



December 23, 2013

## FOUNDATION INVESTIGATION AND DESIGN REPORT

**LITTLE EAST RIVER BRIDGE NO. 4, SITE NO. 44-177  
HIGHWAY 592 - REPLACEMENT OF SIX STRUCTURES  
MINISTRY OF TRANSPORTATION, ONTARIO  
GWP 5265-07-00 WP 5268-07-01**

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REPORT

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# **PART A**

## **FOUNDATION INVESTIGATION REPORT**

**LITTLE EAST RIVER BRIDGE NO. 4 – Site No. 44-177**

**HIGHWAY 592 – REPLACEMENT OF SIX STRUCTURES**

**MINISTRY OF TRANSPORTATION, ONTARIO**

**GWP 5265-07-00; WP 5268-07-01**



## **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 Little East River Bridge No. 4 (site No. 44-177) over Highway 592 in Huntsville, Ontario. The proposed work is part of the replacement of six bridge structures along Highway 592. The Little East River Bridge No. 4 is located approximately 1.5 km south of Bay Lake Road and approximately 4.5 km north of the Highway 11/Novar Road interchange, in between Emsdale and Novar, 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 October 2011. Golder's proposal (Scope of Work) for foundation engineering services associated with the Little East River Bridge 4 structure is contained in Section 6.8 of MH's Technical Proposal of 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 Little East River Bridge No. 4 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, 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 within the project limits 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 treed area and standing water, with Little East River flowing westerly at this location. Residential/recreational properties are located north of the proposed bridge structure. The existing ground surface within the limits of the proposed structure and approach embankments is at about Elevations 328.0 m and 327.7 m, at borehole locations, referenced to Geodetic datum. The Highway 592 south and north approach embankments along the centreline are at about Elevations 327.7 m and 327.9 m, respectively.

## **3.0 INVESTIGATION PROCEDURES**

### **3.1 Foundation Investigation**

The field work for the proposed bridge structure was carried out between May 22 and 28, 2013 during which time a total of four boreholes and one Dynamic Cone Penetration Test (DCPT) were advanced at the location of the structure foundation footprints and approach embankments. In addition, Dynamic Cone Penetration Tests were carried out from the bottom of Boreholes B4-01 to B4-03 to determine depth to refusal at these locations.



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A summary of the respective boreholes advanced at each foundation element and approach embankment is presented below.

Foundation Unit	Borehole
South Approach Embankment	B4-01
South Abutment	B4-02
North Abutment	B4-03 and B4-DC03
North Approach Embankment	B4-04

The results of the borehole investigation and dynamic cone penetration test are presented on the Record of Borehole 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 outside diameter (O.D.) continuous flight hollow-stem augers and 'NW' casing. Soil samples were obtained at intervals of depth of about 0.75 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). Where cobbles and boulders were encountered, samples were recovered using an 'NQ' size rock core barrel. The boreholes and DCPTs were advanced to depths of up to about 31.1 m and 38.0 m below existing ground surface, respectively.

The groundwater conditions in the open boreholes were observed during and upon completion of drilling operations, and a standpipe piezometer was installed in a borehole advanced immediately adjacent to Borehole B4-02 to permit monitoring of the water level at this locations. 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 was also abandoned with cement grout by tremie technique on June 26, 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 soil 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	Borehole / DCPT Depth
	Northing	Easting		
B4-01	5038289.2	324225.6	327.7 m	17.4 m / 21.3 m
B4-02	5038309.5	324223.1	328.0 m	31.1 m / 37.2 m
B4-03	5038318.8	324215.8	327.9 m	31.1 m / 38.0 m
B4-DC03	5038320.2	324214.6	327.9 m	15.9 m
B4-04	5038339.1	324212.7	327.9 m	9.8 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 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/asphalt fragments over a deposit of fill associated with the Highway 592 embankments. The fill is underlain by a deposit of organic silt and/or silt to sand, which is in turn underlain by a deposit of sand and gravel.

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 25 mm to 460 mm thick layer of asphalt/asphalt fragments was encountered at the ground surface in all boreholes.

#### **4.2.2 Sand and Gravel to Sand Fill**

A deposit of non-cohesive fill comprised of sand and gravel to sand, trace gravel was encountered below the asphalt layer in all boreholes. The top of the fill deposit is at between Elevations 327.7 m and 327.4 m and the thickness of the deposit ranges from 0.9 m to 1.5 m.

The SPT 'N'-values measured within the non-cohesive fill deposit range from 7 blows to 19 blows per 0.3 m of penetration, indicating a loose to compact relative density.

The natural water content measured on one sample of the fill is about 7 per cent.

#### **4.2.3 Organic Silt**

A 1.6 m thick pocket of dark grey organic silt, trace to some sand was encountered underlying the fill deposit in Borehole B4-03. Top of the deposit was encountered at Elevation 326.5 m.

The SPT 'N'-values measured within the organic silt pockets range from 2 blows to 3 blows per 0.3 m of penetration, indicating a very loose relative density.

The natural water content and organic content measured on one sample of the organic silt is about 83 per cent and 10 per cent, respectively.

#### **4.2.4 Silt to Sand**

A non-cohesive deposit of silt to sand was encountered underlying the fill deposit in Boreholes B4-01, B4-02 and B4-04 and below the organic silt in Borehole B4-03. The overall silt to sand deposit is comprised of an upper portion of silt and sand to silty sand and a lower portion of silt to sandy silt. The deposit contains trace to some clay and trace organics to a depth of about 11.3 m. In addition, a 6.1 m thick stratum of organic silt and sand in Borehole B4-01. The top of the overall silt to sand deposit ranges from Elevations 326.5 m to 324.9 m and the thickness of the deposit ranges between 8.4 m and 21.7 m, including the thickness of the organic silt and sand in Borehole B4-01. Boreholes B4-01 and B4-04 were terminated within this deposit.

The SPT 'N'-values measured within the overall silt to sand deposit range from 0 blows (weight of hammer) to 26 blows per 0.3 m of penetration, indicating a very loose to compact relative density. The silty sand to silt and sand upper portion of the deposit may be described as very loose to loose, the silt to sandy silt lower portion of the deposit may be described as very loose to compact and the sand pockets are compact in relative density.

The natural water content measured on twenty six samples of the overall deposit ranges from about 20 per cent to 77 per cent, but is generally lower than 36 per cent.

The organic content measured on four samples of the deposit ranges from about 2 per cent to 4 per cent.

The results of grain size distribution tests completed on fourteen samples of the overall silt to sand deposit are shown on Figures B1 in Appendix B, presented for the silt and sand to silty sand upper portion of the deposit on



Figures B1A and B1B, for the sand pockets on Figure B1C and for the silt to sandy silt lower portion of the deposit on Figure B1D.

Atterberg limits test carried out on one sample of the silt and sand portion of this deposit in Borehole B4-04 indicates the fine material to be non-plastic.

Atterberg limits tests were carried out on two upper samples of the silt and sand deposit that contains organics. The limits test carried out on Sample 3 from Borehole B4-04 indicates the fine material to be non-plastic while the test carried out on Sample 7 from Borehole B4-03 measured a liquid limit of about 43 per cent, a plastic limit of about 40 per cent and a plastic index of about 3 per cent. The result of the Atterberg limits test from Borehole B4-03 is presented on Figure B2 and indicates that the fines material of this upper portion of the deposit is classified as silt and sand of medium plasticity.

Within the silt to sand in Borehole B4-01, an approximately 6.1 m thick stratum of organic silt and sand, trace clay, containing fibrous peat layers was encountered at Elevation 322.1 m. The SPT 'N'-values measured within the organic silt and sand ranges from 3 blows to 6 blows per 0.3 m of penetration, indicating a very loose to loose relative density. The natural water content measured on four samples of the organic stratum ranges from about 66 per cent to 110 per cent, while the organic content measured on one sample is about 13 percent. The result of grain size distribution tests completed on a sample of the organic stratum is shown on Figure B3 in Appendix B.

#### **4.2.5 Sand and Gravel**

A non-cohesive deposit of grey sand and gravel was encountered below the silt to sand deposit in Boreholes B4-02 and B4-03. Cobbles were encountered in Borehole B4-02 at about Elevation 301.5 m and a 0.9 m long core sample was recovered. The top of this deposit was encountered at Elevations 304.8 m and 304.1 m and the thickness of deposit is 7.9 m and 7.3 m in Boreholes B4-02 and B4-03, respectively, and potentially up to about 14.2 m thick as inferred from the DCPT driven from the bottom of Borehole B4-03. The DCPT advanced from the bottom of these boreholes were inferred to be terminated within this deposit at depths of 37.2 m and 38.0 m below ground surface, corresponding to Elevations 290.8 m and 290.0 m. The SPT 'N'-values measured within the deposit range from about 12 blows to 34 blows per 0.3 m of penetration, indicating a compact to dense relative density.

