FOUNDATION INVESTIGATION AND DESIGN REPORT

Sign Support Structures
Highway 401/Kingston Road 38 Interchange Improvements, Kingston, Ontario
MTO GWP 4049-11-00
Agreement No. 9016-E-0007, Assignment No. 1

Submitted to:

Ministry of Transportation, Ontario
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FOUNDATION INVESTIGATION REPORT
SIGN SUPPORT STRUCTURES
HIGHWAY 401/KINGSTON ROAD 38 INTERCHANGE IMPROVEMENTS, KINGSTON, ONTARIO
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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by the Ministry of Transportation, Ontario (MTO) under MTO's Foundation Engineering Retainer at Various Locations for Eastern Region and Central Region (Areas of York, Simcoe, Toronto and Durham) as Assignment No.1 of Agreement No. 9016-E-0007 to provide foundation engineering services for two proposed sign support structures as part of the proposed improvements to the Highway 401 interchange at Kingston Road 38, in the City of Kingston, Ontario. The locations of the sites are shown on the Key Plan on Drawings 1 and 2.

The Terms of Reference (TOR) and Scope of Work for the foundation investigation are outlined in MTO's Request for Proposal titled, "To Provide Foundation Engineering Services on Retainer, Various Locations in Eastern Region and Central Region (Areas of York, Simcoe, Toronto and Durham), Assignment Number: 9016-E-0007 & 9016-E-0008" dated September 12, 2016 and associated clarifications. The detailed Scope of Work for this assignment is presented in the Golder's Understanding of Scope documents for Work Orders No. 1 and No. 2 and respective Appendices 3B (dated December 20, 2016). Authorization to proceed with this assignment was provided by MTO on January 16, 2017 (Work Order No. 1) and July 28, 2017 (Work Order No. 2).

2.0 SITE DESCRIPTION

The sites of the two proposed sign support structures are located on the south side of the eastbound lanes of Highway 401 at about STA 16+503 and STA 17+503, in the City of Kingston. The sign support at STA 16+503 is located approximately 1.4 km west of the Kingston Road 38 overpass and the sign support at STA 17+503 is located approximately 0.5 km west of the Kingston Road 38 overpass, in the City of Kingston. This section of Highway 401 consists of a five lane divided highway, comprised of three westbound and two eastbound lanes. The site locations and general topography and features are shown on Drawings 1 and 2. Photographs of the general site conditions at the proposed sign support locations are presented on Figure 1.

3.0 INVESTIGATION PROCEDURES

The field work for the foundation investigation was carried out on January 17 and 18 and March 1, 2018 during which time a total of two boreholes, designated Boreholes 7 and 8, were advanced at the locations shown on Drawings 1 and 2.

The fieldwork was completed using CME-55 truck-mounted and CME-55 track-mounted drill rigs supplied and operated by Marathon Drilling Co. Ltd. of Greely, Ontario. The boreholes were advanced through the overburden using 210 mm outer diameter hollow stem augers. In general, soil samples were obtained at 0.75 m intervals of depth, using a 50 mm outer diameter split-spoon sampler driven by automatic hammers in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586-08)[1]. Bedrock coring at the boreholes was carried out using a 'NQ' core barrel. The open boreholes were backfilled with bentonite upon completion in accordance with Ontario Regulation 903 (Wells) (as amended). The groundwater conditions and water levels in the open boreholes were observed during drilling and immediately following drilling operations.

[1] ASTM D1586-08a – Standard Test Method for Standard Penetration Tests and Split Barrel Sampling of soils.

The field work was observed by a member of Golder's engineering and technical staff who located the boreholes, arranged for the clearance of underground utility services, observed the drilling, sampling and in-situ testing operations, and logged the boreholes. The samples were identified in the field, placed in appropriate containers, labelled and transported to Golder's Whitby and Mississauga geotechnical laboratories where the samples underwent further visual examination. Classification testing (water content, Atterberg limits and grain size distribution) was carried out on selected soil samples and unconfined compression (UC) tests were completed on two selected specimens of the bedrock core, all in accordance with MTO and/or ASTM standards, as applicable.

Two selected samples (one soil and one rock) were submitted to AGAT Laboratories, a Standards Council of Canada (SCC) accredited laboratory of Mississauga, Ontario, for chemical analysis. The sample was analyzed for a suite of corrosivity parameters, including conductivity, resistivity, soluble chloride, soluble sulphate, sulfide, redox potential and pH.

The borehole locations and ground surface elevations were obtained using a GPS (Trimble XH 3.5G), having an accuracy of 0.1 m in the vertical and 0.1 m in the horizontal directions. The borehole locations given on the borehole records and shown on Drawings 1 and 2 are positioned relative to MTM NAD 83 (Zone 9) northing and easting coordinates and the ground surface elevations are referenced to Geodetic Datum. The borehole locations, ground surface elevations and borehole depths are summarized below.

Borehole No.	Foundation Element	Location		Ground Surface Elevation (m)	Borehole Depth (m)
		Northing (m) (Latitude, °)	Easting (m) (Longitude, °)		
7	Sign STA 16+503	4905455.9 (44.289074)	297899.6 (-76.586463)	95.5	8.0*
8	Sign STA 17+503	4905333.4 (44.287980)	298806.8 (-76.575093)	94.8	4.5*

Note: *includes coring for lengths of 3.4 m and 3.9 m in the respective boreholes.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

This project area is located within the Napanee Plain physiographic region, as delineated in *The Physiography of Southern Ontario* (Chapman and Putman, 1894)[2]. The Napanee Plain is flat to undulating, and is characterized by relatively shallow soil deposits overlying bedrock. Geologic mapping[3] indicates that the bedrock within the Napanee Plain consists of grey limestone of the Gull River Formation (of the Trenton-Black River Group), which contains some shale partings and seams.

[2] Chapman, L.J. and Putman, D.F., 1984, The Physiography of Southern Ontario, Ontario Geological Society, Special Volume 2, Third Edition. Accompanied by Map p. 2715, Scale 1:600,000.)

[3] Map 2544, Ontario Geological Society, 1991. Geology of Ontario. Special Volume 4, Part 1. Eds. P.C. Thurston, H.R. Williams, R.H. Sutcliffe and G.M. Stott. Ministry of Northern Development and Mines, Ontario.

The overburden soils within the Napanee Plain generally consist of glacial till, although alluvium is present in river and stream valleys and, in the southern portion of the Plain, low-lying areas are typically covered with deposits of stratified clay. Water well records indicate that the average depth to bedrock within the Napanee Plain is approximately 2 m. However, in many areas, bedrock outcrops exist at ground surface, while deeper soil deposits (on the order of 10 m) are present in the southern portion of the Napanee Plain, and within and adjacent to river valleys throughout the Plain.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions as encountered in the boreholes advanced during this investigation and the results of the laboratory tests carried out on selected soil and bedrock core samples/specimens are presented on the borehole records provided in Appendix A. The results of the in-situ field tests (i.e. SPT “N”-values) as presented on the borehole/drillhole records and in Section 4.2 are uncorrected. The geotechnical laboratory testing plots, photographs of the bedrock core samples and photographs of the UC test specimens and laboratory test sheets are contained in Appendix B. The detailed results of the analytical laboratory testing of one soil (fill) and one rock sample are sample is presented in Appendix C.

The stratigraphic boundaries shown on the borehole records and on the cross-sections on Drawings 1 and 2 are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests and in-situ field tests. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Furthermore, subsurface conditions will vary between and beyond the borehole locations, however; the factual data presented in the borehole records governs any interpretation of the site conditions. It should be noted that the interpreted stratigraphy shown on Drawings 1 and 2 is a simplification of the subsurface conditions.

In general, the subsurface conditions consist of: asphaltic concrete and granular fill, underlain by bedrock in the area of proposed Sign Support at STA 16+503; and topsoil / crushed rock fill underlain by a clayey silt deposit, underlain by bedrock in the area of proposed Sign Support at STA 17+503. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Asphalt

Borehole 7 (Sign at STA 16+503) was advanced through the existing pavement structure and penetrated asphaltic concrete approximately 255 mm thick.

4.2.2 Topsoil

A 50 mm thick layer of clayey topsoil was encountered at the ground surface in Borehole 8 (Sign at STA 17+503).

4.2.3 Fill

Granular fill was encountered underlying the asphalt pavement in Borehole 7 (sign at STA 16+503). The fill is comprised of interlayered moist, brown to grey sand, some silt, sand and gravel containing silty clay pockets, and silty sandy gravel, trace to some clay and trace organics. The surface of the fill was encountered at Elevation 95.2 m and the overall thickness of the deposit is 3.8 m.

A 50 mm thick layer of crushed rock (rock fragments) was encountered underlying the topsoil in Borehole 8 (Sign at STA 17+503).

The SPT “N”-values measured within the granular fill range from 5 blows to 39 blows per 0.3 m of penetration indicating a loose to dense compactness condition.

The result of a grain size distribution test completed on one sample of the silty sandy gravel layer of the fill is shown on Figure B1 in Appendix B. Atterberg limit testing was completed on one sample of the silty sandy gravel layer of the fill and the fines portion of the material was determined to be non-plastic. The water content measured on three samples of the fill deposit ranges from about 10 per cent to about 16 per cent.

4.2.4 Clayey Silt

A clayey silt deposit was encountered underlying the topsoil / rock fragments in Borehole 8 (Sign at STA 17+503). The cohesive stratum is comprised of moist, brown clayey silt, containing trace organics (rootlets) and trace gravel. The surface of the deposit was encountered at Elevation 94.7 m and the thickness of the deposit is 0.5 m.

The natural water content measured on one sample of the deposit is about 17 per cent.

4.2.5 Boulder

A 0.5 m thick boulder was encountered at a depth of 4 m below ground surface (Elevation 91.5 m) in Borehole 7 (Sign at STA 16+503), underlying the granular fill. A photograph of the recovered portions of the boulder is shown on Figure B2 in Appendix B.

4.2.6 Bedrock

Bedrock was encountered underlying the fill / boulder in Borehole 7 and underlying the clayey silt deposit in Borehole 8 at depths of 4.6 m and 0.6 m below ground surface (Elevations 90.9 m and 94.2 m), respectively. Bedrock was cored in Borehole 7 for a length of 3.4 m and in Borehole 8 for a length of 3.9 m. Based on review of the bedrock core samples, the bedrock is described as moderately to slightly weathered, fine grained, thinly to medium bedded, slightly porous, grey limestone. Photographs of the bedrock cores are shown on Figures B2 and B3 in Appendix B.

The total core and solid core recovery ranges from 52 per cent to 100 per cent, and from about 5 per cent to 100 per cent, respectively. The Rock Quality Designation (RQD) measured on the recovered bedrock core samples ranges between 60 per cent and 100 per cent, with a 0.1 m length of core with an RQD of about 17 per cent, generally indicating the rock is of fair to excellent quality in accordance with Table 3.10 of the *Canadian Foundation Engineering Manual* (CFEM), 2006[4].

Two unconfined compression (UC) tests were performed on select specimens of the bedrock core and measured Uniaxial Compressive Strengths of 76 MPa and 88 MPa, indicating that the rock is strong (50<R4<100 MPa), in accordance with Table 3.5 of the CFEM (2006)[4]. The Uniaxial Compressive Strength laboratory test results and photographs of the condition of the tested specimens are shown on Figures B4 and B5 in Appendix B.

4.2.7 Groundwater Conditions

The overburden samples obtained from the boreholes advanced during the investigation were generally moist. The open boreholes were observed to be dry prior to the start of bedrock coring operations.

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

4.2.8 Analytical Testing Results

As discussed in Section 3.0, a soil sample taken from Borehole 7 (for Sign at STA 16+503) and a rock sample from Borehole 8 (for Sign at STA 17+503) were submitted for analysis of parameters used to assess the potential corrosivity of the site soils and rock to steel and concrete. The detailed test results are presented in Appendix C and are summarized as follows:

Parameter	Borehole No. 7 (SA#3 - Soil) (STA 16+503)	Borehole No. 8 (0.61 m to 0.75 m – Rock) (STA 17+503)
pH	8.49	8.85
Resistivity (ohm-cm)	1150	3820
Conductivity (umho/cm)	868	0.262
Chlorides (ug/g)	382	52
Sulphate (ug/g)	196	39
Sulfide (%)	0.07	<0.05
Redox Potential (mV)	142	120

[4] Canadian Geotechnical Society. 2006. Canadian Foundation Engineering Manual, 4th Edition, BiTech Publications.

5.0 CLOSURE

The fieldwork was carried out by Lindsay Palmer and Katelyn Nero, under the direction of Ms. Sarah Poot, P.Eng.. This report was prepared by Ms. Katelyn Nero and reviewed by Ms. Sarah Poot, P.Eng., Senior Engineer and Associate of Golder. Ms. Lisa C. Coyne, P.Eng., a Designated MTO Contact and Principal of Golder, conducted a technical and quality control review of the report.

