



March 12, 2015

FOUNDATION INVESTIGATION AND DESIGN REPORT

SHAWANAGA RIVER SBL BRIDGE STRUCTURE

SITE NO. 44-443/2

**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. 5188-06-01 (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



GEOCRES No: 41H-146

Report Number: 07-1111-0029-10

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

REPORT



**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..... | 4 |
| 4.1 Regional Geology..... | 4 |
| 4.2 Subsurface Conditions | 4 |
| 4.3 South Abutment and Approach Area..... | 5 |
| 4.3.1 Topsoil..... | 5 |
| 4.3.2 Silty Sand | 5 |
| 4.3.3 Cobbles / Rock Fragments and Bedrock | 5 |
| 4.3.4 Groundwater Conditions | 7 |
| 4.4 South Pier (Pier 1)..... | 7 |
| 4.4.1 Shawanaga River | 7 |
| 4.4.2 Peat | 7 |
| 4.4.3 Sand to Sand and Silt (Upper Deposit)..... | 8 |
| 4.4.4 Clayey Silt to Silty Clay..... | 8 |
| 4.4.5 Cobbles | 9 |
| 4.4.6 Silty Sand to Sand to Sand and Gravel (Lower Deposit) | 9 |
| 4.4.7 Bedrock | 9 |
| 4.5 North Pier (Pier 2) | 10 |
| 4.5.1 Shawanaga River | 10 |
| 4.5.2 Organic Sand and Organic Clayey Silt | 10 |
| 4.5.3 Clayey Silt to Clay | 11 |
| 4.5.4 Sand to Silty Sand to Sandy Silt | 11 |
| 4.5.5 Clay | 11 |
| 4.5.6 Bedrock | 12 |
| 4.6 North Abutment and Approach Embankment..... | 13 |



FOUNDATION REPORT – SHAWANAGA RIVER SBL BRIDGE STRUCTURE – HIGHWAY 69 G.W.P. 5111-07-00

| | | |
|--|--|-----------|
| 4.6.1 | Topsoil/Peat..... | 13 |
| 4.6.2 | Organic Sandy Silt..... | 13 |
| 4.6.3 | Sand and Gravel to Sand and Silt to Sandy Silt (Upper Deposit) | 13 |
| 4.6.4 | Clayey Silt to Clay | 14 |
| 4.6.5 | Sand and Silt to Sand and Gravel (Lower Deposit) | 14 |
| 4.6.6 | Bedrock/Refusal | 15 |
| 4.6.7 | Groundwater Conditions..... | 16 |
| 5.0 | CLOSURE | 16 |
| PART B – FOUNDATION DESIGN REPORT | | |
| 6.0 | DISCUSSION AND ENGINEERING RECOMMENDATIONS | 17 |
| 6.1 | General | 17 |
| 6.2 | Foundation Options..... | 18 |
| 6.3 | Spread Footings..... | 19 |
| 6.3.1 | Founding Level Alternatives | 19 |
| 6.3.1.1 | South Abutment | 19 |
| 6.3.1.2 | North Abutment..... | 20 |
| 6.3.2 | Geotechnical Axial Resistances / Reactions | 22 |
| 6.3.3 | Resistance to Lateral Loads | 23 |
| 6.3.4 | Frost Protection | 24 |
| 6.3.5 | Footing Set-Back from Rock Faces | 24 |
| 6.4 | Pile Foundations | 25 |
| 6.4.1 | Pile Options | 25 |
| 6.4.1.1 | South Pier (Pier 1)..... | 25 |
| 6.4.1.2 | North Pier (Pier 2) | 26 |
| 6.4.2 | Geotechnical Axial Resistances / Reactions | 27 |
| 6.4.2.1 | 0.457 m Diameter Steel Pipe Piles..... | 28 |
| 6.4.2.2 | 0.609 m Diameter Drilled Steel Casings | 29 |
| 6.4.2.3 | 0.273 m Diameter Micropiles..... | 29 |
| 6.4.2.4 | HP 310x110 Steel Piles | 29 |
| 6.4.2.5 | 0.9 m Diameter Concrete Caissons | 29 |
| 6.4.3 | Downdrag Load (Negative Skin Friction) | 30 |
| 6.4.4 | Resistance to Lateral Loads | 30 |



FOUNDATION REPORT – SHAWANAGA RIVER SBL BRIDGE STRUCTURE – HIGHWAY 69 G.W.P. 5111-07-00

| | | |
|---------|---|----|
| 6.4.4.1 | Lateral Pile Capacity – Bedrock | 30 |
| 6.4.4.2 | Lateral Pile Deflections – Bedrock | 31 |
| 6.4.4.3 | Group Effects | 31 |
| 6.4.5 | Frost protection..... | 32 |
| 6.5 | Seismic Site Coefficient..... | 32 |
| 6.5.1 | Site Coefficient | 32 |
| 6.5.2 | Seismic Analysis Coefficient..... | 32 |
| 6.6 | Lateral Earth Pressures..... | 33 |
| 6.7 | Approach Embankment Design..... | 34 |
| 6.7.1 | Stability | 35 |
| 6.7.1.1 | Methodology..... | 35 |
| 6.7.1.2 | Parameter Selection..... | 35 |
| 6.7.1.3 | Results of Analysis | 36 |
| 6.7.1.4 | Embankment Fill Types..... | 37 |
| 6.7.2 | Settlement | 38 |
| 6.7.2.1 | Methodology..... | 38 |
| 6.7.2.2 | Parameter Selection..... | 38 |
| 6.7.2.3 | Settlement of Foundation Soils | 39 |
| 6.7.2.4 | Settlement of New Embankment Fill | 39 |
| 6.7.2.5 | Embankment Platform Widening..... | 41 |
| 6.8 | Subgrade Preparation and Embankment Construction | 41 |
| 6.8.1 | Removal of Organic Materials | 41 |
| 6.8.2 | Embankment Fill Placement..... | 42 |
| 6.8.3 | Temporary Shoring / Roadway Protection..... | 42 |
| 6.9 | Design and Construction Considerations | 43 |
| 6.9.1 | Overburden Excavation | 43 |
| 6.9.2 | Control of Groundwater and Surface Water..... | 43 |
| 6.9.3 | Cofferdam Construction..... | 44 |
| 6.9.4 | Obstructions | 45 |
| 6.9.5 | Erosion Protection | 45 |
| 6.9.6 | Permit To Take Water | 45 |
| 6.10 | Recommendations for Rock Excavations and Blasting | 45 |



FOUNDATION REPORT – SHAWANAGA RIVER SBL BRIDGE STRUCTURE – HIGHWAY 69 G.W.P. 5111-07-00

| | | |
|------------|----------------------|-----------|
| 6.10.1 | Rock Excavation..... | 45 |
| 6.10.2 | Blasting..... | 46 |
| 7.0 | CLOSURE | 46 |

REFERENCES

TABLES

| | |
|---------|--|
| Table 1 | Evaluation of Foundation Alternatives – South Abutment |
| Table 2 | Evaluation of Foundation Alternatives – North Abutment |
| Table 3 | Evaluation of Foundation Alternatives – Central Piers |

DRAWINGS

| | |
|-----------|------------------------------------|
| Drawing 1 | Site Location Plan |
| Drawing 2 | Borehole Locations and Soil Strata |
| Drawing 3 | Soil Strata |

FIGURES

| | |
|----------|---|
| Figure 1 | Shawanaga River SBL Bridge Structure – North Approach Stability Analysis – Station 17+761 |
|----------|---|

APPENDICES

Appendix A Record of Boreholes and Drillholes

| | |
|--|---|
| List of Symbols and Abbreviations | |
| Lithological and Geotechnical Rock Description Terminology | |
| Record of Boreholes | B5-01 to B5-22 |
| Record of Drillholes | B5-01, B5-02, B5-05, B5-07 to B5-16 and B5-18 |

Appendix B Laboratory Test Results

| | |
|-------------|--|
| 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 B5-14, Run No. 1 |
| Table B2-3 | Unconfined Compression (UC) Test – Borehole B5-07, Run No. 3 |
| Table B2-4 | Unconfined Compression (UC) Test – Borehole B5-05, Run No. 1 |
| Figure B1-1 | Grain Size Distribution – Sand and Silt (Upper Deposit), South Pier (Pier 1) |
| Figure B1-2 | Plasticity Chart – Clayey Silt to Silty Clay, South Pier (Pier 1) |
| Figure B1-3 | Grain Size Distribution – Sand (Lower Deposit), South Pier (Pier 1) |
| Figure B1-4 | Plasticity Chart – Silty Clay to Clay, North Pier (Pier 2) |
| Figure B1-5 | Plasticity Chart – Clay, North Pier (Pier 2) |
| Figure B1-6 | Grain Size Distribution – Sand and Silt to Sandy Silt (Upper Deposit), North Abutment and Approach |
| Figure B1-7 | Plasticity Chart – Clay, North Abutment and Approach |

Appendix C Non-Standard Special Provisions / Operational Constraint

| | |
|--|------------|
| Dowels Into Rock | – Item No. |
| Obstructions | – Item No. |
| Granular Pad Construction In-the-Wet | |
| Earth Excavation and Granular Pad Construction | |



PART A

FOUNDATION INVESTIGATION REPORT

SHAWANAGA RIVER SBL BRIDGE STRUCTURE, SITE NO. 44-443/2

**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. 5188-06-01



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 provide foundation engineering services for the proposed three-span Shawanaga River Southbound Lanes (SBL) Bridge structure over the Shawanaga River (Site No. 44-443/2). The proposed work is part of the four-laning of Highway 69 from 1.0 km north of the new Highway 559 Interchange northerly to 1.5 km north of Highway 7182 (Shebeshekong Road), which involves high fill embankments and embankments over swamps, the New Woods Road and Shebeshekong Road interchanges and structures, the Shawanaga River and Site 9 Road structures, as well as culvert crossings. The general location of this bridge structure along the new 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 SBL Bridge 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 Supplementary Specialty Plan for foundation engineering services for this project, dated July 4, 2007. The General Arrangement (GA) Drawing and the subsequent updated drawing for the proposed Shawanaga River SBL Bridge were provided to Golder by MRC on January 12, 2009 and October 28, 2013, respectively.

This report addresses the investigation carried out for the Shawanaga River SBL Bridge and the associated approach embankments only. Separate reports address the foundation investigations for the swamp crossings, high fill areas associated with 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 bridge structure centerline and the foundation units/limits for this investigation were located in the field prior to drilling 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 SBL Bridge is located approximately 450 m north of the intersection of the existing Shebeshekong Road and Highway 69 and is approximately 20.5 km northwest of Nobel, Ontario. The proposed new Highway 69 alignment will extend generally in a southeast-northwest direction along the west side of the existing Highway 69, which will become part of the future Shawanaga River Service Road (Site No. 9) in this area.

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 and rivers. The proposed bridge and associated approach embankments are to be situated on a bedrock outcrop on the south side of the Shawanaga River and on a flood plain/low-lying area on the north side of the river. On the south side of the river, the ground surface within the limits of the proposed structure generally slopes down from about Elevation 212.3 m at the south approach embankment, to about Elevation 205.2 m at the south abutment and to about Elevation 198.7 m (river level) near the south pier (Pier 1). On the north side of the river, the ground surface rises from about Elevation 199.1 m (river level) near the north pier (Pier 2) up to about Elevation 199.6 m at the north abutment and to about Elevation 200.7 m at the north approach. The water



level of Shawanaga River near the proposed pier locations ranges from about Elevation 198.5 m to a high water level of about Elevation 199.1 m, although it is noted that the water level in the river was as high as Elevation 199.7 m at the time of the investigation. All elevations are referenced to Geodetic datum.

3.0 INVESTIGATION PROCEDURES

3.1 Foundation Investigation

The field work for the Shawanaga River SBL Bridge subsurface investigation was carried out between February 2 and 8, 2009, between April 8 and 15, 2009 and on July 24 and 25, 2009 during which time a total of twenty-two (22) boreholes were advanced: five (5) boreholes at the south abutment; five (5) boreholes at the south pier (Pier 1); five (5) boreholes at the north pier (Pier 2); five (5) boreholes at the north abutment; and one (1) borehole at each approach embankment. The boreholes, designated as Boreholes B5-01 to B5-22, were advanced at the locations shown in plan on Drawing 2. In addition, Dynamic Cone Penetration Tests (DCPTs) were advanced adjacent to Boreholes B5-03, B5-04 and B5-06 to confirm the depth to refusal at these locations. The results of the DCPTs are presented on the Record of Borehole sheets presented in Appendix A.

The field investigation was carried out using a Diedrich D-25 track- and skid/barge-mounted drill rig supplied and operated by Walker Drilling Ltd. of Utopia, Ontario and portable equipment supplied and operated by OGS Inc. of Almonte, Ontario. Hand excavation methods were used at three of the borehole locations to expose the bedrock in shallow overburden areas. The boreholes were advanced through the overburden using 108 mm inside diameter hollow-stem augers, 165 mm O.D. solid-stem augers and 'BW' or '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 automatic hammers in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586 Standard Test Method for Standard Penetration Tests and Split Barrel Sampling of the Soil), or using 76 mm O.D. thin-walled 'Shelby' tubes (ASTM D1587 Standard Practice for Thin-Walled Tube Sampling of Soil for Geotechnical Purposes) for obtaining relatively undisturbed samples in cohesive soils. Boreholes advanced by portable equipment employed one-third ($\frac{1}{3}$) weight hammers lifted manually. Chunk samples were obtained in two (2) boreholes at locations of thin overburden over bedrock outcrops. Field vane shear tests were conducted in cohesive soils for assessment of undrained shear strengths (ASTM D2573 Standard Test Method for Field Vane Shear Test in Cohesive Soil) using a MTO Standard 'N'-size vane. Samples of the bedrock were obtained using 'NQ' and 'BQ' size rock core barrels.

The boreholes at the foundation elements were typically advanced to auger and/or sampler refusal (i.e. inferred bedrock) and bedrock was confirmed by coring in selected boreholes. The boreholes at the approaches were advanced to the bedrock surface / sampler refusal. The boreholes were advanced to depths of up to about 10 m below existing ground surface/riverbed (or up to 12.7 m below river water surface), including coring of between about 1.5 m and 9.0 m into the bedrock, at Boreholes B5-01, B5-02, B5-05, B5-07 to B5-16 and B5-18.

The groundwater conditions in the open boreholes were observed during the drilling operations and a piezometer was installed in each of Boreholes B5-01 and B5-18 to permit monitoring of the water level at these locations. The piezometers consist of 32 mm diameter PVC pipe, with a slotted screen sealed at a select depth within the borehole. The boreholes and annulus surrounding the piezometer pipe above the sand pack/screen were backfilled to the surface with bentonite pellets/grout. Piezometer installation details and water level measurements are described on the Record of Borehole sheets presented in Appendix A. All



FOUNDATION REPORT – SHAWANAGA RIVER SBL BRIDGE STRUCTURE – HIGHWAY 69 G.W.P. 5111-07-00

boreholes in which standpipe piezometers were not installed were backfilled with bentonite to the ground surface or river bottom surface upon completion in accordance with Ontario Regulation 903 (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 soil samples. Strength tests such as unconfined compression strength and point load index, were carried out on specimens of the rock core. The results of the laboratory testing are included in Appendix B.

The boreholes at the pier locations were located relative to the fixed centerline of the bridge structure and surveyed stations on either side of the riverbanks. The as-drilled borehole locations and ground surface elevations were surveyed by a member of our technical staff, referenced to the survey stakes installed by Callon Dietz. The borehole locations given in the Record of Borehole/Drillhole sheets and shown on Drawings 2 and 3 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 / Water Surface Elevation* (m) | Depth Drilled* (m) |
|----------|-----------------------|----------|--|-----------------------|
| | Northing | Easting | | |
| B5-01 | 5045861.4 | 243441.1 | 199.5 | 6.3 |
| B5-02 | 5045856.8 | 243435.3 | 198.7 | 5.9 |
| B5-03 | 5045860.2 | 243431.8 | 199.4 | 3.0 |
| B5-04 | 5045863.4 | 243448.8 | 199.1 | 2.4 |
| B5-05 | 5045866.7 | 243445.3 | 199.6 | 4.9 |
| B5-06 | 5045875.2 | 243426.7 | 200.7 | 0.7 |
| B5-07 | 5045840.5 | 243462.7 | 199.7 (193.6) | 9.8 |
| B5-08 | 5045835.6 | 243458.7 | 199.4 (192.9) | 12.7 |
| B5-09 | 5045840.3 | 243456.5 | 199.2 (194.4) | 11.1 |
| B5-10 | 5045845.0 | 243466.4 | 199.2 (196.2) | 5.6 |
| B5-11 | 5045840.6 | 243468.5 | 199.1 (193.7) | 9.6 |
| B5-12 | 5045812.7 | 243491.5 | 199.1 (192.7) | 12.0 |
| B5-13 | 5045807.9 | 243487.4 | 199.0 (193.4) | 11.3 |
| B5-14 | 5045812.5 | 243485.2 | 198.9 (193.0) | 11.8 |



| Borehole | Location (MTM Nad 83) | | Ground Surface / Water Surface Elevation* (m) | Depth Drilled* (m) |
|----------|-----------------------|----------|--|-----------------------|
| | Northing | Easting | | |
| B5-15 | 5045817.2 | 243495.0 | 198.8 (193.5) | 12.0 |
| B5-16 | 5045812.8 | 243497.3 | 198.7 (193.1) | 8.0 |
| B5-17 | 5045786.4 | 243508.2 | 207.1 | 0.2 |
| B5-18 | 5045790.7 | 243505.3 | 205.2 | 10.0 |
| B5-19 | 5045791.9 | 243513.0 | 208.3 | 0.0 |
| B5-20 | 5045793.0 | 243521.7 | 209.7 | 0.3 |
| B5-21 | 5045795.7 | 243518.8 | 209.1 | 0.1 |
| B5-22 | 5045778.0 | 243527.4 | 212.3 | 0.1 |

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

As delineated in *The Physiography of Southern Ontario*¹, this section of Highway 69 lies within the physiographic region known as the Georgian Bay Fringe, which extends along the east side of Georgian Bay through the Parry Sound and Muskoka areas, then eastward from Muskoka in patches into the area north of the Kawartha Lakes.

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

The bedrock in the area consists typically of gneisses of the Britt Domain of the Central Gneiss Belt, a subdivision of the Grenville Structural Province, as described in *Geology of Ontario, OGS Special Volume 4*². Deposition of Palaeozoic strata initially covered, and later erosion during glaciation subsequently exposed, these Precambrian rocks.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions encountered in the boreholes advanced for this investigation, together with the results of the laboratory tests carried out on selected soil and bedrock core

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

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



samples, are presented in the record of Boreholes sheets provided in Appendix A. The results of the laboratory tests are also provided in Appendix B. The results of the in situ field tests (i.e. SPT 'N'-values and undrained shear strengths from the field vanes) as presented on the Record of Borehole sheets and in Section 4.3 to 4.6 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. Further, subsurface conditions will vary between and beyond the borehole locations. It should be noted that the interpreted stratigraphy shown on Drawings 2 and 3 is a simplification of the subsurface conditions.

In general, the subsurface conditions in the area of the proposed south abutment consist of bedrock outcrops or surficial layers of topsoil underlain by a thin layer of silty sand over bedrock, while in the areas of the proposed piers and the north abutment consist of organic sand to organic sandy silt or topsoil underlain by alternating layers of sand to sandy silt and clayey silt to clay underlain by bedrock.

A detailed description of the subsurface conditions encountered in the boreholes at the abutments, piers, and approach areas is provided in the following sections.

4.3 South Abutment and Approach Area

A total of five (5) boreholes (Boreholes B5-17 to B5-21) were advanced at the location of the proposed south abutment and one (1) borehole (Borehole B5-22) was advanced on the centerline at the proposed south approach. In general, the subsurface conditions consist of topsoil, underlain by silty sand at some locations over cobbles / rock fragments and bedrock.

4.3.1 Topsoil

An approximately 0.1 m to 0.2 m thick layer of topsoil was encountered at the ground surface in Boreholes B5-17 and B5-20 to B5-22.

4.3.2 Silty Sand

In Borehole B5-20, a localized deposit of silty sand containing some gravel, trace clay, trace organics was encountered below the topsoil. The top of this deposit was encountered at about Elevation 209.6 m and the thickness of the deposit is about 0.2 m.

A Standard Penetration Test (SPT) 'N'-value measured within the silty sand deposit is 4 blows per 0.3 m of penetration indicating a very loose relative density.

4.3.3 Cobbles / Rock Fragments and Bedrock

A layer of cobbles / rock fragments ranging from gravel to cobbles sizes was encountered at ground surface in Borehole B5-18 at about Elevation 205.2 m and the thickness of this layer is about 1 m. In general, the size of the recovered cobbles/rock fragments ranges from about 30 mm to 110 mm.



FOUNDATION REPORT – SHAWANAGA RIVER SBL BRIDGE STRUCTURE – HIGHWAY 69 G.W.P. 5111-07-00

Bedrock was encountered and core samples were recovered in Borehole B5-18 and bedrock was observed to outcrop at Borehole B5-19. The presence of bedrock was inferred by refusal to split-spoon advancement in Boreholes B5-20 and B5-21 and identified by hand excavations at Boreholes B5-17 and B5-22.

The top of the bedrock surface generally varies between about Elevations 209.4 m and 204.2 m at the proposed south abutment and is at about Elevation 212.3 m at the south approach area. The depth to bedrock below ground surface and corresponding bedrock surface elevation is summarized below.

| Foundation Element / Approach Area | Borehole No. | Depth to Bedrock Surface (m) | Bedrock Surface Elevation (m) | Refusal Type |
|------------------------------------|--------------|------------------------------|-------------------------------|-----------------|
| South Approach Area | B5-22 | 0.1 | 212.2 | Hand Shovel |
| | B5-17 | 0.2 | 206.9 | Hand Shovel |
| South Abutment | B5-18 | 1.0 | 204.2 | Bedrock Cored |
| | B5-19 | 0.0 | 208.3 | Bedrock Outcrop |
| | B5-20 | 0.3 | 209.4 | Split-Spoon |
| | B5-21 | 0.1 | 209.0 | Split-Spoon |

Across the south abutment from the southeast corner to the northwest corner of the abutment footprint (a distance of approximately 16.5 m between borehole locations), the bedrock surface elevation varies by about 5.2 m, corresponding to an approximately 3.2H:1V slope or a dip angle of approximately 17° from the horizontal.

Based on the bedrock core samples, the bedrock consists of granite gneiss. In general the bedrock samples are described as slightly weathered to fresh, coarse grained, highly to moderately foliated, slightly to moderately porous, light grey to pink. The Rock Quality Designation (RQD) measured on the core samples ranges from 0 percent to 100 percent, indicating a rock mass of very poor to excellent quality according to Table 3.10 in CFEM (2006)³. The Total Core Recovery (TCR) of the samples recovered is between 7 percent and 100 percent and the Solid Core Recovery (SCR) of samples recovered is between 19 percent and 96 percent.

Point load strength tests were performed on selected samples of the rock core. The diametral point load strength index values are shown on the Record of Drillhole sheets in Appendix A and are presented in Table B1 in Appendix B. The diametral tests carried out on three (3) samples of the gneiss bedrock from this location measured Is_{50} values ranging from about 6.2 MPa to 9.9 MPa.

Also presented in Table B1 are the estimated Unconfined 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 and Application to Rock Strength Classification), which may vary 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 Unconfined Compression (UC) test results and the average of the corresponding point load strength test

³ Canadian Geotechnical Society. 2006. Canadian Foundation Engineering Manual, 4th Edition. The Canadian Geotechnical Society c/o BiTech Publisher Ltd, British Columbia.



results. These values have been given for comparison only and should be interpreted together with the results of the UC tests.

Based on the point load test results, according to Table 3.5 in CFEM (2006)³, the gneiss bedrock at this location is classified as strong (R4, 50 MPa < UCS < 100 MPa) to very strong (R5, 100 MPa < UCS < 250 MPa).

4.3.4 Groundwater Conditions

A standpipe piezometer was installed in Borehole B5-18 to allow monitoring of the groundwater level at this location. Details of the piezometer installation are shown on the Record of Borehole and Drillhole sheets in Appendix A. In general, the overburden samples taken in the boreholes advanced in this area were moist to wet. 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 | B5-18 | 205.2 | 203.0 202.8 | July 28, 2009 August 26, 2009 |

It should be noted that the groundwater level in the area is subject to seasonal fluctuations due to snow melt and precipitation events. The water level in the adjacent Shawanaga River is also affected by run-off during periods of the year which can influence the groundwater conditions on the adjacent banks. Groundwater levels should be expected to be higher during wet periods of the year.

4.4 South Pier (Pier 1)

A total of five (5) boreholes (Boreholes B5-12 to B5-16) were completed to investigate the subsurface conditions at the proposed south pier (Pier 1), located within the Shawanaga River. In general, the subsurface conditions consist of peat at some locations, underlain by alternating layers of non-cohesive soils comprised of sand and gravel, sand and sandy silt and cohesive soils comprised of clayey silt and silty clay, underlain by bedrock.

4.4.1 Shawanaga River

Boreholes B5-12 to B5-16 are in-water boreholes. At the time of the foundation investigation, the river level or water surface varied and was measured to range from about Elevations 199.1 m to 198.7 m, and the depth of water to the river bed ranges between about 5.3 m and 6.4 m. The surface of the riverbed varies between about Elevations 193.5 m and 192.7 m.

It should be noted that the elevation of the river level fluctuates on a seasonal basis especially during periods of rain or snow melt and run-off.

4.4.2 Peat

A deposit of brown, wet, amorphous peat containing wood fragments and interlayers of sand was encountered at the riverbed in Boreholes B5-12 and B5-15. The top of the peat deposit was encountered at



about Elevations 192.7 m and 193.5 m and the thickness of the deposit is about 0.6 m and 1.0 m at the respective boreholes.

The Standard Penetration Test (SPT) 'N'-values measured within the peat deposit are 0 blows (weight of hammer) and 1 blow per 0.3 m of penetration, suggesting a very soft consistency.

The natural water content measured on a sample of this deposit is about 279 percent.

4.4.3 Sand to Sand and Silt (Upper Deposit)

A non-cohesive deposit comprised of sand trace gravel and sand and silt trace clay, trace organics and containing wood fragments was encountered below the whe peat deposit in Boreholes B5-12 and B5-15 and at the riverbed in Boreholes B5-13 and B5-14. The top of this deposit varies between about Elevations 193.4 m and 192.1 m and the thickness of the deposit ranges from about 0.5 m to 2.5 m.

The SPT 'N'-values measured within the sand to sand and silt deposit range from 0 blows (weight of rod and weight of hammer) to 6 blows per 0.3 m of penetration, indicating a very loose to loose relative density. A SPT 'N'-value of 25 blows per 0.08 m of penetration was recorded at the interface of this deposit with the underlying layer of cobbles.

The natural water content measured on samples of this deposit ranges from about 24 percent to 66 percent.

A grain size distribution of a sample of the sand and silt portion of this deposit is shown on Figure B1-1 in Appendix B.

4.4.4 Clayey Silt to Silty Clay

A cohesive deposit of clayey silt to silty clay containing silt interlayers was encountered below the sand to sand and silt deposit in Boreholes B5-12 to B5-14. The top of this deposit varies between about Elevations 192.9 m and 190.5 m and the thickness of the deposit ranges from about 0.3 m to 0.7 m.

The SPT 'N'-values measured within the clayey silt to silty clay deposit range from 0 blows (weight of rod and weight of hammer) to 6 blows per 0.3 m of penetration. An in situ field vane test carried out within this deposit measured an undrained shear strength of about 27 kPa, and the sensitivity is calculated to be about 5. The SPT 'N'-values together with the field vane test result suggest that the clayey silt to silty clay has a very soft to firm consistency.

The natural water content measured on two (2) samples of this deposit is about 47 percent and 53 percent.

Atterberg limits tests carried out on two (2) samples of the clayey silt to silty clay deposit yielded liquid limits of about 34 percent and 48 percent, plastic limits of about 15 percent and 16 percent and corresponding plasticity indices of about 19 percent and 32 percent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure B1-2 in Appendix B and indicate that this material is classified as clayey silt of low plasticity to silty clay of intermediate plasticity.



4.4.5 Cobbles

An approximately 0.4 m and 0.2 m thick layer of cobbles was encountered below the silty clay deposit at about Elevation 191.4 m in Borehole B5-12 and below the sand deposit at about Elevation 191.7 m in Borehole B5-15.

4.4.6 Silty Sand to Sand to Sand and Gravel (Lower Deposit)

A lower non-cohesive deposit comprised of silty sand, sand trace to some silt and trace to some gravel, and sand and gravel was encountered underlying the layer of cobbles in Borehole B5-12 and below the clayey silt to silty clay deposit in Boreholes B5-13 and B5-14. The top of this deposit varies between about Elevations 192.5 m and 189.8 m and the thickness of the deposit ranges from about 0.5 m to 1.1 m.

The SPT 'N'-values measured within this deposit range from 8 blows to 55 blows per 0.3 m of penetration, indicating a loose to very dense relative density. SPT 'N'-values of 100 blows per 0.18 m of penetration and 20 blows per 0.02 m of penetration were measured in Boreholes B5-13 and B5-14, respectively, immediately above the bedrock surface.

The natural water content measured on two (2) samples of this deposit is about 8 percent and 22 percent.

A grain size distribution of a sample from the sand portion of the deposit is shown on Figure B1-3 in Appendix B.

4.4.7 Bedrock

Bedrock was encountered below the subsoils in Boreholes B5-12 to B5-15 and at the river bed in Borehole B5-16, and core samples were recovered at the borehole locations.

The top of the bedrock surface varies between about Elevations 193.1 m and 188.7 m. The depth to bedrock from the river bed and the corresponding bedrock surface elevation is summarized below.

| Foundation Element | Borehole No. | Depth to Bedrock Surface from Riverbed (m) | Bedrock Surface Elevation (m) | Refusal Type |
|---------------------------|---------------------|---|--------------------------------------|---------------------|
| South Pier (Pier 1) | B5-12 | 2.1 | 190.6 | Bedrock Cored |
| | B5-13 | 1.8 | 191.7 | Bedrock Cored |
| | B5-14 | 4.3 | 188.7 | Bedrock Cored |
| | B5-15 | 2.0 | 191.5 | Bedrock Cored |
| | B5-16 | 0.0 | 193.1 | Bedrock Cored |

Across the south pier, from the southeast corner to the northwest corner of the pier footprint (a distance of approximately 12.5 m between borehole locations), the bedrock surface elevation varies by about 4.4 m, corresponding to an approximately 2.8H:1V slope or a dip angle of approximately 19° from the horizontal.

Based on the bedrock core samples, the bedrock consists of granite gneiss. In general, the bedrock samples are described as slightly weathered to fresh, fine to coarse grained with feldspar banding, foliated, black, pink, dark grey and grey. The Rock Quality Designation (RQD) measured on the core samples ranges between 0 percent and 80 percent, indicating a rock mass of very poor to good quality according to



Table 3.10 in CFEM (2006)³. The Total Core Recovery (TCR) and Solid Core Recovery (SCR) of the core samples are between about 91 percent and 100 percent and 11 percent and 88 percent, respectively.

Point load strength tests were performed on selected sample of the rock core. The diametral point load strength index values are shown on the Record of the Drillhole sheets in Appendix A and in Table B1 in Appendix B. The diametral tests carried out in six (6) samples of the granite gneiss bedrock measured Is_{50} values ranging from approximately 3.8 MPa to 7.0 MPa.

One (1) Unconfined Compression (UC) test was carried out in accordance with ASTM D7102 on a selected sample of the granite gneiss bedrock measured a compressive strength of about 99 MPa, as summarized in Table B2-1 and detailed in Table B2-2 in Appendix B.

Also presented in Table B1 are the estimated Unconfined Compressive Strength (UCS) values for each sample tested for point load strength based on a relationship between Is_{50} and UCS and a 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 test and the point load test results, according to Table 3.5 in CFEM (2006)³, the granite gneiss bedrock at this location is classified as strong (R4, 50 MPa < UCS < 100 MPa).

4.5 North Pier (Pier 2)

A total of five (5) boreholes (Boreholes B5-07 to B5-11) were completed to investigate the subsurface conditions at the proposed north pier (Pier 2), located within the Shawanaga River. In general, the subsurface conditions consist of organic sand and organic clayey silt at some locations underlain by alternating layers of non-cohesive soils comprised of sand, silty sand and sandy silt and cohesive soils comprised of clayey silt, silty clay and clay, underlain by bedrock.

4.5.1 Shawanaga River

Boreholes B5-07 to B5-11 are in-water boreholes. At the time of the foundation investigation, the river level or water surface varied and was measured to range from about Elevations 199.7 m to 199.1 m, and the depth of water to the river bed ranges between about 3.0 m and 6.6 m. The surface of the riverbed varies between about Elevations 196.2 m and 192.9 m.

