



July 3, 2015

FOUNDATION INVESTIGATION AND DESIGN REPORT

**SHAWANAGA RIVER SERVICE ROAD (SITE NO. 9) BRIDGE STRUCTURE
SITE NO. 44-65
HIGHWAY 69 FOUR-LANING FROM 1.0 KM NORTH OF THE NEW HIGHWAY
559 INTERCHANGE NORTHERLY TO 1.5 KM NORTH OF HIGHWAY 7182
(SHEBESHEKONG ROAD) FOR 17 KM
MINISTRY OF TRANSPORTATION, ONTARIO
G.W.P. 5111-07-00, W.P. 5186-06-01 (Phase 2 of G.W.P. 5402 05 00)**

Submitted to:

McCormick Rankin, a member of MMM Group Limited
2655 North Sheridan Way
Mississauga, Ontario
L5K 2P8



REPORT

GEOCREs No.: 41H-143

Report Number: 07-1111-0029-4

Distribution:

- 3 Copies Ministry of Transportation, Ontario, North Bay, Ontario, (Northeastern Region)
- 1 Copy Ministry of Transportation, Ontario, Downsview, Ontario (Foundations Section)
- 2 Copies McCormick Rankin, a member of MMM Group Limited, Mississauga, Ontario
- 2 Copies Golder Associates Ltd. Mississauga, Ontario


**A world of
capabilities
delivered locally**





Table of Contents

PART A – FOUNDATION INVESTIGATION REPORT

1.0 INTRODUCTION	1
2.0 SITE DESCRIPTION	1
3.0 INVESTIGATION PROCEDURES.....	2
3.1 Foundation Investigation	2
4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS.....	3
4.1 Regional Geology.....	3
4.2 Subsurface Conditions	4
4.2.1 Topsoil / Asphalt.....	4
4.2.2 Sand and Gravel to Gravelly Sand Fill.....	4
4.2.3 Rock Fill.....	5
4.2.4 Sand and Gravel.....	5
4.2.5 Bedrock	5
4.2.6 Groundwater Conditions.....	7
5.0 CLOSURE	7

PART B – FOUNDATION DESIGN REPORT

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS	8
6.1 General	8
6.2 Foundation Options.....	9
6.3 Spread Footings.....	9
6.3.1 Footing Founding Options	9
6.3.1.1 South Abutment	9
6.3.1.2 North Abutment.....	11
6.3.2 Geotechnical Axial Resistances / Reactions	13
6.3.3 Resistance to Lateral Loads	13
6.3.4 Frost Protection	14
6.3.5 Other Considerations.....	14
6.3.5.1 Removal of Existing Concrete Abutments.....	14
6.3.5.2 Footing Set-Back from Rock Faces.....	15



FOUNDATION REPORT – SERVICE ROAD (SITE NO. 9) BRIDGE STRUCTURE – HIGHWAY 69 G.W.P. 5111-07-00

6.3.5.3	Sequence of Rock Excavation	15
6.4	Seismic Considerations.....	15
6.4.1	Site Coefficient	15
6.4.2	Seismic Analysis Coefficient.....	15
6.5	Lateral Earth Pressures.....	15
6.6	Approach Embankment Design.....	17
6.6.1	Stability	18
6.6.1.1	Methodology.....	18
6.6.1.2	Parameter Selection.....	18
6.6.1.3	Results of Analysis	19
6.6.1.4	Embankment Fill Types.....	19
6.6.2	Settlement	20
6.6.2.1	Methodology.....	20
6.6.2.2	Parameter Selection.....	20
6.6.2.3	Settlement of Foundation Soils	21
6.6.2.4	Settlement of New Embankment Fill	22
6.6.2.5	Embankment Platform Widening.....	23
6.7	Subgrade Preparation and Embankment Construction	23
6.7.1	Removal of Organic Materials	23
6.7.2	Embankment Fill Placement.....	24
6.8	Design and Construction Consideration	24
6.8.1	Overburden Excavation	24
6.8.2	Control of Groundwater and Surface Water.....	24
6.8.3	Obstructions	25
6.8.4	Existing Concrete Abutments	25
6.8.5	Permit To Take Water	25
6.9	Recommendations for Rock Excavations and Blasting	25
6.9.1	Rock Excavation.....	25
6.9.2	Blasting.....	26
7.0	CLOSURE	26



FOUNDATION REPORT – SERVICE ROAD (SITE NO. 9) BRIDGE STRUCTURE – HIGHWAY 69 G.W.P. 5111-07-00

REFERENCES

TABLES

Table 1	Evaluation of Foundation Alternatives
---------	---------------------------------------

DRAWINGS

Drawing 1	Site Location Plan
Drawing 2	Borehole Location and Soil Strata
Drawing 3	Soil Strata

FIGURES

Figure 1	Stability Analysis Results – South Approach – STA 12+419
Figure 2	Stability Analysis Results – North Approach – STA 12+463

APPENDICES

Appendix A **Record of Boreholes and Drillholes**

List of Symbols and Abbreviations
Lithological and Geotechnical Rock Description Terminology
Record of Borehole Sheets B3-01 to B3-12
Record of Drillhole Sheets B3-01 to B3-04, B3-06, B3-08 and B3-11

Appendix B **Laboratory Test Results**

Figure B1-1	Grain Size Distribution – Sand to Sand and Gravel Fill – South Abutment
Figure B1-2	Grain Size Distribution – Sand and Gravel Fill – North Abutment
Figure B1-3	Grain Size Distribution – Sand and Gravel – North Approach
Table B1	Point Load Test Results on Rock Samples
Table B2-1	Summary of Unconfined Compression Test Results
Table B2-2	Unconfined Compression (UC) Test – Borehole B3-02, Run No. 1
Table B2-3	Unconfined Compression (UC) Test – Borehole B3-06, Run No. 1
Table B2-4	Unconfined Compression (UC) Test – Borehole B3-08, Run No. 1

Appendix C **Non-Standard Special Provisions**

Dowels Into Rock	– Item No.
Obstructions	– Item No.



PART A

FOUNDATION INVESTIGATION REPORT

SHAWANAGA RIVER SERVICE ROAD (SITE NO. 9) BRIDGE STRUCTURE

SITE NO. 44-65

HIGHWAY 69 FOUR-LANING

FROM 1.0 KM NORTH OF THE NEW HIGHWAY 559

INTERCHANGE NORTHERLY TO 1.5 KM NORTH OF

HIGHWAY 7182 (SHEBESHEKONG ROAD) FOR 17 KM

MINISTRY OF TRANSPORTATION, ONTARIO

G.W.P. 5111-07-00, W.P. 5186-06-01 (Phase 2 of G.W.P. 5402-05-00)



1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by McCormick Rankin (MRC), a member of MMM Group Limited on behalf of Ministry of Transportation, Ontario (MTO) to carry out a detail foundation investigation services for the proposed Shawanaga River Service Road (Site No. 9) one-span bridge structure over the Shawanaga River (Site No. 44-65). The proposed work is part of the detail design for the four-laning of Highway 69 from 1.0 km north of the new Highway 559 Interchange northerly to 1.5 km north of Highway 7182 (Shebeshekong Road), which involves high fill embankments and embankments over swamps, the New Woods Road and Shebeshekong Road interchanges and structures, the Shawanaga River and Site 9 Road structures, as well as culvert crossings. The general location of this section of the Highway 69 four-laning alignment is shown on Drawing 1.

The terms of reference and the scope of work for the foundation investigation are outlined in MTO's Request for Proposal, dated July 2006. Golder's proposal for foundation engineering services associated with the Shawanaga River Service Road (Site No. 9) bridge structure is contained in Section 6.8 of MRC's Technical Proposal for this assignment. The work has been carried out in accordance with Golder's Supplemental Specialty Quality Control Plan for foundation engineering services for this project, dated July 4, 2007. The General Arrangement (GA) Drawing and the subsequent updated GA Drawing for the proposed Shawanaga River Service Road bridge were provided to Golder by MRC on June 27, 2008 and September 10 and 19, 2014.

This report addresses the investigation carried out for the Shawanaga River Service Road bridge structure and immediately adjacent approach embankments only. Separate reports address the foundation investigations and design for the related swamp crossings and high fill areas for the associated interchange ramps and roadways, culverts and other bridge structures for the project.

The purpose of this investigation is to establish the subsurface conditions at the proposed structure, including the associated approach embankments, by borehole drilling, rock coring and laboratory testing on selected samples. The foundation units/limits for this investigation were located in the field by Callon Dietz Inc. (Callon Dietz), a professional surveying company retained by MRC. The investigation area is shown in plan on Drawing 2.

2.0 SITE DESCRIPTION

The proposed Shawanaga River Service Road (Site No. 9) bridge structure is located approximately 400 m north of the intersection of the existing Shebeshekong Road and Highway 69 and is approximately 20.5 km northwest of Nobel, Ontario. The existing Highway 69, which will become part of the future Shawanaga River Service Road (Site No. 9), runs generally in a southeast-northwest direction on the east side of the proposed new Highway 69.

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 bridge structure and associated approach embankments are to be situated on a relatively flat, moderately treed area and a bedrock outcrop at the north abutment/approach embankment. The ground surface within the limits of the proposed structure and approach embankment areas is between about Elevation 205.1 m and 208.5 m, referenced to Geodetic datum, and is sloping upward from south to north.



3.0 INVESTIGATION PROCEDURES

3.1 Foundation Investigation

The field work for the Shawanaga River Service Road bridge structure investigation was carried out between October 21 and 27, 2008 during which time a total of twelve (12) boreholes were advanced: five (5) boreholes at the south abutment; five (5) boreholes at the north abutment; and two (2) boreholes at the approach embankments (i.e. one (1) borehole at each approach). The boreholes, designated as Boreholes B3-01 to B3-12, were advanced at the locations shown in plan on Drawing 2.

The field investigation was carried out using a Diedrich D-50 Turbo track-mounted drill rig supplied and operated by Walker Drilling Co. Ltd. of Utopia, Ontario. The boreholes were advanced through the overburden using 'NW' casing. Soil samples were obtained at intervals of depth of about 0.75 m using a 50 mm outside diameter (O.D.) split spoon sampler driven by an automatic hammer in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586 Standard Test Method for Standard Penetration Test); a chunk sample was obtained in one borehole containing thin overburden over a bedrock outcrop. Samples of the bedrock were obtained using an 'NQ' size rock core barrel.

The boreholes at the foundation elements were typically advanced to casing and/or sampler refusal (i.e. inferred bedrock) while the boreholes at the approach embankments were advanced to a depth approximately equal to the height of the proposed embankment or to sampler refusal. It should be noted that some of the boreholes were terminated on refusal to shovel excavation. Four (4) boreholes (Boreholes B3-05, B3-07, B3-09 and B3-10) are located on an exposed bedrock outcrop; the remaining boreholes were drilled to depths ranging from about 0.4 m to 6.9 m below existing ground surface, including coring of bedrock for core lengths between about 2.2 m and 3.2 m in Boreholes B3-01 to B3-04, B3-06, B3-08 and B3-11.

The groundwater conditions in the open boreholes were observed during the drilling operations and a piezometer was installed in Borehole B3-03 to permit monitoring of the water level at this location. The piezometer consists of 32 mm diameter PVC pipe, with a slotted screen sealed at a select depth within the borehole. The borehole and annulus surrounding the piezometer pipe above the screen sand pack was backfilled to the surface with bentonite pellets/grout. Piezometer installation details and water level readings are described on the Record of Borehole sheets presented in Appendix A. All boreholes in which standpipe piezometers were not installed were backfilled with bentonite upon completion in accordance with Ontario Regulation 903 Wells (as amended).

The field work was monitored by members of our engineering and technical staff, who located the boreholes, arranged for the clearance of underground services, observed the drilling, sampling and in situ testing operations, logged the boreholes, and examined and cared for the soil and rock samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Mississauga geotechnical laboratory where the samples underwent further visual examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate. Classification testing (water content, Atterberg limits and grain size distribution) was carried out on selected samples. Strength testing, such as uniaxial compression and point load index tests were carried out on specimen of the rock core. The results of the laboratory testing are included in Appendix A.

The perimeter limits of each foundation unit were located in the field by Callon Dietz prior to drilling. The as-drilled borehole locations and ground surface elevations were surveyed by a member of our technical staff, referenced to the survey stakes put down by Callon Dietz. The borehole locations given in the Record of



FOUNDATION REPORT – SERVICE ROAD (SITE NO. 9) BRIDGE STRUCTURE – HIGHWAY 69 G.W.P. 5111-07-00

Borehole/Drillhole sheets and shown on Drawing 2 are positioned relative to MTM NAD 83 northing and easting coordinates and the ground surface elevations are referenced to Geodetic datum and are summarized below.

Borehole	Location (MTM NAD 83)		Ground Surface Elevation (m)	Depth Drilled (m)
	Northing	Easting		
B3-01	5045823.0	243591.4	205.6	2.6
B3-02	5045830.5	243575.2	206.4	4.6
B3-03	5045835.9	243585.8	205.5	6.9
B3-04	5045834.6	243579.0	205.9	4.4
B3-05	5045833.4	243572.1	205.1	0.0
B3-06	5045838.8	243582.7	205.6	5.9
B3-07	5045860.4	243559.5	208.5	0.0
B3-08	5045855.1	243548.9	205.6	3.7
B3-09	5045855.8	243556.3	207.9	0.0
B3-10	5045857.5	243562.6	207.8	0.0
B3-11	5045852.2	243552.0	205.6	5.5
B3-12	5045869.4	243541.7	211.6	0.4

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

As delineated in The Physiography of Southern Ontario¹, 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; numerous bare knobs and ridges of bedrock are present throughout the area. Localized 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.

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.



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 Palaeozoic 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 for this investigation, together with the results of the laboratory tests carried out on selected soil and bedrock core samples, are provided in Appendix A and B, respectively. The results of the in situ field tests (i.e. SPT 'N'-values) as presented on the Record of Borehole sheets and in Sections 4.2.1 to 4.2.5 are uncorrected. The stratigraphic boundaries shown on the Record of Borehole and Drillhole sheets are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. It should be noted that the interpreted stratigraphy shown on Drawings 2 and 3 is a simplification of the subsurface conditions. Variation in the stratigraphic boundaries between and beyond boreholes will exist and is to be expected.

In general, the subsurface conditions in the area of the proposed bridge structure consist of a surficial layer of sand to sand and gravel fill associated with the existing north and south abutments of the original Highway 69 structure and embankments. The sand to sand and gravel fill is in turn underlain by granitic rock fill in places, underlain by granite gneiss/ syenite gneiss bedrock. Bedrock was observed to outcrop within the vicinity of the proposed foundation elements and the north approach embankment.

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

4.2.1 Topsoil / Asphalt

Approximately 40 mm to 80 mm of topsoil or asphalt was encountered at the ground surface in Boreholes B3-01, B3-03, B3-08, B3-11 and B3-12.

4.2.2 Sand and Gravel to Gravelly Sand Fill

A deposit of fill comprised of brown sand to sand and gravel was encountered at the ground surface or below the topsoil/asphalt in Boreholes B3-01 to B3-04, B3-06, B3-08 and B3-11. The top of the fill was encountered at about Elevation 205.5 m and 205.4 m in the boreholes drilled at the south abutment/approach embankment and between about Elevation 207.9 m and 205.5 m at the north abutment/approach embankment. The thickness of the fill deposit is highly variable across the site, ranging from about 0.4 m to 2.4 m. In general, the fill deposit is thicker at the south abutment than at the north abutment.

The fill varies in composition from sand some gravel, to gravelly sand, to sand and gravel, trace to some silt, trace clay containing cobbles, boulders, rock and asphalt fragments, organics, topsoil and rootlets. In some boreholes, oxidation zones were encountered to a depth of about 2.4 m below ground surface. The grain size

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



distributions of seven (7) samples from the fill are presented on Figures B1-1 and B1-2, in Appendix B. The organic content measured on three (3) samples of the fill is about 2 percent and 3 percent, indicating that the fill is slightly organic. The natural water content measured on samples of the sand to sand and gravel fill range from about 3 percent to 18 percent.

The Standard Penetration Test (SPT) 'N'-values measured within the fill deposit range from 6 blows to 46 blows per 0.3 m of penetration, but typically greater than 11 blows per 0.3 m of penetration, indicating a loose to dense relative density.

4.2.3 Rock Fill

In Boreholes B3-03 and B3-06 the sand fill is underlain locally by rock fill extending to the bedrock at depths of about 3.7 m and 2.7 m, respectively, below ground surface. The top of the rock fill was encountered at Elevation 204.9 m and 205 m and the thickness of the deposit is 3.1 m and 2.1 m in Boreholes B3-03 and B3-06, respectively. The rock fill consists of cobble and boulder sizes containing gravel, sand and silt.

The natural water content measured on two (2) samples of the rock fill is about 1 percent and 4 percent.

The SPT 'N'-values measured within the rock fill range from 5 blows per 0.3 m of penetration to 50 blows per 0.08 m of penetration, generally indicating a loose to very dense relative density.

4.2.4 Sand and Gravel

In Borehole B3-12, a layer of sand and gravel was encountered immediately below the ground surface at Elevation 211.6 m. The sand and gravel deposit extends to refusal on inferred bedrock at a depth of 0.4 m below ground surface corresponding to Elevation 211.2 m.

The sand and gravel contains some silt and trace clay. A grain size distribution of one (1) sample from the sand and gravel is presented on Figure B1-3, in Appendix B.

