THURBER ENGINEERING LTD.

FINAL
FOUNDATION INVESTIGATION AND DESIGN REPORT
REHABILITATION OF BRIDGE STRUCTURE No. 40-023
HIGHWAY 35 GULL RIVER NORTH BRIDGE
LUTTERWORTH TOWNSHIP
G.W.P. 5087-11-00
AGREEMENT NO.: 5015-E-0043

GEOCRES NUMBER: 31D-693

SUBMITTED TO

MCINTOSH PERRY CONSULTING ENGINEERS

Location:
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Longitude: -78.802908°

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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 North 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 Geocres Library and reviewed during 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. 40-023) is located on Highway 35, approximately 0.4 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 1.2 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 have been constructed in 1968 with the north abutment of the bridge founded on steel H piles driven to bedrock and the south abutment founded on spread footings on bedrock.

The embankment slopes located adjacent to the abutment are inclined at approximately 2H:1V with the surface consisting of granular fill. Based on the drawing provided by MPCE, the elevation of the center line of roadway is approximately 274.0 m and 273.5 m at the north and south abutments, respectively.

Flow in the river is from west to east. Water control dams are located in close proximity to the downstream side of the bridge. Since the Gull River Bridge is located upstream 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 from May 9th to 10th 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	4 963 088.2	359 963.9	274.3	17.6
17-2	South Abutment – northbound lane	4 963 103.5	359 926.2	273.4	8.7

Both boreholes were advanced through the roadway embankment with a truck mounted CME 75 drill rig equipped with 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. Shallow bedrock 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. The backfill within Borehole 17-2 was supplemented with 4 bags of clean gravel due to a 0.6 m void encountered during drilling. 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 for the bridge area 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.

The stratigraphy encountered in the boreholes is generally characterized by the asphalt pavement, granular and embankment fill overlying native overburden at the north abutment and bedrock at the south abutment. At the north abutment, the native soil consists of clay underlain by silt, sandy silt and silty sand with gravel.

4.2 Asphalt

Both boreholes were advanced through the Highway 35 pavement structure. The thickness of the asphalt ranged from 210 mm to 355 mm.

4.3 Embankment Fill

A granular fill layer consisting predominantly of sand with silt and gravel to gravel with sand was encountered below the asphalt in both boreholes. The fill ranged in thickness from 3.5 to 4.8 m (bottom elevation of 268.2 m to 270.6 m). Occasional cobbles and boulders were present in the fill with frequent cobbles present in the gravel with sand fill in Borehole 17-2. The SPT ‘N’ values typically ranged from 9 blows to 100 blows for 200 mm of penetration; indicating a loose to very dense condition. In Borehole 17-02, a 0.6 m void was present in the fill from elevation 271.1 m to 270.5 m. In order to further understand the limits and extent of the void a ground penetrating radar

(GPR) survey was undertaken in the area of both the north and south bridge approaches. The final GPR report has been included in Appendix E for reference.

The moisture content of the granular fill samples tested ranged from 6% to 17%. 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	23 to 56
Sand	40 to 67
Silt and Clay	4 to 10

4.4 Clay (CL to CI)

A deposit of clay was encountered below the fill in Borehole 17-1. The clay had a thickness of 1.6 m with a base depth of 5.3 m (base elevation 268.9 m). The SPT ‘N’ values in the clay ranged from 5 to 9 blows per 300 mm penetration; indicating a firm to stiff condition.

The moisture content of the clay samples tested ranged from 33% to 37%. The results of a grain size analysis conducted on a sample of clay is summarized in Table 4-2 and is illustrated on Figure C2 in Appendix C.

Table 4-2: Gradation Results for Clay (CL to CI)

Soil Particle	%
Gravel	0
Sand	1
Silt	68
Clay	31

Atterberg Limit testing was completed on one sample of the clay. The results are summarized on the Record of Borehole sheets in Appendix B and the Atterberg Limit graph is included on Figure C5 in Appendix C. The laboratory results are summarized in Table 4-3 and indicate that the clay ranges from low to intermediate plasticity (CL to CI).

Table 4-3: Atterberg Results for Clay (CL to CI)

Parameter	Value
Liquid Limit	35
Plastic Limit	24
Plasticity Index	11

4.5 Silt (ML)

A deposit of silt was encountered below the clay in Borehole 17-01. The silt had a thickness of 6.9 m with a base depth of 12.2 m (base elevation 262.1 m). The silt deposit was interbedded with clay from 5.3 m to 7.2 m. The SPT 'N' values in the silt ranged from 5 to 12 blows per 300 mm penetration; indicating a loose to compact condition.

The moisture content of the silt samples tested ranged from 23% to 35%. The results of a grain size analysis conducted on a sample of silt is summarized in Table 4-4 and is illustrated on Figure C3 in Appendix C.

Table 4-4: Gradation Results for Silt (ML)

Soil Particle	%
Gravel	0
Sand	2
Silt	95
Clay	3

Atterberg Limit testing was completed on one sample of the silt and the results indicated the material was not plastic.

4.6 Sandy Silt (ML)

A deposit of sandy silt was encountered below the silt in Borehole 17-01. The sandy silt had a thickness of 4.6 m with a base depth of 16.8 m (base elevation 257.5 m). The SPT 'N' values in the sandy silt ranged from 4 to 9 blows per 300 mm penetration; indicating a loose condition.

The moisture content of the sandy silt samples tested ranged from 19% to 29%. The results of a grain size analysis conducted on a sample of sandy silt is summarized in Table 4-5 and is illustrated on Figure C4 in Appendix C.

Table 4-5: Gradation Results for Sandy Silt

Soil Particle	%
Gravel	1
Sand	26
Silt	71
Clay	2

Atterberg Limit testing was completed on one sample of the sandy silt and the results indicated the material was not plastic.

4.7 Silty Sand with Gravel

A deposit of silty sand with gravel was encountered below the sandy silt in Borehole 17-01. The borehole was terminated at SPT refusal in this deposit at a depth of 17.6 m (base elevation 256.7 m). The SPT 'N' values in the silty sand with gravel ranged from 22 to 100 blows per 25 mm penetration; indicating a compact to very dense condition. One moisture content recorded in the silty sand with gravel was 3%.

4.8 Bedrock

Granite bedrock was encountered below the embankment fill in Borehole 17-02 which was advanced into the bedrock by coring. Borehole 17-01 was terminated upon spoon refusal on inferredd bedrock. The elevation of the bedrock surface is summarized in the table below:

Table 4-6: Summary of Bedrock Elevation

Location	Borehole No.	Depth Below Existing Ground Surface (m)	Top of Bedrock Elevation (m)
North Abutment	17-1	17.6	256.7*
South Abutment	17-2	5.2	268.2

(*) – Inferred by split spoon refusal

The Total Core Recovery (TCR) ranged from 95 to 100%, the Solid Core Recovery (SCR) ranged from 89 to 98% and the Rock Quality Designation (RQD ranged from 0 to 80% with typical values from 76 to 80%. Based on the RQD values the bedrock below a surficial weathered zone is classified as good quality.

4.9 Groundwater

The ground water level was measured to be at elevation 271.1 m upon completion of drilling in Borehole 17-2. No water level was obtained in Borehole 17-1 during drilling due to the introduction of water into the casing by the drilling method used. The hydrology report should be referenced for water levels in the Gull River.

Due to the permeable nature of the granular fill and approach embankments; it is expected that the groundwater level will respond rapidly to the water level changes in Gull River.

4.10 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 4-7: Analytical Results Summary

Borehole	Sample	Depth (m)	pH	Conductivity (uS/cm)	Resistivity (Ohm-cm)	Chloride (µg/g)	Sulphate (µg/g)
17-1	SS5	4.0	7.32	321	3120	149	7
17-2	SS7	4.9	7.13	585	1710	210	149

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 abutment GPR survey was completed by Geophysics GPR International Inc. of Longueuil, Québec. 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 Miss. Deanna Pizycki M.Eng., E.I.T. 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.

Deanna Pizycki June 7, 2018

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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 North 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 structure is understood to have different supporting foundation types, with the north abutment of the bridge founded on steel H piles driven to bedrock and the south abutment founded on spread footings on bedrock.

The proposed rehabilitation of the Gull River North Bridge is indicated on the May 28th, 2018 60% Contract Drawing Package. The rehabilitation of the bridge 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 a road widening up to 1 m wide with a minimal grade raise along the embankments will be carried out as part of the permanent rehabilitation works. The proposed construction staging shows that a temporary roadway protection system placed along 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 (Sa(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 F.

The site coefficients used to determine the design spectral acceleration and displacement values are a function of the Site Class and the peak ground acceleration (PGA), 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 in Borehole 17-1, the site has been classified as a Site Class D 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 native non-cohesive 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 the north approach will be difficult due to the presence of cobbles and boulders in the fill. It is recommended that an NSSP be included in the tender documents to alert the Contractor to the potential for cobbles and boulders within the fill. 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 at the south abutment, 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 OPSD 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
γ = unit weight of retained soil (kN/m^3), 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 7-1.

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
Soil Unit Weight, kN/m^3, γ	22.8	21.2	20.0
Angle of Internal Friction, ϕ	35°	32°	30°
Undrained Shear Strength, kPa	-	-	-
Coefficient of at Rest Earth Pressure, K_o (Restrained Wall)	0.43	0.47	0.50
Coefficient of Active Earth Pressure, K_a (Unrestrained Wall)	0.27	0.31	0.33
Coefficient of Passive Earth Pressure, K_P	3.7	3.3	3.0

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 = ½ * 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 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 D, PGA with a 2% probability of exceedance in 50 years of 0.071g (Geological Survey of Canada – Fifth Generation) and a F(PGA) of 1.29 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
Soil Unit Weight, kN/m^3, γ	22.8	21.2	20.0
Angle of Internal Friction, ϕ	35°	32°	30°
Active, K_{AE} Yielding Wall	0.30	0.33	0.36
Active, K_{AE} Non-Yielding Wall	0.32	0.36	0.39

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 or II. Where newly placed embankment fill is placed against a sloping ground surface, benching of the existing slope should be carried out in accordance with OPSD 208.010.

