



August 1, 2014

FOUNDATION INVESTIGATION AND DESIGN REPORT

**CN OVERHEAD STRUCTURE, SITE NO. 44-166
HIGHWAY 592 - REPLACEMENT OF SIX STRUCTURES
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5270-07-00; WP 5270-07-01**

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REPORT

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PART A

FOUNDATION INVESTIGATION REPORT

CN OVERHEAD STRUCTURE – Site No. 44-166

HIGHWAY 592 – REPLACEMENT OF SIX STRUCTURES

MINISTRY OF TRANSPORTATION, ONTARIO

GWP 5270-07-00; WP 5270-07-01



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 the Canadian National Rail (CNR) Overhead Structure (Site No. 44-166) on Highway 592 near Huntsville, Ontario. The proposed work is part of the replacement of six bridge structures along Highway 592. The CN Overhead Structure is located approximately 0.5 km north of Super Sign Road and approximately 1 km north of the Highway 11/Emsdale Road interchange in Emsdale, Ontario. The location of the existing bridge structure along Highway 592 is shown on the Key Map on Drawing 1.

The Terms of Reference (TOR) and Scope of Work for the foundation investigation are outlined in MTO's Request for Proposal, dated September 2011. Golder's proposal for foundation engineering services associated with the CN Overhead 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.

The report addresses the investigation carried out for the CN Overhead 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 within the CN Overhead Structure limits is oriented in an east-west direction, although the Highway 592 alignment is oriented generally in a north-south direction.

In general, the topography along Highway 592 consists of rolling terrain, including lakes, low-lying swamps containing areas of standing water, and sparsely to densely populated tree covered areas. Land use in some areas consists of residential/recreational communities. The existing bridge structure and associated approach embankments are situated in a densely treed area crossing over the CN tracks, which runs in a north-south direction in this area. The existing ground surface within the limits of the proposed structure and its approach embankments ranges from Elevations 330.0 m to 328.9 m, referenced to Geodetic datum. The existing Highway 592 east and west approach embankments near the centreline are at Elevations 328.9 m and 329.3 m, respectively.

3.0 INVESTIGATION PROCEDURES

3.1 Foundation Investigation

The field investigation for the proposed bridge structure was carried out between June 17 and 25, 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 separate borehole was advanced immediately adjacent to the borehole at the east abutment to install a piezometer, and a Dynamic Cone Penetration Test (DCPT) was advanced from



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the bottom of the borehole at the west abutment 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
East Approach Embankment	B6-01
East Abutment	B6-02
West Abutment	B6-03
West Approach Embankment	B6-04

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 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). Boulder 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 DCPT were advanced to depths of up to about 47.0 m (including coring of bedrock for a core length of about 3.3 m) below existing ground surface. The DCPT driven from the bottom of borehole at the west abutment was terminated on refusal to further dynamic cone penetration.

The groundwater conditions in the open boreholes were observed during and upon completion of drilling operations, and a standpipe piezometer was installed in a borehole immediately adjacent to Borehole B6-02 to permit monitoring of the water level at this locations. The piezometer consists of 38 mm diameter PVC pipe, with a slotted screen surrounded with sand sealed at a select depth within the borehole. The borehole and annulus surrounding the piezometer pipe above the screen and sand pack were backfilled to the surface with bentonite pellets/grout. Piezometer installation details and water level readings are described on the Record of Borehole sheet B6-02 in Appendix A. All open boreholes were backfilled with cement grout by tremie technique upon completion and the piezometer was also abandoned with cement grout by tremie technique on June 19, 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, grain size distribution and Atterberg limits) was carried out on selected soil samples. Strength testing, consisting of 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.



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Classification of the rock 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 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 Depth / DCPT
	Northing	Easting		
B6-01	5042389.4	319940.4	328.9 m	18.9 m
B6-02	5042383.8	319921.5	329.8 m	47.0 m
B6-03	5042386.0	319891.8	330.0 m	46.0 m / 46.2 m
B6-04	5042382.1	319872.8	329.3 m	18.9 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,

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

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.



observations of drilling progress and the results of Standard Penetration Test (SPTs). These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Further, subsurface conditions will vary between and beyond the borehole locations. It should be noted that the interpreted stratigraphy shown on Drawing 2 is a simplification of the subsurface conditions.

In general, the subsurface conditions in the area of the proposed bridge structure consist of a surface layer of asphalt underlain by a deposit of non-cohesive fill associated with the Highway 592 embankments. The fill is underlain by a relatively thick deposit of silt to sandy silt to silty sand, in turn underlain in places by a deposit of gravelly sand to sand and gravel, which is underlain 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 thick layer of asphalt was encountered at the ground surface in all boreholes.

4.2.2 Sand and Gravel to Silt and Sand (Fill)

A deposit of brown to grey non-cohesive fill was encountered below the asphalt layer in all boreholes. The fill deposit is generally comprised of an upper layer of sand and gravel, trace silt and a lower layer of silt, trace to some sand to silt and sand, trace clay, trace gravel, trace organics and sand pockets in places. The top of the fill deposit ranges from Elevations 330.0 m to 328.9 m and the thickness of the deposit ranges from 2.2 m to 4.5 m.

The SPT 'N'-values measured within the non-cohesive fill deposit generally range from 2 blows to 7 blows per 0.3 m of penetration, indicating a very loose to loose relative density, with SPT 'N'-values between 11 blows and 16 blows per 0.3 m of penetration recorded within the upper portion of the fill immediately underlying the asphalt layer, indicating a compact relative density.

The natural water content measured on six samples of the silt to silt and sand portion of the fill ranges from about 2 per cent to 34 per cent.

The results of grain size distribution tests completed on two samples of the silt to silt and sand portion of the fill are shown on Figure B1 in Appendix B.

4.2.3 Silt to Silty Sand

A non-cohesive deposit comprised of silt to sandy silt to silty sand was encountered underlying the fill deposit in all the boreholes. The overall silt to silty sand deposit is comprised of an upper portion of silt trace to some sand to sandy silt to silt and sand, and a lower portion of silt trace to some sand to silt and sand to silty sand. The deposit generally contains trace to some clay and trace gravel. The top of the silt to silty sand deposit ranges from Elevations 326.7 m to 325.5 m, and the thickness of the deposit ranges from 15.2 m to 39.4 m. Boreholes B6-01 and B6-04 were terminated within this deposit.



The SPT 'N'-values measured within the overall silt to silty sand deposit range from 3 blows to 40 blows per 0.3 m of penetration, indicating a very loose to dense relative density. The silt to sandy silt to silt and sand upper portion of the deposit may be described as very loose to compact, and the silt to silt and sand to silty sand lower portion of the deposit may be described as very loose to dense.

The natural water content measured on thirty four samples of the overall silt to silty sand deposit ranges from about 16 per cent to 77 per cent, but is generally below 33 per cent.

The results of grain size distribution tests completed on nineteen samples of the overall silt to silty sand deposit are shown on Figures B2 in Appendix B: presented for the silt to sandy silt and for the silt to silt and sand upper portion of the deposit on Figures B2A and B2B; and for the silt to silt and sand to silty sand lower portion of the deposit on Figure B2C.

Atterberg limits tests were carried out on four samples of the silt portion of the deposit, and the results indicate that the material is non-plastic.

4.2.4 Gravelly Sand to Sand and Gravel

An approximately 2.3 m thick deposit of gravelly sand to sand and gravel was encountered below the silt to silty sand deposit in Boreholes B6-02 and B6-03. The gravelly sand deposit encountered in Borehole B6-03 contains some silt, trace clay; and a boulder was encountered and cored for a total length of approximately 1.4 m within this deposit. The top of this deposit is at Elevations 288.4 m and 286.1 m in Boreholes B6-02 and B6-03, respectively. The DCPT advanced from the bottom of sampled Borehole B6-03 is inferred to terminate within the gravelly sand deposit after penetrating 0.2 m into this deposit to Elevation 283.8 m. Both boreholes were terminated within this deposit.

SPT 'N'-values of 34 blows per 0.3 m of penetration and 75 blows per 0.15 m of penetration were measured within this deposit, indicating a dense to very dense relative density. A Total Core Recovery (TCR) of 79 per cent was recorded within the cored boulder.

The natural water content measured on one sample of the deposit is about 12 per cent.

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

The photograph of the cored boulder is presented on Figure B4 in Appendix B.

4.2.5 Refusal / Bedrock

Refusal to further DCPT advancement was encountered in Borehole B6-03 at a depth of 46.2 m below ground surface corresponding to Elevation 283.8 m. Bedrock was encountered in Borehole B6-02 at a depth of 43.7 m below the existing ground surface, corresponding to Elevation 286.1 m, and about 3.3 m of core samples were recovered.

Based on a review of the bedrock core samples, the bedrock consists of granitic gneiss with bands and lenses of mafic rich minerals. In general, the bedrock samples are described as fresh, medium to coarse crystalline, strong, light grey with light pink veins and dark green bands, as presented in the Record of Drillhole sheet in Appendix A, and as shown on the photograph of the recovered core samples on Figure B5 in Appendix B.



The Rock Quality Designation (RQD) measured on the core samples is between about 96 per cent and 100 per cent, indicating a rock mass of excellent quality. The Total Core Recovery (TCR) of the core samples recovered is 100 per cent and the Solid Core Recovery (SCR) of the core samples is between 97 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 samples of the bedrock core measured Is_{50} values ranging from 4.8 MPa to 8.7 MPa and the diametral tests carried out on four samples of the bedrock core measured Is_{50} values ranging from 5.5 MPa to 7.9 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 73 MPa. The details of UC test are 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 15.

Based on the laboratory UC test and the point load index test results the approximate uniaxial compressive strength of the bedrock range from 48 MPa to 79 MPa. The granitic gneiss bedrock is generally classified as strong (R4, 50 MPa < UCS < 100 MPa) to very strong (R5, 100 MPa < UCS < 250 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 Elevations 321.3 m to 317.7 m measured at depths between 7.6 m and 11.6 m below ground surface.

A standpipe piezometer was installed in a borehole immediately adjacent to Borehole B6-02 to allow monitoring of the groundwater level at the site. Details of the piezometer installation are shown on the Record of Borehole No. B6-02 in Appendix A and the groundwater level measured in the piezometer is summarized below.

Borehole	Ground Surface Elevation	Depth to Water Level	Groundwater Elevation	Date of Measurement
B6-02	329.8 m	10.2 m	319.6 m	June 19, 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. Veronica Ayetan, P.Eng., a geotechnical engineer with Golder. Mr. Jorge M. A. Costa, P.Eng., Golder's Designated MTO Contact for this project and Principal with Golder, conducted an independent quality control review of the report.



