

December 23, 2013

## FOUNDATION INVESTIGATION AND DESIGN REPORT

RAGGED CREEK BRIDGE, SITE NO. 44-178  
HIGHWAY 592 - REPLACEMENT OF SIX STRUCTURES  
MINISTRY OF TRANSPORTATION, ONTARIO  
GWP 5265-07-00 WP 5269-07-01

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REPORT

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## **PART A**

FOUNDATION INVESTIGATION REPORT  
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## 1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Morrison Hershfield Limited. (MH) on behalf of the Ministry of Transportation, Ontario (MTO) to provide detail foundation engineering services for the replacement of the Ragged Creek Bridge (Site No. 44-178) over Highway 592 in Huntsville, Ontario. The proposed work is part of the replacement of six bridge structures along Highway 592. The Ragged Creek Bridge is located approximately 200 m north of Supersign Road and approximately 1 km east of Highway 11/Emsdale Road Interchange in Emsdale, Ontario. The location of the existing bridge structure along Highway 592 is shown on the Key Map on Drawing 1.

The Terms of Reference (TOR) for the foundation investigation are outlined in MTO's Request for Proposal, dated September 2011. Golder's proposal (Scope of Work) for foundation engineering services associated with the Ragged Creek Bridge structure is contained in Section 6.8 of MH's Technical Proposal for this assignment. The work was carried out in accordance with Golder's Project Specific Supplementary Specialty Plan for foundation engineering services, dated March 21, 2012.

This report addresses the investigation carried out for the Ragged Creek Bridge structure and the associated approach embankments only.

The purpose of this investigation is to establish the subsurface conditions at the replacement bridge structure location, including the associated approach embankments, by borehole drilling techniques, in situ testing and laboratory testing on selected soil samples. The borehole locations for this investigation were surveyed by Tulloch Geomatics Inc. (Tulloch), a professional surveying company retained by MH. The investigation area is shown in plan on Drawing 2.

## 2.0 SITE DESCRIPTION

The existing Highway 592 alignment is oriented generally in a south-north direction.

In general, the topography along Highway 592 consists of rolling terrain, including lakes, low-lying swamps containing areas of standing water, sparsely to densely populated tree covered areas. Land use in some areas consists of residential/recreational communities. The existing bridge is a single-span rigid frame structure with a span length of 6.1 m. The bridge structure and associated approach embankments are situated on a relatively flat, sparsely to densely treed area with residential/recreational properties to the north and south and with Ragged Creek flowing westerly at this location. The existing ground surface within the limits of the proposed structure and approach embankments is between Elevations 319.7 m and 319.6 m, referenced to Geodetic datum. The existing Highway 592 south and north approach embankments at/near the abutments along the centreline are at Elevation 319.6 m.

## 3.0 INVESTIGATION PROCEDURES

### 3.1 Foundation Investigation

The field work for the proposed bridge structure was carried out between May 28 and 31, 2013 during which time a total of four boreholes (designated as Boreholes B5-01 to B5-04) were advanced at the location of the structure foundation footprints and approach embankments. In addition, Dynamic Cone Penetration Test (DCPT) B5-DC02 was advanced from the bottom of Boreholes B5-01 to B5-04 to determine the depth to refusal



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at these locations and immediately adjacent to Borehole B5-02 and subsequently augered to a specified depth to install a piezometer. A summary of the respective boreholes/DCPT advanced at each foundation element and approach embankment is presented below.

Foundation Unit	Borehole/DCPT
South Approach Embankment	B5-01
South Abutment	B5-02 and B5-DC02
North Abutment	B5-03
North Approach Embankment	B5-04

The results of the borehole investigation and dynamic cone penetration tests are presented on the Record of Borehole/DCPT sheets in Appendix A. The boreholes and DCPT were advanced at the locations shown in plan on Drawing 2.

The field borehole investigation was carried out using a truck-mounted CME 55 drill rig supplied and operated by Landcore Drilling of Chelmsford, Ontario. The boreholes were advanced through the overburden using 120 mm outer diameter (O.D.) continuous flight hollow-stem augers and 'NW' casing. Soil samples were obtained at intervals of depth of about 0.75 m, 1.5 m and 3.0 m, using a 50 mm O.D. split-spoon sampler driven by an automatic hammer in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586 – Standard Test Method for Standard Penetration Test). The boreholes and DCPTs were advanced to depths of up to about 31.1 m and 32.0 m below existing ground surface, respectively. The DCPTs were terminated on refusal to further dynamic cone penetration.

The groundwater condition in the open boreholes as observed upon completion of drilling operations, and a standpipe piezometer was installed in a borehole immediately adjacent to Borehole B5-02 to permit monitoring of the water level at that location. The piezometer consists of 38 mm diameter PVC pipe, with a slotted screen surrounded with sand sealed at a select depth within the borehole. The borehole and annulus surrounding the piezometer pipe above the screen and sand pack were backfilled to the surface with bentonite pellets/grout. Piezometer installation details and water level readings are described on the Record of Borehole sheets in Appendix A. All open boreholes were backfilled with cement grout by tremie technique upon completion and the piezometer in the borehole immediately adjacent to Borehole B5-02 was also abandoned with cement grout by tremie technique on June 21, 2013 in accordance with Ontario Regulation 903, Wells (as amended).

The field work was observed by a member of our engineering and technical staff, who located the boreholes, arranged for the clearance of underground services, observed the drilling and sampling operations, logged the boreholes, and examined and cared for the soil samples. The soil samples were identified in the field, placed in appropriate containers, labelled and transported to our Mississauga geotechnical laboratory where samples underwent further visual examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate. Classification testing (water content, organic content, grain size distribution and Atterberg limits) was carried out on selected samples. The results of the laboratory testing are included in Appendix B.

The as-drilled borehole locations and ground surface elevations were surveyed by Tulloch. The locations given in the Record of Borehole / DCPT sheets and shown on Drawing 2 are positioned relative to MTM NAD 83 northing and easting coordinates and the ground surface elevations are referenced to Geodetic datum. The borehole locations, ground surface elevations and drilled depths are summarized below.

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Borehole	Location (MTM NAD 83)		Ground Surface Elevation (m)	Borehole / DCPT Depth (m)
	Northing	Easting		
B5-01	5042269.0	320122.8	319.6 m	9.8 m / 22.6 m
B5-02	5042286.8	320113.2	319.6 m	31.1 m / 31.5 m
B5-DC02	5042285.6	230114.2	319.6 m	19.8 m
B5-03	5042293.1	320102.8	319.6 m	31.1 m / 32.0 m
B5-04	5042311.6	320093.1	319.7 m	9.8 m / 23.2 m

## 4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

### 4.1 Regional Geology

As delineated in *The Physiography of Southern Ontario*<sup>1</sup>, this section of Highway 592 lies within the physiographic region known as the "Number 11 Strip", with portions of Highway 592 in contact with the "Georgian Bay Fringe" region. The Number 11 Strip is a narrow belt that extends from Gravenhurst to North Bay and is characterized by deposits of sand, silt and clay, together with more recent swamp deposits between rock knobs and ridges. The bedrock in the area is typically highly deformed gneiss of the Moon River Domain of the Central Gneiss Belt, a subdivision of the Grenville Structural Province (Geology of Ontario, 1991)<sup>2</sup>.

### 4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions as encountered in the boreholes advanced during this investigation, together with the results of the laboratory tests carried out on selected soil samples, are provided in Appendix A and Appendix B, respectively. The results of the in situ field tests (i.e. SPT 'N'-values) as presented on the Record of Borehole sheets and in Section 4.2 are uncorrected. The stratigraphic boundaries shown on the Record of Borehole sheets and on the profile on Drawing 2 are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Test (SPTs). These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Further, subsurface conditions will vary between and beyond the borehole locations. It should be noted that the interpreted stratigraphy shown on Drawing 2 is a simplification of the subsurface conditions.

In general, the subsurface conditions in the area of the proposed bridge structure consist of a surficial layer of asphalt underlain by a deposit of non-cohesive fill associated with the Highway 592 embankments, underlain in places by a deposit of organic silt. The fill and organic silt deposits are in turn underlain by a deposit of silt to sand which extends to refusal.

A detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

<sup>1</sup> Chapman, L.J. and D. F. Putnam, 1984. *The Physiography of Southern Ontario*, Ontario Geological Survey, Special Volume 2, Third Edition. Accompanied by Map P. 2715, Scale 1:600,000.

<sup>2</sup> Ontario Geological Society. 1991. *Geology of Ontario*, Special Volume 4, Part 2. Eds. P.C. Thurston, H.R. Williams, R.H. Sutcliffe and G.M. Stott. Ministry of Northern Development and Mines, Ontario.

#### 4.2.1 Asphalt

A 50 mm to 480 mm thick layer of asphalt was encountered at the ground surface in all boreholes.

#### 4.2.2 Sand and Gravel to Sand Fill

A deposit of dark brown to dark grey non-cohesive fill was encountered below the asphalt layer in all boreholes. The fill deposit generally comprised of an upper layer of sand and gravel, trace silt containing silt seams and a lower layer of sand, trace to some silt, trace gravel, trace organics and containing rootlets. The top of the fill deposit ranges from Elevations 319.6 m to 319.1 m and the thickness of the fill deposit ranges from 1.4 m to 2.1 m.

The SPT 'N'-values measured within the non-cohesive fill deposit generally range from 2 blows to 8 blows per 0.3 m of penetration, indicating a very loose to loose relative density, with SPT 'N'-values up to 27 blows per 0.3 m of penetration recorded within the upper portion of the fill immediately underlying the asphalt layer, indicating a compact relative density.

The natural water content measured on four samples of the fill ranges from about 23 per cent to 45 per cent. An organic content measured on one sample of the sand portion of the fill deposit is about 4 per cent.

#### 4.2.3 Organic Silt

A deposit of dark brown to dark grey to black organic silt was encountered underlying the fill deposit in Boreholes B5-01 to B5-03. The deposit generally contains trace to some sand and wood fragments. The top of the organic silt deposit is at about Elevation 317.4 m and the thickness of the deposit ranges between 0.8 m and 1.5 m.

The SPT 'N'-values measured within this deposit ranges between 2 blows and 4 blows per 0.3 m of penetration, indicating a very loose relative density.

The natural water content measured on four samples of the organic silt deposit ranges from about 46 per cent to 196 per cent. An organic content measured on one sample of this deposit is about 26 per cent.

#### 4.2.4 Silt to Sand

A deposit comprised of non-cohesive silt to sand was encountered underlying the organic silt deposit in Boreholes B5-01 to B5-03 and below the fill deposit in Borehole B5-04. The overall silt and sand deposit is comprised of an upper portion of sandy silt to silt and sand, an interlayer of silt trace to some sand, a middle portion of silt to silty sand and a lower portion of sand some silt. The deposit generally contains trace to some clay, trace gravel. The 0.7 m and 2.7 m thick upper portion of sandy silt to silt and sand in Boreholes B5-01, B5-03 and B5-04 contains trace organics, rootlets and fibrous peat layers. The top of the overall silt to sand deposit ranges from Elevations 318.3 m to 315.1 m and the thickness of the deposit ranges from about 6.8 m to 27.4 m. The DCPTs advanced from the bottom of sampled Boreholes B5-01 to B5-04 and in DCPT B5-DC02 are inferred to be terminated within this deposit at depths between 19.8 m to 32.0 m below ground surface (Elevations 299.8 m to 287.6 m).



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The SPT 'N'-values measured within the overall silt to sand deposit range from 1 blows to 20 blows per 0.3 m of penetration, indicating a very loose to compact relative density. The sandy silt to silt and sand upper portion of the deposit may be described as very loose to compact, the silt interlayer may be described as very loose, the silt to silty sand middle portion of the deposit may be described as very loose to compact and the sand lower portion of the deposit may be described as loose to compact in relative density.

The natural water content measured on twenty four samples of the overall silt to sand deposit ranges from 22 per cent to 56 per cent. The organic content measured on four samples of the sandy silt to silt and sand upper portion of the deposit ranges between about 1 per cent and 4 per cent.

