



December 23, 2013

FOUNDATION INVESTIGATION AND DESIGN REPORT

**LITTLE EAST RIVER BRIDGE NO. 1, SITE NO. 44-174
HIGHWAY 592 - REPLACEMENT OF SIX STRUCTURES
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5265-07-00 WP 5265-07-01**

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REPORT

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PART A

FOUNDATION INVESTIGATION REPORT

LITTLE EAST RIVER BRIDGE NO. 1 – SITE NO. 44-174

HIGHWAY 592 – REPLACEMENT OF SIX STRUCTURES

MINISTRY OF TRANSPORTATION, ONTARIO

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



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. 1 (Site No. 44-174) 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. 1 is located approximately 75 m south of Savage Settlement Road and approximately 1 km north of the Highway 11/Novar Road interchange in 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 September 2011. Golder's proposal (Scope of Work) for foundation engineering services associated with the Little East River Bridge No. 1 structure is contained in Section 6.8 of MH's Technical Proposal for this assignment. The work was carried out in accordance with Golder's Project Specific Supplementary Specialty Plan for foundation engineering services, dated March 21, 2012.

This report addresses the investigation carried out for the Little East River Bridge No. 1 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 and coring techniques, in situ testing and laboratory testing on selected soil samples. The borehole locations for this investigation were surveyed by Tulloch Geomatics Inc. (Tulloch), a professional surveying company retained by MH. The investigation area is shown in plan on Drawing 2.

2.0 SITE DESCRIPTION

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

In general, the topography along Highway 592 consists of rolling terrain, including lakes, low-lying swamps containing areas of standing water, sparsely to densely populated tree covered areas. Land use in some areas consists of residential/recreational communities. The existing bridge is a single-span rigid frame structure with a span length of 6.1 m. The bridge structure and associated approach embankments are situated on a relatively flat, sparsely treed area surrounded by residential/recreational properties to the north and south and with Little East River flowing easterly at this location. The existing ground surface within the limits of the proposed structure and approach embankments is at about Elevation 322.5 m, referenced to Geodetic datum. The existing Highway 592 south and north approach embankments along the centreline are at Elevations 322.6 m and 322.5 m, respectively.

3.0 INVESTIGATION PROCEDURES

3.1 Foundation Investigation

The field work for the proposed bridge structure was carried out between May 14 and June 6, 2013 during which time a total of five boreholes 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 Borehole B1-02 and from the ground surface adjacent to Borehole B1-03 to determine the depth to refusal at



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

Foundation Unit	Borehole
South Approach Embankment	B1-01
South Abutment	B1-02
North Abutment	B1-03 and B1-05
North Approach Embankment	B1-04

The results of the borehole investigation and dynamic cone penetration tests are presented on the Record of Borehole sheets in Appendix A. The boreholes 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 and 203 mm outer diameter (O.D.) continuous flight hollow-stem augers and 'NW' casing. Soil samples were obtained at intervals of depth of about 0.75 m and 3.0 m, using a 50 mm outer diameter (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). Cobbles and boulders were cored using an 'NQ' size rock core barrel. The boreholes and DCPTs were advanced to depths of up to about 23.5 m and 25.4 m below existing ground surface, respectively. The DCPTs were terminated on refusal to further dynamic cone penetration.

The groundwater conditions in the open boreholes were observed during and upon completion of drilling operations, and a standpipe piezometer was installed in Borehole B1-04 to permit monitoring of the water level at that location. The piezometers consist of 38 mm diameter PVC pipe, with a slotted screen surrounded with sand sealed at a select depth within the borehole. The borehole and annulus surrounding the piezometer pipe above the screen and sand pack were backfilled to the surface with bentonite pellets/grout. Piezometer installation details and water level readings are described on the Record of Borehole sheets in Appendix A. All open boreholes were backfilled with cement grout by tremie technique upon completion and the piezometer in Borehole B1-04 was also abandoned with cement grout by tremie technique on May 15, 2013 in accordance with Ontario Regulation 903, Wells (as amended).

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

The as-drilled borehole locations and ground surface elevations were surveyed by Tulloch. The locations given in the Record of Borehole 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	Eastings		
B1-01	5035109.1	324438.5	322.6 m	8.2 m
B1-02	5035127.1	324448.4	322.5 m	19.8 m / 25.4 m
B1-03	5035139.4	324447.6	322.5 m	6.1 m / 18.9 m
B1-04	5035156.5	324456.9	322.5 m	9.8 m
B1-05	5035139.6	324452.1	322.5 m	23.5 m

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

As delineated in *The Physiography of Southern Ontario*¹, 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)².

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 and cross-section 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 over a deposit of fill associated with the Highway 592 embankments. The fill is underlain by a near surface layer of organic sand in places, and by deposits of silt and sand to sand and/or clayey silt. These deposits are in turn underlain by a deposit of sand and gravel to sandy gravel.

A detailed description of the subsurface conditions encountered in the boreholes at the abutments and approach embankments is provided in the following sections.

¹ 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.

² 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

An approximately 25 mm to 90 mm thick layer of asphalt was encountered at the ground surface in all boreholes.

4.2.2 Fill

A fill deposit comprised of brown sand and gravel to gravelly sand to sand some gravel was encountered in all boreholes below the asphalt layer. The gravelly sand and sand some gravel portions of the fill contain trace to some silt, trace clay, organics. Pieces of wood were encountered within the fill deposit at the location of north abutment in Boreholes B1-03, B1-04 and B1-05, as shown on Figure B1 in Appendix B, and are inferred to be remnants of an existing corduroy roadbed. The top of the fill deposit is at between Elevations 322.5 m and 322.4 m and the thickness of the deposit ranges from 1.4 m to 3.7 m.

The SPT 'N'-values measured within the fill deposit range from 8 blows to 30 blows per 0.3 m of penetration, and 16 blows per 0.2 m of penetration, indicating a loose to compact relative density. In Boreholes B1-03 and B1-04, it is inferred that the split-spoon sampler was bouncing on wood pieces while obtaining the SPT 'N'-values for Samples 2 and 3, as such, the SPT 'N'-values are considered not representative of the fill material's relative density.

The natural water content measured on eight samples of the fill ranges from about 4 per cent to 27 per cent.

The organic content measured on two samples of fill is about 3 per cent and 7 per cent, with the greater organic content measured on a sample containing wood pieces.

The results of grain size distribution tests completed on four samples of the fill deposit are shown on Figure B2 in Appendix B.

4.2.3 Organic Sand

An approximately 0.8 m thick layer of dark brown organic sand, trace to some gravel, trace to some silt was encountered underlying the fill deposit in Borehole B1-05. The top of the deposit was encountered at Elevation 320.3 m

An SPT 'N'-value of 5 blows per 0.3 m of penetration was measured within this layer, indicating a loose relative density.

The natural water content measured on a sample of the organic sand layer is about 31 per cent.

The result of a grain size distribution test completed on a sample of the organic sand layer is shown on Figure B3 in Appendix B.

4.2.4 Silt and Sand to Sand

A deposit of non-cohesive soil comprised of brown to grey silt and sand to silty sand to sand trace to some silt was encountered underlying the fill deposit in Boreholes B1-01 and B1-02. The deposit contains trace to some clay and trace gravel as well as clayey silt seams within the upper 1.9 m portion of this deposit in Borehole B1-01. The top of the silt and sand to sand deposit is at Elevations 318.9 m and 319.5 m, and the



thickness of the deposit is 4.5 m and 2.6 m in Boreholes B1-01 and B1-02, respectively. Borehole B1-01 was terminated within this deposit at a depth of 8.2 m below ground surface (Elevation 314.4 m)

The SPT 'N'-values measured within this deposit range from 4 blows to 11 blows per 0.3 m of penetration, indicating a loose to compact relative density.

The natural water content measured on four samples of this deposit ranges from about 20 per cent and 24 per cent.

The results of grain size distribution tests completed on four samples of the deposit are shown on Figure B4 in Appendix B.

4.2.5 Clayey Silt

A deposit of grey clayey silt was encountered underlying the silt and sand deposit in Borehole B1-02, below the fill deposit in Borehole B1-03 and below the organic sand layer in Borehole B1-05. The deposit generally contains trace to some sand. The top of the clayey silt deposit was encountered between Elevations 319.5 m and 316.9 m and the thickness of deposit varies between 1.3 m and 2.6 m.

The SPT 'N'-values measured within this deposit range from about 3 blows to 7 blows per 0.3 m of penetration, suggesting a soft to firm consistency.

The natural water content measured on five samples of the deposit ranges from about 25 per cent to 36 per cent.

The results of grain size distribution tests completed on two samples of the clayey silt deposit are shown on Figure B5 in Appendix B.

Atterberg limits tests were carried out on three samples of the clayey silt deposit and measured liquid limits ranging from about 26 per cent to 31 per cent, plastic limits ranging from about 17 per cent to 21 per cent and plasticity indices ranging from about 9 per cent to 10 per cent. The results of Atterberg limits tests are shown on plasticity chart on Figure B6 in Appendix B and indicate that the material is classified as clayey silt of low plasticity.

4.2.6 Sand and Gravel to Sandy Gravel

A deposit of brown to grey sand and gravel to sandy gravel was encountered underlying the clayey silt deposit in Boreholes B1-02, B1-03 and B1-05 and below the organic sand layer in Borehole B1-04. The top of the deposit ranges from Elevations 319.5 m to 314.3 m and the thickness of the deposit ranges from 0.9 m to 19.2 m. Boreholes B1-02 to B1-05 were terminated within this deposit between Elevations 316.4 m and 299.0 m. The DCPTs advanced from the bottom of the sampled Borehole B1-02 (at a depth of 19.8 m below ground surface (Elevation 302.7 m)) and adjacent to Borehole B1-03 penetrated 5.6 m and 13.7 m into this deposit (based on the resistance to dynamic cone penetration) and is inferred to terminate within this deposit at a depth of 25.4 m and 18.9 m below ground surface (Elevations 297.1 m and 303.6 m), respectively.

The sand and gravel to sandy gravel deposit generally contains trace to some silt, trace clay and the upper 0.9 m portion of the deposit contains clayey silt seams in Borehole B1-05. Cobbles and/or boulders were encountered at varying depths throughout the deposit and were cored using an 'NQ' size rock core barrel as



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summarized below. In general, the sizes range from about 107 mm to 671 mm. The photographs of the recovered cobbles and boulders are shown on Figure B7 in Appendix B.

