FOUNDATION INVESTIGATION AND DESIGN REPORT
CEDAR CREEK BRIDGE REHABILITATION
HIGHWAY 17
DISTRICT OF THUNDER BAY, ONTARIO

G.W.P. 6068-09-00, Site No: 48E-27

Geocres Number: 42C-33

Report to:

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted at the Cedar Creek Bridge on Highway 17, in the District of Thunder Bay, Ontario.

The purpose of this investigation was to explore the subsurface conditions at the site and, based on the data obtained, to provide a borehole location plan, records of boreholes, stratigraphic profile, laboratory test results and a written description of the subsurface conditions. A model of the subsurface conditions was developed from the data obtained in the course of the investigation.

Thurber carried out the investigation as a sub-consultant to MMM Group Limited, under the Ministry of Transportation Ontario (MTO) Agreement Number 6010-E-0011.

2 SITE DESCRIPTION

The Cedar Creek Bridge is located on Highway 17, between Marathon and White River, Ontario, approximately 1.8 km west of the intersection of Highways 614 and 17. The existing bridge over Cedar Creek is a single span structure with a span of about 13.7 m, a skew of 25°, and a width of 13.4 m. The approach embankments are approximately 4 to 5 m high.

Cedar Creek flows out of Little Cedar Lake and meanders in a north westerly direction. The lands immediately surrounding the bridge site are generally forested. Gold mining operations including an open pit are located approximately 2 km west of the site.

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

The site lies at the boundary between the Quetico and Wawa Subprovinces of the Superior Province of the Canadian Shield. The site is underlain by metasedimentary rocks primarily comprising wacke and siltstone. The bedrock is overlain by silty to sandy till and thin drift deposits.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project were carried out between July 7 and 9, 2013 and consisted of drilling and sampling four boreholes, identified as Boreholes CED-01 to CED-04. The boreholes were advanced to depths of 6.6 to 10.4 m, including recovery of 3.1 to 3.3 m of bedrock core from Boreholes CED-02 to CED-04 and 1.6 m from Borehole CED-01. Dynamic Cone Penetration Tests (DCPTs) were carried out adjacent to Boreholes CED-02 and CED-03.

The approximate locations of the boreholes are shown on the Borehole Locations and Soil Strata Drawing in Appendix E.

The borehole locations were marked in the field and utility clearances were obtained prior to drilling. The coordinates and ground surface elevations for the boreholes were estimated from topographic plans provided by MMM Group Limited.

A truck-mounted CME 75 drill rig was used to advance the boreholes using a combination of NW casing/ wash boring techniques and NQ coring. Soil samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). All rock cores were logged in the field and the Total Core Recovery (TCR), Rock Quality Designation (RQD) and Fracture Indices (FI) were determined.

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

Groundwater conditions in the open boreholes were observed throughout the drilling operations. Groundwater conditions observed after completion of drilling were not representative of site conditions as water was introduced into the boreholes during wash boring and coring operations. Standpipe piezometers were installed in two boreholes to monitor the groundwater level after drilling. The piezometers were subsequently decommissioned and the boreholes without piezometers were backfilled in general accordance with MOE Regulation 903. Completion details of the piezometers and boreholes are summarized in Table 3.1.

Table 3.1 – Borehole Completion Details

Foundation Unit	Boreholes	Piezometer Tip Depth/ Elevation (m)	Completion Details
West Approach	CED-01	None installed	Borehole backfilled with holeplug to 0.1 m, then asphalt to surface.
West Abutment	CED-02	10.4/ 305.1	Sand from 10.4 m to 6.6 m, holeplug from 6.6 m to 0.15 m, then asphalt to surface.
East Abutment	CED-03	9.9/ 305.7	Sand from 9.9 m to 6.4 m, holeplug from 6.4 m to 0.2 m, then asphalt to surface.
East Approach	CED-04	None installed	Borehole backfilled with holeplug to 0.1 m, then asphalt to surface.

4 LABORATORY TESTING

All recovered soil samples were subjected to Visual Identification (VI) and to natural moisture content determination. Selected samples were also subjected to grain size distribution analyses (sieve and hydrometer). 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.

Bedrock core samples were subjected to geological logging. Point load tests were carried out on selected samples of intact bedrock in the laboratory to evaluate the unconfined compressive strength (UCS) of the bedrock. The UCS values of the rock assessed from the point load test data are reported on the borehole logs in Appendix A.

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 E. 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 borehole locations.

The site stratigraphy typically comprises asphalt pavement overlying existing granular embankment fill underlain by relatively thin native deposits of gravelly sand and silty sand till, which in turn overlie bedrock. More detailed descriptions of the individual strata are presented below.

5.1 Asphalt

Asphalt was encountered at the roadway surface in all boreholes. The asphalt layer was between 100 and 200 mm thick.

5.2 Embankment Fill

The existing embankment fill encountered in the boreholes comprises brown gravelly sand to sand and gravel with trace to some silt and occasional cobbles. The fill is between 2.9 and 4.9 m thick, with a lower boundary at depths ranging from 3.0 to 5.0 m (Elev. 312.5 to 310.5).

SPT 'N' values recorded in the fill ranged from 13 to 57 blows for 0.3 m penetration, indicating a compact to very dense relative density. SPT 'N' values of 100 blows for 0.0 to 0.225 m of penetration were recorded in Borehole CED-02, indicative of cobbles or possible boulders in the fill. Moisture contents ranged from 6% to 23%, typically 6% to 12%.

Samples of fill underwent laboratory grain size analysis testing, the results of which are summarized below. These results are also presented on the Record of Borehole sheets included in Appendix A and on the grain size distribution curves shown on Figure B1 of Appendix B.

Soil Particles	Sand and Gravel Fill (%)
Gravel	23 to 40
Sand	43 to 68
Silt & Clay	6 to17

5.3 Gravelly Sand

A native deposit of gravelly sand with occasional cobbles and boulders was encountered beneath the embankment fill in Boreholes CED-01, CED-03 and CED-04. The thickness of the gravelly sand deposit was between 2.0 and 2.1 m, with the lower boundary at depths of 5.0 and 6.2 m (Elev. 310.5 and 309.4). The native sand and overlying fill are similar in composition and identification of the boundary between fill and native materials is subject to interpretation.