It is understood that a permanent road widening up to 1.0 m, with a minimal grade raise (<100 mm) is currently being proposed. It is anticipated that settlements in the order of 10 mm are expected to be induced with this grade widening. These settlements should occur during fill placement and negligible settlement should occur following the construction phase.

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 4.7.

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 the Table 4.7 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 SP 517F01. 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. 270.5 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 sand embankment fill. 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 completion 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.

9.4 Potential Voids within the Embankment

A 0.6 m thick void was encountered in Borehole 17-02 at the south approach within the embankment fill from elevation 271.1 m to 270.5 m. To further understand the limits and extents of the void a ground penetrating radar (GPR) survey was undertaken by Geophysics GPR International in the area of both the north and south bridge approaches.

The GPR survey report indicates two possible buried voids at the bridge site. The first was observed at the south bridge approach in close proximity to Borehole 17-02 and was estimated to be approximately 1.5 m below surface and have a thickness of 0.75 m. The second possible void was encountered at the north approach and was estimated to be approximately 1.5 m below ground surface and have a thickness of 0.6 m. The void at the north approach appears to cover a greater area than the void at the south approach including a portion under the current north approach slab. For further details on the voids please refer to the Final GPR survey report included in Appendix E of this report.

One option to fix the voids is to excavate the approach fills to 15 m behind the abutments and to a depth of at least 2.5 m during the abutment rehabilitation. A taper of 10H:1V should be included away from the bridge. The frost taper requirement should be reviewed with the pavement engineer for this assignment. The base of the excavation should terminate on bedrock should it be encountered. Should voids be observed they should be backfilled with OPSS Granular B Type II following receipt of written notice to proceed in accordance with SP109S12. A non-woven Class II geotextile with an FOS of 75 to 150 µm (OPSS 1860) should be placed at the base of the excavation prior to backfilling. This option may require a full road closure to be completed successfully.

Alternatively, the voids can be treated in-place with injection methods to reduce the requirements for large excavations. One available material option is chemical compaction grouting with a high density, hydro-insensitive expanding polymer resin. The installation would be completed with drilled injection probes from the ground surface to fill any voids and stabilize the existing embankment fill. It is understood that these materials may not exist on MTO's approved list however, this site presents an opportunity for a demonstration project. The extent of soil stabilization by injection methods should also be to a distance of at least 15 m behind the existing abutment and to a minimum depth of at least 2.5 m. The injection grid spacing and depths should be determined by a specialist contractor experienced in such designs.

10 CONSTRUCTION CONCERNS

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

- Cobbles and boulders or other buried obstructions will be encountered in the existing approach embankments which may make installation of sheet piles difficult. An NSSP should be included in the contract alerting the Contractor to these conditions.
- Seasonal fluctuations of the groundwater and river level are to be expected which may impact the construction.
- There is a possibility of encountering voids within the embankment fill during excavation of the bridge approaches.

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

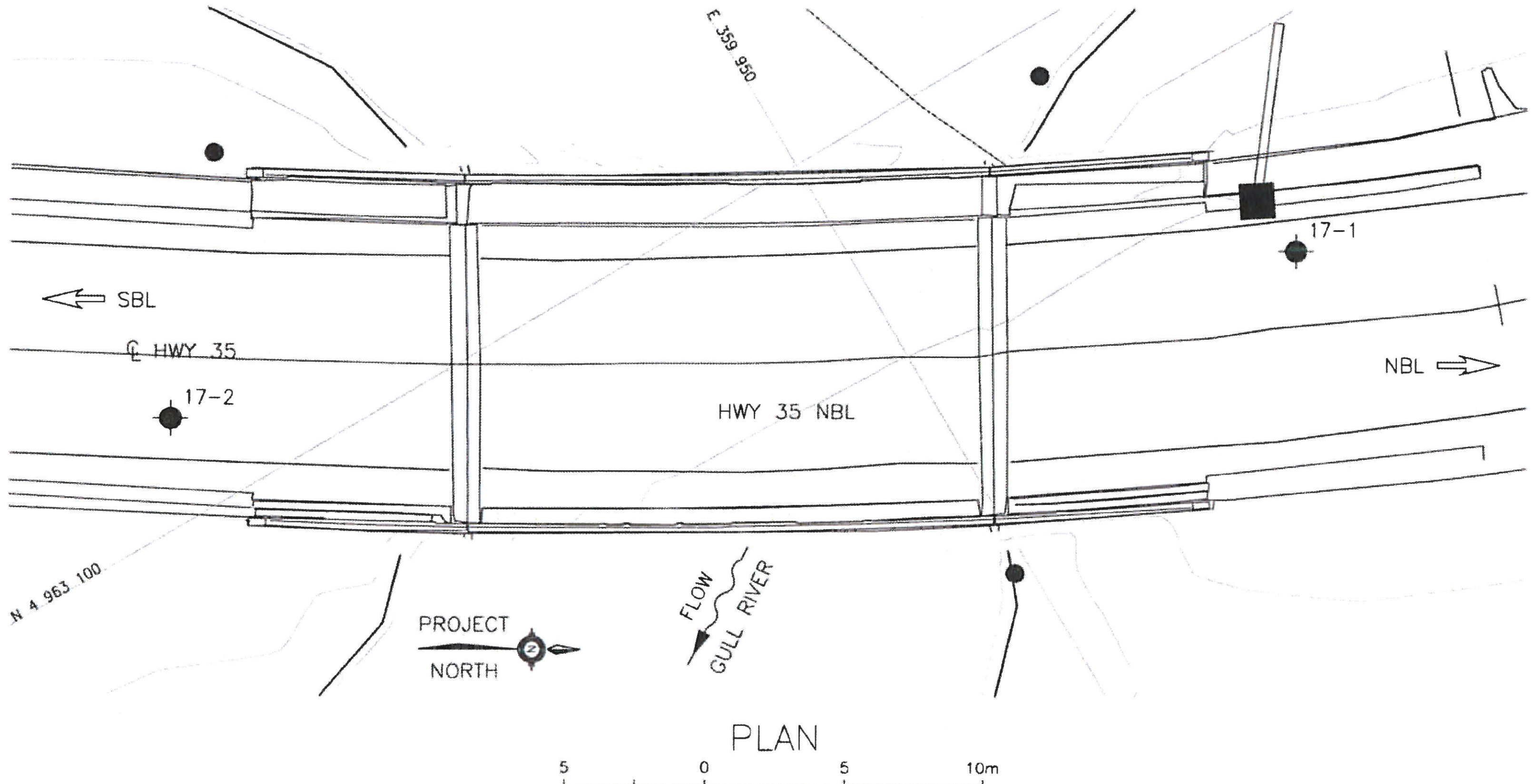
CONT No
GWP No 5087-11-00

SHEET

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

McINTOSH PERRY

THURBER ENGINEERING LTD.

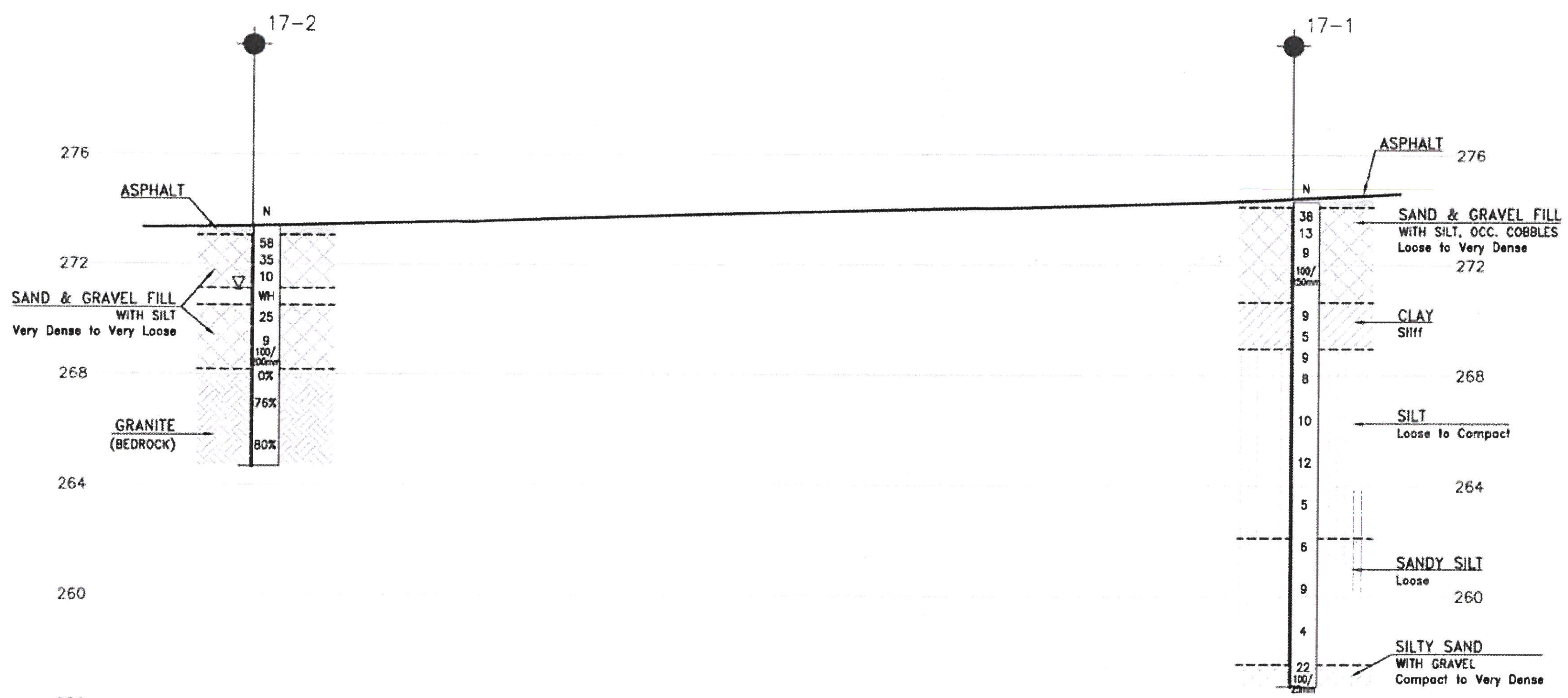


PLAN

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
June 7/18
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
F. J. GRIFFITHS
June 7/18
PROVINCE OF ONTARIO

