



**November 4, 2015**

## **FOUNDATION INVESTIGATION AND DESIGN REPORT**

**SHEBESHEKONG ROAD NBL OVERPASS STRUCTURE, SITE NO. 44-452/C1  
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. 5184-06-01 (Phase 2 of G.W.P. 5402-05-00)**

**Submitted to:**

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**REPORT**

**GEOCRES No: 41H-154**

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## FOUNDATION REPORT – SHEBESHEKONG ROAD NBL OVERPASS STRUCTURE – HIGHWAY 69 G.W.P. 5111-07-00

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# **PART A**

## **FOUNDATION INVESTIGATION REPORT**

### **SHEBESHEKONG ROAD NBL OVERPASS STRUCTURE**

#### **SITE NO. 44-452/C1**

**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. 5184-06-01 (Phase 2 of G.W.P. 5402-05-00)**



## **1.0 INTRODUCTION**

Golder Associates Ltd. (Golder) has been retained by McCormick Rankin (MRC), a member of MMM Group Limited on behalf of Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the proposed single-span Highway 69-Shebeshekong Road Northbound Lane (NBL) Overpass structure. The proposed work is part of the four-laning of Highway 69 from 1.0 km north of the new Highway 559 Interchange northerly to 1.5 km north of Highway 7182 (Shebeshekong Road), which involves high fill embankments and embankments over swamps, the new Woods Road and Shebeshekong Road interchanges and structures, the Shawanaga River and Site No. 9 Road structures, as well as culvert crossings. The general location of the overpass structure along the new Highway 69 alignment is shown on Drawing 1.

The Terms of Reference and the Scope of Work for the foundation engineering services are outlined in MTO's Request for Proposal, dated July 2006. Golder's original proposal for foundation engineering services associated with this section of four-laning of Highway 69 is contained in Section 6.8 of MRC's Technical Proposal for this assignment. Golder's scope of work for the Shebeshekong Road NBL Overpass is contained in Addendum No. 7, dated February 14, 2013. The work has been carried out in accordance with Golder's Supplementary Specialty Plan for this project, dated July 4, 2007. The General Arrangement (GA) Drawing for the proposed Shebeshekong Road NBL Overpass Structure was provided to Golder by MRC on November 10, 2014.

This report addresses the investigation carried out for the Shebeshekong Road NBL Overpass rigid frame structure and the associated approach embankments only. Separate reports address the foundation investigations for the swamp crossings, high fill areas associated with interchange ramps and roadways, culverts and bridge structures for the project.

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

## **2.0 SITE DESCRIPTION**

The proposed Shebeshekong Road NBL Overpass is located approximately 150 m southwest of the existing intersection of Highway 69 and Shebeshekong Road and approximately 550 m south of Shawanaga River. The proposed new Highway 69 alignment runs generally in a southeast-northwest direction on the west side of the existing Highway 69, which will become part of the future Shawanaga River Service Road (Site No. 9 Road) in this area. For the purposes of this report the NBL Overpass structure is considered oriented North-South for the ease of reference.

In general, the topography in the area of the overall project limits consists of rolling terrain including densely treed areas and numerous bedrock outcrops separated by low-lying swamps and rivers. The proposed overpass structure and associated approach embankments are to be situated on a relatively flat, densely treed area with bedrock outcrops. The existing ground surface within the limits of the proposed structure and approach embankments, as encountered at borehole and test pit locations advanced for the foundations investigation, varies between Elevations 207.7 m and 203.7 m, referenced to Geodetic datum.



### **3.0 INVESTIGATION PROCEDURES**

The fieldwork for the Shebeshekong Road NBL Overpass Structure subsurface investigation was carried out between January 15 and 21, 2015 during which time a total of four (4) boreholes and four (4) test pits were advanced: two (2) test pits (Test Pits B7-01 and B7-02) and one (1) borehole (Borehole B7-03) were advanced on the south side of the structure; two (2) test pits (Test Pits B7-05 and B7-06) and one (1) borehole (Borehole B7-04) were advanced on the north side of the structure; and one (1) borehole was advanced at each of the south and north approach embankments (Boreholes B7-07 and B7-08, respectively). The Record of Borehole/Drillhole sheets, Field Test Pit Logs and the results of the laboratory testing are presented in Appendices A and B, respectively. The locations of the boreholes are shown in plan on Drawing 2.

The field investigation was carried out using a modified rubber tire backhoe-loader equipped with a CME 550 drill rig and 135D John Deere excavator supplied and operated by Landcore Drilling of Sudbury, Ontario.

The boreholes were advanced through the overburden using 108 mm inside diameter hollow-stem augers and/or NW casing. In general, soil samples were obtained at intervals of depth of about 0.75 m, using a 50 mm O.D. split-spoon sampler operated by an automatic hammer on the drill rig, performed in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586). Bedrock coring was carried out using an 'NQ' core barrel in two of the boreholes. All open boreholes were backfilled with soil cuttings and/or bentonite upon completion in accordance with Ontario Regulation 903-Wells (as amended).

The boreholes at the location of the foundation elements and approach embankments were advanced to auger, or split-spoon sampler refusal (i.e. inferred bedrock) and bedrock was confirmed by coring. Test pits located at the foundation elements were advanced to bedrock. The boreholes and test pits were advanced to depths ranging from 0.3 m to 6.8 m below existing ground surface, including coring of bedrock for core lengths of about 5.5 m and 5.0 m in Boreholes B7-03 and B7-04, respectively. The groundwater conditions and water levels in the open boreholes were observed during the drilling and test pitting operations and are described on the Record of Borehole sheets in Appendix A.

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

The as-drilled borehole locations and the ground surface elevations for Test Pits/Boreholes B7-01 to B7-06 were surveyed by Callon Dietz and Boreholes B7-07 and B7-08 were surveyed by a member of our technical staff, referenced to the test pit/borehole locations surveyed by Callon Dietz. The test pit and borehole locations given in the Record of Borehole/Drillhole sheets and Field Test Pit Logs and shown on Drawing 2 are positioned relative to MTM NAD83 northing and easting coordinates and the ground surface elevations are referenced to Geodetic datum and are summarized below.



Borehole / Test Pit	Location (MTM NAD83)		Ground Surface Elevation (m)	Depth of Borehole (m)
	Northing	Easting		
B7-01	5045465.0	243894.0	207.0	1.8
B7-02	5045470.7	243899.5	205.8	1.9
B7-03	5045476.8	243905.5	205.5	6.1
B7-04	5045469.4	243889.4	206.4	6.8
B7-05	5045475.1	243894.9	205.2	1.7
B7-06	5045481.2	243900.7	204.1	0.3
B7-07	5045465.8	243904.6	207.7	1.2
B7-08	5045480.1	243889.7	203.7	0.7

## 4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

### 4.1 Regional Geology

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

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

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

### 4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions encountered in the test pits and boreholes advanced for this investigation, together with the results of the laboratory tests carried out on selected soil and bedrock core samples, are presented in the Field Test Pit Logs and Record of Boreholes sheets provided in Appendix A. The results of the laboratory tests as well as photographs of the recovered rock core samples are also provided in Appendix B. The results of the in situ field tests (i.e. SPT 'N'-values) as presented on the Record of Borehole sheets and in Section 4.3 and 4.4 are uncorrected. The stratigraphic boundaries shown on the Record of Borehole 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

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

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



exact planes of geological change. Further, subsurface conditions will vary between and beyond the test pit and borehole locations. It should be noted that the interpreted stratigraphy shown on Drawing 2 is a simplification of the subsurface conditions.

In general, the subsurface conditions at the Shebeshekong Road NBL Overpass site consist of a surficial layer of topsoil or fill underlain by a silt to sand non-cohesive deposit, a sand and gravel layer and bedrock. A detailed description of the subsurface conditions encountered in the boreholes and test pits advanced at the footing locations and approach areas is provided in the following sections.

### **4.3 South Footing and Approach Embankment**

A total of two test pits (Test Pits B7-01 and B7-02) and one borehole (Borehole B7-03) were advanced at the location of the south footing and one borehole (Borehole B7-07) was advanced on the centerline at the south approach. In general, the subsurface conditions consist of topsoil, underlain by a deposit of silt to sand, sand and gravel at one location, and bedrock.

#### **4.3.1 Topsoil**

A layer of topsoil 0.2 m to 0.6 m thick was encountered at ground surface in Boreholes B7-01 to B7-03.

A SPT 'N'-value measured within the topsoil is 5 blows per 0.3 m of penetration, indicating a loose relative density.

The natural water content measured on two (2) samples of the topsoil is about 81 per cent and 109 per cent.

#### **4.3.2 Silt to Sand**

A deposit of silt to silty sand to sand was encountered below the topsoil in Test Pits B7-01 and B7-02 and at ground surface in Borehole B7-07. The top of this deposit was encountered at Elevations between 207.7 m and 205.5 m and its thickness ranges from 0.9 m to 1.6 m.

An SPT 'N'-value measured within this deposit is 7 blows per 0.3 m of penetration, indicating a loose relative density; however, an SPT 'N'-value of 65 blows per 0.3 m of penetration was measured at the interface with inferred bedrock in Borehole B7-07.

The natural water content measured on six (6) samples of this deposit range between about 8 per cent and 43 per cent, with typical values ranging between about 20 per cent and 27 per cent.

The result of one (1) grain size distribution test completed on a sample of silty sand and one (1) grain size distribution test completed on a sample of silt are shown in Figures B1 and B2 in Appendix B, respectively.

#### **4.3.3 Sand and Gravel with Cobbles and Boulders**

A 0.7 m thick deposit of sand and gravel with cobbles and boulders was encountered below the silt to sand deposit in Test Pit B7-02 at Elevation 204.6 m. The cobbles and boulders encountered within this deposit were observed to be up to about 0.4 m in size.



