July 20, 2015

FOUNDATION INVESTIGATION AND DESIGN REPORT

SHEBESHEKONG ROAD UNDERPASS STRUCTURE
SITE NO. 44-442
HIGHWAY 69 FOUR-LANING FROM 1.0 KM NORTH OF THE NEW HIGHWAY 559 INTERCHANGE NORTHERLY TO 1.5 KM NORTH OF HIGHWAY 7182 FOR 17 KM
MINISTRY OF TRANSPORTATION, ONTARIO
G.W.P. 5111-07-00, W.P. 5183-06-01 (Phase 2 of G.W.P. 5402-05-00)

Submitted to:
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REPORT

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PART A

FOUNDATION INVESTIGATION REPORT

SHEBESHEKONG ROAD UNDERPASS STRUCTURE, SITE NO. 44-442

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. 5183-06-01 (Phase 2 of G.W.P. 5402-05-00)

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by McCormick Rankin, a member of MMM Group Limited on behalf of Ministry of Transportation, Ontario (MTO) to carry out detail foundation investigation services for the proposed Shebeshekong Road Interchange two-span underpass structure (Site No. 44-442) over the proposed new Highway 69 four-laning alignment. The proposed work is part of the detail design for the four-laning of Highway 69 from 1.0 km north of the new Highway 559 Interchange northerly to 1.5 km north of Highway 7182 (Shebeshekong Road), which involves high fill embankments and embankments over swamps, the New Woods Road and Shebeshekong Road interchanges and structures, the Shawanaga River and Site 9 Road bridge structures, as well as culvert crossings. The general location of this interchange structure along the new Highway 69 four-laning alignment is shown on Drawing 1.

The terms of reference and the scope of work for the foundation investigation are outlined in MTO's Request for Proposal, dated July 2006. Golder's proposal for foundation engineering services associated with the Shebeshekong Road underpass structure is contained in Section 6.8 of MRC's Technical Proposal for this assignment. The work has been carried out in accordance with Golder's Supplementary Specialty Plan for foundation engineering services for this project, dated July 4, 2007. The General Arrangement (GA) drawing for the proposed underpass structure of Highway 69 and Shebeshekong Road was provided to Golder by MRC on September 19, 2008.

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

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

2.0 SITE DESCRIPTION

The proposed Shebeshekong Road underpass structure is located approximately 2 km south of the intersection of the existing Shebeshekong Road and Highway 69 and is approximately 18 km northwest of Nobel, Ontario. The existing Highway 69, which will be the future Site No. 9 Service Road in this area, runs generally in a southeast-northwest direction along the northeast side of the proposed new Highway 69 alignment.

In general, the topography in the area of the overall project limits consists of rolling terrain including densely treed areas and numerous bedrock outcrops separated by low-lying swamps. The proposed underpass structure and associated approach embankments are to be situated on a relatively flat, densely treed area. The ground surface within the limits of the proposed structure and approach embankments is between about Elevation 216.6 m and Elevation 214.2 m, referenced to geodetic datum, and is gently sloping downward from west to east.

3.0 INVESTIGATION PROCEDURES

3.1 Foundation Investigation

The field work for the Shebeshekong Road underpass structure investigation was carried out between October 27 and November 4, 2008 during which time a total of seventeen (17) boreholes were advanced: five (5) boreholes at the east abutment; five (5) boreholes at the centre pier; five (5) boreholes at the west abutment; and two (2) boreholes at the approach embankments (i.e. one borehole at each approach). The boreholes, designated as Boreholes B2-01 to B2-17, were advanced at approximately the locations shown in plan on Drawing 2.

The field investigation was carried out using a Diedrich D-50 Turbo track-mounted drill rig supplied and operated by Walker Drilling Ltd. of Utopia, Ontario. The boreholes were advanced through the overburden using 115 mm outside diameter solid stem augers and/or 'NW' casing and wash boring methods. Soil samples were obtained at intervals of depth of about 0.75 m and 1.5 m, using a 50 mm outer diameter (O.D.) split-spoon sampler driven by an automatic hammer in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586 Standard Test Method for Standard Penetration Test). Samples of the bedrock were obtained using an 'NQ' size rock core barrel.

The boreholes at the foundation elements were typically advanced to auger and/or sampler refusal (i.e. inferred bedrock) and bedrock was confirmed by coring in selected boreholes, while the boreholes at the approach embankments were advanced to sampler refusal (a depth less than the height of the proposed embankments at this site). The depths of the boreholes range from about 0.2 m to 9.7 m below existing ground surface, including coring of between about 1.5 m and 9.2 m into the bedrock, at Boreholes B2-01, B2-03, B2-05 to B2-11, B2-13 and B2-15 to B2-17.

The groundwater conditions in the open boreholes were observed during the drilling operations and piezometers were installed in Boreholes B2-01, B2-09, B2-10 and B2-15 to permit monitoring of the water level at these locations. The piezometers consist of 32 mm diameter PVC pipe, with a slotted screen sealed at a select depth within the borehole. The boreholes and annulus surrounding the piezometer pipe above the sand pack/screen were backfilled to the surface with bentonite pellets/grout. Piezometer installation details and water level 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 to the ground surface upon completion in accordance with Ontario Regulation 903 Wells (as amended).

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

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

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

Borehole	Location (MTM NAD 83)		Ground Surface Elevation (m)	Depth Drilled (m)
	Northing	Easting		
B2-01	5043992.0	245158.5	215.7	5.3
B2-02	5043985.7	245166.2	216.4	0.9
B2-03	5043990.8	245164.0	216.5	6.1
B2-04	5043996.0	245161.8	216.3	1.0
B2-05	5043989.5	245169.4	216.6	4.9
B2-06	5044022.1	245183.3	215.7	9.7
B2-07	5044015.7	245191.1	215.8	3.0
B2-08	5044020.9	245188.8	215.9	3.4
B2-09	5044026.1	245186.6	216.6	3.2
B2-10	5044019.6	245194.2	215.4	9.3
B2-11	5044052.2	245208.2	214.6	6.3
B2-12	5044045.8	245215.9	214.2	3.0
B2-13	5044050.9	245213.6	214.9	6.0
B2-14	5044056.2	245211.4	215.2	0.2
B2-15	5044049.7	245219.1	214.6	2.4
B2-16	5043979.2	245154.4	215.3	3.7
B2-17	5044062.5	245223.2	214.9	1.7

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

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

Parry Sound and Muskoka areas, then eastward from Muskoka in patches into the area north of the Kawartha Lakes.

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

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 and later erosion during glaciation left behind these Precambrian rocks.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions as encountered in the boreholes advanced for this investigation, together with the results of the laboratory tests carried out on selected soil and bedrock core samples, are provided in Appendix A and B, respectively. The results of the in situ field tests (i.e. SPT ‘N’-values) as presented on the Record of Borehole sheets and in Sections 4.2.1 to 4.2.6 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. It should be noted that the interpreted stratigraphy shown on Drawings 2 and 3 is a simplification of the subsurface conditions. Variation in the stratigraphic boundaries between and beyond the boreholes will exist and is to be expected.

In general, the subsurface conditions in the area of the proposed underpass consist of a surficial layer of topsoil up to about 0.3 m thick, underlain by a deposit of silty sand to sand and gravel between about 0.7 m and 2.8 m thick (where present). The topsoil or sand deposit is underlain by syenite/granite/biotite gneiss bedrock.

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

4.2.1 Topsoil

Up to about 0.3 m of topsoil was encountered immediately below ground surface in all of the boreholes advanced at this site.

4.2.2 Silty Clay

A localized stratum of brown silty clay, trace gravel, containing fine sand seams and rootlets was encountered below the topsoil in Borehole B2-12. The top of this deposit is at about Elevation 214.0 m and its thickness is about 0.9 m.

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

The Standard Penetration Test (SPT) 'N'-values measured within the silty clay deposit are 4 blows and 26 blows per 0.3 m of penetration, indicating a firm to very stiff consistency.

The natural water content measured on one (1) sample of the silty clay is about 26 percent. The organic content measured on one (1) sample of the upper portion of this deposit is about 7 percent.

An Atterberg limit test was carried out on one (1) sample of the silty clay deposit. The liquid limit is about 36 percent and the plastic limit is about 20 percent, corresponding to plasticity index of about 16 percent. The results of the Atterberg limits test are shown on the plasticity charge on Figure B1 in Appendix B and indicate that this material is classified as silty clay of intermediate plasticity.

4.2.3 Sandy Silt to Silty Sand

A deposit of brown to mottled grey-brown sandy silt to silty sand, trace clay and containing rootlets was encountered below the topsoil in Boreholes B2-11, B2-13 and B2-15 and below the silty clay in Borehole B2-12. The top of this deposit varies from about Elevation 214.7 m to 213.1 m and the thickness of the deposit ranges from about 1.0 m to 2.2 m.

The SPT 'N'-values measured within the sand to silt deposit range from 4 blows to 38 blows per 0.3 m of penetration, indicating a loose to dense relative density.

The natural water content measured on samples of this deposit range from about 17 percent to 24 percent and the organic content measured on one (1) sample of the upper portion this deposit is about 4 percent.

The grain size distributions of two (2) samples from the sandy silt to silty sand deposit are shown on Figure B2 in Appendix B.

An Atterberg limits test was carried out on one (1) sample of sandy silt to silty sand deposit which confirmed that it is non-plastic.

4.2.4 Silty Sand to Sand Till

A deposit of brown cohesionless till comprised of silty sand to sand with some silt, trace gravel and clay, containing cobbles, roots and rootlets was encountered below the topsoil in Boreholes B2-01, B2-02 and B2-16. The top of this deposit varies from about Elevation 216.2 m to 215.2 m and the thickness of the deposit ranges from about 0.4 m to 1.2 m.

The SPT 'N'-values measured within the silty sand to sand till deposit typically range from 3 blows to 34 blows per 0.3 m of penetration, indicating a loose to dense relative density. A SPT 'N'-value of 20 blows per 0.15 m of penetration was recorded in Borehole B2-01 prior to split-spoon refusal on bedrock.

The natural water content measured on samples of this deposit range from about 19 percent to 44 percent and the organic content measured on one (1) sample of the upper portion of this deposit is about 10 percent.

The grain size distributions of two (2) samples from the silty sand to sand till deposit are shown on Figure B3 in Appendix B.

4.2.5 Sand and Gravel Till

A deposit of brown cohesionless till comprised of sand and gravel with some silt to silty sand and gravel, trace clay and containing cobbles and boulders was encountered below the topsoil in Borehole B2-04, below the silty sand till deposit in Borehole B2-02, and below the sandy silt deposit in Boreholes B2-11 to B2-13. The top of this deposit varies from about Elevation 216.1 m and 212.1 m and its thickness ranges from 0.3 m to 0.9 m.

The SPT 'N'-values measured within the sand and gravel till deposit range from 35 blows per 0.3 m of penetration to 25 blows per 0.1 m of penetration, indicating a dense to very dense relative density. A SPT 'N'-value of 9 blows per 0.3 m of penetration was recorded within this layer in Borehole B2-04, indicating a very loose relative density at the top of the deposit immediately underlying the peat.

The natural water content measured on samples of this deposit range from about 9 percent to 32 percent.

The grain size distribution on one (1) sample of the sand and gravel till deposit is presented on Figure B4 in Appendix B.

4.2.6 Bedrock

Bedrock was encountered and core samples were recovered from Boreholes B2-01, B2-03, B2-05 to B2-11, B2-13 and B2-15 to B2-17 and the presence of bedrock was inferred from refusal to split-spoon and/or auger advance in Boreholes B2-02, B2-04 and B2-12 and from refusal to shovel penetration in Borehole B2-14. The depth to the bedrock surface is variable and ranges from about 0.1 m to 3.0 m below ground surface. Across the west abutment and centre pier, the bedrock surface elevation varies by up to about 1.8 m and 1.2 m, respectively. Across the east abutment, from the northeast corner to the southwest corner (a distance of about 11 m), the bedrock surface elevation varies by about 3.8 m (approximately 2.9H:1V slope or a dip of approximately 19° from the horizontal). The depth to bedrock below ground surface and corresponding bedrock surface elevation as encountered at the borehole locations is summarized below.

Foundation Element / Approach Embankment	Borehole No.	Depth to Bedrock Surface (m)	Bedrock Surface Elevation (m)	Refusal Type
West Approach Embankment	B2-16	1.3	214.0	Bedrock Cored
West Abutment	B2-01	1.0	214.7	Bedrock Cored
	B2-02	0.9	215.5	Spoon/Auger Refusal
	B2-03	0.0	216.5	Bedrock Cored
	B2-04	1.0	215.3	Auger Refusal
	B2-05	0.3	216.3	Bedrock Cored
Centre Pier	B2-06	0.1	215.6	Bedrock Cored
	B2-07	0.1	215.7	Bedrock Cored
	B2-08	0.3	215.6	Bedrock Cored
	B2-09	0.1	216.5	Bedrock Cored
	B2-10	0.1	215.3	Bedrock Cored
East Abutment	B2-11	2.0	212.6	Bedrock Cored
	B2-12	3.0	211.2	Spoon/Auger Refusal
	B2-13	1.7	213.2	Bedrock Cored
	B2-14	0.2	215.0	Shovel Refusal
	B2-15	2.4	212.2	Bedrock Cored
East Approach Embankment	B2-17	0.2	214.7	Bedrock Cored

Based on the samples of bedrock obtained from the boreholes, the bedrock generally consists of granite/syenite/biotite gneiss. In general, the bedrock samples are described as moderately weathered to fresh, fine to coarse crystalline, slightly foliated to foliated, black and white syenite gneiss. The Rock Quality Designation (RQD) measured on the core samples is typically between about 75 percent and 100 percent, indicating a rock mass of good to excellent quality, according to Table 3.10 in CFEM (2006)[3]. However, portions of core recovered from Boreholes B2-06 and B2-07 on the west side of the centre pier and B2-16 and B2-17 under the approach embankments, contain zones of moderately to highly weathered, fractured rock with RQD values between about 20 percent and 67 percent, indicating a rock mass of poor to fair quality. The Total Core Recovery (TCR) and Solid Core Recovery (SCR) of samples recovered are typically between about 89 percent and 100 percent and about 76 percent and 100 percent, respectively however, at Borehole B2-16 the SCR was noted to be as low as about 33 percent.

Point load strength tests were performed on selected samples of the rock core. The axial and diametral point load strength index values are shown on the Record of Drillhole sheets and in Table B1 in Appendix B. The point load index (Is_{50}) results from the axial laboratory tests carried out on seven (7) samples of the syenitic bedrock range from approximately 2.6 MPa to 8.4 MPa, with an average of about 6.8 MPa. The point load index (Is_{50}) results from the diametral laboratory tests carried out on three (3) samples of the syenitic bedrock range from approximately 6.1 MPa to 8.8 MPa, with an average of about 7.3 MPa.

Four (4) Unconfined Compression (UC) tests were carried out in accordance with ASTM D7102 (Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens) on selected samples of the syenite gneiss bedrock, that measured compressive strengths between about 78 MPa and 121 MPa with an average value of about 98 MPa, as summarized in Tables B2-1 to B2-4 in Appendix B.

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

Based on the laboratory UC tests and point load testing results in accordance with Table 3.5, CFEM 2006[3], the the syenitic bedrock is classified as strong (R4, 50 MPa < UCS < 100 MPa) to very strong (R5, 100 MPa < UCS < 250 MPa).

4.2.7 Groundwater Conditions

The water level in the boreholes as noted during and upon completion of drilling operations was typically between about Elevation 215.9 m and Elevation 213.5 m, measured at ground surface to about 3.1 m below ground surface. In general, the samples taken in the overburden boreholes were noted to be moist to wet. Standpipe piezometers were installed in Boreholes B2-01, B2-09, B2-10 and B2-15 to permit monitoring of the water levels at this site. Details of the piezometer installations are shown the Record of Borehole and Drillhole sheets in Appendix A. The groundwater levels measured in the piezometer installations are summarized below.

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

Foundation Element	Borehole No.	Ground Surface Elevation (m)	Groundwater Elevation (m)	Date of Measurement
West Abutment	B2-01	215.7	215.3 215.5 215.3	November 5, 2008 April 12, 2009 June 10, 2009
Centre Pier	B2-09	216.6	215.6 214.8 213.5	November 5, 2008 April 12, 2009 June 10, 2009
	B2-10	215.4	213.6 213.7 213.6	November 5, 2008 April 12, 2009 June 10, 2009
East Abutment	B2-15	214.6	214.2 214.2 214.2	November 5, 2008 April 12, 2009 June 10, 2009

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

5.0 CLOSURE

The field technician directing the drilling and sampling program was Mr. Chris Radway. This report was prepared by Mr. Matthew Kelly, 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

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PART B

FOUNDATION DESIGN REPORT

SHEBESHEKONG ROAD UNDERPASS STRUCTURE, SITE NO. 44-442

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. 5183-06-01 (Phase 2 of G.W.P. 5402-05-00)

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides engineering design recommendations for the proposed Shebeshekong Road underpass structure. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the design of the structure 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, a member of MMM Group Limited on behalf of Ministry of Transportation, Ontario (MTO) to provide recommendations on foundation aspects for the New Shebeshekong Road underpass structure and approach embankments. The scope of work includes carrying out stability and settlement analyses, the assessment of foundation options and recommendations of geotechnical resistances. The work also includes addressing foundation aspects for the final design and construction of the structure foundations and approach embankments, including requirements for sub-excavation of soft / organic materials and placement of new fill as well as requirements for rock blasting and excavation.

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

It is understood that the Shebeshekong Road underpass structure will consist of a two-span, pre-cast concrete girder bridge of 39 m span lengths, with the centre pier located in the median.

Based on the General Arrangement (GA) Drawing provided by MRC on September 19, 2008, the grade of the proposed Shebeshekong Road bridge deck will vary between about Elevation 220.1 m and 220.9 m, about 4 m to 6 m above the existing ground surface. In comparison, the grade of the proposed new Highway 69 NBL and SBL in the area of the proposed underpass is to be at about Elevation 212.8 m, about 2 m to 5 m below the existing ground surface, constructed in cut within the shallow bedrock. The proposed Shebeshekong Road west and east approach embankments will be up to about 5.5 m high on both sides of the bridge.

6.2 Foundation Options

Shallow foundations comprised of spread footings founded either directly on the bedrock (or mass concrete on bedrock) or “perched” abutments founded on compacted granular pads within the approach embankments fill are considered appropriate to support the bridge structure given the proximity of bedrock to the existing ground surface and proposed underside of the footings.

Due to the shallow nature of the overburden deposits at the site, pile foundations will not be practical and a significant amount of excavation/trenching into the very strong bedrock would be required to achieve the minimum required pile lengths for an integral abutment design which would likely be cost prohibitive.

The following sections provide recommendations for shallow spread footings to support the structure. From a foundations perspective, a shallow foundation is considered most practical for construction at this site and is the preferred alternative.

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

6.3 Spread Footings

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

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

6.3.1 Footing Founding Options

There are several shallow foundation alternatives/founding elevations that can be considered for support of the abutments and pier footings at the site. The alternatives are summarized below.

