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FOUNDATION INVESTIGATION AND DESIGN REPORT

**SITE 9 ROAD UNDERPASS STRUCTURE, SITE NO. 44-444
HIGHWAY 69 FOUR-LANING
FROM 0.4 KM NORTH OF HIGHWAY 7182
(SHEBESHEKONG ROAD) NORTHERLY 11 KM
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5403-05-00, WP 5189-06-01**

Submitted to:

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Thornhill, Ontario
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REPORT



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PART A

**FOUNDATION INVESTIGATION REPORT
SITE 9 ROAD UNDERPASS STRUCTURE, SITE NO. 44-444
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FROM 0.4 KM NORTH OF HIGHWAY 7182
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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by MMM Group (MMM) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the detail design of the proposed Site 9 Road Underpass structure over the new four-laned Highway 69. The proposed work is part of the detail design of the four-laning of Highway 69 from 0.4 km north of Highway 7182 (Shebeshekong Road) northerly for a total distance of 11 km. The general location of this section of the Highway 69 four-laning alignment is shown on the Key Plan on Drawing 1.

The terms of reference and scope of work for the foundation investigation are outlined in MTO's Request for Proposal dated March 28, 2007. Golder's proposal (P7-1191-0020, dated April 24, 2007) for foundation engineering services associated with the structure is contained in Section 6.8 of MMM's Technical Proposal that forms part of the Consultant's Agreement (Purchase Order Number 5006-E-0031) for this project. The work was carried out in accordance with Golder's Supplemental Specialty Quality Control Plan for this project dated September 2007. The final General Arrangement (GA) Drawing, dated July 2010, for the proposed Site 9 Road Underpass structure was provided to Golder by MMM on July 17, 2010.

This report addresses the investigation carried out for the Site 9 Road Underpass structure and the associated approach embankments and Retained Soil System (RSS) walls. Separate reports will be submitted detailing the foundation investigations for the related swamp and pond crossings, culverts and other bridge structures for the project.

The purpose of this investigation is to establish the subsurface conditions at the proposed structure, including the associated approach embankments, by borehole drilling, rock coring and laboratory testing on selected samples. The area of the investigation is shown in plan on Drawing 1.

The investigation was supplemented with information contained in the available existing data supplied by the MTO, specifically:

- Preliminary Foundation and Design Report – Structural Areas, Foundation Investigation – 2, Highway 69 Route Selection Study, 3.5 km North of Highway 559 to 3.8 km North of Highway 522, GWP 5377-02-00, Highway 69, GEOCRE No. 41H-57, July, 2006, by AMEC Earth and Environmental (AMEC).

2.0 SITE DESCRIPTION

The proposed Site 9 Road Underpass structure is located at the intersection of Dumont Road and the existing Highway 69, approximately 900 m north of Highway 7182 (Shebeshekong Road). The existing Highway 69, which will become the future Southbound Lane (SBL) of the proposed four-lane highway, runs generally in a southeast-northwest direction.

In general, the topography in the area of the overall project limits consists of rolling terrain including densely treed areas and numerous bedrock outcrops separated by low-lying swamps. The proposed structure and associated approach embankments are to be situated on a relatively flat, moderately treed area. The ground surface at the borehole locations advanced within the limits of the proposed structure and approach embankment areas is between about Elevation 209.8 m and Elevation 213.3 m, referenced to Geodetic datum, and is gently sloping downward from east to west.



3.0 INVESTIGATION PROCEDURES

The fieldwork for the Site 9 Road Underpass structure investigation was carried out between January 6 and 23, 2009, during which time a total of eight (8) boreholes and three (3) Dynamic Cone Penetration Tests (DCPTs) were advanced: two (2) boreholes were advanced at the west abutment; three (3) boreholes and three (3) DCPTs were advanced at the centre pier; one (1) borehole was advanced at the east abutment; and two (2) boreholes were advanced at the approach embankments (i.e. one borehole at each approach). The boreholes are designated as Boreholes B1-1 to B1-8 and the DCPTs are designated as B1-DC2, B1-DC3 and B1-DC3a. Two (2) probeholes were advanced in July 2009 for the proposed RSS wall. In addition, one (1) borehole (S16-2) and a DCPT (S16-DC2) from the swamp investigations for this area were included to supplement the information for the RSS walls. The details are shown on the Record of Borehole and Drillhole, Record of Penetration Test and Record of Geotechnical Probehole sheets in Appendix A and the locations are shown on Drawing 1.

The locations of two (2) boreholes and one (1) DCPT advanced by AMEC (AMEC, 2006) during the preliminary design phase of the project are also shown on Drawing 1. Borehole ST-3 and ST-4 and DCPT ST-3, are located at the proposed east and west abutments. Copies of these Borehole and Penetration Test Sheets are provided in Appendix B.

The current field investigation was carried out using a track mounted CME-55 drill rig supplied by Landcore Drilling Inc. of Sudbury, Ontario. The boreholes were advanced through the overburden using either 108 mm inside diameter continuous flight hollow stem augers or NW casing and wash boring. Soil samples were obtained at intervals of depth of about 0.75 m, 1.5 m and 3.0 m, using a 50 mm outside diameter split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586-08a). Samples of the bedrock were obtained using an NQ size rock core barrel. All boreholes were backfilled with bentonite upon completion in accordance with Ontario Regulation (O.Reg.) 903 (as amended by O.Reg.372).

At least one borehole at each foundation element was advanced to the bedrock surface while the boreholes at the approach embankments were advanced to a depth approximately equal to the height of the proposed embankment. The depths of the boreholes range from about 2.8 m to 26.8 m below existing ground surface, including coring of between about 3.0 m and 3.9 m of bedrock at Boreholes B1-1 to B1-4.

The groundwater conditions in the open boreholes were observed during the drilling operations and piezometers were installed in Boreholes B1-1 and B1-4 to permit monitoring of the groundwater level at these locations. The piezometers consisted of 51 mm diameter PVC pipe, with a 1.5 m long slotted screen sealed at a selected depth within the boreholes. The non-instrumented boreholes and the annulus surrounding the piezometer pipe above the sand pack were backfilled to the surface with bentonite pellets/grout. The piezometer installation details and water level readings are described on the Record of Borehole sheets in Appendix A.

The fieldwork was observed by members of our engineering and technical staff, who located the boreholes in the field based on the survey carried out by MMM, 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 Sudbury geotechnical laboratory where the samples underwent further visual examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate. Classification testing (water content, Atterberg limits and grain size distribution) was carried out on selected samples. Strength testing (uniaxial compression and point load index) was carried out on selected specimens of the rock core.



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MMM surveyed the location of eight (8) boreholes at the site in December 2008, prior to drilling. Several boreholes were relocated from the original staked location due to uneven ground or overhead or underground utilities. Where boreholes were relocated from their original staked location, Golder laid out the new borehole location and surveyed the ground surface at the new location referencing the original staked location. The borehole locations shown on Drawing 1 and summarized below are positioned relative to MTM NAD 83 northing and easting coordinates and the ground surface elevations are referenced to Geodetic datum.

| Borehole | Location (m) | | Ground Surface Elevation (m) | Borehole Depth (m) |
|----------|--------------|----------|------------------------------|--------------------|
| | Northing | Easting | | |
| B1-1 | 5046414.4 | 242674.9 | 209.9 | 26.8 |
| B1-2 | 5046431.3 | 242708.4 | 211.0 | 6.0 |
| B1-3 | 5046437.7 | 242699.1 | 212.3 | 8.6 |
| B1-4 | 5046448.7 | 242742.1 | 213.3 | 15.8 |
| B1-5 | 5046410.0 | 242657.2 | 209.8 | 9.8 |
| B1-6 | 5046462.4 | 242760.6 | 213.3 | 11.3 |
| B1-7 | 5046420.2 | 242665.2 | 209.9 | 16.8 |
| B1-8 | 5046434.2 | 242704.3 | 211.4 | 2.8 |

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

As delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, 1984¹), 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 surficial soils in this area consist of very shallow deposits of sand, silt and clay overlying metamorphic bedrock and numerous bare knobs and ridges of bedrock are present throughout the area. Localised low-lying swampy areas, containing peat and/or organic soils overlying 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 4*². Deposition of Paleozoic strata and later erosion during glaciation left behind these Precambrian rocks.

¹ 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.

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



4.2 Subsurface Conditions

The detailed subsurface soil 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 given on the attached Record of Borehole, Drillhole and Penetration Test sheets and Probehole in Appendix A. The results of the laboratory tests carried out on selected soil and rock samples are presented in Appendix C. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non continuous sampling, observations of drilling progress and the results of SPTs. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. It should be noted that the interpreted stratigraphy shown on Drawings 1 and 2 is a simplification of the subsurface conditions. Variation in the stratigraphic boundaries between boreholes will exist and is to be expected.

In general, the subsurface conditions in the area of the proposed underpass consist of a surficial layer of topsoil, underlain by a cohesionless deposit of sand to sand and silt. In Boreholes B1-1 and B1-7, a clayey silt seam was encountered at a depth of about 2.7 m within the granular deposit. In Boreholes B1-4 and B1-7, a layer of cobbles and boulders was found below the sand to sand and silt deposit. Gneiss bedrock was encountered in Boreholes B1-1 to B1-4 below the cohesionless deposit. The ground surface at the borehole locations varies between Elevation 209.8 m and Elevation 213.3 m decreasing towards the west.

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

4.2.1 Topsoil/Fill

Approximately 0.1 m to 0.6 m of topsoil was encountered immediately below ground surface in Boreholes B1-2 to B1-8.

A 1.0 m thick layer of brown fill comprising of sand, trace to some gravel, trace silt containing organics was encountered beneath the topsoil in Borehole B1-3 at Elevation 212.2 m. An SPT 'N'-value measured within the fill is 10 blows per 0.3 m of penetration, indicating a compact relative density.

The measured water content on one sample of the fill is about 32 percent.

4.2.2 Sand to Sand and Silt

A cohesionless deposit consisting of sand, silty sand, sand and silt, sandy silt and gravelly sand was encountered in all boreholes below the surficial topsoil/fill layer or from ground surface. In general, this deposit ranges from a sand to sand and silt. The deposit contained trace to some gravel and trace clay.

The top of this deposit was encountered between Elevation 209.2 m and Elevation 213.1 m and the deposit is between 1.9 m and 23.6 m thick, being thickest at the proposed west abutment and thinnest at the median (centre) pier. The lower approximately 4 m of the deposit in Borehole B1-1 and a 0.6 m thick layer at a depth of 4.6 m in Borehole B1-6 is comprised of gravelly sand and gravelly silty sand, respectively.



SPT 'N'-values measured within this deposit typically range from 3 blows to 37 blows per 0.3 m of penetration, indicating a very loose to dense relative density. The portion of this deposit between depths of about 15.2 m (Elevation 194.7 m) and 18.9 m (Elevation 191.0 m) in Borehole B1-1 and the lower portion of the deposit in Borehole B1-7 (located at the west abutment) have SPT 'N'-values greater than 100 blows per 0.3 m of penetration which would typically suggest a very dense relative density; however, in these boreholes, the higher 'N'-values are likely indicative of the presence of cobbles and boulders and/or gravelly layers.

Heaving sands were noted in several boreholes despite a full head of water being maintained inside the augers/casing. Lower SPT 'N'-values were typically recorded below the elevation where heaving sands were encountered, typically between depths of about 4.6 m and 5.6 m.

Difficult casing/auger advancement was noted near the base of the deposit in Boreholes B1-1, B1-3 and B1-5 to B1-8, inferred to be due to the presence of gravelly material or cobbles and boulders.

Grain size analyses were carried out on twenty-five (25) samples of this deposit and the results are presented on Figures C-1 to C-5 in Appendix C.

Measured water contents on samples of the sand to sand and silt deposit range from about 12 percent to 32 percent.

4.2.3 Clayey Silt

In Boreholes B1-1 and B1-7 (located at the proposed west abutment), a 0.1 m to 0.5 m layer of thick clayey silt was encountered at Elevations 207.1 m and 207.2 m, respectively, within the sand to sand and silt deposit.

Atterberg limits testing carried out on two samples of the clayey silt layer indicate liquid limits of about 31 percent and 33 percent and plastic limits of about 18 percent and 19 percent, yielding plasticity indices of about 12 percent and 15 percent. The results of the Atterberg limits testing are shown on the plasticity chart on Figure C-6, and indicate that the stratum is a clayey silt of low plasticity.

Measured water contents on samples of the clayey silt range from about 47 percent to 50 percent.

4.2.4 Cobbles and Boulders

A deposit of cobbles and boulders containing sand and gravel was encountered in Boreholes B1-4 and B1-7. The top of this layer was encountered at Elevation 201.3 m and Elevation 195.3 m and the layer is 0.2 m and 2.2 m thick in Borehole B1-4 and B1-7, respectively.

4.2.5 Bedrock/Refusal

Bedrock was encountered and core samples were recovered from Boreholes B1-1 to B1-4. Based on coring and refusal to auger advance, the depth to the bedrock surface is variable and ranges from 12.2 m (at the proposed east abutment) to 2.1 m (at the proposed centre pier) to 23.6 m (at the proposed west abutment) below ground surface. Between the centre pier and the west abutment (a distance of about 39 m), the bedrock surface



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elevation drops by about 20 m (approximately 2H:1V slope or a dip of approximately 27° from the horizontal). Between the centre pier and the east abutment (a distance of about 39 m), the bedrock surface elevation drops by about 9 m (approximately 4.5H:1V slope or a dip of approximately 13° from the horizontal). The presence of sloping bedrock was confirmed in Borehole B1-8 when auger refusal was encountered and the augers were noted to be sliding to the north. The depth to refusal/bedrock and corresponding refusal/bedrock surface elevation is summarised below.

| Location | Boring No. | Depth to Refusal/Bedrock Surface (m) | Refusal/Bedrock Surface Elevation (m) | Refusal Type |
|------------------------|---|--|--|---|
| Proposed West Approach | B1-5 | N/A | N/A | N/A |
| Proposed West Abutment | B1-1 B1-7 | 23.6 N/A | 186.3 N/A | Bedrock Cored N/A |
| Proposed Centre Pier | B1-2 B1-3 B1-8 B1-DC2 B1-DC3 B1-DC3a | 2.1 5.6 2.8 2.6 4.3 5.9 | 208.9 206.7 208.6 208.4 207.7 206.9 | Bedrock Cored Bedrock Cored Auger Refusal Cone Refusal Cone Refusal Cone Refusal |
| Proposed East Abutment | B1-4 ST-3(DCPT) | 12.2 12.0 | 201.1 201.0 | Bedrock Cored Cone Refusal |
| Proposed East Approach | B1-6 | N/A | N/A | N/A |

N/A: Refusal or bedrock not encountered to a drilled depth of 9.8 m at B1-5, 16.8 m at B1-7 and 11.3 m at B1-6.

Based on the cored bedrock samples, the bedrock generally consists of fresh to slightly weathered, fine to medium grained, pinkish grey gneiss. The Rock Quality Designation (RQD) measured on the core samples is between about 78 percent and 100 percent, indicating a rock mass of good to excellent quality. The Total Core Recovery (TCR) for all samples was 100 percent and the Solid Core Recovery (SCR) of the samples recovered was between 69 percent and 98 percent. In Borehole B1-2, the bedrock core is fractured between Elevations 207.3 m and 207.5 m.

Uniaxial Compressive Strength (UCS) tests carried out on selected samples of the gneiss bedrock obtained from Boreholes B1-1 to B1-4, measured strengths ranging from 64 MPa to 144 MPa, as summarised on Table C-1 in Appendix C.

Point load strength tests were performed on selected samples of the bedrock. Diametral point load strength index values are shown on the Record of Drillhole sheets and in Table C-2 in Appendix C. The diametral point load index (I_{s50}) results from the laboratory tests carried out on core samples of the bedrock range from 2.5 MPa to 10 MPa. These index values correspond to estimated UCS values ranging between 50 MPa and 197 MPa, based on a relationship between I_{s50} and UCS which is given by a correlation factor (K) in accordance with ASTM D5731-08, which varies depending on the size of the core samples and the strength of the rock. For this site, these UCS values are based on an estimated average correlation factor (K) of 20. These values have been given for comparison only and should be interpreted together with the results of the UCS tests.



Based on the laboratory UCS test and point load testing results, the estimated intact strength of the gneiss bedrock ranges from strong (R4, 50 MPa < UCS < 100 MPa) to very strong (R5, 100 MPa < UCS < 250 MPa).

4.2.6 Groundwater Conditions

The water level in the boreholes was observed during and upon completion of drilling operations and was typically between Elevation 208.2 m and Elevation 212.0 m. In general, the samples of the overburden were noted to be moist to wet. An external source of water was pumped into each borehole as it was being advanced to maintain a constant head of water while obtaining SPT samples. However, several boreholes had sand flow into the augers and casing. In some boreholes, it was not possible to continue using hollow stem augers and NW casing with wash boring techniques was then used in order to advance the boreholes through the sand to sand and silt deposit.

Standpipe piezometers were installed in Boreholes B1-1 and B1-4 to permit monitoring of the water levels at this site. Details of the piezometer installations are shown the Record of Borehole sheets in Appendix A. The groundwater levels measured in the piezometer installations are summarised below.

| Foundation Element | Borehole No. | Ground Surface Elevation (m) | Groundwater Elevation (m) | Date of Measurement |
|---------------------------|---------------------|-------------------------------------|----------------------------------|----------------------------|
| West Abutment | B1-1 | 209.9 | 209.1 | March 20, 2009 |
| East Abutment | B1-4 | 213.3 | 212.0 | March 20, 2009 |

Groundwater levels in the area are subject to seasonal fluctuations and to fluctuations after precipitation events and snowmelt.

5.0 CLOSURE

The field drilling program was supervised by Mr. Ed Savard. This report was prepared by Mr. Evan Childerhose, B.Eng., and Mr. André Bom, P.Eng., and the technical aspects were reviewed by Ms. Sarah Coyne, P.Eng., Associate. Mr. Jorge M. A. Costa, P.Eng., Golder's Designated MTO Contact for this project and Principal with Golder, conducted a quality control review of the report.



Report Signature Page

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PART B

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6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides engineering design recommendations for the proposed Site 9 Road Underpass structure and approach embankments. The recommendations are based on interpretation of the factual data obtained from the borings advanced during the subsurface investigation. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the design of the structure foundation and approach embankments. Where comments are made on construction, they are provided in order to highlight those aspects which could affect the design of the project. Those requiring information on 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

We understand that the Site 9 Road Underpass structure will consist of two 39 m long spans with the abutments located east and west of the proposed four-lane divided Highway 69 and the centre pier located in the median of the highway.

Based on the GA drawing dated July 2010, the proposed Site 9 Road grade is at Elevation 221.3 m and Elevation 221.2 m at the west and east abutments, respectively and rises on a vertical curve to Elevation 221.4 m at the centre pier. The finished grade of the proposed Highway 69 NBL and SBL under the proposed Site 9 Road crossing is at approximately Elevation 213 m. As the existing ground surface is at about Elevation 209.8 m and 213.3 m at the west and east approaches, respectively, the proposed approach embankments will be about 11.5 m high and 7.9 m high above existing ground surface at these locations. The details of the proposed grades are shown on Drawing 1. The new SBL is oriented on approximately the same alignment as the existing Highway 69. RSS walls are proposed at the front of the west and east abutments to reduce the overall front slope length to fit within the available right-of-way and to provide the required lateral clearance to the edge of the roadway.

The boreholes advanced at the site generally encountered a surficial layer of topsoil (or fill at the existing Highway 69 location) underlain by a deposit of sand to sand and silt, between 2.1 m and 5.6 m thick at the centre pier and up to 23.8 m and 12.2 m thick at the west and east abutments, respectively. A seam of clayey silt between 0.1 m and 0.5 m thick was encountered within the sand to sand and silt deposit in the boreholes located in the vicinity of the west approach embankment. Cobbles and boulders were encountered within the lower portion of the sand to sand and silt deposit at the west abutment and overlying the bedrock in some boreholes. Gneiss bedrock was encountered at Elevations 186.3 m and 201.1 m at the west and east abutment, respectively. At the centre pier, the bedrock surface was encountered much higher than at the abutment locations and ranges between Elevations 206.7 m and 208.9 m. Based on the bedrock surface elevations noted above, the bedrock surface slopes from the high point at the centre pier significantly down to either abutment. The groundwater level in the boreholes ranges from Elevation 208.2 m to 212.0 m, generally at or up to 1.6 m below existing ground surface.



6.2 Bridge Foundation Options

Due to the loose to generally compact relative density of the sand to sand and silt deposit underlying the site and the thickness of the deposit (11.5 m to 23.6 m), shallow foundations are not recommended for support of the bridge abutments. Deep foundations consisting of steel H-piles driven to bedrock or caissons founded on the bedrock are considered to be feasible for the support of the abutments. From a foundations perspective, it is recommended that the bridge abutments be supported on steel H-piles driven to bedrock. Table 1 summarises the foundation alternatives for the abutments.

At the centre pier, the bedrock surface is present at a relatively shallow depth below ground surface (2.1 m to 5.6 m) and appears to slope down towards both abutments as well as down towards the north. For the centre pier, feasible foundation alternatives include the following:

- Steel H-piles founded on bedrock within pre-drilled holes or driven to refusal on bedrock if minimum pile length can be accommodated into the design;
- Caissons socketted a minimum of 2 m into the strong to very strong bedrock;
- Drilled, rock-socketted, concrete-filled, 340 mm diameter steel tube piles;
- Micropiles socketted into bedrock; or
- Spread footings founded on bedrock or on mass concrete or on a granular fill pad.

We recommend that the centre pier be founded on steel H-piles driven to or socketted into bedrock. The advantages, disadvantages, relative costs and risks/consequences for each of the foundation alternatives for the centre pier are summarised in Table 2.

The following sections provide design recommendations for steel H-piles founded on/in bedrock, caissons socketted into bedrock and steel tube piles founded on/in bedrock. Recommendations for micropiles and spread footings have not been discussed below due to their lower ranking. Recommendations for these two alternatives could be provided under separate cover at the request of the structural designer, should either of these alternatives be ultimately chosen as the preferred foundation option to satisfy other non-foundations criteria.

