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REPORT ON

FOUNDATION INVESTIGATION AND DESIGN
DETAIL DESIGN
HIGHWAY 11 AND MUSKOKA ROAD 169 UNDERPASS STRUCTURE
HIGHWAY 11 / MUSKOKA ROAD 169 INTERCHANGE
GRAVENHURST, ONTARIO
G.W.P 314-00-00

Submitted to:

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by McCormick Rankin Corporation (MRC) to provide foundation engineering services for the following components for the Highway 11 Interchange with Muskoka Road 169 (G.W.P. 314-00-00) in Gravenhurst, Ontario:

- Highway 11 and Pinedale Road/Hewitt Street underpass structure;

- Rehabilitation of the existing Gull Lake Northbound and Southbound Bridges and proposed widening of the southbound bridge structure;

- Highway 11 and Muskoka Road 169 underpass structure;

- Swamp crossing between approximate Hwy 11 NBL Stations 11+510 and 11+940 and SBL Stations 11+550 and 11+970.

This report addresses the new Highway 11 / Muskoka Road 169 Interchange underpass structure and the associated approach embankments. Two phases of foundation investigations have been carried out to assess the subsurface conditions at this site. The foundation investigations for the related swamp crossing, Gull Lake Bridge structure widening, and Pinedale / Hewitt underpass structure for the project are provided in separate reports.

The initial investigation at this site was carried out in May 2005, consistent with the scope of work as outlined in Golder's proposal P41-1349 dated May 2004 that formed part of the Consultant's Agreement (P.O. Number 5005-A-000363) for this project. Based on the variable bedrock conditions encountered during the initial investigation and after a meeting with MTO and MRC, a supplemental investigation was carried out in November 2005, consistent with the scope of work as outlined in our letter to MRC titled "Proposal for Supplementary Investigation for Foundation Engineering Services, Highway 11/Muskoka Road 169 Underpass", dated November 2, 2005. The supplemental investigation was carried out to provide additional information on the variable subsurface conditions (i.e. bedrock surface) at the proposed foundation locations and to investigate other potential foundation locations (i.e. assuming lengthening or shifting of the bridge location to the east and/or west of the original proposed location).

All of the work was carried out in accordance with the Quality Control Plan for this project dated August 2004. The general arrangement drawing for the proposed new underpass structure at Highway 11 / Muskoka Road 169 was provided to Golder by MRC in October 2005.

The purpose of these investigations were to establish the subsurface conditions at the proposed structure by borehole drilling, rock coring, in-situ testing and laboratory testing on selected samples. The boreholes for the initial investigation were located in the field by a member of Golder's staff based on the information and survey layout provided by MRC. The supplemental boreholes were located on a plan by Golder and surveyed and staked in the field by J.D. Barnes Ltd. prior to drilling. The general location of the investigated area is shown in the Key Plan on Drawings 1, 2 and 3.

2.0 SITE DESCRIPTION

The site is located about 350 m east of the existing Highway 11 alignment and about 600 m west of Jevins Lake in Gravenhurst, Ontario. The proposed underpass is to be constructed at about Station 11+200 along the proposed Highway 11 NBL alignment. The proposed Highway 11 in this area will consist of a four-lane freeway, with northbound and southbound lanes (i.e. two lanes in each direction) separated by a grass median. In addition, the Highway 11 E-S ramp and W-N ramp lanes will be located in this area, within the span limits of the underpass structure. The proposed Muskoka Road 169 highway in this area will consist of two lanes (one lane in the east and west direction).

The majority of the site is currently situated within the limits of what appears to be an old sand pit. Adjacent to the open sand pit, the site consists of undulating terrain including densely treed areas and numerous steeply sloping bedrock outcrops. The ground surface within the limits of the proposed Highway 11 / Muskoka Road 169 underpass and approach embankment area generally lies between about Elevation 253 m and 260 m, referenced to Geodetic Datum. Bedrock outcrops are exposed at the ground surface at the north and east limits of the site, specifically near the east abutment and east approach embankment location. Currently, there are private residences located directly west of the site and the existing MTO Gravenhurst Patrol Yard is located immediately south of the site. Beyond these locations, and to the immediate north and east along the proposed highway alignments, the site consists of dense trees, swamp areas and steeply sloping bedrock outcrops.

3.0 INVESTIGATION PROCEDURES

3.1 Foundation Investigation

As mentioned previously, two separate field investigations were performed at this site. In total, seventeen (17) boreholes were advanced and two (2) test pits were excavated as part the investigation. The initial field work for the two-span bridge underpass investigation was carried out between May 16 to May 19, 2005, during which time a total of eight (8) sampled boreholes were put down at the site. The supplemental field investigation was carried out between November 10 and November 17, 2005, during which time a total of nine (9) additional boreholes were advanced and two (2) test pits were excavated at the site.

A total of five (5) boreholes were drilled at each of the centre pier and west abutment footing locations, three (3) boreholes and two (2) test pits were advanced/excavated at the east abutment footing location and two (2) boreholes were advanced at each of the proposed east and west approach embankments / possible bridge lengthening areas. All of the boreholes were advanced to refusal on inferred bedrock. In ten (10) of the boreholes at the pier, abutment and approach embankment locations, bedrock coring was carried out for a length of about 3 m.

The boreholes were advanced using a D-50 track-mounted drill rig supplied and operated by Walker Drilling Ltd. of Barrie, Ontario. The boreholes put down with the drill rig were advanced using 108 mm inner diameter (I.D.) hollow stem augers. Soil samples were obtained, where possible, continuously or at intervals of about 0.75 m to 1.5 m depth, using a 50 mm outer diameter (O.D.) split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586-99). Samples of the bedrock were obtained using an ‘NQ’ size rock core barrel. The boreholes were backfilled with bentonite in accordance with Ontario Reg. 128. The test pits were excavated and backfilled using a CASE 9030 backhoe supplied and operated by personnel from the nearby MTO patrol yard.

The boreholes were all advanced to auger and/or sampler refusal (i.e. inferred bedrock) which occurred at depths ranging from 1.9 m to about 12.3 m below the existing ground surface (not including rock coring). A Dynamic Cone Penetration Test (DCPT) was advanced from the bottom of borehole BH17 to refusal which was reached at 12.3 m below ground surface. At boreholes BH02, BH04, BH06, BH09 to BH14 and BH16, located within the limits of the proposed foundation locations, the drilling was further advanced into the bedrock by coring about 3.0 m. The test pits were excavated to backhoe bucket refusal on inferred bedrock which occurred at depths of 0.9 m and 1.5 m for TP1 and TP2, respectively. The groundwater level in the open boreholes was observed and recorded throughout the drilling operations. A piezometer was installed in BH05 to monitor groundwater levels at this location.

The field work was supervised throughout by members of our technical staff, who located the boreholes, arranged for the clearance of underground service locations, supervised 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 detailed visual examination and appropriate laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards as appropriate. Classifications testing such as water content, organic content and grain size distribution were carried out on samples of the overburden soils. Strength testing such as point load index was carried out on specimens from the rock core.

All investigated borehole locations were surveyed and referenced to the NAD83 MTM co-ordinate system and the geodetic datum for elevation. The surveying of the ground surface elevations of the as-drilled boreholes from the initial investigation (BH01 to BH08) was carried out by members of our engineering staff, referenced to benchmark geodetic elevations at the corners of each foundation footprint provided by J.D. Barnes Ltd. The northing and easting coordinates of the borehole locations were calculated based on measurements from adjacent survey control points provided by J.D. Barnes Ltd. The locations of the supplemental boreholes (Boreholes BH09 to BH17=4) and test pits (TP1 and TP2) were located on a plan by Golder and surveyed and staked by J.D. Barnes Ltd. prior to drilling.

The borehole locations are summarized in the following table and are shown on Drawing 1.

Borehole/Test Pit Number	Borehole Location	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)
BH01	East Abutment	4973548.3	315735.4	255.8
BH02	East Abutment	4973541.1	315733.2	253.8
BH03	Center Pier	4973561.0	315681.5	255.1
BH04	Center Pier	4973553.5	315683.0	253.8
BH05	West Abutment	4973561.8	315639.0	257.4
BH06	West Abutment	4973569.6	315642.6	257.6
BH07	East Approach	4973544.0	315746.4	255.8
BH08	West Approach	4973566.6	315627.7	257.3
BH09	West Approach	4973573.3	315629.5	257.7
BH10	West Abutment	4973572.3	315640.4	257.8
BH11	West Abutment	4973568.8	315639.3	257.5
BH12	West Abutment	4973561.1	315640.4	257.5
BH13	Center Pier	4973560.4	315684.3	254.3
BH14	Center Pier	4973557.3	315682.2	254.1
BH15	Center Pier	4973554.1	315680.4	254.0
BH16	East Abutment	4973544.9	315734.1	254.1
BH17	East Approach	4973539.1	315746.1	254.4
TP1	East Abutment	4973552.8	315735.9	256.7
TP2	East Abutment	4973550.8	315735.4	256.3

4.0 GENERAL SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Geology

From published geologic information, the site is located mostly within the physiographic region known as the Number 11 Strip with portions of Highway 11 in contact with the Georgian Bay Fringe region. The Number 11 Strip is a narrow belt that extends from Gravenhurst to North Bay and is categorized by deposits of sand, silt and clay between rock outcrops. The Georgian Bay Fringe is a broad belt characterized by shallow soil and bare bedrock knobs and ridges (The Physiography of Southern Ontario; Third Edition). Quaternary deposits of lacustrine and fluvial origin together with more recent swamp sediments have been accumulated between the bedrock ridges and, consequently, the overburden thickness and bedrock surface can be variable. The bedrock in the area is typically highly deformed gneiss of the Moon River Domain of the Central Gneiss Belt, a subdivision of the Grenville Structural Province (Geology of Ontario; OGS Special Volume 4). Deposition of Paleozoic strata and later erosion during glaciation left behind these Precambrian rocks covered only in a few places by the flat-lying Palaeozoic bedrock strata.

4.2 Subsurface Conditions and General Overview

The detailed subsurface soil and groundwater conditions as encountered in the boreholes and test pits advanced during this investigation, together with the results of the laboratory tests carried out on selected soil samples, are given on the attached Record of Borehole sheets and Test Pit logs following the text of this report. The results from the laboratory testing are provided in Appendix A. 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 (SPTs). These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Further, subsurface conditions will vary between and beyond the borehole locations.

The inferred soil stratigraphy as encountered in the boreholes and test pits at the proposed two-span bridge location are shown on Drawings 1, 2 and 3.

In general, the subsoils at the proposed structure site consist of sand and silty sand to sand and silt deposits, underlain by gneissic bedrock. The surficial sand and silt deposits contained trace amounts of organics and the silt content of the granular deposit generally increased towards the east side of the site. The total overburden thickness ranged from no cover (i.e. bedrock outcrops present at ground surface near the east abutment location) to about 12.3 m below ground surface. Seven (7) boreholes were terminated at the inferred bedrock surface; ten (10) boreholes were cored to a depth of about 3 m into the bedrock.

It should be noted that most of the boreholes located in the southeast portion of the site (BH02 to BH04, BH13 to BH15, and BH16) were located in a depressed area, understood to be an abandoned sand pit which is about 3.6 m lower in elevation compared to the surrounding area to the north and west.

In one borehole put down at the proposed centre pier (i.e. BH04), a layer of cobbles and/or boulders was encountered directly above the bedrock.

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

4.2.1 Topsoil / Asphalt

A layer of topsoil was encountered at the ground surface in boreholes BH10 to BH12 (inclusive) and in both test pits (TP1 and TP2). The topsoil ranged from about 0.2 m to 0.3 m thick.

Asphalt was encountered at the ground surface in one borehole (BH09) located at the dead end of the existing Holmes Road. The asphalt was about 90 mm thick and was underlain by about 0.1 m of sand and gravel fill.

4.2.2 Sand, Silty Sand, Sand and Silt

Underlying the topsoil and asphalt (BH09 to BH12, TP1 and TP2) and at the ground surface in the remaining boreholes (BH01 to BH08, BH13 to BH17), a deposit of cohesionless sand and silty sand to sand and silt was encountered. These granular soils typically contained trace gravel and clay; and trace organics were encountered within about 0.3 m of the ground surface. A layer of silty sand to gravelly sand was encountered at the ground surface in Borehole BH01.

A predominantly sand layer was encountered at all borehole locations (BH01 to BH17) except BH07. The top of the sand layer was encountered at depths ranging from ground surface to 4.6 m depth (Elevation 257.6 m to Elevation 249.5 m) and the thickness ranged from 1.2 m to 4.5 m. Generally, the sand layer slopes downward from west to east and north to south across the site. The soils encountered above and below the sand layer consisted of silty sand to sand and silt.

A layer of cobbles / boulders was encountered in BH04 at a depth of 7.3 m (Elevation 246.5 m), below the silty sand deposit and directly above the bedrock surface. The cobble/boulder layer was 1.1 m thick. Although not encountered within any other borehole locations, cobbles and boulders may be present within the cohesionless soils near the bedrock interface.

Standard Penetration Testing (SPT) measured ‘N’ values in the cohesionless soils generally ranged from 10 blows to 95 blows per 0.3 m of penetration indicating a generally compact to very dense relative density. Three measured 'N' values from the upper 1 m of surficial sandy soil were 3, 8 and 9 blows per 0.3 m of penetration. Also, five measured 'N' values ranged from 0 (i.e. weight of hammer) to 7 blows per 0.3 m of penetration within the saturated silty sand to sand soil at a depth below about 8 m in BH02, BH16 and BH17; this may be the result of 'blowing' sands into the augers during drilling operations. Typically, higher 'N' values were measured at or near the bedrock interface.

The natural water content measured on selected samples of the sand, silty sand, and sand and silt soils ranged from 2 to 25 percent. The higher water content values were generally measured from samples obtained below the water table and with higher silt content. Grain size distribution curves taken on selected samples of the sand, silty sand and sand and silt are shown on Figures A1, A3, A4 and A5 respectively, in Appendix A. A sample of the silty sand to gravelly sand is shown on Figure A2.

4.2.3 Bedrock

Visible bedrock outcrops are located at the north and east limits of the site as shown on Drawing 1. Bedrock was encountered and cored for 3 m in boreholes BH02, BH04, BH06, BH09 to BH14 and BH16. The presence of bedrock was inferred from auger and/or sampler refusal in all remaining boreholes and excavator bucket refusal in the test pits. The surface of the bedrock is highly variable and fluctuates within the boreholes from about 1.9 m to 12.3 m depth below ground surface, corresponding to elevations ranging from Elevation 242.1 m to 255.8 m. The depth to bedrock below ground surface and corresponding bedrock surface elevation encountered at each borehole and test pit location is listed in the following table.

Borehole/Test Pit Number	Borehole/Test Pit Location	Depth to Bedrock Surface (m)	Bedrock Surface Elevation (m)	Comments
BH08	West Approach	1.9	255.4	Auger Refusal
BH09	West Approach	3.6	254.1	Bedrock Cored
BH05	West Abutment	3.2	254.2	Auger Refusal
BH06	West Abutment	7.3	250.3	Bedrock Cored
BH10	West Abutment	7.3	250.5	Bedrock Cored
BH11	West Abutment	6.6	250.9	Bedrock Cored
BH12	West Abutment	3.8	253.7	Bedrock Cored
BH03	Centre Pier	6.3	248.9	Auger Refusal
BH04	Centre Pier	8.4	245.4	Bedrock Cored
BH13	Centre Pier	5.9	248.4	Bedrock Cored
BH14	Centre Pier	6.6	247.5	Bedrock Cored
BH15	Centre Pier	6.9	247.1	Auger Refusal

Borehole/Test Pit Number	Borehole/Test Pit Location	Depth to Bedrock Surface (m)	Bedrock Surface Elevation (m)	Comments
BH01	East Abutment	5.0	250.8	Auger Refusal
BH02	East Abutment	11.0	242.8	Bedrock Cored
BH16	East Abutment	9.1	245.0	Bedrock Cored
TP1	East Abutment	0.9	255.8	Bucket Refusal
TP2	East Abutment	1.5	254.8	Bucket Refusal
BH07	East Approach	2.0	253.9	Auger Refusal
BH17	East Approach	12.3	242.1	DCPT Refusal

Based on the cored bedrock samples, the bedrock generally consists of biotite gneiss and granite gneiss. The bedrock typically transitions from biotite to granite gneiss from southeast to northwest across the site. Rock cores obtained from BH02, BH04, BH12, BH14 and BH16 consists partially or entirely of biotite gneiss. Granite gneiss was encountered in all cored boreholes (except BH14 and BH16). A localized area of biotite schist was encountered from a depth of about 9.5 m below ground surface in BH16.

In general, the biotite gneiss bedrock samples (i.e. BH02, BH04, BH12, BH14 and BH16) are described as slightly weathered to fresh, foliated black and white, fine to medium grained, and strong to very strong. The bedrock samples typically contained distinct foliation planes and medium to coarse grained quartz and feldspar rich veins/banding. The Rock Quality Designation (RQD) measured on the core samples typically ranged from about 73 to 98 percent, indicating a rock mass of fair to excellent quality. However, the upper portion of boreholes BH02, BH04 and BH16 contained zones of broken rock and the RQD measured on core samples ranged from about 40 to 48 percent indicating a rock mass of poor quality. The Total Core Recovery was between 97 percent and 100 percent.

The granite gneiss bedrock samples obtained from BH06, BH09 to BH13, and at depth in BH02 and BH04, are described as slightly weathered to fresh, foliated black, white and pink, fine to coarse grained, very strong. The bedrock samples typically contained distinct foliation planes and fine to medium grained biotite-rich bands/clusters and thinly banded quartz. The Rock Quality Designation (RQD) measured on the core samples typically ranged from about 73 to 100 percent, indicating a rock mass of fair to excellent quality. The Total Core Recovery was between 92 percent and 100 percent.

The biotite schist bedrock samples obtained from BH16, located at the east abutment, are described as slightly weathered to fresh, foliated black, fine to medium grained, and weak to strong. The bedrock samples typically contained distinct foliation planes of biotite and very little quartz minerals. The Rock Quality Designation (RQD) measured on the core samples was about

42 percent in the upper portion and 87 percent below a depth of 10.7 m, indicating a rock mass of poor to good quality. The low RQD values measured in the upper portion of borehole BH16 can be attributed to zones of broken rock (about 0.1 m to 0.2 m thick) which were encountered within about 1.1 m of the bedrock surface. The Total Core Recovery was between 97 percent and 100 percent.

Point load strength tests were performed on samples of the rock core. Diametral and axial point load strength index values are shown on the Record of Drillhole Sheets and on Table 1 following the text of this report. The point load index (Is_{50}) results from the laboratory tests on the gneissic bedrock range from approximately 2.8 MPa to 8.0 MPa with an average of about 5.1 MPa for diametral tests (i.e. testing carried out perpendicular to the core axis). Axial tests (i.e. testing carried out parallel to the core axis) performed on the gneissic bedrock range from approximately 3.1 to 8.6 with an average of about 6.3 MPa. Diametral and axial point load tests performed on the biotite gneiss bedrock samples typically gave lower values (i.e. compared to the granite gneiss bedrock) with an average of about 3.9 MPa and 5.6 MPa, respectively. The granite gneiss bedrock gave an average of about 6.0 MPa and 6.5 MPa for diametral and axial point load tests, respectively.

Diametral and axial point load tests performed on the biotite schist bedrock samples typically gave lower values compared to the biotite and granite gneiss. Diametral point load index results from the laboratory tests performed on the biotite schist gave values of 1.1 MPa and 1.2 MPa. Axial tests performed on the biotite schist gave values of 2.6 MPa and 1.8 MPa; however, the rock cores broke along the layers of schist and laboratory strength values should be considered to be conservative compared to actual in-situ conditions.

A summary of the average point load index values on the rock core from the ten boreholes where coring was carried out is shown in the following table.

Borehole (Drillhole) No.	Average Diametral Point Load Index, Is_{50} (MPa)			Average Axial Point Load Index, Is_{50} (MPa)		
	Biotite Gneiss	Biotite Schist	Granite Gneiss	Biotite Gneiss	Biotite Schist	Granite Gneiss
BH02	3.8	-	6.0	-	-	-
BH04	5.4	-	6.5	-	-	-
BH06	-	-	6.7	-	-	-
BH09	-	-	6.4	-	-	3.8
BH10	-	-	5.9	-	-	8.0
BH11	-	-	5.7	-	-	7.8
BH12	2.8	-	4.4	-	-	5.4
BH13	-	-	5.7	-	-	7.3
BH14	5.3	-		5.6	-	-
BH16	6.1	1.2		5.6	2.2	-

Based on the laboratory point load testing results and approximate field measurement techniques (see Drillhole Sheets), the estimated intact strength of the biotite and granite gneiss bedrock typically varies from strong (50 MPa < UCS < 100 MPa) to very strong (100 MPa < UCS < 250 MPa). The estimated intact strength of the biotite schist (encountered in BH16 only) is typically medium strong (25 MPa < UCS < 50 MPa) to strong (50 MPa < UCS < 100 MPa).

4.2.4 Groundwater Conditions

In general, the samples taken in the overburden boreholes were noted to be moist to wet. Details of the groundwater conditions and water levels observed in the open boreholes at the time of drilling are summarized on the Record of Borehole sheets following the text of this report. One piezometer was installed in BH05 to permit monitoring of water levels. The piezometer was sealed within the sand layer, directly above the bedrock surface at a depth of 3.2 m (Elevation 254.2 m). Details of the piezometer installation are shown on the Record of Borehole Sheet following the text of this report. The water levels encountered during drilling operations and in the piezometer are summarized in the table below.

Borehole	*Installation*	*Ground Surface Elevation (m)*	*Drilled Depth (m)*	*Water Level Depth (m)*	*Water Level Elevation (m)*	*Date*
BH1	Open Borehole	255.8	5.0	2.1	253.7	May 18, 2005
BH2	Inside Augers	253.8	14.0	4.0	249.8*	May 18, 2005
BH3	Open Borehole	255.1	6.3	1.7	253.4	May 17, 2005
BH4	Open Borehole	253.8	11.3	0.5	253.3	May 16, 2005
BH5	Piezometer	257.4	3.2	dry	dry	Sept. 30, 2005 Nov. 10, 2005
BH6	Open Borehole	257.6	10.4	3.8	253.8	May 19, 2005
BH7	Open Borehole	255.8	2.0	dry	dry	May 19, 2005
BH8	Open Borehole	257.3	1.9	dry	dry	May 19, 2005
BH9	Open Borehole	257.7	6.3	dry	dry	Nov.15, 2005
BH10	Open Borehole	257.8	10.4	6.3	251.5	Nov. 14, 2005
BH11	Open Borehole	257.5	9.6	5.2	252.3	Nov. 15, 2005
BH12	Open Borehole	257.5	6.9	dry	dry	Nov. 16,2005
BH13	Open Borehole	254.3	9.0	1.2	253.1	Nov. 14, 2005
BH14	Open Borehole	254.1	9.2	3.2	250.9	Nov. 11, 2005
BH15	Open Borehole	254.0	6.9	2.3	251.7	Nov. 11, 2005
BH16	Open Borehole	254.1	12.2	4.8	249.3*	Nov. 10, 2005
BH17	Open Borehole	254.4	12.3	3.0	251.4	Nov. 17, 2005

*water level taken inside hollow stem augers prior to rock coring; therefore, the measured water level may not represent stabilized ground water condition.

Groundwater elevations typically varied from 253.3 m to 253.8 m during the investigation performed in May 2005; however, a groundwater elevation of 249.8 m was measured in BH02 which may not represent the stabilized groundwater condition. During the supplemental investigation performed in November 2005, groundwater elevations typically varied from 250.9 m to 253.1 m; however, a groundwater elevation of 249.3 m was measured in BH16 which may not represent the stabilized groundwater condition. The water levels may indicate perched water conditions within the cohesionless soils on top of the sloping bedrock surface. The groundwater levels in November were typically about 1 m to 2 m lower than the water levels measured in May; thus, it should be noted that groundwater conditions in the area are subject to seasonal fluctuations.

4.3 Closure

The field technician supervising the drilling program was Mr. Suresh Bainey. This report was prepared by Ms. Shannon Palmer, EIT and Mr. Kevin J. Bentley, P.Eng., a geotechnical engineer, and reviewed by Ms. Anne S. Poschmann, P.Eng., and quality control review was provided by Mr. Fintan J. Heffernan, P.Eng., a Designated MTO Contact for Golder.

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PART B

FOUNDATION DESIGN REPORT
DETAIL DESIGN
HIGHWAY 11 AND MUSKOKA ROAD 169 UNDERPASS STRUCTURE
HIGHWAY 11 / MUSKOKA ROAD 169 INTERCHANGE
GRAVENHURST, ONTARIO
G.W.P 314-00-00

5.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides recommendations on the foundation aspects of the proposed Highway 11 and Muskoka Road 169 underpass structure. The recommendations are based on interpretation of the factual geotechnical data obtained from the boreholes advanced during the subsurface investigation carried out at the site.

The interpretation and recommendations provided are intended only to provide the designers with sufficient information to assess the feasible foundation alternatives and to design the proposed structure foundations. As such, where comments are made on construction they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods, scheduling and the like.

5.1 General

It is understood that a new two-span structure is to be constructed over the proposed Highway 11 realignment with abutments located on the east and west sides of the new highway. Highway 11 in this area will consist of a four-lane freeway, an on-ramp west of the southbound lanes (SBL), an on-ramp east of the northbound lanes (NBL), and a grass median separating the NBL and SBL. A centre pier is to be located within the proposed grass median. The proposed underpass structure is to be constructed at about Station 11+200 along the Highway 11 NBL and from about Station 9+950 to Station 10+060 along the proposed Muskoka Road 169 centreline chainage. The span lengths are to be 53 m long and 42 m long and the bridge width is to be about 13 m.

It is important to note that, based on the steeply sloping bedrock encountered during the initial investigation (BH01 to BH08), consideration was given to lengthening or moving the bridge in an attempt to locate more favourable abutment foundation conditions. A supplemental investigation was performed (BH09 to BH17) to further define the bedrock conditions at the proposed foundation locations and to determine if more favourable foundation conditions exist east and west of the as proposed abutment locations. The results of the supplemental investigation confirmed that the steeply sloping bedrock surface was still present to the east of the east abutment. West of the west abutment, the slope of the bedrock was not as steep but is still sloping. As a result, it was considered that lengthening or moving the bridge and shifting the abutment locations will not provide a substantial practical or feasible advantage from a geotechnical perspective.

The existing grade/ground surface at the underpass site ranges from about Elevation 254 m to 260 m. The centre pier and southern half of the east abutment are located in what appears to be an old sand pit which is about 3 m to 4 m lower than the surrounding area; thus, the west abutment location and the north side of the proposed structure site are typically at a higher elevation. There are exposed bedrock outcrops located north and east of the site as shown on Drawing 1.

