December 23, 2013

FOUNDATION INVESTIGATION AND DESIGN REPORT

LITTLE EAST RIVER BRIDGE NO. 3 - SITE NO. 44-176
HIGHWAY 592 - REPLACEMENT OF SIX STRUCTURES
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5265-07-00 WP 5267-07-01

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REPORT

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FOUNDATION INVESTIGATION REPORT

LITTLE EAST RIVER BRIDGE NO. 3 – SITE NO. 44-176

HIGHWAY 592 – REPLACEMENT OF SIX STRUCTURES

MINISTRY OF TRANSPORTATION, ONTARIO

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Morrison Hershfield Limited (MH) on behalf of the Ministry of Transportation, Ontario (MTO) to provide detail foundation engineering services for the replacement of Little East River Bridge No. 3 (Site No. 44-176) 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. 3 is located approximately 775 m north of Savage Settlement Road and approximately 2 km north of 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. 3 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. 3 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 low-lying areas to the north and south with Little East River flowing easterly at this location. The existing ground surface within the limits of the proposed structure and approach embankments is between Elevation 326.2 m and 326.0 m, referenced to Geodetic datum. The existing Highway 592 south and north approach embankments along the centerline are at Elevations 326.1 m and 326.2 m, respectively.

3.0 INVESTIGATION PROCEDURES

3.1 Foundation Investigation

The field work for the proposed bridge structure was carried out between April 29 and May 7, 2013 during which time a total of four boreholes (designated as Boreholes B3-01 to B3-04) were advanced at the location of the structure foundation footprints and approach embankments. In addition, a Dynamic Cone Penetration Test (DCPT B3-DC02) was advanced immediately adjacent to Borehole B3-02 and subsequently augered to a specified depth to install a piezometer. DCPTs were also carried out from the bottom of Boreholes B3-02 to B3-04 to determine the depth to refusal at these locations. A summary of the respective boreholes/DCPTs advanced at each foundation element and approach embankment is presented below.

Foundation Unit	Borehole/DCPT
South Approach Embankment	B3-01
South Abutment	B3-02 and B3-DC02
North Abutment	B3-03
North Approach Embankment	B3-04

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

The field borehole investigation was carried out using a truck-mounted CME 55 drill rig supplied and operated by Landcore Drilling of Chelmsford, Ontario. The boreholes were advanced through the overburden using 120 mm outer diameter (O.D.) continuous flight hollow-stem augers and 'NW' casing. Soil samples were obtained at intervals of depth of 0.75 m, 1.5 m and 3.0 m, using a 50 mm O.D. split-spoon sampler driven by an automatic hammer in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586 – Standard Test Method for Standard Penetration Test). Where encountered, cobbles and boulders were cored using an 'NQ' size rock core barrel. The boreholes and DCPTs were advanced to depths of up to about 31.1 m and 32.1 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 upon completion of drilling operations and a standpipe piezometer was installed in a borehole immediately adjacent to Borehole B3-02 to permit monitoring of the water level at this location. The piezometer consists of 38 mm diameter PVC pipe, with a slotted screen surrounded with sand. The annulus surrounding the piezometer pipe above the screen and sand pack was backfilled with bentonite pellets/grout. Piezometer installation details and water level readings are described on the Record of Borehole sheets in Appendix A. All open boreholes were backfilled with cement grout by tremie technique upon completion and the piezometer in the borehole immediately adjacent to Borehole B3-02 was also abandoned with cement grout by tremie technique on June 26, 2013 in accordance with Ontario Regulation 903, Wells (as amended).

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

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

Borehole	Location (MTM NAD 83)		Ground Surface Elevation	Borehole / DCPT Depth
	Northing	Easting		
B3-01	5035947.3	324343.2	326.0 m	9.8 m
B3-02	5035967.5	324345.3	326.2 m	31.1 m / 32.1 m
B3-DC02	5035965.7	324345.7	326.2 m	19.3 m
B3-03	5035978.4	324340.7	326.1 m	31.1 m / 31.5 m
B3-04	5035999.2	324343.1	326.0 m	9.8 m / 29.3 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 Appendix B, respectively. The results of the in situ field tests (i.e. SPT ‘N’-values) as presented on the Record of Borehole sheets and in Section 4.2 are uncorrected. The stratigraphic boundaries shown on the Record of Borehole sheets and on the profile on Drawing 2 are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Test (SPTs). These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Further, subsurface conditions will vary between and beyond the borehole locations. It should be noted that the interpreted stratigraphy shown on Drawing 2 is a simplification of the subsurface conditions.

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

[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.

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

4.2.1 Asphalt

A 25 mm to 50 mm thick layer of asphalt was encountered at the ground surface in all boreholes. Asphalt fragments were encountered below the asphalt in a 0.5 m thick layer in Borehole B3-02.

4.2.2 Sand and Gravel to Silt and Sand Fill

A non-cohesive deposit of fill was encountered below the asphalt layer in all boreholes. The fill deposit is comprised of various layers of dark brown to dark grey sand and gravel trace silt, to sand trace to some gravel, to gravelly silt and sand trace clay, to silt and sand trace gravel and trace clay. The sand and gravel to gravelly silt and sand portions of the fill contain asphalt fragments, trace organics, rootlets, wood fragments and clayey silt pockets. The top of the fill deposit ranges from Elevations 326.1 m to 325.7 m and the thickness of the deposit ranges from 1.7 m to 5.6 m.

The SPT 'N'-values measured within the non-cohesive fill deposit range from 2 blows to 56 blows per 0.3 m of penetration, indicating a very loose to very dense relative density. Typically, the higher SPT 'N'-values were recorded within the upper portion of the fill immediately underlying the asphalt layer.

The natural water content measured on ten samples of the fill ranges from about 2 per cent to 31 per cent.

The results of the grain size distribution tests completed on two samples of the gravelly sand to sand and gravel and one sample of the silt and sand portions of the fill deposit are shown on Figure B1 and Figure B2, respectively in Appendix B.

4.2.3 Organic Sand to Silty Peat

An organics deposit comprised of organic sand trace to some silt to silty peat some sand was encountered underlying the fill deposit in all boreholes. The deposit generally contains trace gravel, rootlets, roots and wood fragments. The top of this deposit ranges from Elevations 324.0 m to 320.4 m and the thickness of this deposit ranges from 1.6 m to 3.4 m.

The SPT 'N'-values measured within this deposit range from 3 blows to 6 blows per 0.3 m of penetration, indicating a very loose to loose relative density.

The natural water content measured on seven samples of the organic sand to silty peat deposit typically ranges from about 38 per cent to 76 per cent, but is up to about 338 per cent. The organic content measured on one sample each of the organic sand and the silty peat is about 13 per cent and 37 per cent, respectively.

The result of a grain size distribution test completed on one sample of the organic sand is presented on Figure B3 in Appendix B. An Atterberg limits test carried out on one sample of the organic sand indicates the fine material to be non-plastic.

4.2.4 Silt

A deposit of grey silt, trace to some clay, trace sand, was encountered underlying the organic sand to silty peat deposit in all the boreholes. The top of the silt deposit ranges from Elevations 320.6 m to 318.8 m and the thickness of the silt deposit ranges from 2.6 m to 6.9 m. In Borehole B3-02, the silt deposit is underlain by a 1.6 m thick pocket of gravelly sand trace to some silt, trace to some clay, at a depth of 11.7 m below ground surface, corresponding to Elevation 314.5 m. Boreholes B3-01 and B3-04 were terminated within this deposit at a depth of 9.8 m below ground surface (Elevations 316.3 m and 316.2 m, respectively).

The SPT ‘N’-values measured within the silt deposit range from 0 blows(weight of hammer) to 6 blows per 0.3 m of penetration, indicating a very loose to loose relative density. A SPT ‘N’-value of 5 blows per 0.3 m of penetration was measured within the gravelly sand pocket.

The natural water content measured on six samples of the silt deposit ranges from 29 per cent to 36 per cent, and the natural water measured on one sample of the gravelly sand pocket is about 15 per cent.

The results of the grain size distribution test completed on five samples of the silt deposit and one sample of the gravelly sand pocket are presented on Figures B4 and B5, respectively in Appendix B. Atterberg limits tests carried out on three samples of the silt deposit indicate the material to be non-plastic.

4.2.5 Clayey Silt with Sand

A cohesive deposit of grey clayey silt with sand containing trace to some gravel was encountered below the silt deposit in Boreholes B3-02 and B3-03. The top of the clayey silt with sand deposit is at Elevations 312.9 m and 313.6 m and the thickness of the deposit is3.8 m and 2.3 m in Boreholes B3-02 and B3-03, respectively. This deposit is also inferred to be present underlying the silt deposit in Borehole B3-04 based on the DCPT advanced from the bottom of the borehole.

The SPT ‘N’-values measured within this deposit range from 0 blows (weight of hammer) to 8 blows per 0.3 m of penetration, suggesting a very soft to firm consistency.

The natural water content measured on three samples of the clayey silt with sand deposit ranges between about 16 per cent and 22 per cent.

The results of the grain size distribution test completed on two samples of tthe clayey silt with sand deposit are presented on Figure B6 in Appendix B. Atterberg limits tests were carried out on two samples of the clayey silt with sand deposit and measured liquid limits of about 18 per cent and 21 per cent, plastic limits of about 13 per cent and 15 per cent and corresponding plasticity indices of about 5 per cent and 6 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure B7 in Appendix B and indicate the material to be clayey silt of low plasticity.

4.2.6 Gravelly Sandy Silt to Sand and Gravel

A deposit of non-cohesive soil comprised of grey gravelly sandy silt to sand and gravel was encountered underlying the clayey silt with sand deposit in Boreholes B3-02 and B3-03. The top of this deposit is at Elevations 309.1 m and 311.3 m and the thickness of the deposit is 14.0 m and 16.3 m in Boreholes B3-02 and B3-03, respectively. The DCPTs advanced from the bottom of the sampled Boreholes B3-02 to B3-04 are inferred to terminate within this deposit at depths between 29.3 m and 32.1 m (Elevations 296.7 m to 294.1 m).

The DCPT advanced adjacent to Borehole B3-02 (i.e. DCPT B3-DC02) encountered refusal to further penetration at a depth of 19.3 m below ground surface, corresponding to Elevation 306.7 m.

Cobbles and/or boulders were encountered at varying depths throughout the deposits and were cored using an 'NQ' size rock core barrel as summarized below.

Foundation Element	Borehole	Top Elevation of Cored Cobbles and/or Boulder	Thickness
South Abutment	B3-02	306.3 m	1.4 m
		303.7 m	0.4 m
		298.3 m	3.2 m
North Abutment	B3-03	308.4 m	0.6 m
		306.6 m	1.5 m
		300.3 m	1.5 m
		297.4 m	1.2 m

The SPT 'N'-values measured within the gravelly sandy silt to sand and gravel deposit typically range from 11 blows to 49 blows per 0.3 m of penetration, indicating a compact to dense relative density. A SPT 'N'-value of 40 blows per 0.1 m of penetration, indicating a compact to very dense relative density, was recorded prior to split-spoon sampler refusal on cobbles within this deposit. The DCPT advanced from the bottom of Boreholes B3-02 to B3-04 and DCPT B3-DC02 (advanced adjacent to B3-DC02) extend to effective refusal at 100 blows per 0.10 m of penetration, and at 60 Blows and 150 Blows per 0.08 m of penetration. The Total Core Recovery of the cored cobbles/boulders samples ranges between about 17 per cent and 80 per cent.

The natural water content measured on four samples of the gravelly sandy silt to sand and gravel deposit ranges from about 9 per cent to 19 per cent.

