



March 12, 2015

FOUNDATION INVESTIGATION AND DESIGN REPORT

**SHAWANAGA RIVER NBL BRIDGE STRUCTURE, SITE NO. 44-443/1
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. 5187-06-01 (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



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REPORT



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PART A

FOUNDATION INVESTIGATION REPORT

SHAWANAGA RIVER NBL BRIDGE STRUCTURE, SITE NO. 44-443/1

**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. 5187-06-01



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 detail foundation engineering services for the proposed three-span Shawanaga River Northbound Lanes (NBL) Bridge structure over the Shawanaga River (Site No. 44-443/1). 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 engineering services are outlined in MTO's Request for Proposal, dated July 2006. Golder's proposal for foundation engineering services associated with the Shawanaga River NBL 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 this project, dated July 4, 2007. The General Arrangement (GA) Drawing for the proposed Shawanaga River NBL Bridge structure was provided to Golder by MRC on January 15, 2009.

This report addresses the investigation carried out for the Shawanaga River NBL 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, test pitting and geological mapping of bedrock outcrops 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 NBL 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 runs generally in a southeast-northwest direction on 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 structure and associated approach embankments are to be situated on a bedrock outcrop on the south side of the Shawanaga River and on the side slope of the existing Highway 69 rock fill embankment on the north side of the river. On the south side of the river, the ground surface within the limits of the proposed structure ranges from about Elevation 212.9 m at the south approach embankment, to about Elevation 207.5 m at the south abutment and to as low as about Elevation 202.0 m at the south pier. On the north side, the ground surface ranges from about Elevation 201.0 m at the north pier to about Elevations 202 m to 205.4 m at the north abutment and about Elevations 200.0 m to 202.5 m at the north approach. All elevations are referenced to Geodetic datum.



3.0 INVESTIGATION PROCEDURES

3.1 Foundation Investigation

The field work for the Shawanaga River NBL Bridge subsurface investigation was carried out between February 6 and 9, 2009, between March 21 and 26, 2009, on April 2, 2009 and between July 20 and 26, 2009 during which time a total of twenty-one (21) boreholes and three (3) test pits were advanced: five (5) boreholes at the south abutment; five (5) boreholes at the south pier (Pier 3); four (4) boreholes and three (3) test pits at the north pier (Pier 4); five (5) boreholes at the north abutment; and one (1) borehole at each approach embankment. The boreholes, designated as Boreholes B4-01 to B4-21 and the test pits, designated as B4-TP1 to B4-TP3, were advanced at the locations shown in plan on Drawing 2.

The boreholes were drilled using a track-mounted Diedrich D-25 drill rig supplied and operated by Walker Drilling Co. Ltd. of Utopia, Ontario and portable equipment supplied and operated by OGS Inc. of Almonte, Ontario. The test pits were excavated using a 240 John Deere excavator operated by Weeks Construction Inc. of Parry Sound, Ontario. Hand excavation methods were used as appropriate depending on the terrain. The boreholes were advanced through the overburden using 165 mm O.D. solid-stem augers and/or 'BW' and 'NW' casing. Soil samples were obtained at intervals of depth of about 0.75 m using a 50 mm outside diameter (O.D.) split-spoon sampler driven by an automatic hammer in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586 Standard Test Method for Standard Penetration Tests and Split Barrel Sampling of the Soil). Boreholes advanced by portable equipment employed one-third ($\frac{1}{3}$) weight hammers lifted manually. Chunk samples were obtained in five (5) boreholes at locations of thin overburden over bedrock outcrops. Samples of the bedrock were obtained using an 'NQ' or 'BQ' size rock core barrel.

The boreholes at the foundation elements were typically advanced to casing 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 and extended into bedrock by coring at the north approach. The boreholes were drilled to depths of up to about 10.2 m below existing ground surface, including coring of bedrock for core lengths between about 3.0 m and 10.2 m in Boreholes B4-01 to B4-11, B4-15 and B4-18.

The test pits at the proposed north pier were excavated to bucket refusal to depths between 0.8 m to 2.1 m below existing ground surface and the bedrock surface was confirmed by exposure at the base of the test pits.

The groundwater conditions in the open boreholes and test pit excavations were observed during the drilling and on completion of test pitting operations and a total of four (4) piezometers were installed in Boreholes B4-01, B4-07, B4-15 and B4-18 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 boreholes. The boreholes and annulus surrounding the piezometer pipe above the screen sand pack were backfilled to the surface with bentonite pellets/grout. Piezometer installation details and water level readings are described on the Record of Borehole sheets presented in Appendix A. All boreholes in which standpipe piezometers were not installed were backfilled with bentonite upon completion in accordance with Ontario Regulation 903 Wells (as amended). The test pit excavations were backfilled with the excavated soil which was nominally compacted by the backhoe bucket and the ground surface was graded to the surrounding ground surface.

The field work was observed by members of our engineering and technical staff who located the boreholes and test pits, arranged for the clearance of underground services, observed the drilling, sampling and in situ testing operations, logged the boreholes and test pits, 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



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testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate. Classification testing (water content, Atterberg limits and grain size distribution) was carried out on selected samples. Strength tests such as unconfined compression and point load index, were carried out on specimen of the rock core. The results of the laboratory testing are included in Appendix B.

The as-drilled borehole and test pits locations and ground surface elevations were surveyed by a member of our technical staff, referenced to the survey stakes put down by Callon Dietz. The borehole / test pit locations given in the Record of Borehole/Drillhole sheets and Field Test Pit Logs 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 / Test Pit	Location (MTM Nad 83)		Ground Surface Elevation (m)	Depth Drilled / Excavated (m)
	Northing	Easting		
B4-TP1	5045843.4	243508.3	199.3	1.1
B4-TP2	5045846.6	243510.3	199.6	2.1
B4-TP3	5045847.0	243506.5	201.4	0.8
B4-01	5045857.8	243498.8	203.5	6.0
B4-02	5045853.5	243493.5	201.7	4.3
B4-03	5045856.8	243490.0	201.2	5.3
B4-04	5045871.6	243474.4	200.2	3.8
B4-05	5045846.6	243510.3	201.8	6.0
B4-06	5045847.0	243506.5	201.5	5.5
B4-07	5045850.8	243512.1	202.4	6.9
B4-08	5045846.8	243514.4	201.8	8.1
B4-09	5045866.1	243505.4	205.3	7.0
B4-10	5045860.9	243510.8	205.5	6.2
B4-11	5045815.2	243536.2	207.6	10.2
B4-12	5045818.8	243534.4	204.5	0.0
B4-13	5045818.8	243539.1	205.0	0.1
B4-14	5045819.1	243544.2	202.3	0.2
B4-15	5045823.7	243541.0	202.0	6.5
B4-16	5045802.5	243546.3	208.3	1.1



Borehole / Test Pit	Location (MTM Nad 83)		Ground Surface Elevation (m)	Depth Drilled / Excavated (m)
	Northing	Easting		
B4-17	5045805.3	243543.4	207.5	0.3
B4-18	5045807.7	243550.6	207.2	6.4
B4-19	5045809.0	243559.7	208.2	0.1
B4-20	5045811.8	243556.8	206.5	0.1
B4-21	5045793.8	243565.0	212.9	0.1

Additional field work was completed on August 26, 2009 for geological mapping of the bedrock outcrops to supplement the foundation investigation for the Shawanaga River NBL Bridge structure. The location of rock mapping is shown in plan on Figure C1 found in Appendix C.

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 and test pits advanced for this investigation, together with the results of the laboratory tests carried out on selected soil and bedrock

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.



core 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 of the south abutment and south pier consist of bedrock outcrops and surficial layers of topsoil over bedrock, while the areas of the north abutment and north pier consist of sand fill and rock fill over bedrock.

A detailed description of the subsurface conditions encountered in the boreholes and test pits advanced 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 B4-16 to B4-20) were advanced at the location of the south abutment and one (1) borehole (Borehole B4-21) was advanced on the centerline at the south approach. In general, the subsurface conditions consist of topsoil, underlain by silty sand at some locations over bedrock.

4.3.1 Topsoil

Up to about 0.1 m of topsoil was encountered at the ground surface at all borehole locations.

4.3.2 Silty Sand

A deposit of silty sand containing trace to some gravel, trace to some clay, rootlets and rock fragments was encountered below the topsoil in Boreholes B4-16 and B4-17. The top of this deposit was encountered at about Elevations 208.2 m and 207.4 m and its thickness is about 1.0 m and 0.2 m at the respective boreholes.

The Standard Penetration Test (SPT) 'N'-values measured within this deposit range from about 1 blow to 23 blows per 0.3 m of penetration, indicating a very loose to compact relative density.

The natural water content measured on two (2) samples of this deposit is about 14 percent and 64 percent with the higher water content value indicating the presence of organics.

4.3.3 Bedrock

Bedrock was encountered and core samples were recovered below the topsoil in Borehole B4-18. The presence of bedrock was inferred by refusal to split-spoon advancement in Boreholes B4-16 and B4-17 and by exposure in hand excavations at B4-19 to B4-21.

The depth to bedrock below ground surface and corresponding bedrock surface elevation is summarized below.



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Foundation Element / Approach Area	Borehole No.	Depth to Bedrock Surface (m)	Bedrock Surface Elevation (m)	Refusal Type
South Approach	B4-21	0.1	212.8	Hand Shovel
South Abutment	B4-16	1.1	207.2	Split-Spoon
	B4-17	0.3	207.2	Split-Spoon
	B4-18	0.1	207.1	Bedrock Cored
	B4-19	0.1	208.1	Hand Shovel
	B4-20	0.1	206.4	Hand Shovel

Across the east edge of the south abutment footprint (a distance of approximately 4.0 m between borehole locations), the bedrock surface elevation varies by about 1.7 m corresponding to an approximately 2.4H:1V slope or a dip of approximately 23° 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, coarse grained, highly foliated, slightly to moderately porous, dark to light grey and containing quartz veins. The Rock Quality Designation (RQD) measured on the core samples is between 83 percent to 100 percent, indicating a rock mass of good to excellent 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 100 percent and between 58 percent and 100 percent, respectively.

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 four (4) core samples of the granite gneiss bedrock from this location measured Is_{50} values ranging from about 1.1 MPa to 4.9 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 11, which was calculated based on a comparison of the two UC test results and the average of nine (9) of the corresponding point load strength test results. These values have been given for comparison only and should be interpreted together with the results of the UCS tests.

Based on the point load testing results, according to Table 3.5 in CFEM (2006)³ the granite gneiss bedrock at this location is classified as weak (R2, 5 MPa < UCS < 25 MPa) to medium strong (R3, 25 MPa < UCS < 50 MPa) at this location but is noted to be classified as strong (R4, 50 MPa < UCS < 100 MPa) in immediately adjacent areas.

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



4.3.4 Rock Mapping

Geological mapping of the bedrock outcrops within the vicinity of the existing south approach and abutment was carried out to collect information on the geological conditions of the general area of the south abutment of the Shawanaga River NBL Bridge structure and the Service Road (Site No. 9) bridge structure. The data collected includes estimates of rock strength, fracture characteristics and orientation, and groundwater conditions, which, combined with the geotechnical borehole investigation described in Section 4.3.3, addresses specific rock foundations engineering considerations.

The inspected areas are shown on Figure C1, in Appendix C. The data mapped from the rock outcrops is shown in stereonet format on Figure C2, in Appendix C. Joint condition data is presented below and refers to major joint sets defined on Figure C2.

Joint Set	Spacing (m)*	Surface condition
J1	0.2-3 (2.0)	Planar to Undulating, Slightly Rough to Rough
J2	0.5-2 (1.5)	Planar to Undulating, Smooth to Slightly Rough
J3	0.4-1.5 (1)	Planar to Undulating, Smooth to Slightly Rough

*Average spacing values given in parentheses.

Based on the site mapping (observations), the bedrock outcrops are described as consisting of dark orangeish to reddish grey, slightly weathered, coarse-grained, non-porous, strong to very strong igneous intrusive (granite). The bedrock is faintly foliated parallel to the J2 set and weathering on joint surfaces comprises of orange iron oxide staining or slight alteration. The rock mass mapped was drained above the water line of the adjacent Shawanaga River.

In general the bedrock exhibits strong rock characteristics. The stability of the rock cut slopes (faces) excavated for the bridge abutment will be controlled by the orientation, spacing and persistency of the discontinuities that exist within the rock mass.

The south abutment area bedrock mapping indicates two sub-vertical sets (J1 and J2) and a moderately dipping set J3, as shown on Figure C2. These joints appear to be moderately to widely spaced (0.5 m to greater than 1 m). For a cut face excavated with a strike angle of 245°, the outcrop data presented on Figure C2 suggests that the cut slope will be kinematically favourable with respect to wedge, planar and toppling failure. Should the cut face be oriented to align with the J3 set, there would be increased potential for planar failure along a joint surface with that orientation.

Based on the above observations, pre-supporting the rock face with dowels does not appear necessary for the possible cut orientation shown on Figure C2. Once excavated, the cut face and bench surface should be inspected by the geotechnical engineer to verify rock conditions.

4.3.5 Groundwater Conditions

A standpipe piezometer was installed in Borehole B4-18 to allow monitoring of the groundwater level at this location. Details of the piezometer installations 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. The groundwater levels measured in the piezometer installation are summarized below.



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Foundation Element	Borehole No.	Ground Surface Elevation (m)	Groundwater Elevation (m)	Date of Measurement
South Abutment	B4-18	207.2	203.3 202.5	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 parts 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 3)

A total of five (5) boreholes (Borehole B4-11 to B4-15) were advanced at the location of proposed south pier (Pier 3). In general, the subsurface conditions consist of bedrock (exposed at ground surface) or topsoil over bedrock.

4.4.1 Topsoil

A layer of topsoil up to about 0.2 m thick was encountered at the ground surface in Boreholes B4-13 and B4-14.

4.4.2 Cobbles/Rock Fragments and Bedrock

An approximately 0.5 m thick layer of cobbles/rock fragments was encountered at ground surface in Borehole B4-15. In general, the size of the recovered cobbles/rock fragments ranges from about 38 mm to 102 mm.

Bedrock was encountered and core samples were recovered in Boreholes B4-11 and B4-15, and bedrock was also observed to outcrop at Borehole B4-12. At Boreholes B4-13 and B4-14, the bedrock surface was identified upon being exposed by shovel excavation.

The depth to bedrock below ground surface and corresponding bedrock surface elevation is summarized below.

Foundation Element	Borehole No.	Depth to Bedrock Surface (m)	Bedrock Surface Elevation (m)	Refusal Type
South Pier (Pier 3)	B4-11	0.0	207.6	Bedrock Cored
	B4-12	0.0	204.5	Bedrock Outcrop
	B4-13	0.1	204.9	Hand Shovel
	B4-14	0.2	202.1	Hand Shovel
	B4-15	0.5	201.5	Bedrock Cored

Across the south pier from the southwest corner to the northeast corner of the pier footprint (a distance of approximately 9.5 m between borehole locations), the bedrock surface elevation varies by about 6.1 m corresponding to an approximately 1.6H:1V slope or a dip angle of approximately 32° 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, coarse grained, highly foliated, slightly to moderately porous, dark to light grey. The Rock Quality Designation (RQD) measured on the cores samples is between 65 percent to 100 percent, indicating a rock mass of fair to excellent quality, according to Table 3.10 in CFEM (2006)³, except for the near-surface core sample in Borehole B4-15 where a RQD of 45 percent was recorded, indicating a rock mass of poor quality. The Total Core Recovery (TCR) and Solid Core Recovery (SCR) of core samples are 100 percent and between 33 percent and 100 percent, respectively.

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 nine (9) core samples of the granite gneiss bedrock measured Is_{50} values ranging from about 2.9 MPa to 8.8 MPa.

Also presented in Table B1 are the estimated Uniaxial Compressive Strength (UCS) values for each sample tested for point load strength based on a relationship between Is_{50} and UCS and an estimated average correlation factor (K) of 11, which was calculated based on a comparison of the two UC test results and the average of nine (9) of the corresponding point load strength test results. These values have been given for comparison only and should be interpreted together with the results of the UCS test.

Based on the point load testing results, according to Table 3.5 in CFEM (2006)³ the granite gneiss bedrock at this location is classified as medium strong (R3, 25 MPa < UCS < 50 MPa) to strong (R4, 50 MPa < UCS < 100 MPa).

4.4.3 Groundwater Conditions

A standpipe piezometer was installed in Borehole B4-15 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 the boreholes advanced in this area were moist. 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 Pier (Pier 3)	B4-15	202.0	202.6* 201.7	July 28, 2009 August 26, 2009

*Water level above ground surface due to heavy rain.

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 parts 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.

4.5 North Pier (Pier 4)

A total of four (4) boreholes (Boreholes B4-05 to B4-08) were drilled and a total of three (3) test pits (Test Pits B4-TP1 to B4-TP3) were excavated at the location of the proposed north pier (Pier 4). In general, the subsurface conditions consist of sand fill and rock fill over bedrock.



4.5.1 Sand Fill

A deposit of fill comprised of sand containing some gravel, trace to some silt, trace clay and occasional cobbles was encountered at the ground surface in all borehole at this location. The top of the sand fill varies between about Elevations 202.4 m and 201.5 m and the thickness ranges from about 1.1 m to 2.2 m. A 0.1 m thick layer of sand and gravel fill containing some silt, organics and wood fragments was encountered in Borehole B4-06 immediately over the bedrock surface.

The SPT 'N'-values measured within the fill deposit range from 5 blows to 20 blows per 0.3 m of penetration, indicating a loose to compact relative density. A SPT 'N'-value of 23 blows per 0.2 m of penetration was recorded in Borehole B4-08 immediately above the rock fill deposit.

The natural water content measured on samples of this fill deposit ranges from about 5 percent to 12 percent.

A grain size distribution of a sample from this deposit is shown on Figure B1-1 in Appendix B.

4.5.2 Rock Fill

Rock fill containing sand, topsoil and rootlets was encountered at the ground surface in Test Pits B4-TP1 to B4-TP3 and below the sand fill in all boreholes except at Borehole B4-06. The top of the rock fill deposit varies between about Elevations 201.4 m and 199.3 m and the thickness of the deposit ranges from about 0.4 m to 2.1 m.

4.5.3 Bedrock

Bedrock was encountered below the rock fill or the sand to sand and gravel fill and core samples were recovered at all borehole locations. The bedrock surface was exposed in all test pits locations.

The depth to bedrock below ground surface and corresponding bedrock surface elevation is summarized below.

Foundation Element	Borehole / Test Pit No.	Depth to Bedrock Surface (m)	Bedrock Surface Elevation (m)	Refusal Type
North Pier (Pier 4)	B4-05	2.6	199.2	Bedrock Cored
	B4-06	1.9	199.6	Bedrock Cored
	B4-07	2.5	199.9	Bedrock Cored
	B4-08	3.1	198.7	Bedrock Cored
	B4-TP1	1.1	198.2	Bucket Refusal
	B4-TP2	2.1	197.5	Bucket Refusal
	B4-TP3	0.8	200.6	Bucket Refusal

Across the north pier from about the centre of the pier to the northwest corner of the pier footprint (a distance of approximately 4 m between test pit locations) the bedrock surface elevation varies by about 3.1 m corresponding to an approximately 1.3H:1V slope or a dip angle of approximately 38° 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 medium grained with feldspar banding, highly foliated, black, pink and grey colour. The Rock Quality Designation (RQD) measured on the core samples are typically between 30 percent and 80 percent, indicating a rock mass of poor to good quality according to Table 3.10 in



CFEM (2006)³. RQD values between 0 percent and 13 percent were measured on the cores samples from near the bedrock surface, indicating an upper rock mass of poor quality. The Total Core Recovery (TCR) and Solid Core Recovery (SCR) of samples recovered are typically between 74 percent and 100 percent and 11 percent and 80 percent, respectively.

