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FOUNDATION INVESTIGATION AND DESIGN REPORT
GULLWING CREEK BRIDGE REPLACEMENT
HENDERSON LOOP ROAD, SITE No. 41S-24
NEAR DRYDEN, ONTARIO
W.O. No. 2016-11032, AGREEMENT # 6015-E-0023

GEOCRES Number: 52F-49

Report

to

MINISTRY OF TRANSPORTATION

Northwest Region

Date: April 7, 2017
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PART 1: FACTUAL INFORMATION

1. INTRODUCTION

This report presents the factual data obtained from a foundation investigation carried out by Thurber Engineering Ltd. (Thurber) for the proposed replacement of the Gullwing Creek Bridge on Henderson Loop Road in the Township of Britton, near Dryden, Ontario.

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

Thurber was retained by the Ministry of Transportation (MTO), Northwest Region to carry out this foundation investigation under the Agreement Number 6015-E-0023, Assignment #1, W.O. 2016-11032.

The existing MTO Foundation Investigation Report titled "Gullwing Creek Structure (Henderson Loop Road), Township of Britton, District of Kenora, Lot 6, Con. II and III, W.O. 77-67009, Site 41S-24, District 20, Kenora", dated April 5, 1978 , Geocres No. 52F-18 prepared for the then-proposed replacement of the original five span timber structure was reviewed. The foundation investigation documented in the report consisted of drilling one borehole to 16.2 m (53 ft) depth.

It should be noted that the elevations of the ground surface and the structure shown in the Geocres Report No. 52F-18 and on the drawing of the 1977 proposed replacement bridge differ significantly from the ground surface elevations presented on the Survey Plan of November 2016. It is probable that the archive documents utilized a local benchmark. However, no description of the location of the local benchmark was indicated in the available archive information. The Borehole Location Plan and Record of Borehole sheet of the Geocres Report No. 52F-18 are included in Appendix E for information.

2. SITE DESCRIPTION

The Gullwing Creek Bridge site is located on Henderson Loop Road, approximately 0.85 km west of Highway 665, in the Township of Britton, near Dryden, Ontario. The key plan showing the general location of the bridge site is presented on the Borehole Location and Soil Strata Drawing enclosed in Appendix D.

Henderson Loop Road runs in the general east-west direction with the bridge perpendicular to the centreline of the road. Gullwing Creek flows from north to south at the structure location. At this location, the Gullwing Creek is a relatively low energy stream with well developed meandering morphology. Adjacent to the site is forested land. Sporadic farmhouses and farmlands (mostly pastures) are present in the vicinity of the site.

As indicated in the Terms of Reference, the existing bridge is a three span (2.9 m, 33.6 m and 3.2 m lengths) structure with a total length of deck of 39.7 m. The existing bridge was built in 1979 and appears to be supported on timber piles, as indicated on the archive drawing titled, "Gullwing Bailey Bridge, Henderson Loop Road", dated 1979. Each bridge abutment is shown to be supported on ten (10) No. 14 timber piles.

A Biennial Inspection Report dated November 20, 2014 indicated that the structure was generally in good to fair condition. The signs of surface decay and weathering of the structural timber, some impact damage to the sidewalks and barriers, and loss of steel coating were noted in the inspection report.

The general area of the site is located within the physiographic region known as the Severn Upland of the Canadian Shield, and is characterized by rounded knobs and ridges of the Pre-Cambrian bedrock and depressions occupied by lakes and swamps. The relief is typically less than 50 m in this region. Locally, the site lies in a shallow valley surrounded by rolling terrain with soils characterized by Lake Agassiz glaciolacustrine deposits of silts and clays.

Photographs of the bridge and surrounding area are presented in Appendix C.

3. INVESTIGATION PROCEDURES

The field investigation and testing program for this project was carried out on September 14 and 15, 2016, and consisted of drilling and sampling two (2) boreholes, designated as Borehole 16-01 and 16-02. The boreholes were located on each side of the bridge, on the shoulders of Henderson Loop Road and in proximity to the existing bridge abutments. The boreholes were drilled to a depth of 31.1 m from the ground surface, and then Dynamic Cone Penetration Testing (DCPT) was conducted below the drilled portions of both boreholes. In Borehole 16-01, a refusal to further cone penetration was encountered at 46.3 m depth. In Borehole 16-02, the DCPT was terminated at 50.3 m depth without reaching practical refusal to cone penetration. Utility clearances were obtained prior to the start of drilling. The ground surface elevations at the borehole locations were derived from the Survey Plan dated November 2016 provided to Thurber by MTO. The coordinate system MTM NAD 83, Zone 16 was used to determine the locations of the boreholes. The approximate locations of the boreholes are shown on the Borehole Locations and Soil Strata Drawing enclosed in Appendix D.

A rubber track mounted CME 750 drill rig was used to advance the boreholes using hollow stem augers. Samples of the soils were obtained from the boreholes at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT) procedures as per ASTM D1586. Undrained shear strength was measured in the very soft to firm silty clay using the field vane in N size. 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 samples for transport to Thurber's laboratory for further examination and testing.

Groundwater conditions were observed in the open boreholes throughout the drilling operations and upon completion of drilling. A standpipe piezometer consisting of 25 mm diameter Schedule 40 PVC pipe with a 3.0 m slotted screen was installed in Borehole 16-01. The boreholes and standpipe piezometer were decommissioned following final water level reading in general accordance with Ontario Regulation 903. Completion details of the borehole are summarized in Table 3.1.

Table 3.1 – Borehole Completion Details

Borehole Number	Borehole Depth / Base Elevation (m)	Completion Details
16-01 (West Abutment)	31.1 / 322.8	Piezometer installed. Tip of piezometer screen at 24.4 m depth. Sand from base of borehole to 20.7 m depth and bentonite holeplug and cuttings to surface.
16-02 (East Abutment)	31.1 / 322.8	Borehole backfilled with bentonite holeplug and cuttings to surface.

The existing MTO report (Geocres No 52F-18) documented one borehole drilled at this site in 1978, designated as Borehole 1. Borehole 1 was advanced near the east abutment to a depth of approximately 16.2 m with soil sampling, and then a DCPT was conducted to a depth of 30.5 m. A DCPT was also conducted near the borehole within the sampled depth. The approximate location of Borehole 1 is shown on the Borehole Locations and Soil Strata Drawing included in Appendix E.

4. LABORATORY TESTING

All recovered soil samples were subjected to Visual Identification (VI) and to natural moisture content determination. Selected samples were subjected to grain size distribution analyses (MTO LS702) and Atterberg Limits testing according to MTO LS703, where appropriate. The results of the laboratory testing program are shown on the Record of Borehole sheets included in Appendix A and on the figures included in Appendix B.

In order to assess the potential for sulphate attack on concrete foundations, as well as the potential for corrosion associated with the below ground portion of the structure, a sample of the existing native soil, and a sample of the surface water from the creek upstream of the existing bridge were collected. The samples were submitted to SGS Canada Inc., a CALA accredited analytical laboratory in Lakefield, Ontario, for analytical testing of corrosivity parameters and sulphate content. The results of the analytical testing are summarized in Section 6 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 on the Record of Borehole sheets and on the "Borehole Locations and Soil Strata" drawing included in Appendix D. A general description of the stratigraphy, based on the conditions encountered in the boreholes, is given in the following paragraphs. However, the factual data presented on the Record of Borehole sheets takes precedence over this general description and should be used for interpretation of the site conditions. It should be recognized that soil conditions may vary between and beyond the borehole locations.

In general, the subsurface conditions encountered in the boreholes consisted of embankment fill underlain by an extensive deposit of silty clay extending to the depths investigated in the boreholes. Descriptions of the individual strata are presented below.

5.1 Embankment Fill

Granular embankment fill was encountered in both boreholes extending from the ground surface. The fill composition ranged from sand and gravel with trace silt to gravelly sand with some silt and some clay. The fill was 1.4 m to 2.3 m thick with the underside encountered at Elev. 352.5 and Elev. 351.6 in Boreholes 16-01 and 16-02, respectively. The relative density of the fill was compact with the recorded SPT-N values between 12 and 23 blows per 0.3 m of penetration.

The measured moisture content of the fill generally ranged from 5% to 9%. The results of grain size analyses conducted on two samples of the fill are presented on the record of Borehole sheets included in Appendix A, and on Figure B1 in Appendix B.

The results are summarized in the following table:

Soil Particle	Percentage (%)	
	Sand and Gravel	Gravelly Sand
Gravel	50	21
Sand	46	56
Silt and Clay	4	-
Silt	-	12
Clay	-	11

The fill encountered in the borehole drilled in 1978 on the east side of the bridge consisted of 0.6 m of sand and gravel underlain by approximately 1.2 m of “mixture of sand and black organics”, as noted on the Record of Borehole sheet in Appendix E.

5.2 Silty Clay

A deposit of silty clay with trace sand was encountered below the fill in all boreholes. Trace of rootlets were noted in the silty clay samples collected immediately below the fill material. The silty clay was sampled to a depth of 31.1 m (Elev. 322.8). In Boreholes 16-01 and 16-02, the upper 4.2 m and 3.2 m of the silty clay was brown to grey in colour and appeared to be typically firm to stiff with the SPT-N values ranging from 2 to 15 blows per 0.3 m of penetration. This zone appears to form a weathered crust to the underlying very soft to firm silty clay deposit. The base of the weathered crust was estimated to be at 5.6 m and 5.5 m depth (Elev. 348.3 and Elev. 348.4) in Boreholes 16-01 and 16-02, respectively.

Underlying the weathered crust was a grey, very soft to firm silty clay, with trace of sand. Occasional seams of silt and clayey silt were noted at depth in the deposit. Field vane shear tests measured undrained shear strength ranging from 5 kPa to 79 kPa, typically 5 kPa to 40 kPa and increasing with depth. Vane tests measured that the sensitivity of the silty clay ranged from 1 to 6 indicating that the silty clay has low to medium sensitivity.

The DCPT carried out below that depth indicated the SPT N values gradually increased to 100 blows per 0.150 m of penetration at 46.3 m depth in Borehole 16-01, and the SPT N values reached approximately 60 blows per 0.15 m penetration at 50.3 m depth in Borehole 16-02.

The results of grain size analyses conducted on samples of the silty clay are provided on the Record of Borehole sheets in Appendix A, and are illustrated in Figures B2 to B3 of Appendix B. The results of the grain size distribution tests are summarized below.

Particle Size	Percentage (%)
Gravel	0
Sand	0 to 8
Silt	32 to 81
Clay	19 to 68

The results of Atterberg Limits testing conducted on samples of the silty clay are provided on the Record of Borehole sheets in Appendix A and are illustrated on the Plasticity Charts (Figures B4 and B5) in Appendix B. Liquid limits ranged from 21% to 53% and the plasticity indices ranged from 8% to 34%, indicating plasticity of the deposit ranging from low to high. Moisture contents of the silty clay varied from 22% to 80%.

5.3 Groundwater Conditions

Water levels were monitored in the open boreholes during drilling operations. Wash boring was used to advance boreholes and therefore water levels recorded during or upon completion of drilling may not reflect natural groundwater conditions. A standpipe piezometer was installed in Borehole 16-01 after completion of drilling. The water level measured in the piezometer and in open boreholes are presented in Table 5.1.

Table 5.1 – Water Level Measurements

Borehole	Date	Water Level		Comment
		Depth (m)	Elev. (m)	
16-01	September 15, 2016	6.1	347.8	In piezometer
16-02	September 15, 2016	4.5	349.4	In open borehole

The water level in Gullwing Creek was shown on the Survey Plan at Elev. 349.34 m in October 2016.

The recorded levels are short-term readings and seasonal fluctuations of the groundwater and river 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 silty clay crust from Borehole 16-01, and a sample of the surface water from the creek were submitted for analytical testing of corrosivity parameters and sulphate. 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	
			BH 16-01, SS#3, 2.3 m – 2.9 m (Silty Clay)	Gullwing Creek Water
Sulphide	%	mg/L	<0.02	0.026
Chloride	µg/g	mg/L	37	0.95
Sulphate	µg/g	mg/L	18	1.8
pH	No unit	No unit	6.37-6.84	7.96
Electrical Conductivity	µS/cm	µS/cm	145	109
Resistivity	Ohms.cm	Ohms.cm	6900	917
Redox Potential	mV	mV	270	218
Corosivity Index	-	-	1	14

7. MISCELLANEOUS

Thurber obtained subsurface utility clearances prior to drilling. Thurber obtained the northing and easting coordinates and ground surface elevations from measurements taken in the field relative to the topographic feature and comparing with the Survey Plan provided by MTO.

RPM Drilling Inc. of Thunder Bay, Ontario supplied and operated the drilling, sampling and in-situ testing equipment for the field investigation. The field investigation was supervised on a full time basis by Mr. Troy MacKinnon of Thurber. Overall supervision of the field program was provided by Mr. Mark Farrant, P.Eng. of Thurber.

Geotechnical laboratory testing was carried out at Thurber's geotechnical laboratory. Analytical laboratory testing was conducted by SGS Canada Inc.

Interpretation of the field data and preparation of this report was carried out by Ms. Anna Piascik, P.Eng and Mark Farrant, P.Eng. The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

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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 recommendations for design of the proposed Gullwing Creek Bridge Replacement on Henderson Loop Road, west of Highway 665 in the Township of Britton, near Dryden, Ontario.

This foundation investigation and design report with the interpretation and recommendations is 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. Contractors 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.

As indicated in the Terms of Reference, the existing bridge is a three span (2.9 m, 33.6 m and 3.2 m lengths) structure with a total length of deck of 39.7 m. The existing bridge was built in 1979 to replace the original five span timber bridge supported on timber bents. The existing bridge appears to be supported on ten (10) Number 14 timber piles with incorporated timber caps at each abutment. As shown on the Survey Plan, dated November 2016, provided by MTO, the level of Henderson Loop Road at the existing approaches to the bridge was at an approximate Elev.353.9. The creek water level was indicated at Elev. 349.34 in October 2016.

