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**REPORT ON**

**FOUNDATION INVESTIGATION AND DESIGN  
EXISTING HIGHWAY 69 / BLAIR (SLY) CREEK  
REPLACEMENT STRUCTURE  
HIGHWAY 69 FROM 1.5 KM SOUTH OF HIGHWAY 559  
TO 3.5 KM NORTH OF HIGHWAY 559  
PARRY SOUND, ONTARIO  
G.W.P 335-00-00  
MINISTRY OF TRANSPORTATION, ONTARIO**

Submitted to:

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## **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 carry out a detailed foundation investigation as part of the detailed design for the Existing Highway 69 (Service Road) replacement structure over Blair (Sly) Creek. The proposed work is part of the detailed design for the four-laning of Highway 69 and re-alignment of Highway 559 north of Nobel, Ontario including the construction of associated new highway on- and off-ramps, access and service roads, bridges and overhead truss sign structures. The general location of the Highway 69 and Highway 559 alignments are shown on the Site Location Map on Figure 1.

The terms of reference for the scope of work are outlined in Golder's proposal P31-1270 dated July 2003 that forms part of the Consultant's Agreement (Number P.O.5005-A-000320) for this project. The addendum work to assess the stability of Service Road grade raise between Station 11+600 and 11+660 adjacent to the realignment of Blair (Sly) Creek was carried out in accordance with Golder's proposal letter titled, "Proposed Additional Work – Revision 1", dated June 2, 2006. All work was carried out in accordance with the Quality Control Plan for this project dated October 2003. The General Arrangement (GA) Drawing for the proposed replacement structure was provided to Golder by URS on November 11, 2005.

This report addresses only the investigation for the Existing Highway 69 (Service Road) replacement structure over Blair (Sly) Creek, the associated approach embankments and the grade raise on the Service Road between 11+600 and 11+660. Separate reports detail the foundation investigations for the swamp crossings, high fill areas, other bridge structures and overhead truss sign structures for the project.

The purpose of this investigation is to establish the subsurface conditions at the proposed replacement structure and in the area adjacent to the Service Road near the proposed Blair (Sly) Creek realignment by borehole drilling, rock coring, in situ testing and laboratory testing on selected samples. The boreholes for the current investigation were located in the field by Callon Dietz Incorporated (Callon Dietz), a professional surveying company retained by URS. The locations of the investigated areas are shown in plan on Drawings 1A and 1C.

## **2.0 SITE DESCRIPTION**

The proposed replacement structure site is located along Existing Highway 69 (north of Nobel, Ontario) at Blair (Sly) Creek, approximately 300 m north of Existing Highway 559 (as shown on Figure 1).

In general, the topography in the area of the overall project site consists of rolling terrain including densely treed areas and numerous bedrock outcrops separated by low-lying swamp areas. The proposed replacement structure is to be constructed at the existing bridge location which spans over a relatively narrow section of Blair (Sly) Creek and its floodplain. The existing highway approach embankments at the bridge site and near the creek realignment rise about 3 m above a lower-lying, moderately treed area. The existing ground surface within the limits of the proposed structure, approach embankments and creek realignment generally lies between about Elevation 199.7 m (approximate existing top of pavement elevation) and Elevation 195.0 m (approximate creek bed elevation), referenced to Geodetic Datum. Photographs of the site are provided in Appendix B.

### **3.0 INVESTIGATION PROCEDURES**

#### **3.1 Foundation Investigation**

The first phase of the field work for the proposed Existing Highway 69 (Service Road) replacement structure investigation was carried out between October 28 and November 2, 2004 during which time a total of four (4) sampled boreholes (BCB-1, BCB-3, BCB-5 and BCB-6) were put down at the site. One borehole was drilled and cored at each of the proposed south and north foundation element locations and one borehole was advanced to refusal at each of the existing south and north approach embankments. All of these boreholes were advanced to refusal on inferred bedrock. In boreholes BCB-3 and BCB-5, at each foundation element location, bedrock coring was carried out for a minimum length of 3 m.

The second phase of the field work was carried out on January 11, 2006 during which time borehole BCB-2 was advanced near the west toe of the south approach of the existing Highway 69 embankment. Blair (Sly) Creek runs roughly parallel and in relatively close proximity to embankment toe in this area; therefore, the purpose of this borehole was to gather additional subsurface information in order to better assess the stability of the proposed embankment and optimize side berm configuration relative to the creek alignment. Note that no borehole named BCB-4 was advanced during either investigation phase.

The third phase of the field work was carried out on June 29, 2006 during which time five (5) sampled boreholes (06-1 to 06-5) were advanced along the left (east) toe of slope of the Service Road between Station 11+600 and 11+660 in the area of the proposed Blair (Sly) Creek realignment. The purpose of these boreholes was to gather additional subsurface information in order to assess the stability of the proposed embankment grade raise in this area and assess if there was a need for stability mitigation measures.

The first phase of drilling was carried out with a track-mounted CME 55 drill rig using 108 mm inside diameter (I.D.) continuous flight hollow stem augers, while the second and third phases of drilling utilized portable drilling equipment with 'NQ' size casing; both supplied and operated by Marathon Drilling Co. Ltd. of Ottawa, Ontario. Soil samples were obtained, where possible, continuously or at intervals of about 0.75 m to 1.5 m depth, using a 50 mm outer diameter (O.D.) split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586-99). Field vane shear tests were conducted in cohesive soils for determination of undrained shear strengths (ASTM D 2573-01). Samples of the bedrock were obtained using an 'NQ' size rock core barrel.

With the exception of BCB-2, 06-1 and 06-3 to 06-5, the boreholes were advanced to auger and/or sampler refusal (i.e. inferred bedrock) which occurred at depths ranging from 4.4 m to

15.8 m below the existing ground surface (not including rock coring). Borehole 06-2 near the creek realignment was terminated upon refusal on a possible boulder or bedrock at a depth of about 0.7 m. At boreholes BCB-3 and BCB-5, located within the footprints of the proposed foundation units, the drilling was further advanced into the bedrock by coring 3.3 m and 3.7 m, respectively. The groundwater level in the open boreholes was observed throughout the drilling operations and piezometers were installed in BCB-1 and BCB-5 to permit monitoring of the groundwater level at these locations. The piezometers consisted of 38 mm O.D. threaded PVC pipe with a slotted screen at depth surrounded by a sand filter and sealed with bentonite above the filter to ground surface within the boreholes. The installation details and water level readings are described on the Record of Borehole sheets that follow the text of this report. All boreholes and piezometers were abandoned in accordance with O. Reg. 128 (amendment to O. Reg. 903). The piezometers were abandoned on January 4, 2006.

The field work was supervised throughout by members of our engineering and technical staff, who confirmed the locations of the boreholes, arranged for the clearance of underground service locations, supervised the drilling, sampling and in situ testing operations, logged the boreholes, and examined and cared for the soil and rock samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Mississauga geotechnical laboratory where the samples underwent further detailed visual examination and appropriate laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards as appropriate. Classification testing such as water content, grain size distribution and Atterberg limits were carried out on samples of the overburden soils. Strength testing such as point load index were carried out on specimens from the rock core.

All of the borehole locations (except BCB-2 and 06-1 to 06-5) were located in the field by Callon Dietz prior to drilling operations. The surveying of the elevations of the as-drilled boreholes and the locations of boreholes BCB-2 and 06-1 to 06-5 was carried out by members of our engineering staff, referenced to staked benchmark geodetic elevations provided by Callon Dietz. The borehole locations and ground surface elevations are shown on Drawing 1A.

## **4.0 GENERAL SITE GEOLOGY AND SUBSURFACE CONDITIONS**

### **4.1 Geology**

From published geologic information, the site is located in the physiographic region known as the Georgian Bay Fringe. The Georgian Bay Fringe borders Georgian Bay as a broad belt characterized by shallow soil and bare bedrock knobs and ridges (The Physiography of Southern Ontario; Third Edition, 1984) however; Quaternary deposits of lacustrine and fluvial origin together with more recent swamp sediments have been accumulated between the bedrock ridges and, consequently, the overburden thickness and bedrock surface can be variable. The bedrock in the area are typically highly deformed gneisses and migmatites of the Britt Domain of the Central Gneiss Belt, a subdivision of the Grenville Structural Province (Geology of Ontario; OGS Special Volume 4, 1991). Deposition of Paleozoic strata and later erosion during glaciation left behind these Precambrian rocks covered only in a few places by flat-lying Palaeozoic bedrock strata.

### **4.2 Subsurface Conditions and General Overview – Replacement Structure**

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 given on the attached Record of Borehole sheets following the text of this report. The results from the laboratory testing are provided in Appendix A. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests (SPTs). These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Further, subsurface conditions will vary between and beyond the borehole locations.

The inferred soil stratigraphy as encountered in the boreholes at the proposed Existing Highway 69 (Service Road) replacement structure over Blair (Sly) Creek is shown on Drawings 1A and 1B. The inferred stratigraphy as encountered in the boreholes advanced in the area of the proposed Blair (Sly) Creek realignment adjacent to the Service Road between Station 11+600 and 11+660 is shown on Drawing 1C. Site photographs are provided in Appendix B.

In general, the subsoils at the structure site consist of embankment fill of the existing highway or peat in the floodplain adjacent to the embankment underlain by successive deposits silty sand to sandy silt, clayey silt to clay, silt to sandy silt and silty sand to sand over bedrock. The total overburden thickness as encountered in boreholes BCB-1, BCB-3, BCB-5 and BCB-6 ranges from 4.4 m, northeast of the existing bridge to 15.8 m, southwest of the existing bridge. Boreholes BCB-1 and BCB-6 were terminated at the inferred bedrock surface, while boreholes BCB-3 and BCB-5 were cored at least three metres into the bedrock.

A more detailed description of the subsurface conditions encountered in the boreholes advanced within the vicinity of the Replacement Bridge Structure (BCB-1, BCB-2, BCB-3, BCB-5 and BCB-6) is provided in the following sections. A description of the subsurface conditions encountered in the area of the proposed Blair (Sly) Creek realignment (06-1 to 06-5) is provided in Section 4.3.

#### **4.2.1 Existing Embankment Fill**

Boreholes BCB-1, BCB-3, BCB-5 and BCB-6 were advanced through embankment fill on the shoulder of the existing highway. The fill is composed of sand containing trace to some gravel and silt with trace asphalt noted in several boreholes and some organics observed at the fill-native ground interface in BCB-3. The ground surface elevations at the borehole locations ranged between Elevation 199.6 m and 199.4 m and the fill-native ground interface was found to range between about Elevations 197.7 m and 196.7 m, indicating an embankment thickness ranging from about 1.7 m to 2.9 m.

Standard Penetration Testing (SPT) carried out within the embankment fill measured 'N' values ranging from 2 blows to 18 blows per 0.3 m of penetration, indicating a very loose to compact relative density.

The natural water content measured on samples of the embankment fill ranged between 7 percent and 27 percent, typically less than 10 percent.

#### **4.2.2 Peat**

Borehole BCB-2 was advanced in the floodplain immediately west of the existing highway embankment toe, south of the existing bridge structure. Soft, wet, black fibrous to sandy peat (with depth) was encountered from ground surface (Elevation 196.3 m) to about 0.8 m depth in the borehole.

#### **4.2.3 Silty Sand to Sandy Silt**

A grey, oxidized silty sand to sandy silt deposit containing trace to some clay and trace organics was encountered below the existing highway embankment fill or below the peat in all of the boreholes. As noted in Section 4.2.1, some organics were encountered at the fill-native ground interface in BCB-3. The top of the silty sand to sandy silt deposit ranged from Elevation 197.7 m to 195.5 m and the thickness ranged from 0.5 m to 1.5 m.

Standard Penetration Testing (SPT) carried out within this stratum measured 'N' values ranging from 2 blows to 22 blows per 0.3 m of penetration, typically less than 10 blows per 0.3 m of penetration. The 'N' values indicate a very loose to compact relative density within the deposit.

The natural water content measured on two samples of this deposit were 23 percent and 25 percent.

#### **4.2.4 Clayey Silt to Clay**

A deposit of reddish brown to grey silty clay to clay (clayey silt to silty clay in borehole BCB-2) containing trace to some sand and trace organics was encountered below the silty sand to sandy silt in all boreholes for the replacement structure. The soil structure of this deposit was noted to be mottled and thin sand seams were observed within this stratum in certain boreholes. The top of this layer varied between Elevation 196.9 m and 194.0 m and the thickness ranged from 1.7 m to 3.8 m. The bottom of this deposit was defined by refusal to further auger advancement in borehole BCB-6.

Standard Penetration Testing (SPT) carried out within this stratum measured 'N' values ranging from 0 blows (i.e. weight of rods/hammer) to 7 blows per 0.3 m of penetration.

In situ field vane testing carried out within this stratum below the existing highway embankment measured undrained shear strengths ranging from 21 kPa to 86 kPa, indicating a soft to stiff consistency. In situ field vane testing carried out within this stratum in the floodplain adjacent to the existing embankment measured undrained shear strengths ranging from 12 kPa to 16 kPa, indicating a very soft to soft consistency.

The natural water content measured on samples of this deposit ranged between 28 percent and 44 percent with an average of 37 percent.

A grain size distribution for one (1) sample from this deposit is shown on Figure A-3 of Appendix A. Atterberg limits testing was carried out on four (4) samples of the silty clay to clay. The liquid limit ranged from about 39 to 55 percent and the plastic limit ranged from about 12 to 20 percent yielding a plasticity index ranging from about 23 to 35 percent. The results of the Atterberg limits testing are shown on the plasticity chart on Figure A-4 in Appendix A.

#### **4.2.5 Silt to Sandy Silt**

A grey silt to sandy silt deposit containing trace to some clay and occasional clay seams and silt layers was encountered below the clayey silt to clay in the boreholes south of Blair (Sly) Creek. The top of this deposit ranged from Elevation 193.7 m to 192.3 m and the thickness ranged from 0.7 m to 1.9 m.

Standard Penetration Testing (SPT) carried out within this stratum measured 'N' values of 0 blows (i.e. weight of rods / hammer) per 0.3 m of penetration. The 'N' values indicate a very loose relative density within the deposit.

The natural water content measured on a sample of this deposit was 21 percent.

Grain size distributions for two (2) samples from this deposit are shown on Figure A-1 of Appendix A.

#### **4.2.6 Silty Sand to Sand**

A grey silty sand to sand deposit containing trace gravel, silt and clay was found to transition from the overlying sandy silt deposit in boreholes BCB-1, BCB-2, BCB-3 and BCB-5, typically becoming coarser with depth. The top of this deposit ranged from Elevation 193.1 m north of Blair (Sly) Creek to 191.6 m south of Blair Creek, with a thickness ranging from 3.5 m to 8.2 m. The bottom of this deposit was defined by refusal to further auger advancement and was confirmed by rock coring in boreholes BCB-3 and BCB-5. Borehole BCB-2 was terminated within this layer.

Standard Penetration Testing (SPT) carried out within this stratum measured 'N' values of 0 blows (i.e. weight of rods / hammer) to 15 blows per 0.3 m of penetration, typically greater than 5 blows per 0.3 m of penetration. The 'N' values indicate a very loose to compact relative density within the deposit.

The natural water content measured on samples of this deposit ranged between 16 percent and 22 percent.

A grain size distribution for one (1) sample from this deposit is shown on Figure A-2 of Appendix A.

#### **4.2.7 Bedrock**

Bedrock was encountered and cored in boreholes BCB-3 and BCB-5. The presence of bedrock was inferred from refusal to further drilling advance in boreholes BCB-1 and BCB-6. At these borehole locations, the bedrock surface ranges from Elevation 195.1 m to Elevation 189.5 m north of the existing bridge and from Elevation 185.4 m to 183.6 m south of the existing bridge.

The upper portion of the bedrock samples are described as fresh to slightly weathered, light grey to pink, medium grained, faintly porous granitic gneiss containing near horizontal, distinct foliation. Below about Elevation 183.5 m in BCB-3 and Elevation 187.4 m in BCB-5, the pinkish grey granitic gneiss quickly transitioned to a fresh, grey and black, fine to medium grained, faintly porous, mafic-rich gneiss containing significantly more biotite and amphibole (segregated into distinct foliation planes) than the overlying granitic gneiss. The Total Core Recovery measured on the core samples was between 95 percent and 100 percent. The Rock

Quality Designation (RQD) measured on the core samples ranged from 41 percent to 100 percent, typically greater than 60 percent, indicating a rock mass of poor to excellent quality.

Axial and diametral point load strength tests were performed on samples of the rock core. Diametral point load strength index values are shown on the Record of Drillhole Sheets. For the upper granitic gneiss, axial point load strength index values ranged from 7.2 MPa to 8.6 MPa and diametral point load strength index values ranged from 3.2 MPa to 5.9 MPa, indicating a strong to very strong rock mass. For the lower mafic-rich gneiss, an axial point load strength index value of 4.6 MPa was measured and diametral point load strength index values ranged from 2.6 MPa to 7.2 MPa, indicating a strong to very strong rock mass. A summary of the point load index values on the rock core from the two (2) boreholes where coring was carried out is shown in the following table. Table 1 following the text of this report presents a detailed list of all point load index testing results performed for this investigation along with the associated approximate Unconfined Compressive Strength (UCS) value for each test.

<b>Borehole (Drillhole) No.</b>	<b>Rock Type</b>	<b>Average Axial Point Load Index (MPa)</b>	<b>Average Diametral Point Load Index (MPa)</b>
BCB-3	Granitic Gneiss	7.4	5.4
BCB-3	Mafic-rich Gneiss	-	4.0
BCB-5	Granitic Gneiss	8.6	3.7
BCB-5	Mafic-rich Gneiss	4.6	6.1

#### **4.2.8 Groundwater Conditions**

The groundwater levels in the piezometers installed at the soil / bedrock interface in borehole BCB-1 and in the bedrock of borehole BCB-5 were measured at about Elevations 196.5 m (2.9 m depth) and 196.6 m (3.0 m depth), respectively on November 14, 2004 and on January 4, 2006. The water level of Blair (Sly) Creek was measured at Elevation 196.2 m in July 2004 and 196.3 m in January 2006. Details of the piezometer installations, groundwater conditions and water levels observed in the open boreholes at the time of drilling are summarized on the Record of Borehole sheets following the text of this report. It should be noted that groundwater levels in the area are subject to seasonal fluctuations.

#### **4.3 Subsurface Conditions – Blair (Sly) Creek Realignment Area**

In general, the subsoils in the area of the proposed Blair (Sly) Creek realignment (i.e. at the left toe of the Service Road between Station 11+600 and 11+660) are similar to those in the area of the replacement structure and consist of a thin surficial layer of organics/topsoil underlain by successive deposits silty sand to sandy silt, silty clay to clay and silty sand to sand. Most of the

boreholes were terminated in the silty sand to sand stratum at depths of about 2 m to 3.5 m. Bedrock was not confirmed in any of the boreholes, however, borehole 06-2 was terminated at a shallow depth (about 0.7 m) at refusal on either a large boulder or possible bedrock.

#### **4.3.1 Silty Sand to Sandy Silt**

A brown to grey, silty sand to sand to sandy silt deposit containing trace clay, trace to some gravel and occasional organics was encountered below the surficial topsoil/ground surface in all of the boreholes. The top of the silty sand to sandy silt deposit ranged from Elevation 196.8 m to 198.2 m and the thickness ranged from about 0.5 m to greater than 2.1 m. Boreholes 06-2 and 06-4 were terminated in this layer.

Standard Penetration Testing (SPT) carried out within this stratum measured 'N' values ranging from 1 blows to 20 blows per 0.3 m of penetration, but typically less than 5 blows per 0.3 m of penetration. The 'N' values indicate a typically very loose to loose relative density within the deposit.

#### **4.3.2 Silty Clay to Clay**

A deposit of reddish brown to grey silty clay to clay containing trace silt was encountered below the silty sand to sandy silt in boreholes 06-1, 06-3 and 06-5. The top of this layer varied between Elevation 197.2 m and 195.4 m and the thickness ranged from about 0.8 m to 2.1 m.

A single Standard Penetration Test (SPT) carried out within this stratum measured an 'N' value of 3 blows per 0.3 m of penetration suggesting a soft consistency.

In situ field vane testing carried out within this stratum measured undrained shear strengths ranging from about 12 kPa to 95 kPa (indicating a very soft to stiff consistency), but the shear strengths generally ranged from about 25 kPa to 45 kPa indicating a firm consistency.

#### **4.3.3 Sand to Silty Sand**

A grey sand to silty sand deposit containing some gravel was encountered below the clayey silt to clay in boreholes 06-1, 06-3 and 06-5. The top of this deposit ranged from Elevation 194.1 m to 195.6 m. The boreholes were terminated at depths ranging from 2.7 m to 3.5 m within this stratum.

Standard Penetration Testing (SPT) carried out within this stratum measured 'N' values of 2 to 14 blows per 0.3 m of penetration. The 'N' values indicate a very loose to compact relative density within the deposit.


#### 4.3.4 Groundwater Conditions

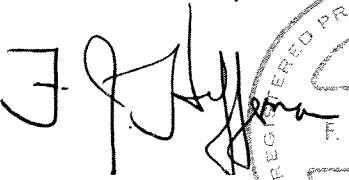
The groundwater levels in the boreholes as encountered during drilling operations ranged from depths of about 0.3 m to 0.6 m below ground surface (Elevation 196.4 m to 197.9 m). Details of the groundwater conditions and water levels observed in the open boreholes at the time of drilling are summarized on the Record of Borehole sheets following the text of this report. It should be noted that groundwater levels in this area are subject to seasonal fluctuations.

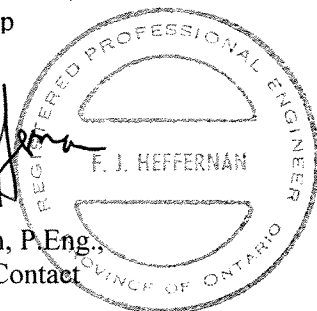
#### 4.4 CLOSURE


This Foundation Investigation Report was prepared by Mr. Chad Gilfillan and reviewed by Dr. J. Paul Dittrich, Ph.D., P.Eng., an Associate with Golder. Mr. Fintan Heffernan, P.Eng., Golder's Designated MTO Contact for this project, conducted an independent quality review of the report.

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**PART B**

**FOUNDATION DESIGN  
EXISTING HIGHWAY 69 / BLAIR (SLY) CREEK  
REPLACEMENT STRUCTURE  
HIGHWAY 69 FROM 1.5 KM SOUTH OF HIGHWAY 559  
TO 3.5 KM NORTH OF HIGHWAY 559  
PARRY SOUND, ONTARIO  
G.W.P 335-00-00  
MINISTRY OF TRANSPORTATION, ONTARIO**

## **5.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS**

This section of the report provides recommendations on the foundation aspects of the proposed existing Highway 69 (Service Road) Replacement Structure over Blair (Sly) Creek. The recommendations are based on interpretation of the factual geotechnical data obtained from the boreholes advanced during the subsurface investigation.

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

### **5.1 General**

It is understood that the existing Highway 69 (Service Road) / Blair (Sly) Creek Replacement Structure will consist of a 39 m single span, slab-on-girder structure with abutments located north and south of Blair (Sly) Creek.

