



THURBER ENGINEERING LTD.

**FINAL
FOUNDATION INVESTIGATION AND DESIGN REPORT
REHABILITATION OF BRIDGE STRUCTURE No. 40-024
HIGHWAY 35 GULL RIVER SOUTH BRIDGE
LUTTERWORTH TOWNSHIP
G.W.P. 5087-11-00
AGREEMENT NO.: 5015-E-0043**

GEOCRES NUMBER: 31D-692

**SUBMITTED TO
MCINTOSH PERRY CONSULTING ENGINEERS**

Location:

Latitude: 44.806067°

Longitude: -78.803566°

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Thurber File: 16284

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual data obtained from a foundation investigation conducted by Thurber Engineering Ltd. (Thurber) for the proposed rehabilitation of the Gull River South Bridge located on Highway 35, within Lutterworth Township. Thurber carried out the investigation as a subconsultant to McIntosh Perry Consulting Engineers (MPCE) as part of Agreement No. 5015-E-0043.

The purpose of this investigation was to explore the subsurface conditions at the site and, based on this data, provide a borehole location plan, record of boreholes, a stratigraphic profile, laboratory test results and a written description of the subsurface conditions. A base plan survey drawing was provided by MPCE for the preparation of this report.

An earlier foundation investigation report that has been obtained from the online Geocres Library in preparation of this report is as follows:

Foundation Investigation Report for New Structure at Gull River & Hwy. #35, Moore's Falls, District #11 (Huntsville), W.J. 67-F-56 - W.P. 425-65 & 106-65 (Geocres 31D00-128), dated August 1967.

The boreholes from this historic report were drilled off the current alignment of Highway 35 and therefore may not reflect conditions at the existing bridge foundations. Furthermore, the position of the boreholes from the historical report relative to the boreholes completed as part of the current investigation are not known. For these reasons the historic boreholes have not been included in the description of the subsurface conditions within this report.

2 SITE DESCRIPTION

The existing structure (No. 40-024) is located on Highway 35, approximately 0.35 km north of Haliburton Road 2 (Deep Bay Rd) near Miner's Bay, Ontario. It is noted that for project orientation purposes, Highway 35 within the project limits, will be described with a north-south alignment. The location of the bridge is shown on the inset Key Plan on Drawing No. 1 in Appendix A.

Within the project limits, Highway 35 is a two-lane highway. Based on the September 2017 drawing provided by MPCE, the roadway cross-section consists of two, 3.75 m wide lanes, and paved shoulders with a width of 1.5 m and 1.3 m on the SBL and NBL respectively. There is a 1.5 m wide sidewalk just outside the shoulder on the south bound side. Steel guide rails are located on both sides of the highway for a short distance from the bridge.

The existing bridge is a 20 m single span concrete bridge. The bridge is noted in the RFP to be constructed in 1968 with abutments founded on spread footings on bedrock.

The embankment slopes located adjacent to the north abutment are inclined at approximately 2H:1V with the surface consisting of granular fill. The southwest embankment slope was also inclined at approximately 2H:1V with granular fill noted at the surface but the southeast embankment slope was found to be inclined at approximately 1.5H:1V and rockfill with some granular infill was noted at the surface. Based on the drawing provided by MPCE, the elevation of the center line of roadway was reported to be approximately 273.0 m and 272.9 m at the north and south abutments, respectively.

Water within Gull River flows from the west to the east. Water control dams are located in close proximity on the upstream side of the bridge. Since the Gull River Bridge is located downstream of the water control structures, it is expected that relatively quick changes in water levels may be encountered. The topography adjacent to the river at the site is rolling forested lands with frequent bedrock outcrops. The land in the vicinity of the bridge is occupied mainly by single-family dwellings and cottages with the exception of a restaurant which is present southwest of the bridge site. Traffic volumes are understood to be 3150 AADT (2013).

Site photographs showing the general conditions at the site during the time of the field investigation are presented in Appendix D.

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 proposed boreholes.

The field investigation for this site included advancing two boreholes drilled on May 10th and 11th, 2017. The northing, easting and elevation of the boreholes are shown on the Borehole Location and Soil Strata Drawing No. 1 in Appendix A and are summarized in Table 3-1. The site is within MTM Zone 10.

Table 3-1: Borehole Summary

Borehole No.	Drilled Location	Northing (m)	Easting (m)	Ground Surface Elevation (m)	Borehole Termination Depth Below Existing Ground Surface (m)
17-1	North Abutment – southbound lane	4 963 116.8	359 915.1	273.1	7.3
17-2	South Abutment – northbound lane	4 963 136.5	359 877.2	272.9	7.3

Both boreholes were advanced through the roadway embankment with a truck mounted CME 75 drill rig equipped with HW/NW casing. The subsurface stratigraphy encountered in the boreholes was recorded in the field by Thurber personnel. Split spoon samples were collected at regular depth intervals in the boreholes via the completion of Standard Penetration Tests (SPT), following the methods described in ASTM Standard D1586. Rock was cored and collected using NQ coring

equipment. All soil and rock core samples recovered from the boreholes were transported to Thurber's Ottawa geotechnical laboratory for further examination and testing.

The boreholes were backfilled with a low-permeability mixture of auger cuttings and bentonite pellets in accordance with Ontario MOE Regulation 903. Boreholes advanced within paved areas were capped with cuttings followed by 150 mm of cold patch asphalt to reinstate the travelling surface.

The as-drilled locations and ground surface elevation of the boreholes were surveyed by MPCE in July 2017.

3.1 Laboratory Testing

Geotechnical laboratory testing consisted of natural moisture content determination and visual identification of all retained soil samples in accordance with the current MTO standards. Grain size distribution analyses testing was also carried out on selected samples to MTO and ASTM standards. All rock cores were photographed and their total core recovery (TCR), solid core recover (SCR) and rock quality designation (RQD) were determined. Chemical analysis for determination of pH, conductivity, resistivity, soluble sulphate and chloride concentrations was carried out on two soil samples.

