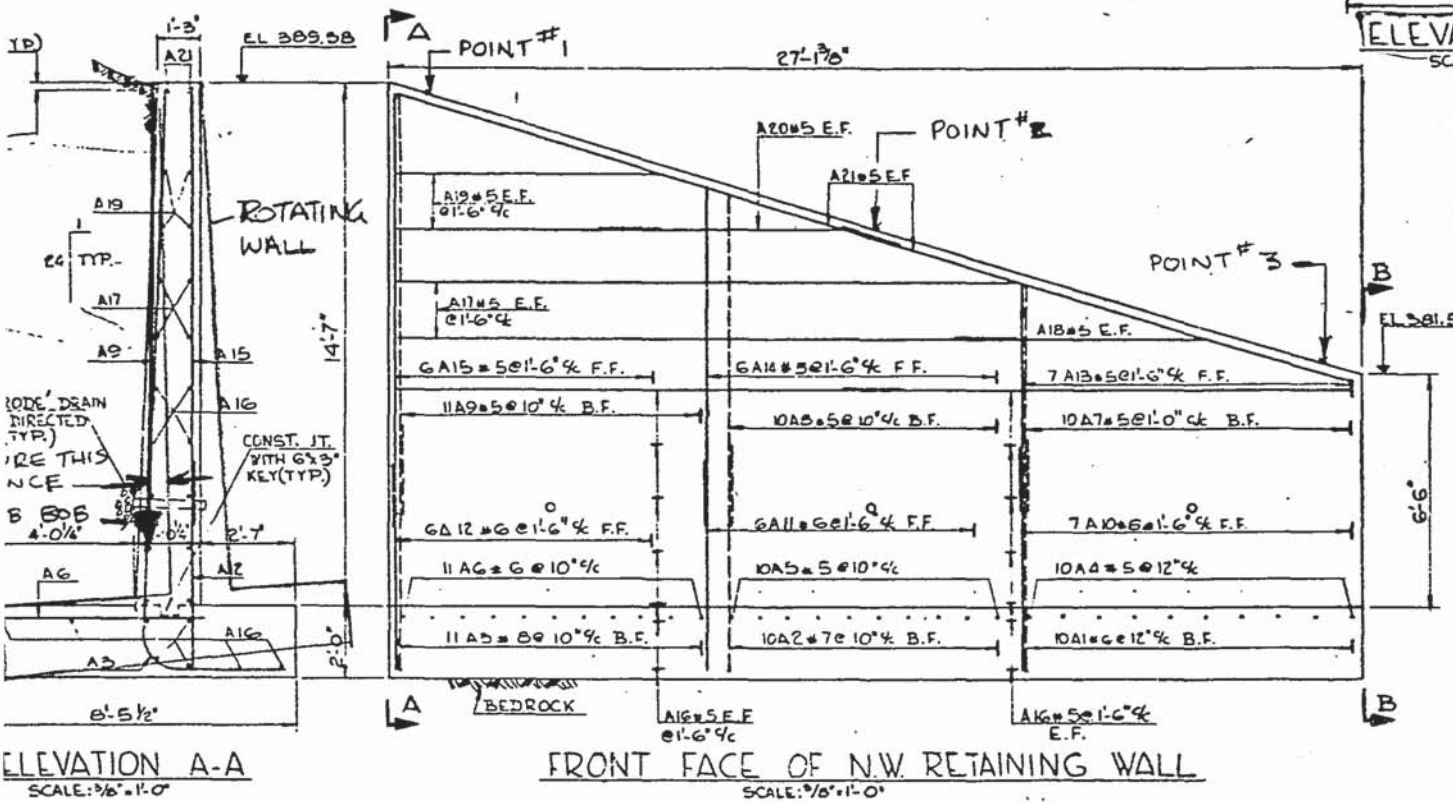
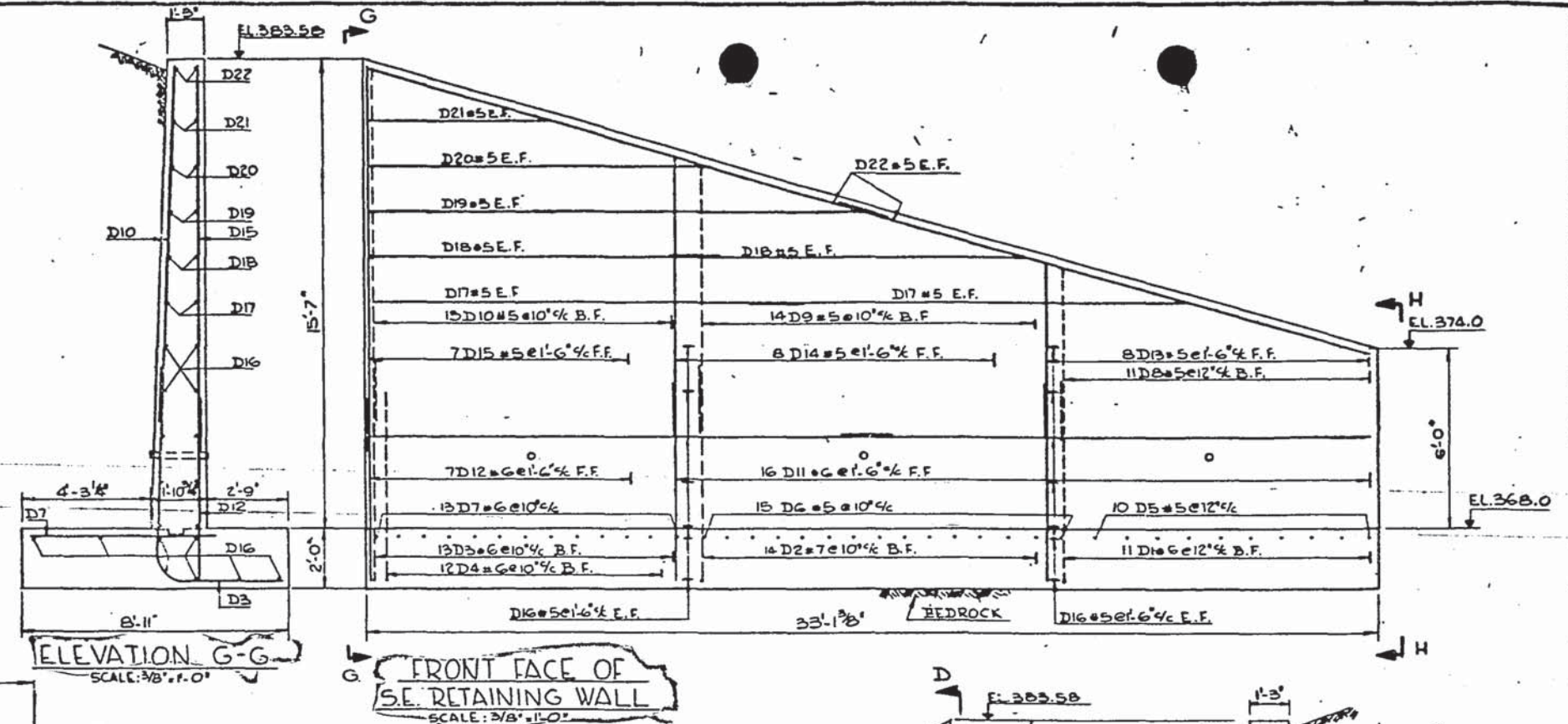
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DESIGN	G.D.	DATE	REVISIONS
DRAWING	G.D.	DATE	REVISIONS
DATE	1963	LOWING	220-516



D-5305-1



ELEVATION F-F
SCALE: 3/8" = 1'-0"



LEGEND
F.F. = FRONT FACE
B.F. = BACK FACE
E.F. = EACH FACE

REVISIONS	DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS ONTARIO EDGE DESIGN			
DOUGLAS CREEK CULVERT			
4 MILES WEST OF HWY 17			
RD NO 3 HIGHWAY No 60		DIST. No 10	
CD. RENEW		TWP. ADMASTON	
LOT 29 & 36		CDL IV	
RETAINING WALLS			
APPROVED	DATE	BY	DESCRIPTION
DESIGN	GP	CHK	CHK
DRAWING	GP	CHK	CHK
DATE	OCT. 1963	LOADING	H20-20
D-5305-3			

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).

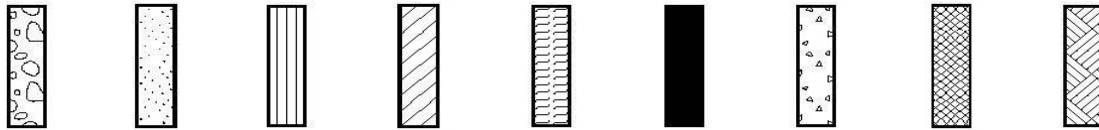
DYNAMIC CONE PENETRATION TEST (DCPT):

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

RECORD OF BOREHOLE No 18-1

1 OF 1

METRIC

GWP# 4076-13-00 LOCATION Lat: 45.521191°, Long: -76.860966° MTM Zone 9: N 5 042 427.0 E 276 601.0 ORIGINATED BY NW
 HWY 60 BOREHOLE TYPE Portable Drill NW Casing / NTW Coring COMPILED BY KE
 DATUM Geodetic DATE 2018.02.06 - 2018.02.09 CHECKED BY KP

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							WATER CONTENT (%)					
								20	40	60	80	100			W _P	W	W _L			
118.4																				
0.0	Sand with silt some gravel - frost to 0.75 m Loose to compact Brown FILL		1	SS	100/ 305mm		118													
			2	SS	5															
			3	SS	5		117													
			4	SS	11															
			5	SS	13		116													
			6	SS	5															
114.7							115													
3.7	SILTY SAND (SM) with gravel TILL - occasional cobbles Compact Grey		7	SS	13															
			8	SS	100/ 150mm		114													
			9	SS	25		113													
			10	SS	24															
112.1	- cobble at 6.0 m						112													
6.3	GRANITE BEDROCK Slightly weathered to fresh Close joint spacing Very strong strength Poor to good quality Grey to black		1	RUN																
			2	RUN			111													
			3	RUN																
			4	RUN			110													
109.8	- 100 mm silt filled fracture at 8.3 m																			
8.6	End of Borehole																			
	Note: A 50% (32 kg) drop hammer was used to advance the splitspoon sampler. The SPT N values presented above have been corrected to provide an estimate of the SPT N value that would have been obtained with a standard full weight hammer.																			

