



THURBER ENGINEERING LTD.

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**DETAILED FOUNDATION INVESTIGATION AND DESIGN REPORT
HIGHWAY 527 WABINOSH RIVER CULVERT
99.6 KM NORTH OF ON-811, THUNDER BAY UNORGANIZED
SITE NO.: 48C-124/C
ASSIGNMENT NO. 6017-E-0013**

G.W.P. 6829-14-00

Geocres No.:

Report to:

Hatch Corporation

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PART 1. FACTUAL INFORMATION

1 INTRODUCTION

This section of the report presents the factual findings obtained from a foundation investigation completed for Wabinoash River Culvert (formerly known as the Waweig Lake Culvert) on Highway 527. The culvert is located approximately 99.6 km north of Tertiary Highway 811 within the Unorganized Thunder Bay District and conveys the Wabinoash River between Nameiben Lake and Waweig Lake. Thurber Engineering Limited (Thurber) carried out the current investigation as a sub-consultant to Hatch Corporation (Hatch) under Assignment No. 6017-E-0013.

The purpose of this investigation was to explore the subsurface conditions at the site and, based on the data obtained, to provide a borehole location plan, records of boreholes, stratigraphic profile, laboratory test results and a written description of the subsurface conditions. A model of the subsurface conditions influencing design and construction was developed in the course of the current investigation. No previous foundation investigation information was available for the subject culvert site within the online Geocres Library.

2 SITE DESCRIPTION

For project purposes, Highway 527 will be assumed to be oriented in a north-south direction.

The existing culvert, conveying the Wabinoash River under Highway 527 from Nameiben Lake to Waweig Lake, is a Steel Plate Corrugated Steel Pipe Arch (SPCSPA) culvert with an unknown construction date. A site survey plan from Hatch indicates that the culvert is approximately 6.3 m wide, 3.9 m high and approximately 18.9 m long at the obvert and 26.8 m long at the invert. The culvert alignment is generally east-west with the flow through the culvert toward the east.

At the location of the culvert, Highway 527 is a two-lane highway with a rural cross-section and narrow gravel shoulders. Steel cable guide rails are present along both sides of the highway in the area of the culvert. The embankment fill height above the culvert is approximately 1.6 m. The elevation of the road surface at the centreline is approximately 317.4 m. The existing embankment slopes are inclined approximately 1.9H:1V to 2H:1V. The land adjacent to the highway is undeveloped and densely vegetated with trees. The traffic volume for this section of Highway 527 is understood to be 230 AADT (2016).

Photographs showing the existing conditions in the area of the culvert are included in Appendix D for reference.

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3 SITE INVESTIGATION AND FIELD TESTING

Thurber contacted Ontario One Call in advance of the field investigation to obtain utility locate clearances in the vicinity of the intended boreholes.

The site investigation and field testing program for this site was carried out in two phases; the first phase was carried out between June 9th and August 10th, 2018 and the second phase was carried out between November 23rd and November 29th, 2018. The northing, easting and elevation of the boreholes and test pit are shown on the Borehole Location and Soil Strata Drawings in Appendix A and are summarized in Table 3-1. The site is within MTM Zone 15.

Table 3-1: Borehole and Test Pit Summary

Borehole or Test Pit No.	Location	Northing (m)	Easting (m)	Ground Surface Elevation (m)	Termination Depth (m)
BH 18-501	North of culvert – SB lane	5 555 280.8	367 398.5	317.4	12.8
BH 18-502	South of culvert – NB lane	5 555 271.8	367 395.9	317.4	9.8
BH 18-503	East end – culvert outlet	5 555 263.7	367 402.5	312.5	2.6
BH 18-504*	West end – culvert inlet	5 555 301.1	367 390.4	313.7	1.4
BH 18-505	South of culvert – SB lane	5 555 271.3	367 386.8	317.5	9.2
BH 18-506	South of culvert – TMB footing	5 555 253.4	367 371.6	317.5	6.9
BH 18-507	North of culvert – TMB footing	5 555 292.0	367 420.1	317.3	9.2
TP 18-508	Northwest of culvert	5 555 309.2	367 387.4	313.8	5.2

** Five attempts were made to drill this borehole due to the frequent boulders present. None of the other attempts penetrated deeper than 0.9 m.*

The drilling was carried out using a truck mounted CME 75 drill rig for on-road Boreholes 18-501 and 18-502, a track mounted Mobile B54 drill rig for on-road Boreholes 18-505, 18-506 and 18-507, and portable drilling equipment for off-road Boreholes 18-503 and 18-504. Test pit 18-508 was excavated with a Volvo ECR305CL Excavator.

Soil samples in the boreholes were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). A standard weight (64 kg) hammer was used for the SPT testing in the on-road boreholes and portable Borehole 18-504. A quarter-weight hammer was used for SPT testing at portable Borehole 18-503 due to difficult site access conditions. The N-values presented on the Record of Borehole

sheet for Borehole 18-503 have been corrected to provide an estimate of the N-value that would have been obtained with a standard 64 kg hammer.

Test Pit 18-508 was advanced to observe the general excavating conditions at the site, including relative groundwater inflow from the gravel till deposit. The soils exposed on the sides of the test pits were classified by visual and tactile examination. Bulk soil samples were collected from the excavator bucket from selected depths. The groundwater level and seepage conditions within the open test pit were noted and, upon reaching the target elevation, a field pumping test was performed in the open test pit using a 75 mm diameter trash pump. The test pit was backfilled with the excavated material to the original ground surface elevation.

A 19 mm diameter standpipe piezometer was installed in Borehole 18-503 to allow for measurements of the groundwater level after completion of drilling. A 51 mm diameter monitoring well with a flush mount cover was installed in Borehole 18-505 to allow for measurements of the groundwater level and to perform a rising head test. The piezometer and monitoring well installation details are illustrated on the respective Record of Borehole Sheets provided in Appendix B. All other boreholes were backfilled with a low-permeability mixture of cuttings and bentonite pellets in accordance with Ontario MOE Regulation 903 as amended. Boreholes advanced within paved areas were capped with granular fill followed by 150 mm of cold patch asphalt to reinstate the travelling surface. The piezometer installed during the phase one investigation was decommissioned in accordance with Ontario MOE Regulation 903 on August 11th, 2018. The monitoring well installed in Borehole 18-508 was tagged in accordance with MOE Regulation 903 and will need to be decommissioned during construction of the replacement culvert.

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

4 LABORATORY TESTING

Geotechnical laboratory testing consisted of natural moisture content determination and visual identification of all retained soil samples. Testing for grain size distribution was also carried out on selected samples to MTO and ASTM standards. Chemical analysis for determination of pH, conductivity, resistivity, sulphate and chloride concentrations was carried out on two soil samples and one surface water sample. Chemical analysis for determination of sulphide concentration was carried out on one soil sample.

In addition to the chemical analyses for sulphate and corrosion parameters, additional chemical analyses were carried out on a sample of groundwater collected from the monitoring well installed in Borehole 18-508 as part of a hydrogeological assessment to provide information on groundwater quality for the purpose of discharging pumped groundwater. The suite of parameters that were tested include metals, e. coli, total coliform, turbidity, alkalinity, ammonia, phosphorous, and total suspended solids (TSS).

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

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Details of the encountered soil stratigraphy are presented on the Record of Borehole and Test Pit Sheets included in Appendix B and the Borehole Locations and Soil Strata Drawings included in Appendix A. Photographs of the test pit are provided in Appendix E. A general description of the stratigraphy, based on the conditions encountered in the boreholes, is given in the following paragraphs. However, the factual data presented on the Record of Borehole and Test Pit Sheets takes precedence over this general description for interpretation of the site conditions. It must be recognized that the soil and groundwater conditions may vary between and beyond borehole locations.

In general terms, subsurface conditions at the site consist of prime surface treatment and granular embankment fill overlying native glacial till.

5.1 Embankment Fill

5.1.1 Prime Surface Treatment

Boreholes 18-501, 18-502, 18-505, 18-506 and 18-507 were drilled through the travelled lanes of the Highway 527 embankment and encountered a layer of prime surface treatment with a thickness of 20 mm.

5.1.2 Fill: Sand with Silt and Gravel to Gravel with Sand

Below the prime surface treatment in the on-road boreholes were layers of granular embankment fill ranging in composition from sand with silt and gravel to silty sand with gravel to gravel with sand. Frequent cobbles were noted to be present within the granular fill layers. The underside of the embankment fill ranged from 3.8 to 5.3 m below the existing roadway surface (elev. 312.1 to 313.7 m).

The SPT tests conducted in the fill gave N-values ranging from 8 blows for 300 mm of penetration to 100 blows for 50 mm of penetration indicating a relative density of loose to very dense, but typically compact to dense. Recorded moisture contents typically ranged from 5 to 26%.

Gradation analyses were completed on six samples of the granular fill. The grain size distribution curves for these samples are included in Figure C1 of Appendix C. The results of the tests are summarized in Table 5-1 below and are presented on the corresponding Record of Borehole Sheets in Appendix B.

Table 5-1: Gradation Results for Granular Fill

Soil Particle	Percentage (%)
Gravel	30 to 64
Sand	35 to 56
Silt and Clay	1 to 14

5.2 Surficial Sand

A surficial sand deposit ranging in composition from sand some silt and gravel to sand with silt and gravel was encountered in Boreholes 18-503 and 18-504. Trace organics were noted in this layer in Borehole 18-504. The underside depths of this layer ranged from 0.4 to 2.4 m below ground surface (elev. 310.1 to 313.3 m).

SPT tests conducted in this deposit gave N-values ranging from 17 blows for 300 mm of penetration to 100 blows for 230 mm of penetration, indicating a relative density of compact to very dense. The recorded moisture contents ranged from 9 to 13%.

Gradation analyses were completed on two samples of the surficial sand layer. The grain size distribution curves are included in Figure C2 of Appendix C and indicate an SP material. The results of the tests are summarized in Table 5-2 below and are presented on the corresponding Record of Borehole sheets in Appendix B.

Table 5-2: Gradation Results for Surficial Sand

Soil Particle	Percentage (%)
Gravel	13 to 45
Sand	49 to 82
Silt and Clay	5 to 6

5.3 Gravel with Sand to Sand with Silt and Gravel (Glacial Till)

A deposit of glacial till was encountered below the embankment fill in Boreholes 18-501, 18-502, 18-505, 18-506 and 18-507, below the sand layer in Boreholes 18-503 and 18-504 and from ground surface in Test Pit 18-508. The glacial till varies in composition from gravel with sand trace silt, to gravel with sand some silt, to gravel with silt and sand, to sand with silt and gravel. Cobbles and boulders were noted in the till in all boreholes and the test pit.

The test pit and all of the boreholes (except 18-501) were terminated within the glacial till layer at base elevations ranging from 307.6 to 312.3 m. Within Borehole 18-501, the upper portion of glacial till consisted of gravel with silt and sand and contained frequent cobbles and boulders. This upper portion of glacial till had an underside depth of 10.1 m below the existing ground surface (elev. 307.3 m) Below this depth the glacial till consisted of sand with silt and gravel and contained occasional cobbles. Borehole 18-501 was terminated within the lower glacial till at 12.8 m below ground surface (elev. 304.6 m).

SPT tests conducted in the upper portion of the glacial till deposit (above elev. 307.3 m) gave N-values ranging from 37 blows for 300 mm of penetration to 100 blows for 25 mm indicating a relative density of dense to very dense. Very poor sample recovery within the split spoon sampler was noted within this layer. SPT tests conducted in the glacial till below elevation 307.3 m gave N-values of 19 and 75 blows, indicating a relative density of compact to very dense. The recorded moisture contents ranged from 4 to 29%.

Gradation analyses were completed on eight samples of the gravel till and two samples of the sand till. The grain size distribution curves are included in Figures C3 through C5 of

Appendix C. The results of the tests are summarized in Table 5-3 below and are presented on the corresponding Record of Borehole and Test Pit sheets in Appendix B.

Table 5-3: Gradation Results for Glacial Till

Soil Particle	Percentage (%)	
	Gravel Till	Sand Till
Gravel	46 to 73	21 to 44
Sand	24 to 46	51 to 71
Fines	3 to 10	5 to 8

5.4 Groundwater and Hydraulic Conductivity

The water level was measured in the piezometer installed in Borehole 18-503, in the monitoring well installed in Borehole 18-505, in the open hole of Boreholes 18-501 and 18-506, and in the open Test Pit 18-508. A hydraulic conductivity (rising head) test was also carried out in the monitoring well installed in Borehole 18-505. The groundwater level measurements and interpreted hydraulic conductivity value are presented in Table 5-4 below. The analysis of the rising head test data is provided in Appendix F.

Table 5-4: Groundwater Level Observations

Borehole or Test Pit	Groundwater Level		Date of Measurement	Hydraulic Conductivity (cm/s)
	Depth (mbgs)	Elevation (m)		
18-501	4.4	313.0	June 10, 2018	-
18-503	0.0	312.5	August 12, 2018	-
18-505	6.4	311.1	November 30, 2018	1.7×10^{-2}
18-506	4.7	312.8	November 25, 2018	-
18-508	3.9	309.9	November 29, 2018	-

The river water level was surveyed at the culvert inlet and outlet and the measured elevations are provided in Table 5-5 below:

Table 5-5: River Water Level Observations

Location	Surface Water Elevation (m)	Date of Measurement
Culvert Inlet	312.5	August 9, 2018
Culvert Outlet	312.5	August 9, 2018

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

A pumping test was carried out within the Test Pit 18-508. The results of the pumping test are provided in Appendix F.

5.5 Analytical Testing

Two samples of soil were submitted to Paracel Laboratories in Ottawa, Ontario for analysis of pH, water soluble sulphate and chloride concentrations, resistivity and conductivity. One of the submitted samples was also tested for sulphide content. The analysis results are provided in Appendix C and are summarized in Table 5-6 below:

Table 5-6: Analytical Results Summary (Soil)

Borehole	18-502	18-503
Sample	SS8	SS2
Depth (m)	5.3 – 5.9	0.6 – 1.2
Chloride (µg/g)	13	6
Sulphate (µg/g)	10	< 5
Sulphide (%)	< 0.02	-
pH (-)	7.1	7.4
Resistivity (Ohm-cm)	21,300	16,600
Conductivity (uS/cm)	47	60

A surface water sample obtained upstream of the existing culvert on November 30 was also submitted to Paracel Laboratories in Ottawa, Ontario for analysis of conductivity, pH, resistivity, chloride and sulphate. The analysis results (identified as the waveig sample)_are provided in Appendix C and are summarized in Table 5-7 below.

Table 5-7: Analytical Results Summary (Surface Water)

Parameter	Result
Chloride (mg/L)	1
Sulphate (mg/L)	<1
pH (-)	7.4
Resistivity (Ohm-cm)	18,000
Conductivity (µS/cm)	56

The results of the analytical testing carried out as part of the hydrogeological assessment are also provided in Appendix C.

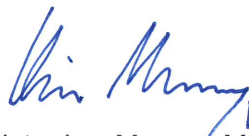
6 MISCELLANEOUS

Borehole and test pit locations were selected in consultation with Hatch and the Ministry of Transportation relative to the existing culvert and the existing site features. The as-drilled locations and ground surface elevations were surveyed by Thurber.

George Downing Estate Drilling Ltd. of Hawkesbury, Ontario and Maple Leaf Drilling of Winnipeg, Manitoba supplied and operated the drilling equipment for the on-road drilling and CCC Drilling of Ottawa, Ontario supplied and operated the drilling equipment for the off-road boreholes. The drillers were responsible for drilling, soil sampling, in-situ testing, piezometer installation and borehole decommissioning. LTL Contracting of Shuniah, Ontario supplied and operated the equipment required to excavate and backfill the test pit. Traffic control signage was provided by Thurber Engineering. The field investigation was supervised on a full-time basis by Mr. Nick Weil and Mr. Sean O'Bryan, C.E.T., of Thurber. Overall supervision of the investigation program was conducted by Mr. Stephen Dunlop, P.Eng.