The existing bridge structure spans approximately 27.5 m over Blair (Sly) Creek on a southeast-northwest skew. The existing top of pavement grade at the structure is estimated to range from about Elevation 199.6 m to 199.8 m. The results of a foundation investigation for the existing structure completed in October 1956 is included in Appendix D. It is understood that the existing structure, including the abutments and piers (to 0.6 m below final grade and 0.6 m below creek bed, respectively), are to be removed.

Based on the information provided on the General Arrangement (GA) Drawing provided by URS on November 11, 2005, the grade of the new bridge deck varies between about Elevations 201.0 m and 201.3 m. It is understood that the grade of the existing approach embankments is to be raised, with new approach embankments up to about 1.8 m above the existing top of pavement grade. This grade raise will result in a total embankment height of up to about 4.3 m south of the creek and 3.8 m north of the creek relative to the inferred natural ground surface (i.e. below existing embankment fill). In addition, as part of the removal of the existing bridge, the front slopes of the existing approach embankments will be excavated back (by up to 10 m long and 2 m deep at the south approach and up to 4 m long and 1.8 m deep at the north approach). The inferred natural ground surface is estimated to range from about Elevation 197.7 m to 196.3 m at the borehole locations. The high water level (HWL) for Blair (Sly) Creek has been assumed to be at Elevation 196.75 m based on the information on the GA Drawing.

Although not within the limits of the proposed replacement bridge structure and its associated approach embankments, it is noted that improvements to the vertical grade of the Service Road (existing Highway 69) are proposed to the north and south of the bridge site. It is our understanding that a grade raise ranging from about 0.4 m to 1.8 m is proposed between about Stations 11+400 and 11+712 (station at north abutment) to the north of the bridge structure. The toe of the raised embankment between about Stations 11+600 and 11+660 will be adjacent to an old channel meander of Blair (Sly) Creek. Given the soft subsoil conditions prevalent within this area of the site (i.e. close to the creek bed and its floodplains) and given the stability problems associated with the proposed grade raise at the south approach embankment at this replacement bridge site (as discussed in Section 5.6.1), additional foundation investigation was carried out within this area to assess the stability of the proposed embankment configuration between Stations 11+600 and 11+660. The results of this stability analysis are presented in Section 5.6.2.

Further to this, it is our understanding that Blair (Sly) Creek, over which the replacement structure will cross, is considered to be an environmentally sensitive area. The recommendations given in the following sections have taken this into account as it pertains to foundation design and construction, excavation, drainage and other considerations.

## **5.2 Bridge Foundation Options**

The native soils underlying the fill of the existing embankment or peat in the floodplain at the existing bridge site consist of successive deposits of sandy silt to silty sand, clayey silt to clay, silt to sandy silt (south of Blair Creek) and silty sand to sand over bedrock. The overburden thickness at the investigated locations ranges from about 2.4 m to 14.1 m, north and south of the creek, respectively. The native overburden soils are underlain by strong to very strong granitic (to mafic-rich) gneiss bedrock. The bedrock surface at the proposed foundation elements, as established in the single boreholes advanced for each, is at about Elevation 185.4 m and 189.5 m within the footprint of the south and north foundation elements, respectively. Based on this, it should be noted that the bedrock surface elevation at the north foundation is approximately 4.1 m higher (potentially up to about 6 m as per the results from the October 1956 investigation) than at the south foundation.

For the foundations, spread footings founded at shallow depth on either the upper loose sandy silt to silty sand or on the generally soft to firm silty clay to clay are not recommended due to the low axial resistance and expected settlement of these strata. Spread footings founded on the underlying cohesionless soils or bedrock are also not recommended due to the deep excavation, groundwater control and the temporary shoring that would be required adjacent to Blair Creek for construction.

Abutment footings perched within the existing (or new) embankment fill (i.e. on well compacted granular) is not considered a suitable alternative at either foundation element due to the compressible nature of the underlying native soils which would result in settlement of the footings. In addition, sub-excavation and removal of the clayey foundation soils followed by placement of granular in compacted lifts would need to be carried out in the dry and would require extensive dewatering which may be cost prohibitive.

The foundation alternatives noted above are summarized in Table 2. It is considered that supporting the abutments on piles driven to bedrock is the most feasible option at this location. The details of this option are presented in the following sections.

### 5.3 Steel H-Pile Foundations

As noted in Section 5.2, steel H-piles driven to refusal on the granitic gneiss bedrock is recommended for support of an integral abutment at both sides of the replacement structure.

If corrugated steel pipes (CSPs) are installed as part of the integral abutment design (through which the piles will be driven), the CSPs should be backfilled with a loose, fine to medium sand. A Non-Standard Special Provision (NSSP) detailing the gradation of this sand should be included in the Contract Documents (see example in Appendix C).

For design, the following pile tip elevations may be assumed for piles terminating on the bedrock surface. The elevations have been assessed based on a review of the depth to bedrock as encountered in the single boreholes put down at the area of the south and north abutments as well as a review of foundation investigation information for the existing bridge structure, completed in October 1956 (Appendix D). There should be a provision made in the Contract for dealing with varying pile lengths considering the variability of the elevation of the bedrock surface in this area.

<i>Foundation Unit</i>	<i>Approximate Design Pile Tip Elevation (m)</i>
South abutment	184 – 185.5
North abutment	187 – 189.5

#### 5.3.1 Axial Geotechnical Resistance

For HP 310 x 110 piles driven to practical refusal on the granitic gneiss bedrock, a factored axial resistance at ULS of 2,000 kN may be assumed for design. In the case of the driven H-piles, this value represents a structural limitation for the pile rather than a geotechnical limitation. The geotechnical resistance at SLS for 25 mm of settlement (for the length of piles required at this

site) will be greater than the factored axial resistance at ULS, since the granitic gneiss bedrock is considered to be unyielding; as such, ULS conditions will govern for this foundation type.

### **5.3.2 Downdrag Load (Negative Skin Friction)**

The loading from the grade raise of the approach embankments will cause consolidation and settlement of the underlying generally soft to firm silty clay to clay strata. If the piles are installed prior to completion of this settlement, because the piles are end-bearing on bedrock, a small amount of settlement of the silty clay to clay relative to the stiff pile will result in the development of negative skin friction on the piles. In this case, downdrag loads will need to be taken into account for design of the piles supporting the abutments.

Where the clayey foundation soils remain in place and are not preloaded, and if an integral abutment design is employed that does not utilize 3 m long corrugated steel pipe (CSP) around the upper portion of the pile, the abutment pile structural design should be based on the full downdrag loads acting on the piles. The estimated unfactored downdrag load acting on the HP 310x110 piles for this case may be taken as 80 kN per pile at the south abutment location and 100 kN per pile at the north abutment location. If the integral abutment design utilizes a CSP that surrounds the portion of the entire pile length embedded in the silty clay to clay, downdrag loads may be neglected.

The downdrag loads calculated in this manner are unfactored loads. The structural capacity of the piles must be checked for the factored dead and downdrag loads in accordance with Section 6.8.4 of the *CHBDC* for ULS conditions. The piles at this location are designed as end-bearing on the bedrock. For this condition (basically classified as non-yielding foundations), the settlement of the piles is largely governed by compression of the pile and will not be greater than 25 mm under the combined SLS and downdrag loading.

Downdrag loads can be reduced or eliminated by constructing preload embankments in the abutment areas (as discussed in Section 5.7) and allowing the settlement to occur prior to installing the piles. Another option to reduce downdrag loads would be to remove and replace the clayey subsoils; however, given the close proximity of Blair (Sly) Creek, excavation would likely be difficult at this site.

### **5.3.3 Lateral Loads (due to Horizontal Soil Deformations)**

In addition to downdrag loads, the effect of lateral loading on the piles caused by horizontal soil deformations (i.e. due to consolidation of clayey strata and lateral spreading under new embankment loading) may also have to be considered in the pile design.

Where the clayey foundation soils remain in place and are not preloaded prior to pile installation, the abutment pile structural design should include additional lateral loads acting on the piles. The estimated unfactored lateral load acting on the portion of the HP 310 x 110 piles embedded in the firm to soft clayey strata for this case are shown in the table below.

<i>Location</i>	<i>Soil Unit</i>	<i>Elevation (m)</i>	<i>Unfactored Lateral Load, <math>P_h</math> (kN/m length)</i>
South Abutment	Firm to Soft Silty Clay	195.0 – 193.5	90 (at top) to 60 (at bottom)
North Abutment	Firm to Soft Silty Clay to Clay	195.0 – 193.1	90 (at top) to 60 (at bottom)

Lateral loads on the piles can be reduced or eliminated by constructing preload embankments in the abutment areas and allowing the settlement and lateral movement to occur prior to pile installation. It is estimated (as discussed in Section 5.7) that the embankment preloading would have to remain in place for at least for at least 5 months and 7 months, at the south and north approach embankments, respectively, to reduce the lateral loads on the abutment piles. The time for preloading at the north approach could be reduced to about 5 months if a 0.5 m high surcharge is constructed on top of the preload embankment. Further details of the requirements for preloading and surcharging are discussed in Section 5.7. As noted above, another option to the reduce the lateral loads would be to remove and replace the clayey subsoils; again, given the close proximity of Blair (Sly) Creek, excavation could be difficult.

#### **5.3.4 Set Criteria**

Set criteria are highly dependent on pile driving hammer type and the selected pile. The set criteria can be established through a variety of methods, including empirical correlations and wave equation analyses, at the time of construction once the hammer and pile types are known. The choice of set criteria is dependent on the experience of the engineer and traditional use where a substantial database has been developed over the years. The criteria needs to be set to also avoid overdriving and possible damage to the piles.

Based on our experience, consideration should be given to the following preliminary criteria. The piles should be driven to an initial set equal to or greater than 10 blows per 12 mm of penetration (unless abrupt peaking occurs) using a hammer with rated energy of about 50 kilojoules but not exceeding 60 kilojoules. On reaching the required set, the hammer energy should be reduced by about 75 percent and the pile should then be re-driven by increasing the hammer energy slowly up to the maximum rated energy over about 40 blows. This procedure is intended to improve the process of the seating of the pile on the sloping bedrock surface. A final set of no less than 10 blows per 12 mm of penetration should be obtained at the maximum hammer energy. Provision should be made to re-tap all piles to confirm the set after adjacent piles have been driven. A

refusal rate of 20 blows per 25 mm should not be exceeded in order to prevent/minimize damage to the hammer and the pile.

All pile installation/driving should be in accordance with SP 903S01. The piles should be provided with rock points, Titus Injector or equivalent, for adequate seating on the sloping bedrock surface. A NSSP should be included in the Contract Documents to address the requirements for rock points (see example in Appendix C).

### **5.3.5 Pile Driving Note**

The pile driving note to be added to the drawings is Note 4 in Clause 2.5.11 of the Structural Manual – “Piles to be driven to bedrock”.

### **5.3.6 Resistance to Lateral Loads**

The design of piles subjected to lateral loads should take into account such factors as the batter of the piles (if any), the relative rigidity of the pile to the surrounding soil, the fixity condition at the head of the pile (pile cap level), the structural capacity of the pile to withstand bending moments, the soil resistance that can be mobilized, the tolerable lateral deflections at the head of the pile and pile group effects. For a longer, more flexible pile, the maximum yield moment of the pile may be reached prior to mobilization of the lateral geotechnical resistance. For design purposes, both the structural and geotechnical resistances should be evaluated to establish the governing case.

It is understood that an integral abutment foundation is being considered for the bridge. Where stiff or dense soils are present near the level of the pile cap, the integral abutment design typically consists of surrounding the upper portion of each H-pile with either a double corrugated steel pipe (CSP) liner (with the annulus between the two CSPs unfilled) or a single CSP liner with the space between the pile and the liner filled with uniform grained, uncompacted sand. In either case, this design allows the upper portion of the H-pile to flex more freely. With this design, the passive lateral resistance over the length of the CSP liner may be neglected. However, at sites where the soil at and below the pile cap level is softer or in a looser state, the CSP liner system may not be required because the low lateral resistance of the soil may provide adequate freedom of movement in the system.

Based on the information provided on the General Arrangement (GA) Drawing provided by URS on November 11, 2005 and the depth to bedrock encountered in boreholes BCB-3 and BCB-5, the length of the H-piles at the borehole locations will be about 10.4 m for the south abutment and 6.5 m for the north abutment. As noted above, due to the variability of the bedrock surface in this area, there should be a provision made in the Contract for dealing with varying pile lengths.

For the relatively long HP 310 x 110 piles driven to bedrock through the stiff to soft clays and very loose to compact sandy silts and sands at the abutments, the horizontal resistance at Ultimate Limit States (ULS) will be controlled by structural limitations such as the yield moment ( $M_{YIELD}$ ) of the pile. In this case, as described in the Canadian Foundation Engineering Manual (1992), the lateral loading will create bending moments in the pile and generate excessive bending stresses in the pile material.

At Serviceability Limit States (SLS), the horizontal resistance of the piles will be controlled by deflections of the pile heads being too large to be compatible with the superstructure. In this case, the horizontal resistance of the pile is calculated based on the coefficient of horizontal subgrade reaction ( $k_h$ ) of the soil.

The horizontal soil reaction to a vertical pile can be estimated using the following formulae depending on the soil type supporting the pile:

For cohesive soils:

$$k_h = \frac{67s_u}{B} \quad \text{where} \quad s_u \text{ is the undrained shear strength of the soil, as given below; and } B \text{ is the pile diameter (m).}$$

For cohesionless soils:

$$k_h = \frac{n_h z}{B} \quad \text{where} \quad n_h \text{ is the constant of horizontal subgrade reaction, as given below; } z \text{ is the depth (m); and } B \text{ is the pile diameter (m).}$$

The following ranges for the values of  $s_u$  and  $n_h$  may be assumed in the structural analysis. The range in values reflects the variability in the subsurface conditions at the abutment locations:

#### SOUTH ABUTMENT

<i>Soil Unit</i>	<i>Elevation (m)</i>	<i><math>s_u</math> (kPa)</i>	<i><math>n_h</math> (MPa/m)</i>
Firm to Stiff Silty Clay	196.2 – 195.0	75	-
Soft Silty Clay	195.0 – 193.5	25	-
Very loose to loose Sandy Silt	193.5 – 191.7	-	1.3
Very loose to compact Sand	191.7 – 185.4	-	2.8

**NORTH ABUTMENT**

<i>Soil Unit</i>	<i>Elevation (m)</i>	<i>s<sub>u</sub> (kPa)</i>	<i>n<sub>h</sub> (MPa/m)</i>
Firm Silty Clay to Clay	196.8 – 195.0	75	-
Soft Silty Clay to Clay	195.0 – 193.1	25	-
Loose Silty Sand to Sand	193.1 – 189.5	-	1.3

For a single HP 310 x 110 pile embedded about 10.4 m (south abutment) or 6.5 m (north abutment) into the soft to stiff silty clay to clay and very loose to compact sandy silt to sand, the estimated factored lateral resistances at ULS and at SLS (for 10 mm and 20 mm of horizontal deflection at the pile cap) are presented in the following table. These values have been estimated based on the solution proposed by Broms (1964) and based on analyses carried out using the commercially available program LPILE Plus (Version 5.0), produced by EnSoft Inc.

<i>Abutment Location (Pile Length)</i>	<i>Factored Lateral Resistance (kN)</i>		
	<i>ULS</i>	<i>SLS (10 mm of deflection)</i>	<i>SLS (20 mm of deflection)</i>
South (10.4 m long pile)	140	50	75
North (6.5 m long pile)	140	50	75

It should be noted that the above values are based on the assumption that the pile cap is located at Elevation 195.8 m and 196.0 m at the south and north abutment, respectively. In addition, the analysis carried out in LPILE assumed a free-headed pile and that the lateral loading was applied to the weak axis of the pile. The factored lateral resistances at ULS and SLS for the south and north abutment piles are similar given the similar soils conditions adjacent to the upper 5 m section of the piles.

If CSPs are installed as part of the integral abutment design, the ULS and SLS values for the lateral pile resistance will have to be re-evaluated.

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

<i>Pile Spacing in Direction of Loading <math>d = \text{Pile Diameter}</math></i>	<i>Subgrade Reaction Reduction Factor (R)</i>
8d	1.00
6d	0.70
4d	0.40
3d	0.25

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

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

### **5.3.7 Frost Protection**

The pile caps should be provided with a minimum of 1.8 m of conventional soil cover for frost protection.

Where rock fill is employed as a cover material, the minimum cover thickness required will be approximately twice that of a conventional soil cover given the open nature of the rock fill structure.

Alternatively, rigid insulation could be used to reduce the required thickness of soil cover over the foundation units. For preliminary design, it can be assumed that 25 mm of rigid insulation is equivalent to 0.6 m of conventional soil cover. The insulation should be installed on the abutment stem extending down from ground surface to the top of the pile cap and then extend to a distance of 1.8 m beyond the perimeter of each foundation unit.

### **5.4 Site Coefficient**

For seismic design purposes, the Site Coefficient,  $S$ , for this site in accordance with Section 4.4.6 of the CHBDC may be taken as 1.5, consistent with Soil Profile Type III.

### **5.5 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.

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

- Select free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) Granular 'A' or Granular 'B' Type II but with less than 5 per cent passing the 200 sieve should be used as backfill behind the walls. This fill should be compacted in loose lifts not greater than 200 mm in thickness to 95 per cent of the material's Standard Proctor maximum dry density in accordance with OPSS 501. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3501.00 and 3504.00.
- 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 Northeastern Region Directive for backfill to structures adjacent to rock embankments, dated November 2002. Other aspects of rock backfill requirements should be in accordance with OPSD 3505.00.
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with *CHBDC* Section 6.9.3 and Figure 6.9.3. Compaction equipment should be used in accordance with OPSS 501.06. Other surcharge loadings should be accounted for in the design, as required.
- The granular fill may be placed either in a zone with width equal to at least 1.8 m behind the back of the wall stem (Case I in Figure C6.9.1(l) of the *Commentary to the CHBDC*) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case II in Figure C6.9.1(l) of the *Commentary to the CHBDC*).
- For Case I, the pressures are based on the proposed embankment fill materials and the following parameters (unfactored) may be used assuming the use of Select Subgrade Material (SSM) or rock fill:

	SSM	Rock Fill
Soil unit weight:	20 kN/m <sup>3</sup>	19 kN/m <sup>3</sup>
Coefficients of static lateral earth pressure:		
Active, $K_a$	0.35	0.24
At rest, $K_o$	0.50	0.38

- 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:

	<b>Granular 'A'</b>	<b>Granular 'B'</b>
Soil unit weight:	22 kN/m <sup>3</sup>	<b>Type II</b> 21 kN/m <sup>3</sup>
Coefficients of static lateral earth pressure:		
Active, $K_a$	0.27	0.31
At rest, $K_o$	0.43	0.47

If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design. The movement required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.9.1 and Table C6.9.1(a) of the *Commentary to the CHBDC*.

Restrained structures are typically concrete box culverts or rigid frame bridge structures where the rotational and/or horizontal movement is not sufficient to mobilize the active pressure condition. For this condition, an at-rest pressure plus any compaction surcharge should be included in the design of the structure.

Seismic (earthquake) loading must also be taken into account in the design in accordance with Section 4.6 of the CHBDC. In this regard, the following should be included in the assessment of lateral earth pressures:

- Seismic loading will result in increased lateral earth pressures acting on the abutment stem and retaining walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure. According to Table A3.1.7 of the CHBDC, this site is located in Seismic Zone 1. The site-specific zonal acceleration ratio for Parry Sound is 0.05. Based on experience, for the subsurface conditions at this site, a 50 per cent amplification of the ground motion may occur (i.e. Site Coefficient,  $S = 1.5$ ), resulting in an increase in the ground surface acceleration from 0.05g to 0.075g. The seismic lateral earth pressure coefficients given below have been derived based on a design acceleration ratio of  $A = 0.075$ .
- In accordance with Sections 4.6.4 and C.4.6.4 of the CHBDC and its *Commentary*, for structures which allow lateral yielding, the horizontal seismic coefficient,  $k_h$ , used in the calculation of the seismic active pressure coefficient, is taken as 0.5 times the zonal acceleration ratio (i.e.  $k_h = 0.04$ ). For structures that do not allow lateral yielding,  $k_h$  is taken as 1.5 times the zonal acceleration ratio (i.e.  $k_h = 0.11$ ). The seismic active earth pressure coefficient is also dependent on the vertical component of the earthquake acceleration,  $k_v$ . Three discrete values of vertical acceleration are typically selected for analysis, corresponding to  $k_v = +2/3 k_h$ ,  $k_v = 0$ , and  $k_v = -2/3 k_h$ .
- The following seismic active pressure coefficients ( $k_{AE}$ ) for the two cases (Case I and Case II) may be used in design; these coefficients reflect the maximum  $K_{AE}$  obtained using the  $k_h$

and three values of  $k_v$  as described above. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is flat.

#### SEISMIC ACTIVE PRESSURE COEFFICIENTS, $K_{AE}$

	Case I	Case II	
		Granular A	Granular B Type II
Yielding wall	0.34	0.27	0.30
Non-yielding wall	0.40	0.32	0.36

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

- The above  $K_{AE}$  values for yielding walls are applicable provided that the wall can move up to  $250A$  (mm), where  $A$  is the design zonal acceleration ratio of 0.075. This corresponds to displacements of up to 19 mm at this site.
- The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e. an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

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

- Where
- |           |   |
|-----------|---|
| $K$       | is either the static active earth pressure coefficient ( $K_a$ ) or the static at rest earth pressure coefficient ( $K_o$ );  |
| $K_{AE}$  | is the seismic active earth pressure coefficient;   |
| $\gamma'$ | is the effective unit weight of the soil ( $\text{kN/m}^3$ ) <ul style="list-style-type: none"> <li>• taken as soil unit weights given above for fill materials</li> <li>• taken as soil unit weights as provided in Section 5.6.1</li> </ul> |
| $d$       | is the depth below the top of the wall (m); and   |
| $H$       | is the height of the wall above the toe (m).  |

## **5.6 Approach Embankment Design**

The construction of the existing Highway 69 (Service Road) Replacement Structure over Blair (Sly) Creek will require placement of up to about 1.8 m of fill above the existing top of pavement grade. This grade raise will result in a final approach embankment height of up to about 4.3 m south of the creek and 3.8 m north of the creek relative to the inferred natural ground surface (i.e. below existing embankment fill). For a grade raise between 1.3 m and 2.0 m, based on the pavement design recommendations (Final Pavement Design Report – Service Road – Golder Associates, October 2005) the existing asphalt can be left in place and rock fill (or additional Granular B, Type II) can be used for a base below 150 mm Granular B, Type II material, 150 mm Granular A and 60 mm asphalt. In addition, a grade raise of up to about 1.8 m is also required along the Service Road between Stations 11+600 to 11+660 where the toe of the embankment will be adjacent to the realigned Blair (Sly) Creek.

Based on the investigated locations at this site, the existing approach embankments are founded on sandy silt to silty sand underlain by silty clay to clay over silt, sandy silt and/or sand over bedrock. The overburden thickness at the south approach ranges from about 11.3 m to 14.1 m (with the silty clay to clay up to about 3.1 m thick) while at the north approach the overburden thickness ranges from about 2.4 m to 8.0 m (with the silty clay to clay up to about 3.8 m thick).