The results of grain size distribution tests completed on four samples of the gravelly sandy silt to sand and gravel deposit are presented on Figure B8 in Appendix B. An Atterberg limits test was carried out on one sample of the gravelly sandy silt portion of this deposit and measured a liquid limit of about 18 per cent, a plastic limit of about 15 per cent and a corresponding plasticity index of about 3 per cent. The result of the Atterberg limits test is shown on the plasticity chart on Figure B9 in Appendix B and indicates that the fines material of the gravelly sandy silt is classified as silt of slight plasticity.

4.3 Groundwater Conditions

In general, the soil samples taken in the boreholes were moist to wet. The groundwater levels measured in the open boreholes upon completion of drilling range from 1.3 m to 2.3 m below ground surface, corresponding to Elevations 324.8 m to 323.9 m.

A standpipe piezometer was installed in a borehole immediately adjacent to Borehole B3-02 to allow monitoring of the groundwater level at the site. The piezometer was decommissioned on June 26, 2013. Details of the piezometer installation and the groundwater levels are shown on the Record of Borehole No. B3-02 in Appendix A and the groundwater level measured in the piezometer is summarized below.

Borehole	Ground Surface Elevation	Depth to Water Level	Groundwater Elevation	Date of Measurement
B3-02	326.2 m	1.9 m	324.3 m	May 2, 2013
		2.4 m	323.8 m	June 26, 2013

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

5.0 CLOSURE

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

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PART B

FOUNDATION DESIGN REPORT

LITTLE EAST RIVER BRIDGE NO. 3 – Site No. 44-176

HIGHWAY 592 – REPLACEMENT OF SIX STRUCTURES

MINISTRY OF TRANSPORTATION, ONTARIO

GWP 5265-07-00; WP 5267-07-01

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides engineering design recommendations for the proposed Little East River Bridge No. 3 on Highway 592 (Site No.44-176). 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. 3 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. 3 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 326.3 m, which corresponds to a raise of the existing approach embankments of up to about 0.2 m.

6.2 Foundation Options

Given that very loose to loose organic deposits are present in the 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/or boulders were encountered within the gravelly sandy silt to sand and gravel deposits; stage construction will be required in a narrow right-of-way; there is an overhead Hydro line along the existing structure which cannot readily be relocated or de-energized, deep foundations comprised of soil-bonded micropiles is considered the preferred alternative for the support of the structure. Driven steel H-piles or drilled steel casings may be considered for design, however, the geotechnical axial capacity will be relatively low as the H-piles and steel casings will develop capacities through friction only within the generally very loose to compact granular deposits.

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

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

6.3 Spread Footings

Shallow foundations comprised of spread/strip footings founded on native overburden are not recommended for support of the proposed bridge abutments given that the soils down to immediately below the depth of frost penetration are very loose to loose organic sand. In the event that shallow foundations are considered further for the support of the proposed structure, recommendations for design are provided below.

6.3.1 Geotechnical Axial Resistance and Reaction

For 11.5 m long by 2 m wide footings founded on the native overburden (a deposit of very loose to loose organic sand underlain by a deposit of very loose to loose silt) at Elevation 322.2 m at the abutments, the factored geotechnical axial resistance at Ultimate Limits States (ULS) and geotechnical reaction at Serviceability Limits States (SLS) for 25 mm of settlement are provided below.

Foundation Location	Factored Geotechnical Axial Resistance at ULS	Geotechnical Reaction at SLS for 25 mm of Settlement
South and North Abutments	300 kPa	45 kPa

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

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

6.3.2 Resistance to Lateral Loads

The resistance to lateral forces/sliding resistance between the concrete footings and the natural subgrade materials should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction, tan δ', 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 δ')
Concrete footing on very loose to loose organic sand	0.30

The value presented above represents an unfactored value.

6.3.3 Frost Protection

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

- The required thickness of conventional soil cover for frost protection of the footings is 1.8 m, as per OPSD 3090.010 (*Frost Penetration Depths for Southern Ontario)*) as measured perpendicular to/from the face of the abutment slope to the edge of the underside of the footing (it is not simply a vertical dimension when the footing is adjacent to a slope).
- If adequate soil cover cannot be provided for the footing, rigid styrofoam insulation should be installed to compensate for the lack of soil cover and provide protection from frost penetration. In this regard, the MTO has adopted an equivalent thickness of 25 mm of styrofoam equal to 300 mm of soil cover.

6.4 Driven Steel H-Pile Foundations

Given the presence of very loose to loose organic sand to silt peat deposit/silt deposits, the very soft to firm clayey silt deposit and the thick underlying granular deposit, friction piles consisting of steel H-piles driven into the compact to dense gravelly sandy silt to sand and gravel deposit could be considered for the support of the proposed structure. However, cobbles and boulders were encountered within the gravelly sandy silt to sand and gravel deposit in the boreholes drilled at this site and there is a risk associated with potential difficulty in driving steel H-piles through the cobbles and 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 east side of the bridge, there may not be adequate construction platform width to accommodate piling equipment necessary to the required depth to drive long H-piles to achieve the desired axial capacities for design. Furthermore, piles cannot be battered for lateral resistance due to the proximity of the temporary shoring (cofferdam).

6.4.1 Geotechnical Axial Resistance and Reaction

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

Foundation Location	Elevation of Underside of Pile Cap [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	322.2 m	321.0 m	302.2 m	20 m	875 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 gravelly sandy silt to sand and gravel deposit, provisions should be made in the Contract Documents to deal with varying pile lengths at the abutments.

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

6.4.2 Set Criteria

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

The pile termination or set criteria will be dependent on the pile driving hammer type and the selected pile type. The set criteria can be established through a variety of methods, including empirical correlations, such as the use of the Hiley Formula, and wave equation analyses, at the time of construction once the hammer and pile types are known. The choice of set criteria is dependent on the experience of the engineer and traditional use where a substantial database has been developed over the years. The criteria need to be set to also avoid overdriving and possibly damaging the pile.

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

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

- Piles to be driven in accordance with Standard SS 103-11 using an ultimate geotechnical resistance of 2,000 kN per pile, but must be driven below El. 302.2 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 gravelly sandy silt to sand and gravel deposit, it is recommended that a down-the-hole (DTH) hammer drilling system be used for the installation of the drilled steel casing. However, due to the proposed construction sequencing/staging, narrow right-of-way and the presence of overhead Hydro lines along the east side of the bridge, there may not be adequate construction platform width to accommodate drilling equipment necessary to advance long steel casing to achieve the desired geotechnical axial capacities for design.

6.5.1 Geotechnical Axial Resistance and Reaction

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

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	322.2 m	321.0 m	302.2 m	20 m	1,425 kN	N/A

Notes:

1. As per the GA Drawing provided by MH on November 6, 2013.
2. The geotechnical reaction at SLS for 25 mm of settlement will be greater than or equal to the factored geotechnical axial resistance at ULS and therefore, the SLS condition does not apply.

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

6.5.2 Frost Protection

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

6.6 Micropiles

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

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

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

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

6.6.1 Geotechnical Axial Resistance and Reaction

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

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	322.2 m	321.0 m	303.7 m	273 mm	18.5 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.

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 (B3-02) and North Abutment (B3-03)	Loose Organic Sand	321.0 m to 320.6 m	3,000 kPa/m	-
	Very Loose to Loose Silt	320.6 m to 313.6 m	3,000 kPa/m	-
	Very Soft to Firm Clayey Silt	313.6 m to 311.3 m	-	50 kPa
	Compact to Very Dense Sand and Gravel	309.1 m to 294.6 m	20,000 kPa/m	-

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

Foundation Location	Pile Type	Axial Load Applied at the Top of Pile/Casing	Factored Geotechnical Lateral Resistance at ULS [1]	Geotechnical Lateral Reaction at SLS for 10 mm of Deflection [1]
South and North Abutment	HP 310 x 110	875 kN	75 kN	30 kN
	610 mm diameter drilled steel casing	1,425 kN	65 kN	40 kN

Note:

1. Analyses assume a fixed-head condition.

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

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:

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. 3 structure will be at about Elevation 326.3 m, requiring placement of up to about 0.2 m of fill to raise the existing south and north approach embankment grades.

Based on the investigated locations at this site, the south and north approach embankments are founded on deposits of very loose to loose organic sand to silty peat/silt, underlain by a deposit of very soft to firm clayey silt with sand, which is in turn underlain by a deposit of compact to very dense gravelly sandy silt to sand and gravel.

It is understood that a partial excavation of the organic soils of up to 2 m deep and backfilling with rock fill along the existing side slopes will be carried out as part of the embankment widening at the approach embankments. However, excavation will not be carried out along the existing embankment and as such, the existing fill material and the underlying organic sand to silty peat will remain in place. It is also understood that rock fill will be utilized for the embankment widening at this site. Further, it is understood that a preload period of one year will be included in the construction schedule to allow for the settlement/consolidation of the underlying organic soils, inorganic deposits as well as the rock fill.

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

6.10.1 Stability

6.10.1.1 Methodology

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

6.10.1.2 Parameter Selection

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

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

For the purpose of the stability analysis, the groundwater level was assumed to be at Elevation 323.8 m, which is based on groundwater level measurements in the open boreholes upon completion of drilling.

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

Embankment	Soil Type	Unit Weight, γ	Undrained Shear Strength, s_u	Cohesion, c'	Effective Friction Angle, ϕ'
South Approach Embankment	New Granular Fill	20 kN/m^3	-	0 kPa	34°
	New Rock Fill	19 kN/m^3	-	0 kPa	40°
	Existing Loose to Compact Sand and Gravel Fill	20 kN/m^3	-	0 kPa	30°
	Loose Organic Sand to Silty Peat	18 kN/m^3	-	0 kPa	27°
	Very Loose to Loose Silt	18 kN/m^3	-	0 kPa	28°
	Loose Gravelly Sand	20 kN/m^3	-	0 kPa	29°
	Very Soft to Firm Clayey Silt with Sand	17 kN/m^3	25 kPa	-	-
	Compact to Very Dense Sand and Gravel	20 kN/m^3	-	0 kPa	34°

Embankment	Soil Type	Unit Weight, γ	Undrained Shear Strength, s_u	Cohesion, c'	Effective Friction Angle, ϕ'
North Approach Embankment	New Granular Fill	20 kN/m^3	-	0 kPa	34°
	New Rock Fill	19 kN/m^3	-	0 kPa	40°
	Existing Loose Silt and Sand to Sand and Gravel Fill	20 kN/m^3	-	0 kPa	30°
	Very Loose to Loose Organic Sand to Silty Peat	18 kN/m^3	-	0 kPa	27°
	Very Loose to Loose Silt	18 kN/m^3	-	0 kPa	28°
	Firm Clayey Silt with Sand	17 kN/m^3	25 kPa	-	-
	Compact Gravelly Sandy Silt	20 kN/m^3	-	0 kPa	30°
	Compact to Dense Sand and 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 and North Approach Embankments	2.2 m	1.25H:1V	≥ 1.3

Note:

1. Embankment height includes an approximately 0.2 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);
- Elastic compression of the cohesive soils (short-term);
- Primary and secondary time-dependent consolidation of organic soils (long-term); and,

- Self-weight compression of the new embankment fill (long-term).

The analyses were carried out at the critical sections of the approach embankments where the thickness of compressible foundation soils is up to about 29.9 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 with sand deposit encountered overlying the gravelly sand and/or the silt 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. The compression of the organic soils was modelled by estimating deformation parameters based on correlations proposed by Mesri and Ajlouni (2007) and the National Research Council of Canada (1969).

