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FOUNDATION INVESTIGATION AND DESIGN REPORT

**HIGHWAY 529 IC UNDERPASS, SITE NO. 44-445
HIGHWAY 69 FOUR-LANING FROM 0.4 KM
NORTH OF HIGHWAY 7182 (SHEBESHEKONG ROAD)
NORTHERLY 11 KM
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5403-05-00, WP 5190-06-01**

Submitted to:
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REPORT



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PART A

**FOUNDATION INVESTIGATION REPORT
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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by MMM Group (MMM) on behalf of the Ministry of Transportation, Ontario (MTO) to provide engineering services for the detail foundation investigation and design for the proposed Highway 529 Underpass structure at the Highway 69/Highway 529 Interchange (IC). 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 dated March 28, 2007. Golder's proposal (P7-1191-0020, dated April 24, 2007) for foundation engineering services associated with the structure is contained in Section 6.8 of MMM's Technical Proposal that forms part of the Consultant's Agreement (Purchase Order Number 5006-E-0031) for this project. The work was carried out in accordance with Golder's Supplemental Specialty Quality Control Plan for this project dated September 2007. The General Arrangement (GA) drawing, dated August 2008, for the proposed Highway 529 IC Underpass structure was provided to Golder by MMM on June 30 2009.

This report addresses the investigation carried out for the Highway 529 IC Underpass structure and the associated approach embankments only. Separate reports will be submitted detailing the foundation investigations for the related swamp and pond crossings, culverts and other bridge structures for the project.

The purpose of this investigation is to establish the subsurface conditions at the proposed structure 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 in plan on Drawing 1.

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

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

2.0 SITE DESCRIPTION

The proposed approximately 80 m long 2-span structure is located in the Township of Harrison along Highway 69, 6.0 km north of Shebeshekong Road and 2.7 km south of South Shore Road. The proposed structure will form part of the Pointe au Baril Station Interchange. The proposed grade of Highway 529 in the bridge area will be at about Elevation 208 m, which is about 9 m above the existing ground surface at the west approach and about 2 m above ground at the east approach. In the area of the proposed structure, the proposed Highway 69 northbound lanes (NBL) will be mainly in a rock cut whereas the southbound lanes (SBL) will be in fill, both with a grade at about Elevation 200 m. Detour 1B, required for the construction staging of Highway 69, will be located parallel to the existing Highway 69 (future SBL) and will cross immediately adjacent to, and west of the proposed west abutment. The proposed embankment for Detour 1B will be about 1.5 m high with the surface grade at about Elevation 200 m. The detour will be constructed prior to construction of the bridge and approach embankments.



In general, the topography in the area of the overall project limits consists of rolling terrain including densely treed areas and numerous bedrock outcrops separated by low-lying swamps. The proposed west abutment and approach are situated in a swamp. Bedrock is generally exposed within the footprint of the proposed centre pier and east abutment and rises towards the east abutment and east approach. The ground surface at the borehole locations within the limits of the proposed structure and approach embankment areas is at between Elevation 198.5 m and Elevation 208.0 m at the west and east ends of the site, respectively.

3.0 INVESTIGATION PROCEDURES

The fieldwork for the proposed structure was carried out in three stages, with a total of seventeen (17) boreholes advanced at the site. The borehole locations and groundwater surface elevations are shown on Drawings 1 and 2 and noted on the respective Record of Borehole and Drillhole sheets in Appendix A. The field investigation was carried out as follows:

- Between February 27 and 29, 2008, five (5) boreholes (Boreholes S5-1 to S5-5) were drilled for the proposed west approach embankment and swamp area using portable equipment, supplied and operated by Walker Drilling Ltd. (Walker) of Utopia, Ontario.
- Between January 12 and 19, 2009, six (6) boreholes (Boreholes B2-1 to B2-6) were drilled for the proposed west abutment and centre pier using portable equipment, supplied and operated by OGS Inc. (OGS) of Ottawa, Ontario.
- Between January 27 and 28, 2009, six (6) boreholes (Boreholes B2-7 to B2-12) were drilled for the proposed east abutment and east approach embankment using a track mounted CME-55, supplied and operated by Landcore Drilling Ltd. (Landcore) of Sudbury, Ontario or by hand auger methods.

The results from one (1) borehole advanced by AMEC (AMEC 2006) on January 31 and February 1, 2006, designated as Borehole ST-6, near the proposed west abutment as shown on Drawing 1, are provided in Appendix B.

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. Two boreholes were advanced using hand auger methods. Soil samples were obtained, where possible, continuously or 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 D1586-08a). Field vane shear tests were conducted in cohesive soils for assessment of undrained shear strengths (ASTM D2573-08) using an MTO standard "B" size vane. 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. All boreholes were backfilled with bentonite upon completion in accordance with Ontario Regulation (O.Reg.) 903 (as amended by O.Reg. 372).

The total depth of the boreholes ranged from 0.1 m to 14.6 m below existing ground surface or ice/water surface, including between 2.8 m and 8.6 m coring lengths in Boreholes B2-1, B2-3 to B2-5, B2-7, B2-8 and B2-11.



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The groundwater conditions in the open boreholes were observed during the drilling operations and a piezometer was installed in Borehole B2-1 to permit monitoring of the groundwater level at this location. The piezometer consists of 51 mm diameter PVC pipe, with a 5 m long slotted screen sealed at a selected depth within the borehole. The borehole annulus surrounding the piezometer screen was backfilled to Elevation 193.3 m with sand/cuttings and a bentonite seal was installed between Elevation 193.3 m and 196.0 m. The remainder of the borehole was then backfilled with cuttings and a 0.3 m thick bentonite seal at ground surface. 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 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. In addition, a one-dimensional consolidation (oedometer) test was carried out on one Shelby tube sample of the cohesive soil deposit in Borehole S5-5. Strength testing (uniaxial compression and point load index) was also carried out on selected specimens of the rock core.

The centreline of Highway 529 was surveyed and staked in the field by MMM in the fall of 2007 prior to drilling Boreholes S5-1 to S5-5. Golder surveyed the ice surface at Boreholes S5-1 to S5-5 at the time of the fieldwork, referenced to the existing ground surface at the proposed centreline alignment stakes. MMM also surveyed and staked the locations of Boreholes B2-1 to B2-12 in December 2008, prior to drilling. The borehole locations shown on Drawing 1 are positioned relative to MTM NAD 83 northing and easting coordinates and the ground surface elevations are referenced to Geodetic datum. The borehole/drillhole locations and ground surface elevations are as follows:

Borehole	Location (m)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing	Easting		
B2-1	5048925.4	238215.2	198.5	14.6
B2-2	5048954.3	238238.1	200.6	0.6
B2-3	5048958.6	238240.6	202.8	3.6
B2-4	5048959.0	238235.0	200.9	3.2
B2-5	5048959.4	238229.5	199.2	4.1
B2-6	5048963.7	238232.0	201.1	0.9
B2-7	5048993.4	238255.4	208.2	8.6
B2-8	5048988.7	238258.4	208.0	2.9
B2-9	5048993.1	238261.0	207.7	0.2
B2-10	5048993.8	238249.8	208.0	0.1
B2-11	5048998.1	238252.4	207.5	3.3
B2-12	5049006.4	238263.0	206.6	1.0



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.

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. Deposition of Paleozoic strata and later erosion during glaciation left behind 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 attached Record of Borehole and Drillhole sheets in Appendix A. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling and observations of drilling progress and cuttings. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Further, subsurface conditions will vary between and beyond the borehole locations. The inferred soil stratigraphy based on the results of the boreholes is shown on Drawings 1 and 2.

In general, the subsoils in the area of the proposed west abutment consist of organics/peat underlain by deposits of sand, gravelly sand, sand and silt and/or silt and silty clay. The total thickness of overburden is variable at the site, ranging from about 11 m in the west abutment area to none at the east abutment area where bedrock is exposed.

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

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

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



4.2.1 Fill

Fill was encountered at the ground surface in Borehole B2-1 and reportedly under a thin layer of asphalt at Borehole ST-6, at Elevation 198.5 m and 200.4 m, respectively. The fill in Borehole B2-1 consists of 0.6 m of wet, black peat underlain by 1.2 m of wet, grey to black sand and gravel containing organics and cobble-sized rock fill for a total thickness of 1.8 m. In Borehole ST-6, 50 mm of asphalt was underlain by 0.3 m of moist, brown sand and gravel fill.

An SPT 'N'-value measured within the fill is 4 blows per 0.3 m of penetration, although several 'N'-values greater than 100 blows per 0.3 m of penetration were recorded, indicating the presence of rock fill and/or cobbles within the fill. The overall 'N'-values indicate that the deposit ranges from very loose to very dense relative density.

The natural water content measured on one sample of the fill is 80 percent.

4.2.2 Organics/Peat

A deposit of moist, brown to black organics or peat was encountered under 0.3 m to 0.6 m of ice/water in Boreholes S5-1 through S5-5, at ground surface in B2-2 and B2-8 to B2-12 and underlying the fill layer in Borehole B2-1. At the west end of the site, the top of the peat deposit was encountered between Elevation 198.5 m and 196.7 m and the thickness of the deposit ranges from 0.3 m to 2.6 m. At the central and eastern portion of the site, the top of the organics layer was encountered between Elevation 208.0 m and 199.2 m and the thickness of the deposit ranges between 0.1 m and 0.6 m.

SPT 'N'-values measured within the organics/peat range from 0 (weight of hammer or rods) to 11 blows per 0.3 m of penetration, indicating a very soft to stiff consistency.

The natural water content measured on samples of the peat range between about 123 percent and 227 percent.

4.2.3 Sand to Silty Sand to Silt

A cohesionless deposit was encountered below the surficial organics/peat in Boreholes S5-1, S5-3 and S5-4, B2-6 and B2-12, underlying the fill and a silty clay deposit in Borehole ST-6 (AMEC 2006) and underlying the clay deposit in Boreholes S5-2, S5-5 and B2-1. The deposit is comprised of sand containing trace to some gravel, silty sand or silt containing trace to some clay. In Borehole B2-1, the sand deposit contains occasional clay seams.

In Boreholes S5-1, S5-4 and ST-6, the sand deposit is intersected by a stratum of silty clay (see Section 4.2.4). The upper portion of sand/silty sand/silt deposit is between 0.7 m and 3.6 m thick and the surface of the deposit was encountered between Elevation 206.4 m and 197.5 m. The lower (main) portion of sand deposit encountered below the silty clay is between 0.5 m and 5.5 m thick and the surface of this lower portion of the deposit was encountered between Elevation 198.3 m and 190.5 m.



SPT 'N'-values measured within the sand to silty sand to silt deposit range between 1 blow and greater than 100 blows per 0.3 m of penetration. Typically, the 'N'-values are less than about 45 blows per 0.3 m of penetration, indicating a very loose to dense relative density.

Grain size distributions of four samples of the sand deposit and one sample of a sand and silt seam are shown on Figures C-1 and C-2, respectively, in Appendix C.

The natural water content measured on samples of the deposit range between 12 percent and 37 percent.

4.2.4 Silty Clay

A deposit of wet, brown to grey, silty clay containing trace to some sand was encountered below the organics/peat in Boreholes S5-2, S5-5 and B2-1 and as a layer within the sand deposit in Boreholes S5-1 and S5-4 and Borehole ST-6 (AMEC 2006). The surface of the deposit was encountered between Elevation 196.6 m and 194.9 m and ranges in thickness from 0.5 m to 6.0 m.

SPT 'N'-values measured within the silty clay in Boreholes S5-1, S5-2, S5-4, S5-5 and B2-1 range from 0 (weight of hammer or weight of rods) to 1 blow per 0.3 m of penetration and in situ field vane testing within this stratum measured undrained shear strengths ranging from about 10 kPa to 21 kPa. The SPT 'N'-values and in situ vane shear strengths indicate that the silty clay deposit has a very soft to soft consistency. In Borehole ST-6, the 'N'-values range from 2 blows to 5 blows per 0.3 m of penetration and the undrained shear strengths measured range between 36 kPa and 106 kPa, indicating that in this borehole the silty clay has a very soft to stiff consistency.

Atterberg limits testing carried out on four samples of the silty clay deposit indicate liquid limits ranging from about 36 percent to 42 percent, plastic limits ranging from about 15 percent to 24 percent, yielding plasticity indices ranging from about 12 percent to 22 percent. The results of the Atterberg limits testing are shown on the plasticity charts on Figure C-3.

For the silty clay sample from Borehole S5-2, the Atterberg limit test results plot on the A-line, as shown on the plasticity chart on Figure C-3 and the measured water content was of about 105 percent, indicative of the presence of organics, possibly from the peat deposit directly above this deposit.

Grain size distribution tests were carried out on two samples of the silty clay deposit and the results are shown on Figure C-4.

The natural water content measured on six samples of this deposit range from about 37 percent to 105 percent.

One laboratory consolidation (oedometer) test was carried out on a specimen of the silty clay obtained from Borehole S5-5 and the test results are shown on Figure C-5. The preconsolidation pressure (σ_p') was estimated from the Void Ratio versus logarithmic Pressure plot using the Casagrande method as well as from the Total Work versus Pressure plot. The relevant consolidation test results are summarized below.



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Borehole/ Sample Number	Elevation (m)	σ_{vo}' (kPa)	σ_p' (kPa)	$\sigma_p' - \sigma_{vo}'$ (kPa)	OCR	e_o	C_r	C_c	c_v^* (cm ² /s)
S5-5/5	194.4	18	33	15	1.8	1.24	0.06	0.33	0.002

*For stress range of approximately $60 \leq \sigma_v' \leq 230$ kPa

where: σ_{vo}' effective overburden pressure in kPa

σ_p' preconsolidation pressure in kPa

OCR overconsolidation ratio

e_o initial void ratio

C_c compression index (based on void ratio)

C_r recompression index (based on void ratio)

c_v coefficient of consolidation in cm²/s in the normally consolidated range

4.2.5 Bedrock

Bedrock was encountered and cored in Boreholes B2-1, B2-3 to B2-5, B2-7, B2-8 and B2-11. The bedrock surface was inferred from refusal to dynamic cone penetration in Borehole ST-6 (AMEC 2006) and from hand auger, auger, casing or split-spoon refusal in the remaining boreholes. The bedrock surface (inferred or actual) was encountered at depths ranging from ground surface to 11.1 m below ground/ice surface and ranges from Elevation 208.2 m to 187.4 m, as presented in Table C-1. Bedrock was generally encountered at shallow depths at the boreholes advanced for the centre pier and east abutment.

Based on a review of the bedrock core samples, the bedrock at the site consisted of grey/pink, fine to medium grained, fresh to slightly weathered gneiss to granite gneiss. In Borehole B2-5, the upper 1.8 m of core was heavily fractured with zones of broken core.

The Total Core Recovery (TCR) during bedrock coring was 100 percent. The Rock Quality Designation (RQD) measured on the core samples ranges from about 26 percent to 100 percent, indicating a rock mass of poor to excellent quality. The RQD typically increased with depth and is generally higher at the eastern portion of the site. The Solid Core Recovery (SCR) ranges from 39 percent to 100 percent, increasing with depth and is generally greater than 60 percent.

Laboratory Uniaxial Compressive Strength (UCS) testing was carried out on seven core samples of the bedrock, representing one core sample for each drillhole. The UCS ranges from about 83 MPa to 112 MPa, indicating strong to very strong rock, as summarized in Table C-2.

Point load strength tests were performed on selected samples of the bedrock. Diametral point load strength index values are shown on the Record of Drillhole Sheets and are summarized in Table C-3. The diametral point load index (I_{s50}) results from the laboratory tests carried out on core samples of the gneiss range from 1 MPa to 6 MPa. These index values correspond to estimated UCS values ranging between 28 MPa and 127 MPa, based on a relationship between I_{s50} and UCS which is given by a correlation factor (K) in accordance with ASTM D5731-08, which varies depending on the size of the core samples and the strength of the rock. For this site, these UCS values are based on an estimated average correlation factor (K) of 20. These values have been given for comparison only and should be interpreted together with the results of the UCS tests.



Based on the laboratory UCS test and the point load test results, in accordance with Table 3.5 of the Canadian Foundation Engineering Manual (2006)³, the bedrock is classified as medium strong to very strong (R4, 25 MPa < UCS < 250 MPa).

4.2.6 Groundwater Conditions

The water levels were noted immediately after the drilling operations in the boreholes. In general, the soil samples taken in the boreholes were noted to be moist to wet. A standpipe piezometer was installed in Borehole B2-1 and the screen sealed within the sand deposit. The details of the piezometer installation are shown on the Record of Borehole sheet in Appendix A.

The water levels in the open boreholes at the west portion of the site were generally measured at the ground/ice surface. At the central and eastern portions of the site where bedrock was either exposed or was encountered at shallow depth below ground surface, the open boreholes were dry. The water level measured in the standpipe in Borehole B2-1 was at a depth of 0.6 m below ground surface, corresponding to Elevation 197.9 m on March 19, 2009.

5.0 CLOSURE

The field personnel supervising the drilling program were Mr. Ed Savard and Mr. Indulis Dumpis under the direction of Mr. André Bom, P.Eng. This report was prepared by Mr. Tim Rancourt, E.I.T. and Mr. André Bom, P.Eng., and the technical aspects were reviewed by Ms. Sarah E. M. Coyne, P.Eng., an Associate with Golder. A quality control review of the report was provided by Mr. Jorge M. A. Costa, P.Eng., Golder's Designated MTO Contact for this project.

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



Report Signature Page

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TR/AB/SEMC/JMAC/lb



PART B

FOUNDATION DESIGN REPORT

HWY 529 IC UNDERPASS, SITE NO. 44-445

HIGHWAY 69 FOUR LANING FROM 0.4 KM

NORTH OF HIGHWAY 7182 (SHEBESHEKONG ROAD)

NORTHERLY 11 KM

MINISTRY OF TRANSPORTATION, ONTARIO

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6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides engineering design recommendations for the proposed Highway 529 Interchange Underpass structure. 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

We understand that the Highway 529 Interchange Underpass structure will consist of two approximately 40 m long spans. Based on the GA drawing provided by MMM, the proposed Highway 529 grade is at Elevation 207.6 m, 208.0 m and 208.4 m at the west abutment, centre pier and east abutment, respectively. The final grade of the proposed Highway 69 NBL and SBL in the area of the proposed Highway 529 crossing is approximately Elevation 200 m. The new NBL are located within a bedrock cut and the new SBL centreline will be located approximately 8 m to the east of the existing Highway 69 embankment centreline. The west approach will be located within a swamp area where the existing ground surface is at about Elevation 198 m with about 0.5 m of ponded water in the swamp at the time of our investigation. At the east abutment, the existing ground surface is at about Elevation 208 m. At the centre pier, the ground surface varies between about Elevation 199 m and 203 m and bedrock is present at or just below ground surface. The proposed west approach embankment will be about 10 m high above existing ground surface whereas the east approach will require less than 1 m of filling to raise the grade to the prepared roadway surface level. The details of the proposed grades are shown on Drawing 1.

Boreholes advanced in the west approach embankment area (Boreholes S5-1 to S5-5 and B2-1) encountered a peat deposit up to 3.6 m thick, underlain by a silty clay deposit up to 3.2 m thick underlain by a sand deposit up to 5.5 m thick. The inferred bedrock surface was encountered at depths between about 0.9 m and 11.1 m, corresponding to between Elevations 197.8 m and 187.4 m. The bedrock was deepest in the abutment area and was noted to outcrop northwest of the west approach. The groundwater level in the piezometer installed in Borehole B2-1 was measured at 0.6 m below existing ground surface on March 19, 2009, corresponding to Elevation 197.9 m, which corresponds to the water level in the adjacent swamp. Reportedly, Borehole ST-6 was advanced by AMEC about 4 m southeast of the proposed abutment, and encountered 4.0 m of fill overlying 6.0 m silty clay and 2.7 m of sand. Refusal (inferred bedrock surface) was encountered at a depth of 12.8 m (Elevation 187.7 m) and this borehole was reportedly dry upon completion of drilling.



At the centre pier and east approach embankment areas, bedrock was encountered at or within 1 m below the ground surface. At the centre pier, the bedrock surface is variable, ranging between Elevation 202.8 m and 199.0 m. At the east abutment, the bedrock surface ranges from Elevation 208.2 m to 207.4 m. In Borehole B2-12 at the east approach, the inferred bedrock surface was encountered at Elevation 205.6 m.

The construction sequence is critical to the design of the bridge. We understand that a detour (Detour 1B) will be constructed parallel to and immediately west of the existing highway, which will run directly through the west approach area. Following re-routing of traffic onto Detour 1B, the NBL will be constructed and potentially the foundation preparation for the centre pier and east abutment. Following re-routing of traffic onto the new NBL, the west abutment and approach embankment and remaining structure will then be constructed together with the new SBL.