The overburden soils at the site generally consist of sand, silty sand and sand and silt deposits, underlain by gneissic bedrock. There are typically trace to some amounts of organic material or topsoil present within about 0.3 m of the ground surface. There are isolated areas of very loose sand within the upper 1 m of the surficial material; however, the cohesionless deposit is generally compact to dense. The groundwater level in the sand, silty sand to sand and silt was found at about Elevation 254 m in the Spring and about Elevation 252 m in the Fall. The sand, silty sand to sand and silt overburden soils are typically underlain by strong to very strong biotite gneiss and granite gneiss bedrock of fair to excellent quality; with the exception of BH04 (at the proposed pier location) and BH16 (east abutment location). In BH04, the upper portion (about 0.1 m) of bedrock contains zones of broken rock of poor quality. In BH16, the bedrock consists predominantly of medium strong to strong biotite schist with zones of broken rock of poor quality in the upper 1 m of bedrock.

In the area of the proposed centre pier (BH04), a layer of cobbles / boulders was encountered directly above the bedrock surface. The probable cobble / boulder layer extended from a depth of 7.3 m to 8.4 m below ground surface (i.e. about 1.1 m thick).

Based on the General Arrangement drawing provided by MRC, the proposed pile cap (or shallow foundation) base for the west abutment, east abutment, and centre pier are to be founded at approximate elevations of 254.2 m, 254.0 m, and 249.2 m, respectively. In order to accommodate the above proposed founding elevations, excavations (relative to the existing ground surface at the time of our investigation) within the cohesionless deposits up to about 3.6 m and 1.8 m deep at the west and east abutment locations, and up to about 6 m deep at the centre pier location, are required. If final road and ditch grading is provided prior to construction of the foundations, the founding elevations at the west and east abutment locations can be graded as part of the construction works (i.e. no further sub-excavation); however, subexcavation up to about 3 m deep (relative to the final grading) at the centre pier location will still be required. At the east abutment, about 1.8 m of excavation into the bedrock will also be required to reach a founding elevation of 254.0 m based on the results of test pit TP1. Despite what foundation option is chosen, it is recommended that the proposed highway be graded prior to excavation/construction of the foundations in order to limit subexcavation depths and reduce dewatering efforts. Due to the undulating bedrock surface, depth of cut required, and high groundwater table, it is recommended that the base of the pile cap or shallow foundation be as high as possible.

5.2 Bridge Foundation Options

Various alternatives for the abutment and pier foundations are considered in the sections below and a summary of these alternatives is presented in Tables 2 and 3 following the text of this report. Tables 2 and 3 outline the advantages, disadvantages, relative costs, and risks/consequences associated with each alternative. It should be noted that the development of the tables included extensive communication between MRC, MTO Foundations and Structural Sections, Golder, and various deep foundations contractors and provided the screening process which led to the preferred foundation alternatives from a geotechnical / foundation perspective.

It should be noted that there is significant variation in bedrock surface elevation across the site and more importantly, across the footprint of each foundation unit. This variability in bedrock surface presents some difficulties with foundation construction. Consideration was given to shifting the foundation locations by increasing the bridge span lengths; however, the results of the supplemental field investigation indicated that the sloping bedrock was also present at the proposed relocated foundations.

It is considered that a combination of spread footings on bedrock and small diameter drilled pipe piles (concrete filled) socketed into the bedrock for the support of the bridge abutments are the preferred alternative from a geotechnical / foundation perspective. At the pier location, spread footings founded on mass concrete (likely tremied concrete methods) with extensive excavation and dewatering effort is the preferred alternative from a geotechnical /foundation perspective.

5.2.1 Option No. 1 - Spread Footings founded on compact to dense sand to sand and silt deposit or "perched" on granular fill

Consideration could be given to the use of spread footings supported on the typically compact to dense sandy deposits and/or perched on engineered fill. Due to the depth and steeply sloping bedrock at the site, however, some areas of the footings will be founded partially on bedrock. In addition, the thickness of granular deposits below the footings is variable across the footing length which will result in differential settlement.

The following table summarizes the spread footing founding elevations (as per the current design), founding soil or rock and, if applicable, the thickness of the underlying compressible founding soil above the bedrock.

Foundation Element	Proposed Footing Elevation (depth below existing ground)		Founding Soil or Bedrock (soil thickness above bedrock, m)	
	North Side (m)	South Side (m)	North Side	South Side
West Abutment	254.2 (3.6)	254.2 (3.3)	Compact sand (3.9)	Bedrock (0)
Center Pier	249.2 (5.9)	249.2 (4.8)	Dense sand and silt (0.3)	Dense sand and silt (3.8)
East Abutment	254.0 (2.7)	254.0 (*-0.2)	Bedrock (0)	Very dense to very loose silty sand and sand (11.2)

*Existing ground surface is actually below founding elevation.
Note: depths and elevations shown represent the maximum anticipated range within each foundation element.

The following details regarding shallow spread footings should be considered:

- Differential settlement between the north and south sides of each foundation is a major concern due to the varying thickness of the overburden soils and makes this option impractical for "settlement sensitive" structures. However, differential settlements can be reduced by excavating (blasting) the bedrock and replacing with approved granular material;

- Some blasting of the existing bedrock will be required on the south side of the west abutment and the north side of the east abutments to reach the proposed founding elevation and obtain a level surface; and

- Dewatering for groundwater control at the location of the centre pier during foundation construction and possibly both abutment locations, depending on the timing of the construction of the highway.

To reduce the potential differential settlement at the abutment locations, consideration could be given to subexcavating (i.e. blasting and removing) the upper portion of the bedrock and replacing with granular fill to allow for perched spread footings. However, difficulties associated with dewatering for excavation of the bedrock, and replacement and compaction of the granular materials in the dry should be anticipated if the highway grades are not subexcavated and graded to allow for gravity drainage of the abutment foundation areas prior to excavation. At the east abutment location, even if the proposed highway grades are met prior to abutment foundation construction, bedrock subexcavation and granular backfill up to 7 m below the groundwater level is needed to provide a level bedrock surface and consistent engineered fill thickness (i.e. to prevent differential settlement within the foundation footprint). As a result, this option is not considered practical.

5.2.2 Option No. 2 – Spread Footings Founded on Bedrock and/or on Mass Concrete Placed on Bedrock

Consideration could be given to subexcavating each foundation footprint to bedrock and replacing with controlled low strength material (i.e. mass concrete) up to the founding elevations.

The following table summarizes the proposed spread footing founding elevations (as per the current design), estimated bedrock elevations and corresponding estimated thickness of mass concrete placement.

Foundation Element	Proposed Footing Elevation		Bedrock Elevation		Estimated Mass Concrete Thickness	
	North Side (m)	South Side (m)	North Side (m)	South Side (m)	North Side (m)	South Side (m)
West Abutment	254.2	254.2	250.3	254.2	3.9	0
Center Pier	249.2	249.2	248.9	245.4	0.3	3.8
East Abutment	254.0	254.0	254.0	242.8	0	11.2

Note: depths and elevations shown in the above table represent the maximum anticipated range within each foundation element.

Referring to the table and the results of the field investigation, this method is not feasible at the east abutment location where bedrock and groundwater was encountered up to about 11 m and 10 m below existing ground surface, respectively. This option could be considered at the west abutment and pier location. A NSSP should be made in the Contract Document for additional mass concrete placement to accommodate variations in the bedrock surface; an example is provided in Appendix B.

The groundwater table was generally around Elevation 253 m to 254 m during drilling in May and around Elevation 251 m to 253 m in November. As a result, subexcavation for spread footings will require dewatering in the sand, silty sand to sand and silt deposit, especially at the centre pier location. A combined dewatering system using sheetpiling and wells or sheetpiling and tremie concrete methods may be required (see Section 5.9.2). Difficulties achieving a watertight seal between the sheetpiles and the sloping bedrock or cobbles/boulders (encountered within the pier footprint) should be anticipated.

Dewatering efforts can be minimized at the west abutment foundation location if the proposed Highway 11 road is graded and perimeter ditches sloped to final design elevations prior to construction of the abutment foundations. This will allow for gravity drainage of the silts and sands at the abutment locations and to a lesser extent at the pier location.

Rock blasting may be required at shallow depths to ensure a level founding surface where spread footings or mass concrete is to be used. Below the groundwater level, spread footings / mass concrete could be anchored using rock dowels.

5.2.3 Option No. 3 – Combined Spread Footings Founded on Bedrock and Drilled Piles Socketed into Bedrock

Alternatively, consideration could be given to combining spread footings founded on bedrock with drilled piles socketed into bedrock. This option allows for optimizing open cut excavation and deep foundation options in order to minimize extensive subexcavation and placement of mass concrete below the groundwater table. Due to the variable bedrock surface encountered at the borehole locations during the drilling investigation, the shallow and deep foundations could be constructed in a stepped pattern to follow the general contour of the bedrock surface and minimize rock cut/blasting or extensive subexcavation. The following elevations may be assumed at each foundation location:

Foundation Element	Estimated Depth to Bedrock from underside of spread footing/pile cap (recommended foundation type)		Estimated Bedrock Elevation	
	North Side	South Side	North Side	South Side
West Abutment	3.9 m (mass concrete and spread footing or drilled pile)	0 m (spread footing)	250.3 m	254.2 m
Centre Pier	0.3 m (mass concrete and spread footing)	3.8 m (mass concrete and spread footing or drilled pile)	248.9 m	245.4 m
East Abutment	0 m (spread footing)	11.2 m (drilled pile)	254.0 m	242.8 m

Note: Depths and elevations shown represent the maximum anticipated range within each foundation element

After conversations with the designer, if combined spread footings and drilled piles are considered for design, it can be assumed that about 55% to 65% of the abutment foundations will consist of drilled piles. The actual footing area supported on spread footings and drilled piles will vary depending on field conditions. A NSSP should be made in the Contract Document for additional mass concrete placement to accommodate variations in the bedrock surface; an example is provided in Appendix B.

Rock blasting may be required at shallow depths to ensure a level founding surface where spread footings are used. Alternatively, the spread footings could be anchored using rock dowels. The drilled piles located on sloping bedrock surface should be socketted into the bedrock to maintain a level base or anchored using rock dowels or equivalent. The sloping bedrock will also present difficulties in the socketting as well as the drilling for the rock anchors since a seal will be required at the base of the caisson or drilled pile to prevent inflow of the surrounding sands and silts during cleaning, rock drilling and placement of concrete. As a result (and as described in detail in Section 5.5), small diameter (i.e. 324 mm O.D.) pipe piles (concrete filled) installed using specialized down-the-hole hammer drilling techniques are preferred in lieu of larger diameter caissons which are less likely to achieve the required socket and/or water tight seal for rock anchors within the hard, steeply sloping bedrock without more specialized equipment. As a result, if higher capacities are needed (see Section 5.5.1), larger diameter piles/caissons would not be economical and other foundation alternatives should be investigated.

As discussed in the previous section, the groundwater table was generally around Elevation 253 m to 254 m during drilling in May and around Elevation 251 m to 253 m in November. As a result, subexcavation for spread footings will require dewatering in the sand, silty sand to sand and silt deposit, especially at the centre pier location. A combined dewatering system using sheetpiling and wells or sheetpiling and tremie concrete methods may be required (see Section 5.9.2). Dewatering efforts can be minimized at the abutment foundation locations if the proposed Highway 11 road is graded and perimeter ditches sloped to final design elevations prior to construction of the abutment foundations. This will allow for gravity drainage of the silts and sands at the abutment locations and to a lesser extent at the pier location.

5.2.4 Option No. 4 – Steel H-piles Driven to Found on Bedrock (Subexcavate Bedrock and Replace with Granular Fill where necessary)

This option involves excavating and replacing bedrock with granular backfill to achieve minimum pile lengths and could be adopted to support either a conventional or integral abutment type structure. The minimum required pile lengths are about 3 m and 6 m for conventional and integral abutments, respectively. The rock excavation could be completed in conjunction with roadway/ditching blasting; however this option would require mass subexcavation of bedrock and replacement with granular fill below the water level. At the pier location, bedrock would need to be removed up to 5 m below the final road grade and estimated groundwater level (i.e. the final stormwater ditch elevation) if construction of the roadway was completed prior to the construction of the pier foundation. Otherwise, bedrock would need to be removed up to 7 m below the existing groundwater level. As a result, this option is not considered practical and not the preferred alternative for any of the foundations.

The use of steel H-piles would minimize differential settlements encountered after construction; however, considerable bedrock blasting/removal and backfilling with select granular material (minus 50 mm size) will be required. The following table provides estimated bedrock subexcavation depths in order to provide a minimum 3 m embedment for the steel H-piles below the underside of the proposed pile cap elevation.

Foundation Element	**Proposed Underside of Pile Cap Elevation (m)**	**Estimated Thickness of Bedrock Sub-excavation Required**		**Estimated Pile Tip Elevation (i.e. Final Bedrock Surface Elevation, m)**	
		North Side	**South Side**	**North Side**	**South Side**
West Abutment	254.2	n/a	3.0	250.3	251.2
Center Pier	249.2	2.6	0.3*	246.2	245.4
East Abutment	254.0	5.7	n/a	251.0	242.8

Notes:

n/a - indicates bedrock surface is greater than or equal to 3 m below underside of pile cap.

Depths and elevations shown represent the maximum anticipated range within each foundation element.

*Cobbles/boulders present within 3 m below underside of pile cap elevation will need to be removed based on BH04

The steeply sloping bedrock surface at all founding locations will present some difficulties with seating of the piles, particularly at the east abutment. In this regard, piles should be fitted with appropriate rock points (i.e. Titus "Rock Injector Design", Oslo Points as per OPSD 3000.201 or equivalent) but even with the rock points, there may be difficulties due to the combination of steeply sloping bedrock and relatively thin overburden soils (i.e. short pile embedment lengths). Based on approximate founding levels provided by MRC and assuming bedrock subexcavation and granular backfill to provide a minimum 3 m pile embedment, the proposed pile embedment lengths at the west, east and central pier foundation footprints would range from about 3 m to 4 m, 3 m to 11 m, and about 3 m to 4 m, respectively. As a result, this option is not preferred from a foundations perspective.

5.2.5 Option No. 5 – Combination of Spread Footings and Steel H-piles driven to found on bedrock

Another option that could be considered for support of the abutments (less likely at the pier) is combining spread footings and steel H-piles founded on bedrock. Shallow spread footings may be constructed on bedrock where driven pile embedment lengths are less than 3 m. The estimated founding elevations for spread footings and piles would be the same as those shown in the table for Option No.3; however, driven steel H-piles founded on bedrock would be used instead of drilled piles socketed into bedrock. After conversations with the designer, for design purposes, it can be assumed that about 55% to 65% of the abutment foundation will consist of piles.

As discussed in Option No. 3, the steeply sloping bedrock surface at all founding locations will present some difficulties with seating of the piles, particularly at the east abutment. In addition, a layer of cobbles/boulders and broken rock was encountered at the pier location (BH04) which will also present difficulties with seating of the piles. In this regard, all piles should be fitted with appropriate rock points (i.e. Titus “Rock Injector Design”, Oslo Points as per OPSD 3000.201 or equivalent) but even with the rock points, there may be difficulties.

5.2.6 Option No. 6 – Micropiles

Consideration was given to using micropiles for support of the abutments and pier location. For the purpose of this report, a micropile is defined as a small-diameter (typically less than 300 mm), drilled and grouted replacement pile that is typically reinforced. Contrary to conventional pipe piles (concrete filled) / caissons where most of the applied load is resisted by the reinforced concrete, micropile structural capacities rely on high-capacity steel elements (typically threaded bars or reinforcing steel) to resist most or all of the applied load. The special drilling and grouting methods used in micropile installation allow for high grout/ground bond values along the grout/ground interface. The grout transfers the load through friction from the reinforcement to the ground in the micropile bond zone in a manner similar to that of ground anchors. Due to the small pile diameter, any end-bearing contribution in micropiles is generally neglected. Vertical micropiles may be limited in lateral capacity and cost effectiveness.

For this site, considering bedrock is generally encountered at shallow depth, there is not enough overburden thickness to develop sufficient axial capacity. As a result, the micropiles would require socketing into the bedrock. As previously mentioned, a threaded bar or reinforcing bar is typically placed into the cased borehole for both structural and lateral support and the borehole casing subsequently removed. As a result, lateral capacities are based entirely on the bending stiffness of the steel bars with limited resistance offered from the grouted zone. As a result, this option is not preferred from a foundations perspective.

For higher axial and lateral loads, steel-pipe reinforcement can be used. In fact, Option No. 3 is basically large diameter micropiles (we have referenced this system as “drilled pipe piles (concrete filled)”) combined with spread footings where bedrock was encountered at or near the founding level. Refer to Option No. 3 for details.

5.2.7 Option No. 7 – Caissons

Consideration was given to using caissons for support of the abutments and pier location. Due to the steeply sloping bedrock, the presence of cobbles and boulders with the subsoils and the potential difficulties associated with achieving a seal at the base of the caisson (i.e. groundwater inflow), caissons are not a feasible option. Based on conversations with several deep foundations contractors, large diameter caissons (diameter greater than or equal to 0.75 m) socketed into the sloping bedrock are not considered economical.

5.3 Spread Footings

The bridge abutments and pier may be supported on shallow spread footings founded on the compact to dense sandy deposits and/or properly prepared biotite/granite gneiss bedrock at the founding elevations provided in the tables presented in Section 5.2.1 and 5.2.2, respectively.

For design of spread footings on bedrock, based on the borehole results and exposed bedrock outcrops, there is high variability in the bedrock surface elevation within the limits of each foundation element – particularly at the east abutment. For the spread footing option, all loose or fractured rock at the bedrock surface will need to be subexcavated and removed which may result in lower founding elevations than those shown in the table in Section 5.2.2. Ideally, where bedrock is encountered, spread footings should be placed either directly on the properly prepared bedrock surface or placed on mass concrete constructed on the properly prepared bedrock surface which should minimize the bedrock excavation difficulties. If any areas will require the employment of mass concreting, it will be necessary to clean, scale and remove any loose debris to ensure a proper bond to the bedrock. 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 5.3.2). All bedrock excavation within and near the footing areas should be carried out using line drilling and pre-shearing techniques to minimize shattering and over-break. Additional recommendations on bedrock excavation are provided in Section 5.10.

For design of spread footings on the native sand, silty sand to sand and silt soils, based on the design founding elevations, all topsoil and very loose to loose soils will need to be removed and all footings will be founded within the compact to very dense soils. At all foundation locations, particularly at the centre pier location and below the water table, a suitable dewatering/diversion scheme (as discussed in Section 5.9.2) will be required in order to maintain a dry and stable excavation especially during periods of high groundwater levels. The surficial sand, silty sand to sand and silt soils (which are present below the founding elevation) on the south side of the east abutment are not considered to be suitable for the subgrade support of bridge foundations or engineered fill materials that support the foundation. These surficial soils will need to be removed from within the proposed foundation footprint to a depth of about 0.5 m or until all loose soils or soils containing organics have been removed. The subexcavation should then be replaced with engineered fill (Granular ‘A’) to reach design founding elevations. The native soils and engineered fill materials are susceptible to disturbance by construction activity especially during wet or freezing weather and care should be taken to preserve the integrity of the materials. If the concrete for the footings on the native or engineered fill soil cannot be poured immediately after excavation and inspection, it is recommended that a working mat of lean concrete be placed in the excavation to protect the integrity of the bearing stratum.

5.3.1 Geotechnical Resistance

For the abutments and centre pier, spread footings may be placed on the native compact to very dense sand, silty sand to sand and silt deposit and/or the surface of the properly prepared biotite/granite gneiss bedrock using the design values provided in the table below.

Spread Footing Location	***Geotechnical Resistance***		***Geotechnical Resistance***	
	Native Sand, Silty Sand, Sand and Silt		***Bedrock***	
	ULS (kPa)	**SLS (kPa)**	**ULS (kPa)**	**SLS (kPa)**
West Abutment	800	300	10,000	n/a
Pier Location	800	300	10,000	n/a
East Abutment	800	300	10,000	n/a

According to the GA drawing, the proposed founding elevations for the centre pier, west and east abutments are to be at 249.2 m, 254.2 m and 254.0 m, respectively. To maintain these proposed elevations, the south side of the west abutment and the north side of the east abutment will be founded on gneissic bedrock; all other portions of footings would be founded on the native silty sand, silt and sand soils of varying thickness above the bedrock. The centre pier will be founded several metres below the groundwater level. If the pier is founded on the sandy overburden, full dewatering to below the founding level prior to excavation for the footing will be required to avoid loosening of the foundation soil.

It is important to understand that the SLS values provided above for the native sand, silty sand to sand and silt indicate up to 25 mm of settlement could occur; however, this settlement is expected to occur differentially within the limits of each foundation footprint, especially at the east and west abutment locations as well as between foundation units. As a result, spread footings on the native soils are not a preferred option. Based on conversations with the bridge designer, this option is not feasible given the bridge structure's low tolerance for differential settlement, especially across the width of the abutments.

Alternatively, to reduce differential settlements at the abutment locations, spread footings placed (or perched) on a compacted Granular 'A' core (after subexcavation and removal of the native soils and sloping bedrock) could be considered. For design, a factored geotechnical resistance at ULS of 900 kPa may be assumed for spread footings placed on the compacted Granular 'A' pad. The geotechnical resistance at SLS (for 25 mm of settlement) will depend on the thickness of the Granular 'A' pad and the consistency and thickness of the underlying soils; a value of 350 kPa may be assumed for design. If this "perched" abutment option is adopted for the design of the foundations, these resistances would have to be confirmed once the elevation of the abutment footing is known. The granular fill should be placed in maximum 200 mm loose lifts and uniformly compacted to at least 100 percent of Standard Proctor maximum dry density under foundations.

Shallow spread footings placed on bedrock may be designed based on a factored geotechnical resistance at Ultimate Limit States (ULS) of 10,000 kPa, as indicated in the table above. For footings placed on a mass concrete pad, the factored geotechnical resistance at Ultimate Limit States (ULS) is as given above for bedrock assuming that the strength of the concrete used to form the pad is at least 25 MPa. The geotechnical resistance at Serviceability Limit States (SLS) for 25 mm of settlement will be greater than the factored axial resistance at ULS, since the bedrock is considered to be an unyielding material; as such, ULS conditions will govern for this foundation type.

All loose, broken and/or fractured rock within the foundation footprint and at the footing level should be cleaned and scaled prior to replacement with concrete and in accordance with Special Provision No. 902S01. Difficulties removing the cobble/boulder layer encountered at the pier location (BH4) should be anticipated.

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

5.3.2 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the base of the concrete footings and the underlying soils or bedrock should be calculated in accordance with Section 6.7.5 of the *CHBDC*.

For cast-in-place concrete footings placed directly on the native soils or Granular ‘A’ pad, the coefficient of friction, tan δ, may be taken as 0.58 between the base of the concrete footings and native sand/silt soils, and as 0.62 between the base of the concrete footings and the compacted Granular ‘A’ pad. This represents an unfactored value; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating the horizontal resistance.

In the case of mass concrete placed on the bedrock surface, the design must check the sliding resistance between the base of the concrete footings and the top of the mass concrete, and between the base of the mass concrete and the bedrock. The coefficient of friction, tan δ, may be taken as 0.62 between the base of the concrete footings and mass concrete, and as 0.70 between the base of mass concrete/concrete footings and bedrock.

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

If necessary, the sliding resistance 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 intact rock mass is essentially as strong as or stronger than concrete, the design of the dowels in the rock may be handled in the same way as the dowel embedment into the concrete. This assumes that the unconfined compressive strength of the grout will be similar to that of the concrete. The dowels should have a minimum embedded length within the unfractured (intact) bedrock of 1 m, and the structural strength of the dowel and compressive strength of the grout should not be exceeded.

An Ultimate Limit State (ULS) design value of 400 kPa may be assumed for the grout-to-rock bond strength, based on applying a resistance factor of 0.4 (according to Table 6.6.2.1 of the CHBDC) to the ultimate bond strength of 1,000 kPa. The geotechnical resistance at Serviceability Limit State (SLS) for 25 mm of displacement will be greater than the factored resistance at ULS; as such, ULS conditions will govern for this installation. The upper 0.5 m of the bond length should be ignored in the calculation of required bond length since the rock near surface may be weathered or disturbed. The actual bond strength for the rock – grout interface may vary from the typical design value given and should be verified in the field. Dowels should be checked to ensure that the rock mobilized around the anchor can support the design load (i.e. check against conical rock mass failure). Closely spaced dowels should be checked for group interaction. If dowelling into bedrock is adopted at this site, an NSSP should be included in the

Contract Document to specify the installation, materials and testing of the dowels (an example is provided in Appendix B).

5.3.3 Frost Protection

For spread footings or mass concrete founded on the properly prepared intact gneiss bedrock at this site, frost susceptibility is not an issue. However, where founding subsoils consist of native sandy deposits or if perched abutments are being considered, footings should be provided with a minimum of 1.7 m of soil cover for frost protection. Alternatively, rigid insulation could be used to reduce the thickness of soil cover needed. As a guideline, one inch of rigid polystyrene foam insulation may be used for every 0.3 m reduction in soil cover. A layer of granular material (75 mm of Granular 'A') should be provided as bedding and cover where the insulation is placed adjacent to rock fill.

5.4 Steel H-Pile Foundations

The bedrock surface at the east abutment and centre pier slopes steeply from the north down to the south (about 0.7H:1V and 1.5H:1V, respectively) and the bedrock surface encountered at the west abutment slopes from the south down to the north (about 2H:1V). At the abutment and pier locations, the foundation may be partially or completely supported on steel H-piles driven to bedrock, depending on which option is chosen (i.e. partially combined with spread footings or whether bedrock is completely removed by blasting and replaced with granular material).

The estimated pile tip elevations range from 242.8 m at the east abutment to 251.2 m at the west abutment, as shown in the table in Section 5.2.4. As stated previously in Option No. 4 (Section 5.2.4), in order to provide a minimum pile embedment length of 3 m, as much as 5.7 m of bedrock will need to be subexcavated/blasted and replaced with granular fill in some areas. Based on the borehole information, the bedrock in about half of the total area of each foundation footprint will need to subexcavated and replaced with granular fill in order to achieve a minimum pile embedment of 3 m. Based on the design drawings and if bedrock removal and replacement with granular fill is carried out, the length of piles will range between about 3 m and 11 m, typically between about 3 m and 5 m. There should be provision made in the contract for dealing with varying pile lengths due to the variable bedrock surface.

A review of the borehole logs indicated that difficulty in augering to bedrock was only encountered in Borehole BH04; this is inferred to be due to the presence of cobbles/boulders and/or broken bedrock. Consideration must be given to potential difficulties driving the piles due to the presence of cobbles and boulders within the native deposits and the sloping bedrock at this site. As a result, piles should be fitted with appropriate rock points (i.e. Titus "Rock Injector Design", Oslo Points as per OPSD 3000.201, or equivalent). A NSSP should be included in the contract to address this issue and is included in Appendix B for reference. Pile installation and rock points should be in accordance with Special Provision SP903S01.

5.4.1 Axial Geotechnical Resistance

For steel HP 310 x 110 piles driven to refusal on the biotite/granite gneiss bedrock, a factored axial resistance at Ultimate Limit States (ULS) of 1,200 kN may be assumed for design. The ULS value of 1,200 kN has been reduced to account for the combination of the following factors at this site:

i) the potential for difficulties in dealing with the steeply sloping bedrock and potential for the piles sliding along the bedrock surface;

ii) to account for the relatively low pile embedment lengths (typically ranges between 3 m and 5 m, assuming bedrock subexcavation and granular backfill); and

iii) takes into consideration the weaker biotite schist that was encountered in BH16 and cobbles/boulders encountered in BH4.

