



May 10, 2010

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## PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

ENGLEHART (BLANCHE) RIVER BRIDGE REPLACEMENT  
HIGHWAY 573, SITE NO. 47-030  
MUNICIPALITY OF CHARLTON AND DACK, ONTARIO  
MINISTRY OF TRANSPORTATION, ONTARIO  
GWP 5109-05-00

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REPORT



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**FOUNDATION REPORT - ENGLEHART (BLANCHE) RIVER  
BRIDGE REPLACEMENT - SITE 47-030, HIGHWAY 573, GWP 5109-05-00**

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# PART A

FOUNDATION INVESTIGATION REPORT  
ENGLEHART (BLANCHE) RIVER BRIDGE REPLACEMENT  
HIGHWAY 573, SITE NO. 47-030  
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. 47-030, the most easterly 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. 47-030 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 in 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 0.3 km west of the junction with Highway 560. The existing road grade is about 1.3 m to 1.7 m above the adjacent reservoir water level sloping steeply downwards to the north. 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 landscaped grass and small shrubs and bedrock is exposed in several areas. 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 river flows from south to north.

The existing three-span concrete bridge was constructed in 1927 and has a width of 5.7 m and a length of 17.4 m. The existing highway grade is at between Elevation 259.2 m and 259.6 m and the water level in the reservoir immediately 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 256.92 m at the location of the existing bridge.

Based on on-site discussions with the local public works personnel, the east abutment experiences some settlement each year and asphalt patching is required/carried out to smooth out the transition between the abutment and the approach embankment. This was confirmed by our site observations of the roadway surface.



### 3.0 INVESTIGATION PROCEDURES

The fieldwork at the bridge site was carried out on October 5 and 6, 2009, at which time a total of four (4) boreholes (BH09-1 to BH09-4) 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 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. 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.

The boreholes were advanced to auger refusal at depths ranging from about 1.8 m to 2.8 m below the existing ground surface.

The groundwater conditions in the open boreholes were observed during the drilling operations and a piezometer was installed in Borehole BH09-1, 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, Atterberg limits 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 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		
BH09-1	5297319.6	379771.5	259.5	2.2
BH09-2	5297321.8	379770.8	259.6	1.8
BH09-3	5297309.2	379750.5	259.2	2.1
BH09-4	5297312.4	379749.1	259.2	2.8

## 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)<sup>1</sup>.

Terrain mapping by the Ontario Geological Survey<sup>2</sup> 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 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 and granular material comprised of sand to gravelly sand fill and/or rock fill. A thin deposit of native sandy silt to silt or sand and gravel containing organics was encountered below the granular fill in the boreholes advanced for the proposed east abutment.

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

<sup>1</sup> 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.

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



#### **4.2.1 Asphalt**

All of the boreholes were drilled through the pavement. Approximately 340 mm to 480 mm of asphalt was encountered in the boreholes. In Boreholes BH09-2, BH09-3 and BH09-4, a 100 mm to 180 mm thick interlayer of gravelly sand fill containing trace silt was encountered between asphalt layers. The ground surface at the boreholes ranges from Elevation 259.6 m to 259.2 m.

#### **4.2.2 Fill**

Fill was encountered underlying the asphalt in all boreholes. The top of the fill was encountered between Elevation 259.1 m and 258.8 m. On the east side of the river (BH09-1 and BH09-2), the fill material is up to 1.1 m thick and is comprised of grey to brown sand to sand and gravel and contains trace silt and cobbles and boulders. On the west side of the river (BH09-3 and BH09-4), the fill is up to 1.8 m thick and is comprised of gravelly sand, trace silt and/or rock fill in a gravelly sand to silty sand matrix. Difficult drilling was noted throughout the fill material at this site and observations of the augers sliding/dipping were noted in some boreholes. A void was encountered at a depth of 0.6 m in Borehole BH09-3 as indicated by the lack of cutting returns below a depth of 0.4 m. Also, the split-spoons did not recover any samples from within the rock fill in the boreholes on the west side of the river.

SPT 'N'-values measured within the fill range from 8 to 26 blows per 0.3 m of penetration where the sampler was able to penetrate the full sample length, indicating a loose to compact relative density. In some boreholes, the split-spoon sampler did not penetrate the full sample depth indicating the presence of cobbles and boulders and/or rock fill.

The natural water content measured on samples of the fill ranges between about 2 and 8 percent.

Grain size distributions for two samples of the sand and gravel fill are shown on Figure B-1 in Appendix B.

#### **4.2.3 Sandy Silt to Sand and Gravel**

A 0.3 m to 0.7 m thick deposit of moist to wet, brown to grey, sandy silt to silt containing trace clay and organics or sand and gravel containing organics was encountered beneath the fill in Boreholes BH09-1, BH09-2 and BH09-4. The top of this deposit was encountered between Elevation 258.1 m and 257.1 m.

One SPT 'N'-value measured within this deposit was 26 blows per 0.3 m of penetration where the sampler was able to penetrate the full sample length, indicating a compact relative density. Elsewhere within the native deposits, the 'N'-values were 30 for 0.08 m of penetration and 29 for 0.20 m of penetration suggesting the presence of cobbles and boulders and indicated loose to very dense relative density.

Grain size distributions for two samples of the native sand and gravel are shown on Figure B-2 in Appendix B.