The surface of the bedrock at the proposed foundation elements, as established at the investigated locations, ranges from about Elevation 211.2 m to 216.5 m. Given the shallow nature of the overburden (or lack thereof at some locations), the near surface gneiss bedrock is considered most suitable for the support of the proposed bridge foundations. The bedrock surface elevation as encountered in the boreholes at each proposed foundation element is summarized below.

Foundation Element	Boreholes	Bedrock Surface Elevation (m)
West Abutment	B2-01 to B2-05	214.7 to 216.5
Centre Pier	B2-06 to B2-10	215.3 to 216.5
East Abutment	B2-11 to B2-15	211.2 to 215.0

Based on the borehole results there is some variability in the bedrock surface elevation within the limits of each foundation element. In addition, the upper portion of the bedrock is, in a few local areas, moderately fractured (RQD values of about 35 percent and 51 percent as encountered in Boreholes B2-06 and B2-07) and it may be necessary to sub-excavate loose or fractured rock from within some areas of the foundation footprints prior to construction. For design, three options for founding levels may be considered as presented below.

1. The following foundation elevations may be assumed for footings on mass concrete:

West Abutment:	Elevation 216.5 m
Centre Pier:	Elevation 216.5 m (or lower – see below)
East Abutment:	Elevation 215.0 m

   In this case, following removal of the overburden, the bedrock surface would have to be cleaned (including removal of all loose or fractured rock) and then mass concrete would be placed to raise the grade to the footing founding level. All mass concrete construction should be in accordance with OPSS.PROV 904 (*Concrete Structures*). The advantage of this approach is that excavation into the strong to very strong bedrock is avoided.

2. Alternatively, the following foundation elevations may be assumed for footings constructed directly on bedrock:

West Abutment:	Elevation 214.7 m
Centre Pier:	Elevation 215.3 m
East Abutment:	Elevation 211.2 m

   In this case, following the removal of the overburden, excavation of the higher portions of the bedrock will be required within the foundation footprints. Based on the borehole results, sub-excavation of up to about 3.8 m of bedrock will be required in some foundation areas. It is noted that the bedrock is classified as strong to very strong rock (i.e. estimated unconfined compressive strengths in the range of about 80 MPa to about 120 MPa) and the level of fracturing in the upper portions of the bedrock is variable. This will make excavation of the bedrock potentially difficult particularly in areas where only small depths and narrow zones of removal are needed. Recommendations for bedrock excavation and blasting are provided in Section 6.9.

3. As a third option, an intermediate founding level may be assumed for design. In this case, a combination of bedrock sub-excavation and mass concrete placement will be required.

Based on the GA Drawing provided by MRC, the underside of the pier footing is proposed to be located below about Elevation 210 m, about 6 m below the existing bedrock surface and about 2 m below the final grade in the median area. Based on the quality of the bedrock encountered at Boreholes B2-06 and B2-10, the bedrock at this elevation is considered suitable for support of the pier footing at this level. However, additional bedrock excavation through the strong to very strong bedrock will be required.

It should be noted that footing excavations to expose the bedrock surface will, in some places, extend through relatively thin, water-bearing sand and silt deposits. Groundwater control measures (as discussed in Section 6.8.2) may be required in order to maintain a dry and stable excavation especially during periods of high groundwater levels.

As an alternative to supporting the abutment footings on bedrock (or mass concrete), consideration could be given to the use of abutment footings “perched” within the approach embankments. This option would require that the spread footing be founded on a pad of well compacted granular fill (i.e. not founded on rock fill) and that the overburden soils are removed prior to the placing of granular fill. However, given the shallow and variable depth to the bedrock surface, in order to avoid stress concentrations within the footing that could occur due to differential settlement at transition points between a portion of the footing on bedrock and a portion on the granular pad, it may be necessary to remove bedrock across some portions of the foundation footprints to ensure that the granular pad is a minimum 1 m thick at all locations below the footings.

The simplest option for the bridge foundations, from a technical perspective, is spread footings placed on the bedrock surface or on mass concrete placed on the bedrock surface which minimizes the amount of bedrock excavation required.

6.3.2 Footing Set-Back from Rock Faces

The abutment footings will be situated on the bedrock above the adjacent new Highway 69 road grade. The footings must be maintained an adequate distance away from the edge of the new rock cut, and the rock face adequately cleaned and/or protected such that the integrity of the rock face/founding rock surface is maintained. In this regard, the abutment footings should be located away from the rock face at least a distance as defined by an imaginary line projected at 0.5 horizontal to 1 vertical (0.5H:1V) from the toe of the rock cut and not closer than about 2 m from the crest of the nearest new rock cut. If the bridge layout does not allow for this setback zone, special measures (such as the installation of vertical rock dowels behind the crest of the rock face prior to rock excavation) are required to control and pre-support the rock face during blasting and following construction.

6.3.3 Geotechnical Axial Resistance

The following summarizes the design factored geotechnical axial resistances for spread footings constructed directly on bedrock, mass concrete on bedrock or on a compacted granular pad.

Spread Footing Alternatives	Factored Geotechnical Axial Resistance at Ultimate Limit States (ULS) (kPa)	Geotechnical Resistance at Serviceability Limit States (SLS) for 25 mm of Settlement (kPa)
Spread footing on properly prepared gneiss bedrock	10,000	--
Spread footing on mass concrete on properly prepared gneiss bedrock[1]	10,000	--
Spread footing on a minimum 1 m thick Granular 'A' Pad	900	350

Note: [1] Assuming that the strength of the mass concrete is at least 25 MPa.

The geotechnical resistances provided above are given under the assumption that the loads will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with Sections 6.7.2 and 6.7.4 of the *Canadian Highway Bridge Design Code (CHBDC), 2006* and its *Commentary*.

For spread footings founded on the properly prepared and inspected bedrock, the geotechnical resistance at SLS for 25 mm of settlement will be greater than the factored geotechnical axial resistance at ULS and as a result the SLS condition does not apply. For footings placed on mass concrete, the factored geotechnical axial resistance at ULS as given above assumes that the strength of the concrete used to form the pad is at least 25 MPa.

Following excavation of the overburden and prior to placing any concrete, it will be necessary to clean, scale and remove all loose, shattered and/or fractured rock within the footprint of the footings to ensure a proper bond to the bedrock. 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 6.3.4).

6.3.4 Resistance to Lateral Loads

The resistance to lateral forces/sliding resistance between the concrete footings and the bedrock (or the compacted Granular ‘A’ pad, if applicable), should be calculated in accordance with Section 6.7.5 of the *Canadian Highway Bridge Design Code (CHBDC)*. The following summarises the coefficient of friction, tan δ, for the various interface materials.

Interface Materials	Coefficient of Friction (tan δ)
Mass Concrete or Concrete Footing on Bedrock	0.70
Concrete Footing on Compacted Granular ‘A’	0.58

These values represent unfactored values; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating horizontal resistance.

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

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

The dowels should have a minimum embedded length of 1 m within the competent bedrock, and the structural strength of the dowel and compressive strength of the grout should not be exceeded. Depending on the selected founding elevation for the footings at this site, it is recommended that the upper portion of the bedrock where RQDs are less than 50% (i.e. poor or very poor quality) should be ignored in the calculation of required bond length.

If dowelling into bedrock is adopted for resistance to sliding at this site, a NSSP should be included in the Contract Documents to specify the installation, material and testing of the dowels; an example NSSP is included in Appendix C.

6.3.5 Frost Protection

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

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

Perched abutment footings founded on Granular ‘A’ pads should be provided with a minimum of 1.8 m of soil cover for frost protection.

In addition, the following should also be noted for the design of spread footings perched 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 cover and provide protection from frost action. In this case, the MTO has adopted an equivalent of 25 mm of styrofoam to 300 mm of soil cover.

6.4 Site Considerations

6.4.1 Site Coefficient

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

6.4.2 Seismic Analysis

According to the National Building Code of Canada (1995) seismic hazard values (as referenced in the CHBDC and its Commentary), the site specific peak horizontal ground acceleration for the Parry Sound area is 0.051 (for a probability of exceedance of 10 percent in 50 years). According to Table 4.1 of the *CHBDC,* this site is located in Seismic Performance Zone 1 and the corresponding site–specific zonal acceleration ratio, A, is 0.05.

Given this assessment and the fact that that the proposed bridge structure is not designated as a lifeline or truss bridge, and in accordance with Section 4.4.5.3 of the *CHBDC*, no seismic analysis is required for structures located in Seismic Performance Zone 1.

6.5 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 do not need to be analysed for this bridge structure.

The following recommendations are made concerning the design of the walls. It should be noted that these design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Select, free draining granular fill meeting the specifications of OPSS.PROV 1010 (*Aggregates*) Granular 'A' or Granular 'B' Type II, but with less than 5 per cent passing the No. 200 sieve (0.075 mm), 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 OPSS 501 (*Compacting*) and Special Provision 105S21 (*Water Requirements*). 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 in a zone with the width equal to at least 1.8 m behind the back of the wall (in accordance with Figure C6.20(a) of the *Commentary* to the *CHBDC*). For unrestrained walls, 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:

Fill Type	**Soil Unit Weight**	**Coefficients of Static Lateral Earth Pressure**	
		At-Rest, K_o	**Active, K_a**
Granular 'A'	22 kN/m^3	0.43	0.27
Granular 'B' Type II	21 kN/m^3	0.43	0.27
Granular Fill	21 kN/m^3	0.31	0.47

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 foundation 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 approaches to the Shebeshekong Road underpass will be up to about Elevation 220.1 m and Elevation 220.9 m, at the east and west embankments respectively, requiring placement of up to about 5.5 m of fill within the limits of the approach embankments. In addition, the grade of the proposed new Highway 69 NBL and SBL in the area of the proposed underpass is to be at about Elevation 212.8 m, about 2 m to 5 m below the existing ground surface, constructed in cut within the shallow bedrock immediately adjacent to the front slope of the new approach fills.

Based on the investigation locations at this site, the approach embankments will be founded on the loose to compact sandy silt and silty sand and gravel till (east abutment area) or loose to dense silty sand to sand till underlain by bedrock, or directly on the bedrock. All topsoil, organic matter and any localized deposits of silty clay should be stripped from below the approach embankment areas, and all subgrade soils should be proof-rolled prior to fill placement.

Where the toe of the new approach embankments will be located close to the excavated rock cut faces (i.e. the rock cuts made for the new Highway 69 NBL and SBL), the set-back from the crest of the rock cut to the toe of the embankment should be a minimum of 1.5 m. Good quality, controlled blasting methods, under the guidance of a blasting specialist, will be critical to maintain the excavation lines and preserve the integrity of the rock crest and mass. Recommendations for bedrock excavation and blasting are provided in Section 6.9.

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

6.6.1.2 Parameter Selection

The soils encountered below the proposed approach embankments consist primarily of cohesionless materials with bedrock at shallow depth.

For the loose to dense cohesionless subsoils, the effective stress parameters employed in the analysis were estimated from empirical correlations based on the results of the in situ Standard Penetration Tests (SPT). The correlations proposed by Peck et al (1974) and NAVFAC (1986) were employed and the results were tempered by engineering judgment based on precedent experience in similar soils.

At all areas, the analyses assume that any organic soils and localized deposits of silty clay have been removed prior to construction of the new embankments. The piezometric conditions required in the analysis were based on the groundwater levels noted during the drilling of the boreholes and measured in the piezometer installations. In general, the groundwater table was assumed to be located at the existing ground surface level.

The following summarize the simplified stratigraphy and the associated strengths and unit weights employed for the different soil types in the approach embankment areas. For the purpose of analysis, both granular fill and rock fill have been considered for the construction of the approach embankments as indicated below. Granular fill is assumed to have side slopes at 2H:1V and 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	**Soil Type**	**Unit Weight (kN/m^3)**	**Undrained Shear Strength (kPa)**	**Cohesion, c' (kPa)**	**Angle of Internal Friction, ϕ' (degrees)**
West Approach STA 9+961	New Rock Fill	19	--	--	40
	New Granular Fill (sand and gravel)	21	--	--	35
	Loose to Dense Native Silty Sand to Sand (Till)	19	--	--	32
East Approach STA 10+040	New Rock Fill	19	--	--	40
	New Granular Fill (sand and gravel)	21	--	--	35
	Dense Native Silty Sand and Gravel (Till)	19	--	--	32

6.6.1.3 Results of Analysis

The results of the stability analyses for the two embankment fill options (granular fill and rock fill) are summarized below for the east and west approach embankments. The minimum factor of safety is based on a deep-seated, global trial failure surface that would impact the operation of the roadway.

Approach Embankment	Embankment Height at Critical Section (m)	Granular Fill Option		Rock Fill Option	
		Recommended Side Slope Profile	Minimum Factor of Safety	Recommended Side Slope Profile	Minimum Factor of Safety
West Approach STA 9+961	5.5	2H : 1V	≥ 1.3	1.25H : 1V	≥ 1.3
East Approach STA 10+040	5.5				

6.6.1.4 *Embankment Fill Types*

The 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), side slope, construction cost and time, and ease of construction/availability.

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, this option will require a larger volume of fill and wider right-of-way because the side slopes will be flatter than rock fill slopes.

Rock Fill

The main advantage of using rock fill is the ability to achieve steeper embankment side slopes. This is useful in areas with limited right-of-ways and rock fill would likely be available locally. The disadvantage of using rock fill for the construction of high embankments is that some post-construction settlement of the embankment fill itself will occur within about the first year of construction.

6.6.2 Settlement

Some relatively small settlement of the approach embankments can be expected as a result of the loading from the new fills on the thin, primarily cohesionless foundation soils at this site. In addition, for the proposed rock fill materials to be employed in the construction of the embankments, some settlement will also occur due to compression of the new embankment fill itself.

The majority of rock fill settlement will occur within the first year following construction, but some settlement will occur over the life of the embankment.

6.6.2.1 *Methodology*

To estimate the magnitude of the expected settlements, analyses were carried out on the critical sections of the proposed approach embankments using the commercially available program Settle^{3D} (version 2.003), produced by Rocscience Inc.

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

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

6.6.2.2 Parameter Selection

At both approaches, the foundation soils are composed primarily of loose to dense sand to sandy silt to silty sand and gravel (till).

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 and surficial organic soils will be removed prior to construction and that rock fill will be used for the new embankment construction.

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

The following summarizes the simplified stratigraphy, unit weights and deformation parameters employed for the different soils types in the approach areas. For the purpose of analysis, both granular fill and rock fill have been considered for the construction of the approach embankments as indicated below. Granular fill is assumed to have side slopes at 2H:1V and 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	Soil	Thickness (m)	Unit Weight (kN/m^3)	Estimated Deformation Properties
West Approach STA 9+961	Rock Fill	5.5	19	Refer to Section 0
	Loose to Dense Native Silty Sand to Sand (Till)	1.2	19	E' = 20 MPa
East Approach STA 10+040	Rock Fill	5.5	19	Refer to Section 0
	Loose to Dense Native Sand to Sandy Silt to Silty Sand and Gravel (Till)	2.2	19	E' = 20 MPa

6.6.2.3 Settlement of Foundation Soils

The results of the estimated settlement of the foundation soils for the east and west approach embankments are summarized below.

Approach Embankment	Maximum New Embankment Height[1] (m)	Estimated Settlement of Foundation Soils (mm)
West Approach STA 9+961	5.5 + 0.3 = 5.8	5
East Approach STA 10+040	5.5 + 0.3 = 5.8	10

Note: [1] Includes additional fill required after removal of maximum depth of topsoil/organic matter

Provided that the surficial organic soils and any localized deposits of silty clay are removed prior to construction, these settlements are expected to occur rapidly (i.e. during or shortly after construction) in response to filling based on the estimated relatively high hydraulic conductivity of the native soils as indicated by results of the grain size distributions.

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 for the fill thickness at this location, 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 up to 5.5 m high approach embankments is presented below.

Approach Embankment	Maximum New Embankment Height[1] (m)	Estimated Short-Term Settlement of Rock Fill (mm)
West Approach STA 9+962	5.5 + 0.3 = 5.8	45
East Approach STA 10+038	5.5 + 0.3 = 5.8	45

Note: [1] Includes additional fill required after removal of maximum depth of topsoil/organic 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 as follows.

Approach Embankment	Maximum New Embankment Height[1] (m)	Estimated Long-Term Settlement of Rock Fill (mm)
West Approach STA 9+962	5.5 + 0.3 = 5.8	5
East Approach STA 10+038	5.5 + 0.3 = 5.8	5

Note: [1] Includes additional fill required after removal of maximum depth of topsoil/organic 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 75 days is required for the west and east approach embankments to allow for settlement of the rock fill to occur in the short term.

6.6.2.5 Embankment Platform Widening

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

The minimum required embankment platform widening (per embankment side) is calculated based on the estimated consolidation settlement of the foundation soils (including creep) and long-term settlement/compression of the embankment fill plus an additional 200 mm for the future pavement overlay, multiplied by the horizontal component of the side slope of the pavement structure (3H:1V), but cannot be less than the minimum platform widening requirements as described above.

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

6.7 Subgrade Preparation and Embankment Construction

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

6.7.1 Removal of Organic Materials and Localized Silty Clay

Based on the information from the borings obtained during the field investigation, organic deposits (i.e. topsoil) up to about 0.3 m thick can be expected in some areas of the new approach embankments. In addition, a localized deposit of silty clay, up to about 0.9 m thick, was encountered near the southwest corner of the east approach. These organic and silty clay layers should be stripped from the plan limits of the approach embankments areas prior to fill placement.

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 in final position, but should be deposited on and pushed forward over the end of the layer being constructed. Blading, dozing and ‘chinking’ the rock to form a dense, compact mass will be required to minimize voids and bridging. Side slopes for rock fill embankments should be no steeper than 1.25 horizontal to 1 vertical (1.25H:1V).

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

6.8 Design and Construction Consideration

6.8.1 Overburden Excavation

In order to construct the bridge foundations on the bedrock (or on mass concrete on the bedrock), excavations up to about 3 m below the existing ground surface will be required and will be made primarily through the loose to dense sand to sandy silt to silty sand and gravel till. These overburden soils are considered Type 3 soils according to Occupational Health and Safety Act and Regulation for Construction Projects (OHSA). The excavation in the overburden should be carried out with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V). Excavations within the bedrock may be made with vertical or near vertical cut as discussed in Section 6.9.

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

6.8.2 Control of Groundwater and Surface Water

The groundwater level at this site is generally between about 0 m (i.e. at existing ground surface) and about 3.1 m below existing ground surface. Whichever foundation option is selected for the bridge structure, excavation to expose the bedrock surface, plus removal of bedrock at some locations, will be required.

At the centre pier, the water table was measured to be at about Elevation 213.5 m (about 2 m below the elevation of the bedrock surface). Depending on the selected design elevation for the underside of the pier footing, groundwater control may be required during construction at the pier location.

At the east and west abutments, the groundwater table was measured to be above the bedrock surface elevation and coincident with the ground surface at some locations. As such, unwatering will likely be required at the east and west abutment areas to achieve a properly prepared subgrade and to construct the footings in the dry.