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PART B

FOUNDATION DESIGN REPORT

CN OVERHEAD STRUCTURE – SITE NO. 44-166

HIGHWAY 592 – REPLACEMENT OF SIX STRUCTURES

MINISTRY OF TRANSPORTATION, ONTARIO

GWP 5270-07-00; WP 5270-07-01



6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides engineering design recommendations for the proposed CN Overhead Structure on Highway 592 (Site No.44-166). 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 foundations 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 the CN Overhead Structure on Highway 592 in Huntsville, Ontario.

Based on the General Arrangement (GA) drawing provided by MH on January 29, 2014, the proposed CN Overhead Structure will consist of a single-span, pre-cast girder structure with a span length of 28 m. The grade of the proposed bridge deck will be at about Elevation 331.1 m, corresponding to a raise of the existing approach embankments of up to about 2.2 m, and the proposed underside of the spread footings or pile caps will be at about Elevation 322.8 m. In addition, the replacement structure will have retained soil system (RSS) walls adjacent to the east and west abutments and wingwalls, between about 3 m and 5 m high oriented parallel to Highway 592 on both sides of the highway.

6.2 Foundation Options

Given the presence of an up to about 39.4 m thick deposit of very loose to compact silt to silty sand, a shallow foundation system comprised of spread/strip footings founded directly on the native overburden is not recommended for support of the abutments. Given the high fines content of the deposit and the lack of lateral confinement to protect against migration of aggregates out of the stone column into the native overburden, a foundation support system comprised of aggregate piers and vibro-replacement stone columns is not appropriate at this site and is not considered further. Deep soil mixing may be considered as means to improve the in situ soil conditions, however, the increased geotechnical axial resistance derived from the improved foundation deposit may not be sufficient for design of spread/strip footing foundations.

Given that refusal/bedrock is up to about 39 m below the proposed pile cap elevation, deep foundations comprised of driven steel H-piles terminated within the dense zone of the silt to silty sand deposit is considered the preferred alternative for the support of the overhead structure. Drilled steel casing terminated within the compact to dense silt to silty sand deposit as well as steel H-piles driven to refusal/bedrock may also be considered for design.

Soil-bonded micropiles could be considered for design, however, unlike the other five bridge replacement structures crossing the Little East River and Ragged Creek associated with the Highway 592 Rehabilitation assignment, where the subsurface conditions (i.e. thick granular deposits containing cobbles and boulders) and



site constraints (i.e. narrow right-of-way) for equipment access/setup detrimentally affects construction of other types of deep foundations, a micropile foundation system at this site does not offer any technical advantage over driven steel H-piles or drilled steel casings.

The following sections provide recommendations for alternative foundation systems, comprised of spread footings constructed on the upper compact native overburden and deep soil mixing columns, driven H-pile, drilled steel casing foundations and soil-bonded micropiles installed to within the lower compact to dense silt to silty sand deposit.

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

6.3 Spread Footings

Shallow foundations comprised of spread/strip footings founded on native overburden are not recommended for support of the proposed bridge abutments given that the very loose to compact nature of the upper portions of the silt to silty sand deposit. In the event that shallow foundations are considered further for the support of the proposed structure, recommendations for design are provided below.

6.3.1 Geotechnical Axial Resistance and Reaction

For 12 m long by 4.5 m wide footings founded on the very loose to compact silt to silty sand deposit at Elevation 323.0 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:

Foundation Location	Factored Geotechnical Axial Resistance at ULS	Geotechnical Reaction at SLS for 25 mm of Settlement
East and West Abutments	500 kPa	50 kPa

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

Given the low geotechnical reaction at SLS, consideration can be given to the use of deep soil mixing to improve the strength and stiffness of the native overburden. Deep soil mixing is a ground improvement technique which mechanically mixes the native overburden with cementitious binder slurry to create columns of in situ composite soil mass. Discussions with a Contractor specializing in the soil mixing ground improvement technique indicate that a geotechnical reaction up to 250 kPa at SLS for 25 mm of settlement is achievable. For preliminary analysis and design, the following summarizes the diameter of the soil mixed columns, the spacing between columns, depth of the treatment, the tip elevation of the columns as well as the geotechnical reaction at SLS at 25 mm of settlement:



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Foundation Location	Diameter of Soil Mixed Column	Centre-to-Centre Spacing Between Columns	Depth of Treatment ^(a)	Tip Elevation of Column	Geotechnical Reaction at SLS for 25 mm of Settlement
East Abutment	1.8 m	3 m	7.5 m	314.8 m	150 kPa
	1.8 m	2 m	7.5 m	314.8 m	250 kPa
West Abutment	1.8 m	3 m	12.5 m	309.8 m	150 kPa
	1.8 m	2 m	12.5 m	309.8 m	250 kPa

Notes:

- (a) Assumes underside of spread/strip footing is at Elevation 322.8 m and that spread/strip footing will be founded on 0.5 m of compacted Granular 'A' pad (i.e. load transfer layer).

6.3.2 Resistance to Lateral Loads

The resistance to lateral forces/sliding resistance between the concrete footings, the natural subgrade materials and compact Granular 'A' pad 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 various materials is given below.

Interface Material(s)	Coefficient of Friction ($\tan \delta$)
Concrete footing on very loose to compact silt	0.30
Concrete footing on compacted Granular 'A' over soil mixed columns	0.55

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 a thickness of 25 mm of styrofoam equivalent to 300 mm of soil cover.



6.4 Driven Steel H-Pile

Given the presence of the up to about 41.7 m thick overburden deposit at this site, a deep foundation support system consisting of friction steel H-piles driven into the compact portion of the silt to silty sand deposit is considered the preferred foundation alternative for the support of the proposed structure at this site.

Consideration could also be given to steel H-pile driven to refusal/bedrock; however, up to about 39 m long piles would be required for this foundation system.

6.4.1 Geotechnical Axial Resistance and Reaction

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

Foundation Type	Foundation Location	Elevation of Underside of Pile Cap ^(a)	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 ^(b)
Friction H-Pile	East Abutment	322.8 m	295.8 m	27 m	1,000 kN	N/A
	West Abutment	322.8 m	295.8 m	27 m	1,000 kN	N/A
End-Bearing H-Pile	East Abutment (Bedrock)	322.8 m	286.1 m	36.7 m	2,000 kN	N/A
	West Abutment (Refusal)	322.8 m	283.8m	39 m	1,600 kN ^(c)	N/A

Notes:

- (a) As per the GA Drawing provided by MH on January 29, 2014.
- (b) 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.
- (c) A reduction factor of 0.8 is applied to account for possible refusal on boulders.

For end-bearing H-piles, provisions should be made in the Contract Documents to deal with varying pile lengths at the abutments given the possibility of encountering refusal on boulders within the gravelly sand to sand and gravel deposit. In addition, all end-bearing H-piles should be fitted with driving shoes and flange plates (reinforced tips) in accordance with OPSD 3000.100 (Steel H-Pile Driving Shoe) to minimize damage to the pile during driving and penetration through the granular deposits containing boulders.

6.4.2 Set Criteria

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



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

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

The following pile driving notes should be added to the Contract Drawings (i.e. relevant notes in Clause 3.3.3 of the Structural Manual (MTO, 2011)):

For friction H-piles (Note 2):

- Piles to be driven in accordance with Standard SS 103-11 using an ultimate geotechnical resistance of 2,500 kN per pile, but must be driven below El. 295.8 m.

For end-bearing H-piles at the east abutment (Note 5):

- Piles to be driven to bedrock.

For end-bearing H-piles at the west abutment (Note 2):

- Piles to be driven in accordance with Standard Drawing SS 103-11 using an ultimate geotechnical resistance of 4,000 kN per pile, but must be driven below El. 286.1 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

Deep foundations consisting of steel casing drilled into the lower compact portion of the silt to silty sand deposit could be considered for the support of the proposed structure at this site.

Consideration could also be given to socketting the steel casing into bedrock/refusal; however, up to 40.5 m long steel casings would be required.

6.5.1 Geotechnical Axial Resistance and Reaction

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



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Foundation Type	Foundation Location	Elevation of Underside of Pile Cap ^(a)	Casing Tip Elevation	Length of Casing from Underside of Pile Cap	Factored Geotechnical Axial Resistance at ULS	Geotechnical Reaction at SLS for 25 mm of Settlement ^(b)
Friction Steel Casing	East Abutment	322.8 m	303.8 m	18 m	1,000 kN	N/A
	West Abutment	322.8 m	303.8 m	18 m	1,000 kN	N/A
End-Bearing Steel Casing	East Abutment (Bedrock)	322.8 m	284.6 m ^(c)	38.2 m	3,700 kN	N/A
	West Abutment (Refusal)	322.8 m	282.3 m ^(c)	40.5 m	2,950 kN ^(d)	N/A

Notes:

- (a) As per the GA Drawing provided by MH on January 29, 2014.
- (b) 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.
- (c) Steel casings socketted 1.5 m into bedrock/refusal.
- (d) A reduction factor of 0.8 is applied to account for possible refusal on boulders.

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

Deep foundations consisting of soil-bonded micropiles advanced into the lower compact portion of the silt to silty sand deposit could be considered for the support of the proposed structure at this site. However, given that cobbles and boulders are not present within the silt to silty sand deposit (i.e. not encountered in the boreholes at this site) and that there physical constraints are not apparent at this site, a micropile foundation system does not offer any technical advantage over driven steel H-piles or drilled steel casings.

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, the micropile tip elevation as well as the diameter and length of the micropiles.



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Foundation Location	Elevation of Underside of Pile Cap ^(a)	Type and Size of Central Bar	Diameter of Micropile Bond Zone	Micropile Tip Elevation	Length of Micropile from Underside of Pile Cap
East and West Abutment	322.8 m	Hollow-Core Bar 103/78	263 mm	294.5 m	28.3 m ^(b)

Note:

(a) As per the GA Drawing provided by MH on January 29, 2014.

(b) Assumes a 3 m long cased length.

The following summarizes the preliminary factored geotechnical axial resistance at ULS and the geotechnical reaction at SLS for 25 mm of settlement for a 263 mm diameter micropile at the proposed abutments.

