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FOUNDATION INVESTIGATION AND DESIGN REPORT
HAWKEYE LAKE EAST ROAD LRB BRIDGE REPLACEMENT
TOWNSHIP OF FOWLER, DISTRICT OF THUNDER BAY, ONTARIO
SITE NO. 48C-355**

ASSIGNMENT NO. 6015-E-0023

GEOCRES No.

Report

to

MINISTRY OF TRANSPORTATION ONTARIO

Lat: 48.680945

Long. -89.443967

Date: July 19, 2017
File: 17792

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PART 1: FACTUAL INFORMATION

1. INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted at the existing Hawkeye Lake East Road LRB Bridge on East Hawkeye Lake Road, in the District of Thunder Bay, Ontario.

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

Thurber was retained by the Ministry of Transportation Ontario (MTO) to carry out this foundation investigation under the MTO Retainer Agreement Number 6015-E-0023.

2. SITE DESCRIPTION

The site is located on Hawkeye Lake East Road, approximately 500 m east of Gilbride Road (Highway 591) in the Township of Fowler within the District of Thunder Bay, Ontario. The East Hawkeye Lake Road runs in an east-west direction at the bridge site. The existing structure is a 3.7 m long single span timber bridge and has an unknown construction date. The bridge superstructure consists of timber decking with gravel on top, timber curbs on each side and timber ties resting on 11 timber girders. The substructure consists of timber abutments resting on boulders (rock fill).

The creek that is crossed by the bridge, flows in a south to north direction. The land surrounding the site is largely forested and is generally of low relief, undulating with some bedrock ridges and plains.

Photographs in Appendix C show the general nature of the site and the existing bridge.

Based on published geological information, the bridge site lies within a ground moraine of sand and sandy till over granite bedrock.

3. INVESTIGATION PROCEDURES

The site investigation and field testing consisted of drilling and sampling four (4) boreholes (17-05, 17-05R, 17-06 and 17-06R) to depths ranging between 2.0 m and 6.8 m below the existing ground surface.

The original scope of work for the foundation investigation of this assignment included advancing two boreholes at the site. Boreholes 17-05 and 17-06 were originally drilled on April 19, 2017. However, the soil samples retrieved from the boreholes were lost by the shipping company during their transit from Thunder Bay to Thurber's laboratory. Therefore, a second mobilization was made by Thurber on June 6, 2017 during which time Boreholes 17-05R and 17-06R were drilled to obtain samples of the overburden soils for laboratory testing. This was discussed and agreed upon with the MTO Foundation Office.

Boreholes 17-05 and 17-05R were drilled on the west side of the existing bridge and Boreholes 17-06 and 17-06R were drilled on the east side.

The approximate locations of the boreholes are shown on the Boreholes Locations and Soil Strata Drawing included in Appendix D.

Utility clearances were obtained prior to the start of the drilling. The ground surface elevations for the boreholes were estimated from the topographic drawings provided to Thurber by MTO. The coordinate system MTM NAD 83, Zone 15 was used for these boreholes.

A rubber tired buggy mounted drill rig and rubber track mounted drill rig were used to advance the boreholes using hollow stem augers. Samples were obtained in the boreholes at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). Boreholes 17-05 and 17-06 were advanced into bedrock using an NQ core barrel.

Groundwater conditions in the open boreholes were observed throughout the drilling operations. The boreholes were backfilled in general accordance with MOE Regulation 903. Completion details of the boreholes are summarized in Table 3.1.

Table 3.1 – Borehole Completion Details

Borehole Number	Coordinates (MTM NAD 83, Zone 15)		Ground Surface Elevation (m)	Termination Depth (m)	Completion Details
	Northing (m)	Easting (m)			
17-05	5,393,745.1	345,742.3	446.3	6.8	Bentonite holeplug to 0.6 m, sand to 0.3 m and cold patch to surface
17-06	5,393,747.1	345,751.6	446.3	6.1	Bentonite holeplug to 0.6 m, sand to 0.3 m and cold patch to surface

Borehole Number	Coordinates (MTM NAD 83, Zone 15)		Ground Surface Elevation (m)	Termination Depth (m)	Completion Details
	Northing (m)	Easting (m)			
17-05R	5,393,743.6	345,744.4	446.3	2.0	Bentonite holeplug and cuttings to ground surface.
17-06R	5,393,749.7	345,750.4	446.4	2.1	Bentonite holeplug and cuttings to ground surface.

4. LABORATORY TESTING

All recovered soil samples from Borehole 17-05R and 17-06R were subjected to visual identification and natural moisture content determination. Selected soil samples were subjected to grain size distribution analyses (sieve and hydrometer) and point load testing was conducted on selected bedrock cores. The results of this testing program are summarized on the Record of Borehole sheets included in Appendix A and on the figures presented in Appendix B.

In order to assess the potential for sulphate attack on foundation concrete and the potential for corrosion, a sample of the native soil and a sample of the surface water from the creek upstream of the bridge were collected. The samples were submitted to SGS Canada Inc., a CALA accredited laboratory in Lakefield, Ontario, for analytical testing of corrosivity parameters and sulphate content. The results of the analytical testing are summarized in Section 6 below and are presented in Appendix B.

5. DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets included in Appendix A. Details of the encountered soil stratigraphy are presented in these sheets and on the “Borehole Locations and Soil Strata” drawing included in Appendix D. An overall description of the stratigraphy is given in the following paragraphs. However, the factual data presented in the Record of Borehole sheets governs any interpretation of the site conditions. It must be recognized that soil conditions may vary between and beyond the borehole locations.

In general, the subsurface conditions encountered consisted of a sand fill overlying a loose to compact native gravelly sand deposit which is in turn underlain by bedrock. More detailed descriptions of the individual strata are presented below.

5.1 Asphalt

All boreholes were drilled through about 25 mm of the existing asphalt pavement on Hawkeye Lake East Road. The thickness of asphalt may vary along the road.

5.2 Sand Fill

Sand fill containing some silt to silty, trace to some gravel, and trace organics was encountered below the asphalt in all boreholes. The fill was brown in color and extended to depths of approximately 0.6 m to 0.8 m below existing ground surface (Elevations 445.7 m to 445.5 m). Moisture contents of two samples of the sand fill were 6% and 9%.

The results of grain size analyses conducted on a selected sample of the fill are provided on the Record of Borehole sheets in Appendix A and plotted in Figure B1 of Appendix B. The results are summarized below.

Soil Particle	Percentage (%)
Gravel	14
Sand	62
Silt	20
Clay	4

5.3 Clayey Silt

An interlayer of clayey silt was encountered in Borehole 17-05 below the fill at a depth of 0.8 m. The clayey silt layer was approximately 1.5 m thick and contained some sand and trace gravel. Cobbles (within the clayey silt) were inferred from split spoon sampler bouncing and hard augering between depths of 0.8 m and 1.4 m.

5.4 Gravelly Sand

Gravelly sand was encountered beneath the fill in Boreholes 17-06, 17-05R and 17-06R and below the clayey silt in Borehole 17-05 at depths ranging between 0.6 m and 2.3 m. The sand extended to bedrock surface in Boreholes 17-05 and 17-06 at depths of 3.8 m and 3.0 m below ground surface (Elevation 442.5 m and 443.3 m). Boreholes 17-05R and 17-06R were terminated at depths of 2.0 m and 2.1 m (Elevations 444.3 m) due to split spoon bouncing and/or auger refusal. The gravelly sand contained trace to some silt and was brown to grey in colour.

SPT 'N' values in the gravelly sand deposit ranged from 3 to 34 blows per 0.3 m of penetration, indicating the a very loose to dense relative density, predominantly loose to compact.

Cobbles and boulders were inferred within the deposit in Borehole 17-06, 17-05R and 17-06R due to split spoon bouncing or auger refusal at depths ranging between 0.8 m and 2.1 m.

Measured moisture contents of selected sample of the gravelly sand ranged from 8% to 23%.

The results of grain size analyses conducted on a sample of the gravelly sand are provided on the Record of Borehole sheets in Appendix A and plotted in Figure B2 of Appendix B. The results are summarized below.

Soil Particle	Percentage (%)
Gravel	32
Sand	61
Silt and Clay	7

5.5 Bedrock

Auger refusal on Granite bedrock was encountered and proven by coring in Boreholes 17-05 and 17-06. The bedrock was encountered at depths of 3.8 m and 3.0 m (Elevations 442.5 m and 443.3 m), respectively.

The bedrock was described as grey to white granite. Total Core Recovery (TCR) in the bedrock were 97% and 100% with Solid Core Recovery (SCR) ranging from 77% to 98%. The Rock Quality Designation (RQD) determined from the recovered cores generally ranged from 77% to 100%, indicating good to excellent rock quality. Average unconfined compressive strengths (UCS) of the rock ranged between 172 MPa to 220 MPa based on correlations with the point load tests (PLT), indicating the bedrock is very strong.