6.3 Deep Foundations

Driven steel H-piles are recommended for support of the bridge abutments. The results of the borehole investigation at the west abutment indicate cobbles and boulders were encountered within a sand to sand and silt deposit between a depth of about 13.7 m and 23.6 m depth. At the borehole at the east abutment, a 0.2 m thick deposit of cobbles and boulders was encountered overlying the bedrock. Due to the presence of cobbles and boulders, it may be difficult to advance piles or caissons at these locations below these depths, particularly at the west abutment.

We recommend founding the centre pier on steel H-piles driven to or socketted into bedrock. If the minimum pile length cannot be achieved by driving the piles, then 600 mm diameter holes will be required to be drilled into the bedrock to incorporate minimum pile lengths into the design.



6.3.1 Steel H-Pile Foundations

Steel H-piles driven to bedrock are the recommended foundation alternative for support of the bridge abutments and pier.

Given the depth to bedrock at the abutments at this site, an integral abutment design may be considered. If corrugated steel pipes (CSPs) are installed as part of the integral abutment design (through which the piles will be driven), the CSPs should be backfilled with a loose, fine to medium sand. An NSSP detailing the installation method and gradation of this sand should be included in the Contract Documents; an example is included in Appendix D.

For design of the abutments, the estimated tip elevations for the piles and estimated pile lengths based on the underside of pile caps given on the GA drawing are presented below. There should be a provision made in the Contract for dealing with varying pile lengths, but the lengths given below should be considered minimum lengths.

| Foundation Element | Location | Borehole Numbers | Proposed Underside of Pile Cap (m) | Bedrock Surface Elevation (m) | Estimated Design Pile Length (m) |
|--------------------|----------|------------------|------------------------------------|-------------------------------|----------------------------------|
| West Abutment | South | B1-1 | 215.8 | 186.3 | 29.5 |
| | North | B1-7 | | 186.3* | >22.7 |
| East Abutment | North | ST-3 (DCPT) | 216.4 | 201.0 | 15.4 |
| | South | B1-4 | | 201.1 | 15.3 |

*Bedrock not proven at this location – depth to bedrock assumed to be the same as south end of the foundation unit

At the abutments, the pile caps will be perched within the new rock fill embankment with the underside of the west and east abutment pile caps, respectively, about 6.5 m and 3.6 m above the existing ground surface. Since CSPs cannot readily be installed after construction of the rock fill embankment, consideration should be given to extending the CSPs to the native ground surface. The CSPs could be surrounded by granular fill for support or the embankment could locally be constructed using a granular pad below the CSPs through which the piles could be driven. The CSPs will extend a minimum of 3 m below the pile cap. At the west abutment, about 3.5 m of granular fill would be required to raise the embankment to the underside of the CSP, while at the east abutment about 0.6 m of granular fill would be required after the organic materials are removed.

At the west and east abutments, the elevations given above should be assumed to be the design pile tip elevations. However, practically, the piles could “hang up” on or within the deposit containing cobbles and boulders overlying the bedrock at the west abutment. Provisions for cutting off the excess pile stick up should be included in the contract.

For design of the centre pier, the estimated tip elevations for piles terminating on the bedrock surface or within the 600 mm diameter drilled holes and the estimated pile lengths based on the underside of pile cap given on the GA drawing are presented below.



| Foundation Element | Location | Borehole Numbers | Proposed Underside of Pile Cap Elevation (m) | Bedrock Surface Elevation (m) | Estimated Design Pile Length if Piles Driven to Bedrock (m) | Approximate Depth of 600 mm Diameter Drilling Required for Minimum 3 m Pile (m) |
|--------------------|----------|------------------|--|-------------------------------|---|---|
| Centre Pier | South | B1-2 | 211.1 | 208.9 | 2.2 | 0.8 |
| | Centre | B1-8 | | 208.6 | 2.5 | 0.5 |
| | North | B1-3 | | 206.7 | 4.4 | N/A |

The piles installed in the 600 mm diameter drilled holes will have to be fixed at the base in sufficient depth of concrete (to be determined by the structural engineer) to achieve fixity of the lower section of the pile. As such, the pile tip elevations indicated above may have to be lowered or adjusted as required for structural considerations and the depth of bedrock drilling indicated above may increase.

6.3.1.1 Geotechnical Axial Resistance

The east abutment and the centre pier should be supported on HP 310X110 piles driven to bedrock (or socketted into bedrock where required at the pier) and a factored geotechnical axial resistance at ULS of 2,000 kN may be assumed for design. This value represents a structural limitation for the pile rather than a geotechnical limitation. The geotechnical resistance at SLS for 25 mm of settlement (for the length of piles required at this site) will be greater than the factored axial resistance at ULS. Since the bedrock is considered to be an unyielding material, ULS conditions will govern for this foundation type.

At the west abutment, it is likely that piles will hang up on the cobbles and boulders within the sand to sand and silt deposit. At this location, for HP 310X110 piles hanging up on the cobbles and boulders, a geotechnical axial resistance at ULS of 1,600 kN and at SLS for 25 mm of settlement of 1,400 kN should be used for design. Consideration could be given to the use of a heavier pile section to facilitate driving through the cobbles and boulders. Factored geotechnical axial resistances at ULS of 1,800 kN and 2,200 kN should be used for HP 310X125 and HP 360X152 piles, respectively, corresponding to geotechnical resistances at SLS of 1,600 kN and 1,900 kN, respectively. A reduction factor of 0.8 has been applied to the ULS value to account for refusal in the cobble/boulder layer and for potential damage to the piles when penetrating into/through the boulders and into bedrock.

Piles at the abutments should be fitted with driving shoes and flange plates (reinforced tips) in accordance with OPSD 3000.100 (Steel H-Pile Driving Shoe) to minimize damage to the pile during driving and penetration through the cobbles and boulders overlying the bedrock.



6.3.1.2 *Downdrag*

The settlement of the less than 0.5 m thick clayey silt deposit at the west abutment due to the loading imposed by the 11.4 m high approach embankment will be small (less than 60 mm) and therefore downdrag loads do not need to be taken into account in the pile design.

Downdrag loads do not need to be taken into account in the pile design at the centre pier.

6.3.1.3 *Set Criteria*

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

Based on our experience, consideration should be given to the following preliminary criteria for seating the pile on bedrock. The piles should be driven to an initial set equal to or greater than 10 blows per 12 mm of penetration (unless abrupt peaking occurs) using a hammer with rated energy of about 50 kilojoules but not exceeding 60 kilojoules. On reaching the required set, the hammer energy should be reduced by about 75 percent and the pile should then be re-driven by increasing the hammer energy slowly in stages up to the maximum rated energy over about 40 blows. This procedure is intended to improve the process of the seating of the pile on the bedrock surface, which is likely to be sloping at this site. A final set of not less than 10 blows per 12 mm of penetration should be obtained at the maximum hammer energy. Provision should be made to re-tap a minimum of 10% of the piles to confirm the set after adjacent piles have been driven.

6.3.1.4 *Pile Driving Note*

The pile driving note to be added to the drawings for this project is:

“Piles to be driven to bedrock”.

6.3.1.5 *Resistance to Lateral Loads*

The design of piles subjected to lateral loads should take into account such factors as the batter of the piles (if any), the relative rigidity of the pile to the surrounding soil, the fixity condition at the head of the pile (pile cap level), the structural capacity of the pile to withstand bending moments, the soil resistance that can be mobilised, the tolerable lateral deflections at the head of the pile and pile group effects. For a longer, more flexible pile, the maximum yield moment of the pile may be reached prior to mobilisation of the lateral geotechnical resistance. For design purposes, both the structural and geotechnical resistances should be evaluated to establish the governing case.

Lateral loading could be resisted fully or partially by the use of battered piles. For vertical piles, the resistance to lateral loading will have to be derived from the soil in front of the piles. Where integral abutments are under consideration, there will also be a requirement for the piles to move sufficiently to accommodate the bridge deck deflections.



The resistance to lateral loading in front of a vertical pile may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h (kPa/m), is based on the equation for cohesionless soils given below.

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

where: n_h = the constant of horizontal subgrade reaction (kPa/m)
 z = the depth (m)
 B = the pile diameter or width (m)

For the very loose to very dense cohesionless soils (below the groundwater table) at this site, a value for n_h of 4,400 kPa/m may be assumed in the structural analysis of the lateral pile deflections.

It is understood that an integral abutment foundation design is being considered for both abutments of the bridge. Where the integral design includes the installation of 3 m long CSP liners (with the annular space between the pile and the liner filled with uniform grained, uncompacted sand), the upper portion of the H-piles will be free to flex and move laterally. With this design, the passive lateral resistance over the length of the CSP liner should be neglected.

At the abutments, the lateral resistance of the piles will be developed from the passive resistance of the soil over the portion of the piles below the CSP liners. For a single HP 310X110, HP 310X125 or HP 360X152 pile surrounded by a 3 m long CSP liner below the pile cap and embedded into the cohesionless soils below the CSP at the abutments, the estimated maximum lateral resistance at ULS is about 110 kN and at SLS, for 10 mm of deflection, is about 40 kN (assuming a steel yield strength of 300 MPa).

At the centre pier, based on the short pile lengths, the lateral resistance of the piles will be developed primarily from the batter of the piles where driven to bedrock and/or the fixity (presumably in concrete) at the base of the 600 mm diameter drilled holes. The structural resistance of the pile will govern the lateral resistance at the centre pier.

Group action for lateral loading should also be considered when the pile spacing in the direction of the loading is less than eight pile diameters. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading by a reduction factor, R (NAVFAC DM-7.2), as follows:

| Pile Spacing in Direction of Loading d = Pile Diameter | Subgrade Reaction Reduction Factor |
|--|---|
| 8d | 1.00 |
| 6d | 0.70 |
| 4d | 0.40 |
| 3d | 0.25 |

The subgrade reaction reduction factor should be interpolated for pile spacings in between those listed above.



6.3.1.6 Frost Protection

All pile caps should be provided with a minimum of 1.8 m of soil cover for frost protection, as per OPSD 3090.101 (Foundation Frost Depths for Southern Ontario).

6.3.2 Caissons

An alternative to steel H-piles, the abutments and centre pier could be supported on caissons socketted into the bedrock. The caissons could be installed without the need for temporary shoring and dewatering. The high axial resistance of the caissons would result in fewer units being required to support the abutment than that required for the H-pile design as well as the possible elimination of the pile cap. At the centre pier, we understand that individual caissons would likely be required, one under each pier column eliminating the need for the pile cap.

There may be difficulty in socketting large diameter caissons within the strong to very strong sloping gneiss bedrock at this site and achieving an adequate seal and temporary liners and tremie concrete would be required to install the caissons. At the abutment locations, the cobbles and boulders encountered within the sand to sand and silt deposit as well as the cobble and boulder layer above the bedrock may also provide some difficulties in caisson installation.

The elevation of base of the rock socket is given below.

| Foundation Element | Side | Estimated Surface of Bedrock* (m) | Base of Bedrock Socket (m) |
|--------------------|-------|-----------------------------------|----------------------------|
| West Abutment | North | 186.3* | 184.3 |
| | South | 186.3 | 184.3 |
| Centre Pier | North | 206.7 | 204.7 |
| | South | 208.9 | 206.9 |
| East Abutment | North | 201.0 | 199.0 |
| | South | 201.1 | 199.1 |

*Bedrock not proven at this location – depth to bedrock assumed to be the same as the south end of the foundation unit.

6.3.2.1 Geotechnical Axial Resistance

Caissons at this site will derive their axial resistance mainly from the shaft resistance of the rock socket with some capacity derived from end bearing depending on the adequacy of removal of loose/weathered material from the bottom of the caisson. The contribution from end-bearing will be neglected due to the difficulties in cleaning and inspecting the base of the sockets which will be below the water level and/or a great depth. The factored geotechnical axial resistance at ULS for two different caisson diameters socketted a minimum of 2 m into the bedrock are given below.



| Caisson Diameter(m) | Gneiss Bedrock (minimum 2 m socket) | |
|------------------------|--|------------------|
| | ULS (kN) | SLS for 25 mm |
| 1.5 | 8,000 | N/A |
| 1.8 | 10,000 | N/A |

The resistance required to achieve 25 mm of settlement is greater than that given for ULS for caissons socketted into the bedrock and, therefore, SLS conditions do not apply.

It should be noted that blow-up of the base of the caisson could occur during installation through the overburden at the abutments and a sufficient head of water should be maintained inside the caisson liner at all times to balance the hydrostatic pressures.

6.3.2.2 *Downdrag*

As discussed in Section 6.3.1.2, downdrag can be neglected for caissons installed at this site.

6.3.2.3 *Resistance to Lateral Loads*

The geotechnical resistance to lateral loading for the caissons should be calculated in accordance with Section 6.3.1.5 using the horizontal subgrade reaction formula. As caissons are not the preferred alternative for the abutments and pier, lateral capacities for such foundation elements at these locations are not applicable. Estimated maximum lateral resistances for caissons could be provided upon request.

6.3.2.4 *Frost Protection*

The pile caps (if any) for the caissons should be provided with a minimum of 1.8 m of conventional soil cover for frost protection, as per OPSD 3090.101 (Foundation Frost Depths for Southern Ontario).

6.3.3 *Steel Tube Pile Foundations*

As an alternative to steel H-piles, the centre pier could be supported on drilled, rock-socketted, concrete-filled 340 mm outer diameter steel tube piles. The tube piles need to be socketted into the bedrock to achieve a level founding surface at the base of the pile and to minimize the potential for sliding along the inclined bedrock surface.

Specialized equipment and drilling techniques will be required for this foundation alternative. In general, the drilled pile system uses a four step process. The first step is to weld a non-salvageable ring (i.e. crown) to the end of a steel pipe pile that will be used to drill into the bedrock and allow rotation of the shoe without rotation of the steel pipe. The next step is to insert the pilot bit into the steel pipe pile, which locks into the crown by rotating



clockwise. The next step involves drilling through the overburden and bedrock by rotating the lower part of the crown (called the driver) and the pilot bit while the upper part of the crown and the steel pipe casing do not rotate. The last step (after the steel pipe casing reaches the required bedrock socket depth) involves reversing the drill direction to unlock and retrieve the pilot bit, and leaving the steel pipe and non-salvageable crown in place. The steel pipe can then be filled with tremie concrete (if there is water inflow through the bedrock) after reinforcing steel is added, if required.

The drilled pile rock socket must be inspected by qualified geotechnical personnel to ensure that the founding stratum has been reached and is consistent with the design assumptions and that the base has been properly cleaned and is dry. In this regard, temporary liners (i.e. the steel pipe piles) will be required to permit downhole inspection.

Section 6.3.1 provides a summary of the bedrock surface elevations at the centre pier relating to estimated pile lengths.

6.3.3.1 Geotechnical Axial Resistance

The drilled piles will derive their axial resistance in part from end-bearing and in part from shaft friction. For this site, the majority of the resistance will be derived from base resistance. A factored axial geotechnical resistance at ULS of 2,400 kN should be used for design, assuming a 13 mm thick tremie concrete filled steel pipe and socketted a minimum of 0.6 m into the bedrock. The ULS value depends on structural capacity of the pile and may need to be adjusted depending on final configuration, pipe steel grade, concrete strength, bedrock socket details, and reinforcing steel, if applicable.

For drilled piles founded in the bedrock, the resistance required to achieve 25 mm of settlement is greater than that given for ULS and therefore SLS conditions do not apply.

6.3.3.2 Downdrag

Downdrag loads do not need to be taken into account in the pile design at the centre pier.

6.3.3.3 Resistance to Lateral Loads

The resistance to lateral loading developed by the soils in front of the vertical drilled piles, and the reductions due to group effects, may be determined as per Section 6.3.1.5.

6.3.3.4 Frost Protection

All pile caps should be provided with a minimum of 1.8 m of soil cover for frost protection, as per OPSD 3090.101 (Foundation Frost Depths for Southern Ontario).



6.4 Seismic Considerations

6.4.1 Site Coefficient

For seismic design purposes, the Site Coefficient, S , for this site, based on experience and considering the guidelines in Section 4.4.6 of the *CHBDC* may be taken as 1.5, consistent with Soil Profile Type III.

6.4.2 Seismic Analysis Coefficient

The potential for seismic (earthquake) loading must also be considered for the design of abutment stems/retaining walls in accordance with Section 4.6 of the *CHBDC*. According to Table A3.1.7 of the Canadian Highway Bridge Design Code (*CHBDC*), this site is located in Seismic Zone 1. The site-specific zonal acceleration ratio for the Parry Sound area is 0.05. Based on experience, for the subsurface conditions at this site, a 50 percent amplification of the ground motion may occur (i.e. Site Coefficient, $S=1.5$ for Soil Profile III from Table 4.4 of *CHBDC*), resulting in an increase in the Peak Horizontal Acceleration (PHA) from 0.05 g to 0.075 g at the ground surface.

It is understood from correspondence with MMM that based on Section 4.4.4 of the *CHBDC*, that this bridge structure is assigned Seismic Performance Zone 1. Given this, and in accordance with Section 4.4.5.1 of the *CHBDC*, no seismic analysis is required for structures located in Seismic Zone Performance 1.

6.5 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment stems and any associated wing walls/retaining walls will depend on the type and method of placement of the backfill materials, the nature of the soils behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls. As discussed in Section 6.4.2, seismic (earthquake) loading need not be analyzed.

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

- Select free-draining granular fill meeting the specifications of Special Provision SP 110S13 (Aggregates – Base, Subbase, Select Subgrade and Backfill) Granular ‘A’ or Granular ‘B’ Type II but containing less than 5 percent passing the No. 200 sieve size should be used as backfill behind the walls. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS 501 (Compaction). Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3101.150 (Walls, Abutment, Backfill Minimum Granular Requirements) and 3121.150 (Walls, Retaining, Backfill Minimum Granular Requirements).



- For structures that are not comprised of integral or semi-integral abutments, rock fill may be used as backfill behind the walls and the material should meet the specifications as outlined in the Northern Region Directive for backfill to structures adjacent to rock fill embankments, dated November 2002. Other aspects of rock backfill requirements should be in accordance with OPSD 3101.200 (Walls, Abutment, Backfill Rock).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with CHBDC Section 6.9.3 and Figure 6.6. Other surcharge loadings should be accounted for in the design, as required.
- For restrained structures, the granular fill should be placed in a zone with width equal to at least 1.8 m behind the back of the walls (in accordance with Figure C6.20(a) of the *Commentary to the CHBDC*). For unrestrained structures, granular fill should be placed within the wedge shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the base of the footing (in accordance with Figure C6.20(b), Case II, of the *Commentary to the CHBDC*).
- For restrained structures, the pressures are based on the proposed embankment fill materials and the existing overburden soils and the following parameters (unfactored) may be used assuming the use of granular fill or rock fill:

| | | |
|--|----------------------|----------------------|
| | Earth Fill | Rock Fill |
| Soil unit weight: | 21 kN/m ³ | 19 kN/m ³ |
| Coefficients of static lateral earth pressure: | | |
| Active, K_a | 0.31 | 0.22 |
| At rest, K_o | 0.47 | 0.36 |

- For unrestrained structures, the pressures are based on the rock fill as above or on the granular fill as placed and the following parameters (unfactored) may be assumed:

| | | |
|--|----------------------|----------------------|
| | Granular 'A' | Granular 'B' |
| | | Type II |
| Soil unit weight: | 22 kN/m ³ | 21 kN/m ³ |
| Coefficients of static lateral earth pressure: | | |
| Active, K_a | 0.27 | 0.27 |
| At rest, K_o | 0.43 | 0.43 |

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

A restrained structure is typically a concrete box culvert or a rigid frame bridge where the rotational and/or horizontal movement is not sufficient to mobilize the active earth pressure condition. For this condition, an at-rest pressure plus any compaction surcharge should be included in the design of the structure.



6.6 Retained Soil System (RSS) Walls

RSS walls are required at the west abutment extending about 15 m north of the north end and about 7 m south of the south end of the abutment. RSS walls are required at the east abutment extending 7 m north of the north end and 15 m south of the south end of the abutment. The RSS walls are required in order to reduce the overall length of the front slope to fit within the available right-of-way.

An RSS wall consists generally of granular fill placed and compacted in layers, and reinforced with fabric strips or grids. A facing material, typically pre-cast concrete panels mechanically fastened to the reinforcing strips or grids, is used to form the vertical face of the retained soil structure and to prevent loss of fill material.

A typical RSS wall has the front facing supported on a strip footing placed at shallow depth below the ground surface in front of the wall. The final grading design should be checked to provide approximately 0.3 m of embedment for the RSS wall facing footing. The base of the RSS wall at the west and east abutments is currently shown on the GA to be located at about Elevation 213.7 m and 213.0 m, respectively. A granular fill pad comprised on Granular 'B' Type II or Granular 'A' will be required under the footing and the RSS mass. The granular fill pad should be constructed over the native cohesionless soils after the removal of topsoil, loose fill or unsuitable native soils and should extend a minimum of 1 m beyond the edges of the footing and soil mass.

Assuming that the RSS wall acts as a unit and utilizes the full width of the reinforced soil mass, which has been taken as 0.8 times the height of the height of the wall, the factored geotechnical resistances at ULS and the geotechnical resistance at SLS (25 mm of settlement) given below may be used for assessment of the reinforced mass founded on the properly prepared granular fill pad constructed over the native cohesionless soils.

| | Wall Height (m) | Assumed Reinforced Width* (m) | Factored Geotechnical Resistance at ULS | Geotechnical Resistance at SLS |
|---------------|--------------------|-------------------------------------|--|-----------------------------------|
| West Abutment | 3.0 m | 2.4 | 600 kPa | 100 kPa |
| East Abutment | 5.0 m | 4.0 | 300 kPa | 100 kPa |

* Assumed equivalent to 80% of the wall height.

The resistance to lateral forces / sliding resistance between the compacted fill of the RSS wall and the subgrade should be calculated in accordance with Section 6.7.5 of the CHBDC. The coefficient of friction, $\tan \phi'$, between the compacted granular fills of the RSS wall and the properly prepared subgrade may be taken as 0.6. This represents an unfactored value; in accordance with the CHBDC, a factor of 0.8 is to be applied in calculating the horizontal resistance.