Given the relative density and grain size distribution for the sand and silt soils in these areas, and using the limits of dewatering proposed by Powers (1992), it is considered likely that pumping from within the excavations with adequately sized and properly filtered pumps will be sufficient to control the groundwater inflow. Surface water should be directed away from the excavations at all times.

6.8.2.1 Permit to Take Water

A preliminary drawdown/seepage analysis has been carried out to estimate the volume(s) of groundwater flow that may have to be pumped at the west abutment and east abutment in order to keep the groundwater level below the base of the excavations during spread footing construction. Based on upper bound estimates of hydraulic conductivity (k) for the soil deposits at and below the base of the proposed excavated areas (from Hazen (1911)), it is anticipated that pumping in excess of 60,000 litres/day may be required if more than one foundation area is constructed simultaneously. Further, the pumping volumes could increase depending on weather conditions (i.e. precipitation) and time of construction (i.e. snow melt). It is however considered unlikely that pumping more than 400,000 litres/day will be required.

6.9 Recommendations for Rock Excavations and Blasting

6.9.1 Rock Excavation

It should be noted that the bedrock at the site is classified as strong (R4) to very strong (R5) (i.e. estimated uniaxial compressive strengths in the range of about 80 MPa to about 120 MPa) and the degree of fracturing in the upper portions of the bedrock is variable. 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 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 6.3.2, the abutment footings will be founded on the new rock cuts above the new Highway 69 NBL and SBL road grade. As such, an adequate set-back distance between the footings and the crest of the final rock face must be provided. If the layout does not allow for the footing set-back recommended in Section 6.3.2, a NSSP should be provided for vertical rock dowels to be installed behind the crest of the rock face (prior to any blasting or 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 Mr. Matthew Kelly, P. Eng., Ms. T. Veronica Ayetan, P.Eng., and Mr. Christopher Ng, P. Eng. The technical aspects were reviewed by Mr. J. Paul Dittrich, Ph.D., P. Eng., a senior geotechnical engineer and Principal with Golder. Mr. Jorge M. A. Costa, P. Eng., the Designated MTO Contact for this project and Principal with Golder, conducted an independent quality control review of the report.

Report Signature Page

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July 20/2015
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TVA/MWK/CN/JPD/JMAC/jl

\\golder.gds\gal\mississauga\active\2007\1111\07-1111-0029 - mrc - hwy 69 four-laning -\report\final\3 - shebeshekong road underpass\07-1111-0029-3 fidr 15jul20 highway 69 shebeshekong road underpass.docx

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Chapman, L.J., and Putnam, D.F. 1984. The Physiography of Southern, 3rd Edition. Ontario Geological Survey, Special Volume 2. Ontario Ministry of Natural Resources.

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Hazen, A. 1911. Discussions of “Dams on Sand Foundations” by A.C. Koenig, Transactions, ASCE, Vol. 73, pp. 199-203.

Kulhawy, F.H. and Mayne, P.W. 1990. Manual on Estimating Soil Properties for Foundation Design. EL 6800, Research Project 1493 6. Prepared for Electric Power Research Institute, Palo Alto, California.

Peck, R.B., Hanson, W.E., and Thornburn, T.H. 1974. Foundation Engineering, Second Edition, John Wiley and Sons, New York.

Powers, J.P. 1992. Construction Dewatering – New Methods and Applications, 2nd Edition, John Wiley and Sons, New York.

Unified Facilities Criteria. NAVFAC Design Manual 7.02. Soil Mechanics, Foundation and Earth Structures. Alexandria, Virginia, 1986.

STANDARDS:

ASTM International:

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

Contract Design Estimating and Documentation (CDED):

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

Ministry Transportation Ontario:

MTO Guideline for Rock Fill Settlement and Rock Fill Quantity Estimates. September 2010.

Embankment Settlement Criteria for Design. July 2010.

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

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

Ontario Occupational Health and Safety Act:

Ontario Regulation 213/91	Construction Projects.
Ontario Regulation 443/09	Amendment to Ontario Regulation 213.

Ontario Provisional Standard Drawing:

OPSD 202.010	Slope Flattening Using Surplus Excavated Material on Earth or Rock Embankments.
OPSD 3090.101	Foundation, Frost 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 902	Construction Specification for Excavating and Backfilling – Structures.
OPSS.PROV 904	Construction Specification for Concrete Structures.
OPSS.PROV 206	Construction Specification for Grading.
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade, and Backfill Material.

Ontario Water Resources Act:

Ontario Regulation 903	Wells.

TABLES

TABLE 1
EVALUATION OF FOUNDATION ALTERNATIVES – SHEBESHEKONG ROAD UNDERPASS STRUCTURE
W.P. 5183-06-01 / G.W.P. 5111-07-00

Foundation Option	*Rank*	*Advantages*	*Disadvantages*	*Relative Costs*	*Risks / Consequences*
Spread Footings on Bedrock and/or Mass Concrete	1	• Relative east of construction. • Minimizes bedrock excavation depending on design footing level. • Reduced bedrock excavation (as compared with pile option). • Negligible post-construction settlement.	• Variable bedrock surface will require soil and bedrock excavation followed by mass concrete placement to achieve level footing. • Bedrock will have to be blasted using controlled blasting techniques to minimize shattering and over-break. • Does not allow for integral abutment design.	• Lower relative cost than piled foundation option. • Additional costs associated with excavation of bedrock and mass concrete. • Additional costs for vertical dowels, if required, to reinforce the rock face in front of the abutments.	• If bedrock surface elevation is higher than anticipated, additional bedrock excavation is required. • Variability in bedrock surface will impact mass concrete quantities and excavation depth. • Must take measures to ensure integrity of rock face in front of abutment footings or 'dental' concreting/repair or rock dowelling may be required during construction.
Spread Footings on Granular 'A' Pads	2	• Relative east of construction. • Eliminates the need for mass concrete. • Reduces the requirements for bedrock excavation. • Small post-construction settlement.	• Subexcavation of an additional 1 m of bedrock to allow for placement of a minimum 1 m thick Granular 'A' pad may be required in some areas, otherwise stress concentrations will occur on the footing. • Does not allow for integral abutment design.	• Lower relative cost than pile foundation. • Lower relative cost than mass concrete. • Additional cost associated with sub-excavation of an additional 1 m into bedrock in some areas. • Additional costs for vertical dowels, if required, to reinforce the rock face in front of the abutments. • Possible higher relative cost than spread footing on bedrock since lower allowable bearing capacity may require larger footing size.	• Must ensure proper compaction of Granular 'A' pad to minimize post-construction settlement. • Must take measures to ensure integrity of rock face in front of abutment footings or 'dental' concreting/repair or rock dowelling may be required during construction.

TABLE 1
EVALUATION OF FOUNDATION ALTERNATIVES – SHEBESHEKONG ROAD UNDERPASS STRUCTURE
W.P. 5183-06-01 / G.W.P. 5111-07-00

Foundation Option	***Rank***	***Advantages***	***Disadvantages***	***Relative Costs***	***Risks / Consequences***
H-piles in Bedrock Trenches	Not Recommended	• Allows for integral abutment design.	• Bedrock excavation to form trenches will be required to achieve minimum required pile lengths.	• Higher relative cost than spread footings due to significant costs for excavating trenches in bedrock.	• Not recommended due to significant depth of excavation required in strong bedrock.

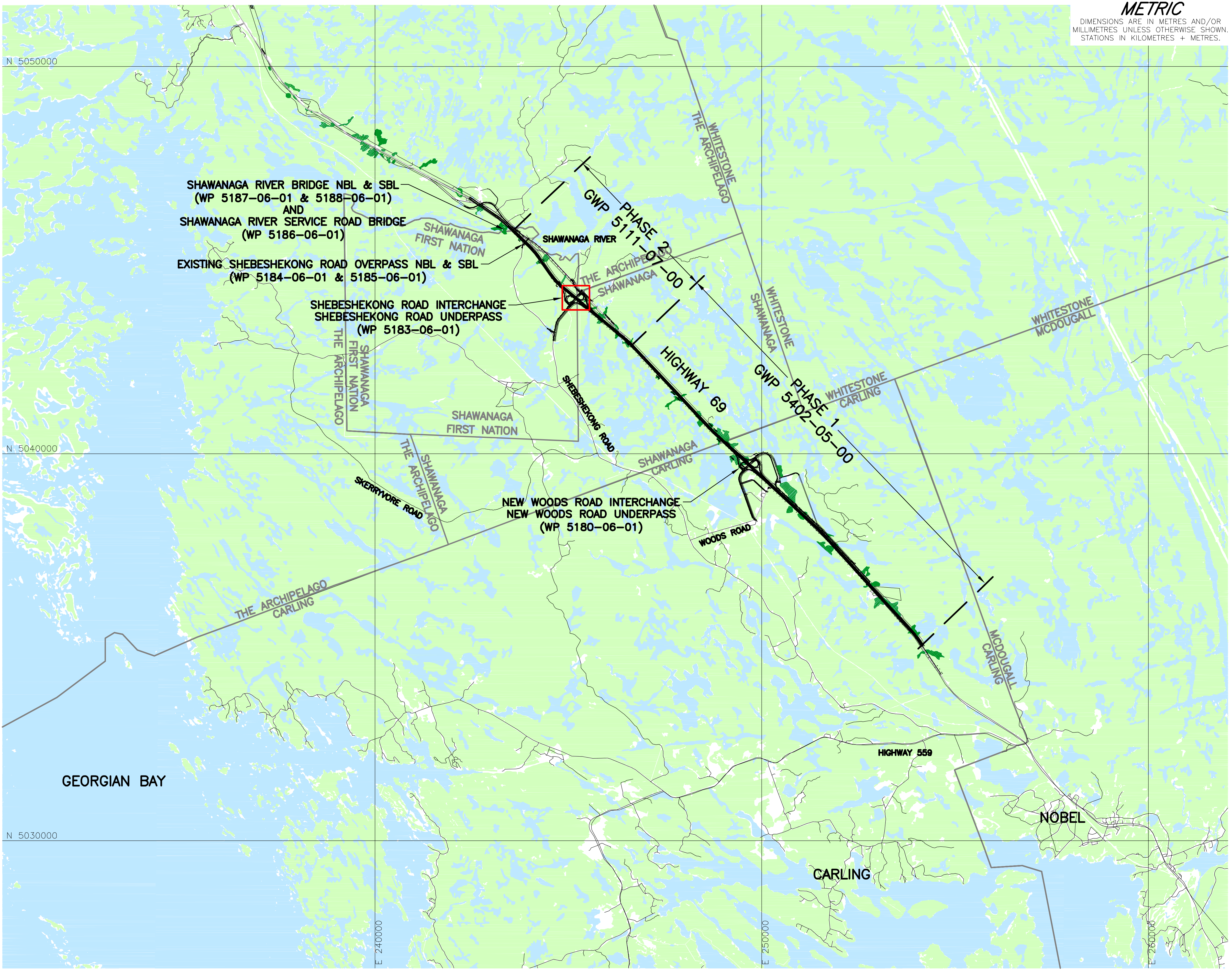
Prepared By: CN

Reviewed By: JPD/JMAC

DRAWINGS

MINISTRY OF TRANSPORTATION, ONTARIO

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



PLAN

SCALE
1 0 1 2 km

CONT No.
WP No. 5183–06–01

SHEET

HIGHWAY 69
SITE LOCATION PLAN

Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

KEY PLAN
NOT TO SCALE

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

NO.	DATE	BY	REVISION

Geocres No. 41H–140

HWY. 69	PROJECT NO. 07–1111–0029		DIST.
SUBM'D. VA	CHKD. VA	DATE: Jul. 2015	SITE: 44–442
DRAWN: DD/CD	CHKD. CN	APPD. JPD/JMAC	DWG. 1

PLOT DATE: July 3, 2015
FILENAME: T:\Projects\2007\07-1111-0029 (MRC, Parry Sound)\-KB- Shebeshekong Road Structure (5020) (5023) - Final\07111110029KB001.dwg

MINISTRY OF TRANSPORTATION, ONTARIO

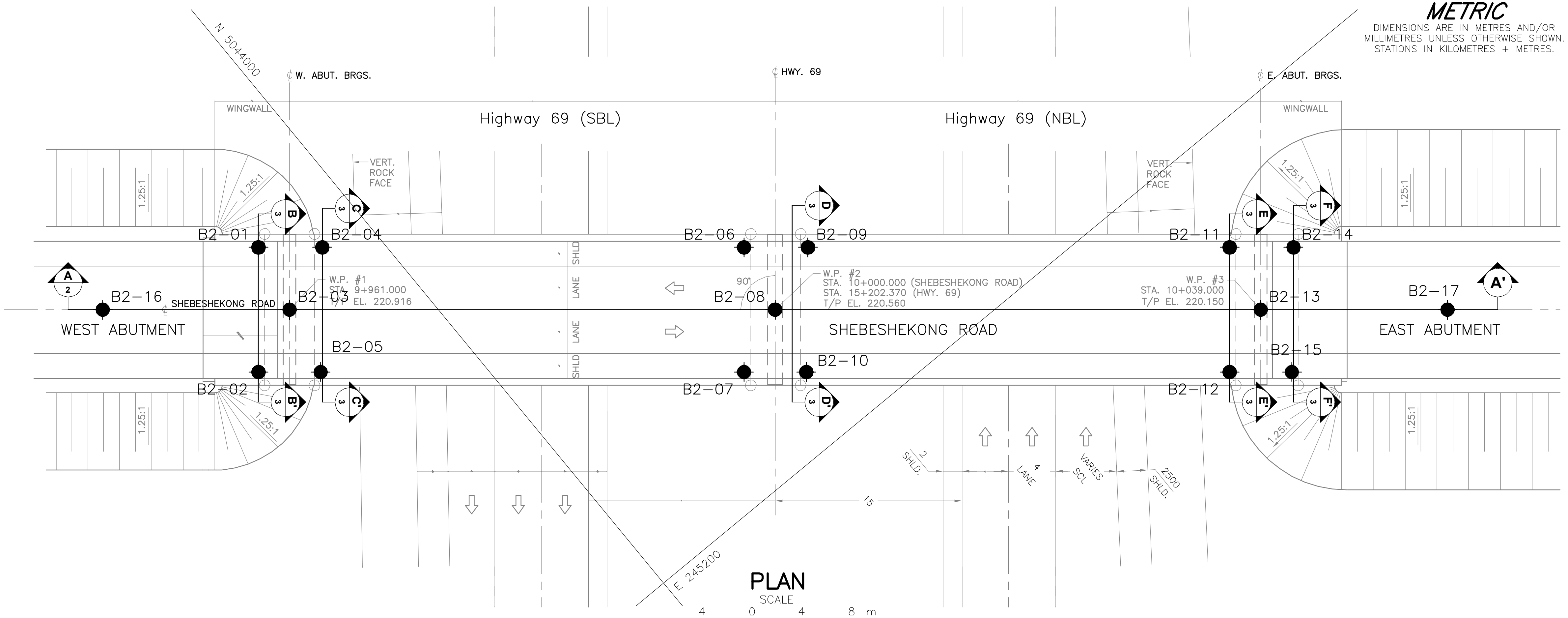
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 5183-06-01

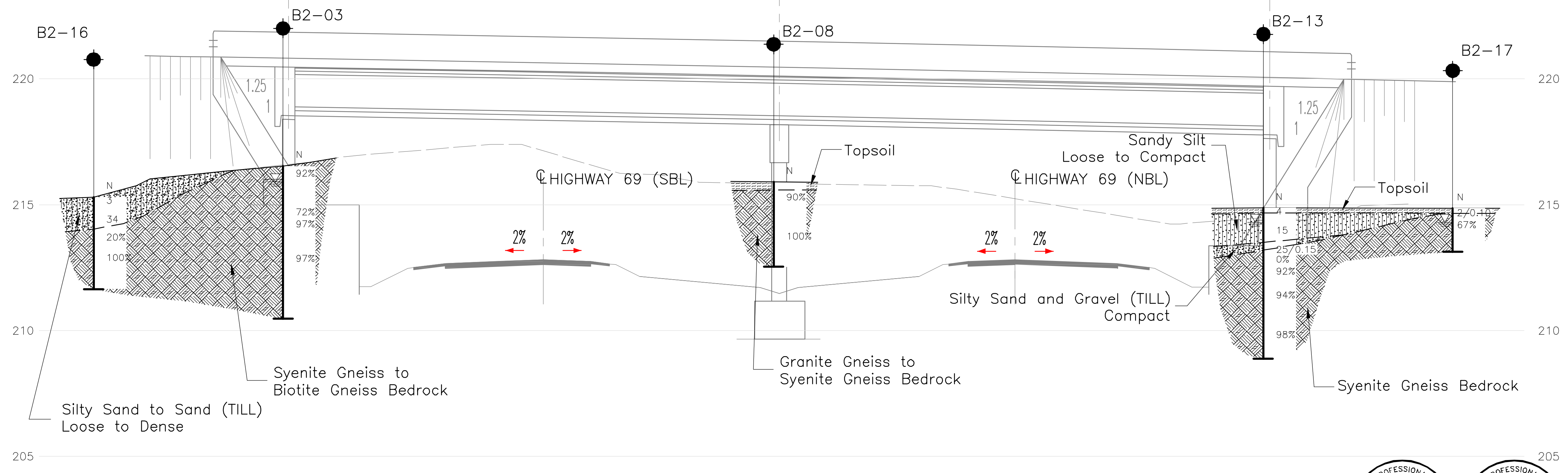
HIGHWAY 69
SHEBESHEKONG ROAD UNDERPASS
BOREHOLE LOCATION
AND SOIL STRATA

SHEET

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



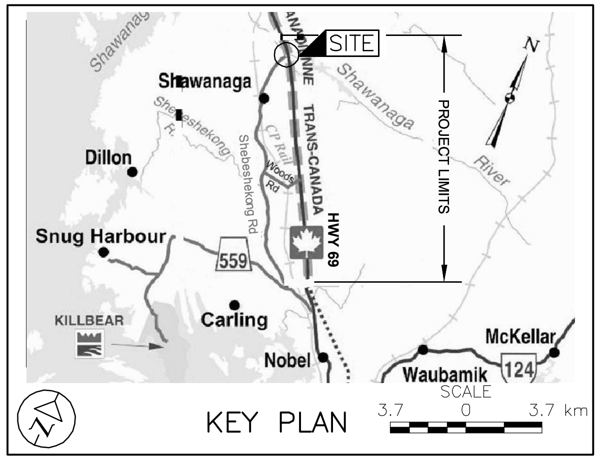
PLAN
SCALE
4 0 4 8 m



CENTRELINE PROFILE
SHEBESHEKONG ROAD UNDERPASS

A-A'
2

SCALE HORIZONTAL
4 0 4 8 m
2 0 2 4 m
SCALE VERTICAL



KEY PLAN

LEGEND

●	Borehole – Current Investigation
▮	Seal
▯	Piezometer
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
100%	Rock Quality Designation (RQD)
▽	WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
B2-01	215.7	5043992.0	245158.5
B2-02	216.4	5043985.7	245166.2
B2-03	216.5	5043990.8	245164.0
B2-04	216.3	5043996.0	245161.8
B2-05	216.6	5043989.5	245169.4
B2-06	215.7	5044022.1	245183.3
B2-07	215.8	5044015.7	245191.1
B2-08	215.9	5044020.9	245188.8
B2-09	216.6	5044026.1	245186.6
B2-10	215.4	5044019.6	245194.2
B2-11	214.6	5044052.2	245208.2
B2-12	214.2	5044045.8	245215.9
B2-13	214.9	5044050.9	245213.6
B2-14	215.2	5044056.2	245211.4
B2-15	214.6	5044049.7	245219.1
B2-16	215.3	5043979.2	245154.4
B2-17	214.7	5044062.5	245223.2

NOTES

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

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

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

J. M. A. COSTA
Jun. 03, 2015

J.P. DITTRICH
Jun. 03, 2015

NO.	DATE	BY	REVISION

Geocres No. 41H-140

HWY. 69		PROJECT NO. 07-1111-0029	DIST.
SUBM'D. MWK	CHKD. MWK	DATE: Jul. 2015	SITE: 44-442
DRAWN: DD/RJ	CHKD. CN	APPD. JPD/JMAC	DWG. 2

REFERENCE

Base plans provided in digital format by MRC, drawing file no. S6878-325-001GA.dwg, received on September 19, 2008.