The natural water content measured on two samples of the deposit is about 11 per cent and 24 per cent.

### **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 Elevations 327.0 m to 326.4 m measured at depths of between 0.9 m and 1.6 m below ground surface.

A standpipe piezometer was installed in a borehole advanced immediately adjacent to Borehole B4-02 to allow monitoring of the groundwater level at the site. Details of the piezometer installation are shown on the Record of Borehole No. B4-02 in Appendix A and the groundwater level measured in the piezometer is summarized below.



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Borehole	Ground Surface Elevation	Depth to Water Level	Groundwater Elevation	Date of Measurement
B4-02	328.0 m	1.6 m	326.4 m	May 24, 2013
		1.5 m	326.5 m	May 27, 2013
		1.5 m	326.5 m	May 31, 2013
		2.2 m	325.8 m	June 26, 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 Mr. Al Varshoi, M.E.Sc., and reviewed by Mr. 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**

**LITTLE EAST RIVER BRIDGE NO. 4 – SITE NO 44-177**

**HIGHWAY 592 – REPLACEMENT OF SIX STRUCTURES**

**MINISTRY OF TRANSPORTATION, ONTARIO**

**GWP 5265-07-00; WP 5268-07-01**



## **6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS**

This section of the report provides engineering design recommendations for the proposed Little East River Bridge No. 4 on Highway 592 (Site No.44-177). 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 Little East River Bridge No. 4 on Highway 592 in Huntsville, Ontario.

Based on the General Agreement (GA) Drawing provided by MH on November 6, 2013, the proposed Little East River Bridge No. 4 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 328.3 m, which corresponds to a raise of the existing approach embankments of up to about 0.6 m.

### **6.2 Foundation Options**

Given that very loose organic silt and silty 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 support of the abutments.

Given that: bedrock was not encountered to the depths drilled; cobbles were encountered within the sand and gravel deposit at the south abutment area; stage construction will be required in a narrow right-of-way; there is an overhead Hydro line along the existing structure which cannot readily be relocated or de-energized, 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 are very loose and in places have a high organic content. 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 and/or underlain by a deposit of very loose silt to silty sand) at Elevation 325.2 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.

<b>Foundation Location</b>	<b>Factored Geotechnical Axial Resistance at ULS</b>	<b>Geotechnical Reaction at SLS for 25 mm of Settlement</b>
South and North Abutments	350 kPa	65 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.

<b>Interface Material(s)</b>	<b>Coefficient of Friction (<math>\tan \delta'</math>)</b>
Concrete footing on very loose silty sand and/or 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 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 the presence of very loose to compact nature of the silt to sand deposit and the underlying compact to dense sand and gravel deposit, friction piles consisting of steel H-piles driven into the sand and gravel deposit could be considered for the support of the proposed structure. However, cobbles were encountered within the sand and gravel deposit in the boreholes drilled at this site and there is a risk associated with potential difficulty in driving steel H-piles through the cobbles and/or the potential for the steel H-piles refusing on the cobbles depending on the thickness and lateral extent of the cobbles deposit. In addition, due to the proposed construction sequencing/staging, the narrow right-of-way and the presence of overhead Hydro lines along the west side of the bridge, there may not be adequate construction platform width to accommodate piling equipment necessary to the required depth to drive long H-piles 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.

Foundation Location	Elevation of Underside of Pile Cap <sup>1</sup>	Elevation of Underside of Tremie Plug <sup>1</sup>	Pile 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	325.2 m	324.0 m	302.5 m	22.7 m	725 kN	N/A

Notes:

1. As per the GA Drawing provided by MH on November 6, 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.

Taking into consideration the possibility of encountering refusal on cobbles within the sand and gravel deposit, provisions should be made in the Contract Documents to deal with varying pile lengths at the abutments.





All piles should be fitted with driving shoes and flange plates (reinforced tips) in accordance with OPSP 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 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 1,825 kN per pile, but must be driven below El. 302.5 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 presence of cobbles within the sand and gravel deposits, it is recommended that a down-the-hole (DTH) hammer drilling system be used for the installation of the drilled steel casing. However, due to the proposed construction sequencing/staging, narrow right-of-way and the presence of overhead Hydro lines along the east side of the bridge, 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	325.2 m	324.0 m	302.5 m	22.7 m	1,125 kN	N/A

Notes:

1. As per the GA Drawing provided by MH on November 6, 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 granular deposits and presence of cobbles) 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	325.2 m	324.0 m	311.7 m	273 mm	13.5 m

Note:

1. As per the GA Drawing provided by MH on November 6, 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):

for non-cohesive soils:

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



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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 (B3-02) and North Abutment (B3-03)	Very Loose to Loose Silt to Sand	321.0 m to 316.0 m	3,000 kPa/m	-
	Compact Silt to Sand	316.0 m to 304.1 m	8,000 kPa/m	-
	Compact to Dense Sand and Gravel	304.1 m to 290.0 m	20,000 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	725 kN	85 kN	20 kN
	610 mm diameter drilled steel casing	1,125 kN	75 kN	25 kN

Note:

- 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*).



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:

<b>Pile Spacing in Direction of Loading (<math>d</math> = pile diameter)</b>	<b>Subgrade Reaction Reduction Factor, <math>R</math></b>
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 0.078g 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 Little East River Bridge No. 4 structure will be at about Elevation 328.3 m, requiring placement of up to about 0.6 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 deposits of very loose to compact silt to sand and/or organic silt, underlain by a deposit of very loose to compact silt to sand, which is in turn underlain by a deposit of compact to dense sand and gravel.

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. Further, it is understood that a preload period of one year will be included in the construction schedule to allow for the settlement/consolidation of the underlying organic soils, inorganic deposits as well as the rock fill.

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 cohesive soils, total stress parameters were employed in the analyses assuming undrained conditions. The total stress parameters (i.e. average mobilized undrained shear strength –  $s_u$ ) for the cohesive soils were estimated from correlations with the SPT results and other laboratory test data (i.e. natural water content), where appropriate.





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For the purpose of the stability analysis, the groundwater level was assumed to be at Elevation 325.8 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.

Embankment	Soil Type	Unit Weight, $\gamma$	Undrained Shear Strength, $s_u$	Cohesion, $c'$	Effective Friction Angle, $\phi'$
South Approach Embankment	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 Loose to Compact Sand to Sand and Gravel Fill	20 kN/m <sup>3</sup>	-	0 kPa	30°
	Very Loose to Compact Silt to Sand	19 kN/m <sup>3</sup>	-	0 kPa	28°
	Very Loose to Loose Organic Silt and Sand	18 kN/m <sup>3</sup>	-	0 kPa	27°
	Compact Sand and Gravel	20 kN/m <sup>3</sup>	-	0 kPa	34°
North Approach Embankment	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 Loose to Compact Sand and Gravel Fill	20 kN/m <sup>3</sup>	-	0 kPa	30°
	Very Loose Organic Silt	18 kN/m <sup>3</sup>	-	0 kPa	27°
	Very Loose to Compact Silt to Sand	19 kN/m <sup>3</sup>	-	0 kPa	28°
	Compact to Dense Sand and Gravel	20 kN/m <sup>3</sup>	-	0 kPa	34°

### 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	2.1 m	1.25H:1V	≥ 1.3

Note:

1. Embankment height includes an approximately 0.6 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.

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 36.6 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.



## FOUNDATION REPORT - LITTLE EAST RIVER BRIDGE NO.4 - HIGHWAY 592 GWP 5265-07-00; WP 5268-07-01

Embankment	Soil Type	Thickness <sup>1</sup>	Unit Weight, $\gamma$	Deformation Parameter(s)
South Approach Embankment	Existing Loose to Compact Sand to Sand and Gravel Fill	~1.5 m	20 kN/m <sup>3</sup>	$E' = 5 \text{ MPa}$
	Very Loose to Compact Silt to Sand	4.1 m to 21.7 m	19 kN/m <sup>3</sup>	$E' = 3 \text{ MPa to } 8 \text{ MPa}$
	Very Loose to Loose Organic Silt and Sand	~6.1 m	18 kN/m <sup>3</sup>	$e_o = 2.0$ $C_c = 1.0$ $C_{\alpha(\epsilon)} = 0.06$ $c_v = 1.0 \times 10^{-3} \text{ cm}^2/\text{s}$
	Compact Sand and Gravel	~14 m	20 kN/m <sup>3</sup>	$E' = 25 \text{ MPa}$
North Approach Embankment	Existing Loose to Compact Sand and Gravel Fill	~1.4 m	20 kN/m <sup>3</sup>	$E' = 5 \text{ MPa}$
	Very Loose Organic Silt	~1.6 m	18 kN/m <sup>3</sup>	$e_o = 2.0$ $C_c = 1.0$ $C_{\alpha(\epsilon)} = 0.06$ $c_v = 1.0 \times 10^{-3} \text{ cm}^2/\text{s}$
	Very Loose to Compact Silt to Sand	~20.8 m	19 kN/m <sup>3</sup>	$E' = 3 \text{ MPa to } 8 \text{ MPa}$
	Compact to Dense Sand and Gravel	~14.2 m	20 kN/m <sup>3</sup>	$E' = 25 \text{ MPa}$

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 (cm<sup>2</sup>/s)

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 325.8 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.