It should be noted that elevation of the river level fluctuates on a seasonal basis especially during periods of rain or snow melt and run-off.

4.5.2 Organic Sand and Organic Clayey Silt

A deposit of organic sand containing wood fragments and rootlets was encountered at the riverbed in Boreholes B5-07 and B5-08. The top of the organic sand deposit is at about Elevations 193.6 m and 192.9 m and the thickness of the deposit is about 0.6 m and 1.5 m in the respective boreholes. In Borehole B5-09, an approximately 0.5 m thick deposit of organic clayey silt was encountered at the river bed at about Elevation 194.4 m. The organic clayey silt deposit contains trace gravel, trace sand and wood fragments.



The Standard Penetration Test (SPT) 'N'-values measured within the organic sand and organic clayey silt deposits are 0 blows (weight of rod) per 0.3 m of penetration, indicating a very loose relative density and very soft consistency, respectively.

The natural water content measured on one (1) sample of the organic clayey silt is about 93 percent.

4.5.3 Clayey Silt to Clay

A cohesive deposit comprised of clayey silt to silty clay to clay was encountered below the organic sand and organic clayey silt deposits in Boreholes B5-07 to B5-09. The top of this deposit varies between about Elevations 193.9 m and 191.3 m and the thickness of the deposit ranges from about 0.7 m to 2.0 m.

The SPT 'N'-values measured within the cohesive deposit are 0 blows (weight of the rod) per 0.3 m of penetration. In situ field vane tests carried out within this deposit measured undrained shear strength of about 17 kPa and 18 kPa; and the sensitivity is calculated to be between about 4 and 5. The field vane test results indicate that the clayey silt to clay deposit has a soft consistency.

The natural water content measured on two (2) samples of the clayey silt to clay deposit is about 39 percent and 67 percent.

Atterberg limits tests were carried out on two (2) samples of the cohesive deposit and yielded liquid limits of about 45 percent and 53 percent, plastic limits of about 16 percent and 20 percent, and corresponding plasticity indices of about 29 percent and 33 percent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure B1-4 in Appendix B and indicate that the cohesive material tested is classified as silty clay of intermediate plasticity to clay of high plasticity.

4.5.4 Sand to Silty Sand to Sandy Silt

A non-cohesive deposit of sand some silt to silty sand to sandy silt was encountered below the clayey silt to clay deposit in Boreholes B5-07 to B5-09 and at the river bed in Boreholes B5-10 and B5-11. The deposit encountered in Boreholes B5-10 and B5-11 contains trace gravel, organics and wood fragments. The top of the non-cohesive deposit varies between about Elevations 192.3 m and 189.3 m in Boreholes B5-07 to B5-09, while the top of the deposit was encountered at a higher Elevations 196.2 m and 193.7 m in Boreholes B5-10 and B5-11, respectively. The overall thickness of this deposit ranges from about 0.2 m to 1.1 m.

The SPT 'N'-values measured within this deposit range between 0 blows (weight of rod) and 3 blows per 0.3 m of penetration, 10 blows per 0.03 m of penetration and 5 blows per 0.13 m of penetration, indicating a very loose to loose relative density.

The natural water content measured on a sample of the silty sand and on a sample of the sandy silt portion of this deposit is about 22 percent and 94 percent, respectively. The high water content indicates the presence of organics within the sandy silt material.

4.5.5 Clay

In Borehole B5-08, a localized deposit of clay containing silt interlayers was encountered below the silty sand deposit at about Elevation 188.9 m. The thickness of this deposit is about 0.7 m.



FOUNDATION REPORT – SHAWANAGA RIVER SBL BRIDGE STRUCTURE – HIGHWAY 69 G.W.P. 5111-07-00

An SPT 'N'-value of 1 blow per 0.3 m of penetration was recorded within this deposit, suggesting a very soft consistency.

The natural water content measured on a sample of the clay deposit is about 66 percent.

An Atterberg limits test was carried out in one (1) sample of this deposit and yielded a liquid limit of about 54 percent, a plastic limit of about 20 percent and a plasticity index of about 34 percent. The result of the Atterberg limits test is shown on the plasticity chart on Figure B1-5 in Appendix B and indicates that this material is clay of high plasticity.

4.5.6 Bedrock

Bedrock was encountered below the overburden soils or at the riverbed surface and core samples were recovered at all borehole locations.

The top of bedrock surface generally varies between about Elevations 195.1 m and 188.2 m. The depth to the bedrock from the river bed and the corresponding bedrock surface elevation is summarized below.

| Foundation Element | Borehole No. | Depth to Bedrock Surface from Riverbed (m) | Bedrock Surface Elevation (m) | Refusal Type |
|------------------------|--------------|--|-------------------------------|---------------|
| North Pier (Pier 2) | B5-07 | 1.5 | 192.1 | Bedrock Cored |
| | B5-08 | 4.6 | 188.2 | Bedrock Cored |
| | B5-09 | 2.9 | 191.5 | Bedrock Cored |
| | B5-10 | 1.1 | 195.1 | Bedrock Cored |
| | B5-11 | 0.4 | 193.3 | Bedrock Cored |

Across the north pier from the northeast corner to the southwest corner of the pier footprint (a distance of approximately 12.5 m between borehole locations), the bedrock surface elevation varies by about 6.9 m, corresponding to an approximately 1.8H:1V slope and a dip angle of approximately 29° from the horizontal.

Based on the bedrock core samples, the bedrock consists of granite gneiss. In general, the bedrock samples are describes as slightly weathered to fresh, fine to coarse grained with feldspar banding, foliated, black, pink, dark grey and grey. The Rock Quality Designation (RQD) measured on the core samples ranges between 39 percent and 78 percent, indicating a rock mass of poor to good quality according to Table 3.10 in CFEM (2006)³. The Total Recovery (TCR) and Solid Core Recovery (SCR) of core samples are between about 92 percent and 100 percent and about 39 percent and 92 percent, respectively.

Point load strength tests were performed on selected sample of the rock core. The diametral point load strength index values are shown on the Record of the Drillhole sheets in Appendix A and in Table B1 in Appendix B. The diametral tests carried out on four (4) samples of granite gneiss bedrock measured Is_{50} values ranging from approximately 7.7 MPa to 9.3 MPa.

One (1) Unconfined Compression (UC) test was carried out in accordance with ASTM D7102, on a selected sample of the granite gneiss bedrock and measured a compressive strength of about 118 MPa, as summarized in Table B2-1 and detailed in Table B2-3 in Appendix B.

Also presented in Table B1 are the estimated Unconfined Compressive Strength (UCS) values for each sample tested for point load strength based on a relationship between Is_{50} and UCS and a 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 test and the point load test results, according to Table 3.5 in CFEM (2006)³, the granite gneiss bedrock at this location is classified as very strong (R5, 100 MPa < UCS < 250 MPa).

4.6 North Abutment and Approach Embankment

A total of five (5) boreholes (Boreholes B5-01 to B5-05) was advanced at the location of the proposed north abutment and one (1) borehole (Borehole B5-06) was advanced on the centerline at the north approach embankment. In general, the subsurface conditions consist topsoil/peat and organic sandy silt underlain by alternating layers of non-cohesive soils comprised of sand and gravel, sandy silt and sand and silt and cohesive soils comprised of clayey silt, silty clay and clay, underlain by bedrock.

4.6.1 Topsoil/Peat

An approximately 0.2 m to 0.9 m thick deposit of topsoil/peat was encountered at the ground surface in all boreholes except for Borehole B5-04. The peat deposit contains wood fragments. In general, the top of the topsoil/peat deposit varies from about Elevations 200.7 m to 198.7 m.

The Standard Penetration Test (SPT) 'N'-values measured within the topsoil deposit range from 2 blows to 6 blows per 0.3 m of penetration, indicating a very loose to loose relative density. A SPT 'N'-value of 53 blows per 0.3 m of penetration was measured within the peat deposit in Borehole B5-05, suggesting a hard consistency. This high SPT 'N' value could be a result of wood fragments present within the peat deposit.

The natural water content measured on one (1) sample of topsoil is about 44 percent and the natural water content measured on one (1) sample of peat deposit is about 229 percent.

4.6.2 Organic Sandy Silt

A deposit of dark brown to grey organic sandy silt trace to some clay, containing silty sand layers and rootlets was encountered below the topsoil in Borehole B5-03 and at the ground surface in B5-04. The top of this deposit is at about Elevations 199.2 m and 199.1 m and the thickness of this deposit is about 1.2 m and 0.6 m at the respective boreholes.

The SPT 'N'-values measured within this deposit are 2 blows and 3 blows per 0.3 m of penetration, indicating a very loose relative density.

The natural water content measured on samples of this deposit ranges from about 28 percent to 87 percent. The organic content measured on one (1) sample this deposit in Borehole B5-04 is about 13 percent.

4.6.3 Sand and Gravel to Sand and Silt to Sandy Silt (Upper Deposit)

A non-cohesive deposit comprised of sand and gravel trace silt, sand and silt trace to some clay and sandy silt trace clay was encountered either below the topsoil/peat or organic sandy silt in all the boreholes except



for Borehole B5-04. In Boreholes B5-01, B5-02 and B5-05, the deposit contains organics and rootlets. The top of this deposit at the boreholes typically ranges from about Elevations 198.9 m to 198.0 m, while in Borehole B5-06 the surface of the deposit is at about Elevation 200.5 m. The thickness of this deposit ranges from about 0.5 m to 1.4 m. The bottom of this deposit was defined by refusal to auger advancement and/or dynamic cone penetration in Boreholes B5-03 and B5-06.

The SPT 'N'-values measured within the sand and silt to sandy silt deposit typically range from 2 blows to 14 blows per 0.3 m of penetration, and a SPT 'N'-value of 30 blows per 0.3 m of penetration was recorded within the sand and gravel portion of deposit, all indicating a very loose to dense relative density. In addition, a SPT 'N'-value of 69 blows per 0.1 m of penetration was recorded in Borehole B5-05 immediately above the bedrock surface.

The natural water content measured on one (1) sample of the sand and gravel portion of the deposit in Borehole B5-06 is about 8 percent. The natural water content measured on five (5) samples of the sand and silt to sand silt portions of the deposit ranges from about 20 percent to 82 percent, with the high water content recorded within the upper portion of the deposit containing organics.

The grain size distributions of two (2) samples of the sand and silt to sandy silt portion of this deposit are shown on Figure B1-6 in Appendix B.

4.6.4 Clayey Silt to Clay

A cohesive deposit comprised of clayey silt, silty clay and clay was encountered below the sand and silt deposit in Boreholes B5-01 and B5-02 and below the organic sandy silt deposit in Borehole B5-04. The clayey silt deposit encountered in Borehole B5-04 contains trace to some sand. The top of this deposit varies between about Elevations 198.5 m and 197.2 m and the thickness of the deposit ranges from about 0.1 m to 1.8 m. The bottom of this deposit was defined by refusal to split-spoon and auger advancement and dynamic cone penetration in Boreholes B5-04.

The SPT 'N'-values measured within this deposit range from 1 blow to 8 blows per 0.3 m of penetration. Two in situ field vane tests carried out within the clay deposit measured undrained shear strengths of about 21 kPa and 53 kPa, and the sensitivity is calculated to be about 3 and 6. The SPT 'N'-values and the field vane test results suggest that the clayey silt to clay deposit has a very soft to stiff consistency.

The natural water content measured on three (3) samples of this deposit ranges from about 21 percent to 66 percent.

Atterberg limits tests were carried out on two (2) samples of the clay portion of this deposit, and measured liquid limits of about 51 percent, plastic limits of about 18 percent and the corresponding plasticity indices of about 33 percent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure B1-7 in Appendix B, and indicate that the material is classified as clay of high plasticity.

4.6.5 Sand and Silt to Sand and Gravel (Lower Deposit)

A lower non-cohesive deposit of sand and silt containing trace clay was encountered immediately below the silty clay to clay deposit in Boreholes B5-01 and B5-02. In Borehole B5-02, the sand and silt deposit grades to sand and gravel immediately over the bedrock surface. The sand and gravel layer contains trace silt and



cobbles. The top of this deposit was encountered at about Elevations 197.7 m and 196.6 m, and the thickness of the deposit is about 0.7 m and 0.6 m at the respective boreholes.

The SPT 'N'-values measured within this deposit are 4 blows and 11 blows per 0.3 m of penetration, indicating a loose to compact relative density. A SPT 'N'-value of 100 blows per 0.07 m of penetration was recorded at the transition zone of sand and silt to sand and gravel in Borehole B5-01, which is immediately above the bedrock surface.

The natural water content measured on two (2) samples of this deposit is about 14 percent and 25 percent.

4.6.6 Bedrock/Refusal

In Boreholes B5-03, B5-04 and B5-06, the presence of bedrock was inferred from refusal to auger advancement and resistance to cone penetration. Bedrock was encountered and core samples were recovered in Boreholes B5-01, B5-02 and B5-05.

The top of bedrock surface varies between about Elevations 197.8 m and 195.9. The depth to bedrock surface from the ground surface and the corresponding bedrock elevation is summarized below.

| Foundation Element / Approach Embankment | Borehole No. | Depth to Bedrock Surface (m) | Bedrock Surface Elevation (m) | Refusal Type |
|---|---------------------|-------------------------------------|--------------------------------------|---------------------|
| North Abutment | B5-01 | 2.5 | 197.0 | Bedrock Cored |
| | B5-02 | 2.8 | 195.9 | Bedrock Cored |
| | B5-03 | 2.8 | 196.6 | Auger Refusal |
| | B5-04 | 2.4 | 196.7 | Auger Refusal |
| | B5-05 | 1.8 | 197.8 | Bedrock Cored |
| North Approach Embankment | B5-06 | 0.7 | 200.0 | Auger Refusal |

Across the north abutment from the northeast corner to the southwest corner of the abutment footprint (a distance of approximately 14.5 m between borehole locations), the bedrock surface elevation varies by about 1.9 m, corresponding to an approximately 7.6H:1V slope and a dip angle of approximately 7° from the horizontal.

Based on the bedrock core samples, the bedrock consists of granite gneiss. In general the bedrock samples are described as slightly weathered to fresh, fine to coarse grained with feldspar banding, foliated, black, pink and grey. The Rock Quality Designation (RQD) measured on the cored samples ranges from 45 percent to 88 percent, indicating a rock mass of poor to good quality according to Table 3.10 in CFEM (2006)³. The Total Core Recovery (TCR) and Solid Core Recovery (SCR) of the core samples are between about 91 percent and 100 percent and between about 29 percent and 90 percent, respectively.

Point load strength tests were performed on selected samples of the rock core. The axial and diametral point load strength index values are shown on the Record of Drillhole sheets in Appendix A and are presented in Table B1 in Appendix B. The axial point load test carried out on two (2) samples of the granite gneiss bedrock measured Is_{50} values of 4.2 MPa and 4.5 MPa. The diametral tests carried out on six (6) core samples of the granite gneiss bedrock measured Is_{50} values ranging from approximately 5.3 MPa to 8.5 MPa.



One (1) Unconfined Compression (UC) test was carried out in accordance to ASTM D7102, on a selected sample of the granite gneiss bedrock and measured a compressive strength of about 58 MPa, as summarized on Table B2-1 and detailed on Table B2-4 in Appendix B.

Also presented in Table B1 are the estimated Unconfined Compressive Strength (UCS) values for each sample tested for point load strength based on a relationship between Is_{50} and UCS and a 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 test and the point load test results, according to Table 3.5 in CFEM (2006)³, the granite gneiss bedrock at this location is classified as strong (R4, 50 MPa < UCS < 100 MPa) to very strong (R5, 100 MPa < UCS < 250 MPa).

4.6.7 Groundwater Conditions

A standpipe piezometer was installed in Borehole B5-01 to allow monitoring of the groundwater level at the site. Details of the piezometer installation are shown on the Record of Borehole and Drillhole sheets in Appendix A. In general, the overburden samples taken in boreholes advanced in this area were moist to wet. 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 |
|--------------------|--------------|------------------------------|---------------------------|---------------------|
| North Abutment | B5-01 | 199.5 | 199.3 | April 16, 2009 |
| | | | 199.0 | August 26, 2009 |

It should be noted that groundwater level in the area is subject to seasonal fluctuations, snow melt and precipitation events. The water level in the adjacent Shawanaga River is also affected by run-off during periods of the year which can influence the groundwater conditions on the adjacent banks. Groundwater levels should be expected to be higher during wet periods of the year.

5.0 CLOSURE

Messrs. Matt Rhody and Chris Radway, senior technicians with Golder, directed the drilling program. This report was prepared 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.



FOUNDATION REPORT – SHAWANAGA RIVER SBL BRIDGE STRUCTURE – HIGHWAY 69 G.W.P. 5111-07-00

Report Signature Page

Christopher Ng, P. Eng.
Geotechnical Engineer, Associate



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

TVA/CN/JPD/JMAC/jl



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

\\golder.gds\gal\mississauga\active\2007\1111\07-1111-0029 - mrc - hwy 69 four-laning -\report\final\10 - shawanaga river sbl bridge\07-1111-0029-10 fidr
15mar12 highway 69 shawanaga river sbl bridge structure.docx



PART B

FOUNDATION DESIGN REPORT

SHAWANAGA RIVER SBL BRIDGE STRUCTURE, SITE NO. 44-443/2

**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. 5188-06-01



6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides engineering design recommendations for the proposed Shawanaga River SBL Bridge structure. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the design of the structure 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 Shawanaga River SBL Bridge and approaches. 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 approaches, 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 bridge structures, the Shawanaga River and Site 9 Road structures, as well as culvert crossings.

It is understood that the Shawanaga River SBL Bridge will be a three-span, pre-cast concrete girder structure consisting of two end-spans 30 m long and a centre span 40 m long, with abutments located south and north of the Shawanaga River and the south and north piers located in the river. The alignment for the proposed Shawanaga River SBL Bridge is west of the newly proposed Shawanaga NBL Bridge and existing Highway 69.

Based on the General Arrangement (GA) Drawing provided by MRC on October 28, 2013 and subsequent revision to the GA drawing provided on June 05, 2014, the grade of the proposed Shawanaga River SBL Bridge deck varies between about Elevation 208.1 m (south abutment) and Elevation 207.5 m (north abutment). The existing ground surface within the south approach embankment and abutment area generally varies from about Elevations 212.3 m to 205.2 m (at the investigated locations), but is generally above Elevation 208.1 m. Therefore rock cuts up to about 4.2 m below the existing ground surface will be required in the area of the south approach and a portion of the south abutment and some localized filling (up to about 2.9 m deep) may be required at the west edge of the south abutment. The existing ground surface within the north approach embankment and abutment area varies from about Elevations 200.7 m to 198.7 m (at the investigated locations); therefore, the north approach embankment and abutment area will require fills up to about 8.8 m high relative to the existing ground surface. At the south pier (Pier 1) which is located in the river, the river level varied from about Elevations 199.1 m to 198.7 m (at the time of the investigation) and the surface of the river bed varies from about Elevations 193.5 m to 192.7 m (at the investigated locations). At the north pier (pier 2) which is located in the river, the river level varied from about Elevations 199.7 m to 199.1 m (at the time of the investigation) and the



surface of the river bed varies from about Elevations 196.2 m to 192.9 m (at the investigated locations). The normal (local) surface water level and the high water level of the Shawanaga River are indicated on the GA Drawing provided by MRC to be at about Elevations 198.5 m and 202.2 m, respectively.

6.2 Foundation Options

Given the proximity of the bedrock surface to the existing ground surface, the proposed elevations of underside of footings and the variation in the overburden at the site, especially the presence of organic deposits and weak/soft cohesive deposits at the north abutment, shallow foundations comprised of spread footings founded either directly on the bedrock (at the south abutment) or on a compacted granular pad constructed on the bedrock (at the north abutment) are considered appropriate to support the bridge structure abutments. Pile foundations (H-piles) have also been considered as an alternative system for support of the bridge structure at the abutments, however, given the shallow depth to bedrock, installation of the piles would require significant excavation/trenching into the strong to very strong bedrock (at the south abutment) as well as the drilling of shallow sockets into the bedrock (at the north abutment) to achieve the minimum required pile lengths for an integral abutment design and this option would be more expensive than the shallow foundation option.

For the south pier (Pier 1) and north pier (Pier 2) located in the river, several deep (pile) foundations options have been considered including:

- steel pipe piles (drilled into bedrock and filled with concrete);
- drilled steel casings (socketed into bedrock and filled with concrete);
- micropiles;
- H-piles (fixed into bedrock sockets backfilled with concrete); and,
- caissons (socketed into bedrock)

The use of shallow foundations to support the in-water piers was also considered, however the overburden deposits below the river bed are comprised of variable thickness of organic deposits and loose sands, silts and very soft clays that are unsuitable to support spread footings. Supporting the piers on spread footings founded directly on the bedrock (or on mass concrete over bedrock) was also considered, however, given the depth of water at the piers (ranging from about 3.0 m to 6.6 m), the variable overburden thickness (ranging from 0 m, i.e. bedrock exposed at riverbed, to up to about 4.6 m), the presence of cobbles in the overburden, and the sloping and strong to very strong nature of the bedrock, construction of the footings in the river would be difficult and present serious challenges. Conventional cofferdam construction would be required, and as discussed further in Section 6.9.3, given the conditions at the site, achieving an adequate seal at the base of the cofferdam to allow dewatering for construction in-the-dry would be difficult, carry high costs and pose a risk to the successful completion of the foundations.

From a foundations perspective, shallow foundations are considered most practical for construction and the preferred foundation alternative at the south and north abutments, while deep foundations (comprised of drilled steel pipe piles or casings) are the preferred foundation alternative for the south and north in-water piers.



The following sections provide recommendations for the design of shallow foundations (spread footings) at the abutments, and deep foundations (drilled steel pipe piles, casings, micropiles, steel H-piles and caissons) at the piers to support the proposed bridge foundation elements.

The advantages, disadvantages, relative costs and risks/consequences for each of the foundation alternatives at the south abutment, north abutment and piers are summarized in Tables 1 to 3, respectively.

6.3 Spread Footings

At this site, shallow foundations comprised of spread footings founded either directly on the bedrock (or on mass concrete over bedrock), and on a granular pad over bedrock are considered the preferred alternative for support of the south and north abutments, respectively. The overburden deposits encountered at the location of the north abutment are either too weak/soft or too loose to be able to support the bridge structure and are not suitable as a bearing stratum for a shallow bridge foundation. A comparison between shallow and deep foundation alternatives (and associated relative costs) at the north and south abutments is presented in Tables 1 and 2, respectively.

6.3.1 Founding Level Alternatives

There are several founding level alternatives for support of the south and north abutments as discussed below. The following sections outline the recommendations for footing founding options, geotechnical resistances, resistance to lateral loads and requirements for frost protection.

6.3.1.1 South Abutment

The existing ground surface in the area of the south abutment footprint varies from about Elevations 209.7 m to 205.2 m. A layer of topsoil about 0.2 m thick was encountered at the ground surface, underlain by a localized very loose silty sand deposit and gravel, cobbles/rock fragments at the southwest and northeast corners, respectively, of the proposed abutment. The overburden is underlain by gneiss bedrock.

Based on the GA Drawing, the underside of the 1.5 m wide footing for the south abutment is proposed to be at about Elevation 204.1 m (considering a 0.5 m thick footing).

The details of the ground surface elevation, bedrock surface elevation, the depth to the bedrock surface below existing ground surface and the depth into the bedrock to the underside of the proposed footing as encountered at the boreholes are summarized below.



FOUNDATION REPORT – SHAWANAGA RIVER SBL BRIDGE STRUCTURE – HIGHWAY 69 G.W.P. 5111-07-00

| Borehole Location Within South Abutment | Borehole | Ground Surface Elevation (m) | Bedrock Surface Elevation (m) | Depth to Bedrock Below Existing Ground Surface (m) | Depth of Bedrock Excavation Required to Underside of Proposed Footing (m) |
|---|----------|------------------------------|-------------------------------|--|---|
| West Side | B5-17 | 207.1 | 206.9 | 0.2 | 2.8 |
| | B5-18 | 205.2 | 204.2 | 1.0 | 0.1 |
| Centre | B5-19 | 208.3 | 208.3 | 0.0 | 4.2 |
| East Side | B5-20 | 209.7 | 209.4 | 0.3 | 5.3 |
| | B5-21 | 209.1 | 209.0 | 0.1 | 4.9 |

Based on the borehole results, the underside of the proposed spread footing at the south abutment is up to about 5.3 m below the existing bedrock surface, therefore, bedrock excavation up to about 5.3 m deep and overburden material/cobbles/topsoil excavation up to about 1.0 m deep will be required to reach the proposed founding level at Elevation 204.1 m. In general, the bedrock at or immediately below the proposed founding level at the borehole locations is of very poor to excellent quality, but is typically fair to good quality, with the RQD generally ranging from about 22 percent to 100 percent. However, the quality of the bedrock may be variable in places and any loose or fractured bedrock encountered at the founding level will need to be sub-excavated and removed prior to footing construction and replaced with mass concrete. Recommendations for excavation of the bedrock and sequence of construction are provided in Section 6.10.

The preferred foundation option at the south abutment is a spread footing founded either directly on the granite gneiss bedrock at Elevation 204.1 m or on mass concrete over bedrock because of the bedrock strength characteristics, ease of footing construction and overall long-term stability of the footing (i.e. minimal potential for undermining of the foundation over the life of the structure). All mass concrete construction should be in accordance with OPSS.PROV 904 (*Concrete Structures*).

6.3.1.2 North Abutment

The existing ground surface in the area of the proposed north abutment footprint varies from about Elevations 199.6 m to 198.7 m, from the western edge to the eastern edge. In general, a deposit of topsoil/peat up to about 0.9 m thick and/or an organic sandy silt deposit up to about 1.2 m thick were encountered at the ground surface at the borehole locations advanced within the proposed footprint. The organic deposits are underlain by an upper deposit of very loose to compact sand and silt to sandy silt, which is then underlain by an up to approximately 1.8 m thick deposit of soft to firm clayey silt to clay. The clayey silt to clay deposit is in turn underlain by a lower deposit of loose to compact sand and silt to sand and gravel. The native soils are underlain by granite gneiss bedrock.

Based on the GA Drawing and following discussions with MRC, the underside of the 8 m wide footing for the north abutment is proposed to be at about Elevation 201.4 m (considering a 1.2 m thick footing) founded on a Granular 'A' or 'B' Type II pad.



FOUNDATION REPORT – SHAWANAGA RIVER SBL BRIDGE STRUCTURE – HIGHWAY 69 G.W.P. 5111-07-00

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 of the proposed footing as encountered at the boreholes are 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 Underside of Proposed Footing (m) |
|---|----------|------------------------------|-------------------------------|--|--|
| West Side | B5-02 | 198.7 | 195.9 | 2.8 | 5.5 |
| | B5-03 | 199.4 | 196.6 | 2.8 | 4.8 |
| Centre | B5-01 | 199.5 | 197.0 | 2.5 | 4.4 |
| East Side | B5-04 | 199.1 | 196.7 | 2.4 | 4.7 |
| | B5-05 | 199.6 | 197.8 | 1.8 | 3.6 |

In general, based on the borehole results within the footprint of the north abutment, the proposed elevation of the underside of the spread footing is up to 2.7 m above the existing ground surface and between about 3.6 m and 5.5 m above the bedrock surface. The bedrock at the borehole locations is of poor to good quality with RQD ranging from about 45 percent to 88 percent. The existing overburden at this location is not suitable for support of the footing and will have to be removed and replaced by a Granular 'A' or 'B' Type II pad prior to footing construction; therefore overburden excavation up to about 2.8 m will be required at the foundation footprint. The plan area extent of sub-excavation to bedrock should be taken as 1 m beyond the abutment footing plus a downward extension at 1H:1V granular pad slope to bedrock.

In order to found the footing at the proposed Elevation 201.4 m, the construction of a granular pad up to about 5.5 m thick will be required at the north abutment and a minimum 1.8 m of soil cover will be required for frost protection of the spread footing (refer to Section 6.4.5 (Frost Protection) for details).

Given the thickness of the granular pad, the depth of overburden excavation and the normal water level of 198.5 m expected at this site, dewatering and temporary shoring would be required to construct the granular pad in-the-dry. If the granular pad is constructed in-the-dry, the granular pad could be constructed with Granular 'A' material.

Alternatively, in order to reduce the dewatering and temporary shoring requirements at the north abutment, a portion of the granular pad below the normal water level of Shawanaga River could be constructed in-the-wet by end-dumping the granular material in lifts (up to a maximum elevation of 198.5 m) with the remaining portion of the granular pad above water constructed using proper compaction in-the-dry. If this approach is employed, the granular pad should be constructed of Granular 'B' Type II to the underside of the footing. The maximum depth of Granular 'B' Type II that should be placed in-the-wet for the un-compacted portion of the granular pad is 2.5 m above the lowest bedrock surface. In addition, the highest elevation of the groundwater table at the north abutment and the maximum allowable river level at the time of construction at this site should be 198.5 m. An Operational Constraint (OC) providing a method specification for the construction of the uncompacted portion of the granular pad in-the-wet and the allowable water level for construction is included in Appendix C.



The earth excavation for the granular pad and construction of the granular pad in-the-wet should be carried out simultaneously. In addition, the construction for the pad and the adjacent north embankment should be constructed concurrently to allow for the granular pad to be preloaded to reduce immediate settlement that may result from the portion of the Granular 'B' Type II pad placed below water (refer to Section 6.7.2.4).

Given the elevation of the high water level for the Shawanaga River at this site (which is at Elevation 202.2 m), it is recommended that suitable erosion protection be provided to the granular pad and footing during construction and on/around the front and side slopes of the approach fills to above the elevation of the footing founding level. An Operational Constraint (OC) providing a method specification for the excavation and construction of the granular pad as well as the protection of the granular pad with Rip-Rap during and immediately upon completion of construction of the granular pad is included in Appendix C.

Consideration could be given to founding the spread footing on mass concrete over bedrock, however, given the elevation of the bedrock surface at the north abutment and the proposed footing elevation, the volume of mass concrete required would be significant and very expensive.

The recommended foundation option at the north abutment is to found the footing at about Elevation 201.4 m and construct the footing on a partially compacted Granular 'B' Type II pad over bedrock after sub-excavation of all organic deposits and native soils within the north abutment footprint.

6.3.2 Geotechnical Axial Resistances / Reactions

The following summarizes the factored geotechnical axial resistances at Ultimate Limits States (ULS) for spread footings placed on properly prepared granite gneiss bedrock or mass concrete (founded on the properly prepared bedrock) or on compacted Granular 'A' or 'B' Type II pad (founded on the properly prepared bedrock). For spread footings founded on the properly prepared and inspected bedrock or on mass concrete on bedrock, the geotechnical reaction at SLS for 25 mm of settlement will be greater than the factored geotechnical axial resistance at ULS since the granite gneiss bedrock is considered to be an unyielding material; as such, ULS conditions will govern for this foundation type.

| Foundation Element | Founding Alternative for the Proposed Spread Footings | Factored Geotechnical Axial Resistance at Ultimate Limit States (ULS) | Geotechnical Reaction at Serviceability Limit States (SLS) for 25 mm of Settlement |
|--------------------|--|---|--|
| South Abutment | Spread Footing on Granite Gneiss Bedrock or on Mass Concrete placed directly on Bedrock ¹ | 10,000 kPa | N/A |
| | Spread Footing on compacted Granular 'A' pad placed directly on Bedrock ² | 900 kPa | 350 kPa |
| North Abutment | Spread Footing on Granular 'B' Type II pad placed directly on Bedrock ³ | 750 kPa | 250 kPa |

Note: ¹ Assumes mass concrete will have compressive strength greater than 25 MPa.

² For Granular 'A' pad constructed in-the-dry.

³ For Granular 'B' Type II constructed in-the-wet. The maximum depth of Granular B Type II that can be placed in-the-wet is 2.5 m and the maximum allowable river level at the time of construction is 198.5 m.



The geotechnical resistances provided above are given for loads that 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 is at least 25 MPa.

For footings founded on a Granular 'A' or 'B' Type II pad, the top edge of the spread footing must be located at least 1.0 m (measured perpendicular) from the crest of the adjacent pad slope. If the footing is any closer than 1.0 m from the crest of the slope, the allowable capacity of the footing will decrease due to edge effects.

Following excavation of the overburden and bedrock, and prior to placing any concrete and/or constructing the engineered fill granular pad (for Granular 'A' pad constructed in-the-dry), 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 of the mass concrete/concrete footing and proper compaction of the granular pad on the bedrock. Field inspection should be carried out when the excavation is dry and in accordance with OPSS 902 (*Excavating and Backfilling*).