The recommendations in this report take into consideration the construction staging, scheduling and proximity of traffic to construction of the new structure and approach embankments. Foundation recommendations for the Highway 69 SBL embankment and Detour 1B through the swamp area are provided in a separate report dealing with the swamp crossings.

6.2 Bridge Foundation Options

6.2.1 West Abutment

The foundation design for the west abutment will be complex due to the following factors:

- The existing Highway 69 embankment located immediately east of the proposed abutment;
- The low-lying swampy area at the west approach embankment area;
- The approximately 10 m high west approach embankment (to be constructed over the swamp); and
- The Detour 1B embankment located within the footprint of the west approach embankment.

In order to determine the preferred foundation alternative for the west abutment, the stability and settlement issues were first assessed for the west approach embankment. As discussed in Section 6.7.4, the preferred stability and settlement mitigation option for the west approach is full sub-excavation of the peat and silty clay.

Given that full sub-excavation of the peat and silty clay (to a depth of about 5.8 m) will take place at the west approach and the excavation will be backfilled with uncompacted fill below the water level, settlements will be too large for shallow foundations to be considered feasible for the abutment. Therefore, deep foundations (i.e. steel H-piles driven to bedrock or caissons socketted into the bedrock) are recommended for support of the west abutment. The advantages, disadvantages, relative costs and risks/consequences for each of the foundation alternatives for the west abutment are summarised in Table 1.



Procedures for subgrade preparation and embankment construction are discussed in Section 6.8 and procedures for the sub-excavation and backfilling at the west approach are discussed in Section 6.9.1. Based on discussion with MTO Foundations, we recommend that 75 mm minus rock fill be used for swamp backfill/embankment construction below the abutment area to facilitate pile driving.

6.2.2 Centre Pier and East Abutment

At the centre pier and east abutment, we recommend founding the bridge on spread footings constructed on bedrock. Bedrock excavation will be required at these locations to reach the founding elevation and prepare the surface for footing construction. We understand that this excavation may be completed as part of the blasting requirements for the new NBL.

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

6.3 Shallow Foundations

We recommend supporting the east abutment and centre pier on spread footings placed on properly prepared gneiss to granite gneiss bedrock. The stratigraphy and depths/elevations to/of the bedrock surface are shown on Drawing 1 and the details of the bedrock surface elevation at the foundation elements are summarized below.

Foundation Element	Relevant Borehole Numbers	Bedrock Surface Elevation (m)	Proposed Underside of Footing* (m)
Centre Pier	B2-2 to B2-6	202.8 m to 199.0 m	196.7
East Abutment	B2-7 to B2-11	208.2 m to 207.4 m	203.8

Note * From GA drawing dated August 2008

As noted above, bedrock excavation for the foundation elements at the east abutment and centre pier may be carried out during construction of the new NBL. It is anticipated that the bedrock at the founding depth will be of good quality and that the founding surface will be properly prepared assuming that controlled rock excavation/blasting techniques are utilized for removing bedrock in the founding areas. Recommendations on bedrock excavation in the footing areas are provided in Section 6.9.3.

The east abutment will be founded on a blasted bedrock ledge about 4 m above the adjacent Highway 69 road grade. The footings must be maintained an adequate distance away from the edge of the rock cut and the rock face adequately cleaned and/or protected such that the integrity of the rock face/founding rock is maintained. In this regard, the abutment footing should be located away from the rock face at least a distance as defined by an imaginary line projected at 0.5 horizontal to 1 vertical from the toe of the rock cut. If the footing layout does not allow for this setback zone, vertical rock dowels should be installed along the crest of the cut prior to blasting of the lower roadway area in order to control and pre-support the rock face.



If less bedrock excavation/higher founding level is desired, then consideration should be given to the use of rock dowels/anchors between the footings and bedrock to enhance the sliding resistance on the lower quality bedrock at this level.

6.3.1 Geotechnical Resistance

At the centre pier and east abutment, spread footings placed on the surface of the properly prepared gneiss to granite gneiss bedrock may be designed based on a factored geotechnical axial resistance at 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 is considered to be unyielding materials and, as such, ULS conditions will govern for this foundation type.

All loose, shattered and/or fractured rock at the footing level should be removed and replaced with concrete. OPSS 902 (Excavating and Backfilling – Structures) should be included in the Contract Documents to address the requirements for construction and inspection of footings on bedrock.

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 footing, 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 should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction, $\tan \delta$, may be taken as 0.70 between the base of the poured concrete footings and the bedrock for construction in-the-dry. This value represents an unfactored value; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating the horizontal resistance.

The sliding/lateral resistance between the concrete footing and the bedrock may be supplemented by dowelling/anchoring into the bedrock particularly if the founding level is to be raised. 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 in 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 minimum embedded length within the bedrock of 1 m, 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 D.

6.3.3 Frost Protection

For spread footings founded directly on the bedrock at this site, frost susceptibility is not an issue.



6.4 Deep Foundations

As discussed in Section 6.2, we recommend that the west abutment be founded on steel H-piles, driven to bedrock. Caissons socketted into the bedrock can also be considered at the west abutment. In either case, the deep foundations will have to be installed through the swamp backfill material and the native sand/sand and silt overlying the bedrock surface. As noted previously, the excavation for the removal of peat and silty clay in the immediate abutment area will extend to a depth of about 6.8 m and below the groundwater level. For this site, the excavation should be backfilled with rock fill with a maximum particle size of 75 mm to allow for pile driving through this backfill.

6.4.1 Steel H-Pile Foundations

For design of the west abutment, we recommend that HP310X110 piles be driven to bedrock, encountered at Elevation 187.4 m in the one borehole at this location. This results in an estimated pile length of 12.1 m based on the underside of pile cap at Elevation 199.5 m. Since the bedrock is sloping, the pile lengths will vary.

6.4.1.1 Geotechnical Axial Resistance

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

6.4.1.2 Downdrag

As indicated previously, we recommend that the compressible silty clay deposit be sub-excavated at the west abutment. Therefore, downdrag loads do not need to be considered in the design.

6.4.1.3 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 is evident from the centreline profile shown on Drawing 1, the bedrock surface is sloping in an east-west direction perpendicular to the west abutment and could potentially also be sloping in a north-south direction along the west abutment footprint. Therefore, based on our experience, consideration should be given to the following preliminary set criteria and procedures, which is 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 kilojoules.
- 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 OPSS 903 (Deep Foundations). The piles should be provided with rock points, Titus Injector or equivalent. A NSSP should be included in the Contract Documents to address the requirements for rock points; an example is included in Appendix D.

6.4.1.4 *Pile Driving Note*

The pile driving note to be added to the drawings for this project is: "Piles to be driven to bedrock".

6.4.1.5 *Resistance to Lateral Loads*

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

The recommendations provided below assume that the silty clay deposit is sub-excavated and replaced with 75 mm minus rock fill at the west abutment.

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.



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

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

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

At the west abutment, the lateral load response of the piles will be developed from the passive resistance of the soil. 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 given below.

Soil Unit	Elevation (m)	n_h (kPa/m)
Compacted Rock Fill (75 mm minus) Placed Above Water Level	199.5 – 198.0	6,600
Uncompacted Rock Fill (75 mm minus) Placed Below Water Level	198.0 – 192.9	1,300
Very Loose Sand	192.9 – 187.4	1,300

For a single HP310X110 pile embedded into the rock fill and cohesionless soils, the estimated maximum lateral resistance at ULS is about 100 kN and at SLS, for 10 mm of deflection, is about 40 kN (assuming a steel yield strength of 300 MPa).

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 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.



6.4.1.6 Frost Protection

All pile caps should be provided with a minimum of 1.8 m of soil cover for frost protection in accordance with Ontario Provincial Standard Drawing (OPSD) 3090.101 (Foundation Frost Depths for Southern Ontario).

6.4.2 Caissons

Caissons socketted into the bedrock could be considered as an alternative to steel H-piles driven to bedrock at the west abutment. The caissons should be socketted a minimum of 2 m into the strong to very strong gneiss to granite gneiss bedrock. However, there will be difficulty in socketting large diameter caissons within the strong to very strong sloping bedrock at this site and achieving an adequate seal.

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 rock socket. The contribution from end-bearing will be neglected due to the difficulties in cleaning and inspecting the base of the sockets which will be below the water level and at great depth. The factored geotechnical axial resistance at ULS for three different caisson diameters socketted a minimum of 2 m into the bedrock are given below.

Caisson Diameter (m)	Gneiss Bedrock (minimum 2 m socket)	
	ULS (kN)	SLS for 25 mm
1.2	6,000	n/a
1.5	8,000	n/a
1.8	10,000	n/a

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

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

6.4.2.2 Downdrag

As discussed in Section 6.4.1.2, downdrag can be neglected for caissons installed at this site.



6.4.2.3 Resistance to Lateral Loads

The geotechnical resistance to lateral loading for the caissons should be calculated in accordance with Section 6.1.2.5 using the horizontal subgrade reaction formulas. As caissons are not proposed for the west abutment at this time, lateral capacities for the foundation element at this location are not required by the designer.

6.4.2.4 Frost Protection

The pile caps for the caissons 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, based on experience and considering the guidelines in Section 4.4.6 of the *CHBDC* may be taken as 1, consistent with Soil Profile Type I, assuming the silty clay deposit is sub-excavated and replaced with rock fill at the west abutment as recommended previously.

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, a 50 percent amplification of the ground motion may occur (i.e. Site Coefficient, $S=1.0$ for Soil Profile I from Table 4.4 of *CHBDC*), resulting in an increase in the Peak Horizontal Acceleration (PHA) from 0.05 g to 0.075 g at the ground surface.

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

6.6 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 on 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 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 (Compaction). Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3101.150 and 3121.150 (Walls Abutment; and Walls Retaining, Backfill Minimum Granular Requirement, respectively).
- For structures that are not comprised of integral or semi-integral abutments, rock fill may be used as backfill behind the walls and the material should meet the specifications as outlined in the Northern Region Directive for backfill to structures adjacent to rock fill embankments, dated November 2002. Other aspects of rock backfill requirements should be in accordance with OPSD 3101.200 (Walls Abutment, Backfill Rock).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with CHBDC Section 6.9.3 and Figure 6.6. Other surcharge loadings should be accounted for in the design, as required.
- For restrained structures, the granular fill should be placed in a zone with width equal to at least 1.8 m behind the back of the walls (in accordance with Figure C6.20(a) of the *Commentary to the CHBDC*). For unrestrained structures, granular fill should be placed within the wedge shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the base of the footing (in accordance with Figure C6.20(b), Case II, of the *Commentary to the CHBDC*).
- For restrained structures, the pressures are based on the proposed embankment fill materials and the existing overburden soils and the following parameters (unfactored) may be used assuming the use of granular fill or rock fill:

	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 placed or on the granular fill as placed and the following parameters (unfactored) may be assumed:

	Granular 'A'	Granular 'B' Type II	Rock Fill
Soil unit weight:	22 kN/m ³	21 kN/m ³	19 kN/m ³
Coefficients of static lateral earth pressure:			
Active, K_a	0.27	0.27	0.22
At rest, K_o	0.43	0.43	0.36

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 CHDBC*.

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 west approach of the new Highway 529 IC Underpass structure will be up to 9.7 m above the existing ground surface (top of peat). At the east approach, there will be less than 1 m of filling required.

As discussed previously, the subsoils at the west approach consist of peat, silty clay and sand overlying bedrock. At the east approach, bedrock is exposed or within 1 m of the ground surface. The water level used for design is Elevation 198 m, consistent with the water level in the piezometer in Borehole B2-1.

The analysis is focused on the nearly 10 m high embankment over the compressible soils at the west approach embankment. As the east approach will essentially be in a rock cut at the abutment and about 1 m high or less about 20 m behind the abutment, stability and settlement analysis are not applicable.

We understand that rock fill is the preferred embankment fill material for the Highway 69 project and, as such, 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. Rock fill embankments have side slopes at 1.25H:1V.

The analysis assumes that as a minimum, the peat will be removed from below the west approach embankment and after the detour is no longer in use.

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



6.7.1 Stability

Analyses were performed on the south side slope 6 m and 15 m behind the west abutment as these are considered to be the critical sections for the west approach embankment based on the height of embankment (up to about 9.7 m) and the thickness of the silty clay deposit (up to 3.2 m thick). Analyses were also carried out for the front slope at the west abutment, but this area was not considered to be critical for stability.

Details of the stability analysis for Detour 1B are given in a separate report addressing embankments over swamps for this area. The analysis for the proposed west approach embankment assumes the traffic has been moved to the new NBL and Detour 1B is no longer in operation.

6.7.1.1 Methodology

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

For the cohesive layers, total stress parameters were employed in the analysis. The total stress parameters (i.e. undrained shear strength – s_u) for the cohesive soils were assessed based on the results of the in situ field vane tests, oedometer test and estimated from correlations with the SPT results and other laboratory test data. Where appropriate, a correction factor as a function of the plasticity index of the soil, based on Bjerrum (1973), was employed to estimate the average mobilized undrained shear strength from the results of the in situ field vane tests.

The stability analyses assume that the existing fill (where present) and peat have been removed prior to construction of the new embankments. The slope stability analyses model geometry and stratigraphy utilized for the embankment cross-sections at 6 m and 15 m behind the abutment is similar to that shown on Figures 1 and 2, respectively, but with a continuous silty clay deposit under the full extent of the embankment. Summarized below are the simplified stratigraphy and the associated strengths and unit weights employed for the different soil types in the proposed west approach area.



West Approach Embankment

Soil Type (West Embankment Area)	Unit Weight (kN/m³)	Undrained Shear Strength (kPa)	Angle of Internal Friction
Rock Fill	19	--	40°
Rock Fill (75 mm minus)	20	--	38°
Peat	12	--	27°
Silty Clay (Very Soft to Soft)	17	10	--
Sand (Very Loose to Dense)	19	--	28°

6.7.1.3 Results of Analysis

The results of the stability analyses are summarized below for the critical sections referred 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 Deposit Thickness at Centreline	Greatest Embankment Height	Factor of Safety
West Approach South Side Slope 6 m Behind Abutment	S5-5 and B2-1	Silty Clay = 3.2 m Sand = 5.5 m	9.1 m	< 1
West Approach South Side Slope 15 m Behind Abutment	S5-1	Silty Clay = 0.5 m Sand = 2.4 m	9.6 m	< 1

Since the Factor of Safety is not only less than the target value of 1.3, but is less than 1, mitigation measures will be required as discussed in Section 6.7.4.

6.7.2 Liquefaction Potential and Seismic Analysis

As noted in Section 6.5.2, this site is located in Seismic Zone 1 with a PHA less than 0.08. Further, the bridge structure is not a lifeline structure. As such, based on Section 4.4.4 of the *CHBDC*, the site is assigned a Seismic Performance of 1 and, therefore, in accordance with Section 4.4.5.1 of the *CHBDC*, no liquefaction analysis is required.



6.7.3 Settlement

Settlement is anticipated at the west approach as a result of embankment construction over the compressible soils at this site. Settlement of the cohesionless soils will occur during construction while settlement of the cohesive deposits is time-dependent and will occur during and after construction. In addition, the rock fill will also undergo settlement, a component of which will occur after construction.

As discussed previously, Detour 1B will be constructed in advance of the west approach embankment. Details of magnitude and time rate of settlement under the detour are discussed in a separate report.

6.7.3.1 Methodology

To estimate the magnitude of the expected settlements at the west approach, analyses were carried out on the critical sections as given in Section 6.7.1 using the commercially available program Settle^{3D} (by Rocscience Inc.) and spreadsheet and hand calculations. The model geometry and stratigraphy is similar to that shown on Figures 1 and 2, except that the silty clay deposit is continuous under the full extent of the embankment.

The following sections summarize the simplified stratigraphy, parameters and results of the analysis. A discussion on the rate of settlement is also included.

6.7.3.2 Parameter Selection

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

The consolidation settlement of the silty clay was assessed using the results of the in situ field vane and SPT tests and/or laboratory consolidation tests to estimate the deformation parameters for these soils. In addition, the results of the laboratory index testing were also employed to estimate deformation parameters using empirical correlations proposed in literature by Terzaghi and Peck (1967) and Kulhawy and Mayne (1990).

The degree of over-consolidation in the cohesive strata, required in the analyses, was estimated from the results of the in situ field vane tests and the following correlations relating mobilized undrained shear strength to preconsolidation pressure:

$$s_{u(mob)} = 0.22\sigma_p' \text{ (after Mesri, 1975)}$$

where: $s_{u(mob)}$ = average mobilized undrained shear strength (kPa)

$$\sigma_p' = \text{preconsolidation pressure (kPa)}$$

and $s_{u(mob)} = \mu s_{u(FV)}$ (after Bjerrum, 1973)

where: $s_{u(FV)}$ = undrained shear strength from field vane tests (kPa)

$$\mu = \text{Bjerrum's correction factor based on Plasticity Index (i.e. about 1 for this site since the PI is less than about 20 percent)}$$



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It is known that some secondary consolidation settlement occurs following the completion of primary settlement. This secondary settlement, or creep settlement, occurs over the long term (i.e. decades) for the normally consolidated clays at this site. The magnitude of secondary (creep) settlement (Mesri, 1975 as quoted in Holtz and Kovacs, 1981) was estimated using the following:

$$S_c = C_{\alpha\epsilon} \times L_o \times (\Delta \log t)$$

Based on Mesri (1975), the following empirical correlation was utilized to estimate $C_{\alpha\epsilon}$ from water content:

$$C_{\alpha\epsilon} = w_n / 100$$

where: S_c = secondary (creep) settlement (mm)
 $C_{\alpha\epsilon}$ = modified secondary compression index (%)
 L_o = initial thickness of compressible deposit (mm) in the normally consolidated portion of the deposit
 w_n = water content (decimal)
 t = time period of interest (years)

The following simplified stratigraphy (assuming the peat has been removed) and deformation parameters have been developed for and employed in the settlement analysis for the west approach at 6 m behind the abutment (based on Boreholes B2-1 and S5-5) and 15 m behind the abutment (based on Borehole S5-1).

Location	Material	Maximum Thickness (m)	Unit Weight (kN/m ³)	Estimated Deformation Properties
6 m Behind West Abutment	Rock Fill	11.7 (9.1 m above WL 2.1 m below WL)	19	Refer to Section 6.7.3.3
	Silty Clay	3.2	17	(see below)
	Sand	2.7	19	E' = 5 MPa
15 m Behind West Abutment	Rock Fill	9.9 (9.6 m above WL 0.3 m below WL)	19	Refer to Section 6.7.3.3
	Silty Clay	0.5	17	(see below)
	Sand	2.4	19	E' = 5 MPa

The following consolidation parameters were estimated for the silty clay deposit based primarily on the results of a laboratory consolidation test performed on a specimen of the silty clay obtained from Borehole S5-5. These results were compared with values estimated from empirical correlations using the results of the in situ tests and laboratory index testing as described previously.



Location	Elevation (m)	σ_{vo}' (kPa)	σ_p' (kPa)	OCR	e_o	C_r	C_c	C_y (cm ² /s)	$C_{\alpha\epsilon}^*$
West Approach	195.9 to 192.7	18	33	1.8	1.24	0.06	0.33	0.002	0.0059

* Based on average water content of deposit.

6.7.3.3 Settlement – New Embankment Fill

Granular Fill

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

Rock Fill

Where standard rock fill is to be used for the construction of the proposed embankments (and for swamp backfilling below the west abutment area when 75 mm minus rock fill has been specified), 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	Long-Term Rock Fill Settlement	
	Compacted Rock Fill	Dumped Rock Fill
Up to 15 m	0.1% H	0.2% H

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

6.7.3.4 Results of Analysis

The estimated total settlement of the rock fill and cohesionless soils, as well as the primary and secondary consolidation settlement of the silty clay deposit at the west approach is summarized in Table 2.

Based on a coefficient of consolidation (c_v) of $0.002 \text{ cm}^2/\text{s}$ from the results of the consolidation test and assuming two-way drainage of the 3.2 m thick silty clay layer, it is estimated that about 90 percent of the consolidation settlement will be completed in about 5 months.

The magnitude of creep settlement for the silty clay strata is expected to be 20 mm per log-cycle of time at this location, corresponding to a settlement of about 20 mm for the life of the pavement.



Since the post-construction settlement will be greater than 25 mm within 20 m of the abutment, mitigation of post-construction settlement (vertical and differential) will be required at the west approach. Alternatives to mitigate this settlement are discussed in Section 6.7.4.

6.7.4 Mitigation of Stability and Settlements

As discussed in Section 6.7.1.3, the Factor of Safety for stability is less than 1.3 for the proposed 9.1 m to 9.6 m high west approach embankment constructed over the silty clay and sand and silt subsoils after the peat has been removed. As presented in Table 2, post-construction settlements between about 480 mm and 560 mm are anticipated at 6 m behind the abutment.