The results of grain size distribution tests completed on sixteen samples of the overall silt to sand deposit are shown on Figure B1 in Appendix B, broken down into the sandy silt to silt and sand upper portion of the deposit on Figure B1A, the silt interlayer on Figure B1B, the silt to silty sand middle portion of the deposit on Figure B1C and the sand lower portion of the deposit on Figure B1D. Atterberg limits tests carried out on one sample each of the silty sand and the silt portions of this deposit indicates the fine material to be non-plastic.

### 4.3 Groundwater Conditions

In general, the soil samples taken in the boreholes were moist to wet. The groundwater levels measured in the open boreholes upon completion of drilling range from 1.4 m to 1.8 m below ground surface, corresponding to about Elevations 318.3 m to 317.8 m.

A standpipe piezometer was installed in a borehole immediately adjacent to Borehole B5-02 to allow monitoring of the groundwater level at the site. The piezometer was decommissioned on June 21, 2013. Details of the piezometer installation and the groundwater levels are shown on the Record of Borehole No. B5-02 in Appendix A and the groundwater level measured in the piezometer is summarized below.

Borehole	Ground Surface Elevation	Depth to Water Level	Groundwater Elevation	Date of Measurement
B5-02	319.6 m	2.0 m	317.6 m	May 29, 2013
		1.8 m	317.8 m	May 31, 2013
		1.9 m	317.7 m	June 21, 2013

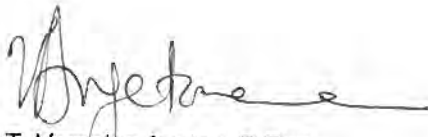
It should be noted that groundwater levels in the area are subject to seasonal fluctuations and precipitation events, and should be expected to be higher during wet periods of the year.

### 5.0 CLOSURE

Mr. Indulis Dumpis, a senior technician with Golder, directed the drilling program. This report was prepared by Ms. Madison C. Kennedy and Ms. T. Veronica Ayetan, P.Eng., and was reviewed by Christopher Ng, P.Eng., a geotechnical engineer and Associate with Golder. Mr. Jorge M. A. Costa, P.Eng., Golder's Designated MTO Contact for this project and Principal with Golder, conducted an independent quality control review of the report.

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## **PART B**

FOUNDATION DESIGN REPORT

RAGGED CREEK BRIDGE

HIGHWAY 592 – REPLACEMENT OF SIX STRUCTURES

MINISTRY OF TRANSPORTATION, ONTARIO

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## 6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides engineering design recommendations for the proposed Ragged Creek Bridge on Highway 592 (Site No.44-178). The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the design of the structure foundation and approach embankments. Where comments are made on construction, they are provided in order to highlight those aspects which could affect the design of the project. Those requiring information on the aspects of construction should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

### 6.1 General

Golder Associates Ltd. (Golder) has been retained by Morrison Hershfield Limited (MH) on behalf of the Ministry of Transportation, Ontario (MTO) to provide recommendations on foundation aspects for the detail design of the proposed replacement of Ragged Creek Bridge on Highway 592 in Huntsville, Ontario.

Based on the General Agreement (GA) Drawing provided by MH on August 23, 2013, the proposed Ragged Creek Bridge will consist of a single-span, pre-cast girder structure with a span length of 12.6 m. The grade of the proposed bridge deck will be at about Elevation 320.1 m, which corresponds to a raise of the existing approach embankments of up to about 0.3 m.

### 6.2 Foundation Options

Given that very loose organic silt and silt to sand deposits are present in the areas of the abutments down to below the depth of frost penetration, the relatively shallow depth to the groundwater table and the proximity to the adjacent river, a shallow foundation system is not recommended for the support of the abutments.

Given that bedrock was not encountered to the depths drilled and that stage construction will be required in a narrow right-of-way, deep foundations comprised of soil-bonded micropiles is considered the preferred alternative for the support of the structure. Driven steel H-piles or drilled steel casings may be considered for design, however, the geotechnical axial capacity will be relatively low as the H-piles and steel casings will develop capacities through friction only within the generally very loose to compact granular deposits.

The following sections provide recommendations for alternative foundation systems, comprised of spread footings constructed on the native overburden, driven H-pile and drilled steel casing foundations, as well as soil-bonded micropiles.

The advantages, disadvantages, relative costs and risks/consequences for the various foundation options are summarized in Table 1.

### 6.3 Spread Footings

Shallow foundations comprised of spread/strip footings founded on native overburden are not recommended for support of the proposed bridge abutments given that the soils down to well below the depth of frost penetration

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are very loose organic silt/silt and sand. In the event that shallow foundations are considered further for the support of the proposed structure, recommendations for design are provided below.

### 6.3.1 Geotechnical Axial Resistance and Reaction

For 11.5 m long by 2 m wide footings founded on the native overburden (a deposit of very loose organic silt underlain by a deposit of very loose silt to sand) at Elevation 316.7 m at the abutments, the factored geotechnical axial resistance at Ultimate Limits States (ULS) and geotechnical reaction at Serviceability Limits States (SLS) for 25 mm of settlement are provided below.

Foundation Location	Factored Geotechnical Axial Resistance at ULS	Geotechnical Reaction at SLS for 25 mm of Settlement
South and North Abutments	300 kPa	50 kPa

The geotechnical resistances provided above are given for loads applied perpendicular to the surface of the footing. Where the load are not applied perpendicular to the surface of the footings, inclination of the load should be taken into account in accordance with Sections 6.7.2 and 6.7.4 of the *Canadian Highway Bridge Design Code (CHBDC, 2006)* and its *Commentary*.

The construction of the cast-in-place footings must be carried out within a dry excavation. Given that the groundwater level and the river water level at the abutments is above the underside of the proposed footings, cofferdam construction and unwatering will be required to allow for construction of the footings in dry conditions.

### 6.3.2 Resistance to Lateral Loads

The resistance to lateral forces/sliding resistance between the concrete footings and the natural subgrade materials should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction,  $\tan \delta'$ , for the soil-structure interface between the cast-in-place concrete footing and the native overburden is given below.

Interface Material(s)	Coefficient of Friction ( $\tan \delta'$ )
Concrete footing on very loose organic silt	0.30

The value presented above represents an unfactored value.

### 6.3.3 Frost Protection

The following should be noted for the design of footings founded on the native overburden:

- The required thickness of conventional soil cover for frost protection of the footings is 1.8 m, as per OPSD 3090.010 (*Frost Penetration Depths for Southern Ontario*) as measured perpendicular to/from the



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face of the abutment slope to the edge of the underside of the footing (it is not simply a vertical dimension when the footing is adjacent to a slope).

- If adequate soil cover cannot be provided for the footing, rigid styrofoam insulation should be installed to compensate for the lack of soil cover and provide protection from frost penetration. In this regard, the MTO has adopted an equivalent thickness of 25 mm of styrofoam equal to 300 mm of soil cover.

## **6.4 Driven Steel H-Pile Foundations**

Given presence of the very loose to loose organic silt and the thick underlying granular deposit, friction piles consisting of steel H-piles driven into the compact portion of the silt to sand deposit could be considered for the support of the proposed structure. However, given the loose to compact relative density of the silt to sand deposit, the geotechnical axial capacity will be relative low. In addition, due to the proposed construction sequencing/staging and the narrow right-of-way, there may not be adequate construction platform width to accommodate piling equipment necessary to drive long H-piles to the required depth to achieve the desired axial capacities for design. Furthermore, piles cannot be battered for lateral resistance due to the proximity of the temporary shoring (cofferdam).

### **6.4.1 Geotechnical Axial Resistance and Reaction**

The following summarizes the proposed elevation of the underside of the pile cap and tremie plug, the pile tip elevation, pile length, as well as the factored geotechnical axial resistance at ULS and the geotechnical reaction at SLS for 25 mm of settlement for driven steel HP 310 x 110 piles at the proposed abutments.

<b>Foundation Location</b>	<b>Elevation of Underside of Pile Cap <sup>1</sup></b>	<b>Elevation of Underside of Tremie Plug <sup>1</sup></b>	<b>Pile Tip Elevation</b>	<b>Length of Pile from Underside of Pile Cap</b>	<b>Factored Geotechnical Axial Resistance at ULS</b>	<b>Geotechnical Reaction at SLS for 25 mm of Settlement <sup>2</sup></b>
South and North Abutment	316.7 m	315.5 m	293.5 m	23.2 m	450 kN	N/A

Notes:

1. As per the GA Drawing provided by MH on August 23, 2013.
2. The geotechnical reaction at SLS for 25 mm of settlement will be greater than or equal to the factored geotechnical axial resistance at ULS and therefore, the SLS condition does not apply.

All piles should be fitted with driving shoes and flange plates (reinforced tips) in accordance with OPSS 3000.100 (*Steel H-Pile Driving Shoe*) to minimize damage to the pile during driving and penetration through the granular deposits containing cobbles and boulders.

### **6.4.2 Set Criteria**

All pile installation/driving should be in accordance with OPSS 903 (*Deep Foundations*).

The pile termination or set criteria will be dependent on the pile driving hammer type and the selected pile type. The set criteria can be established through a variety of methods, including empirical correlations, such as the

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use of the Hiley Formula, and wave equation analyses, at the time of construction once the hammer and pile types are known. The choice of set criteria is dependent on the experience of the engineer and traditional use where a substantial database has been developed over the years. The criteria need to be set to also avoid overdriving and possibly damaging the pile.

For friction piles, the pile capacity must be verified in the field by the use of the Hiley Formula (MTO's Standard Drawing SS103-11 Pile Driving Control (2008)) during the final stages of driving for the ultimate capacity at the elevations provided in Section 6.4.1.

The following pile driving note should be added to the Contract Drawings (i.e. Note 2 in Clause 3.3.3 of the Structural Manual (MTO, 2008)):

- Piles to be driven in accordance with Standard SS 103-11 using an ultimate geotechnical resistance of 1,125 kN per pile, but must be driven below El. 293.5 m.

### 6.4.3 Frost Protection

The pile cap at the abutment locations should be provided with a minimum of 1.8 m of conventional soil cover or equivalent insulation for frost protection.

## 6.5 Drilled Steel Casing

Consideration could also be given to the use of drilled steel casing for support of the abutments. Due to the proposed construction sequencing/staging and narrow right-of-way, there may not be adequate construction platform width to accommodate drilling equipment necessary to advance long steel casing to achieve the desired geotechnical axial capacities for design.

### 6.5.1 Geotechnical Axial Resistance and Reaction

The following summarizes the proposed elevation of the underside of pile cap and tremie plug, the casing tip elevation, casing length as well as the factored geotechnical axial resistance at ULS and the geotechnical reaction at SLS for 25 mm of settlement for a 610 mm diameter drilled steel casing at the proposed abutments.

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Foundation Location	Elevation of Underside of Pile Cap <sup>1</sup>	Elevation of Underside of Tremie Plug <sup>1</sup>	Casing Tip Elevation	Length of Pile from Underside of Pile Cap	Factored Geotechnical Axial Resistance at ULS	Geotechnical Reaction at SLS for 25 mm of Settlement <sup>2</sup>
South and North Abutment	316.7 m	315.5 m	293.5 m	23.2 m	575 kN	N/A

**Notes:**

1. As per the GA Drawing provided by MH on August 23, 2013.
2. The geotechnical reaction at SLS for 25 mm of settlement will be greater than or equal to the factored geotechnical axial resistance at ULS and therefore, the SLS condition does not apply.

It should be noted that a smaller casing diameter (i.e. less than 610 mm) does not offer any significant advantages, in terms of capacity, over driven steel H-piles.

## 6.5.2 Frost Protection

The pile cap at the abutment locations should be provided with a minimum of 1.8 m of conventional soil cover or equivalent insulation for frost protection.

## 6.6 Micropiles

Due to the nature of the subsurface conditions (i.e. thick very loose to compact granular deposits) and the site constraints for equipment access/setup which detrimentally affect construction of other deep foundation types, micropiles are considered to be the preferred foundation alternative at this site. The advantages that micropiles have over driven steel H-piles and drilled steel casing include:

- Micropiles can readily penetrate through cobbles and boulders in the overburden; and,
- Micropile drilling equipment is relatively small (for use in confined spaces and/or low headroom situations) as compared to pile-driving and/or casing-drilling equipment.