The static global stability of the RSS wall has been analyzed and the results of the analysis are discussed in Section 6.7.1. The internal stability of the wall should be checked by the RSS supplier/designer.

The assessment of settlement of the foundation soils below the approach embankments is presented in Section 6.7.3. The settlements of the foundation soils are anticipated to occur generally during and upon completion of construction of the approach embankments. Consideration should be given to completing the construction of the embankments to their full height prior to constructing the RSS wall footings as discussed in Section 6.7.4.



6.7 Approach Embankment Design

The construction of the Site 9 Road underpass structure will require placement of up to about 12.2 m of fill within the limits of the west approach embankment and up to about 8.6 m of fill within the limits of the east approach embankment after stripping of approximately up to 0.6 m of surficial organic soils (i.e. topsoil). The rock fill embankment will be constructed essentially all above the present groundwater level.

At both approaches, the foundation soils are composed primarily of very loose to dense sand to sand and silt. A thin clayey silt layer, up to 0.5 m thick, was encountered within the cohesionless deposit at the proposed west abutment.

Based on the subsurface conditions at this site, the approach embankments will be founded on very loose to dense sand to sand and silt, underlain by bedrock. All topsoil, organic matter and fill materials should be stripped from below the approach embankment areas.

At all areas, the analyses assume that organic soils have been removed prior to construction of the new embankments. The piezometric conditions required in the analysis were based on the groundwater levels noted during the drilling of the boreholes and measured in the piezometer installations. In general, the groundwater table was assumed to be located within 1 m of the existing ground surface elevation.

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

6.7.1 Stability

Analyses were performed on the critical (i.e. highest or thickest fill) cross-sections of the proposed new approach embankments to assess the stability for the proposed stratigraphy, embankment heights and slope, which includes the geometry of the proposed RSS walls.

6.7.1.1 Methodology

Limit equilibrium slope stability analyses were performed using the commercially available program GeoStudio 2007 (Version 7.13), produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis. For all analyses, the Factor of Safety of numerous potential failure surfaces was computed in order to establish the minimum Factor of Safety. The Factor of Safety is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. A target minimum Factor of Safety of 1.3 is normally adopted for the design of embankment slopes under static conditions. This Factor of Safety is considered adequate for the embankments at this site considering the design requirements and the field data available. The stability analyses were performed to check that the target minimum Factor of Safety was achieved for the design embankment height and geometry. In general, circular slip surfaces were analysed in the design.



6.7.1.2 Parameter Selection

For the very loose to dense cohesionless soils, the effective stress parameters employed in the analysis were estimated from empirical correlations based on the results of the in situ SPT. The correlations proposed by Peck et al. (1974) and NAVFAC (1982) were employed and the results were tempered by engineering judgment based on precedent experience in similar soils.

For the cohesive layer, total stress parameters were employed in the analysis. The total stress parameters (i.e. average mobilized undrained shear strength – s_u) for the cohesive soils were estimated from correlations with the SPT results and laboratory test data.

Summarized below are the simplified stratigraphy and the associated strengths and unit weights employed for the different soil types in the proposed approach/abutment areas. For the purpose of analysis, both granular fill and rock fill have been considered for the construction of the approach embankments as indicated below. Granular fill is assumed to have side slopes at 2H:1V and rock fill is assumed to have side slopes at 1.25H:1V. A discussion on the different fill types, with respect to stability, is provided in Section 6.7.1.3; however, it is understood that rock fill will be used for the construction of the approach embankments.

| Soil Type | Unit Weight (kN/m³) | Undrained Shear Strength (kPa) | Angle of Internal Friction |
|---|---|---|---------------------------------------|
| New Rock Fill | 19 | -- | 40° |
| New Granular Fill | 21 | -- | 35° |
| Existing Embankment Fill (existing Highway 69 embankment, assumed to be sand and gravel with rock fill – beyond the toe of west abutment front slope) | 20 | -- | 30° |
| Very Loose to Dense Sand to Sand and Silt | 19 | -- | 28° |
| Clayey Silt (West Abutment only) | 17 | 50 | -- |

6.7.1.3 Embankment Fill Types and Benching Requirements

The different embankment fill alternatives (i.e. granular fill and rock fill) provide relative advantages and disadvantages in terms of weight (i.e. driving force and applied load to founding subsoils/bedrock), construction cost and time, and ease of construction/availability.

Granular Fill

The main advantage of using granular fill for embankment construction is the ease of construction and the reduction of post-construction settlements within the fill embankment itself. However, this option will require a larger volume of fill and wider right-of-way because the side slopes will be flatter than for rock fill slopes.



For the granular fill option, the incorporation of a 2 m wide bench (or berm) into the uniform side slope profile is required for embankment which exceed a height of 8 m such that the uninterrupted embankment slope does not exceed a height of 8 m, as per OPSD 202.010 (Slope Flattening). Given that the west and east approach embankment heights will be greater than 8 m, benches will be required if granular fill is to be used embankment construction.

Rock Fill

The main advantage of using rock fill for embankment construction is the ability to achieve steeper embankment side slopes and therefore less property requirements. This is useful in areas with a limited right-of-way, which we understand is not of concern at this site. The disadvantage of using rock fill for the construction of high embankments is that some post-construction settlement of the embankment fill itself will occur.

For the rock fill option, the incorporation of 2 m wide benches (or successive berms) into the uniform side slope profile is required wherever the embankment will exceed a height of 10 m such that the uninterrupted rock fill slope does not exceed a height of 10 m as per OPSD 202.010 (Slope Flattening). At the west approach, where the maximum approach embankment height is about 12.2 m, a bench would be required if rock fill is to be used for embankment construction. Given that the maximum east approach embankment height is about 8.6 m, a bench would not be required.

6.7.1.4 Results of Analysis

The results of the stability analyses for the two embankment fill options (granular and rock fill) are summarized below for the west and east approach embankments. The minimum Factor of Safety is based on a deep-seated, global trial failure surface that would impact the operation of the roadway.

| Location | Embankment Height at Critical Section (m) | Earth Fill Option | | Rock Fill Option | |
|---------------|---|--------------------------------|--------------------------|--------------------------------|--------------------------|
| | | Recommended Side Slope Profile | Minimum Factor of Safety | Recommended Side Slope Profile | Minimum Factor of Safety |
| West Approach | 12.2 | 2H : 1V | ≥ 1.3 | 1.25H : 1V | ≥ 1.3 |
| East Approach | 8.6 | | | | |

Since the Factor of Safety is greater than 1.3 for the proposed approach embankments, stability mitigation is not required.

6.7.2 Liquefaction Potential and Seismic Analysis

As noted in Section 6.4.2, this site is located in Seismic Zone 1 with a PHA less than 0.08. Further, the bridge structure is not a lifeline structure. As such, based on Section 4.4.4 of the *CHBDC*, the site is assigned to Seismic Performance Zone 1 and, therefore, in accordance with Section 4.4.5.1 of the *CHBDC*, no liquefaction analysis is required.



6.7.3 Settlement

Settlement of the approach embankments and associated RSS walls can be expected as a result of the loading from the new fills on the cohesionless foundation soils at this site and the thin cohesive layer below the proposed west abutment. In addition, settlement due to compression of the new embankment fill itself, assumed to be rock fill, will also occur.

For the settlement analyses, the critical sections were assessed considering the location of the greatest new embankment height at each approach area.

6.7.3.1 Methodology

To estimate the magnitude of the expected settlements, analyses were carried out on the critical sections of the proposed approach embankments using both the commercially available program Settle^{3D} (by Rocscience Inc.) and hand calculations.

6.7.3.2 Parameter Selection

The immediate compression of the cohesionless foundation strata was assessed by estimating an elastic modulus of deformation based on the SPT 'N'-values and empirical correlations found in literature by Bowles (1984) and Kulhawy and Mayne (1990).

The cohesive layer encountered below the footprint of the west abutment will generally consolidate as fill is placed due to the relatively small thickness of this layer. Pore pressures are expected to dissipate as filling progresses, such that the settlement is anticipated to be elastic and not time sensitive.

The following table summarizes the simplified stratigraphy, unit weights and deformation parameters employed for the different soils types in the approach areas.

| Location of Embankment | Material | Thickness (m) | Unit Weight (kN/m³) | Estimated Deformation Properties |
|-------------------------------|---|----------------------|---------------------------------------|---|
| West Approach | Rock Fill | 12.2 | 19 | Refer to Section 6.7.3.3 |
| | Very Loose to Dense Sand to Sand and Silt | 23.6 | 19 | E' = 20 MPa |
| | Clayey Silt | up to 0.5 | 17 | E' = 2 MPa |
| East Approach | Rock Fill | 8.6 | 19 | Refer to Section 6.7.3.3 |
| | Very Loose to Compact Sand to Sand and Silt | 11.5 | 19 | E' = 20 MPa |



6.7.3.3 *Settlement of New Embankment Fill*

Granular Fill

If granular fill is employed for the construction of the new approach embankments at this site, very little additional settlement due to compression of the embankment fill itself will occur additional to the estimated settlement of the foundation soils. In this case, the additional settlement from properly compacted granular fills is expected to be less than about 25 mm and will occur during construction.

Rock Fill

Where rock fill is to be used for the construction of the proposed embankments, there will be settlement due to compression of the rock fill itself under self weight, in addition to the settlement of the underlying foundation soils as described above. The magnitude of settlement of the rock fill depends on the following factors:

- type of rock/strength of particles;
- size and shape of rock particles;
- gradation of rock fill;
- total height/thickness of rock fill (stress level); and
- method of construction and sequence of placement (including lift thickness, compactive effort and state of packing).

The settlement of rock fill occurs as a result of re-arrangement of rock particles under load and wetting and as a result of localized crushing of rock particles at point contacts. The magnitude of both the short-term and long-term post-construction settlement of the rock fill is a function of the height of fill as well as the method of fill placement (i.e. compacted versus dumped rock fill) as outlined in MTO Foundations Guideline, "Post-Construction Rock Fill Settlement and Guidelines For Estimating Rock Fill Quantity", dated April 2010.

Rock fill should be placed, whenever possible, in a controlled manner (i.e. not end-dumped) in accordance with OPSS 206 (Grading). Blading, dozing and 'chinking' the rock fill to form a dense, compact mass is required to minimize voids and bridging and reduce settlements and should be used to construct rock fill embankments above the existing groundwater table. Where rock fill cannot be placed in a controlled manner (i.e. below the groundwater table), the post-construction settlement of the rock fill is expected to be greater.

Short-Term Rock Fill Settlement

The magnitude of short-term post-construction settlement associated with compacted and end-dumped rock fill may be estimated in accordance with the MTO Foundations Guideline (April 2010), as follows:



| Total Height of Rock Fill, H | Short-Term Rock Fill Settlement | |
|------------------------------|---------------------------------|------------------|
| | Compacted Rock Fill | Dumped Rock Fill |
| Up to 5 m | 0.5% H | 1.0% H |
| 5 m to 10 m | 0.75% H | 1.5% H |
| 10 m to 15 m | 1.0% H | 2.0% H |

Approximately 90 percent of the short-term settlement may be expected to occur within the first six (6) months following construction of the embankment to full height. The short-term settlement is expected to be fully completed within one (1) year following the completion of embankment construction to full height.

Long-Term Rock Fill Settlement

The magnitude of long-term post-construction settlement for compacted and end-dumped rock fill may be estimated in accordance with the MTO Foundations Guideline (April 2010), as follows:

| Total Height of Rock Fill, H | Short-Term Rock Fill Settlement | |
|------------------------------|---------------------------------|------------------|
| | Compacted Rock Fill | Dumped Rock Fill |
| Up to 15 m | 0.1% H | 0.2% H |

The long-term rock fill settlement is expected to occur from one (1) year following the completion of construction over the life of the embankment.

6.7.3.4 Results of Analysis

Presented below is the estimated settlement of the foundation soils and embankment rock fill as a result of the embankment construction in the area of the bridge approaches.

| Location of Embankment | Estimated Maximum Proposed Embankment Height (m) | Thickness of Cohesionless Deposit (m) | Estimated Settlement of Foundation Soils (mm) | Estimated Settlement of Rock Fill (mm) |
|-----------------------------|--|---------------------------------------|---|--|
| West Approach STA 13+644 | 12.2 | 23.6* | 175 | 135 |
| East Approach STA 13+567 | 8.6 | 11.5 | 75 | 75 |

*Includes 0.5 m thick cohesive layer

The settlements of the foundation soils are expected to occur rapidly (i.e. during or shortly after construction) in response to filling based on the estimated relatively high hydraulic conductivity of the native soils as indicated by results of the grain size distributions.



It is anticipated that 90% of the short-term rock fill settlement will occur in the first 6 months after embankment construction and long-term settlement will occur 1 year following embankment construction, resulting in post-embankment construction settlement of 135 mm and 75 mm at the west and east approaches, respectively. Mitigation measures as discussed in Section 6.7.4 will therefore be required.

Construction of the embankment to the full height will also cause settlement of the RSS wall footings and soil masses behind the walls if these are constructed prior to construction of the full approach embankments; mitigation will also be required in this regard.

6.7.3.5 *Embankment Widening*

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

6.7.4 *Mitigation of Settlements*

The following will need to be considered for embankment and RSS wall construction at the west and east approaches:

- Settlement of cohesionless foundation soils will occur as the embankment is constructed.
- Settlement of RSS wall footings and soil masses will occur due to the settlement of the underlying foundation soils as a result of embankment construction to full height which is higher than the top of the walls.
- Post-construction settlement of rock fill embankment will occur after the embankment is constructed to the full height.

As discussed in Section 6.3.1, about 6.5 m of granular fill would be required to raise the west embankment to the underside of the west abutment and about 3.6 m of granular fill to the underside of the east abutment. As such, the embankments will be partially constructed in the area of the abutments and some settlement of the foundation soils will occur during construction of the granular pad. If the RSS wall footings and soil masses are constructed after granular pad construction and prior to embankment construction to the full embankment height, the estimated settlement of the foundation soils below the RSS wall footing and soil mass is estimated to be about 80 mm and 40 mm under the west and east walls, respectively. If the walls cannot accommodate this settlement, we recommend that the approach embankment within the vicinity of the RSS be constructed to full height and preloaded for at least 2 months prior to the construction of the RSS footings, walls and soil masses. This would require localized sub-excavation of the new embankment fill to allow wall construction.

Where rock fill will be used as embankment fill material, beyond the granular pads at the abutments, settlement of both the east and west approach embankments are anticipated to occur due to the compression of rock fill. The magnitude of the post-construction settlement of the rock fill settlement is estimated to be about 135 mm and 75 mm at the west and east approaches, respectively, as noted in Section 6.7.3.5. If the embankments are preloaded for 6 months, the estimated post-construction settlement would be about 20 mm for the west and east approaches. Consideration could be given to constructing the embankments entirely out of granular fill which would eliminate post-construction settlement of the embankment fill at this site.



6.8 Subgrade Preparation and Embankment Construction

The existing native subsoils are considered to be an appropriate subgrade for the proposed approach embankments. Prior to the placement of fill, topsoil/organic deposits (up to 0.6 m thick) and any softened or loosened soil should be stripped from the plan limits of the proposed works.

Placement of granular fill or rock fill material should be carried out in accordance with the requirements as outlined in SP206S03 (Earth Excavation, Grading; Rock Excavation, Grading). Side slopes for rock fill embankments should be no steeper than 1.25H:1V as per OPSD 202.010 (Slope Flattening).

In accordance with OPSD 202.010 (Slope Flattening), and as required by MTO, the incorporation of 2 m wide berms into the uniform side slope profile is required at the west approach embankment.

6.9 Design and Construction Considerations

6.9.1 Excavation and Groundwater Control

At the proposed abutments, the pile caps will be perched within the new rock fill embankments. Excavations at the approaches and abutments will generally consist of stripping the existing surficial organic matter (i.e. topsoil).

At the centre pier, the underside of the pile cap will be at Elevation 211.1 m, which is generally at or near ground surface at the south end of the pier and about 1.2 m below existing ground surface at the north end of the pier.

All excavations must be carried out in accordance with the latest edition of the Ontario Occupational Health and Safety Act (OHSA) and Regulations for Construction Projects and good construction practice. The native soils are classified as Type 3 soil, according to the OHSA.

Excavations below the water level are not anticipated at the abutments or centre pier. Surface water should be directed away from the excavation at all times.

6.9.2 Obstructions

Cobbles and boulders were encountered within the cohesionless soils and/or overlying the bedrock at the proposed abutments. Further, boulders and/or rock fill may be encountered within the existing highway fill. We recommend that an Operational Constraint (OC) be included in the Contract Documents to alert the Contractor of the presence of these obstructions; an example is included in Appendix D.

7.0 CLOSURE

This report was prepared by Mr. André Bom, P.Eng., and the technical aspects were reviewed by Ms. Sarah E. M. Coyne, P.Eng., an Associate with Golder. Mr. Jorge M. A. Costa, P.Eng., Golder's Designated MTO Contact for this project and Principal with Golder, conducted an independent quality control review of the report.




Report Signature Page

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STANDARDS:

ASTM International:

| | |
|----------------|--|
| ASTM D1586-08a | Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils |
| ASTM D5731-08 | Standard Test Method for Determination of the Point Load Strength Index of Rock and Application to Rock Strength Classifications |

Ministry of Transportation:

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|--------------------|---|
| Foundation Section | Post-Construction Rock Fill Settlement and Guidelines For Estimating Rock Fill Quantity, April 2010 |
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Ministry of Transportation Ontario Special Provisions

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|-----------|--|
| SP 110S13 | Material Specifications for Aggregates – Base, Subbase, Select Subgrade and Backfill |
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Northern Region Directive: Backfill to Structures Adjacent to Rock Embankment Approaches, November 2002

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

Ontario Provincial Standard Specification:

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|----------|--|
| OPSS 206 | Construction Specification for Grading, November 2009 |
| OPSS 501 | Construction Specification for Compacting, November 2005 |

Ontario Provincial Standard Drawings Ontario Provincial Standard Drawings

| | |
|---------------|--|
| OPSD 202.010 | Slope Flattening using Surplus Excavated Material on Earth or Rock Embankment, November 2009 |
| OPSD 3000.100 | Foundation Piles Steel H-Pile Driving Shoe, November 2005 |
| OPSD 3000.201 | Foundation Piles Steel HP 310 Oslo Point, November 2005 |
| OPSD 3090.101 | Foundation Frost Depths for Southern Ontario, November 2005 |
| OPSD 3101.150 | Walls Abutment, Backfill Minimum Granular Requirement, November 2005 |
| OPSD 3101.200 | Walls Abutment, Backfill Rock, November 2005 |
| OPSD 3121.150 | Walls Retaining, Backfill Minimum Granular Requirement, November 2005 |

Ontario Water Resources Act:

| | |
|---------------------------|-------------------------------------|
| Ontario Regulation 372/97 | Amendment to Ontario Regulation 903 |
| Ontario Regulation 903/90 | Wells |



FOUNDATION REPORT - SITE 9 ROAD UNDERPASS STRUCTURE HIGHWAY 69 GWP 5403-05-00, WP 5189-06-01

Table 1: Evaluation of Foundation Alternatives – Abutments

| Options | Rank | Advantages | Disadvantages | Relative Costs | Risks/Consequences |
|---------------------------------|------|--|--|---|--|
| Steel H-Piles Driven to Bedrock | 1 | <ul style="list-style-type: none"> ■ Conventional construction. ■ Bottom of pile cap/abutment above existing ground surface. ■ Axial resistance depends on structural capacity of pile not on the subsurface soils. ■ Eliminates post-construction settlement. | <ul style="list-style-type: none"> ■ Possibility of piles “hanging up” on the very dense stratum at Elev. 194.5 m or on cobbles and boulders below Elev. 195 m at west abutment. ■ Difficulty seating piles into sloping bedrock. ■ Pile tip needs to be reinforced with driving shoes. ■ Relatively long piles required (about 29 m) therefore use of a heavier pile section (HP 310X125 or HP 360X152) to facilitate driving through the very dense stratum and/or cobbles and boulders may be required. | <ul style="list-style-type: none"> ■ Higher cost associated with potential heavier pile sections. ■ Higher cost associated with provisions for additional piles if piles hang-up on the very dense stratum or are damaged or piles are driven out of alignment if sloping bedrock surface is present. ■ Lower relative costs compared with caisson option. | <ul style="list-style-type: none"> ■ Risk of pile deviating from alignment from boulders encountered during pile driving at west abutment. |
| Caissons Socketted into Bedrock | 2 | <ul style="list-style-type: none"> ■ Reduced number of deep elements compared to steel H-piles. ■ Possible elimination of pile cap. ■ Eliminates post-construction settlement. | <ul style="list-style-type: none"> ■ Temporary liners would be required for groundwater control and support through the granular overburden. ■ Relatively long caissons (about 29 m) may have to penetrate boulders at some locations. ■ Concrete for caissons would have to be placed by tremie methods below the groundwater level. ■ May be difficult socketting caissons into the strong and potentially sloping gneiss bedrock. | <ul style="list-style-type: none"> ■ Higher cost than steel H-piles driven to bedrock. | <ul style="list-style-type: none"> ■ Risk of difficulties in penetrating the cobble and boulder layer and achieving seal and drilling large diameter bedrock socket. ■ Anticipated slow rate of drilling production for large diameter in strong to very strong bedrock. |
| Spread Footing on Granular Pad | 3 | <ul style="list-style-type: none"> ■ Conventional construction. | <ul style="list-style-type: none"> ■ Does not allow for integral abutment design. ■ Preloading may be required. ■ Inadequate axial resistance of subsoils requiring large foundation units. | <ul style="list-style-type: none"> ■ Lower cost than for deep foundations. | <ul style="list-style-type: none"> ■ Settlement of the footing due to embankment fill over loose cohesionless foundation soils. |



