



December 14, 2011

FOUNDATION INVESTIGATION AND DESIGN REPORT

**SUCKER CREEK SBL BRIDGE, SITE 44-447/2
HIGHWAY 69 FOUR-LANING FROM 0.4 KM NORTH OF HIGHWAY 7182
(SHEBESHEKONG ROAD) NORTHERLY 11 KM
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5005-08-00, WP 5194-06-01**

Submitted to:
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REPORT





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PART A

FOUNDATION INVESTIGATION REPORT
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HIGHWAY 69 FOUR-LANING FROM 0.4 KM
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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by MMM Group (MMM) on behalf of Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the proposed Highway 69 southbound lane (SBL) bridge crossing Sucker Creek. This project is part of the detail design for the four-laning of Highway 69 from 0.4 km north of Highway 7182 (Shebeshekong Road) northerly for 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 (RFP) dated March 28, 2007. Golder's proposal (P7-1191-0020, dated April 24, 2007) for foundation engineering services associated with the new SBL 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 General Arrangement (GA) Drawing for the proposed Sucker Creek SBL structure was provided to Golder by MMM.

This report addresses the investigation carried out for the Highway 69 SBL bridge structure crossing Sucker Creek and the associated approach embankments. Separate reports detail the foundation investigations for the related NBL bridge structure, 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 location, including the associated approach embankments, by borehole drilling, rock coring and laboratory testing on selected soil and rock core samples. The investigated areas are shown on Drawing 1.

2.0 SITE DESCRIPTION

The proposed SBL bridge crossing Sucker Creek is a 97 m long 3-span structure, located in the Township of Harrison along the new Highway 69 alignment, about 1.5 km south of Highway 529 and about 400 m east of the existing Highway 69 alignment. The proposed grade at the new Highway 69 south and north approach embankments will be at about Elevation 201 m and 203 m, respectively, which is up to about 9 m above the existing ground surface at the south approach and about 16 m above the existing ground surface at the north approach.

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. At the crossing location, Sucker Creek is about 15 m wide, situated in a valley more than 100 m wide. The ground surface at the borehole locations within the limits of the proposed structure and approach embankment areas is between Elevation 183.6 m and Elevation 192.6 m.



3.0 INVESTIGATION PROCEDURES

The fieldwork for the investigation at the location of the proposed structure was carried out in two stages and included the drilling of a total of fifteen (15) boreholes and one (1) Dynamic Cone Penetration Test (DCPT) at approximately the locations shown on Drawing 1. The field investigation was carried out as follows:

- Between February 10 and February 24, 2009, the following boreholes and DCPT were advanced using a track mounted CME-55 supplied and operated by Landcore Drilling Ltd. (Landcore) of Sudbury, Ontario:
 - Five (5) boreholes at the south abutment (Boreholes B5-1 to B5-5);
 - One (1) borehole at the south pier (Borehole B5-6);
 - One (1) borehole at the north pier (Borehole B5-7);
 - Two (2) boreholes and one (1) DCPT at the north abutment (Boreholes B5-10 and B5-11 and DCPT B5-DC1); and
 - One (1) borehole for each of the south and north approach embankments (Boreholes B5-13 and B5-14, respectively).
- On March 16 and March 17, 2009, four (4) boreholes (Boreholes B5-6a to B5-6d) were drilled at the south pier using portable equipment, supplied and operated by OGS Inc. (OGS) of Ottawa, Ontario.

The boreholes were advanced using 108 mm inside diameter (I.D.) continuous flight hollow stem augers, 'NW' casing and wash boring or portable equipment using 'BW' or 'NW' casing and wash boring. Soil samples were obtained, where possible, at intervals of depth of 0.75 m to 1.5 m, using a 50 mm outer diameter (O.D.) split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586). Rock core samples were obtained in 'NQ' size using either an 'NQ'-size core barrel or a thin-walled core barrel which fits inside 'BW' casing. The groundwater conditions in the open boreholes were observed during the drilling operations. All boreholes were backfilled with bentonite upon completion in accordance with Ontario Regulation (O.Reg.) 903 (as amended).

The boreholes were advanced to auger/split-spoon refusal or cored into the bedrock, to depths ranging from 0.1 m to 21.8 m below existing ground surface or casing and included coring bedrock for lengths of between 2.9 m and 3.9 m in Boreholes B5-1, B5-3, B5-5, B5-6, B5-6b, B5-6c, B5-7 and B5-10.

A piezometer was installed in Borehole B5-11 to permit monitoring of the groundwater level at this location. The piezometer consists of 51 mm diameter PVC pipe, with a 1.5 m long slotted screen sealed at a selected depth within the borehole. The borehole annulus surrounding the piezometer screen was backfilled to Elevation 186.2 m with sand and sand cuttings from the borehole and a bentonite seal was installed between Elevation 186.2 m and 187.1 m. The piezometer was abandoned in accordance with Ontario Regulation 903 on March 3, 2009. The piezometer installation details and water level readings are described on the Record of Borehole sheets in Appendix A. The fieldwork was supervised throughout by members of our engineering and technical staff who: located the boreholes 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



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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. A consolidated drained direct shear test was performed on a sample of the sand from Borehole B507 for determination of the effective angle of internal friction (ϕ'). Strength testing (uniaxial compression and point load index) was also carried out on selected specimens of the rock core.

The centreline of Highway 69 was surveyed and staked in the field by MMM in September 2009, and the borehole locations were staked by MMM in December 2008, and February 2009. Where boreholes were relocated from the original staked locations, Golder resurveyed and located the new borehole relative to MMM's stakes. 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		
B5-1	5051523.5	236836.8	192.2	4.7
B5-2	5051528.4	236835.7	191.5	0.7
B5-3	5051524.8	236831.4	191.5	5.0
B5-4	5051521.3	236827.0	190.4	1.1
B5-5	5051526.2	236825.9	190.5	5.2
B5-6	5051554.1	236824.8	185.2	3.8
B5-6a	5051550.6	236820.5	185.0	0
B5-6b	5051555.5	236819.4	183.6	3.4
B5-6c	5051552.8	236830.2	185.9	5.5
B5-6d	5051557.6	236829.1	184.9	2.1
B5-7	5051589.2	236816.9	184.0	21.8
B5-10	5051618.5	236810.3	187.3	20.1
B5-11	5051615.0	236806.0	186.9	14.0
B5-13	5051510.2	236834.7	192.6	0.7
B5-14	5051663.1	236807.1	188.8	2.1
B5-DC1	5051622.0	236814.7	187.8	9.6

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

As delineated in The Physiography of Southern Ontario (Chapman and Putnam, 1984)¹, 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.

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.



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 4² (OGS, 1991). Deposition of Paleozoic strata initially covered the bedrock and later erosion during glaciation exposed these Precambrian rocks.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions, as encountered in the boreholes advanced during this investigation, together with the results of the laboratory tests carried out on selected soil and rock samples, are presented on the Record of Borehole and Drillhole sheets in Appendix A. The results of the laboratory tests carried out on selected soil and rock samples are presented in Appendix B. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling and observations of drilling progress and the results of SPT. 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, on the south side of the creek, a surficial layer of topsoil or fill materials is underlain by thin deposits of sandy silt to silty sand or silty clay, underlain by bedrock at shallow depths. On the north side of the creek, a surficial layer of topsoil is underlain by deposits of sand and sand and gravel. The total thickness of overburden is variable at the site, ranging from approximately 0 m to 2.1 m on the south side of the creek and from approximately 2.1 m to 18.7 m on the north side of the creek.

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

4.2.1 Fill

In Boreholes B5-5, B5-6c and B5-6d, a deposit of moist to wet, brown fill consisting of sandy organics and/or sand and gravel was encountered at ground surface. The surface of the fill was encountered between Elevation 190.5 and 184.9 m and the thickness of the fill ranges between 0.5 m and 2.1 m.

4.2.2 Organics/Topsoil

A deposit of moist, brown organics or topsoil was encountered at ground surface at all the boreholes except B5-6b to B5-6d. The top of the organics layers were encountered between Elevation 192.6 m and 184.0 m and the thickness of the deposit ranges between 0.1 m and 0.8 m. The topsoil in Borehole B5-6 was noted to contain cobbles and boulders.

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



4.2.3 Sandy Silt to Silty Sand

A deposit of moist to wet, brown sandy silt to silty sand was encountered below the topsoil in Boreholes B5-1 to B5-5. The surface of the deposit was encountered between Elevation 192.0 m and 189.7 m and ranges in thickness from 0.5 m to 1.4 m. The deposit was noted to contain cobbles in Borehole B5-5 below a depth of 1.6 m.

The measured SPT 'N'-values within the sandy silt to silty sand range between 2 and 30 blows per 0.3 m of penetration, with a value of 19 blows per 0.1 m of penetration, indicating a very loose to compact relative density.

4.2.4 Silty Clay

A stratum of moist, brown silty clay was encountered below the topsoil in Borehole B5-13. The surface of this layer was encountered at Elevation 192.4 m, and its thickness is 0.5 m.

A measured SPT 'N'-value at the base of the stratum is 6 blows per 0.15 m of penetration, suggesting a stiff consistency.

An Atterberg limits test carried out on the sample of the silty clay yielded a liquid limit of 39 percent, a plastic limit of 24 percent and a plasticity index of 15 percent. The results of the Atterberg limits testing are shown on the plasticity chart on Figure B-1 in Appendix B and indicate that the material is a silty clay of intermediate plasticity.

The natural moisture content of the sample is 36 percent.

4.2.5 Sand to Gravelly Sand

On the south side of the creek, a deposit of moist, brown gravelly sand was encountered below the topsoil in Borehole B5-3. On the north side of the creek, a deposit of moist to wet, brown to grey sand, trace to some gravel, trace to some silt was encountered below the organics/topsoil in Boreholes B5-7, B5-10, B5-11 and B5-14. The deposit contained occasional cobbles in Borehole B5-7 and cobbles and boulders in Borehole B5-10. On the south side of the creek, the gravelly sand deposit was encountered at Elevation 190.5 m and the thickness is 1.1 m. On the north side of the creek, the surface of the deposit was encountered between Elevation 188.6 m and 183.8 m and the thickness of the deposit ranges between 1.9 m and 11.4 m.

The measured SPT 'N'-values within the sand/gravelly sand deposit range between 0 blows (i.e. weight of hammer) and greater than 100 blows per 0.3 m of penetration. Typically, the 'N'-values range between 11 and 26 blows per 0.3 m of penetration, indicating a compact relative density, and the presence of very loose or very dense layers.

The grain size distributions of eleven samples of the sand/gravelly sand deposit are shown on Figure B-2.

The natural water content measured on samples of the deposit range between 2 percent and 27 percent.



A laboratory consolidated drained direct shear (DS) test was carried out on one selected sample of the sand deposit from Borehole B5-7. The detailed test results are shown on Figure B-3 in Appendix B and the results are summarized below.

Borehole/Sample Number	Depth / Elevation (m)	Effective Cohesion Intercept, c' (kPa)	Effective Angle of Internal Friction, ϕ' (degrees)
B5-7/6	4.1/179.9	0	37

Note: The assessed shear strength parameters are only valid over the range of stress conditions employed in the direct shear test.

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 3.6 m and 10.7 m.

Difficult casing/auger advancement was noted near the base of the deposit in Boreholes B5-7 and B5-10, inferred to be due to the presence of gravelly material or cobbles and boulders.

4.2.6 Sand and Gravel

In Boreholes B5-7, B5-10 and B5-11, a deposit of wet, brown or grey, sand and gravel, containing cobbles and boulders was encountered below the sand to gravelly sand deposit. The surface of the deposit was encountered between Elevations 176.4 m and 172.4 m and the thickness of the deposit ranges between 3.5 m and 7.1 m.

The measured SPT 'N'-values within the sand and gravel deposit range between 71 blows per 0.3 m of penetration and 20 blows per 0.3 m of penetration, indicating a very dense relative density, and essentially refusal to split-spoon advancement at several sample depths in Borehole B5-7.

A grain size distribution test was carried out on one sample of the sand and gravel deposit in Borehole B5-10 and the results are shown on Figure B-4.

The natural moisture content of one sample of the deposit is 15 percent.

4.2.7 Refusal/Bedrock

Bedrock was encountered and cored in Boreholes B5-1, B5-3, B5-5, B5-6, B5-6b, B5-6c, B5-7 and B5-10. The bedrock surface was inferred from refusal to further auger, casing or split-spoon advancement or dynamic cone penetration in Boreholes B5-2, B5-4, B5-6a, B5-6d, B5-11, B5-13, B5-14 and DCPT B5-DC1. The bedrock surface (inferred or actual) was encountered in the boreholes at depths ranging from ground surface to 18.7 m below ground surface, ranging from Elevation 191.9 m and 165.3 m, as presented in Table B-1.



Based on a review of the bedrock core samples, the bedrock at the site consists of gneiss and the core samples are described as reddish brown or pinkish grey to grey, fine to coarse grained and fresh to slightly weathered, except in Borehole B5 7 where the bedrock is a mafic intrusive, and the core samples are greenish grey, fine to medium grained and moderately weathered.

The Total Core Recovery (TCR) is 100 percent for all core samples, except for Run No. 2 in Borehole B5-10 which had a TCR of 91 percent. The Rock Quality Designation (RQD) measured on the core samples typically ranges from about 65 percent to 100 percent, indicating a rock mass of fair to excellent quality. The uppermost core sample in Borehole B5-6c measured a RQD value of 45 percent and the lowermost core sample in Boreholes B5-7 measured a RQD value of 24, indicating a rock mass of very poor to poor quality. The RQD typically increased with depth. The Solid Core Recovery (SCR) typically ranges from 50 percent to 100 percent, typically increasing with depth. The uppermost core samples in Boreholes B5-6b and B5 6c measured a SCR value of 48 percent and 45 percent, respectively. The lowermost core sample in Boreholes B5-7 measured a SCR value of 24 percent.

Laboratory Uniaxial Compressive Strength (UCS) testing was carried out on seven core samples of the bedrock. The UCS ranges from about 81 MPa to 123 MPa for the gneiss bedrock indicating strong to very strong rock, as summarized in Table B-2.

Point load strength tests were carried out on selected core samples of the bedrock. Diametral point load strength index values are shown on the Record of Drillhole sheets and are summarized in Table B-3 in Appendix B. The diametral point load index (I_{s50}) results from the laboratory tests carried out on core samples range from about 2 MPa to 8 MPa for the gneiss bedrock. These index values correspond to estimated UCS values between 48 MPa and 178 MPa, based on a relationship between I_{s50} and UCS which is given by a correlation factor (K) in accordance with ASTM D5731-08 and 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 22 for the gneiss, which was calculated based on a comparison of the UCS test results and the point load strength test results. 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 tests and the point load strength test results, in accordance with Table 3.5 in CFEM (2006)³, the gneiss is medium strong (R3, 25 MPa < UCS < 50 MA) to very strong (R5, 100 MPa < UCS < 250 MPa).

4.2.8 Groundwater Conditions

The water levels in the boreholes were noted immediately after the drilling operations. In general, the soil samples taken in the boreholes were noted to be moist to wet. Where bedrock either was exposed or was encountered at shallow depth below ground surface, the open boreholes were dry. The water level measured in Borehole B5-5, B5-6d, B5 7, B5-10 and B5-11 is at depths ranging between 0.9 m to 5.5 m below ground surface, between Elevation 189.0 m and 181.8 m.

³ Canadian Geotechnical Society, 2006. Canadian Foundation Engineering Manual, 4th Edition.



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A standpipe piezometer was installed in Borehole B5-11 to permit monitoring of the water level at this site. Details of the piezometer installations are shown in the Record of Borehole sheets in Appendix A. The groundwater level measured in the piezometer installation is summarised below.

Foundation Element	Borehole No.	Ground Surface Elevation (m)	Groundwater Elevation (m)	Date of Measurement
North Abutment	B5-11	186.9	182.3	March 3, 2009

It should be noted that groundwater levels in the area are subject to seasonal fluctuations and precipitation events.

5.0 CLOSURE

The field personnel supervising the drilling program were Mr. Ed Savard and Mr. Trevor Moxam. This report was prepared by Mr. Adam Wissink, EIT and André Bom, P.Eng. The technical aspects were reviewed by Mr. Jorge M. A. Costa, P.Eng., Golder's Designated MTO Contact for this project, who also carried out a quality control review of the report.