SPT 'N' values recorded in the gravelly sand ranged from 21 blows for 0.3 m of penetration to 100 blows for no penetration, indicating a compact to very dense relative density. The higher values also reflect the presence of cobbles and boulders. Moisture contents of 11% and 22% were measured.

5.4 Silty Sand to Sand and Silt Till

A layer of till comprising silty sand to sand and silt was encountered underlying the fill in Borehole CED-02 and the gravelly sand in Boreholes CED-03 and CED-04. The till contained trace to some gravel, trace clay and occasional cobbles and boulders. The till layer has a thickness between 0.6 and 2.1 m, and a lower boundary at depths of 6.8 to 7.1 m (Elev. 308.8 to 308.4).

SPT "N" values of 26 to 49 blows for 0.3 m penetration were recorded, indicating a compact to very dense relative density. An N-value of 100 blows/0.025 m was recorded on a probable boulder in Borehole CED-02. Moisture contents of 9% and 10% were measured in Boreholes CED-03 and CED-04, and 30% in one sample from Borehole CED-02.

Three samples of till underwent laboratory grain size analysis testing, the results of which are summarized below. These results are also presented on the Record of Borehole sheets included in Appendix A, and the grain size distribution curves are shown on Figure B2 of Appendix B.

Soil Particles	Silty Sand to Sand and Silt Till (%)
Gravel	1 to 19
Sand	51 to 62
Silt	25 to 39
Clay	4 to 8

5.5 Bedrock

Metasedimentary bedrock comprising wacke and siltstone was encountered below the gravelly sand and silty sand till in all boreholes. The depths and elevations of the bedrock surface, proven by coring in all boreholes, are summarized in Table 5.1.

Table 5.1 – Depths and Elevations of Bedrock

Borehole	Top of Bedrock	
	Depth (m)	Elevation
CED-01	5.0	310.5
CED-02	7.1	308.4
CED-03	6.8	308.8
CED-04	7.0	308.6

Rock core lengths between 1.6 m to 3.3 m were recovered from the boreholes. Total core recovery in the bedrock was generally 100%. A lower recovery of 74% was recorded in Run 3 of Borehole CED-04.

The Rock Quality Designation (RQD) was typically between 74% and 100%, indicating good to excellent rock quality. Lower RQD values of 44% and 58% were recorded in the intial core runs in Boreholes CED-02 and CED-04, indicating a poor to fair rock quality.

The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m core, generally ranged from 0 to 5. Fracture indices greater than 10 were noted in Run 1 of Borehole CED-04.

The unconfined compressive strength of rock cores estimated from point load tests ranged from 86 to 158 MPa, indicating a strong to very strong rock. The results are presented on the Record of Borehole Sheets in Appendix A (as average per run).

5.6 Water Levels

Wash boring and rock coring methods were used to advance the boreholes and therefore water levels during or upon completion of drilling may not reflect natural groundwater levels. Standpipe piezometers were installed in two boreholes to monitor the groundwater level after completion. The water levels measured in the piezometers are summarized in Table 5.2.

Table 5.2 – Water Level Measurements

Borehole	Date	Water Level		Comment
		Depth (m)	Elev. (m)	
CED-02	May 2, 2014	3.2	312.3	In piezometer
CED-03	May 2, 2014	3.1	312.5	In piezometer

The preliminary GA drawing indicates a creek water level at Elev. 310.9 and a high water level at Elev. 312.4.

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

6 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 MMM Group Limited.

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

Eastern Ontario Diamond Drilling of Hawkesbury, Ontario supplied a truck mounted CME-75 drill rig and conducted the drilling, sampling and in-situ testing operations. The drilling operations were supervised by Ms. Eckie Siu of Thurber.

Overall supervision of the field program was conducted by Mr. Mark Farrant P.Eng. Interpretation of the data and preparation of the report were carried out by Ms. Mei Cheong., P.Eng.

The report was reviewed by Mr. Murray Anderson, P.Eng. and Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 GENERAL

This report presents interpretation of the geotechnical data in the factual report and presents geotechnical recommendations for the proposed rehabilitation of the existing Cedar Creek Bridge on Highway 17 in the District of Thunder Bay, Ontario.

The existing Cedar River Bridge was constructed in 1958 and consists of a single span concrete rigid frame structure with a span of about 13.7 m, a skew of 25°, and a width of 13.4 m. The approach embankments are approximately 4 to 5 m high.

Archive drawings indicate that the structure is supported on spread footings founded a minimum 0.3 m below the bedrock surface. The abutment footings are approximately 14.7 m long, 1.2 m wide and at least 0.6 m thick, with the top of the footing at Elev. 308.6. The wingwall footings are generally 3.7 m wide and vary in length from 6.7 to 8.2 m.

Based on the preliminary General Arrangement drawing provided by MMM Group, rehabilitation of the bridge will involve replacement of the existing bridge deck with a new precast prestressed box girder deck. Rehabilitation will also include modification to the tops of the abutments and wingwalls, installation of rock anchors through the abutment walls, and replacement of the existing approach slabs.

The discussions and recommendations presented in this report are based on the information provided by MMM Group Limited and on the factual data obtained in the course of the investigation.

8 ASSESSMENT OF EXISTING FOUNDATIONS

The archive drawings indicate that the existing abutment footings are founded at least 0.3 m below the bedrock surface at an anticipated level of Elev. 308.0. The bedrock surface was encountered at Elev. 308.4 and 308.8 in two boreholes drilled adjacent to the abutments during the current investigation. This information is consistent with the existing footings being constructed on bedrock.

For footings on bedrock, a factored geotechnical resistance at Ultimate Limit State (ULS) of 2,500 kPa is recommended to assess the capability of the existing abutment foundations to carry the revised loads from the rehabilitated structure. The geotechnical resistance at SLS will not govern for footings founded on bedrock.