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

4 DESCRIPTION OF SUBSURFACE CONDITIONS

4.1 Overview / General

Reference is made to the Record of Borehole sheets in Appendix B for details of the soil stratigraphy encountered in the boreholes. A stratigraphic profile is presented on Drawing No. 1 in Appendix A for illustrative purposes. An overall description of the stratigraphy is given in the following paragraphs; however, the factual data presented in the Record of Boreholes governs any interpretation of the site conditions. It must be recognized that soil and groundwater conditions may vary between and beyond sampled locations.

The stratigraphy encountered in the boreholes through the embankment near the abutment is generally characterized by the asphalt pavement structure underlain by granular fill and rockfill embankments overlying bedrock.

4.2 Asphalt

Both boreholes were advanced through the Highway 35 pavement structure and encountered asphalt ranging from 225 mm to 400 mm in thickness.

4.3 Fill

4.3.1 Granular Fill

A granular fill layer consisting predominantly of sand with silt and gravel to silty sand with gravel was encountered below the asphalt in both boreholes. This layer has a thickness ranging from 1.3 m to 3.3 m (bottom elevation of 269.4 m to 271.4 m). Cobbles and boulders were observed

in this unit within Borehole 17-1. The SPT 'N' values ranged from 6 to 42 blows; indicating a loose to dense condition.

The moisture content of the granular fill samples tested ranged from 9% to 20%. The results of three grain size analyses conducted on samples of granular fill are summarized in Table 4-1 and illustrated on Figure C1 in Appendix C.

Table 4-1: Gradation Results for Granular Fill

Soil Particle	%
Gravel	4 to 26
Sand	61 to 91
Silt and Clay	5 to 13

4.3.2 Rock Fill

A layer consisting predominantly of rock fill was encountered beneath the granular fill in Borehole 17-2. This layer has a top elevation of 271.4 m, and a thickness of 3.2 m. The borehole was advanced through the rockfill using casing and coring techniques. Sampling was attempted, however due to the nature of this material sample recovery was poor or not feasible. A single SPT 'N' value of 100 blows for 75 mm of penetration was obtained within this layer; indicating a very dense condition.

Rockfill pieces were cored and indicated particles with diameters up to 575 mm. Boulders estimated as large as 1 m in diameter were observed on the side slopes of the embankment adjacent to Borehole 17-2.

4.4 Bedrock

The fill was underlain by granite bedrock in both boreholes and was proven with coring techniques. The bedrock surface ranges from elevation 268.2 to 269.4 m and is summarized in the table below:

Table 4-2 Summary of Bedrock Elevation

Location	Borehole No.	Depth Below Existing Ground Surface (m)	Top of Bedrock Elevation (m)
North Abutment	17-1	3.7	269.4
South Abutment	17-2	4.7	268.2

The Total Core Recovery (TCR) ranged from 87 to 100%, the Solid Core Recovery (SCR) ranged from 87 to 100% and the Rock Quality Designation (RQD) typically ranged from 53 to 98%. The diamond core barrel wore out near the beginning of Run 3 on Borehole 17-2 and no further penetration was achievable. This resulted in a core recovery for Run 3 of less than 100 mm and an insufficient length of core to calculate an RQD for that run. Based on the RQD values the bedrock is classified as fair to excellent quality.

4.5 Groundwater

No water levels were obtained during drilling due to the introduction of water into the casing by the drilling method used in Borehole 17-1 and 17-2. The hydrology report should be referenced for water levels in the Gull River.

Due to the permeable nature of the granular fill and the open nature of the rockfill and approach embankments located within close proximity to upstream water control structures, it is expected that the groundwater level will respond rapidly to the water level changes in Gull River.

4.6 Results of Analytical Tests

Two samples of soil recovered from within the boreholes were selected and submitted for analytical testing including pH, conductivity, resistivity, chloride and sulphate. The results are summarized below and presented in the Certificate of Analysis included in Appendix C.

Table 4-3: Analytical Results Summary

Borehole	Sample	Depth (m)	pH	Conductivity (uS/cm)	Resistivity (Ohm-cm)	Chloride (µg/g)	Sulphate (µg/g)
17-1	SS4	2.8	7.2	1530	654	681	299
17-2	SS2	1.1	7.6	2980	336	1890	18

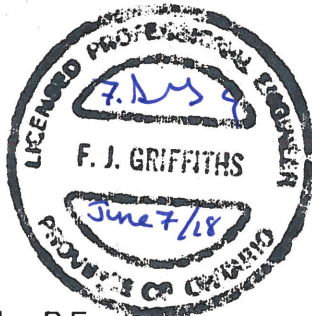
5 MISCELLANEOUS

Thurber obtained utility clearances prior to drilling and the borehole locations were positioned relative to existing site features and proposed works. MPCE surveyed the borehole locations and ground surface elevations. George Downing Estate Drilling Ltd. of Hawkesbury, Ontario supplied and operated the drilling equipment to carry out the drilling, sampling, in-situ testing and borehole decommissioning. The drilling, and sampling operations in the field were supervised on a full-time basis by Mr. Jeffery Morrison, E.I.T. of Thurber. Laboratory testing was carried out in Thurber's MTO-approved laboratory in Ottawa.

Overall project management and direction of the field program was provided by Mr. Stephen Peters, P.Eng. Interpretation of the field data and preparation of this report was completed by Mr. Christopher Murray, M.Sc., P.Eng.. The report was reviewed by Dr. Fred Griffiths, P.Eng. and Dr. P.K. Chatterji, P.Eng., the Designated Principal Contact for MTO Foundations Projects.



Christopher Murray, M.Sc., P.Eng.
Geotechnical Engineer



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



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

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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

6 GENERAL

This section of the report presents interpretation of the factual data in Part 1 of this report for the proposed rehabilitation of the Gull River South Bridge located on Highway 35, near Miner's Bay, Ontario. Geotechnical assessment and recommendations are provided to assist the design team with the design of a temporary protection system for rehabilitation of the abutments.