Foundation Element/ Approach Embankment	Borehole	Top Elevation of Cored Cobbles and/or Boulders	Thickness
South Abutment	B1-02	313.0 m	0.9 m
		311.8 m	1.5 m
		309.4 m	0.3 m
		307.7 m	0.4 m
		304.1 m	1.4 m
North Abutment	B1-05	317.3 m	0.7 m
		315.5 m	0.6 m
		311.2 m	0.9 m
		309.4 m	0.3 m
		306.3 m	0.6 m
North Approach Embankment	B1-04	304.7 m	0.5 m
		317.2 m	0.8 m
		315.5 m	0.3 m

The SPT 'N'-values measured within the sand and gravel to sandy gravel deposit typically range from about 10 blows to 46 blows per 0.3 m of penetration, indicating a compact to dense relative density. SPT 'N'-values of about 34 blows to 50 blows per 0.15 m of penetration were recorded prior to split-spoon sampler refusal on cobbles and boulders within this deposit. The DCPT advanced from the bottom of Borehole B1-02 extends to effective refusal at 30 blows per 0.13 m of penetration and noticeable bouncing of the drive hammer, while the DCPT advanced adjacent to Borehole B1-03 encountered effective refusal at greater than 137 blows per 0.3 m of penetration. The Total Core Recovery of the cored cobbles/boulders samples generally ranges between about 20 per cent and 100 per cent, except in few instances where percentage of recovery was not recorded.

The natural water content measured on ten samples of the deposit ranges from about 6 per cent to 14 per cent.

The results of grain size distribution tests completed on six samples of the sand and gravel to sandy gravel deposit are shown on Figure B8 in Appendix B.

4.3 Groundwater Conditions

In general, the soil samples taken in the boreholes were moist to wet. During the drilling operations, artesian groundwater conditions were noted in Boreholes B1-02 and B1-05 when advancing the casing between depths of 5.2 m and 22.3 m below ground surface (between Elevations 317.3 m and 300.2 m) and in Borehole B1-03 upon completion of the dynamic cone penetration test at a depth of 18.9 m below ground surface (Elevation 303.6 m). The series of groundwater levels recorded in the drill casing during and upon completion of drilling/penetration were measured at depths ranging from 0.3 m to 0.8 m above ground surface (Elevations 322.8 m to 323.3 m). The groundwater levels measured in the open boreholes upon completion of drilling range from about 1.0 m to 1.6 m below ground surface (Elevations 321.5 m and 321.0 m) and 0.8 m above ground surface (Elevation 323.3 m).



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A standpipe piezometer was installed in Borehole B1-04 to allow monitoring of the groundwater level at the site. The water level in the piezometer was monitored for five hours upon completion of the installation and then the piezometer was decommissioned. Details of the piezometer installation are shown on the Record of Borehole No. B1-04 in Appendix A, and the groundwater level measured in the piezometer is summarized below.

Borehole	Ground Surface Elevation	Depth to Water Level	Groundwater Elevation	Date of Measurement
B1-04	322.5 m	-0.1 m	322.6 m ¹	May 15, 2013

Notes:

1. Artesian Conditions

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 Ms. Veronica Ayetan, P.Eng., a geotechnical engineer 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. 1 – SITE NO. 44-174

HIGHWAY 592 – REPLACEMENT OF SIX STRUCTURES

MINISTRY OF TRANSPORTATION, ONTARIO

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



6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides engineering design recommendations for the proposed Little East River Bridge No. 1 on Highway 592 (Site No. 44-174). 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. 1 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. 1 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 323.1 m, which corresponds to a raise of the existing approach embankments of about 0.6 m.

6.2 Foundation Options

Given that loose granular deposits and/or soft to firm cohesive deposits are present areas of the abutment down to below the depth of frost penetration, the relatively shallow depth to the groundwater table and proximity to the adjacent river, a shallow foundation system is not recommended for the support of the abutments.

Given that: bedrock was not encountered to the depths drilled; cobbles and boulders and artesian condition were encountered within the sand and gravel to sandy gravel deposit; 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 loose to compact granular deposits.

The following sections provide recommendations for the alternative foundations 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 footings founded on native overburden are not recommended for support of the proposed bridge abutments given that the soils down to immediately below the depth of frost penetration are loose granular and soft to firm cohesive deposits. 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 spread footings founded on the native overburden (a deposit of loose silt and sand underlain by a deposit of soft to firm clayey silt at the south abutment and firm clayey silt deposit at the north abutment), the factored geotechnical axial resistance at Ultimate Limits States (ULS) and geotechnical reaction at Serviceability Limits States (SLS) for 25 mm of settlement are provided below.

Foundation Location	Factored Geotechnical Axial Resistance at ULS	Geotechnical Reaction at SLS for 25 mm of Settlement
South Abutment	135 kPa	50 kPa
North Abutment	85 kPa	50 kPa

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

The construction of the cast-in-place spread 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 spread footings, cofferdam construction and unwatering will be required to allow for construction of the footings in dry conditions.

6.3.2 Resistance to Lateral Loads

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

Interface Material(s)	Coefficient of Friction ($\tan \delta'$)
Concrete footing on loose silt and sand or soft to firm clayey 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 spread 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 loose/soft nature of the upper sand and silt/clayey silt deposits and the thick underlying granular deposit, friction piles consisting of steel H-piles driven into the compact to dense sand and gravel to sandy gravel deposit could be considered for the support of the proposed structure. However, cobbles and boulders were encountered within the granular 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 boulders and/or the potential for the steel H-piles refusing on the cobbles and/or boulders. 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 drive long H-piles to the required depth to achieve the desired axial capacities for design. Furthermore, piles cannot be battered for lateral resistance due to the proximity of the temporary shoring (cofferdam).

6.4.1 Geotechnical Axial Resistance and Reaction

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

Foundation Location	Elevation of Underside of Pile Cap ¹	Elevation of Underside of Tremie Plug ¹	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 ²
South and North Abutment	319.3 m	317.8 m	299.3 m	20 m	950 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 and boulders within the sand and gravel to sandy 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 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 Drawing SS 103-11 using an ultimate geotechnical resistance of 2,060 kN per pile, but must be driven below El. 299.3 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 and boulders within the sand and gravel to sandy 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 west 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 ¹	Elevation of Underside of Tremie Plug ¹	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 ²
South and North Abutment	319.3 m	317.8 m	299.3 m	20 m	1,400 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, presence of cobbles and boulders and artesian conditions) 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;
- 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; and,
- Higher geotechnical resistances can be achieved from micropiles when compared to the same length of driven H-pile and drilled steel casing.

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.

Foundation Location	Elevation of Underside of Pile Cap ¹	Elevation of Underside of Tremie Plug ¹	Micropile Tip Elevation	Diameter of Micropile	Length of Micropile from Underside of Pile Cap
South and North Abutment	319.3 m	317.8 m	308.8 m	273 mm	10.5 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 ¹
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.



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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:

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

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

and for cohesive soils:

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

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 (B1-02)	Loose Silt and Sand	317.8 m to 316.9 m	5,000 kPa/m	-
	Soft to Firm Clayey Silt	316.9 m to 314.3 m	-	25 kPa
	Sand and Gravel to Sandy Gravel	314.3 m to 297.1 m	20,000 kPa/m	-
North Abutment (B1-05)	Sand and Gravel to Sandy Gravel	317.8 m to 299.0 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.



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Foundation Location	Pile Type	Axial Load Applied at the Top of Pile/Casing	Factored Geotechnical Lateral Resistance at ULS ¹	Geotechnical Lateral Reaction at SLS for 10 mm of Deflection ¹
South Abutment	HP 310 x 110	950 kN	45 kN	25 kN
	610 mm diameter drilled steel casing	1,400 kN	50 kN	30 kN
North Abutment	HP 310 x 110	950 kN	60 kN	25 kN
	610 mm diameter drilled steel casing	1,400 kN	55 kN	35 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:

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

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

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



6.8 Seismic Considerations

6.8.1 Site Coefficient

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

6.8.2 Seismic Analysis Coefficient

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



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Fill Type	Soil Unit Weight	Coefficients of Static Lateral Earth Pressure	
		At-Rest, K_o	Active, K_a
Rock Fill	19 kN/m ³	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 ³	0.43	0.27
Granular 'B' Type II	21 kN/m ³	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. 1 structure will be at about Elevation 323.1 m, requiring placement of up to about 0.6 m of fill to raise the existing approach embankment grade.

Based on the investigated locations at this site, the south approach embankment is founded on a deposit of loose silt and sand to sand, underlain by a deposit of soft to firm clayey silt which in turn is underlain by a deposit of compact to dense sand and gravel to sandy gravel, while the north approach embankment is founded on a deposit of organic sand which is underlain by a deposit of compact to dense sand and gravel. Near the north abutment the existing approach embankment is founded on a deposit of organic sand which is underlain by a deposit of firm clayey silt, in turn underlain by a deposit of compact to dense sand and gravel. Where excavations are not required, it is understood that the organic sand deposit and the existing fill materials will remain in place.

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.

For the purpose of the stability analysis, the groundwater level was assumed to be at Elevation 321.5 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, γ	Undrained Shear Strength, s_u	Cohesion, c'	Effective Friction Angle, ϕ'
South Approach Embankment (2H:1V side slopes)	New Granular Fill	21 kN/m ³	-	0 kPa	34°
	Existing Loose to Compact Sand to Sand and Gravel Fill	20 kN/m ³	-	0 kPa	30°
	Loose to Compact Silt and Sand to Sand	18 kN/m ³	-	0 kPa	30°
	Soft to Firm Clayey Silt	17 kN/m ³	25 kPa	-	-
	Compact to Dense Sand and Gravel to Sandy Gravel	20 kN/m ³	-	0 kPa	34°



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Embankment	Soil Type	Unit Weight, γ	Undrained Shear Strength, s_u	Cohesion, c'	Effective Friction Angle, ϕ'
North Approach Embankment (2H:1V side slopes)	New Granular Fill	21 kN/m ³	-	0 kPa	34°
	Existing Loose to Compact Sand to Sand and Gravel Fill	20 kN/m ³	-	0 kPa	30°
	Loose to Compact Organic Sand	18 kN/m ³	-	0 kPa	27°
	Soft to Firm Clayey Silt	17 kN/m ³	25 kPa	-	-
	Compact to Dense Sand and Gravel to Sandy Gravel	20 kN/m ³	-	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 ¹	Side Slope Profile	Minimum Factor of Safety
South Approach Embankments	4.3 m	2H:1V	≥ 1.3
North Approach Embankment	2.0 m	2H:1V	≥ 1.3

Note:

- 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^{3D} (Version 2.0), developed by Rocscience Inc.

The sources of settlement are considered to include:

- Immediate settlement of the granular soils (short-term); and,
- Elastic compression of the cohesive soils (short-term).