We understand that deck replacement will increase the design foundation load at ULS from 430 to 434 kPa. The loading at SLS (328 kPa) will remain unchanged. Supporting the new deck on the existing spread footings is therefore considered to be acceptable from a geotechnical viewpoint. The Structural Designer must verify that the strength and integrity of the existing foundations is adequate to carry the foundation loads.

The resistance values provided are for a vertical, concentric load. Where eccentric or inclined loads are applied, the resistance used in the design must be reduced in accordance with Clause 6.7.3 and Clause 6.7.4 of the CHBDC.

The lateral resistance of the footings may be computed using an unfactored friction coefficient of 0.70 for concrete on bedrock. Additional lateral resistance, if required, may be provided by rock anchors.

The frost penetration depth at this site is 2.3 m. Footings on bedrock at this site do not require frost protection.

9 ABUTMENTS

9.1 Backfill and Lateral Earth Pressures

Any new backfill behind the modified abutment and wing walls should be placed in accordance with OPSS 902. All backfill material should consist of Granular A, Granular B Type II or Granular B Type III material meeting the specifications of OPSS.PROV. 1010. Compaction equipment to be used adjacent to the walls should be restricted in accordance with OPSS 501.

Lateral earth pressures acting on the walls may be assumed to be distributed triangularly and to be governed by the characteristics of the abutment backfill and the underlying soils. 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 = earth pressure coefficient (see Table 9.1)

γ = unit weight of retained soil (see Table 9.1)

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 shown in Table 9.1.

Table 9.1 – Earth Pressure Coefficients (K)

Loading Condition	Earth Pressure Coefficient (K)			
	OPSS Granular A or Granular B Type II $\phi = 35°, \gamma = 22.8$ kN/m^3		OPSS Granular B Type I, Granular B Type III, or Existing Granular Fill $\phi = 32°, \gamma = 21.2$ kN/m^3	
	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)*	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)*
Active (Unrestrained Wall)	0.27	0.38	0.31	0.46
At-rest (Restrained Wall)	0.43	-	0.47	-
Passive	3.7	-	3.3	-

* For wing walls.

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

The factors in Table 9.1 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.

In accordance with Clause 6.9.3 of the CHBDC, a compaction surcharge should be added. The magnitude should be 12 kPa at the top of fill and decreasing to 0 kPa at a depth of 2.0 m for Granular B Type I or III, or at a depth of 1.7 m for Granular A or Granular B Type II.

9.2 Rock Anchors

We understand that the proposed rehabilitation may increase the horizontal loads on the abutments by 75% to 80%. The use of rock anchors is being considered to resist the increased horizontal loads.

The factored rock-grout bond strength at ULS recommended for design of anchors developed in the metasedimentary bedrock at the site is 700 kPa. This value includes a geotechnical resistance factor of 0.4 as per Table 6.1 of the CHBDC.

The length of the unbonded (free-stressing) zone should be at least 3.0 m for a steel bar anchor and 4.5 m for a steel strand anchor. The minimum bond length should be 3.0 m. The section of drill hole through the cohesionless fill, sand and till will need to be cased to maintain an open hole during installation of the anchor.

The above recommendations are provided to estimate the anchor capacity for design purposes only. It is necessary that selected anchors be performance tested and all remaining production anchors on site be proof tested to confirm their carrying capacities. Anchor testing and other relevant anchor installation details should be in accordance with OPSS 942.

10 SEISMIC CONDITIONS

The following seismic parameters should be used for design:

- Velocity Related Seismic Zone 0
- Zonal Velocity Ratio 0.0
- Acceleration Related Seismic Zone 0
- Zonal Acceleration Ratio 0.0
- Peak Horizontal Acceleration 0.011g

The soil profile type at this site has been classified as Type I. Therefore, according to Clause 4.4.6.1 Table 4.4 of the CHBDC, a Site Coefficient "S" (ground motion amplification factor) of 1.0 should be used in seismic design.

In accordance with Clause 4.6.4 of the CHBDC, retaining structures should be designed using active (K_{AE}) and passive (K_{PE}) earth pressure coefficients that incorporate the effects of earthquake loading. The coefficients of horizontal earth pressure for seismic loading presented in Table 10.1 may be used:

Table 10.1 – Earth Pressure Coefficients for Earthquake Loading

Condition	Earth Pressure Coefficient (K)	
	OPSS Granular A or Granular B Type II ϕ = 35°, γ = 22.8 kN/m^3	OPSS Granular B Type I/III or Existing Granular Fill ϕ = 32°, γ = 21.2 kN/m^3
Active (K_{AE})*	0.28	0.32
Passive (K_{PE})	3.7	3.2
At Rest (K_{OE})**	0.45	0.50

* After Mononobe and Okabe, passive case assumes a horizontal surface in front of the wall.
** After Woods

As the existing footings are founded on bedrock, liquefaction is not a concern at this site.

THURBER

11 SCOUR AND EROSION PROTECTION

Erosion protection must be provided along any soil surfaces that may be in contact with the river flow. In particular, erosion should be provided along the toe of the embankment slopes where not protected by wingwalls.

A vegetation cover should be established on all other exposed earth surfaces to protect against surficial erosion, in general accordance with OPSS 804.

12 EXCAVATION AND GROUNDWATER CONTROL

Excavation to carry out modifications to the existing abutments and wingwalls is expected to be limited to the existing granular embankment fill above the river and groundwater levels.

All excavations must be carried out in accordance with OPSS 902 and the requirements of the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the approach fill within the depth of excavation may be classed as Type 3 soil above the water table.

The selection of the method of excavation is the responsibility of the Contractor and must be based on his equipment, experience and interpretation of the site conditions. It is anticipated that a hydraulic excavator will be suitable. Provision must be made for the handling of pavement materials, potential obstructions in the fill, and cobbles and boulders.

Bridge rehabilitation will be carried out in stages to maintain one traffic lane operational at all times. Roadway protection will be required to facilitate staging. Roadway protection should be provided in accordance with OPSS 539 and designed for Performance Level 2.

