



May 10, 2010

CELEBRATING
50
YEARS
in 2010

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

ENGLEHART (BLANCHE) RIVER BRIDGE REPLACEMENT
HIGHWAY 573, SITE NO. 47-029
MUNICIPALITY OF CHARLTON AND DACK, ONTARIO
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5109-05-00

Submitted to:
URS Canada Inc.
75 Commerce Valley Drive East
Markham, Ontario
L3T 7N9



GEOCRES NO.: 31M-85

Report Number: 09-1191-0027-2

Distribution:

- 5 Copies - Ministry of Transportation, Ontario, North Bay, ON (Northeastern Region)
- 1 Copy - Ministry of Transportation, Ontario, Downsview, ON (Foundations Section)
- 2 Copies - URS Canada Inc., Markham, ON
- 2 Copies - Golder Associates Ltd., Sudbury, ON

REPORT



A world of
capabilities
delivered locally





Table of Contents

PART A – FOUNDATION INVESTIGATION REPORT

1.0 INTRODUCTION.....	1
2.0 SITE DESCRIPTION.....	1
3.0 INVESTIGATION PROCEDURES	2
4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS	3
4.1 Regional Geology	3
4.2 Subsurface Conditions.....	3
4.2.1 Asphalt	3
4.2.2 Fill	4
4.2.3 Sand and Silt to Silt.....	4
4.2.4 Refusal/Bedrock.....	4
4.2.5 Groundwater Conditions	5
5.0 CLOSURE.....	5

PART B – FOUNDATION DESIGN REPORT

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS.....	7
6.1 General.....	7
6.2 Shallow Foundations	8
6.2.1 Geotechnical Axial Resistance.....	8
6.2.2 Resistance to Lateral Loads.....	8
6.2.3 Frost Protection.....	9
6.3 Lateral Earth Pressures for Design.....	9
6.3.1 Static.....	9
6.3.2 Dynamic	10
6.4 Approach Embankment Design	11
6.4.1 Subgrade Preparation and Embankment Construction.....	11
6.4.2 Approach Embankment Stability	11
6.4.3 Approach Embankment Settlement.....	12
6.5 Detail Design and Construction Considerations.....	12



FOUNDATION REPORT - ENGLEHART (BLANCHE) RIVER BRIDGE REPLACEMENT - SITE 47-029, HIGHWAY 573, GWP 5109-05-00

6.5.1	Additional Investigation Requirements.....	12
6.5.2	Excavations.....	12
6.5.3	Groundwater and Surface Water Control for Foundation Excavation.....	12
6.5.4	Temporary Shoring	13
6.5.5	Footing Subgrade Preparation	13
6.5.6	Obstructions.....	13
6.5.7	Removal of Existing Bridge	13
7.0	CLOSURE.....	13

DRAWINGS

Drawing 1 Englehart (Blanche) River, Highway 573, Bridge 47-029, Borehole Location and Soil Strata

APPENDICES

Appendix A Record of Boreholes

List of Symbols and Abbreviations

Lithological and Geotechnical Rock Description Terminology

Record of Borehole and Drillhole Sheets

Appendix B Laboratory Test Results

Figure B-1 Grain Size Distribution – Gravelly Sand (Fill)

Figure B-2 Grain Size Distribution – Sand and Silt



PART A

FOUNDATION INVESTIGATION REPORT
ENGLEHART (BLANCHE) RIVER BRIDGE REPLACEMENT
HIGHWAY 573, SITE NO. 47-029
MUNICIPALITY OF CHARLTON AND DACK, ONTARIO
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5109-05-00



1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by URS Canada Inc. (URS) on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary design services for the replacement of three (3) structures carrying Highway 573 over Englehart (Blanche) River in the Municipality of Charlton and Dack (northwest of Englehart), Ontario. This report addresses Bridge Site No. 47-029, the middle of the three structures.

The terms of reference and scope of work for the foundation investigation are outlined in MTO's Request for Proposal (RFP) dated November 2008 and Request for Clarification letter dated December 29, 2009. Golder's proposal P81-1712, dated January 2009, for foundation engineering services associated with the bridge at site No. 47-029 is contained in Section 5.8 of URS' Technical Proposal that forms part of the Consultant's Agreement Number 5008-E-0026 for this project. The work was carried out in accordance with Golder's Supplemental Specialty Quality Control Plan for this project dated May 7, 2009. The General Arrangement drawing for the bridge structure was provided to Golder by URS in January 2010.

The purpose of this investigation is to establish the subsurface conditions at the proposed replacement structure by borehole drilling, in situ testing and laboratory testing on selected samples. The location of the site is shown on the key plan on Drawing 1.

2.0 SITE DESCRIPTION

The site is situated in the Township of Charlton and Dack on Highway 573 crossing the Englehart (Blanche) River, approximately 0.5 km west of the junction with Highway 560. The existing road grade is about 1.3 m above the river water level. The surrounding land is mainly used for recreational activities, with grass and tree cover extending beyond the limits of the site. The banks adjacent to the river are vegetated with grass and small shrubs and bedrock is exposed in several areas. At this bridge site, the river appears to be flowing along a channel blasted into the bedrock. The river is a regulated watercourse used for power generation by Kagawong Power Inc. A dam and footbridge are located to the south (upstream) of the existing bridge structure and the water flows from south to north.

The existing single-span concrete girder bridge was constructed in 1927 with a width of 6.4 m and a length of 8.7 m. The existing highway grade is at between about Elevation 259.2 m and 259.3 m and the water level upstream of the dam was measured at approximately Elevation 257.9 m in October 2009 at the time of drilling. The water level in the river, measured by others in September 2009, is Elevation 256.04 m at the location of the existing bridge.

The highway surface at the east abutment has been repaired by asphalt padding and a transverse crack is present across the highway at the east abutment wall. A slight trough is visible across the highway at the west abutment stem wall; however, asphalt padding is not evident in this area.



3.0 INVESTIGATION PROCEDURES

The fieldwork at the bridge site was carried out on October 6, 2009, at which time a total of four (4) boreholes (BH09-5 to BH09-8) were advanced at the site, two boreholes at each proposed abutment location. The borehole locations and groundwater surface elevations are shown on Drawing 1 and noted on the respective Record of Borehole and Drillhole sheets in Appendix A. All boreholes were drilled using a CME 75 truck-mounted drill rig supplied and operated by George Downing Estate Drilling Ltd. (Downing) of Grenville-Sur-La-Rouge, Quebec.

The boreholes were advanced using 108 mm inside diameter (I.D.) continuous flight hollow stem augers or NW casing with wash boring. Soil samples were obtained, where possible, at intervals of depth of 0.75 m, using a 50 mm outer diameter (O.D.) split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586). Auger samples were typically taken just below the asphalt or at the ground surface. Rock core samples at one borehole were obtained using an NQ size core barrel.

The boreholes were advanced to auger refusal at depths ranging from about 1.2 m to 2.4 m below the existing ground surface and 0.6 m of bedrock core was obtained from Borehole BH09-8.

The groundwater conditions in the open boreholes were observed during the drilling operations and a piezometer was installed in Borehole BH09-8, to allow monitoring of the groundwater level at this location. The piezometer consists of a 19 mm O.D. rigid PVC tubing with a 1.5 m long slotted screen and a flush mounted cap. The water level readings are presented on the Record of Borehole sheets in Appendix A. The boreholes were backfilled with bentonite as per Ontario Regulation 903 (as amended by O. Reg. 372) upon completion of drilling.