FOUNDATION REPORT - SITE 9 ROAD UNDERPASS STRUCTURE HIGHWAY 69 GWP 5403-05-00, WP 5189-06-01

Table 2: Evaluation of Foundation Alternatives – Centre Pier

| Options | Rank | Advantages | Disadvantages | Relative Costs | Risks/Consequences |
|---|------|---|--|---|--|
| H-piles (HP310X110) Installed Within 0.6 m Diameter Pre-Drilled Holes and Socketted into Bedrock (or driven to refusal on bedrock if minimum pile length can be achieved) | 1a | <ul style="list-style-type: none"> Shored excavation and dewatering may not be required if base of pile cap located above groundwater level. Eliminates post-construction settlement. Easier to install 0.6 m diameter H-pile sockets in bedrock compared to larger diameter caissons. Typically, shorter length of rock socket compared to caissons. | <ul style="list-style-type: none"> Minimum pile length for driven H-piles (without rock sockets) may not be achievable at all locations. Pre-drilled holes may be required for socketting piles into bedrock followed by tremie placement of concrete for pile toe fixity. Temporary liners would be required for groundwater control and support through overburden where rock socketting required. Specialised drilling equipment required for creating 0.6 m diameter socket in strong to very strong bedrock. | <ul style="list-style-type: none"> Assuming 12 piles and 2/3 of piles require rock socketting, estimated total cost = \$175,000. | <ul style="list-style-type: none"> At locations where piles are not socketted in rock, there is a risk of driven piles being out of alignment/location due to sloping bedrock, or of piles becoming damaged. Potential for difficulties seating drill casing in/on sloping bedrock. |
| Caissons Socketted 2 m into Bedrock | 1b | <ul style="list-style-type: none"> Possible elimination of pile cap. Dewatering may not be required for either caisson installation or caisson cap construction. Shored excavation not required. Eliminates post-construction settlement. | <ul style="list-style-type: none"> Temporary liners would be required for groundwater control and support through overburden. Concrete for caissons would have to be placed by tremie methods below the groundwater level. Will require careful drilling operations as it may be difficult socketting caissons into strong to very strong and sloping gneiss bedrock. Requires specialist foundation contractor. More difficult to advance larger diameter caissons than smaller diameter caissons or H-pile sockets. | <ul style="list-style-type: none"> For the limited number of required foundation elements, costs will likely be more expensive comparable to (or more expensive than) other alternatives which may require extensive shoring/cofferdams. Assuming 2 – 1.5 m diameter caissons socketted 3 m into bedrock, estimated total cost = \$300,000. | <ul style="list-style-type: none"> Risk of difficulties achieving seal and drilling large diameter bedrock socket. Potential for difficulties seating caisson drill casing in/on sloping bedrock. Anticipated slow rate of drilling production for large diameter in strong to very strong bedrock. |



FOUNDATION REPORT - SITE 9 ROAD UNDERPASS STRUCTURE HIGHWAY 69 GWP 5403-05-00, WP 5189-06-01

Table 2: Evaluation of Foundation Alternatives – Centre Pier (Continued)

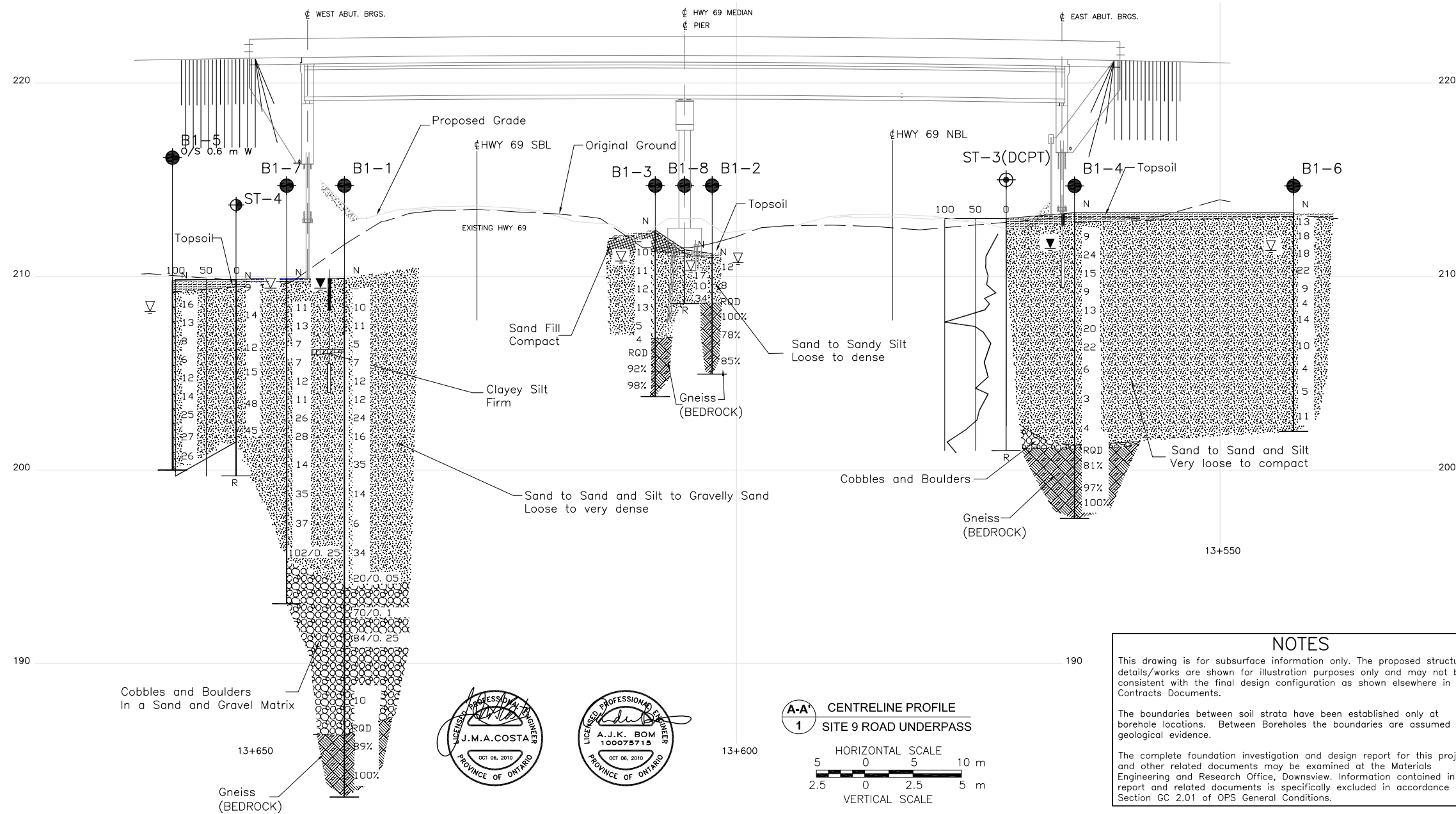
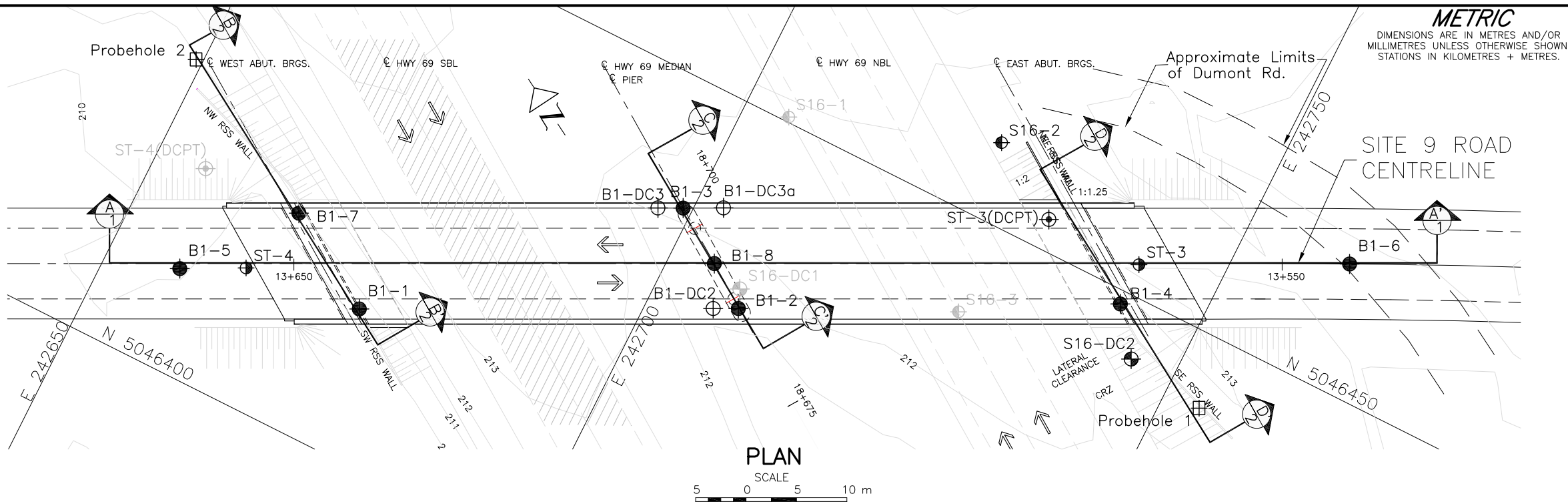
| Options | Rank | Advantages | Disadvantages | Relative Costs | Risks/Consequences |
|--|------|--|---|--|---|
| Drilled, Rock-Socketted Concrete-Filled Tube Piles (0.34 m diameter) | 1c | <ul style="list-style-type: none"> Possible increased capacity as compared to steel H-piles. Shored excavation and dewatering may not be required if pile cap constructed above groundwater level. Eliminates post-construction settlement. | <ul style="list-style-type: none"> Specialised drilling equipment required. Non MTO standard pile types. | <ul style="list-style-type: none"> Assuming 10 – 0.34 m diameter tube piles socketted 1 m into bedrock, estimated total cost = \$225,000. | <ul style="list-style-type: none"> Potential for difficulties seating drill casing in/on sloping bedrock. |
| Micropiles (0.27 m diameter grouted pile with central reinforcing bar and permanent steel casing on upper section of pile) | 2 | <ul style="list-style-type: none"> Shored excavation and dewatering may not be required if pile cap constructed above groundwater level. Eliminates post-construction settlement. | <ul style="list-style-type: none"> Rock socketting required for fixity. Specialised drilling equipment required. Specialised detail foundation design required for micropile foundation. | <ul style="list-style-type: none"> Assuming 14 – 0.27 m diameter micropiles with a minimum 3 m bond length in bedrock, estimated total cost = \$275,000. Additional costs required for detailed design and load testing during construction. | <ul style="list-style-type: none"> Specialised foundation design to achieve required lateral resistance with small diameter piles. |



FOUNDATION REPORT - SITE 9 ROAD UNDERPASS STRUCTURE HIGHWAY 69 GWP 5403-05-00, WP 5189-06-01

Table 2: Evaluation of Foundation Alternatives – Centre Pier (Continued)

| Options | Rank | Advantages | Disadvantages | Relative Costs | Risks/Consequences |
|---|------|---|--|--|---|
| Spread Footings on Mass Concrete and/or Bedrock | 3 | <ul style="list-style-type: none"> Eliminates post-construction settlement. Straightforward construction. | <ul style="list-style-type: none"> Variable bedrock surface may require large quantity of mass concrete to achieve level footing. Temporary shoring/cofferdam is required adjacent to existing Highway 69 roadway embankment. May require steel dowels to anchor mass concrete to sloping bedrock surface. Sub-excavation of cohesionless, water-bearing soils could be up to 5.6 m below existing ground surface and up to 4 m below the groundwater level. Dewatering, cleaning bedrock surface and concrete placement required within shored and dewatered excavation (i.e. cofferdam). Installation of cofferdam on sloping bedrock and achieving adequate seal around base to restrict seepage inflow may be difficult. | <ul style="list-style-type: none"> Lower relative costs compared with piled foundation options. Additional costs required for installation of temporary shoring/cofferdam and dewatering. | <ul style="list-style-type: none"> Risk that bedrock excavation may be required if encountered at different/higher elevations than that identified in boreholes. Risk of not being able to properly seal cofferdam and achieving full dewatering within the cofferdam. |
| Spread Footings on Granular 'A' Pad | 4 | <ul style="list-style-type: none"> Bedrock excavation/mass concrete not required. | <ul style="list-style-type: none"> Large footprint required to accommodate a granular pad beyond the footing limits and therefore more extensive shoring/excavation/dewatering operation. Temporary shoring is required adjacent to existing Highway 69 roadway embankment. Sub-excavation of cohesionless, water-bearing soils could be up to 5.6 m below existing ground surface and up to 4 m below the groundwater level. Dewatering and placement of Granular 'A' engineered fill required within shored and dewatered excavation (i.e. cofferdam). Installation of cofferdam on sloping bedrock and achieving adequate seal around base to restrict seepage inflow may be difficult. | <ul style="list-style-type: none"> Lower relative costs compared with piled foundation options and footing on mass concrete option. Additional costs required for installation of temporary shoring and dewatering. Additional shoring costs compared to shallow footing on bedrock due to larger footprint required. | <ul style="list-style-type: none"> Risk of differential settlement between abutment foundation elements founded on piles on bedrock. Risk of groundwater seepage into the cofferdam resulting in inability to achieve adequate and consistent density (compaction) of the Granular 'A' pad. |



CONT No.
WP No. 5189-06-01

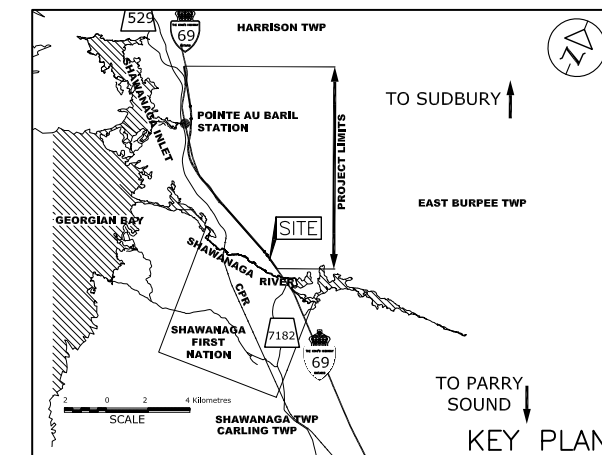
HIGHWAY 69
SITE 9 ROAD UNDERPASS
BOREHOLE LOCATION
AND SOIL STRATA



SHEET



Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA



LEGEND

- Borehole - Current Investigation
- ⊕ DCPT - Current Investigation
- Previous Borehole (Golder, 2008)
- ⊕ Previous DCPT (Golder, 2008)
- Previous Borehole (AMEC, 2006)
- ⊕ Previous DCPT (AMEC, 2006)
- ⊕ Probehole
- N Standard Penetration Test Value
- 4 Blows/0.3 m unless otherwise stated (Std. Pen. Test, 475j/blow)
- ▽ WL upon completion of drilling
- ▽ WL in piezometer, measured on March 20, 2009
- ⊕ Seal
- ⊕ Piezometer
- R Refusal

| No. | ELEVATION | CO-ORDINATES | |
|------------|-----------|--------------|----------|
| | | NORTHING | EASTING |
| B1-1 | 209.9 | 5046414.4 | 242674.9 |
| B1-2 | 211.0 | 5046431.3 | 242708.4 |
| B1-3 | 212.3 | 5046437.7 | 242699.1 |
| B1-4 | 213.3 | 5046448.7 | 242742.1 |
| B1-5 | 209.8 | 5046410.0 | 242657.2 |
| B1-6 | 213.3 | 5046462.4 | 242760.6 |
| B1-7 | 209.9 | 5046420.2 | 242665.2 |
| B1-8 | 211.4 | 5046434.2 | 242704.3 |
| B1-DC2 | 211.0 | 5046430.2 | 242706.2 |
| B1-DC3 | 212.0 | 5046436.6 | 242696.8 |
| B1-DC3a | 212.8 | 5046439.6 | 242702.6 |
| S16-2 | 213.1 | 5046457.7 | 242724.4 |
| S16-DC2 | 212.1 | 5046444.3 | 242745.5 |
| ST-3 | 213.7 | 5046453.0 | 242742.0 |
| ST-4 | 209.9 | 5046413.0 | 242663.0 |
| ST-3(DCPT) | 213.0 | 5046453.0 | 242732.0 |
| PROBEHOLE1 | | 5046443.0 | 242753.7 |
| PROBEHOLE2 | | 5046429.2 | 242649.3 |

REFERENCE

Base plans provided in digital format by MMM Group Ltd., drawing file no. P1-GENERAL ARRANGEMENT (38m).dwg, received July 17, 2010.

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.



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

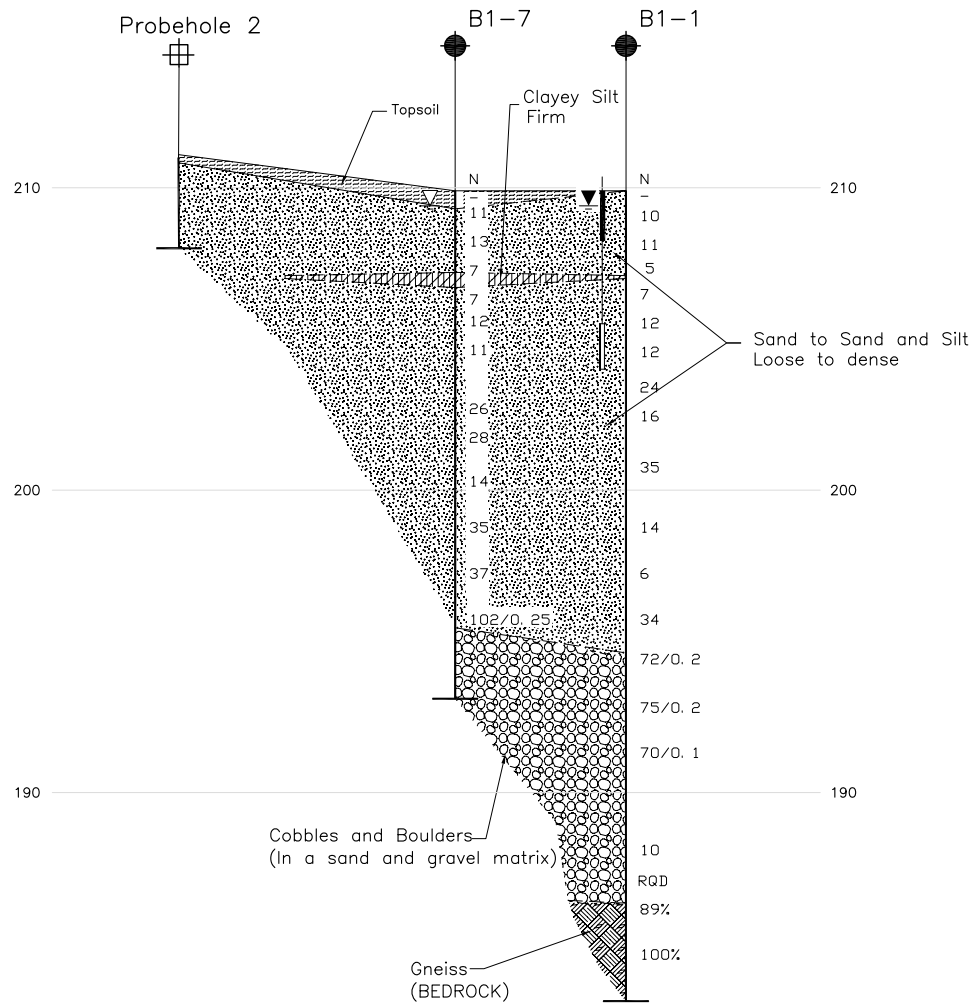
CONT No.
WP No. 5189-06-01

HIGHWAY 69
SITE 9 ROAD UNDERPASS
SOIL STRATA

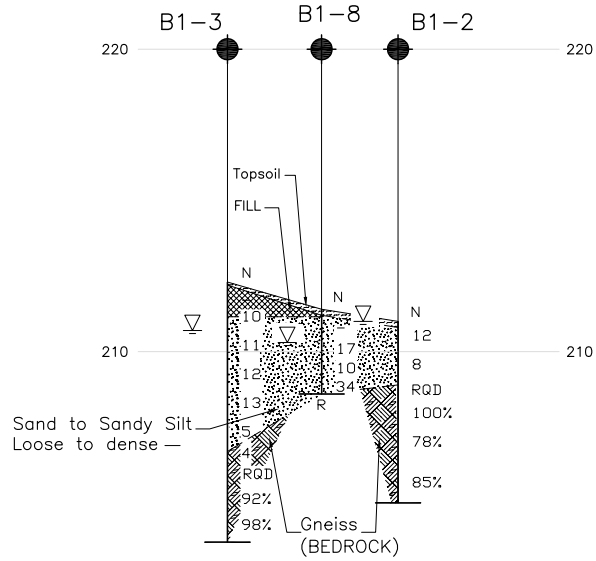
SHEET



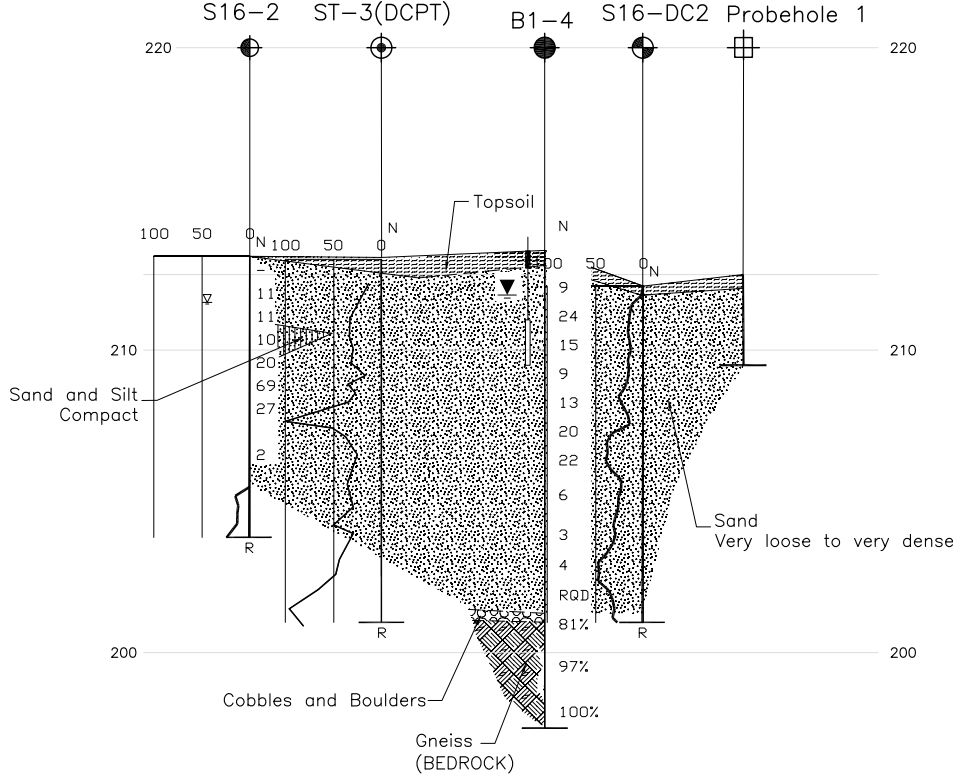
Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA



B-B' WEST ABUTMENT CROSS-SECTION
2 SITE 9 ROAD UNDERPASS
HORIZONTAL SCALE
VERTICAL SCALE



C-C' CENTRE PIER CROSS-SECTION
2 SITE 9 ROAD UNDERPASS
HORIZONTAL SCALE
VERTICAL SCALE



D-D' EAST ABUTMENT CROSS-SECTION
2 SITE 9 ROAD UNDERPASS
HORIZONTAL SCALE
VERTICAL SCALE

LEGEND

- Borehole - Current Investigation
- DCPT - Current Investigation
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- Previous DCPT (Golder, 2008)
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| B1-4 | 213.3 | 5046448.7 | 242742.1 |
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| B1-7 | 209.9 | 5046420.2 | 242665.2 |
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| B1-DC2 | 211.0 | 5046430.2 | 242706.2 |
| B1-DC3 | 212.0 | 5046436.6 | 242696.8 |
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| PROBEHOLE2 | | 5046429.2 | 242649.3 |

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| NO. | DATE | BY | REVISION |
|--------------------|--------------------------|----------------|--------------|
| Geocres No. 41H-72 | | | |
| HWY. 69 | PROJECT NO. 07-1191-0020 | | DIST. |
| SUBM'D. EC | CHKD. AB | DATE: OCT 2010 | SITE: 44-444 |
| DRAWN: MM | CHKD. SEMC | APPD. JMAC | DWG. 2 |





APPENDIX A

Record of Boreholes, Drillholes, Penetration Tests and Probeholes



LIST OF SYMBOLS

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

1. 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 |
| V | volume |
| W | weight |

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

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

(a) Index Properties (continued)

| | |
|-----------|--|
| w | water content |
| w_l | liquid limit |
| w_p | plastic limit |
| I_p | 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_a | coefficient of secondary consolidation |
| m_v | coefficient of volume change |
| c_v | coefficient of consolidation |
| T_v | time factor (vertical direction) |
| U | degree of consolidation |
| σ'_p | pre-consolidation pressure |
| 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 |
| SS | Split-spoon |
| DS | Denison type sample |
| FS | Foil sample |
| RC | Rock core |
| SC | Soil core |
| ST | Slotted tube |
| TO | Thin-walled, open |
| TP | Thin-walled, piston |
| WS | Wash sample |

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance, N_d :

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

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

Piezocone 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 (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

(b) Cohesive Soils

| Consistency | C_u, S_u |
|-------------|--------------------------------|
| | <u>kPa</u> <u>psf</u> |
| 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.



LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE

Fresh: no visible sign of weathering

Faintly weathered: weathering limited to the surface of Major discontinuities

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

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

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

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

BEDDING THICKNESS

| <u>Description</u> | <u>Bedding Plane Spacing</u> |
|---------------------|------------------------------|
| Very thickly bedded | > 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 | < 6 mm |

JOINT OR FOLIATION SPACING

| <u>Description</u> | <u>Spacing</u> |
|--------------------|----------------|
| Very wide | > 3 m |
| Wide | 1 – 3 m |
| Moderately close | 0.3 – 1 m |
| Close | 50 – 300 mm |
| Very close | < 50 mm |

GRAIN SIZE

| <u>Terms</u> | <u>Size*</u> |
|---------------------|-------------------|
| Very Coarse Grained | > 60 mm |
| Coarse Grained | 2 – 60 mm |
| Medium Grained | 60 microns – 2 mm |
| Fine Grained | 2 – 60 microns |
| Very Fine Grained | < 2 microns |

* Note: Grains > 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 varies 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 separation) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

Dip with Respect to (W.R.T.) 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 abbreviated description of the discontinuities, whether naturally occurring separation such as fractures, bedding planes and foliation planes or mechanically induced fractures 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

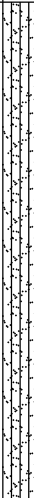



| | |
|------------------------------|----------------------|
| B - Bedding | ⊥ - Perpendicular To |
| FO - Foliation / Schistosity | - Parallel To |
| CL - Cleavage | P - Polished |
| SH - Shear Plane / Zone | K - Slickensided |
| VN - Vein | SM - Smooth |
| F - Fault | R - Rough |
| CO - Contact | ST - Stepped |
| J - Joint | PL - Planar |
| FR - Fracture | U - Undulating |
| MF - Mechanical Fracture | C - Curved |

| PROJECT 07-1191-0020 | | | | RECORD OF BOREHOLE No B1-1 | | | | 1 OF 2 METRIC | | | | | |
|----------------------|---|------------|---------|--|------------|----------------------------|--------------------|---|--|---|--|---|--|
| W.P. 5189-06-01 | | | | LOCATION N 5046414.4; E 242674.9 | | | | ORIGINATED BY EHS | | | | | |
| DIST _____ HWY 69 | | | | BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring | | | | COMPILED BY MM | | | | | |
| DATUM Geodetic | | | | DATE January 13 and 19, 2009 | | | | CHECKED BY AB | | | | | |
| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT | | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH kPa | | WATER CONTENT (%) | | | |
| 209.9 0.0 | GROUND SURFACE SAND to SAND and SILT Loose to very dense Brown Moist to wet | | 1 | AS | - | | | | | | | | |
| | | | 2 | SS | 10 | | | | | | | | 0 90 (10) |
| | | | 3 | SS | 11 | | | | | | | | |
| 207.1 | | | 4 | SS | 5 | | | | | | | | |
| 2.9 | CLAYEY SILT Brown Wet | | 5 | SS | 7 | | | | | | | | 0 49 (51) |
| | SAND to SAND and SILT Loose to very dense Brown Wet | | 6 | SS | 12 | | | | | | | | |
| | | | 7 | SS | 12 | | | | | | | | |
| | | | 8 | SS | 24 | | | | | | | | 0 52 (48) |
| | | | 9 | SS | 16 | | | | | | | | |
| | | | 10 | SS | 35 | | | | | | | | |
| | Difficult augering below 9.1 m depth. | | | | | | | | | | | | |
| 199.2 10.7 | SAND and SILT, some gravel Loose to very dense Brown Wet | | 11 | SS | 14 | | | | | | | | 13 55 (32) |
| | | | 12 | SS | 6 | | | | | | | | |
| | | | 13 | SS | 34 | | | | | | | | |
| | Switch to NW Casing, Wash Boring at 13.7 m depth. | | | | | | | | | | | | |

MIS-MTO 001 07-1191-0020 B1 BH LOGS METRIC.GPJ GAL-MISS.GDT 06/10/10 DATA INPUT:

Continued Next Page

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

| PROJECT 07-1191-0020 | | RECORD OF BOREHOLE No B1-1 | | | | 2 OF 2 METRIC | | | | | | | | | | | |
|----------------------|---|--|--------|------|-------------------------|----------------------|---|--------------------|----|----|---|-------------------|----|--|---------------------------------------|--|------------|
| W.P. 5189-06-01 | | LOCATION N 5046414.4; E 242674.9 | | | | ORIGINATED BY EHS | | | | | | | | | | | |
| DIST HWY 69 | | BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring | | | | COMPILED BY MM | | | | | | | | | | | |
| DATUM Geodetic | | DATE January 13 and 19, 2009 | | | | CHECKED BY AB | | | | | | | | | | | |
| SOIL PROFILE | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | | PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT | | | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) | | |
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | | | "N" VALUES | SHEAR STRENGTH kPa | | | | WATER CONTENT (%) | | | | | |
| | --- CONTINUED FROM PREVIOUS PAGE --- | | | | | | 20 | 40 | 60 | 80 | 100 | W _p | W | W _L | | | |
| | | | | | | | ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED | | | | | | | | | | |
| | | | | | | | 20 | 40 | 60 | 80 | 100 | 10 | 20 | 30 | | | |
| 189.9 | SAND and SILT, some gravel Loose to very dense Brown Wet Occasional cobbles and boulders below 15.4 m depth. |  | 14 | SS | 20/0.05 | | | | | | | | ○ | | | | 16 58 (26) |
| | | | | | | | 194 | | | | | | | | | | |
| | | | | | | | 193 | | | | | | | | | | |
| | | | | | | | 192 | | | | | | | | | | |
| | | | | | | | 191 | | | | | | | | | | |
| 189 | Advanced a bi-cone from 19.5 m depth to 21.3 m depth. Gravelly silty SAND Compact Brown Wet |  | 15 | SS | 75/0.2 | | | | | | | | | | | | |
| | | | | | | | 190 | | | | | | | | | | |
| | | | | | | | 189 | | | | | | | | | | |
| | | | | | | | 188 | | | | | | | ○ | | | 27 46 (27) |
| | | | | | | | 187 | | | | | | | | | | |
| 186.3 | GNEISS (BEDROCK) Bedrock cored from 23.6 m to 26.8 m depth. For coring details refer to Record of Drillhole B1-1. |  | 16 | SS | 70/0.1 | | | | | | | | | | | | |
| | | | | | | | 186 | | | | | | | | | | |
| | | | | | | | 185 | | | | | | | | | | |
| | | | | | | | 184 | | | | | | | | | | |
| 183.1 | GNEISS (BEDROCK) Bedrock cored from 23.6 m to 26.8 m depth. For coring details refer to Record of Drillhole B1-1. |  | 17 | SS | 10 | | | | | | | | | | | | |
| | | | | | | | 183 | | | | | | | | | | |
| 26.8 | End of Borehole | | | | | | | | | | | | | | | | |
| | Notes: | | | | | | | | | | | | | | | | |
| | 1. Water level measured in open borehole at a depth of 0.5 m below ground surface (Elev. 209.4 m) upon completion of drilling. | | | | | | | | | | | | | | | | |
| | 2. Water level measured in piezometer at a depth of 0.8 m below ground surface (Elev. 209.1 m) on March 20, 2009. | | | | | | | | | | | | | | | | |

MIS-MTO 001 07-1191-0020 B1 BH LOGS METRIC.GPJ GAL-MISS.GDT 06/10/10 DATA INPUT:

PROJECT: 07-1191-0020

RECORD OF DRILLHOLE: B1-1

SHEET 1 OF 1

LOCATION: N 5046414.4 ; E 242674.9

DRILLING DATE: January 13 and 19, 2009

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 850

DRILLING CONTRACTOR: Landcore

| DEPTH SCALE METRES | DRILLING RECORD | DESCRIPTION | SYMBOLIC LOG | ELEV. DEPTH (m) | RUN No. | COLOUR % RETURN | 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. | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
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| | | | | | | | RECOVERY | | R.Q.D. % | FRACT. INDEX PER 0.3 m | DISCONTINUITY DATA | | | | | | | | | | HYDRAULIC CONDUCTIVITY K, cm/sec | | Diametral Point Load Index (MPa) | RMC -Q' AVG. | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | TOTAL CORE % | SOLID CORE % | | | B Angle | DIP w.r.t. CORE AXIS | TYPE AND SURFACE DESCRIPTION | Jr | Ja | Jn | 10 ⁻⁶ | 10 ⁻⁵ | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
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| 24 | NQ Coring 01/19/2009 | Refer to Previous Page | | 186.3 23.6 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |

UCS = 144 MPa

DEPTH SCALE

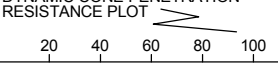
1 : 50



LOGGED: EHS

CHECKED: AB

SUD-RCK 07-1191-0020 B1 BH LOGS METRIC.GPJ GAL-MISS.GDT 06/10/10 DATA INPUT:

| PROJECT 07-1191-0020 | | | RECORD OF BOREHOLE No B1-2 | | | 1 OF 1 METRIC | | | | | |
|----------------------|---|------------|--|------|------------|----------------------------|-----------------|--|---|---------------------------------------|--|
| W.P. 5189-06-01 | | | LOCATION N 5046431.3; E 242708.4 | | | ORIGINATED BY EHS | | | | | |
| DIST _____ HWY 69 | | | BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers | | | COMPILED BY MM | | | | | |
| DATUM Geodetic | | | DATE January 8 and 9, 2009 | | | CHECKED BY AB | | | | | |
| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT  SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED | PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L WATER CONTENT (%) | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | | | | |
| 211.0 | GROUND SURFACE | | | | | | | | | | |
| 0.0 | TOPSOIL | | | | | | | | | | |
| 0.2 | Brown Moist | | | | | | | | | | |
| | SAND, trace to some silt, trace clay Loose to compact Brown Wet | | 1 | SS | 12 | | 210 | | | | |
| | | | 2 | SS | 8 | | 209 | | | | 0 92 6 2 |
| 208.9 | GNEISS (BEDROCK) | | 1 | RC | REC 100% | | 208 | | | | RQD = 100% |
| 2.1 | Bedrock cored from 2.1 m to 6.0 m depth. For coring details refer to Record of Drillhole B1-2. | | 2 | RC | REC 100% | | 207 | | | | RQD = 78% |
| | | | 3 | RC | REC 100% | | 206 | | | | RQD = 85% |
| 205.0 | End of Borehole | | | | | | 205 | | | | |
| 6.0 | Note: 1. Water level measured at ground surface upon completion of drilling. | | | | | | | | | | |

MIS-MTO 001 07-1191-0020 B1 BH LOGS METRIC.GPJ GAL-MISS.GDT 06/10/10 DATA INPUT:

PROJECT: 07-1191-0020

RECORD OF DRILLHOLE: B1-2

SHEET 1 OF 1

LOCATION: N 5046431.3 ; E 242708.4

DRILLING DATE: January 8 and 9, 2009

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 850

DRILLING CONTRACTOR: Landcore

| DEPTH SCALE METRES | DRILLING RECORD | DESCRIPTION | SYMBOLIC LOG | ELEV. DEPTH (m) | RUN No. | COLOUR % RETURN | | | | | | | | | | | | | | BR - Broken Rock | | | | | | | | | |
|--|-----------------|-----------------|--------------|-----------------------|---------|------------------------|--|-----------------------|--|-----------------------|--|------------------------------|--|-----------------------|--|-----------------------|--|--|--|----------------------------------|--|----------------------------------|--|--------------|--|--|--|--|--|
| | | | | | | JN - Joint | | BD - Bedding | | PL - Planar | | PO - Polished | | BR - Broken Rock | | K - Slickensided | | NOTE: For additional abbreviations refer to list of abbreviations & symbols. | | | | | | | | | | | |
| | | | | | | FLT - Fault | | FO - Foliation | | CU - Curved | | K - Slickensided | | SM - Smooth | | Ro - Rough | | MB - Mechanical Break | | | | | | | | | | | |
| | | | | | | SHR - Shear | | CO - Contact | | UN - Undulating | | ST - Stepped | | IR - Irregular | | MB - Mechanical Break | | MB - Mechanical Break | | | | | | | | | | | |
| VN - Vein | | OR - Orthogonal | | ST - Stepped | | IR - Irregular | | MB - Mechanical Break | | MB - Mechanical Break | | MB - Mechanical Break | | MB - Mechanical Break | | MB - Mechanical Break | | MB - Mechanical Break | | MB - Mechanical Break | | | | | | | | | |
| CJ - Conjugate | | CL - Cleavage | | CL - Cleavage | | CL - Cleavage | | CL - Cleavage | | CL - Cleavage | | CL - Cleavage | | CL - Cleavage | | CL - Cleavage | | CL - Cleavage | | CL - Cleavage | | | | | | | | | |
| FLUSH | | RECOVERY | | R.Q.D. % | | FRACT. INDEX PER 0.3 m | | B Angle | | DIP w.r.t. CORE AXIS | | TYPE AND SURFACE DESCRIPTION | | Jr | | Ja | | Jn | | HYDRAULIC CONDUCTIVITY K, cm/sec | | Diametral Point Load Index (MPa) | | RMC -Q' AVG. | | | | | |
| 100 | | 100 | | 100 | | 100 | | 100 | | 100 | | 100 | | 100 | | 100 | | 100 | | 100 | | 100 | | 100 | | | | | |
| Refer to Previous Page | | | | 208.9 | | | | | | | | | | | | | | | | | | | | | | | | | |
| GNEISS | | | | 2.1 | | | | | | | | | | | | | | | | | | | | | | | | | |
| Fine to medium grained | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Slightly weathered | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Very strong | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Pinkish grey | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Heavily jointed from 3.5 m to 3.7 m depth. | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| End of Drillhole | | | | 205.0 | | | | | | | | | | | | | | | | | | | | | | | | | |
| | | | | 6.0 | | | | | | | | | | | | | | | | | | | | | | | | | |
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DEPTH SCALE

1 : 50



LOGGED: EHS

CHECKED: AB

SUD-RCK 07-1191-0020 B1 BH LOGS METRIC.GPJ GAL-MISS.GDT 06/10/10 DATA INPUT:

| PROJECT 07-1191-0020 | | | RECORD OF BOREHOLE No B1-3 | | | 1 OF 1 METRIC | | | | | | | | | | | |
|----------------------|---|------------|--|------|------------|--|-----------------|--------------------|---|---|----------------|-------------|-------------------|--|---------------------------------------|--|-------------|
| W.P. 5189-06-01 | | | LOCATION N 5046437.7; E 242699.1 | | | ORIGINATED BY EHS | | | | | | | | | | | |
| DIST _____ HWY 69 | | | BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers | | | COMPILED BY MM | | | | | | | | | | | |
| DATUM Geodetic | | | DATE January 12, 2009 | | | CHECKED BY AB | | | | | | | | | | | |
| 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 | | | | | WATER CONTENT (%) | | | UNIT WEIGHT γ kN/m ³ | GR SA SI CL |
| | | | | | | | 20 40 60 80 100 | 20 40 60 80 100 | W _p | W | W _L | 10 20 30 | | | | | |
| 212.3 | GROUND SURFACE | | | | | | | | | | | | | | | | |
| 0.0 | TOPSOIL Brown Moist | | 1 | AS | - | | 212 | | | | | | | | | | |
| 211.2 | Sand, trace to some gravel, trace silt, trace to some organics (FILL) Compact Brown Moist to wet | | 2 | SS | 10 | | 211 | | | | | | | | | | |
| 1.1 | SAND to SANDY SILT, trace clay Loose to compact Brown Wet | | 3 | SS | 11 | | 210 | | | | | | | | | | |
| | | | 4 | SS | 12 | | 209 | | | | | | | | | | 0 80 (20) |
| | | | 5 | SS | 13 | | 208 | | | | | | | | | | 0 30 (70) |
| | | | 6 | SS | 5 | | 207 | | | | | | | | | | |
| | Approximately 0.6 m of heaving sands at a depth of 4.6 m and 4.1 m of heaving sands at a depth of 5.6 m. | | 7 | SS | 4 | | 206 | | | | | | | | | | |
| 206.7 | GNEISS (BEDROCK) | | | | | | 205 | | | | | | | | | | |
| 5.6 | Bedrock cored from 5.6 m to 8.6 m depth. For coring details refer to Record of Drillhole B1-3. | | 1 | RC | REC 100% | | 204 | | | | | | | | | | RQD = 92% |
| | | | 2 | RC | REC 100% | | | | | | | | | | | | RQD = 98% |
| 203.7 | End of Borehole | | | | | | | | | | | | | | | | |
| 8.6 | Notes: 1. Water level measured at a depth of 1.6 m below ground surface (Elev. 210.7 m) upon completion of drilling. | | | | | | | | | | | | | | | | |

MIS-MTO 001 07-1191-0020 B1 BH LOGS METRIC.GPJ GAL-MISS.GDT 06/10/10 DATA INPUT:

PROJECT: 07-1191-0020

RECORD OF DRILLHOLE: B1-3

SHEET 1 OF 1

LOCATION: N 5046437.7 ; E 242699.1

DRILLING DATE: January 12, 2009

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 850

DRILLING CONTRACTOR: Landcore

| DEPTH SCALE METRES | DRILLING RECORD | DESCRIPTION | SYMBOLIC LOG | ELEV. | RUN No. | COLOUR % RETURN | 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 | | | | | | | | |
| | | | | (m) | | | | | | | | |
| | | | | | | | | | | | | |
| DISCONTINUITY DATA | | | | | | | | | | | | |
| RECOVERY | | R.Q.D. % | FRACT. INDEX PER 0.3 m | B Angle | DIP w.r.t. CORE AXIS | TYPE AND SURFACE DESCRIPTION | Jr | Ja | Jn | HYDRAULIC CONDUCTIVITY K, cm/sec | Diametral Point Load Index (MPa) | RMC -Q AVG. |
| TOTAL CORE % | SOLID CORE % | | | | | | | | | | | |
| | | | | | | | | | | | | |
| | | Refer to Previous Page | | 206.7 | | | | | | | | |
| 6 | | GNEISS Fine to medium grained Slightly weathered Very strong Grey | | 5.6 | 1 | Grey 100% | | | | | | |
| 7 | NQ Coring 01/12/2009 | | | | | | | | | | | |
| 8 | | | | | 2 | Grey 100% | | | | | | |
| | | End of Drillhole | | 203.7 | | | | | | | | |
| 9 | | | | 8.6 | | | | | | | | |
| 10 | | | | | | | | | | | | |
| 11 | | | | | | | | | | | | |
| 12 | | | | | | | | | | | | |
| 13 | | | | | | | | | | | | |
| 14 | | | | | | | | | | | | |
| 15 | | | | | | | | | | | | |