The design of any road protection or dewatering system that may be required is the responsibility of the Contractor. All shoring systems should be designed by a Professional Engineer experienced in such designs.

13 CONSTRUCTION CONCERNS

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

- The existing embankment fill and native sand/till deposits contain cobbles and boulders which may interfere with excavation, anchor installation or the installation of roadway protection.

- Water levels in the river may fluctuate during construction.

14 CLOSURE

Engineering analysis and preparation of the report were carried out by Ms. Mei Cheong., P.Eng.

The report was reviewed by Mr. Murray R. Anderson, P.Eng. and Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

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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

C_{pen} 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.

<u>UNIFIED SOILS CLASSIFICATION</u>

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			

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)	(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.

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No CED-01

METRIC

WP#	6068-09-00	LOCATION	Cedar Creek Bridge N 5 395 921.0 E 387 202.6	ORIGINATED BY	ES
HWY	17	BOREHOLE TYPE	Casing/NQ Coring	COMPILED BY	AN
DATUM	Geodetic	DATE	2013.07.08 - 2013.07.08	CHECKED BY	MEF

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
315.5										
0.0	**ASPHALT:** (125mm)									
0.1	**SAND** and **GRAVEL**, some silt, occasional cobbles Dense to Compact Brown Damp to Wet (FILL)				315					
		1	SS	40		○				
					314					
		2	SS	46		○	40	43	17 (SI+CL)	
					313					
		3	SS	25		○				
312.5 / 3.0	Gravelly **SAND**, occassional cobbles Compact Brown Wet	4	SS	21		○				
					312					
					311					
	Cobbles and boulders (cored from 4.6 to 5.0m)	5	SS	100/ 0.0						
310.5 / 5.0	**METASEDIMENTARY BEDROCK** (wacke and siltstone), grey				310					
308.9 / 6.6	END OF BOREHOLE AT 6.6m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 0.1m, THEN ASPHALT TO SURFACE.				309					

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); 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/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

+ ³, × ³: Numbers refer to Sensitivity

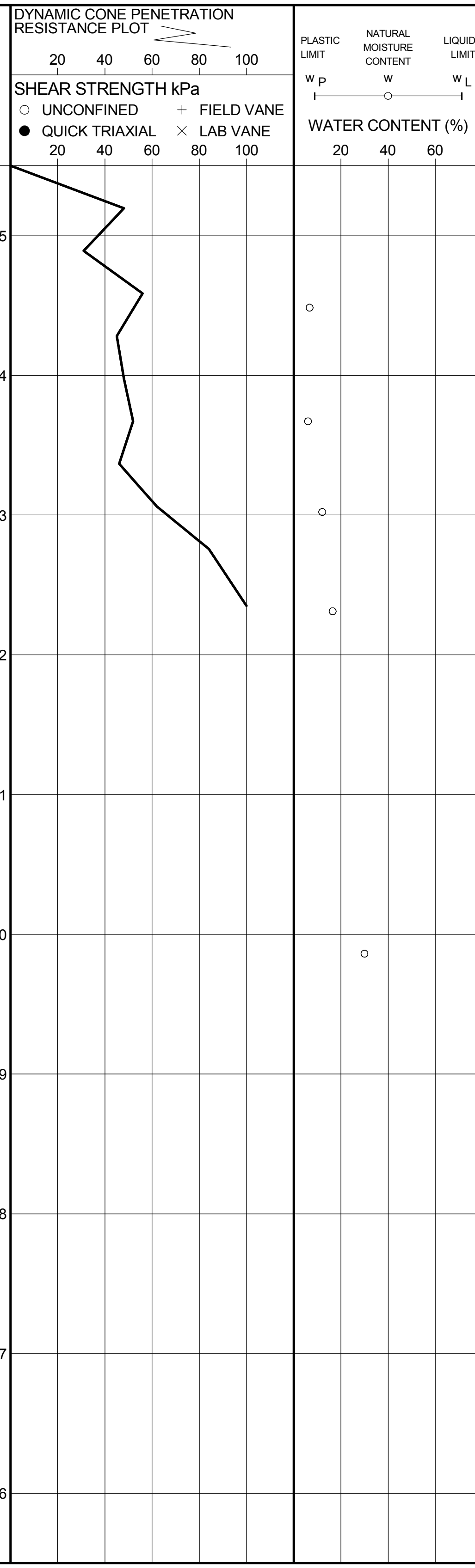
20 / 15 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No CED-02

METRIC

WP# 6068-09-00	LOCATION Cedar Creek Bridge N 5 395 933.5 E 387 205.1		ORIGINATED BY ES
HWY 17	BOREHOLE TYPE Casing/NQ Coring		COMPILED BY AN
DATUM Geodetic	DATE 2013.07.08 - 2013.07.08		CHECKED BY MEF

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
315.5							
0.0	**ASPHALT:** (175mm)						
0.2	Gravelly **SAND**, some silt, occasional cobbles Dense to Very Dense Brown Moist to Wet (FILL)				315		
		1	SS	31			23 64 13 (SI+CL)
					314		
		2	SS	44			
		3	SS	101/ 0.225	313		
		4	SS	50/ 0.125			
					312		
		5	SS	100/ 0.0	311		
310.5 5.0	Silty **SAND**, trace clay, trace gravel, occasional cobbles Compact Dark Brown Wet (TILL)				310		
		6	SS	26			5 62 29 4
	Boulder from 6.2 to 6.6m	7	SS	100/ 0.025	309		
308.4 7.1	**METASEDIMENTARY BEDROCK** (wacke and siltstone), grey Sub-vertical fractures (50mm) at 7.2m Strong	1	RUN		308	FI 5 0	RUN #1 TCR=100% SCR=58% RQD=58% UCS=126MPa (Average)
		2	RUN			1 1	RUN #2 TCR=100% SCR=100% RQD=100% UCS=102MPa (Average)
	Quartz vein (25mm) at 9.0m				307	0 0	
	Sub-vertical fracture (25mm) at 9.5m, (75mm) at 9.7m, (150mm) at 10.0m					0 1	
		3	RUN		306	5	RUN #3 TCR=100% SCR=100%



