

**Golder Associates Ltd.**

2390 Argentia Road  
Mississauga, Ontario, Canada L5N 5Z7  
Telephone: (905) 567-4444  
Fax: (905) 567-6561



**REPORT ON**

**FOUNDATION INVESTIGATION AND DESIGN  
SOUTHBOUND HIGHWAY 69 / BLAIR (SLY) CREEK 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:

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

GEOCRES No. 41H-54

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**PART A**

**FOUNDATION INVESTIGATION REPORT  
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**Golder Associates**

## **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 new Southbound Highway 69 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 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 southbound structure at Blair Creek was provided to Golder by URS on January 11, 2005.

This report addresses the investigation for the proposed Southbound Highway 69 structure over Blair (Sly) Creek and the associated approach embankments and potential retaining wall; however where relevant, boreholes and dynamic cone penetration tests (DCPTs) advanced for the swamp crossing investigation immediately adjacent to this area have been utilized and are included as part of this report. Separate reports detail the foundation investigations for the related 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 structure 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 location of the investigated area is shown in plan on Drawing 1A.

## **2.0 SITE DESCRIPTION**

The site is located approximately 80 m east of existing Highway 69 (north of Nobel, Ontario) at the proposed southbound Highway 69 crossing of Blair (Sly) Creek, approximately 300 m north of existing Highway 559 (as shown on Figure 1). An oblique aerial photograph of Blair Creek in this area has been included in Appendix B.

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 southbound Highway 69 alignment is diverted away from the median centreline in this area so that the southbound structure over Blair (Sly) Creek crosses near perpendicular to the natural alignment of the creek (i.e. avoiding a meander/bend in the creek that would require a greater span than that of the proposed structure). The area surrounding the floodplain of the creek is a relatively low-lying grassland containing well spaced trees and small shrubs to the south of the creek and transitioning into a low-lying, wooded area with open water in areas to the north of the creek. The embankment of a narrow, abandoned access road / trail was found in the vicinity of the site passing through the area of the proposed north abutment. The existing ground surface within the limits of the proposed structure and approach embankments generally lies between Elevation 198 m and Elevation 195 m (approximate creek bed elevation), referenced to Geodetic Datum.

### **3.0 INVESTIGATION PROCEDURES**

#### **3.1 Foundation Investigation**

The field work for the Southbound Highway 69 bridge structure investigation was carried out between November 3 and December 4, 2004 during which time a total of four (4) sampled boreholes (SBL-1 to SBL-4) were put down at the site. One borehole was drilled at each of the proposed south and north foundation element locations and one borehole was advanced to refusal within the footprint of each of the proposed south and north approach embankments. All of the boreholes were advanced to refusal on inferred bedrock. In the boreholes at the foundation element locations, bedrock coring was carried out to a minimum depth of 3 m. In the area of the potential retaining wall (located adjacent to the south abutment), three (3) boreholes and two (2) dynamic cone penetration tests (DCPTs) advanced as part of the swamp crossing investigation to the south of Blair (Sly) Creek in this area have also been utilized. The logs for these relevant boreholes and DCPTs are included as part of this report, however, the details of the investigation procedures at these locations are described in a separate report.

The field investigation was carried out using a track-mounted CME 55 drill rig supplied and operated by Marathon Drilling Co. Ltd. of Ottawa, Ontario. The boreholes put down with the drill rig were advanced using 108 mm inside diameter (I.D.) continuous flight hollow stem augers. Soil samples were obtained, where possible, continuously or at intervals of about 0.75 m to 1.5 m depth, using a 50 mm outer diameter (O.D.) split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586-99) or using a 76 mm O.D. thin-walled 'Shelby' tube (ASTM D1587-00) for relatively undisturbed samples in cohesive soils. Field vane shear tests were conducted in cohesive soils for assessment of undrained shear strengths (ASTM D 2573-01). Samples of the bedrock were obtained using an 'NQ' size rock core barrel.