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

Interpretation of the factual data and preparation of this report were carried out by Mr. Christopher Murray, P.Eng. and Mr. Stephen Dunlop, P.Eng. The report was reviewed by Dr. Fred Griffiths, P.Eng., a Designated Principal Contact for MTO Foundation Projects.



Dec 21/18

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PART 2. ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 INTRODUCTION

This section of the report provides an interpretation of the factual data from Part 1 of this report and presents geotechnical recommendations to assist the project team in designing a suitable foundation for the proposed replacement of the existing Wabinoash River Culvert (formerly known as the Waweig Lake Culvert) crossing Highway 527. The discussion and recommendations presented in this report are based on the information provided by Hatch and on the factual data obtained during the course of the investigation.

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

The existing culvert, conveying the Wabinoash River from Nameiben Lake to Waweig Lake under Highway 527, is a SPCSPA culvert with an unknown construction date. A site survey plan from Hatch indicates that the culvert is approximately 6.3 m wide, 3.9 m high and approximately 18.9 m long at the obvert and 26.8 m long at the invert. The culvert alignment is generally east to west with the flow through the culvert toward the east.

Invert elevations of 312.215 m and 312.068 m at the inlet and outlet, respectively are given in the CAD site survey information provided by Hatch. River bottom elevations of 311.9 and 311.8 m, at the inlet and outlet respectively, were surveyed by Thurber personnel during the field investigation. The embankment fill height above the culvert is approximately 1.6 m. The elevation of the road surface at the centreline is approximately 317.4 m. The existing embankment slopes are inclined between approximately 1.9H:1V and 2H:1V.

No previous foundation investigation information for the subject culvert was available in the online Geocres Library.

7.1 Preferred Structure

Drawings provided by Hatch indicate that the preferred culvert replacement option is an approximately 32 m long, 16 m wide, open footed corrugated steel box constructed on approximately the same alignment as the existing culvert. It is understood that the streambed elevations will be similar to the existing culvert.

It is also understood that the preferred construction methodology is an open-cut excavation with a one-lane temporary modular bridge. The temporary modular bridge is indicated to have spread footing foundations that are 1.5 m wide and 6.5 m long, with a road elevation that is approximately equal to the existing Highway 527 vertical profile. It is assumed that the temporary bridge will not be needed in the winter months.

7.2 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 (CHBDC), version CSA S6-14.

It is assumed that the proposed culvert structure has a consequence classification of *Typical Consequence*, 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 for this structural culvert.

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

The site coefficients used to determine the design spectral acceleration and displacement values are a function of the Site Class and the peak ground acceleration (PGA). At this site, the PGA for a reference Site Class C with a 2% probability of exceedance in 50 years (2475-year event) is 0.038g. This value is to be scaled by the $F(PGA)$ based on the site specific Site Class.

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. This site has been classified as a Site Class C in accordance with Section 4.4.3.2 of the CHBDC (S6-14).

8.3 Seismic Liquefaction

The Seed & Idriss Simplified Method was used to assess the potential for liquefaction at this site. Based on the low reference PGA and the subsurface conditions encountered at

the drilled locations at this site, the foundation soils are considered not susceptible to liquefaction during a seismic event.

9 DESIGN OPTIONS

9.1 Culvert Type and Foundation Alternatives

Selection of the culvert type must consider the proposed construction procedures, staging requirement, geotechnical resistance available in the foundation soils, the depth to suitable bearing stratum and post-construction settlement criteria. From a geotechnical perspective, the following culvert types were considered:

- Circular Pipes (Concrete, HDPE, Steel)
From a foundation engineering perspective, pipe culverts are a feasible culvert option. However, it is understood that numerous circular pipe culverts would be required to meet hydraulic requirements and therefore this option is not preferred.
- Closed Bottom Culvert (Box)
A precast segmental box culvert is considered a feasible option from a foundation engineering perspective. Precast sections, rather than cast-in-place construction, can be installed expediently with less potential for disturbance of the founding soils during installation. However, it is understood that this option is not preferred from a structural perspective due to the large span and high cover required.
- Open Bottom Culvert (Box, Arch)
Open bottom culverts are considered feasible for this site from a foundation engineering perspective but would require greater excavation and dewatering efforts during construction to place the foundation in the dry. Dewatering efforts could be reduced by constructing the lower portion of the footings with tremie concrete below the groundwater level.
- Steel Sheet Pile Walls with Precast Concrete Slab
A culvert consisting of two rows of parallel sheet pile walls supporting precast concrete slabs is not considered feasible at this site because of the high risk of the sheet piles refusing at an insufficient depth in the very dense glacial till, which contains cobbles and boulders.

A comparison of these alternatives, based on their respective advantages and disadvantages, is included in Appendix G. It is not considered economical or practical to support a culvert on deep foundations at this site and therefore this option is not presented in this report.

9.2 Construction Methodology Alternative

For the proposed culvert replacement, the following construction methods were considered.

- Open Cut with Full Road Closure and Temporary Detour
Installation of a new culvert using open cut techniques and a full road closure would allow for an expedited construction schedule and could reduce costs associated with requiring roadway protection and ground/surface water control. However, it is

understood that an acceptable detour route is not available and therefore this option is not feasible.

- Open Cut with Temporary Modular Bridge Spanning Excavation

The culvert replacement can take place within a full width open cut excavation with a single lane temporary modular bridge (TMB) spanning the excavation provided that there is sufficient vertical clearance beneath the bridge to complete the installation.

- Open Cut with Staged Temporary Widening

Widening of the existing highway and/or construction of a temporary detour embankment to accommodate traffic passage during construction is considered feasible from a geotechnical perspective. A review of the environmental acceptability for placing fill within the river, the requirement for property acquisition, and alteration to highway geometry is also needed to assess this option.

- Open Cut with Staged Replacement and Temporary Protection System

The use of open cut techniques in conjunction with staged culvert replacement is a potentially feasible construction option from a geotechnical perspective. This option will require roadway protection, as discussed further in Section 11.2, installed along the embankment centerline to maintain a single lane of traffic flow along the current highway alignment. Installation of sheet piles will be difficult through the dense till with cobbles and boulders. Due to the required height of soil to be retained, the roadway protection may require lateral support in the form of rakers, struts, or deadman anchors to reduce lateral deflections.

- Trenchless Techniques

Trenchless techniques would have the advantage of minimum disruption to traffic and would avoid a large excavation through the existing highway embankment. However, this option will be high risk due to the possibility of encountering obstructions and dewatering challenges. The limited tunnel cover is also problematic and presents a significant risk. The anticipated size of replacement culvert will also limit the available installation methods. A trenchless installation is not recommended at this site due to the high risk involved.

9.3 Recommended Approach for the Culvert Replacement

From a foundation engineering perspective, the preferred culvert replacement options are a circular pipe (e.g. corrugated steel) or a precast segmental box culvert using open cut techniques. However, it is understood that these options are not preferred from a hydraulics and structural perspective and are not discussed further herein. Alternatively, an open bottom culvert is feasible, but will require frost protection and therefore deeper excavations with additional dewatering requirements. To reduce the dewatering requirements, the lower portion of the footing could be constructed with tremie mass concrete. A temporary modular bridge, temporary protection systems (TPS), or a temporary widening would be needed to facilitate construction.

10 FOUNDATION DESIGN RECOMMENDATIONS

Foundation design aspects for the replacement culvert include subgrade conditions, geotechnical resistances, settlement of the founding soils, imposed loading pressures, erosion control, temporary modular bridge and temporary protection system design, groundwater control and stability of stage construction. The culvert must be designed to resist loading including lateral earth pressures, hydrostatic pressure, weight of embankment fill, traffic loading and any surcharge due to construction equipment and activities under static and seismic conditions.

10.1 Culvert Foundation Bearing Resistances

An open bottom culvert may be founded on the very dense glacial till below the frost depth (see Section 10.3). It is understood that the founding elevation will be 309.4 m. The existing stratigraphy at this founding elevation consists of very dense glacial till. For a footing width of up to 2.0 m wide, the design can be based on the following factored geotechnical resistance values:

- Factored Geotechnical Resistance at ULS of 540 kPa
- Factored Geotechnical Resistance at SLS of 395 kPa

The factored geotechnical resistances include the following factors:

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

The bearing resistance values are for vertical, concentric loading. In the case of eccentric or inclined loading, the bearing resistance must be reduced in accordance with CHBDC Clause 6.10.3 and Clause 6.10.4. Foundation settlement, based on the supplied SLS resistance, is expected to be up to 25 mm.

Resistance to lateral forces/sliding resistance between concrete and native glacial till or compacted Granular 'A' (Section 10.2) should be evaluated in accordance with the CHBDC assuming an unfactored coefficient of 0.55 for cast-in-place concrete. A geotechnical resistance factor against sliding (ϕ_{gu}) of 0.80 may be used.

It is noted that construction will extend below the observed river water level. Water diversion and dewatering (Section 11.3) will be required to construct the footings in the dry.

10.2 Subgrade Preparation and Backfilling

Subgrade preparation for the culvert replacement should include excavation and removal of the existing culvert and backfill materials. All organics, soft or loose deposits, disturbed soils, and deleterious materials must be stripped from the footprint of the foundation to expose competent subgrade at or below the desired founding elevations.

The exposed final subgrade must be inspected to confirm that the subgrade is suitable and uniformly competent. Any soft or organic materials at the subgrade level should be sub-excavated and backfilled with granular fill consisting of OPSS.PROV 1010 Granular A

material as soon as practical to protect the subgrade from disturbance during construction. The granular fill should be compacted as per OPSS.PROV 501. Any boulders or cobbles encountered at the subgrade elevation that become loosened/disturbed should be removed and the excavation backfilled with compacted Granular A or mass concrete.

There is no bedding requirement for open footed culverts; however, the glacial till could be disturbed during excavation and it may be difficult to achieve a level bearing surface due to the presence of cobbles and boulders. Therefore, a leveling pad of mass concrete or compacted Granular A should be placed promptly after excavation and inspection in accordance with SP109S12. After the leveling pad is completed and the concrete, if applicable, has set, the culvert footings can be constructed directly on the leveling pad without the need for an additional granular pad or bedding material.

The glacial till subgrade may be disturbed when saturated from both construction traffic and weather. Construction equipment should not be permitted to travel on the exposed subgrade. It is noted that construction will extend below the river elevation. Water diversion and dewatering will be required to prepare the subgrade in the dry. Refer to Section 11.3 for additional comments on groundwater and surface water control.

It is recommended that culvert cover be in accordance with OPSS 902 and consist of free-draining, non-frost susceptible granular materials such as Granular A, or Granular B Type II with a maximum particle size of 26.5 mm, meeting the requirements of OPSS.PROV 1010.

The backfill requirements for an open bottom culvert should be consistent with Section 7 of the CHBDC, OPSS.PROV 501 and OPSS 902 and consist of material meeting the requirements of OPSS Select Subgrade Material or Granular B Type I or III and should be compacted in regular lifts as per OPSS.PROV 501 and the CHBDC. Care must be exercised when compacting the fill adjacent to and above the culvert in order not to damage the culvert. Heavy compaction equipment used adjacent to the culvert must be restricted in accordance with OPSS.PROV 501. Backfill requirements of the metal box culvert supplier must also be confirmed.

10.3 Frost Depth

The depth of frost penetration at this site is estimated to be 2.6 m (OPSD 3090.100), which will need to be considered for open bottom culverts and wing walls, if required.

A frost taper is not considered necessary at this site since the existing granular embankment fill has a low susceptibility to frost and there does not appear to be any frost related pavement issues at the site. Additional details on the pavement investigation for the frost taper are provided within the Pavement Design Report.

10.4 Lateral Earth Pressures

Lateral earth pressures parameters provided in Table 10-1 and Table 10-2 in the sections below are based on the assumptions that the pressures are acting on a vertical plane at the outside edge of the culvert and the backfill is fully drained so that there are no unbalanced hydrostatic pressures above the permanent groundwater level. If adequate drainage cannot be confirmed, the potential for buildup of hydrostatic pressures should be considered in design. Typically, a proprietary design of metal arch/box culverts is carried out by the

manufacturer. The manufacturer will need to assess the soil-structure interaction inside these planes to complete the design. Where ground surfaces are horizontal or sloped at 2H:1V (for head walls or wing walls) behind vertical wall/plane, the corresponding coefficients provided in Tables 10-1 and 10-2 should be used. For other backfill and wall geometries, Thurber will need to calculate the appropriate earth pressure coefficients.

10.4.1 Static Lateral Earth Pressure

Lateral earth pressures acting on structures should be computed in accordance with the CHBDC. Under drained conditions the lateral earth pressure is generally given by the following expression:

$$p_h = K * (\gamma h + q)$$

where:

- p_h = horizontal pressure on the wall at depth h (kPa)
- K = earth pressure coefficient (see table below)
(K_a for yielding walls, K_o for non-yielding walls)
- γ = unit weight of retained soil (see table below), use submerged unit weight below groundwater level
- h = depth below top of fill where pressure is computed (m)
- q = value of any surcharge (kPa)

A lateral earth pressure due to backfill compaction should be added to the calculated lateral earth pressure in accordance with Clause 6.12.3 of the CHBDC. Typical earth pressure coefficients for backfill are shown in Table 10-1.

Table 10-1. Static Earth Pressure Coefficients

Condition	Earth Pressure Coefficient (K)					
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I or III $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$		OPSS SSM and Existing Sand Fill $\phi = 30^\circ, \gamma = 21.0 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active, K_A (Yielding Wall)	0.27	0.39	0.31	0.47	0.33	0.54
At Rest, K_o (Non-Yielding Wall)	0.43	-	0.47	-	0.50	-
Passive, K_P (Movement towards Soil Mass)	3.7	-	3.3	-	3.0	-
Soil Group ^(*)	"medium dense sand"		"loose to medium dense sand"		"loose sand"	

Note: (*) for use with Figure C6.16 of the Commentary to the CHBDC.

The use of a material with a high friction angle and low earth pressure coefficients (Granular A or Granular B Type II) is preferred as it results in lower earth pressures acting on the culvert.

The parameters in the table correspond to full mobilization of active and passive earth pressures and require certain relative movements between the wall and adjacent soil to produce these conditions. The values to be used in design can be assessed from Figure C6.16 of the Commentary to the CHBDC using the soil group designation as outlined in Table 10-1. Active earth pressures should be used for any head/wing walls or unrestrained walls. For rigid structures, at-rest horizontal earth pressures would apply for design.

10.4.2 Combined Static and Seismic Lateral Earth Pressure

In accordance with Clause 4.6.5 of the CHBDC (S6-14), retaining structures should be designed using dynamic earth pressure coefficients that incorporate the effects of earthquake loading. 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:

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

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

The coefficients of horizontal earth pressure for seismic loading presented in Table 10-2 may be used. The provided earth pressure coefficients are based on a Seismic Site Class C and a PGA with a 2% probability of exceedance in 50 years of 0.038g (Geological Survey of Canada – Fifth Generation) and a $F(PGA)$ of 1.00 as per Table 4.8 of the CHBDC (S6-14 update No. 2, July 2017).

Table 10-2. Dynamic Earth Pressure Coefficients

Condition	Earth Pressure Coefficient (K)			
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I or III $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Slope Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Slope Surface Behind Wall (2H:1V)
Active, K_{AE} Yielding Wall	0.28	0.42	0.32	0.50
Active, K_{AE} Non-Yielding Wall	0.29	0.44	0.33	0.54

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 soils 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_A for yielding walls, K_o for non-yielding walls)
γ	=	unit weight of retained soil, use submerged unit weight below groundwater level
K_{AE}	=	combined static and seismic earth pressure coefficient
H	=	total height of the wall (m)

10.5 Embankment Design and Reinstatement

10.5.1 Embankment Reconstruction

Embankment reconstruction after culvert replacement should be carried out in accordance with OPSS.PROV 206. The embankment should be reinstated with side slopes of 2H:1V (or flatter) if constructed using Select Subgrade Material (SSM) or Granular B Type I/III (OPSS.PROV 1010). The fill should be placed and compacted in accordance with OPSS.PROV 501.

Backfill requirements of the metal box culvert supplier will also need to be confirmed.

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

10.5.2 Embankment Settlement and Stability

Provided the subgrade is prepared as outlined above and construction of the embankment is carried out in accordance with recommendations provided within this report, the embankment side slopes should remain stable.