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

Embankment	Soil Type	Thickness [1]	Unit Weight, γ	Deformation Parameter(s)
South Approach Embankment	Existing Loose to Compact Sand and Gravel Fill	2.2 m to 4.5 m	20 kN/m^3	E' = 5 MPa
	Loose Organic Sand to Silty Peat	1.9 m to 3.4 m	18 kN/m^3	e_o = 2.0 C_c = 0.75 $C_{\alpha(\varepsilon)}$ = 0.045 c_v =1.0 x 10^{-3} cm^2/s
	Very Loose to Loose Silt	~6.1 m	18 kN/m^3	E' = 3 MPa
	Loose Gravelly Sand	~1.1 m	20 kN/m^3	E' = 5 MPa
	Very Soft to Firm Clayey Silt with Sand	~3.8 m	17 kN/m^3	m_v = 5 x 10^{-4} kPa^{-1}
	Compact to Very Dense Sand and Gravel	~15.0 m	20 kN/m^3	E' = 25 MPa

Embankment	Soil Type	Thickness [1]	Unit Weight, γ	Deformation Parameter(s)
North Approach Embankment	Existing Loose Silt and Sand to Sand and Gravel Fill	3.0 m to 5.6 m	20 kN/m^3	E' = 5 MPa
	Very Loose to Loose Organic Sand to Silty Peat	1.6 m to 2.6 m	18 kN/m^3	e_o = 2.0 C_c = 0.75 $C_{\alpha(\varepsilon)}$ = 0.045 c_v =1.0 x 10^{-3} cm^2/s
	Very Loose to Loose Silt	~6.9 m	18 kN/m^3	E' = 3 MPa
	Firm Clayey Silt with Sand	~2.3 m	17 kN/m^3	m_v = 5 x 10^{-4} kPa^{-1}
	Compact Gravelly Sandy Silt	~1.9 m	20 kN/m^3	E' = 10 MPa
	Compact to Dense Sand and Gravel	~14.8 m	20 kN/m^3	E' = 25 MPa

Note:

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

where: E' is the elastic modulus (MPa)
m_v is the coefficient of volume compressibility (kPa^{-1})
e_o is the initial void ratio
C_c is the primary compression index
C_α is the secondary compression index
c_v is the coefficient of consolidation (cm^2/s)

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

For the purpose of settlement analyses, the groundwater level was assumed to be located on average at Elevation 323.8 m, based on several groundwater level measurements in the open boreholes upon completion of drilling.

6.10.2.3 Settlement of Foundation Soils

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

Embankment	Settlement During Construction (including 1 year Preload Period)		Post-Construction Settlement (10 Years After Preload Period)		Total Settlement
	Organic Soils [1]	Inorganic Soils [2]	Organic Soils [1]	Inorganic Soils [2]	
South Approach Embankment Centreline	30 mm	30 mm	55 mm	~0 mm	115 mm
South Approach Embankment Side Slope [3]	560 mm	35 mm to 60 mm	130 mm	~0 mm	725 mm to 750 mm
North Approach Embankment Centreline	35 mm	30 mm	40 mm	~0 mm	105 mm
North Approach Embankment Side Slope [3]	560 mm	35 mm to 60 mm	130 mm	~0 mm	725 mm to 750 mm

Notes:

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

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

6.10.2.4 Settlement of Rock Fill Embankment

It is understood that rock fill is to be used for the construction of the approach embankments widening as a result of the narrow right-of-way and as such, there will be settlement due to compression of the rock fill itself under self-weight along the east side of the approach embankments. The magnitude of settlement of the rock fill depends on the type of rock/strength of particles, size and shape of particles, gradation of rock fill, total height/thickness of fill and the method of construction and sequence of placement. Rock fill should be placed, whenever possible, in a controlled manner (i.e. not end-dumped) in accordance with SP 206S03 (Rock Excavation, Grading). Where rock fill cannot be placed in a controlled manner (i.e. below the groundwater table), the post-construction settlement of the rock fill is expected to be greater. Based on MTO's Guideline for Rock Fill Settlement and Rock Fill Quantity Estimates (2010), the estimated settlements of rock fill for the approach embankments are presented below.

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

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

6.10.3 Liquefaction Potential below Embankments

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

6.10.4 Embankment Platform Widening

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

6.10.5 Embankment Fill Placement

Placement 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 322.2 m and the underside of the tremie plug at Elevation 321.0 m, excavations up to about 5.2 m deep below the existing ground surface will be required and will be made through the existing fill material and overburden soils. The existing fill materials and native overburden soils are considered Type 3 soils according to the Occupational Health and Safety Act and Regulation for Construction Projects (OHSA) and as such, temporary open-cut excavations above the groundwater level should be carried out with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V). In addition, temporary shoring (cofferdam) will be required for the excavation to the underside of the tremie plug as it will be below the groundwater level and the water level of Little East River.

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

6.11.2 Temporary Roadway Protection

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

6.11.3 Control of Groundwater and Surface Water

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

6.11.4 Obstructions

It should be noted that cobbles and boulders were encountered within the native gravelly sandy silt to sand and 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.

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Reese, L.C., 1975. Laterally Loaded Piles. GESA Report D-75-14, UCCC Report 75-14, Geotechnical Engineering Software Activity, University of Colorado Computing Centre, Boulder.

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ASTM International:

ASTM D1586	Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils

Commercial Software:

LPILE Plus (Version 5.0) by Ensoft Inc.

Settle^{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 206S03	Amendment to OPSS 206 – Earth Excavation, Grading; Rock Excavation, Grading.
Special Provision 539S02	Amendment to OPSS 539 – Protection System

Ministry of Transportation Ontario:

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

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

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

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

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

Ontario Occupational Health and Safety Act:

Ontario Regulation 213	Construction Projects (as amended)

Ontario Provisional Standard Drawing:

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

Ontario Provincial Standard Specification:

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

Ontario Water Resources Act:

Ontario Regulation 903	Wells (as amended)

TABLES

Table 1: Evaluation of Foundation Alternatives

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks / Consequences
Spread/strip footings (11.5 m long by 2 m wide)	NR[1]	■ Relative ease of construction.	■ Allows only for semi-integral abutment design. ■ Axial capacity on the loose organic sand to silty peat or at greater depth on the very loose to loose silt 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.	■ 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.	■ Large footings will be required to develop adequate axial capacity.
Driven steel H-piles (HP 310x110)	2	■ 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.	■ Integral abutment design may not be possible due to constraints in achieving free length of pile to allow for lateral movement due to the presence of the tremie plug. ■ 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	■ 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.	■ 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 gravelly sandy silt to sand and 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.

Table 1: Evaluation of Foundation Alternatives

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks / Consequences
			to the proximity of the sheetpile cofferdam. ■ Excavation for pile cap will be below water table. ■ 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 east side of the bridge will be in close proximity to overhead hydro lines.		■ Overhead hydro lines will need to be de-energized during portions of the piling operation.
Drilled steel casings using DTH hammer drilling system (610 mm)	3	■ 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.	■ 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	■ 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.	■ 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 gravelly sandy silt to sand and gravel deposit. ■ May require additional construction platform width and/or temporary closure of the roadway to accommodate larger

Table 1: Evaluation of Foundation Alternatives

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

Note: 1. NR – Not Recommended

DRAWINGS

MINISTRY OF TRANSPORTATION, ONTARIO

PLAN

SCALE

500 0 500 1000m

METRIC

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

CONT No.

WP No. 5267–07–01

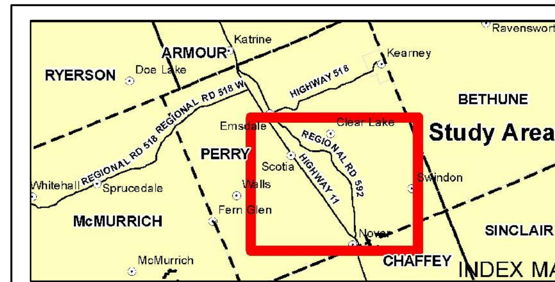
SHEET

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–332

HWY. 592		PROJECT NO. 11–1111–0149	DIST.
SUBM'D. AV	CHKD. CN	DATE: Dec. 2013	SITE:
DRAWN: JFC	CHKD.	APPD.	DWG. 1

PLOT DATE: December 23, 2013

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

CONT No.
WP No. 5267-07-01

SHEET

HIGHWAY 592
LITTLE EAST RIVER BRIDGE #3
BOREHOLE LOCATIONS AND SOIL STRATA

Golder Associates
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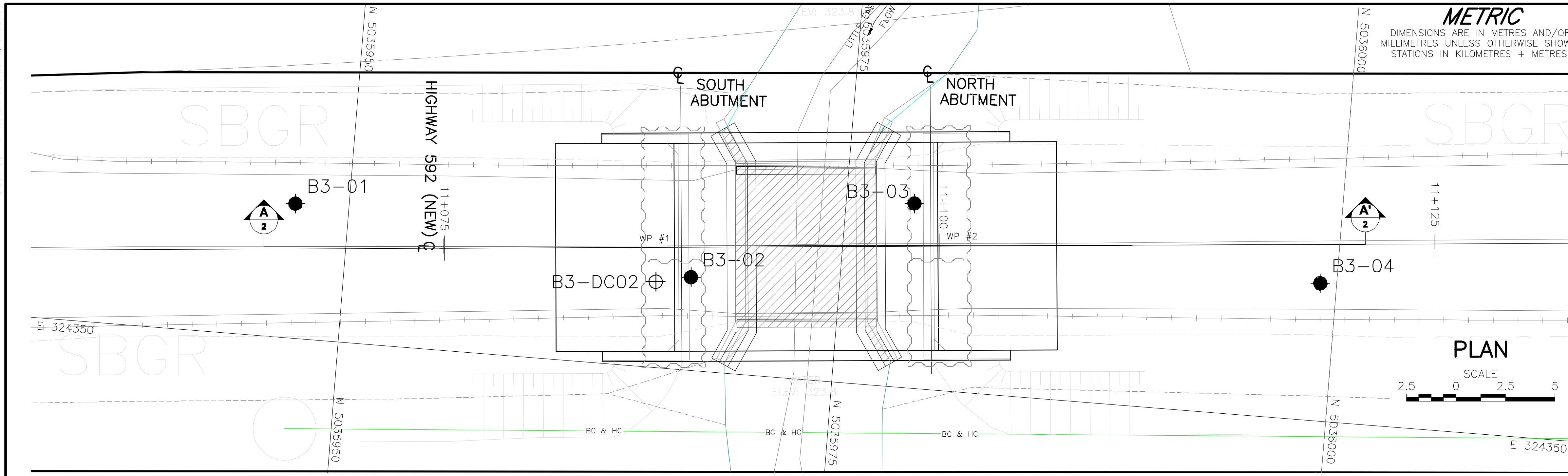
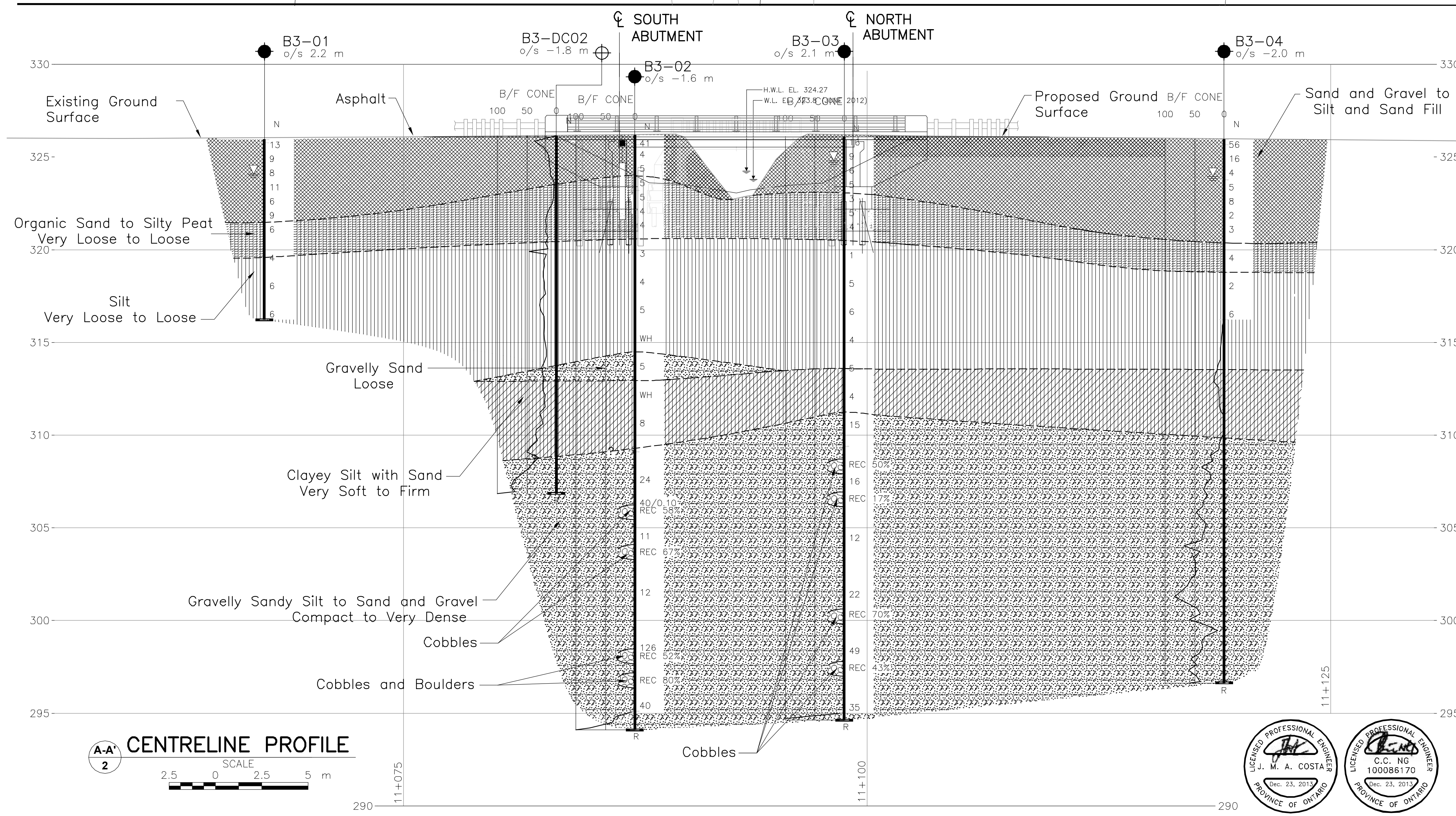
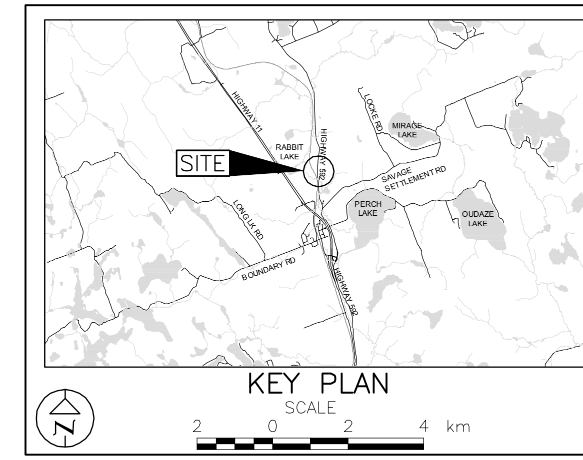
MISSISSAUGA, ONTARIO, CANADA