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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 Highway 69 SBL structure crossing Sucker Creek. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the design of the structure foundation and approach embankments. Where comments are made on construction, they are provided in order to highlight those aspects which could affect the design of the project. Those requiring information on the aspects of construction should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

6.1 General

The new Highway 69 SBL structure crossing Sucker Creek will be a three-span structure, comprised of a 35 m long centre span and 31 m long south and north spans. Based on the GA drawing dated August 2011, the proposed Highway 69 SBL grade is at Elevation 200.8 m and 202.5 m at the south and north abutments, respectively. The ground surface in the south approach area is generally sloping downwards to the north towards the creek and ranges from about Elevation 193 m about 20 m south of the abutment to about Elevation 191.0 m at the abutment. Similarly, the ground surface in the north approach area is generally sloping downwards to the south towards the creek and ranges from about Elevation 189.0 m, about 20 m north of the abutment to about Elevation 187.0 m at the abutment. The proposed south and north approach embankments (closest to the abutments) will be up to about 9 m and 16 m high above existing ground surface, respectively. The ground surface varies between about Elevation 183.6 m and 186.0 m at the south pier while the ground surface is at about Elevation 184.0 m at the north pier. The water level in Sucker Creek at the time of MMM's survey in December 2007, was Elevation 181.9 m. We understand that the high water level (100 year storm event) is Elevation 183.15 m.

At the south abutment, the overburden generally consists of a layer of topsoil up to 0.3 m thick underlain by a deposit of very loose to dense sand and silt to silty sand and gravelly sand up to 1.8 m thick overlying bedrock. A 0.5 m thick silty clay layer was encountered below 0.2 m of topsoil about 20 m behind the abutment. At the south approach (from the abutment to 20 m behind the abutment), the bedrock surface was encountered at depths between 0.7 m and 2.1 m below ground surface, ranging between Elevations 191.9 m and 188.3 m.

From the centre of the south pier to the west, bedrock is either exposed or covered by up to 0.8 m of topsoil. The east side of the pier is located in the existing snowmobile trail and up to 2.1 m of fill materials consisting of organics and/or sand and gravel are present overlying bedrock. The bedrock surface is variable, ranging between Elevation 182.8 m at the northeast corner of the proposed footing and Elevation 185.0 m at the southwest corner of the proposed footing.

At the north pier, a 0.2 m thick layer of topsoil is underlain by up to 11.4 m of very loose to compact sand and 7.1 m of very dense sand and gravel. The bedrock surface was encountered at a depth of 18.7 m below ground surface, which is Elevation 165.3 m.



At the north abutment, a 0.2 m to 0.3 m thick layer of topsoil/organics is underlain by between 10.4 m and 11.1 m of loose to very dense sand and 3.5 m to 5.8 m of very dense sand and gravel. The bedrock surface, cored in Borehole B5-10 and inferred from casing refusal in Borehole B5-11, was encountered at depths below ground surface of 17.1 m (Elevation 170.2 m) and 14.0 m (Elevation 172.9 m), respectively. Refusal to further penetration in DCPT B5-DC1 was encountered at 9.6 m, (Elevation 178.2 m). About 20 m north of the abutment, a 0.2 m thick layer of organics was encountered overlying 1.9 m of compact to dense sand over inferred bedrock.

6.2 Bridge Foundation Options

At the proposed south abutment, the following foundation alternatives have been considered and are technically feasible:

- A spread footing founded directly on the bedrock surface. Due to the variability of the bedrock surface (Elevation 188.3 m to 190.8 m), mass concrete or bedrock excavation would be required to level the footing area.
- A spread footing founded on a compacted granular pad constructed over the bedrock (upon removal of the overburden).
- Steel H-piles driven through an approximately 6.5 m thick compacted granular pad to the bedrock surface or socketted into the bedrock (for fixity or to achieve required pile length) for an integral abutment configuration.
- Caissons advanced through a compacted granular pad and socketted into the bedrock.

At the proposed north abutment, the following foundation alternatives were considered and are technically feasible:

- A spread footing founded on a compacted granular pad constructed over the native soil. In this case, the granular pad must be at least 2 m thick.
- Steel H-piles driven through an approximately 9.5 m thick compacted granular pad to the bedrock surface for an integral abutment configuration.
- Caissons advanced through a compacted granular pad and socketted into the bedrock.

A summary of the advantages, disadvantages, relative costs and risks/consequences for the south and north abutment foundation alternatives are summarised in Tables 1 and 2, respectively. Given that the new Sucker Creek SBL crossing will likely be an integral abutment structure, we recommend the use of steel H-piles to support the abutments. However, non-integral abutments should be considered and spread footings founded either directly on the bedrock or on a compacted granular pad over the native soils are technically feasible in this case, from a foundation perspective.

The proposed south pier should be founded on a spread footing constructed on the bedrock given that the depth to bedrock is relatively shallow at this location. Mass concrete or bedrock excavation would be required to level the footing area as the rock is sloping at this location.



At the proposed north pier, shallow foundations are not considered feasible due to the low relative density of the thick layer of cohesionless soils overlying bedrock at this location. Deep foundations consisting of steel H-piles driven to bedrock or caissons socketted into bedrock are both technically feasible alternatives. Since piles are recommended for the abutments, we recommend that steel H-piles also be used at the north pier, as the preferred foundation alternative. Table 3 summarises the foundation alternatives for the north pier.

The following sections provide further discussion and design recommendations for shallow and deep foundations as discussed above.

6.3 Shallow Foundations

We recommend supporting the proposed south pier on a spread footing placed on properly prepared gneiss bedrock. The south abutment could also be founded on a spread footing placed directly on the bedrock surface. The bedrock surface elevation at the proposed foundation elements are summarized below.

Foundation Element	Relevant Borehole Numbers	Bedrock Surface Elevation (m)	Recommended Underside of Footing Elevation (m)
South Abutment	B5-1 to B5-5	188.3 to 190.8	190.8
South Pier	B5-6 and B5-6a to B5-6d	182.8 to 185.0	185.0

Based on the proposed footing levels, mass concrete should be used to raise the founding level to the underside of the footing level. A thickness of up to 2.5 m and 2.2 m of mass concrete would be required at the south abutment and south pier, respectively.

If a lower founding elevation is required for structural reasons, then bedrock excavation will be required. Recommendations on bedrock excavation (using controlled blasting methods, etc.) are provided in Section 6.9.1.

Spread footings founded on a properly prepared compacted granular pad could also be considered at both the south and north abutments. In this case, the granular pad would be placed directly on the bedrock surface at the south abutment and on the surface of the compact to dense sand deposit at Elevation 186.5 m at the north abutment. The granular pad should be designed and constructed as described in Section 6.8.

6.3.1 Geotechnical Resistance

Spread footings placed on the surface of the properly prepared gneiss bedrock or on mass concrete of compressive strength equal to or greater than the concrete footings (assumed to be 25 MPa or greater) may be designed based on a factored geotechnical axial resistance at Ultimate Limit States (ULS) of 10,000 kPa. The geotechnical resistance at Serviceability Limit States (SLS) for 25 mm of settlement will be greater than the factored geotechnical axial resistance at ULS, since bedrock (or mass concrete) is considered to be an unyielding material and, as such, ULS conditions will govern for this foundation type.



All loose, shattered and/or fractured rock within the footprint at the founding level should be removed and replaced with mass concrete. MTO Special Provision (SP) 902S01 should be included in the Contract Documents to address the requirements for construction and inspection of footings on bedrock.

For spread footings placed on a minimum 2 m thick compacted granular pad, a factored geotechnical axial resistance at ULS of 700 kPa may be used. A corresponding SLS value of 350 kPa may be used assuming a 2 m to 3 m wide footing. These resistance values assume the granular pad is placed in dry conditions directly over the surface of the bedrock at the south abutment and on the native sand deposit founded no higher than Elevation 186.5 m at the north abutment. The granular pad should be designed and constructed as described in Section 6.8.

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

6.3.2 Resistance to Lateral Loads

Resistance to lateral forces/sliding resistance between the concrete footings and the bedrock or the compacted granular pad should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction, $\tan \delta$, may be taken as 0.7 between the base of the concrete footings or mass concrete and the bedrock and 0.6 between the concrete footings and the granular pad (NAVFAC, 1982) for construction in-the-dry. These values represent unfactored values; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating the horizontal resistance.

For footings on bedrock, the sliding/lateral resistance between the concrete footing/mass concrete and the bedrock may be supplemented by dowelling/anchoring into the bedrock if required. The horizontal resistance of the dowels is dependent on the strength of the bedrock, grout and steel. For this site, where the rock mass is essentially as strong as or stronger than concrete, the design of the dowels into the rock may be considered in the same way as dowels embedded into the concrete. This assumes that the unconfined compressive strength of the grout will be similar to that of the concrete. The dowels should have a 1 m minimum embedded length within the bedrock, and the structural strength of the dowel and compressive strength of the grout should not be exceeded. If dowelling into bedrock is adopted at this site, an NSSP should be included in the Contract Documents to specify the installation, materials and testing of the dowels; an example is provided in Appendix C.

6.3.3 Frost Protection

For spread footings founded directly on the bedrock or mass concrete over bedrock, frost susceptibility is not an issue. Footings constructed on a granular pad should be provided with a minimum of 1.8 m of soil cover or equivalent thickness of insulation for frost protection (OPSD 3090.101).



6.4 Deep Foundations

6.4.1 Steel H-Pile Foundations

We recommend that the abutments and north pier be founded on steel H-piles. A minimum length of 5 m is required for an integral abutment.

For corrugated steel pipes (CSPs) installed as part of the integral abutment design (through which the piles will be driven), the CSPs should be backfilled with loose, fine to medium sand. An NSSP detailing the installation method and gradation of this sand should be included in the Contract Documents and an example is included in Appendix C.

For design, the estimated tip elevations 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 due to the variability of the bedrock surface and the lengths given below should be considered minimum lengths.

Foundation Element	Location within Foundation Element	Borehole Numbers	Proposed Underside of Pile Cap* (m)	Bedrock Surface/Refusal Elevation (m)	Design Tip Elevation (m)	Estimated Design Pile Length (m)
South Abutment	West	B5-4 & B5-5	194.8	189.3 to 188.3	188.3	6.5
	Centre	B5-3		189.4	189.4	5.4
	East	B5-1 & B5-2		190.8 to 190.6	189.8	5**
North Pier	Centre	B5-7	181.1	165.3	165.3	15.5
North Abutment	West	B5-11	196.7	172.9	172.9	23.8
	Centre	B5-10		170.2	170.2	26.5
	East	B5-DC1		178.2	178.2	18.5

Note * From GA drawing dated August 2011.

** Bedrock trenching or pre-drilling holes into bedrock will be required to achieve adequate pile length.

Due to the proposed grade raise at the south and north abutments, a compacted granular pad/core will be required below the abutments through which the piles will be driven. The granular pad should be designed and constructed as described in Section 6.8.

At the east side of the south abutment, consideration should be given to bedrock excavation to the pile tip level such that a compacted granular pad can be constructed. Bedrock excavation at the east end of the south abutment will be less than 1 m. This will avoid the need for pre-drilled holes/trenching on only a portion of the abutment.



6.4.1.1 *Geotechnical Axial Resistance*

For HP310X110 piles driven to refusal on the gneiss bedrock at the south and north abutments and mafic intrusive bedrock at the north pier, a factored axial resistance at ULS of 2,000 kN per pile may be used 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.

Due to the presence of boulders observed at the ground surface at Borehole B5-10 and a boulder encountered at a depth of 0.3 m in the original location for DCPT B5-DC1, we recommend that the abutment area be cleared of surficial and near surface boulders prior to constructing the granular pads to minimize the potential for the piles to “hang up” at this level, which may not be otherwise possible to avoid once the pad is constructed. Details are given in Section 6.9.5.

6.4.1.2 *Set Criteria*

For piles driven to bedrock, set criteria are highly dependent on pile driving hammer type 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.

At the north abutment, the bedrock surface is sloping down towards the south, as shown on Drawing 1. Therefore, consideration should be given to the following set criteria and pile driving procedures, which are intended to improve the process of seating of the piles on a sloping bedrock surface:

- 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.
- A final set of no less than 10 blows per 12 mm of penetration should then be obtained at the maximum hammer energy.

All pile installation/driving should be in accordance with SP903S01. The piles should be provided with rock points, Titus Injector or equivalent. An NSSP should be included in the Contract Documents to address the requirements for rock points; an example is included in Appendix C.



6.4.1.3 Pile Driving Note

The pile driving note to be added to the drawings for this project is Note 5 in Clause 3.3.3 of the Structural Manual (MTO, 2008):

“Piles to be driven to bedrock”.

6.4.1.4 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.

The lateral load response of a single vertical pile may be calculated using subgrade reaction theory (CHBDC 5.9.2, CFEM 1992) 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)

At the south abutment, north pier and north abutment, the lateral load response of the piles will be developed from the passive resistance of the soil or backfill. The values of n_h to be used to calculate the coefficient of horizontal subgrade reaction (k_h) to be assumed in the structural analysis for the piles at this location are presented below:

Foundation Element (Relevant Borehole)	Soil Unit	Elevation (m)	n_h (kPa/m)
South Abutment (B5-1 to B5-5)	Loose Sand within CSP	194.8 – 191.8	1,300
	Compacted Granular Fill Core*	191.8 – 189.8	6,600
North Pier (B5-7)	Very Loose to Compact Sand	181.1 – 172.4	1,300
	Very Dense Sand and Gravel	172.4 – 165.3	4,400
North Abutment (B5-10 and B5-11)	Loose Sand within CSP	196.7 – 193.7	1,300
	Compacted Granular Fill Core*	193.6 – 187.2	6,600
	Loose to compact Sand	187.2 – 176.0	1,300
	Very dense Sand to Sand and Gravel	176.0 – 170.2	4,400

* Granular fill placed and compacted in accordance with SP206S03.



At the south and north abutments and north pier, for a single HP310X110 pile embedded into the native cohesionless soils (and new granular fill at the south and north abutments), the estimated maximum lateral resistance at ULS is 100 kN and at SLS, for 10 mm of deflection, is 50 kN (assuming a steel yield strength of 300 MPa). These values are based on analysis carried out using the commercially available program LPILE Plus (Version 5.0), produced by Ensoft Inc.

Group action for lateral loading should also be considered when the pile spacing in the direction of the loading is less than six to 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 (R)
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.

6.4.1.5 Frost Protection

All pile caps should be provided with a minimum of 1.8 m of soil cover for frost protection (OPSD 3090.101).

6.4.2 Caissons

Caissons socketted into the bedrock could be considered as an alternative to steel H-piles at the south and north abutments and north pier. The high axial capacity of the caissons would result in fewer units being required to support the abutments than those required for the H-pile design, as well as the possible elimination of a pile cap. There will be difficulty in socketting the large diameter caissons within the bedrock and achieving an adequate seal. Temporary liners and tremie concrete will likely be required to install caissons at this site.

Based on our experience with caissons socketted into bedrock at sites with similar bedrock conditions, caissons have been successfully advanced with drilling equipment employing a down-hole hammering system consisting of an external liner advanced as the bit is rotated and driven into the bedrock.



6.4.2.1 Geotechnical Axial Resistance

If caissons are considered as a foundation alternative, the caissons at this site will derive their axial resistance mainly from the shaft resistance of the bedrock socket. The contribution from end-bearing will be neglected due to the difficulties in cleaning and inspecting the base of the sockets, which will generally be below the water level and at great depth below ground surface at the north pier and at the north abutment. The factored geotechnical axial resistance at ULS for different caisson diameters socketted a minimum of 2 m into the bedrock is given below.

Caisson Diameter (m)	Mafic Intrusive Bedrock (North Pier) and Gneiss Bedrock (South and North Abutment) (minimum 2 m long socket)	
	ULS (kN)	SLS for 25 mm
0.9	4,000	n/a
1.5	8,000	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 north pier and north abutment and a sufficient head of water should be maintained inside the liner at all times to balance the hydrostatic pressures.

6.4.2.2 Resistance to Lateral Loads

The geotechnical resistance to lateral loading for the caissons should be calculated in accordance with Section 6.4.1.4 using the horizontal subgrade reaction formulas. However, as we understand that caissons are not proposed for the foundation elements at this time, lateral capacities are not required by the designer.

6.4.2.3 Frost Protection

The pile caps for the caissons, if constructed at/below ground surface, should be provided with a minimum of 1.8 m of conventional soil cover for frost protection (OPSD 3090.101).

6.5 Seismic Considerations

6.5.1 Site Coefficient

For seismic design purposes, the Site Coefficient, *S*, for this site, in accordance with Section 4.4.6 of the CHBDC may be taken as 1.0, consistent with Soil Profile Type I.