The analyses were carried out at the critical sections of the approach embankment where the thickness of compressible foundation soils is up to about 25.4 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. Due to the relatively thin clayey silt deposit encountered overlying the sand and gravel to sandy gravel deposit, the compression of the cohesive deposit was modelled by estimating a coefficient of volume compressibility based on the SPT 'N'-values and engineering judgement. For the purpose of the settlement analyses, it is assumed that the settlement of the existing fill materials is negligible.

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

Embankment	Soil Type	Thickness	Unit Weight, γ	Elastic Modulus, E'	Coefficient of Volume Compressibility, m_v
South Approach Embankment	Existing Loose to Compact Sand to Sand and Gravel Fill	~1.5 m	20 kN/m ³	5 MPa	-
	Loose to Compact Silt and Sand to Sand	2.6 m to 4.5 m ¹	18 kN/m ³	5 MPa	-
	Soft to Firm Clayey Silt	~ 2.6 m	17 kN/m ³	-	2×10^{-4} kPa ⁻¹
	Compact to Dense Sand and Gravel to Sandy Gravel	~ 17.2 m ¹	20 kN/m ³	30 MPa	-
North Approach Embankment	Existing Loose to Compact Sand to Sand and Gravel Fill	~0.7 m	20 kN/m ³	5 MPa	-
	Loose to Compact Organic Sand	~ 1.6 m	18 kN/m ³	5 MPa	-
	Soft to Firm Clayey Silt	~ 1.5 m	17 kN/m ³	-	2×10^{-4} kPa ⁻¹
	Compact to Dense Sand and Gravel to Sandy Gravel	6.8 m to 19.2 m ¹	20 kN/m ³	30 MPa	-

Note:

1. Applicable borehole(s) (i.e. borehole(s) advanced in the vicinity of the respective approach embankment) terminated within the deposit.



For the purpose of settlement analyses, the groundwater level was assumed to be located on average at Elevation 321.5 m, based on several groundwater level measurements in the open boreholes upon completion of drilling.

6.10.2.3 Settlement of Foundation Soils

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

Embankment	Estimated Settlement of Foundation Soils
South Approach Embankment	25 mm to 35 mm
North Approach Embankment	15 mm to 20 mm

These settlements are expected to occur relatively quickly (i.e. within 30 days after construction) in response to the grade raise, based on the predominantly non-cohesive nature of the foundation soils. However, it should be noted that some long-term (i.e. creep) settlement of the organic sand deposit along the north approach embankment is expected and as such, future maintenance of the highway may be required.

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 319.3 m and the underside of the tremie plug at Elevation 317.8 m, excavations up to about 4.7 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 an inferred remnants of an existing corduroy roadbed were encountered along the north approach embankment underlying the granular fill. In addition, cobbles and boulders were encountered within the native sand and gravel to sandy gravel deposit during borehole advancement. The presence of such obstructions could affect the excavation works and/or installation of temporary shoring/cofferdam as well as 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.



Report Signature Page

Tomasz Zalucki, P.Eng.,
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Designated MTO Contact, Principal

TZ/CN/JMAC/sm

n:\active\2011\11-1111-0149 mh - highway 592 - huntsville\reporting\verb no. 1\final\11-1111-0149-1 rpt 13dec23 fidr little east river bridge no.1.docx



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- | | |
|------------|---|
| 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^{3D} (Version 2.0) by Rocscience Inc.
- Slide (Version 6.0) by Rocscience Inc.
- Contract Design Estimating and Documentation (CDED):
- | | |
|--------------------------|--|
| Special Provision 105S21 | Amendment to OPSS 501 – Water Requirements and Quality Control for Compaction – Method B |
| Special Provision 539S02 | Amendment to OPSS 539 – Protection System |
- Ministry of Transportation Ontario:



FOUNDATION REPORT - LITTLE EAST RIVER BRIDGE NO. 1 - HIGHWAY 592 GWP 5265-07-00 WP 5265-07-01

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

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

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

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

Ontario Occupational Health and Safety Act:

Ontario Regulation 213 Construction Projects (as amended)

Ontario Provisional Standard Drawing:

OPSD 3000.100	Foundation, Piles, Steel H-Pile Driving Shoe
OPSD 3090.010	Foundation, Frost Penetration Depths for Southern Ontario
OPSD 3101.200	Walls – Abutment, Backfill – Rock
OPSD 3121.150	Walls – Retaining, Backfill – Minimum Granular Requirement

Ontario Provincial Standard Specification:

OPSS 501	Construction Specification for Compacting
OPSS 539	Construction Specification for Temporary Protection Systems
OPSS 804	Construction Specification for Seed and Cover
OPSS 903	Construction Specification for Deep Foundations
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade, and Backfill Material

Ontario Water Resources Act:

Ontario Regulation 903 Wells (as amended)



TABLES



FOUNDATION REPORT - LITTLE EAST RIVER BRIDGE NO. 1 - HIGHWAY 592 GWP 5265-07-00 WP 5265-07-01

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 ¹	<ul style="list-style-type: none"> Relative ease of construction. 	<ul style="list-style-type: none"> Allows only for semi-integral abutment design. Axial capacity on the loose/soft native overburden will be low. Excavation for footings will be below water table. Cofferdam (with concrete tremie plug) and unwatering will be required for construction of the footings within a dry excavation. 	<ul style="list-style-type: none"> Lower relative cost than driven pile, drilled steel casing and micropile foundation options. Additional cost for cofferdam construction and unwatering for construction of the footings. 	<ul style="list-style-type: none"> Large footings will be required to develop adequate axial capacity.
Driven steel H-piles (HP 310x110)	2	<ul style="list-style-type: none"> Negligible post-construction settlement. Higher axial capacity than spread/strip footings. Straight forward construction; except that site constraints may preclude use of pile driving equipment. 	<ul style="list-style-type: none"> Integral abutment design may not be possible due to constraints in achieving free length of pile to allow for lateral movement. Given the thickness of the overburden, axial capacity will be developed through shaft resistance (i.e. friction piles) only. Reinforced pile tips and/or heavier pile section will be required for piles to penetrate through cobbles and boulders. Piles cannot be battered for lateral resistance due to the proximity of the sheetpile cofferdam. Excavation for pile cap will 	<ul style="list-style-type: none"> Higher relative cost than spread/strip footing foundation option. Higher cost associated with pile reinforcement and/or heavier pile section to advance the H-piles through cobbles and boulders. Additional cost for cofferdam construction and unwatering for construction of the pile cap. 	<ul style="list-style-type: none"> Potential for requirement to drive piles deeper to develop adequate axial capacity during construction. Potential difficulty driving piles through the cobbles and boulder present in the sand and gravel to sandy gravel deposit. Potential for pile damage when driving through cobbles and boulders. May require additional construction platform width and/or temporary closure of the roadway to accommodate larger (pile driving) equipment. Overhead hydro lines will



FOUNDATION REPORT - LITTLE EAST RIVER BRIDGE NO. 1 - HIGHWAY 592 GWP 5265-07-00 WP 5265-07-01

Table 1: Evaluation of Foundation Alternatives

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks / Consequences
			<p>be below water table.</p> <ul style="list-style-type: none"> ■ Cofferdam (with concrete tremie plug) and unwatering will be required for construction of the pile caps within a dry excavation. ■ Requires larger (pile driving) equipment as compared to micropile drilling equipment. ■ Piling operation along the west side of the bridge will be in close proximity to overhead hydro lines. 		<p>need to be de-energized during portions of the piling operation.</p>
Drilled steel casings using DTH hammer drilling system (610 mm)	3	<ul style="list-style-type: none"> ■ Reduced number of deep foundation elements compared to steel H-piles. ■ DTH drilling can readily penetrate through cobbles and boulders in overburden. ■ Relatively straightforward construction; except that site constraints may preclude the use of drilling equipment. ■ Negligible post-construction settlement. 	<ul style="list-style-type: none"> ■ Allows only for semi-integral abutment design. ■ Given the thickness of the overburden, axial capacity will be developed through shaft resistance (i.e. friction steel casing) only. ■ Drilling slurry will be required to balance groundwater pressures and minimize basal heave. ■ Excavation for pile cap will be below water table. ■ Cofferdam (with concrete tremie plug) and unwatering will be required for construction of the pile cap within a dry excavation. 	<ul style="list-style-type: none"> ■ Higher relative cost than spread/strip footing and driven pile foundation options. ■ Additional cost for specialized drilling equipment. ■ Additional cost associated with the need for drilling slurry and temporary liners. ■ Additional cost for cofferdam construction and unwatering for construction of the pile cap. 	<ul style="list-style-type: none"> ■ Potential for unbalanced head in liners during installation may result in base heave and possible loss of ground. ■ Specialized drilling equipment and/or method could be required to penetrate cobbles and boulders present in the sand and gravel to sandy gravel deposit. ■ May require additional construction platform width and/or temporary closure of the roadway to accommodate larger (drilling) equipment. ■ Overhead hydro lines will need to be de-energized