Traffic protection was carried out for the boreholes drilled within the roadway in accordance with our Traffic Protection Plan and the MTO Book 7 Temporary Conditions Manual.

The fieldwork was supervised throughout by members of our engineering and technical staff who located the boreholes, arranged for the clearance of underground service locations, supervised the drilling and sampling operations, logged the boreholes, and examined and cared for the soil samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Sudbury geotechnical laboratory where the samples underwent further visual examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate. Classification testing (water content and grain size distribution) was carried out on selected soil samples.

The locations of the proposed foundation elements were laid out in the field by Golder relative to the proposed abutment locations based on the dimensions shown on a preliminary drawing provided by URS dated October 2009. Golder surveyed the ground surface elevation of the boreholes once completed, referencing an existing benchmark located on the south concrete wing wall between Sites 47-030 and 47-029 (BM ONR No. 8010845206). The ground surface and water surface elevations are referenced to geodetic datum. The northing and easting coordinates (MTM NAD83) were determined by plotting the boreholes relative to the existing bridge on the January 2010 General Arrangement provided by URS and converting to the coordinate system. The northing and easting coordinates, ground surface elevations and the borehole depth are summarised below.



Borehole	Borehole Location		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing	Easting		
BH09-5	5297293.3	379710.5	259.3	1.2
BH09-6	5297298.1	379710.6	259.3	1.2
BH09-7	5297297.1	379722.9	259.2	1.9
BH09-8	5297301.2	379719.7	259.2	2.4

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

Published literature indicates that the site is located in the transition zone between the Western Abitibi Subprovince of the Superior Province (to the north) and the Huronian Supergroup (to the south). The bedrock geology follows the river valley and consists of mafic metavolcanic rock (Geology of Ontario; OGS Special Volume 4)¹.

Terrain mapping by the Ontario Geological Survey² describes the subsurface soils in the vicinity of the site as silty colluvial slopewash and debris creep sheet with minor talus.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions, as encountered in the boreholes advanced during this investigation, together with the results of the laboratory tests carried out on selected soil samples, are presented on the attached Record of Borehole and Drillhole sheets in Appendix A. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling and observations of drilling progress and cuttings. 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 based on the results of the boreholes is shown on Drawing 1.

In general, the subsoils at the structure site consist of asphalt underlain by granular fill and/or rock fill. A thin layer of native sand and silt or silt /organics was encountered below the rock fill in two boreholes.

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

4.2.1 Asphalt

All of the boreholes were drilled through the pavement or shoulder of the highway. In all boreholes, a layer of sand or gravelly sand between 60 mm and 150 mm thick was encountered above (i.e. at the ground surface) or within the asphalt layers. Approximately 210 mm to 240 mm of asphalt was encountered in the boreholes. The ground surface at the boreholes is at Elevation 259.2 m at the west abutment and at Elevation 259.3 m at the east abutment.

¹ Geology of Ontario, 1991. Ontario Geological Survey, Special Volume 4, Part 1. Eds. P.C. Thurston, H.R. Williams, R.H. Sutcliffe and G.M. Stott. Ministry of Northern Development and Mines, Ontario.

² Northern Ontario Engineering Geology Terrain Study, OGS Map Reference Numbers 5020 and 5021.



4.2.2 Fill

Fill was encountered in all boreholes either directly underlying the asphalt pavement or the granular shoulder material interlayered with the asphalt. The top of the fill was encountered between Elevation 258.9 m and 259.1 m. The fill material is comprised of brown gravelly sand containing trace silt or rock fill in a gravelly sand with silt matrix. On the west side of the river (BH09-5 and BH09-6), the fill is between 0.3 m and 0.9 m thick and on the east side of the river (BH09-7 and BH09-8), the fill is between 1.3 m and 1.6 m thick.

Several instances of difficult drilling or augers split-spoons sliding/dipping on obstructions were noted throughout advancement of the boreholes through the fill material at this site. Empty split-spoons were recorded in a few locations in the fill indicative of larger gravel sizes or rock fill. In Borehole BH09-8, casing had to be used from ground surface in the second attempt to advance the borehole.

SPT 'N'-values measured within the fill range from 17 to 22 blows per 0.3 m of penetration where the sampler was able to penetrate its full length, indicating a compact relative density. In one borehole, the split-spoon sampler did not penetrate the full sample depth due to the presence of rock fill.

The natural water content measured on samples of the fill ranges between about 7 percent and 16 percent.

A grain size distribution for one sample of the gravelly sand fill is shown on Figure B-1 in Appendix B.

4.2.3 Sand and Silt to Silt

In Borehole BH09-5, located on the west side of the river, a 0.7 m thick deposit of wet, brown (oxidized) sand and silt containing trace clay, trace gravel and cobbles and boulders was encountered beneath the fill at Elevation 258.8 m. In Borehole BH09-8, located on the east side of the river, a 0.1 m thick layer of silt and organics was encountered beneath the fill at Elevation 257.7 m.

The split-spoon sampler did not penetrate its full length in this deposit in BH09-5 due to the presence of cobbles and boulders within the deposit. The SPT 'N'-value recorded is 50 blows per 0.3 m of penetration, but the deposit is inferred to be loose in relative density.

The natural water content measured on one sample of the native soil was about 16 percent.

A grain size distribution for one sample of the native sand and silt is shown on Figure B-2 in Appendix B.

4.2.4 Refusal/Bedrock

All of the boreholes were terminated upon encountering auger refusal on either the inferred bedrock surface or within the rock fill. In Borehole BH09-5, located on the west side of the river, refusal is likely on the bedrock surface as the fill was penetrated to the native soil. In Borehole BH09-6, also located on the west side of the river, the augers were sliding to the south along the inferred bedrock surface. In Borehole BH09-7, the borehole was terminated on auger refusal within the rock fill or on the inferred bedrock surface as difficult drilling was noted throughout the rock fill layer. In Borehole BH09-8, the surface of the bedrock (metagabbro) was confirmed by coring. The depth to refusal below ground surface and the elevation at which refusal/bedrock or inferred bedrock was noted is presented below.



Borehole	Depth to Refusal/Bedrock Surface (m)	Refusal/Bedrock Surface Elevation (m)	Comments
BH09-5	1.2	258.1	Auger refusal on inferred bedrock surface
BH09-6	1.2	258.1	Auger refusal on inferred bedrock surface; augers sliding south
BH09-7	1.9	257.3	Auger refusal within rock fill or on inferred bedrock surface
BH09-8	1.6	257.6	Bedrock confirmed by coring

Exposed bedrock downstream of the existing bridge appears to be metavolcanic which is consistent with the geology of the area. The river in this area appears to be flowing in a blasted bedrock channel as evidenced by the rock cuts surrounding the bridge. Bedrock is visibly sloping downwards towards the river to the north beyond the blasted channel.

4.2.5 Groundwater Conditions

In general, the samples taken in the boreholes were moist to wet with free water noted in some samples. All boreholes were dry upon the completion of drilling. In Boreholes BH09-5 and BH09-6, caving of the borehole walls was observed at a depth of 0.6 m (Elevation 258.7 m) and 0.9 m (Elevation 258.4 m) below ground surface, respectively, which is indicative of the groundwater level. A piezometer was installed in Borehole BH09-8 and the piezometer was dry on November 26, 2009 to a depth of 2.4 m below ground surface (Elevation 256.8 m).

The water level in the reservoir upstream of the existing structure was measured at Elevation 257.9 m on October 6, 2009, at the time of the subsurface exploration program. The water level in the river, measured by others in September 2009, is Elevation 256.04 m at the location of the existing bridge.