A check on the sliding resistance between the mass concrete or concrete footing on the bedrock and the concrete footing on the compacted Granular 'A' or 'B' Type II pad 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 mass concrete or concrete footings, compacted Granular 'A' pad and the bedrock should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction, $\tan \delta$, for the various interface materials is summarized below.

| Interface Materials | Coefficient of Friction ($\tan \delta$) |
|---|---|
| Mass Concrete or Concrete Footing on Bedrock | 0.70 |
| Concrete Footing on Compacted Granular 'A' or 'B' Type II Pad | 0.58 |

The values presented above represent unfactored values; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating 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 into the bedrock may be considered in the same way as dowels embedded into the concrete. This assumes that the Uniaxial Compressive Strength (UCS) of the grout will be similar to that of the 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. Depending on the selected founding elevation for the footings at this site, it is recommended that the upper portion of the bedrock where RQDs are less than 50% (i.e. poor or very poor quality) should be ignored in the calculation of required bond length.

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 Penetration 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 granite gneiss bedrock at the south abutment at this site (i.e. all loose or shattered rock to be removed prior to construction), frost susceptibility is not an issue.

For the north abutment footing founded on the compacted Granular 'A' or 'B' Type II pad, the footing should be provided with a minimum of 1.8 m of soil cover for frost protection. In addition, the following should also be noted for the design of a spread footing founded on a compacted Granular 'A' or 'B' Type II pad:

- The required thickness of conventional soil cover for frost protection of the footing (1.8 m) is measured perpendicular from the face of the abutment slope to the edge of the underside of the footing (it is not simply a vertical dimension when the footing is adjacent to a slope).
- Where the Granular 'A' or 'B' Type II pad is constructed with a 1 horizontal to 1 vertical (1H:1V) side slope, it is typical to cover the pad slope with a 2 horizontal to 1 vertical (2H:1V) conventional earth slope (to promote vegetation growth). Alternatively, the pad slope will need to be covered with suitably sized rip-rap for erosion protection.
- If adequate soil cover cannot be provided for the footing, rigid styrofoam insulation shall be installed to compensate for the lack of cover and provide protection from frost action. In this case, the MTO has adopted an equivalent of 25 mm of styrofoam to 300 mm of soil cover.

6.3.5 Footing Set-Back from Rock Faces

Steep bedrock outcrops are present in the area of the south abutment foundation and the south abutment footing will be situated (perched) on the bedrock above the Shawanaga River, requiring bedrock excavation to create a new rock face in front of the proposed south abutment. The footing 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 protected such that the integrity of the rock face/founding rock is maintained. In this regard, the footing should be located away from the rock face at least a distance as defined by an imaginary line projected at 0.5 horizontal to 1 vertical (0.5H:1V) from the toe of the rock face up to the underside of the footing and not closer than 2 m from the edge of 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 included in Appendix C.

6.4 Pile Foundations

At this site, deep (pile) foundations comprised of drilled steel pipe piles or casings advanced into the bedrock are considered the preferred alternative for support of the south and north piers. The depth of water as well as the variable thickness and composition of the overburden deposits at the piers would pose significant challenges to the successful construction of spread footings and as such shallow foundations are not recommended at these locations.

6.4.1 Pile Options

As noted in Section 6.2, a number of different types of piles have been considered for support of the pier foundations, the details of which are presented and advantages and disadvantages as well as relative costs and risks/consequences compared in Table 3.

The thickness of the overburden at the piers ranges from about 0 m to 4.7 m and is comprised of very loose/very soft organic sands and peats, overlying loose sands and silts and very soft clays. As such, the contribution of the overburden soils to the axial and lateral capacity of the piles will be negligible and all pile foundations will have to be advanced to and into the bedrock.

Based on the GA Drawing, the underside of the pile cap for the south and north piers is proposed to be at about Elevation 196.2 m (considering a 1.5 m thick pile cap). The following sections outline the expected range of pile lengths at each of the pier foundation units as well as the geotechnical axial resistances for the different pile types, giving due consideration to the quality of the bedrock at each location.

6.4.1.1 South Pier (Pier 1)

The details of the river bed elevation, bedrock surface elevation, thickness of overburden, depth to bedrock below the underside of the pile cap and minimum recommended length of pile at the south pier (based on conditions encountered at the boreholes) are summarized below.



FOUNDATION REPORT – SHAWANAGA RIVER SBL BRIDGE STRUCTURE – HIGHWAY 69 G.W.P. 5111-07-00

| Borehole Location Within South Pier | Borehole | River Bed Elevation (m) | Bedrock Surface Elevation (m) | Thickness of Overburden (m) | Depth to Bedrock Below Underside of Pile Cap (m) | Minimum Embedment Length of Pile into Bedrock* (m) | Minimum Length of Pile Below U/S Pile Cap (m) |
|-------------------------------------|----------|-------------------------|-------------------------------|-----------------------------|--|--|---|
| West Side | B5-13 | 193.4 | 191.7 | 1.7 | 4.5 | 3.7 | 8.2 |
| | B5-14 | 193.0 | 188.7 | 4.3 | 7.5 | 1.5 | 9.0 |
| Centre | B5-12 | 192.7 | 190.6 | 2.1 | 5.6 | 2.6 | 8.2 |
| | B5-15 | 193.5 | 191.5 | 2.0 | 4.7 | 4.3 | 9.0 |
| East Side | B5-16 | 193.1 | 193.1 | 0.0 | 3.1 | 1.9 | 5.0 |

Note: * Minimum embedment length of pile into bedrock (not including uncased socket length required for some options) to provide the axial geotechnical resistance presented in Section 6.4.2. Additional embedment length into bedrock may be required to satisfy the lateral loads on the pier; to be determined by structural engineer per Section 6.4.4.

The minimum length of pile below the underside of the pile cap summarized above has been selected considering the variation in the RQD of the bedrock at each of the boreholes and the requirement to embed the steel pipe piles at least 1.5 m into the good quality rock as identified by the higher RQD zones. Given the size of the footprint of the pier foundation, and the nature of the sloping bedrock surface, it is recommended that the piles be installed with a common length and that the tips of all the piles in the group supporting the pier all be terminated at about the same lowest elevation. As such, at this location, it is recommended that all piles be terminated at Elevation 187.2 m, or lower depending on the minimum embedment length to satisfy the lateral loads on the pier (refer to Section 6.4.4).

6.4.1.2 North Pier (Pier 2)

The details of the river bed elevation, bedrock surface elevation, thickness of overburden, depth to bedrock below the underside of the pile cap and minimum recommended length of pile at the north pier (based on the conditions encountered at the boreholes) are summarized below.



FOUNDATION REPORT – SHAWANAGA RIVER SBL BRIDGE STRUCTURE – HIGHWAY 69 G.W.P. 5111-07-00

| Borehole Location Within South Pier | Borehole | River Bed Elevation (m) | Bedrock Surface Elevation (m) | Thickness of Overburden (m) | Depth to Bedrock Below Underside of Pile Cap (m) | Minimum Embedment Length of Pile into Bedrock* (m) | Minimum Length of Pile Below U/S Pile Cap (m) |
|-------------------------------------|----------|-------------------------|-------------------------------|-----------------------------|--|--|---|
| West Side | B5-08 | 192.8 | 188.2 | 4.6 | 8.0 | 1.5 | 9.5 |
| | B5-09 | 194.4 | 191.5 | 2.9 | 4.7 | 1.7 | 6.4 |
| Centre | B5-07 | 193.6 | 192.1 | 1.5 | 4.1 | 1.5 | 5.6 |
| East Side | B5-10 | 196.2 | 195.1 | 1.1 | 1.1 | 1.5 | 2.6 |
| | B5-11 | 193.7 | 193.3 | 0.4 | 2.9 | 2.5 | 5.4 |

Note: * Minimum embedment length of pile into bedrock (not including uncased socket length required for some options) to provide the axial geotechnical resistance presented in Section 6.4.2. Additional embedment length into bedrock may be required to satisfy the lateral loads on the pier; to be determined by structural engineer per Section 6.4.4.

The minimum length of pile below the underside of the pile cap summarized above has been selected considering the variation in the RQD of the bedrock at each of the boreholes and the requirement to embed the steel pipe piles at least 1.5 m into the good quality rock as identified by the higher RQD zones. Given the size of the footprint of the pier foundation, and the nature of the sloping bedrock surface, it is recommended that the piles be installed with the following common lengths and pile tip elevations across the pier foundation, or lower depending on the minimum embedment length to satisfy the lateral loads on the pier (refer to Section 6.4.4):

| Location Within Pile Cap | Minimum Pile Length Below U/S Pile Cap (m) | Pile Tip Elevation (m) |
|---------------------------|--|------------------------|
| Western 1/3 of Foundation | 9.5 | 186.7 |
| Eastern 2/3 of Foundation | 7.0 | 189.2 |

6.4.2 Geotechnical Axial Resistances / Reactions

As noted in Section 6.2, several deep (pile) foundations options have been considered for support of the pier foundations, including:

- steel pipe piles (0.457 m diameter, socketed into bedrock with reinforcing cage or central bar(s), filled with 30 MPa concrete);
- drilled steel casings (0.609 m diameter, socketed into bedrock, filled with 30 MPa concrete);
- micropiles (0.273 m diameter, socketed into bedrock, 2-#18 central bars, filled with 30 MPa grout);
- H-piles (HP 310x110, placed into 0.609 m diameter cored bedrock sockets, backfilled with 30 MPa concrete); and,
- caissons (0.9 m diameter, socketed into bedrock, 30 MPa concrete).

The following summarizes the factored axial geotechnical resistance and reaction for the different foundation options at the pier locations.



FOUNDATION REPORT – SHAWANAGA RIVER SBL BRIDGE STRUCTURE – HIGHWAY 69 G.W.P. 5111-07-00

| Pile Foundation Alternative | Factored Axial Geotechnical Resistance at ULS* (kN) (Compression) | Axial Geotechnical Reaction at SLS** (kN) (Compression) | Factored Axial Geotechnical Resistance at ULS (Tension) | Uncased Socket Details for Tensile Resistance |
|--|---|---|---|--|
| 0.457 m diameter Steel Pipe Piles | 3,000 | N/A | 2,000 kN | $L_s = 2 \text{ m (min)}$ $D_s = 0.457 \text{ m}$ |
| | | | 250 kN/m | $D_s = 0.10 \text{ m}$ |
| | | | 350 kN/m | $D_s = 0.15 \text{ m}$ |
| 0.609 m diameter Drilled Steel Casings | 3,700 | N/A | 2,700 kN | $L_s/D_s = 3.0 \text{ (min)}$ |
| 0.273 m diameter Micropiles | 2,000 | N/A | 1,500 kN | $L_s = 2.5 \text{ m (min)}$ |
| HP 310x110 Steel Piles | 2,000 | N/A | 1,000 kN | $L_s = 1.5 \text{ m (min)}$ |
| 0.9 m diameter Concrete Caissons | Not Recommended | | | |

Note: * Structural Capacity of pile must be checked.

** The SLS reaction for 25 mm of settlement is greater than the ULS resistance and therefore ULS governs.

L_s = Length of socket

D_s = Diameter of socket

The installation details for each of the above pile types are briefly summarized in the following sections.

For all options, the recommended embedment lengths into bedrock are the minimum required to satisfy the axial loads provided above. Additional embedment length into bedrock may be required to satisfy lateral loads on the pier (s) and is to be determined by the structural engineer (refer to Section 6.4.4).

6.4.2.1 0.457 m Diameter Steel Pipe Piles

To be installed by rotary duplex drilling using a sacrificial ring bit on the bottom of the pipe or casing and a Down-the-Hole (DTH) hammer to clean out the centre of the pile and also to create a socket within the bedrock below the bottom of the pile to develop tensile capacity. All pipe piles/steel casings would be embedded a minimum depth of 1.5 m below the top of the bedrock. However, depending on how high the tensile loads are, the following alternatives can be considered for the rock socket details below the bottom of the pipe/casing:

- 0.457 m diameter uncased socket
- 0.10 m diameter uncased socket
- 0.15 m diameter uncased socket

The tensile capacities associated with each of the above are summarized in the table in Section 6.4.2.

For each of these options, central reinforcement, in the form of a reinforcing cage or Dywidag bar(s) would be required to be installed within the pile and into the rock socket. Efforts should be made to clean/flush the base of the pipe/casing and rock socket prior to tremie concreting. In the event that cleaning/flushing cannot be relied upon, it is recommended that the depth of the rock socket(s) for all of the above alternatives be lengthened by a minimum of 0.5 m.



6.4.2.2 0.609 m Diameter Drilled Steel Casings

To be installed by rotary duplex drilling using a sacrificial ring bit on the bottom of the casing and a Down-the-Hole (DTH) hammer to clean out the centre of the pile and also to create a socket within the bedrock below the bottom of the casing. In order to develop sufficient capacity in compression and tension, an uncased rock socket with a Length/Diameter (L/D) ratio of at least 3 is recommended. It is recommended that the casing be installed a minimum of 1.5 m into the bedrock, but additional cased embedment length may be required to satisfy the lateral loads on the piers. Because the pile would develop its axial capacity based on the shear resistance in the rock socket wall (i.e. between the concrete and bedrock) and not rely on end-bearing at the base of the socket, the requirement to properly clean and inspect the base of the socket would be lessened. A rebar cage would have to be lowered through the casing and into the rock socket prior to concreting.

6.4.2.3 0.273 m Diameter Micropiles

To be installed by rotary duplex drilling using a sacrificial ring bit on the bottom of the casing and a Down-the-Hole (DTH) hammer to clean out the centre of the pile and also to create a socket within the bedrock below the bottom of the casing. In order to develop sufficient capacity in compression and tension, the uncased portion of the rock socket would have to have a minimum length of 2.5 m below the bottom of the micropile casing. It is recommended that the cased portion of the micropiles be installed a minimum of 1.5 m into the bedrock, but additional cased embedment length may be required to satisfy the lateral loads on the piers. Because the pile would develop its axial capacity based on the shear resistance in the rock socket wall (i.e. between the concrete and bedrock) and not rely on end-bearing at the base of the socket, the requirement to properly clean and inspect the base of the socket would be lessened. Central reinforcement, in the form of two (2) #18 Dywidag bars would be installed through the casing and into the rock socket prior to grouting.

6.4.2.4 HP 310x110 Steel Piles

To be placed inside of 0.609 m diameter sockets drilled into the bedrock a minimum of 1.5 m by rotary duplex drilling using a temporary casing with ring bit and a Down-the-Hole (DTH) hammer to clean out the centre of the pile and also to create the socket within the bedrock. Upon completion of drilling of the rock socket, the H-pile would be lowered through the temporary casing and rest on the bedrock and the socket would be backfilled with tremie concrete prior to removing the casing. Given the sloping nature of the bedrock surface, the variable thickness of overburden and the variable length between the underside of the pile cap and top of bedrock, it would not be possible to install steel H-piles by driving to refusal on bedrock at these locations.

6.4.2.5 0.9 m Diameter Concrete Caissons

To be advanced into bedrock using permanent casings and conventional caisson drilling equipment. It is anticipated that difficulties would arise when attempting to seal the large diameter casings in the very strong, sloping and fractured bedrock at some locations. If a proper seal cannot be formed, there will be difficulties forming the rock socket below the casing. Given the risks associated with the larger diameter drilling, the use of caissons is not recommended at this site.



6.4.3 Downdrag Load (Negative Skin Friction)

Soft clayey silt and clay strata up to about 2 m thick were encountered in the boreholes advanced at the pier foundation units. However, given that no filling is proposed to be carried out within the river in the vicinity of the piers and considering that the fills to be placed in the north abutment area are more than 10 m away from the closest pier foundation, consolidation and settlement of the clay at the piers should not occur and as such, no downdrag loads are expected on the pile foundations.

6.4.4 Resistance to Lateral Loads

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

Lateral loading can be resisted fully or partially by the use of battered piles. For vertical piles, the resistance to lateral loading will have to be derived from the soil and/or the bedrock in front of the piles. At this site, given that the overburden soils are very weak and of variable thickness (i.e. non-existent at some locations, but in general less than about 2 m thick) it is recommended that the contribution of the soils to the lateral resistance of the piles be ignored and only the lateral resistance provided by the bedrock be relied upon for design.

Given the long free-length (i.e. unsupported length) of pile through the river water column at the locations of the piers and considering that the river water level fluctuates at the site, the potential for steel section loss and the requirements for a minimum sacrificial thickness at the pile wall will need to be considered as part of the structural design.

The following sections provide geotechnical recommendations to assist with the evaluation of the lateral capacity (or passive resistance) as well as the lateral response (or deflection) of the piles. The information provided should be used by the structural engineer to evaluate the minimum required embedment of the piles into bedrock to satisfy the lateral loads on the piers.

6.4.4.1 Lateral Pile Capacity – Bedrock

The passive resistance of the portion of the pile socketed into bedrock has been analysed using a Rock Mass Rating (RMR) profile for the bedrock based on the RQD values and UCS strength values measured on the rock core recovered from the boreholes. The ultimate lateral capacity of the bedrock versus elevation is outlined below.



| Depth below Bedrock Surface (m) | Ultimate Lateral Capacity (MPa) |
|--|--|
| 0 – 2.5 | 2.5 |
| Below 2.5 | 7.5 |

The capacity per metre length of pile within the bedrock (in kN) can be determined by multiplying the Ultimate Lateral Capacity, given above, by the diameter of the pile.

A geotechnical resistance factor of 0.5 should be applied to the calculated values of the ultimate lateral/passive resistance of the pile, based on the methods described above, for the section of the pile within the bedrock.

6.4.4.2 Lateral Pile Deflections – Bedrock

The lateral load response of a single pile may be calculated using subgrade reaction theory and the coefficient of horizontal subgrade reaction, k_h , (kPa/m) for the granitic gneiss bedrock. It is anticipated that the rock will remain in the elastic range for the design loading; however, this assumption should be checked once the design is finalized and the maximum lateral loads on a single pile are available. For loading within the elastic range, closed form solutions are applicable for the estimation of the coefficient of horizontal subgrade reaction, k_h .

Based on the lateral rock mass elastic modulus of the 'upper' granitic gneiss bedrock (from 0 m to 2.5 m below the bedrock surface where the lower RQD values are present), $E_h = 2,000$ MPa, and a Poisson's ratio of 0.2, the lateral rock mass spring constant is given by:

$$k_h = \frac{4\pi(1-\nu)E_h}{(3-4\nu)(1+\nu)\ln(r_o/r_i)} \frac{1}{\ln(10)} = \frac{4\pi(0.8)(2,000)}{(2.2)(1.2)} \frac{1}{\ln(10)} = 3,300 \text{ MN/m/m}$$

r_i = radius of caisson

r_o = radius of 'zero' deformation; typically 10 to 15 caisson diameters.

Based on the lateral rock mass elastic modulus of the 'lower' granitic gneiss bedrock, $E_h = 10,000$ MPa (below a depth of 2.5 m from the bedrock surface where the relatively higher RQD values are present), and a Poisson's ratio of 0.2, the lateral rock mass spring constant is given by:

$$k_h = \frac{4\pi(1-\nu)E_h}{(3-4\nu)(1+\nu)\ln(r_o/r_i)} \frac{1}{\ln(10)} = \frac{4\pi(0.8)(10,000)}{(2.2)(1.2)} \frac{1}{\ln(10)} = 16,500 \text{ MN/m/m}$$

6.4.4.3 Group Effects

Group action for lateral loading should also be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction (NAVFAC, 1982) in the direction of loading by a reduction factor, R , as follows:



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

Where a pile group is oriented perpendicular to the direction of loading, group action may be considered by reducing the coefficient of horizontal subgrade reaction (NAVFAC, 1982) by a reduction factor R as follows:

| Pile Spacing Perpendicular to Direction of Loading d = Pile Diameter | Horizontal Subgrade Reaction Reduction Factor, R |
|--|--|
| 4 d | 1.00 |
| 1 d | 0.50 |

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

6.4.5 Frost protection

Based on the GA Drawing, the underside of the pile cap at the piers is proposed to be located about 2.3 m below the surface of the river. For this configuration, no frost protection measures are required.

6.5 Seismic Site Coefficient

6.5.1 Site Coefficient

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

6.5.2 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 percent in 50 years). According to Table 4.1 of the CHBDC, this site is located in Seismic Performance Zone 1 and the corresponding site-specific zonal acceleration ratio, A , is 0.05.



Given this assessment and the fact that that 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*, no seismic analysis is required for structures located in Seismic Performance Zone 1.

6.6 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, and the drainage conditions behind the walls. As discussed in Section 6.5.2, 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 | Coefficients of Static Lateral Earth Pressure | |
|-----------|----------------------|---|---------------|
| | | At-Rest, K_o | Active, K_a |
| Rock Fill | 19 kN/m ³ | 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*.

6.7 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 Elevations 208.1 m and 207.5 m, respectively.

Within the limits of the proposed south approach area, the existing ground surface varies from about Elevation 212.3 m (as encountered at the borehole within the approach area about 20 m south of the south abutment) to about Elevation 205.2 m (at the western portion of the embankment immediately behind the abutment). Bedrock outcrops are present at the south approach area extending towards the south abutment and the south pier. Given these existing conditions, up to about 2.9 m of new embankment fill will be required at the western edge of the approach immediately behind the abutment. The remainder of the approach will consist of rock cuts up to about 4.2 m deep (or more to accommodate the pavement structure thickness).

Within the limits of the proposed north approach embankment, the existing ground surface varies from about Elevation 200.7 m (as encountered at the borehole within the approach area about 20 m north of the north abutment) to about Elevation 198.7 m recorded at the boreholes advanced at the western portion of the embankment immediately behind the north abutment). In general, new embankment fill up to about 8.8 m high will be required at the north approach.

At the south approach, where some filling will be required immediately behind the abutment, bedrock is either outcropping or covered with a thin layer of topsoil or cobbles / rock fragments. It is anticipated that any organic soils will be removed as part of the bedrock excavation required for abutment construction, in addition to all topsoil and organic matter to be stripped from the approach areas prior to fill placement. Construction of the remainder of the south approach area will require excavation through the existing bedrock outcrops and fill placement will be limited to that required for the roadway structure itself immediately behind the abutment area.

At the north approach, the overburden in the embankment fill area is variable, comprising topsoil/peat, organic sandy silt, sand and silt to sandy silt to sand and gravel (in places) and clayey silt to clay over bedrock. All



topsoil, organic matter and clayey silt to clay are considered unsuitable as subgrade for the new embankment fill and should be stripped from the plan limits of the approach area and the exposed subgrade proof-rolled prior to placement of new fill. However, it is noted that the clayey silt to clay was encountered in the boreholes advanced in the abutment area and as such the majority of this strata should be removed as part of granular pad and footing construction.

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

6.7.1 Stability

Analyses were performed on the critical (i.e. highest or thickest) fill sections of the proposed new approach embankment to assess the stability for the proposed heights and geometries. Critical sections include those through the side slopes of the new approaches and the back face of the north abutment, assuming that the cohesive deposit present immediately behind or adjacent to the abutment is not completely removed as recommended above.

6.7.1.1 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 employed in the analysis.

6.7.1.2 Parameter Selection

The soils that will remain in place below the proposed approaches (following stripping of organics and clayey silt to clay deposit) consist primarily of thin layers of non-cohesive native soils (i.e. loose to compact sand and silt to sandy silt and dense to very dense sand and gravel at the north approach) underlain by bedrock.

For the native deposits, the effective stress parameters employed 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 non-cohesive deposit, the values selected for the parameters were generally based on the lower relative density as assessed from the SPT “N”-values.

At all areas, the analyses assume that all organic soils, and clayey silt to clay soils have been removed from below the embankment footprint prior to construction of the new approach fills. The groundwater level measured in the piezometer installation in Borehole B5-18 at the south abutment was at about Elevation 202.8 m, within



FOUNDATION REPORT – SHAWANAGA RIVER SBL BRIDGE STRUCTURE – HIGHWAY 69 G.W.P. 5111-07-00

the bedrock, on August 26, 2009 and the ground water level measured in the piezometer installation in Borehole B5-01 at the north abutment was at about Elevation 199.0 m, within the overburden, on August 26, 2009. For the stability analysis, it is assumed that the groundwater level is located at the ground surface.

The following summarizes the simplified stratigraphy and the associated strengths and unit weights employed for the different soil types in both approach embankment areas. It is understood that rock fill is to be used for construction of the new approach embankments, 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 proposed side slopes for the Shawanaga River NBL bridge structure approaches). A discussion on different fill types, with respect to stability, is provided in Section 6.7.1.4.

| 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 |
| | Very Loose to Silty Sand | 19 | -- | 28 |
| | Cobbles/Rock Fragments | 19 | -- | 30 |
| | New Rock Fill | 19 | -- | 40 |
| | Existing Rock Fill | 19 | -- | 35 |
| North Approach | Loose to Compact Sand and Silt to Sandy Silt | 19 | -- | 28 |
| | Dense to Very Dense Sand and Gravel | 19 | -- | 30 |
| | Soft to Firm* Clayey Silt to Clay | 17 | 22 | -- |
| | Stiff Clay* | 19 | 57 | -- |
| | | | | |

Note: * To be removed from below the embankment footprint prior to construction of the new approach fills.

6.7.1.3 Results of Analysis

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



FOUNDATION REPORT – SHAWANAGA RIVER SBL BRIDGE STRUCTURE – HIGHWAY 69 G.W.P. 5111-07-00

| Location | Embankment Height at Critical Section (m) | Rock Fill Option | |
|--------------------------------|---|-----------------------------------|-----------------------------|
| | | Recommended Side Slope Profile | Minimum Factor of Safety |
| South Approach STA 17+653.5 | ≤2.8* | 1.25H : 1V | ≥ 1.3 |
| North Approach STA 17+753.5 | 10.9** | | |

Note: * Majority of South Approach area is constructed through a rock cut.

** Including required depth of excavation.

At the north approach, a critical section across the back face of the north abutment and the wildlife passage was analysed, assuming that the cohesive deposit encountered within the foundation footprint was left in place and extends along the embankment footprint. The result of the analysis indicates that the embankment will have a factor of safety of 1.3 or greater for a deep seated, global failure surface that would impact the operation of the roadway (see Figure 1). However, for settlement consideration, it is recommended that all cohesive deposit within the north abutment and embankment footprint be removed prior to construction.

6.7.1.4 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. It is understood that terrain constraints within the approaches will not accommodate the flatter slopes required for granular fill, and as such granular fill is not an option at this location.

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 existing Highway 69 and the new north approach embankment alignment. Rock fill would likely be available locally (from the rock cuts required in the south approach area and beyond). 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.



6.7.2 Settlement

Some relatively small settlement of the proposed rock fill material will occur during construction of the embankment due to compression of the new rock fill itself. The majority of rock fill settlement will occur within the first year of construction, but some relatively small settlement will continue to occur over the life of the embankment.

6.7.2.1 Methodology

To estimate the magnitude of the expected settlements, analyses were carried out on the critical sections of the proposed approach embankments using hand and spreadsheet calculations.

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.

The sources of settlement were considered to include:

- Immediate settlement of the native granular soils (where present); and,
- Self-weight compression of the embankment fill materials (short-term and long-term).

6.7.2.2 Parameter Selection

The immediate compression of the non-cohesive 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 6.7.1.2. The analyses performed assume that all topsoil, surficial organic soils and clayey silt to clay deposits will be removed prior to embankment construction and that rock fill will be used for the new embankment.

The piezometric conditions required in the analyses were based on the groundwater levels noted during drilling and measured in the piezometer installations. In general, the groundwater level was assumed to be at the ground surface.

The following summarizes the simplified stratigraphy, unit weights and deformation parameters employed for the foundation soils in the critical sections of 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.



| Approach Embankment | Foundation Soil | Thickness (m) | Unit Weight (kN/m³) | Estimated Deformation Property (E) (MPa) |
|--------------------------------|--|----------------------|---------------------------------------|---|
| South Approach STA 17+653.5 | N/A ¹ | N/A ¹ | N/A ¹ | N/A ¹ |
| North Approach STA 17+753.5 | Loose to Compact Sand and Silt to Sandy Silt | 0.7 | 19 | 10 |

Note: ¹ South Approach is predominantly in a rock cut; where localized filling is required on the west side immediately behind the abutment area, all thin overburden soils will be stripped prior to filling; there are no foundation soils at the critical section.

6.7.2.3 Settlement of Foundation Soils

At the south approach, where the overburden is very thin or non-existent (due to bedrock either at shallow depth or outcropping) and where the majority of the approach will be constructed in a rock cut, there will be no settlement of the foundations soils.

At the north approach, at the critical section where stripping of organics and removal of clayey silt to clay deposit up to about 2.1 m deep below ground surface is required (immediately behind the abutment area), resulting in a total embankment fill up to about 10.9 m high, up to about 15 mm of settlement of the foundation soils are estimated to occur. However, given the non-cohesive nature of the foundation soils, this settlement is expected to occur during construction.

6.7.2.4 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 embankments 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 6.7.2.3. 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



FOUNDATION REPORT – SHAWANAGA RIVER SBL BRIDGE STRUCTURE – HIGHWAY 69 G.W.P. 5111-07-00

be nominal and the magnitude is estimated to be up to about 0.5 percent for effective height of the rock fill embankment height up to about 5 m, and up to about 0.75 percent for effective height of the rock fill embankment between 5 m and 10 m. The estimated short-term settlement of rock fill for the approach embankments is presented below.

| Approach Embankment | Maximum New Embankment Height¹ (m) | Estimated Short-Term Settlement of Rock Fill (mm) |
|--------------------------------|--|--|
| South Approach STA 17+653.5 | 2.8 | 15 |
| North Approach STA 17+753.5 | 8.8 + 2.1 = 10.9 | 80 |

Note: ¹ Includes additional fill required after removal of maximum depth of topsoil/organics and silty clay deposit.

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 10 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 present below.

| Approach Embankment | Maximum New Embankment Height¹ (m) | Estimated Long-Term Settlement of Rock Fill (mm) |
|--------------------------------|--|---|
| South Approach STA 17+653.5 | 2.8 | <5 |
| North Approach STA 17+753.5 | 8.8 + 2.1 = 10.9 | 10 |

Note: ¹ Includes additional fill required after removal of maximum depth of topsoil/organics and silty clay deposit.

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.

In order to meet 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, a minimum preload period of 150 days is required for the north approach rock fill embankment to allow for the majority of the short-term settlement to occur.

As detailed in Section 6.3.1.2, the recommended foundation option for the north abutment is a spread footing founded on a partially compacted Granular 'B' Type II pad over bedrock. Given that approximately 2.5 m thick portion of the Granular 'B' Type II pad will be placed below water and un-compacted, it is recommended that the extent of the preload area for the north approach embankment be extended southward to cover the Granular 'B' Type II pad up to the limit and maximum elevation of the final grade for the abutment front slope, to reduce any immediate settlement that may result from the uncompacted portion of the granular pad. The recommended minimum duration of the preload area over the granular pad area is 30 days.



6.7.2.5 *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 the minimum required platform widening on major highways (i.e. including Highway 69), for some subgrade conditions, is 1 m or 2 m per side, unless the preferred mitigation option eliminates uncertainty regarding embankment settlement/performance (i.e. full sub-excavation to bedrock and backfilling with granular material).

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 profile of the pavement structure (4H:1V), but cannot be less than the minimum platform widening requirement as described above.

The required platform widening for the north approach embankment should be a minimum of 2 m given the immediately adjacent swamp to the north and greater thickness of rock fill. Platform widening is not required at the south approach area given that the embankment fill is being placed directly on bedrock. Further, the embankment widening component on the north approach should transition within the 20 m length of the approach embankment into the greater widening recommended for the overall embankment crossing the swamp to the north of the north approach embankment (refer to Swamp and Pond Crossing Foundation Investigation and Design Report by Golder Associates Ltd. (2011)).

6.8 Subgrade Preparation and Embankment Construction

The layers of topsoil/peat, organic sandy silt and silty clay deposit should be stripped from the plan limits of the proposed works and the subgrade soils should be proof-rolled prior to new fill placement. In addition, all existing overburden soils within the footprint of the north abutment foundation area should be sub-excavated to the bedrock surface and replaced with compacted Granular ‘A’ or ‘B’ Type II. The extent of sub-excavation for the Granular ‘A’ or ‘B’ Type II pad should extend a minimum of 1 m beyond the perimeter at the underside of the north abutment footing and extend downwards to the bedrock surface at a slope no steeper than 1H:1V.

The following sections provide details on the recommendations for subgrade preparation and embankment construction.

6.8.1 Removal of Organic Materials

Based on the information from the boreholes obtained during the field investigation, layers of topsoil/peat up to about 0.9 m thick and an organic sandy silt deposit up to about 1.2 m can be expected in some areas of the new approaches. At the north approach areas, a cohesive deposit up to about 1.8 m thick was encountered in the



boreholes adjacent to the north approach area (advanced immediately at the front of the north abutment), which extends to a depth of about 2.4 m below ground surface. All topsoil/organic materials and the cohesive deposit (clayey silt to silty clay to clay) deposit should be stripped from the plan limits of the approach embankment areas prior to fill placement for the new embankment(s).