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 following sections address the foundation aspects of the design and installation of a temporary protection system required for the rehabilitation of the bridge. The discussions and recommendations presented in this report are based on the information provided by MPCE and on the factual data obtained during the course of the investigation.

6.1 Proposed Structure

The existing bridge, as described in the RFP, is an 18.3 m single span concrete bridge noted to be constructed in 1968. The structure supports two lanes of traffic and a sidewalk adjacent to the southbound lane. The north and south abutments are understood to be founded on spread footings on bedrock.

The proposed rehabilitation of the Gull River South Bridge is indicated on the May 28th, 2018 60% Contract Drawing Package. The rehabilitation includes the removal and patching of deteriorated areas of concrete in the deck soffit, wingwalls, abutment walls and girders, the reconstruction of the ballast walls, as well as conversion to semi-integral abutments. Based on cross sectional drawings received from MPCE on April 27th, 2018 it is indicated that minimal road widening and a negligible grade raise will be carried out as part of the permanent rehabilitation works. The proposed construction staging shows that a temporary roadway protection system placed near the highway centerline will be required to maintain a single lane of traffic during rehabilitation/conversion of the abutments.

6.2 Applicable Codes and Design Considerations

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

7 SEISMIC CONSIDERATIONS

7.1 Spectral and Peak Acceleration Hazard Values

The seismic hazard data for the CHBDC is based on the fifth-generation seismic model developed by the Geological Survey of Canada (GSC). 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 E.

The site coefficients used to determine the design spectral acceleration and displacement values are a function of the Site Class and the peak ground acceleration (PGA), which is 0.071g at this site.

7.2 CHBDC Seismic Site Classification

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

Based on the soil conditions encountered below road surface, the site has been classified as a Site Class C in accordance with Section 4.4.3.2 of the CHBDC (S6-14).

7.3 Seismic Liquefaction

Based on the PGA value and the subsurface conditions encountered at the drilled locations at this site, the embankment soils are considered not susceptible to liquefaction during a seismic event.

8 DESIGN RECOMMENDATIONS

8.1 Temporary Protection Systems

Temporary protection systems (TPS) should be provided in accordance with OPSS.PROV 539 and designed for Performance Level 2 (maximum 25 mm horizontal deflection). The actual pressure distribution acting on the shoring systems is a function of the construction sequence and relative flexibility of the wall and these factors must be considered when design the shoring system.

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

Driving of sheet piles as roadway protection at this site will be difficult due to the presence of rockfill at the south abutment and cobbles and boulders within the fill at the north abutment. It is recommended that an NSSP be included in the tender documents to alert the Contractor to the presence of obstructions and difficulty of driving sheet piles. Suggested text for this NSSP has been included in Appendix F.

Drilled in soldier piles and lagging is another TPS option. Bedrock is at shallow depth, therefore depending on the depth of excavation, socketing of soldier piles in bedrock may be required to provide lateral stability of the temporary roadway protection and should be verified by the roadway protection Designer. Bracing of the shoring could also be considered.

8.2 Backfill and Lateral Earth Pressure

Backfill to the structure should be placed in accordance with OPSS 902. All backfill material should consist of Granular A, or Granular B Type II meeting OPSS.PROV 1010 specifications. The backfill must be in accordance with OPSS 902 and placed to the extents shown on OPD 3101.150.

Compaction equipment to be used adjacent to the walls should be restricted in accordance with OPSS.PROV 501. If adequate drainage cannot be confirmed, the potential of hydrostatic pressures should be considered.

8.2.1 Static Lateral Earth Pressure Coefficients

Lateral earth pressures acting on structures should be computed in accordance with the CHBDC but, under fully drained conditions, is generally given by the expression:

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

where:

P_h = horizontal pressure on the wall (kPa)

K = earth pressure coefficient (see table below)

γ = unit weight of retained soil (see table below), adjusted for water 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. The recommended lateral earth pressure parameters for use in the design for a horizontal back-slope are provided in Table 8-1.

Table 8-1 Static Lateral Earth Pressure Coefficients, Horizontal Backslope

Parameter	OPSS Granular A & B Type II	OPSS Granular B Type I	Existing Granular Fill	Existing Rockfill
Soil Unit Weight, kN/m^3 , γ	22.8	21.2	20.0	18.0
Angle of Internal Friction, ϕ	35°	32°	30°	42°
Coefficient of at Rest Earth Pressure, K_0 (Restrained Wall)	0.43	0.47	0.50	0.33
Coefficient of Active Earth Pressure, K_a (Unrestrained Wall)	0.27	0.31	0.33	0.20
Coefficient of Passive Earth Pressure, K_p	3.7	3.3	3.0	5.0

For rigid structures, it is recommended that at-rest horizontal lateral earth pressure parameters be used for design. Active pressures should be used for the design of unrestrained walls. The parameters in the table correspond to full mobilization of active and passive earth pressure and require certain relative movements between the wall and adjacent soil to produce these conditions. The values used in design can be assessed from Figure C6.16 of the Commentary to the CHBDC. The earth pressure coefficients should be adjusted where ground surfaces are sloped behind the walls.

Passive earth resistance in front of the walls should be ignored for all permanent structures.

8.2.2 Combined Static and Seismic Lateral Earth Pressure Parameters

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 Mononobe-Okabe Method with:

- $k_h = \frac{1}{2} * F(\text{PGA}) * \text{PGA}$, for structures that allow 25 to 50 mm of movement, and
- $k_h = F(\text{PGA}) * \text{PGA}$, 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 8-2 for use in the design of a horizontal back-slope may be used. The provided earth pressure coefficients are based on a Seismic Site Class C, PGA with a 2% probability of exceedance in 50 years of 0.071g (Geological Survey of Canada – Fifth Generation) and a $F(\text{PGA})$ of 1.00 as per Table 4.8 of the CHBDC (S6-14 update No. 2, July 2017).