#### **4.3.4 Bedrock/Refusal**

Bedrock was encountered and core samples were recovered below the topsoil deposit in Borehole B7-03. Photographs of the recovered rock samples are shown on Figure B3 in Appendix B. The presence of bedrock was inferred by split-spoon sampler refusal in Borehole B7-07 and by exposing bedrock in Test Pits B7-01 and B7-02.

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

<b>Foundation Element / Approach Area</b>	<b>Borehole No.</b>	<b>Depth to Bedrock Surface (m)</b>	<b>Bedrock Surface Elevation (m)</b>	<b>Refusal Type</b>
South Approach	B7-07	1.2	206.5	Split-Spoon
South Footing	B7-01	1.8	205.2	Test Pitting
	B7-02	1.9	203.9	Test Pitting
	B7-03	0.6	204.9	Bedrock Cored

Based on the bedrock core samples, the bedrock consists of granite gneiss. In general the bedrock samples are described as fresh to slightly weathered, fine to coarse grained, foliated, non-porous and grey, white and pink. The Rock Quality Designation (RQD) measured on the core samples are between 65 per cent and 100 per cent, indicating a rock mass of fair to excellent quality, according to Table 3.10 in CFEM (2006)<sup>3</sup>. The Total Core Recovery (TCR) and Solid Core Recovery (SCR) of the core samples is 100 per cent and between 67 per cent and 100 per cent, respectively.

Point load index tests were carried out in accordance to ASTM D5731 on selected samples of the granite gneiss bedrock core and the point load strength index values are shown on the Record of Drillhole sheets in Appendix A and are presented in Table B1 in Appendix B. The axial tests carried out on four (4) core samples of granite gneiss bedrock measured  $Is_{50}$  values ranging from about 6.0 MPa to 11.8 MPa. The diametral tests carried out on three (3) core samples of granite gneiss bedrock measured  $Is_{50}$  values of about 0.3 MPa and 2.7 MPa.

One (1) Unconfined Compression (UC) test was carried out in accordance to ASTM D7102, on a selected core sample of the granite gneiss bedrock from Borehole B7-03 and measured a compressive strength of about 56 MPa, as detailed in Table B2 in Appendix B.

According to Table 3.5 in CFEM (2006), the granite gneiss bedrock is classified as strong ( $R4$ ,  $50 \text{ MPa} < \text{UCS} < 100 \text{ MPa}$ ) to extremely strong ( $R6$ , Point Load Index  $> 10 \text{ MPa}$ ).

#### **4.3.5 Groundwater Conditions**

In general, the test pits and boreholes were dry upon completion of drilling and the overburden samples recovered in the boreholes were moist. The water level in Borehole B7-07 upon completion of drilling operations was measured at the base of the borehole (i.e. at refusal/inferred bedrock) at a depth of 1.2 m below ground surface, corresponding to Elevation 206.5 m. The groundwater level in the area is subject to seasonal fluctuations, snow melt and variation due to precipitation events.

<sup>3</sup>Canadian Geotechnical Society, 2006. Canadian Foundation Engineering Manual, 4th Edition.





## **4.4 North Footing and Approach Embankment**

A total of two test pits (Test Pits B7-05 and B7-06) and one borehole (Borehole B7-04) were advanced at the location of the north footing and one borehole (Borehole B7-08) was advanced on the centerline at the north approach. In general, the subsurface conditions consist of topsoil, underlain by a deposit of silt to silty sand underlain by bedrock.

### **4.4.1 Topsoil**

A 0.2 m to 0.3 m thick layer of topsoil was encountered at the ground surface in Test Pits B7-05, B7-06 and in Borehole B7-04.

The natural water content measured on one (1) sample of the topsoil is about 36 per cent.

### **4.4.2 Silt to Silty Sand**

A deposit of silt to sandy silt to silt and sand to silty sand was encountered below the topsoil layer in Test Pits B7-05 and B7-06 and Borehole B7-04 and at the ground surface in Borehole B7-08. The top of this deposit was encountered between Elevations 206.1 m and 203.7 m and the thickness of the deposit ranges from 0.1 m to 1.5 m.

The SPT 'N'-values measured within the silt to silty sand deposit are 4 blows and 5 blows per 0.3 m of penetration, indicating a loose relative density; however an SPT 'N'-value of 41 blows per 0.3 m of penetration was measured at the interface with bedrock in Borehole B7-04.

The natural water content measured on seven (7) samples of this deposit range between about 19 per cent and 52 per cent, with typical values ranging between 19 per cent and 34 per cent.

The result of two (2) grain size distribution test completed on samples of the silt and sandy silt portions of the deposit are shown in Figure B2 in Appendix B.

### **4.4.3 Bedrock/Refusal**

Bedrock was encountered and core samples were recovered below the sandy silt in Borehole B7-04. Photographs of the recovered rock samples are shown on Figure B3 in Appendix B. The presence of bedrock was inferred by refusal to auger advancement in Borehole B7-08 and by bedrock exposure by excavation in Test Pits B7-05 and B7-06.

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

<b>Foundation Element / Approach Area</b>	<b>Borehole No.</b>	<b>Depth to Bedrock Surface (m)</b>	<b>Bedrock Surface Elevation (m)</b>	<b>Refusal Type</b>
North Approach	B7-08	0.7	203.0	Auger
North Footing	B7-04	1.8	204.6	Bedrock Cored Test Pitting Test Pitting
	B7-05	1.7	203.5	
	B7-06	0.3	203.8	



Based on the bedrock core samples, the bedrock consists of granite gneiss. In general the bedrock samples are described as fresh to slightly weathered, fine to coarse grained, foliated, non-porous, dark grey, white and pink. The Rock Quality Designation (RQD) measured on the core samples is 100 per cent, indicating a rock mass of excellent quality, according to Table 3.10 in CFEM (2006). The Total Core Recovery (TCR) and Solid Core Recovery (SCR) of the core samples is 100 per cent and between 94 per cent and 100 per cent, respectively.

Point load index tests were carried out in accordance to ASTM D5731 on selected samples of the granite gneiss bedrock core and the point load strength index values are shown on the Record of Drillhole sheets in Appendix A and are presented in Table B1 in Appendix B. The axial tests carried out on four (4) core samples of granite gneiss bedrock measured  $Is_{50}$  values ranging from about 3.7 MPa to 9.7 MPa. The diametral tests carried out on three (3) core samples of granite gneiss bedrock measured  $Is_{50}$  values between about 2.2 MPa and 3.7 MPa.

According to Table 3.5 in CFEM (2006), the granite gneiss bedrock is classified as strong (R4, 2 MPa < Point Load Index < 4 MPa) to very strong (R5, 4 MPa < Point Load Index < 10 MPa).

#### **4.4.4 Groundwater Conditions**

In general, test pits and boreholes were dry upon completion of drilling and the overburden samples recovered in the boreholes were moist. The perched water level in Borehole B7-04 upon completion of drilling operations was measured below the bedrock surface at a depth of 3.7 m below ground surface, corresponding to Elevation 202.7 m. The groundwater level in the area is subject to seasonal fluctuations, snow melt and variation due to precipitation events and in Borehole B7-04 is due to the water introduced during coring.

## **5.0 CLOSURE**

The field personnel supervising the drilling program was Mr. Indulis Dumpis, a senior technician with Golder. This report was prepared by Ms. Madison C. Kennedy, B.A.Sc. and reviewed by Mr. Christopher Ng, P.Eng., geotechnical engineer and an Associate of Golder. Mr. Jorge M. A. Costa, P.Eng., Golder's Designated MTO Contact for this project and Principal of Golder, carried out a quality control review of the report.



# FOUNDATION REPORT – SHEBESHEKONG ROAD NBL OVERPASS STRUCTURE – HIGHWAY 69 G.W.P. 5111-07-00

## Report Signature Page

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# **PART B**

**FOUNDATION DESIGN REPORT**

**SHEBESHEKONG ROAD NBL OVERPASS STRUCTURE**

**SITE NO. 44-452/C1**

**HIGHWAY 69 FOUR-LANING FROM 1.0 KM NORTH OF THE NEW  
HIGHWAY 559 INTERCHANGE NORTHERLY TO 1.5 KM NORTH OF  
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**MINISTRY OF TRANSPORTATION, ONTARIO**

**G.W.P. 5111-07-00; W.P. 5184-06-01 (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 Highway 69-Shebeshekong Road NBL Overpass structure. The recommendations are based on interpretation of the factual data obtained from the test pits and boreholes advanced during the subsurface investigation. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the design of the structure foundation and approach embankments. Where comments are made on construction, they are provided in order to highlight those aspects which could affect the design of the project. Those requiring information on the aspects of construction should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

### **6.1 General**

Golder Associates Ltd. (Golder) has been retained by McCormick Rankin (MRC), a member of MMM Group Limited on behalf of the Ministry of Transportation, Ontario (MTO) to provide recommendations on foundation aspects for the detail design of the Shebeshekong Road NBL Overpass structure as part of the four-laning of Highway 69 in the Township of Shawanaga. It is understood that the Shebeshekong Road NBL Overpass structure will consist of a 9 m long single span, rigid frame structure with walls located on the north and south sides of Shebeshekong Road.

Based on the General Arrangement (GA) Drawing provided by MRC on November 10, 2014, the Highway 69 grade for the proposed Shebeshekong Road NBL Overpass Structure will be about Elevation 210.7 m, between about 3.0 m and 5.2 m above the existing ground surface. In comparison, the proposed grade for the new Shebeshekong Road in the area of the proposed NBL Overpass is at about Elevation 204.3 m, up to about 3.4 m below the existing ground surface.

### **6.2 Foundation Options**

Given the proximity of the bedrock to the existing ground surface (i.e. between 0.3 m and 1.9 m deep), and considering the proposed grade for the Shebeshekong Road alignment, shallow foundations comprised of spread footings founded directly on bedrock is considered the preferred foundation alternative to support the NBL Overpass structure.