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 six (6) samples of the granite gneiss bedrock measured Is_{50} values ranging from about 6.9 MPa to 11.1 MPa.

One (1) Unconfined Compression (UC) test was carried out in accordance to ASTM D7102 Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens on a selected core sample of granite gneiss bedrock from Borehole B4-06 and measured a compressive strength of about 78 MPa, as summarized in Table B2-1 and detailed in Table B2-2 in Appendix B.

Also presented in Table B1 are the estimated Uniaxial Compressive Strength (UCS) values for each sample tested for point load strength based on a relationship between Is_{50} and UCS and an estimated average correlation factor (K) of 11, which was calculated based on a comparison of the two UC test results and the average of nine (9) of the corresponding point load strength test results. These values have been given for comparison only and should be interpreted together with the results of the UCS 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 ($R4, 50 \text{ MPa} < \text{UCS} < 100 \text{ MPa}$) to very strong ($R5, 100 \text{ MPa} < \text{UCS} < 250 \text{ MPa}$).

4.5.4 Groundwater Conditions

A standpipe piezometer was installed in Borehole B4-07 to allow monitoring the groundwater level at the site. Details of the piezometer installation are shown on the Record of the Borehole and Drillhole sheets in Appendix A. In general the overburden samples taken in the boreholes advanced in this area were moist. 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 Pier (Pier 4)	B4-07	202.4	199.2	March 26, 2009
			199.2	April 16, 2009
			198.5	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 parts 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.

4.6 North Abutment and Approach Embankment

A total of five (5) boreholes (Boreholes B4-01 to B4-03, B4-09 and B4-10) were advanced at the location of the proposed north abutment and one (1) borehole (Borehole B4-04) was advanced on the west side of the north



approach. In general, the subsurface conditions consist of sand fill and rock fill underlain in places by layers of sand to sand and silt or clayey silt to silt, over bedrock.

4.6.1 Asphalt

An approximately 0.2 m thick layer of asphalt was encountered at the ground surface in Boreholes B4-09 and B4-10.

4.6.2 Topsoil

An approximately 0.1 m thick layer of topsoil was encountered at the ground surface in Borehole B4-03.

4.6.3 Sand to Sand and Gravel Fill

A deposit of fill comprised of sand containing trace gravel, trace silt, cobbles, boulders and organics to sand and gravel containing trace silt, organics and wood fragments was encountered at the ground surface in Borehole B4-01 and underlying the asphalt layer in Boreholes B4-09 and B4-10. The top of this fill deposit varies between about Elevations 205.3 m and 203.5 m and the thickness of this deposit ranges from about 0.7 m to 2.1 m.

A 0.5 m thick deposit of fill comprised of silt, some sand, trace to some clay, containing organics was encountered in Borehole B4-04 at ground surface.

The SPT 'N'-values measured within the sand to sand and gravel fill typically range from 20 blows to 98 blows per 0.3 m of penetration indicating a compact to very dense relative density. The higher blow counts measured are attributed to the presence of cobbles or boulders. A SPT 'N'-value of 5 blows per 0.3 m of penetration was recorded with the silt fill, indicating a loose relative density.

The natural water content measured on samples of the sand fill ranges from about 4 percent to 15 percent while the natural water content of about 22 percent was measured on a sample of the silt fill.

4.6.4 Rock Fill

Rock fill was encountered underlying the sand fill deposit in Borehole B4-01. The top of rock fill deposit was encountered at about Elevation 202.8 m and its thickness is about 2.0 m.

4.6.5 Sand to Sand and Silt

A cohesionless deposit comprised of sand containing trace gravel, organics and rootlets to sand and silt containing trace to some gravel and trace clay was encountered at the ground surface and below the sand and gravel fill in Boreholes B4-02 and B4-09, respectively. The top of this deposit was encountered at about Elevations 201.7 m and 203.0 m and its thickness is about 0.7 m and 0.6 m in the respective boreholes.

Two (2) SPT 'N'-values measured within this deposit are 4 blows and 5 blows per 0.3 m of penetration, indicating a loose relative density.



The natural water contents measured on two (2) samples within the sand and silt to sand deposit are about 16 percent and 21 percent.

The grain size distribution of a sample of this deposit is shown on Figure B1-2 in Appendix B.

4.6.6 Clayey Silt

A cohesive stratum comprised of clayey silt containing trace sand, organics and rootlets was encountered underlying the topsoil in Borehole B4-03. The top of the clayey silt stratum is at about Elevation 201.1 m and its thickness is about 0.6 m.

An SPT 'N'-value measured within clayey silt stratum is 4 blows per 0.3 m of penetration, suggesting a soft to firm consistency.

The natural water content and organic content measured on a sample of clayey silt is about 30 percent and 4 percent, respectively.

An Atterberg limits test was carried out on a sample of the clayey silt and yielded liquid and plastic limit of about 24 percent and 18 percent, respectively, corresponding a plasticity index of about 6 percent. The result of the Atterberg limits test is shown on the plasticity chart on Figure B1-3 in Appendix B and indicates that this material is a clayey silt of low plasticity.

4.6.7 Silt

A 0.7 m thick deposit of silt, trace sand and trace clay was encountered underlying the cohesive layer in Borehole B4-03, at about Elevation 200.5 m.

An SPT 'N'-value measured within the silt deposit is 26 blows per 0.3 m of penetration, indicating a compact relative density.

4.6.8 Cobbles and Boulders

A deposit of cobbles and boulders was encountered below the silt deposit in Borehole B4-03. The cobbles and boulders were encountered at about Elevation 199.8 m and the deposit is about 0.9 m thick.

4.6.9 Bedrock

Bedrock was encountered and core samples were recovered all boreholes at this location. The depth to the bedrock surface from the ground surface and the corresponding bedrock elevation is summarized below.



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Foundation Element / Approach Embankment	Borehole No.	Depth to Bedrock Surface (m)	Bedrock Surface Elevation (m)	Refusal Type
North Abutment	B4-01	2.7	200.8	Bedrock Cored
	B4-02	0.7	201.0	Bedrock Cored
	B4-03	2.3	198.9	Bedrock Cored
	B4-09	2.9	202.4	Bedrock Cored
	B4-10	2.3	203.2	Bedrock Cored
North Approach Embankment	B4-04	0.5	199.7	Bedrock Cored

Across the north abutment from the northwest corner to the southwest corner of the abutment footprint (a distance of approximately 4.7 m between borehole locations) the bedrock surface varies in elevation by about 2.1 m corresponding to an approximately 2.2H:1V slope or a dip angle of about approximately 24° from the horizontal.

Based on the cored samples of bedrock, the bedrock consists of granite gneiss bedrock. In general, the bedrock samples are described as fresh to slightly weathered, fine to medium grained with feldspar banding, faintly to moderately porous, weakly to moderately foliated, black, pink and grey colour. In general, the Rock Quality Designation (RQD) measured on the core samples ranges between about 54 percent and 100 percent, with values typically increasing with depth, indicating a rock mass of fair quality to excellent quality, according to Table 3.10 in CFEM (2006)³. RQD values ranging between about 0 percent and 39 percent were encountered on the upper portions of the bedrock in some boreholes, indicating an upper rock mass of very poor to poor quality at some locations. The Total Core Recovery (TCR) and Solid Core Recovery (SCR) of the core samples are typically between about 75 percent and 100 percent and 30 percent and 100 percent, respectively. TCR and SCR values ranging between 40 percent and 58 percent and between 0 percent and 17 percent, respectively, were measured on some of the cored samples generally corresponding to the zones of lower RQD.

Point load strength tests were performed on selected sample of the rock core. The axial and 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 axial point load index (Is_{50}) results from the laboratory tests carried out on three (3) core samples of the granite gneiss bedrock range from approximately 10.3 MPa to 10.8 MPa. The diametral tests carried out on fifteen (15) core samples of granite gneiss bedrock measured Is_{50} values ranging from about 4.6 MPa to 8.6 MPa.

One (1) Unconfined Compression (UC) test was carried out in accordance to ASTM D7102 Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens, on a selected core sample of the granite gneiss bedrock from Borehole B4-09 and measured a compressive strength of about 86 MPa, as summarized in Table B2-1 and detailed in Table B2-3 in Appendix B.

Also presented in Table B1 are the estimated Uniaxial Compressive Strength (UCS) values for each sample tested for point load strength based on a relationship between Is_{50} and UCS and an estimated average correlation factor (K) of 11, which was calculated based on a comparison of the two UC test results and the average of nine (9) of the corresponding point load strength test results. These values have been given for comparison only and should be interpreted together with the results of the UCS test.



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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 is classified as strong (R4, 50 MPa < UCS < 100 MPa) to very strong (R5, 100 MPa < UCS < 250 MPa).

4.6.10 Groundwater Conditions

A standpipe piezometer was installed in Borehole B4-01 to allow monitoring of the groundwater level at the site. Details of the piezometer installations are shown 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
North Abutment	B4-01	203.5	202.0 201.6	April 16, 2009 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 parts 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, Chris Radway and Indulis Dumpis, senior technicians with Golder, directed the drilling program. Messrs. Marc Rougier, David Chesser and Adam Horwitz completed the field work for rock mapping. This report was prepared by Ms. T. Veronica Ayetan, P.Eng. and Mr. Christopher Ng, 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.



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

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PART B

FOUNDATION DESIGN REPORT

SHAWANAGA RIVER NBL BRIDGE STRUCTURE, SITE NO. 44-443/1

**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. 5187-06-01



6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides foundation engineering design recommendations for the proposed Shawanaga River NBL Bridge structure. The recommendations are based on interpretation of the factual data obtained from the boreholes and test pits 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 NBL Bridge structure and approaches. The scope of work includes carrying out stability and settlement analyses, the assessment of foundation options and provision of geotechnical resistances for foundation alternatives. 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 NBL Bridge structure will be a three-span, pre-cast concrete girder structure consisting of two spans 16 m long and a center span 40 m long with the abutments and piers located south and north of the Shawanaga River. The alignment for the proposed Shawanaga River NBL Bridge is west of the existing Highway 69. The south approach and south abutment are located further to the west of the existing Highway 69 whereas the north approach embankment and north abutment are located adjacent to the existing Highway 69.

Based on the General Arrangement (GA) Drawing provided by MRC on January 15, 2009, the grade of the proposed Shawanaga River NBL bridge deck varies between about Elevation 208.2 m (south abutment) and Elevation 207.8 m (north abutment). The existing ground surface within the south approach embankment and abutment area varies from about Elevations 206.5 m to 212.9 m (at the investigated locations), but is generally above Elevation 208.1 m. Therefore rock cuts to as much as about 4.7 m below the existing ground surface are required at the location of the south approach embankment, and some small localized filling areas (up to about 1 m deep) may be required immediately behind the south abutment. The existing ground surface within the north approach embankment and abutment areas varies from about Elevations 200.2 m to 205.5 m (at the investigated locations); therefore the north approach embankment and abutment will require fills up to about 7.6 m high relative to the ground surface. At the south pier (Pier 3) the existing ground surface, comprised of bedrock outcrops, varies from about Elevations 202.0 m to 207.6 m (at the investigated locations). The ground surface at the north pier (Pier 4) varies from about Elevations 199.3 m to 202.4 m (at the investigated locations).



The normal (local) surface water level of the Shawanaga River is indicated on the GA Drawing provided by MRC to be at about Elevation 198.5 m.

6.2 Foundation Options

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

Pile foundations (H-piles and micropiles) have also been considered as an alternative system for support of the bridge structure, however, given the shallow depth to bedrock and considering the presence of the existing rock fill at the north pier (Pier 4) and north abutment of the proposed bridge structure, the installation of piles will be difficult. In addition, to achieve the minimum required pile lengths for an integral abutment design, installation of the piles would require significant excavation/trenching into the strong to very strong bedrock, which was generally encountered at the bridge location, and would likely be expensive.

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

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

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 are considered the preferred alternative for support of the structure.

6.3.1 Founding Level Alternatives

There are several founding level alternatives for support of the south abutment, south pier (Pier 3), north pier (Pier 4) and north abutment footings at the site. The alternatives are summarized below.

The following sections outline the recommendations for footing founding options, geotechnical resistance, 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 206.5 m to 208.3 m. A layer of topsoil about 0.1 m thick was encountered at the ground surface, underlain by a very loose to compact silty sand deposit, containing and rock fragments on the western edge of the proposed abutment. The overburden is underlain by gneiss bedrock.



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Based on the GA Drawing, the underside of the footing for the south abutment is proposed to be at about Elevation 204.2 m.

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

Borehole Location Within South Abutment	Borehole	Ground Surface Elevation (m)	Bedrock Surface Elevation (m)	Depth to Bedrock Below Existing Ground Surface (m)	Depth of Bedrock Excavation Required to Underside of Proposed Footing (m)
West Side	B4-16	208.3	207.2	1.1	3.0
	B4-17	207.5	207.2	0.3	3.0
Centre	B4-18	207.2	207.1	0.1	2.9
East Side	B4-19	208.2	208.1	0.1	3.9
	B4-20	206.5	206.4	0.1	2.2

Based on the borehole results, the underside of the proposed spread footing for the south abutment is up to about 3.9 m below the existing bedrock surface, therefore, bedrock excavation up to about 3.9 m deep and overburden material/topsoil excavation up to about 1.1 m deep will be required to reach the proposed founding level at Elevation 204.2 m. In general, the bedrock at or immediately below the proposed founding level at the borehole locations is of good to excellent quality with the RQD ranging from about 97 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. Recommendations for excavation of the bedrock and sequence of construction are provided in Section 6.9.

The preferred foundation option at the south abutment is a spread footing founded on granite gneiss bedrock at Elevation 204.2 m 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).

6.3.1.2 South Pier (Pier 3)

The existing ground surface in the area of the south pier (Pier 3) footprint varies between about Elevation 202.0 m and 207.6 m. In general, bedrock outcrops were encountered at the borehole locations at the south pier. Gneiss bedrock is overlain in places by topsoil or cobbles/rock fragments.

Based on the GA Drawing, the underside of the footing for Pier 3 is proposed to be at about Elevation 200.7 m.

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



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Borehole Location Within South Pier	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	B4-11	207.6	207.6	0.0	6.9
	B4-12	204.5	204.5	0.0	3.8
Centre	B4-13	205.0	204.9	0.1	4.2
East Side	B4-14	202.3	202.1	0.2	1.4
	B4-15	202.0	201.5	0.5	0.8

Based on the borehole results, the underside of the proposed spread footing for Pier 3 is up to about 6.9 m below the existing ground surface/bedrock surface, therefore, bedrock excavation up to about 6.9 m deep and topsoil excavation up to about 0.2 m deep and cobbles excavation up to about 0.5 m deep will be required to reach the founding level at Elevation 200.7 m. In general, the bedrock at or below the proposed founding level at the borehole locations is of fair to excellent quality with RQD ranging from about 65 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. Recommendations for excavation of the bedrock and sequence of construction are provided in Section 6.9.

The preferred foundation option at Pier 3 is a spread footing founded on granite gneiss bedrock, at Elevation 200.7 m, because of the bedrock high 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) as well as comparable to the founding level of the proposed adjacent wild life crossing culvert.

6.3.1.3 North Pier (Pier 4)

The existing ground surface in the area of the north pier (Pier 4) footprint varies between about Elevations 199.3 m and 202.4 m. In general a deposit of fill consisting of loose to compact sand intermixed with cobbles and organics was encountered at the ground surface at the borehole locations. A deposit of rock fill was encountered underlying the sand fill or at the ground surface in places. The fill deposit is underlain by gneiss bedrock.

Based on the GA Drawing, the underside of the footing for the Pier 4 is proposed to be at about Elevation 200.3 m.

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 in the boreholes and test pits at the south abutment is summarized below.



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Borehole Location Within North Pier	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 ¹ (m)
West Side	B4-06	201.5	199.6	1.9	0.7
	B4-TP1	199.3	198.2	1.1	2.1
	B4-TP3	201.4	200.6	0.8	-0.3
Centre	B4-05	201.8	199.2	2.6	1.1
	B4-TP2	199.6	197.5	2.1	2.8
East Side	B4-07	202.4	199.9	2.5	0.4
	B4-08	201.8	198.7	3.1	1.6

Note: ¹ Negative (-) implies underside of proposed footing (at Elevation 200.3 m) is below the existing bedrock surface.

Based on the borehole and test pit results within the footprint of the proposed Pier 4, the proposed elevation of the underside of the spread footing (at Elevation 200.3 m) is generally between about 0.4 m and 2.8 m above the bedrock surface, except at the location of Test Pit B4-TP3 where the proposed footing elevation is about 0.3 m below the bedrock surface elevation. Therefore, it will be necessary to either lower the footing level or excavate/expose the bedrock surface and place mass concrete up to the proposed footing level to create a level bearing surface. However, as the upper portion of the bedrock is of very poor to fair quality with RQD ranging from about 9 percent to 57 percent, consideration should be given to lowering the footing level to the lowest elevation of the bedrock surface as encountered in the boreholes at the Pier 4 location (i.e. to about Elevation 197.5 m) which will require rock excavation up to about 2.8 m deep and overburden excavation up to about 3.1 m deep across the foundation footprint. Alternatively, a combination of bedrock excavation and mass concrete placement could be considered to balance the volume of bedrock excavation and mass concrete required. Any loose or fractured bedrock encountered at the founding level or bedrock surface will need to be sub-excavated and removed prior to footing construction and/or mass concrete placement. All mass concrete construction should be in accordance with OPSS.PROV 904 (*Concrete Structures*).

As an alternative to supporting the north pier footing on bedrock (or mass concrete), consideration could be given to constructing a well compacted granular pad on the bedrock surface. However, the use of a granular pad poses several risks to the design and performance of the footing at this location. Firstly, in order to avoid stress concentrations within the footing that could occur due to differential settlement at the transition between a portion of the footing on bedrock and a portion on the granular pad, it would be necessary to remove additional bedrock across the footprint to about Elevation 199.3 m to ensure that the granular pad is a minimum 1 m thick at all locations below the footing, requiring up to about 1.3 m of rock cutting.

Secondly, given that the estimated depth of frost penetration at this site is 1.8 m, for a spread footing founded on Granular 'A' pad, a minimum 1.8 m of soil cover for frost protection will be required. In addition, the following should also be noted for the design of a spread footing founded on a Granular 'A' 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' 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).
- If adequate soil cover cannot be provided for the footing, rigid styrofoam insulation should be installed to compensate for the lack of soil cover and provide protection from frost penetration. In this regard, the MTO has adopted a thickness of 25 mm of styrofoam equivalent to 300 mm of soil cover.