The geotechnical resistance at SLS for 25 mm of settlement will be greater than the factored axial resistance at ULS, since the bedrock is considered to be an unyielding material; as such, ULS conditions will govern for this foundation type.

5.4.2 Resistance to Lateral Loads

Resistance to lateral loading can be derived using vertical piles, with enhanced support offered by battered piles, which is likely required at this site. The maximum pile batter should be 1H:3V in accordance with Section 3.1.3 of the MTO Structural Manual. If vertical piles are used, the resistance to lateral loading will have to be derived from the soil in front of the piles. If integral or semi-integral abutments are considered, there will also be a requirement for the piles to move sufficiently to accommodate the bridge deck deflections. However, given that pile embedment lengths are typically less than 5 m in most areas (3 m embedment in some areas), integral abutments are likely not a feasible option (i.e. a minimum 5 m embedment length is typically preferred for integral abutments).

The resistance to lateral loading in front of the pile may be calculated using subgrade reaction theory, where the coefficient of horizontal subgrade reaction, k_h (MPa/m) for pile width B (m), is based on the following equation for granular soils:

$$k_h = \frac{n_h z}{B}$$

where n_h is the constant of horizontal subgrade reaction (MPa/m);
z is the depth (m); and
B is the pile diameter (m).

The following table provides the recommended range for the value of n_h to be used in the structural analysis. The range in values reflects the variability in the subsurface conditions and values used will depend on the design elevation of the pile cap. Design values are provided for the full stratigraphic sequence at the site.

Location	*Soil Unit*	*n_h(MPa/m)*
West Abutment	Embankment fill, if applicable (assumed to be compacted Granular 'A' fill above groundwater level (above El. 254 m))	10
	Embankment fill or backfill in bedrock trench excavation, assumed to be compacted granular fill below the groundwater level (below El. 254 m)	6 to 8
	Existing very loose to very dense native sand and silt, silty sand and sand above groundwater level (above El. 254 m)	10 to 15
	Existing compact to very dense native sand and silt, silty sand and sand below groundwater level (below Elevation 254 m)	8 to 10
East Abutment	Embankment fill, if applicable (assumed to be compacted Granular 'A' fill above groundwater level (above El. 254 m))	10
	Embankment fill or backfill in bedrock trench excavation, assumed to be compacted granular fill below the groundwater level (below El. 254 m)	6 to 8
	Existing loose to dense native sand and silt, silty sand and sand above groundwater level (above El. 254 m)	12 to 15
	Existing very loose to very dense native sand and silt, silty sand and sand below groundwater level (below Elevation 254 m)	5 to 10

A maximum lateral resistance of 60 kN at ULS and 30 kN at SLS is recommended for vertical HP 310x110 piles driven with a minimum embedment length of 3 m within the existing compact to dense sandy soils or within compacted granular fill placed after bedrock sub-excavation. Higher lateral capacities can be achieved for greater pile embedment depths.

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

Pile Spacing in Direction of Loading (d = Pile Diameter)	*Reduction Factor (R)*
8d	1.0
6d	0.7
4d	0.4
3d	0.25

Reference: Foundations and Earth Structures – Design Manual 7.2, NAVFAC DM-7.2. Department of the Navy, Naval Facilities Engineering Command (1982).

The subgrade reaction reduction factor should be interpolated for pile spacings in between those provided in the above table.

5.4.3 Frost Protection

The pile caps should be provided with a minimum of 1.7 m of soil cover for frost protection. Alternatively, rigid insulation could be used to reduce the thickness of soil cover needed. As a guideline, one inch of rigid polystyrene foam insulation may be used for every 0.3 m reduction in soil cover. A layer of granular material (75 mm of Granular 'A') should be provided as bedding and cover where the insulation is placed adjacent to rock fill.

5.5 Drilled Piles

Consideration could be given to the use of drilled piles for the support of the abutments and pier. The range of estimated bedrock surface elevations that may be used for design are provided in the table shown for Option No. 3 (Section 5.2.3).

The drilled piles need to be socketted into the bedrock to achieve a level founding surface at the base of the pile and to minimize the potential for sliding along the inclined bedrock surface. In order to achieve a bedrock socket into the steeply sloping bedrock, specialized equipment and drilling techniques will be required. After conversations with several deep foundations contractors, the general procedure involves using smaller diameter (less than 324 mm in outer diameter) steel pipe piles advanced through the overburden soils and into the bedrock using down-the-hole hammer techniques. In general, the drilled pile system uses a four step process. The first step is to weld a non-salvageable ring (i.e. crown) to the end of a steel pipe pile that will be used to drill into the bedrock and allow rotation of the shoe without rotation of the steel pipe. The next step is to insert the pilot bit into the steel pipe pile, which locks into the crown by rotating clockwise. The next step involves drilling through the overburden and bedrock by rotating the lower part of the crown (called the driver) and the pilot bit while the upper part of the crown and the steel pipe casing do not rotate. The last step (after the steel pipe casing reaches the required bedrock socket depth) involves reversing the drill direction to unlock and retrieve the pilot bit, and leaving the steel pipe and non-salvageable crown in place. The steel pipe can than be filled with tremie concrete (if there is water inflow through the bedrock) and reinforcing steel added, if required.

The drilled pile excavations must be inspected by qualified geotechnical personnel to ensure that the founding stratum has been reached and is consistent with the design assumptions and that the base has been properly cleaned and is dry. In this regard, temporary liners (i.e. the steel pipe piles) will be required to permit downhole inspection.

5.5.1 Axial Geotechnical Resistance

The drilled piles will derive their axial resistance in part from end-bearing and in part from shaft friction. For this site, the majority of the resistance will be derived from base resistance. The factored axial geotechnical resistance at ULS that may be used for design are given in the table below:

Drilled Pile Type	***Socket / Anchor Details***	***Axial Resistance***	
		Bedrock	
		ULS	**SLS**
300 mm Diameter Drilled Pile (tremie concrete filled, 13 m thick steel pipe)	Nominal socketing into bedrock; however, small diameter rock dowel installed about 1.5 m into rock	1,200 kN	n/a
300 mm Diameter Drilled Pile (tremie concrete filled, 13 mm thick steel pipe)	Socketed a minimum 0.6 m into bedrock (measured from low side of sloping bedrock/pile interface)	*2,000 kN	n/a
324 mm Diameter Drilled Pile (tremie concrete filled, 13 mm thick steel pipe)	Socketed a minimum 0.6m into bedrock (measured from low side of sloping bedrock/pile interface)	*2,400 kN	n/a

*values depend on structural capacity of the pile and may need to be adjusted depending on final configuration, pipe steel grade, concrete strength, bedrock socket details, and reinforcing steel, if applicable.

For drilled piles founded in the bedrock, the resistance required to achieve 25 mm of settlement is greater than that given for ULS and therefore SLS conditions do not apply.

For larger diameter drilled piles (i.e. greater than about 324 mm diameter), an installation method similar to the system described previously would be required to achieve adequate socketing and in order to achieve larger axial resistance capacities. However, for larger diameter piles the steeply sloping bedrock becomes more difficult to excavate and requires more specialized equipment and construction techniques. As a result, larger diameter drilled piles or caissons would be uneconomical.

5.5.2 Resistance to Lateral Loads

The resistance to lateral loading developed by the soils in front of the vertical drilled piles, and the reductions due to group effects, may be determined as per Section 5.4.2.

5.5.3 Frost Protection

The pile caps should be provided with a minimum of 1.7 m of soil cover for frost protection. Alternatively, rigid insulation could be used to reduce the thickness of soil cover needed. As a guideline, one inch of rigid polystyrene foam insulation may be used for every 0.3 m reduction in soil cover. A layer of granular material (75 mm of Granular ‘A’) should be provided as bedding and cover where the insulation is placed adjacent to rock fill.

5.6 Earthquake Consideration

For seismic design purposes, the Site Coefficient, S, for this site in accordance with Section 4.4.6 of the CHBDC may be taken as 1.0, consistent with Soil Profile Type I.

5.7 Lateral Earth Pressures for Design

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, on the nature of the soils behind the backfill, on the magnitude of surcharge including construction loadings, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls.

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 Ontario Provincial Standard Specifications (OPSS) Granular ‘A’ or Granular ‘B’ Type II but with less than 5 per cent passing the 200 sieve should be used as backfill behind the walls. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3501.00 and 3504.00.

- 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 6.9.3. Compaction equipment should be used in accordance with Special Provision SP105S10. Other surcharge loadings should be accounted for in the design, as required.

- The granular fill may be placed either in a zone with width equal to at least 1.7 m behind the back of the wall stem (see Case I in Figure C6.9.1(l)(i) of the *Commentary to the CHBDC*) or 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 (see Case II in Figure C6.9.1(l)(ii) of the *Commentary to the CHBDC)*.

- For Case I, the pressures are based on the proposed embankment fill materials and the following parameters (unfactored) may be used assuming the use of Select Subgrade Material (SSM):

	SSM (sand fill)	**SSM (rock fill)**
Soil / rock unit weight:	20 kN/m^3	19 kN/m^3
Coefficients of static lateral earth pressure:		
Active, K_a	0.33	0.24
At rest, K_o	0.50	0.38
Passive, K_p	3.0	4.1

- For Case II, the pressures are based on the granular fill as placed and the following parameters (unfactored) may be assumed:

	Granular 'A'	**Granular 'B' Type II**
Soil unit weight:	22 kN/m^3	21 kN/m^3
Coefficients of static lateral earth pressure:		
Active, K_a	0.27	0.27
At rest, K_o	0.43	0.43
Passive, K_p	3.7	3.7

- If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design. The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure, may be taken as:

  - Rotation of approximately 0.002 about the base of a vertical wall;
  - Horizontal translation of 0.001 times the height of the wall; or
  - A combination of both.

- If semi-integral abutment design allows for movement of the bridge deck ends, passive earth pressures may be used in the geotechnical design of the structure. The movements required to fully mobilize passive pressure or resistance are much larger than those required to mobilize active pressure. In practice, movements may not be sufficient to mobilize full passive resistance; if this is the case, at-rest earth pressures should be assumed for geotechnical design. A resistance factor equal to 0.5 should be applied to the passive resistance (see Table 6.6.2.1, see *Commentary to the CHBDC*). The movement to allow passive pressures to develop within the backfill may be taken as:

- Rotation of approximately 0.1 about the base of a vertical wall;
- Rotation of approximately 0.02 about the top of a vertical wall;
- Horizontal translation of 0.05 times the height of the wall; or
- A combination of the above.

- Seismic loading will result in increased lateral earth pressures acting on the abutment stem and retaining walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure. According to Table A3.1.7 of the CHBDC, this site is located in Seismic Zone 1. The site-specific zonal acceleration ratio (A) for Gravenhurst is 0.05. Based on experience, for the thin overburden soils at the site and embankment heights of up to about 8 m, a 10 to 20 per cent amplification of the ground motion may occur, resulting in an increase in the ground surface acceleration from 0.05g to between 0.055g and 0.06g. The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio of A = 0.06.

- In accordance with Sections 4.6.4 and C.4.6.4 of the *CHBDC* and its *Commentary*, for structures which allow lateral yielding, the horizontal seismic coefficient, k_h, used in the calculation of the seismic active pressure coefficient, is taken as 0.5 times the zonal acceleration ratio (i.e. $k_h = 0.03$). For structures that do not allow lateral yielding, k_h is taken as 1.5 times the zonal acceleration ratio (i.e. $k_h = 0.09$). The seismic active and passive earth pressure coefficients are also dependent on the vertical component of the earthquake acceleration, k_v. Three discrete values of vertical acceleration are typically selected for analysis, corresponding to $k_v = +2.3k_h$, $k_v = 0$, and $k_v = -2/3$.

- The following seismic active earth pressure coefficients (K_{AE}) for the two cases (Case I and Case II) may be used in design; these coefficients reflect the maximum K_{AE} obtained using the k_h and three values of k_v as described above. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is flat.

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

	Case I (sand)	Case II	
		Granular A	Granular B Type II
Yielding wall	0.32	0.26	0.26
Non-yielding wall	0.37	0.30	0.30

Note : These CHBDC seismic K_{AE} values include the effect of wall friction ($\delta=\phi'/2$) and are less than the static values of K_a and K_o reported above for the very low zonal acceleration ratio for this site.

- The above K_{AE} values for yielding walls are applicable provided that the wall can move up to 250A (mm), where A is the design zonal acceleration ratio of 0.06. This corresponds to displacements of up to 15 mm at this site.

- The earthquake-induced dynamic active lateral pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e. an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

$$p = K\,\gamma' d + (K_{AE} - K)\,\gamma'\,H$$

Where: p is the total (static plus seismic) pressure distribution (kPa)
K is either the static active earth pressure coefficient (K_a) or the static at rest earth pressure coefficient (K_o);
K_{AE} is the seismic active earth pressure coefficient;
γ' is the effective unit weight of the soil (kN/m^3)
  - taken as soil unit weights given above for fill materials
  - taken as 19 kN/m^3 for the native materials

d is the depth below the top of the wall (m); and
H is the height of the wall above the toe (m).

- The following seismic passive earth pressure coefficients (K_{PE}) may be used in design of the bridge deck ends for semi-integral abutment design; these coefficients reflect the maximum K_{PE} obtained using the k_h and three values of k_v as described above. It should be noted that these seismic earth pressure coefficients assume that the back of the wall/deck is vertical and the ground surface behind the wall is flat.

SEISMIC PASSIVE PRESSURE COEFFICIENTS, K_{PE}

Case I (sand)	Case II	
	Granular A	Granular B Type II
4.6	6.8	6.8

- The earthquake-induced dynamic passive lateral pressure distribution, which is to be subtracted from the static passive earth pressure distribution, is a linear distribution with maximum pressure at the base of the wall and minimum pressure at the top (i.e. a triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

$$p = K_p \gamma' d - (K_{PE} - K_p) \gamma' H$$

Where: p is the total (static plus seismic) pressure distribution (kPa)
K_p is the static passive earth pressure coefficient;
K_{PE} is the seismic passive earth pressure coefficient;
γ' is the effective unit weight of the soil (kN/m^3);
- taken as soil unit weights given above for fill materials
- taken as 19 kN/m^3 for the native materials

d is the depth below the top of the wall (m); and
H is the height of the wall above the toe (m).

5.8 Approach Embankment Design and Construction

Based on the information provided on the General Arrangement Drawing for the site, the proposed top of grade for Muskoka Road 169 at the abutment structure locations is at about Elevation 261 m. The existing ground surface ranges from approximate Elevation 257.5 m to 258 m at the west approach embankment location, and ranges from approximately Elevation 254 m to 259 m at the east approach embankment location. As a result, the embankments are up to about 3.5 m and 7.3 m in height at the west and east approach embankments, respectively.

It should be noted that the proposed Highway 11 grade in the area of the approach embankment foreslopes involves cutting the existing grade by up to 5 m and 3 m, resulting in elevations as low as about Elevation 252.5 m and 250.5 m at the west and east abutment foreslope toe locations. As a result, the embankment foreslopes are up to about 8.5 m and 10.5 m in height in some localized areas at the west and east approach embankments, respectively. Although global stability for the higher foreslopes is not a concern, the surficial stability and other design considerations need be addressed as discussed in Section 5.8.4.

Based on the borehole results, the subgrade soils at the proposed approach embankment locations consist of overburden soils consisting of topsoil, sand, silty sand to sand and silt with trace to some organics/roots at the surface, in turn, underlain by bedrock. Bedrock is exposed at the ground surface immediately north and east of the east abutment location. All topsoil and organic matter should be stripped from below the approach embankment areas prior to fill placement.

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

5.8.1 Stability

Analyses were performed on the critical (i.e. highest) sections of the proposed new approach embankments to assess stability and liquefaction potential.

At the east approach area, bedrock ranges from visible outcrops at the ground surface to over 12.3 m below ground surface (BH17). At the west approach, bedrock ranges from about 1.9 m to 3.6 m below ground surface. The overburden soils, where present, are generally composed of compact to very dense cohesionless soils. For these soils, effective stress parameters were employed in the analysis assuming drained conditions using the results of the in situ Standard Penetration Tests (SPT) tempered by engineering judgment based on precedent experience in similar soils.

At all areas, the analyses assume that all topsoil, existing asphalt and underlying fill, and native soils containing organics (encountered at or immediately below the ground surface during field investigation operations) have been removed prior to construction of the new embankments. The piezometric conditions required in the analyses were based on the groundwater levels noted during drilling of the boreholes. In general, the overburden soils within the approach embankment areas were not saturated and groundwater was typically observed at about Elevation 254 m or below.

The following table summarizes the simplified stratigraphy and the associated strengths and unit weights employed for the different soil types in the approach areas. For the purposes of analysis, both earth fill and rock fill were considered for the construction of the approach embankments, and as indicated in the table below. Rock fill is assumed to have minimum side slopes at 1.25H:1V and the earth fill is assumed to have side slopes at 2H:1V. A discussion on the different fill types, with respect to stability, is provided in Section 5.8.1.1.

East and West Approach Embankments

Soil Type	Unit Weight (kN/m^3)	Strength Parameters
Rock Fill	19	$c' = 0$ kPa, $\phi' = 38^o$
Earth Fill (Sand and Gravel)	21	$c' = 0$ kPa, $\phi' = 35^o$
Compact to very dense Sand, Silty Sand to Sand and Silt	20	$c' = 0$ kPa, $\phi' = 28^o$ to 30^o

The results of the stability analyses (using the commercially available program SLOPE/W produced by Geo-Slope International Ltd.) for the two embankment fill options are summarized in the following table. At each abutment location, the highest (i.e. most critical) embankment section has been analyzed. The result of the stability analysis for the east approach embankment is shown in Figure 1. The minimum factor of safety is based on a deep-seated, global type failure surface that would impact the operation of the roadway.

<table>
<tr><th rowspan="2">Location</th><th rowspan="2">Embankment Height at Critical Section (m)</th><th colspan="2">Earth Fill Option</th><th colspan="2">Rock Fill Option</th></tr>
<tr><th>Recommended Side Slope Profile</th><th>Minimum Factor of Safety</th><th>Recommended Side Slope Profile</th><th>Minimum Factor of Safety</th></tr>
<tr><td>East Approach</td><td>7.3</td><td rowspan="2">2H : 1V</td><td rowspan="2">≥ 1.3</td><td rowspan="2">1.25H : 1V</td><td rowspan="2">≥ 1.3</td></tr>
<tr><td>West Approach</td><td>3.5</td></tr>
</table>

For the approach embankment heights being considered, the incorporation of a 2 m wide bench (or berm) into the uniform side slope profile is generally not required as per OPSD 202.010 and MTO Northern Region guidelines. However, locally at the east abutment foreslope, where the slope height is greater than 8 m for earth fill or 10 m for rock fill, a 2 m wide bench will be required.

5.8.1.1 Embankment Fill Types and Berm Requirements

Based on the anticipated embankment fill heights and existing soil conditions, either earth fill or rock fill embankment options may be considered. The different fill alternatives (i.e. earth fill and rock fill) provide relative advantages and disadvantages in terms of weight (i.e. driving force and applied load to founding subsoils / bedrock), construction cost and time, and ease of construction / availability.

It should be noted that the use of similar adjacent fill materials should be ensured to prevent problems caused by the migration of fines between dissimilarly graded fill types as well as potential variation in thermal effects related to different materials.

5.8.1.1.1 Earth Fill

The main advantage of using earth fill (i.e. sand and gravel) 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. For this project, acceptable earth fill is considered to be suitable locally available and/or imported, granular material.

For the earth fill option, the incorporation of a 2 m wide mid-height bench (or berm) into the uniform side slope profile is required for surficial stability wherever the embankment will exceed a height of 8 m.

5.8.1.1.2 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. In addition, rock fill will likely be available from the rock cuts proposed for the underpass, thus providing an advantage in cost. 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 and second year of construction.

For the rock fill option, the incorporation of 2 m wide berms (or successive benches) into the uniform side slope profile is required for surficial stability wherever the embankment will exceed a height of 10 m such that the uninterrupted rock fill slope never exceeds a height of 10 m (as per most recent MTO Northern Region guidelines).

5.8.2 Liquefaction Potential

The liquefaction potential of the soils below the approach embankments under seismic loading has been considered using the empirical method outlined in Section C.4.6.2 of the CHBDC Commentary, which correlates the cyclic resistance ratio of the soils with their normalized penetration resistance and fines content. Based on this assessment, and assuming a ground surface acceleration of 0.06 g, a factor of safety of greater than 3 against liquefaction is generally obtained for a magnitude 7.0 earthquake event. At the south side of the east abutment, where low SPT values were obtained at a depth of about 8 m (Elevation 246 m), a factor of safety of less than 1 is obtained within this deposit for a thickness of about 1 m. However, as previously mentioned, the low SPT values measured may be a result of the native material being forced into the augers by water pressure during drilling, and therefore, may not be representative of the typical conditions at depth. Total seismic settlements within the native soils are calculated to be up to about 100 mm based on analysis performed in accordance with Tokimatsu and Seed (1987); about 75% of this settlement is due to the low SPT values recorded below Elevation 247 m. Pseudo-static methods of embankment stability analysis indicate that a yield acceleration of approximately 0.12 g results in a factor of safety against side slope instability of 1.0. Based on this yield acceleration and the correlation proposed by Makdisi and Seed (1978), it is estimated that very little additional deformations (i.e. less than about 5 mm) of the embankment could result under the design earthquake event.

5.8.3 Settlement

Settlement analyses were performed on the critical sections of the proposed approach embankments. For these analyses, the critical sections are assumed to correspond to the greatest new embankment heights that support the highway, approximately 7.3 m at the east approach and about 3.5 m at the west approach. The unit weights and slope profiles for the embankment fill described in Section 5.8.1 were employed in the analyses.

As noted previously, within the east and west approach embankment areas, bedrock is sloping and ranges from shallow depth up to about 12.3 m below ground surface. The overburden soils are composed of cohesionless soils. Surficial deposits of topsoil were encountered at some of the investigated locations.

Provided that the topsoil and surficial soils containing organics is removed prior to the new embankment fill placement (as discussed in Section 5.8), long-term settlements of the new approach embankments, due to compression of the foundation soils, are expected to be small. For embankment fills constructed with rock fill, the majority of the settlement of the approach embankments is expected due to compression of the rock fill itself. Estimated total post-construction settlements are summarized in Table 4.

The following sections describe the estimated settlement of the foundation soils and the estimated settlements of the embankment fill due to the loading imposed by the new approach embankments.

5.8.3.1 Settlement of Cohesionless Foundation Soils

The immediate compression of the typically compact to very dense sand, silty sand and sand and silt subsoils encountered in the boreholes in the area of the approaches were modelled by estimating an elastic modulus of deformation based on the SPT 'N' values and correlations proposed by Bowles (1984) and Kulhawy and Mayne (1990).

The following table presents the results of the estimated settlements of the foundation soils as a result of the new embankment construction in the area of the approaches.

Area of Embankment (Muskoka Road 169 Chainage)	***Approximate Location***	***Maximum New Embankment Thickness* (m)***	***Estimated Settlement of Foundation Soils (mm)***
West Approach (9+935 to 9+960)	North Side South Side	3.5 3.0	up to 25 up to 20
East Approach (10+050 to 10+075)	North Side South Side	2.0 7.3	*0 to 15 up to 75

Notes : *estimated settlement of fill embankment placed on bedrock is zero

These settlements are expected to occur rapidly (i.e. during or shortly after construction) in response to the filling based on the granular nature of the native soils as indicated by the results of the grain size distributions.

It is noted that these settlements are conditional on the topsoil and organic soils being stripped and removed from the area of the embankment footprint prior to fill placement.

5.8.3.2 Settlement of Rock Fill

If rock fill is used for the construction of the embankments, in addition to the settlement due to compression of the foundation soils described above, there will be settlement due to compression of the rock fill itself. Settlement of the rock fill depends on the type of rock and on the method and sequence of placement and compaction of the fill. Assuming that the rock fill is not end dumped in its final position and is placed in accordance with the requirements as outlined in the Special Provision 206S03 (dated January 2004), the settlement of the newly placed rock fill is expected to be relatively small. In general, it is estimated that for the granitic gneiss rock fill likely to be used at this site, for the up to 7.3 m high approach embankments, the settlement of the rock fill will be about 1% of the new effective height of rock fill. Estimated total and differential settlements within the approach embankments are shown in the table below.

Area of Embankment (Muskoka Road 169 Chainage)	*Approximate Location*	*Maximum Rock Fill Embankment Thickness (m)*	*Estimated Maximum Settlement of Rock Embankment (mm)*	*Estimated Differential Settlement (mm)*
West Approach (9+935 to 9+960)	North Side South Side	3.5 3.0	35 30	less than 5
East Approach (10+050 to 10+075)	North Side South Side	2.0 7.3	20 75	less than 55

It is anticipated that the majority (approximately 60%) of the total settlements shown above will occur within the first year following construction. It is estimated that about 30% of the total settlements will occur within 6 months following construction.

5.8.3.3 Settlement of Earth Fill

Where earth fill (granular fill) is used for the construction of the embankments, the settlement of the approved new embankment fill itself is expected to be less than 25 mm. The majority of settlement will occur during construction.

5.8.4 Mitigation of Approach Embankment Settlement and Stability

Based on the design drawings and conversations with the designer, the approach slabs will be supported directly on the approach embankment fill, and the slabs cannot tolerate more than about 50 mm of settlement relative to the top of the abutment walls. As a result, it is recommended that earth fill (i.e. compacted granular fill) be used beneath the plan limits of the approach slab to limit settlement of the embankment fill to less than 25 mm as described in the previous section and as shown on Table 4. The granular fill can be tapered beyond the approach slab footprint (in the direction away from the abutment) to allow for transition to rock fill (similar to OPSD 3501.000). Generally rock fill placed above earth (granular) fill is preferred to prevent loss of finer material. However, if earth (granular) fill is placed above rock fill, the surface of the rock fill should be compacted and chinked prior to placing the granular material on top (as per SP206S03, January 2004, Sect. 206.07.08). Within the approach slab footprint and any settlement sensitive areas, the earth (granular) fill placed above rock fill should consist of Granular 'A' or Granular 'B' Type II material (OPSS 1010). Granular 'B' Type II fill is preferred as more loss of material through the voids is expected if Granular 'A' fill is used.

Although the use of granular earth fill mitigates settlement issues related to the approach slab, there is not enough distance between the crest of the embankment and the minimum clearance required at the embankment toe (i.e. adjacent to Highway 11) at the foreslope location to allow for the required 2H:1V side-slope profile for earth fill. In order to achieve steeper side-slopes while maintaining earth (granular) fill below the approach slab, a detail similar to that shown in Figure 2A could be incorporated into the design. Figure 2A and 2B shows typical sections at the east abutment approach slab location which uses temporary earth fill side-slopes at 1.5H:1V, covered with rock fill having permanent side-slopes at 1.25H:1V.