DOUBLE_LINE 20479 DOUGLAS CREEK CULVERT.GPJ 2012TEMPLATE(MTO).GDT 28/18

RECORD OF BOREHOLE No 18-2

1 OF 1

METRIC

GWP# 4076-13-00 LOCATION Lat: 45.521235°, Long: -76.860998° MTM Zone 9: N 5 042 431.9 E 276 598.6 ORIGINATED BY NW
 HWY 60 BOREHOLE TYPE Portable Drill NW Casing / NTW Coring COMPILED BY KE
 DATUM Geodetic DATE 2018.02.13 - 2018.02.13 CHECKED BY KP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
								20 40 60 80 100							
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE							
116.1								20 40 60 80 100							GR SA SI CL
0.0	Sand with silt some gravel Compact Brown FILL		1	SS	12		116								10 84 6 (SI+CL)
115.4															
0.7	Clay with sand and silt Stiff to firm Grey FILL		2	SS	13		115								
								1.0							
								3.8							
			3	SS	4		114								1 13 39 47
	- becoming clayey silt below 2.7 m - cored 150 mm cobble at 2.8 m														
113.3			4	SS	100/										
2.8	700 mm CONCRETE				100mm										
			1	RUN			113								
112.5															
3.6	SANDY CLAY (CL) with gravel TILL Stiff to very stiff grey		5	SS	100/ 280mm										18 27 30 25
111.9	- Cored 100 mm cobble at 4.0 m														
4.2	GRANITE BEDROCK Slightly weathered to fresh Close joint spacing Strong strength Poor to fair quality Grey to black				30mm										RUN #2 TCR=71% SCR=66% RQD=49%
			2	RUN											
			3	RUN			111								RUN #3 TCR=100% SCR=95% RQD=62% UCS=85MPa
110.5															
5.6	End of Borehole Note: A 50% (32 kg) drop hammer was used to advance the splitspoon sampler. The SPT N values presented above have been corrected to provide an estimate of the SPT N value that would have been obtained with a standard full weight hammer.														

DOUBLE_LINE 20479 DOUGLAS CREEK CULVERT.GPJ 2012TEMPLATE(MTO).GDT 2/8/18

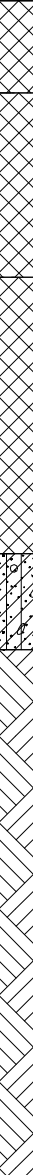
+³, ×³: Numbers refer to Sensitivity 20 15 10 5 0 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 18-3

1 OF 1

METRIC

GWP# 4076-13-00 LOCATION Lat: 45.521175°, Long: -76.860786° MTM Zone 9: N 5 042 425.2 E 276 615.1 ORIGINATED BY NW/KE
 HWY 60 BOREHOLE TYPE Portable Drill NW Casing / NTW Coring COMPILED BY KE
 DATUM Geodetic DATE 2018.02.15 - 2018.02.16 CHECKED BY KP

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					
118.3													
0.0	Sand with silt some gravel - grass and roots at surface Compact Brown FILL		1	SS	12								
117.7			2	SS	104								
0.6	Silty sand some gravel - frost to 1.2 m Dense to very dense brown FILL		3	SS	37								
116.5													
1.8	Clay with sand and silt some gravel Stiff Grey FILL		4	SS	11								
			5	SS	25								
			6	SS	16								
114.6													
3.7	SANDY CLAY (CL) with gravel TILL - occasional cobbles and boulders Stiff Grey		7	SS	4								
114.0			8	SS	100/								
4.3	GRANITE BEDROCK Slightly weathered to fresh Close joint spacing Very strong strength Fair to excellent quality Grey to black - 150 mm silt filled fracture at 4.6 m		1	RUN	20 mm								
			2	RUN									
			3	RUN									
			4	RUN									
			5	RUN									
			6	RUN									
110.5													
7.8	End of Borehole Note: A 50% (32 kg) drop hammer was used to advance the splitspoon sampler. The SPT N values presented above have been corrected to provide an estimate of the SPT N value that would have been obtained with a standard full weight hammer.												

DOUBLE_LINE 20479 DOUGLAS CREEK CULVERT.GPJ 2012TEMPLATE(MTO).GDT 2/8/18

RECORD OF BOREHOLE No 18-4

1 OF 1

METRIC

GWP# 4076-13-00 LOCATION Lat: 45.521212°, Long: -76.860792° MTM Zone 9: N 5 042 429.3 E 276 614.6 ORIGINATED BY NW
 HWY 60 BOREHOLE TYPE Portable Drill NW Casing / NTW Coring COMPILED BY KE
 DATUM Geodetic DATE 2018.02.18 - 2018.02.18 CHECKED BY KP

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						
116.0								20	40	60	80	100		
0.0	COBBLES / BOULDERS													
115.4														
0.6	Sandy clay with silt some gravel Stiff Grey FILL		1	SS	48		115							14 35 29 22
			2	SS	28									
114.2														
1.8	GRAVELLY CLAY (CL) with sand TILL		3	SS	100/ 50mm		114							27 21 29 23
113.8	Very stiff Grey													RUN #1 TCR=100% SCR=100% RQD=89% UCS=110MPa
2.2	GRANITE BEDROCK Slightly weathered to fresh Close joint spacing Very strong strength Good quality Grey to black		1	RUN										RUN #2 TCR=100% SCR=100% RQD=89%
			2	RUN			113							
112.4														
3.6	End of Borehole Note: A 50% (32 kg) drop hammer was used to advance the splitspoon sampler. The SPT N values presented above have been corrected to provide an estimate of the SPT N value that would have been obtained with a standard full weight hammer.													