5.0 CLOSURE

The field drilling program was supervised by Mr. Ed Savard. This report was prepared by Mr. Evan Childerhose, E.I.T., and the technical aspects were reviewed by Ms. Sarah E.M. Coyne, P.Eng., Associate. An independent quality control review of the report was provided by Mr. Jorge M.A. Costa, P.Eng., Principal and Golder's Designated MTO Contact for this project.



Report Signature Page

GOLDER ASSOCIATES LTD.



Sarah E.M. Coyne, P.Eng.
Senior Geotechnical Engineer, Associate



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

EC/SEMC/JMAC/lb/lb

n:\active\2009\1190 sudbury\1191\09-1191-0027 urs blanche 3 bridges\7000 reporting\final report\site 47-029\09-1191-0027 final rpt 10may10 site 47-029.docx



PART B

FOUNDATION DESIGN REPORT
ENGLEHART (BLANCHE) RIVER BRIDGE REPLACEMENT
HIGHWAY 573, SITE NO. 47-029
MUNICIPALITY OF CHARLTON AND DACK, ONTARIO
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5109-05-00



6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides foundation design recommendations for the preliminary design of the proposed new Highway 573 replacement structure crossing the Englehart (Blanche) River (Site No. 47-029), which is the middle bridge in a series of three bridges west of Highway 560. The preliminary recommendations are based on interpretation of the factual data obtained from the boreholes advanced during this preliminary subsurface investigation. The discussion and preliminary recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the preliminary design of the structure foundations and approach embankments. Where comments are made on construction, they are provided in order to highlight those aspects that could affect the preliminary design of the project, and for which special provisions are expected to be required as the project proceeds through detail design and into contract preparation. Those requiring information on the aspects of construction should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

Further borehole investigation and analysis will be required during the detail design phase of the project, once the configuration of the proposed structure is finalized, to confirm and expand on the preliminary foundation recommendations provided in this report.

6.1 General

We understand that the existing bridge carrying Highway 573 over the Englehart (Blanche) River is a single-span concrete girder bridge with a width of 6.4 m and a length of 8.7 m and was constructed in 1927. A dam and footbridge are located to the south (upstream) of the existing bridge.

We understand that it is proposed to replace the existing structure with a 12 m long single span bridge, approximately 8 m wide to be located on approximately the existing bridge alignment. The proposed grade is Elevation 259.3 m which is about the same as the existing highway grade.

The subsurface conditions in the immediate vicinity of the existing structure generally consist of asphalt underlain by granular fill and/or rock fill. A thin deposit of native soil/organics was encountered below the fill in two boreholes. At the investigated locations for the proposed abutments, the total overburden thickness ranges between 1.2 m and 1.9 m. The inferred bedrock surface varied between Elevation 257.3 m and 258.1 m, generally sloping downwards from west to east.

Due to the shallow depth to bedrock and the poor quality of the overburden soils, we recommend founding the new bridge abutments on shallow spread footings placed directly on the prepared bedrock surface. Deep foundations are not considered practical at this site due to the shallow thickness of overburden and may also be problematic due to the presence of the fill material comprising the immediate approach embankments adjacent to the existing abutments.



6.2 Shallow Foundations

We recommend supporting the bridge abutments on spread footings placed directly on the properly prepared bedrock surface. Based on the results of the borehole investigation, the variability in the bedrock surface across the foundation elements is less than about 0.7 m. The inferred bedrock surface elevations at the borehole locations as well as the recommended founding elevation are presented below. The bedrock elevations will vary between and beyond the borehole locations at each site.

Foundation Element	Borehole Numbers	Depth to Bedrock	Inferred Bedrock Surface Elevation	Recommended Foundation Elevation
West Abutment	BH09-05/BH09-06	1.2 m	258.1 m	258.1 m
East Abutment	BH09-07/BH09-08	1.9 m/1.6 m	257.3 m*/257.6 m	257.6 m

* Refusal likely within the rock fill.

In order to provide a level surface for the footings, the bedrock should be exposed, loosened material removed and the surface cleaned. The foundation area should be raised using mass concrete to the founding elevation indicated above, specifically to the highest elevation of the bedrock encountered. If a lower founding elevation is desired, or if more variability of the bedrock surface is encountered during the detailed foundation investigation, then bedrock excavation may be required. A higher foundation elevation would require additional mass concrete. The footings should be constructed "in-the-dry". Details of mass concrete placement and bedrock preparation are given in Section 6.5.5.

6.2.1 Geotechnical Axial Resistance

For spread footings placed on the properly prepared bedrock surface, which is assumed to be strong metagabbro bedrock, or on mass concrete of the same compressive strength as the footings, which is assumed to be 25 MPa or greater, a factored geotechnical axial resistance at Ultimate Limit States (ULS) of 10 MPa may be used for design. The geotechnical axial resistance at Serviceability Limit States (SLS) for 25 mm of settlement will be greater than the factored geotechnical axial resistance at ULS since the bedrock is considered to be an unyielding material and therefore ULS conditions will govern for this foundation type.

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.

6.2.2 Resistance to Lateral Loads

Resistance to lateral forces/sliding resistance between the base of the mass concrete and the bedrock subgrade should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction, $\tan \delta$, may be taken as 0.70 between the concrete footing/mass concrete and the properly prepared bedrock surface for construction "in-the-dry". This value represents an unfactored value; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating the horizontal resistance. The mass concrete could also be dowelled into the bedrock surface to provide for additional sliding resistance on the sloping bedrock surface. Consideration should also be given to the proximity of the existing dam to the new bridge footings and any potential lateral loading on these footings that might develop due to the retained water column.



6.2.3 Frost Protection

For spread footings founded directly on the bedrock at this site, frost susceptibility is not an issue.

6.3 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. Seismic (earthquake) loading must also be taken into account in the design.

6.3.1 Static

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 Special Provision (SP) 110S13 (Material Specification for Aggregates) Granular 'A' or Granular 'B' Type II but containing less than 5 percent passing the No. 200 sieve size should be used as backfill behind the walls. This fill should be compacted in loose lifts not greater than 200 mm thick to 95 percent of the material's Standard Proctor maximum dry density in accordance with SP 105S10 (Compaction). 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 Ontario Provincial Standard Drawings (OPSD) 3101.150 (Walls Abutment, Backfill) and 3121.150 (Walls Retaining, Backfill).
- For structures that are not comprised of integral or semi-integral abutments, rock fill may be used as backfill behind the walls and the material should meet the specifications as outlined in the Northern Region Directive for backfill to structures adjacent to rock fill embankments, dated November 2002. Other aspects of rock backfill requirements should be in accordance with OPSD 3101.200 (Walls Abutment, Backfill Rock).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with CHBDC Section 6.9.3 and Figure 6.6. Compaction equipment should be used in accordance SP 105S10 (Compaction). 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 2.4 m behind the back of the wall stem (as outlined on Figure C6.20(a), Case 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 base of the footing/pile cap (as outlined in Figure C6.20(b), Case II, of the Commentary to the CHBDC).



- For Case I, the pressures are based on the proposed embankment fill materials and the existing overburden soils and the following parameters (unfactored) may be used assuming the use of earth fill or rock fill:

	Earth Fill	Rock Fill
Soil unit weight:	21 kN/m ³	19 kN/m ³
Coefficients of static lateral earth pressure:		
Active, K_a	0.31	0.22
At rest, K_o	0.47	0.36

- For Case II, the pressures are based on the rock fill as above or 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

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 should be calculated in accordance with Section C6.9.1 and Table C6.6 of the Commentary to the *CHBDC*.