LEGEND

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

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
B3-01	326.0	5035947.3	324343.2
B3-02	326.2	5035967.5	324345.3
B3-03	326.1	5035978.4	324340.7
B3-04	326.0	5035999.2	324343.1
B3-DC02	326.2	5035965.7	324345.7

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 file Plan and Profile no. 44176-01.dwg, received November 7, 2013.

LICENSED PROFESSIONAL ENGINEER J. M. A. COSTA Dec. 23, 2013 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER C.C. NG 100086170 Dec. 23, 2013 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No. 31E-332			
HWY. 592	PROJECT NO. 11-1111-0149		DIST.
SUBM'D. AV	CHKD. CN	DATE: Dec. 2013	SITE: 44-176
DRAWN: JFC	CHKD. TVA	APPD.	DWG. 2

PLOT DATE: December 23, 2013
FILENAME: T:\Projects\2011\11-1111-0149 (MH, Huntsville)\--CB--Little East River Bridge #3 Stamped\1111110149C8002.dwg

APPENDIX A

Record of Borehole/DCPT 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
υ	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

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

(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 $= \sigma'_p / \sigma'_{vo}$
(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

Notes: 1 $\tau = c' + \sigma' \tan \phi'$

2 shear strength = (compressive strength)/2

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

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, 60º cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

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

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm^2 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 **Relative Density**	**N** **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	c_u, s_u 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

Golder Associates

RECORD OF BOREHOLE No B3-01

SHEET 1 OF 1 **METRIC**

PROJECT 11-1111-0149
W.P. 5267-07-01 LOCATION N 5035947.3 ;E 324343.2 ORIGINATED BY ID
DIST ___ HWY 592 BOREHOLE TYPE 120 mm O.D. Hollow Stem Augers and NW Casing COMPILED BY GRL/AV
DATUM Geodetic DATE May 7, 2013 CHECKED BY CN

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
326.0	GROUND SURFACE						
0.0	Asphalt (25 mm)						
	Sand and gravel, trace silt (FILL) Loose to compact Dark brown becoming grey below a depth of 2.3 m Moist to wet	1A / 1B	SS	13			
		2	SS	9		325	
		3	SS	8	∇	324	44 52 (4)
		4	SS	11		323	
		5	SS	6		322	
		6	SS	9			
321.5 / 4.5	SILTY PEAT, some sand, trace gravel, containing wood fragments Loose Black Wet	7	SS	6		321	OC=36.6% (unit weight column)
						320	
319.6 / 6.4	SILT, trace to some clay, trace sand Loose Grey Wet	8A / 8B	SS	4		319	
		9	SS	6		318	0 1 90 9 Non-plastic
						317	
		10	SS	6			
316.2 / 9.8	END OF BOREHOLE						

NOTE:

1. Water level in open borehole at a depth of 1.9 m below ground surface (Elev. 324.1 m) upon completion of drilling.

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100
PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60
UNIT WEIGHT γ kN/m³

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

Golder Associates

RECORD OF BOREHOLE No B3-02

SHEET 1 OF 3 **METRIC**

PROJECT 11-1111-0149
W.P. 5267-07-01 LOCATION N 5035967.5 ;E 324345.3 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 April 29 to May 2, 2013 CHECKED BY CN

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	GR	SA	SI	CL
326.2	GROUND SURFACE								
0.0 / 0.1	Asphalt (50 mm)								
	Asphalt fragments	1A / 1B	SS	41					
325.7 / 0.5	Sand and gravel (FILL) Loose Brown Moist	2	SS	4	○				
	Containing clayey silt pockets below a depth of 1.5 m.	3	SS	5					
324.0 / 2.2	ORGANIC SAND, trace to some silt, trace gravel, containing rootlets Loose Dark grey to black Wet	4	SS	5					
		5	SS	5	○				
		6	SS	4	○	0	74	16	0
		7	SS	4	○				
320.6 / 5.6	SILT, trace clay to some clay, trace sand Very loose to loose Grey Wet	8	SS	3	○	0	1	91	8
		9	SS	4					
		10	SS	5	○	0	0	86	14
		11	SS	WH					
314.5 / 11.7	Gravelly SAND, trace to some silt, trace to some clay Loose Grey Wet	12	SS	5	○	25	53	11	11
312.9 / 13.3	CLAYEY SILT with SAND, trace gravel Very soft to firm Grey Wet	13	SS	WH	H○	1	45	34	20

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (326 to 312); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No B3-02

SHEET 2 OF 3 — METRIC

PROJECT: 11-1111-0149			
W.P.: 5267-07-01	LOCATION: N 5035967.5 ;E 324345.3	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: April 29 to May 2, 2013	CHECKED BY: CN	

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAYEY SILT with SAND, trace gravel Very soft to firm Grey Wet	14	SS	8	311		
					310		
309.1 17.1	SAND and GRAVEL, trace to some silt Compact to very dense Brown to grey Wet				309		
		15	SS	24	308		
		16	SS	40/0.10	307		
	Cobbles encountered between depths of 19.9 and 21.3 m.	1	RC	REC 58%	306		
		17	SS	11	305	○	48 49 (3)
	Cobbles encountered between depths of 22.5 and 22.9 m.	2	RC	REC 67%	304		
					303		
		18	SS	12	302		
					301		
					300		
		19	SS	126	299	○	55 37 (8)
	Cobbles and Boulders encountered between depths of 27.9 and 31.1 m.	3	RC	REC 52%	298		
		4	RC	REC 80%	297		

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20, 40, 60, 80, 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (20, 40, 60); UNIT WEIGHT γ kN/m³; GROUND WATER CONDITIONS; STRAT PLOT.

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity   ○ 3% STRAIN AT FAILURE

Foundation Design

Golder Associates

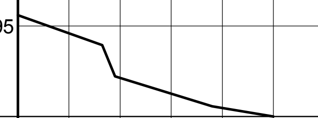
RECORD OF BOREHOLE No B3-02

SHEET 3 OF 3

METRIC

PROJECT 11-1111-0149
W.P. 5267-07-01
LOCATION N 5035967.5 ;E 324345.3
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 April 29 to May 2, 2013
CHECKED BY CN

ELEV DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED) 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
295.1	SAND and GRAVEL, trace to some silt Compact to very dense Brown to grey Wet		4	RC	REC 80%		296				
			20	SS	40						
294.1 31.1	END OF BOREHOLE Dynamic Cone Penetration Test (DCPT)						295				
32.1	END OF DCPT Refusal to Further Penetration (100 Blows / 0.10 m)										



NOTES:

1. Water level in open borehole at a depth of 2.3 m below ground surface (Elev. 323.9 m) upon completion of drilling.

2. An additional borehole was advanced about 1.5 m South of Borehole B3-02 to install a piezometer and carry out Dynamic Cone Penetration Test, see B3-DC02 for results of cone penetration test.

3. Water level measurements in Piezometer:

Date	Depth (m)	Elev. (m)
05/02/13	1.9	324.3
06/26/13	2.4	323.8

4. Piezometer decommissioned on June 26, 2013.

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

$O^{3\%}$ STRAIN AT FAILURE

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

RECORD OF BOREHOLE No B3-03

SHEET 1 OF 3

METRIC

PROJECT 11-1111-0149			
W.P. 5267-07-01	LOCATION N 5035978.4 ;E 324340.7		ORIGINATED BY ID
DIST	HWY 592	BOREHOLE TYPE 120 mm O.D. Hollow Stem Augers and NW Casing	COMPILED BY GRL/AV
DATUM Geodetic	DATE May 2 to 6, 2013		CHECKED BY CN

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100) | SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20, 40, 60, 80, 100) | PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20, 40, 60 | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
326.1	GROUND SURFACE					
0.0	Asphalt (25 mm)	1A	SS	16		
325.7 / 0.4	Sand and gravel (FILL) Compact Dark grey Moist	1B				
	Silt and sand, trace gravel, trace clay (FILL) Loose Brown Moist	2	SS	9		
		3	SS	6		3 61 33 3
		4	SS	5		
323.1 / 3.0	ORGANIC SAND, some silt, containing wood fragments Very loose to loose Black Wet	5	SS	3	OC=12.9%	Non-plastic
		6	SS	5		
		7	SS	4		
320.5 / 5.6	SILT, trace to some clay Very loose to loose Grey Wet	8	SS	1		
		9	SS	5		0 0 92 8 Non-plastic
		10	SS	6		
		11	SS	4		
313.6 / 12.5	CLAYEY SILT with SAND, some gravel Firm Grey Wet	12A	SS	5		
		12B				14 61 11 14
		13	SS	4		
311.3 / 14.8						

Ground water level symbol (∇) shown at approximately elev. 324.9.