6.5.2 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 *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, no amplification of the ground motion is recommended for design (i.e. Site Coefficient, $S=1$ for Soil Profile I from Table 4.4 of *CHBDC*), and the Peak Horizontal Acceleration (PHA) is 0.05 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 Performance Zone 1.

6.6 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 6.5.2, 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) 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 (Compacting). 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 may be placed either in a zone with width equal to at least 1.8 m behind the back of the wall stem (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 (as outlined in Figure C6.20(b), 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:

	Granular 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 indicated 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.

6.7 Approach Embankment Design

The proposed south and north approach embankments of the new Highway 69 SBL structure crossing Sucker Creek will be up to about 9 m and 16 m high above existing ground surface, respectively.

At the south approach, the subsoils consist of topsoil underlain by about 2.1 m of sandy silt to silty sand, gravelly sand and/or silty clay. At the north approach, the subsoils consist of organics underlain by up to about 1.9 m of loose to compact sand. At the toe of the new north embankment front slope (i.e. near the proposed north pier), up to 11.4 m of very loose to compact sand was encountered. The water level used in the analysis of the embankment stability and settlement is at about Elevation 182.3 m at the north abutment and decreasing to the water level in Sucker Creek at Elevation 181.9 m.



The stability and settlement analysis are focused on the 16 m high embankment over the cohesionless soils at the north approach. As the south approach embankment will essentially be founded on bedrock upon removal of the shallow overburden, stability and settlement are not a concern. We recommend that the shallow overburden at the south approach be sub-excavated prior to embankment construction.

As noted in Section 6.2, the recommended foundation alternative for the north abutment is to construct a compacted granular pad over the native soils (after removal of organics) to the underside of pile cap through which the piles will be driven. Generally, rock fill is the preferred embankment fill material for the Highway 69 project. In this regard, the stability and settlement analyses discussed in the following sections have been carried out on the basis that all roadway embankments will be constructed of rock fill, except for the zone of granular pad within the south and north abutments. The granular pad, to be constructed at side slopes at 1H:1V, should be constructed concurrently with the rock fill embankment as discussed in Section 6.8. Based on MMM's current design, the rock fill embankment on the south approach front slope and side slopes will be constructed at 1.50H:1V and on the north approach, the front slope and side slopes will be constructed at 1.25H:1V.

For rock fill, 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. At the north approach, a 2 m wide bench is required at 10 m below final grade.

The analysis assumes that as a minimum, the 0.3 m thick layer of organics will be removed from below the north approach embankment footprint.

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

6.7.1 Stability

Analyses were performed on the north embankment front slope as well as the east side slope located 6 m behind the north abutment as these are considered to be the critical sections for the north approach embankment.

6.7.1.1 Methodology

Limit equilibrium slope stability analyses were performed using the commercially available program GeoStudio 2007 (Version 7.17), 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, slope geometry and stratigraphy. In general, circular slip surfaces were analysed in the design.



6.7.1.2 Parameter Selection

For the very loose to compact sand deposit and the underlying dense sand and gravel deposit encountered at depth at the toe of the front slope and at ground surface at the north abutment, the effective stress parameters employed in the analysis were estimated from empirical correlations based on the results of the in situ SPTs. 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. In addition, for the very loose to compact cohesionless subsoils encountered within the upper approximately 7 m of the borehole advanced near the toe of the north front slope (i.e. Borehole B5-7 at the north pier), the effective stress parameter employed in the analysis was also estimated giving consideration to the laboratory consolidated drained direct shear (DS) tests carried out on one selected sample of the sand from Borehole B5-7. The details of the test results are shown on Figure B-3 in Appendix B.

Summarized below are the simplified stratigraphy and the associated strengths and unit weights employed for the different soil types in the proposed north approach.

Soil Type	Unit Weight (kN/m ³)	Angle of Internal Friction
Rock Fill	19	40°
Granular pad below abutment	21	35°
Sand	20	32°
Sand and Gravel	21	35°

6.7.1.3 Results of Analysis

The results of the stability analyses are summarized below for the critical sections described in Section 6.7.1.2. The minimum Factor of Safety is based on a deep-seated, global trial failure surface that would impact the operation of the roadway.

Location	Relevant Boreholes at Centreline	Greatest Embankment Height (m)	Factor of Safety
North Approach Front Slope	B5-7, B5-10 and B5-14	16	1.2
North Approach East Side Slope 6 m behind abutment	B5-10 and B5-14	~ 14 (based on contours)	>1.3

Since the front slope has a Factor of Safety less than the target value of 1.3, stability mitigation measures will be required. We recommend that mitigation of stability consist of the incorporation of a 6 m wide stability berm at crest Elevation 186.3 m along the toe of the north front slope. The inclusion of such a berm into the front slope geometry results in a Factor of Safety of 1.3 for the embankment, as presented on Figure 2. The toe berm should extend to the embankment side slopes at crest Elevation 186.3 m, tapering to the adjacent existing ground surface.



6.7.2 Settlement

Settlement of the north approach embankment can be expected as a result of the loading from the new fills on the cohesionless foundation soils. In addition, settlements may also occur due to compression of the embankment fill itself at both the south and north approach embankments. As discussed previously, we recommend that the relatively thin overburden below the new south approach embankment from the abutment to 20 m behind the abutment be stripped prior to fill placement.

6.7.2.1 Methodology

To estimate the magnitude of the expected settlements at the north approach, analyses were carried out on the critical sections at the north approach (i.e. 6 m and 20 m behind the abutment) and the critical section at the south approach (i.e. 6 m behind the abutment) using hand calculations. The analyses assume that all native soils are removed from below the south approach embankment footprint and organic materials are removed from below the north approach embankment footprint. The following sections summarize the simplified stratigraphy, parameters and results of the analysis.

6.7.2.2 Settlement Criteria

Based on MTO's guideline "Embankment Settlement Criteria For Design" (March 2, 2010), the following post-construction settlement and differential settlement criteria are considered acceptable within 20 years post-paving for the bridge approach embankments and the new Highway 69 embankment at this site:

Location	Distance from Transition Point (i.e. Abutment)	Total Post-Construction Settlement* (mm)	Differential Settlement Rate
Transition/Taper to Bridge Abutments	0 m to 20 m	25	--
	20 m to 50 m	50	--
	50 m to 75 m	75	--
New Highway 69 Embankment	>75 m	100	200:1

These criteria have been used for determining whether mitigation measures are required to limit post-construction settlement of the approach embankments.

6.7.2.3 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 following simplified stratigraphy and deformation parameters have been employed in the settlement analysis for the south and north approaches at the critical sections.



Location of Embankment and Relevant Boreholes	Material	Maximum Thickness (m)	Unit Weight (kN/m ³)	Estimated Deformation Properties
South Approach (6 m behind abutment) (B5-1 to B5-5 and B5-13)	Rock Fill	9	19	Refer to Section 6.7.3.4
North Approach (behind abutment) (B5-10, B5-11 and B5-DC1)	Granular Backfill behind abutment	6.0	19	Refer to Section 6.7.3.4
	Granular Pad	9.5	21	Refer to Section 6.7.3.4
	Loose to very dense Sand	11.1	20	E' = 15 MPa
North Approach (20 m behind abutment) (B5-14)	Rock Fill	13	19	Refer to Section 6.7.3.4

6.7.2.4 Settlement – New Embankment Fill

Granular Fill

Settlement of granular fill placed behind and below the new abutments will be nominal provided the granular material is properly placed and compacted. For this case of the compacted granular pad and the granular fill behind the abutment, the settlement from properly compacted granular fill 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.

6.7.2.5 Results of Analysis

Presented below are the results of the estimated settlement of the foundation soils and embankment rock fill as a result of embankment construction.



Location of Embankment	Maximum Proposed Embankment Height (m)	Thickness of Cohesionless Deposit (m)	Estimated Settlement		
			Foundation Soils (mm)	Rock Fill (mm)	Granular Fill (mm)
South Approach (6 m behind abutment)	9	N/A	N/A	80	N/A
North Approach (behind abutment)	15.5	~17	220	N/A	<25
North Approach (20 m behind abutment)	13	N/A	35	140	N/A

Settlement of the foundation soils is 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.

The estimated settlement of the new rock fill after construction is estimated to be 80 mm and 140 mm at the south and north approaches, respectively. Approximately 90 percent of the estimated short-term rock fill settlement is expected to occur within six months following construction, with the remaining settlement expected to occur over the remaining design life of the roadway embankment. Since the estimated post construction settlement of the new embankment rock fill within 20 m of the abutments is greater than the settlement criteria referenced in Section 6.7.3.2, preloading the rock fill embankment for six months is required to mitigate (i.e. reduce) the magnitude of post-construction settlement. After a six-month preload period, the post-construction settlement is estimated to be approximately 10 mm and 20 mm for the immediate south and north approach embankment, respectively.

6.8 Subgrade Preparation and Embankment Construction

Due to the variability of soil type and limited thickness of the existing native soils at the south approach, we recommend that these soils be removed at the same time as the surficial organics, such that the bedrock is exposed prior to constructing the approach embankment.

At the north approach, the foundation soils should be left in place and only topsoil/organic deposits and any softened or loosened soil should be stripped from the approach embankment footprint prior to fill placement.

Embankment fill at the north approach should be keyed into the existing sloping ground surface (provided it is not exposed bedrock and it is steeper than 3H:1V) as per OPSD 208.010.

Placement of granular and rock fill material should be carried out in accordance with the requirements as outlined in SP206S03. Side slopes for rock fill embankments should be no steeper than 1.25H:1V as per OPSD 202.010.

At the south and north abutments, the compacted granular pad/core for installation of the piles should be constructed in accordance with the geometry shown on Figure 1 such that it is integrated into the rock fill embankment. We recommend the pad be constructed using Granular 'A' material. Alternatively, Granular 'B' Type II with maximum particle size of 75 mm could be considered. The granular pad/core should extend at least 1 m beyond the plan limits



of the pile cap and be sloped no steeper than 1H:1V. The granular pad should be constructed in accordance with SP206S03. At the south abutment, we recommend that the overburden be removed to expose the bedrock surface prior to construction of the pad. The granular pad will be up to about 6.5 m and 6.7 m thick at the south and north abutments, although the sloping ground could locally require the pad to be thicker.

As discussed in Section 6.7, the granular pad/core should be constructed concurrently with the rock fill embankment to reduce the potential for differential settlement occurring and for constructability reasons; an example Operational Constraint (OC) to be included in the Contract is presented in Appendix C.

6.8.1 Platform Widening

In accordance with the requirements of MTO NRE 98 200 (Embankment Design Guidelines), the minimum required embankment widening to account for future pavement overlays is 2 m per embankment side.

6.9 Design and Construction Considerations

6.9.1 Blasting for Rock Excavations

At the south abutment and south pier, bedrock excavation may be required to install deep foundations in a trench or to create a level platform for the shallow foundations. For bedrock excavation, the overall slope of the cut face may be formed vertical or at a steep slope (i.e. 0.25H:1V). All bedrock excavation within and near footing areas should be carried out using controlled blasting techniques in order to minimize shattering and over-break. The use of line drilling, pre-shearing or cushion blasting are recommended in order to provide a neat excavation line and minimize face instabilities resulting from blast damage to the rock mass. Good blasting practices will be critical to maintaining the excavation lines and preserving the integrity of the rock mass in the area of the structure foundations. We recommended that the Contractor retain a blast engineer and submit proposed blast plans for review at least three weeks in advance of rock excavation.

6.9.2 Excavations

At the north pier, excavations up to 3.5 m within the native soils adjacent to Sucker Creek will be required for pile cap construction. Excavation will extend below the groundwater level.

Temporary excavation side slopes above the water level within the native sand should be no steeper than 1H:1V. The native sand material at this site may be classified as a Type 3 soil. Below the water level, temporary shoring and unwatering will be required to carry out the construction of the pile cap in-the-dry at this location. All excavations must be carried out in accordance with the latest edition of the Occupational Health and Safety Act and Regulation for Construction Projects (OHSA) and good construction practices.

6.9.3 Groundwater and Surface Water Control

Excavations at the south abutment/approach, south pier and north abutment/approach will likely be dry given the existing and proposed ground surface elevations at these locations and the groundwater level noted during the subsurface investigation. Groundwater/surface water is anticipated to drain towards Sucker Creek. Therefore, groundwater control is not anticipated for these areas. Surface water should be directed away from the excavations at all times.



The proposed north pier is located immediately adjacent to Sucker Creek and groundwater inflow should be expected during excavation for the pile cap and controlled unwatering within temporary shoring will be required. The shoring should be advanced to an appropriate depth to control groundwater inflow, in conjunction with controlled unwatering or with fluid support, in order to prevent boiling/blow-out of the soils at the base of the excavation and to minimize ground loss during excavation, backfilling and concrete placement. The shoring designer should consider the use of a tremie plug below the pile cap for this purpose, through which the piles would be driven. The Contractor is responsible to ensure that appropriate construction procedures and equipment are used for construction.

The Contractor should be alerted that excavations will be advanced through cohesionless soils which are expected to be unstable below the groundwater level at this site. The Contract should include an OC to alert the Contractor to the requirement for ground control, an example of which is included in Appendix C.

6.9.4 Temporary Shoring

Given the depth of excavation (up to about 3.5 m) required to construct the north pier pile cap and the proximity to the creek, a temporary cut-off wall (cofferdam) will most likely be required at this location.

Temporary excavation support systems should be designed and constructed in accordance with OPSS 539. The lateral movement of the temporary shoring system should meet Performance Level 2.

6.9.5 Obstructions

During the field investigation for the north abutment, a boulder was observed at ground surface at or near the location of Borehole B5-10 and refusal was encountered at a depth of 0.3 m, likely on a boulder, at the original location of DCPT B5-1. As noted in Section 6.4.1.1, boulders at ground surface within the north abutment foundation footprint should be removed prior to construction of the granular pad to ensure that there are no obstructions that could impact the piling operations. An NSSP should be included in the Contract Documents for this work; an example is provided in Appendix C.

7.0 CLOSURE

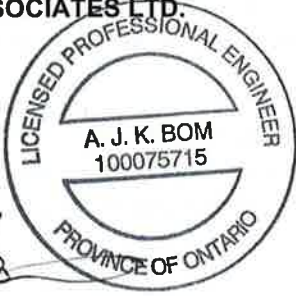

This report was prepared by Mr. André Bom, P.Eng., and the technical aspects were reviewed by Ms. Sarah E. M. Coyne, P.Eng., Associate. Mr. Jorge M. A. Costa, P.Eng., Golder's Designated MTO Contact for this project and Principal, conducted an independent quality control review of the report.



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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

Commercial Software

LPile (Version 5.0) by Ensoft Inc.

GeoStudio (Version 7.17) by Geo-Slope International Ltd.

Ministry of Transportation, Ontario, 2008. Structural Manual. Quality and Standards, Transportation Engineering Branch, Bridge Office, Design Section.

Ministry of Transportation, Ontario. 2010. Post Construction Rock Fill Settlement and Guidelines for Estimating Rock Fill Quantity, April 12, 2010.

Ministry of Transportation Ontario, 2010. Embankment Settlement Criteria for Design, Final Draft, March 2, 2010



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Ministry of Transportation, Ontario. 2002. Backfill to Structures Adjacent to Rock Embankment Approaches, Northern Region Directive, November 2002.

Ministry of Transportation, Ontario. 1998. Northern Region Embankment Design Guidelines, Northern Region Directive, NRE, 98-200, issued by Geotechnical Section, October 1998.