It is understood that no permanent grade raise or widening is anticipated along the alignment of Highway 527 and therefore negligible settlement is expected to occur beneath the embankment.

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

10.6 Cement Type and Corrosion Potential

Analytical tests were completed to determine the potential for degradation of the concrete in the presence of soluble sulphates and the potential for corrosion of exposed steel. The concentration of soluble sulphate provides an indication of the degree of sulphate attack that is expected for concrete in contact with soil and groundwater at the site. Soluble sulphate concentrations less than 1000 µg/g in soil generally indicate a low degree of

sulphate attack is expected for concrete in contact with soil and groundwater. The class of concrete selected should consider the effects of road de-icing salts.

The pH, resistivity and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. The test results provided in Section 5.5 may be used to aid in the selection of coatings and corrosion protection systems for buried steel objects. The corrosion effects of road de-icing salts should also be considered.

10.7 Temporary Modular Bridge Design

Although this bridge is to be a temporary structure, the following recommendations have assumed a consequence level of “Typical”. Should the Ministry consider that a “Low” consequence level is appropriate, the Consequence Factor and the recommended geotechnical resistance values could be increased.

10.7.1 Foundation Bearing Resistances

It is understood that it is proposed to found the temporary modular bridge on spread footings embedded 1.2 m below grade. Spread footings for the temporary modular bridge can be constructed on an engineered pad consisting of a minimum 0.5 m thick layer of Granular A material placed directly over the existing embankment fill soils. The Granular A pad must be placed in lifts no thicker than 150 mm and compacted in accordance with OPSS.PROV 501. The top of the Granular A pad must extend to 1.0 m beyond the edge of all sides of the footing and be sloped away from the footing at 1H:1V, or flatter.

The subgrade for the temporary modular bridge should be inspected prior to placement of the footing to ensure that the soils are as described within this report. Any exposed organic or soft/loose materials should be sub-excavated and replaced with Granular A compacted in accordance with OPSS.PROV 501.

The footings should be founded within the existing embankment geometry and not extend onto widened fill (if present). The following factored geotechnical resistance values are applicable for a 1.5 m wide cast-in-place footing with minimum setback distances and temporary slopes inclined at the values presented in Section 10.7.2 below. If different slope geometries are required, the bearing resistance values provided below will need to be re-evaluated.

- Factored Geotechnical Resistance at ULS of 200 kPa
- Factored Geotechnical Resistance at SLS of 135 kPa

The geotechnical bearing resistances provided above include a resistance factor of 0.5 (ϕ_{gu}) and 0.8 (ϕ_{gs}) for the ULS and SLS values, respectively, as per Table 6.2 of the CHBDC (static analysis – typical understanding). The geotechnical resistances are for vertical concentric loading only on cast-in-place footings and should be adjusted for the effects of inclined or eccentric loadings, where applicable, in accordance with CHBDC Clause 6.10.3 and 6.10.4. Foundation settlement, based on the supplied SLS resistance, is expected to be up to 25 mm.

The horizontal resistance against sliding for a cast-in-place concrete footing founded on engineered fill can be computed using a friction factor of 0.55. Appropriate resistance factors should be applied for the design.

Frost protection is not required for the modular bridge footings provided the bridge is decommissioned prior to the onset of winter.

10.7.2 Global Slope Stability

Global slope stability of the temporary slopes in front, and to the sides, of the temporary modular bridge footings has been modeled under static loading conditions using the commercially available slope stability program Slope/W (Version 9) of the GeoStudio software package developed by Geo-Slope International with the option for Morgenstern-Price method of slices for limit equilibrium analyses.

The results of the analyses indicate that the factor of safety against global slope instability in the longitudinal and transverse directions is equal to 1.5 provided that the following conditions are met:

- The temporary forward slope and temporary widened side slopes are no steeper than 1.5H:1V.
- The TMB footings are set back from the crest of the forward slope by a lateral distance of at least 5 m measured from the base of the footing.
- The TMB footings are set back from the crest of the side slopes by a lateral distance of at least 2 m measured from the base of the footing.
- The bearing pressures do not exceed the ULS bearing resistance value provided in Section 10.7.1 above.

It is important that the construction activities do not undermine the slopes or steepen the maximum 1.5H:1V slope geometry. Appropriate scour and erosion protection measures (see Section 11.4) should remain in place and functional for the duration of construction.

11 CONSTRUCTION CONSIDERATIONS

11.1 Excavations

All excavations must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of OHSA, the fill and native soils may be classified as Type 3 soil. Below the water table, the soils are also classified as Type 3 soils.

Excavations for the culvert replacement must be carried out in accordance with OPSS 902 and will be carried out through the existing embankment fill and extend into the underlying native deposits (sand, and glacial till). Selection of the equipment and methodology to excavate and prepare the founding surface is the responsibility of the Contractor. Stockpiling or surface surcharge should not be allowed on the embankment or side slopes.

At locations where there are space restrictions or where a slope has to be retained, the excavations will need to be carried out within a protection system. Further discussion on temporary protection systems (TPS) is presented in Section 11.2.

11.2 Temporary Protection Systems

Temporary Protection Systems (TPS) may be required during various stages of construction and must be implemented in accordance with OPSS.PROV 539.

If the TPS will be in close proximity to the temporary modular bridge footings, the TPS must be designed to resist the loadings from the temporary bridge footings as well as all associated loadings from construction activities. In that case, the stability of the footings will also be reliant on the stability of the TPS; therefore, the TPS should be designed for Performance Level 1b (10 mm of horizontal deflection) and should not be removed until the temporary modular bridge is removed from service. If the TPS is not supporting the modular bridge footings, a Performance Level 2 (maximum 25 mm horizontal deflection) would be appropriate.

The actual pressure distribution acting on the shoring system is a function of the construction sequence and the relative flexibility of the wall and these factors must be considered when designing the shoring system. The protection system near the culvert could be left in place and cut off in accordance with OPSS.PROV 539 to limit the disturbance of subgrade under the new culvert during removal of the TPS.

Lateral earth pressure coefficients, under fully mobilized conditions, that can be used in design for the embankment fill and culvert backfill are provided in Table 10-1. The lateral earth pressure coefficients for the underlying native soils are given below for a vertical wall and a horizontal backslope:

Sand

γ	=	19	(kN/m ³ , bulk unit weight of soil)
K_A	=	0.33	
K_P	=	3.0	

Glacial Till

γ	=	21	(kN/m ³ , bulk unit weight of soil)
K_A	=	0.27	
K_P	=	3.7	

Submerged unit weight should be used below the groundwater level.

The design of roadway protection is the responsibility of the Contractor. All protection systems should be designed by a licensed Professional Engineer experienced in such designs and retained by the Contractor. The design of the roadway protection system must incorporate traffic loading and surcharge loading due to construction equipment and operations.

It is recommended that an NSSP be included in the tender documents to alert the Contractor to the potential for cobbles and boulders and obstructions within the fill and glacial till.

Given the presence of obstructions and shallow refusal in Boreholes 18-503 and 18-504, installation of sheet piles will be difficult. It is anticipated that a soldier pile and lagging system will be utilized for the TPS. Deadman or bedrock anchors, struts and/or raker supports may be required to achieve the specified performance level.

11.3 Surface and Groundwater Control

11.3.1 General

The depth of excavations required to construct the culvert footings will extend below the river 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 river/surface water flow at the site to permit the replacement of the culvert in stable excavation.

Culvert construction, subgrade preparation and placement and compaction of granular materials should be carried out in the dry. However, it is noted that the soils at this site have a high hydraulic conductivity and dewatering rates are expected to be significant. To reduce the amount of dewatering at the site, consideration can be given to constructed the lower portion of the footings below the water level by means of tremie mass concrete, noting that dewatering will still be required to construct the upper portion of the footing.

11.3.2 Groundwater Inflow Estimate

The potential requirement for dewatering was assessed following a review of the geologic and hydrogeologic conditions at the site as well as the current understanding of the construction details. Based on the Preliminary General Arrangement drawing provided by Hatch, the work entails removal of the existing culvert and replacement with an open footing SPCSP arch/box culvert. Two cast-in-place concrete footings will be poured, and granular bedding material will be placed between the footings. The culvert will be supported by the footings.

Two construction methodologies were evaluated as part of the hydrogeological assessment, as follows:

Scenario 1) The excavation for both footings are carried out concurrently and the excavations are dewatered below the proposed underside of footing elevation (El. 309.4 m).

Scenario 2) The excavation for each footing is carried out separately (only one trench is dewatered at a time) and the excavations are dewatered to an elevation of 311.2 m. Below this level, the footings would be constructed below the water level by means of tremie mass concrete.

A summary of the key assumptions for each of the scenarios is provided in Table 11-1 below.

Table 11-1. Hydrogeological Assessment Assumptions

Parameter	Scenario 1 (Full Depth)	Scenario 2 (Tremie)
Groundwater Elevation	313.4 m	313.4 m
Dewatering Target Elevation	308.4 m	311.2 m
Excavation Area	35 m x 20 m	35 m x 10 m
Primary Soil Unit to be Dewatered	Gravel Till	
Hydraulic Conductivity	1.7×10^{-2} cm/s	

Aquifer Bottom Elevation	Not defined – assumed 304.6 m
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For each scenario, a base groundwater extraction flow rate was estimated, and a factor of safety of 3 was applied to this flow rate to provide an allowance for removal of water from soil storage, variation in hydraulic conductivity, actual excavation dimensions and geometry, and groundwater levels due to seasonality or other factors, etc. Further, an additional amount of water taking was estimated to account for rainfall or surface runoff into the excavations, in the amount of 75 mm of rainfall over 24 hours, to determine a budgeted peak rate of extraction. A radius of influence for the water taking was also estimated based on an assumed number of days of extraction and other geologic factors. This analysis assumes that no surface water taking has been included in the analysis, since it has been assumed that the river water would be diverted around the work area.

The calculations for the budgeted peak flow rate as well as the radius of influence are provided in Appendix I, which includes the theory and formulae. Based on the assumed parameters presented in Table 11-1, the peak budgeted flow rates for Scenarios 1 and 2 were estimated to be approximately 3,464,000 and 2,228,000 L/day, respectively. These values were calculated based on base groundwater flow estimates of 1,137,000 and 734,000 L/day, respectively, which were multiplied by a safety factor of 3, and to which flow rates of 53,000 and 26,000 L/day, respectively, were added to account for rainfall into the excavation. As indicated previously, this estimate does not include any surface water (river) flows.

The radius of influence was estimated to be approximately 240 metres assuming 60 days of continuous pumping and a storage coefficient of 0.3.

11.3.3 Water Quality

The groundwater sample that was taken from the monitoring well at BH 18-505 was analyzed for metals, ammonia, phosphorus, alkalinity, pH, and total suspended solids (see Appendix C for the results). Several of the parameters exceeded the Provincial Water Quality Objectives (or Interim Objectives). It is noted that the quality of the water taken from a monitoring well can be poorer than that of the true groundwater conditions due to silt buildup and agitation in the vicinity of the well.

11.3.4 Recommendations

Given that the peak budgeted flow rate for both scenarios exceeds 400,000 L/day, a Category 3 Permit To Take Water (PTTW) will be required. This will require a Hydrogeological Study that is in accordance with guidance provided by the Ministry of the Environment, Conservation and Parks (MECP). This will consider potential impacts of dewatering on water supply, structures, surface water and the water quality. A pre-consultation meeting or conference call may be arranged with MECP to review the anticipated requirements of the Hydrogeological Study. An application for the Category 3 PTTW will need to be completed, and payment of the PTTW fee to MECP will be required. It may take three months or more to receive the permit once the application and Hydrogeological Study have been received in good order by MECP.

It is anticipated that some form of water treatment will be required prior to discharge to the natural environment, to remove sediment and metals at a minimum, to meet the requirements of the PTTW for this project. In addition, given the high permeability of the soil, it is anticipated that the dewatering method may include the use of wells or well-points in addition to sumps. However, the specific dewatering and treatment design will be 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. Also, it is highly 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. An NSSP amendment to FOUN0003 is recommended. Suggested wording is provided in Appendix J.

To limit the amount of dewatering, consideration could be given to carrying out the excavation below the drawn-down water level in the wet and constructing a layer of mass concrete using tremie methods up to the drawn-down water level, which should be drawn down to an elevation of at least 311.2 m. The mass concrete should extend at least 0.3 m beyond the edge of the footings in all directions to allow for potential variations in the footing size below the drawn down water level. Footings for the open bottom culvert could then be constructed on the surface of the mass concrete.

11.4 Scour Protection and Erosion Control

The Contractor should provide silt fences and erosion control blankets as per OPSS 805 throughout the duration of construction to prevent transport of silt/sediment. Slope protection and drainage measures will be required to ensure the long-term surficial stability of the embankment slopes. Slope vegetation should be established as soon as possible after completion of the embankment fills in order to limit surficial erosion.

Particle size analyses in conjunction with the Wischmeier Nomograph indicate that the granular fill, sand and glacial tills encountered at this site have a low potential for soil erodibility.

Scour and erosion protection should be provided for the culvert inlet and outlet areas. Design of the scour and erosion protection measures must consider hydrologic and hydraulic concerns and should be carried out by specialists experienced in this field.

Typically, rock protection should be provided over all earth surfaces subjected to flowing water in accordance with OPSS 511. Treatment at the outlet 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.

11.5 Disturbed Subgrade

The excavation for Test Pit 18-508 was substantial in size. If the founding area of the new culvert footings overlaps any subgrade that was disturbed, the disturbed soils should be excavated and replaced with mass concrete or compacted Granular A.

12 CONSTRUCTION CONCERNS

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

- Disturbance of the soil subgrade. Construction equipment should not be on the final subgrade. The final subgrade should be levelled with a layer of either compacted Granular A or mass concrete.
- Cobbles/boulders and/or buried obstructions will be encountered in the existing embankment fill and in the native tills at this site and could interfere with installation of the roadway protection system and excavation progress.
- River water levels will fluctuate. Excavation will involve lowering the water level below the excavation base to maintain a dry excavation and stable side slopes. The dewatering scheme will be critical for culvert construction at this site.
- The Contractor's selection of construction equipment and methodology must include assessment of the capability of the existing embankment to support the proposed construction equipment and any temporary structure fill (i.e., as a pad for crane support).

The successful performance of the culvert installation will depend largely upon good workmanship and quality control during construction. Subgrade examination should be carried out by qualified geotechnical personal during construction in accordance with SP109S12 to confirm that foundation recommendations are correctly implemented and material specifications are met.

13 CLOSURE

Engineering analysis and preparation of this report were carried out by Mr. Christopher Murray, P.Eng. and Mr. Stephen Dunlop, P.Eng. The report was reviewed by Dr. Fred Griffiths, P.Eng., a Designated Principal Contact for MTO Foundation Projects.