The results of stability and settlement analysis for the new approach embankments and the results of the stability analysis for the embankment grade raise near the Blair (Sly) Creek realignment are presented in the following sections.

### **5.6.1 Stability – Approach Embankments**

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

Limit equilibrium slope stability analyses were performed using the commercially available program SLOPE/W (GeoStudio 2004, Version 6.16), produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis. For all analyses, the factor of safety of numerous potential failure surfaces were computed in order to establish the minimum factor of safety. The factor of safety is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. A target minimum factor of safety of 1.3 is normally used in the design of embankment slopes under static conditions. This factor of safety is considered adequate for the embankments at this site considering the design requirements and the field data available. The stability analyses were performed to check that the target minimum factor of safety was achieved for the proposed embankment heights and geometries.

As noted above, the subsoils encountered below the existing approach embankments are composed of a combination of cohesionless and cohesive soils. For the cohesionless layers, effective stress parameters were employed in the analysis assuming drained conditions and the shear strength parameters were estimated from empirical correlations using the results of the in situ Standard Penetration Tests (SPT). The correlations proposed by Peck et al. (1974), Schmertmann (1975) and US Navy (1971) were employed and the results were tempered by engineering judgment based on precedent experience in similar soils.

For the cohesive layers, total stress parameters were employed in the analysis. The total stress parameters (i.e. average mobilized undrained shear strength –  $s_u$ ) for the cohesive soils were assessed based on the results of the field vane tests and estimated from the correlations with the SPT results and other laboratory test data. Where appropriate, Bjerrum's correction factor (1973) was employed to estimate the average mobilized undrained shear strength from the results of the in situ field vane tests.

The analyses assume that the existing embankment fill (including asphalt), with the exception of the fill to be removed for the construction of the new abutments, will remain in place. The piezometric conditions required in the analyses were based on water levels measured in piezometers installed in boreholes BCB-1 and BCB-5 and the water level of Blair (Sly) Creek. In general, the groundwater level is located at about 3 m depth below the existing top of pavement grade (i.e. from the natural ground surface to about 1 m below).

The following table summarizes the simplified stratigraphy and the associated strengths and unit weights employed for the different soil types in the approach areas. For the purpose of analysis, only rock fill has been considered as the base for the grade raise of the approach embankments as indicated in the table below. Rock fill is assumed to have side slopes at 1.25H:1V. A discussion on different fill types is provided in Section 5.6.1.1.

**South Approach Embankment**

<i>Soil Type</i>	<i>Unit Weight (kN/m<sup>3</sup>)</i>	<i>Strength Parameters</i>
Rock Fill	19	$c' = 0 \text{ kPa}, \phi' = 38^\circ$
Very loose to compact Existing Sand and Gravel Fill	21	$c' = 0 \text{ kPa}, \phi' = 32^\circ$
Peat (floodplain)	10.5	$c' = 2 \text{ kPa}, \phi' = 27^\circ$
Loose Silty Sand to Sandy Silt	19	$c' = 0 \text{ kPa}, \phi' = 29^\circ$
Firm to soft Silty Clay to Clay (centreline)	17	$s_u = 50 \text{ kPa to } 20 \text{ kPa}$
Very soft Clayey Silt to Clay (west toe)	17	$s_u = 12 \text{ kPa}$
Very loose to loose Silt to Sandy Silt	19	$c' = 0 \text{ kPa}, \phi' = 29^\circ$
Loose to compact Sand	20	$c' = 0 \text{ kPa}, \phi' = 31^\circ$

**North Approach Embankment**

<i>Soil Type</i>	<i>Unit Weight (kN/m<sup>3</sup>)</i>	<i>Strength Parameters</i>
Rock Fill	19	$c' = 0 \text{ kPa}, \phi' = 38^\circ$
Very loose to compact Existing Sand and Gravel Fill	21	$c' = 0 \text{ kPa}, \phi' = 32^\circ$
Loose Sandy Silt	19	$c' = 0 \text{ kPa}, \phi' = 29^\circ$
Firm to soft Silty Clay to Clay (centreline)	17	$s_u = 50 \text{ kPa to } 20 \text{ kPa}$
Soft Silty Clay to Clay (toes)	17	$s_u = 20 \text{ kPa to } 15 \text{ kPa}$
Loose Silty Sand to Sand	20	$c' = 0 \text{ kPa}, \phi' = 31^\circ$

The results of the stability analyses for the two embankment fill options are summarized in the following table. At each area, the highest (i.e. most critical) embankment section has been analyzed. In addition, the stability of the front slopes of the embankments (i.e. in the direction towards Blair Creek) were also analyzed. The minimum factor of safety is based on a deep-seated, global trial failure surface that would impact the operation of the roadway.

<i>Approach Embankment</i>	<i>Location</i>	<i>Embankment Height at Critical Section (m)*</i>	<i>Rock Fill Option</i>	
			<i>Recommended Side Slope Profile</i>	<i>Minimum Factor of Safety</i>
South Approach	Right of Centreline	4.3	1.25H : 1V	1.1
	Left of Centreline	4.3	1.25H : 1V	$\geq 1.3$
	Front Slope	4.3	1.25H : 1V	$\geq 1.3$
North Approach	Right of Centreline	3.8	1.25H : 1V	$\geq 1.3$
	Left of Centreline	3.8	1.25H : 1V	$\geq 1.3$
	Front Slope	3.8	1.25H : 1V	$\geq 1.3$

\*Note : Embankment height relative to the inferred natural ground surface (i.e. below existing embankment fill).

Based on results of the stability analyses for the up to 3.8 m high embankment proposed north of Blair (Sly) Creek, no berms are required to achieve a Factor of Safety of 1.3.

For the up to 4.3 m high embankment proposed south of Blair (Sly) Creek, no berm is required to the left of centreline; however, right of centreline, where the creek is in close proximity to the proposed new embankment toe, a stability berm is required. To achieve a Factor of Safety of 1.3 it would be necessary to construct a 1.5 m high (not including removal of surficial organics/peat) by 8 m wide by about 50 m long (Station 11+750 to 11+800) rock fill berm at the right toe of the new embankment (Figure 2).

The incorporation of an additional 2 m wide bench (or berm) into the uniform side slope profile is not required at these sections of the proposed approach embankments because the embankments are less than 6 m high (refer to Section 5.6.1.1).

#### **5.6.1.1 Embankment Fill Types and Berm Requirements**

The different fill alternatives (i.e. earth fill and rock fill) provide relative advantages and disadvantages in terms of weight (i.e. driving force and applied load to founding subsoils), post-construction performance (i.e. settlement), construction cost and time, and ease of construction / availability.

##### **Earth Fill**

The main advantage of using earth fill (i.e. sand and gravel) is the ease of construction and the lack of post-construction settlements within the fill embankment itself. However, this option will require a larger volume of fill and wider right-of-way because the side slopes will be flatter than

rock fill slopes. For this project, acceptable earth fill is considered to be suitable locally available and/or imported, granular material.

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

### **Rock Fill**

The main advantage of using rock fill is the ability to achieve steeper embankment side slopes. This is useful in areas with limited right-of-ways. In addition, rock fill will likely be available from the rock cuts proposed for the new Highway 69 alignment, thus providing an advantage in cost. The disadvantage of using rock fill for the construction of high embankments is that some post-construction settlement of the embankment fill itself will occur within about the first year of construction.

For the rock fill option, the incorporation of 2 m wide berms (or successive benches) into the uniform side slope profile is required wherever the embankment will exceed a height of 6 m such that the uninterrupted rock fill slope never exceeds a height of 6 m (as per MTO Northeastern Region guidelines). We understand that the Northeastern Region requirements for berms have recently changed from 6 m to 10 m height. However, we have been instructed to maintain the original guidelines for this project.

#### **5.6.2 Stability – Service Road – 11+600 to 11+660**

Limit equilibrium slope stability analyses were also performed on sections of the proposed Service Road embankment between Station 11+600 to 11+660 to assess the stability of the new grade raise adjacent to the proposed Blair (Sly) Creek realignment. The parameter assessment and analyses were carried out in the same manner as that described for the approach embankments in Section 5.6.1.

The following table summarizes the simplified stratigraphy and the associated strengths and unit weights employed for the different soil types in this area. For the purpose of analysis, only rock fill has been considered as the base for the grade raise of the approach embankments as indicated in the table below. Rock fill is assumed to have side slopes at 1.25H:1V.

**Service Road – Left Toe – Station 11+600 to 11+660**

<i>Soil Type</i>	<i>Unit Weight (kN/m<sup>3</sup>)</i>	<i>Strength Parameters</i>
Rock Fill	19	$c' = 0 \text{ kPa}, \phi' = 38^\circ$
Very loose to compact Existing Sand and Gravel Fill	21	$c' = 0 \text{ kPa}, \phi' = 32^\circ$
Very loose to loose Silty Sand to Sandy Silt	19	$c' = 0 \text{ kPa}, \phi' = 29^\circ$
Firm to soft Silty Clay to Clay (beneath embankment)	17	$s_u = 50 \text{ kPa to } 20 \text{ kPa}$
		$s_u = 20 \text{ kPa}$
Soft Silty Clay to Clay (at embankment toe)	17	$s_u = 50 \text{ kPa to } 20 \text{ kPa}$
		$s_u = 20 \text{ kPa}$
Very loose to compact Silty Sand to Sand	19	$c' = 0 \text{ kPa}, \phi' = 29^\circ$

The results of the stability analyses indicates a Factor of Safety greater than 1.3 for the proposed embankment grade raise in this area adjacent to the Blair (Sly) Creek realignment (see Figure 3). The minimum factor of safety is based on a deep-seated, global trial failure surface that would impact the operation of the roadway. Based on results of the stability analyses for the up to 1.8 m high grade raise in this area, no stability berms are required to achieve the target factor of safety.

**5.6.3 Liquefaction Potential**

The liquefaction potential of the soils below the approach embankments under seismic loading has been considered using the empirical method outlined in Section C.4.6.2 of the *CHBDC* Commentary, which correlates the cyclic resistance ratio of the soils with their normalized penetration resistance and fines content. Based on this assessment, and assuming a ground surface acceleration of 0.075 g, a factor of safety of greater than 1.0 against liquefaction is obtained for magnitude 7.0 earthquake events under the approach embankments. Pseudo-static methods of embankment stability analysis indicate that a yield acceleration of approximately 0.10 g results in a factor of safety against side slope instability of 1.0. Based on this yield acceleration and the correlation proposed by Makdisi and Seed (1978), it is estimated that very little additional deformations (i.e. less than about 5 mm) of the embankment could result under the design earthquake event. Localized failures at the embankment toe, resulting in steepening of the embankment side slopes, could occur. Since deep-seated global instability is not anticipated under the design earthquake event, localized toe failures would be mainly a maintenance issue. This should be considered in the life-cycle costing when assessing the relative costs of the works. Alternatively, consideration could be given to sub-excavation and removal of these silty subsoils prior to construction of the approach embankments in order to eliminate the potential for

seismically induced instability at the embankment toes; however, as the existing embankment fill is to remain in place, this alternative would not be practical.

#### **5.6.4 Settlement**

Settlement of the approach embankments can be expected as a result of the loading from the new fills (over the existing embankment fill) on the compressible foundation soils at this site. In addition, depending on the type of fill materials employed in the construction, settlements may also occur due to compression of the embankment fill itself.

To estimate the magnitude of the expected settlements, analyses were carried out on the critical sections of the proposed approach embankments using the commercially available program UNISETTLE (Version 3.2). The rate of settlement of the cohesive foundation soils was assessed by spreadsheet calculations using Terzaghi's one-dimensional consolidation theory.

For these settlement analyses, the critical sections are assumed to correspond to those areas where the greatest amounts of fill (up to about 1.8 m) are required to reach the final new proposed embankment heights (i.e. approximately 4.3 m and 3.8 m relative to the inferred natural ground surface for the south and north approaches, respectively). The unit weights and slope profiles for the embankment fill described in Section 5.6.1 were employed in the analyses. The analyses performed assume that the existing embankment fill (including asphalt), with the exception of the fill to be removed for the construction of the new abutments, will remain in place and that rock fill will be used for the majority of the grade raise. The piezometric conditions required in the analyses were based on the groundwater levels noted during drilling and measured in the piezometer installations (BCB-1 and BCB-5). In general, the groundwater level is located at about 3 m depth below the existing top of pavement grade (about 1 m below the inferred natural ground surface).

As noted previously, the foundation soils at this site are composed of a combination of cohesionless (i.e. sands) and cohesive (i.e. clays) strata of varying thickness. The immediate compression of the very loose to compact sandy silt to silty sand and sand layers was assessed by estimating an elastic modulus of deformation based on the SPT 'N'-values and empirical correlations found in literature by Bowles (1984) and Kulhawy and Mayne (1990).

The consolidation settlement of the soft to firm silty clay to clay layers was assessed using the results of the in situ field vane and SPT tests to estimate the deformation parameters for these soils. In addition, the results of the laboratory index testing were also employed to estimate deformation parameters using empirical correlations proposed in literature by Terzaghi and Peck (1967), Kulhawy and Mayne (1990), Azzouz et al. (1976) and Britto and Gunn (1987). These estimated values were then checked using the results of laboratory consolidation tests performed on specimens of silty clay obtained from boreholes adjacent to this site.

The degree of over-consolidation in the cohesive strata, required in the analyses, was estimated from the results of the in situ field vane tests and the following correlations relating mobilized undrained shear strength to preconsolidation pressure:

$$s_{u(mob)} = 0.22\sigma_p' \quad (\text{after Mesri, 1975})$$

where:  $s_{u(mob)}$  = average mobilized undrained shear strength (kPa)  
 $\sigma_p'$  = preconsolidation pressure (kPa)

and

$$s_{u(mob)} = \mu s_{u(FV)} \quad (\text{after Bjerrum, 1973})$$

where :  $s_{u(mob)}$  = average mobilized undrained shear strength (kPa)  
 $s_{u(FV)}$  = undrained shear strength from field vane test (kPa)  
 $\mu$  = Bjerrum's correction factor based on Plasticity Index

The following sections summarize the simplified stratigraphy, unit weights and deformation parameters employed for the different soils types in the approach areas. In these sections, the maximum estimated settlement of the existing fill / foundation soils in these areas (due to the loading imposed by the grade raise of the new approach embankment fills) is presented and a discussion on the rate of settlement is included.

#### 5.6.4.1 Settlement of Foundation Soils (South Approach)

The following simplified stratigraphy and deformation parameters have been developed for and employed in the settlement analysis of the proposed 4.3 m high (relative to the inferred natural ground surface) embankment at the south approach.

<i>Soil</i>	<i>Thickness (m)</i>	<i>Unit Weight (kN/m<sup>3</sup>)</i>	<i>Estimated Deformation Properties</i>
Rock fill (1.5 m grade raise – incl. about 1.1 m rock fill)	1.1	19	Refer to Section 5.6.3.3
Existing Embankment Sand and Gravel Fill	2.9	21	E' = 15 MPa
Silty Sand to Sandy Silt	0.8	19	E' = 5 MPa
Silty Clay to Clay	3.1	17	(see below)
Sandy Silt	1.9	19	E' = 4 MPa
Sand	8.2	20	E' = 10 MPa

The following consolidation parameters were estimated for the silty clay to clay layer based on empirical correlations using the results of the in situ tests and laboratory index testing as

described previously. In addition, the parameters were compared with the results of laboratory consolidation tests performed on specimens of clay obtained to the northeast of the project site.

<i>Location</i>	<i>Elevation (m)</i>	$\sigma_{vo}'$ (kPa)	$\sigma_p'$ (kPa)	<i>OCR</i>	$e_o$	$C_r$	$C_c$	$c_v$ (cm <sup>2</sup> /s)
South Approach	196.8 to 195.8	75	300	4.0	1.2	0.055	0.55	1.6 x 10 <sup>-3</sup>
	195.8 to 193.5	90	100	1.1				

Note: values above are estimated at the middle of the silty clay to clay sub-layers.

Based on the results of the settlement analysis, the maximum total settlement of the foundation soils and existing embankment fill in the area of the south approach is estimated to be about 90 mm. This total settlement is estimated to be comprised of about 40 mm of immediate settlement due to compression of the cohesionless soil layers (including the existing embankment fill) and about 50 mm of time dependent settlement of the cohesive soil layers.

Assuming a coefficient of consolidation ( $c_v$ ) of about 1.6x10<sup>-3</sup> m<sup>2</sup>/s (based on empirical correlations with liquid limit using US Navy (1971) and on results of laboratory consolidation tests on samples from a site located about 150 m to the northeast) and assuming two-way drainage of the approximately 3.1 m thick silty clay to clay layer, it is estimated that the about 90 percent of the consolidation settlement will be completed in about 5 months.

The magnitude of creep settlement for the silty clay to clay strata is expected to be less than 10 mm per log-cycle of time at this site.

#### **5.6.4.2 Settlement of Foundation Soils (North Approach)**

The following simplified stratigraphy and deformation parameters have been developed for and employed in the settlement analysis of proposed 3.8 m high (relative to the inferred natural ground surface) embankment at the north approach.

<i>Soil</i>	<i>Thickness (m)</i>	<i>Unit Weight (kN/m<sup>3</sup>)</i>	<i>Estimated Deformation Properties</i>
Rock fill (1.8 m grade raise – incl. about 1.4 m rock fill)	1.4	19	Refer to Section 5.6.3.3
Existing Embankment Sand and Gravel Fill	2.1	21	E' = 8 MPa
Sandy Silt	0.7	19	E' = 7 MPa
Silty Clay to Clay	3.8	17	(see below)
Silty Sand to Sand	3.5	20	E' = 6 MPa

The following consolidation parameters were estimated for the silty clay to clay layer based on empirical correlations using the results of the in situ tests and laboratory index testing as described previously. In addition, the parameters were compared with the results of laboratory consolidation tests performed on specimens of clay obtained to the northeast of the project site.

<i>Location</i>	<i>Elevation (m)</i>	<i><math>\sigma_{vo}'</math> (kPa)</i>	<i><math>\sigma_p'</math> (kPa)</i>	<i>OCR</i>	<i><math>e_o</math></i>	<i><math>C_r</math></i>	<i><math>C_c</math></i>	<i><math>c_v</math> (cm<sup>2</sup>/s)</i>
North Approach	196.9 to 195.9	65	275	4.2	1.2	0.055	0.55	1.6 x 10 <sup>-3</sup>
	195.9 to 193.1	80	90	1.1				

Note: values above are estimated at the middle of the silty clay to clay sub-layers.

Based on the results of the settlement analysis, the maximum total settlement of the foundation soils and existing embankment fill in the area of the north approach is estimated to be about 110 mm. This total settlement is estimated to be comprised of about 30 mm of immediate settlement due to compression of the cohesionless soil layers (including the existing embankment fill) and about 80 mm of time dependent settlement of the cohesive soil layers.

Assuming a coefficient of consolidation ( $c_v$ ) of about 1.6x10<sup>-3</sup> cm<sup>2</sup>/s (based on empirical correlations with liquid limit using US Navy (1971) and on results of laboratory consolidation tests on samples from a site located about 150 m to the northeast) and assuming two-way drainage of the approximately 3.8 m thick silty clay to clay layer, it is estimated that the about 90 percent of the consolidation settlement will be completed in about 7 months.

The magnitude of creep settlement for the silty clay to clay strata is expected to be about 10 mm per log-cycle of time at this site.

### 5.6.4.3 Settlement of Rock Fill

If rock fill is used for the grade raise of the embankments, in addition to the settlement due to compression of the foundation soils described above, there will be settlement due to compression of the rock fill itself. Settlement of the rock fill depends on the type of rock and on the method and sequence of placement and compaction of the fill. Assuming that the rock fill is not end dumped in its final position and is placed in accordance with the requirements as outlined in Special Provision SP 206S03 dated January 2004, the settlement of the newly placed rock fill is expected to be small. In general, it is estimated that for the granitic gneiss rock fill likely to be used at this site, the settlement of the rock fill will be about 0.5% of the thickness of rock fill.

<i>Location of Embankment</i>	<i>Approximate Chainage</i>	<i>Maximum New Embankment Height (m)</i>	<i>Rock Fill Component of Grade Raise (m)</i>	<i>Estimated Settlement of Rock Fill (mm)</i>
South Approach	11+751 to 11+771	4.3 (up to 1.8 m grade raise)	1.1	<10
North Approach	11+692 to 11+712	3.8 (up to 1.8 m grade raise)	1.4	<10

It is anticipated that the majority (approximately 60%) of this settlement will occur in the first year following construction.

## 5.7 Mitigation of Stability Issues / Time Dependent Settlements

As discussed in Section 5.6.1, the presence of the very soft to firm clayey subsoils and the location of the proposed south approach embankment relative to Blair (Sly) Creek create stability problems for the grade raise at the south approach. In addition, as discussed in Section 5.6.3, the soft and compressible subsoils will result in some time dependent settlements at the new south and north approach embankments. In the area of the south and north approach, consideration needs to be given to adopting a design and/or following a construction sequence to achieve a minimum Factor of Safety of 1.3 for the grade raises and to limit the post-construction settlements and subsequent maintenance of the new roadway pavement structure.

For the south and north approach areas (i.e. within 20 m north and 50 m south of the bridge abutments for this report), the following sections outline the options and recommendations for achieving the target factor of safety for the required embankment geometries and for minimizing the time dependent, post-construction settlements that could effect the performance of the roadway. The advantages, disadvantages, relative costs and risks/consequences for the mitigation options at the south and north approach areas are summarized and ranked in Table 3 and 4,

respectively. At this site, preloading with toe berms (where required) is considered to be the most practical and preferred settlement and stability mitigation option.

### **5.7.1 Preloading and Toe Berm**

For the up to 3.1 m thick clayey strata located in the footprint of the south approach and the up to 3.8 m thick clayey strata located in the footprint of the north approach it is estimated that 90 percent of the post-construction foundation soil settlements will be completed in about 5 months and 7 months, respectively from the time of completion of construction. Given that the existing embankments are to remain in place for the grade raise, if the construction schedule can accommodate these periods, pre-loading the foundation soils by building the embankment as early as possible will provide the most practical solution in terms of the stability and long-term performance of the roadway. A small surcharge on the north approach embankment could also be considered (as discussed in the following section) to accelerate the rate of consolidation such that both the north and south approaches would approach the end of consolidation at about the same time.

For this alternative, based on our stability analyses, a toe berm approximately 1.5 m high (not including removal of surficial organics and peat) and 8 m wide will be required on the west side of the south approach between about Stations 11+750 and 11+800 in order to achieve a target Factor of Safety of 1.3. If this size of berm cannot be constructed due to environmental restrictions (i.e. encroaches into Blair Creek), consideration could be given to reducing the size of the berm required by sub-excavating the very soft clayey subsoils located in the floodplain to the west of the south approach embankment. However, temporary support systems would have to be installed near the edge of the creek (to prevent the creek from being under-mined) and at the toe of the existing embankment (to protect the existing embankment and roadway). Also, the limited space between the creek and existing embankment would make the sub-excavation difficult.

It should be noted that total immediate and consolidation settlements of all subsoils and embankment fills is expected to be on the order of 85 mm for the south approach and 110 mm for the north approach. As such, the embankments will need to be over-built by this amount in order to allow for such settlements during the preload period.

Some small amount of additional long-term settlements due to secondary consolidation (i.e. creep) of the clayey strata (about 10 mm per log-cycle of time) should be expected with this option.