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Embankment	Settlement During Construction (including 1 year Preload Period)		Post-Construction Settlement (10 Years After Preload Period)		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	85 mm	40 mm	125 mm	~0 mm	250 mm
South Approach Embankment Side Slope <sup>3</sup>	~0 mm	55 mm	~0 mm	~0 mm	55 mm
North Approach Embankment Centreline	75 mm	45 mm	35 mm	~0 mm	155 mm
North Approach Embankment Side Slope <sup>3</sup>	~0 mm	45 mm	~0 mm	~0 mm	45 mm

Notes:

1. Organic soils include the organic silt and organic silt and sand deposits.
2. Inorganic soils include the silt to sand and sand and gravel 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 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 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 1.5 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 and compaction of granular fill for the grade raise and widening of the approach embankment should be carried out 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 325.2 m and the underside of the tremie plug at Elevation 324.0 m, excavations up to about 4 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 Little East River.

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.

### **6.11.4 Obstructions**

It should be noted that cobbles were encountered within the native sand and gravel deposit during borehole advancement. The presence of such obstructions could affect the construction of deep foundations. It is recommended that a NSSP be included in the Contract Documents to warn the Contractor of these obstructions and to ensure that the Contractor is equipped to handle such obstructions; an example NSSP is included in Appendix C.

## **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.



FOUNDATION REPORT - LITTLE EAST RIVER BRIDGE NO.4 -  
HIGHWAY 592 GWP 5265-07-00; WP 5268-07-01

## Report Signature Page

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TZ/CN/JMAC/sm

n:\active\2011\11-1111-0149-4 - highway 592 - huntsville\reporting\verb no.4\final\11-1111-0149-4 rpt 13dec23 fidr little east river bridge no.4.docx



## REFERENCES

- Broms, B. B., 1964. Lateral Resistance of Piles in Cohesive Soils. Journal of the Soil Mechanics and Foundations Divisions, ASCE, Vol. 90, No. SM2, Proc. Paper 3825, pp. 27-63.
- Bowles, J.E. 1984. Physical and Geotechnical Properties of Soils, Second Edition. McGraw Hill Book Company, New York.
- Canadian Geotechnical Society. 1992. Canadian Foundation Engineering Manual – 3<sup>rd</sup> Edition.
- Canadian Highway Bridge Design Code (CHBDC) and Commentary on CAN/CSA-S6-06. 2006. CSA Special Publication, S6.1-06. Canadian Standard Association.
- Chapman, L.J. and Putnam, D.F., 1984. *The Physiography of Southern Ontario*, Ontario Geological Survey, Special Volume 2, Third Edition. Accompanied by Map P.2715, Scale 1:600,000.
- 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.
- Kulhawy, F.H. and Mayne, P.W. 1990. Manual on Estimating Soil Properties for Foundation Design. EL-6800, Research Project 1493-6. Prepared for Electric Power Research Institute, Palo Alto, California.
- Mesri, G. and Ajlouni, M. 2007. Engineering Properties of Fibrous Peat. Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 133, Issue 7, pp. 850-866.
- National Research Council of Canada. Associate Committee on Geotechnical Research. Muskeg Subcommittee. 1969. Muskeg Engineering Handbook. *Edited by* MacFarlane, I.C. University of Toronto Press.
- Peck, R.B., Hanson, W.E., and Thornburn, T.H. 1974. Foundation Engineering, Second Edition, John Wiley and Sons, New York.
- Reese, L.C., 1975. Laterally Loaded Piles. GESA Report D-75-14, UCCC Report 75-14, Geotechnical Engineering Software Activity, University of Colorado Computing Centre, Boulder.
- Terzaghi, K., 1955. Evaluation of Coefficients of Subgrade Reaction. Geotechnique, Vol. 5, pp. 297-326.
- Unified Facilities Criteria, U.S. Navy. 1986. NAVFAC Design Manual 7.02. Soil Mechanics, Foundation and Earth Structures. Alexandria, Virginia.
- ASTM International:
- |            |   |
|------------|---|
| ASTM D1586 | Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils |
|------------|---|
- Commercial Software:
- 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):





## FOUNDATION REPORT - LITTLE EAST RIVER BRIDGE NO.4 - HIGHWAY 592 GWP 5265-07-00; WP 5268-07-01

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 SS 103-11. 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



## FOUNDATION REPORT - LITTLE EAST RIVER BRIDGE NO.4 - HIGHWAY 592 GWP 5265-07-00; WP 5268-07-01

**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 very loose to loose organic silt and silt to sand deposits 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 sheetpile cofferdam.</li> <li>Excavation for pile cap will be below water table.</li> <li>Cofferdam (with concrete</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> </ul>	<ul style="list-style-type: none"> <li>Potential for requirement to drive piles deeper to develop adequate axial capacity during construction.</li> <li>Potential difficulty driving or damage piles through the cobbles present in the sand and gravel deposit.</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 de-energized during portions of the piling operation.</li> </ul>



## FOUNDATION REPORT - LITTLE EAST RIVER BRIDGE NO.4 - HIGHWAY 592 GWP 5265-07-00; WP 5268-07-01

**Table 1: Evaluation of Foundation Alternatives**

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks / Consequences
			<p>tremie plug) and unwatering will be required for construction of the pile caps within a dry excavation.</p> <ul style="list-style-type: none"> <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>		
Drilled steel casings using DTH hammer drilling system (610 mm)	3	<ul style="list-style-type: none"> <li>Reduced number of deep foundation elements compared to steel H-piles.</li> <li>DTH drilling can readily penetrate through cobbles in overburden.</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>Allows only for semi-integral abutment design.</li> <li>Given the thickness of the overburden, axial capacity will be developed through shaft resistance (i.e. friction steel casing) only.</li> <li>Drilling slurry will be required to balance groundwater pressures and minimize basal heave.</li> <li>Excavation for pile cap will be below water table.</li> <li>Cofferdam (with concrete tremie plug) and unwatering will be required for construction of the pile cap within a dry excavation.</li> <li>Requires larger (drilling) equipment as compared to</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> </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>Specialized drilling equipment and/or methods could be required to penetrate cobbles present in the sand and gravel deposit.</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 de-energized during portions of the drilling operation.</li> </ul>



## FOUNDATION REPORT - LITTLE EAST RIVER BRIDGE NO.4 - HIGHWAY 592 GWP 5265-07-00; WP 5268-07-01

**Table 1: Evaluation of Foundation Alternatives**

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks / Consequences
			micropile drilling equipment. ■ Drilling operation along the east side of the bridge will be in close proximity to overhead hydro lines.		
Micropiles (273 mm diameter)	1	■ Negligible post-construction settlement. ■ Potential for achieving high axial capacity in the overburden using pressure grouting techniques. ■ Drilling equipment will readily penetrate cobbles in the sand and gravel deposit. ■ Requires smaller drilling equipment as compared to steel casing drilling equipment.	■ Allows only for semi-integral abutment design. ■ Detail micropile design will be required. ■ Pile load tests required to confirm capacity for design. ■ Cofferdam and unwatering will be required for construction of the pile cap within a dry excavation.	■ Higher relative cost than footings and driven pile foundation options. ■ Additional cost associated with the detail micropile design. ■ Additional cost for specialized drilling equipment. ■ Additional cost for cofferdam construction and unwatering for construction of the pile cap. ■ Additional cost for the micropile pile load tests.	■ Few contractors have experience with soil-bonded micropile installation on MTO projects.