KEYPLAN



PLAN ALONG ℄ HWY 35

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
17-1	274.3	4 963 088.2	359 963.9
17-2	273.4	4 963 103.5	359 926.2

-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-693

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

FILENAME: H:\Drafting\18000\18284\TED-18284-BHPP-GR.dwg
PLOTDATE: 3/19/2018 9:41 AM

MINISTRY OF TRANSPORTATION, ONTARIO

APPENDIX B

RECORD OF BOREHOLE SHEETS

THURBER

SYMBOLS, ABBREVIATIONS AND TERMS USED ON TEST HOLE RECORDS

TERMINOLOGY DESCRIBING COMMON SOIL GENESIS

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

TERMINOLOGY DESCRIBING SOIL STRUCTURE:

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

RECOVERY:

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

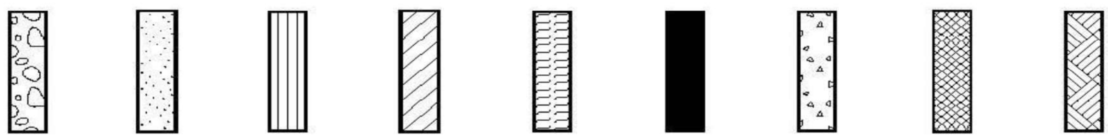
N-VALUE:

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

DYNAMIC CONE PENETRATION TEST (DCPT):

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

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



Boulders Cobbles Gravel	Sand	Silt	Clay	Organics	Asphalt	Concrete	Fill	Bedrock

TEXTURING CLASSIFICATION OF SOILS

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

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 (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.

TERMS DESCRIBING CONSISTENCY (COHESIONLESS SOILS ONLY)

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

THURBER

MODIFIED UNIFIED SOIL CLASSIFICATION

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

Note - W_L= Liquid Limit

THURBER

EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION

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

TERMS

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

DISCONTINUITY SPACING

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

STRENGTH CLASSIFICATION

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

RECORD OF BOREHOLE No 17-1

METRIC

GWP# 5087-11-00	LOCATION Gull River North Bridge, MTM z10: N 4 963 088.2 E 359 963.9	ORIGINATED BY JM
HWY 35	BOREHOLE TYPE NW Casing	COMPILED BY CM
DATUM Geodetic	DATE 2017.05.09 - 2017.05.09	CHECKED BY SP

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	GR	SA	SI	CL
274.3									
0.0	**210 mm ASPHALT**								
0.2	SAND with Silt and Gravel, Occasional Cobbles, Loose to Very Dense, Brown **FILL**	1	SS	38	~5				
		2	SS	13	~7	45	47	8 (SI+CL)	
		3	SS	9	~9				
		4	SS	100/ 250mm	~10				
	-840 mm Boulder at 2.8 m								
270.6 / 3.7	**CLAY (CL-CI)** Firm to Stiff, Brownish Grey	5	SS	9	~33 (range ~24–37)	0	1	68	31
		6	SS	5	~37				
268.9 / 5.3	**SILT (ML)** Loose to Compact, Grey, - interbedded with Clay from 5.3 m to 7.2 m	7	SS	9	~33				
		8	SS	8	~36				
		9	SS	10	~23	0	2	95	3
		10	SS	12	~24				

Elevation scale: 274, 273, 272, 271, 270, 269, 268, 267, 266, 265

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

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

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 17-1

METRIC

GWP# 5087-11-00 LOCATION Gull River North Bridge, MTM z10: N 4 963 088.2 E 359 963.9 ORIGINATED BY JM

HWY 35 BOREHOLE TYPE NW Casing COMPILED BY CM

DATUM Geodetic DATE 2017.05.09 - 2017.05.09 CHECKED BY SP

Continued From Previous Page

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%) (approx.)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILT (ML) Loose to Compact Grey				264		
		11	SS	5		~25	
					263		
262.1 / 12.2	**Sandy SILT (ML)** Loose Grey	12	SS	6	262	~27	
					261		
		13	SS	9	260	~21	
					259		
		14	SS	4		~19	1 26 71 2
					258		
257.5 / 16.8	**Silty SAND** with Gravel Compact Grey	15	SS	22	257	~3	
256.7 / 17.6	End of Borehole	16	SS	100/ 25mm			

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

+ ³, × ³: Numbers refer to Sensitivity

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 17-2

METRIC

GWP# 5087-11-00	LOCATION Gull River North Bridge, MTM z10: N 4 963 103.5 E 359 926.2	ORIGINATED BY JM
HWY 35	BOREHOLE TYPE NW Casing / NQ Coring	COMPILED BY CM
DATUM Geodetic	DATE 2017.05.10 - 2017.05.10	CHECKED BY SP

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
273.4					
0.0	**355 mm ASPHALT**				
273.0 / 0.4	SAND with Silt and Gravel, Very Dense to Very Loose, Brown, **FILL**	1	SS	58	
		2	SS	35	23 67 10 (SI+CL)
		3	SS	10	
271.1 / 2.3	**VOID** - Possible 0.6 m void at 2.3 m	4	SS	WH	
270.5 / 2.9	SAND with Silt and Gravel, Compact, Brown, **FILL**	5	SS	25	
269.7 / 3.7	GRAVEL with Sand, Frequent Cobbles, Loose to Very Dense, Brown, **FILL**	6	SS	9	56 40 4 (SI+CL)
		7	SS	100/ 200mm	
268.2	-150 mm Boulder at 5.1 m				
5.2	**GRANITE BEDROCK**, Chlorite, Pyrite and Quartz Infills, Very Strong, Moderatly Weathered, Grey	1	RUN		RUN #1 TCR=100% SCR=89% RQD=0%
		2	RUN		RUN #2 TCR=100% SCR=98% RQD=76%
		3	RUN		RUN #3 TCR=95% SCR=90% RQD=80%
264.6 / 8.7	End of Borehole. Water at 271.1 m on completion of drilling				

Ground water: ▼ at 271.1 m

Elevation scale: 273, 272, 271, 270, 269, 268, 267, 266, 265

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

+ 3, × 3: Numbers refer to Sensitivity

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

APPENDIX C

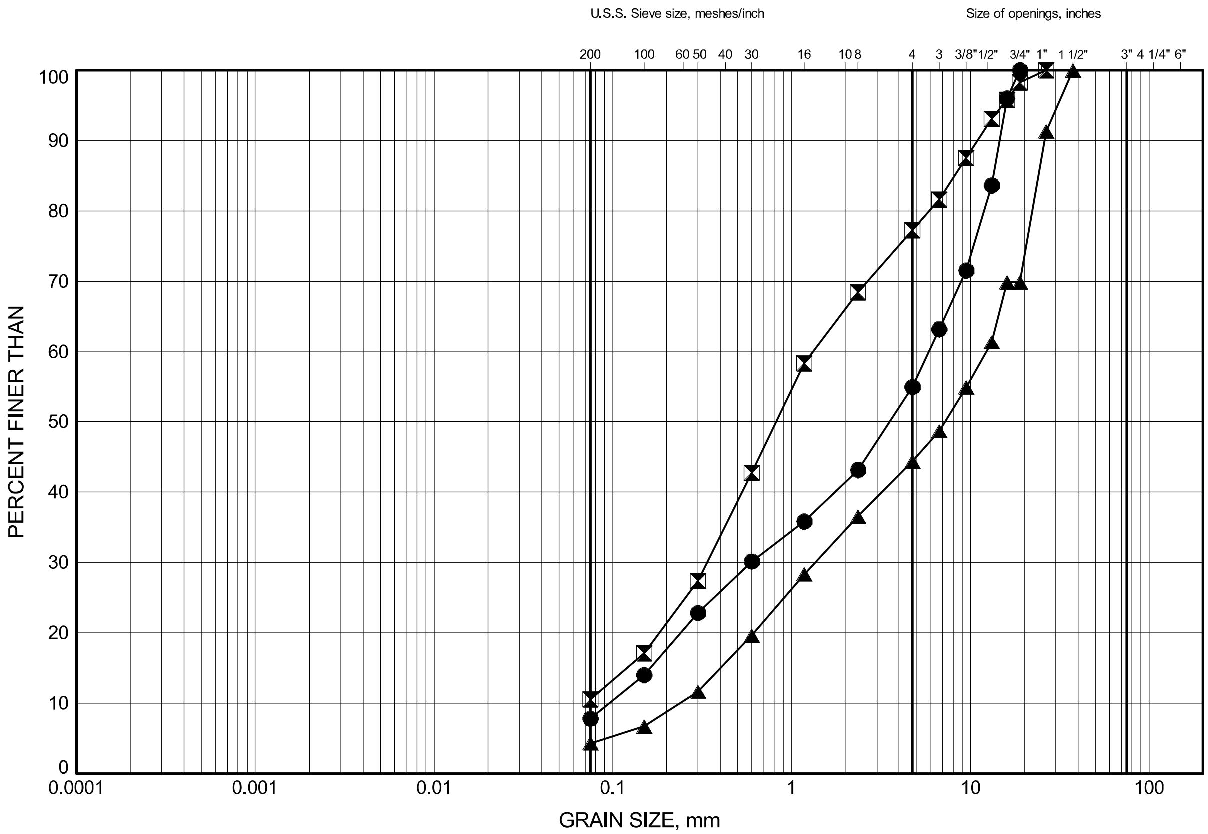
LABORATORY TEST RESULTS

Gull River North Bridge

GRAIN SIZE DISTRIBUTION

FIGURE C1

Fill



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	17-1	1.12	273.14
⧗	17-2	1.27	272.11
▲	17-2	4.42	268.96