The following sections outline the options and recommendations for achieving the target Factor of Safety for the required embankment geometry and for minimizing post-construction settlements that could affect roadway performance. The advantages, disadvantages, relative costs and risks/consequences for the stability and settlement mitigation options at the west approach embankment are summarized and ranked in Table 3. We recommend full sub-excavation of the very soft to soft silty clay below the footprint of west approach embankment as the preferred alternative from a foundation perspective to mitigate both stability and settlement. This sub-excavation is in addition to the fill (from the existing Highway 69 and Detour 1B embankments) and peat removal. A discussion of the alternatives is given below.

6.7.4.1 Full Sub-Excavation

The bottom of the cohesive deposit is up to 5.8 m below the existing ground surface at Borehole S5-5 corresponding to Elevation 192.7 m. Reportedly, Borehole ST-6 (AMEC 2006) indicates that to the southeast of the abutment area, the bottom of the clay could be as low as Elevation 190.5 m. Full sub-excavation of the cohesive deposit to this depth in this area is considered feasible and would be the best technical solution in terms of the long-term performance of the roadway and considering the complex construction staging.

As the west approach is located within a low-lying swamp area, the majority of the sub-excavation would have to be carried out 'in the wet' (i.e. below the water level). Excavation 'in the wet' results in less risk of instability and base heave than under dry conditions but will create more uncertainty regarding full removal of the cohesive deposit. Excavation 'in the wet' to remove the cohesive deposit in this area should be carried out with side slopes no steeper than 1H:1V to limit the risk of instability. Removal of the cohesive deposit should extend to a horizontal distance beyond the toe of the proposed embankment equal to the horizontal component of the side slope profile (i.e. 1.25H for rock fill) multiplied by the depth to the bottom of the cohesive deposit below the ground surface (in accordance with OPSD 203.010, Embankments Over Swamps, New Construction).

For the full sub-excavation option, assuming the embankment is preloaded for a minimum of 6 months, the estimated post-construction settlement is between 20 mm and 35 mm, as presented in Table 4. For a preload period of 1 year, the post-construction settlement is less than 20 mm. Since this post-construction settlement and preloading requirement is directly related to settlement of the rock fill, the following alternatives to mitigate settlement of the new rock fill embankment are recommended:



- Delay paving in this area as long as possible;
- Backfill the embankment with granular material (i.e. sand and gravel) in a wedge to a minimum distance of 20 m behind the abutment; and
- Monitor the settlement of the new rock fill embankment (i.e. settlement pins at the top of the embankment) for an estimation to be made of post-construction settlement for future maintenance.

Full sub-excavation is the recommended option to mitigate settlement and stability. Stability analyses for the critical embankment sections as constructed after full sub-excavation of the silty clay indicate a Factor of Safety greater than 1.3, as shown on Figures 1 and 2.

6.7.4.2 *Preloading/Surcharging with Toe Berms*

A combination of preloading and surcharging (to mitigate settlement) and toe berms (to mitigate stability) could also be considered depending on the overall construction schedule and property requirements. This would eliminate the requirement for sub-excavation of the silty clay below the water level and use a smaller quantity of rock fill. Fill or peat sub-excavation would still be required.

For this foundation alternative, toe berms about 2 m high by 15 m wide on the south side and 10 m wide on the north side along the outside of the embankment toes, respectively, would be required in order to maintain a Factor of Safety equal to or greater than 1.3.

If the construction schedule can accommodate a preload period by constructing the embankment as early as possible and there is sufficient property available to accommodate toe berms, then preloading the foundation soils could be considered a technically feasible option to mitigate stability and settlement.

However, preloading alone before paving would not mitigate the post-construction settlements (remaining primary consolidation and secondary (creep) consolidation of the silty clay deposit) to an acceptable level. We recommend a 2 m high surcharge be placed and preloaded for 5 months to reduce the magnitude of post-construction settlement. The surcharge applies greater stress to the underlying clayey soils and increases the rate of primary consolidation over that achieved by preloading only, resulting in over-consolidation of the underlying compressible foundations soils. At the end of primary consolidation, the portion of the surcharge fill remaining above the required embankment height (sub-base level) is removed. The surcharge fill can also be left in place for a longer duration to further reduce the long-term, secondary (creep) settlements. Larger toe berms would be required for the higher embankment associated with the surcharge fill. For the alternative of a 2 m surcharge and 5-month preload period, the total post-construction settlement is between 35 mm and 45 mm. For a preload period of 1 year, the total post-construction settlement is less than 15 mm.

For this alternative, the magnitude and time rate of settlement as well as the dissipation of pore pressures during and after construction of the west approach over the swamp area should be assessed with monitoring instrumentation.



Consideration could be given to incorporating the Detour 1B embankment as part of the toe berms for the west approach, provided the peat is removed from below the footprint of the detour embankment at the west approach embankment. Since it is not currently being recommended that the peat be removed from the detour construction area, the peat would have to be removed before the approach embankment is constructed.

6.7.4.3 Other Alternatives

Mitigation alternatives that were assessed but are not considered feasible at this site are also presented in Table 3 and include the following:

- The loading imposed by the new embankment fill on the soft to firm compressible foundation soils in this area could be reduced by using ultra-lightweight expanded polystyrene (EPS) fill. However, use of this material (up to 8 m thick) would not increase the Factor of Safety for stability to greater than 1.3 without the requirement for toe berms.
- Ground improvement techniques such as rammed aggregate piers to densify the silty clay deposit resulting in increased stability (although toe berms may still be required depending on the final design of the piers) and reduced settlements. Since the site is a relatively small low-lying swamp area, there is a risk that the desired results would not be obtained and careful slope and ground monitoring would be required. This alternative would not be considered practical or cost effective at this site.
- Wick drains to enhance the rate of consolidation settlement of the silty clay deposit would not be practical at this site since the silty clay deposit is relatively thin. The drainage path would not be significantly decreased such that the time required for preloading would be reduced and the strength gain within the silty clay would not be sufficient to support the proposed embankment fill. Therefore, the extra cost of design, installation and monitoring for this alternative is not considered feasible.
- Increasing the length of the bridge could have a positive impact on reducing the settlement and enhancing stability of the approach embankments. If the bridge was lengthened so that the west abutment was at least 30 m west of the currently proposed location, then stability and settlement would not be an issue. If the final road grade was lowered, then the load on the foundation soils would be reduced and stability and settlement would not be as much of a concern (depending on the final grade). However, the substantial increase in cost associated with a longer bridge is considered to make this alternative not cost effective.

6.8 Subgrade Preparation and Embankment Construction

Prior to embankment construction, all peat and topsoil/vegetation/organic soils must be removed from below the footprint of the proposed embankment. In addition, sub-excavation of the silty clay deposit as recommended in Section 6.7.4.1 should be carried out. Sub-excavation of the peat and silty clay below the west approach embankment should be in accordance with OPSS 209 (Embankments Over Swamps and Compressible Soils).



As discussed with MMM, the following construction sequence is recommended for removal of the existing fill, peat and silty clay and for construction of the west abutment/approach:

- Construct Detour 1B embankment directly over existing ground (i.e. without peat/silty clay removal) as recommended in a separate report addressing embankment over swamps for this area. Settlement will occur but the embankment will be stable as discussed in the separate report. In this case, extensive temporary roadway protection will not be required for detour construction adjacent to the existing Highway 69.
- Move traffic to Detour 1B and construct the NBL, east abutment and centre pier (rock blasting required).
- Move traffic to the NBL. Sub-excavate existing Highway 69 fill, the Detour 1B embankment, peat and silty clay at the west approach to 5.8 m depth below the water level and backfill with rock fill. Backfill should consist of rock fill 75 mm maximum minus particle sizes within 5 m of the pile locations to facilitate pile driving.
- Construct west abutment and west approach embankment, followed by construction of the new SBL.

Placement of rock fill material should be carried out in accordance with the requirements as outlined in OPSS 209 (Embankments Over Swamps and Compressible Soils) below the water level and SP206S03 (Rock Excavation Grading; Rock Embankment) above the water level. Side slopes for rock fill embankments should be no steeper than 1.25H:1V as per OPSD 202.010 (Slope Flattening). An Operational Constraint (OC) should be included in the contract for the 75 mm minus rock fill recommended for use within the footprint of the west abutment; an example is included in Appendix D.

The final lift of fill prior to placement of the granular subbase and base courses should be compacted to 100 percent of the Standard Proctor maximum dry density. Inspection and field density testing should be carried out by qualified personnel during placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

6.8.1 Embankment Widening

In accordance with the requirements of MTO NRE 98-200, the minimum required embankment widening to account for estimated post-construction settlement and for future pavement overlays is 1 m per embankment side. To account for the settlement of the rock fill and for future pavement overlays at this site, platform widening of 1.5 m is recommended.

6.9 Design and Construction Considerations

6.9.1 Excavations

At the east abutment, removal of the bedrock will be required. At the west approach, sub-excavation of the peat and silty clay deposit will be required and the Detour 1B embankment and existing Highway 69 embankment will need to be removed in order to carry out this sub-excavation.



Excavation and backfilling should follow the procedures in OPSS 209 (Embankments Over Swamps and Compressible Soils), although some organics and/or soft compressible subsoils will likely remain in place below the new embankment due to the difficulties in excavating these materials below the water level. However, if any remaining peat/clay is confined to beneath the toe area of the proposed embankment, it should not have a significant effect (stability and settlement) on the performance of the proposed roadway.

Temporary excavation side slopes above or below water should be no steeper than 1H:1V. 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).

6.9.2 Groundwater and Surface Water Control

Excavations for the removal of the peat and silty clay deposits will extend below the water table at the west abutment and approach. Conventional or long-reach type excavators should be suitable for the excavating operations at these locations. The excavation can be carried out below the water level without dewatering measures, in accordance with OPSS 209 (Embankments Over Swamps and Compressible Soils).

Excavations at the centre pier, east abutment and east approach will generally be carried out in-the-dry given the existing and proposed ground surface elevations at these locations. Groundwater/surface water is anticipated to drain into the new highway cut.

Surface water should be directed away from the excavations at all times.

6.9.3 Blasting for Rock Excavations

For excavations into the bedrock, 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. It is recommended that the Contractor retain a blast engineer and submit proposed blast plans for review at least three weeks in advance of rock excavation.

6.9.4 Obstructions

The Contractor should be alerted in an OC to the presence of blast rock fill within the existing Highway 69 embankment which will impact sub-excavation at the west abutment area; an example is included 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., an Associate with Golder. 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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REFERENCES

- AMEC Earth and Environmental. July 2006. Preliminary Foundation and Design Report – Structural Areas, Foundation Investigation – 2, Highway 69 Route Selection Study, 3.5 km North of Highway 559 to 3.8 km North of Highway 522, GWP 5377-02-00, Highway 69, GEOCRE No. 41H-57.
- Bjerrum, L., 1973. Problems of Soil Mechanics and Construction of Soft Clays and Structurally Unstable Soils. State of the art Report, Session 4. Proceedings, 8th International Conference on Soil Mechanics and Foundation Engineering, Moscow, Vol. 3, pp. 111-159.
- Bowles, J.E., 1984. Physical and Geotechnical Properties of Soils, Second Edition. McGraw Hill Book Company, New York.
- Canadian Geotechnical Society, 2006. Canadian Foundation Engineering Manual, Fourth Edition.
- Canadian Highway Bridge Design Code (CHBDC) and Commentary on CAN/CSA S6 06, 2006. CSA Special Publication, S6.1 06. Canadian Standard Association.
- 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.
- Holtz, D.R. and Kovacs, W.D., 1981. An Introduction to Geotechnical Engineering, Prentice Hall Inc.
- Kulhawy, F.H. and Mayne, P.W., 1990. Manual on Estimating Soil Properties for Foundation Design. EL 6800, Research Project 1493 6. Prepared for Electric Power Research Institute, Palo Alto, California.
- Lo, K.Y., 1982. Rockfill in the Foundation Design of Highway Structures. Research and Development Branch. Ministry of Transportation Ontario.
- Mesri, G., 1975. Discussion on new design procedure for stability of soft clays. ASCE Journal of the Geotechnical Engineering Division, Vol. 101, GT4, pp. 409-412.
- NAVFAC Design Manual DM 7.2. Soil Mechanics, Foundation and Earth Structures. U.S. Navy, 1982. Alexandria, Virginia.
- Ontario Geological Society, 1991. Geology of Ontario, Special Volume 4, Part 2. Ministry of Northern Development and Mines, Ontario.
- Peck, R.B., Hanson, W.E., and Thornburn, T.H., 1974. Foundation Engineering, Second Edition, John Wiley and Sons, New York.
- Terzaghi, K. and Peck, R.B., 1967. Soil Mechanics in Engineering Practice, 2nd Edition, John Wiley and Sons, New York.



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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 D2573-08	Standard Test Method for Field Vane Shear Test in Cohesive Soil
ASTM D5731-08	Standard Test Method for Determination of the Point Load Strength Index of Rock and Application to Rock Strength Classifications

Ministry of Transportation:

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

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

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

Ontario Provincial Standard Specification:

OPSS 206	Construction Specification for Grading. November 2009
OPSS 209	Construction Specification For Embankments Over Swamps And Compressible Soils, April 2009
OPSS 501	Construction Specification for Compacting. November 2005
OPSS 902	Construction Specification for Excavating and Backfilling – Structures. November 2009
OPSS 903	Construction Specification for Deep Foundations, November 2009

Ontario Provincial Standard Drawings

OPSD 202.010	Slope Flattening using Surplus Excavated Material on Earth or Rock Embankment, November 2009
OPSD 203.010	Embankments Over Swamp, New Construction, November 2005
OPSD 3000.201	Foundation Piles Steel HP 310 Oslo Point, November 2005
OPSD 3090.101	Foundation Frost Depths for Southern Ontario, November 2005
OPSD 3101.150	Walls Abutment, Backfill Minimum Granular Requirement, November 2005
OPSD 3101.200	Walls Abutment, Backfill Rock, November 2005
OPSD 3121.150	Walls Retaining, Backfill Minimum Granular Requirement, November 2005

Ontario Water Resources Act:

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



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Table 1: Evaluation of Foundation Alternatives – West Abutment

Recommended Foundation Treatment and Abutment Foundation Type	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
<ul style="list-style-type: none"> ■ Sub-excavation of peat and silty clay, replacement with 75 mm minus rock fill. ■ Steel H-piles driven to bedrock. 	1	<ul style="list-style-type: none"> ■ Straightforward construction. ■ Allows for embankment construction using smaller size rock fill which settles less than uncompacted Granular 'B' Type II below the water level. ■ Allows for integral abutment design. 	<ul style="list-style-type: none"> ■ Heavy pile sections required to facilitate driving through the 75 mm minus rock fill used to backfill the swamp. ■ Pile tip needs to be provided with driving rock points. ■ Difficulty seating piles into sloping bedrock. ■ Difficulty controlling placement of 75 mm minus rock fill in the abutment area. 	<ul style="list-style-type: none"> ■ Higher cost associated with heavier pile sections. 	<ul style="list-style-type: none"> ■ Risk of larger pieces of rock fill (i.e. greater than 75 mm) placed below the abutment footprint causing difficulties for pile driving.
<ul style="list-style-type: none"> ■ Sub-excavation of peat and silty clay, replacement with 75 mm minus rock fill. ■ Caissons socketted 2 m 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 overburden including 75 mm minus rock fill. ■ Concrete for caissons would have to be placed by tremie methods below the water level. ■ Will be difficult installing caissons through rock fill and socketting caissons into strong to very strong bedrock. ■ Does not allow for integral abutment design. 	<ul style="list-style-type: none"> ■ Cost many times higher than steel H-piles driven to bedrock. 	<ul style="list-style-type: none"> ■ Risk of difficulties achieving seal and drilling large diameter bedrock socket. ■ Risk of augering caissons through smaller size rock fill. ■ Risk of not being able to extract temporary liners.
<ul style="list-style-type: none"> ■ Sub-excavation of peat and silty clay, replacement with Granular 'B' Type II, 75 mm minus rock fill or conventional size rock fill. ■ Spread footing constructed on granular pad over backfill. 	3	<ul style="list-style-type: none"> ■ Straightforward construction. ■ Pile installation is potentially easier than through rock fill. 	<ul style="list-style-type: none"> ■ Post-construction settlement of the swamp backfill will occur under the foundation; the magnitude is dependent on time period fill is in place prior to footing construction but time period will likely be too great to achieve adequate resistance. ■ Differential settlement will occur between west abutment and centre pier/east abutment which will be founded on rock. 	<ul style="list-style-type: none"> ■ Lower cost than deep foundations. 	<ul style="list-style-type: none"> ■ Risk of greater than 25 mm differential settlement between foundation elements. ■ Risk of greater than 25 mm post-construction settlement of rock fill below footing.



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Table 2: Results of Settlement Analyses - West Approach Embankment

Deposit	Estimated Settlement (mm) [Refer to Section 6.7.3.4]		Estimated Settlement Rate
	6 m Behind Abutment	15 m Behind Abutment	
New Rock Fill (above and below water level)	100	80	■ Assumes 90% of short-term settlement occurs in the first 6 months following construction and long-term settlement occurs from one year following construction
Silty Clay Primary Consolidation	360 to 440	50 to 70	■ 90% primary consolidation expected within 5 months ■ 100% primary consolidation within 13 to 15 months
Silty Clay Secondary Consolidation (end of primary to 20 years)	20	<5	■ Post-construction (after 100% primary consolidation)
Cohesionless Deposits (Sand)	70 to 90	40 to 60	■ During construction
Total Settlement	630 to 730	245 to 295	■ During and post-construction
Total Post-Construction Settlement	Rock Fill: 100 Silty Clay: 380 to 460 Total: 480 to 560	Rock Fill: 80 Silty Clay: 50 to 70 Total: 130 to 150	■ Post-construction (i.e. after completion of embankment to final height)
Total Differential Post-Construction Settlement between 6 m and 15 m behind the Abutment	330 to 430		■ Post-construction (i.e. after completion of embankment to final height)

Note: Assumes peat has been sub-excavated below the embankment and replaced with rock fill and a short duration of embankment construction.



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Table 3: Evaluation of Stability/Settlement Mitigation Alternatives - West Approach Embankment

Options	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Sub-excavation of Peat, Existing Fill and Silty Clay	1	<ul style="list-style-type: none"> Improved stability at west approach. Reduced post-construction and differential settlement. Eliminates downdrag loads on piles. Conventional construction operations. 	<ul style="list-style-type: none"> Removal of silty clay requires excavation up to about 6 m below the water level which is at ground surface. Removal of the existing highway embankment and Detour 1B required (rock fill). Requires a larger volume of rock fill. 	<ul style="list-style-type: none"> Increased costs for double handling of existing embankment material. Generally less expensive than other mitigation alternatives. Increased cost for additional rock fill. 	<ul style="list-style-type: none"> Risk of post-construction settlement if the silty clay is removed.
Preloading and Stabilizing Toe Berms	2	<ul style="list-style-type: none"> Detour embankment and underlying cohesive deposit can remain in place and incorporated into west approach as toe berms. If preloading conducted prior to pile construction, downdrag loads can be reduced. 	<ul style="list-style-type: none"> Large stabilizing toe berms will be required to mitigate stability. Time for preloading is about 3 to 5 months after embankment construction to achieve 95% consolidation. Requires a much large volume of rock fill. 	<ul style="list-style-type: none"> Less expensive than removing detour fill and cohesive deposit. Increased cost if construction schedule is delayed. Increased cost of fill material for berms. 	<ul style="list-style-type: none"> Primary and secondary (creep) settlement of foundation soils will occur. Risk of impacting construction schedule.
Lightweight Fill (EPS)	NF	<ul style="list-style-type: none"> Reduces load on compressible soils thereby increasing stability and reducing settlement. Reduces magnitude of post-construction and differential settlement. Stabilizing toe berms not required. 	<ul style="list-style-type: none"> Significant thickness (8 m) of EPS achieves a Factor of Safety <1.3. High cost of EPS fill material. 	<ul style="list-style-type: none"> Relative cost of EPS is up to an order of magnitude higher than for the other materials. 	<ul style="list-style-type: none"> Reduce risk of stability and/or settlement issues.