5.6 Groundwater Conditions

Where possible, water levels were monitored in the open boreholes during drilling operations and the results are shown in Table 5.1.

Table 5.1 – Water Level Measurements

Borehole	Date	Water Level		Comment
		Depth (m)	Elev. (m)	
17-05	April 19, 2017	1.2	445.1	In open borehole
17-05R	June 6, 2017	1.4	444.9	In open borehole
17-06R	June 6, 2017	1.4	445.0	In open borehole

The normal and the high creek water levels were reported to be about 445.0 m and 445.7 m at the site.

The water levels measured in the boreholes are short-term readings and seasonal fluctuations of the groundwater and creek level are to be expected. In particular, the water level may be at a higher elevation after the spring snowmelt or after periods of heavy rainfall.

6. CORROSIVITY AND SULPHATE TEST RESULTS

A sample of the native soil and a sample of the surface water from the creek were tested for corrosivity parameters and sulphate content. The results of the analytical tests are shown in Table 6.1. The laboratory certificates of analysis are presented in Appendix B.

Table 6.1 – Analytical Test Results

Parameter	Units (Soil)	Units (Water)	Test Results	
			17-06R SS#2	Creek Water
Sulphide	%	mg/L	0.02	<0.006
Chloride	µg/g	mg/L	40	1.4
Sulphate	µg/g	mg/L	26	1.9
pH	pH Units	pH Units	7.89	6.71
Electrical Conductivity	µS/cm	µS/cm	89	40
Resistivity (Calculated)	ohm.cm	ohm.cm	11200	25000
Redox Potential	mV	mV	285	213

7. MISCELLANEOUS

Borehole locations were selected and established in the field by Thurber Engineering Ltd. The coordinates and the ground surface elevations for the boreholes were established based on topographic survey information provided by MTO.

Thurber obtained utility clearances for the borehole locations prior to drilling.

RPM Drilling of Thunder Bay, Ontario supplied a track-mounted CME-45 drill rig, a rubber tire buggy mounted drill rig and conducted the drilling, sampling and in-situ testing operations for the boreholes. The drilling operations were supervised by Mr. Amir Fereidouni and Mr. Ryan McCourt of Thurber.

Overall supervision of the field program and interpretation of the data was carried out by Mr. Cory Zanatta, EIT.

The report was prepared by Mr. Cory Zanatta, EIT., and Mr. Mehdi Mostakhdemi, P.Eng., and reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.



60 YEARS

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File No.: 17792
E file: H:\17000-17999\17792 NWR Foundations Retainer - Assignment 3 - 4 Bridges\Reports & Memos\Hawkeye Lake Bridge\17792 - Hawkeye Lake Bridge_Draft FIDR.docx

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Date: July 12, 2017
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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

8. GENERAL

This report provides an interpretation of the geotechnical data in the factual report, and presents foundation design recommendations for the proposed LRB Bridge replacement on Hawkeye Lake East Road within the township of Fowler in the District of Thunder Bay, Ontario.

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 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 site is located on Hawkeye Lake East Road, approximately 500 m east of Highway 591 in the Township of Fowler, Ontario. The Hawkeye Lake East Road runs in an east-west direction at the bridge site. The existing structure is a 3.7 m long single span timber bridge and has an unknown construction date. The bridge superstructure consists of timber decking with gravel on top, timber curbs on each side and timber ties resting on 11 timber girders. The substructure consists of timber abutments resting on boulders (rock fill).

The November 2014 structural inspection report indicates that the bridge is in overall poor conditions due to rotting and insects inhabitation of timber girders and abutment members.

8.1 Structure Replacement Alternatives

This section presents discussions on proposed replacement options and foundation alternatives, and provides foundation design recommendations for the replacement of the Hawkeye Lake East Road LRB Bridge.

The Structural Design Report (SDR), prepared by Hatch, has discussed four (4) replacement options for this site:

- A single Cell Precast Concrete box (closed) culvert;
- Corrugated Steel Pipe (CSP) Culvert(s);
- An Open footing (metal) Culvert on precast footings; and
- A 9.3 m long Lessard Modular Bridge with sheet pile wall abutments and wingwalls. The replacement bridge will be longer than the existing bridge. A 300 mm grade raise is proposed for the modular bridge option.

In general, the foundation soil stratigraphy at the site consists of a sand fill over loose to compact gravelly sand underlain by the bedrock. The bedrock surface varies between Elevations 442.5 m and 443.3 m. The short-term water level in the boreholes ranged from 444.9 m to 445.1 m. The normal and the high-water levels in the creek are reported to be 445.0 m and 445.7 m. The groundwater level will likely reflect the creek water level.

A comparison of the replacement options based on their respective advantages and disadvantages is included in Appendix E. The SDR recommended the modular bridge option as the preferred alternative and does not consider the other two alternatives to be feasible, based on the following considerations:

- Both Pre-cast box culvert and open footing culvert would require significant road grade raise;
- The depth to the bedrock is shallow at the site. Placement of open footing culverts with precast foundations would require bedrock excavation; and
- The open footing culvert option would require stream diversion and in-water work.

Recommendations for the design and installation of modular bridge are presented below. The other replacement options are not discussed further.

9. STRUCTURE FOUNDATION DESIGN FOR MODULAR BRIDGE

Based on the subsurface conditions at this site, both shallow and deep foundation options have been considered for support of the abutments of a modular bridge at the site. A summary of the advantages and disadvantages associated with each option is provided below:

- **Footings on an engineered granular pad in the approach embankments:** Footings placed on at least 1.0 m thick compacted engineered granular pad are feasible for support of abutments for the proposed bridge replacement. Recommendation for design of shallow foundations is provided in the following sections.
- **Deep Foundations:** Deep foundations (Driven steel H-piles or drilled shafts/caissons) are not recommended for support of new abutments at the site due to the relatively shallow depth to the bedrock surface (3.0 m to 3.8 m) and the potential presence of cobbles/boulders within the gravelly sand. This option is not discussed in the report further.

9.1 Spread Footings on Engineered Fill

The preliminary GA drawing for the modular bridge option indicates a footing founding elevation of about 445.3 m. At that elevation, both abutments will be founded in the existing loose to compact native gravelly sand or clayey silt interlayer. The following construction sequence is recommended for the footing constructed on engineered fill:

1. Excavate to remove all timber abutments, boulders, rock protection and other deleterious material from the footprint of the new foundations.
2. The minimum depth of excavation must accommodate the concrete foundation slab and at least 1.0 m of engineered fill below the slab (about Elevation 444.8 m), as described below.
3. The subgrade below the 1 m engineered fill pad should be inspected to detect and sub-excavate soft spots and confirm that the subgrade is uniformly compacted.
4. The dimensions of the base of the excavation must be determined by assuming a granular pad 1.0 m wider than the footing at the level of the footing base and projecting outward at 2H:1V.

The excavation for the footings will be conducted after installation of the sheet pile abutment walls. The base of the excavation may be below the creek water level and/or the groundwater table at the site. Temporary dewatering within the area of the proposed foundation footprint may be required depending on the groundwater level at the time of construction.

The engineered fill pad should consist of OPSS Granular "A" or Granular B Type II placed in 150 mm lifts and compacted to 100% of its SPMDD at $\pm 2\%$ of optimum moisture content. The top of the engineered fill pad should be at least 1 m wider than the footprint of the spread footing at the underside of the footing and must be at least 1.0 m thick.

Excavations for the engineered pad construction and footing placement may require the existing superstructure to be removed or temporarily supported during construction.

The following axial geotechnical resistances may be used for design of 1.5 m to 2 m wide spread footings of founded at or below Elevation 445.8 m on at least 1 m thick engineered fill founded at or below Elevation 444.8 m:

- Factored Geotechnical Resistance at Ultimate Limit State (ULS) of 200 kPa
- Factored Geotechnical Resistance at Serviceability Limit State (SLS) of 150 kPa for a settlement of 25 mm.

The consequence factor of 1 was utilized in this design adopting a “typical” consequence level. The geotechnical resistance factor of 0.5 for bearing, and 0.8 for settlement (both adopted for “typical” degree of understanding) were used to obtain the above values, in accordance with Section 6.9 of the Canadian Highway Bridge Design Code (CHBDC) 2014.

The ULS resistance and settlement are dependent on the footing size, depth of burial, configuration and applied loads; the geotechnical resistances should therefore be reviewed if the foundation width or founding elevation differs significantly from that given above.

The lateral resistance developed along the base of the footings founded on the engineered fill should be computed using an ultimate friction coefficient of 0.6 for cast-in-place concrete and 0.5 for pre-cast concrete. The friction coefficients provided above are “ultimate” values and require a degree of sliding movement to occur to fully mobilize the resistance.