6.8.2 Embankment 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 into 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).

In accordance with OPSD 202.010 (*Slope Flattening*) and as required by MTO, 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. Given that the maximum height of the rock fill approach embankment at the critical section 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.

6.8.3 Temporary Shoring / Roadway Protection

At the south abutment, given that the excavations for the new foundation construction will be primarily through bedrock and considering that the new foundation areas are located more than 10 m away from the proposed Shawanaga River NBL Bridge, temporary shoring and/or roadway protection is not anticipated to be required in this area.

Temporary shoring may however be required to support the overburden soils at the north abutment during the excavations to expose the bedrock surface prior to construction of the compacted Granular 'A' or 'B' Type II pad, depending on the sequence of construction relative to the new north approach embankment for the Shawanaga River NBL Bridge. Design and construction of temporary shoring in these areas should take into account the following:

- Strong to very strong granite gneiss bedrock;
- Irregular / sloping surface of bedrock; and,
- Groundwater level at the north abutment area up to about 3.1 m above the lowest bedrock surface.

Each of the above will affect the choice of shoring type for design as well as the methodology for shoring installation during construction. A method of providing lateral fixity to the toe of the shoring at the bedrock surface (i.e. toe pins) may also be required.

Temporary excavation support systems should be designed and constructed in accordance with OPSS 539 (*Temporary Protection Systems*). The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS 539 (*Temporary Protection Systems*).



6.9 Design and Construction Considerations

6.9.1 Overburden Excavation

Based on the proposed elevations of the underside of the abutment footings and the recommendation to remove all overburden down to the top of bedrock prior to mass concrete/footing and granular pad construction, excavations for the bridge abutment foundations will extend through the soils and depths summarized below. The elevation of the groundwater table as measured in the piezometers installed at the south and north abutments are also summarized below for comparison with the founding elevations and/or maximum depth of excavation required.

| Foundation Element | Proposed / Recommended Underside of Footing Elevation (m) | Depth(s) of Excavation to Founding Level (m) | Anticipated Soil Types Encountered | Groundwater / River Level Elevation (m) |
|--------------------|---|--|--|---|
| South Abutment | 204.1 | 1.0 (Soil) | Very Loose Silty Sand, Cobbles / Rock Fragments | 202.8 |
| | | 5.3 (Rock) | -- | |
| North Abutment | 201.4 (195.9)* | 2.4 (Soil – East Side) | Very Loose Peat / Organic Sandy Silt, Compact Sand and Silt, Firm to Stiff Clayey Silt to Clay | 199.0 |
| | | 2.8 (Soil – West Side) | Very Loose Organic Sandy Silt, Very loose to Compact Sand and Silt to Sandy Silt, Soft Clay | |

Note: * 195.9 m indicates lowest elevation of the bedrock surface (as encountered in the boreholes) to be excavated to or exposed prior to granular pad construction.

The overburden soils requiring excavation as part of foundation construction are considered Type 3 and Type 4 soils according to the 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 the latest edition of the Ontario Regulation 213 OHSA for Construction Projects (as amended by Ontario Regulation 443).

6.9.2 Control of Groundwater and Surface Water

The elevation of the groundwater table in comparison to the footing founding level at the abutments, as well as the lowest elevation of the bedrock surface required to be exposed prior to placement of mass concrete is summarized in Section 6.9.1. Based on this information, the following comments are provided regarding the requirements for un-watering / groundwater control.

At the south abutment, the measured elevation of the groundwater is below (about 1.3 m) the proposed founding level. As such, groundwater control is not anticipated to be required at this location.

At the north abutment, the measured elevation of the groundwater is about 3.2 m above the lowest elevation of the bedrock surface to be exposed prior to placement of mass concrete or concrete footings or granular pad



construction. Un-watering / groundwater control may be required at this location depending on the foundation option adopted at this location as described in Section 6.3.1.2. If the abutment footing is founded on mass concrete or on a granular pad with the full depth of the granular pad constructed in-the-dry, dewatering is required to control groundwater at this location. However, if the abutment is founded on a granular pad in which the lower portion of the granular pad is constructed in-the-wet with the remaining portion constructed in-the-dry; it is anticipated that groundwater control can be accomplished by pumping from well filtered sumps to lower the groundwater table to Elevation 198.5 m prior to construction at this location.

Surface water should be directed away from the excavations at all times. It should be noted that groundwater level in the area is subject to seasonal fluctuations, precipitation events and run-offs from Shawanaga River (Refer to Sections 4.3.4, 4.4.1, 4.5.1 and 4.6.7); however, 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.

6.9.3 Cofferdam Construction

Construction of the pile caps for the in-water piers will require some form of cofferdam. As noted in Section 6.2, it is anticipated that conventional cofferdam construction (i.e. the use of interlocking sheet piles driven through the overburden to form a water tight box structure) will be difficult at this site because of the following challenges at the pier locations:

- Depth of water (ranging from 3.0 m to 6.6 m) plus maximum thickness of overburden (up to 4.6 m) will require cofferdams up to about 11.2 m deep which will result in very high pressures acting on the sides and base of the sheet piles.
- Presence of cobbles in the overburden creating obstructions that could impede the installation of sheet piles.
- Variable thickness of overburden ranging from 0 m (i.e. bedrock exposed at riverbed) to 4.6 m, and variable composition (organics, loose sands/silts and very soft clays) offers little in the way of toe fixity at the base of the sheet piles; the use of toe pins/anchors into bedrock would likely be required to achieve adequate fixity and stability.
- Strong to very strong, sloping bedrock (up to about 30°) that will not allow penetration of the driven sheet piles, resulting in stepping of the sheets along the bedrock surface, leaving gaps and therefore make sealing at the base of the cofferdam difficult.
- Requirement to excavate or 'muck-out' the base of the cofferdam in-the-wet (due to challenges associated with creating a proper seal at bedrock surface) and pour a tremie concrete plug in-the-wet prior to dewatering. If mucking is not successful in removing all of the overburden materials and producing a clean, debris-free bedrock surface, any sediment trapped below the tremie concrete plug will affect the geotechnical resistance(s) – axial and lateral – of the spread footings and could affect the long-term performance of the structure.

Given the above, the use of conventional cofferdams for pier construction at this site would likely carry high costs and as well as risks to the successful completion.

As such, it is recommended that consideration be given to using prefabricated cofferdam(s), constructed with pre-drilled holes and steel tube sleeves through the base large enough to accommodate the foundation pile



elements. These types of cofferdams could be floated and then anchored into place, act as a template during pile installation and upon completion of piling, could be backfilled with concrete to form the pile cap(s).

6.9.4 Obstructions

Cobbles/rock fragment were encountered at the south abutment and south pier during borehole drilling. Conventional excavation equipment should be suitable for the majority of the excavation through the organic deposits and overburden soils. However, the presence of cobbles / boulders and rock fragments 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.

6.9.5 Erosion Protection

The requirements for erosion protection of the north abutment/approach front and side slopes shall be evaluated by the hydraulic engineer in consideration of the river hydraulics, in order to protect the approach embankment and abutment foundations from undermining/erosion by the river water flow. At a minimum, the erosion protection should consist of R-50 Rip-Rap or Rock Protection material as specified in OPSS.PROV 1004 (*Aggregates – Miscellaneous*), as appropriate.

6.9.6 Permit To Take Water

At the north abutment, the groundwater level as measured at the time of the investigation was about 3.1 m above the lowest elevation of the bedrock surface. However, it is known that the water level at the adjacent river can fluctuate widely depending on the time of the year, precipitation events and amount of snow melt which will influence the groundwater level on the adjacent river banks. Considering the above and given that the type of shoring and method of dewatering selected by the Contractor for the north abutment is unknown, a permit to take water will be required for this site.

6.10 Recommendations for Rock Excavations and Blasting

6.10.1 Rock Excavation

It should be noted that the bedrock at the site is generally classified as strong (R4) to very strong (R5). The three (3) Unconfined Compression (UC) tests carried out on bedrock core samples, measured Uniaxial Compressive Strength (UCS) between about 58 MPa and 118 MPa. Bedrock of such strength 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 (i.e. at the south abutment) 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, the south abutment footing will be founded on the bedrock outcrop above the Shawanaga River and 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, 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.

6.10.2 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 Quality Verification Engineer (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. T. Veronica Ayetan, P. Eng., and 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.



FOUNDATION REPORT – SHAWANAGA RIVER SBL BRIDGE STRUCTURE – HIGHWAY 69 G.W.P. 5111-07-00

Report Signature Page



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

TVA/CN/JPD/JMAC/jl



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

\\golder.gds\gal\mississauga\active\2007\1111\07-1111-0029 - mrc - hwy 69 four-laning -\report\final\10 - shawanaga river sbl bridge\07-1111-0029-10 fidr
15mar12 highway 69 shawanaga river sbl bridge structure.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.

Golder Associates Ltd. 2011. *Foundation Investigation and Design Report, Swamp and Pond Crossing – Phase 1, Highway 69 Four Laning, From 0.4 km North of Highway 7182 (Shebeshekong Road) Northerly 11 km*, Ministry of Transportation, Ontario, G.W.P. 5403-05-00.

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

Ontario Provincial Standard Specification:

| | |
|----------------|---|
| OPSS 120 | General Specification for Use of Explosives. |
| OPSS 501 | Construction Specification for Compacting. |
| OPSS 539 | Construction Specification for Temporary Protection Systems. |
| OPSS 902 | Construction Specification for Excavating and Backfilling - Structures. |
| OPSS.PROV 206 | Construction Specification for Grading. |
| OPSS.PROV 904 | Construction Specification for Concrete Structures. |
| OPSS.PROV 1010 | Material Specification for Aggregates. |
| OPSS.PROV 1004 | Material Specification for Aggregates – Miscellaneous. |

Ontario Water Resources Act:

Ontario Regulation 903 Wells.



TABLES



FOUNDATION REPORT – SHAWANAGA RIVER SBL BRIDGE STRUCTURE HIGHWAY 69 G.W.P. 5111-01-00; W.P. 5188-06-01

TABLE 1
EVALUATION OF FOUNDATION ALTERNATIVES – SOUTH ABUTMENT
G.W.P. 5111-07-00 / W.P. 5188-06-01

| Options | Rank | Advantages | Disadvantages | Relative Costs | Risks/Consequences |
|--|------|--|---|--|--|
| Spread Footing on 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"> Removal of overburden (containing boulders) up to about 1 m thick and bedrock excavation up to about 6.9 m deep is required. Careful and controlled blasting of variable bedrock surface will be required to achieve a level surface for footing construction. Mass concrete may be required if over-blasting occurs. May require steel dowels to anchor footing (or mass concrete) to improve lateral resistance. Allows for semi-integral abutment design only. | <ul style="list-style-type: none"> Lower cost in comparison to pile foundation option. Additional cost for mass concrete if over-blasting occurs. | <ul style="list-style-type: none"> Additional bedrock excavation may be required if encountered at different/higher elevations than that identified in boreholes. Placement of mass concrete or 'dental' concrete may be required if controlled blasting practices not followed. |
| H-piles (HP310x110) installed within Bedrock Trench (backfilled with concrete at base) | NP | <ul style="list-style-type: none"> Allows for integral abutment design. Negligible post construction settlement. | <ul style="list-style-type: none"> Removal of overburden and deeper depth of bedrock excavation required (more than 6.9 m deep) in order to excavate trench to achieve minimum pile lengths. | <ul style="list-style-type: none"> Higher cost than shallow foundation option. Additional costs associated with deeper bedrock excavation to create trench in bedrock. | <ul style="list-style-type: none"> Bedrock trench excavation destabilizing face of rock outcrop if controlled blasting not employed. |



FOUNDATION REPORT – SHAWANAGA RIVER SBL BRIDGE STRUCTURE HIGHWAY 69 G.W.P. 5111-01-00; W.P. 5188-06-01

TABLE 1
EVALUATION OF FOUNDATION ALTERNATIVES – SOUTH ABUTMENT
G.W.P. 5111-07-00 / W.P. 5188-06-01

| | | | | | |
|------------------------------------|----|---|--|--|---|
| Spread Footing on Granular 'A' Pad | NR | <ul style="list-style-type: none"> ■ Mass concrete not required. Granular 'A' used as filler between bedrock surface and underside of footing. | <ul style="list-style-type: none"> ■ 1 m minimum thickness of Granular 'A' pad required below footing otherwise differential settlement and stress concentrations may occur leading to cracking of footing. ■ Deeper bedrock excavation required as compared with Footing on Bedrock option to ensure minimum 1 m thick granular pad constructed below footing. ■ Allows for semi-integral abutment design. | <ul style="list-style-type: none"> ■ Lower relative costs compared with piled foundation option. ■ Additional costs for additional bedrock excavation. | <ul style="list-style-type: none"> ■ Must ensure proper compaction of Granular 'A' to minimise post construction settlement. |
|------------------------------------|----|---|--|--|---|

Reviewed By: JPD/JMAC



FOUNDATION REPORT – SHAWANAGA RIVER SBL BRIDGE STRUCTURE HIGHWAY 69 G.W.P. 5111-01-00; W.P. 5188-06-01

TABLE 2
EVALUATION OF FOUNDATION ALTERNATIVES – NORTH ABUTMENT
G.W.P. 5111-07-00 / W.P. 5188-06-01

| Options | Rank | Advantages | Disadvantages | Relative Costs | Risks/Consequences |
|--|------|--|---|--|---|
| Spread Footing on Granular 'A' Pad | 1 | <ul style="list-style-type: none"> ■ Mass concrete not required. | <ul style="list-style-type: none"> ■ Approximately up to about 3.0 m excavation of the overburden material required to expose bedrock. ■ Dewatering, cleaning of bedrock surface and placement of Granular 'A' engineered fill required within shored and dewatered excavation (i.e. cofferdam). ■ Installation of cofferdam on sloping bedrock and achieving adequate seal around base to restrict seepage inflow may be difficult. ■ Lower geotechnical axial resistance at ULS and SLS as compared with footing on mass concrete option. ■ Allows for semi-integral abutment design only. | <ul style="list-style-type: none"> ■ Lower relative costs compared with piled foundation option and footing on mass concrete option. ■ Additional costs required for installation of temporary shoring and dewatering. | <ul style="list-style-type: none"> ■ Must ensure proper compaction of Granular 'A' to minimise post construction settlement. ■ Groundwater seepage into the cofferdam resulting in inability to achieve adequate and consistent density (compaction) of the Granular 'A' pad. |
| H-piles (HP310x110) installed within 0.6 m diameter Pre-Drilled Holes and Socketted into Bedrock | 2 | <ul style="list-style-type: none"> ■ Allows for integral abutment design. ■ Shored excavation and dewatering not required. ■ Negligible post construction settlement. | <ul style="list-style-type: none"> ■ Temporary liners required for groundwater control and support though overburden where rock socketting required. ■ Special drilling equipment required for creating 0.6 m diameter socket. | <ul style="list-style-type: none"> ■ Additional costs associated with special equipment required for pre-drilling 0.6 m diameter holes in bedrock. | <ul style="list-style-type: none"> ■ Difficulty achieving adequate seal of temporary liners into hard, sloping bedrock when creating bedrock sockets. |



FOUNDATION REPORT – SHAWANAGA RIVER SBL BRIDGE STRUCTURE HIGHWAY 69 G.W.P. 5111-01-00; W.P. 5188-06-01

TABLE 2
EVALUATION OF FOUNDATION ALTERNATIVES – NORTH ABUTMENT
G.W.P. 5111-07-00 / W.P. 5188-06-01

| | | | | | |
|---|----|--|---|---|--|
| Caisson 0.6 m diameter socketed into bedrock | 3 | <ul style="list-style-type: none"> Shored excavation and dewatering not required. Negligible post construction settlement. High axial capacity in compression and tension; smaller number of pile elements required. | <ul style="list-style-type: none"> Does not allow for integral abutment design. Temporary liners required for groundwater control and support though overburden where rock socketting required. Special drilling equipment required for creating 0.6 m diameter socket. Concrete for caissons would have to be placed by tremie methods below the water level. | <ul style="list-style-type: none"> Additional costs associated with special equipment required for pre-drilling 0.6 m diameter holes in bedrock. | <ul style="list-style-type: none"> Difficulty achieving adequate seal of temporary liners into hard, sloping bedrock when creating bedrock sockets. |
| Spread Footings on Mass Concrete over Bedrock | NP | <ul style="list-style-type: none"> Relative ease of construction. Reduced bedrock excavation (as compared with pile option). Higher geotechnical axial resistance at ULS and SLS as compared with footing on granular pad option. Negligible post-construction settlement. | <ul style="list-style-type: none"> Approximately up to about 3 m excavation of the overburden material required to expose bedrock. Dewatering, cleaning bedrock surface and placement of mass concrete required within shored and dewatered excavation (i.e. cofferdam). Installation of cofferdam on sloping bedrock and achieving adequate seal around base to restrict seepage inflow may be difficult. Allows for semi-integral abutment design only. May require steel dowels to anchor mass concrete to sloping bedrock surface. | <ul style="list-style-type: none"> Relatively higher cost in comparison to footing on granular pad and pile foundation options due to significant quantity of mass concrete required. Additional costs required for installation of temporary shoring/cofferdam and dewatering. | <ul style="list-style-type: none"> Potential of not being able to properly seal cofferdam and achieving full dewatering within the cofferdam. |

Reviewed By: JPD/JMAC



FOUNDATION REPORT – SHAWANAGA RIVER SBL BRIDGE STRUCTURE HIGHWAY 69 G.W.P. 5111-01-00; W.P. 5188-06-01

TABLE 3
EVALUATION OF FOUNDATION ALTERNATIVES – CENTRAL PIERS
G.W.P. 5111-07-00 / W.P. 5188-06-01

| Options | Rank | Advantages | Disadvantages | Relative Costs | Risks / Consequences |
|---|------|---|---|---|---|
| Steel Pipe Piles 0.457 m dia. (advanced into bedrock with DTH drilling and filled with concrete) | 1 | <ul style="list-style-type: none"> ■ High axial capacity in compression; governed by structural capacity of pile element. ■ Larger diameter pile element offers greater section stiffness, especially where unsupported through water column. ■ DTH drilling method offers good chance of advancing pile through variable overburden (including cobbles) and into hard and sloping bedrock. ■ Conventional cofferdam construction (sheet piling driven to bedrock) and dewatering can be avoided if pre-fabricated, floating cofferdam utilized as template for pile installation and form for pile cap. ■ Does not require a drilled uncased rock socket. | <ul style="list-style-type: none"> ■ Negligible axial capacity in tension (since grouted rock socket not constructed at tip of piles), if required. ■ Base of rock socket must be cleaned prior to concreting since pile capacity relying on end-bearing on bedrock. ■ Given thin and variable overburden in river, lateral fixity at base of piles cannot be relied on unless additional efforts (i.e. larger diameter temporary drill casing) employed to ensure proper grouting of annulus between pile and rock socket. ■ Specialized drilling equipment required. ■ Larger equipment required for installation of larger diameter pile elements. ■ Concrete piles would have to be placed by tremie methods below the water level. | <ul style="list-style-type: none"> ■ Higher cost per pile than driven piles or smaller diameter micropiles, but potentially fewer piles required as result of higher axial capacity. ■ Additional costs associated with special drilling equipment. | <ul style="list-style-type: none"> ■ Potential for difficulties cleaning base of rock socket prior to filling pipe pile with concrete. |



FOUNDATION REPORT – SHAWANAGA RIVER SBL BRIDGE STRUCTURE HIGHWAY 69 G.W.P. 5111-01-00; W.P. 5188-06-01

TABLE 3
EVALUATION OF FOUNDATION ALTERNATIVES – CENTRAL PIERS
G.W.P. 5111-07-00 / W.P. 5188-06-01

| Options | Rank | Advantages | Disadvantages | Relative Costs | Risks / Consequences |
|--|------|---|--|---|---|
| Drilled Steel Casings 0.406 m to 0.609 m dia. (socketed into bedrock using DTH drilling, grouted in place and backfilled with grout) | 2 | <ul style="list-style-type: none"> High axial capacity in compression and tension, if required. Larger diameter pile element offers greater section stiffness, especially where unsupported through water column. Cleaning of base of rock socket not required since axial capacity developed based on shaft resistance in socket only. DTH drilling method offers good chance of advancing pile through variable overburden (including cobbles) and into hard and sloping bedrock. Conventional cofferdam construction (sheet piling driven to bedrock) and dewatering can be avoided if pre-fabricated, floating cofferdam utilized as template for pile installation and form for pile cap. | <ul style="list-style-type: none"> Given thin and variable overburden in river, lateral fixity at base of piles cannot be relied on unless additional efforts (i.e. larger diameter temporary drill casing) employed to ensure proper grouting of annulus between pile and rock socket. Specialized drilling equipment required. Larger equipment required for installation of larger diameter pile elements. Requires additional depth of uncased drilling to create rock socket to penetrate through fractured zone. | <ul style="list-style-type: none"> Higher cost per pile than driven piles and/or smaller diameter micropiles, but potentially fewer piles required as result of higher capacity. Additional costs associated with special drilling equipment. | <ul style="list-style-type: none"> If casings not adequately seated and sealed into bedrock, there is a potential for debris and overburden materials impeding rock socket construction. |



FOUNDATION REPORT – SHAWANAGA RIVER SBL BRIDGE STRUCTURE HIGHWAY 69 G.W.P. 5111-01-00; W.P. 5188-06-01

TABLE 3
EVALUATION OF FOUNDATION ALTERNATIVES – CENTRAL PIERS
G.W.P. 5111-07-00 / W.P. 5188-06-01

| Options | Rank | Advantages | Disadvantages | Relative Costs | Risks / Consequences |
|---|------|--|--|---|---|
| Micropiles (0.273 m dia. grouted pile with central reinforcing bar and permanent steel casing seated into bedrock) | 3 | <ul style="list-style-type: none"> Smaller diameter pile elements require smaller drilling equipment for installation. Potentially easier for over-water construction work. Small diameter combined with DTH drilling offers best chance of advancing pile through variable overburden (including cobbles) and into hard and sloping bedrock without difficulties. Conventional cofferdam construction (sheet piling driven to bedrock) and dewatering can be avoided if pre-fabricated, floating cofferdam utilized as template for pile installation and form for pile cap. | <ul style="list-style-type: none"> Specialized drilling equipment required. Lower axial capacity in compression and tension as compared with larger diameter pile elements; larger number of piles required. Given thin and variable overburden in river, lateral fixity at base of piles cannot be relied on unless additional efforts (i.e. larger diameter temporary drill casing) employed to ensure proper grouting of annulus between pile and rock socket. Smaller diameter pile element offers lower section stiffness, especially where unsupported through water column. Specialized detail foundation design required for micropile foundations considering small diameter elements offer lower section stiffness. | <ul style="list-style-type: none"> Lower relative cost per pile due to smaller diameter, but larger number of piles required. Additional costs associated with special drilling equipment. Additional costs required for detail design and load testing during construction. | <ul style="list-style-type: none"> Possible specialized foundation design to achieve required lateral resistance with small diameter piles. Possible requirement to properly grout annulus between casing and bedrock in order to achieve best fixity at base of piles. |



FOUNDATION REPORT – SHAWANAGA RIVER SBL BRIDGE STRUCTURE HIGHWAY 69 G.W.P. 5111-01-00; W.P. 5188-06-01

TABLE 3
EVALUATION OF FOUNDATION ALTERNATIVES – CENTRAL PIERS
G.W.P. 5111-07-00 / W.P. 5188-06-01

| Options | Rank | Advantages | Disadvantages | Relative Costs | Risks / Consequences |
|---|------|--|---|--|---|
| H-piles (HP 310x110) driven to bedrock or installed within 0.6 m diameter pre-drilled holes and socketted into bedrock where minimum pile length cannot be achieved | 4 | <ul style="list-style-type: none"> Relatively straight forward construction where minimum pile length can be achieved. Lateral fixity at base of piles is achieved by placing concrete in annulus between rock socket and H-pile using tremie methods. Conventional cofferdam construction (sheet piling driven to bedrock) and dewatering can be avoided if pre-fabricated, floating cofferdam utilized as template for pile installation and form for pile cap. Typically, shorter length of rock socket required for installation of H-piles as compared to other pile options. | <ul style="list-style-type: none"> Negligible axial capacity in tension except where H-piles are grouted into rock sockets. Minimum pile length for driven H-piles (without rock sockets) will not be achievable at all locations (since distance between underside of pile cap and top of bedrock varies from 1 m to 7.9 m). Rock socket drilling potentially required at >50% of pile locations. Rock points will be required to seat driven piles on hard, sloping bedrock surface. Specialized DTH drilling equipment required to create 0.6 m diameter socket in strong to very strong bedrock. | <ul style="list-style-type: none"> Lower relative cost per pile for H-piles as compared to other pile options. Additional costs associated with special equipment required for pre-drilling 0.6 m diameter holes in bedrock. | <ul style="list-style-type: none"> At locations where piles are not socketted into rock, there is a risk of driven piles being out of alignment / location due to sloping bedrock, or of piles becoming damaged during installation. Potential for difficulties seating drill casing in / on sloping bedrock. Lateral resistance of H-pile section through unsupported water column would have to be considered as part of the design. |



FOUNDATION REPORT – SHAWANAGA RIVER SBL BRIDGE STRUCTURE HIGHWAY 69 G.W.P. 5111-01-00; W.P. 5188-06-01

TABLE 3
EVALUATION OF FOUNDATION ALTERNATIVES – CENTRAL PIERS
G.W.P. 5111-07-00 / W.P. 5188-06-01

| Options | Rank | Advantages | Disadvantages | Relative Costs | Risks / Consequences |
|---|------|---|--|---|---|
| Caissons 0.9 m to 1.2 m diameter socketed into bedrock | NR | <ul style="list-style-type: none"> Very high axial capacity in compression and tension; smaller number of pile elements required. Larger diameter pile element offers very high section stiffness, especially where unsupported through water column. Conventional cofferdam construction (sheet piling driven to bedrock) and dewatering can be avoided if pre-fabricated, floating cofferdam utilized as template for pile installation and form for pile cap, or if raised pile cap employed as part of design. | <ul style="list-style-type: none"> Permanent steel liners/casings would be required for groundwater control and support through overburden and water column. Concrete for caissons would have to be placed by tremie methods below the water level. Will require careful drilling operations as it will be difficult to seat and seal liners/casings and drill large diameter sockets into strong to very strong and sloping gneiss bedrock. Requires specialist foundation contractor and large equipment working over water to install large diameter pile elements. More difficult to advance larger diameter caissons into hard, sloping bedrock than smaller diameter drilled casings or H-pile sockets. | <ul style="list-style-type: none"> Higher relative cost per pile due to large diameter sockets into bedrock, but smaller number of piles required. | <ul style="list-style-type: none"> Potential difficulties seating liners/casings and achieving seal and drilling large diameter bedrock sockets in strong to very strong and sloping rock. Anticipated slow rate of drilling production for larger diameter in strong to very strong bedrock. |



FOUNDATION REPORT – SHAWANAGA RIVER SBL BRIDGE STRUCTURE HIGHWAY 69 G.W.P. 5111-01-00; W.P. 5188-06-01

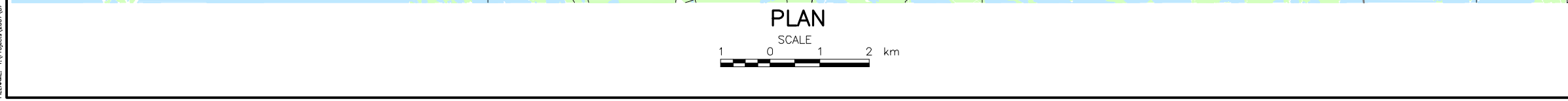
TABLE 3
EVALUATION OF FOUNDATION ALTERNATIVES – CENTRAL PIERS
G.W.P. 5111-07-00 / W.P. 5188-06-01

| Options | Rank | Advantages | Disadvantages | Relative Costs | Risks / Consequences |
|---|------|--|--|--|--|
| Spread Footings on Mass Concrete and / or Bedrock | NR | <ul style="list-style-type: none"> ■ Straightforward construction (following successful completion of cofferdam). ■ High geotechnical axial resistance on strong to very strong bedrock. | <ul style="list-style-type: none"> ■ Variable bedrock surface will require large quantity of mass concrete to achieve level footing. ■ May require steel dowels to anchor mass concrete to sloping bedrock surface. ■ Sub-excavation of cohesionless, water-bearing soils required up to about 11.2 m below river surface level and up to about 4.7 m below surface of river bed. ■ Dewatering, cleaning bedrock surface and concrete placement required within shored and dewatered excavation (i.e. cofferdam) up to about 11.2 m deep. ■ Installation of deep cofferdam on strong to very strong and sloping bedrock and achieving adequate seal around base to restrict seepage inflow will be difficult. Toe pins likely required to achieve fixity of sheet piles to bedrock where overburden is thin or not present. | <ul style="list-style-type: none"> ■ Additional costs required for installation of temporary shoring / cofferdam and dewatering. ■ Potentially higher relative costs compared with piled foundation option(s), due to likely high cost of deep cofferdam construction and sealing. | <ul style="list-style-type: none"> ■ Difficulties achieving proper seal of cofferdam and achieving full dewatering within the cofferdam to allow footing construction in dry condition. |

Reviewed By: JPD/JMAC



DRAWINGS



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

CONT No.
WP No. 5188-06-01



HIGHWAY 69
SITE LOCATION PLAN



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



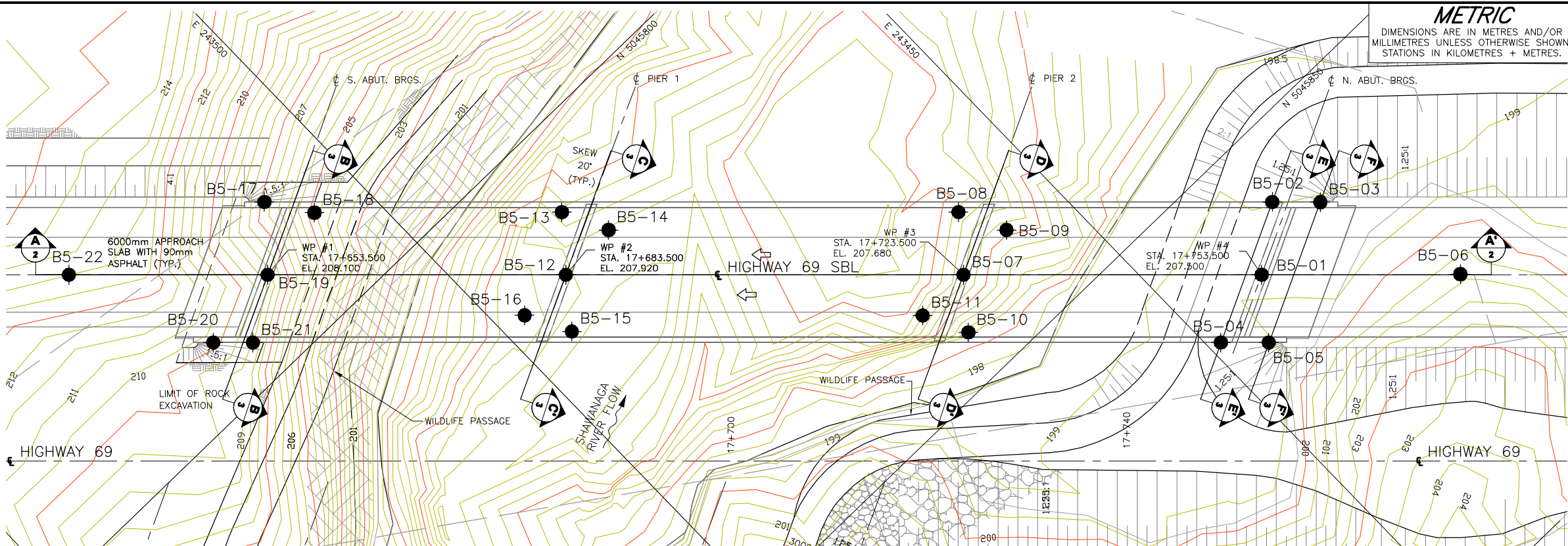
KEY PLAN
NOT TO SCALE



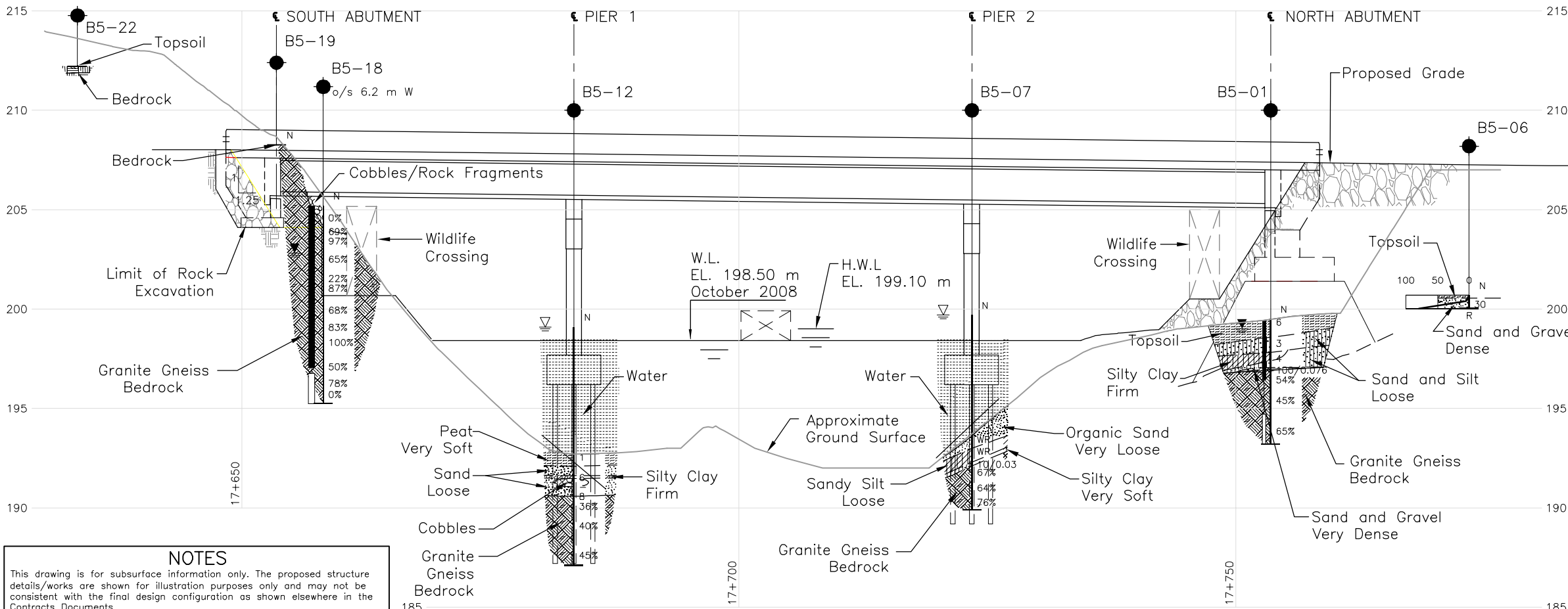
REFERENCE

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

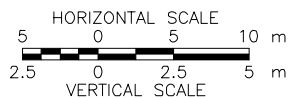
| | | | |
|---------------------|----------|--------------------------|----------------|
| | | | |
| | | | |
| NO. | DATE | BY | REVISION |
| Geocres No. 41H-146 | | | |
| HWY. 69 | | PROJECT NO. 07-1111-0029 | DIST. |
| SUBM'D. VA | CHKD. VA | DATE: Mar. 2015 | SITE: 44-443/2 |
| DRAWN: DD/CD | CHKD. CN | APPD. JPD/JMAC | DWG. 1 |



