



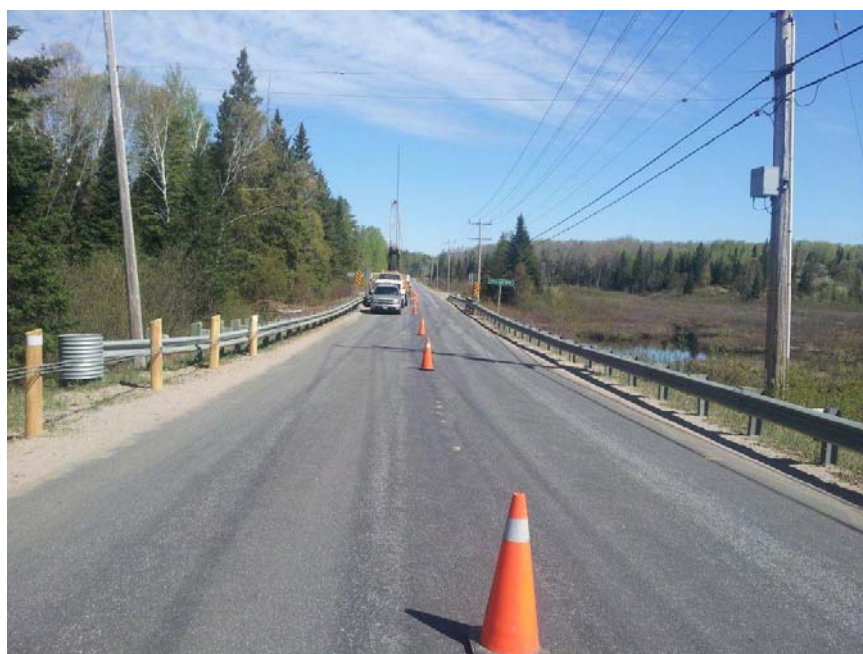
December 23, 2013

FOUNDATION INVESTIGATION AND DESIGN REPORT

**LITTLE EAST RIVER BRIDGE NO. 2, SITE NO. 44-175
HIGHWAY 592 - REPLACEMENT OF SIX STRUCTURES
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5265-07-00 WP 5266-07-01**

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REPORT

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PART A

FOUNDATION INVESTIGATION REPORT

LITTLE EAST RIVER BRIDGE NO. 2 – SITE No. 44-175

HIGHWAY 592 – REPLACEMENT OF SIX STRUCTURES

MINISTRY OF TRANSPORTATION, ONTARIO

GWP 5265-07-00; WP 5266-07-01



1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Morrison Hershfield Limited. (MH) on behalf of the Ministry of Transportation, Ontario (MTO) to provide detail foundation engineering services for the replacement of Little East River Bridge No. 2 (Site No. 44-175) 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. 2 is located approximately 400 m north of Savage Settlement Road and approximately 1.7 km north of the Highway 11/Novar Road interchange in Novar, Ontario. The location of the existing bridge structure along Highway 592 is shown on the Key Map on Drawing 1.

The Terms of Reference (TOR) for the foundation investigation are outlined in MTO's Request for Proposal, dated September 2011. Golder's proposal (Scope of Work) for foundation engineering services associated with the Little East River Bridge No. 2 structure is contained in Section 6.8 of MH's Technical Proposal of this assignment. The work was carried out in accordance with Golder's Project Specific Supplementary Specialty Plan for foundation engineering services, dated March 21, 2012.

This report addresses the investigation carried out for the Little East River Bridge No. 2 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, rock coring, in situ testing and laboratory testing on selected soil and rock core samples. The borehole locations for this investigation were surveyed in the field 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 area to the north and residential/recreational properties to the south with Little East River flowing westerly at this location. The existing ground surface within the limits of the proposed structure and approach embankments is between about Elevations 326.5 m and 326.3 m, referenced to Geodetic datum. The existing Highway 592 south and north approach embankments along the centreline are at Elevations 326.4 m and 326.3 m, respectively.

3.0 INVESTIGATION PROCEDURES

3.1 Foundation Investigation

The field work for the proposed bridge structure was carried out between May 7 and 14, 2013 during which time a total of four boreholes were advanced at the location of the structure foundation footprints and approach embankments. In addition, a Dynamic Cone Penetration Test (DCPT) was advanced immediately adjacent to Borehole B2-03 and subsequently augered to a specified depth to install a piezometer. A DCPT was advanced



FOUNDATION REPORT - LITTLE EAST RIVER BRIDGE NO.2 - HIGHWAY 592 GWP 5265-07-00; WP 5266-07-01

from the bottom of Borehole B2-04 to determine the depth to refusal. A summary of the respective boreholes advanced at each foundation element and approach embankment is presented below.

Foundation Unit	Borehole
South Approach Embankment	B2-01
South Abutment	B2-02
North Abutment	B2-03
North Approach Embankment	B2-04

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

The field borehole investigation was carried out using a truck-mounted CME 55 drill rig supplied and operated by Landcore Drilling of Chelmsford, Ontario. The boreholes were advanced through the overburden using 120 mm and 203 mm outer diameter (O.D.) continuous flight hollow-stem augers and 'NW' casing. Soil samples were obtained at intervals of depth of about 0.75 m and 3.0 m, using a 50 mm O.D. split-spoon sampler driven by an automatic hammer in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586 – Standard Test Method for Standard Penetration Test). Cobbles and/or boulders and bedrock samples were recovered using an 'NQ' size rock core barrel and photographs of the recovered core samples are provided in Appendix B. The boreholes and DCPTs were advanced to depths of up to about 24.4 m (including coring of bedrock for core length of about 3.4 m) and 19.3 m below existing ground surface, respectively.

The groundwater conditions in the open boreholes were observed during and upon completion of drilling operations, and a standpipe piezometer was installed in a borehole immediately adjacent to Borehole B2-03 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 sealed at a select depth within the borehole. The borehole and annulus surrounding the piezometer pipe above the screen and sand pack were backfilled to the surface with bentonite pellets/grout. Piezometer installation details and water level readings are described on the Record of Borehole sheets in Appendix A. All open boreholes were backfilled with cement grout by tremie technique upon completion and the piezometer in Borehole B2-03 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 and rock samples. The soil and rock samples were identified in the field, placed in appropriate containers, labelled and transported to our Mississauga geotechnical laboratory where the 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. Strength testing, such as uniaxial (unconfined) compression and point load index, was carried out on selected specimens of the rock core. The results of the laboratory testing are included in Appendix B.

Classification of the mass quality of the bedrock with respect to the Rock Quality Designation (RQD) is described based on Table 3.10 of the Canadian Foundation Engineering Manual (CFEM, 2006)¹ while the strength of the

¹ Canadian Geotechnical Society, 2006. Canadian Foundation Engineering Manual, 4th Edition.



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bedrock core samples is based on Table 3.5 of CFEM (2006). The degree of weathering of the bedrock samples and the strength classification of the intact rock mass based on field identification are described in accordance with Table B.3 and Table B.6, respectively, of the International Society for Rock Mechanics (ISRM, 1985)² standard classification system

The as-drilled borehole locations and ground surface elevations were surveyed by Tulloch. The locations given in the Record of Borehole/Drillhole 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		
B2-01	5035575.8	324374.4	326.4 m	9.8 m
B2-02	5035596.1	324376.8	326.4 m	24.3 m
B2-03	5035607.5	324372.2	326.5 m	24.4 m / 19.3 m
B2-04	5035627.7	324374.7	326.3 m	9.8 m / 19.3 m

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

As delineated in *The Physiography of Southern Ontario*³, 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)⁴.

4.2 Subsurface Conditions

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

² International Society for Rock Mechanics Commission on Test Methods, 1985. Int. J. Rock Mech.Min. Sci. & Geomech. Abstr. Vol 22, No. 2, pp. 51-60.

³ 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.

⁴ 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.



subsurface conditions will vary between and beyond the borehole locations. It should be noted that the interpreted stratigraphy shown on Drawing 2 is a simplification of the subsurface conditions.

In general, the subsurface conditions in the area of the proposed bridge structure consist of a surficial layer of asphalt over a deposit of fill associated with the Highway 592 embankments. The fill is underlain by a deposit of sand and gravel containing pockets of organic sand and silt to clayey silt. The sand and gravel deposit is underlain by deposits of sand to silt and sand, which in turn is underlain in places by a lower deposit of sand and gravel and in places by granitic gneiss bedrock.

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

4.2.1 Asphalt

An approximately 25 mm to 40 mm thick layer of asphalt was encountered at the ground surface in all boreholes.

4.2.2 Silty Sand/ Sand to Sand and Gravel (Fill)

A non-cohesive deposit of fill comprised of brown to grey silty sand to sand to sand and gravel was encountered in all boreholes below the asphalt layer. The deposit occasionally contains trace clay, trace organics and silt seams. The top of the fill deposit is at about Elevations 326.5 m and 326.3 m and the thickness of the deposit ranges from about 2.2 m to 3.0 m.

The SPT 'N'-values measured within the non-cohesive fill deposit range from 3 blows to 29 blows per 0.3 m of penetration, indicating a very loose to compact relative density.

The natural water content measured on five samples of the fill ranges from about 7 per cent to 19 per cent.

The results of grain size distribution tests completed on three samples of the fill deposit are shown on Figure B1 in Appendix B.

4.2.3 Sand and Gravel (Upper Deposit)

An upper deposit of grey sand and gravel was encountered below the fill in Boreholes B2-02 to B2-04 and below a 0.8 m thick organic sand pocket in Borehole B2-01 and a 0.7 m thick clayey silt pocket in Borehole B2-04. The deposit generally contains trace to some silt and occasional silt seams. The deposit contains layers or pockets of silt to clayey silt up to about 1.6 m and 2.1 m thick at the location of south abutment and approach, in Boreholes B2-01 and B2-02. Cobbles and boulders were encountered within the lower portion of the deposit at about or below Elevation 319.0 m in Boreholes B2-02 and B2-03. The top of the deposit ranges between Elevations 324.2 m and 322.6 m and the thickness of the deposit ranges from 1.9 m to 7.6 m, including the thickness of the silt to clayey silt layers and organic sand pocket. Borehole B2-01 was terminated within this deposit at a depth of 9.8 m below ground surface, corresponding to Elevation 316.6 m.

The SPT 'N'-values measured within the organic sand pocket and the overall sand and gravel deposit generally range from 6 blows to 44 blows per 0.3 m of penetration, indicating a loose to dense relative density. In



Borehole B2-03, an SPT 'N'-value as high as 39 blows per 0.08 m of penetration was recorded prior to split-spoon refusal on cobbles within the deposit.

The natural water content measured on eight samples of this overall non-cohesive deposit ranges from about 5 per cent to 21 per cent and is about 32 per cent in the organic sand pocket.

The results of grain size distribution tests completed on three samples of this deposit are shown on Figure B2 in Appendix B.

4.2.4 Clayey Silt to Silt (Pockets)

An approximately 0.7 m thick pocket of clayey silt was encountered immediately below the fill deposit in Borehole B2-04 and approximately 1.6 m to 2.1 m thick pockets of clayey silt to silt, some gravel, trace to some sand were encountered within the upper deposit of sand and gravel in Boreholes B2-01 and B2-02. The top of the clayey silt to silt pockets ranges between Elevations 323.3 m and 320.8 m.