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PART B

FOUNDATION DESIGN REPORT
SIGN SUPPORT STRUCTURES
HIGHWAY 401/KINGSTON ROAD 38 INTERCHANGE IMPROVEMENTS, KINGSTON, ONTARIO
MTO GWP 4049-11-00
AGREEMENT NO. 9016-E-0007, ASSIGNMENT NO. 1

6.0 DISCUSSION AND RECOMMENDATIONS

This section of the report provides foundation design recommendations for two proposed sign support structures. The recommendations are based on interpretation of the factual data obtained from the borehole and drillholes advanced during the subsurface investigation at this site and from site observations. The interpretation and recommendations presented in this report are intended only to provide the designers with sufficient information to assess feasible foundation design alternatives and to design the proposed sign foundations. The Foundation Design Report's, discussion and recommendation are intended for the use of the Ministry of Transportation, Ontario (MTO) and shall not be used or relied upon for any other purpose or by any other parties, including the construction contractor or the design-build contractor. The contractor must make their own interpretation based on the factual data in Part A (Foundation Investigation) of the report. Where comments are made on construction, they are provided only in order to highlight those aspects which could affect the design of the project and for which special provisions may be required in the Contract Documents. Those requiring information on 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 the Ministry of Transportation, Ontario (MTO) to provide an assessment of foundation options, geotechnical parameters and recommendations on foundation aspects for the two proposed sign support structures located on Highway 401 Eastbound, STA 16+503 and STA 17+503 in the City of Kingston.

The recommendations provided in this report assume that the existing road embankment configuration will be maintained, as the details of the proposed construction staging for the eastbound lanes are unknown at this time.

6.1.1 Sign Support Structure at STA 16+503

It is understood that the proposed ground-mounted sign support structure at STA 16+503 will be designated either a "steel column sign support" or a "timber post sign support" as defined in MTO's *Sign Support Manual* (2015). Based on information provided by Stantec, the structural designer for this project, the sign supports will be designed as "Non-Breakaway Type".

According to the Sign Support Manual (2015), Section 5, steel column sign supports are categorized into five different Types of Supports (2-2, 2-3, 3-2, 3-3 and 3-4) based on the designed number of columns and crossarms. Each Type is dependent on the sign area, the eccentricity and the 10-year reference wind pressure at the proposed sign location. For timber post sign supports, in accordance with the Sign Support Manual (2015), Section 6, supports generally consist of two to four timber posts, designated under three different Types (Type II, Type III, or Type IV). Each Type of support is dependent on the sign area and post height. Golder understands that the Type of support for this sign has not yet been determined.

Due to access constraints and the presence of the embankment side slope south of the guiderail, Borehole 7 was advanced at about STA 16+503 on the eastbound lanes right paved shoulder. The subsurface conditions encountered at this location consist of 255 mm of asphalt at the surface (Elevation 95.5 m) underlain by a 3.7 m thick deposit of loose to dense granular fill to a depth of 4.0 m below ground surface. Boulders/blast rock underlies the granular fill from a depth of 4.0 m to a depth of 4.6 m below ground surface. Limestone bedrock underlies the boulder/blast rock at Elevation 90.9 m and is classified as strong and of fair to excellent quality.

6.1.2 Sign Support Structure at STA 17+503

It is understood that the proposed overhead sign support structure(s) at STA 17+503 will be designated a "single cantilever static sign support" as defined in MTO's Sign Support Manual (2015). According to the Sign Support Manual (2015), Section 3, single cantilever static sign supports typically follow one of four pre-determined designs (Classes 1 through 4). Each class has a specified set of member dimensions, which results in different load carrying capacities. The Class is dependent on the area of the sign, the eccentricity and the 50-year reference wind pressure at the proposed sign location. Golder understands that the Class for this sign has not yet been determined.

Borehole 8 was advanced at about STA 17+503 at approximately the proposed location of the sign. The subsurface conditions encountered at this location consist of topsoil at ground surface (Elevation 94.8 m) underlain by a thin layer of rock fragments, underlain by a 0.5 m thick deposit of clayey silt to a depth of 0.6 m below ground surface. Limestone bedrock underlies the stiff clayey silt deposit at Elevation 94.2 m and is classified as strong and of fair to excellent quality.

6.2 Sign Support Structure Foundations

6.2.1 Steel Column Sign Support

A steel column supported sign (Non-Breakaway Type) is one of two proposed options at STA 16+503 as discussed in Section 6.1.1. Steel-mounted non-breakaway sign supports are typically designed with the steel column extending below the ground surface within a 450 mm diameter concrete-filled augered hole (or socketted into bedrock, if applicable) in accordance with the requirements in MTO's Sign Support Manual (2015) (Section 5 and Drawing SS118-33). The footing depths identified in Figure 5.4.3 of the Manual range from 1.6 m to 2.8 m, depending on the sign area, the steel column size and the number of columns (two or three). Section 5.1.5 of the *Sign Support Manual* (2015) specifies that the minimum footing depths identified in Figure 5.4.3 are based on a passive earth pressure of 68 kPa at Serviceability Limit States (SLS), derived from Brom's equation for pole foundations in cohesive soils with a shear strength of 50 kPa. Further, the footing depth assumes that the lateral soil resistance is based on full depth, without reduction for the depth of frost penetration into the soil. If it is deemed that for a specific site the soil parameters for are less than specified, a site-specific footing design must be carried out.

The results of the investigation at Borehole 7 indicate that sufficient overburden thickness of suitable quality/condition is present at the proposed sign location. The granular fill has friction angles greater than the input parameters described above for lateral resistance.

Where required (for example, for larger than standard board sizes), a site-specific footing design can be carried out by the structural designer using the parameter values provided in Table 1 following the text of this report to calculate the unfactored passive lateral earth pressure P_p (kPa), distributed along the length of the caisson foundation.

6.2.2 Timber Post Sign Support

A timber post-supported sign (Non-Breakaway Type) is one of two proposed options at STA 16+503 as discussed in Section 6.1.1. Timber post sign supports are typically designed with a timber post extended below the ground surface within a 600 mm diameter augered hole in accordance with the requirements in MTO's Sign Support Manual (2015) (Section 6 and Drawing SS118-34). The augered hole is backfilled with compacted OPSS.PROV 1010 (Aggregates) Granular 'A' material and overfilled by approximately 75 mm in order to allow for settlement and to promote water runoff. Granular material around footings should be placed in accordance with OPSS.PROV 501 (Compacting).

The footing depths for a non-breakaway timber post sign support range from 1.2 m to 1.5 m, depending on the sign area and the number of columns (two or three). Section 6.1.5 of the *Sign Support Manual* (2015) specified that the minimum timber post depths identified are based on a passive earth pressure of 68 kPa at SLS. If it is deemed that for a specific site the soil parameters are less than specified, a site-specific footing deign must be carried out.

The results of the investigation at Borehole 7 indicate that sufficient overburden thickness of suitable quality/condition is present at the proposed sign location. The loose to dense granular fill has friction angles that result in a passive lateral resistance equal to or greater than the minimum required passive lateral resistance described above for a cohesive soil.

Due to the difference in ground surface elevation at the borehole location versus the sign support location, it is possible that bedrock may be encountered within the anticipated foundation depth. The contract should include a quantity allowance for coring the bedrock, as applicable, and appropriate construction procedures and equipment will be required to penetrate the bedrock.

Where required, a site-specific footing design can be carried out by the structural designer using the parameters provided in Table 1 following the text of this report to calculate the un-factored passive lateral earth pressure Pp (kPa), distributed along the length of the foundation.

6.2.3 Single Cantilever Static Sign Support

A single cantilever static sign is proposed at STA 17+503 as discussed in Section 6.1.2. Single cantilever static sign supports are typically designed with a standard single concrete caisson foundation in accordance with the requirements of the Sign Support Manual (2015) for ground mounted signs. Typically, the caisson design depth and diameter are determined based on the design Class selected. The standard caisson design assumes normal competent soil conditions of uniform composition, with minimum soil parameters as specified in the *Sign Support Manual* (2015). Based on the results of the foundation investigation, the subsurface conditions at the proposed STA 17+503 sign location encountered bedrock at shallow depth below ground surface (i.e. at a depth of about 0.6 m) which is within the typical caisson depth and therefore requires a site-specific design.

Based on the existing ground conditions, the foundation for the sign support can be designed as a single concrete caisson socketed into the rock depending on the specified depth, or alternatively, the sign can be supported on a spread footing, founded on the bedrock surface. A comparison between these two foundation options based on the advantages, disadvantages, relative costs and risks/consequences for each of the foundation options is presented in Table 2 following the text of this report. Recommendations for these foundation options are provided in Sections 6.2.3.1 and 6.2.3.2.

6.2.3.1 Caisson Foundation

As noted above, a caisson foundation for a overhead sign support at STA 17+503 should be designed in accordance with the standard caisson foundation design for cantilever static sign supports presented in the *Sign Support Manual* (2015), Section 3 and Drawing SS118-3. The standard design specifies a caisson diameter of 1.2 m to 1.35 m, and a length ranging from 5 m to 6.5 m depending on the sign design Class, below the frost depth, except where bedrock is encountered within this depth. For this sign/site, the frost depth is approximately 1.6 m as interpreted from Ontario Provincial Standards Drawing (OPSD) 3090.101 (Foundation, Frost Penetration Depths from Southern Ontario).

In accordance with Note 1 of the Notes to Design on Standard Drawing SS118-3 of the *Sign Support Manual* (2015), where bedrock is encountered at a depth (Y) less than the required depth (L_{req}) for design of a Class 1, Class 2, Class 3 or Class 4 caisson foundation, the required depth of the caisson foundation may be taken as follows:

$$L = Y + \frac{L_{req} - Y}{2}$$

Where:

L = Length of caisson below depth of neglected overburden (m) which included the frost depth and the depth of any poor soils

Y = Distance between depth of neglected overburden soils and bedrock (m)

Based on the above equation from the standards in the *Sign Support Manual*, the total length of the caisson as well as the length of caisson socketed into the limestone bedrock for Class 1, Class 2, Class 3 and Class 4 design for single cantilevered static sign support for Overhead Sign STA 17+503 are summarized below. It is noted that the depth/elevation of the bedrock surface may vary slightly at the sign location, as compared with the borehole location, as discussed above.

Design Class	Caisson length required below neglected overburden, L_{req} (m)	Depth of neglected overburden (m)	Depth to Bedrock (m)	Distance between depth of neglected overburden and depth to Bedrock, Y (m)	Caisson length below depth of neglected overburden, L (m)	Total Caisson Length (m)	Length of Caisson Socketted into Bedrock
Class 1 (1.2 m Dia.)	5.0	0.6	0.6	0	2.5	0.6+2.5 = 3.1	3.1 - 0.6 = 2.5
Class 2 (1.2 m Dia.)	5.0	0.6	0.6	0	2.5	0.6+2.5 = 3.1	3.1 – 0.6 = 2.5

Design Class	Caisson length required below neglected overburden, L_{req} (m)	Depth of neglected overburden (m)	Depth to Bedrock (m)	Distance between depth of neglected overburden and depth to Bedrock, Y (m)	Caisson length below depth of neglected overburden, L (m)	Total Caisson Length (m)	Length of Caisson Socketted into Bedrock
Class 3 (1.2 m Dia.)	6.0	0.6	0.6	0	3.0	0.6+3.0 = 3.6	3.6 – 0.6 = 3.0
Class 4 (1.35 m Dia.)	6.5	0.6	0.6	0	3.3	0.6+3.3 = 3.9	3.9 – 0.6 = 3.3

For a concrete caisson socketed into bedrock, the lateral resistance will be developed primarily from the fixity (in concrete) within the drilled socket. The factored passive lateral resistance may be taken as 25 MPa. Notwithstanding these values, a minimum socket length as noted above is recommended to satisfy MTO's *Sign Support Manual*.

The bedrock at the proposed overhead sign location is classified as strong and of fair to excellent quality; in addition, the limestone bedrock in this area is considered abrasive to construction equipment. As such, appropriate equipment and construction procedures (such as coring or churn drilling techniques) would be required to advance the sockets into the bedrock.

6.2.3.2 Spread Footing

As an alternative to a caisson socketed into bedrock, consideration could be given to using a spread footing founded on bedrock to support the sign at STA 17+503. The founding surface should consist of properly prepared bedrock, after all shattered, loose and fractured bedrock has been removed. The highest recommended founding elevation for the design of the footing is Elevation 94.2 m. In addition, if the bedrock surface is sloping, mass concrete and/or hoe ramming may be required to achieve a level footing subgrade design; if mass concrete is used to raise the subgrade to the footing elevation it should consist of the same type of concrete as that used for the footing. Given the potential of encountering an uneven and sloping bedrock surface, consideration could be given to including a Non-Standard Special Provisions (NSSP) for mass concrete and levelling of the bedrock surface in the contract documents; example NSSPs are provided in Appendix D.

Given that the sign is to be founded on limestone bedrock, or mass concrete over limestone bedrock, frost protection is not required.

Inspection and approval of the foundation area prior to spread footing construction should be carried out by a Foundation Engineer in accordance with OPSS 902 (Excavating and Backfilling), as amended by Special Provision (SP) 109S12, to ensure that all fractured rock has been removed from the foundation area and that the foundation base has been properly prepared for the placement of concrete.

Construction of the footing foundation for the sign support structures should be in accordance with OPSS.PROV 915 (Sign Support Structures).

6.2.3.2.1 Geotechnical Axial Resistance

For the spread footing bearing directly on the limestone bedrock surface, or on mass concrete over bedrock, a factored ultimate geotechnical axial resistance of 25 MPa may be used for design. Serviceability Limit States (SLS) conditions do not apply for footing founded on the limestone bedrock or on mass concrete at this site.