PLAN



A-A' CENTRELINE PROFILE



NOTES

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

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

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

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

CONT No.
WP No. 5188-06-01

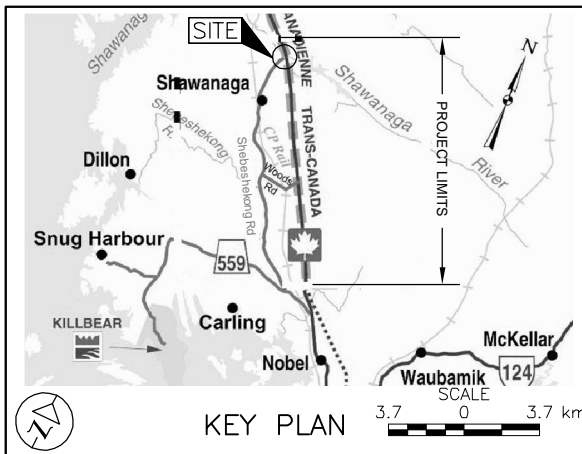
HIGHWAY 69
SHAWANAGA RIVER BRIDGE (SBL)
BOREHOLE LOCATIONS
AND SOIL STRATA



SHEET



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 26/08/2009
- WL upon completion of drilling
- R Refusal

| No. | ELEVATION | CO-ORDINATES | |
|-------|-----------|--------------|----------|
| | | NORTHING | EASTING |
| B5-01 | 199.5 | 5045861.4 | 243441.1 |
| B5-02 | 198.7 | 5045856.8 | 243435.3 |
| B5-03 | 199.4 | 5045860.2 | 243431.8 |
| B5-04 | 199.1 | 5045863.4 | 243448.8 |
| B5-05 | 199.6 | 5045866.7 | 243445.3 |
| B5-06 | 200.7 | 5045875.2 | 243426.7 |
| B5-07 | 199.7 | 5045840.5 | 243462.7 |
| B5-08 | 199.4 | 5045835.6 | 243458.7 |
| B5-09 | 199.2 | 5045840.3 | 243456.5 |
| B5-10 | 199.2 | 5045845.0 | 243466.4 |
| B5-11 | 199.1 | 5045840.6 | 243468.5 |
| B5-12 | 199.1 | 5045812.7 | 243491.5 |
| B5-13 | 199.0 | 5045807.9 | 243487.4 |
| B5-14 | 198.9 | 5045812.5 | 243485.2 |
| B5-15 | 198.8 | 5045817.2 | 243495.0 |
| B5-16 | 198.7 | 5045812.8 | 243497.3 |
| B5-17 | 207.1 | 5045786.4 | 243508.2 |
| B5-18 | 205.2 | 5045790.7 | 243505.3 |
| B5-19 | 208.3 | 5045791.9 | 243513.0 |
| B5-20 | 209.7 | 5045793.0 | 243521.7 |
| B5-21 | 209.1 | 5045795.7 | 243518.8 |
| B5-22 | 212.3 | 5045778.0 | 243527.4 |

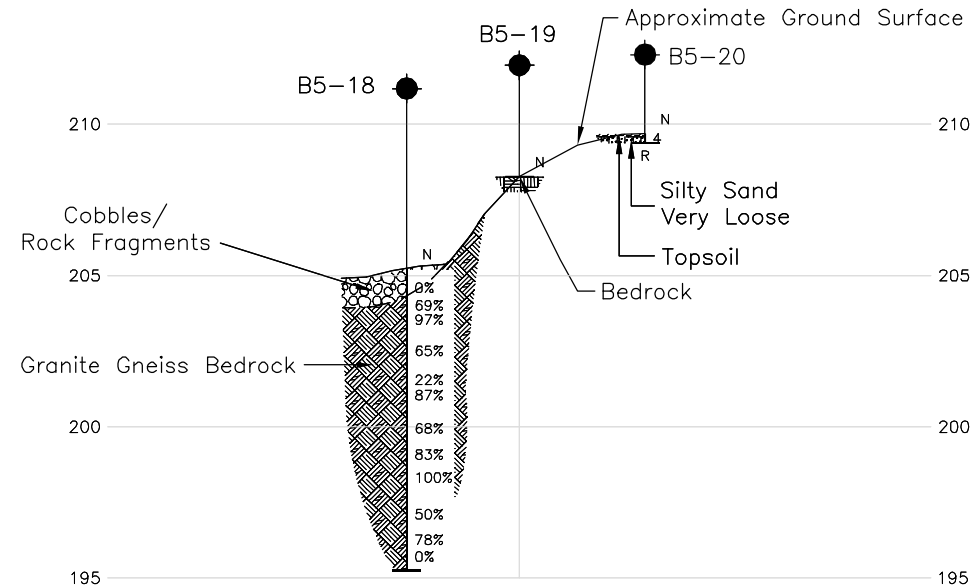
REFERENCE

Base plans provided in digital format by MRC, drawing files 271XB01.DWG, 5271-XPD-SHAWANAGA.dwg, PR # 377-02-00-PR-1.dwg, received October 1, 2007, and by MMM, file S6878-310-001GA.dwg, received February 9, 2015.
Approximate centerline profile based on topographic data provided by MRC, drawing file h6878xb07 Phase-2 contours 1m intervals.dwg, received September 10, 2014.

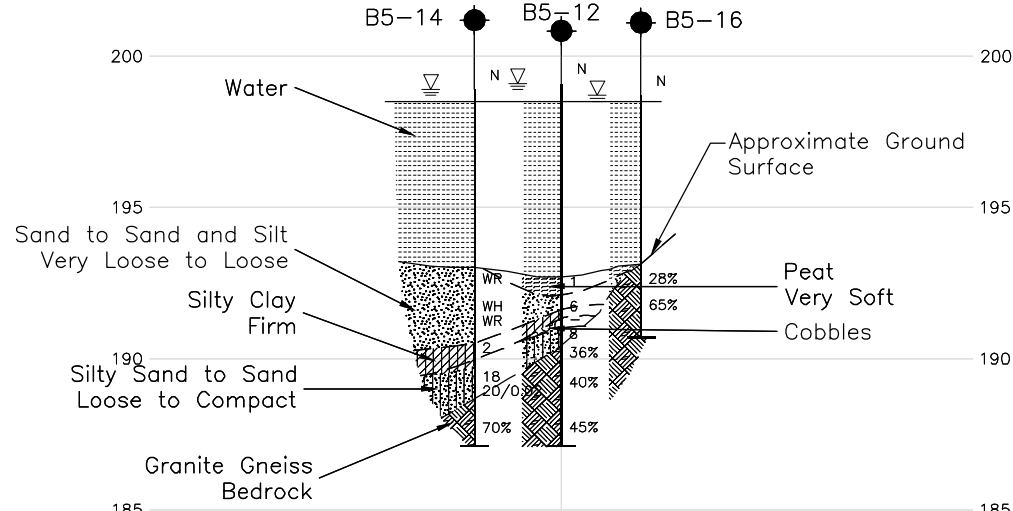
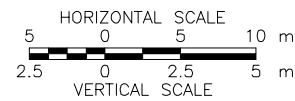
| NO. | DATE | BY | REVISION |
|-----|---------|----------|---------------|
| 1 | 11/2015 | JPD/JMAC | Initial Issue |
| 2 | 11/2015 | JPD/JMAC | Revised |

| | | |
|---------------------|--------------------------|-----------------|
| Geocres No. 41H-146 | PROJECT NO. 07-1111-0029 | DIST. |
| HWY. 69 | CHKD. VA/OK | DATE: Mar. 2015 |
| SUBM'D. VA | CHKD. CN | APPD. JPD/JMAC |
| DRAWN: JS/JFC | CHKD. CN | DWG. 2 |

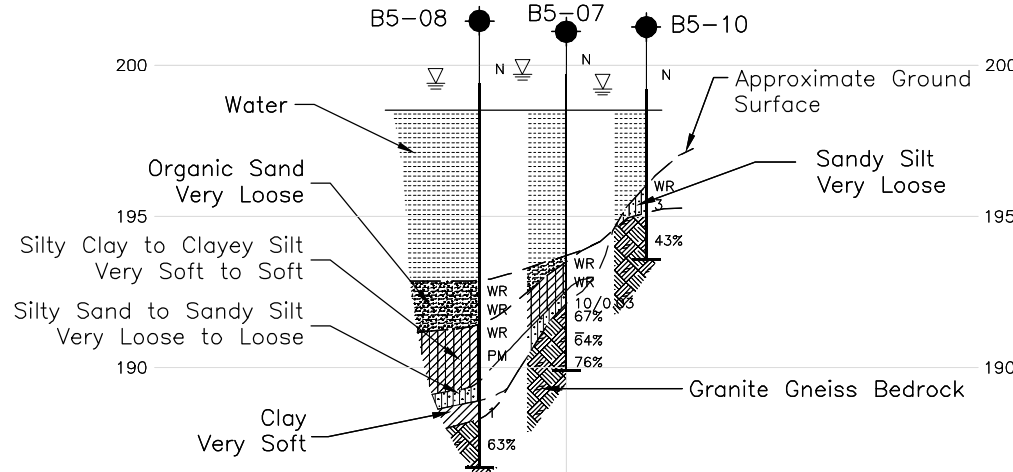
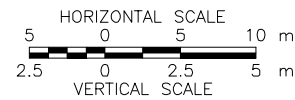




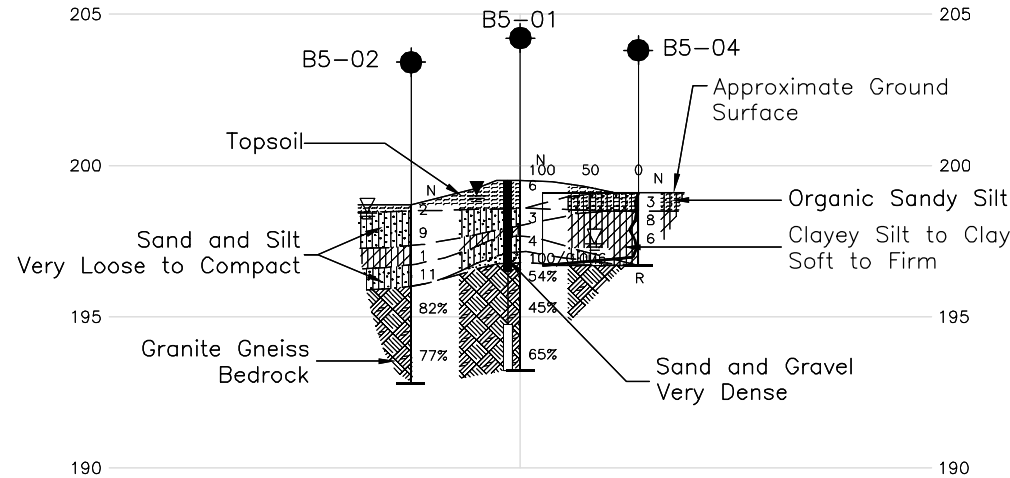
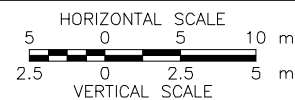
B-B'
2 SOUTH ABUTMENT



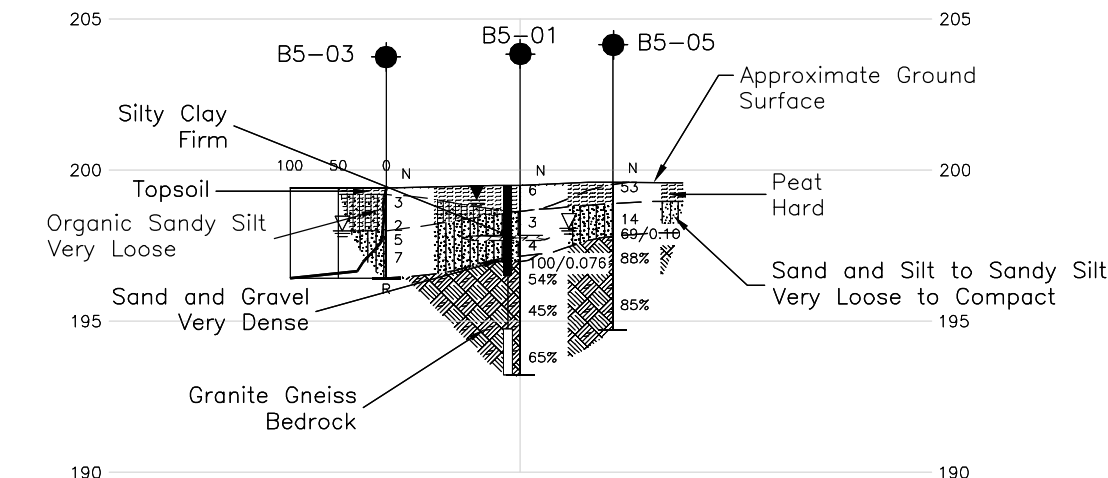
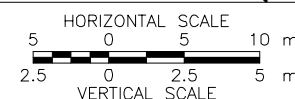
C-C'
2 PIER 1



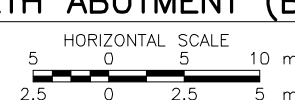
D-D'
2 PIER 2



E-E'
2 NORTH ABUTMENT (FRONT)



F-F'
2 NORTH ABUTMENT (BACK)



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

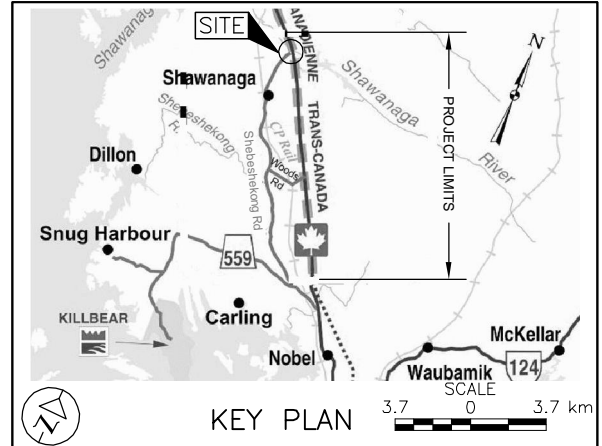
CONT No.
WP No. 5188-06-01

HIGHWAY 69
SHAWANAGA RIVER BRIDGE (SBL)
SOIL STRATA

SHEET



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 26/08/2009
- WL upon completion of drilling
- R Refusal

| No. | ELEVATION | CO-ORDINATES | |
|-------|-----------|--------------|----------|
| | | NORTHING | EASTING |
| B5-01 | 199.5 | 5045861.4 | 243441.1 |
| B5-02 | 198.7 | 5045856.8 | 243435.3 |
| B5-03 | 199.4 | 5045860.2 | 243431.8 |
| B5-04 | 199.1 | 5045863.4 | 243448.8 |
| B5-05 | 199.6 | 5045866.7 | 243445.3 |
| B5-07 | 199.7 | 5045840.5 | 243462.7 |
| B5-08 | 199.4 | 5045835.6 | 243458.7 |
| B5-10 | 199.2 | 5045845.0 | 243466.4 |
| B5-12 | 199.1 | 5045812.7 | 243491.5 |
| B5-14 | 198.9 | 5045812.5 | 243485.2 |
| B5-16 | 198.7 | 5045812.8 | 243497.3 |
| B5-18 | 205.2 | 5045790.7 | 243505.3 |
| B5-19 | 208.3 | 5045791.9 | 243513.0 |
| B5-20 | 209.7 | 5045793.0 | 243521.7 |

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.



| | | | |
|---------------|-------------|----------------|--------------|
| NO. | DATE | BY | REVISION |
| Geocres No. | 41H-146 | | |
| HWY. | 69 | PROJECT NO. | 07-1111-0029 |
| SUBM'D. VA | CHKD. VA/OK | DATE: | Mar. 2015 |
| DRAWN: JS/JFC | CHKD. CN | APPD: JPD/JMAC | DWG. 3 |



FIGURES



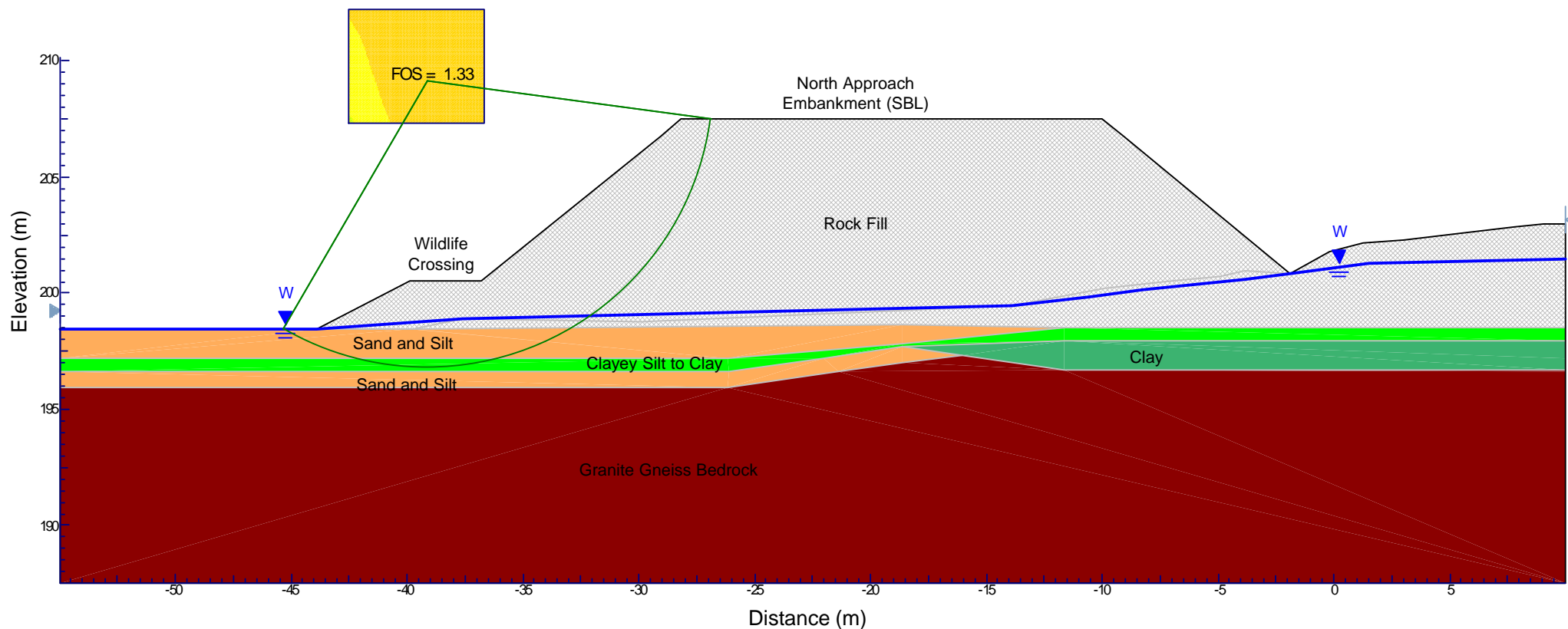
Highway 69 SBL – Shawanaga River SBL Bridge Structure – North Approach Stability Analysis – Station 17+761

Figure 1

| Material Name | Unit Weight (kN/m ³) | Cohesion (kPa) | Friction Angle (degrees) |
|------------------------|----------------------------------|-------------------|--------------------------|
| Rock Fill | 19 | 0 | 40 |
| Sand and Silt | 19 | 0 | 28 |
| Clayey Silt to Clay | 17 | 22 | - |
| Clay | 19 | 57 | - |
| Granite Gneiss Bedrock | 25 | Infinite Strength | |

NOTES:

1. All dimensions are in metres.
2. All rock fill slopes are at 1.25H:1V.
3. Cross-section of the North Approach Embankment provided in pdf format by MRC on June 05, 2014.





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 $= \sigma'_p / \sigma'_{vo}$ |

(d) Shear Strength

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

Notes: 1
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

WEATHERING STATE

Fresh: no visible sign of weathering

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

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

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

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

Completely weathered: rock is wholly decomposed and in a friable condition but the rock 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 B5-01 | | SHEET 1 OF 1 | | METRIC | | | | | | | | | | | | | | | |
|-----------------|--|--------------------|---------|--------------------------|------------|--|-----------------|---|--|------------------------------------|-------------------------------------|-----------------------------------|---------------------|---|------|-----------|-----------|----------|-----|-------|----------|-----|-------|
| W.P. 5188-06-01 | | LOCATION | | N 5045861.4 ; E 243441.1 | | ORIGINATED BY | | MR | | | | | | | | | | | | | | | |
| DIST | | HWY 69 | | BOREHOLE TYPE | | 165 mm O.D. Continuous Flight Solid Stem Augers and NW Casing, Wash Boring | | COMPILED BY | | | | | | | | | | | | | | | |
| DATE | | Geodetic | | DATE | | February 2 & 3, 2009 | | CHECKED BY | | | | | | | | | | | | | | | |
| | | | | | | | | OK | | | | | | | | | | | | | | | |
| 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 | | | | | | | | | | | | | | | |
| 199.5 | GROUND SURFACE | | | | | | | 20 40 60 80 100 | | | | | | | | | | | | | | | |
| 0.0 | TOPSOIL | | 1 | SS | 6 | | 199 | | | | | | 43.6 | | | | | | | | | | |
| 198.6 | | | 2A | | | | | | | | | | | | | | | | | | | | |
| 0.9 | SAND and SILT, trace clay, trace organics, containing rootlets | | 2B | SS | 3 | | | | | | | | 81.3 | 0 42 54 4 | | | | | | | | | |
| 197.8 | Loose Brown to grey Moist | | 3A | | | | 198 | | | | | | | | | | | | | | | | |
| 1.8 | SILTY CLAY Firm Brown Wet | | 3B | SS | 4 | | | | | | | | | | | | | | | | | | |
| 197.1 | | | 3C | | | | | | | | | | | | | | | | | | | | |
| 2.5 | SAND and SILT, trace clay Loose Grey Wet | | 4A | SS | 100/0.07 | | 197 | | | | | | | | | | | | | | | | |
| | SAND and GRAVEL, trace silt, inferred cobbles Very dense Grey Wet | | 1 | RC | REC 97% | | 196 | | | | | | | RQD = 54% | | | | | | | | | |
| | Granite Gneiss (BEDROCK) | | 2 | RC | REC 91% | | 195 | | | | | | | RQD = 45% | | | | | | | | | |
| | Bedrock cored from depths of 2.5 m to 6.3 m | | | | | | | | | | | | | | | | | | | | | | |
| | For bedrock coring details, refer to Record of Drillhole B5-01 | | 3 | RC | REC 100% | | 194 | | | | | | | RQD = 65% | | | | | | | | | |
| 193.2 | END OF BOREHOLE | | | | | | | | | | | | | | | | | | | | | | |
| 6.3 | NOTES: 1. Water level in open borehole at a depth of 1.8 m below ground surface (Elev. 197.7 m) upon completion of overburden drilling. 2. Water level measurements in Piezometer: <table border="1" style="margin-top: 10px;"> <tr> <th>Date</th> <th>Depth (m)</th> <th>Elev. (m)</th> </tr> <tr> <td>16/04/09</td> <td>0.2</td> <td>199.3</td> </tr> <tr> <td>26/08/09</td> <td>0.5</td> <td>199.0</td> </tr> </table> | | | | | | | | | | | | | | Date | Depth (m) | Elev. (m) | 16/04/09 | 0.2 | 199.3 | 26/08/09 | 0.5 | 199.0 |
| Date | Depth (m) | Elev. (m) | | | | | | | | | | | | | | | | | | | | | |
| 16/04/09 | 0.2 | 199.3 | | | | | | | | | | | | | | | | | | | | | |
| 26/08/09 | 0.5 | 199.0 | | | | | | | | | | | | | | | | | | | | | |

SHEET 1 OF 1

DATUM: Geodetic

DRILLING CONTRACTOR: Walker Drilling

CHECKED: OK

| PROJECT 07-1111-0029 | | | RECORD OF BOREHOLE No B5-02 | | | SHEET 1 OF 1 | | | METRIC | | | | | | | | |
|--|---|------------|--|------|------------|----------------------------|--------------------|---|--------|--|--|--|------------------------------------|-------------------------------------|-----------------------------------|---------------------|---|
| W.P. 5188-06-01 | | | LOCATION N 5045856.8 ; E 243435.3 | | | ORIGINATED BY MR | | | | | | | | | | | |
| DIST HWY 69 | | | BOREHOLE TYPE 165 mm O.D. Continuous Flight Solid Stem Augers and NW Casing, Wash Boring | | | COMPILED BY VA | | | | | | | | | | | |
| DATUM Geodetic | | | DATE February 3, 2009 | | | CHECKED BY OK | | | | | | | | | | | |
| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | | | PLASTIC LIMIT W _p | NATURAL MOISTURE CONTENT W | LIQUID LIMIT W _L | UNIT WEIGHT γ | REMARKS & GRAIN SIZE DISTRIBUTION (%) |
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH kPa | | | | | | | | | |
| 198.7 | GROUND SURFACE | | | | | | | | | | | | | | | | |
| 0.0 | TOPSOIL | | | | | | | | | | | | | | | | |
| 0.2 | SAND and SILT, trace to some clay, trace organics, containing rootlets Very loose to loose Brown to brown and grey Wet | | 1 | SS | 2 | | | | | | | | | | | | |
| 197.2 | | | 2 | SS | 9 | | | | | | | | | | | | |
| 1.5 | CLAY Soft Mottled brown Wet | | 3 | SS | 1 | | | | | | | | | | | | |
| 196.6 | | | 4 | SS | 11 | | | | | | | | | | | | |
| 2.1 | SAND and SILT, trace clay Compact Grey Wet | | | | | | | | | | | | | | | | |
| 195.9 | | | | | | | | | | | | | | | | | |
| 2.8 | Granite Gneiss (BEDROCK) | | 1 | RC | REC 93% | | | | | | | | | | | | |
| | Bedrock cored from depths of 2.8 m to 5.9 m For bedrock coring details, refer to Record of Drillhole B5-02 | | 2 | RC | REC 95% | | | | | | | | | | | | |
| 192.8 | | | | | | | | | | | | | | | | | |
| 5.9 | END OF BOREHOLE | | | | | | | | | | | | | | | | |
| NOTES: | | | | | | | | | | | | | | | | | |
| 1. Water level in open borehole at a depth of 0.3 m below ground surface (Elev. 198.4 m) upon completion of overburden drilling. | | | | | | | | | | | | | | | | | |
| 2. An additional borehole was drilled 1.0 m east of Borehole B5-02 to carry out in situ vane testing at a depth of 1.8 m below ground surface (Elev. 196.9 m). | | | | | | | | | | | | | | | | | |

PROJECT: 07-1111-0029

RECORD OF DRILLHOLE: B5-02

SHEET 1 OF 1

LOCATION: N 5045856.8 ;E 243435.3

DRILLING DATE:

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: D25

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 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
|-----------------------|-------------------------------|--|--------------|-----------------------|---------|--------------------|--|-----------------|-----------------|-------------|---------------------------------|---------|---------------------------|---------------------------------|----|----|----|--|----------------|--|---|------------------|--|--|--|--|--|--|--|--|--|--|---|--|--|--|--|--|--|--|--|--|--|--|--|---|--|--|--|--|--|--|--|--|--|--|--|--|---|--|--|--|--|--|--|--|--|--|--|--|--|-------|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|
| | | | | | | | FLUSH | RECOVERY | | R.Q.D. % | FRACT. INDEX PER 0.3 m | B Angle | DIP w.r.t CORE AXIS | TYPE AND SURFACE DESCRIPTION | Jr | Ja | Jn | HYDRAULIC CONDUCTIVITY K, cm/sec | | | Diametral Point Load Index (MPa) | RMC -Q AVG | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | TOTAL CORE % | SOLID CORE % | | | | | | | | | 10 10 10 | 10 10 10 | 10 10 10 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 3 | NW Casing February 3, 2009 | Continued from Record of Borehole B5-02 GRANITE GNEISS Slightly weathered to fresh, fine to coarse grained, foliated, black and grey | | 195.90 2.80 | 1 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |

DEPTH SCALE

1 : 50





LOGGED: MR

CHECKED: OK

GTA-RCK 018 T:\PROJECTS\2007\07-1111-0029 (MRC, PARRY SOUND)\LOG\07-1111-0029-SHAWANAGA NBL & SBL-PHASE III.GPJ GAL-MISS.GDT 03/12/15 SAC

| PROJECT | | RECORD OF BOREHOLE | | No B5-03 | | SHEET 1 OF 1 | | METRIC | | | | | | |
|--|---|--------------------|---------|--------------------------|------------|--|-----------------|---|--|------------------------------------|-------------------------------------|-----------------------------------|---------------------|---|
| W.P. 07-1111-0029 | | LOCATION | | N 5045860.2 ; E 243431.8 | | ORIGINATED BY | | MR | | | | | | |
| DIST | | HWY 69 | | BOREHOLE TYPE | | 108 mm I.D. Continuous Flight Hollow Stem Augers | | COMPILED BY | | | | | | |
| VA | | DATE | | February 5, 2009 | | CHECKED BY | | OK | | | | | | |
| 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 | | | | | | |
| 199.4 | GROUND SURFACE | | | | | | | | | | | | | |
| 0.0 | TOPSOIL | | | | | | | | | | | | | |
| 0.2 | Organic Sandy SILT, some clay, containing rootlets Very loose Brown to grey Moist | | 1A | SS | 3 | | 199 | | | | | | | |
| | | | 1B | | | | | | | | | | | |
| | | | 2A | | | | | | | | | | | |
| | | | 2B | SS | 2 | | | | | | | | | |
| 198.0 | Sandy SILT, trace clay Loose Grey Wet | | 2C | | | | 198 | | | | | | | |
| 1.4 | | | 3 | SS | 5 | | | | | | | | | |
| | | | 4 | SS | 7 | | | | | | | | | |
| 196.6 | END OF BOREHOLE AUGER REFUSAL | | | | | | | | | | | | | 0 24 71 5 |
| 3.0 | END OF DCPT Refusal to Further Penetration (100 Blows / 0.23 m) | | | | | | | | | | | | | |
| NOTES: 1. Water level in open borehole at a depth of 1.4 m below ground surface (Elev. 198.0 m) upon completion of drilling. 2. A Dynamic Cone Penetration Test was carried out at 1.5 m east of Borehole B5-03; refusal encountered at a depth of 3.0 m below ground surface (Elev. 196.4 m). | | | | | | | | | | | | | | |

| PROJECT | | RECORD OF BOREHOLE | | No B5-04 | | SHEET 1 OF 1 | | METRIC | |
|-----------------|--|--------------------|--|--------------------------|--|--|--|-------------|--|
| W.P. 5188-06-01 | | LOCATION | | N 5045863.4 ; E 243448.8 | | ORIGINATED BY | | MR | |
| DIST | | HWY 69 | | BOREHOLE TYPE | | 108 mm I.D. Continuous Flight Hollow Stem Augers | | COMPILED BY | |
| VA | | DATE | | February 5, 2009 | | CHECKED BY | | OK | |
| DATUM | | Geodetic | | | | | | | |