Thirdly, given the elevation of the Regional Water Level for the river at this site (which is at Elevation 201.1 m), a footing founded on a Granular 'A' pad would require suitable erosion protection.

Given these concerns, the use of a granular pad to support the north pier footing is not recommended.

The preferred foundation option at the north pier (Pier 4) is to raise the underside of the footing to about Elevation 200.6 m and sub-excavate all fill and/or loose or fractured bedrock and to use mass concrete to raise the grade to Elevation 200.6 m. Given the strength compatibility between the concrete footing, mass concrete and bedrock as well as ease of placement for pad/footing construction and overall long-term stability of the concrete mass (i.e. minimal potential for undermining of the foundation over the life of the structure), this approach is considered the most suitable at this location.

6.3.1.4 North Abutment

The existing ground surface in the area of the proposed north abutment footprint varies from about Elevations 201.2 m to 205.5 m, from the western edge to the eastern edge. In general, a deposit of fill consisting of sand to sand and gravel, containing organics and cobbles was encountered at the ground surface across the eastern and center portion of the proposed footprint, underlain by rock fill or sand and silt, in places. At the western portion of the proposed footprint, a loose sand deposit or soft to firm clayey silt deposit was encountered at the ground surface. The clayey silt deposit is underlain by a compact silt layer and cobbles and boulders. The fill and the native soils are underlain by granite gneiss bedrock.

The proposed north abutment is located immediately to the west of the existing Highway 69 alignment and the new highway alignment merges with the existing roadway at the north approach embankment, as shown in plan on Drawing 2.

Based on the GA Drawing, the underside of the footing for the north abutment is proposed to be at about Elevation 201.5 m.

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



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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 ¹ (m)
West Side	B4-02	201.7	201.0	0.7	0.5
	B4-03	201.2	198.9	2.3	2.6
Centre	B4-01	203.5	200.8	2.7	0.7
East Side	B4-09	205.3	202.4	2.9	-0.9
	B4-10	205.5	203.2	2.3	-1.7

Note: ¹ Negative (-) implies underside (u/s) of proposed footing (at Elevation 201.5 m) is below the existing bedrock surface.

In general, based on the borehole results within the footprint of the proposed north abutment, the proposed elevation of the underside of the spread footing is between about 0.5 m and 2.6 m above the bedrock surface at three (3) borehole locations (Boreholes B4-01 to B4-03), and between about 0.9 m and 1.7 m below the bedrock surface at two (2) borehole locations (Boreholes B4-09 and B4-10). The bedrock at or immediately below the proposed founding level at the borehole locations is of poor to excellent quality with RQD ranging from about 34 percent to 100 percent. The existing fill and native soils at this location are not suitable for support of the footing and will have to be removed prior to footing construction or placement of mass concrete. In order to found the footing on a competent and level bearing stratum, consideration could be given to lowering the footing level to the lowest elevation of the bedrock surface (to about Elevation 198.9 m); however this option would require overburden excavation up to about 2.9 m and bedrock excavation up to about 4.3 m at the east side of the foundation footprint. To reduce the amount of bedrock excavation required, consideration should be given to lowering the founding level of the footing to Elevation 201.0 m and placement of mass concrete (up to 2.1 m thick) to raise the grade to the proposed founding level at the north-western corner of the footing, combined with bedrock excavation to depth of up to about 2.2 m below the bedrock surface on the east side of the proposed footprint to found the footing directly on the bedrock. Any loose or fractured bedrock encountered at the founding level or bedrock surface will need to be sub-excavated and removed prior to footing construction and/or mass concrete placement. Recommendations for excavation of the bedrock and sequence of construction are provided in Section 6.9. All mass concrete construction should be in accordance with OPSS.PROV 904 (*Concrete Structures*).

Consideration of other alternatives such as supporting the footing on a granular pad to avoid the use of mass concrete, is not recommended at this location given the additional depth of bedrock excavation that would be required to ensure a minimum 1 m thickness of granular below all areas of the footing (to avoid stress concentrations) as well as the special design and construction measures (and possibly future maintenance or inspection) that would likely be required to ensure that no loss of material from the granular pad occurs (in particular along the southern edge of the footing) over the life of the footing.

The recommended foundation option at the north abutment is to lower the founding level of the footing to Elevation 201.0 m and use mass concrete to raise the grade to the founding level on the north-western portion of the footing and excavate bedrock on the eastern portion of the footing depths of up to 2.2 m below the bedrock surface. This is considered the preferred foundation option due to the strength characteristics and compatibility



between the concrete footing, mass concrete and bedrock as well as ease of placement of concrete and overall long-term stability of the concrete mass (i.e. minimal potential for undermining of the foundation over the life of the structure), as well as comparable to the founding level of the proposed adjacent wild life crossing culvert.

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). 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	10,000 kPa	N/A
South Pier (Pier 3)	Spread Footing on Granite Gneiss Bedrock	10,000 kPa	N/A
North Pier (Pier 4)	Spread Footing on Mass Concrete placed directly on Granite Gneiss Bedrock*	10,000 kPa	N/A
North Abutment	Spread Footings on Granite Gneiss Bedrock and on Mass Concrete placed directly on Bedrock*	10,000 kPa	N/A

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

The geotechnical resistances provided above are given 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 used to form the pad is at least 25 MPa.

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



6.3.3 Resistance to Lateral Loads

The resistance to lateral forces/sliding resistance between the concrete footings and the bedrock should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction, $\tan \delta$, for the interface between the mass concrete / concrete footing and bedrock is:

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

The value presented above represents an unfactored value; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating 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 D.

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 this site (i.e. all loose or shattered rock to be removed prior to construction), frost susceptibility is not an issue.



6.3.5 Other Considerations

Given some of the unique features at this site, where steep bedrock outcrops are present in the areas of the foundation elements and the requirement to create a new sloped rock face in front of the proposed south abutment, the following additional foundation recommendations should be considered in the design.

6.3.5.1 Footing Set-Back from Rock Faces

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

6.4 Seismic Site Coefficient

6.4.1 Site Coefficient

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

6.4.2 Seismic Analysis Coefficient

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

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

6.5 Lateral Earth Pressures

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



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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*.

6.6 Approach Embankment Design

Based on the GA Drawing provided by MRC the proposed road grade for the new south approach and north approach embankments will be at about Elevation 208.0 m.

Within the limits of the proposed south approach embankment, the existing ground surface varies from about Elevation 206.5 m (at the eastern portion of the embankment immediately behind the abutment) to about Elevation 212.9 m as encountered at the borehole within the approach area about 20 m south of the south abutment. Bedrock outcrops are present at the south approach area extending towards the south abutment and the south pier. Given these current conditions, up to about 1.5 m of new embankment fill will be required at the eastern edge of the approach immediately behind the abutment. The remainder of the approach will consist of rock cuts up to about 4.9 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 200.2 m at the western edge of the approach to about Elevation 205.5 m recorded at the boreholes located on the existing Highway 69 along the eastern side of the approach. In general, embankment fill up to about 7.8 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 organics/topsoil. It is anticipated that any existing overburden 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.

At the north approach, the overburden in the embankment fill area is variable, comprising asphalt/sand to sand and gravel fill of the existing Highway 69 roadway embankment and/or topsoil, native clayey silt, sand and silt, silt and cobbles and boulders over bedrock. All topsoil, organic matter and clayey silt is 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.

For the analysis, the groundwater level is assumed to be at Elevation 199.8 m; however, the groundwater level could vary due to seasonal fluctuations and precipitation events.

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



6.6.1 Stability

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

6.6.1.1 Methodology

Limit equilibrium slope stability analyses were performed using the commercially available program Slide (Version 6), produced by Rocscience Inc., employing the Morgenstern-Price method of analysis. For all analyses, the factors of safety of numerous potential failure surfaces were computed in order to establish the minimum factor of safety. The factor of safety is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. A target minimum factor of safety of 1.3 is normally adopted in the design of embankment slopes under static conditions. This factor of safety is considered adequate for the embankments at this site considering the design requirements and the field data available. The stability analyses were performed to check that the target minimum factor of safety was achieved for the proposed embankment heights and geometries. In general, circular slip surfaces were employed in the analysis.

6.6.1.2 Parameter Selection

The soils that will remain in place below the proposed approach embankments (following stripping of organics and clayey silt) consist primarily of thin layers of cohesionless fill/native soils (i.e. loose silt fill, rock fill and loose to compact silty sand) underlain by bedrock.

For the cohesionless fill and native deposit, 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 fill, the values selected for the parameters were generally based on the lower relative density.

At all areas, the analyses assume that all organic soils, and clayey silts have been removed from below the embankment footprint prior to construction of the new embankments. The groundwater level measured in the piezometer installation in Borehole B4-18 at the south abutment was at about Elevation 202.5 m, within the bedrock, on August 26, 2009 and the ground water level measured in the piezometer installation in Borehole B4-01 at the north abutment was at about Elevation 201.6 m, within the existing rock fill, on August 26, 2009. For the stability analysis, it is assumed that the groundwater level is located at or slightly above the bedrock surface.

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



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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 Compact Silty Sand	19	--	28
North Approach	New Rock Fill	19	--	40
	Existing Rock Fill	19	--	35
	Compact to Very Dense Sand and Gravel Fill	20	--	32
	Loose to Compact Sand and Silt to Silt	19	--	28
	Cobbles and Boulders	19	--	30

6.6.1.3 Results of Analysis

The results of the stability analyses for the new rock fill embankments are shown below for the south and north approach embankments, respectively. The minimum factor of safety 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).

Location	Embankment Height at Critical Section (m)	Rock Fill Option	
		Recommended Side Slope Profile	Minimum Factor of Safety
South Approach STA 17+637.5	≤1.5*	1.25H : 1V	≥ 1.3
North Approach STA 17+715.0	7.5**		

Note: * South Approach Embankment is constructed through a rock cut.

** Including required depth of excavation.

6.6.1.4 Embankment Fill Types

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



Granular Fill

The main advantage of using granular fill is the ease of construction and the lack of post-construction settlements within the fill embankment itself. However, granular fill would require a larger volume of fill and wider right-of-way because the side slopes will be flatter than rock fill slopes. 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 north approach embankment and at the south approach embankment. Rock fill would likely be available locally. The disadvantage of using rock fill for the construction of high embankments is that some post-construction settlement of the embankment fill itself will occur within about the first year of construction.

6.6.2 Settlement

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

6.6.2.1 Methodology

To estimate the magnitude of the expected settlements, analyses were carried out on the critical sections of the proposed approach embankments using the commercially available program Settle3D (version 2.0), produced by Rocscience Inc.

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

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

6.6.2.2 Parameter Selection

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

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



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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 or slightly above the bedrock surface.

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

Approach Embankment	Foundation Soil	Thickness (m)	Unit Weight (kN/m ³)	Estimated Deformation Properties (MPa)
South Approach STA 17+637.5	N/A ¹	N/A ¹	N/A ¹	N/A ¹
North Approach STA 17+715.0	Loose to Compact Sand and Silt to Silt	0.7	19	10
	Cobbles and Boulders	0.9	19	20

Note: ¹ South Approach is in a rock cut, there are no foundation soils.

6.6.2.3 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 new embankment fill up to about 7.5 m high will be required (immediately behind the abutment area following stripping of organics and removal of clayey silt), up to about 15 mm of settlement of the foundation soils are estimated to occur. However, given the cohesionless nature of the foundation soils, this settlement is expected to occur during construction.

6.6.2.4 Settlement of New Embankment Fill

Granular Fill

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



Rock Fill

It is understood that rock fill is to be used for the construction of the 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 6.6.2.3. The magnitude of settlement of the rock fill depends on the type of rock/strength of particles, size and shape of rock particles, gradation of rock fill, total height/thickness of fill and the method of construction and sequence of placement. Rock fill should be placed, in a controlled manner (i.e. not end dumped) in accordance with OPSS.PROV 206 (*Grading*), as amended by SP 206F04 (*Rock Excavation, Grading*). In accordance with the “MTO Guideline for Rock Fill Settlement and Rock Fill Quantity Estimates” dated September 2010, settlement of rock fill placed in this manner is expected to be nominal and the magnitude is estimated to be up to about 0.75 percent of the effective height of the rock fill embankment. The estimated short-term settlement of rock fill for the approach embankments is presented below.

Approach Embankment	Maximum New Embankment Height ¹ (m)	Estimated Short-Term Settlement of Rock Fill (mm)
South Approach STA 17+637.5	1.5	10
North Approach STA 17+715.0	6.8 + 0.7 = 7.5	55

Note: ¹ Includes additional fill required after removal of maximum depth of topsoil/organics and clayey silt 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. about 5 mm) is expected to occur in the following (6) months.

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

Approach Embankment	Maximum New Embankment Height ¹ (m)	Estimated Long-Term Settlement of Rock Fill (mm)
South Approach STA 17+637.5	1.5	<5
North Approach STA 17+715.0	6.8 + 0.7 = 7.5	10

Note: ¹ Includes additional fill required after removal of maximum depth of topsoil/organics and clayey silt 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 100 days is required for the north approach rock fill embankment to allow for the majority of the short-term settlement to occur. The preload area for the north



approach embankment should extend further to the north of the north approach area to about STA 17+800 (refer to Project GWP: 5403-05-00, Swamp and Pond Crossing Foundation Investigation and Design Report by Golder Associates Ltd. (2011)) at which chainage the proposed Highway NBL grade crosses the existing Highway 69 embankment.

6.6.2.5 *Embankment Platform Widening*

In accordance with the requirements of MTO “Northern Region Engineering Directive NRE 98-200”, the construction of the embankments should include an allowance for platform widening (in 0.5 m increments) to accommodate settlements during construction as well as post-construction settlements, so that the minimum standard shoulder widths are maintained if future grade raises on the embankments are required. According to NRE 98-200, the need for future raises in road grade could occur due to settlement/compression of the embankment fill, settlement of the foundation soils and to accommodate future pavement overlays up to 200 mm thick. It is understood that this directive applies to all rock fill embankments as well as for granular fill embankments where widening restrictions are present (i.e. due to space/property issues, presence of a sensitive body of water and so on). It is further understood that 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 2 m given the thickness of the rock fill required in this area. Platform widening is not required at the south approach embankment given that the embankment fill is being placed directly on bedrock. Recommendations for the proposed Highway 69 NBL embankment beyond the north approach embankment area are contained in Project GWP: 5403-05-00, Swamp and Pond Crossing Foundation Investigation and Design Report by Golder Associates Ltd. (2011).

6.7 Subgrade Preparation and Embankment Construction

The layers of topsoil/organics and clayey silt 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. The following sections provide details on the recommendations for subgrade preparation and embankment construction.

6.7.1 Removal of Organic Materials

Based on the information from the boreholes obtained during the field investigation, layers of topsoil up to about 0.2 m thick can be expected in some areas of the new approaches. A deposit of soft clayey silt up to about 0.7 m thick was encountered at the north approach area. All topsoil/organic materials and the clayey silt layer



should be stripped from the plan limits of the approach embankment areas prior to fill placement for the new embankment(s).

6.7.2 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).

Considering that the new embankment fills for the north approach will be constructed on, over and adjacent to the existing Highway 69 embankment, the new fills should be keyed into / benched into the existing embankment fill slope in accordance with the requirements of OPSD 208.010 (*Benching of Earth Slopes*).

Given that the maximum height of the rock fill approach embankment at this site is less than 8 m, the incorporation of a mid-height berms (or successive benches) is not required at this site.

6.7.3 Temporary Shoring / Roadway Protection

At the south abutment and south pier, 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 existing adjacent Highway 69 roadway and existing Shawanaga River Bridge, temporary shoring and/or roadway protection is not anticipated to be required.

The new north pier and north abutment will, however, be located immediately adjacent to the west side of the existing Highway 69 roadway embankment. Depending on the construction staging, if the existing roadway is to remain in operation during the construction of the new north pier and abutment foundations, excavation support for protection of the existing roadway at both foundation areas will be required during construction. 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*).

Temporary shoring may also be required to support the fills and overburden soils at the north pier and north abutment during the excavations to expose the bedrock surface prior to placement of mass concrete. Design and construction of temporary shoring in these areas should take into account the following:

- Presence of rock fill;
- Presence of cobbles and boulders;
- Strong to very strong granite gneiss bedrock;
- Irregular / sloping surface of bedrock; and
- Groundwater table up to about 2.7 m above 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. It is noted that pre-drilling or excavation and replacement of rock fill and cobbles



/ boulders may be required prior to shoring installation. A method of providing lateral fixity to the toe of the shoring at the bedrock surface (i.e. toe pins) may also be required.

6.8 Design and Construction Consideration

6.8.1 Overburden Excavation

Based on the proposed or recommended elevations of the underside of the abutment footings and pier footings, and the recommendation to remove all overburden/fills down to the top of bedrock prior to mass concrete/footing construction, excavations for the bridge foundations will extend through the soils and depths summarized below. The elevation of the groundwater table as measured in the piezometers installed at each foundation unit is 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 Elevation (m)
South Abutment	204.2	1.1 (Soil)	Loose to Compact Silty Sand	202.5
		3.9 (Rock)	--	
South Pier (Pier 3)	200.7	0.5 (Rock)	Cobbles	201.7
		6.9 (Rock)	--	
North Pier (Pier 4)	200.3	3.1 (Soil)	Compact Sand and Gravel Fill; Rock Fill	198.5
	197.5 ¹	0.3 (Rock)	--	
North Abutment	201.0	2.9 (Soil – East Side)	Dense Sand and Gravel Fill, Rock Fill	201.6
		2.3 (Soil – West Side)	Soft Clayey Silt; Compact Silt; Cobbles and Boulders	
	198.9 ¹	2.2 (Rock)	--	

Note: ¹Lowest anticipated elevation of bedrock surface required to be excavated to / exposed prior to placement of mass concrete.

The fills and overburden soils requiring excavation as part of foundation construction are considered Type 3 soils according to the Occupational Health and Safety Act and Regulation for Construction Projects (OHSA). The excavation through the fills / 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).



6.8.2 Control of Groundwater and Surface Water

The elevation of the groundwater table in comparison to the founding level at the abutments and piers as well as the lowest elevation of the bedrock surface required to be exposed prior to placement of mass concrete is summarized in Section 6.8.1. Based on this information, the following comments are provided regarding the requirements for unwatering / groundwater control.

At the south abutment, since the measured elevation of the groundwater is below the proposed founding level, groundwater control is not anticipated to be required.

At the south pier, although the measured elevation of the groundwater is slightly (about 1 m) above the proposed founding level, it is expected that this may reflect a local perched water condition (or an unstabilized condition remaining from the rock coring operations) and should not affect the construction. As such, groundwater control is not anticipated to be required at this location.

At the north pier and north abutment, the measured level of the groundwater is about 1 m and 2.7 m, respectively, above the lowest elevation of the bedrock surface required to be exposed prior to placement of mass concrete. At these locations, it is anticipated that the groundwater condition is localized, therefore pumping from properly filtered sumps should be sufficient to control the groundwater during construction.

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 4.3.5, 4.4.3, 4.5.4 and 4.6.10); 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.

6.8.3 Obstructions

Boulders/rock fragment were encountered at the south pier and rock fill and cobbles and boulders were encountered at the north pier and north abutment, during borehole drilling and test pitting.