4.2.5 Bedrock

Bedrock was encountered and core samples were recovered from Boreholes B3-01 to B3-04, B3-06, B3-08 and B3-11. Bedrock outcrops were observed towards the west edge of the south abutment near the location of Borehole B3-05 and across the north abutment from the middle to the east edge near the locations of Boreholes B3-07, B3-09 and B3-10; the presence of bedrock was inferred from shovel refusal at Borehole B3-12. The depth of the surface of the bedrock is variable and ranges from ground surface to 3.7 m below ground surface. Across the south abutment and north abutment, from west to east (a distance of about 10 m between borehole locations), the bedrock surface elevation varies between about 2.2 m and 3.9 m; equivalent to approximately 4.5H:1V (4.5H:1V) slope or a dip of approximately 12 degrees from the horizontal at the south abutment and approximately 2.6H:1V slope or a dip of approximately 21 degrees from the horizontal at the north abutment.

The depth to bedrock below ground surface, corresponding bedrock surface elevation and refusal type is summarised below.



FOUNDATION REPORT – SERVICE ROAD (SITE NO. 9) BRIDGE STRUCTURE – HIGHWAY 69 G.W.P. 5111-07-00

Foundation Element / Approach Embankment	Borehole No.	Depth to Bedrock Surface (m)	Bedrock Surface Elevation (m)	Refusal Type
South Approach Embankment	B3-01	0.4	205.2	Bedrock Cored
South Abutment	B3-02	1.4	205.0	Bedrock Cored
	B3-03	3.7	201.8	Bedrock Cored
	B3-04	1.3	204.6	Bedrock Cored
	B3-05	0.0	205.1	Bedrock Outcrop
	B3-06	2.7	202.9	Bedrock Cored
North Abutment	B3-07	0.0	208.5	Bedrock Outcrop
	B3-08	1.0	204.6	Bedrock Cored
	B3-09	0.0	207.9	Bedrock Outcrop
	B3-10	0.0	207.8	Bedrock Outcrop
	B3-11	2.4	203.2	Bedrock Cored
North Approach Embankment	B3-12	0.4	207.5	Bedrock Outcrop

Based on the cored bedrock samples, the bedrock generally consists of syenite gneiss (at the south abutment) and granite gneiss (at the north abutment). In general, the bedrock samples are described as slightly weathered to fresh, fine to coarse crystalline, slightly foliated to foliated, fine to coarse grained/fine crystalline, black, white and pink syenite/granite containing pink veins and porphyry. The Rock Quality Designation (RQD) measured on the core samples is typically between about 53 percent and 100 percent, indicating a rock mass of fair to excellent quality. However, upper portions of core recovered from Boreholes B3-01 at the south abutment contains zones of highly weathered rock with RQD of about 0 percent, indicating a rock mass of very poor quality. The Total Core Recovery (TCR) and Solid Core Recovery (SCR) of samples recovered is typically between 92 percent and 100 percent and 61 percent and 94 percent, respectively; however at Boreholes B3-02 to B3-04 and B3-08 the SCR of samples recovered within the upper portion of the core is between about 33 percent and 57 percent.

Point load strength tests were performed on selected samples of the rock core. The diametral and axial point load strength index values are shown on the Record of Drillhole sheets and are presented in Table B1 in Appendix B. The axial point load index (Is_{50}) results from the laboratory tests carried out on three (3) samples of the syenite gneiss bedrock range from approximately 5.6 MPa to 8.5 MPa with an average of about 6.9 MPa. The diametral tests carried out on two (2) samples of the syenite gneiss bedrock measured Is_{50} values of about 7.1 MPa and 7.7 MPa. The axial point load index (Is_{50}) results from the laboratory tests carried out on two (2) samples of the granite gneiss bedrock were approximately 5.4 MPa and 6.9 MPa. The diametral tests carried out on four (4) samples of the granite gneiss bedrock measured Is_{50} values ranging from 6.5 MPa to 11.4 MPa with an average of 7.3 MPa. A lower axial Is_{50} value of 0.6 MPa and a lower diametral Is_{50} value of 2.4 MPa were measured on two samples within the upper weathered portion of the granite gneiss bedrock and may not be representative of the overall strength of the granite gneiss bedrock.

Two (2) Unconfined Compression (UC) tests were carried out, in accordance to ASTM D7102 (Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures), on selected samples of the syenite gneiss bedrock measured compressive strengths of about 79 MPa and 115 MPa, and one (1) UC test completed on a sample of the granite gneiss bedrock



FOUNDATION REPORT – SERVICE ROAD (SITE NO. 9) BRIDGE STRUCTURE – HIGHWAY 69 G.W.P. 5111-07-00

measured a compressive strength of about 70 MPa, as summarised on Table B2-1 and detailed in Tables B2-2 to B2-4 in Appendix B.

Also presented in Table B1 are the estimated Uniaxial Compressive Strength (UCS) values for each sample tested for point load strength based on a relationship between Is_{50} and UCS which is given by a correlation factor (K) in accordance with (ASTM D5731 Standard Test Method for Determination of the Point Load Strength Index of Rock), which varies depending on the size of the core sample and the strength of the rock. For this site, the UCS values are based on an estimated average correlation factor (K) of 14 which was calculated based on a comparison of the UC test results and the point load strength test results. These values have been given for comparison only and should be interpreted together with the results of the UC test.

Based on the laboratory UC tests and point load testing results in accordance with Table 3.5, CFEM (2006)³ the syenite gneiss bedrock is classified as strong (R4, 50 MPa < UCS < 100 MPa) to very strong (R5, 100 MPa < UCS < 250 MPa) and the granitic gneiss bedrock is classified as weak (R2, 5 MPa < UCS < 25 MPa) within the upper weathered zones to strong (R4, 50 MPa < UCS < 100 MPa).

4.2.6 Groundwater Conditions

The water level noted during and upon completion of drilling operations in Borehole B3-11 is at about Elevation 203.3 m, measured at about 2.3 m below ground surface and the remaining open boreholes were dry. In general, the samples taken in all the overburden boreholes advanced in this area were moist. A standpipe piezometer was installed in Borehole B3-03 to permit monitoring of the groundwater level at this location. Details of the piezometer installation are shown the Record of Borehole and Drillhole sheets in Appendix A. The groundwater levels measured in the piezometer installation are summarized below.

Foundation Element	Borehole No.	Ground Surface Elevation (m)	Groundwater Elevation (m)	Date of Measurement
South Abutment	B3-03	205.5	201.6	November 5, 2008
			201.5	November 6, 2008

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

5.0 CLOSURE

The field technician directing the drilling program was Mr. Chris Radway. This report was prepared by Ms. T. Veronica Ayetan, P. Eng., and was reviewed by Mr. J. Paul Dittrich, Ph.D., P. Eng., a senior geotechnical engineer and Principal with Golder. Mr. Jorge M. A. Costa, P.Eng., Golder's Designated MTO Contact for this project and Principal with Golder, conducted an independent quality control review of the report.

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



FOUNDATION REPORT – SERVICE ROAD (SITE NO. 9) BRIDGE STRUCTURE – HIGHWAY 69 G.W.P. 5111-07-00

Report Signature Page



J. Paul Dittrich, Ph.D., P. Eng.,
Senior Geotechnical Engineer, Principal



Jorge M.A. Costa, P. Eng.,
Designated MTO Contact, Principal

TVA/JPD/JMAC/tva/jl

\\golder.gds\gal\mississauga\active\2007\1111\07-1111-0029 - mrc - hwy 69 four-laning -report\final\4 - site 9 road bridge structure (shawanaga river)\07-1111-0029-4 fdr 15jul03 highway 69 - site 9 road bridge.docx



PART B

FOUNDATION DESIGN REPORT

**SHAWANAGA RIVER SERVICE ROAD (SITE NO. 9) BRIDGE STRUCTURE
SITE NO. 44-65**

HIGHWAY 69 FOUR-LANING

FROM 1.0 KM NORTH OF THE NEW HIGHWAY 559

INTERCHANGE NORTHERLY TO 1.5 KM NORTH OF

HIGHWAY 7182 (SHEBESHEKONG ROAD) FOR 17 KM

MINISTRY OF TRANSPORTATION, ONTARIO

G.W.P. 5111-07-00, W.P. 5186-06-01 (Phase 2 of G.W.P. 5402-05-00)



6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides engineering design recommendations for the proposed Shawanaga River Service Road (Site No. 9) bridge. 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 foundations 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

Golder Associates Ltd. (Golder) was retained by McCormick Rankin (MRC), a member of MMM Group Limited on behalf of Ministry of Transportation, Ontario (MTO) to provide recommendations on foundation aspects for the new Shawanaga River Service Road (Site 9) structure and approach embankments. The scope of work includes carrying out stability and settlement analyses, the assessment of foundation options and provision of geotechnical resistances. The work also includes addressing foundation aspects for the final design and construction of the structure foundations and approach embankments, including requirements for sub-excavation of organic materials and placement of new fill as well as requirements for rock excavation and blasting.

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

It is understood that the Shawanaga River Service Road (Site 9) bridge structure will be a one-span, pre-cast concrete girder bridge with a 31 m span length with the abutments located north and south of the Shawanaga River. The alignment for the proposed Service Road bridge structure is east of the existing Highway 69.

Based on the General Arrangement (GA) Drawing provided by MRC on July 24, 2009 and subsequent revision to the GA Drawing provided on September 10 and 19 2014, the grade of the proposed Shawanaga River Service Road bridge deck varies between about Elevation 207.8 m (south abutment) and Elevation 207.6 m (north abutment). The existing ground surface within the south approach embankment area varies from about Elevation 205 m to 206.5 m; therefore, the new south approach embankment will require fills up to about 2.5 m above the existing ground surface. Within the majority of the north approach embankment, the bedrock is at or near the existing ground surface. On the east side of the north abutment and along much of the approach, the exposed bedrock outcrop gradually rises from about Elevation 208 m to 211.5 m; therefore in this area, a rock cut of up to about 4 m will be required. On the west side of the north abutment and along a short length of the approach, the existing ground/old roadway surface is at about Elevation 205.6 m; therefore the new north approach embankment will require fills up to about 2 m above the existing ground surface in this area. The normal (local) surface water level of the Shawanaga River is indicated on the GA Drawing provided by MRC to be at about Elevation 199.7 m.



It should be noted that the old concrete bridge abutments from the previous Highway 69 alignment (pre-1957) are present at the site and are located partially within the new proposed Service Road structure alignment. It is our understanding that the old bridge abutments are to be removed in whole (or partially) as part of the new construction.

6.2 Foundation Options

Shallow foundations comprised of spread footings founded either directly on the bedrock (or mass concrete on bedrock) are considered appropriate to support the bridge structure given the proximity of bedrock to the existing ground surface and proposed underside of footings.

Pile foundations have also been considered, however, given the shallow depth to bedrock and considering that the existing fill behind the old bridge abutments/within the vicinity of the proposed abutments is comprised of rock fill at some depths, the installation of piles will be difficult. In addition, to achieve the minimum required pile lengths for an integral abutment design, installation of the piles would require significant excavation/trenching into the strong to very strong bedrock which would likely be cost prohibitive.

The following sections provide recommendations for shallow spread footings to support the proposed structure. Given the significant excavation/trenching of the bedrock that would be required to install pile foundations, this option is considered impractical at this site and is not discussed further in this report. From a foundations perspective, the shallow foundation option is considered more practical for construction and is the preferred foundation alternative.

The advantages, disadvantages, relative costs and risks/consequences for each of the foundation options are summarised in Table 1.

6.3 Spread Footings

At this site, shallow foundations comprised of spread footings founded on either the bedrock or mass concrete on bedrock are considered the preferred alternative for support of the structure.

The following sections outline the recommendations for footing founding options, geotechnical axial resistance, resistance to lateral loads, requirements for frost protection and other considerations such as minimum set-back etc.

6.3.1 Footing Founding Options

There are several founding level alternatives for support of the north and south abutment footings at the site. The alternatives are summarised below.

6.3.1.1 South Abutment

The existing ground surface in the area of the south abutment footprint varies from about Elevation 205.5 m to 206.5 m. Fill consisting of loose to compact sand to sand and gravel containing rootlets and topsoil was encountered immediately below the ground surface. At the boreholes drilled along the eastern edge of the



FOUNDATION REPORT – SERVICE ROAD (SITE NO. 9) BRIDGE STRUCTURE – HIGHWAY 69 G.W.P. 5111-07-00

proposed abutment, the sand and gravel fill is underlain by rock fill. The fill materials are underlain by syenite gneiss bedrock.

The south abutment for the old Highway 69 bridge alignment (pre-1957) is located immediately in front (i.e. to the north) of the proposed new south abutment and it is anticipated that the majority of the old abutment will be removed for the construction of the south abutment footing as depicted on the GA Drawing.

Based on the GA Drawing, the underside of the 3 m wide footing for the south abutment is proposed to be at Elevation 203.6 m (considering a 1 m thick footing). It is recommended that the new footing extend below all fill materials and any existing concrete from the old abutment, if present within the footprint, and be founded on competent bedrock. The surface of the bedrock at the proposed south abutment, as encountered at the investigated locations, is variable over the length of the proposed footing and ranges from about Elevation 201.8 m at the eastern portion of the proposed footing to about Elevation 205.1 m at the western portion. In general, the bedrock at or below the proposed founding level at the borehole locations is of fair to excellent quality with RQD ranging from about 53 percent to 94 percent. However, the quality of the bedrock may, in places, be variable and any loose or fractured bedrock encountered at the founding level will need to be sub-excavated and removed prior to footing construction.

The details of the ground surface elevation, bedrock surface elevation, depth to the bedrock surface below existing ground surface and depth to bedrock below the underside (u/s) of the proposed footing, as encountered in the boreholes at the south abutment, is summarized below.

Borehole Location Within South Abutment	Borehole	Ground Surface Elevation (m)	Bedrock Surface Elevation (m)	Depth to Bedrock below Existing Ground Surface (m)	Depth to Bedrock below U/S of Proposed Footing ¹ (m)
West Side	B3-02	206.4	205.0	1.4	-1.4
West Side	B3-05	205.1	205.1	--	-1.5
Centre	B3-04	205.9	204.6	1.3	-1.0
East Side	B3-03	205.5	201.8	3.7	1.8
East Side	B3-06	205.6	202.9	2.7	0.7

Note: ¹ Negative (-) implies underside (u/s) of proposed footing (at Elevation 203.6 m) is lower than the top of existing bedrock surface.

Based on the borehole results within the western portion of the footprint of the proposed abutment, the bedrock surface is between about 1 m and 1.5 m above the proposed footing elevation and therefore will need to be removed to reach the founding level at Elevation 203.6 m. Recommendations for excavation of the bedrock and sequence of construction are provided in Section 6.9.

Along the eastern portion of the proposed footing, the bedrock surface is between 0.7 m and 1.8 m below the proposed footing level at Elevation 203.6 m. Consideration could be given to lowering the footing level to the minimum elevation of the bedrock surface as encountered in the boreholes (i.e. to Elevation 201.8 m), however, this option would require substantial additional rock excavation (up to 3.3 m) across the footprint and it would also result in a much higher abutment stem wall. In order to minimize the height of the abutment stem wall and



the amount of bedrock excavation, it is recommended that mass concrete be placed to raise the grade to the proposed founding level. All mass concrete construction should be in accordance with OPSS.PROV 904 (*Concrete Structures*).

As an alternative to supporting the abutment footing on bedrock (or mass concrete), consideration could be given to employing a well compacted granular pad placed on top of the bedrock surface. However, the use of a granular pad poses several risks to the design and performance of the footing at this location. Firstly, in order to avoid stress concentrations within the footing that could occur due to differential settlement at the transition between a portion of the footing on bedrock and a portion on the granular pad, it would be necessary to remove additional bedrock across the footprint (i.e. down to about Elevation 202.6 m) to ensure that the granular pad is a minimum 1 m thick at all locations below the footing, requiring up to about 2.5 m of rock cutting.

Secondly, given the unprotected nature of some portions of the footing at this location (i.e. a section of the northern and eastern part of the footing will be exposed, effectively 'perched' on the bedrock outcrop), special design and construction measures (and possibly future maintenance or inspection) would be required to ensure that no loss of material from the granular pad occurs over the life of the footing.

Given these concerns, the use of granular pad to support the footing is not recommended at this site.

The preferred foundation option at the south abutment is to found the spread footing on bedrock along the western portion of the proposed footing and to use mass concrete to raise the grade to Elevation 203.6 m along the eastern portion of the proposed footing. Given the strength compatibility between the concrete footing, mass concrete and bedrock as well as ease of placement for footing construction and overall long-term stability of the concrete mass (i.e. minimal potential for undermining of the foundation over the life of the structure), this approach is considered the most suitable at this location.

6.3.1.2 North Abutment

In the area of the new north abutment there is an existing near vertical rock face/cut which is about coincident with the centre of the proposed north abutment footprint. Across the western portion of the proposed footing the ground/old roadway surface is at about Elevation 205.6 m. At the eastern portion of the proposed footing, the top of the near vertical rock face/cut is at about Elevation 208 m, which corresponds to a 2.4 m difference in existing ground surface elevation. Fill was encountered in Boreholes B3-08 and B3-11, drilled at the northwest and southwest corner of the north abutment, and extended to about 1.0 m and 2.4 m below existing ground/old roadway surface, respectively. The fill consists of loose to dense sand to sand and gravel with some silt. The fill material is underlain by granitic gneiss bedrock.

The north abutment for the old Highway 69 bridge alignment (pre-1957) is located immediately in front (i.e. to the south) on the west side of the proposed new north abutment and it is anticipated that a portion of the old abutment will be removed when the existing rock face is cut back to the geometry shown on the GA Drawing.