Table 8-2 Dynamic Lateral Earth Pressure Coefficients, Horizontal Backslope

Parameter	OPSS Granular A & B Type II	OPSS Granular B Type I	Existing Granular Fill	Existing Rockfill
Soil Unit Weight, kN/m ³ , γ	22.8	21.2	20.0	18.0
Angle of Internal Friction, ϕ	35°	32°	30°	42°
Active, K_{AE} Yielding Wall	0.29	0.33	0.35	0.21
Active, K_{AE} Non-Yielding Wall	0.31	0.35	0.38	0.23

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, adjusted for water level
K_{AE}	=	combined static and seismic earth pressure coefficient
H	=	total height of the wall (m)

8.3 Embankment Design and Reinstatement

Embankment reconstruction 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. Alternatively, the embankments could be reconstructed using rockfill with side slopes reinstated at 1.5H:1V (or flatter). If granular is used over rockfill then the surface of the rockfill must be chinked and a geotextile (Class II non-woven FOS 50 to 150 μ m) should be used as a separation layer. Where newly placed embankment fill is placed against a sloping ground surface or on existing fill, benching of the existing slope should be carried out in accordance with OPSD 208.010.

It is understood that minimal permanent widening and grade raise (<100 mm) is currently proposed. Provided no additional increase in embankment width or height is proposed, negligible foundation settlement is expected to occur.

8.4 Cement Type and Corrosion Potential

Two samples of the soil were submitted to Paracel Laboratories in Ottawa, Ontario for analysis of pH, water soluble sulphate and chloride concentrations, resistivity and conductivity. The analysis

was completed to determine the potential for degradation of the concrete in the presence of soluble sulphates and the potential for corrosion of exposed steel used in foundations and buried infrastructure. The analysis results are summarized in the Table 4-3.

The concentration of soluble sulphate provides an indication of the degree of sulphate attack that is expected for concrete in contact with soil and groundwater at the site. Soluble sulphate concentrations less than 1000 µg/g generally indicate that a low degree of sulphate attack is expected for concrete in contact with soil and groundwater. The class of concrete selected should consider the effects of road de-icing salts.

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

9 CONSTRUCTION CONSIDERATIONS

9.1 Excavations

All excavations must be conducted in accordance with the requirements of the Occupational Health & Safety Act & Regulations (OHSA) for Construction Projects. The fills and native soils above the water table at the site should be classified as Type 3 soils. All soils below the water table are considered to be Type 4 soils unless dewatering is carried out.

At locations where there are space restrictions or where a slope must be retained, the excavations will need to be carried out within a protection system as discussed in Section 8.1. Design of the temporary protection system is the responsibility of the Contractor.

9.2 Dewatering

Dewatering design and decisions regarding dewatering, must be carried out by the Contractor and carried out in accordance with SP517F01. The Contractor must be prepared to control the groundwater and surface water flow at the site to permit construction in a dry and stable excavation.

The depth of excavation is not expected to extend below the river level as noted on the general arrangement drawing (elev. 269.9 m on May 17, 2017). Therefore, dewatering is not expected to be a concern. Surface water should be directed away from excavations.

If the excavation extends below the groundwater level at the time of construction, the following inputs for *Table A* within SP 517F01 may be used: ***** = No and ***** = N/A. It is anticipated that conventional sump and pump techniques should be sufficient for controlling normal surface water and groundwater infiltration into excavations in the upper granular fill.

The river level is dam controlled and should excavations need to extend to below the river level there will be significant challenges to maintaining dry conditions due to the presence of the rock fill at the north approach. The excavation could require full enclosure by shoring and the placement of a tremie concrete plug prior to dewatering.

9.3 Erosion Control and Scour Protection

Slope protection and drainage measures will be required to ensure the long-term surficial stability of the reinstatement of the embankment slopes. Slope vegetation should be established as soon as possible after placement of the granular embankment fills in order to control surficial erosion in general accordance with OPSS.PROV 804. The contractor should provide silt fences and erosion control blankets, as required, throughout the duration of the construction to prevent silt/sediment from running off the site as per OPSS 805.

Due to the proximity of water control structures, scour and erosion protection will be paramount and should be reviewed for adequacy along the river banks in the area of the bridge. Design of the scour and erosion protection measures must consider hydrologic and hydraulic concerns and should be carried out by specialists experienced in the field. Typically, rock protection should be provided over all earth surfaces subjected to flowing water in accordance with OPSS 511. The embankment material consists of sand with silt and gravel and rockfill and is considered to have a low erosion potential.

10 CONSTRUCTION CONCERNS

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

- Cobbles, boulders and rockfill or other buried obstructions will be encountered in the existing approach embankments. An NSSP should be included in the contract alerting the Contractor to these conditions. Driving of sheet piles will be difficult. Obstructions will be encountered during excavation.
- Seasonal fluctuations of the groundwater and river level are to be expected which may impact the construction. Dewatering in rockfill below the water level will be difficult.

The successful outcome of the project will depend largely upon good workmanship and quality control during construction. Observation of the excavation and backfilling operations by qualified geotechnical personnel in accordance with SP109S12 will be required during construction to confirm that the foundation recommendations are correctly implemented, and material specifications are met.

11 CLOSURE

Engineering analysis and preparation of this report was completed by Mr. Christopher Murray, M.Sc., P.Eng.. The report was reviewed by Dr. Fred Griffiths, P.Eng. and Dr. P.K. Chatterji, P.Eng., the Designated Principal Contact for MTO Foundations Projects.