Based on the design drawings, the proposed span lengths require embankment foreslopes to act as one side of the drainage ditch at both abutment locations. As a result, we understand that rock protection is needed at the base of the foreslopes. Using a detail similar to that provided in Figure 2B (which is compatible with the detail shown in Figure 2A) will result in rock protection (i.e. rock fill) at the foreslopes, sloped at 1.25H:1V with a minimum thickness of 500 mm. As a result, the rock fill allows for steeper foreslopes which provide the minimum clearances required and also provides rock protection within the drainage ditches. The rock fill used along the drainage ditch should be sized according to the hydraulic conditions.

5.8.5 Subgrade Preparation and Embankment Construction

The existing native subsoils are considered to be an appropriate subbase for the proposed approach embankments; however, prior to the placement of any fill, all surface and near surface layers of topsoil/organic deposits and any softened or loosened soils should be stripped from the plan limits of the proposed works and the remaining subgrade soils should be proof-rolled, where possible.

Table 4 summarizes the recommended fill composition type(s) to be placed for the embankments, the locations and depth of topsoil/organics, the recommended side slope profiles, the requirements for side berms, the anticipated differential settlements, platform widenings (in accordance with NRE 98-200 and due to the unique embankment construction required to minimize approach slab settlements), and the recommended method of removal of topsoil/organics. Although the platform widening is not needed for foundation/settlement reasons, it may be required for future overlays.

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

5.8.6 Removal of Organics / Asphalt

Based on the subsurface information obtained during the field investigation, topsoil and native sandy soils containing organics (i.e. roots and wood pieces) was typically encountered within about 0.5 m below ground surface. Thicker organic deposits can be expected in some areas of the new approach embankments that are located within densely treed areas (i.e. at the south side of the west abutment). These organic layers should be stripped from the plan limits of the approach areas prior to fill placement. Where encountered, the existing asphalt (BH09) and any underlying fill or soil containing organics should be removed from the plan limits of the approach embankment.

5.8.7 Embankment Fill Placement

If earth fill (i.e. granular fill meeting OPSS 1010 Select Subgrade Material) is to be used for construction of the new embankments, placement of all granular fill material should be carried out in accordance with SP 206S03 (dated January 2004), in regular lifts with loose thickness not exceeding 300 mm, and be compacted to at least 95 percent of the Standard Proctor maximum dry density. The final lift prior to placement of the granular sub-base or base course should be placed and compacted to current MTO requirements for pavements. Inspection and field density testing should be carried out by qualified geotechnical personnel during all earth fill placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved. Side slopes for exposed earth fill embankments should be no steeper than 2H:1V. For temporary earth fill embankments that are to be covered with rock fill (see Figure 2), side slopes should be no steeper than 1.5H:1V and should be protected from surficial erosion until the rock fill cover is provided.

If rock fill is used for the construction of the new embankments, placement of all rock fill material should be carried out in accordance with the requirements as outlined in the Special Provision SP 206S03 (dated January 2004). 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. Voids and bridging shall be minimized by blading, dozing and ‘chinking’ the rock to form a dense, compact mass. Side slopes for rock fill embankments should be no steeper than 1.25H:1V.

Vegetation cover should be established on all permanent soil slopes to protect embankment fill against surficial erosion.

5.9 Design and Construction Considerations

5.9.1 Excavations

The proposed Highway 11 road and ditch grades will require permanent cuts within the native cohesionless soils up to about 5 m below the existing ground surface and up to about 3.5 m below the current groundwater level. Consequently, if the proposed Highway 11 roadway and drainage ditches are graded prior to construction of the underpass foundations, the current groundwater level is anticipated to gravity drain down to the proposed drainage ditch elevation of 252.5 m at the west abutment and 250.5 m at the east abutment (about 1.5 m to 3.5 m lower than the current groundwater level), provided positive drainage is provided away from the bridge location. A discussion of groundwater control during excavation is discussed in the next section.

All excavations must be carried out in accordance with the latest edition of the Ontario Occupational Health and Safety Act (OHSA) and Regulations for Construction Projects. The typically compact to very dense sandy silt, sand and silt deposits at this site are classified as Type 3 soil according to the OHSA. Temporary excavations (i.e. those that are open only for a relatively short period and consequently backfilled) greater than 1.2 m deep through the native soils may be made with side slopes no steeper than about 1H:1V above the groundwater table or where the deposit is fully dewatered, assuming that sufficient drainage is provided by the ditches. At the east and west abutment locations, the lower portion of the embankment foreslopes will consist of cut native sand to silty sand, which will be covered by rock fill (see Figure 2B). For these cuts, the rock fill should be placed immediately after the cut is completed since these materials are susceptible to loosening. A geotextile filter between the native sandy soils and the rock protection should be provided to control surficial erosion during temporary conditions and in areas where the ditch is below the groundwater level. For these temporary cut slopes that will be covered with a thin veneer of rock fill, a minimum side slope of 1.5H:1V should be maintained.

Where required, any temporary excavation support system should be designed and constructed in accordance with MTO's Special Provision 105S19. The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in SP 105S19.

Depending on the foundation option that is chosen, subexcavation into the underlying bedrock may be required. It is noted that the bedrock is generally classified as strong to very strong. This will make rock excavation potentially difficult, particularly in areas where only small depths and narrow zones of removal are needed. Bedrock excavation in the vicinity of the proposed structure foundations should be carried out using line drilling and pre-shearing techniques (as discussed in

Section 5.10). This method would provide better control over the configuration of the founding surface, and this procedure would be the preferred approach where deeper excavation into the bedrock is required for footing construction.

5.9.2 Groundwater and Surface Water Control

Groundwater is anticipated to be encountered at about Elevation 251 m to 254 m (depending on the construction season), with some areas having higher perched water levels. Groundwater levels at the abutment borehole locations were generally encountered at or just above the proposed spread footing/pile cap founding elevations. However, the groundwater level at the center pier location was encountered about 3 metres higher than the proposed pier spread footing/pile cap founding elevation.

Filtered sump pumps or a more elaborate de-watering system may be required at the foundation locations if the proposed Highway 11 grade and ditches allowing for gravity run-off are not constructed prior to the abutment and pier foundation construction. Consideration should be given to raising the spread footing or pile cap founding elevations as high as possible to limit excavation below the groundwater table.

The centre pier location will most likely require a more elaborate dewatering system, in combination with sheetpiling in order to control groundwater seepage. If subexcavation to bedrock and mass concrete placement is chosen as a foundation option or excavation for construction of a pile cap, difficulties achieving a watertight seal between the sheetpiles and the sloping bedrock (about 1.5H:1V in this area) should be anticipated, particularly given the presence of cobbles/boulders. It is recommended that the water level inside the sheetpile box be maintained higher than the external groundwater level (Elevation 254 m during our investigation) and that a seal be placed at the sheetpile and soil/rock interface in order to minimize the potential for fines migrating into the base of the excavation. Consideration should be given to using tremied concrete methods for the placement of the mass concrete pad and installation of the rock dowels to support a shallow foundation, or to provide a seal for the pile cap if deep foundations are used. Prior to introducing the tremied concrete, consideration should also be given to placing sand bags or dry concrete bags along the interface between the sheetpiles and underlying soil/rock in order to fill the voids (i.e. assuming sloping bedrock) and minimize disturbance and possible infilling of fines/soil at the founding level. Tremied concrete construction should be in accordance with SP 105S19 and OPSS 904, specifically Section 904.07.03.15.09.

After the tremied concrete seal has been placed, water can be pumped from the excavation and construction of the spread footing or pile cap can be performed in the “dry”. The thickness of the tremied concrete plug should be designed using a Factor of Safety of at least 1.3 for non-structural components (a Factor of Safety of 1.5 is preferred considering the range in seasonal ground water levels). The Contractor's designer should check these thicknesses against his design. A note should be included on the contract drawings to this effect. In addition, a NSSP should be included in the contract to address the issue of encountering cobbles and boulders; a NSSP is included in Appendix B for reference.

For the spread footing option, the bedrock must be clean and the quality of the tremied concrete should be ascertained by obtaining concrete cores in the tremie or an equivalent method to ensure that limited air voids are present.

Alternatively, a vacuum well point system consisting of a series of well points installed with a sand filter surround and connected to a vacuum pump to lower the water table and allow for open cut excavation down to bedrock could be considered. However, this option would require a larger excavation and close spaced well points to reduce the risk of unstable excavations and basal instability leading to piping or boiling of the sand and silt soils near the bedrock interface.

It is anticipated that for open-cut excavations through the soil or rock, perched groundwater, if encountered, and surface water can be adequately controlled by pumping from properly filtered sumps and diverting any stormwater drainage paths to promote run-off away from or around the proposed construction areas.

5.9.3 Obstructions

The soils at the site may contain cobbles and boulders, particularly near the soil/bedrock interface at the pier locations. Conventional excavation equipment should be suitable for the majority of excavation through the on-site soils; however, the presence of rock fragments or boulders may interfere with or slow the progress of augering, piling, excavation and dewatering. An NSSP should be included in the contract documents to alert the contractor of such potential construction difficulties, an example is included in Appendix B for reference.

Difficulties excavating or driving sheetpiling (if required for dewatering purposes) through the cobble/boulder layer to the bedrock surface should be anticipated at the pier location. If typical sheetpiling installation methods cannot penetrate below the cobble/boulder layers, specialized construction techniques can be employed. Such techniques could involve excavating from within the sheetpiling to remove the cobbles/boulders and then continuing driving the sheetpiling, or line drilling ahead of and to facilitate driving of sheetpiling.

Ultimately, provision will have to be made in the Contract Specifications to ensure that the Contractor is equipped to handle such obstructions.

5.9.4 Proposed Permanent Rock Cut Slopes

For the rock cuts which are planned to be excavated to create the required clear zone (i.e. at the north side of the east abutment location), the newly excavated rock faces are expected to be relatively less weathered and in better condition than the existing faces provided good blasting practises are implemented. However, with time the rock faces will weather due to the blocky nature of the jointing and especially if biotite schist is present, as encountered in one borehole (BH16). As such, it is recommended that the east abutment founding level be lowered to the Highway 11 road grade level if spread footings or mass concrete is being considered. If the founding level of the abutment footings are located above the Highway 11 road surface, a setback of at least 3 m behind the face of the existing and/or proposed rock cut should be maintained assuming the rock consists of biotite or granite gneiss. For rock cuts less than about 3 m in height, the abutment footing should be located such that it is fully outside of a line drawn at an angle of about 1H:1V from the toe of the gneissic rock cut. It is important to note that weaker biotite schist bedrock was encountered in one borehole at depth at the east abutment (BH16). If the abutment founding level is above the Highway 11 road surface, the type and condition of the bedrock (i.e. after blasting and excavation) should be inspected by a rock engineer and setback distances increased or mitigation measures (e.g. lower founding elevation or rock bolting) implemented if biotite schist bedrock is encountered.

For permanent cut slopes through the bedrock, the overall slope to the cut face may be formed vertical to near vertical (i.e. 0.25H:1V). The use of carefully controlled drill and blast excavation techniques will be required to ensure a neat excavation line and minimize face instabilities and long-term maintenance problems resulting from blast damage to the rock mass as discussed in the following sections.

5.10 Blasting Recommendations for Rock Excavations

5.10.1 Excavation Considerations

For excavations into the bedrock, the overall slope to the cut face may be formed vertical or at a steep near vertical slope (i.e. 0.25H:1V). The use of controlled blasting techniques (such as pre-shearing or cushion blasting) are recommended, particularly along footing areas, in order to provide a neat excavation line and minimize face instabilities resulting from damage to the rock mass.

5.10.2 Special Provisions

5.10.2.1 Blasting

Good blasting practices will be critical to maintaining the excavation lines and preserving the integrity of the rock mass in the area of the structure foundations and proposed rock cuts. The use of controlled blasting techniques is recommended for all of the bedrock excavation. It is recommended that the Contractor retain a blast engineer and submit proposed blast plans to the Contract Administrator at least 3 weeks in advance of rock excavation. It is recommended that a separate NSSP for the control of all blasting operations be prepared; an example is provided in Appendix B. The NSSP should include, but not be limited to, the following:

- Outlining the requirements, procedure and extent of a pre-blast survey. This would include all structures within a radius of about 100 m of the blasting operations, as well as notification to all individuals working or living within 500 m.
- Submission of a blast proposal by the blasting contractor or their blast consultant detailing the blast methodology, including drill hole patterns, hole size and depths, size of blasts, explosive and initiation product details, as well as all blast control procedures. Blast control procedures would include details on controlling flyrock, temporary road closures, blast signalling and site clearing procedures, as well as procedures to deal with debris clean-up. This submission would be required prior to the commencement of any blasting operations.
- The requirement for trial blasts for all proposed production and wall control blast procedures.
- The requirements for ground and air vibration monitoring during the blasting operations. This would include details on instrumentation, number and location of monitoring sites, blast recording and reporting procedures, and procedures to be followed in the event of excessive vibration readings.
- At all locations where structures are located adjacent to rock cuts, new rock cut faces should inspected a rock engineering specialist contracted by the Contractor Administrator and provisions made for rock bolting, if necessary.

We recommend limiting ground vibration levels to 50 mm/s for adjacent services and buildings. Continuous monitoring of all blasting operations would dictate when changes to the blast procedures become necessary to meet these limits and how close to the blasting approaches the adjacent structures.

It is recommended that the specification for the blasting require a minimum of 80 percent half barrels (drill hole traces) visible on the cut face after scaling.

5.11 Closure

This report was prepared by Ms. Shannon Palmer, EIT and Mr. Kevin Bentley, P.Eng, a geotechnical engineer, and reviewed by Ms. Anne S. Poschmann, P.Eng., a Principal and senior geotechnical engineer. Mr. Fintan J. Heffernan, P.Eng., a Designated MTO Contact for Golder conducted a quality control review of the report.

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TABLES

TABLE 1

SUMMARY OF POINT LOAD TESTS ON ROCK CORE SAMPLES

PROJECT NO.:04-1111-039D

TITLE: Highway 11 and Muskoka Rd 169, Gravenhurst

DATE: May, 2006

Borehole Number	Sample Number	Sample Depth (m)	Bedrock Type	Test Type	Core Length (mm)	Core [2] Diameter (mm)	Ram Pressure (kPa)	Load (P) (kN)	Is Diametral (MPa)	**Is (50mm) (MPa)**	**Approx. [1] UCS** (MPa)
2	2	11.4	Biotite Gneiss	D	62.0	46.5	6701.9	6.48	3.000	2.903	67
	1	11.5	Biotite Gneiss	D	116.2	47.3	7901.7	7.64	3.413	3.329	77
	3	11.7	Biotite Gneiss	D	71.4	46.5	7881.0	7.62	3.528	3.414	79
	4	12.1	Biotite Gneiss	D	114.3	46.5	5543.6	5.36	2.481	2.401	55
	5	12.2	Biotite Gneiss	D	78.0	46.5	8336.1	8.06	3.731	3.611	83
	6	12.5	Biotite Gneiss	D	111.8	47.3	11459.5	11.08	4.955	4.832	111
	7	13.6	Granite Gneiss	D	120.4	47.3	14203.7	13.74	6.134	5.984	138
4	1	9.4	Biotite Gneiss	D	126.4	47.3	16237.7	15.70	7.013	6.841	157
	2	10.4	Biotite Gneiss	D	55.1	47.3	9363.4	9.06	4.053	3.952	91
	3	11.3	Granite Gneiss	D	102.4	47.3	15362.1	14.86	6.628	6.467	149
6	1	8.2	Granite Gneiss	D	181.8	47.3	15362.1	14.86	6.635	6.472	149
	2	9.9	Granite Gneiss	D	76.8	47.3	16437.7	15.90	7.092	6.920	159

[1] Is_{50} x 23 (actual value will have to be confirmed by UCS testing), from ISRM ("Suggested Methods for Determining Point Load Strength", International Society for Rock Mechanics Commission on Testing Methods, Int. J. Rock. Mech. Min. Sci. and Geomechanical Abstr., Vol 22, No. 2 1985, pp. 51-60.

[2] Actual distance between point load cones at time of failure.

TABLE 1 (cont'd)
SUMMARY OF POINT LOAD TESTS ON ROCK CORE SAMPLES

PROJECT NO.:04-1111-039D
TITLE: Highway 11 and Muskoka Rd 169, Gravenhurst
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Borehole Number	Sample Number	Sample Depth (m)	Bedrock Type	Test Type	Core Length (mm)	Core [(2)] Diameter (mm)	Equivalent Diameter (mm)	Ram Pressure (kPa)	Load (P) (kN)	Is Axial (MPa)	Is Diametral (MPa)	**Is (50mm) (MPa)**	**Approx. [(1)] UCS** (MPa)
9	1	3.9	Granite Gneiss	A	66.5	63.2	73.20	14362.3	13.89	2.592		3.077	71
	2	4.1	Granite Gneiss	D	69.9	63.2		24339.4	23.54		5.885	6.541	150
	3	4.6	Granite Gneiss	A	75.7	63.5	78.23	23201.7	22.44	3.667		4.485	103
	4	6.2	Granite Gneiss	D	63.5	63.5		23118.9	22.36		5.545	6.174	142
10	1	7.5	Granite Gneiss	D	85.9	63.5		18078.7	17.48		4.336	4.828	111
	2	7.7	Granite Gneiss	A	69.3	63.5	74.88	38122.5	36.87	6.576		7.887	181
	3	9.2	Granite Gneiss	D	126.5	63.5		26042.4	25.19		6.246	6.955	160
	4	9.3	Granite Gneiss	A	58.7	63.5	68.88	34364.7	33.23	7.006		8.092	186
11	1	6.7	Granite Gneiss	D	131.1	63.8		17161.7	16.60		4.083	4.555	105
	2	7.2	Granite Gneiss	A	60.7	63.8	70.20	30572.4	29.57	6.000		6.990	161
	3	8.0	Granite Gneiss	D	148.1	63.8		25697.7	24.85		6.114	6.821	157
	4	8.5	Granite Gneiss	A	64.3	63.8	72.22	39101.5	37.81	7.249		8.554	197
12	1	4.1	Granite Gneiss	A	66.0	63.5	73.07	29324.4	28.36	5.312		6.300	145
	2	4.2	Granite Gneiss	D	99.6	63.5		16658.3	16.11		3.995	4.449	102
	3	6.2	Granite Gneiss	A	64.0	63.5	71.94	20643.6	19.96	3.858		4.544	105
	4	6.3	Biotite Gneiss	D	67.6	63.5		10659.7	10.31		2.557	2.847	65

(1) Is_{50} x 23 (actual value will have to be confirmed by UCS testing), from ISRM ("Suggested Methods for Determining Point Load Strength", International Society for Rock Mechanics Commission on Testing Methods, Int. J. Rock. Mech. Min. Sci. and Geomechanical Abstr., Vol 22, No. 2 1985, pp. 51-60.
(2) Actual distance between point load cones at time of failure

TABLE 1 (cont'd)
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Borehole Number	Sample Number	Sample Depth (m)	Bedrock Type	Test Type	Core Length (mm)	Core [2] Diameter (mm)	Equivalent Diameter (mm)	Ram Pressure (kPa)	Load (P) (kN)	Is Axial (MPa)	Is Diametral (MPa)	**Is (50mm)** (MPa)	**Approx. [1] UCS** (MPa)
13	1	6.4	Granite Gneiss	A	68.3	63.5	74.33	33102.9	32.01	5.795		6.927	159
	2	6.5	Granite Gneiss	D	58.4	63.5		15120.7	14.62		3.627	4.038	93
	3	7.7	Granite Gneiss	A	63.2	63.5	71.51	34419.8	33.29	6.510		7.647	176
	4	7.8	Granite Gneiss	D	76.2	63.5		27717.9	26.81		6.648	7.403	170
14	1	6.7	Biotite Gneiss	A	63.8	63.5	71.80	28738.4	27.79	5.392		6.345	146
	2	6.8	Biotite Gneiss	D	67.8	63.5		26401.0	25.53		6.332	7.051	162
	3	8.5	Biotite Gneiss	A	63.2	63.5	71.51	21581.4	20.87	4.082		4.795	110
	4	8.6	Biotite Gneiss	D	64.0	63.5		10721.7	10.37		2.571	2.863	66
16	1	9.4	Biotite Gneiss	A	68.3	63.5	74.33	26842.2	25.96	4.699		5.617	129
	2	9.5	Biotite Gneiss	D	71.1	63.5		22919.0	22.16		5.497	6.121	141
	3	10.7	Biotite Schist*	A*	70.9	63.5	75.71	12741.5	12.32	2.150		2.591	60
	4	11.5	Biotite Schist*	A*	65.0	63.5	72.49	8091.0	7.82	1.489		1.760	40
	5	11.6	Biotite Schist	D	100.6	63.5		4040.5	3.91		0.969	1.079	25
	6	12.0	Biotite Schist	D	71.1	63.5		4640.3	4.49		1.113	1.239	29

[1] Is_{50} x 23 (actual value will have to be confirmed by UCS testing), from ISRM ("Suggested Methods for Determining Point Load Strength", International Society for Rock Mechanics Commission on Testing Methods, Int. J. Rock. Mech. Min. Sci. and Geomechanical Abstr., Vol 22, No. 2 1985, pp. 51-60.
[2] Actual distance between point load cones at time of failure
*Strength test results to be used with casution as rock cores broke along schist plane and not completely through core sample

TABLE 2
EVALUATION OF ABUTMENT FOUNDATION ALTERNATIVES
Highway 11 / Muskoka Road 169 Underpass
G.W.P. 314-00-00

Footing Option	*Rank*	*Advantages*	*Disadvantages*	*Relative Costs*	*Risks/Consequences*
Combined Spread Footing Founded on Bedrock and Drilled Piles (Concrete Filled) / Socketed into bedrock	1	• Allows for semi-integral abutment design; • Can drill through cobbles/boulders and any broken bedrock to design elevation; • With down hole hammer, allows for better control of the socketing into strong to very strong bedrock; • Increased capacity when compared to Steel H-piles; • With pile cap maintained as high as possible, excavation and dewatering requirements are minimized.	• Sequenced construction procedure required for both shallow and deep foundations for use on single foundation footprint; • Specialized drilling equipment required and sloping bedrock may cause difficulties; • Potential for differential behaviour using both shallow and deep foundation options at the same location if piles not socketed into rock.	• Higher relative costs than driven piled foundations since more specialized equipment required; • Lower dewatering costs for construction of pile cap compared to Rank 4.	• Design elevations can be achieved by drilling through any cobbles/boulders encountered and into the bedrock.
Combined Spread Footing and Driven Steel H-Piles on bedrock	2	• Relatively straight forward construction procedure; • Allows for semi-integral abutment design; • With pile cap maintained as high as possible, excavation and dewatering requirements are minimized.	• Sequenced construction procedure required to install shallow and deep foundations for each abutment foundation footprint; • Potential for piles to “hang-up” on bedrock ridges or knobs, or on cobbles / boulders typical of the area; • Potential for differential behaviour using both shallow and deep foundation options at the same location.	• Lower relative costs than using drilled piles/caissons; • Lower dewatering costs for construction of pile cap compared to Rank 4.	• Potential for piles to “hang-up” on cobbles, boulders or be deflected away from vertical (i.e. seating problems) during driving (especially with low overburden thickness).
Steel H-Piles driven to found on Bedrock (Subexcavate/Blast bedrock and replace with compacted granular fill where	3	• Allows for semi-integral abutment design (i.e. integral abutment design requires at least 5 m embedment length);	• Significant amount of bedrock needs to be removed / blasted and replaced with granular fill; • Bedrock will have to be subexcavated (i.e. blasted) using controlled blasting techniques	• Higher costs for bedrock subexcavation / blasting and replacement with compacted granular	• Undulating bedrock may require additional bedrock removal quantities; • Potential difficulties subexcavating strong to very

TABLE 2 (CONT'D)

required to achieve minimum 3 m pile embedment depth)			to minimize shattering and over-break; • Blasting and compaction of replacement granular soils below groundwater table; • Even with blasting of bedrock, pile embedment lengths are near the minimum required.	fill.	strong bedrock; • Blasting below water table may be required if highway ditches are not graded and allowed to drain by gravity prior to foundation construction; • Potential for piles to be deflected away from vertical (i.e. seating problems) during driving (especially with low overburden thickness).
Spread Footings founded on bedrock and/or on mass concrete placed on bedrock	4	• Allows for semi-integral abutment design; • Can minimize or eliminate bedrock excavation. • Depending on water level at the time of construction, this option may be ranked higher at the west abutment location only.	• Subexcavation depths through native cohesionless soils may be up to 7.3 m and 11.0 m below existing ground surface and up to 4 m and 10 m below the groundwater table for the west and east abutments, respectively. As a result, this option likely not feasible at east abutment location; • Tremied concrete placement or dewatering required. Sheetpile box or formwork likely required to limit size of excavation, control groundwater and control mass concrete quantities. • Variable bedrock surface will likely require bedrock excavation or installation of rock dowels (below groundwater level).	• Comparable costs to other options due to large amount of subexcavation and placement of mass concrete below water table.	• Variability in bedrock surface will impact excavation depths and mass concrete quantities; • Difficulties in achieving seal for sheetpiles including dewatering and quality of tremied concrete placement in permeable granular soils; • Difficulties installing dowels or blasting below groundwater level.
Spread Footings founded on native sand to sand and silt soils or perched within embankment fill	5 (NP)	• Can minimize or eliminate bedrock excavation	• Perched abutment locations are underlain by sloping bedrock and differential settlement of native soils across the foundation footprint is a concern; • Differential settlement between the abutments and pier, and within the abutment footprint itself (due to compression of embankment fill and native sand/silt soils on sloping bedrock); • Low geotechnical resistance.	• Low relative costs compared to other options	• Potential difficulties in achieving compaction with high groundwater table and silty native soils; • Differential settlement of structure is not desirable; • High risk of differential settlement within foundation

TABLE 2 (CONT'D)

					footprint limits make this option not practical.
Micropiles (grouted diameter less than 300 mm with reinforced threaded bar)	6 (NP)	-	• Low geotechnical resistance for small diameter friction pile design in thin overburden soils (as a result, end-bearing or rock socketing required); • Low lateral geotechnical resistance makes this option not practical.	• Specialized equipment required and specialized design in order to achieve adequate lateral resistance.	• High risk of difficulties achieving required lateral resistance with small diameter piles.
Caissons	7 (NP)	-	• High water table and steeply sloping bedrock make this option not feasible.	• Not economical	• High risk of difficulties achieving seal and bedrock socket.