The factored geotechnical resistance provided above is given for loads that will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the footing, inclination of the load should be taken into account in accordance with Section 6.10.4 of the *Canadian Highway Bridge Design Code* (CHBDC, 2014).

6.2.3.2.2 Resistance to Lateral Loads

Resistance to lateral forces/sliding resistance between the cast-in-place concrete footing and the bedrock should be calculated in accordance with Section 6.10.5 of the CHBDC (2014). The following presents the coefficient of friction, tan δ, for the interface between the concrete footing on bedrock as interpreted from NAVFAC (1982):

Founding Material	Coefficient of Friction (tan δ)
Mass Concrete and / or Bedrock	0.70

For a footing on bedrock, the sliding/lateral resistance between the concrete footing/mass concrete and the bedrock, and the passive earth pressure, may be supplemented by dowelling into the bedrock if necessary. The horizontal resistance of the dowels is dependent on the strength of the bedrock, grout and steel. The measured Unconfined Compression (UC) tests indicate that the bedrock is strong (50<R4<100 MPa). The design of the dowels into the bedrock may be handled in the same way as the dowel embedded into the concrete. This assumes that the uniaxial compressive strength (UCS) of the grout will be similar to that of concrete. The actual bond stress along the rock-grout interface may vary from the design value and should therefore be verified in the field as noted below. The dowels should have a 1 m minimum embedded length within the competent bedrock, and the structural strength of the dowel and compressive strength of the grout should not be exceeded. If dowelling is required for structural considerations a Non-Standard Special Provision (NSSP) should be included in the Contract Documents to specify the installation, materials and testing of the dowels; and the current NSSP is provided in Appendix D.

6.3 Frost Protection

For footings founded directly on bedrock, frost protection is not required.

6.4 Construction Considerations

The following sections identify potential construction considerations that may impact the detail design and construction of the proposed sign support foundations.

6.4.1 Temporary Excavations and Temporary Protection Systems

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

Open-cut excavations of short duration through the granular fill, clayey silt deposit and shattered rock layer should be carried out in accordance with the latest Occupational Health and Safety Act for Construction Projects (OHSA). When referencing OHSA, the granular fill, clayey silt deposit and shattered rock should be considered as a “Type-3 Soil”. As such, excavations should be sloped at an inclination of 1 Horizontal to 1 Vertical (1H:1V) or flatter. For excavations into good quality bedrock, if necessary, the overall slope to the cut face may be formed vertically, or near vertically (i.e. about 0.5H:1V).

Temporary protection systems are not expected to be required for the construction of sign support foundations where augered holes are adopted. However, for general guidance should such systems be required to accommodate construction staging and selected foundation types and construction methods, it is anticipated that driven sheetpiles will not be feasible at the sign support sites due to the presence of rock fill and/or shallow bedrock. Soldier pile and lagging would likely need to be employed with the soldier piles pre-drilled into bedrock. Given these constraints and requirements, if deep excavations are required, a caisson option noted above would eliminate the need for an excavation and protection systems.

Where required, temporary protection systems should be designed and constructed in accordance with OPSS.PROV 539 (Temporary Protection System) as amended by SP105S09, and the lateral movement should meet Performance Level 2 provided that any existing adjacent utilities can tolerate this magnitude of deformation. The temporary protection system should be removed after completion of construction or cut-off at a depth of at least 1.6 m below ground surface. The following design parameters may be used for the design of the temporary protection systems at the proposed overhead sign locations.

Soil Deposit	Unit Weight	Friction Angle	Coefficient of Earth Pressure		
			Active, K_a	At Rest, K_o	Passive, K_p
Granular Fill	21 kN/m^3	28°	0.31	0.53	2.8

The selection and design of the protection system will be the responsibility of the contractor.

During construction, stockpiles should be placed well away from the edge of the excavation, and their height should be controlled so they do not surcharge the sides of the excavation and/or overall existing highway embankment slopes. Generally, the distance between the crest of the excavation and the toe of the stockpile should be greater than 1.5 times the depth of the excavation.

6.4.2 Embankment Fill Re-Placement

The excavation around and above the spread footing foundation and the cover over the caisson foundation may be backfilled using granular material such as OPSS.PROV 1010 (Aggregates) Granular 'A' or 'B' (Type II) placed in 300 mm thick loose lifts and uniformly compacted not less than 100 per cent of the Standard Proctor maximum dry density of the material and as outlined in OPSS.PROV 501 (Compacting).

The final grade surrounding the sign foundations should be sloped to promote surface water drainage away from the pavement and sign supports, to the adjacent ditch, and, if located on the vegetated embankment side slope, surfaced with top soil and seed, in accordance with OPSS 804 (Seed and Cover), or granular sheeting, in accordance with OPSS.PROV 1004 (Aggregates - Miscellaneous). If the resulting side slopes exceed 2 Horizontal to 1 Vertical, R-10 Rip-Rap, in accordance with OPSS.PROV 1004 (Aggregates - Miscellaneous), should be used to reduce the potential for erosion of the slope locally.

6.4.3 Concrete Caissons

It is recommended that temporary liners be employed if and where caissons are advanced through overburden soils, to minimize disturbance to the surrounding ground and minimize the potential for materials to enter the bedrock socket and impact the foundation performance. MTO's Standard Special Provision for caisson foundations for sign supports and HML poles should be included in the Contract Documents.

6.5 Corrosion Assessment and Protection

Soil corrosivity may affect the concrete and reinforced steel foundations of the sign buried in the soil. The long-term performance and durability of the foundations are directly related to their respective corrosion resistance. Generally, the corrosivity potential to a structure depends on the soil resistivity / electrical conductivity, hydrogen ion concentration, and salts (chloride and sulphate) concentrations. The analytical results for the sample from Borehole 7 (STA 16+503) submitted for testing are presented in Section 4.2.8 and included in Appendix C.

6.5.1 Potential for Sulphate Attack

The analytical test results were compared to CSA Standard, Can/CSA-A23.1-14 Table 3 ("*Additional requirements for concrete subjected to sulphate attack*") for potential sulphate attack on concrete. The sulphate concentration measured in the sample is less than 0.1 per cent, which is below the exposure class of Moderate. Therefore, based on the test result on the soil sample from the borehole, the effects of sulphates from within the existing fill around the foundation may not need to be considered (i.e. at both Overhead Signs STA 16+503 and STA 17+503).

6.5.2 Potential for Corrosion

The test results of the soil and rock indicate a pH of about 8.5 to 8.85 and a resistivity of about 1,150 to 3820 ohm-cm. According to the *Gravity Pipe Design Guidelines* (MTO, 2014), the pH is not considered detrimental to concrete durability. However, the resistivity of 1,150 ohm-cm indicates that the soil corrosiveness is “Severe” (R<2,000 ohm-cm), as per Table 3.2 of the Gravity Pipe Design Guidelines (MTO, 2014), and some level of corrosion protection should be applied to the foundation element / materials. Further, given that the sign support foundation at each site (Sta 16+503 and 17+503) is located adjacent to the roadway shoulder and will be exposed to de-icing salt, consideration should be given to the selection of a “C” type exposure class as defined by CSA A23.1 Table 1.

It is ultimately up to the structural designer to determine the appropriate exposure class and to ensure that all aspects of CSA A23.1 Section 4.1.1 “Durability Requirements” are followed.

7.0 CLOSURE

This report was prepared by Ms. Katelyn Nero, and reviewed by Ms. Sarah Poot, P.Eng., a senior geotechnical engineer and Associate of Golder. Ms. Lisa C. Coyne, P.Eng., a MTO Foundations Designated Contact and Principal of Golder, conducted a technical and quality control review of the report.

Golder Associates Ltd.

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KN/AC/SEMP/JMAC/ljv/cr

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Ontario Provisional Standard Drawing:

OPSD 3090.101Foundation Frost Penetration Depths for Southern Ontario

Ontario Provincial Standard Specification and Special Provisions:

OPSS.PROV 501 Construction Specifications for Compacting

OPSS.PROV 539 Construction Specification for Temporary Protection Systems

OPSS 902 Construction Specification for Excavation and Backfill for Structures

OPSS.PROV 903Construction Specification for Deep Foundations

OPSS.PROV 915 Construction Specification for Sign Support Structures

OPSS.PROV 804 Construction Specification for Seed and Cover

OPSS.PROV 1004 Material Specification for Aggregates – Miscellaneous

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

SP105S09 Amendment to OPSS 539, November 2014

SP109S12 Amendment to OPSS 902, November 2010

SP109F57Amendment to OPSS 903, April 2016

ASTM International:

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

Ontario Water Resources Act:

Ontario Regulation 903 Wells (as amended)

Ontario Occupational Health and Safety Act:

Ontario Regulation 213/91 Construction Projects (as amended)

Table 1 - Geotechnical Design Parameters for Steel Column and Timber Sign Support Foundations for OHS STA 16+503

Foundation Element / Sign Station	Borehole No.	Stratum	Depth[1] (m)	Elevation (m)	Design Parameters					
					S_u (kPa)	Φ' (°)	ϒ (kN/m³)	ϒ' (kN/m³)	K_p	f_c (MPa)
OH Sign STA 16+503	7	Compact Sand Fill	0.3 – 1.5	95.2 – 94.0	-	29	20	-	2.9	-
		Dense Sand and Gravel Fill	1.5 – 2.2	94.0 – 93.3	-	29	20	-	2.9	-
		Loose to Compact Silty Sandy Gravel Fill	2.2 – 4.0	93.3 – 91.5	-	28	20	-	2.8	-
		Strong, Fair to Excellent Quality Limestone Bedrock	4.0	91.5	-	45	24	-	5.9	75

NOTES:

1. Depths are given relative to the existing ground surface elevation at the proposed sign location. The existing ground surface elevation should be compared to the proposed design ground surface elevation at the actual sign support foundation location, and the depths to the various soil strata adjusted accordingly.

2. Design Parameters: S_u= undrained shear strength (kPa);
   Φ'= effective friction angle (degrees);
   ϒ = bulk unit weight (kN/m³);
   ϒ' = effective unit weight below the groundwater level (kN/m³);
   K_p = passive earth pressure coefficient;
   f_c= compressive strength (MPa)

Table 2 - Evaluation of Foundation Alternatives for Cantilever Sign Support STA 17+503

Options	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Caisson Socketed into Bedrock	2	▪ No post-construction settlement. ▪ Soil cover for frost protection is not required.	▪ Coring or churn drilling into the bedrock will be required to advance sockets. ▪ Temporary liner required for soil support during installation to prevent sloughing and caving of cohesionless soil above the bedrock. ▪ Large diameter caisson advanced into the very strong bedrock is costly.	▪ Much higher cost of installation compared to spread footings. ▪ Additional cost associated with specialized drilling equipment to advance the caisson holes into the bedrock.	▪ Specialized drilling equipment will be required to socket caissons into bedrock. ▪ May be considered cost prohibitive.
Spread Footings Founded on and Doweled into Bedrock	1	▪ Relative ease of construction. ▪ No bedrock coring and/or churn drilling required. ▪ No post-construction settlement. ▪ Soil cover for frost protection is not required. ▪ Very high geotechnical axial resistance available. ▪ Bedrock near ground surface.	▪ Larger excavation of overburden is required producing a larger volume of excavation spoils. ▪ Larger volume of concrete may be required to achieve level footing. ▪ Dowels may be required to anchor spread footings (due to structural considerations) ▪ Temporary protection system may be required to maintain the existing lane of traffic. ▪ Removal of fractured bedrock will be required for construction of the footings.	▪ Relatively lower cost in comparison to caissons socketed into bedrock. ▪ Additional cost required for the disposal of larger volumes of excavation soils. ▪ Additional costs required for installation of dowels into the bedrock. ▪ Additional costs required for temporary protection systems, if required. ▪ Additional costs required for pull out test on dowels.	▪ Risk that additional excavation and greater volume of concrete may be required if bedrock is sloping below the design founding elevation. ▪ Must ensure foundation base is properly prepared. ▪ Must ensure temporary protection systems are in place to prevent damage to the existing infrastructure.

Eastbound Highway 401 at STA 16+503 (City of Kingston), Looking East. January 2018.

Eastbound Highway 401 at STA 17+503 (City of Kingston), Looking West. January 2018.