Conventional excavation equipment should be suitable for the majority of the excavation through the fills and overburden soils. However, the presence of boulders and rock fill 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 D.

6.8.4 Erosion Protection

The recommendation for the foundation elements is to found all footings either directly on bedrock or on mass concrete over bedrock. The need for and design of erosion protection for the footings founded at an elevation(s) within the fluctuating level of the Shawanaga River water levels should be assessed by the hydraulic engineer.

6.8.5 Permit To Take Water

Although the groundwater level as measured at the time of the investigation was only about 1 m above the lowest elevation of the bedrock surface, 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. Considering the above and given that the type of shoring and method of dewatering selected by the Contractor for the north pier and north abutment is unknown, a permit to take water may be required for this site.

6.9 Recommendations for Rock Excavations and Blasting

6.9.1 Rock Excavation

It should be noted that the bedrock at the site is generally classified as strong (R4) to very strong (R5). The two (2) Unconfined Compression (UC) tests carried out on bedrock core samples, measured Uniaxial Compressive Strength (UCS) of about 78 MPa and 86 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 6.3.5.1, the abutments and piers footings will be founded on the bedrock outcrops 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 6.3.5.1, a NSSP should be provided for vertical rock dowels to be installed behind the crest of the rock face (prior to any new rock excavation) in order to provide additional support to the rock face during blasting and following construction. All rock faces should be adequately cleaned and/or protected such that the integrity of the rock face/founding rock is maintained.

6.9.2 Blasting

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

It is recommended that all new rock cut faces in the area of the proposed structure foundations be inspected by a 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 Mr. Christopher Ng, 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.



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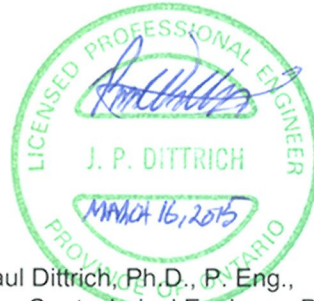
Report Signature Page

Christopher Ng, P. Eng.
Geotechnical Engineer, Associate



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

TVA/CN/JPD/JMAC/J



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

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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:

Settle3D (Version 2.0) by Rocscience Inc.
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.



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

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 208.010	Benching of Earth Slopes.
OPSD 3090.101	Foundation, Frost Penetration Depths for Southern Ontario.
OPSD 3101.150	Walls, Abutment, Backfill, Minimum Granular Requirement.
OPSD 3101.200	Walls, Abutment, Backfill, Rock.
OPSD 3102.100	Walls, Abutment, Backfill, Drain.
OPSD 3121.150	Walls, Retaining, Backfill, Minimum Granular Requirement.
OPSD 3190.100	Walls, Retaining and Abutment, Wall Drain.

Ontario Provincial Standard Specification:

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

Ontario Water Resources Act:

Ontario Regulation 903 Wells.



TABLES



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

TABLE 1
EVALUATION OF FOUNDATION ALTERNATIVES
G.W.P. 5111-07-00 / W.P. 5187-06-01

Option	Rank/ Option	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Spread Footings on Mass Concrete and/or Bedrock	1	<ul style="list-style-type: none"> ■ Relative ease of construction. ■ Reduced bedrock excavation (as compared with pile option). ■ Negligible post-construction settlement. 	<ul style="list-style-type: none"> ■ Bedrock excavation is required at the foundation elements as follows: <ul style="list-style-type: none"> ■ Up to about 3.9 m of rock cut at the south abutment. ■ Up to about 6.9 m of rock cut at the south pier (pier 3). ■ Up to about 2.2 m of rock cut at the north abutment. ■ Overburden excavation is required at the foundation elements as follows: <ul style="list-style-type: none"> ■ Up to about 1.1 m of silty sand excavation at the south abutment. ■ Up to about 0.5 m of boulder excavation at the south pier (pier 3). ■ Up to about 1.8 m of sand fill /rock fill silty sand excavation at the north pier (pier 4). ■ Up to about 4.5 m of sand to sand and gravel fill /rock fill, clayey silt, sand and silt and cobbles/boulders excavation at the north abutment. ■ Variable bedrock surface may require large quantities of mass concrete to achieve level footing, in particular at north pier and north abutment. ■ May require steel dowels to anchor mass concrete or footings to sloping bedrock surface. ■ Dewatering, cleaning bedrock surface and concrete placement required within shored and dewatered excavation (i.e. cofferdam) at the north 	<ul style="list-style-type: none"> ■ Lower relative costs compared with piled foundation options. ■ Additional costs required for rock blasting and mass concrete placement. ■ Additional costs required for installation of temporary shoring/cofferdam and dewatering. 	<ul style="list-style-type: none"> ■ Risk of not being able to properly seal cofferdam and achieving full dewatering within the cofferdam. ■ Risk of encountering cobbles and boulders during installation of cofferdam. Pre-drilling of sub-excavation may be required. ■ Variability in bedrock surface will impact mass concrete quantities and excavation depths. ■ Must take measures to control blasting techniques to ensure integrity of rock face below/in front of the footings or 'dental' concreting/repair may be required during construction.



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TABLE 1
EVALUATION OF FOUNDATION ALTERNATIVES
G.W.P. 5111-07-00 / W.P. 5187-06-01

Option	Rank/ Option	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Spread Footings on Granular 'A' Pads	Not Recommended	<ul style="list-style-type: none"> Relative ease of construction. Small post-construction settlement. 	<ul style="list-style-type: none"> Installation of cofferdam through cobbles and boulders on sloping bedrock and achieving adequate seal around base to restrict seepage inflow may be difficult. Does not allow for integral abutment design. 		
			<ul style="list-style-type: none"> Large footprint required to accommodate a granular pad beyond the footing limits and therefore more extensive rock blasting/shoring/excavation/dewatering will be necessary. Dewatering and placement of Granular 'A' engineered fill required within shored and dewatered excavation (i.e. cofferdam). Installation of cofferdam through cobbles and boulders on sloping bedrock and achieving adequate seal around base to restrict seepage inflow may be difficult. Sub-excavation of an additional 1 m of bedrock below footing level to allow for placement of a minimum 1 m thick Granular 'A' pad otherwise stress concentrations and potential cracking will occur on the footing. Does not allow for integral abutment design. 	<ul style="list-style-type: none"> Potentially lower relative costs compared with piled foundation options and footing on mass concrete or on bedrock option. Additional costs required for installation of temporary shoring and dewatering. Additional shoring costs compared to shallow footing on bedrock due to larger footprint required. Additional cost associated with sub-excavation of an additional 1 m into bedrock. 	<ul style="list-style-type: none"> Risk of groundwater seepage into the cofferdam resulting in inability to achieve adequate and consistent density (compaction) of the Granular 'A' pad; could result in differential settlement across footing. Moderate risk of undermining of the abutment in the long term if the granular pad erodes or sloughs. Inspection and future maintenance may be required.



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TABLE 1
EVALUATION OF FOUNDATION ALTERNATIVES
G.W.P. 5111-07-00 / W.P. 5187-06-01

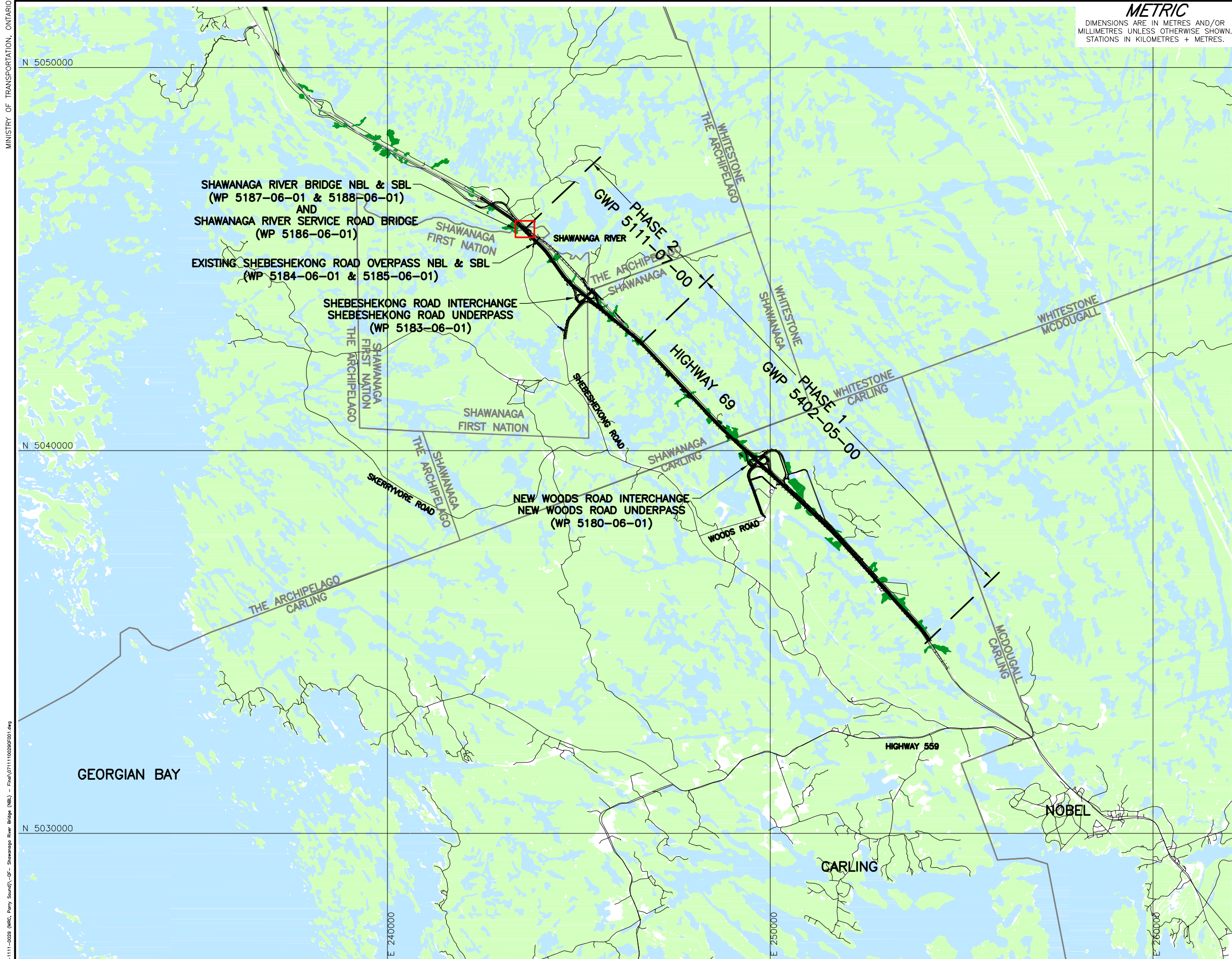
Option	Rank/ Option	Advantages	Disadvantages	Relative Costs	Risks/Consequences
H-Piles (HP310x110) installed within 0.6 m diameter Pre-Drilled Holes and Socketted into Bedrock (or driven to refusal on bedrock if minimum pile length can be achieved)	Not Recommended	<ul style="list-style-type: none"> Allows for integral abutment design provided minimum pile length is installed. Negligible post-construction settlement. Sub-excavation to expose bedrock surface not required. 	<ul style="list-style-type: none"> Heavier pile sections required to penetrate through cobbles and boulders/rock fill if pre-drilling is not carried out. Where rock socketting is required, temporary liners will also be required for groundwater control and support though overburden material. Minimum pile length may not be achieved due to shallow bedrock: bedrock trenching or rock socketting may be required. 	<ul style="list-style-type: none"> Additional costs associated with special equipment that will be required for creating sockets in bedrock where minimum pile lengths cannot be achieved. Additional costs associated with temporary liners. 	<ul style="list-style-type: none"> At the locations where piles are not socketted in rock, there is a risk of driven piles being out of alignment/location due to sloping bedrock, or of piles becoming damaged; larger pile cap to allow greater alignment tolerances would be required.
Micropiles (270 mm diameter grouted pile with central reinforcing bar and permanent steel casing on upper section of pile)	Not Recommended	<ul style="list-style-type: none"> Sub-excavation to expose bedrock surface not required. Negligible post construction settlement. 	<ul style="list-style-type: none"> Rock socketting required for fixity. Detailed micropile design and testing will be required. Shallow and variable depth to bedrock below north pier and north abutment footings may result in a “mixed” foundation (i.e. partially on piles, partially on bedrock). May not allow for integral abutment design. 	<ul style="list-style-type: none"> Additional cost associated with the detailed micropile design and pile load testing. 	<ul style="list-style-type: none"> Specialized foundation design to achieve required lateral resistance with small diameter piles may be required. Minimum pile length may not be achieved unless pile cap level is raised.

Prepared By: CN

Reviewed By: JPD/JMAC



DRAWINGS



PLAN

SCALE

1 0 1 2 km

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

CONT No.
WP No. 5187-06-01



HIGHWAY 69
SITE LOCATION PLAN

SHEET



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



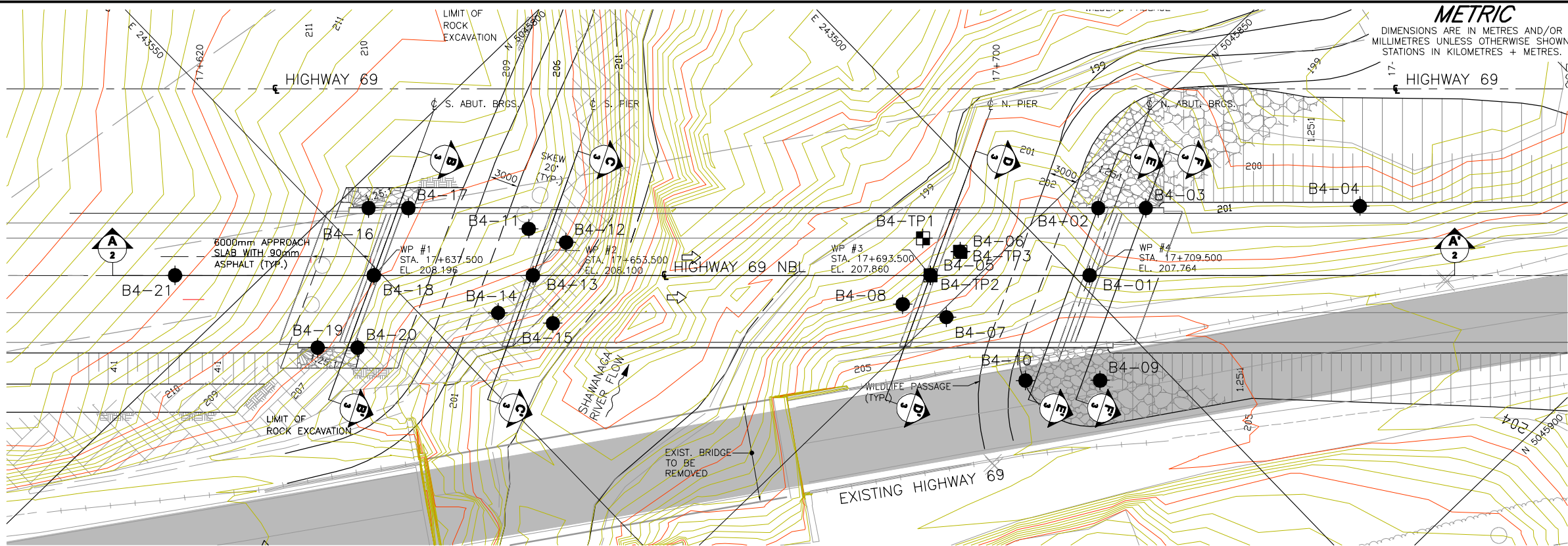
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NOT TO SCALE



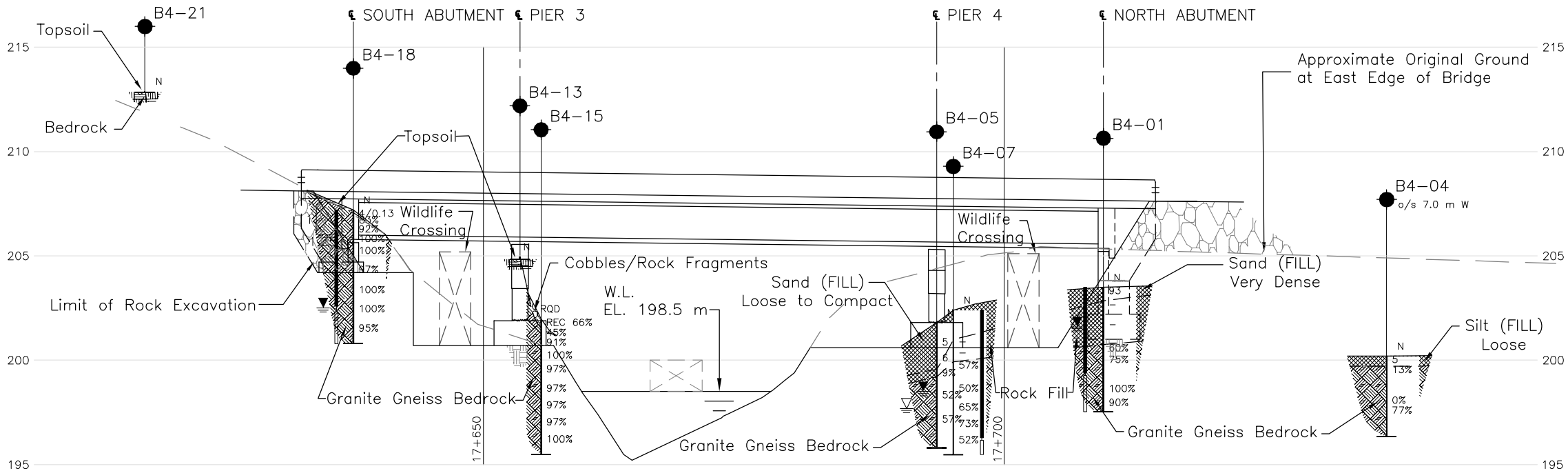
REFERENCE

Base Data - MNR NRVS, obtained 2004, CANMAP v2006.4
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Datum : NAD 83 Projection : MTM Zone 10

NO.	DATE	BY	REVISION
Geocres No. 41H-145			
HWY. 69	PROJECT NO. 07-1111-0029		DIST.
SUBM'D. VA	CHKD. VA	DATE: Mar. 2015	SITE: 44-443/1
DRAWN: DD/CD	CHKD. CN	APPD. JPD/JMAC	DWG. 1



PLAN



NOTES

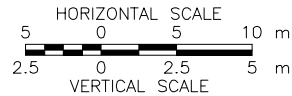
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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