Note: 1. NR – Not Recommended



# **DRAWINGS**



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

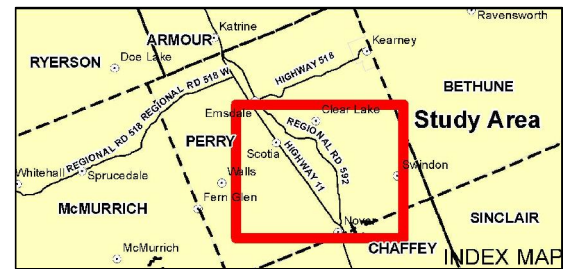
CONT No.  
WP No. 5268-07-01

HIGHWAY 592  
REPLACEMENT OF SIX STRUCTURES  
KEY MAP

SHEET



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



N.T.S

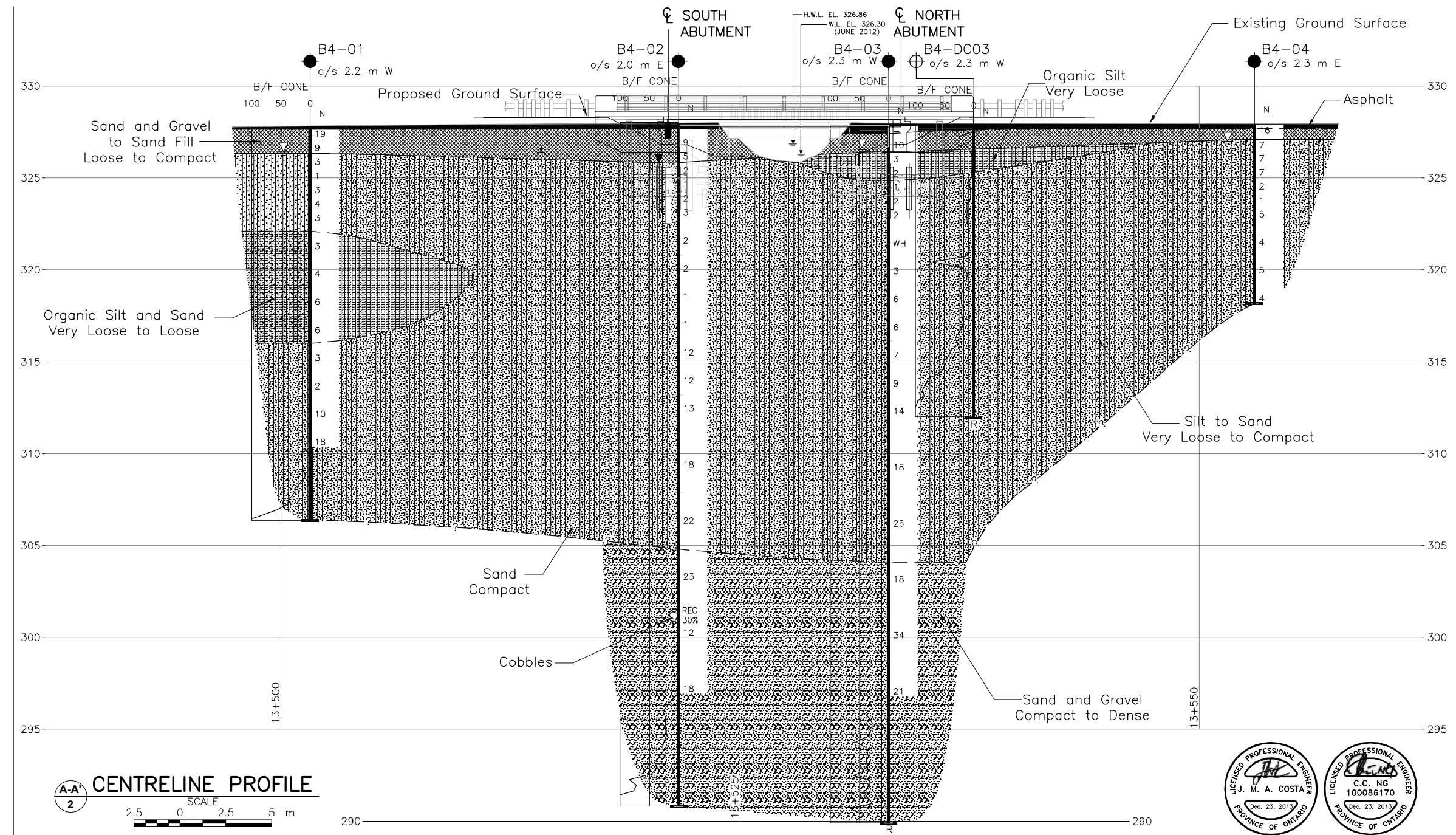
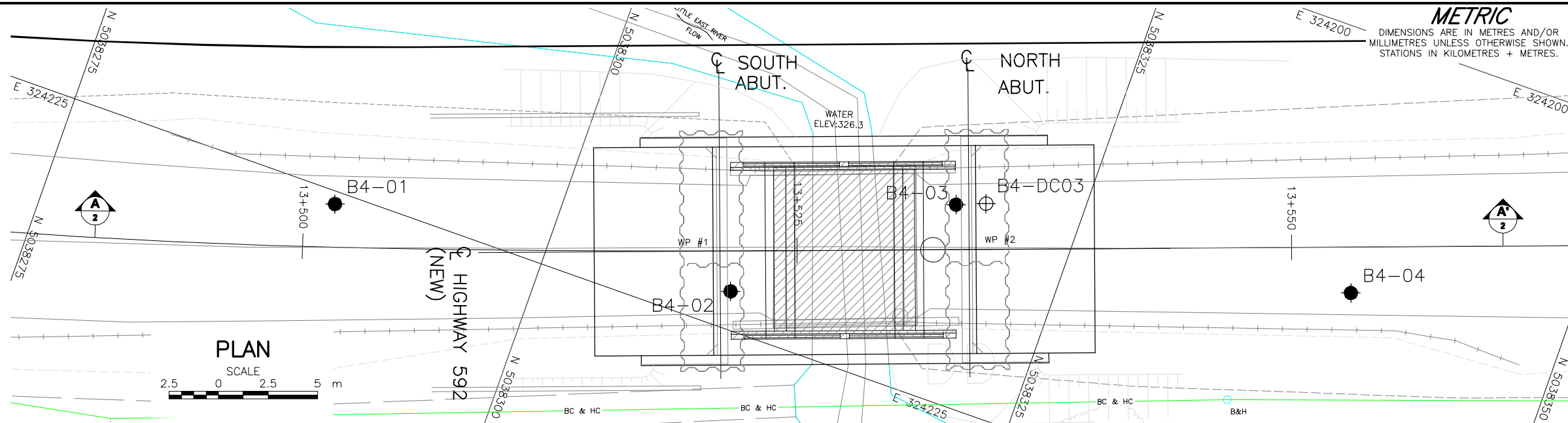


REFERENCE


Base data – MNR NRVIS, obtained 2004, CANMAP v2006.4 Produced by Golder Associates Ltd. under licence from Ontario Ministry of Natural Resources

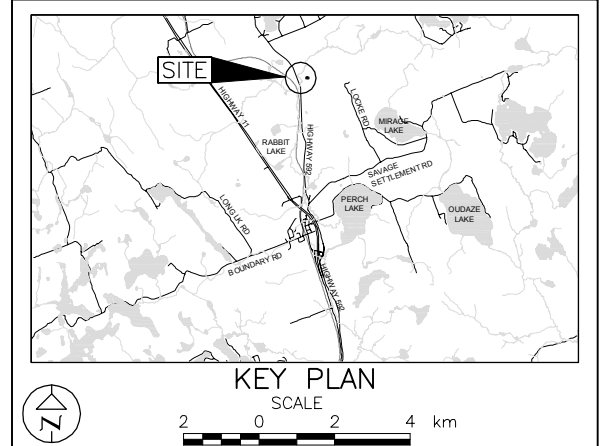
NO.	DATE	BY	REVISION	
Access No. 31E-333				
Y. 592		PROJECT NO. 11-1111-0149		DIST.
BM'D. AV		CHKD. CN	DATE: Dec. 2013	SITE:
AWN: JFC		CHKD.	APPD.	DWG. 1











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

CONT No. WP No. 5268-07-01		
HIGHWAY 592 LITTLE EAST RIVER BRIDGE #4 BOREHOLE LOCATIONS AND SOIL STRATA		SHEET



- | LEGEND  |  |
|---|--|
|    | Borehole — Current Investigation                                   |
|    | Dynamic Cone Penetration Test                                      |
|    | Seal   |
|    | Piezometer   |
| N   | Standard Penetration Test Value                                    |
| 16  | Blows/0.3m unless otherwise stated<br>(Std. Pen. Test, 475 j/blow) |
| REC   | Total core recovery  |
|  | WL in piezometer, measured on June 26, 2013                        |
|  | WL upon completion of drilling                                     |
| R   | Refusal  |

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
B4-01	327.7	5038289.2	324225.6
B4-02	328.0	5038309.5	324223.1
B4-03	327.9	5038318.8	324215.2
B4-04	327.9	5038339.1	324212.7
B4-DC03	327.9	5038320.2	324214.6

## NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

## REFERENCE

Base plans provided in digital format by MH, drawing file nos.  
 x1114246\_44-174-44-175\_44-176align.dwg, x1114246\_44177align.dwg,  
 x1114246\_44178\_44166align.dwg and  
 x1114246\_44-174\_44-175\_44-176base.dwg, x1114246\_44177base.dwg  
 and x1114246\_44178\_44166base.dwg, received June 11, 2013 and  
 General Arrangement Plan and Profile file no. 44177-01.dwg, received  
 November 7, 2013.