Based on the GA Drawing, the underside of the approximately 3.7 m wide footing for the north abutment is proposed to be at Elevation 204.0 m (considering a 0.5 m thick footing). It is recommended that the new footing extend below all fill materials and any existing concrete from the old abutment, if present within the footprint, and be founded on competent bedrock. The surface of the bedrock at the proposed north abutment, as encountered at the investigated locations is variable over the length of the proposed footing and ranges from about Elevation 203.2 m at the western portion of the proposed footing to about Elevation 208.5 m at the eastern



FOUNDATION REPORT – SERVICE ROAD (SITE NO. 9) BRIDGE STRUCTURE – HIGHWAY 69 G.W.P. 5111-07-00

portion. In general, the bedrock at or below the proposed founding level at the borehole locations is of fair to good quality with RQD ranging from about 50 percent to 89 percent. However, the quality of the bedrock may, in places, be variable and any loose or fractured bedrock encountered at the founding level will need to be sub-excavated and removed prior to footing construction.

The details of the ground surface elevation, bedrock surface elevation, depth to the bedrock surface below existing ground surface and depth to bedrock below the underside (u/s) of the proposed footing as encountered in the boreholes at the north abutment is summarized below.

Borehole Location Within North Abutment	Borehole	Ground Surface Elevation (m)	Bedrock Surface Elevation (m)	Depth to Bedrock below Existing Ground Surface (m)	Depth to Bedrock below U/S of Proposed Footing ¹ (m)
West Side	B3-11	205.6	203.2	2.4	0.8
West Side	B3-08	205.6	204.6	1.0	-0.6
Centre	B3-09	207.9	207.9	--	-3.9
East Side	B3-10	207.8	207.8	--	-3.8
East Side	B3-07	208.5	208.5	--	-4.5

Note: ¹ Negative (-) implies underside (u/s) of proposed footing (at Elevation 204.0 m) is lower than the top of existing bedrock surface.

Based on the borehole results, over much of the footprint of the proposed north abutment, the surface of the bedrock is located above the proposed footing elevation and between about 0.6 m and 4.5 m of rock excavation will therefore be required to reach the founding level at about Elevation 204.0 m. Recommendations for excavation of the bedrock and sequence of construction are provided in Section 6.9.

At the southwest portion of the proposed footing, the bedrock surface is about 0.8 m below the proposed footing level at Elevation 204.0 m. For this area, it is recommended that mass concrete be placed to raise the grade to the proposed founding level. All mass concrete construction should be in accordance with OPSS.PROV 904 (*Concrete Structures*).

The consideration of other alternatives, as discussed in Section 6.3.1.1 for the south abutment such as lowering the founding level or the use of a granular pad to avoid the use of mass concrete, is not considered warranted at this location given the relatively small area that requires mass concrete relative to the rest of the footing.

Therefore, the recommended foundation option at the north abutment is to found the spread footing on bedrock over much of the footprint and to use mass concrete to raise the grade to Elevation 204.0 at the southwest portion of the proposed footing, due to its strength characteristics, compatibility between the concrete footing, mass concrete and bedrock, ease of placement and its over-all long term durability (i.e. minimal potential for undermining over the life of the structure).



6.3.2 Geotechnical Axial Resistances / Reactions

The following summarises the design factored geotechnical axial resistances for spread footings placed on properly prepared syenite or granitic gneiss bedrock or mass concrete (founded on the properly prepared bedrock). For spread footings founded on the properly prepared and inspected bedrock or on mass concrete, the geotechnical resistance at SLS for 25 mm of settlement will be greater than the factored geotechnical axial resistance at ULS since the syenite/granite gneiss bedrock is considered to be an unyielding material; as such, ULS conditions will govern for this foundation type.

Foundation Element	Spread Footing Alternatives For the Proposed Abutments	Factored Geotechnical Axial Resistance at Ultimate Limit States (ULS)	Geotechnical Reaction at Serviceability Limit States (SLS) for 25 mm of Settlement
South Abutment	Spread Footings on Syenite Gneiss Bedrock and on Mass Concrete placed directly on Bedrock*	10,000 kPa	N/A
North Abutment	Spread Footings on Granite Gneiss Bedrock and on Mass Concrete placed directly on Bedrock*	10,000 kPa	N/A

Note: * Assumes mass concrete will have compressive strength greater than 25 MPa.

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*.

For footings placed on mass concrete, the factored geotechnical axial resistance at ULS is as given above for bedrock assuming that the compressive strength of the concrete used to form the pad is at least 25 MPa.

Following excavation of the overburden/fills and prior to placing any concrete, it will be necessary to clean, scale and remove all loose, shattered and/or fractured rock within the footprint of the footings to ensure a proper bond to the bedrock. The bedrock should be inspected by a Quality Verification Engineer (QVE) following excavation, in accordance with OPSS 902 (Excavating and Backfilling Structures), to ensure that it has been properly prepared. Field inspection should be carried out when the excavation is dry and in accordance with OPSS 902 (*Excavating and Backfilling*). In addition, a check on the sliding resistance between the mass concrete and the bedrock should be carried out (in accordance with the recommendations provided in Section 6.3.3).

6.3.3 Resistance to Lateral Loads

The 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, $\tan \delta$, for the interface between the mass concrete / concrete footing and bedrock is:

Interface Materials	Coefficient of Friction ($\tan \delta$)
Mass Concrete or Concrete Footing on Bedrock	0.70



The value presented above 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 mass concrete/concrete footing and the bedrock may be supplemented by dowelling into the bedrock, if necessary. The horizontal resistance of the dowels is dependent on the strength of the bedrock, grout and steel. A value of 750 kPa (factored) may be assumed for the grout-to-rock unit bond stress assuming minimum 30 MPa grout strength. This value is based on a factor of 0.4 for static analysis in tension (*CHBDC, 2006*).

For this site, where the rock mass is essentially as strong or stronger than concrete, the design of the dowels in the rock may be handled in the same way as the dowel embedment into the concrete. This assumes that the Uniaxial Compressive Strength (UCS) of the grout will be similar to that of concrete.

The dowels should have a minimum embedded length of 1 m within the competent bedrock, and the structural strength of the dowel and compressive strength of the grout should not be exceeded. If dowelling into bedrock is adopted for resistance to sliding at this site, a NSSP should be included in the Contract Documents to specify the installation, material and testing of the dowels; an example NSSP is included in Appendix C.

6.3.4 Frost Protection

The estimated depth of frost penetration at this site is 1.8 m, based on OPSD 3090.101 (*Foundation Frost Depths for Southern Ontario*).

The RQD of the upper portion of the bedrock below the proposed footing level(s) is generally greater than 50 percent, therefore for spread footings or mass concrete founded on the properly prepared syenite/granite gneiss bedrock at this site (i.e. all loose or shattered rock to be removed prior to construction), frost susceptibility is not an issue.

6.3.5 Other Considerations

Given some of the unique features at this site (including the presence of old concrete bridge abutments and steep bedrock outcrops), the following additional foundation concerns and recommendations should be considered.

6.3.5.1 Removal of Existing Concrete Abutments

The existing north and south concrete abutments from the old Highway 69 bridge alignment (pre-1957) are in close proximity to the new proposed abutments. Although concrete was not encountered in any of the core samples obtained from the boreholes advanced at the site, it is recommended that if concrete is encountered within the footprint of the proposed abutments during excavation, it should be removed prior to construction the new footings.



6.3.5.2 Footing Set-Back from Rock Faces

The abutment footings will be situated (perched) on the bedrock above the Shawanaga River. The footings must be maintained an adequate distance away from the edge of the final rock face (i.e. existing and maintained face or new cut face) and all rock faces should be adequately cleaned and/or protected such that the integrity of the rock face/founding rock is maintained. In this regard, the abutment footings 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 (0.5H:1V) from the toe of the rock face and not closer than about 2 m from the nearest rock slope crest. If the layout does not allow for this footing set-back, a NSSP should be included in the Contract Documents for vertical rock dowels to be installed behind the crest of the rock face (prior to any new rock excavation, where applicable) in order to provide additional support to the rock face during blasting and following construction. An example is provided in Appendix C.

6.3.5.3 Sequence of Rock Excavation

At the south abutment, up to about 1.5 m of bedrock excavation is required to establish the proposed footing founding level. In order to maintain the integrity of the new rock face (and reduce the risk of blast damage from the front slope excavation affecting the founding level for the footing), it is recommended that all bedrock excavation should be carried out in accordance with OPSS.PROV 206 (*Grading*), as amended by SP 206F04 (*Rock Excavation, Grading*). Additional recommendations for excavation of the bedrock and sequence of construction are provided in Section 6.9.

6.4 Seismic Considerations

6.4.1 Site Coefficient

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

6.4.2 Seismic Analysis Coefficient

According to the National Building Code of Canada (1995) seismic hazard values (as referenced in the CHBDC and its Commentary), the site specific peak horizontal ground acceleration for the Parry Sound area is 0.051 (for a probability of exceedance of 10 per cent in 50 years). For the thicknesses and type of overburden soils at the site, an amplification factor of 1.0 of the ground motion is recommended for design. As such, the ground surface acceleration would be about 0.05.

Given the proposed bridge structure is not designated as a lifeline or truss bridge, and in accordance with Section 4.4.5.3 of the CHBDC, seismic analysis is not required for this structure.

6.5 Lateral Earth Pressures

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,



FOUNDATION REPORT – SERVICE ROAD (SITE NO. 9) BRIDGE STRUCTURE – HIGHWAY 69 G.W.P. 5111-07-00

and the drainage conditions behind the walls. As discussed in Section 6.4.2, seismic (earthquake) loading need not be analyzed for this bridge structure.

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 OPSS.PROV 1010 (*Aggregates*) Granular 'A' or Granular 'B' Type II but with less than 5 percent passing the 200 sieve should be used as backfill behind the walls. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill in accordance with OPSD 3102.100 (*Walls, Abutment, Backfill Drain*) and OPSD 3190.100 (*Walls, Retaining and Abutment, Wall Drain*). Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with Special Provision (SP) 105S21 (*Water Requirements*) and OPSS 501 (*Compacting*). Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3101.150 (*Walls, Abutment, Backfill, Minimum Granular Requirement*) and OPSD 3121.150 (*Walls, Retaining, Backfill, Minimum Granular Requirement*).
- 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 specification as outlined in the Northeastern Region Directive (2002) for backfill of structures adjacent to rock embankments. Other aspects of rock backfill requirements should be in accordance with OPSD 3101.200 (*Walls, Abutment, Backfill, Rock*). The following parameters (unfactored) may be used for rock backfill:

Fill Type	Soil Unit Weight (kN/m ³)	Coefficients of Static Lateral Earth Pressure	
		At-Rest, K _o	Active, K _a
Rock Fill	19	0.36	0.22

- 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 walls, granular fill should be placed either in a zone with the 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 walls, 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 footing (in accordance with Figure C6.20(b) of the *Commentary* to the CHBDC). 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:

Fill Type	Soil Unit Weight (kN/m ³)	Coefficients of Static Lateral Earth Pressure	
		At-Rest, K _o	Active, K _a
Granular 'A'	22	0.43	0.27
Granular 'B' Type II	21	0.43	0.27
Rock Fill	19	0.36	0.22



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 required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.9.1 and Table C6.6 of the *Commentary* to the *CHBDC*.

6.6 Approach Embankment Design

Based on the GA Drawing provided by MRC, the proposed road grade for the new south approach and north approach embankments will be at about Elevation 208 m.

Within the limits of the proposed south approach embankment, the existing ground surface varies from about Elevation 206.4 m to Elevation 205.5 m. Therefore the construction of the Shawanaga River Service Road (Site No. 9) bridge structure will require placement of up to about 2.5 m of new fill within the limits of the south approach.

Within the limits of the proposed north approach embankment, the existing ground surface varies from about Elevation 205.5 m in the area of the old roadway to about Elevation 208 m in the area of the exposed bedrock outcrop. The rock outcrop is a near vertical rock face located at about the centre of the north abutment, extending in a north-westerly direction and intersecting the western limit of the approach embankment at a distance of about 10 m north of the north abutment. Given this, up to about 2.5 m of new embankment fill will be required to construct only a portion of the north approach (i.e. within a zone that is about 5 m wide at the abutment, tapering down to no fill at about 10 m north of the abutment). The remainder of the approach area will be rock cut. The ground surface at the top of the rock face cut is at about Elevation 208 m at the north abutment and rises to Elevation 211.6 m at the northern extent of the north approach embankment. Therefore, up to about 3.6 m of the bedrock excavation (or more to accommodate the pavement structure thickness) will be required within the majority of the north approach embankment area.

Based on the borehole results, the embankment subgrade soils at the south approach will consist of generally compact sand fill underlain by either sand and gravel fill or rock fill which in turn is underlain by syenite/granite gneiss bedrock. At the north approach embankment within the portion of the embankment where fill is required, it is anticipated that the subgrade soils will consist of compact to dense sand and gravel fill. Considering that the existing embankment fill has been in place since the construction of the original (pre-1957) alignment for the old Highway 69 and that the fills generally have a compact relative density, it is considered suitable as a subgrade for the new approach fills and may remain in place.

All topsoil and organic matter should be stripped from below the approach embankment areas, and all subgrade soils should be proof-rolled prior to fill placement.

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



6.6.1 Stability

Analyses were performed on the critical (i.e. highest or thickest fill) sections of the proposed new approach embankments to assess the stability for the proposed heights and geometries. Critical sections include those through the side slopes of the new approaches.

6.6.1.1 Methodology

Limit equilibrium slope stability analyses were performed using the commercially available program Slide (version 6), produced by Rocscience Inc., employing the Morgenstern Price method of analysis. For all analyses, the factors of safety of numerous potential failure surfaces were 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 in 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 proposed embankment heights and geometries. In general, circular slip surfaces were analysed in the design.

6.6.1.2 Parameter Selection

The soils encountered below the proposed approach embankments consist primarily of granular fill soils underlain by bedrock.

For the loose to dense granular fill, the effective stress parameters employed in the analysis were estimated from empirical correlations based on the results of the in situ Standard Penetration Tests (SPT). The correlations proposed by Peck et al. (1974) and U.S. Navy (1986) were employed and the results were tempered by engineering judgement based on precedent experience in similar soils. Considering the variable nature of the fill, the values selected for the parameters were generally based on the lower relative density.

At all areas, the analyses assume that any organic soils have been removed prior to construction of the new embankments. The groundwater level measured in the piezometer installation in Borehole B3-03 at the south abutment was at Elevation 201.5 m on June 11, 2008, which is within the bedrock and slightly above the adjacent river water level. Based on this, it is assumed that the groundwater level is located below the overburden/fill materials in the stability analysis and that hydrostatic conditions do not apply.

The following summarizes the simplified stratigraphy and the associated strengths and unit weights employed for the different soil types in both of the approach embankment areas. It is understood that rock fill is to be used for construction of the new approach embankments since granular fill, which would have to be constructed at a flatter side slope, will not fit the existing geometric constraints within the approach areas. As such, only rock fill has been utilized in the analyses as indicated below. Approach embankment side slopes have been defined as shown on the cross-sections provided by MRC (i.e. 1.25H:1V for new rock fill and matching current side slopes for existing fills). A discussion on different fill types, with respect to stability, is provided in Section 6.6.1.4.



FOUNDATION REPORT – SERVICE ROAD (SITE NO. 9) BRIDGE STRUCTURE – HIGHWAY 69 G.W.P. 5111-07-00

Approach Embankment	Soil Type	Unit Weight (kN/m ³)	Undrained Shear Strength (kPa)	Angle of Internal Friction, ϕ' (degrees)
South Approach	New Rock Fill	19	--	40
	Existing Loose to Dense Sand to Sand and Gravel Fill	19	--	32
	Existing Rock Fill	19	--	40
North Approach	New Rock Fill	19	--	40
	Existing Loose to Dense Sand to Sand and Gravel Fill	19	--	32

6.6.1.3 Results of Analysis

The results of the stability analyses for the new rock fill embankments below and are shown on Figure 1 and Figure 2 for the south and north approach embankments, respectively. The minimum factor of safety is based on a deep-seated, global trial failure surface that would impact the operation of the roadway (i.e. typically extending between about 0.5 m to 1 m behind the crest of the new fill embankment).

Location	Embankment Height at Critical Section (m)	Rock Fill Option	
		Recommended Side Slope Profile	Minimum Factor of Safety
South Approach	2.5	1.25H : 1V	≥ 1.3
North Approach	2.5		

6.6.1.4 Embankment Fill Types

Different embankment fill alternatives (i.e. granular fill and rock fill) provide relative advantages and disadvantages in terms of weight (i.e. driving force and applied load to founding subsoils/bedrock), construction cost and time, and ease of construction/availability. A brief description of each alternative is described below.

Granular Fill

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



Rock Fill

The main advantage of using rock fill is the ability to achieve steeper embankment side slopes. This is useful in areas with limited right-of-ways, such as between the east side of the south abutment and the Shawanaga River at this site and areas where rock fill would likely be available locally. The disadvantage of using rock fill for the construction of high embankments is that some post-construction settlement of the embankment fill itself will occur within about the first year of construction.

6.6.2 Settlement

Some relatively small settlement of the approach embankments can be expected as a result of the loading from the new fills on the granular foundation soils at this site. In addition, for the proposed rock fill materials to be employed in the construction of the embankments, some settlement will also occur due to compression of the new embankment rock fill itself. The majority of rock fill settlement will occur within the first year following construction, but some settlement will occur over the life of the embankment.

6.6.2.1 Methodology

To estimate the magnitude of the expected settlements, analyses were carried out on the critical sections of the proposed approach embankments using the commercially available program Settle^{3D} (version 2) produced by Rocscience Inc.

For the settlement analyses, the critical sections were assessed considering the location of the following at each approach area:

- The greatest new embankment height; and/or,
- The thickest overburden deposit.

6.6.2.2 Parameter Selection

At both approaches, the foundation soils are composed primarily of compact to dense sand to sand and gravel fill and/or rock fill underlain by bedrock.

The immediate compression of the granular 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 unit weights and slope profiles for the embankment fill are as described in Section 6.6.1.2. The analyses performed assume that all topsoil and surficial organic soils will be removed prior to construction and that rock fill will be used for the new embankment construction.