FOUNDATION REPORT - LITTLE EAST RIVER BRIDGE NO. 1 - HIGHWAY 592 GWP 5265-07-00 WP 5265-07-01

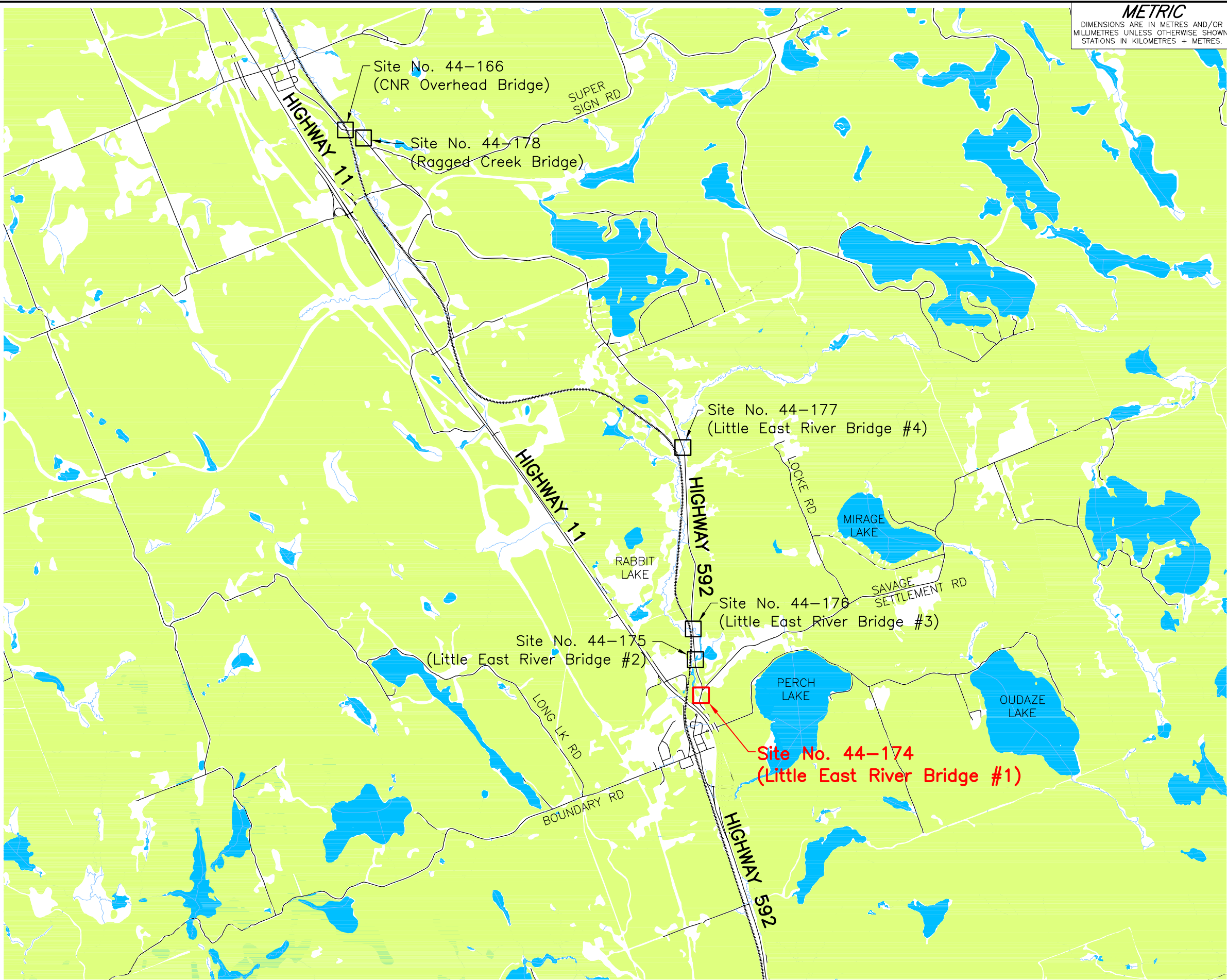
Table 1: Evaluation of Foundation Alternatives

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks / Consequences
			<ul style="list-style-type: none"> Requires larger (drilling) equipment as compared to micropile drilling equipment. Drilling operation along the west side of the bridge will be in close proximity to overhead hydro lines. 		during portions of the drilling operation.
Micropiles (273 mm diameter)	1	<ul style="list-style-type: none"> Negligible post-construction settlement. Potential for achieving high axial capacity in the non-cohesive overburden using pressure grouting techniques. Drilling equipment will readily penetrate cobbles and boulders in the granular deposits. Requires smaller drilling equipment as compared to steel casing drilling equipment. 	<ul style="list-style-type: none"> 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. 	<ul style="list-style-type: none"> 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. 	<ul style="list-style-type: none"> Few contractors have experience with soil-bonded micropile installation on MTO projects.


Note: 1. NR – Not Recommended



DRAWINGS

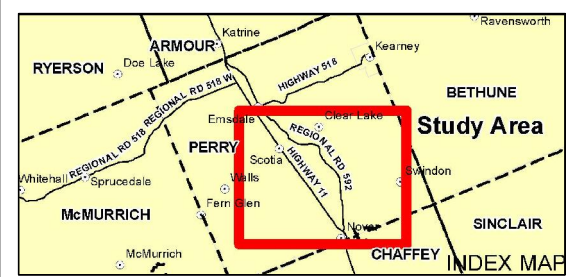


METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No. WP No. 5265-07-01		
HIGHWAY 592 REPLACEMENT OF SIX STRUCTURES KEY MAP		



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

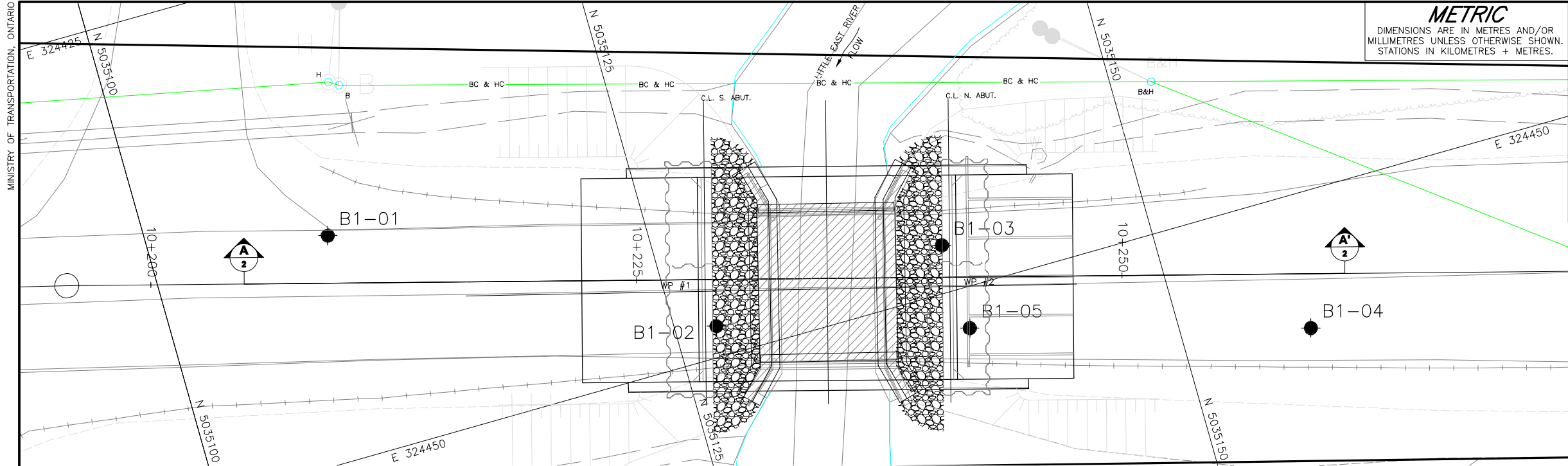


 N.T.S



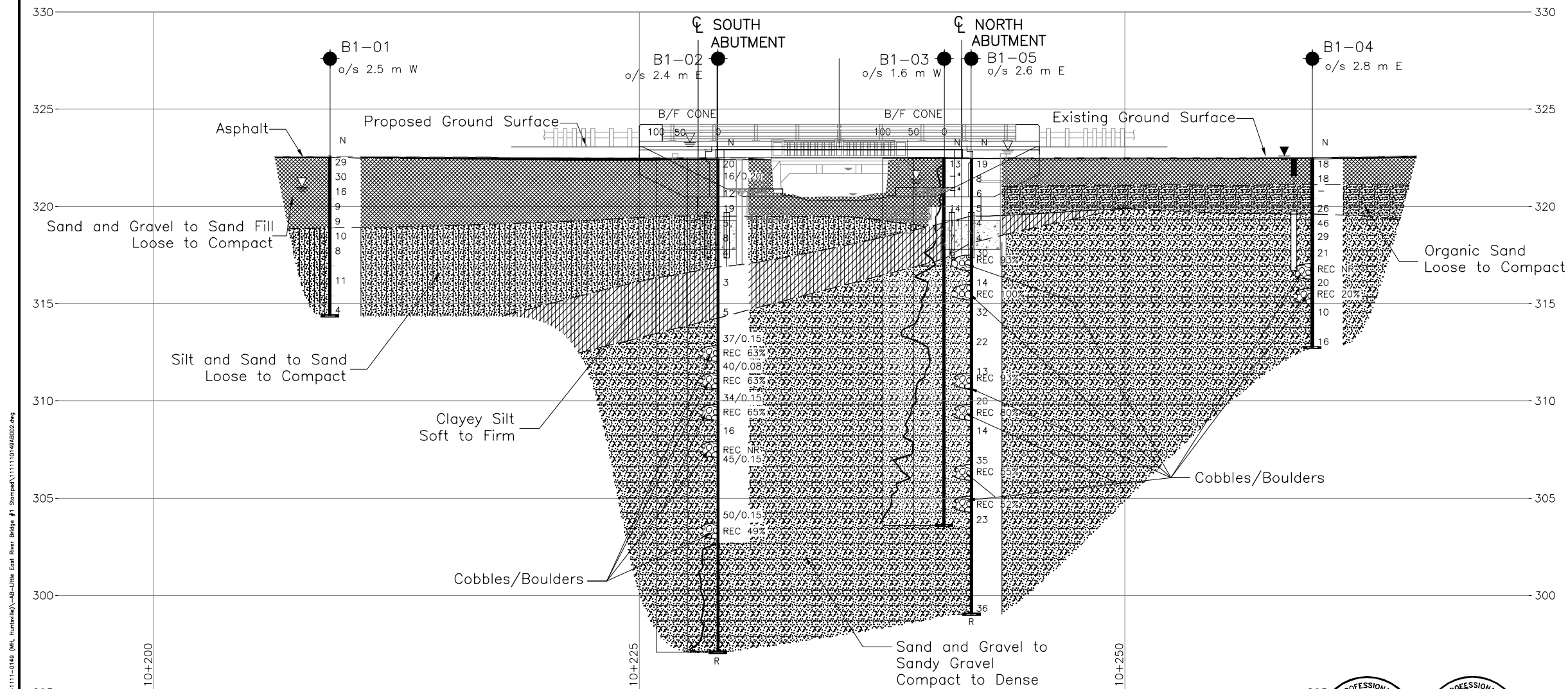
REFERENCE
Base data — MNR NRVIS, obtained 2004, CANMAP v2006.4 Produced by
Golder Associates Ltd. under licence from Ontario Ministry of Natural
Resources

NO.	DATE	BY	REVISION	
Geocres No. 31E-330				
HWY. 592		PROJECT NO. 11-1111-0149		DIST.
SUBM'D. AV		CHKD. CN	DATE: Dec. 2013	SITE:
DRAWN: JFC		CHKD.	APPD.	DWG. 1



PLAN

SCALE
2.5 0 2.5 5 m



CENTRELINE PROFILE

SCALE
2.5 0 2.5 5 m

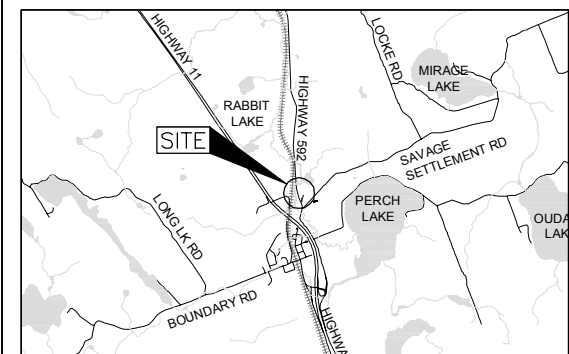
METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 5265-07-01

HIGHWAY 592
LITTLE EAST RIVER BRIDGE #1
BOREHOLE LOCATIONS AND SOIL STRATA



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN

SCALE
1.2 0 1.2 2.4 km

LEGEND

- Borehole - Current Investigation
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- REC Total core recovery
- WL in piezometer, measured on May 14, 2013
- WL upon completion of drilling
- R Refusal

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
B1-01	322.6	5035109.1	324438.5
B1-02	322.5	5035127.1	324448.4
B1-03	322.5	5035139.4	324447.6
B1-04	322.5	5035156.5	324456.9
B1-05	322.5	5035139.6	324452.1

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. 44174-01.dwg, received November 07, 2013.