PLOT DATE: July 3, 2015
FILENAME: T:\Projects\2007\07-1111-0029 (MRC, Parry Sound)\-KB- Shebeshekong Road Structure (5020) (5023) - Final\07111110029KB002.dwg

MINISTRY OF TRANSPORTATION, ONTARIO

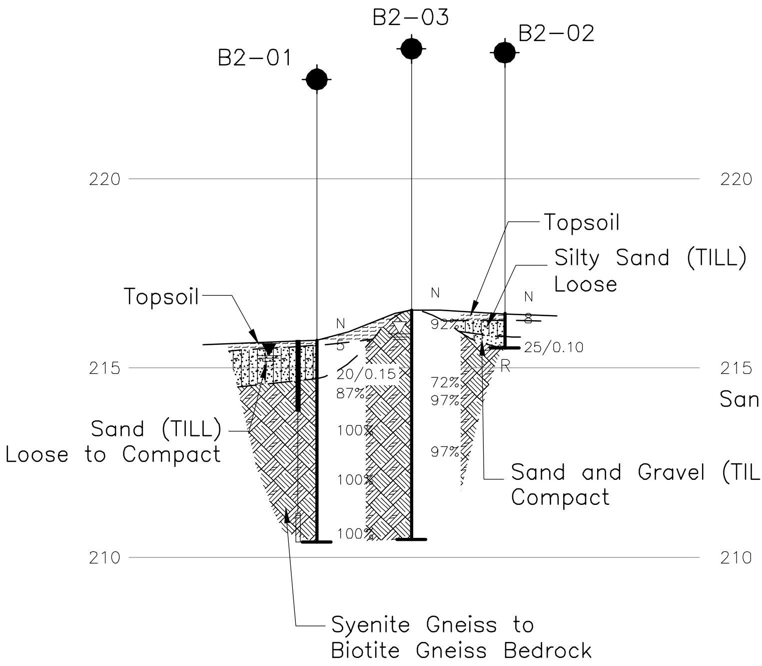
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 5183-06-01

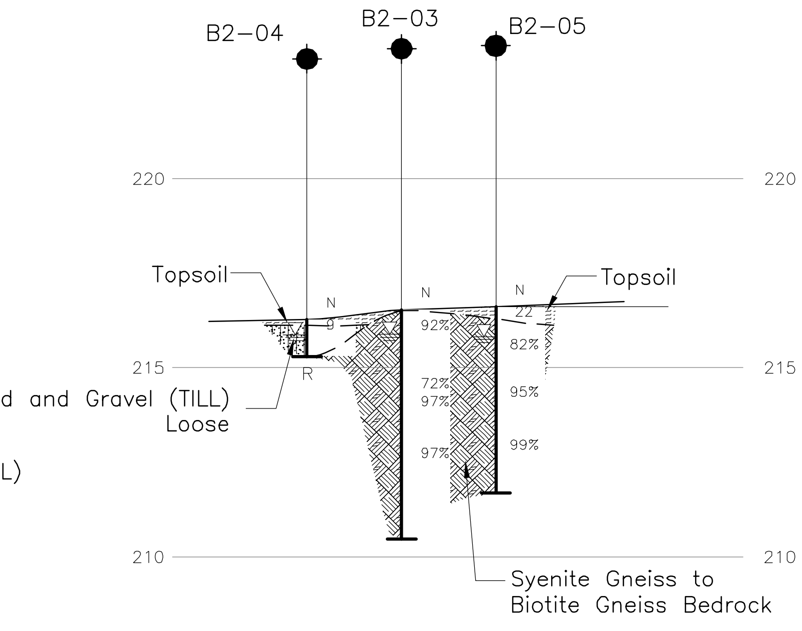
HIGHWAY 69
SHEBESHEKONG ROAD UNDERPASS
SOIL STRATA

SHEET

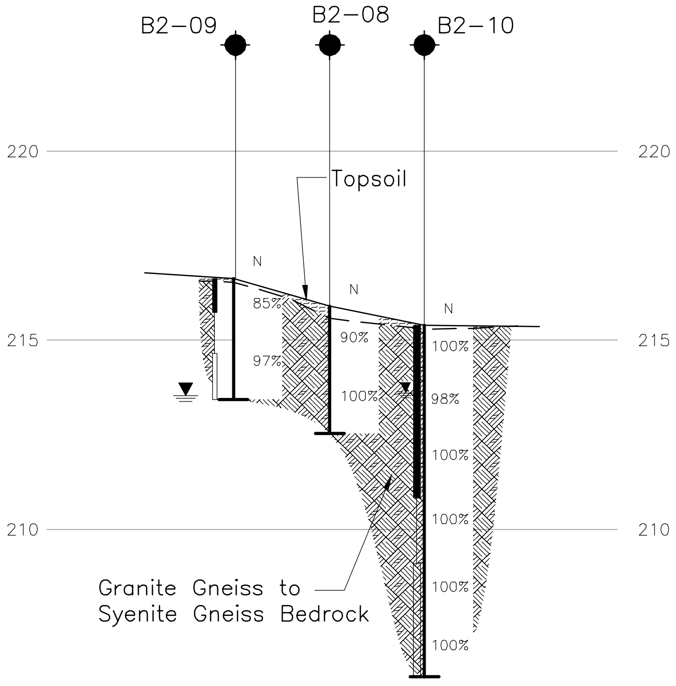
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



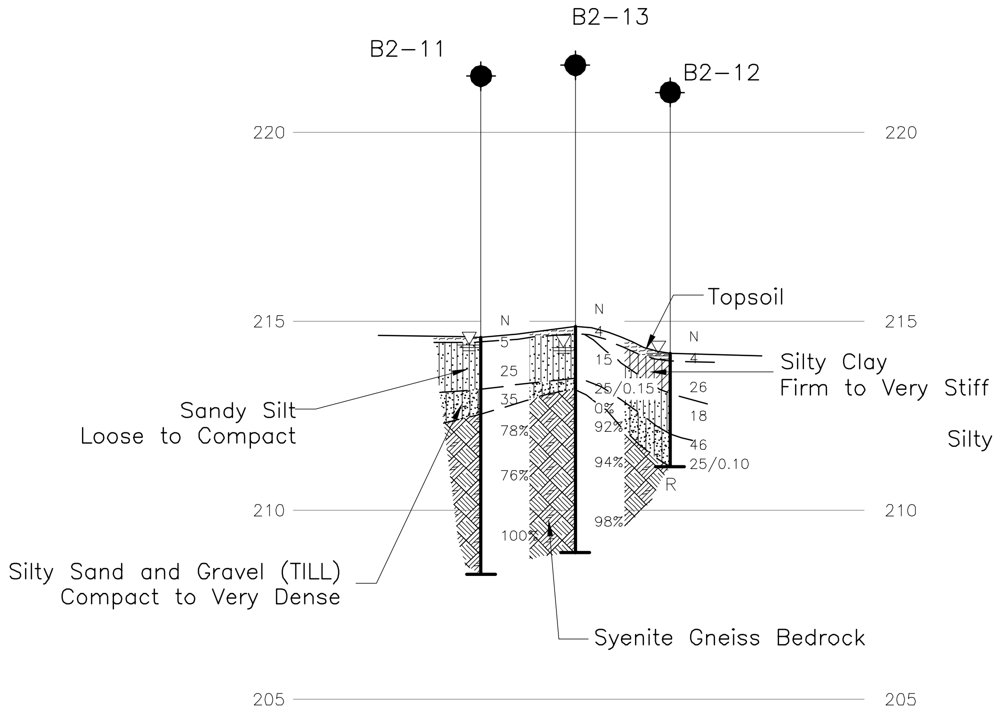
B-B'
2
WEST ABUTMENT (BACK)



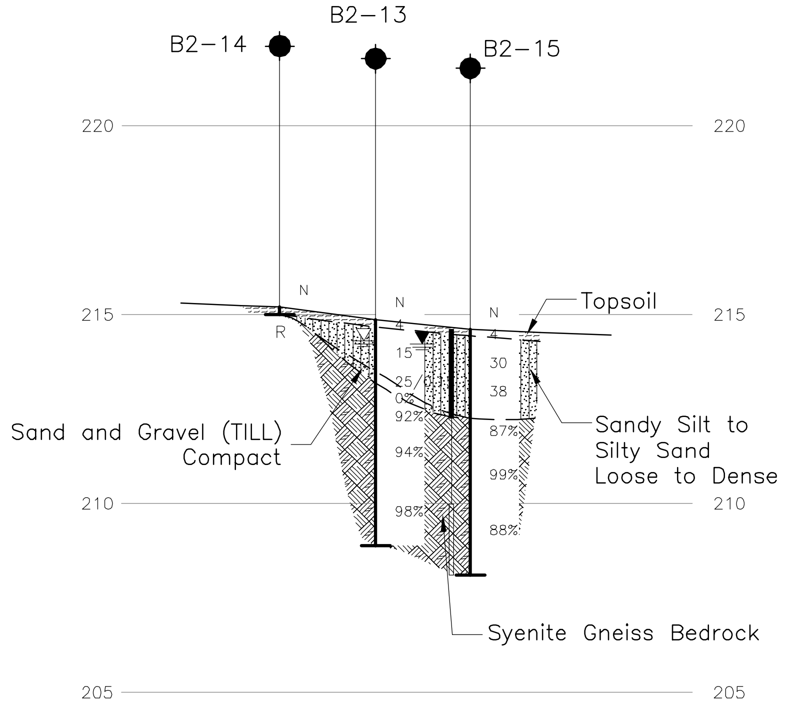
C-C'
2
WEST ABUTMENT (FRONT)



D-D'
2
CENTRE PIER

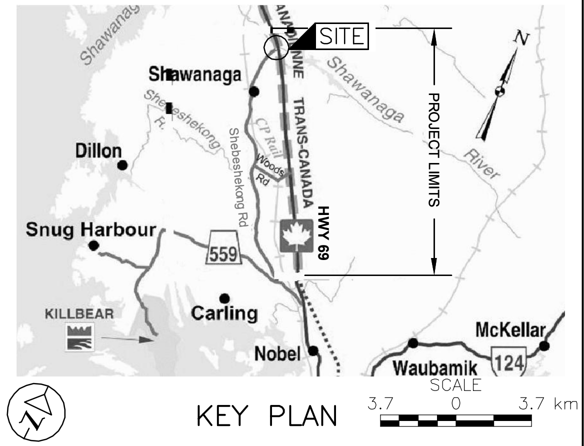


E-E'
2
EAST ABUTMENT (FRONT)



F-F'
2
EAST ABUTMENT (BACK)

SCALE HORIZONTAL
4 0 4 8 m
2 0 2 4 m
SCALE VERTICAL



KEY PLAN

LEGEND

●	Borehole – Current Investigation
▮	Seal
▯	Piezometer
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
100%	Rock Quality Designation (RQD)
▼	WL in piezometer, measured on June 10, 2009
▽	WL upon completion of drilling
R	Refusal

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
B2-01	215.7	5043992.0	245158.5
B2-02	216.4	5043985.7	245166.2
B2-03	216.5	5043990.8	245164.0
B2-04	216.3	5043996.0	245161.8
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B2-08	215.9	5044020.9	245188.8
B2-09	216.6	5044026.1	245186.6
B2-10	215.4	5044019.6	245194.2
B2-11	214.6	5044052.2	245208.2
B2-12	214.2	5044045.8	245215.9
B2-13	214.9	5044050.9	245213.6
B2-14	215.2	5044056.2	245211.4
B2-15	214.6	5044049.7	245219.1

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J. M. A. COSTA
Jun. 03, 2015

J.P. DITTRICH
Jun. 03, 2015

NO.	DATE	BY	REVISION

Geocres No. 41H-140

HWY. 69		PROJECT NO. 07-1111-0029	DIST.
SUBM'D. MWK	CHKD. MWK	DATE: Jul. 2015	SITE: 44-442
DRAWN: DD/RJ	CHKD. CN	APPD. JPD/JMAC	DWG. 3

PLOT DATE: July 3, 2015
FILENAME: T:\Projects\2007\07-1111-0029 (MRC, Parry Sound)\-KB- Shebeshekong Road Structure (5020) (5023) - Final\07111110029KB003.dwg

APPENDIX A

Record of Boreholes and Drillholes

LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\,\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
υ	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
σ_1, σ_2, σ_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 $\tau = c' + \sigma' \tan\phi'$

2 shear strength = (compressive strength)/2

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance; N_d:

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60º 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 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

(b) Cohesive Soils

Consistency

	c_u, s_u	
	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

WEATHERINGS STATE

Fresh: no visible sign of weathering

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

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

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

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

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

BEDDING THICKNESS

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^{o} 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		

RECORD OF BOREHOLE No B2-01

SHEET 1 OF 1

METRIC

PROJECT 07-1111-0029
W.P. 5186-06-01
DIST HWY 69
DATUM Geodetic
LOCATION N 5043992.0 ;E 245158.5
BOREHOLE TYPE NW Casing, Wash Boring
DATE October 27 and 30, 2008
ORIGINATED BY CR
COMPILED BY MWK
CHECKED BY CN

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × REMOULDED
20 40 60 80 100

PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L
WATER CONTENT (%) 10 20 30

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
215.7	GROUND SURFACE							
0.0	TOPSOIL							
0.1	SAND, some silt, trace gravel, trace clay, containing roots and rootlets (TILL) Loose to compact Brown Wet	1	SS	5		44.2		OC = 9.7%
		2	SS	20/0.15	215			4 78 16 2
214.7 1.0	Syenite Gneiss to Biotite Gneiss (BEDROCK) Bedrock cored from depths of 1.0 m to 5.3 m For bedrock coring details, refer to Record of Drillhole B2-01	1	RC	REC 100%	214			RQD = 87%
		2	RC	REC 98%	213			RQD = 100%
		3	RC	REC 100%	212 211			RQD = 100%
210.4		4	RC	REC 100%				RQD = 100%
5.3	END OF BOREHOLE							

NOTE:

1. Water level measured in piezometer:

Date	Depth (m)	Elev (m)
05/11/08	0.4	215.3
12/04/09	0.2	215.5
10/06/09	0.4	215.3

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

PROJECT: 07-1111-0029

RECORD OF DRILLHOLE: B2-01

SHEET 1 OF 1

LOCATION: N 5043992.0 ;E 245158.5

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE:

DRILL RIG: D-50 Turbo (Track-Mounted)

DRILLING CONTRACTOR: Walker Drilling

DATUM: Geodetic

JN - Joint	BD - Bedding	PL - Planar	PO - Polished
FLT - Fault	FO - Foliation	CU - Curved	K - Slickensided
SH - Shear	CO - Contact	UN - Undulating	SM - Smooth
VN - Vein	OR - Orthogonal	ST - Stepped	RO - Rough
CJ - Conjugate	CL - Cleavage	IR - Irregular	VR - Very Rough

MB - Mechanical Break
BR - Broken Rock
NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	FLUSH	COLOUR % RETURN	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	NOTES
		Continued from Record of Borehole B2-01	214.70					
1	NQ RC October 27 and 30, 2008	SYENITE GNEISS Slightly weathered to fresh, fine to medium grained, foliated, black and white	1.00	1	0.3	Light brown 80 - 90	BR,IR,VR BR-FR,IR,Ro FR,IR,VR	
2				2	0.3	Light brown 80 - 90		
3							JN,PL,Ro	
		BIOTITE GNEISS Fresh, fine to medium, crystalline, foliated, black and white	212.20 3.50				JN,PL,Ro-SM	
4				3	0.3	Light brown 80 - 90	FO-JN,PL-ST,Ro FO,PL,SM	
5				4	0.3		FO-JN,PL,SM-Ro	
		END OF BOREHOLE	210.40 5.30					
6								
7								
8								
9								
10								
11								

Column headings: SYMBOLIC LOG; RECOVERY (TOTAL CORE % 80 60 40 20; SOLID CORE % 80 60 40 20); R.Q.D. % 80 60 40 20; FRACT. INDEX PER 0.3 m 5 10 15 20; B Angle 0 90 180 270; DIP w.r.t. CORE AXIS 0 30 60 90; Jr Ja Jn; HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-6} 10^{-5} 10^{-4} 10^{-3}; Diametral Point Load Index (MPa) 2 4 6; RMC -Q' AVG.

GTA-RCK 018 T:\PROJECTS\2007\07-1111-0029 (MRC, PARRY SOUND)\LOG\07-1111-0029-SHEBESHEKONG RD-PHASE II.GPJ GAL-MISS.GDT 07/09/15 DD/SAC

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: CR
CHECKED: CN

RECORD OF BOREHOLE No B2-02

SHEET 1 OF 1

METRIC

PROJECT 07-1111-0029

W.P. 5186-06-01

DIST HWY 69

DATUM Geodetic

LOCATION N 5043985.7 ;E 245166.2

BOREHOLE TYPE 115 mm O.D. Solid Stem Augers

DATE October 30, 2008

ORIGINATED BY CR

COMPILED BY MWK

CHECKED BY CN

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES						
216.4	GROUND SURFACE									
0.0	TOPSOIL									
215.8	Silty SAND, some gravel, trace clay, containing cobbles and roolets (TILL) Loose Brown Moist	1	SS	8	▽	216				
215.5 0.9	SAND and GRAVEL, some silt, trace clay, containing cobbles (TILL) Compact Brown Moist	2	SS	25/0.10				○		
	END OF BOREHOLE SPOON AND AUGER REFUSAL									

NOTE:

1. Water level in open borehole at a depth of 0.8 m below ground surface (Elev. 215.6 m) upon completion of drilling.

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

RECORD OF BOREHOLE No B2-03

SHEET 1 OF 1

METRIC

PROJECT 07-1111-0029

W.P. 5186-06-01

LOCATION N 5043990.8 ;E 245164.0

ORIGINATED BY CR

DIST HWY 69

BOREHOLE TYPE NW Casing, Wash Boring

COMPILED BY MWK

DATUM Geodetic

DATE October 29, 2008

CHECKED BY CN

SOIL PROFILE			SAMPLES								
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
216.5	GROUND SURFACE										
0.0	TOPSOIL Syenite Gneiss to Biotite Gneiss (BEDROCK) Bedrock cored from depths of 0.03 m to 6.1 m For bedrock coring details, refer to Record of Drillhole B2-03		1	RC	REC 100%	∇	216				RQD = 92%
			2	RC	REC 96%		215				RQD = 72%
			3	RC	REC 100%		214				RQD = 97%
			4	RC	REC 99%		213 212 211				RQD = 97%
210.4 6.1	END OF BOREHOLE NOTE: 1. Water level in open borehole at a depth of 0.6 m below ground surface (Elev. 215.9 m) upon completion of drilling.										