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

Borehole 18-1
Box 1 (of 2)
Elevation 112.1 m to 110.7 m

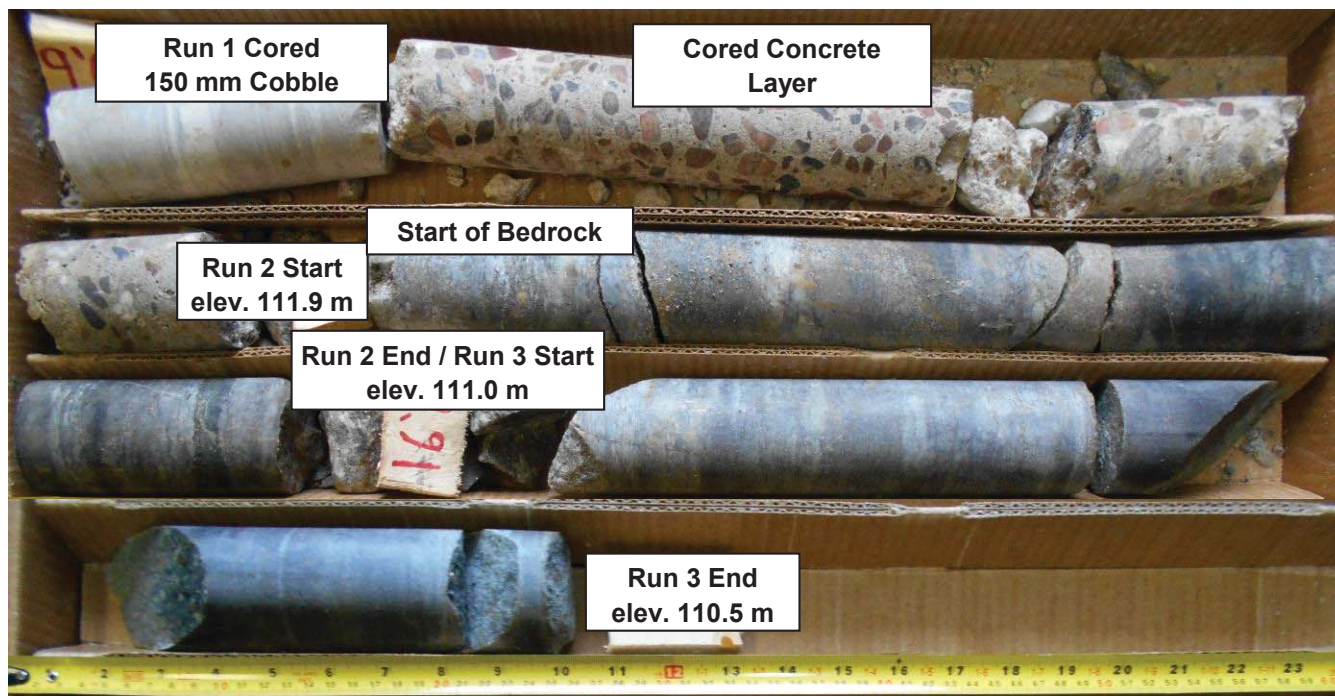


Borehole 18-1
Box 2 (of 2)
Elevation 110.7 m to 109.8 m

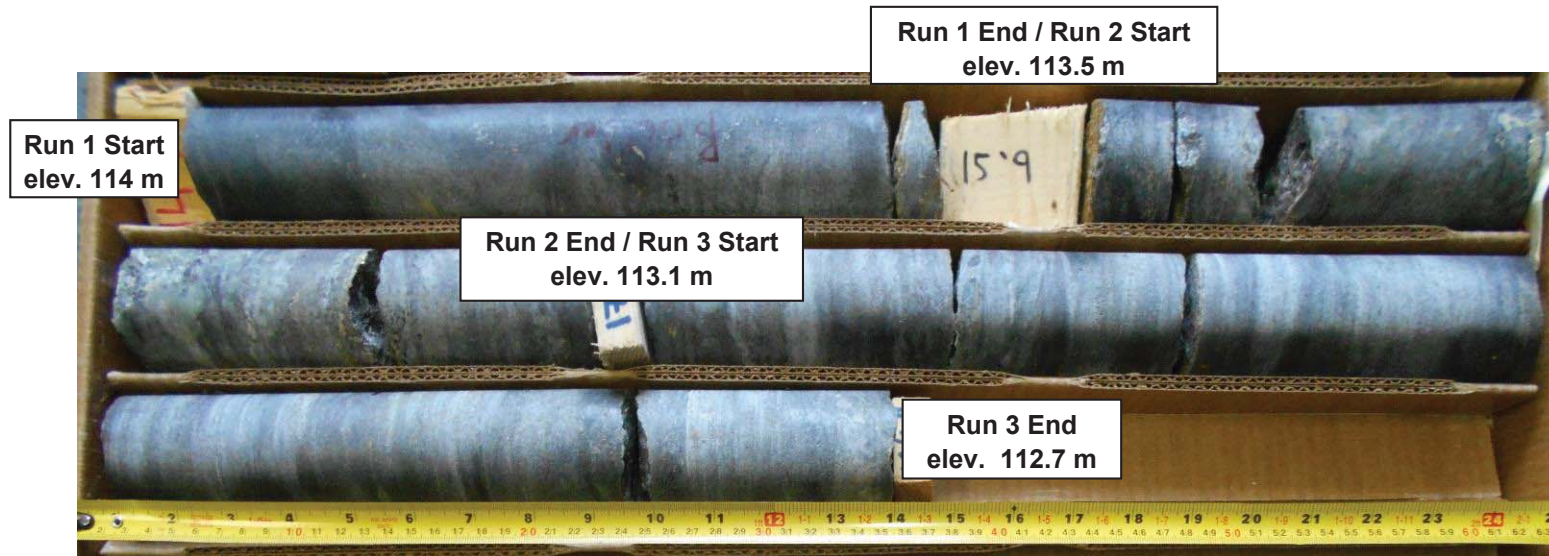


Borehole 18-2

Run 1 and 2 (of 2)
Elevation 111.9 m to 110.5 m



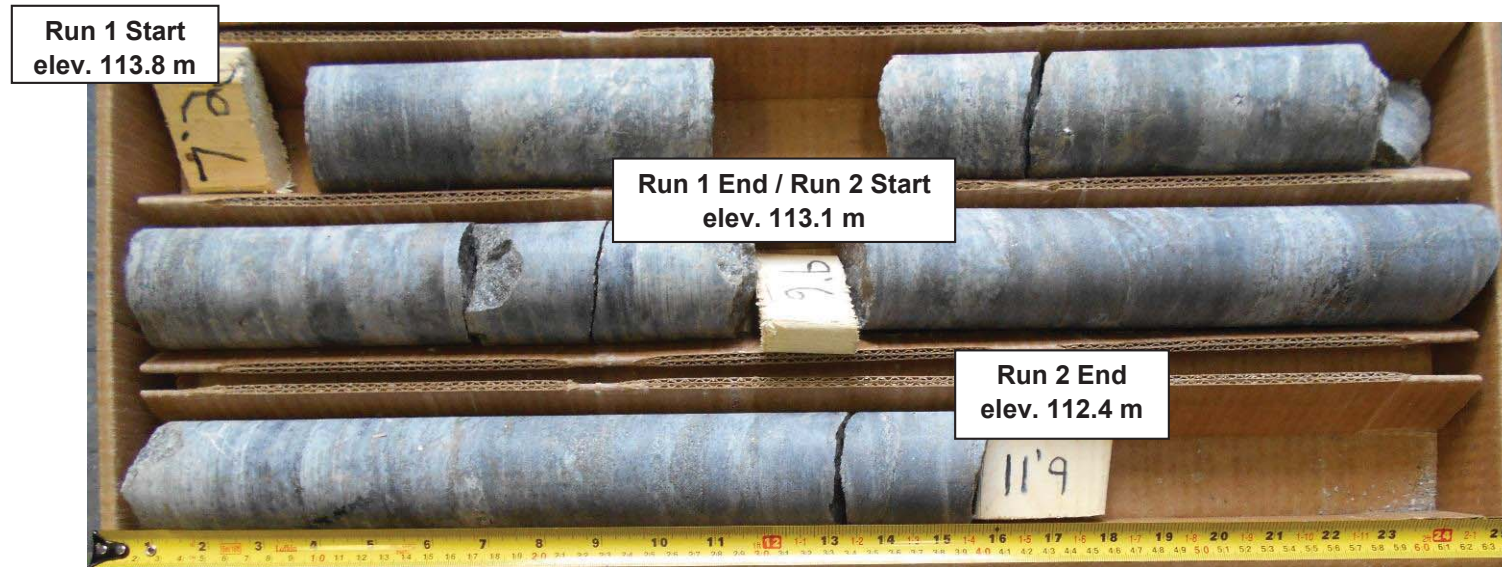
Borehole 18-3
Box 1 (of 2)
Elevation 114.0 m to 112.7 m



Borehole 18-3
Box 2 (of 2)
Elevation 112.7 m to 110.5 m



Borehole 18-4
Box 1 (of 1)
Elevation 113.8 m to 112.4 m

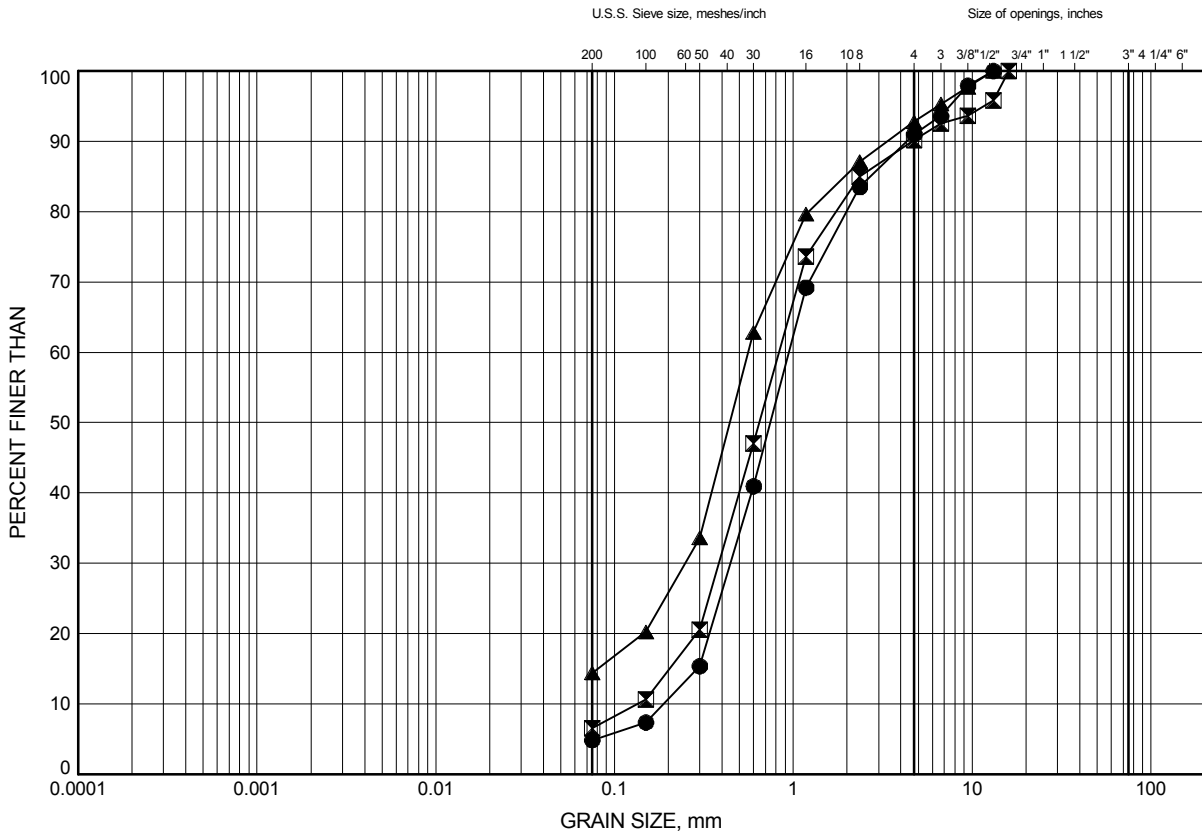


APPENDIX C
LABORATORY TEST RESULTS

Site 29-155/C Douglas Creek Culvert
GRAIN SIZE DISTRIBUTION

FIGURE 1

Fill: Sand with Silt some Gravel to Silty Sand some Gravel



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	18-1	0.91	117.49
⊠	18-2	0.30	115.79
▲	18-3	1.52	116.78