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A-A'
2

CENTRELINE PROFILE



CONT No.
WP No. 5187-06-01

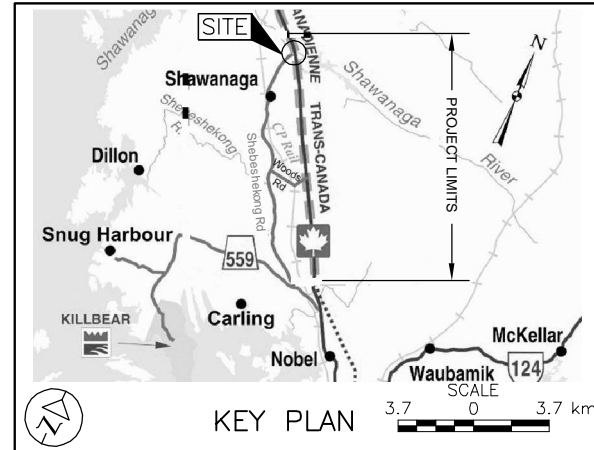


HIGHWAY 69
SHAWANAGA RIVER BRIDGE (NBL)
BOREHOLE LOCATIONS
AND SOIL STRATA

SHEET



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



LEGEND

- Borehole - Current Investigation
- Test Pit - Approximate Location
- 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)
- REC Recovery
- WL in piezometer, measured on 26/08/2009
- WL upon completion of drilling
- R Refusal

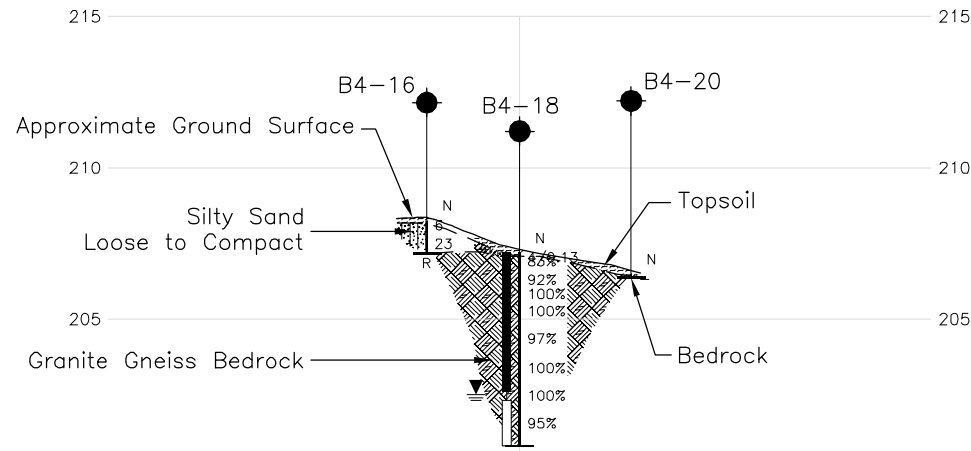
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		NORTHING	EASTING
B4-TP1	199.3	5045843.4	243508.3
B4-TP2	199.6	5045846.6	243510.3
B4-TP3	201.4	5045847.0	243506.5
B4-01	203.5	5045857.8	243498.8
B4-02	201.7	5045853.5	243493.5
B4-03	201.2	5045856.8	243490.0
B4-04	200.2	5045871.6	243474.4
B4-05	201.8	5045846.6	243510.3
B4-06	201.5	5045847.0	243506.5
B4-07	202.4	5045850.8	243512.1
B4-08	201.8	5045846.8	243514.4
B4-09	205.3	5045866.1	243505.4
B4-10	205.5	5045860.9	243510.8
B4-11	207.6	5045815.2	243536.2
B4-12	204.5	5045818.8	243534.4
B4-13	205.0	5045818.8	243539.1
B4-14	202.3	5045819.1	243544.2
B4-15	202.0	5045823.7	243541.0
B4-16	208.3	5045802.5	243546.3
B4-17	207.5	5045805.3	243543.4
B4-18	207.2	5045807.7	243550.6
B4-19	208.2	5045809.0	243559.7
B4-20	206.5	5045811.8	243556.8
B4-21	212.9	5045793.8	243565.0

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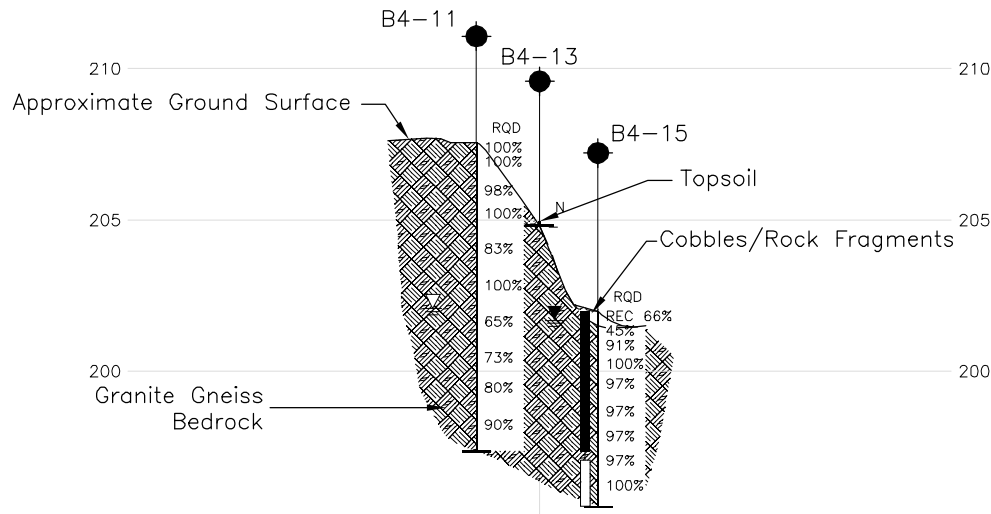
Base plans provided in digital format by MRC, drawing files 271XB01.DWG, 5271-XPB-SHAWANAGA.dwg, PR # 5377-02-00-PR-1.dwg, received October 1, 2007, and S6878-305-001GA.dwg, received October 31, 2014.

NO.	DATE	BY	REVISION
Geocres No. 41H-145			
HWY. 69	PROJECT NO. 07-1111-0029		DIST.
SUBM'D. VA	CHKD. VA/OK	DATE: Mar. 2015	SITE: 44-443/1
DRAWN: RJ/JS	CHKD. CN	APPD. JPD/JMAC	DWG. 2

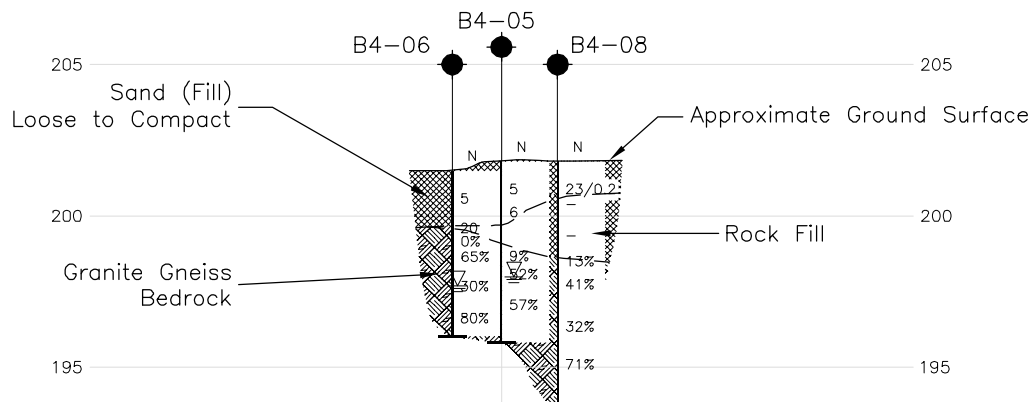




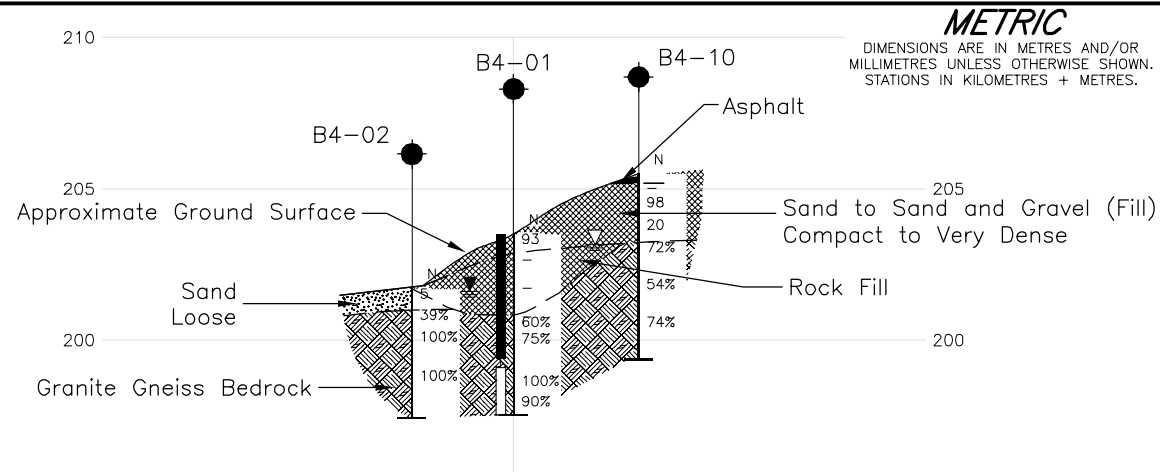
SOUTH ABUTMENT
B-B' 2
HORIZONTAL SCALE: 0 to 10 m
VERTICAL SCALE: 2.5 to 5 m



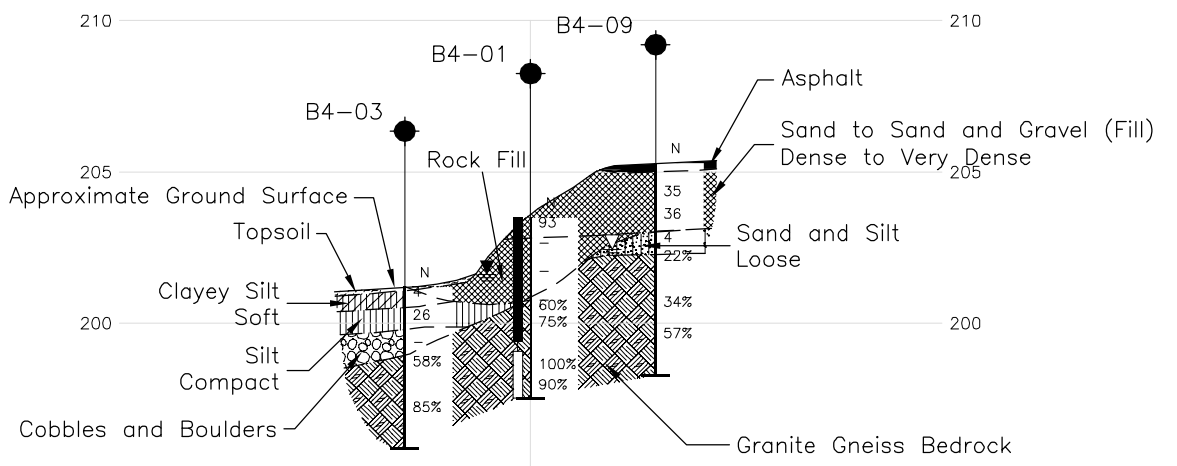
PIER 3
C-C' 2
HORIZONTAL SCALE: 0 to 10 m
VERTICAL SCALE: 2.5 to 5 m



PIER 4
D-D' 2
HORIZONTAL SCALE: 0 to 10 m
VERTICAL SCALE: 2.5 to 5 m



NORTH ABUTMENT (FRONT)
E-E' 2
HORIZONTAL SCALE: 0 to 10 m
VERTICAL SCALE: 2.5 to 5 m



NORTH ABUTMENT (BACK)
F-F' 2
HORIZONTAL SCALE: 0 to 10 m
VERTICAL SCALE: 2.5 to 5 m

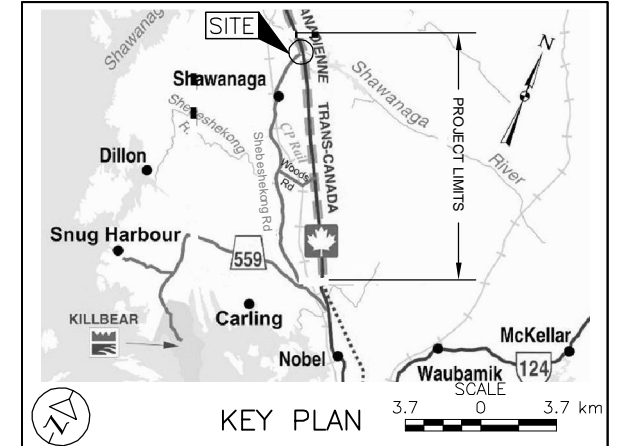
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 5187-06-01

HIGHWAY 69
SHAWANAGA RIVER BRIDGE (NBL)
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)		
REC	Recovery		
	WL in piezometer, measured on 26/08/2009		
	WL upon completion of drilling		
R	Refusal		

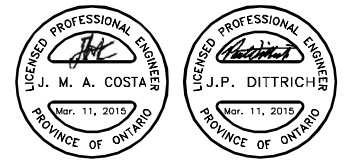
No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
B4-01	203.5	5045857.8	243498.8
B4-02	201.7	5045853.5	243493.5
B4-03	201.2	5045856.8	243490.0
B4-05	201.8	5045846.6	243510.3
B4-06	201.5	5045847.0	243506.5
B4-08	201.8	5045846.8	243514.4
B4-09	205.3	5045866.1	243505.4
B4-10	205.5	5045860.9	243510.8
B4-11	207.6	5045815.2	243536.2
B4-13	205.0	5045818.8	243539.1
B4-15	202.0	5045823.7	243541.0
B4-16	208.3	5045802.5	243546.3
B4-17	207.5	5045805.3	243543.4
B4-18	207.2	5045807.7	243550.6
B4-19	208.2	5045809.0	243559.7
B4-20	206.5	5045811.8	243556.8

NOTES

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NO.	DATE	BY	REVISION
Geocres No. 41H-145			
HWY. 69		PROJECT NO. 07-1111-0029	DIST.
SUBM'D. VA	CHKD. VA/OK	DATE: Mar. 2015	SITE: 44-443/1
DRAWN: RJ/JS	CHKD. CN	APPD. JPD/JMAC	DWG. 3



APPENDIX A

RECORD OF BOREHOLES, DRILLHOLES and FIELD TEST PIT LOGS



LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

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

(a) Index Properties (continued)

w	water content
w_l or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	plasticity index = $(w_l - w_p)$
w_s	shrinkage limit
I_L	liquidity index = $(w - w_p) / I_p$
I_C	consistency index = $(w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index = $(e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

Notes: 1
2

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



LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

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

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.

(b) Cohesive Soils Consistency

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

IV. SOIL TESTS

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

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

V. MINOR SOIL CONSTITUENTS

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



LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE

Fresh: no visible sign of weathering

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

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

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

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

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

BEDDING THICKNESS

Description	Bedding Plane Spacing
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

Description	Spacing
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

Term	Size*
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	

FIELD TEST PIT LOG

JOB NUMBER:	07-1111-0029	JOB NAME:	MRC / Highway 69 Four-Laning / Shawanaga NBL Structure		DATE:	Mar 21, 2009
TEST PIT NUMBER:	B4-TP1	APPROXIMATE LOCATION:	N 5045843.4; E 243508.3		APPROXIMATE ELEVATION:	199.3 m
MACHINE TYPE:	240 John Deere Excavator	TEST PIT SIZE:	N/A		DATUM:	Geodetic
TEMP/WEATHER:	Sunny, 2.5°C	CONTRACTOR:	Weeks Construction Inc.			

Elev.		Soil Description	Samples		Remarks
Depth			No.	Depth (m)	
From (m)	To (m)				
0.0	1.1	Rock fill, containing topsoil and rootlets	--	--	Exposed bedrock at a depth of 1.1 m
198.2					
1.1		End of Test Pit Bedrock			

Comments:

Water Conditions in Test Pit: Dry



JOB No.	07-1111-0029
TEST PIT No.:	B4-TP1
TECHNICIAN:	MR
COMPILED BY:	VA

FIELD TEST PIT LOG

JOB NUMBER:	07-1111-0029	JOB NAME:	MRC / Highway 69 Four-Laning / Shawanaga NBL Structure	DATE:	Mar 21, 2009
TEST PIT NUMBER:	B4-TP2	APPROXIMATE LOCATION:	N 5045846.6; E 243510.3	APPROXIMATE ELEVATION:	199.6 m
MACHINE TYPE:	240 John Deere Excavator	TEST PIT SIZE:	N/A	DATUM:	Geodetic
TEMP/WEATHER:	Sunny, 2.5°c	CONTRACTOR:	Weeks Construction Inc.		

Elev.		Soil Description	Samples		Remarks
Depth			No.	Depth (m)	
From (m)	To (m)				
0.0	2.1	Rock fill, containing sand, topsoil and rootlets	--	--	Exposed bedrock at a depth of 2.1 m
197.5					
2.1		End of Test Pit Bedrock			

Comments:

Water Conditions in Test Pit: Dry

For additional soil and bedrock details, see Record of Borehole B4-05



JOB No. 07-1111-0029
 TEST PIT No.: B4-TP2
 TECHNICIAN: MR
 COMPILED BY: VA

FIELD TEST PIT LOG

JOB NUMBER:	07-1111-0029	JOB NAME:	MRC / Highway 69 Four-Laning / Shawanaga NBL Structure		DATE:	Mar 21, 2009
TEST PIT NUMBER:	B4-TP3	APPROXIMATE LOCATION:	N 5045847.0; E 243506.5		APPROXIMATE ELEVATION:	201.4 m
MACHINE TYPE:	240 John Deere Excavator	TEST PIT SIZE:	N/A		DATUM:	Geodetic
TEMP/WEATHER:	Sunny, 2.5°C	CONTRACTOR:	Weeks Construction Inc.			