9.2 Frost Protection

The depth of frost penetration at this site is approximately 2.2 m as per Ontario Provincial Standard Drawing (OPSD) 3090.100 (Foundation Frost Depths for Northern Ontario). However, concrete slab foundations for modular bridges may be founded on an engineered fill pad with a minimum embedment of 0.5 m.

10. EXCAVATION AND GROUNDWATER CONTROL

The bridge design should attempt to keep the base of the temporary excavation above the creek water level. Where excavations extend below the water level, the Contractor must implement effective dewatering procedures.

All excavations must be carried out in accordance with the requirements of the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the existing fill and the native clayey silt may be classified as Type 3 soil. The native sand below the water table may be classified as Type 4 soil.

The excavation and backfilling for foundations must be carried out in accordance with OPSS 902.

The dewatering system on site should conform to OPSS 518 (Construction Specifications for Control of Water from Dewatering Operation). The design of an effective dewatering system that may be required is the responsibility of the Contractor and the Contract Documents must alert him to this responsibility and the need to engage a dewatering specialist. Suggesting wording for an NSSP in this regard is included in Appendix F. Additional assessment should be made to determine if a Permit to Take Water (PTTW) is required.

Stockpile of excavated materials and heavy construction equipment should be kept at least the same horizontal distance from the edge of excavation as the depth of the excavation to prevent local instabilities.

11. TEMPORARY SUPPORT SYSTEM

If required, the temporary excavation support system must be designed and constructed in accordance with OPSS 539. The protection system should be designed for Performance Level 2 (maximum 25 mm horizontal deflection). The Contractor should select the wall type and design taking into account the soil conditions encountered in the boreholes. Designing of sheet pile for temporary roadway protection is likely to encounter obstruction such as cobbles and boulders in the embankment fill and in the native gravelly sand deposit. Soldier pile and lagging is the another option.

The following parameters may be used for design of the temporary shoring system:

γ	=	21 kN/m ³	(bulk unit weight for fill and native sand)
γ'	=	11 kN/m ³	(submerged unit weight for fill and native sand)
K_a	=	0.33	(active pressure coefficient for fill and native sand)
K_p	=	3.0	(passive pressure coefficient for fill and native sand)

Full hydrostatic pressure should be considered assuming a water level at least equal to the design stream water level.

The design of temporary protection system is the responsibility of the Contractor. The actual pressure distribution acting on the protection/shoring system is a function of the construction sequence and the relative flexibility of the retaining system, and these factors have to be considered when designing the shoring system. All protection systems should be designed by a Professional Engineer experienced in such designs, who will determine an appropriate support system.

12. LATERAL EARTH PRESSURES

Backfill to the abutments for the modular bridge should consist of free-draining, non-frost susceptible granular materials such as Granular A or B Type II conforming to the requirements of OPSS.PROV 1010. Reference should be made to the backfill arrangements stipulated in OPSD 3101.150, as appropriate.

Earth pressures acting on the structure may be assumed to be distributed triangularly and to be governed by the characteristics of the abutment backfill. For a fully drained condition, the pressures should be computed in accordance with the CHBDC but generally are given by the expression:

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

Where: p_h = horizontal pressure on the wall at depth h (kPa)
 K = coefficient of lateral earth pressure (see Tables 12.1 and 12.2)
 γ = unit weight of retained soil (see Tables 12.1 and 12.2)
 h = depth below top of fill where pressure is computed (m)
 q = value of any surcharge (kPa)

Earth pressure coefficients for backfill to the abutment wall are dependent on the material used as backfill. Typical values are given in Table 12.1.

Table 12.1 – Coefficients of Lateral Earth Pressure (K)

Loading Condition	OPSS Granular A or Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I or Type III $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$	
	Horizontal Backfill	Sloping Backfill (2H:1V)	Horizontal Backfill	Sloping Backfill (2H:1V)
Active K_A (Unrestrained Wall)	0.27	0.38*	0.31	0.46*
At-rest K_0 (Restrained Wall)	0.43	-	0.47	-
Passive K_P	3.7	-	3.3	-

* For wing walls

The use of a material with a high friction angle and low active pressure coefficient (e.g. Granular A, Granular B Type II) is preferred as it results in lower earth pressures acting on the wall.

The active and passive earth pressure coefficients in Table 12.1 are “ultimate” values and require certain movements for the respective conditions to be mobilized. The values to use in design can

be estimated from Figure C6.16 in the Commentary to the Canadian Highway Bridge Design Code (CHBDC).

In general the lateral earth pressure applied to a retaining structure depends on the lateral movement of the structure to activate active, passive or at rest earth pressure. If the wall support does not allow lateral movement (restrained wall), at rest earth pressures should be assumed for design. If the wall support allows lateral movements (unrestrained stem), active earth pressure should be used in the design of the wall. The minimum lateral movement to allow active pressures to develop within the backfill is outlined in Section C6.12 of the Commentary on CHBDC 2014.

In accordance with Clause 6.12.3 of the CHBDC 2014, a lateral pressure representing the compaction surcharge should be added in design of retaining walls. The magnitude of the lateral pressure should be 12 kPa at the top of fill which linearly decreases to zero at a depth of 1.7 m (for Granular B Type I) or at a depth of 2.0 m (for Granular A or B Type II). If the wall is retaining sloping backfill, appropriate earth pressure parameters from Table 12.1 for sloping backfill should be used.

12.1 Sheet Pile Abutment Walls for Modular Bridge Option

The SDR considers the use of sheet pile wall abutments (and wingwalls) for the modular bridge configuration. The sheet piles will provide containment and resistance to lateral earth pressures applied from the approach fill. The sheet piles should be installed behind the existing timber abutments or the timber abutments should be removed prior to sheet pile installation. Sheet piles may be difficult to drive at this site since cobbles and/or boulders are anticipated to be present within the embankment fill and gravelly sand at the site.

The stability of the sheet pile wall system (including but not limited to global stability, basal stability, anchor design, bending) should be evaluated by the wall designer and the depths of penetration (or sheet pile tip elevations) be determined for a minimum factor of safety of 1.5 using the geotechnical design parameters presented in Table 12.2. The lateral impact of the footing foundation loads on the sheet pile wall system should be taken into account in the design if shallow spread footings on engineered fill is considered to support the modular bridge foundations.

The interaction between the sheet pile wall and the adjacent soil may be analysed using a soil-spring model and a coefficient of horizontal subgrade reaction, k_s . In cohesionless soils, the horizontal subgrade reaction per linear meter varies with depth and can be calculated as follows:

$$k_s = n_h z \quad (\text{kN/m}^3)$$

where z = depth of embedment in metres

n_h = coefficient related to soil density, see table below (kN/m^3)

For soil-spring analysis, the spring constant, K_s , may be obtained by the expression $K_s = k_s L$ (kN/m), where k_s is the coefficient of horizontal subgrade reaction (kN/m³) and L is the length (m) of the pile segment or element used in the analysis. The ultimate passive pressure mobilized per unit length of the pile should not exceed the value provided below:

$$P_{ult} = k_p \cdot \gamma' \cdot z$$

The coefficients of passive earth pressure (K_p) are provided for horizontal ground surface in front of the sheet pile wall. For sloping ground in front of the sheet pile wall, the recommended values for the coefficients of passive earth pressure (K_p) should be reduced.

Table 12.2 – Soil Parameters for Sheet Pile Analysis

Foundation Element (Reference Borehole)	Soil Unit	Elevation (m)		γ' (kN/m ³)	K_a	K_0	K_p	n_h (kN/m ³)
		Top	Bottom					
West Abutment (17-05 and 17-05R)	Existing Fill	446.3*	445.5	21	0.33	0.5	3.0	3,000
	Gravelly Sand above Water Table	445.5	445.1	21	0.33	0.5	3.0	3,000
	Gravelly Sand below Water Table	445.1	442.5	11	0.33	0.5	3.0	2,000
East Abutment (17-06 and 17-06R)	Existing Fill	446.3*	445.7	21	0.33	0.5	3.0	3,000
	Gravelly Sand above Water Table	445.7	445.0	21	0.33	0.5	3.0	3,000
	Gravelly Sand below Water Table	445.0	443.3	11	0.33	0.5	3.0	2,000

Note: * Elevation of top of sheet pile varies.

In general, backfill to the sheet pile walls should be in accordance with OPSS 902 and should consist of Granular A, Granular B Type II or III material. All granular material should meet the specifications of OPSS.PROV 1010. Compaction equipment to be used adjacent to retaining structures should be restricted in accordance with OPSS.PROV 501.

Design of the permanent sheet pile walls must consider environmental conditions such as road salts or fluctuating water levels that may cause corrosion and reduce the service life of the structure. The native soils in front of the sheet piles should be protected from creek erosion so that the sheet piles do not lose lateral support.