The SPT 'N'-values measured within the clayey silt pockets range between 9 blows and 14 blows per 0.3 m of penetration, suggesting a stiff consistency; and the SPT 'N'-values measured within the silt pockets are 3 blows and 12 blows to per 0.3 m of penetration, indicating a very loose to compact relative density.

The natural water content measured on two samples of the clayey silt pockets are about 27 per cent and 34 per cent and on a sample of the silt pocket is about 24 per cent.

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

An Atterberg limits test carried out on a sample of the silt pocket measured a liquid limit of about 24 per cent, a plastic limit of about 18 per cent and corresponding plastic index of about 2 per cent, indicating the material is classified as silt of slight plasticity. Atterberg limits tests carried out on two samples of the clayey silt pockets measured liquid limits of about 27 per cent and 29 per cent, plastic limits of about 18 per cent and 21 per cent and plasticity indices of about 7 per cent and 8 per cent, indicating that the material is classified as clayey silt of low plasticity. The Atterberg limits test results are shown on plasticity chart on Figure B4 in Appendix B.

4.2.5 Silt and Sand to Sand

A deposit of grey silt and sand to sand was encountered below the upper sand and gravel deposit in Boreholes B2-02 to B2-04. The deposit contains trace to some silt, trace clay and generally transitions from an upper deposit of sand to a lower deposit of silty sand to silt and sand. In Borehole B2-02, cobbles were encountered between depths of 19.5 m and 20.9 m. The top of the deposit ranges from Elevations 320.7 m to 317.7 m and the thickness of the deposit ranges between 4.2 m and 12.2 m.

The SPT 'N'-values measured within the silt and sand to sand deposit range from 5 blows to 18 blows per 0.3 m of penetration, indicating a loose to compact relative density.

The natural water content measured on seven samples of this deposit ranges from about 20 per cent to 32 per cent.



The results of grain size distribution tests completed on five samples of this deposit are shown on Figure B5 in Appendix B.

4.2.6 Sand and Gravel (Lower Deposit)

An approximately 7.3 m thick deposit of sand and gravel, trace to some silt was encountered below the silt and sand to sand deposit in Borehole B2-03 and is inferred present from a DCPT for a thickness of about 3 m underlying the sand deposit in Borehole B2-04. In Borehole B2-03, cobbles and boulders were encountered between depths of 19.4 m and 21.3 m as well as 21.7 m and 24.4 m. The top of this deposit was encountered at Elevation 309.4 m in Borehole B2-03 and inferred at Elevation 310.0 m in the DCPT in Borehole B2-04 and the deposit terminated on split-spoon sampler and DCPT refusal.

SPT 'N'-values of 17 blows and 54 blows per 0.3 m of penetration were measured within this deposit, indicating a compact to very dense relative density.

The natural water content measured on a sample of the deposit is about 11 per cent.

The result of a grain size distribution test completed on a sample of this deposit is shown on Figure B6 in Appendix B.

4.2.7 Refusal/Bedrock

Refusal to further split-spoon and DCPT advancement was encountered in Boreholes B2-03 and B2-04, respectively, at depths of about 24.4 m and 19.3 m below ground surface, corresponding to Elevations 302.1 m and 307.0 m, respectively.

Bedrock was encountered and about 3.4 m of core samples were recovered from Borehole B2-02. The bedrock surface was encountered at about Elevation 305.5 m corresponding to a depth of about 20.9 m below ground surface.

Based on a review of the bedrock core samples, the bedrock consists of granitic gneiss. In general, the bedrock samples are described as fresh, medium to coarse crystalline, medium strong to very strong, light greyish-pink, with bands and lenses of mafic rich minerals, as presented in the Record of Drillhole sheet in Appendix A, and shown on the photograph of the recovered core samples on Figure B7 in Appendix B.

The Rock Quality Designation (RQD) measured on the core samples ranges from about 95 per cent to 100 per cent, indicating a rock mass of excellent quality. The Total Core Recovery (TCR) of the core samples recovered is about 100 per cent and Solid Core Recovery (SCR) of the core samples recovered is between about 96 per cent and 100 per cent.

Point load strength index tests (ASTM D5731 – Standard Test Method for Determination of the Point Load Strength Index of Rock and Application to Rock Strength Classification) were carried out on selected samples of the bedrock core. The axial and diametral point load strength index values are shown on the Record of Drillhole sheet and are presented in Table B1 in Appendix B. The axial tests carried out on four (4) samples of the bedrock core measured Is_{50} values ranging from about 6.7 MPa to 15.2 MPa and the diametral tests carried out on four (4) samples of the bedrock core measured Is_{50} values ranging from about 1.7 MPa to 4.8 MPa.



An Unconfined Compression (UC) test (ASTM D7012 – Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens) was carried out on a selected core sample of the bedrock and measured a uniaxial compressive strength of about 140 MPa. The details of UC test is presented in Table B2 in Appendix B.

Also presented in Table B1 are the estimated Uniaxial Compressive Strength (UCS) values for each sample tested for point load strength index based on a relationship between Is_{50} of the axial point load tests and UCS, which is given by a correlation factor (K) which varies depending on the size of the core sample and the strength of the rock. For this site, the UCS values are based on an estimated average correlation factor (K) of 13.

Based on the laboratory UC test and the point load index test results the approximate uniaxial compressive strength of the bedrock range from 22 MPa to 198 MPa but is generally greater than 41 MPa, and the granitic gneiss bedrock is generally classified as medium strong (R3, 25 MPa < UCS < 50 MPa) to very strong (R5, 100 MPa < UCS < 250 MPa) with one diametral test indicating weak (R2, 5 MPa < UCS < 25 MPa) rock.

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 about Elevations 324.5 m to 323.7 m measured at between about 1.8 m and 2.7 m below ground surface.

A standpipe piezometer was installed in Borehole B2-03 to allow monitoring of the groundwater level at the site. Details of the piezometer installation are shown on the Record of Borehole No. B2-03 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
B2-03	326.5 m	2.1 m	324.4 m	May 9, 2013
		2.0 m	324.5 m	May 10, 2013
		2.7 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 Mr. Al Varshoi, M.E.Sc., and reviewed by Ms. T. Veronica Ayetan, P.Eng., a geotechnical engineer with Golder. Mr. Jorge M. A. Costa, P.Eng., Golder's Designated MTO Contact for this project and Principal with Golder, conducted an independent quality control review of the report.



Report Signature Page

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PART B

FOUNDATION DESIGN REPORT

LITTLE EAST RIVER BRIDGE NO. 2 – Site No. 44-175

HIGHWAY 592 – REPLACEMENT OF SIX STRUCTURES

MINISTRY OF TRANSPORTATION, ONTARIO

GWP 5265-07-00; WP 5266-07-01



6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides engineering design recommendations for the proposed Little East River Bridge No. 2 on Highway 592 (Site No.44-175). 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. 2 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. 2 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 326.6 m, which corresponds to a raise of the existing approach embankments of up to about 0.3 m.

6.2 Foundation Options

Given that loose to compact granular deposits are present in the areas of the abutments down to below the depth of frost penetration, the relatively shallow depth to the groundwater table and proximity to the adjacent river, a shallow foundation support system is not considered to be the preferred alternative for the support of the structure, although such foundations could be considered.

Given that: bedrock was not encountered to the depths drilled at the north abutment; cobbles and/or boulders were encountered within the silt and sand/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 for the north abutment will be relatively low as the H-piles and steel casings will develop capacities through friction only within the loose to compact granular deposits.

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

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



6.3 Spread Footings

Shallow foundations comprised of spread/strip footings founded on native overburden could be considered for the support of the proposed structure. However, given the loose to compact nature of the sand and gravel deposit down to immediately below the depth of the frost penetration, geotechnical axial capacity (specifically, at Serviceability Limits States (SLS) for 25 mm of settlement) will be low.

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 compact sand and gravel underlain by a deposit of very loose silt at the south abutment and by a deposit of compact to dense sand and gravel at the north abutment) at Elevation 322.7 m at the abutments, the factored geotechnical axial resistance at Ultimate Limits States (ULS) and geotechnical reaction at Serviceability Limits States (SLS) for 25 mm of settlement are provided below.

Foundation Location	Factored Geotechnical Axial Resistance at ULS	Geotechnical Reaction at SLS for 25 mm of Settlement
South Abutment	200 kPa	70 kPa
North Abutment	675 kPa	110 kPa

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

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

6.3.2 Resistance to Lateral Loads

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

Interface Material(s)	Coefficient of Friction ($\tan \delta'$)
Concrete footing on compact sand and gravel	0.45

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

Steel H-piles end-bearing on bedrock at the south abutment and friction steel H-piles driven into the compact to very dense silt and sand/sand and gravel deposits at the north abutment could be considered for the support of the proposed structure. However, cobbles and boulders were encountered within the granular deposits 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.



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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 Abutment (End-bearing pile)	322.7 m	321.5 m	305.5 m ³	17.2 m	1,600 kN ⁴	N/A
North Abutment (Friction pile)	322.7 m	321.5 m	302.1 m	20.6 m	950 kN	N/A

Notes:

1. As per the GA Drawing provided by MH on November 6, 2013.
2. The geotechnical reaction at SLS for 25 mm of settlement will be greater than or equal to the factored geotechnical axial resistance at ULS and therefore, the SLS condition does not apply.
3. Elevation of bedrock as encountered in Borehole B2-02.
4. A reduction factor of 0.8 is applied to account for possible refusal of piles on cobbles overlying the bedrock.

Taking into consideration the possibility of encountering refusal on cobbles and boulders within the silt and sand/sand and gravel deposits, 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 OPSS 3000.100 (*Steel H-Pile Driving Shoe*) to minimize damage to the pile during driving and penetration through the granular deposits containing cobbles and boulders.

6.4.2 Set Criteria

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

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

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

The following pile driving notes should be added to the Contract Drawings (i.e. Notes 2 and 5 in Clause 3.3.3 of the Structural Manual (MTO, 2008)):

At the south abutment (Note 5):

- Piles to be driven to bedrock.

At the north abutment (Note 2):



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- Piles to be driven in accordance with Standard Drawing SS 103-11 using an ultimate geotechnical resistance of 2,250 kN per pile, but must be driven below El. 302.1 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 silt and sand/sand and gravel deposits, it is recommended that a down-the-hole (DTH) hammer drilling system be used for the installation of the drilled steel casing. However, due to the proposed construction sequencing/staging, narrow right-of-way and the presence of overhead Hydro lines along the east side of the bridge, there may not be adequate construction platform width to accommodate drilling equipment necessary to advance long steel casing to achieve the desired geotechnical axial capacities for design.

6.5.1 Geotechnical Axial Resistance and Reaction

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

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 Abutment	322.7 m	321.5 m	304.0 m ³	18.7 m	4,250 kN	N/A
North Abutment	322.7 m	321.5 m	302.7 m	20 m	1,575 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.
3. Based on steel casing socketed 1.5 m into the bedrock.

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

6.5.2 Frost Protection

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



6.6 Micropiles

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

- Micropiles can readily penetrate through cobbles and boulders in the overburden;
- Micropile drilling equipment is relatively small (for use in confined spaces and/or low headroom situations) as compared to pile-driving and/or casing-drilling equipment; and,
- Higher geotechnical resistances can be achieved from micropiles when compared to the same length of driven H-pile and drilled steel casing.