There are two types of micropiles: the conventional micropile system and the hollow bar micropile system. The conventional micropile system advances a borehole into the overburden using a steel casing, and upon completion of drilling, a solid steel reinforcing bar is lowered to the bottom of the borehole and grouted in place for the length required to achieve the design axial capacity. The hollow bar micropile system installs a hollow steel bar into the overburden as the borehole is advanced, and of itself serves as the drill-string during drilling, and is grouted in place as the drilling advances.

There are advantages and disadvantages to each type of micropile and an assessment of each should be carried out at the time of the detail micropile design.

### 6.6.1 Geotechnical Axial Resistance and Reaction

For preliminary analysis and design, the following summarizes the proposed elevation of the underside of the pile cap and tremie plug, the micropile tip elevation as well as the diameter and length of the micropiles.

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Foundation Location	Elevation of Underside of Pile Cap <sup>1</sup>	Elevation of Underside of Tremie Plug <sup>1</sup>	Micropile Tip Elevation	Diameter of Micropile	Length of Micropile from Underside of Pile Cap
South and North Abutment	316.7 m	315.5 m	301.7 m	273 mm	15 m

Note:

1. As per the GA Drawing provided by MH on August 23, 2013.

The following summarizes the preliminary factored geotechnical axial resistance at ULS and the geotechnical reaction at SLS for 25 mm of settlement for a 273 mm diameter micropile at the proposed abutments.

Foundation Location	Factored Geotechnical Axial Resistance at ULS	Geotechnical Reaction at SLS for 25 mm of Settlement <sup>1</sup>
South and North Abutment	550 kPa	N/A

Note:

1. The geotechnical reaction at SLS for 25 mm of settlement will be greater than or equal to the factored geotechnical axial resistance at ULS and therefore, the SLS condition does not apply.

It should be noted that the geotechnical axial capacities will vary depending on the diameter and the length of the micropile selected during detailed design.

### 6.6.2 Frost Protection

The pile caps at the abutment locations should be provided with a minimum of 1.8 m of conventional soil cover or equivalent insulation for frost protection.

## 6.7 Resistance to Lateral Loads

The design of piles subjected to lateral loads should take into account such factors as the batter of the pile (if any), the relative rigidity of the pile to the surrounding soil, the fixity condition at the head of the pile (pile cap level), the structural capacity of the pile to withstand bending moments, the soil resistance that can be mobilized, the tolerable lateral deflections at the head of the pile and pile group effects. For a longer, more flexible pile, the maximum yield moment of the pile may be reached prior to mobilization of the lateral geotechnical resistance. For design purposes, both the structural and geotechnical resistances should be evaluated to establish the governing case.

Lateral loading could be resisted fully or partially by the use of battered piles/casings.

The resistance to lateral loading in front of a single pile may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction,  $k_h$  (kPa/m), is based on the following equations (CFEM, 1992 as referenced in the CHBDC Commentary, 2006):



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for non-cohesive soils:

$$k_h = \frac{n_h z}{B}$$

where:

$n_h$	=	constant of subgrade reaction (kPa/m)
$z$	=	depth (m)
$B$	=	pile diameter or width (m)

and for cohesive soils:

$$k_h = \frac{67s_u}{B}$$

where:

$s_u$	=	undrained shear strength of the soil (kPa)
$B$	=	pile diameter or width (m)

The values of  $n_h$  (Terzaghi, 1955 and Reese, 1975) and  $s_u$  to be incorporated into the calculations of the coefficient of horizontal subgrade reaction ( $k_h$ ) within the native subsoils to be utilized for the structural analysis of the piles and casings at this site are summarized below.

Foundation Element (Relevant Borehole)	Soil Unit	Elevation	$n_h$	$s_u$
South Abutment (B5-02) and North Abutment (B5-03)	Very Loose to Loose Silt to Sand	315.5 m to 293.5 m	3,000 kPa/m	-
	Compact Sand	293.5 m to 287.6 m	5,000 kPa/m	-

For a single HP 310x110 vertical pile or a single 610 mm diameter drilled steel casing, the estimated factored lateral resistances at ULS as well as the estimated lateral reactions at SLS (for 10 mm of horizontal deflection at the pile caps) are presented below. These values are based on analysis carried out using the commercially available program LPILE Plus (Version 5.0), developed by Ensoft Inc.

Foundation Location	Pile Type	Axial Load Applied at the Top of Pile/Casing	Factored Geotechnical Lateral Resistance at ULS <sup>1</sup>	Geotechnical Lateral Reaction at SLS for 10 mm of Deflection <sup>1</sup>
South and North Abutment	HP 310 x 110	450 kN	110 kN	20 kN
	610 mm diameter drilled steel casing	575 kN	125 kN	20 kN

Note:

1. Analyses assume a fixed-head condition.

Based on the above, it is considered that both structural and geotechnical resistances of the piles should be evaluated to establish the governing case at ULS. At SLS, the horizontal resistance of the piles will be controlled by deflections and the horizontal resistance of the pile should be calculated based on the coefficient of horizontal subgrade reaction ( $k_h$ ) of the soil as discussed above. The SLS resistance should be taken as that corresponding to a horizontal deflection of 10 mm at the underside of the pile cap for units supporting the abutments (*CHBDC Commentary C6.8.7.1*).



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The upper zone of the soil (down to a depth below the H-pile concrete tremie plug equal to about  $1.5 \cdot B$  (after Broms, 1964, where  $B$  is the pile diameter) should be neglected in the calculation of lateral resistance of the pile to account for disturbance effects during installation.

Group action for lateral loading should also be considered when the spacing in the direction of loading is less than eight (8) pile diameters between rows of driven steel H-pile or drilled steel casing. Group action can be evaluated by reducing the coefficient of lateral subgrade reaction in the direction of loading by a reduction factor,  $R$  (U.S. Navy, 1986), as follows:

Pile Spacing in Direction of Loading ( $d$ = pile diameter)	Subgrade Reaction Reduction Factor, $R$
8d	1.00
6d	0.70
4d	0.40
3d	0.25

The subgrade reaction reduction factor should be interpolated for H-pile/casing spacing in between those listed above.

It should be noted that the recommendations for lateral load-deflection behaviour for a single micropile and group effects for micropile groups is to be provided in the detailed micropile design.

## **6.8 Seismic Considerations**

### **6.8.1 Site Coefficient**

For seismic design purposes, the Site Coefficient,  $S$ , for this site, based on experience and considering the guidelines in Section 4.4.6 of the *CHBDC* may be taken as 1.2, consistent with Soil Profile Type II.

### **6.8.2 Seismic Analysis Coefficient**

According to the National Building Code of Canada (1995) seismic hazard values (as referenced in the *CHBDC* and its *Commentary*), the site specific peak horizontal ground acceleration for the Huntsville area is 0.065g (for a probability of exceedance of 10 per cent in 50 years). For the thicknesses and type of overburden soils at the site, an amplification factor of 1.2 of the ground motion is recommended for design. As such, the ground surface acceleration is about 0.078 and this site is classified as Seismic Performance Zone 1.

Given that the proposed structure is a single-span bridge and in accordance with Sections 4.4.5.2 of the *CHBDC*, seismic analysis is not required for this structure.

## **6.9 Lateral Earth Pressures**

The lateral earth pressures acting on the abutment stems and any associated wing walls/retaining walls will depend on the type and method of placement of the backfill material, the nature of the soils behind the backfill,

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the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls.

The following recommendations are made concerning the design of abutment walls. It should be noted that these design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Select, free draining granular fill meeting the specifications of OPSS.PROV 1010 Granular 'A' or Granular 'B' Type II, but with less than 5 per cent passing the No. 200 sieve, should be used as backfill behind the walls. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS 501 (Compacting) and Special Provision 105S21 (Water Requirements). Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3121.150 (Walls, Retaining, Backfill, Minimum Granular Requirement).
- For structures that are not comprised of integral or semi-integral abutments, rock fill may be used as backfill behind the walls and the material should meet the specification as outlined in the Northeastern Region Directive (2002) for backfill of structures adjacent to rock embankments. Other aspects of rock backfill requirements should be in accordance with OPSD 3101.200 (Walls, Abutment, Backfill, Rock). The following parameters (unfactored) may be used for rock backfill:

Fill Type	Soil Unit Weight	Coefficients of Static Lateral Earth Pressure	
		At-Rest, $K_o$	Active, $K_a$
Rock Fill	19 kN/m <sup>3</sup>	0.36	0.22

- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with CHBDC Section 6.9.3 and Figure 6.6. Other surcharge loadings should be accounted for in the design as required.
- For restrained walls, granular fill should be placed in a zone with the width equal to at least 1.8 m behind the back of the wall (in accordance with Figure C6.20(a) of the *Commentary* to the CHBDC). For unrestrained walls, fill should be placed within the wedge shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (in accordance with Figure C6.20(b) of the *Commentary* to the CHBDC). The pressures are based on the proposed embankment fill material and the following parameters (unfactored) may be used:

Fill Type	Soil Unit Weight	Coefficients of Static Lateral Earth Pressure	
		At-Rest, $K_o$	Active, $K_a$
Granular 'A'	22 kN/m <sup>3</sup>	0.43	0.27
Granular 'B' Type II	21 kN/m <sup>3</sup>	0.43	0.27

If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the foundation design of the structure. If the wall support and superstructure does not allow lateral yielding, at-rest earth pressures should be assumed for foundation design. The movement required to allow active

pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.9.1 and Table C6.6 of the *Commentary* to the CHBDC.

## **6.10 Approach Embankment Design**

Based on the GA Drawing provided by MH, the proposed grade for the Ragged Creek Bridge structure will be at about Elevation 320.1 m, requiring placement of up to about 0.3 m of fill to raise the existing south and north approach embankment grades.

Based on the investigated locations at this site, the south and north approach embankments are founded on a deposit of very loose to loose organic silt, underlain by a deposit of very loose to compact silt to sand.

It is understood that a partial excavation of the organic soils of up to 2 m deep and backfilling with rock fill along the existing side slopes will be carried out as part of the embankment widening at the approach embankments. However, excavation will not be carried out along the existing embankment and as such, the existing fill material and the underlying organic silt will remain in place. It is also understood that rock fill will be utilized for the embankment widening at this site.

The results of stability and settlement analysis for the approach embankments are presented in the following sections.

### **6.10.1 Stability**

#### **6.10.1.1 Methodology**

Limit equilibrium slope stability analyses were performed using the commercially available program Slide (Version 6.0), developed by Rocscience Inc., employing the Morgenstern-Price method of analysis. For all analyses, the Factors of Safety (FoS) of numerous potential failure surfaces were computed in order to establish the minimum FoS. The FoS is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. A target minimum FoS of 1.3 is normally used in the design of embankment slopes under static conditions.

#### **6.10.1.2 Parameter Selection**

For the non-cohesive soils, the effective stress parameters employed in the analysis were estimated from empirical correlations based on the results of the in situ Standard Penetration Tests (SPT). The correlations proposed by Peck et al (1974) and U.S. Navy (1986) were employed and the results were adjusted by engineering judgment based on precedent experience in similar soils.

For the purpose of the stability analysis, the groundwater level was assumed to be at Elevation 317.7 m, which is based on groundwater level measurements in the open boreholes upon completion of drilling.

The following presents the simplified stratigraphy and the associated strengths and unit weights employed for the existing embankment fill and the native overburden deposits encountered at the approach embankment areas.