12.2 Cellular Bin Type Abutment Walls

The bedrock surface elevation at the site varies between 442.5 m and 443.3 m. As indicated earlier the designing of sheet piles is likely to be difficult at this site due to the presence of cobbles and boulders in the fill and the native soils. Consideration may be given to the use of cellular bin

type abutment walls. This type of walls, if used, may be supported on an engineered granular pad resting on the compact gravelly sand or the bedrock subgrade. Any topsoil/organic must be removed from the wall subgrade and replaced with granular fill compacted as per OPSS 501. The engineered pad is required to provide subgrade uniformity along the wall alignment. This pad should consist of compact Granular A materials and have a minimum thickness of 0.5 m. Local sub-excavation may be required to accommodate the design grades or to remove unsuitable subgrade materials. The walls should be founded at or below Elevation 445.4 m and the base of the granular pad should be founded at or below 444.9 m. For a 1 m to 2 m high wall founded on a 0.5 m thick granular pad a factored geotechnical resistance at ULS of 200 kPa and a geotechnical resistance at SLS of 150 kPa (for 25 mm of settlement) may be used for design.

Resistance to lateral forces / sliding resistance between wall base and the underlying engineered gravel pad should be evaluated in accordance with the CHBDC 2014 assuming an ultimate coefficient of friction of 0.4.

Lateral earth pressures acting on the walls should be computed as described in Section 12. If the wall is retaining sloping backfill, appropriate earth pressure parameters for sloping backfill should be used.

The base of the bin wall should be protected from scour and erosion so that the wall is not undermined.

13. SEISMIC CONSIDERATIONS

The new structure is considered as a Seismic performance category 1 based on Table 4.10 of the CHBDC 2014; therefore, it does not need to be analyzed for seismic loads regardless of its importance and geometry in accordance with Section 4.4.5.1 of the CHBDC 2014.

14. EMBANKMENT RESTORATION

The existing road embankment slopes appear to be performing satisfactorily. Provided that the embankment is reconstructed at the same slope inclination as the existing embankment, but not steeper than 2H:1V, the restored embankment slope should remain stable.

It is anticipated that there may be up to 300 mm of grade raise at this site for the bridge replacement, and for such a small grade raise settlement of the embankment is not a concern. Any settlement due to changes in the bridge configuration is expected to be less than 25 mm.

Embankment restoration after completion of the bridge replacement should be carried out in accordance with OPSS.PROV 206. The embankment material may consist of imported Granular A, Granular B Type II, or Granular B Type III material. Alternatively, the existing

embankment fill may be used, provided it is unfrozen, free of organics, and at a moisture content that is suitable for compaction.

In general, surface vegetation, topsoil, organic deposits, disturbed material or otherwise loose/soft soils should be stripped from the foundation footprints, and within the embankment footprints. Inspection and approval of the foundation surfaces by qualified geotechnical personnel should be conducted.

15. SCOUR AND EROSION PROTECTION

Erosion protection should be provided at the bridge abutment. Design of the erosion protection measures should consider hydrologic and hydraulic factors and should be carried out by specialists experienced in this field.

Typically, rock protection should be provided over all surfaces with which creek water is likely to be in contact. A vegetation cover should be established on all other exposed earth surfaces to protect against surficial erosion in general accordance with OPSS.PROV 804.

16. CORROSION & SULPHATE ATTACK POTENTIAL

The results of the corrosivity and sulphate analytical tests conducted on the native soil and the creek water indicate the following:

- The potential for sulphate attack on concrete foundations from the surrounding soil or surface water is considered to be negligible due to the low concentration of sulphate in the samples tested.
- The potential for corrosion on metal structural elements is considered to be very mild to mild.
- The effect of road de-icing salt should be considered in the choice of concrete and metal structure elements.

17. CONSTRUCTION CONCERNS

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

- Seasonal fluctuations of the groundwater and creek level are to be expected. In particular, the water level may be at a higher elevation after the spring snowmelt or after periods of heavy rainfall, which may impact the construction.

- The Contractor's selection of construction equipment and methodology should include assessment of the capability of the existing embankment to support the proposed construction equipment and any temporary structures or fill (i.e., as a pad for crane support). Site conditions may limit the type of equipment suitable for use during construction. The design and safety of any temporary works is the responsibility of the Contractor.

18. CLOSURE

Engineering analysis and preparation of this report was carried out by Mr. Mehdi Mostakhdemi, M.Sc., P.Eng. The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.



Thurber Engineering Ltd.

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Senior Geotechnical Engineer

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Review Principal, Designated MTO Contact

Client: MTO Foundations
File No.: 17792
E file: H:\17000-17999\17792 NWR Foundations Retainer - Assignment 3 - 4 Bridges\Reports & Memos\Hawkeye Lake Bridge\17792 - Hawkeye Lake
Bridge_Draft FIDR.docx

DRAFT

Date: July 12, 2017
Page: 21 of 21

STATEMENT OF LIMITATIONS AND CONDITIONS

1. STANDARD OF CARE

This Report has been prepared in accordance with generally accepted engineering or environmental consulting practices in the applicable jurisdiction. No other warranty, expressed or implied, is intended or made.

2. COMPLETE REPORT

All documents, records, data and files, whether electronic or otherwise, generated as part of this assignment are a part of the Report, which is of a summary nature and is not intended to stand alone without reference to the instructions given to Thurber by the Client, communications between Thurber and the Client, and any other reports, proposals or documents prepared by Thurber for the Client relative to the specific site described herein, all of which together constitute the Report.

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3. BASIS OF REPORT

The Report has been prepared for the specific site, development, design objectives and purposes that were described to Thurber by the Client. The applicability and reliability of any of the findings, recommendations, suggestions, or opinions expressed in the Report, subject to the limitations provided herein, are only valid to the extent that the Report expressly addresses proposed development, design objectives and purposes, and then only to the extent that there has been no material alteration to or variation from any of the said descriptions provided to Thurber, unless Thurber is specifically requested by the Client to review and revise the Report in light of such alteration or variation.

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5. INTERPRETATION OF THE REPORT

- a) Nature and Exactness of Soil and Contaminant Description: Classification and identification of soils, rocks, geological units, contaminant materials and quantities have been based on investigations performed in accordance with the standards set out in Paragraph 1. Classification and identification of these factors are judgmental in nature. Comprehensive sampling and testing programs implemented with the appropriate equipment by experienced personnel may fail to locate some conditions. All investigations utilizing the standards of Paragraph 1 will involve an inherent risk that some conditions will not be detected and all documents or records summarizing such investigations will be based on assumptions of what exists between the actual points sampled. Actual conditions may vary significantly between the points investigated and the Client and all other persons making use of such documents or records with our express written consent should be aware of this risk and the Report is delivered subject to the express condition that such risk is accepted by the Client and such other persons. Some conditions are subject to change over time and those making use of the Report should be aware of this possibility and understand that the Report only presents the conditions at the sampled points at the time of sampling. If special concerns exist, or the Client has special considerations or requirements, the Client should disclose them so that additional or special investigations may be undertaken which would not otherwise be within the scope of investigations made for the purposes of the Report.
- b) Reliance on Provided Information: The evaluation and conclusions contained in the Report have been prepared on the basis of conditions in evidence at the time of site inspections and on the basis of information provided to Thurber. Thurber has relied in good faith upon representations, information and instructions provided by the Client and others concerning the site. Accordingly, Thurber does not accept responsibility for any deficiency, misstatement or inaccuracy contained in the Report as a result of misstatements, omissions, misrepresentations, or fraudulent acts of the Client or other persons providing information relied on by Thurber. Thurber is entitled to rely on such representations, information and instructions and is not required to carry out investigations to determine the truth or accuracy of such representations, information and instructions.
- c) Design Services: The Report may form part of design and construction documents for information purposes even though it may have been issued prior to final design being completed. Thurber should be retained to review final design, project plans and related documents prior to construction to confirm that they are consistent with the intent of the Report. Any differences that may exist between the Report's recommendations and the final design detailed in the contract documents should be reported to Thurber immediately so that Thurber can address potential conflicts.
- d) Construction Services: During construction Thurber should be retained to provide field reviews. Field reviews consist of performing sufficient and timely observations of encountered conditions in order to confirm and document that the site conditions do not materially differ from those interpreted conditions considered in the preparation of the report. Adequate field reviews are necessary for Thurber to provide letters of assurance, in accordance with the requirements of many regulatory authorities.

6. RELEASE OF POLLUTANTS OR HAZARDOUS SUBSTANCES

Geotechnical engineering and environmental consulting projects often have the potential to encounter pollutants or hazardous substances and the potential to cause the escape, release or dispersal of those substances. Thurber shall have no liability to the Client under any circumstances, for the escape, release or dispersal of pollutants or hazardous substances, unless such pollutants or hazardous substances have been specifically and accurately identified to Thurber by the Client prior to the commencement of Thurber's professional services.