Atterberg limits testing was carried out on one sample of the sandy silt to silt containing trace to some clay and yielded a liquid limit of 25 percent, a plastic limit of 20 percent and a plasticity index of 5 percent. The results of the Atterberg limits testing are shown on the plasticity chart on Figure B-3 in Appendix B and indicate that the material is classified as a silt of slight plasticity.

The natural water content measured on samples of this deposit ranges between about 12 percent and 29 percent.



#### 4.2.4 Refusal

All of the boreholes were terminated upon encountering auger refusal on either the inferred bedrock surface or within the rock fill. In Borehole BH09-2, located on the east side of the river, the augers were observed to be sliding/dipping to the north along the inferred bedrock surface. In Borehole BH09-3, the borehole was terminated when the drill rig was noted to be 'shaking violently', suggesting that refusal may have been reached within the rock fill. The inferred bedrock surface/refusal was encountered at the depths and elevations presented below.

Borehole	Depth to Refusal (m)	Refusal/Bedrock Surface Elevation (m)	Comments
BH09-1	2.2	257.3	Auger refusal on inferred bedrock surface
BH09-2	1.8	257.8	Auger refusal on inferred bedrock surface; augers sliding to the north
BH09-3	2.1	257.1	Auger refusal within rock fill
BH09-4	2.8	256.4	Auger refusal on inferred bedrock surface

Exposed bedrock downstream of the existing bridge, at approximately the existing bridge founding level, appears to be metavolcanic which is consistent with the geology of the area. Bedrock is visibly sloping downwards towards the river to the north through the river 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. Boreholes BH09-2 to BH09-4 were dry upon the completion of drilling. In Borehole BH09-3, the borehole walls caved at a depth of about 1.0 m below ground surface (Elevation 258.2 m) which could be indicative of the groundwater level. In Borehole BH09-1, the water level was encountered at a depth of 1.9 m below ground surface (Elevation 257.6 m) upon completion of drilling. A piezometer was installed in Borehole BH09-1 and the piezometer was dry to a depth of 2.2 m below ground surface (Elevation 257.3 m) on November 26, 2009.

The water level in the upstream reservoir of the dam was measured at Elevation 257.9 m in the first week of October 2009, at the time of the subsurface exploration program. The water level in the river, measured by others in September 2009, is Elevation 256.92 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

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# **PART B**

**FOUNDATION DESIGN REPORT  
ENGLEHART (BLANCHE) RIVER BRIDGE REPLACEMENT  
HIGHWAY 573, SITE NO. 47-030  
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. 47-030), which is the easternmost 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.

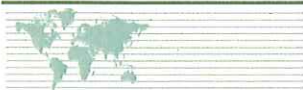
### 6.1 General

We understand that the existing bridge carrying Highway 573 over the Englehart (Blanche) River is a three-span concrete bridge with a width of 5.7 m and a length of 17.4 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 22 m long, 7 m wide single-span bridge, to be located essentially on approximately the existing alignment, with a centreline to centreline shift to the north by about 3 m. The proposed grade is Elevation 259.6 m and 259.9 m at the west and east abutments, respectively, which is about 0.5m higher than 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 containing trace to some clay and/or organics was encountered below the fill. At the investigated locations for the proposed abutments, the total overburden thickness ranges between about 1.8 m and 2.8 m. The inferred bedrock surface varies between Elevation 257.8 m and 256.4 m, generally sloping downwards from east to west.

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 nature of the rock 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
East Abutment	BH09-01/BH09-02	2.2 m/1.8 m	257.3 m/257.8 m	257.8 m
West Abutment	BH09-03/ BH09-04	2.1 m/2.8 m	257.1 m*/256.4 m	256.4 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 footing foundation area should be raised using mass concrete to the founding elevation indicated above, specifically to the highest elevation at 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.

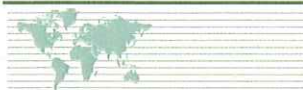
### 6.2.1 Geotechnical Axial Resistance

For spread footings placed on the properly prepared bedrock surface, which is assumed to be strong metavolcanic bedrock or on mass concrete of the same compressive strength at 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 with 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 <sup>3</sup>	19 kN/m <sup>3</sup>
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 <sup>3</sup>	21 kN/m <sup>3</sup>
Coefficients of static lateral earth pressure:		
Active, $K_a$	0.27	0.27
At rest, $K_o$	0.43	0.43

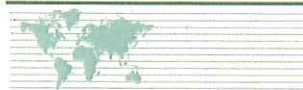
If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding, at rest earth pressures should be assumed for geotechnical design. The movement to allow 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 about 1 m wider and shifted about 3 m north (centreline to centreline) of the existing bridge and will have a final grade about 0.5 m higher than 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 abutments as well as the thickness of asphalt measured in the boreholes (up to 450 mm and 370 mm at the existing east and west abutments, respectively), indicate that asphalt padding has taken place in the past. This is consistent with the soil conditions encountered in the boreholes, specifically the traces of clay and/or organics noted within the native material at the east abutment and the rock fill containing voids at the west abutment. We therefore recommend that prior to the placement of any fill, all existing fill and native soil be removed and replaced with granular fill or rock fill in accordance with SP 206S03 (Earth, Grading; Rock Embankment).