Continued Next Page

+³, ×³: Numbers refer to Sensitivity

(%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No CED-02

METRIC

WP# 6068-09-00 | LOCATION Cedar Creek Bridge N 5 395 933.5 E 387 205.1 | ORIGINATED BY ES

HWY 17 | BOREHOLE TYPE Casing/NQ Coring | COMPILED BY AN

DATUM Geodetic | DATE 2013.07.08 - 2013.07.08 | CHECKED BY MEF

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	"N" VALUES	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/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page										
305.1										0 1	RQD=100% UCS=121MPa (Average)
10.4	END OF BOREHOLE AT 10.4m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) 2014.05.12 3.2 312.3										

+ ³, × ³: Numbers refer to Sensitivity

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No CED-03

METRIC

WP#	6068-09-00	LOCATION	Cedar Creek Bridge N 5 395 938.5 E 387 231.9	ORIGINATED BY	ES
HWY	17	BOREHOLE TYPE	Casing/NQ Coring	COMPILED BY	AN
DATUM	Geodetic	DATE	2013.07.09 - 2013.07.09	CHECKED BY	MEF

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
315.6	
0.0	**ASPHALT:** (200mm)
0.2	Gravelly **SAND**, some silt Dense to Compact Brown Moist to Wet (FILL) Occasional cobbles No recovery
311.5 / 4.1	Gravelly **SAND**, occasional cobbles Very Dense Grey Wet
309.4 / 6.2	Silty **SAND**, some gravel, trace clay Dense Grey Moist (TILL)
308.8 / 6.8	**METASEDIMENTARY BEDROCK** (wacke and siltstone), grey Quartz vein (50mm) at 7.3m Sub-vertical fracture (25mm) at 7.4m, (50mm) at 7.5m
305.7	

SAMPLES

NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	37	○	
2	SS	17	○	32 58 10 (SI+CL)
3	SS	30	○	
4	SS	16		
5	SS	61	○	
6	SS	43	○	19 51 25 5
1	RUN			RUN #1 TCR=100% SCR=94% RQD=94% UCS=99MPa (Average)
2	RUN			RUN #2 TCR=100% SCR=100% RQD=100% UCS=113MPa (Average)
3	RUN			RUN #3 TCR=100% SCR=100% RQD=100% UCS=98MPa (Average)

FI: 2, 0, 2, 1, 0, 1, 0, 2, 0, 1

GROUND WATER CONDITIONS | ELEVATION SCALE: 315, 314, 313, 312, 311, 310, 309, 308, 307, 306

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

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No CED-03

METRIC

WP#	6068-09-00	LOCATION	Cedar Creek Bridge N 5 395 938.5 E 387 231.9	ORIGINATED BY	ES
HWY	17	BOREHOLE TYPE	Casing/NQ Coring	COMPILED BY	AN
DATUM	Geodetic	DATE	2013.07.09 - 2013.07.09	CHECKED BY	MEF

SOIL PROFILE			SAMPLES					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
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
	Continued From Previous Page										
9.9	END OF BOREHOLE AT 9.9m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) 2014.05.02 3.1 312.5										

+ 3 , × 3 : Numbers refer to Sensitivity

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

ONTMT4S 1197.GPJ 2012TEMPLATE(MTO).GDT 6/25/14

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No CED-04

METRIC

WP# 6068-09-00 LOCATION Cedar Creek Bridge N 5 395 950.9 E 387 234.6 ORIGINATED BY ES
HWY 17 BOREHOLE TYPE Casing/NQ Coring COMPILED BY AN
DATUM Geodetic DATE 2013.07.07 - 2013.07.07 CHECKED BY MEF

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
315.6								
0.0	**ASPHALT:** (100mm)							
0.1	Gravelly **SAND**, trace silt, occasional cobbles; Very Dense to Compact; Brown; Damp to Moist; (FILL)				315			
		1	SS	49		o		
		2	SS	57	314	o		
		3	SS	13	313	o		26 68 6 (SI+CL)
		4	SS	28	312	o		
311.5 / 4.1	Gravelly **SAND**, occasional cobbles; Grey; Wet							
		5	SS	100/ 0.050	311			
					310			
309.5 / 6.1	**SAND** and **SILT**, trace gravel, trace clay; Dense; Grey; Moist; (TILL)	6	SS	49	309	o		1 52 39 8
308.6 / 7.0	**METASEDIMENTARY BEDROCK** (wacke and siltstone), grey; Sub-vertical fractures (25mm to 75mm) at 7.1m, 7.2m, 7.4m, 8.0m, 8.1m	1	RUN		308		FI: 3, >10, >10, >15	RUN #1 TCR=100% SCR=60% RQD=44% UCS=158MPa (Average)
	Highly broken zone (50mm to 100mm) at 7.3m, 7.7m, 8.0m, 8.3m, 8.4m, 8.9m; Quartz veins (25mm) at 8.9m, 9.2m; 100mm at 9.0m	2	RUN		307		0, 0, 0	RUN #2 TCR=100% SCR=100% RQD=100% UCS=113MPa (Average)
	Sub-vertical fracture (50mm) at 9.5m, 9.7m	3	RUN		306		0, 2, 3	RUN #3 TCR=74% SCR=74% RQD=74% UCS=86MPa (Average)

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE
PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L
UNIT WEIGHT γ kN/m^3

Continued Next Page

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No CED-04

METRIC

WP#	6068-09-00	LOCATION	Cedar Creek Bridge N 5 395 950.9 E 387 234.6	ORIGINATED BY	ES
HWY	17	BOREHOLE TYPE	Casing/NQ Coring	COMPILED BY	AN
DATUM	Geodetic	DATE	2013.07.07 - 2013.07.07	CHECKED BY	MEF

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
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	Continued From Previous Page										
305.5 10.1	END OF BOREHOLE AT 10.1m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 0.1m, THEN ASPHALT TO SURFACE.										