NO.	DATE	BY	REVISION		
Geocres No. 31E-333					
HWY. 592			PROJECT NO. 11-1111-0149		DIST.
SUBM'D. AV		CHKD. CN	DATE: Dec. 2013	SITE: 44-177	
DRAWN: JFC		CHKD.	APPD.	DWG. 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

<b>(a)</b>	<b>Index Properties</b>
$\rho(\gamma)$	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

$$\tau = c' + \sigma' \tan \phi'$$

$$\text{shear strength} = (\text{compressive strength})/2$$



## 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

### 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.

### 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

#### (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 B4-01		SHEET 2 OF 2		METRIC								
W.P.		5268-07-01		LOCATION		N 5038289.2 ; E 324225.6		ORIGINATED BY								
DIST		HWY 592		BOREHOLE TYPE		120 mm O.D. Hollow Stem Augers and NW Casing		COMPILED BY								
DATUM		Geodetic		DATE		May 22, 2013		CHECKED BY								
								TVA								
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 ---															
310.3	SILT, trace to some sand, trace to some clay, trace organics Very loose to compact Grey Wet		14	SS	10		312									
							311									
17.4	END OF BOREHOLE Dynamic Cone Penetration Test (DCPT)		15	SS	18		310									
							309									
							308									
							307									
306.4	END OF DCPT															
21.3	NOTES:  1. Water level in open borehole at a depth of 1.3 m below ground surface (Elev. 326.4 m) upon completion of drilling.  2. Borehole caved at a depth of 11.6 m (Elev. 316.1 m) below ground surface upon completion of drilling.															

PROJECT 11-1111-0149		<b>RECORD OF BOREHOLE No B4-02</b>		SHEET 1 OF 3		<b>METRIC</b>	
W.P. 5268-07-01		LOCATION N 5038309.5 ; E 324223.1		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 23 and 24, 2013		CHECKED BY TVA			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT  γ  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR   SA   SI   CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)							
								20	40	60	80	100	W <sub>p</sub>	W	W <sub>L</sub>					
328.0	GROUND SURFACE																			
0.0	Asphalt fragments (300 mm)		1A	AS	-															
0.3	Sand, trace gravel, trace silt (FILL) Compact Brown Moist		1B																	
			2	SS	9															
326.5																				
1.5	Silty SAND, trace organics Very loose to loose Dark brown Moist		3	SS	5															
			4	SS	2															
			5	SS	1															
324.2																				
3.8	SAND, some silt, trace to some clay, trace organics Very loose Dark grey Wet		6	SS	2															
			7	SS	3															
			8	SS	2															
320.8																				
7.2	Sandy SILT, trace clay, trace organics to a depth of 11.3 m Very loose to compact Dark grey becoming brown below a depth of 11.3 m Moist to wet		9	SS	2															
			10	SS	1															
			11	SS	1															
			12	SS	12															
			13	SS	12															
313.2																				
14.8																				

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

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+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3%</sup> STRAIN AT FAILURE

PROJECT		11-1111-0149		RECORD OF BOREHOLE No B4-02		SHEET 3 OF 3		METRIC							
W.P.		5268-07-01		LOCATION		N 5038309.5 ; E 324223.1		ORIGINATED BY							
DIST		HWY 592		BOREHOLE TYPE		120 mm O.D. Hollow Stem Augers and NW Casing		COMPILED BY							
DATUM		Geodetic		DATE		May 23 and 24, 2013		CHECKED BY							
TVA															
SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS		ELEVATION SCALE		DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES										
296.9	SAND and GRAVEL Compact Grey Wet Granite rock fragments below a depth of 30.5 m.		19	SS	18										
31.1	END OF BOREHOLE Dynamic Cone Penetration Test (DCPT)														
290.8	END OF DCPT Refusal to Further Penetration														
37.2	NOTES:  1. Water level in open borehole measured at a depth of 1.6 m below ground surface (Elev. 326.4 m) upon completion of drilling.  2. An additional borehole was advanced about 1.5 m South of Borehole B4-02 to install a piezometer.  3. Water level measurements in Piezometer:  Date      Depth (m)      Elev. (m) 05/24/13      1.6      326.4 05/27/13      1.5      326.5 05/31/13      1.5      326.5 06/26/13      2.2      325.8  4. Piezometer decommissioned on June 26, 2013.														



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+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3%</sup> STRAIN AT FAILURE


PROJECT 11-1111-0149			<b>RECORD OF BOREHOLE No B4-03</b>			SHEET 2 OF 3			<b>METRIC</b>								
W.P. 5268-07-01			LOCATION N 5038318.8 ; E 324215.2			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 27 and 28, 2013			CHECKED BY TVA											
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 $\gamma$ 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									WATER CONTENT (%)
--- CONTINUED FROM PREVIOUS PAGE ---																	
	SILT, trace to some sand, trace clay Loose to compact Grey Wet		14	SS	14		312										
							311										
							310										
			15	SS	18		309										
							308										
							307										
			16	SS	26		306										0 5 91 4
							305										
304.1							304										
23.8	SAND and GRAVEL Compact to dense Grey Wet		17	SS	18		303										
							302										
							301										
			18	SS	34		300										
							299										
							298										

Continued Next Page

+ <sup>3</sup>, × <sup>3</sup>: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

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PROJECT <u>11-1111-0149</u>		<b>RECORD OF BOREHOLE No B4-03</b>		SHEET 3 OF 3		<b>METRIC</b>	
W.P. <u>5268-07-01</u>		LOCATION <u>N 5038318.8 ; E 324215.2</u>		ORIGINATED BY <u>ID</u>			
DIST <u>          </u> HWY <u>592</u>		BOREHOLE TYPE <u>120 mm O.D. Hollow Stem Augers and NW Casing</u>		COMPILED BY <u>GRL/AV</u>			
DATUM <u>Geodetic</u>		DATE <u>May 27 and 28, 2013</u>		CHECKED BY <u>TVA</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			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		W <sub>p</sub>	W	W <sub>L</sub>		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	WATER CONTENT (%)					
						20 40 60 80 100	20 40 60 80 100	20 40 60						
296.8	SAND and GRAVEL Compact to dense Grey Wet		19	SS	21		297							
31.1	END OF BOREHOLE Dynamic Cone Penetration Test (DCPT)						296							
							295							
							294							
							293							
							292							
							291							
290.0	END OF DCPT Refusal to Further Penetration (85 Blows / 0.15 m)						290							
38.0	NOTE:  1. Water level in open borehole at a depth of 1.2 m below ground surface (Elev. 326.7 m) upon completion of drilling.													

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

PROJECT 11-1111-0149		RECORD OF BOREHOLE No B4-04		SHEET 1 OF 1		METRIC						
W.P. 5268-07-01		LOCATION N 5038339.1 ; E 324212.7		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 22 and 23, 2013		CHECKED BY TVA								
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					
327.9	GROUND SURFACE											
0.0	Asphalt (200 mm)		1A	SS	16							
0.2	Sand and gravel, trace silt (FILL) Loose to compact Dark grey Moist		1B									
			2	SS	7							
326.5												
1.4	SILT and SAND, trace organics to a depth of 4.4 m, trace clay to a depth of 5.2 m Very loose to loose Dark grey becoming brown below a depth of 5.2 m Moist to wet		3	SS	7						OC=1.8%	0 60 34 6 Non-plastic
			4	SS	7							
			5	SS	2							
			6	SS	1							
			7	SS	5							0 34 60 6
			8	SS	4							
			9	SS	5							
			10	SS	4							
318.2												
9.8	END OF BOREHOLE											
	NOTE: 1. Water level in open borehole at a depth of 0.9 m below ground surface (Elev. 327.0 m) upon completion of drilling.											