Borehole SBL-2 was advanced in the floodplain of Blair (Sly) Creek which was identified as an environmentally sensitive area by the Ministry of Natural Resources (MNR). Special provisions were taken to access this borehole location and to contain the sediment created by drilling activities. As approved by the MNR, swamp mats were used to create a temporary access road from the south bank into the floodplain, thereby minimizing rutting of the soft ground and disturbance to vegetation. In addition, a temporary silt fence was placed between the floodplain waters (up to about 0.6 m deep) where the drilling took place and the main channel of the creek to contain the flow of sediment from the borehole. Following completion of drilling operations and the settling of sediment, the silt fence and swamp mats were removed.

The boreholes and DCPTs at this location were advanced to auger and/or SPT or DCPT testing refusal (i.e. inferred bedrock) which occurred at depths ranging from about 3.1 m to 11.4 m below

the existing ground surface (not including rock coring). At boreholes SBL-2 and SBL-3, located within the footprints of the proposed foundation units, the drilling was further advanced into the bedrock by coring 3.3 m and 3.4 m, respectively. The groundwater level in the open boreholes was observed throughout the drilling operations and a piezometer was installed in SBL-4 to permit monitoring of the groundwater level at this location. The piezometer installed within the borehole consisted of 38 mm O.D. threaded PVC tubing with a slotted screen at depth surrounded by a sand filter and sealed with bentonite above the filter to ground surface. The installation details and water level readings are described on the Record of Borehole sheets that follow the text of this report. The piezometer and all boreholes were abandoned in accordance with O.Reg. 128 (amendment to O.Reg. 903). The piezometer was 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 services/utilities, supervised the set-up of the drill rig in the floodplain of the creek, supervised 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. In addition, the results of a consolidation (oedometer) test carried out as part of the investigation and design of the immediately adjacent swamp crossing (i.e. on a Shelby tube sample from a borehole located approximately 80 m south of Blair (Sly) Creek) were also utilized. Strength testing such as point load index were carried out on specimens from the rock core.

The borehole locations within the foot print of the proposed structure foundations were located in the field by Callon Dietz prior to drilling operations. The locating of the other boreholes and DCPTs and the surveying of the elevations of all of the as-drilled boreholes was carried out by members of our engineering staff, referenced to benchmark geodetic elevations provided by URS. 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). 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). Deposition of Paleozoic strata and later erosion during glaciation left behind these Precambrian rocks covered only in a few places by the flat-lying Palaeozoic bedrock strata.

### **4.2 Subsurface Conditions and General Overview**

The detailed subsurface soil and groundwater conditions as encountered in the boreholes 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 from the boreholes at the proposed Southbound Highway 69 structure over of Blair (Sly) Creek and at the associated potential retaining wall location is shown on Drawings 1A and 1B.

In general, the subsoils at the structure site consist of topsoil, peat and/or organic sandy silt underlain by successive deposits of sandy silt to silty sand (upper); silty clay to clay; silt to sandy silt to sand and silt (lower); and sand over bedrock. The total overburden thickness at the investigated locations ranges from about 3.1 m (south of Blair Creek, at the south end of the retaining wall) to up to about 11.4 m (north of Blair Creek). All of the boreholes were terminated at the inferred bedrock surface; with the exception of the two (2) boreholes at the proposed abutment foundation areas which were cored at least three metres into the bedrock.

A more detailed description of the subsurface conditions encountered in the boreholes (SBL-1 to SBL-4, BH04-73, BH04-75 and BH04-105) is provided in the following sections.

#### **4.2.1 Topsoil, Fibrous Peat and Organic Sandy Silt to Silty Sand**

A dark brown to black topsoil, fibrous peat and/or organic sandy silt to silty sand layer was encountered in all of the boreholes, except SBL-2 (located within the flood plain of Blair Creek). In most of the boreholes, this layer was encountered at the ground surface, however, in borehole SBL-3 it was encountered below a layer of silty sand fill. The top of this layer (i.e. ground surface) ranged from about Elevation 197.1 m to 197.8 m and the thickness ranged from about 0.2 m to 2.3 m.