ONTMT4S 1197.GPJ 2012TEMPLATE(MTO).GDT 6/25/14

+ 3 , × 3 : Numbers refer to Sensitivity

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

Appendix B

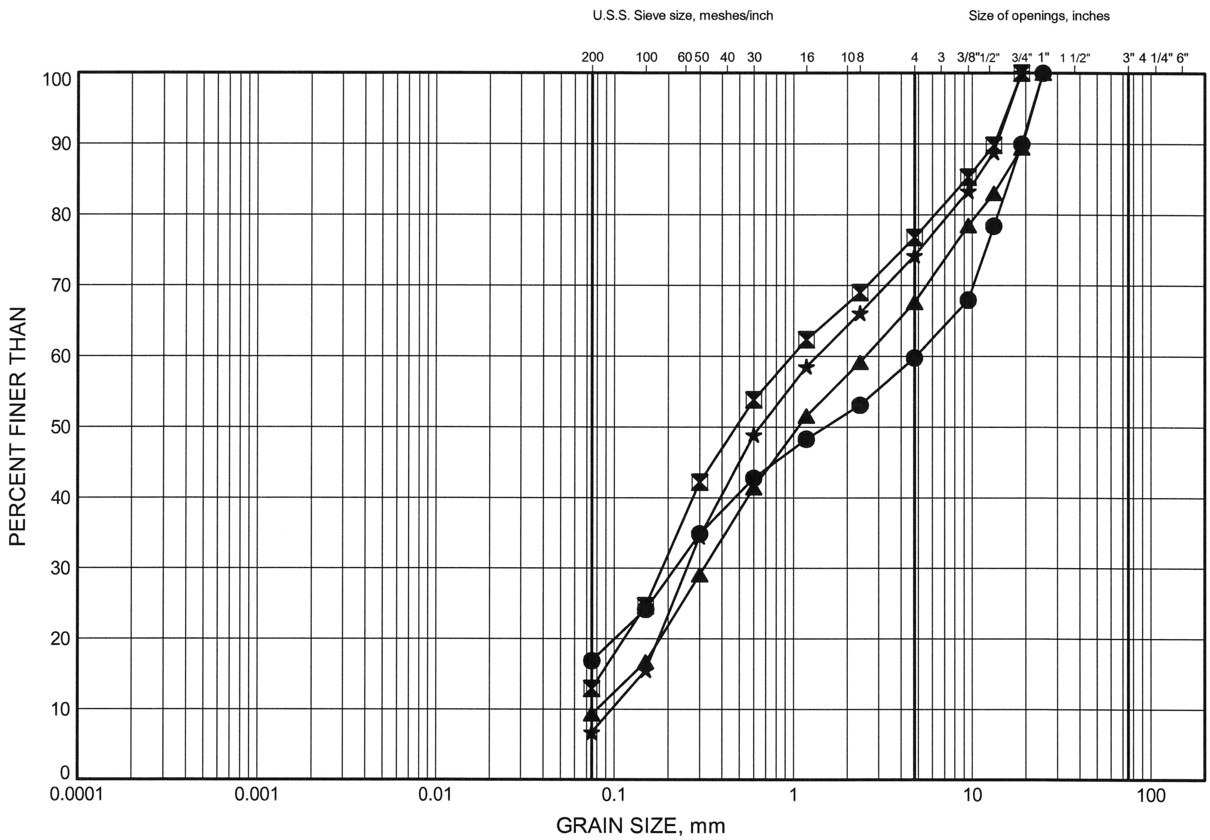
Laboratory Test Results

Cedar Creek Bridge

GRAIN SIZE DISTRIBUTION

FIGURE B1

GRAVELLY SAND TO SAND & GRAVEL FILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	CED-01	1.83	313.67
⧗	CED-02	1.07	314.43
▲	CED-03	1.83	313.77
★	CED-04	2.59	313.01