FOUNDATION REPORT - HIGHWAY 529 IC UNDERPASS
HIGHWAY 69, GWP 5403-05-00, WP 5190-06-01

Table 3: Evaluation of Stability/Settlement Mitigation Alternatives – West Approach Embankment (Continued)

Options	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Ground Improvement	NF	<ul style="list-style-type: none"> ■ Increase strength of soils thereby decreasing settlement and improving stability. 	<ul style="list-style-type: none"> ■ Not widely used for bridge construction. ■ Limited area for ground improvement, not cost effective. 	<ul style="list-style-type: none"> ■ Increased cost of specialized equipment and design of ground improvement system. 	<ul style="list-style-type: none"> ■ Risk of unforeseen results using construction methods that have not been widely tested. ■ Reduce risk of stability and/or settlement issues.
Wick Drains	NF	<ul style="list-style-type: none"> ■ Decreases the time required for settlement to occur though time not a significant factor because of limited thickness of clay. 	<ul style="list-style-type: none"> ■ Should be carried out in conjunction with staged construction and preloading. ■ Extra cost of design, installation and monitoring not warranted for the thin silty clay deposit (<3.2 m thick) at this site. 	<ul style="list-style-type: none"> ■ Increased cost of design, installation and monitoring. 	<ul style="list-style-type: none"> ■ Primary and secondary (creep) settlement of foundation soils will still occur.
Change of Bridge Geometry (lengthen bridge)	NF	<ul style="list-style-type: none"> ■ Longer bridge would minimize filling over swamp. ■ Longer bridge would provide for better stability of approach embankment based on soil conditions. 	<ul style="list-style-type: none"> ■ Lengthening bridge would increase overall cost of the project. 	<ul style="list-style-type: none"> ■ Cost would increase substantially if bridge is lengthened. 	<ul style="list-style-type: none"> ■ Reduce risk of stability and/or settlement issues.

NF indicates that alternative has been considered but is not feasible.

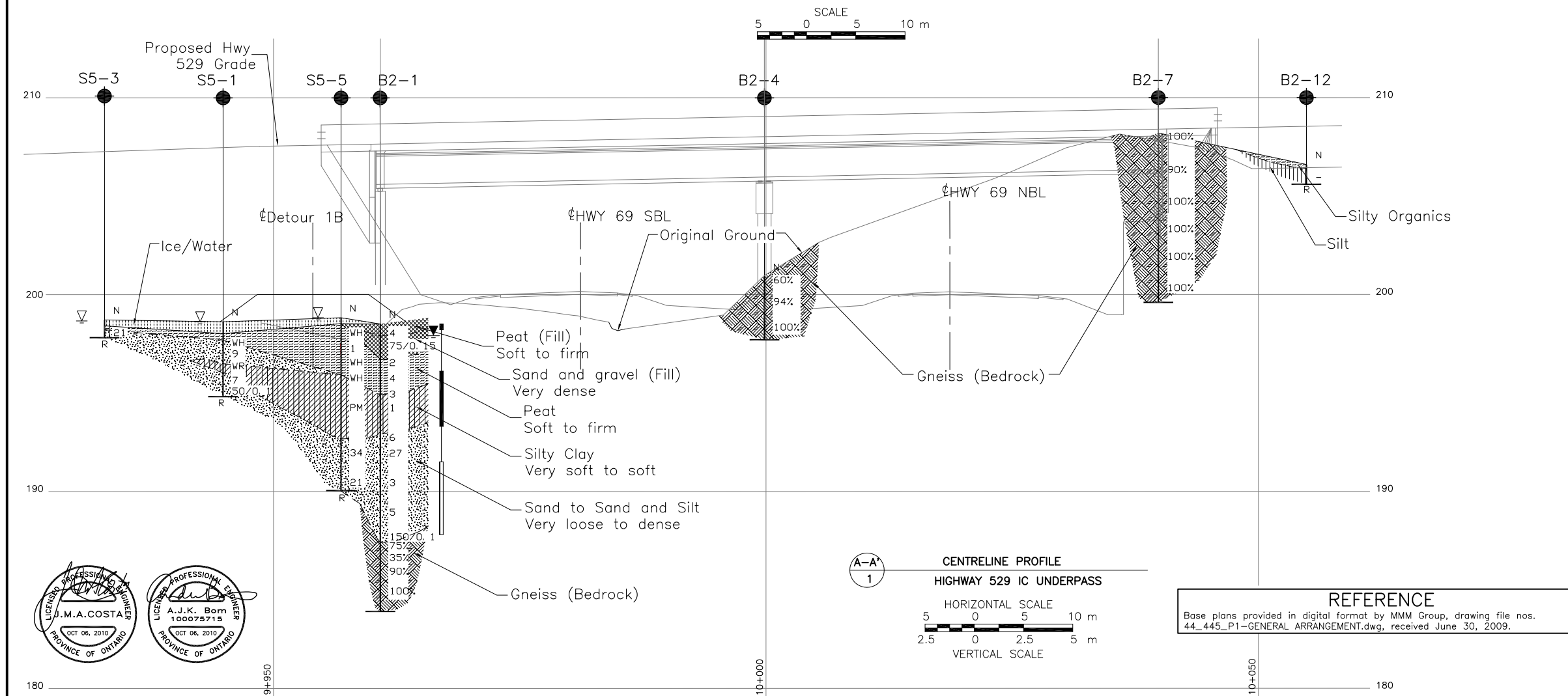
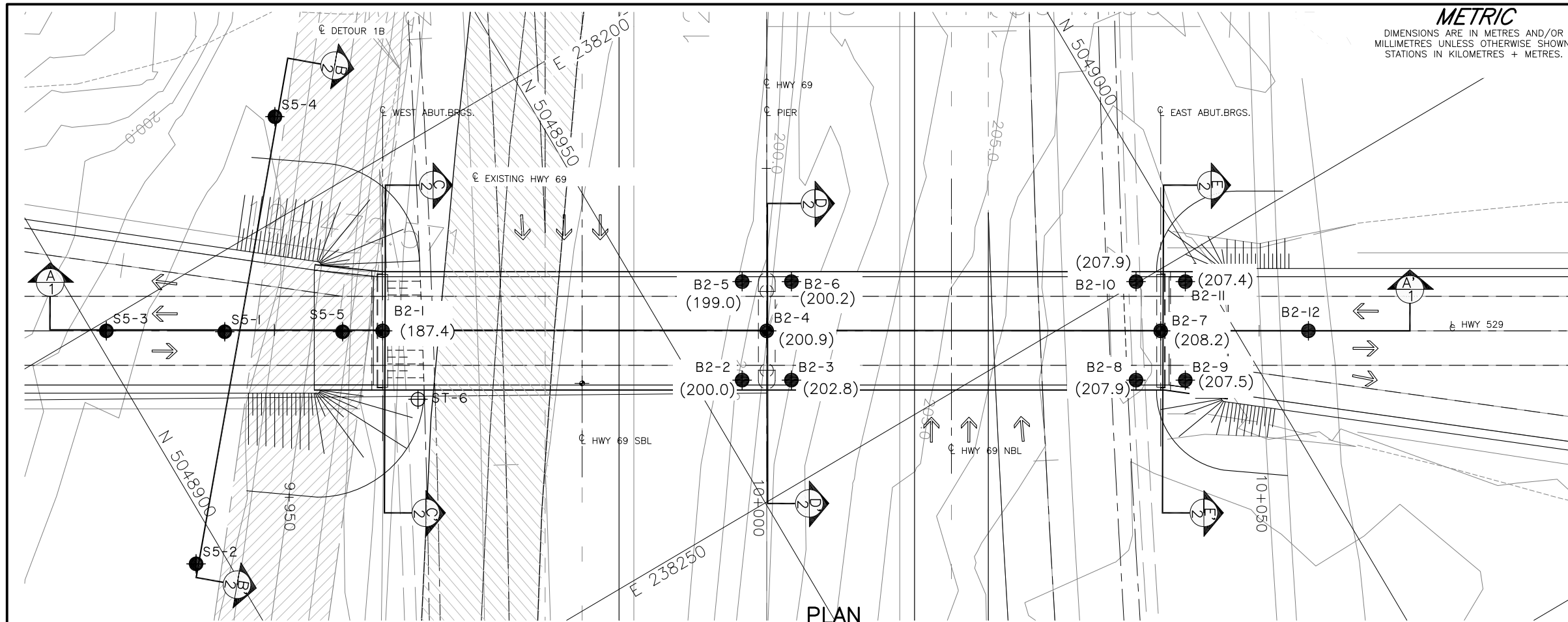


FOUNDATION REPORT - HIGHWAY 529 IC UNDERPASS HIGHWAY 69, GWP 5403-05-00, WP 5190-06-01

Table 4: Summary of Post-Construction Settlement for Mitigation Options of West Approach Embankment

	Estimated Post-Construction Settlement with Full Sub-Excavation of Peat and Silty Clay (mm)				Estimated Post-Construction Settlement with Preloading and 2 m Surcharging (mm)			
	After 6-month Preload Duration		After 1-year Preload Duration		After 5-month Preload Duration		After 1-year Preload Duration	
	6 m Behind Abutment	15 m Behind Abutment	6 m Behind Abutment	15 m Behind Abutment	6 m Behind Abutment	15 m Behind Abutment	6 m Behind Abutment	15 m Behind Abutment
Rock Fill	35	20	20	10	20	20	10	10
Silty Clay	N/A	N/A	N/A	N/A	25	15	<5	<5
Total	35	20	20	10	45	35	<15	<15
Total Differential between 6 m and 15 m behind the Abutment	15		20		10		~0	

*Assumes only peat has been sub-excavated below the embankment and replaced with rock fill. A 2 m surcharge has been assumed to accelerate the rate of settlement.



CONT No.
WP No. 5190-06-01

HIGHWAY 69
HWY 529 IC UNDERPASS
BOREHOLE LOCATION
AND SOIL STRATA

Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA

SHEET

KEY PLAN

LEGEND

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

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
B2-1	198.5	5048925.4	238215.2
B2-2	200.6	5048954.3	238238.1
B2-3	202.8	5048958.6	238240.6
B2-4	200.9	5048959.0	238235.0
B2-5	199.2	5048959.4	238229.5
B2-6	201.1	5048963.7	238232.0
B2-7	208.2	5048993.4	238255.4
B2-8	208.0	5048988.7	238258.4
B2-9	207.7	5048993.1	238261.0
B2-10	208.0	5048993.8	238249.8
B2-11	207.5	5048998.1	238252.4
B2-12	206.6	5049006.4	238263.0
S5-1	198.6	5048911.6	238207.1
S5-2	198.5	5048897.1	238225.9
S5-3	198.7	5048901.3	238200.9
S5-4	198.7	5048927.1	238190.9
S5-5	198.8	5048921.9	238213.2
ST-6	200.5	5048925	238223

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.

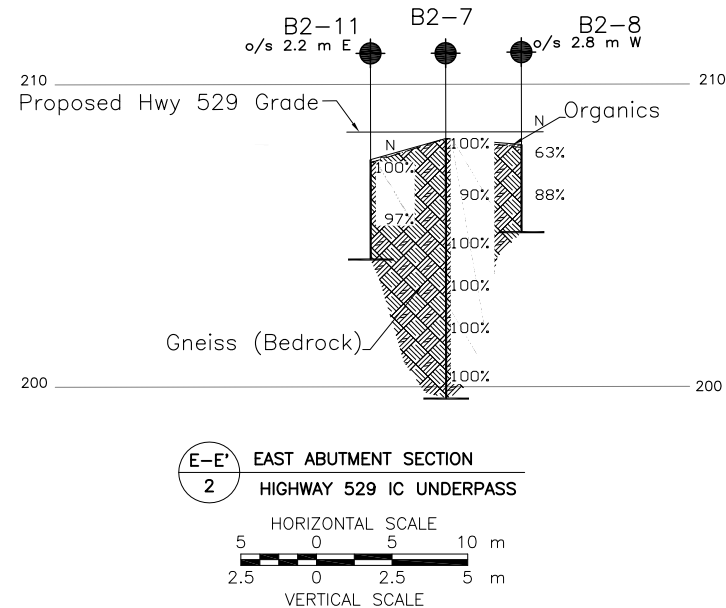
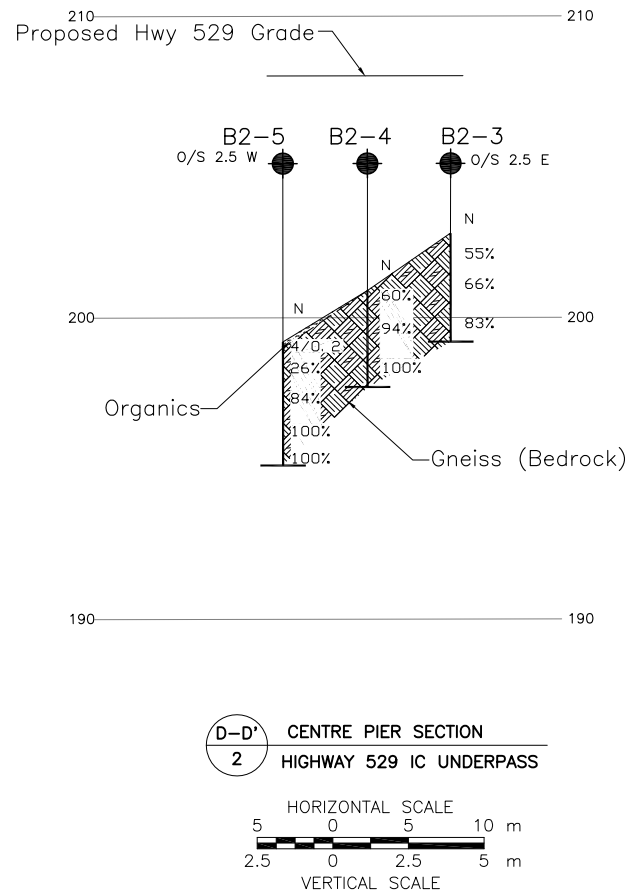
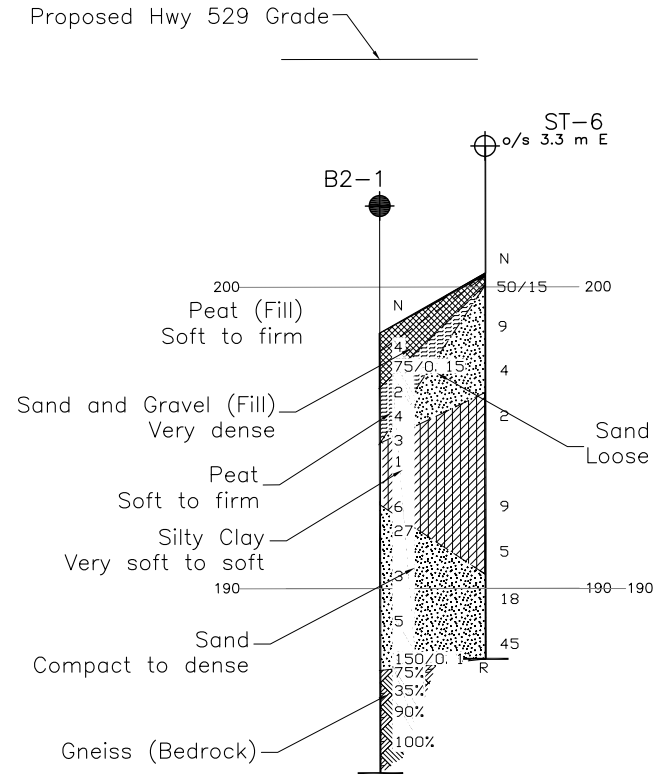
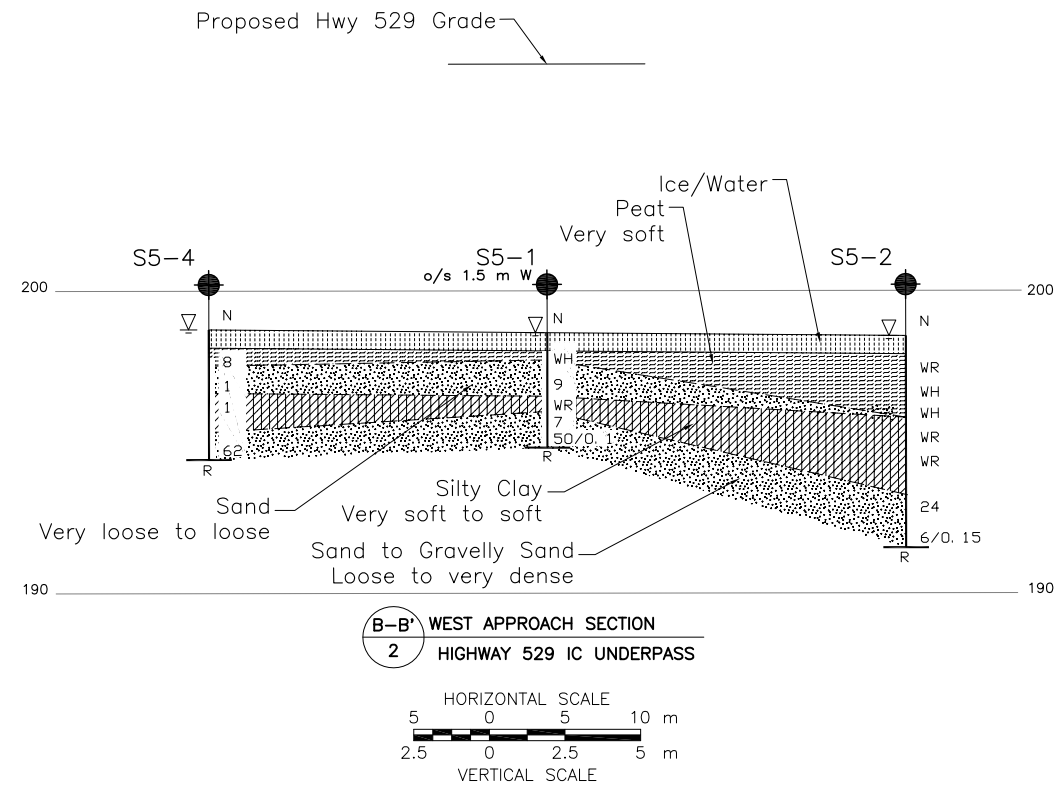
The boundaries between soil strata have been established only at borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

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.

NO.	DATE	BY	REVISION

Geocres No. 41H-74

HWY. 69	PROJECT NO. 07-1191-0020	DIST.
SUBM'D. TRR	CHKD. AB	DATE: OCT 2010
DRAWN: MM	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. 5190-06-01

HIGHWAY 69
HWY 529 IC UNDERPASS
SOIL STRATA

SHEET



Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA

LEGEND

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

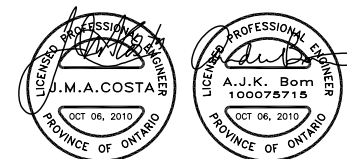
No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
B2-1	198.5	5048925.4	238215.2
B2-2	200.6	5048954.3	238238.1
B2-3	202.8	5048958.6	238240.6
B2-4	200.9	5048959.0	238235.0
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B2-12	206.6	5049006.4	238263.0
S5-1	198.6	5048911.6	238207.1
S5-2	198.5	5048897.1	238225.9
S5-3	198.7	5048901.3	238200.9
S5-4	198.7	5048927.1	238190.9
S5-5	198.8	5048921.9	238213.2
ST-6	200.5	5048925.0	238223.0

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.