The piezometric conditions required in the analyses were based on the groundwater levels noted during drilling and measured in the piezometer installation. Since the groundwater level measured within the standpipe piezometer was within the bedrock, it is assumed that the groundwater level is located below the overburden/fill materials and therefore no pore water pressures were utilized in the settlement analysis.



FOUNDATION REPORT – SERVICE ROAD (SITE NO. 9) BRIDGE STRUCTURE – HIGHWAY 69 G.W.P. 5111-07-00

The following summarize the simplified stratigraphy, unit weights and deformation parameters employed for the different soils types in the approach areas. For the purpose of analysis, rock fill has been considered for the construction of the approach embankments as indicated below. Rock fill is assumed to have side slopes at 1.25H:1V. A discussion on the different embankment fill types, with respect to stability, is provided in Section 6.6.1.4, however, it is understood that rock fill is to be used for the construction of the approach embankments.

Approach Embankment	Soil	Thickness (m)	Unit Weight (kN/m ³)	Estimated Deformation Properties
South Approach STA 12+405	New Rock Fill	2.5	19	Refer to Section 6.6.2.4
	Existing Loose to Dense Sand to Sand and Gravel Fill	Up to 1.4	19	E' = 20 MPa
	Existing Rock Fill	Up to 3.1	19	E' = 15 MPa
North Approach STA 12+466	New Rock Fill	2.5	19	Refer to Section 6.6.2.4
	Existing Loose to Dense Sand and Gravel Fill	Up to 2.4	19	E' = 20 MPa

The settlement analyses have been carried out at the critical section of the north and south approaches (i.e. the thickest overburden deposits and highest new embankment fills) as well as assuming the lowest relative density of the existing fills. At the south approach embankment, the conditions at Borehole B3-03 are considered the most critical case as the thickness of fill is greatest at this location. At the north embankment the soil conditions at Borehole B3-11 are considered the most critical as the thickness of fill is greatest at this location.

6.6.2.3 Settlement of Foundation Soils

The results of the estimated settlement of the foundation soils the south and north approach embankments are summarized below.

Approach Embankment	Maximum New Embankment Height ¹ (m)	Estimated Settlement of Foundation Soils (mm)
South Approach STA 12+405	2.5	15
North Approach STA 12+466	2.5	15

Provided that the topsoil and any surficial organics are removed prior to the new embankment fill placement, settlements of the new approach embankments, due to compression of the foundation soils, are expected to be relatively small and occur rapidly during construction (i.e. during or shortly after construction) in response to filling.



6.6.2.4 Settlement of New Embankment Fill

Granular Fill

If granular material is employed for the construction of the new approach embankments at this site, very little additional settlement due to compression of the embankment fill itself will occur beyond the estimated settlement of the foundation soils (as described above). In this case, the additional settlement from properly compacted granular fills is expected to be less than about 25 mm and will occur during construction. It is recommended that the fines content of the granular fill used for embankment construction be minimized to avoid long-term settlement and maintenance issues.

Rock Fill

It is understood that rock fill is to be used for the construction of the new embankment and as such, there will be settlement due to compression of the rock fill itself under self weight, in addition to the settlement of the underlying foundation soil deposits as described in Section 6.6.2.3. The magnitude of settlement of the rock fill depends on the type of rock/strength of particles, size and shape of rock particles, gradation of rock fill, total height/thickness of fill and the method of construction and sequence of placement. Rock fill should be placed, in a controlled manner (i.e. not end dumped) in accordance with OPSS.PROV 206 (*Grading*), as amended by SP 206F04 (*Rock Excavation, Grading*). In accordance with the “MTO Guideline for Rock Fill Settlement and Rock Fill Quantity Estimates” dated September 2010, settlement of rock fill placed in this manner is expected to be nominal and for the fill thickness at this location, the magnitude is estimated to be up to about 0.5 percent of the effective height of the rock fill embankment. The estimated short-term settlement of rock fill, for the up to 2.5 m high approach embankments is presented below.

Approach Embankment	Maximum New Embankment Height (m)	Estimated Settlement of Rock Fill (mm)
South Approach STA 12+405	2.5	15
North Approach STA 12+466	2.5	15

About 90 percent of the short-term settlement of the rock fill estimated above is expected to occur within six (6) months following construction to the full height of the embankment and the remaining 10 percent (i.e. less than about 5 mm) is expected to occur in the following (6) months.

In addition, the expected long-term post-construction settlement of the embankment rock fill is as follows.

Approach Embankment	Maximum New Embankment Height (m)	Estimated Long-Term Settlement of Rock Fill (mm)
South Approach STA 12+405	2.5	5
North Approach STA 12+466	2.5	5



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.

Given the above, the estimated settlement of the rock fill embankment is within the settlement performance criterion of 25 mm of settlement over a 20-year period following the completion of construction in accordance with Section 1.2 of MTO's "Embankment Settlement Criteria for Design" dated July 2010.

6.6.2.5 *Embankment Platform Widening*

In accordance with the requirements of MTO "Northern Region Engineering Directive NRE 98-200", the construction of the embankments should include an allowance for platform widening (in 0.5 m increments) to accommodate settlements during construction as well as post-construction settlements, so that the minimum standard shoulder widths are maintained if future grade raises on the embankments are required. According to NRE 98-200, the need for future raises in road grade could occur due to settlement/compression of the embankment fill, settlement of the foundation soils and to accommodate future pavement overlays up to 200 mm thick. It is understood that this directive applies to all rock fill embankments as well as for granular fill embankments where widening restrictions are present (i.e. due to space/property issues, presence of a sensitive body of water and so on). It is further understood that on non-major highways and roadways (i.e. ramps and side roads), the minimum required platform widening over swamp crossings is 1 m per side.

The minimum required embankment platform widening (per embankment side) is calculated based on the estimated consolidation settlement of the foundation soils (including creep) and long-term settlement/compression of the embankment fill plus an additional 200 mm for the future pavement overlay, multiplied by the horizontal component of the side slope of the pavement structure (4H:1V), but cannot be less than the minimum platform widening requirements as described above.

As a result, the minimum required embankment widening at this site to account for the estimated settlements and for future pavement overlays is 1 m per embankment side.

6.7 Subgrade Preparation and Embankment Construction

The existing overburden/fills are considered to be appropriate subbase for the proposed approach embankments, however, prior to the placement of any fill, any surface and near surface layer of topsoil/asphalt/organic deposits and any softened or loosened soil should be stripped from the plan limits of the proposed works and the subgrade soils should be proof-rolled. The following sections provide details on the recommendations for subgrade preparation and embankment construction.

6.7.1 Removal of Organic Materials

Based on the information from the boreholes obtained during the field investigation, surficial topsoil of up to 0.1 m thick can be expected in some areas of the new approach embankments. This surficial topsoil layer, where encountered, should be stripped from the plan limits of the approach areas prior to fill placement.



6.7.2 Embankment Fill Placement

Where the existing/old approach embankments are composed of rock fill (such as those along the east slope at the proposed new south approach), any loose or deleterious material should be removed from the toe and slopes prior to any new fill placement.

Placement of rock fill material should be carried out in accordance with the requirements as outlined in OPSS.PROV 206 (*Grading*). The rock should not be dumped in final position, but should be deposited on and pushed forward over the end of the layer being constructed. Blading, dozing and ‘chinking’ the rock to form a dense, compact mass will be required to minimize voids and bridging. Side slopes for rock fill embankments should be no steeper than 1.25 horizontal to 1 vertical (1.25H:1V).

The incorporation of 2 m wide berms (or successive benches) into the uniform side slope profile (i.e. slope flattening) is required wherever rock fill embankments exceed a height of 10 m such that the uninterrupted rock fill slope does not exceed a height of 10 m as per OPSD 202.010 (*Slope Flattening*). Given that the maximum height of the rock fill approach embankment is less than 10 m, the incorporation of a mid-height berm (or successive benches) is not required at this site. Vegetation cover should be established on all soil slopes to protect embankment fill against surficial erosion.

6.8 Design and Construction Consideration

6.8.1 Overburden Excavation

Based on the currently proposed elevations of the underside of the abutment footings and the recommendation to remove all overburden/fills down to the top of bedrock prior to mass concrete/footing construction, excavation for the bridge foundations will extend to depths of up to about 3.7 m below existing ground surface and will be primarily through compact to dense sand to sand and gravel fill and rock fill. The groundwater level as measured in a standpipe piezometer at the south abutment was located within the bedrock at a depth of about 4 m below ground surface.

These fill soils/rock fill are considered Type 3 soils according to Occupational Health and Safety Act and Regulation for Construction Projects (OHSA). The excavation through the overburden should be carried out with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V).

All excavations must be carried out in accordance with Ontario Regulation 213 Ontario Occupational Health and Safety Act for Construction Projects (as amended by Ontario Regulation 443).

6.8.2 Control of Groundwater and Surface Water

The groundwater level as measured in a standpipe piezometer at the south abutment was below the surface of the bedrock at about Elevation 201.6 m (about 4 m below existing ground surface). At the north and south abutments, it is anticipated that excavations to expose the bedrock surface for spread footing construction will be above the groundwater level.

Surface water should be directed away from the excavation at all times. It is anticipated that removal of any surface water that may seep into the excavations can be handled by pumping from well filtered sumps located along the perimeter of the excavation. Where required during construction, sedimentation control measures



such as silt fencing should be installed in accordance with OPSS 805 (*Temporary Erosion and Sediment Control Measures*).

6.8.3 Obstructions

The sand and gravel fill and rock fill at the site contain cobble and boulder sized materials as encountered during the drilling. Conventional excavation equipment should be suitable for the majority of the excavation through the overburden soils. However, the presence of boulders may interfere with or slow the progress of stripping and excavation. It is recommended that a Non-Standard Special Provision (NSSP), be included in the Contract Document to warn the Contractor of these obstructions and to ensure that the Contractor is equipped to handle such obstructions; an example NSSP is included in Appendix C.

6.8.4 Existing Concrete Abutments

As noted in Section 6.3.5.1, the existing north and south concrete abutments from the old Highway 69 bridge (pre-1957) are in close proximity to the new proposed abutments. Although concrete was not encountered in any of the core samples obtained from the boreholes advanced at the site, it is recommended that if concrete is encountered within the footprint of the proposed abutments during excavation, it should be removed prior to construction of the new footings.

6.8.5 Permit To Take Water

Considering that the groundwater level (as measured in the piezometer) is located below the surface of the bedrock in the vicinity of the abutments, we do not anticipate any significant groundwater flows into the excavations during abutment footing construction and as such, a permit to take water is not anticipated to be required for this site.

6.9 Recommendations for Rock Excavations and Blasting

6.9.1 Rock Excavation

It should be noted that the bedrock is generally classified as strong (R4) to very strong (R5) (i.e. the Uniaxial Compressive Strength (UCS) is in the range of about 70 MPa to 110 MPa) and the level of fracturing in the upper portions of the bedrock is variable. This will make excavation potentially difficult particularly in areas where only small depths and narrow zones of rock removal are needed. Bedrock excavation in the vicinity of the proposed structure foundations should be carried out in accordance with OPSS.PROV 206 (*Grading*), as amended by SP 206F04 (*Rock Excavation, Grading*), by wall control blasting techniques such as line drilling and pre-shearing to minimize blast damage to the rock (i.e. shattering and over-break) and provide better control over the configuration of the founding surface.

As discussed in Section 6.3.5.2, the abutment footings will be founded on the bedrock outcrops above the Shawanaga River. As such, an adequate set-back distance between the footing and the edge of the final rock face (i.e. existing and maintained face or new cut face) must be provided. If the layout does not allow for the footing set-back recommended in Section 6.3.5.2, a NSSP should be provided for vertical rock dowels to be



installed behind the crest of the rock face (prior to any new rock excavation) in order to provide additional support to the rock face during blasting and following construction. All rock faces should be adequately cleaned and/or protected such that the integrity of the rock face/founding rock is maintained.

6.9.2 Blasting

The use of explosives should follow the specifications outlined in OPSS 120 (*Use of Explosives*). It is recommended that control of all blasting operations, including removal of all loose, unstable rock from the cut faces, be carried out in accordance with OPSS.PROV 206 (*Grading*).

It is recommended that all new rock cut faces in the area of the proposed structure foundations be inspected by a QVE soon after blasting to assess if the blasting operations have affected the integrity of the rock mass that will ultimately be supporting the new abutment footings.

7.0 CLOSURE

This report was prepared by Ms. Sandra McGaghran, M.Eng., P. Eng., and Ms. T. Veronica Ayetan, P.Eng., with inputs provided by Mr. Christopher Ng, P. Eng. The technical aspects were reviewed by Mr. J. Paul Dittrich, Ph.D., P. Eng., a senior geotechnical engineer and Principal with Golder. Mr. Jorge M. A. Costa, P. Eng., the Designated MTO Contact for this project and Principal with Golder, conducted an independent quality control review of the report.



Report Signature Page

Sandra McGaghran, M. Eng., P. Eng.,
Geotechnical Engineer, Associate



J. Paul Ditttrich, Ph.D., P. Eng.,
Senior Geotechnical Engineer, Principal



Jorge M. A. Costa, P. Eng.,
Designated MTO Contact, Principal

TVA/SMM/CN/JPD/JMAC/tva/jl

\\golder.gds\gal\mississauga\active\2007\1111\07-1111-0029 - mrc - hwy 69 four-laning -report\final\4 - site 9 road bridge structure (shawanaga river)\07-1111-0029-4 fidr 15jul03 highway 69 - site 9 road bridge.docx



REFERENCES

- 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, 4th Edition. The Canadian Geotechnical Society c/o BiTech Publisher Ltd., British Columbia.
- 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.
- Geology of Ontario. 1991. Ontario Geological Society, Special Volume 4, Part 2. Eds. P.C. Thurston, H.R. Williams, R.H. Sutcliffe and G.M. Stott. Ministry of Northern Development and Mines, Ontario.
- 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.
- Peck, R.B., Hanson, W.E., and Thornburn, T.H. 1974. Foundation Engineering, Second Edition, John Wiley and Sons, New York.
- Unified Facilities Criteria, U.S. Navy. 1986. NAVFAC Design Manual 7.02. Soil Mechanics, Foundation and Earth Structures. Alexandria, Virginia.

STANDARDS:

ASTM International:

ASTM D1586	Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils.
ASTM D5731	Standard Test Method for Determination of the Point Load Strength Index of Rock and Application to Rock Strength Classifications.
ASTM D7102	Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures.

Contract Design Estimating and Documentation (CDED):

- Special Provision 105S21 Amendment to OPSS 501 – Compacting.
- Special Provision 206F04 Rock Excavation, Grading.

Commercial Software:

- Slide (Version 6.0) by Rocscience Inc.

Ministry of Transportation Ontario:

- MTO Guideline for Rock Fill Settlement and Rock Fill Quantity Estimates. September 2010.
- Embankment Settlement Criteria for Design. July 2010.
- Northern Region Engineering Directive NRE 98-200. Northern Region Embankment Design Guidelines. October 1998.



FOUNDATION REPORT – SERVICE ROAD (SITE NO. 9) BRIDGE STRUCTURE – HIGHWAY 69 G.W.P. 5111-07-00

Northeastern Region Engineering Directive. Backfill to Structures Adjacent to Rock Embankment Approaches. November 2002.

Ontario Occupational Health and Safety Act:

Ontario Regulation 213/91 Construction Projects.

Ontario Regulation 443/09 Amendment to Ontario Regulation 213.

Ontario Provisional Standard Drawing:

OPSD 202.010	Slope Flattening Using Surplus Excavated Material on Earth or Rock Embankments.
OPSD 3090.101	Foundation, Frost Penetration Depths for Southern Ontario.
OPSD 3101.150	Walls, Abutment, Backfill, Minimum Granular Requirement.
OPSD 3101.200	Walls, Abutment, Backfill, Rock.
OPSD 3102.100	Walls, Abutment, Backfill, Drain.
OPSD 3121.150	Walls, Retaining, Backfill, Minimum Granular Requirement.
OPSD 3190.100	Walls, Retaining and Abutment, Wall Drain.

Ontario Provincial Standard Specification:

OPSS 120	General Specification for Use of Explosives.
OPSS 501	Construction Specification for Compacting.
OPSS 805	Construction Specification for Temporary Erosion and Sediment Control Measures
OPSS 902	Construction Specification for Excavating and Backfilling.
OPSS.PROV 206	Construction Specification for Grading.
OPSS.PROV 904	Construction Specification for Concrete Structures.
OPSS.PROV 1010	Material Specification for Aggregates.

Ontario Water Resources Act:

Ontario Regulation 903 Wells.