PROJECT

Overhead Sign Support Structure
Hwy 401 EBL, City of Kingston
Ministry of Transportation, Ontario, G.W.P. 4049-11-00

TITLE

Site Photographs

GOLDER

PROJECT No.	1664176(0001)		FILE No.	----	
DESIGN	KN	27/04/2018	SCALE	NTS	REV.
CADD	-- --				
CHECK			**FIGURE 1**		
REVIEW	JMAC	16/05/2018			

MINISTRY OF TRANSPORTATION, ONTARIO

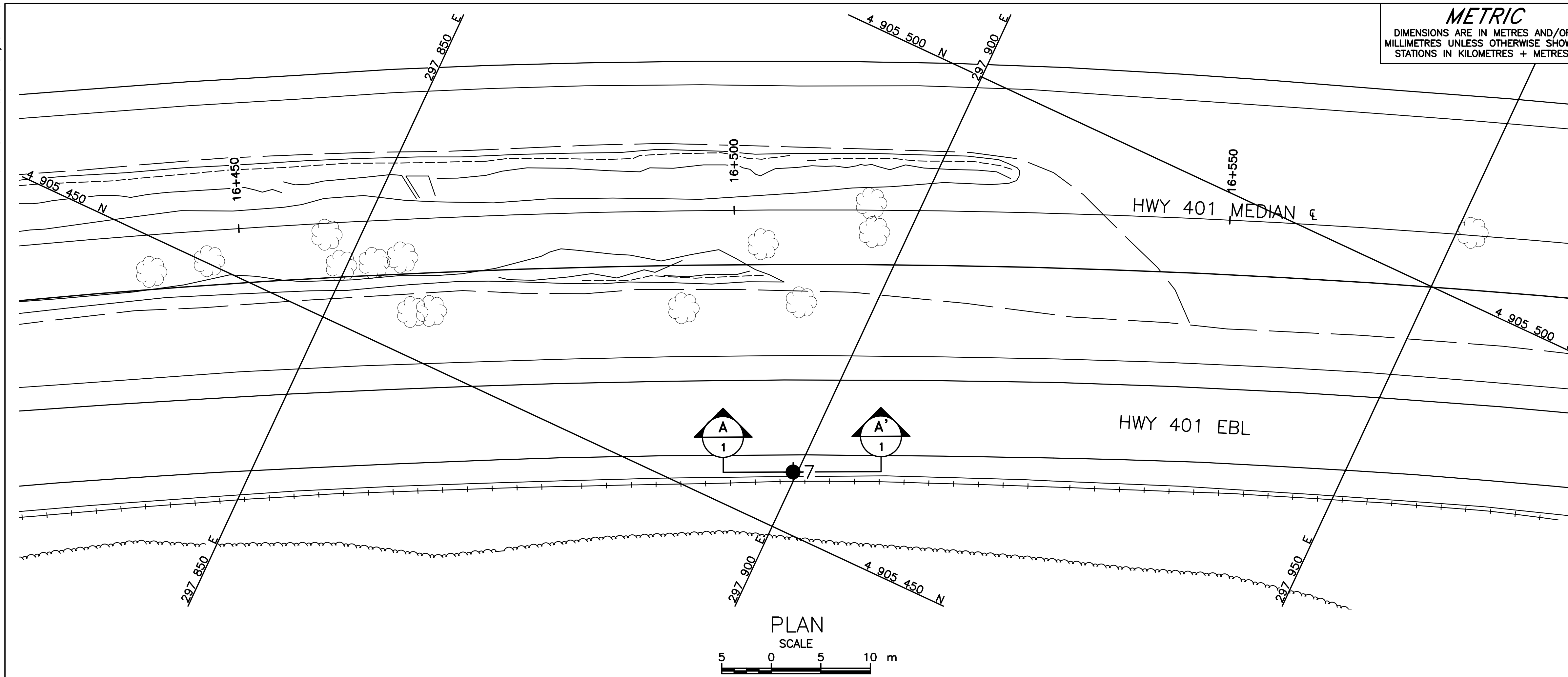
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

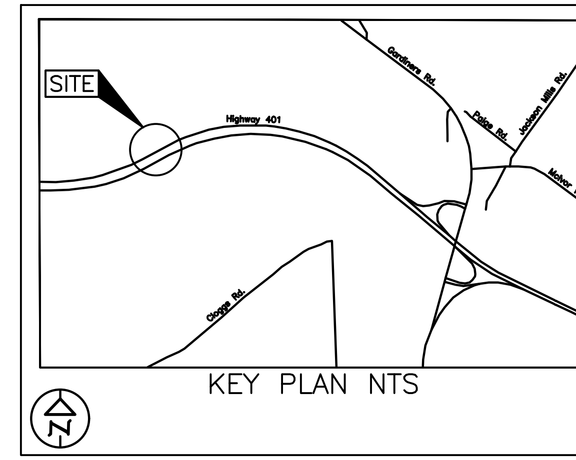
CONT No.
GWP No. 4040-11-00

HIGHWAY 401 EBL
OVERHEAD SIGN AT STA 16+503
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

GOLDER





LEGEND

- Borehole – Current Investigation
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- WL upon completion of drilling – Dry on May 1, 2018

BOREHOLE CO-ORDINATES (NAD83 MTM ZONE 9)

No.	ELEVATION	NORTHING	EASTING
7	95.5	4905455.9	297899.6

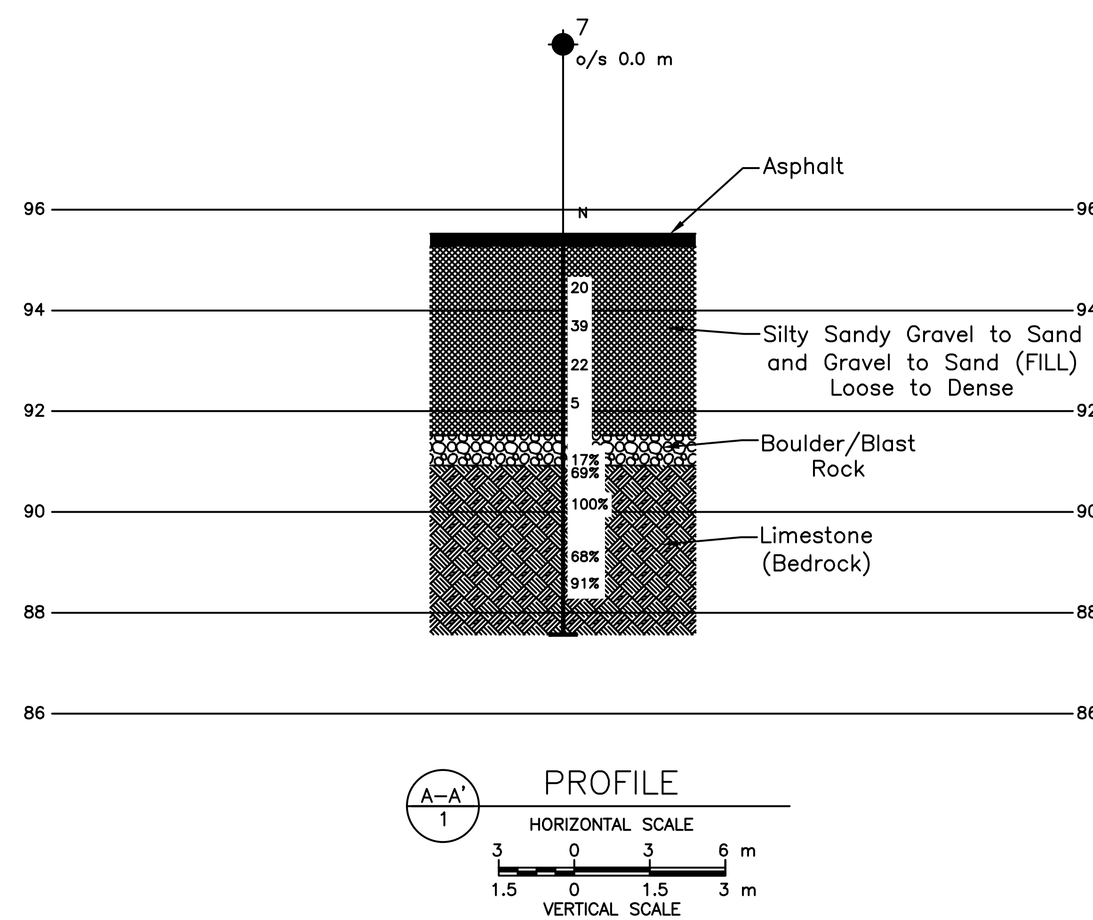
NOTES

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

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

REFERENCE

Base plans provided in digital format by Stantec, drawing file no. x-401-38lc.dgw, received DEC 07, 2017.



LICENSED PROFESSIONAL ENGINEER L.C. COYNE PROVINCE OF ONTARIO June 29, 2018

LICENSED PROFESSIONAL ENGINEER S.E.M. POOT 90471921 PROVINCE OF ONTARIO June 29, 2018

NO.	DATE	BY	REVISION

Geocres No. 31C-273

HWY. 401	PROJECT NO. 1664176		DIST.
SUBM'D. KN	CHKD. KN	DATE: 6/29/2018	SITE:
DRAWN: TR	CHKD. SEMP	APPD. JMAC/LCC	DWG. 1

MINISTRY OF TRANSPORTATION, ONTARIO

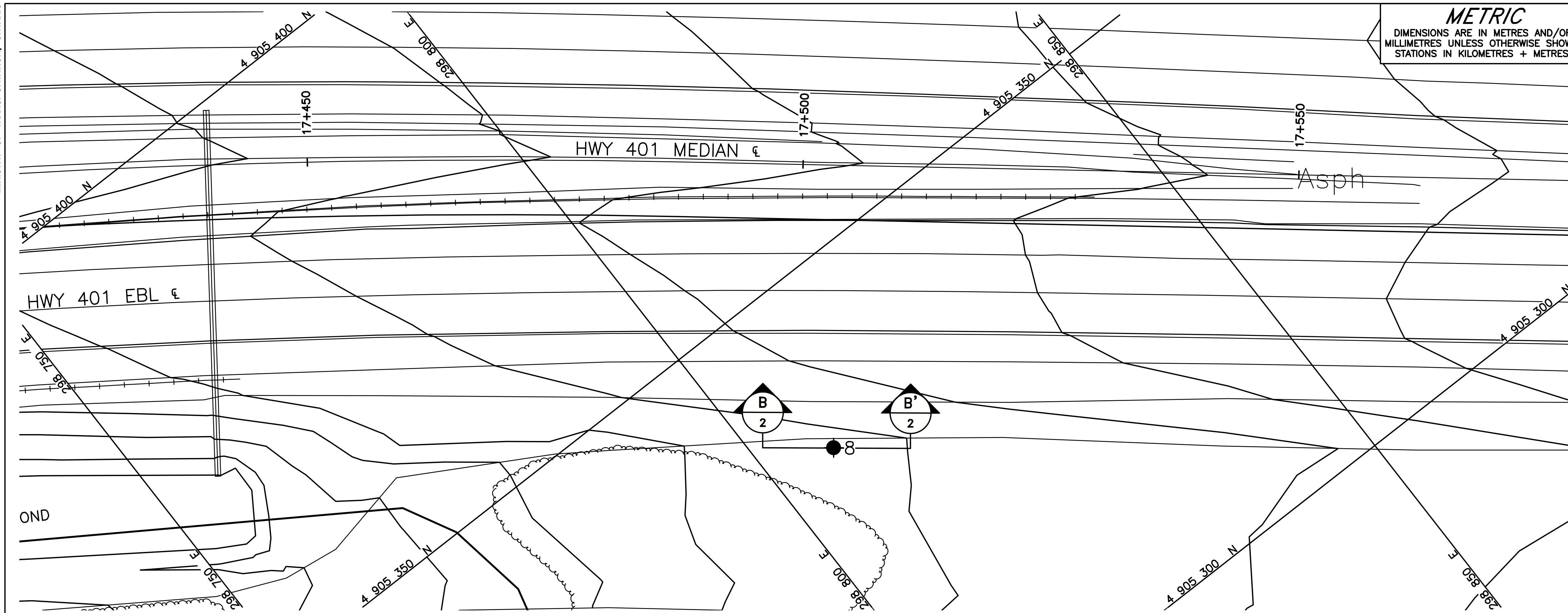
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 4049–11–00