Thurber Engineering Ltd.
Report Prepared By:



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Geotechnical Engineer



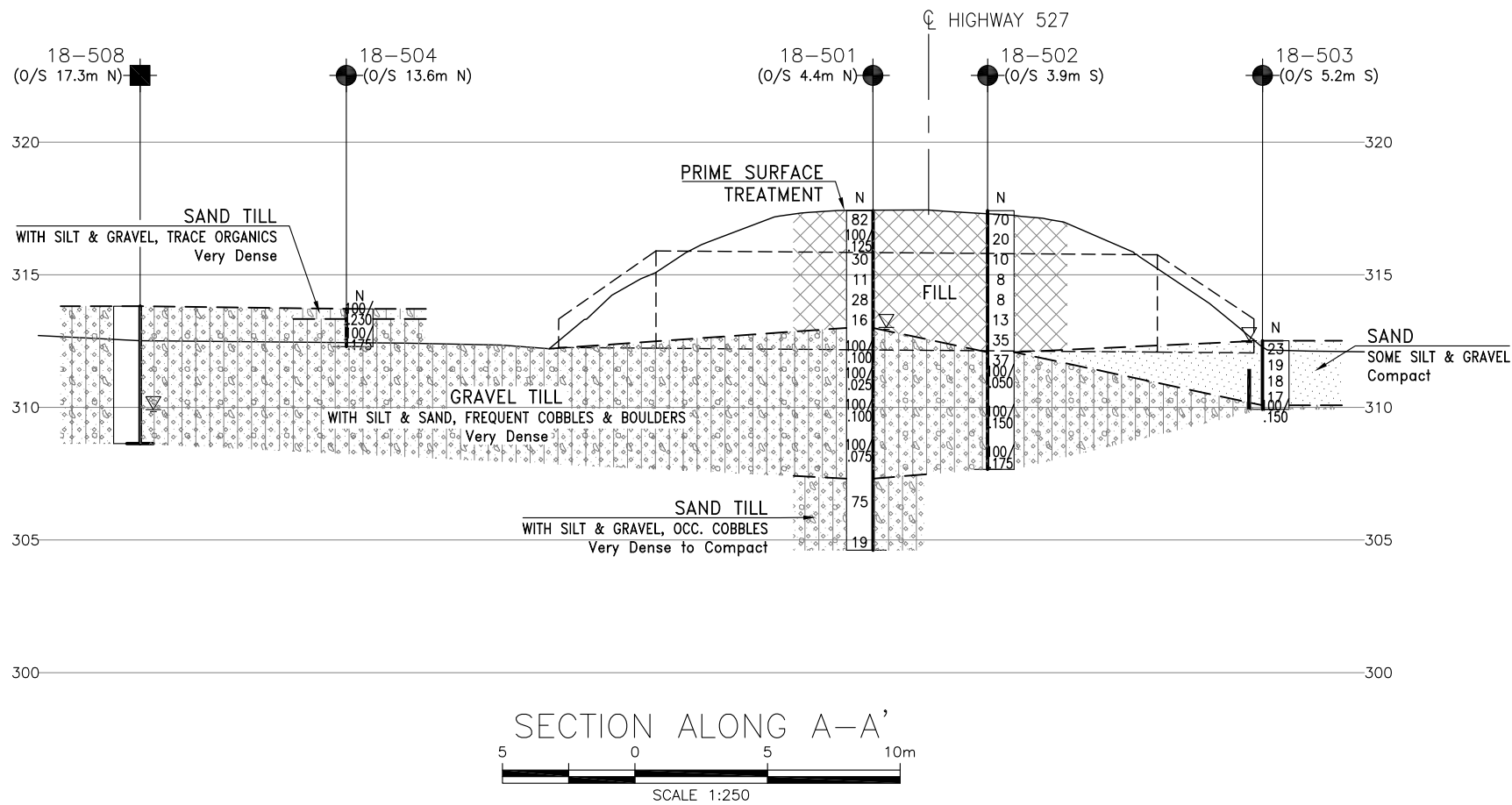
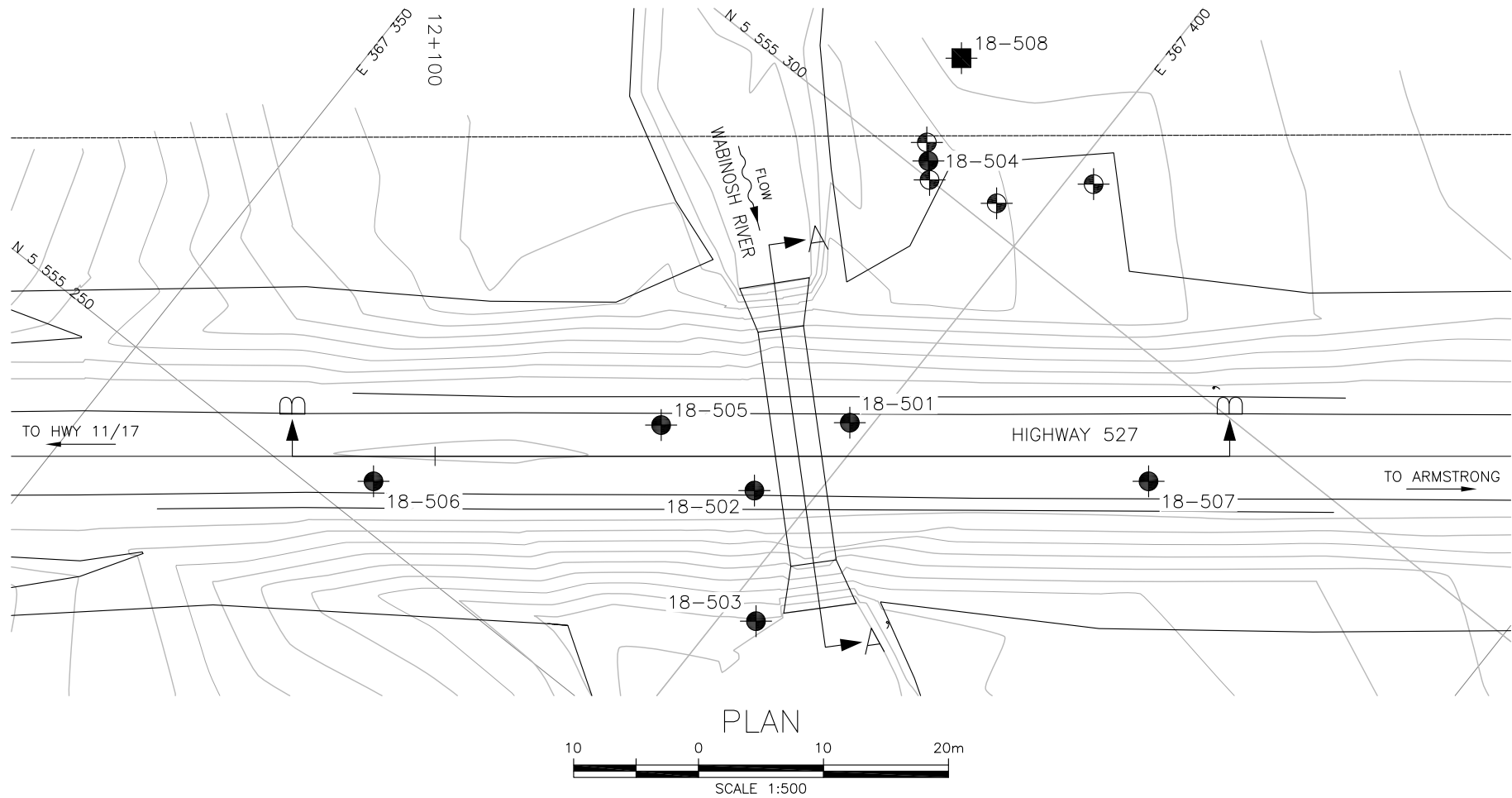
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DRAFT

Appendix A.
Drawings

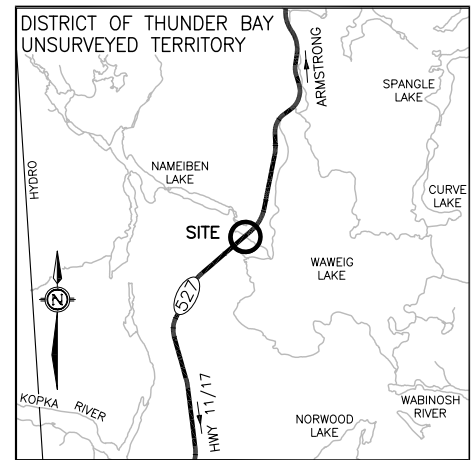


METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT No
GWP No 6829-14-00

HIGHWAY 527
WABINOSH RIVER CULVERT
REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA

HATCH



KEYPLAN

LEGEND

●/■	Borehole / Test Pit
⊕	Failed Borehole Attempt
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

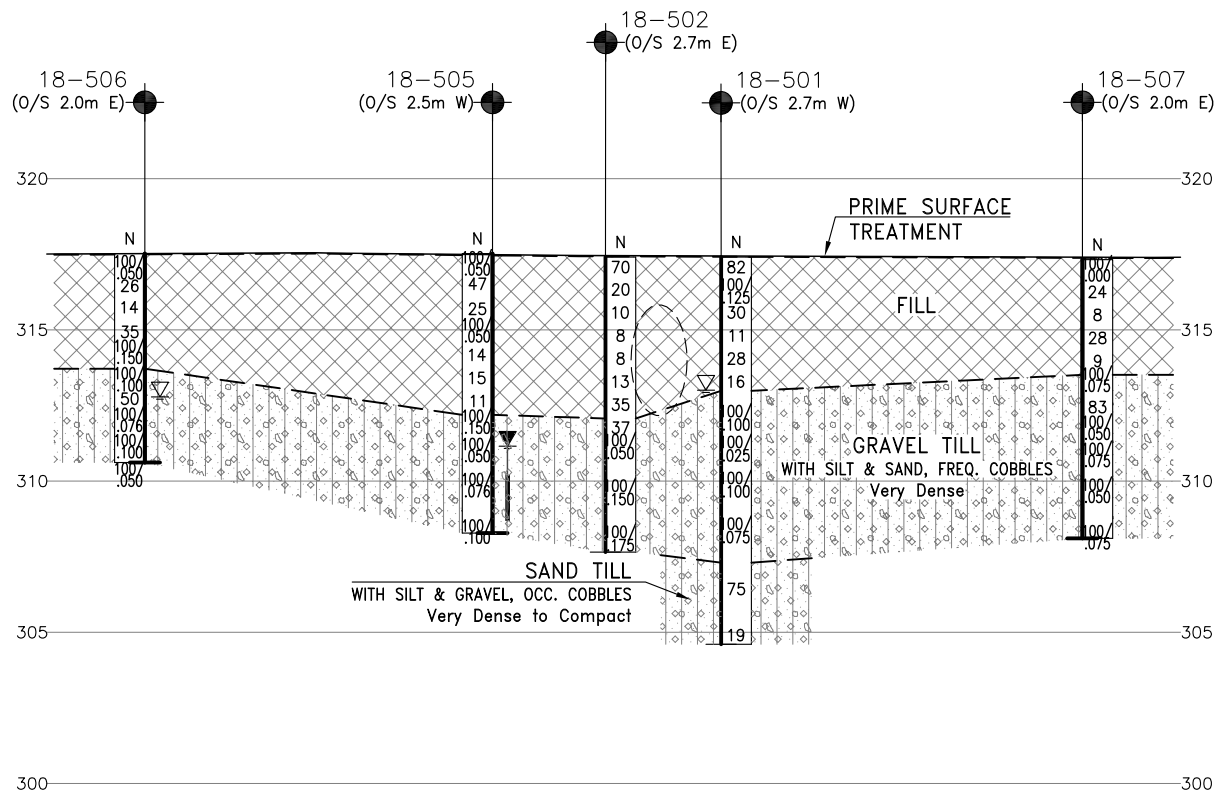
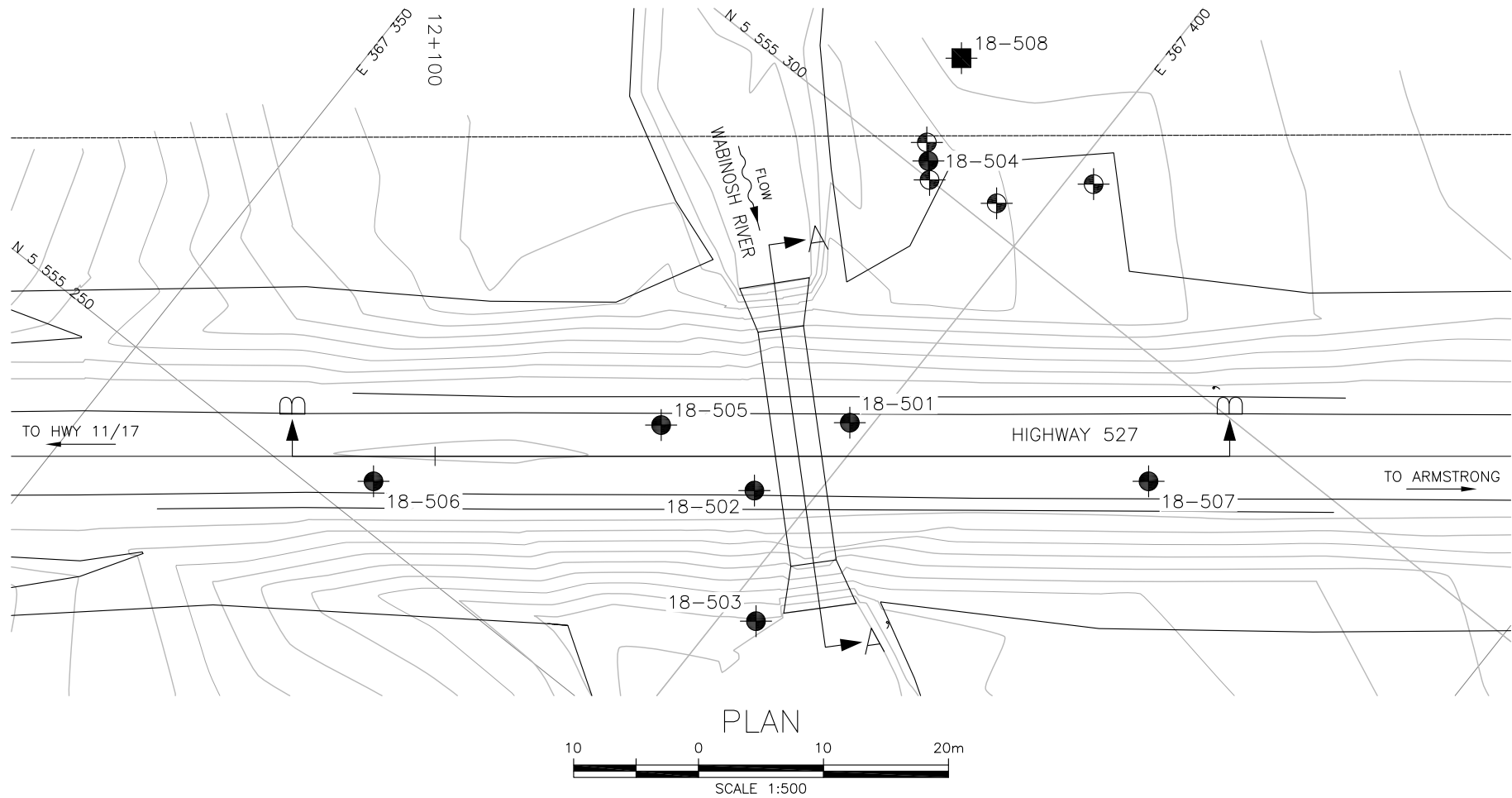
NO	ELEVATION	NORTHING	EASTING
18-501	317.4	5 555 280.8	367 398.5
18-502	317.4	5 555 271.8	367 395.9
18-503	312.5	5 555 263.7	367 402.5
18-504	313.7	5 555 301.1	367 390.4
18-505	317.5	5 555 271.3	367 386.8
18-506	317.5	5 555 253.4	367 371.6
18-507	317.3	5 555 292.0	367 420.1
18-508	313.8	5 555 309.2	367 387.4

-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 15.