TABLES



FOUNDATION REPORT – SERVICE ROAD (SITE NO. 9) BRIDGE STRUCTURE – HIGHWAY 69 G.W.P. 5111-07-00

TABLE 1
EVALUATION OF FOUNDATION ALTERNATIVES – SHAWANAGA RIVER SERVICE ROAD (SITE NO. 9) BRIDGE STRUCTURE
G.W.P. 5111-07-00 / W.P. 5180-06-01

Option	Rank/ Option	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Spread Footings on Mass Concrete and/or Bedrock	1	<ul style="list-style-type: none"> Relative ease of construction. Reduced bedrock excavation (as compared with pile option) Negligible post-construction settlement. 	<ul style="list-style-type: none"> Requires removal (blasting) of approximately 1.5 m of bedrock at the south abutment and up to 4.5 m at the north abutment to reach the proposed founding level. Variable bedrock surface will require soil excavation followed by mass concrete to achieve level footing. Bedrock will have to be blasted using controlled blasting techniques to minimize shattering and overbreak. 	<ul style="list-style-type: none"> Lower relative cost than piled foundation option. Additional cost associated with excavation of shallow bedrock and mass concrete. Additional costs for vertical dowels, if required, to reinforce the rock face in front of the proposed south abutment. 	<ul style="list-style-type: none"> If bedrock surface elevation is higher than anticipated, additional bedrock excavation is required. Variability in bedrock surface will impact mass concrete quantities and excavation depths. Must take measures to ensure integrity of rock face below/in front of the footings or 'dental' concreting/repair may be required during construction.
Spread Footings on Granular 'A' Pads	Not Recommended	<ul style="list-style-type: none"> Relative ease of construction Small post-construction settlement. 	<ul style="list-style-type: none"> Subexcavation of an additional 1 m into bedrock to allow for placement of a minimum 1 m thick Granular 'A' pad otherwise stress concentrations will occur on the footing. Special design considerations required to ensure no loss of material from the granular pad over the life of the structure. 	<ul style="list-style-type: none"> Lower relative cost than piled foundations. Lower relative cost than mass concrete. Additional cost associated with subexcavation of an additional 1 m into bedrock. Additional costs for vertical dowels, if required, to reinforce the rock face in front of the proposed south abutment. 	<ul style="list-style-type: none"> Moderate risk of undermining of the abutment in the long term if the granular pad erodes or sloughs away. Inspection and future maintenance may be required.



FOUNDATION REPORT – SERVICE ROAD (SITE NO. 9) BRIDGE STRUCTURE – HIGHWAY 69 G.W.P. 5111-07-00

TABLE 1
EVALUATION OF FOUNDATION ALTERNATIVES – SHAWANAGA RIVER SERVICE ROAD (SITE NO. 9) BRIDGE STRUCTURE
G.W.P. 5111-07-00 / W.P. 5180-06-01

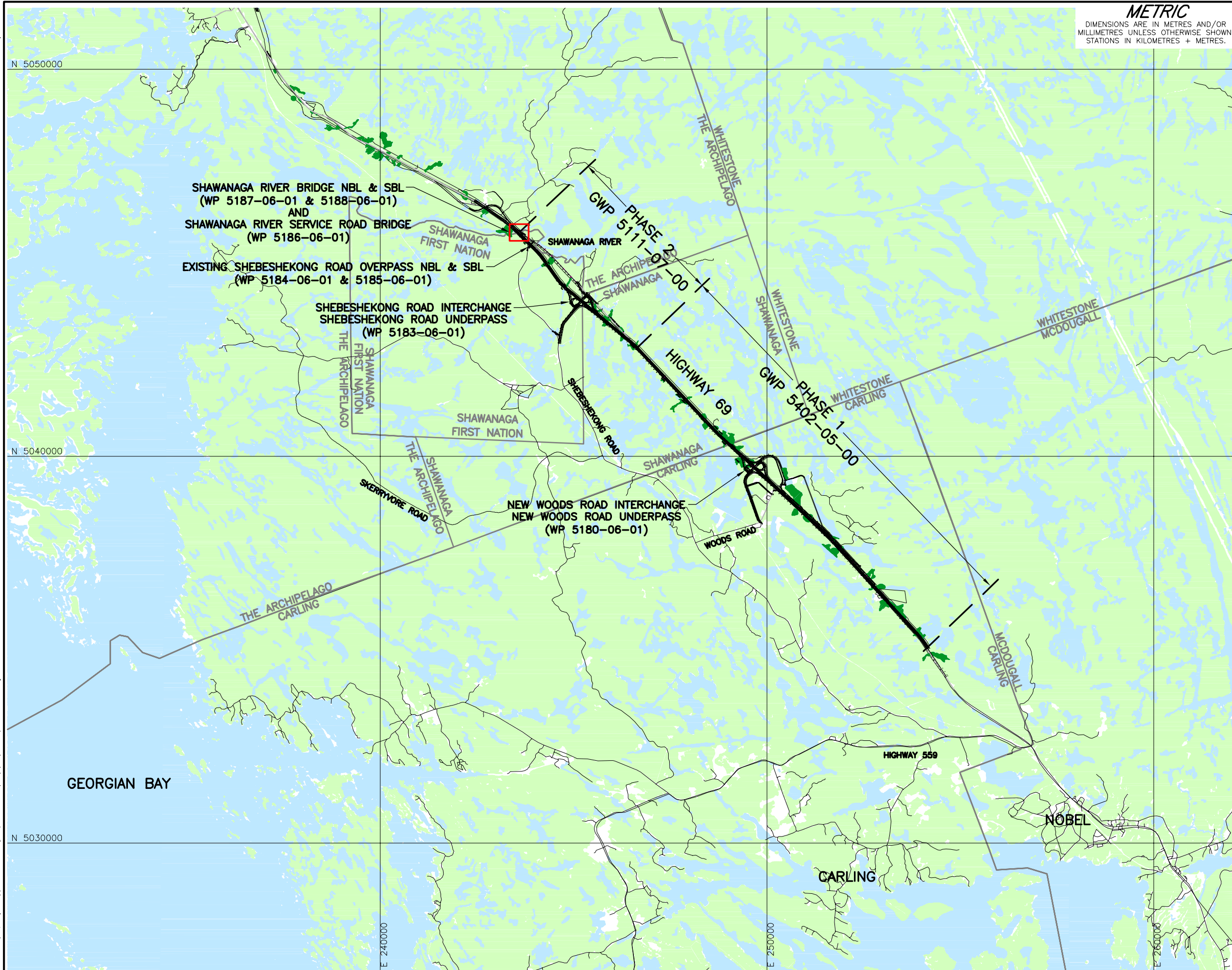
Option	Rank/ Option	Advantages	Disadvantages	Relative Costs	Risks/Consequences
H-piles in Bedrock Trenches	Not Recommended	<ul style="list-style-type: none">Allows for integral abutment designNegligible post-construction settlement	<ul style="list-style-type: none">Bedrock excavation to form trench will be required to achieve minimum required pile lengths.	<ul style="list-style-type: none">Higher relative cost than spread footings due to significant costs for excavating trench in bedrock.	<ul style="list-style-type: none">Not recommended due to significant depth of excavation required in strong to very strong bedrock.

Prepared By: CN/SMM

Reviewed By: JPD/JMAC




DRAWINGS



PLAN



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

CONT No. WP No. 5186-06-01	
HIGHWAY 69 SITE LOCATION PLAN	SHEET

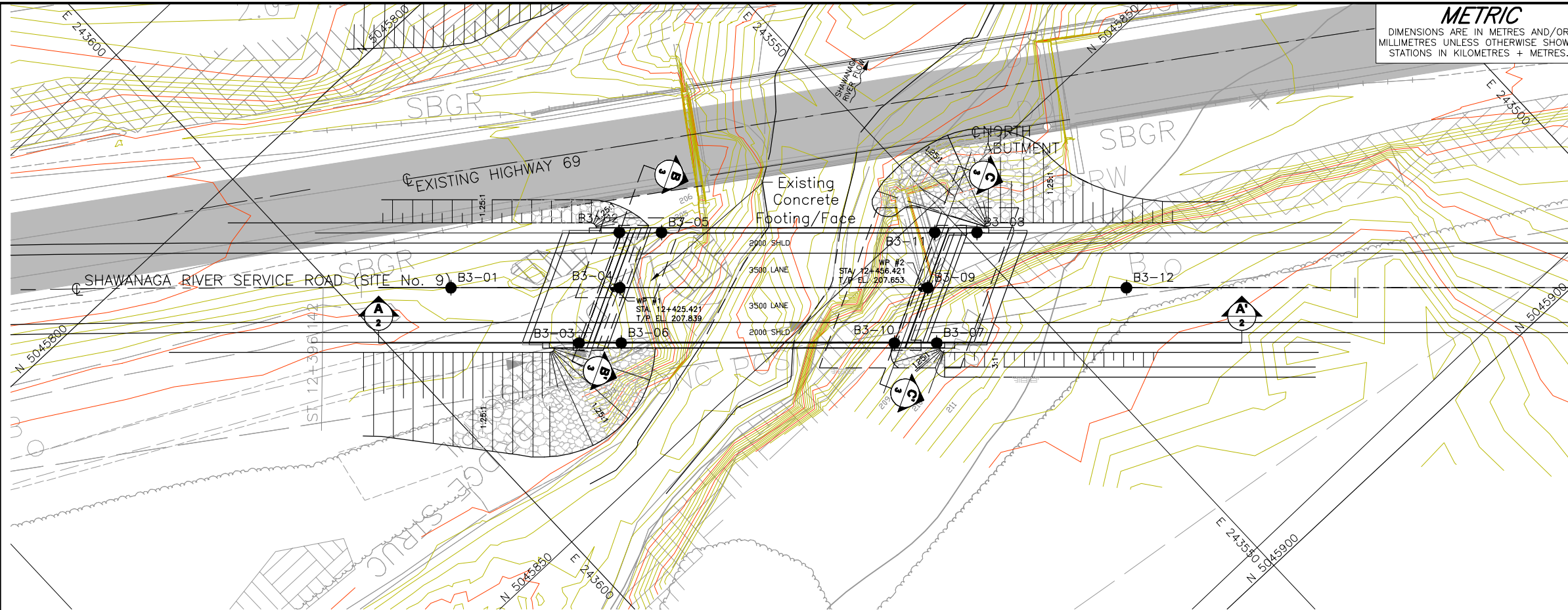


Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

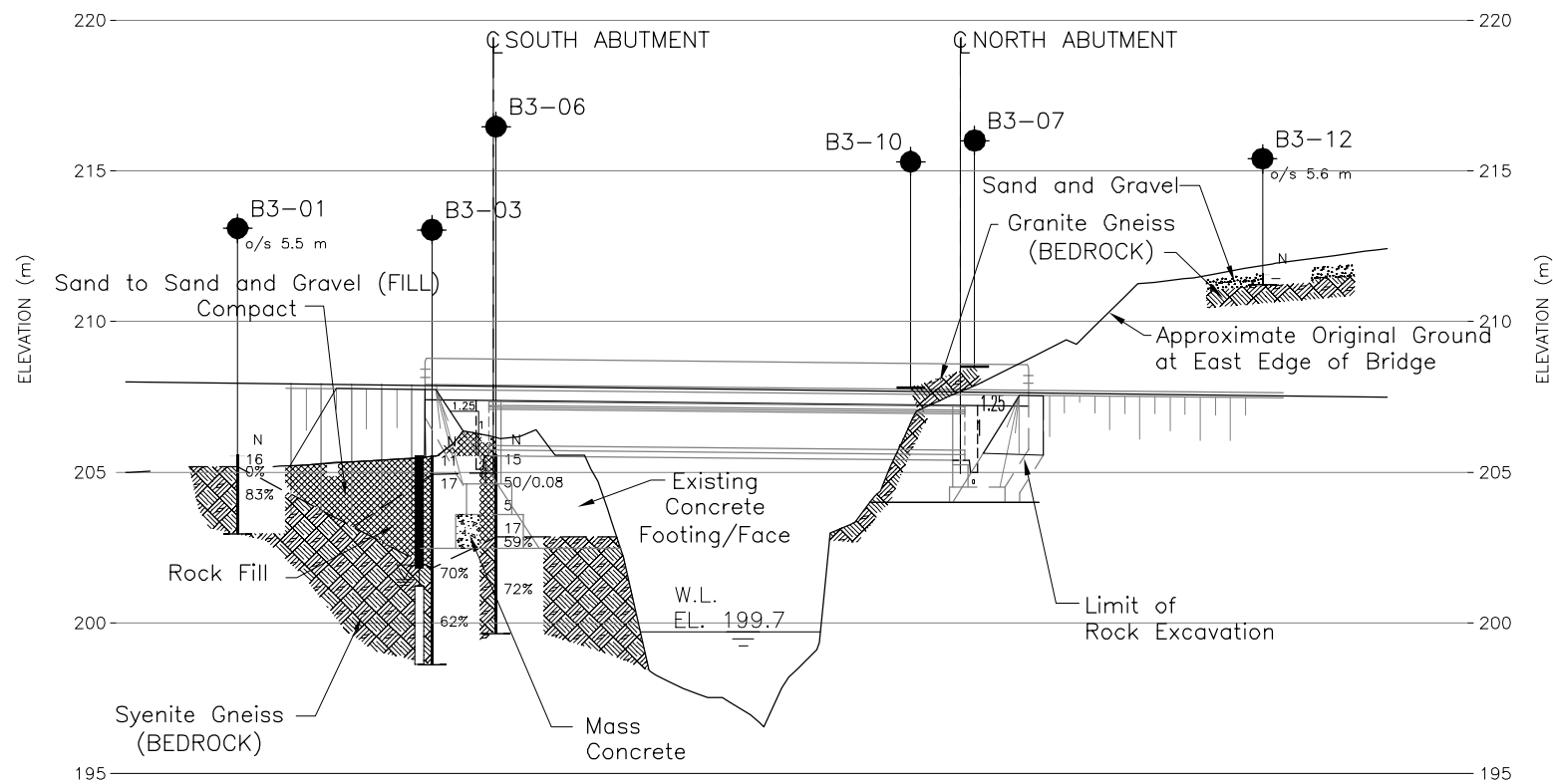


KEY PLAN
NOT TO SCALE

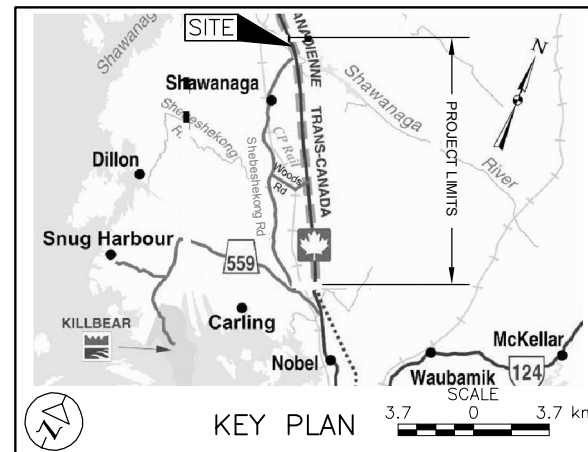
REFERENCE				
Base Data - MNR NRVS, obtained 2004, CANMAP v2006.4 Produced by Golder Associates Ltd under licence from Ontario Ministry of Natural Resources, © Queens Printer 2008 Datum : NAD 83 Projection : MTM Zone 10				
Geocres No. 41H-143				
NO.	DATE	BY	REVISION	
HWY. 69	PROJECT NO. 07-1111-0029		DIST.	
SUBM'D. VA	CHKD. VA	DATE: Jul. 2015	SITE: 44-65	
DRAWN: DD/CD	CHKD. CN	APPD. JPD/JMAC	DWG. 1	



PLAN

SCALE
5 0 5 10 mA-A'
2EAST EDGE OF BRIDGE
HIGHWAY 69 - SHAWANAGA RIVER SERVICE ROAD (SITE No. 9)HORIZONTAL SCALE
5 0 5 10 m
VERTICAL SCALE
2.5 0 2.5 5 m**METRIC**
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.CONT No.
WP No. 5186-06-01HIGHWAY 69
SHAWANAGA RIVER SERVICE ROAD (SITE No. 9)
BOREHOLE LOCATION
AND SOIL STRATA

SHEET

**Golder Associates Ltd.**
MISSISSAUGA, ONTARIO, CANADA

KEY PLAN

LEGEND

- Borehole - Current Investigation
- ⊕ Dynamic Cone Penetration Test
- ⊥ Seal
- ⊥ Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated
(Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- ≡ WL in piezometer, measured on June 11, 2008
- ≡ WL upon completion of drilling
- R Refusal

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
B3-01	205.6	5045823.0	243591.4
B3-02	206.4	5045830.5	243575.2
B3-03	205.5	5045835.9	243585.8
B3-04	205.9	5045834.6	243579.0
B3-05	205.1	5045833.4	243572.1
B3-06	205.6	5045838.8	243582.7
B3-07	208.5	5045860.4	243559.5
B3-08	205.6	5045855.1	243548.9
B3-09	207.9	5045855.8	243556.3
B3-10	207.8	5045857.5	243562.6
B3-11	205.6	5045852.2	243552.0
B3-12	211.6	5045869.4	243541.7

NOTES

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

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

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



REFERENCE

Base plans provided in digital format by MRC, drawing file 5271XB01.DWG, 5271-XPB-ARCHIPELAGO.dwg, 5271-XPB-Carling.dwg, 5271-XPB-SHAWANAGA.dwg, PR # 5377-02-00-PR-1.dwg, received October 1, 2007, and h6878xb1.dwg, h6878xb07 Phase-2 contours 1m intervals.dwg, S6878-315-001GA.dwg, received September 10, 2014.

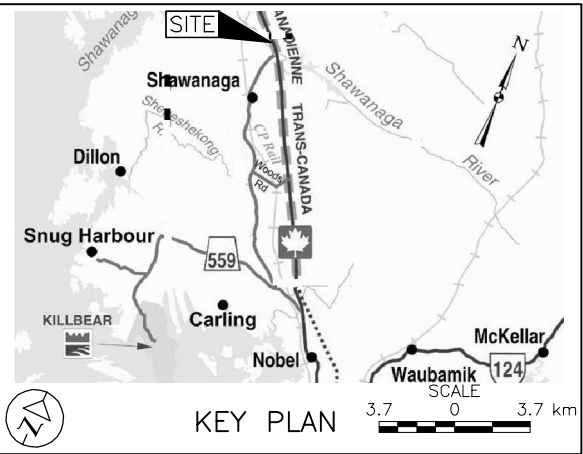
NO.	DATE	BY	REVISION
Geocres No. 41H-143			
HWY. 69	PROJECT NO. 07-1111-0029		DIST.
SUBM'D. VA	CHKD. CN	DATE: JUL. 2015	SITE: 44-65
DRAWN: JFC/RJ	CHKD. SMM	APPD. JPD	DWG. 2

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

CONT No.
WP No. 5186-06-01

HIGHWAY 69
SHAWANAGA RIVER SERVICE ROAD (SITE No. 9)
SOIL STRATA

**Golder Associates Ltd.**
MISSISSAUGA, ONTARIO, CANADA



LEGEND

- Borehole - Current Investigation
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- WL in piezometer, measured on June 11, 2008
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
B3-02	206.4	5045830.5	243575.2
B3-03	205.5	5045835.9	243585.8
B3-04	205.9	5045834.6	243579.0
B3-09	207.9	5045855.8	243556.3
B3-10	207.8	5045857.5	243562.6
B3-11	205.6	5045852.2	243552.0

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.

