



June 2, 2016

FINAL FOUNDATION INVESTIGATION AND DESIGN REPORT

**OVERHEAD SIGNS - HIGHWAY 69 FOUR-LANING FROM 1.0 KM NORTH OF
THE NEW HIGHWAY 559 INTERCHANGE NORTHERLY TO 1.5 KM NORTH OF
HIGHWAY 7182 (SHEBESHEKONG ROAD) FOR 17 KM
MINISTRY OF TRANSPORTATION, ONTARIO
G.W.P. 5402-05-00**

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REPORT





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PART A

FOUNDATION INVESTIGATION REPORT

OVERHEAD SIGNS

HIGHWAY 69 FOUR-LANING

FROM 1.0 KM NORTH OF THE NEW HIGHWAY 559

INTERCHANGE NORTHERLY TO 1.5 KM NORTH OF

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MINISTRY OF TRANSPORTATION, ONTARIO

G.W.P. 5402-05-00



1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by McCormick Rankin, a member of MMM Group Limited (MRC) on behalf of Ministry of Transportation, Ontario (MTO) to provide detailed foundation engineering services for two (2) proposed overhead signs (OHS) within the Phase 1 limits of the new Highway 69 alignment. The proposed work is part of the four-laning of Highway 69 from 1.0 km north of the new Highway 559 Interchange northerly to 1.5 km north of Highway 7182 (Shebeshekong Road), which involves: high fill embankments and embankments over swamps; the New Woods Road and Shebeshekong Road interchanges and structures; the Shawanaga River and Site 9 Road structures; as well as culvert crossings. The general location/extent of the Phase 1 and Phase 2 of new Highway 69 four-laning alignment within which the overhead signs are located is shown on the Site Location Plan on Drawing 1. The general locations of the two overhead signs are shown on Drawings A1 and B1.

The Terms of Reference and the scope of work for the foundation investigation are outlined in MTO's Request for Proposal, dated January 2007. Golder's proposal for foundation engineering services associated with the overhead signs is contained in an Additional Scope of Work letter subsequent to MRC's Technical Proposal for this assignment. The work has been carried out in accordance with Golder's Project Specific Supplementary Specialty Plan for foundation engineering services for this project, dated July 4, 2007.

This report addresses the investigation carried out for two proposed overhead signs along Highway 69 near the north end of the Phase 1 section of the project. Separate reports address the foundation investigations and design for the related swamp crossings, high fill areas, culverts, bridge structures for the project, and other overhead signs.

The purpose of this investigation is to establish the subsurface conditions at the proposed overhead sign foundation locations by borehole drilling, rock coring, in situ testing and laboratory testing on selected samples. The foundation limits for this investigation were located in the field using survey stakes positioned by Callon Dietz Inc. (Callon Dietz), a professional surveying company. The area of the investigations is shown in plan on Drawings A1 and B1.

2.0 SITE DESCRIPTION

The location of the two proposed Overhead Signs, which are the subject of this report, are as follows:

- Overhead Sign 5 (OHS-5) located at Sta. 11+281 Shawanaga Twp.
- Overhead Sign 6 (OHS-6) located at Sta. 13+014 Shawanaga Twp. and was investigated as part of this assignment as Overhead Sign 6 (OHS-6).

OHS-5 is located on the existing alignment of Highway 69, which will become the northbound lanes of the new four-laned highway. OHS-6 is located approximately 30 m west of existing Highway 69, over the new southbound lanes of the four-laned highway. Based on plan and elevation drawings of the proposed design provided by MRC we understand that the anticipated final design conditions at the location of OHS-6 will vary from the existing conditions. Based on the general agreement drawings provided to us, the highway embankment will be constructed to about Elevation 215.0 m to support the new Southbound lanes of the four-laned highway. The embankment is expected to be between about 1.3 m and 3.0 m high relative to the existing ground surface (as measured at the borehole locations) and will be constructed of either rock or earth fill.



In general, the topography in the area of the sign locations consists of rolling terrain including densely treed areas and numerous bedrock outcrops separated by low-lying swamps containing areas of standing water and various types of vegetation and organic soils. The proposed overhead signs are located in areas of bedrock outcrop or areas with thin overburden or existing fills over bedrock.

The ground surface at the proposed overhead sign locations is at about Elevation 216.5 m at the OHS-5, and between about Elevation 213.7 m and 212.0 m at OHS-6.

3.0 INVESTIGATION PROCEDURES

The field work for the overhead sign investigation was carried out from December 14 to 16, 2015 during which time a total of four (4) boreholes were advanced, two (2) boreholes at each of the proposed overhead sign structures (i.e., one (1) borehole at each foundation element). The boreholes, designated as Boreholes OHS-5A, OHS-5B, OHS-6A, and OHS-6B were advanced at the locations shown in plan on Drawings A1 and B1 in Appendices A and B.

The field investigation was carried out using a track-mounted CME-55 drill rig supplied and operated by Landcore Drilling of Chelmsford, Ontario. The boreholes were advanced through the native overburden and fill material using 'NW' casing. Soil samples were obtained continuously or at intervals of depth of about 0.75 m using a 50 mm outside diameter (O.D.) split-spoon sampler driven by an automatic hammer in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586 Standard Test Method for Standard Penetration Tests and Split Barrel Sampling of the Soil). Samples of the bedrock were obtained using an 'NQ' size rock core barrel.

The boreholes were advanced through the overburden to auger and/or sampler refusal (i.e. inferred bedrock), or bedrock was exposed by shovel excavation of this overburden and bedrock was confirmed by coring in all of the boreholes. The boreholes were advanced to depths of up to about 4.9 m below existing ground surface, including coring of bedrock for core lengths between about 3.0 m and 3.7 m.

The groundwater conditions in the open boreholes were observed during the drilling operations and are described on the Record of Borehole sheets in Appendices A and B. It should be noted groundwater elevations will vary depending on seasonal fluctuations, precipitation and local soil permeability. All boreholes were backfilled with bentonite upon completion in accordance with Ontario Regulation 903 (as amended).

The field work was supervised by a member of our engineering staff who located the boreholes, arranged for the clearance of underground services, observed the drilling, sampling and in situ testing operations, logged the boreholes, and examined and cared for the soil and rock samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Mississauga geotechnical laboratory where the samples underwent further visual examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate. Classification testing, including as water content, grain size distribution and organic content was carried out on selected soil samples. Classification of the bedrock rock mass quality with respect to the Rock Quality Description (RQD) is described based on Table 3.10 of the Canadian Foundation Engineering Manual (2006) ¹. The results of the laboratory testing are included in Appendices A and B.

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



The boreholes were located in the field with stakes positioned by Callon Dietz. Ground surface elevations were obtained from cross-sections of existing ground conditions at the overhead sign locations provided by MRC. The borehole locations provided in the Record of Borehole and Drillhole sheets as well as on Drawings A1 and B1, are positioned relative to MTM NAD 83 northing and easting coordinates and the ground surface elevations are referenced to Geodetic datum. The borehole coordinates, ground surface elevation, and drilled depths are summarized below.

Borehole No.	Location (MTM NAD 83)		Collar Elevation (Geodetic Datum) (m)	Depth Drilled (m)
	Northing	Easting		
OHS-5A	5041344.9	248047.5	216.5	4.8
OHS-5B	5041332.4	248034.4	216.5	4.9
OHS-6A	5042579.9	246831.4	213.7	4.6
OHS-6B	5042567.6	246818.3	212.0	3.4

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

As delineated in The Physiography of Southern Ontario², this section of Highway 69 lies within the physiographic region known as the Georgian Bay Fringe, which extends along the east side of Georgian Bay through the Parry Sound and Muskoka areas, then eastward from Muskoka in patches into the area north of the Kawartha Lakes.

This part of the Georgian Bay Fringe physiographic region was never submerged during periods of glacial recession. As a result, the native surficial soils in this area consist of very shallow deposits of sand, silt and clay underlain by metamorphic bedrock; numerous bare knobs and ridges of bedrock are present throughout the area. Localized low lying swampy areas, containing peat and/or organic soils underlain by soft/loose native soils, are present in valleys between the bedrock knobs and ridges.

The bedrock in the area consists typically of gneisses of the Britt Domain of the Central Gneiss Belt, a subdivision of the Grenville Structural Province, as described in Geology of Ontario, OGS Special Volume 4³. Deposition of Palaeozoic strata initially covered the bedrock and later erosion during glaciation exposed these Precambrian rocks.

² Chapman, L.J. and Putnam, D.F., 1984. The Physiography of Southern Ontario, Ontario Geological Survey Special Volume 2, Third Edition. Accompanied by Map P.2715, Scale 1:600,000.

³ Geology of Ontario, 1991. Ontario Geological Society, Special Volume 4, Part 2. Ministry of Northern Development and Mines Ontario.



4.2 Subsurface Conditions

The detailed subsurface soil, bedrock and groundwater conditions as encountered in the boreholes advanced for this investigation, together with the results of the laboratory tests carried out on selected soil samples, are presented on the attached Record of Borehole and Drillhole sheets in Appendices A and B. The stratigraphic boundaries shown on the Record of Borehole and Drillhole sheets are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Variation in the stratigraphic boundaries between and beyond boreholes will exist and is to be expected.

In general, the subsurface conditions in the area of the proposed overhead signs typically consist of a cohesionless deposit sand and gravel fill or native silt and sand underlain by granitic gneiss bedrock. The following sections provide information on the subsoils and groundwater conditions encountered in the boreholes advanced at each of the proposed overhead sign location.

4.3 Overhead Sign 5 - STA 11+281 (Township of Shawanaga)

Two (2) boreholes (Boreholes OHS-5A and OHS-5B) were advanced at the proposed locations of the foundation support elements for OHS-5, within the shoulder of the existing Highway 69 embankments, as shown on drawing A1. In general, bedrock was encountered underlying sand and gravel fill from the existing highway shoulders.

4.3.1 Granular Fill

Granular fill material was encountered at the ground surface at both borehole locations. The granular fill is characterized by an upper granular base layer, approximately 250 mm to 330 mm thick, consisting of sand and gravel, trace to some silt and trace to some reclaimed asphalt pavement fragments. The Standard Penetration Test (SPT) 'N'-values measured within the granular base is 11 blows and 15 blows per 0.3 m of penetration, indicating a compact relative density. The natural water content measured on two (2) sample of this layer is 6 percent. The grain size distribution of two (2) samples of the granular base is presented on Figure A1 in Appendix A.

The lower granular subbase layer extends from the bottom of the granular base layer to the top of bedrock at approximately 1.2 m to 1.8 m below existing ground surface. The granular subbase material is characterized as gravelly sand, trace silt and cobbles are inferred present within the subbase fill from observations of drilling progress. The Standard Penetration Test (SPT) 'N'-values measured within the granular subbase layer range from 6 blows per 0.3 m of penetration to SPT Refusal (greater than 100 blows for less than 0.3 m penetration), indicating a loose to very dense relative density.

4.3.2 Bedrock

Bedrock was encountered below the granular fill in Boreholes OHS-5A and OHS-5B, and core samples were recovered as shown on Figure A2 and A3. The depth to bedrock below ground surface and the corresponding bedrock surface elevation are summarized below.



Borehole No.	Depth to Bedrock Surface (m)	Bedrock Surface Elevation (m)	Refusal Type
OHS-5A	1.8	214.7	Split spoon; Bedrock Cored
OHS-5B	1.2	215.3	Casing advancement; Bedrock Cored

Based on the rock core samples, the bedrock consists of granitic gneiss. In general the bedrock samples are described as moderately weathered to fresh, crystalline, medium to coarse grained, and grey with pink banding. The Rock Quality Designation (RQD) measured on the core samples ranges from 38 percent to 97 percent, indicating a rock mass of poor to excellent quality. The Total Core Recovery (TCR) of the core samples is between 86 percent and 100 percent and the Solid Core Recovery (SCR) of samples recovered is between 52 percent and 100 percent.