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

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

6.6.1 Geotechnical Axial Resistance and Reaction

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

Foundation Location	Elevation of Underside of Pile Cap ¹	Elevation of Underside of Tremie Plug ¹	Micropile Tip Elevation	Diameter of Micropile	Length of Micropile from Underside of Pile Cap
South and North Abutment	322.7 m	321.5 m	307.5 m	273 mm	15.2 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:

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

where:

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



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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 (B2-02)	Very Loose Silt	321.5 m to 320.8 m	3,000 kPa/m	-
	Stiff Clayey Silt	320.8 m to 319.8 m	-	50 kPa
	Loose Sand and Gravel	319.8 m to 317.7 m	12,000 kPa/m	-
	Loose to Compact Sand to Silt and Sand	317.7 m to 305.5 m	8,000 kPa/m	-
North Abutment (B2-03)	Compact to Dense Sand and Gravel	321.5 m to 317.7 m	12,000 kPa/m	-
	Compact Silt and Sand to Sand	317.7 m to 309.4 m	8,000 kPa/m	-
	Compact to Very Dense Sand and Gravel	309.4 m to 302.1 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 Abutment	HP 310 x 110	1,600 kN	35 kN	30 kN
	610 mm diameter drilled steel casing	4,250 kN	15 kN	N/A ²
North Abutment	HP 310 x 110	950 kN	75 kN	30 kN
	610 mm diameter drilled steel casing	1,575 kN	55 kN	35 kN

Note:

- Analyses assume a fixed-head condition.
- The geotechnical reaction at SLS for 10 mm of deflection will be greater than or equal to the factored geotechnical lateral resistance at ULS and therefore, the SLS condition does not apply.

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,



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

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

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

Fill Type	Soil Unit Weight	Coefficients of Static Lateral Earth Pressure	
		At-Rest, K_o	Active, K_a
Rock Fill	19 kN/m 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. 2 structure will be at about Elevation 326.6 m, requiring placement of up to about 0.3 m of fill to raise the existing south approach embankment grade and no grade raise at the existing north approach embankment.

Based on the investigated locations at this site, the south approach embankment is founded on a deposit of loose to dense sand and gravel deposit containing pockets of organic sand and clayey silt to silt, underlain by a deposit of loose to compact silt and sand to sand overlying bedrock, while the north approach embankment is founded on an upper deposit of sand and gravel containing pockets of clayey silt, underlain by a deposit of loose to compact silt and sand to sand, which is in turn underlain by a deposit of compact to very dense lower sand and gravel. Where excavations are not required, it is understood that the organic sand deposit and the existing fill materials will remain in place.

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

6.10.1 Stability

6.10.1.1 Methodology

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

6.10.1.2 Parameter Selection

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

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

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



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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	21 kN/m ³	-	0 kPa	34°
	New Rock Fill	19 kN/m ³	-	0 kPa	40°
	Existing Loose to Compact Sand to Sand and Gravel Fill	20 kN/m ³	-	0 kPa	30°
	Loose Organic Sand	18 kN/m ³	-	0 kPa	27°
	Compact to Dense Sand and Gravel (Upper Deposit)	20 kN/m ³	-	0 kPa	32°
	Very Loose Silt	18 kN/m ³	-	0 kPa	28°
	Stiff Clayey Silt	17 kN/m ³	50 kPa	-	-
	Loose to Compact Silt and Sand to Sand	19 kN/m ³	-	0 kPa	28°
North Approach Embankment	New Granular Fill	21 kN/m ³	-	0 kPa	34°
	New Rock Fill	19 kN/m ³	-	0 kPa	40°
	Existing Very Loose to Compact Silty Sand to Sand and Gravel Fill	20 kN/m ³	-	0 kPa	30°
	Stiff Clayey Silt	17 kN/m ³	50 kPa	-	-
	Compact to Dense Sand and Gravel (Upper Deposit)	20 kN/m ³	-	0 kPa	32°
	Loose to Compact Silt and Sand to Sand	19 kN/m ³	-	0 kPa	28°
	Compact to Very Dense Sand and Gravel (Lower Deposit)	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.



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Embankment	Embankment Height at Critical Section ¹	Side Slope Profile	Minimum Factor of Safety
South Approach Embankments	2.4 m	2H:1V (West Side) 1.25H:1V (Rock Fill, East Side)	≥ 1.3
North Approach Embankment	3.0 m	2H:1V (West Side) 1.25H:1V (Rock Fill East Side)	≥ 1.3

Note:

1. Embankment height includes an approximately 0.3 m high grade raise at the south approach embankment.

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); 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 21.4 m and as such, the estimated settlements represent the maximum value along the approach embankments.

6.10.2.2 Parameter Selection

The following presents the simplified stratigraphy and the associated unit weights and strengths employed for the estimation of settlement of the foundation soils at the approach embankment areas. The immediate compression of the non-cohesive overburden soils were modelled by estimating an elastic modulus of deformation based on the SPT 'N'-values and using correlations proposed by Bowles (1984) and Kulhawy and Mayne (1990). These estimated values were compared with the typical range of expected values for similar soil types, as outlined in *CHBDC* and adjusted, as appropriate. Due to the relatively thin clayey silt deposit encountered overlying the sand and gravel to sandy gravel deposit, the compression of the cohesive deposit was modelled by estimating a coefficient of volume compressibility based on the SPT 'N'-values and engineering judgement. For the purpose of the settlement analyses, it is assumed that the settlement of the existing fill materials is negligible.

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



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Embankment	Soil Type	Thickness	Unit Weight, γ	Elastic Modulus, E'	Coefficient of Volume Compressibility, m_v
South Approach Embankment	Existing Loose to Compact Sand to Sand and Gravel Fill	~2.2 m	20 kN/m ³	5 MPa	-
	Loose Organic Sand	~0.8 m	18 kN/m ³	5 MPa	-
	Compact to Dense Sand and Gravel (Upper Deposit)	2.1 m to 2.6 m ¹	20 kN/m ³	20 MPa	-
	Very Loose Silt	1.1 m to 1.6 m	18 kN/m ³	3 MPa	-
	Stiff Clayey Silt	~1 m	17 kN/m ³	-	5 x 10 ⁻⁴ kPa ⁻¹
	Loose to Compact Silt and Sand to Sand	~12.7 m	19 kN/m ³	5 MPa to 13 MPa	-
North Approach Embankment	Existing Very Loose to Compact Silty Sand to Sand and Gravel Fill	~3.0 m	20 kN/m ³	5 MPa	-
	Stiff Clayey Silt	~0.7 m	17 kN/m ³	-	5 x 10 ⁻⁴ kPa ⁻¹
	Compact to Dense Sand and Gravel (Upper Deposit)	2.9 m to 5.8 m	20 kN/m ³	20 MPa	-
	Loose to Compact Silt and Sand to Sand	~8.3 m ¹	19 kN/m ³	5 MPa to 13 MPa	-
	Compact to Very Dense Sand and Gravel (Lower Deposit)	~7.3 m ¹	20 kN/m ³	20 MPa	-

Note:

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

For the purpose of settlement analyses, the groundwater level was assumed to be located on average at Elevation 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	Estimated Settlement of Foundation Soils
South Approach Embankment	10 mm to 20 mm
North Approach Embankment	5 mm to 15 mm

These settlements are expected to occur relatively quickly (i.e. within 15 days after construction) in response to the grade raise, based on the predominantly non-cohesive nature of the foundation soils.

6.10.2.4 Settlement of Rock Fill Embankment

It is understood that rock fill is to be used for the construction of the east side of the approach embankments where there is limited right-of-way to accommodate the embankment widening 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, in a controlled manner (i.e. not end-dumped) in accordance with SP 206S03 (Rock Excavation, Grading). According to MTO's Guideline for Rock Fill Settlement and Rock Fill Quantity Estimates (2010), the settlement of rock fill placed in this manner is expected to be nominal and the magnitude is estimated to be about 0.6 per cent (0.5 percent for short-term and 0.1 per cent for long-term) of the effective thickness of rock fill. As such, the estimated settlement of rock fill for the approach embankments is presented below.

Embankment	Thickness of Rock Fill Along East Slope	Estimated Settlement of Rock Fill		
		Short-Term	Long-Term	Total
South and North Approach Embankment	Up to about 2 m	10 mm	5 mm	15 mm

The majority of the settlement of the rock fill as estimated above is expected to occur during construction.

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 322.7 m and the underside of the tremie plug at Elevation 321.5 m, excavations up to about 5 m below deep 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 the 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 granular 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.



FOUNDATION REPORT - LITTLE EAST RIVER BRIDGE NO.2 -
HIGHWAY 592 GWP 5265-07-00; WP 5266-07-01

Report Signature Page

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- | | |
|------------|---|
| ASTM D1586 | Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils |
|------------|---|
- Commercial Software:
- LPILE Plus (Version 5.0) by Ensoft Inc.
- Settle^{3D} (Version 2.0) by Rocscience Inc.
- Slide (Version 6.0) by Rocscience Inc.
- Contract Design Estimating and Documentation (CDED):
- | | |
|--------------------------|--|
| Special Provision 105S21 | Amendment to OPSS 501 – Water Requirements and Quality Control for Compaction – Method B |
|--------------------------|--|



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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.

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

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

Ontario Occupational Health and Safety Act:

Ontario Regulation 213 Construction Projects (as amended)

Ontario Provisional Standard Drawing:

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

OPSD 3090.010 Foundation, Frost Penetration Depths for Southern Ontario

OPSD 3101.200 Walls – Abutment, Backfill – Rock

OPSD 3121.150 Walls – Retaining, Backfill – Minimum Granular Requirement

Ontario Provincial Standard Specification:

OPSS 501 Construction Specification for Compacting

OPSS 539 Construction Specification for Temporary Protection Systems

OPSS 804 Construction Specification for Seed and Cover

OPSS 903 Construction Specification for Deep Foundations

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

Ontario Water Resources Act:

Ontario Regulation 903 Wells (as amended)



TABLES



FOUNDATION REPORT - LITTLE EAST RIVER BRIDGE NO.2 - HIGHWAY 592 GWP 5265-07-00; WP 5266-07-01

Table 1: Evaluation of Foundation Alternatives

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks / Consequences
Spread/strip footings (11.5 m long by 2 m wide)	4	<ul style="list-style-type: none"> Relative ease of construction. 	<ul style="list-style-type: none"> Allows only for semi-integral abutment design. Axial capacity on the thick loose to compact sand and gravel deposit will be low. Excavation for footings will be below water table. Cofferdam (with concrete tremie plug) and unwatering will be required for construction of the footings within a dry excavation. 	<ul style="list-style-type: none"> Lower relative cost than driven pile, drilled steel casing and micropile foundation options. Additional cost for cofferdam construction and unwatering for construction of the footings. 	<ul style="list-style-type: none"> Large footings will be required to develop adequate axial capacity.
Driven steel H-piles (HP 310x110)	2	<ul style="list-style-type: none"> Negligible post-construction settlement. Higher axial capacity than spread/strip footings. Straight forward construction; except that site constraints may preclude use of pile driving equipment. 	<ul style="list-style-type: none"> Integral abutment design may not be possible due to constraints in achieving free length of pile to allow for lateral movement due to the presence of the tremie plug. The south abutment will be supported on end-bearing H-piles while the north abutment will be supported on friction H-piles, due to the potential for piles to "hang up" on cobbles and boulders. Reinforced pile tips and/or heavier pile section will be required for piles to penetrate through cobbles 	<ul style="list-style-type: none"> Higher relative cost than spread/strip footing foundation option. Higher cost associated with pile reinforcement and/or heavier pile section to advance the H-piles through cobbles and boulders. Additional cost for cofferdam construction and unwatering for construction of the pile cap. 	<ul style="list-style-type: none"> Potential for requirement to drive piles deeper at the north abutment to develop adequate axial capacity during construction. Potential difficulty driving piles through the cobbles and boulder present in the granular deposits. Potential for pile damage when driving through cobbles and boulders. May require additional construction platform width and/or temporary closure of the roadway to accommodate larger (pile driving) equipment. Overhead hydro lines will