GEOCRES No. 52I-02

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	DP	CHK SP	CODE
DRAWN	AN	CHK CM	SITE
			LOAD
			STRUCT
			DWG 1
			DATE DEC 2018

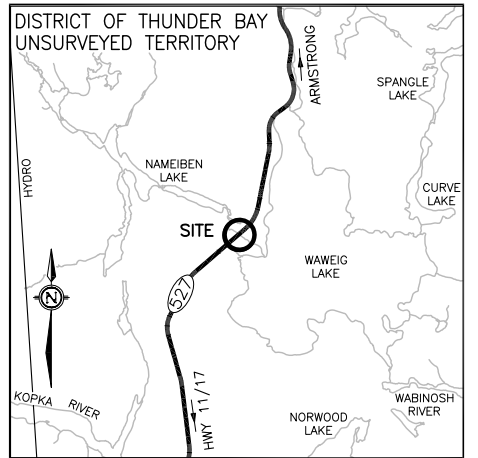


METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT No
GWP No 6829-14-00

HIGHWAY 527
WABINOSH RIVER CULVERT
REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA

HATCH



KEYPLAN

LEGEND

●/■	Borehole / Test Pit
⊕	Failed Borehole Attempt
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

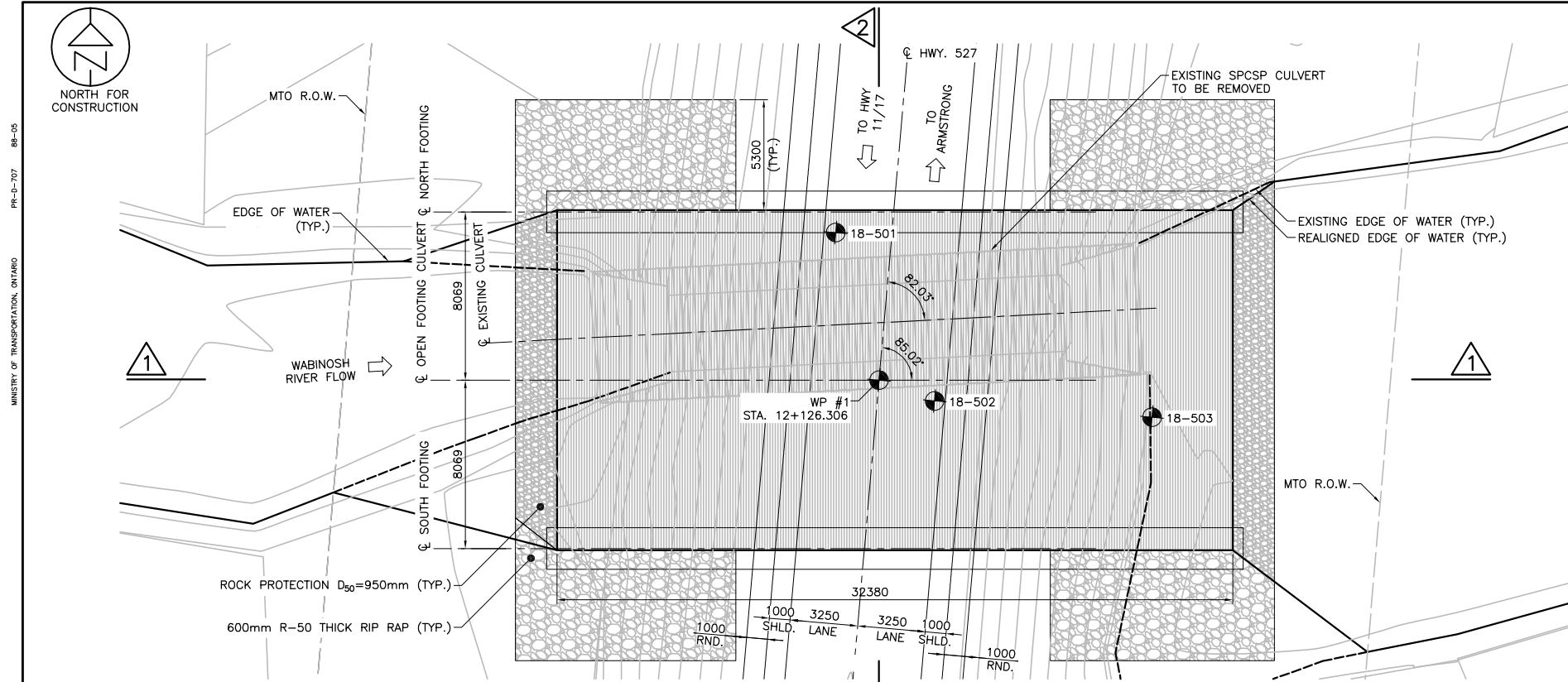
NO	ELEVATION	NORTHING	EASTING
18-501	317.4	5 555 280.8	367 398.5
18-502	317.4	5 555 271.8	367 395.9
18-503	312.5	5 555 263.7	367 402.5
18-504	313.7	5 555 301.1	367 390.4
18-505	317.5	5 555 271.3	367 386.8
18-506	317.5	5 555 253.4	367 371.6
18-507	317.3	5 555 292.0	367 420.1
18-508	313.8	5 555 309.2	367 387.4

-NOTES-

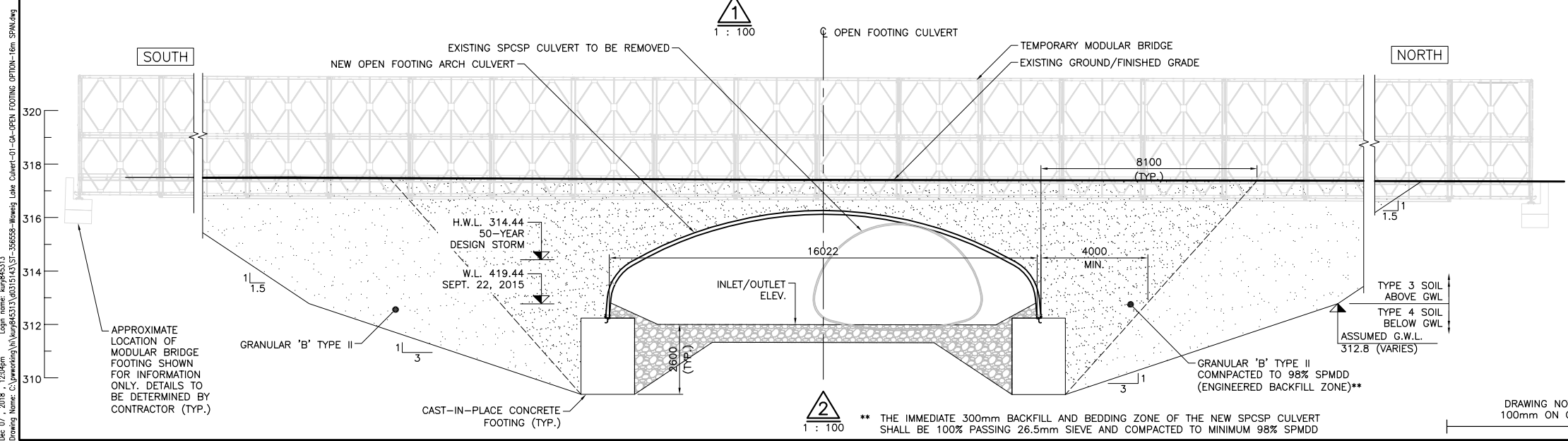
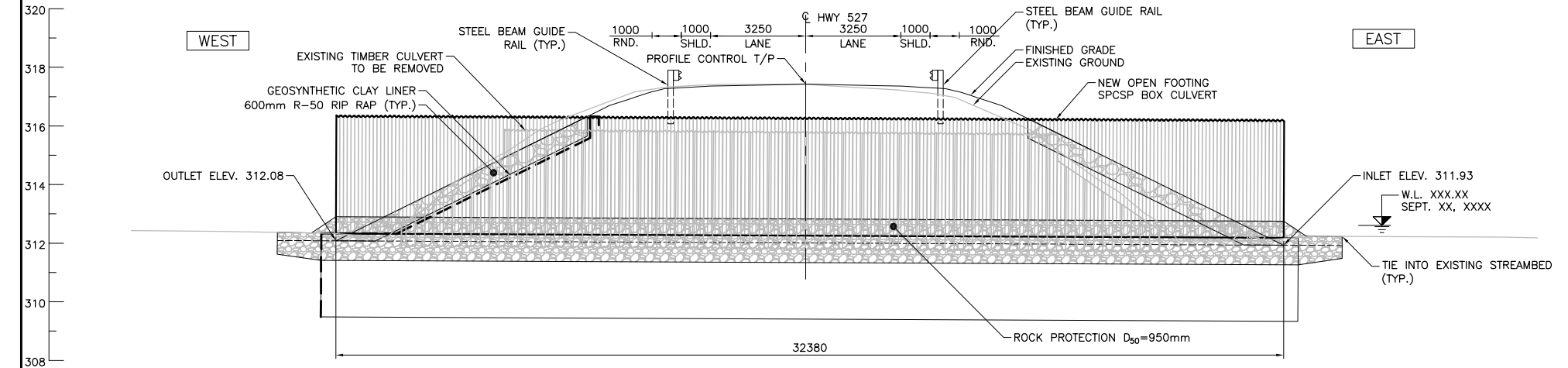
- The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- This drawing is for subsurface information only. Surface details and features are for conceptual illustration.
- Coordinate system is MTM NAD 83 Zone 15.

GEOCRES No. 52I-02

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	DP	CHK SP	CODE
DRAWN	MFA	CHK CM	SITE
			LOAD
			STRUCT
			DWG 2
			DATE DEC 2018



PLAN
1 : 100



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

WP	NORTHING	EASTING
#1	5 555 274.415	367 394.791

CONT No.
WP No. 6830-14-01

WABINOSH RIVER (WAVEIG LAKE)
CULVERT REPLACEMENT
GENERAL ARRANGEMENT
OPEN FOOTING OPTION

SHEET
XX

HATCH

PRELIMINARY
NOT FOR CONSTRUCTION

DATE	REV.	DESCRIPTION
DESIGN	AK	CHK
DRAWN	CHK	
CODE CAN/CSA S6-14	LOAD CL-625-ONT	DATE DEC. 2018
SITE 48C-124/C		DWG 1

Dec 07 - 2018 - 12:04pm Login name: kury45313
Drawing Name: C:\working\h\kury45313\60315143\ST-356556-Waveig Lake Culvert-01-GA-OPEN FOOTING OPTION-16m SPAN.dwg

** THE IMMEDIATE 300mm BACKFILL AND BEDDING ZONE OF THE NEW SPCSP CULVERT SHALL BE 100% PASSING 26.5mm SIEVE AND COMPACTED TO MINIMUM 98% SPMD

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

Appendix B.
Record of Borehole Sheets



SYMBOLS, ABBREVIATIONS AND TERMS USED ON TEST HOLE RECORDS

TERMINOLOGY DESCRIBING COMMON SOIL GENESIS

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

TERMINOLOGY DESCRIBING SOIL STRUCTURE:

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

RECOVERY:

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

N-VALUE:

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

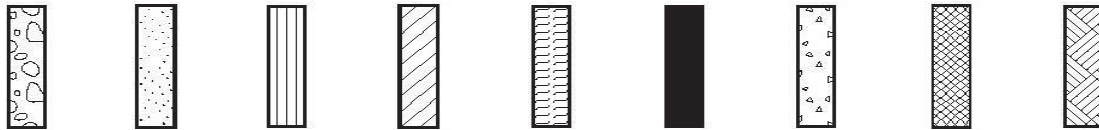
DYNAMIC CONE PENETRATION TEST (DCPT):

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



STRATA PLOT:

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



Boulders
Cobbles
Gravel Sand Silt Clay Organics Asphalt Concrete Fill Bedrock

TEXTURING CLASSIFICATION OF SOILS

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

TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

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

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

SAMPLE TYPES

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

TERMS DESCRIBING CONSISTENCY (COHESIONLESS SOILS ONLY)

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

MODIFIED UNIFIED SOIL CLASSIFICATION

Major Divisions		Group Symbol	Typical Description
COARSE GRAINED SOIL	GRAVEL AND GRAVELLY SOILS	GW	Well-graded gravels or gravel-sand mixtures, little or no fines.
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines.
		GM	Silty gravels, gravel-sand-silt mixtures.
		GC	Clayey gravels, gravel-sand-clay mixtures.
	SAND AND SANDY SOILS	SW	Well-graded sands or gravelly sands, little or no fines.
		SP	Poorly-graded sands or gravelly sands, little or no fines.
		SM	Silty sands, sand-silt mixtures.
		SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS	SILT AND CLAY SOILS $W_L < 35\%$	ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
		OL	Organic silts and organic silty-clays of low plasticity.
	SILT AND CLAY SOILS $35\% < W_L < 50\%$	MI	Inorganic compressible fine sandy silt with clay of medium plasticity, clayey silts.
		CI	Inorganic clays of medium plasticity, silty clays.
		OI	Organic silty clays of medium plasticity.
	SILT AND CLAY SOILS $W_L > 50\%$	MH	Inorganic silts, micaceous or diatomaceous fine sandy 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 18-501

1 OF 2

METRIC

GWP# 6829-14-00 LOCATION Lat: 50.131459°, Long: -89.124404° Wabinoish River Culvert, MTM 215: N 5 555 280.8 E 367 398.5 ORIGINATED BY NW
 HWY 527 BOREHOLE TYPE NW Casing COMPILED BY SOB
 DATUM Geodetic DATE 2018.06.09 - 2018.06.10 CHECKED BY SD

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							
								UNCONFINED							
								FIELD VANE							
317.4						20	40	60	80	100	W P	W	W L		
0.0	20mm Prime Surface Treatment														
	SAND with Silt and Gravel, frequent Cobbles, FILL Very Dense to Compact Brown		1	SS	82										
			2	SS	100/ 125mm										
	- 90 mm and 75 mm Cobbles cored between 1.2 and 1.5 m														
	- 150 mm Cobble cored at 2.1 m		3	SS	30										
			4	SS	11										
314.4															
3.0	Silty SAND with Gravel, frequent Cobbles, FILL Compact Brown		5	SS	28										30 56 14 (SI+CL)
			6	SS	16										
313.0															
4.4	GRAVEL (GW) with Silt and Sand, frequent Cobbles and Boulders, TILL Very Dense Brown - 200 mm Boulder cored at 4.5 m														
			7	SS	100/ 100mm										59 35 6 (SI+CL)
			8	SS	100/ 25mm										
	- Several Cobbles cored between 6.3 and 7.6 m														
			9	SS	100/ 100mm										
	- 260 mm Boulder cored at 8.5 m														
			10	SS	100/ 75mm										
	- Several Cobbles cored between 9.2 and 10.1 m														

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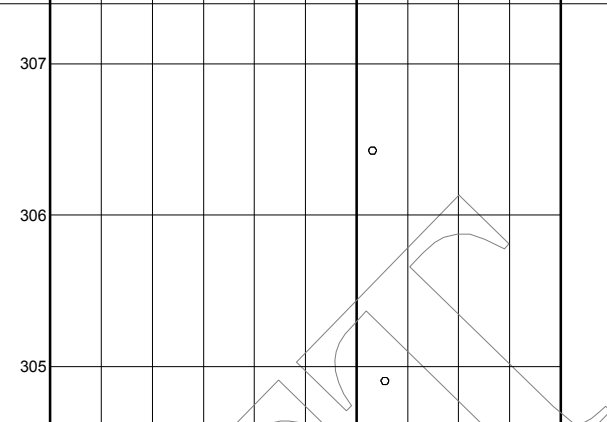
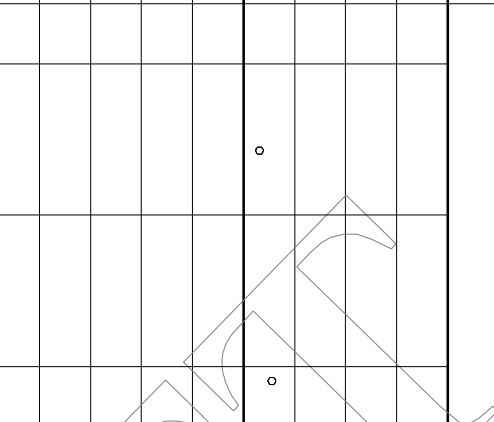
+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

METRIC

ELEV DEPTH	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			
	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES										
								20	40				60	80	100
								SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE		w P w w L ————— WATER CONTENT (%) 20 40 60					
	Continued From Previous Page							20	40	60	80	100			

307.3	10.1	SAND (SP) with Silt and Gravel, occasional Cobbles, TILL Very Dense to Compact Brown						307		44	51	5 (SI+CL)	
				11	SS	75					306		
											305		
304.6	12.8	End of Borehole - Advanced past refusal Water at 4.4 m B.G.S. (elev. 313.0 m) on completion of drilling								21	71	8 (SI+CL)	

DOUBLE LINE 19773 WABINOSH RIVER.GPJ 2012TEMPLATE(MTO).GDT 18/12/18

+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 18-502

1 OF 1

METRIC

GWP# 6829-14-00 LOCATION Lat: 50.131378°, Long: -89.124442° Wabinoish River Culvert, MTM z15: N 5 555 271.8 E 367 395.9 ORIGINATED BY NW
 HWY 527 BOREHOLE TYPE NW Casing COMPILED BY SOB
 DATUM Geodetic DATE 2018.06.12 - 2018.06.12 CHECKED BY SD