Laboratory point load strength tests were completed on selected samples of the rock core. The axial and diametral point load strength index values are shown on the Record of Drillhole sheets and are presented in Table A1 in Appendix A. The axial tests carried out on six (6) core samples of the granitic gneiss bedrock resulted in measured Is_{50} values ranging from about 3.4 MPa to 6.5 MPa and the diametral tests carried out on five (5) core samples of the granite gneiss bedrock resulted in measured Is_{50} values ranging from about 2.1 MPa to 5.0 MPa.

Based on the point load testing results and in accordance with Table 3.5 in CFEM (2006)¹ the granite gneiss bedrock is classified as strong (R4, 2 MPa < Point Load Index < 4 MPa) to very strong (R5, 4 MPa < Point Load Index < 10 MPa).

4.3.3 Groundwater Conditions

The granular fill samples obtained from the boreholes were moist to wet. The water level observed in Borehole OHS-5B during drilling was 1.1 m below existing ground surface, corresponding to Elevation of 215.4 m. The water level in Borehole OHS-5A was dry upon completion of overburden drilling, before the introduction of drilling water for coring operations and was not measured upon completion of coring.

It should be noted that the groundwater level as measured during the drilling operation is not stabilized and is subject to seasonal fluctuations and precipitation events.

4.4 Overhead Sign 6 - STA 13+014 (Township of Shawanaga)

Two (2) boreholes (Boreholes OHS-6A and OHS-6B) were advanced at the proposed locations of the foundation support elements for OHS-6, as shown on Drawing B1. In general, the subsurface conditions consist of topsoil and a silt and sand deposit overlying bedrock.

4.4.1 Topsoil

A layer of topsoil was encountered at the ground surface at both borehole locations, ranging in thickness from 100 mm to 200 mm and was characterized as sandy organic silt to sandy fibrous peat, some silt. The topsoil layer extends to the bedrock surface in Borehole OHS6-B, at about Elevation 211.9 m. The natural water content measured in two (2) samples of this deposit is 28 and 44 per cent. The organic content on one sample of topsoil tested is about resulted in a value of 7 per cent.



4.4.2 Silt Sand

A 0.4 m thick deposit of silt and sand was encountered underlying the topsoil in Borehole OHS-6A, and extends to the top of bedrock, approximately 1.1 m below existing ground surface corresponding to Elevation 212.6 m. The silt and sand was characterized as brown to red-brown in colour and contains traces of gravel and organics and cobbles are inferred from drilling progress. The Standard Penetration Test (SPT) 'N'-values measured within the silt and sand deposit are 9 blows per 0.3 m penetration and 100 blows per 0.18 m of penetration (Refusal) indicating a loose to very dense relative density. The natural water content measured on two (2) samples of this deposit is 13 and 17 per cent. The grain size distribution of one (1) sample of the silt and sand deposit is presented on Figure B1 in Appendix B.

4.4.3 Bedrock

Bedrock was encountered below the silt and sand deposit and topsoil layer overburden soils and core samples were recovered in Boreholes OHS-6A and OHS-6B. The depth to bedrock below ground surface and the corresponding bedrock surface elevation are summarized below.

Borehole No.	Depth to Bedrock Surface (m)	Bedrock Surface Elevation (m)	Refusal Type
OHS-6A	1.1	212.6	Split Spoon; Bedrock Cored
OHS-6B	0.1	211.9	Bedrock Cored

Based on the rock core samples, the bedrock mainly consists of granitic gneiss with a zone of gabbro overlying the granitic gneiss from 1.1 m to 1.5 m below ground surface in Borehole OHS-6A. In general the granitic gneiss samples are described as moderately weathered to fresh, crystalline, medium to coarse grained and grey with pink banding, while gabbro samples were described as highly weathered, crystalline, dark green, medium to coarse grained with large feldspathic clasts. The Rock Quality Designation (RQD) measured on the core samples ranges between 61 percent and 96 percent, indicating a rock mass of fair to excellent quality. The Total Core Recovery (TCR) and Solid Core Recovery (SCR) of the samples recovered are between 94 percent and 100 percent, and between 66 percent and 97 percent, respectively.

Laboratory point load strength tests were completed on selected samples of the rock core. The axial and diametral point load strength index values are shown on the Record of Drillhole sheets and are presented in Table B1 in Appendix B. The axial and diametral tests carried out on one (1) core sample of the gabbro bedrock measured Is_{50} values of about 1.4 MPa and 3.1 MPa, respectively. The axial tests carried out on five (5) core samples of the granitic gneiss bedrock measured Is_{50} values ranging from about 6.8 MPa to 12.7 MPa and the diametral tests carried out on five (5) core samples of the granitic gneiss bedrock resulted Is_{50} values ranging from about 1.9 MPa to 8.4 MPa.

Based on the point load testing results and in accordance with Table 3.5 in CFEM (2006)¹ the gabbro is classified as medium strong (R3, 1 MPa < Point Load Index < 2 MPa) to strong (R4, 2MPa < Point Load Index < 4 MPa); and the granitic gneiss bedrock is classified as medium strong (R3, 1 MPa < Point Load Index < 2 MPa) to extremely strong (R6, Point Load Index > 10 MPa).



4.4.4 Groundwater Conditions

In general, the overburden samples taken in the boreholes advanced were moist to wet. The water level was measured during drilling in Borehole OHS-6A at 0.6 m below existing ground surface, at Elevation 213.1 m. The groundwater level was not encountered in Borehole OHS-6B upon completion of bedrock coring, but the limited thickness of overburden encountered indicate dry conditions.

It should be noted that the groundwater level as measured during the drilling operation is not stabilized and is subject to seasonal fluctuations and precipitation events.

5.0 CLOSURE

Mr. David Marmor, E.I.T. directed the drilling program and prepared this foundation investigation report. The report was reviewed by Mr. John Hagan, P. Eng., a geotechnical engineer with Golder. Mr. Jorge M. A. Costa, P. Eng., Golder's Designated MTO Contact for this project and Senior Consultant with Golder, completed an independent quality control review of the report.



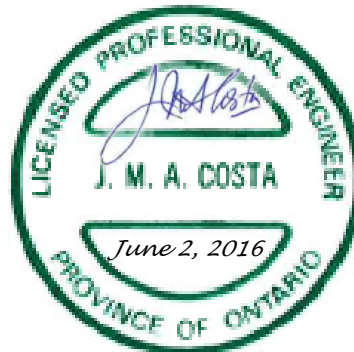
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PART B

FOUNDATION DESIGN REPORT

OVERHEAD SIGNS

HIGHWAY 69 FOUR-LANING

FROM 1.0 KM NORTH OF THE NEW HIGHWAY 559

INTERCHANGE NORTHERLY TO 1.5 KM NORTH OF

HIGHWAY 7182 (SHEBESHEKONG ROAD) FOR 17 KM

MINISTRY OF TRANSPORTATION, ONTARIO

G.W.P. 5402-05-00



6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides geotechnical parameters and recommendations for the design and construction of foundations for the proposed overhead signs. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation for this project. The interpretation and recommendations presented are intended to provide the designers with sufficient information to design the proposed sign foundations. Where comments are made on construction, they are provided in order to highlight those aspects which could affect the planning of the project, and for which special provisions or operational constraints may be required during construction. Those requiring information on the aspects of construction should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

6.1 General

Golder Associates Ltd. (Golder) was retained by McCormick Rankin, a member of MMM Group Limited (MRC) on behalf of Ministry of Transportation, Ontario (MTO) to provide an assessment of foundation options, geotechnical parameters and recommendations on foundation aspects for two (2) proposed overhead signs to be located above the existing and new Highway 69 NBL and SBL at approximately STA 11+281 and 13+014, in the Township of Shawanaga.

OHS-5 is located on the existing alignment of Highway 69 at STA 11+281, Shawanaga Twp., which will become the northbound lanes of the new four-laned highway. The anticipated final design conditions at the location of OHS-5 will be similar to the existing conditions. OHS-6 is located approximately 30 m west of existing Highway 69 at STA 13+014, Shawanaga Twp., over the new southbound lanes of the four-laned highway. Based on plan and elevation drawings of the proposed design provided by MRC we understand that the anticipated final design conditions at the location of OHS-6 will vary from the existing conditions. Based on the drawings, a highway embankment will be constructed to about Elevation 215.0 m to support the new Southbound Lanes (SBL) of the four-laned highway. The embankment is expected to be between about 1.3 m and 3.0 m high above existing ground surface (as measured at the borehole locations) and will be constructed of either rock fill or earth fill.

The overall project involves the design of a 17 km section of the new Highway 69 four-laning alignment north of Nobel, Ontario, including high fill embankments and embankments over swamps, the Woods Road and Shebeshekong Road interchanges and structures, the Shawanaga River and Site 9 Road structures, as well as culvert crossings and overhead signs.

6.2 Overhead Sign Foundations

Support of overhead signs, understood to be tri-chord static signs for these sites, are typically designed with a "standard" caisson foundation design, in accordance with the requirements in MTO's *Sign Support Manual* (2015). However, given that the bedrock at the proposed two overhead sign locations at these sites is present at relatively shallow depths below ground surface, the foundations for the support of the overhead signs can be designed as caissons socketed into bedrock as specified in Note 1 of Notes to Designer on Standard Drawing SS118-3 in the *Sign Support Manual* or as spread footings founded on, and potentially dowelled into, the bedrock. Recommendations for these two foundation options are provided in Sections 6.2.1 and 6.2.2.



At OHS-5, spread footings founded on and doweled into bedrock are considered the most appropriate and feasible option from a foundations perspective. Bedrock is present at shallow depth below ground surface at both foundation elements, given that, relatively shallow excavations will be required. Construction of shallow footings avoids the need to mobilize expensive and relatively specialized equipment for coring and churn drilling, which is considered relatively more contractor specific than the installation of spread footings founded on and doweled into bedrock. The use of steel liners to prevent sloughing or caving of the cohesionless fill material would also likely be required if caissons are to be installed.

At OHS-6 caissons socketed into bedrock are considered the most appropriate and feasible foundation option from a foundations perspective. The excavation depth required for shallow footings would be as deep as 3.1 m through the embankment fill. Because of the depth of excavation required for spread footings, caissons are considered the most feasible option, despite the need for specialized equipment and steel liners. In addition, the caisson option allows for the use of the standard caisson foundation design, for a modified length given the presence of bedrock at shallow depth as outlined in Note 1 of Notes to Designer on Standard Drawing SS118-3 in MTO's Sign Support Manual (2015).

The advantages, disadvantages, relative costs and risks/consequences for each of the foundation options are also summarized in Table 1 and Table 2 for OHS-5 and OHS-6, respectively.

6.2.1 Caissons

As noted above, caisson foundations for overhead sign supports should be designed in accordance with the requirements in MTO's *Sign Support Manual (2015)*. The *Sign Support Manual* includes a standard caisson foundation design for tri-chord static sign supports (Section 4 and Standard Drawings SS118-3, SS118-4 and SS118-5), in which the caisson extends 5 m below the design frost penetration depth, except where bedrock is encountered within this depth as stated in Note 1 of the Notes to Designer on Standard Drawing SS118-3. As shown on the depth of frost penetration isopleths for Southern Ontario in OPSD 3090.101 (Foundation Frost Penetration Depths), the estimated depth of frost penetration at the site is approximately 1.8 m.

In accordance with Note 1 of the Notes to Design on Standard Drawing SS118-3 of MTO's *Sign Support Manual (2015)*, where bedrock is encountered at a depth less than 5 m below the bottom of the depth of frost penetration, the required depth of the caisson foundation below the frost depth may be taken as follows:

$$L = Y + \frac{5-Y}{2}$$

where: L = length of caisson below depth of frost penetration (m)
 Y = distance between depth of frost penetration and depth to bedrock (m)

Based on the above equation, the length of caisson as well as the length of caisson socketed into the gneiss bedrock for the two overhead signs are summarized below.