### **5.7.2      Surcharging and Toe Berms**

If the construction schedule cannot accommodate the 5 months (south approach) and 7 months (north approach) required for primary consolidation settlement under the preloading option, consideration could be given to reducing these times through the use of a surcharge.

The addition of a 0.5 m high surcharge on the north approach embankment would accelerate the consolidation such that approximately 90% of the primary consolidation (of the required grade raise) would be achieved in about 5 months (i.e. similar to that required under the preloading option for the south approach). However, this option will require a 1.5 m wide mid-height toe berm be added to the front and side slopes of the north approach embankment to maintain a Factor of Safety of 1.3.

Consideration could be given to employing higher surcharges on both approach embankments to further accelerate the rate of consolidation. For example, under the influence of a 1 m high surcharge, it is estimated that 90 % consolidation (of the required new grade raise) would be achieved in approximately 2 months at the south approach and 3 months at the north approach. However, this option would require much larger toe berms on both the north and south approach embankments to maintain stability. Note that, the size of the toe berm required to maintain stability on the west side of the south approach embankment under surcharge loading would likely encroach into the adjacent Blair (Sly) Creek. As such, this option may not be feasible due to environmental restrictions.

Some additional long-term settlements due to secondary consolidation (i.e. creep) of the clayey strata (about 10 mm per log-cycle of time) should be expected with this option.

### **5.7.3      Light Weight (EPS) Fill**

In order to reduce the loads imposed by the up to 1.8 m grade raise for the approach embankments over the soft and compressible foundation soils in this area, the use of light weight (i.e. expanded polystyrene (EPS)) fill could be considered. The use of this material for the new embankment fill would eliminate the need for a stabilizing toe berm on the west side of the south approach and would result in very little time-dependent (consolidation) settlement of the foundation soils.

It is estimated that up to 1.8 m of EPS fill would be required between about Stations 11+750 and 11+800 (to mitigate stability and consolidation settlements) of the south approach and Stations 11+590 and 11+710 of the north approach and beyond (to mitigate consolidation settlements). North (down-chainage) of about Station 11+690, further investigation may be required as discussed previously. For installation, a minimum conventional fill/pavement structure cover of

at least 1 m is required over the EPS. Therefore, to accommodate this, the existing embankment fill would have to be partially sub-excavated by about 1 m prior to installation of the EPS fill and completion of the new pavement structure.

#### **5.7.4 Full Sub-excavation**

It is understood that the existing approach embankments are to remain in place; however, consideration could be given to sub-excavating and removing the underlying clayey subsoils, along with the existing embankments, to negate the need for a toe berm on the west side of the south approach embankment and to help minimize settlement of both the south and north approaches.

The bottom of the clayey strata is located at a depth of up to about 4.0 m at the south approach and up to about 4.4 m at the north approach, below the original/natural ground surface (i.e. between about 6 m and 7 m below existing top of embankment fill). If the existing approach embankments can be removed, sub-excavation and removal of the clayey strata to these depths is considered feasible; however, the sub-excavation could undermine Blair (Sly) Creek which we understand is considered an environmentally sensitive area. Therefore, to minimize the impact of the excavation on the creek, a sheet pile support system could be installed at the excavation limit to allow the work to be carried out while minimizing the encroachment on the creek. Recommendations with respect to sheet pile support are given in Section 5.9. In addition, since the groundwater table is located at about the level of the ground surface, the sub-excavation would likely have to be carried out 'in-the-wet' (i.e. below the water level). This approach is recommended since the cost of de-watering could be significant. Also, excavation 'in-the-wet' would help to maintain steeper side slope stability and minimize the chance of base heave failure.

Assuming that the water table is maintained at the ground surface (and the work is carried out 'in-the-wet'), an unsupported side slope profile of about 1H:1V is recommended to maintain the stability of the works during excavation. Where required, a steeper side slope profile could be utilized if the excavation is carried out in stages or strips with limited width as discussed in Section 5.9. This methodology for excavation in strips (and 'in-the-wet') should also be adopted in areas adjacent to the sheet pile support system near the edges of Blair (Sly) Creek. If this approach is not employed (and the width of excavation is not limited), the sheet pile wall may have to be supported by tie-backs (i.e. with anchors installed within the very loose to compact sand deposit at depth or within the bedrock) to maintain stability of the wall and the adjacent creek.

Adopting this alternative will result in increasing the effective thickness of the new embankment fill by up to about 6.1 m at the south approach and 6.6 m at the north approach because of the additional fill required to replace the existing embankment and the fill required below the natural

ground surface. The fill below the natural ground surface should be constructed with the same side slope profile as that used for the above grade embankment. The increase in fill height will result in additional (i.e. up to about 60 mm to 65 mm) post-construction settlement of the embankment rock fill.

### **5.7.5 Wick Drains**

Due to the limited thickness of the clayey subsoils below the existing approach embankments (i.e. on the order of about 2 m to 4 m thick), the use of wick drains to reduce the amount of time required for primary consolidation is not considered to be practical at this location. In addition, the presence of the existing embankment fill could also make the installation of wick drains problematic. As such, wick drains are not recommended as a mitigation option for this site.

## **5.8 Subgrade Preparation and Embankment Construction**

The existing embankment fill and the underlying native subsoils are considered to be appropriate subbase for the proposed grade raise; however, prior to the placement of any fill, all surface and near surface layers of topsoil/organic/peat deposits and any softened or loosened soils should be stripped from the plan limits of the proposed works (i.e. in the areas adjacent to the existing embankment toes, as required) and the subgrade soils should be proof-rolled. For filling on the existing roadway and side slopes, the geotechnical/pavement recommendations outlined in the Final Pavement Design Report – Service Road – by Golder Associates, dated October 2005 should be followed.

Table 5 summarizes the recommended fill type to be placed for the grade raise, the location and depth of organics, the recommended side slope profiles, the requirements for side berms, the anticipated differential settlements, platform widenings (in accordance with NRE 98-200) and the recommended method of removal of organics. The following sections provide details on the recommendations for subgrade preparation and embankment construction.

It should be noted that if sub-excavation and removal of the soft clayey strata at depth is adopted as the method to mitigate the time-dependent settlements at the approach embankments, it will be necessary to backfill the excavation in the area of the abutment foundations with a granular fill with less than 75 mm particles sizes (not rock fill) so that pile driving to bedrock can be carried out without encountering obstructions.

### **5.8.1 Removal of Organics**

Prior to the placement of any fill for the grade raise, all surficial and near surface layers of organic deposits (i.e. vegetation/topsoil/peat) should be stripped from the plan limits of the

proposed works that include the existing embankment side slopes and the areas adjacent to the existing embankment toes. It is anticipated that topsoil up to about 0.2 m to 0.3 m thick can be expected in most areas of the widened approach embankment footprints. On the west side of the south approach, peat about 0.8 m thick can be expected as evidenced in borehole BCB-2.

Where new fill is required to be placed immediately on top of the existing Highway 69 roadway embankments, construction procedures should implement the guidelines of OPSD 203.020. These guidelines require that the slopes of the existing embankment be temporarily excavated to a 1H:1V profile to allow for removal of a larger extent of organic material. However, in all cases, during excavation, measures must be adopted to ensure that the existing roadways are protected and that the stability of the existing embankments are maintained. A provision for traffic control measures should also be included in the Contract to maintain the safe operation of the Service Road/existing Highway 69 during any adjacent excavation works.

### **5.8.2 Embankment Fill Placement and Erosion Protection**

Since the existing embankments are composed of sand and gravel fill, benching into the existing side slopes should be carried out as per OPSD 208.010 during the construction of the grade raises/widenings. In addition, any loose or deleterious material should be removed from the toe and slopes prior to any new fill placement.

If earth fill (granular) is to be used for construction of the new embankments, placement of all granular fill material should be carried out in accordance with Special Provision 206S03 (January 2004) – Section 206.07.07, in regular lifts with loose thickness not exceeding 300 mm, and be compacted to at least 95 percent of the Standard Proctor maximum dry density. The final lift prior to placement of the granular sub-base or base course should be placed and compacted to current MTO requirements for pavements. Inspection and field density testing should be carried out by qualified geotechnical personnel during all earth fill placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved. Side slopes for earth fill embankments should be no steeper than 2H:1V.

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

If rock fill is used for the construction of the new embankments, placement of all rock fill material should be carried out in accordance with Special Provision 206S03 (January 2004) – Section 206.07.08. The rock should not be dumped in final position, but should be deposited on and pushed forward over the end of the layer being constructed. Voids and bridging shall be minimized by blading, dozing and ‘chinking’ the rock to form a dense, compact mass. Side slopes for rock fill embankments should be no steeper than 1.25H:1V.

Erosion protection (i.e. rip-rap) of a suitable size and thickness should be placed (as required) on the front and adjacent side slopes to protect the approach embankment and abutment foundations from undermining/erosion by the river water flow. As part of the rip-rap design and installation, provision should also be made to ensure measures are adopted to protect the loss of any fine materials from the underlying approach embankment fill (or retaining wall fill) through the erosion protection.

## **5.9 Design and Construction Considerations**

### **5.9.1 Excavations**

As discussed in Section 5.1, excavation of the front slopes of the existing approach embankments (by up to 10 m long and 2 m deep at the south approach and up to 4 m long and 1.8 m deep at the north approach) will be carried out as part of the removal of the old bridge structure. Following removal of the front slopes of the existing approaches, excavations up to about 1.8 m deep will be required in the area of the new abutments to construct the pile caps. In addition, excavation within the plan limits of the new approach embankment widenings will be required to remove topsoil/organic peat deposits up to about 0.8 m deep within the flood plain adjacent to the west toe of the south approach and up to about 0.3 m deep elsewhere around the existing embankment toes, prior to fill placement.

The embankment fill soils are classified as Type 3 and the native silty clays and sandy silts are classified as Type 4 according to the *Occupational Health and Safety Act and Regulations for Construction Projects*.

At the south and north abutment, temporary excavation to a depth of about 1.8 m for pile installation and construction of the pile cap should be carried out with side slopes no steeper than 2H:1V to ensure stability of the excavation at the base of the existing approach embankment fills. Where excavation with 2H:1V side slopes cannot be carried out, a shoring protection system (per the recommendation in Section 5.9.1.1) should be utilized especially for excavations adjacent to the existing roadway. Assuming the excavations are carried out 'in-the-dry' to a maximum depth of 1.8 m and the groundwater conditions at this area are as described in Section 4.2.8, it is calculated that the Factor of Safety against base heave is approximately 1.2 at the south abutment and about 1.3 at the north abutment. Although a Factor of Safety of 1.3 is desirable for one-dimensional conditions, a slightly lower factor of safety is acceptable here due to the three-dimensional/limited excavation extents and considering the temporary nature of the works.

As noted in Section 5.7, full sub-excavation and removal of the clayey strata could be considered as a settlement and stability mitigation measure for the approach embankments at this site. If this alternative is adopted, the excavations would extend up to about 4.5 m deep (below the

original/natural ground surface) and up to about 7 m deep below the existing top of embankments. In the vicinity of the approaches away from the creek, where space permits, temporary excavations (i.e. those that are open only for a relatively short period) to a depth of up to about 7 m for removal of the clayey strata, should be carried out with side slopes of about 1H:1V assuming that the work is done 'in-the-wet'. If steeper side slopes are required (i.e. in areas immediately adjacent to the creek), the recommendations for temporary shoring or staged excavation described in the sections below could be considered. In addition, to maintain stability of the existing roadway embankment, a minimum offset of 8 m should be maintained at all times between the crest of the excavation and the toe of the existing embankment.

Groundwater inflows will have to be controlled as discussed in Section 5.9.2. Alternatively, a temporary sheet pile shoring system could be installed to limit the extent of the excavation and cut-off or reduce groundwater inflows.

Conventional excavation equipment should be suitable for the excavation through the on-site soils and embankment fills.

All excavations must be carried out in accordance with the latest edition of the Ontario Occupational Health and Safety Act and Regulations for Construction Projects.

#### **5.9.1.1 Temporary (or Permanent) Shoring**

Where space and/or high groundwater levels restrict the use of open cuts and where excavations immediately adjacent to the existing roadway are required, a temporary support system should be constructed to support excavations. As discussed above, the use of temporary shoring could also be considered for construction of the abutment foundations and for excavations immediately adjacent to Blair (Sly) Creek. The temporary excavation support system should be in accordance with Special Provision 105S19 (dated June 2006) and designed to Performance Level 2. Roadway protection should also be in accordance with Special Provision 105S19.

If temporary shoring is employed for pile installation and pile cap construction at the abutments, groundwater inflows may occur (especially at the south abutment) considering that up to about 0.5 m of sandy silt exists at the ground surface/base of existing embankment and below the high water level in Blair (Sly) Creek. Ground water inflow into the shored excavation should be controlled by dewatering (as discussed in Section 5.9.2) so that the pile cap construction can be carried out in the dry.

#### **5.9.1.2 Staged Excavation**

At the approaches, if sub-excavation and removal is adopted as the settlement/stability mitigation measure at this site, and if excavation 'in-the-wet' for removal of the clayey strata is to be carried

out with side slopes steeper than 1H:1V or adjacent to a sheet pile shoring system that does not employ a tie-back anchor system to maintain support, it is recommended that the work be done in stages or strips with limited width. The recommendations for staged excavation are as follows:

- Removal of the organics and clayey strata should be carried out in short sections perpendicular to the critical areas, sheet piling and/or creek alignment;
- Excavation and backfilling operations should be carried out simultaneously in a manner that the excavation is not left open for more than 2 m in width (at the base of the excavation) at any given time.

### **5.9.2 Groundwater and Surface Water Control**


In the area of the abutments and approach embankments, the groundwater level is generally within about 1 m of the natural ground surface (i.e. below existing embankment fill). In the area adjacent to the existing embankments (i.e. within the footprint of the widenings located in the floodplain of Blair (Sly) Creek), the groundwater level is generally at the ground surface. If sub-excavation is adopted as the settlement and stability mitigation measure at this location, groundwater flow into the approximately 4.5 m to 7 m deep excavations can be expected to occur due to the high groundwater levels and permeable nature of the sand strata underlying the clayey subsoils. In addition, it is recommended that the excavation work be carried out 'in-the-wet' to maintain side slope and basal stability in the excavation. In this case, groundwater control will not be required.

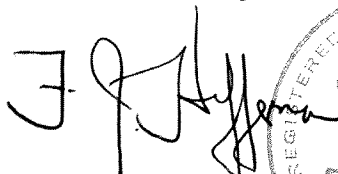
Where excavation is only required in the immediate vicinity of the south and north abutment foundations (i.e. for construction of the new pile caps), the approximately 1.8 m deep excavations may require groundwater control (depending on the water level in Blair (Sly) Creek at the time of excavation) in addition to directing surface water away from the excavation at all times. Based on grain size distributions for the silt and sand strata from boreholes adjacent to this site (i.e. at the Highway 69 NBL and SBL/Blair (Sly) Creek Structure locations), and considering the limits of dewatering proposed by Powers (1992), it is considered that the volume of seepage through the near surface sandy silt layer may be low enough that adequate groundwater control could be attained through the use of pumping from properly filtered sumps in the excavation. In all cases, surface water should be directed away from the excavations at all times.

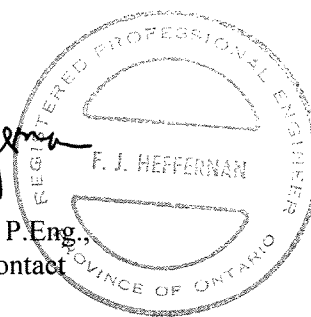
**5.10 CLOSURE**


This Foundation Design Report was prepared by Mr. Chad Gilfillan and reviewed by Dr. J. Paul Dittrich, Ph.D., P.Eng., an Associate with Golder. Mr. Fintan Heffernan, P.Eng., Golder's Designated MTO Contact for this project, conducted an independent quality review of the report.

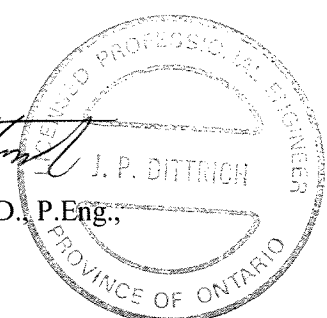
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## LIST OF ABBREVIATIONS

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

### 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

### 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.)

#### 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

#### Dynamic Cone Penetration Resistance; $N_d$ :

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

**PH:** Sampler advanced by hydraulic pressure

**PM:** Sampler advanced by manual pressure

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

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

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

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

### 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 <sup>1</sup>
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement <sup>1</sup>
$D_R$	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 <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.

## LIST OF SYMBOLS

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

### I. General

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

### II. STRESS AND STRAIN

$\gamma$	shear strain
$\Delta$	change in, e.g. in 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

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

# LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

## WEATHERING STATE

**Fresh:** no visible sign of weathering.

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

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

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

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

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

## BEDDING THICKNESS

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

## JOINT OR FOLIATION SPACING

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

## GRAIN SIZE

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

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

## CORE CONDITION

### Total Core Recovery

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

### Solid Core Recovery (SCR)

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

### Rock Quality Designation (RQD)

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

## DISCONTINUITY DATA

### Fracture Index

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

### Dip with Respect to (W.R.T.) Core Axis

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

### Description and Notes

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

### Abbreviations



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

## **RECORD OF BOREHOLE SHEETS**

PROJECT 03-1111-028			RECORD OF BOREHOLE No BCB-1			1 OF 2 METRIC			
W.P. 335-00-00			LOCATION N 5032671.2 ; E 256659.6			ORIGINATED BY EHS			
DIST 52 HWY 69			BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger			COMPILED BY KG			
DATUM Geodetic			DATE October 28, 2004			CHECKED BY CG			
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100 PLASTIC LIMIT W <sub>p</sub> NATURAL MOISTURE CONTENT W LIQUID LIMIT W <sub>L</sub> WATER CONTENT (%) 20 40 60 UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
199.4	GROUND SURFACE								
0.0	Sand, trace to some gravel, trace silt (FILL) Compact Brown and oxidized Moist		1	SS	14		199		
			2	SS	16		198		
197.7									
1.8	Silty Sand, trace organics Compact to loose Grey, oxidized Wet		3	SS	22		197		
196.8			4	SS	6		196		
2.6	Clay, trace sand and organics Stiff to soft Mottled reddish brown and grey Wet						195		
195.4			5	SS	WH		194		
4.0	Silty Clay, trace sand, occasional sand seams Soft to firm Brownish grey Wet						193		
193.7			6	SS	WH		192		
5.7	Sandy Silt, trace clay Very loose Grey Wet		7	SS	WR		191		
191.8							190		
7.6	Sand, trace gravel and silt Loose to compact Grey Wet		8	SS	15		189		
			9	SS	4		188		
							187		
			10	SS	6		186		
							185		
			11	SS	11				
			12	SS	11				

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

PROJECT <u>03-1111-028</u>		<b>RECORD OF BOREHOLE No BCB-1</b>				2 OF 2 <b>METRIC</b>												
W.P. <u>335-00-00</u>		LOCATION <u>N 5032671.2 ; E 256659.6</u>				ORIGINATED BY <u>EHS</u>												
DIST <u>52</u> HWY <u>69</u>		BOREHOLE TYPE <u>Power Auger 108 mm I.D. Hollow Stem Auger</u>				COMPILED BY <u>KG</u>												
DATUM <u>Geodetic</u>		DATE <u>October 28, 2004</u>				CHECKED BY <u>CG</u>												
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  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										
	--- CONTINUED FROM PREVIOUS PAGE ---						20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					20 40 60 WATER CONTENT (%)						
183.6 15.8	Sand, trace gravel and silt Loose to compact Grey Wet Becoming Silty Sand at 15.0 m depth End of Borehole Auger refusal  Note: 1. Borehole caved from about 8.5 m to 13.4 m depth during piezometer installation. 2. Water level in piezometer at 2.9 m depth on November 14, 2004.		13	SS	13		184											


PROJECT 03-1111-028			RECORD OF BOREHOLE No BCB-2			1 OF 1 METRIC											
W.P. 335-00-00			LOCATION N 5032649.7 ; E 256645.6			ORIGINATED BY CMG											
DIST 52 HWY 69			BOREHOLE TYPE Portable Drilling Equipment			COMPILED BY DD											
DATUM Geodetic			DATE January 12, 2006			CHECKED BY JPD											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ	GR SA SI CL
								20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100			
196.3	GROUND SURFACE																
0.0	Fibrous Peat, some wood fragments, trace to some sand																
195.5	Soft Black wet																
0.8	Sandy Silt, some clay, trace organics		1	SS	2												
	Very loose																
	Grey		2	SS	2												
	Wet																
194.0	Clayey Silt to Silty Clay, trace to some sand		3	SS	WH												
2.3	Very soft to soft																
	Grey																
	Wet																
192.3	Silt to Sandy Silt, some clay, occasional silt layers		4	SS	WH												
4.0	Very loose																
	Grey																
	Wet																
191.6	Sand, trace silt, trace gravel		5	SS	14												
191.3	Compact																
5.0	Grey																
	Wet																
	End of Borehole																
	Note:																
	1. Water level in open borehole at ground surface on completion of drilling.																

PROJECT 03-1111-028			RECORD OF BOREHOLE No BCB-3			1 OF 2 METRIC		
W.P. 335-00-00			LOCATION N 5032679.6 ; E 256647.3			ORIGINATED BY EHS/CG		
DIST 52 HWY 69			BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger			COMPILED BY KG		
DATUM Geodetic			DATE October 28 and 29, 2004			CHECKED BY CG		
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100 20 40 60 80 100 PLASTIC LIMIT W <sub>p</sub> NATURAL MOISTURE CONTENT W LIQUID LIMIT W <sub>L</sub> WATER CONTENT (%) 20 40 60
199.6	GROUND SURFACE							
0.0	Sand, trace gravel, silt and asphalt (FILL) Very loose to compact Light brown and oxidized Moist		1	SS	12		199	
			2	SS	2			
			3	SS	10		198	
	Some organics at fill/native ground interface		4	SS	18		197	
196.7								
2.9	Sandy Silt, some organics (topsoil, wood fragments)							
196.2	Loose		5	SS	5		196	
3.4	Dark brown to grey Moist to wet Silty Clay, trace sand Stiff to soft Mottled reddish brown to grey Moist to wet						195	
			6	SS	WH			
							194	
193.5								
6.1	Sandy Silt, trace clay, occasional clay seams Very loose to loose Grey Wet		7	SS	PM		193	
							192	
191.7			8	SS	8		191	
7.9	Sand, trace gravel and silt Compact to very loose Grey Wet						190	
			9	SS	8			
							189	
			10	SS	14			
							188	
			11	SS	6		187	
							186	
185.4			12	SS	WR			
14.2	Bedrock						185	
	Refer to Record of Drillhole BCB-3 for details							

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

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PROJECT <u>03-1111-028</u>		<b>RECORD OF BOREHOLE No BCB-3</b>				2 OF 2 <b>METRIC</b>											
W.P. <u>335-00-00</u>		LOCATION <u>N 5032679.6 ; E 256647.3</u>				ORIGINATED BY <u>EHS/CG</u>											
DIST <u>52</u> HWY <u>69</u>		BOREHOLE TYPE <u>Power Auger 108 mm I.D. Hollow Stem Auger</u>				COMPILED BY <u>KG</u>											
DATUM <u>Geodetic</u>		DATE <u>October 28 and 29, 2004</u>				CHECKED BY <u>CG</u>											
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  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									WATER CONTENT (%)
	--- CONTINUED FROM PREVIOUS PAGE ---						<div style="display: flex; justify-content: space-between; font-size: small;"> <span>20 40 60 80 100</span> <span>20 40 60 80 100</span> </div> <div style="display: flex; justify-content: space-between; font-size: x-small;"> <span>○ UNCONFINED    + FIELD VANE</span> <span>● QUICK TRIAXIAL    × REMOULDED</span> </div>										
	Bedrock																
	Refer to Record of Drillhole BCB-3 for details																
182.1																	
17.6	End of Borehole																
	Notes:																
	1. Spoon refusal at 14.2 m depth.																
	2. Water level in open borehole at 2.7 m depth upon completion of drilling.																