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | PLASTIC LIMIT W _p | NATURAL MOISTURE CONTENT W | LIQUID LIMIT W _L | UNIT WEIGHT γ | REMARKS & GRAIN SIZE DISTRIBUTION (%) | | | |
|--------------|--|---|---------|------|------------|---|-----------------|--|----|---------------------------------|-------------------------------|--------------------------------|------------------|---------------------------------------|----|----|-----|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | 20 | 40 | | | | | | 60 | 80 | 100 |
| 199.1 | GROUND SURFACE | | | | | | | | | | | | | | | | |
| 0.0 | Organic Sandy SILT, trace clay, containing silty sand layers |  | 1 | SS | 3 |  | 199 | | | | | | | | | | |
| 198.5 | Very loose Dark brown Moist | | 2 | SS | 8 | | 198 | | | | | | | | | | |
| 197.9 | CLAYEY SILT, trace to some sand | | 3 | SS | 6 | | 197 | | | | | | | | | | |
| 1.2 | Firm Brown to grey Wet | | | | | | | | | | | | | | | | |
| 196.7 | CLAY Stiff Brown Wet | | | | | | | | | | | | | | | | |
| 2.4 | END OF BOREHOLE SPOON AND AUGER REFUSAL | | | | | | | | | | | | | | | | |
| | END OF DCPT Refusal to Further Penetration (100 Blows / 0.28 m) | | | | | | | | | | | | | | | | |
| | NOTES: | | | | | | | | | | | | | | | | |
| | 1. Water level in open borehole at a depth of 1.7 m below ground surface (Elev. 197.4 m) upon completion of drilling. | | | | | | | | | | | | | | | | |
| | 2. A Dynamic Cone Penetration Test was carried out 1.5 m west of Borehole B5-04; refusal encountered at a depth of 2.4 m below ground surface (Elev. 196.7 m). | | | | | | | | | | | | | | | | |
| | 3. An additional borehole was drilled 1.5 m north-west of Borehole B5-04 to obtain a Shelby tube sample between depths of 1.8 m and 2.3 m. | | | | | | | | | | | | | | | | |

GTA-MTO 001 T:\PROJECTS\2007\07-1111-0029 (MRC, PARRY SOUND)\LOG\07-1111-0029-SHAWANAGA NEL & SBL-PHASE III.GPJ GAL-GTA.GDT 03/12/15 SAC

| PROJECT 07-1111-0029 | | RECORD OF BOREHOLE No B5-05 | | SHEET 1 OF 1 | | METRIC | | | | | | | | | | | |
|----------------------|---|--|---------|------------------|------------|--|-----------------|--------------------|---|---|----------------|-------------|-------------------|--|---------------------------------------|------------------------|-------------|
| W.P. 5188-06-01 | | LOCATION N 5045866.7 ; E 243445.3 | | ORIGINATED BY MR | | | | | | | | | | | | | |
| DIST HWY 69 | | BOREHOLE TYPE 165 mm O.D. Continuous Flight Solid Stem Augers and NW Casing, Wash Boring | | COMPILED BY VA | | | | | | | | | | | | | |
| DATUM Geodetic | | DATE February 8, 2009 | | CHECKED BY OK | | | | | | | | | | | | | |
| SOIL PROFILE | | | SAMPLES | | | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT | | | UNIT WEIGHT | | | REMARKS & GRAIN SIZE DISTRIBUTION (%) | | |
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | SHEAR STRENGTH kPa | | | | | WATER CONTENT (%) | | | γ kN/m ³ | GR SA SI CL |
| | | | | | | | 20 40 60 80 100 | 20 40 60 80 100 | W _p | W | W _L | 10 20 30 | | | | | |
| 199.6 0.0 | GROUND SURFACE | | | | | | | | | | | | | | | | |
| 198.9 0.7 | PEAT, containing wood fragments Hard Brown Moist | | 1 | SS | 53 | | 199 | | | | | | | | | 229.2 | |
| | SAND and SILT, trace clay, trace organics, containing rootlets to a depth of 1.1 m | | 2A | SS | 14 | | | | | | | | | | | 54.9 | |
| | Compact Dark brown to grey Moist to wet | | 2B | SS | | | | | | | | | | | | | |
| 197.8 1.8 | Granite Gneiss (BEDROCK) | | 3 | SS | 59/0.10 | | 198 | | | | | | | | | | |
| | Bedrock cored from depths of 1.8 m to 4.9 m | | 1 | RC | REC 100% | | 197 | | | | | | | | | | RQD = 88% |
| | For bedrock coring details, refer to Record of Drillhole B5-05 | | 2 | RC | REC 100% | | 196 | | | | | | | | | | RQD = 85% |
| 194.7 4.9 | END OF BOREHOLE | | | | | | 195 | | | | | | | | | | |
| | NOTE: 1. Water level in open borehole at a depth of 1.5 m below ground surface (Elev. 198.1 m) upon completion of overburden drilling. | | | | | | | | | | | | | | | | |

GTA-MTO 001 T:\PROJECTS\2007\07-1111-0029 (MRC, PARRY SOUND)\LOG\07-1111-0029-SHAWANAGA NEL & SBL-PHASE III.GPJ GAL-GTA.GDT 03/12/15 SAC

SHEET 1 OF 1

DATUM: Geodetic

DRILLING CONTRACTOR: Walker Drilling

T:\PROJECTS\2007\07-1111-0029 (MRC, PARRY SOUND)\LOG\07-1111-0029-SHAWANAGA NBL & SBL-PHASE III.GPJ GAL-MISS.GDT 03/12/15 SAC

CHECKED: OK

| PROJECT | | RECORD OF BOREHOLE | | No B5-06 | | SHEET 1 OF 1 | | METRIC | | | | | | | | | |
|---|---|--------------------|---------|--------------------------|------------|---|-----------------|---|--|--|--|--|------------------------------------|-------------------------------------|-----------------------------------|---------------------|---|
| W.P. 5188-06-01 | | LOCATION | | N 5045875.2 ; E 243426.7 | | ORIGINATED BY | | MR | | | | | | | | | |
| DIST | | HWY 69 | | BOREHOLE TYPE | | 165 mm O.D. Continuous Flight Solid Stem Augers | | COMPILED BY | | | | | | | | | |
| VA | | DATE | | February 6, 2009 | | CHECKED BY | | OK | | | | | | | | | |
| Geodetic | | | | | | | | | | | | | | | | | |
| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | | | PLASTIC LIMIT W _p | NATURAL MOISTURE CONTENT W | LIQUID LIMIT W _L | UNIT WEIGHT γ | REMARKS & GRAIN SIZE DISTRIBUTION (%) |
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH kPa | | | | | | | | | |
| 200.7 | GROUND SURFACE | | | | | | | | | | | | | | | | |
| 0.0 | TOPSOIL | | 1A | | | | | | | | | | | | | | |
| 0.2 | SAND and GRAVEL, trace silt | | 1B | SS | 30 | | | | | | | | | | | | |
| 200.0 | Dense Brown | | | | | | | | | | | | | | | | |
| 0.7 | Moist END OF BOREHOLE AUGER REFUSAL | | | | | | | | | | | | | | | | |
| NOTES: 1. Open borehole dry upon completion of drilling. 2. A Dynamic Cone Penetration Test was carried out 1.5 m east of Borehole B5-06; refusal encountered at a depth of 0.5 m below ground surface (Elev. 200.2 m). | | | | | | | | | | | | | | | | | |

GTA-MTO 001 T:\PROJECTS\2007\07-1111-0029 (MRC, PARRY SOUND)\LOG\07-1111-0029-SHAWANAGA NEL & SBL-PHASE III.GPJ GAL-GTA.GDT 03/12/15 SAC

| PROJECT 07-1111-0029 | | RECORD OF BOREHOLE No B5-07 | | SHEET 1 OF 1 | | METRIC | | | | | | | | | | | |
|----------------------|---|--------------------------------------|---------|------------------|------------|--|-----------------|--------------------|---|----------------|---|----------------|-------------------|--|---------------------------------------|------------|-------------|
| W.P. 5188-06-01 | | LOCATION N 5045840.5 ; E 243462.7 | | ORIGINATED BY MR | | | | | | | | | | | | | |
| DIST HWY 69 | | BOREHOLE TYPE NW Casing, Wash Boring | | COMPILED BY TZ | | | | | | | | | | | | | |
| DATUM Geodetic | | DATE April 8, 2009 | | CHECKED BY OK | | | | | | | | | | | | | |
| SOIL PROFILE | | | SAMPLES | | | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT | | | UNIT WEIGHT | | | REMARKS & GRAIN SIZE DISTRIBUTION (%) | | |
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | SHEAR STRENGTH kPa | | | | | WATER CONTENT (%) | | | γ kN/m³ | GR SA SI CL |
| | | | | | | | | 20 40 60 80 100 | 20 40 60 80 100 | W _p | W | W _L | 10 20 30 | | | | |
| 199.7 0.0 | WATER SURFACE Water | | | | | ▽ | | | | | | | | | | | |
| 193.6 6.1 | Organic SAND, containing wood fragments Very loose Brown Wet | | 1 | SS | WR | | | | | | | | | | | | |
| 193.0 6.7 | SILTY CLAY Very soft Brown Wet | | 2 | SS | WR | | | | | | | | | | | | |
| 192.3 7.6 | Sandy SILT, trace gravel Loose Grey Wet | | 3 | SS | 10/0.03 | | | | | | | | | | | | |
| | Granite Gneiss (BEDROCK) | | | | | | | | | | | | | | | | |
| | Bedrock cored from depths of 7.6 m to 9.8 m | | | | | | | | | | | | | | | | |
| | For bedrock coring details, refer to Record of Drillhole B5-07 | | | | | | | | | | | | | | | | |
| 189.9 9.8 | END OF BOREHOLE | | | | | | | | | | | | | | | | |

GTA-MTO 001 T:\PROJECTS\2007\07-1111-0029 (MRC, PARRY SOUND)\LOG\07-1111-0029-SHAWANAGA NEL & SBL-PHASE III.GPJ GAL-GTA.GDT 03/12/15 SAC

PROJECT: 07-1111-0029

RECORD OF DRILLHOLE: B5-07

SHEET 1 OF 1

LOCATION: N 5045840.5 ;E 243462.7

DRILLING DATE:

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: D25

DRILLING CONTRACTOR: Walker Drilling

| DEPTH SCALE METRES | DRILLING RECORD | DESCRIPTION | SYMBOLIC LOG | ELEV. DEPTH (m) | RUN No. | FLUSH | 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 | | 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 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | 88 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |

DEPTH SCALE

1 : 50



LOGGED: MR

CHECKED: OK

GTA-RCK 018 T:\PROJECTS\2007\07-1111-0029 (MRC, PARRY SOUND)\LOG\07-1111-0029-SHAWANAGA NBL & SBL-PHASE III.GPJ GAL-MISS.GDT 03/12/15 SAC

| PROJECT 07-1111-0029 | | RECORD OF BOREHOLE No B5-08 | | SHEET 1 OF 1 | | METRIC | | | | | | | | | | | | |
|----------------------|--|--------------------------------------|---------|------------------|------------|--|-----------------|--------------------|---|----------------|---|----------------|-------------------|---|---------------------------------------|--|--|--|
| W.P. 5188-06-01 | | LOCATION N 5045835.6 ; E 243458.7 | | ORIGINATED BY MR | | | | | | | | | | | | | | |
| DIST HWY 69 | | BOREHOLE TYPE NW Casing, Wash Boring | | COMPILED BY TZ | | | | | | | | | | | | | | |
| DATUM Geodetic | | DATE April 9, 2009 | | CHECKED BY OK | | | | | | | | | | | | | | |
| 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 (%) | | | | | |
| 199.4 0.0 | WATER SURFACE Water | | | | | | | 20 40 60 80 100 | 20 40 60 80 100 | W _p | W | W _L | 10 20 30 | γ | GR SA SI CL | | | |
| 192.8 6.6 | Organic SAND, containing wood fragments and rootlets Very loose Black Wet | | 1 | SS | WR | | 199 | | | | | | | | | | | |
| | | | 2 | SS | WR | | 198 | | | | | | | | | | | |
| 191.3 8.1 | CLAYEY SILT Soft Grey Wet | | 3 | SS | WR | | 197 | | | | | | | | | | | |
| | | | 4 | TO | PM | | 196 | | | | | | | | | | | |
| 189.3 | | | | | | | 195 | | | | | | | | | | | |
| 188.9 10.5 | Silty SAND Very loose Grey Wet | | 5A | | | | 194 | | | | | | | | | | | |
| 188.2 11.2 | CLAY, containing silt interlayers Very soft Brown Wet | | 5B | SS | 1 | | 193 | | | | | | | | | | | |
| | Granite Gneiss (BEDROCK) | | | | | | 192 | | | | | | | | | | | |
| | Bedrock cored from depths of 11.2 m to 12.7 m | | 1 | RC | REC 98% | | 191 | | | | | | | | | | | |
| 186.7 12.7 | For bedrock coring details, refer to Record of Drillhole B5-08 END OF BOREHOLE | | | | | | 190 | | | | | | | | | | | |
| | | | | | | | 189 | | | | | | | | | | | |
| | | | | | | | 188 | | | | | | | | | | | |
| | | | | | | | 187 | | | | | | | | | | | |

SHEET 1 OF 1

DATUM: Geodetic

DRILLING CONTRACTOR: Walker Drilling

T:\PROJECTS\2007\07-1111-0029 (MRC, PARRY SOUND)\LOG\07-1111-0029-SHAWANAGA NBL & SBL-PHASE III.GPJ GAL-MISS.GDT 03/12/15 SAC

CHECKED: OK

| PROJECT 07-1111-0029 | | RECORD OF BOREHOLE No B5-09 | | SHEET 1 OF 1 | | METRIC | | | | | | | | | | | | |
|----------------------|--|--------------------------------------|---------|------------------|------------|--|-----------------|--|---|--|--|-------------|---|--|---------------------------------------|-------------|--|--|
| W.P. 5188-06-01 | | LOCATION N 5045840.3 ; E 243456.5 | | ORIGINATED BY MR | | | | | | | | | | | | | | |
| DIST _____ HWY 69 | | BOREHOLE TYPE NW Casing, Wash Boring | | COMPILED BY TZ | | | | | | | | | | | | | | |
| DATUM Geodetic | | DATE April 10, 2009 | | CHECKED BY OK | | | | | | | | | | | | | | |
| 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 (%) | | | | | |
| 199.2 | WATER SURFACE | | | | | | | 20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED | | | | | W _p W W _L 10 20 30 | | | GR SA SI CL | | |
| 0.0 | Water | | | | | | | | | | | | | | | | | |
| 194.4 | Organic CLAYEY SILT, trace gravel, trace sand, containing wood fragments | | 1A | SS | WR | | 199 | | | | | | | | | | | |
| 193.9 | Very soft Brown Wet | | 1B | SS | WR | | 198 | | | | | | | | | | | |
| 5.3 | CLAY Soft Grey Wet | | 2 | SS | WR | | 197 | | | | | | | | | | | |
| | | | | | | | 196 | | | | | | | | | | | |
| | | | | | | | 195 | | | | | | | | | | | |
| 192.0 | Sandy SILT Grey Wet | | 3A | TO | PM | | 194 | | | | | | | | | | | |
| 7.2 | | | 3B | | | | 193 | | | | | | | | | | | |
| 191.5 | Granite Gneiss (BEDROCK) | | | | | | 192 | | | | | | | | | | | |
| 7.7 | Bedrock cored from depths of 7.7 m to 11.1 m | | 1 | RC | REC 92% | | 191 | | | | | | | | | RQD = 39% | | |
| | For bedrock coring details, refer to Record of Drillhole B5-09 | | 2 | RC | REC 100% | | 190 | | | | | | | | | RQD = 70% | | |
| | | | 3 | RC | REC 100% | | 189 | | | | | | | | | RQD = 58% | | |
| 188.1 | END OF BOREHOLE | | | | | | | | | | | | | | | | | |
| 11.1 | | | | | | | | | | | | | | | | | | |

GTA-MTO 001 T:\PROJECTS\2007\07-1111-0029 (MRC, PARRY SOUND)\LOG\07-1111-0029-SHAWANAGA NEL & SBL-PHASE III.GPJ GAL-GTA.GDT 03/12/15 SAC

SHEET 1 OF 1

DATUM: Geodetic

DRILLING CONTRACTOR: Walker Drilling

T:\PROJECTS\2007\07-1111-0029 (MRC, PARRY SOUND)\LOG\07-1111-0029-SHAWANAGA NBL & SBL-PHASE III.GPJ GAL-MISS.GDT 03/12/15 SAC

CHECKED: OK

| PROJECT 07-1111-0029 | | | RECORD OF BOREHOLE No B5-10 | | | SHEET 1 OF 1 | | | METRIC | | | | | | | | |
|----------------------|--|------------|--------------------------------------|------|------------|----------------------------|-----------------|---|--------|----|----|-----|------------------------------------|-------------------------------------|-----------------------------------|-------------------------|---|
| W.P. 5188-06-01 | | | LOCATION N 5045845.0 ; E 243466.4 | | | ORIGINATED BY MR | | | | | | | | | | | |
| DIST HWY 69 | | | BOREHOLE TYPE NW Casing, Wash Boring | | | COMPILED BY TZ | | | | | | | | | | | |
| DATUM Geodetic | | | DATE April 10, 2009 | | | CHECKED BY OK | | | | | | | | | | | |
| 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 | | | | | | | | | |
| 199.2 0.0 | WATER SURFACE Water | | | | | | | 20 | 40 | 60 | 80 | 100 | | | | | |
| 196.2 3.0 | Sandy SILT, trace gravel, trace organics, containing wood fragments Very loose Grey Wet | | 1 | SS | WR | | | | | | | | | | | | |
| 195.1 4.1 | Granite Gneiss (BEDROCK) | | 2 | SS | 3 | | | | | | | | | | | | |
| | Bedrock cored from depths of 4.1 m to 5.6 m | | | | | | | | | | | | | | | | |
| | For bedrock coring details, refer to Record of Drillhole B5-10 | | 1 | RC | REC 98% | | | | | | | | | | | | |
| 193.6 5.6 | END OF BOREHOLE | | | | | | | | | | | | | | | | |

GTA-MTO 001 T:\PROJECTS\2007\07-1111-0029 (MRC, PARRY SOUND)\LOG\07-1111-0029-SHAWANAGA NBL & SBL-PHASE III.GPJ GAL-GTA.GDT 03/12/15 SAC

PROJECT: 07-1111-0029

RECORD OF DRILLHOLE: B5-10

SHEET 1 OF 1

LOCATION: N 5045845.0 ;E 243466.4


DRILLING DATE:

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: D25

DRILLING CONTRACTOR: Walker Drilling

| DEPTH SCALE METRES | DRILLING RECORD | | DESCRIPTION | SYMBOLIC LOG | ELEV. DEPTH (m) | RUN No. | FLUSH | 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 | | | 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 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 5 | NQ RC April 10, 2009 | NW Casing April 10, 2009 | Continued from Record of Borehole B5-10 GRANITE GNEISS Slightly weathered to fresh, fine to medium grained with feldspar banding, dark grey to pink |  | 195.09 4.11 | 1 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |

DEPTH SCALE

1 : 50



LOGGED: MR

CHECKED: OK

GTA-RCK 018 T:\PROJECTS\2007\07-1111-0029 (MRC, PARRY SOUND)\LOG\07-1111-0029-SHAWANAGA NBL & SBL-PHASE III.GPJ GAL-MISS.GDT 03/12/15 SAC

| PROJECT 07-1111-0029 | | RECORD OF BOREHOLE No B5-11 | | SHEET 1 OF 1 | | METRIC | | | | | | | | | | | |
|----------------------|--|--------------------------------------|---------|------------------|------------|--|-----------------|---|---|--|--|-------------|--|--|---------------------------------------|---|-------------|
| W.P. 5188-06-01 | | LOCATION N 5045840.6 ; E 243468.5 | | ORIGINATED BY MR | | | | | | | | | | | | | |
| DIST HWY 69 | | BOREHOLE TYPE NW Casing, Wash Boring | | COMPILED BY TZ | | | | | | | | | | | | | |
| DATUM Geodetic | | DATE April 11, 2009 | | CHECKED BY OK | | | | | | | | | | | | | |
| SOIL PROFILE | | | SAMPLES | | | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT | | | UNIT WEIGHT | | | REMARKS & GRAIN SIZE DISTRIBUTION (%) | | |
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED | | | | | WATER CONTENT (%) W _p — W — W _L | | | γ | GR SA SI CL |
| 199.1 0.0 | WATER SURFACE Water | | | | | ▽ | 199 | | | | | | | | | | |
| | | | | | | | 198 | | | | | | | | | | |
| | | | | | | | 197 | | | | | | | | | | |
| | | | | | | | 196 | | | | | | | | | | |
| | | | | | | | 195 | | | | | | | | | | |
| | | | | | | | 194 | | | | | | | | | | |
| 193.7 5.8 | SAND, some silt, trace gravel, trace organics, containing wood fragments Loose Black Wet Granite Gneiss (BEDROCK) Bedrock cored from depths of 5.8 m to 9.6 m For bedrock coring details, refer to Record of Drillhole B5-11 | | 1 | SS | 5/0.13 | | 193 | | | | | | | | | | RQD = 43% |
| | | | 1 | RC | REC 96% | | 192 | | | | | | | | | | RQD = 70% |
| | | | 2 | RC | REC 93% | | 191 | | | | | | | | | | RQD = 78% |
| | | | 3 | RC | REC 100% | | 190 | | | | | | | | | | |
| 189.5 9.6 | END OF BOREHOLE | | | | | | | | | | | | | | | | |

GTA-MTO 001 T:\PROJECTS\2007\07-1111-0029 (MRC, PARRY SOUND)\LOG\07-1111-0029-SHAWANAGA NEL & SBL-PHASE III.GPJ GAL-GTA.GDT 03/12/15 SAC

PROJECT: 07-1111-0029

RECORD OF DRILLHOLE: B5-11

SHEET 1 OF 1

LOCATION: N 5045840.6 ;E 243468.5

DRILLING DATE:

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: D25

DRILLING CONTRACTOR: Walker Drilling

| DEPTH SCALE METRES | DRILLING RECORD | DESCRIPTION | SYMBOLIC LOG | ELEV. | | RUN No. | COLOUR % RETURN | FLUSH | RECOVERY | | R.Q.D. % | FRACT INDEX PER 0.3 m | DISCONTINUITY DATA | | | | | | HYDRAULIC CONDUCTIVITY | | | | Diametral Point Load Index (MPa) | RMC -Q AVG | NOTES | | | | | | |
|-----------------------|-----------------------------|---|-----------------|-----------------|--|---------|--------------------|-------|-----------------|-----------------|-------------|--------------------------------|--------------------|---------------------------|---------------------------------|----|----|----|---------------------------|---------|---------|---------|---|------------------|-------|---------|------------|--------------|-------------|---------------|-----------------------|
| | | | | DEPTH (m) | FLUSH | | | | TOTAL CORE % | SOLID CORE % | | | B Angle | DIP w.r.t CORE AXIS | TYPE AND SURFACE DESCRIPTION | Jr | Ja | Jn | K, cm/sec | 10 ° | 10 ° | 10 ° | | | | 10 ° | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | | | | | | JN - Joint | BD - Bedding | PL - Planar | PO - Polished | MB - Mechanical Break |
| | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| SH - Shear | | CO - Contact | UN - Undulating | SM - Smooth | NOTE: For additional abbreviations refer to list of abbreviations & symbols. | | | | | | | | | | | | | | | | | | | | | | | | | | |
| VN - Vein | | OR - Orthogonal | ST - Stepped | RO - Rough | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| CJ - Conjugate | | CL - Cleavage | IR - Irregular | VR - Very Rough | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 6 | NW Casing April 11, 2009 | Continued from Record of Borehole B5-11 | | 193.34 | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 7 | | GRANITE GNEISS Slightly weathered to fresh, fine to medium grained with feldspar banding, foliated, black, pink and grey | | 5.76 | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 8 | NQ RC April 11, 2009 | | | | 1 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 9 | | | | | 2 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 10 | | END OF DRILLHOLE | | 189.53 | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 11 | | | | 9.57 | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 12 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 13 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 14 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 15 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |

DEPTH SCALE

1 : 50



LOGGED: MR

CHECKED: OK

GTA-RCK 018 T:\PROJECTS\2007\07-1111-0029 (MRC, PARRY SOUND)\LOG\07-1111-0029-SHAWANAGA NBL & SBL-PHASE III.GPJ GAL-MASS.GDT 03/12/15 SAC

| PROJECT 07-1111-0029 | | RECORD OF BOREHOLE No B5-12 | | SHEET 1 OF 1 | | METRIC | | | | | | | | | | |
|----------------------|---------------|--|---------|------------------|----------------------------|-----------------|---|--------------------|----|------------------------------------|-------------------------------------|-----------------------------------|---------------------|---|-------------------|--|
| W.P. 5188-06-01 | | LOCATION N 5045812.7 ; E 243491.5 | | ORIGINATED BY MR | | | | | | | | | | | | |
| DIST _____ HWY 69 | | BOREHOLE TYPE NW Casing, Wash Boring | | COMPILED BY TZ | | | | | | | | | | | | |
| DATUM Geodetic | | DATE April 11 & 12, 2009 | | CHECKED BY OK | | | | | | | | | | | | |
| 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 (%) | |
| 199.1 | WATER SURFACE | | | | | | 20 | 40 | 60 | 80 | 100 | | | | | |
| 0.0 | Water | | | | | | 20 | 40 | 60 | 80 | 100 | | | | | |
| 192.7 | 6.4 | PEAT (Amorphous), containing wood fragments | | | | | | | | | | | | | | |
| 192.1 | 7.0 | Very soft Brown Wet | 1 | SS | 1 | | | | | | | | | | | |
| 191.6 | | SAND, trace organics, containing wood fragments | 2A | SS | 6 | | | | | | | | | | | |
| 191.1 | 8.0 | Loose Brown Wet | 2B | | | | | | | | | | | | | |
| 190.6 | 8.5 | SILTY CLAY | 2C | | | | | | | | | | | | | |
| | | Firm Brown Wet | 3 | RC | - | | | | | | | | | | | |
| | | COBBLES | 4 | SS | 8 | | | | | | | | | | | |
| | | SAND, trace to some silt, trace gravel | 1 | RC | REC 97% | | | | | | | | | | | |
| | | Loose Grey Wet | | | | | | | | | | | | | | |
| | | Granite Gneiss (BEDROCK) | 2 | RC | REC 100% | | | | | | | | | | | |
| | | Bedrock cored from depths of 8.5 m to 12 m | | | | | | | | | | | | | | |
| | | For bedrock coring details, refer to Record of Drillhole B5-12 | 3 | RC | REC 100% | | | | | | | | | | | |
| 187.1 | 12.0 | END OF BOREHOLE | | | | | | | | | | | | | | |

GTA-MTO 001 T:\PROJECTS\2007\07-1111-0029 (MRC, PARRY SOUND)\LOG\07-1111-0029-SHAWANAGA NEL & SBL-PHASE III.GPJ GAL-GTA.GDT 03/12/15 SAC

SHEET 1 OF 1

DATUM: Geodetic

DRILLING CONTRACTOR: Walker Drilling

T:\PROJECTS\2007\07-1111-0029 (MRC, PARRY SOUND)\LOG\07-1111-0029-SHAWANAGA NBL & SBL-PHASE III.GPJ GAL-MISS.GDT 03/12/15 SAC

CHECKED: OK

| PROJECT 07-1111-0029 | | | RECORD OF BOREHOLE No B5-13 | | | SHEET 1 OF 1 | | | METRIC | | | | | | | | |
|----------------------|--|------------|--------------------------------------|------|------------|----------------------------|--------------------|---|--------|--|--|--|------------------------------------|-------------------------------------|-----------------------------------|---------------------|---|
| W.P. 5188-06-01 | | | LOCATION N 5045807.9 ; E 243487.4 | | | ORIGINATED BY MR | | | | | | | | | | | |
| DIST _____ HWY 69 | | | BOREHOLE TYPE NW Casing, Wash Boring | | | COMPILED BY TZ | | | | | | | | | | | |
| DATUM Geodetic | | | DATE April 12 & 13, 2009 | | | CHECKED BY OK | | | | | | | | | | | |
| 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 | | | | | | | | | |
| 199.0 | WATER SURFACE | | | | | | | | | | | | | | | | |
| 0.0 | Water | | | | | | | | | | | | | | | | |
| 193.4 | | | | | | | | | | | | | | | | | |
| 5.6 | SAND, trace gravel, trace organics, containing wood fragments | | 1A | | | | | | | | | | | | | | |
| 192.9 | Very loose | | 1B | SS | WR | | | | | | | | | | | | |
| 6.1 | Brown | | | | | | | | | | | | | | | | |
| 192.5 | Wet | | 2 | SS | 55 | | | | | | | | | | | | |
| 6.6 | CLAYEY SILT, containing silt interlayers | | | | | | | | | | | | | | | | |
| 191.7 | Very soft | | 3 | SS | 100/0.18 | | | | | | | | | | | | |
| 7.4 | Brown | | | | | | | | | | | | | | | | |
| | Wet | | 1 | RC | REC 100% | | | | | | | | | | | | RQD = 53% |
| | SAND and GRAVEL | | | | | | | | | | | | | | | | |
| | Very dense | | | | | | | | | | | | | | | | |
| | Grey | | | | | | | | | | | | | | | | |
| | Wet | | | | | | | | | | | | | | | | |
| | Granite Gneiss (BEDROCK) | | 2 | RC | REC 91% | | | | | | | | | | | | RQD = 16% |
| | Bedrock cored from depths of 7.4 m to 11.3 m | | | | | | | | | | | | | | | | |
| | For bedrock coring details, refer to Record of Drillhole B5-13 | | 3 | RC | REC 100% | | | | | | | | | | | | RQD = 0% |
| | | | | | | | | | | | | | | | | | |
| | | | 4 | RC | REC 100% | | | | | | | | | | | | RQD = 64% |
| | | | | | | | | | | | | | | | | | |
| 187.7 | | | | | | | | | | | | | | | | | |
| 11.3 | END OF BOREHOLE | | | | | | | | | | | | | | | | |

SHEET 1 OF 1

DATUM: Geodetic

DRILLING CONTRACTOR: Walker Drilling

T:\PROJECTS\2007\07-1111-0029 (MRC, PARRY SOUND)\LOG\07-1111-0029-SHAWANAGA NBL & SBL-PHASE III.GPJ GAL-MISS.GDT 03/12/15 SAC