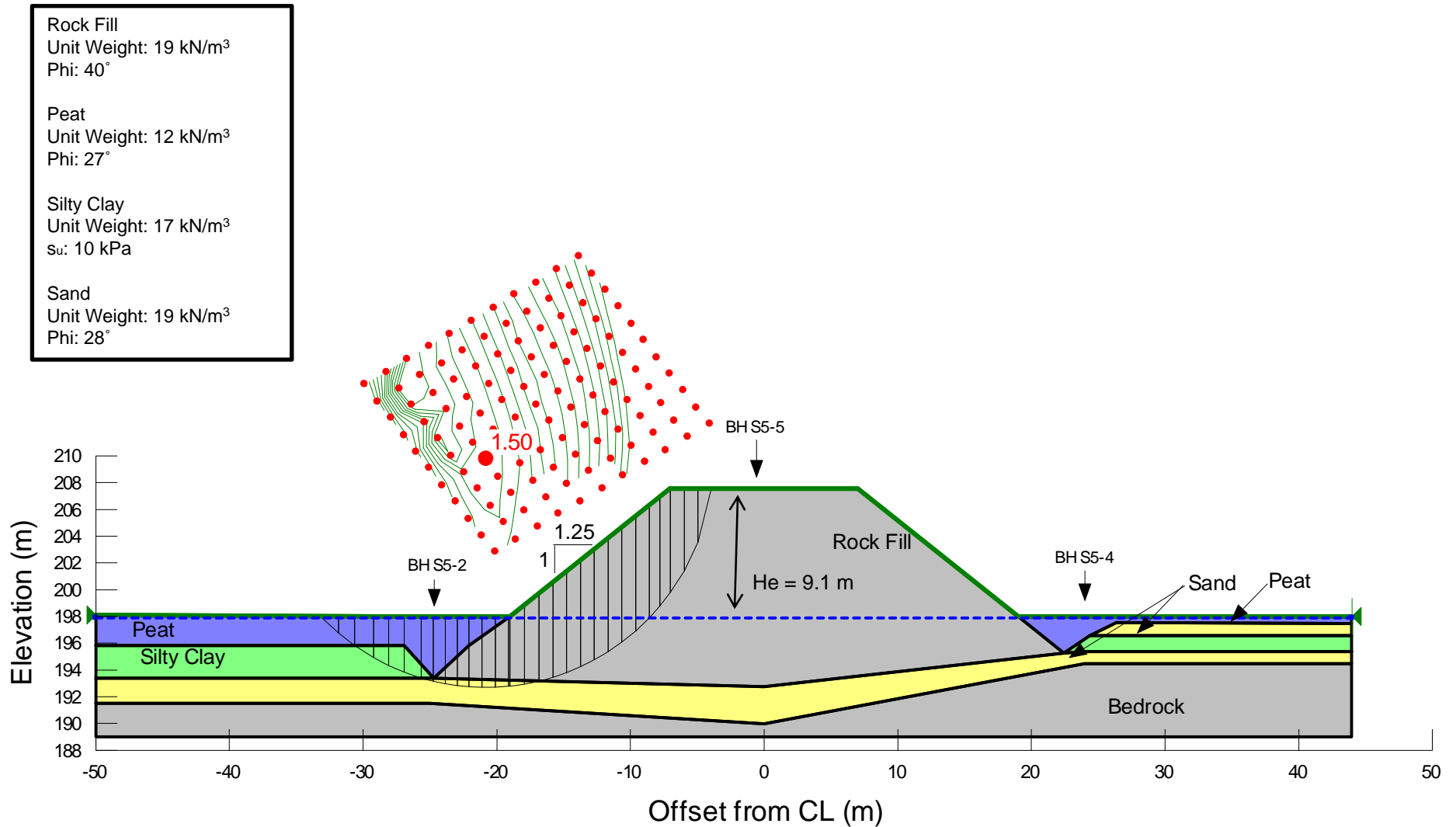
The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS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



NO.	DATE	BY	REVISION
Geocres No. 41H-74			
HWY. 69	PROJECT NO. 07-1191-0020		DIST.
SUBM'D. TRR	CHKD. AB	DATE: OCT 2010	SITE: 44-445
DRAWN: MM	CHKD. SEMC	APPD. JMCA	DWG. 2

Slope Stability Analysis West Approach Embankment (6 m Behind Abutment) - Full Sub-Excavation

FIGURE 1



DATE: October 2010

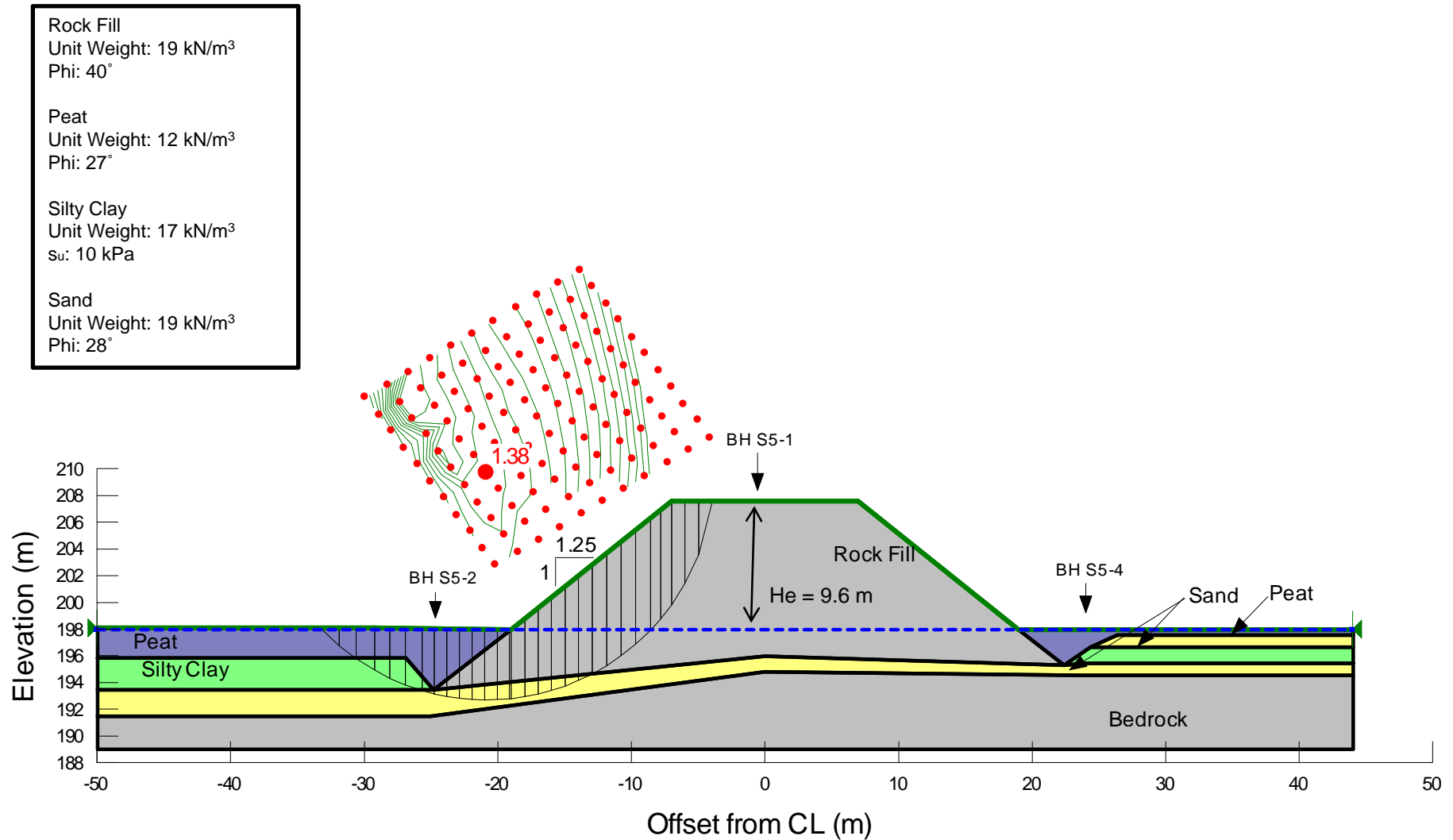
PROJECT: 07-1191-0020 – B2



Drawn by: AB Checked by: SEMC

Slope Stability Analysis **West Approach Embankment (15 m Behind Abutment) - Full Sub-Excavation**

FIGURE 2



DATE: October 2010

PROJECT: 07-1191-0020 – B2



Drawn by: AB Checked 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

Piezocone Penetration Test (CPT)

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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

(b) Cohesive Soils

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

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO ₄	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

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



LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE

Fresh: no visible sign of weathering

Faintly weathered: weathering limited to the surface of Major discontinuities

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

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

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

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

BEDDING THICKNESS

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

JOINT OR FOLIATION SPACING

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

GRAIN SIZE

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

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

CORE CONDITION

Total Core Recovery (TCR)

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

Solid Core Recovery (SCR)

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

Rock Quality Designation (RQD)

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

DISCONTINUITY DATA

Fracture Index

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

Dip with Respect to (W.R.T.) Core Axis

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

Description and Notes

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

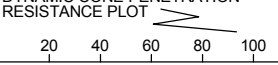



Abbreviations

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

PROJECT 07-1191-0020			RECORD OF BOREHOLE No S5-1			1 OF 1 METRIC															
W.P. 5403-05-00			LOCATION N 5048911.6; E 238207.1			ORIGINATED BY ID															
DIST HWY 69			BOREHOLE TYPE Portable Equipment, NW Casing, Wash Boring			COMPILED BY MM															
DATUM Geodetic			DATE February 29, 2008			CHECKED BY AB															
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ			GR SA SI CL		
198.6	ICE SURFACE							20 40 60 80 100													
0.0	ICE																				
198.3																					
198.0	WATER																				
197.7	PEAT (Fibrous) Very soft Black Wet		1a 1b	SS	WH		198											123.2			
0.9	SAND containing organics Very loose to loose Brown Wet		2	SS	9		197														
196.5																					
2.1	SILTY CLAY, trace sand Very soft Brown Wet		3	SS	WR		196											41.1			
196.0																					
2.6	SAND, trace to some gravel Loose Grey Wet		4	SS	7		195														
194.8			5	SS	50/10 T																
3.8	End of Borehole Spoon Refusal																				
	Note: 1. Water level measured at ice surface (Elev. 198.6 m) upon completion of drilling.																				

PROJECT 07-1191-0020			RECORD OF BOREHOLE No S5-2			1 OF 1 METRIC											
W.P. 5403-05-00			LOCATION N 5048897.1; E 238225.9			ORIGINATED BY ID											
DIST _____ HWY 69			BOREHOLE TYPE Portable Equipment, NW Casing, Wash Boring			COMPILED BY MM											
DATUM Geodetic			DATE February 27, 2008			CHECKED BY AB											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ kN/m ³	GR SA SI CL
								20 40 60 80 100	20 40 60 80 100	W _p	W	W _L	10 20 30				
198.5	ICE SURFACE																
0.0	ICE																
198.2																	
197.9	WATER																
0.6	PEAT (Fibrous) Very soft Black Wet		1	SS	WR		198										
197.2																	
1.3	PEAT (Amorphous) Very soft Black Wet		2	SS	WH		197									202.2	
			3	SS	WH		196										
195.8																	
2.7	SILTY CLAY, some sand Very soft to soft Grey Wet		4	SS	WR		195									195.4	0 17 62 21
			5	SS	WR		194									65.1	
193.3																	
5.2	SAND, trace to some silt Loose to compact Grey Wet		6	SS	24		193										
191.5			7	SS	6/0.15		192										10 74 (16)
7.0	End of Borehole Spoon Refusal (Hammer Bouncing) Note: 1. Water level measured at ice surface (Elev. 198.5 m) upon completion of drilling.																

MIS-MTO 001 07-1191-0020 S5 METRIC.GPJ GAL-MISS.GDT 07/10/10 DATA INPUT:

PROJECT 07-1191-0020		RECORD OF BOREHOLE No S5-3				1 OF 1 METRIC					
W.P. 5403-05-00		LOCATION N 5048901.3; E 238200.9				ORIGINATED BY ID					
DIST HWY 69		BOREHOLE TYPE Portable Equipment, NW Casing, Wash Boring				COMPILED BY MM					
DATUM Geodetic		DATE February 28, 2008				CHECKED BY AB					
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT  SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L WATER CONTENT (%)	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
198.7	ICE SURFACE										
0.0 198.4	ICE										
	PEAT (Fibrous) Black Wet		1	SS	21		198				
197.8 0.9	SAND, some gravel Compact Grey Wet End of Borehole Spoon Refusal (Hammer Bouncing) Note: 1. Advanced a DCPT 1.0 m west of Borehole S5-3. Refusal at a depth of 0.5 m. 2. Water level measured at ice surface (Elev. 198.7 m) upon completion of drilling.										

PROJECT		RECORD OF BOREHOLE No S5-4				1 OF 1 METRIC								
W.P. 5403-05-00		LOCATION N 5048927.1; E 238190.9				ORIGINATED BY ID								
DIST HWY 69		BOREHOLE TYPE Portable Equipment, NW Casing, Wash Boring				COMPILED BY MM								
DATUM Geodetic		DATE February 27, 2008				CHECKED BY AB								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
198.7	ICE SURFACE													
0.0	ICE													
198.1														
0.6	PEAT (Fibrous) Firm to stiff Black Wet		1	SS	8									
197.5														
1.2	SAND, containing organics Very loose Brown Wet		2	SS	1									
196.6														
2.1	SILTY CLAY, trace sand Very soft to soft Grey Wet		3	SS	1									
195.3														
3.4	SAND, some gravel, some silt Very dense Grey Wet		4	SS	62									
194.4														
4.3	End of Borehole Spoon Refusal (Hammer Bouncing) Note: 1. Water level measured at ice surface (Elev. 198.7 m) upon completion of drilling.													

MIS-MTO 001 07-1191-0020 S5 METRIC.GPJ GAL-MISS.GDT 07/10/10 DATA INPUT:

PROJECT 07-1191-0020			RECORD OF BOREHOLE No S5-5			1 OF 1 METRIC															
W.P. 5403-05-00			LOCATION N 5048921.9; E 238213.2			ORIGINATED BY ID															
DIST HWY 69			BOREHOLE TYPE Portable Equipment, NW Casing, Wash Boring			COMPILED BY MM															
DATUM Geodetic			DATE February 28, 2008			CHECKED BY AB															
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ			GR SA SI CL		
198.8	ICE SURFACE							20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W _p — W — W _L 10 20 30			228.9 17.8 NP					
0.0	ICE/SNOW																				
0.3	PEAT (Fibrous) Very soft Black Wet		1	SS	WH		198														
			2	SS	1																
197.1	PEAT (Amorphous) Very soft Black Wet		3	SS	WH		197														
			4a	SS	WH		196														
195.9	SILTY CLAY, trace sand Very soft to soft Grey Wet		4b	SS	WH		195	3 + 3													
			5	TO	PM		194	2 + 3					67.1 17.8								
192.7	SAND, trace to some gravel, trace to some silt, trace clay Compact to dense Grey Wet		6	SS	34		192						○			NP			9 79 7 5		
			7	SS	21		191														
190.0	End of Borehole Casing Refusal						190														
8.8	Note: 1. Water level measured at ice surface (Elev. 198.8 m) upon completion of drilling.																				

MIS-MTO 001 07-1191-0020 S5 METRIC.GPJ GAL-MISS.GDT 07/10/10 DATA INPUT:

PROJECT 07-1191-0020				RECORD OF BOREHOLE No B2- 1				1 OF 2 METRIC					
W.P. 5190-06-01				LOCATION N 5048925.4; E 238215.2				ORIGINATED BY ID					
DIST HWY 69				BOREHOLE TYPE Portable Equipment, BW Casing, Wash Boring				COMPILED BY MM					
DATUM Geodetic				DATE January 15 to 19, 2009				CHECKED BY AB					
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT		UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	W _p W W _L	WATER CONTENT (%)		
198.5	GROUND SURFACE												
0.0	Peat (FILL) Soft to firm Black Wet		1	SS	4							80.1	
197.9			2	SS	75/0.15								
0.6	Sand and gravel, some organics, containing blast rock (FILL) Very dense Grey to black Wet												
196.7			3	SS	2							60.5	
1.8	PEAT Soft to firm Black Wet		4	SS	4								
			5	SS	3								
194.9			6	SS	1							41	0 6 52 42
3.6	SILTY CLAY, trace to some sand Very soft to soft Grey Wet												
			7	SS	6								
192.9			8	SS	27								4 78 11 7
5.6	SAND, trace to some silt, trace to some clay, trace gravel, occasional clay seams Very loose to compact Grey Wet												
			9	SS	3								
			10	SS	5								11 41 (48)
	Sand and silt seam at 9.1 m to 9.7 m depth.												
			11	SS	150/0.1								
187.4													
11.1	GNEISS (BEDROCK)		1	RC	REC 100%								RQD = 75%
	Bedrock cored from 11.1 m depth to 14.6 m depth.		2	RC	REC 100%								RQD = 35%
	For coring details see Record of Drillhole B2-1.		3	RC	REC 100%								RQD = 90%
			4	RC	REC 100%								RQD = 100%
183.9													
14.6	End of Borehole												

MIS-MTO 001 07-1191-0020 B2 BH LOGS METRIC.GPJ GAL-MISS.GDT 04/10/10 DATA INPUT:

Continued Next Page

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

PROJECT <u>07-1191-0020</u>		RECORD OF BOREHOLE No B2- 1		2 OF 2 METRIC	
W.P. <u>5190-06-01</u>		LOCATION <u>N 5048925.4; E 238215.2</u>		ORIGINATED BY <u>ID</u>	
DIST <u> </u> HWY <u>69</u>		BOREHOLE TYPE <u>Portable Equipment, BW Casing, Wash Boring</u>		COMPILED BY <u>MM</u>	
DATUM <u>Geodetic</u>		DATE <u>January 15 to 19, 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 (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W _p	W	W _L		GR	SA	SI	CL	
								○ UNCONFINED	● QUICK TRIAXIAL	+	×	FIELD VANE	REMOULDED	WATER CONTENT (%)							
	--- CONTINUED FROM PREVIOUS PAGE ---																				
	Note: 1. Water level measured in piezometer at a depth of 0.6 m below ground surface (Elev. 197.9 m) on March 19, 2009.																				


MIS-MTO 001 07-1191-0020 B2 BH LOGS METRIC.GPJ GAL-MISS.GDT 04/10/10 DATA INPUT:

SHEET 1 OF 1


DATUM: Geodetic

DRILLING CONTRACTOR: OGS Inc.

CHECKED: AB

PROJECT <u>07-1191-0020</u>				RECORD OF BOREHOLE No B2- 2				1 OF 1 METRIC									
W.P. <u>5190-06-01</u>		LOCATION <u>N 5048954.3; E 238238.1</u>				ORIGINATED BY <u>ID</u>											
DIST <u> </u> HWY <u>69</u>		BOREHOLE TYPE <u>Portable Equipment, Hand Sampling</u>				COMPILED BY <u>MM</u>											
DATUM <u>Geodetic</u>		DATE <u>January 14, 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									
200.6	GROUND SURFACE							20	40	60	80	100					
0.0	ORGANICS Stiff Brown Moist		1	SS	11												
200.0	End of Borehole Spoon Refusal (Hammer Bouncing) Probable Bedrock																
0.6	Note: 1. Borehole dry upon completion of drilling.																

MIS-MTO 001 07-1191-0020 B2 BH LOGS METRIC.GPJ GAL-MISS.GDT 04/10/10 DATA INPUT:

PROJECT <u>07-1191-0020</u>		RECORD OF BOREHOLE No B2- 3				1 OF 1 METRIC											
W.P. <u>5190-06-01</u>		LOCATION <u>N 5048958.6; E 238240.6</u>				ORIGINATED BY <u>ID</u>											
DIST <u> </u> HWY <u>69</u>		BOREHOLE TYPE <u>Thin-wall N Coring</u>				COMPILED BY <u>MM</u>											
DATUM <u>Geodetic</u>		DATE <u>January 13 and 14, 2009</u>				CHECKED BY <u>AB</u>											
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					WATER CONTENT (%)				
202.8 0.0	GROUND SURFACE GNEISS (BEDROCK) Bedrock cored from 0.0 m depth to 3.6 m depth. For coring details see Record of Drillhole B2-3.		1	RC	REC 100%												
			2	RC	REC 100%												
			3	RC	REC 100%												
199.2 3.6	End of Borehole																

MIS-MTO 001 07-1191-0020 B2 BH LOGS METRIC.GPJ GAL-MISS.GDT 04/10/10 DATA INPUT:

PROJECT: 07-1191-0020

RECORD OF DRILLHOLE: B2- 3

SHEET 1 OF 1

LOCATION: N 5048958.6 ;E 238240.6

DRILLING DATE: January 13 and 14, 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	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.																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																				
							RECOVERY		R.Q.D. %		FRACT. INDEX PER 0.3 m		DISCONTINUITY DATA										HYDRAULIC CONDUCTIVITY K, cm/sec		Diametral Point Load Index (MPa)		RMC -Q AVG.																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																		
							TOTAL CORE %	SOLID CORE %					B Angle		DIP w.r.t. CORE AXIS		TYPE AND SURFACE DESCRIPTION						Jr	Ja	Jn	10	10	10	10	10	10	10	10																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																												
							FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH	FLUSH

UCS = 80 MPa

DEPTH SCALE

1 : 50



LOGGED: ID

CHECKED: AB

SUD-RCK 07-1191-0020 B2 BH LOGS METRIC.GPJ GAL-MISS.GDT 04/10/10 DATA INPUT:

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

PROJECT: 07-1191-0020

RECORD OF DRILLHOLE: **B2- 4**

SHEET 1 OF 1

LOCATION: N 5048959.0 ;E 238235.0


DRILLING DATE: January 13, 2009

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Portable Equipment

DRILLING CONTRACTOR: OGS Inc.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV.		RUN No.	COLOUR % RETURN	FLUSH	RECOVERY		R.Q.D. %	FRACT INDEX PER 0.3 m	DISCONTINUITY DATA					HYDRAULIC CONDUCTIVITY				Diametral Point Load Index (MPa)	RMC -Q AVG.		
				DEPTH (m)					TOTAL CORE %	SOLID CORE %			B Angle	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	K, cm/sec	10 ⁻⁶	10 ⁻⁵			10 ⁻⁴	10 ⁻³
0		GROUND SURFACE		200.9																					
0	01/13/2009 Thin-wall N Coring	GNEISS to GRANITE GNEISS Fine to medium grained Slightly weathered Strong Pinkish grey		0.0	1																				
1		All Joints noted to be undulating and rough.																							
2		2																							
3		3																							
		End of Drillhole		197.7	3.2																				
4																									
5																									
6																									
7																									
8																									
9																									
10																									

DEPTH SCALE

1 : 50



LOGGED: ID

CHECKED: AB

SUD-RCK 07-1191-0020 B2 BH LOGS METRIC.GPJ GAL-MISS.GDT 04/10/10 DATA INPUT:

PROJECT 07-1191-0020		RECORD OF BOREHOLE No B2- 5				1 OF 1 METRIC										
W.P. 5190-06-01		LOCATION N 5048959.4; E 238229.5				ORIGINATED BY ID										
DIST HWY 69		BOREHOLE TYPE BW Casing, Thin-wall N Coring				COMPILED BY MM										
DATUM Geodetic		DATE January 12, 2009				CHECKED BY AB										
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa								
199.2	GROUND SURFACE															
0.0	ORGANICS		1	SS	4/0.2											
0.2	Brown Moist GNEISS (BEDROCK)		1	RC	REC 100%											RQD = 26%
	Bedrock cored from 0.2 m depth to 4.1 m depth.		2	RC	REC 100%											RQD = 84%
	For coring details see Record of Drillhole B2-5.		3	RC	REC 100%											RQD = 100%
195.1	End of Borehole		4	RC	REC 100%											RQD = 100%
4.1	Note: 1. Borehole dry at the start of coring.															