The preliminary General Arrangement drawing dated April 2017 indicates that the replacement structure will consist of a single span modular bridge constructed on the existing alignment with a proposed grade raise and widening of the roadway platform. The grade raise consists of 0.3 to 0.5 m over the existing platform, and 1 m on the side slopes.

9. STRUCTURE FOUNDATIONS

In general, the soil stratigraphy below the existing embankment fill consists of a deep deposit of silty clay. The silty clay was sampled to a depth of 31.1 m (Elev. 322.8). The upper 4.2 m and 3.2 m of the silty clay in Boreholes 16-01 and 16-02, respectively, was typically firm to stiff and appeared to form a weathered crust to the underlying very soft to firm silty clay deposit. The base of the weathered crust was estimated to be at 5.6 m and 5.5 m depth (Elev. 348.3 and Elev. 348.4) in the boreholes. Underlying the weathered crust was a grey, very soft to firm silty clay, with trace sand and trace gravel. Occasional seams of silt and clayey silt were noted at depth in the silty clay deposit. Field vane shear testing performed below the crust zone measured undrained shear strength ranging from 5 kPa to 79 kPa. The DCPT carried out below the sampled depth indicated gradual increase in the SPT N values to 100 blows per 0.150 m of penetration at 46.3 m depth in Borehole 16-01, and the SPT N values recording approximately 60 blows per 0.15 m penetration at 50.3 m depth in Borehole 16-02 without reaching refusal.

The Gullwing Creek water level shown on the Survey Plan was at Elev. 349.34 on October 2016. The groundwater level measured in the piezometer installed in Borehole 16-01 was at 6.1 m depth or Elev. 347.8. It is anticipated that the groundwater level will be influenced by the water level in Gullwing Creek.

Given the soil stratigraphy encountered and the requirements of modular bridge design, the following foundation options were considered for the support of this bridge:

- spread footings placed on engineered fill pads, and
- driven steel H-piles.

Recommendations for design of the feasible foundation options are presented in the following sections along with the corresponding geotechnical design parameters, where applicable. A preferred foundation option from a geotechnical perspective was indicated.

9.1 Spread Footings on Engineered Fill Pads

9.1.1 Founding Elevations

A modular bridge supported on concrete spread footings placed on minimum 2 m thick granular fill pad can be considered at this site. The Survey Plan dated November 2016 indicates the finished road grade at approximate Elev. 353.9 at the abutments. The base of a granular engineered fill pad located below approximate Elev. 352 and no lower than Elev. 351.5 may be assumed for design, so that the engineered fill pad will be constructed on the firm to stiff silty clay (weathered crust) above the water level in the creek.

9.1.2 Engineered Fill Construction

A sketch of the abutment placed on compacted fill is enclosed in Appendix G. Excavations for the engineered fill pad construction will require the existing timber pile caps removed and part of the existing timber piles to be cut-off. For construction of the engineered fill pad, the following construction sequence may be followed:

1. The minimum depth of excavation should accommodate the concrete foundation slab and the thickness of engineered fill pad below the slab;
2. Excavate to remove all timber pile caps and any posts. The existing timber piles located within the footprint of the proposed granular pads should be cut-off to approximately 0.3 m below the design base elevations of the granular pad;
3. The subgrade for the engineered fill pad should be inspected and all organic matter, soft/loose soils, and any deleterious materials should be removed from the footprint of the excavation and replaced with Granular A or Granular B Type II.
4. A separation layer consisting of a non-woven geotextile should be placed on the prepared silty clay subgrade.
5. The engineered fill pads should consist of OPSS Granular A or Granular B Type II placed in 150 mm lifts and compacted to 100% of its SPMDD at ±2% of optimum moisture content;
6. Dewatering measures should be provided, as required, to place the engineered fill in the dry;
7. The dimensions of the base of the excavation should be determined by assuming a granular pad 1.0 m wider than the footing at the level of the footing base and projecting outward and downward no steeper than 1H:1V.

It will be beneficial to place/locate the new abutments/spread footings some distance behind the existing abutments to take the advantage of the slope stabilizing effect of the existing timber piles. As noted above, the existing timber piles should not be extracted at this site. If piles are located within the footprint of the granular pad; they should be cut-off at approximately 0.3 m depth below the founding level of the engineered fill pad.

9.1.3 Axial Geotechnical Resistance and Reaction

The following values of factored Geotechnical Resistance at ULS and Geotechnical Reaction at SLS may be used for design of a minimum 1.5 m wide spread footing placed on the above prepared engineered fill pad, with the base of the engineered fill pad at below Elev. 352:

Factored Geotechnical Resistance at ULS (kPa)	-	165 kPa
Geotechnical Reaction at SLS (kPa)	-	125 kPa

The value of the Geotechnical Reaction at SLS given corresponds to up to 25 mm to 35 mm of settlement.

The value of a Factored Geotechnical Resistance at ULS was assessed assuming a Consequence Factor of 1.0 (Typical), and a Resistance Factor of 0.5 (Typical), as per CHBDC 2014. The Geotechnical Reaction at SLS was assessed assuming a factor of 0.8 for typical degree of understanding of the subsurface conditions

The geotechnical resistance provided above is for concentric, vertical loading conditions only. In the case of eccentric or inclined loading, the geotechnical resistance should be calculated as indicated in the CHBDC 2014 Clause 6.10.3 and Clause 6.10.4.

The lateral resistance of the footings founded on engineered fill may be computed using an unfactored friction coefficient of 0.5.

Due to the proposed grade raise of 0.3 to 0.5 m, the spread footing would be subject to approximately 40 – 50 mm of settlement, and therefore the footing option is less desirable than driven piles.

9.2 Driven H-Pile Foundations

The ground conditions at the site are considered to be suitable for the use of driven steel H-pile foundations to support the bridge abutments primarily through shaft friction. In light of the very soft to stiff consistency of the silty clay underlying the site, the piles may need to be driven to significant depths, depending on the resistance required.

9.2.1 Axial Geotechnical Resistance and Reaction

The axial factored geotechnical resistance at Ultimate Limit States (ULS) and geotechnical reaction at Serviceability Limit States (SLS) for a steel HP 310x110 piles are provided in the table below for two pile lengths. It was assumed that the underside of the pile caps will be located at approximately Elev. 352.

Table 9.1 – Axial Factored Geotechnical Resistance and Reaction for HP 310x110

Location (Relevant Borehole)	Top of Pile Elevation (m)	Pile Length / Tip Elevation (m)	Factored Geotechnical Resistance at ULS (kN)	Geotechnical Reaction at SLS (kN)
West Abutment (16-01)	351.4	28 / 323.4	450	325
East Abutment (16-02)		28 / 323.4	450	325

The axial pile resistances shown in Table 9.1 were derived based on the resistances along the pile shaft embedded within the silty clay.

9.2.2 Pile Installation

Pile installation should be in accordance with OPSS 903.

Pile driving should be controlled in accordance with Standard Drawing SS103-11 (Hiley Formula) and the designer should specify an ultimate pile resistance. Controlling of pile installation using the Hiley formula could start when the piles are within 2.0 m of the design tip elevation. The appropriate pile driving note is "Piles to be driven in accordance with Standard SS103-11 using an ultimate resistance of 'R' kN per pile". 'R' should have a minimum value of twice the design load at ULS.

If the proposed bridge design requires that the deviation at the top of the pile be limited to tight tolerance, a driving template or other means may be required to achieve the specified maximum deviation.

Pile tip protection or pile shoe will not be required and should not be used for friction piles at this bridge site.

The alignment of the H-piles should be carefully selected to be outside of the footprint of the existing abutments and away from river banks. The timber piles supporting the existing abutments may be cut-off at approximately 0.3 m below the new base of pile cap.

9.2.3 Lateral Resistance

The geotechnical lateral resistance acting on a pile in cohesive soils may be estimated using the coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) as follows:

k_s = $67\ S_u / D$ (kN/m^3)

p_{ult} = $9\ S_u$ (kPa)

Where S_u = undrained shear strength (kPa)

D = pile width or diameter (m)

The above equations and recommended parameters in Table 9.2 below may be used to analyse the interaction between a pile and the surrounding soil. The lateral pressures obtained from the analysis should not exceed the ultimate lateral resistance.

Table 9.2 – Soil Parameters for Lateral Pile Resistance

Soil Unit	Elevation (m)		γ' (kN/m^3)	n_h (kN/m^3)	K_p	S_u (kPa)
	Top	Bottom				
West Abutment (16-01)						
Sand and Gravel Fill (Compact)	GS	352.6	21	3,000	1.6 *	-
Silty Clay (Firm to Stiff)	352.6	348.3	8	-	-	60
Silty Clay (very soft)	348.3	343.0	7	-	-	5
Silty Clay (soft)	343.0	335.3	8	-	-	25
Silty Clay (firm)	335.3	322.9	8	-	-	40
East Abutment (16-02)						
Gravelly Sand Fill (compact)	GS	351.6	21	3,000	1.6 *	-
Silty Clay (Firm to Stiff)	351.6	348.4	8	-	-	60
Silty Clay (Soft)	348.4	343.0	8	-	-	20
Silty Clay (Soft to Firm)	343.0	338.5	8	-	-	30
Silty Clay (Firm)	338.5	322.8	8	-	-	40

Note: * Kp accounts for 2H:1V fill slope

The spring constant, K_s, for analysis may be obtained by the expression, $K_s = k_s\ L\ D$ (kN/m), where k_s is the coefficient of horizontal subgrade reaction (kN/m^3), D is the pile width (m) and L is the length (m) of the pile segment or element used in the analysis. The ultimate lateral resistance, P_{ult}, may be obtained from the expression, $P_{ult} = p_{ult}\ L\ D$. This represents the ultimate load at which the pile fails and will not support any additional load at greater displacements.

The modulus of subgrade reaction and ultimate lateral resistance may have to be reduced, based on the pile spacing. The reduction factors to be used for a pile group oriented perpendicular or parallel to the direction of loading are provided in Table 9.3. Intermediate values may be obtained by linear interpolation.

Table 9.3 – Subgrade Reaction Reduction Factors for Pile Spacing

Condition	Pile Spacing (Centre to Centre)	Reduction Factor
Pile group oriented ***perpendicular*** to direction of loading	4D	1.0
	1D	0.5
Pile group oriented ***parallel*** to direction of loading	8D	1.0
	6D	0.7
	4D	0.4
	3D	0.25

In the case of conventional abutments, i.e. not integral type, horizontal loads may be resisted by means of battered piles.

9.3 Downdrag

Widening of the roadway platform is proposed, which results in grade raises of approximately 0.3 to 0.5 m above the existing roadway and 1 m on the side slopes to accommodate the replacement bridge. The new fill placement at the abutments will result in development of downdrag forces along the length of abutment piles associated with consolidation of the silty clay foundation under the weight of the new fill.

For design purposes, an unfactored downdrag load of 500 kN per pile is recommended to evaluate the impact of downdrag on the abutment piles.

This downdrag load should be multiplied by a load factor of 1.25 as per CHBDC Commentary Clause C6.11.4.10 to obtain a factored downdrag load. In accordance with Section 6.11.4.10 of the CHBDC and Clause C6.11.4.10 of the Commentary, in the structural design of a pile, the factored downdrag load should be added to the factored permanent loads to assess the effects of downdrag. The factored dead and downdrag load should not exceed the factored structural resistance of a pile at the neutral plane.

9.4 Integral Abutment Considerations

The soil conditions at this site are suitable for the design of an integral abutment structure. The pile flexibility requirements of this design should be checked by the structural designer, taking account of the lateral resistance of the soil surrounding the piles.

9.5 Recommended Foundation

From cost effectiveness and constructability perspectives, spread footings supported on granular engineered fill pads placed within the weathered crust of the silty clay deposit is the preferred foundation option at this site taking account of the soil stratigraphy and the requirements of modular bridge design.

9.6 Frost Cover

The depth of frost penetration at this site is approximately 2.5 m. If piles are used, the base of pile caps should be provided with a minimum 2.5 m of earth cover as protection against frost action.

If it is not practical to provide 2.5 m of earth cover, consideration can be given to augmenting the frost protection by using expanded polystyrene insulation (EPS). Typically, 25 mm of EPS can be considered equivalent to 600 mm of earth cover. If EPS is used, it must be provided with long term protection against erosion, environmental degradation and spills.

Concrete bearing slab foundations for a modular bridge founded on an engineered fill pads should be provided with a minimum 500 mm embedment.

10. EXCAVATION AND DEWATERING

The Gullwing Creek water level was shown on the Survey Plan at Elev. 349.34 in October 2016. Groundwater level measured in the standpipe piezometer in Borehole 16-01 was at 6.1 m depth or at Elev. 347.8 at the time of completion of fieldwork. It should be assumed that the groundwater level will be governed by the water level in the creek.

Excavations for abutment construction should be kept above the creek level. Where excavations extend below the groundwater or creek level, the Contractor should implement effective dewatering procedures to lower the water level to a minimum 0.5 m below the base of excavation. Dewatering scheme consisting of sump and pump with surface runoff diversion is anticipated to be appropriate and effective at the abutments. If the construction is carried out during period of high water level in the creek, then more positive water control measures will be required, such as the use of sheet pile cofferdam and pumping from within the excavation. However, design and implementation of the dewatering procedures is the responsibility of the Contractor.

All excavations should 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 may be classified as Type 3 soil above the water table and as Type 4 soil below the water table. The native silty clay within the depth of excavation may be classified as Type 3 soil. Flatter slopes may be required at locations where water seepage affects surficial stability.

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

The selection of the method of excavation is the responsibility of the Contractor and depends on the equipment available, experience and interpretation of the site conditions by the Contractor. It is anticipated that a hydraulic excavator will be suitable to conduct the excavation at this site.