Date October 2017

GWP# 5087-11-00

THURBER

Prep'd CM

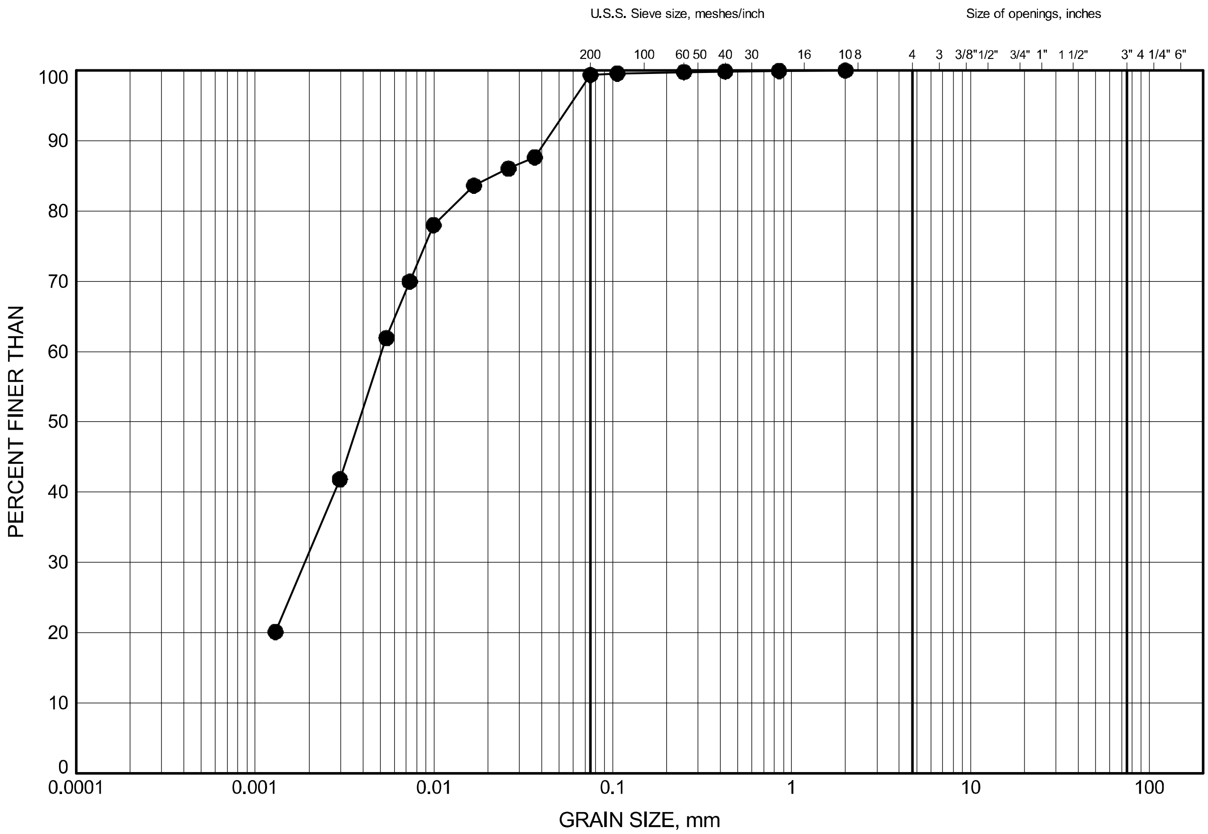
Chkd. FJG

Gull River North Bridge

GRAIN SIZE DISTRIBUTION

FIGURE C2

Clay (CL-CI)



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	17-1	4.11	270.14

Date October 2017

GWP# 5087-11-00

THURBER

Prep'd CM

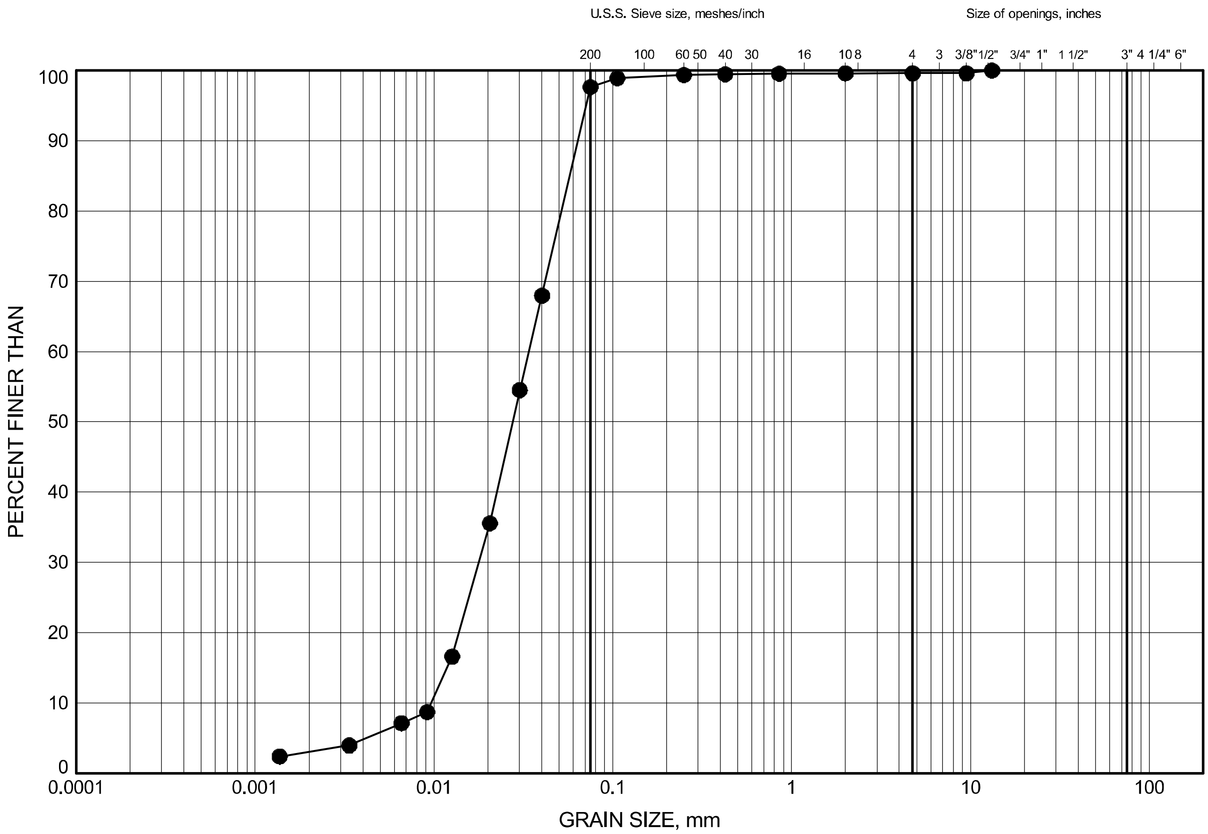
Chkd. FJG

Gull River North Bridge

GRAIN SIZE DISTRIBUTION

FIGURE C3

Silt (ML)