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Overhead Sign	Borehole No.	Depth to Bedrock (m)	Depth of Frost Penetration (m)	Distance between Depth of Frost Penetration and Depth to Bedrock Y (m)	Caisson Length Below Depth of Frost Penetration $L = Y + (\frac{5-Y}{2})$ (m)	Total Caisson Length (m)	Length of Caisson Socketed into Bedrock (m)
OHS-5	OHS-5A	1.8	1.8	$1.8 - 1.8 = 0.0$	2.5	$2.5 + 1.8 = 4.3$	$4.3 - 1.8 = 2.5$
	OHS-5B	1.2	1.2	$1.2 - 1.2 = 0.0$	2.5	$2.5 + 1.2 = 3.7$	$3.7 - 1.2 = 2.5$
OHS-6	OHS-6A	2.4	1.8	$2.4 - 1.8 = 0.6$	2.8	$2.8 + 1.8 = 4.6$	$4.6 - 2.4 = 2.2$
	OHS-6B	3.1	1.8	$3.1 - 1.8 = 1.3$	3.2	$3.2 + 1.8 = 5.0$	$5 - 3.1 = 1.9$

It should be noted that the recommendations provided above are applicable for the anticipated final design conditions. For OHS-5, these conditions are expected to be very similar to the existing conditions. For OHS-6, the anticipated final design conditions will consist of a highway embankment constructed to approximate Elevation 215.0 m and will be about 1.3 m and 3.0 m high above existing ground surface at the locations of Boreholes OHS-6A and OHS-6B, respectively.

For concrete caissons socketed into bedrock, the lateral resistance will be developed primarily from the fixity (in concrete) within the drilled sockets. In this case, the structural resistance of the caisson will govern the ultimate lateral resistance. A minimum socket length as noted above is required.

Based on the existing subsurface soil conditions encountered above bedrock at OHS-5 (loose to very dense non-cohesive materials and relatively shallow depth to the groundwater level and the subsurface soil conditions above bedrock at OHS-6 (proposed non-cohesive embankment fill and a relatively shallow depth to the groundwater level), the construction of the caissons should be carried out within a temporary steel liner to avoid the open hole from caving or sloughing prior to pouring concrete. As concrete is placed in the liner-protected hole (by tremie placement method) the temporary steel liner should be removed progressively to the extent that the surface of the concrete is always within the steel liner and above the discharge point to prevent caving-in of the hole and mitigate the potential for segregation or the formation of voids in the concrete, and removal of the liner can occur simultaneously. As the caisson will be socketed into the bedrock, consideration can be given to leaving the steel liner in place permanently.

The bedrock at the proposed overhead sign locations is classified as poor to excellent quality bedrock based on the RQD of the rock core samples taken at the sign foundation locations and as such, appropriate equipment and construction procedures (such as coring or churn drilling techniques) would be required to advance the sockets into the bedrock.



6.2.2 Spread Footings

As an alternative to caissons socketed into bedrock, consideration could be given to using spread footings to support the overhead signs. At the proposed sign locations, all of the spread footings should be founded on bedrock. However, because of the shallow depth to bedrock it may be necessary to anchor the footing to bedrock to achieve adequate lateral resistance. In addition, variations in the bedrock surface elevation are to be expected in the areas of the proposed overhead signs and as such, mass concrete and/or hoe ramming may be required to achieve a level footing at the design elevations.

The bedrock encountered at the proposed overhead sign locations is generally of good quality, but nevertheless, the founding surface should be properly prepared (i.e., sub-excavated of any loose and fractured bedrock). Where the bedrock surface is above the design elevation of the footing, hoe ramming may be required to achieve the founding grade of the underside of the footing. Conversely, where bedrock surface is below the design elevation of the footing, mass concrete would be required to raise the founding grade to the design elevation of the underside of the footing. Given the potential of encountering an uneven and slopping bedrock surface, a Non-Standard Special Provision (NSSP) for mass concrete should be included in the Contract Documents in the event that a thicker footing is required; an example NSSP is provided in Appendix C.

Open cut excavations of short duration through the granular fill and silt and sand deposit are considered feasible for the proposed footing construction. Excavations for the proposed spread footings should be carried out in accordance with the latest Occupational Health and Safety Act for Construction Projects (OHSA). When referencing OHSA, the granular fills and native sand and silt deposit should be considered as "Type 3 Soil". As such, excavations should be sloped at a gradient of 1 Horizontal to 1 Vertical (1H:1V) or flatter. For excavations into the bedrock, if necessary, the overall slope to the cut face may be formed vertically, or near vertically (i.e. about 0.5H:1V).

Given the anticipated limited size of the excavation and limited overburden thickness, as well as the presence of groundwater in the boreholes essentially at/near the bedrock surface, seepage into the excavation should be adequately controlled by pumping from properly filtered sumps. However, it should be noted that the groundwater levels are subject to seasonal fluctuations and precipitation events and as such, the proposed construction method and/or the construction schedule should be planned accordingly.

Based on email corresponded with MRC, Golder understands that OHS-5 will be constructed in Phase 1, Stage 2 of construction period over the existing Highway 69 after traffic has been diverted onto the newly constructed Southbound lanes (i.e. there will be no live traffic while the sign is being constructed). In addition Golder understands that OHS-6 will be constructed in Phase 1, Stage 1 during construction of the new Southbound lanes while traffic continues to use the existing Highway 69 (i.e. there will be no live traffic while the sign is being constructed).

At the location of OHS-5, the need to protect the existing pavement structure is limited as there will be no live traffic using the highway during construction and the existing structure will likely be reconstructed/rehabilitated as part of the project. If shoring is required for footing construction it should be designed/constructed in accordance with Ontario Provincial Standard Specifications (OPSS.PROV) 539 (Temporary Protection Systems), designed to meet Performance Level 3. The following design parameters may be used for the design of temporary shoring at OHS-5:



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Design Parameter	Existing Granular Fill
Unit Weight above Groundwater Level γ (kN/m ³)	20
Unit Weight below Groundwater Level γ' (kN/m ³)	10
Friction Angle ϕ (°)	28
K_a^*	0.36
K_p^*	2.8
K_o^*	0.53

* Earth pressure coefficients for horizontal backfill.

At OHS-6 the sign foundations will be constructed within the outer sections of the new SBL embankment and protecting the existing pavement structure would not be relevant if the pavement structure has not yet been constructed. However, the need for shoring may be necessary due to a limited working platform on the newly constructed embankment. If shoring is required it should be designed/constructed in accordance with OPSS.PROV 539 (Temporary Protection Systems), designed to meet Performance Level 3. The following design parameters may be used for the design of temporary shoring at OHS-6:

Design Parameter	New Granular Fill	New Rock Fill
Unit Weight above Groundwater Level γ (kN/m ³)	21	19
Unit Weight below Groundwater Level γ' (kN/m ³)	11	9
Friction Angle ϕ (°)	32	40
K_a^*	0.31	0.22
K_p^*	3.3	4.6
K_o^*	0.47	0.36

* Earth pressure coefficients for horizontal backfill.

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, for the sign proposed to be located on the existing highway, the distance between the crest of the excavation and the toe of the stockpile should be greater than the depth of adjacent excavation.

Inspection and approval of the foundation areas by the Quality Verification Engineer prior to footing construction should be carried out as required in accordance with OPSS 902 (Excavating and Backfilling), to ensure that all loose soils and/or fractured rock has been removed from the foundation areas and that the foundation base has been properly prepared for the placement of concrete.

6.2.2.1 Geotechnical Axial Resistance

For spread footings bearing directly on the granite or gabbro bedrock surface or on mass concrete over bedrock, a factored geotechnical axial resistance at Ultimate Limit States (ULS) of 1.5 MPa may be used for design. Serviceability Limit States (SLS) conditions do not apply for footings founded on bedrock or on mass concrete.



The geotechnical resistances provided above are given for loads applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with Clauses 6.7.4 and C6.7.4 of the *Canadian Highway Bridge Design Code (CHBDC 2006)* and its *Commentary*.

6.2.2.2 Resistance to Lateral Loads

Resistance to lateral forces/sliding resistance between the concrete footings and the prepared bedrock surface should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The following summarizes the coefficient of friction, $\tan \delta$, for the interface materials.

Interface Material	Coefficient of Friction ($\tan \delta$)
Concrete footing on Bedrock	0.70

This value represents an unfactored value.

At the location of OHS-5, spread footings, if adopted, will be founded at bedrock surface at depths of 1.3 m and 1.8 m below existing ground surface. The resistance to lateral forces/sliding will develop between the concrete footings and the prepared bedrock surface. The resistance to lateral forces/sliding should be calculated in accordance with Section 6.7.5 of the *CHBDC (2006)*, based on the coefficient of friction, $\tan \delta$, equal to 0.70 for the bedrock/concrete interface as above. This value represents an unfactored value. The sliding/lateral resistance between the concrete footing/mass concrete and the bedrock may also be supplemented by dowelling into the bedrock, if evaluated to be necessary by the designer as discussed further below.

At the location of OHS-6 the anticipated final design conditions will consist of a newly constructed highway embankment with final grade at approximately Elevation 215 m (fill thickness of approximately 1.3 m to 3.0 m high relative to the existing ground surface at the east and west sign foundation elements, respectively) constructed of either granular fill or rock fill. For spread footings founded at bedrock at OHS-6, the founding levels will be at approximately Elevation 212.6 m at the east foundations element (Borehole OHS-6A) and Elevation 211.9 m at the west foundation element (Borehole OHS-6B). Based on an embankment grade at approximately Elevation 215.0 m there will be about 2.4 m and 3.1 m of new embankment fill and native soil above the founding elevation for spread footings at the east and west foundation elements, respectively. Resistance to lateral forces/sliding resistance between the concrete footings and the prepared bedrock surface should be calculated in accordance with Section 6.7.5 of the *CHBDC (2006)*, based on a coefficient of friction, $\tan \delta$, for the interface materials equal to 0.70 as above. This value represents an unfactored value. In addition, passive earth pressure for the component of fill/native soil below the depth of frost penetration will also be developed and can be calculated on the basis of a passive earth pressure coefficient, K_p , equal to 3.3 for an embankment constructed of new granular fill or 4.6 for embankment constructed of rock fill. The sliding/lateral resistance between the concrete footing/mass concrete and the bedrock may also be supplemented by dowelling into the bedrock, if evaluated to be necessary by the designer as discussed further below.

For footings on bedrock, at both OHS-5 and OHS-6, the sliding/lateral resistance between the concrete footing/mass concrete and the bedrock, and the passive earth pressure at OHS-6, 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. For this site, where the rock mass uniaxial compressive strength is estimated to be between about 30 MPa and



75 MPa for the gabbro bedrock and between about 40 MPa and 250 MPa for the granite bedrock, as correlated from the point load test results as per Table 3.5 of CFEM (2006), indicating that the bedrock is essentially as strong or stronger than concrete (assuming that 30 MPa concrete will be used for construction of the footings and for “mass concrete” filling). The design of the dowels into the rock may be handled in the same way as the dowel embedment into the concrete. This assumes that the Unconfined Compressive Strength (UCS) of the grout will be similar to that of the 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 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; an example is provided in Appendix C.

6.3 Construction Considerations

The excavation around and above the spread footing may be backfilled using granular material such as OPSS.PROV 1010 (Aggregates) Granular ‘A’ or ‘B’ (Type I or II) placed in 300 mm thick loose lifts and uniformly compacted to the requirements outlined in OPSS.PROV 501 (Compacting).

The final grade surrounding the sign should be sloped to promote surface water drainage away from the pavement and sign, to the adjacent ditch, and surfaced with top soil and seed, in accordance with OPSS.PROV 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.

As the excavations, either for spread footings or for caissons will extend through granular overburden and likely to below the groundwater level(s), it is recommended that a NSSP be included in the Contract Documents to warn the Contractor of these conditions which may affect the installation of the overhead sign foundations. An example NSSP is provided in Appendix C.