Elev.		Soil Description	Samples		Remarks
Depth			No.	Depth (m)	
From (m)	To (m)				
0.0	0.8	Rock fill, containing topsoil and rootlets	--	--	Exposed bedrock at a depth of 0.8 m
200.6					
0.8		End of Test Pit Bedrock			

Comments:

Water Conditions in Test Pit: Dry

For additional soil and bedrock details, see Record of Borehole B4-06



JOB No.	07-1111-0029
TEST PIT No.:	B4-TP3
TECHNICIAN:	MR
COMPILED BY:	VA

PROJECT		RECORD OF BOREHOLE		No B4-01		SHEET 1 OF 1		METRIC									
W.P. 07-1111-0029		LOCATION		N 5045857.8 ; E 243498.8		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		DATE		February 6 & 7, 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			SHEAR STRENGTH kPa									
203.5	GROUND SURFACE							20	40	60	80	100					
0.0	Sand, trace gravel, trace silt, containing organics and cobbles (FILL)		1	SS	93												
202.8	Very dense Brown Wet ROCK FILL		2	RC	-												
0.7			3	RC	-												
			4	RC	-												
200.8	Granite Gneiss (BEDROCK)		1	RC	REC 95%												RQD = 60%
2.7	Bedrock cored from depths of 2.7 m to 6.0 m		2	RC	REC 100%												RQD = 75%
	For bedrock coring details, refer to Record of Drillhole B4-01		3	RC	REC 100%												RQD = 100%
			4	RC	REC 96%												RQD = 90%
197.5	END OF BOREHOLE																
6.0	NOTE: 1. Water level measurements in Piezometer. Date Depth (m) Elev. (m) 16/04/09 1.5 202.0 26/08/09 1.9 201.6																

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: B4-01

SHEET 1 OF 1

LOCATION: N 5045857.8 ;E 243498.8

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	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
NO RC February 7, 2009		Continued from Record of Borehole B4-01		200.76 2.74	1								
3		GRANITE GNEISS Fresh to slightly weathered, medium grained, faintly to moderately porous, strong, weakly to moderately foliated, grey with black stippling, minor chloritic alteration and oxidation											
4					2								
5		GRANITE GNEISS Slightly weathered, medium grained, faintly to moderately porous, medium strong to strong, moderately to strongly foliated, grey and green, striping and stippling, moderate chloritic alteration		198.75 4.75	3								
6					4								8.6 MPa
6		END OF DRILLHOLE		197.53 5.97									
7													
8													
9													
10													
11													
12													

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 <u>07-1111-0029</u>	RECORD OF BOREHOLE No B4-02	SHEET 1 OF 1	METRIC
W.P. <u>5187-06-01</u>	LOCATION <u>N 5045853.5 ; E 243493.5</u>	ORIGINATED BY <u>MR</u>	
DIST <u></u> HWY <u>69</u>	BOREHOLE TYPE <u>165 mm O.D. Continuous Flight Solid Stem Augers and NW Casing, Wash Boring</u>	COMPILED BY <u>VA</u>	
DATUM <u>Geodetic</u>	DATE <u>February 7, 2009</u>	CHECKED BY <u>OK</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT						PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						WATER CONTENT (%)								
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED						w _p w w _L								
201.7	GROUND SURFACE							20	40	60	80	100										
0.0	SAND, trace gravel, containing organics and rootlets		1	SS	5																	
201.0	Loose Brown Wet																					
0.7	Granite Gneiss (BEDROCK)		1	RC	REC 89%															RQD = 39%		
	Bedrock cored from depths of 0.7 m to 4.3 m		2	RC	REC 100%															RQD = 100%		
	For bedrock coring details, refer to Record of Drillhole B4-02		3	RC	REC 100%															RQD = 100%		
197.4	END OF BOREHOLE																					
4.3	NOTE: 1. Open borehole dry 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

PROJECT: 07-1111-0029

RECORD OF DRILLHOLE: B4-02

SHEET 1 OF 1

LOCATION: N 5045853.5 ;E 243493.5

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	JN - Joint FLT - Fault SH - Shear VN - Vein CJ - Conjugate	BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage	PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular	PO - Polished K - Slickensided SM - Smooth RO - Rough VR - Very Rough	MB - Mechanical Break BR - Broken Rock NOTE: For additional abbreviations refer to list of abbreviations & symbols.	NOTES
1	NW casing February 7, 2009	Continued from Record of Borehole B4-02		201.00 0.70	1								
2	NQ RC February 7, 2009	GRANITE GNEISS Slightly weathered to fresh, fine to medium grained with feldspar banding, foliated, black, pink and grey			2								
3													
4		END OF DRILLHOLE		197.43 4.27	3								
5													
6													
7													
8													
9													
10													

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 B4-03		SHEET 1 OF 1		METRIC							
W.P. 5187-06-01		LOCATION N 5045856.8 ; E 243490.0		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		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					
201.2	GROUND SURFACE						20 40 60 80 100						
0.0	TOPSOIL		1	SS	4	201							OC=3.7%
200.5	CLAYEY SILT, trace sand, containing organics and rootlets Soft to firm		2	SS	26	200							
0.7	Brown to grey Moist												
199.8	SILT, trace sand, trace clay Compact												
1.4	Brown Wet		3	RC	-	199							
198.9	COBBLES AND BOULDERS												
2.3	Granite Gneiss (BEDROCK)		1	RC	REC 95%	198							RQD = 58%
	Bedrock cored from depths of 2.3 m to 5.3 m												
	For bedrock coring details, refer to Record of Drillhole B4-03		2	RC	REC 92%	197							RQD = 85%
195.9	END OF BOREHOLE					196							
5.3	NOTE: 1. Open borehole dry upon completion of overburden drilling.												

PROJECT: 07-1111-0029

RECORD OF DRILLHOLE: B4-03

SHEET 1 OF 1

LOCATION: N 5045856.8 ;E 243490.0

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	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 %														
								80 60 40 20	80 60 40 20	80 60 40 20	80 60 40 20												
								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												
NOTE: For additional abbreviations refer to list of abbreviations & symbols.																							
3	NQ RC February 8, 2009	Continued from Record of Borehole B4-03		198.91 2.29	1																		
		GRANITE GNEISS Slightly weathered to fresh, fine to medium grained with feldspar banding, foliated, black, pink and grey																					
4					2																		
5																							
6																							
7																							
8																							
9																							
10																							
11																							
12																							
END OF DRILLHOLE																							

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 B4-04		SHEET 1 OF 1		METRIC											
W.P. 5187-06-01		LOCATION N 5045871.6; E 243474.4		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 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 (%)			γ kN/m³	GR SA SI CL
							20 40 60 80 100	○ UNCONFINED + FIELD VANE	● QUICK TRIAXIAL × REMOULDED	Wp	W	WL	10 20 30				
200.2	GROUND SURFACE																
0.0	Silt, some sand, trace to some clay, containing organics (FILL)		1	SS	5		200										
199.7	Loose Brown Moist																
0.5	Granite Gneiss (BEDROCK)		1	RC	REC 75%		199										RQD = 13%
	Bedrock cored from depths of 0.5 m to 3.8 m		2	RC	REC 76%		198										RQD = 0%
	For bedrock coring details, refer to Record of Drillhole B4-04		3	RC	REC 98%		197										RQD = 77%
196.4	END OF BOREHOLE																
3.8	NOTE: 1. Open borehole dry 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

PROJECT: 07-1111-0029

RECORD OF DRILLHOLE: **B4-04**

SHEET 1 OF 1

LOCATION: N 5045871.6 ;E 243474.4

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.	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												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	K ₁₀	K ₅₀	K ₉₀																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																															
1	NW Casing February 8, 2009	Continued from Record of Borehole B4-04 GRANITE GNEISS Slightly weathered to fresh, fine to coarse grained with feldspar banding, foliated, black, pink and grey		199.71 0.49	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 B4-05		SHEET 1 OF 1		METRIC																					
W.P.		LOCATION		ORIGINATED BY		MR																							
DIST		BOREHOLE TYPE		COMPILED BY		TZ																							
DATUM		DATE		CHECKED BY		OK																							
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS			ELEVATION SCALE			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT			NATURAL MOISTURE CONTENT			LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES																								
201.8	GROUND SURFACE																												
0.0	Sand, some gravel, trace silt (FILL) Loose Brown Moist		1	SS	5																								
			2	SS	6																								
199.6	ROCK FILL																												
199.2	Granite Gneiss (BEDROCK)		1	RC	REC 74%																								
2.6	Bedrock cored from depths of 2.6 m to 6 m For bedrock coring details, refer to Record of Drillhole B4-05		2	RC	REC 93%																								
			3	RC	REC 100%																								
195.8	END OF BOREHOLE																												
6.0	NOTES: 1. Water level in open borehole at a depth of 4.1 m below ground surface (Elev. 197.7 m) upon completion of drilling. 2. Sand fill placed as backfill to Test Pit B4-TP2 to allow for bedrock coring.																												

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

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 B4-06			SHEET 1 OF 1			METRIC														
W.P. 5187-06-01			LOCATION N 5045847.0 ; E 243506.5			ORIGINATED BY MR																	
DIST HWY 69			BOREHOLE TYPE 165 mm O.D. Continuous Flight Solid Stem Augers and NW Casing, Wash Boring			COMPILED BY TZ																	
DATUM Geodetic			DATE March 24, 2009			CHECKED BY OK																	
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS			ELEVATION SCALE			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES																		
201.5	GROUND SURFACE																						
0.0	Sand, some gravel, trace to some silt, trace clay (FILL) Loose to compact Brown Moist		1	SS	5																		
199.7			2A	SS	20																		
1.9	Sand and gravel, some silt, containing organics and wood fragments (FILL) Compact Brown Moist Granite Gneiss (BEDROCK) Bedrock cored from depths of 1.9 m to 5.5 m For bedrock coring details, refer to Record of Drillhole B4-05		2B	RC	REC 100%																		
			2	RC	REC 96%																		
			3	RC	REC 95%																		
			4	RC	REC 100%																		
196.0	END OF BOREHOLE																						
5.5	NOTES: 1. Open borehole dry upon completion of drilling. 2. Sand fill placed as backfill to Test Pit B4-TP3 to allow for bedrock coring.																						

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: B4-06

SHEET 1 OF 1

LOCATION: N 5045847.0 ;E 243506.5

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	DISCONTINUITY DATA					HYDRAULIC CONDUCTIVITY K, cm ² /sec				Diametral Point Load Index (MPa)	RMC -Q AVG	NOTES																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																						
								TOTAL CORE %	SOLID CORE %			B Angle	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	10 °	10 °	10 °				10 °																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																					
2	NW Casing March 24, 2009	Continued from Record of Borehole B4-06		199.61 1.89	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 B4-07		SHEET 1 OF 1		METRIC															
W.P. 5187-06-01		LOCATION N 5045850.8 ; E 243512.1		ORIGINATED BY MR																	
DIST HWY 69		BOREHOLE TYPE 165 mm O.D. Continuous Flight Solid Stem Augers and NW Casing, Wash Boring		COMPILED BY TZ																	
DATUM Geodetic		DATE March 24, 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			GR SA SI CL		
202.4	GROUND SURFACE							20 40 60 80 100	20 40 60 80 100	10 20 30											
0.0	Sand, some gravel, containing cobbles (FILL) Brown Moist						202														
201.1	ROCK FILL		1	RC	-		201														
1.3			2	RC	-		200														
199.9	Granite Gneiss (BEDROCK)		1	RC	REC 92%		199														RQD = 57%
2.5	Bedrock cored from depths of 2.5 m to 6.9 m For bedrock coring details, refer to Record of Drillhole B4-07		2	RC	REC 100%		198														RQD = 50%
			3	RC	REC 100%		197														RQD = 65%
			4	RC	REC 100%		196														RQD = 73%
			5	RC	REC 94%																RQD = 52%
195.5	END OF BOREHOLE																				
6.9	NOTES: 1. Water level in open borehole at a depth of 3.1 m below ground surface (Elev. 199.3 m) upon completion of drilling. 2. Water level measurements in piezometer: Date Depth (m) Elev. (m) 26/03/09 3.2 199.2 16/04/09 3.2 199.2 26/08/09 3.9 198.5																				

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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 B4-08		SHEET 1 OF 1		METRIC											
W.P. 5187-06-01		LOCATION N 5045846.8 ; E 243514.4		ORIGINATED BY MR													
DIST HWY 69		BOREHOLE TYPE 165 mm O.D. Continuous Flight Solid Stem Augers and NW Casing, Wash Boring		COMPILED BY TZ													
DATUM Geodetic		DATE March 25 & 26, 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	GR SA SI CL
							20 40 60 80 100	20 40 60 80 100	W _p	W	W _L	10 20 30	kN/m ³				
201.8	GROUND SURFACE																
0.0	Sand, some gravel, trace silt (FILL) Compact Brown Moist		1	SS	23/0.2		201										
200.7	ROCK FILL		2	RC	-		200										
1.1			3	RC	-		199										
198.7	Granite Gneiss (BEDROCK)		1	RC	REC 100%		198										RQD = 13%
3.1	Bedrock cored from depths of 3.1 m to 8.1 m For bedrock coring details, refer to Record of Drillhole B4-08		2	RC	REC 98%		197										RQD = 41%
			3	RC	REC 100%		196										RQD = 32%
			4	RC	REC 98%		195										RQD = 71%
193.7	END OF BOREHOLE						194										
8.1	NOTE: 1. Water level in open borehole at a depth of 3.8 m below ground surface (Elev. 198.0 m) upon completion of drilling.																

PROJECT: 07-1111-0029

RECORD OF DRILLHOLE: B4-08

SHEET 1 OF 1

LOCATION: N 5045846.8 ;E 243514.4

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.	COLOUR % RETURN	FLUSH	RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.3 m	B Angle	DIP w.r.t CORE AXIS	DISCONTINUITY DATA			HYDRAULIC CONDUCTIVITY			Diametral Point Load Index (MPa)	RMC -Q AVG	NOTES			
								TOTAL CORE %	SOLID CORE %					TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	K, cm/sec	10 10 10 10				10 10 10 10		
		Continued from Record of Borehole B4-08		198.66																					
		GRANITE GNEISS Slightly weathered to fresh, fine to coarse grained with feldspar banding, foliated, black, pink and grey		3.14																					
4				1																					
				2																					
5																									
				3																					
6																									
				4																					
7																									
8				END OF DRILLHOLE		193.72																			
						8.08																			
9																									
10																									
11																									
12																									
13																									

8.6 MPa
11.1 MPa

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 B4-09		SHEET 1 OF 1		METRIC											
W.P. 5187-06-01		LOCATION N 5045866.1 ;E 243505.4		ORIGINATED BY ID													
DIST HWY 69		BOREHOLE TYPE 165 mm O.D. Continuous Flight Solid Stem Augers and NW Casing, Wash Boring		COMPILED BY TZ													
DATUM Geodetic		DATE April 2, 2009		CHECKED BY OK													
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC NATURAL LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ	GR SA SI CL
							20 40 60 80 100	20 40 60 80 100	W _p	W	W _L	10 20 30	kN/m ³				
205.3	GROUND SURFACE																
0.0	ASPHALT																
0.2	Sand, trace gravel, trace silt (FILL) Dense Brown Moist		1A	SS	35		205										
204.0			1B				204										
1.3	Sand and gravel, trace silt, containing wood fragments (FILL) Dense Brown Moist		2	SS	36												
203.0							203										
2.3	SAND and SILT, trace to some gravel, trace clay Loose Brown Moist		3	SS	4												9 39 51 1
202.4							202										
2.9	Granite Gneiss (BEDROCK) Bedrock cored from depths of 2.9 m to 7 m For bedrock coring details, refer to Record of Drillhole B4-09		1	RC	REC 56%												RQD = 22%
			2	RC	REC 40%		201										RQD = 34%
			3	RC	REC 63%		200										
							199										RQD = 57%
198.3	END OF BOREHOLE																
7.0	NOTE: 1. Water level in open borehole at a depth of 2.9 m below ground surface (Elev. 202.4 m) upon completion of drilling.																

PROJECT: 07-1111-0029

RECORD OF DRILLHOLE: B4-09

SHEET 1 OF 1

LOCATION: N 5045866.1 ;E 243505.4

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				FRACT INDEX PER 0.3 m	DISCONTINUITY DATA										HYDRAULIC CONDUCTIVITY				Diametral Point Load Index (MPa)	RMC -Q AVG	NOTES																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																			
				DEPTH (m)					TOTAL CORE %	SOLID CORE %	R.Q.D. %	B Angle		DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	K, cm ³ /sec	10 °	10 °	10 °	10 °																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																										
																								JN - Joint	BD - Bedding	PL - Planar	PO - Polished				MB - Mechanical Break																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																		
																																FLT - Fault	FO - Foliation	CU - Curved	K - Slickensided	BR - Broken Rock																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													
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																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													
3	NQ RC April 2, 2009	Continued from Record of Borehole B4-09		202.40																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													
		GRANITE GNEISS Slightly weathered to fresh, fine to medium grained with feldspar banding, foliated, black, pink and grey		2.90																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													</

DEPTH SCALE

1 : 50



LOGGED: ID

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 B4-10		SHEET 1 OF 1		METRIC											
W.P. 5187-06-01		LOCATION N 5045860.9 ; E 243510.8		ORIGINATED BY ID													
DIST HWY 69		BOREHOLE TYPE 165 mm O.D. Continuous Flight Solid Stem Augers and NW Casing, Wash Boring		COMPILED BY TZ													
DATUM Geodetic		DATE April 2, 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	GR SA SI CL
							20 40 60 80 100	○ UNCONFINED + FIELD VANE	● QUICK TRIAXIAL × REMOULDED	W _p	W	W _L	10 20 30				
205.5	GROUND SURFACE																
0.0	ASPHALT																
0.2	Sand, some gravel, trace silt (FILL) Brown Moist		1	AS	-		205										
204.6																	
0.9	Sand and gravel, trace silt, containing organics and cobbles (FILL) Compact to very dense Brown Moist		2	SS	98		204										
			3	SS	20												
203.2																	
2.3	Granite Gneiss (BEDROCK)						203										
	Bedrock cored from depths of 2.3 m to 6.2 m		1	RC	REC 98%		202										
	For bedrock coring details, refer to Record of Drillhole B4-10		2	RC	REC 98%		201										
			3	RC	REC 99%		200										
199.3																	
6.2	END OF BOREHOLE																
	NOTE: 1. Water level in open borehole at a depth of 2.3 m below ground surface (Elev. 203.2 m) upon completion of drilling.																

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 B4-11		SHEET 1 OF 1		METRIC								
W.P. 07-1111-0029		LOCATION		N 5045815.2 ; E 243536.2		ORIGINATED BY		CR								
DIST		HWY 69		BOREHOLE TYPE		Portable Equipment, BW Casing, Wash Boring		COMPILED BY								
VA		DATE		Geodetic		July 22 & 23, 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								
207.6	GROUND SURFACE						20	40	60	80	100					
0.0	Granite Gneiss (BEDROCK)		1	RC	REC 100%											RQD = 100%
	Bedrock cored from ground surface to a depth of 10.2 m		2	RC	REC 100%											RQD = 100%
	For bedrock coring details, refer to Record of Drillhole B4-11		3	RC	REC 100%											RQD = 98%
			4	RC	REC 100%											RQD = 100%
			5	RC	REC 100%											RQD = 83%
			6	RC	REC 100%											RQD = 100%
			7	RC	REC 100%											RQD = 65%
			8	RC	REC 100%											RQD = 73%
			9	RC	REC 100%											RQD = 80%
			10	RC	REC 100%											RQD = 90%
197.4	END OF BOREHOLE															
10.2	NOTE: 1. Water level in open borehole at a depth of 5.5 m below ground surface (Elev. 202.1 m) 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

SHEET 1 OF 2

DATUM: Geodetic

DRILLING CONTRACTOR: OGS

[illegible]

CHECKED: OK

STA-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: B4-11

SHEET 2 OF 2

LOCATION: N 5045815.2 ;E 243536.2

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 °	10 °			10 °																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																
10		-- CONTINUED FROM PREVIOUS PAGE -- Becoming dark grey at a depth of 9.9 m		197.36 10.21	10																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	

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




+ 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		RECORD OF BOREHOLE				No B4-13		SHEET 1 OF 1		METRIC							
W.P. 07-1111-0029		LOCATION				N 5045818.8 ; E 243539.1		ORIGINATED BY CR									
DIST		HWY 69		BOREHOLE TYPE		Hand Excavation		COMPILED BY VA									
DATUM Geodetic		DATE		July 20, 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									
205.0	GROUND SURFACE																
0.0	TOPSOIL	///	1	CS	-												
0.1	END OF EXCAVATION BEDROCK																
	NOTE: 1. Hand digging carried out at proposed borehole location to expose bedrock (Dry).																