HIGHWAY 401 EBL
OVERHEAD SIGN STA 17+503
BOREHOLE LOCATIONS AND SOIL STRATA

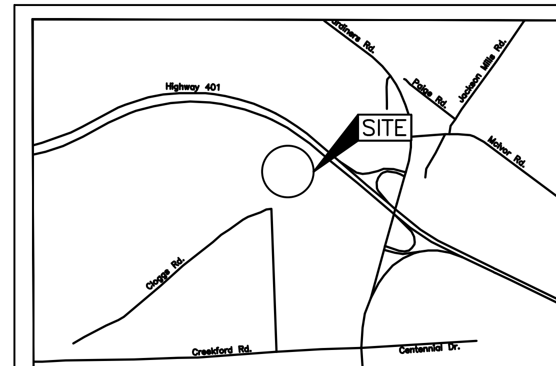
SHEET

GOLDER

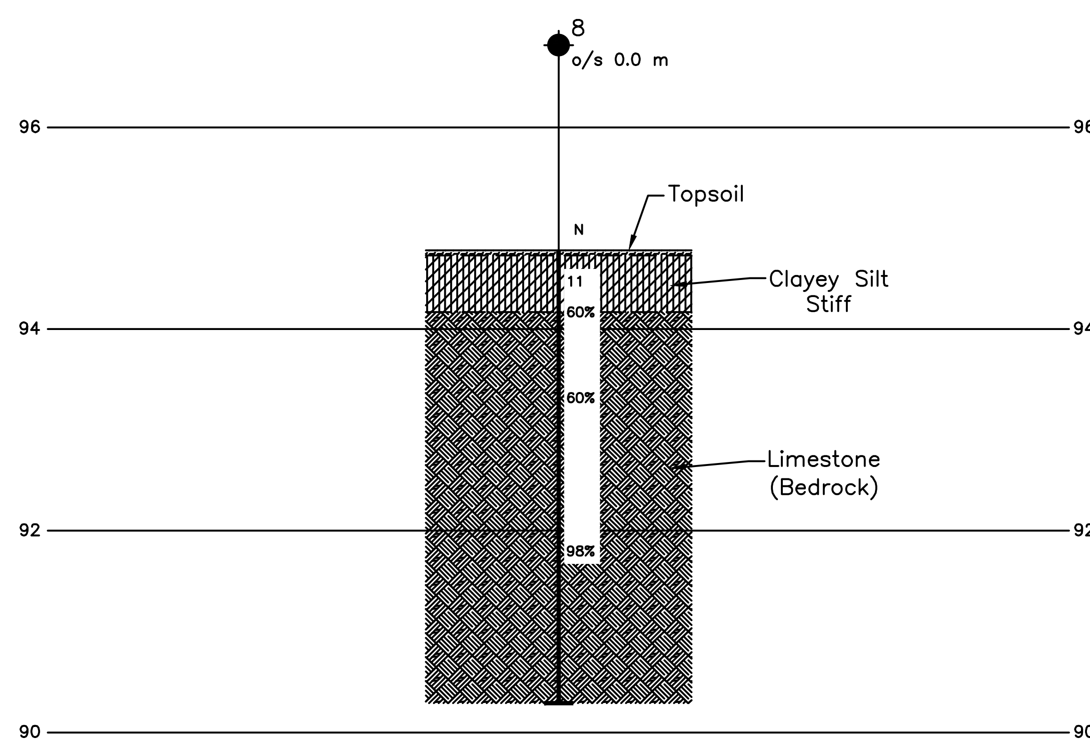


PLAN

SCALE 5 0 5 10 m



KEY PLAN NTS



B–B' 2 PROFILE

HORIZONTAL SCALE 3 0 3 6 m

0.75 0 0.75 1.5 m
VERTICAL SCALE

LEGEND

●	Borehole – Current Investigation
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
100%	Rock Quality Designation (RQD)
▽	WL upon completion of drilling – Dry Jan 17, 2018

BOREHOLE CO–ORDINATES (NAD83 MTM ZONE 9)

No.	ELEVATION	NORTHING	EASTING
8	94.8	4905333.4	298806.8

NOTES

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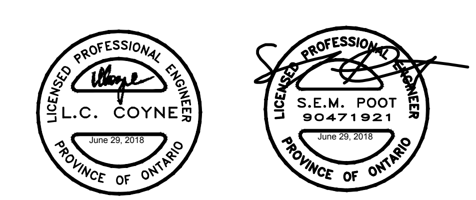


NO.	DATE	BY	REVISION
.	.	.	.

Geocres No. 31C–273

HWY. 401		PROJECT NO. 1664176	DIST.
SUBM'D. KN	CHKD. KN	DATE: 6/29/2018	SITE:
DRAWN: TR	CHKD. SEMP	APPD. JMAC/LCC	DWG. 2

PLOT DATE: June 28, 2018

APPENDIX A

Record of Boreholes and Drillholes

LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

Notes:
1. $\tau = c' + \sigma' \tan\phi'$
2. shear strength = (compressive strength)/2

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)

Dynamic Cone Penetration Resistance; N_d:

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60º cone attached to “A” size drill rods for a distance of 300 mm (12 in.).

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

Piezo-Cone Penetration Test (CPT)

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

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

Compactness Condition	**N** **Blows/300 mm or Blows/ft**
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils

Consistency

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

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

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

V. MINOR SOIL CONSTITUENTS

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

WEATHERINGS 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

Description	Bedding Plane Spacing
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

Description	Spacing
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

Term	Size*
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 90º 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		

GOLDER

Foundation Design

RECORD OF BOREHOLE No 7

SHEET 1 OF 1 **METRIC**

PROJECT 1664176 / 0001
G.W.P. 4049-11-00
LOCATION N 4905455.9; E 297899.6 MTM NAD 83 ZONE 9 (LAT. 44.289074; LONG. -76.586463)
ORIGINATED BY LP
DIST Eastern HWY 401
BOREHOLE TYPE 210 mm O.D. Hollow Stem Augers and NQ Casing / Coring
COMPILED BY SEMP
DATUM GEODETIC
DATE March 1, 2018
CHECKED BY SEMP

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
95.5	GROUND SURFACE						
0.0 / 95.2	ASPHALT (255 mm)						
0.3	Sand, some silt, some gravel (FILL) Compact Brown Moist				95		
		1	SS	20		○	
94.0 / 1.5	Sand and gravel, silty clay pockets (FILL) Dense Brown to grey Moist				94		
		2	SS	39			
93.3 / 2.2	Silty sandy gravel, trace to some clay, trace organics (FILL) Loose to compact Brown Moist						
		3	SS	22	93	○	43 20 26 11 Non-Plastic
		4	SS	5		○	
	Auger refusal at 4.0 m depth				92		
91.5 / 4.0	Boulder / Blast rock cored from a depth of 4.0 m to 4.6 m	-	RC	REC 42%			
90.9 / 4.6	LIMESTONE (BEDROCK)	1	RC	REC 52%	91		RQD = 17%
	Bedrock cored from a depth of 4.6 m to 8.0 m For bedrock coring details, refer to Record of Drillhole No. 7	2	RC	REC 80%			RQD = 69%
		3	RC	REC 100%	90		RQD = 100%
		4	RC	REC 89%	89		RQD = 68%
		5	RC	REC 100%	88		RQD = 91%
87.5 / 8.0	END OF BOREHOLE NOTE: 1. Borehole dry prior to rock coring.						

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

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

GTA-MTO 001 \\GOLDER.GDS\GAL\MISSISSAUGA\SM\CLIENTS\MTO\HWY_401_COLLINS_CREEK_BRIDGE\02_DATA\GINT\1664176.GPJ GAL-GTA.GDT 5-18-18 TB

RECORD OF DRILLHOLE: 7

PROJECT: 1664176 / 0001

SHEET 1 OF 1

LOCATION: N 4905455.9 ;E 297899.6

DATUM: GEODETIC

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: March 1, 2018

DRILL RIG: CME 75 Truck

DRILLING CONTRACTOR: Marathon Drilling Co. Ltd.

JN - Joint, FLT - Fault, SHR- Shear, VN - Vein, CJ - Conjugate; BD- Bedding, FO- Foliation, CO- Contact, OR- Orthogonal, CL - Cleavage; PL - Planar, CU- Curved, UN- Undulating, ST - Stepped, IR - Irregular; PO- Polished, K - Slickensided, SM- Smooth, Ro - Rough, MB- Mechanical Break; BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	FEATURES
		Continued From Record of Borehole No. 7	91.0			
	NQ Coring	LIMESTONE (Gull River Formation) moderately weathered, medium bedded, non-porous, medium strong, grey	4.6	1		
5				2	BD,IR,Ro BD,IR,Ro	
		LIMESTONE (GULL RIVER FORMATION) Slightly weathered to fresh, Medium bedded, Non-porous, Strong, Grey	90.2 / 5.4	3	BD,IR,Ro BD,IR,Ro BD,ST,Ro	
6				4	BD,IR,Ro BD,IR,Ro BD,IR,Ro	UCS = 88.2 MPa
7				5	BD,ST,Ro BD,IR,Ro BD,IR,Ro	
8		END OF DRILLHOLE	87.5 / 8.0			

Depth scale: 9, 10, 11, 12, 13, 14, 15, 16

DEPTH SCALE 1 : 60

Golder Associates

LOGGED: LP

CHECKED: SEMP

GOLDER

RECORD OF BOREHOLE No 8

SHEET 1 OF 1

METRIC

PROJECT 1664176 / 0001

G.W.P. 4049-11-00

LOCATION N 4905333.4; E 298806.8 MTM NAD 83 ZONE 9 (LAT. 44.287979; LONG. -76.575093)

ORIGINATED BY KN

DIST Eastern HWY 401

BOREHOLE TYPE 210 mm O.D. Hollow Stem Augers and NQ Coring

COMPILED BY SEMP

DATUM GEODETIC

DATE January 17 and 18, 2018

CHECKED BY SEMP

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
94.8	GROUND SURFACE								
0.0	Clayey silt TOPSOIL (FILL) (50 mm)	1	SS	11			~17 (natural moisture)		
0.1	Rock Fragments (FILL) (50 mm)								
94.2 / 0.6	CLAYEY SILT, trace rootlets, trace gravel Stiff Brown Moist Auger grinding at 0.6 m depth								
	LIMESTONE (BEDROCK) Bedrock cored from a depth of 0.6 m to 4.5 m For bedrock coring details, refer to Record of Drillhole No. 8	1	RC	REC 100%		94			RQD = 60%
		2	RC	REC 100%		93 / 92			RQD = 60%
		3	RC	REC 100%		91			RQD = 98%
90.3 / 4.5	END OF BOREHOLE NOTES: 1. Borehole dry prior to rock coring. 2. Moved 1.5 m east and advanced another borehole to refusal at 0.6 m depth.								

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED: 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%): 20 40 60

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

RECORD OF DRILLHOLE: 8

PROJECT: 1664176 / 0001
LOCATION: N 4905333.4 ;E 298806.8
INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: January 17 and 18, 2018
DRILL RIG: CME 55
DRILLING CONTRACTOR: Marathon Drilling Co. Ltd.

SHEET 1 OF 1
DATUM: GEODETIC

JN - Joint, FLT - Fault, SHR- Shear, VN - Vein, CJ - Conjugate
BD- Bedding, FO- Foliation, CO- Contact, OR- Orthogonal, CL - Cleavage
PL - Planar, CU- Curved, UN- Undulating, ST - Stepped, IR - Irregular
PO- Polished, K - Slickensided, SM- Smooth, Ro - Rough, MB- Mechanical Break
BR - Broken Rock
NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	Jr	Ja	FEATURES / NOTES
		Continued From Record of Borehole No. 8	94.2					
	NQ Coring	LIMESTONE (GULL RIVER FORMATION) Moderately weathered Thinly to medium bedded Slightly porous Medium strong Grey	0.6					
1				1	BD,UN,Ro	3	4	
		LIMESTONE (GULL RIVER FORMATION) Slightly weathered Thinly to medium bedded Slightly porous Strong Grey	93.3 1.5					
					BD,UN,Ro	3	1	
2					BD,UN,Ro	3	1	
				2	BD,UN,Ro	3	1	
					BD,UN,Ro	3	1	
					BD,UN,Ro	3	1	
3					BD,PL,Ro	1.5	4	UCS = 76 MPa
				3				
4			90.3 4.5					
		END OF DRILLHOLE						

Depth scale: 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12

DEPTH SCALE
1 : 60

Golder Associates

LOGGED: KN
CHECKED: SEMP

GTA-RCK 054 \\GOLDER.GDS\GAL\MISSISSAUGA\SIM\CLIENTS\MTO\HWY_401_COLLINS_CREEK_BRIDGE\02_DATA\GINT\1664176.GPJ GAL-MISS.GDT 5-23-18 TB

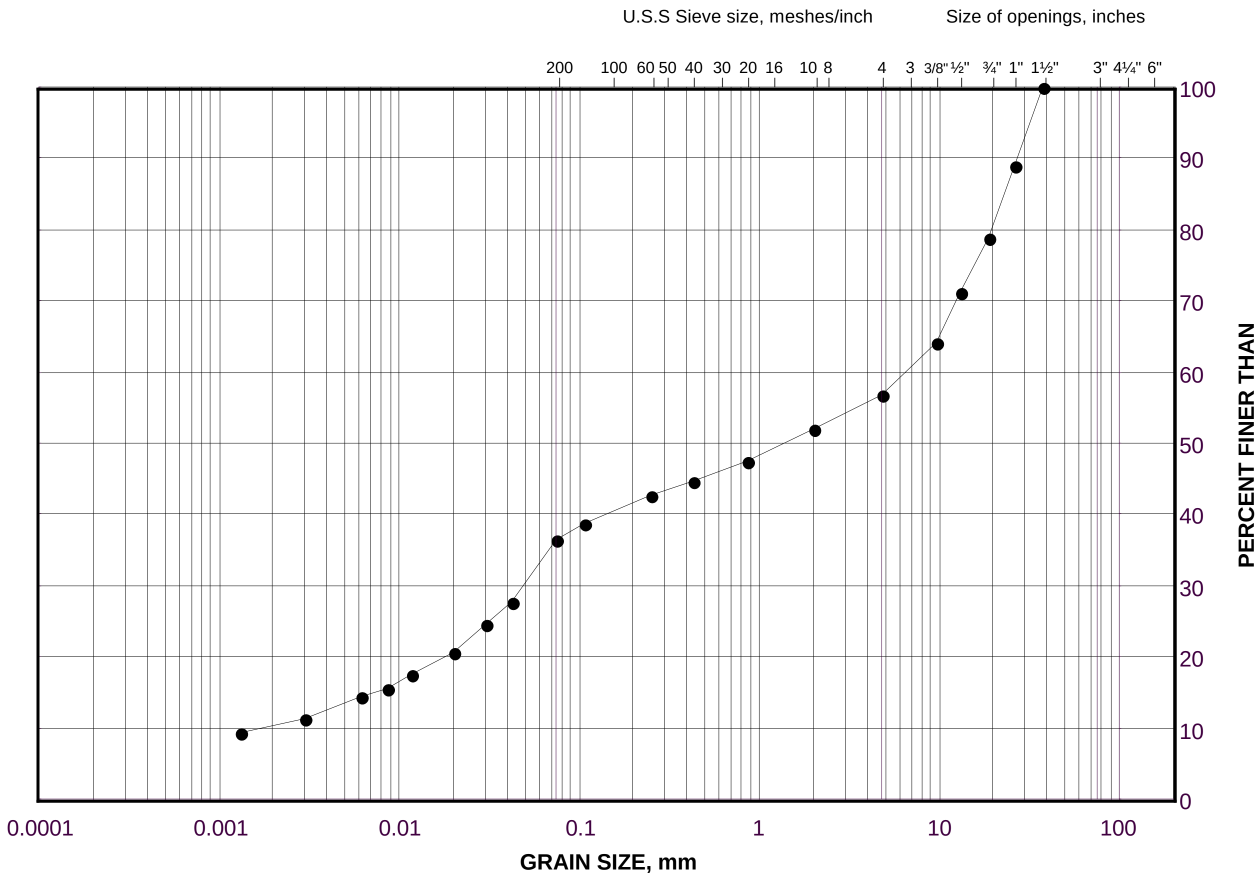
APPENDIX B

Laboratory Test Results

GRAIN SIZE DISTRIBUTION

Silty Sand Gravel (FILL)