PROJECT <u>11-1111-0149</u>		<b>RECORD OF DCPT No B4-DC03</b>				SHEET 2 OF 2		<b>METRIC</b>												
W.P. <u>5268-07-01</u>		LOCATION <u>N 5038320.2 ; E 324214.6</u>				ORIGINATED BY <u>ID</u>														
DIST <u>          </u> HWY <u>592</u>		BOREHOLE TYPE <u>Dynamic Cone Penetration Test</u>				COMPILED BY <u>GRL/AV</u>														
DATUM <u>Geodetic</u>		DATE <u>May 27, 2013</u>				CHECKED BY <u>TVA</u>														
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 $\gamma$ 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 ---						<div style="display: flex; justify-content: space-between;"> <span>20 40 60 80 100</span> <span>20 40 60 80 100</span> </div> <div style="display: flex; justify-content: space-between;"> <span>○ UNCONFINED</span> <span>+ FIELD VANE</span> </div> <div style="display: flex; justify-content: space-between;"> <span>● QUICK TRIAXIAL</span> <span>× REMOULDED</span> </div>													
312.1	Dynamic Cone Penetration Test (DCPT)					312	127													
15.9	END OF DCPT Refusal to Further Penetration (127 Blows)																			

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# **APPENDIX B**

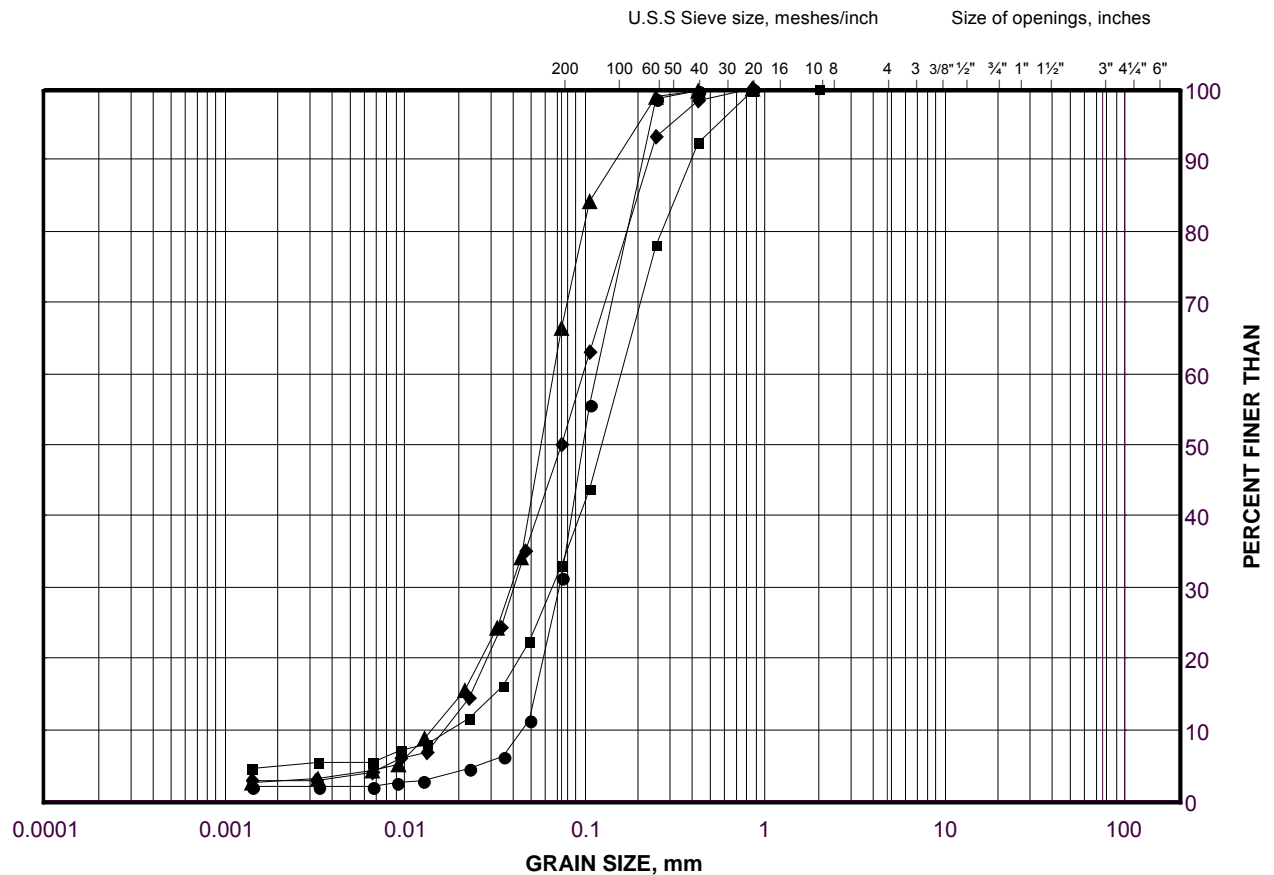
## **Laboratory Test Results and Bedrock Core Photographs**



# GRAIN SIZE DISTRIBUTION

Silt and Sand to Silty 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)
●	B4-03	11	316.9
■	B4-03	5	324.5
◆	B4-03	7	323.0
▲	B4-03	9	319.9

Project Number: 11-1111-0149

Checked By: AV

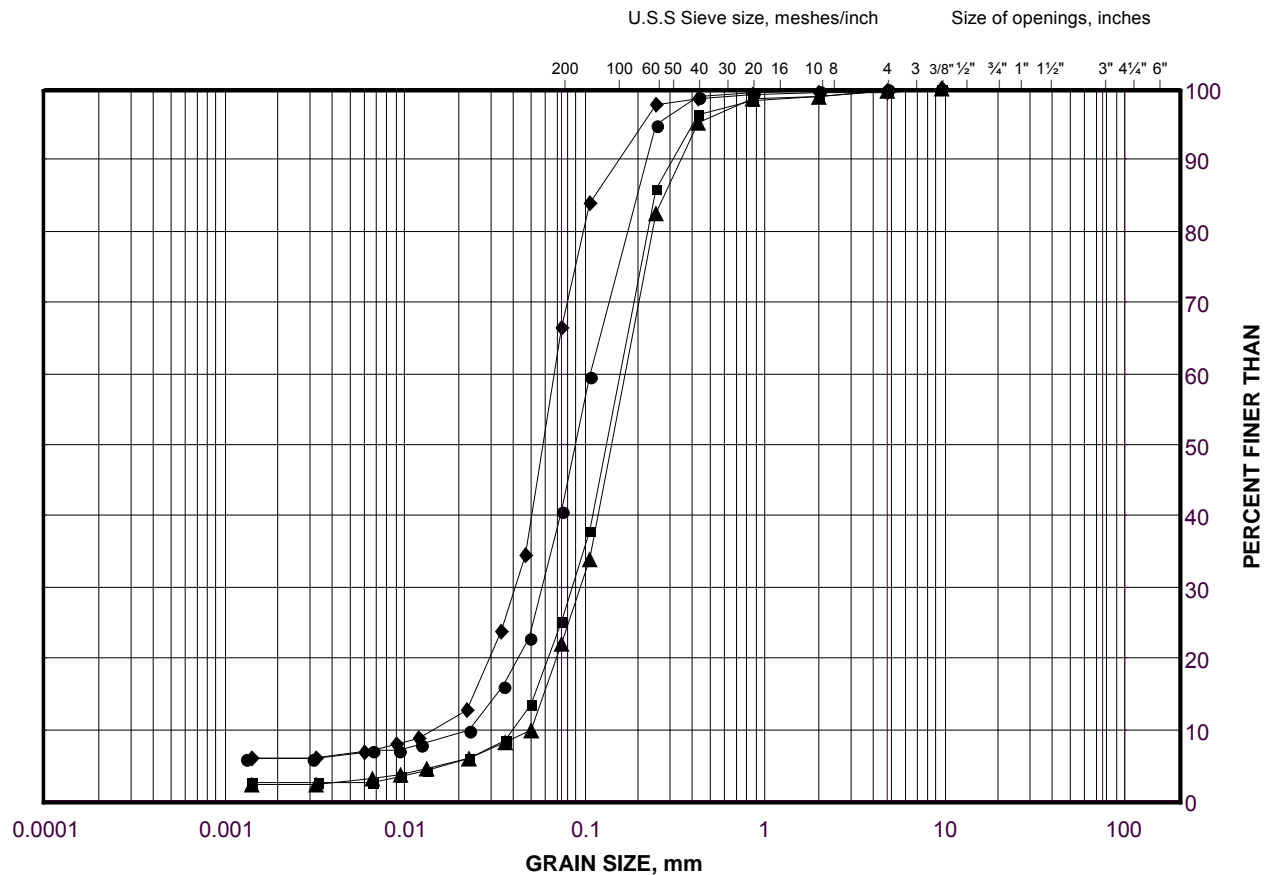
**Golder Associates**

Date: 27-Nov-13

# GRAIN SIZE DISTRIBUTION

Silt and Sand to Silty Sand

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)
●	B4-04	3	326.1
■	B4-01	4	325.1
◆	B4-04	7	323.1
▲	B4-01	7	322.8