7.0 CLOSURE

Mr. David Marmor, E.I.T prepared this Foundation Design report. The report was reviewed by Mr. John Hagan, P. Eng., a geotechnical engineer with Golder. Mr. Jorge M. A. Costa, P. Eng., Golder’s Designated MTO Contact for this project and Senior Consultant with Golder, completed an independent quality control review of the report.



Report Signature Page

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- ASTM International
- | | |
|------------|---|
| ASTM D1586 | Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils |
|------------|---|
- Ontario Provincial Standard Drawings
- | | |
|---------------|---|
| OPSD 3090.101 | Foundation, Frost Penetration Depths for Southern Ontario |
|---------------|---|
- Ontario Provincial Standard Specifications
- | | |
|----------------|--|
| OPSS.PROV 501 | Construction Specification for Compacting |
| OPSS.PROV 539 | Construction Specification for Temporary Protection Systems |
| OPSS.PROV804 | Construction Specifications for Seed and Cover |
| OPSS.PROV 902 | Construction Specification for Excavating and Backfilling - Structures |
| OPSS.PROV 1004 | Material Specification for Aggregates – Miscellaneous |
| OPSS.PROV 1010 | Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material |
- Ontario Water Resources Act
- Ontario Regulation 903/90



ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS

PARTICLE SIZES OF CONSTITUENTS

Soil Constituent	Particle Size Description	Millimetres	Inches (US Std. Sieve Size)
BOULDERS	Not Applicable	>300	>12
COBBLES	Not Applicable	75 to 300	3 to 12
GRAVEL	Coarse Fine	19 to 75 4.75 to 19	0.75 to 3 (4) to 0.75
SAND	Coarse Medium Fine	2.00 to 4.75 0.425 to 2.00 0.075 to 0.425	(10) to (4) (40) to (10) (200) to (40)
SILT/CLAY	Classified by plasticity	<0.075	< (200)

MODIFIERS FOR SECONDARY AND MINOR CONSTITUENTS

Percentage by Mass	Modifier
>35	Use 'and' to combine major constituents (i.e., SAND and GRAVEL, SAND and CLAY)
> 12 to 35	Primary soil name prefixed with "gravelly, sandy, SILTY, CLAYEY" as applicable
> 5 to 12	some
≤ 5	trace

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.) split-spoon sampler for a distance of 300 mm (12 in.).

Cone Penetration Test (CPT)

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

Dynamic Cone Penetration Resistance (DCPT); N_d:

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

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

SAMPLES

AS	Auger sample
BS	Block sample
CS	Chunk sample
DO or DP	Seamless open ended, driven or pushed tube sampler – note size
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
SS	Split spoon sampler – note size
ST	Slotted tube
TO	Thin-walled, open – note size
TP	Thin-walled, piston – note size
WS	Wash sample

SOIL TESTS

w	water content
PL, w _p	plastic limit
LL, 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
GS	specific gravity
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 (FV)	field vane (LV-laboratory vane test)
Y	unit weight

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

NON-COHESIVE (COHESIONLESS) SOILS

Compactness²

Term	SPT 'N' (blows/0.3m) ¹
Very Loose	0 - 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	>50

- SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects.
- Definition of compactness descriptions based on SPT 'N' ranges from Terzaghi and Peck (1967) and correspond to typical average N₆₀ values.

Field Moisture Condition

Term	Description
Dry	Soil flows freely through fingers.
Moist	Soils are darker than in the dry condition and may feel cool.
Wet	As moist, but with free water forming on hands when handled.

COHESIVE SOILS

Consistency

Term	Undrained Shear Strength (kPa)	SPT 'N' ¹ (blows/0.3m)
Very Soft	<12	0 to 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	>200	>30

- SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects; approximate only.

Water Content

Term	Description
w < PL	Material is estimated to be drier than the Plastic Limit.
w ~ PL	Material is estimated to be close to the Plastic Limit.
w > PL	Material is estimated to be wetter than the Plastic Limit.



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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

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

Notes: 1
2

$$\tau = c' + \sigma' \tan \phi'$$

$$\text{shear strength} = (\text{compressive strength})/2$$



LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

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

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

JOINT OR FOLIATION SPACING

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

GRAIN SIZE

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

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

CORE CONDITION

Total Core Recovery (TCR)

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

Solid Core Recovery (SCR)

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

Rock Quality Designation (RQD)

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

DISCONTINUITY DATA

Fracture Index

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

Dip with Respect to Core Axis

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

Description and Notes

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

Abbreviations

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



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Table 1: Evaluation of Foundation Alternatives – Overhead Sign 5

Option	Rank	Advantages	Disadvantages	Relative Costs	Risks / Consequences
Caissons socketed into Bedrock	2	<ul style="list-style-type: none"> ■ No post-construction settlement ■ Soil cover for frost protection is not required for caissons socketed into bedrock ■ Bedrock at/near ground surface ■ Standard caisson foundation design can be used modified for shallow bedrock, as outlined in MTO's Sign Support Manual. 	<ul style="list-style-type: none"> ■ Coring or churn drilling into the strong to very strong bedrock will be required to advance sockets for caisson construction. ■ Temporary liner for soil support during installation to prevent sloughing and caving of cohesionless soil. 	<ul style="list-style-type: none"> ■ Relatively higher cost of installation compared to spread footings; and, ■ Additional cost associated with specialized drilling equipment to advance the caisson holes into the bedrock. 	<ul style="list-style-type: none"> ■ Specialized drilling equipment will be required to socket caissons into bedrock.
Spread Footings founded on and Dowelled into Bedrock	1	<ul style="list-style-type: none"> ■ Relative ease of construction ■ No bedrock coring and/or churn drilling required ■ No post-construction settlement ■ Soil cover for frost protection is not required for footings on bedrock ■ Very high geotechnical axial resistance available ■ Bedrock at/near ground surface 	<ul style="list-style-type: none"> ■ Larger excavation of overburden is required producing a larger volume of excavation spoils ■ Larger volume of mass concrete may be required to achieve level footing; ■ Dowels may be required to anchor spread footings (due to structural considerations) ■ Groundwater control may be required. 	<ul style="list-style-type: none"> ■ Relatively lower cost in comparison to caissons socketed into bedrock; ■ Additional cost required for the disposal of larger volume of excavation spoils; and, ■ Additional costs required for installation of dowels into the bedrock. 	<ul style="list-style-type: none"> ■ Risk that additional excavation and mass concrete may be required if bedrock is encountered below the design founding elevation during construction; and, ■ Must ensure foundation base is properly prepared for concrete placement.

Prepared By: DPM

Reviewed By: JMAC



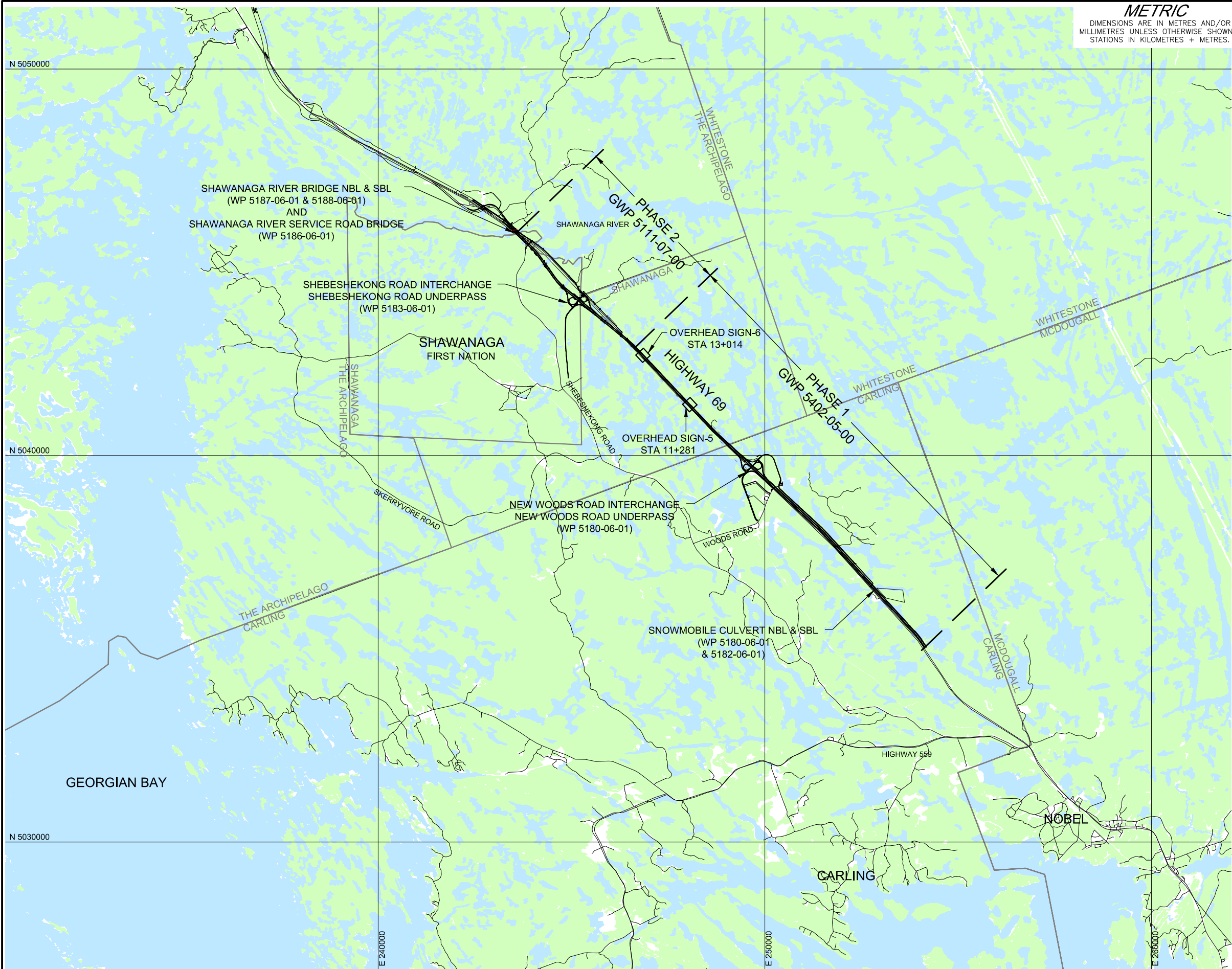
FOUNDATION REPORT - OVERHEAD SIGNS - HIGHWAY 69 - GWP 5402-05-00

Table 2: Evaluation of Foundation Alternatives – Overhead Sign 6

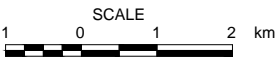
Option	Rank	Advantages	Disadvantages	Relative Costs	Risks / Consequences
Caissons socketed into Bedrock	1	<ul style="list-style-type: none"> No post-construction settlement Soil cover for frost protection is not required for caissons socketed into bedrock Standard caisson foundation design can be used modified for shallow bedrock, as outlined in MTO's Sign Support Manual. 	<ul style="list-style-type: none"> Coring or churn drilling into the strong to very strong bedrock will be required to advance sockets for caisson construction. Temporary liner for soil support during installation to prevent sloughing and caving of cohesionless soil. 	<ul style="list-style-type: none"> Relatively higher cost of installation compared to spread footings; and, Additional cost associated with specialized drilling equipment to advance the caisson holes into the bedrock. 	<ul style="list-style-type: none"> Specialized drilling equipment will be required to socket caissons into bedrock.
Spread Footings founded on and Dowelled into Bedrock	2	<ul style="list-style-type: none"> Relative ease of construction No bedrock coring and/or churn drilling required No post-construction settlement Soil cover for frost protection is not required for footings on bedrock Very high geotechnical axial resistance available 	<ul style="list-style-type: none"> Larger excavation of overburden is required producing a larger volume of excavation spoils Up to 3.1 m excavation required to found on bedrock. Larger volume of mass concrete may be required to achieve level footing; Dowels may be required to anchor spread footings (due to structural considerations) Groundwater control may be required. 	<ul style="list-style-type: none"> Relatively lower cost in comparison to caissons socketed into bedrock; Additional cost required for the disposal of larger volume of excavation spoils; and, Additional costs required for installation of dowels into the bedrock. 	<ul style="list-style-type: none"> Risk that additional excavation and mass concrete may be required if bedrock is encountered below the design founding elevation during construction; and, Must ensure foundation base is properly prepared for concrete placement.