Date July 2018
 GWP# 4076-13-00

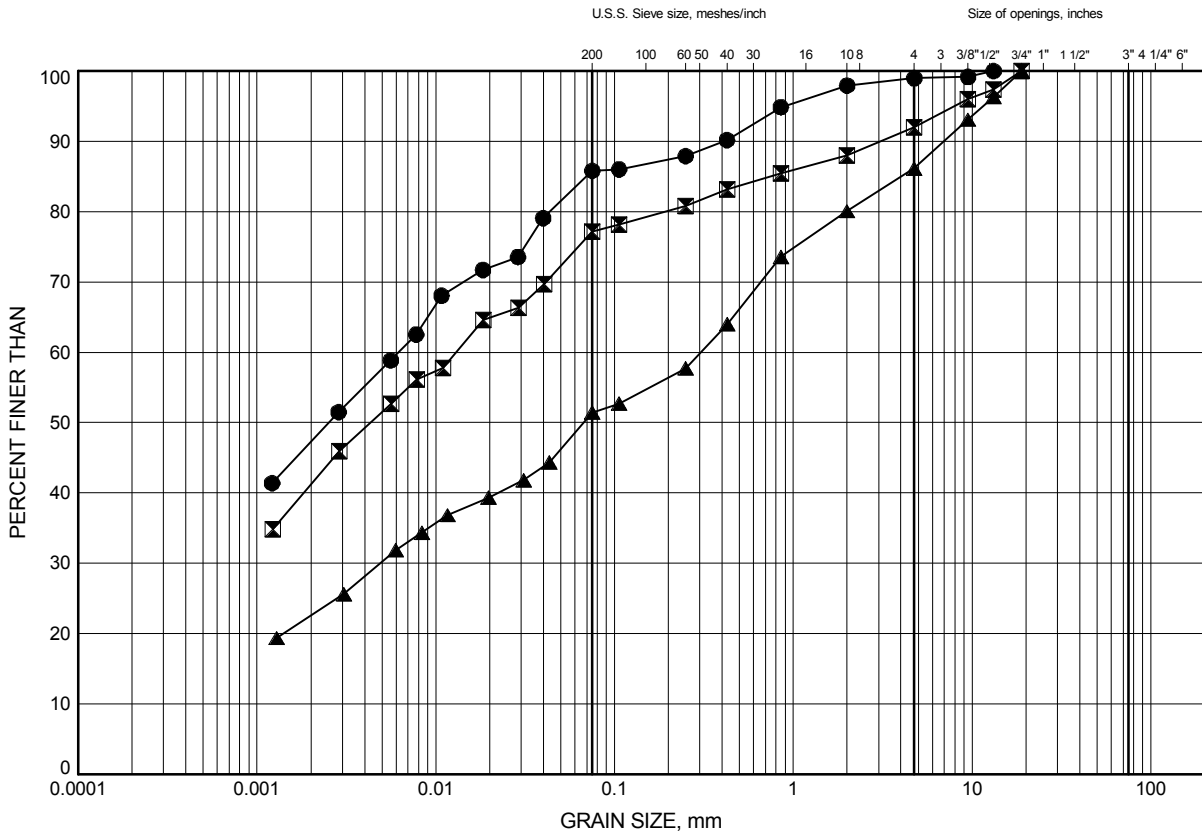


Prep'd KCP
 Chkd. FG

Site 29-155/C Douglas Creek Culvert
GRAIN SIZE DISTRIBUTION

FIGURE 2

Fill: Clay with Sand and Silt to Sandy Clay with Silt some Gravel



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	18-2	2.13	113.96
⊠	18-3	2.74	115.56
▲	18-4	0.91	115.09

Date July 2018
 GWP# 4076-13-00



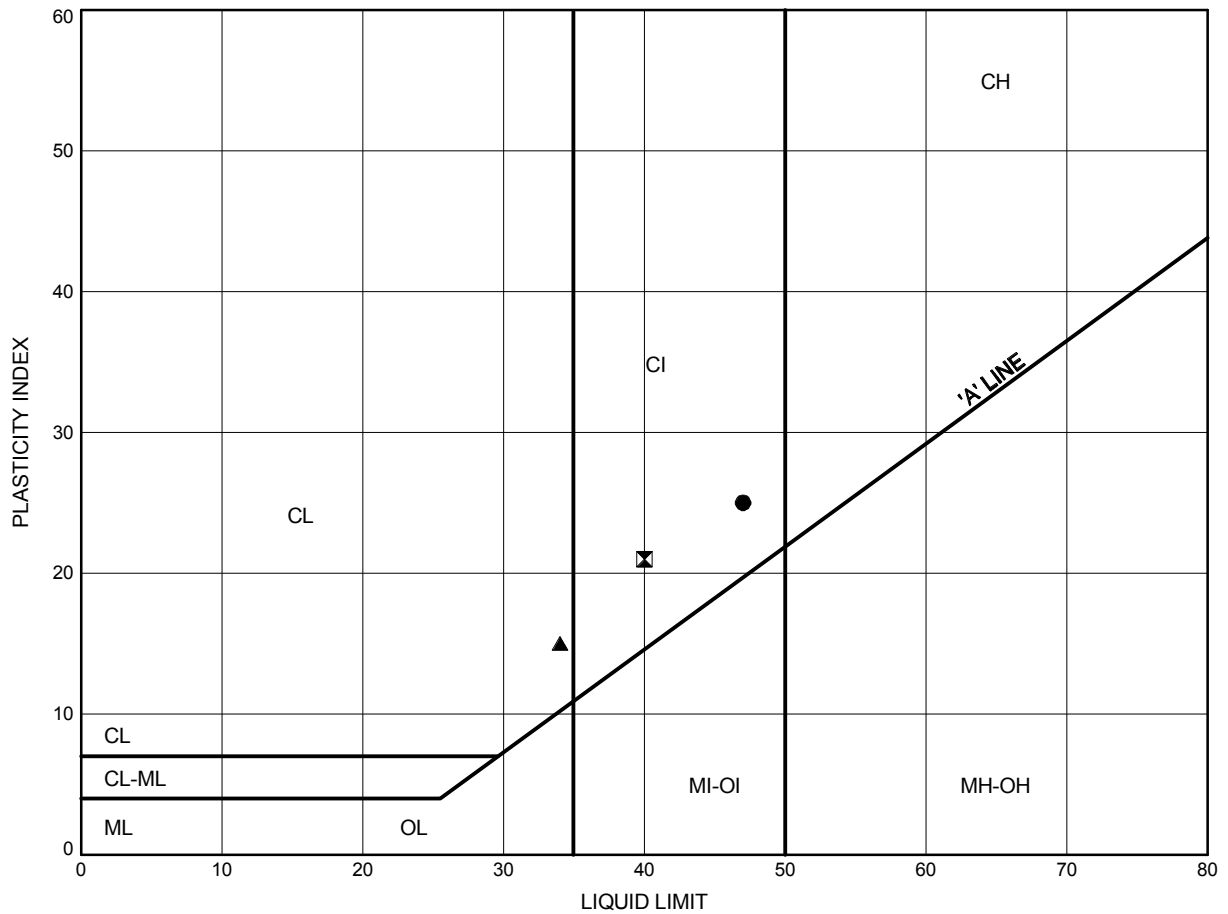
Prep'd KCP
 Chkd. FG

Site 29-155/C Douglas Creek Culvert

ATTERBERG LIMITS TEST RESULTS

FIGURE 3

Fill: Clay with Sand and Silt to Sandy Clay with Silt some Gravel



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	18-2	2.13	113.96
⊠	18-3	2.74	115.56
▲	18-4	0.91	115.09