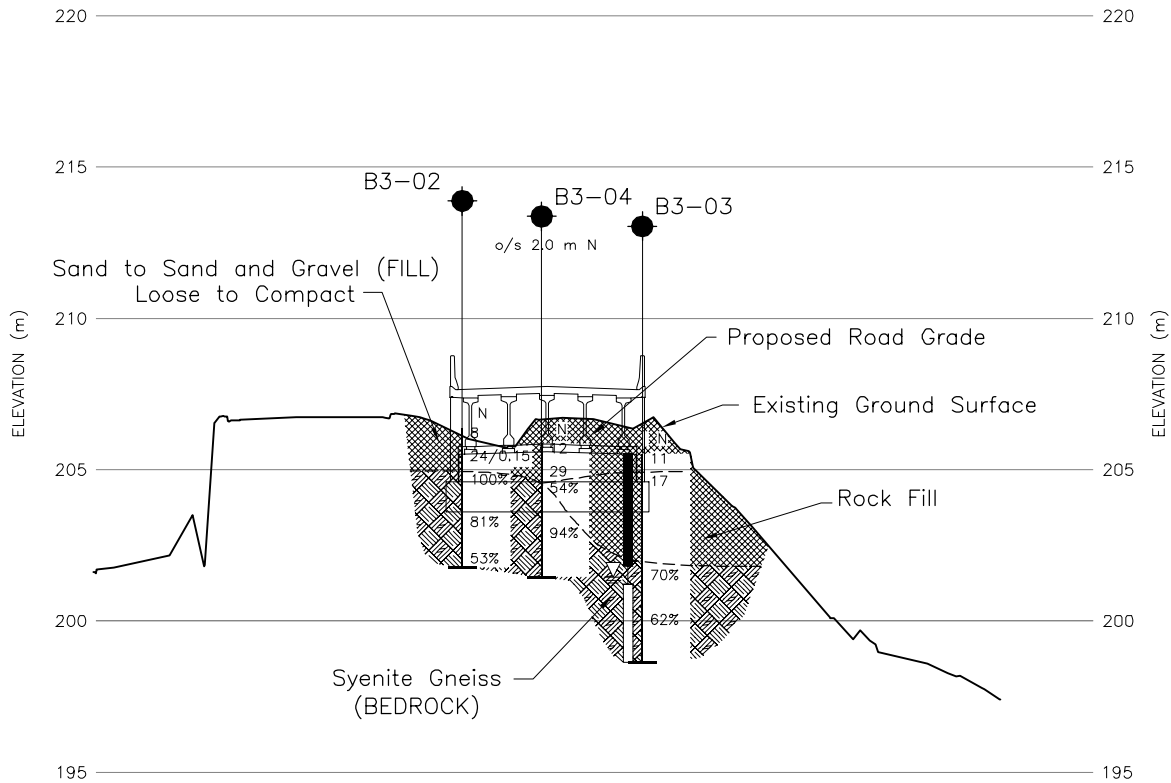
The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS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

REFERENCE

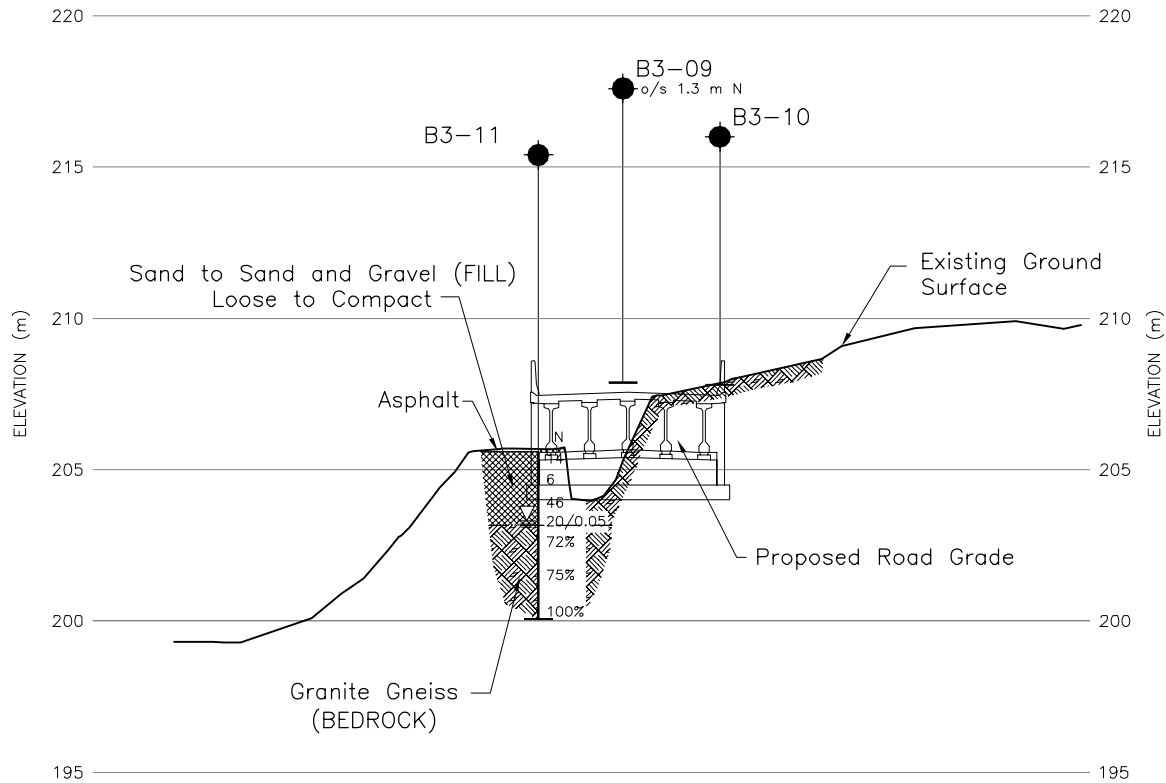
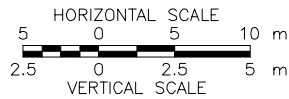
Base plans provided in digital format by MRC, drawing file 5271XB01.DWG, 5271-XPB-ARCHIPELAGO.dwg, 5271-XPB-Carling.dwg, 5271-XPB-SHAWANAGA.dwg, PR # 5377-02-00-PR-1.dwg, received October 1, 2007, and S6878-315-001GA_1mSOUTH.dwg, received July 27, 2009

Existing ground surface provided in digital format by MMM, drawing file 6878 jh XS at Site 9 Abut for Fnd-4.dwg received September 16, 2014.

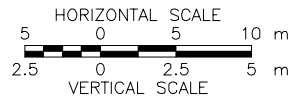
Proposed bridge and abutments provided in digital format by MMM, drawing file Shawanaga Sections_Sept 26 2014.dwg, received September 26, 2014.



B-B'
2 SOUTH ABUTMENT
HIGHWAY 69 - SHAWANAGA RIVER SERVICE ROAD (SITE No. 9)



C-C'
2 NORTH ABUTMENT
HIGHWAY 69 - SHAWANAGA RIVER SERVICE ROAD (SITE No. 9)



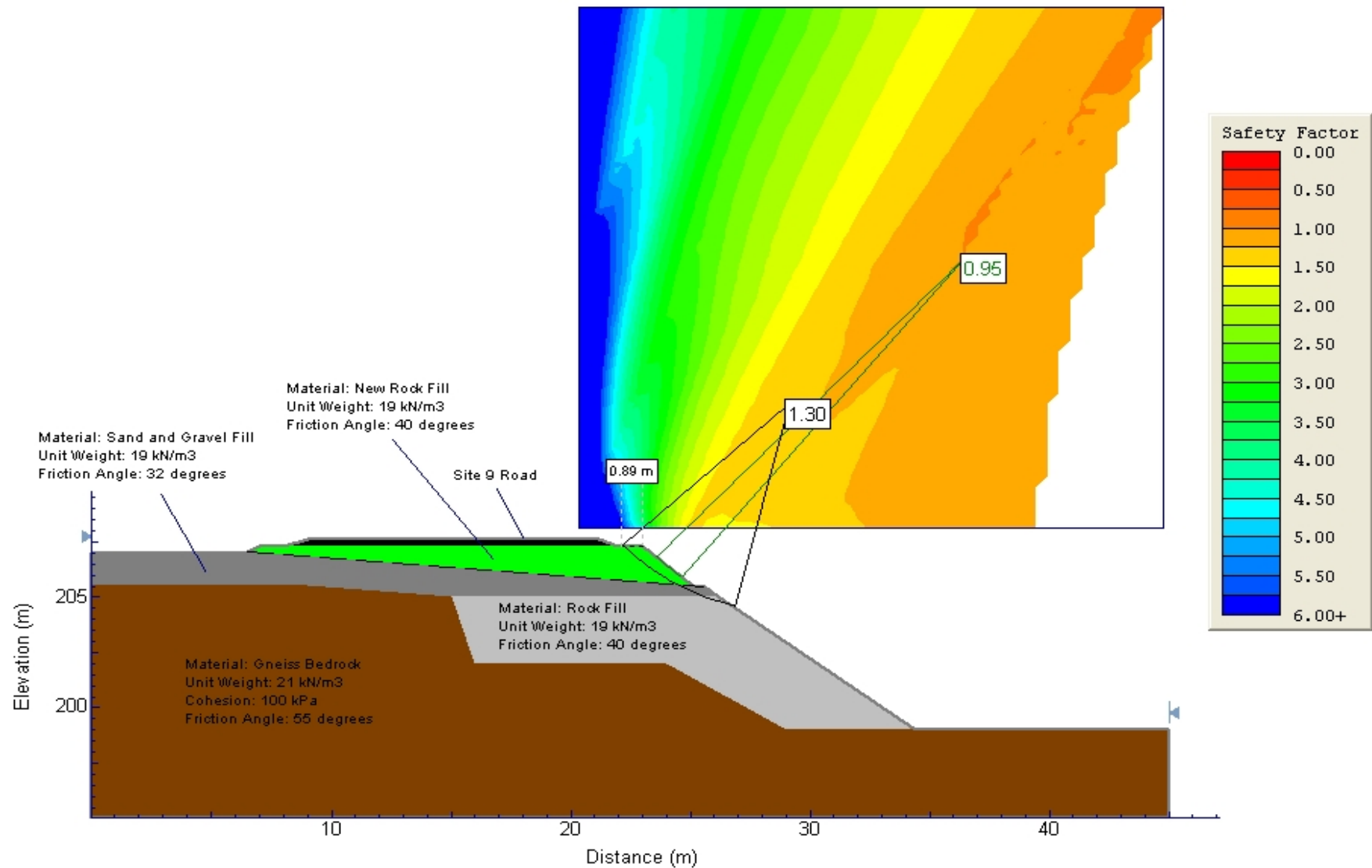
NO.	DATE	BY	REVISION
Geocres No. 41H-143			
HWY. 69	PROJECT NO. 07-1111-0029		DIST.
SUBM'D. CR	CHKD. CN	DATE: JUL. 2015	SITE: 44-65
DRAWN: JFC/RJ	CHKD. SMM	APPD. JPD	DWG. 3



FIGURES

Stability Analysis Results South Approach - STA 12+419

Figure 1



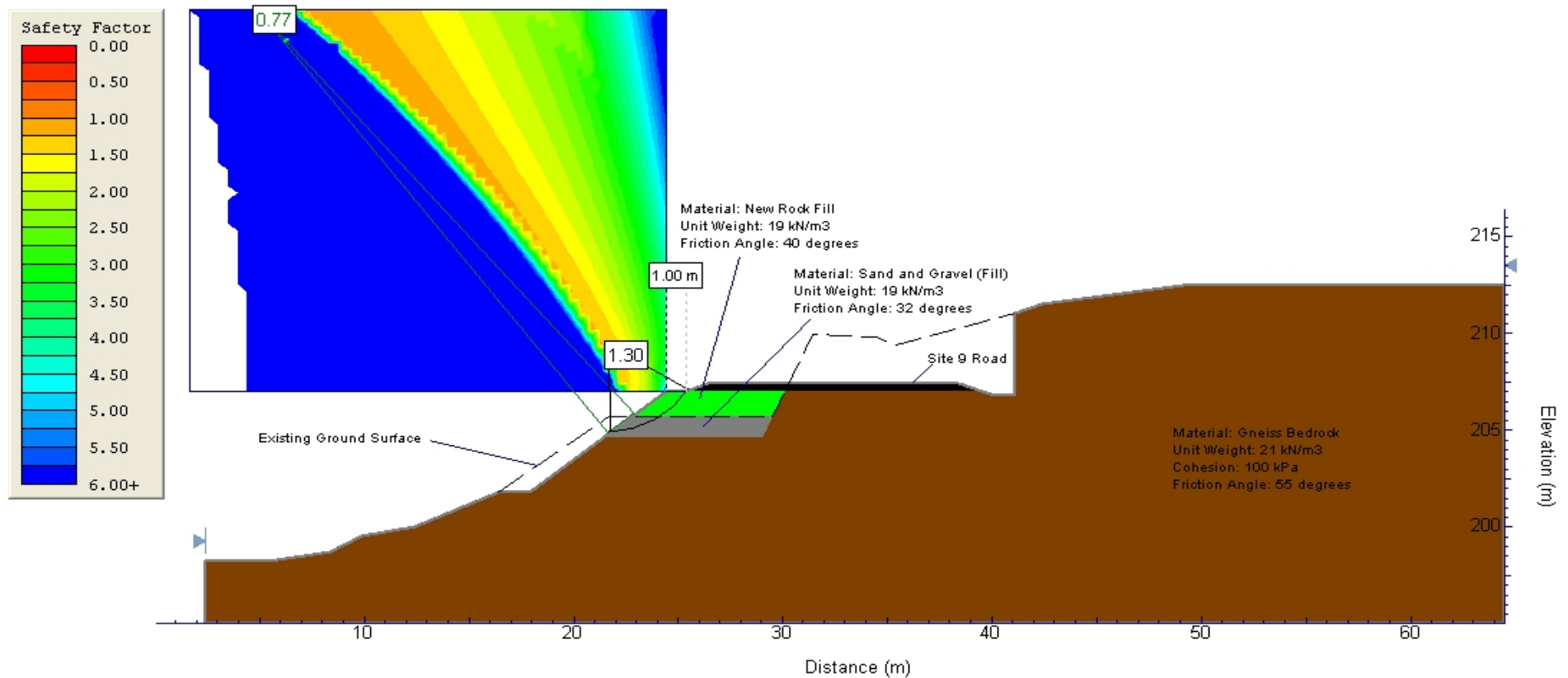
Date: September 2009
Project: 07-1111-0029-4

Golder Associates

Drawn: TB
Checked: SMM

Stability Analysis Results
North Approach - STA 12+463

Figure 2





APPENDIX A

RECORD OF BOREHOLES AND DRILLHOLES



LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta \sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
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 or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	plasticity index = $(w_l - w_p)$
w_s	shrinkage limit
I_L	liquidity index = $(w - w_p) / I_p$
I_C	consistency index = $(w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index = $(e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

Notes: 1
2

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



LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance; N_d :

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

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

(b) Cohesive Soils Consistency

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

IV. SOIL TESTS

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

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

V. MINOR SOIL CONSTITUENTS

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



LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERINGS STATE

Fresh: no visible sign of weathering

Faintly weathered: weathering limited to the surface of major discontinuities.

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

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

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

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

BEDDING THICKNESS

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

JOINT OR FOLIATION SPACING

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

GRAIN SIZE

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

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

CORE CONDITION

Total Core Recovery (TCR)

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

Solid Core Recovery (SCR)

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

Rock Quality Designation (RQD)

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

DISCONTINUITY DATA

Fracture Index

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

Dip with Respect to Core Axis

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

Description and Notes

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

Abbreviations

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

PROJECT		RECORD OF BOREHOLE		No B3-01		SHEET 1 OF 1		METRIC									
W.P. 5111-07-00		LOCATION		N 5045823.0 ; E 243591.4		ORIGINATED BY		CR									
DIST		HWY 69		BOREHOLE TYPE		NW Casing, Wash Boring		COMPILED BY									
DATUM		Geodetic		DATE		October 27, 2008		CHECKED BY									
								SMM									
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									
205.6	GROUND SURFACE																
0.0	TOPSOIL		1	SS	16												
205.2	Sand and gravel, some silt, containing organics, rootlets and topsoil (FILL)		1	RC	REC 100%												RQD = 0%
0.4	Compact Brown Moist Syenite Gneiss (BEDROCK)																
	Bedrock cored from depths of 0.4 m to 2.6 m.		2	RC	REC 100%												RQD = 83%
203.0	For bedrock coring details, refer to Record of Drillhole B3-01.																
2.6	END OF BOREHOLE																
	NOTE: 1. Open borehole dry prior to coring.																