FIGURE B1



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	7	3	92.9

Project Number: 1664176 (0001)(02)

Checked By: SEMP

Golder Associates

Date: 16-May-18

Scale

0 m — 0.40 m

0 ft — 1.0 ft

PROJECT	**Agreement No.: 9016-E-0007** **Overhead Signs** **Kingston, Ontario**				
TITLE	**BEDROCK CORE PHOTOGRAPHS** **BOREHOLE 7 (4.03 m to 7.95 m)**				
GOLDER	PROJECT No. 1664176/0001			FILE No. ----	
	DESIGN	KN	20180326	SCALE NTS	VER. 1.
	CADD	--		**FIGURE B2**	
	CHECK	SEMP	20180522		
	REVIEW	JMAC	20180516		

Start of Run No. 1 (0.61 m)

Start of Run No. 2 (1.46 m)

Start of Run No. 3 (2.98 m)

End of Borehole (4.49 m)

Scale

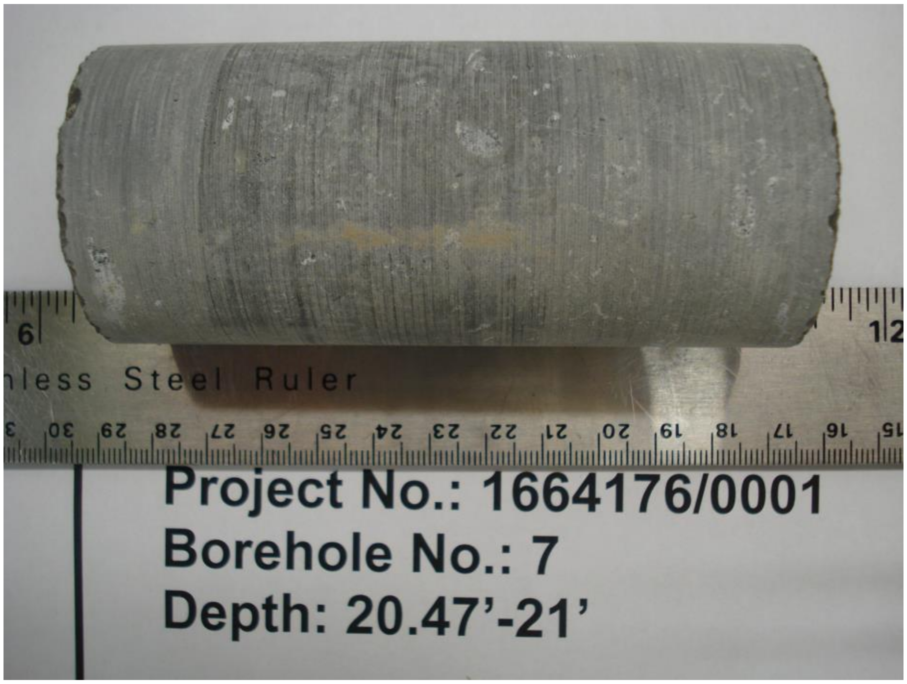
0 m — 0.40 m

0 ft — 1.0 ft — 2.0 ft

PROJECT	Agreement No.: 9016-E-0007 Overhead Signs Kingston, Ontario				
TITLE	BEDROCK CORE PHOTOGRAPHS BOREHOLE 8 (0.61 m to 4.49 m)				
GOLDER	PROJECT No. 1664176/0001			FILE No. ----	
	DESIGN	KN	20180326	SCALE NTS	VER. 1.
	CADD	--		FIGURE B3	
	CHECK	SEMP	20180522		
	REVIEW	JMAC	20180516		

UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS
ASTM D7012

FIGURE B4A



BEFORE COMPRESSION
CORE RUN # 3

AFTER COMPRESSION
CORE RUN # 3

Date Mar. 29, 2018
Project 1664176/0001

Golder Associates

Drawn Frank
Chkd. SEMP

Figure B4B

UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS
ASTM D7012

SAMPLE IDENTIFICATION

PROJECT NUMBER	1664176 (0001)	SAMPLE NUMBER	-
PROJECT NAME	MTO/MERO East Fnd Ret/Ontario	SAMPLE DEPTH, m	6.24-6.40
BOREHOLE NUMBER	7	DATE:	March 28,2018

TEST CONDITIONS

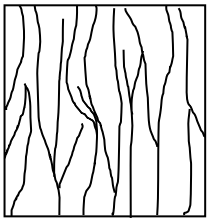
MACHINE SPEED, mm/min	N/A	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.28

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.79	WATER CONTENT, (specimen) %	0.10
SAMPLE DIAMETER, cm	4.74	UNIT WEIGHT, kN/m^3	26.47
SAMPLE AREA, cm^2	17.66	DRY UNIT WT., kN/m^3	26.45
SAMPLE VOLUME, cm^3	190.61	SPECIFIC GRAVITY	-
WET WEIGHT, g	514.73	VOID RATIO	-
DRY WEIGHT, g	514.22		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	N/A	COMPRESSIVE STRENGTH, MPa	88.2

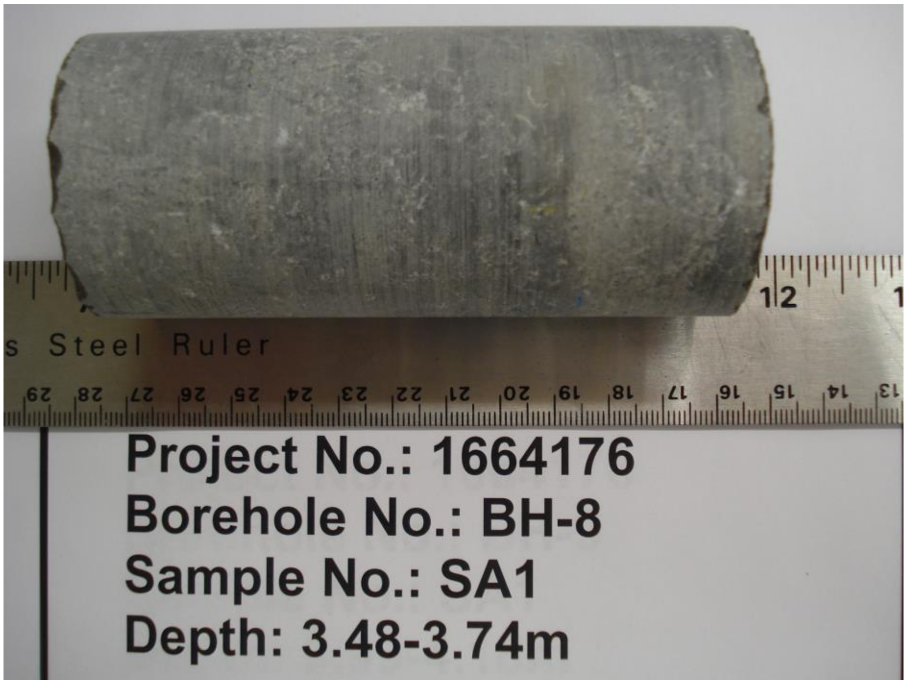
REMARKS: L/D Ratio not in accordance with ASTM Standard

Checked By: SEMP

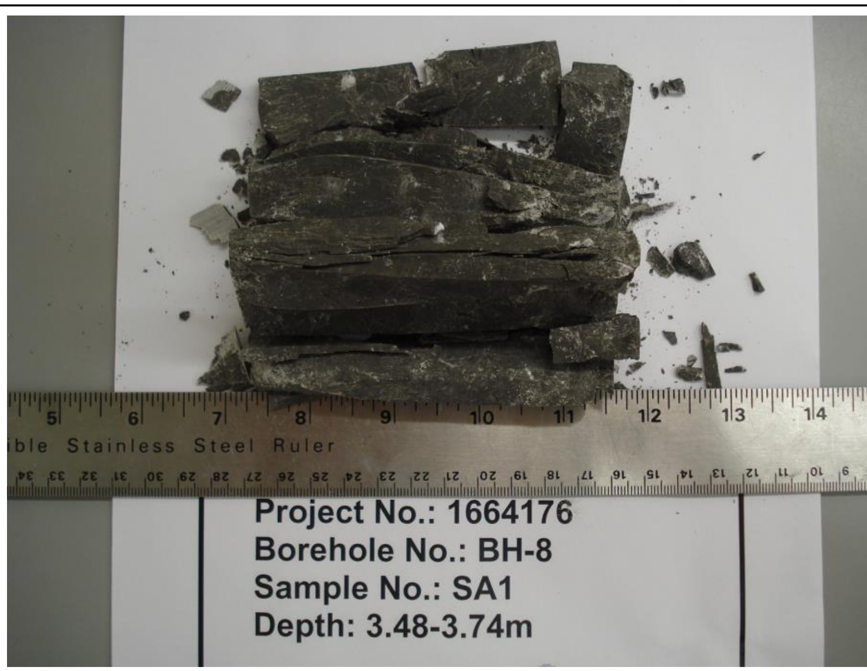
Golder Associates

UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS
ASTM D7012

FIGURE B5A



BEFORE COMPRESSION
CORE RUN # 3



AFTER COMPRESSION
CORE RUN # 3

Date Feb. 5, 2018
Project 1664176

Golder Associates

Drawn Frank
Chkd. SEMP

Form G.A.-D-4 (imperial) FORM PRODUCED JUNE 1986

Figure B5B

UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS
ASTM D7012

SAMPLE IDENTIFICATION

PROJECT NUMBER	1664176	SAMPLE NUMBER	SA 1
PROJECT NAME	MTO/MERO East Fnd Ret/Ontario	SAMPLE DEPTH, m	3.48-3.74
BOREHOLE NUMBER	BH-8	DATE:	01/29/2018

TEST CONDITIONS

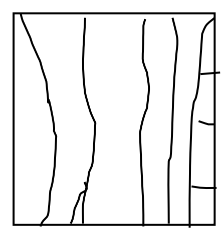
MACHINE SPEED, mm/min	N/A	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.24

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.60	WATER CONTENT, (specimen) %	0.04
SAMPLE DIAMETER, cm	4.74	UNIT WEIGHT, kN/m^3	26.47
SAMPLE AREA, cm^2	17.62	DRY UNIT WT., kN/m^3	26.46
SAMPLE VOLUME, cm^3	186.78	SPECIFIC GRAVITY	-
WET WEIGHT, g	504.38	VOID RATIO	-
DRY WEIGHT, g	504.18		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	N/A	COMPRESSIVE STRENGTH, MPa	75.7

REMARKS:

Checked By: SEMP

Golder Associates

APPENDIX C

Analytical Laboratory Test Results

5835 COOPERS AVENUE
MISSISSAUGA, ONTARIO
CANADA L4Z 1Y2
TEL (905)712-5100
FAX (905)712-5122
http://www.agatlabs.com

CLIENT NAME: GOLDER ASSOCIATES LTD.
100 SCOTIA COURT
WHITBY, ON L1N8Y6
(905) 723-2727

ATTENTION TO: Katie Nero

PROJECT: Collins Creek 1664176/001

AGAT WORK ORDER: 18T321750

SOIL ANALYSIS REVIEWED BY: Amanjot Bhela, Inorganic Coordinator

DATE REPORTED: Mar 27, 2018

PAGES (INCLUDING COVER): 6

VERSION*: 1

Should you require any information regarding this analysis please contact your client services representative at (905) 712-5100

*NOTES

All samples will be disposed of within 30 days following analysis. Please contact the lab if you require additional sample storage time.

5835 COOPERS AVENUE
MISSISSAUGA, ONTARIO
CANADA L4Z 1Y2
TEL (905)712-5100
FAX (905)712-5122
http://www.agatlabs.com

Certificate of Analysis

AGAT WORK ORDER: 18T321750

PROJECT: Collins Creek 1664176/001

CLIENT NAME: GOLDER ASSOCIATES LTD.