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 <u>07-1111-0029</u>		RECORD OF BOREHOLE No B4-14		SHEET 1 OF 1		METRIC												
W.P. <u>5187-06-01</u>		LOCATION <u>N 5045819.1 ;E 243544.2</u>		ORIGINATED BY <u>CR</u>														
DIST <u> </u> HWY <u>69</u>		BOREHOLE TYPE <u>Hand Excavation</u>		COMPILED BY <u>VA</u>														
DATUM <u>Geodetic</u>		DATE <u>July 21, 2009</u>		CHECKED BY <u>OK</u>														
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ	GR SA SI CL	
202.3	GROUND SURFACE							20 40 60 80 100	○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× REMOULDED	20 40 60 80 100	10 20 30				
0.0	TOPSOIL		1	CS	-													
0.2	END OF EXCAVATION BEDROCK																	
	NOTE: 1. Hand digging carried out at proposed borehole location to expose bedrock (Dry).																	

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 B4-15		SHEET 1 OF 1		METRIC											
W.P. 5187-06-01		LOCATION N 5045823.7 ; E 243541.0		ORIGINATED BY CR													
DIST HWY 69		BOREHOLE TYPE Portable Equipment, BW Casing, Wash Boring		COMPILED BY VA													
DATUM Geodetic		DATE July 21 & 22, 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	GR SA SI CL
								20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	10 20 30	10 20 30	10 20 30				
202.0	GROUND SURFACE																
0.0	Cobbles/Rock Fragments		1	RC	REC 66%												
201.5	Granite Gneiss (BEDROCK)		2	RC	REC 100%		201										RQD = 45%
0.5	Bedrock cored from depths of 0.5 m to 6.5 m For bedrock coring details, refer to Record of Drillhole B4-15		3	RC	REC 100%												RQD = 91%
			4	RC	REC 100%		200										RQD = 100%
			5	RC	REC 100%												RQD = 97%
			6	RC	REC 100%		199										RQD = 97%
			7	RC	REC 100%		198										RQD = 97%
			8	RC	REC 100%		197										RQD = 97%
			9	RC	REC 100%		196										RQD = 100%
195.5	END OF BOREHOLE																
6.5	NOTE: 1. Water level measurements in piezometer: Date Depth (m) Elev. (m) 28/07/09 -0.6* 202.6 26/08/09 0.3 201.7 *Water level above ground surface due to heavy rain.																

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: OGS

CHECKED: OK

PROJECT		RECORD OF BOREHOLE		No B4-16		SHEET 1 OF 1		METRIC									
W.P.		LOCATION		ORIGINATED BY		DIST		BOREHOLE TYPE		COMPILED BY		DATE		CHECKED BY			
07-1111-0029		N 5045802.5 ; E 243546.3		CR		HWY 69		Portable Equipment		VA		Geodetic		July 20, 2009		OK	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100	W _p	W	W _L		
208.3	GROUND SURFACE																
0.0	TOPSOIL		1A														
0.1	Silty SAND, some gravel, rock fragments, containing rootlets Loose to compact Brown Moist		1B	SS	6												
207.2			2	SS	23												
1.1	END OF BOREHOLE SPOON REFUSAL																
NOTES: 1. Borehole advanced using portable drilling equipment with one-third weight hammer. SPT 'N' values shown have been adjusted to infer values that would be obtained with standard weight hammer. 2. Open borehole dry upon completion of 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

PROJECT		RECORD OF BOREHOLE No B4-17				SHEET 1 OF 1		METRIC								
W.P. 07-1111-0029		LOCATION N 5045805.3 ; E 243543.4				ORIGINATED BY CR										
DIST _____ HWY 69		BOREHOLE TYPE Portable Equipment				COMPILED BY VA										
DATUM Geodetic		DATE July 20, 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								
207.5	GROUND SURFACE															
0.0	TOPSOIL		1A	SS	1											
0.3	Silty SAND, trace to some clay, trace gravel, containing rootlets Very loose Brown Moist 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 was 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		RECORD OF BOREHOLE		No B4-18		SHEET 1 OF 1		METRIC								
W.P. 07-1111-0029		LOCATION		N 5045807.7 ; E 243550.6		ORIGINATED BY		CR								
DIST		HWY 69		BOREHOLE TYPE		Portable Equipment, BW Casing, Wash Boring		COMPILED BY								
VA		DATE		July 20 & 21, 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								
207.2	GROUND SURFACE						20	40	60	80	100					
0.0	TOPSOIL		1	SS	4/0.13											
0.1	Granite Gneiss (BEDROCK)		1	RC	REC 100%											RQD = 83%
	Bedrock cored from depths of 0.1 m to 6.4 m		2	RC	REC 100%											RQD = 92%
	For bedrock coring details refer to Record of Drillhole B4-18		3	RC	REC 100%											RQD = 100%
			4	RC	REC 100%											RQD = 100%
			5	RC	REC 100%											RQD = 97%
			6	RC	REC 100%											RQD = 100%
			7	RC	REC 100%											RQD = 100%
			8	RC	REC 100%											RQD = 95%
200.8	END OF BOREHOLE															
6.4	NOTES:															
	1. Borehole advanced using portable drilling equipment with one-third weight hammer. SPT 'N' value shown has been adjusted to infer value that was obtained with standard weight hammer.															
	2. Water level measurements in piezometer:															
	Date Depth (m) Elev. (m)															
	28/07/09 3.9 203.3															
	26/08/09 4.7 202.5															

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: OGS

CHECKED: OK

PROJECT		RECORD OF BOREHOLE				No B4-19		SHEET 1 OF 1		METRIC							
W.P. 5187-06-01		LOCATION				N 5045809.0 ; E 243559.7		ORIGINATED BY CR									
DIST		HWY 69		BOREHOLE TYPE		Hand Excavation		COMPILED BY VA									
DATUM Geodetic		DATE		July 21, 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									
208.2	GROUND SURFACE																
8.9	TOPSOIL		1	CS													
	END OF EXCAVATION BEDROCK																
	NOTE: 1. Hand digging carried out at proposed borehole location to expose bedrock (DRY).																

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 <u>07-1111-0029</u>		RECORD OF BOREHOLE No B4-20				SHEET 1 OF 1		METRIC								
W.P. <u>5187-06-01</u>		LOCATION <u>N 5045811.8 ; E 243556.8</u>				ORIGINATED BY <u>CR</u>										
DIST <u> </u> HWY <u>69</u>		BOREHOLE TYPE <u>Hand Excavation</u>				COMPILED BY <u>VA</u>										
DATUM <u>Geodetic</u>		DATE <u>July 21, 2009</u>				CHECKED BY <u>OK</u>										
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa								
206.5	GROUND SURFACE															
0.0	TOPSOIL	///	1	CS	-											
0.1	END OF EXCAVATION BEDROCK															
NOTE: 1. Hand digging carried out at proposed borehole location to expose bedrock (DRY).																

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 B4-21				SHEET 1 OF 1		METRIC								
W.P. 5187-06-01		LOCATION N 5045793.8 ;E 243565.0				ORIGINATED BY CR										
DIST HWY 69		BOREHOLE TYPE Hand Excavation				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								
212.9	GROUND SURFACE															
8.9	TOPSOIL															
	END OF EXCAVATION BEDROCK															
	NOTE: 1. Hand digging carried out at proposed borehole location to expose bedrock (DRY).															

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



APPENDIX B

LABORATORY TEST RESULTS

TABLE B1
POINT LOAD TEST RESULTS ON ROCK SAMPLES

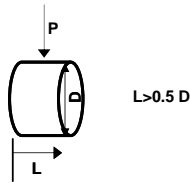
Borehole Number	Run Number	Sample Depth (m)	Sample Elevation (m)	Bedrock Description	Test Type	Is (50mm) (MPa)	Approx. ⁽¹⁾ UCS (MPa)
B4-01	3	5.31	198.2	Granite Gneiss	Diametral	6.252	69.0
B4-01	4	5.47	198.0	Granite Gneiss	Diametral	7.504	83.0
B4-01	4	5.54	195.7	Granite Gneiss	Diametral	8.573	94.0
B4-02	1	1.0	200.7	Granite Gneiss	Diametral	6.185	68.0
B4-02	1	1.0	200.7	Granite Gneiss	Diametral	6.394	70.0
B4-02	1	1.2	200.5	Granite Gneiss	Diametral	5.175	57.0
B4-02	2	2.0	199.7	Granite Gneiss	Diametral	5.501	61.0
B4-02	2	2.1	199.6	Granite Gneiss	Diametral	6.052	67.0
B4-02	2	2.2	199.5	Granite Gneiss	Diametral	4.591	51.0
B4-06	2	2.6	198.9	Granite Gneiss	Diametral	6.995	77.0
B4-06	2	2.8	198.7	Granite Gneiss	Diametral	8.072	89.0
B4-06	2	2.9	198.6	Granite Gneiss	Diametral	9.291	102.0
B4-08	2	5.0	196.8	Granite Gneiss	Diametral	8.640	95.0
B4-08	2	5.1	196.7	Granite Gneiss	Diametral	11.102	122.0
B4-08	2	5.2	196.6	Granite Gneiss	Diametral	6.862	75.0
B4-09	1	3.1	202.2	Granite Gneiss	Axial	10.322	114.0
B4-09	1	3.2	202.1	Granite Gneiss	Diametral	5.543	61.0
B4-09	1	3.3	202.0	Granite Gneiss	Axial	10.509	116.0
B4-09	1	3.3	202.0	Granite Gneiss	Axial	10.828	119.0
B4-09	1	3.3	202.0	Granite Gneiss	Diametral	5.735	63.0
B4-09	1	3.4	201.9	Granite Gneiss	Diametral	6.386	70.0
B4-09	2	4.8	200.5	Granite Gneiss	Diametral	7.830	86.0
B4-09	2	4.9	200.4	Granite Gneiss	Diametral	8.272	91.0
B4-09	2	5.0	200.3	Granite Gneiss	Diametral	6.928	76.0
B4-11	6	4.6	203.0	Granite Gneiss	Diametral	6.331	70.0
B4-11	7	6.3	201.3	Granite Gneiss	Diametral	4.853	53.0
B4-11	9	8.6	199.0	Granite Gneiss	Diametral	2.940	32.0
B4-11	10	9.3	198.3	Granite Gneiss	Diametral	4.878	54.0
B4-11	10	9.8	197.8	Granite Gneiss	Diametral	5.797	64.0
B4-15	5	3.1	198.9	Granite Gneiss	Diametral	6.717	74.0
B4-15	7	4.1	197.9	Granite Gneiss	Diametral	8.827	97.0
B4-15	8	5.1	196.9	Granite Gneiss	Diametral	8.573	94.0
B4-15	9	6.3	195.8	Granite Gneiss	Diametral	4.073	45.0
B4-18	6	3.9	203.3	Granite Gneiss	Diametral	3.375	37.0
B4-18	6	4.6	202.7	Granite Gneiss	Diametral	2.110	23.0
B4-18	7	5.1	202.1	Granite Gneiss	Diametral	1.059	12.0
B4-18	9	5.9	201.3	Granite Gneiss	Diametral	4.927	54.0

TABLE B1
POINT LOAD TEST RESULTS ON ROCK SAMPLES

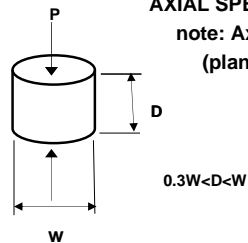
Borehole Number	Run Number	Sample Depth (m)	Sample Elevation (m)	Bedrock Description	Test Type	Is (50mm) (MPa)	Approx. ⁽¹⁾ UCS (MPa)
⁽¹⁾ $Is_{50} \times K$ (actual value could be confirmed by UCS testing), from ISRM. This range has been given based on $K = 11$, calculated from Is_{50} Average (9 tests) equal to 7.2 MPa on Diametral orientation on samples from BH B4-06 and B4-09, and UCS Average (2 tests) equal to 81.5 MPa on a sample from end of BH B4-06 and B4-09. "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: OK
Checked By: CN
Reviewed By: JPD/JMAC

TABLE B2-1
SUMMARY OF UNCONFINED COMPRESSION TEST RESULTS
SHAWANAGA RIVER NBL BRIDGE STRUCTURE
HIGHWAY 69, TOWNSHIP OF PARRY SOUND
GWP 5111-07-00, W.P 5187-06-01

Borehole Number (Core Run)	Sample Depth (m)	Sample Elevation (m)	Rock Type	Core Diameter (mm)	Unconfined Compressive Strength (MPa)
B4-06 (2)	3.3	198.2	Granite Gneiss	47.4	78.1
B4-09 (2)	4.7	200.6	Granite Gneiss	47.5	85.5

Compiled By: OKChecked By: CNReviewed By: JPD/JMAC

TABLE B2-2
UNCONFINED COMPRESSION (UC) TEST
ASTM D 7012-07

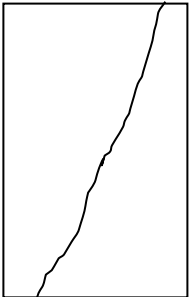
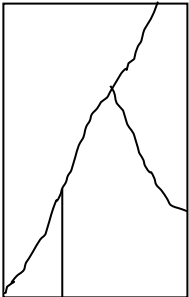
SAMPLE IDENTIFICATION			
PROJECT NUMBER	07-1111-0029	RUN NUMBER	2
BOREHOLE NUMBER	B4-06	SAMPLE DEPTH, m	3.1-3.4
TEST CONDITIONS			
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.11
SPECIMEN INFORMATION			
SAMPLE HEIGHT, cm	10.00	WATER CONTENT, (specimen) %	0.13
SAMPLE DIAMETER, cm	4.74	UNIT WEIGHT, kN/m ³	26.10
SAMPLE AREA, cm ²	17.65	DRY UNIT WT., kN/m ³	26.07
SAMPLE VOLUME, cm ³	176.46	SPECIFIC GRAVITY, assumed	2.70
WET WEIGHT, g	469.90	VOID RATIO	0.02
DRY WEIGHT, g	469.29		
VISUAL INSPECTION	FAILURE SKETCH		
			
TEST RESULTS			
STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	78.1
REMARKS:	N/A	DATE:	6/16/2009
CHECKED BY:	CN	REVIEWED BY:	JPD/JMAC

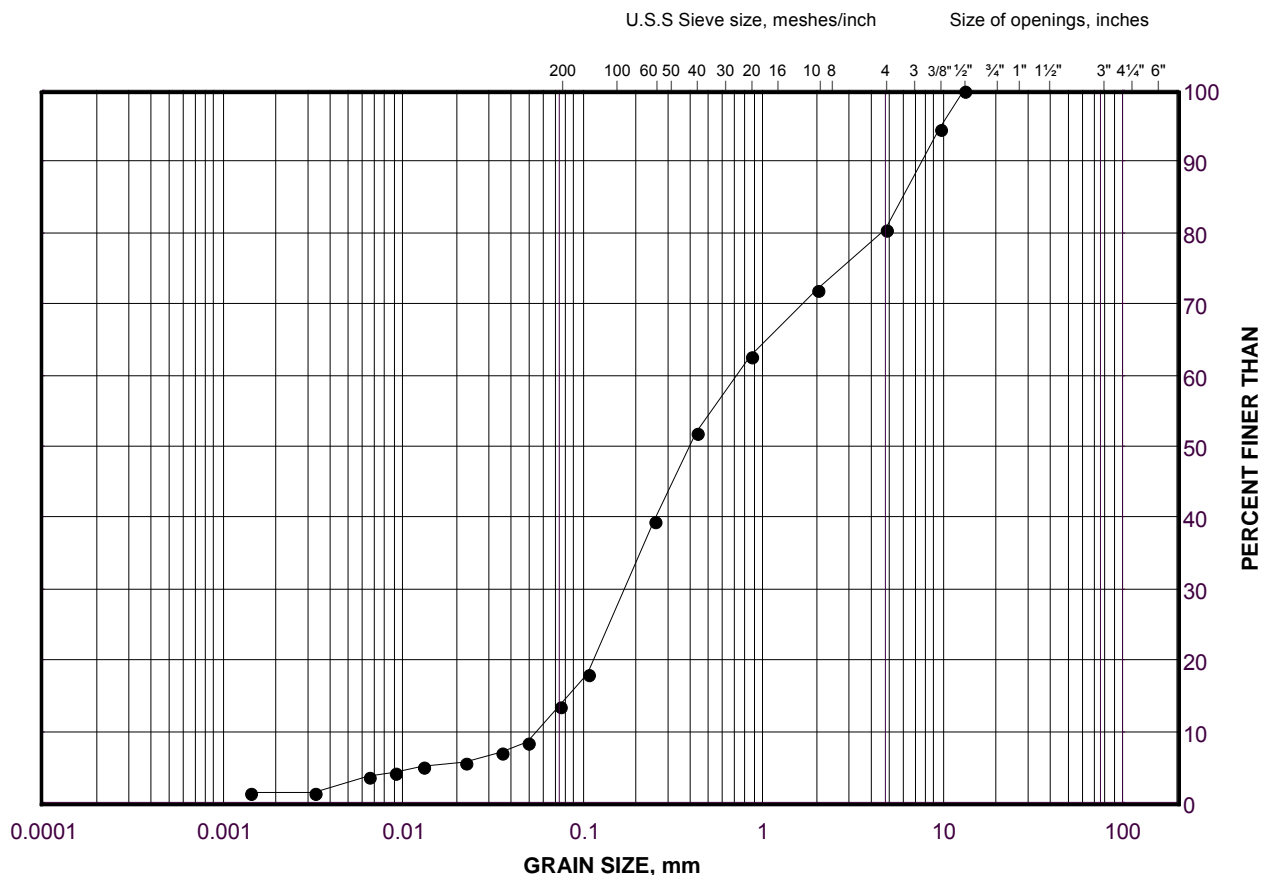
TABLE B2-3
UNCONFINED COMPRESSION (UC) TEST
ASTM D 7012-04

SAMPLE IDENTIFICATION			
PROJECT NUMBER	07-1111-0029	RUN NUMBER	2
BOREHOLE NUMBER	B4-09	SAMPLE DEPTH, m	4.5-4.8
TEST CONDITIONS			
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.16
SPECIMEN INFORMATION			
SAMPLE HEIGHT, cm	10.25	WATER CONTENT, (specimen) %	0.12
SAMPLE DIAMETER, cm	4.75	UNIT WEIGHT, kN/m ³	26.91
SAMPLE AREA, cm ²	17.68	DRY UNIT WT., kN/m ³	26.88
SAMPLE VOLUME, cm ³	181.25	SPECIFIC GRAVITY, assumed	2.70
WET WEIGHT, g	497.61	VOID RATIO	-0.02
DRY WEIGHT, g	497.01		
VISUAL INSPECTION	FAILURE SKETCH		
			
TEST RESULTS			
STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	85.5
REMARKS:	N/A	DATE:	6/16/2009
CHECKED BY:	CN	REVIEWED BY:	JPD/JMAC

GRAIN SIZE DISTRIBUTION

Sand (Fill)
North Pier (Pier 4)