NO.	DATE	BY	REVISION
Geocres No. 31E-330			
HWY. 592		PROJECT NO. 11-1111-0149	
SUBM'D. AV		CHKD. TVA	DATE: Dec. 2013
DRAWN: JFC		CHKD.	APPD. CN/JMAC
		DIST. SITE: 44-174	
		DWG. 2	





APPENDIX A

Record of Borehole Sheets



LIST OF SYMBOLS

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

I. GENERAL

π	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

γ	shear strain
Δ	change in, e.g. in stress: $\Delta \sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a)	Index Properties
$\rho(\gamma)$	bulk density (bulk unit weight)*
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	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_α	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
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio = σ'_p / σ'_{vo}

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	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 ρ . Unit weight symbol is γ 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 cm² 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 _p	plastic limit
w _l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D _R	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 ₄	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	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 B1-01		SHEET 1 OF 1		METRIC								
W.P.		5265-07-01		LOCATION		N 5035109.1 ; E 324438.5		ORIGINATED BY								
DIST		HWY 592		BOREHOLE TYPE		120 mm O.D. Hollow Stem Augers and NW Casing		COMPILED BY								
DATUM		Geodetic		DATE		May 21, 2013		CHECKED BY								
								TVA								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
322.6	GROUND SURFACE															
0.0	Asphalt (90 mm)		1	SS	29											
	Sand and gravel (FILL) Compact Brown Moist		2	SS	30											
321.2																
1.4	Sand, some gravel, trace silt, trace organics (FILL) Loose to compact Brown Wet		3	SS	16											
			4	SS	9											
			5	SS	9											
318.9																
3.7	SILT and SAND, trace gravel, trace clay, containing clayey silt seams Loose Brown Wet		6	SS	10											
			7	SS	8											
317.0																
5.6	SAND, trace gravel, trace to some silt Compact Brown Wet		8	SS	11											
315.4																
7.2	Silty SAND, trace to some clay Loose Grey Wet		9	SS	4											
314.4																
8.2	END OF BOREHOLE															
	NOTE: 1. Water level at a depth of 1.6 m below ground surface (Elev. 321.0 m) upon completion of drilling.															

PROJECT 11-1111-0149		RECORD OF BOREHOLE No B1-02		SHEET 1 OF 2	METRIC
W.P. 5265-07-01		LOCATION N 5035127.1 ; E 324448.4		ORIGINATED BY ID	
DIST HWY 592		BOREHOLE TYPE 203 mm O.D. Hollow Stem Augers and NW Casing		COMPILED BY AV	
DATUM Geodetic		DATE May 15, 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/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL	
								20	40	60	80	100	W _p	W	W _L						
322.5	GROUND SURFACE																				
0.0	Asphalt (90 mm)		1	SS	20																
	Sand and gravel (FILL)		2	SS	16/0.20																
	Compact																				
	Brown		3	SS	12																
	Moist to wet		4	SS	19																
319.5																					
3.0	SILT and SAND, trace to some clay		5	SS	5																
	Loose		6	SS	8																
	Brown becoming grey below a depth of 3.7 m		7	SS	4																
	Wet																				
316.9																					
5.6	CLAYEY SILT, trace sand		8	SS	3																
	Soft to firm																				
	Grey		9	SS	5																
	Wet																				
314.3																					
8.2	SAND and GRAVEL, trace to some silt																				
	Dense		10	SS	37/0.15																
	Grey		3	RC	REC 63%																
	Wet																				
	Artesian condition encountered when advanced casing to a depth of 9.1 m, water level recorded at 0.3 m above ground surface.																				
	Cobbles encountered between depths of 9.5 m and 10.4 m.																				
	Cobbles and boulders encountered between depths of 10.7 m and 12.2 m.		11	SS	40/0.08																
			4	RC	REC 63%																
			12	SS	34/0.15																
	Artesian condition encountered when advanced casing to a depth of 12.8 m, water level recorded at 0.5 m above ground surface.		5	RC	REC 65%																
	Cobbles encountered between depths of 13.1 m and 13.4 m.																				
308.8																					
13.7	Sandy GRAVEL, trace silt		13	SS	16																
	Compact to dense																				
	Grey		6	RC	NR																
	Wet																				

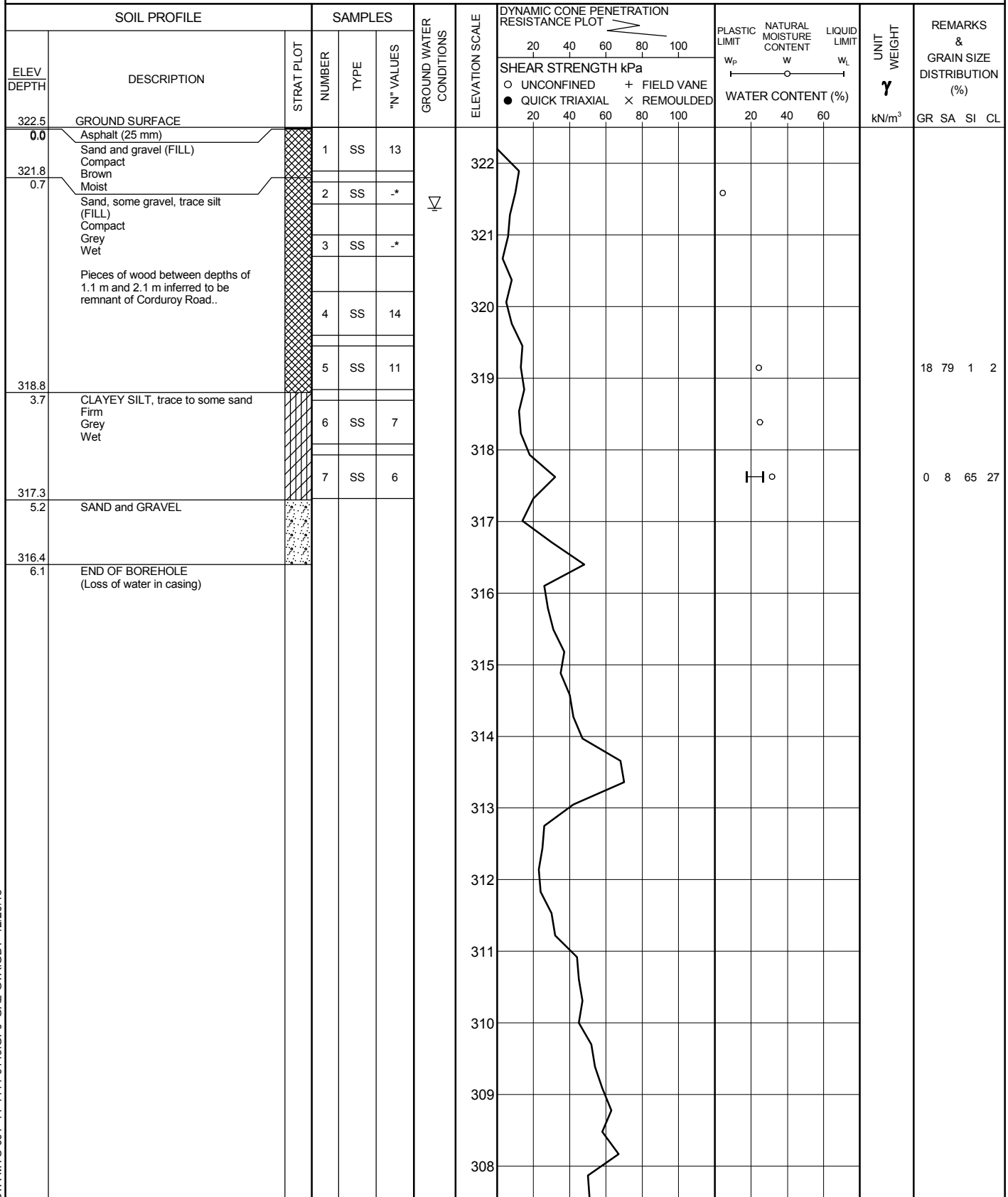
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+ ³, × ³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

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

PROJECT 11-1111-0149				RECORD OF BOREHOLE No B1-02				SHEET 2 OF 2				METRIC					
W.P. 5265-07-01				LOCATION N 5035127.1 ; E 324448.4				ORIGINATED BY ID									
DIST HWY 592				BOREHOLE TYPE 203 mm O.D. Hollow Stem Augers and NW Casing				COMPILED BY AV									
DATUM Geodetic				DATE May 15, 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/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
	--- CONTINUED FROM PREVIOUS PAGE ---						20	40	60	80	100	20	40	60			
							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W _p — W — W _L					
							20 40 60 80 100					20 40 60					
302.7	Artesian condition encountered when advanced casing to a depth of 14.3 m, water level recorded at 0.6 m above ground surface.	STRAT PLOT	6	RC	NR												
	Cobbles encountered between depths of 14.8 m and 15.2 m.		14	SS	45/0.15												
	Artesian condition encountered when advanced casing to a depth of 15.8 m, water level recorded at 0.8 m above ground surface.																
	Cobbles encountered between depths of 18.4 m and 19.8 m.		15	SS	50/0.15												
			7	RC	REC 49%												
19.8	END OF BOREHOLE																
	Dynamic Cone Penetration Test (DCPT)																
297.1	END OF DCPT																
25.4	Refusal to Further Penetration (30 Blows / 0.13 m)																
	NOTES:																
	1. Artesian conditions encountered during drilling:																
	Date Depth (m) W.L. Water/Casing Elev. (m)																
	05/15/13 -0.3/9.1 322.8																
	05/16/13 -0.5/12.8 323.0																
	05/16/13 -0.6/14.3 323.1																
	05/16/13 -0.8/15.8 323.3																
	NR - Not Recorded																