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	17-1	7.92	266.33

Date October 2017

GWP# 5087-11-00

THURBER

Prep'd CM

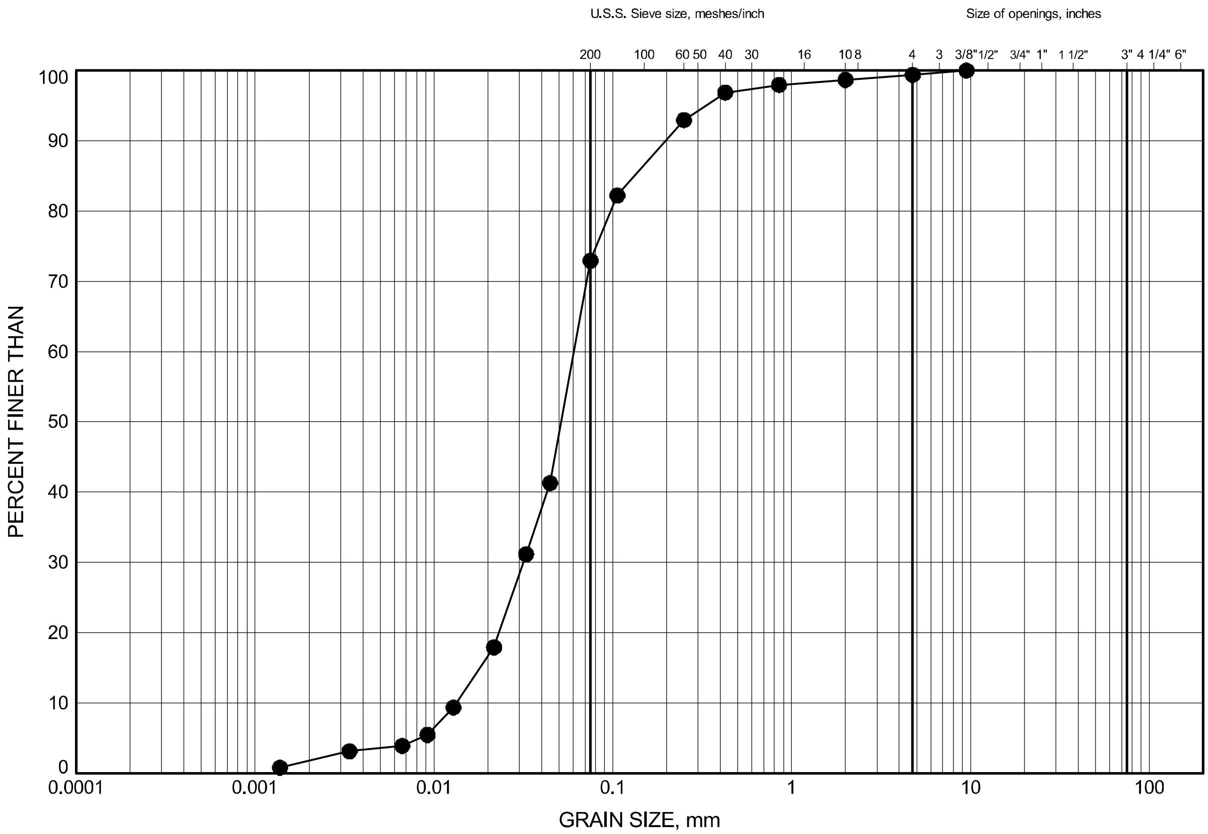
Chkd. FJG

Gull River North Bridge

GRAIN SIZE DISTRIBUTION

FIGURE C4

Sandy Silt (ML)



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	17-1	15.54	258.71

Date October 2017

GWP# 5087-11-00

THURBER

Prep'd CM

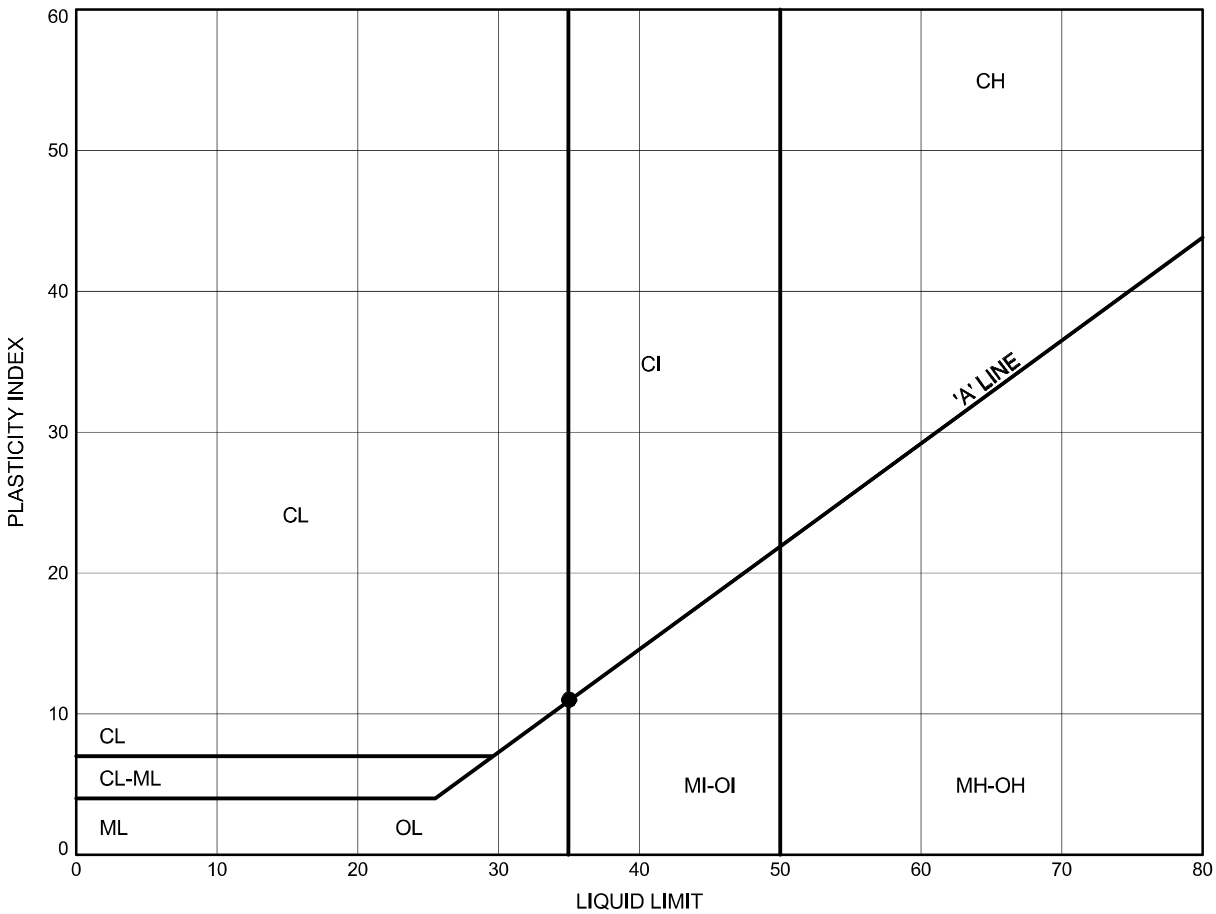
Chkd. FJG

Gull River North Bridge

ATTERBERG LIMITS TEST RESULTS

FIGURE C5

Clay (CL-CI)



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	17-1	4.11	270.14

THURBALT 16284 GULL RIVER BRIDGE NORTH.GPJ 20/10/17

Date October 2017

GWP# 5087-11-00

THURBER

Prep'd CM

Chkd. FJG

Borehole 17-2

Runs 1 and 2 (of 3)

Elevation 268.2 m to 266.2 m

THURBER ENGINEERING LTD.	**Foundation Investigation** **Gull River North Bridge** **Lutterworth Township, Ontario**	**GWP: 5087-11-00** **Project No.: 16284**

Borehole 17-2

Run 3 (of 3)

Elevation 266.2 m to 264.6 m

THURBER ENGINEERING LTD.

Foundation Investigation
Gull River North Bridge
Lutterworth Township, Ontario

GWP: 5087-11-00

Project No.: 16284

300 - 2319 St. Laurent Blvd
Ottawa, ON, K1G 4J8
1-800-749-1947
www.paracellabs.com

Certificate of Analysis

Thurber Engineering Ltd.

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

Client PO: 16284
Project: North Gull River Bridge
Custody: 14057

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

Order #: 1721508

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

Paracel ID	Client ID
1721508-01	17-1, SS5, 12'6"-14'6"
1721508-02	17-2, SS7, 15'6"-16'8"

Approved By:

Dale Robertson, BSc
Laboratory Director

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	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
Report Date: 01-Jun-2017
Client: Thurber Engineering Ltd.
Order Date: 26-May-2017
Client PO: 16284
Project Description: North Gull River Bridge

	Client ID:	17-1, SS5, 12'6"-14'6"	17-2, SS7, 15'6"-16'8"	-	-
	Sample Date:	09-May-17	10-May-17	-	-
	Sample ID:	1721508-01	1721508-02	-	-
	MDL/Units	Soil	Soil	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	74.9	86.4	-	-
General Inorganics					
Conductivity	5 uS/cm	321	585	-	-
pH	0.05 pH Units	7.32	7.13	-	-
Resistivity	0.10 Ohm.m	31.2	17.1	-	-
Anions					
Chloride	5 ug/g dry	149	210	-	-
Sulphate	5 ug/g dry	7	149	-	-

Certificate of Analysis

Client: Thurber Engineering Ltd.

Client PO: 16284

Report Date: 01-Jun-2017

Order Date: 26-May-2017

Project Description: North 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: North 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: North 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			

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 40-023 looking South (05/11/2018)

Figure 2: Roadway Platform at Bridge 40-023 looking North (05/10/2017)

Figure 3: East Side of Bridge Looking North (05/11/2017)

Figure 4: West Side of Bridge Looking North (05/11/2017)

APPENDIX E

GPR SURVEY REPORT

GEOPHYSICS GPR INTERNATIONAL INC.
6741 Columbus Road
Unit 14
Mississauga, Ontario
Canada L5T 2G9
Tel.: (905) 696-0656
Fax: (905) 696-0570
gprtor@gprtor.com
www.geophysicsgpr.com

September 12, 2017 Our File: T17029B

Stephen Peters, P.Eng
Geotechnical Engineer
Thurber Engineering Ltd
104, 2460 Lancaster Road
Ottawa ON
K1B4S5

RE: GPR Scanning of Gull River North Bridge

Geophysics GPR International Inc. was requested by Thurber Engineering Ltd to perform a high-resolution ground radar survey at the address above. The purpose of the survey was to search and delineate voids in the abutments of the bridge.

The survey was carried out on August 8, 2017.

This investigation utilized two ground penetrating radar antennas to generate a pseudo-cross section of the top 3m of the subsurface. A 1500 MHz antenna was utilized for it's high accuracy of the upper 50 to 70 centimeters. A 350 MHz antenna was also used for a deeper probe, reaching 3-4m below the surface, however, there is always a sacrifice of detail.

Profiles were collected perpendicular to the road with a line spacing of one meter.

<u>South Abutment</u>

Four types of anomalies were found in the southern abutment (west side in terms of compass) of the Gull River North Bridge. The positions of all features are shown in Figure 1.

Anomalies found with the 1500 MHz are given an 'A' designation because they are shallow and a 'B' designation is assigned to the larger 350 MHz antenna because the targets are deeper.

Figure 2 highlights both anomaly types observed with the 1500MHz antenna.