ATTENTION TO: Katie Nero

SAMPLING SITE:

SAMPLED BY:

Corrosivity Package

DATE RECEIVED: 2018-03-20

DATE REPORTED: 2018-03-27

			SAMPLE DESCRIPTION:	BH7 SA3
			SAMPLE TYPE:	Soil
			DATE SAMPLED:	2018-03-01
Parameter	**Unit**	**G / S**	**RDL**	**9139856**
Sulfide (S2-)	%		0.05	0.07
Chloride (2:1)	µg/g	NA	2	382
Sulphate (2:1)	µg/g		2	196
pH (2:1)	pH Units		NA	8.49
Electrical Conductivity (2:1)	mS/cm	0.57	0.005	**0.868**
Resistivity (2:1)	ohm.cm		1	1150
Redox Potential (2:1)	mV		5	142

Comments: RDL - Reported Detection Limit; G / S - Guideline / Standard: Refers to Table 1: Full Depth Background Site Condition Standards - Soil - Residential/Parkland/Institutional/Industrial/Commercial/Community Property Use
Guideline values are for general reference only. The guidelines provided may or may not be relevant for the intended use. Refer directly to the applicable standard for regulatory interpretation.

9139856 EC/Resistivity, pH, Chloride, Sulphate and Redox Potential were determined on the extract obtained from the 2:1 leaching procedure (2 parts DI water: 1 part soil).
*Sulphide analyzed at AGAT 5623 McAdam

Certified By: Amanjot Bhela

Guideline Violation

AGAT WORK ORDER: 18T321750

PROJECT: Collins Creek 1664176/001

5835 COOPERS AVENUE
MISSISSAUGA, ONTARIO
CANADA L4Z 1Y2
TEL (905)712-5100
FAX (905)712-5122
http://www.agatlabs.com

CLIENT NAME: GOLDER ASSOCIATES LTD. **ATTENTION TO: Katie Nero**

SAMPLEID	SAMPLE TITLE	GUIDELINE	ANALYSIS PACKAGE	PARAMETER	UNIT	GUIDEVALUE	RESULT
9139856	BH7 SA3	ON T1 S RPI/ICC	Corrosivity Package	Electrical Conductivity (2:1)	mS/cm	0.57	0.868

AGAT Laboratories
5835 COOPERS AVENUE
MISSISSAUGA, ONTARIO
CANADA L4Z 1Y2
TEL (905)712-5100
FAX (905)712-5122
http://www.agatlabs.com

Quality Assurance

CLIENT NAME: GOLDER ASSOCIATES LTD. | **AGAT WORK ORDER: 18T321750**

PROJECT: Collins Creek 1664176/001 | **ATTENTION TO: Katie Nero**

SAMPLING SITE: | **SAMPLED BY:**

Soil Analysis

RPT Date: Mar 27, 2018			DUPLICATE				REFERENCE MATERIAL			METHOD BLANK SPIKE			MATRIX SPIKE		
PARAMETER	Batch	Sample Id	Dup #1	Dup #2	RPD	Method Blank	Measured Value	Acceptable Limits Lower	Acceptable Limits Upper	Recovery	Acceptable Limits Lower	Acceptable Limits Upper	Recovery	Acceptable Limits Lower	Acceptable Limits Upper
Corrosivity Package															
Sulfide (S2-)	9139853	9139853	< 0.05	< 0.05	NA	< 0.05	98%	80%	120%						
Chloride (2:1)	9139777		103	88	15.7%	< 2	104%	80%	120%	94%	80%	120%	86%	70%	130%
Sulphate (2:1)	9139777		55	46	17.8%	< 2	98%	80%	120%	111%	80%	120%	111%	70%	130%
pH (2:1)	9139777		7.88	7.85	0.4%	NA	100%	90%	110%	NA			NA		
Electrical Conductivity (2:1)	9139777		0.292	0.272	7.1%	< 0.005	97%	90%	110%	NA			NA		
Redox Potential (2:1)	9139777		176	169	4.1%	< 5	101%	70%	130%	NA			NA		

Comments: NA signifies Not Applicable.
Duplicate Qualifier: As the measured result approaches the RL, the uncertainty associated with the value increases dramatically, thus duplicate acceptance limits apply only where the average of the two duplicates is greater than five times the RL.

Certified By: Amanjot Bhela

AGAT Laboratories

5835 COOPERS AVENUE
MISSISSAUGA, ONTARIO
CANADA L4Z 1Y2
TEL (905)712-5100
FAX (905)712-5122
http://www.agatlabs.com

Method Summary

CLIENT NAME: GOLDER ASSOCIATES LTD.
PROJECT: Collins Creek 1664176/001
SAMPLING SITE:

AGAT WORK ORDER: 18T321750
ATTENTION TO: Katie Nero
SAMPLED BY:

PARAMETER	AGAT S.O.P	LITERATURE REFERENCE	ANALYTICAL TECHNIQUE
Soil Analysis			
Sulfide (S2-)	MIN-200-12025	ASTM E1915-09	GRAVIMETRIC
Chloride (2:1)	INOR-93-6004	McKeague 4.12 & SM 4110 B	ION CHROMATOGRAPH
Sulphate (2:1)	INOR-93-6004	McKeague 4.12 & SM 4110 B	ION CHROMATOGRAPH
pH (2:1)	INOR 93-6031	MSA part 3 & SM 4500-H+ B	PH METER
Electrical Conductivity (2:1)	INOR-93-6036	McKeague 4.12, SM 2510 B	EC METER
Resistivity (2:1)	INOR-93-6036	McKeague 4.12, SM 2510 B,SSA #5 Part 3	CALCULATION
Redox Potential (2:1)		McKeague 4.12 & SM 2510 B	REDOX POTENTIAL ELECTRODE

5835 COOPERS AVENUE
MISSISSAUGA, ONTARIO
CANADA L4Z 1Y2
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FAX (905)712-5122
http://www.agatlabs.com

CLIENT NAME: GOLDER ASSOCIATES LTD.
100 SCOTIA COURT
WHITBY, ON L1N8Y6
(905) 723-2727

ATTENTION TO: Sarah Poot

PROJECT: 1664176

AGAT WORK ORDER: 18T341964

SOIL ANALYSIS REVIEWED BY: Amanjot Bhela, Inorganic Coordinator

DATE REPORTED: Jun 01, 2018

PAGES (INCLUDING COVER): 5

VERSION*: 1

Should you require any information regarding this analysis please contact your client services representative at (905) 712-5100

*NOTES

All samples will be disposed of within 30 days following analysis. Please contact the lab if you require additional sample storage time.

5835 COOPERS AVENUE
MISSISSAUGA, ONTARIO
CANADA L4Z 1Y2
TEL (905)712-5100
FAX (905)712-5122
http://www.agatlabs.com

Certificate of Analysis

AGAT WORK ORDER: 18T341964

PROJECT: 1664176

CLIENT NAME: GOLDER ASSOCIATES LTD.

ATTENTION TO: Sarah Poot

SAMPLING SITE:

SAMPLED BY:

Corrosivity Package

DATE RECEIVED: 2018-05-23

DATE REPORTED: 2018-06-01

			SAMPLE DESCRIPTION:	BH8_R1_0.61 m to 0.75 m
			SAMPLE TYPE:	Rock
			DATE SAMPLED:	2018-01-18
Parameter	Unit	G / S	RDL	9264359
Sulfide (S2-)	%		0.05	<0.05
Chloride (2:1)	µg/g		2	52
Sulphate (2:1)	µg/g		2	39
pH (2:1)	pH Units		NA	8.85
Electrical Conductivity (2:1)	mS/cm		0.005	0.262
Resistivity (2:1)	ohm.cm		1	3820
Redox Potential (2:1)	mV		5	120

Comments: RDL - Reported Detection Limit; G / S - Guideline / Standard

9264359 EC/Resistivity, pH, Chloride, Sulphate and Redox Potential were determined on the extract obtained from the 2:1 leaching procedure (2 parts DI water: 1 part soil).
*Sulphide analyzed at AGAT 5623 McAdam

Certified By: Amanjot Bhela

AGAT Laboratories

5835 COOPERS AVENUE
MISSISSAUGA, ONTARIO
CANADA L4Z 1Y2
TEL (905)712-5100
FAX (905)712-5122
http://www.agatlabs.com

Quality Assurance

CLIENT NAME: GOLDER ASSOCIATES LTD. | AGAT WORK ORDER: 18T341964
PROJECT: 1664176 | ATTENTION TO: Sarah Poot
SAMPLING SITE: | SAMPLED BY:

Soil Analysis

RPT Date: Jun 01, 2018			DUPLICATE				REFERENCE MATERIAL			METHOD BLANK SPIKE			MATRIX SPIKE		
PARAMETER	Batch	Sample Id	Dup #1	Dup #2	RPD	Method Blank	Measured Value	Acceptable Limits		Recovery	Acceptable Limits		Recovery	Acceptable Limits	
								Lower	Upper		Lower	Upper		Lower	Upper
Corrosivity Package															
Sulfide (S2-)	9264359	9264359	< 0.05	< 0.05	NA	< 0.05	99%	80%	120%						
Chloride (2:1)	9262257		11	11	0.0%	< 2	105%	80%	120%	96%	80%	120%	99%	70%	130%
Sulphate (2:1)	9262257		20	21	4.9%	< 2	95%	80%	120%	99%	80%	120%	105%	70%	130%
pH (2:1)	9262257		7.66	7.72	0.8%	NA	98%	90%	110%						
Electrical Conductivity (2:1)	9270072		0.408	0.405	0.7%	< 0.005	99%	90%	110%						
Redox Potential (2:1)	9263410		168	170	1.2%	< 5	103%	70%	130%						

Comments: NA signifies Not Applicable.
Duplicate Qualifier: As the measured result approaches the RL, the uncertainty associated with the value increases dramatically, thus duplicate acceptance limits apply only where the average of the two duplicates is greater than five times the RL

Certified By: Amanjot Bhela

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Method Summary

CLIENT NAME: GOLDER ASSOCIATES LTD.

PROJECT: 1664176

SAMPLING SITE:

AGAT WORK ORDER: 18T341964

ATTENTION TO: Sarah Poot

SAMPLED BY:

PARAMETER	AGAT S.O.P	LITERATURE REFERENCE	ANALYTICAL TECHNIQUE
Soil Analysis			
Sulfide (S2-)	MIN-200-12025	ASTM E1915-09	GRAVIMETRIC
Chloride (2:1)	INOR-93-6004	McKeague 4.12 & SM 4110 B	ION CHROMATOGRAPH
Sulphate (2:1)	INOR-93-6004	McKeague 4.12 & SM 4110 B	ION CHROMATOGRAPH
pH (2:1)	INOR 93-6031	MSA part 3 & SM 4500-H+ B	PH METER
Electrical Conductivity (2:1)	INOR-93-6036	McKeague 4.12, SM 2510 B	EC METER
Resistivity (2:1)	INOR-93-6036	McKeague 4.12, SM 2510 B,SSA #5 Part 3	CALCULATION
Redox Potential (2:1)		McKeague 4.12 & SM 2510 B	REDOX POTENTIAL ELECTRODE

APPENDIX D

Non-Standard Special Provisions

Mass Concrete – Item No.

Non-Standard Special Provision

Scope of Work

The scope of work for the above noted tender item includes the supply and placement of mass concrete under the overhead sign spread footing at Highway 401 STA 17+503, City of Kingston to raise the founding grade to the design level of the underside of the footings.

Construction

Concrete shall be the same strength as the footing concrete and placed in accordance with OPSS 904 (Concrete Structures).

Basis of Payment

Payment at the Contract Price for the above tender item includes full compensation for all labour, equipment and material to do the required work.

END OF SECTION

<u>Levelling Bedrock Surface</u> – Item No.

Non-Standard Special Provision

Scope of Work

The scope of work for the above noted tender item includes bedrock excavation at the cantilever overhead static sign at Highway 401 STA 17+503, City of Kingston to provide a level founding surface for the footing.

Construction

Prior to placing concrete for the proposed footing, the bedrock shall be levelled using mechanical means (i.e. hoe ram, line drilling, or equivalent) such that the surface of the bedrock is sloping less than 10 degrees throughout the footprint of the footing. The exposed bedrock must be cleaned by removing loose debris and rock shatter. The Foundations Engineer shall review the footing subgrade prior to placing concrete in accordance with OPSS 902, as amended by SP 109S12.

Basis of Payment

Payment at the Contract Price for the above tender item includes full compensation for all labour, equipment and material to do the required work.

END OF SECTION

<u>DOWELS INTO ROCK</u> - Item No.

Special Provision

CONSTRUCTION SPECIFICATION FOR THE SUPPLY, INSTALLATION AND TESTING OF DOWELS INTO ROCK FOR PIER FOOTINGS

1.0 SCOPE

The work for the above noted tender item shall be in accordance with OPSS 904, including all Special Provisions, except as extended herein. This document specifies additional requirements for the supply, installation and testing of Dowels into Rock for the pier footing.

2.0 REFERENCES

This Special Provision refers to the following standards, specifications, or publications:

ASTM International

D1143M Standard Test Methods for Deep Foundations Under Static Axial Compressive Load

3.0 DEFINITIONS

For the purpose of this Special Provision, the following definitions apply:

Dowels into Rock means reinforcing steel bar and non-shrink grout.