Embankments could be constructed of granular fill with side slopes 2H:1V or of rock fill constructed with side slopes of 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 is 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 gap graded or 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 2.6 m and 3.2 m at the east and west 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 340 mm and 450 mm of asphalt and granular layers, which is considered a significant 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 were 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 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 2.8 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 steeply 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 OPSS902 (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, particularly on the west side, are comprised of rock fill in a gravelly sand matrix and containing voids. Large fragments of rock fill could be present within the fill. Further, cobbles and boulders were noted within the sand to gravelly sand fill and the native soils.

A PVC pipe was observed to outlet the slope on the northeast side of the proposed bridge. The designers should account for removal or relocation of this pipe as part of the overall design, as may be required.

#### 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 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.

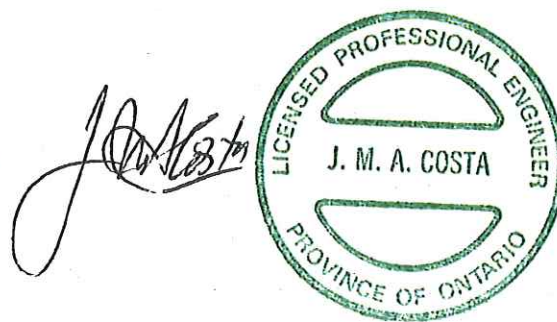


## Report Signature Page

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

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## 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