7. INDEPENDENT JUDGEMENTS OF CLIENT

The information, interpretations and conclusions in the Report are based on Thurber's interpretation of conditions revealed through limited investigation conducted within a defined scope of services. Thurber does not accept responsibility for independent conclusions, interpretations, interpolations and/or decisions of the Client, or others who may come into possession of the Report, or any part thereof, which may be based on information contained in the Report. This restriction of liability includes but is not limited to decisions made to develop, purchase or sell land.

Appendix A

Record of Borehole Sheets

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

CLASSIFICATION	PARTICLE SIZE	VISUAL IDENTIFICATION
Boulders	Greater than 200mm	same
Cobbles	75 to 200mm	same
Gravel	4.75 to 75mm	5 to 75mm
Sand	0.075 to 4.75mm	Not visible particles to 5mm
Silt	0.002 to 0.075mm	Non-plastic particles, not visible to the naked eye
Clay	Less than 0.002mm	Plastic particles, not visible to the naked eye

2. COARSE GRAIN SOIL DESCRIPTION (50% greater than 0.075mm)

TERMINOLOGY	PROPORTION
Trace or Occasional	Less than 10%
Some	10 to 20%
Adjective (e.g. silty or sandy)	20 to 35%
And (e.g. sand and gravel)	35 to 50%

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

DESCRIPTIVE TERM	UNDRAINED SHEAR STRENGTH (kPa)	APPROXIMATE SPT ⁽¹⁾ 'N' VALUE
Very Soft	12 or less	Less than 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	Greater than 200	Greater than 30

NOTE: Hierarchy of Soil Strength Prediction

- 1) Laboratory Triaxial Testing
- 2) Field Insitu Vane Testing
- 3) Laboratory Vane Testing
- 4) SPT value
- 5) Pocket Penetrometer


4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

DESCRIPTIVE TERM	SPT "N" VALUE
Very Loose	Less than 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	Greater than 50

5. LEGEND FOR RECORDS OF BOREHOLES

SYMBOLS AND ABBREVIATIONS FOR SAMPLE TYPE	SS Split Spoon Sample	WS Wash Sample	AS Auger (Grab) Sample
	TW Thin Wall Shelby Tube Sample	TP Thin Wall Piston Sample	
	PH Sampler Advanced by Hydraulic Pressure	PM Sampler Advanced by Manual Pressure	
	WH Sampler Advanced by Self Static Weight	RC Rock Core	SC Soil Core

$$\text{Sensitivity} = \frac{\text{Undisturbed Shear Strength}}{\text{Remoulded Shear Strength}}$$

 Water Level
 Shear Strength Determination by Pocket Penetrometer

- (1) SPT 'N' Value Standard Penetration Test 'N' Value – refers to the number of blows from a 63.5kg hammer free falling a height of 0.76m to advance a standard 50 mm outside diameter split spoon sampler for 0.3 m depth into undisturbed ground.
- (2) DCPT Dynamic Cone Penetration Test – Continuous penetration of a 50 mm outside diameter, 60° conical steel point attached to "A" size rods driven by a 63.5 kg hammer free falling a height of 0.76 m. The resistance to cone penetration is the number of hammer blows required for each 0.3 m advance of the conical point into undisturbed ground.

EXPLANATION OF ROCK LOGGING TERMS


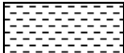



ROCK WEATHERING CLASSIFICATION

Fresh (FR)	No visible signs of weathering.
Fresh Jointed (FJ)	Weathering limited to the surface of major discontinuities.
Slightly Weathered (SW)	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock material.
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 structure are preserved.

DISCONTINUITY SPACING

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

SYMBOLS

	CLAYSTONE
	SILTSTONE
	SANDSTONE
	COAL
	BEDROCK

STRENGTH CLASSIFICATION

Rock Strength	Approximate Uniaxial Compressive Strength (MPa)	Approximate Uniaxial Compressive Strength (psi)	Field Estimation of Hardness*
Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.
Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail

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 0.1m in length or larger as a % of total core run length.
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.

UNIFIED SOILS CLASSIFICATION


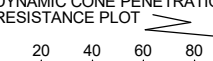






MAJOR DIVISIONS		GROUP SYMBOL	TYPICAL DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW	Well-graded gravels or gravel-sand mixtures, little or no fines.
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines.
		GM	Silty gravels, gravel-sand-silt mixtures.
		GC	Clayey gravels, 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	SILTS AND CLAYS $W_L < 50\%$	ML	Inorganic silts and 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. ($W_L < 30\%$).
		CI	Inorganic clays of medium plasticity, silty clays. ($30\% < W_L < 50\%$).
		OL	Organic silts and organic silty-clays of low plasticity.
	SILTS AND CLAYS $W_L > 50\%$	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
		OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils.
CLAY SHALE			
SANDSTONE			
SILTSTONE			
CLAYSTONE			
COAL			

RECORD OF BOREHOLE No 17-05

1 OF 1

METRIC

W.P. _____ LOCATION Hawkeye Lake East Bridge N 5 393 745.1 E 345 742.3 ORIGINATED BY AHF
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2017.04.19 - 2017.04.19 CHECKED BY CZ

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				
446.3	GROUND SURFACE											
0.8	ASPHALT: (25mm)						20 40 60 80 100			W P W W L		
	SAND, some silt, trace gravel Brown Wet (FILL)		1	GS			○ UNCONFINED + FIELD VANE			○		
445.5							● QUICK TRIAXIAL x LAB VANE			WATER CONTENT (%)		
0.8	Clayey SILT, some sand, trace gravel Brown Moist		1	SS	85/ 0.100		20 40 60 80 100			20 40 60		
	Cobbles at 1.4m due to hard augering											
444.0												
2.3	Gravelly SAND, some gravel, some silt Loose to Dense Brown Wet		2	SS	7							
			3	SS	34							
442.5												
3.8	GRANITE, very strong, grey to white (BEDROCK)											
	Horizontal fracture (25mm) at 4.0m, 5.0m and 5.1m		1	RUN								
	Sub-vertical fracture (75mm) at 4.9m											
	Sub-horizontal fracture (75mm) at 5.3m and (50mm) at 6.1m											
	Horizontal fracture (25mm) at 5.4m, 5.7m and 6.0m		2	RUN								
439.5												
6.8	END OF BOREHOLE AT 6.8m. WATER LEVEL AT 1.2m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 0.6m, SAND TO 0.3m AND COLD PATCH TO SURFACE.											

ONTMT4S MTO-17792.GPJ 2017TEMPLATE(MTO).GDT 7/19/17

+³, ×³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 17-05R

1 OF 1

METRIC

W.P. _____ LOCATION Hawkeye Lake East Bridge N 5 393 743.6 E 345 744.4 ORIGINATED BY BRM
 HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2017.06.06 - 2017.06.06 CHECKED BY CZ

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					WATER CONTENT (%)						
						20	40	60	80	100	20	40	60				
446.3	GROUND SURFACE																
0.0	ASPHALT: (25mm)		1	GS												14 62 20 4	
445.5	Silty SAND, some gravel, trace organics Dark Brown Moist to Wet																
0.8	(FILL)		1	SS	3												
	Gravelly SAND Very Loose to Compact Dark Brown Wet																
444.3			2	SS	13												
2.0	END OF BOREHOLE AT 2.0m DUE TO SPLIT SPOON REFUSAL ON COBBLES/BOULDERS. WATER LEVEL AT 1.4m.																

RECORD OF BOREHOLE No 17-06

1 OF 1

METRIC

W.P. _____ LOCATION Hawkeye Lake East Bridge N 5 393 747.1 E 345 751.6 ORIGINATED BY AHF
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2017.04.19 - 2017.04.19 CHECKED BY CZ

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE					w _P w w _L				
446.3	GROUND SURFACE						20	40	60	80	100	20	40	60	FI 0 1 1 1 0 0 4 4 1 0	RUN #1 TCR=100% SCR=98% RQD=100% UCS=172MPa RUN #2 TCR=100% SCR=77% RQD=77% UCS=182MPa	
0.0	ASPHALT: (25mm)																
445.7	SAND, some silt, trace gravel Brown Moist (FILL)		1	GS													
0.6	Gravelly SAND, some silt, split spoon bouncing on cobbles/boulders Compact Brown Moist		1	SS	100/ 0.050												
			2	SS	23												
			3	SS	11												
443.3																	
3.0	GRANITE, very strong, grey to white (BEDROCK)		1	RUN													
	Sub-vertical fracture (75mm) at 4.9m, (125mm) at 5.1m and (75mm) at 5.2m																
	Sub-horizontal fracture (25mm to 50mm) at 5.0m, 5.1m and 5.2m		2	RUN													
	Horizontal fracture (25mm) at 5.6m and 6.0m																
440.2																	
6.1	END OF BOREHOLE AT 6.1m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 0.6m, SAND TO 0.3m AND COLD PATCH TO SURFACE.																