Elevation scale: 326, 325, 324, 323, 322, 321, 320, 319, 318, 317, 316, 315, 314, 313, 312

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity $\bigcirc^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No B3-03

METRIC

PROJECT 11-1111-0149			
W.P. 5267-07-01	LOCATION N 5035978.4 ;E 324340.7	ORIGINATED BY ID	
DIST HWY 592	BOREHOLE TYPE 120 mm O.D. Hollow Stem Augers and NW Casing	COMPILED BY GRL/AV	
DATUM Geodetic	DATE May 2 to 6, 2013	CHECKED BY CN	

DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20, 40, 60, 80, 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (20, 40, 60); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%)

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	GR	SA	SI	CL
	Gravelly Sandy SILT, some clay Compact Grey Wet					311				
		14	SS	15			28	28	30	14
						310				
309.4 16.7	SAND and GRAVEL, trace to some silt Compact to dense Wet					309				
	Cobbles encountered between depths of 17.7 m and 18.3 m.	1	RC	REC 50%		308				
		15	SS	16		307				
	Cobbles encountered between depths of 19.5 m and 21.0 m.	2	RC	REC 17%		306				
						305				
		16	SS	12		304				
						303				
						302				
		17	SS	22		301				
	Cobbles encountered between depths of 25.8 m and 27.3 m.	3	RC	REC 70%		300				
						299				
		18	SS	49		298				
	Cobbles encountered between depths of 28.7 m and 29.9 m.	4	RC	REC 43%		297				

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity   ○ $^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No B3-03

SHEET 3 OF 3 **METRIC**

PROJECT 11-1111-0149			
W.P. 5267-07-01	LOCATION N 5035978.4 ;E 324340.7		ORIGINATED BY ID
DIST	HWY 592	BOREHOLE TYPE 120 mm O.D. Hollow Stem Augers and NW Casing	COMPILED BY GRL/AV
DATUM Geodetic		DATE May 2 to 6, 2013	CHECKED BY CN

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	SAND and GRAVEL, trace to some silt Compact to dense Wet						296				
			19	SS	35				○		50 43 (7)
295.0	END OF BOREHOLE Dynamic Cone Penetration Test (DCPT)						295	150			
294.6 31.5	END OF DCPT Refusal to Further Penetration (150 Blows / 0.08 m) NOTE: 1. Water level in open borehole at a depth of 1.3 m below ground surface (Elev. 324.8 m) upon completion of drilling.										

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

$\circ^{3\%}$ STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No B3-04

SHEET 1 OF 3

METRIC

PROJECT 11-1111-0149

W.P. 5267-07-01

LOCATION N 5035999.2 ;E 324343.1

ORIGINATED BY ID

DIST HWY 592

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

COMPILED BY GRL/AV

DATUM Geodetic

DATE May 2, 2013

CHECKED BY CN

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100) | SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20, 40, 60, 80, 100) | PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L | WATER CONTENT (%) (20, 40, 60) | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
326.0	GROUND SURFACE											
0.0	Asphalt (40 mm)		1A	SS	56							
325.7 0.3	Sand and gravel, containing asphalt fragments (FILL) Very dense Brown Moist		1B									
	Sand, trace to some gravel (FILL) Loose to very dense Brown and black Moist		2	SS	16		325	○				
			3	SS	4	▽	324					
323.8 2.2	Gravelly silt and sand, some gravel, trace clay, trace organics, containing rootlets and wood fragments (FILL) Very loose to loose Brown becoming grey below a depth of 3.7 m Wet		4	SS	5			○	22	40	34	4
			5	SS	8		323					
			6	SS	2		322					
			7	SS	3		321	○				
320.4 5.6	SILTY PEAT, some sand, containing roots and wood fragments Very loose Black Wet						320					
			8	SS	4			○ 338				
318.8 7.2	SILT, trace to some clay Very loose to loose Grey Wet						319					
			9	SS	2		318					
							317					
			10	SS	6			○	0	0	91	9
									Non-plastic			
316.2 9.8	END OF BOREHOLE Dynamic Cone Penetration Test (DCPT)						316					
							315					
							314					
							313					
							312					

Continued Next Page

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

○ 3% STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No B3-04

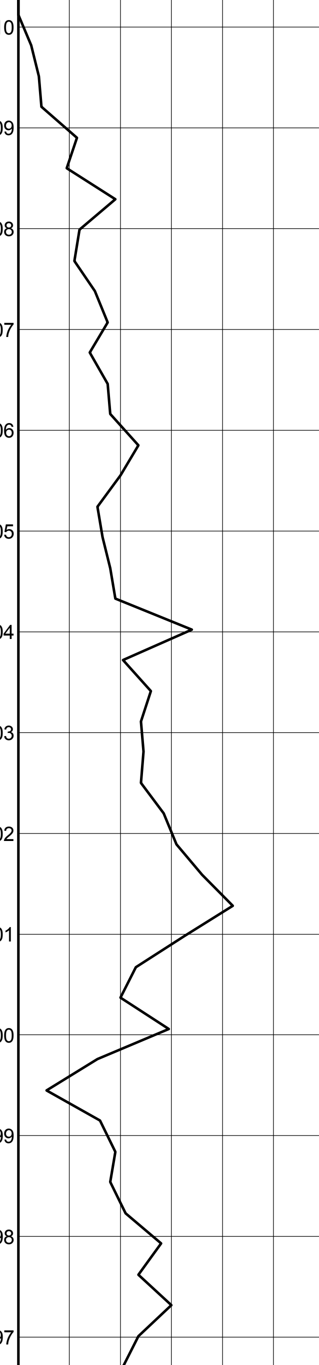
METRIC

PROJECT 11-1111-0149

W.P. 5267-07-01 LOCATION N 5035999.2 ;E 324343.1 ORIGINATED BY ID

DIST HWY 592 BOREHOLE TYPE 120 mm O.D. Hollow Stem Augers and NW Casing COMPILED BY GRL/AV

DATUM Geodetic DATE May 2, 2013 CHECKED BY CN

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
							310 309 308 307 306 305 304 303 302 301 300 299 298 297				
296.7 29.3											

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity ○ $^{3\%}$ STRAIN AT FAILURE

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

Golder Associates

RECORD OF BOREHOLE No B3-04

SHEET 3 OF 3

METRIC

PROJECT 11-1111-0149

W.P. 5267-07-01 LOCATION N 5035999.2 ;E 324343.1 ORIGINATED BY ID

DIST HWY 592 BOREHOLE TYPE 120 mm O.D. Hollow Stem Augers and NW Casing COMPILED BY GRL/AV

DATUM Geodetic DATE May 2, 2013 CHECKED BY CN

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	END OF DCPT Refusal to Further Penetration (60 Blows / 0.08 m) NOTE: 1. Water level in open borehole at a depth of 2.0 m below ground surface (Elev. 324.0 m) upon completion of drilling.										

$+^3, \times^3$: Numbers refer to Sensitivity $\bigcirc^{3\%}$ STRAIN AT FAILURE

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

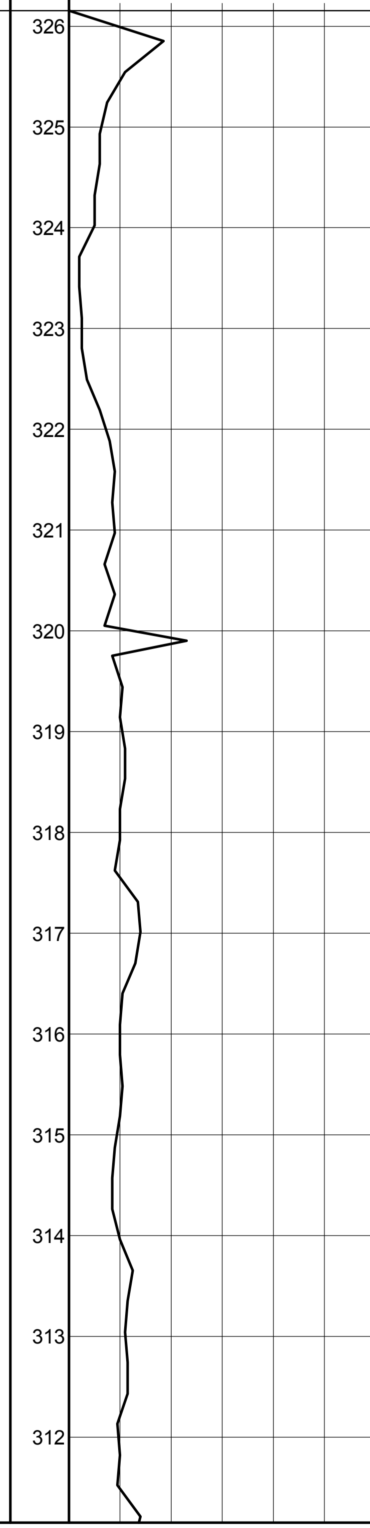
RECORD OF DCPT No B3-DC02

SHEET 1 OF 2

METRIC

PROJECT 11-1111-0149			
W.P. 5267-07-01	LOCATION N 5035965.7 ;E 324345.7		ORIGINATED BY ID
DIST ___ HWY 592	BOREHOLE TYPE Dynamic Cone Penetration Test		COMPILED BY GRL/AV
DATUM Geodetic	DATE May 2, 2013		CHECKED BY CN

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
326.2	GROUND SURFACE										
0.0	Dynamic Cone Penetration Test (DCPT)										



Continued Next Page

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

○ 3% STRAIN AT FAILURE

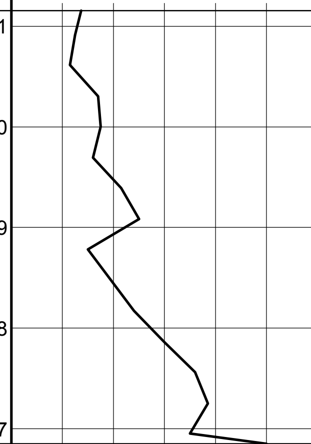
RECORD OF DCPT No B3-DC02

SHEET 2 OF 2

METRIC

PROJECT	11-1111-0149				
W.P.	5267-07-01	LOCATION	N 5035965.7 ;E 324345.7	ORIGINATED BY	ID
DIST	HWY 592	BOREHOLE TYPE	Dynamic Cone Penetration Test	COMPILED BY	GRL/AV
DATUM	Geodetic	DATE	May 2, 2013	CHECKED BY	CN

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20 40 60 80 100)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	Dynamic Cone Penetration Test (DCPT)						311				
							310				
							309				
							308				
306.9							307				
19.3	END OF DCPT Refusal to Further Penetration (100 Blows / 0.10 m)										