Prepared By: DPM

Reviewed By: JMAC



PLAN



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

CONT No.
GWP No. 5402-05-00

HIGHWAY 69

SITE LOCATION PLAN

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN
NOT TO SCALE

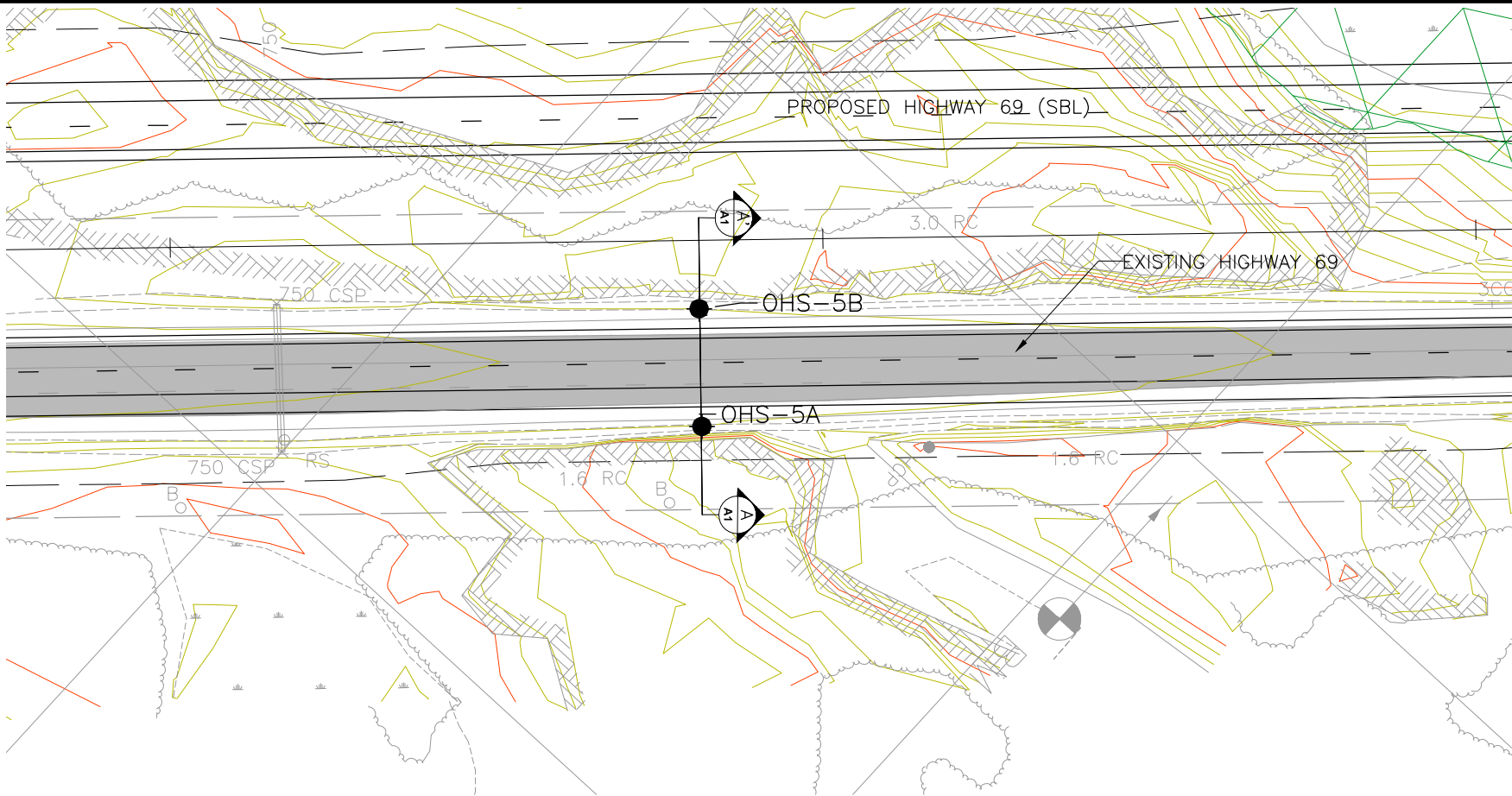
REFERENCE			
Base Data – MNR NRVS, obtained 2004, CANMAP v2006.4			
Produced by Golder Associates Ltd under licence from			
Ontario Ministry of Natural Resources@Queens Printer 2008			
Datum : NAD 83 Projection : MTM Zone 10			

NO.	DATE	BY	REVISION
Geocres No. 41H-162			
HWY. 69		PROJECT NO. 14-1181-0014	
SUBMD. DM		CHKD. DM	DIST.
DRAWN: STB		DATE: May 2016	SITE:
		APPD. JMAC	DWG. 1



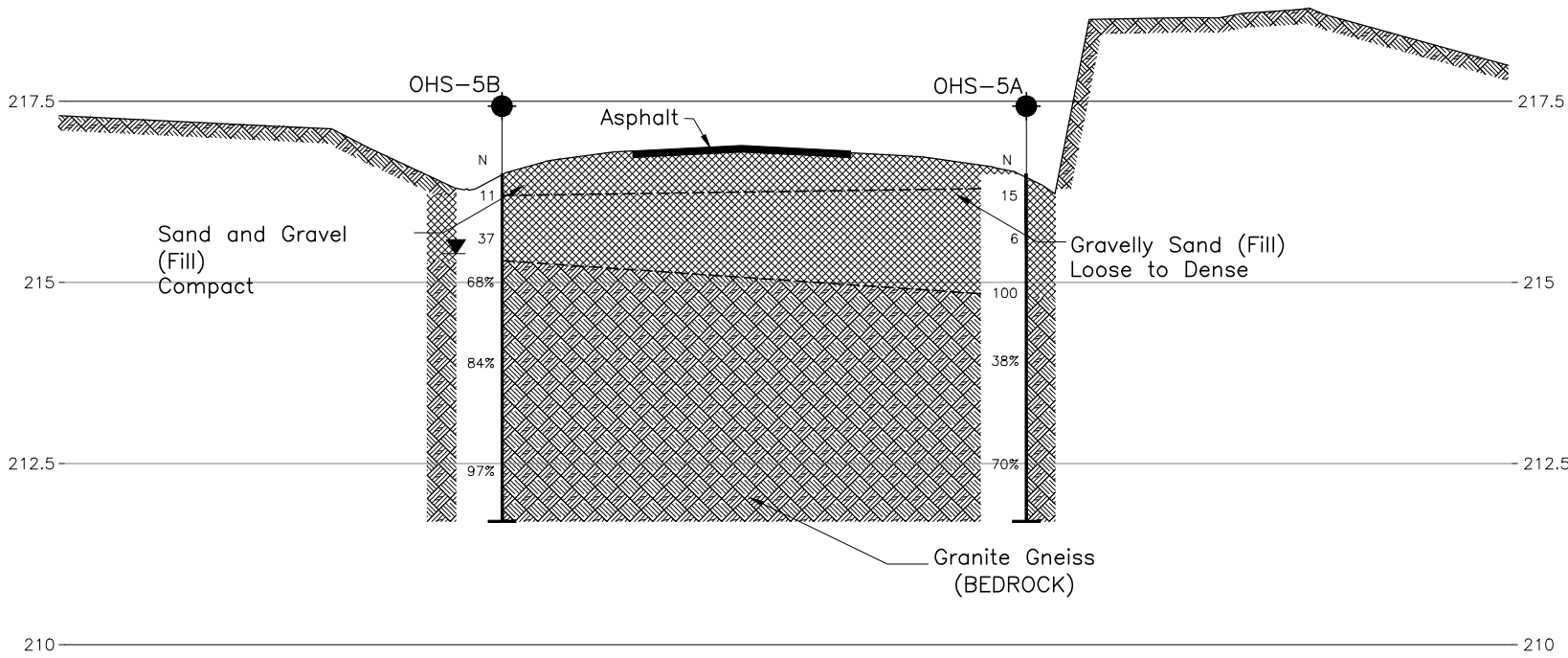
APPENDIX A

Overhead Sign 5 - STA 11+281 Shawanaga Twp



PLAN

SCALE
0 5 10 25 m
1:1000



A-A'
A1

CROSS-SECTION A-A' STA. 11+281
HIGHWAY 69 (NBL)

HORIZ. SCALE 1:500
0 5 m
2.5 0 2.5 m
VERT. SCALE 1:250

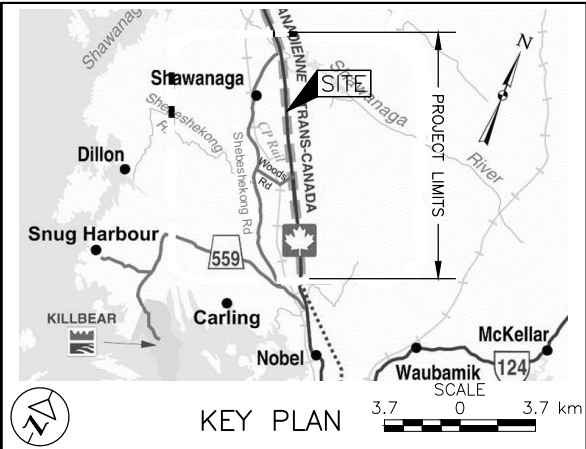
METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 5402-05-00

HIGHWAY 69 (NBL)
OVERHEAD SIGN 5, STA 11+281
BOREHOLE LOCATIONS AND SOIL STRATA



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



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)
- ▼ Recorded WL in open borehole

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
OHS5-A	216.5	5041344.9	248047.5
OHS5-B	216.5	5041332.4	248034.4

NOTES

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

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

The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

REFERENCE

Base plans provided in digital format by MRC, drawing file 5271XB01.DWG, 5271-XPD-ARCHIPELAGO.dwg, 5271-XPD-Carling.dwg, 5271-XPD-SHAWANAGA.dwg, PR # 5377-02-00-PR-1.dwg, received October 1, 2007 and h6878_PHASE1_XA1, h6878_PHASE1_XN1.dwg, received January 21, 2009 and h6878_PHASE1_XN1.dwg, received September 19, 2011.



NO.	DATE	BY	REVISION
Geocres No. 41H-162			
HWY. 69	PROJECT NO. 14-1181-0014		DIST.
SUBM'D. DM	CHKD. DM	DATE: May 2016	SITE:
DRAWN: STB	CHKD. JH	APPD. JMAC	DWG. A1

PROJECT		1411810014 (14000)		RECORD OF BOREHOLE No OHS5-A		SHEET 1 OF 1		METRIC									
G.W.P.		5402-05-00		LOCATION		N 5041344.9 ; E 248047.5		ORIGINATED BY									
DIST		HWY 69		BOREHOLE TYPE		NW Casign and NQ Coring		COMPILED BY									
DATUM		Geodetic		DATE		December 14, 2015		CHECKED BY									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
216.5	GROUND SURFACE																
0.0	Sand and gravel, trace to some silt, some asphalt coated particles (FILL)		1	SS	15												49 46 (5)
0.3	Compact Black to brown Moist		2	SS	6												
214.7	Gravelly sand, trace silt, trace cobbles inferred from drilling progress (FILL)		3	SS	100/0.23												
1.8	Loose to compact Black to brown Moist																
	GNEISS (BEDROCK)																
	Bedrock cored from 1.8 m depth to 4.8 m depth.		1	RC	REC 89%												RQD = 38%
	For coring details see Record of Drillhole OHS5-A.																
			2	RC	REC 90%												RQD = 70%
211.7	End of Borehole																
4.8	NOTE: 1. Water level not recorded due to introduction of drilling water.																