ONTMT4S MTO-17792.GPJ 2017TEMPLATE(MTO).GDT 7/19/17

RECORD OF BOREHOLE No 17-06R

1 OF 1

METRIC

W.P. _____ LOCATION Hawkeye Lake East Bridge N 5 393 749.7 E 345 750.4 ORIGINATED BY BRM
 HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2017.06.06 - 2017.06.06 CHECKED BY CZ

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
						20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					WATER CONTENT (%)						
											20	40	60				
446.4	GROUND SURFACE																
0.8	ASPHALT: (25mm)		1	GS													
445.6	SAND, some gravel, trace silt Loose Brown Moist (FILL)		1	SS	7												
0.8	Gravelly SAND Loose to Compact Black Brown Moist		2	SS	10												
444.3	END OF BOREHOLE AT 2.1m DUE TO AUGER REFUSAL. WATER LEVEL AT 1.4m.																
2.1																	

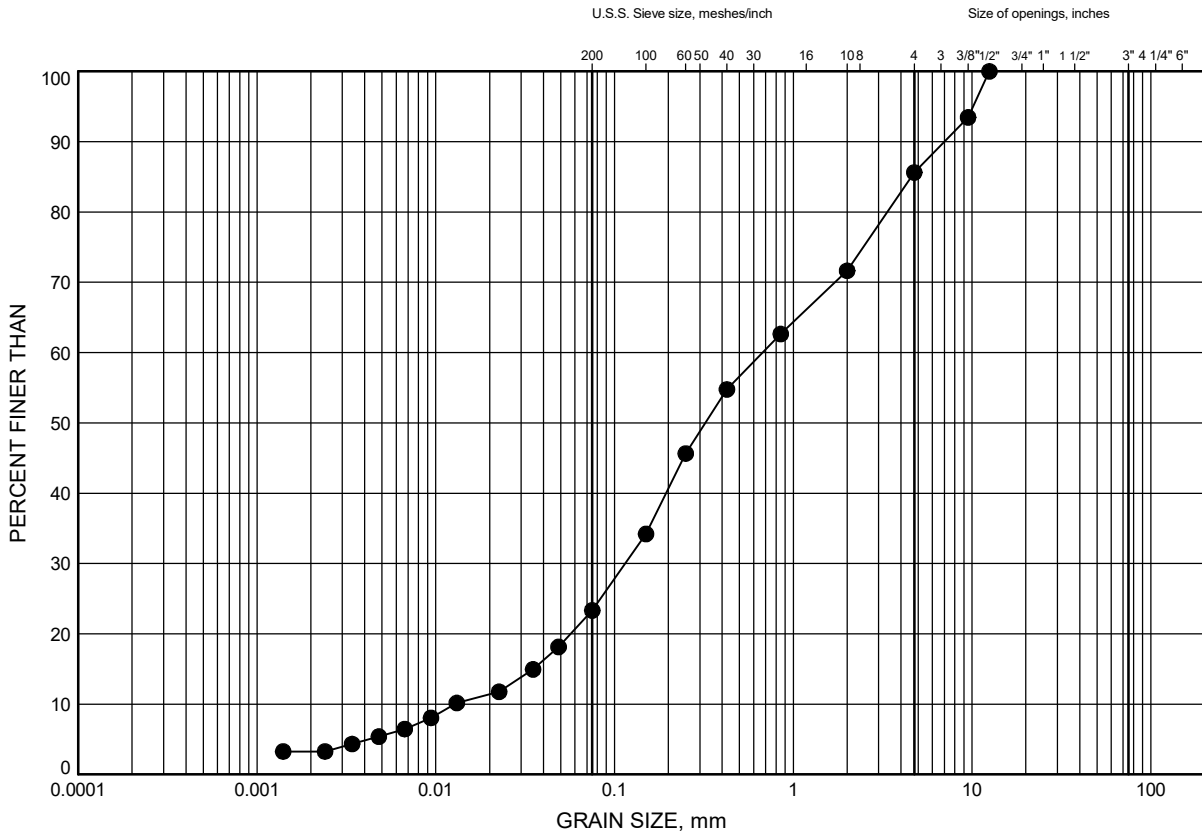
Appendix B

Laboratory Test Results

Hawkeye Lake East Bridge GRAIN SIZE DISTRIBUTION

FIGURE B1

Sand Fill



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	17-05R	0.3	446.0

Date July 2017
W.P.

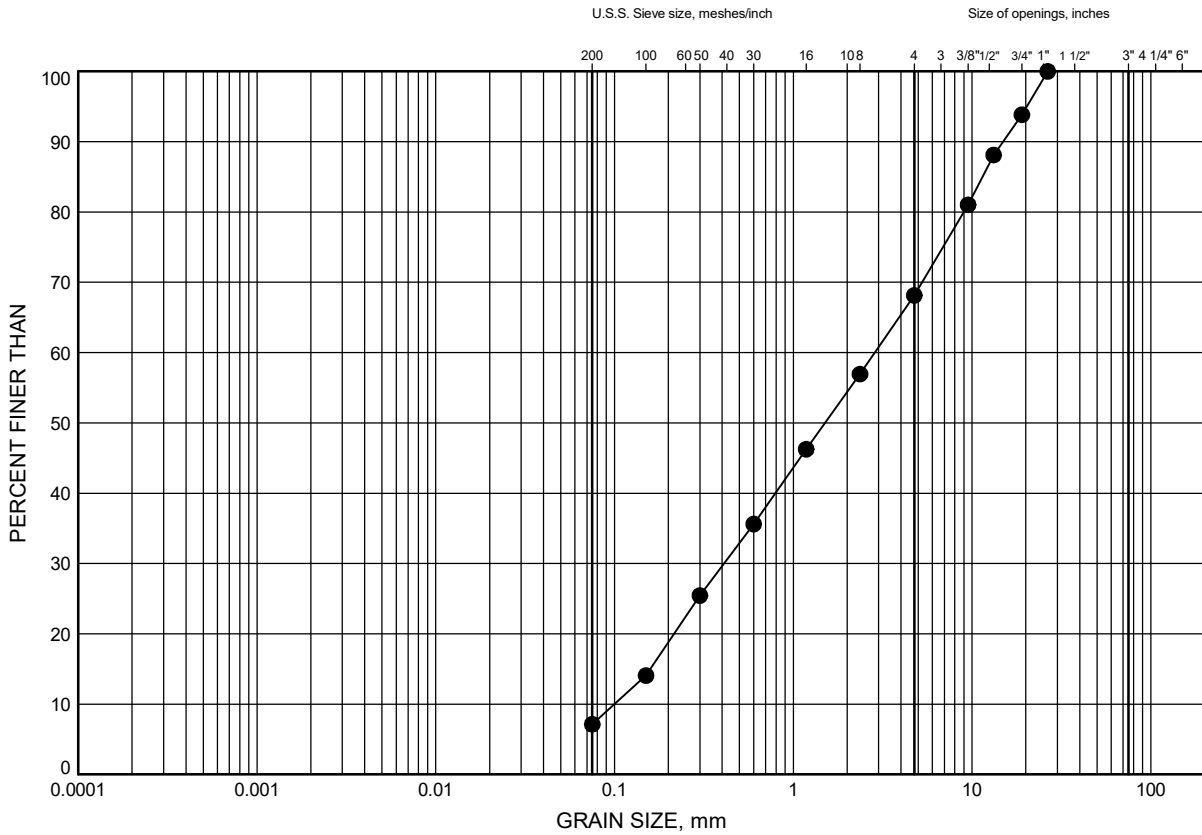


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Chkd. MM

Hawkeye Lake East Bridge GRAIN SIZE DISTRIBUTION

FIGURE B2

Gravelly Sand



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	17-06R	1.8	444.6

Date July 2017
W.P.



Prep'd MFA
Chkd. MM

POINT LOAD TEST SHEET

ASTM D5731-08

Job No:	17792
Client:	MTO
Project Name:	Hawkeye Lake East Bridge
Core Size:	NQ BH No : 17-05

Date Drilled:	19-Apr-17
Date Tested:	03-May-17
Tester:	TF
Reviewed by:	CZ

[illegible]



ASTM D5731-08

Date Drilled:	19-Apr-17
Date Tested:	03-May-17
Tester:	TF
Reviewed by:	CZ

[illegible]



SGS Canada Inc.

P.O. Box 4300 - 185 Concession St.