PROJECT: 03-1111-028

**RECORD OF DRILLHOLE: BCB-3**

SHEET 1 OF 1

LOCATION: N 5032679.6 ;E 256647.3

DRILLING DATE: November 1, 2004

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: TRACK CME 55

DRILLING CONTRACTOR: MARATHON DRILLING LTD.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH	COLOUR % RETURN	JN - Joint		BD - Bedding		PL - Planar		PO - Polished		BR - Broken Rock	NOTES WATER LEVELS INSTRUMENTATION		
									FLT - Fault	SHR - Shear	FO - Foliation	CU - Curved	K - Slickensided							
									VN - Vein	OR - Orthogonal	UN - Undulating	SM - Smooth								
									CJ - Conjugate	CL - Cleavage	ST - Stepped	Ro - Rough	MB - Mechanical Break							
RECOVERY										FRACT INDEX PER 0.3 m		DISCONTINUITY DATA				HYDRAULIC CONDUCTIVITY K, cm/sec		Diametral Point Load Index (MPa)	RMC -Q' AVG.	
TOTAL CORE %	SOLID CORE %	R.Q.D. %	B Angle	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION		10	10	10	10										
		- continued from Record of Borehole -		185.40																
15	NQ CORING	Granitic Gneiss Medium grained Fresh to slightly weathered Faintly porous Light grey to pink Distinct near horizontal foliation		14.20	1															
					2															
16		183.45		3																
				4																
17		Mafic-rich Gneiss Fine to medium grained Fresh Faintly porous Grey and black Distinct foliation (biotite/amphibole in particular)		16.15																
18		END OF DRILLHOLE		182.00														Refer to Record of Borehole BCB-3		
17.60																				
18																				
19																				
20																				
21																				
22																				
23																				
24																				

PROJECT 03-1111-028			RECORD OF BOREHOLE No BCB-5			1 OF 2 METRIC		
W.P. 335-00-00			LOCATION N 5032689.4 ; E 256607.2			ORIGINATED BY BL/CG		
DIST 52 HWY 69			BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger			COMPILED BY KG		
DATUM Geodetic			DATE November 1, 2004			CHECKED BY CG		
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100
199.6	GROUND SURFACE							PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W <sub>p</sub> W W <sub>L</sub> WATER CONTENT (%)
0.0	Sand, some gravel (FILL) Very loose to loose Brown Moist		1	SS	2		199	
197.5			2	SS	5		198	
2.1	Sandy Silt, trace to some clay Loose Grey Moist		3	SS	9		197	
196.8			4	SS	7		196	
2.8	Silty Clay to Clay, trace sand Stiff to soft Brownish grey to grey Wet		5	SS	1		195	
			6	SS	WH		194	
			7	SS	-		193	
193.1			8	SS	5		192	
6.6	Silty Sand to Sand Loose Grey Wet		9	SS	7		191	
189.5							190	
10.1	Bedrock  Refer to Record of Drillhole BCB-5 for details						189	
							188	
							187	
185.9							186	
13.7								

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

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+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3</sup>% STRAIN AT FAILURE

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SHEET 1 OF 1

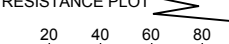
DATUM: Geodetic

DRILLING CONTRACTOR: MARATHON DRILLING LTD.

CHECKED: CG

MIS-RCK 002 031111028AARCK.GPJ GAL-MASS.GDT 5/10/06 JFC



PROJECT 03-1111-028			RECORD OF BOREHOLE No BCB-6			1 OF 1 METRIC		
W.P. 335-00-00			LOCATION N 5032697.7 ;E 256594.5			ORIGINATED BY BL		
DIST 52 HWY 69			BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Augers			COMPILED BY KG		
DATUM Geodetic			DATE November 2, 2004			CHECKED BY CG		
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100
199.5	GROUND SURFACE							PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W <sub>p</sub> W W <sub>L</sub> WATER CONTENT (%)
0.0	Silty Sand (FILL) Very loose to Loose Brown Moist		1	SS	4		199	
			2	SS	3		198	
197.5			3	SS	7		197	
2.0	Sandy Silt, trace clay Loose Brown Moist		4	SS	7		196	
196.9			5	SS	5			
2.6	Silty Clay, trace sand Reddish grey Stiff to Firm Wet							
195.1								
4.4	End of Borehole Auger refusal							
	Note: 1. Water level in open borehole at 2.4 m depth upon completion of drilling.							

PROJECT <u>03-1111-028</u>			RECORD OF BOREHOLE <b>No 06-1</b>			1 OF 1 <b>METRIC</b>					
W.P. <u>335-00-00</u>			LOCATION <u>N 5032733.0 ; E 256578.0</u>			ORIGINATED BY <u>BML</u>					
DIST <u>52</u> HWY <u>69</u>			BOREHOLE TYPE <u>Portable Drilling Equipment</u>			COMPILED BY <u>DD</u>					
DATUM <u>Geodetic</u>			DATE <u>June 29, 2006</u>			CHECKED BY <u>JPD</u>					
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT  SHEAR STRENGTH kPa ○ UNCONFINED    + FIELD VANE ● QUICK TRIAXIAL    × REMOULDED	PLASTIC LIMIT W <sub>p</sub> NATURAL MOISTURE CONTENT W    LIQUID LIMIT W <sub>L</sub> WATER CONTENT (%)	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
197.9	GROUND SURFACE										
0.0	Topsoil		1	SS	4						
0.2	Silty Sand										
197.2	Loose Grey Moist										
0.7	Silty Clay to Clay, trace silt Stiff to very soft Reddish brown to grey Wet		2	SS	3						
195.1											
2.8	Sand, some gravel Compact Grey Wet		3	SS	14						
194.6											
3.4	End of Borehole										

PROJECT <u>03-1111-028</u>		<b>RECORD OF BOREHOLE No 06-2</b>				1 OF 1 <b>METRIC</b>									
W.P. <u>335-00-00</u>		LOCATION <u>N 5032745.0 ;E 256559.0</u>				ORIGINATED BY <u>BML</u>									
DIST <u>52</u> HWY <u>69</u>		BOREHOLE TYPE <u>Portable Drilling Equipment</u>				COMPILED BY <u>DD</u>									
DATUM <u>Geodetic</u>		DATE <u>June 29, 2006</u>				CHECKED BY <u>JPD</u>									
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED		WATER CONTENT (%) W <sub>p</sub> W W <sub>L</sub>			γ	GR SA SI CL	
198.0	GROUND SURFACE							20 40 60 80 100							
0.0	Topsoil		1	SS	20										
0.2	Sand, some gravel, occasional rootlets														
197.3	Compact Brown Dry														
0.7	End of Borehole Refusal on large boulder/Possible Bedrock  Borehole dry upon completion of drilling.														

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PROJECT 03-1111-028			RECORD OF BOREHOLE No 06-3			1 OF 1 METRIC											
W.P. 335-00-00			LOCATION N 5032775.0 ; E 256535.0			ORIGINATED BY BML											
DIST 52 HWY 69			BOREHOLE TYPE Portable Drilling Equipment			COMPILED BY DD											
DATUM Geodetic			DATE June 29, 2006			CHECKED BY JPD											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ	GR SA SI CL
							20 40 60 80 100	20 40 60 80 100	W <sub>p</sub>	W	W <sub>L</sub>	20 40 60	kN/m <sup>3</sup>				
197.6	GROUND SURFACE																
0.0	Topsoil																
0.2	Sandy Silt, trace clay, trace gravel and organics Very loose Brown Wet		1	SS	1		197										
196.4	Silty Clay to Clay, trace silt Firm to Stiff Reddish brown Wet						196		5.4								
195.6																	
2.0	Sand Very loose Grey Wet		2	SS	2		195										
194.9																	
2.7	End of Borehole																

PROJECT <u>03-1111-028</u>		<b>RECORD OF BOREHOLE No 06-4</b>				1 OF 1 <b>METRIC</b>										
W.P. <u>335-00-00</u>		LOCATION <u>N 5032754.0 ; E 256546.0</u>				ORIGINATED BY <u>BML</u>										
DIST <u>52</u> HWY <u>69</u>		BOREHOLE TYPE <u>Portable Drilling Equipment</u>				COMPILED BY <u>DD</u>										
DATUM <u>Geodetic</u>		DATE <u>June 29, 2006</u>				CHECKED BY <u>JPD</u>										
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 $\gamma$ 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								
198.2	GROUND SURFACE						20	40	60	80	100					
0.0	Silty Sand to Sand, some gravel Loose to very loose Brown to grey Wet		1	SS	5		198									
							197									
196.1	End of Borehole		2	SS	2											
2.1	Note: 1. Water level at 0.3 m (Elev. 197.9 m) below ground surface upon completion of drilling.															

PROJECT 03-1111-028			RECORD OF BOREHOLE No 06-5			1 OF 1 METRIC		
W.P. 335-00-00			LOCATION N 5032757.0 ; E 256556.0			ORIGINATED BY BML		
DIST 52 HWY 69			BOREHOLE TYPE Portable Drilling Equipment			COMPILED BY DD		
DATUM Geodetic			DATE June 29, 2006			CHECKED BY JPD		
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100
197.0	GROUND SURFACE							PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W <sub>p</sub> W W <sub>L</sub> WATER CONTENT (%)
0.0	Organics/Topsoil		1	SS			196	
0.2	Silty Sand, occasional organics Very loose Grey Wet		2	SS	1			
195.4			3	SS	1		195	
1.6	Silty Clay to Clay, trace silt Firm Grey Wet							
194.1								
2.9	Silty Sand Compact Grey Wet		4	SS	13		194	
193.5								
3.5	End of Borehole							
Note: 1. Water level at 0.6 m (Elev. 196.4 m) below ground surface upon completion of drilling.								

## **TABLES**

**TABLE 1 - SUMMARY OF POINT LOAD TESTS ON ROCK CORE SAMPLES**

PROJECT NO 03-1111-028							
LOCATION: Existing Highway 69 / Blair (Sly) Creek Replacement Structure							
DATE: January 06, 2005							
Borehole Number	Sample Number	Rock Type	Sample Depth (ft)	Sample Depth (m)	Test Type	Is (50mm) (MPa)	Approx. UCS <sup>1</sup> (Is <sub>50</sub> x23)(MPa)
BCB-3	1	Granitic Gneiss	47.4	14.4	D	5.901	136
BCB-3	2	Granitic Gneiss	49.5	15.1	A	7.221	166
BCB-3	3	Granitic Gneiss	50.2	15.3	D	4.931	113
BCB-3	4	Granitic Gneiss	52.3	15.9	A	7.603	175
BCB-3	5	Mafic-rich Gneiss	54.3	16.5	D	5.441	125
BCB-3	6	Mafic-rich Gneiss	54.9	16.7	D	2.606	60
BCB-5	1	Granitic Gneiss	33.2	10.1	D	3.168	73
BCB-5	2	Granitic Gneiss	35.3	10.8	A	8.605	198
BCB-5	3	Granitic Gneiss	35.8	10.9	D	4.300	99
BCB-5	4	Mafic-rich Gneiss	39.9	12.2	A	4.610	106
BCB-5	5	Mafic-rich Gneiss	42.5	13.0	D	7.196	165
BCB-5	6	Mafic-rich Gneiss	44.5	13.6	D	5.067	117
<b>SUMMARY</b>				<b>Granitic Gneiss</b>			
				Average Axial		7.810	180
				Average Diametral		4.575	105
				St. Dev. Axial		0.715	16
				St. Dev. Diametral		1.146	26
				Number of Axial Tests		3	
				Number of Diametral Tests		4	
				<b>Mafic-rich Gneiss</b>			
				Average Axial		4.610	106
				Average Diametral		5.077	117
				St. Dev. Axial		-	-
				St. Dev. Diametral		1.891	43
				Number of Axial Tests		1	
				Number of Diametral Tests		4	

<sup>1</sup> UCS = Is x 23 is based on previous experience and would require UCS testing to further validate this relationship

Note: Rock specimens tend to be anisotropic (ie. stronger in axial than diametral).

**TABLE 2**  
**EVALUATION OF FOUNDATION ALTERNATIVES**  
**Existing Highway 69 Replacement Structure at Blair (Sly) Creek**  
**G.W.P. 335-00-00**

<i>Footing Option</i>	<i>Rank</i>	<i>Advantages</i>	<i>Disadvantages</i>	<i>Relative Costs</i>	<i>Risks/Consequences</i>
Steel H Piles	1	Relatively straight forward construction. Allows for integral abutment design.	<p>Downdrag loads and lateral loads have to be considered unless embankment preloading is carried out prior to pile installation or unless sub-excavation and removal of soft clay strata adopted as part of approach embankment settlement and stability mitigation alternative.</p> <p>If sub-excavation and removal mitigation alternative adopted, granular fill (less than 75 mm sizes), not rock fill, will have to be placed in abutment area to allow pile installation.</p>	Lower relative costs than spread footings on bedrock or spread footings on compact sand at depth.	
Spread Footings perched within embankment fill	2	Can eliminate temporary shoring and groundwater control required for excavation to expose bedrock.	<p>Potential for differential settlement between north and south abutment due to compression of embankment fill.</p> <p>Only viable if sub-excavation and removal of soft clay strata adopted as part of approach embankment settlement and stability mitigation alternative.</p>	Lower relative costs than piled foundations.	Not recommended due to potential for some differential settlements anticipated between north and south abutment.
Spread Footings on bedrock or mass concrete pad	X		<p>Deep (5 m to 16 m) excavations required. Temporary shoring and groundwater control required to expose bedrock surface.</p> <p>Highly variable bedrock surface will require mass concrete placement to achieve level footing.</p>	High cost for groundwater control and temporary shoring as compared with shallower footings.	Not recommended due to significant depth of excavations and close proximity to Blair Creek.
Shallow Spread Footings on near surface loose silty sand.	X		Low geotechnical resistance. Differential settlements between north and south abutment due to consolidation of underlying soft to firm silty clay under footing and approach embankment loading.	Lower relative costs than piled foundations.	Not recommended due to potential for some differential settlements anticipated between north and south abutment.

n:\active\2003\1111\03-1111-028 urs hwy 69 parry sound\reporting\final\7 - blair creek - replacement bridge\tables\table2\_evaluation foundation alternatives\_final.doc

**X:** Indicates that the founding option is not recommended for this site.

**TABLE 3**  
**EVALUATION OF SETTLEMENT / STABILITY MITIGATION ALTERNATIVES**  
**South Approach Embankment – Existing Highway 69 Replacement Structure at Blair (Sly) Creek**  
**G.W.P. 335-00-00**

<i>Stability/ Settlement Mitigation Option</i>	<i>Rank</i>	<i>Advantages</i>	<i>Disadvantages</i>	<i>Relative Costs</i>	<i>Risks/Consequences</i>
Pre-Loading and Stability Berm (1.5 m high x 8 m wide by 50 m long right of centreline only)	1	Relatively simple operation; no deep subexcavation or temporary shoring required.	Lengthened construction time required. Settlement of foundation soil may take 5 months to reach 90% consolidation. Toe berm required right of centreline.	Low cost. However some additional costs required for berm construction right of centreline (in the floodplain of Blair Creek).	Settlement of embankment/ foundation soils will occur.
Light Weight Fill (EPS) for Grade Raise (50 m long x 14 m wide x 1.5 m high)	2	Reduces load on compressible soils thereby increasing stability and reducing settlement of foundation soils. Settlement of embankment fill minimized. No berm required.	Very high material costs. Existing embankment grade would need to be lowered in order to install the EPS while allowing for a minimum 1 m of conventional fill over the EPS.	Cost savings in berm fill or sub-excavation and replacement, but relative cost of EPS fill is up to an order of magnitude higher than for the other options.	Factor of safety for stability increased. Settlements of foundation soils and embankment fill minimized.
Surcharging and Stability Berms	3	Relatively simple operation; no deep subexcavation or temporary shoring required. Less construction time required relative to preloading option.	Toe berms required to maintain stability under surcharge fill. For the area right of centreline, surcharging may not be able to be carried out as the much larger toe berm required would encroach into Blair Creek, which we understand is considered environmentally sensitive.	Increased cost of construction and material for surcharge and wider toe berms.	Not recommended due to environmental issues associated with in-filling Blair Creek with large toe berms to maintain stability of surcharge.
Full Subexcavation and Replacement (up to 4.0 m deep – below natural ground surface; up to 6.0 m deep below existing top of pavement)	4	Stability and long-term settlement issues minimized since all or nearly all weak, soft and compressible materials are removed. Stability berms not required.	Significant additional excavation required to remove existing embankment fill. Permanent sheet pile shoring would be required to support the adjacent creek on the west and north sides of the embankment. High groundwater table and permeable layers below the clayey strata would require sub-excavation and removal 'in-the-wet' and the use of rock fill for placement under water. Staged excavation in short strips required to maintain stability of sheet pile shoring.	Additional costs for sub-excavation and disposal of soft soils. Additional costs for partial removal of existing embankment, permanent sheet pile shoring along the edge of the adjacent Blair Creek.	Lower risk with respect to stability and long term settlement of foundation soils. Additional fill settlement due to increased effective embankment height. Potential for some impact on creek still exists.

**TABLE 3**  
**EVALUATION OF SETTLEMENT / STABILITY MITIGATION ALTERNATIVES**  
**South Approach Embankment – Existing Highway 69 Replacement Structure at Blair (Sly) Creek**  
**G.W.P. 335-00-00**

<i>Stability/ Settlement Mitigation Option</i>	<i>Rank</i>	<i>Advantages</i>	<i>Disadvantages</i>	<i>Relative Costs</i>	<i>Risks/Consequences</i>
Wick Drains	X	Reduce the preload / surcharge duration	Increased time for installation of wicks. Potential difficulties installing through existing embankment fill. Monitoring of settlements and pore pressures required.	Additional costs associated with wicks, possible pre-drilling costs for installation, and instrumentation and monitoring.	Not practical due to the limited thickness of the clayey subsoils in this area.

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**TABLE 4**  
**EVALUATION OF SETTLEMENT / STABILITY MITIGATION ALTERNATIVES**  
**North Approach Embankment – Existing Highway 69 Replacement Structure at Blair (Sly) Creek**  
**G.W.P. 335-00-00**

<i>Stability/ Settlement Mitigation Option</i>	<i>Rank</i>	<i>Advantages</i>	<i>Disadvantages</i>	<i>Relative Costs</i>	<i>Risks/Consequences</i>
Pre-Loading	1	Relatively simple operation; no deep subexcavation or temporary shoring required. No stability berms required.	Lengthened construction time required. Settlement of foundation soil may take 7 months to reach 90% consolidation. For the area north of the limits of the north approach embankment, further investigation and analysis may be required in order to assess if toe berms are required left of centreline.	Relatively low cost.	Settlement of embankment/ foundation soils will occur.
Surcharging	2	Relatively simple operation; no deep subexcavation or temporary shoring required. Less construction time required relative to preloading option. It is estimated that a 0.5 m high surcharge will reduce time to reach 90% consolidation to 5 months (versus 7 months for preloading only option)	1.5 m wide mid-height berm required to maintain stability under 0.5 m high surcharge fill. For the area north of the limits of the north approach embankment, further investigation and analysis may be required in order to determine whether or not toe berms would be required left of centreline.	Increased cost of construction and material for surcharge and toe berms.	Not recommended for surcharge heights greater than 0.5 m due to requirements for much larger stability berms and possible environmental issues associated with potential in-filling of Blair Creek due to larger berms.
Light Weight Fill (EPS) for Grade Raise (20 m long* x 14 m wide x 1.8 m high)  *EPS fill may also be required beyond the north limit of the north approach embankment – Station 11+590 to 11+710 (up to 120 m long)	3	Reduces load on compressible soils thereby increasing stability and reducing settlement of foundation soils. Settlement of embankment fill minimized.	Very high material costs. Existing embankment grade would need to be lowered in order to install the EPS while allowing for a minimum 1 m of conventional fill over the EPS.	Relative cost of fill is up to an order of magnitude higher than for the other options.	Settlements of foundation soils and embankment fill minimized. Factor of safety for stability increased between Stations 11+590 and 11+710.

**TABLE 4**  
**EVALUATION OF SETTLEMENT / STABILITY MITIGATION ALTERNATIVES**  
**North Approach Embankment – Existing Highway 69 Replacement Structure at Blair (Sly) Creek**  
**G.W.P. 335-00-00**

<i>Stability/ Settlement Mitigation Option</i>	<i>Rank</i>	<i>Advantages</i>	<i>Disadvantages</i>	<i>Relative Costs</i>	<i>Risks/Consequences</i>
Full Subexcavation and Replacement (up to 4.4 m deep – below natural ground surface; up to 7.0 m deep below existing top of pavement)	4	Stability and long-term settlement issues minimized since all or nearly all weak, soft and compressible materials are removed. Stability berms not required.	Permanent sheet pile shoring would be required to support the adjacent creek on the east and south sides of the embankment. High groundwater table and permeable layers below the clayey strata would require sub-excavation and removal 'in-the-wet' and the use of rock fill for placement under water. Staged excavation in short strips required to maintain stability of sheet pile shoring.	Additional costs for sub-excavation and disposal of soft soils. Additional costs for partial removal of existing embankment, permanent sheet pile shoring along the edge of the adjacent Blair Creek.	Lower risk with respect to stability and long term settlement of foundation soils. Additional fill settlement due to increased effective embankment height. Potential for some impact on creek still exists.
Wick Drains	X	Reduce the preload / surcharge duration	Increased time for installation of wicks. Potential difficulties installing through existing embankment fill. Monitoring of settlements and pore pressures required.	Additional costs associated with wicks, possible pre-drilling costs for installation, and instrumentation and monitoring.	Not practical due to the limited thickness of the clayey subsoils in this area.