NP = not practical or not feasible

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TABLE 3
EVALUATION OF PIER FOUNDATION ALTERNATIVES
Highway 11 / Muskoka Road 169 Underpass
G.W.P. 314-00-00

Footing Option	*Rank*	*Advantages*	*Disadvantages*	*Relative Costs*	*Risks/Consequences*
Spread Footings founded on bedrock and/or on mass concrete placed on bedrock	1	• Can minimize or eliminate bedrock excavation.	• Subexcavation depths through native cohesionless soils may be up to about 8.5 m below existing ground surface and up to 8 m below the existing groundwater table; • Tremied concrete placement and/or extensive dewatering required. Sheetpile box or formwork required to limit size of excavation and control mass concrete quantities. • Variable bedrock surface will likely require bedrock excavation (to create level surface) or installation of rock dowels (below groundwater level).	• Comparable costs to other options due to large amount of subexcavation, dewatering, and placement of mass concrete.	• Variability in bedrock surface will impact excavation depths and mass concrete quantities; • Difficulties in achieving seal for sheetpiles including dewatering and quality of tremied concrete placement in highly permeable granular soils; • Potential difficulties penetrating / excavating through cobble/boulder layer to expose bedrock surface; • Difficulties installing dowels or blasting bedrock below groundwater level.
Combined Spread Footing Founded on Bedrock and Drilled Piles (Concrete Filled) Socketed into Bedrock	2	• Can drill through cobbles/boulders and any broken bedrock to design elevation.	• Sequenced construction procedure required for both shallow and deep foundations for use on pier foundation footprint; • Variable bedrock surface within relatively small foundation footprint may lead to limited use of piles (i.e. only a small number of piles may actually be used); • Specialized drilling equipment required and sloping bedrock may cause difficulties; • Tremied concrete placement and/or dewatering required for construction of pile cap and spread footing in the dry. Sheetpile box or formwork likely required to limit size	• Higher relative costs than shallow foundations since more specialized equipment required.	• Design elevations can be achieved by drilling through any cobbles/boulders encountered and into the bedrock; • Redesign of drilled piles required if assumed bedrock levels / orientation differs.

TABLE 3 (CONT'D)

			of excavation and control concrete quantities. • Variable bedrock surface may require bedrock excavation (to create level surface at base of pile cap)		
Combined Spread Footing and Driven Steel H-Piles on bedrock	3		• Sequenced construction procedure required to install shallow and deep foundations for pier foundation footprint; • Variable bedrock surface within relatively small foundation footprint may lead to limited use of piles (i.e. only a small number of piles may actually be used); • Tremied concrete placement and/or dewatering required for construction of pile cap and spread footing in the dry. Sheetpile box or formwork likely required to limit size of excavation and control concrete quantities; • Likely not feasible at current pile cap base elevation due to high potential for piles to "hang-up" prior to achieving 3 m embedment depth.	• Lower relative costs than using drilled piles/caissons; • Comparable costs to spread footings with mass concrete placement due to dewatering and sheeting costs. • If pile cap can be raised by about 1 m, driven H-pile option will be less costly.	• Potential for piles to "hang-up" on cobbles, boulders or be deflected away from vertical (i.e. seating problems) during driving.
Steel H-Piles driven to Bedrock (Subexcavate/Blast bedrock and replace with compacted granular fill required to achieve a minimum 3 m pile embedment depth)	4	• One type of foundation is used for entire pier footing.	• Significant amount of bedrock needs to be removed / blasted and replaced with granular fill under water; • Bedrock will have to be subexcavated (i.e. blasted) up to 3 m within the rock itself using controlled blasting techniques to minimize shattering and over-break; • Subexcavation depths through native cohesionless soils may be up to about 8.5 m below existing ground surface and up to 8 m below the existing groundwater table (not including up to 3 m additional bedrock subexcavation); • Compaction of replacement granular soils	• High costs for bedrock subexcavation / blasting and replacement with compacted granular fill below groundwater table.	• Undulating bedrock may require additional bedrock removal quantities; • Potential difficulties subexcavating strong to very strong bedrock; • Difficulties blasting bedrock and replacing with granular soils below the water table expected.

TABLE 3 (CONT'D)

			below groundwater table.		
Spread Footings founded on native sand to sand and silt soils	5		• Potential for differential settlement of native soils across the foundation plan area is a concern; • Potential for differential settlement between the abutments and pier; • Low geotechnical resistance; • Subexcavation depths through native cohesionless soils may be up to about 6 m below existing ground surface and up to 4 m below existing groundwater table; • Tremied concrete placement and dewatering required. Sheetpile box or formwork likely required to limit size of excavation, control groundwater, and control mass concrete quantities.	• Reduced costs compared to Option No. 2;	• Difficulties in achieving seal for sheetpiles and quality of tremied concrete placement in highly permeable granular soils; • Potential difficulties in maintaining integrity of cohesionless soils at foundation base due to high groundwater table ("piping" of sandy soils); • Differential settlement in structure is not desirable, especially for a single column integral pier.
Micropiles (grouted diameter less than 300 mm with reinforced threaded bar)	6 (NP)	-	• Low geotechnical resistance for small diameter friction pile design in thin overburden soils (as a result, end-bearing or rock socketing required); • Low lateral geotechnical resistance makes this option not practical.	• Specialized equipment required and specialized design in order to achieve adequate lateral resistance.	• High risk of difficulties achieving required lateral resistance with small diameter piles.
Caissons	7 (NP)	-	• High water table and steeply sloping bedrock make this option not feasible.	• Not economical	• High risk of difficulties achieving seal and bedrock socket.

NP = not practical or not feasible

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TABLE 4
Summary of Recommendations at Structure Approach Embankments (incl. Platform Widening)
Highway 11 and Muskoka Road 169 Underpass
G.W.P. 314-00-00

Highway	Approx. Station (Muskoka Rd. 169 centreline chainage)	Proposed Works	Overburden Conditions	Recommended Embankment Fill Type	Organics Encountered along alignment	Recommended Side Slope	Side Berm Recommended	Estimated Post-Construction Settlements (δ)* *and Platform Widening (w)***	Stripping / Organics Removal
Highway 11 and Muskoka Road 169 Underpass	9+035 to 9+060	West Approach (fill ranges from about 3 m to 8.5 m high within footprint, and fill typically ranges from 3 m to 3.5 m thick immediately behind abutment within footprint of approach slab)	Sand to sand and silt at ground surface to a depth ranging from 2 m to 7 m below ground surface, underlain by bedrock	Granular Fill below approach slab (see Figure 2) Rock Fill beyond approach slab	Yes. Organics/ topsoil within about 0.3 m bgs. Also, asphalt/fill within about 0.2 m bgs.	1.25H:1V for surficial rock fill; 1.5H:1V for temporary granular fill covered with rock fill (see Figure 2)	No.	δ_{max}= 35mm (rock fill) δ_{max}< 25mm (granular fill) δ_{diff}< 25 mm (within approach slab footprint location, see Figure 2) w = 2000 mm (up to 5 m beyond limit of wingwalls, see Figure 2) transitioning to w = 1000 mm in all other areas	Strip and remove all organics/ asphalt/fill within footprint of embankment.
	10+050 to 10+075	East Approach (fill ranges from 2 m to 10.5 m high within embankment footprint, and fill ranges from 2 m to 7.3 m thick immediately behind abutment within footprint of approach slab)	Silty sand to sand and silt ranging from 2 m to 11 m below ground surface, underlain by bedrock	Granular Fill below approach slab (see Figure 2) Rock Fill beyond approach slab	Yes Organics/ topsoil within about 0.3 m bgs.	1.25H: 1V for surficial rock fill; 1.5H:1V for temporary granular fill to be covered with rock fill (see Figure 2)	Yes, at foreslope only where embankment height exceeds 10 m); A 2 m wide berm should be placed at mid-height.	δ_{max}= 75 mm (rock fill) δ_{max}< 25 mm (granular fill) δ_{diff}< 25 mm (within approach slab footprint location, see Figure 2) w = 2000 mm (up to 5 m beyond limit of wingwalls, see Figure 2) transitioning to w = 1000 mm in all other areas	Strip and remove all organics/ topsoil within footprint of embankment.

Notes :
*** Settlements below travelled lanes include compression of rockfill and earth fill but do not include settlements of foundations soils which should occur during construction.**
**** Recommended embankment platform widening (per embankment side) where rock fill is used for construction based on guidelines in NRE 98-200.**
δ_{max}= maximum calculated settlement within footprint of approach embankment
δ_{diff}= anticipated differential settlement that could occur directly behind the abutment and in the proposed approach slab footprint

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RECORD OF BOREHOLE SHEETS AND TEST PIT LOGS

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance; N_d:

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

PH:	Sampler advanced by hydraulic pressure
PM:	Sampler advanced by manual pressure
WH:	Sampler advanced by static weight of hammer
WR:	Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 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.

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LIST OF SYMBOLS

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

I. General

π	3.1416
in x,	natural logarithm of x
$\log_{10}$	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
F	factor of safety
V	volume
W	weight

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
v	poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma$-u)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1+\sigma_2+\sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

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

(a) Index Properties (continued)

w	water content
w_1	liquid limit
w_p	plastic limit
l_p	plasticity index $= (w_1 - w_p)$
w_s	shrinkage limit
I_L	liquidity index $= (w - w_p)/I_p$
I_C	consistency index $= (w_1 - w)/I_p$
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_a	coefficient of secondary consolidation
m_v	coefficient of volume change
c_v	coefficient of consolidation
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation pressure
OCR	over-consolidation ratio $= \sigma'_p/\sigma'_{vo}$

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = tan δ
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

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

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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 texture and structure are preserved.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	> 2 m
Thickly bedded	0.6 m to 2m
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	< 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	> 3 m
Wide	1 - 3 m
Moderately close	0.3 - 1 m
Close	50 - 300 mm
Very close	< 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	> 60 mm
Coarse Grained	2 - 60 mm
Medium Grained	60 microns - 2 mm
Fine Grained	2 - 60 microns
Very Fine Grained	< 2 microns

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

CORE CONDITION

Total Core Recovery

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 varies 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 (W.R.T.) 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 abbreviated 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

B - Bedding	P - Polished
FO - Foliation/Schistosity	S - Slickensided
CL - Cleavage	SM - Smooth
SH - Shear Plane/Zone	R - Ridged/Rough
VN - Vein	ST - Stepped
F - Fault	PL - Planar
CO - Contact	FL - Flexured
J - Joint	UE - Uneven
FR - Fracture	W - Wavy
MF - Mechanical Fracture	C - Curved
$\parallel$ - Parallel To	
$\perp$ - Perpendicular To	

Form...G.A.-R-3

Golder Associates

RECORD OF BOREHOLE No BH01

METRIC

PROJECT 04-1111-039
W.P. 314-00-00
LOCATION N 4973548.3 ;E 315735.4
ORIGINATED BY SB
DIST HWY 11
BOREHOLE TYPE 108 mm I.D. Hollow Stem Power Auger
COMPILED BY DD
DATUM Geodetic
DATE May 18, 2005
CHECKED BY SLP

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
255.8	GROUND SURFACE						
0.0	Silty SAND to Gravelly SAND, trace clay, trace organics Loose Brown Moist					255	
		1	SS	8			24 48 24 4
254.6 1.2	Silty SAND to SAND and SILT, trace clay Compact to dense Grey to brown Moist to wet	2	SS	25	∇	254	
		3	SS	35		253	
		4	SS	39			0 53 46 1
252.0 3.8	SAND, trace silt Dense to very dense Brown to grey Wet					252	
		5	SS	30			
		6	SS	24/.20		251	
250.8 5.0	End of Borehole Auger Refusal						

Note:

1. Water level in open borehole at 2.1 m depth (Elev. 253.7 m) upon completion of drilling operations.

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O 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

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

RECORD OF BOREHOLE No BH02

 METRIC

PROJECT 04-1111-039
W.P. 314-00-00 LOCATION N 4973541.1 ;E 315733.2 ORIGINATED BY SB
DIST HWY 11 BOREHOLE TYPE 108 mm I.D. Hollow Stem Power Auger COMPILED BY DD
DATUM Geodetic DATE May 18, 2005 CHECKED BY SLP

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
253.8	GROUND SURFACE					
0.0	SAND and SILT, trace clay Compact to dense Grey Moist	1	SS	32		
		2	SS	36		0 41 57 2
		3	SS	22		
		4	SS	13		
250.0 3.8	SAND, trace to some silt Compact to very dense Grey Wet	5	SS	29	∇	
		6	SS	56		
		7	SS	28		
246.2 7.6	Silty SAND to SAND, some silt, trace clay Very loose to compact Brown to grey Wet	8	SS	WH		1 76 21 2
		9	SS	1		
		10	SS	18/.15		
242.8 11.0	BIOTITE AND GRANITE GNEISS Slightly weathered to fresh Strong to very strong Fine to coarse crystalline Foliated, black/white/pink Bedrock cored from 11.0 m to 14.0 m For bedrock coring details refer to Record of Drillhole BH02					
239.8 14.0						

ELEVATION SCALE: 253, 252, 251, 250, 249, 248, 247, 246, 245, 244, 243, 242, 241, 240

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O 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

Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No BH02

 METRIC

PROJECT 04-1111-039
W.P. 314-00-00
LOCATION N 4973541.1 ;E 315733.2
ORIGINATED BY SB
DIST HWY 11
BOREHOLE TYPE 108 mm I.D. Hollow Stem Power Auger
COMPILED BY DD
DATUM Geodetic
DATE May 18, 2005
CHECKED BY SLP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O 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						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	End of Borehole Note: 1. Water level inside hollow stem augers at 4.0 m depth (Elev. 249.8 m) upon completion of drilling operations. As a result this may not represent the stabilized water level.										

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

RECORD OF DRILLHOLE: BH02

PROJECT: 04-1111-039
SHEET 1 OF 1

LOCATION: N 4973541.1 ;E 315733.2
DRILLING DATE: May 18, 2005
DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---
DRILL RIG: D-50
DRILLING CONTRACTOR: Walker Drilling Ltd.

JN - Joint, FLT - Fault, SHR - 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, MB - Mechanical Break, BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION
	GROUND SURFACE			
11	BIOTITE GNEISS Slightly weathered to fresh Strong to very strong Fine to medium crystalline Foliated, black/white Coarse Quartz-Rich Banding	11.00	1	,IR,Ro ,PL,Ro/SM FO,PL,Ro/SM ,IR,Ro FO,PL,Ro/SM PL,IR,Ro FO,PL,Ro IR,PL,Ro IR,PL,Ro FO,PL,Ro FO,PL,Ro JN,IR,Ro
12–13			2	,PL,Ro ,PL,Ro ,PL,Ro ,PL,Ro FO,PL,SM ,PL/IR,Ro JN,PL,Ro
	GRANITE GNEISS Fresh Very strong Fine to coarse, crystalline Foliated, pink/black/white	13.26		JN,PL,Ro VN,IR,Ro JN,PL,Ro
	BIOTITE GNEISS Fresh Very strong Fine to coarse, crystalline Foliated, black/white	13.70		FO,PL,Ro
14	End of Drillhole	14.00		

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: SB
CHECKED: SLP

MIS-RCK 002 04111039AARCK.GPJ GAL-MISS.GDT 21/8/06 JDR

RECORD OF BOREHOLE No BH03

 METRIC

PROJECT 04-1111-039
W.P. 314-00-00 LOCATION N 4973561.0 ;E 315681.5 ORIGINATED BY SB
DIST HWY 11 BOREHOLE TYPE 108 mm I.D. Hollow Stem Power Auger COMPILED BY DD
DATUM Geodetic DATE May 17, 2005 CHECKED BY SLP

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
255.1	GROUND SURFACE					255		
0.0	SAND, trace silt Compact Brown Moist to wet							
		1	SS	17		254	~22	
		2	SS	14	▽		~20	
253.0 2.1	SAND and SILT, trace clay Dense Brown and grey Wet					253		
		3	SS	37			~21	
		4	SS	31		252	~20	0 35 62 3
		5	SS	35		251		
		6	SS	31		250	~18	0 41 56 3
248.9 6.3	End of Borehole Auger Refusal Note: 1. Water level in open borehole at 1.7 m depth (Elev. 253.4 m) upon completion of drilling operations.					249		

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

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

RECORD OF BOREHOLE No BH04

 METRIC

PROJECT 04-1111-039
W.P. 314-00-00
LOCATION N 4973553.5 ;E 315683.0
ORIGINATED BY SB
DIST HWY 11
BOREHOLE TYPE 108 mm I.D. Hollow Stem Power Auger
COMPILED BY DD
DATUM Geodetic
DATE May 16, 2005
CHECKED BY SLP

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
253.8	GROUND SURFACE									
0.0	SAND, trace silt, trace organics Compact Brown and grey Moist to wet				253					
		1	SS	14		~20				
		2	SS	14	252	~20				
251.7 / 2.1	Silty SAND, trace clay Compact Brown and grey Wet	3	SS	27	251	~20	0	68	30	2
		4	SS	24		~18				
250.1 / 3.7	SAND and SILT, trace clay, trace gravel Dense to very dense Brown Wet	5	SS	50	250		1	66	32	1
		6	SS	48	249	~18				
					248					
		7	SS	34	247					
246.5 / 7.3	Cobbles/Boulders Grinding of Augers				246					
245.4 / 8.4	BIOTITE AND GRANITE GNEISS Slightly weathered to fresh Strong to very strong Fine to coarse crystalline Foliated, black/white/pink Bedrock cored from 8.4 m to 11.3 m For bedrock coring details refer to Record of Drillhole BH04				245, 244, 243					
242.5 / 11.3	End of Borehole Note: 1. Water level in open borehole at 0.5 m depth (Elev. 253.3 m) upon completion of drilling operations.									

Ground water conditions: water level symbol at approx. Elev. 253.3.

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (O 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

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

PROJECT: 04-1111-039

RECORD OF DRILLHOLE: BH04

SHEET 1 OF 1

LOCATION: N 4973553.5 ;E 315683.0

DRILLING DATE: May 17, 2005

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: D-50

DRILLING CONTRACTOR: Walker Drilling Ltd.

JN - Joint, FLT - Fault, SHR - 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, MB - Mechanical Break, BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	DISCONTINUITY DATA – TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
	GROUND SURFACE				
	BIOTITE GNEISS Slightly weathered to fresh Strong to very strong Fine to medium crystalline Foliated, black/white Coarse Quartz-Rich Banding	8.28	1	,, BR ,, BR IR,,Ro IR,,Ro FO,,SM ,PL,Ro FO,PL,Ro FO,PL,Ro FO,PL,Ro FO,PL,Ro FO,PL,Ro VN,PL,Ro/SM FO,PL,Ro FO,PL,Ro	
9					
10			2	FO,PL,Ro/SM FO,PL,Ro/SM FO,PL,SM FO,PL,SM FO,PL,Ro/SM	
	GRANITE GNEISS Fresh Very strong Fine to coarse crystalline Foliated, black/white/pink	10.67		,IR,Ro ,PL, FO,PL,Ro/SM	
11					
	End of Drillhole	11.33			
12					
13					
14					
15					
16					
17					
18					

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: SB

CHECKED: SLP

MIS-RCK 002 041111039AARCK.GPJ GAL-MISS.GDT 21/8/06 JDR

Golder Associates

RECORD OF BOREHOLE No BH05

 METRIC

PROJECT 04-1111-039
W.P. 314-00-00
LOCATION N 4973561.8 ;E 315639.0
ORIGINATED BY SB
DIST HWY 11
BOREHOLE TYPE 108 mm I.D. Hollow Stem Power Auger
COMPILED BY DD
DATUM Geodetic
DATE May 19, 2005
CHECKED BY SLP

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
257.4	GROUND SURFACE								
0.0	SAND, trace to some gravel, trace silt and clay Compact to very dense Brown to grey Moist				257				
		1	SS	17		10	85	4	1
					256				
		2	SS	24					
					255				
		3	SS	23		2	93	4	1
254.2		4	SS	12/.08					
3.2	End of Borehole Auger Refusal Note: 1. Borehole dry upon completion of drilling operations. 2. Piezometer dry on September 30, 2005 and November 10, 2005.								

Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O 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

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

RECORD OF BOREHOLE No BH06

 METRIC

PROJECT 04-1111-039			
W.P. 314-00-00	LOCATION N 4973569.6 ;E 315642.6		ORIGINATED BY SB
DIST	HWY 11	BOREHOLE TYPE 108 mm I.D. Hollow Stem Power Auger	COMPILED BY DD
DATUM Geodetic	DATE May 19, 2005		CHECKED BY SLP

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
257.6	GROUND SURFACE					
0.0	SAND, trace gravel, trace silt, trace organics and roots Very loose Brown Moist	1	SS	3		
256.1 1.5	SAND, trace to some silt, trace clay Compact Brown and grey Moist	2	SS	23		
		3	SS	19		0 89 8 3
		4	SS	23		
253.8 3.8	SAND and SILT, trace clay Compact Brown to grey Moist to wet	5	SS	22	∇ (253.8)	0 49 50 1
		6	SS	24		
		7	SS	14		
250.3 7.3	GRANITE GNEISS Slightly weathered to fresh Very strong Fine to coarse crystalline Foliated, black/white/pink Bedrock cored from 7.3 m to 10.4 m For bedrock coring details refer to Record of Drillhole BH06					
247.3 10.4	End of Borehole Note: 1. Water level in open borehole at 3.8 m depth (Elev. 253.8 m) upon completion of drilling operations.					

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (257–248); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O 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

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

PROJECT: 04-1111-039

RECORD OF DRILLHOLE: BH06

SHEET 1 OF 1

LOCATION: N 4973569.6 ;E 315642.6

DRILLING DATE: May 19, 2005

DATUM: Geodetic

DRILL RIG: D-50

INCLINATION: -90° AZIMUTH: ---

DRILLING CONTRACTOR: Walker Drilling Ltd.

JN - Joint, FLT - Fault, SHR- 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, MB- Mechanical Break, BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
	GROUND SURFACE				
	GRANITE GNEISS Slightly weathered to fresh Very strong Fine to coarse crystalline Foliated, black/white/pink Biotite-Rich Banding	7.32			
8			1	,PL,Ro/SM ,PL,Ro/SM	
9			2	FO,PL,Ro	
10					
	End of Drillhole	10.36			
11					
12					
13					
14					
15					
16					
17					

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: SB

CHECKED: SLP

Golder Associates

RECORD OF BOREHOLE No BH07

 METRIC

PROJECT 04-1111-039

W.P. 314-00-00 LOCATION N 4973544.0 ;E 315746.4 ORIGINATED BY SB

DIST HWY 11 BOREHOLE TYPE 108 mm I.D. Hollow Stem Power Auger COMPILED BY DD

DATUM Geodetic DATE May 19, 2005 CHECKED BY SLP

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
255.8	GROUND SURFACE												
0.0	SAND and SILT, trace gravel, trace clay, contains cobbles above 0.8 m depth Compact to very dense Brown Moist to wet						255						
			1	SS	16								
			2	SS	67/.08		254			0	52	47	1
253.9 2.0	End of Borehole Auger Refusal Note: 1. Borehole dry upon completion of drilling operations.												

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

REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

RECORD OF BOREHOLE No BH08

 METRIC

PROJECT 04-1111-039
W.P. 314-00-00
LOCATION N 4973566.6 ;E 315627.7
ORIGINATED BY SB
DIST HWY 11
BOREHOLE TYPE 108 mm I.D. Hollow Stem Power Auger
COMPILED BY DD
DATUM Geodetic
DATE May 19, 2005
CHECKED BY SLP

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
257.3	GROUND SURFACE												
0.0	SAND, trace gravel, silt and clay Compact to very dense Brown Moist						257						
			1	SS	12		256						
255.4			2	SS	26/.18					0	95	4	1
1.9	End of Borehole Auger Refusal Note: 1. Borehole dry upon completion of drilling operations.												

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O 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
REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

RECORD OF BOREHOLE No BH09

 METRIC

PROJECT 04-1111-039			
W.P. 314-00-00	LOCATION N 4973573.3 ;E 315629.5		ORIGINATED BY SB
DIST	HWY 11	BOREHOLE TYPE 108 mm I.D. Hollow Stem Power Auger	COMPILED BY MSM
DATUM Geodetic	DATE November 15, 2005		CHECKED BY KB

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
257.7	GROUND SURFACE						
0.0	Asphalt						
0.2	Sand and gravel (FILL)						
	SAND, trace to some silt Compact to very dense Brown and grey Moist				257		
		1	SS	24			
		2	SS	51	256		
		3	SS	47	255		
		4	SS	69			0 76 (24)
254.1 3.6	GRANITE GNEISS Slightly weathered to fresh Strong to very strong Fine to coarse crystalline Foliated black/white/pink Bedrock cored from 3.6m to 6.3m For bedrock coring details refer to Record of Drillhole BH09				254 253 252		
251.4 6.3	End of borehole Note: 1. Borehole dry upon completion of drilling operations.						

Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O 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

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

PROJECT: 04-1111-039

RECORD OF DRILLHOLE: BH09

SHEET 2 OF 2

LOCATION: N 4973573.3 ;E 315629.5

DRILLING DATE: November 15, 2005

DATUM: Geodetic

DRILL RIG: CME 55 Bombardier

INCLINATION: -90° AZIMUTH: ---

DRILLING CONTRACTOR: Walker Drilling Ltd.

JN - Joint, FLT - Fault, SHR - 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, MB - Mechanical Break; BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	DISCONTINUITY DATA – TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
	- continued from Record of Borehole -	254.10			
	GRANITE GNEISS Slightly weathered to fresh Strong to very strong Fine to coarse crystalline Foliated, black/white/pink	3.60			
4			1		Axial
				FO,PL,SM-Ro	
					Axial
				FO,PL,Ro	
5			2	JN,IR,Ro	
				FO,PL,SM	
				JN,PL,Ro	
6		251.38 6.32			
	End of Drillhole				
7					
8					
9					
10					
11					
12					
13					

DEPTH SCALE

1 : 50

LOGGED: SB

CHECKED: KB

MIS-RCK 002 041111039AARCK.GPJ GAL-MISS.GDT 21/8/06 JDR

Golder Associates

RECORD OF BOREHOLE No BH10

 METRIC

PROJECT 04-1111-039
W.P. 314-00-00
LOCATION N 4973572.3 ;E 315640.4
ORIGINATED BY SB
DIST
HWY 11
BOREHOLE TYPE 108 mm I.D. Hollow Stem Power Auger
COMPILED BY MSM
DATUM Geodetic
DATE November 14, 2005
CHECKED BY KB

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
257.8	GROUND SURFACE						
0.0	TOPSOIL						
257.5 0.3	SAND Loose to dense Brown to brown and grey Moist	1	SS	9		o	
		2	SS	31			
		3	SS	38		o	
		4	SS	39			
		5	SS	37		o	
253.2 4.6	SAND and SILT Compact to very dense Brown	6	SS	69			
	Becoming wet at 6.1m depth	7	SS	24	∇	o	0 32 62 2
250.5 7.3	GRANITE GNEISS Slightly weathered to fresh Very Strong Fine to coarse crystalline Foliated black/white/pink Bedrock cored from 7.3m to 10.4m For bedrock coring details refer to Record of Drillhole BH10						
247.4 10.4	End of Borehole Note: 1. Water level in open borehole at 6.3m depth upon completion of drilling operations.						

Elevation scale: 257, 256, 255, 254, 253, 252, 251, 250, 249, 248

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O 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

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

PROJECT: 04-1111-039

RECORD OF DRILLHOLE: BH10

SHEET 2 OF 2

LOCATION: N 4973572.3 ;E 315640.4

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: November 14, 2005

DRILL RIG: CME 55 Bombardier

DRILLING CONTRACTOR: Walker Drilling Ltd.