$+^3, \times^3$: Numbers refer to Sensitivity   ○ $^{3\%}$ STRAIN AT FAILURE

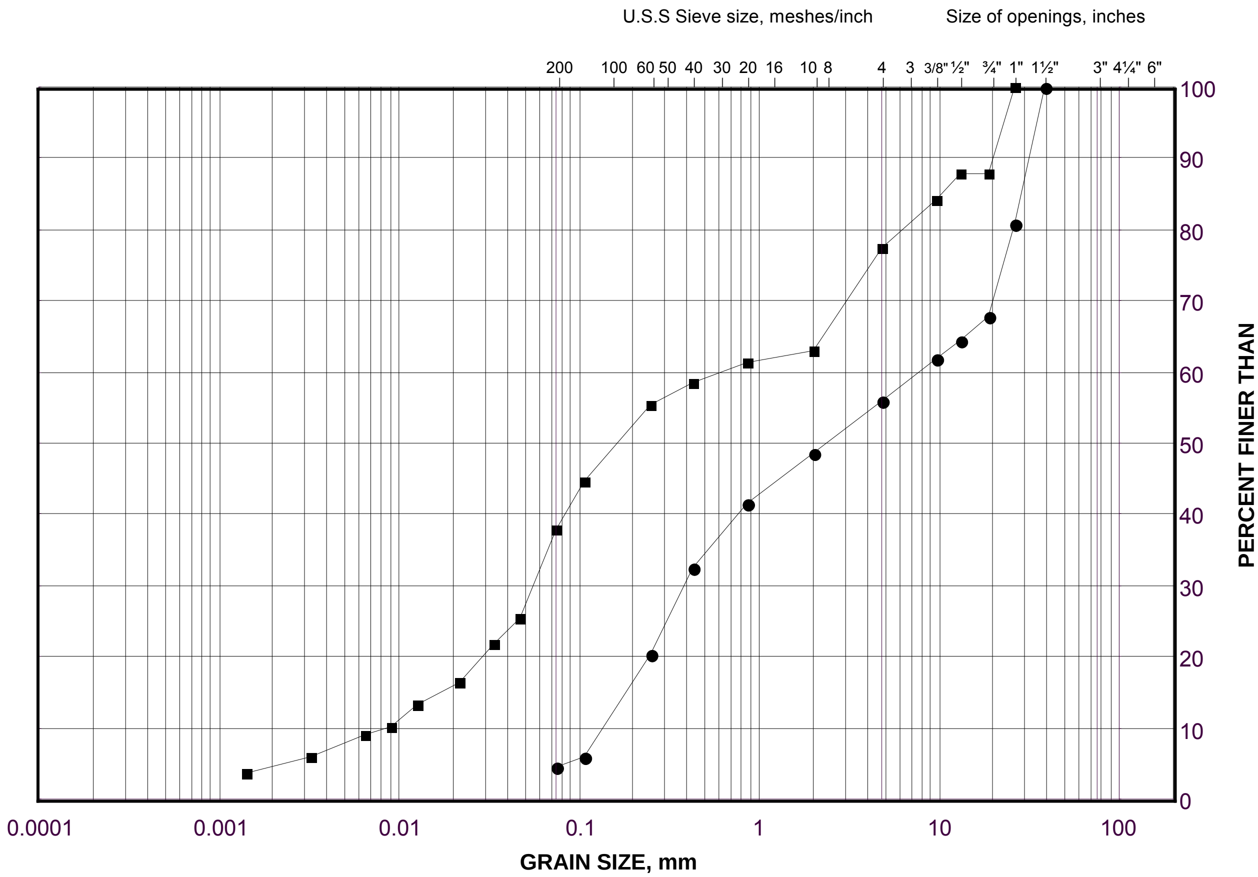
APPENDIX B

Laboratory Test Results

GRAIN SIZE DISTRIBUTION

Gravelly Silt and Sand to Sand and Gravel (Fill)

FIGURE B1



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	B3-01	3	324.2
■	B3-04	4	323.4

Project Number: 11-1111-0149

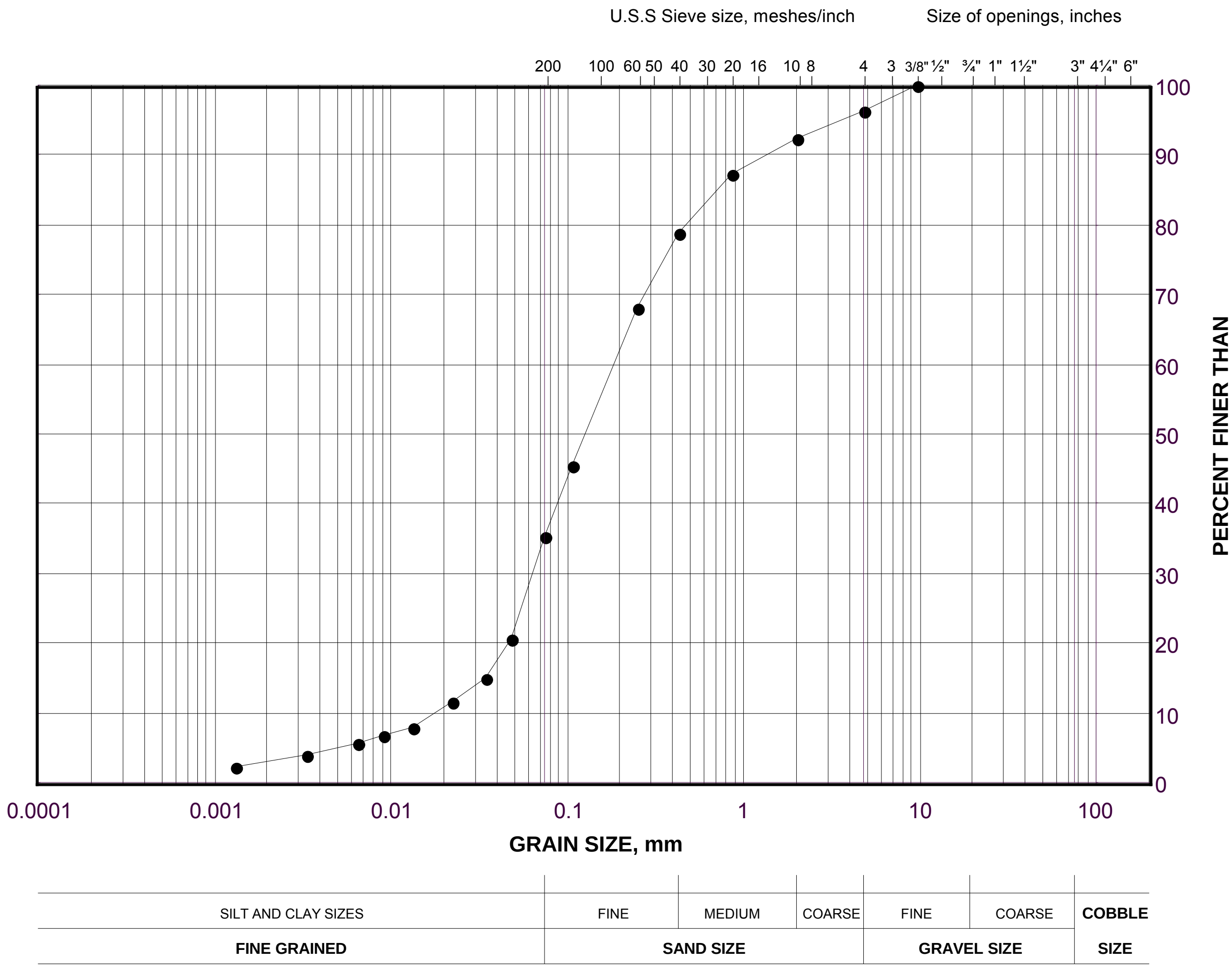
Checked By: MCK/TVA

Golder Associates

Date: 26-Aug-13

GRAIN SIZE DISTRIBUTION
Silt and Sand (Fill)

FIGURE B2



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	B3-03	3	324.3

Project Number: 11-1111-0149

Checked By: MCK/TVA

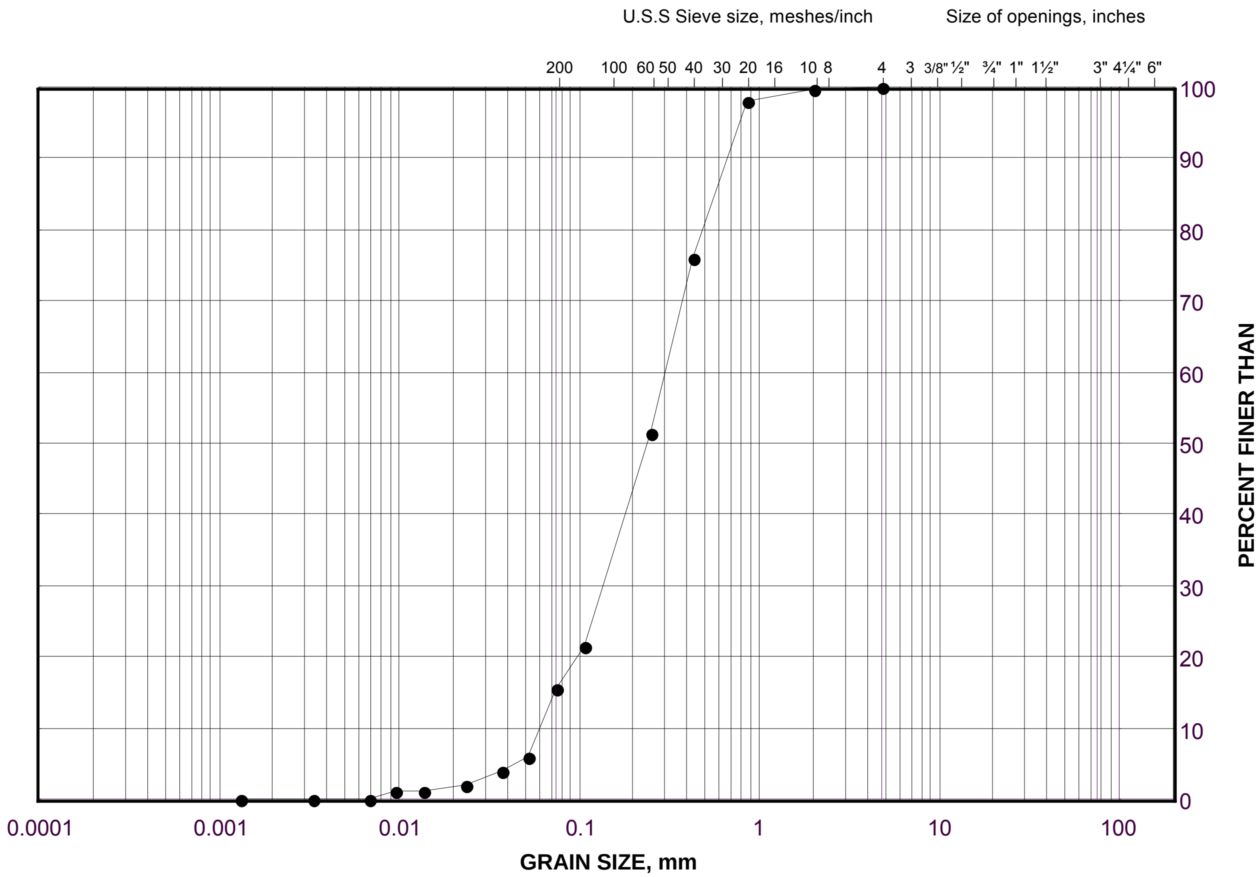
Golder Associates

Date: 08-Aug-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)
●	B3-02	6	322.1