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**TABLE 5**  
**Summary of Recommendations at Structure Approach Embankments (incl. Platform Widening)**  
**Existing Highway 69 (Service Road) Replacement Structure at Blair Creek**  
**G.W.P. 335-00-00**

Highway	Approx. Station	Proposed Works	Surface Conditions	Recommended Embankment Fill Type	Organics Encountered Along Alignment	Recommended Side Slope	Side Berm Recommended	Estimated Post-Construction Settlement ( $\delta$ ) and Platform Widening (w)**** (mm)	Swamp Excavation / Organic Removal OPSD
Existing Highway 69 (Service Rd) Replacement Structure at Blair (Sly) Creek	11+692 to 11+712	<u>North Approach</u> Grade raise up to 1.8 m high (final embank. height up to about 3.8 m*)	Existing embankment founded on bank of Blair Creek.	Rock fill above existing fill (and pavement structure) for grade raise between 1.3 m and 2.0 m high**	Yes. Up to about 0.3 m adjacent to existing embankment.	1.25H : 1V	No.	$\delta = 80 + 10 = 90$ w = 1000	203.020 (excavate existing slopes to 1H:1V)
	11+751 to 11+800	<u>South Approach and about 30 m southerly.</u> Grade raise up to 1.5 m high (final embank. height up to about 4.3 m*)	Existing embankment founded on bank / floodplain of Blair Creek. Channel meander located immediately to west of south approach.	Rock fill above existing fill (and pavement structure) for grade raise between 1.3 m and 2.0 m high**	Yes. Up to about 0.3 m adjacent to existing embankment. About 0.8 m in floodplain right of centreline (as encountered in borehole BCB-2).	1.25H: 1V	Yes. Right of Centreline (west side): 1.5 m high by 8 m wide by 50 m long	$\delta = 50 + 10 = 60$ w = 1000	203.020 (excavate existing slopes to 1H:1V)

n:\active\2003\1111\03-1111-028 urs hwy 69 parry sound\reporting\draft\7 - blair creek - replacement bridge\tables\table5\_summaryapproachembankmentrecommendations (incl nre 98-200).doc

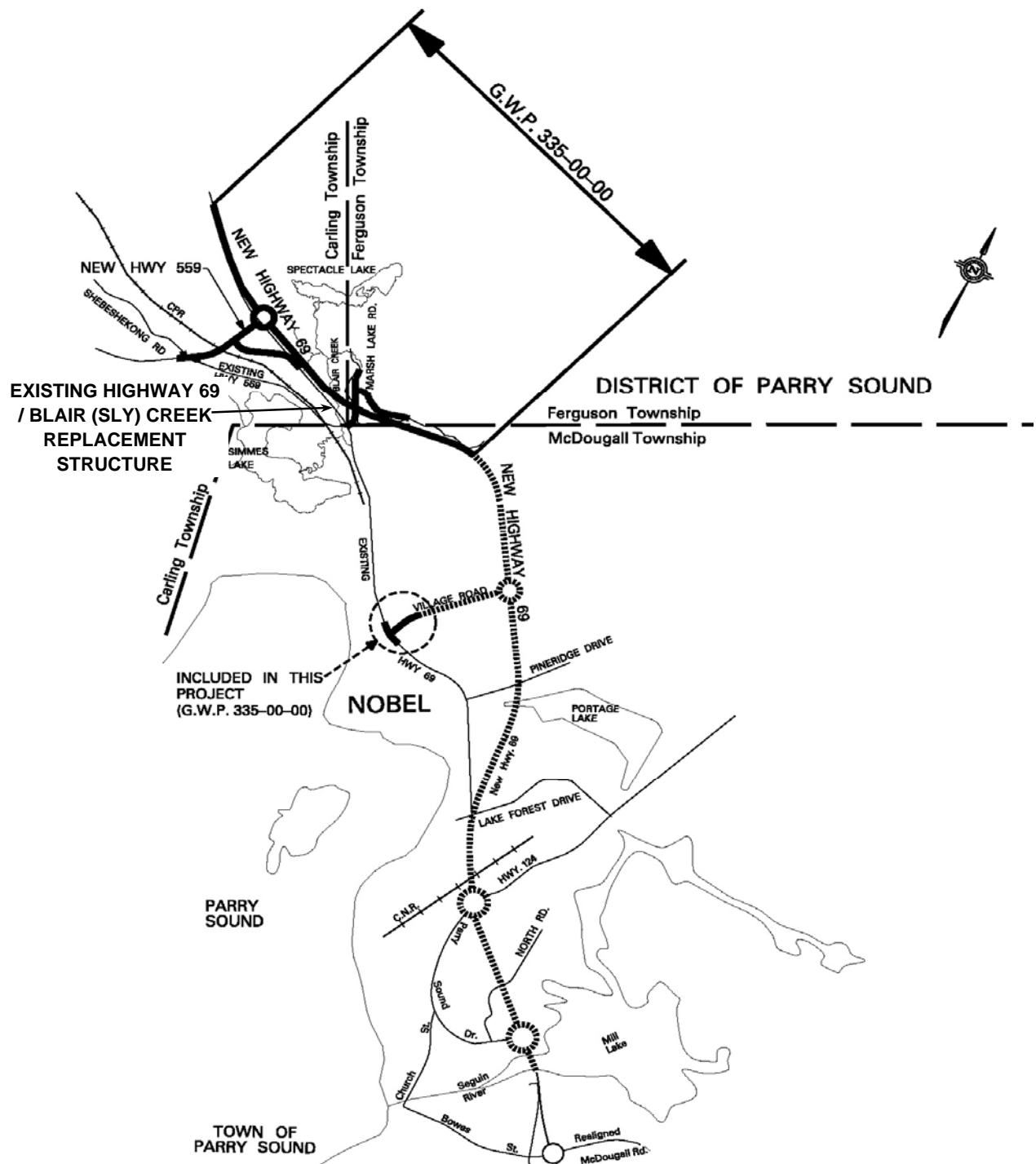
- Notes :**
- \* Relative to the inferred natural ground surface (i.e. below existing approach embankment fill).
  - \*\* As per Final Pavement Design Report – Service Road – Golder Associates Ltd., October 2005.
  - \*\*\* Settlements include compression of rockfill plus compression of cohesive layers below embankment (where encountered).
  - \*\*\*\* Recommended embankment platform widening (per embankment side) based on guidelines in NRE 98-200.

## **FIGURES**

# SITE LOCATION MAP

HIGHWAY 69 FROM 1.5 KM SOUTH OF HIGHWAY 559  
TO 3.5 KM NORTH OF HIGHWAY 559

FIGURE 1



GWP No. 335-00-00  
Date: March 2006  
Project: 03-1111-028-7

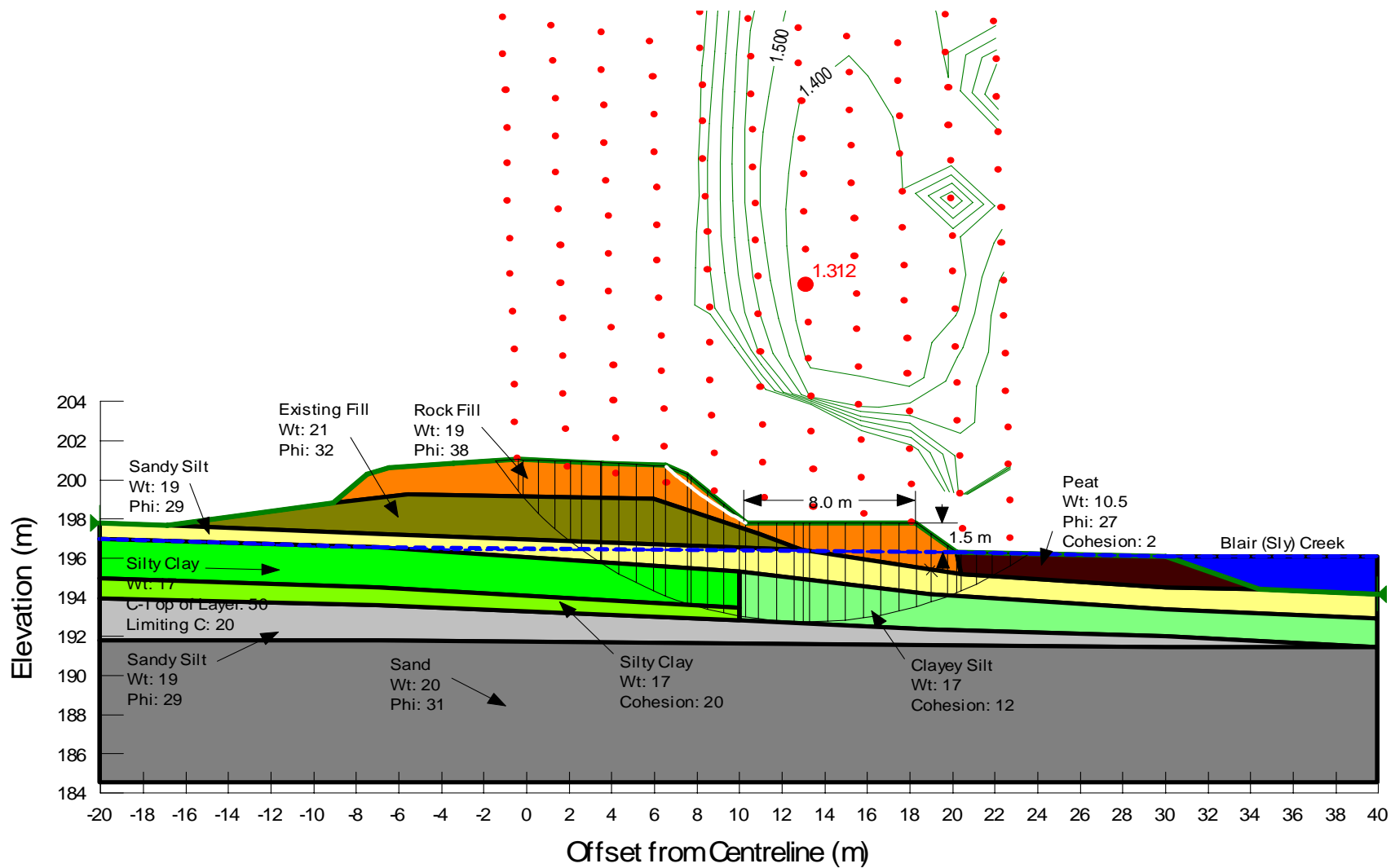
Drawn by: CMG  
Checked by: JPD

Golder Associates

Provided in digital format by URS on January 7, 2005

**EXISTING HIGHWAY 69 REPLACEMENT STRUCTURE OVER BLAIR (SLY) CREEK**  
**SOUTH APPROACH EMBANKMENT - RIGHT OF CENTRELINE**

**FIGURE 2**



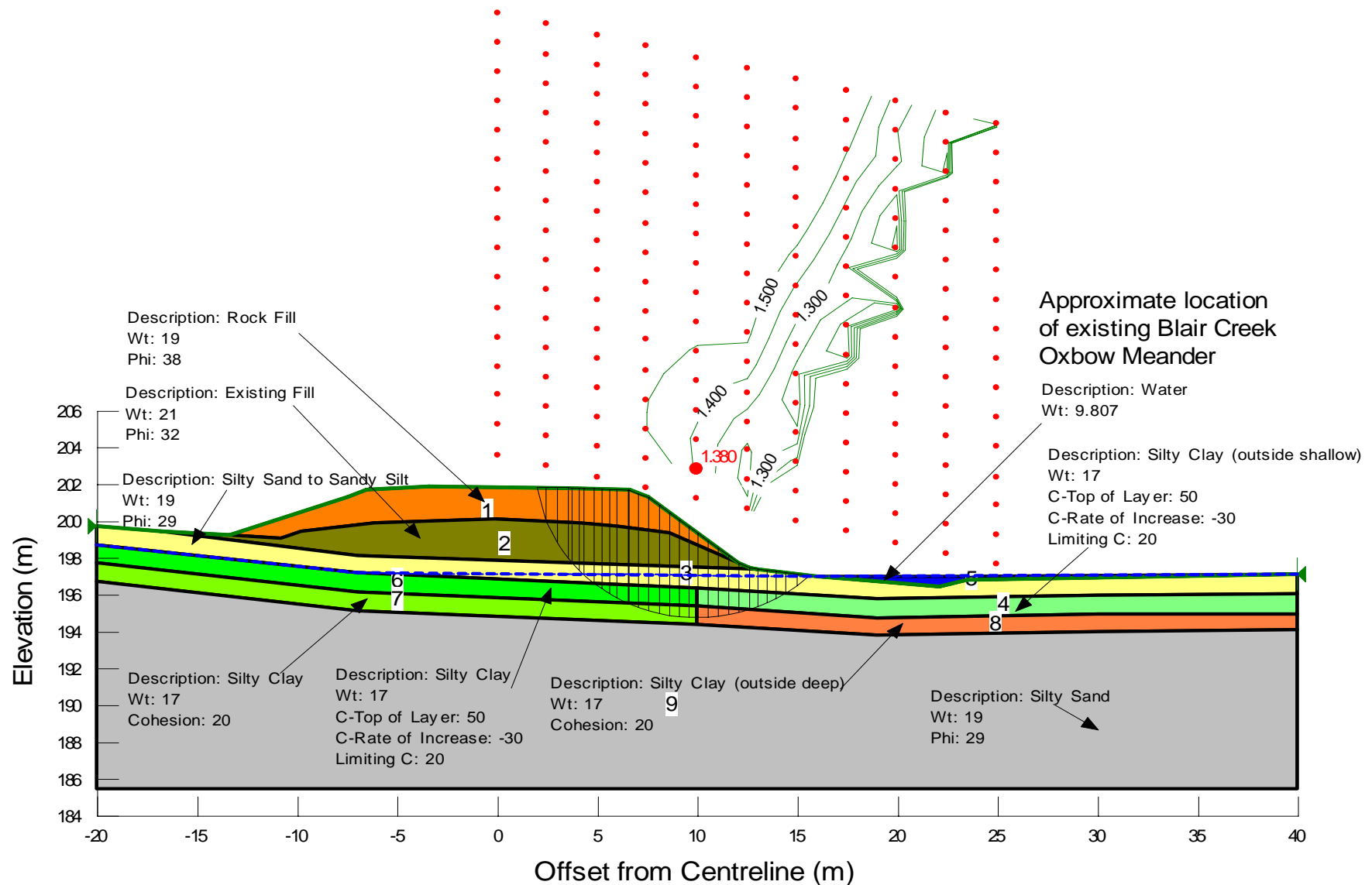
Date: Sep-06  
 Project: 03-1111-028-7

**Golder Associates**

Drawn: CMG  
 Checked: JPD

**EXISTING HIGHWAY 69 REPLACEMENT STRUCTURE OVER BLAIR (SLY) CREEK**  
 SERVICE ROAD - STA. 11+600 to STA. 11+660 (LEFT OF CENTRELINE)

**FIGURE 3**

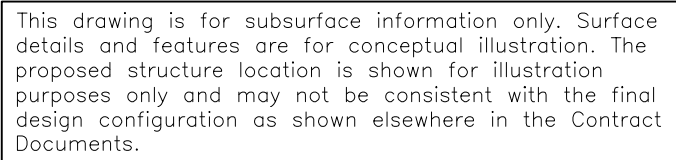


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 Project: 03-1111-028

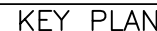
**Golder Associates**

Drawn: BML  
 Checked: JPD

## **DRAWINGS**



SHEET



500 0 500 1000m

	Borehole – Current Investigation
R	Refusal
	Seal
	Piezometer
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
100%	Rock Quality Designation (RQD)
	WL in piezometer, measured on Nov. 14, 2004
	WL upon completion of drilling

No.	ELEVATION	CO—ORDINATES	
		NORTHING	EASTING
BCB—1	199.4	5032671.2	256659.
BCB—2	196.3	5032649.7	256645.
BCB—3	199.6	5032679.6	256647.
BCB—5	199.6	5032689.4	256607.
BCB—6	199.5	5032697.7	256594.

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

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

For subsurface information only.

Base plans provided in digital format by URS, drawing file no. BlairCreekReplaceGA.dwg, dated Nov. 2005 received Nov. 11, 2005.

NO.	DATE	BY	REVISION		
<b>Geocres No. 41H-59</b>					
HWY. 69		PROJECT NO. 03-1111-028		DIST. 52	
SUBM'D. CMG	CHKD. CMG	DATE: SEPT. 2006		SITE:	
DRAWN: JFC	CHKD. JPD	APPD.		DWG. 1A	

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

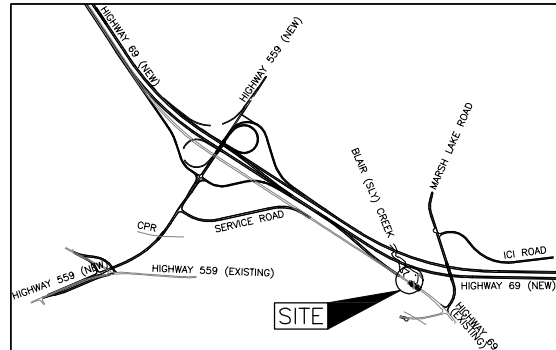
CONT No.  
GWP No. 335-00-00

EXIST. HIGHWAY 69 (SERVICE ROAD)  
PROPOSED REPLACEMENT BRIDGE OVER  
BLAIR (SLY) CREEK  
BOREHOLE SOIL STRATA

SHEET



**Golder Associates Ltd.**  
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN

SCALE

500 0 500 1000m

### LEGEND

- Borehole - Current Investigation
- R Refusal
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated  
(Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- WL in piezometer, measured on Nov. 14, 2004
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
BCB-1	199.4	5032671.2	256659.6
BCB-2	196.3	5032649.7	256645.6

### NOTES

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

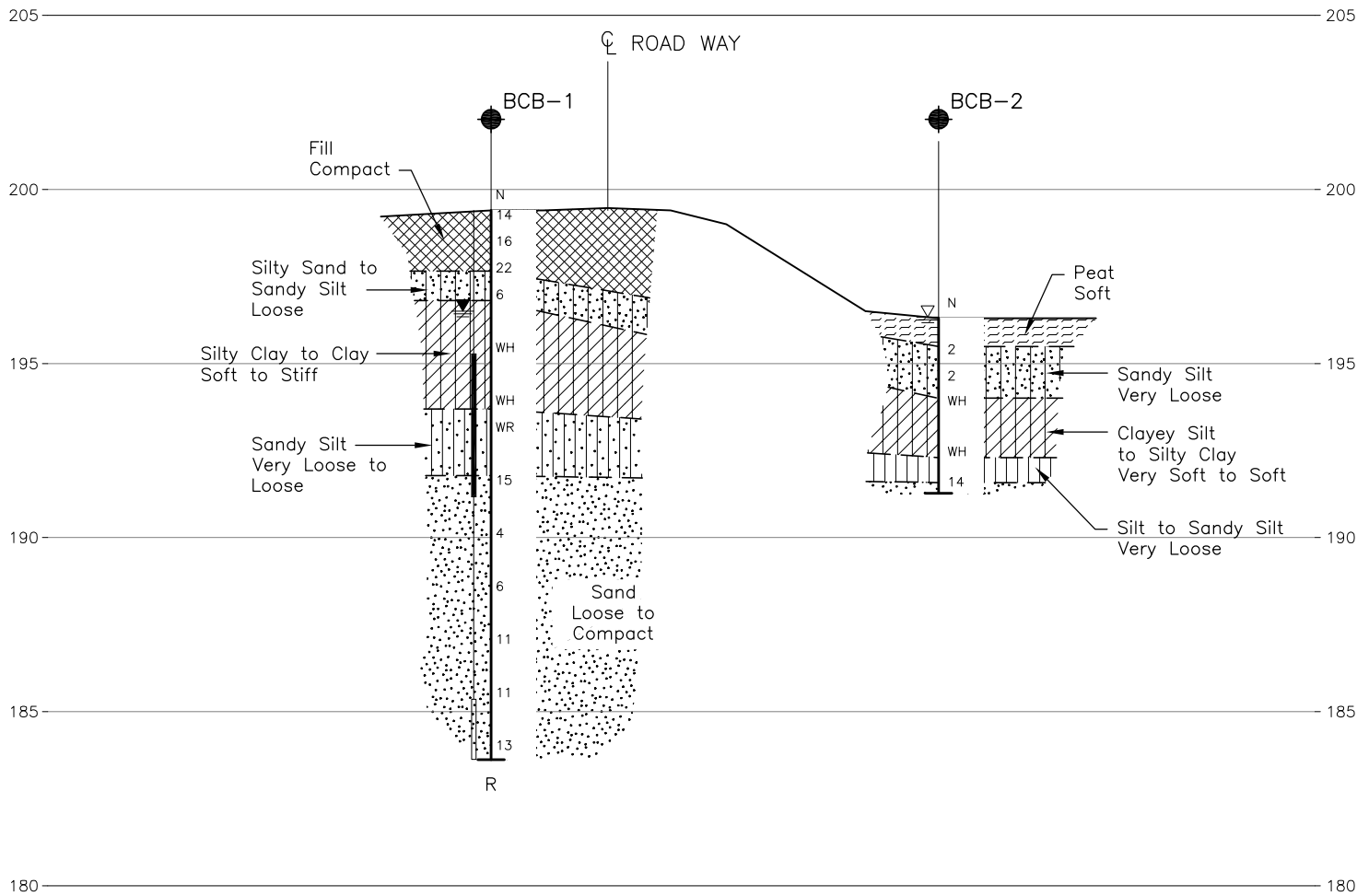
The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS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

For subsurface information only.