Ministry of Transportation Ontario Special Provisions

SP 110S13	Material Specifications for Aggregates – Base, Subbase, Select Subgrade and Backfill
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Ontario Provincial Standard Specification:

OPSS 206	Construction Specification for Grading
OPSS 501	Construction Specification for Compacting
OPSS 539	Construction Specification for Temporary Protection Systems

Ontario Provincial Standard Drawings Ontario Provincial Standard Drawings

OPSD 202.010	Slope Flattening using Surplus Excavated Material on Earth or Rock Embankment
OPSD 208.010	Benching of Earth Slopes
OPSD 3090.101	Foundation Frost Depths for Southern Ontario
OPSD 3101.150	Walls Abutment, Backfill Minimum Granular Requirement
OPSD 3101.200	Walls Abutment, Backfill Rock
OPSD 3121.150	Walls Retaining, Backfill Minimum Granular Requirement

Ontario Water Resources Act:

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



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Table 1: Evaluation of Foundation Alternatives - South Abutment

Options	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Steel H-Piles Driven to Bedrock	1	<ul style="list-style-type: none"> Allows for integral abutment construction. 	<ul style="list-style-type: none"> Minor bedrock excavation required to achieve 5 m pile length. Requires granular pad/core below the abutment. Requires removal of overburden for pile cap construction. 	<ul style="list-style-type: none"> Lower relative costs compared with caisson option. 	<ul style="list-style-type: none"> Low risk of not achieving minimum required pile length.
Spread Footings Founded on Bedrock Surface	2	<ul style="list-style-type: none"> Straightforward construction. Eliminates potential settlement of granular pad. Higher axial resistance than spread footings on granular pad. 	<ul style="list-style-type: none"> Requires removal of overburden. Mass concrete or bedrock excavation required to level the foundation due to sloping rock surface. Does not allow for integral abutment construction. 	<ul style="list-style-type: none"> Lower relative cost compared to deep foundation alternatives but higher than for footings on granular pad due to mass concrete/bedrock excavation or socketting of piles for integral abutment design. 	<ul style="list-style-type: none"> Low risk of difficulties levelling foundation on sloping rock. May require rock anchors/dowels for lateral sliding resistance.
Spread Footings Founded on Compacted Granular Pad over Bedrock	3	<ul style="list-style-type: none"> Straightforward construction. 	<ul style="list-style-type: none"> Requires granular pad below the abutment. Lower axial resistance than steel H-piles or spread footings on bedrock. Requires removal of overburden to expose bedrock and replacement with compacted granular fill. Does not allow for integral abutment construction. 	<ul style="list-style-type: none"> Lower relative cost compared to deep foundation alternatives. 	<ul style="list-style-type: none"> Low risk of settlement of granular pad. Risk of differential settlement if pad thickness varies over bedrock surface. Risk of differential settlement between foundation elements.



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Options	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Caissons Socketted into Bedrock	4	<ul style="list-style-type: none">■ Reduced number of deep elements compared to steel H-piles (higher axial resistance per unit).■ Possible elimination of pile cap.	<ul style="list-style-type: none">■ Difficulty socketting caissons into bedrock.■ Does not allow for integral abutment construction.	<ul style="list-style-type: none">■ Cost many times higher than for piles.	<ul style="list-style-type: none">■ Risk of difficulties drilling large diameter bedrock socket.



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Table 2: Evaluation of Foundation Alternatives - North Abutment

Options	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Steel H-Piles Driven to Bedrock	1	<ul style="list-style-type: none"> ■ Straightforward construction. ■ Allows for integral abutment construction. 	<ul style="list-style-type: none"> ■ Pile tips need to be reinforced with rock points to help seat pile on sloping bedrock. ■ Requires granular pad below the abutment to allow for pile driving. ■ Removal of boulders from at/near ground surface required. 	<ul style="list-style-type: none"> ■ Lower relative costs compared with caisson option. 	<ul style="list-style-type: none"> ■ Risk of not achieving design capacity if pile “hangs up” on boulders at/near ground surface.
Spread Footings Founded on Compacted Granular Pad over Native Soils	2	<ul style="list-style-type: none"> ■ Relatively straightforward construction. 	<ul style="list-style-type: none"> ■ Lower axial resistance than steel H-piles. ■ Requires granular pad below the abutment. ■ Does not allow for integral abutment construction. 	<ul style="list-style-type: none"> ■ Lower cost than for deep foundations. 	<ul style="list-style-type: none"> ■ Risk of settlement of granular pad. ■ Risk of differential settlement compared to other foundation elements on bedrock.
Caissons Socketted into Bedrock	3	<ul style="list-style-type: none"> ■ Reduced number of deep elements compared to steel H-piles (higher axial resistance per unit). ■ Possible elimination of pile cap. 	<ul style="list-style-type: none"> ■ Temporary liners would be required for groundwater control and support through the granular overburden. ■ Concrete for caissons would have to be placed by tremie methods below the water level. ■ Difficulty socketting caissons into bedrock. ■ Does not allow for integral abutment construction. ■ Removal of boulders from at/near ground surface required. 	<ul style="list-style-type: none"> ■ Cost many times higher than for piles. 	<ul style="list-style-type: none"> ■ Risk of difficulties achieving seal and drilling large diameter bedrock socket.

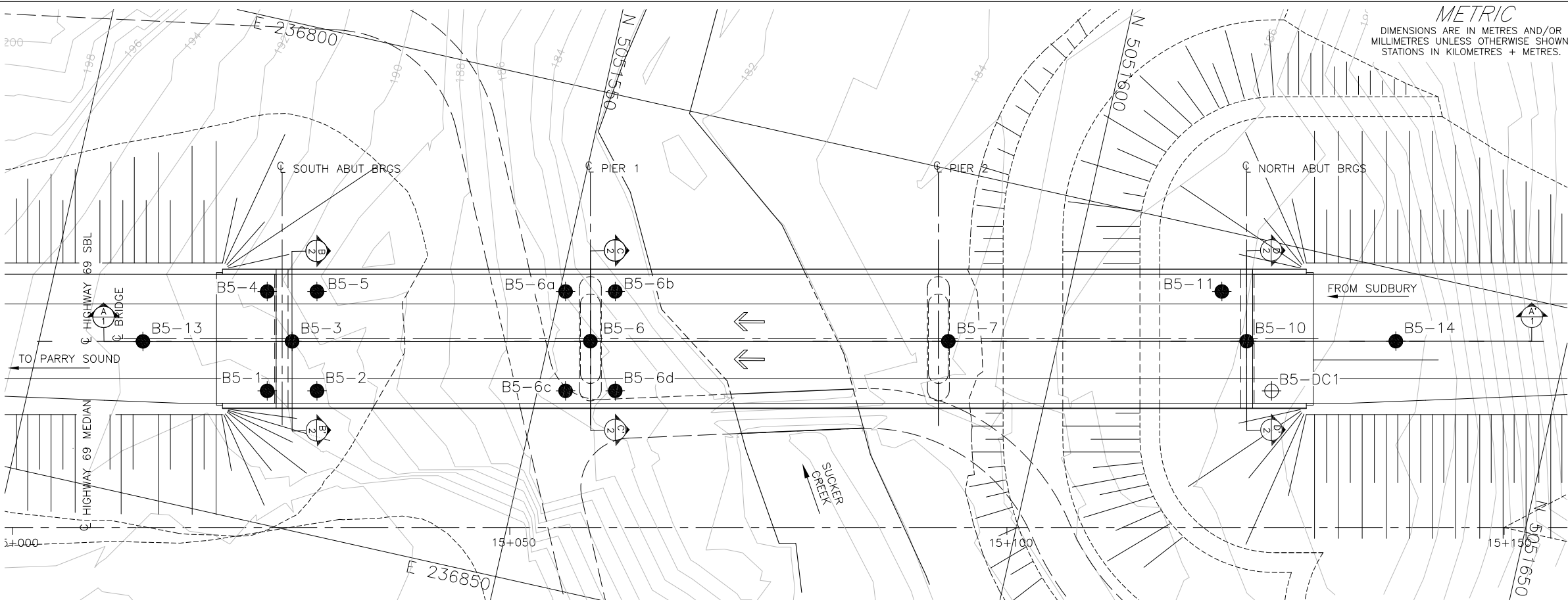


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HIGHWAY 69, GWP 5005-08-00, WP 5194-06-01

Table 3: Evaluation of Foundation Alternatives - North Pier

Options	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Steel H-Piles Driven to Bedrock	1	<ul style="list-style-type: none"> ■ Straightforward construction. ■ Higher axial resistance than shallow foundations. 	<ul style="list-style-type: none"> ■ Pile tips need to be reinforced with rock points to help seat pile on sloping bedrock. ■ Pile cap construction below water level requires cofferdam, temporary shoring and unwatering. 	<ul style="list-style-type: none"> ■ Lower relative costs compared with caisson option. ■ Increased cost for cofferdam. 	<ul style="list-style-type: none"> ■ Risk of heaving/boiling sands within cofferdam.
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. 	<ul style="list-style-type: none"> ■ Temporary liners would be required for groundwater control and support through the granular overburden. ■ Concrete for caissons would have to be placed by tremie methods below the water level. ■ Difficulty socketting caissons into bedrock. 	<ul style="list-style-type: none"> ■ Cost many times higher than for piles. ■ Increased cost for cofferdam if pile cap constructed below/at grade. 	<ul style="list-style-type: none"> ■ Risk of difficulties achieving seal and drilling large diameter bedrock socket.
Shallow Foundations	NF		<ul style="list-style-type: none"> ■ Low geotechnical axial resistance. ■ Would require extensive unwatering to found footing on proper stratum at depth. 	<ul style="list-style-type: none"> ■ Lower cost of footing but increased cost for cofferdam and unwatering. 	<ul style="list-style-type: none"> ■ Risk of settlement of the footing due to relative density of cohesionless soils.



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MILLIMETRES UNLESS OTHERWISE SHOWN.
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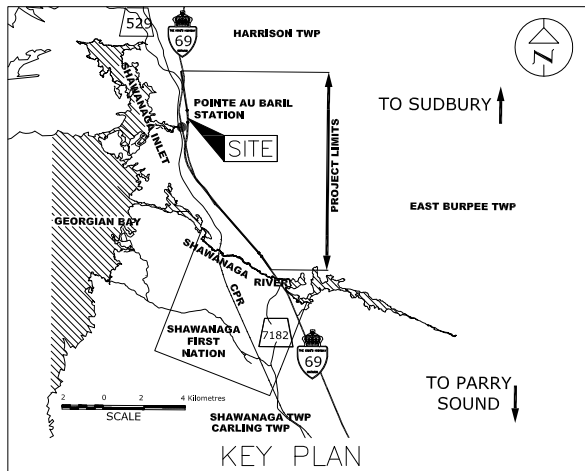


HIGHWAY 69
SUCKER CREEK (SBL)
BOREHOLE LOCATIONS
AND SOIL STRATA

SHEET



Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA



LEGEND

- Borehole
- ⊕ DCPT
- N Standard Penetration Test Value
- 4 Blows/0.3 m unless otherwise stated (Std. Pen. Test, 475j/blow)
- ▽ WL upon completion of drilling
- R Refusal
- 100% Rock Quality Designation (RQD)

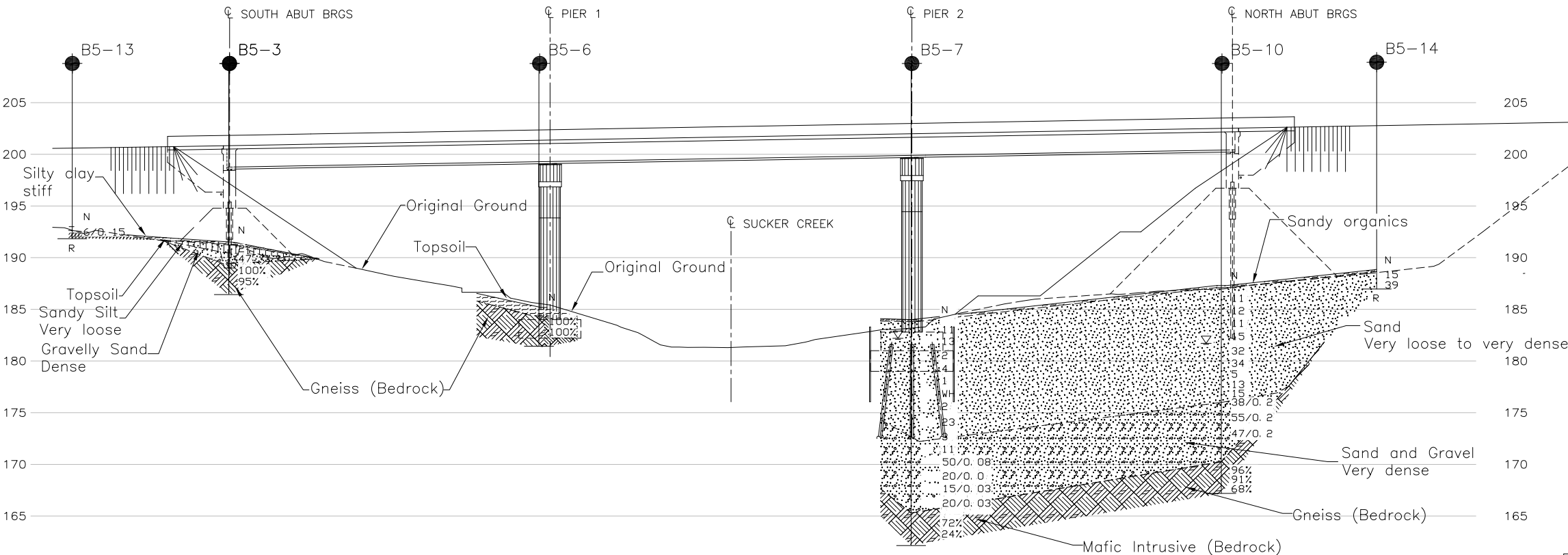
No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
B5-1	192.2	5051523.5	236836.8
B5-2	191.5	5051528.4	236835.7
B5-3	191.5	5051524.8	236831.4
B5-4	190.4	5051521.3	236827.0
B5-5	190.5	5051526.2	236825.9
B5-6	185.2	5051554.1	236824.8
B5-6a	185.0	5051550.6	236820.5
B5-6b	183.6	5051555.5	236819.4
B5-6c	185.9	5051552.8	236830.2
B5-6d	184.9	5051557.6	236829.1
B5-7	184.0	5051589.2	236816.9
B5-10	187.3	5051618.5	236810.3
B5-11	186.9	5051615.0	236806.0
B5-13	192.6	5051510.2	236834.7
B5-14	188.8	5051633.1	236807.1
B5-DC1	187.8	5051622.0	236814.7

NOTES

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

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

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



A-A'
1

CENTRELINE PROFILE

HIGHWAY 69 SBL

SCALE

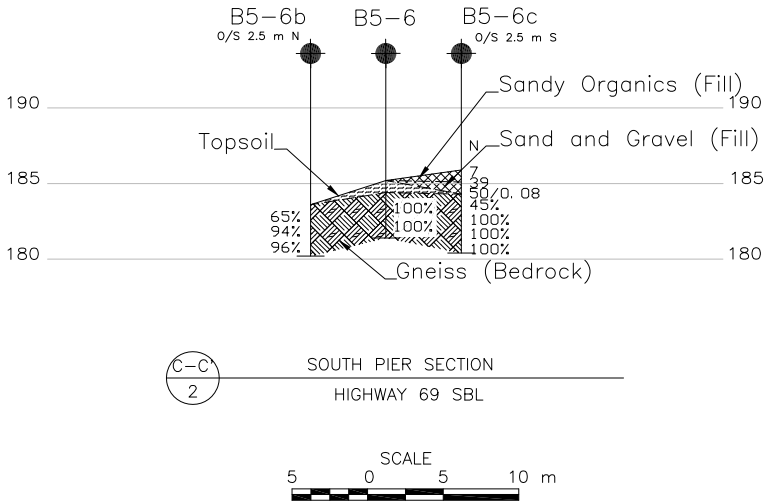
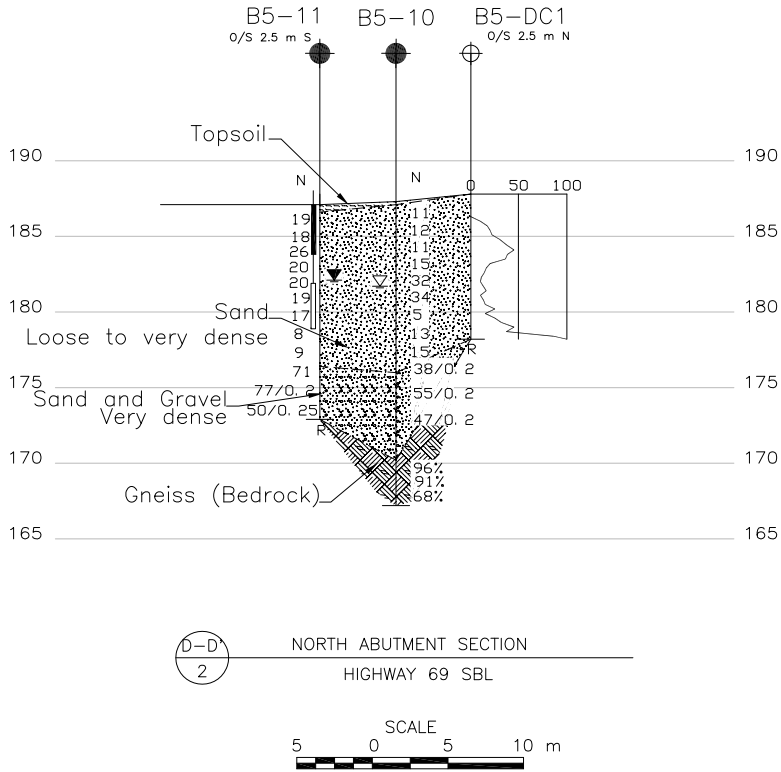
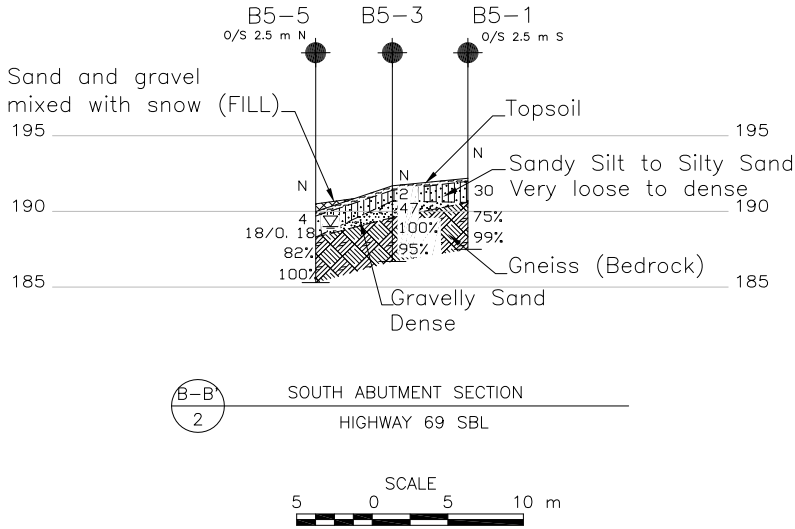
5 0 5 10 m