A1: This appears to be a bold reflection from the base of the asphalt when compared to the rest of the survey area. There are two possibilities, something was laid on top of the granular before the asphalt was laid, which seems unlikely or there is a thin void.

A2: The base of the asphalt within a specific area has the appearance of caving slightly but a void has not formed. If there is a void it is very thin.

B1: Is a possible location where there was a former repair of granular base to a depth of 50 cm.

B3: Is a possible void observed with the 350 MHz antenna at an approximate depth of at least 1.5 m. Figure 4 is an example radar profile image over this target. The top of the void is apparent and the anomalous zone is only centimeters away from an existing borehole that encountered a void at 2.3 meters deep and 60 cm thick. Void thickness is somewhat difficult to measure for ground radar because the radar pulse travels so quickly through the air so it is difficult to observe a reflection from the bottom of the void. The reflector from the top of the void is at 1.5 meters and the follow-up reflection could be 15 cm later. This distance must be multplied by 5 to account for the speed of the radar pulse in air so the void could be 75 cm thick. There is a stronger follow-up reflector that could double this thickness to 1.5 meters but this is quite uncertain.

There is roughly 7 meters of concrete (from the expansion joint) overlain by asphalt. The asphalt thickness is from 17 to 20 cm thick and the concrete is 15 to 17 cm thick.

The asphalt away from the abutment (> 7 meters from the expansion joint) is difficult to judge because there appears to be more than two lifts and if it is combined with the granular there may be 30 to 35 cm thick roughly.

Figure 3 is an example profile closer to the expansion joint that shows the asphalt over the concrete. From this area there is gleaned the following:

- 17 to 20 cm of asphalt
- depth of rebar is 7.5 to 10 into the concrete
- thickness of concrete 15 to 17 cm.
- spacing between rebars is 20 cm roughly.

North Abutment

Three types of anomalies were found in the northern abutment (east side in terms of compass) of the Gull River North Bridge. The positions of all features are shown in Figure 5.

Anomalies found with the 1500 MHz are given an 'A' designation because they are shallow and a 'B' designation is assigned to the larger 350 MHz antenna because the targets are deeper.

Figure 6 highlights both anomaly types observed with the 1500MHz antenna.

A3: The base of the asphalt within a specific area has the appearance of caving slightly within the granular base but if there is a void it is likely very thin.

A4: This appears to be a bold reflection from the base of the asphalt when compared to the rest of the survey area. There are two possibilities, something was laid on top of the granular before the asphalt was laid, which seems unlikely or there is a thin void.

B2: Is a possible void at an approximate depth of at least 1.5 m. It is not certain but it may be partly under the concrete portion of the abutment. This is more difficult to observe due to the reinforcing steel within the concrete. Once again void thickness is somewhat difficult to measure for ground radar because the radar pulse travels so quickly through the air but the follow-up reflector after the top-of-void reflector is 60 cm if we factor in the speed of the radar pulse through air. It is not certain how reliable a thickness measurement can be when the void is likely an irregular shape.

There is roughly 4 meters of concrete (from the expansion joint) overlain by asphalt. The asphalt thickness is from 17 to 20 cm thick and the concrete is 15 to 17 cm thick.

The asphalt away from the abutment (> 7 meters from the expansion joint) is difficult to judge because there appears to be more than two lifts and if it is combined with the granular there may be 30 to 35 cm thick roughly.

Figure 7 is an example profile closer to the expansion joint that shows the asphalt over the concrete. From this area there is gleaned the following:

- 15 to 20 cm of asphalt
- depth of rebar is 7 to 10 into the concrete
- thickness of concrete 15 to 17 cm, although not certain, could be double this
- spacing between rebars is 20 cm roughly.

If you have any questions please feel free to contact me.

Sincerely,

Milan Situm, P.Geo.
Manager

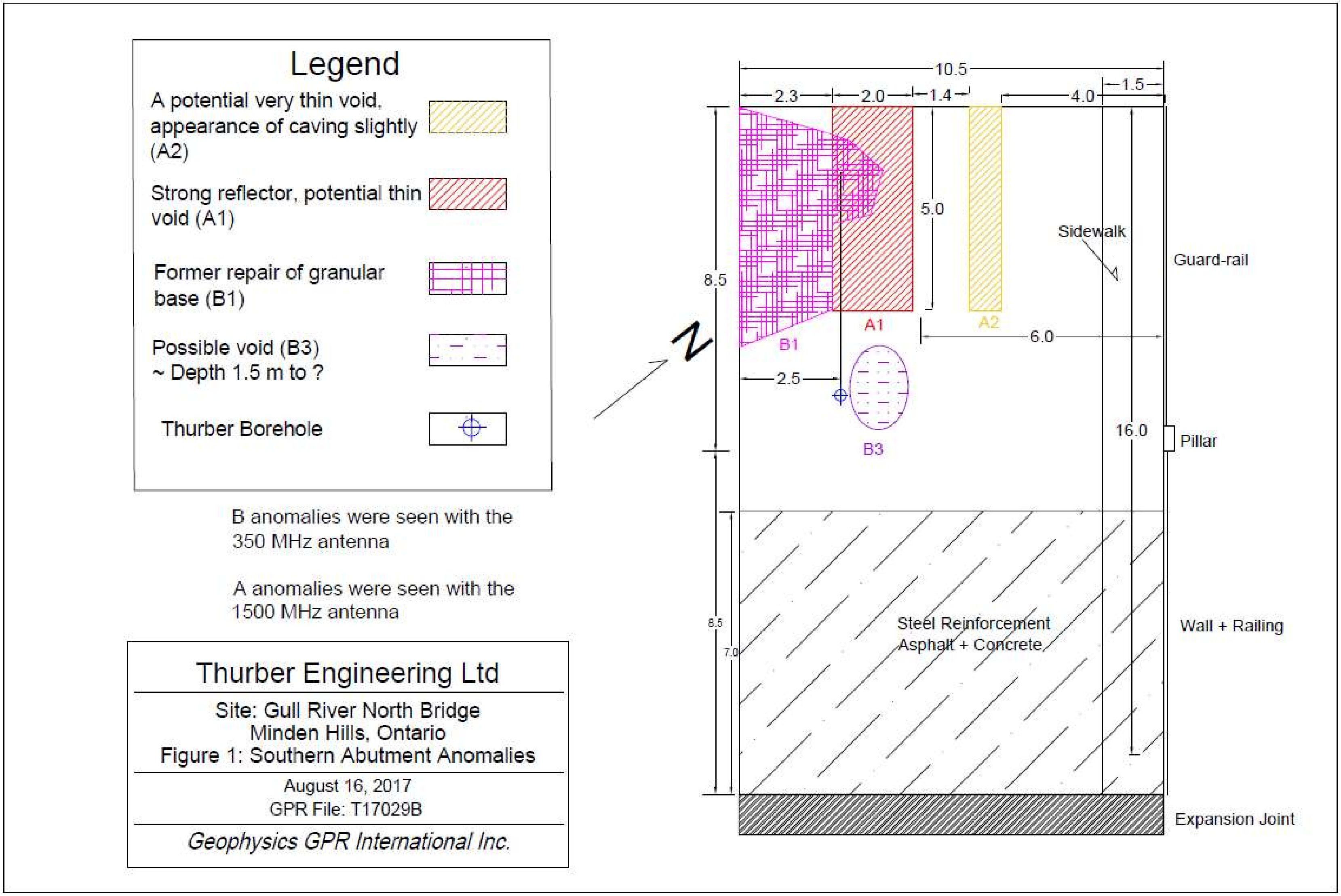


Figure 1: Gull River North Bridge, Southern section. Anomalies represented by coloured hatched areas.