11. TEMPORARY SUPPORT SYSTEM

If required, the temporary excavation support system should be designed and constructed in accordance with OPSS. PROV 539 for Performance Level 2. The Contractor should select the temporary support system and design taking into account the soil conditions encountered in the boreholes.

The following parameters apply for design of the temporary support system:

γ	=	21 kN/m^3	(bulk unit weight of fill)
	=	18 kN/m^3	(bulk unit weight of native silty clay)
γ'	=	11 kN/m^3	(submerged unit weight of fill)
	=	8 kN/m^3	(submerged unit weight of native silty clay)
K_a	=	0.30	(active earth pressure coefficient of fill)
	=	0.39	(active earth pressure coefficient of native silty clay)
K_p	=	3.0	(passive earth pressure coefficient of fill)
	=	2.6	(passive earth pressure coefficient of native silty clay)

The actual lateral earth pressure distribution acting on the shoring system is a function of construction sequence and the relative rigidity of the shoring wall and these factors should be accounted for when designing the temporary protection system.

Vibratory methods must not be used at this site to install roadway protection.

The design of temporary shoring system should be the responsibility of the contractor. All shoring systems should be designed by a Professional Engineer experienced in such design.

12. ABUTMENT BACKFILL AND LATERAL EARTH PRESSURES

If any new backfill is placed behind the modified abutments, it 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.PROV 501.

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 Table below)

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

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, if included in the design, 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 $\phi = 35°$, $\gamma = 22.8$ kN/m^3		OPSS Granular B Type I or Type III $\phi = 32°$, $\gamma = 21.2$ 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 abutment walls, if required

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 accordance with Clause 6.12.3 of the CHBDC, a compaction surcharge should be added.

13. SEISMIC CONSIDERATIONS

In accordance with the CHBDC 2014, the selection of the seismic site classification is based on the average soil conditions encountered in the upper 30 m of the stratigraphy. The stratigraphy of the site includes compact granular fill and very soft to stiff silty clay deposit extending to depth at least 31.1 m. Based on Table 4.1, Clause 4.4.3.2 of the CHBDC, the site was categorized as the Seismic Site Class E. The peak ground acceleration, PGA, for a 2% in 50 year probability of exceedance at this site is 0.043 g, as per the National Building Code of Canada (NBCC).

In accordance with Clause 4.6.5 of the CHBDC 2014, 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 13.1 may be used:

Table 13.1 – Earth Pressure Coefficients for Earthquake 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 or Type III $\phi = 32°, \gamma = 21.2$ kN/m^3
Active (K_{AE})*	0.29	0.33
Passive (K_{PE})	3.6	3.2
At Rest (K_{OE})**	0.51	0.55

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

Given the presence of deep silty clay deposit with relatively high content of clay, the potential for liquefaction at this site is assessed to be low.

14. APPROACH EMBANKMENTS

No evidence of instability or excessive settlements of the existing approach embankments were noted during the foundation site investigation or indicated in the MTO Structure Inspection Report.

Widening of the roadway platform is proposed, which will involve grade raises of approximately 0.3 to 0.5 m above the existing roadway and 1 m on the side slopes. The additional fill is expected to cause settlements in the order of 15 - 20 mm in the roadway platform and 30 – 40 mm on the side slopes. This settlement is expected to occur over a 2 – 3 year period. Periodic roadway maintenance may be required during this period.

Oversteepening of the river banks/front slopes at the bridge were indicated on the Survey Plan. The oversteepening could be attributed to the ongoing erosion by the creek flow; design of appropriate erosion protection for the river banks/front slopes will be required.

Embankment restoration after completion of the bridge replacement should be carried out in accordance with OPSS PROV 206 and OPSS PROV 209. 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 reused, provided it is unfrozen, free of organics and at a moisture content that it is suitable for compaction. Fill placement for embankment restoration should follow the requirements of OPSD 208.010 (Benching of Earth Slopes) to integrate the existing and new embankment fill.

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

The embankments should be reconstructed with side slopes inclined not steeper than 2 horizontal to 1 vertical.

Global stability of the approach embankments near the river valley slopes is not considered to be an issue for the structure provided that the front slope is constructed no steeper than the current slope inclination with proper erosion protection implemented. However, additional assessment of the approach embankment stability should be conducted when the structure design and approach embankment geometries are finalized.

15. SCOUR AND EROSION PROTECTION

Erosion protection should be provided at the bridge along soil surfaces that may be in contact with the creek flow. 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 AND SULPHATE ATTACK POTENTIAL

The results of the corrosivity and sulphate analytical tests conducted on the native soil and creek water collected during the current investigation indicate the following conditions at the locations tested:

- The potential for corrosion or sulphate attack on concrete foundations from the surrounding native soil or surface water is considered to be negligible due to the low concentration of sulphate and chloride in the samples tested.
- The potential for soil corrosion on metal is considered to be mild.
- Appropriate protection measures are recommended if metal structural elements are used.

17. CONSTRUCTION CONCERNS

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

- A suitable dewatering / unwatering system should be employed to enable construction of the foundations in the dry and prevent instability of the excavation walls.

- The water level in the creek may fluctuate and be at higher elevation at the time of construction than indicated in the report.
- The existing timber piles should not be extracted, but cut-off at a depth 0.3 m below the founding level to allow for construction of new foundations.
- Cobbles or other buried obstructions may be encountered during excavation in the existing embankment fill and may interfere with installation of the temporary roadway protection system, if required. Suggested wording for an NSSP on obstructions is included in Appendix F.
- The Contractor's selection of construction equipment and methodology should include assessment of the capability of the existing embankment and very soft to firm silty clay foundation 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. An NSSP to this effect is included in Appendix F.

18. CLOSURE

Engineering analysis and preparation of this report was carried out by Ms. Anna Piascik, P.Eng. and Mr. Mark Farrant, P.Eng. The report was reviewed by 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.

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			

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 16-01

METRIC

W.P. 2016-11032 | LOCATION Gullwing Creek Bridge N 5 531 009.3 E 314 811.3 | ORIGINATED BY TM

HWY Henderson Loop Rd. | BOREHOLE TYPE Hollow Stem Augers/Dynamic Cone Penetration Test | COMPILED BY AN

DATUM Geodetic | DATE 2016.09.14 - 2016.09.14 | CHECKED BY AMP

ELEV / DEPTH (m)	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
353.9	GROUND SURFACE				
0.0	**SAND** and **GRAVEL**, trace silt Compact Brown Moist (FILL)	1	SS	23	50 46 4 (SI+CL)
352.5 / 1.4	Silty **CLAY**, trace sand Firm to Stiff Brown to Grey Moist to Wet (Weathered Crust)	2	SS	15	
		3	SS	10	
		4	SS	7	0 8 63 29
		5	SS	5	
348.3 / 5.6	Silty **CLAY**, trace sand, occasional clayey silt and silt seams Very Soft to Soft Grey Wet	6	SS	0	
		7	SS	0	0 0 32 68
		8	SS	0	

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (353, 352, 351, 350, 349, 348, 347, 346, 345, 344); 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

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 16-01

METRIC

W.P. 2016-11032	LOCATION Gullwing Creek Bridge N 5 531 009.3 E 314 811.3		ORIGINATED BY TM
HWY Henderson Loop Rd.	BOREHOLE TYPE Hollow Stem Augers/Dynamic Cone Penetration Test		COMPILED BY AN
DATUM Geodetic	DATE 2016.09.14 - 2016.09.14		CHECKED BY AMP

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/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	Continued From Previous Page										
	Silty **CLAY**, trace sand, occasional clayey silt and silt seams Very Soft to Soft Grey Wet							+			
			9	SS	0		343		○		
							342	+ 1.7			
			10	SS	0		341		○		
							340	+ 2.4			
			11	SS	0				○		
							339	+ 1.8			
			12	SS	0		338		○		
							337	+ 1.2			
			13	SS	0		336		○		
								+ 1.6			
			14	SS	0		335		├─┤○		0 0 77 23
								+ 1.0			
	clayey silt seam						334				

Continued Next Page

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

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

RECORD OF BOREHOLE No 16-01

METRIC

W.P. 2016-11032	LOCATION Gullwing Creek Bridge N 5 531 009.3 E 314 811.3	ORIGINATED BY TM
HWY Henderson Loop Rd.	BOREHOLE TYPE Hollow Stem Augers/Dynamic Cone Penetration Test	COMPILED BY AN
DATUM Geodetic	DATE 2016.09.14 - 2016.09.14	CHECKED BY AMP

ELEV DEPTH	SOIL PROFILE DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page						
	Silty **CLAY**, trace sand, occasional clayey silt and silt seams Very Soft to Soft Grey Wet						
					333		
		15	SS	0			
					332		
					331		
					330		
		16	SS	3			
					329		
					328		
					327		
		17	SS	0			0 0 75 25
					326		
					325		
					324		

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

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

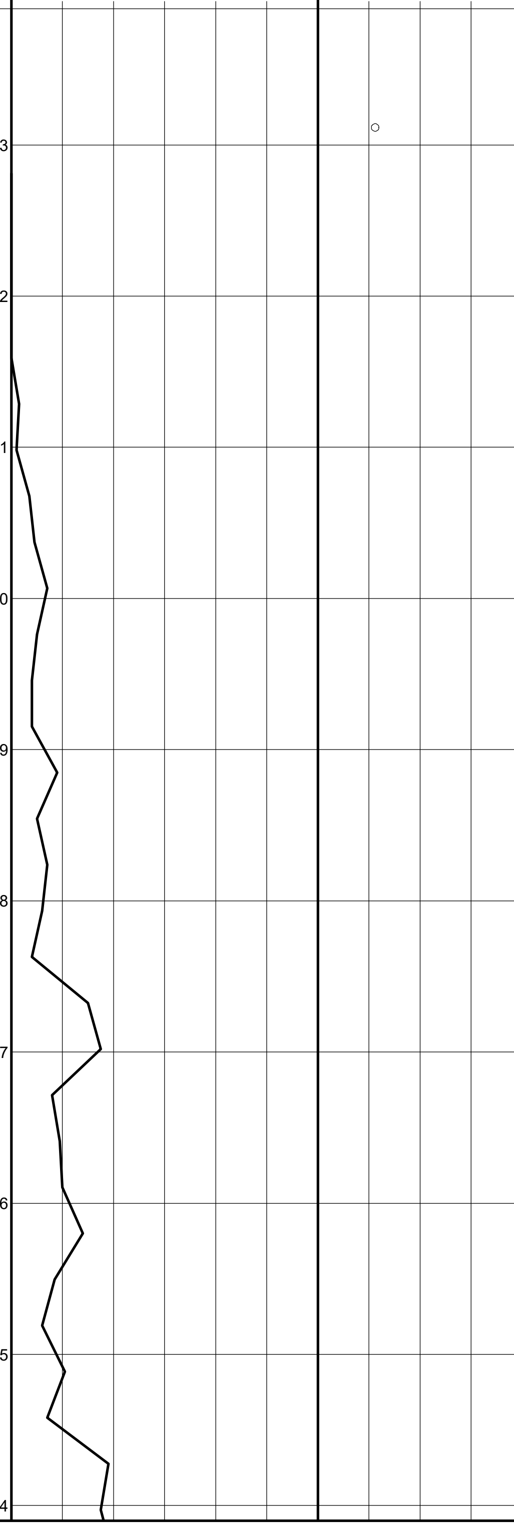
20 / 15 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 16-01

METRIC

W.P.	2016-11032	LOCATION	Gullwing Creek Bridge N 5 531 009.3 E 314 811.3	ORIGINATED BY	TM
HWY	Henderson Loop Rd.	BOREHOLE TYPE	Hollow Stem Augers/Dynamic Cone Penetration Test	COMPILED BY	AN
DATUM	Geodetic	DATE	2016.09.14 - 2016.09.14	CHECKED BY	AMP

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/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	Continued From Previous Page										
322.8	Silty **CLAY**, trace sand, occasional clayey silt and silt seams Very Soft to Soft Grey Wet		18	SS	0		323				
31.1	End of sampling and start DCPT at 31.1m						322, 321, 320, 319, 318, 317, 316, 315, 314				



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+ ³, × ³: Numbers refer to Sensitivity

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

Ministry of Transportation Ontario

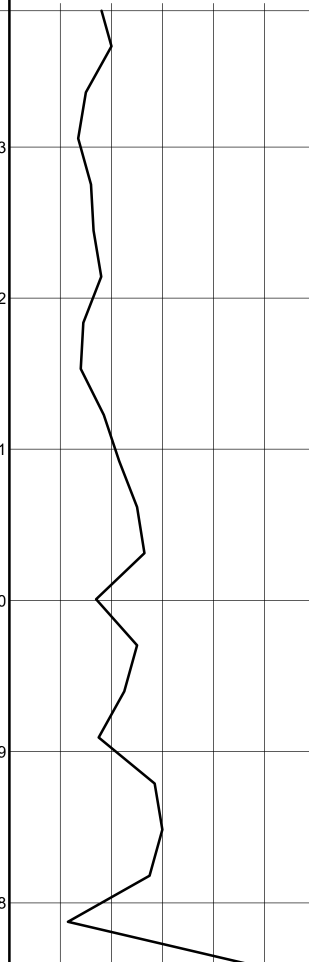
THURBER

RECORD OF BOREHOLE No 16-01

METRIC

W.P.	2016-11032	LOCATION	Gullwing Creek Bridge N 5 531 009.3 E 314 811.3	ORIGINATED BY	TM
HWY	Henderson Loop Rd.	BOREHOLE TYPE	Hollow Stem Augers/Dynamic Cone Penetration Test	COMPILED BY	AN
DATUM	Geodetic	DATE	2016.09.14 - 2016.09.14	CHECKED BY	AMP

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/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	Continued From Previous Page						313				
							312				
							311				
							310				
							309				
							308				
307.6 / 46.3	END OF BOREHOLE AT 46.3m DUE TO CONE REFUSAL. Well installation consists of 25mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen.										