Due to the shallow nature of the overburden deposits at this site, pile foundations would not be practical and a significant amount of excavation/trenching into the strong to extremely strong bedrock would be required to achieve the minimum required pile lengths for deep foundation, which would likely be cost prohibitive. Similarly, short caissons are not considered suitable as the caisson would have to be socketted into the strong to extremely strong bedrock, which would be difficult to drill.

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

The following sections provide recommendations for shallow spread footings to support the structure.



## 6.3 Spread Footings

At this site, shallow foundations comprised of spread footings founded directly on bedrock is considered the preferred alternative for support of the structure.

### 6.3.1 Geotechnical Axial Resistance and Reaction

Based on the GA Drawing provided by MRC, the footings are proposed to be founded at approximately Elevation 202.5 m, which will require up to about 1.9 m of overburden and 2.7 m of bedrock excavation to construct the spread footing on a properly prepared granite gneiss bedrock surface.

The following summarizes the factored geotechnical axial resistance at Ultimate Limit States (ULS) for spread footings on properly prepared bedrock. The geotechnical reaction at Serviceability Limit States (SLS) for 25 mm of settlement will be greater than the factored geotechnical resistance at ULS at this site and as a result the SLS condition does not apply.

Foundation Option	Factored Geotechnical Axial Resistance at Ultimate Limit States (ULS)	Geotechnical Reaction at Serviceability Limit States (SLS) for 25 mm of Settlement
Spread footing on properly prepared granite gneiss bedrock	10,000 kPa	N/A <sup>1</sup>

Note: 1. The geotechnical reaction at SLS for 25 mm of settlement will be greater than the factored geotechnical resistance at ULS for spread footings on bedrock and as a result the SLS condition does not apply.

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

### 6.3.2 Resistance to Lateral Loads

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

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

The value presented above represents an unfactored value.

If necessary, the sliding resistance between the concrete footings and the bedrock can be supplemented by dowelling into the bedrock. The horizontal resistance of the dowels is dependent on the strength of the bedrock, grout and steel. For this site, where the rock mass is essentially as strong or stronger than concrete, the design of the dowels into the rock may be handled in the same way as the dowel embedment into the concrete. This





assumes that the unconfined compressive strength of the grout will be similar to that of the concrete. The dowels should have a minimum embedded length of 1 m within the sound/non-shattered bedrock, and the structural strength of the dowels and compressive strength of the grout should not be exceeded. If dowelling into bedrock is adopted at this site, a NSSP should be included in the Contract Documents to specify the installation, materials and testing of the dowels, as presented in the example NSSP included in Appendix C.

### **6.3.3 Frost Protection**

For spread footings founded directly on the properly prepared granite gneiss bedrock at this site, a minimum soil cover for frost protection is not required.

## **6.4 Seismic Site Consideration**

### **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 (2006) may be taken as 1.0, consistent with Soil Profile Type I.

### **6.4.2 Seismic Analysis Coefficient**

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

Given that the proposed Shebeshekong Road NBL Overpass structure is a single-span structure, and in accordance with Sections 4.4.5.2 of the CHBDC, seismic analysis is not required for this structure.

## **6.5 Lateral Earth Pressures**

The lateral earth pressures acting on the walls and any associated retaining (stub) walls will depend on the type and method of placement of the backfill material, 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.

The following recommendations are made concerning the design of 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 Granular 'A' or Granular 'B' Type II but with less than 5 per cent passing the No. 200 (0.075 mm) sieve should be used as backfill behind the walls. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS.PROV 501 (Compacting).



- For rigid frame structures, granular fill should be placed in a zone with the width equal to at least 1.8 m behind the back of the walls (in accordance with Figure C6.20(a) of the *Commentary* to the CHBDC for restrained walls). For the retaining walls, fill should be placed within a 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 for unrestrained walls). The pressures are based on the fill as placed and the following parameters (unfactored) may be assumed:

Fill Type	Soil Unit Weight	Coefficients of Static Lateral Earth Pressure	
		At-Rest, $K_o$	Active, $K_a$
Granular 'A'	22 kN/m <sup>3</sup>	0.43	0.27
Granular 'B' Type II	21 kN/m <sup>3</sup>	0.43	0.27
Rock Fill	19 kN/m <sup>3</sup>	0.36	0.21

If the walls allow lateral yielding, active earth pressures may be used in the geotechnical design of the structure. If the walls do not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design. The movement required to allow active pressure 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 Retaining (Stub) Walls

It is understood that retaining (stub) walls doweled into bedrock are proposed on the west end of the rigid frame structure to retain the Highway 69 proposed approach embankment fill. As outlined in Section 6.3.2, dowels should have a minimum embedded length of 1 m within the sound/non-shattered bedrock, and the structural strength of the dowels and compressive strength of the grout should not be exceeded. A NSSP should be included in the Contract Documents to specify the installation, materials and testing of the dowels, as presented in the example NSSP included in Appendix C.

It is also understood that the underside of the strip footings for the retaining (stub) wall is to be founded at the same elevation as the footings for the rigid frame structure (i.e. at Elevation 202.5 m). As such, refer to Section 6.3 for geotechnical axial resistance/reaction, resistance to lateral loads and frost protection requirements for the design of the strip footing.

## 6.7 Approach Embankment Design

Based on the investigated locations at this site, the approach embankments will be up to about 6.5 m high and founded on either the silt to sand deposits or directly on bedrock. Where present, all topsoil should be stripped from below the approach embankment areas.

Discussions on stability and settlement of the new approach embankments are presented in the following sections.



### **6.7.1 Stability**

Taking into consideration that the south approach to the NBL structure will be located in and immediately adjacent to a rock cut and the north approach embankment is to be constructed with rock fill with 1.25H:1V or flatter side slopes, stability issues of the approach embankments are not anticipated.

### **6.7.2 Settlement**

#### **6.7.2.1 Methodology**

To estimate the magnitude of the expected embankment settlements, analyses were carried out at the critical sections of the proposed approach embankments. For the approach embankments, the critical sections correspond to the greatest embankment height at about 6.5 m at the north approach.

The settlement analysis assumes that the organic soils encountered at/below ground surface have been removed.

The sources of settlement are considered to include:

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

#### **6.7.2.2 Parameter Selection**

The following summarize the simplified stratigraphy and the associated strengths and unit weights employed for the foundation soils at the approach embankment areas. The immediate compression of the silty sand/ silt and sand deposit were modelled by estimating an elastic modulus of deformation based on the SPT 'N'-values and using correlation proposed by Bowles (1984) and Kulhawy and Mayne (1990).

<b>Embankment</b>	<b>Soil Type</b>	<b>Thickness</b>	<b>Unit Weight</b>	<b>Elastic Modulus, E'</b>
South Approach Embankment	Silty Sand	Up to 1.2 m	19 kN/m <sup>3</sup>	2 MPa
North Approach Embankment	Silt and Sand	Up to 0.7 m	19 kN/m <sup>3</sup>	3.5 MPa

For the purpose of the settlement analysis, the groundwater table was conservatively assumed to be located at the existing ground surface.

#### **6.7.2.3 Settlement of Foundation Soils**

The results of the estimated settlement of the foundation soils at the approach embankments are summarized below.



<b>Embankment</b>	<b>Soil Type</b>	<b>Estimated Settlement of Foundation Soils (mm)</b>
South Approach Embankment	Silty Sand	Up to 20
North Approach Embankment	Silt and Sand	Up to 50

These settlements are expected to occur rapidly (i.e. during construction) in response to filling based on the non-cohesive and granular nature of the foundation soils.

#### **6.7.2.4 Settlement of Embankment Fill**

It is understood that rock fill is to be used for the construction of the approach embankments and as such, there will be settlement due to compression of the rock fill itself under self-weight. The magnitude of settlement of the rock fill depends on the type of rock/strength of particles, size and shape of 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). According to MTO's Guideline for Rock Fill Settlement and Rock Fill Quantity Estimates (2010), the settlement of rock fill placed in this manner is expected to be nominal and the magnitude is estimated to be up to about 0.75 per cent of the effective height of the rock fill embankment. As such, the estimated settlement of rock fill for the approach embankments is presented below.

<b>Embankment</b>	<b>Maximum New Embankment Height<sup>1</sup> (m)</b>	<b>Estimated Settlement of Rock Fill (mm)</b>
South Approach Embankment	3	20
North Approach Embankment	7	60

Note: 1. Includes additional fill required after removal organic and native soils if required.

The majority of the settlement of the rock fill is expected to occur during and within a 6-month period following construction. However, some post-construction time-dependent settlement will occur. 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 (2010), a minimum preload period of 100 days is required for the north approach.

If Granular 'A' or 'B' Type II is used for the construction of the approach embankments (compacted in accordance with OPSS.PROV 501 (Compacting)), it is anticipated that the post-construction settlement of the embankment fill will be negligible (i.e. less than 5 mm).

#### **6.7.2.5 Embankment Platform Widening**

In accordance with the requirements of MTO Northern Region Engineering Directive NRE 98-200, Northern Region Embankment Design Guidelines (1998), a minimum embankment platform widening of 2 m per side is



required for approach embankment of major highways (i.e. including Highway 69). However, given that the embankment fill is to be placed on thin native non-cohesive overburden over bedrock or directly on bedrock, a minimum platform widening is not required at the approach embankments.

## **6.8 Subgrade Preparation and Embankment Construction**

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

### **6.8.1 Removal of Organic Materials**

Prior to the placement of any fill, all surface and near surface layers of topsoil should be stripped from the plan limits of the proposed works.

### **6.8.2 Embankment Fill Placement**

Placement of rock fill material for approach embankment construction should be carried out in accordance with the requirements as outlined in the OPSS.PROV 206 (Grading). The rock fill 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 fill 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).