MIS-MTO 001 07-1191-0020 B2 BH LOGS METRIC.GPJ GAL-MISS.GDT 04/10/10 DATA INPUT:

PROJECT: 07-1191-0020

RECORD OF DRILLHOLE: B2- 5

SHEET 1 OF 1

LOCATION: N 5048959.4 ;E 238229.5

DRILLING DATE: January 12, 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.	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																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													
								RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.3 m	DISCONTINUITY DATA				HYDRAULIC CONDUCTIVITY K, cm/sec				Diametral Point Load Index (MPa)	RMC -Q AVG.																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																
								TOTAL CORE %	SOLID CORE %			TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	K ₁₀	K ₅₀	K ₉₀																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																			
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DEPTH SCALE


1 : 50




LOGGED: ID

CHECKED: AB

SUD-RCK 07-1191-0020 B2 BH LOGS METRIC.GPJ GAL-MISS.GDT 07/10/10 DATA INPUT:

PROJECT <u>07-1191-0020</u>				RECORD OF BOREHOLE No B2- 6				1 OF 1 METRIC									
W.P. <u>5190-06-01</u>		LOCATION <u>N 5048963.7; E 238232.0</u>				ORIGINATED BY <u>ID</u>											
DIST <u> </u> HWY <u>69</u>		BOREHOLE TYPE <u>Portable Equipment, Hand Sampling</u>				COMPILED BY <u>MM</u>											
DATUM <u>Geodetic</u>		DATE <u>January 14, 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									
201.1	GROUND SURFACE							20	40	60	80	100					
0.0	ORGANICS		1	SS	2		201										
0.2	Brown Moist																
200.2	Silty SAND, trace organics		2	SS	1/0.15												
0.9	Very loose Brown Moist																
	End of Borehole Spoon Refusal (Hammer Bouncing)																
	Note: 1. Borehole dry upon completion of drilling.																

MIS-MTO 001 07-1191-0020 B2 BH LOGS METRIC.GPJ GAL-MISS.GDT 04/10/10 DATA INPUT:

PROJECT <u>07-1191-0020</u>				RECORD OF BOREHOLE No B2-7				1 OF 1 METRIC												
W.P. <u>5190-06-01</u>				LOCATION <u>N 5048993.4; E 238255.4</u>				ORIGINATED BY <u>EHS</u>												
DIST <u> </u> HWY <u>69</u>				BOREHOLE TYPE <u>NQ Coring</u>				COMPILED BY <u>MM</u>												
DATUM <u>Geodetic</u>				DATE <u>January 27, 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										WATER CONTENT (%)		
								20	40	60	80	100						20	40	60
208.2 0.0	GROUND SURFACE GNEISS (BEDROCK) Bedrock cored from 0.0 m depth to 8.6 m depth. For coring details see Record of Drillhole B2-7.		1	RC	REC 100%													RQD = 100%		
			2	RC	REC 100%													RQD = 90%		
			3	RC	REC 100%													RQD = 100%		
			4	RC	REC 100%													RQD = 100%		
			5	RC	REC 100%													RQD = 100%		
			6	RC	REC 100%													RQD = 100%		
199.6 8.6	End of Borehole																			

MIS-MTO001 07-1191-0020 B2 BH LOGS METRIC.GPJ GAL-MISS.GDT 04/10/10 DATA INPUT:

PROJECT: 07-1191-0020

RECORD OF DRILLHOLE: B2-7

SHEET 1 OF 1

LOCATION: N 5048993.4 ;E 238255.4

DRILLING DATE: January 27, 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	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.																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																							
							RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.3 m	DISCONTINUITY DATA										HYDRAULIC CONDUCTIVITY K, cm/sec										Diametral Point Load Index (MPa)										RMC -Q AVG.																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																							
							TOTAL CORE %	SOLID CORE %			B Angle °	DIP w.r.t. CORE AXIS °	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	K ₁	K ₂	K ₃	K ₄	K ₅	K ₆	K ₇	K ₈	K ₉	K ₁₀	K ₁₁	K ₁₂	K ₁₃	K ₁₄	K ₁₅	K ₁₆	K ₁₇	K ₁₈	K ₁₉	K ₂₀	K ₂₁	K ₂₂	K ₂₃	K ₂₄	K ₂₅	K ₂₆	K ₂₇	K ₂₈	K ₂₉	K ₃₀	K ₃₁	K ₃₂	K ₃₃	K ₃₄	K ₃₅	K ₃₆	K ₃₇	K ₃₈	K ₃₉	K ₄₀	K ₄₁	K ₄₂	K ₄₃	K ₄₄	K ₄₅	K ₄₆	K ₄₇	K ₄₈	K ₄₉	K ₅₀	K ₅₁	K ₅₂	K ₅₃	K ₅₄	K ₅₅	K ₅₆	K ₅₇	K ₅₈	K ₅₉	K ₆₀	K ₆₁	K ₆₂	K ₆₃	K ₆₄	K ₆₅	K ₆₆	K ₆₇	K ₆₈	K ₆₉	K ₇₀	K ₇₁	K ₇₂	K ₇₃	K ₇₄	K ₇₅	K ₇₆	K ₇₇	K ₇₈	K ₇₉	K ₈₀	K ₈₁	K ₈₂	K ₈₃	K ₈₄	K ₈₅	K ₈₆	K ₈₇	K ₈₈	K ₈₉	K ₉₀	K ₉₁	K ₉₂	K ₉₃	K ₉₄	K ₉₅	K ₉₆	K ₉₇	K ₉₈	K ₉₉	K ₁₀₀	K ₁₀₁	K ₁₀₂	K ₁₀₃	K ₁₀₄	K ₁₀₅	K ₁₀₆	K ₁₀₇	K ₁₀₈	K ₁₀₉	K ₁₁₀	K ₁₁₁	K ₁₁₂	K ₁₁₃	K ₁₁₄	K ₁₁₅	K ₁₁₆	K ₁₁₇	K ₁₁₈	K ₁₁₉	K ₁₂₀	K ₁₂₁	K ₁₂₂	K ₁₂₃	K ₁₂₄	K ₁₂₅	K ₁₂₆	K ₁₂₇	K ₁₂₈	K ₁₂₉	K ₁₃₀	K ₁₃₁	K ₁₃₂	K ₁₃₃	K ₁₃₄	K ₁₃₅	K ₁₃₆	K ₁₃₇	K ₁₃₈	K ₁₃₉	K ₁₄₀	K ₁₄₁	K ₁₄₂	K ₁₄₃	K ₁₄₄	K ₁₄₅	K ₁₄₆	K ₁₄₇	K ₁₄₈	K ₁₄₉	K ₁₅₀	K ₁₅₁	K ₁₅₂	K ₁₅₃	K ₁₅₄	K ₁₅₅	K ₁₅₆	K ₁₅₇	K ₁₅₈	K ₁₅₉	K ₁₆₀	K ₁₆₁	K ₁₆₂	K ₁₆₃	K ₁₆₄	K ₁₆₅	K ₁₆₆	K ₁₆₇	K ₁₆₈	K ₁₆₉	K ₁₇₀	K ₁₇₁	K ₁₇₂	K ₁₇₃	K ₁₇₄	K ₁₇₅	K ₁₇₆	K ₁₇₇	K ₁₇₈	K ₁₇₉	K ₁₈₀	K ₁₈₁	K ₁₈₂	K ₁₈₃	K ₁₈₄	K ₁₈₅	K ₁₈₆	K ₁₈₇	K ₁₈₈	K ₁₈₉	K ₁₉₀	K ₁₉₁	K ₁₉₂	K ₁₉₃	K ₁₉₄	K ₁₉₅	K ₁₉₆	K ₁₉₇	K ₁₉₈	K ₁₉₉	K ₂₀₀	K ₂₀₁	K ₂₀₂	K ₂₀₃	K ₂₀₄	K ₂₀₅	K ₂₀₆	K ₂₀₇	K ₂₀₈	K ₂₀₉	K ₂₁₀	K ₂₁₁	K ₂₁₂	K ₂₁₃	K ₂₁₄	K ₂₁₅	K ₂₁₆	K ₂₁₇	K ₂₁₈	K ₂₁₉	K ₂₂₀	K ₂₂₁	K ₂₂₂	K ₂₂₃	K ₂₂₄	K ₂₂₅	K ₂₂₆	K ₂₂₇	K ₂₂₈	K ₂₂₉	K ₂₃₀	K ₂₃₁	K ₂₃₂	K ₂₃₃	K ₂₃₄	K ₂₃₅	K ₂₃₆	K ₂₃₇	K ₂₃₈	K ₂₃₉	K ₂₄₀	K ₂₄₁	K ₂₄₂	K ₂₄₃	K ₂₄₄	K ₂₄₅	K ₂₄₆	K ₂₄₇	K ₂₄₈	K ₂₄₉	K ₂₅₀	K ₂₅₁	K ₂₅₂	K ₂₅₃	K ₂₅₄	K ₂₅₅	K ₂₅₆	K ₂₅₇	K ₂₅₈	K ₂₅₉	K ₂₆₀	K ₂₆₁	K ₂₆₂	K ₂₆₃	K ₂₆₄	K ₂₆₅	K ₂₆₆	K ₂₆₇	K ₂₆₈	K ₂₆₉	K ₂₇₀	K ₂₇₁	K ₂₇₂	K ₂₇₃	K ₂₇₄	K ₂₇₅	K ₂₇₆	K ₂₇₇	K ₂₇₈	K ₂₇₉	K ₂₈₀	K ₂₈₁	K ₂₈₂	K ₂₈₃	K ₂₈₄	K ₂₈₅	K ₂₈₆	K ₂₈₇	K ₂₈₈	K ₂₈₉	K ₂₉₀	K ₂₉₁	K ₂₉₂	K ₂₉₃	K ₂₉₄	K ₂₉₅	K ₂₉₆	K ₂₉₇	K ₂₉₈	K ₂₉₉	K ₃₀₀	K ₃₀₁	K ₃₀₂	K ₃₀₃	K ₃₀₄	K ₃₀₅	K ₃₀₆	K ₃₀₇	K ₃₀₈	K ₃₀₉	K ₃₁₀	K ₃₁₁	K ₃₁₂	K ₃₁₃	K ₃₁₄	K ₃₁₅	K ₃₁₆	K ₃₁₇	K ₃₁₈	K ₃₁₉	K ₃₂₀	K ₃₂₁	K ₃₂₂	K ₃₂₃	K ₃₂₄	K ₃₂₅	K ₃₂₆	K ₃₂₇	K ₃₂₈	K ₃₂₉	K ₃₃₀	K ₃₃₁	K ₃₃₂	K ₃₃₃	K ₃₃₄	K ₃₃₅	K ₃₃₆	K ₃₃₇	K ₃₃₈	K ₃₃₉	K ₃₄₀	K ₃₄₁	K ₃₄₂	K ₃₄₃	K ₃₄₄	K ₃₄₅	K ₃₄₆	K ₃₄₇	K ₃₄₈	K ₃₄₉	K ₃₅₀	K ₃₅₁	K ₃₅₂	K ₃₅₃	K ₃₅₄	K ₃₅₅	K ₃₅₆	K ₃₅₇	K ₃₅₈	K ₃₅₉	K ₃₆₀	K ₃₆₁	K ₃₆₂	K ₃₆₃	K ₃₆₄	K ₃₆₅	K ₃₆₆	K ₃₆₇	K ₃₆₈	K ₃₆₉	K ₃₇₀	K ₃₇₁	K ₃₇₂	K ₃₇₃	K ₃₇₄	K ₃₇₅	K ₃₇₆	K ₃₇₇	K ₃₇₈	K ₃₇₉	K ₃₈₀	K ₃₈₁	K ₃₈₂	K ₃₈₃	K ₃₈₄	K ₃₈₅	K ₃₈₆	K ₃₈₇	K ₃₈₈	K ₃₈₉	K ₃₉₀	K ₃₉₁	K ₃₉₂	K ₃₉₃	K ₃₉₄	K ₃₉₅	K ₃₉₆	K ₃₉₇	K ₃₉₈	K ₃₉₉	K ₄₀₀	K ₄₀₁	K ₄₀₂	K ₄₀₃	K ₄₀₄	K ₄₀₅	K ₄₀₆	K ₄₀₇	K ₄₀₈	K ₄₀₉	K ₄₁₀	K ₄₁₁	K ₄₁₂	K ₄₁₃	K ₄₁₄	K ₄₁₅	K ₄₁₆	K ₄₁₇	K ₄₁₈	K ₄₁₉	K ₄₂₀	K ₄₂₁	K ₄₂₂	K ₄₂₃	K ₄₂₄	K ₄₂₅	K ₄₂₆	K ₄₂₇	K ₄₂₈	K ₄₂₉	K ₄₃₀	K ₄₃₁	K ₄₃₂	K ₄₃₃	K ₄₃₄	K ₄₃₅	K ₄₃₆	K ₄₃₇	K ₄₃₈	K ₄₃₉	K ₄₄₀	K ₄₄₁	K ₄₄₂	K ₄₄₃	K ₄₄₄	K ₄₄₅	K ₄₄₆	K ₄₄₇	K ₄₄₈	K ₄₄₉	K ₄₅₀	K ₄₅₁	K ₄₅₂	K ₄₅₃	K ₄₅₄	K ₄₅₅	K ₄₅₆	K ₄₅₇	K ₄₅₈	K ₄₅₉	K ₄₆₀	K ₄₆₁	K ₄₆₂	K ₄₆₃	K ₄₆₄	K ₄₆₅	K ₄₆₆	K ₄₆₇	K ₄₆₈	K ₄₆₉	K ₄₇₀	K ₄₇₁	K ₄₇₂	K ₄₇₃	K ₄₇₄	K ₄₇₅	K ₄₇₆	K ₄₇₇	K ₄₇₈	K ₄₇₉	K ₄₈₀	K ₄₈₁	K ₄₈₂	K ₄₈₃	K ₄₈₄	K ₄₈₅	K ₄₈₆	K ₄₈₇	K ₄₈₈	K ₄₈₉	K ₄₉₀	K ₄₉₁	K ₄₉₂	K ₄₉₃	K ₄₉₄	K ₄₉₅	K ₄₉₆	K ₄₉₇	K ₄₉₈	K ₄₉₉	K ₅₀₀	K ₅₀₁	K ₅₀₂	K ₅₀₃	K ₅₀₄	K ₅₀₅	K ₅₀₆	K ₅₀₇	K ₅₀₈	K ₅₀₉	K ₅₁₀	K ₅₁₁	K ₅₁₂	K ₅₁₃	K ₅₁₄	K ₅₁₅	K ₅₁₆	K ₅₁₇	K ₅₁₈	K ₅₁₉	K ₅₂₀	K ₅₂₁	K ₅₂₂	K ₅₂₃	K ₅₂₄	K ₅₂₅	K ₅₂₆	K ₅₂₇	K ₅₂₈	K ₅₂₉	K ₅₃₀	K ₅₃₁	K ₅₃₂	K ₅₃₃	K ₅₃₄	K ₅₃₅	K ₅₃₆	K ₅₃₇	K ₅₃₈	K ₅₃₉	K ₅₄₀	K ₅₄₁	K ₅₄₂	K ₅₄₃	K ₅₄₄	K ₅₄₅	K ₅₄₆	K ₅₄₇	K ₅₄₈	K ₅₄₉	K ₅₅₀	K ₅₅₁	K ₅₅₂	K ₅₅₃	K ₅₅₄	K ₅₅₅	K ₅₅₆	K ₅₅₇	K ₅₅₈	K ₅₅₉	K ₅₆₀	K ₅₆₁	K ₅₆₂	K ₅₆₃	K ₅₆₄	K ₅₆₅	K ₅₆₆	K ₅₆₇	K ₅₆₈	K ₅₆₉	K ₅₇₀	K ₅₇₁	K ₅₇₂	K ₅₇₃	K ₅₇₄	K ₅₇₅	K ₅₇₆	K ₅₇₇	K ₅₇₈	K ₅₇₉	K ₅₈₀	K ₅₈₁	K ₅₈₂	K ₅₈₃	K ₅₈₄	K ₅₈₅	K ₅₈₆	K ₅₈₇	K ₅₈₈	K ₅₈₉	K ₅₉₀	K ₅₉₁	K ₅₉₂	K ₅₉₃	K ₅₉₄	K ₅₉₅	K ₅₉₆	K ₅₉₇	K ₅₉₈	K ₅₉₉	K ₆₀₀	K ₆₀₁	K ₆₀₂	K ₆₀₃	K ₆₀₄	K ₆₀₅	K ₆₀₆	K ₆₀₇	K ₆₀₈	K ₆₀₉	K ₆₁₀	K ₆₁₁	K ₆₁₂	K ₆₁₃	K ₆₁₄	K ₆₁₅	K ₆₁₆	K ₆₁₇	K ₆₁₈	K ₆₁₉	K ₆₂₀	K ₆₂₁	K ₆₂₂	K ₆₂₃	K ₆₂₄	K ₆₂₅	K ₆₂₆	K ₆₂₇	K ₆₂₈	K ₆₂₉	K ₆₃₀	K ₆₃₁	K ₆₃₂	K ₆₃₃	K ₆₃₄	K ₆₃₅	K ₆₃₆	K ₆₃₇	K ₆₃₈	K ₆₃₉	K ₆₄₀	K ₆₄₁	K ₆₄₂	K ₆₄₃	K ₆₄₄	K ₆₄₅	K ₆₄₆	K ₆₄₇	K ₆₄₈	K ₆₄₉	K ₆₅₀	K ₆₅₁	K ₆₅₂	K ₆₅₃	K ₆₅₄	K ₆₅₅	K ₆₅₆	K ₆₅₇	K ₆₅₈	K ₆₅₉	K ₆₆₀	K ₆₆₁	K ₆₆₂	K ₆₆₃	K ₆₆₄	K ₆₆₅	K ₆₆₆	K ₆₆₇	K ₆₆₈	K ₆₆₉	K ₆₇₀	K ₆₇₁	K ₆₇₂	K ₆₇₃	K ₆₇₄	K ₆₇₅	K ₆₇₆	K ₆₇₇	K ₆₇₈	K ₆₇₉	K ₆₈₀	K ₆₈₁	K ₆₈₂	K ₆₈₃	K ₆₈₄	K ₆₈₅	K ₆₈₆	K ₆₈₇	K ₆₈₈	K ₆₈₉	K ₆₉₀	K ₆₉₁	K ₆₉₂	K ₆₉₃	K ₆₉₄	K ₆₉₅	K ₆₉₆	K ₆₉₇	K ₆₉₈	K ₆₉₉	K ₇₀₀	K ₇₀₁	K ₇₀₂	K ₇₀₃	K ₇₀₄

DEPTH SCALE

1 : 50



LOGGED: EHS

CHECKED: AB

SUD-RCK 07-1191-0020 B2 BH LOGS METRIC.GPJ GAL-MISS.GDT 04/10/10 DATA INPUT:

PROJECT		RECORD OF BOREHOLE No B2- 8				1 OF 1 METRIC										
W.P. 5190-06-01		LOCATION N 5048988.7; E 238258.4				ORIGINATED BY EHS										
DIST _____ HWY 69		BOREHOLE TYPE NW Casing, NQ Coring				COMPILED BY MM										
DATUM Geodetic		DATE January 28, 2009				CHECKED BY AB										
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					WATER CONTENT (%)			
208.0	GROUND SURFACE						20	40	60	80	100	W _p	W	W _L		
0.1	ORGANICS Brown GNEISS (BEDROCK)															
	Bedrock cored from 0.1 m depth to 2.9 m depth.		1	RC	REC 100%											RQD = 63%
	For coring details see Record of Drillhole B2-8.		2	RC	REC 100%											RQD = 88%
205.1	End of Borehole															
2.9	Note: 1. Borehole dry at the start of coring.															