DEPTH SCALE

1 : 50



LOGGED: EHS

CHECKED: AB

SUD-RCK 07-1191-0020 B1 BH LOGS METRIC.GPJ GAL-MISS.GDT 06/10/10 DATA INPUT:

| PROJECT 07-1191-0020 | | | | RECORD OF BOREHOLE No B1-4 | | | | 1 OF 2 METRIC | | | | | |
|----------------------|---|------------|---------|--|------------|-------------------------|-----------------|--|-----------------|---|-------------------|--|---------------------------------------|
| W.P. 5189-06-01 | | | | LOCATION N 5046448.7; E 242742.1 | | | | ORIGINATED BY EHS | | | | | |
| DIST _____ HWY 69 | | | | BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring | | | | COMPILED BY MM | | | | | |
| DATUM Geodetic | | | | DATE January 7 and 8, 2009 | | | | CHECKED BY AB | | | | | |
| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT | | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) |
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | 20 40 60 80 100 | 20 40 60 80 100 | W _p W W _L | WATER CONTENT (%) | | |
| 213.3 | GROUND SURFACE | | | | | | | | | | | | |
| 0.0 | TOPSOIL Brown Moist | | | | | | | | | | | | |
| 212.8 | SAND, trace to some silt, trace to some gravel Very loose to compact Brown Moist to wet | | 1 | SS | 9 | | | | | | | | |
| 0.5 | | | 2 | SS | 24 | | | | | | | | |
| | | | 3 | SS | 15 | | | | | | | | 0 86 (14) |
| | | | 4 | SS | 9 | | | | | | | | |
| | | | 5 | SS | 13 | | | | | | | | |
| | Approximately 0.9 m of heaving sands at a depth of 4.6 m. Switch to NW Casing, Wash Boring | | 6 | SS | 20 | | | | | | | | 10 85 (5) |
| | | | 7 | SS | 22 | | | | | | | | |
| | | | 8 | SS | 6 | | | | | | | | 0 90 9 1 |
| | | | 9 | SS | 3 | | | | | | | | |
| | | | 10 | SS | 4 | | | | | | | | |
| 201.3 | COBBLES and BOULDERS | | | | | | | | | | | | |
| 12.2 | GNEISS (BEDROCK) | | 1 | RC | REC 100% | | | | | | | | RQD = 81% |
| | Bedrock cored from 12.2 m to 15.8 m depth. For coring details refer to Record of Drillhole B1-4. | | 2 | RC | REC 100% | | | | | | | | RQD = 97% |

MIS-MTO 001 07-1191-0020 B1 BH LOGS METRIC.GPJ GAL-MISS.GDT 06/10/10 DATA INPUT:

Continued Next Page

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



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

MIS-MTO001 07-1191-0020 B1 BH LOGS METRIC.GPJ GAL-MISS.GDT 06/10/10 DATA INPUT:

PROJECT: 07-1191-0020

RECORD OF DRILLHOLE: B1-4

SHEET 1 OF 1

LOCATION: N 5046448.7 ; E 242742.1

DRILLING DATE: January 7 and 8, 2009

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 850

DRILLING CONTRACTOR: Landcore

| DEPTH SCALE METRES | DRILLING RECORD | DESCRIPTION | SYMBOLIC LOG | ELEV. DEPTH (m) | RUN No. | COLOUR % RETURN | 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. | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
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----|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|
| | | | | | | | RECOVERY | | R.Q.D. % | FRACT. INDEX PER 0.3 m | B Angle | DIP w.r.t. CORE AXIS | DISCONTINUITY DATA | | | HYDRAULIC CONDUCTIVITY K, cm/sec | | | Diameter Point Load Index (MPa) | RMC -Q AVG. | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | TOTAL CORE % | SOLID CORE % | | | | | Type and Surface Description | Jr | Ja | Jh | 10 cm | 10 cm | | | 10 cm | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH | FLUSH |

DEPTH SCALE

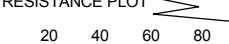

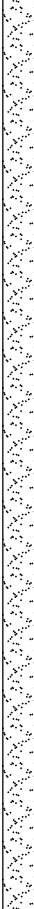
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LOGGED: EHS

CHECKED: AB

SUD-RCK 07-1191-0020 B1 BH LOGS METRIC.GPJ GAL-MISS.GDT 06/10/10 DATA INPUT:

| PROJECT 07-1191-0020 | | | RECORD OF BOREHOLE No B1-5 | | | | 1 OF 1 METRIC | | | | | |
|----------------------|---|--|--|------|------------|----------------------------|----------------------|--|---|---------------------------------------|--|-----------|
| W.P. 5189-06-01 | | | LOCATION N 5046410.0; E 242657.2 | | | | ORIGINATED BY EHS | | | | | |
| DIST _____ HWY 69 | | | BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers | | | | COMPILED BY MM | | | | | |
| DATUM Geodetic | | | DATE January 13, 2009 | | | | CHECKED BY AB | | | | | |
| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT  SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED | PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L WATER CONTENT (%) | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL | |
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | | | | | |
| 209.8 | GROUND SURFACE | | | | | | | | | | | |
| 0.0 | TOPSOIL Brown Wet |  | | | | | | | | | | |
| 209.2 | SAND to SAND and SILT Loose to compact Brown Wet |  | 1 | SS | 16 | | 209 | | | | | |
| | | | 2 | SS | 13 | | 208 | | | | | |
| | | | 3 | SS | 8 | | 207 | | | | | 0 82 (18) |
| | | | 4 | SS | 6 | | 206 | | | | | |
| | | | 5 | SS | 12 | | 205 | | | | | 0 48 (52) |
| | | | 6 | SS | 14 | | 204 | | | | | |
| | | | 7 | SS | 25 | | 203 | | | | | |
| | | | 8 | SS | 27 | | 202 | | | | | 0 57 (43) |
| | | | 9 | SS | 26 | | 201 | | | | | |
| 200.0 | End of Borehole | | | | | | 200 | | | | | |
| 9.8 | Note: 1. Water level measured at a depth of 1.6 m below ground surface (Elev. 208.2 m) upon completion of drilling. | | | | | | | | | | | |

MIS-MTO 001 07-1191-0020 B1 BH LOGS METRIC.GPJ GAL-MISS.GDT 06/10/10 DATA INPUT:

| PROJECT 07-1191-0020 | | | RECORD OF BOREHOLE No B1-6 | | | | 1 OF 1 METRIC | | | | | | |
|----------------------|--|------------|--|------|------------|-------------------------|-------------------|---|---------------------------------|-------------------------------|--------------------------------|------------------|---------------------------------------|
| W.P. 5189-06-01 | | | LOCATION N 5046462.4; E 242760.6 | | | | ORIGINATED BY EHS | | | | | | |
| DIST HWY 69 | | | BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring | | | | COMPILED BY MM | | | | | | |
| DATUM Geodetic | | | DATE January 6 and 7, 2009 | | | | CHECKED BY AB | | | | | | |
| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT <div style="display: flex; justify-content: space-around; font-size: small;"> 20 40 60 80 100 </div> | 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 | | | | | | | | |
| 213.3 | GROUND SURFACE | | | | | | | | | | | | |
| 0.0 | TOPSOIL | | | | | | | | | | | | |
| 0.2 | Brown Moist | | 1 | SS | 13 | | 213 | | | | | | |
| | SAND to SAND and SILT | | | | | | | | | | | | |
| | Loose to compact | | 2 | SS | 18 | | 212 | | | | | | |
| | Brown to grey | | | | | | | | | | | | |
| | Moist to wet | | 3 | SS | 18 | | 211 | | | | | | 0 66 (34) |
| | | | | | | | | | | | | | 16 76 (8) |
| | | | 4 | SS | 22 | | 210 | | | | | | |
| | | | 5 | SS | 9 | | 209 | | | | | | |
| | | | 6 | SS | 4 | | 208 | | | | | | |
| | Approximately 0.6 m of heaving sands at a depth of 4.6 m. Switch to NW casing, Wash Boring. | | 7 | SS | 14 | | 207 | | | | | | 29 60 (11) |
| | Gravelly sand layer 0.6 m thick at 4.6 m depth. Advanced a bi-cone from a 5.8 m depth to a 6.1 m depth. | | 8 | SS | 10 | | 206 | | | | | | |
| | | | 9 | SS | 4 | | 205 | | | | | | 0 89 (11) |
| | | | 10 | SS | 5 | | 204 | | | | | | |
| | Difficulty advancing casing at 9.9 m depth. | | 11 | SS | 11 | | 203 | | | | | | |
| 202.0 | End of Borehole | | | | | | 202 | | | | | | |
| 11.3 | Note: 1. Water level measured at a depth of 2.0 m below ground surface (Elev. 211.3 m) upon completion of drilling. | | | | | | | | | | | | |


MIS-MTO 001 07-1191-0020 B1 BH LOGS METRIC.GPJ GAL-MISS.GDT 06/10/10 DATA INPUT:

| PROJECT 07-1191-0020 | | | RECORD OF BOREHOLE No B1-7 | | | 1 OF 2 METRIC | | | | | | | |
|----------------------|---|------------|--|------|------------|-------------------------|---|--|---------------------------------|-------------------------------|--------------------------------|------------------|---------------------------------------|
| W.P. 5189-06-01 | | | LOCATION N 5046420.2; E 242665.2 | | | ORIGINATED BY EHS | | | | | | | |
| DIST HWY 69 | | | BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring | | | COMPILED BY MM | | | | | | | |
| DATUM Geodetic | | | DATE January 22 and 23, 2009 | | | CHECKED BY AB | | | | | | | |
| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | | | | | | | |
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | 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 (%) |
| 209.9 | GROUND SURFACE | | | | | | SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED | | WATER CONTENT (%) | | | | |
| 0.0 | TOPSOIL Brown Moist | | 1 | AS | - | | | | | | | | |
| 209.3 | | | | | | | | | | | | | |
| 0.6 | SAND to SAND and SILT Loose to very dense Brown Wet | | 2 | SS | 11 | 209 | | | | | | | |
| | | | 3 | SS | 13 | 208 | | | | | | | 0 81 (19) |
| | | | 4 | SS | 7 | 207 | | | | | | | |
| 207.2 | | | | | | | | | | | | | |
| 2.7 | CLAYEY SILT Firm Brown Wet | | 5 | SS | 7 | 206 | | | | | | | |
| 206.7 | | | 6 | SS | 12 | 205 | | | | | | | |
| 3.2 | SAND to SAND and SILT Loose to very dense Brown Wet | | 7 | SS | 11 | 204 | | | | | | | |
| | | | 8 | SS | 26 | 203 | | | | | | | |
| | | | 9 | SS | 28 | 202 | | | | | | | 1 52 (47) |
| | Approximately 1.2 m of heaving sands at a depth of 7.6 m. Switch to NW Casing, Wash Boring | | 10 | SS | 14 | 201 | | | | | | | |
| | | | 11 | SS | 35 | 200 | | | | | | | |
| | | | 12 | SS | 37 | 199 | | | | | | | |
| | | | 13 | SS | 102/0.25 | 198 | | | | | | | |
| | Heaving sands in casing at 9.1 m and 12.2 m depths. | | | | | 197 | | | | | | | |
| | | | | | | 196 | | | | | | | 17 72 (11) |
| | Difficulty advancing casing at 13.7 m depth due to gravelly layer. | | | | | 195 | | | | | | | |
| 195.3 | | | | | | | | | | | | | |
| 14.6 | COBBLES and BOULDERS, some sand and gravel | | | | | | | | | | | | |

MIS-MTO 001 07-1191-0020 B1 BH LOGS METRIC.GPJ GAL-MISS.GDT 06/10/10 DATA INPUT:




Continued Next Page

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

| PROJECT <u>07-1191-0020</u> | | RECORD OF BOREHOLE No B1-7 | | | | 2 OF 2 METRIC | | | | | | | | | | | | |
|--------------------------------------|--|---|--------|------|----------------------------|--------------------------|--|--------------------|--|--|--|--|-------------------|--|---|--|--|--|
| W.P. <u>5189-06-01</u> | | LOCATION <u>N 5046420.2; E 242665.2</u> | | | | ORIGINATED BY <u>EHS</u> | | | | | | | | | | | | |
| DIST <u> </u> HWY <u>69</u> | | BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring</u> | | | | COMPILED BY <u>MM</u> | | | | | | | | | | | | |
| DATUM <u>Geodetic</u> | | DATE <u>January 22 and 23, 2009</u> | | | | CHECKED BY <u>AB</u> | | | | | | | | | | | | |
| SOIL PROFILE | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | | | PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT | | | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL | | |
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | | | "N" VALUES | SHEAR STRENGTH kPa | | | | | WATER CONTENT (%) | | | | | |
| | --- CONTINUED FROM PREVIOUS PAGE --- | | | | | | <div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 20 40 60 80 100 </div> <div style="display: flex; justify-content: space-between;"> ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED </div> | | | | | <div style="display: flex; justify-content: space-between;"> W_p W W_L </div> | | | | | | |
| 193.1 | COBBLES and BOULDERS, some sand and gravel |  | | | | 194 | | | | | | | | | | | | |
| 16.8 | End of Borehole Notes: 1. Water level measured at a depth of 0.5 m below ground surface (Elev. 209.4 m) upon completion of drilling. 2. Cored through a 0.5 m thick boulder at a depth of 14.9 m. | | | | | | | | | | | | | | | | | |

| PROJECT <u>07-1191-0020</u> | | RECORD OF BOREHOLE No B1-8 | | | | 1 OF 1 METRIC | | | | | | | | | | | | |
|--------------------------------------|--|---|---------|------|------------|----------------------------|-----------------|---|----|----|----|-----|------------------------------------|-------------------------------------|-----------------------------------|--|--|--|
| W.P. <u>5189-06-01</u> | | LOCATION <u>N 5046434.2; E 242704.3</u> | | | | ORIGINATED BY <u>EHS</u> | | | | | | | | | | | | |
| DIST <u> </u> HWY <u>69</u> | | BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers</u> | | | | COMPILED BY <u>MM</u> | | | | | | | | | | | | |
| DATUM <u>Geodetic</u> | | DATE <u>January 9, 2009</u> | | | | CHECKED BY <u>AB</u> | | | | | | | | | | | | |
| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | | | PLASTIC LIMIT W _p | NATURAL MOISTURE CONTENT W | LIQUID LIMIT W _L | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL | |
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH kPa | | | | | | | | | | |
| 211.4 | GROUND SURFACE | | | | | | | 20 | 40 | 60 | 80 | 100 | | | | | | |
| 0.0 | TOPSOIL | | | | | | | | | | | | | | | | | |
| 0.2 | Brown Frozen | | 1 | AS | - | ▽ | 211 | | | | | | | | | | | |
| | SAND to Sandy SILT | | 2 | SS | 17 | | 210 | | | | | | | | | | | |
| | Compact to dense | | 3 | SS | 10 | | | | | | | | | | | | | |
| | Brown Wet | | 4 | SS | 34 | | 209 | | | | | | | | | | | |
| 208.6 | Augers grinding at 2.6 m depth. | | | | | | | | | | | | | | | | | |
| 2.8 | End of Borehole Auger Refusal | | | | | | | | | | | | | | | | | |
| | Note: 1. Water level at a depth of 1.1 m below ground surface (Elev. 210.3 m) upon completion of drilling. 2. Augers sliding to the north upon refusal. | | | | | | | | | | | | | | | | | |

MIS-MTO 001 07-1191-0020 B1 BH LOGS METRIC.GPJ GAL-MISS.GDT 06/10/10 DATA INPUT:

| PROJECT 07-1191-0020 | | | | RECORD OF BOREHOLE No S16-2 | | | | 1 OF 1 METRIC | | | | | | | | | | | | |
|----------------------|---|---|---------|--|------------|----------------------------|-----------------|--|--|--|--|--|------------------------------------|-------------------------------------|-----------------------------------|---|--|-----------|----------|--|
| W.P. 5403-05-00 | | | | LOCATION N 5046457.7; E 242724.4 | | | | ORIGINATED BY EC | | | | | | | | | | | | |
| DIST _____ HWY 69 | | | | BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers | | | | COMPILED BY MM | | | | | | | | | | | | |
| DATUM Geodetic | | | | DATE February 19, 2008 | | | | CHECKED BY AB | | | | | | | | | | | | |
| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | | | PLASTIC LIMIT W _p | NATURAL MOISTURE CONTENT W | LIQUID LIMIT W _L | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL | | | |
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH kPa | | | | | | | | | | | | |
| | | | | | | | | <div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 20 40 60 80 100 </div> <div style="display: flex; justify-content: space-between;"> ○ UNCONFINED + FIELD VANE </div> <div style="display: flex; justify-content: space-between;"> ● QUICK TRIAXIAL × REMOULDED </div> | | | | | | | | | | | | |
| 213.1 | GROUND SURFACE | | | | | | | | | | | | | | | | | | | |
| 0.0 | SAND, trace to some silt Compact Brown Wet |  | 1 | AS | - | ▽ | 213 | | | | | | | | | | 0 93 (7) | | | |
| | | | 2 | SS | 11 | | 212 | | | | | | | | | | | | | |
| | | | 3 | SS | 11 | | 211 | | | | | | | | | | | | | |
| 211.0 | SAND and SILT, trace clay Compact Brown Wet |  | 4 | SS | 10 | | 210 | | | | | | | | | | | 0 63 32 5 | | |
| 2.1 | | | 5 | SS | 20 | | | | | | | | | | | | | | | |
| | | | 6 | SS | 69 | | 209 | | | | | | | | | | | | 6 87 (7) | |
| 209.3 | SAND, trace to some silt, trace to some gravel Very loose to very dense Brown Wet |  | 7 | SS | 27 | | 208 | | | | | | | | | | | | | |
| 3.8 | | | | | | | 207 | | | | | | | | | | | | | |
| | | | 8 | SS | 2 | 206 | | | | | | | | | | | | | | |
| 205.5 | Start of DCPT Approximately 1.5 m of heaving sands measured in augers at 7.6 m depth. | | | | | | 205 | | | | | | | | | | | | | |
| 7.6 | | | | | | | 204 | | | | | | | | | | | | | |
| 203.8 | End of DCPT Refusal to Further Penetration (Hammer Bouncing) | | | | | | | | | | | | | | | | | | | |
| 9.3 | Note: 1. Water level at a depth of 1.5 m below ground surface (Elev. 211.6 m) upon completion of drilling. | | | | | | | | | | | | | | | | | | | |

MIS-MTO 001 07-1191-0020 S16 METRIC.GPJ GAL-MISS.GDT 07/10/10 DATA INPUT:



| PROJECT | | RECORD OF PENETRATION TEST | | | | No B1-DC2 | | 1 OF 1 | | METRIC | | | | | | | | | |
|---------------|--|----------------------------|--------|------|------------|--|-----------------|--------------------|--|--------------------------|--|--------------|--|----------------|--|---------------------------------------|--|-------------|--|
| W.P. | | LOCATION | | | | ORIGINATED BY | | EHS | | | | | | | | | | | |
| DIST | | BOREHOLE TYPE | | | | COMPILED BY | | MM | | | | | | | | | | | |
| DATUM | | DATE | | | | CHECKED BY | | AB | | | | | | | | | | | |
| 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 | | W _p | | W | | W _L | | γ | | GR SA SI CL | |
| 211.0 0.0 | GROUND SURFACE | | | | | | | 20 40 60 80 100 | | 10 20 30 | | | | | | | | | |
| 208.4 2.6 | End of DCPT Refusal to Further Penetration (Hammer Bouncing) | | | | | | | 20 40 60 80 100 | | 10 20 30 | | | | | | | | | |

MIS-MTO 001 07-1191-0020 B1 BH LOGS METRIC.GPJ GAL-MISS.GDT 06/10/10 DATA INPUT:



| PROJECT | | RECORD OF PENETRATION TEST | | | | No B1-DC3 | | 1 OF 1 | | METRIC | | | | | | | |
|--------------|-------|--|------------|--------|------|--|-------------------------|-----------------|--------------------|--------------------------|-------------------|--------------|---|-------------|-------------|---------------------------------------|--|
| W.P. | | LOCATION | | | | ORIGINATED BY | | EHS | | | | | | | | | |
| DIST | | BOREHOLE TYPE | | | | COMPILED BY | | MM | | | | | | | | | |
| DATUM | | DATE | | | | CHECKED BY | | AB | | | | | | | | | |
| 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 | | WATER CONTENT (%) | | γ | | GR SA SI CL | | |
| 212.0 | 0.0 | GROUND SURFACE | | | | | | | 20 40 60 80 100 | 20 40 60 80 100 | 10 20 30 | | | | | | |
| 207.7 | 4.3 | End of DCPT Refusal to Further Penetration (Hammer Bouncing) | | | | | | | | | | | | | | | |

MIS-MTO001 07-1191-0020 B1 BH LOGS METRIC.GPJ GAL-MISS.GDT 06/10/10 DATA INPUT:

RECORD OF GEOTECHNICAL PROBEHOLES
SITE 9 ROAD UNDERPASS STRUCTURE RSS WALLS
Station 18+655 and 18+730, Referenced to Hwy 69 Median C/L

07-1191-0020
July 2009

Probehole 1

18+655 35.0 Rt C/L D+1.5 PA

0 - 240 Dk Br Sa Tps
240 - 1.80 Br Sa Tr Si Tr Gr, Moist, Comp*
1.80 - 3.00 Gr Sa Tr Si Tr Gr, Wet, Fr Wat
@1.90, Sat, Comp**
- 3.00 NFP Sloughing

* Sample Depth = 1.00 – 1.30

**Sample Depth = 2.00 – 2.30

W = 22 %

Passing 150 mm = 100 %

26.5 mm = 100 %

4.75 mm = 100 %

1.18 mm = 100 %

300 µm = 82 %

75 µm = 6 %

Probehole 2

18+730 35.0 Lt C/L D-3.0 PA

0 - 250 Dk Br Sa Tps
1.00 - 3.00 Gr Sa Tr Si Tr Gr, Wet, Fr Wat
@900, Sat, Comp*
- 3.00 NFP Sloughing