GTA-MTO 001 T:\PROJECTS\2014\14-1181-0014 (NORTHEASTERN REGION RETAINER)\LOG\14-1181-0014_14000_MTO.GPJ GAL-GTA.GDT 2-8-16 STB

PROJECT		1411810014 (14000)		RECORD OF BOREHOLE No OHS5-B		SHEET 1 OF 1		METRIC								
G.W.P.		5402-05-00		LOCATION		N 5041332.4 ; E 248034.4		ORIGINATED BY								
DIST		HWY 69		BOREHOLE TYPE		NW Casign and NQ Coring		COMPILED BY								
DATUM		Geodetic		DATE		December 14, 2015		CHECKED BY								
								JH								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
216.5	GROUND SURFACE															
0.0	Sand and gravel, some silt, some asphalt coated particles (FILL)		1	SS	11											40 52 (8)
216.2	Compact Black to brown Moist		2	SS	37											
0.3	Gravelly sand, trace silt, trace cobbles inferred from drilling progress (FILL)		1	RC	REC 87%											RQD = 68%
215.3	Compact to dense Brown Wet		2	RC	REC 100%											RQD = 84%
1.2	GNEISS (BEDROCK)		3	RC	REC 100%											RQD = 97%
	Bedrock cored from 1.2 m depth to 4.9 m depth.															
	For coring details see Record of Drillhole OHS5-B.															
211.6	End of Borehole															
4.9	NOTE: 1. Water Level in open borehole measured at a depth of 1.1 m below ground surface (Elev. 215.4 m) prior to introduction of drill water.															

[illegible]

SHEET 1 OF 1

DATUM: Geodetic

DRILLING CONTRACTOR: Landcore Drilling

GTA-RCK 004 T:\PROJECTS\2014\14-1181-0014 (NORTHEASTERN REGION RETAINER)\LOG\ SUPERSEDED\14-1181-0014 14000 MTO RCK.GPJ GAL-MISS.GDT 2-8-16

CHECKED: JH

TABLE A1

POINT LOAD TEST ON ROCK SAMPLES

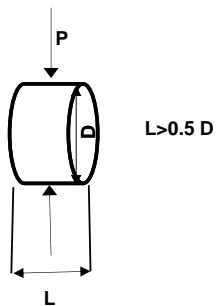
PROJECT NO. 14-1181-0014(14000)
 TITLE MTO\5013-E-0034\NE Region Retainer
 DATE January, 2016

Borehole Number	Run Number	Sample Depth (m)	Sample Elevation (m)	Bedrock Description	Test Type	Core Length (mm)	Core Diameter (mm)	Core ⁽¹⁾ Is Axial (MPa)	Is Diametral (MPa)	Is (50mm) (MPa)
OHS5-A	1	2.50	214.0	Granite Gneiss	Axial	27.45	47.63	3.702	-	3.378
OHS5-A	1	2.92	213.6	Granite Gneiss	Axial	28.55	47.63	6.943	-	6.392
OHS5-A	1	2.92	213.6	Granite Gneiss	Diametral	104.75	44.16	-	5.250	4.965
OHS5-A	2	4.29	212.2	Granite Gneiss	Axial	25.13	47.63	7.228	-	6.467
OHS5-A	2	4.29	212.2	Granite Gneiss	Diametral	62.95	43.93	-	4.028	3.800
OHS5-B	1	1.47	215.0	Granite Gneiss	Axial	26.13	47.64	6.520	-	5.884
OHS5-B	1	1.47	215.0	Granite Gneiss	Diametral	71.34	44.41	-	2.211	2.096
OHS5-B	2	2.81	213.7	Granite Gneiss	Axial	25.02	47.66	4.746	-	4.242
OHS5-B	2	2.81	213.7	Granite Gneiss	Diametral	75.13	45.36	-	3.502	3.352
OHS5-B	3	4.14	212.4	Granite Gneiss	Axial	24.20	47.65	5.191	-	4.606
OHS5-B	3	4.14	212.4	Granite Gneiss	Diametral	68.42	42.47	-	4.341	4.034

⁽¹⁾ Actual distance between point load cones at time of failure.

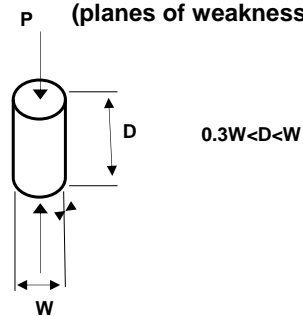
DIAMETRAL SPECIMEN SHAPE REQUIREMENTS

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



AXIAL SPECIMEN SHAPE REQUIREMENTS

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

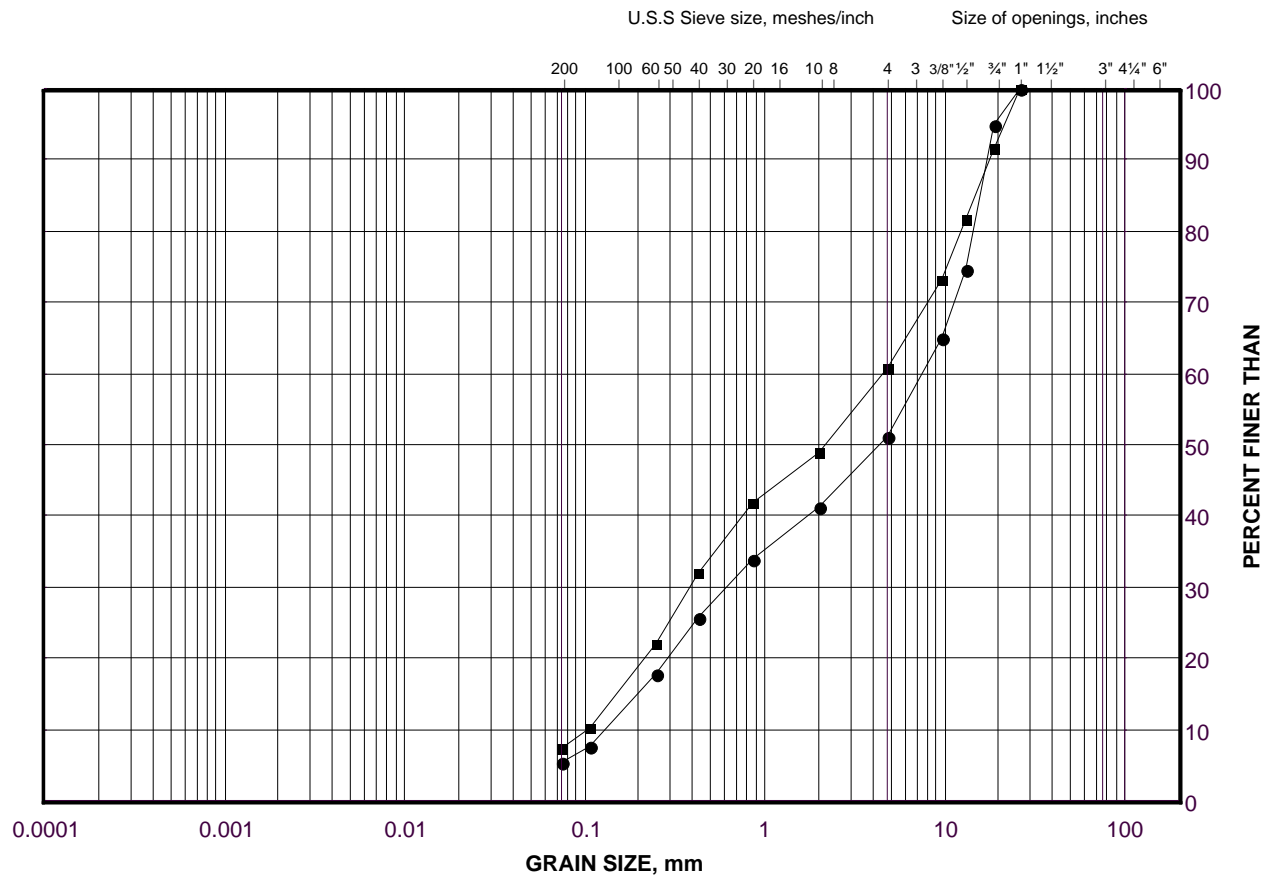


Compiled by: DPM
 Reviewed by: JMAC

GRAIN SIZE DISTRIBUTION

Sand and Gravel (Fill)

FIGURE A1



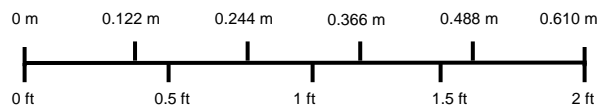
SILT AND CLAY SIZES		FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED		SAND SIZE			GRAVEL SIZE		SIZE


LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	OHS5-A	1	215.9
■	OHS5-B	1	215.9



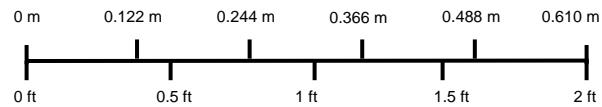
Borehole OHS-5A
Elevation 214.7 m to 211.7 m




PROJECT		OVERHEAD SIGNS HIGHWAY 69 FOUR-LANNING G.W.P 5402-05-00			
TITLE		OHS-5A BEDROCK CORE PHOTOGRAPH			
		PROJECT No. 14-1181-0014-14000		FILE No. ----	
		DESIGN	DPM	JAN 2016	SCALE AS SHOWN
		CADD	--		REV.
		CHECK	JH	JAN 2016	FIGURE A2
		REVIEW	JMAC	Feb 2016	



Borehole OHS-5B
Elevation 215.3 m to 211.6 m

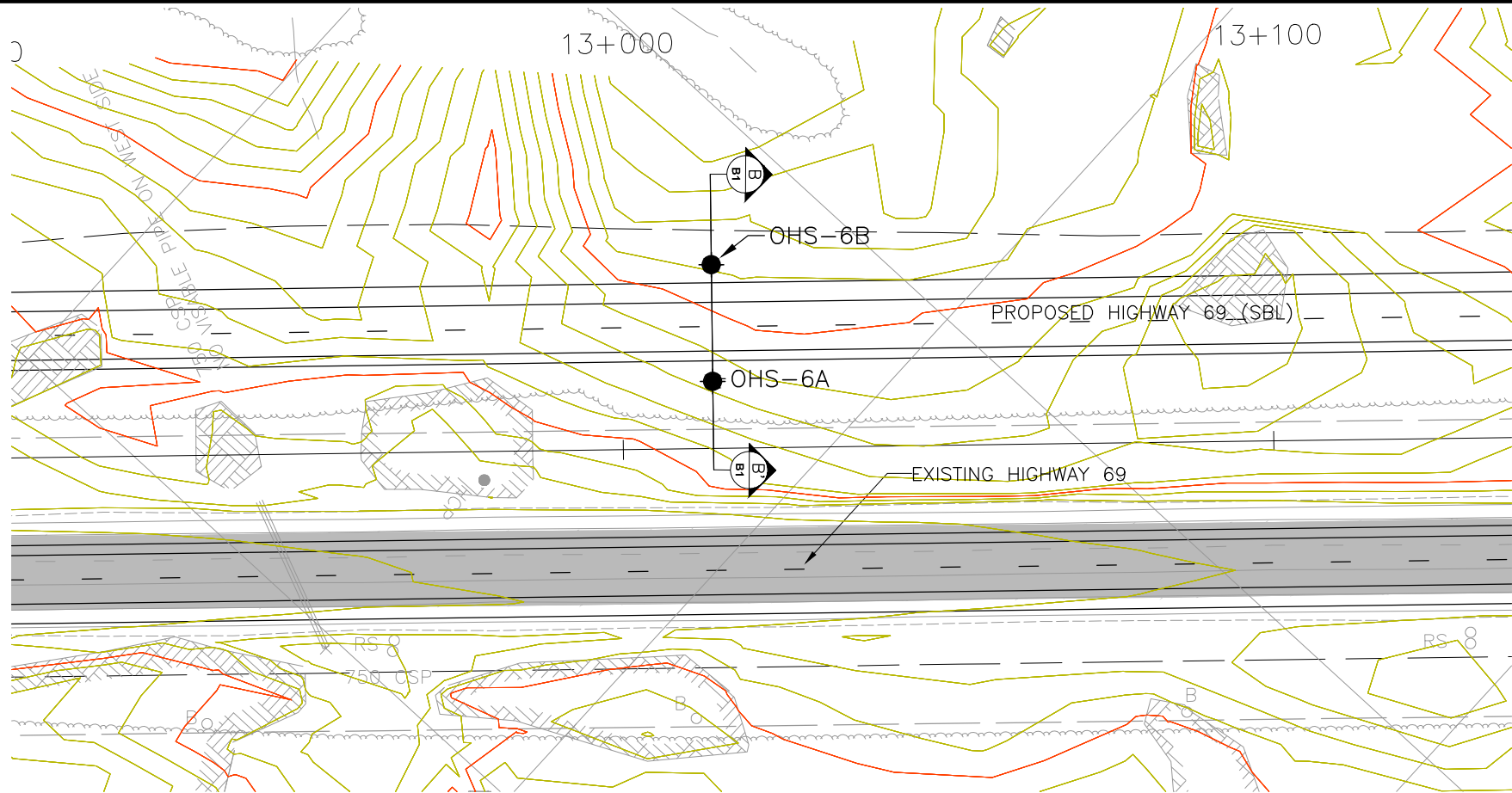


PROJECT		OVERHEAD SIGNS HIGHWAY 69 FOUR-LANNING G.W.P 5402-05-00			
TITLE		OHS-5B BEDROCK CORE PHOTOGRAPH			
		PROJECT No. 14-1181-0014-14000		FILE No. ----	
		DESIGN	DPM	JAN 2016	SCALE AS SHOWN
		CADD	--		REV.
		CHECK	JH	JAN 2016	FIGURE A3
		REVIEW	JMAC	Feb 2016	