Date July 2018

GWP# 4076-13-00



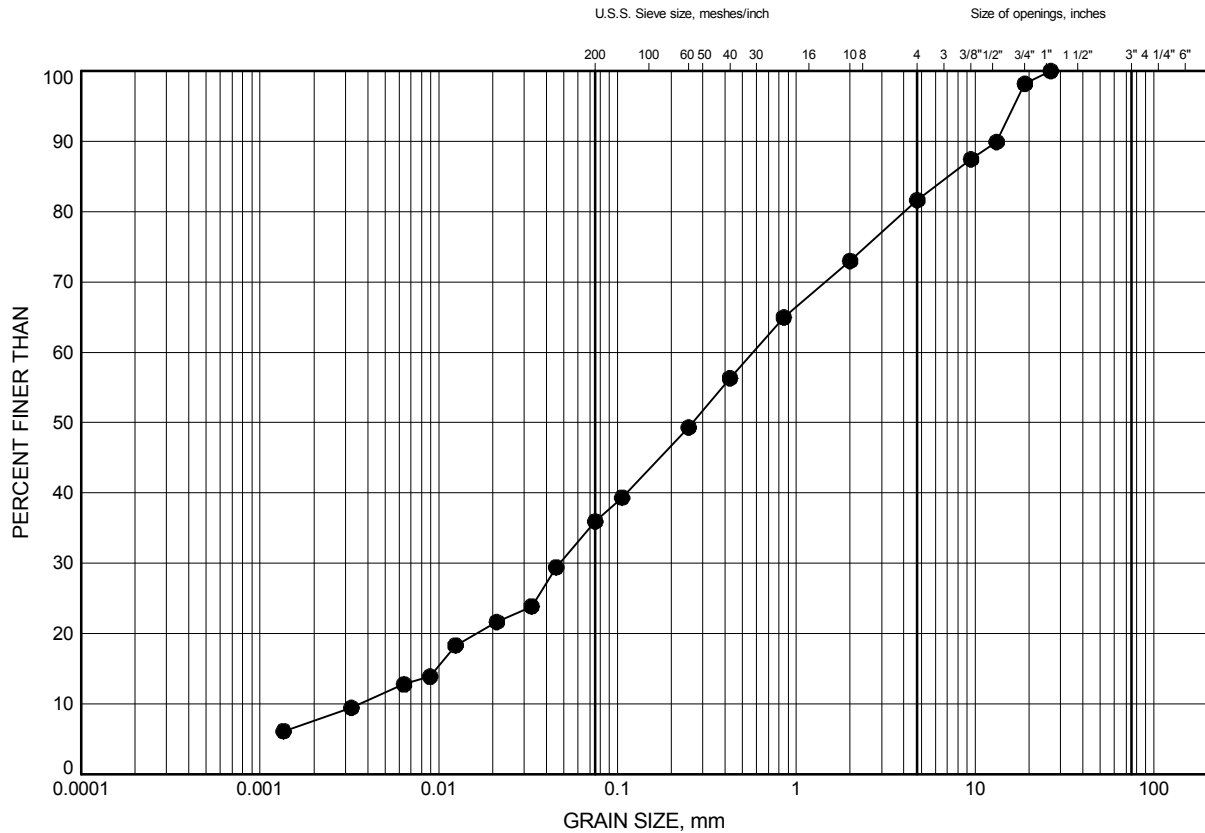
Prep'd KCP

Chkd. FG

Site 29-155/C Douglas Creek Culvert
GRAIN SIZE DISTRIBUTION

FIGURE 4

Silty Sand with Gravel Till



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	18-1	5.18	113.22

Date July 2018
 GWP# 4076-13-00

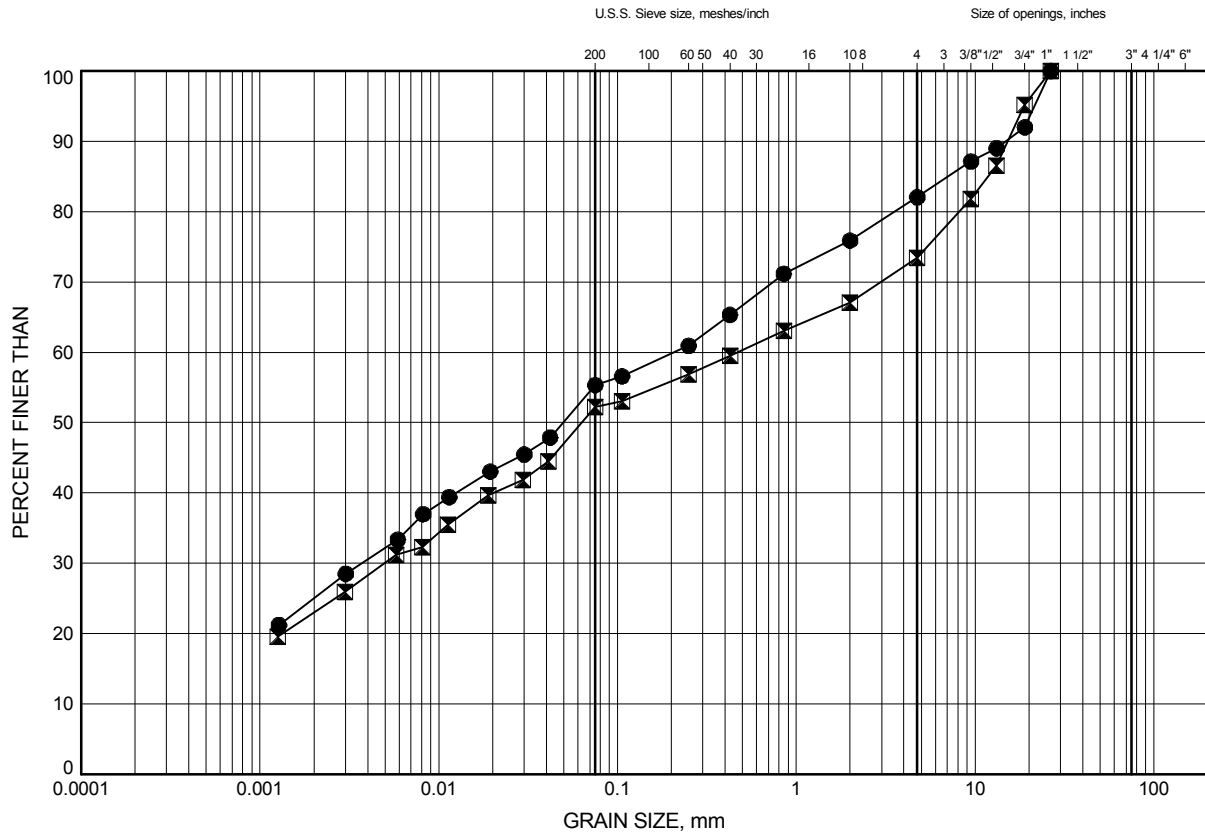


Prep'd KCP
 Chkd. FG

Site 29-155/C Douglas Creek Culvert
GRAIN SIZE DISTRIBUTION

FIGURE 5

Sandy Clay with Gravel to Gravely Clay with Sand Till



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	18-2	3.76	112.34
⊠	18-4	2.01	114.00

Date July 2018
 GWP# 4076-13-00

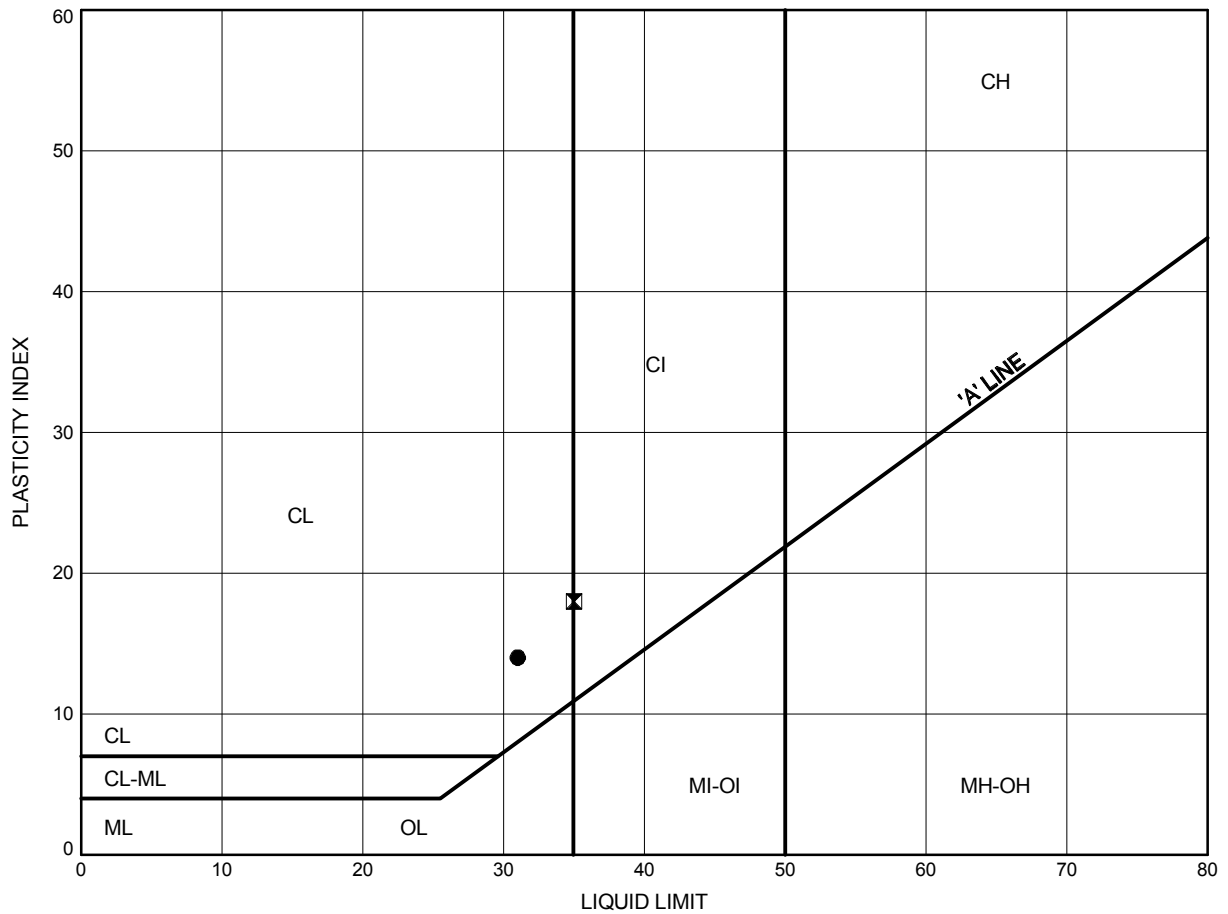


Prep'd KCP
 Chkd. FG

Site 29-155/C Douglas Creek Culvert
ATTERBERG LIMITS TEST RESULTS

FIGURE 6

Sandy Clay with Gravel to Gravely Clay with Sand Till



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	18-2	3.76	112.34
⊠	18-4	2.01	114.00

Date July 2018

GWP# 4076-13-00



Prep'd KCP

Chkd. FG



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

Stantec

March 28, 2018
File: 122410864

Attention: Thurber Engineering Ltd., File #20479

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

The table below summarizes five (5) rock core unconfined compressive strength results.