Foundation Location	Factored Geotechnical Axial Resistance at ULS	Geotechnical Reaction at SLS for 25 mm of Settlement ^(a)
East and West Abutment	1,000 kPa	N/A

Note:

(a) 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:



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

where: n_h = coefficient related to soil density (kPa/m)
 z = depth (m)
 B = pile diameter or width (m)

and for cohesive soils:

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

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

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

Foundation Element (Relevant Borehole)	Soil Unit	Elevation	n_h	s_u
East Abutment (B6-02) and West Abutment (B6-03)	Compact Silt to Sandy Silt	322.8 m to 320.0 m	10,000 kPa/m	-
	Very Loose to Loose Silt to Silty Sand	320.0 m to 314.0 m	5,000 kPa/m	-
	Compact Sandy Silt to Silty Sand	314.0 m to 305.0 m	8,000 kPa/m	-
	Compact to Dense Silt to Silty Sand	305.0 m to 288.5 m	15,000 kPa/m	-
	Very Dense Gravelly Sand to Sand and Gravel	288.5 m to 283.8 m	30,000 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 ^(a)	Axial Load Applied at the Top of Pile/Casing	Factored Geotechnical Lateral Resistance at ULS ^(a)	Geotechnical Lateral Reaction at SLS for 10 mm of Deflection ^(a)
East and West Abutment	HP 310 x 110	1,000 kN	80 kN	20 kN
	610 mm diameter drilled steel casing	1,000 kN	165 kN	20 kN

Notes:

- (a) Analyses assume 1.8 m of soil cover above the pile cap.
- (b) Analyses assume a pinned-head condition.



Based on the above, it is considered that both the structural and geotechnical resistances of the piles/drilled steel casings 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 pile cap equal to about $1.5 \cdot B$ (after Broms, 1964, where B is the pile diameter) should be neglected in the calculation of lateral resistance of the pile to account for disturbance effects during installation.

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

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

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

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

6.8 Seismic Considerations

6.8.1 Site Coefficient

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

6.8.2 Seismic Analysis Coefficient

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

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



6.9 Lateral Earth Pressures

The lateral earth pressures acting on the abutment stems and any associated wing walls/retaining walls will depend on the type and method of placement of the backfill material, the nature of the soils behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls.

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

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

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

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



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

If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the foundation design of the structure. If the wall support and superstructure does not allow lateral yielding, at-rest earth pressures should be assumed for foundation design. The movement required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.9.1 and Table C6.6 of the *Commentary* to the CHBDC.

6.10 Retained Soil System (RSS) Walls

It is understood that mechanically-reinforced soil retaining systems (retained soil system or RSS walls) will be required on both sides of the east and west abutments.

6.10.1 Founding Elevations

A typical RSS wall has front facing panels supported on a strip footing placed at shallow depth below the ground surface in front of the wall. Based on the GA drawing provided by MH, the proposed RSS walls are to be stepped into the approach embankment fill at founding levels between Elevation 326 m and 328 m. As such, it is recommended that Granular 'B' Type II (i.e. not rock fill) be used as embankment fill within the limits of the RSS walls to minimize the post-construction settlement of the RSS walls.

The facing footings should be placed on a minimum 300 mm thick compacted Granular 'A' pad. The compacted granular pad should extend at least 1 m beyond the outside edge of the facing footing, then downward at 1H:1V, as shown on Figure 5.2 in the MTO RSS Design Guidelines.

6.10.2 Global Stability

The static global slope stability of the RSS walls adjacent to the CN Overhead Structure has been analyzed using the commercially available program Slide (Version 6.0), developed by Rocscience Inc., employing the Morgenstern-Price method of analysis. For all analyses, the Factor of Safety of numerous potential failure surfaces was computed in order to establish the minimum Factor of Safety. A target Factor of Safety of 1.5 against deep-seated global instability of the RSS walls is normally adopted by MTO for design under static conditions, considering the design requirements and the field data available.

The soil parameters used in the analysis were estimated from empirical correlations using results of in situ Standard Penetration Testing and geotechnical classification testing. The following presents the simplified stratigraphy and the associated strengths and unit weights employed for the new embankment fill and the native overburden deposits encountered at the approach embankment areas.



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Embankment	Soil Type	Unit Weight, γ	Undrained Shear Strength, s_u	Cohesion, c'	Effective Friction Angle, ϕ'
East and West Approach Embankment	Granular 'A' Soil Mass	21 kN/m ³	-	0 kPa	34°
	Granular 'B' Type II Embankment Fill	22 kN/m ³	-	0 kPa	34°
	Compact Silt to Sandy Silt (Elev. 322.8 m to 320.0 m)	19 kN/m ³	-	0 kPa	31°
	Very Loose to Loose Silt to Silty Sand (Elev. 320.0 m to 314.0 m)	19 kN/m ³	-	0 kPa	28°
	Compact Sandy Silt to Silty Sand (Elev. 314.0 m to 305.0 m)	19 kN/m ³	-	0 kPa	30°
	Compact to Dense Silt to Silty Sand (Elev. 305.0 m to 288.5 m)	19 kN/m ³	-	0 kPa	34°
	Very Dense Gravelly Sand to Sand and Gravel (below Elev. 288.5 m)	22 kN/m ³	-	0 kPa	36°

Two RSS walls sections were analyzed for the varying wall heights as shown on the GA drawing provided by MH. In these analyses, the height of the RSS walls was considered to extend from the top of the pavement structure to the top of the front facing footing as shown on the GA drawing. The analyses were carried out using a 1.4 m thick soil cover over the front facing footing (as per GA drawing) and 1.5H:1V granular fill slope in front of the toe of the RSS wall. If the wall configuration/geometry changes during the course of the detail design and is different from that assumed, further stability analyses should be carried out.

Given the required height of the RSS walls, the required minimum reinforced width of the RSS walls to obtain a Factor of Safety of 1.5 or greater against deep-seated global instability has been analyzed. The ratios of minimum reinforcement mass width to wall height for the two RSS walls heights are provided below.

RSS Wall Height	Minimum Ratio of Reinforced Mass Width to Wall Height
2.8 m	1.0
4.8 m	0.7

The Contract Drawings will need to specify the width of the reinforced soil mass as presented above.

6.10.3 Geotechnical Axial Resistance

Assuming that the RSS wall acts as a unit and uses the full width of the reinforced soil mass, as presented in Section 6.10.2, the factored geotechnical axial resistance at ULS given below may be used for assessment of the reinforced mass founded on well compacted Granular 'B' Type II.



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RSS Wall Height	Factored Geotechnical Axial Resistance at ULS
2.8 m	200 kPa
4.8 m	180 kPa

6.10.4 Resistance to Lateral Loads

The resistance to lateral forces/sliding resistance between the compacted granular fill of the RSS walls and the Granular 'B' Type II (i.e. embankment fill) subgrade should be calculated in accordance with Section 6.7.5 of the CHBDC. The coefficient of friction, $\tan \delta$, for the compacted reinforced soil mass and the compacted Granular 'B' Type II subgrade is given below.

Interface Material(s)	Coefficient of Friction ($\tan \delta$)
Reinforced Soil Mass on compacted Granular 'B' Type II	0.55

6.11 Gabion Walls

It is understood that up to 2 m high gabions walls may be required along these sections of the approach embankments beyond the east and west abutments as follows:

Embankment	Station/Location
East Approach	STA 19+985 to 20+030 (South and North side slopes)
West Approach	STA 20+090 to 20+120 (North side slope) STA 20+130 to 20+155 (Side side slope)

Where gabion walls are required, all gabion walls should be constructed in accordance with OPSS 512 (Installation of Gabions) with gabion stones meeting the G-10 gradation requirement as specified in Table 8 of OPSS.PROV 1004 (Aggregates – Miscellaneous).

6.11.1 Lateral Earth Pressures

Based on the cross-sections provided by MH, it is understood that rock fill will be used as back fill behind the gabion walls. As such, the following parameters (unfactored) for the coefficient of lateral pressure may be used for design:

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

If the gabion walls allow lateral yielding, active earth pressures may be used in the foundation design of the structure. If the gabion walls do not allow for 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*.

6.11.2 Resistance to Lateral Loads

The resistance to lateral forces/sliding resistance between the rock back fill of the gabion walls and various subgrade materials should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction, $\tan \delta$, for the compacted reinforced soil mass and various materials is given below.

Interface Material(s)	Coefficient of Friction ($\tan \delta$)
Gabion walls on very loose to compact silt	0.30
Gabion walls on very loose to compact existing sand and gravel to silt and sand fill	0.40
Gabion walls on compacted Granular 'B' Type II	0.55

The value presented above represents an unfactored value.

6.12 Approach Embankment Design

Based on the GA Drawing provided by MH, the proposed grade for the CN Overhead Structure will be at about Elevation 331.1 m, requiring placement of up to about 2 m and 1.7 m of fill to raise the existing east and west approach embankment grades, respectively.

Based on the investigated locations at this site, the east and west approach embankments are founded on deposits of very loose to compact silt to silty sand, underlain by a deposit of dense to very dense sand and gravel.

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

6.12.1 Stability

6.12.1.1 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.



6.12.1.2 Parameter Selection

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

For the purpose of the stability analysis, the groundwater level was assumed to be at Elevation 320 m, which is based on the piezometric groundwater level measured in Borehole B6-02 and groundwater level measurements in the open boreholes upon completion of drilling.

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

Embankment	Soil Type	Unit Weight, γ	Cohesion, c'	Effective Friction Angle, ϕ'
East and West Approach Embankment	Granular 'B' Type II Embankment Fill	22 kN/m ³	0 kPa	34°
	Rock Fill	19 kN/m ³	0 kPa	40°
	Existing Loose to Compact Silt to Silt and Sand Fill	20 kN/m ³	0 kPa	29°
	Compact Silt to Sandy Silt (Elev. 322.8 m to 320.0 m)	19 kN/m ³	0 kPa	31°
	Very Loose to Loose Silt to Silty Sand (Elev. 320.0 m to 314.0 m)	19 kN/m ³	0 kPa	28°
	Compact Sandy Silt to Silty Sand (Elev. 314.0 m to 305.0 m)	19 kN/m ³	0 kPa	30°
	Compact to Dense Silt to Silty Sand (Elev. 305.0 m to 288.5 m)	19 kN/m ³	0 kPa	34°
	Very Dense Gravelly Sand to Sand and Gravel (below Elev. 288.5 m)	22 kN/m ³	0 kPa	36°

6.12.1.3 Results of Analysis

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



Embankment	Embankment Height at Critical Section ^(a)	Side Slope Profile (Rock Fill Widening)	Minimum Factor of Safety
East Approach Embankment	5.0 m	1.25H:1V	≥ 1.3
West Approach Embankment	5.5 m	1.25H:1V	≥ 1.3

Note:

- (a) Embankment height includes an approximately 2 m high and 1.7 m high grade raise at east and west approach embankment, respectively.