WATER LEVEL READINGS

DATE	DEPTH(m)	ELEV.(m)
2016.09.15	6.1	347.8

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

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

RECORD OF BOREHOLE No 16-02

METRIC

W.P.	2016-11032	LOCATION	Gullwing Creek Bridge N 5 531 004.9 E 314 858.0	ORIGINATED BY	TM
HWY	Henderson Loop Rd.	BOREHOLE TYPE	Hollow Stem Augers/Dynamic Cone Penetration Test	COMPILED BY	AN
DATUM	Geodetic	DATE	2016.09.15 - 2016.09.15	CHECKED BY	AMP

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa	GR	SA	SI	CL
353.9	GROUND SURFACE										
0.0	Gravelly **SAND**, some silt, some clay Compact Brown Moist (FILL)	1	SS								
		2	SS	21		353		21	56	12	11
		3	SS	12		352					
351.6 2.3	Silty **CLAY**, trace sand, some organics (rootlets) in the upper 1.0m zone Soft to Stiff Brown to Grey Moist (Weathered Crust)	4	SS	9		351					
		5	SS	9				0	0	38	62
					∇	350					
		6	SS	2		349					
348.4 5.5	Silty **CLAY**, trace sand, occasional clayey silt and silt seams Soft to Firm Grey Wet					348					
		7	SS	0		347	+ 3.0				
		8	SS	0		346		0	0	32	68
						345	+ 2.1				
		9	SS	0		344					

Column headings: SOIL PROFILE; STRAT PLOT; 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 (%)

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

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

RECORD OF BOREHOLE No 16-02

METRIC

W.P. 2016-11032	LOCATION Gullwing Creek Bridge N 5 531 004.9 E 314 858.0	ORIGINATED BY TM	
HWY Henderson Loop Rd.	BOREHOLE TYPE Hollow Stem Augers/Dynamic Cone Penetration Test	COMPILED BY AN	
DATUM Geodetic	DATE 2016.09.15 - 2016.09.15	CHECKED BY AMP	

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa (Field Vane, Sensitivity)	WATER CONTENT (%)	GR	SA	SI	CL
	Silty **CLAY**, trace sand, occasional clayey silt and silt seams Soft to Firm Grey Wet					+ 2.7					
		10	SS	0	343		○				
						+ 1.9					
					342						
		11	SS	0			○				
					341	+ 2.2					
		12	SS	0	340		├─┤ ○	0	0	73	27
						+ 1.7					
					339						
		13	SS	0			○				
					338	+ 2.1					
		14	SS	0	337		○				
					336						
		15	SS	0			○				
					335						
					334						

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

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

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Ontario

THURBER

RECORD OF BOREHOLE No 16-02

METRIC

W.P. 2016-11032
LOCATION Gullwing Creek Bridge N 5 531 004.9 E 314 858.0
ORIGINATED BY TM
HWY Henderson Loop Rd.
BOREHOLE TYPE Hollow Stem Augers/Dynamic Cone Penetration Test
COMPILED BY AN
DATUM Geodetic
DATE 2016.09.15 - 2016.09.15
CHECKED BY AMP

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	GR	SA	SI	CL
	Silty **CLAY**, trace sand, occasional clayey silt and silt seams Soft to Firm Grey Wet						333				
			16	SS	0		332	0	0	71	29
							331				
							330				
			17	SS	0		329				
							328				
							327				
			18	SS	0		326				
							325				
							324				

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 (%)

Continued Next Page

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

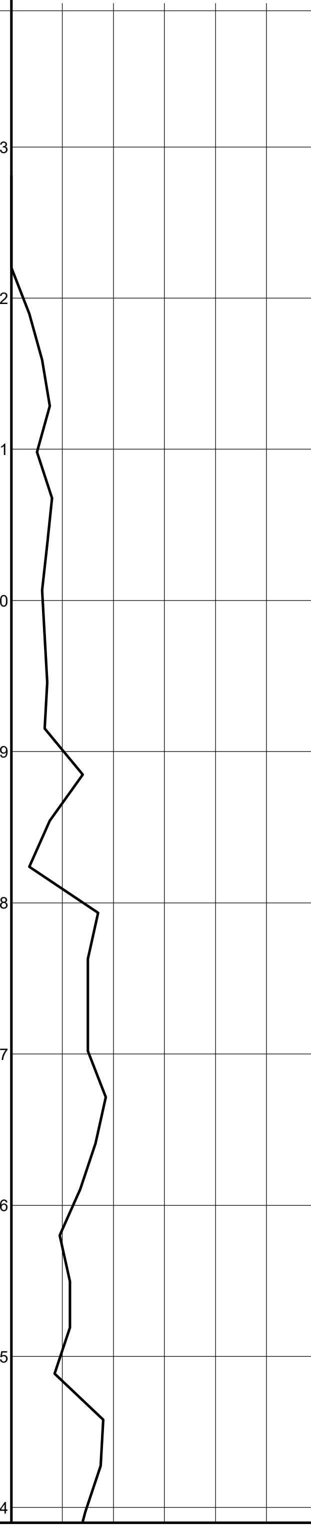
RECORD OF BOREHOLE No 16-02

METRIC

W.P.	2016-11032	LOCATION	Gullwing Creek Bridge N 5 531 004.9 E 314 858.0	ORIGINATED BY	TM
HWY	Henderson Loop Rd.	BOREHOLE TYPE	Hollow Stem Augers/Dynamic Cone Penetration Test	COMPILED BY	AN
DATUM	Geodetic	DATE	2016.09.15 - 2016.09.15	CHECKED BY	AMP

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT kN/m3	GR	SA	SI	CL
	Silty **CLAY**, trace sand, occasional clayey silt and silt seams Soft to Firm Grey Wet	19	SS	0	323			0	0	81	19
322.8 31.1	End of sampling and start DCPT at 31.1m				322 – 314						



Continued Next Page

+ 3, × 3 : Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

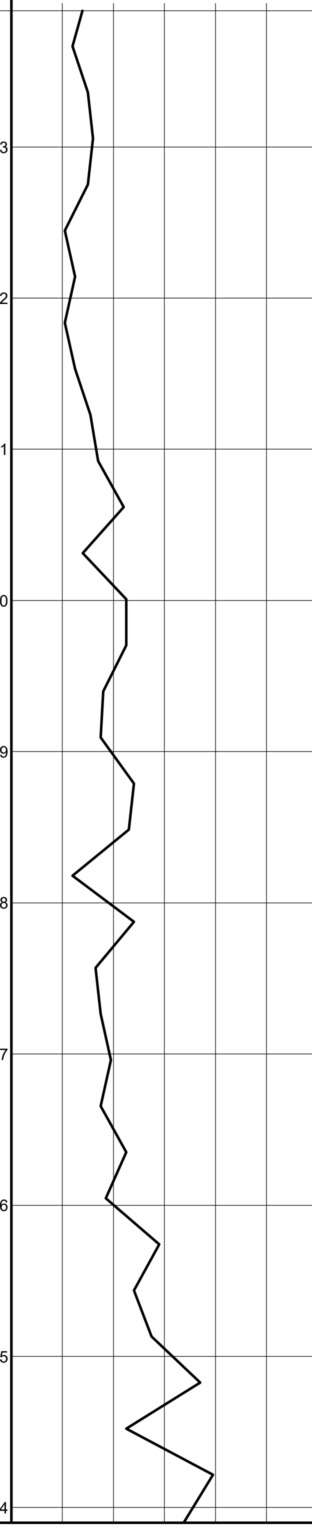
THURBER

RECORD OF BOREHOLE No 16-02

METRIC

W.P.	2016-11032	LOCATION	Gullwing Creek Bridge N 5 531 004.9 E 314 858.0	ORIGINATED BY	TM
HWY	Henderson Loop Rd.	BOREHOLE TYPE	Hollow Stem Augers/Dynamic Cone Penetration Test	COMPILED BY	AN
DATUM	Geodetic	DATE	2016.09.15 - 2016.09.15	CHECKED BY	AMP

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/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	Continued From Previous Page						313				
							312				
							311				
							310				
							309				
							308				
							307				
							306				
							305				
							304				



Continued Next Page

+³, ×³: Numbers refer to Sensitivity

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

ONTMT4S MTO-14504.GPJ 2015TEMPLATE(MTO).GDT 12/8/16

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 16-02

METRIC

W.P. 2016-11032 LOCATION Gullwing Creek Bridge N 5 531 004.9 E 314 858.0 ORIGINATED BY TM

HWY Henderson Loop Rd. BOREHOLE TYPE Hollow Stem Augers/Dynamic Cone Penetration Test COMPILED BY AN

DATUM Geodetic DATE 2016.09.15 - 2016.09.15 CHECKED BY AMP

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/m3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	Continued From Previous Page										
303.6 50.3	END OF BOREHOLE AT 50.3m. WATER LEVEL IN OPEN BOREHOLE AT 4.5m DEPTH. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND CUTTINGS TO SURFACE.										

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

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

ONTMT4S MTO-14504.GPJ 2015TEMPLATE(MTO).GDT 12/8/16

Appendix B

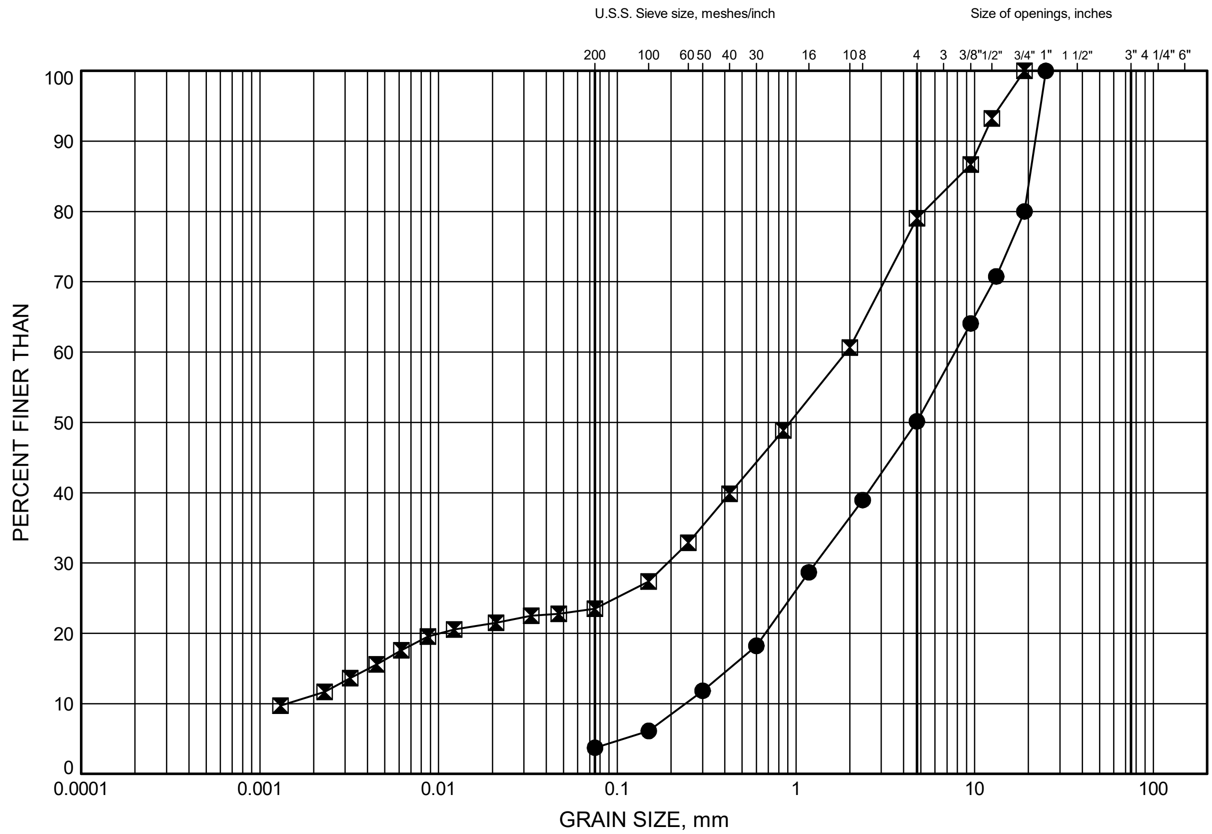
Geotechnical and Analytical Laboratory Test Results

Gullwing Creek Bridge

GRAIN SIZE DISTRIBUTION

FIGURE B1

Sand and Gravel to Gravelly Sand Fill



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	16-01	1.07	352.87
⧗	16-02	1.07	352.85