DATUM: Geodetic

JN - Joint, FLT - Fault, SHR- 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, MB- Mechanical Break, BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	RMC -Q' AVG.	NOTES WATER LEVELS INSTRUMENTATION
	- continued from Record of Borehole -	250.47				
	GRANITE GNEISS Slightly weathered to fresh Very strong Fine to coarse crystalline Foliated black/white/pink	7.30	1	PL,,Ro		Axial
8						
				JN,PL,Ro JN,PL,Ro		
9			2	FO,IR,Ro JN,PL-IR,Ro	3.09	Axial
10		247.41				
	End of Drillhole	10.36				
11						
12						
13						
14						
15						
16						
17						

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: SB

CHECKED: KB

Golder Associates

RECORD OF BOREHOLE No BH11

METRIC

PROJECT 04-1111-039			
W.P. 314-00-00	LOCATION N 4973568.8 ;E 315639.3		ORIGINATED BY SB
DIST	HWY 11	BOREHOLE TYPE 108 mm I.D. Hollow Stem Power Auger	COMPILED BY MSM
DATUM Geodetic	DATE November 15, 2005		CHECKED BY KB

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
257.5	GROUND SURFACE										
0.0	TOPSOIL										
0.2	SAND, trace to some silt, trace organics and gravel Compact to dense Brown and grey Moist					257					
		1	SS	11							
		2	SS	27		256	o				
		3	SS	35		255					
		4	SS	37		254	o	0	89	11	0
253.7 3.8	SAND and SILT Dense to very dense Brown moist to wet	5	SS	54							
		6	SS	57	∇	253	o				
	Becoming wet at 6.1m depth	7	SS	50		252 / 251					
250.9 6.6	GRANITE GNEISS Slightly weathered to fresh Very Strong Fine to coarse crystalline Foliated black/white/pink Bedrock cored from 6.6m to 9.6m For bedrock coring details refer to Record of Drillhole BH11					250 / 249					
247.9 9.6	End of Borehole Note: 1. Water level in open borehole at 5.18m depth upon completion of drilling operations.					248					

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O 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 (%)

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

PROJECT: 04-1111-039

RECORD OF DRILLHOLE: BH11

LOCATION: N 4973568.8 ;E 315639.3

DRILLING DATE: November 16, 2005

DATUM: Geodetic

DRILL RIG: CME 55 Bombardier

INCLINATION: -90° AZIMUTH: ---

DRILLING CONTRACTOR: Walker Drilling Ltd.

JN - Joint, FLT - Fault, SHR- 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, MB- Mechanical Break, BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	DISCONTINUITY TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
	- continued from Record of Borehole -				
	GRANITE GNEISS Slightly weathered to fresh Very strong Fine to coarse crystalline Foliated black/white/pink	250.99 / 6.55			
7					Axial
			1	JN,PL-ST,Ro JN,PL,Ro	
8				FO,PL,Ro	
					8.55 Axial
			2	,PL,IR,Ro	
9				FO,PL,SM-Ro	
	End of Drillhole	247.94 / 9.60			
10					
11					
12					
13					
14					
15					
16					

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: SB

CHECKED: KB

Golder Associates

RECORD OF BOREHOLE No BH12

 METRIC

PROJECT 04-1111-039
W.P. 314-00-00 LOCATION N 4973561.1 ;E 315640.4 ORIGINATED BY SB
DIST HWY 11 BOREHOLE TYPE 108 mm I.D. Hollow Stem Power Auger COMPILED BY MSM
DATUM Geodetic DATE November 16, 2005 CHECKED BY KB

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 (O 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
257.5	GROUND SURFACE										
0.0	TOPSOIL										
0.2	SAND, trace to some gravel, trace silt and clay Compact to dense Brown to brown and grey Moist						257				
			1	SS	14		256				
			2	SS	37				o		6 90 (4)
			3	SS	28		255				
			4	SS	34		254		o		
253.7 3.8	GRANITE and BIOTITE GNEISS Slightly weathered to fresh Strong to very strong Fine to coarse crystalline Foliated black/white/pink Bedrock cored from 3.8m to 6.9m For bedrock coring details refer to Record of Drillhole BH12						253 252 251				
250.6 6.9	End of Borehole Note: 1. Borehole dry upon completion of drilling operations.										

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

MIS-MTO 001 041111039AAMTO.GPJ GAL-MISS.GDT 21/8/06

PROJECT: 04-1111-039

RECORD OF DRILLHOLE: BH12

LOCATION: N 4973561.1 ;E 315640.4

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: November 16, 2005

DRILL RIG: CME 55 Bombardier

DRILLING CONTRACTOR: Walker Drilling Ltd.

DATUM: Geodetic

JN - Joint, FLT - Fault, SHR- 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, MB- Mechanical Break, BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	FLUSH	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
	- continued from Record of Borehole -	253.70				
	GRANITE GNEISS Slightly weathered to fresh Very strong Fine to coarse crystalline Foliated, black/white/pink	3.81				
4					,IR,Ro	
					,, JN,PL,Ro	Axial
					JN,IR,Ro	
			1			
5					FO,PL,SM-Ro	
6			2	<<	JN,PL,Ro ,IR,Ro FO,IR,Ro	
	BIOTITE GNEISS Slightly weathered to fresh Strong Fine to medium crystalline Black/white Coarse quartz banding	251.26 6.25				Axial
					,IR,Ro	
					FO,PL,Ro	
					,ST,Ro	
7	End of Drillhole	250.65 6.86				
8						
9						
10						
11						
12						
13						

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: SB

CHECKED: KB

MIS-RCK 002 04111039AARCK.GPJ GAL-MISS.GDT 21/8/06 JDR

Golder Associates

RECORD OF BOREHOLE No BH13

 METRIC

PROJECT 04-1111-039

W.P. 314-00-00 LOCATION N 4973560.4 ;E 315684.3 ORIGINATED BY SB

DIST ____ HWY 11 BOREHOLE TYPE 108 mm I.D. Hollow Stem Power Auger COMPILED BY MSM

DATUM Geodetic DATE November 14, 2005 CHECKED BY KB

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
254.3	GROUND SURFACE					
0.0	SAND Compact Brown Moist to wet	1	SS	23	∇ (water level)	
252.8 / 1.5	Silty SAND to SAND and SILT, trace clay Dense to very dense Brown Wet	2	SS	36		
		3	SS	35		0 64 34 2
		4	SS	35		
		5	SS	95		
		6	SS	43		
248.4 / 5.9	GRANITE GNEISS Slightly weathered to fresh Strong to very strong Fine to coarse crystalline Foliated black/white/pink Bedrock cored from 5.9m to 9.0m For bedrock coring details refer to Record of Drillhole BH13					
245.3 / 9.0	End of Borehole Note: 1. Water level in open borehole at 1.20m depth upon completion of drilling operations.					

Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; ELEVATION SCALE (254 to 246); 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

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

MIS-MTO 001 041111039AAMTO.GPJ GAL-MISS.GDT 21/8/06

PROJECT: 04-1111-039

RECORD OF DRILLHOLE: BH13

LOCATION: N 4973560.4 ;E 315684.3

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: November 14, 2005

DRILL RIG: CME 55 Bombardier

DRILLING CONTRACTOR: Walker Drilling Ltd.

DATUM: Geodetic

JN - Joint, FLT - Fault, SHR- 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, MB- Mechanical Break, BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
	- continued from Record of Borehole -	248.37			
6	GRANITE GNEISS Slightly weathered to fresh Strong to very strong Fine to coarse crystalline Foliated black/white/pink	5.94	1	FO,PL,Ro JN,PL,Ro JN,PL,Ro	Axial
7					
8			2	FO,PL,Ro	Axial
9	End of Drillhole	245.31 9.00			
10					
11					
12					
13					
14					
15					

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: SB

CHECKED: KB

Golder Associates

RECORD OF BOREHOLE No BH14

 METRIC

PROJECT 04-1111-039

W.P. 314-00-00 LOCATION N 4973557.3 ;E 315682.2 ORIGINATED BY SB

DIST HWY 11 BOREHOLE TYPE 108 mm I.D. Hollow Stem Power Auger COMPILED BY MSM

DATUM Geodetic DATE November 11, 2005 CHECKED BY KB

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
254.1	GROUND SURFACE						
0.0	SAND, trace silt Compact Brown Moist					254	
		1	SS	20		253	
		2	SS	20		252	
		3	SS	21			
251.2 2.9	Silty SAND to SAND and SILT, trace clay Dense to very dense Brown Wet	4	SS	51	▽	251	
		5	SS	51		250	
		6	SS	44		249	
		7	SS	70		248	0 61 37 2
247.5 6.6	BIOTITE GNEISS Slightly weathered to fresh Strong to very strong Fine to medium crystalline Foliated black/white Bedrock cored from 6.6m to 9.2m For bedrock coring details refer to Record of Drillhole BH14					247 246	
244.9 9.2	End of Borehole Note: 1. Water level in open borehole at 3.20m depth upon completion of drilling operations.					245	

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O 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.

PROJECT: 04-1111-039

RECORD OF DRILLHOLE: BH14

SHEET 2 OF 2

LOCATION: N 4973557.3 ;E 315682.2

DRILLING DATE: November 11, 2005

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55 Bombardier

DRILLING CONTRACTOR: Walker Drilling Ltd.

JN - Joint, FLT - Fault, SHR- 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, MB- Mechanical Break, BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR / % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3 m	B Angle	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec	Diametral Point Load Index (MPa)	RMC -Q' AVG.	NOTES WATER LEVELS INSTRUMENTATION
		- continued from Record of Borehole -		247.58														
		BIOTITE GNEISS Slightly weathered to fresh Strong to very strong Fine to medium crystalline Foliated black/white Coarse quartz - rich banding		6.55														Axial
7					1									FO,PL,Ro				
														JN,PL,Ro FO,PL,SM				
8														JN,IR,Ro				
					2		<<							JN,IR,Ro				Axial
9				244.93 9.20										,PL,SM-Ro				
		End of Drillhole																
10																		
11																		
12																		
13																		
14																		
15																		
16																		

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: SB

CHECKED: KB

Golder Associates

RECORD OF BOREHOLE No BH15

 METRIC

PROJECT 04-1111-039
W.P. 314-00-00
LOCATION N 4973554.1 ;E 315680.4
ORIGINATED BY SB
DIST HWY 11
BOREHOLE TYPE 108 mm I.D. Hollow Stem Power Auger
COMPILED BY MSM
DATUM Geodetic
DATE November 11, 2005
CHECKED BY KB

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
254.0	GROUND SURFACE					
0.0	SAND, trace to some silt and clay Compact Brown becoming grey at 1.83m depth Moist to wet	1	SS	26		0 87 (10)
		2	SS	26	∇	
		3	SS	19		
		4	SS	14		
250.2 3.8	Silty SAND to SAND and SILT Dense to very dense Brown/Grey Wet	5	SS	67		
		6	SS	45		
		7	SS	40/0.05		
247.1 6.9	End of Borehole Auger Refusal at 6.86 m. Note: 1. Water level in open borehole at 2.29 m upon completion of drilling operations.					

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O 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
ELEVATION SCALE: 253 252 251 250 249 248

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

Golder Associates

RECORD OF BOREHOLE No BH16

 METRIC

PROJECT 04-1111-039

W.P. 314-00-00 LOCATION N 4973544.9 ;E 315734.1 ORIGINATED BY SB

DIST HWY 11 BOREHOLE TYPE 108 mm I.D. Hollow Stem Power Auger COMPILED BY MSM

DATUM Geodetic DATE November 10, 2005 CHECKED BY KB

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
254.1	GROUND SURFACE					
0.0	Silty SAND to SAND and SILT, trace clay Dense to very dense Brown to grey Moist to wet	1	SS	50		
	Becoming grey at 1.52m depth	2	SS	50		
	Becoming wet at 1.98m depth	3	SS	41		
		4	SS	30		
		5	SS	39		0 37 59 4
249.5 4.6	SAND Very loose to dense Grey Wet	6	SS	34	∇	
		7	SS	13		
		8	SS	WH		
245.0 9.1	BIOTITE GNEISS and BIOTITE SCHIST Slightly weathered to fresh Medium strong to very strong Fine to medium crystalline Foliated black/white Bedrock cored from 9.1m to 12.2m For bedrock coring details refer to Record of Drillhole BH16					
241.9 12.2	End of Borehole Note: 1. Water level in open borehole at 4.80m depth upon completion of drilling operations.					

Column headings also printed: STRAT PLOT; ELEVATION SCALE (254 to 242); 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

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

PROJECT: 04-1111-039

RECORD OF DRILLHOLE: BH16

SHEET 2 OF 2

LOCATION: N 4973544.9 ;E 315734.1

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: November 10, 2005

DRILL RIG: CME 55 Bombardier

DRILLING CONTRACTOR: Walker Drilling Ltd.

DATUM: Geodetic

JN - Joint, FLT - Fault, SHR- 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, MB- Mechanical Break, BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
	- continued from Record of Borehole -				
	BIOTITE GNEISS Slightly weathered to fresh Strong to very strong Fine to medium crystalline Foliated black/white Coarse quartz - rich banding	244.96 9.14		JN,PL,Ro	Axial
10	BIOTITE SCHIST Slightly weathered to fresh Medium strong Fine to medium crystalline Foliated black	244.60 9.50	1	JN,PL,Ro ,PL,Ro JN,IR,Ro ,IR,Ro JN,IR,Ro ,IR,Ro ,PL,Ro	Axial
11			2	JN,IR,Ro ,PL,Ro ,PL,Ro	Axial
12				FO,PL,Ro	
	End of Drillhole	241.91 12.19			

Note:

1. Broken core encountered at following intervals:

- 9.5m to 9.6m
- 9.7m to 9.9m
- 10.1m to 10.2m

Depth scale: 13, 14, 15, 16, 17, 18, 19

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: SB

CHECKED: KB

Golder Associates

RECORD OF BOREHOLE No BH17

 METRIC

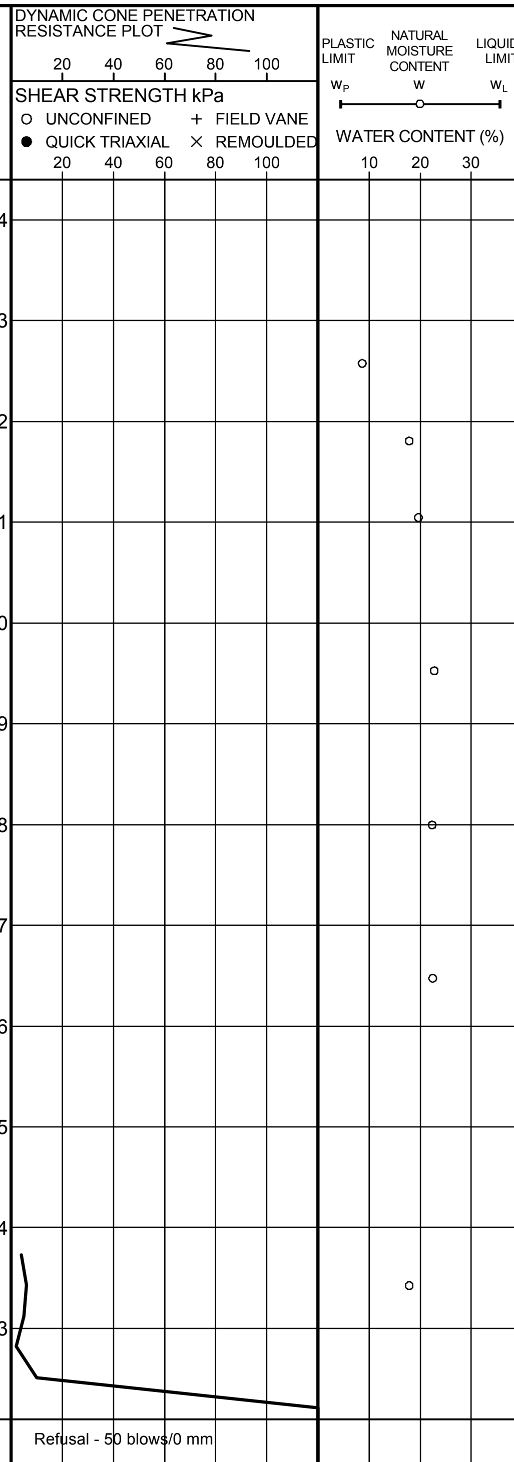
PROJECT 04-1111-039

W.P. 314-00-00 LOCATION N 4973539.1 ;E 315746.1 ORIGINATED BY SB

DIST HWY 11 BOREHOLE TYPE 108 mm I.D. Hollow Stem Power Auger COMPILED BY MSM

DATUM Geodetic DATE November 17, 2005 CHECKED BY KB

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	W (%)	GR	SA	SI	CL
254.4	GROUND SURFACE									
0.0	Silty SAND to SAND and SILT, trace clay Compact to very dense Brown and grey Moist to wet	1	SS	10						
		2	SS	51						
		3	SS	37			0	40	57	3
251.5 2.9	SAND, trace silt Dense to very dense Grey Wet	4	SS	55	∇					
		5	SS	46						
		6	SS	52						
248.9 5.5	Silty SAND to SAND and SILT, trace clay Loose to compact Grey Wet	7	SS	23			0	46	53	1
		8	SS	3						
		9	SS	7						
		10	SS	25						
242.8 11.6	End of Borehole									
242.1 12.3	End of DCPT Refusal of Dynamic Cone Penetration Test (DCPT) Note: 1. Water level in open borehole at 3.0m depth upon completion of drilling operations.									



UNIT WEIGHT γ kN/m³

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

FIELD TEST PIT LOG

JOB NUMBER:	04-1111-039	**JOB NAME:**	Highway 11/169 Bridge/Gravenhurst	**DATE:**	November 11, 2005
TEST PIT NUMBER:	TP1	**LOCATION:**	N 4973552.8; E315735.9	**ELEVATION:**	256.7
MACHINE TYPE:	CASE 9030 BACKHOE	**TEST PIT SIZE:**	0.9 m deep X 1 m wide X 2 m long	**DATUM:**	Geodetic
TEMP/WEATHER:	Overcast	**CONTRACTOR:**	MTO Patrol Yard		

Depth		Soil Description	Samples		In-situ Density Test		Remarks
From (m)	To (m)		No.	Depth (m)	No.	Depth (m)	
0.00	0.30	Topsoil	1	0.00 – 0.30			
0.30	0.91	Brown Sand, trace silt to Silty Sand	2	0.30 – 0.91			
0.91	0.92	Bedrock					

Comments:

Bedrock was sharply sloping

Water Conditions in Test Pit:

Test Pit dry upon completion

☒ Test Pit dry

JOB No. 04-1111-039
TEST PIT No.: TP1
Technician: SB

Golder Associates

FIELD TEST PIT LOG

JOB NUMBER:	04-1111-039	**JOB NAME:**	Highway 11/169 Bridge/Gravenhurst	**DATE:**	November 11, 2005
TEST PIT NUMBER:	TP2	**LOCATION:**	N 4973550.8; E315735.4	**ELEVATION:**	256.3
MACHINE TYPE:	CASE 9030 BACKHOE	**TEST PIT SIZE:**	1.5 m deep X 1 m wide X 3 m long	**DATUM:**	Geodetic
TEMP/WEATHER:	Overcast	**CONTRACTOR:**	MTO Patrol Yard		

Depth		Soil Description	Samples		In-situ Density Test		Remarks
From (m)	To (m)		No.	Depth (m)	No.	Depth (m)	
0.00	0.30	Topsoil	1	0.00 – 0.30			
0.30	1.52	Brown Sand, trace silt to Silty Sand	2	0.30 – 1.52			
1.52	1.53	Bedrock					

Comments:

Bedrock was sharply sloping

Water Conditions in Test Pit:

Test Pit dry upon completion

☒ Test Pit dry

JOB No. 04-1111-039
TEST PIT No.: TP2
Technician: SB

Golder Associates

MIS_TPS_001 04111039AATPS.GPJ GAL-MISS.GDT 3/1/06

DRAWINGS

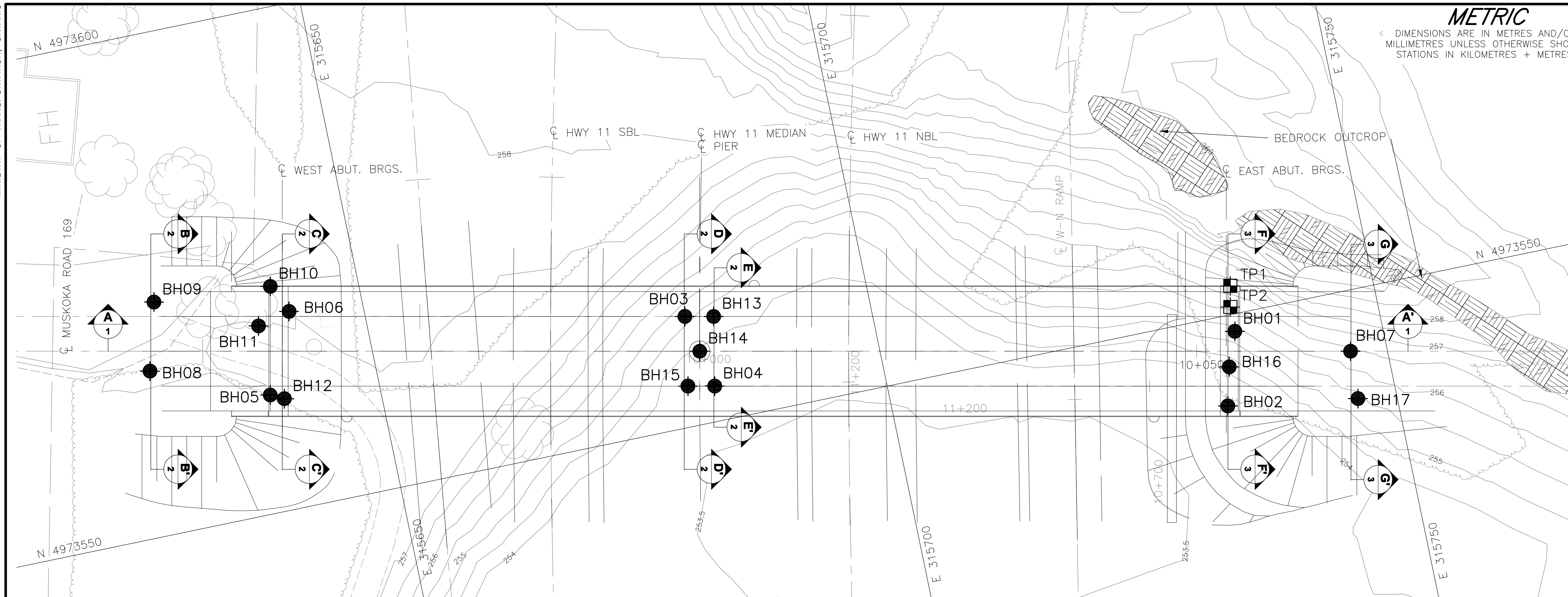
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No.314-00-00

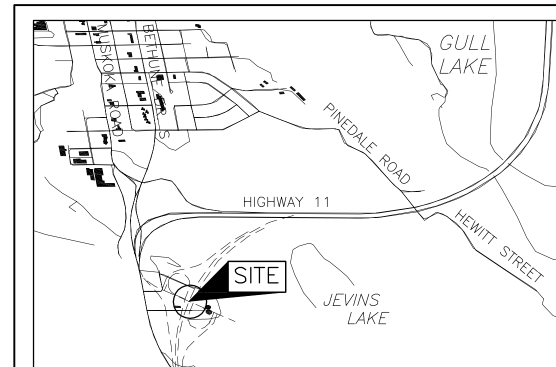
SHEET

HIGHWAY 11
HWY 11/MUSKOKA ROAD 169 UNDERPASS
BOREHOLE LOCATIONS AND SOIL STRATA

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



PLAN
SCALE 5 0 5 10 m

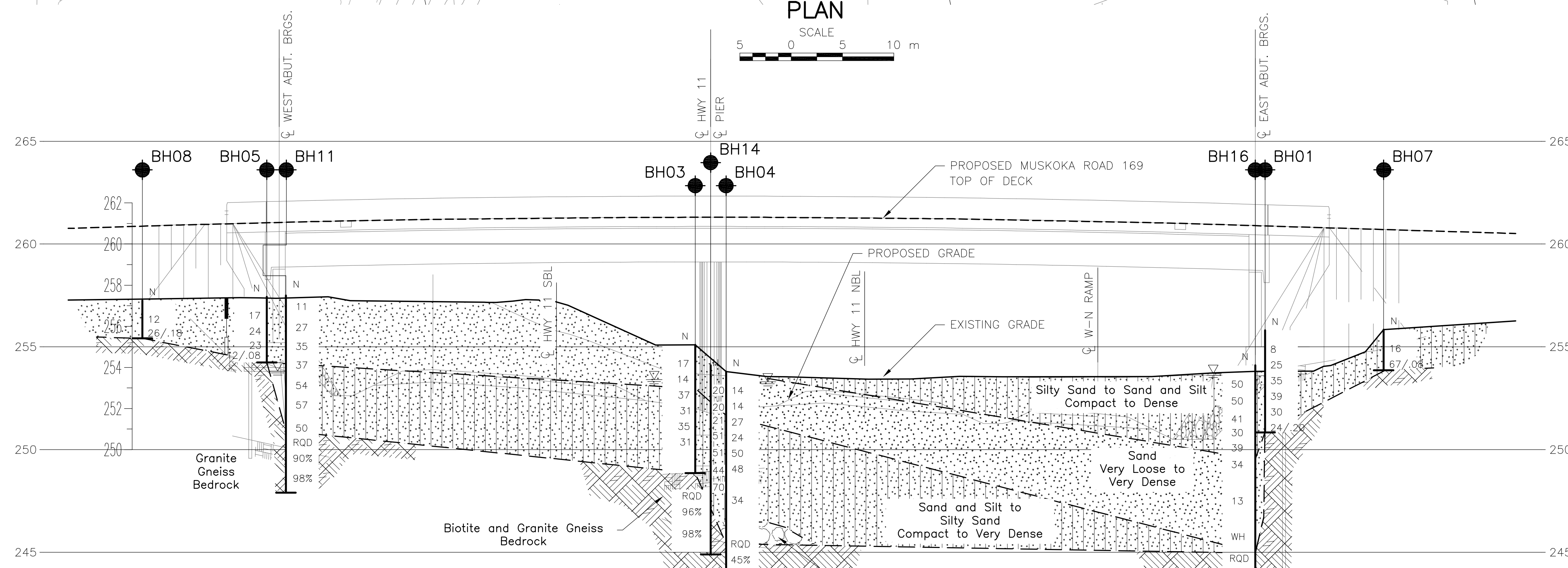


KEY PLAN
APPROX. SCALE 1 : 50,000
0.5 0 0.5 1 km

LEGEND

Borehole – Current Investigation
Test Pit – 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 Sept 30, 2005
WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
BH01	255.8	4973548.3	315735.4
BH02	253.8	4973541.1	315733.2
BH03	255.1	4973561.0	315681.5
BH04	253.8	4973553.5	315683.0
BH05	257.4	4973561.8	315639.0
BH06	257.6	4973569.6	315642.6
BH07	255.8	4973544.0	315746.4
BH08	257.3	4973566.6	315627.7
BH09	257.7	4973573.3	315629.5
BH10	257.8	4973572.3	315640.4
BH11	257.5	4973568.8	315639.3
BH12	257.5	4973561.1	315640.4
BH13	254.3	4973560.4	315684.3
BH14	254.1	4973557.3	315682.2
BH15	254.0	4973554.1	315680.4
BH16	254.1	4973544.9	315734.1
BH17	254.4	4973539.1	315746.1
TP1	256.7	4973552.8	315735.9
TP2	256.3	4973550.8	315735.4



NOTES

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.