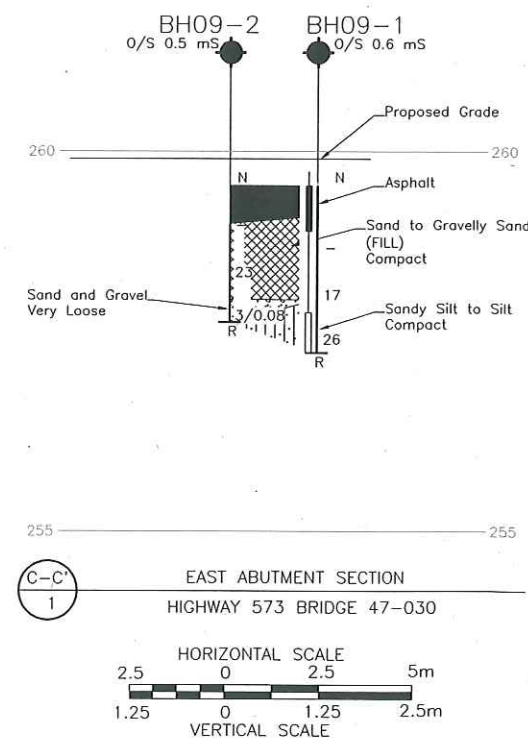
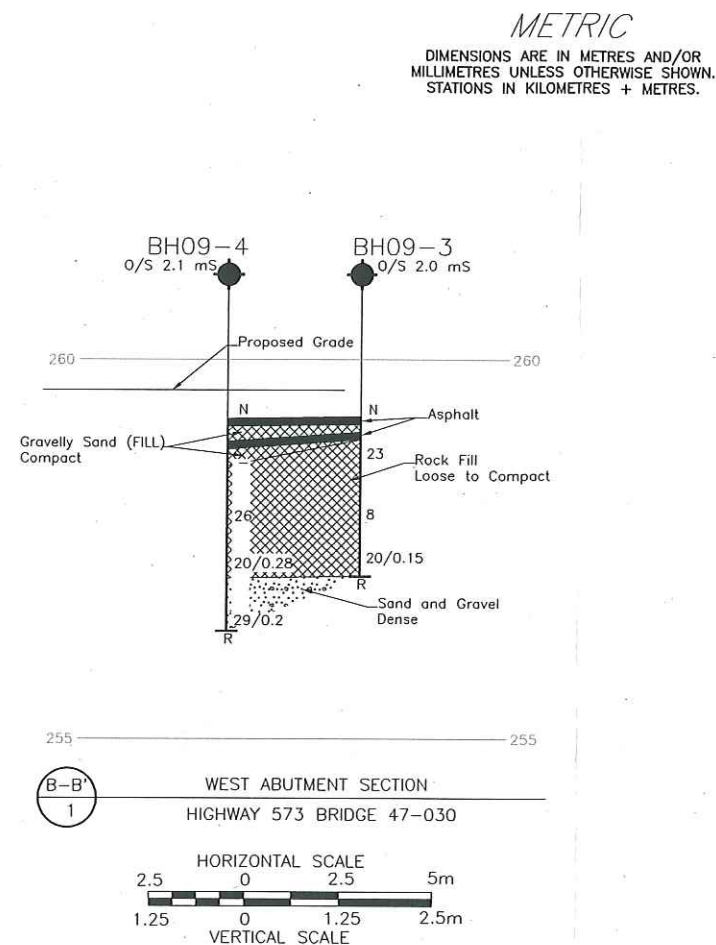
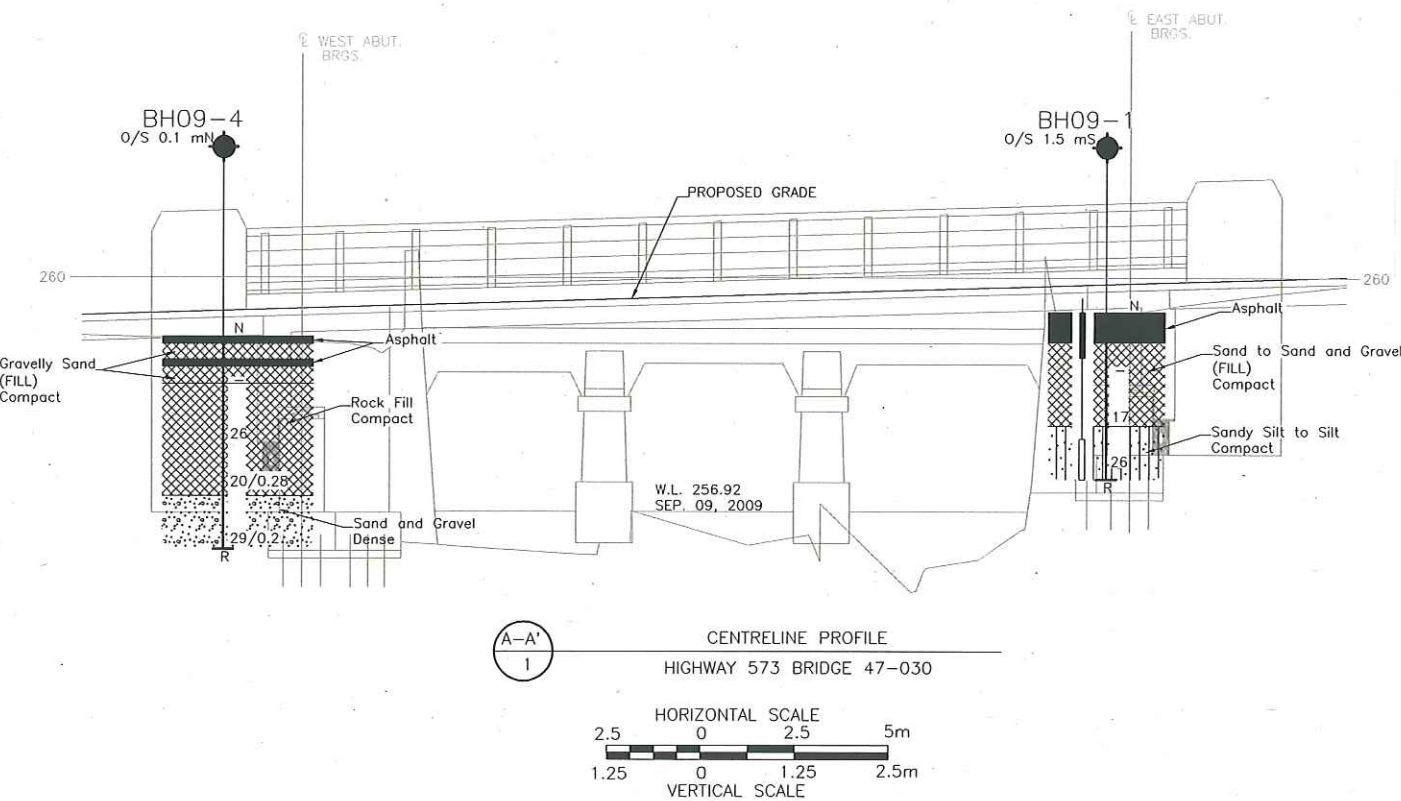
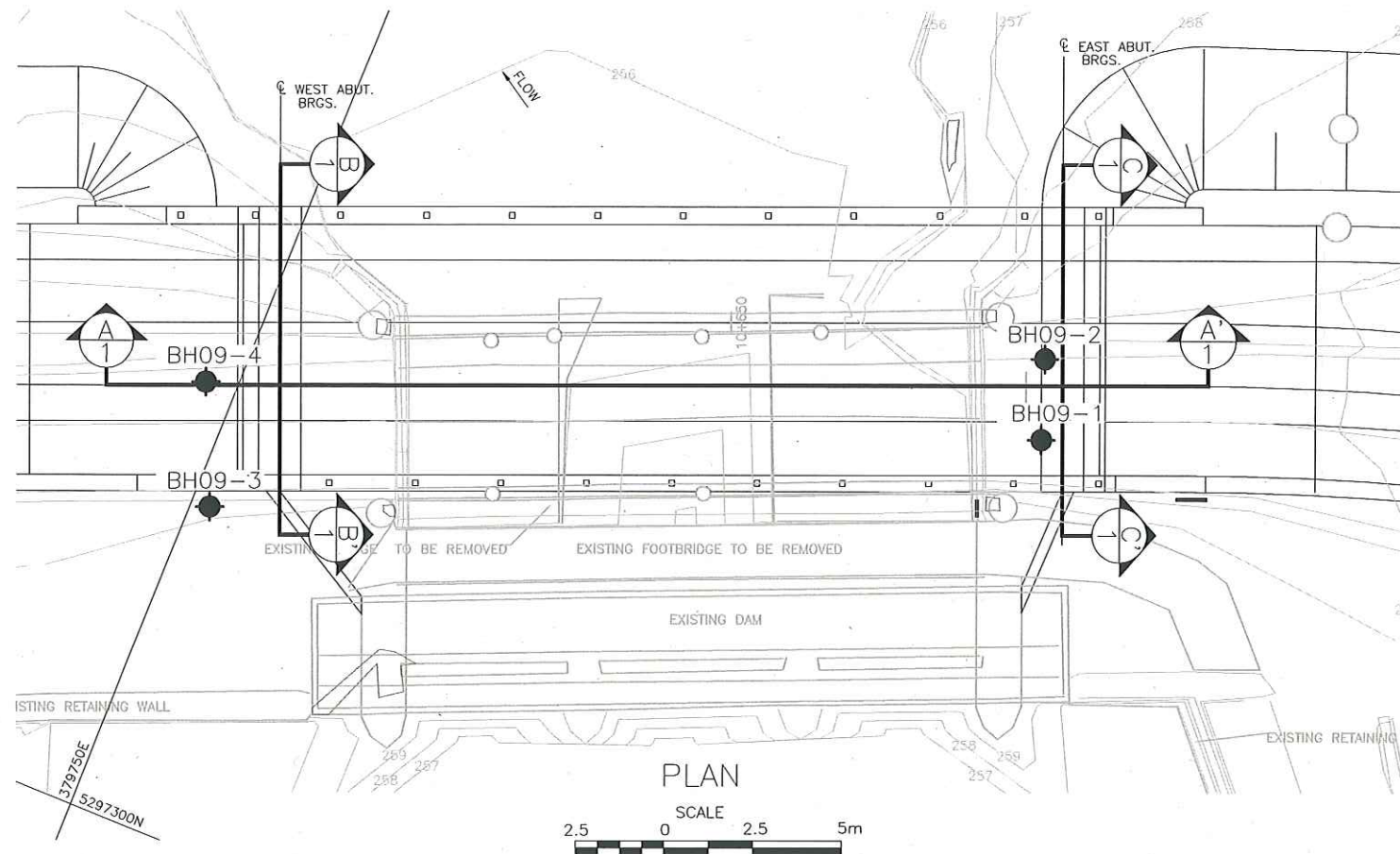
### Ministry of Transportation Ontario Special Provisions