GRAIN SIZE DISTRIBUTION - THURBER MTO-14504.GPJ 11/10/16

Date April 2017

W.P. 2016-11032

THURBER

Prep'd MFA

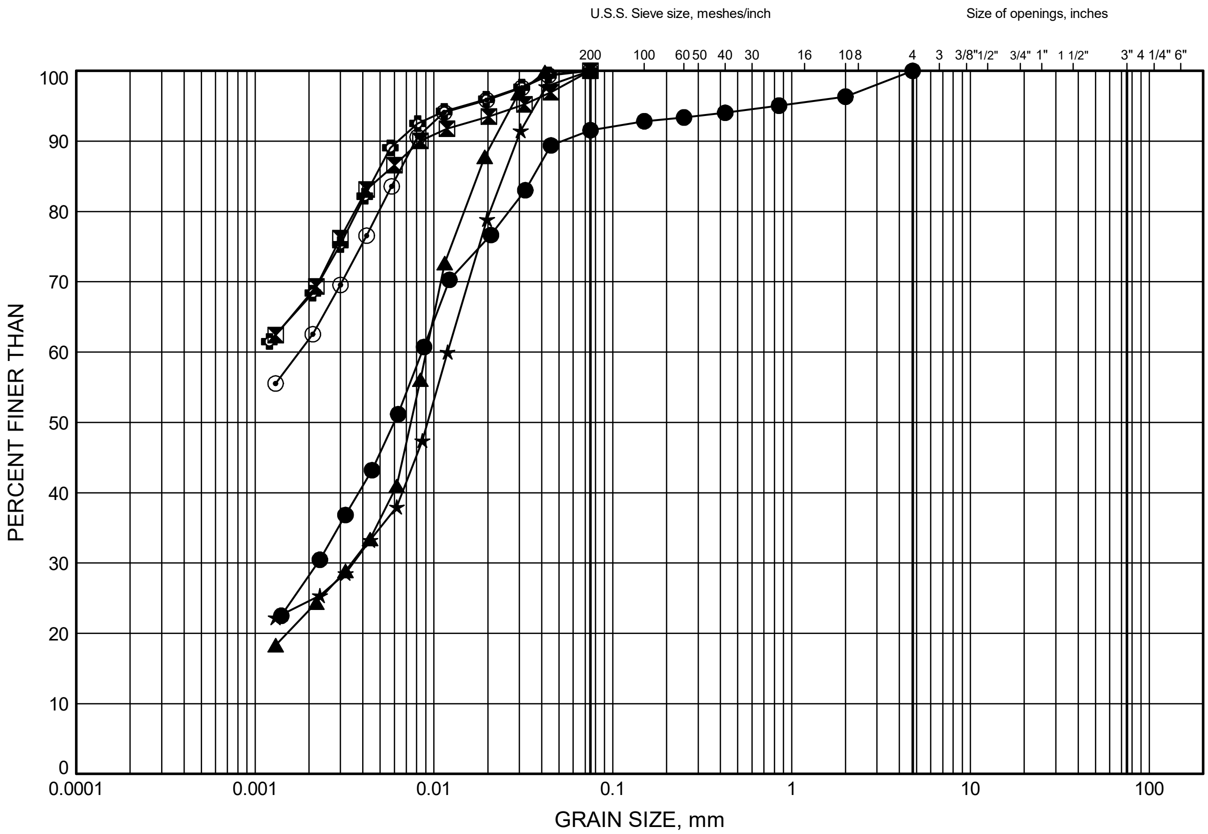
Chkd. AMP

Gullwing Creek Bridge

GRAIN SIZE DISTRIBUTION

FIGURE B2

Silty Clay



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	16-01	3.35	350.59
⧗	16-01	7.92	346.02
▲	16-01	18.59	335.35
★	16-01	27.74	326.20
⊙	16-02	3.35	350.56
✿	16-02	7.92	345.99

GRAIN SIZE DISTRIBUTION - THURBER MTO-14504.GPJ 11/10/16

Date April 2017

W.P. 2016-11032

THURBER

Prep'd MFA

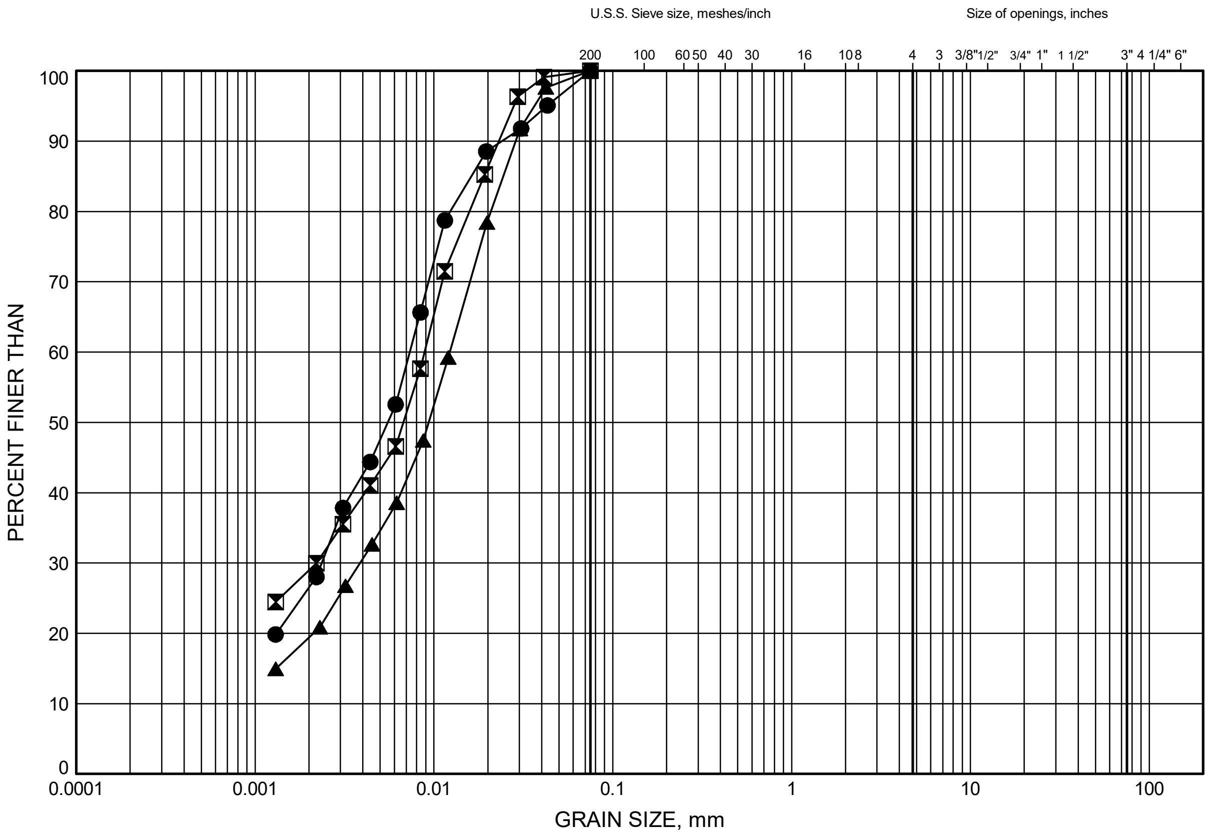
Chkd. AMP

Gullwing Creek Bridge

GRAIN SIZE DISTRIBUTION

FIGURE B3

Silty Clay



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	16-02	14.02	339.89
⧗	16-02	21.64	332.27
▲	16-02	30.78	323.13

GRAIN SIZE DISTRIBUTION - THURBER MTO-14504.GPJ 11/10/16

Date April 2017

W.P. 2016-11032

THURBER

Prep'd MFA

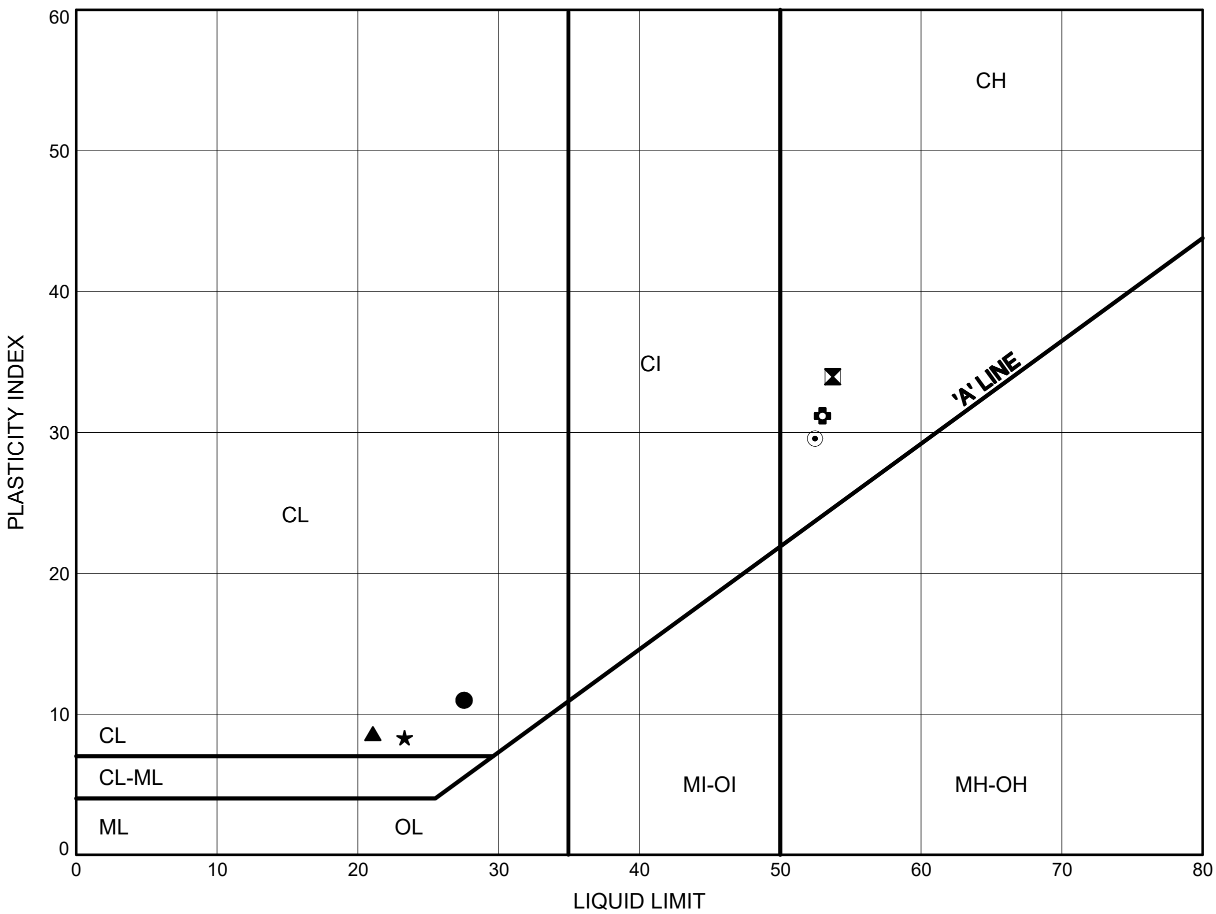
Chkd. AMP

Gullwing Creek Bridge

ATTERBERG LIMITS TEST RESULTS

FIGURE B4

Silty CLAY



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	16-01	3.35	350.59
⧗	16-01	7.92	346.02
▲	16-01	18.59	335.35
★	16-01	27.74	326.20
⊙	16-02	3.35	350.56
✿	16-02	7.92	345.99

THURBALT MTO-14504.GPJ 11/29/16

Date April 2017
W.P. 2016-11032

THURBER

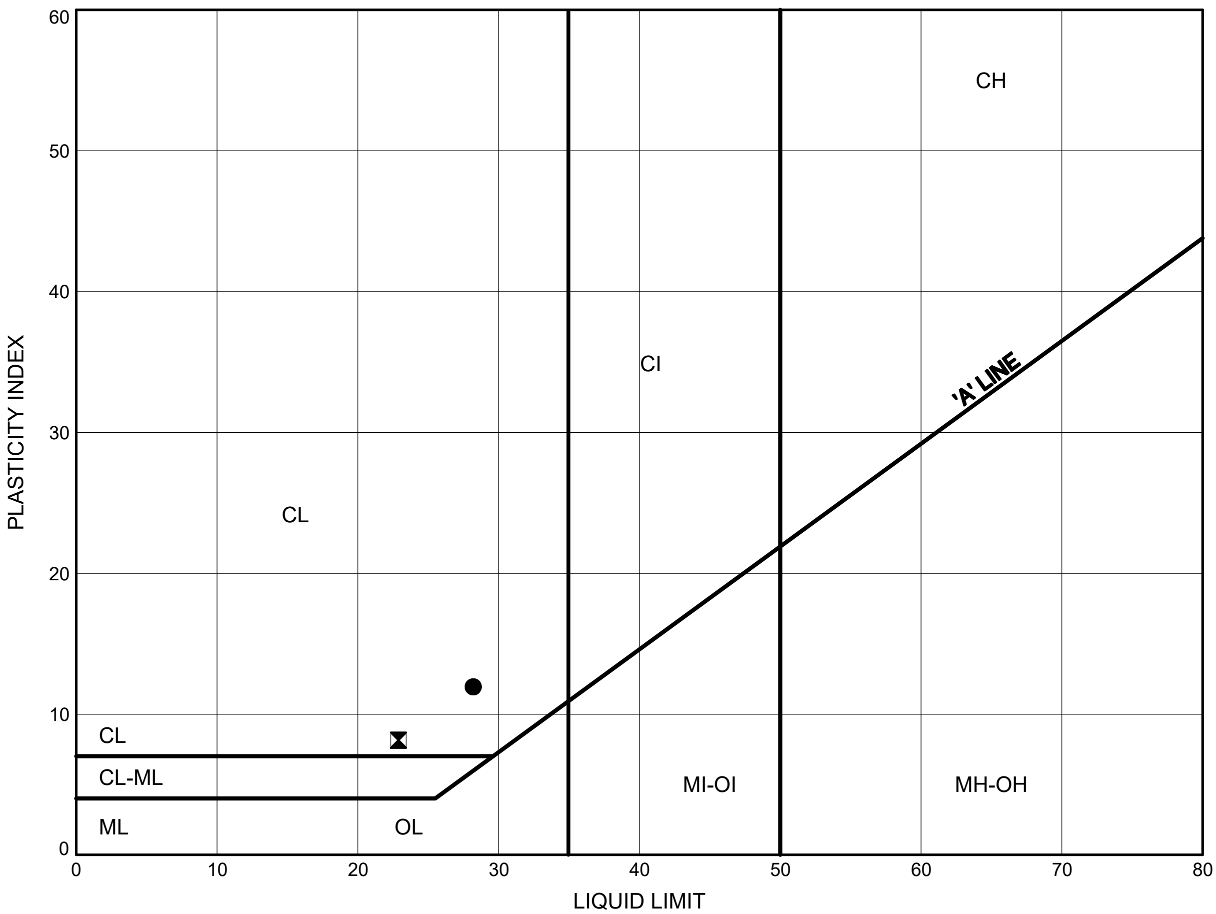
Prep'd AN
Chkd. AMP

Gullwing Creek Bridge

ATTERBERG LIMITS TEST RESULTS

FIGURE B5

Silty CLAY



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	16-02	14.02	339.89
⧗	16-02	30.78	323.13