For subsurface information only. The proposed structure details are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contract Documents

A-A' 1 PROFILE ALONG MUSKOKA ROAD ℄

SCALE
HORT. 5 0 5 10 m

SCALE
VERT. 2.5 0 2.5 5 m

REFERENCE

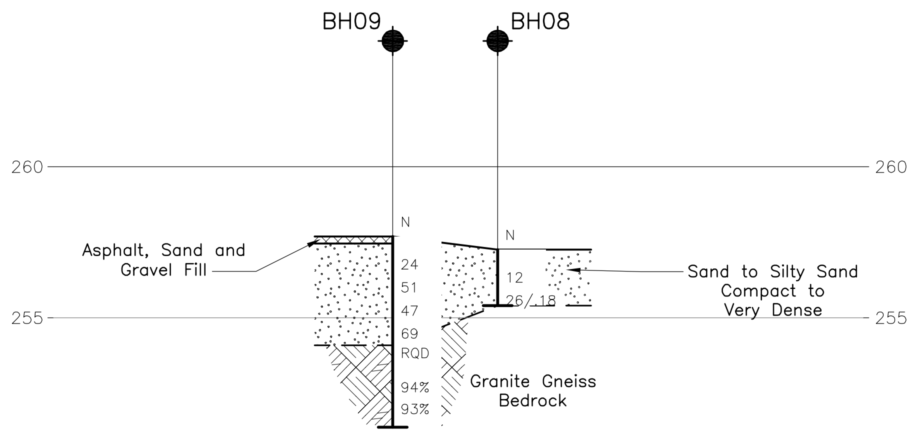
Base plans and General Arrangement dwg provided in digital format by McCORMICK RANKIN CORPORATION, drawing file nos. 5799–align–dec13–04.dwg , 5799–sp plan.dwg and 05799_XB1.dwg, received December 15, 2004. General Arrangement dwg file no. 5799–303–001.dwg received May 5, 2006.

NO.	DATE	BY	REVISION

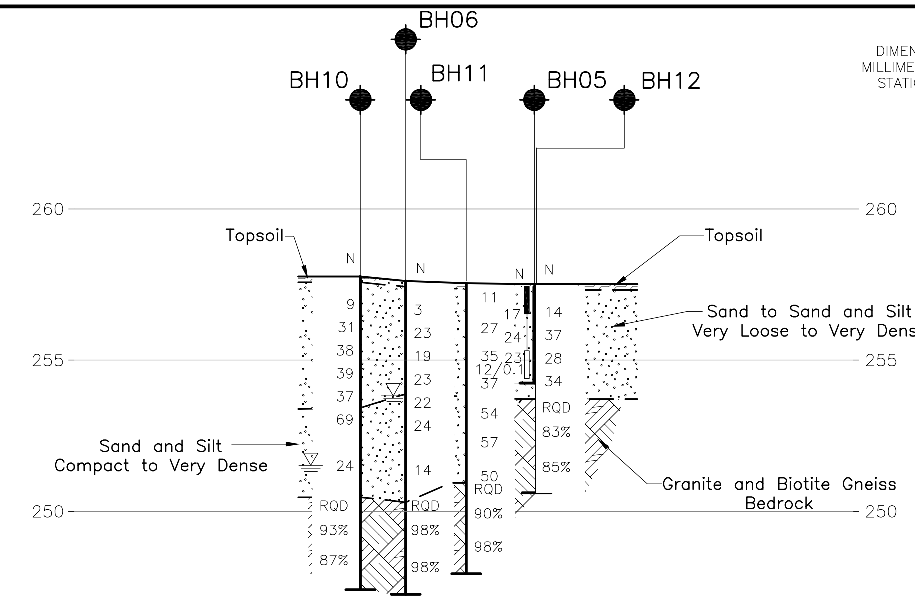
Geocres No. 31D–415			
HWY. 11	PROJECT NO. 04–1111–039D		DIST.
SUBM'D. KJB	CHKD. JPD	DATE: AUG 2006	SITE: 42–139
DRAWN: MSM	CHKD. KJB	APPD.	DWG. 1

METRIC

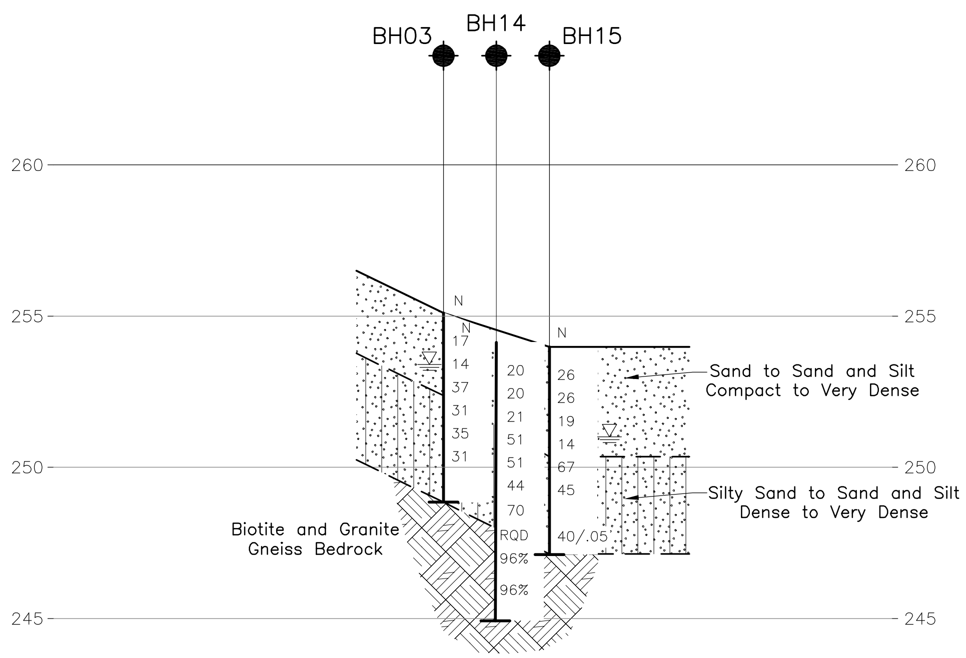
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.



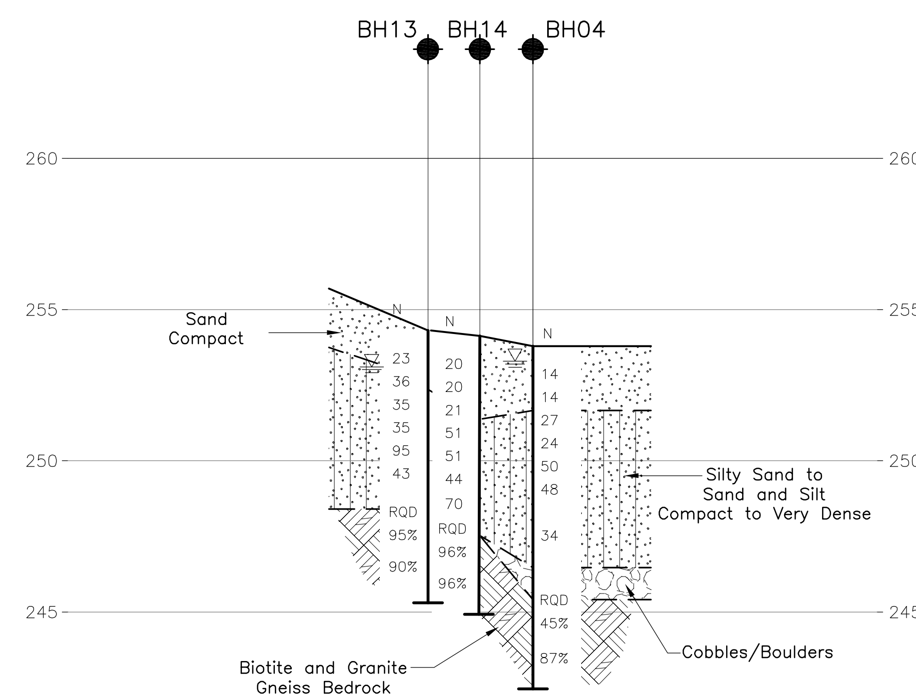
(B-B' / 1) SECTION ALONG WEST APPROACH



(C-C' / 1) SECTION ALONG WEST ABUTMENT



(D-D' / 1) SECTION ALONG CENTRE PIER (WEST SIDE)



(E-E' / 1) SECTION ALONG CENTRE PIER (EAST SIDE)

SCALE
HORT. 5 0 5 10 m

SCALE
VERT. 2.5 0 2.5 5 m

NOTES

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.

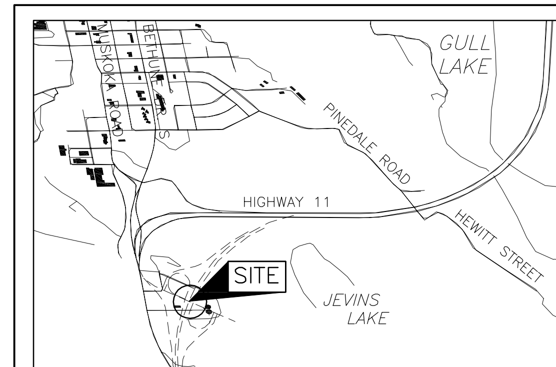
For subsurface information only. The proposed structure details are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contract Documents.

CONT No.
WP No.314-00-00

SHEET

HIGHWAY 11
HWY 11/MUSKOKA ROAD 169 UNDERPASS
BOREHOLE SOIL STRATA

Golder Associates
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN

APPROX. SCALE 1 : 50,000
0.5 0 0.5 1 km

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 Sept 30, 2005
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES NORTHING	CO-ORDINATES EASTING
BH03	255.1	4973561.0	315681.5
BH04	253.8	4973553.5	315683.0
BH05	257.4	4973561.8	315639.0
BH06	257.6	4973569.6	315642.6
BH08	257.3	4973566.6	315627.7
BH09	257.7	4973573.3	315629.5
BH10	257.8	4973572.3	315640.4
BH11	257.5	4973568.8	315639.3
BH12	257.5	4973561.1	315640.4
BH13	254.3	4973560.4	315684.3
BH14	254.1	4973557.3	315682.2
BH15	254.0	4973554.1	315680.4

REFERENCE

Base plans and General Arrangement dwg provided in digital format by McCORMICK RANKIN CORPORATION, drawing file nos. 5799–align–dec13–04.dwg , 5799–sp plan.dwg and 05799_XB1.dwg, received December 15, 2004. General Arrangement dwg file no. 57989–302–001.dwg, received February 10, 2005, revised October, 2005.

NO.	DATE	BY	REVISION

Geocres No. 31D–415

HWY. 11	PROJECT NO. 04–1111–039D		DIST.
SUBM'D. KJB	CHKD. JPD	DATE: AUG 2006	SITE: 42–139
DRAWN: MSM	CHKD. KJB	APPD.	DWG. 2

METRIC

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

CONT No.
WP No.314-00-00

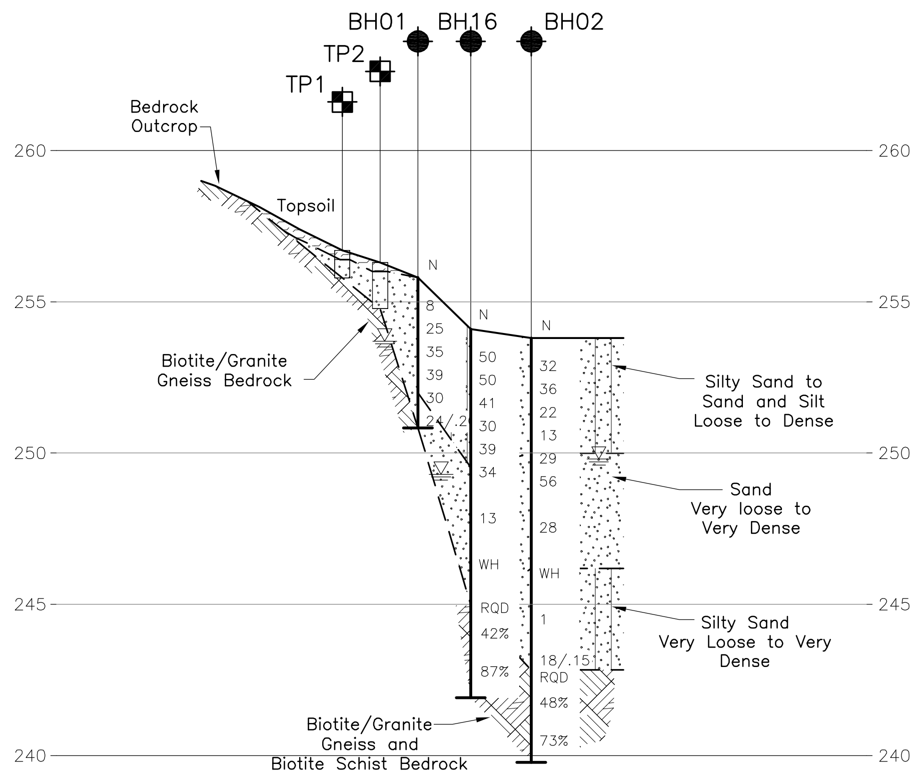
SHEET

HIGHWAY 11

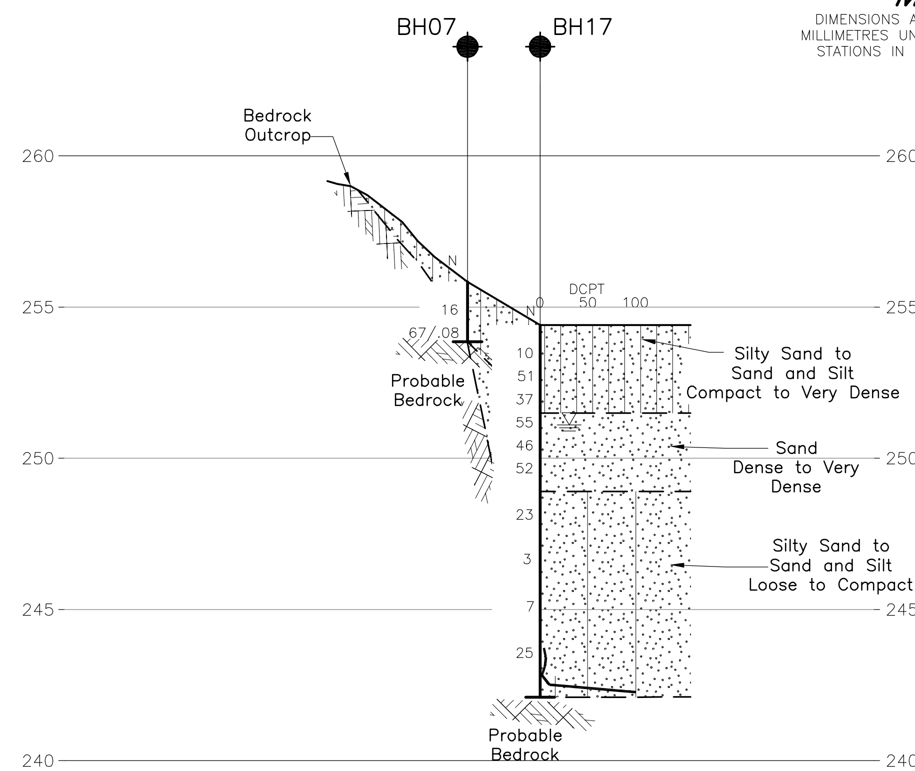
HWY 11/MUSKOKA ROAD 169 UNDERPASS

BOREHOLE SOIL STRATA

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



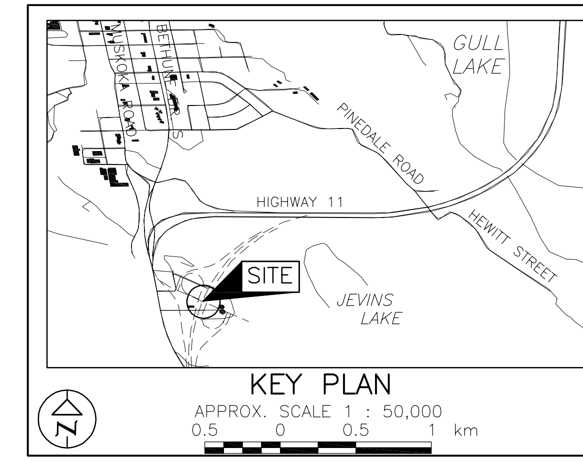
F-F' 1 SECTION ALONG EAST ABUTMENT



G-G' 1 SECTION ALONG EAST APPROACH

SCALE
HORT. 5 0 5 10 m

SCALE
VERT. 2.5 0 2.5 5 m



KEY PLAN

APPROX. SCALE 1 : 50,000
0.5 0 0.5 1 km

LEGEND

- Borehole – Current Investigation
- Test Pit – Current Investigation
- 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
TP1	256.7	4973552.8	315735.9
TP2	256.3	4973550.8	315735.4
BH01	255.8	4973548.3	315735.4
BH02	253.8	4973541.1	315733.2
BH07	255.8	4973544.0	315746.4
BH16	254.1	4973544.9	315734.1
BH17	254.4	4973539.1	315746.1

NOTES

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.

For subsurface information only. The proposed structure details are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contract Documents

REFERENCE

Base plans and General Arrangement dwg provided in digital format by McCORMICK RANKIN CORPORATION, drawing file nos. 5799-align-dec13-04.dwg , 5799-sp plan.dwg and 05799_XB1.dwg, received December 15, 2004. General Arrangement dwg file no. 57989-302-001.dwg, received February 10, 2005, revised October, 2005.

NO.	DATE	BY	REVISION

Geocres No. 31D-415

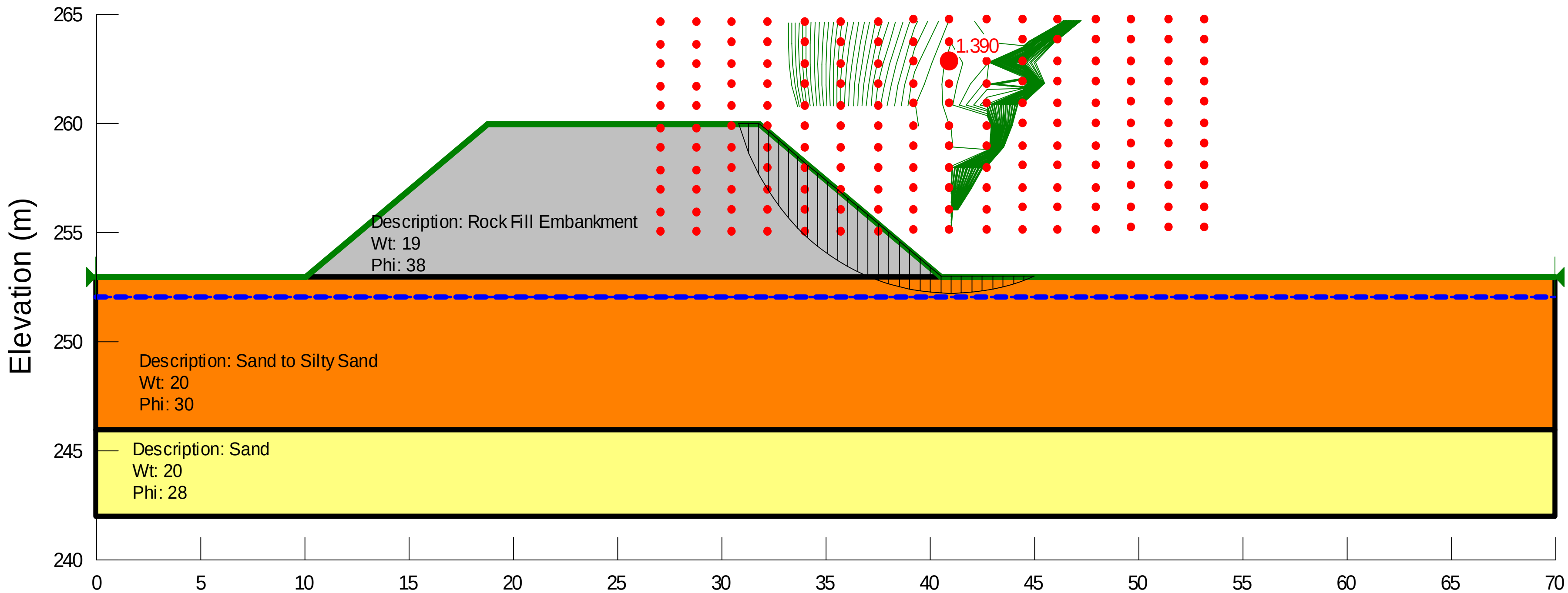
HWY. 11	PROJECT NO. 04-1111-039D		DIST.
SUBM'D. KJB	CHKD. JPD	DATE: AUG 2006	SITE: 42-139
DRAWN: MSM	CHKD. KJB	APPD.	DWG. 3

FIGURES

EMBANKMENT STABILITY ANALYSIS - EAST APPROACH
HIGHWAY 11 AND MUSKOKA ROAD 169 UNDERPASS
7.3 m HIGH ROCKFILL EMBANKMENT (1.25H:1V)

FIGURE 1

Description: 04-1111-039D Hwy 11 / Muskoka Road 169 IC Underpass - Gravenhurst
Comments: 7m High Rockfill Embankment 1.25H:1V
File Name: 04-1111-039D 7m High Rockfill Embankment.gsz
Analysis Method: Morgenstern-Price



NOT TO SCALE

Date: Aug-06
Project: 04-1111-039D

Golder Associates

Drawn: SLP
Checked: KJB

PLOT DATE: August 21, 2006
FILENAME: T:\Projects\2004\04-1111-039 (MRC, Gravenhurst)\-FA- (FIGURES)\041111039FA2.dwg