Location	Sample Depth	Compressive Strength (MPa)	Description of Break
18-1 (Run 1)	21'11"-22'4"	107.4	Two well formed cones on either end
18-2 (Run 2)	14'2"-14'9"	85.7	Well formed cone on bottom, diagonal cracks through top
18-3 (Run 1)	14'1"-15'1"	217.7	Two well formed cones on either end
18-3 (Run 3)	17'-17'5"	189.1	Well formed cone on bottom, cracks through rest of core
18-4 (Run 1)	7'2"-7'9"	110.2	Two well formed cones on either end

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: 20479 Douglas
Custody: 39591

Report Date: 27-Feb-2018
Order Date: 22-Feb-2018

Order #: 1808293

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

Paracel ID	Client ID
1808293-01	SS3 4'-6' BH18-1
1808293-02	SS4 6'-8' BH18-3

Approved By:



Mark Foto, M.Sc.
Lab Supervisor

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

Report Date: 27-Feb-2018

Order Date: 22-Feb-2018

Project Description: 20479 Douglas

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	23-Feb-18	24-Feb-18
pH, soil	EPA 150.1 - pH probe @ 25 °C, CaCl buffered ext.	23-Feb-18	23-Feb-18
Resistivity	EPA 120.1 - probe, water extraction	27-Feb-18	27-Feb-18
Solids, %	Gravimetric, calculation	23-Feb-18	23-Feb-18

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

Report Date: 27-Feb-2018

Order Date: 22-Feb-2018

Project Description: 20479 Douglas

Client ID:	SS3 4'-6' BH18-1	SS4 6'-8' BH18-3	-	-
Sample Date:	08-Feb-18	15-Feb-18	-	-
Sample ID:	1808293-01	1808293-02	-	-
MDL/Units	Soil	Soil	-	-

Physical Characteristics

% Solids	0.1 % by Wt.	95.6	79.5	-	-
----------	--------------	------	------	---	---

General Inorganics

pH	0.05 pH Units	8.46	8.34	-	-
Resistivity	0.10 Ohm.m	105	31.7	-	-

Anions

Chloride	5 ug/g dry	19	61	-	-
Sulphate	5 ug/g dry	<5	14	-	-

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

Report Date: 27-Feb-2018

Order Date: 22-Feb-2018

Project Description: 20479 Douglas

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: 27-Feb-2018

Order Date: 22-Feb-2018

Project Description: 20479 Douglas

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	8.3	5	ug/g dry	8.6			3.7	20	
Sulphate	6.88	5	ug/g dry	7.50			8.5	20	
General Inorganics									
pH	8.05	0.05	pH Units	8.07			0.2	10	
Resistivity	11.5	0.10	Ohm.m	11.5			0.5	20	
Physical Characteristics									
% Solids	86.1	0.1	% by Wt.	88.3			2.6	25	

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

Report Date: 27-Feb-2018

Order Date: 22-Feb-2018

Project Description: 20479 Douglas

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	100	5	ug/g	8.6	91.5	78-113			
Sulphate	112	5	ug/g	7.50	104	78-111			

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

Report Date: 27-Feb-2018
Order Date: 22-Feb-2018
Project Description: 20479 Douglas

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.

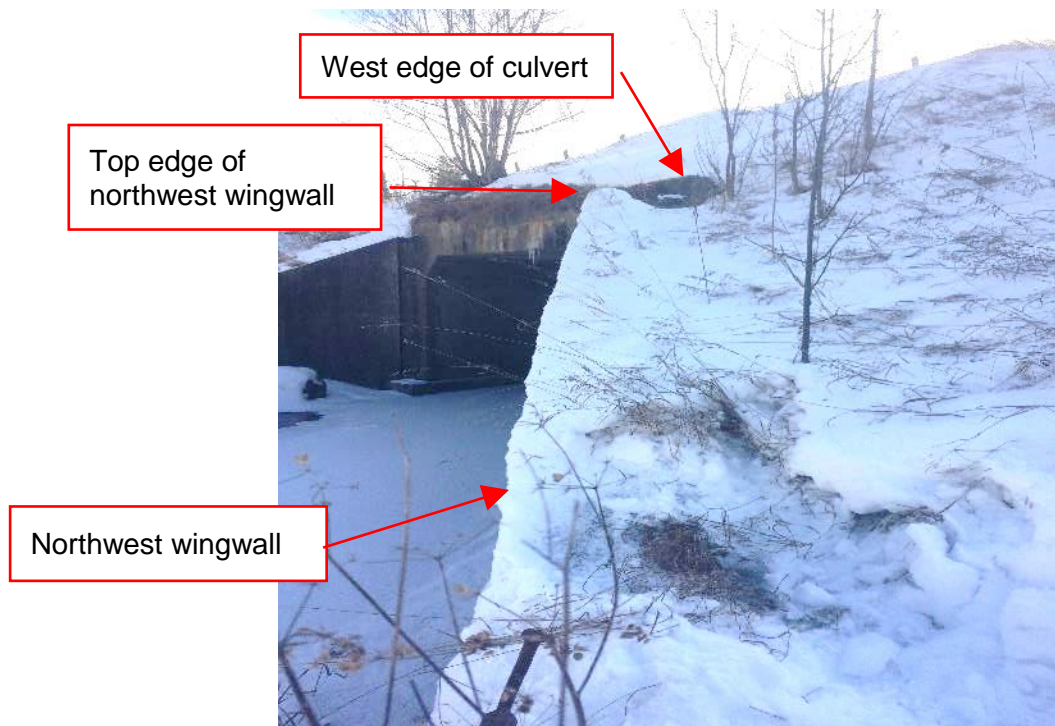
APPENDIX D
SITE PHOTOGRAPHS



**Photograph 1: Looking east along the north embankment towards the culvert inlet
(2017/12/20)**



Photograph 2: Looking downstream towards the culvert inlet (2017/12/20)



Photograph 3: Looking southeast along existing the northwest wingwall (2017/12/20)



Photograph 4: Rotation of northwest wingwall (2018/02/16)



Photograph 5: Rotation of northeast wingwall (2017/01/17)



Photograph 6: Looking west along embankment behind northwest wingwall (2017/12/20)



Photograph 7: Looking east along embankment behind northeast wingwall (2017/12/20)



Photograph 8: Looking upstream north of the inlet (2018/01/15)

APPENDIX E

2015 NBC SEISMIC HAZARD CALCULATION SLOPE STABILITY ANALYSIS RESULTS

2015 National Building Code Seismic Hazard Calculation

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

April 26, 2018

Site: 45.5212 N, 76.8608 W User File Reference: Douglas 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)
0.342	0.406	0.340	0.259	0.186	0.095	0.046	0.012	0.0047	0.218	0.155

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)	0.030	0.100	0.175
Sa(0.1)	0.043	0.132	0.219
Sa(0.2)	0.042	0.119	0.191
Sa(0.3)	0.034	0.095	0.149
Sa(0.5)	0.025	0.070	0.109
Sa(1.0)	0.013	0.037	0.057
Sa(2.0)	0.0052	0.018	0.028
Sa(5.0)	0.0011	0.0041	0.0068
Sa(10.0)	0.0006	0.0017	0.0028
PGA	0.024	0.073	0.120
PGV	0.017	0.054	0.087

References

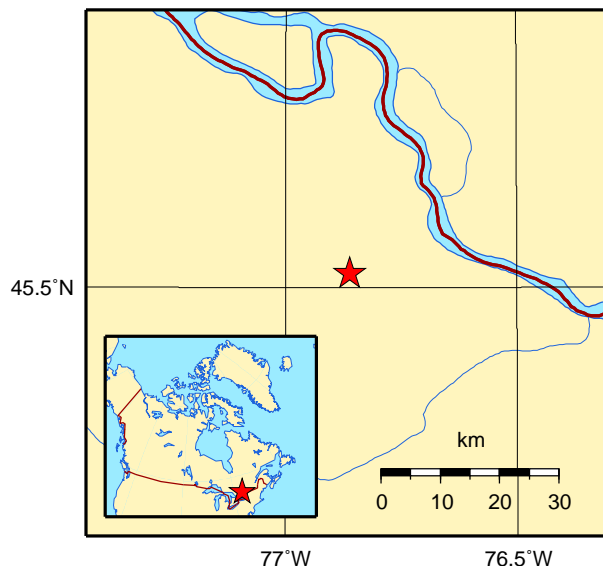
National Building Code of Canada 2015 NRCC no. 56190;
Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada

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: Douglas Creek Culvert Rehabilitation
Comments:
Name: 1.0 East Wall - Static