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)
•	B4-06	2A	199.9

Project Number: 07-1111-0029

Checked By: CN

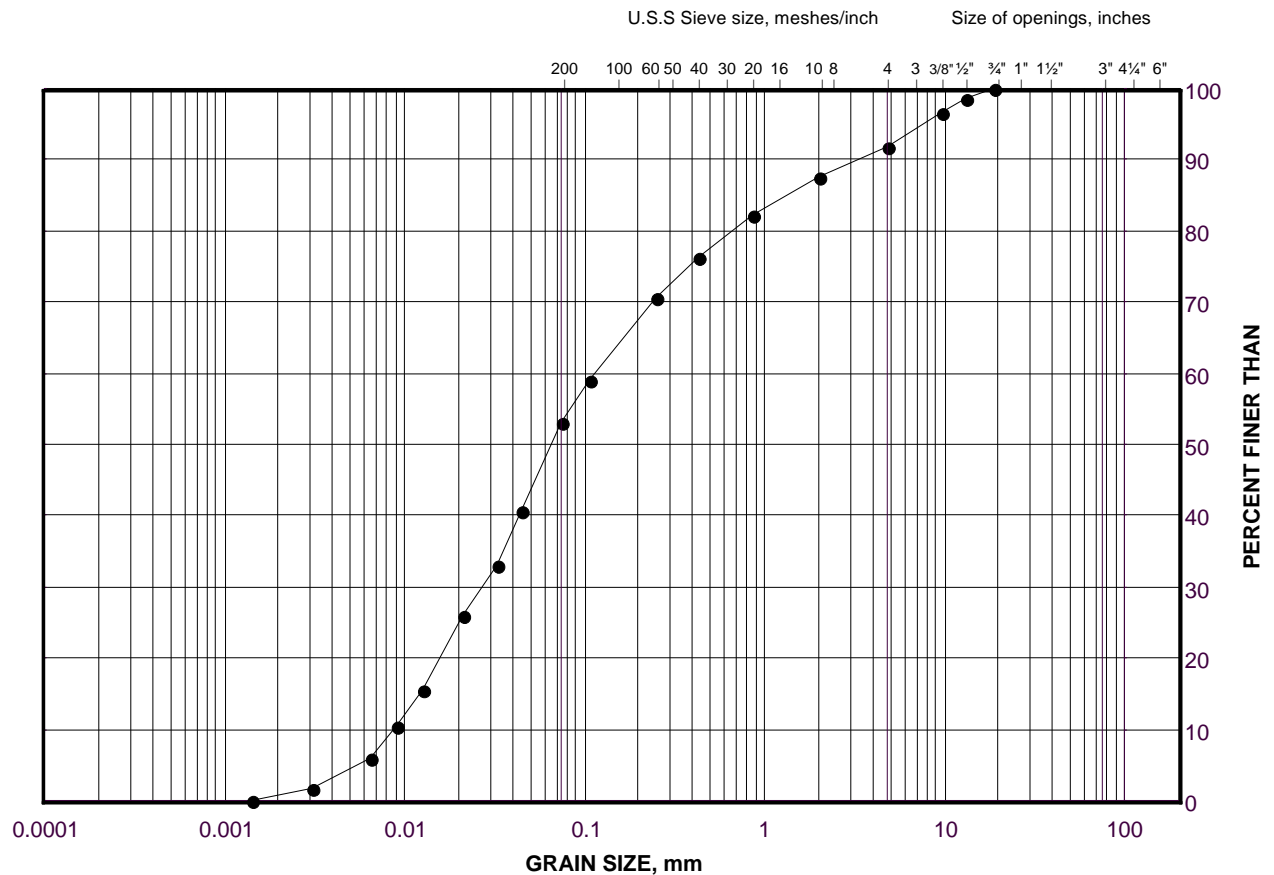
Golder Associates

Date: 12-Aug-10

GRAIN SIZE DISTRIBUTION

Sand and Silt
North Abutment

FIGURE B1-2



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

LEGEND

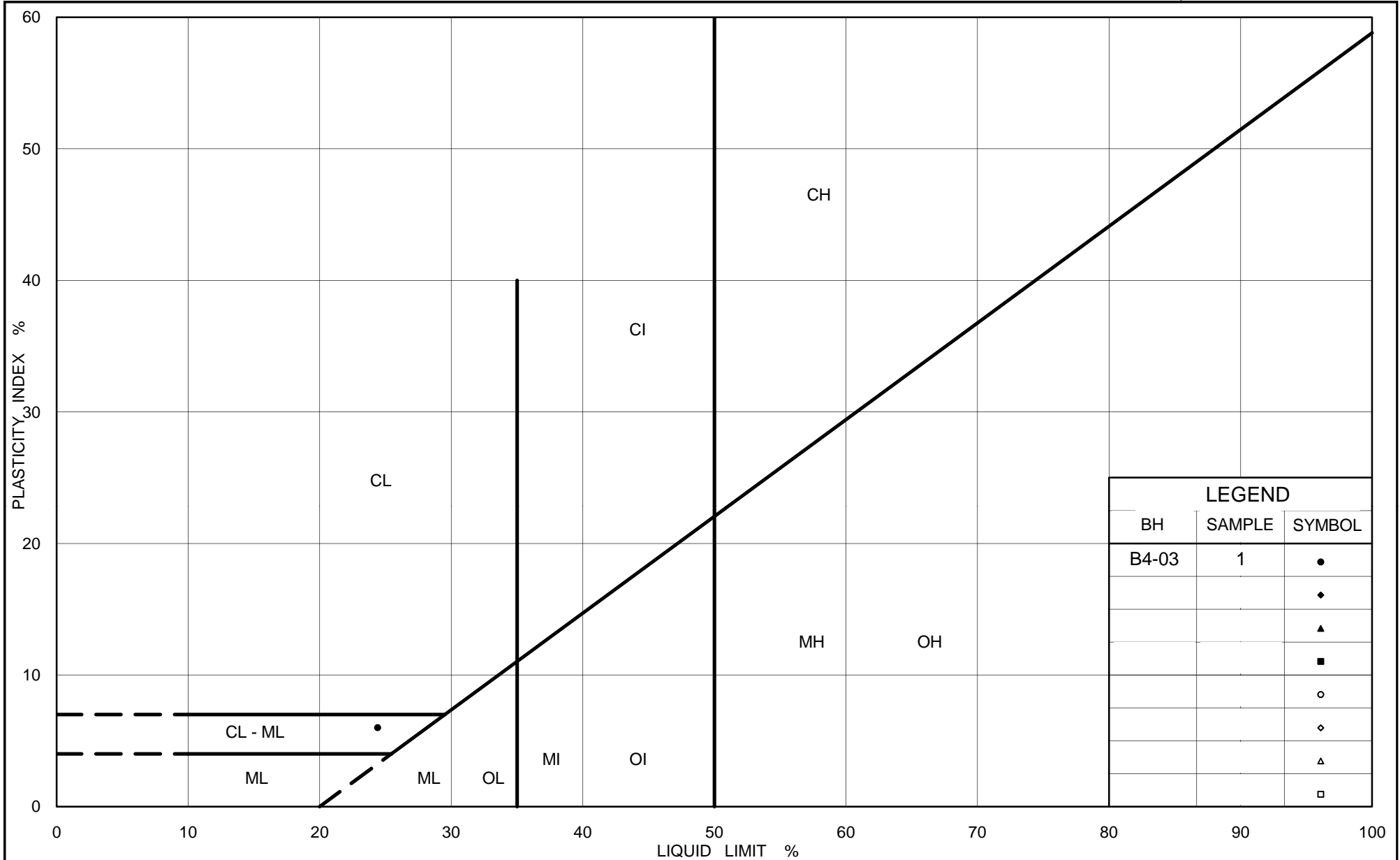
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	B4-09	3	202.7

Project Number: 07-1111-0029

Checked By: CN

Golder Associates

Date: 12-Aug-10



Ministry of Transportation

Ontario

PLASTICITY CHART Clayey Silt North Abutment

Figure No. B1-3

Project No. 07-1111-0029

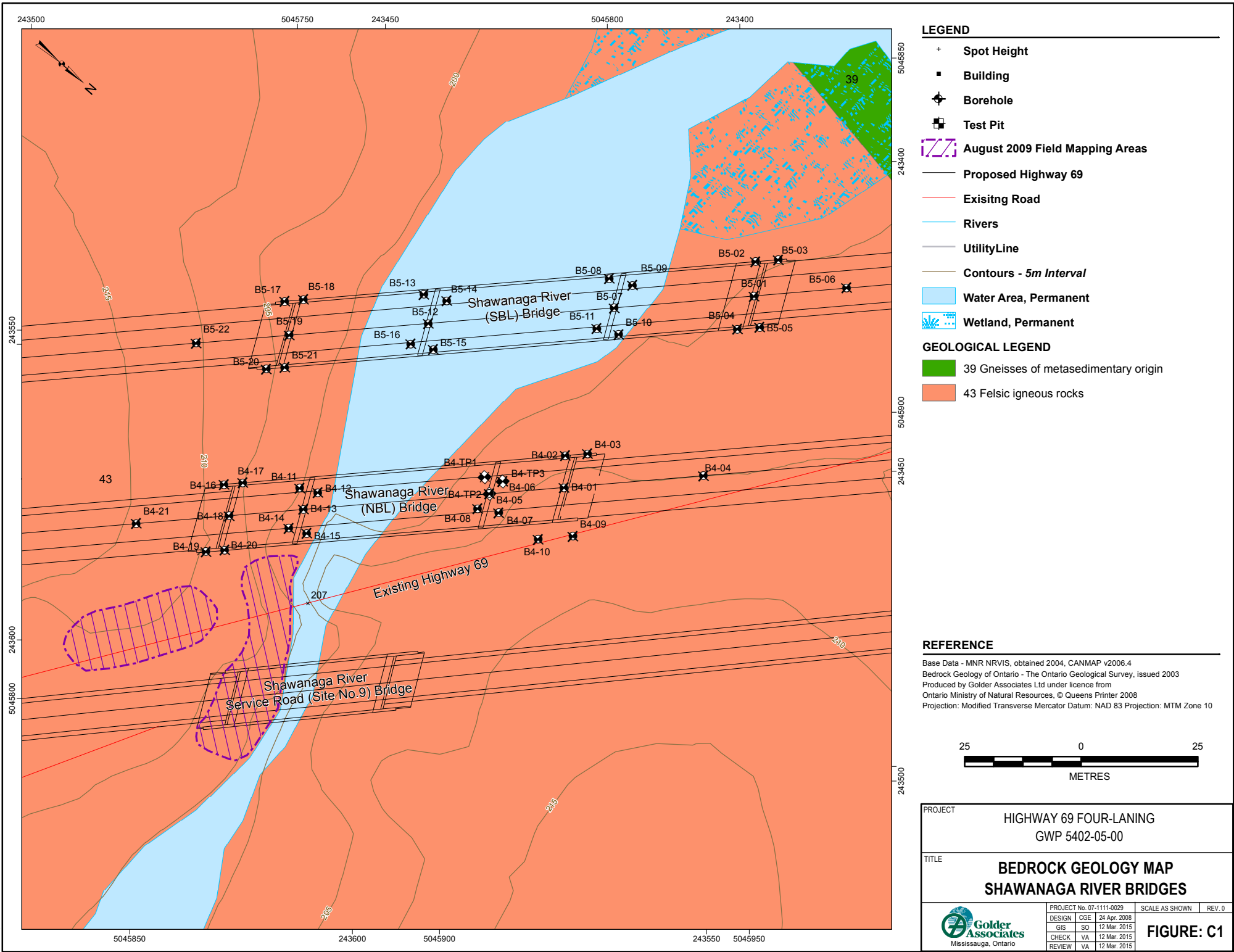
Checked By: CN

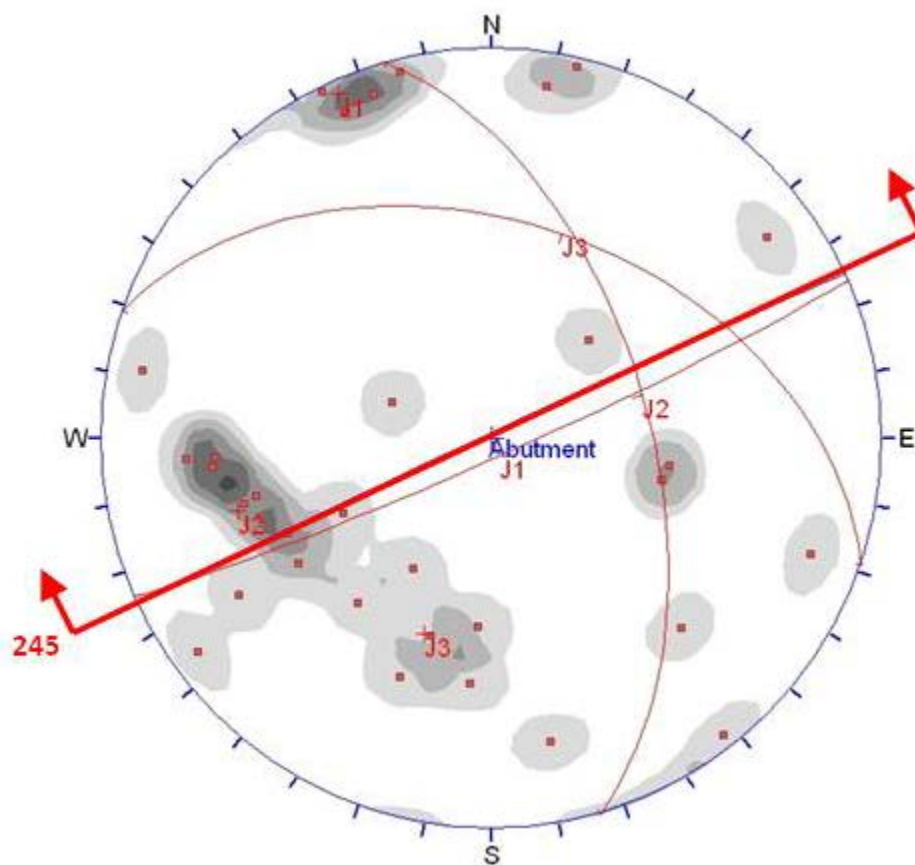


APPENDIX C

ROCK MAPPING

G:\Projects\2007\07-1111-0029_MRC_ParrySoundGIS\MXDs\Draft\Bedrock-Geology_8.5x11_Rotated.mxd





Fisher
Concentrations
% of total per 1.0 % area



0.00 ~ 2.00 %
2.00 ~ 4.00 %
4.00 ~ 6.00 %
6.00 ~ 8.00 %
8.00 ~ 10.00 %
10.00 ~ 12.00 %
12.00 ~ 14.00 %

No Bias Correction
Max. Conc. = 10.7163%

Equal Area
Lower Hemisphere
32 Poles
32 Entries

Orientations

ID		Dip / Direction
J1	m	86 / 156
J2	m	57 / 074
J3	m	44 / 019

PROJECT Highway 69 Shawanaga River Bridge (NBL)
Structure

TITLE
**Shawanaga River Bridge (NBL) South Abutment
Bedrock Outcrop Data**



PROJECT No. 07-1111-0029			FILE No. ----	
DESIGN	DAC	APR 2011	SCALE	AS SHOWN
CADD	DAC	APR 2011	REV.	
CHECK	JPD	APR 2011		
REVIEW	JMAC	APR 2011		


Figure C2

Old Highway 69 South Abutment (pre-1957)



Existing Highway 69 South Abutment




PROJECT		Highway 69 Shawanaga River Bridge (NBL) Structure			
TITLE		Rock Mapping Photos			
		PROJECT No. 07-1111-0029		FILE No. ----	
		DESIGN	DAC	APR 2011	SCALE AS SHOWN
		CADD	DAC	APR 2011	REV.
		CHECK	JPD	APR 2011	
		REVIEW	JMAC	APR 2011	
					Figure C3

Proposed Shawanaga River Bridge (NBL) South Abutment




East Side of Proposed Highway 69 NBL between about STA 17+ 620 and 17+650



PROJECT		Highway 69 Shawanaga River Bridge (NBL) Structure			
TITLE		Rock Mapping Photos			
		PROJECT No. 07-1111-0029		FILE No. ----	
		DESIGN	DAC	APR 2011	SCALE AS SHOWN
		CADD	DAC	APR 2011	REV.
		CHECK	JPD	APR 2011	
		REVIEW	JMAC	APR 2011	
					Figure C4

East Side of Proposed Shawanaga River Bridge (NBL) South Abutment



PROJECT		Highway 69 Shawanaga River Bridge (NBL) Structure			
TITLE		Rock Mapping Photo			
	PROJECT No. 07-1111-0029			FILE No. ----	
	DESIGN	DAC	APR 2011	SCALE AS SHOWN	REV.
	CADD	DAC	APR 2011	Figure C5	
	CHECK	JPD	APR 2011		
	REVIEW	JMAC	APR 2011		



APPENDIX D

NON-STANDARD SPECIAL PROVISIONS

DOWELS INTO ROCK - Item No.

Special Provision

1.0 SCOPE

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

2.0 MATERIALS AND INSTALLATION

Dowels into rock shall be constructed in accordance with OPSS 904. All reinforcing steel supplied shall be in accordance with OPSS 1440 (dowel bars conforming to CSA Standard G30.18, Grade 400).

Where dowels are to be placed in rock, holes shall be drilled to the required depth and size. Each hole shall be cleaned out, grouted and the dowel set in place. Grout shall be of the same strength as the footing concrete (but not less than 30 MPa at 28 days).

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

3.0 ROCK DOWEL TESTING

All proposed testing procedures shall be in general conformance with ASTM D3689-07, 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 (NBL)	South Abutment	2
Highway 69 / Shawanaga River Bridge (NBL)	South Pier	2
Highway 69 / Shawanaga River Bridge (NBL)	North Pier	2
Highway 69 / Shawanaga River Bridge (NBL)	North Abutment	2

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

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

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

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

Displacement measurements shall be carried out at each load increment using calibrated displacement gauges capable of measuring movements of 0.0025 cm. Measurements shall be referenced to an independent fixed referenced 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 lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

OBSTRUCTIONS - Item No.

Non-Standard Special Provision

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

Basis of Payment

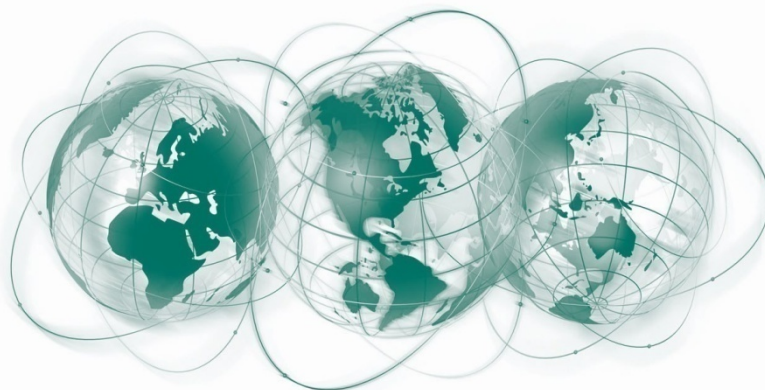
Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION

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

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

solutions@golder.com
www.golder.com



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