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; Rack Embankment

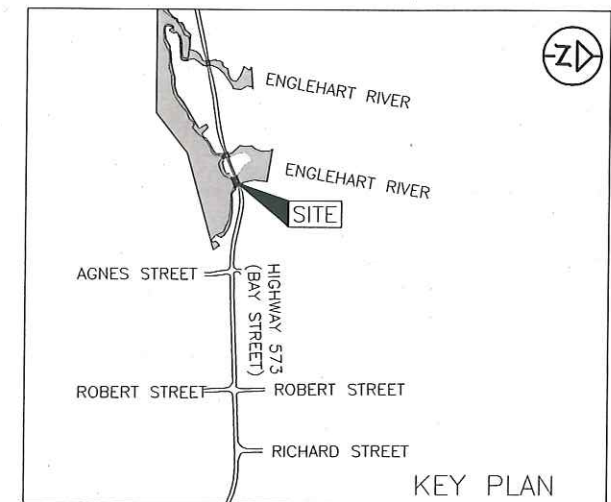
Northern Region Directive; Backfill to Structures Adjacent to Rock Embankment Approaches, November 2002



CONT No. 5109-05-00  
GWP No. 5109-05-00  
ENGLEHART (BLANCHE) RIVER  
HIGHWAY 573 BRIDGE 47-030  
BOREHOLE LOCATION  
AND SOIL STRATA



Golder Associates Ltd.  
SUDBURY, ONTARIO, CANADA



#### LEGEND

- Borehole
- 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
- R Refusal

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
BH09-01	259.5	5297319.6	379771.5
BH09-02	259.6	5297321.8	379770.8
BH09-03	259.2	5297309.2	379750.5
BH09-04	259.2	5297312.4	379749.1

#### REFERENCE

General Arrangement Drawing provided in digital format by URS, drawing file nos B#\_47-030.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-86  
HWY. 573 PROJECT NO. 09-1191-0027 DIST.  
SUBM'D. EC CHKD. DATE: MAY 2010 SITE: 47-030  
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

$\pi$	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

$\gamma$	shear strain
$\Delta$	change in, e.g. stress: $\Delta\sigma$
$\epsilon$	linear strain
$\epsilon_v$	volumetric strain
$\eta$	coefficient of viscosity
$\nu$	Poisson's ratio
$\sigma$	total stress
$\sigma'$	effective stress ( $\sigma' = \sigma - u$ )
$\sigma_{vo}$	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
$\sigma_{oct}$	mean stress or octahedral stress = $(\sigma_1 + \sigma_2 + \sigma_3)/3$
$\tau$	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

### III. SOIL PROPERTIES

#### (a) Index Properties

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

\* Density symbol is  $\rho$ . Unit weight symbol is  $\gamma$  where  $\gamma = \rho g$  (i.e. mass density 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
$\sigma'_p$	pre-consolidation pressure
OCR	over-consolidation ratio = $\sigma'_p / \sigma'_{vo}$

#### (d) Shear Strength

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

Notes: 1  $\tau = c' + \sigma' \tan \phi'$   
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.).