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

PROJECT: 07-1111-0029

RECORD OF DRILLHOLE: B2-03

SHEET 1 OF 1

LOCATION: N 5043990.8 ;E 245164.0

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE:

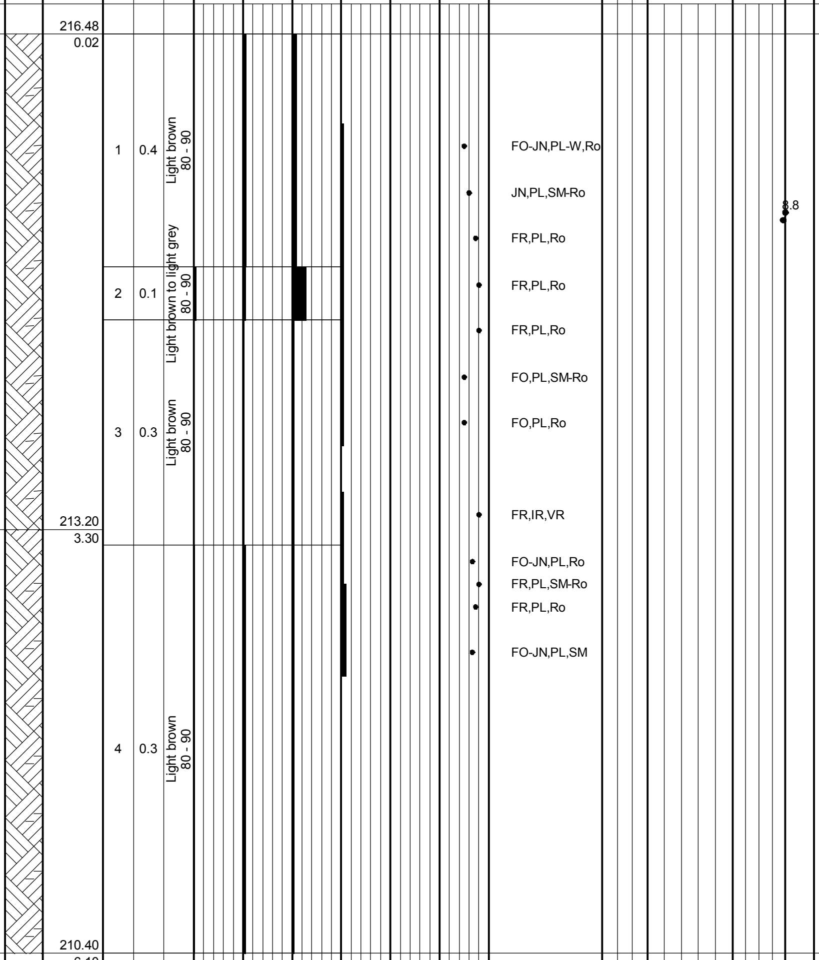
DRILL RIG: D-50 Turbo (Track-Mounted)

DRILLING CONTRACTOR: Walker Drilling

DATUM: Geodetic

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	

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.		FLUSH COLOUR % RETURN
		Continued from Record of Borehole B2-03	216.48			
	NQ RC October 29, 2008	SYENITE GNEISS Slightly weathered, fine to medium, crystalline, foliated, black and white	0.02	1	0.4	Light brown 80 - 90
1				2	0.1	Light brown to light grey 80 - 90
2				3	0.3	Light brown 80 - 90
3			213.20 3.30			
		BIOTITE GNEISS Slightly weathered to fresh, fine to medium, crystalline, foliated, black and white		4	0.3	Light brown 80 - 90
4						
5						
6			210.40 6.10			
		END OF DRILLHOLE				
7						
8						
9						
10						



TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn
FO-JN,PL-W,Ro			
JN,PL,SM-Ro			
FR,PL,Ro			
FR,PL,Ro			
FR,PL,Ro			
FO,PL,SM-Ro			
FO,PL,Ro			
FR,IR,VR			
FO-JN,PL,Ro			
FR,PL,SM-Ro			
FR,PL,Ro			
FO-JN,PL,SM			

NOTES

GTA-RCK 018 T:\PROJECTS\2007\07-1111-0029 (MRC, PARRY SOUND)\LOG\07-1111-0029-SHEBESHEKONG RD-PHASE II.GPJ GAL-MISS.GDT 07/09/15 DD/SAC

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: CR

CHECKED: CN

RECORD OF BOREHOLE No B2-04

SHEET 1 OF 1

METRIC

PROJECT 07-1111-0029
W.P. 5186-06-01
DIST HWY 69
DATUM Geodetic
LOCATION N 5043996.0 ;E 245161.8
BOREHOLE TYPE 115 mm O.D. Solid Stem Augers
DATE October 30, 2008
ORIGINATED BY CR
COMPILED BY MWK
CHECKED BY CN

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						GR	SA	SI	CL
216.3	GROUND SURFACE													
0.0	TOPSOIL		1A	SS	9	▽	216		○		50	38	11	1
0.2	SAND and GRAVEL, some silt, trace clay, containing cobbles (TILL) Loose Brown Wet		1B											
215.3 1.0	END OF BOREHOLE AUGER REFUSAL													

NOTE:

1. Water level in open borehole at a depth of 0.4 m below ground surface (Elev. 215.9 m) upon completion of drilling.

$+^3, \times^3$: Numbers refer to Sensitivity

$\bigcirc^{3\%}$ STRAIN AT FAILURE

Golder Associates

Foundation Design

RECORD OF BOREHOLE No B2-05

SHEET 1 OF 1

METRIC

PROJECT 07-1111-0029

W.P. 5186-06-01

LOCATION N 5043989.5 ;E 245169.4

ORIGINATED BY CR

DIST HWY 69

BOREHOLE TYPE NW Casing, Wash Boring

COMPILED BY MWK

DATUM Geodetic

DATE October 29, 2008

CHECKED BY CN

SOIL PROFILE			SAMPLES								
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
216.6	GROUND SURFACE										
0.0	TOPSOIL		1	SS	22/0.05						
216.3 0.3	Syenite Gneiss (BEDROCK) Bedrock cored from depths of 0.3 m to 4.9 m For bedrock coring details, refer to Record of Drillhole B2-05		1	RC	REC 99%	▽	216 215				RQD = 82%
			2	RC	REC 100%		214				RQD = 95%
			3	RC	REC 99%		213 212				RQD = 99%
211.7 4.9	END OF BOREHOLE NOTE: 1. Water level in open borehole at a depth of 0.8 m below ground surface (Elev. 215.8 m) upon completion of drilling.										

GTA-MTO 001 T:\PROJECTS\2007\07-1111-0029 (MRC, PARRY SOUND)\LOG\07-1111-0029-SHEBESHEKONG RD-PHASE II.GPJ GAL-GTA.GDT 07/09/15 DD/SAC

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

PROJECT: 07-1111-0029

RECORD OF DRILLHOLE: B2-05

SHEET 1 OF 1

LOCATION: N 5043989.5 ;E 245169.4

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE:

DRILL RIG: D-50 Turbo (Track-Mounted)

DRILLING CONTRACTOR: Walker Drilling

DATUM: Geodetic

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	

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.		FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE % 80 60 40 20	RECOVERY SOLID CORE % 80 60 40 20	R.Q.D. % 80 60 40 20	FRACT. INDEX PER 0.3 m 5 10 15 20	DISCONTINUITY DATA B Angle 0 90 180 270	DISCONTINUITY DATA DIP w.r.t. CORE AXIS 0 30 60 90	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-6} 10^{-5} 10^{-4} 10^{-3}	Diametral Point Load Index (MPa) 2 4 6	RMC -Q' AVG.	NOTES
		Continued from Record of Borehole B2-05		216.30																	
	NQ RC October 29, 2008	SYENITE GNEISS Slightly weathered to fresh, fine to medium grained, foliated, black and white, contains nepheline vains and pink eudialyte grains		0.30										BD,IR,Ro BD,IR,Ro FR,IR,Ro FR,IR-PL,Ro FR,PL,Ro							
1					1	0.3	Light brown 80 - 90							FR-JN,PL,Ro FR,PL,Ro FR,IR,Ro							
2																					
					2	0.4	Light brown 80 - 90							FR,IR-PL,Ro							
3														FR,IR,Ro FO, JN,PL,Ro-SM							
4					3	0.3	Light brown 80 - 90							FO,PL,SM-Ro							
				211.70																	
5		END OF DRILLHOLE		4.90																	
6																					
7																					
8																					
9																					
10																					

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: CR

CHECKED: CN

RECORD OF BOREHOLE No B2-06

SHEET 1 OF 1

METRIC

PROJECT 07-1111-0029

W.P. 5186-06-01

DIST ____ HWY 69

DATUM Geodetic

LOCATION N 5044022.1 ;E 245183.3

BOREHOLE TYPE NW Casing, Wash Boring

DATE November 5, 2008

ORIGINATED BY CR

COMPILED BY MWK

CHECKED BY CN

SOIL PROFILE		SAMPLES							
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
215.7	GROUND SURFACE									
0.0	TOPSOIL									
0.1	Weathered syenite gneiss (BEDROCK)		RC	-		215				
214.7 1.0	Syenite Gneiss (BEDROCK) Bedrock cored from depths of 1.0 m to 9.7 m For bedrock coring details, refer to Record of Drillhole B2-06	1	RC	REC 89%	∇	214				RQD = 51%
		2	RC	REC 100%		213				RQD = 100%
		3	RC	REC 100%		212 211				RQD = 97%
		4	RC	REC 100%		210				RQD = 99%
		5	RC	REC 99%		209 208				RQD = 100%
		6	RC	REC 100%		207				RQD = 100%
206.0 9.7	END OF BOREHOLE NOTE: 1. Water level in open borehole at a depth of 1.2 m below ground surface (Elev. 214.5 m) upon completion of drilling.					206				

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

PROJECT: 07-1111-0029

RECORD OF DRILLHOLE: B2-06

SHEET 1 OF 1

LOCATION: N 5044022.1 ;E 245183.3

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE:

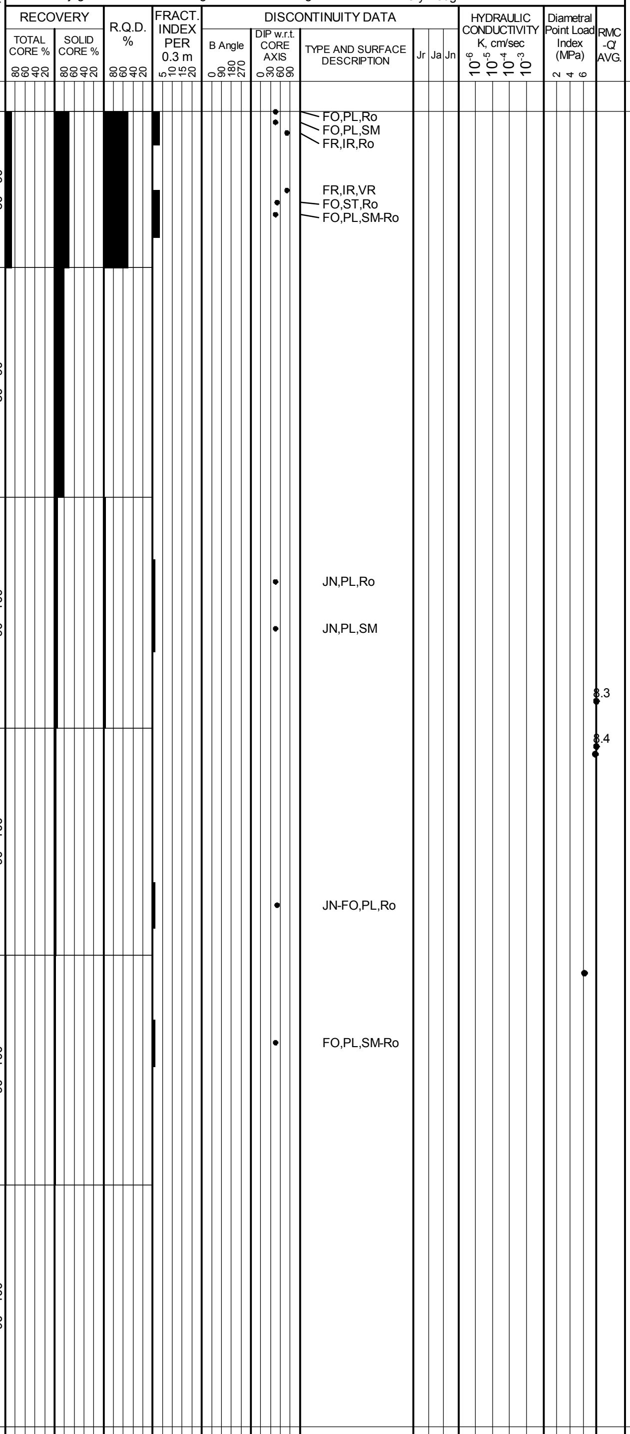
DRILL RIG: D-50 Turbo (Track-Mounted)

DRILLING CONTRACTOR: Walker Drilling

DATUM: Geodetic

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	

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.		COLOUR % RETURN / FLUSH	NOTES
		Continued from Record of Borehole B2-06		214.70				
1	NQ RC November 05, 2008	SYENITE GNEISS Moderately weathered to fresh, fine to medium crystalline, foliated, black and white, contains nepheline veins and pink eudialyte grains		1.00	1	0.4	Brown to light brown 80 - 90	
2					2	0.3	Light brown 80 - 90	
3								
4					3	0.3	Light brown 90 - 100	
5								
6					4	0.3	Light brown 90 - 100	
7					5	0.3	Light brown to grey 90 - 100	
8								
9				206.00 9.70	6	0.3	Light brown to grey 90 - 100	
10		END OF DRILLHOLE						
11								



DEPTH SCALE
1 : 50

LOGGED: CR
CHECKED: CN

Golder Associates

RECORD OF BOREHOLE No B2-07

SHEET 1 OF 1

METRIC

PROJECT 07-1111-0029
W.P. 5186-06-01
LOCATION N 5044015.7 ;E 245191.1
ORIGINATED BY CR
DIST
HWY 69
BOREHOLE TYPE NW Casing, Wash Boring
COMPILED BY MWK
DATUM Geodetic
DATE October 28, 2008
CHECKED BY MWK/CN

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L / WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
215.8	GROUND SURFACE										
0.0	TOPSOIL										
0.1	Granite Gneiss (BEDROCK) Bedrock cored from depths of 0.1 m to 3.0 m For bedrock coring details, refer to Record of Drillhole B2-07		1	RC	REC 98%	∇	215				RQD = 35%
			2	RC	REC 100%		214 213				RQD = 77%
212.8 3.0	END OF BOREHOLE NOTE: 1. Water level in open borehole at a depth of 0.6 m below ground surface (Elev. 215.2 m) upon completion of drilling.										

$+^3, \times^3$: Numbers refer to Sensitivity　　$\bigcirc^{3\%}$ STRAIN AT FAILURE

PROJECT: 07-1111-0029

RECORD OF DRILLHOLE: B2-07

SHEET 1 OF 1

LOCATION: N 5044015.7 ;E 245191.1

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE:

DRILL RIG: D-50 Turbo (Track-Mounted)

DRILLING CONTRACTOR: Walker Drilling

DATUM: Geodetic

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.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.		COLOUR / % RETURN (FLUSH)
		Continued from Record of Borehole B2-07	215.70			
	NQ RC October 28, 2008	GRANITE GNEISS Slightly weathered, fine crystalline, foliated, black and white	0.10	1	0.4	Light brown 90 - 100
1						
2				2	0.3	Light brown 80 - 90
3		END OF DRILLHOLE	212.80 3.00			
4						
5						
6						
7						
8						
9						
10						

DISCONTINUITY DATA – TYPE AND SURFACE DESCRIPTION:

JN,PL,VR
FO,PL,Ro
JN,PL,Ro
FR,PL,Ro
FO,PL,Ro
JN,PL,Ro
FR,IR,Ro
FR,PL,Ro
JN,PL-IR,Ro
FO,PL,Ro
JN,PL,Ro
FO-JN,PL,Ro
FR,PL,Ro
FR,PL,Ro
FO,PL,Ro
FO,PL,Ro

Jr	Ja	Jn	HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6} 10^{-5} 10^{-4} 10^{-3})	Diametral Point Load Index (MPa) (2 4 6)	RMC -Q' AVG.	NOTES

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: CR

CHECKED: MWK/CN

RECORD OF BOREHOLE No B2-08

SHEET 1 OF 1

METRIC

PROJECT 07-1111-0029

W.P. 5186-06-01

DIST HWY 69

DATUM Geodetic

LOCATION N 5044020.9 ;E 245188.8

BOREHOLE TYPE NW Casing, Wash Boring

DATE October 28, 2008

ORIGINATED BY CR

COMPILED BY MWK

CHECKED BY CN

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
215.9	GROUND SURFACE										
0.0	TOPSOIL										
215.6 0.3	Granite Gneiss to Syenite Gneiss (BEDROCK) Bedrock cored from depths of 0.3 m to 3.4 m For bedrock coring details, refer to Record of Drillhole B2-08		1	RC	REC 97%		215				RQD = 90%
							214				
			2	RC	REC 100%		213				RQD = 100%
212.5 3.4	END OF BOREHOLE NOTE: 1. Water level in open borehole not noted upon completion of drilling/coring.										

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

PROJECT: 07-1111-0029

RECORD OF DRILLHOLE: B2-08

SHEET 1 OF 1

LOCATION: N 5044020.9 ;E 245188.8

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE:

DRILL RIG: D-50 Turbo (Track-Mounted)

DRILLING CONTRACTOR: Walker Drilling

DATUM: Geodetic

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	

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.		COLOUR / % RETURN / FLUSH	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	NOTES
		Continued from Record of Borehole B2-08		215.60					
	NQ RC October 28, 2008	GRANITE GNEISS Slightly weathered, fine crystalline, foliated, black and white		215.60 0.30	1	0.4	Light brown 80 - 90	FO-JN,W,Ro FO,PL-W,Ro FO,PL,Ro-SM FR,IR-PL,Ro	
1 2 3		SYENITE GNEISS Slightly weathered to fresh, fine to medium grained, black and white, contains pink porphyr		214.99 0.91				FO,PL,Ro FR,IR,VR FO-JN,PL,Ro	
					2	0.4	Light brown 90 - 100	FO,PL,Ro	
4 5 6 7 8 9 10		END OF DRILLHOLE		212.50 3.40					

Jr | Ja | Jn

HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-6} 10^{-5} 10^{-4} 10^{-3}

Diametral Point Load Index (MPa) 2 4 6

RMC -Q' AVG.