THURBALT MTO-14504.GPJ 11/29/16

Date April 2017

W.P. 2016-11032

THURBER

Prep'd AN

Chkd. AMP

SGS Canada Inc.
P.O. Box 4300 - 185 Concession St.
Lakefield - Ontario - KOL 2HO
Phone: 705-652-2000 FAX: 705-652-6365

Project : 14504

02-November-2016

Thurber Engineering Ltd.
Attn : Mark Farrant

103, 2010 Winston Park Drive
Oakville, ON
L6H 5R7,

Phone: 905-829-8666 x 228
Fax:

Date Rec. : 27 October 2016
LR Report: **CA14590-OCT16**
Reference: 14504 Mark Farrant

Copy: #1

CERTIFICATE OF ANALYSIS

Final Report

Analysis	**1: Analysis Start Date**	**2: Analysis Start Time**	**3: Analysis Approval Date**	**4: Analysis Approval Time**	**5: BH-1, SS#3, 7.5'-9.5'**
Sample Date & Time					20-Oct-16
Temperature Upon Receipt [°C]	---	---	---	---	14.0
Corrosivity Index [none]	02-Nov-16	16:55	02-Nov-16	16:55	1
pH [no unit]	31-Oct-16	10:29	01-Nov-16	08:56	6.37
Soil Redox Potential [mV]	28-Oct-16	17:11	31-Oct-16	13:39	270
Sulphide [%]	31-Oct-16	13:26	31-Oct-16	14:04	< 0.02
% Moisture (wet wt) [%]	28-Oct-16	08:04	31-Oct-16	08:58	23.9
pH [no unit]	28-Oct-16	07:54	31-Oct-16	08:49	6.84
Chloride [µg/g]	28-Oct-16	20:20	01-Nov-16	11:12	37
Sulphate [µg/g]	28-Oct-16	20:20	01-Nov-16	11:12	18
Conductivity [uS/cm]	28-Oct-16	07:54	31-Oct-16	08:49	145
Resistivity (calculated) [Ohms.cm]	02-Nov-16	16:54	02-Nov-16	16:54	6900

Temperature of Samples upon receipt 14 degrees C
Cooling agent present
Custody Seal not present

Corrosivity Index is based on the American Water Works 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

Deanna Edwards, B.Sc, C.Chem
Project Specialist
Environmental Services, Analytical

OnLine LIMS

0000823873

SGS Canada Inc.
P.O. Box 4300 - 185 Concession St.
Lakefield - Ontario - KOL 2HO
Phone: 705-652-2000 FAX: 705-652-6365

Method Descriptions

Parameter	SGS Method Code	Reference Method Code
Anions by IC	ME-CA-[ENV]IC-LAK-AN-001	EPA300/MA300-Ions1.3
Carbon/Sulphur	ME-CA-[ENV]ARD-LAK-AN-020	ASTM E1918
Conductivity	ME-CA-[ENV]EWL-LAK-AN-006	SM 2510
Metals Prep	ME-CA-[ENV]ARD-LAK-AN-013	
pH	ME-CA-[ENV]EWL-LAK-AN-001	SM 4500

SGS Canada Inc.
P.O. Box 4300 - 185 Concession St.
Lakefield - Ontario - KOL 2HO
Phone: 705-652-2000 FAX: 705-652-6365

Project : 14504
LR Report : **CA14590-OCT16**

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: DIO0421-OCT16												
Chloride	0.4	µg/g	<0.4		1	20	104	80	120	103	75	125
Sulphate	0.4	µg/g	<0.4		12	20	98	80	120	100	75	125
Carbon/Sulphur - QCBatchID: ECS0038-OCT16												
Sulphide	0.02	%	<0.02		NV	20	102	80	120			
pH - QCBatchID: ARD0091-OCT16												
pH	0.05	no unit			0	20	101	80	120			

SGS Canada Inc.
P.O. Box 4300 - 185 Concession St.
Lakefield - Ontario - KOL 2HO
Phone: 705-652-2000 FAX: 705-652-6365

Project : 14504

17-November-2016

Thurber Engineering Ltd.
Attn : Mark Farrant

103, 2010 Winston Park Drive
Oakville, ON
L6H 5R7,

Phone: 905-829-8666 x 228
Fax:

Date Rec. : 19 September 2016
LR Report: **CA13497-SEP16**
Reference: 14504 Mark Farrant

Copy: #1

CERTIFICATE OF ANALYSIS

Final Report

Analysis	1: Analysis Start Date	2: Analysis Start Time	3: Analysis Approval Date	4: Analysis Approval Time	5: MDL	6: Gullwing Creek
Sample Date & Time						13-Sep-16 11:00
Temperature Upon Receipt [°C]	---	---	--	--	---	19.0
pH [no unit]	20-Sep-16	07:23	21-Sep-16	11:11	0.05	7.96
Conductivity [µS/cm]	20-Sep-16	07:23	21-Sep-16	11:11	2	109
Resistivity (calculated) [Ohms.cm]	21-Sep-16	10:38			---	917
Redox Potential [mV]	19-Sep-16	16:42	20-Sep-16	10:53	---	218
Chloride [mg/L]	20-Sep-16	07:42	21-Sep-16	10:05	0.04	0.95
Sulphate [mg/L]	20-Sep-16	07:42	21-Sep-16	10:05	0.04	1.8
Sulphide [mg/L]	20-Sep-16	15:10	21-Sep-16	09:09	0.006	0.026
Corrosivity Index [none]	21-Sep-16	12:25	21-Sep-16	12:25		14

Temperature of samples upon receipt 19 degrees C
Cooling Agent Present
Custody Seal Present and Intact

Sulphide bottle received broken, solution from the general bottle containing zero headspace was used to fill a new Sulphide bottle.

Corrosivity Index is based on the American Water Works 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

Deanna Edwards, B.Sc, C.Chem
Project Specialist
Environmental Services, Analytical

OnLine LIMS

0000837390

SGS Canada Inc.
P.O. Box 4300 - 185 Concession St.
Lakefield - Ontario - KOL 2HO
Phone: 705-652-2000 FAX: 705-652-6365

Project : 14504
LR Report : **CA13497-SEP16**

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

SGS Canada Inc.
P.O. Box 4300 - 185 Concession St.
Lakefield - Ontario - KOL 2HO
Phone: 705-652-2000 FAX: 705-652-6365

Project : 14504

LR Report : **CA13497-SEP16**

Quality Control Report

Inorganic Analysis												
Parameter	Reporting Limit	Unit	Method Blank		RPD	Acceptance Criteria	LCS / Spike Blank			Matrix Spike / Reference Material		
							Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
						%		Low	High		Low	High
Anions by IC - QCBatchID: DIO0257-SEP16												
Chloride	0.04	mg/L	<0.04		4	20	100	80	120	104	75	125
Sulphate	0.04	mg/L	<0.04		7	20	95	80	120	103	75	125
Conductivity - QCBatchID: EWL0255-SEP16												
Conductivity	2	µS/cm	< 2		0	10	99	90	110	NA		
pH - QCBatchID: EWL0255-SEP16												
pH	0.05	no unit	NA		0		100			NA		
Redox Potential - QCBatchID: EWL0252-SEP16												
Redox Potential	no	mV	NA		9	20	100	80	120	NA		
Sulphide by SFA - QCBatchID: SKA0140-SEP16												
Sulphide	0.006	mg/L	<0.02		ND	20	95	80	120	NV	75	125

Appendix C

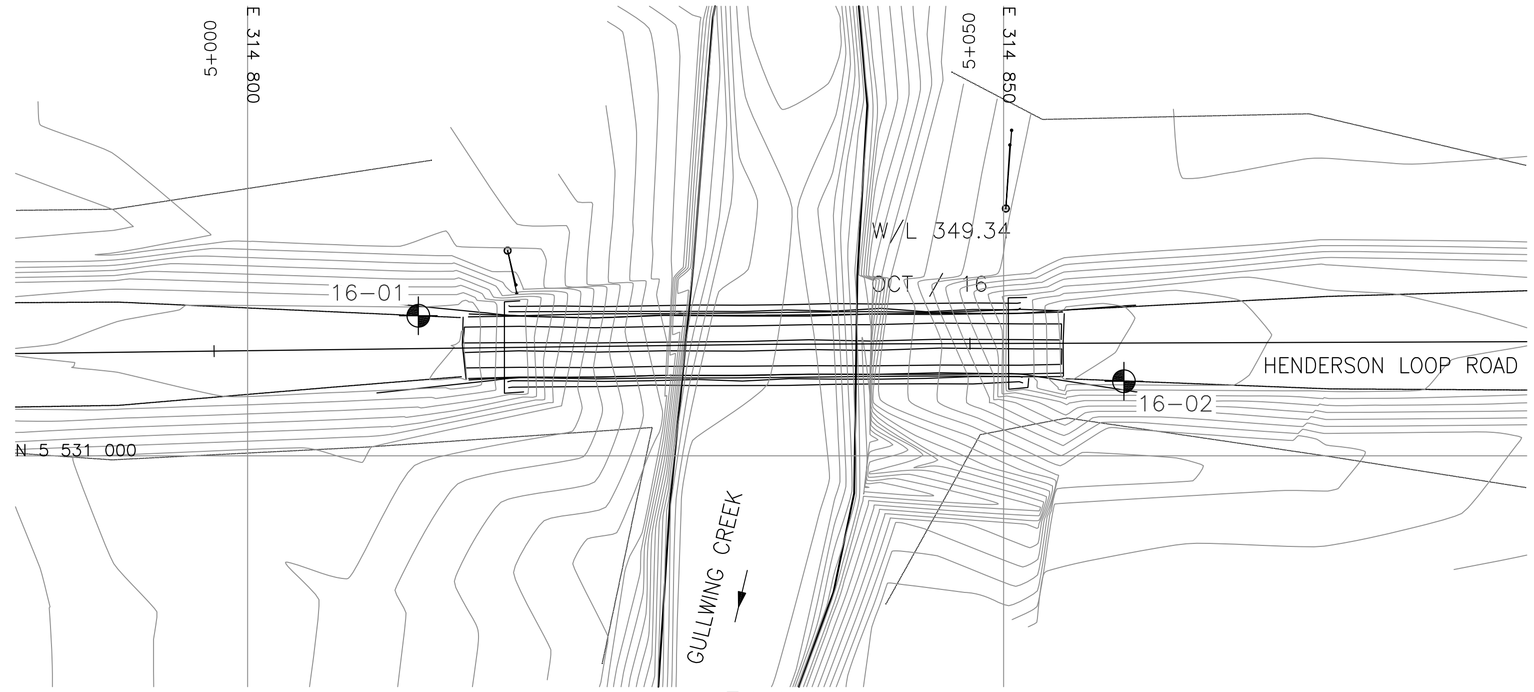
Selected Site Photographs

Photograph 1 – Henderson Loop Road and Gullwing Creek Bridge; Looking West

Photograph 2 – Gullwing Creek Bridge; Looking West

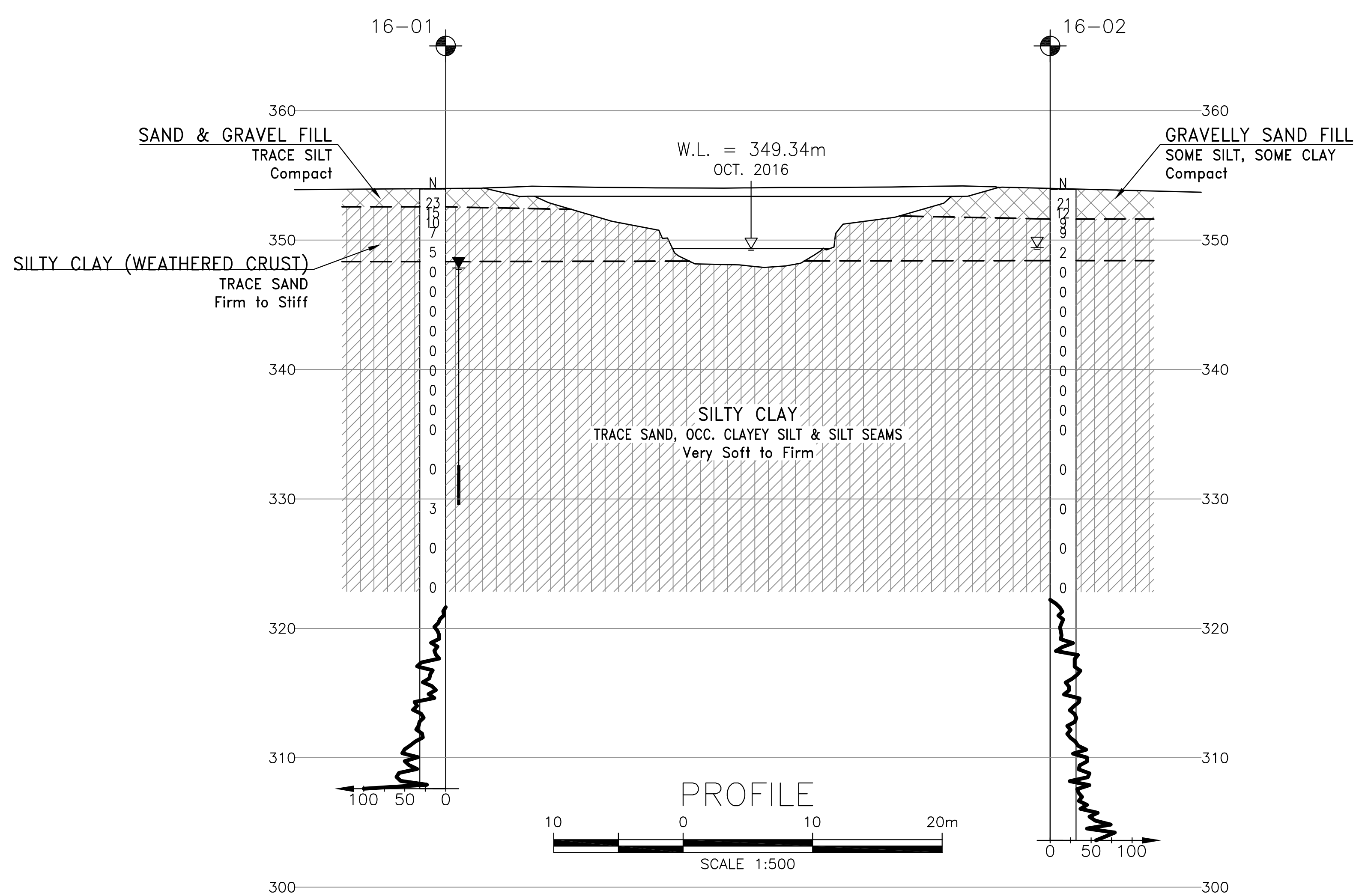
Appendix D

Borehole Locations and Soil Strata Drawing



PLAN

10 0 10 20m
SCALE 1:500



PROFILE

10 0 10 20m
SCALE 1:500

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

LICENSED PROFESSIONAL ENGINEER
P. R. CHATTERJI
April 7/17
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
M.E. FARRANT
100053767
Apr. 7/17
PROVINCE OF ONTARIO