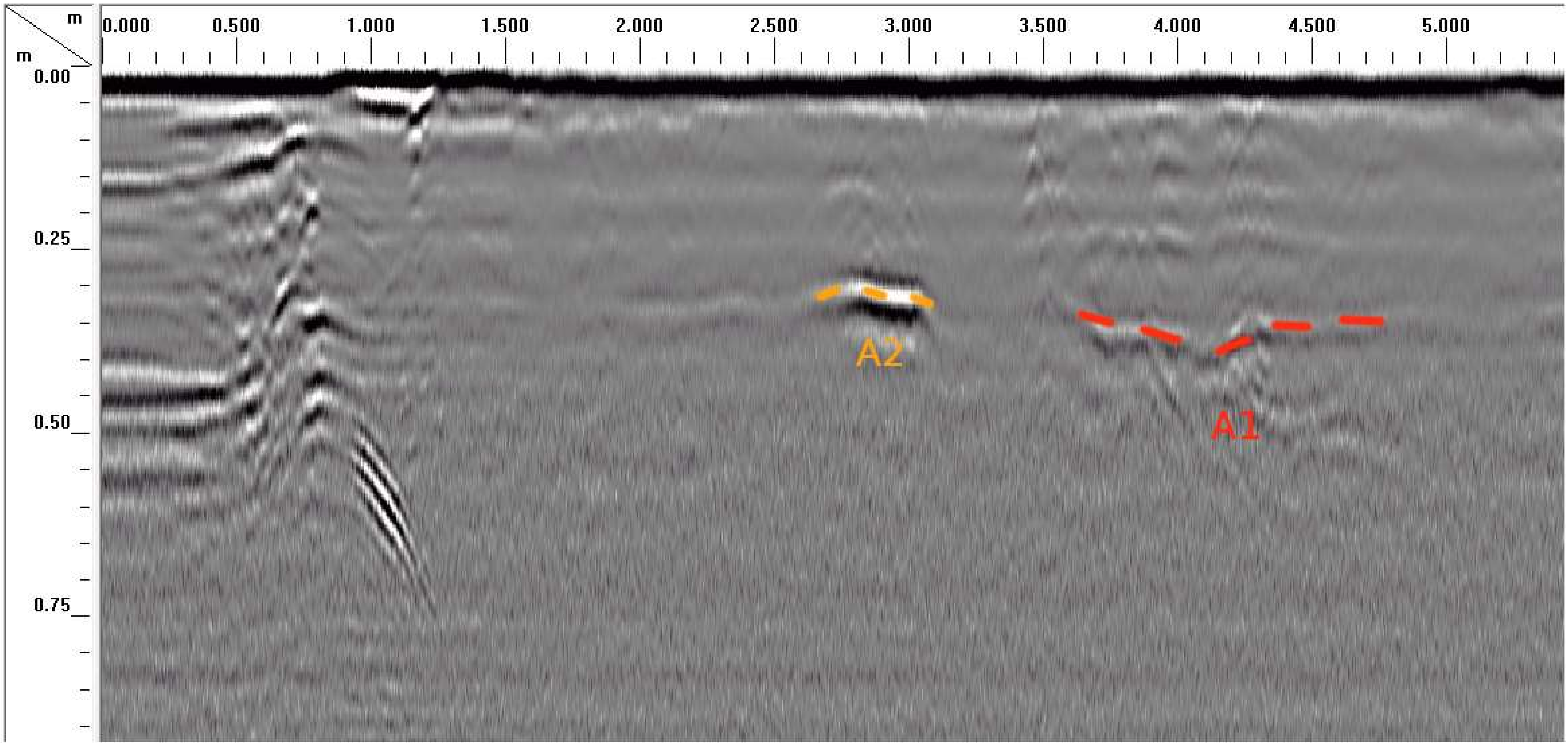


Figure 2: Data collected with the 1500 MHz antenna, showcasing anomalies A1 and A2.

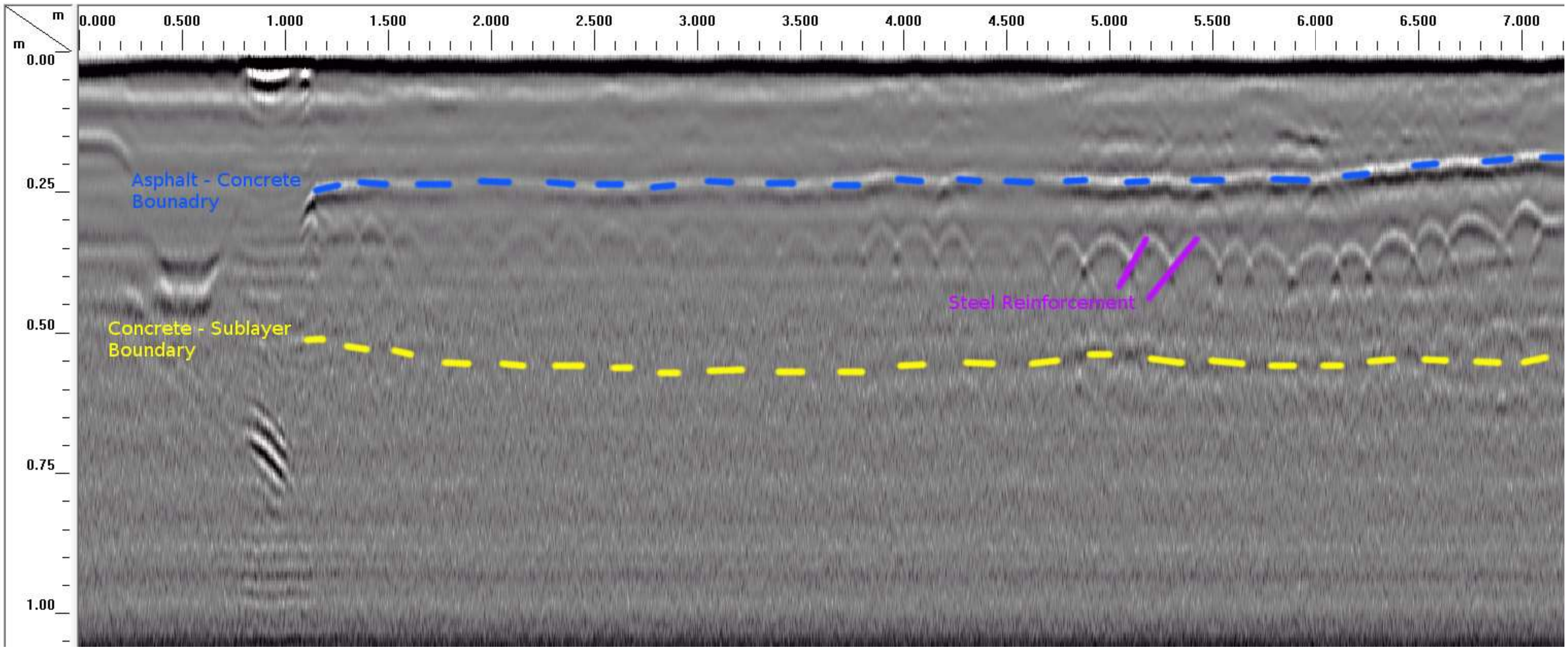


Figure 3: Example of steel reinforcement and subsurface boundaries from the Northern section. Evenly spaced steel reinforcement; 15cm spacing.

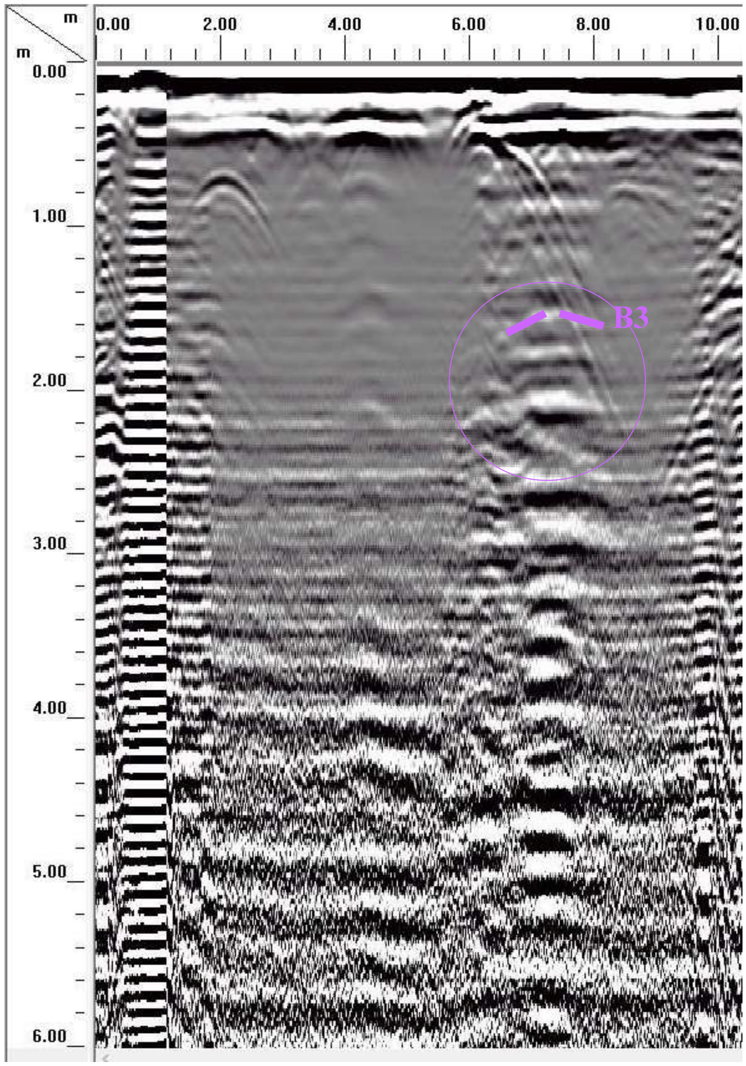


Figure 4: Data collected with the 350 MHz antenna, showcasing possible void at B3.

Legend

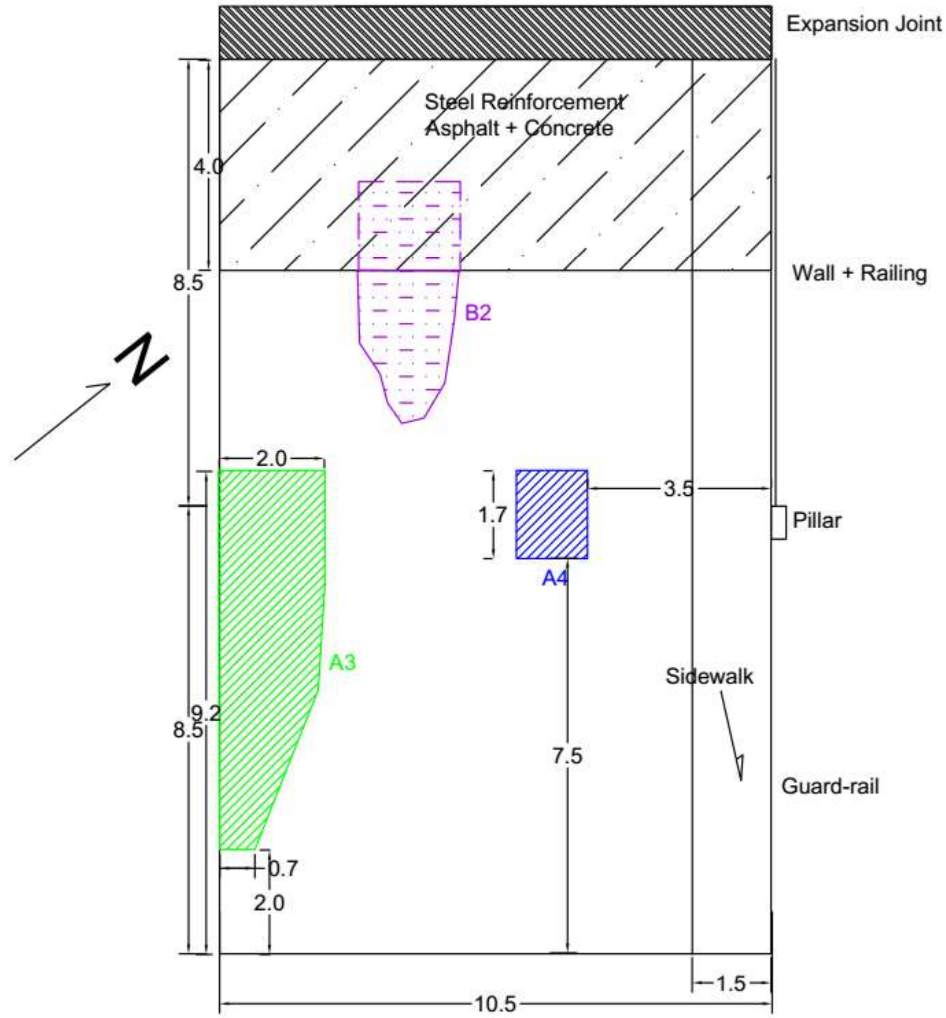
A potential very thin void, appearance of caving slightly within the granular base (A3)

Strong reflector, potential thin void (A4)

Possible void (B2)
~ Depth 1.5 m to ?