GTA-MTO 001 T:\PROJECTS\2007\07-1111-0029 (MRC, PARRY SOUND)\LOG\07-1111-0029-SERVICE ROAD #9-PHASE III.GPJ GAL-GTA.GDT 07/09/15 DD/SAC

PROJECT: 07-1111-0029

RECORD OF DRILLHOLE: B3-01

SHEET 1 OF 1

LOCATION: N 5045823.0 ;E 243591.4

DRILLING DATE:

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: D-50

DRILLING CONTRACTOR: Walker Drilling

DEPTH SCALE METRES	DRILLING RECORD		DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN	JN - Joint FLT - Fault SH - 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 VR - Very Rough MB - Mechanical Break BR - Broken Rock NOTE: For additional abbreviations refer to list of abbreviations & symbols.														NOTES	
								RECOVERY			FRACT. INDEX PER 0.3 m	DISCONTINUITY DATA						HYDRAULIC CONDUCTIVITY K, cm/sec			Diameter 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					
																			FLUSH	FLUSH			
1	NW Casing October 27, 2008		SYENITE GNEISS Slightly weathered to fresh, fine to coarse grained, foliated, black, white, and pink, contains pink veins and porphyr		205.18 0.41	1	0.2	Brown to light brown 90 - 80															

DEPTH SCALE

1 : 50



LOGGED: CR

CHECKED: SMM

GTA-RCK 018 T:\PROJECTS\2007\07-1111-0029 (MRC, PARRY SOUND)\LOG\07-1111-0029-SERVICE ROAD #9-PHASE III.GPJ GAL-MISS.GDT 07/09/15 DD/SAC

PROJECT		RECORD OF BOREHOLE		No B3-02		SHEET 1 OF 1		METRIC								
W.P. 5111-07-00		LOCATION		N 5045830.5 ; E 243575.2		ORIGINATED BY		CR								
DIST		HWY 69		BOREHOLE TYPE		NW Casing, Wash Boring		COMPILED BY								
MWK/VA		DATE		October 22, 2008		CHECKED BY		SMM								
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								
206.4	GROUND SURFACE															
0.0	Sand, some gravel, some silt, containing rootlets and topsoil (FILL)		1A	SS	8											OC = 2.1%
0.3	Loose Brown Moist		1B													
			2	SS	24/0.15											60 34 5 1
205.0	Sand and gravel, trace silt, trace clay, containing cobbles/boulders and rootlets (FILL)															
1.4	Loose to compact Brown Moist															
	Syenite Gneiss (BEDROCK)		1	RC	REC 100%											RQD = 100%
	Bedrock cored from depths of 1.4 m to 4.6 m.															
	For bedrock coring details, refer to Record of Drillhole B3-02.		2	RC	REC 100%											RQD = 81%
			3	RC	REC 95%											RQD = 53%
201.8	END OF BOREHOLE															
4.6	NOTE: 1. Open borehole dry prior to coring.															

GTA-MTO 001 T:\PROJECTS\2007\07-1111-0029 (MRC, PARRY SOUND)\LOG\07-1111-0029-SERVICE ROAD #9-PHASE III.GPJ GAL-GTA.GDT 07/09/15 DD/SAC

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

PROJECT: 07-1111-0029

RECORD OF DRILLHOLE: B3-03

SHEET 1 OF 1

LOCATION: N 5045835.9 ;E 243585.8


DRILLING DATE:

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: D-50

DRILLING CONTRACTOR: Walker Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR FLUSH % RETURN	JN - Joint FLT - Fault SH - 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 VR - Very Rough MB - Mechanical Break BR - Broken Rock NOTE: For additional abbreviations refer to list of abbreviations & symbols.																NOTES																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																			
							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																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																								
							80-90	90-100																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																		
							80-90	90-100																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																		
4	NW (Facing) October 24, 2008	Continued from Record of Borehole B3-03 SYENITE GNEISS Slightly weathered to fresh, fine to coarse grained, foliated, black, white, and pink, contains pink feldspar veins and porphyr		201.81 3.73	1	Brown to light brown 80 - 90																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																				

DEPTH SCALE

1 : 50



LOGGED: CR

CHECKED: SMM

GTA-RCK 018 T:\PROJECTS\2007\07-1111-0029 (MRC, PARRY SOUND)\LOG\07-1111-0029-SERVICE ROAD #9-PHASE III.GPJ GAL-MISS.GDT 07/09/15 DD/SAC

PROJECT		RECORD OF BOREHOLE		No B3-04		SHEET 1 OF 1		METRIC																					
W.P. 07-1111-0029		LOCATION		N 5045834.6 ; E 243579.0		ORIGINATED BY		CR																					
DIST		HWY 69		BOREHOLE TYPE		NW Casing, Wash Boring		COMPILED BY																					
MWK/VA		DATE		October 23, 2008		CHECKED BY		SMM																					
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS			ELEVATION SCALE			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																								
205.9	GROUND SURFACE																												
0.0	Gravelly sand, trace to some silt, trace clay, containing asphalt fragments and oxidation zones (FILL)		1	SS	12																								
205.2	Compact Brown Moist		2	SS	29																								
0.7																													
204.6	Sand, some gravel, some silt, containing rock fragments, cobbles and rootlets (FILL)		1	RC	REC 97%																								
1.3	Compact Brown Moist																												
	Syenite Gneiss (BEDROCK)																												
	Bedrock cored from depths of 1.3 m to 4.4 m.																												
	For bedrock coring details, refer to Record of Drillhole B3-04.		2	RC	REC 100%																								
201.5	END OF BOREHOLE																												
4.4	NOTE: 1. Open borehole dry prior to coring.																												

GTA-MTO 001 T:\PROJECTS\2007\07-1111-0029 (MRC, PARRY SOUND)\LOG\07-1111-0029-SERVICE ROAD #9-PHASE III.GPJ GAL-GTA.GDT 07/09/15 DD/SAC

SHEET 1 OF 1

DATUM: Geodetic

DRILLING CONTRACTOR: Walker Drilling



T:\PROJECTS\2007\07-1111-0029 (MRC, PARRY SOUND)\LOG\07-1111-0029-SERVICE ROAD #9-PHASE III.GPJ GAL-MISS.GDT 07/09/15 DD/SAC

CHECKED: SMM



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

GTA-MTO 001 T:\PROJECTS\2007\07-1111-0029 (MRC, PARRY SOUND)\LOG\07-1111-0029-SERVICE ROAD #9-PHASE III.GPJ GAL-GTA.GDT 07/09/15 DD/SAC

PROJECT 07-1111-0029		RECORD OF BOREHOLE No B3-06				SHEET 1 OF 1		METRIC									
W.P. 5111-07-00		LOCATION N 5045838.8 ; E 243582.7				ORIGINATED BY CR											
DIST _____ HWY 69		BOREHOLE TYPE NW Casing, Wash Boring				COMPILED BY MWK/VA											
DATUM Geodetic		DATE October 23, 2008				CHECKED BY SMM											
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									
205.6	GROUND SURFACE							20	40	60	80	100					
0.0	Sand, some gravel, some silt, containing asphalt fragments and organics (FILL)		1	SS	15												OC = 2.4%
205.0	Compact Brown Moist ROCK FILL		2	SS	50/0.08												
0.6			3	SS	5												
			4	SS	17												
202.9	Syenite Gneiss (BEDROCK)																
2.7	Bedrock cored from depths of 2.7 m to 5.9 m. For bedrock coring details, refer to Record of Drillhole B3-06.		1	RC	REC 100%												RQD = 59%
			2	RC	REC 100%												RQD = 72%
199.7	END OF BOREHOLE																
5.9	NOTE: 1. Open borehole dry prior to coring.																

GTA-MTO 001 T:\PROJECTS\2007\07-1111-0029 (MRC, PARRY SOUND)\LOG\07-1111-0029-SERVICE ROAD #9-PHASE III.GPJ GAL-GTA.GDT 07/09/15 DD/SAC

PROJECT: 07-1111-0029

RECORD OF DRILLHOLE: B3-06

SHEET 1 OF 1

LOCATION: N 5045838.8 ;E 243582.7

DRILLING DATE:

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: D-50

DRILLING CONTRACTOR: Walker Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR FLUSH % RETURN	JN - Joint FLT - Fault SH - 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 VR - Very Rough MB - Mechanical Break BR - Broken Rock NOTE: For additional abbreviations refer to list of abbreviations & symbols.														NOTES																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																												
							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																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													
							80 80 80 80	80 80 80 80																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																									
3	NW Casing October 23, 2008	Continued from Record of Borehole B3-06 SYENITE GNEISS Slightly weathered to fresh, fine to coarse grained, foliated, black, white, and pink, contains pink veins and porphyr		202.85 2.74	1	0.4	Brown to light brown 80 - 90																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																										

DEPTH SCALE

1 : 50



LOGGED: CR

CHECKED: SMM



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

GTA-MTO 001 T:\PROJECTS\2007\07-1111-0029 (MRC, PARRY SOUND)\LOG\07-1111-0029-SERVICE ROAD #9-PHASE III.GPJ GAL-GTA.GDT 07/09/15 DD/SAC

PROJECT		RECORD OF BOREHOLE		No B3-08		SHEET 1 OF 1		METRIC									
W.P. 07-1111-0029		LOCATION		N 5045855.1 ; E 243548.9		ORIGINATED BY		CR									
DIST		HWY 69		BOREHOLE TYPE		NW Casing, Wash Boring		COMPILED BY									
MWK/VA		DATE		October 22, 2008		CHECKED BY		SMM									
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									
205.6	GROUND SURFACE							20	40	60	80	100					
8.9	TOPSOIL		1	SS	21												43 46 9 2
204.6	Sand and gravel, trace to some silt, trace clay, containing cobbles and boulders (FILL) Compact to dense Brown Moist		2	SS	40/0.10												39 46 12 3
1.0	Granite Gneiss (BEDROCK)		1	RC	REC 100%												RQD = 50%
	Bedrock cored from depths of 1.0 m to 3.7 m.																
	For bedrock coring details, refer to Record of Drillhole B3-08.		2	RC	REC 100%												RQD = 89%
201.9	END OF BOREHOLE																
3.7	NOTE: 1. Open borehole dry prior to coring.																

GTA-MTO 001 T:\PROJECTS\2007\07-1111-0029 (MRC, PARRY SOUND)\LOG\07-1111-0029-SERVICE ROAD #9-PHASE III.GPJ GAL-GTA.GDT 07/09/15 DD/SAC

PROJECT: 07-1111-0029

RECORD OF DRILLHOLE: B3-08

SHEET 1 OF 1

LOCATION: N 5045855.1 ;E 243548.9

DRILLING DATE:

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: D-50

DRILLING CONTRACTOR: Walker Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR FLUSH % RETURN	JN - Joint FLT - Fault SH - 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 VR - Very Rough	MB - Mechanical Break BR - Broken Rock NOTE: For additional abbreviations refer to list of abbreviations & symbols.	NOTES
1	NW Casing October 22, 2008	Continued from Record of Borehole B3-08 GRANITE GNEISS Slightly weathered, fine grained, slightly foliated, black, white, and pink		204.99 0.61								
2	NQ RC October 22, 2008				1	Brown to light brown 90 - 100						
3					2	Brown to light brown 90 - 100						
4		END OF DRILLHOLE		201.87 3.73								
5												
6												
7												
8												
9												
10												

DEPTH SCALE

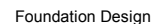
1 : 50



LOGGED: CR

CHECKED: SMM

GTA-RCK 018 T:\PROJECTS\2007\07-1111-0029 (MRC, PARRY SOUND)\LOG\07-1111-0029-SERVICE ROAD #9-PHASE III.GPJ GAL-MISS.GDT 07/09/15 DD/SAC



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

PROJECT		RECORD OF BOREHOLE		No B3-11		SHEET 1 OF 1		METRIC									
W.P. 07-1111-0029		LOCATION		N 5045852.2 ; E 243552.0		ORIGINATED BY		CR									
DIST		HWY 69		BOREHOLE TYPE		NW Casing, Wash Boring		COMPILED BY									
MWK/VA		DATE		October 21, 2008		CHECKED BY		SMM									
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									WATER CONTENT (%)
205.6	GROUND SURFACE							20	40	60	80	100					
0.7	ASPHALT																
	Sand and gravel, trace to some silt, trace clay, containing rock fragments (FILL) Loose to dense Brown Moist		1	SS	14												
			2	SS	6												
	Cobbles/boulders and oxidation zones encountered below a depth of 0.7 m		3	SS	46												
203.2			4	SS	20/0.05												
2.4	Granite Gneiss (BEDROCK)																
	Bedrock cored from depths of 2.4 m to 5.5 m. For bedrock coring details, refer to Record of Drillhole B3-11.		1	RC	REC 100%												
			2	RC	REC 100%												
			3	RC	REC 100%												
200.1	END OF BOREHOLE																
5.5	NOTE: 1. Water level in open borehole at a depth of 2.3 m below ground surface (Elev. 203.3 m) prior to coring.																

PROJECT: 07-1111-0029

RECORD OF DRILLHOLE: B3-11

SHEET 1 OF 1

LOCATION: N 5045852.2 ;E 243552.0

DRILLING DATE:

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: D-50

DRILLING CONTRACTOR: Walker Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN	FLUSH	JN - Joint FLT - Fault SH - 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 VR - Very Rough	MB - Mechanical Break BR - Broken Rock NOTE: For additional abbreviations refer to list of abbreviations & symbols.	NOTES
RECOVERY	TOTAL CORE %	SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3 m	B Angle	DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	HYDRAULIC CONDUCTIVITY K, cm/sec	Diametral Point Load Index (MPa)	RMC -Q AVG
Continued from Record of Borehole B3-11													
203.18													
2.44													
1	0.3	Brown 10 - 20											
2	0.2												
3	0.2												
200.08													
5.54													
END OF DRILLHOLE													

DEPTH SCALE

1 : 50



LOGGED: CR

CHECKED: SMM

GTA-RCK 018 T:\PROJECTS\2007\07-1111-0029 (MRC, PARRY SOUND)\LOG\07-1111-0029-SERVICE ROAD #9-PHASE III.GPJ GAL-MISS.GDT 07/09/15 DD/SAC

PROJECT		RECORD OF BOREHOLE		No B3-12		SHEET 1 OF 1		METRIC												
W.P. 5111-07-00		LOCATION		N 5045869.4 ;E 243541.7		ORIGINATED BY		CR												
DIST		HWY 69		BOREHOLE TYPE		Shovel Excavation		COMPILED BY												
VA		DATE		October 27, 2008		CHECKED BY		SMM												
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
								20	40	60	80	100	W _p	W	W _L					
211.6	GROUND SURFACE																			
0.0	TOPSOIL		1	CS	-															31 48 19 2
211.2	SAND and GRAVEL, some silt, trace clay, containing rootlets																			
0.4	Brown Moist END OF EXCAVATION BEDROCK OUTCROP																			

GTA-MTO 001 T:\PROJECTS\2007\07-1111-0029 (MRC, PARRY SOUND)\LOG\07-1111-0029-SERVICE ROAD #9-PHASE III.GPJ GAL-GTA.GDT 07/09/15 DD/SAC

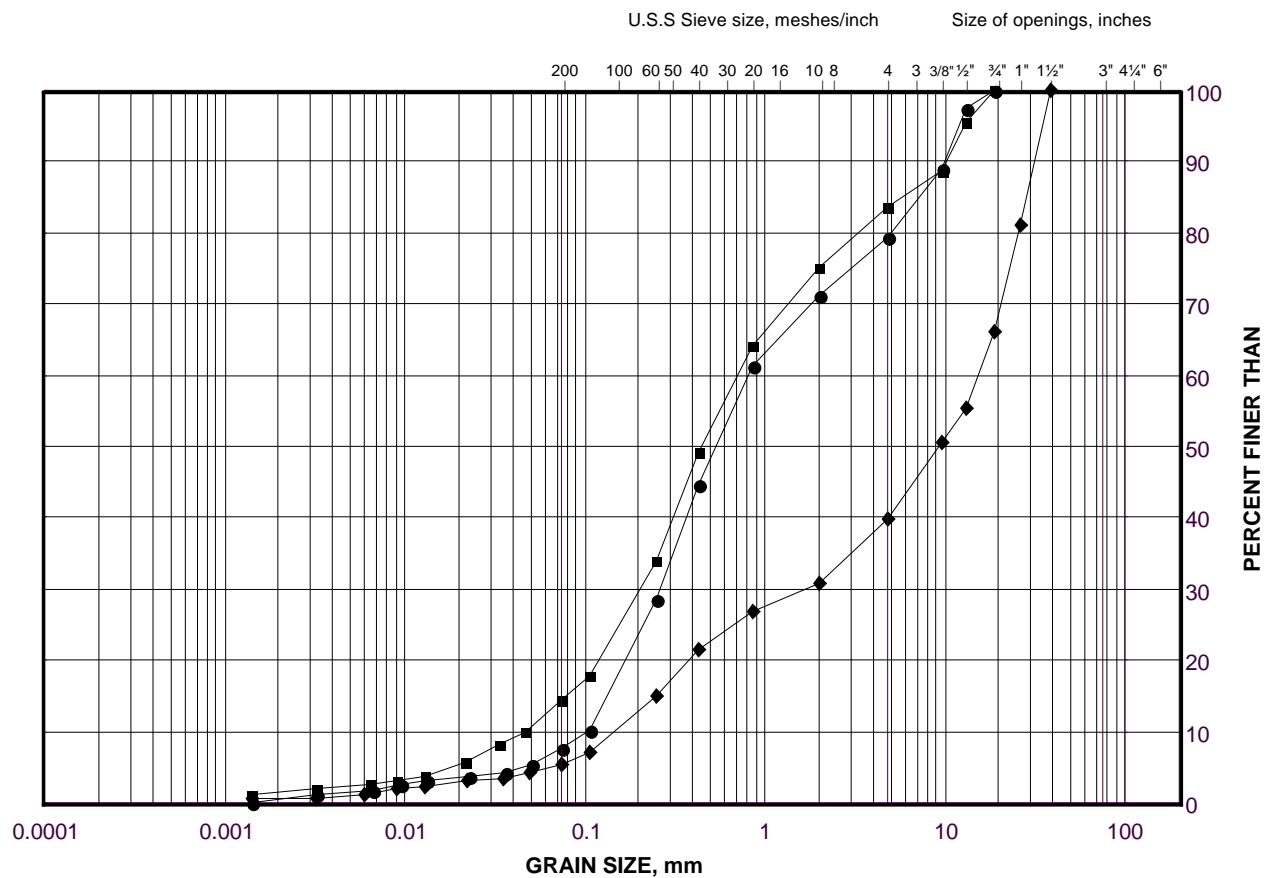


APPENDIX B

LABORATORY TEST RESULTS

Sand to Sand and Gravel Fill South Abutment

FIGURE B1-1



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	B3-04	1	205.7
■	B3-04	2	204.9
◆	B3-02	2	205.5