*Sample Depth = 1.00 – 1.30

W = 25 %

Passing 150 mm = 100 %

26.5 mm = 100 %

4.75 mm = 100 %

1.18 mm = 99 %

300 µm = 81 %

75 µm = 5 %

**Sample Depth = 2.00 – 2.30



APPENDIX B

AMEC Record of Boreholes and Penetration Tests

| | | | |
|---|----------------------|---|-------------------|
| G.W.P. 5377-02-00 | | LOCATION Dumont Road, Township of the Archipelago, Co-ords: 5046453 N; 242742 E | ORIGINATED BY MAH |
| DIST 54 | HWY 69 | BOREHOLE TYPE Solid Stem Augering | COMPILED BY SN |
| DATUM Geodetic | DATE 30 January 2006 | | CHECKED BY IH |
| PROJECT Highway 69 Route Selection Study, from 3.5 km North of HWY 559 to 3.8 km North of HWY 522 | | | JOB NO. TT53126 |

[illegible]





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

| SOIL PROFILE | | SAMPLES | | | GROUND WATER CONDITIONS | DEPTH m | ELEVATION m | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT | | | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) | | |
|-----------------------|--|------------|--------|------|-------------------------|------------|----------------|--|----|---|----|----|--|---------------------------------------|---|----------------|
| ELEV DEPTH (m) | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | | | | "N" VALUES | 20 | 40 | 60 | 80 | | | 100 | W _p |
| | SAND brown, very loose to very dense, moist to wet | | | | | | | | | | | | | | | |
| | with silt | | 7 | SS | 10 | | | | | | | | | | | 0 70 (30) |
| | | | | | | | | | | | | | | | | |
| 201.5 | End of Borehole | | 9 | SS | 50/0 | | | | | | | | | | | |
| 12.2 201.3 12.4 | Auger refusal at 12.2 m depth Dynamic Cone Penetration Test was conducted below 12.2 m depth. Groundwater in open borehole on completion: 1.0 m End of DCPT Refusal to Dynamic Cone Penetration Test at 12.4 m depth due to possible bedrock DCPT was conducted in another location ST-3 (DCPT) located at 10 m west of ST-3. Borehole was backfilled with bentonite. | | | | | | | | | | | | | | DCPT blow count = 100 / 20 cm at 12.4 m | |

RECORD OF BOREHOLE No ST-4

G.W.P. 5377-02-00 LOCATION Dumont Road, Township of the Archipelago, Co-ords: 5046413 N; 242663 E 1 OF 2
 DIST 54 HWY 69 BOREHOLE TYPE Solid Stem Augering ORIGINATED BY MAH
 DATUM Geodetic DATE 30 January 2006 COMPILED BY SN
 PROJECT Highway 69 Route Selection Study, from 3.5 km North of HWY 559 to 3.8 km North of HWY 522 CHECKED BY IH
 JOB NO. TT53126

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | DEPTH m | ELEVATION SCALE m | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | | | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL | | | |
|----------------------|---|---|---------|------|------------|---|------------|----------------------|---|----|----|----|-----|---|--|------------------------------------|-------------------------------------|-----------------------------------|
| ELEV DEPTH (m) | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | | SHEAR STRENGTH kPa | | | | | | | WATER CONTENT (%) | | |
| | | | | | | | | | 20 | 40 | 60 | 80 | 100 | | | PLASTIC LIMIT w _p | NATURAL MOISTURE CONTENT w | LIQUID LIMIT w _L |
| 209.9 0.1 | about 75 mm TOPSOIL trace rootlets SAND some organics and rootlets in SS1, trace silt brown, loose to compact, moist to wet |  | 1 | SS | 9 |  | 1 | 209 | | | | | | | | | | |
| | some silt, trace clay | | 2 | SS | 14 | | 2 | 208 | | | | | | | | | | |
| | | | 3 | SS | 12 | | 3 | 207 | | | | | | | | | | |
| 206.0 4.0 | SILTY SAND brown, compact, moist |  | 4 | SS | 15 | | 4 | 206 | | | | | | | | | | |
| | | | 5 | | | | 5 | 205 | | | | | | | | | | |
| 204.4 5.5 | SAND trace clay brown, dense, moist to wet |  | 5 | SS | 48 | | 6 | 204 | | | | | | | | | | |
| | | | 6 | | | | 7 | 203 | | | | | | | | | | |
| | | | 6 | SS | 45 | | | 202 | | | | | | | | | | |

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

RECORD OF BOREHOLE No ST-4

| | | | |
|---|---|----------------|-------------------|
| G.W.P. 5377-02-00 | LOCATION Dumont Road, Township of the Archipelago, Co-ords: 5046413 N; 242663 E | 2 OF 2 | ORIGINATED BY MAH |
| DIST 54 HWY 69 | BOREHOLE TYPE Solid Stem Augering | COMPILED BY SN | |
| DATUM Geodetic | DATE 30 January 2006 | CHECKED BY IH | |
| PROJECT Highway 69 Route Selection Study, from 3.5 km North of HWY 559 to 3.8 km North of HWY 522 | | | JOB NO. TT53126 |

| SOIL PROFILE | | | | SAMPLES | | | GROUND WATER CONDITIONS | DEPTH m | ELEVATION SCALE m | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT | | | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) | | | | | | | |
|----------------------|--|------------|--------|---------|------------|--|----------------------------|------------|----------------------|---|----|---|----|----|---|---|----|----|--|----|----|----|---|
| ELEV DEPTH (m) | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | SHEAR STRENGTH kPa | | | | WATER CONTENT (%) | | | | | | | | | | | | | |
| | | | | | | ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE | | | | w _p w w _L | | | | | | | | | | | | | |
| | | | | | | 20 | 40 | 60 | 80 | 100 | 20 | 40 | 60 | 80 | 100 | 10 | 20 | 30 | | GR | SA | SI | CL |
| 201.9 8.1 | End of Borehole Groundwater in open borehole on completion: 1.0 m Dynamic Cone Penetration Test was conducted below 8.1 m depth. | | | | | | | | | | | | | | | | | | | | | | |
| 199.4 10.5 | End of DCPT Refusal to Dynamic Cone Penetration Test at 10.5 m depth due to possible bedrock DCPT was conducted in another location ST-4 (DCPT) located at 10 m northwest of ST-4. Borehole was backfilled with bentonite. | | | | | | | | | | | | | | | | | | | | | | DCPT blow count = 100 / 28 cm at 10.5 m |

1 OF 2

| | | |
|--|--|-------------------------|
| G.W.P. <u>5377-02-00</u> | LOCATION <u>Dumont Road, Township of the Archipelago, Co-ords: 5046453 N; 242732 E</u> | ORIGINATED BY <u>JF</u> |
| DIST <u>54</u> HWY <u>69</u> | BOREHOLE TYPE <u>Dynamic Cone Penetration</u> | COMPILED BY <u>SN</u> |
| DATUM <u>Geodetic</u> | DATE <u>22 February 2006</u> | CHECKED BY <u>IH</u> |
| PROJECT <u>Highway 69 Route Selection Study, from 3.5 km North of HWY 559 to 3.8 km North of HWY 522</u> | | JOB NO. <u>TT53126</u> |

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

RECORD OF BOREHOLE No ST-3 (DCPT)

| | | | |
|---|---|----------------|------------------|
| G.W.P. 5377-02-00 | LOCATION Dumont Road, Township of the Archipelago, Co-ords: 5046453 N; 242732 E | 2 OF 2 | ORIGINATED BY JF |
| DIST 54 HWY 69 | BOREHOLE TYPE Dynamic Cone Penetration | COMPILED BY SN | |
| DATUM Geodetic | DATE 22 February 2006 | CHECKED BY IH | |
| PROJECT Highway 69 Route Selection Study, from 3.5 km North of HWY 559 to 3.8 km North of HWY 522 | | | JOB NO. TT53126 |

| SOIL PROFILE | | | | SAMPLES | | | GROUND WATER CONDITIONS | DEPTH m | ELEVATION SCALE m | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | PLASTIC LIMIT w _p | NATURAL MOISTURE CONTENT w | LIQUID LIMIT w _L | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL | | |
|----------------------|---|------------|--------|---------|------------|--------------------|----------------------------|------------|----------------------|---|--|------------------------------------|-------------------------------------|-----------------------------------|--|--|--|--|
| ELEV DEPTH (m) | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | SHEAR STRENGTH kPa | | | | WATER CONTENT (%) | | | | | | | | |
| | | | | | | ○ UNCONFINED | | | | + FIELD VANE | | | | | | | | |
| | | | | | | ● QUICK TRIAXIAL | × LAB VANE | | | | | | | | | | | |
| | DCPT | | | | | | | | | | | | | | | | | |
| 201.0 | | | | | | | | | | | | | | | | | | |
| 12.0 | End of DCPT Refusal to Dynamic Cone Penetration Test at 12.1 m depth due to possible bedrock ST-3 (DCPT) was located at 10 m west of ST-3. | | | | | | | | | | | | | | | DCPT blow count = 100/15 cm at 12.0 m | | |



APPENDIX C

Laboratory Test Results

**TABLE C-1
UNIAXIAL COMPRESSION STRENGTH TEST RESULTS
SITE 9 ROAD UNDERPASS STRUCTURE
HIGHWAY 69, POINTE AU BARIL
GWP 5403-05-00**

| Borehole Number | Sample Depth (m) | Sample Elevation (m) | Rock Type | Core Diameter (mm) | Load (kN) | Unconfined Compressive Strength (MPa) |
|----------------------------|---------------------------------|-------------------------------------|----------------------|-------------------------------|----------------------|--|
| B1-1 | 24.7 | 185.2 | Gneiss | 48.0 | 255.1 | 144 |
| B1-2 | 3.0 | 208.0 | Gneiss | 48.0 | 228.5 | 129 |
| B1-3 | 7.6 | 204.7 | Gneiss | 48.0 | 180.7 | 102 |
| B1-4 | 12.7 | 200.6 | Gneiss | 48.0 | 113.9 | 64 |

Compiled by: EC
Checked by: SEMC

TABLE C-2
POINT LOAD STRENGTH TEST RESULTS
SITE 9 ROAD UNDERPASS STRUCTURE
HIGHWAY 69, POINTE AU BARIL
GWP 5403-05-00

| Borehole Number | Sample Depth ¹ (m) | Sample Elevation (m) | Rock Type | Test Type ² | Core Diameter (mm) | Ram Pressure (MPa) | Load (kN) | I _s Diametral ² (MPa) | I _s 50 mm ² (MPa) | Approximate UCS ² (MPa) |
|-----------------|----------------------------------|-------------------------|-----------|------------------------|-----------------------|-----------------------|--------------|--|--|------------------------------------|
| B1-1 | 24.4 | 185.5 | Gneiss | D | 48 | 20.4 | 0.019 | 8.4 | 8.2 | 165 |
| B1-1 | 25.3 | 184.6 | Gneiss | D | 48 | 19.7 | 0.019 | 8.1 | 8.0 | 159 |
| B1-1 | 26.2 | 183.7 | Gneiss | D | 48 | 18.5 | 0.018 | 7.6 | 7.5 | 150 |
| B1-2 | 2.6 | 208.4 | Gneiss | D | 48 | 12.9 | 0.012 | 5.3 | 5.2 | 104 |
| B1-2 | 4.8 | 206.2 | Gneiss | D | 48 | 15.1 | 0.014 | 6.2 | 6.1 | 122 |
| B1-2 | 5.5 | 205.5 | Gneiss | D | 48 | 24.4 | 0.023 | 10.0 | 9.9 | 197 |
| B1-3 | 5.9 | 206.4 | Gneiss | D | 48 | 9.6 | 0.009 | 3.9 | 3.9 | 77 |
| B1-3 | 6.8 | 205.5 | Gneiss | D | 48 | 11.9 | 0.011 | 4.9 | 4.8 | 96 |
| B1-3 | 8.0 | 204.3 | Gneiss | D | 48 | 6.2 | 0.006 | 2.5 | 2.5 | 50 |
| B1-4 | 13.0 | 200.3 | Gneiss | D | 48 | 14.6 | 0.014 | 6.0 | 5.9 | 118 |
| B1-4 | 13.4 | 199.9 | Gneiss | D | 48 | 16.4 | 0.016 | 6.8 | 6.6 | 133 |
| B1-4 | 14.9 | 198.4 | Gneiss | D | 48 | 16.6 | 0.016 | 6.8 | 6.7 | 134 |

NOTES:

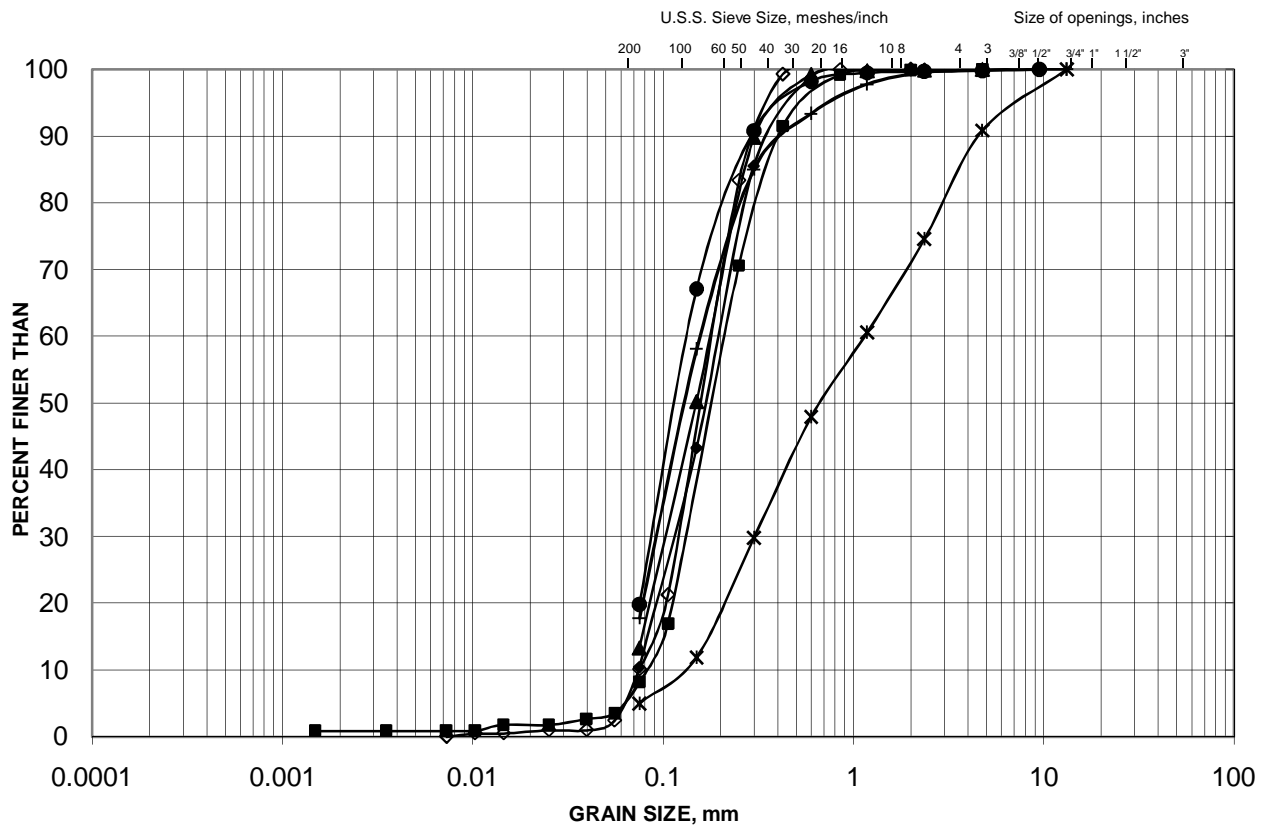
1. Depths are given below the ground surface at the borehole location.
2. Where: D = Diametral test;
I_s Diametral = Uncorrected point load strength;
I_s 50 mm = Corrected point load strength; and
UCS = Uniaxial compressive strength = I_s 50 mm X K. A value of 20 has been used, based on correlation with UCS for this site as per ASTM D5731-08 (cross-referenced to "Suggested Methods for Determining Point Load Strength", International Society for Rock Mechanics Commission on Testing Methods, Int. J. Rock Mech. Sci. and Geomechanical Abst., Vol 22, No. 2, 1985, pp. 51-60).

Compiled by: EC
Checked by: SEMC

GRAIN SIZE DISTRIBUTION

Sand

FIGURE C-1



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

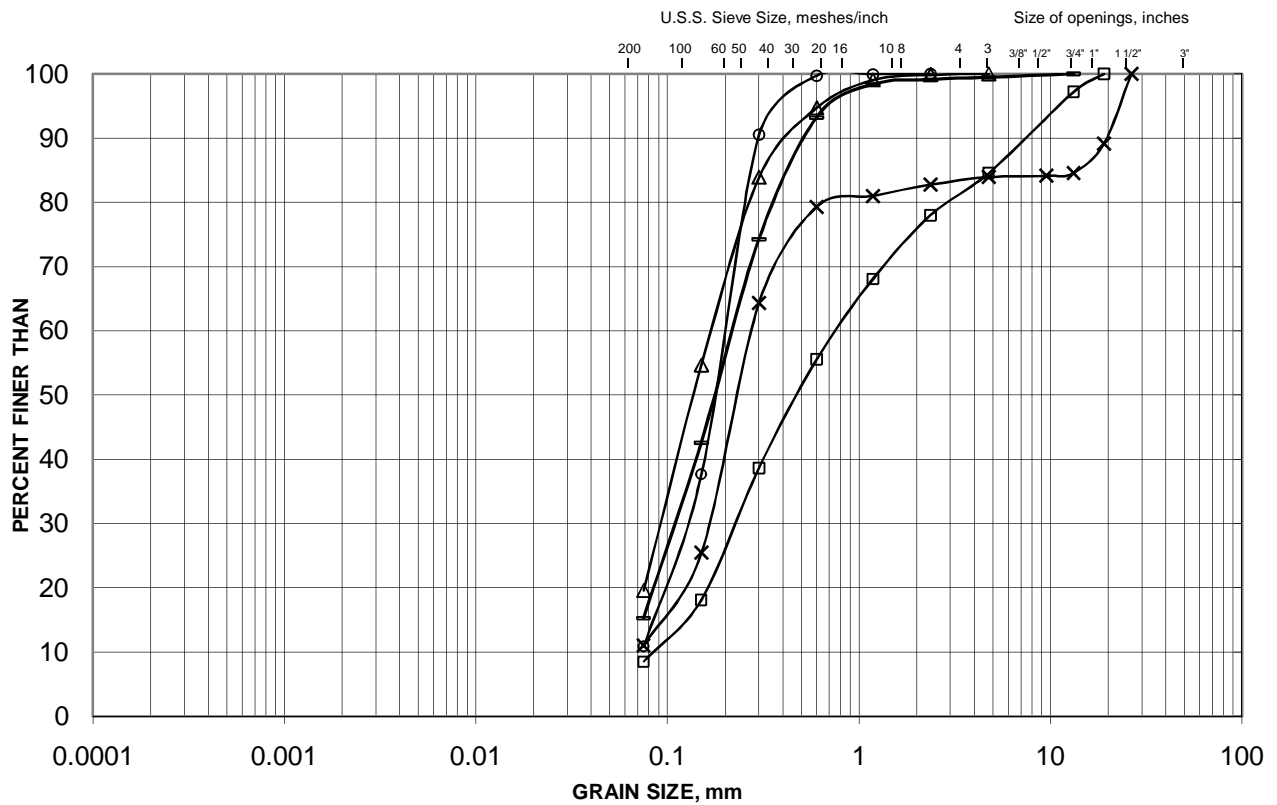
LEGEND

| SYMBOL | BOREHOLE | SAMPLE | ELEVATION (m) |
|--------|----------|--------|---------------|
| ◆ | B1-1 | 2 | 208.8 |
| ■ | B1-2 | 2 | 209.2 |
| ● | B1-3 | 4 | 209.7 |
| ▲ | B1-4 | 3 | 210.7 |
| ✱ | B1-4 | 6 | 208.4 |
| ◇ | B1-4 | 8 | 205.4 |
| + | B1-5 | 3 | 207.2 |

GRAIN SIZE DISTRIBUTION

Sand (cont'd)

FIGURE C-1



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

LEGEND

| SYMBOL | BOREHOLE | SAMPLE | ELEVATION (m) |
|--------|----------|--------|---------------|
| □ | B1-6 | 3b | 211.3 |
| ○ | B1-6 | 9 | 205.4 |
| △ | B1-7 | 3 | 208.1 |
| × | B1-7 | 13 | 195.9 |
| — | B1-8 | 3 | 209.6 |