B anomalies were seen with the 350 MHz antenna

A anomalies were seen with the 1500 MHz antenna



Thurber Engineering Ltd

Site: Gull River North Bridge
Minden Hills, Ontario
Figure 5: Northern Abutment Anomalies

August 16, 2017
GPR File: T17029B

Geophysics GPR International Inc.

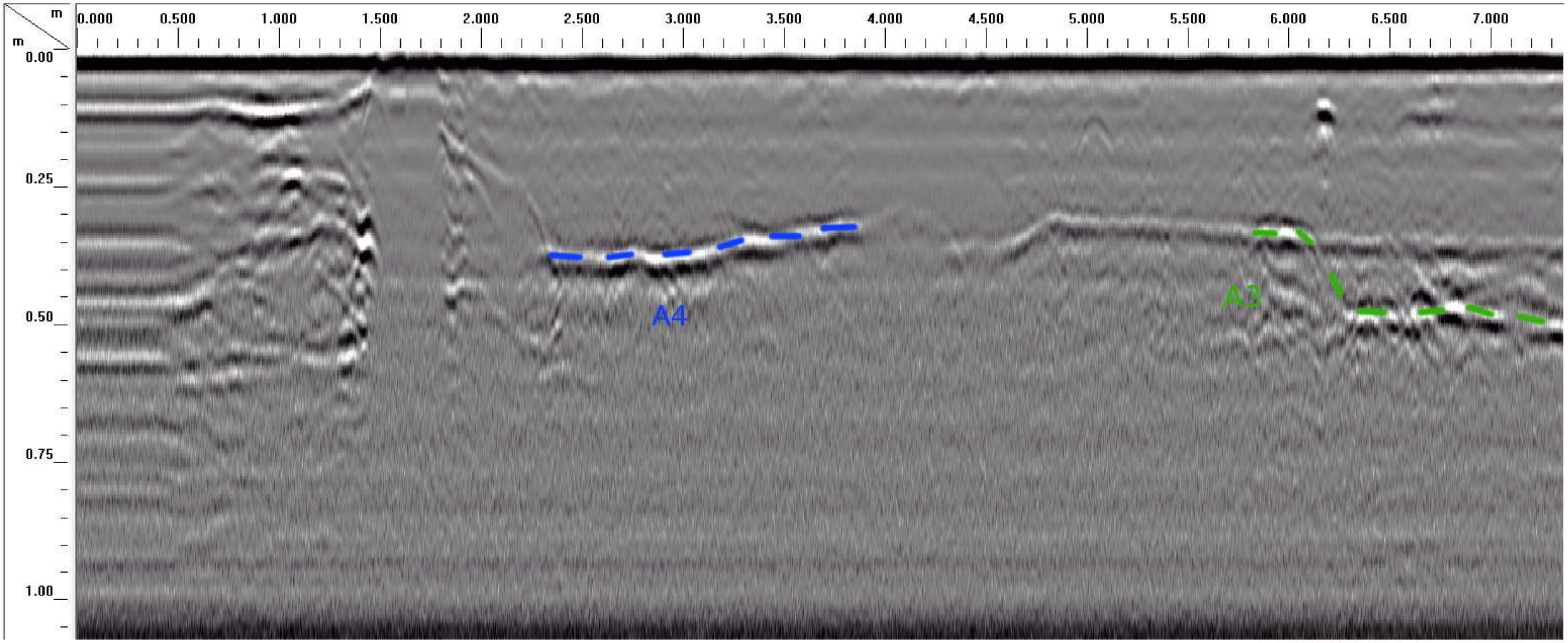


Figure 6: Data collected with the 1500 MHz antenna, showcasing anomalies A4 and A5.

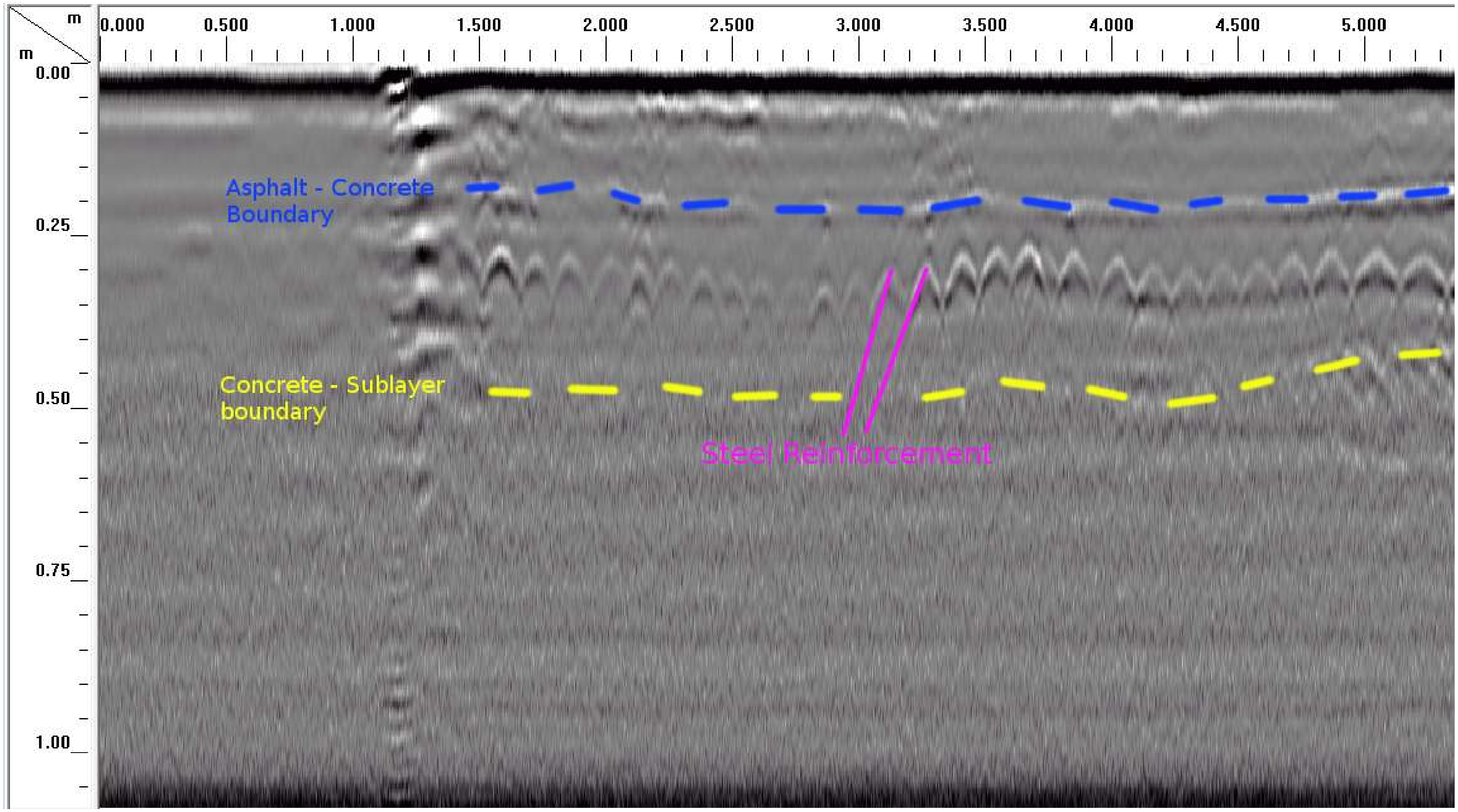


Figure 7: Example of steel reinforcement from the Southern segment, as well as subsurface boundaries. Evenly spaced steel reinforcement; 15cm spacing.

APPENDIX F

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
OPSS 1860	Material Specification for Geotextiles
SP 109S12	Amendment to OPSS 902, November 2010 – QVE, Backfilling Compaction, and Certificate of Conformance
SP 517F01	Standard Special Provision for Dewatering of Pipeline, Utility, and Associated Structure Excavation

SUGGESTED NON-STANDARD SPECIAL PROVISIONS

Suggested text for a NSSP on “Obstructions”

“The presence of cobbles and boulders within the fill as well as the presence of shallow bedrock in some locations may have an impact on excavations and on 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 (Sa(T), where T is the period in seconds) and peak ground acceleration (PGA) values are given in units of g (9.81 m/s^2). Peak ground velocity is given in m/s. Values are for "firm ground" (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