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Embankment	Soil Type	Unit Weight, $\gamma$	Cohesion, $c'$	Effective Friction Angle, $\phi'$
South and North Approach Embankments	New Granular Fill	20 kN/m <sup>3</sup>	0 kPa	34°
	New Rock Fill	19 kN/m <sup>3</sup>	0 kPa	40°
	Existing Very Loose to Compact Sand to Sand and Gravel Fill	20 kN/m <sup>3</sup>	0 kPa	30°
	Very Loose to Loose Organic Silt	18 kN/m <sup>3</sup>	0 kPa	27°
	Very Loose to Loose Silt to Sand	19 kN/m <sup>3</sup>	0 kPa	28°
	Compact Sand	19 kN/m <sup>3</sup>	0 kPa	30°

### 6.10.1.3 Results of Analysis

The results of the stability analyses for the approach embankments are summarized below. The minimum factor of safety is based on a deep-seated, global trial failure surface that would impact the operation of the highway.

Embankment	Embankment Height at Critical Section <sup>1</sup>	Side Slope Profile	Minimum Factor of Safety
South and North Approach Embankments	1.7 m	1.25H:1V	≥ 1.3

Note:

1. Embankment height includes an approximately 0.3 m high grade raise at both approach embankments.

## 6.10.2 Settlement

### 6.10.2.1 Methodology

To estimate the magnitude of expected settlement of the embankments, analyses were carried out at the critical section of the south and north approach embankments, corresponding to the highest grade raise and/or largest widening. Settlement analyses were carried out using both the commercially available program *Settle*<sup>3D</sup> (Version 2.0), developed by Rocscience Inc.



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The sources of settlement are considered to include:

- Immediate settlement of the granular soils (short-term);
- Primary and secondary time-dependent consolidation of organic soils (long-term); and,
- Self-weight compression of the new embankment fill (long-term).

The analyses were carried out at the critical sections of the approach embankments where the thickness of compressible foundation soils is up to about 30.3 m and as such, the estimated settlements represent the maximum value along the approach embankments.

### 6.10.2.2 Parameter Selection

The following presents the simplified stratigraphy and the associated unit weights and strengths employed for the estimation of settlement of the foundation soils at the approach embankment areas. The immediate compression of the non-cohesive overburden soils were modelled by estimating an elastic modulus of deformation based on the SPT 'N'-values and using correlations proposed by Bowles (1984) and Kulhawy and Mayne (1990). These estimated values were compared with the typical range of expected values for similar soil types, as outlined in CHBDC and adjusted, as appropriate. The compression of the organic soils was modelled by estimating deformation parameters based on correlations proposed by Mesri and Ajlouni (2007) and the National Research Council of Canada (1969).

The following summarize the simplified stratigraphy and the associated unit weights and deformation parameters employed for the existing fill materials and the native soil deposits encountered at the approach embankment areas.

Embankment	Soil Type	Thickness <sup>1</sup>	Unit Weight, $\gamma$	Deformation Parameter(s)
South and North Approach Embankment	Existing Very Loose to Compact Sand to Sand and Gravel Fill	1.4 m to 2.1 m	20 kN/m <sup>3</sup>	$E' = 5 \text{ MPa}$
	Very Loose to Loose Organic Silt	0.8 m to 1.5 m	18 kN/m <sup>3</sup>	$e_o = 2.5$ $C_c = 1.0$ $C_{\alpha(e)} = 0.06$ $c_v = 1.0 \times 10^{-3} \text{ cm}^2/\text{s}$
	Very Loose to Loose Silt to Sand	22.5 m to 23.2 m	19 kN/m <sup>3</sup>	$E' = 3 \text{ MPa}$
	Compact Sand	5.3 m to 5.8 m	19 kN/m <sup>3</sup>	$E' = 10 \text{ MPa}$

Note:

1. Thickness based on applicable borehole(s) (i.e. borehole(s) advanced in the vicinity of the respective approach embankment) terminated within the respective deposit

where:  $E'$  is the elastic modulus (MPa)  
 $e_o$  is the initial void ratio  
 $C_c$  is the primary compression index  
 $C_{\alpha}$  is the secondary compression index  
 $c_v$  is the coefficient of consolidation ( $\text{cm}^2/\text{s}$ )



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It should be noted that the parameters for organic deposits are based on estimates from empirical correlations established in published literature and as such, these parameters should be considered as general approximations.

For the purpose of settlement analyses, the groundwater level was assumed to be located on average at Elevation 317.7 m, based on several groundwater level measurements in the open boreholes upon completion of drilling.

#### **6.10.2.3 Settlement of Foundation Soils**

The results of the analyses of the estimated settlement of the foundation soils at the approach embankments are presented below.

Embankment	Settlement During Construction		Post-Construction Settlement (10 Years After Construction)		Total Settlement
	Organic Soils <sup>1</sup>	Inorganic Soils <sup>2</sup>	Organic Soils <sup>1</sup>	Inorganic Soils <sup>2</sup>	
South Approach Embankment Centreline	15 mm	45 mm	50 mm	~0 mm	110 mm
South Approach Embankment Side Slope <sup>3</sup>	~0 mm	60 mm	~0 mm	~0 mm	60 mm
North Approach Embankment Centreline	15 mm	45 mm	30 mm	~0 mm	90 mm
North Approach Embankment Side Slope <sup>3</sup>	~0 mm	60 mm	~0 mm	~0 mm	60 mm

Notes:

1. Organic soils include the organic silt deposit.
2. Inorganic soils include the silt to sandy silt to silt and sand and sand and gravel fill deposits.
3. Analyses assume an up to about 2 m deep sub-excavation and replacement with rock fill.

As a result of the differential settlement between the embankment centreline and the side slopes, future maintenance of the highway may be required.

#### **6.10.2.4 Settlement of Rock Fill Embankment**

It is understood that rock fill is to be used for the construction of the approach embankments widening as a result of the narrow right-of-way and as such, there will be settlement due to compression of the rock fill itself under self-weight along the east and west side of the approach embankments. The magnitude of settlement of the rock fill depends on the type of rock/strength of particles, size and shape of particles, gradation of rock fill, total height/thickness of fill and the method of construction and sequence of placement. Rock fill should be placed, whenever possible, in a controlled manner (i.e. not end-dumped) in accordance with SP 206S03 (Rock Excavation, Grading). Where rock fill cannot be placed in a controlled manner (i.e. below the groundwater table), the post-construction settlement of the rock fill is expected to be greater. Based on MTO's Guideline for

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Rock Fill Settlement and Rock Fill Quantity Estimates (2010), the estimated settlements of rock fill for the approach embankments are presented below.

Embankment	Thickness of Rock Fill Along East and West Slope	Estimated Settlement of Rock Fill		
		Short-Term	Long-Term	Total
South and North Approach Embankment	Up to about 2 m (above groundwater table)	10 mm	5 mm	15 mm
	Up to about 2 m (below groundwater table)	20 mm	5 mm	25 mm

The majority of the settlement of the rock fill is expected to occur during construction; however, some post-construction time-dependent settlement will occur, as noted above.

### 6.10.3 Liquefaction Potential below Embankments

The liquefaction potential of the soils below the approach embankments under seismic loading has been considered using the empirical method outlined in Section C.4.6.2 of the *Commentary* to the CHBDC, which correlates the cyclic resistance ratio (CRR) of the soils with their normalized penetration resistance and fines content. Based on this assessment and with a site specific peak horizontal acceleration of 0.078g, the subsoils are not considered liquefiable for an earthquake of magnitude 7.0. Localized failures at the embankment toe, resulting in steepening of the embankment side slopes, could occur, however, the probability of this occurrence is considered to be low.

### 6.10.4 Embankment Platform Widening

In accordance with the requirements of MTO Northern Region Engineering Directive NRE 98-200, Northern Region Embankment Design Guidelines (1998), the minimum required embankment widening at this site to account for the estimated post-construction settlement and for future pavement overlays is 1 m per embankment side.

### 6.10.5 Embankment Fill Placement

Placement of granular fill for the grade raise and rock fill for the widening of the approach embankment should be carried out in accordance with SP 206S03 (Earth Excavation, Grading; Rock Excavation, Grading) and compaction of granular fill should be in accordance with OPSS 501 (Compacting) as modified by SP 105S21, with inspection and field testing by qualified personnel during construction to confirm that appropriate materials are used and that adequate levels of compaction are achieved. Where embankment widening and/or grade raise is carried out using earth fill and in areas of exposed earth fill, it is recommended that topsoil and seeding or pegged sod be placed as soon as practical after completion of the grade raise and embankment widening to reduce erosion of the embankment side slopes due to surface water runoff. The erosion protection should be carried out in accordance with OPSS 804 (Seed and Cover).

## **6.11 Design and Construction Considerations**

### **6.11.1 Overburden Excavation**

In order to construct the pile cap for the abutments at the currently proposed base at Elevation 316.7 m and the underside of the tremie plug at Elevation 315.5 m, excavations up to about 4.1 m deep below the existing ground surface will be required and will be made through the existing fill material and overburden soils. The existing fill materials and native overburden soils are considered Type 3 soils according to the Occupational Health and Safety Act and Regulation for Construction Projects (OHSA) and as such, temporary open-cut excavations above the groundwater level should be carried out with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V). In addition, temporary shoring (cofferdam) will be required for the excavation to the underside of the tremie plug as it will be below the groundwater level and the water level of Ragged Creek.

All excavations must be carried out in accordance with Ontario Regulation 213 Ontario Occupational Health and Safety Act for Construction Projects (as amended).

### **6.11.2 Temporary Roadway Protection**

Given that the existing roadway (i.e. Highway 592) is to remain partially open to traffic during construction, temporary roadway protection systems will be required to protect the excavations. The temporary protection system should be constructed in accordance with OPSS 539 (Temporary Protection Systems) as modified by SP 539S02 and the lateral movement should meet Performance Level 2.

### **6.11.3 Control of Groundwater and Surface Water**

Excavations to construct pile caps will extend below the groundwater level and therefore will require temporary shoring with unwatering to allow for construction of the pile caps in dry conditions. Temporary shoring and unwatering could be in the form of sheetpile cut-off wall or cofferdam advanced to an appropriate depth to control groundwater inflow. In addition, a tremie concrete "plug" will also be required at the base of the cofferdam to mitigate potential for base instability due to groundwater pressures.

## **7.0 CLOSURE**

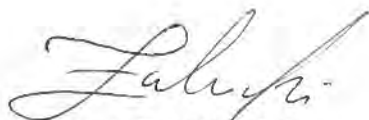
This report was prepared by Mr. Tomasz Zalucki, P.Eng., and was reviewed by Christopher Ng, P.Eng., a geotechnical engineer and Associate with Golder. Mr. Jorge M. A. Costa, P.Eng., Golder's Designated MTO Contact for this project and a Principal with Golder, conducted an independent quality control review of the report.

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## Report Signature Page



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|------------|---|
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- |   |
|---|
| LPILE Plus (Version 5.0) by Ensoft Inc.               |
| Settle <sup>3D</sup> (Version 2.0) by Rocscience Inc. |
| Slide (Version 6.0) by Rocscience Inc.                |
- Contract Design Estimating and Documentation (CDED):



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**FOUNDATION REPORT - RAGGED CREEK BRIDGE - HIGHWAY  
592 GWP 5265-07-00; WP 5269-07-01**

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Special Provision 105S21    Amendment to OPSS 501 – Water Requirements and Quality Control for Compaction – Method B

Special Provision 206S03    Amendment to OPSS 206 – Earth Excavation, Grading; Rock Excavation, Grading.

Special Provision 539S02    Amendment to OPSS 539 – Protection System

**Ministry of Transportation Ontario:**

MTO Guideline for Rock Fill Settlement and Rock Fill Quantity Estimates. September 2010.

Northern Region Engineering Directive NRE 98-200. Northern Region Embankment Design Guidelines. October 1998.

Northeastern Region Engineering Directive. Backfill to Structures Adjacent to Rock Embankment Approaches. November 2002.

Standard Drawing SS103-11. Pile Driving Control. April 2008.

Structural Manual. Provincial Highway Management Division, Highway Standards Branch, Bridge Office. April 2008.