PROJECT 11-1111-0149		RECORD OF BOREHOLE No B1-03		SHEET 1 OF 2		METRIC	
W.P. 5265-07-01		LOCATION N 5035139.4 ; E 324447.6		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 June 3, 2013		CHECKED BY TVA			



Continued Next Page

+ 3, X 3: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

PROJECT <u>11-1111-0149</u>		RECORD OF BOREHOLE No B1-03		SHEET 2 OF 2		METRIC	
W.P. <u>5265-07-01</u>		LOCATION <u>N 5035139.4 ; E 324447.6</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>June 3, 2013</u>		CHECKED BY <u>TVA</u>			

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20 40 60 80 100	20 40 60 80 100	W _p W W _L	20 40 60		
	--- CONTINUED FROM PREVIOUS PAGE ---												
303.6	END OF BOREHOLE (Loss of water in casing)												
18.9	END OF DCPT Refusal to Further Penetration (137 Blows / 0.3 m) NOTES: 1. Borehole terminated upon loss of water head in drill string. 2. Artesian condition encountered upon completion of DCPT and removal of the penetration cone, water flowing from ground surface. 3. Water level in open borehole measured at a depth of 1.1 m below ground surface (Elev. 321.4 m) upon completion of drilling. 4. A dynamic cone penetration test was advanced 1.5m north of Borehole B1-03 to confirm refusal. * Split-spoon sampler bouncing on wood pieces; N-value not representative of soil relative density.												

PROJECT		11-1111-0149		RECORD OF BOREHOLE No B1-04				SHEET 1 OF 1		METRIC						
W.P.		5265-07-01		LOCATION		N 5035156.5 ; E 324456.9		ORIGINATED BY		ID						
DIST		HWY 592		BOREHOLE TYPE		203 mm O.D. Hollow Stem Augers and NW Casing		COMPILED BY		AV						
DATUM		Geodetic		DATE		May 14 and 15, 2013		CHECKED BY		TVA						
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
322.5	GROUND SURFACE						20	40	60	80	100					
0.8	Asphalt (40 mm) Sand and gravel (FILL) Compact Brown Moist		1	SS	18											
			2	SS	18											
321.1																
1.4	Sand, some silt, some gravel, trace clay (FILL) Compact Dark brown Wet		3A	SS	-*								o		OC = 6.7 %	12 68 16 4
			3B													
	Pieces of wood at a depth of 1.7 m inferred to be remnant of Corduroy Road.		4	SS	26								o		OC = 2.8 %	
319.5																
3.0	SAND and GRAVEL, trace silt, trace clay Compact to dense Brown Wet		5	SS	46								o			
			6	SS	29											
			7	SS	21								o			43 51 (6)
	Cobbles encountered between depths of 5.3 m and 6.1 m.		1	RC	REC NR											
			8	SS	20											
			2	RC	REC 20%											
	Cobbles encountered between depths of 7.0 m and 7.3 m.		9	SS	10											
			10	SS	16								o			42 53 4 1
312.8																
9.8	END OF BOREHOLE															
NOTES: 1. Water level in open borehole at a depth of 1.0 m below ground surface (Elev. 321.5 m) upon completion of drilling. 2. Water level measurements in Piezometer: Date Depth (m) Elev. (m) 05/15/13 -0.1** 322.6 3. Water level in piezometer monitored for 5 hours with pumping test carried out to confirm the water level; piezometer decommissioned on May 15, 2013 upon completion of monitoring. NR - Not Recorded * Split-spoon sampler bouncing on wood pieces; N-value is not representative of soil relative density.																

PROJECT 11-1111-0149			RECORD OF BOREHOLE No B1-05			SHEET 2 OF 2			METRIC																															
W.P. 5265-07-01			LOCATION N 5035139.6 ; E 324452.1			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 June 4 to 6, 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																																			
--- CONTINUED FROM PREVIOUS PAGE ---																																								
	SAND and GRAVEL, trace to some silt, containing clayey silt seams to a depth of 5.2 m Compact to dense Brown Wet		14	SS	35																																			
	Artesian condition noted at a depth of 15.2 m, water flowing from around the casing.		5	RC	REC 55%																																			
	Cobbles encountered between depths of 16.2 m and 16.8 m.																																							
	Cobbles encountered between depths of 17.8 m and 18.3 m.		6	RC	REC 52%																																			
			15	SS	23																																			
299.0	Artesian condition encountered at a depth of 22.3 m, water level recorded at 0.4 m above ground surface.		16	SS	36																																			
23.5	END OF BOREHOLE																																							
NOTES: 1. Water level in open borehole at a depth of 1.1 m below ground surface (Elev. 321.4 m) during drilling, measured on June 4, 2013. 2. Artesian conditions encountered during drilling: <table border="1" style="width:100%; border-collapse: collapse;"> <thead> <tr> <th>Date</th> <th>Depth (m)</th> <th>W.L. Water/Casing Elev. (m)</th> </tr> </thead> <tbody> <tr> <td>06/05/13</td> <td>-*/5.2</td> <td>-*</td> </tr> <tr> <td>06/05/13</td> <td>-0.4/9.8</td> <td>322.9</td> </tr> <tr> <td>06/06/13</td> <td>-*/15.2</td> <td>-*</td> </tr> <tr> <td>06/06/13</td> <td>-0.4/22.3</td> <td>322.9</td> </tr> </tbody> </table> * Water flowing around casing. 3. water level in open borehole at a depth of 0.3 m above ground surface (Elev. 322.8 m) upon completion of drilling.																										Date	Depth (m)	W.L. Water/Casing Elev. (m)	06/05/13	-*/5.2	-*	06/05/13	-0.4/9.8	322.9	06/06/13	-*/15.2	-*	06/06/13	-0.4/22.3	322.9
Date	Depth (m)	W.L. Water/Casing Elev. (m)																																						
06/05/13	-*/5.2	-*																																						
06/05/13	-0.4/9.8	322.9																																						
06/06/13	-*/15.2	-*																																						
06/06/13	-0.4/22.3	322.9																																						

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APPENDIX B

Laboratory Test Results and Bedrock Core Photographs


Boreholes B1-03, B1-04 and B1-05



Borehole B1-03 Sample 3

Borehole B1-05 Sample 3

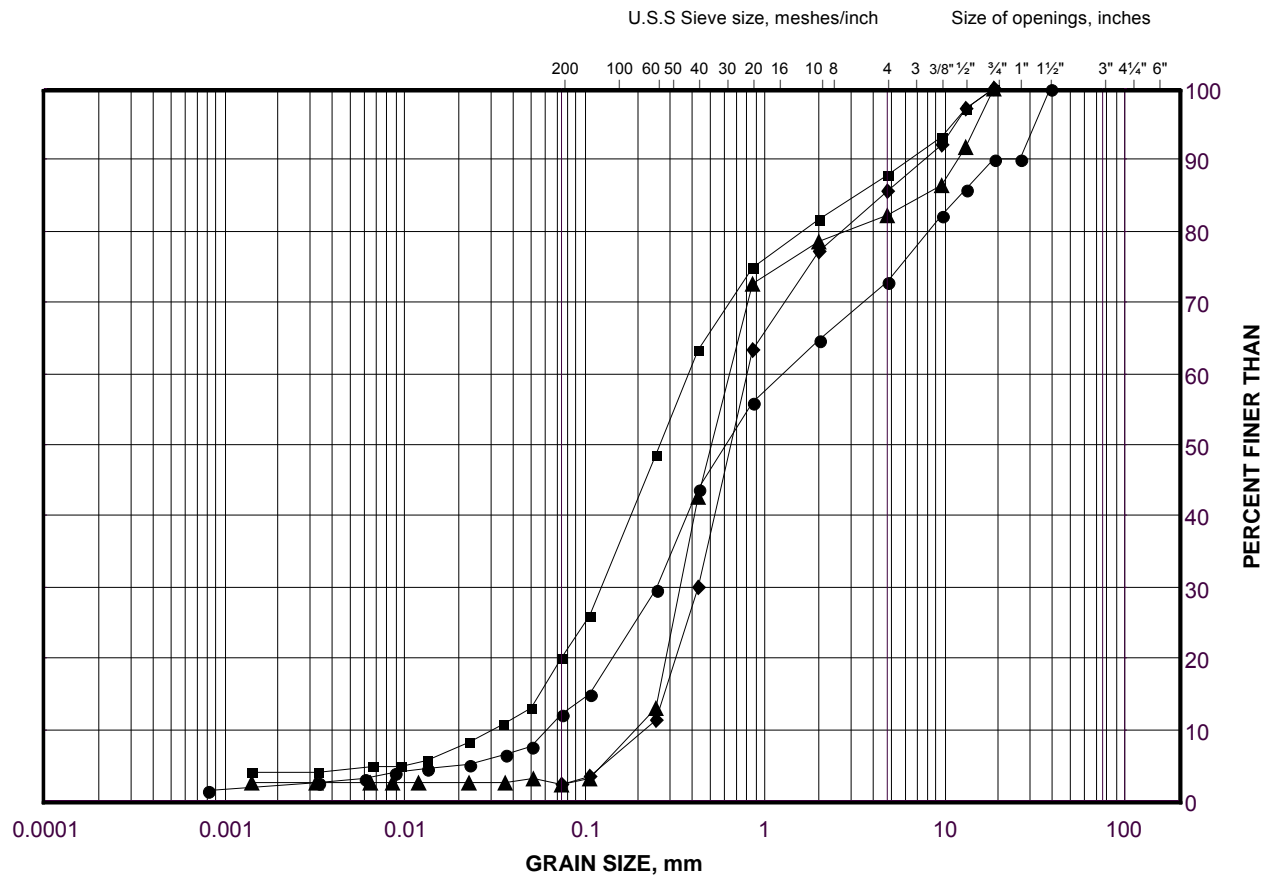
Borehole B1-04 Sample 3

PROJECT		Little East River Bridge No.1 Highway 592 GWP 5265-07-00; WP 5265-07-01			
TITLE		Wood Pieces Photograph – B1-02, B1-04 and B1-05 Highway 592			
		PROJECT No. 11-1111-0149		FILE No. ----	
		DESIGN	AV	AUG 13	SCALE NTS
		CADD	--		REV.
		CHECK	TZ	AUG 13	FIGURE B1
		REVIEW	JMAC	AUG 13	