Lakefield - Ontario - KOL 2H0

Phone: 705-652-2000 FAX: 705-652-6365

Project : 17792

LR Report : CA14503-JUN17

Quality Control Report

Inorganic Analysis												
Parameter	Reporting Limit	Unit	Method Blank				LCS / Spike Blank			Matrix Spike / Reference Material		
					RPD	Acceptance Criteria	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
						%		Low	High		Low	High
Anions by IC - QCBatchID: DIO0308-JUN17												
Chloride	0.4	µg/g	<0.4		6	20	97	80	120	95	75	125
Sulphate	0.4	µg/g	<0.4		6	20	95	80	120	96	75	125
Carbon/Sulphur - QCBatchID: ECS0027-JUN17												
Sulphide	0.02	%	<0.02		NV	20	116	80	120			
pH - QCBatchID: EWL0329-JUN17												
pH	0.05	no unit	NA		1		100			NA		
pH - QCBatchID: EWL0330-JUN17												
pH	0.05	no unit	NA		0		100			NA		

Certificate of Analysis

SGS Canada Inc.
185 Concession St. Box 4300
Lakefield, Ont., Canada, K0L 2H0



Client
SGS LIMS Number
Analysis Package:

Attention: Cory Zanatta
Project#: 17792
Thurber Engineering
CA14503-JUN17
Corrosivity

Sample ID	Unit	Analysis Start Date	Analysis Approval Date	17-09R SS6	17-07R SS3	17-06R SS4
				07-Jun-17	05-Jun-17	06-Jun-17
Temperature Upon Receipt	°C			4.0	4.0	4.0
Corrosivity Index	none	01-Jun-17	01-Jun-17	8.5	4.0	4.5
Soil Redox Potential	mV	29-May-17	30-May-17	12	233	285
Sulphide	%	01-Jun-17	01-Jun-17	0.03	<0.02	0.02
% Moisture (wet wt)	%	30-May-17	01-Jun-17	26.0	16.8	13.30
pH	no unit	30-May-17	31-May-17	8.48	7.08	7.89
Chloride	µg/g	31-May-17	01-Jun-17	1.9	8.0	40
Sulphate	µg/g	31-May-17	01-Jun-17	51	11	26
Conductivity	uS/cm	30-May-17	31-May-17	157	43	89
Resistivity (calculated)	ohms.cm	30-May-17	01-Jun-17	6370	23300	11200

Corrosivity Index is based on the AWWA
Corrosivity Scale according to AWWA C-105.
An index greater than 10 indicates the
soil matrix may be corrosive to cast iron alloys.

Deanna Edwards B.Sc., C.Chem
Project Specialist
Environment, Health and Safety

Data reported represents the sample submitted to SGS. Reproduction of this analytical report in full or in part is prohibited without prior written approval. Please refer to SGS General Conditions of Services located at http://www.sgs.com/terms_and_conditions_service.htm. (Printed copies are available upon request.). Test Method information available upon request. "Temperature Upon Receipt" is representative of the whole shipment and may not reflect the temperature of individual samples.

**SGS Canada Inc.**

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

Project : 17840/17792

16-May-2017

Thurber Engineering Ltd**Attn :** Cory Zanatta

2010 Winston Park Dr
Oakville, ON
L6H 5R7,

Date Rec. : 10 May 2017**LR Report:** CA14294-MAY17**Reference:** 17840/17792 Cory Zanatta**Copy:** #1

Phone: 905-829-8666 x 240

Fax:

CERTIFICATE OF ANALYSIS

Final Report

Sample ID	Sample Date & Time	Temperature Upon Receipt °C	pH no unit	Conductivity µS/cm	Resistivity (calculated) ohms.cm	Redox Potential mV	Chloride mg/L	Sulphate mg/L	Sulphide mg/L
1: Analysis Start Date		---	11-May-17	11-May-17	---	11-May-17	15-May-17	15-May-17	11-May-17
2: Analysis Start Time		---	10:30	10:41	---	13:57	18:20	18:20	12:10
3: Analysis Approval Date		--	15-May-17	15-May-17	---	15-May-17	16-May-17	16-May-17	12-May-17
4: Analysis Approval Time		--	10:54	10:51	---	10:32	13:24	13:24	16:01
5: MDL		---	0.05	2	---	---	0.04	0.04	0.006
6: Rossmere Creek	25-Apr-17	9.0	6.35	115	8700	197	24	1.1	0.014
7: Two Island Lake	25-Apr-17	9.0	6.42	35	28700	218	2.0	2.0	< 0.006
8: Wawing Creek	25-Apr-17	9.0	6.30	47	21200	221	5.8	1.8	0.009
9: Hawkeye Lake	25-Apr-17	9.0	6.71	40	25000	213	1.4	1.9	< 0.006
10: Pinewood River	25-Apr-17	9.0	6.89	78	12900	207	0.68	0.43	0.018

Temperature of Sample upon Receipt: 9 degrees C

Cooling Agent Present: yes

Custody Seal Present: no

Brian Graham B.Sc.**Project Specialist****Environmental Services, Analytical**

**SGS Canada Inc.**

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

Project : 17840/17792**LR Report :** CA14294-MAY17

Method Descriptions

Parameter	SGS Method Code	Reference Method Code
Anions by IC	ME-CA-[ENV]IC-LAK-AN-001	EPA300/MA300-Ions1.3
Conductivity	ME-CA-[ENV]EWL-LAK-AN-006	SM 2510
pH	ME-CA-[ENV]EWL-LAK-AN-006	SM 4500
Redox Potential		SM 2580
Sulphide by SFA	ME-CA-[ENV]SFA-LAK-AN-008	SM 4500



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Project : 17840/17792

LR Report : CA14294-MAY17

Quality Control Report

Inorganic Analysis												
Parameter	Reporting Limit	Unit	Method Blank		RPD		LCS / Spike Blank			Matrix Spike / Reference Material		
					Acceptance Criteria	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)		
							Low	High		Low	High	
Anions by IC - QCBatchID: DIO0256-MAY17												
Chloride	0.04	mg/L	<0.04		2	20	97	80	120	100	75	125
Sulphate	0.04	mg/L	<0.04		0	20	96	80	120	89	75	125
Anions by IC - QCBatchID: DIO0269-MAY17												
Chloride	0.04	mg/L	<0.04		0	20	100	80	120	119	75	125
Sulphate	0.04	mg/L	<0.04		0	20	97	80	120	102	75	125
Conductivity - QCBatchID: EWL0183-MAY17												
Conductivity	2	µS/cm	< 2		0	10	99	90	110	NA		
pH - QCBatchID: EWL0182-MAY17												
pH	0.05	no unit	NA		1		100			NA		
Redox Potential - QCBatchID: EWL0192-MAY17												
Redox Potential	no	mV	NA		0	20	103	80	120	NA		
Sulphide by SFA - QCBatchID: SKA0095-MAY17												
Sulphide	0.006	mg/L	<0.006		ND	20	80	80	120	NV	75	125
Sulphide by SFA - QCBatchID: SKA0105-MAY17												
Sulphide	0.006	mg/L	0.009		ND	20	96	80	120	125	75	125