**Ontario Occupational Health and Safety Act:**

Ontario Regulation 213    Construction Projects (as amended)

**Ontario Provisional Standard Drawing:**

OPSD 3000.100              Foundation, Piles, Steel H-Pile Driving Shoe

OPSD 3090.010              Foundation, Frost Penetration Depths for Southern Ontario

OPSD 3101.200              Walls – Abutment, Backfill – Rock

OPSD 3121.150              Walls – Retaining, Backfill – Minimum Granular Requirement

**Ontario Provincial Standard Specification:**

OPSS 501                      Construction Specification for Compacting

OPSS 539                      Construction Specification for Temporary Protection Systems

OPSS 804                      Construction Specification for Seed and Cover

OPSS 903                      Construction Specification for Deep Foundations

OPSS.PROV 1010              Material Specification for Aggregates – Base, Subbase, Select Subgrade, and Backfill Material

**Ontario Water Resources Act:**

Ontario Regulation 903    Wells (as amended)

## TABLES

Table 1: Evaluation of Foundation Alternatives

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks / Consequences
Spread/strip footings (11.5 m long by 2 m wide)	NR <sup>1</sup>	<ul style="list-style-type: none"> <li>Relative ease of construction.</li> </ul>	<ul style="list-style-type: none"> <li>Allows only for semi-integral abutment design.</li> <li>Axial capacity on the loose organic silt or at greater depth on very loose to compact silt to sand deposit will be low.</li> <li>Excavation for footings will be below water table.</li> <li>Cofferdam (with concrete tremie plug) and unwatering will be required for construction of the footings within a dry excavation.</li> </ul>	<ul style="list-style-type: none"> <li>Lower relative cost than driven pile, drilled steel casing and micropile foundation options.</li> <li>Additional cost for cofferdam construction and unwatering for construction of the footings.</li> </ul>	<ul style="list-style-type: none"> <li>Large footings will be required to develop adequate axial capacity.</li> </ul>
Driven steel H-piles (HP 310X110)	2	<ul style="list-style-type: none"> <li>Negligible post-construction settlement.</li> <li>Higher axial capacity than spread/strip footings.</li> <li>Straight forward construction, except that site constraints may preclude use of pile driving equipment.</li> </ul>	<ul style="list-style-type: none"> <li>Integral abutment design may not be possible due to constraints in achieving free length of pile to allow for lateral movement due to the presence of the tremie plug.</li> <li>Given the thickness of the overburden, axial capacity will be developed through shaft resistance (i.e. friction piles) only.</li> <li>Piles cannot be battered for lateral resistance due to the proximity of the sheelpile cofferdam.</li> <li>Excavation for pile cap will be below water table.</li> </ul>	<ul style="list-style-type: none"> <li>Higher relative cost than spread/strip footing foundation option.</li> <li>Additional cost for cofferdam construction and unwatering for construction of the pile cap.</li> <li>Additional cost associated with acquiring additional right-of-way and/or the relocation of Hydro lines to accommodate larger (pile driving) equipment.</li> </ul>	<ul style="list-style-type: none"> <li>Potential for requirement to drive piles deeper to develop adequate axial capacity during construction.</li> <li>May require additional construction platform width and/or temporary closure of the roadway to accommodate larger (pile driving) equipment.</li> <li>Overhead hydro lines will need to be relocated and/or de-energized for/during the piling operation.</li> </ul>

FOUNDATION REPORT - RAGGED CREEK BRIDGE - HIGHWAY 592 GWP 5265-07-00; WP 5269-07-01

Table 1: Evaluation of Foundation Alternatives

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks / Consequences
Drilled steel casings (610 mm)	3	<ul style="list-style-type: none"><li>■ Reduced number of deep foundation elements compared to steel H-piles.</li><li>■ Relatively straightforward construction; except that site constraints may preclude the use of drilling equipment.</li><li>■ Negligible post-construction settlement.</li></ul>	<ul style="list-style-type: none"><li>■ Cofferdam (with concrete tremie plug) and unwatering will be required for construction of the pile caps within a dry excavation.</li><li>■ Requires larger (pile driving) equipment as compared to micropile drilling equipment.</li><li>■ Piling operation along the east side of the bridge will be in close proximity to overhead hydro lines.</li></ul>	<ul style="list-style-type: none"><li>■ Higher relative cost than spread/strip footing and driven pile foundation options.</li><li>■ Additional cost for specialized drilling equipment.</li><li>■ Additional cost associated with the need for drilling slurry and temporary liners.</li><li>■ Additional cost for cofferdam construction and unwatering for construction of the pile cap.</li><li>■ Additional cost associated with acquiring additional right-of-way and/or the relocation of Hydro lines to accommodate larger (pile</li></ul>	<ul style="list-style-type: none"><li>■ Potential for unbalanced head in liners during installation may result in base heave and possible loss of ground.</li><li>■ May require additional construction platform width and/or temporary closure of the roadway to accommodate larger (drilling) equipment.</li><li>■ Overhead hydro lines will need to be relocated and/or de-energized for/during the piling operation.</li></ul>



Table 1: Evaluation of Foundation Alternatives

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks / Consequences
Micropiles (273 mm diameter)	1	<ul style="list-style-type: none"> <li>Negligible post-construction settlement.</li> <li>Potential for achieving high axial capacity in the overburden using pressure grouting techniques.</li> <li>Requires smaller drilling equipment as compared to steel casing drilling equipment.</li> </ul>	<ul style="list-style-type: none"> <li>Requires larger (drilling) equipment as compared to micropile drilling equipment.</li> <li>Drilling operation along the east side of the bridge will be in close proximity to overhead hydro lines.</li> </ul>	<ul style="list-style-type: none"> <li>Higher relative cost than footings and driven pile foundation options.</li> <li>Additional cost associated with the detail micropile design.</li> <li>Additional cost for specialized drilling equipment.</li> <li>Additional cost for cofferdam construction and unwatering for construction of the pile cap.</li> <li>Additional cost for the micropile pile load tests.</li> </ul>	<ul style="list-style-type: none"> <li>Few contractors have experience with soil-bonded micropile installation on MTO projects.</li> </ul>

Note: 1 NR – Not Recommended

DRAWINGS



**METRIC**  
DIMENSIONS ARE IN METRES AND/OR  
MILLIMETRES UNLESS OTHERWISE SHOWN  
STATIONS IN KILOMETRES + METRES.

# HIGHWAY 592

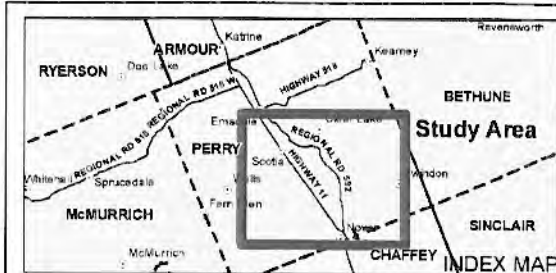
## REPLACEMENT OF SIX STRUCTURES

### KEY MAP

SHEET



**Golder Associates Ltd.**  
MISSISSAUGA, ONTARIO, CANADA



N.T.S

## PLAN

SCALE  
500 0 500 1000m

## REFERENCE

Base data = MNR NRIIS, obtained 2004, DANMAP v2006.4 Produced by  
Order Associates Ltd. Under licence from Ontario Ministry of Natural  
Resources

NO.	DATE	BY	REV'S ON
Geodes No. 31E-334			
HWY 592	PROJECT NO. 11-1111-0149		DIST
SUBM'D AV	CHCKD CN	DATE Dec, 2013	SFE
DRAWN: JFC	CHKD:	APPD:	PAGE 1



**METRIC**  
DIMENSIONS ARE IN METERS AND/OR  
MILIMETERS UNLESS OTHERWISE SHOWN  
STATIONS ARE IN KILOMETERS METERS

CONT No.  
WP No. 5269-07-01

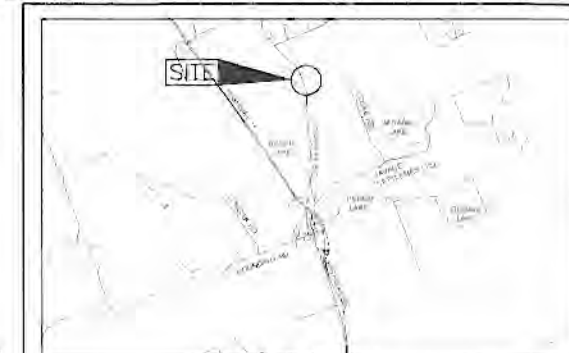


SHEET

HIGHWAY 592  
RAGLAND CREEK BRIDGE  
BOREHOLE LOCATIONS AND SOIL STRATA



**Golder Associates Ltd.**  
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN



### LEGEND

- **Baseline** (current) investigation
- ⊕ **Diagnostic** (not definitive) test
- **Definitive**
- ↑ **Prognostic**
- ④ **Screening** (detection) test (you)
- ③ **Screening** (for) disease (diagnosis) test (SB, OG, Test, OG, OG, OG)
- ▼ **Wt. is diagnostic** (diagnosis) test
- ② **Wt. Up** (diagnosis) test (diagnosis) test
- ① **Referral**

CORRELATION COEFFICIENTS			
$r_{ij}$	ELEVATION	NORTHING	EASTING
BS-01	0.98	0.942290	0.923939
BS-02	0.98	0.940989	0.921133
BS-03	0.98	0.942243	0.923939
BS-04	0.98	0.942213	0.923939
BS-05-06	0.98	0.942286	0.923939

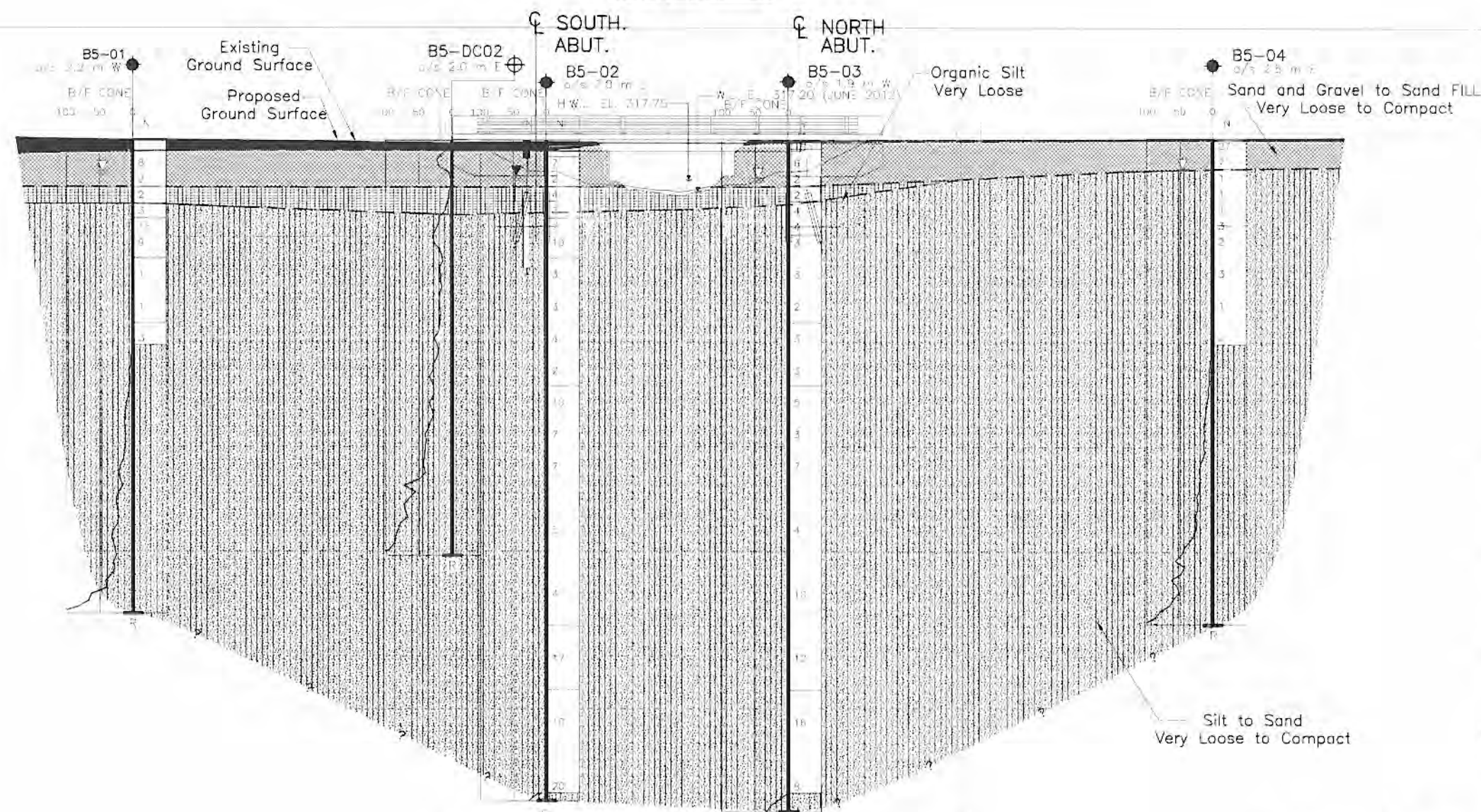
## NOTES

This drawing is for illustrative information only. The proposed structure details shown are shown for illustration purposes only and do not constitute a contract with the final design configuration as shown on drawings or the controls. See drawing.