Project Number: 11-1111-0149

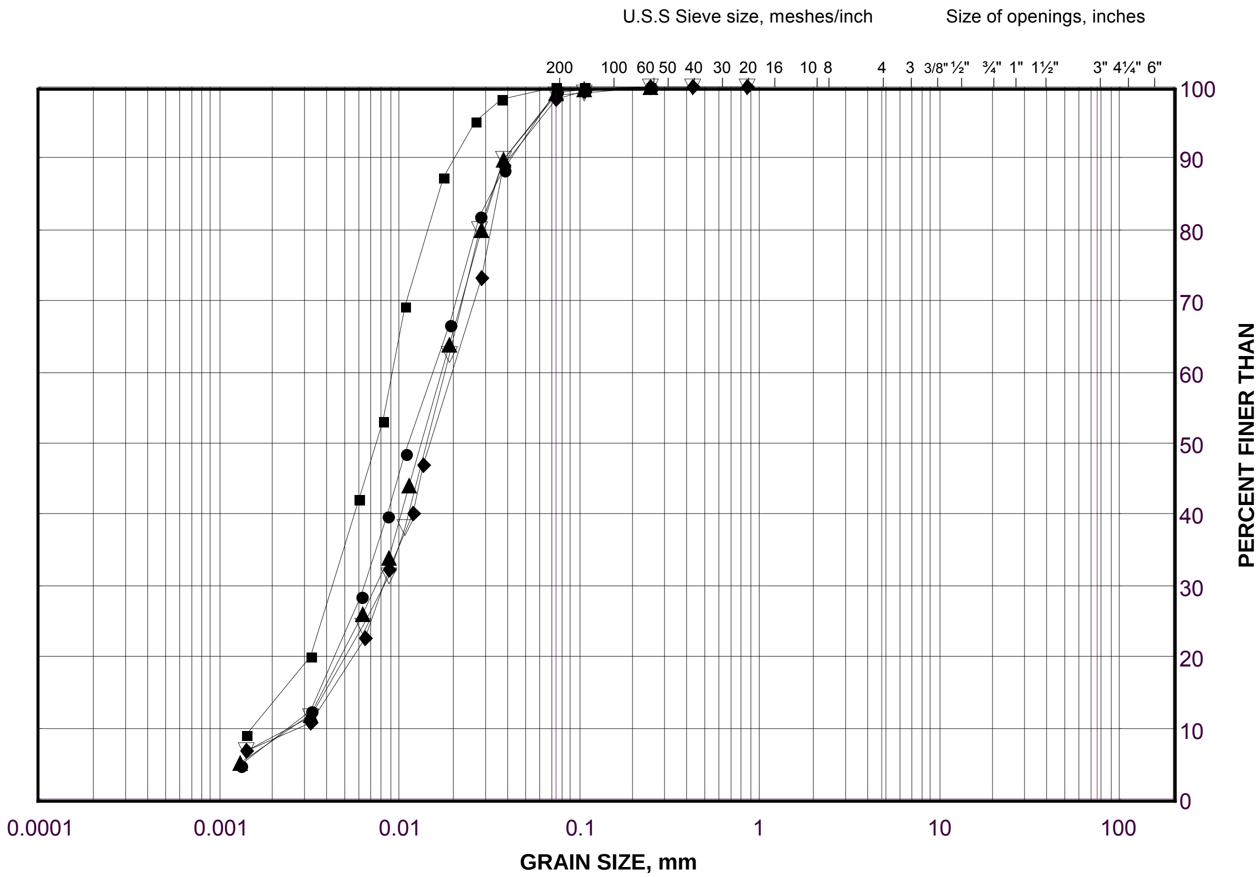
Checked By: MCK/TVA

Golder Associates

Date: 26-Aug-13

GRAIN SIZE DISTRIBUTION
Silt

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)
●	B3-04	10	316.5
■	B3-02	10	316.7
◆	B3-02	8	319.8
▲	B3-03	9	318.2
▽	B3-01	9	318.1

Project Number: 11-1111-0149

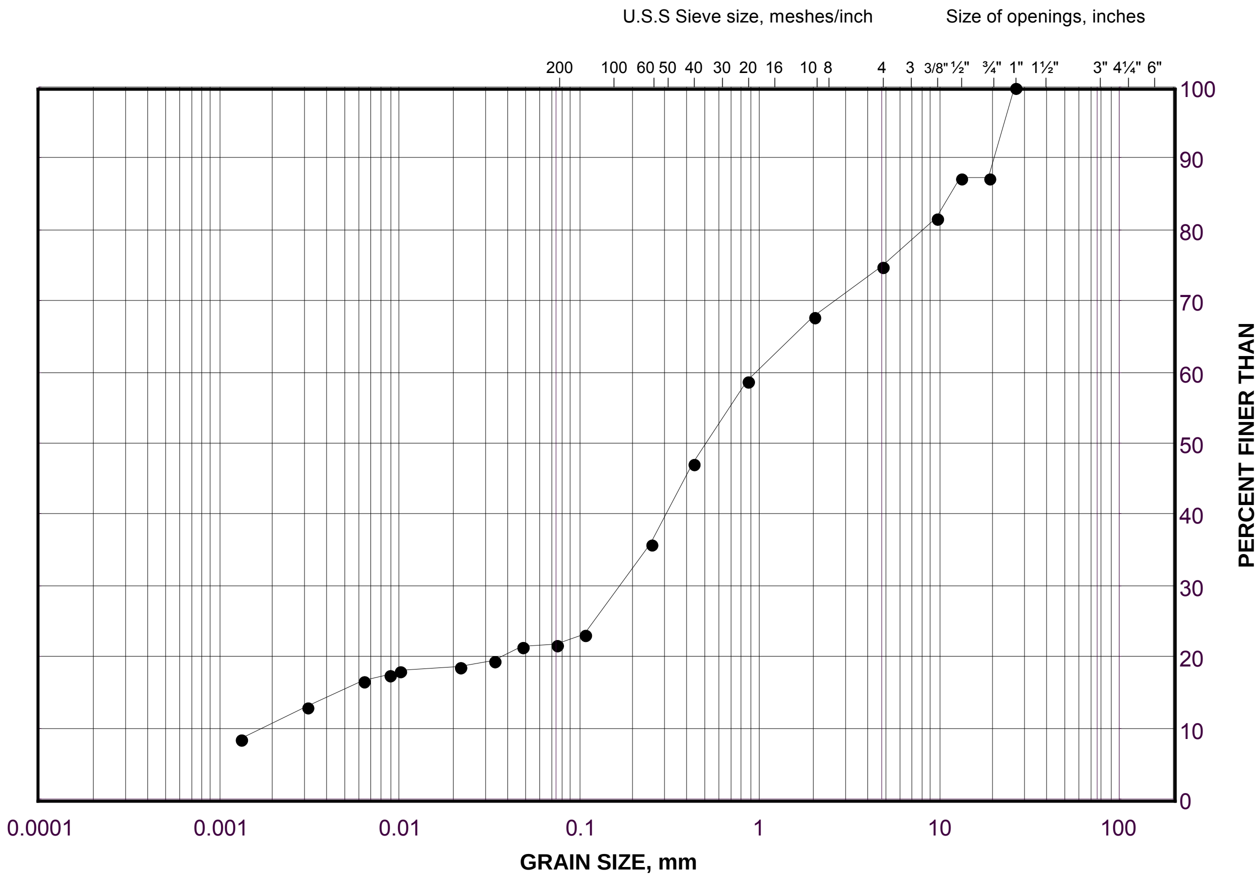
Checked By: MCK/TVA

Golder Associates

Date: 08-Aug-13

GRAIN SIZE DISTRIBUTION
Gravelly Sand (Pocket)

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)
●	B3-02	12	313.7

Project Number: 11-1111-0149

Checked By: MCK/TVA

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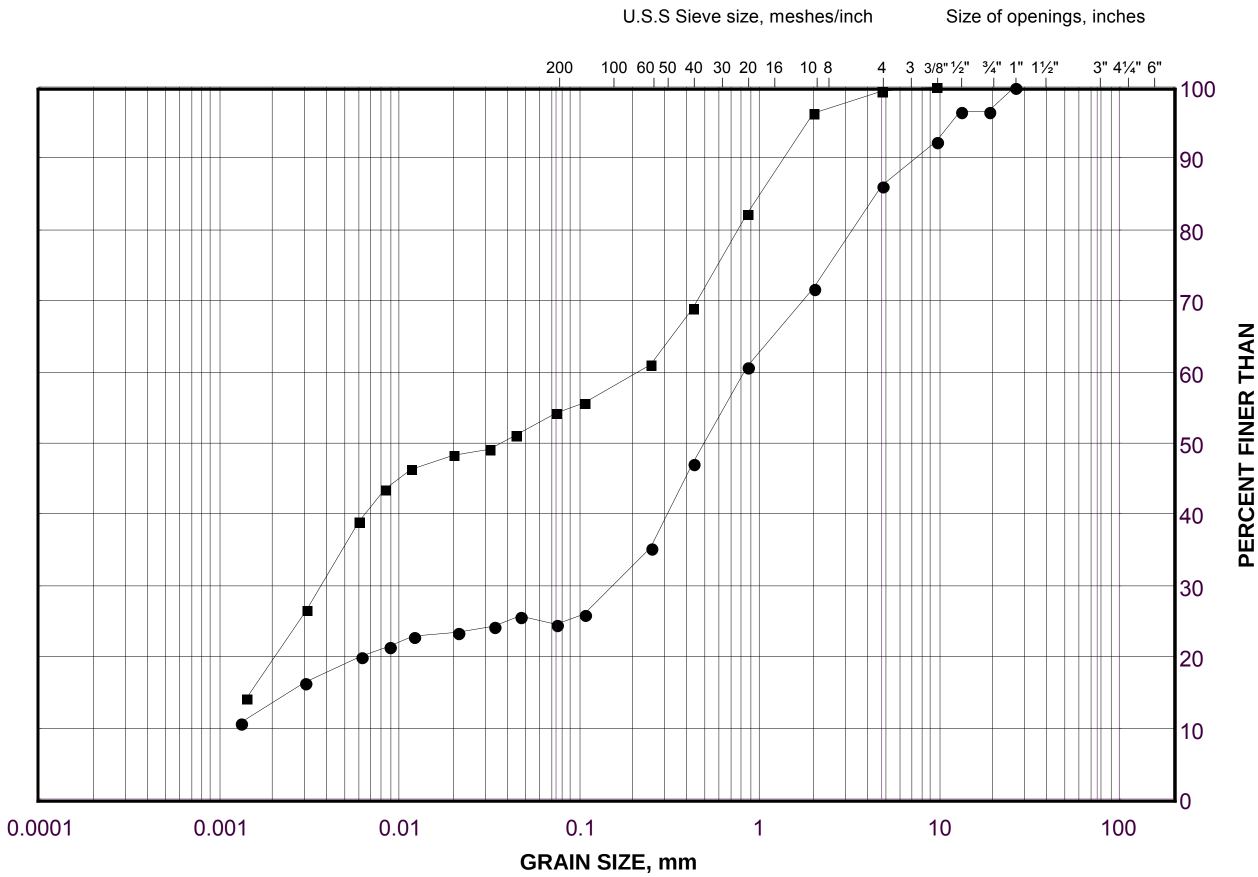
Date: 08-Aug-13

GRAIN SIZE DISTRIBUTION Clayey Silt with Sand	FIGURE B6



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	B3-03	12B	313.5
■	B3-02	13	312.2