A restrained structure is typically a rigid frame bridge where the rotational and/or horizontal movement is not sufficient to mobilize the active earth pressure condition. For this condition, an at-rest pressure plus any compaction surcharge should be included in the design of the structure.

6.3.2 Dynamic

The potential for seismic (earthquake) loading must also be considered for the design of abutment stems/retaining walls in accordance with Section 4.6 of the *CHBDC* (if applicable). In this regard, the following should be taken into account in the lateral earth pressures.

- Seismic loading may 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. The site-specific zonal acceleration ratio for the Englehart area is 0.05 (Table A3.1.1, *CHBDC*). According to Table C4.2 of the *CHBDC*, this site is located in Seismic Zone 1. Based on experience, for the subsurface conditions at this site, there will be no amplification of the ground motion (i.e. Site Coefficient, $S=1.0$), resulting in a peak horizontal acceleration (PHA) of 0.05 g.



Since this highway route/bridge is not designated as a lifeline bridge, based on Section 4.4.4 of the *CHBDC*, this bridge structure is assigned to Seismic Performance Zone 1. Given this, and in accordance with Section 4.4.5.1 of the *CHBDC*, structures located in Seismic Performance Zone 1 need not be analysed for seismic loads.

6.4 Approach Embankment Design

The existing bridge will be replaced by a new bridge of about the same width as the existing structure and shifted about 3 m northwards (centreline to centreline) of the existing bridge and will have a final grade at about the same Elevation as the existing highway. The existing approach embankment side slopes are steeper than 1.5 horizontal to 1 vertical.

6.4.1 Subgrade Preparation and Embankment Construction

Observations of patching at the existing east abutment as well as the thickness of asphalt measured in the boreholes (up to 210 mm and 240 mm at the existing abutments), indicate that asphalt padding has taken place in the past. This is consistent with the soil conditions encountered in the boreholes and our site observations. We therefore recommend that all existing fill and native soil be removed and replaced with granular fill or rock fill in accordance with SP 206S03 (Earth Excavation, Grading; Rock Embankment).

Embankments could be constructed of granular fill with side slopes 2H:1V or of rock fill constructed with side slopes 1.25H:1V. If granular fill is used, it should be free-draining (i.e. SP 110S13 (Aggregates) Granular 'B' Type II, Granular 'A' or Select Subgrade Material) to ensure no build-up of excess pore pressure within the fill given the presence of the adjacent reservoir and the groundwater level which may be close to the bedrock surface. Granular 'B' Type I is not recommended due to the potential for a high variability in gradation or potential for supply of uniform (poorly) graded material which could result in potential post-construction settlement.

Embankment fill should be placed and compacted in accordance with SP 206S03 (Grading) and SP 105S10 (Compaction). To reduce erosion of the embankment side slopes (if constructed with granular material) due to surface water runoff, placement of topsoil and seeding or pegged sod is recommended as soon as practicable after construction of the embankments. The erosion protection should be in accordance with OPSS 572 (Seed and Cover).

6.4.2 Approach Embankment Stability

For embankments constructed directly on the bedrock surface and comprised of granular fill or rock fill for the anticipated thicknesses of 1.2 m and 2.0 m at the west and east abutments, respectively, there are no stability issues at this site.



6.4.3 Approach Embankment Settlement

Given our visual observations of the condition of the existing pavement and as confirmed by the results of the borehole investigation, the asphalt padding carried out at the existing abutments is an indicator of past/ongoing settlement of the existing fill and underlying native materials behind the abutments. The borehole investigation for this bridge encountered between 210 mm and 240 mm of asphalt and granular layers, which is considered a substantial thickness of pavement structure. Provided that all existing fill and native materials are removed and replaced with properly placed/compacted granular fill or rock fill, settlement of the approach embankments at this site should be less than 25 mm and should occur during and shortly after construction.

6.5 Detail Design and Construction Considerations

6.5.1 Additional Investigation Requirements

An additional borehole investigation and laboratory testing program and analysis of these supplemental data will be required during the detail design phase, once the proposed location of the foundation elements is finalized, to confirm the preliminary foundation recommendations presented herein, including founding elevations and sub-excavation requirements, geotechnical resistances, settlement, unwatering and temporary shoring requirements, etc.

In particular, a sufficient number of boreholes should be drilled at the proposed foundation elements to confirm the bedrock surface elevation and bedrock quality using coring techniques. Due to the difficulties experienced during the preliminary investigation with borehole drilling through the existing rock fill materials, specialized drilling equipment such as tri-cone and coring methods may be required to advance the boreholes. Consideration will also have to be given to proper backfilling of the boreholes as voids may be present within the rock fill. Portable rock coring equipment may be required to advance boreholes north of the existing highway at the north limit of the proposed foundation elements since the ground surface (generally a blasted bedrock channel and/or exposed bedrock) slopes/steps steeply downwards away from the existing bridge.

6.5.2 Excavations

Excavations for shallow foundations (footings) to depths of up to 1.9 m below existing ground surface will be made through rock fill (containing voids), granular fill and native soils containing cobbles and boulders and should be in accordance with OPSS902 (Earth Excavation, Structures). The overburden soils are considered Type 4 soil according to the Occupational Health and Safety Act and Regulation for Construction Projects (OHSA). The excavation work should be carried out in accordance with the requirements of the OHSA, with side slopes no steeper than 1H:1V and good construction practice.

6.5.3 Groundwater and Surface Water Control for Foundation Excavation

It is likely that groundwater will be flowing along the downward sloping surface of the bedrock from south (i.e. from the reservoir located upstream of the bridge) to north. Fluctuating water levels and flowing groundwater conditions due to the proximity of the bridge site to the local water retention dam should be taken into consideration. As such, a suitable unwatering scheme in conjunction with temporary shoring may be required to maintain a dry and stable excavation during construction, including for the placement of mass concrete "in-the-dry".



6.5.4 Temporary Shoring

It is likely that the existing dam, of considerable age, will not be suitable to act as stand-alone temporary shoring system during the construction of the new bridge footings and sub-excavation for embankment construction. The designers should ensure that there is sufficient room to construct temporary shoring between the proposed foundations and the existing dam. Given the presence of rock fill and cobbles and boulders, installation of steel sheet-piling will likely not be possible. Other shoring methods such as a soldier pile and lagging system with the piles drilled and socketted into the bedrock may be required. Given that the ground surface slopes downward towards the north, rakers may need to be used to support the shoring wall. Tie-backs/anchors will likely not be possible due to the proximity of the dam.

6.5.5 Footing Subgrade Preparation

All loose, shattered and/or fractured rock within the footprint of the footings at the founding level should be removed and replaced with mass concrete in accordance with OPSS 902 (Excavation and Backfilling for Structures). Where mass concrete is used to level the founding area, it should be of the same compressive strength as will be used for the actual footing. If bedrock excavation is required to level the founding area, it should be carried out using controlled blasting techniques (i.e. line drilling, pre-shearing or cushion blasting) in order to minimize shattering and over break resulting from blast damage to the rock mass.

6.5.6 Obstructions

As noted above, the existing embankments are comprised of rock fill in a gravelly sand matrix where large fragments of rock fill could be present. Further, cobbles and boulders were noted within the native soils.