The boundaries between soil-dwelling fungi have long been fuzzy, as in bacterial taxonomy. (Between lichens, for example, are required for

The complete Foundation investigation and design report for this project and other related documents may be obtained at the Materials Engineering and Research Office. For more information, contact: (415) 893-2100 ext. 2100 or 2101. General Conditions.

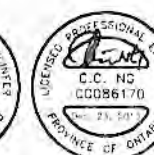
## REFERENCE

[illegible]

### CENTRELINE PROFILE



SCALE



NO.		DATE		BY		REVISION	
Geodesy No		31E-334					
JOB 592		PROJECT NO. 11-1111-D149		DIS.			
SUBMIT AV		CHD. CN		DATE Dec. 2013		SITE 44-17	
DRAWN JFC		CREDIT TVA		APP.		REV. 2	



# APPENDIX A

## Record of Borehole/DCPT Sheets

## LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

### I. GENERAL

$\pi$	3.1416
$\ln x$ ,	natural logarithm of x
$\log_{10}$	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
FoS	factor of safety

### II. STRESS AND STRAIN

$\gamma$	shear strain
$\Delta$	change in, e.g. in stress: $\Delta \sigma$
$\epsilon$	linear strain
$\epsilon_v$	volumetric strain
$\eta$	coefficient of viscosity
$\nu$	Poisson's ratio
$\sigma$	total stress
$\sigma'$	effective stress ( $\sigma' = \sigma - u$ )
$\sigma'_{vo}$	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
$\sigma_{oct}$	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
$\tau$	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

### III. SOIL PROPERTIES

#### (a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight)*
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
$\gamma'$	unit weight of submerged soil ( $\gamma' = \gamma - \gamma_w$ )
$D_R$	relative density (specific gravity) of solid particles ( $D_R = \rho_s / \rho_w$ ) (formerly $G_s$ )
e	void ratio
n	porosity
S	degree of saturation

#### (a) Index Properties (continued)

w	water content
$w_l$ or LL	liquid limit
$w_p$ or PL	plastic limit
$I_p$ or PI	plasticity index $= (w_l - w_p)$
$w_s$	shrinkage limit
$I_L$	liquidity index $= (w - w_p) / I_p$
$I_c$	consistency index $= (w_l - w) / I_p$
$e_{max}$	void ratio in loosest state
$e_{min}$	void ratio in densest state
$I_D$	density index $= (e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

#### (b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

#### (c) Consolidation (one-dimensional)

$C_c$	compression index (normally consolidated range)
$C_r$	recompression index (over-consolidated range)
$C_s$	swelling index
$C_{\alpha}$	secondary compression index
$m_v$	coefficient of volume change
$c_v$	coefficient of consolidation (vertical direction)
$c_h$	coefficient of consolidation (horizontal direction)
$T_v$	time factor (vertical direction)
U	degree of consolidation
$\sigma'_p$	pre-consolidation stress
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

#### (d) Shear Strength

$\tau_p, \tau_r$	peak and residual shear strength
$\phi'$	effective angle of internal friction
$\delta$	angle of interface friction
$\mu$	coefficient of friction $= \tan \delta$
$c'$	effective cohesion
$c_u, s_u$	undrained shear strength ( $\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
$p'$	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
$q_u$	compressive strength $(\sigma_1 - \sigma_3)$
$S_t$	sensitivity

\* Density symbol is  $\rho$ . Unit weight symbol is  $\gamma$  where  $\gamma = \rho g$  (i.e. mass density multiplied by acceleration due to gravity)

Notes: 1  
2

$\tau = c' + \sigma' \tan \phi'$   
shear strength = (compressive strength)/2

## LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

### I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
SS	Split-spoon
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

### III. SOIL DESCRIPTION

#### (a) Non-Cohesive (Cohesionless) Soils

Density Index	N
Relative Density	Blows/300 mm or Blows/ft
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

### II. PENETRATION RESISTANCE

#### Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)

#### Dynamic Cone Penetration Resistance; $N_d$ :

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH:	Sampler advanced by hydraulic pressure
PM:	Sampler advanced by manual pressure
WH:	Sampler advanced by static weight of hammer
WR:	Sampler advanced by weight of sampler and rod

#### Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm<sup>2</sup> pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance ( $Q_t$ ), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

#### (b) Cohesive Soils Consistency

	$c_u, s_u$	
	kPa	psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

### IV. SOIL TESTS

w	water content
w <sub>p</sub>	plastic limit
w <sub>l</sub>	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test <sup>1</sup>
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement <sup>1</sup>
D <sub>R</sub>	relative density (specific gravity, $G_s$ )
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO <sub>4</sub>	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
$\gamma$	unit weight

**Note:** 1 Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU

### V. MINOR SOIL CONSTITUENTS

Per cent by Weight	Modifier	Example
0 to 5	Trace	Trace sand
5 to 12	Trace to Some (or Little)	Trace to some sand
12 to 20	Some	Some sand
20 to 30	(ey) or (y)	Sandy
over 30	And (non-cohesive (cohesionless)) or With (cohesive)	Sand and Gravel Silty Clay with sand / Clayey Silt with sand

PROJECT 11-1111-0149

**RECORD OF BOREHOLE No B5-01**

SHEET 1 OF 2

**METRIC**

W.P. 5269-07-01

LOCATION N 5042269.0 ; E 320122.8

ORIGINATED BY ID

DIST HWY 592

BOREHOLE TYPE 120 mm O.D. Hollow Stem Augers and NW Casing

COMPILED BY GRL/AV

DATUM Geodetic

DATE May 29 and 30, 2013

CHECKED BY CN

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	N° VALUES			SHEAR STRENGTH kPa							
								20 40 60 80 100							
								○ UNCONFINED + FIELD VANE							
								● QUICK TRIAXIAL × REMOULDED							
319.6	GROUND SURFACE														
0.0	Asphalt (460 mm)		1A	AS	-										
319.1			1B	AS											
0.5	Sand, trace to some silt, trace organics (FILL) Very loose to loose Brown Moist to wet		2	SS	8		319								
			3	SS	2		318								
317.4															
2.2	ORGANIC SILT Very loose Dark grey Wet		4	SS	2		317								
316.6															
3.0	Sandy SILT, trace clay, trace organics, containing rootlets Very loose Grey Wet		5	SS	3		316								
315.9			6	SS	15		315								
3.7	SILT and SAND Loose to compact Grey Wet		7	SS	9										
314.0															
5.6	SILT, trace clay, trace sand Very loose Grey Wet		8	SS	1		314								
			9	SS	1		313								
310.9															
8.7	SILT and SAND, trace clay Very loose Grey Wet		10	SS	3		311								
309.8															
9.8	END OF BOREHOLE Dynamic Cone Penetration Test (DCPT)						310								
							309								
							308								
							307								
							306								
							305								

Continued Next Page

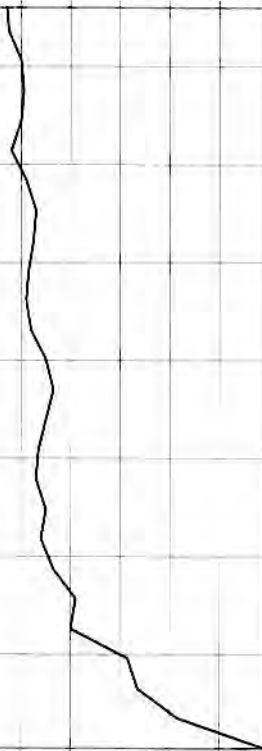
+ 3 x 3

Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

GT-AMT001 11-1111-0149 GPJ GAL-GTA GDT 12/20/13



PROJECT 11-1111-0149		RECORD OF BOREHOLE No B5-01				SHEET 2 OF 2		METRIC					
W.P. 5269-07-01		LOCATION N 5042269.0 E 320122.8				ORIGINATED BY ID							
DIST. HWY 592		BOREHOLE TYPE 120 mm O.D. Hollow Stem Augers and NW Casing				COMPILED BY GRL/AV							
DATUM Geodetic		DATE May 29 and 30, 2013				CHECKED BY CN							
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT $w_p$	NATURAL MOISTURE CONTENT $w$	LIQUID LIMIT $w_L$	UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					
— CONTINUED FROM PREVIOUS PAGE —							20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x REMOULDED	20 40 60					
	END OF BOREHOLE Dynamic Cone Penetration Test (DCPT)												
297.0 22.6	END OF DCPT Refusal to Further Penetration (118 Blows/0.3 m)  NOTES: 1. Water level in open borehole at a depth of 1.5 m below ground surface (Elev. 318.1 m) upon completion of drilling. 2. Borehole caved at a depth of 3.4 m below ground surface (Elev. 316.2 m) upon completion of drilling.												

GTA-MTO-001 11-1111-0149.GPJ GAL-GTA GDT 12/20/13

PROJECT 11-1111-0149		<b>RECORD OF BOREHOLE No B5-02</b>		SHEET 1 OF 3		<b>METRIC</b>	
W.P. 5269-07-01		LOCATION N 5042266 8 E 320113.2		ORIGINATED BY ID			
DIST HWY 592		BOREHOLE TYPE 120 mm O.D. Hollow Stem Augers and NW Casing		COMPILED BY GRL/AV			
DATUM Geodetic		DATE May 28 and 29, 2013		CHECKED BY CN			

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20					
319.6	GROUND SURFACE		1A	AS									
0.0	Asphalt (480 mm)		1B	AS									
319.1													
318.8	Sand and gravel (FILL) Brown Moist		2	SS	7								
0.8	Sand, trace to some silt, trace gravel, trace organics (FILL) Very loose to loose Brown to grey Moist to wet		3	SS	2								
317.4													
2.2	ORGANIC SILT, containing fibrous peat layers Very loose Dark brown to grey Wet		4	SS	4								
			5	SS	3								
315.9													
3.7	SILT and SAND, trace clay Loose Brown Wet		6	SS	7								
			7	SS	10								
314.0													
5.6	SILT, trace to some sand, trace clay Very loose Grey Wet		8	SS	3								
313													
312			9	SS	3								
311													
310			10	SS	4								
309													
308			11	SS	2								
307.9													
11.7	SILT and SAND, trace clay, trace gravel Loose Gray Wet		12	SS	10								
307													
306			13	SS	7								
305													