## **6.9 Design and Construction Considerations**

### **6.9.1 Overburden Excavation**

In order to construct the rigid frame structure foundations on the bedrock at the currently proposed footing elevations, excavations up to about 4.5 m below the existing ground surface will be required and will be made through the native non-cohesive overburden/bedrock. The overburden soils are considered Type 3 soils according to Occupational Health and Safety Act and Regulation for Construction Projects (OHSA). Excavations in the overburden soils 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 6.10.

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

### **6.9.2 Control of Groundwater and Surface Water**

The groundwater level measured in two boreholes at this site was at or below existing bedrock surface while all the test pits and other boreholes were dry upon completion of drilling. Construction of the rigid frame structure will require excavation of up to 4.5 m below existing ground surface, to the proposed founding level at Elevation 202.5 m, for both the north and south footings.

Given the thin overburden and granite gneiss bedrock of excellent quality and that the proposed rigid frame structure location is higher than the surrounding area, it is expected that pumping from within the excavations



with adequately sized and properly filtered pumps will be sufficient to control the groundwater inflow. All surface water should be directed away from the excavations.

### **6.9.3 Obstructions**

Cobbles and boulders (up to about 400 mm in diameter) were encountered directly above the bedrock at the south footing during test pitting.

Conventional excavation equipment should be suitable for the majority of the excavation through the fills and overburden soils. However, the presence of cobbles and boulders may interfere with or slow the progress of stripping and excavation. It is recommended that a Non-Standard Special Provision (NSSP) be included in the Contract Document to warn Contractor of these obstructions and to ensure that the contractor has the equipment to handle such obstructions, as presented in the example NSSP included in Appendix C.

## **6.10 Recommendations for Rock Excavation and Blasting**

### **6.10.1 Rock Excavation**

It should be noted that the bedrock at the site is classified as strong (R4) to extremely strong (R6) (i.e. estimated unconfined compressive strengths of about 56 MPa and point load index between 3.7 MPa and 11.8 MPa). As such, bedrock excavation in the vicinity of the proposed structure foundations should be carried out using line drilling and pre-shearing techniques to minimize blast damage to the rock (i.e. shattering and over-break) and provide better control over the configuration of the founding surface. The overall slope of the rock face may be formed vertically, or near vertical (i.e. about 0.25H:1V). In addition, following excavation, it will be necessary to remove all loose, shattered and/or fractured rock within the footprint of the foundations and to ensure that the founding rock is cleaned and protected such that the integrity of the rock is maintained.

### **6.10.2 Blasting**

The use of explosives should follow the specifications outlined in OPSS.PROV 120 (Use of Explosives). It is recommended that control of all blasting operations 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 footings.





## **7.0 CLOSURE**

This Foundation Design Report was prepared by Ms. Madison C. Kennedy, B.A.Sc., a member of Golder's geotechnical engineering group. The technical aspects were reviewed by Mr. Christopher Ng, P.Eng. a geotechnical engineer and an Associate of Golder. Mr. Jorge M. A. Costa, P.Eng., Golder's Designated MTO Contact for this project and Principal of Golder, carried out an independent quality control review of the report.



# FOUNDATION REPORT – SHEBESHEKONG ROAD NBL OVERPASS STRUCTURE – HIGHWAY 69 G.W.P. 5111-07-00

## Report Signature Page

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## REFERENCES

Bowles, J.E. 1984. *Physical and Geotechnical Properties of Soils*, Second Edition. McGraw Hill Book Company, New York.

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

Canadian Highway Bridge Design Code (CHBDC) and Commentary on CAN/CSA-S6-06. 2006. CSA Special Publication, S6.1-06. Canadian Standard Association.

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

Geology of Ontario. 1991. Ontario Geological Society, Special Volume 4, Part 2. Eds. P.C. Thurston, H.R. Williams, R.H. Sutcliffe and G.M. Stott. Ministry of Northern Development and Mines, Ontario.

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

### 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.

### Ministry of Transportation Ontario:

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

Embankment Settlement Criteria for Design. July 2010.

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

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 as amended by O. Reg. 443/09

### Ontario Provincial Standard Specification:

OPSS.PROV 120	General Specification for Use of Explosives.
OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 206	Construction Specification for Grading.
OPSS.PROV 1010	Material Specifications for Aggregates – Base, Subbase, Select Subgrade, and Backfill Material



## FOUNDATION REPORT – SHEBESHEKONG ROAD NBL OVERPASS STRUCTURE – HIGHWAY 69 G.W.P. 5111-07-00

Ontario Water Resources Act:

Ontario Regulation 903      Wells.



# TABLES



## FOUNDATION REPORT – SHEBESHEKONG ROAD NBL OVERPASS STRUCTURE – HIGHWAY 69 G.W.P. 5111-07-00

**TABLE 1**  
**EVALUATION OF FOUNDATION ALTERNATIVES**  
**G.W.P. 5111-07-00 / W.P. 5184-06-01**

Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Spread Footings on Bedrock	1	<ul style="list-style-type: none"> <li>■ Bedrock/foundation level present at shallow depth below ground surface</li> <li>■ Relative ease of construction.</li> <li>■ Reduced bedrock excavation (as compared with pile option).</li> <li>■ Negligible post-construction settlement.</li> </ul>	<ul style="list-style-type: none"> <li>■ None</li> </ul>	<ul style="list-style-type: none"> <li>■ Lower relative costs compared with piled foundation options.</li> </ul>	<ul style="list-style-type: none"> <li>■ Must take measures to employ controlled blasting techniques to ensure integrity of rock below the footings founding levels or repair using mass concrete may be required during construction in areas of overbreak/overshatter.</li> </ul>
H-Piles in Bedrock Trenches	NR	<ul style="list-style-type: none"> <li>■ Negligible post-construction settlement.</li> </ul>	<ul style="list-style-type: none"> <li>■ Bedrock excavation to form trench (or drilling to form sockets) will be required to achieve minimum required pile lengths.</li> <li>■ Lower axial resistance than shallow foundation as structural strength of pile govern.</li> </ul>	<ul style="list-style-type: none"> <li>■ Higher relative cost than spread footings due to additional costs for excavating trenches in bedrock.</li> </ul>	<ul style="list-style-type: none"> <li>■ Not recommended due to the additional depth of excavation required in strong to extremely strong bedrock.</li> </ul>