CHECKED: OK

| PROJECT 07-1111-0029 | | | RECORD OF BOREHOLE No B5-14 | | | SHEET 1 OF 1 | | | METRIC | | | | | |
|----------------------|--|------------|--------------------------------------|------|------------|-------------------------|-----------------|--|--------|--|--|--|--|---------------------------------------|
| W.P. 5188-06-01 | | | LOCATION N 5045812.5 ; E 243485.2 | | | ORIGINATED BY MR | | | | | | | | |
| DIST HWY 69 | | | BOREHOLE TYPE NW Casing, Wash Boring | | | COMPILED BY TZ | | | | | | | | |
| DATUM Geodetic | | | DATE April 13, 2009 | | | CHECKED BY OK | | | | | | | | |
| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | | | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) |
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH kPa | | | | | | |
| 198.9 0.0 | WATER SURFACE Water | | | | | | | | | | | | | |
| 193.0 5.9 | SAND, trace gravel, containing organics and wood fragments to a depth of 7.3 m Very loose Brown Wet | | 1 | SS | WR | | | | | | | | | |
| | | | 2A | SS | WH | | | | | | | | | |
| | | | 2B | SS | WH | | | | | | | | | |
| | | | 3 | SS | WR | | | | | | | | | |
| 190.9 | SAND and SILT, trace clay | | | | | | | | | | | | | |
| 190.5 | Very loose | | 4A | SS | 2 | | | | | | | | | |
| 8.4 | Grey Wet | | 4B | SS | 2 | | | | | | | | | |
| 189.8 | SILTY CLAY | | | | | | | | | | | | | |
| 9.1 | Firm Brown Wet | | | | | | | | | | | | | |
| 189.2 | Silty SAND | | 5A | SS | 18 | | | | | | | | | |
| 9.8 | Compact Grey Wet | | 5B | SS | 18 | | | | | | | | | |
| 188.7 | SAND, some gravel | | 6 | SS | 20.0.02 | | | | | | | | | |
| 10.2 | Compact Grey Wet | | | | | | | | | | | | | |
| | Granite Gneiss (BEDROCK) | | 1 | RC | REC 100% | | | | | | | | | |
| | Bedrock cored from depths of 10.2 m to 11.8 m | | | | | | | | | | | | | |
| 187.1 | For bedrock coring details, refer to Record of Drillhole B5-14 | | | | | | | | | | | | | |
| 11.8 | END OF BOREHOLE | | | | | | | | | | | | | |

GTA-MTO 001 T:\PROJECTS\2007\07-1111-0029 (MRC, PARRY SOUND)\LOG\07-1111-0029-SHAWANAGA NEL & SBL-PHASE III.GPJ GAL-GTA.GDT 03/12/15 SAC

PROJECT: 07-1111-0029

RECORD OF DRILLHOLE: B5-14

SHEET 1 OF 1

LOCATION: N 5045812.5 ;E 243485.2

DRILLING DATE:

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: D25

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 | |
|-----------------------|-----------------------------|---|--------------|-----------------------|---------|--------------------|---|-----------------|-----------------|-------------|---------------------------------|---------|---------------------------|---------------------------------|----|----|--|----------------------|----------------------|---|-------|------------------|
| | | | | | | | FLUSH | RECOVERY | | R.Q.D. % | FRACT. INDEX PER 0.3 m | B Angle | DIP w.r.t CORE AXIS | DISCONTINUITY DATA | | | HYDRAULIC CONDUCTIVITY K, cm/sec | | | Diametral Point Load Index (MPa) | | RMC -Q AVG |
| | | | | | | | | TOTAL CORE % | SOLID CORE % | | | | | TYPE AND SURFACE DESCRIPTION | Jr | Ja | Jn | 10 10 10 10 | 10 10 10 10 | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | NW Casing April 13, 2009 | Continued from Record of Borehole B5-14 | | 188.70 | | | | | | | | | | | | | | | | | | |
| | | GRANITE GNEISS Slightly weathered to fresh, fine to medium grained with feldspar banding, foliated, dark grey and pink | | 10.20 | 1 | | | | | | | | | | | | | | | UC=99.2MPa | | |
| 11 | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | </ | | | | | | | | | | | | | | | | |

DEPTH SCALE

1 : 50



LOGGED: MR

CHECKED: OK

GTA-RCK 018 T:\PROJECTS\2007\07-1111-0029 (MRC, PARRY SOUND)\LOG\07-1111-0029-SHAWANAGA NBL & SBL-PHASE III.GPJ GAL-MISS.GDT 03/12/15 SAC

| PROJECT 07-1111-0029 | | RECORD OF BOREHOLE No B5-15 | | SHEET 1 OF 1 | | METRIC | | | | | | | | | | | | | | | |
|----------------------|--|--------------------------------------|---------|------------------|------------|--|-----------------|--------------------|---|----------------|---|----------------|-------------------|------|---------------------------------------|---|-----------|--|--|--|--|
| W.P. 5188-06-01 | | LOCATION N 5045817.2 ; E 243495.0 | | ORIGINATED BY MR | | | | | | | | | | | | | | | | | |
| DIST HWY 69 | | BOREHOLE TYPE NW Casing, Wash Boring | | COMPILED BY TZ | | | | | | | | | | | | | | | | | |
| DATUM Geodetic | | DATE April 14, 2009 | | CHECKED BY OK | | | | | | | | | | | | | | | | | |
| 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 | | | | | |
| 198.8 0.0 | WATER SURFACE Water | | | | | | | 20 40 60 80 100 | 20 40 60 80 100 | W _p | W | W _L | 10 20 30 | 45.8 | GR SA SI CL | | | | | | |
| 193.5 5.3 | PEAT (Amorphous), containing wood fragments and interlayers of sand Very soft Brown Wet | | 1 | SS | WH | | | | | | | | | | | | | | | | |
| 192.6 6.3 | SAND, trace gravel, trace organics, containing wood fragments Very loose Grey Wet | | 2 | SS | 1 | | | | | | | | | | | | | | | | |
| 191.7 7.3 | Cobbles Granite Gneiss (BEDROCK) | | 3 | SS | 25/0.08 | | | | | | | | | | | | | | | | |
| | | | 4 | RC | - | | | | | | | | | | | | | | | | |
| | Bedrock cored from depths of 7.3 m to 12 m For bedrock coring details, refer to Record of Drillhole B5-15 | | 1 | RC | REC 96% | | | | | | | | | | | | RQD = 28% | | | | |
| | | | 2 | RC | REC 100% | | | | | | | | | | | | RQD = 58% | | | | |
| | | | 3 | RC | REC 92% | | | | | | | | | | | | RQD = 14% | | | | |
| | | | 4 | RC | REC 98% | | | | | | | | | | | | RQD = 80% | | | | |
| 186.8 12.0 | END OF BOREHOLE | | | | | | | | | | | | | | | | | | | | |

GTA-MTO 001 T:\PROJECTS\2007\07-1111-0029 (MRC, PARRY SOUND)\LOG\07-1111-0029-SHAWANAGA NEL & SBL-PHASE III.GPJ GAL-GTA.GDT 03/12/15 SAC

PROJECT: 07-1111-0029

RECORD OF DRILLHOLE: B5-15

SHEET 1 OF 1

LOCATION: N 5045817.2 ;E 243495.0

DRILLING DATE:

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: D25

DRILLING CONTRACTOR: Walker Drilling

| DEPTH SCALE METRES | DRILLING RECORD | DESCRIPTION | SYMBOLIC LOG | ELEV. | | RUN No. | COLOUR % RETURN | FLUSH | RECOVERY | | | | R.Q.D. % | FRACT. INDEX PER 0.3 m | DISCONTINUITY DATA | | | | | | HYDRAULIC CONDUCTIVITY | | | | Diameter Point Load Index (MPa) | RMC -Q AVG | NOTES | | | | | |
|--|-----------------------------|---|--------------|--------------|--|---------|--------------------|-------|-----------------|-----------------|---------|----------------------------|-------------|---------------------------------|---------------------------------|----|--|----|-------------------------|-----------------|---|-----------------|----|--|---|------------------|-------|--|---|--|--|--|
| | | | | DEPTH (m) | | | | | TOTAL CORE % | SOLID CORE % | B Angle | DIP w.r.t. CORE AXIS | | | TYPE AND SURFACE DESCRIPTION | Jr | Ja | Jn | K, cm ³ /sec | 10 ⁶ | 10 ⁴ | 10 ² | 10 | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 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. | | | |
| | | Continued from Record of Borehole B5-15 | | 191.52 | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | NW Casing April 14, 2009 | GRANITE GNEISS Slightly weathered to fresh, fine to medium grained with feldspar banding, foliated, dark grey and pink laminated | | 7.28 | | 1 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 8 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | 2 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 9 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 10 | NQ RC April 14, 2009 | | | | | 3 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 11 | | | | | | 4 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 12 | | END OF DRILLHOLE | | 186.80 | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | | | | 12.00 | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 13 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 14 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 15 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 16 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 17 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |

DEPTH SCALE

1 : 50



LOGGED: MR

CHECKED: OK

GTA-RCK 018 T:\PROJECTS\2007\07-1111-0029 (MRC, PARRY SOUND)\LOG\07-1111-0029-SHAWANAGA NBL & SBL-PHASE III.GPJ GAL-MISS.GDT 03/12/15 SAC

| PROJECT 07-1111-0029 | | RECORD OF BOREHOLE No B5-16 | | SHEET 1 OF 1 | | METRIC | | | | | | | | | | | |
|----------------------|---|--------------------------------------|---------|------------------|------------|--|-----------------|--------------------|---|---|----------------|-------------|-------------------|---------------------------------------|--|------------------------|-------------|
| W.P. 5188-06-01 | | LOCATION N 5045812.8 ; E 243497.3 | | ORIGINATED BY MR | | | | | | | | | | | | | |
| DIST HWY 69 | | BOREHOLE TYPE NW Casing, Wash Boring | | COMPILED BY TZ | | | | | | | | | | | | | |
| DATUM Geodetic | | DATE April 15, 2009 | | CHECKED BY OK | | | | | | | | | | | | | |
| SOIL PROFILE | | | SAMPLES | | | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT | | | UNIT WEIGHT | | REMARKS & GRAIN SIZE DISTRIBUTION (%) | | | |
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | SHEAR STRENGTH kPa | | | | | WATER CONTENT (%) | | | γ kN/m ³ | GR SA SI CL |
| | | | | | | | 20 40 60 80 100 | 20 40 60 80 100 | W _p | W | W _L | 10 20 30 | | | | | |
| 198.7 0.0 | WATER SURFACE Water | | | | | ▽ | | | | | | | | | | | |
| | | | | | | | 198 | | | | | | | | | | |
| | | | | | | | 197 | | | | | | | | | | |
| | | | | | | | 196 | | | | | | | | | | |
| | | | | | | | 195 | | | | | | | | | | |
| | | | | | | | 194 | | | | | | | | | | |
| 193.1 5.6 | Granite Gneiss (BEDROCK) Bedrock cored from depths of 5.6 m to 8 m For bedrock coring details, refer to Record of Drillhole B5-16 | | 1 | RC | REC 94% | | 193 | | | | | | | | | RQD = 28% | |
| | | | 2 | RC | REC 100% | | 192 | | | | | | | | | RQD = 65% | |
| 190.7 8.0 | END OF BOREHOLE | | | | | | 191 | | | | | | | | | | |

GTA-MTO 001 T:\PROJECTS\2007\07-1111-0029 (MRC, PARRY SOUND)\LOG\07-1111-0029-SHAWANAGA NEL & SBL-PHASE III.GPJ GAL-GTA.GDT 03/12/15 SAC

PROJECT: 07-1111-0029

RECORD OF DRILLHOLE: B5-16

SHEET 1 OF 1

LOCATION: N 5045812.8 ;E 243497.3










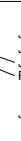



















DRILLING DATE:

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: D25

DRILLING CONTRACTOR: Walker Drilling

| DEPTH SCALE METRES | DRILLING RECORD | DESCRIPTION | SYMBOLIC LOG | ELEV. DEPTH (m) | RUN No. | COLOUR % RETURN | FLUSH | RECOVERY | | | | R.Q.D. % | FRACT. INDEX PER 0.3 m | B Angle | DIP w.r.t CORE AXIS | DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION | Jr | Ja | Jn | HYDRAULIC CONDUCTIVITY K, cm/sec | Diametral Point Load Index (MPa) | RMC -Q AVG | NOTES |
|-----------------------|-----------------------------|--|---|-----------------------|---------|---|---|---|---|---|---|---|---|---|---|---|---|---|---|--|---|------------------|-------|
| | | | | | | | | TOTAL CORE % | SOLID CORE % | R.O.D. % | FRACT. INDEX PER 0.3 m | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | | |
| 6 | NW Casing April 15, 2009 | Continued from Record of Borehole B5-16 GRANITE GNEISS Slightly weathered to fresh, fine to coarse grained with feldspar banding, foliated, dark grey and pink |  | 193.12 5.58 | 1 |  |  |  |  |  |  |  |  |  |  |  |  |  |  | | | | |
| | | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | | |
| 7 | NQ RC April 15, 2009 | | | | 2 |  |  |  |  |  |  |  |  |  |  |  |  |  |  | | | | |
| | | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | | |
| 8 | | END OF DRILLHOLE | | 190.71 7.99 | | | | | | | | | | | | | | | | | | | |
| 9 | | | | | | | | | | | | | | | | | | | | | | | |
| 10 | | | | | | | | | | | | | | | | | | | | | | | |
| 11 | | | | | | | | | | | | | | | | | | | | | | | |
| 12 | | | | | | | | | | | | | | | | | | | | | | | |
| 13 | | | | | | | | | | | | | | | | | | | | | | | |
| 14 | | | | | | | | | | | | | | | | | | | | | | | |
| 15 | | | | | | | | | | | | | | | | | | | | | | | |

DEPTH SCALE

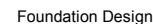
1 : 50



LOGGED: MR

CHECKED: OK

GTA-RCK 018 T:\PROJECTS\2007\07-1111-0029 (MRC, PARRY SOUND)\LOG\07-1111-0029-SHAWANAGA NBL & SBL-PHASE III.GPJ GAL-MISS.GDT 03/12/15 SAC



+ 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-SHAWANAGA NBL & SBL-PHASE III.GPJ GAL-GTA.GDT 03/12/15 SAC

| PROJECT 07-1111-0029 | | RECORD OF BOREHOLE No B5-18 | | SHEET 1 OF 1 | | METRIC | | | | | | | | | | | | |
|----------------------|---|--|---------|------------------|-------------|--|-----------------|--------------------|---|-----------------|----------|-------------|-------------------|--|---------------------------------------|------------------------|-------------|------------|
| W.P. 5188-06-01 | | LOCATION N 5045790.7 ; E 243505.3 | | ORIGINATED BY CR | | | | | | | | | | | | | | |
| DIST _____ HWY 69 | | BOREHOLE TYPE Portable Equipment, BW Casing, Wash Boring | | COMPILED BY VA | | | | | | | | | | | | | | |
| DATUM Geodetic | | DATE July 24 & 25, 2009 | | CHECKED BY OK | | | | | | | | | | | | | | |
| SOIL PROFILE | | | SAMPLES | | | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT | | | UNIT WEIGHT | | | REMARKS & GRAIN SIZE DISTRIBUTION (%) | | | |
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | SHEAR STRENGTH kPa | | | | | WATER CONTENT (%) | | | γ kN/m ³ | GR SA SI CL | |
| | | | | | | | 20 40 60 80 100 | 20 40 60 80 100 | 20 40 60 80 100 | 20 40 60 80 100 | 10 20 30 | 10 20 30 | 10 20 30 | | | | | |
| 205.2 0.0 | GROUND SURFACE BOULDERS/ROCK FRAGMENTS | | | | | | 205 | | | | | | | | | | | |
| 204.2 1.0 | Granite Gneiss (BEDROCK) Coring started at 0.3 m below ground surface. Bedrock cored from depths of 1.0 m to 10.0 m For bedrock coring details, refer to Record of Drillhole B5-18 | | 1 | RC | REC 48% | | 205 | | | | | | | | | | | RQD = 0% |
| | | | 2 | RC | REC 100% | | 204 | | | | | | | | | | | RQD = 69% |
| | | | 3 | RC | REC 100% | | 203 | | | | | | | | | | | RQD = 97% |
| | | | 4 | RC | REC 98% | | 202 | | | | | | | | | | | RQD = 65% |
| | | | 5 | RC | REC 100% | | 201 | | | | | | | | | | | RQD = 22% |
| | | | 6 | RC | REC 100% | | 200 | | | | | | | | | | | RQD = 87% |
| | | | 7 | RC | REC 100% | | 199 | | | | | | | | | | | RQD = 68% |
| | | | 8 | RC | REC 100% | | 198 | | | | | | | | | | | RQD = 83% |
| | | | 9 | RC | REC 100% | | 197 | | | | | | | | | | | RQD = 100% |
| | | | 10 | RC | REC 100% | | 196 | | | | | | | | | | | RQD = 50% |
| | | | 11 | RC | REC 100% | | | | | | | | | | | | | RQD = 78% |
| | | | 12 | RC | REC 17% | | | | | | | | | | | | | RQD = 0% |
| 195.2 10.0 | END OF BOREHOLE NOTES: 1. Water level measurements in piezometer: Date Depth (m) Elev. (m) 28/07/09 2.2 203.0 26/08/09 2.4 202.8 | | | | | | | | | | | | | | | | | |

GTA-MTO 001 T:\PROJECTS\2007\07-1111-0029 (MRC, PARRY SOUND)\LOG\07-1111-0029-SHAWANAGA NEL & SBL-PHASE III.GPJ GAL-GTA.GDT 03/12/15 SAC

PROJECT: 07-1111-0029

RECORD OF DRILLHOLE: B5-18

SHEET 1 OF 2

LOCATION: N 5045790.7 ;E 243505.3

DRILLING DATE:

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: Portable Equipment

DRILLING CONTRACTOR: OGS

| DEPTH SCALE METRES | DRILLING RECORD | DESCRIPTION | SYMBOLIC LOG | ELEV. DEPTH (m) | RUN No. | COLOUR % RETURN | FLUSH | RECOVERY | | | | FRACT INDEX PER 0.3 m | DISCONTINUITY DATA | | | | HYDRAULIC CONDUCTIVITY K, cm/sec | | | | Diametral Point Load Index (MPa) | RMC -Q AVG | NOTES | | | | | | | | | | | | | |
|-----------------------|----------------------------|---|--------------|-----------------------|---------|--------------------|-------|-----------------|-----------------|-------------|---------|--------------------------------|----------------------------|---------------------------------|----|----|--|---------|---------|---------|---|------------------|-------|---------|----|----|----|----|----|----|----|----|----|----|----|----|
| | | | | | | | | 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 ° | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | | | | JN | PL | RO | JN | PL | RO | JN | PL | RO | JN | PL | RO |
| | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 0 | | GROUND SURFACE | | 205.21 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | BW Casing July 24, 2009 | COBBLES/ROCK FRAGMENTS | | 0.00 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 1 | | | | | 1 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | | GRANITE GNEISS Slightly weathered, coarse grained, highly foliated, slightly to moderately porous, light grey to pink | | 204.17 1.04 | 2 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 2 | | | | | 3 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 3 | | | | | 4 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | | GRANITE GNEISS Slightly weathered to fresh, coarse grained, moderately foliated, slightly to moderately porous, light grey to pink | | 201.86 3.35 | 5 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 4 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | 6 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 5 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | 7 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 6 | | | | | 8 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | | GRANITE GNEISS Slightly weathered, coarse grained, moderately foliated, slightly to moderately porous, light grey to pink | | 198.63 6.58 | 9 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 7 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | 10 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 8 | | | | | 11 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 9 | | | | | 12 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 10 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | | CONTINUED NEXT PAGE | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |

DEPTH SCALE

1 : 50



LOGGED: CR

CHECKED: OK

GTA-RCK 018 T:\PROJECTS\2007\07-1111-0029 (MRC, PARRY SOUND)\LOG\07-1111-0029-SHAWANAGA NBL & SBL-PHASE III.GPJ GAL-MISS.GDT 03/12/15 SAC

PROJECT: 07-1111-0029

RECORD OF DRILLHOLE: B5-18

SHEET 2 OF 2

LOCATION: N 5045790.7 ;E 243505.3

DRILLING DATE:

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: Portable Equipment

DRILLING CONTRACTOR: OGS

| 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 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
|-----------------------|-----------------|--|--------------|-----------------------|---------|--------------------|---|-----------------|-----------------|-------------|---------------------------------|--------------------|---------------------------|---------------------------------|----|----|----|--|------------------|------------------|---|------------------|-------|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|--|----|
| | | | | | | | FLUSH | 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 ⁻⁵ | 10 ⁻⁴ | 10 ⁻³ | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 10 | | -- CONTINUED FROM PREVIOUS PAGE -- END OF DRILLHOLE | | 9.96 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | </ |

DEPTH SCALE

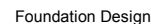
1 : 50



LOGGED: CR

CHECKED: OK

GTA-RCK 018 T:\PROJECTS\2007\07-1111-0029 (MRC, PARRY SOUND)\LOG\07-1111-0029-SHAWANAGA NBL & SBL-PHASE III.GPJ GAL-MISS.GDT 03/17/15 SAC



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

| PROJECT | | RECORD OF BOREHOLE | | No B5-20 | | SHEET 1 OF 1 | | METRIC | | | | | | | | | |
|-----------------|---|--------------------|---------|--------------------------|------------|--|-----------------|---|--|----|--|--|------------------------------------|-------------------------------------|-----------------------------------|---|--|
| W.P. 5188-06-01 | | LOCATION | | N 5045793.0 ; E 243521.7 | | ORIGINATED BY | | CR | | | | | | | | | |
| DIST | | HWY 69 | | BOREHOLE TYPE | | Portable Equipment, BW Casing, Wash Boring | | COMPILED BY | | VA | | | | | | | |
| DATUM | | Geodetic | | DATE | | July 24, 2009 | | CHECKED BY | | OK | | | | | | | |
| 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 | | | | | | | | | |
| 209.7 | GROUND SURFACE | | | | | | | | | | | | | | | | |
| 0.0 | TOPSOIL | | 1A | SS | 4 | | | | | | | | | | | | |
| 0.3 | Silty SAND, trace clay, some gravel, trace organics Very loose Brown Wet END OF BOREHOLE SPOON REFUSAL NOTES: 1. Borehole advanced using Portable drilling equipment with one-third weight hammer. SPT 'N' value shown has been adjusted to infer value that would obtained with standard weight hammer. 2. Open borehole dry upon completion of drilling. | | 1B | | | | | | | | | | | | | | |

GTA-MTO 001 T:\PROJECTS\2007\07-1111-0029 (MRC, PARRY SOUND)\LOG\07-1111-0029-SHAWANAGA NEL & SBL-PHASE III.GPJ GAL-GTA.GDT 03/12/15 SAC

| PROJECT 07-1111-0029 | | RECORD OF BOREHOLE No B5-21 | | | | SHEET 1 OF 1 | | METRIC | | | | | | | | |
|----------------------|---|--|--------|------|----------------------------|------------------|---|--------------------|----|----|-----|------------------------------------|-------------------------------------|-----------------------------------|---|--|
| W.P. 5188-06-01 | | LOCATION N 5045795.7 ; E 243518.8 | | | | ORIGINATED BY CR | | | | | | | | | | |
| DIST HWY 69 | | BOREHOLE TYPE Portable Equipment, BW Casing, Wash Boring | | | | COMPILED BY VA | | | | | | | | | | |
| DATUM Geodetic | | DATE July 24, 2009 | | | | CHECKED BY OK | | | | | | | | | | |
| 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 | | | | | | | | |
| 209.1 | GROUND SURFACE | | | | | | 20 | 40 | 60 | 80 | 100 | | | | | |
| 0.0 | TOPSOIL | | 1 | CS | | 209 | | | | | | | | | | |
| 0.1 | END OF BOREHOLE SPOON REFUSAL | | | | | | | | | | | | | | | |
| | NOTE: 1. Open borehole dry upon completion of drilling. | | | | | | | | | | | | | | | |

GTA-MTO 001 T:\PROJECTS\2007\07-1111-0029 (MRC, PARRY SOUND)\LOG\07-1111-0029-SHAWANAGA NBL & SBL-PHASE III.GPJ GAL-GTA.GDT 03/12/15 SAC



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



APPENDIX B

LABORATORY TEST RESULTS

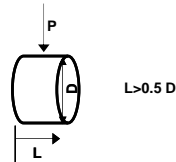
TABLE B1
POINT LOAD TEST RESULTS ON ROCK SAMPLES

| Borehole Number | Run Number | Sample Depth (m) | Sample Elevation (m) | Bedrock Description | Test Type | Is (50mm) (MPa) | Approx. ⁽¹⁾ UCS (MPa) |
|-----------------|------------|------------------|----------------------|---------------------|-----------|-----------------|----------------------------------|
| B5-02 | 1 | 3.2 | 195.5 | Granite Gneiss | Diametral | 6.737 | 94.0 |
| B5-02 | 1 | 3.4 | 195.3 | Granite Gneiss | Diametral | 5.311 | 74.0 |
| B5-02 | 1 | 3.6 | 195.1 | Granite Gneiss | Diametral | 5.253 | 74.0 |
| B5-05 | 1 | 2.0 | 197.6 | Granite Gneiss | Diametral | 8.523 | 119.0 |
| B5-05 | 1 | 2.1 | 197.5 | Granite Gneiss | Diametral | 6.741 | 94.0 |
| B5-05 | 1 | 2.2 | 197.4 | Granite Gneiss | Axial | 4.539 | 64.0 |
| B5-05 | 1 | 2.2 | 197.4 | Granite Gneiss | Axial | 4.225 | 59.0 |
| B5-05 | 1 | 2.3 | 197.3 | Granite Gneiss | Diametral | 6.311 | 88.0 |
| B5-07 | 1 | 8.1 | 191.6 | Granite Gneiss | Diametral | 8.264 | 116.0 |
| B5-07 | 1 | 8.1 | 191.6 | Granite Gneiss | Diametral | 7.917 | 111.0 |
| B5-07 | 3 | 9.6 | 190.1 | Granite Gneiss | Diametral | 9.274 | 130.0 |
| B5-07 | 3 | 9.7 | 190.0 | Granite Gneiss | Diametral | 7.688 | 108.0 |
| B5-14 | 1 | 10.3 | 188.6 | Granite Gneiss | Diametral | 7.020 | 98.0 |
| B5-14 | 1 | 10.4 | 188.5 | Granite Gneiss | Diametral | 5.205 | 73.0 |
| B5-14 | 1 | 10.9 | 188.0 | Granite Gneiss | Diametral | 3.827 | 54.0 |
| B5-15 | 1 | 7.5 | 191.3 | Granite Gneiss | Diametral | 3.923 | 55.0 |
| B5-15 | 2 | 8.3 | 190.5 | Granite Gneiss | Diametral | 5.676 | 79.0 |
| B5-15 | 2 | 8.7 | 190.1 | Granite Gneiss | Diametral | 6.636 | 93.0 |
| B5-18 | 6 | 4.9 | 200.3 | Granite Gneiss | Diametral | 9.870 | 138.0 |
| B5-18 | 8 | 5.9 | 199.3 | Granite Gneiss | Diametral | 9.870 | 138.0 |
| B5-18 | 9 | 7.7 | 197.5 | Granite Gneiss | Diametral | 6.179 | 87.0 |

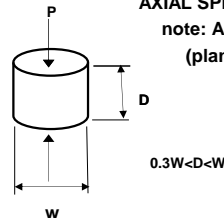
⁽¹⁾ $I_{s50} \times K$ (actual value could be confirmed by UCS testing), from ISRM. This range has been given based on $K=14$, calculated from I_{s50} Average (12 tests) equal to 6.6 Mpa on Diametral orientation and UCS Average (3 tests) equal to 91.7 Mpa.
 "Suggested Methods for Determining Point Load 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. 51-60.

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/OK
 Reviewed by: CN
 Reviewed By: JPD/JMAC

TABLE B2-1
SUMMARY OF UNCONFINED COMPRESSION TEST RESULTS
SHAWANAGA RIVER SBL BRIDGE STRUCTURE
HIGHWAY 69, TOWNSHIP OF PARRY SOUND
GWP 5111-07-00 AND WP 5188-06-01

| Borehole Number (Core Run) | Sample Depth (m) | Sample Elevation (m) | Rock Type | Core Diameter (mm) | Unconfined Compressive Strength (MPa) |
|---|-----------------------------|-------------------------------------|------------------|-----------------------------------|--|
| B5-14 (1) | 10.6 | 188.3 | Granite Gneiss | 47.8 | 99 |
| B5-07 (3) | 9.3 | 190.4 | Granite Gneiss | 47.5 | 118 |
| B5-05 (1) | 2.6 | 197.0 | Granite Gneiss | 47.5 | 58 |

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

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

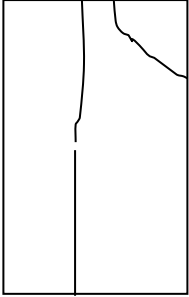
| SAMPLE IDENTIFICATION | | | |
|--------------------------------|--|---------------------------------|-----------|
| PROJECT NUMBER | 07-1111-0029 | RUN NUMBER | 1 |
| BOREHOLE NUMBER | B5-14 | SAMPLE DEPTH, m | 10.5-10.7 |
| TEST CONDITIONS | | | |
| MACHINE SPEED, mm/min | - | TYPE OF SPECIMEN | Rock Core |
| DURATION OF TEST,min | >2 <15 | L/D | 2.18 |
| SPECIMEN INFORMATION | | | |
| SAMPLE HEIGHT, cm | 10.42 | WATER CONTENT, (specimen) % | 0.19 |
| SAMPLE DIAMETER, cm | 4.78 | UNIT WEIGHT, kN/m ³ | 25.70 |
| SAMPLE AREA, cm ² | 17.95 | DRY UNIT WT., kN/m ³ | 25.65 |
| SAMPLE VOLUME, cm ³ | 186.99 | SPECIFIC GRAVITY, assumed | 2.70 |
| WET WEIGHT, g | 490.13 | VOID RATIO | 0.03 |
| DRY WEIGHT, g | 489.20 | | |
| VISUAL INSPECTION | FAILURE SKETCH | | |
| |  | | |
| TEST RESULTS | | | |
| STRAIN AT FAILURE, % | - | COMPRESSIVE STRESS, MPa | 99.2 |
| REMARKS: | DATE: | | 6/16/2009 |
| CHECKED BY: TVA | REVIEWED BY: | | JPD/JMAC |

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

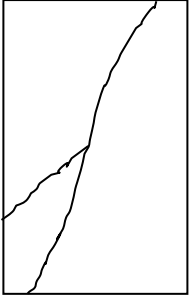
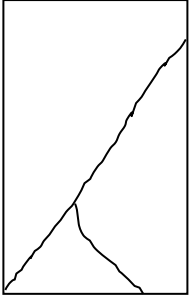
| SAMPLE IDENTIFICATION | | | |
|--------------------------------|--|---------------------------------|-----------|
| PROJECT NUMBER | 07-1111-0029 | RUN NUMBER | 3 |
| BOREHOLE NUMBER | B5-07 | SAMPLE DEPTH, m | 9.1-9.4 |
| TEST CONDITIONS | | | |
| MACHINE SPEED, mm/min | - | TYPE OF SPECIMEN | Rock Core |
| DURATION OF TEST,min | >2 <15 | L/D | 2.30 |
| SPECIMEN INFORMATION | | | |
| SAMPLE HEIGHT, cm | 10.94 | WATER CONTENT, (specimen) % | 0.17 |
| SAMPLE DIAMETER, cm | 4.75 | UNIT WEIGHT, kN/m ³ | 25.35 |
| SAMPLE AREA, cm ² | 17.72 | DRY UNIT WT., kN/m ³ | 25.30 |
| SAMPLE VOLUME, cm ³ | 193.86 | SPECIFIC GRAVITY, assumed | 2.70 |
| WET WEIGHT, g | 501.22 | VOID RATIO | 0.05 |
| DRY WEIGHT, g | 500.37 | | |
| VISUAL INSPECTION | FAILURE SKETCH | | |
| |  | | |
| TEST RESULTS | | | |
| STRAIN AT FAILURE, % | - | COMPRESSIVE STRESS, MPa | 118.0 |
| REMARKS: | DATE: | | 6/16/2009 |
| CHECKED BY: TVA | REVIEWED BY: | | JPD/JMAC |