FOUNDATION REPORT - LITTLE EAST RIVER BRIDGE NO.2 - HIGHWAY 592 GWP 5265-07-00; WP 5266-07-01

Table 1: Evaluation of Foundation Alternatives

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks / Consequences
			<p>and boulders.</p> <ul style="list-style-type: none"> ■ Piles cannot be battered for lateral resistance due 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. 		<p>need to be de-energized during portions of the piling operation.</p>
Drilled steel casings using DTH hammer drilling system (610 mm)	3	<ul style="list-style-type: none"> ■ Reduced number of deep foundation elements compared to steel H-piles. ■ DTH drilling can readily penetrate through cobbles and boulders in overburden. ■ Relatively straightforward construction; except that site constraints may preclude the use of drilling equipment. ■ Negligible post-construction settlement. 	<ul style="list-style-type: none"> ■ Allows only for semi-integral abutment design. ■ The south abutment will be supported on steel casings socketed into bedrock while the north abutment will be supported on friction steel casings as bedrock was not encountered to the depth drilled. ■ Drilling slurry will be required to balance 	<ul style="list-style-type: none"> ■ Higher relative cost than spread/strip footing and driven pile foundation options. ■ Additional cost for specialized drilling equipment. ■ Additional cost associated with the need for drilling slurry and temporary liners. ■ Additional cost for cofferdam construction 	<ul style="list-style-type: none"> ■ Potential for unbalanced head in liners during installation may result in base heave and possible loss of ground. ■ Specialized drilling equipment and/or method could be required to penetrate cobbles and boulders present in the granular deposits. ■ May require additional construction platform width and/or temporary closure



Table 1: Evaluation of Foundation Alternatives

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks / Consequences
			<p>groundwater pressures and minimize basal heave.</p> <ul style="list-style-type: none"> ■ Excavation for pile cap will be below water table. ■ Cofferdam (with concrete tremie plug) and unwatering will be required for construction of the pile cap within a dry excavation. ■ 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. 	and unwatering for construction of the pile cap.	<p>of the roadway to accommodate larger (drilling) equipment.</p> <ul style="list-style-type: none"> ■ Overhead hydro lines will need to be de-energized during portions of the drilling operation. ■ Drilling deep to found/socket casings on/into the bedrock could result in much longer casing or casings of varying lengths depending on the elevation of the bedrock.
Micropiles (273 mm diameter)	1	<ul style="list-style-type: none"> ■ 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 granular deposits. ■ Requires smaller drilling equipment as compared to steel casing drilling equipment. 	<ul style="list-style-type: none"> ■ Allows only for semi-integral abutment design. ■ Detail micropile design will be required. ■ Pile load tests required to confirm capacity for design. ■ Cofferdam and unwatering will be required for construction of the pile cap within a dry excavation. 	<ul style="list-style-type: none"> ■ Higher relative cost than footings and driven pile foundation options. ■ Additional cost associated with the detail micropile design. ■ Additional cost for specialized drilling equipment. ■ Additional cost for cofferdam construction and unwatering for construction of the pile cap. ■ Additional cost for the micropile pile load tests. 	<ul style="list-style-type: none"> ■ Few contractors have experience with soil-bonded micropile installation on MTO projects.



DRAWINGS



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

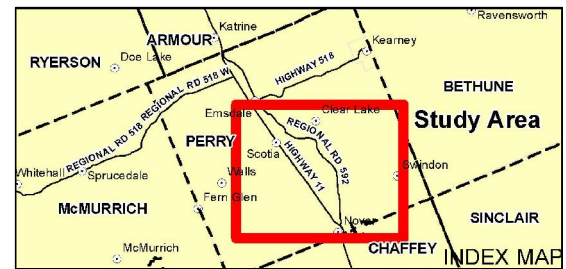
CONT No.
WP No. 5266-07-01

HIGHWAY 592
REPLACEMENT OF SIX STRUCTURES
KEY MAP

SHEET



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



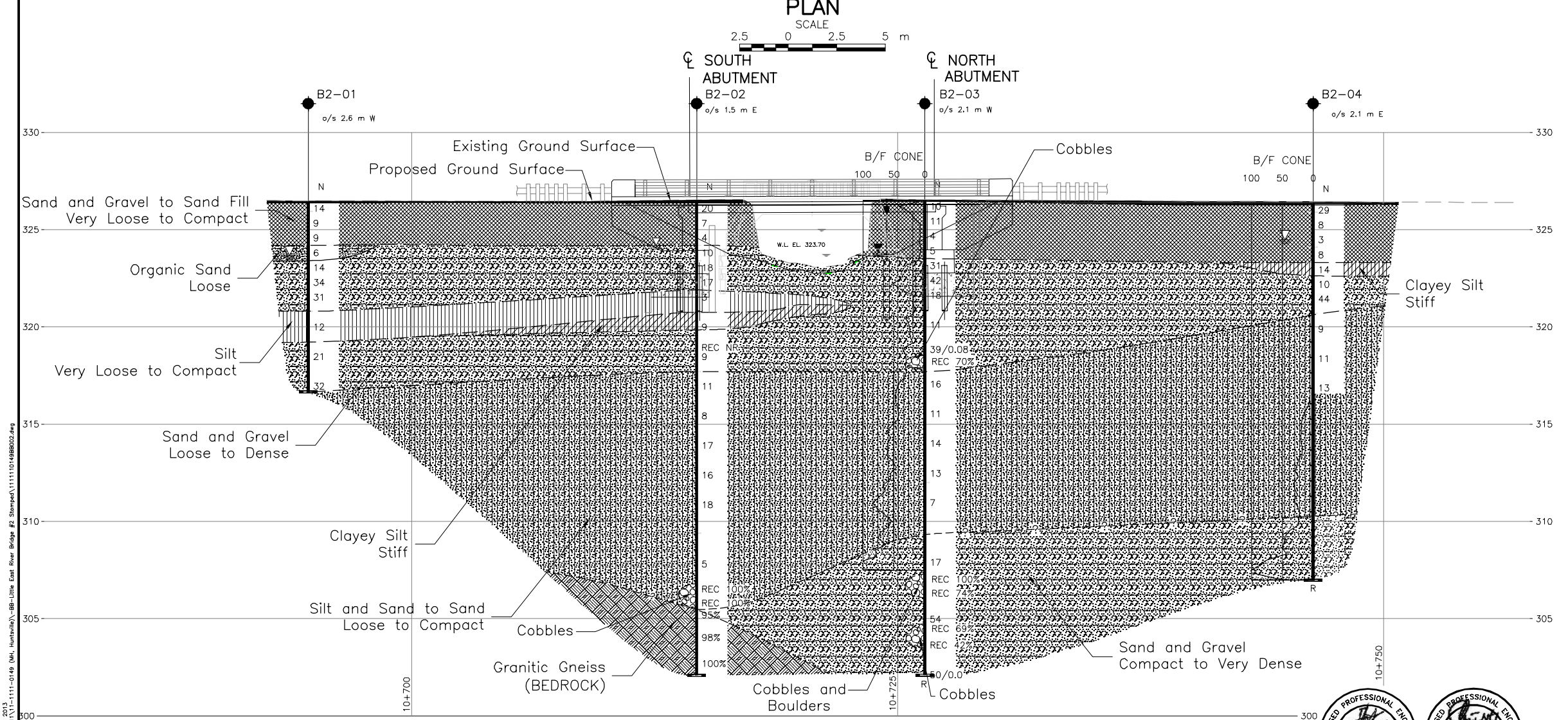
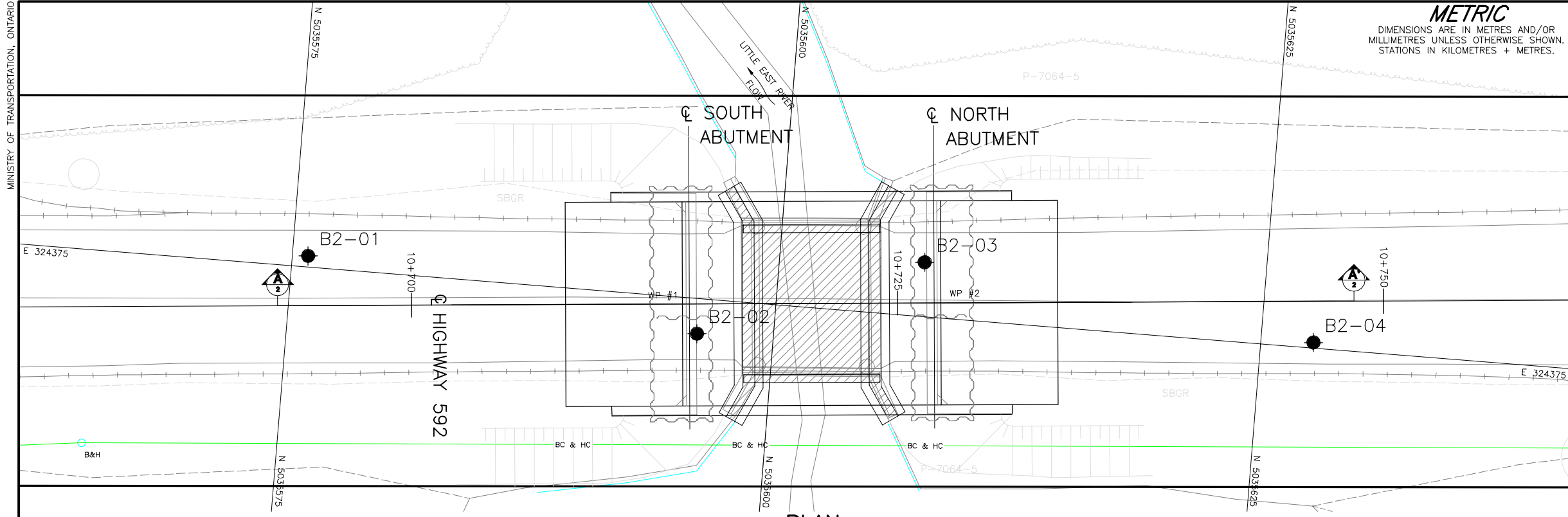
N.T.S



REFERENCE

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

NO.	DATE	BY	REVISION	
Geocres No. 31E-331				
HWY. 592		PROJECT NO. 11-1111-0149		DIST.
SUBM'D. AV		CHKD. CN	DATE: Dec. 2013	SITE:
DRAWN: JFC		CHKD.	APPD.	DWG. 1



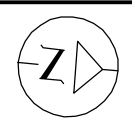
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METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

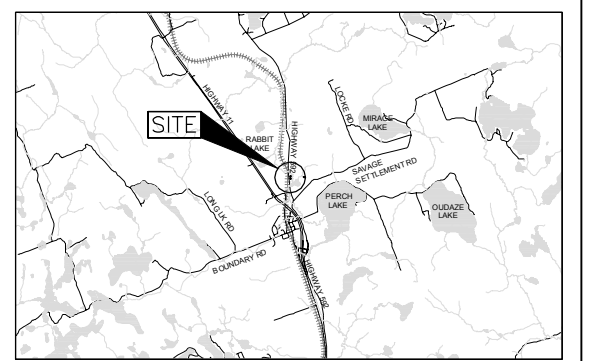
CONT No.
WP No. 5266-07-01

HIGHWAY 592
LITTLE EAST RIVER BRIDGE #2

BOREHOLE LOCATIONS AND SOIL STRATA



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN
SCALE
2 0 2 4 km

LEGEND	
	Borehole - Current Investigation
	Seal
	Piezometer
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
100%	Rock Quality Designation (RQD)
REC	Total core recovery
	WL in piezometer, measured on June 26, 2013
	WL upon completion of drilling
R	Refusal

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
B2-01	326.4	5035575.8	324374.4
B2-02	326.4	5035596.1	324376.8
B2-03	326.5	5035607.5	324372.2
B2-04	326.3	5035627.7	324374.7

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 Plan and Profile file no. 44175-01.dwg, received November 7, 2013.