GRAIN SIZE DISTRIBUTION

Sand to Gravelly Sand (Fill)

FIGURE B2



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	B1-05	2	321.4
■	B1-04	3A	320.8
◆	B1-01	4	320.0
▲	B1-03	5	319.1

Project Number: 11-1111-0149

Checked By: AV

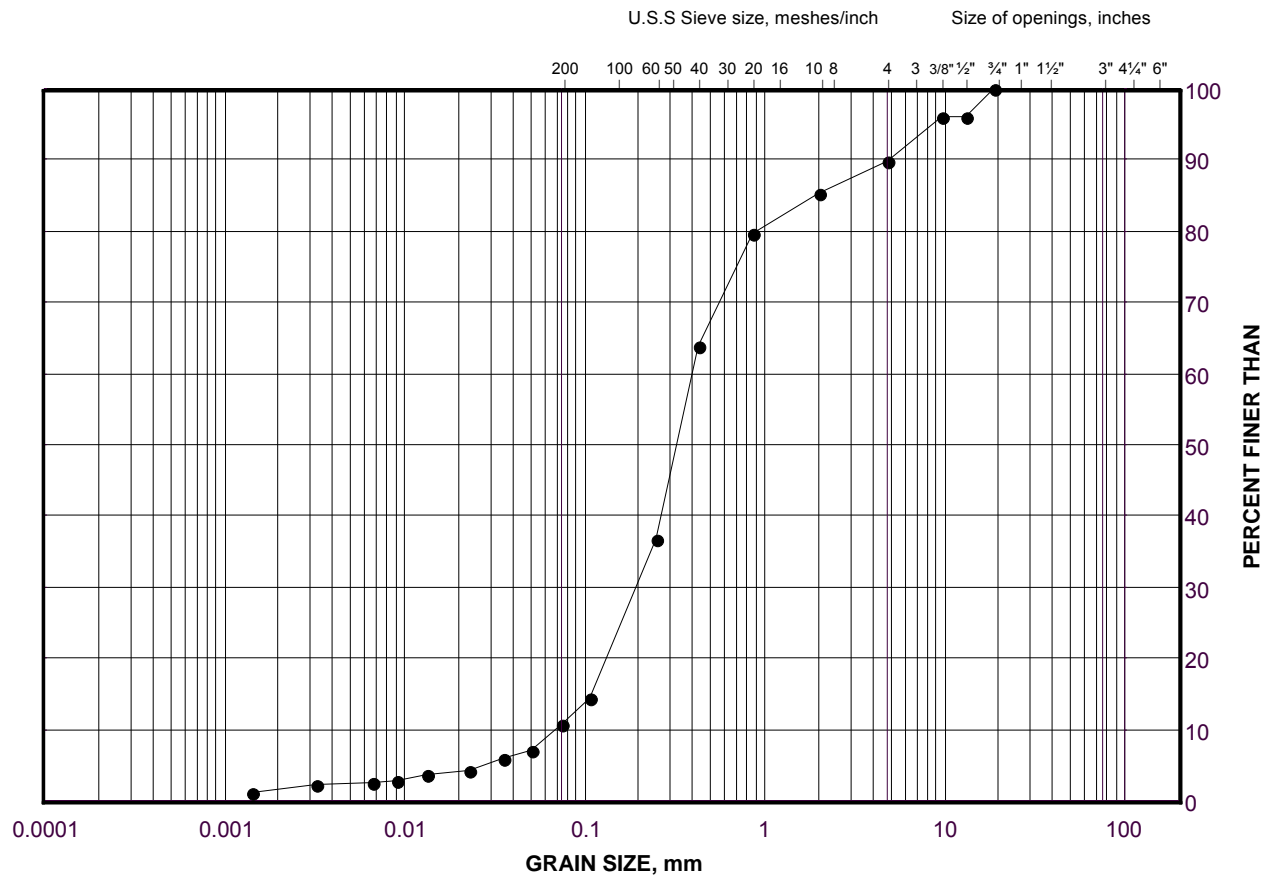
Golder Associates

Date: 21-Oct-13

GRAIN SIZE DISTRIBUTION

Organic 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)
•	B1-05	4	319.9

Project Number: 11-1111-0149

Checked By: AV

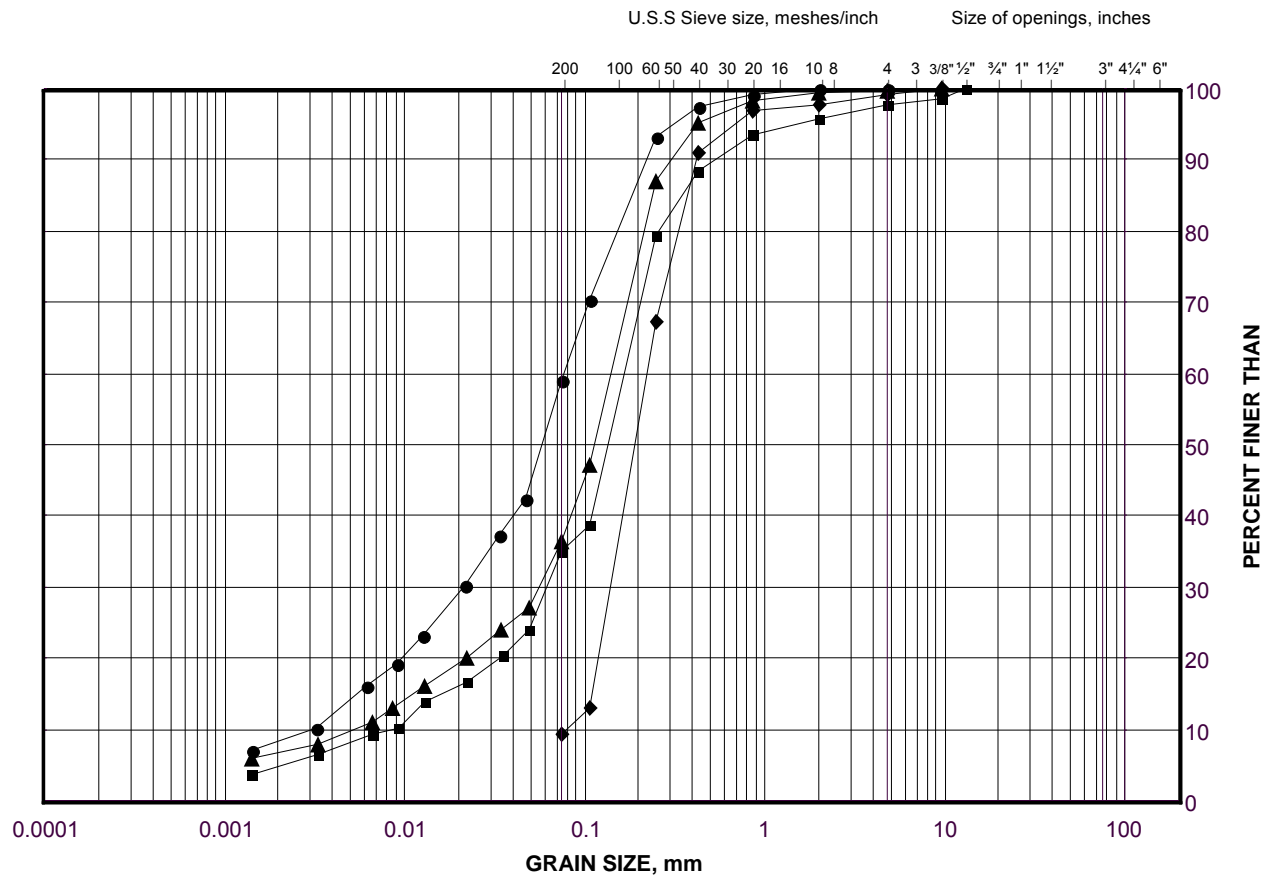
Golder Associates

Date: 21-Oct-13

GRAIN SIZE DISTRIBUTION

Silt and Sand to Sand

FIGURE B4



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	B1-02	6	318.4
■	B1-01	7	317.7
◆	B1-01	8	316.2
▲	B1-01	9	314.7