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: CR

CHECKED: CN

RECORD OF BOREHOLE No B2-09

SHEET 1 OF 1

METRIC

PROJECT 07-1111-0029
W.P. 5186-06-01
DIST HWY 69
DATUM Geodetic
LOCATION N 5044026.1 ;E 245186.6
BOREHOLE TYPE NW Casing, Wash Boring
DATE October 28, 2008
ORIGINATED BY CR
COMPILED BY MWK
CHECKED BY CN

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L / WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
216.6	GROUND SURFACE										
0.0	TOPSOIL										
0.1	Granite Gneiss to Syenite Gneiss (BEDROCK) Bedrock cored from depths of 0.1 m to 3.2 m For bedrock coring details, refer to Record of Drillhole B2-09		1	RC	REC 100%		216				RQD = 85%
			2	RC	REC 100%		215 214				RQD = 97%
213.4 3.2	END OF BOREHOLE										

NOTE:

1. Water level measured in piezometer:

Date	Depth (m)	Elev (m)
05/11/08	1.0	215.6
12/04/09	1.8	214.8
10/06/09	3.1	213.5

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

PROJECT: 07-1111-0029

RECORD OF DRILLHOLE: B2-09

SHEET 1 OF 1

LOCATION: N 5044026.1 ;E 245186.6

DRILLING DATE:

DATUM: Geodetic

DRILL RIG: D-50 Turbo (Track-Mounted)

INCLINATION: -90° AZIMUTH: ---

DRILLING CONTRACTOR: Walker Drilling

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	

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.		COLOUR / % RETURN (FLUSH)	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6} 10^{-5} 10^{-4} 10^{-3})	Diametral Point Load Index (MPa) (2 4 6)	RMC -Q' AVG.	NOTES
		Continued from Record of Borehole B2-09													
	NQ RC October 28, 2008	GRANITE GNEISS Slightly weathered, fine crystalline, foliated, black and white		216.50 0.10	1	0.4	Light brown 80 - 90	FO,PL,Ro FO,PL,Ro							
1 2 3		SYENITE GNEISS Slightly weathered to fresh, fine to medium grained, black and white, contains pink porphyr		215.69 0.91				FO,PL,Ro							
					2	0.3	Light brown 80 - 90	FR,IR,VR FR,PL,VR JN-FO,PL,Ro							
4 5 6 7 8 9 10		END OF DRILLHOLE		213.40 3.20											

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: CR

CHECKED: CN

Golder Associates

RECORD OF BOREHOLE No B2-10

SHEET 1 OF 1

METRIC

PROJECT 07-1111-0029
W.P. 5186-06-01
DIST ____ HWY 69
DATUM Geodetic

LOCATION N 5044019.6 ;E 245194.2
BOREHOLE TYPE NW Casing, Wash Boring
DATE November 4, 2008

ORIGINATED BY CR
COMPILED BY MWK
CHECKED BY CN

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 | SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100 | PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30 | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)
215.4	GROUND SURFACE						
0.0 0.1	TOPSOIL Syenite Gneiss (BEDROCK) Bedrock cored from depths of 0.1 m to 9.3 m For bedrock coring details, refer to Record of Drillhole B2-10					215	
			1	RC	REC 100%	214	RQD = 72%
			2	RC	REC 98%	213	RQD = 98%
			3	RC	REC 100%	212 211	RQD = 100%
			4	RC	REC 100%	210	RQD = 100%
			5	RC	REC 100%	209 208	RQD = 100%
			6	RC	REC 100%	207	RQD = 100%
206.1 9.3	END OF BOREHOLE						

NOTE:

1. Water level measured in piezometer:

Date	Depth (m)	Elev (m)
05/11/08	1.8	213.6
12/04/09	1.7	213.7
10/06/09	1.8	213.6

GTA-MTO 001 T:\PROJECTS\2007\07-1111-0029 (MRC, PARRY SOUND)\LOG\07-1111-0029-SHEBESHEKONG RD-PHASE II.GPJ GAL-GTA.GDT 07/09/15 DD/SAC

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

PROJECT: 07-1111-0029

RECORD OF DRILLHOLE: B2-10

SHEET 1 OF 1

LOCATION: N 5044019.6 ;E 245194.2

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE:

DRILL RIG: D-50 Turbo (Track-Mounted)

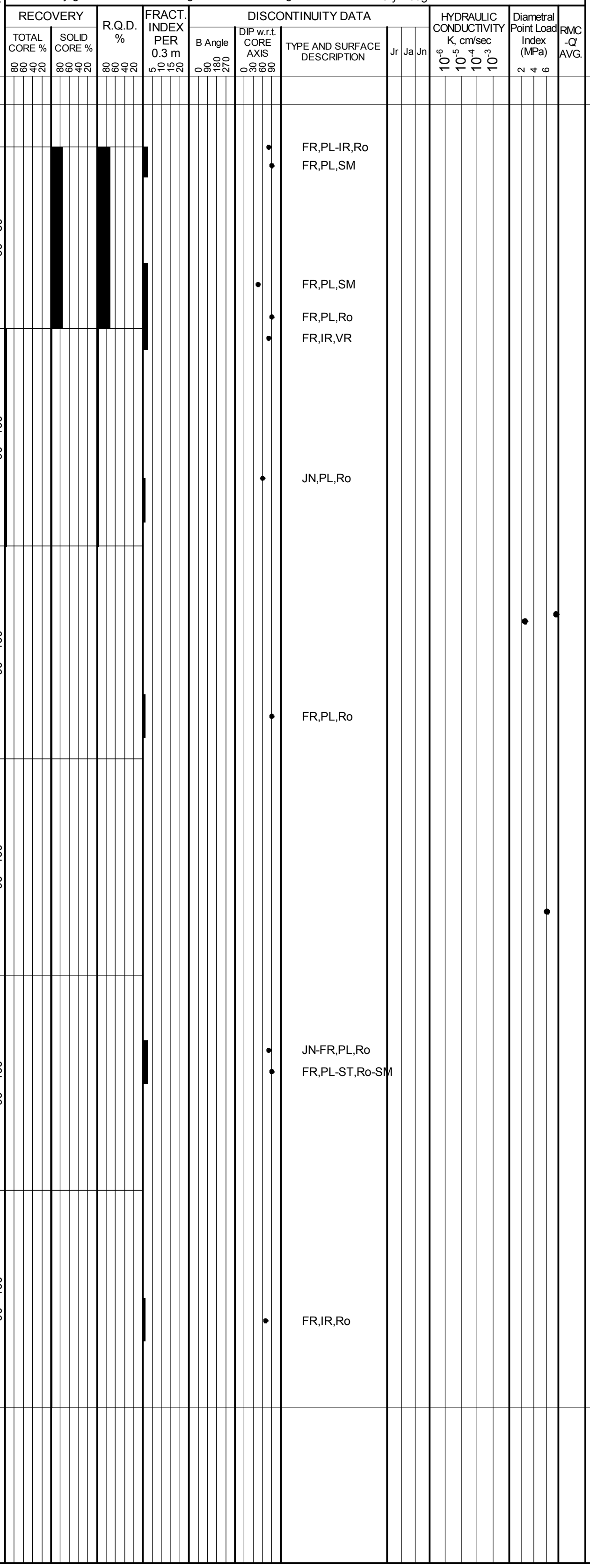
DRILLING CONTRACTOR: Walker Drilling

DATUM: Geodetic

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	

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)
		Continued from Record of Borehole B2-10	215.30
	NQ RC November 04, 2008	SYENITE GNEISS Slightly weathered, fine to medium crystalline, foliated, black and white, contains red porphyr	0.10
			206.10
		END OF DRILLHOLE	9.30

RUN No.		FLUSH COLOUR % RETURN
1	0.5	Light brown 90 - 80
2	0.3	Light brown 90 - 100
3	0.3	Light brown to brown 90 - 100
4	0.2	Light brown 90 - 100
5	0.3	Light brown 90 - 100
6	0.4	Light brown 90 - 100



FR,PL-IR,Ro
FR,PL,SM
FR,PL,SM
FR,PL,Ro
FR,IR,VR
JN,PL,Ro
FR,PL,Ro
JN-FR,PL,Ro
FR,PL-ST,Ro-SM
FR,IR,Ro

NOTES

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: CR

CHECKED: CN

RECORD OF BOREHOLE No B2-11

SHEET 1 OF 1

METRIC

PROJECT 07-1111-0029			
W.P. 5186-06-01	LOCATION N 5044052.2 ;E 245208.2		ORIGINATED BY CR
DIST	HWY 69	BOREHOLE TYPE 115 mm O.D. Solid Stem Augers and NW Casing, Wash Boring	COMPILED BY MWK
DATUM Geodetic		DATE October 30 and 31, 2008	CHECKED BY CN

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L / WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
214.6	GROUND SURFACE										
0.0	TOPSOIL										
0.1	Sandy SILT, trace clay, containing rootlets Loose to compact Brown Wet		1	SS	5	∇			○		OC = 3.7%
							214				
			2	SS	25						0 21 75 4
213.2 1.4	Silty SAND and GRAVEL, trace clay (TILL) Dense Brown Wet		3	SS	35		213		○		
212.6 2.0	Syenite gneiss (BEDROCK) Bedrock cored from depths of 2.0 m to 6.3 m For bedrock coring details, refer to Record of Drillhole B2-11		1	RC	REC 100%		212				RQD = 78%
							211				
			2	RC	REC 100%						RQD = 76%
							210				
			3	RC	REC 100%		209				RQD = 100%
208.3 6.3	END OF BOREHOLE NOTE: 1. Water level in open borehole at a depth of 0.2 m below ground surface (Elev. 214.4 m) upon completion of drilling.										

$+^3, \times^3$: Numbers refer to Sensitivity

$\bigcirc^{3\%}$ STRAIN AT FAILURE

PROJECT: 07-1111-0029

RECORD OF DRILLHOLE: B2-11

SHEET 1 OF 1

LOCATION: N 5044052.2 ;E 245208.2

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE:

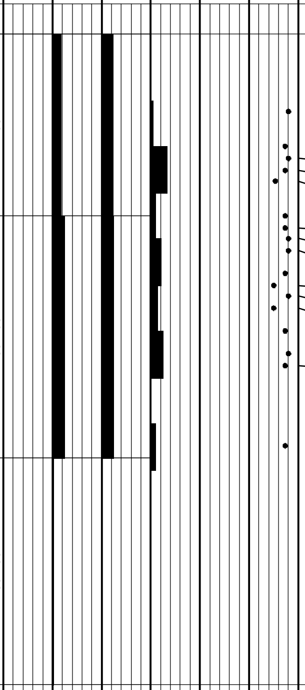
DRILL RIG: D-50 Turbo (Track-Mounted)

DRILLING CONTRACTOR: Walker Drilling

DATUM: Geodetic

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	

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	FLUSH	COLOUR / % RETURN	NOTES
		Continued from Record of Borehole B2-11	212.60				
2	NQ RC October 30 and 31, 2008	SYENITE GNEISS Slightly weathered to fresh, slightly foliated, fine to coarse crystalline, black and white, contains red porphyr	2.00	1	0.2	Light brown 90	
3–4				2	0.3	Light brown 80 - 90	
5–6			208.30 6.30	3	0.3	Light brown 80 - 90	
7							
8							
9							
10							
11							
12							



DISCONTINUITY DATA – TYPE AND SURFACE DESCRIPTION (Jr, Ja, Jn; HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-6} 10^{-5} 10^{-4} 10^{-3}; Diametral Point Load Index (MPa) 2 4 6; RMC -Q' AVG.)

- FR,IR,VR
- FR,PL,SM-Ro
- FR,IR,Ro
- FR,IR,Ro
- JN,PL,SM-Ro
- FR,PL,Ro
- FR,PL,Ro
- FR,PL,Ro
- FR,IR,Ro
- FR,IR,Ro
- FR,PL-IR,Ro
- FR,IR,VR
- VN-JN,PL,Ro-SM
- FO-JN,PL,Ro
- FR,PL,Ro
- FO-JN,PL,Ro
- FR,PL-IR,Ro

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: CR

CHECKED: CN

RECORD OF BOREHOLE No B2-12

SHEET 1 OF 1

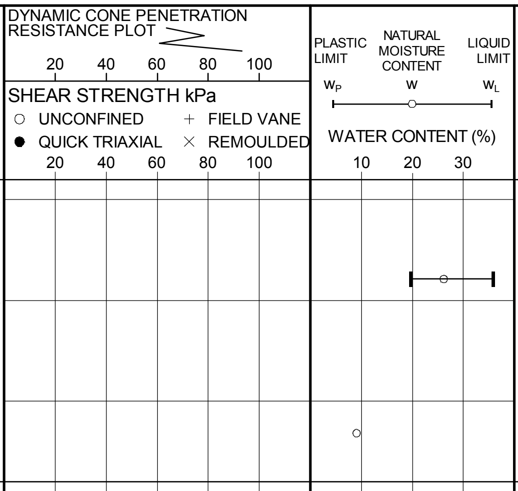
METRIC

PROJECT 07-1111-0029
W.P. 5186-06-01
LOCATION N 5044045.8 ;E 245215.9
ORIGINATED BY CR
DIST
HWY 69
BOREHOLE TYPE 115 mm O.D. Solid Stem Augers
COMPILED BY MWK
DATUM Geodetic
DATE November 4, 2008
CHECKED BY CN

SOIL PROFILE			SAMPLES					UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	kN/m³	GR	SA	SI	CL
214.2	GROUND SURFACE					▽						
0.0	TOPSOIL											
0.2	SILTY CLAY, trace gravel, containing rootlets and some sand seams Firm to very stiff Brown Wet		1	SS	4		214		OC = 6.7%			
			2A 2B	SS	26							
213.1 / 1.1	Sandy SILT, trace clay Compact Mottled grey-brown Wet						213					
			3	SS	18				0	26	71	3
212.1 / 2.1	Silty SAND and GRAVEL, trace clay, containing cobbles (TILL) Dense to very dense Brown Wet						212					
			4	SS	46							
			5	SS	25/0.10							
211.2 / 3.0	END OF BOREHOLE SPOON AND AUGER REFUSAL											

NOTE:

1. Water level in open borehole at ground surface (Elev. 214.2 m) upon completion of drilling.



$+^{3}$, $\times^{3}$: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No B2-13

SHEET 1 OF 1

METRIC

PROJECT 07-1111-0029
W.P. 5186-06-01
DIST HWY 69
DATUM Geodetic
LOCATION N 5044050.9 ;E 245213.6
BOREHOLE TYPE 115 mm O.D. Solid Stem Augers and NW Casing, Wash Boring
DATE October 31 and November 3, 2008
ORIGINATED BY CR
COMPILED BY MWK
CHECKED BY CN

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × REMOULDED
20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L
WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

SOIL PROFILE ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)
214.9	GROUND SURFACE						
0.0	TOPSOIL						
0.2	Sandy SILT, trace clay, containing silty sand seams, roots and rootlets Loose to compact Brown Moist to wet	1	SS	4	▽		
		2	SS	15		214	
213.5	Silty SAND and GRAVEL, trace clay (TILL) Compact Brown Wet	3	SS	25/0.15			
213.2 1.7	Syenite Gneiss (BEDROCK) Bedrock cored from depths of 1.7 m to 6.0 m For bedrock coring details, refer to Record of Drillhole B2-13	1	RC	REC 100%		213	RQD = 0%
		2	RC	REC 94%		212	RQD = 92%
		3	RC	REC 100%		211	RQD = 94%
		4	RC	REC 100%		210	RQD = 98%
208.9 6.0	END OF BOREHOLE NOTE: 1. Water level in open borehole at a depth of 0.6 m below ground surface (Elev. 214.3 m) upon completion of drilling.					209	

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

PROJECT: 07-1111-0029

RECORD OF DRILLHOLE: B2-13

SHEET 1 OF 1

LOCATION: N 5044050.9 ;E 245213.6

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE:

DRILL RIG: D-50 Turbo (Track-Mounted)

DRILLING CONTRACTOR: Walker Drilling

DATUM: Geodetic

JN - Joint
FLT - Fault
SH - Shear
VN - Vein
CJ - Conjugate

BD - Bedding
FO - Foliation
CO - Contact
OR - Orthogonal
CL - Cleavage

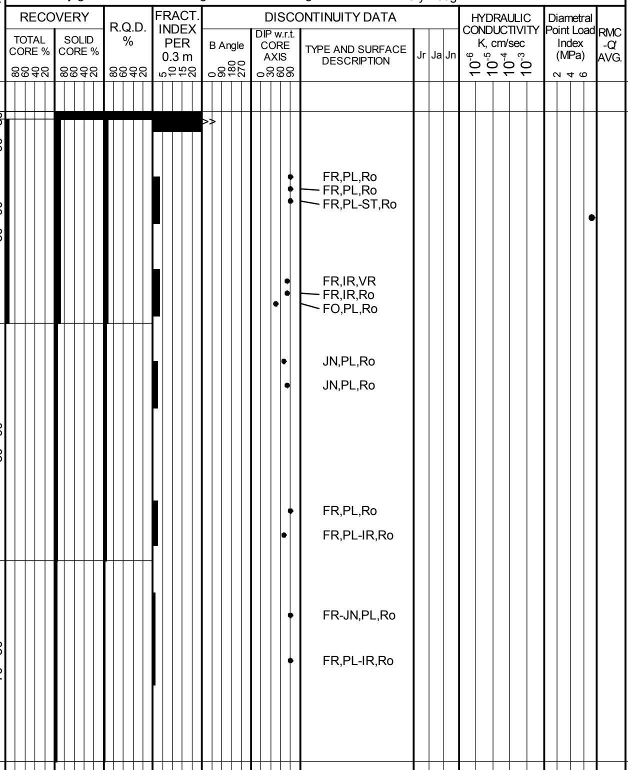
PL - Planar
CU - Curved
UN - Undulating
ST - Stepped
IR - Irregular

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.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.		COLOUR % RETURN FLUSH
		Continued from Record of Borehole B2-13	213.20			
	NQ RC October 30 and November 03, 2008	SYENITE GNEISS Slightly weathered to fresh, fine to medium crystalline, slight to strong foliation, black and white, contains pink porphyr	1.70	1	0.1	90 - 80
2				2	0.4	Light brown 80 - 90
3				3	0.3	Light brown 80 - 90
4, 5				4	0.3	Light brown 70 - 80
6		END OF DRILLHOLE	208.90 6.00			
7						
8						
9						
10						
11						



TYPE AND SURFACE DESCRIPTION:

FR,PL,Ro
FR,PL,Ro
FR,PL-ST,Ro
FR,IR,VR
FR,IR,Ro
FO,PL,Ro
JN,PL,Ro
JN,PL,Ro
FR,PL,Ro
FR,PL-IR,Ro
FR-JN,PL,Ro
FR,PL-IR,Ro

NOTES

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: CR

CHECKED: CN

Golder Associates

RECORD OF BOREHOLE No B2-14

SHEET 1 OF 1

METRIC

PROJECT 07-1111-0029
W.P. 5186-06-01
LOCATION N 5044056.2 ;E 245211.4
ORIGINATED BY CR
DIST
HWY 69
BOREHOLE TYPE Hand Excavation
COMPILED BY MWK
DATUM Geodetic
DATE October 30, 2008
CHECKED BY CN

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
215.2	GROUND SURFACE										
0.0	TOPSOIL						215				
0.2	END OF EXCAVATION (Refusal To Shovel Advance)										

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

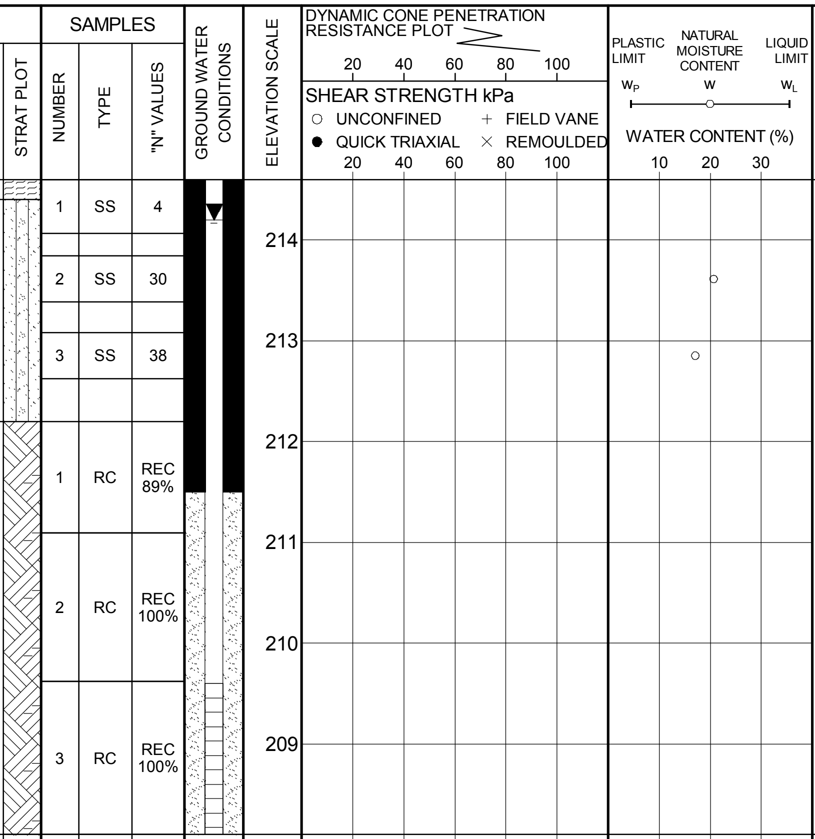
RECORD OF BOREHOLE No B2-15

SHEET 1 OF 1

METRIC

PROJECT 07-1111-0029
W.P. 5186-06-01
DIST HWY 69
DATUM Geodetic
LOCATION N 5044049.7 ;E 245219.1
BOREHOLE TYPE 115 mm O.D. Solid Stem Augers and NW Casing , Wash Boring
DATE November 3, 2008
ORIGINATED BY CR
COMPILED BY MWK
CHECKED BY CN

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
214.6	GROUND SURFACE				
0.0	TOPSOIL				
0.2	Silty SAND, trace clay Loose to dense Brown to mottled grey-brown Moist to wet	1	SS	4	
		2	SS	30	Non-Plastic
212.2		3	SS	38	
2.4	Syenite Gneiss (BEDROCK) Bedrock cored from depths of 2.4 m to 6.5 m For bedrock coring details, refer to Record of Drillhole B2-15	1	RC	REC 89%	RQD = 87%
		2	RC	REC 100%	RQD = 99%
208.1		3	RC	REC 100%	RQD = 88%
6.5	END OF BOREHOLE				



UNIT WEIGHT γ kN/m³

NOTE:

1. Water level measured in piezometer:

Date	Depth (m)	Elev (m)
05/11/08	0.4	214.2
12/04/09	0.4	214.2
10/06/09	0.4	214.2

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

PROJECT: 07-1111-0029

RECORD OF DRILLHOLE: B2-15

SHEET 1 OF 1

LOCATION: N 5044049.7 ;E 245219.1

INCLINATION: -90° AZIMUTH: ---

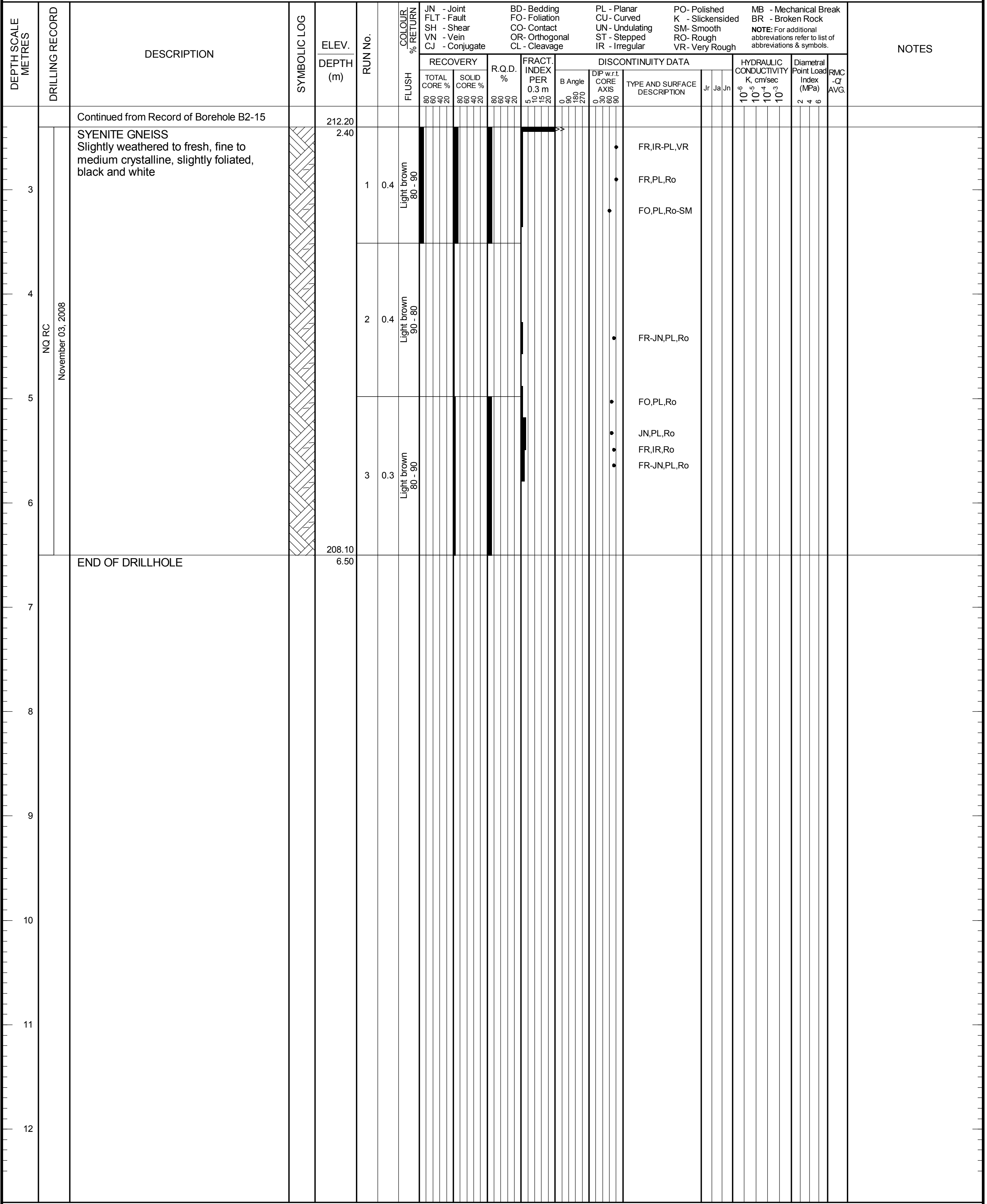
DRILLING DATE:

DRILL RIG: D-50 Turbo (Track-Mounted)

DRILLING CONTRACTOR: Walker Drilling

DATUM: Geodetic

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	



DEPTH SCALE METRES	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.		FLUSH (COLOUR / % RETURN)	TYPE AND SURFACE DESCRIPTION
	Continued from Record of Borehole B2-15	212.20				
	SYENITE GNEISS Slightly weathered to fresh, fine to medium crystalline, slightly foliated, black and white	2.40	1	0.4	Light brown 80 - 90	FR,IR-PL,VR FR,PL,Ro FO,PL,Ro-SM
			2	0.4	Light brown 90 - 80	FR-JN,PL,Ro
			3	0.3	Light brown 80 - 90	FO,PL,Ro JN,PL,Ro FR,IR,Ro FR-JN,PL,Ro
		208.10				
	END OF DRILLHOLE	6.50				

GTA-RCK 018 T:\PROJECTS\2007\07-1111-0029 (MRC, PARRY SOUND)\LOG\07-1111-0029-SHEBESHEKONG RD-PHASE II.GPJ GAL-MISS.GDT 07/09/15 DD/SAC

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: CR

CHECKED: CN

Foundation Design

RECORD OF BOREHOLE No B2-16

SHEET 1 OF 1

METRIC

PROJECT 07-1111-0029
W.P. 5186-06-01
DIST
HWY 69
DATUM Geodetic
LOCATION N 5043979.2 ;E 245154.4
BOREHOLE TYPE 115 mm O.D. Solid Stem Augers and NW Casing , Wash Boring
DATE October 30, 2008
ORIGINATED BY CR
COMPILED BY MWK
CHECKED BY CN

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L / WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						GR	SA	SI	CL
215.3	GROUND SURFACE													
0.0	TOPSOIL													
0.1	Silty SAND to SAND, some silt, trace gravel, trace clay, containing roots and rootlets to a depth of 0.6 m (TILL) Loose to dense Brown Moist to wet		1	SS	3		215							
			2	SS	34				○		7	64	27	2
214.0 1.3	Syenite gneiss (BEDROCK) Bedrock cored from depths of 1.3 m to 3.7 m For bedrock coring details, refer to Record of Drillhole B2-16		1	RC	REC 94%		214				RQD = 20%			
			2	RC	REC 99%		213 212				RQD = 100%			
211.6 3.7	END OF BOREHOLE NOTE: 1. Open borehole dry upon completion of drilling.													

$+^3, \times^3$: Numbers refer to Sensitivity

$\bigcirc^{3\%}$ STRAIN AT FAILURE

GTA-MTO 001 T:\PROJECTS\2007\07-1111-0029\07-1111-0029 (MRC, PARRY SOUND)\LOG\07-1111-0029-SHEBESHEKONG RD-PHASE II.GPJ GAL-GTA.GDT 07/09/15 DD/SAC

PROJECT: 07-1111-0029

RECORD OF DRILLHOLE: B2-16

SHEET 1 OF 1

LOCATION: N 5043979.2 ;E 245154.4

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE:

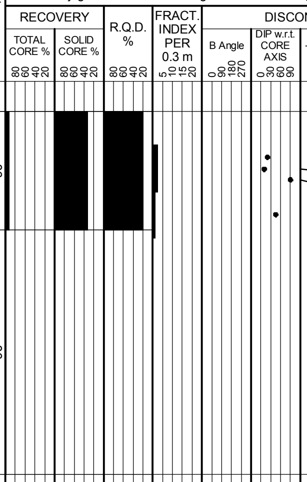
DRILL RIG: D-50 Turbo (Track-Mounted)

DRILLING CONTRACTOR: Walker Drilling

DATUM: Geodetic

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	

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.		COLOUR / % RETURN (FLUSH)	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	NOTES
		Continued from Record of Borehole B2-16	214.00					
2	NQ RC October 30, 2008	SYENITE GNEISS Slightly weathered, fine to medium grained, foliated, black and white	1.30	1	0.3	Light brown 90	FR,PL-ST,Ro FR,PL,Ro FR,IR,Ro FO,PL,SM-Ro	
3				2	0.3	Light brown 90		
			211.60					
4		END OF DRILLHOLE	3.70					
5								
6								
7								
8								
9								
10								
11								



Jr | Ja | Jn

HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-6} 10^{-5} 10^{-4} 10^{-3}

Diametral Point Load Index (MPa) 2 4 6

RMC -Q' AVG.

DEPTH SCALE

1 : 50

LOGGED: CR

CHECKED: CN

RECORD OF BOREHOLE No B2-17

SHEET 1 OF 1

METRIC

PROJECT 07-1111-0029
W.P. 5186-06-01
DIST ____ HWY 69
DATUM Geodetic
LOCATION N 5044062.5 ;E 245223.2
BOREHOLE TYPE 115 mm O.D. Solid Stem Augers and NW Casing , Wash Boring
DATE November 4, 2008
ORIGINATED BY CR
COMPILED BY MWK
CHECKED BY CN

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
214.9	GROUND SURFACE										
0.0	TOPSOIL		1	SS	2/0.10						
0.2	Syenite Gneiss (BEDROCK) Bedrock cored from depths of 1.3 m to 3.7 m For bedrock coring details, refer to Record of Drillhole B2-16		1	RC	REC 97%	▽	214				RQD = 67%
213.2 1.7	END OF BOREHOLE NOTE: 1. Water level in open borehole at a depth of 0.6 m below ground surface (Elev. 214.3 m) upon completion of drilling.										

$+^3$, $\times^3$: Numbers refer to Sensitivity

○ $^{3\%}$ STRAIN AT FAILURE

PROJECT: 07-1111-0029

RECORD OF DRILLHOLE: B2-17

SHEET 1 OF 1

LOCATION: N 5044062.5 ;E 245223.2

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE:

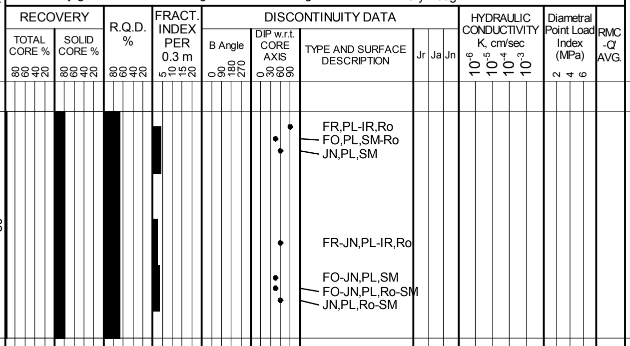
DRILL RIG: D-50 Turbo (Track-Mounted)

DRILLING CONTRACTOR: Walker Drilling

DATUM: Geodetic

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	

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.		COLOUR / % RETURN (FLUSH)	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	NOTES
		Continued from Record of Borehole B2-17		214.70					
1	NQ RC November 04, 2008	SYENITE GNEISS Slightly weathered, medium grained, foliated, black and white		0.20	1	0.3	Light brown 80	FR,PL-IR,Ro FO,PL,SM-Ro JN,PL,SM FR-JN,PL-IR,Ro FO-JN,PL,SM FO-JN,PL,Ro-SM JN,PL,Ro-SM	
				213.20 1.70					
2		END OF DRILLHOLE							
3									
4									
5									
6									
7									
8									
9									
10									