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					
317.4								20 40 60 80 100					
0.0	20mm Prime Surface Treatment												
	SAND with Silt and Gravel, frequent Cobbles, FILL Very Dense to Compact Brown - 75 mm Cobble cored at 0.6 m		1	SS	70		317						41 49 10 (SI+CL)
			2	SS	20								
			3	SS	10		316						
315.3													
2.1	GRAVEL with Sand, frequent Cobbles, FILL Loose to Dense Brown - 90 mm Cobble cored at 2.9 m		4	SS	8		315						
			5	SS	8		314						
	- 75 mm Cobble cored at 3.7 m		6	SS	13								
			7	SS	35		313						64 35 1 (SI+CL)
312.1													
5.3	- 80 mm Cobble cored at 5.2 m		8	SS	37		312						
	GRAVEL (GW) with Silt and Sand, frequent Cobbles and Boulders, TILL Dense to Very Dense Brown		9	SS	100/ 50mm		311						
			10	SS	100/ 150mm		310						
			11	SS	100/ 175mm		309						
307.6							308						
9.8	End of Borehole - Refusal												

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

RECORD OF BOREHOLE No 18-503

1 OF 1

METRIC

GWP# 6829-14-00 LOCATION Lat: 50.131304°, Long: -89.124351° Wabinoah River Culvert, MTM z15: N 5 555 263.7 E 367 402.5 ORIGINATED BY SOB
HWY 527 BOREHOLE TYPE Portable, 1/4 weight hammer COMPILED BY AC
DATUM Geodetic DATE 2018.08.09 - 2018.08.09 CHECKED BY SD

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE								WATER CONTENT (%)
312.5								20	40	60	80	100				
0.0	SAND (SP) some Silt and Gravel Compact Brown		1	SS	23		312									
			2	SS	19											
			3	SS	18		311								13	82 5 (SI+CL)
			4	SS	17											
310.1							310									
308.9	GRAVEL (GW) with Silt and Sand, frequent Cobbles and Boulders, TILL Very Dense Brown		5	SS	100/											
2.6	End of Borehole - Refusal on Boulder A 19mm diameter standpipe was installed DATE DEPTH (m) ELEV. (m) 2018.08.12 0.0 312.5 Note: A quarter-weight (16kg) drop hammer was used to advance the split-spoon sampler. The "N" values presented above have been corrected to provide an estimate of the "N" value that would have been obtained with a standard 64kg hammer				150 mm											

DOUBLE LINE 19773 WABINOSH RIVER.GPJ 2012TEMPLATE(MTO).GDT 18/12/18

RECORD OF BOREHOLE No 18-504

1 OF 1

METRIC

GWP# 6829-14-00 LOCATION Lat: 50.131642°, Long: -89.124514° Wabinoash River Culvert, MTM z15: N 5 555 301.1 E 367 390.4 ORIGINATED BY SOB
 HWY 527 BOREHOLE TYPE Portable COMPILED BY AC
 DATUM Geodetic DATE 2018.08.09 - 2018.08.10 CHECKED BY SD

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 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE							PLASTIC LIMIT W _P NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L				
313.7								20	40	60	80	100							
0.0	SAND (SP) with Silt and Gravel, trace Organics Very Dense Brown		1	SS	100/ 230 mm														
313.3			2	NQ	-														
0.4			3	SS	100/ 175 mm														
			4	NQ	175 mm														
312.3																			
1.4	End of Borehole - Casing Refusal on Boulder				-														

DOUBLE LINE 19773 WABINOSH RIVER.GPJ 2012TEMPLATE(MTO).GDT 18/12/18

RECORD OF BOREHOLE No 18-505

1 OF 2

METRIC

GWP# 6829-14-00 LOCATION Lat: 50.131374°, Long: -89.124569° Wabinoth River Culvert, MTM z15: N 5 555 271.3 E 367 386.8 ORIGINATED BY NW
HWY 527 BOREHOLE TYPE HW Casing / HQ Coring COMPILED BY SOB
DATUM Geodetic DATE 2018.11.23 - 2018.11.24 CHECKED BY SD

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				
								○ UNCONFINED + FIELD VANE				
								● QUICK TRIAXIAL × LAB VANE				
317.5						20 40 60 80 100	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	WATER CONTENT (%)		
0.0	20mm Prime Surface Treatment		1	SS	100/50mm							
	SAND with Silt and Gravel, occasional Cobbles, FILL Compact to Very Dense, Frozen near surface Brown		2	SS	47						36 52 12 (SI+CL)	
			3	SS	25							
			4	SS	100/50mm							
314.5	- 90 mm Cobble cored at 2.9 m		5	SS	14							
3.0	Silty SAND with Gravel, FILL Compact Brown		6	SS	15							
			7	SS	11							
312.2	GRAVEL (GW) with Silt and Sand, frequent Cobbles, TILL Very Dense Brown		8	SS	100/150mm							
5.3	- Several Cobbles cored between 6.1 and 7.0 m		9	SS	100/50mm							
	- 225 mm Boulder at 7.0 m											
	- Several Cobbles cored between 7.6 and 9.1 m		10	SS	100/76mm							
308.3	End of Borehole A 51mm diameter monitoring well was installed		11	SS	100/100mm							
9.2												

Continued Next Page

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

DOUBLE LINE 19773 WABINOSH RIVER.GPJ 2012TEMPLATE(MTO).GDT 18/12/18

METRIC

[illegible]

+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 18-506

1 OF 1

METRIC

GWP# 6829-14-00 LOCATION Lat: 50.131215°, Long: -89.124785° Wabinoah River Culvert, MTM z15: N 5 555 253.4 E 367 371.6 ORIGINATED BY NW
HWY 527 BOREHOLE TYPE HW Casing / HQ Coring COMPILED BY SOB
DATUM Geodetic DATE 2018.11.24 - 2018.11.25 CHECKED BY SD

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					
317.5	20mm Prime Surface Treatment GRAVEL with Sand, occasional Cobbles, FILL Compact to Very Dense, Frozen near surface Brown - Several Cobbles cored between 2.3 and 3.0 m - 100 mm Cobble cored at 3.2 m		1	SS	100/ 50mm		20 40 60 80 100				58 38 4 (SI+CL)		
0.0			2	SS	26		20 40 60						
			3	SS	14		20 40 60						
			4	SS	35		20 40 60						
			5	SS	100/ 150mm		20 40 60						
313.7	GRAVEL (GW) with Silt and Sand, frequent Cobbles, TILL Very Dense Brown - Several Cobbles cored between 3.8 and 4.6 m - 100 mm Cobble cored at 5.0 m - Several Cobbles cored between 5.3 and 6.9 m		6	SS	100/ 100mm		313	20 40 60				59 34 7 (SI+CL)	
3.8			7	SS	50			20 40 60					
			8	SS	100/ 76mm			20 40 60					
			9	SS	100/ 100mm			20 40 60					
			10	SS	100/ 50mm			20 40 60					
310.6	End of Borehole Water at 4.7 m B.G.S. (elev. 312.8 m) on completion of drilling												
6.9													

+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

METRIC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		NATURAL MOISTURE CONTENT	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	SHEAR STRENGTH kPa	W P W L		PLASTIC LIMIT
317.3 0.8	20mm Prime Surface Treatment SAND with Silt and Gravel, occasional Cobbles, FILL Loose to Compact, Frozen near surface Brown - Several Cobbles cored between 0.2 and 1.2 m - 90 mm Cobble cored at 2.1 m - 70 mm Cobble cored at 2.9 m		1	SS	100/ 0mm		317					
			2	SS	24		316					
			3	SS	8		315					
			4	SS	28		314					
			5	SS	9		313					
313.5 3.8	GRAVEL (GW) with Silt and Sand, frequent Cobbles, TILL Very Dense Brown - Several Cobbles cored between 3.8 and 5.2 m - 175 mm Cobble cored at 6.0 m - Several Cobbles cored between 6.1 and 9.1 m		6	SS	100/ 75mm		312					
			7	SS	83		311					
			8	SS	100/ 50mm		310					
			9	SS	100/ 75mm		309					
			10	SS	100/ 50mm							
308.1 9.2	End of Borehole		11	SS	100/ 75mm							

DOUBLE LINE 19773 WABINOSH RIVER.GPJ 2012TEMPLATE(MTO).GDT 18/12/18

+³, ×³: Numbers refer to Sensitivity

RECORD OF TEST PIT No 18-508

1 OF 1

METRIC

GWP# 6829-14-00 LOCATION Lat: 50.131715°, Long: -89.124555° Wabinoah River Culvert, MTM z15: N 5 555 309.2 E 367 387.4 ORIGINATED BY NW
 HWY 527 BOREHOLE TYPE Test Pit / Volvo ECR305CL Excavator COMPILED BY CM
 DATUM Geodetic DATE 2018.11.29 - 2018.11.29 CHECKED BY SD

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE							
313.8															
0.0	GRAVEL (GW) with Sand trace Silt, frequent Cobbles and Boulders, TILL Brown		1	BS	-										65 32 3 (SI+CL)
			2	BS	-										
			3	BS	-										
			4	BS	-										71 26 3 (SI+CL)
			5	BS	-										
			6	BS	-										
			7	BS	-										73 24 3 (SI+CL)
308.6	End of Test Pit Water at 3.9 m B.G.S. (elev. 309.9 m) on completion of digging														
5.2															

DOUBLE LINE 19773 WABINOSH RIVER.GPJ 2012TEMPLATE(MTO).GDT 18/12/18

Appendix C.
Laboratory Testing

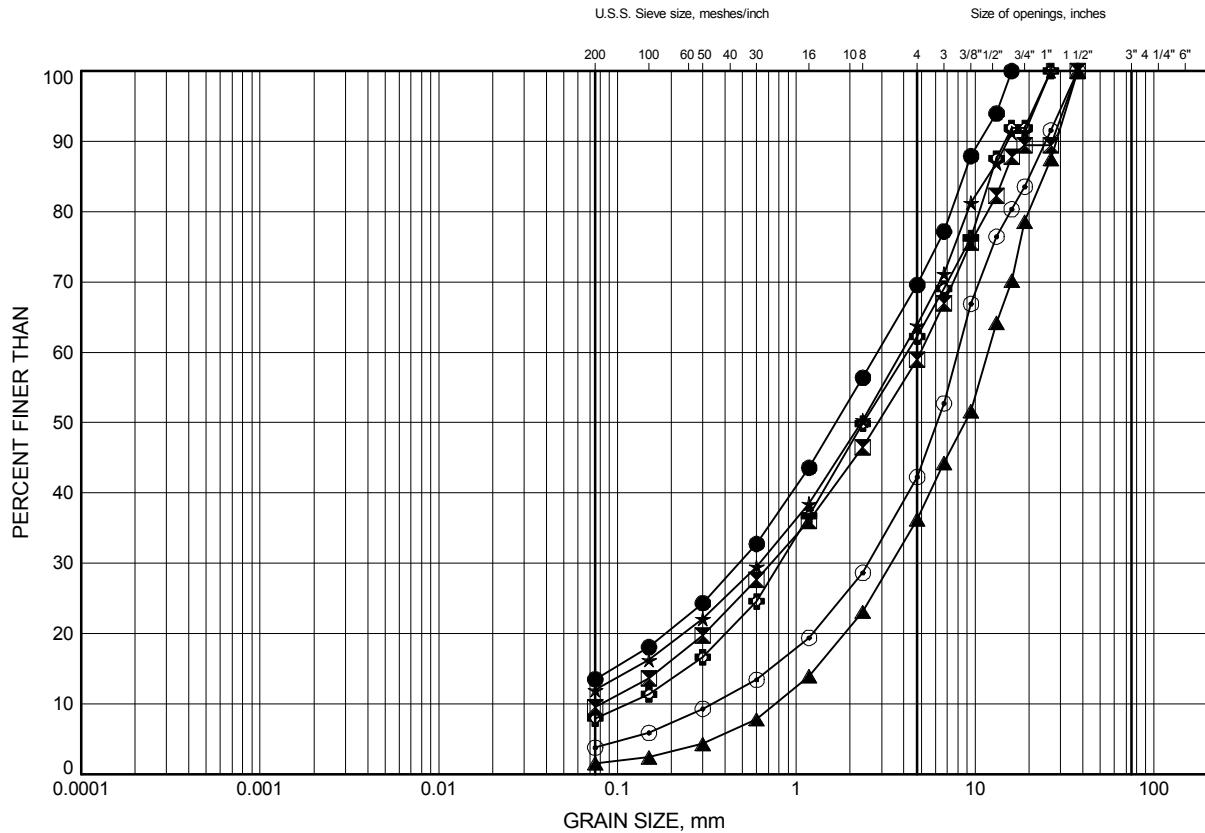
Appendix C.1

Particle Size Analysis Figures

Wabinoash River Culvert GRAIN SIZE DISTRIBUTION

FIGURE C1

Embankment Fill



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	18-501	3.35	314.05
⊠	18-502	0.33	317.07
▲	18-502	4.11	313.29
★	18-505	1.00	316.50
⊙	18-506	1.07	316.43
⊕	18-507	2.59	314.71

Date December 2018

GWP# 6829-14-00



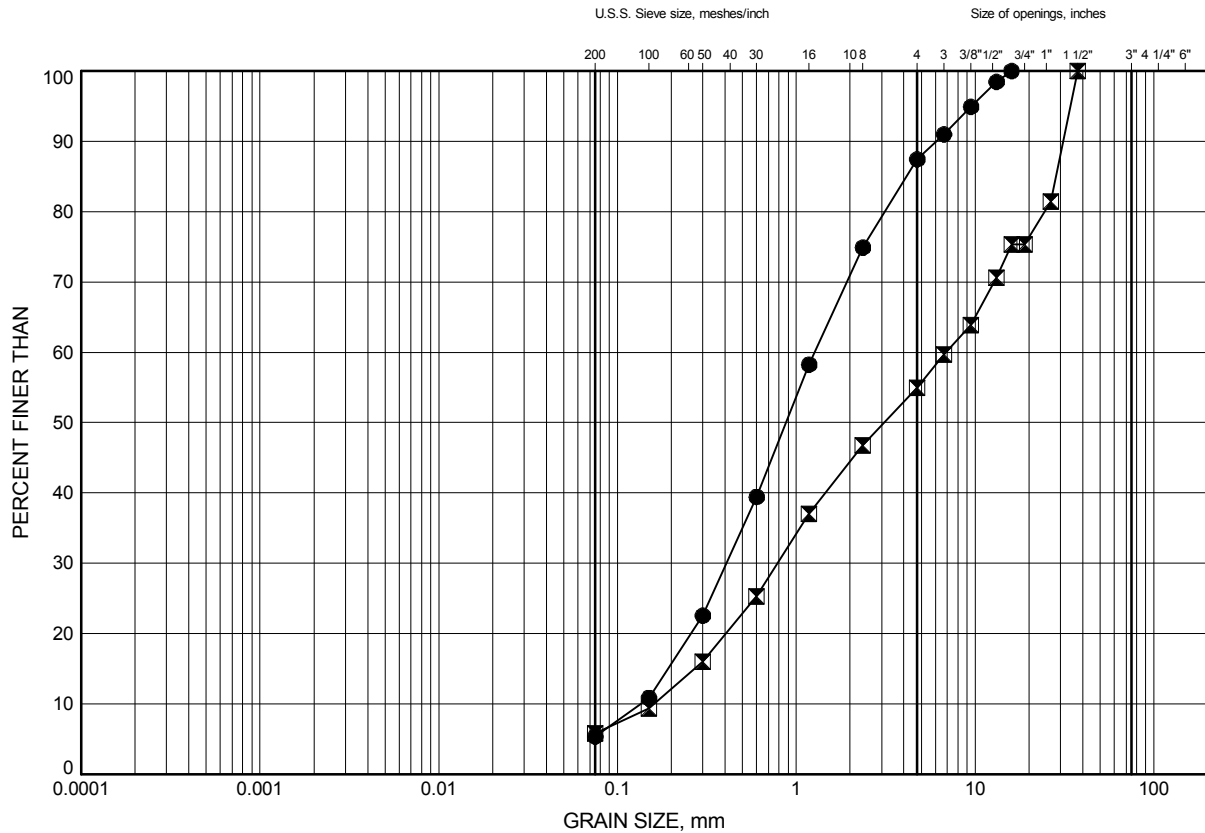
Prep'd CM

Chkd. SD

Wabinoash River Culvert GRAIN SIZE DISTRIBUTION

FIGURE C2

Surficial Sand



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	18-503	1.52	310.98
⊠	18-504	0.19	313.51