Note: 1. NR – Not Recommended

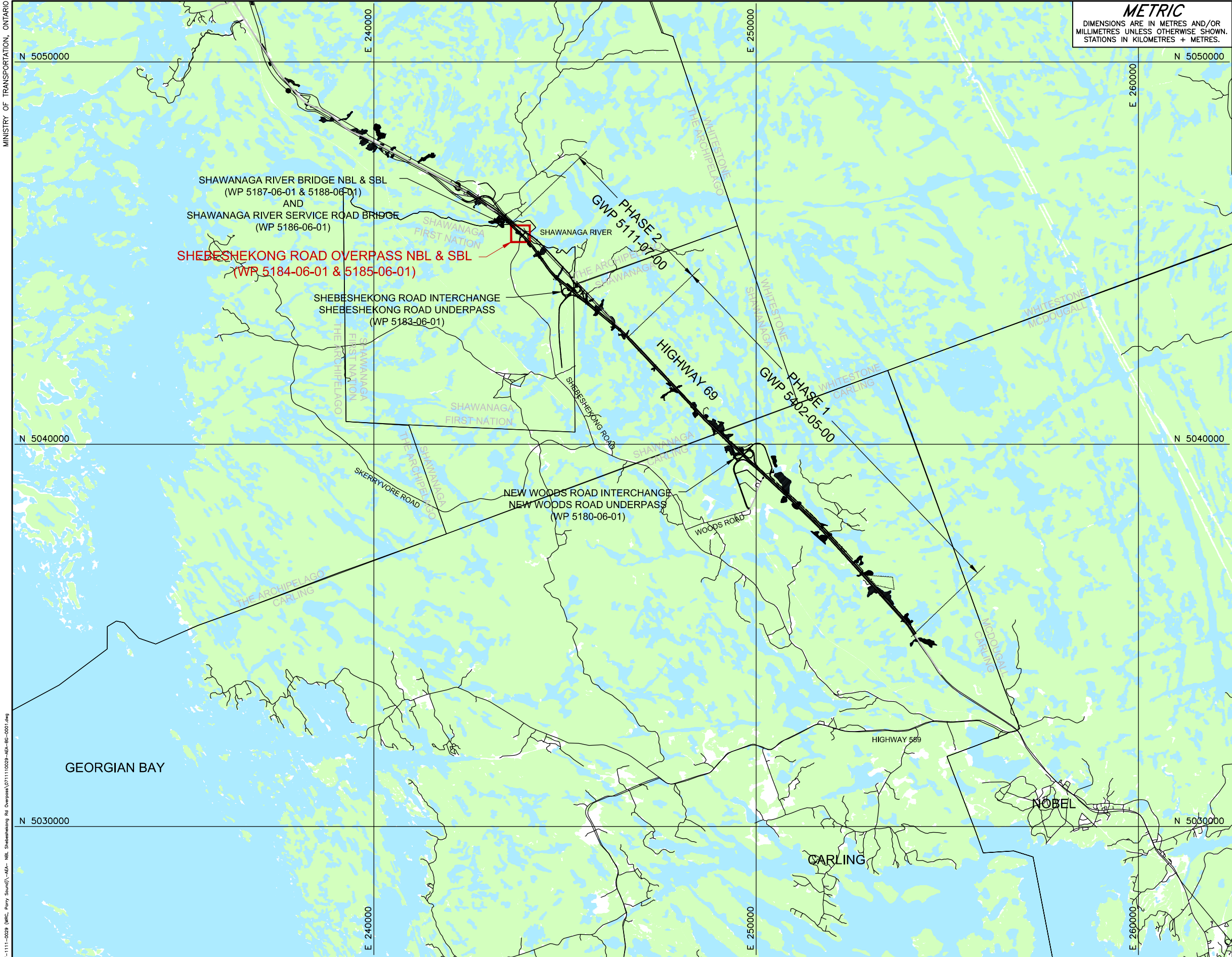
Prepared By: MCK

Reviewed By: CN/JMAC





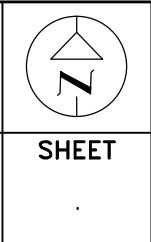
# **DRAWINGS**



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

CONT No. .  
WP No. 5184-06-01

HIGHWAY 69  
SITE LOCATION PLAN



KEY PLAN  
NOT TO SCALE

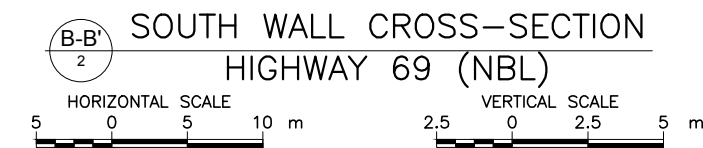
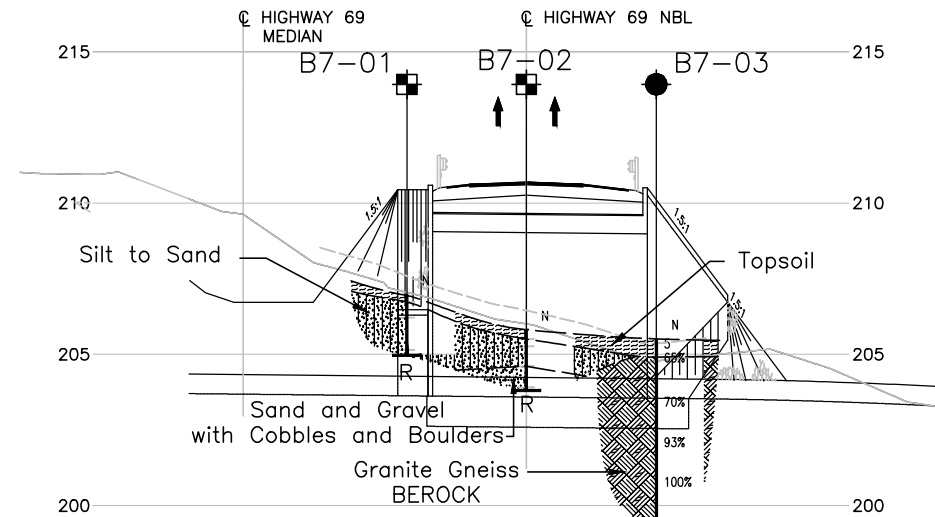
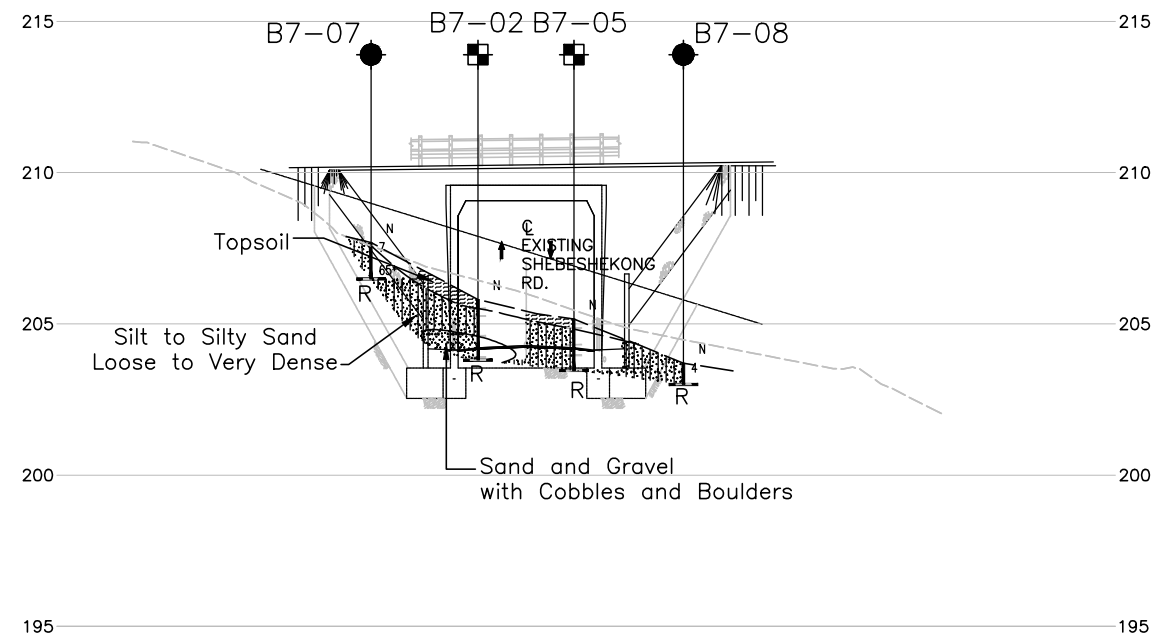
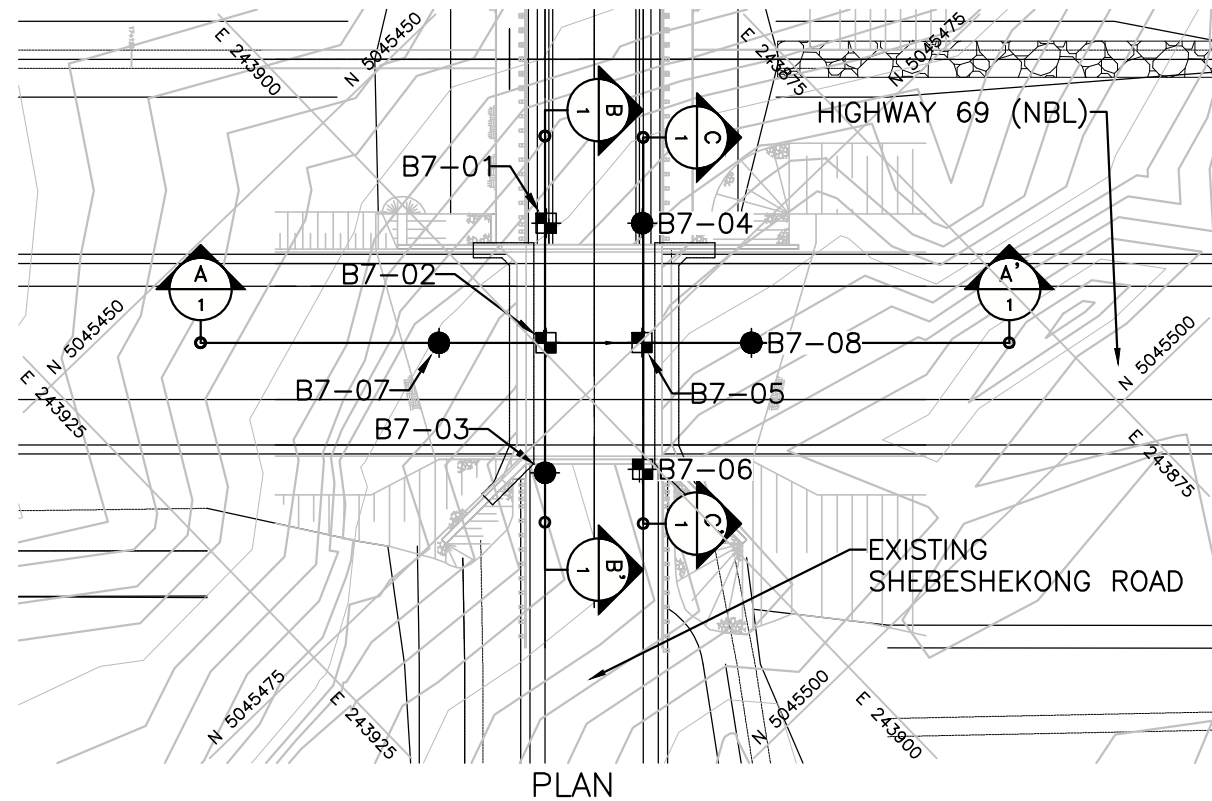


**REFERENCE**  
Base Data — MNR NRVIS, obtained 2004, CANMAP v2006.4  
Produced by Golder Associates Ltd under licence from  
Ontario Ministry of Natural Resources, ©Queens Printer 2008  
Datum : NAD 83 Projection : MTM Zone 10



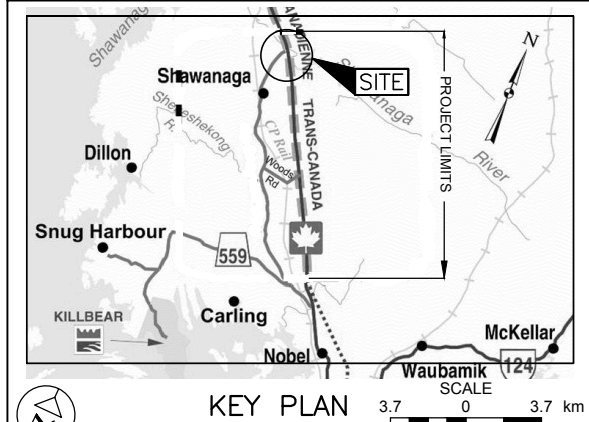
NO.	DATE	BY	REVISION
Geocres No. 41H-154			
HWY. 69		PROJECT NO. 07-1111-0029	DIST. .
SUBM'D. MCK	CHKD. CN	DATE: Nov. 2015	SITE:44-452/C1
DRAWN: JFC	CHKD. .	APPD. JMAC	DWG. 1





CONT No.  
WP No.5184-06-01

HIGHWAY 69  
SHEBESHEKONG ROAD NBL OVERPASS STRUCTURE  
BOREHOLE LOCATIONS AND  
SOIL STRATA



- Borehole - Current Investigation
- Test Pit
- 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
- R Refusal