Christopher Murray, M.Sc., P.Eng.
Geotechnical Engineer

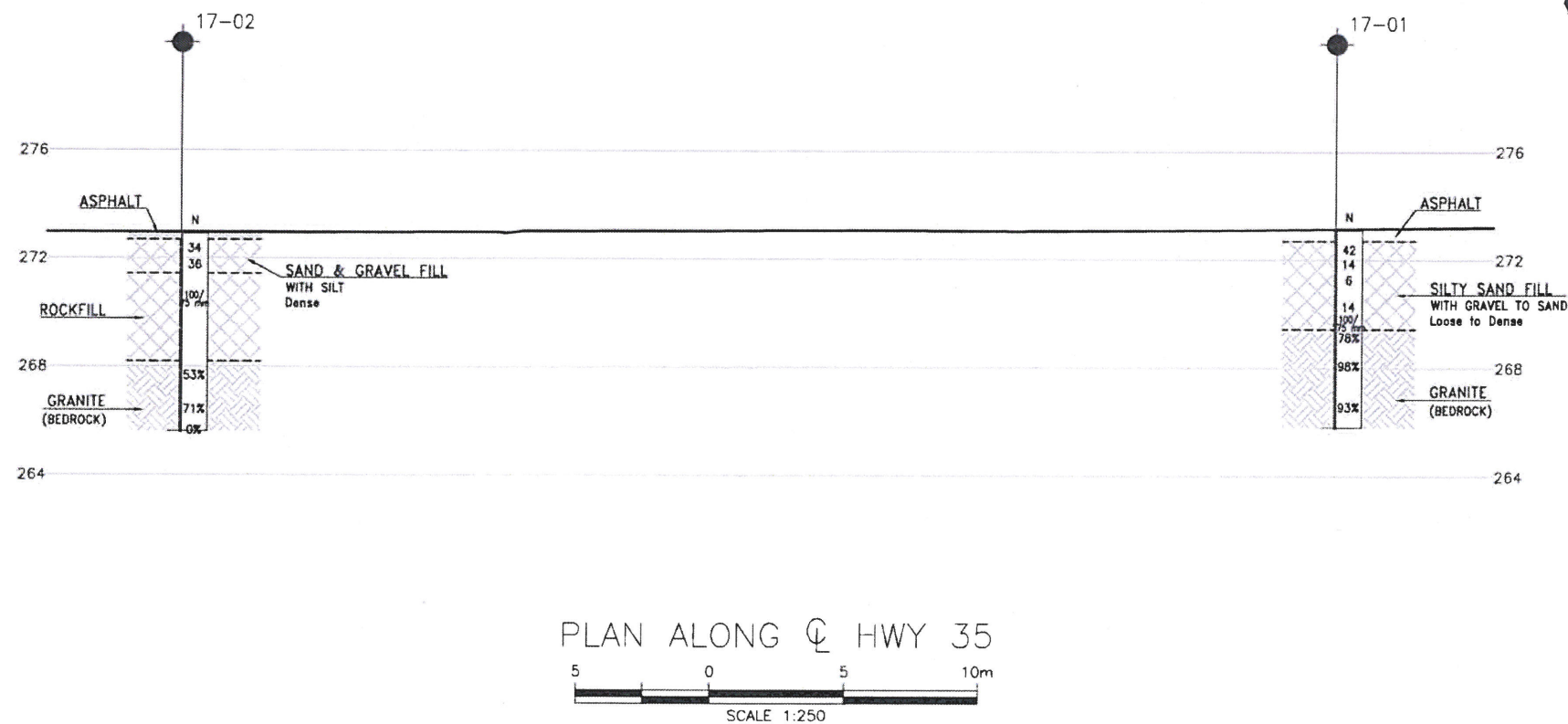
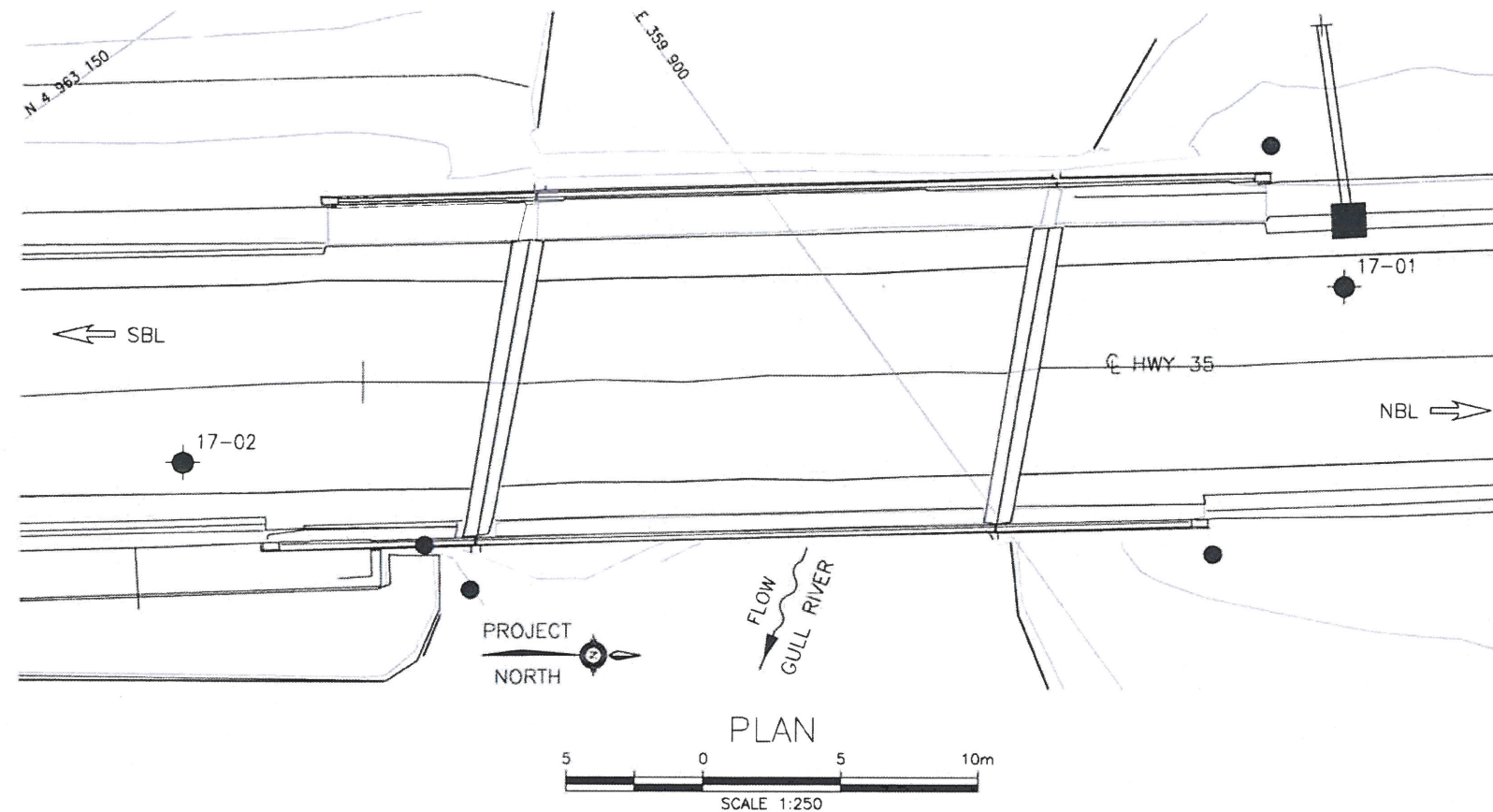