DEPTH SCALE

1 : 50

Golder Associates

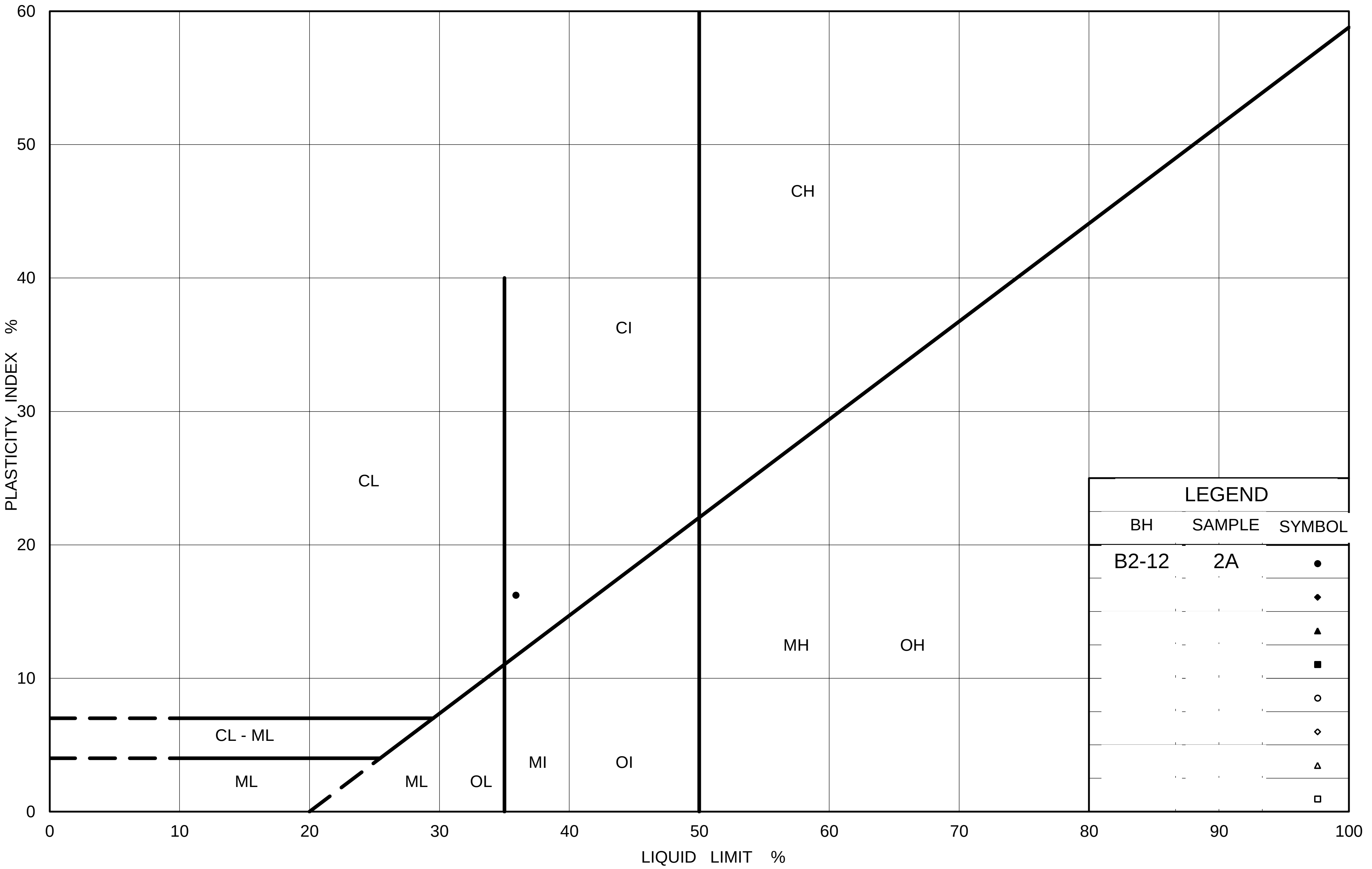
LOGGED: CR

CHECKED: CN

APPENDIX B

Laboratory Test Results

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART
Silty Clay

Figure No. B1

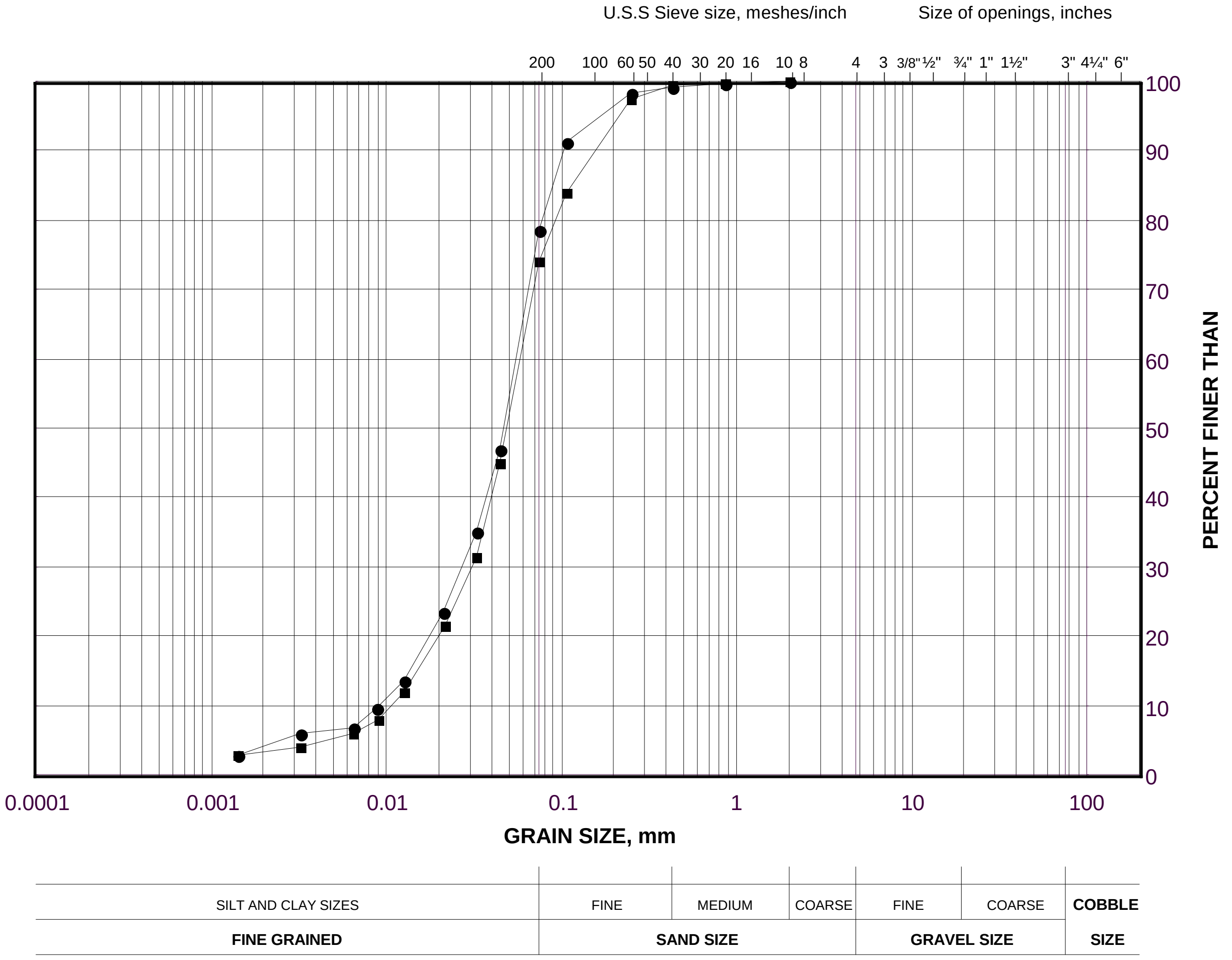
Project No. 07-1111-0029

Checked By: CN

GRAIN SIZE DISTRIBUTION

Sandy Silt

FIGURE B2



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	B2-11	2	213.5
■	B2-12	3	212.5

Project Number: 07-1111-0029

Checked By: CN

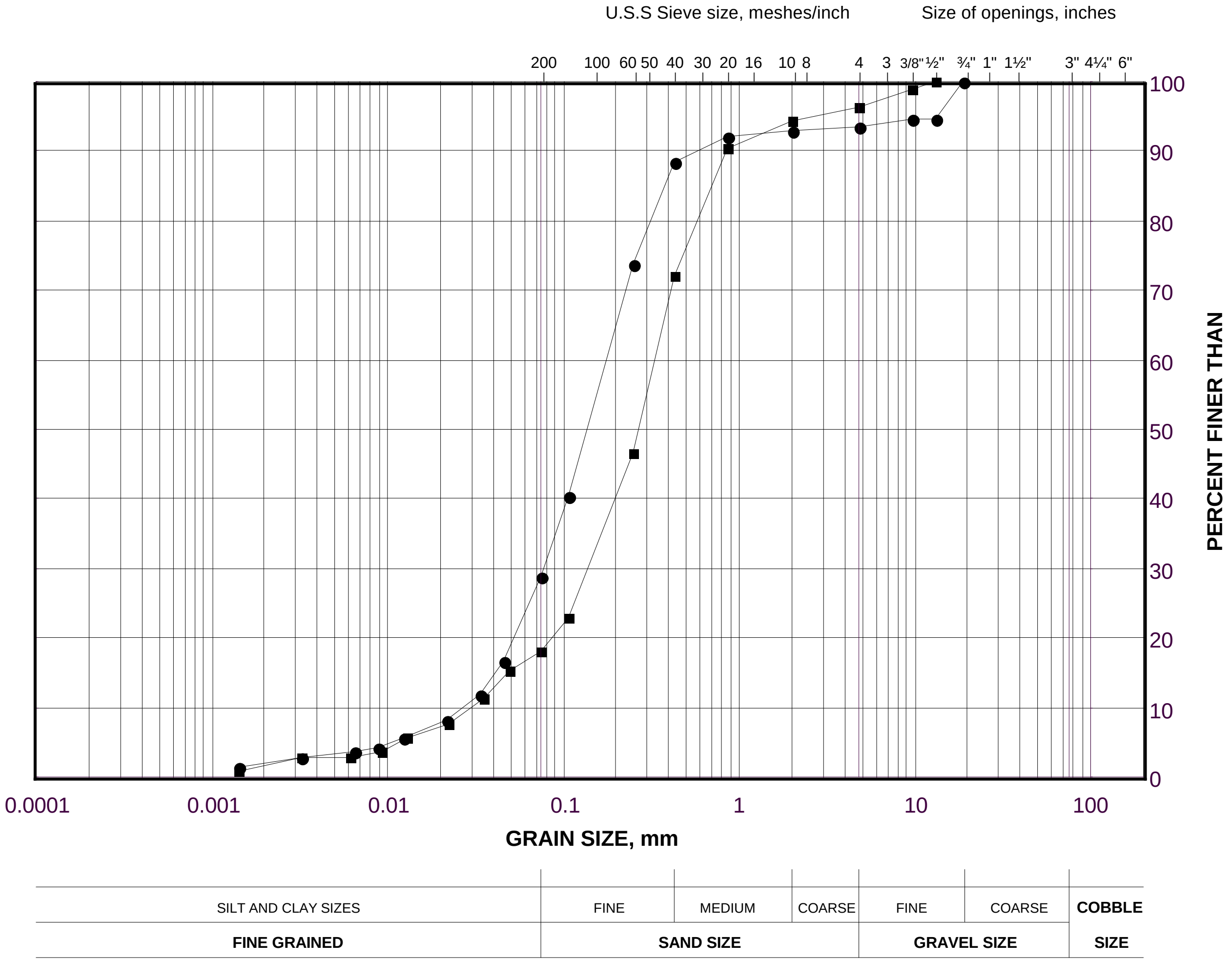
Golder Associates

Date: 30-Sep-09

GRAIN SIZE DISTRIBUTION

Silty Sand to Sand (Till)

FIGURE B3



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	B2-16	2	214.2
■	B2-01	2	214.8

Project Number: 07-1111-0029

Checked By: CN

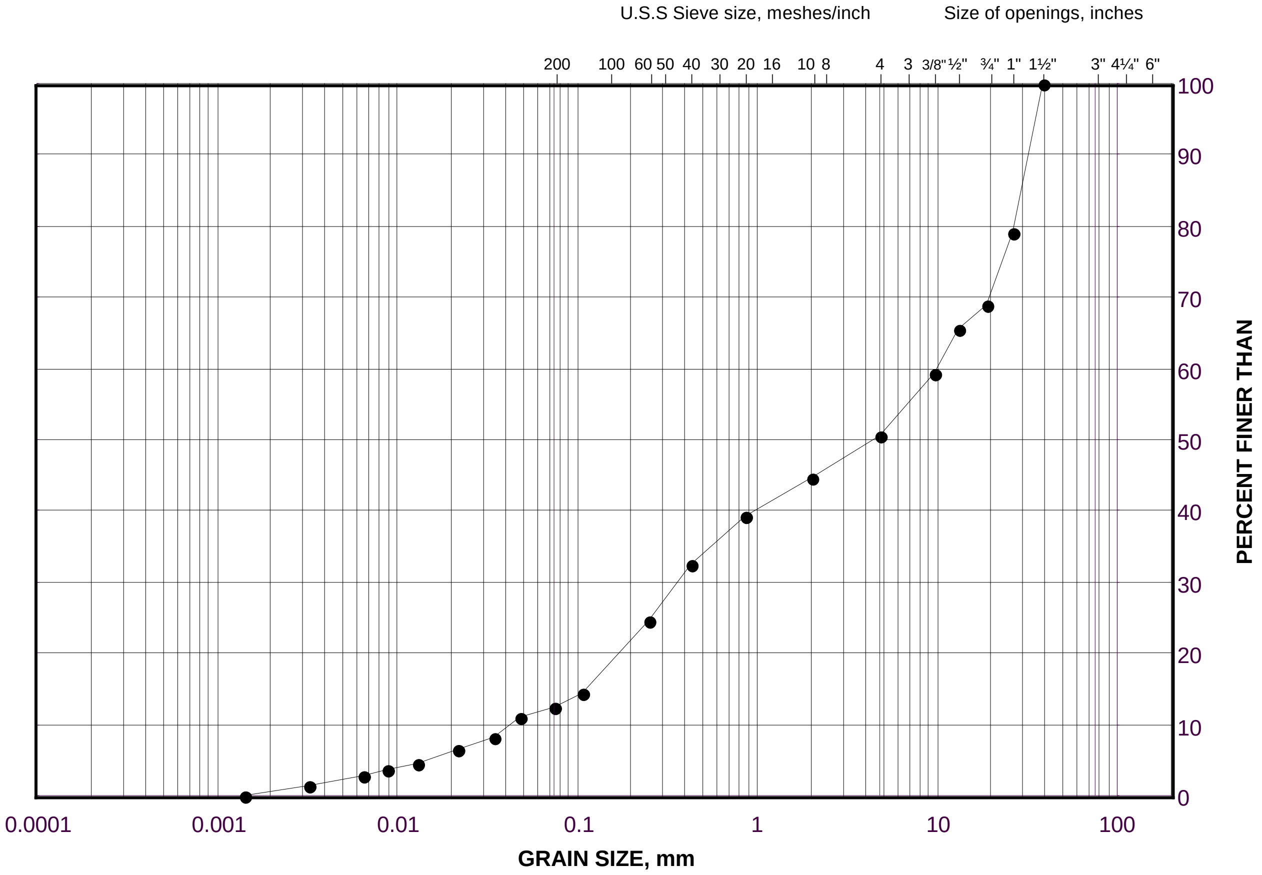
Golder Associates

Date: 30-Sep-09

GRAIN SIZE DISTRIBUTION

Sand and Gravel (Till)

FIGURE B4



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	B2-04	1B	216.0

Project Number: 07-1111-0029

Checked By: CN

Golder Associates

Date: 30-Sep-09

TABLE B1

POINT LOAD TEST ON ROCK SAMPLES

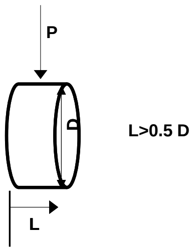
Borehole Number	Run Number	Sample Depth (m)	Sample Elevation (m)	Bedrock Description	Test Type	**Is (50mm)** (MPa)	**Approx. (1) UCS Range** (MPa)
B2-03	1	1.2	215.3	Syenite Gneiss	Axial	7.662	107
B2-03	1	1.2	215.3	Syenite Gneiss	Diametral	8.778	123
B2-06	3	4.9	210.8	Syenite Gneiss	Axial	8.279	116
B2-06	4	5.2	210.5	Syenite Gneiss	Axial	8.437	118
B2-06	4	5.2	210.5	Syenite Gneiss	Diametral	7.111	100
B2-06	5	6.7	209.0	Syenite Gneiss	Axial	6.194	87
B2-10	3	3.7	211.7	Syenite Gneiss	Axial	7.597	106
B2-10	3	3.7	211.7	Syenite Gneiss	Axial	2.582	36
B2-10	4	5.8	209.6	Syenite Gneiss	Diametral	6.109	86
B2-13	2	2.4	212.5	Syenite Gneiss	Axial	7.220	101

(1) Is_{50} x K could from ASTM D5731-08. A value of K = 14 has been estimated for this site based on the results of the UCS testing.

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)

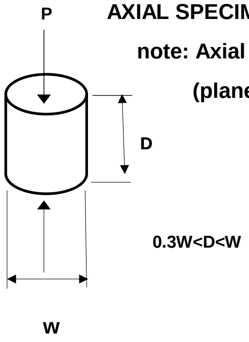


TABLE B2-1

UNCONFINED COMPRESSION (UC) TEST

ASTM D 7012-07

SAMPLE IDENTIFICATION

PROJECT NUMBER	07-1111-0029	SAMPLE NUMBER	-
BOREHOLE NUMBER	B2-03	SAMPLE DEPTH, m	1.83

TEST CONDITIONS

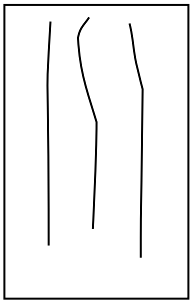
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.25

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.70	WATER CONTENT, (specimen) %	0.07
SAMPLE DIAMETER, cm	4.75	UNIT WEIGHT, kN/m^3	27.06
SAMPLE AREA, cm^2	17.72	DRY UNIT WT., kN/m^3	27.04
SAMPLE VOLUME, cm^3	189.61	SPECIFIC GRAVITY, assumed	2.70
WET WEIGHT, g	523.35	VOID RATIO	-0.02
DRY WEIGHT, g	522.98		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	95.4

REMARKS:

DATE: 12/29/2008

Checked By: MM

Golder Associates

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

SAMPLE IDENTIFICATION

PROJECT NUMBER	07-1111-0029	SAMPLE NUMBER	-
BOREHOLE NUMBER	B2-06	SAMPLE DEPTH, m	4.88

TEST CONDITIONS

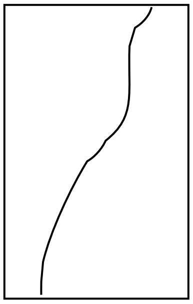
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.25

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.70	WATER CONTENT, (specimen) %	0.08
SAMPLE DIAMETER, cm	4.75	UNIT WEIGHT, kN/m^3	27.01
SAMPLE AREA, cm^2	17.72	DRY UNIT WT., kN/m^3	26.99
SAMPLE VOLUME, cm^3	189.61	SPECIFIC GRAVITY, assumed	2.70
WET WEIGHT, g	522.45	VOID RATIO	-0.02
DRY WEIGHT, g	522.03		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	120.5

REMARKS:

DATE: 12/29/2008

Checked By: MM

Golder Associates

TABLE B2-3
UNCONFINED COMPRESSION (UC) TEST
ASTM D 7012-07

SAMPLE IDENTIFICATION

PROJECT NUMBER	07-1111-0029	SAMPLE NUMBER	-
BOREHOLE NUMBER	B2-10	SAMPLE DEPTH, m	5.79

TEST CONDITIONS

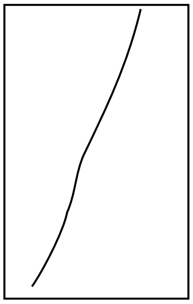
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.28

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.90	WATER CONTENT, (specimen) %	0.08
SAMPLE DIAMETER, cm	4.79	UNIT WEIGHT, kN/m^3	26.69
SAMPLE AREA, cm^2	18.02	DRY UNIT WT., kN/m^3	26.66
SAMPLE VOLUME, cm^3	196.42	SPECIFIC GRAVITY, assumed	2.80
WET WEIGHT, g	534.70	VOID RATIO	0.03
DRY WEIGHT, g	534.27		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	98.7

REMARKS:

DATE: 12/29/2008

Checked By: MM

Golder Associates

TABLE B2-4
UNCONFINED COMPRESSION (UC) TEST
ASTM D 7012-07

SAMPLE IDENTIFICATION

PROJECT NUMBER	07-1111-0029	SAMPLE NUMBER	-
BOREHOLE NUMBER	B2-13	SAMPLE DEPTH, m	2.44

TEST CONDITIONS

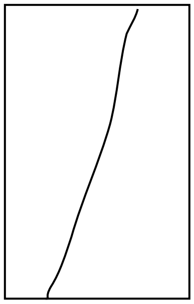
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.34

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	11.00	WATER CONTENT, (specimen) %	0.09
SAMPLE DIAMETER, cm	4.70	UNIT WEIGHT, kN/m^3	27.21
SAMPLE AREA, cm^2	17.35	DRY UNIT WT., kN/m^3	27.19
SAMPLE VOLUME, cm^3	190.84	SPECIFIC GRAVITY, assumed	2.80
WET WEIGHT, g	529.75	VOID RATIO	0.01
DRY WEIGHT, g	529.27		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	77.9

REMARKS:

DATE: 12/29/2008

Checked By: MM

Golder Associates

APPENDIX C

Non-Standard Special Provisions

DOWELS INTO ROCK - Item No.

Special Provision

1.0 SCOPE

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

2.0 MATERIALS AND INSTALLATION

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

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

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

3.0 ROCK DOWEL TESTING

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

4.0 PERFORMANCE TESTS

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

Bridge	Foundation	Number of Dowels for Performance Testing
Highway 69 / Shebeshekong Road Underpass	West Abutment	2
Highway 69 / Shebeshekong Road Underpass	Centre Pier	2
Highway 69 / Shebeshekong Road Underpass	East Abutment	2

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

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

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

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

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

Displacement measurements shall be carried out at each load increment using calibrated displacement gauges capable of measuring movements of 0.0025 cm. Measurements shall be referenced to an independent fixed referenced pint.

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

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

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

5.0 BASIS OF PAYMENT

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

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

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