REFERENCE

Base plans provided in digital format by MMM, drawing file nos. 44_447-2_01-GENERAL ARRANGEMENT INTEGRAL (31m), received August 18, 2011



NO.	DATE	BY	REVISION
Geocres No. 41H-81			
HWY. 69	PROJECT NO. 07-1191-0020		DIST.
SUBM'D.	CHKD. AB	DATE: DEC 2011	SITE: 44-447/2
DRAWN: PL	CHKD. SEMC	APPD. JMAC	DWG. 1



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

CONT No.
WP No. 5194-06-01

HIGHWAY 69
SUCKER CREEK (SBL)
SOIL STRATA

SHEET



Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA

LEGEND

- Borehole
- DCPT
- N Standard Penetration Test Value
- 4 Blows/0.3 m unless otherwise stated (Std. Pen. Test, 475j/blow)
- WL upon completion of drilling
- Seal
- Piezometer
- WL March 3, 2009
- R Refusal
- 100% Rock Quality Designation (RQD)

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
B5-1	192.2	5051523.5	236836.8
B5-2	191.5	5051528.4	236835.7
B5-3	191.5	5051524.8	236831.4
B5-4	190.4	5051521.3	236827.0
B5-5	190.5	5051526.2	236825.9
B5-6	185.2	5051554.1	236824.8
B5-6a	185.0	5051550.6	236820.5
B5-6b	183.6	5051555.5	236819.4
B5-6c	185.9	5051552.8	236830.2
B5-6d	184.9	5051557.6	236829.1
B5-7	184.0	5051589.2	236816.9
B5-10	187.3	5051618.5	236810.3
B5-11	186.9	5051615.0	236806.0
B5-13	192.6	5051510.2	236834.7
B5-14	188.8	5051633.1	236807.1
B5-DC1	187.8	5051622.0	236814.7

NOTES

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

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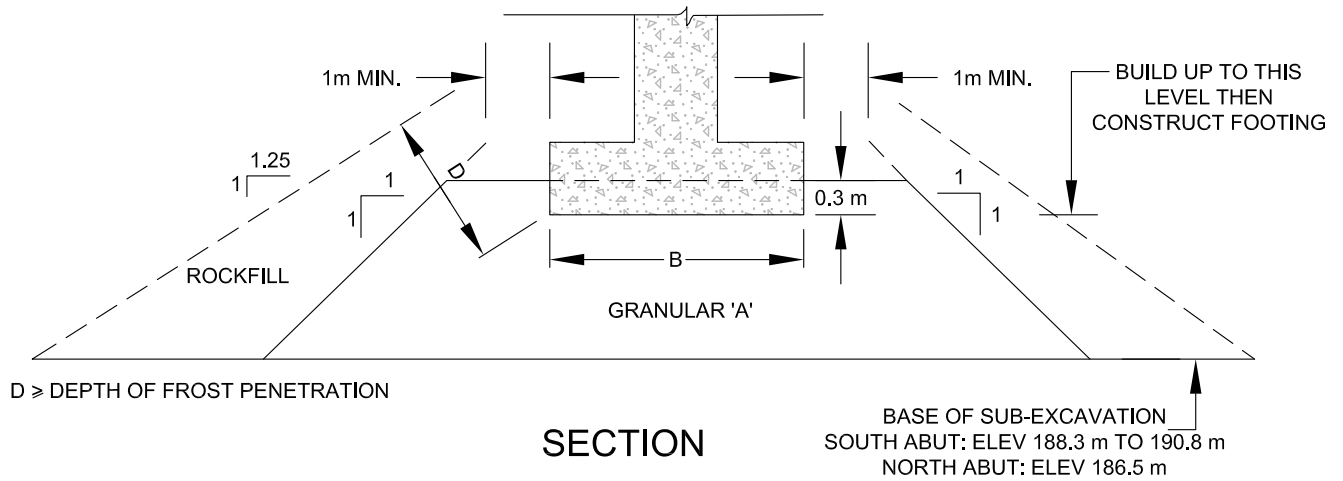
REFERENCE

Base plans provided in digital format by MMM, drawing file nos. 44_447-2_01-GENERAL ARRANGEMENT INTEGRAL (31m), received AUGUST 18, 2011




NO.	DATE	BY	REVISION
Geocres No. 41H-81			
HWY. 69	PROJECT NO. 07-1191-0020		DIST.
SUBM'D.	CHKD. AB	DATE: DEC 2011	SITE: 44-447/2
DRAWN: AMW	CHKD. SEMC	APPD. JMAC	DWG. 2

PLOT DATE: December 12, 2011
FILENAME: N:\Active\2007\1190 Sudbury\1191\07-1191-0020 MMM Hwy 69 Twinning\5000 Drawings\Structure Location Plan\Sucker Creek Structure (B5)\07-1191-0020-B5 FIG1.dwg



CONSTRUCTION SEQUENCE:

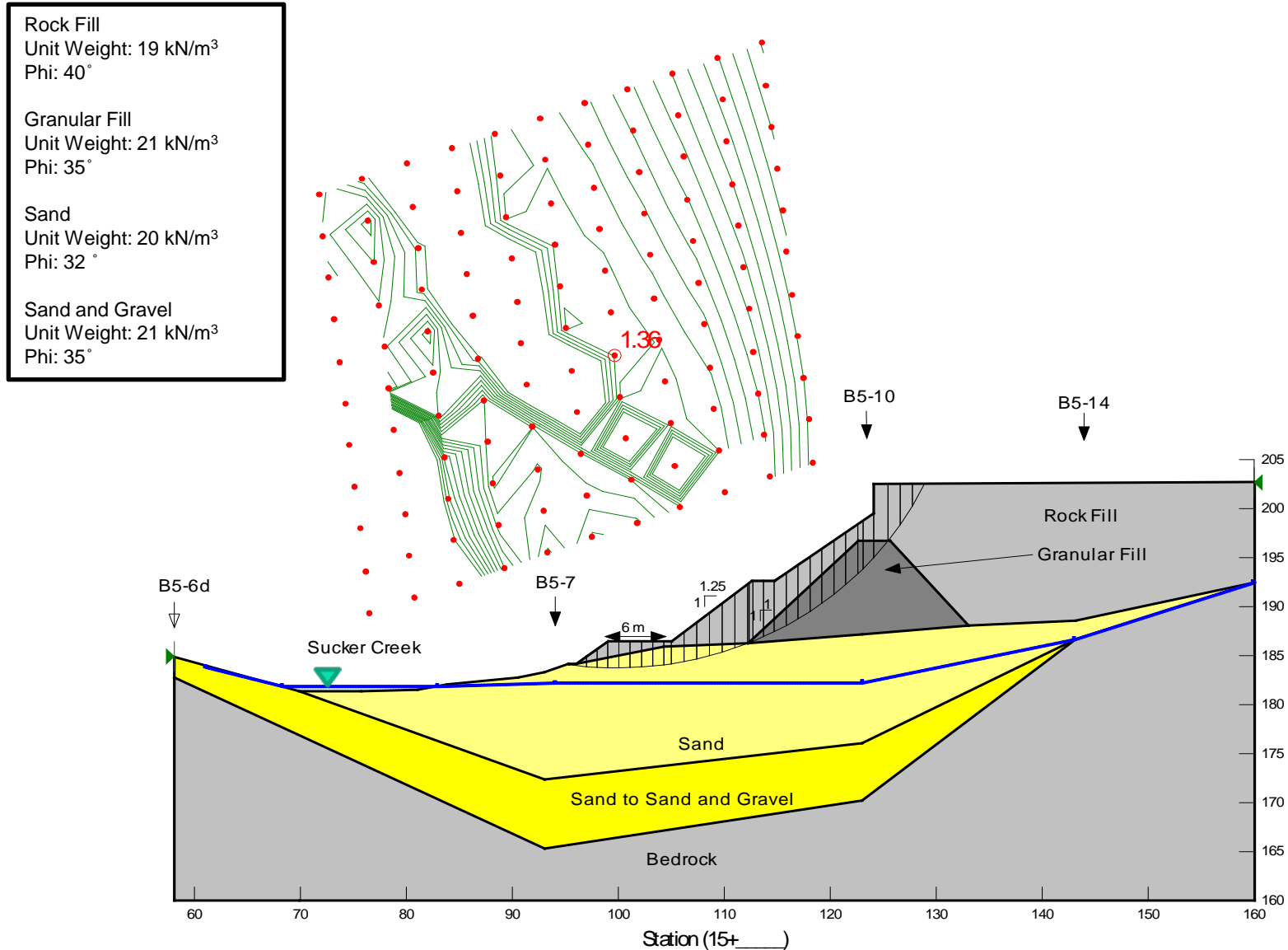
1. REFER TO ACCOMPANYING FOUNDATION DESIGN REPORT, SECTION 6.2.
2. REMOVE SUBSOILS UNDER FOOTPRINT OF COMPACTED GRANULAR CORE TO ELEVATION SPECIFIED.
3. PLACE AND COMPACT GRANULAR 'A' (OR GRANULAR 'B' TYPE II MAXIMUM PARTICLE SIZE 75 mm) IN ACCORDANCE WITH SP206S03 TO UNDERSIDE OF ABUTMENT. CONSTRUCT SURROUNDING ROCK FILL CONCURRENTLY WITH GRANULAR 'A' / 'B' TYPE II.
4. DRIVE PILES.
5. CONSTRUCT CONCRETE PIER CAP (OR SPREAD FOOTING IF APPLICABLE).
5. PLACE REMAINDER OF GRANULAR 'A' AND BACKFILL AS REQUIRED.
6. SOURCE M.T.C 1982.

PROJECT				GWP 5005-08-00 SUCKER CREEK SBL BRIDGE							
TITLE TYPICAL ABUTMENT ON COMPACTED FILL CORE											
				PROJECT No.		07-1191-0020		FILE No. 07-1191-0020-B5 FIG1.dwg			
				DESIGN				SCALE		NTS	
				CAD		AMW		DEC 2011		FIGURE No.	
				CHECK		AB		DEC 2011		1	
				REVIEW		JMAC		DEC 2011			



Sucker Creek SBL Front Slope Stability North Approach (Toe Berm)

FIGURE 2



Date: December 2011

Project No.: 07-1191-0020-B6

Analysis By: EC / AB Reviewed By: SEMC



APPENDIX A

Record of Boreholes and Drillholes



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

Piezo-Cone Penetration Test (CPT)

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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

(b) Cohesive Soils Consistency

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

IV. SOIL TESTS

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

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

V. MINOR SOIL CONSTITUENTS

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



LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE

Fresh: no visible sign of weathering

Faintly weathered: weathering limited to the surface of Major discontinuities

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

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

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

Completely weathered: rock is wholly decomposed and in a friable condition but the rock 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 B5- 1			1 OF 1 METRIC											
W.P. 5194-06-01			LOCATION N 5051523.5; E 236836.8			ORIGINATED BY EHS											
DIST _____ HWY 69			BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers			COMPILED BY AMW											
DATUM Geodetic			DATE February 11, 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 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W _p W W _L WATER CONTENT (%)			γ kN/m ³	GR SA SI CL
							20 40 60 80 100										
192.2	GROUND SURFACE																
0.0	Silty TOPSOIL						192										
0.2	Brown Moist																
	Sandy SILT to Silty SAND		1	SS	30		191										
	Compact to dense																
	Brown																
	Moist																
190.6	GNEISS (BEDROCK)						190									RQD = 75%	
1.6	Bedrock cored from 1.6 m depth to 4.7 m depth.		1	RC	REC 100%												
	For coring details refer to Record of Drillhole B5-1.						189										
			2	RC	REC 100%		188									RQD = 99%	
187.5	End of Borehole																
4.7	Note: 1. Borehole dry upon completion of drilling.																

PROJECT: 07-1191-0020

RECORD OF DRILLHOLE: B5- 1

SHEET 1 OF 1

LOCATION: N 5051523.5 ;E 236836.8

DRILLING DATE: February 11, 2009

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 55

DRILLING CONTRACTOR: Landcore Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH	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	NOTES WATER LEVELS INSTRUMENTATION
DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH	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	NOTES WATER LEVELS INSTRUMENTATION
		Refer to Previous Page		190.6									
2		GNEISS Fine to coarse grained Slightly weathered to 2.5 m, fresh below 2.5 m Very strong Pinkish grey		1.6									
3	02/11/09 NQ Coring	Numerous joints to 2.5 m depth.			1								
4					2								
5		End of Drillhole		187.5									
6				4.7									
7													
8													
9													
10													
11													

DEPTH SCALE


1 : 50



LOGGED: EHS

CHECKED: AB

SUD-RCK 07-1191-0020 B5 BH LOGS METRIC.GPJ GAL-MISS.GDT 12/12/11 DATA INPUT:

PROJECT <u>07-1191-0020</u>				RECORD OF BOREHOLE No B5- 2				1 OF 1 METRIC										
W.P. <u>5194-06-01</u>				LOCATION <u>N 5051528.4; E 236835.7</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>AMW</u>										
DATUM <u>Geodetic</u>				DATE <u>February 11, 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										
191.5	GROUND SURFACE							20	40	60	80	100						
0.0	Silty TOPSOIL		1	AS	-		191											
0.2	Brown Moist																	
190.8	Sandy SILT, some clay with rootlets																	
0.7	Brown Moist																	
	End of Borehole Auger Refusal																	
Note: 1. Borehole dry upon completion of drilling.																		

PROJECT 07-1191-0020			RECORD OF BOREHOLE No B5- 3			1 OF 1 METRIC											
W.P. 5194-06-01			LOCATION N 5051524.8; E 236831.4			ORIGINATED BY EHS											
DIST _____ HWY 69			BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers			COMPILED BY AMW											
DATUM Geodetic			DATE February 10, 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 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W _p W W _L WATER CONTENT (%)			γ kN/m ³	GR SA SI CL
							20 40 60 80 100	20 40 60 80 100	10 20 30								
191.5	GROUND SURFACE																
0.0	Silty TOPSOIL																
191.2	Brown																
0.3	Moist																
190.5	Sandy SILT, some clay with rootlets		1	SS	2		191										
1.0	Very loose																
	Brown																
	Moist																
	Gravelly SAND, some silt		2	SS	47		190										28 56 (16)
	Dense																
	Brown																
	Moist																
189.4	GNEISS (BEDROCK)																
2.1	Bedrock cored from 2.3 m depth to 5.2 m depth.		1	RC	REC 100%		189										RQD = 100%
	For coring details refer to Record of Drillhole B5-3.																
			2	RC	REC 100%		188										RQD = 95%
							187										
186.5	End of Borehole																
5.0	Note: 1. Borehole dry upon completion of drilling. 2. About 0.2 m of snow at BH location.																