Date June 2014

WP# 6068-09-00

THURBER

Prep'd AN

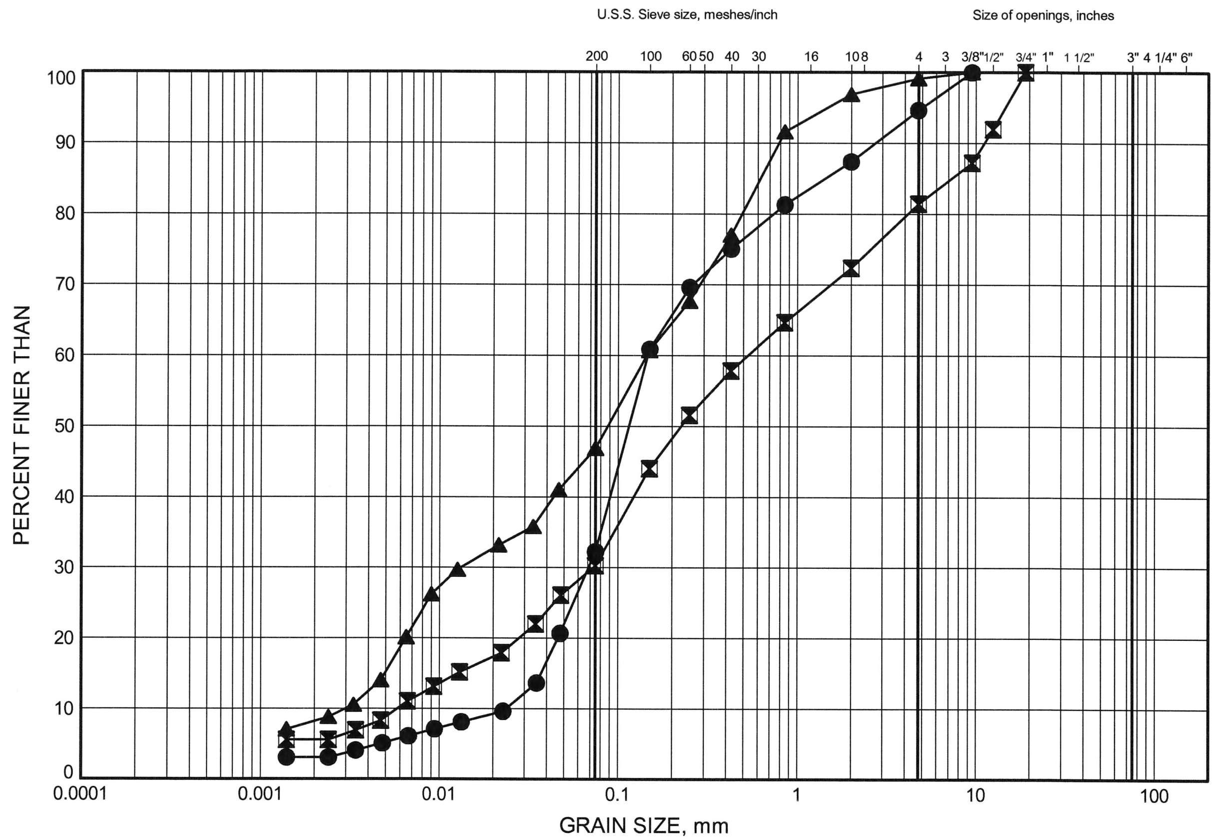
Chkd. MRA

Cedar Creek Bridge

GRAIN SIZE DISTRIBUTION

FIGURE B2

SILTY SAND TO SAND & SILT TILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	CED-02	5.64	309.86
⧗	CED-03	6.40	309.20
▲	CED-04	6.40	309.20

Date June 2014

WP# 6068-09-00

THURBER

Prep'd AN

Chkd. MRA

GRAIN SIZE DISTRIBUTION - THURBER 1197.GPJ 6/24/14

Appendix C

Site Photographs

Photograph 1 – Cedar Creek Bridge Looking East

Photograph 2 – Cedar Creek Bridge Looking West

Photograph 3 – Cedar Creek Looking North

Photograph 4 – Cedar Creek Looking South

Photograph 5 – West Abutment

Photograph 6 – Southeast Wingwall

Appendix D

List of SPs and OPSS

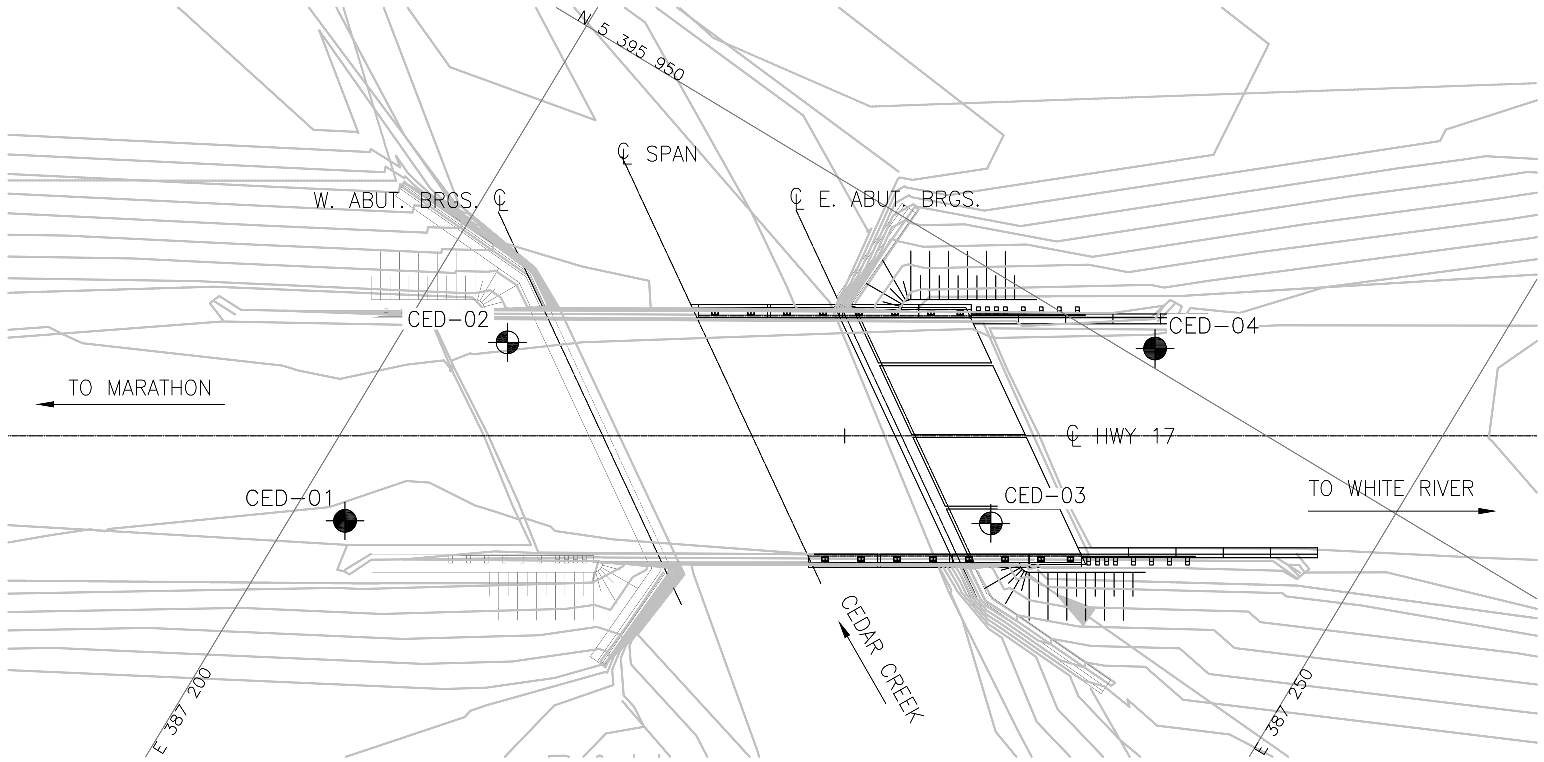
List of Special Provisions and OPSS Documents Referenced in this Report