Project Number: 11-1111-0149

Checked By: AV

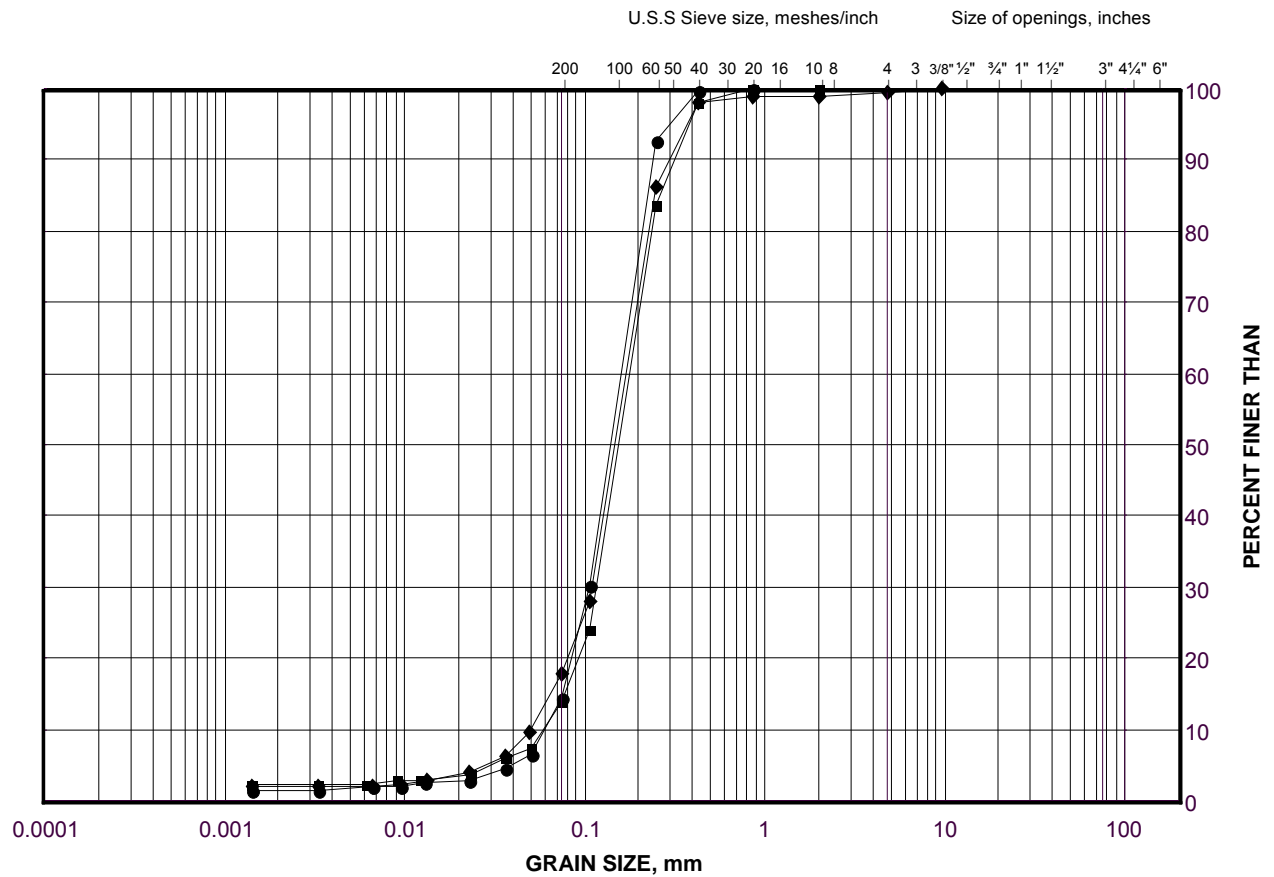
**Golder Associates**

Date: 27-Nov-13

# GRAIN SIZE DISTRIBUTION

Sand (Pockets)

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)
●	B4-02	14	312.5
■	B4-02	6	323.9
◆	B4-02	8	321.7