6.12.2 Settlement

6.12.2.1 Methodology

To estimate the magnitude of expected settlement of the embankments, analyses were carried out at the critical section of the east and west approach embankments, corresponding to the highest grade raise and/or largest widening. Settlement analyses were carried out using 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);
- Self-weight compression of the new embankment fill (long-term).

The analyses were carried out at the critical sections of the approach embankments where the thickness of compressible foundation soils is up to about 36.6 m and as such, the estimated settlements represent the maximum value along the approach embankments.

6.12.2.2 Parameter Selection

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

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



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Embankment	Soil Type	Thickness ^(a)	Unit Weight, γ	Deformation Parameter(s)
East and West Approach Embankment	Existing Loose to Compact Silt to Silt and Sand Fill	1.0 m to 2.0 m	20 kN/m ³	E' = 6 MPa
	Compact Silt to Sandy Silt (Elev. 322.8 m to 320.0 m)	~2.8 m	19 kN/m ³	E' = 10 MPa
	Very Loose to Loose Silt to Silty Sand (Elev. 320.0 m to 314.0 m)	~6.0 m	19 kN/m ³	E' = 6 MPa
	Compact Sandy Silt to Silty Sand (Elev. 314.0 m to 305.0 m)	~9.0 m	19 kN/m ³	E' = 12 MPa
	Compact to Dense Silt to Silty Sand (Elev. 305.0 m to 288.5 m)	~16.5 m	19 kN/m ³	E' = 25 MPa
	Very Dense Gravelly Sand to Sand and Gravel	~2.3 m	22 kN/m ³	E' = 50 MPa

Note:

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

For the purpose of settlement analyses, the groundwater level was assumed to be located on average at Elevation 320 m, based on the piezometric groundwater level measured in Borehole B6-02 and groundwater level measurements in the open boreholes upon completion of drilling.

6.12.2.3 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
East Approach Embankment	30 mm to 40 mm
West Approach Embankment	15 mm to 25 mm

These settlements are expected to occur relatively quickly (i.e. during construction) in response to the grade raise.

6.12.2.4 Settlement of Rock Fill Embankment

It is understood that rock fill is to be used for the construction of the approach embankments widening and grade raise beyond the RSS wall areas as a result of the narrow right-of-way and as such, there will be settlement due to compression of the rock fill itself under self-weight along the east and west side of the approach embankments. The magnitude of settlement of the rock fill depends on the type of rock/strength of particles,



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size and shape of particles, gradation of rock fill, total height/thickness of fill and the method of construction and sequence of placement. Based on MTO's Guideline for Rock Fill Settlement and Rock Fill Quantity Estimates (2010), the estimated settlements of rock fill for the approach embankments are presented below.

Embankment	Thickness of Rock Fill	Estimated Settlement of Rock Fill		
		Short-Term	Long-Term	Total
East Approach Embankment Centreline	Up to about 2 m (above groundwater table)	10 mm	<5 mm	~15 mm
East Approach Embankment Side Slope	Up to about 4 m (above groundwater table)	20 mm	5 mm	25 mm
West Approach Embankment Centreline	Up to about 1.5 m (below groundwater table)	10 mm	<5 mm	~15 mm
West Approach Embankment Side Slope	Up to about 5 m (above groundwater table)	25 mm	5 mm	30 mm

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

6.12.3 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.

6.12.4 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.

6.12.5 Embankment Fill Placement

Placement of granular fill for the grade raise and rock fill for the construction of the approach embankment should be carried out in accordance with SP 206S03 (Earth Excavation, Grading; Rock Excavation, Grading) and compaction of granular fill should be in accordance with OPSS 501 (Compacting) as modified by SP 105S21, with inspection and field testing by qualified personnel during construction to confirm that appropriate materials are used and that adequate levels of compaction are achieved. The placement of rock fill should be carried out in a controlled manner to minimize voids and bridging by blading, dozing and 'chinking' the rock to form a dense, compact mass.



Rock fill embankments should be constructed with side slopes no steeper than 1.25 horizontal to 1 vertical (1.25H:1V).

6.13 Design and Construction Considerations

6.13.1 Overburden Excavation

The existing embankment fill 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).

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

Where rock fill is used for widening of the existing approach embankments, benching of the existing embankment side slopes should be carried out in accordance with OPSD 208.010 (Benching of Earth Slopes).

The excavation for the abutment front slopes will expose the silt to silty sand native overburden which is highly erodible. Consideration should be given to either placement of erosion protection on the exposed slope or cutting into the slope in the area of the exposed silt to silty sand and backfilling to the final slope configuration using Granular 'B' Type II.

6.13.2 Temporary Roadway Protection

Where temporary roadway protection is required, all temporary excavation support systems should be constructed in accordance with OPSS 529 (Temporary Protection Systems) as modified by SP 539S02 and the lateral movement should meet Performance Level 2.

6.13.3 Obstructions

It should be noted that boulders were encountered within the native gravelly sand to sand and gravel deposit during borehole advancement. The presence of such obstructions could affect the construction of deep foundations. If the deep foundation system adopted to support the abutments penetrates into/through the gravelly sand to sand and gravel deposit, 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 Mr. Christopher Ng, P.Eng., a geotechnical engineer and Associate with Golder. Mr. Jorge M. A. Costa, P.Eng., Golder's Designated MTO Contact for this project and a Principal with Golder, conducted an independent quality control review of the report.



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HIGHWAY 592 GWP 5270-07-00; WP 5270-07-01

Report Signature Page

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TZ/CN/JMAC/sm

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- Unified Facilities Criteria, U.S. Navy. 1986. NAVFAC Design Manual 7.02. Soil Mechanics, Foundation and Earth Structures. Alexandria, Virginia.
- ASTM International:
- | | |
|------------|---|
| ASTM D1586 | Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils |
|------------|---|
- Commercial Software:
- LPILE Plus (Version 5.0) by Ensoft Inc.
- Settle^{3D} (Version 2.0) by Rocscience Inc.
- Slide (Version 6.0) by Rocscience Inc.
- Contract Design Estimating and Documentation (CDED):
- | | |
|--------------------------|--|
| Special Provision 105S21 | Amendment to OPSS 501 – Water Requirements and Quality Control for Compaction – Method B |
| Special Provision 206S03 | Amendment to OPSS 206 – Earth Excavation, Grading; Rock Excavation, Grading. |



FOUNDATION REPORT - CN OVERHEAD STRUCTURE - HIGHWAY 592 GWP 5270-07-00; WP 5270-07-01

Special Provision 539S02 Amendment to OPSS 539 – Protection System

Ministry of Transportation Ontario:

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

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

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

RSS Design Guidelines. Engineering Standard Branch. September 2008.

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

Structural Manual. Provincial Highway Management Division, Highway Standards Branch, Bridge Office. April 2011.

Ontario Occupational Health and Safety Act:

Ontario Regulation 213 Construction Projects (as amended)

Ontario Provisional Standard Drawing:

OPSD 208.010	Benching of Earth Slopes
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 512	Construction Specification for Installation of Gabions
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 1004	Material Specification for Aggregates – Miscellaneous
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 - CN OVERHEAD STRUCTURE - HIGHWAY 592 GWP 5270-07-00; WP 5270-07-01

Table 1: Evaluation of Foundation Alternatives

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks / Consequences
Spread/strip footings founded on native silt to silty sand deposit (12 m long by 4.5 m wide)	NR	<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 upper portions of the very loose to compact silt to silty sand deposit will be low. 	<ul style="list-style-type: none"> Lower relative cost than driven pile, drilled steel casing and micropile foundation options. 	<ul style="list-style-type: none"> Large footings will be required to develop adequate axial capacity. However, this will result in large settlement, which will cause underside of the bridge deck to encroach within the CN clearance envelope.
Spread/strip footings founded on 0.5 m of compacted Granular 'A' pad over deep soil mixed columns / improved ground (12 m long by 4.5 m wide)	4	<ul style="list-style-type: none"> Relative ease of construction. Higher axial capacity compared to spread/strip footings on untreated native overburden. 	<ul style="list-style-type: none"> Allows only for semi-integral abutment design. The increased in axial capacity on the deep soil mixed columns may not be sufficient for design. 	<ul style="list-style-type: none"> Higher relative cost than driven piles but lower relative cost than drilled steel casing and micropile foundation options. Additional cost for waste/spoil disposal. 	<ul style="list-style-type: none"> Larger footings may be required to develop adequate axial capacity. Soil mixed columns may not provide a "homogeneous" bearing stratum for the full footprint of the footings.
Driven steel H-piles (HP 310x110)	1	<ul style="list-style-type: none"> Negligible post-construction settlement. Higher axial capacity than spread/strip footings. Allows for integral abutment design. 	<ul style="list-style-type: none"> Axial capacity will be developed through shaft resistance for friction piles foundation option. Long piles will be required for end-bearing piles founded on refusal / bedrock. 	<ul style="list-style-type: none"> Higher relative cost than spread/strip footing on untreated native overburden. 	<ul style="list-style-type: none"> Potential requirement to drive friction piles deeper to develop adequate axial capacity during construction. If H-piles are to be driven to refusal/bedrock, there is potential difficulty for driving or damage piles when penetrating through the boulders present in the gravelly sand to sand and gravel deposit over the bedrock.
Drilled steel casings using DTH hammer drilling system (610 mm)	2	<ul style="list-style-type: none"> Negligible post-construction settlement. Reduced length and/or reduced number of deep 	<ul style="list-style-type: none"> Allows only for semi-integral abutment design. Axial capacity will be 	<ul style="list-style-type: none"> Higher relative cost than spread/strip footing and driven pile foundation options. 	<ul style="list-style-type: none"> Potential for unbalanced head in liners during installation may result in base heave and possible