Project Number: 07-1111-0029

Checked By: SMM

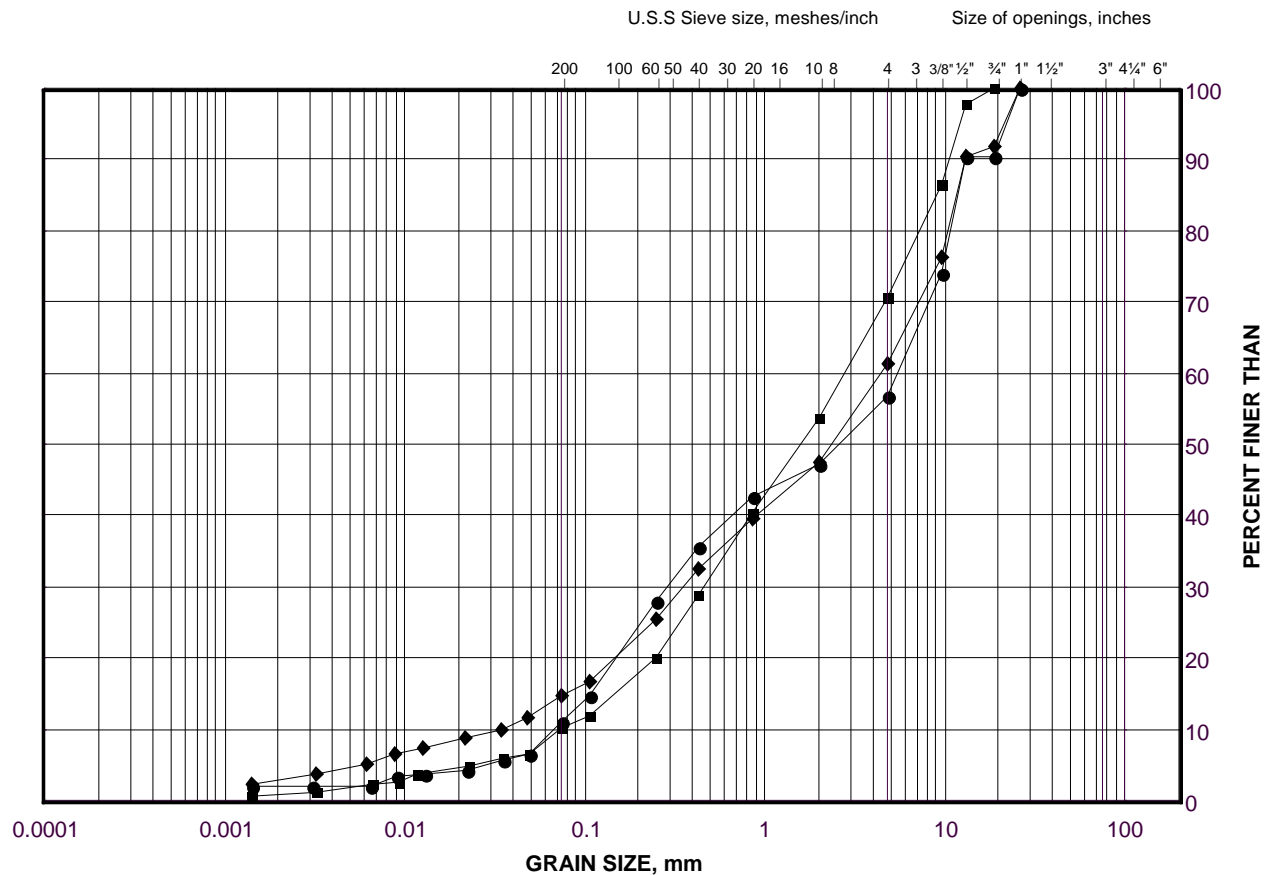
Golder Associates

Date: 27-Jul-09

GRAIN SIZE DISTRIBUTION

Sand and Gravel Fill
North Abutment

FIGURE B1-2



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	B3-08	1	205.4
■	B3-11	2	204.6
◆	B3-08	2	204.7

Project Number: 07-1111-0029

Checked By: SMM

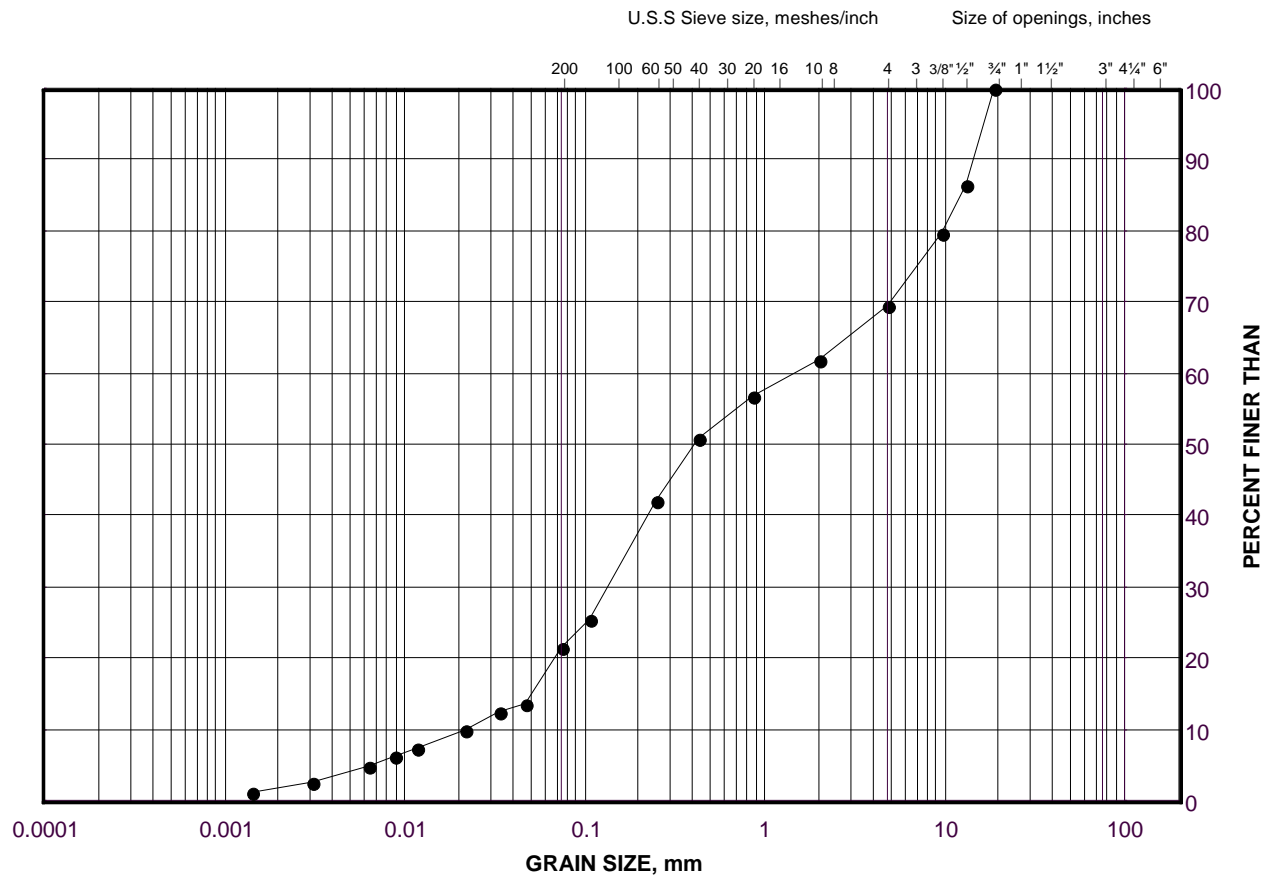
Golder Associates

Date: 21-Sep-09

GRAIN SIZE DISTRIBUTION

Sand and Gravel
North Approach

FIGURE B1-3



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
•	B3-12	1	207.7

Project Number: 07-1111-0029

Checked By: SMM

Golder Associates

Date: 21-Sep-09

TABLE B1
POINT LOAD TEST RESULTS ON ROCK SAMPLES

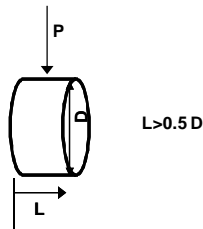
Borehole Number	Run Number	Sample Depth (m)	Sample Elevation (m)	Bedrock Description	Test Type	Is (50mm) (MPa)	Approx. ⁽¹⁾ UCS (MPa)
B3-02	1	1.5	204.9	Syenite Gneiss	Axial	6.521	91
B3-02	1	1.5	204.9	Syenite Gneiss	Diametral	7.085	99
B3-02	1	2.1	204.3	Syenite Gneiss	Axial	8.515	119
B3-06	1	3.0	202.6	Syenite Gneiss	Axial	5.624	79
B3-06	1	3.0	202.6	Syenite Gneiss	Diametral	7.720	108
B3-08	1	1.2-1.5	205.6	Granite Gneiss	Axial	0.555	8
B3-08	1	1.2-1.5	205.6	Granite Gneiss	Diametral	2.422	34
B3-08	1	2	203.6	Granite Gneiss	Axial	5.423	76
B3-08	2	2.3	203.3	Granite Gneiss	Diametral	11.373	159
B3-08	2	2.7	202.9	Granite Gneiss	Diametral	8.936	125
B3-11	1	2.5	203.1	Granite Gneiss	Diametral	6.569	92
B3-11	1	2.9	202.7	Granite Gneiss	Axial	6.938	97
B3-11	1	3.1	202.5	Granite Gneiss	Diametral	7.397	104

⁽¹⁾ $I_{s50} \times C$ (actual value could be confirmed by UCS testing), from ISRM. A value of $C = 14$ has been used and is based on correlation with UCS testing for this site.

("Suggested Methods for Determining Point Load Strength", International Society for Rock Mechanics Commission on Testing Methods, Int. J. Rock. Mech. Min. Sci. and Geomechanical Abstr., Vol 22, No. 2 1985, pp. 53-60.

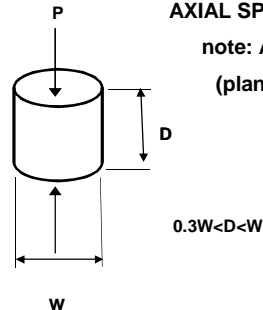
DIAMETRAL SPECIMEN SHAPE REQUIREMENTS

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



AXIAL SPECIMEN SHAPE REQUIREMENTS

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



Compiled By: TVA
 Checked By: SMM/CN
 Reviewed By: JPD/JMAC

TABLE B2-1
SUMMARY OF UNCONFINED COMPRESSION (UC) TEST RESULTS
SHAWANAGA RIVER SERVICE ROAD (SITE NO. 9)
GWP 5186-06-01
HIGHWAY 69, TOWNSHIP OF PARRY SOUND

Borehole Number	Sample Depth (m)	Sample Elevation (m)	Rock Type	Core Diameter (mm)	Uniaxial Compressive Strength (MPa)
B3-02	2.1	203.5	Syenite Gneiss	47.0	115
B3-06	3.1	202.6	Syenite Gneiss	47.1	79
B3-08	1.4	204.2	Granite Gneiss	47.5	70

Compiled by: SMM/CN

Reviewed By: JMAC

TABLE B2-2
UNCONFINED COMPRESSION (UC) TEST
ASTM D 7012-04

SAMPLE IDENTIFICATION

PROJECT NUMBER	07-1111-0029	RUN NUMBER	1
BOREHOLE NUMBER	B3-02	SAMPLE DEPTH, m	2.13
		ELEVATION, m	203.5

TEST CONDITIONS

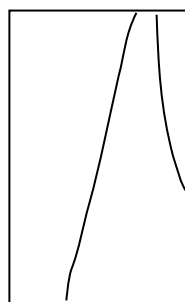
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST, min	>2 <15	L/D	2.19

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.30	WATER CONTENT, (specimen) %	0.15
SAMPLE DIAMETER, cm	4.70	UNIT WEIGHT, kN/m ³	26.45
SAMPLE AREA, cm ²	17.35	DRY UNIT WT., kN/m ³	26.41
SAMPLE VOLUME, cm ³	178.70	SPECIFIC GRAVITY, assumed	2.70
WET WEIGHT, g	482.22	VOID RATIO	0.00
DRY WEIGHT, g	481.50		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	115.4
----------------------	---	-------------------------	-------

REMARKS:

DATE:

12/29/2008

TABLE B2-3
UNCONFINED COMPRESSION (UC) TEST
ASTM D 7012-04

SAMPLE IDENTIFICATION			
PROJECT NUMBER	07-1111-0029	RUN NUMBER	1
BOREHOLE NUMBER	B3-06	SAMPLE DEPTH, m	3.05
		ELEVATION, m	202.6

TEST CONDITIONS			
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST, min	>2 <15	L/D	2.29

SPECIMEN INFORMATION			
SAMPLE HEIGHT, cm	10.80	WATER CONTENT, (specimen) %	0.12
SAMPLE DIAMETER, cm	4.71	UNIT WEIGHT, kN/m ³	26.72
SAMPLE AREA, cm ²	17.42	DRY UNIT WT., kN/m ³	26.69
SAMPLE VOLUME, cm ³	188.17	SPECIFIC GRAVITY, assumed	2.80
WET WEIGHT, g	512.96	VOID RATIO	0.03
DRY WEIGHT, g	512.35		

VISUAL INSPECTION**FAILURE SKETCH**

TEST RESULTS			
STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	79.1

REMARKS:

DATE:

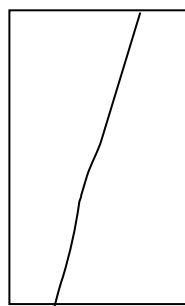
12/29/2008

TABLE B2-4
UNCONFINED COMPRESSION (UC) TEST
ASTM D 7012-04

SAMPLE IDENTIFICATION			
PROJECT NUMBER	07-1111-0029	RUN NUMBER	1
BOREHOLE NUMBER	B3-08	SAMPLE DEPTH, m	1.40
		ELEVATION, m	204.2

TEST CONDITIONS			
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.27

SPECIMEN INFORMATION			
SAMPLE HEIGHT, cm	10.80	WATER CONTENT, (specimen) %	0.27
SAMPLE DIAMETER, cm	4.75	UNIT WEIGHT, kN/m ³	26.56
SAMPLE AREA, cm ²	17.72	DRY UNIT WT., kN/m ³	26.49
SAMPLE VOLUME, cm ³	191.38	SPECIFIC GRAVITY, assumed	2.70
WET WEIGHT, g	518.55	VOID RATIO	0.00
DRY WEIGHT, g	517.15		

VISUAL INSPECTION**FAILURE SKETCH**

TEST RESULTS			
STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	70.3

REMARKS:

DATE:

12/29/2008



APPENDIX C

NON-STANDARD SPECIAL PROVISIONS

DOWELS INTO ROCK - Item No.

Special Provision

1.0 SCOPE

This special provision covers the requirements for the placement and field testing of dowels into rock.

2.0 MATERIALS AND INSTALLATION

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 G30.18, Grade 400).

Where dowels are to be placed in rock, holes shall be drilled to the required depth and size. Each hole shall be cleaned out, grouted and the dowel set in place. Grout shall be of the same strength as the footing concrete (but not less than 30 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.

3.0 ROCK DOWEL TESTING

All proposed testing procedures shall be in general conformance with ASTM D 3689-07, ASTM D1143-07 and ASTM D4435-08. Field testing must be carried out in the presence of, and the results reviewed and approved by, the Contract Administrator.

4.0 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 69 / Shawanaga Service Road (Site No. 9) Bridge	South Abutment	2
Highway 69 / Shawanaga Service Road (Site No. 9) Bridge	North Abutment	2

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

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

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

The design load shall not exceed 288 kN for 35M dowels, 202 kN for 30M dowels, 144 kN for 25M dowels, and 86 kN for 20M dowels. The test loads shall not exceed 80% of the yield strength of the dowel.

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 point.

Rock dowels which fail to meet the acceptance criteria shall be replaced at the Contractor's expense and re-tested. If a rock dowel fails, three (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 percent of the theoretical elastic elongation of the free stressing length and is less than the theoretical elongation of the free stressing length plus 50 percent of the bond length.

5.0 BASIS OF PAYMENT

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

OBSTRUCTIONS - Item No.

Non-Standard Special Provision

The presence of cobbles/boulders and rock fill were encountered at the project site. Consideration of the presence of these obstructions must be made in the selection of appropriate equipment and procedures for sub-excavation through these materials for spread footing construction.

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

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.

Africa	+ 27 11 254 4800
Asia	+ 852 2562 3658
Australasia	+ 61 3 8862 3500
Europe	+ 356 21 42 30 20
North America	+ 1 800 275 3281
South America	+ 55 21 3095 9500

solutions@golder.com
www.golder.com



Golder Associates Ltd.
6925 Century Avenue, Suite #100
Mississauga, Ontario, L5N 7K2
Canada
T: +1 (905) 567 4444