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NO.	DATE	BY	REVISION
Geocres No. 41H-59			
HWY. 69	PROJECT NO. 03-1111-028		DIST. 52
SUBM'D. CMG	CHKD. CMG	DATE: SEPT. 2006	SITE:
DRAWN: MSM	CHKD. JPD	APPD.	DWG. 1B



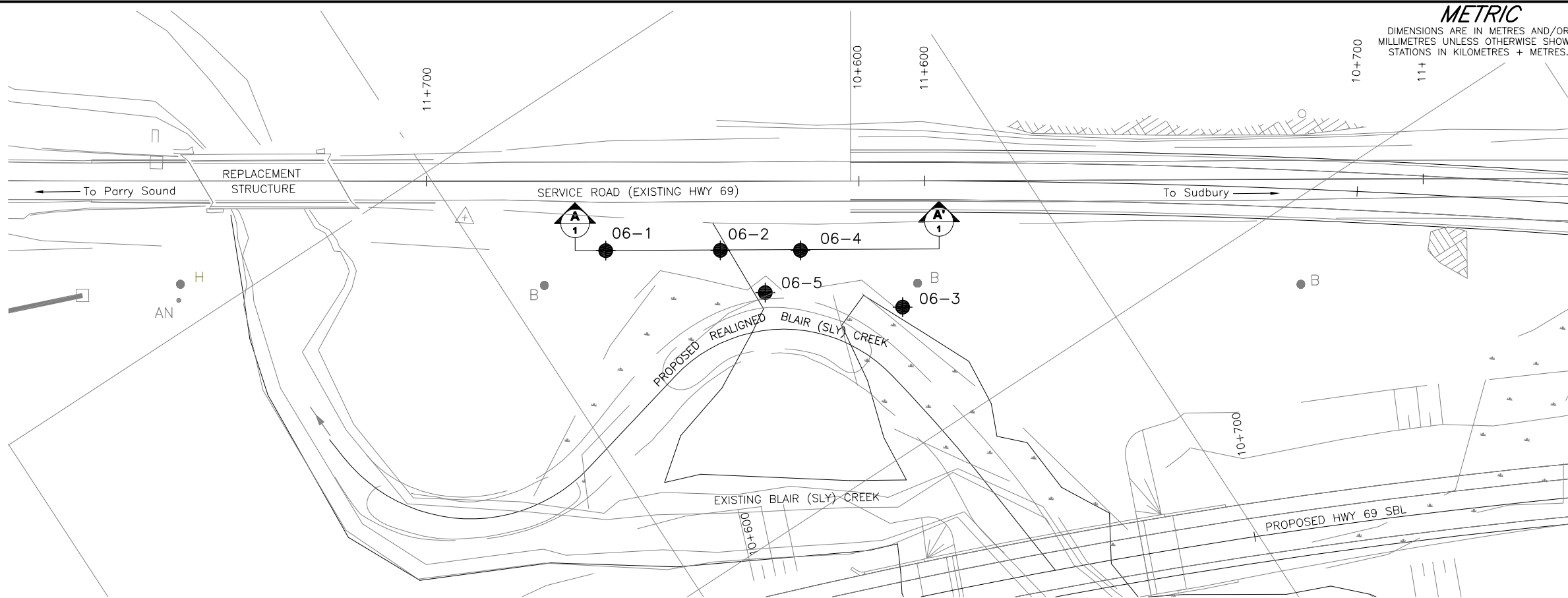
B-B'

### SOUTH APPROACH

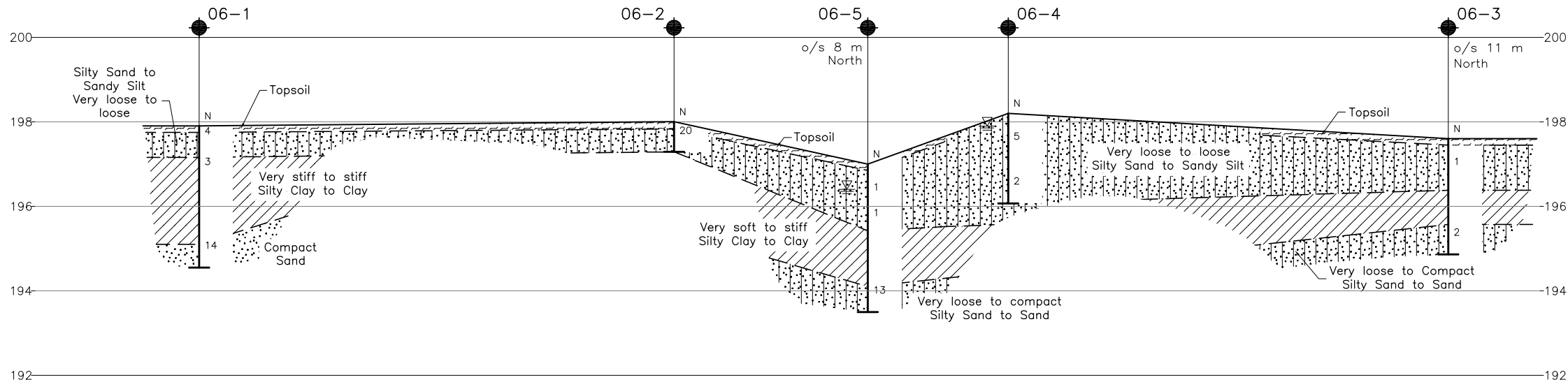
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4 0 4 8 m

VERT. SCALE  
2 0 2 4 m

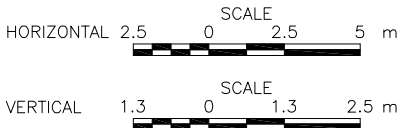




PLAN



PROFILE A-A'



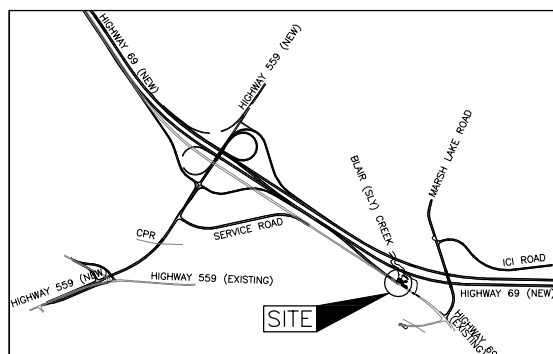
**METRIC**  
DIMENSIONS ARE IN METRES AND/OR  
MILLIMETRES UNLESS OTHERWISE SHOWN.  
STATIONS IN KILOMETRES + METRES.

CONT No.  
WP No. 335-00-00

EXIST. HIGHWAY 69 (SERVICE ROAD)  
PROPOSED REALIGNED  
BLAIR (SLY) CREEK 11+600 TO 11+660  
BOREHOLE LOCATIONS & SOIL STRATA



**Golder Associates Ltd.**  
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN



LEGEND

- Borehole - June 29, 2006 Investigation
- Hydro Pole
- ≡ WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
06-1	197.9	5032733	256578
06-2	198.0	5032745	256559
06-3	197.6	5032775	256535
06-4	198.2	5032754	256546
06-5	197.0	5032757	256556

NOTES

This drawing is for subsurface information only.

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

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

REFERENCE

Base plan provided in digital format by URS, drawing file Blair Creek Plan.dwg, received September 28, 2006.

NO.	DATE	BY	REVISION
Geocres No. 41H-59			
HWY. 69	PROJECT NO. 03-1111-028		DIST. 52
SUBM'D. BML	CHKD. BML	DATE: SEPT. 2006	SITE:
DRAWN: MSM	CHKD. JPD	APPD. FJH	DWG. 1C

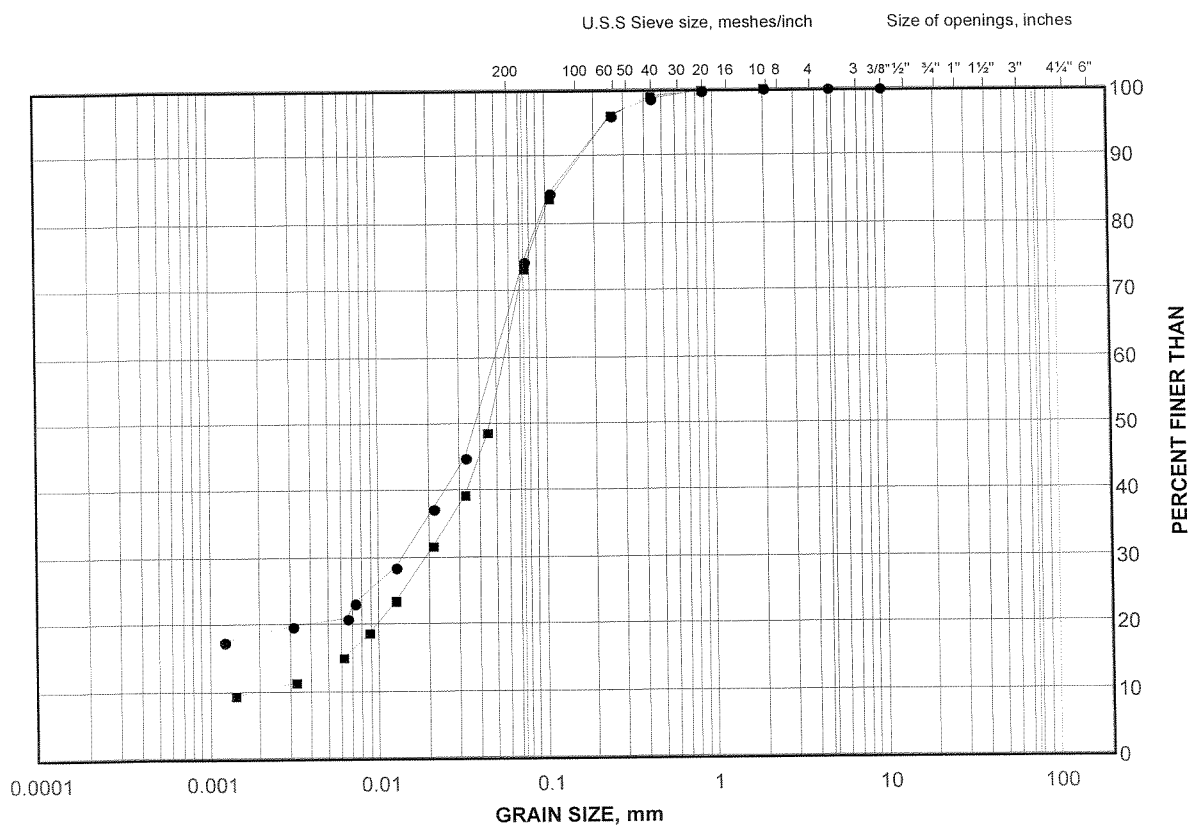


**APPENDIX A**  
**LABORATORY TEST DATA**

# GRAIN SIZE DISTRIBUTION

Sandy Silt

FIGURE A-1



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

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	BCB-2	4	192.2
■	BCB-1	7	193.0

Project Number: 03-1111-028

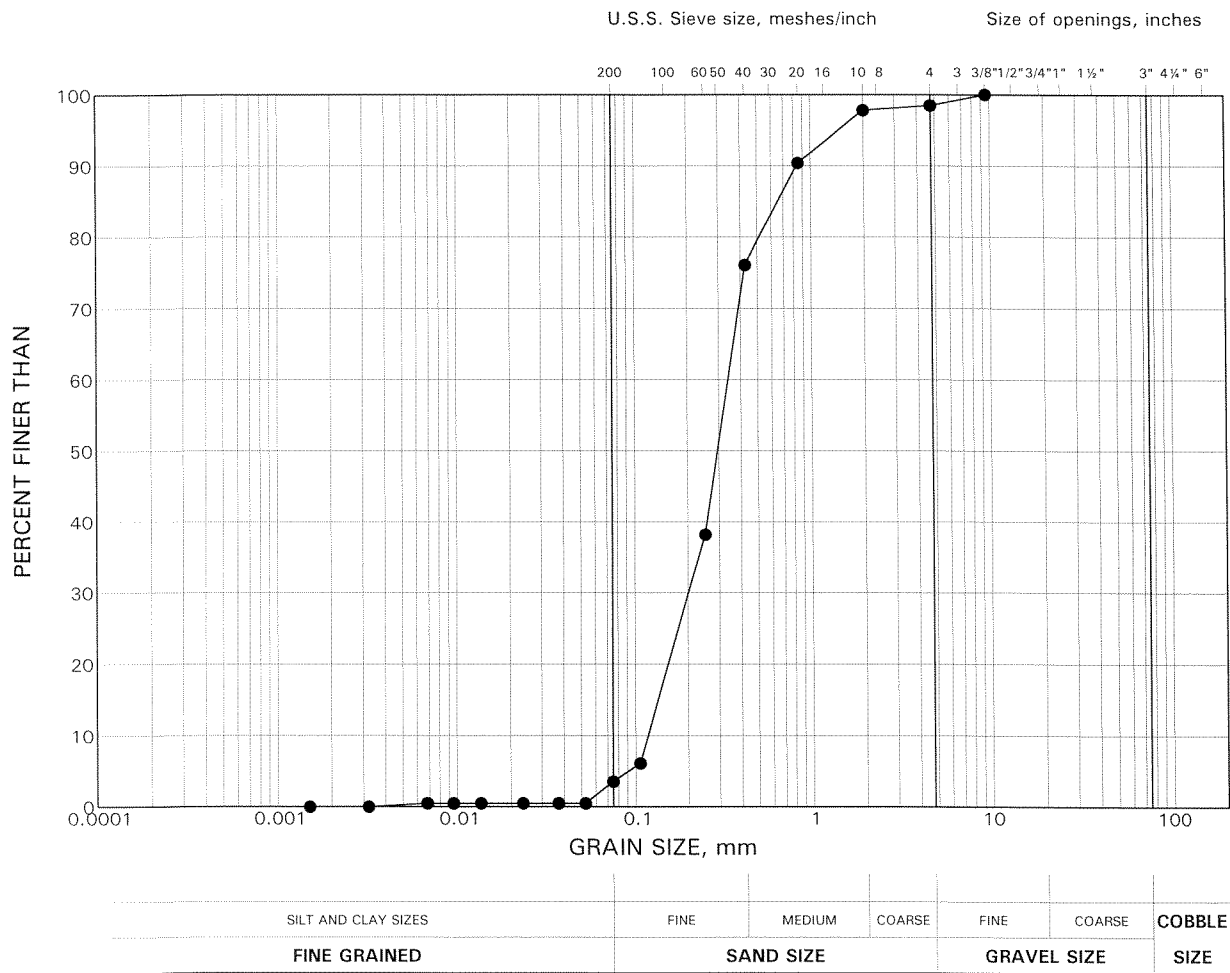
Checked By: *CA*

Golder Associates

Date: 28-Mar-06

# GRAIN SIZE DISTRIBUTION Sand

FIGURE A-2

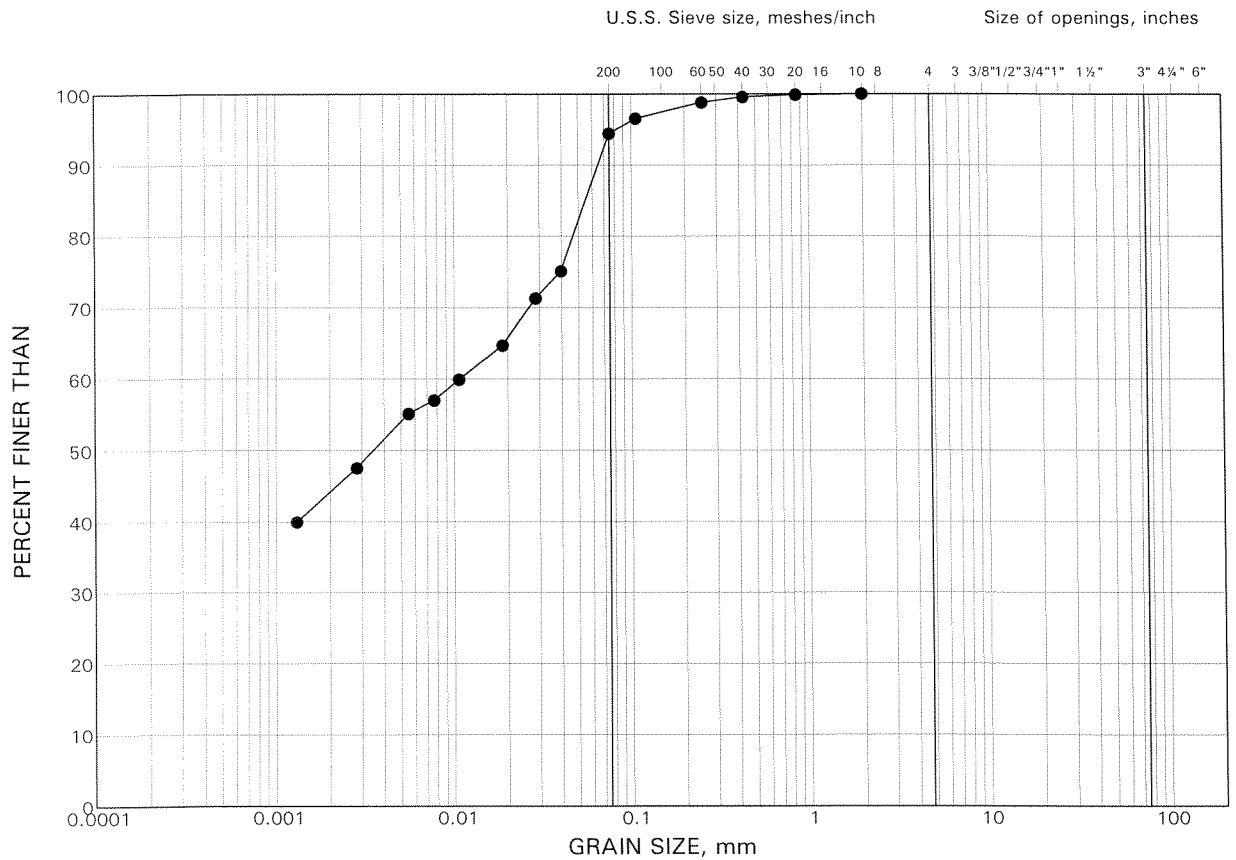


LEGEND			
SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
•	BCB-3	10	188.6

# GRAIN SIZE DISTRIBUTION

Silty Clay to Clay

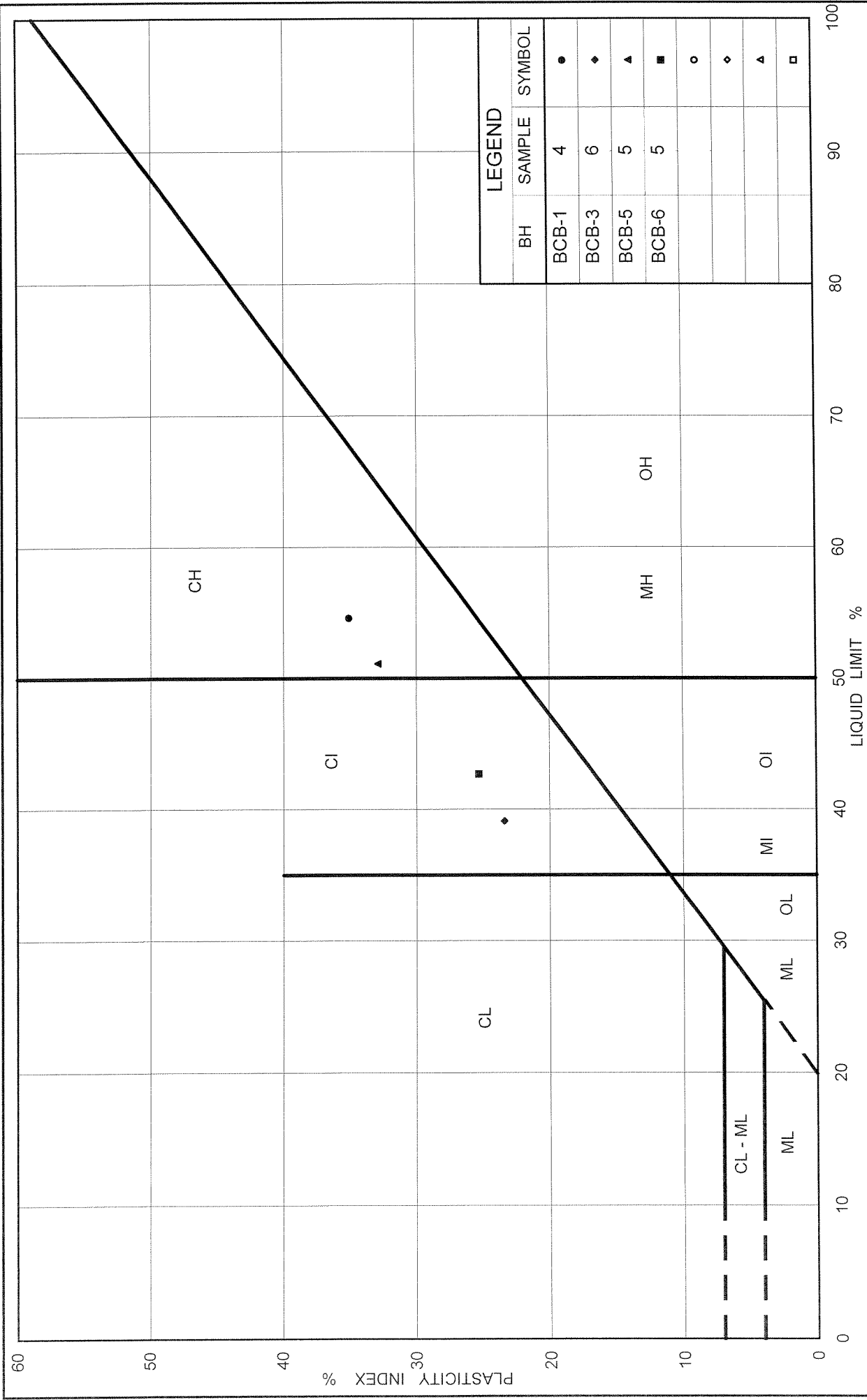
FIGURE A-3



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

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
•	BCB-5	5	195.5



**APPENDIX B**  
**SITE PHOTOGRAPHS**

Oblique Aerial Photograph  
Existing Highway 69 / Blair (Sly) Creek

FIGURE B-1



**SITE PHOTOGRAPHS**  
Existing Highway 69 / Blair (Sly) Creek

**FIGURE B-2**  
Page 1 of 3



**Photo 1** : West side of existing bridge (looking north) - October 2004.



**Photo 2** : Centreline of existing bridge (looking north) - October 2004.

GWP No. 335-00-00  
Date: March 2006  
Project: 03-1111-028-7

Drawn by: CMG  
Checked by: JPD

**Golder Associates**

**SITE PHOTOGRAPHS**  
Existing Highway 69 / Blair (Sly) Creek

**FIGURE B-2**  
Page 2 of 3



**Photo 3** : East side of existing bridge (looking north) - October 2004.



**Photo 4** : Underside of existing bridge (looking east) - October 2004.

GWP No. 335-00-00  
Date: March 2006  
Project: 03-1111-028-7

Drawn by: CMG  
Checked by: JPD

**Golder Associates**

N:\Active\2003\1111\03-1111-028 URS Hwy 69 Parry Sound\Reporting\Draft\7 - Blair Creek - Replacement Bridge\Appendix B\Figure B-2.xls\Figure B-2 (2)

**SITE PHOTOGRAPHS**  
Existing Highway 69 / Blair (Sly) Creek

**FIGURE B-2**  
Page 3 of 3



**Photo 5** : West side of existing bridge (looking north) - January 2006.



**Photo 6** : West side of existing bridge (looking south) - January 2006.

GWP No. 335-00-00  
Date: March 2006  
Project: 03-1111-028-7

Drawn by: CMG  
Checked by: JPD

**Golder Associates**

N:\Active\2003\1111\03-1111-028 URS Hwy 69 Parry Sound\Reporting\Draft\7 - Blair Creek - Replacement Bridge\Appendix B\Figure B-2.xls\Figure B-2 (3)

**APPENDIX C**

**SAMPLE NON-STANDARD SPECIAL PROVISIONS**

Special Provision

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**SCOPE**

This specification covers the requirements for the installation of the corrugated steel pipes (CSPs) at the integral abutments.

**SUBMISSION AND DESIGN REQUIREMENTS**

All submissions shall bear the seal and signature of an Engineer.

At least two weeks prior to commencement of installation of the abutment piles, the Contractor shall submit to the Contract Administer, for information purposes only, three (3) sets of the working drawings.

The Contractor shall have a copy of the submitted working drawings on site at all times. Working drawings shall include at least the following:

1. Layout and elevations of the CSPs;
2. Location of reference points, and location of the centroid of each pile with respect to the reference points;
3. Construction sequence and details;
4. Source of the sand fill, and description of placing methods and equipment;
5. Location and details of all temporary bracing and spacers for the piles and CSPs;
6. Method for preventing water and debris from entering the CSP prior to placing sand; and
7. Method for preventing concrete from abutment pours from entering the CSPs during placement.

The Contractor shall be responsible for the complete detailed design of all temporary bracing, including spacers required to maintain the piles, CSP spacing and abutment stems in their specified positions through all stages of construction until the CSPs have been backfilled. All temporary bracing shall be removed.

**MATERIAL**

**Corrugated Steel Pipe**

CSP shall be in accordance with OPSS 1801, and shall be from a supplier listed under DSM#4.60.80. The CSP shall be of the diameter and wall thickness specified on the Contract drawings, and shall be galvanized in accordance with CSA G164-M.