NO.	DATE	BY	REVISION
Geocres No. 31E-331			
HWY. 592		PROJECT NO. 11-1111-0149	DIST.
SUBM'D. AV	CHKD. CN	DATE: Dec. 2013	SITE: 44-175
DRAWN: JFC	CHKD. TVA	APPD.	DWG. 2





APPENDIX A

Record of Borehole/Drillhole Sheets



LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_α	secondary compression index
m_v	coefficient of volume change
C_v	coefficient of consolidation (vertical direction)
C_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio = σ'_p / σ'_{vo}

(d) Shear Strength

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

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

Notes: 1
2

$\tau = c' + \sigma' \tan \phi'$
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, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

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

(b) Cohesive Soils Consistency

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

IV. SOIL TESTS

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

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

V. MINOR SOIL CONSTITUENTS

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



LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE

Fresh: no visible sign of weathering

Faintly weathered: weathering limited to the surface of major discontinuities.

Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

Moderately weathered: weathering extends throughout the rock mass but the rock material is not friable.

Highly weathered: weathering extends throughout rock mass and the rock material is partly friable.

Completely weathered: rock is wholly decomposed and in a friable condition but the rock and structure are preserved.

BEDDING THICKNESS

<u>Description</u>	<u>Bedding Plane Spacing</u>
Very thickly bedded	Greater than 2 m
Thickly bedded	0.6 m to 2 m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	Less than 6 mm

JOINT OR FOLIATION SPACING

<u>Description</u>	<u>Spacing</u>
Very wide	Greater than 3 m
Wide	1 m to 3 m
Moderately close	0.3 m to 1 m
Close	50 mm to 300 mm
Very close	Less than 50 mm

GRAIN SIZE

<u>Term</u>	<u>Size*</u>
Very Coarse Grained	Greater than 60 mm
Coarse Grained	2 mm to 60 mm
Medium Grained	60 microns to 2 mm
Fine Grained	2 microns to 60 microns
Very Fine Grained	Less than 2 microns

Note: * Grains greater than 60 microns diameter are visible to the naked eye.

CORE CONDITION

Total Core Recovery (TCR)

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varied from 0% for completely broken core to 100% for core in solid sticks.

DISCONTINUITY DATA

Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

Dip with Respect to Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90° angle is horizontal.

Description and Notes

An abbreviation description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes, or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

JN Joint	PL Planar
FLT Fault	CU Curved
SH Shear	UN Undulating
VN Vein	IR Irregular
FR Fracture	K Slickensided
SY Stylolite	PO Polished
BD Bedding	SM Smooth
FO Foliation	SR Slightly Rough
CO Contact	RO Rough
AXJ Axial Joint	VR Very Rough
KV Karstic Void	
MB Mechanical Break	

PROJECT		11-1111-0149		RECORD OF BOREHOLE No B2-01		SHEET 1 OF 1		METRIC								
W.P.		5265-07-01		LOCATION		N 5035575.8 ; E 324374.4		ORIGINATED BY								
DIST		HWY 592		BOREHOLE TYPE		203 mm O.D. Hollow Stem Augers and NW Casing		COMPILED BY								
DATUM		Geodetic		DATE		May 9, 2013		CHECKED BY								
								CN								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
326.4	GROUND SURFACE															
0.0	Asphalt (25 mm)		1A	SS	14											
325.9	Sand and gravel (FILL)		1B													
0.5	Compact Brown Moist															
	Sand, some silt, trace clay, trace organics, containing silt seams (FILL)		2	SS	9											
	Loose Brown Moist															
324.2			3	SS	9											
2.2	ORGANIC SAND, containing wood fragments and rootlets															
323.4	Loose Dark brown Wet		4	SS	6											
3.0	SAND and GRAVEL, containing silt seams															
	Compact to dense Grey Moist to wet		5	SS	14											
			6	SS	34											
			7	SS	31											
320.8																
5.6	SILT, some gravel, some clay, trace to some sand															
	Compact Grey Wet		8	SS	12											
319.2																
7.2	SAND and GRAVEL															
	Compact to dense Grey Wet		9	SS	21											
316.7			10	SS	32											
9.8	END OF BOREHOLE															
NOTE:																
1. Water level in open borehole at a depth of 2.7 m below ground surface (Elev. 323.7 m) upon completion of drilling.																

PROJECT		11-1111-0149		RECORD OF BOREHOLE No B2-02		SHEET 2 OF 2		METRIC									
W.P.		5266-07-01		LOCATION		N 5035596.1 ; E 324376.8		ORIGINATED BY		ID							
DIST		HWY 592		BOREHOLE TYPE		120 mm O.D. Hollow Stem Augers and NW Casing		COMPILED BY		GRL/AV							
DATUM		Geodetic		DATE		May 13 and 14, 2013		CHECKED BY		CN							
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
	--- CONTINUED FROM PREVIOUS PAGE ---																
309.4	SAND, trace to some silt, trace clay Loose to compact Brown Wet		14	SS	18		311										
							310										
17.0	SILT and SAND, trace clay Loose Grey Wet						309										
							308										0 52 46 2
			15	SS	5												
							307										
	Cobbles encountered between depths of 19.5 and 20.9 m.		2	RC	REC 100%		306										
305.5			3	RC	REC 100%												
20.9	Granitic Gneiss (BEDROCK)						305										RQD = 95%
	Bedrock cored from depths of 20.9 m to 24.3 m. Refer to Record of Drillhole B2-02 for bedrock coring details.		4	RC	REC 100%		304										RQD = 98%
			5	RC	REC 100%		303										RQD = 100%
302.1			6	RC	REC 100%												
24.3	END OF BOREHOLE																
	NOTES: 1. Water level in open borehole at a depth of 2.2 m below ground surface (Elev. 324.2 m) upon completion of drilling. NR - Not Recorded																

PROJECT: 11-1111-0149

RECORD OF DRILLHOLE: B2-02

SHEET 1 OF 1

LOCATION: N 5035596.1 ; E 324376.8

DRILLING DATE: May 14, 2013

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: C.M.E. 55 Truck Mount

DRILLING CONTRACTOR: Landcore

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV.	RUN No.	COLOUR % RETURN	FLUSH	RECOVERY				R.Q.D. %	FRACT. INDEX PER 0.3 m	DISCONTINUITY DATA				HYDRAULIC CONDUCTIVITY K, cm/sec				Diametral Point Load Index (MPa)	RMC -Q' AVG.	NOTES	
				DEPTH (m)				TOTAL CORE %	SOLID CORE %	B Angle	DIP w.r.t. CORE AXIS			TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	10 TO 10	10 TO 10	10 TO 10	10 TO 10				
JN - Joint FLT - Fault SH - Shear VN - Vein CJ - Conjugate	BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage	PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular	PO - Polished K - Slickensided SM - Smooth RO - Rough VR - Very Rough	MB - Mechanical Break BR - Broken Rock	NOTE: For additional abbreviations refer to list of abbreviations & symbols.																				
21	NW Casing	Continued from Record of Borehole B2-02		305.47																					
21		Fresh, light greyish-pink, medium to coarse grained, medium strong to strong, gneissic banding, GRANITIC GNEISS		20.93																					
22					4																				
23		with bands and lenses of mafic rich minerals			5																				
24					6																				
		END OF DRILLHOLE		302.08																					
25				24.32																					
26																									
27																									
28																									
29																									
30																									

DEPTH SCALE

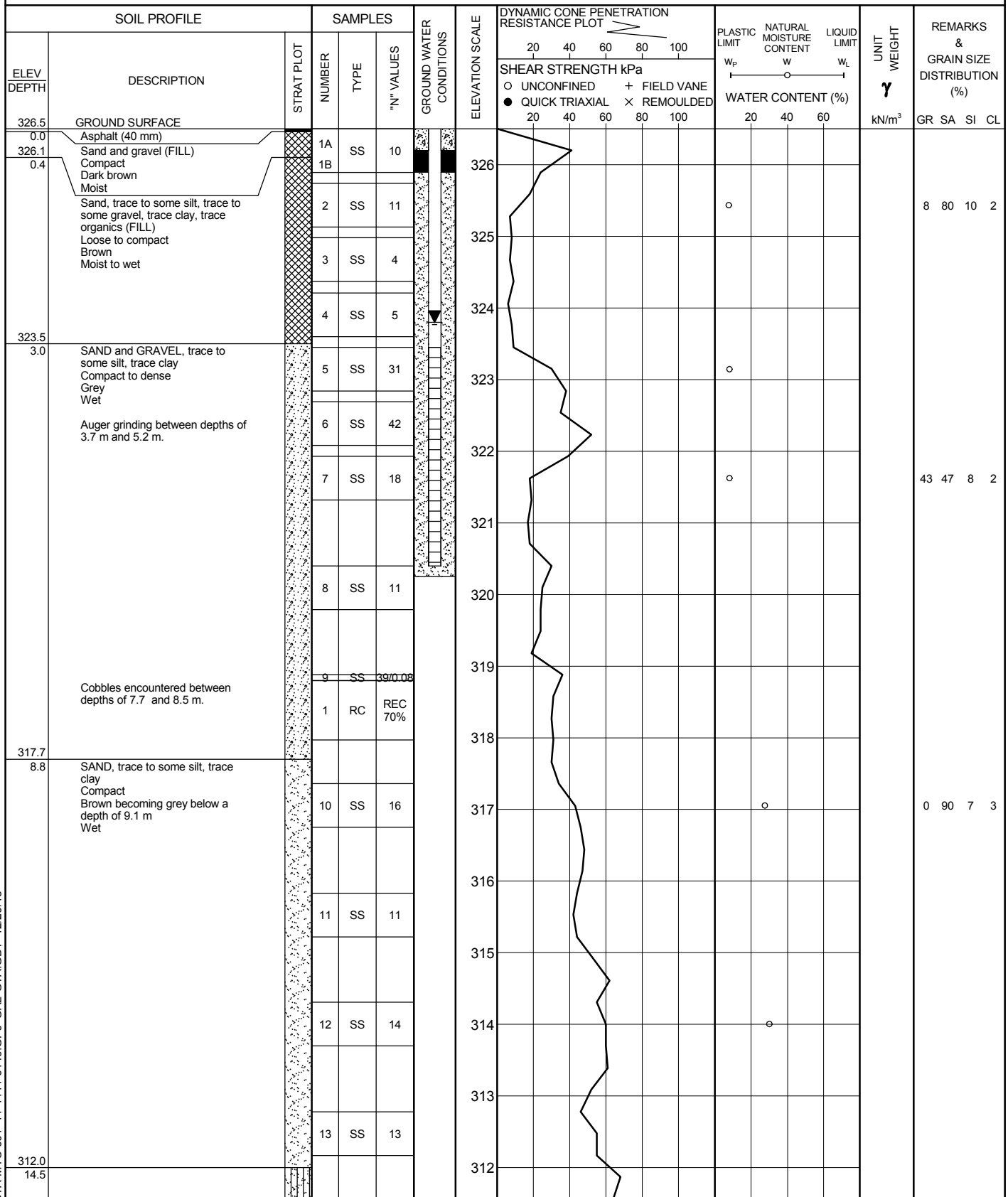
1 : 50



LOGGED:

CHECKED: CN

PROJECT 11-1111-0149		RECORD OF BOREHOLE No B2-03		SHEET 1 OF 2		METRIC	
W.P. 5266-07-01		LOCATION N 5035607.5 ; E 324372.2		ORIGINATED BY ID			
DIST HWY 592		BOREHOLE TYPE 120 mm O.D. Hollow Stem Augers and NW Casing		COMPILED BY AV			
DATUM Geodetic		DATE May 7 to 9, 2013		CHECKED BY CN			



Continued Next Page

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

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

PROJECT 11-1111-0149			RECORD OF BOREHOLE No B2-03			SHEET 2 OF 2			METRIC															
W.P. 5266-07-01			LOCATION N 5035607.5 ; E 324372.2			ORIGINATED BY ID																		
DIST HWY 592			BOREHOLE TYPE 120 mm O.D. Hollow Stem Augers and NW Casing			COMPILED BY AV																		
DATUM Geodetic			DATE May 7 to 9, 2013			CHECKED BY CN																		
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS			ELEVATION SCALE			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES																			
--- CONTINUED FROM PREVIOUS PAGE ---																								
309.4	SILT and SAND, trace clay Loose Grey Wet		14	SS	7																			
17.1	SAND and GRAVEL, trace to some silt Compact to very dense Grey Wet																							
			15	SS	17																			
	Cobbles and boulders encountered between depths of 19.4 and 21.3 m.		2	RC	REC 100%																			
			3	RC	REC 74%																			
			16	SS	54																			
	Cobbles encountered between depths of 21.7 and 24.4 m.		4	RC	REC 69%																			
			5	RC	REC 42%																			
302.1	END OF BOREHOLE SPLIT-SPOON REFUSAL		17	SS	50/0.0																			
24.4	NOTES: 1. Water level in open borehole at a depth of 2.1 m below ground surface (Elev. 324.4 m) upon completion of drilling. 2. An additional borehole was advanced about 1.5 m North of Borehole B2-03 to install a piezometer and to carry out Dynamic Cone Penetration Test. 3. Water level measurements in Piezometer: Date Depth (m) Elev. (m) 05/09/13 2.1 324.4 05/10/13 2.0 324.5 06/26/13 2.7 323.8 4. Piezometer decommissioned on June 26, 2013.																							

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

PROJECT		11-1111-0149		RECORD OF BOREHOLE No B2-04		SHEET 1 OF 2		METRIC						
W.P.		5266-07-01		LOCATION		N 5035627.7 ; E 324374.7		ORIGINATED BY						
DIST		HWY 592		BOREHOLE TYPE		120 mm O.D. Hollow Stem Augers and NW Casing		COMPILED BY						
DATUM		Geodetic		DATE		May 10, 2013		CHECKED BY						
								CN						
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
326.3	GROUND SURFACE							20 40 60 80 100	20 40 60					
0.0	Asphalt (25 mm)		1A	SS	29									
325.9	Sand and gravel (FILL)		1B											
0.4	Compact Brown Moist													
	Sand, trace gravel, trace organics, containing silt seams (FILL)		2	SS	8									
	Very loose to loose Brown Moist													
			3	SS	3									
324.1	Silty sand, trace to some clay, trace gravel (FILL)													
2.2	Loose Grey Wet		4	SS	8									5 67 21 7
323.3	CLAYEY SILT, trace to some sand													
3.0	Stiff Grey Wet		5	SS	14									
322.6	SAND and GRAVEL, trace silt													
3.7	Compact to dense Grey Wet		6	SS	10									
			7	SS	44									
320.7	SAND, trace to some silt, trace clay													
5.6	Loose to compact Brown Wet		8	SS	9									
			9	SS	11									0 84 13 3
			10	SS	13									
316.5	END OF BOREHOLE													
9.8	Dynamic Cone Penetration Test (DCPT)													

Continued Next Page

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

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

PROJECT		11-1111-0149		RECORD OF BOREHOLE No B2-04		SHEET 2 OF 2		METRIC							
W.P.		5266-07-01		LOCATION		N 5035627.7 ; E 324374.7		ORIGINATED BY							
DIST		HWY 592		BOREHOLE TYPE		120 mm O.D. Hollow Stem Augers and NW Casing		COMPILED BY							
DATUM		Geodetic		DATE		May 10, 2013		CHECKED BY							
CN															
SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS		ELEVATION SCALE		DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	20 40 60 80 100	W _p W W _L	WATER CONTENT (%)	γ	GR SA SI CL		
307.0	END OF BOREHOLE Dynamic Cone Penetration Test (DCPT)						311								
310							310								
309							309								
308							308								
307	END OF DCPT Refusal to Further Penetration (30 Blows / 0.13 m)						307								
19.3	NOTES: 1. Water level in open borehole at a depth of 1.8 m below ground surface (Elev. 324.5 m) upon completion of drilling. 2. Borehole caved at a depth of 4.6 m below ground surface (Elev. 321.7 m) upon completion of drilling.														



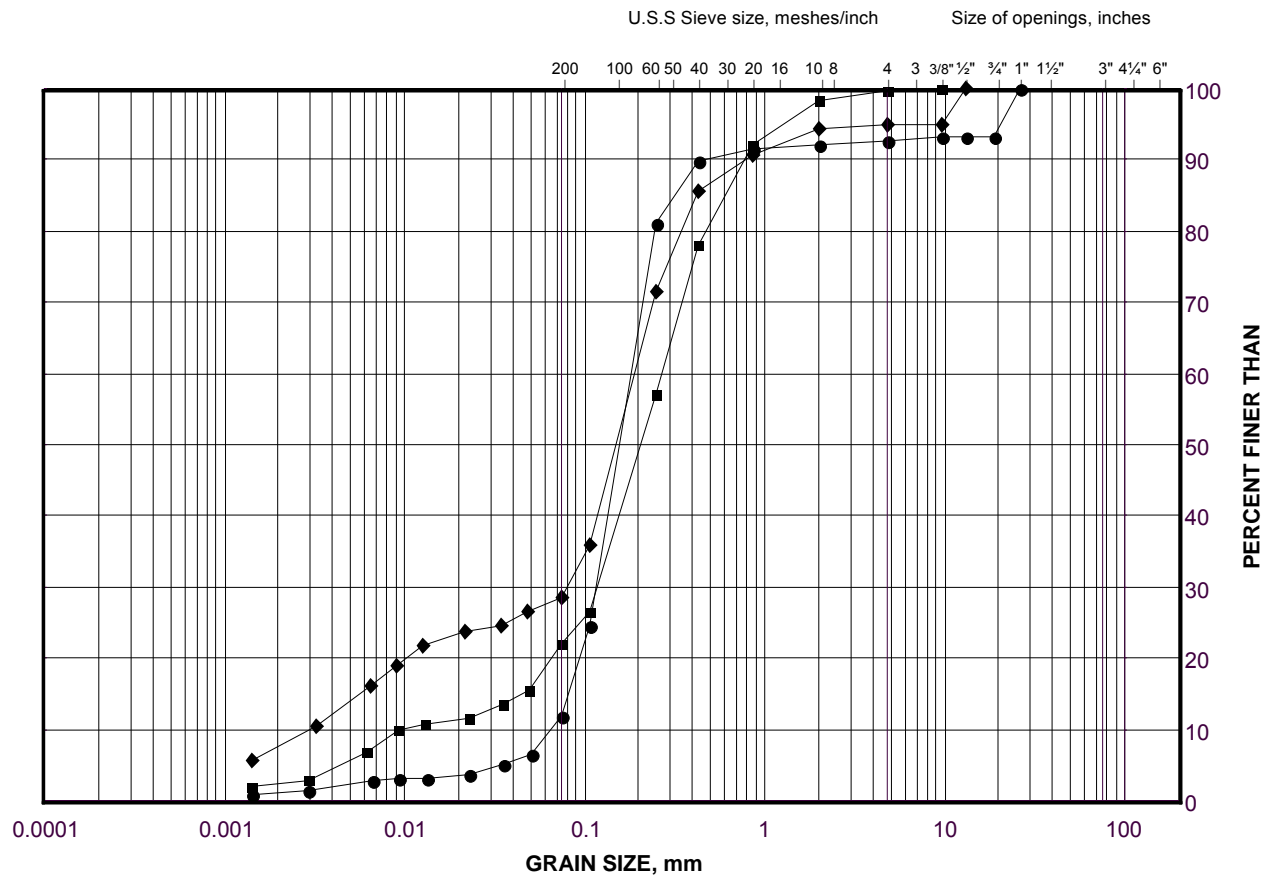
APPENDIX B

Laboratory Test Results and Bedrock Core Photographs

GRAIN SIZE DISTRIBUTION

Silty Sand to Sand (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)
●	B2-03	2	325.4
■	B2-01	3	324.6
◆	B2-04	4	323.7