FOUNDATION REPORT - CN OVERHEAD STRUCTURE - HIGHWAY 592 GWP 5270-07-00; WP 5270-07-01

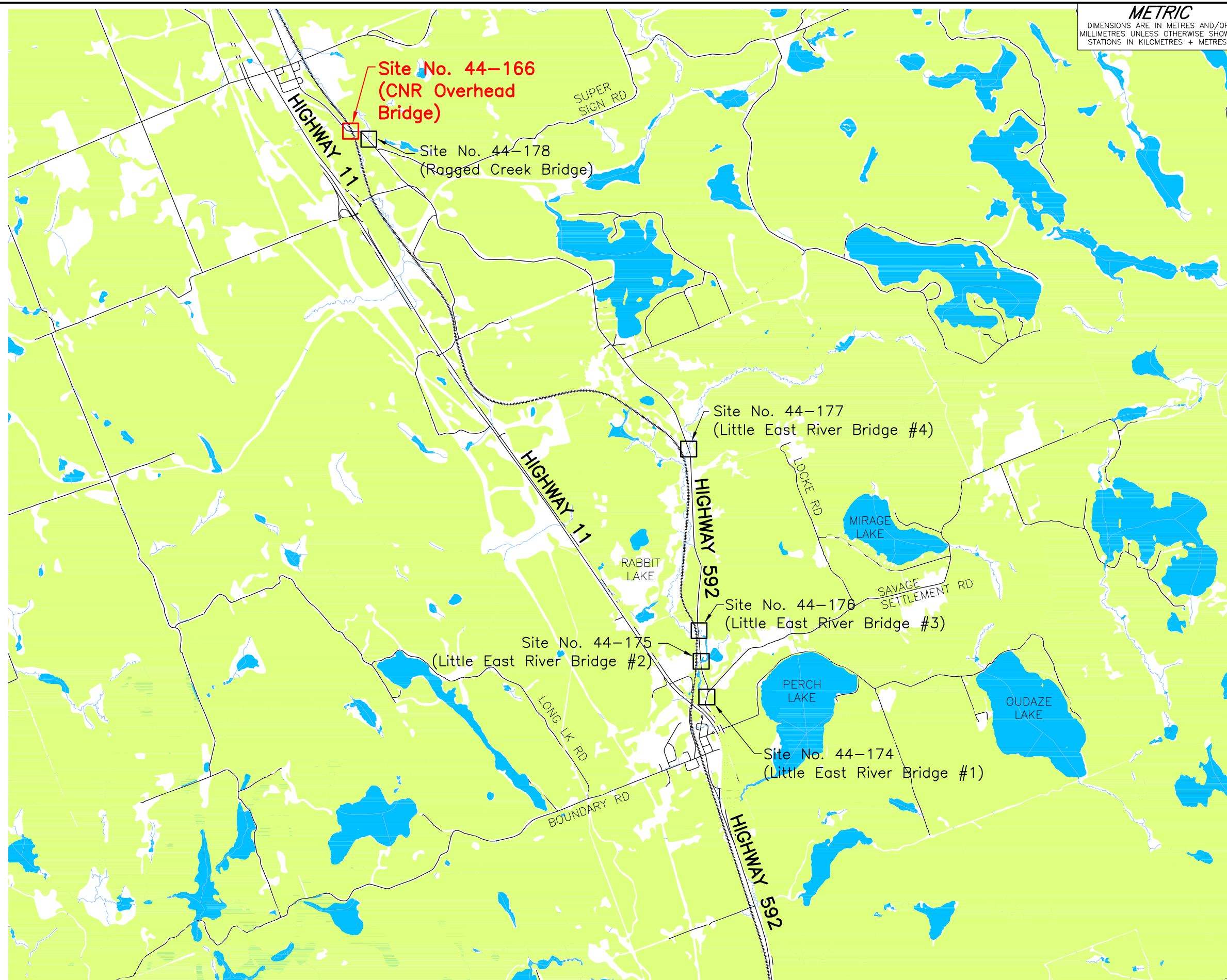
Table 1: Evaluation of Foundation Alternatives

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks / Consequences
		<p>foundation elements compared to steel H-piles.</p> <ul style="list-style-type: none"> ■ DTH drilling can readily penetrate through boulder in overburden, where present. 	<p>developed through shaft resistance for friction steel casing foundation option.</p> <ul style="list-style-type: none"> ■ Long steel casings will be required for casings socketted into bedrock. ■ Drilling slurry will be required to balance groundwater pressures and minimize basal heave. ■ Requires larger (drilling) equipment as compared to micropile drilling equipment. 	<ul style="list-style-type: none"> ■ Additional cost for specialized drilling equipment. ■ Additional cost associated with the need for drilling slurry, management / disposal of cutting returns. 	<p>loss of ground.</p> <ul style="list-style-type: none"> ■ Specialized drilling equipment and/or methods could be required to penetrate cobbles present in the sand and gravel deposit. ■ Advancing drilled steel casing to bedrock and/or refusal conditions through the thick non-cohesive deposits.
Micropiles (263 mm diameter)	3	<ul style="list-style-type: none"> ■ Negligible post-construction settlement. ■ Potential for achieving high axial capacity in the overburden using pressure grouting techniques. ■ 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. 	<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 the micropile pile load tests. 	<ul style="list-style-type: none"> ■ Few contractors have experience with soil-bonded micropile installation on MTO projects could result in higher construction costs and/or require additional oversight during construction.

Note: NR – Not Recommended



DRAWINGS

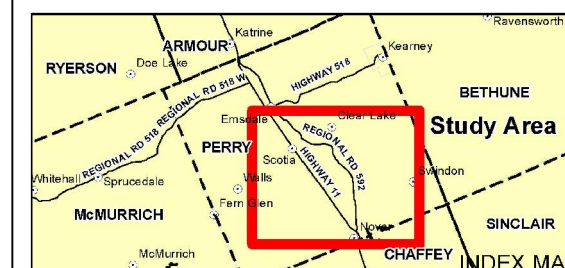


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DIMENSIONS ARE IN METRES AND/OR
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STATIONS IN KILOMETRES + METRES.

SHEET



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

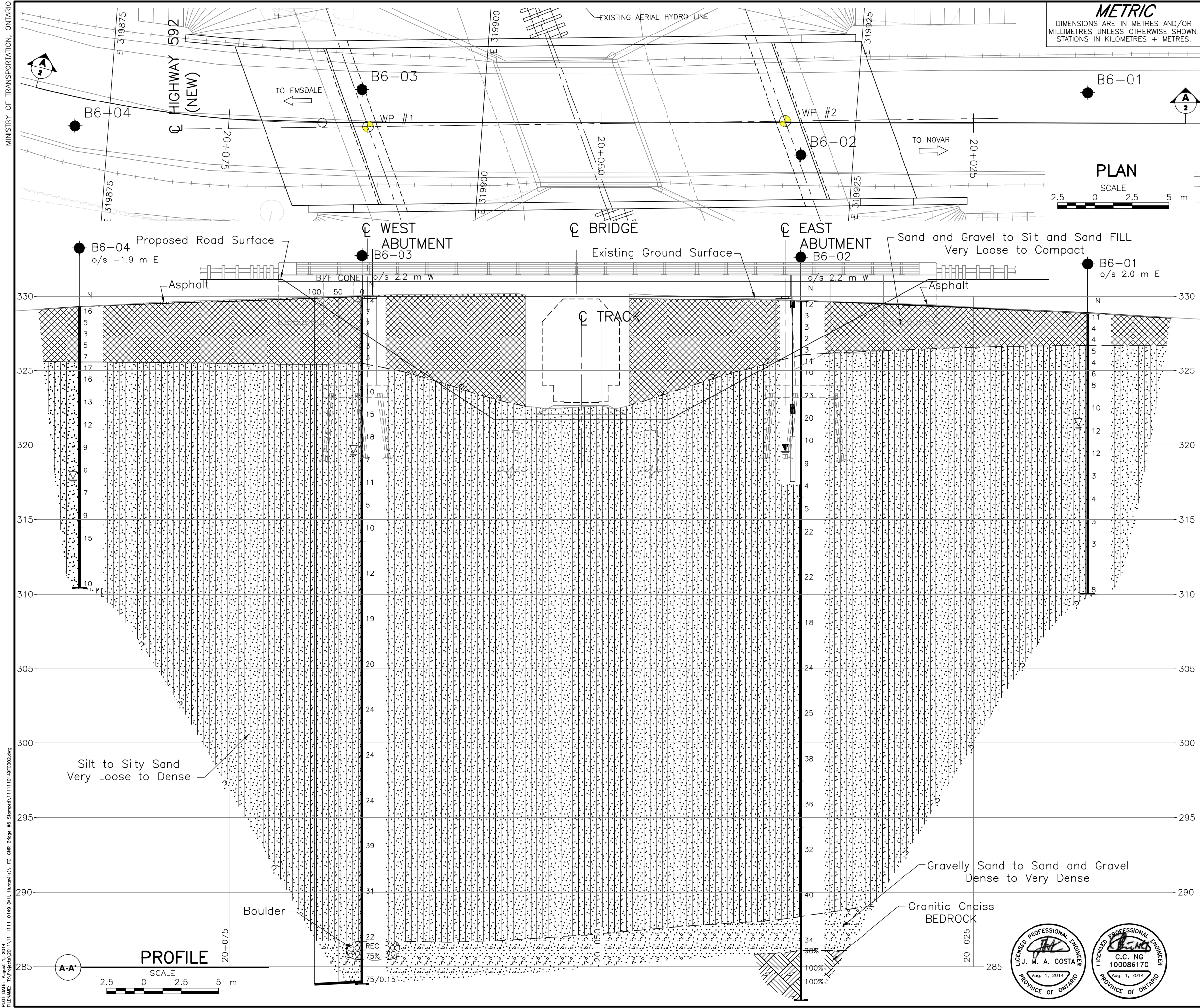


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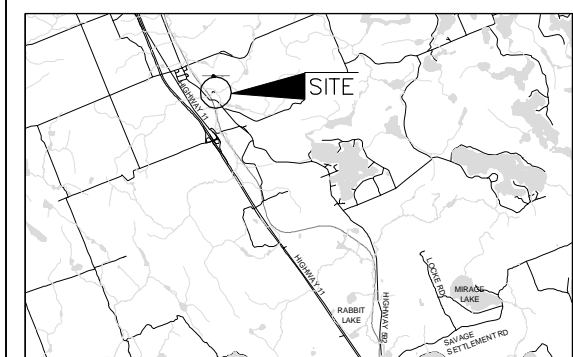
REFERENCE

Base data – MNR NRVIS, obtained 2004, CANMAP v2006.4 Produced by
Golder Associates Ltd. under licence from Ontario Ministry of Natural
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CONT No.
WP No. 5270-07-01HIGHWAY 592
CNR OVERHEAD BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

**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)
- REC Total Recovery
- 100% Rock Quality Designation (RQD)
- WL in piezometer, measured on June 19, 2013
- WL upon completion of drilling
- R Refusal

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
B6-01	328.9	5042389.4	319940.4
B6-02	329.8	5042383.8	319921.5
B6-03	330.0	5042386.0	319891.8
B6-04	329.3	5042382.1	319872.8

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. 44166-01.dwg, received January 28, 2014.