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: (-1.025, 129.85) w/ Radius: 11.01299 m
FoS Contours: 1.5 to 2.5, ++0.1

Embankment Fill	20 kN/m³	0 kPa	32 °
Till	21 kN/m³	0 kPa	35 °
Bedrock			
Concrete	24 kN/m³		

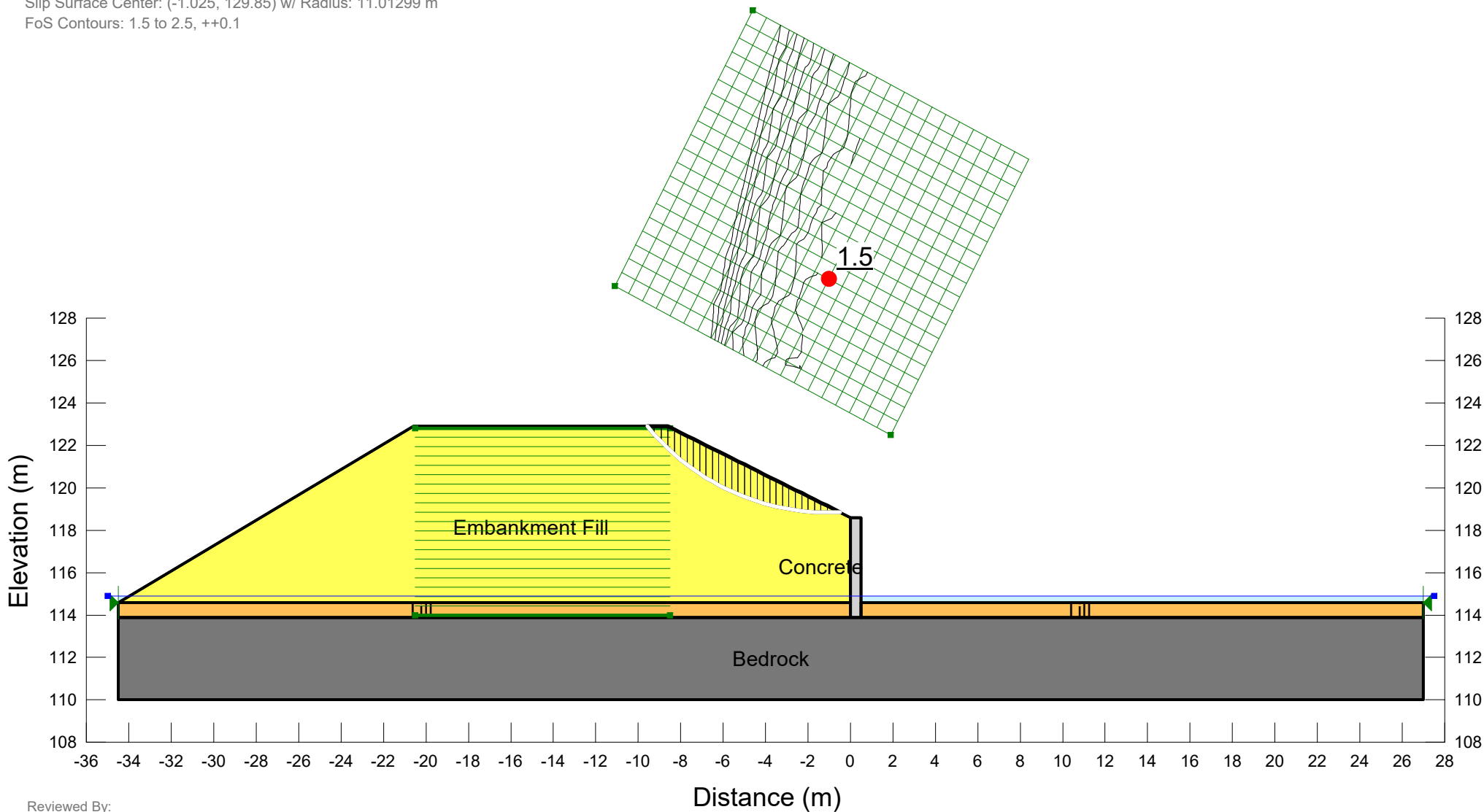


Figure 1

Title: Douglas Creek Culvert Rehabilitation
Comments:
Name: 1.1 East Wall - Pseudo-Static

Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m
PWP Conditions Source: Piezometric Line
Seismic, H: 0.109 V: 0
Slip Surface Center: (-1.025, 129.85) w/ Radius: 11.010707 m
FoS Contours: 1.2 to 2.2, ++0.1

Embankment Fill	20 kN/m³	0 kPa	32 °
Till	21 kN/m³	0 kPa	35 °
Bedrock			
Concrete	24 kN/m³		

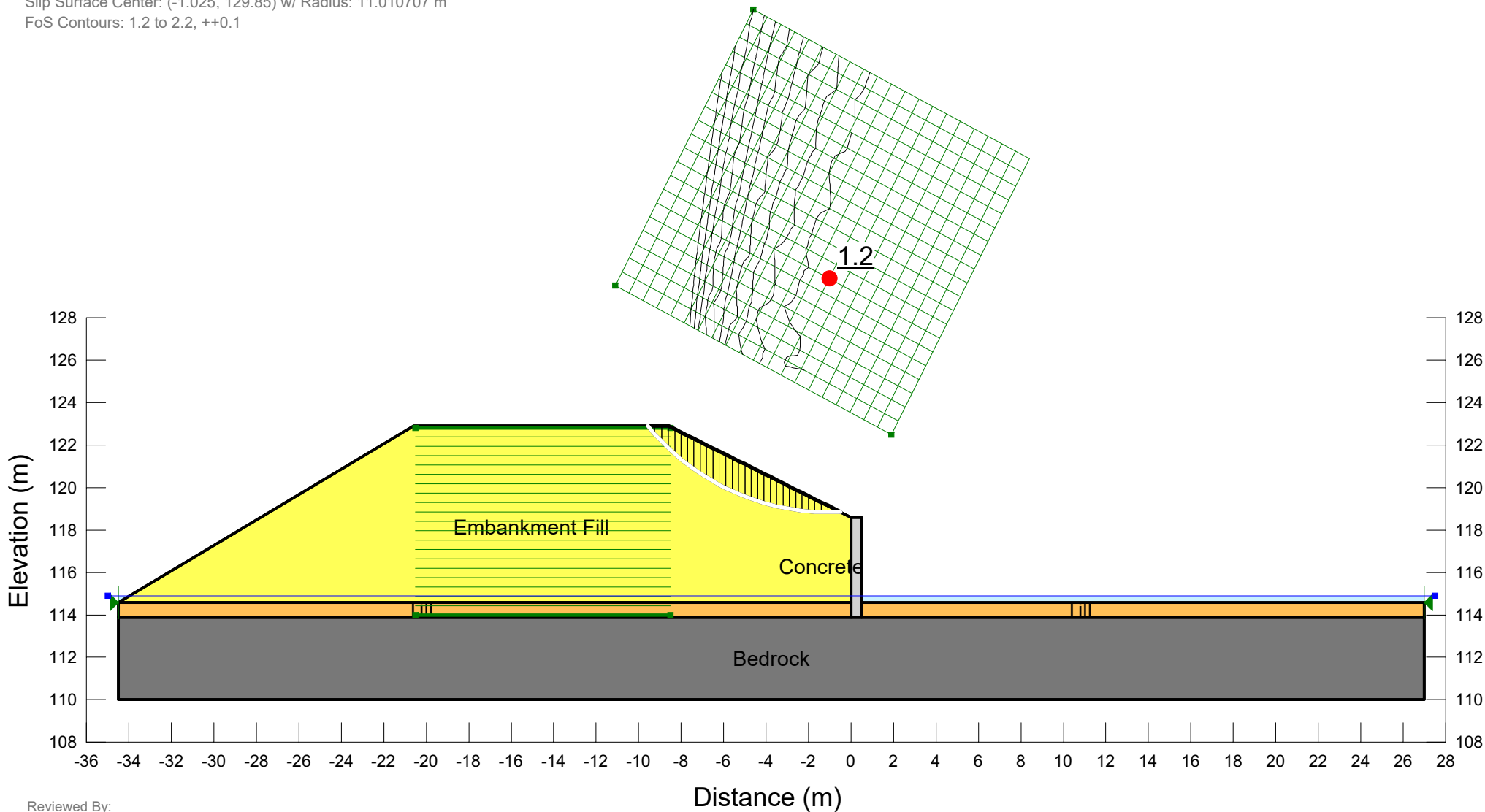


Figure 2

Title: Douglas Creek Culvert Rehabilitation
Comments:
Name: 1.0 West Wall - Static

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: (-1.025, 128.2) w/ Radius: 9.549602 m
FoS Contours: 1.5 to 2.5, ++0.1

Embankment Fill	20 kN/m³	0 kPa	32 °
Till	21 kN/m³	0 kPa	35 °
Bedrock			
Concrete	24 kN/m³		