Date December 2018

GWP# 6829-14-00



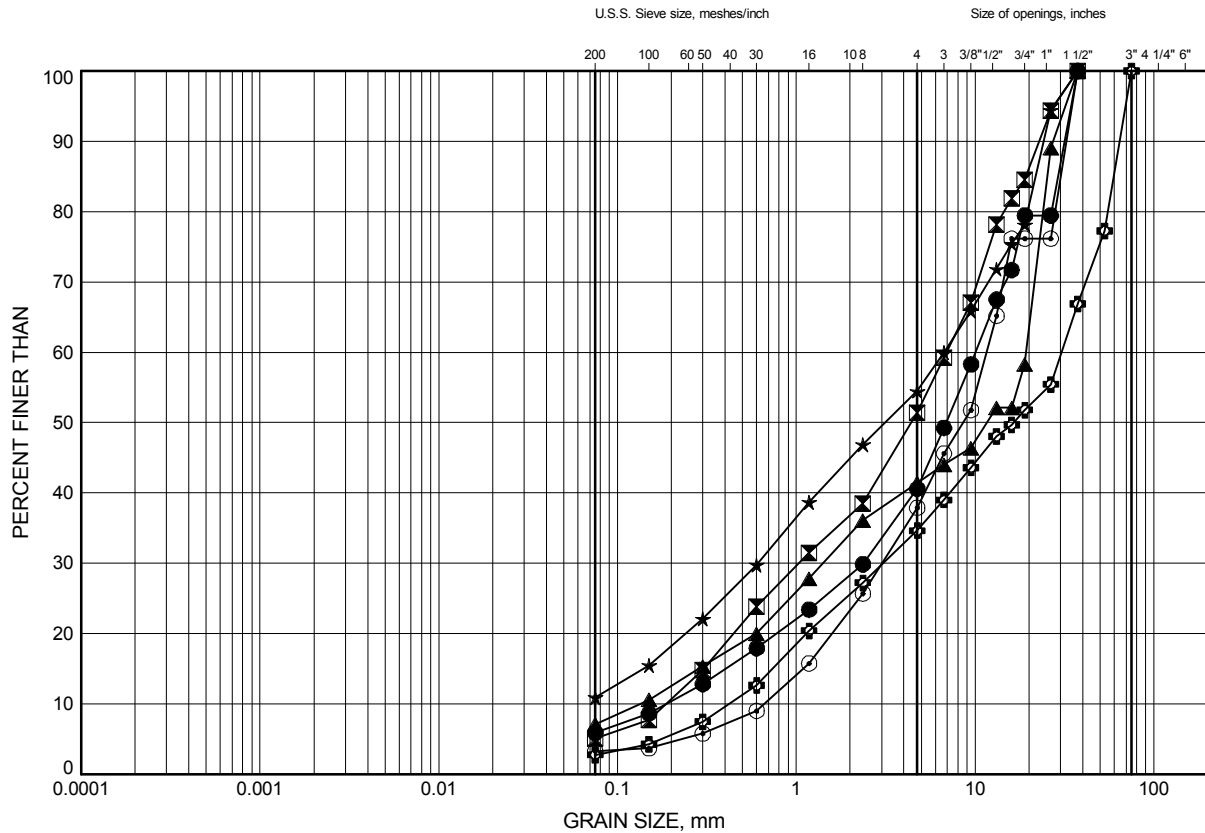
Prep'd CM

Chkd. SD

Wabinoash River Culvert GRAIN SIZE DISTRIBUTION

FIGURE C3

Gravel Till



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	18-501	5.46	311.94
⊠	18-504	1.17	312.53
▲	18-506	4.80	312.70
★	18-507	4.88	312.42
⊙	18-507	9.18	308.12
⊕	18-508	0.38	313.42

Date December 2018

GWP# 6829-14-00



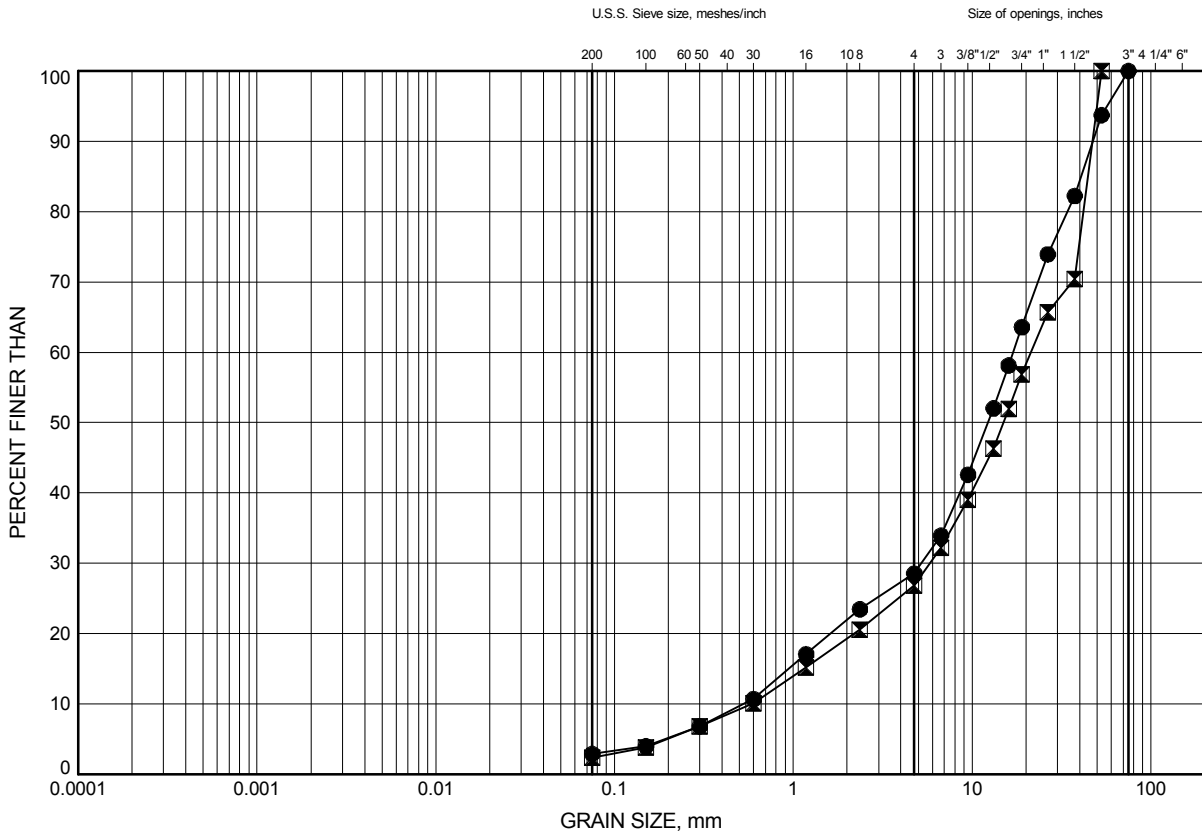
Prep'd CM

Chkd. SD

Wabinoash River Culvert GRAIN SIZE DISTRIBUTION

FIGURE C4

Gravel Till



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	18-508	3.20	310.60
⊠	18-508	5.03	308.77

Date December 2018
GWP# 6829-14-00

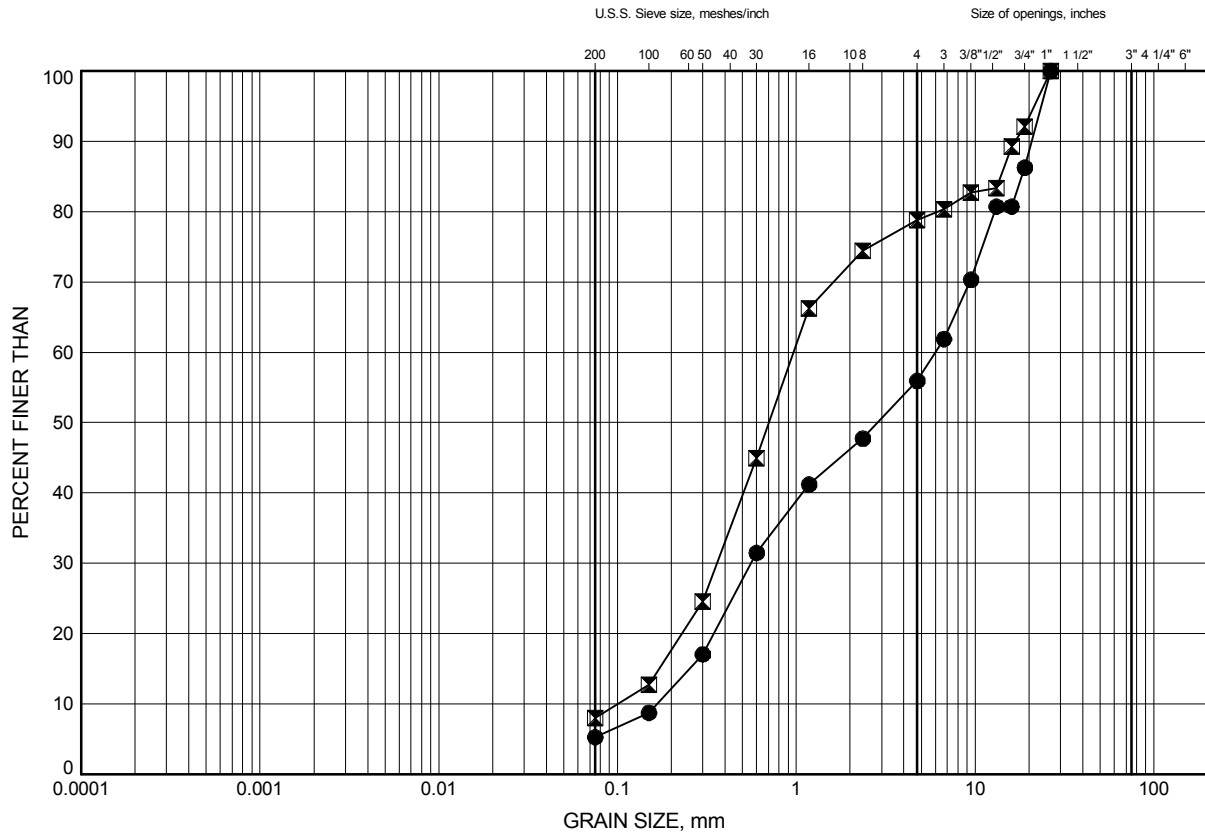


Prep'd CM
Chkd. SD

Wabinoash River Culvert GRAIN SIZE DISTRIBUTION

FIGURE C5

Sand Till



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	18-501	10.97	306.43
⊠	18-501	12.50	304.90

Date December 2018

GWP# 6829-14-00



Prep'd CM

Chkd. SD

Appendix C.2
Analytical Testing Results

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

Report Date: 26-Jun-2018

Order Date: 20-Jun-2018

Project Description: 19773

Client ID:		18-101, SS6, 12'6"-14'6"	18-203, SS3, 5'10"-7'10"	18-204, SS4, 10'4"-12'4"	18-401, SS5, 10'-12'
Sample Date:		05/30/2018 11:00	06/12/2018 14:30	06/13/2018 09:45	06/07/2018 13:30
Sample ID:		1825441-01	1825441-02	1825441-03	1825441-04
MDL/Units		Soil	Soil	Soil	Soil
Physical Characteristics					
% Solids	0.1 % by Wt.	80.0	88.0	89.3	92.4
General Inorganics					
Conductivity	5 uS/cm	135	156	98	90
pH	0.05 pH Units	7.81	7.76	7.76	7.56
Resistivity	0.10 Ohm.m	74.3	64.3	102	111
Anions					
Chloride	5 ug/g dry	9	25	29	9
Sulphate	5 ug/g dry	16	46	7	28
Client ID:		18-502, SS8, 17'6"-19'6"	18-301, SS8A, 17'6"-19'4"	-	-
Sample Date:		06/12/2018 11:15	06/05/2018 15:30	-	-
Sample ID:		1825441-05	1825441-06	-	-
MDL/Units		Soil	Soil	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	89.9	90.0	-	-
General Inorganics					
Conductivity	5 uS/cm	47	50	-	-
pH	0.05 pH Units	7.14	7.38	-	-
Resistivity	0.10 Ohm.m	213	198	-	-
Anions					
Chloride	5 ug/g dry	13	19	-	-
Sulphate	5 ug/g dry	10	6	-	-

**SGS Canada Inc.**

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

Paracel Laboratories

Attn : Dale Robertson

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

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

28-June-2018

Date Rec. : 22 June 2018
LR Report: CA12773-JUN18
Reference: Project#:1825441

Copy: #1

CERTIFICATE OF ANALYSIS

Final Report

Sample ID	Sample Date & Time	Sulphide %
1: Analysis Start Date		28-Jun-18
2: Analysis Start Time		13:23
3: Analysis Completed Date		28-Jun-18
4: Analysis Completed Time		14:45
5: QC - Blank		< 0.02
6: QC - STD % Recovery		105%
7: QC - DUP % RPD		ND
8: RL		0.02
9: 18-101,SS6, 12'6"-14'16"	30-May-18	< 0.02
10: 18-204,SS4, 10'4"-12'4"	13-Jun-18	< 0.02
11: 18-401,SS5, 10'-12'	07-Jun-18	< 0.02
12: 18-502,SS8, 17'6"-19'6"	12-Jun-18	< 0.02
13: 18-301,SS8A, 17'6"-19'4"	05-Jun-18	< 0.02

RL - SGS Reporting Limit
ND - Not Detected

Kimberley Didsbury
Project Specialist
Environmental Services, Analytical

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

Report Date: 17-Aug-2018

Order Date: 13-Aug-2018

Project Description: 19773

Client ID:	18-503, SS2, 2' - 4'	-	-	-
Sample Date:	08/09/2018 09:00	-	-	-
Sample ID:	1833093-01	-	-	-
MDL/Units	Soil	-	-	-

Physical Characteristics

% Solids	0.1 % by Wt.	88.6	-	-	-
----------	--------------	------	---	---	---

General Inorganics

Conductivity	5 uS/cm	60	-	-	-
pH	0.05 pH Units	7.42	-	-	-
Resistivity	0.10 Ohm.m	166	-	-	-

Anions

Chloride	5 ug/g dry	6	-	-	-
Sulphate	5 ug/g dry	<5	-	-	-

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

Report Date: 07-Dec-2018

Order Date: 3-Dec-2018

Project Description: 19773

Client ID:	Rousseau	Max	Rinker	Wabikon
Sample Date:	11/30/2018 12:00	11/30/2018 11:45	11/30/2018 11:30	11/30/2018 11:00
Sample ID:	1849062-01	1849062-02	1849062-03	1849062-04
MDL/Units	Water	Water	Water	Water

General Inorganics

Conductivity	5 uS/cm	125	74	52	84
pH	0.1 pH Units	7.5	7.4	7.3	7.5
Resistivity	0.01 Ohm.m	79.9	135	193	119

Anions

Chloride	1 mg/L	7	2	1	4
Sulphate	1 mg/L	1	1	1	1

Client ID:	Waweig	-	-	-
Sample Date:	11/30/2018 10:00	-	-	-
Sample ID:	1849062-05	-	-	-
MDL/Units	Water	-	-	-

General Inorganics

Conductivity	5 uS/cm	56	-	-	-
pH	0.1 pH Units	7.4	-	-	-
Resistivity	0.01 Ohm.m	180	-	-	-