<b>PH:</b>	Sampler advanced by hydraulic pressure
<b>PM:</b>	Sampler advanced by manual pressure
<b>WH:</b>	Sampler advanced by static weight of hammer
<b>WR:</b>	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 cm<sup>2</sup> 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	$C_u, S_u$	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 <sub>p</sub>	plastic limit
w <sub>l</sub>	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test <sup>1</sup>
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement <sup>1</sup>
D <sub>R</sub>	relative density (specific gravity, $G_s$ )
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 <sub>4</sub>	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
$\gamma$	unit weight

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

PROJECT 09-1191-0027		RECORD OF BOREHOLE No BH09- 1				1 OF 1 METRIC						
W.P. 5109-05-00		LOCATION N 5297319.6 ;E 379771.5		ORIGINATED BY EHS								
DIST HWY 573		BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers		COMPILED BY AMW								
DATUM Geodetic		DATE October 5, 2009		CHECKED BY EC								
SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE								
259.5	GROUND SURFACE						20 40 60 80 100					
0.0	Cold Mix ASPHALT						20 40 60 80 100					
259.1	ASPHALT						20 40 60 80 100					
0.4	Sand to sand and gravel, some silt, trace clay containing cobbles and boulders (FILL)		1	AS	-	259						
	Compact Greyish brown to brown Moist to wet		2	SS	17							45 32 17 -6
258.0						258						
1.5	Difficult drilling to 1.5 m		3	SS	26							
257.3	Sandy SILT to SILT trace to some clay, trace to some gravel containing organics and organic seams											
2.2	Compact Brown Wet Organic seam, 50 mm thick at 1.8 m depth End of Borehole Auger Refusal											
Notes: 1. Water level measured at a depth of 1.9 m below surface (Elev. 257.6 m) upon completion of drilling. 2. Piezometer dry on November 26, 2009.												

MIS-MTO 001 09-1191-0027 URS BLACHE 3 BRIDGES.GPJ GAL-MISS.GDT 4/5/10

PROJECT <u>09-1191-0027</u>		<b>RECORD OF BOREHOLE No BH09-2</b>		1 OF 1 <b>METRIC</b>	
W.P. <u>5109-05-00</u>	LOCATION <u>N 5297321.8 ; E 379770.8</u>	ORIGINATED BY <u>EHS</u>			
DIST <u>HWY 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 NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W <sub>p</sub>	W	W <sub>L</sub>		
259.6	GROUND SURFACE													
0.0	Cold Mix ASPHALT													
259.1	Two 100 mm layers of gravelly sand, trace silt within asphalt													
0.5	Gravelly sand, trace silt containing cobbles and boulders (FILL)		1			259								
	Compact Brown Moist		2	SS	23									
258.1														
257.8	Difficult drilling between 0.76 m and 1.5 m; augers dipping north		3	SS	3/0.08	258								35 47 14 4
1.8	SAND and GRAVEL, some silt, trace clay, trace organics Very loose Brown Moist to wet End of Borehole Auger Refusal; augers dipping north													
	Note:  1. Borehole dry upon completion of drilling.													

MIS-MTO 001 09-1191-0027 URS BLACHE 3 BRIDGES GPJ GAL-MISS.GDT 4/5/10

PROJECT 09-1191-0027		RECORD OF BOREHOLE No BH09-3				1 OF 1 METRIC								
W.P. 5109-05-00		LOCATION N 5297309.2 ; E 379750.5				ORIGINATED BY EHS								
DIST _____ HWY 573		BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers				COMPILED BY AMW								
DATUM Geodetic		DATE October 6, 2009				CHECKED BY EC								
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 $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X REMOULDED 20 40 60 80 100						
259.2	GROUND SURFACE													
0.0	Cold Mix ASPHALT													
0.3	Gravelly sand (FILL) ASPHALT		1	SS	23		259							32 63 (5)
	Rock fill in a sand and gravel, trace silt matrix (FILL)													
	Loose to compact Moist to wet		2	SS	8		258							
	Difficult drilling throughout		3	SS	20/0.15									
257.1	End of Borehole Auger Refusal													
2.1	Notes:  1. Borehole dry upon completion of drilling; caved at 1.0 m depth.  2. Spoon attempted at 1.5 m; spoon empty. Spoon slid along probable boulder.  3. Large void at 0.6 m depth; no cuttings after 0.45 m.													

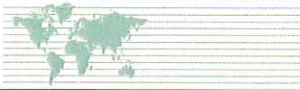
MIS-MTO 001 09-1191-0027 URS BLACHE 3 BRIDGES.GPJ GAL-MISS.GDT 4/5/10

PROJECT 09-1191-0027		<b>RECORD OF BOREHOLE No BH09-4</b>				1 OF 1 <b>METRIC</b>	
W.P. 5109-05-00		LOCATION N 5297312.4 ; E 379749.1				ORIGINATED BY EHS	
DIST HWY 573		BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers				COMPILED BY AMW	
DATUM Geodetic		DATE October 6, 2009				CHECKED BY EC	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X REMOULDED											
						20	40	60	80	100							
259.2	GROUND SURFACE																
0.0	ASPHALT																
	Gravelly sand, trace silt (FILL)		1	AS	-												
	Brown																
0.6	Moist																
	ASPHALT																
	Gravelly sand, trace silt (FILL)		2	SS	26												
	Brown																
	Moist																
	Rock fill in a silty sand, some gravel matrix (FILL)		3	SS	20/0.28												
257.1	Compact																
2.1	Brown																
	Moist to wet		4	SS	29/0.2												
256.4	Difficult drilling below 0.76 m																
2.8	SAND and GRAVEL, some silt, trace clay, trace organics																
	Dense																
	Brown to dark grey																
	Moist to wet																
	End of Borehole																
	Auger Refusal																
Notes: 1. Borehole dry upon completion of drilling. 2. Sample 3: Spoon bouncing and sliding; spoon empty.																	