Project Number: 07-1191-0020-B1

Checked By: SEMC

DRAFT

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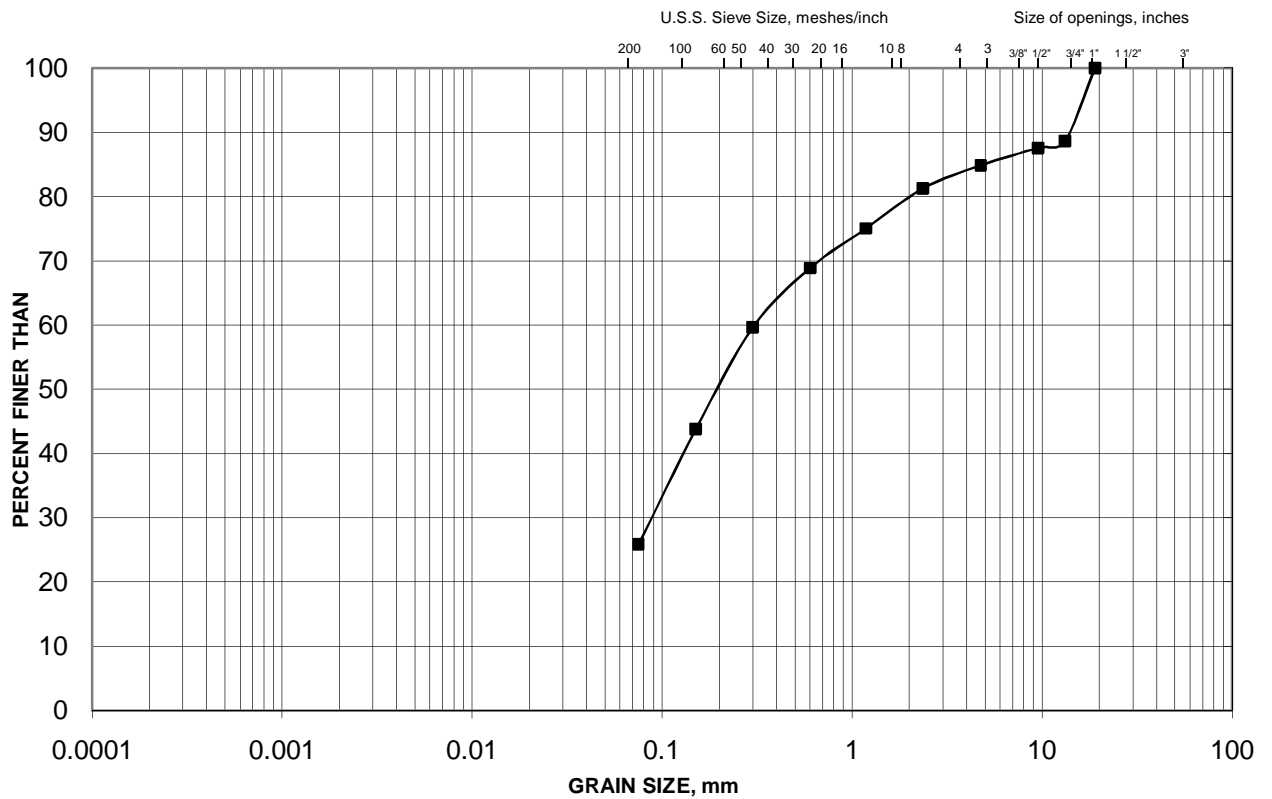
Page 2 of 2

Date: October 2010

GRAIN SIZE DISTRIBUTION

Silty Sand

FIGURE C-2



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

LEGEND

| SYMBOL | BOREHOLE | SAMPLE | ELEVATION (m) |
|--------|----------|--------|---------------|
| —■— | B1-1 | 14 | 194.6 |

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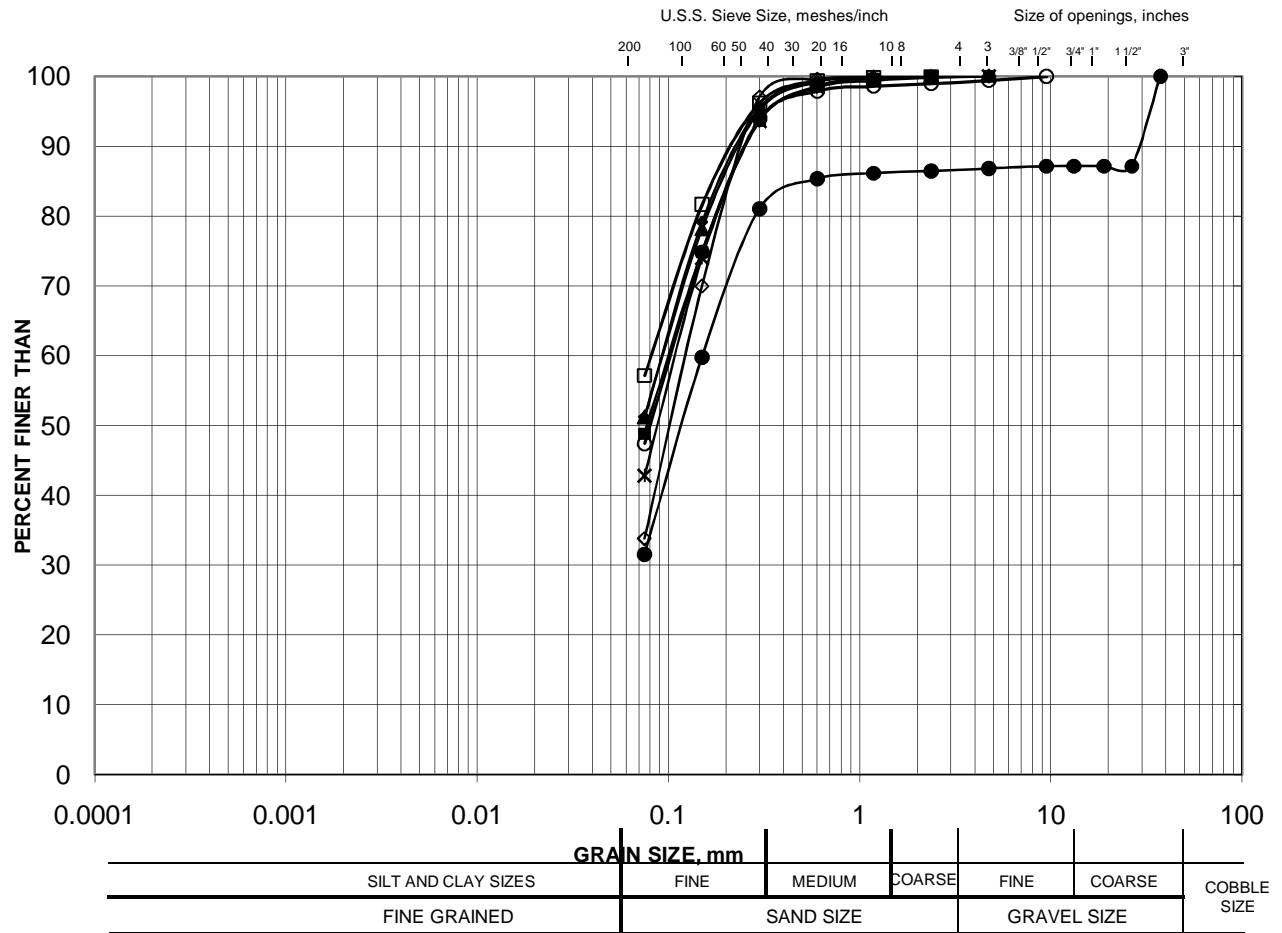
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GRAIN SIZE DISTRIBUTION

Sand and Silt

FIGURE C-3



LEGEND

| SYMBOL | BOREHOLE | SAMPLE | ELEVATION (m) |
|--------|----------|--------|---------------|
| —◆— | B1-1 | 5 | 206.6 |
| —■— | B1-1 | 8 | 203.5 |
| —●— | B1-1 | 11 | 198.9 |
| —▲— | B1-5 | 6 | 204.9 |
| —*— | B1-5 | 8 | 201.9 |
| —◇— | B1-6 | 3a | 211.6 |
| —□— | B1-7 | 5b | 206.5 |
| —○— | B1-7 | 9 | 202.0 |

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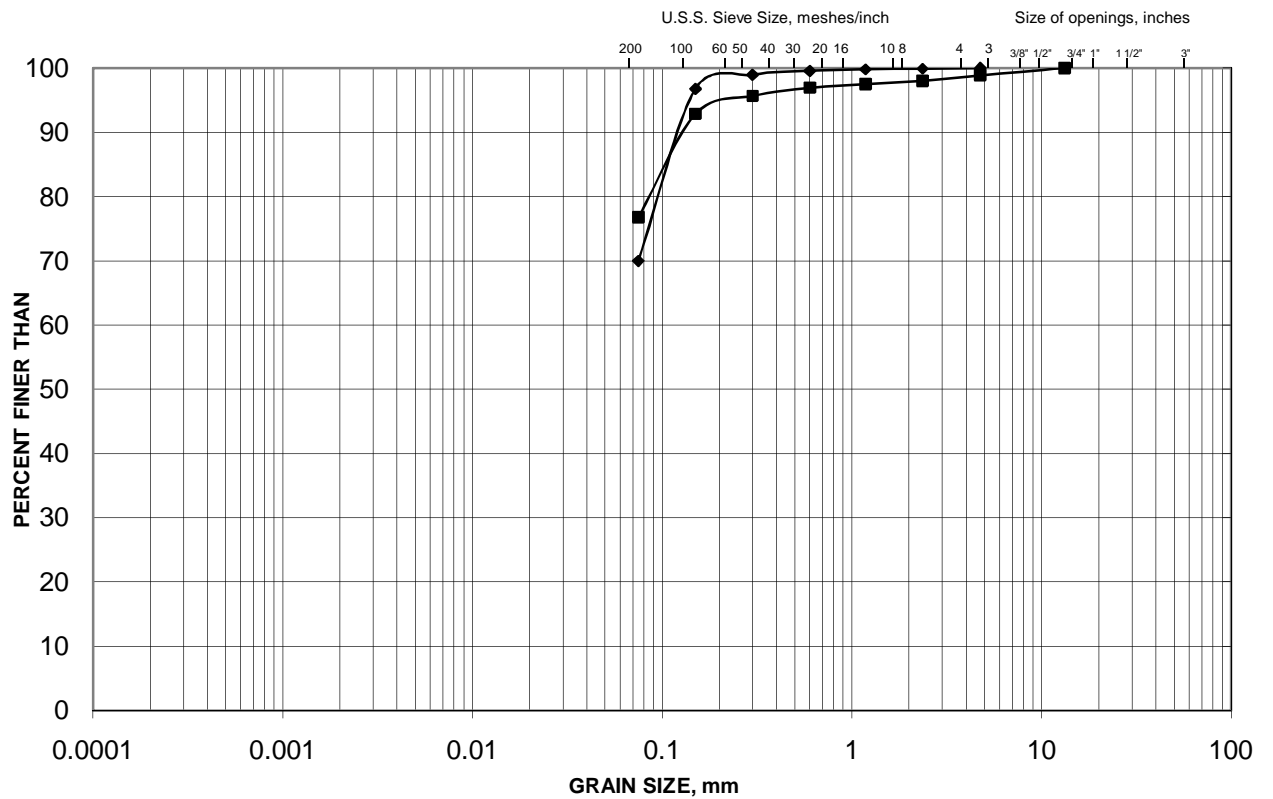
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GRAIN SIZE DISTRIBUTION

Sandy Silt

FIGURE C-4



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

LEGEND

| SYMBOL | BOREHOLE | SAMPLE | ELEVATION (m) |
|--------|----------|--------|---------------|
| ◆ | B1-3 | 6 | 208.2 |
| ■ | B1-8 | 4 | 208.8 |

Project Number: 07-1191-0020-B1

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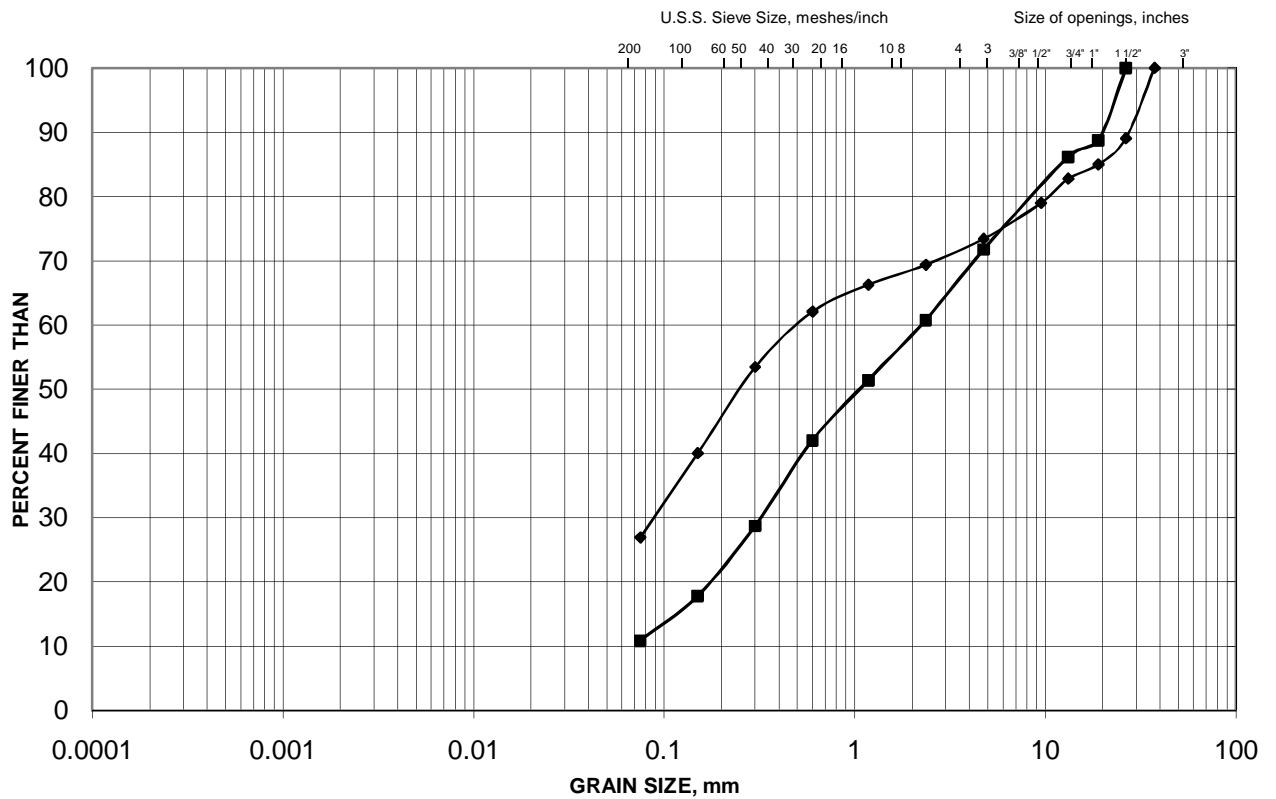
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GRAIN SIZE DISTRIBUTION

Gravelly Sand

FIGURE C-5



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

LEGEND

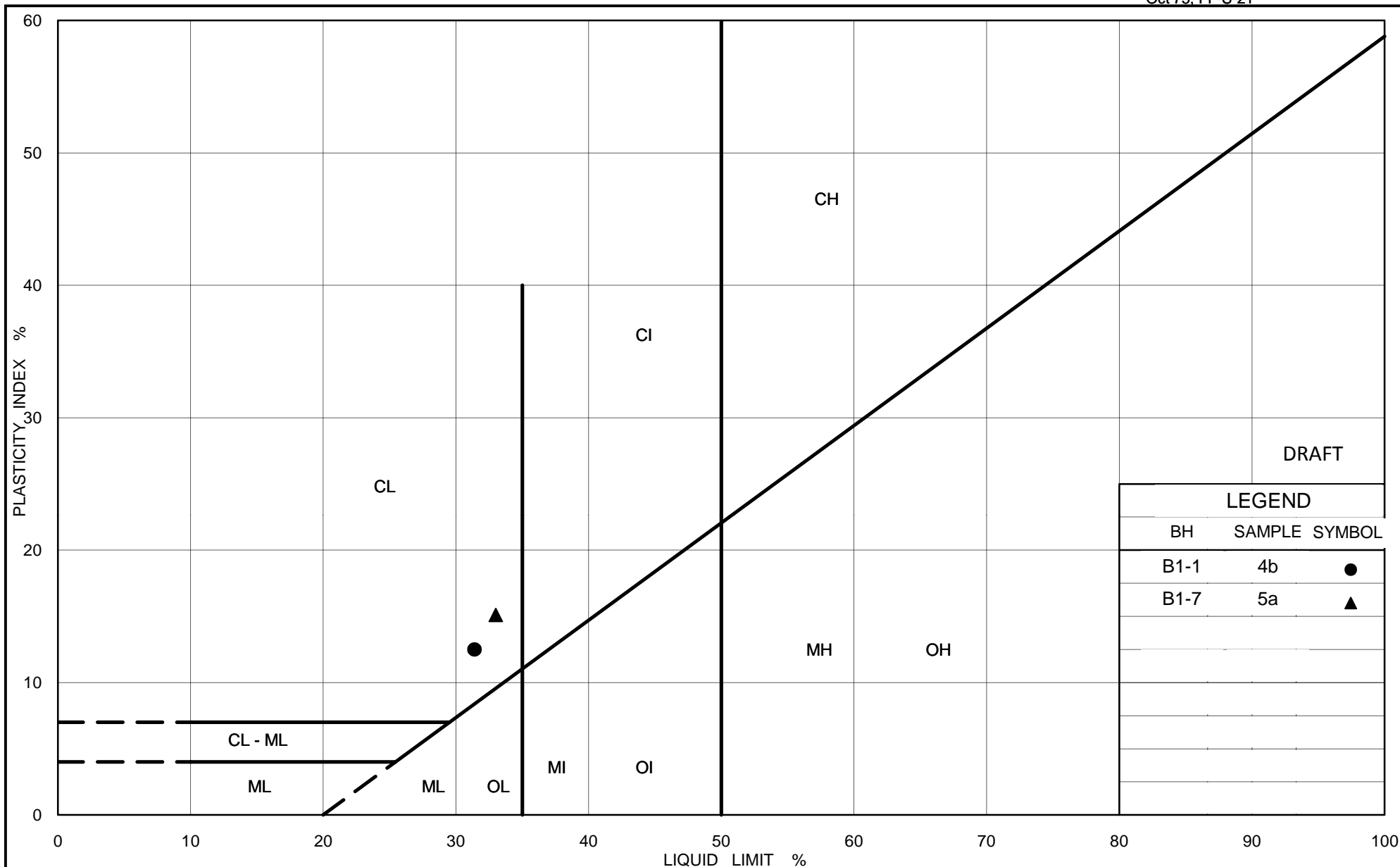
| SYMBOL | BOREHOLE | SAMPLE | ELEVATION (m) |
|--------|----------|--------|---------------|
| ◆ | B1-1 | 17 | 188.3 |
| ■ | B1-6 | 7 | 208.4 |

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Ministry of Transportation

Ontario

PLASTICITY CHART Clayey Silt

Figure C-6

Project No. 07-1191-0020-B1

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APPENDIX D

Non-Standard Special Provisions and Operational Constraints

Special Provision

SCOPE

This specification covers the requirements for the installation of the corrugated steel pipes (CSPs) at the integral abutments.

SUBMISSION AND DESIGN REQUIREMENTS

All submissions shall bear the seal and signature of an Engineer.

At least two weeks prior to commencement of installation of the abutment piles, the Contractor shall submit to the Contract Administrator, for information purposes only, three (3) sets of the working drawings.

The Contractor shall have a copy of the submitted working drawings on site at all times. Working drawings shall include at least the following:

1. Layout and elevations of the CSPs;
2. Location of reference points, and location of the centroid of each pile with respect to the reference points;
3. Construction sequence and details;
4. Source of the sand fill, and description of placing methods and equipment;
5. Location and details of all temporary bracing and spacers for the piles and CSPs;
6. Method for preventing water and debris from entering the CSP prior to placing sand; and
7. Method for preventing concrete from abutment pours from entering the CSPs during placement.

The Contractor shall be responsible for the complete detailed design of all temporary bracing, including spacers required to maintain the piles, CSP spacing and abutment stems in their specified positions through all stages of construction until the CSPs have been backfilled. All temporary bracing shall be removed.

MATERIAL

Corrugated Steel Pipe

CSP shall be in accordance with OPSS 1801, and shall be from a supplier listed under DSM#4.60.80. The CSP shall be of the diameter and wall thickness specified on the Contract drawings, and shall be galvanized in accordance with CSA G164-M.

CSP FOR INTEGRAL ABUTMENTS – Item No.

Special Provision

CSPs shall be supplied in the lengths and with the end treatments, either square or skew, as specified on the Contract drawings; field cutting and splicing of CSPs will not be permitted. Cut ends shall be neat and free of burrs. The planes defined by the end treatments of each CSP shall be parallel to each other.

Handling and storage of CSPs shall be in accordance with the manufacturer's recommendations. Damaged CSPs shall be rejected. Localized areas of damaged galvanizing on otherwise acceptable CSPs shall be repaired with two coats of zinc-rich paint.

Sand Fill

The sand fill for backfilling the CSP shall meet the gradation requirements of Table 1 below:

Table 1 – Sand Fill Gradation Requirements

| MTO Sieve Designation | | Percentage Passing by Mass |
|------------------------------|------|-----------------------------------|
| 2 mm | #10 | 100% |
| 600 µm | #30 | 80% to 100% |
| 425 µm | #40 | 40% to 80% |
| 250 µm | #60 | 5% to 25% |
| 150 µm | #100 | 0% to 6% |

CONSTRUCTION

The sequence of construction shall be in accordance with the working drawings and as follows, unless otherwise approved:

1. Form concrete levelling pad and place CSPs and spacers.
2. Construct concrete levelling pads.
3. Install piles by driving to bedrock.
4. Place loose sand into 600 diameter CSP.
5. Remove temporary spacers.

The CSP shall be positioned such that the piles are centrally positioned within the CSP. Temporary blocking and bracing shall be used to hold the CSP in position.

CSP FOR INTEGRAL ABUTMENTS – Item No.

Special Provision

The Contractor shall ensure the full perimeters of the tops of all CSPs at each abutment are at the elevation and orientation shown on the working drawings.

The CSP at each pile shall be constructed to the following tolerances:

| <u>Criteria</u> | <u>Tolerance</u> |
|--|-------------------------|
| Maximum deviation of CSP from pile centroid | +/- 50 mm |
| Maximum deviation of any point on the top perimeter of the CSP from the specified elevation | +/- 10 mm |

The sand fill shall be placed dry of optimum and free-flowing, completely filling the volume between the CSP and pile. No additional compaction effort other than the action of placing the sand itself shall be applied to the sand fill.

The placing of the sand fill shall be carried out in a manner such as to not damage and displace the CSP.

BASIS OF PAYMENT

Payment at the contract price for the above tender item shall include all labour, equipment and material required to do the work.

Operational Constraint – Obstructions

At the Site 9 Road structure, the cohesionless soils were noted to contain cobbles and boulders and a layer of cobbles and boulders was encountered overlying the bedrock at some borehole locations. Consideration of the presence of these obstructions must be made in the selection of appropriate equipment and procedures for piling for deep foundations and for augering/drilling through these materials.

At Golder Associates we strive to be the most respected global group of companies specializing in ground engineering and environmental services. Employee owned since our formation in 1960, we have created a unique culture with pride in ownership, resulting in long-term organizational stability. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees now operating from offices located throughout Africa, Asia, Australasia, Europe, North America and South America.

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