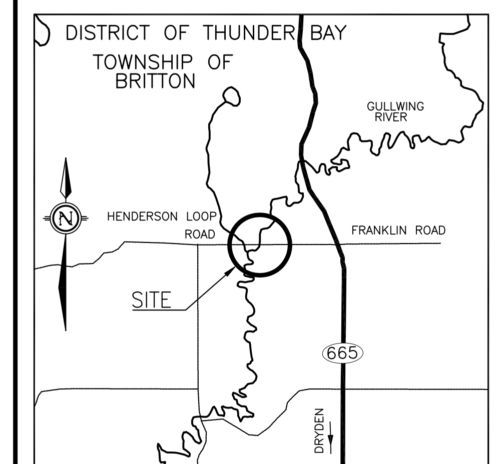
CONT No
WP No 6113-15-00

HENDERSON LOOP ROAD
GULLWING CREEK
BRIDGE REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET
7

Ontario

THURBER ENGINEERING LTD.



KEYPLAN

LEGEND

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

NO	ELEVATION	NORTHING	EASTING
16-01	353.9	5 531 009.3	314 811.3
16-02	353.9	5 531 004.9	314 858.0

-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.
3) MTM Zone 16 co-ordinate system used to obtain borehole Northings and Eastings.

GEOCRES No. 52F-49

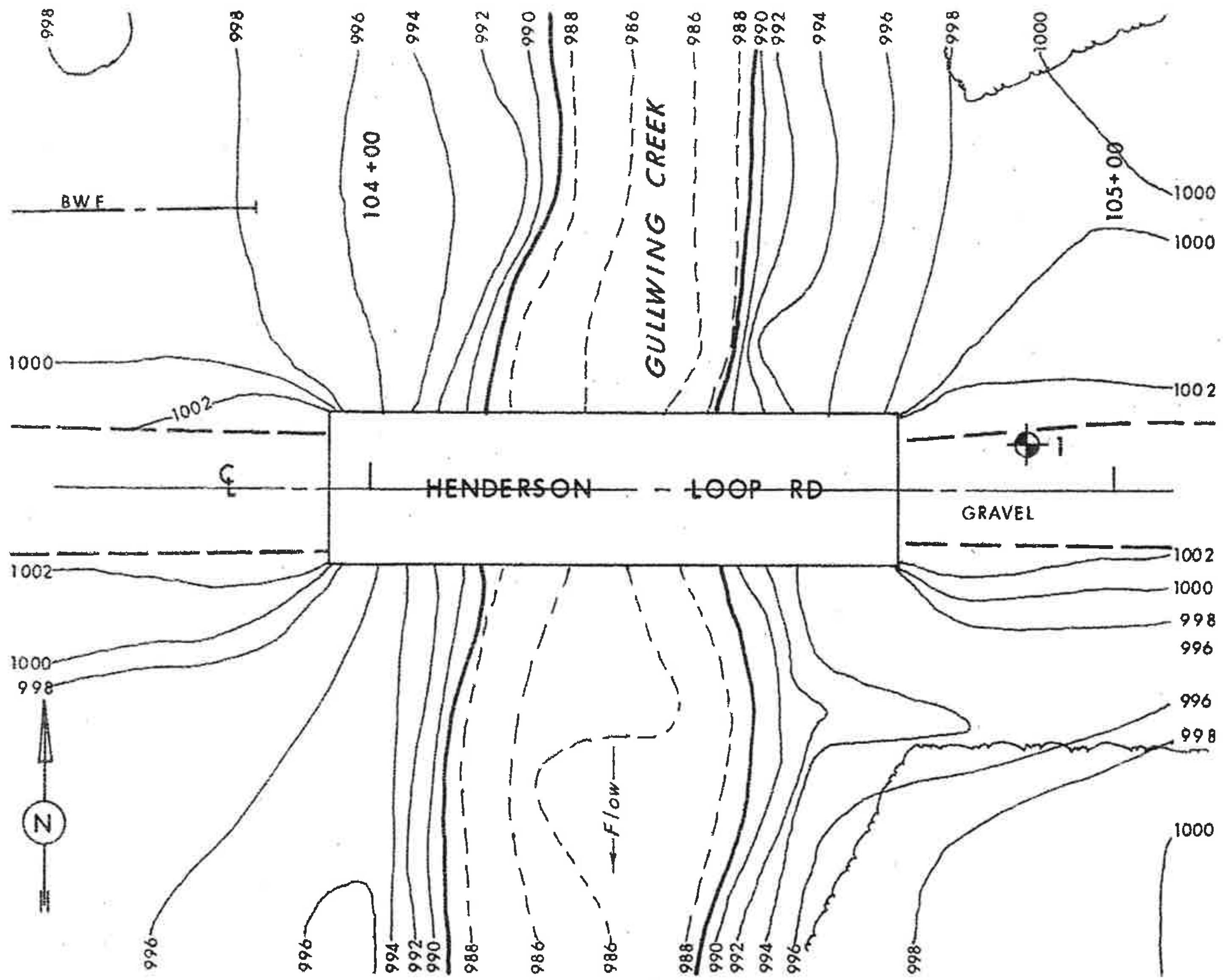
REVISIONS	DATE	BY	DESCRIPTION

DESIGN AMP	CHK PKC	CODE	LOAD	DATE APR 2017
DRAWN MFA	CHK AMP	SITE	STRUCT	DWG 2

FILENAME: H:\Drafting\14000\14504\TED-14504-BHPP-GC.dwg
PLOTDATE: 4/7/2017 1:07 PM

Appendix E

Record of Borehole Sheets and Borehole Location and Soil Strata Drawing
Geocres No 52F-18



LEGEND:

Bore Hole & Cone

Water Level

**GULLWING CREEK
AND
HENDERSON LOOP ROAD**

KENORA DIST.
BRITTON TWP
CON II & III LOT 6

SITE 41S-24 DIST 20

WO 77-67009

PLAN
SCALE: 1" = 20'

REF No E-5605-1; NOV 1977

Figure No 1

EXPLANATION OF TERMS USED IN REPORT

'N' VALUE: AN INDICATOR OF SUBSOIL QUALITY. IT IS OBTAINED FROM THE STANDARD PENETRATION TEST (CSA STD. A119.1). SPT 'N' VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 2 INCH O.D. SPLIT-BARREL SAMPLER TO PENETRATE 12 INCHES INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WEIGHING 140 POUNDS, FALLING FREELY A DISTANCE OF 30 INCHES. FOR PENETRATIONS OF LESS THAN 12 INCHES 'N' VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. 'N' VALUES CORRECTED FOR OVERBURDEN PRESSURE ARE DENOTED THUS $\overline{N}$.

DYNAMIC CONE PENETRATION TEST (CSA STD. A119.3): CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (2" O.D. 60 CONE ANGLE) DRIVEN BY 350 FT-LB IMPACTS ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 12 INCH ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOIL QUALITY: SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSITY.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH AS FOLLOWS:

S_u (PSF)	0 - 250	250 - 500	500 - 1000	1000 - 2000	2000 - 4000	> 4000
	VERY SOFT	*SOFT*	*FIRM*	*STIFF*	*VERY STIFF*	*HARD*

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF SPT 'N' VALUES AS FOLLOWS:

'N' (BLOW/FT)	0 - 5	5 - 10	10 - 30	30 - 50	> 50
	VERY LOOSE	*LOOSE*	*COMPACT*	*DENSE*	*VERY DENSE*

ROCK QUALITY: ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND/OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH DRILLED IN THAT CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE NATURALLY FRACTURED CORE PIECES, 4"+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY, IS:

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	*POOR*	*FAIR*	*GOOD*	*EXCELLENT*

JOINTING AND BEDDING:

SPACING	2"	2" - 12"	1' - 3'	3' - 10'	> 10'
JOINTING	*VERY CLOSE*	*CLOSE*	*MOD. CLOSE*	*WIDE*	*VERY WIDE*
BEDDING	*VERY THIN*	*THIN*	*MEDIUM*	*THICK*	*VERY THICK*

ABBREVIATIONS & SYMBOLS

LABORATORY TESTING

TRIAXIAL TESTS ARE DESCRIBED IN TERMS OF WHETHER THEY ARE CONSOLIDATED (C) OR NOT (U)
ISOTROPICALLY (I) OR NOT (A)
AND SHEARED DRAINED (D) OR UNDRAINED (U)
WITH PORE PRESSURE MEASUREMENTS (BAR OVER SYMBOLS)
EG. $\overline{CIU}$ = CONSOLIDATED ISOTROPIC UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENT
UNLESS OTHERWISE SPECIFIED IN REPORT ALL TESTS ARE IN COMPRESSION

FIELD SAMPLING

S S	SPLIT SPOON
W S	WASH SAMPLE
S T	SLOTTED TUBE SAMPLE
B S	BLOCK SAMPLE
C S	CHUNK SAMPLE
T W	THINWALL OPEN
T P	THINWALL PISTON
O S	OSTERBERG SAMPLE
F S	FOIL SAMPLE
R C	ROCK CORE
P H	T.W. ADVANCED HYDRAULICALLY
P M	T.W. ADVANCED MANUALLY

EARTH PRESSURE TERMS

μ	COEFFICIENT OF FRICTION
δ	ANGLE OF WALL FRICTION
k_o	COEFFICIENT OF EARTH PRESSURE AT REST
k_A	COEFFICIENT OF ACTIVE EARTH PRESSURE
k_p	COEFFICIENT OF PASSIVE EARTH PRESSURE
i	ANGLE OF INCLINATION OF SURCHARGE
ω	SLOPE ANGLE-BACKFACE OF WALL
β	ANGLE OF SLOPE
N_γ, N_q, N_c	BEARING CAPACITY FACTORS
D_f	DEPTH OF FOOTING
B, L	FOOTING DIMENSIONS

INDEX PROPERTIES

γ	UNIT WEIGHT OF SOIL (BULK DENSITY)
γ_w	UNIT WEIGHT OF WATER
γ_d	UNIT DRY WEIGHT OF SOIL (DRY DENSITY)
γ'	UNIT WEIGHT OF SUBMERGED SOIL
G_s	SPECIFIC GRAVITY OF SOLIDS
e	VOIDS RATIO
e_o	INITIAL VOIDS RATIO
e_{max}	e IN LOOSEST STATE
e_{min}	e IN DENSEST STATE
D_r	RELATIVE DENSITY = $\frac{e_{max} - e}{e_{max} - e_{min}}$
n	POROSITY
w	WATER CONTENT
w_L	LIQUID LIMIT
w_p	PLASTIC LIMIT
w_S	SHRINKAGE LIMIT
I_p	PLASTICITY INDEX = $w_L - w_p$
I_L	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$
I_c	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$
A_c	ACTIVITY = $\frac{I_p \text{ of soil}}{\% < 2\mu m \text{ Soil Fraction}}$
Om	ORGANIC MATTER CONTENT
S_r	DEGREE OF SATURATION
S	SENSITIVITY = $\frac{S_u \text{(undisturbed)}}{S_u \text{(remoulded)}}$