Design Engineer means an Engineer who has a minimum of five (5) years experience in all aspects associated with the underwater installation of Dowels into Rock, including drilling, underwater grouting and doweling work. The Design Engineer shall be retained by the Contractor to design various components for the installation and testing for the Dowels into Rock.

Quality Verification Engineer means an Engineer who has a minimum of five (5) years experience in all aspects associated with the underwater installation of Dowels into Rock, including drilling, underwater grouting and doweling work. The Quality Verification Engineer shall be retained by the Contractor to ensure conformance with the contract documents and issue certificate(s) of conformance.

4.0 DESIGN AND SUBMISSION REQUIREMENTS

4.01 Working Drawings

Working Drawings shall consist of drawings, testing and installation records, procedures and reports, and work plans.

The Contractor shall submit Working Drawings to the Contract Administrator as follows:

a) All Working Drawings that include drawing, testing and installation procedures and reports, and work plans shall be sealed and signed by the Design Engineer.

b) All Working Drawings that include testing and installation results and reports shall be signed and sealed by the Quality Verification Engineer.

Upon completion of testing or installation and testing for each component, the Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by a Quality Verification Engineer. The Certificate shall state that the work has been carried out in conformance with the Working Drawings and in general conformance with the contract documents.

Working Drawings consisting of testing an installation records and reports shall be submitted four days after completion of testing and installation. All other Working Drawings shall be submitted two weeks prior to construction.

Working Drawings to be submitted include the following with further details outlined in the remainder of this specification:

a) Design calculations, specifications and shop drawings covering all aspects of fabrication, installation and acceptance testing of Dowels into Rock.

b) Test results verifying the 28 day strength of non-shrink grout.

c) The method for constructing of the holes, maintaining the holes, and placing reinforcing steel bars, grout and other materials in the holes, including casing sizes, bit sizes and tremie grouting methods.

d) The procedures to verify hole length. Records of measurements that verify the hole length.

e) Records of all drilling procedures, rock conditions encountered, and installation times.

f) Test procedures for Dowels into Rock.

g) Drawings and design calculations for a suitable reaction system for the applied test loads.

h) Records of vertical and horizontal movements of the reaction system, and elongation of the reinforcing steel bar.

i) Drawings and details for reference system arrangement.

j) Current calibration curves shall be provided for all gauges.

k) Complete test records for all tests including plots of dowel movement versus dowel load, dowel load versus time, and dowel movement versus time.

l) Remedial measures for unacceptable stressing results.

5.0 MATERIALS

5.01 Non-Shrink Grout

The non-shrink grout shall be an approved product from the MTO's Pre-Qualified Products List.

5.02 Anti-Washout Agent

The anti-washout agent shall be used with the non-shrink grout for the Dowels into Rock. The anti-washout agent shall be one of the following proprietary products:

1) Sikament 100 SC Anti-Washout Admixture
   Sika Canada Inc.
   6915 Davand Drive
   Mississauga, ON, L5T 1L5
   Toll Free Phone: 800-933-7452

2) Rheomac UW 450 Anti-Washout Admixture
   BASF Construction Chemicals Canada Ltd (Master Builders)
   1800 Clark Blvd
   Brampton, ON, L6T 4M7
   Toll Free Phone: 416-520-1392

5.03 Manufacturer Information

The Contractor shall provide the following information from the manufacturer for non-shrink grout and anti-washout agent:

a) Data sheets for the non-shrink grout and anti-washout agent,

b) technical information that proves that the non-shrink grout and anti-washout agent are compatible, and

c) installation procedures.

6.0 EQUIPMENT

All equipment for the installation of the Dowels into Rock shall be suitable for the intended purposes and capable of working on the site under the prevailing access and clearance conditions.

The equipment shall not cause damage to the reinforcing steel bars.

7.0 CONSTRUCTION

7.01 Instructions to Contractor

These instructions are to be read in conjunction with the Contract Drawings.

A total of 2 test Dowels into Rock are required for the Dowels into Rock at the pier.

Dowels into rock at the pier shall be installed prior to unwatering the structure excavation. Dowels shall extend through tremie concrete and into sound bedrock to the specified embedment depth.

7.02 Responsibilities of the Contractor

The Contractor shall prove the allowable bond stress by tests of the Dowels into Rock on non-production Dowels into Rock.

The Contractor shall supply equipment, materials and skilled personnel to install production Dowels into Rock and conduct the specified acceptance tests. It shall be the responsibility of the Contractor to constantly monitor the acceptance tests, maintain specified test loads and record test measurements as specified by the Contract Administrator.

The Contractor is responsible for materials and workmanship. Any remedial measures, required because of defects in materials or workmanship, shall be completed by the Contractor at no cost to the Owner.

The Contractor shall submit 4 copies of all Working Drawings to the Contract Administrator as outlined in Section 4.0.

7.03 Subsurface Conditions

Rock and groundwater conditions are described in the Foundation Investigation Report for this Contract.

7.04 Construction of Holes

The sides and end of the hole shall not be disturbed. The Contractor shall submit Working Drawings to the Contract Administrator that include the method for constructing of the holes, maintaining the holes, and placing reinforcing steel bar, grout and other materials in the holes. All excavated material shall be removed from the site.

The hole diameters and hole length for this project are as specified on the Contract Drawings. Prior to commencing drilling operations, the Contractor shall submit Working Drawings to the Contract Administrator outlining devised procedures to verify hole length. The Contractor shall submit Working Drawings that include drilling operations records to the Contract Administrator that include the above noted records.

At all times, the Contractor shall keep a record of all drilling procedures, rock conditions encountered, and installation times. The Contractor shall submit Working Drawings to the Contract Administrator that include the above noted records.

7.05 Installation of Reinforcing Steel Bar

Reinforcing steel bar shall be installed in strict accordance with the Contract Drawings and installation procedures.

Centering devices shall be provided to ensure that the reinforcing steel bar is located centrally in the hole.

Dowels into Rock at the pier shall be installed prior to unwatering the structure excavation. Dowels shall extend through the tremie concrete for the pier footing and into sound bedrock.

Reinforcing steel bar shall be installed after the dowel hole has been filled with non-shrink grout.

7.06 Grout and Anti-Washout Agent

The non-shrink grout shall entirely fill the annular space between the reinforcing steel bar and side for the dowel hole.

The placement of grout for the test Dowels into Rock shall be identical to the production Dowels into Rock.

Anti-washout agent shall be used in accordance with the specifications of the manufacturer.

Non-shrink grout shall be placed into the dowel hole using tremie placement methods.

8.0 QUALITY ASSURANCE

All work for the installation of Dowels into Rock shall be inspected by the Quality Verification Engineer.

8.01 Qualifications

8.01.01 Qualifications of Staff from Contractor or Sub-Contractor Completing Work for the Dowels into Rock

All work shall be performed under the direction of personnel experienced with all aspects associated with the underwater installation of Dowels into Rock. Such experience shall have been obtained within the preceding five (5) years on projects of similar nature and scope to the work required for this project.

8.01.02 Qualifications of the Quality Verification Engineer

A resume of the work experience of the Quality Verification Engineer shall be submitted to the Contract Administrator for record purposes. The Quality Verification Engineer shall be a Professional Engineer licensed in the Province of Ontario having a minimum of five years of experience on projects of similar nature and scope to the work required for this project.

8.01.03 Qualifications of the Design Engineer

A resume of the work experience of the Design Engineer shall be submitted to the Contract Administrator for record purposes. The Design Engineer shall be a Professional Engineer licensed in the Province of Ontario having a minimum of five years of experience of projects of similar nature and scope to the work required for this project.

8.02 Testing Requirements

All work for the testing of Dowels into Rock shall be inspected by the Quality Verification Engineer.

8.02.01 General Testing Requirements

Refer to the attached Instructions to Contractor and the Contract Drawings for specific test details.

The Contractor shall install the number of Dowels into Rock specified in the contract documents for testing purposes. The purpose of the testing the Dowels into Rock is to prove the adequacy of the proposed anchor configuration and installation procedures under the site conditions, and to provide design parameters.

The equipment, labour and materials for test dowels shall be identical to Dowels into Rock at the pier. The Dowels into Rock for testing shall be 55M dowels grouted into 140 mm diameter holes filled with an approved non-shrink grout with a minimum 4,000 mm embedment into sound bedrock.

The Contractor shall submit Working Drawings that include proposed procedures for testing of the Dowels into Rock to the Contract Administrator. Such testing shall be executed in strict accordance with the proposed procedures of the Contractor.

The Quality Verification Engineer shall supervise the testing of the Dowels into Rock. The Contractor will notify the Contract Administrator of the testing schedule at least 10 days prior to commencement of the testing program. Testing for Dowels into Rock shall be conducted concurrently, as scheduled by the Contract Administrator. The tests shall normally be conducted between 8:00 hrs and 20:00 hrs from Monday to Friday, unless otherwise directed by the Contract Administrator.

The Contractor shall supply materials and skilled personnel to conduct the tests for the Dowels into Rock. The equipment and materials shall be capable of stressing the Dowels into Rock to the specified loads. It shall be the responsibility of the Contractor to constantly monitor the test, maintain specified test loads and to record test measurements as specified by the Quality Verification Engineer.

The test site shall be restored to its pre-test condition. Reinforcing steel bars used in tests shall be cut down 25 mm below the top of the sound bedrock.

8.02.02 Testing Location

The Contractor shall remove all loose rock down to sound bedrock at the test location.

The test Dowels into Rock shall be constructed at locations specified by the Contract Administrator. The water depth at the location of the test shall be at least 0.5 m deep.

If site conditions dictate, changes to the test locations will be considered. The Contractor shall provide the Contract Administrator at least 2 days notice in writing of this operation.

8.02.03 Testing Equipment

The dowels into rock will be carried out generally in accordance with the prevailing requirements of ASTM International D1143M superseded where applicable by the procedures specified in this document.

The Contractor shall submit Working Drawings for a suitable reaction system for the applied test loads to the Contract Administrator. Jacks must be secured with chains to provide adequate protection for the personnel in the event of breakage of the reinforcing steel bar or stressing system.

The Contractor shall submit Working Drawings for the reference system arrangement to the Contract Administrator. All reference beams shall be as follows:

The beams shall be independently supported with the support firmly embedded in the ground.

The testing device shall not apply compression to the bedrock surrounding the test for the Dowels into Rock, within a circle concentric with the dowel hole and a diameter equal to 4.0 m.

Reference beams shall be sufficiently rigid to support instrumentation such that variations in readings do not occur.

The Contractor shall construct suitable enclosures to provide complete protection for equipment and instruments from variations in the weather conditions and disturbances during the test program. These provisions must meet the approval of the Quality Verification Engineer and will include that the test enclosures must be weather-proof and provide a consistent temperature in order to eliminate temperature variations that could affect instrumentation.

8.02.04 Testing for Dowels Into Rock, and Report

At all times, the Contractor shall keep records of vertical and horizontal movements of the reaction system, elongation of reinforcing steel bar, and the record of test enclosure temperature. The movements shall be recorded with respect to an independent fixed reference point. The Contractor shall submit Working Drawings that include the above noted records to the Contract Administrator.

Dial gauges shall have at least a 76.2 mm (3.0 in.) travel. Longer gauge stems or sufficient gauge blocks shall be provided to allow for greater travel if required. Gauges shall have precision of at least 0.025 mm (0.0001 in.). The dial gauges shall be placed on smooth bearing surfaces mounted perpendicular to the direction of movement. All gauges, scales or reference points attached to the test anchor shall be mounted so as to prevent movement relative to the test anchor during the test. The Contractor shall submit Working Drawings that include details for current calibration and curves for all gauges to the Contract Administrator.

Jacks used for reinforcing steel bars shall have a minimum ram dimension of 152.6 mm (6.0 in.). The Contractor shall submit Working Drawings that include details for current calibration and curves for all gauges to the Contract Administrator.

Requirements for Clauses 5.4.1 to 5.4.4 shall be repeated as required at different testing locations.

8.02.05 Testing Loading

The testing procedures shall safely load test the Dowels into Rock in tension at a rate of approximately 100kN per minute to the test load of 1,150 kN. The load shall be increased by an additional 50 kN beyond this level as directed by the Quality Verification Engineer.

Each load shall be maintained for a minimum time of 15 minutes and until the rate of displacement is not greater than 0.25 mm (0.01 inches) per hour.

8.03 Acceptance Criteria

The following acceptance criteria apply:

a) The testing of dowels shall be carried out in advance of the instalment of Dowels into Rock at the pier footing.

b) Tests for Dowels into Rock shall have a capacity of at least 1035 kN. The Quality Verification Engineer shall report on the acceptance of the tests for Dowels into Rock. The Quality Verification Engineer shall report on the testing of the Dowels into Rock including recommendations for increasing embedment depth, if necessary.

9.0 MEASUREMENT FOR PAYMENT

For measurement purposes, a count shall be made of the number of dowels installed.

10.0 BASIS OF PAYMENT

Payment at the contract unit price for the above tender item shall include full compensation for all labour, equipment, and materials to do the work. No additional payment will be made for tests for Dowels into Rock which are deemed as included as part of the work for the above noted item.

WARRANT: Use only in consultation with Regional Structural Section with this non-standard tender item.

CUSTODIAN: Tony Sangiuliano, MERO - Foundation Group.