MIS-MTO 001 09-1191-0027 URS BLACHE 3 BRIDGES GPJ GAL-MISS.GDT 4/5/10



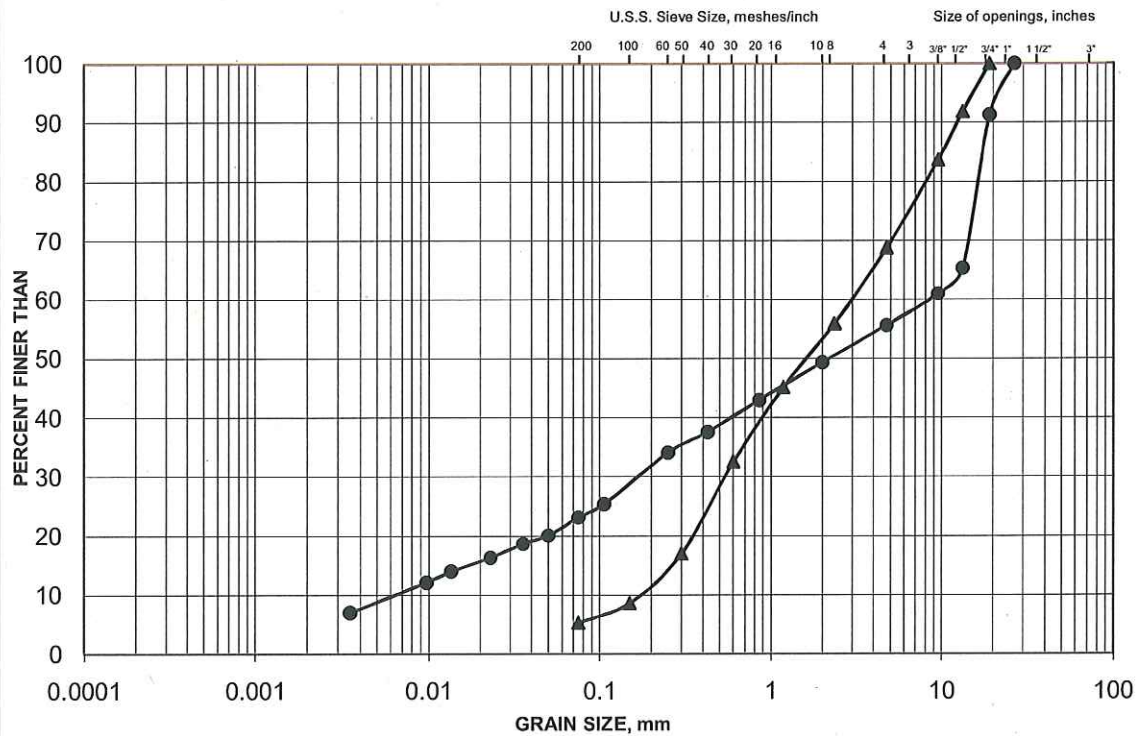
# APPENDIX B

## Laboratory Test Results

# GRAIN SIZE DISTRIBUTION

Sand and Gravel (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)
●	BH09-1	2	258.4
▲	BH09-3	1	258.9