6.5.7 Removal of Existing Bridge

We understand that the existing bridge structure will be removed in a staged manner as the new bridge is constructed. Since it is not known whether the existing bridge/bridge footings are intimately connected to the dam or if the dam is being directly supported by the existing footings, we recommend that the interaction between the existing bridge foundations and the dam be clearly defined and/or the existing footings be left in place if such a condition cannot be clearly determined.


7.0 CLOSURE

This report was prepared by Ms. Sarah Coyne, P.Eng., Associate. Mr. Jorge Costa, P.Eng., Principal and Golder's Designated MTO Contact for this project, conducted an independent quality control review of the report.




Report Signature Page

GOLDER ASSOCIATES LTD.



Sarah E.M. Coyne, P.Eng.
Senior Geotechnical Engineer, Associate



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

SEMC/JMAC/lb/lb

n:\active\2009\1190 sudbury\1191\09-1191-0027 urs blanche 3 bridges\7000 reporting\final reports\site 47-029\09-1191-0027 final rpt 10may10 site 47-029.docx



REFERENCES

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

Occupational Health and Safety Act and Regulation for Construction Projects, January 2006.

Ontario Provincial Standard Specifications

OPSS 572 Construction Specification for Seed and Cover

OPSS 902 Construction Specification for Excavating and Backfilling - Structures

Ontario Provincial Standard Drawings

OPSD 3101.150 Walls Abutment, Backfill Minimum Granular Requirement

OPSD 3101.200 Walls Abutment, Backfill Rock

OPSD 3121.150 Walls Retaining, Backfill Minimum Granular Requirement

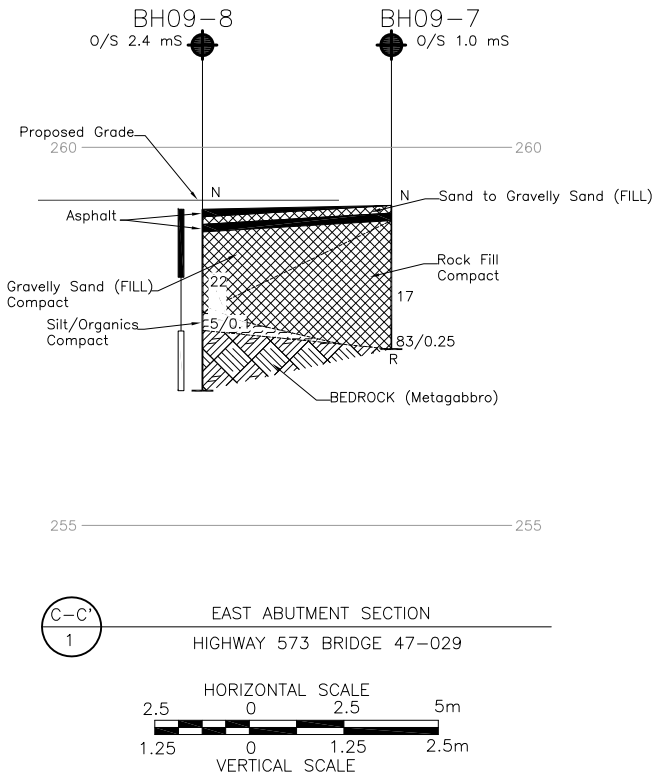
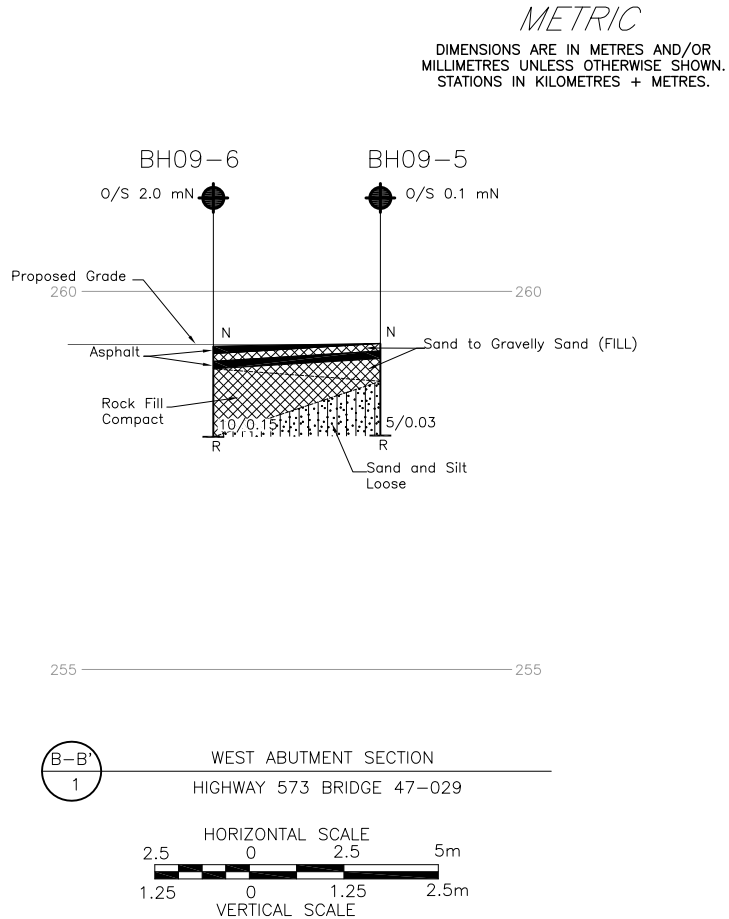
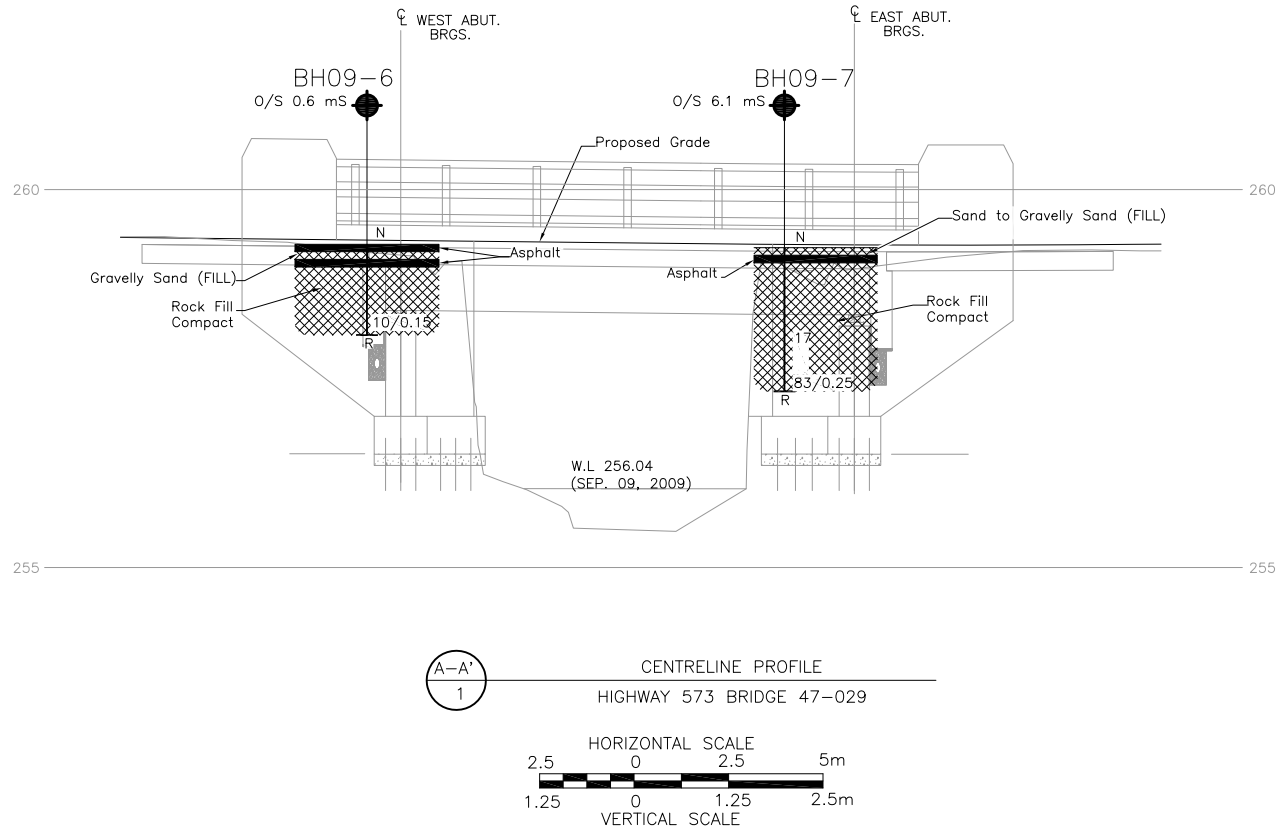
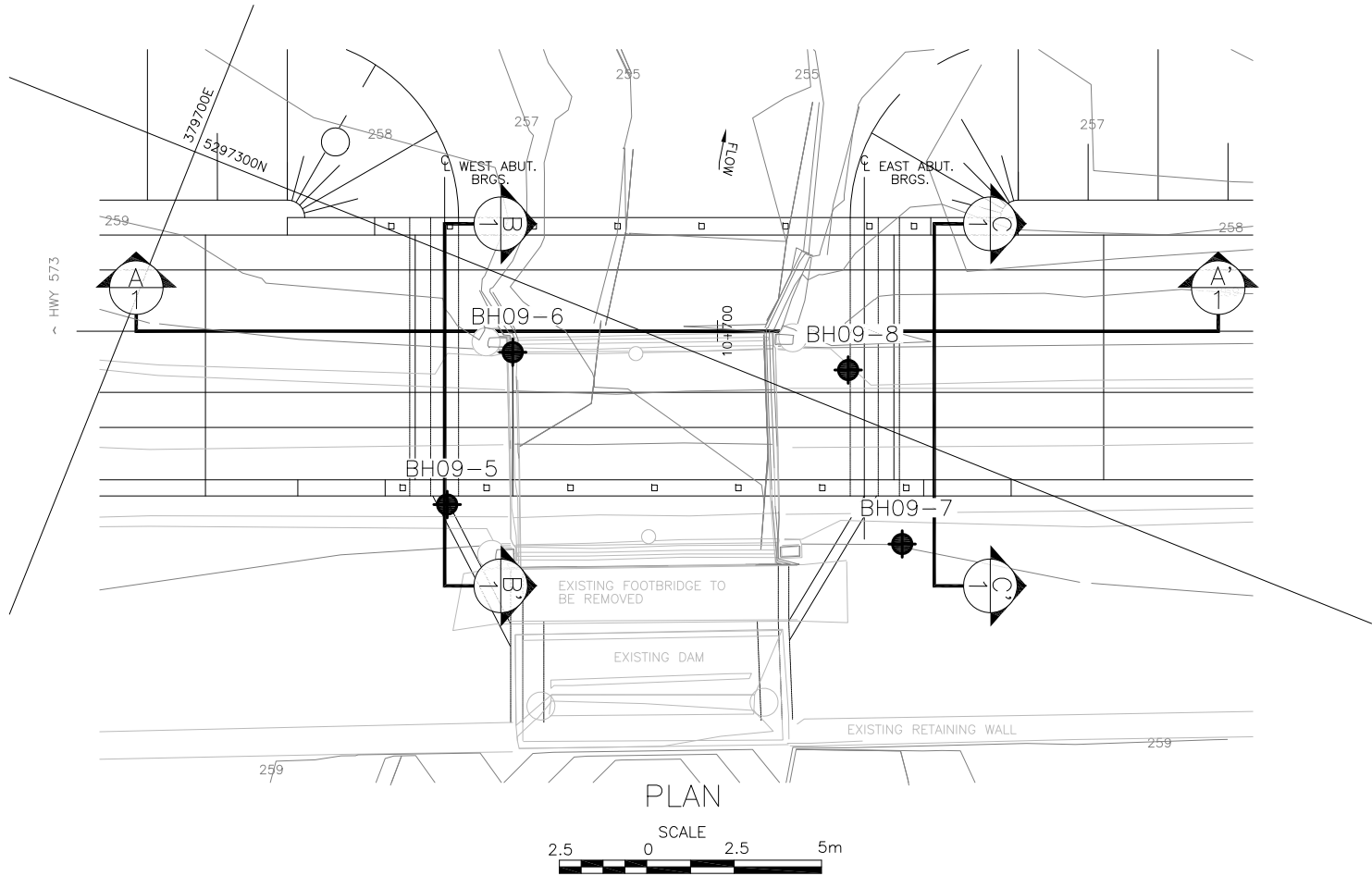
Ministry of Transportation Ontario Special Provisions