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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

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

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 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 B6-01		SHEET 1 OF 2	METRIC
W.P. 5270-07-01	LOCATION N 5042389.4 ; E 319940.4	ORIGINATED BY ID			
DIST HWY 592	BOREHOLE TYPE 120 mm O.D. Hollow Stem Augers and NW Casing	COMPILED BY GRL/AV			
DATUM Geodetic	DATE June 21, 2013	CHECKED BY TVA			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W _p	W	W _L		
								UNCONFINED ○ QUICK TRIAXIAL	FIELD VANE + REMOULDED ×					
328.9	GROUND SURFACE													
0.0	Asphalt (25 mm)		1A	SS	11									
0.4	Sand and gravel (FILL) Compact Brown Moist Silt and sand, trace gravel (FILL) Loose to compact Brown and grey Moist to wet		1B											
			2	SS	4									
			3	SS	4									
326.7														
2.2	Sandy SILT, trace to some clay, trace gravel Very loose to compact Brown becoming grey below a depth of 4.6 m Wet		4	SS	5									
			5	SS	4									
			6	SS	6									
			7	SS	8									
			8	SS	10									
			9	SS	12									
			10	SS	12									
			11	SS	3									
			12	SS	4									
315.6														
13.3	SILT, trace sand, trace clay Very loose Grey Wet		13	SS	3									
314.1														
14.8														

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

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PROJECT 11-1111-0149			RECORD OF BOREHOLE No B6-01			SHEET 2 OF 2			METRIC															
W.P. 5270-07-01			LOCATION N 5042389.4 ;E 319940.4			ORIGINATED BY ID																		
DIST HWY 592			BOREHOLE TYPE 120 mm O.D. Hollow Stem Augers and NW Casing			COMPILED BY GRL/AV																		
DATUM Geodetic			DATE June 21, 2013			CHECKED BY TVA																		
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS			ELEVATION SCALE			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES																			
	--- CONTINUED FROM PREVIOUS PAGE ---																							
	Silty SAND Very loose to loose Grey Wet		14	SS	3																			
310.0			15	SS	8																			
18.9	END OF BOREHOLE																							
	NOTES: 1. Water level at a depth of 7.6 m below ground surface (Elev. 321.3 m) upon completion of drilling. 2. Borehole caved at a depth of 11.3 m below ground surface (Elev. 317.6 m) upon completion of drilling.																							

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PROJECT <u>11-1111-0149</u>		RECORD OF BOREHOLE No B6-02		SHEET 2 OF 4	METRIC
W.P. <u>5270-07-01</u>		LOCATION <u>N 5042383.8 ; E 319921.5</u>		ORIGINATED BY <u>ID</u>	
DIST <u> </u> HWY <u>592</u>		BOREHOLE TYPE <u>120 mm O.D. Hollow Stem Augers and NW Casing</u>		COMPILED BY <u>GRL/AV</u>	
DATUM <u>Geodetic</u>		DATE <u>June 17 to 20, 2013</u>		CHECKED BY <u>TVA</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT			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							W _p	W	W _L
	--- CONTINUED FROM PREVIOUS PAGE ---																
	Silty SAND, trace clay Loose to dense Grey Wet		14	SS	22		314										
							313										
							312										
			15	SS	22		311										
							310										
							309										
			16	SS	18		308										
							307										
							306										
			17	SS	24		305										
							304										
							303										
			18	SS	25		302										
							301										
							300										

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

PROJECT 11-1111-0149				RECORD OF BOREHOLE No B6-02				SHEET 4 OF 4				METRIC					
W.P. 5270-07-01				LOCATION N 5042383.8 ; E 319921.5				ORIGINATED BY ID									
DIST HWY 592				BOREHOLE TYPE 120 mm O.D. Hollow Stem Augers and NW Casing				COMPILED BY GRL/AV									
DATUM Geodetic				DATE June 17 to 20, 2013				CHECKED BY TVA									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
	--- CONTINUED FROM PREVIOUS PAGE ---						20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W _p W W _L 20 40 60					
282.8	Granitic Gneiss (BEDROCK)		2	RC	REC 100%		284										RQD = 100%
			3	RC	REC 100%		283										RQD = 100%
47.0	END OF BOREHOLE																
	NOTES: 1. Water level measured in open borehole at a depth of 8.7 m below ground surface (Elev. 321.1 m) during drilling on June 18, 2013. 2. An additional borehole was advanced about 2.0 m East of Borehole B6-02 to install a piezometer. 3. Water level measurements in Piezometer: Date Depth (m) Elev. (m) 06/19/13 10.2 319.6 4. Piezometer decommissioned on June 19, 2013.																

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SHEET 1 OF 1

DATUM: Geodetic

DRILLING CONTRACTOR: Landcore

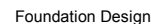
CHECKED: TVA

PROJECT 11-1111-0149		RECORD OF BOREHOLE No B6-03		SHEET 1 OF 4		METRIC											
W.P. 5270-07-01		LOCATION N 5042386.0 ; E 319891.8		ORIGINATED BY ID													
DIST HWY 592		BOREHOLE TYPE 120 mm O.D. Hollow Stem Augers and NW Casing		COMPILED BY AV													
DATUM Geodetic		DATE June 24 and 25, 2013		CHECKED BY TVA													
SOIL PROFILE			SAMPLES			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	W _p	W	W _L	γ	GR	SA	SI	CL
330.0	GROUND SURFACE																
0.0	Asphalt (25 mm)		1A	SS	14												
0.3	Sand and gravel (FILL) Compact Brown Moist		1B														
	Silt, some sand, trace clay (FILL) Very loose to compact Grey Moist becoming wet below a depth of 3.1 m		2	SS	7		329										
			3	SS	2		328										
			4	SS	2												
	Containing sand pockets below a depth of 3.1 m.		5	SS	3		327										
			6	SS	3		326										
325.5	SILT, trace sand, trace clay Loose to compact Grey Wet		7	SS	7		325										
			8	SS	10		324										
			9	SS	15		323										
			10	SS	18		322										
							321										
319.8	SILT and SAND, trace clay Loose to compact Grey Wet		11	SS	7		320										
			12	SS	11		319										
							318										
							317										
316.7	Sandy SILT, trace clay Loose Grey Wet		13	SS	5		316										

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

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

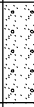
PROJECT <u>11-1111-0149</u>		RECORD OF BOREHOLE No B6-03		SHEET 3 OF 4	METRIC
W.P. <u>5270-07-01</u>	LOCATION <u>N 5042386.0 ; E 319891.8</u>	ORIGINATED BY <u>ID</u>			
DIST <u> </u> HWY <u>592</u>	BOREHOLE TYPE <u>120 mm O.D. Hollow Stem Augers and NW Casing</u>	COMPILED BY <u>AV</u>			
DATUM <u>Geodetic</u>	DATE <u>June 24 and 25, 2013</u>	CHECKED BY <u>TVA</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W _p	W	W _L		GR	SA	SI	CL	
-- CONTINUED FROM PREVIOUS PAGE --								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					WATER CONTENT (%)								
	SILT and SAND, trace clay Compact to dense Grey Wet		19	SS	24		299							○			0	56	40	4	
							298														
							297														
			20	SS	24		296														
							295														
							294														
			21	SS	39		293														
							292														
							291														
			22	SS	31		290														
							289														
							288														
			23	SS	22		287														
							286														
286.1 43.9			1	RC	REC 79%																

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

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PROJECT 11-1111-0149			RECORD OF BOREHOLE No B6-03			SHEET 4 OF 4			METRIC																				
W.P. 5270-07-01			LOCATION N 5042386.0; E 319891.8			ORIGINATED BY ID																							
DIST HWY 592			BOREHOLE TYPE 120 mm O.D. Hollow Stem Augers and NW Casing			COMPILED BY AV																							
DATUM Geodetic			DATE June 24 and 25, 2013			CHECKED BY TVA																							
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS			ELEVATION SCALE			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p			NATURAL MOISTURE CONTENT W			LIQUID LIMIT W _L			UNIT WEIGHT γ			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES																								
284.0	Gravelly SAND, some silt, trace clay Very dense Grey Wet		1	RC	REC 79%																								
46.2	Boulder encountered at a of 44.4 m END OF BOREHOLE END OF DCPT Refusal to Further Penetration (100 Blows / 0.15 m) NOTE: 1. Water level measured at a depth of 10.5 m below ground surface (Elev. 319.5 m) during drilling on June 24, 2013.		24	SS	75/0.15																								

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PROJECT <u>11-1111-0149</u>		RECORD OF BOREHOLE No B6-04		SHEET 1 OF 2		METRIC	
W.P. <u>5270-07-01</u>		LOCATION <u>N 5042382.1 ; E 319872.8</u>		ORIGINATED BY <u>ID</u>			
DIST <u> </u> HWY <u>592</u>		BOREHOLE TYPE <u>120 mm O.D. Hollow Stem Augers and NW Casing</u>		COMPILED BY <u>GRL/AV</u>			
DATUM <u>Geodetic</u>		DATE <u>June 20, 2013</u>		CHECKED BY <u>TVA</u>			

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									
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED									
329.3	GROUND SURFACE						20	40	60	80	100						
0.0	Asphalt (25 mm)		1A	SS	16												
0.3	Sand and gravel (FILL) Compact Brown Moist		1B														
	Silt, trace clay, trace gravel, trace to some sand, trace organics (FILL) Very loose to compact Grey Dry to moist		2	SS	5												
			3	SS	3												
			4	SS	5												
			5	SS	7												
325.6																	
3.7	SILT, trace to some sand, trace clay Loose to compact Grey Wet		6	SS	17												
			7	SS	16											0 7 89 4	
			8	SS	13												
			9	SS	12											0 12 84 4	
			10	SS	9											Non-plastic	
			11	SS	6												
			12	SS	7												
			13	SS	9											0 20 76 4	
314.5																	
14.8																	

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

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PROJECT 11-1111-0149			RECORD OF BOREHOLE No B6-04			SHEET 2 OF 2			METRIC																							
W.P. 5270-07-01			LOCATION N 5042382.1 ; E 319872.8			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 June 20, 2013			CHECKED BY TVA																										
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS			ELEVATION SCALE			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			SHEAR STRENGTH kPa			WATER CONTENT (%)			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES																											
	--- CONTINUED FROM PREVIOUS PAGE ---																															
310.4	SILT and SAND, trace clay Compact Grey Wet		14	SS	15																											
311																																
15			15	SS	10																											
18.9	END OF BOREHOLE																															
	NOTE: 1. Water level at a depth of 11.6 m below ground surface (Elev. 317.7 m) during drilling.																															