- OPSS 501
- OPSS 539
- OPSS 804
- OPSS 902
- OPSS 942
- OPSS.PROV.1010

Appendix E

Borehole Locations and Soil Strata Drawing

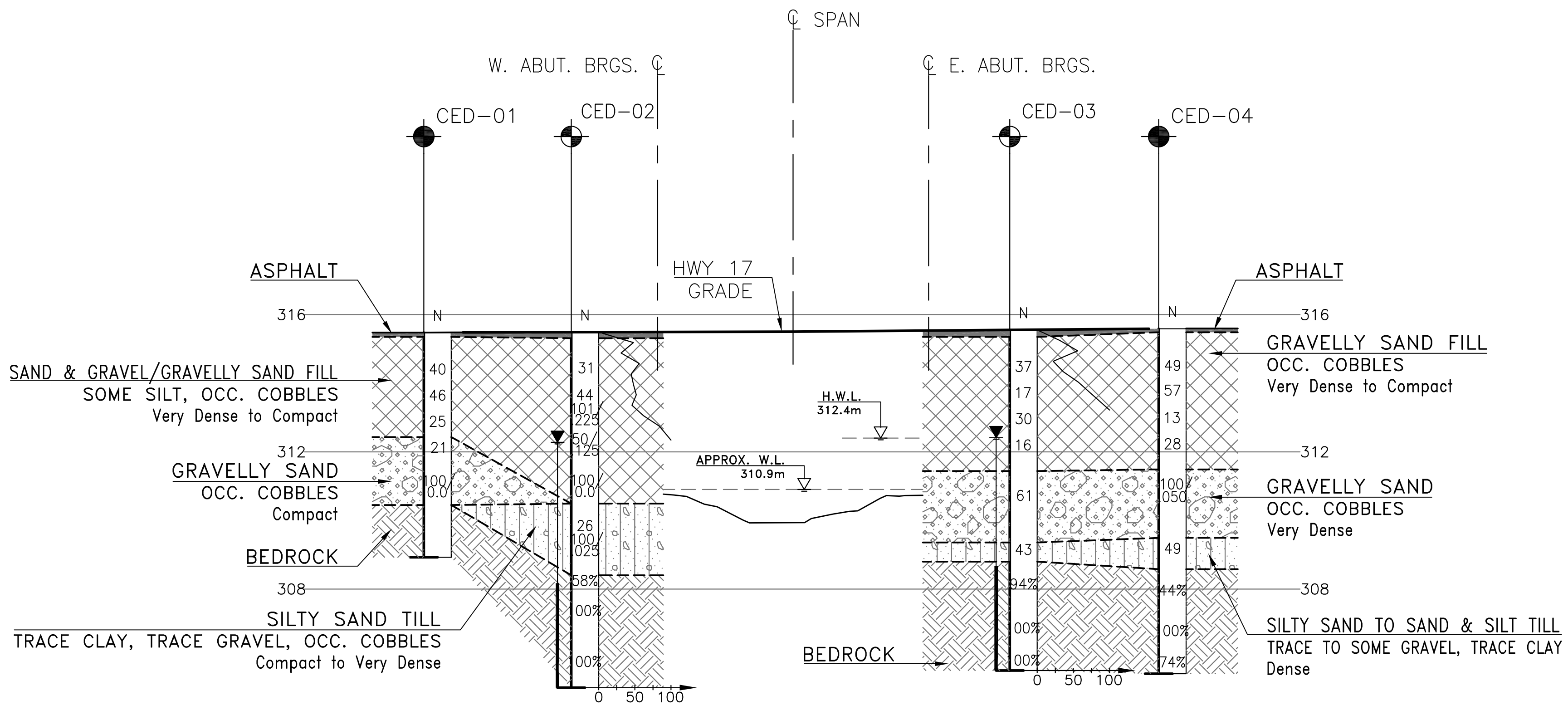
MINISTRY OF TRANSPORTATION, ONTARIO



PLAN

8 0 8 16m

SCALE 1:400



PROFILE ALONG ℄ HWY 17

8 0 8 16m H 1:400

4 0 4 8 V 1:200

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

LICENSED PROFESSIONAL ENGINEER
M. R. ANDERSON
Aug 28/14
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
Aug 28/14
PROVINCE OF ONTARIO

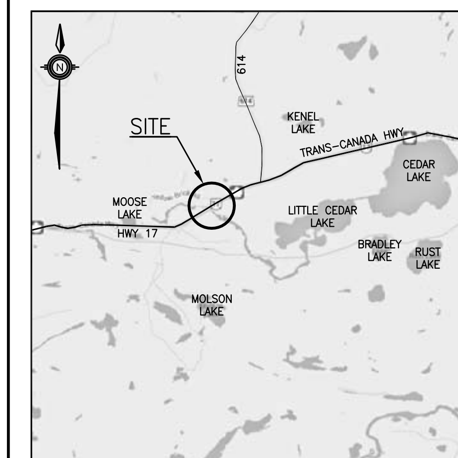
CONT No
WP No 6068-09-00

SHEET

HIGHWAY 17
CEDAR CREEK
BRIDGE REHABILITATION
BOREHOLE LOCATIONS AND SOIL STRATA

MMM GROUP

THURBER ENGINEERING LTD.



KEYPLAN

L E G E N D

	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 During Drilling
	Water Level In Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
CED-01	315.5	5 395 921.0	387 202.6
CED-02	315.5	5 395 933.5	387 205.1
CED-03	315.6	5 395 938.5	387 231.9
CED-04	315.6	5 395 950.9	387 234.6

-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. 42C-33

REVISIONS	DATE	BY	DESCRIPTION

DESIGN MC	CHK MC	CODE	LOAD	DATE AUG 2014
DRAWN AN	CHK	SITE 48E-27	STRUCT	DWG 1

FILENAME: H:\Drafting\19\1351\197\ted1197-CedarCreek(Plan&Profile).dwg
PLOTDATE: 9/3/2014 11:20 AM