Anions

Chloride	1 mg/L	1	-	-	-
Sulphate	1 mg/L	<1	-	-	-

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

Report Date: 04-Dec-2018

Order Date: 3-Dec-2018

Project Description: 19773

Client ID:	Wawieg 18-505	-	-	-
Sample Date:	11/30/2018 10:00	-	-	-
Sample ID:	1849049-01	-	-	-
MDL/Units	Water	-	-	-

Microbiological Parameters

E. coli	1 CFU/100 mL	<10 [1]	-	-	-
Total Coliforms	1 CFU/100 mL	<10 [1]	-	-	-

General Inorganics

Turbidity	0.1 NTU	10.7 [2]	-	-	-
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Certificate of Analysis
Client: Thurber Engineering Ltd.
Client PO:

Report Date: 07-Dec-2018

Order Date: 3-Dec-2018

Project Description: 19773

Client ID: Wawieg 18-505
Sample Date: 11/30/2018 10:00
Sample ID: 1849050-01
MDL/Units: Water

-	-	-
-	-	-
-	-	-
-	-	-

General Inorganics

Alkalinity, total	5 mg/L	28	-	-	-
Ammonia as N	0.01 mg/L	0.03	-	-	-
pH	0.1 pH Units	7.1	-	-	-
Phosphorus, total	0.01 mg/L	0.05	-	-	-
Total Suspended Solids	2 mg/L	39	-	-	-

Metals

Aluminum	10 ug/L	67	-	-	-
Antimony	1 ug/L	<1	-	-	-
Arsenic	10 ug/L	<10 [1]	-	-	-
Barium	10 ug/L	<10	-	-	-
Beryllium	1 ug/L	<1	-	-	-
Boron	50 ug/L	<50	-	-	-
Cadmium	1 ug/L	<1	-	-	-
Calcium	200 ug/L	8710	-	-	-
Chromium	50 ug/L	<50	-	-	-
Cobalt	1 ug/L	2	-	-	-
Copper	5 ug/L	9	-	-	-
Iron	200 ug/L	<200	-	-	-
Lead	1 ug/L	<1	-	-	-
Magnesium	200 ug/L	2330	-	-	-
Manganese	50 ug/L	<50	-	-	-
Molybdenum	5 ug/L	<5	-	-	-
Nickel	5 ug/L	<5	-	-	-
Potassium	200 ug/L	5100	-	-	-
Selenium	5 ug/L	<5	-	-	-
Silver	1 ug/L	<1	-	-	-
Sodium	200 ug/L	5210	-	-	-
Thallium	1 ug/L	<1	-	-	-
Tin	10 ug/L	<10	-	-	-
Vanadium	1 ug/L	1	-	-	-
Zinc	20 ug/L	81	-	-	-

Appendix D.
Site Photographs



Photo 1. Looking east toward culvert inlet (2018/08/10)



Photo 2. Looking northwest toward culvert outlet (2018/08/10)



Photo 3. Looking north along Highway 527 overtop of culvert (2018/08/10)



Photo 4. Looking south along Highway 527 overtop of culvert (2018/08/10)

Appendix E.
Test Pit Photographs



Photo 1. Open test pit to about 2.5 m depth (2018/11/29)



Photo 2. Typical cobble and boulder sizes (2018/11/29)



Photo 3. Open test pit to about 3.5 m depth (2018/11/29)



Photo 4. Open test pit to about 3.9 m depth (2018/11/29)



Photo 5. Excavating below water level, looking at west side wall (2018/11/29)



Photo 6. Excavating below water level, looking at east side wall (2018/11/29)

Appendix F.

Results of Hydraulic Conductivity Testing and Test Pit Pumping Test

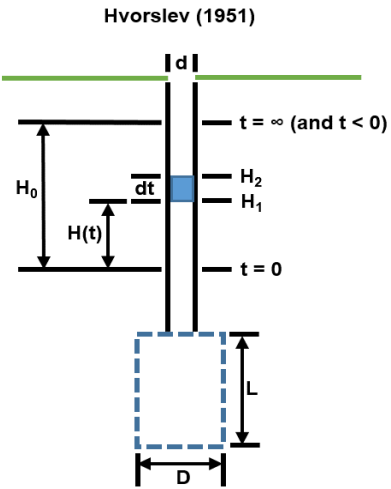
In-Situ Hydraulic Conductivity Test
Hvorslev Analysis
Method based on NAFAC Soil Mechanics Design Manual 7.01

INPUT DATA	Rising Head Test
Borehole BH18-505	
Static Water Level	6.4 mbgs
Well Diameter (d)	0.051 m
Borehole Diameter (D)	0.096 m
Length of Intake (L)	1.52 m
Initial Unbalanced Head (H ₀)	0.15 m
Shape Factor (F)	2.76

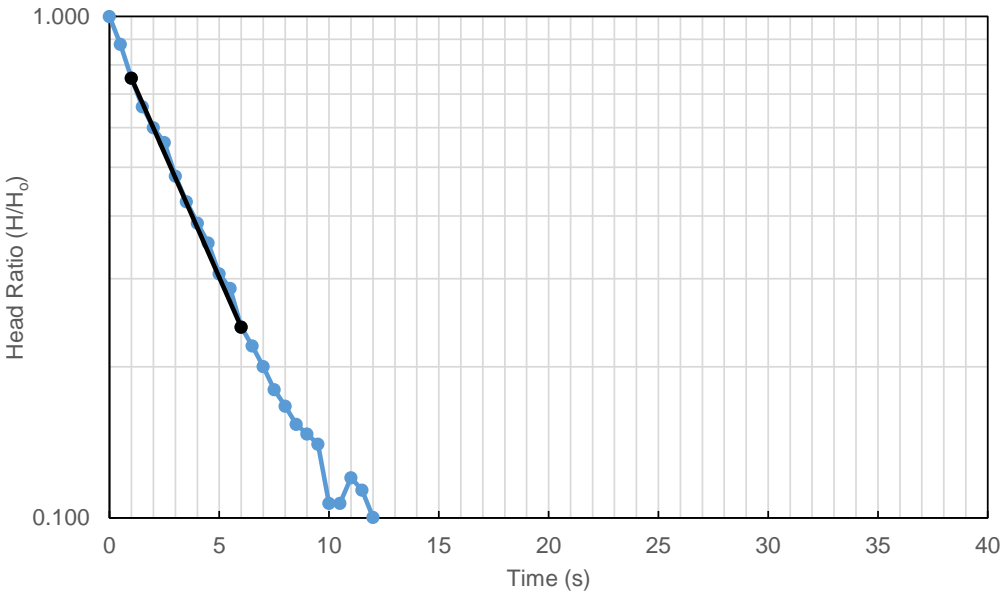
$$K = \frac{A}{F(t_2 - t_1)} \ln \left(\frac{H_1}{H_2} \right)$$

For piezometers of perforated extension of length "L"

$$F = \frac{2\pi L}{\ln \left(\frac{L}{R} \right)}$$



K = 1.7E-02 cm/s



DATE: Nov 28, 2018

PREPARED: YC

PROJECT: 19773

CHECKED: DH



Pumping Test Data Sheet

Project No: 19773

Location: Wabinoash River Culvert

Test Pit No: 18-508

Dimensions: About 3.7 m wide, 4.1 m long, and 1.14 m deep (water filled portion of Test Pit)

Equipment: 76.2 mm (3 in) diameter submersible trash pump

Pumping rate: 202 L/min (53.4 gal/min)

	Time Elapsed (min)	Height of Water (m)
<i>Start of pumping test</i>	0	1.14
	1.0	1.11
	2.0	1.10
	3.0	1.09
	4.0	1.08
	5.0	1.07
	14.0	0.97
	24.0	0.94
	30.0	0.91
	40.0	0.89
<i>Start of recovery test</i>	0.0	0.89
	1.0	0.90
	2.0	0.91
	3.0	0.93
	4.0	0.94
	5.0	0.97
	8.0	0.99
	14.0	1.04

Appendix G.

Foundation Comparison

GEOTECHNICAL COMPARISON OF ALTERNATIVE FOUNDATION TYPES

Type	Circular Pipe Culvert	Closed Box Culvert	Open Bottom Culvert	Precast Concrete Slab on Sheet Pile Culvert
Advantages	<ul style="list-style-type: none"> • Can tolerate larger magnitude of settlement than concrete (rigid frame) culverts. • Relatively expedient installation. 	<ul style="list-style-type: none"> • Relatively expedient installation if precast units are used. • Typically smaller magnitude of settlement than open footing foundation due to lower bearing stress on subgrade. • Minimized differential settlement between culvert and approach fills. 	<ul style="list-style-type: none"> • Limits disturbance to streambed. Typically favourable from an aquatic habitat perspective. • Relatively expedient installation if precast units are used. • Likely will not require offline diversion at this site. 	<ul style="list-style-type: none"> • Minimized volume of excavation compared to other options. • Allows for winter construction. • Eliminates the need for an offline diversion channel.
Disadvantages	<ul style="list-style-type: none"> • Feasibility also depends on flow capacity and other hydraulic properties. May need multiple pipes. • Requires large excavation. • Roadway protection or temporary widening will be required. • Groundwater control is required. 	<ul style="list-style-type: none"> • Requires large excavation. • Roadway protection or temporary widening will be required. • Groundwater control is required. 	<ul style="list-style-type: none"> • Requires deeper excavation for frost protection increasing excavation volume and dewatering efforts. • Potential for post construction settlement. • Roadway protection or temporary widening will be required. 	<ul style="list-style-type: none"> • Quantity and cost of sheet piles. • Cannot penetrate obstructions. • Differential settlement will occur between non-yielding culvert and approach fills. A geogrid may be needed to strengthen the pavement structure.
Risks/Consequences	<ul style="list-style-type: none"> • Cobbles and boulders present a challenge for installing roadway protection. Pre-drilling may be required to advance soldier piles. 	<ul style="list-style-type: none"> • Cobbles and boulders present a challenge for installing roadway protection. Pre-drilling may be required to advance soldier piles. 	<ul style="list-style-type: none"> • Cobbles and boulders present a challenge for installing roadway protection. Pre-drilling may be required to advance soldier piles. 	<ul style="list-style-type: none"> • High risk of encountering obstructions and having inadequate lateral support due to a shallow refusal.
Relative Cost	Low	Low	Moderate	Moderate to High
Recommendation	Feasible	Feasible	Feasible	Not Feasible

Appendix H.

GSC Seismic Hazard Calculation

2015 National Building Code Seismic Hazard Calculation

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

August 21, 2018

Site: 50.1315 N, 89.1243 W User File Reference: Waweig Lake Culvert

Requested by: C.Murray, Thurber Engineering

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)
0.052	0.071	0.065	0.051	0.038	0.019	0.0082	0.0017	0.0008	0.038	0.026

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)	0.0026	0.013	0.025
Sa(0.1)	0.0043	0.020	0.036
Sa(0.2)	0.0049	0.021	0.035
Sa(0.3)	0.0044	0.018	0.029
Sa(0.5)	0.0033	0.013	0.022
Sa(1.0)	0.0014	0.0063	0.011
Sa(2.0)	0.0006	0.0026	0.0045
Sa(5.0)	0.0002	0.0006	0.0009
Sa(10.0)	0.0002	0.0004	0.0005
PGA	0.0024	0.011	0.020
PGV	0.0016	0.0080	0.014

References

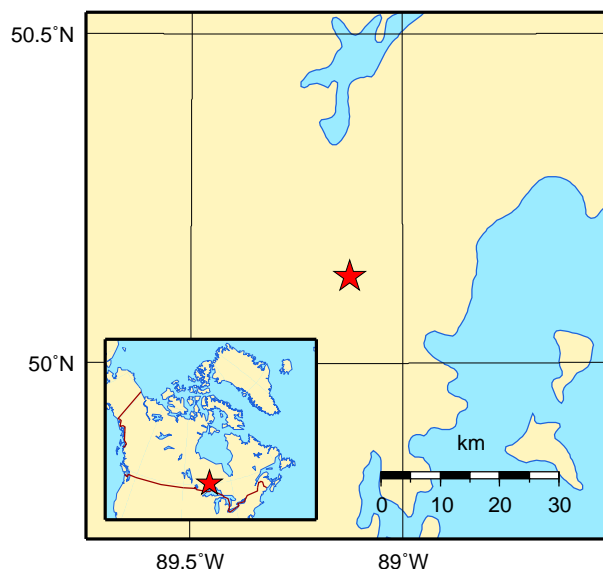
National Building Code of Canada 2015 NRCC no. 56190;
Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada

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

Canada

Appendix I.
Dewatering Estimate

Dewatering Calculations for Highway 527 Waweig Lake Culvert

Parameter	Units	Footing Depth Full Area	Limited Depth (Tremie) One Footing
Geologic Unit to Dewater		Gravel	Gravel
10% diameter (D10)	mm		
Input Hydraulic Conductivity in cm/s (K)	cm/s	1.7E-02	1.7E-02
Hydraulic Conductivity converted to m/day	m/day	14.7	14.7
Input height of groundwater pressure (H)	m	8.8	8.8
Input dewatering height (h)	m	3.4	6.6
Input length of excavation (x, a)	m	35	35
Input width of excavation (b)	m	20	10
Input/calculate radius of trench (rw or rs)	m	10.0	5.0
Length to width ratio	unitless	1.8	3.5
Net water table lowering	m	5.40	2.20
Equation Type		Trench	Trench
Apply reduction for partial aquifer penetration?	yes/no	no	no
Vertical length actively dewatered	m		
Radius of a single extraction well	m		
Radii of Influence			
Sichardt Equation (Ro based on K, H, h)	m	211.2	86.1
Ro = Sichardt + (rw or rs)	m	220	90
Calculate alternative Ro using Bear, 1979: Ro=1.5(Tt/S)^0.5			
T	m²/day	129.25	129.25
t	days	60	60
S	unitless	0.3	0.3
Alternate Ro	m	241	241
*Note: The alternate Ro is for comparison. It is not the Ro used to calculate Q.			
Calculated Flow Rate			
Base groundwater flow	L/day	1,137,000	734,000
Partial Penetration Factor	unitless	1.00	1.00
Safety factor on groundwater flow	unitless	3	3
Groundwater flow with safety factor	L/day	3,411,000	2,202,000
Rainfall entering excavation	mm	75	75
Duration to remove rainfall	hours	24	24
Flow rate to remove rainfall	L/day	53,000	26,000
Budgeted peak flow rate	L/day	3,464,000	2,228,000
=	L/s	40	26
=	gal/min	529	340
Input values for highlighted cells		Flow rate estimates rounded to nearest 1,000 L/day.	

Appendix J.

List of Special Provisions and OPSS Documents Referenced in this Report

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

OPSS.PROV 206	Construction Specification for Grading
OPSS.PROV 501	Construction Specification for Compacting
OPSS 511	Construction Specification for Rip-Rap, Rock Protection, and Granular Sheeting
OPSS.PROV 517	Construction Specification for Dewatering of Pipeline, Utility and Associated Structure Excavation
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 1010	Material Specification for Aggregates Base, Subbase, Select Subgrade, and Backfill Material
OPSS.PROV 1205	Material Specification for Clay Seal
OPSS 1860	Material Specification for Geotextiles
OPSD 208.010	Benching of Earth Slopes
OPSD 810.010	General Rip-Rap Layout for Sewer and Culvert Outlets
OPSD 3090.100	Foundation Frost Depths for Northern Ontario
SP109S12	Amendment to OPSS 902 - QVE, Backfilling Compaction, and Certificate of Conformance
SPFOUN0003	Amendment to OPSS 902 – Dewatering Structure Excavations

2. Suggested text for a NSSP on “Obstructions”

Obstructions such as cobbles and boulders may be encountered within the glacial till. These obstructions may also be present in the fill. Such obstructions may impede the progress of open-cut excavations, and/or installation of temporary protection systems. The contractor shall use appropriate equipment and methodologies to penetrate the obstructions.

3. Suggested text for a NSSP on “Dewatering Structure Excavations”

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

The contractor is notified that a Permit to Take Water (PTTW) has been issued for this project. The contractor shall retain a specialist to design the dewatering and treatment systems to meet the requirements of the PTTW and the contract documents.

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.