MIS-MTO 001 07-1191-0020 B2 BH LOGS METRIC.GPJ GAL-MISS.GDT 04/10/10 DATA INPUT:

SHEET 1 OF 1

DATUM: Geodetic

DRILLING CONTRACTOR: Landcore Drilling

CHECKED: AB



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

PROJECT <u>07-1191-0020</u>		RECORD OF BOREHOLE No B2-10				1 OF 1 METRIC										
W.P. <u>5190-06-01</u>		LOCATION <u>N 5048993.8; E 238249.8</u>				ORIGINATED BY <u>EHS</u>										
DIST <u> </u> HWY <u>69</u>		BOREHOLE TYPE <u>Hand Auger</u>				COMPILED BY <u>MM</u>										
DATUM <u>Geodetic</u>		DATE <u>January 28, 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								
208.0	GROUND SURFACE															
0.0	ORGANICS		1	AS	-											
0.1	Brown															
	End of Borehole Auger Refusal															
	Note:															
	1. Borehole dry upon completion of drilling.															

MIS-MTO 001 07-1191-0020 B2 BH LOGS METRIC.GPJ GAL-MISS.GDT 04/10/10 DATA INPUT:

PROJECT 07-1191-0020				RECORD OF BOREHOLE No B2-11				1 OF 1 METRIC									
W.P. 5190-06-01				LOCATION N 5048998.1; E 238252.4				ORIGINATED BY EHS									
DIST _____ HWY 69				BOREHOLE TYPE NW Casing, NQ Coring				COMPILED BY MM									
DATUM Geodetic				DATE January 27, 2009				CHECKED BY AB									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
207.5	GROUND SURFACE							20	40	60	80	100	W _p	W	W _L		
0.0	ORGANICS Brown																
0.1	GNEISS (BEDROCK)																
	Bedrock cored from 0.1 m depth to 3.3 m depth.		1	RC	REC 100%		207										RQD = 100%
	For coring details see Record of Drillhole B2-11.						206										
			2	RC	REC 100%		205										RQD = 97%
204.2	End of Borehole																
3.3	Note: 1. Borehole dry at the start of coring.																

MIS-MTO 001 07-1191-0020 B2 BH LOGS METRIC.GPJ GAL-MISS.GDT 04/10/10 DATA INPUT:

PROJECT: 07-1191-0020

RECORD OF DRILLHOLE: B2-11

SHEET 1 OF 1

LOCATION: N 5048998.1 ; E 238252.4

DRILLING DATE: January 27, 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	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										FRACT. INDEX PER 0.3 m										DISCONTINUITY DATA										HYDRAULIC CONDUCTIVITY K, cm/sec										Diametral Point Load Index (MPa)										RMC -Q AVG.																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																												
							TOTAL CORE %					SOLID CORE %					R.Q.D. %					B Angle					DIP w.r.t. CORE AXIS					TYPE AND SURFACE DESCRIPTION					Jr					Ja					Jn					10 10 10 10 10					2 2 2 2 2					10 10 10 10 10					2 2 2 2 2					10 10 10 10 10					2 2 2 2 2																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																								
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UCS = 87 MPa

DEPTH SCALE

1 : 50



LOGGED: EHS

CHECKED: AB

SUD-RCK 07-1191-0020 B2 BH LOGS METRIC.GPJ GAL-MISS.GDT 04/10/10 DATA INPUT:

PROJECT <u>07-1191-0020</u>			RECORD OF BOREHOLE No B2-12			1 OF 1 METRIC		
W.P. <u>5190-06-01</u>			LOCATION <u>N 5049006.4; E 238263.0</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>MM</u>		
DATUM <u>Geodetic</u>			DATE <u>January 28, 2009</u>			CHECKED BY <u>AB</u>		
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
206.6	GROUND SURFACE							PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W _p W W _L WATER CONTENT (%)
0.0	Silty ORGANICS							
0.2	Reddish brown Moist		1	AS	-		206	
205.6	SILT, trace to some clay, trace sand							
1.0	Brown to grey Moist to wet							
	End of Borehole Auger Refusal							
	Note: 1. Borehole dry upon completion of drilling.							

MIS-MTO 001 07-1191-0020 B2 BH LOGS METRIC.GPJ GAL-MISS.GDT 04/10/10 DATA INPUT:



APPENDIX B



Record of Borehole, AMEC (2006)

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

2 OF 2

G.W.P. 5377-02-00	LOCATION Highway 529 Interchange, Township of the Archipelago.	2 OF 2	ORIGINATED BY MAH
DIST 54 HWY 69	Co-ords: 5048925 N; 238223 E BOREHOLE TYPE Solid Stem Augering		COMPILED BY SN
DATUM Geodetic	DATE 31 January 2006 - 1 February 2006		CHECKED BY IH
PROJECT Highway 69 Route Selection Study, from 3.5 km North of HWY 559 to 3.8 km North of HWY 522			JOB NO. TT53126

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DEPTH m	ELEVATION SCALE m	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES				SHEAR STRENGTH kPa			WATER CONTENT (%)				
									○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× LAB VANE	W _p	W		

	SILTY CLAY grey, firm, wet														
			6	SS	5										
190.5															
10.0	SAND with silt, some gravel grey, fine to medium grained, compact to dense, moist														
			7	SS	18										
187.9			8	SS	45										17 62 (21)
12.7	End of Borehole														
12.7	Auger refusal at 12.7 m due to possible bedrock														
12.8	Dynamic cone penetration test was conducted below 12.7 m.														
	No noticeable groundwater in open borehole on completion														
	End of DCPT														
	Refusal to Dynamic Cone Penetration Test at 12.8 m depth due to possible bedrock														
	Borehole ST-6 was moved to 8 m east from the specified location (5048925 N; 238215 E) which is on a ditch adjacent to Highway 69.														
	Borehole was backfilled with bentonite.														

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



APPENDIX C

Laboratory Test Results

TABLE C-1
REFUSAL/BEDROCK ELEVATIONS
HWY 529 IC UNDERPASS, SITE NO. 44-445
GWP 5403-05-00, WP 5190-06-01
HIGHWAY 69, POINTE AU BARIL

Borehole	Depth to Refusal/Bedrock Surface (m)	Refusal/Bedrock Surface Elevation (m)	Comments
S5-1	3.8	194.8	Split-Spoon Refusal
S5-2	7.0	191.5	Split-Spoon Refusal
S5-3	0.9	197.8	Split-Spoon Refusal
S5-4	4.3	194.4	Split-Spoon Refusal
S5-5	8.8	190.0	Casing Refusal
B2-1	11.1	187.4	Bedrock Surface
B2-2	0.6	200.0	Split-Spoon Refusal
B2-3	G.S.*	202.8	Bedrock Surface
B2-4	G.S.*	200.9	Bedrock Surface
B2-5	0.2	199.0	Bedrock Surface
B2-6	0.9	200.2	Split-Spoon Refusal
B2-7	G.S.*	208.2	Bedrock Surface
B2-8	0.1	207.9	Bedrock Surface
B2-9	0.2	207.5	Auger Refusal
B2-10	0.1	207.9	Auger Refusal
B2-11	0.1	207.4	Bedrock Surface
B2-12	1.0	205.6	Auger Refusal

*G.S. denotes bedrock was encountered at ground surface.

Compiled by: TRR
 Checked by: SEMC
 Reviewed by: JMAC

TABLE C-2
UNIAXIAL COMPRESSIVE STRENGTH TEST RESULTS
HWY 529 IC UNDERPASS, SITE NO. 44-445
GWP 5403-05-00, WP 5190-06-01
HIGHWAY 69, POINTE AU BARIL

Borehole Number	Sample Depth (m)	Sample Elevation (m)	Rock Type	Core Diameter (mm)	Uniaxial Compressive Strength (MPa)
B2-1	12.8	185.7	Gneiss	51	112
B2-3	2.1	200.7	Gneiss	51	80
B2-4	2.4	198.5	Gneiss to Granite Gneiss	51	98
B2-5	2.1	197.1	Gneiss	50	83
B2-7	7.7	200.5	Gneiss	48	95
B2-8	2.3	205.7	Gneiss	48	92
B2-11	2.2	205.3	Gneiss	48	87

Compiled by: TRR
Checked by: SEMC
Reviewed by: JMAC

TABLE C-3
POINT LOAD STRENGTH TEST RESULTS
HWY 529 IC UNDERPASS, SITE NO. 44-445
GWP 5403-05-00, WP 5190-06-01
HIGHWAY 69, POINTE AU BARIL

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)
B2-1	11.3	187.2	Gneiss	D	50	16.7	0.016	6.3	6.3	127
B2-1	13.1	185.4	Gneiss	D	50	13.9	0.013	5.3	5.3	105
B2-1	14.6	183.9	Gneiss	D	50	16.3	0.015	6.2	6.2	123
B2-3	0.8	201.4	Gneiss	D	50	8.9	0.008	3.4	3.4	67
B2-3	3.4	198.8	Gneiss	D	50	11.5	0.011	4.4	4.4	87
B2-4	0.3	200.6	Gneiss	D	50	9.7	0.009	3.7	3.7	74
B2-4	1.6	199.3	Gneiss	D	50	15.5	0.015	5.9	5.9	118
B2-5	1.9	197.3	Gneiss	D	48	11.2	0.011	4.2	4.2	85
B2-5	3.0	196.2	Gneiss	D	48	11.3	0.011	4.3	4.3	86
B2-5	4.0	195.2	Gneiss	D	48	10.9	0.010	4.1	4.1	83
B2-7	5.6	202.6	Gneiss	D	48	8.7	0.008	3.6	3.5	70
B2-7	7.4	200.8	Gneiss	D	48	11.6	0.011	4.8	4.7	94
B2-7	8.1	200.1	Gneiss	D	48	9.5	0.009	3.9	3.8	77
B2-8	0.9	207.1	Gneiss	D	48	6.7	0.006	2.7	2.7	53
B2-8	1.8	206.2	Gneiss	D	48	9.3	0.009	3.8	3.7	75
B2-8	2.7	205.3	Gneiss	D	48	10.8	0.010	4.5	4.4	88
B2-11	0.9	206.6	Gneiss	D	48	3.5	0.003	1.4	1.4	28
B2-11	1.3	206.2	Gneiss	D	48	8.2	0.008	3.4	3.3	66
B2-11	2.6	204.9	Gneiss	D	48	8.1	0.008	3.3	3.2	66

NOTES: 1. Depths are given below the ground surface at the borehole location.

2. Where: D = Diametral;
 I_s Diametral = Uncorrected point load strength;
 I_s 50 mm = Corrected point load strength; and
 UCS = Uniaxial compressive strength = I_s 50 mm X K. A value of 20, based on correlation with UCS for this site as per ASTM D5731-

Compiled by: TRR

Checked by: SEMC

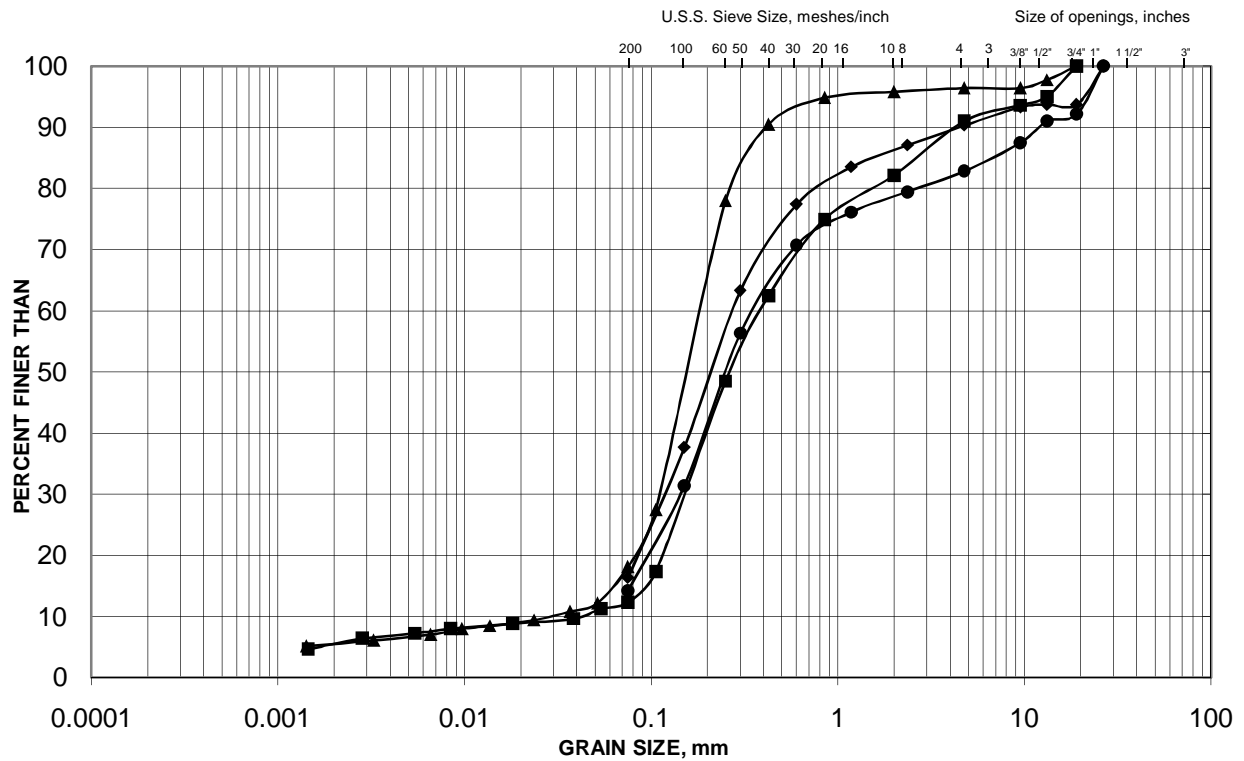
08 (cross-referenced to "Suggested Methods for Determining Point Load Strength", International Society for Rock Mechanics Commission on Testing Methods, Int. J. Rock Mech. Sci. and Geomechanical Abst., Vol 22, No. 2, 1985, pp. 51-60).

GRAIN SIZE DISTRIBUTION

Sand

FIGURE

C-1



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
◆	S5-2	7	191.6
●	S5-4	4	194.7
■	S5-5	6	192.1
▲	B2-1	8	192.1

Project Number: 07-1191-0020-B2

Checked By: SEMC

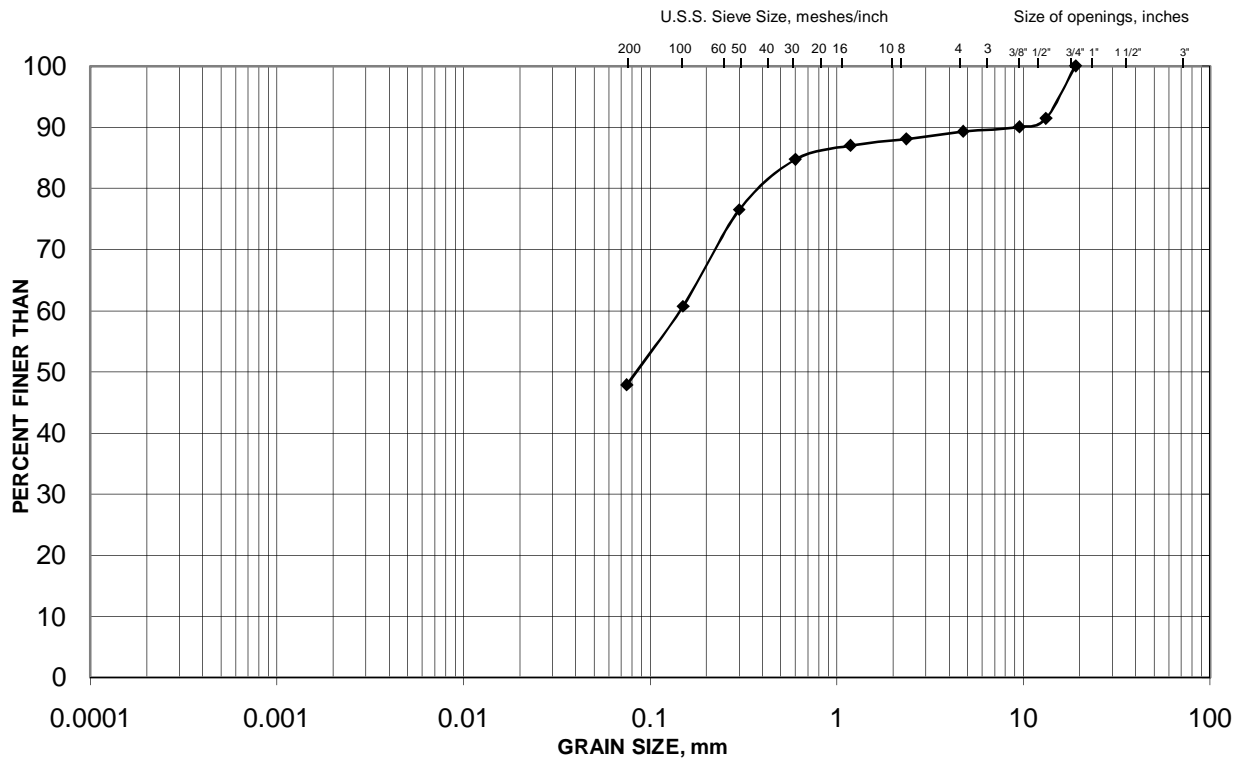
Golder Associates

Date: October 2010

GRAIN SIZE DISTRIBUTION

Sand and Silt

FIGURE
C-2



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

LEGEND

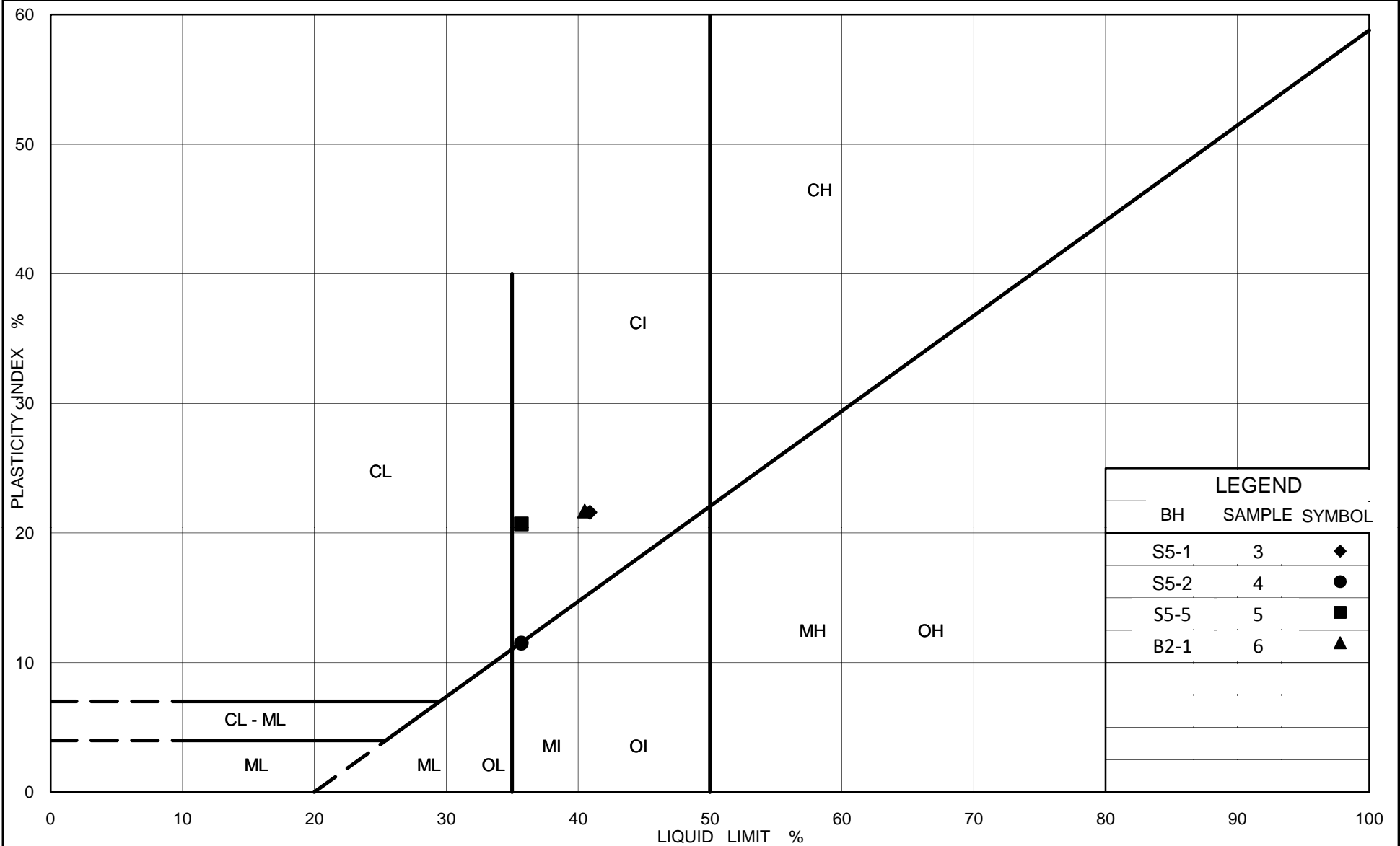
SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
—◆—	B2-1	10	189.1