Project Number: 11-1111-0149

Checked By: AV

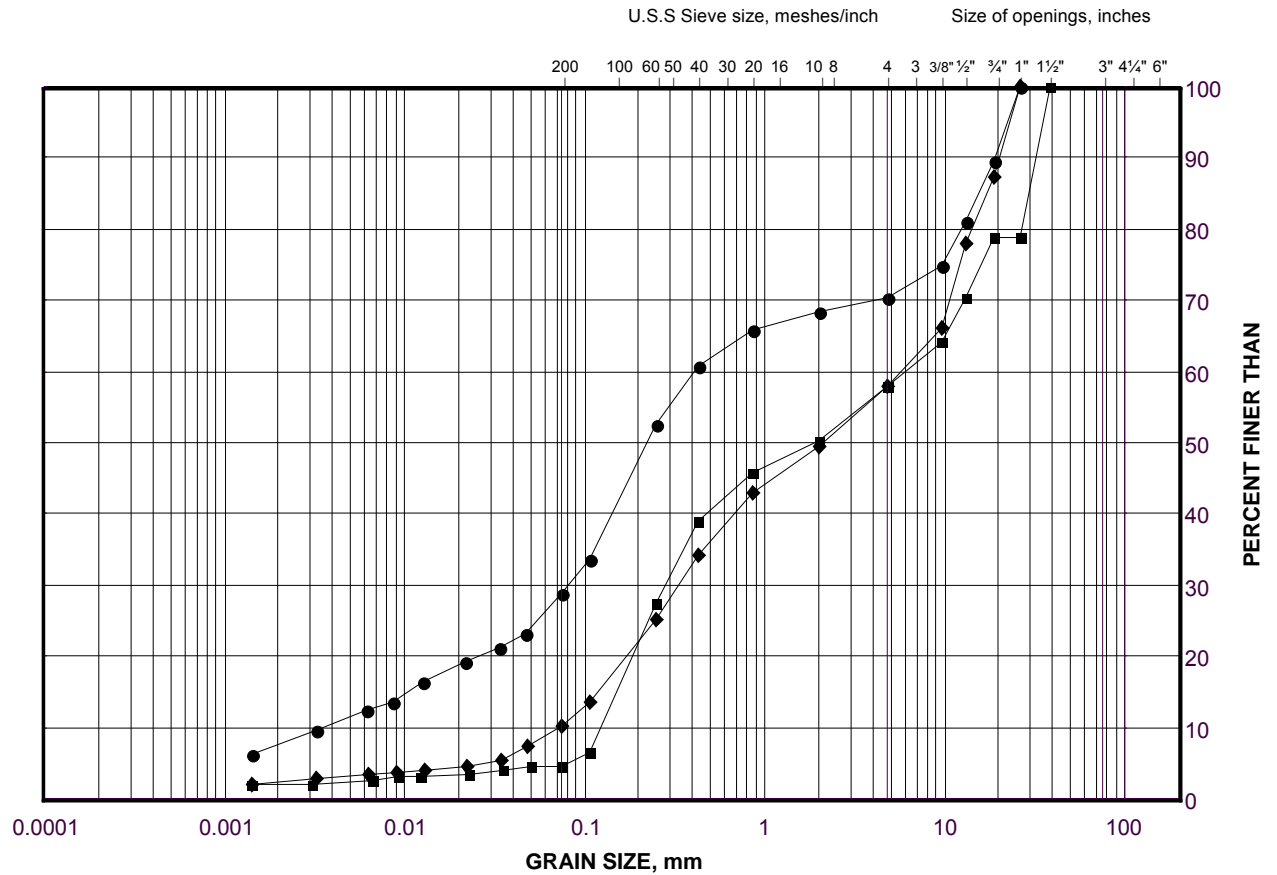
Golder Associates

Date: 07-Aug-13

GRAIN SIZE DISTRIBUTION

Sand and Gravel (Upper Deposit)

FIGURE B2



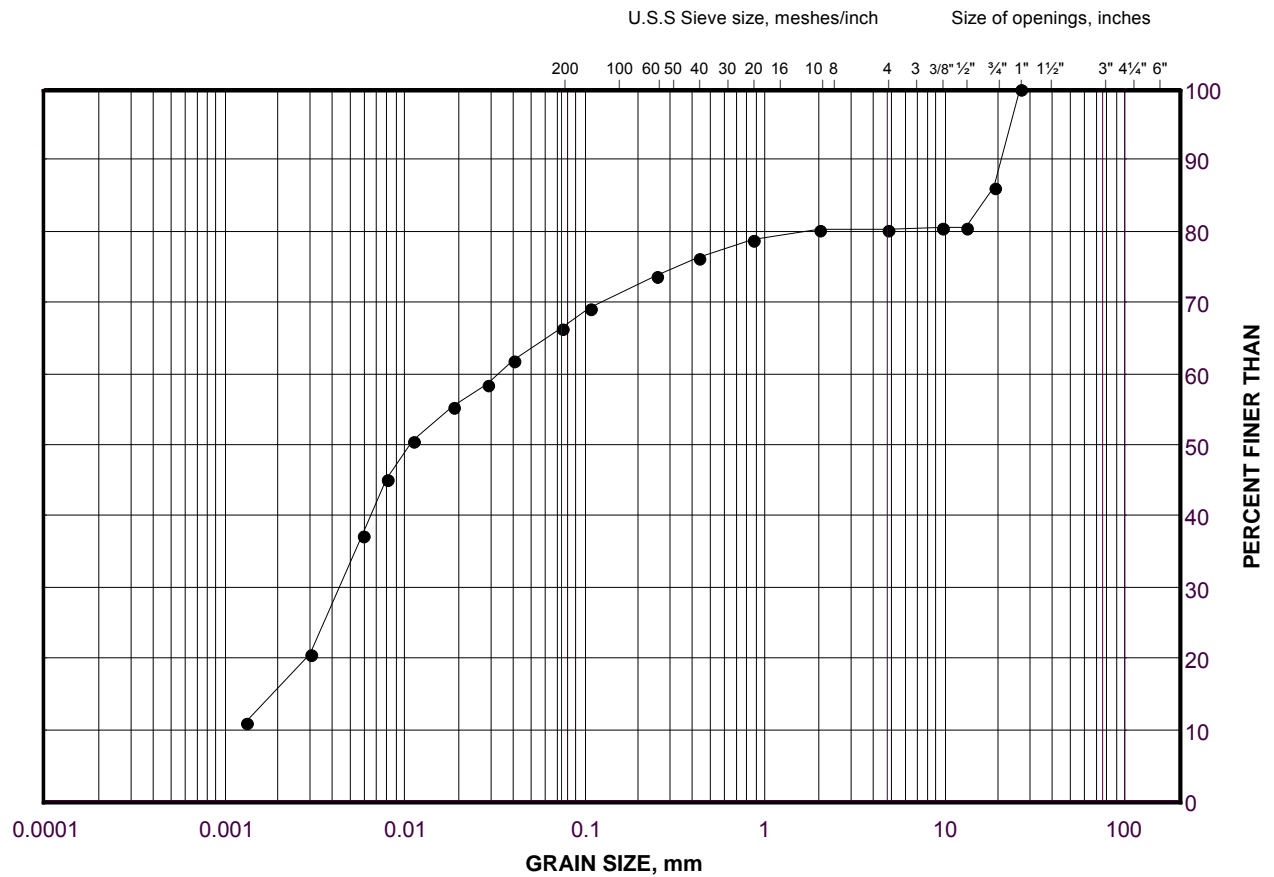
SILT AND CLAY SIZES				FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED				SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	B2-02	5	323.1
■	B2-02	6	322.3
◆	B2-03	7	321.6

Silt (Pockets)

FIGURE B3



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

LEGEND

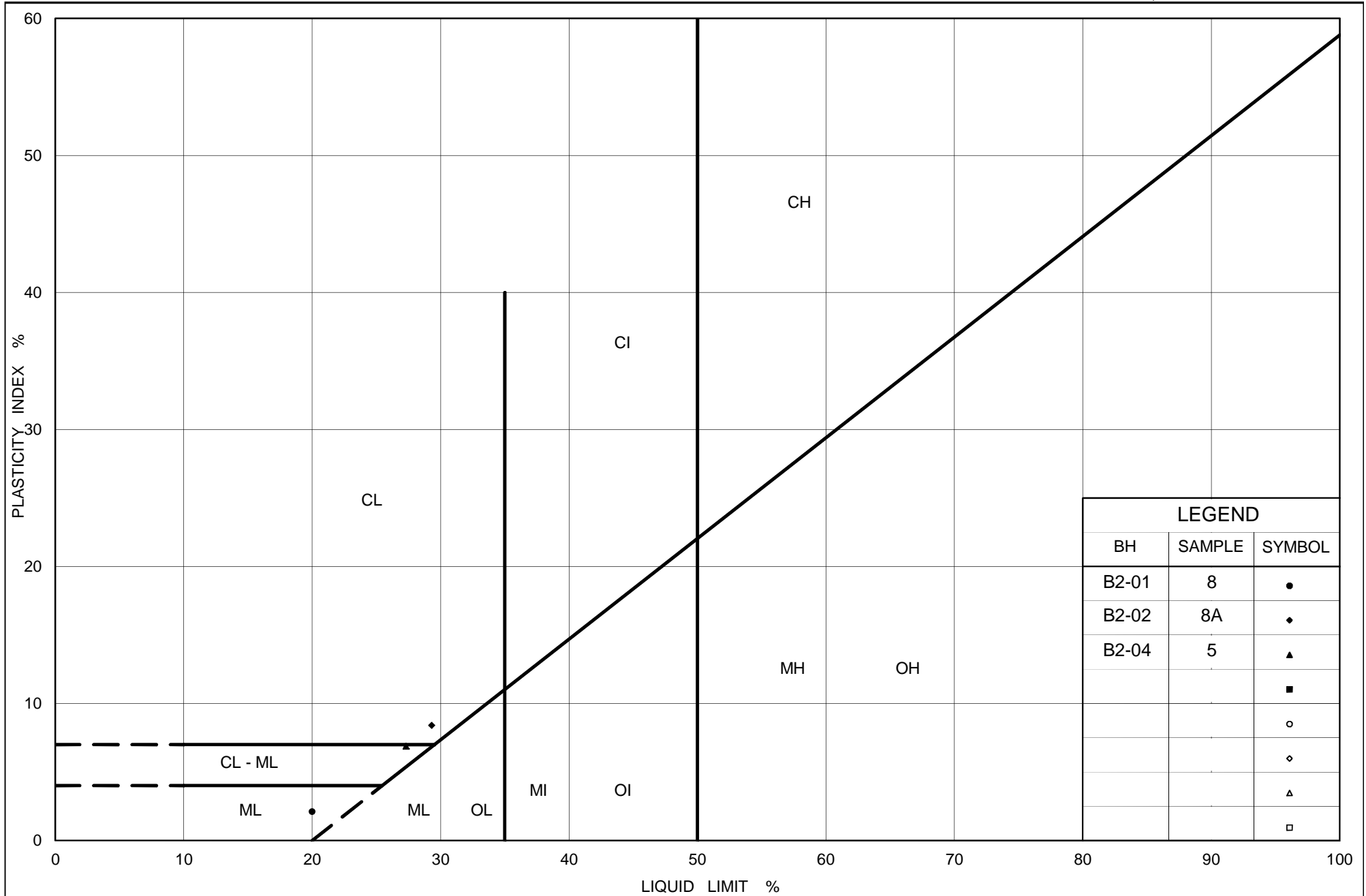
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	B2-01	8	320.0