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
B7-01	207.0	5045465.0	243894.0
B7-02	205.8	5045470.7	243899.5
B7-03	205.5	5045476.8	243905.5
B7-04	206.4	5045469.4	243889.4
B7-05	205.2	5045475.1	243894.9
B7-06	204.1	5045481.2	243900.7
B7-07	207.7	5045465.8	243904.6
B7-08	203.7	5045480.1	243889.7

## NOTES

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

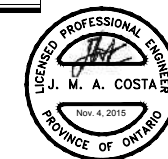
The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

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

## REFERENCE

Base plans provided in digital format by MMM, drawing file no. S6878-330-0015GA.dwg, dated November 2013, h6878\_PHASE2\_XD1 grading.dwg and h6878\_PHASE2\_XN1.dwg, received November 17, 2014

NO.	DATE	BY	REVISION
1	Nov. 4, 2015	JMAC	Geocres No. 41H-154
2	Nov. 4, 2015	JMAC	PROJECT NO. 07-1111-0029
3	Nov. 4, 2015	JMAC	DIST. .
4	Nov. 4, 2015	JMAC	HWY. 69
5	Nov. 4, 2015	JMAC	SUBM'D. MCK
6	Nov. 4, 2015	JMAC	CHKD. CN
7	Nov. 4, 2015	JMAC	DATE: Nov. 2015
8	Nov. 4, 2015	JMAC	SITE:44-452/C1
9	Nov. 4, 2015	JMAC	DRAWN: MR/JFC
10	Nov. 4, 2015	JMAC	CHKD. .
11	Nov. 4, 2015	JMAC	APPD. JMAC
12	Nov. 4, 2015	JMAC	DWG. 2





# APPENDIX A

## FIELD TEST PIT LOGS AND RECORD OF BOREHOLES AND DRILLHOLES



## LIST OF SYMBOLS

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

### I. GENERAL

$\pi$	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

$\gamma$	shear strain
$\Delta$	change in, e.g. in stress: $\Delta \sigma$
$\varepsilon$	linear strain
$\varepsilon_v$	volumetric strain
$\eta$	coefficient of viscosity
$\nu$	Poisson's ratio
$\sigma$	total stress
$\sigma'$	effective stress ( $\sigma' = \sigma - u$ )
$\sigma'_{vo}$	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
$\sigma_{oct}$	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
$\tau$	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
$\gamma'$	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  $\rho$ . Unit weight symbol is  $\gamma$  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
$C_{\alpha}$	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
$\sigma'_p$	pre-consolidation stress
OCR	over-consolidation ratio = $\sigma'_p / \sigma'_{vo}$

#### (d) Shear Strength

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

Notes: 1  
2

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



## LIST OF ABBREVIATIONS

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

### I. SAMPLE TYPE

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

### III. SOIL DESCRIPTION

#### (a) Cohesionless Soils

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

### II. PENETRATION RESISTANCE

#### Standard Penetration Resistance (SPT), N:

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

#### (b) Cohesive Soils Consistency

	kPa	Cu, Su	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

#### Dynamic Cone Penetration Resistance; N<sub>d</sub>:

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

**PH:** Sampler advanced by hydraulic pressure

**PM:** Sampler advanced by manual pressure

**WH:** Sampler advanced by static weight of hammer

**WR:** Sampler advanced by weight of sampler and rod

#### Piezo-Cone Penetration Test (CPT)

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

### IV. SOIL TESTS

w	water content
w <sub>p</sub>	plastic limit
w <sub>l</sub>	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test <sup>1</sup>
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement <sup>1</sup>
D <sub>R</sub>	relative density (specific gravity, G <sub>s</sub> )
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 <sub>4</sub>	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

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

### V. MINOR SOIL CONSTITUENTS

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



# LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

## WEATHERING STATE

**Fresh:** no visible sign of weathering

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

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

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

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

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

## BEDDING THICKNESS

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

## JOINT OR FOLIATION SPACING

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

## GRAIN SIZE

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

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

## CORE CONDITION

### Total Core Recovery (TCR)

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

### Solid Core Recovery (SCR)

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

### Rock Quality Designation (RQD)

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

## DISCONTINUITY DATA

### Fracture Index

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

### Dip with Respect to Core Axis

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

### Description and Notes

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

### Abbreviations

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



# FIELD TEST PIT LOG: B7-01

SHEET: 1/1

PROJECT: 07-1111-0029

DATE: January 15, 2015

LOCATION: N 5045465.0 E 243894.0

CONTRACTOR: Landcore Drilling

ELEVATION: 207.0 m

MACHINE TYPE: 135D John Deere

TEMPERATURE -8 °C

WEATHER: Cloudy with flurries

DEPTH (m)		SOIL DESCRIPTION	SAMPLE		Laboratory Results		Remarks
			No.	Depth (m)	Water Content (%)	Grain Size Distribution Gr Sa Si Cl	
From	To						
0.0	0.2	TOPSOIL	1	0.2			
0.2	0.8	Silty SAND, trace organics Brown Moist	2	0.6	8.1		
0.8	1.8	SAND, some silt Light brown Moist to wet	3	1.5	20.3		
1.8		BEDROCK					
		END OF TEST PIT					

## Notes :

- 1) Side of test pit stable
- 2) Bedrock sloping to the East at about 45 degrees

ORIGINATED BY: ID  
COMPILED BY: MP  
CHECKED BY: MCK

## WATER CONDITIONS IN TEST PIT

☒ Test Pit Dry



# FIELD TEST PIT LOG: B7-02

SHEET: 1/1

PROJECT: 07-1111-0029

DATE: January 15, 2015

LOCATION: N 5045470.7 E 243899.5

CONTRACTOR: Landcore Drilling

ELEVATION: 205.8 m

MACHINE TYPE: 135D John Deere

TEMPERATURE -8 °C

WEATHER: Cloudy with flurries

DEPTH (m)		SOIL DESCRIPTION	SAMPLE		Laboratory Results		Remarks
			No.	Depth (m)	Water Content (%)	Grain Size Distribution Gr Sa Si Cl	
0.0	0.3	TOPSOIL	1	0.2	81.4		
0.3	0.6	Silty SAND, trace gravel, trace organics, with pieces of wood and rootlets Brown Moist	2	0.5	42.7		
0.6	1.2	SILT, some sand Light brown Moist	3	0.9	26.6	0 18 78 4	
1.2	1.9	SAND and GRAVEL, some silt, with cobbles and boulder (0.4 m diameter) Light brown Moist	4	1.5			
1.9		BEDROCK					

END OF TEST PIT

## Notes :

- 1) Side of test pit stable
- 2) Bedrock sloping to the East

ORIGINATED BY: ID  
COMPILED BY: MP  
CHECKED BY: MCK

## WATER CONDITIONS IN TEST PIT

☒ Test Pit Dry



PROJECT 07-1111-0029		<b>RECORD OF BOREHOLE No B7-03</b>				SHEET 1 OF 1		<b>METRIC</b>									
W.P. 5183-06-01		LOCATION N 5045476.8 ; E 243905.5				ORIGINATED BY ID											
DIST _____ HWY 69		BOREHOLE TYPE CME 550, 108 mm I.D. Continuous Flight, Hollow Stem Augers				COMPILED BY MP											
DATUM Geodetic		DATE January 21, 2015				CHECKED BY MCK											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
205.5	GROUND SURFACE							20	40	60	80	100					
0.0	TOPSOIL Loose		1	SS	5												
204.9	Granite Gneiss (BEDROCK)																
0.6	Bedrock cored from depths of 0.6 m to 6.1 m.  For bedrock coring details refer to Record of Drillhole B7-03.		1	RC	REC 100%												
			2	RC	REC 100%												
			3	RC	REC 100%												
			4	RC	REC 100%												
199.4	END OF BOREHOLE																
6.1	NOTE:  1. Water level not recorded upon completion of drilling.																

PROJECT: 07-1111-0029

## RECORD OF DRILLHOLE: B7-03

SHEET 1 OF 1

LOCATION: N 5045476.8 ; E 243905.5

DRILLING DATE: January 21, 2015

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 550

DRILLING CONTRACTOR: Landcore Drilling Inc.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH	COLOUR % RETURN	JN - Joint FLT - Fault SH - Shear VN - Vein CJ - Conjugate	BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage	PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular	PO - Polished K - Slickensided SM - Smooth RO - Rough VR - Very Rough	MB - Mechanical Break BR - Broken Rock NOTE: For additional abbreviations refer to list of abbreviations & symbols.	NOTES
		Continued from Record of Borehole B7-03		204.89 0.61									
1		GRANITE GNEISS Fresh to slightly weathered, foliated, grey, white and pink, fine to coarse grained, non-porous, medium strong to strong			1	100							
2					2	100							
3					3	100							
4					4	100							
5					5	100							
6					6	100							
7					7	100							
8					8	100							
9					9	100							
10					10	100							
		END OF DRILLHOLE		199.40 6.10									

DEPTH SCALE

1 : 50



LOGGED: ID

CHECKED: MCK

GTA-RCK 018 T:\PROJECTS\2007-11-11-0029 (MRC, PARRY SOUND)\LOG\07-11-11-0029-SHEBESHEKONG RD-PHASE II\GPJ GAL-MISS GDT 8/31/15 DD/SAC

PROJECT		07-1111-0029		RECORD OF BOREHOLE No B7-04		SHEET 1 OF 1		METRIC										
W.P.		5183-06-01		LOCATION		N 5045469.4 ; E 243889.4		ORIGINATED BY ID										
DIST		HWY 69		BOREHOLE TYPE		CME 550, 108 mm I.D. Continuous Flight, Hollow Stem Augers		COMPILED BY MP										
DATUM		Geodetic		DATE		January 19, 2015		CHECKED BY MCK										
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)
206.4	GROUND SURFACE							20	40	60	80	100						
0.0	TOPSOIL		1A	SS	5													
0.3	Sandy SILT, trace gravel, trace clay, trace organics Loose Brown Moist		1B	SS	5													
			2	SS	5													
204.6	Granite Gneiss (BEDROCK)		3	SS	41/0.15													
1.8	Bedrock cored from depths of 1.8 m to 6.8 m.  For bedrock coring details refer to Record of Drillhole B7-04.		1	RC	REC 100%													
			2	RC	REC 100%													
			3	RC	REC 100%													
			4	RC	REC 100%													
199.6	END OF BOREHOLE																	
6.8	NOTE:  1. Water level in open borehole at a depth of 3.7 m below ground surface (Elev. 202.7 m) upon completion of drilling.																	