Dr. Fred Griffiths, P.Eng.
Senior Associate
Senior Geotechnical Engineer



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

APPENDIX A
BOREHOLE LOCATION AND SOIL STRATA DRAWINGS



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

CONT No
GWP No 5087-11-00

HIGHWAY 35
GULL RIVER SOUTH
BRIDGE REHABILITATION
BOREHOLE LOCATIONS AND SOIL STRATA






**McINTOSH
PERRY**



THURBER ENGINEERING LTD.



KEYPLAN
LEGEND

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

NO	ELEVATION	NORTHING	EASTING
17-01	273.1	4 963 116.8	359 915.1
17-02	272.9	4 963 136.5	359 877.2

-NOTES-

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

GEOCRES No. 31D-692

[illegible]

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).

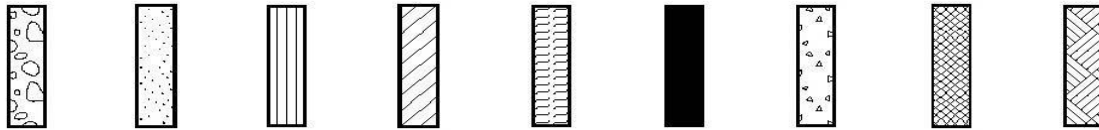
DYNAMIC CONE PENETRATION TEST (DCPT):

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 17-1

1 OF 1

METRIC

GWP# 5087-11-00 LOCATION Gull River South Bridge, MTM z10: N 4 963 116.8 E 359 915.1 ORIGINATED BY JM
 HWY 35 BOREHOLE TYPE NW Casing / NQ Coring COMPILED BY JM
 DATUM Geodetic DATE 2017.05.11 - 2017.05.11 CHECKED BY CM

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					
								WATER CONTENT (%)					
273.1													
0.0	400 mm ASPHALT												
272.7													
0.4	Silty SAND with Gravel to SAND Loose to Dense Brown FILL		1	SS	42								26 61 13 (SI+CL)
			2	SS	14								
			3	SS	6								4 91 5 (SI+CL)
	- 250 mm Boulder at 2.3 m		4	SS	14								
	- Frequent Cobbles below 3.1 m		5	SS	100/ 275 mm								
269.4													
3.7	GRANITE BEDROCK, occasional Chlorite and Quartz seams Slightly Weathered to Fresh Medium Bedded Grey		1	RUN									RUN #1 TCR=87% SCR=87% RQD=78%
			2	RUN									RUN #2 TCR=100% SCR=100% RQD=98%
			3	RUN									RUN #3 TCR=100% SCR=100% RQD=93%
265.8													
7.3	End of Borehole												

ONTMT4S 16284 GULL RIVER BRIDGE SOUTH.GPJ 2012TEMPLATE(MTO).GDT 16/3/18

RECORD OF BOREHOLE No 17-2

1 OF 1

METRIC

GWP# 5087-11-00 LOCATION Gull River South Bridge, MTM z10: N 4 963 136.5 E 359 877.2 ORIGINATED BY JM
HWY 35 BOREHOLE TYPE NW Casing / NQ Coring COMPILED BY JM
DATUM Geodetic DATE 2017.05.10 - 2017.05.10 CHECKED BY CM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				W P W W L							
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	WATER CONTENT (%)										
272.9							20	40	60	80	100	20	40	60		GR	SA	SI	CL
0.0	225 mm ASPHALT																		
0.2	SAND with Silt and Gravel Dense Brown FILL		1	SS	34														15 76 9 (SI+CL)
			2	SS	36														
271.4																			
1.5	ROCK FILL - 150 mm Boulder at 1.5 m - 200 mm Boulder at 1.7 m - 575 mm Boulder at 1.9 m - Frequent Cobbles below 2.5 m																		
			3	SS	100/ 75 mm														
268.2																			
4.7	GRANITE BEDROCK Slightly Weathered Thinly Bedded Grey and Pink		1	RUN															RUN #1 TCR=100% SCR=100% RQD=53%
			2	RUN															RUN #2 TCR=100% SCR=100% RQD=71%
																			RUN #3 TCR=100% SCR=100% RQD=0%
265.6			3	RUN															
7.3	End of Borehole																		