SP 105S10 Amendment to OPSS 501, February 1996

SP 110S13 Material Specification for Aggregates – Base, Subbase, Select Subgrade, and Backfill

SP 206S03 Earth Excavation, Grading; Rock Embankment

Northern Region Directive; Backfill to Structures adjacent to Rock Embankment Approaches, November 2002



CONT No.
GWP No. 5109-05-00

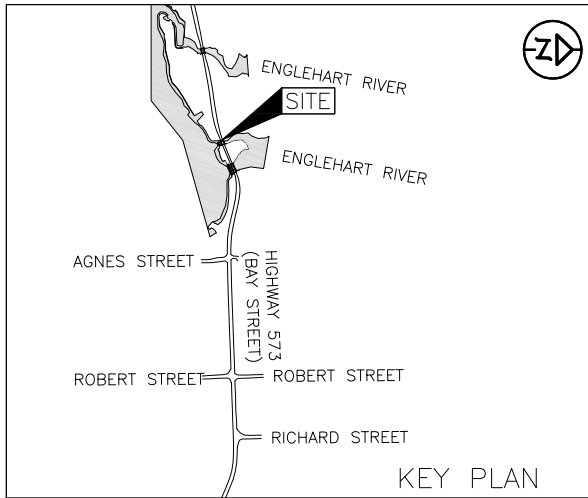


ENGLEHART (BLANCHE) RIVER
HIGHWAY 573 BRIDGE 47-029
BOREHOLE LOCATION
AND SOIL STRATA

SHEET



Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA



LEGEND

- Proposed Borehole - Current Investigation
- N Standard Penetration Test Value
- 4 Blows/0.3 m unless otherwise stated (Std. Pen. Test, 475j/blow)
- WL upon completion of drilling
- WL in piezometer, measured on November 26, 2009
- Seal
- Piezometer
- Refusal

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
BH09-05	259.3	5297293.3	379710.5
BH09-06	259.3	5297298.1	379710.6
BH09-07	259.2	5297297.1	379722.9
BH09-08	259.2	5297301.2	379719.7

REFERENCE

General Arrangement Drawing provided in digital format by URS, drawing file nos B#_47-029.DWG, received Feb, 2010.