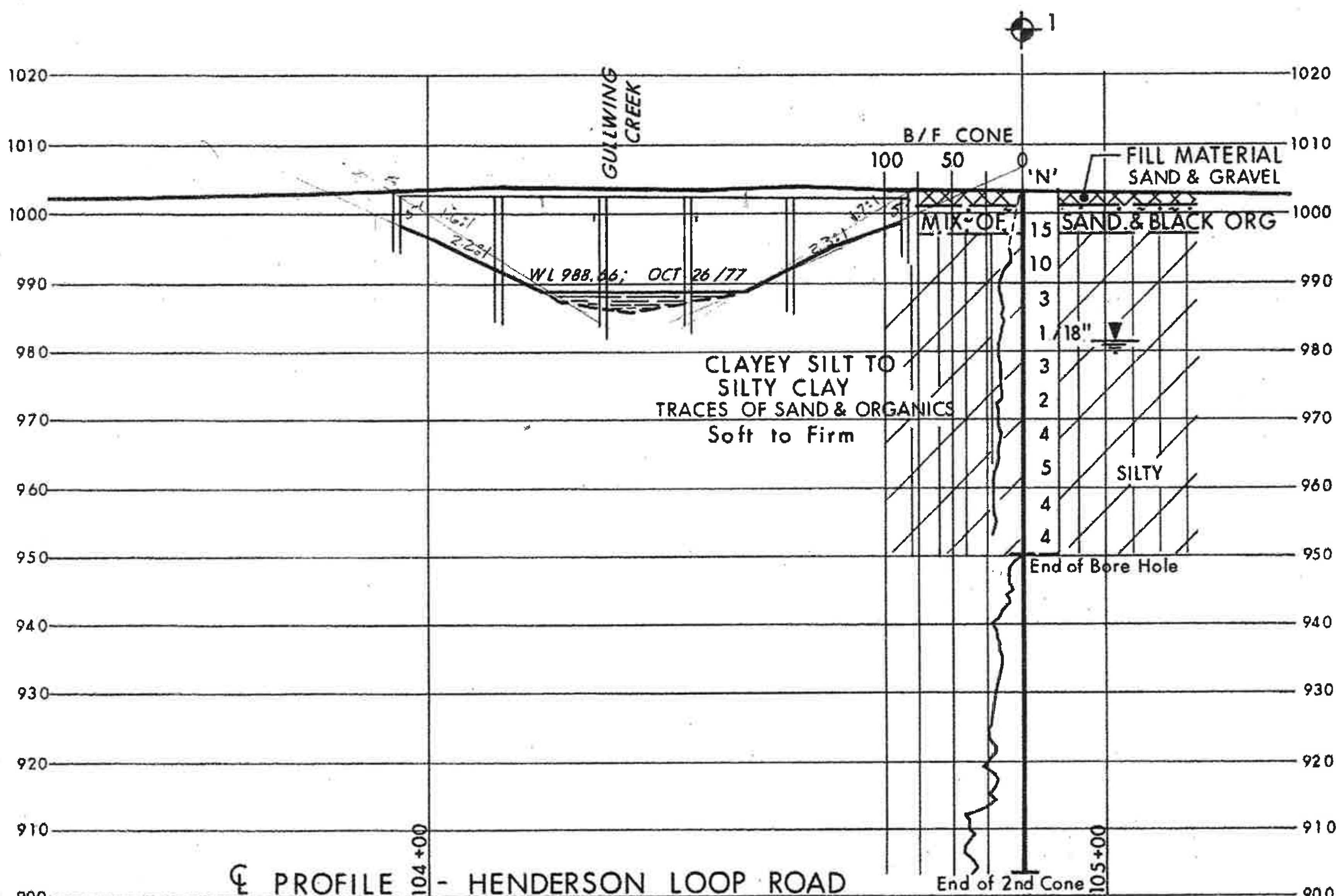
STRENGTH PARAMETERS

ϕ	ANGLE OF SHEARING RESISTANCE
τ_f	PEAK SHEAR STRENGTH
τ_R	RESIDUAL SHEAR STRENGTH
c	COHESION INTERCEPT
$\sigma_1, \sigma_2, \sigma_3$	NORMAL PRINCIPAL STRESSES
u	PORE WATER PRESSURE
u_e	EXCESS u
r_u	PORE PRESSURE RATIO
q_u	UNCONFINED COMPRESSIVE STRENGTH
s_u	UNDRAINED SHEAR STRENGTH
ϵ	LINEAR STRAIN
γ	SHEAR STRAIN
ν	POISSON'S RATIO
E	MODULUS OF ELASTICITY
G	MODULUS OF SHEAR DEFORMATION
k_s	MODULUS OF SUBGRADE REACTION
m, n	STABILITY COEFFICIENTS
A, B	PORE PRESSURE COEFFICIENTS

NOTE: EFFECTIVE STRESS PARAMETERS ARE DENOTED BY USE OF APOSTROPHE ABOVE THE SYMBOL, THUS:
ϕ' = EFFECTIVE ANGLE OF SHEARING RESISTANCE;
σ' = EFFECTIVE NORMAL STRESS

HYDRAULIC TERMS

h	HYDRAULIC HEAD OR POTENTIAL
q	RATE OF DISCHARGE
v	VELOCITY OF FLOW
i	HYDRAULIC GRADIENT
j	SEEPAGE FORCE PER UNIT VOLUME
η	COEFFICIENT OF VISCOSITY
k	COEFFICIENT OF HYDRAULIC CONDUCTIVITY
k_h	k IN HORIZONTAL DIRECTION
k_v	k IN VERTICAL DIRECTION
m_v	COEFFICIENT OF VOLUME CHANGE
c_v	COEFFICIENT OF CONSOLIDATION
C_c	COMPRESSION INDEX
C_r	RECOMPRESSION INDEX
d	DRAINAGE PATH DISTANCE
T_v	TIME FACTOR
U	DEGREE OF CONSOLIDATION
O_r	OVERCONSOLIDATION RATIO (OCR)



℄ PROFILE - HENDERSON LOOP ROAD

SCALE: 1" = 20"

KENORA DIST; BRITTON TWP
CON II & III LOT 6

SITE 41S-24 DIST 20
W O 77-67009

Figure No 2

Ministry of Transportation and Communications
Ontario

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 1

WO 77-67009 LOCATION Sta. 104+88; o/s 6' Lt. ℄ Henderson Loop Road ORIGINATED BY PP

DIST 20 HWY Twp. Rd. BOREHOLE TYPE Continuous Flight Auger & Cone Tests COMPILED BY PP

DATUM Assumed DATE February 12, 1978 CHECKED BY

SOIL PROFILE

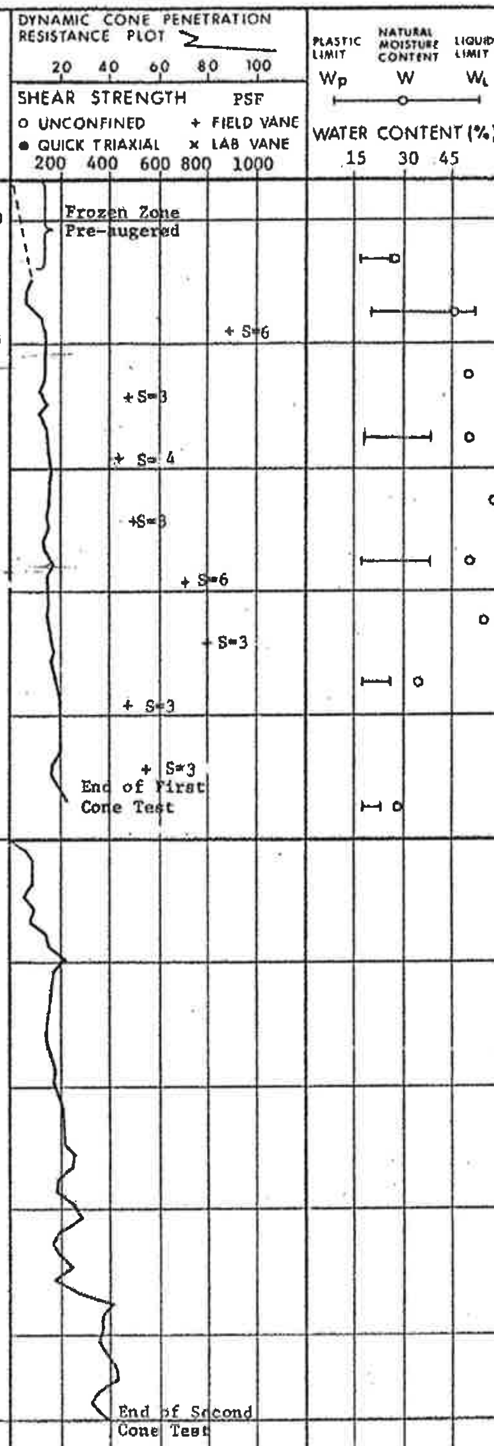
ELEV / DEPTH	DESCRIPTION
1003.2	Ground Level
0.0	Sand & Gravel Fill Material
2.0	Mixture of Sand
997.2	& Black Organics
6.0	Clayey Silt to Silty Clay. Traces of Sand and Organics Soft to Firm Silty
950.2 / 53.0	End of Borehole
903.2 / 100.0	End of Cone Test

Note: The Second Cone Test Was Advanced From The Bottom of Borehole

SAMPLES

NUMBER	TYPE	'N' VALUES	SHEAR STRENGTH (PSF)	WATER CONTENT	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	15			Org. 1.03%
2	SS	10	+ S=6		
3	SS	3	+ S=3		0 0 41 59
4	SS	1/18"	+ S=4		
5	SS	3	+S=3		
6	SS	2	+ S=6		
7	SS	4	+ S=3		0 1 32 67
8	SS	5	+ S=3		
9	SS	4	+ S=3		
10	SS	4			

GROUND WATER CONDITIONS: water level shown at approx. elev. 981



UNIT WEIGHT γ

+3, ×5 : Numbers refer to Sensitivity   15 -◇- 5 (%) STRAIN AT FAILURE (20 above, 10 below)

OFFICE REPORT ON SOIL EXPLORATION

Appendix F

List of Specifications and Suggested Wording for NSSP

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

- OPSS PROV 206
- OPSS PROV 209
- OPSS PROV 501
- OPSS.PROV 517
- OPSS PROV 539
- OPSS PROV 804
- OPSS PROV 902
- OPSS PROV 1010
- OPSD 3090.100

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. 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.

The existing pile foundations should not be extracted. The piles should be cut-off approximately 0.3 m below the founding level of the engineered granular pad or pile cap for the new structure.”

- 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, foundation construction and backfilling should be carried out in the dry."

- Suggested text for NSSP on "Use of Heavy Construction Equipment"

The use of heavy construction equipment and in particular heavy lift cranes may be required during removal of the existing bridge and erection of the new bridge. The impact of the heavy equipment loads on the existing embankment, the soft foundation soils (silty clay) underlying the embankment, and the existing and new bridge foundations must be considered during selection of the methodology and equipment employed for construction.

Prior to commencement of construction, the Contractor shall retain a Geotechnical Consultant to assess the impact of the proposed equipment loads and methodology, and determine requirements and/or restrictions necessary to safely support the loads. All Foundation Engineering services required for this project shall be performed by consultant(s) listed as accepted under the MTO's RAQS for providing services under the specialty of Geotechnical (Structures and Embankments) – High Complexity.

The assessment shall include, but not be limited to, the following:

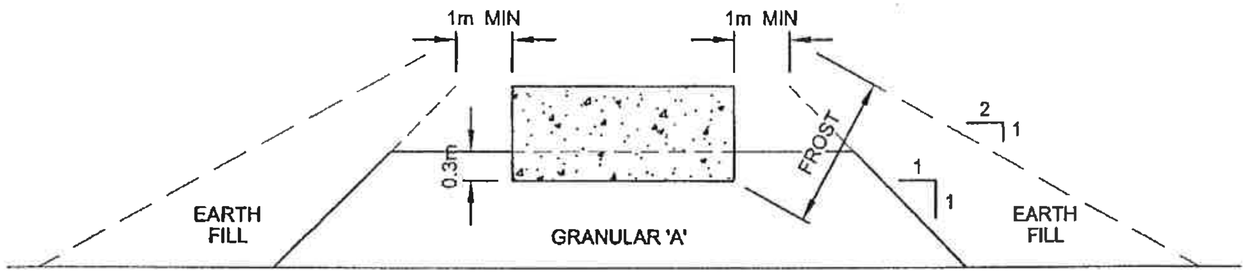
- Determining appropriate setbacks for heavy equipment from the bridge abutments and existing and new foundations;
- Determining the permissible ground pressure that may be applied to the foundation soils by the equipment; and
- Providing recommendations for crane pad design to distribute the crane loads without causing foundation and creek bank failure.

The Contractor shall submit the findings of the geotechnical assessment and details of the proposed equipment and construction methodology to the Contract Administrator for information purposes a minimum of two weeks prior to the start of construction.

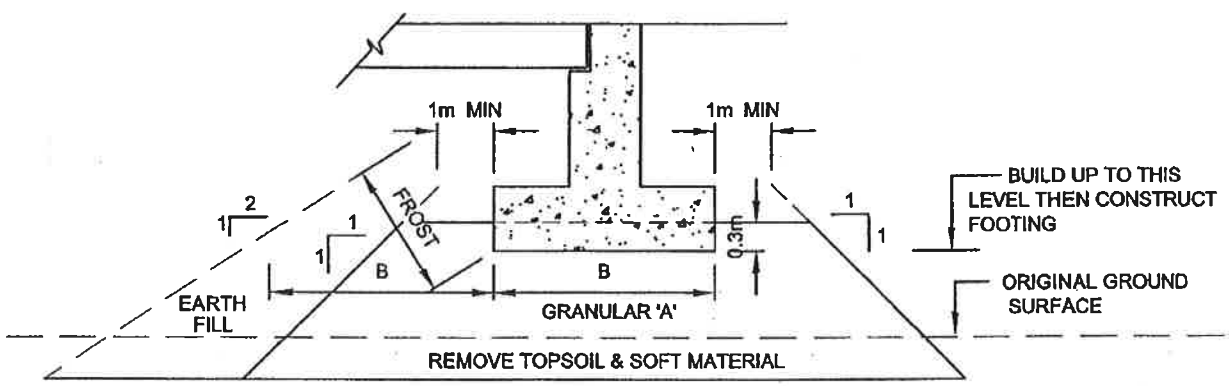
Appendix G

Sketch of Abutment on Compacted Fill (Figure 1)

March , 2004



CROSS-SECTION



LONGITUDINAL SECTION

NOT TO SCALE

NOTES:

1. REMOVE TOPSOIL AND OR SOFT SUBSOIL UNDER AREA OF COMPACTED GRANULAR 'A' AND EARTH FILL.
2. PLACE GRANULAR 'A' AND EARTH FILL TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO O.P.S.S. 501.
3. CONSTRUCT CONCRETE FOOTING.
4. PLACE REMAINDER OF GRANULAR 'A' AND EARTH FILL AS REQUIRED.
5. SOURCE M.T.C. 1982.

ENGINEER AEG	ABUTMENT ON COMPACTED FILL SHOWING GRANULAR A CORE	THURBER
DRAWN SS		
DATE April , 2004		
APPROVED PKC		DWG. NO. FIGURE 1
SCALE NTS		

TED35146 .DWG