Project Number: 11-1111-0149

Checked By: AV

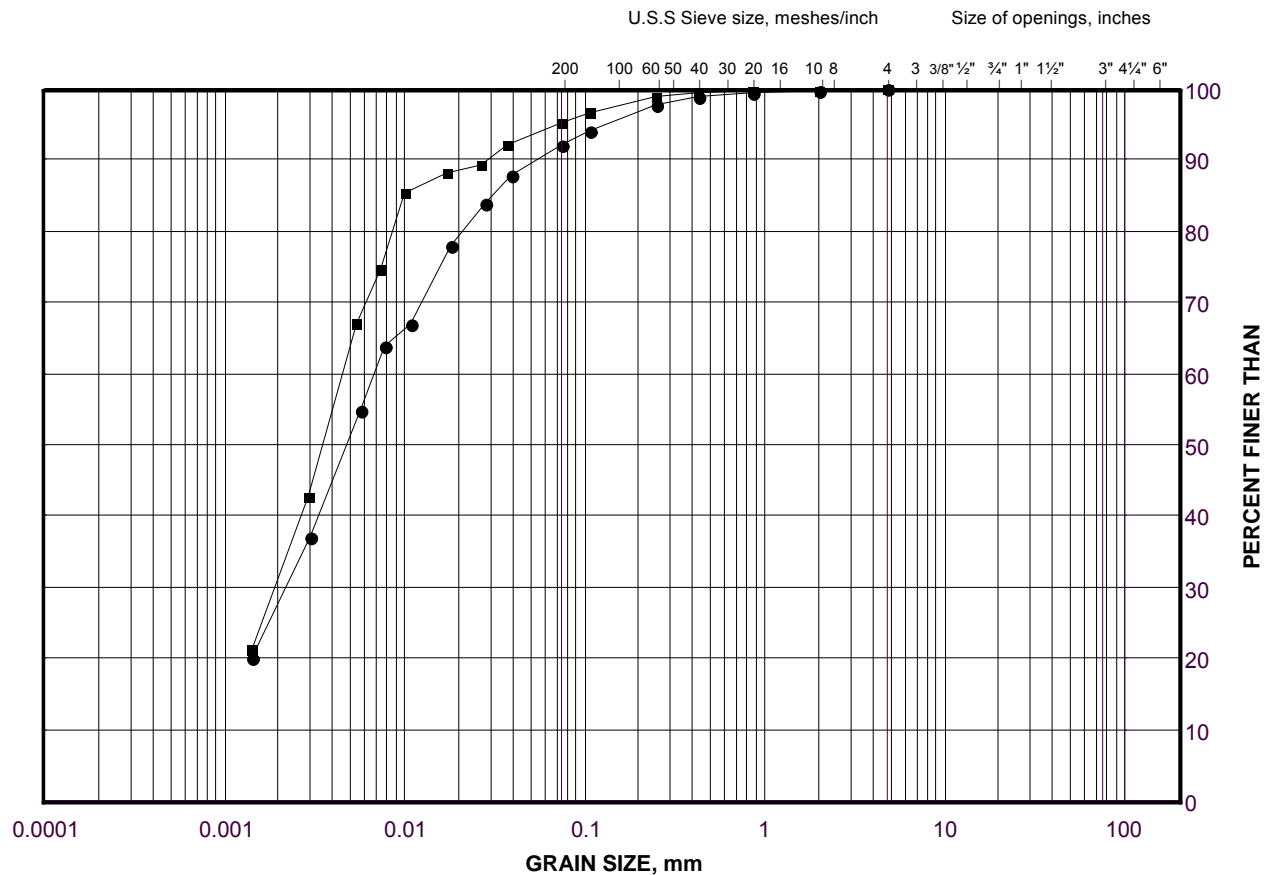
Golder Associates

Date: 07-Aug-13

GRAIN SIZE DISTRIBUTION

Clayey Silt

FIGURE B5



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

LEGEND

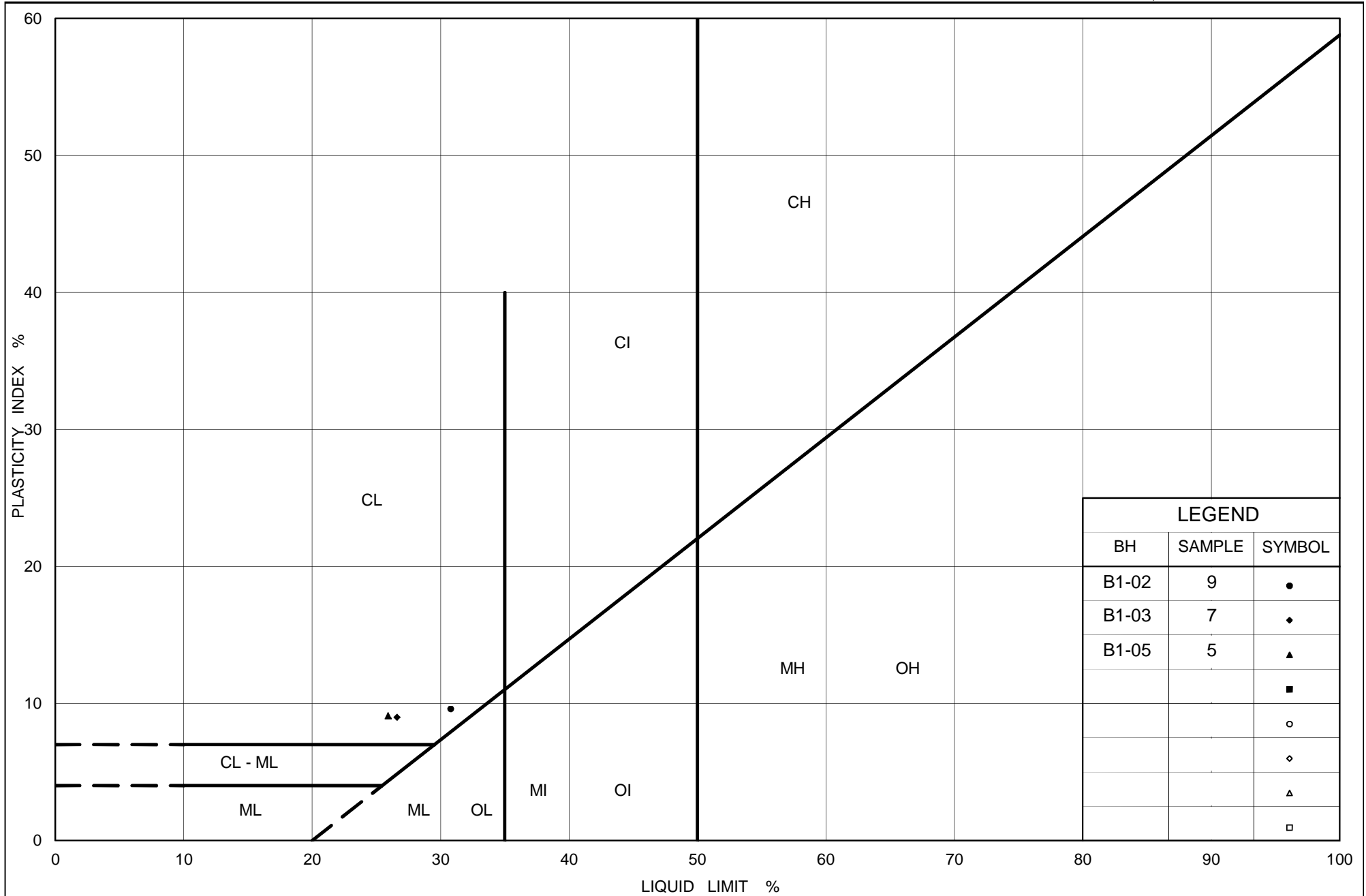
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	B1-03	7	317.6
■	B1-02	9	314.6

Project Number: 11-1111-0149

Checked By: AV

Golder Associates

Date: 07-Aug-13



Ministry of Transportation

Ontario

PLASTICITY CHART Clayey Silt

Figure No. B6

Project No. 11-1111-0149


Checked By: AV

Boreholes B1-02 and B1-05



0 m	0.25 m	0.5 m	0.75 m	1.0 m	1.25 m	1.5 m
0 ft	1 ft	2 ft	3 ft	4 ft	5 ft	

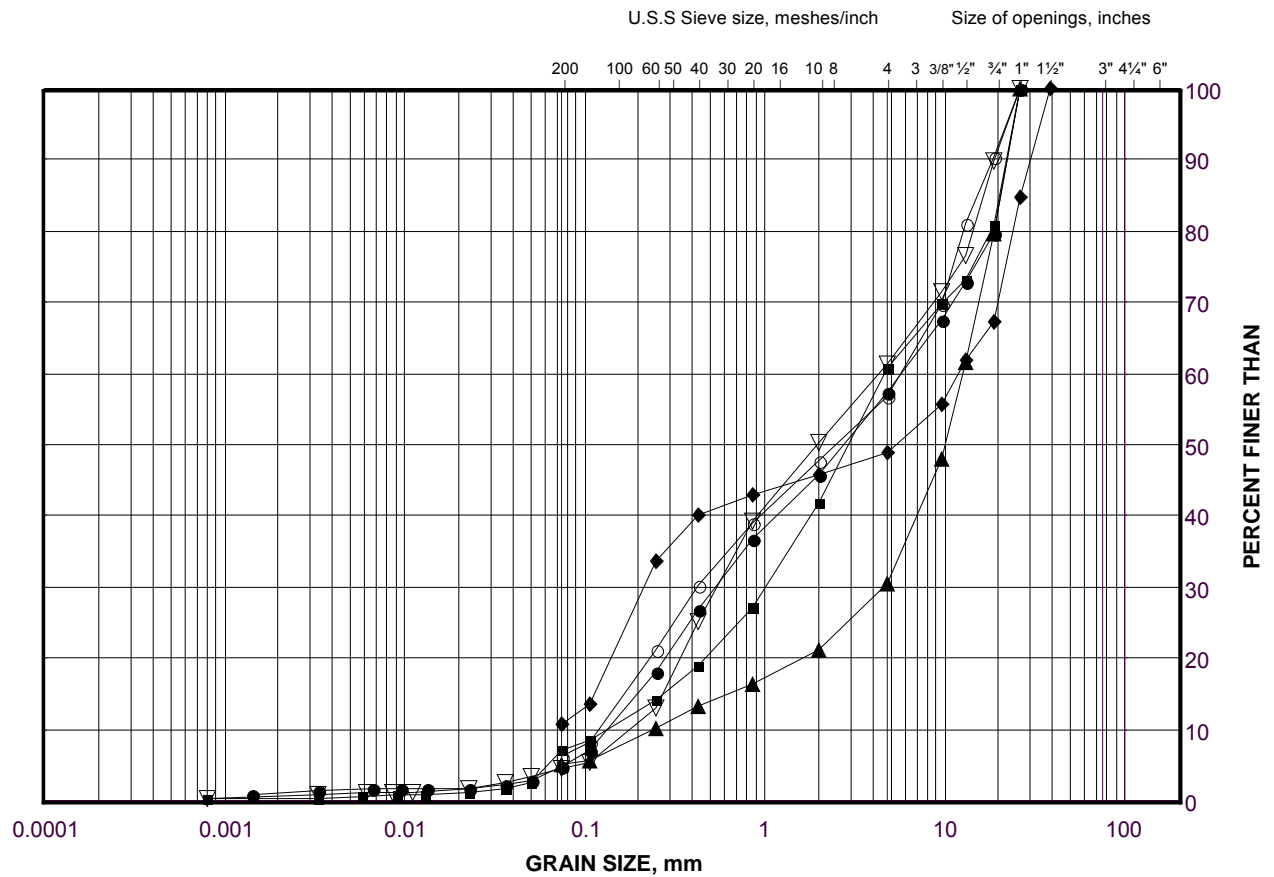
Scale

PROJECT		Little East River Bridge No.1 Highway 592 GWP 5265-07-00; WP 5265-07-01			
TITLE		Cobbles and Boulders Photograph – B1-02 and B1-05 Highway 592			
		PROJECT No. 11-1111-0149		FILE No. ----	
		DESIGN	AV	AUG 13	SCALE NTS
		CADD	--	--	REV.
		CHECK	TVA	AUG 13	FIGURE B7
		REVIEW	JMAC	AUG 13	

GRAIN SIZE DISTRIBUTION

Sand and Gravel to Sandy Gravel

FIGURE B8



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	B1-04	10	313.1
■	B1-05	12	310.0
◆	B1-02	12	310.2
▲	B1-02	14	307.1
▽	B1-05	15	303.9
○	B1-04	7	317.6

Project Number: 11-1111-0149

Checked By: AV

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Date: 07-Aug-13



APPENDIX C

Non-Standard Special Provisions

OBSTRUCTIONS

Special Provision

SCOPE

Inferred remnants of an existing corduroy roadbed were encountered along the north approach embankment underlying the granular fill. In addition, cobbles and boulders were encountered within the sand and gravel to sandy 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 excavation works, installation of temporary shoring/cofferdams as well as 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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