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

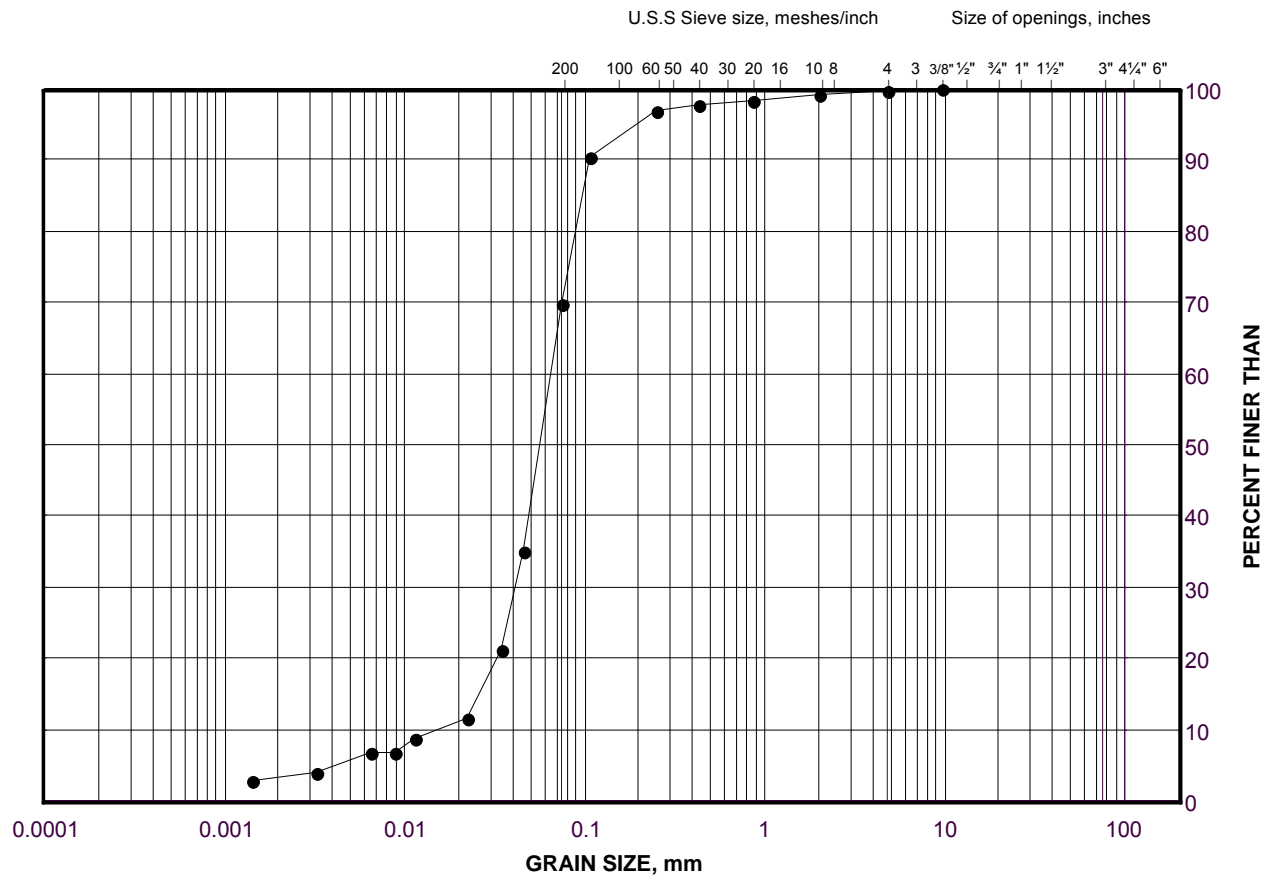
| SAMPLE IDENTIFICATION | | | |
|--------------------------------|--|---------------------------------|-----------|
| PROJECT NUMBER | 07-1111-0029 | RUN NUMBER | 1 |
| BOREHOLE NUMBER | B5-05 | SAMPLE DEPTH, m | 2.5-2.7 |
| TEST CONDITIONS | | | |
| MACHINE SPEED, mm/min | - | TYPE OF SPECIMEN | Rock Core |
| DURATION OF TEST,min | >2 <15 | L/D | 2.10 |
| SPECIMEN INFORMATION | | | |
| SAMPLE HEIGHT, cm | 9.95 | WATER CONTENT, (specimen) % | 0.08 |
| SAMPLE DIAMETER, cm | 4.75 | UNIT WEIGHT, kN/m ³ | 26.84 |
| SAMPLE AREA, cm ² | 17.68 | DRY UNIT WT., kN/m ³ | 26.82 |
| SAMPLE VOLUME, cm ³ | 175.95 | SPECIFIC GRAVITY, assumed | 2.70 |
| WET WEIGHT, g | 481.82 | VOID RATIO | -0.01 |
| DRY WEIGHT, g | 481.43 | | |
| VISUAL INSPECTION | FAILURE SKETCH | | |
| |  | | |
| TEST RESULTS | | | |
| STRAIN AT FAILURE, % | - | COMPRESSIVE STRESS, MPa | 57.9 |
| REMARKS: | DATE: | | 6/16/2009 |
| CHECKED BY: TVA | REVIEWED BY: | | JPD/JMAC |

GRAIN SIZE DISTRIBUTION

Sand and Silt (Upper Deposit)

South Pier (Pier 1)

FIGURE B1-1



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

LEGEND

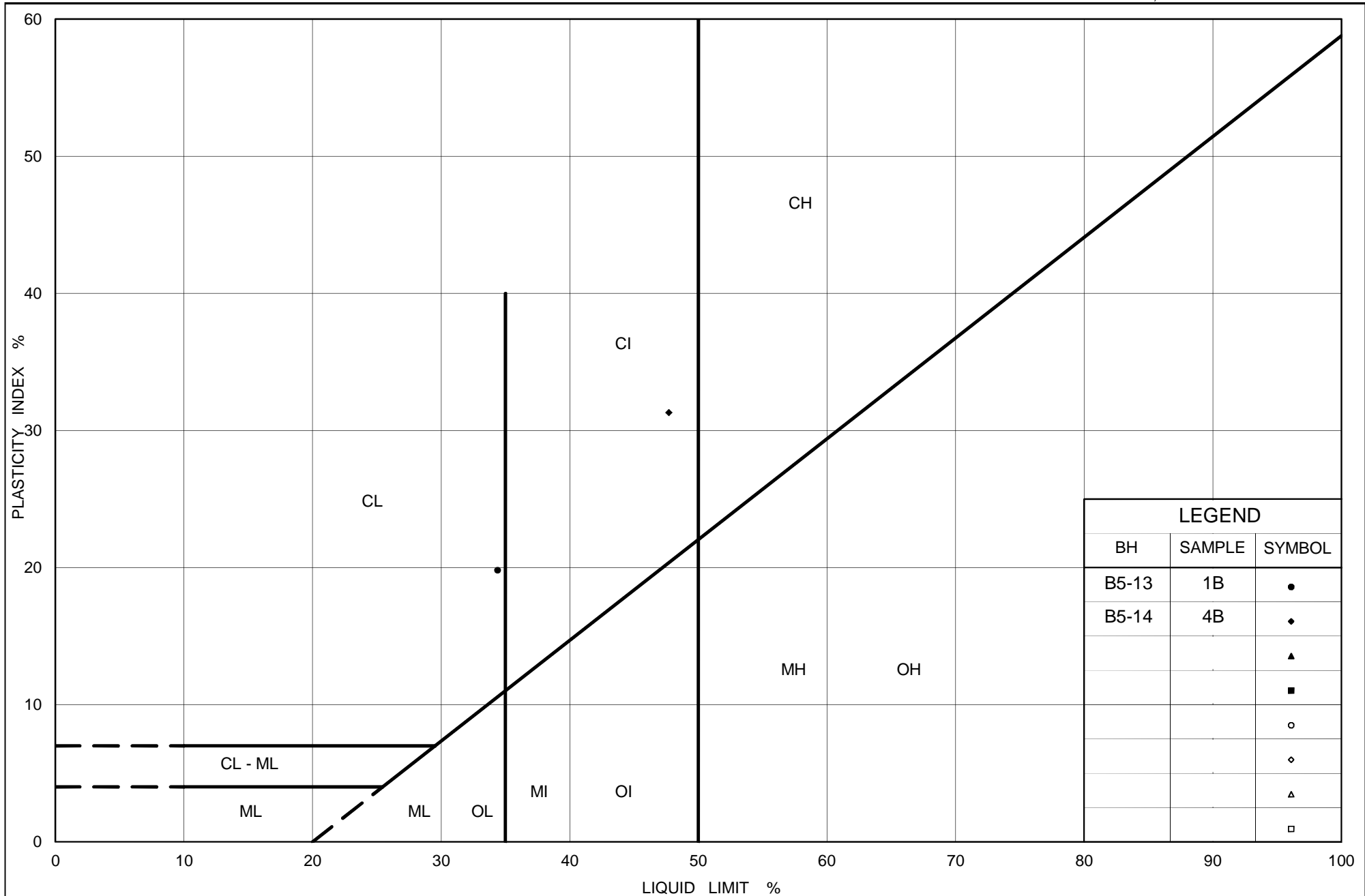
| SYMBOL | BOREHOLE | SAMPLE | ELEVATION(m) |
|--------|----------|--------|--------------|
| • | B5-14 | 4A | 190.7 |

Project Number: 07-1111-0029

Checked By: TVA

Golder Associates

Date: 08-Nov-13



Ministry of Transportation

Ontario

PLASTICITY CHART
Clayey Silt to Silty Clay
South Pier (Pier 1)

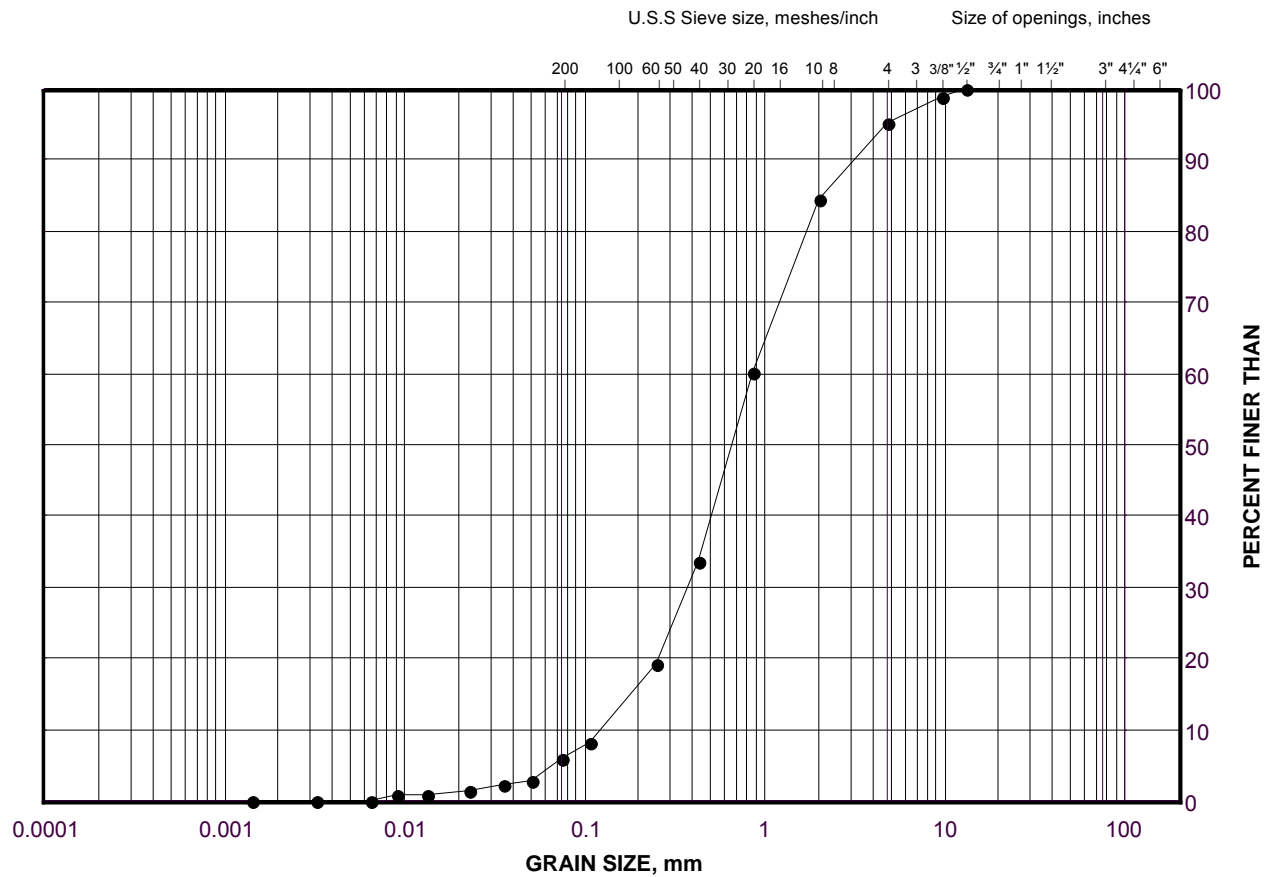
Figure No. B1-2

Project No. 07-1111-0029

Checked By: TVA

Sand (Lower Deposit)
South Pier (Pier 1)

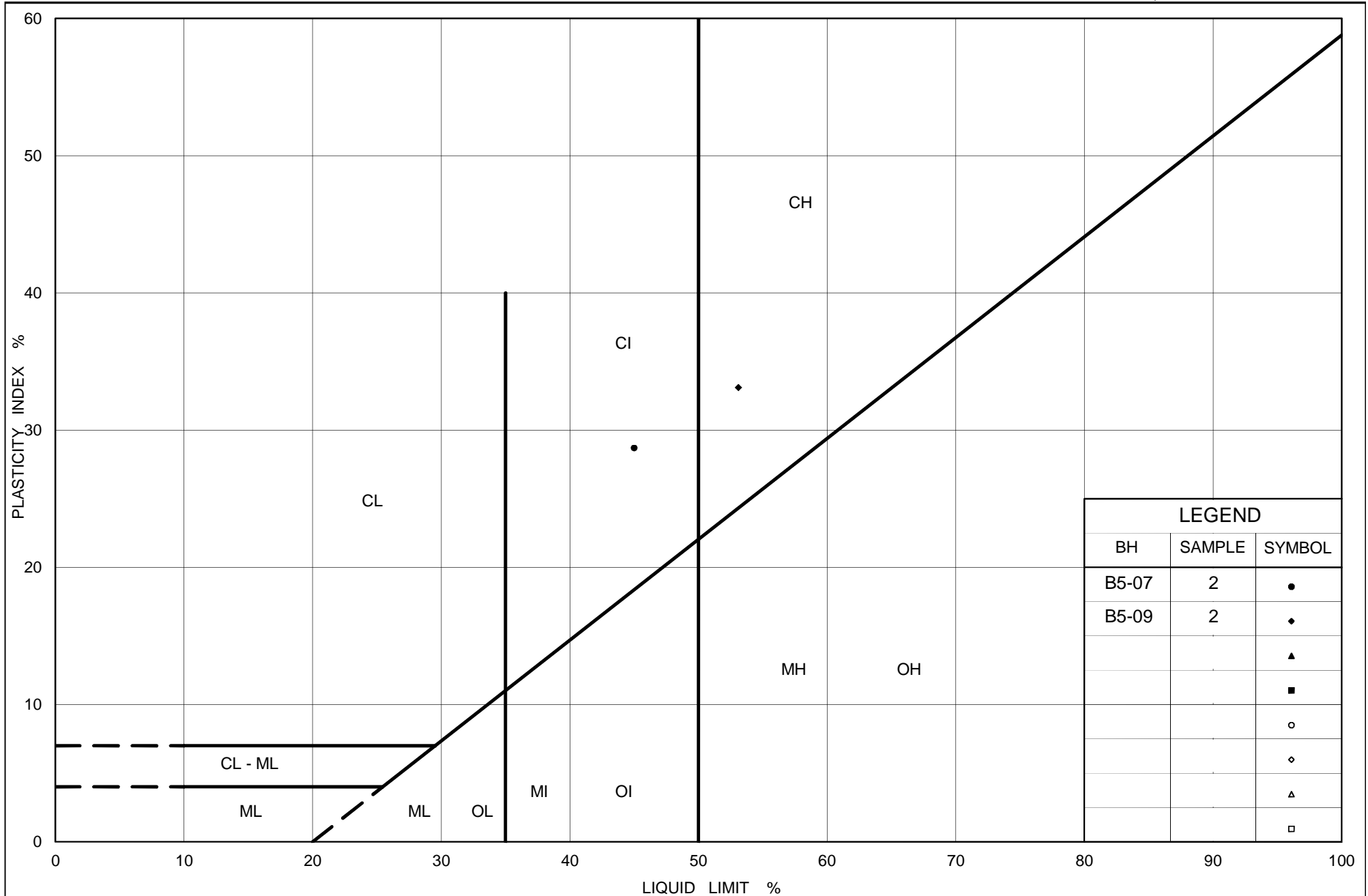
FIGURE B1-3



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

LEGEND

| SYMBOL | BOREHOLE | SAMPLE | ELEVATION(m) |
|--------|----------|--------|--------------|
| ● | B5-12 | 4 | 190.8 |



Ministry of Transportation

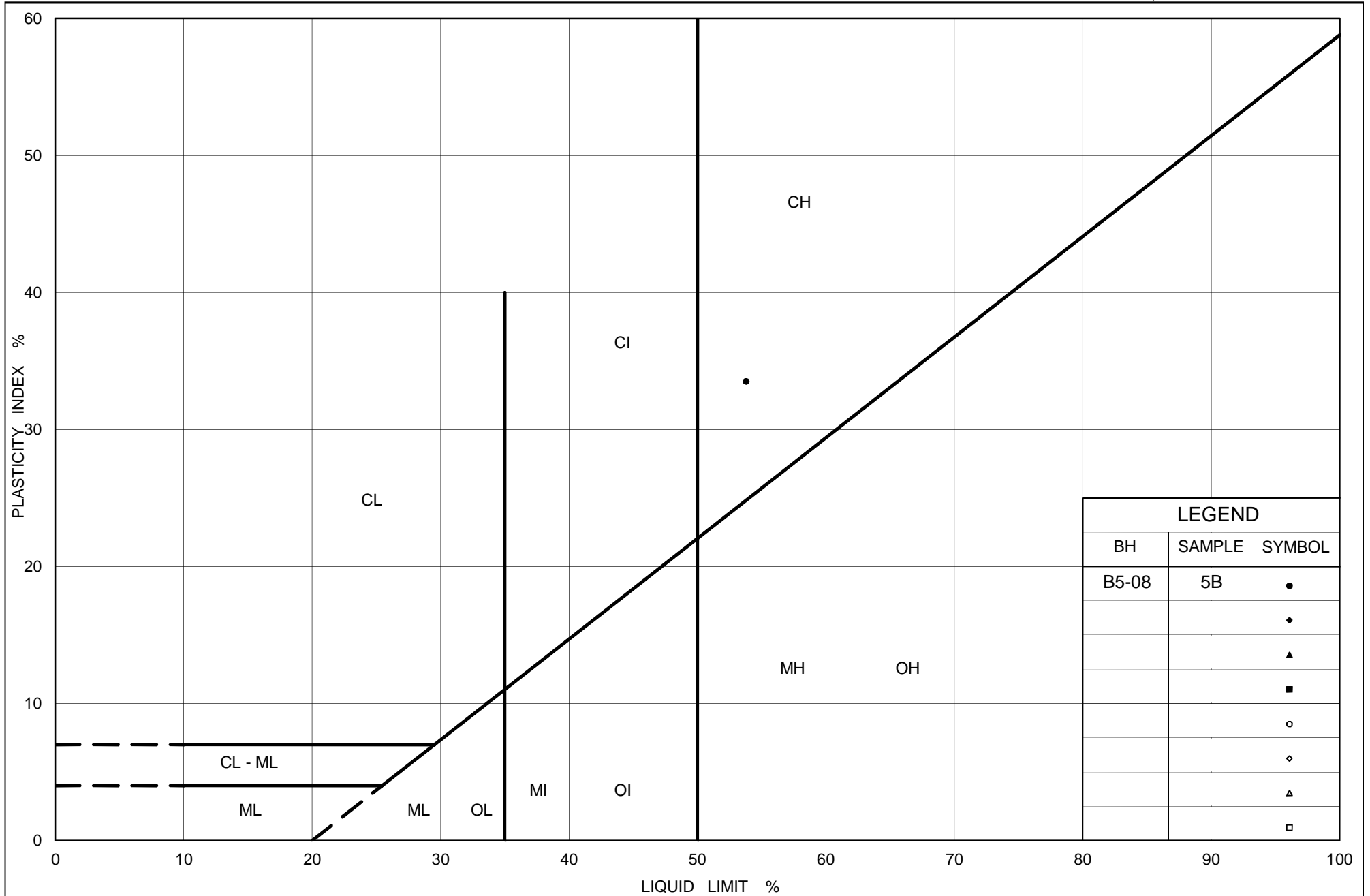
Ontario

PLASTICITY CHART Silty Clay to Clay North Pier (Pier 2)

Figure No. B1-4

Project No. 07-1111-0029

Checked By: TVA



Ministry of Transportation

Ontario

PLASTICITY CHART Clay North Pier (Pier 2)

Figure No. B1-5

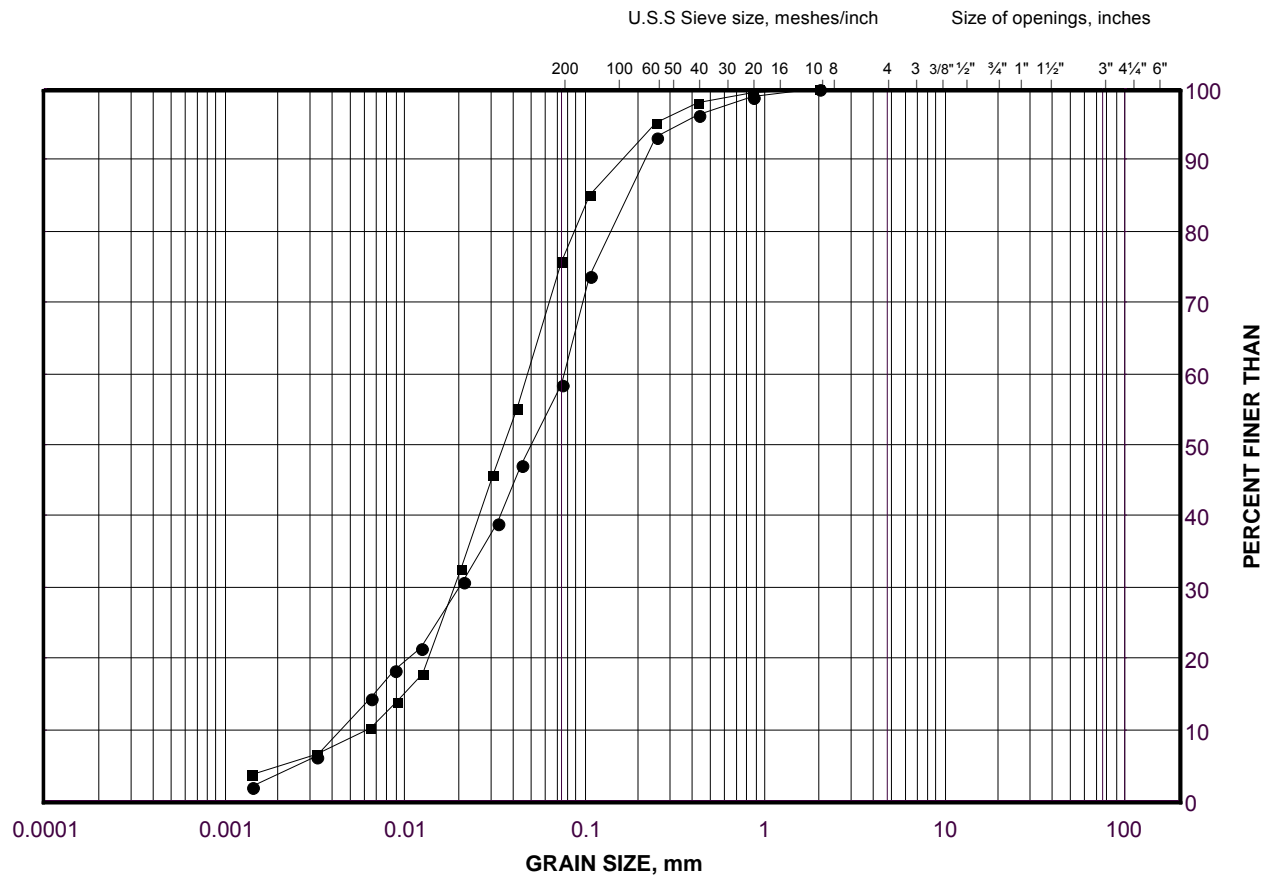
Project No. 07-1111-0029

Checked By: TVA

GRAIN SIZE DISTRIBUTION

Sand and Silt to Sandy Silt (Upper Deposit)
North Abutment and Approach

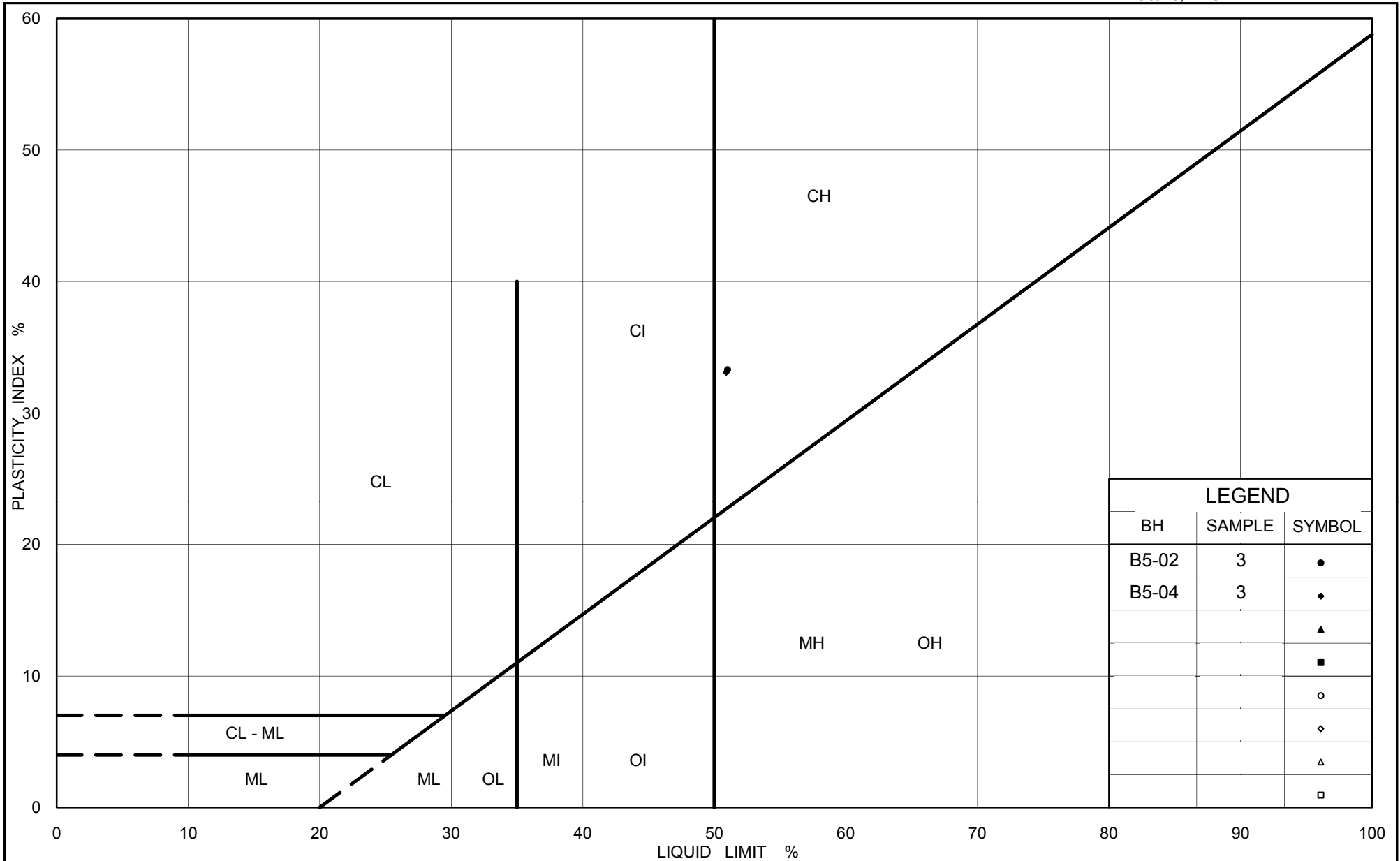
FIGURE B1-6



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

LEGEND

| SYMBOL | BOREHOLE | SAMPLE | ELEVATION(m) |
|--------|----------|--------|--------------|
| ● | B5-01 | 2B | 198.4 |
| ■ | B5-03 | 4 | 197.0 |



Ministry of Transportation

Ontario

PLASTICITY CHART
Clay
North Abutment and Approach

Figure No. B1-7

Project No. 07-1111-0029

Checked By: TVA



APPENDIX C

NON-STANDARD SPECIAL PROVISIONS / OPERATIONAL CONSTRAINT

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 River Bridge (SBL) | South 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 pint.

Rock dowels which fail to meet the acceptance criteria shall be replaced at the Contractor's expense and re-tested. If a rock dowel fails, 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 fragments 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 granular pad and 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

Operational Constraint

Granular Pad Construction In-the-Wet

This Operational Constraint alerts the Contractor of the constraints placed on construction of the Granular 'B' Type II (granular) pad in-the-wet (below water) at the north abutment of the Shawanaga River Southbound Lane (SBL) Bridge.

For the foundation at the north abutment comprised of the concrete footing to be constructed on a Granular 'B' Type II pad over bedrock after sub-excavation of all organic deposits and native soils, an approximately 2.5 m thick portion of the granular pad will be constructed below the water level (Elevation 198.5 m). For the portion of the granular pad constructed in-the-wet, the following construction constraints apply:

- The granular pad can be constructed only during the period(s) when river level is at or below Elevation 198.5 m;
- The elevation of the groundwater table at the north abutment should be at or below Elevation 198.5 m before construction work begins.
- Excavation and construction of the granular pad in-the-wet shall be carried out simultaneously;
- The maximum total thickness of granular pad to be constructed below water (in-the-wet) is 2.5 m; and ,
- The initial lift of backfill material is restricted to 1.0 m, followed by successive fill lifts 0.5 m thick or less.

All costs shall be deemed to be included in the Contract bid price for the various tender items and no additional payment will be made.

Operational Constraint

Earth Excavation and Granular Pad Construction

This Operational Constraint alerts the Contractor of the constraints placed on the Earth Excavation for Structure Foundations and Construction of the Granular Pad in-the-wet at the North Abutment of the Shawanaga River Southbound Lane (SBL)

Excavation and Construction

The excavation for and construction of the granular pad first lift below water shall be carried out simultaneously as specified in the Contract Documents.

The excavation of the overburden soil to expose the bedrock within the north abutment granular pad footprint shall be carried out in strips from north to south, and in segments across the length of the abutment footprint from east to west. For the first segment of excavation, once the removal of the overburden material has been completed exposing the bedrock surface, the construction of the granular pad in-the wet shall commence immediately in general accordance with the following procedure:

- The granular pad shall be constructed of Granular 'B' Type II material meeting the specification in OPSS.PROV 1010 (Aggregates);
- Construction of the granular pad shall be carried out in lifts; the initial lift shall not be more than 1 m thick, placed as the excavation proceeds to cover the entire footprint of the excavation. After placement of granular material for the initial lift, the surface of the lift shall be compacted/tamped throughout by the bucket of a backhoe/excavator of at least 45,000 kg operating weight with a minimum of 2 passes to cover the entire surface of the lift;
- Subsequent lifts shall not be more than 0.5 m thick up to the water surface and each lift shall be compacted using a backhoe/excavator as noted above. The surface of the final lift to or immediately below the water surface shall be compacted by a track bulldozer of minimum 15,000 kg operating weight for a minimum of 6 passes (3 forward and 3 backward) and overlapping successively across the granular pad; and
- The portion of the granular pad above water (in-the-dry) shall be constructed in accordance with the requirements of OPSS 902 (Excavating and Backfilling – Structures) and OPSS 501 (Compacting).

Protection of Granular Pad

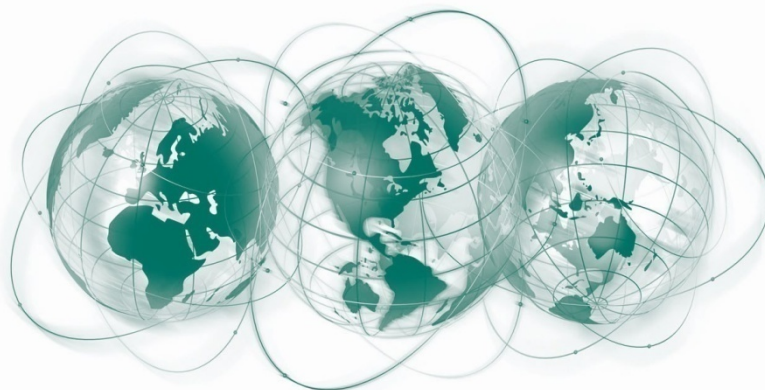
The Contractor shall be responsible for maintaining the stability of the excavation and the integrity of the foundation structure (i.e. the granular pad) constructed in-the-wet (below water level) as the work is carried out. Following the sequence of work outlined above, the granular pad above the water level shall be protected with Rip-Rap progressively as the pad is constructed.

All costs shall be deemed to be included in the Contract bid price for the various tender items and no additional payment will be made.

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