NOTES

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

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

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

NO.	DATE	BY	REVISION
Geocres No. 31M-85			
HWY. 573		PROJECT NO. 09-1191-0027	DIST.
SUBM'D. EC	CHKD.	DATE: MAY 2010	SITE: 47-029
DRAWN: JJL	CHKD. SEMC	APPD. JMAC	DWG. 1



APPENDIX A

Record of Boreholes

LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_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 $= \tan \delta$
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 + \sigma_3)/2$ or $(\sigma'_1 + \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 + \sigma_3)$
S_t	sensitivity

Notes: 1 $\tau = c' + \sigma' \tan \phi'$
2 Shear strength = (Compressive strength)/2

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
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

Piezcone Penetration Test (CPT)

An electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Density Index (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.

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

<u>Description</u>	<u>Bedding Plane Spacing</u>
Very thickly bedded	> 2 m
Thickly bedded	0.6 m to 2 m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	< 6 mm

JOINT OR FOLIATION SPACING

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

GRAIN SIZE

<u>Terms</u>	<u>Size*</u>
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 (TCR)

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

Solid Core Recovery (SCR)

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

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD 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 separation) 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 separation such as fractures, bedding planes and foliation planes or mechanically induced fractures 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	⊥ - Perpendicular To
FO - Foliation / Schistosity	- Parallel To
CL - Cleavage	P - Polished
SH - Shear Plane / Zone	K - Slickensided
VN - Vein	SM - Smooth
F - Fault	R - Rough
CO - Contact	ST - Stepped
J - Joint	PL - Planar
FR - Fracture	U - Undulating
MF - Mechanical Fracture	C - Curved

PROJECT <u>09-1191-0027</u>		RECORD OF BOREHOLE No BH09- 5				1 OF 1 METRIC								
W.P. <u>5109-05-00</u>		LOCATION <u>N 5297293.3 ;E 379710.5</u>				ORIGINATED BY <u>EHS</u>								
DIST <u> </u> HWY <u>573</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers</u>				COMPILED BY <u>AMW</u>								
DATUM <u>Geodetic</u>		DATE <u>October 6, 2009</u>				CHECKED BY <u>EC</u>								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
259.3	GROUND SURFACE													
0.0	Sand, trace to some gravel, trace silt (FILL)		1	AS	-									
258.8	Brown Moist ASPHALT													
0.5	Gravelly sand, trace silt (FILL)		2	SS	5/0.03									5 49 40 6
258.1	Brown Moist SAND and SILT, trace clay, trace gravel, oxidized containing cobbles and boulders													
1.2	Loose Brown Wet													
	Difficult drilling below 0.6 m													
	End of Borehole													
	Auger Refusal; auger sliding north													
	Notes:													
	1. Borehole dry upon completion of drilling; cave at 0.6 m depth.													
	2. Sample 2: Attempted spoon first then recovered auger sample. Spoon bouncing at 0.95 m depth.													

PROJECT <u>09-1191-0027</u>		RECORD OF BOREHOLE No BH09- 6				1 OF 1 METRIC										
W.P. <u>5109-05-00</u>		LOCATION <u>N 5297298.1 ;E 379710.6</u>				ORIGINATED BY <u>EHS</u>										
DIST <u> </u> HWY <u>573</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers</u>				COMPILED BY <u>AMW</u>										
DATUM <u>Geodetic</u>		DATE <u>October 6, 2009</u>				CHECKED BY <u>EC</u>										
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					WATER CONTENT (%)			
259.3	GROUND SURFACE						20 40 60 80 100					W _p W W _L				
0.0	ASPHALT						○ UNCONFINED + FIELD VANE					○ QUICK TRIAXIAL × REMOULDED				
0.3	Gravelly sand, trace silt (FILL) Brown Moist						20 40 60 80 100					10 20 30				
258.1	ASPHALT		1	SS	10/0.15		20 40 60 80 100					10 20 30				
1.2	Rock fill in a gravelly sand, trace silt matrix (FILL) Compact Brown Moist						20 40 60 80 100					10 20 30				
End of Borehole Auger Refusal; augers sliding south Notes: 1. Borehole dry upon completion of drilling; caved at 0.9 m depth. 2. Sample1: Spoon sliding south.							20 40 60 80 100					10 20 30				

PROJECT <u>09-1191-0027</u>		RECORD OF BOREHOLE No BH09- 7				1 OF 1 METRIC											
W.P. <u>5109-05-00</u>		LOCATION <u>N 5297297.1 ;E 379722.9</u>				ORIGINATED BY <u>EHS</u>											
DIST <u> </u> HWY <u>573</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers</u>				COMPILED BY <u>AMW</u>											
DATUM <u>Geodetic</u>		DATE <u>October 6, 2009</u>				CHECKED BY <u>EC</u>											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
259.2	GROUND SURFACE							20	40	60	80	100					
0.0	Sand, some gravel, trace silt (FILL) Brown Moist						259										
0.3	ASPHALT Gravelly sand, trace silt (FILL) Brown Moist		1	SS	17		258										
257.3	Rock fill in a gravelly sand, trace silt matrix (FILL) Compact Brown Moist		2	SS	83/0.25												
1.9	Difficult drilling below 0.3 m depth End of Borehole Auger Refusal																
Note: 1. Borehole dry upon completion of drilling.																	

PROJECT <u>09-1191-0027</u>		RECORD OF BOREHOLE No BH09- 8				1 OF 1 METRIC									
W.P. <u>5109-05-00</u>		LOCATION <u>N 5297301.2 ; E 379719.7</u>				ORIGINATED BY <u>EHS</u>									
DIST <u> </u> HWY <u>573</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers; NW Casing, Wash Boring</u>				COMPILED BY <u>AMW</u>									
DATUM <u>Geodetic</u>		DATE <u>October 6, 2009</u>				CHECKED BY <u>EC</u>									
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa				WATER CONTENT (%)			
259.2	GROUND SURFACE						20 40 60 80 100				20 40 60 80 100				
0.0	ASPHALT						20 40 60 80 100				10 20 30				
0.3	Gravelly sand, trace silt (FILL)						20 40 60 80 100				10 20 30				
0.3	ASPHALT						20 40 60 80 100				10 20 30				
0.3	Gravelly sand with silt, trace clay (FILL)						20 40 60 80 100				10 20 30				
0.3	Compact Brown Moist		1	SS	22		20 40 60 80 100				10 20 30				28 43 21 8
257.7	SILT, trace sand Brown to black Compact Wet		2	SS	5/0.1		20 40 60 80 100				10 20 30				
1.6	ORGANICS		1	RC	REC 100%		20 40 60 80 100				10 20 30				RQD = 0%
256.8	METAGABBRO (BEDROCK)						20 40 60 80 100				10 20 30				
2.4	Bedrock cored from 1.6 m depth to 2.4 m depth.						20 40 60 80 100				10 20 30				
<p>For coring details refer to Record of Drillhole BH09-8.</p> <p>End of Borehole</p> <p>Notes:</p> <p>1. First borehole location: Augers slid off boulder at 0.85 m depth; moved borehole 0.5 m west.</p> <p>2. Second borehole location: Augers slid off boulder at 1.07 m depth; sliding south. Sample 1 retrieved. Borehole moved 1.0 m west and resumed sampling at 1.5 m depth. Casing used from ground surface in second borehole location.</p> <p>3. Spoon refusal at 1.7 m depth (Elev. 257.5 m).</p> <p>4. Borehole dry upon completion of drilling.</p> <p>5. Piezometer dry on November 26, 2009.</p>							20 40 60 80 100				10 20 30				