Project Number: 11-1111-0149

Checked By: AV

Golder Associates

Date: 07-Aug-13



Ministry of Transportation

Ontario

PLASTICITY CHART Silt to Clayey Silt

Figure No. B4

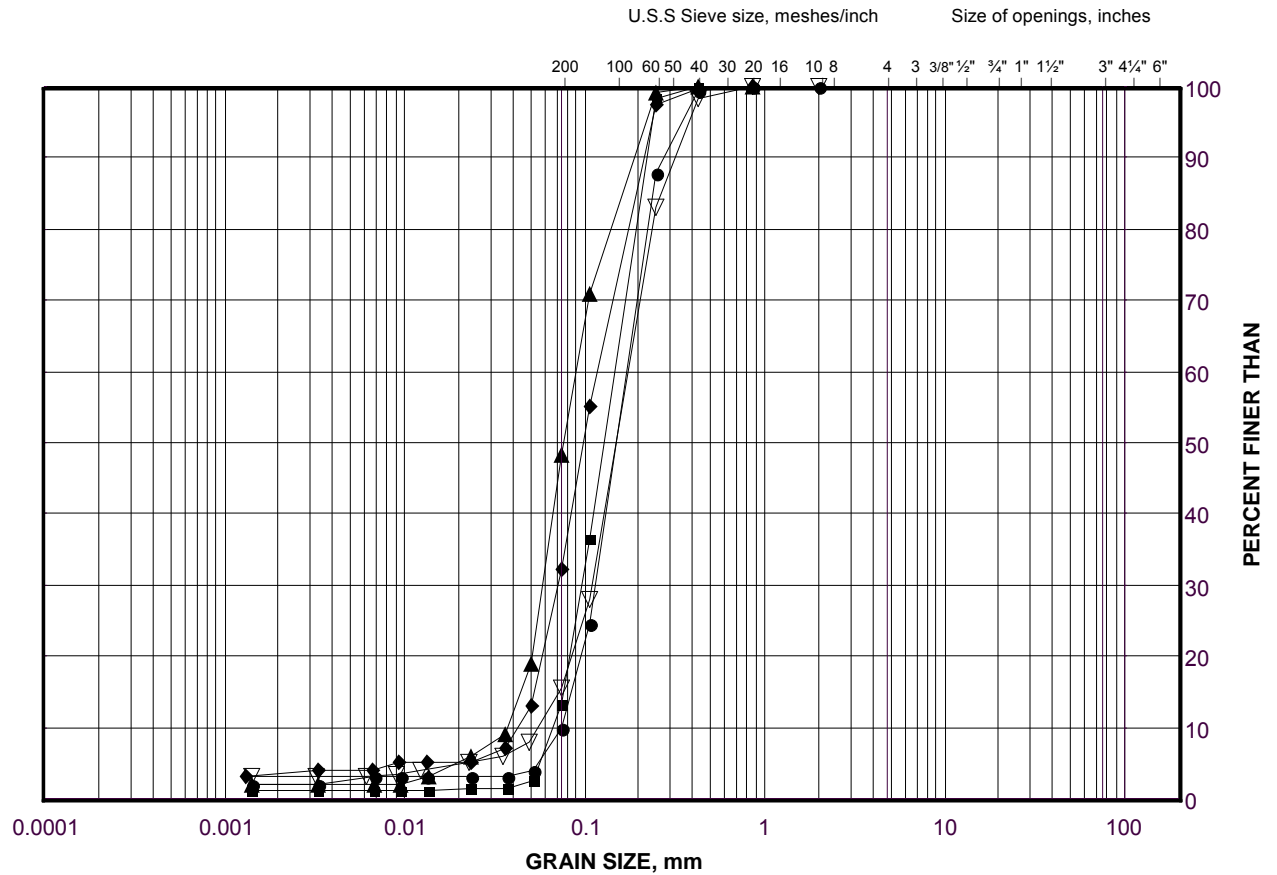
Project No. 11-1111-0149

Checked By: AV

GRAIN SIZE DISTRIBUTION

Silt and Sand to Sand

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)
●	B2-03	10	317.1
■	B2-02	11	315.4
◆	B2-03	14	310.9
▲	B2-02	15	307.8
▽	B2-04	9	318.4

Project Number: 11-1111-0149

Checked By: AV

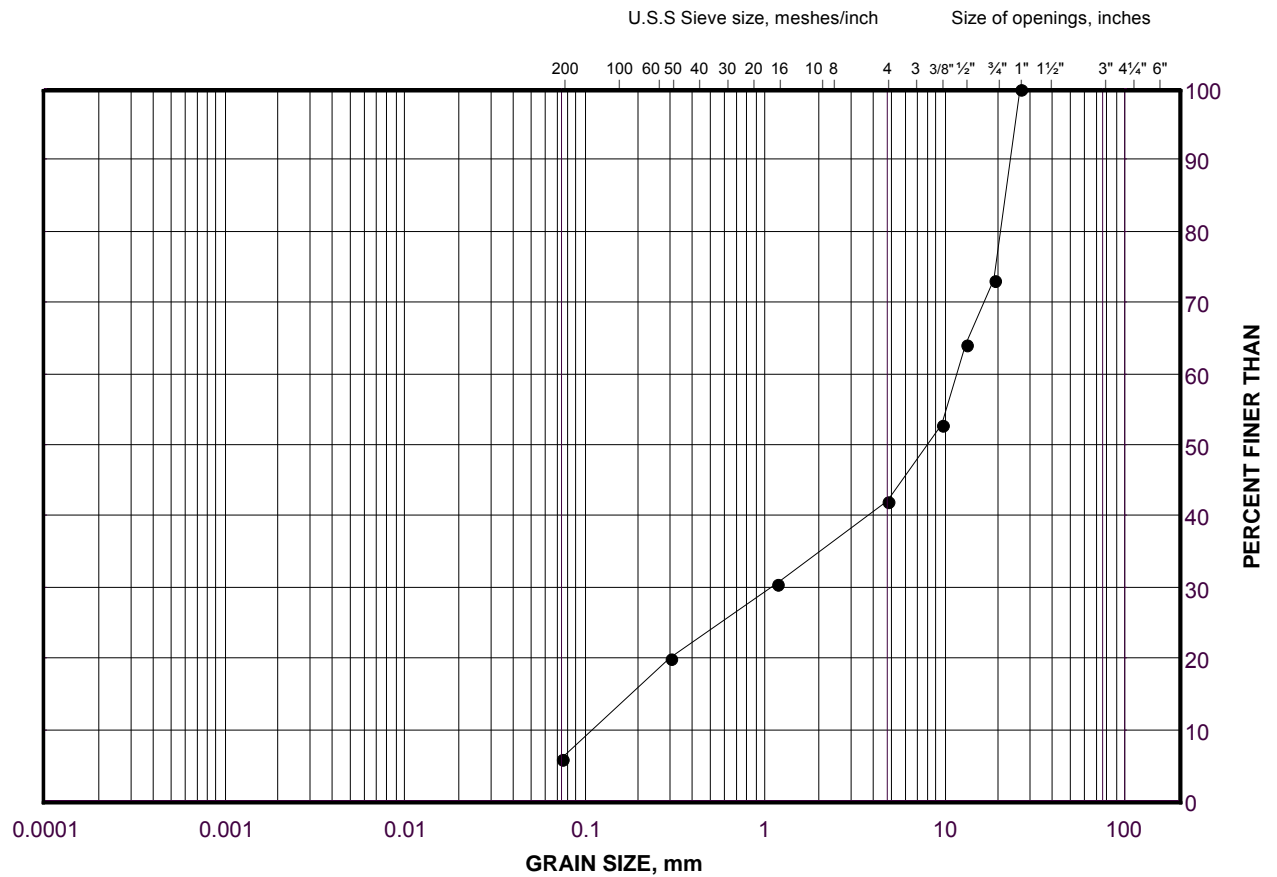
Golder Associates

Date: 07-Aug-13

GRAIN SIZE DISTRIBUTION

Sand and Gravel (Lower Deposit)

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)
•	B2-03	16	304.9

Project Number: 11-1111-0149

Checked By: AV

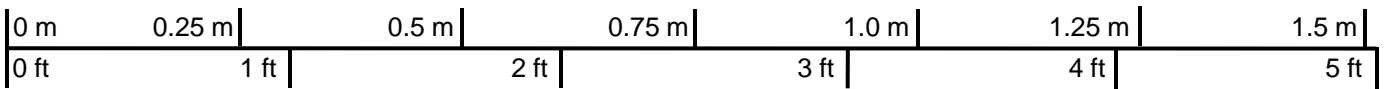
Golder Associates

Date: 20-Aug-13

Borehole B2-02



Box 1: 20.93 m – 24.32 m



Scale


PROJECT				Little East River Bridge No.2 Highway 592 GWP 5265-07-00; WP 5265-07-02			
TITLE				Rock Core Photograph – Borehole B2-02			
				PROJECT No. 11-1111-0149		FILE No. ----	
				DESIGN	AV	AUG 13	SCALE NTS
				CADD	--	--	REV.
				CHECK	CN	AUG 13	FIGURE B7
				REVIEW	JMAC	AUG 13	

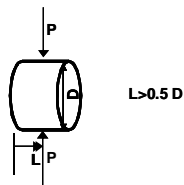
TABLE B1
POINT LOAD TEST RESULTS ON ROCK SAMPLES

Borehole Number	Run Number	Sample Depth (m)	Sample Elevation (m)	Bedrock Description	Test Type	Core Length (mm)	Core Diameter (mm)	Is (50mm) (MPa)	Approx. UCS Value ⁽¹⁾ (MPa)
B2-02	4	21.14	305.3	Granitic Gneiss	Diametral	80.00	47.60	3.15	41
B2-02	4	21.12	305.3	Granitic Gneiss	Axial	35.00	47.60	7.86	102
B2-02	5	22.38	304.0	Granitic Gneiss	Diametral	65.00	47.60	1.71	22
B2-02	5	22.40	304.0	Granitic Gneiss	Axial	30.00	47.60	6.66	87
B2-02	5	23.01	303.4	Granitic Gneiss	Diametral	70.00	47.60	4.73	61
B2-02	5	22.99	303.4	Granitic Gneiss	Axial	35.00	47.60	15.24	198
B2-02	6	24.15	302.3	Granitic Gneiss	Diametral	100.00	47.60	4.78	62
B2-02	6	24.12	302.3	Granitic Gneiss	Axial	50.00	47.60	13.02	169

⁽¹⁾ $Is_{50} \times K$, from ASTM Designation: D 5731 "Standard Test Method for Determination of the Point Load Strength Index of Rock and Application to Rock Strength Classifications". A value of $K = 13$ has been used based on a UCS test result.

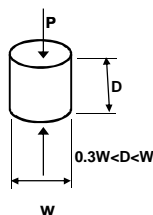
DIAMETRAL SPECIMEN SHAPE REQUIREMENTS

note: Diametral tests are perpendicular to core axis (planes of weakness)



AXIAL SPECIMEN SHAPE REQUIREMENTS

note: Axial tests are parallel to core axis (planes of weakness)



Compiled By: AV
Checked By: TVA
Reviewed By: JMAC

TABLE B2
UNCONFINED COMPRESSION TEST (UC)
ASTM D 7012-07

SAMPLE IDENTIFICATION

PROJECT NUMBER	11-1111-0149	RUN NUMBER	2
BOREHOLE NUMBER	B2-02	SAMPLE DEPTH, m	22.72-22.93

TEST CONDITIONS

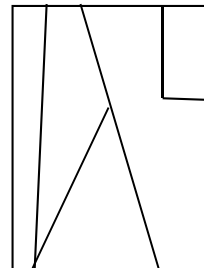
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST, min	>2 <15	L/D	2.21

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.40	WATER CONTENT, (specimen) %	0.05
SAMPLE DIAMETER, cm	4.71	UNIT WEIGHT, kN/m ³	26.04
SAMPLE AREA, cm ²	17.45	DRY UNIT WT., kN/m ³	26.02
SAMPLE VOLUME, cm ³	181.40	SPECIFIC GRAVITY	-
WET WEIGHT, g	481.80	VOID RATIO	-
DRY WEIGHT, g	481.56		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	140.2
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REMARKS:	DATE: 7/2/2013
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CHECKED BY: AV	REVIEWED BY: TVA/JMAC
----------------	-----------------------



APPENDIX C

Non-Standard Special Provisions

OBSTRUCTIONS

Special Provision

SCOPE

Cobbles and boulders were encountered within the sand and gravel and within the silt and sand to sand deposits 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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North America	+ 1 800 275 3281
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