ONTMT4S 16284 GULL RIVER BRIDGE SOUTH.GPJ 2012TEMPLATE(MTO).GDT 16/3/18

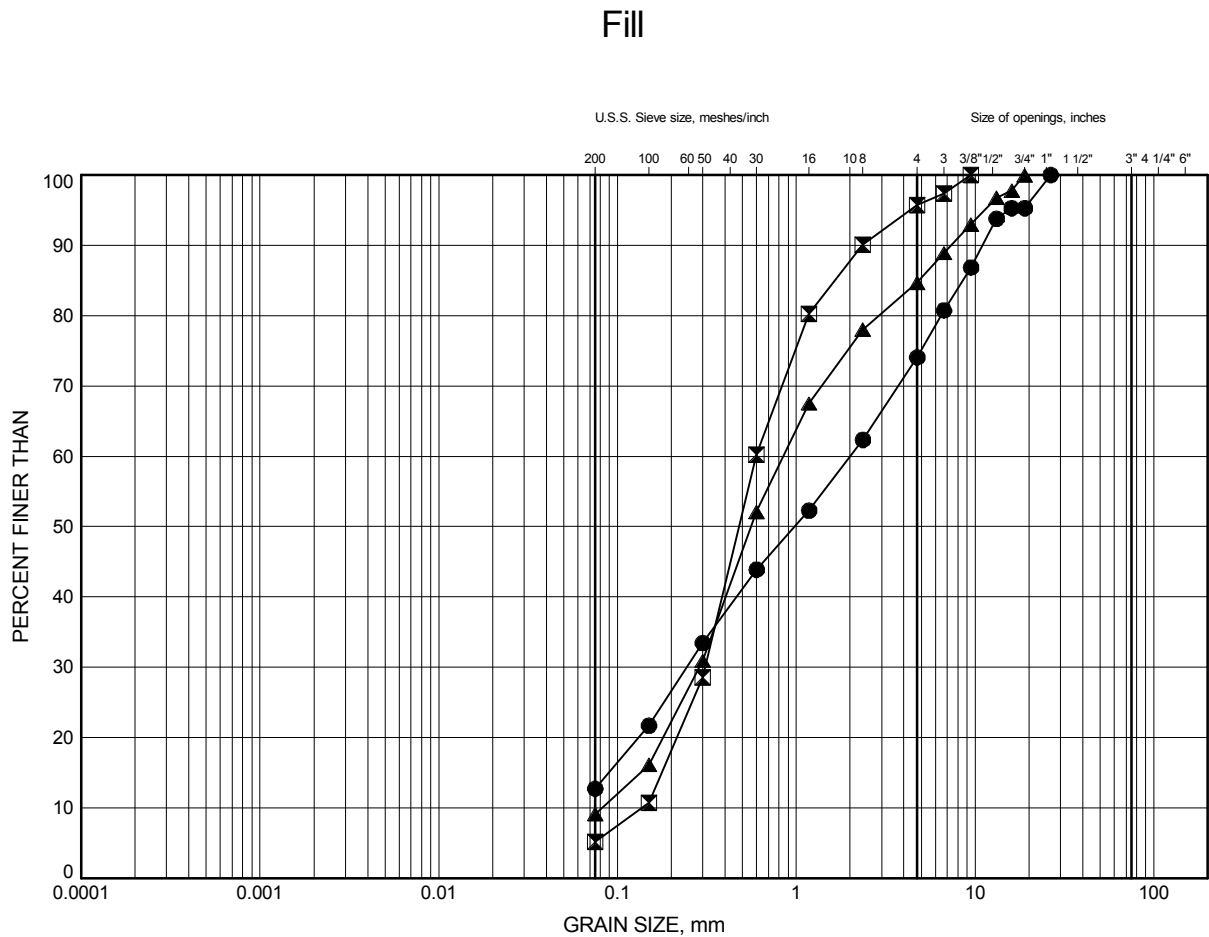
+³, ×³: Numbers refer to Sensitivity
20
15
10
(%) STRAIN AT FAILURE

APPENDIX C
LABORATORY TEST RESULTS

Gull River South Bridge

GRAIN SIZE DISTRIBUTION

FIGURE C1



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	17-1	0.71	
⊠	17-1	1.83	
▲	17-2	0.53	

Date July 2017
GWP# 5087-11-00



Prep'd CM
Chkd. SP

Borehole 17-1
Runs 1 and 2 (of 3)
Elevation 269.4 m to 267.3 m



Borehole 17-1
Run 3 (of 3)
Elevation 267.3 m to 265.8 m

Run 3 Start
elev.267.3m



Run 3 End
elev.265.8m



Foundation Investigation
Gull River South Bridge
Lutterworth Township, Ontario

GWP: 5274-14-00

Project No.: 16284

Borehole 17-2
Runs 1 and 2 (of 3)
Elevation 268.2 m to 266.3 m



Borehole 17-2
Run 2 and 3 (of 3)
Elevation 266.3 m to 265.6 m



Certificate of Analysis

Thurber Engineering Ltd.

2460 Lancaster Rd, Suite 104
Ottawa, ON K1B4S5
Attn: Stephen Peters

Client PO: 16284
Project: South Gull River Bridge
Custody: 14056

Report Date: 1-Jun-2017
Order Date: 26-May-2017

Order #: 1721506

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

Paracel ID	Client ID
1721506-01	BH17-1, SS4, 8'3"-10'3"
1721506-02	BH17-2, SS2, 2'9"-4'9"

Approved By:



Dale Robertson, BSc
Laboratory Director

Certificate of Analysis
Client: Thurber Engineering Ltd.
Client PO: 16284

Report Date: 01-Jun-2017

Order Date: 26-May-2017

Project Description: South Gull River Bridge

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	29-May-17	29-May-17
Conductivity	MOE E3138 - probe @25 °C, water ext	1-Jun-17	1-Jun-17
pH, soil	EPA 150.1 - pH probe @ 25 °C, CaCl buffered ext.	28-May-17	28-May-17
Resistivity	EPA 120.1 - probe, water extraction	1-Jun-17	1-Jun-17
Solids, %	Gravimetric, calculation	28-May-17	28-May-17