SHEET 1 OF 1

DATUM: Geodetic

DRILLING CONTRACTOR: Downing

CHECKED: EC

MIS-RCK 004 09-1191-0027 URS BLACHE 3 BRIDGES.GPJ GAL-MISS.GDT 4/5/10



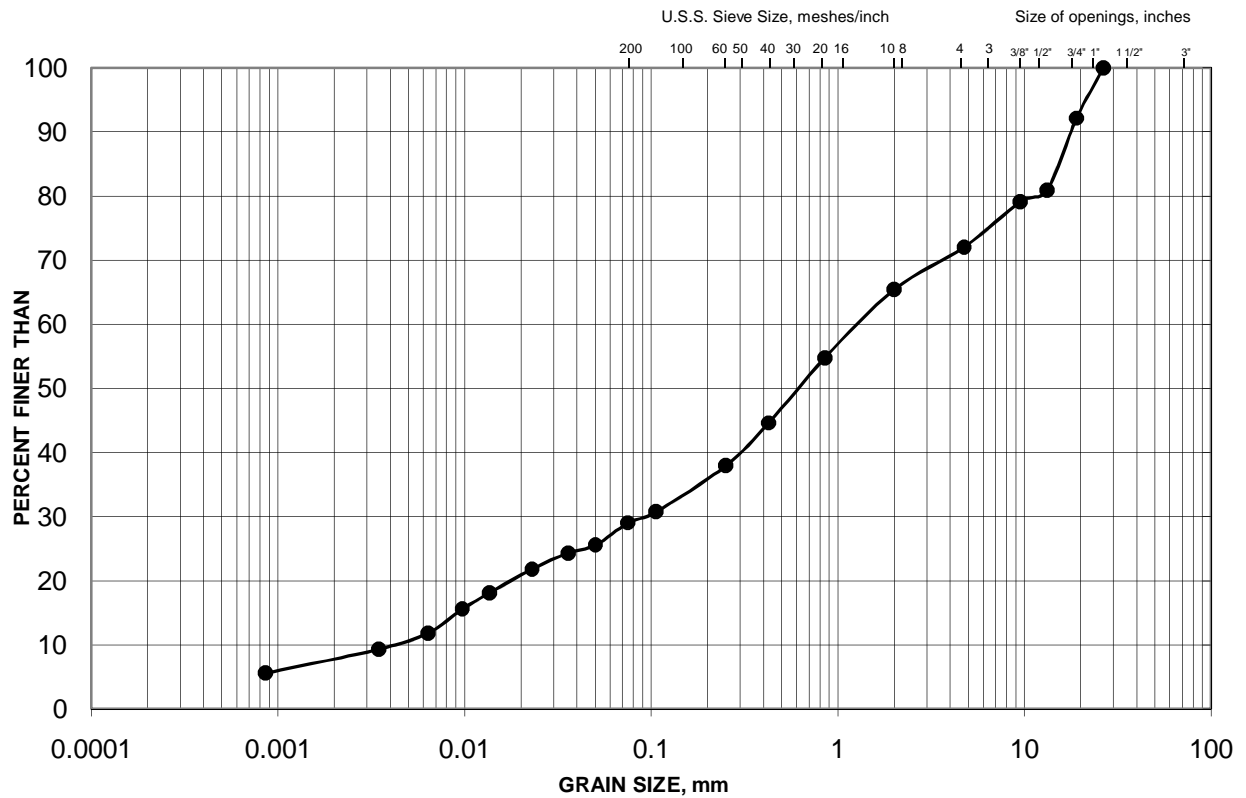
APPENDIX B

Laboratory Test Results

GRAIN SIZE DISTRIBUTION

Gravelly Sand (FILL)

FIGURE
B-1



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
—●—	BH09-8	1	258.1

Project Number: 09-1191-0027

Checked By: SEMC

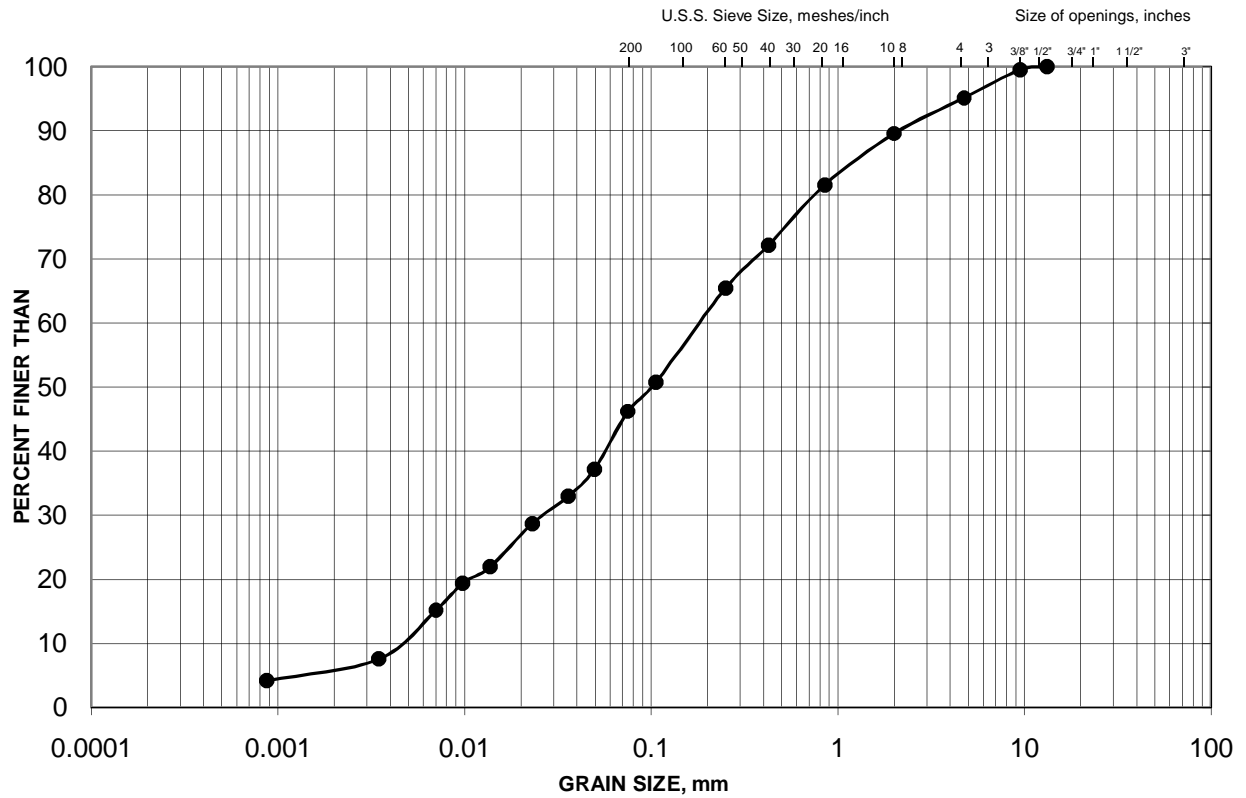
Golder Associates

Date: May 2010

GRAIN SIZE DISTRIBUTION

Sand and Silt

FIGURE
B-2



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
—●—	BH09-5	2	258.4

Project Number: 09-1191-0027

Checked By: SEMC

Golder Associates

Date: May 2010

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

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

solutions@golder.com
www.golder.com

Golder Associates Ltd.
1010 Lorne Street
Sudbury, Ontario, P3C 4R9
Canada
T: +1 (705) 524 6861