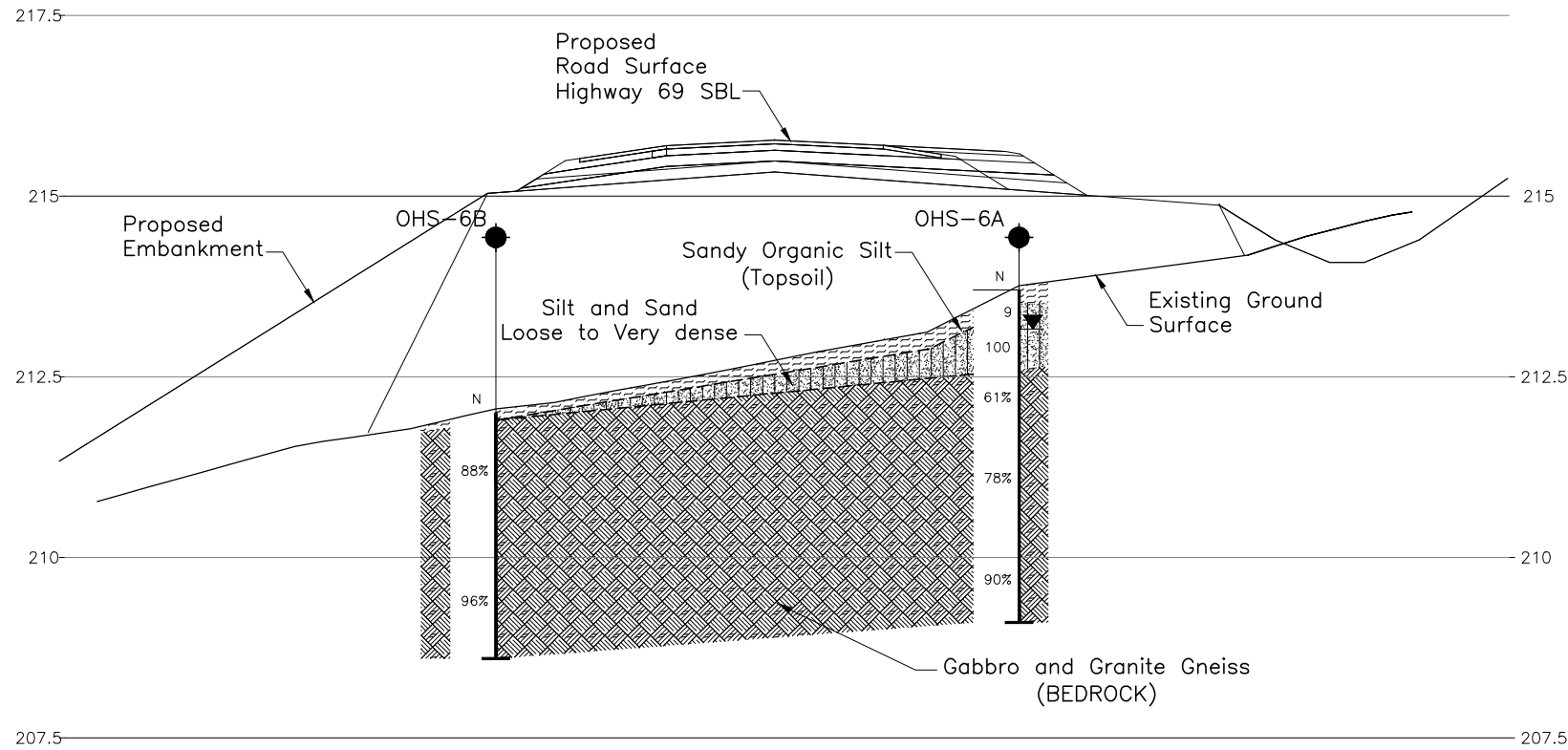
APPENDIX B

Overhead Sign 6 - STA 13+014 Shawanaga Twp



PLAN

SCALE
0 5 10 20 m
1:1000



CROSS-SECTION B-B' STA. 13+014
HIGHWAY 69 (SBL)

HORIZ. SCALE 1:500
5 0 5 m
2.5 0 2.5 m
VERT. SCALE 1:250

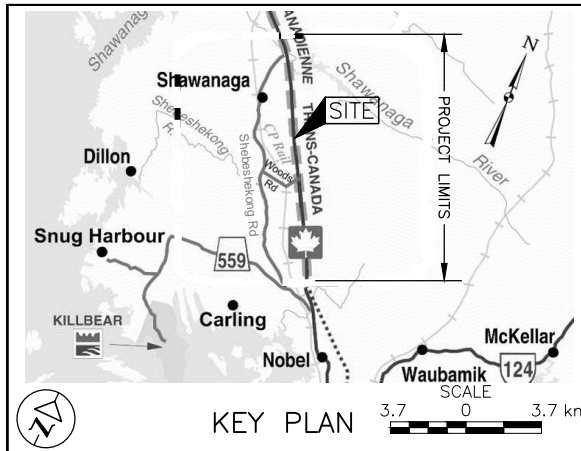
METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 5402-05-00

HIGHWAY 69 (SBL)
OVERHEAD SIGN 6, STA 13+014
BOREHOLE LOCATIONS AND SOIL STRATA



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



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)
- ▼ Recorded WL in open borehole

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
OHS6-A	213.7	5042579.9	246831.4
OHS6-B	212.0	5042567.6	246818.3

NOTES

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

The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

REFERENCE

Base plans provided in digital format by MRC, drawing file 5271XB01.DWG, 5271-XPD-ARCHIPELAGO.dwg, 5271-XPD-Carling.dwg, 5271-XPD-SHAWANAGA.dwg, PR # 5377-02-00-PR-1.dwg, received October 1, 2007 and h6878_PHASE1_XA1, h6878_PHASE1_XN1.dwg, received January 21, 2009 and h6878_PHASE1_XN1.dwg, received September 19, 2011.



NO.	DATE	BY	REVISION
Geocres No. 41H-162			
HWY. 69	PROJECT NO. 14-1181-0014	DIST.	
SUBM'D. DM	CHKD. DM	DATE: May 2016	SITE:
DRAWN: STB	CHKD. JH	APPD. JMAC	DWG. B1

PROJECT		1411810014 (14000)		RECORD OF BOREHOLE No OHS6-A		SHEET 1 OF 1		METRIC					
G.W.P.		5402-05-00		LOCATION		N 5042579.9 ; E 246831.4		ORIGINATED BY					
DIST		HWY 69		BOREHOLE TYPE		NW Casign and NQ Coring		COMPILED BY					
DATUM		Geodetic		DATE		December 15, 2015		CHECKED BY					
								JH					
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT		UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)			
213.7	GROUND SURFACE												
0.0	Sandy ORGANIC SILT, some fibrous peat, trace gravel (TOPSOIL)		1A										
0.2	Black to reddish brown Moist		1B	SS	9								
212.6	SILT and SAND, some gravel, trace fibrous peat		2	SS	100/0.18								
1.1	Loose to very dense Brown to reddish brown Moist to wet		1	RC	REC 94%								
	GABBRO and GNEISS (BEDROCK)												
	Bedrock cored from 1.1 m depth to 4.6 m depth.		2	RC	REC 95%								
	For coring details see Record of Drillhole OHS6-A.												
			3	RC	REC 98%								
209.1	End of Borehole												
4.6	NOTE: 1. Water level in open borehole measured at a depth of 0.6 m below ground surface (Elev. 213.1 m) prior to introduction of drilling water.												

GTA-MTO 001 T:\PROJECTS\2014\14-1181-0014 (NORTHEASTERN REGION RETAINER)\LOG\14-1181-0014_14000_MTO.GPJ GAL-GTA.GDT 2-8-16 STB

PROJECT 1411810014 (14000)		RECORD OF BOREHOLE No OHS6-B		SHEET 1 OF 1		METRIC											
G.W.P. 5402-05-00		LOCATION N 5042567.6 ; E 246818.3		ORIGINATED BY DM													
DIST _____ HWY 69		BOREHOLE TYPE NW Casign and NQ Coring		COMPILED BY SB													
DATUM Geodetic		DATE December 16, 2015		CHECKED BY JH													
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W _p — W — W _L WATER CONTENT (%)			γ kN/m ³	GR SA SI CL
							20 40 60 80 100	20 40 60 80 100	10 20 30								
212.0	GROUND SURFACE																
0.0	Sandy FIBROUS PEAT, some silt (TOPSOIL) Black Moist																
	GNEISS (BEDROCK)																
	Bedrock cored from 0.1 m depth to 3.4 m depth.																
	For coring details see Record of Drillhole OHS6-B.																
			1	RC	REC 96%		211										RQD = 88%
			2	RC	REC 100%		210										RQD = 96%
208.6	End of Borehole						209										
3.4	NOTE: 1. Groundwater not encountered in borehole.																