SUD-MTO 001 07-1191-0020 B5 BH LOGS METRIC.GPJ GAL-MISS.GDT 12/12/11 DATA INPUT:

PROJECT: 07-1191-0020

RECORD OF DRILLHOLE: B5- 3

SHEET 1 OF 1

LOCATION: N 5051524.8 ;E 236831.4

DRILLING DATE: February 10, 2009

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 55

DRILLING CONTRACTOR: Landcore Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN	FLUSH	RECOVERY		R.Q.D. %	FRACT. INDEX METRES	DISCONTINUITY DATA				HYDRAULIC CONDUCTIVITY				Diametral Point Load Index (MPa)	RMC -Q AVG	NOTES WATER LEVELS INSTRUMENTATION				
								TOTAL CORE %	SOLID CORE %			B Angle	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	k, cm/s								
																			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
		Refer to Previous Page		189.4 2.1																						
3	02/10/09 NQ Coring	GNEISS Fine to coarse grained Fresh Very strong Pinkish grey			1																		UCS = 111 MPa			
4					2																					
5		End of Drillhole		186.5 5.0																						
6																										
7																										
8																										
9																										
10																										
11																										
12																										

DEPTH SCALE


1 : 50



LOGGED: EHS

CHECKED: AB

SUD-RCK 07-1191-0020 B5 BH LOGS METRIC.GPJ GAL-MISS.GDT 12/12/11 DATA INPUT:

PROJECT <u>07-1191-0020</u>				RECORD OF BOREHOLE No B5- 4				1 OF 1 METRIC									
W.P. <u>5194-06-01</u>		LOCATION <u>N 5051521.3; E 236827.0</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>AMW</u>											
DATUM <u>Geodetic</u>		DATE <u>February 10, 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									
190.4	GROUND SURFACE							20	40	60	80	100					
0.0	Silty TOPSOIL																
0.2	Brown Moist		1	SS	9	190											
	Sandy SILT, some clay, trace gravel																
189.3	Loose Brown Moist		2	SS	19/0.1												
1.1	End of Borehole Spoon and Auger Refusal																
Note: 1. Borehole dry upon completion of drilling. 2. About 0.6 m of snow at BH location.																	

SUD-MTO 001 07-1191-0020 B5 BH LOGS METRIC.GPJ GAL-MISS.GDT 12/12/11 DATA INPUT:

PROJECT		07-1191-0020		RECORD OF BOREHOLE No B5- 5		1 OF 1 METRIC											
W.P.		5194-06-01		LOCATION		N 5051526.2; E 236825.9											
DIST		HWY 69		BOREHOLE TYPE		108 mm I.D. Continuous Flight Hollow Stem Augers											
DATUM		Geodetic		DATE		February 10, 2009											
						ORIGINATED BY EHS											
						COMPILED BY AMW											
						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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa 20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					WATER CONTENT (%) 10 20 30 W _p W W _L				
190.5	GROUND SURFACE																
0.0	Sand and gravel mixed with snow (FILL)																
190.0							190										
189.7	Silty TOPSOIL																
0.8	Brown Moist																
	Sandy SILT to Silty SAND, trace to some clay, trace gravel, containing cobbles below 1.6 m depth		1	SS	4												
	Very loose to loose		2	SS	18/0.18												
	Brown Moist to wet						189										
188.3	GNEISS (BEDROCK)																
2.2	Bedrock cored from 2.2 m depth to 5.2 m depth.		1	RC	REC 100%		188									RQD = 82%	
	For coring details refer to Record of Drillhole B5-5.																
			2	RC	REC 100%		187										
							186									RQD = 100%	
185.3	End of Borehole																
5.2																	
	Note: 1. Water level measured at a depth of 1.5 m below ground surface (Elev. 189.0 m) upon completion of drilling.																

SHEET 1 OF 1





DATUM: Geodetic

DRILLING CONTRACTOR: Landcore Drilling

CHECKED: AB

SUD-RCK 07-1191-0020 B5 BH LOGS METRIC.GPJ GAL-MISS.GDT 12/12/11 DATA INPUT:

PROJECT <u>07-1191-0020</u>		RECORD OF BOREHOLE No B5- 6		1 OF 1 METRIC	
W.P. <u>5194-06-01</u>		LOCATION <u>N 5051554.1; E 236824.8</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>AMW</u>	
DATUM <u>Geodetic</u>		DATE <u>February 11, 2009</u>		CHECKED BY <u>AB</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED										w _p w w _L		
185.2	GROUND SURFACE							20	40	60	80	100								
0.0	Silty TOPSOIL, containing cobbles and boulders Brown Moist						185													
184.4																				
0.8	GNEISS (BEDROCK)						184										RQD = 100%			
	Bedrock cored from 0.8 m depth to 3.8 m depth. For coring details refer to Record of Drillhole B5-6.		1	RC	REC 100%		183													
			2	RC	REC 100%		182										RQD = 100%			
181.4																				
3.8	End of Borehole																			
	Note: 1. Borehole dry upon completion of drilling.																			

PROJECT: 07-1191-0020

RECORD OF DRILLHOLE: B5- 6

SHEET 1 OF 1

LOCATION: N 5051554.1 ;E 236824.8

DRILLING DATE: February 11, 2009

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 55

DRILLING CONTRACTOR: Landcore Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN	FLUSH	RECOVERY		R.Q.D. %	FRACT. INDEX METRES	DISCONTINUITY DATA				HYDRAULIC CONDUCTIVITY		Diametral Point Load Index (MPa)	RMC -Q AVG	NOTES WATER LEVELS INSTRUMENTATION					
								TOTAL CORE %	SOLID CORE %			B Angle	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn				k, cm/s				
								888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888		888888	888888	888888	888888	888888
								888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888		888888	888888	888888	888888	888888
1	02/11/09 NQ Coring	Refer to Previous Page		184.4 0.8																					
2		GNEISS Fine to coarse grained Fresh Very strong Pinkish grey			1										J, FO, R										
3																									
4																									
5																									
6																									
7																									
8																									
9																									
10																									
		End of Drillhole		181.4 3.8																					

DEPTH SCALE

1 : 50



LOGGED: EHS

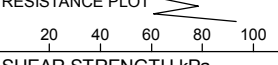
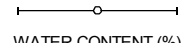

CHECKED: AB

SUD-RCK 07-1191-0020 B5 BH LOGS METRIC.GPJ GAL-MISS.GDT 12/12/11 DATA INPUT:



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

USUD-MTO 001 07-1191-0020 B5 BH LOGS METRIC.GPJ GAL-MISS.GDT 12/12/11 DATA INPUT:

PROJECT <u>07-1191-0020</u>		RECORD OF BOREHOLE No B5- 6b				1 OF 1 METRIC											
W.P. <u>5194-06-01</u>		LOCATION <u>N 5051555.5; E 236819.4</u>				ORIGINATED BY <u>TDM</u>											
DIST <u> </u> HWY <u>69</u>		BOREHOLE TYPE <u>Portable Equipment</u>				COMPILED BY <u>AMW</u>											
DATUM <u>Geodetic</u>		DATE <u>March 16, 2009</u>				CHECKED BY <u>AB</u>											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					 WATER CONTENT (%) Wp — W — Wl			γ kN/m³	GR SA SI CL
							20 40 60 80 100										
183.6 0.0	GROUND SURFACE GNEISS (BEDROCK)		1	RC	REC 100%		183										RQD = 65%
	Bedrock cored from ground surface to 3.4 m depth. For coring details refer to Record of Drillhole B5-6b.		2	RC	REC 100%		182										RQD = 94%
			3	RC	REC 100%		181										RQD = 96%
180.2 3.4	End of Borehole Note: 1. Borehole dry upon completion of drilling.																

SHEET 1 OF 1

DATUM: Geodetic

DRILLING CONTRACTOR: OGS Inc.

CHECKED: AB

SUD-RCK 07-1191-0020 B5 BH LOGS METRIC.GPJ GAL-MISS.GDT 12/12/11 DATA INPUT:

PROJECT		RECORD OF BOREHOLE				No B5- 6c		1 OF 1		METRIC							
W.P.		LOCATION		ORIGINATED BY													
DIST		BOREHOLE TYPE		COMPILED BY													
DATUM		DATE		CHECKED BY													
07-1191-0020		N 5051552.8; E 236830.2		TDM													
5194-06-01		Portable Equipment, BW Casing, Wash Boring		AMW													
HWY 69		March 17, 2009		AB													
Geodetic																	
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									
								20	40	60	80	100					
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED									
								WATER CONTENT (%)									
								20	40	60	80	100	10	20	30		
185.9	GROUND SURFACE																
0.0	Sandy organics (FILL) Loose Brown Moist		1	SS	7												
185.1																	
0.8	Sand and gravel, trace to some silt (FILL) Dense Brown Moist		2	SS	39												
184.3																	
1.6	GNEISS (BEDROCK) Bedrock cored from 1.6 m depth to 5.5 m depth. For coring details refer to Record of Drillhole B5-6c.		3	SS	50/0.08												
			1	RC	REC 100%												RQD = 45%
			2	RC	REC 100%												RQD = 100%
			3	RC	REC 100%												RQD = 100%
			4	RC	REC 100%												RQD = 100%
180.4	End of Borehole																
5.5	Notes: 1. Borehole dry upon completion of drilling. 2. Borehole located on existing snowmobile trail.																

PROJECT: 07-1191-0020

RECORD OF DRILLHOLE: B5- 6c

SHEET 1 OF 1

LOCATION: N 5051552.8 ;E 236830.2

DRILLING DATE: March 17, 2009

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: Portable Equipment

DRILLING CONTRACTOR: OGS Inc.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN	FLUSH	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.	NOTES WATER LEVELS INSTRUMENTATION
DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN	FLUSH	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.	NOTES WATER LEVELS INSTRUMENTATION
		Refer to Previous Page		184.3									
2		GNEISS Fine to coarse grained Slightly weathered to 2.4 m depth, fresh below 2.4 m Very strong Pinkish grey		1.6	1								
3		Broken core between 1.8 and 2.4 m depths.			2								
4					3								
5					4								
6		End of Drillhole		180.4									
7				5.5									
8													
9													
10													
11													

DEPTH SCALE

1 : 50



LOGGED: TDM

CHECKED: AB

SUD-RCK 07-1191-0020 B5 BH LOGS METRIC.GPJ GAL-MISS.GDT 12/12/11 DATA INPUT:

PROJECT <u>07-1191-0020</u>		RECORD OF BOREHOLE No B5- 6d				1 OF 1 METRIC											
W.P. <u>5194-06-01</u>		LOCATION <u>N 5051557.6; E 236829.1</u>				ORIGINATED BY <u>TDM</u>											
DIST <u> </u> HWY <u>69</u>		BOREHOLE TYPE <u>Portable Equipment, BW Casing, Wash Boring</u>				COMPILED BY <u>AMW</u>											
DATUM <u>Geodetic</u>		DATE <u>March 17, 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									
184.9	GROUND SURFACE						20	40	60	80	100						
0.0	Sand and gravel (FILL), trace silt Compact Brown Moist to wet		1	SS	20	▽											
			2	SS	16												
			3	SS	20												
182.8	End of Borehole Refusal to Further Penetration (Hammer Bouncing)																
2.1	Notes: 1. Water level measured at a depth of 0.9 m below ground surface (Elev. 184.0 m) upon completion of drilling. 2. Borehole located on existing snowmobile trail.																

SUD-MTO 001 07-1191-0020 B5 BH LOGS METRIC.GPJ GAL-MISS.GDT 12/12/11 DATA INPUT:

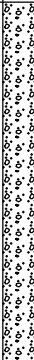

PROJECT <u>07-1191-0020</u>		RECORD OF BOREHOLE No B5- 7		1 OF 2 METRIC	
W.P. <u>5194-06-01</u>		LOCATION <u>N 5051589.2; E 236816.9</u>		ORIGINATED BY <u>EHS</u>	
DIST <u> </u> HWY <u>69</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring</u>		COMPILED BY <u>AMW</u>	
DATUM <u>Geodetic</u>		DATE <u>February 12, 2009</u>		CHECKED BY <u>AB</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)		
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× REMOULDED	w _p			w	w _L	
184.0	GROUND SURFACE																
0.0	Sandy TOPSOIL																
0.2	Brown Moist		1	AS	-												
	SAND, trace to some silt, occasional cobbles																
	Very loose to compact		2	SS	11												
	Brown Moist																
			3	SS	13												
	Becoming grey and wet below 2.3 m depth.		4	SS	2												
			5	SS	4												
	Approximately 0.9 m of heaving sands at a depth of 3.6 m.		6	SS	1												
			7	SS	WH												
			8	SS	2												
	Approximately 1.8 m of heaving sands at a depth of 7.3 m. Switch to NW Casing, wash boring.		9	SS	23												
			10	SS	3												
	Approximately 3.0 m of heaving sands at a depth of 10.7 m. Tricone used to clean out casing at 10.7 m depth.		11	SS	11												
172.4	SAND and GRAVEL, containing cobbles and boulders																
11.6	Very dense																
	Brown to grey																
	Wet																
	Difficulty advancing casing below 13.7 m depth. Tricone used to clean out casing at 13.7 m depth.		12	SS	50/0.08												

SUD-MTO 001 07-1191-0020 B5 BH LOGS METRIC.GPJ GAL-MISS.GDT 12/12/11 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 B5- 7			2 OF 2 METRIC										
W.P. <u>5194-06-01</u>			LOCATION <u>N 5051589.2; E 236816.9</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>AMW</u>										
DATUM <u>Geodetic</u>			DATE <u>February 12, 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								
	--- CONTINUED FROM PREVIOUS PAGE ---						20	40	60	80	100					
							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					WATER CONTENT (%)				
							20	40	60	80	100	10	20	30		
165.3	SAND and GRAVEL, containing cobbles and boulders Very dense Brown to grey Wet			SS	20/0											
				SS	15/0.03											
			13	SS	20/0.03											
18.7	MAFIC INTRUSIVE (BEDROCK) Bedrock cored from 18.7 m depth to 21.8 m depth. For coring details refer to Record of Drillhole B5-7.		1	RC	REC 100%											RQD = 72%
			2	RC	REC 100%											RQD = 24%
162.2 21.8	End of Borehole Note: 1. Water level measured at a depth of 1.8 m below ground surface (Elev. 182.2 m) upon completion of drilling.															

SUD-MTO 001 07-1191-0020 B5 BH LOGS METRIC.GPJ GAL-MISS.GDT 12/12/11 DATA INPUT:

PROJECT: 07-1191-0020

RECORD OF DRILLHOLE: B5- 7

SHEET 1 OF 1

LOCATION: N 5051589.2 ;E 236816.9

DRILLING DATE: February 12, 2009

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 55

DRILLING CONTRACTOR: Landcore Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN	LEGEND														NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																										
							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																																																																																																																																																																																																																																																																									
							RECOVERY		R.Q.D. %	FRACT. INDEX METRES	DISCONTINUITY DATA				HYDRAULIC CONDUCTIVITY		Diameter 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			k, cm/s																																																																																																																																																																																																																																																																												
FLUSH	80 60 40 20 0	80 60 40 20 0	80 60 40 20 0	100 75 50 25 0	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 100	0 25 50 75 10

DEPTH SCALE

1 : 50



LOGGED: EHS

CHECKED: AB



SUD-RCK 07-1191-0020 B5 BH LOGS METRIC.GPJ GAL-MISS.GDT 12/12/11 DATA INPUT:

PROJECT <u>07-1191-0020</u>			RECORD OF BOREHOLE No B5-10			1 OF 2 METRIC							
W.P. <u>5194-06-01</u>			LOCATION <u>N 5051618.5; E 236810.3</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, NW Casing, Wash Boring</u>			COMPILED BY <u>AMW</u>							
DATUM <u>Geodetic</u>			DATE <u>February 20, 2009</u>			CHECKED BY <u>AB</u>							
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DYNAMIC CONE PENETRATION RESISTANCE PLOT					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 ● QUICK TRIAXIAL × REMOULDED </div>						
187.3	GROUND SURFACE												
0.0	Silty TOPSOIL												
0.2	Brown Moist		1	AS	-								
	SAND, some silt, trace to some gravel		2	SS	11								0 81 19 0
	Loose to very dense												
	Brown Moist		3	SS	12								
			4	SS	11								
			5	SS	15								
			6	SS	32								11 80 (9)
			7	SS	34								
	Becoming wet below 5.2 m depth												
			8	SS	5								
			9	SS	13								
	Containing cobbles and boulders below 8.2 m depth												
	Approximately 3.0 m of heaving sands at 9.1 m depth. Switched to NW Casing Wash Boring. Tricone used to clean out casing at 9.1 m depth.		10	SS	15								12 77 (11)
			11	SS	38/0.2								
176.0													
11.3	SAND and GRAVEL, trace silt, containing cobbles and boulders												
	Very dense												
	Grey												
	Wet												
	Difficulty advancing casing below 12.2 m depth.		12	SS	55/0.2								
			13	SS	47/0.2								

Continued Next Page

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

SUD-MTO 001 07-1191-0020 B5 BH LOGS METRIC.GPJ GAL-MISS.GDT 12/12/11 DATA INPUT:

PROJECT <u>07-1191-0020</u>		RECORD OF BOREHOLE No B5-10				2 OF 2 METRIC										
W.P. <u>5194-06-01</u>		LOCATION <u>N 5051618.5; E 236810.3</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, NW Casing, Wash Boring</u>				COMPILED BY <u>AMW</u>										
DATUM <u>Geodetic</u>		DATE <u>February 20, 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								
--- CONTINUED FROM PREVIOUS PAGE ---																
170.2 17.1	SAND and GRAVEL, trace silt, containing cobbles and boulders Very dense Grey Wet		14	RC	-	172								o		33 64 (3)
						171										
	GNEISS (BEDROCK)		1	RC	REC 100%	170										RQD = 96%
	Bedrock cored from 17.1 m depth to 20.1 m depth. For coring details refer to Record of Drillhole B5-10.		2	RC	REC 91%	169										RQD = 91%
			3	RC	REC 100%	168										RQD = 68%
167.2 20.1	End of Borehole Note: 1. Water level measured at a depth of 5.5 m below ground surface (Elev. 181.8 m) upon completion of drilling.															