SCHEMATIC OF TYPICAL EMBANKMENT FILL DETAILS AT ABUTMENTS
HIGHWAY 11/MUSKOKA ROAD 169 UNDERPASS
G.W.P. 314-00-00

FIGURE 2

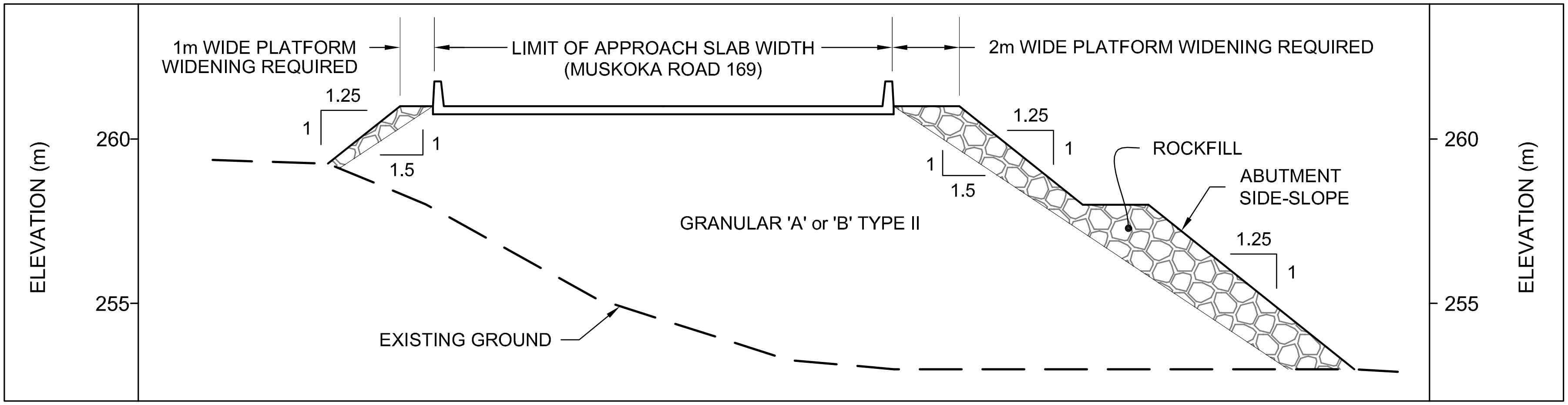


FIGURE 2A - TYPICAL SECTION ACROSS APPROACH SLAB

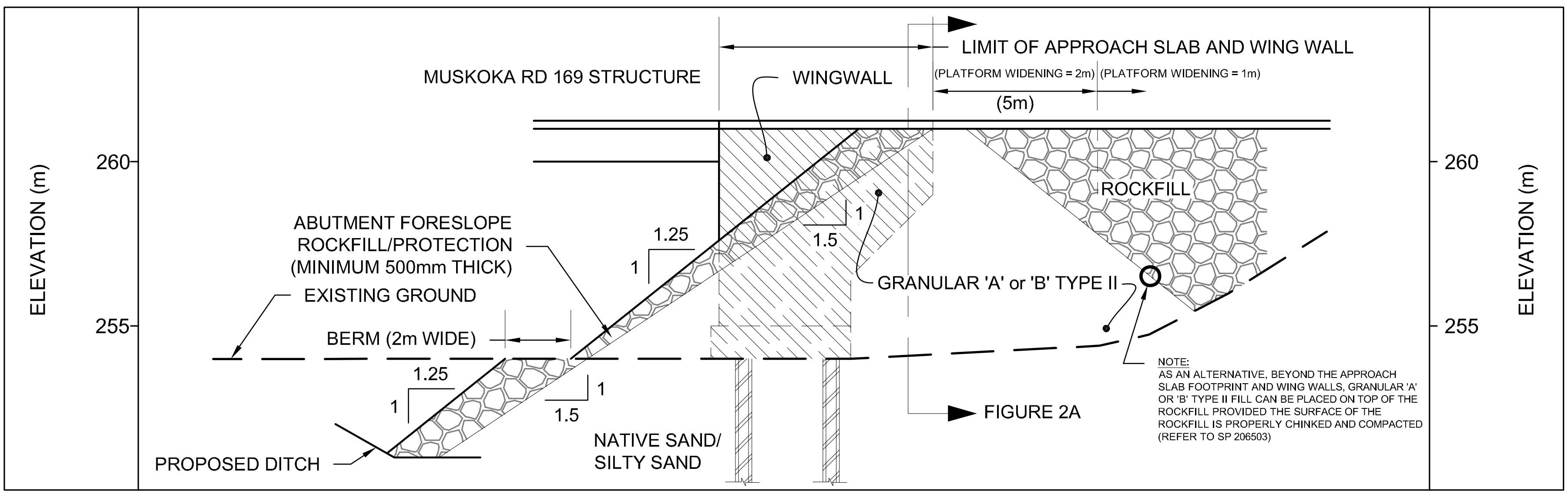


FIGURE 2B - TYPICAL PROFILE SECTION AT EAST ABUTMENT

DATE: AUG 2006
PROJECT: 04-1111-039

Golder Associates

4 0 4 8
SCALE METRES

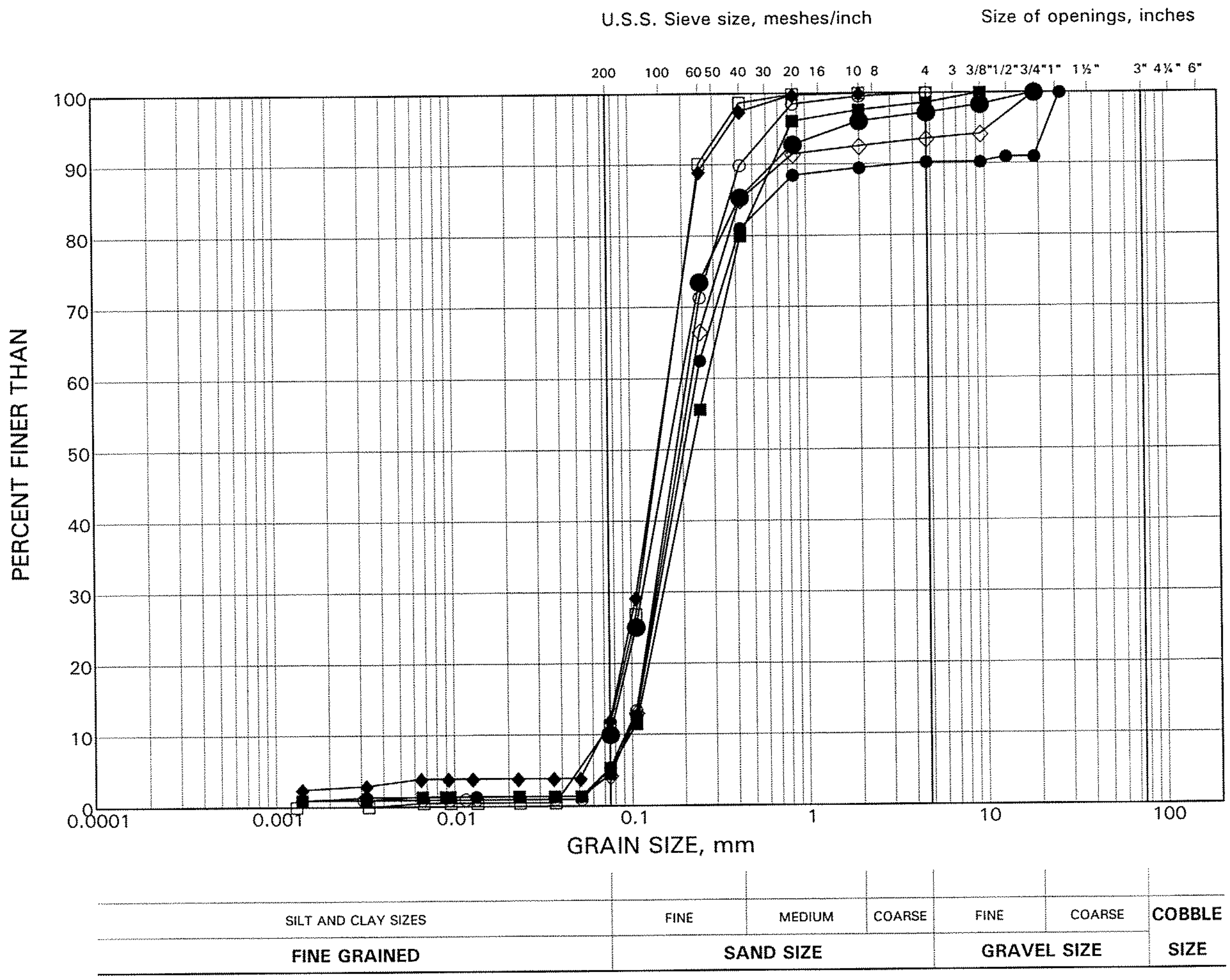
CAD: MSM
CHK: KJB

APPENDIX A

LABORATORY TEST DATA

GRAIN SIZE DISTRIBUTION
Sand

FIGURE A1



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	BH05	1	256.3
■	BH05	3	254.8
◆	BH06	3	255.0
○	BH08	2	255.6
□	BH11	4	247.6
◇	BH12	2	251.9
⬤	BH15	1	246.0

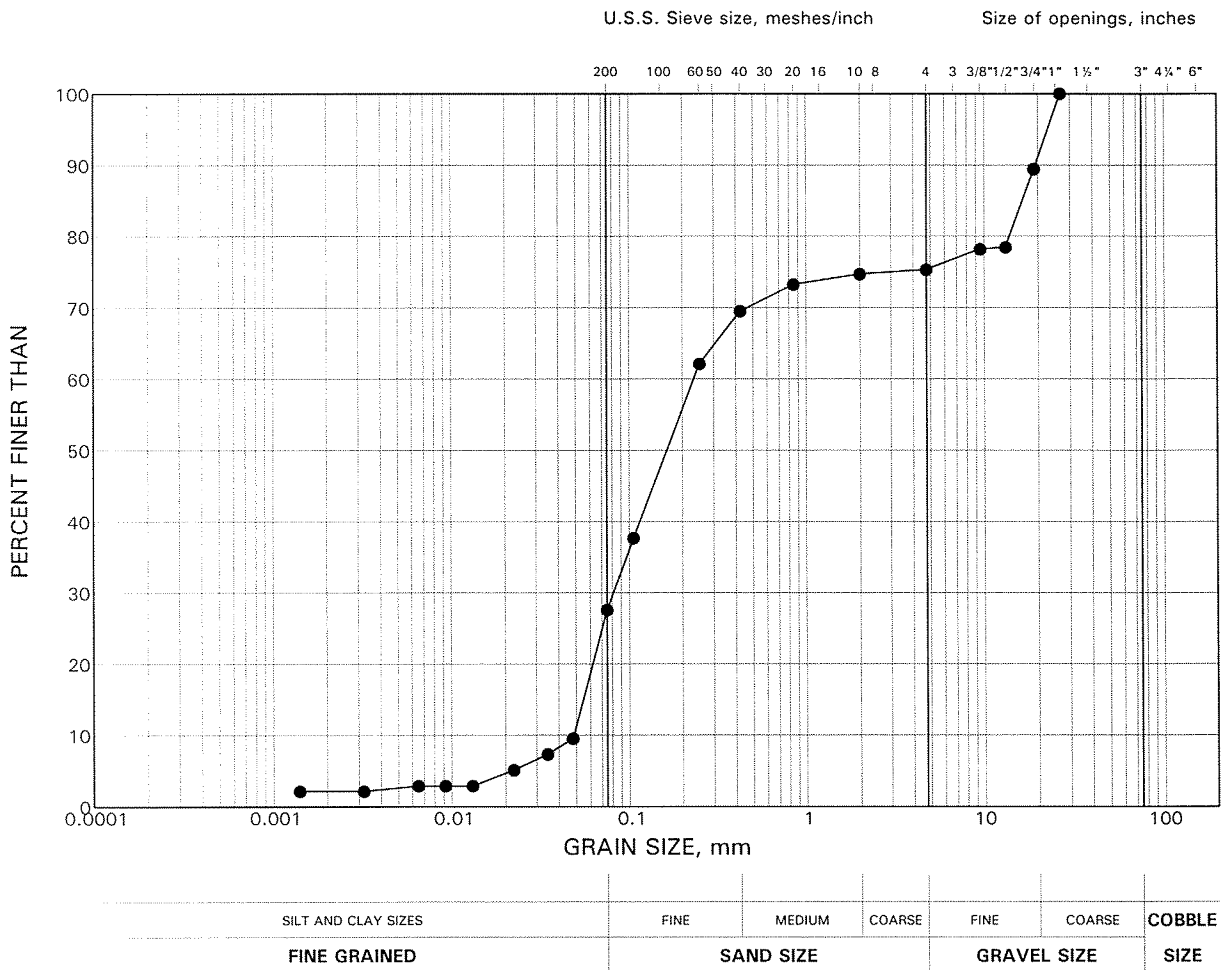
Date December, 2005
Project 04-1111-039

Golder Associates

Prepared by LG
Checked by SLP

GRAIN SIZE DISTRIBUTION
Silty Sand to Gravelly Sand

FIGURE A2



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	BH01	1	254.7

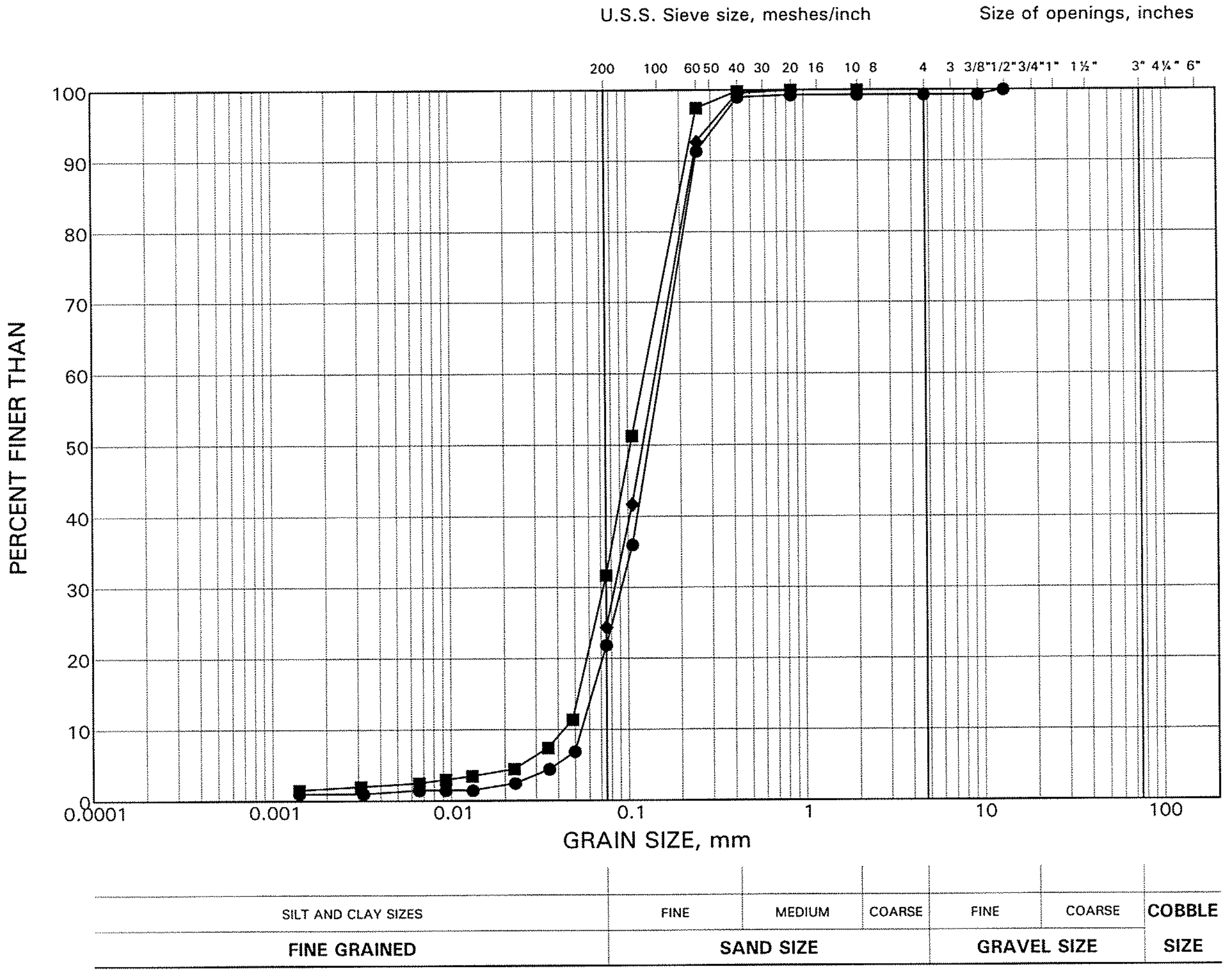
Date October, 2005
Project 04-1111-039

Golder Associates

Prepared by LG
Checked by KJB

GRAIN SIZE DISTRIBUTION
Silty Sand

FIGURE A3



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	BH02	8	246.0
■	BH04	3	251.2
◆	BH09	4	250.8

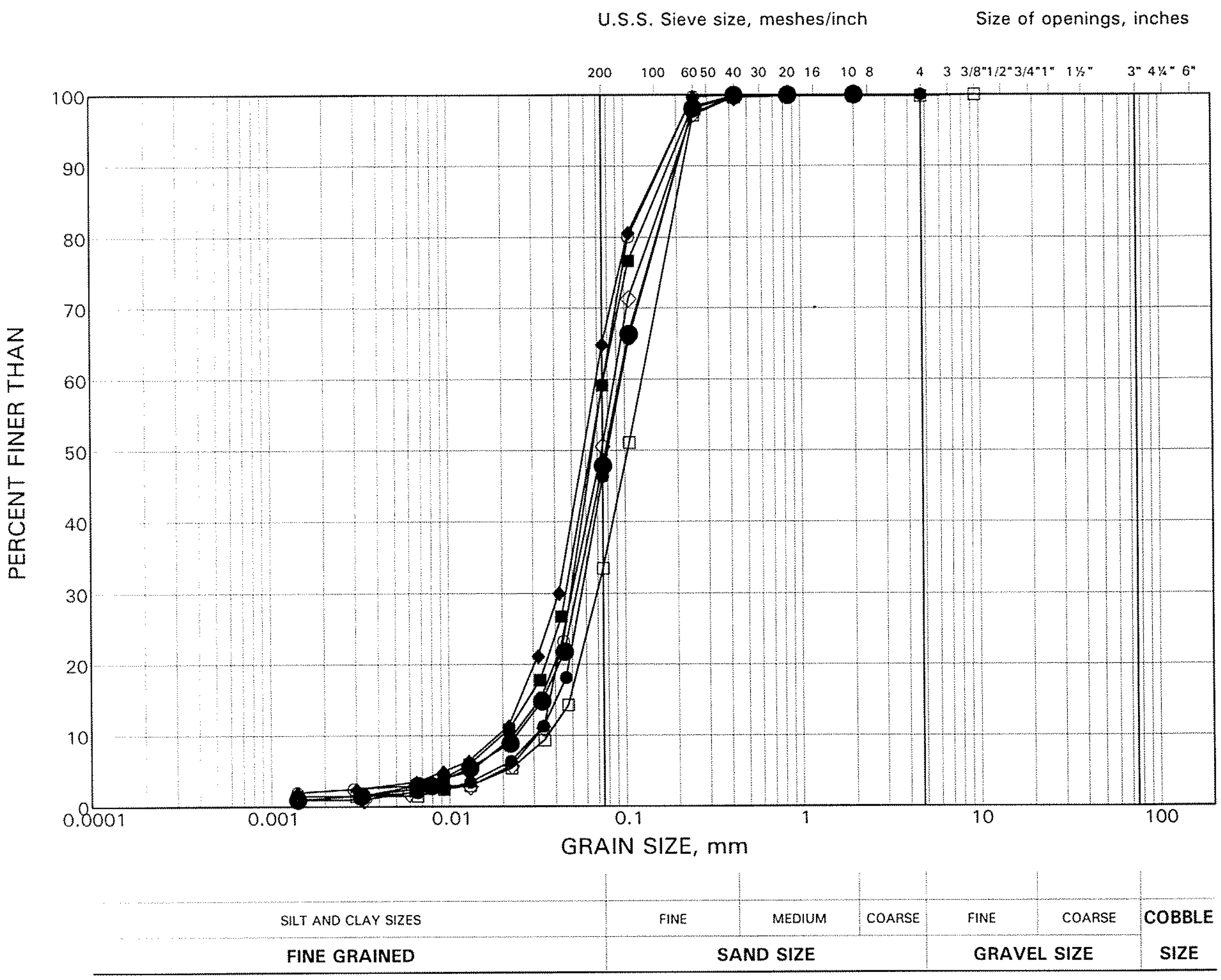
Date December, 2005
Project 04-1111-039

Golder Associates

Prepared by LG
Checked by KJB

GRAIN SIZE DISTRIBUTION
Sand and Silt

FIGURE A4



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	BH01	4	252.4
■	BH02	2	252.0
◆	BH03	4	251.7
○	BH03	6	250.2
□	BH04	5	249.7
◇	BH06	5	253.5
⬤	BH07	2	254.3

Date October, 2005

Project 04-1111-039

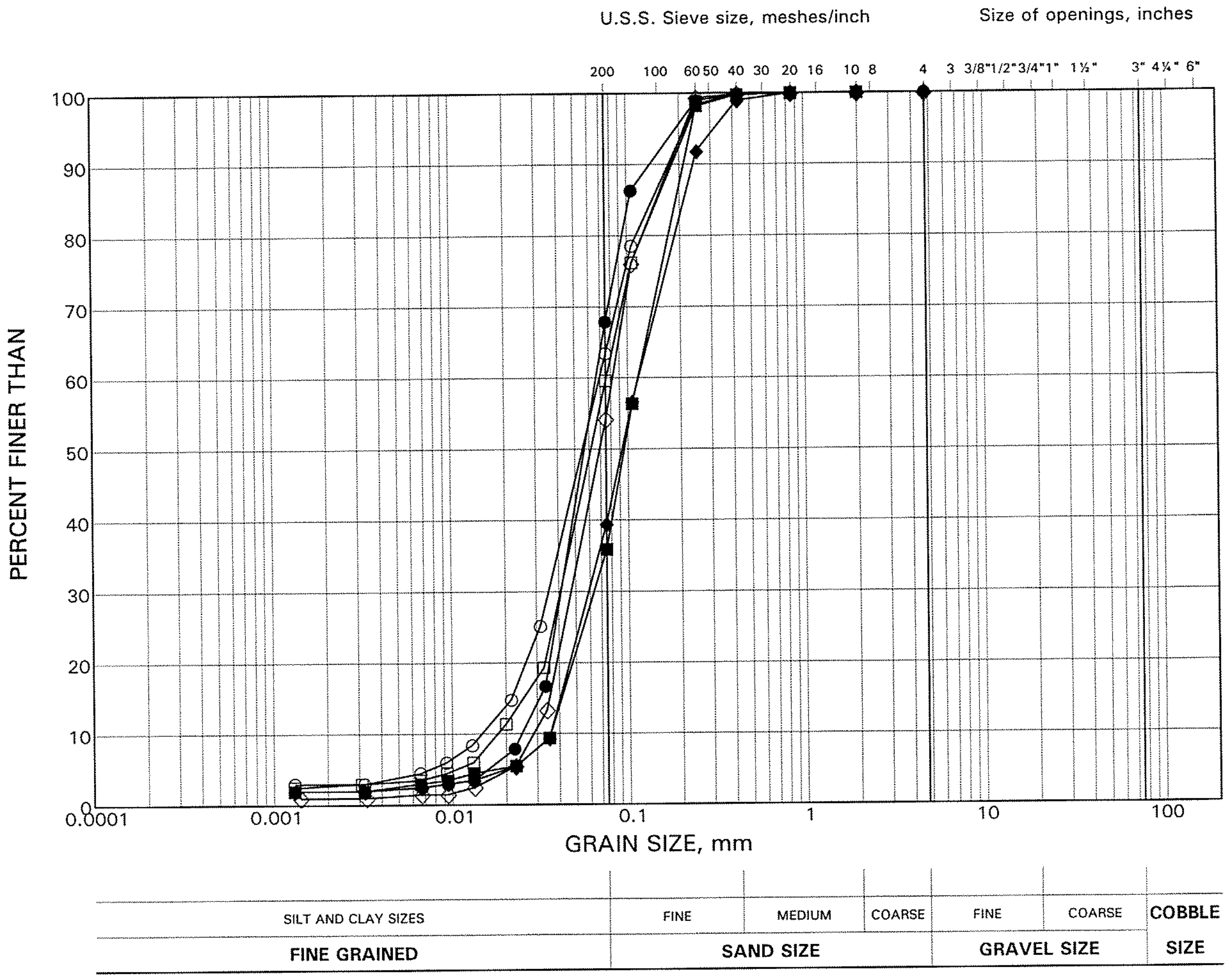
Golder Associates

Prepared by LG

Checked by KJB

GRAIN SIZE DISTRIBUTION
Sand and Silt

FIGURE A5



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	BH10	7	244.1
■	BH13	3	245.8
◆	BH14	7	241.1
○	BH16	5	237.8
□	BH17	3	239.5
◇	BH17	7	235.7

Date December, 2005
Project 04-1111-039

Golder Associates

Prepared by LG
Checked by SLP

APPENDIX B

NON-STANDARD SPECIAL PROVISIONS

MASS CONCRETE – Item No.

Special Provision

Scope of Work

The scope of work for the above noted tender item includes the mass concrete under the East and West abutment footings and the Central column footing. Tremied concrete methods are required at the Central column footing.

Construction

Concrete shall be of the same strength as the footing concrete and placed in accordance with OPSS 904.

Tremied concrete methods will be required for the placement of the mass concrete pad and installation of the rock dowels to support a shallow foundation, or to provide a seal for the pile cap if deep foundations are used. Prior to introducing the tremied concrete, consideration should be given to placing sand bags or dry concrete bags along the interface between the sheetpiles and underlying soil/rock in order to fill the voids (i.e. assuming sloping bedrock) and minimize disturbance and possible infilling of fines/soil at the founding level. Tremied concrete construction should be in accordance with SP 105S19 and OPSS 904, specifically Section 904.07.03.15.09.

For tremied concrete used to support foundation loads, the Contractor will perform at least two concrete cores through the tremied concrete for each foundation footprint to verify the concrete meets the design specifications (i.e. e.g. strength and consistency) to the Contract Administrator.

Basis of Payment

Payment at the contract price for the above noted tender item includes full compensation for all labour, equipment and materials to do the required work.

DOWELS Into Rock – Item No.

Special Provision

Scope of Work

Work under this item is for the placement and field testing of dowels into rock.

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 CSAG30.18, Grade 400).

Where dowels are to be placed in rock, holes 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 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.

Rock Dowel Testing

All proposed testing procedures shall be in general conformance with ASTM D 3689-90 and ASTM D 114381 (Re-approved 1994). 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.

Bridge	Foundation	Number of Dowels for Performance Testing
Highway 11 and Muskoka Road 169 Underpass	West Abutment	2
Highway 11 and Muskoka Road 169 Underpass	Central Pier	2
Highway 11 and Muskoka Road 169 Underpass	East 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, 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% of the theoretical elastic elongation of the free stressing and is less than the theoretical elongation of the free stressing length plus 50% of the bond length.

Basis of Payment

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

<u>ROCK POINTS</u> - Item No.

Non-Standard Special Provision

<u>Scope</u>

As part of the work under the above tender item, the Contractor shall supply Titus "Rock Injector Design" Pile Points on HP 310 x 110 Piles or equivalent. Piles will be driven to bedrock.

<u>References</u>

OPSS 906 – Structural Steel
SP903S01

<u>Materials</u>

The pile points shall be of the following:

Product	**Manufacturer**
HPP-R-12	Titus Steel Company Ltd. 6767 Invader Cr. Mississauga, ON Tel (905) 564-2446

(Or approved equivalent which includes Oslo Points as per OPSD 3000.201)

<u>Basis of Payment</u>

Payment at the Contract Price for the above tender items shall be full compensation for all labour, equipment and material to do the work.

BOULDERS/COBBLES DURING EXCAVATION, DRILLING, PILE INSTALLATION, ETC. - Item No.

Special Provision

The overburden soils at the site consist of water-bearing sand and gravel containing cobbles and boulders. In addition, the soils will be susceptible to cave-in, sloughing and boiling.

Appropriate equipment and procedures will be required to penetrate/remove cobbles/boulders that are encountered during excavation, augering/drilling, pile driving and/or sheet pile installation, etc.

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

CONTROLLED BLASTING and VIBRATION MONITORING at Foundation Locations and Permanent Rock Cuts – Item No.

Special Provision

Scope of Work

Work under this item is for the complete removal of rock using controlled blasting techniques by appropriate controlled drilling and blasting at locations indicated in the contract and disposal of rock material. This includes all rock removal required at the proposed foundation abutment locations.

Construction

The use of explosives shall follow the general specifications outlined in the latest version of OPSS 120.

Drilling equipment shall consist of the following:

> A hydraulic track drill or equivalent capable of drilling the required controlled blasting holes accurately and uniformly across the top of a rock cut, or other suitable equipment, given the site conditions.

Removal shall be carried out in such a manner to minimize disturbance to any surrounding rock beyond the excavation limits.

All material resulting from the operation shall be managed in accordance with OPSS 180 specified elsewhere in the contract.

All costs associated with the management of materials are deemed to be included in the contract unit price.

Trial blasting will be required for all proposed production and wall control blast procedures.

Monitoring and Reporting

Ground and air vibration monitoring is required during the blasting operations. Ground vibration levels should be limited to the maximum peak particle velocity values provided in Table 1 in OPSS 120 for adjacent services, bridges and buildings (i.e. 50 mm/s for frequencies greater than 40 Hz).

The Contractor shall submit the following information to the Contract Administrator at least 3 weeks in advance of rock excavation.

- Blast Contractor: contractor must be fully qualified, experienced and capable of working at heights with approved Ministry of Labour safety full arrest devices. A statement of experience is required;

- An outline of the requirements, procedure, and extent of the pre-blast survey required;

- Proposal prepared by blast contractor or blast consultant detailing the blast methodology, including drill hole patterns, hole size and depths, size of blasts, explosive and initiation product details, as well as all blast control procedures. Blast control procedures would include details on controlling flyrock, temporary road closures, blast signalling and site clearing procedures, as well as procedures to deal with debris clean-up; and

- Details on instrumentation, number and location of monitoring sites, blast recording and reporting procedures, and procedures to be followed in the event of excessive vibration readings.

Instrumentation or monitoring ground and air vibration effects from the blasting should be set up in accordance with the International Society of Explosives Engineers field practice guidelines (1999).

At all locations where structures (existing and proposed) are located adjacent to or within rock cuts, the new or existing rock cut faces and/or structure founding surfaces should be inspected by an independent rock engineering specialist and provisions made for rock bolting/dowelling, if necessary.

A minimum of 80 percent half barrels (drill hole traces) visible on the cut face after scaling is required.

Measurement of Payment

The measurement for payment shall be by Plan Quantity, as may be revised by Adjusted Plan Quantity of the volume or rock in m^3 measured in-place.

Basis of Payment

Payment at the contract price for the above noted tender item includes full compensation for all labour, equipment and materials to do the required work.

n:\active\2004\1111\04-1111-039 mrc hwy 11-169 ic gravenhurst\reports\hwy 11- muskoka road 169 bridge\final report\fidr\nssps\04-1111-039d nssp-controlledblastingatstructuresrev1.doc

Sloping Bedrock - Item No.

Non-Standard Special Provision

Scope

The contractor shall be alerted that the bedrock surface at the Highway 11 / Muskoka Road 169 Bridge site is variable and steeply sloping. Any foundations designed on bedrock should account for the varying founding elevations, pile lengths, etc.

Basis of Payment

Payment at the contract price for the above tender item shall be full compensation for all labour, equipment and materials required to do the work.