Certificate of Analysis
Client: Thurber Engineering Ltd.
Client PO: 16284

Report Date: 01-Jun-2017

Order Date: 26-May-2017

Project Description: South Gull River Bridge

Client ID:	BH17-1, SS4, 8'3"-10'3"	BH17-2, SS2, 2'9"-4'9"	-	-
Sample Date:	11-May-17	10-May-17	-	-
Sample ID:	1721506-01	1721506-02	-	-
MDL/Units	Soil	Soil	-	-

Physical Characteristics

% Solids	0.1 % by Wt.	84.3	86.3	-	-
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General Inorganics

Conductivity	5 uS/cm	1530	2980	-	-
pH	0.05 pH Units	7.23	7.63	-	-
Resistivity	0.10 Ohm.m	6.54	3.36	-	-

Anions

Chloride	5 ug/g dry	681	1890	-	-
Sulphate	5 ug/g dry	299	18	-	-

Certificate of Analysis
Client: Thurber Engineering Ltd.
Client PO: 16284

Report Date: 01-Jun-2017

Order Date: 26-May-2017

Project Description: South Gull River Bridge

Method Quality Control: Blank

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

Certificate of Analysis
Client: Thurber Engineering Ltd.
Client PO: 16284

Report Date: 01-Jun-2017

Order Date: 26-May-2017

Project Description: South Gull River Bridge

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	153	5	ug/g dry	151			1.5	20	
Sulphate	890	5	ug/g dry	884			0.7	20	
General Inorganics									
Conductivity	735	5	uS/cm	758			3.1	6.2	
pH	7.88	0.05	pH Units	7.85			0.4	10	
Resistivity	13.6	0.10	Ohm.m	13.2			3.1	20	
Physical Characteristics									
% Solids	85.6	0.1	% by Wt.	85.9			0.3	25	

Certificate of Analysis
Client: Thurber Engineering Ltd.
Client PO: 16284

Report Date: 01-Jun-2017

Order Date: 26-May-2017

Project Description: South Gull River Bridge

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	256	5	ug/g	151	105	78-113			
Sulphate	972	5	ug/g	884	88.7	78-111			

Certificate of Analysis
Client: Thurber Engineering Ltd.
Client PO: 16284

Report Date: 01-Jun-2017
Order Date: 26-May-2017
Project Description: South Gull River Bridge

Qualifier Notes:

None

Sample Data Revisions

None

Work Order Revisions / Comments:

None

Other Report Notes:

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

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

APPENDIX D
SELECTED PHOTOGRAPHS



Figure 1: Roadway Platform at Bridge 40-024 looking South (05/11/2017)



Figure 2: Roadway Platform at Bridge 40-024 looking North (05/10/2017)



Figure 3: West Side of Bridge Looking South from North Abutment (05/10/2017)



Figure 4: East Side of Bridge Looking South from North Abutment (05/10/2017)

APPENDIX E

**LIST OF REFERENCED SPECIFICATIONS
SUGGESTED NON-STANDARD SPECIAL PROVISIONS
GSC SEISMIC HAZARD CALCULATION**

LIST OF REFERENCED SPECIFICATIONS

OPSD 208.010	Benching of Earth Slopes
OPSD 3101.150	Walls, Abutment, Backfill Minimum Granular Requirements
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 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
SP 517F01	Standard Special Provision for Dewatering of Pipeline, Utility, and Associated Structure Excavation
SP 109S12	Amendment to OPSS 902, November 2010 - QVE, Backfilling Compaction, an Certificate of Conformance

SUGGESTED NON-STANDARD SPECIAL PROVISIONS

Suggested text for an NSSP on "Obstructions"

"The presence of cobbles, boulders and buried obstructions within the fill and rock fill as well as the presence of shallow bedrock may have an impact on excavations as well as the installation of protection systems at this site."

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

October 12, 2017

Site: 44.806 N, 78.8035 W User File Reference: Highway 35 Gull River Bridges

Requested by: , Thurber Engineering Ltd.

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

Sa(0.05)	Sa(0.1)	Sa(0.2)	Sa(0.3)	Sa(0.5)	Sa(1.0)	Sa(2.0)	Sa(5.0)	Sa(10.0)	PGA (g)	PGV (m/s)
0.087	0.121	0.120	0.105	0.087	0.053	0.028	0.0071	0.0031	0.071	0.073

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)	0.013	0.037	0.056
Sa(0.1)	0.021	0.055	0.080
Sa(0.2)	0.022	0.057	0.082
Sa(0.3)	0.020	0.050	0.072
Sa(0.5)	0.015	0.041	0.059
Sa(1.0)	0.0078	0.024	0.035
Sa(2.0)	0.0033	0.012	0.018
Sa(5.0)	0.0007	0.0027	0.0042
Sa(10.0)	0.0005	0.0012	0.0018
PGA	0.012	0.031	0.046
PGV	0.0096	0.030	0.046

References

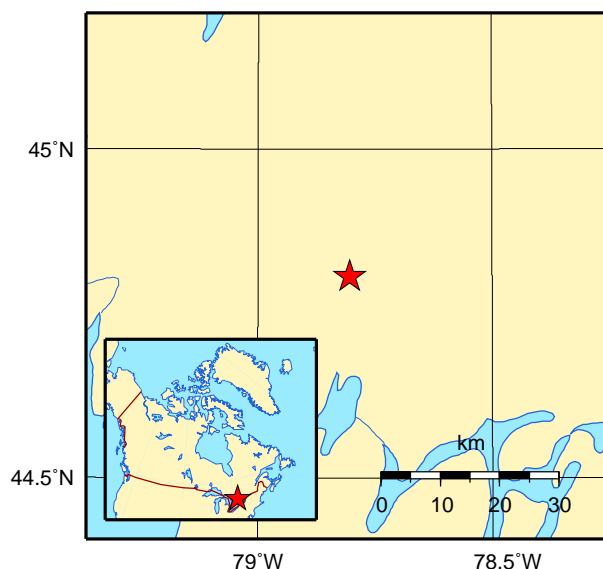
National Building Code of Canada 2015 NRCC no. 56190;
Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada

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



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