Project Number: 11-1111-0149

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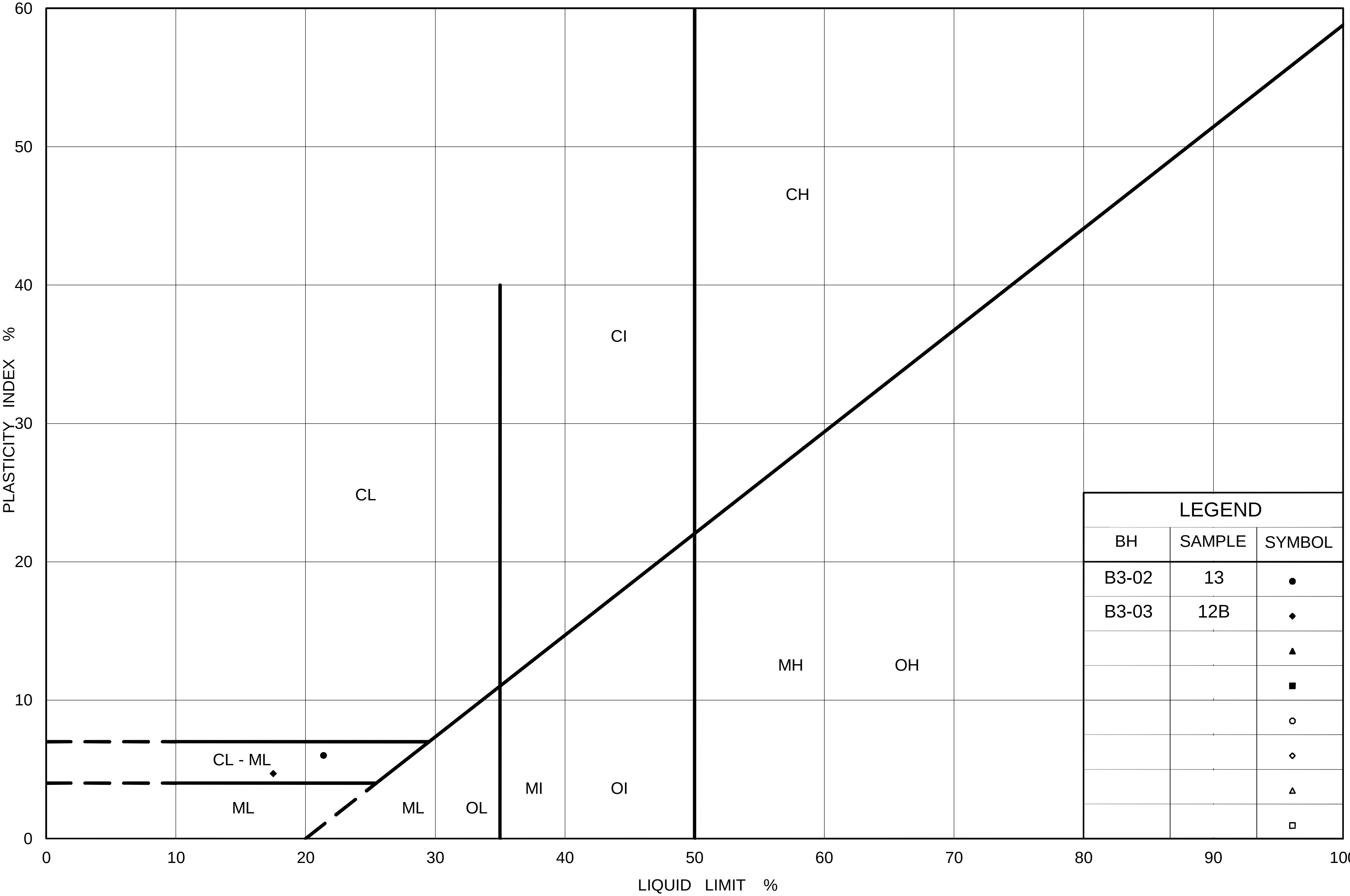
Date: 08-Aug-13



LEGEND

BH	SAMPLE	SYMBOL
B3-02	13	●
B3-03	12B	◆
		▲
		■
		○
		◇
		△
		□

Ministry of Transportation

Ontario

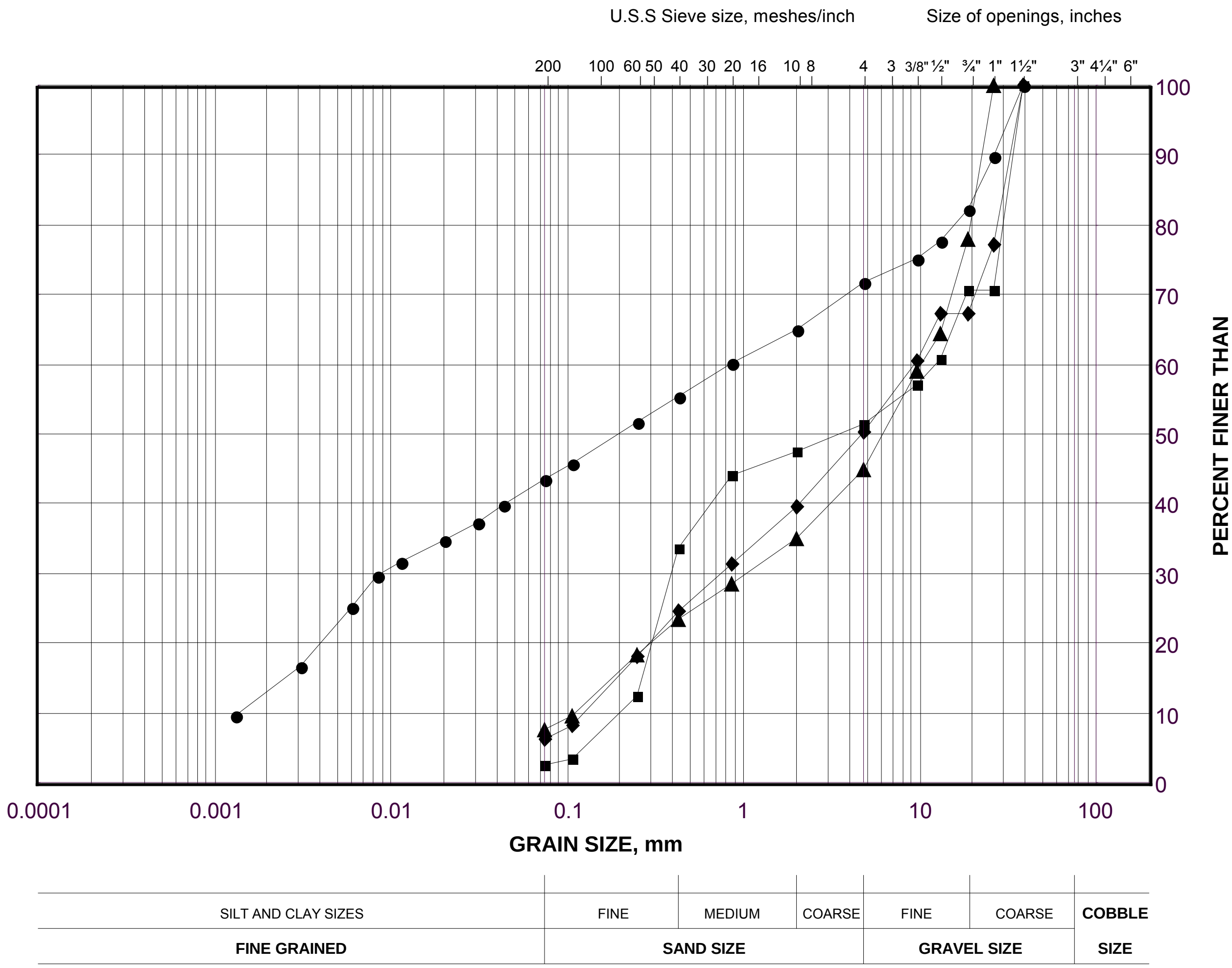
PLASTICITY CHART
Clayey Silt with Sand

Figure No. B7

Project No. 11-1111-0149

Checked By: MCK/TVA

GRAIN SIZE DISTRIBUTION Gravelly Sandy Silt to Sand and Gravel	FIGURE B8



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	B3-03	14	310.6
■	B3-02	17	304.6
◆	B3-03	19	295.3
▲	B3-02	19	298.5

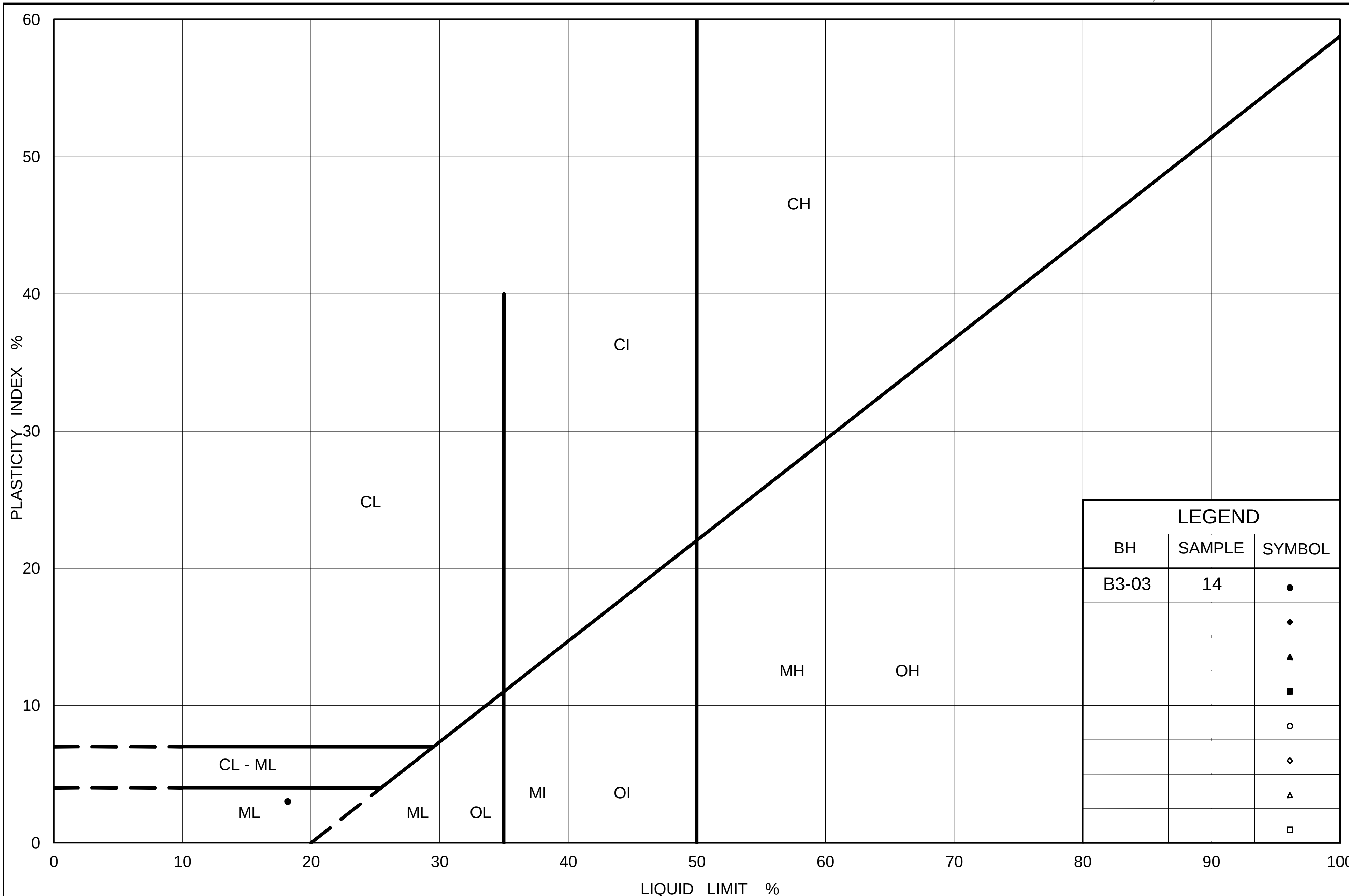
Project Number: 11-1111-0149

Checked By: MCK/TVA

Golder Associates

Date: 08-Aug-13

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART

Sandy Silt

Figure No. B9

Project No. 11-1111-0149

Checked By: MCK/TVA

APPENDIX C

Non-Standard Special Provisions

OBSTRUCTIONS

Special Provision

SCOPE

Cobbles and boulders were encountered within the gravelly sandy silt to sand and gravel deposit during advancement of the boreholes. Consideration of the presence of these obstructions must be made in the selection of appropriate equipment and procedures for 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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Europe	+ 356 21 42 30 20
North America	+ 1 800 275 3281
South America	+ 55 21 3095 9500

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