PROJECT: 07-1111-0029

**RECORD OF DRILLHOLE: B7-04**

SHEET 1 OF 1

LOCATION: N 5045469.4 ; E 243889.4

DRILLING DATE: January 19, 2015

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 550

DRILLING CONTRACTOR: Landcore Drilling Inc.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH	RECOVERY TOTAL CORE %	SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.25 m	B Angle	DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	HYDRAULIC CONDUCTIVITY K, cm/sec	Diametral Point Load Index (MPa)	RMC -Q AVG.	NOTES
		Continued from Record of Borehole B7-04		204.57 1.83																
2	NQRC January 19, 2015	GRANITE GNEISS Fresh to slightly weathered, foliated, dark grey, white and pink, fine to coarse grained, non-porous, medium strong to strong			1	100							JN,IR,RO							
3													JN,IR,RO							
4					2	100							JN,IR,RO JN,IR,RO JN,IR,RO							Axial Axial
5					3	100							JN,IR,RO							
6					4	100														9.7 MPa Axial
7		END OF DRILLHOLE		199.57 6.83																Axial
8																				
9																				
10																				
11																				

DEPTH SCALE

1 : 50



LOGGED: ID

CHECKED: MCK

GTA-RCK 018 T:\PROJECTS\2007-11-11-0029 (MRC, PARRY SOUND)\LOG\07-11-11-0029-SHEBESHEKONG RD-PHASE II\GPJ GAL-MISS.GDT 8/31/15 DD/SAC

# FIELD TEST PIT LOG: B7-05

SHEET: 1/1

PROJECT: 07-1111-0029

DATE: January 15, 2015

LOCATION: N 5045475.1 E 243894.9

CONTRACTOR: Landcore Drilling

ELEVATION: 205.2 m

MACHINE TYPE: 135D John Deere

TEMPERATURE -8 °C

WEATHER: Cloudy with flurries

DEPTH (m)		SOIL DESCRIPTION	SAMPLE		Laboratory Results		Remarks
			No.	Depth (m)	Water Content (%)	Grain Size Distribution Gr Sa Si Cl	
From	To						
0.0	0.3	TOPSOIL	1	0.2	36.3		
0.3	0.9	Silty SAND to SILT and SAND, trace gravel, trace organics Brown Moist	2	0.6	34		
0.9	1.2	Sandy SILT, trace of clay, trace to some gravel Light brown Moist	3	1.1			
1.2	1.7	SILT, some sand, trace clay, trace organics Light brown Moist	4	1.5	19.2	0 17 78 5	
1.7		BEDROCK					

END OF TEST PIT

## Notes :

- 1) Side of test pit stable
- 2) Bedrock gently sloping down to the Northeast

ORIGINATED BY: ID  
COMPILED BY: MP  
CHECKED BY: MCK

## WATER CONDITIONS IN TEST PIT

☒ Test Pit Dry





# FIELD TEST PIT LOG: B7-06

SHEET: 1/1

PROJECT: 07-1111-0029

DATE: January 15, 2015

LOCATION: N 5045481.2 E 243900.7

CONTRACTOR: Landcore Drilling

ELEVATION: 204.1 m

MACHINE TYPE: 135D John Deere

TEMPERATURE -8 ° C

WEATHER: Cloudy with flurries

DEPTH (m)		SOIL DESCRIPTION	SAMPLE		Laboratory Results				Remarks
			No.	Depth (m)	Water Content (%)	Grain Size Distribution			
From	To						Gr	Sa	
0.0	0.2	TOPSOIL	1	0.15					
0.2	0.3	Silty SAND to Sandy SILT, trace organics Reddish brown Moist	2	0.27	33.6				
0.3		BEDROCK							
		END OF TEST PIT							

## Notes :

- 1) Side of test pit stable
- 2) Bedrock gently slopping down to the Northeast


ORIGINATED BY: ID  
COMPILED BY: MP  
CHECKED BY: MCK

## WATER CONDITIONS IN TEST PIT


☒ Test Pit Dry



PROJECT <u>07-1111-0029</u>		<b>RECORD OF BOREHOLE No B7-07</b>		SHEET 1 OF 1		<b>METRIC</b>	
W.P. <u>5183-06-01</u>		LOCATION <u>N 5045465.8 ; E 243904.6</u>		ORIGINATED BY <u>ID</u>			
DIST <u>          </u> HWY <u>69</u>		BOREHOLE TYPE <u>CME 550, 108 mm I.D. Continuous Flight, Hollow Stem Augers</u>		COMPILED BY <u>MP</u>			
DATUM <u>Geodetic</u>		DATE <u>January 20, 2015</u>		CHECKED BY <u>MCK</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT						PLASTIC LIMIT   NATURAL MOISTURE   LIQUID CONTENT   LIMIT			UNIT WEIGHT  γ  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						WATER CONTENT (%)				GR	SA	SI	CL
								○ UNCONFINED      + FIELD VANE ● QUICK TRIAXIAL    × REMOULDED													
207.7	GROUND SURFACE							20	40	60	80	100									
0.0	Silty SAND, trace clay, trace organics Loose to very dense Light brown Moist		1A 1B	SS	7																
206.5			2	SS	65																
1.2	END OF BOREHOLE SPLIT-SPOON REFUSAL  NOTE:  1. Water level in open borehole at a depth of 1.2 m below ground surface (Elev. 206.5 m) upon completion of drilling.																				

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PROJECT		07-1111-0029		RECORD OF BOREHOLE No B7-08		SHEET 1 OF 1		METRIC								
W.P.		5183-06-01		LOCATION		N 5045480.1 ; E 243889.7		ORIGINATED BY								
DIST		HWY 69		BOREHOLE TYPE		CME 550, 108 mm I.D. Continuous Flight, Hollow Stem Augers		COMPILED BY								
DATUM		Geodetic		DATE		January 21, 2015		CHECKED BY								
								MCK								
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa								
203.7	GROUND SURFACE															
0.0	SILT and SAND, trace organics Very loose Brown Moist		1	SS	4										51.5	
203.0	END OF BOREHOLE AUGER REFUSAL															
0.7	NOTES:  1. Open borehole dry upon completion of drilling.  2. Additional boreholes advanced 1 m southeast and 1 m northwest of Borehole B7-08 location encountered auger refusal at a depth of about 0.6 m.															

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# APPENDIX B

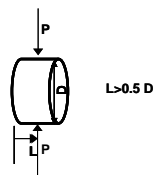
## LABORATORY TEST RESULTS

## Summary of Point Load Test Results on Rock Samples

[illegible]

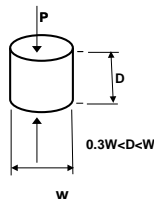
## DIAMETRAL SPECIMEN SHAPE REQUIREMENTS

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



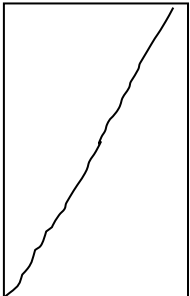
## AXIAL SPECIMEN SHAPE REQUIREMENTS

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



Compiled By: MCK  
Checked By: CN  
Reviewed By: JMAC

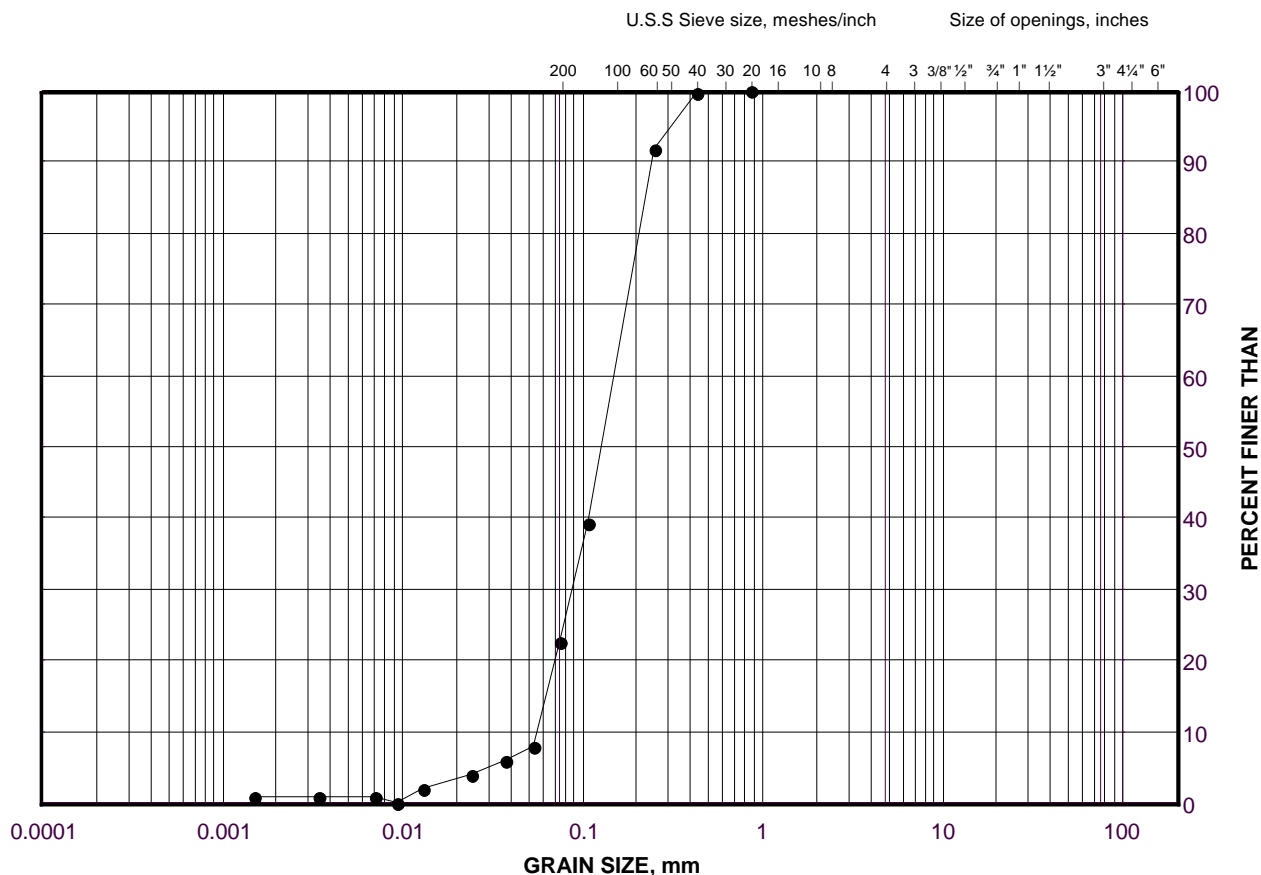
**TABLE B2**  
**UNCONFINED COMPRESSION (UC) TEST**  
**ASTM D7012**