GTA-MTO 001 11-1111-0149.GPJ GAL-GTA.GDT 12/20/13

Continued Next Page

+ 3, x 3. Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE


PROJECT 11-1111-0149		<b>RECORD OF BOREHOLE No B5-02</b>		SHEET 2 OF 3		<b>METRIC</b>	
W.P. 5269-07-01		LOCATION N 5042286.8 E 320113.2		ORIGINATED BY ID			
DIST HWY 592		BOREHOLE TYPE 120 mm O.D. Hollow Stem Augers and NW Casing		COMPILED BY GRL/AV			
DATUM Geodetic		DATE May 28 and 29, 2013		CHECKED BY CN			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT <div>20 40 60 80 100</div> <div>SHEAR STRENGTH kPa</div> <div>○ UNCONFINED + FIELD VANE</div> <div>● QUICK TRIAXIAL × REMOULDED</div>	PLASTIC LIMIT W <sub>p</sub> NATURAL MOISTURE CONTENT w LIQUID LIMIT W <sub>L</sub> WATER CONTENT (%) <div>20 40 60</div>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
— CONTINUED FROM PREVIOUS PAGE —											
	SILT and SAND, trace clay, trace gravel Loose Grey Wet		14	SS	7		304				
							303				
							302				
			15	SS	5		301				0 43 54 3
							300				
							299				
			16	SS	4		298				
							297				
296.4											
23.2	SILT, some sand, trace clay Compact Grey Wet						296				
			17	SS	17		295				0 14 83 3 Non-plastic
							294				
293.4											
26.2	SAND, some silt Compact Grey Wet						293				
			18	SS	10		292				
							291				
							290				

Continued Next Page

+ 3 x 3. Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

GTA-MTO 001 11-1111-0149.GPJ GAL-GTA.GDT 12/20/13

PROJECT 11-1111-0149		RECORD OF BOREHOLE No B5-02		SHEET 3 OF 3		METRIC																						
W.P. 5269-07-01		LOCATION N 5042286.8, E 320113.2		ORIGINATED BY ID																								
DIST HWY 592		BOREHOLE TYPE 120 mm O.D. Hollow Stem Augers and NW Casing		COMPILED BY GRL/AV																								
DATUM Geodetic		DATE May 28 and 29, 2013		CHECKED BY CN																								
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)														
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20	40						60	80	100	20	40	60	80	100						
	— CONTINUED FROM PREVIOUS PAGE —																											
288.5	SAND, some silt Compact Gray Wet		19	SS	20		289																					
288.1	END OF BOREHOLE																											
31.5	Dynamic Cone Penetration Test (DCPT)																											
	END OF DCPT Refusal to Further Penetration (15 Blows / 0.08 m)																											
	NOTES:																											
	1. Water level in open borehole at a depth of 1.7 m below ground surface (Elev. 317.9 m) upon completion of drilling.																											
	2. An additional borehole was advanced 1.5 m South of borehole B5-02 to install a piezometer and carry out Dynamic Cone Penetration Test, see B5-02 for results of DCPT.																											
	3. Water level measurements in Piezometer:																											
	<table border="1" style="width:100%; border-collapse: collapse;"> <thead> <tr> <th>Date</th> <th>Depth (m)</th> <th>Elev. (m)</th> </tr> </thead> <tbody> <tr> <td>05/29/13</td> <td>2.0</td> <td>317.6</td> </tr> <tr> <td>05/31/13</td> <td>1.8</td> <td>317.8</td> </tr> <tr> <td>06/21/13</td> <td>1.9</td> <td>317.7</td> </tr> </tbody> </table>	Date	Depth (m)	Elev. (m)	05/29/13	2.0	317.6	05/31/13	1.8	317.8	06/21/13	1.9	317.7															
Date	Depth (m)	Elev. (m)																										
05/29/13	2.0	317.6																										
05/31/13	1.8	317.8																										
06/21/13	1.9	317.7																										
	4. Piezometer decommissioned on June 21, 2013.																											



PROJECT 11-1111-0149		RECORD OF BOREHOLE No B5-03		SHEET 1 OF 3		METRIC							
W.P. 5269-07-01		LOCATION N 5042293.1 E 320102.8		ORIGINATED BY ID									
DIST HWY 592		BOREHOLE TYPE 120 mm O.D. Hollow Stem Augers and NW Casing		COMPILED BY AV									
DATUM Geodetic		DATE May 30, 2013		CHECKED BY CN									
SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20 40 60 80 100					
319.6	GROUND SURFACE												
0.0	Asphalt (75 mm)		1A	SS	15								
319.2	Sand and gravel, trace silt (FILL)		1B	SS									
0.4	Compact Brown Moist		2	SS	6								
	Sand, trace gravel, trace silt, trace organics, containing rootlets (FILL)		3	SS	2								
	Very loose to loose Brown Wet												
317.4	ORGANIC SILT, trace to some sand, containing wood fragments		4	SS	2								
2.2	Very loose Dark grey to black												
316.6	Wet		5	SS	4								
3.0	SILT and SAND, trace to some clay, containing fibrous peat layers		6	SS	4								
	Very loose Grey Wet		7	SS	3								
			8	SS	3								
			9	SS	2								
			10	SS	3								
			11	SS	3								
			12	SS	5								
			13	SS	3								



Continued Next Page

+ 3 × 3 Numbers refer to  
Sensitivity ○ 3% STRAIN AT FAILURE

GTA-MTO 001 11-1111-0149.GPJ GAL-GTA.GDT 12/20/13

PROJECT 11-1111-0149		<b>RECORD OF BOREHOLE No B5-03</b>		SHEET 2 OF 3		<b>METRIC</b>	
W.P. 5269-07-01		LOCATION N 5042293.1 E 320102.8		ORIGINATED BY ID			
DIST HWY 592		BOREHOLE TYPE 120 mm O.D. Hollow Stem Augers and NW Casing		COMPILED BY AV			
DATUM Geodetic		DATE May 30, 2013		CHECKED BY CN			

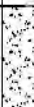
  

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES									
— CONTINUED FROM PREVIOUS PAGE —														
	SILT and SAND, trace to some clay Very loose to compact Grey Wet		14	SS	7		304							
								303						
								302						
								301						
								300						
								299						
								298						
								297						
	SAND, some silt, trace clay Loose to compact Grey Wet		15	SS	4		301						0 65 31 4	
								300						
								299						
								298						0 52 43 5
								297						
								296						
								295						
								294						
293.4							293							
26.2								292						
								291						
								290						

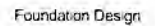
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+ 3 × 3 Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

GTA-MTO 001 11-1111-0149.GPJ GAL-GTA.GDT 12/20/13

PROJECT 11-1111-0149		RECORD OF BOREHOLE No B5-03				SHEET 3 OF 3		METRIC					
W.P. 5269-07-01		LOCATION N 5042293.1 E 320102.8				ORIGINATED BY ID							
DIST HWY 592		BOREHOLE TYPE 120 mm O.D. Hollow Stem Augers and NW Casing				COMPILED BY AV							
DATUM Geodetic		DATE May 30, 2013				CHECKED BY CN							
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					
	— CONTINUED FROM PREVIOUS PAGE —						20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	20 40 60					
288.5	SAND, some silt, trace clay Loose to compact Grey Wet		19	SS	8								0 83 13 4
31.1	END OF BOREHOLE												
287.6	Dynamic Cone Penetration Test (DCPT)												
32.0	END OF DCPT Refusal to Further Penetration (30 Blows / 0.0 m)												
	NOTE:  1. Water level in open borehole at a depth of 1.8 m below ground surface (Elev 317.8 m) upon completion of drilling.												

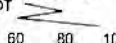
GTAMTO 001 11-1111-0149.GPJ GAL-GTA.GDT 12/20/13

GIA-MTO 001 11-1111-0149.GPJ GAL-GTA.GDT 12/20/13

+ 3, × 3, Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

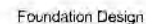
PROJECT 11-1111-0149		RECORD OF BOREHOLE No B5-04				SHEET 2 OF 2		METRIC				
W.P. 5269-07-01		LOCATION N 5042311.6 E 320093.1				ORIGINATED BY ID						
DIST HWY 592		BOREHOLE TYPE 120 mm O.D. Hollow Stem Augers and NW Casing				COMPILED BY GRL/AV						
DATUM Geodetic		DATE May 28, 2013				CHECKED BY CN						
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE								
-- CONTINUED FROM PREVIOUS PAGE --												
296.5	END OF BOREHOLE Dynamic Cone Penetration Test (DCPT)											
296.5	END OF DCPT Refusal to Further Penetration (107 Blows/0.3 m)											
23.2	NOTES:  1. Water level in open borehole at a depth of 1.4 m below ground surface (Elev. 318.3 m) upon completion of drilling.  2. Borehole caved at a depth of 6.7 m (Elev. 313.0 m) below ground surface upon completion of drilling.											



PROJECT 11-1111-0149		RECORD OF DCPT No B5-DC02		SHEET 1 OF 2		METRIC						
W.P. 5269-07-01		LOCATION N 5042285.6 E 320114.2		ORIGINATED BY ID								
DIST HWY 592		BOREHOLE TYPE Dynamic Cone Penetration Test		COMPILED BY GRL								
DATUM Geodetic		DATE May 29 and 31, 2013		CHECKED BY CN								
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT <div style="text-align: center;">  </div>	PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE								
319.6	GROUND SURFACE											
0.0	Dynamic Cone Penetration Test (DCPT)											
319												
318												
317												
316												
315												
314												
313												
312												
311												
310												
309												
308												
307												
306												
305												

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+ 3 × 3 Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE



+3, X3: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

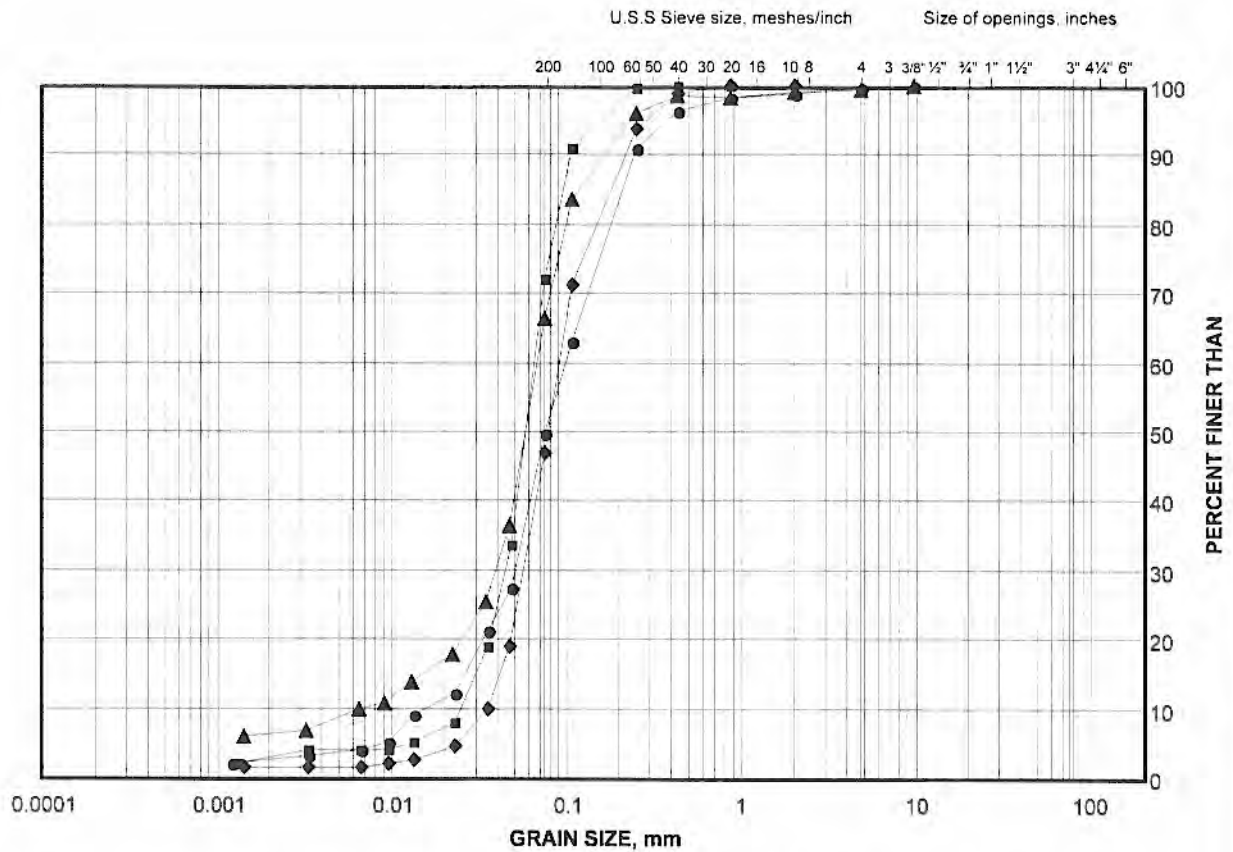
## **APPENDIX B**

### **Laboratory Test Results**

# GRAIN SIZE DISTRIBUTION

Silt and Sand

FIGURE B1A



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED	SAND SIZE			GRAVEL SIZE		SIZE