Project Number: 07-1191-0020-B2

Checked By: SEMC

Golder Associates

Date: October 2010



Ministry of Transportation

Ontario

PLASTICITY CHART Silty Clay

Figure C-3

Project No. 07-1191-0020-B2

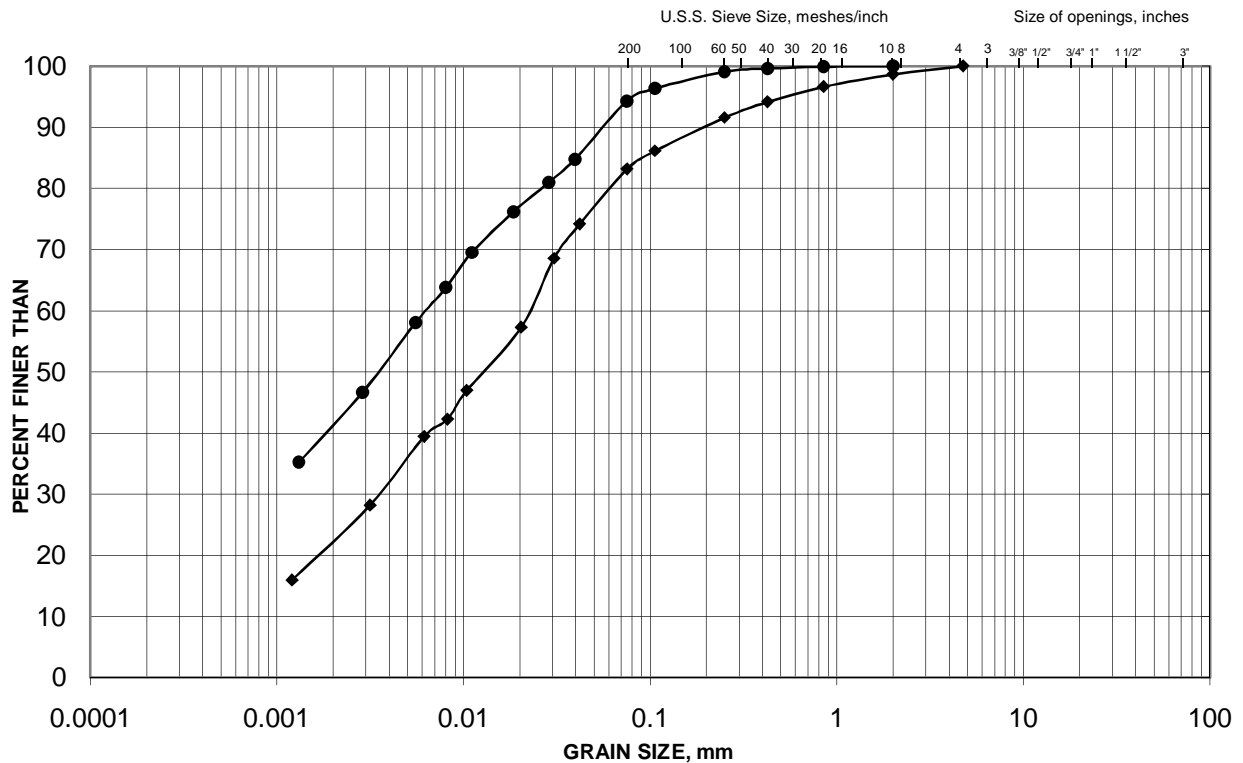
Checked By: SEMC

GRAIN SIZE DISTRIBUTION

Silty Clay

FIGURE

C-4



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
◆	S5-2	4	195.3
●	B2-1	6	194.4

Project Number: 07-1191-0020-B2

Checked By: SEMC

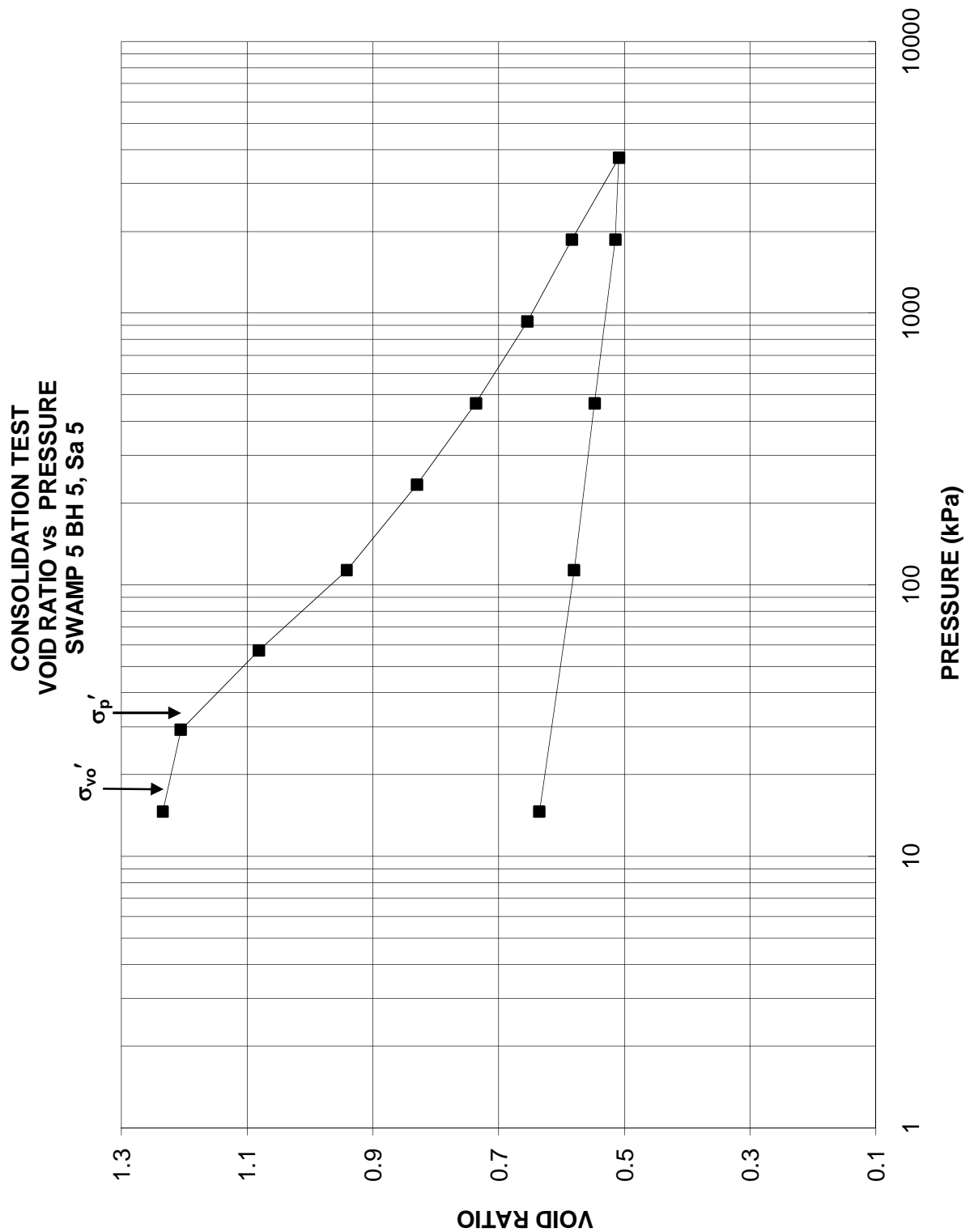
Golder Associates

Date: October 2010

OEDOMETER CONSOLIDATION SUMMARY						FIGURE C-5 Page 1 of 4		
SAMPLE IDENTIFICATION								
Project Number		07-1191-0020-B2		Borehole, Sample		S5-5, Sa 5		
Swamp:		5		Sample Depth, (m)		4.4		
TEST CONDITIONS								
Test Type		Standard		Load Duration, hr		24		
Oedometer Number		1						
Date Started		April 24/08						
Date Completed		May 5/08						
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL								
Sample Height, cm		1.906		Unit Weight, kN/m ³		17.8		
Sample Diameter, cm		5.0		Dry Unit Weight, kN/m ³		11.8		
Area, cm ²		19.63		Specific Gravity, assumed		2.7		
Volume, cm ³		37.41		Solids Height, cm		0.852		
Water Content, %		50.2		Volume of Solids, cm ³		16.73		
Wet Mass, g		67.83		Volume of Voids, cm ³		20.68		
Dry Mass, g		45.16		Degree of Saturation, %		109.6		
TEST COMPUTATIONS								
Pressure	Primary Consolidation	Corr. Height	Void Ratio	Average Height	t ₅₀	cv.	m _v	k
kPa	mm	cm	Ratio	cm	s	cm ² /s	m ² /MN	cm/s
0	0.00	1.906	1.236	1.906				
14.6	0.02	1.904	1.234	1.905	2700	0.00026	0.072	1.856E-09
29.2	0.24	1.880	1.205	1.892	400	0.00175	0.877	1.509E-07
57.4	1.06	1.774	1.081	1.827	710	0.00092	2.004	1.810E-07
113.1	1.19	1.655	0.941	1.714	400	0.00144	1.203	1.700E-07
233.3	0.95	1.560	0.830	1.607	200	0.00253	0.478	1.186E-07
466.0	0.80	1.480	0.736	1.520	90	0.00503	0.220	1.087E-07
933.2	0.70	1.410	0.654	1.445	90	0.00454	0.101	4.515E-08
1864.2	0.60	1.350	0.584	1.380	70	0.00533	0.046	2.390E-08
3728.4	0.64	1.286	0.508	1.318	26	0.01309	0.025	3.266E-08
1864.2	-0.05	1.291	0.514	1.288				
466.0	-0.28	1.319	0.547	1.305				
113.1	-0.28	1.347	0.580	1.333				
14.6	-0.47	1.394	0.635	1.370				
Notes: k calculated using cv based on t ₅₀ values.								
SAMPLE DIMENSIONS AND PROPERTIES - FINAL								
Sample Height, cm		1.394		Unit Weight, kN/m ³		19.6		
Sample Diameter, cm		5.000		Dry Unit Weight, kN/m ³		16.2		
Area, cm ²		19.63		Specific Gravity, assumed		2.7		
Volume, cm ³		27.36		Solids Height, cm		0.852		
Water Content, %		21.2		Volume of Solids, cm ³		16.73		
Wet Mass, g		54.73		Volume of Voids, cm ³		10.64		
Dry Mass, g		45.16		Degree of Saturation, %		90.1		
Prepared By: TG			Golder Associates			Checked By: AB		

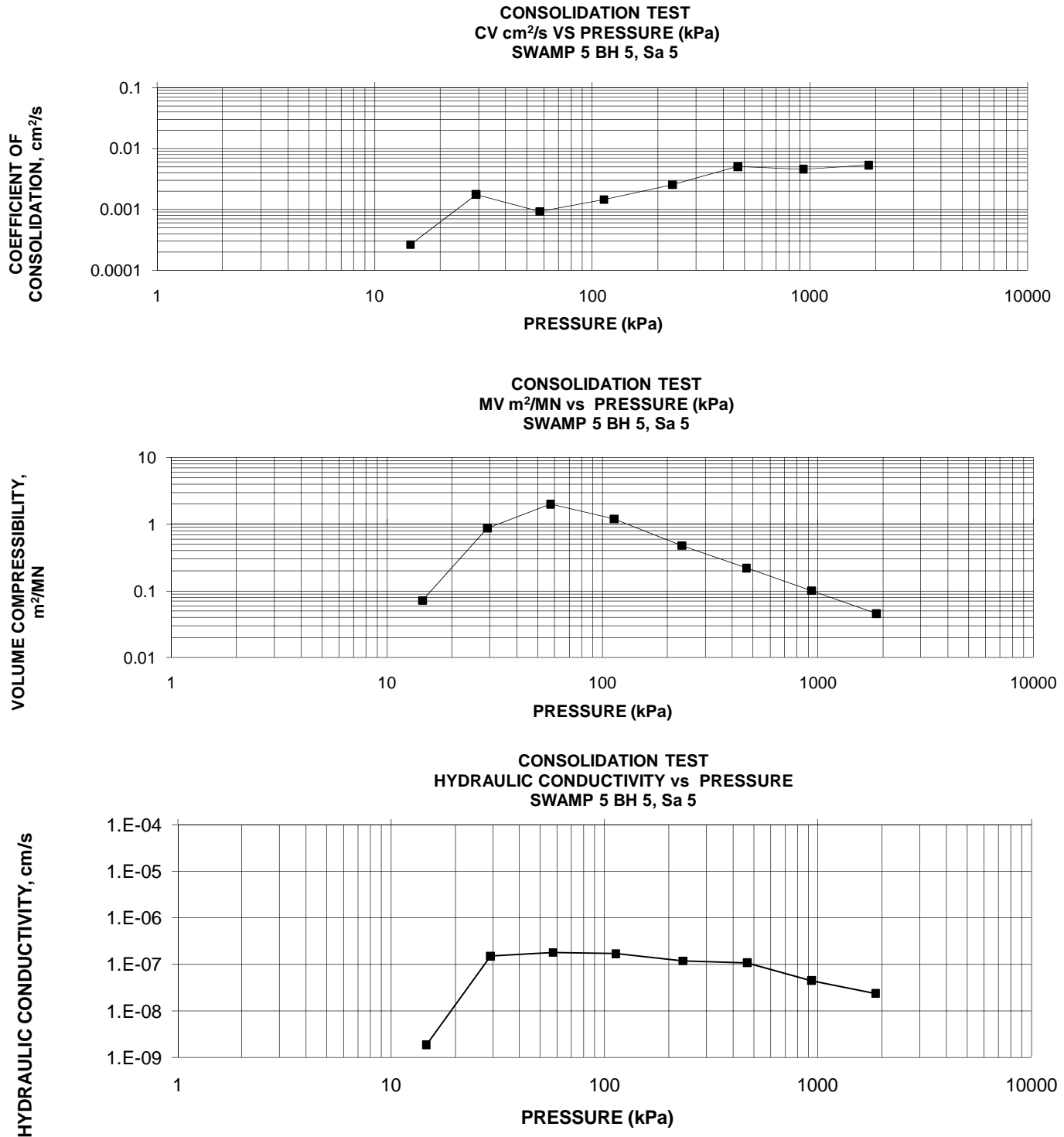
PRIMARY CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

FIGURE C-5
Page 2 of 4



OEDOMETER CONSOLIDATION SUMMARY

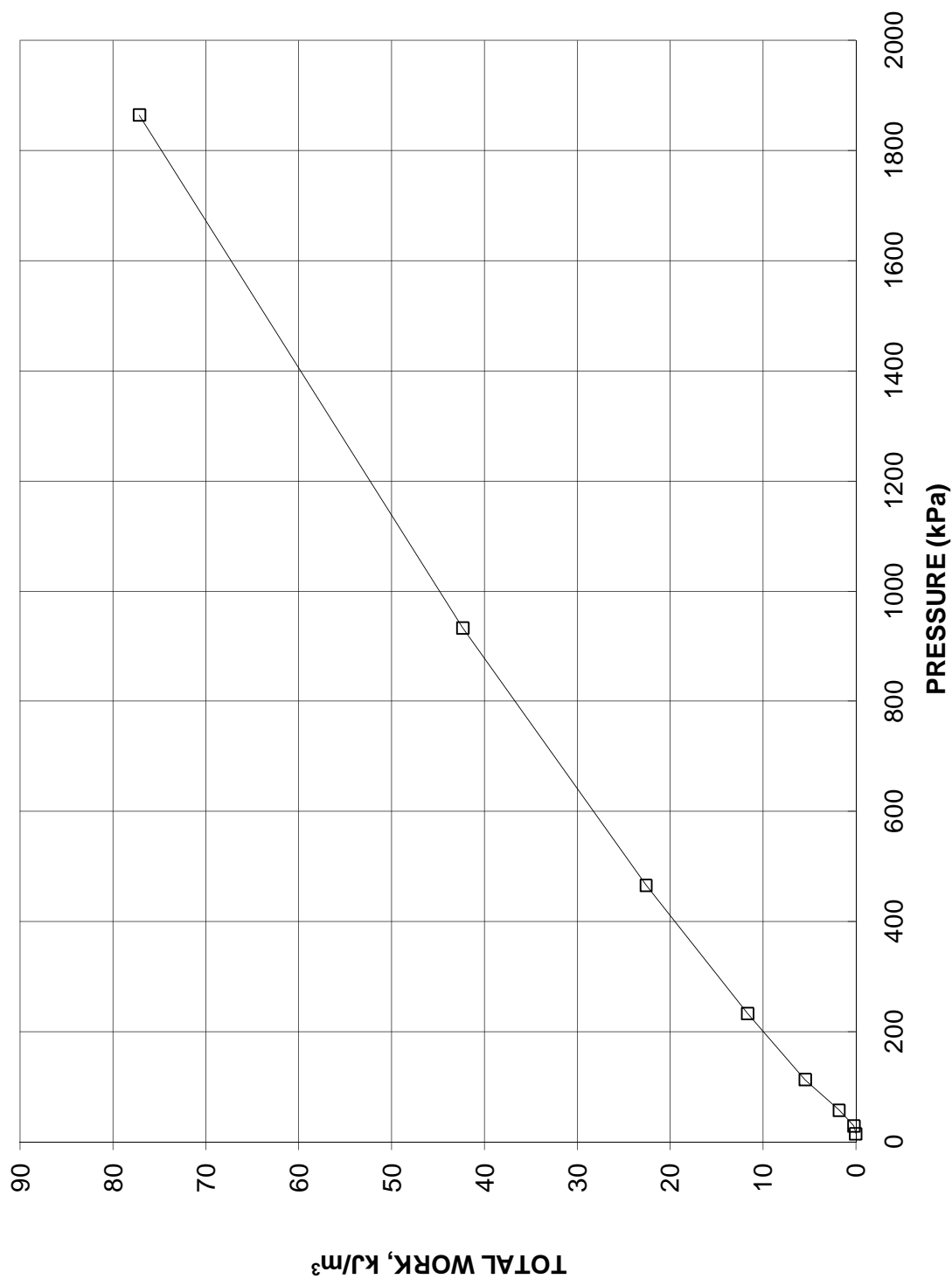
FIGURE C-5
Page 3 of 4



**PRIMARY CONSOLIDATION TEST
TOTAL WORK VS. PRESSURE**

**FIGURE C-5
Page 4 of 4**

**CONSOLIDATION TEST
TOTAL WORK, kJ/m^3 vs PRESSURE
SWAMP 5 BH 5, Sa 5**





APPENDIX D

Non-Standard Special Provisions and Operational Constraints

ROCK DOWELS – 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).

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 rock dowels 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
Highway 529 IC Underpass	Centre Pier and East Abutment	2

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

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

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

ROCK DOWELS – Item No.

Non-Standard Special Provision

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 0.0025 cm. 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.

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 piles, as applicable. Piles will be driven to bedrock.

References

OPSS 906 – Structural Steel

OPSS 903 – Construction Specification for Deep Foundations

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 consistent with 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 – 75 mm minus Rock Fill Placement as Swamp Backfill

The Contactor should provide rock fill with maximum particle size of 75 mm in the area of the west abutment as swamp backfill and embankment fill to facilitate pile driving.

Operational Constraint – Obstructions - Existing Blast Rock Fill

As part of the work for the sub-excavation at the west abutment area, the Contactor shall be alerted that the existing embankment contains blast rock fill.

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

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