SAMPLE IDENTIFICATION			
PROJECT NUMBER	07-1111-0029	SAMPLE NUMBER	03
BOREHOLE NUMBER	B7-03	SAMPLE DEPTH, m	3.43-3.66
TEST CONDITIONS			
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.19
SPECIMEN INFORMATION			
SAMPLE HEIGHT, cm	10.37	WATER CONTENT, (specimen) %	0.17
SAMPLE DIAMETER, cm	4.73	UNIT WEIGHT, kN/m <sup>3</sup>	26.83
SAMPLE AREA, cm <sup>2</sup>	17.53	DRY UNIT WT., kN/m <sup>3</sup>	26.78
SAMPLE VOLUME, cm <sup>3</sup>	181.83	SPECIFIC GRAVITY	-
WET WEIGHT, g	497.60	VOID RATIO	-
DRY WEIGHT, g	496.76		
VISUAL INSPECTION		FAILURE SKETCH	
			
TEST RESULTS			
STRAIN AT FAILURE, %	0.0	COMPRESSIVE STRENGTH, MPa	55.8
REMARKS:		DATE: 2015-03-02	
CHECKED BY: MCK		REVIEWED BY: CN / JMAC	

# GRAIN SIZE DISTRIBUTION

Silty Sand  
Shebeshekong Road NBL Overpass Structure

FIGURE B1



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

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	B7-07	2	206.7

Project Number: 07-1111-0029

Checked By: CN

**Golder Associates**

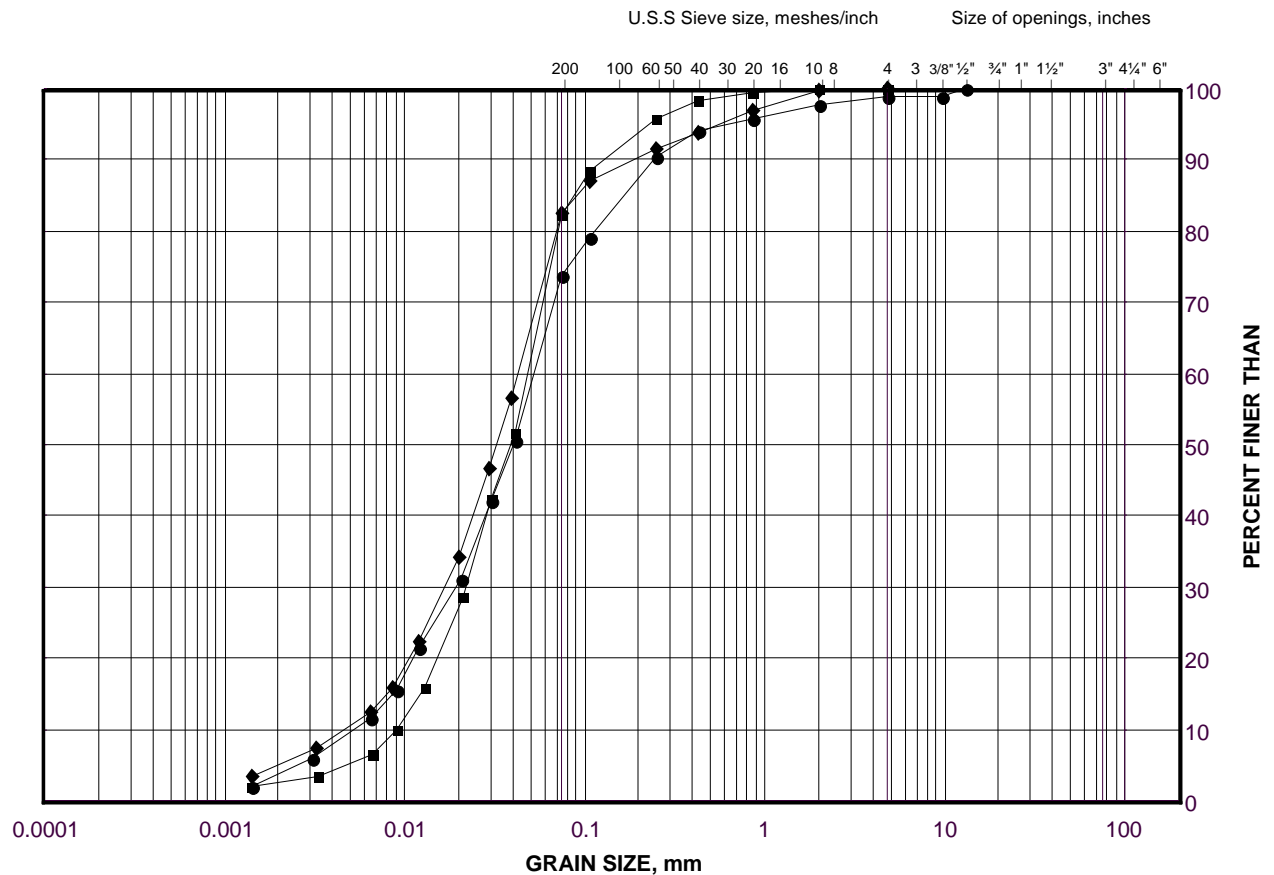
Date: 24-Apr-15



# GRAIN SIZE DISTRIBUTION

Silt to Sandy Silt  
Shebeshekong Road NBL Overpass Structure

FIGURE B2



## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	B7-04	2	205.4
■	B7-02	3	204.9
◆	B7-05	4	203.7

Project Number: 07-1111-0029

Checked By: CN

**Golder Associates**

Date: 24-Apr-15

### Borehole B7-03



Box 1: 0.61 m – 4.72 m



Box 2: 4.72 m – 6.10 m

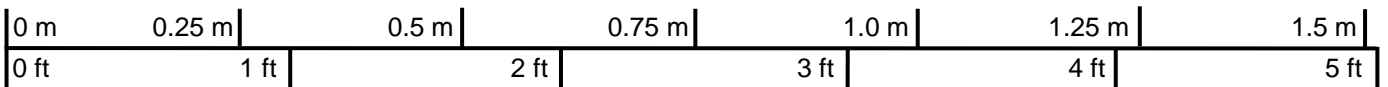
### Borehole B7-04




Box 1: 1.83 m – 6.05 m



Box 2: 6.05 m – 6.83 m



Scale

PROJECT		KEY RIVER (NBL) Highway 69 GWP 511-07-00; WP 5184-06-01			
TITLE		Bedrock Core Photograph – Boreholes B7–03 and B7–04			
		PROJECT No. 07-1111-0029		FILE No. ----	
		DESIGN	MCK	APR 15	SCALE NTS
		CADD	-- --		REV.
		CHECK	CN	APR 15	FIGURE B3
		REVIEW	JMAC	APR 15	



# APPENDIX C

## NON-STANDARD SPECIAL PROVISIONS

## **DOWELS INTO ROCK - Item No.**

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Non-Standard Special Provision

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### **Scope of Work**

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

### **Construction**

Dowels into rock shall be constructed in accordance with OPSS.PROV 904 Concrete Structures. All reinforcing steel supplied shall be in accordance with OPSS.PROV 1440 Steel Reinforcement for Concrete (dowel bars conforming to CAN/CSA G30.18, Grade 400).

Where dowels are to be placed in rock, hole shall be drilled to the required depth and size. Hole diameter shall be two times the nominal diameter of the dowel. Each hole shall be cleaned out, grouted and the dowel set in place. Grout shall be of the same strength as the footing concrete or at least 25 MPa at 28 days.

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

### **Rock Dowel Testing**

All proposed testing procedures shall be in general conformance with ASTM D3689, ASTM D1143/D1143M and ASTM D4435. Field testing must be carried out in the presence of, and the results reviewed and approved by, the Contract Administrator.

### **Performance Tests**

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

<b>Bridge</b>	<b>Foundation</b>	<b>Number of Dowels for Performance Testing</b>
Highway 69 / Shebeshekong Road Overpass Bridge (NBL)	North and South Abutment	2

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

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

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

The design load shall be taken as 360 kN for 35M dowels, 252 kN for 30M dowels, 180 kN, for 25M dowels, and 108 kN for 20M dowels.

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

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

### **Basis of Payment**

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

END OF SECTION

**OBSTRUCTIONS - Item No.**

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Non-Standard Special Provision

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Cobbles and boulders were encountered at the project site above the depth of the footings. Consideration of the presence of these obstructions must be made in the selection of appropriate equipment and procedures for sub-excavation through this material for footing construction.

**Basis of Payment**

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

END OF SECTION

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

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