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APPENDIX B

Laboratory Test Results and Bedrock Core Photographs

Silt to Silt and Sand (Fill)

U.S.S Sieve size, meshes/inch

Size of openings, inches

PERCENT FINER THAN

GRAIN SIZE, mm

Grain Size (mm)	Percent Finer Than (Squares)	Percent Finer Than (Circles)
0.0075	5	3
0.015	8	6
0.03	15	12
0.06	45	35
0.1	85	65
0.2	95	85
0.4	98	95
0.75	100	98
1.5	100	100
3.0	100	100
6.0	100	100

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

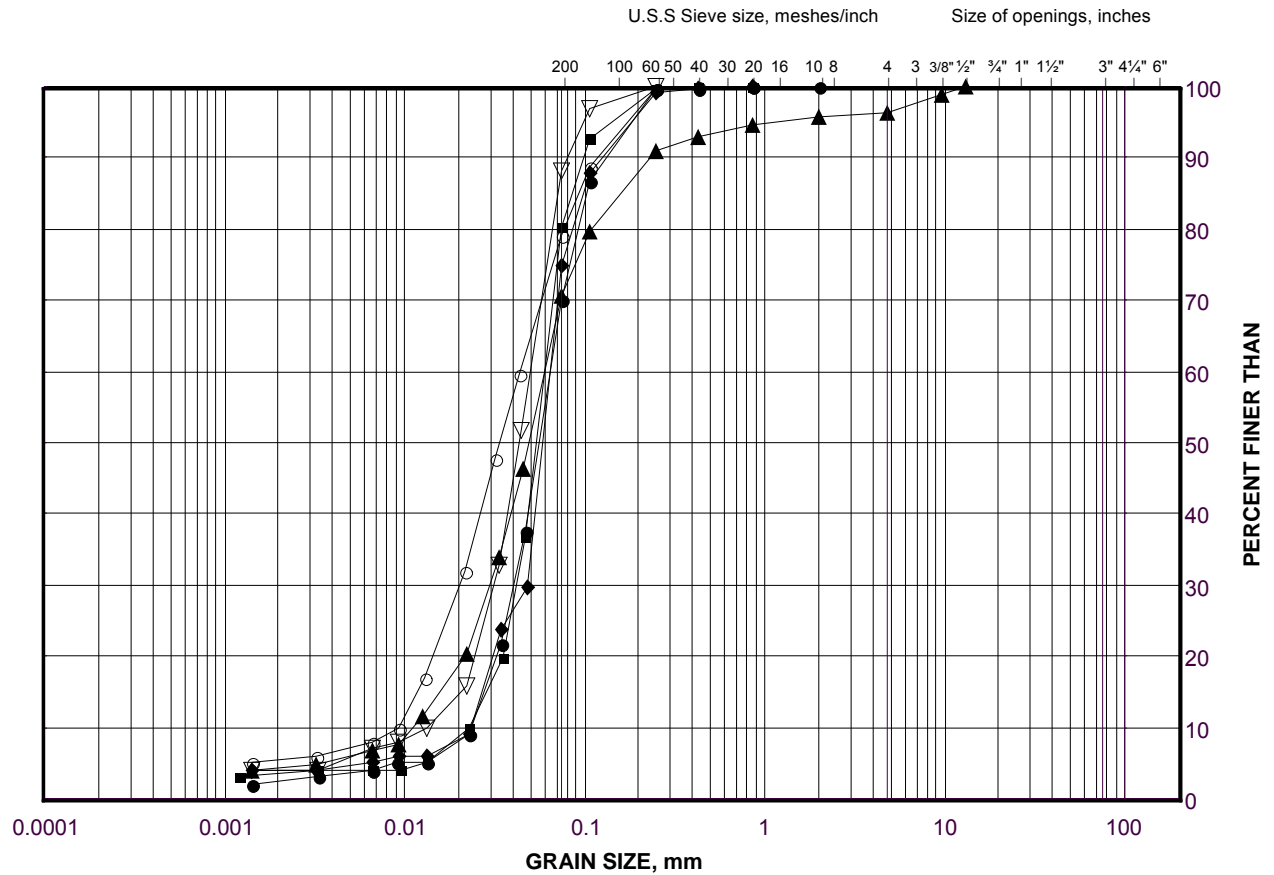
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	B6-02	3	327.9
■	B6-03	4	327.4

Date: 27-Mar-14

GRAIN SIZE DISTRIBUTION

Silt to Sandy Silt

FIGURE B2A



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	B6-01	11	317.9
■	B6-04	13	315.3
◆	B6-03	14	314.4
▲	B6-01	6	324.8
▽	B6-04	9	321.4
○	B6-01	9	321.0

Project Number: 11-1111-0149

Checked By: AV

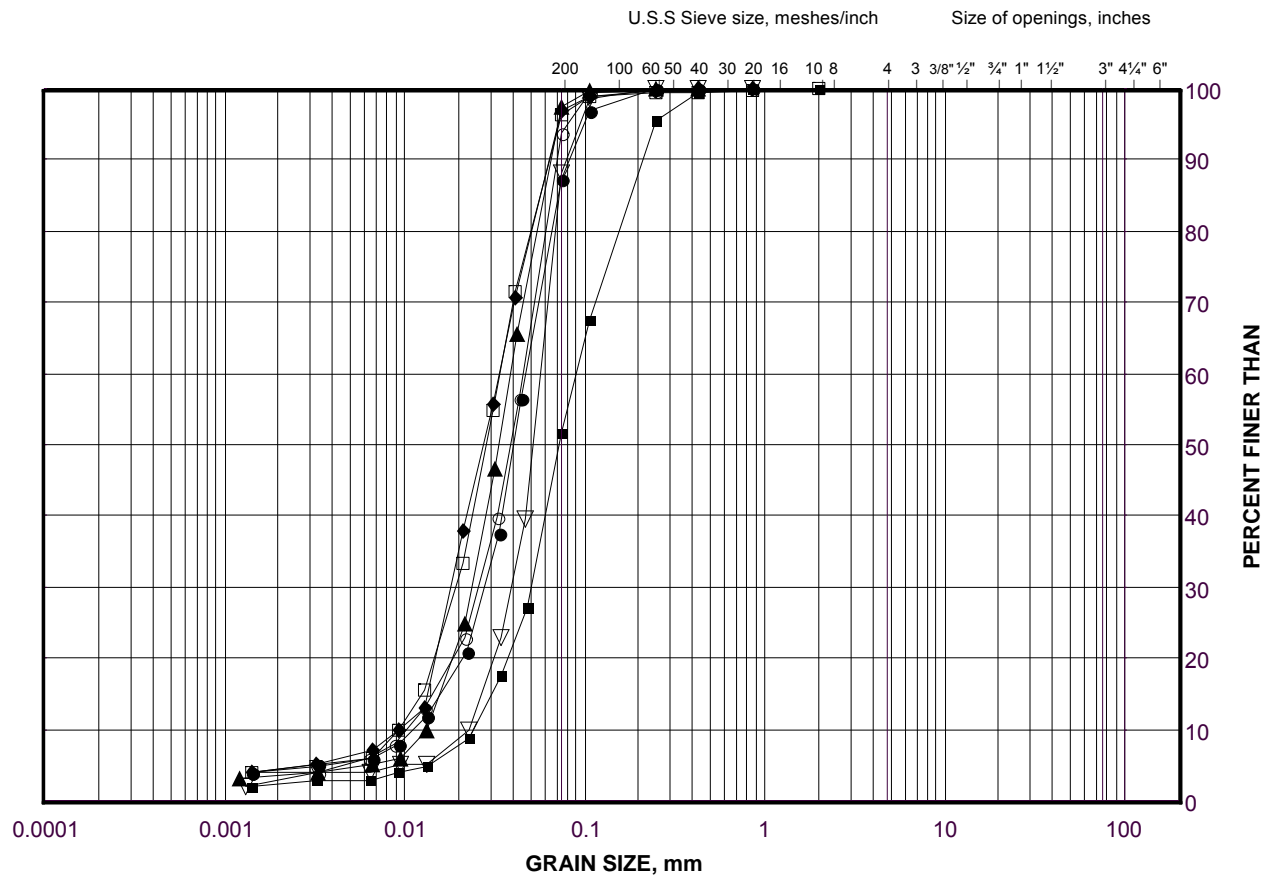
Golder Associates

Date: 27-Mar-14

GRAIN SIZE DISTRIBUTION

Silt to Silt and Sand

FIGURE B2B



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	B6-02	10	320.3
■	B6-03	12	317.5
◆	B6-02	12	317.3
▲	B6-01	13	314.9
▽	B6-02	6	325.7
○	B6-04	7	324.4
□	B6-03	8	323.7