Project Number: 09-1191-0027

Checked By: SEMC

Golder Associates

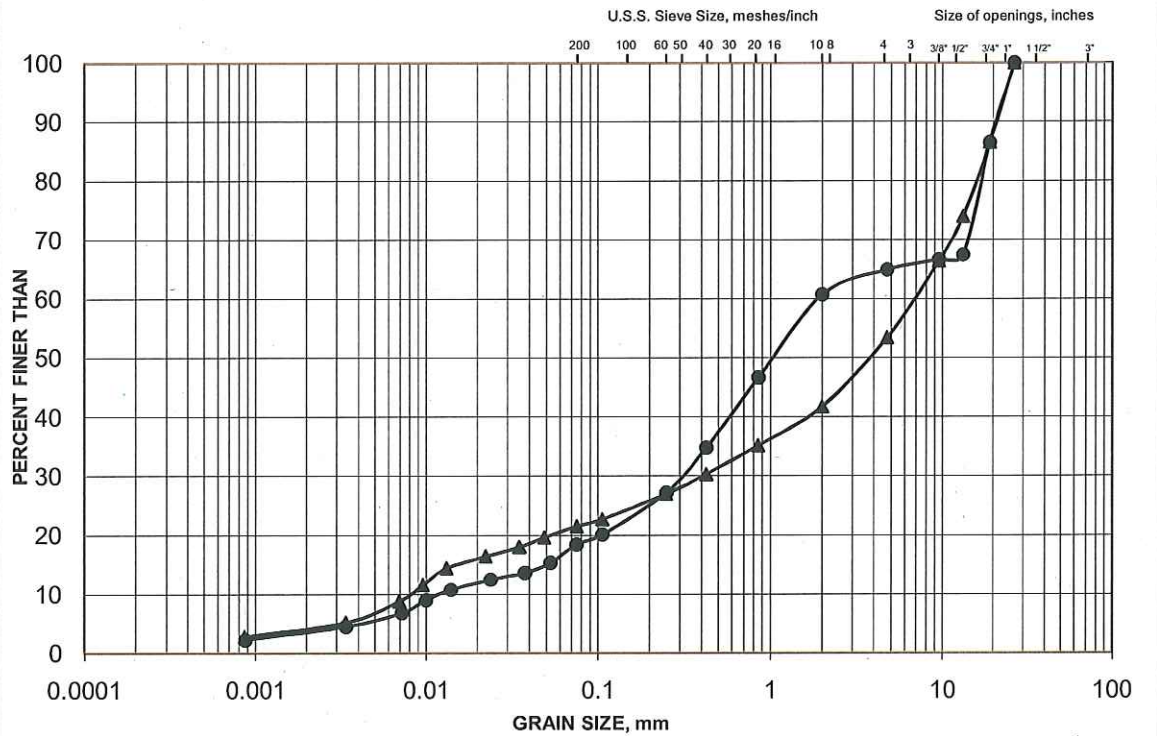
Date: May 2010

# GRAIN SIZE DISTRIBUTION

Sand and Gravel

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)
●	BH09-2	3	257.9
▲	BH09-4	4	256.7

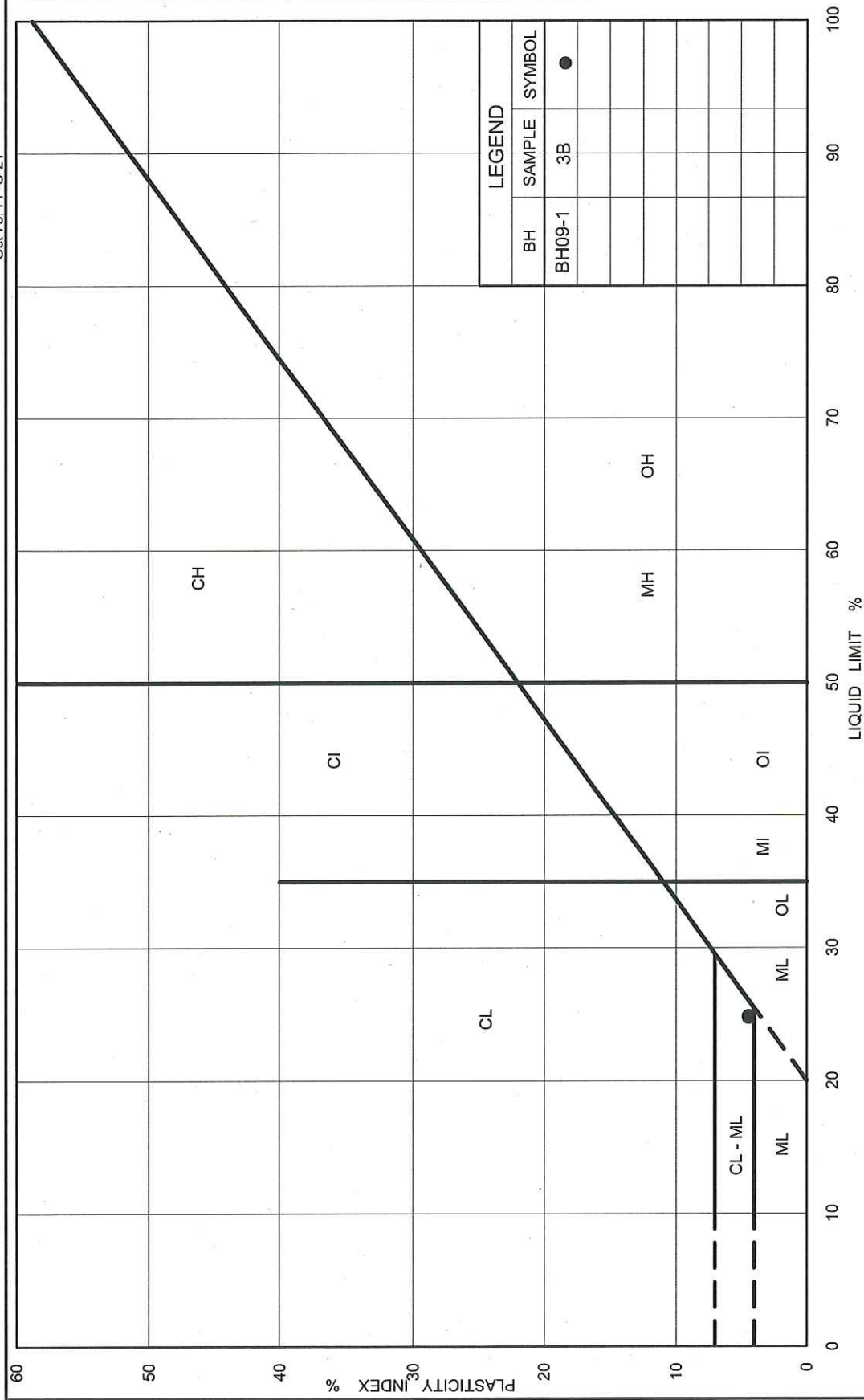
Project Number: 09-1191-0027

Checked By: SEMC

**Golder Associates**

Date: May 2010

Oct 75, FF-S-21



LEGEND		
BH	SAMPLE	SYMBOL
BH09-1	3B	●

Figure B-3  
Project No. 09-1191-0027, Site No. 47-030  
Checked By: SEMC

PLASTICITY CHART  
Silt

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

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