SUD-MTO 001 07-1191-0020 B5 BH LOGS METRIC.GPJ GAL-MISS.GDT 12/12/11 DATA INPUT:

PROJECT: 07-1191-0020

RECORD OF DRILLHOLE: B5-10

SHEET 1 OF 1

LOCATION: N 5051618.5 ;E 236810.3

DRILLING DATE: February 20, 2009

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 55

DRILLING CONTRACTOR: Landcore Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN	FLUSH	RECOVERY		R.Q.D. %	FRACT. INDEX METRES	DISCONTINUITY DATA				HYDRAULIC CONDUCTIVITY				Diametral Point Load Index (MPa)	RMC -Q AVG	NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																		
								TOTAL CORE %	SOLID CORE %			B Angle	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	k, cm/s	10 ⁻⁴				10 ⁻³	10 ⁻²	10 ⁻¹																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																															
JN - Joint	BD - Bedding	PL - Planar	PO - Polished	BR - Broken Rock	NOTE: For additional abbreviations refer to list of abbreviations & symbols.																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																			
FLT - Fault	FO - Foliation	CU - Curved	K - Slickensided																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																					

DEPTH SCALE

1 : 50



LOGGED: EHS

CHECKED: AB

SUD-RCK 07-1191-0020 B5 BH LOGS METRIC.GPJ GAL-MISS.GDT 12/12/11 DATA INPUT:

PROJECT 07-1191-0020			RECORD OF BOREHOLE No B5-11			1 OF 2 METRIC		
W.P. 5194-06-01			LOCATION N 5051615.0; E 236806.0			ORIGINATED BY EHS		
DIST HWY 69			BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring			COMPILED BY AMW		
DATUM Geodetic			DATE February 23 and 24, 2009			CHECKED BY AB		
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100 PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L WATER CONTENT (%)
186.9	GROUND SURFACE							
0.0	Sandy ORGANICS							
186.6	Brown							
0.3	Moist							
	SAND, trace to some gravel, trace to some silt		1	SS	19		186	
	Loose to compact		2	SS	18		185	
	Brown		3	SS	26		184	
	Moist		4	SS	20		183	
			5	SS	20		182	
	Becoming wet below 5.1 m depth		6	SS	19		181	
			7	SS	17		180	
			8	SS	8		179	
	Approximately 0.6 m heaving sands in augers at 9.1 m and 10.7 m depths		9	SS	9		178	
							177	
176.4	SAND and GRAVEL, trace to some silt		10	SS	71		176	
10.5	Very dense						175	
	Brown to grey		11	SS	77/0.2		174	
	Wet						173	
	Containing cobbles and boulders below 11.6 m depth.		12	SS	50/0.25			
	Switched to NW Casing, Wash Boring at 13.7 m depth							
172.9								
14.0								

SUD-MTO 001 07-1191-0020 B5 BH LOGS METRIC.GPJ GAL-MISS.GDT 12/12/11 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 B5-11				2 OF 2 METRIC	
W.P. <u>5194-06-01</u>		LOCATION <u>N 5051615.0; E 236806.0</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>AMW</u>	
DATUM <u>Geodetic</u>		DATE <u>February 23 and 24, 2009</u>				CHECKED BY <u>AB</u>	

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					W _p	W		
							20	40	60	80	100					
	End of Borehole Casing Refusal Notes: 1. Water level measured at a depth of 4.8 m below ground surface (Elev. 182.1 m) upon completion of drilling. 2. Water level measured in piezometer at a depth of 4.6 m below ground surface (Elev. 182.3 m) on March 3, 2009. 3. About 0.2 m of snow at BH location.															
	--- CONTINUED FROM PREVIOUS PAGE ---															

SUD-MTO 001 07-1191-0020 B5 BH LOGS METRIC.GPJ GAL-MISS.GDT 12/12/11 DATA INPUT:

PROJECT <u>07-1191-0020</u>			RECORD OF BOREHOLE No B5-13			1 OF 1 METRIC										
W.P. <u>5194-06-01</u>			LOCATION <u>N 5051510.2; E 236834.7</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>AMW</u>										
DATUM <u>Geodetic</u>			DATE <u>February 11, 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								
192.6	GROUND SURFACE						20	40	60	80	100					
0.0	Silty TOPSOIL															
0.2	Brown Moist															
191.9	SILTY CLAY		1	SS	6/0.15	192										
0.7	Brown Moist End of Borehole Spoon and Auger Refusal															
Note: 1. Borehole dry upon completion of drilling. 2. About 0.4 m of snow at BH location.																

PROJECT <u>07-1191-0020</u>			RECORD OF BOREHOLE No B5-14			1 OF 1 METRIC											
W.P. <u>5194-06-01</u>			LOCATION <u>N 5051633.1; E 236807.1</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>AMW</u>											
DATUM <u>Geodetic</u>			DATE <u>February 24, 2009</u>			CHECKED BY <u>AB</u>											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W _p W W _L WATER CONTENT (%)			γ kN/m ³	GR SA SI CL
							20 40 60 80 100	20 40 60 80 100	10 20 30								
188.8	GROUND SURFACE																
0.0	Sandy ORGANICS																
0.2	Brown Moist																
	SAND, trace to some silt, trace to some gravel																
	Compact to dense																
	Brown Moist		1	SS	15		188										
			2	SS	39		187										
186.7																	
2.1	End of Borehole Auger Refusal																
	Note: 1. Borehole dry upon completion of drilling.																

SUD-MTO 001 07-1191-0020 B5 BH LOGS METRIC.GPJ GAL-MISS.GDT 12/12/11 DATA INPUT:



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



APPENDIX B

Laboratory Test Results

**TABLE B-1
REFUSAL/BEDROCK ELEVATIONS
HIGHWAY 69 SUCKER CREEK SBL
GWP 5005-08-00, WP 5194-06-01**

Borehole	Depth to Refusal/Bedrock Surface (m)	Refusal/Bedrock Surface Elevation (m)	Comments
B5-1	1.6	190.6	Bedrock Surface
B5-2	0.7	190.8	Auger Refusal
B5-3	2.1	189.4	Bedrock Surface
B5-4	1.1	189.3	Spoon and Auger Refusal
B5-5	2.2	188.3	Bedrock Surface
B5-6	0.8	184.4	Bedrock Surface
B5-6a	0.1	185.0	Spoon Refusal
B5-6b	0	183.6	Bedrock Surface
B5-6c	1.6	184.3	Bedrock Surface
B5-6d	2.1	182.8	Spoon Refusal
B5-7	18.7	165.3	Bedrock Surface
B5-10	17.1	170.2	Bedrock Surface
B5-11	14.0	172.9	Casing Refusal
B5-13	0.7	191.9	Spoon and Auger Refusal
B5-14	2.1	186.7	Auger Refusal
B5-DC1	9.6	178.2	DCPT Refusal

Compiled by: AMW
Checked by: AB
Reviewed by: JMAC

TABLE B-2
UNIAXIAL COMPRESSION STRENGTH TEST RESULTS
HIGHWAY 69 SUCKER CREEK SBL
GWP 5005-08-00, WP 5194-06-01

Borehole Number	Sample Depth (m)	Sample Elevation (m)	Rock Type	Core Diameter (mm)	Uniaxial Compressive Strength (MPa)
B5-1	3.4	188.8	Gneiss	48	123
B5-3	3.1	188.4	Gneiss	48	111
B5-5	3.7	186.8	Gneiss	48	103
B5-6	1.6	183.6	Gneiss	48	123
B5-6b	1.2	182.4	Gneiss	51	81
B5-6c	3.3	182.6	Gneiss	51	119
B5-10	19.0	168.3	Gneiss	48	107

Compiled by: AMW
Checked by: AB
Reviewed by: JMAC

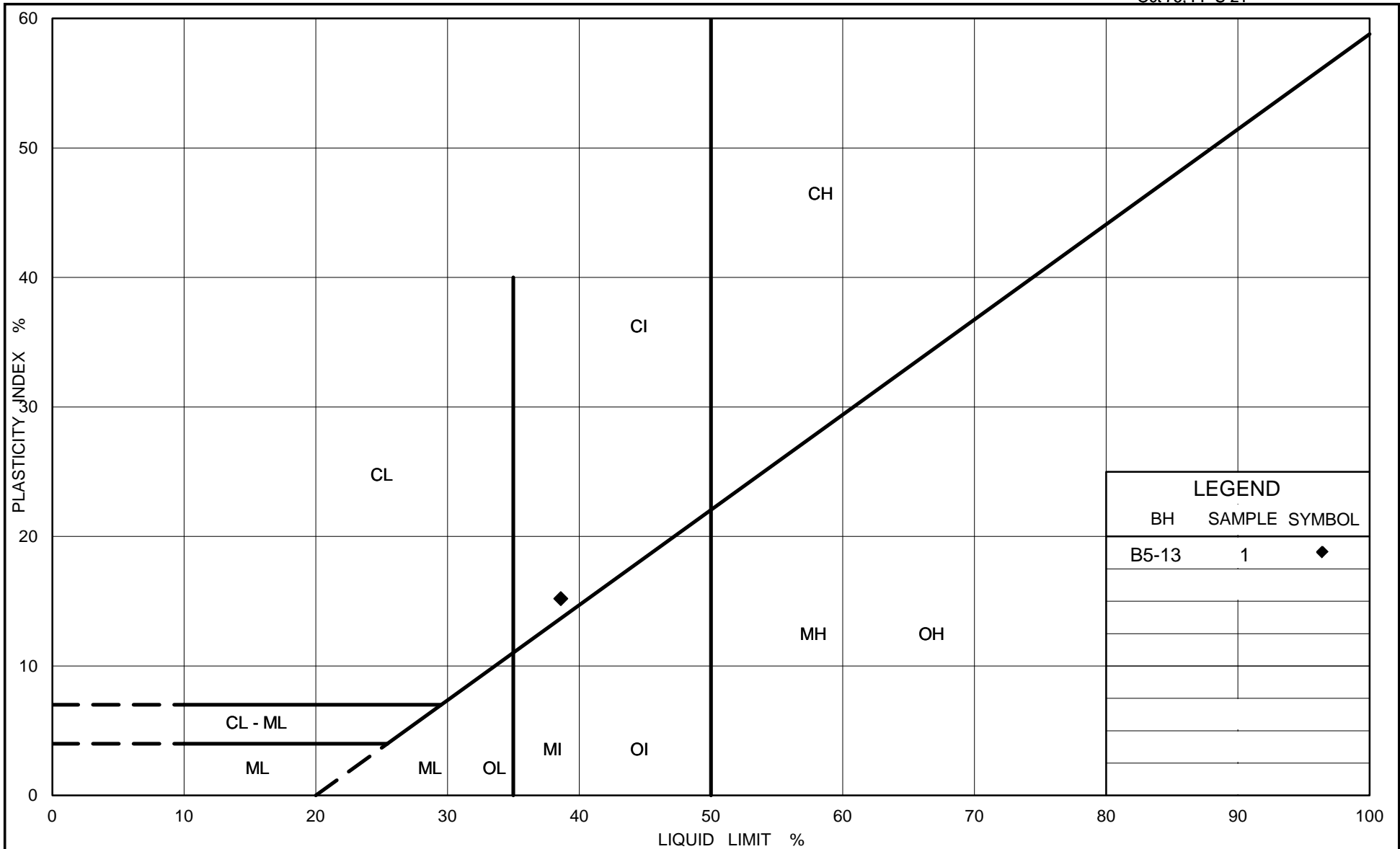
TABLE B-3
POINT LOAD STRENGTH TEST RESULTS
HIGHWAY 69 SUCKER CREEK SBL
GWP 5005-08-00, WP 5194-06-01

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)
B5-1	2.3	189.9	Gneiss	D	48	9.7	0.00916	4.0	4.0	88
B5-1	2.8	189.4	Gneiss	D	48	12.8	0.01217	5.4	5.3	117
B5-1	4.1	188.2	Gneiss	D	48	15.0	0.01418	6.3	6.1	134
B5-3	2.5	189.0	Gneiss	D	48	9.8	0.00927	4.1	4.0	88
B5-3	4.6	186.9	Gneiss	D	48	17.6	0.01672	7.4	7.2	158
B5-3	4.9	185.7	Gneiss	D	48	17.4	0.01646	7.3	7.1	156
B5-5	4.1	186.4	Gneiss	D	48	11.7	0.01113	4.9	4.8	106
B5-5	5.0	185.5	Gneiss	D	48	12.1	0.01149	5.1	5.0	110
B5-6	1.0	184.2	Gneiss	D	48	11.0	0.01047	4.6	4.5	97
B5-6	2.4	182.8	Gneiss	D	48	7.1	0.00669	3.0	2.9	64
B5-6	3.7	181.5	Gneiss	D	48	16.7	0.01583	7.0	6.8	194
B5-6b	1.0	182.6	Gneiss	D	51	11.7	0.01111	4.2	4.3	95
B5-6b	1.7	181.9	Gneiss	D	51	11.6	0.01100	4.2	4.2	92
B5-6b	2.9	180.7	Gneiss	D	51	10.4	0.00988	3.8	3.8	84
B5-6c	2.6	183.3	Gneiss	D	51	6.1	0.00574	2.2	2.2	48
B5-6c	4.3	182.6	Gneiss	D	51	8.7	0.00821	3.1	3.2	70
B5-6c	5.2	181.7	Gneiss	D	51	15.6	0.01481	5.6	5.7	125
B5-10	17.4	169.9	Gneiss	D	48	17.9	0.01695	7.5	7.3	161
B5-10	19.2	168.1	Gneiss	D	48	17.6	0.01665	7.4	7.2	158
B5-10	19.8	167.5	Gneiss	D	48	19.6	0.01856	8.2	8.1	178

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 Values of 22 has been used for the Gneiss, based on correlation with UCS for this site ("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. 53-60, in ASTM D5731.