## **CSP FOR INTEGRAL ABUTMENTS – Item No.**

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### **Special Provision**

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CSPs shall be supplied in the lengths and with the end treatments, either square or skew, as specified on the Contract drawings; field cutting and splicing of CSPs will not be permitted. Cut ends shall be neat and free of burrs. The planes defined by the end treatments of each CSP shall be parallel to each other.

Handling and storage of CSPs shall be in accordance with the manufacturer's recommendations. Damaged CSPs shall be rejected. Localized areas of damaged galvanizing on otherwise acceptable CSPs shall be repaired with two coats of zinc-rich paint.

### **Sand Fill**

The sand fill for backfilling the CSP shall meet the gradation requirements of Table 1 below:

**Table 1 – Sand Fill Gradation Requirements**

MTO Sieve Designation		Percentage Passing by Mass
2 mm	#10	100%
600 µm	#30	80% to 100%
425 µm	#40	40% to 80%
250 µm	#60	5% to 25%
150 µm	#100	0% to 6%

### **CONSTRUCTION**

The sequence of construction shall be in accordance with the working drawings and as follows, unless otherwise approved:

1. Form concrete levelling pad and place CSPs and spacers.
2. Construct concrete levelling pads.
3. Place loose sand into 600 diameter CSP.
4. Install piles by driving to bedrock.
5. Remove temporary spacers.

The CSP shall be positioned such that the piles are centrally positioned within the CSP. Temporary blocking and bracing shall be used to hold the CSP in position.

The Contractor shall ensure the full perimeter of the tops of all CSPs at each abutment are at the elevation and orientation shown on the working drawings.

## **CSP FOR INTEGRAL ABUTMENTS – Item No.**

---

### **Special Provision**

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The CSP at each pile shall be constructed to the following tolerances:

<b><u>Criteria</u></b>	<b><u>Tolerance</u></b>
<b>Maximum deviation of CSP from pile centroid</b>	<b>+/- 50 mm</b>
<b>Maximum deviation of any point on the top perimeter of the CSP from the specified elevation</b>	<b>+/- 10 mm</b>

The sand fill shall be placed dry of optimum and free-flowing, completely filling the volume between the CSP and pile. No additional compaction effort other than the action of placing the sand itself shall be applied to the sand fill.

The placing of the sand fill shall be carried out in a manner such as to not damage and displace the CSP.

### **BASIS OF PAYMENT**

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

## **ROCK POINTS – Item No.**

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### **Special Provision**

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### **SCOPE**

As part of the work under the above tender item the Contractor shall supply and install TITUS Rock Injector Pile Points on HP 310 x 110 Piles.

### **REFERENCES**

OPSS 906 – Structural Steel

### **MATERIALS**

The pile points shall be of the following:

<b><u>Product</u></b>	<b><u>Manufacturer</u></b>
HPP-R-12	Titus Steel Company Ltd. 6767 Invader Cr. Mississauga, ON Tel (905) 564-2446

(Or approved equivalent)

### **BASIS OF PAYMENT**

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

**APPENDIX D**  
**REFERENCE INFORMATION**

#56-F-215C

Hwy #69

SLY CREEK

CROSSING

B.A. 567

# RACEY, MacCALLUM AND ASSOCIATES

LIMITED

A COMPANY OWNED, MANAGED AND OPERATED BY

Consulting Engineers  
AND ASSOCIATED STAFF

MONTREAL



VANCOUVER

TORONTO

DONALD C. MACCALLUM, B.ENG., M.E.I.C., PENG

H. JOHN RACEY, B.SC., M.E.I.C., PENG

A. ERIC RANKINE, B.SC., M.E.I.C., A.M.I.E.E.C.E., PENG

TORONTO DIVISION  
20 CARLTON STREET

REFERENCE: S-500-564/T-470.

15 October 1956.

Department of Highways of Ontario,  
c/o Morrison, Hershfield, Millman and Huggins,  
96 Bloor Street West,  
TORONTO, Ontario.

Attention: Mr. H. Tryhorn

RE: FOUNDATION INVESTIGATION FOR THE  
PROPOSED SLY CREEK CROSSING,  
HIGHWAY NO. 69 REVISION, APPROXIMATELY  
SEVEN MILES NORTH OF PARRY SOUND,  
ONTARIO.

Dear Sirs:

We have completed our investigation of the above noted bridge site and our report on the subject is attached hereto. The depth of bedrock was found to vary to some extent, particularly under the west bank, and its surface elevation was noted to be between El. 605 and El. 627 feet. One correction is required in the bedrock depths previously submitted verbally to you, and this applies to the probing at hole 2A, where bedrock was located at El. 608 rather than at El. 610.5 feet, indicated earlier. We regret any inconvenience this error may have caused you.

In this report we have given little consideration to the competence of the overburden, since support on bedrock seems the obvious means for carrying the bridge weight. We shall be pleased to discuss overburden conditions in greater detail, if you require.

We thank you for this opportunity to be of service to you. If you have any further queries, please do not hesitate to get in touch with us.

Yours very truly,  
RACEY, MacCALLUM AND ASSOCIATES LIMITED

*W.A. Trow*  
W.A. Trow, P. Eng.  
Divisional Soils Engineer.

WAT/MD  
In quadruplicate

FOUNDATION INVESTIGATION FOR THE  
PROPOSED SLY CREEK CROSSING, HIGHWAY  
NO. 69 REVISION, APPROXIMATELY SEVEN  
MILES NORTH OF PARRY SOUND, ONTARIO.

Report No: S-500-564/T-470

Racey, MacCallum and Associates Limited.

15 October 1956.

15 October 1956.

FOUNDATION INVESTIGATION FOR THE  
PROPOSED SLY CREEK CROSSING, HIGHWAY  
NO. 69 REVISION, APPROXIMATELY SEVEN  
MILES NORTH OF PARRY SOUND, ONTARIO.

This report presents the results of a foundation investigation performed at the above noted bridge site, during the period from 27 September to 2 October, 1956. The work consisted of one boring and two probings to rock on each bank of the river, as indicated in enclosure no. 1.

DESCRIPTION OF THE SITE AND SUBSOIL CONDITIONS

Sly Creek is a relatively sluggish river that traverses the area in a general north south direction. It is bounded by soil banks, approximately five feet high, and the general vegetation along them suggests that flood water conditions are not matters for great concern. Bedrock outcrops steeply, approximately two hundred feet south west of the west bank, and comes out at a flat angle about one hundred and fifty feet right of station 12+00. These surface observations indicate generally shallow overburden in the area.

The present highway bridge, in the shape of a large concrete box culvert, appears to be in good condition and about three and a half foot clearance has been left under it to pass flood water. Its clear span is approximately forty two feet. Conversation with local residents indicates that the footings for this bridge were carried thirty feet below the present ground surface.

The soil conditions at the site, as disclosed by borings 1 and 2, are presented in enclosures 2 and 3. They indicate that the site is overlain by approximately fourteen feet of brown silty clay, which is stiffer near the surface, probably the result of dessication. Under this clay the soil changes to a loose, slightly cohesive, silt, which in turn grades into a uniform fine to medium sharp sand existing in a medium dense condition. Some gravel was encountered in this sand, particularly over bedrock in hole 2, on the west bank. Bedrock consisted of biotite gneiss with some granite, both of which exist in a sound state. Evidence from the two borings and the additional two probings on each bank, indicates that the bedrock surface is not uniform and the gradient under the west bank is of the order of one on three.

DISCUSSION OF THE RESULTS

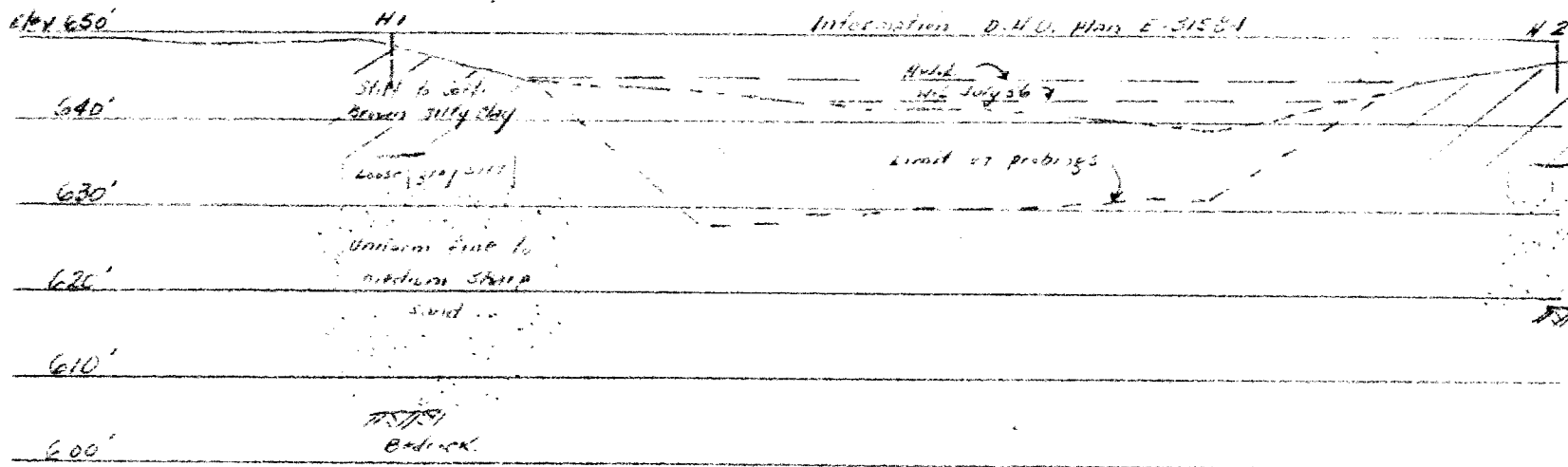
The subsoil and bedrock conditions at the site are such as to warrant little discussion, in view of the foundation

15 October 1956.

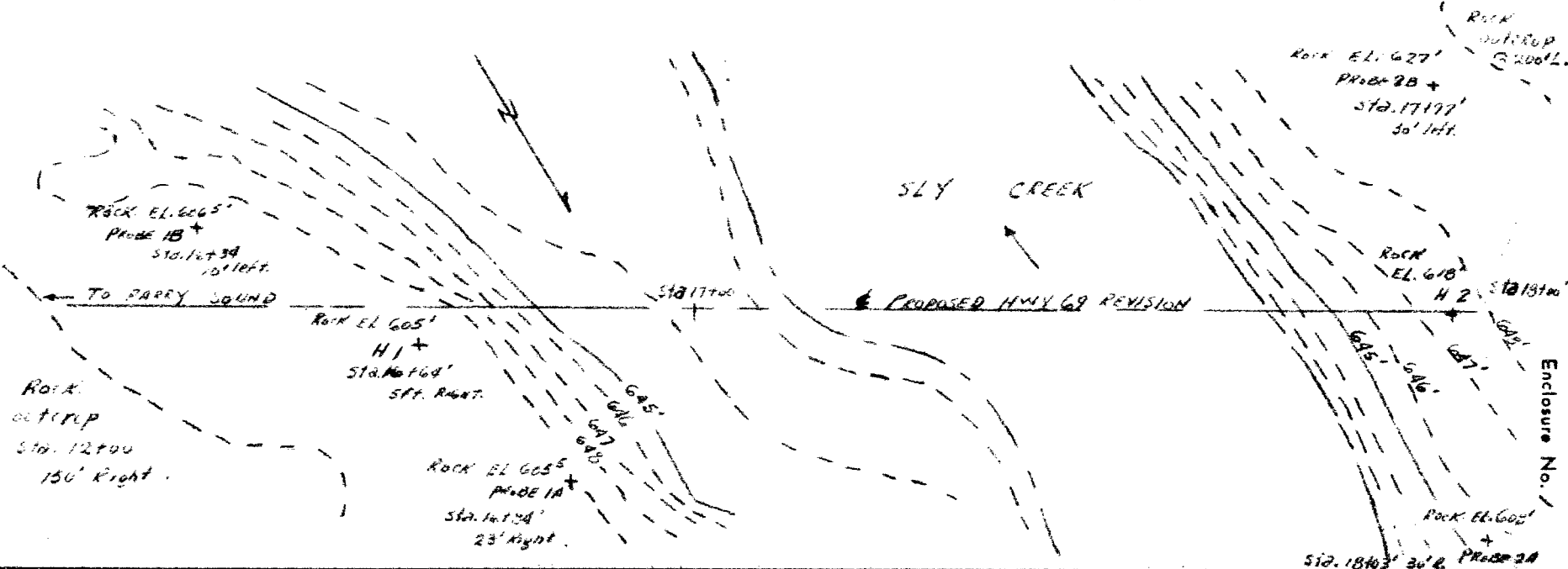
requirements for the bridge structure. It is understood that the footings must carry at least to El.635 feet, for frost and river scour protection, and at this depth loose silt and fine sand deposits are encountered.

On the basis of the empirical standard penetration test, the safe bearing values in this silt and sand, for a limiting settlement of one inch, are of the order of 1000 p.s.f. Recent personal experience, plus information published by the Bureau of Reclamation, suggests that this estimate is conservative, particularly in view of the fact that these granular deposits would undergo instantaneous settlement or adjustment as the bridge load was applied. However, a conservative approach may be warranted, considering that excavation into the silt and fine sand would set up a differential ground water head, promoting "quick sand" or loosening of the soil.

In view of the proximity and competence of bedrock, the foregoing comments are probably of academic interest only, since the support of the structure on piles to bedrock should provide the most economic and sure foundation for the proposed bridge. The embankment heights on the approaches to the site are negligible and, hence, the active thrust against the bridge and its supporting piles, should cause no serious concern. Ample support against movement should be provided by friction at the pile tip-rock contact, and by the passive resistance of the overlying sand. This remark should apply as well to the stability of the pile tips against sliding along the inclined bedrock face, provided the bedrock gradient does not exceed the value of one to three, noted on the west bank. A close record should be kept during pile driving operations, so that any steeper bedrock gradients can be detected and remedial measures taken. The precise resistance to sliding along inclined bedrock can only be determined by a pile load test.



SKETCH SHOWING BOREHOLE LOCATION AND SUBSOIL PROFILE Horiz. & vert. Scale 2" = 1'



Order No. SS00-47470Enclosure No. 2**RACEY MacCALLUM AND ASSOCIATES LTD.**

Foundation Engineering Division

Engineering Data Sheet for Borehole: 1Project: PROPOSED SLY CREEK CROSSINGField Supervision: H. G.Location: HWY 69 REVISION ≈ 7 MI. NORTH PARRY SOUNDDriller: RNT2Hole Location: SEE ENCL 1Prep.: Hole Elevation and Datum: 649'Checked: Field Work Begun: 27-9-56 Ended: 2-10-56Date: **LEGEND****Sampling Method**

2" Dia split tube

2" Shelby tube

**Penetration Resistance**

2" Split tube

2" Dia. Cone

Casing

**Strength**

Unconfined compression

Vane test and sensitivity

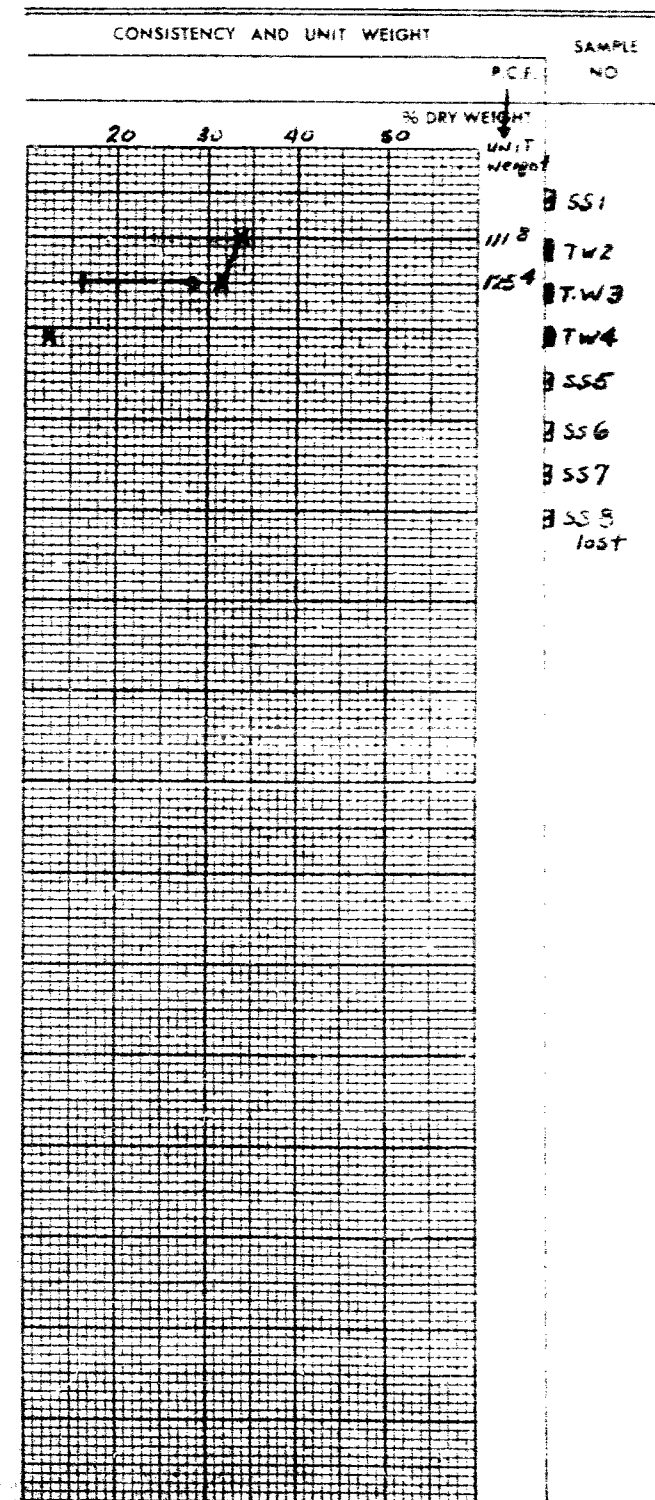
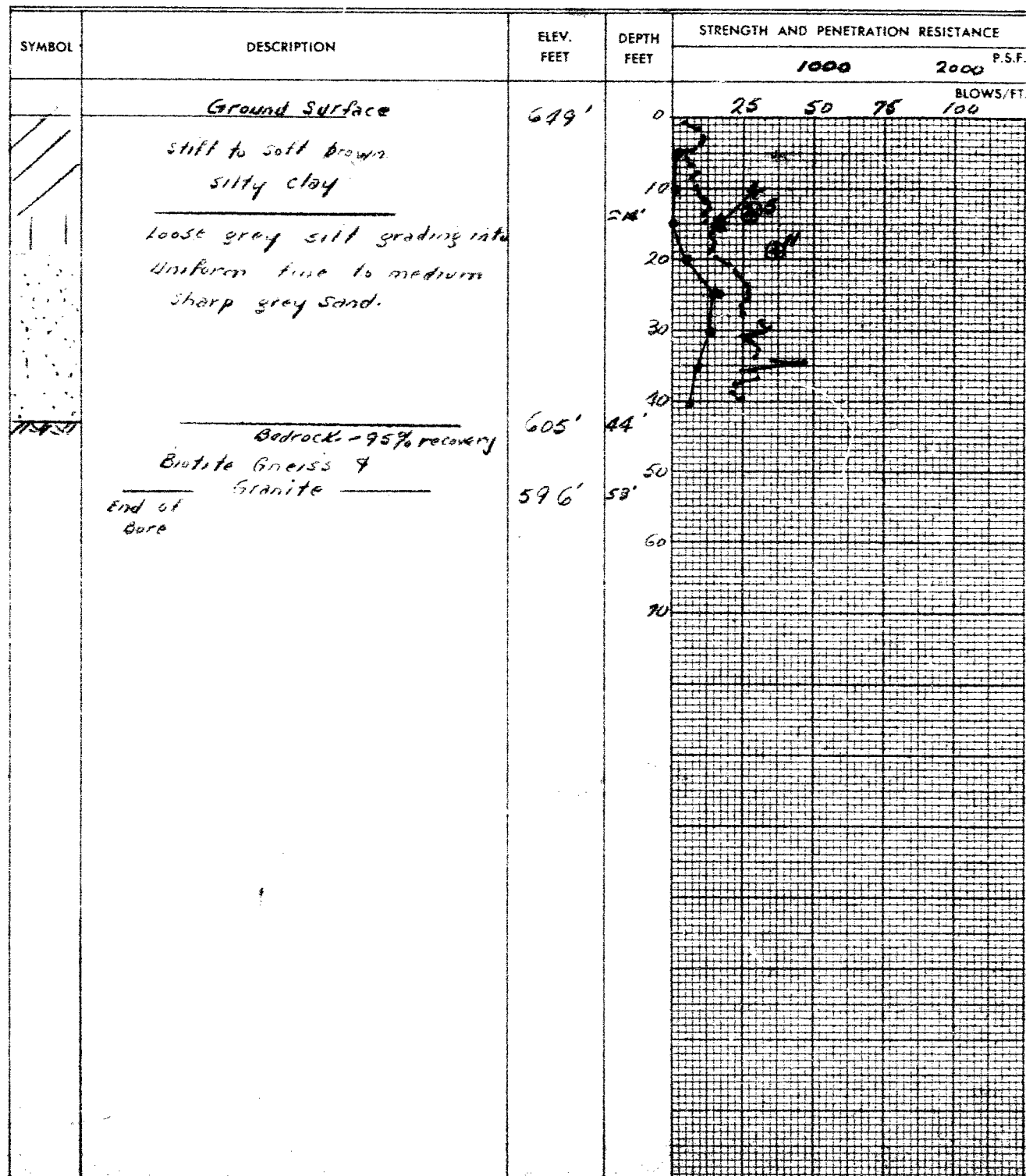
**Consistency**

Natural moisture

Liquid limit

Plastic limit

Natural Unit Weight



**RACEY MacCALLUM AND ASSOCIATES LTD.**

Foundation Engineering Division

Engineering Data Sheet for Borehole: 2

Project: *PROPOSED SLY CREEK CROSSING*

Location: HWY 69 REVISION @ 7 MI. NORTH PARRY SOUND

Hole Location *See FIRST*

Hole Elevation and Datum: 647<sup>5</sup>

Field Work Begun 27-9/56 Ended 2-10/56

Field Supervision: *H. G.*

Driller: *BJP*

Prep.:

Checked:

Date:

### LEGEND

### Sampling Method

2" Dia. split tube

2" Shelby tube

### Penetration Resistance

2" Split tube

2" Dia. Cone

### Casing

### Strength

Unconfined compression

### Yane test and sensitivity

### Consistency

Natural moisture

Liquid limit

Plastic limit

Natural Unit Weight

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE	
				1000 P.S.F.	2000 P.S.F.
	Ground surface	6475	0	25	50
	stiff to soft brown silty clay		10	75	100
	soft slightly cohesive silt grading into uniform fine to medium sharp sand (Some small gravel at 20' and over)		20		
	Bedrock 75% recovery	618	29.5		
	Quartzite Gneiss & Granite		40		
	End of Bore	608	39.5		

CONSISTENCY AND UNIT WEIGHT		P.C.F.	SAMPLE NO.
20	30	40	50
% DRY WEIGHT			
UNIT WEIGHT			
NO. 7			
			T.W.-1
			T.W.-2.
			SS 3
			SS.4
			SS 5