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	B5-04	3	317.9
■	B5-01	5	316.2
◆	B5-02	6	315.5
▲	B5-03	7	314.7

Project Number: 11-1111-0149

Checked By: AV

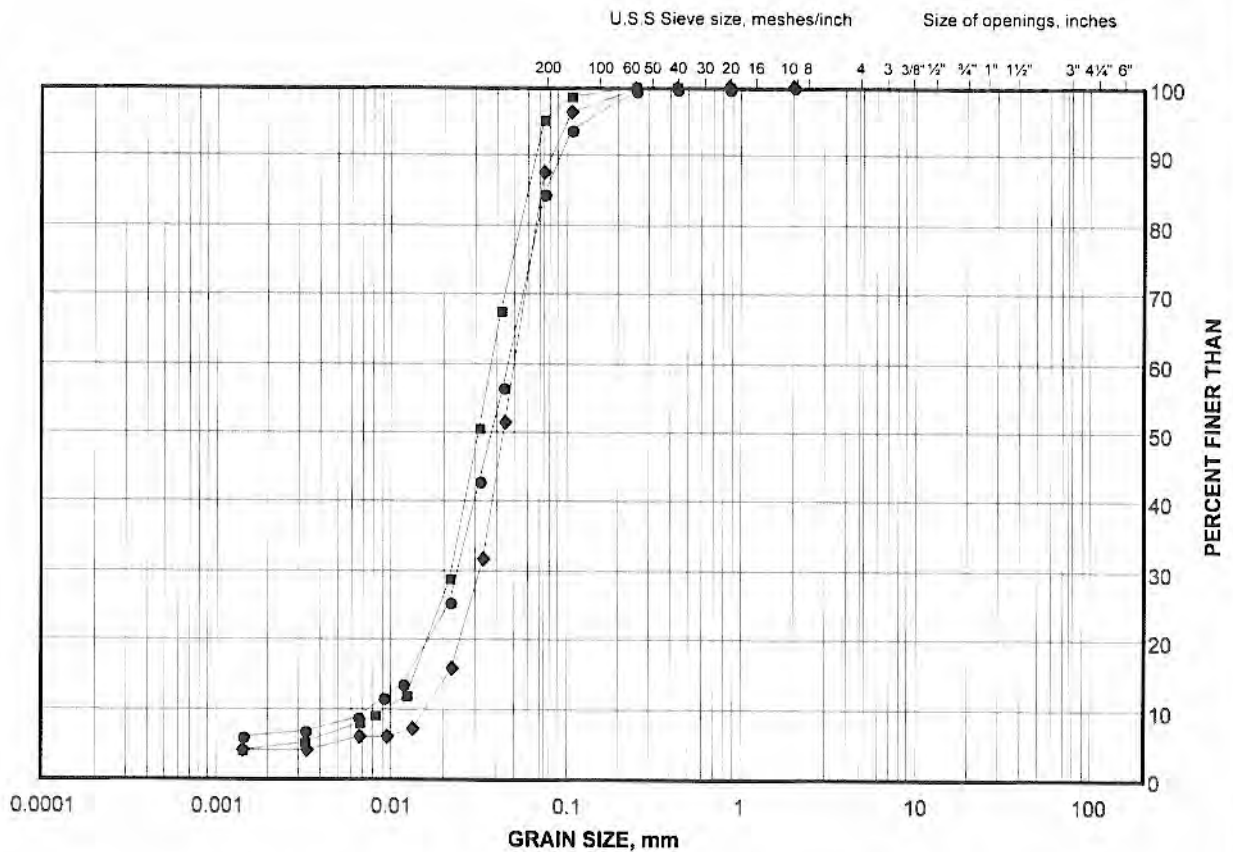
Golder Associates

Date: 06-Sep-13

# GRAIN SIZE DISTRIBUTION

Silt

FIGURE B1B



SILT AND CLAY SIZES		FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED		SAND SIZE			GRAVEL SIZE		SIZE

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	B5-03	10	310.2
■	B5-01	8	313.2
◆	B5-02	9	311.7

Project Number: 11-1111-0149

Checked By: AV

Golder Associates

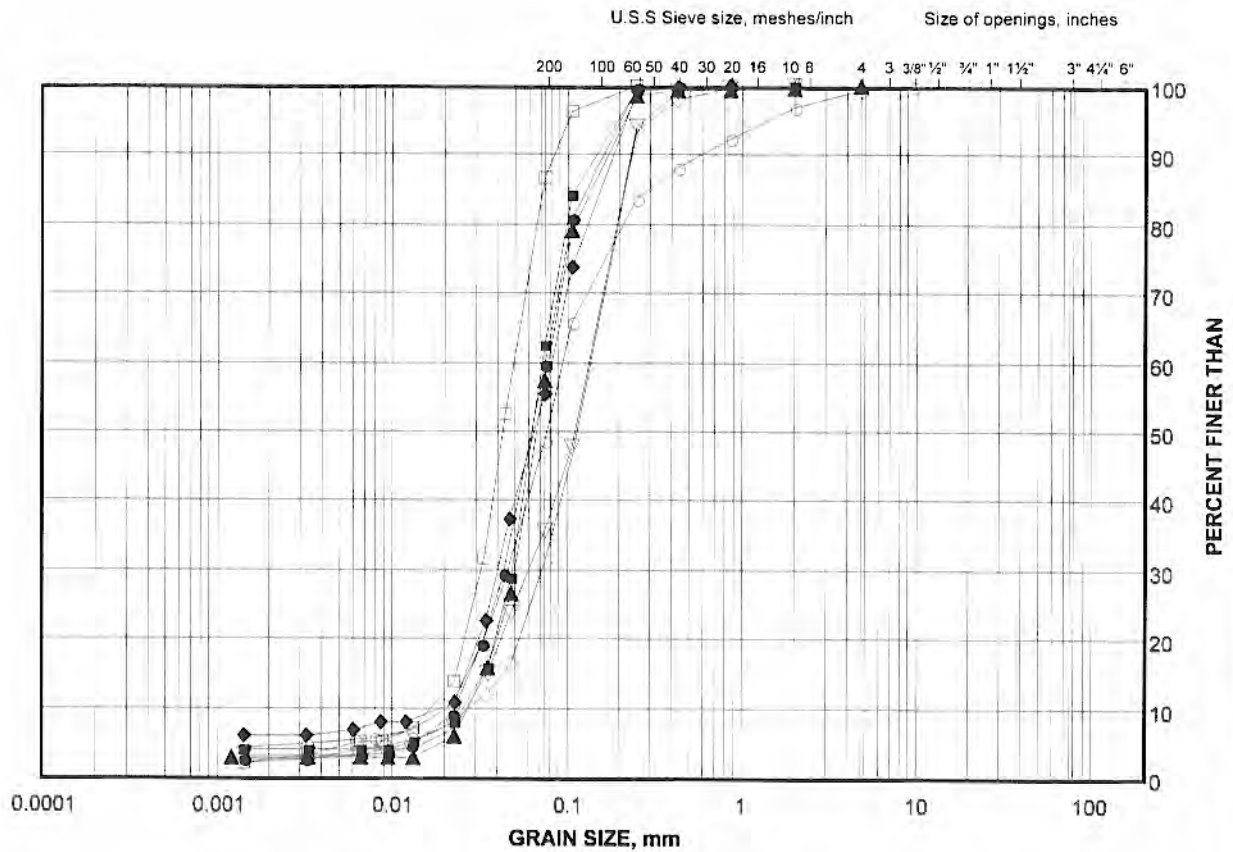
Date: 06-Sep-13



# GRAIN SIZE DISTRIBUTION

Silt to Silty Sand

FIGURE B1C



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED	SAND SIZE			GRAVEL SIZE		SIZE

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	B5-01	10	310.2
■	B5-02	12	307.1
◆	B5-03	13	305.6
▲	B5-02	15	301.0
▽	B5-03	15	301.0
○	B5-03	16	297.9
□	B5-02	17	294.9
△	B5-04	7	314.8

Project Number: 11-1111-0149

Checked By: AV

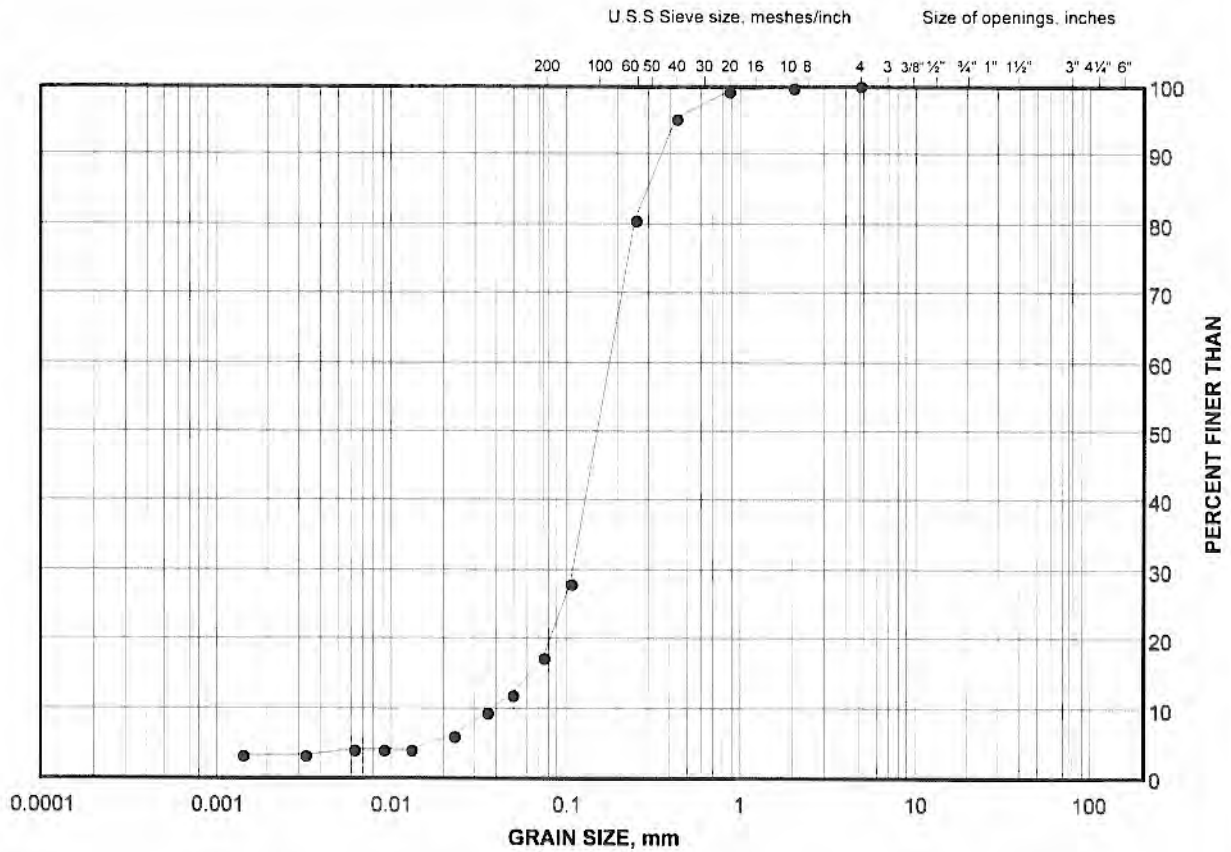
Golder Associates

Date: 06-Sep-13

# GRAIN SIZE DISTRIBUTION

Sand

FIGURE B1D



SILT AND CLAY SIZES			FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED			SAND SIZE			GRAVEL SIZE		

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	B5-03	19	288.8

Project Number: 11-1111-0149

Checked By: AV

**Golder Associates**

Date: 06-Sep-13