Project Number: 11-1111-0149

Checked By: AV

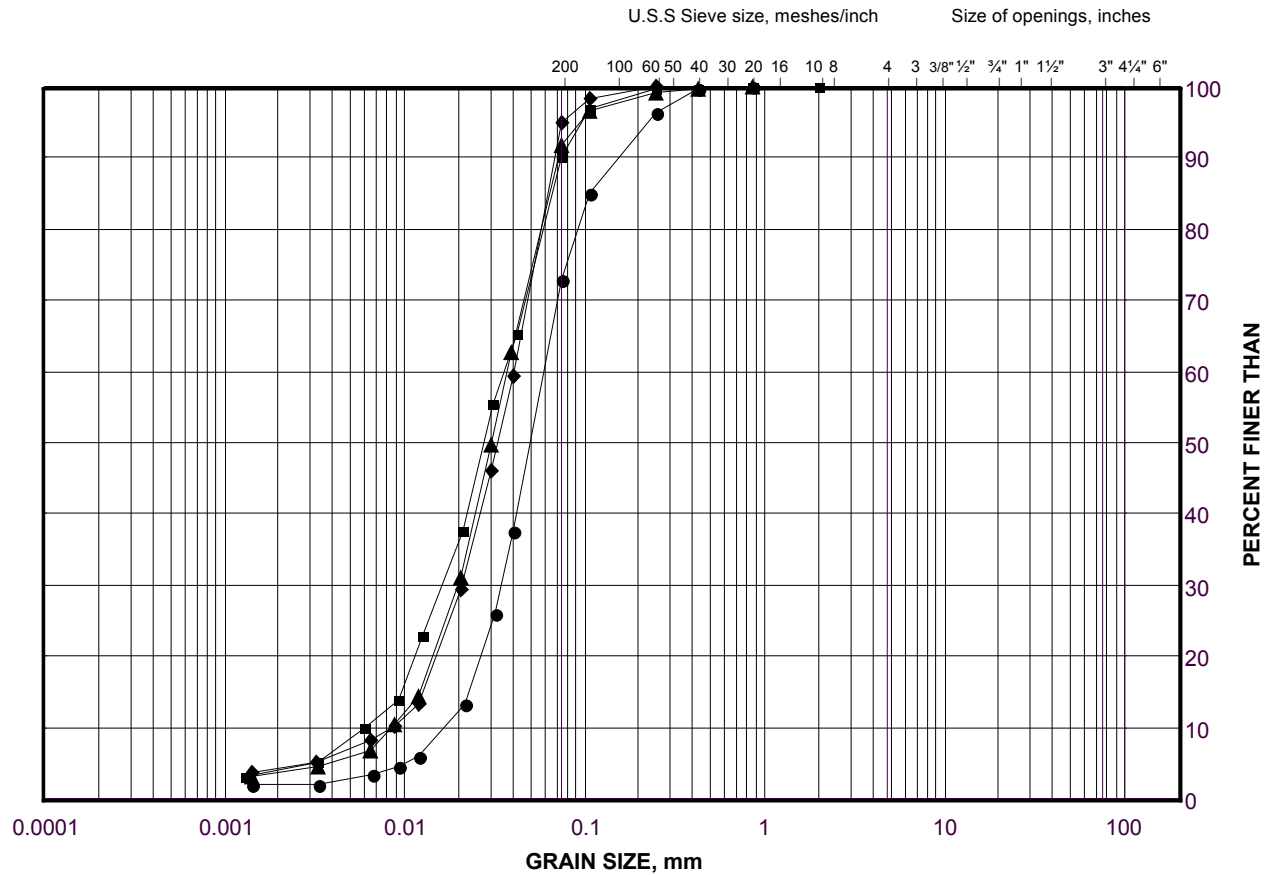
**Golder Associates**

Date: 28-Oct-13

# GRAIN SIZE DISTRIBUTION

Silt to Sandy Silt

FIGURE B1D



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

## LEGEND

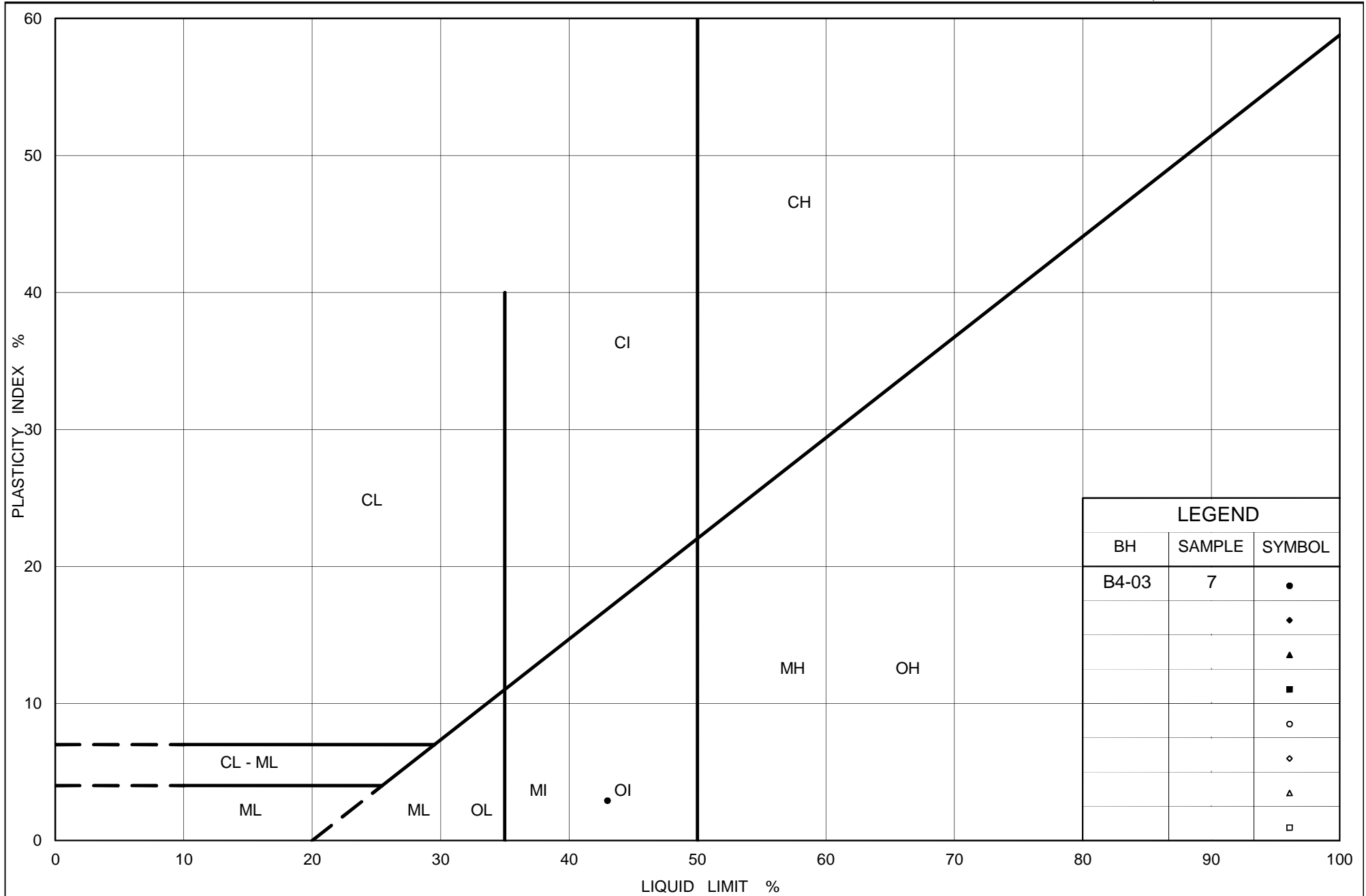
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	B4-02	11	317.1
■	B4-01	13	313.6
◆	B4-03	16	306.2
▲	B4-02	16	306.4

Project Number: 11-1111-0149

Checked By: AV

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Date: 28-Oct-13



Ministry of Transportation

Ontario

# PLASTICITY CHART Organic Silt and Sand

Figure No. B2

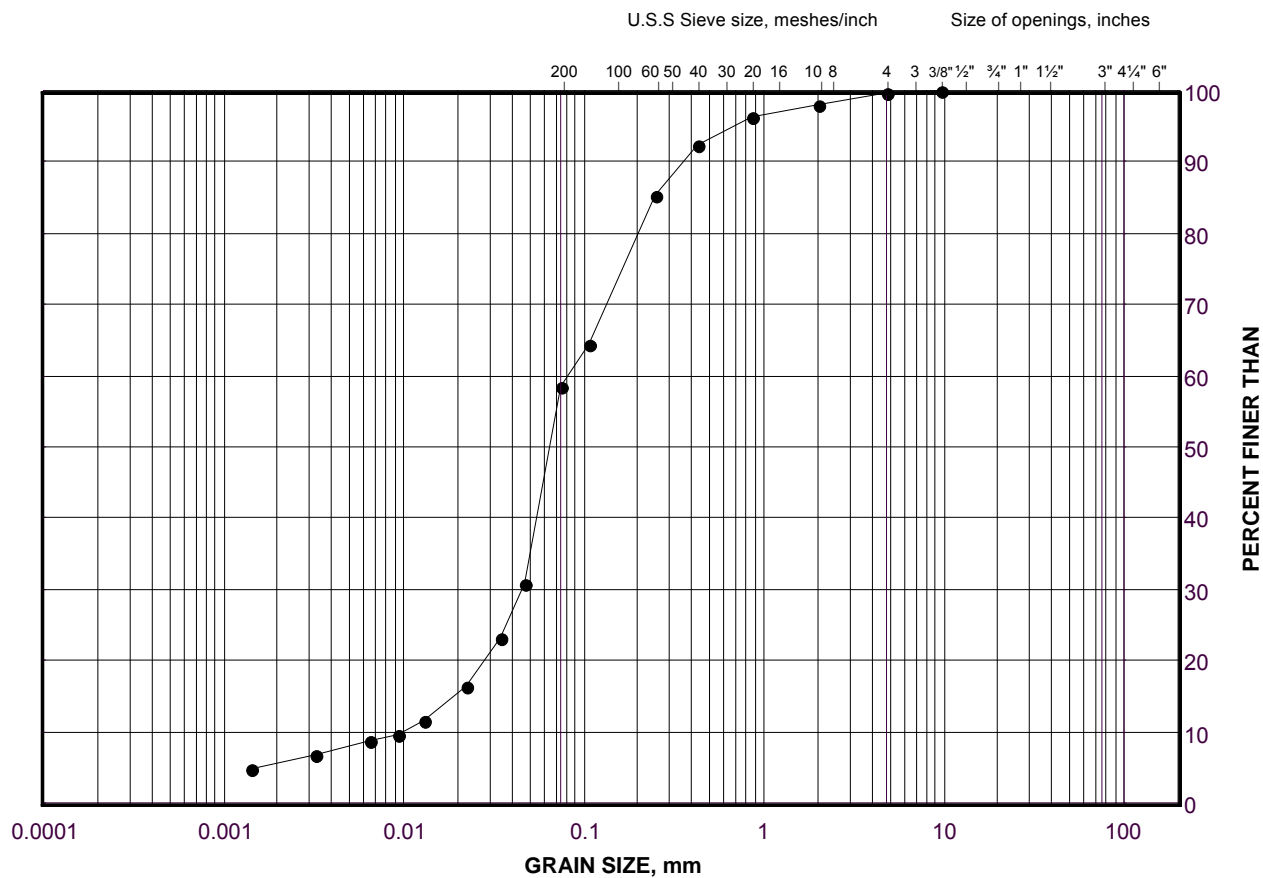
Project No. 11-1111-0149

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# GRAIN SIZE DISTRIBUTION

Organic Silt and Sand

FIGURE B3



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

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	B4-01	10	318.2

Project Number: 11-1111-0149

Checked By: AV

**Golder Associates**

Date: 28-Oct-13



# **APPENDIX C**

## **Non-Standard Special Provisions**



## **OBSTRUCTIONS**

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Special Provision

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### **SCOPE**

Cobbles were encountered within the sand and gravel deposit during advancement of the boreholes. Consideration of the presence of these obstructions must be made in the selection of appropriate equipment and procedures for the construction of deep foundations.

### **BASIS OF PAYMENT**

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION

At Golder Associates we strive to be the most respected global company providing consulting, design, and construction services in earth, environment, and related areas of energy. Employee owned since our formation in 1960, our focus, unique culture and operating environment offer opportunities and the freedom to excel, which attracts the leading specialists in our fields. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees who operate from offices located throughout Africa, Asia, Australasia, Europe, North America, and South America.

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