Compiled by: AMW
Checked by: AB
Reviewed by: JMAC



Ministry of Transportation
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PLASTICITY CHART Silty Clay

Figure B-1

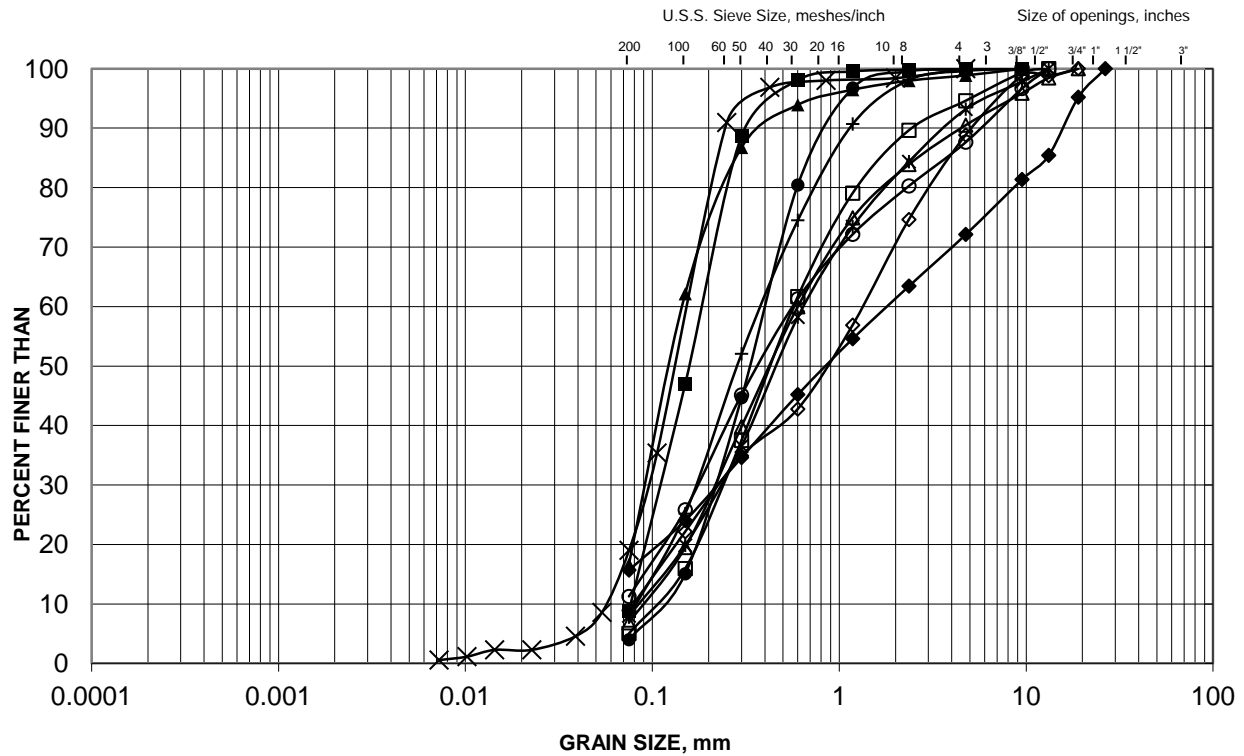
Project No. 07-1191-0020-B5

Checked By: AB

GRAIN SIZE DISTRIBUTION

Sand to Gravelly Sand

FIGURE
B-2



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
◆	B5-3	2	189.9
●	B5-7	3	182.2
■	B5-7	7	179.1
▲	B5-7	10	174.6
×	B5-10	2	186.2
◇	B5-10	6	183.2
○	B5-10	10	177.9
□	B5-11	1	186.0
△	B5-11	5	183.0
+	B5-11	9	177.7
*	B5-14	2	187.1

Project Number: 07-1191-0020-B5
Checked By: AB

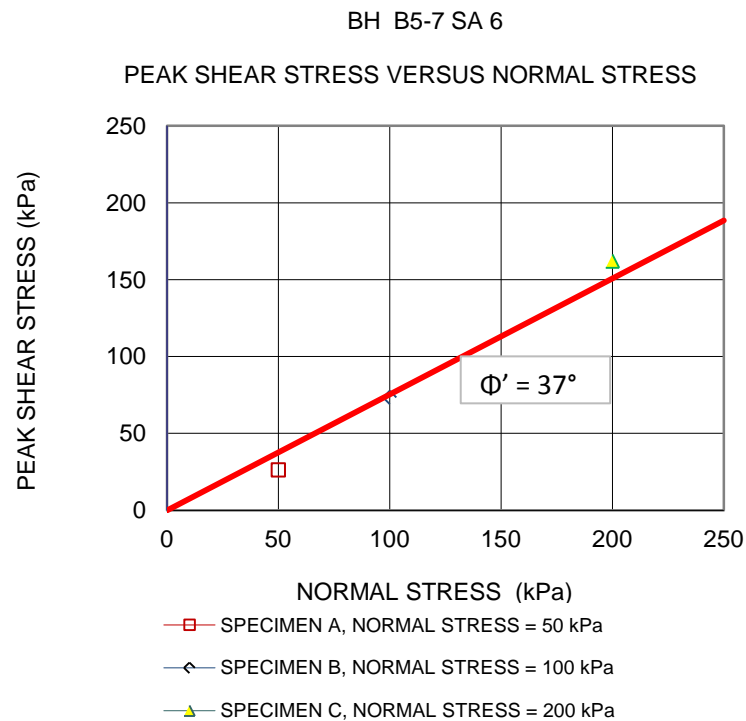
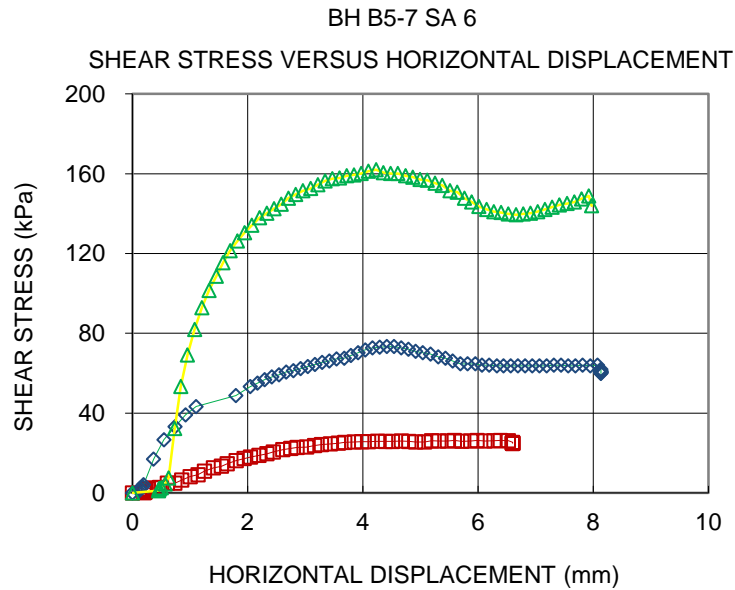
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Date: December 2011

CONSOLIDATED DRAINED DIRECT SHEAR TEST Sand		FIGURE B-3 (Sheet 1 of 3)	
TEST STAGE	A	B	C
BOREHOLE NUMBER	B5-7	B5-7	B5-7
SAMPLE NUMBER	6	6	6
SAMPLE DEPTH, (m)	3.8-4.4	3.8-4.4	3.8-4.4
SAMPLE HEIGHT, (mm)	24.80	25.10	26.10
SAMPLE LENGTH, (mm)	59.80	59.90	59.70
WATER CONTENT, BEFORE TEST, (%)	-	-	-
NORMAL (CONSOLIDATION) STRESS, (kPa)	50.00	100.00	200.00
WATER CONTENT, AFTER TEST, (%)	23.85	25.50	24.44
DISPLACEMENT RATE, mm/min	0.12	0.12	0.12
TIME TO FAILURE, HOURS	0.8	0.6	0.6
PEAK SHEAR STRESS, (kPa)	26.29	73.41	162.02
HORIZONTAL DISPLACEMENT AT PEAK, (mm)	5.59	4.42	4.23
DRY DENSITY, initial, Mg/m ³	1.50	1.53	1.51
WET DENSITY, initial, Mg/m ³	-	-	-
TEST NOTES:			
Each specimen was prepared dry with low compaction, normal stresses applied and then submerged in the shear box.			
Date: December 2011		Prepared By: LH	
Project No. 07-1191-0020-B5		Checked By: MM	
Golder Associates			

CONSOLIDATED DRAINED DIRECT SHEAR TEST
Sand

FIGURE B-3
(Sheet 2 of 3)



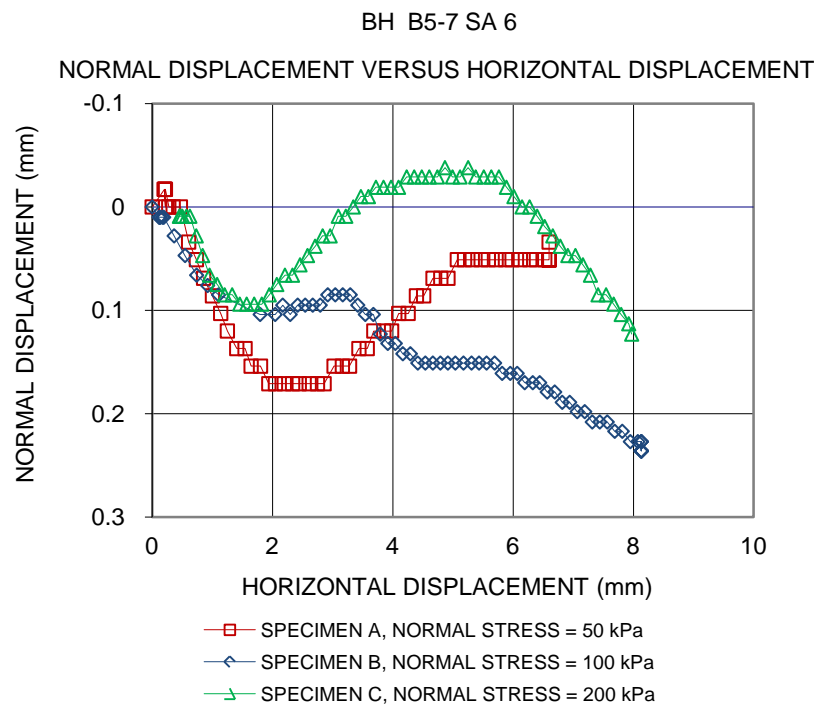
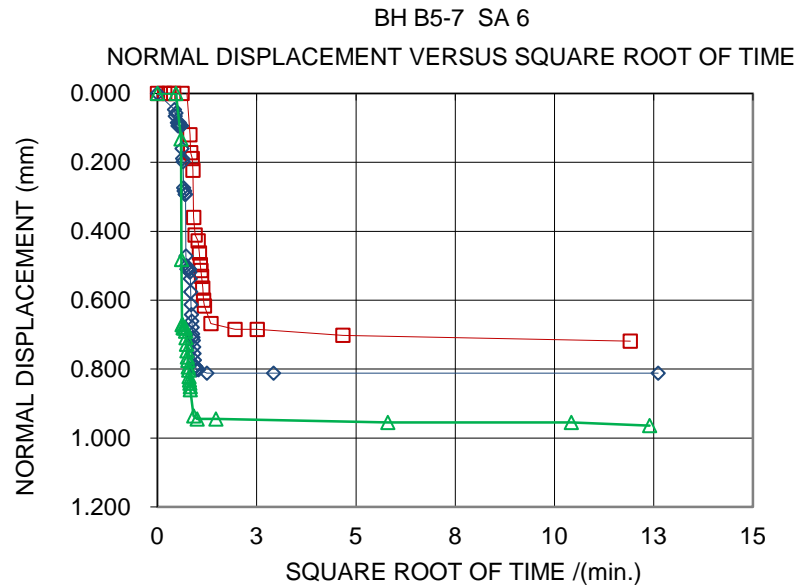
Date: December 2011
Project No. 07-1191-0020-B5

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Prepared By: LH
Checked By: MM

CONSOLIDATED DRAINED DIRECT SHEAR TEST **Sand**

FIGURE B-3
(Sheet 3 of 3)



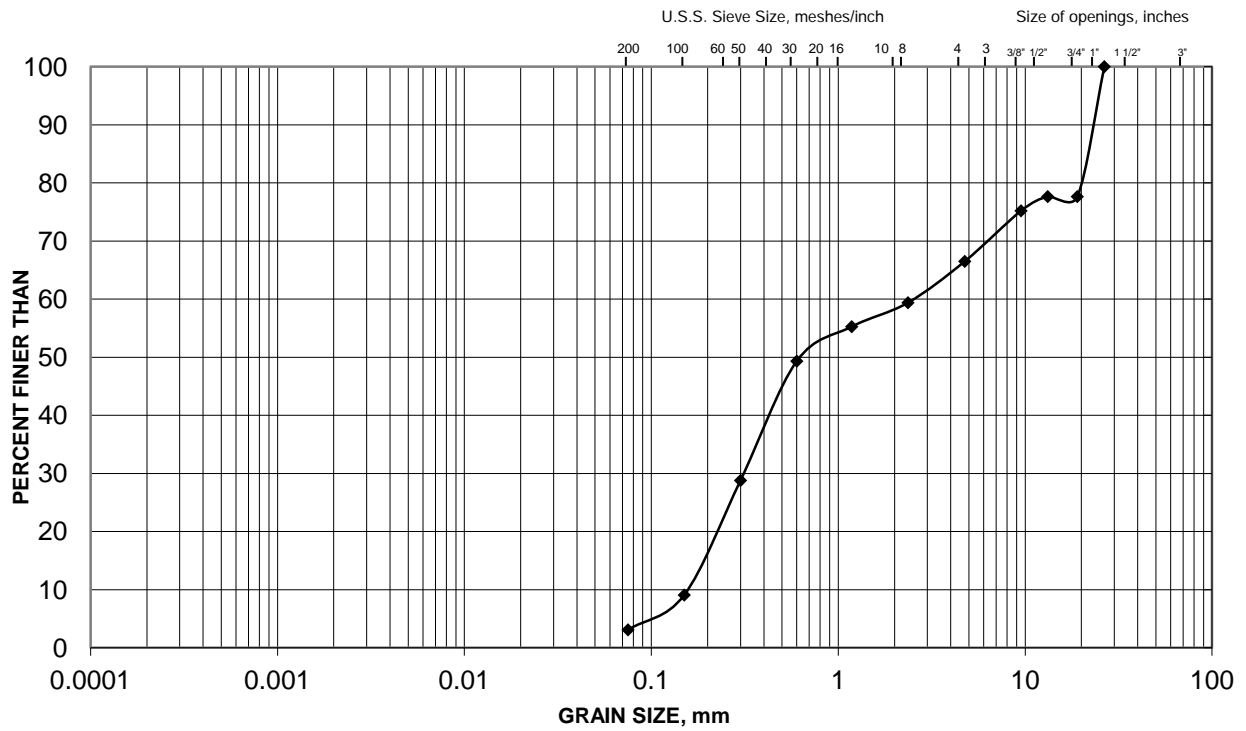
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Prepared By: LH
Checked By: MM

GRAIN SIZE DISTRIBUTION Sand and Gravel

FIGURE
B-4



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
—◆—	B5-10	14	171.8

Project Number: 07-1191-0020-B5
Checked By: AB

Golder Associates

Date: December 2011



APPENDIX C

Non-Standard Special Provisions and Operational Constraints

DOWELS INTO ROCK – Item No.

Non-Standard Special Provision

Scope of Work

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

Construction

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

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

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

Rock Dowel Testing

All proposed testing procedures shall be in general conformance with ASTM D 3689-90 and ASTM D 114381 (Re-approved 1994). Field testing must be carried out in the presence of, and the results reviewed and approved by, the Contract Administrator.

Performance Tests

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

Bridge	Foundation	Number of Dowels for Performance Testing
Sucker Creek SBL	South Abutment and South Pier	2 per footing

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

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

DOWELS INTO ROCK – Item No.

Non-Standard Special Provision

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

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

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

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

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

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

Basis of Payment

Payment at the Contract Price for the above tender items shall include full compensation for all labour, equipment and material to do work.

Special Provision

SCOPE

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

REMOVAL OF BOULDERS AT NORTH ABUTMENT – Item No.

Non-Standard Special Provision

Scope of Work

The scope of work for the above noted tender item includes the removal of native soils to a depth of 0.5 m in order to remove boulders within the footprint of the north abutment to facilitate pile driving in this area.

Basis of Payment

Payment at the contract price for the above noted tender item includes full compensation for all labour, equipment and materials to do the required work.

ROCK POINTS - Item No.

Non-Standard Special Provision

As part of the work under the above tender item, the Contractor shall supply Titus “Rock Injector Design” Pile Points or equivalent on HP 310x110, HP310X125 and HP360X152 Piles, as applicable. Piles will be driven to bedrock.

References

OPSS 906 – Structural Steel
SP903S01

Materials

The pile points shall be of the following:

Product

Manufacturer

HPP-R-12

Titus Steel Company Ltd.
6767 Invader Crescent
Mississauga, Ontario
Tel. 905-564-2446

(Or approved equivalent which includes Oslo Points as per OPSD 3000.201)

Basis of Payment

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

END OF SECTION

Operational Constraint – Embankment Construction

At the Sucker Creek SBL north and south abutments, the granular pad/core should be constructed concurrently with the rock fill.

Operational Constraint – Ground Control

At the Sucker Creek SBL north pier, the Contactor shall be alerted that the cohesionless soils at the site are water-bearing and susceptible to soil cave-in, sloughing and “boiling”. The Contractor is responsible to ensure that appropriate construction procedures and equipment are used for construction at the pier. Backfilling operations should be carried out in-the-dry.

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