Project Number: 11-1111-0149

Checked By: AV

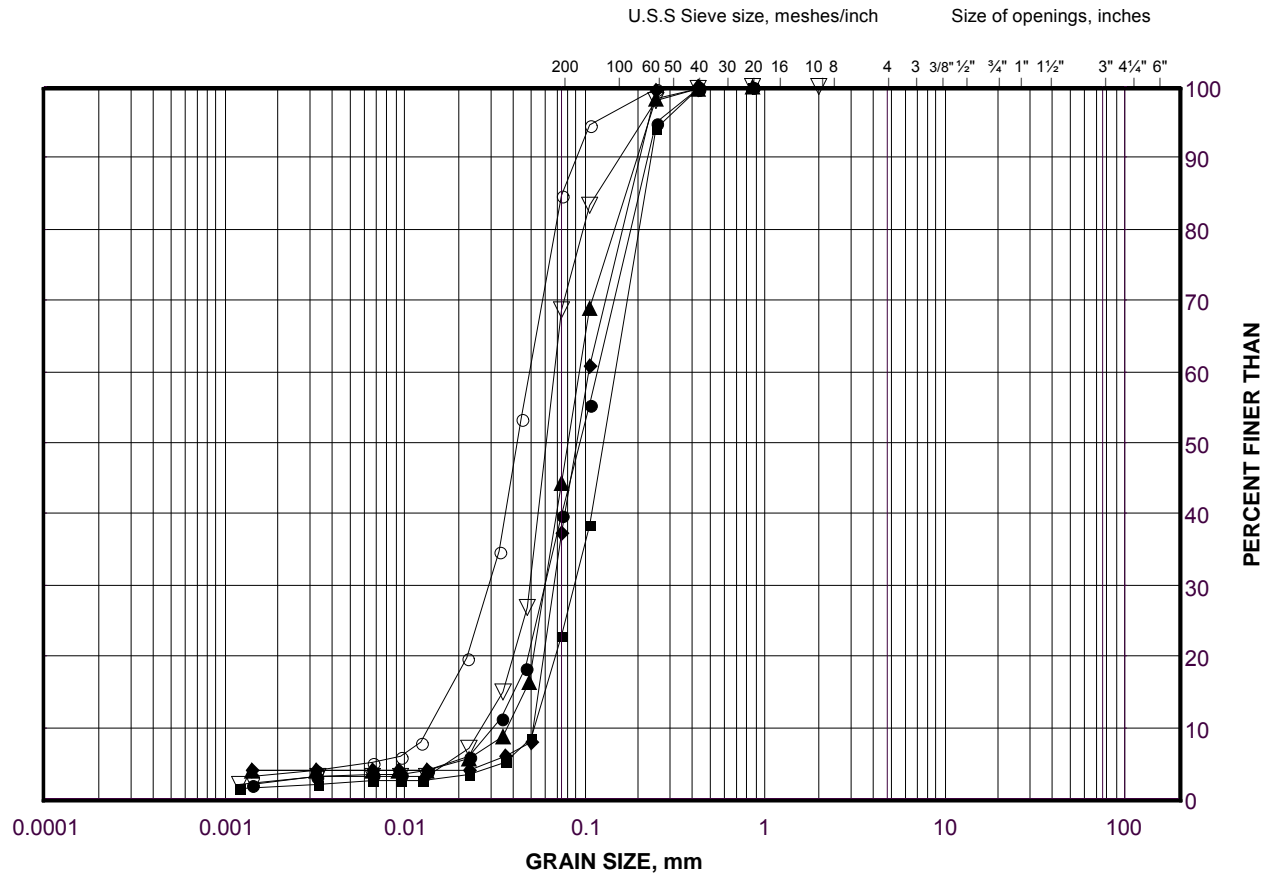
Golder Associates

Date: 27-Mar-14

GRAIN SIZE DISTRIBUTION

Silt to Silt and Sand to Silty Sand

FIGURE B2C



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	B6-04	15	310.7
■	B6-02	16	308.1
◆	B6-03	17	305.3
▲	B6-03	19	299.2
▽	B6-02	20	295.9
○	B6-02	21	292.8

Project Number: 11-1111-0149

Checked By: AV

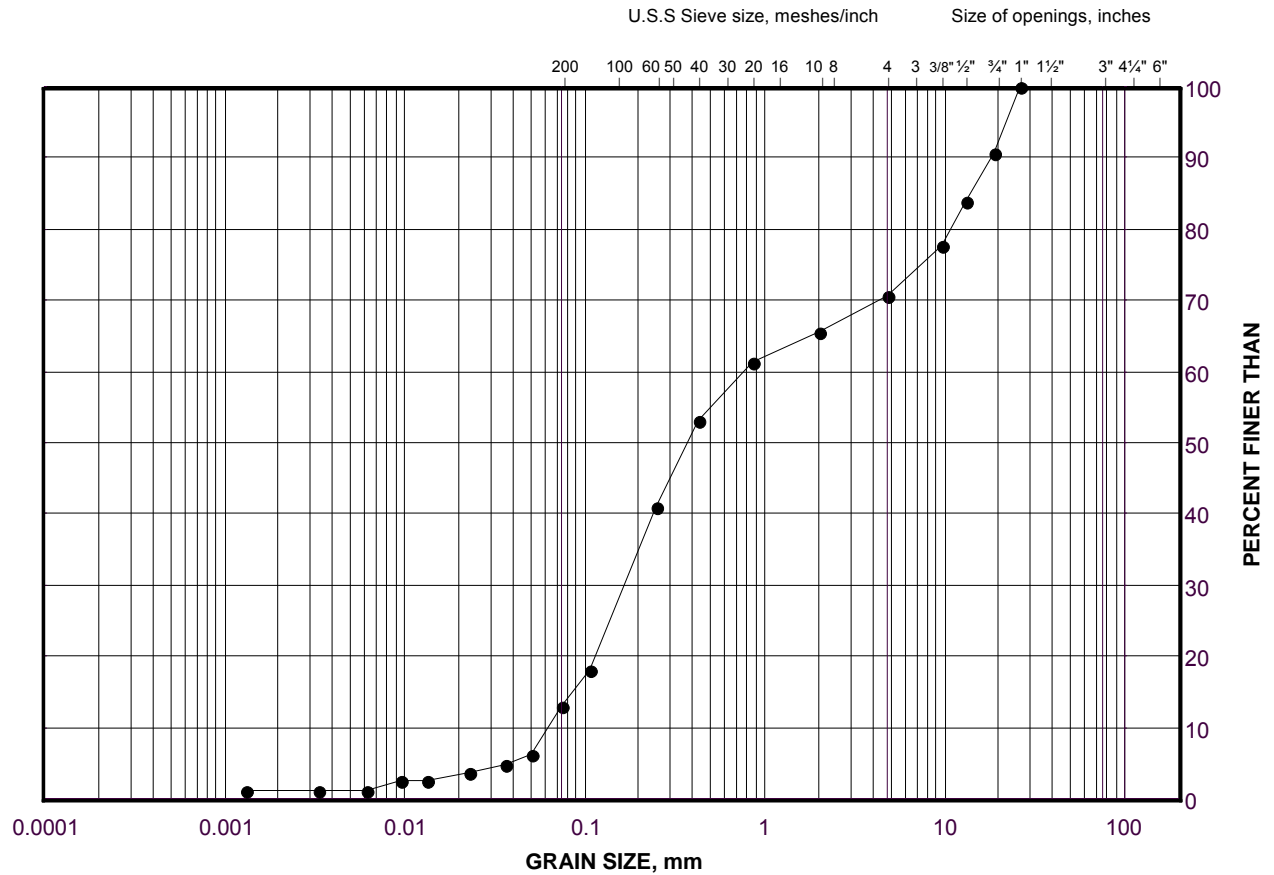
Golder Associates

Date: 27-Mar-14

GRAIN SIZE DISTRIBUTION

Gravelly Sand

FIGURE B3



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	B6-03	24	284.1

Project Number: 11-1111-0149

Checked By: AV

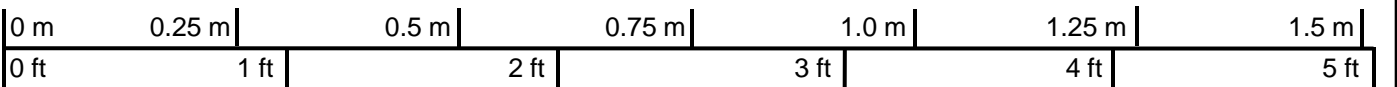
Golder Associates

Date: 27-Mar-14


Borehole B6-03



Box 1: 44.3 m – 45.4 m



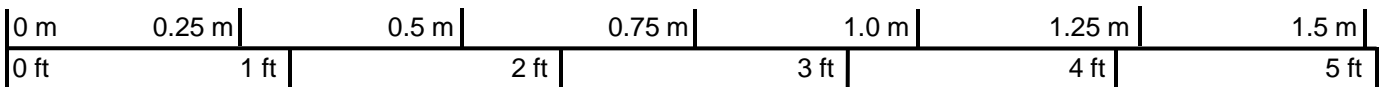
Scale

PROJECT				CN OVERHEAD STRUCTURE			
				Highway 592			
				GWP 5270-07-00; WP 5270-07-01			
TITLE				Boulder Core Photograph – Borehole B6–03			
				Highway 592			
				PROJECT No. 11-1111-0149		FILE No. ----	
				DESIGN	AV	AUG 13	SCALE NTS
				CADD	--	--	REV.
				CHECK	TVA	AUG 13	FIGURE B4
				REVIEW	JMAC	AUG 13	

Borehole B6-02



Box 1: 43.68 m – 46.97 m



Scale


PROJECT				CN OVERHEAD STRUCTURE Highway 592 GWP 5270-07-00; WP 5270-07-01			
TITLE				Bedrock Core Photograph – Borehole B6-02 Highway 592			
				PROJECT No. 11-1111-0149		FILE No. ----	
				DESIGN	AV	AUG 13	SCALE NTS
				CADD	--	--	REV.
				CHECK	TVA	AUG 13	FIGURE B5
				REVIEW	JMAC	AUG 13	

TABLE B1

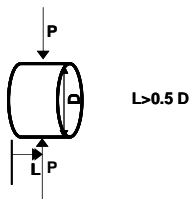
POINT LOAD TEST RESULTS

Borehole Number	Run Number	Sample Depth (m)	Sample Elevation (m)	Bedrock Description	Test Type	Core Length (mm)	Core ⁽²⁾ Diameter (mm)	Is (50mm) (MPa)	Approx. UCS Value ⁽¹⁾ (MPa)
B6-02	1	43.74	286.1	Granitic Gneiss	Diametral	90.00	47.60	7.91	119
B6-02	1	44.80	285.0	Granitic Gneiss	Diametral	85.00	47.60	6.58	99
B6-02	1	44.73	285.1	Granitic Gneiss	Diametral	90.00	47.60	7.41	111
B6-02	1	43.76	286.0	Granitic Gneiss	Axial	45.00	47.60	6.58	99
B6-02	1	44.74	285.1	Granitic Gneiss	Axial	40.00	47.60	7.91	119
B6-02	2	45.56	284.2	Mafic-Rich Grey Gneiss	Diametral	90.00	47.60	5.47	82
B6-02	2	45.54	284.3	Mafic-Rich Grey Gneiss	Axial	40.00	47.60	8.67	130
B6-02	3	46.69	283.1	Mafic-Rich Grey Gneiss	Axial	40.00	47.60	4.77	72

⁽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 $K = 15$ has been used based on a UCS test result.

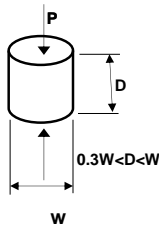
DIAMETRAL SPECIMEN SHAPE REQUIREMENTS

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	3
BOREHOLE NUMBER	B6-02	SAMPLE DEPTH, m	46.75-46.97

TEST CONDITIONS

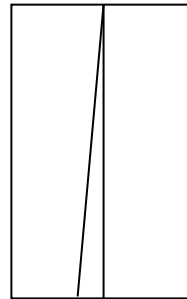
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.21

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.51	WATER CONTENT, (specimen) %	0.05
SAMPLE DIAMETER, cm	4.75	UNIT WEIGHT, kN/m ³	28.18
SAMPLE AREA, cm ²	17.71	DRY UNIT WT., kN/m ³	28.16
SAMPLE VOLUME, cm ³	186.07	SPECIFIC GRAVITY	-
WET WEIGHT, g	534.80	VOID RATIO	-
DRY WEIGHT, g	534.53		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	72.9
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REMARKS:	DATE: 7/2/2013
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CHECKED BY: AV	REVIEWED BY: TVA/JMAC
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APPENDIX C

Non-Standard Special Provisions

OBSTRUCTIONS

Special Provision

SCOPE

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

BASIS OF PAYMENT

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

END OF SECTION

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

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