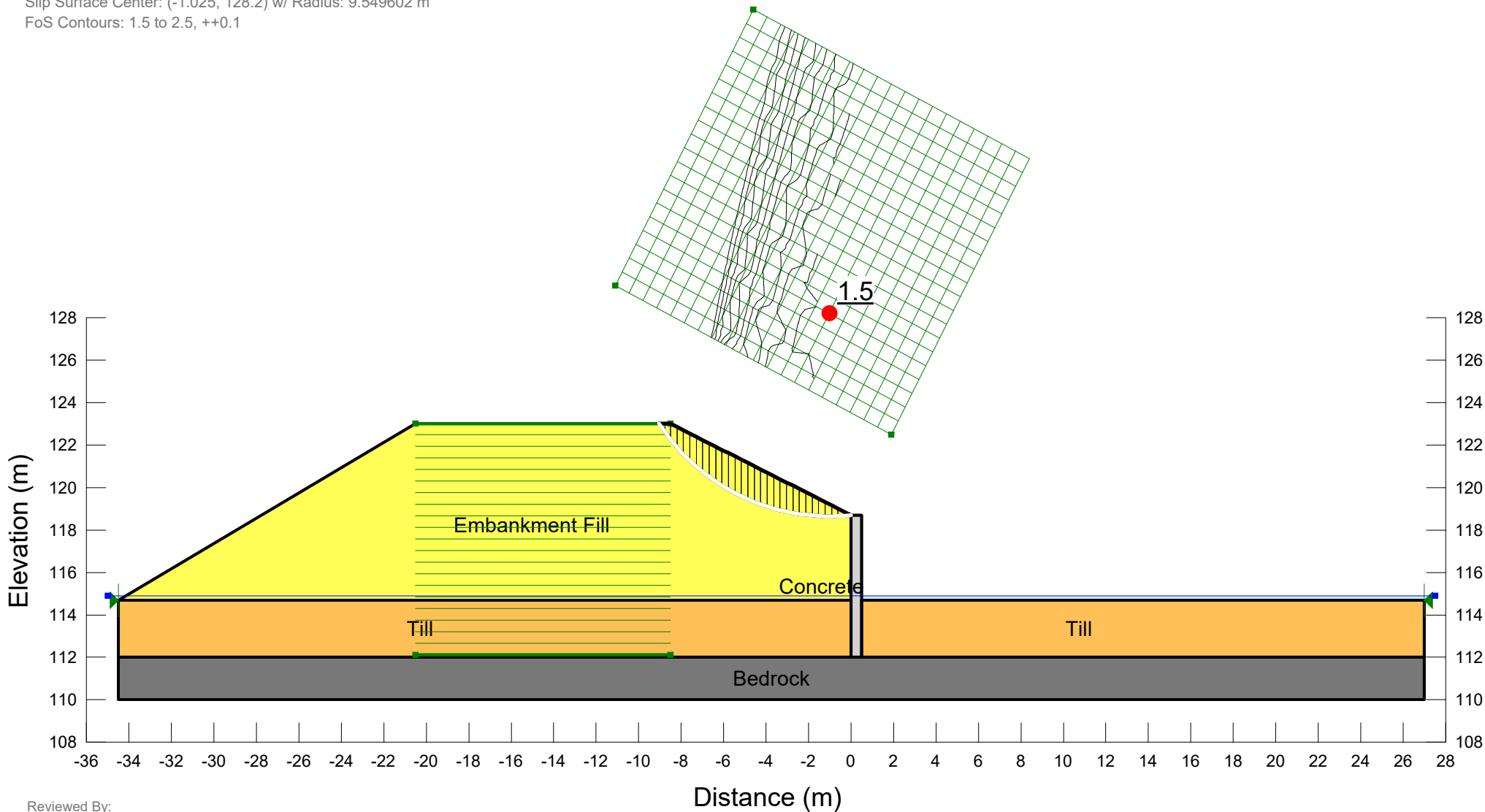


Figure 3

Title: Douglas Creek Culvert Rehabilitation
Comments:
Name: 1.1 West Wall - Pseudo-Static

Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m
PWP Conditions Source: Piezometric Line
Seismic, H: 0.109 V: 0
Slip Surface Center: (-1.025, 128.2) w/ Radius: 9.549602 m
FoS Contours: 1.2 to 2.2, ++0.1

Embankment Fill	20 kN/m³	0 kPa	32 °
Till	21 kN/m³	0 kPa	35 °
Bedrock			
Concrete	24 kN/m³		

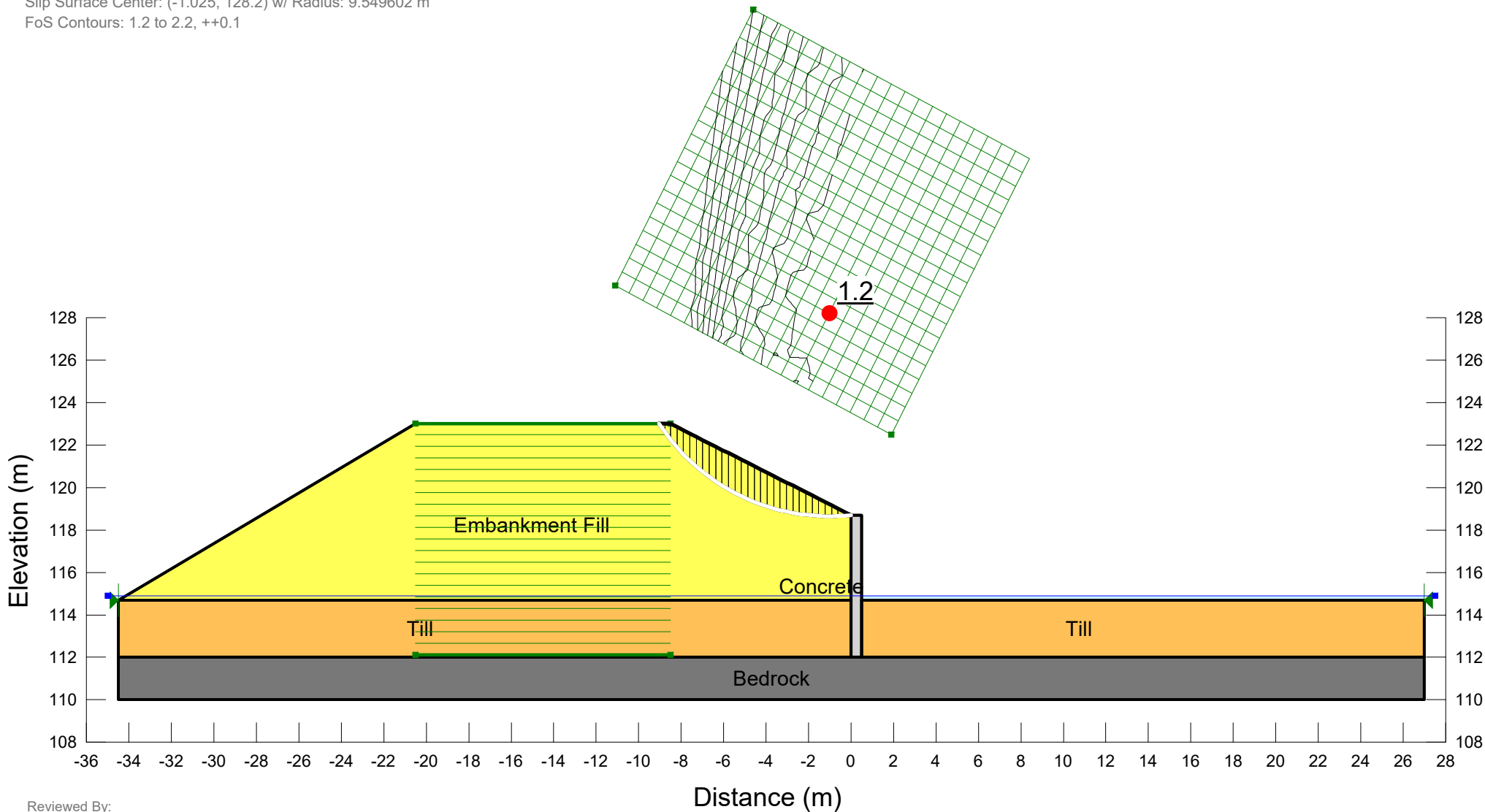


Figure 4

APPENDIX F

LIST OF REFERENCED SPECIFICATIONS NON-STANDARD SPECIAL PROVISIONS

LIST OF REFERENCED SPECIFICATIONS

OPSD 810.010	General Rip-Rap Layout for Sewer and Culvert Outlets
OPSD 3090.101	Foundation, Frost Penetration Depths for Southern Ontario
OPSS.PROV 180	General Specification for the Management of Excess Materials
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 of Structures
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 517F01	Amendment to OPSS 517, July 2017
Special Provision FOUN0003	Dewatering Structure Excavations, March 2018

NON-STANDARD SPECIAL PROVISIONS

RECOMMENDED WORDING FOR “NSSP – TEMPORARY PROTECTION SYSTEMS”

The bedrock profile varies across the site. Temporary protection systems will be installed in ground conditions that include sloping and shallow bedrock. The Contractor's installation method and temporary protection system design shall take into account existing, soil and bedrock conditions.

RECOMMENDED WORDING FOR “NSSP – OBSTRUCTIONS”

Excavations at the site or the installation of cofferdams or temporary protection system could encounter cobbles or boulders, concrete or other buried obstructions. Such obstructions may impede excavation and installation to the designed installation depths.

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

RECOMMENDED WORDING FOR “NSSP – UNDERMINING OF EXISTING CULVERT”

Excavations activities at the site should not undermine the existing culvert foundations. If excavation is required below the culvert foundation, the Contractor shall submit an underpinning strategy to the Contract Administrator for discussion prior to carrying out the excavation.