GTA-MTO 001 T:\PROJECTS\2014\14-1181-0014 (NORTHEASTERN REGION RETAINER)\LOG\14-1181-0014_14000_MTO.GPJ GAL-GTA.GDT 2-8-16 STB

[illegible]

PROJECT: 1411810014 (14000)

RECORD OF DRILLHOLE: OHS6-B

SHEET 1 OF 1

LOCATION: N 5042567.6 ; E 246818.3

DRILLING DATE: December 16, 2015

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55

DRILLING CONTRACTOR: Landcore Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	PENETRATION RATE min(m)	FLUSH	RECOVERY TOTAL CORE %	SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3 m	B Angle	DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	HYDRAULIC CONDUCTIVITY K, cm/sec	Diametral Point Load Index (MPa)	RMC -Q AVG.	NOTES WATER LEVELS INSTRUMENTATION
		TOP OF BEDROCK		211.90																
		GRANITE GNEISS Strong to very strong Moderately weathered to fresh Grey with pink banding Crystalline Medium to coarse Non-porous		0.10									FO							
1					1								JN						1.5	Axial Diametral
													JN							
													JN	3	1	4				
													JN						3.5	Axial Diametral
													JN	3	1	4				
													JN							
													JN	2	0.75	4				
													JN							
													JN	3	1	4				
													JN							
													JN						2.7	Axial Diametral
													JN							
3																				
		End of Drillhole		208.62																
		NOTE: 1. Groundwater not encountered in drillhole.		3.38																
4																				
5																				

DEPTH SCALE

1 : 25



LOGGED: DM

CHECKED: JH

GTA-RCK 004 T:\PROJECTS\2014\14-1181-0014 (NORTHEASTERN REGION RETAINER)\LOG_SUPERSEDED\14-1181-0014_14000_MTO_RCK.GPJ GAL-MISS.GDT 2-8-16

TABLE B1

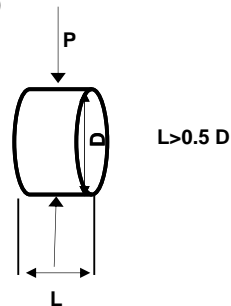
POINT LOAD TEST ON ROCK SAMPLES

Borehole Number	Run Number	Sample Depth (m)	Sample Elevation (m)	Bedrock Description	Test Type	Core Length (mm)	Core Diameter (mm)	Core ⁽¹⁾ Axial (MPa)	Is Diametral (MPa)	Is (50mm) (MPa)
OHS6-A	7	1.25	212.5	Gabbro	Axial	23.12	47.66	1.581	-	1.388
OHS6-A	7	1.25	212.5	Gabbro	Diametral	74.63	41.77	-	3.380	3.117
OHS6-A	8	1.66	212.0	Granite Gneiss	Axial	23.63	47.62	7.676	-	6.772
OHS6-A	8	1.66	212.0	Granite Gneiss	Diametral	92.17	43.50	-	2.074	1.948
OHS6-A	9	3.54	210.2	Granite Gneiss	Axial	21.03	47.67	10.368	-	8.913
OHS6-A	9	3.54	210.2	Granite Gneiss	Diametral	80.72	43.30	-	8.919	8.360
OHS6-B	10	0.39	211.6	Granite Gneiss	Axial	20.91	47.08	13.433	-	11.500
OHS6-B	10	0.39	211.6	Granite Gneiss	Diametral	66.22	44.70	-	7.487	7.119
OHS6-B	11	1.11	210.9	Granite Gneiss	Axial	25.20	47.45	9.515	-	8.510
OHS6-B	11	1.11	210.9	Granite Gneiss	Diametral	53.81	42.75	-	6.712	6.255
OHS6-B	12	2.44	209.6	Granite Gneiss	Axial	21.73	47.45	14.717	-	12.732
OHS6-B	12	2.56	209.4	Granite Gneiss	Diametral	75.86	42.92	-	3.366	3.142

⁽¹⁾ Actual distance between point load cones at time of failure.

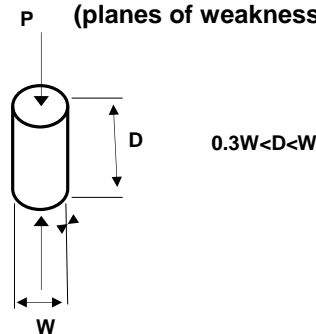
DIAMETRAL SPECIMEN SHAPE REQUIREMENTS

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



AXIAL SPECIMEN SHAPE REQUIREMENTS

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

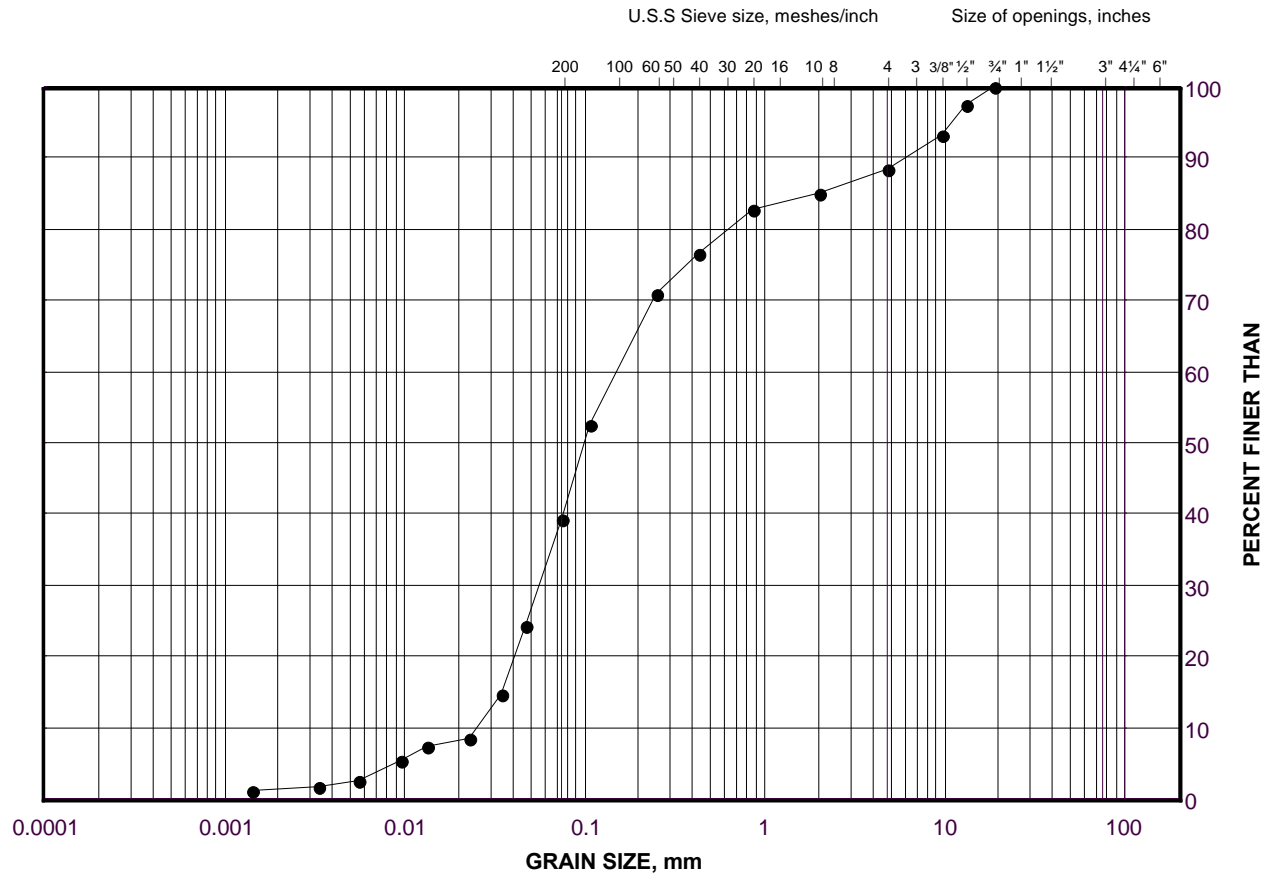


Compiled by: DPM
Reviewed by: JMAC

GRAIN SIZE DISTRIBUTION

Silt and Sand

FIGURE B1



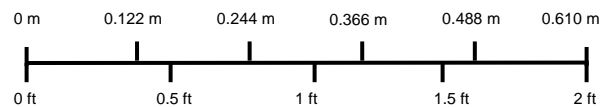
SILT AND CLAY SIZES		FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED		SAND SIZE			GRAVEL SIZE		SIZE


LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	OHS6-A	2	212.9



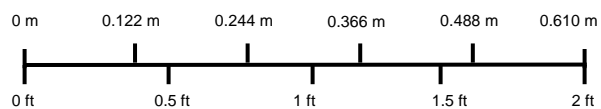
Borehole OHS-6A
Elevation 212.6 m to 209.1 m




PROJECT		OVERHEAD SIGNS HIGHWAY 69 FOUR-LANNING G.W.P 5402-05-00			
TITLE		OHS-6A BEDROCK CORE PHOTOGRAPH			
		PROJECT No. 14-1181-0014-14000		FILE No. ----	
		DESIGN	DPM	JAN 2016	SCALE AS SHOWN
		CADD	--		REV.
		CHECK	JH	JAN 2016	FIGURE B2
		REVIEW	JMAC	FEB 2016	



Borehole OHS-6B
Elevation 211.9 m to 208.6 m



PROJECT		OVERHEAD SIGNS HIGHWAY 69 FOUR-LANNING G.W.P 5402-05-00			
TITLE		OHS-6B BEDROCK CORE PHOTOGRAPH			
		PROJECT No. 14-1181-0014-14000		FILE No. ----	
		DESIGN	DPM	JAN 2016	SCALE AS SHOWN
		CADD	--		REV.
		CHECK	JH	JAN 2016	FIGURE B3
		REVIEW	JMAC	FEB 2016	



APPENDIX C

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 footings 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

Dowels Into Rock – Item No.

Non-Standard Special Provision

Scope of Work

Work under this item is for the placement and field testing of dowels into rock.

Construction

Dowels into rock shall be constructed in accordance with OPSS.PROV 904 Concrete Structures. All reinforcing steel supplied shall be in accordance with OPSS.PROV 1440 (Steel Reinforcement for Concrete) (dowel bars conforming to CSA Standard CSAG30.18, Grade 400).

For dowels into rock, holes shall be drilled to the required depth and size. Hole diameter shall be two times the nominal diameter of the dowel. Each hole shall be cleaned out, grouted and the dowel set in place. Grout shall be of the same strength as the footing concrete, or at least 25 MPa at 28 days.

If the hole contains water, the Contractor shall remove the water, otherwise, a tremie procedure shall be used to completely fill the hole with grout. The dowel shall be forced into the hole after the grout has been placed and while it is still fresh.

Rock Dowel Testing

All proposed testing procedures shall be in general conformance with ASTM D3689-90 and ASTM D1143M-07. Field testing must be carried out in the presence of, and the results reviewed and approved by, the Contract Administrator.

Performance Tests

The following table summarizes the number of dowels into rock where performance testing shall be carried out to confirm that the design load of the rock dowels can be achieved. The Contract Administrator will select the rock dowels to be tested.

Structure	Number of Dowels for Performance Testing
Overhead Sign OHS-5	2 per spread footing
Overhead Sign OHS-6	2 per spread footing

Performance test shall be by axial tensioning using a hydraulic jack with a capacity of at least 1.5 times the ultimate strength of the dowels.

Rock dowels shall be loaded and unloaded in 3 cycles and measurements of the displacement of the dowel shall be carried out at each load increment (step) in accordance with the following schedule:

Cycle-Step	1-1	1-2	1-3	2-1	2-2	2-3	2-4
% Design Load	50	75	25	50	75	100	25

Cycle-Step	3-1	3-2	3-3	3-4	3-5
% Design Load	50	75	100	110	25

The design load shall be taken as 360 kN for 35M dowels, 252 kN for 30M dowels, 180 kN for 25M dowels and 108 kN for 20M dowels.

Displacement measurements shall be carried out at each load increment using calibrated displacement gauges capable of measuring movements of 0.025 mm. Measurements shall be referenced to an independent fixed referenced point.

Rock dowels which fail to meet the acceptance criteria shall be replaced at the Contractor's expense and re-tested. If a rock dowel fails, 3 additional rock dowels shall be tested at the same spread footing as directed by the Contract Administrator.

Acceptance criteria for the rock dowels will be in accordance with the Post-Tensioning Institute (1985) as follows:

- The dowels are acceptable if the total elastic movement is greater than 80 percent of the theoretical elastic elongation of the free stressing length and is less than the theoretical elongation of the free stressing length plus 50 percent of the bond length.

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

Control of Overburden Soils – Item No.

Non-Standard Special Provision

Scope of Work

Excavations for the overhead sign foundations will be advanced through cohesionless soils, which should be expected to be unstable below the groundwater level. Where cohesionless soil deposits are encountered, appropriate construction equipment and procedures will be required to minimize ground loss during excavation and concrete placement.

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

As a global, employee-owned organisation with over 50 years of experience, Golder Associates is driven by our purpose to engineer earth's development while preserving earth's integrity. We deliver solutions that help our clients achieve their sustainable development goals by providing a wide range of independent consulting, design and construction services in our specialist areas of earth, environment and energy.

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