Appendix C

Site Photographs



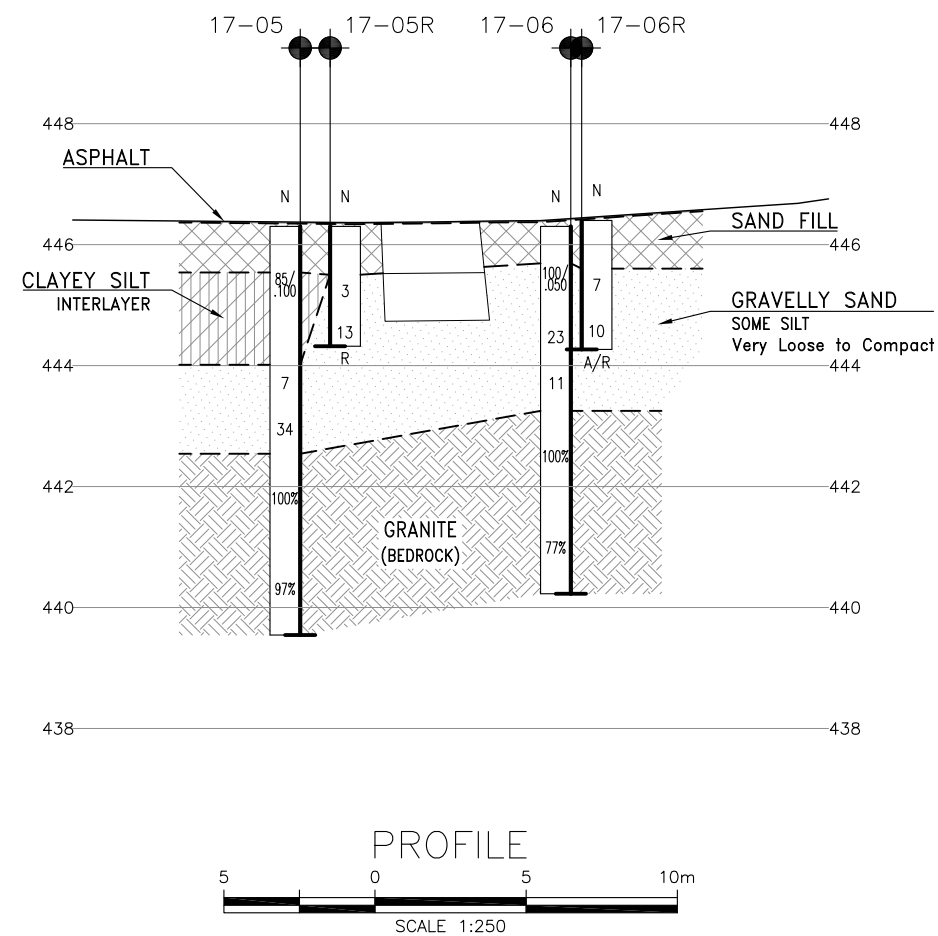
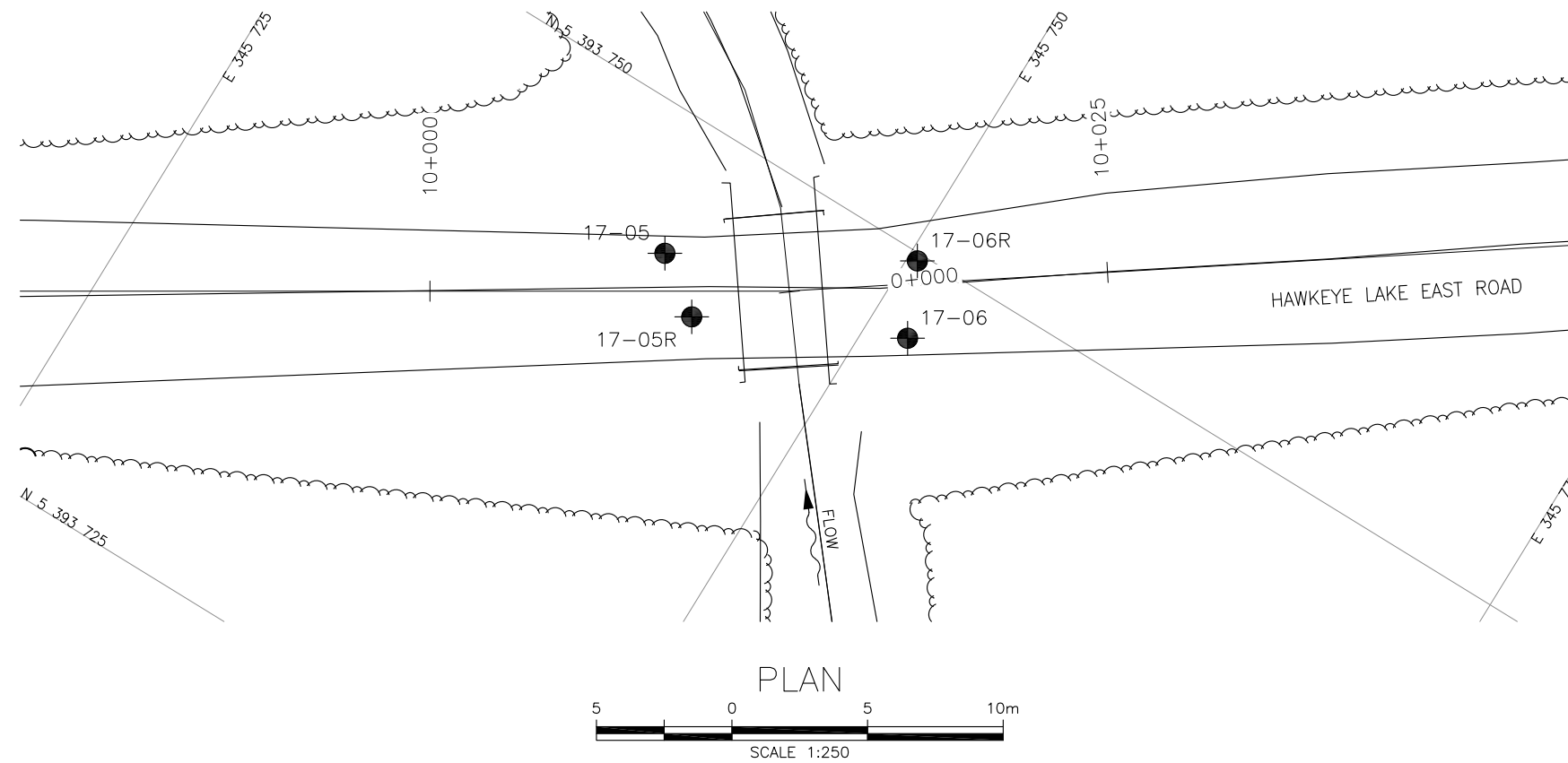
Photograph 1 – Hawkeye Lake Road East Bridge Outlet



Photograph 2 – East Hawkeye Lake Road Bridge – North Abutment Looking South

Appendix D

Borehole Locations and Soil Strata Drawing



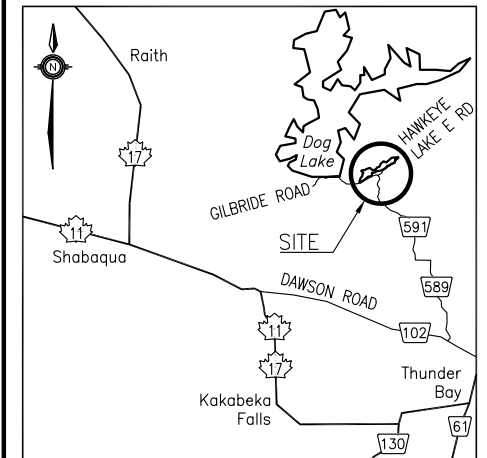
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT No
WP No








HAWKEYE LAKE EAST ROAD
BRIDGE
REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET



KEYPLAN

LEGEND

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

[illegible]

-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.

REVISIONS									
	DATE	BY				DESCRIPTION			
DESIGN	CZ	CHK	PKC			LOAD		DATE	JUL 2017
DRAWN	MFA	CHK	CZ	CODE		STRUCT	DWG	1	

Appendix E

Comparison of Foundation Alternatives

COMPARISON OF FOUNDATION ALTERNATIVES

Modular Bridge	Concrete Box Culvert	Concrete Open Footing Culvert
<u>Advantages:</u> i. Ease of construction. ii. A temporary diversion channel is not required during construction.	<u>Advantages:</u> i. Relatively rapid installation and less disturbance to subgrade soils if precast segments are used. ii. Segmental option can accommodate some potential differential settlement along culvert axis.	<u>Advantages:</u> i. Conventional construction. ii. Possibly less disturbance of creek channel / less environmental issues such as those involving spawning fish species.
<u>Disadvantages:</u> i. Presence of cobbles/boulder would cause difficulties in driving the sheet piles. ii. Would require full road closure during construction.	<u>Disadvantages:</u> i. More expensive than a CSP culvert. ii. Relatively large excavation required to install culvert. iii. Temporary roadway protection system required. iv. Would require road grade raise.	<u>Disadvantages:</u> i. Greater potential for differential settlement. ii. Deeper excavation and potentially longer dewatering requirements. iii. More disturbance of creek. iv. Would require road grade raise.
FEASIBLE - PREFERRED	FEASIBLE	NOT RECOMMENDED

Appendix F

List of SPs and OPSS, and Suggested Text for Selected NSSP

1. List of OPSS and OPSD Documents Relevant to this Project

- OPSS.PROV 206
- OPSS.PROV 209
- OPSS.PROV 422
- OPSS.PROV 501
- OPSS.PROV 539
- OPSS.PROV 804
- OPSS.PROV 902
- OPSS.PROV 903
- OPSS.PROV 1004
- OPSS.PROV 1010
- OPSS.PROV 1205
- OPSS.511
- OPSS.1860
- OPSD.802.010
- OPSD.803.010
- OPSD.3101.150

2. Suggested Wording for NSSP

- Suggested Text for NSSP on "Obstructions"

"Excavations and installation of cofferdams and roadway protection systems could encounter obstructions such as cobbles and boulders embedded in the fill and native soils, or shallow bedrock. Such obstructions may impede excavation progress and/or sheetpile installation. The Contractor shall be prepared to remove, drill through and/or penetrate these obstructions to achieve the design depths."

- Suggested Text for NSSP on "Groundwater and Dewatering"

"The Contractor is notified that the site has high groundwater levels and that these levels may be higher than the water levels shown in the Foundation Investigation Report prepared

for this site. While reference should be made to that report for a description of the encountered conditions, the Contractor must satisfy himself regarding the groundwater levels likely to prevail at the time of construction and be prepared to implement dewatering procedures.

The Contractor is further notified that failure to implement dewatering in advance of excavating below the groundwater table may result in sloughing and boiling of the soil in the excavation and a loss in stability and bearing resistance.

Design and provision of an effective dewatering system is the responsibility of the Contractor. Subgrade preparation, culvert construction and backfilling must be carried out in the dry.