Standard Penetration Testing (SPT) measured 'N' values ranging from 0 blows (weight of hammer) to 2 blows per 0.3 m of penetration, indicating a very loose relative density or very soft to soft consistency.

Natural water contents measured on two (2) samples of the organic sandy silt to silty sand were 26 percent and 108 percent.

#### **4.2.2 Silty Sand Fill**

A brown silty sand fill containing trace organics and rootlets was encountered near the ground surface in borehole SBL-3 adjacent to the north bank of Blair (Sly) Creek. The top of this layer was at about Elevation 197.3 m with a thickness of about 0.5 m. This fill layer may be the remnants of an old access road / trail as discussed in Section 2.0.

A single Standard Penetration Testing (SPT) measured an 'N' value of 3 blows per 0.3 m of penetration, indicating a very loose relative density.

#### **4.2.3 Sandy Silt to Silty Sand (Upper)**

A light brown, oxidized or grey silty sand to sandy silt deposit containing trace to some organics, trace clay and trace gravel was encountered below the topsoil, peat or organic layers in all of the boreholes except borehole SBL-3 (where it was not found) and borehole SBL-2 where it was found at the surface of the creek bed/floodplain. The top of this deposit ranged from about Elevation 197.5 m to 196.1 m and the thickness ranged from about 0.3 m to 1.5 m.

Standard Penetration Testing (SPT) carried out within this stratum measured 'N' values ranging from 1 blow to 10 blows per 0.3 m of penetration indicating a very loose to loose relative density.

The natural water content measured on three (3) samples of this deposit ranged from about 31 percent and 38 percent with an average of about 34 percent.

#### **4.2.4 Silty Clay to Clay**

A deposit of reddish brown to grey, silty clay to clay some silt containing trace sand and organics was encountered below the upper sandy silt to silty sand in all of the boreholes except SBL-3 where it was encountered below the organic sandy silt to silty sand. The structure of this soil deposit was noted in places to be mottled and/or varved and to contain occasional thin sand seams. The top of this stratum varied between Elevation 197.0 m and 194.5 m and the thickness ranged from about 2.2 m to 4.8 m.

Standard Penetration Testing (SPT) carried out within this stratum measured 'N' values ranging from 0 blows (i.e. weight of hammer) to 10 blows per 0.3 m of penetration, but typically less than 1 blow per 0.3 m of penetration.

In situ field vane testing carried out within this stratum measured undrained shear strengths ranging from about 5 kPa to 54 kPa, with an average of about 23 kPa. Sensitivity was found to range from about 2.9 to 11.6, typically less than 7.0. In general, the field vane test results together with the SPT 'N' values suggest the silty clay to clay stratum has a very soft to stiff consistency.

The natural water content measured on samples of this deposit ranged between 23 percent and 89 percent with an average of about 53 percent. The lower water contents measured within the stratum are attributed to the presence of siltier materials at depth.

Atterberg limits testing was carried out on eight (8) samples of the silty clay to clay. The liquid limit ranged from about 24 to 88 percent and the plastic limit ranged from about 15 to 25 percent yielding a plasticity index ranging from about 9 to 64 percent. The results of the Atterberg limits testing are shown on the plasticity chart on Figure A-1 in Appendix A and indicate that the material is typically silty clay of intermediate plasticity to clay of high plasticity becoming clayey silt of low plasticity at depth within the deposit.

#### **4.2.5 Silt, Sandy Silt to Sand and Silt (Lower)**

Beneath the silty clay to clay in boreholes SBL-3, SBL-4 and 04-105, a grey silt, sandy silt to sand and silt deposit was encountered. The top of this deposit ranged from about Elevation 192.5 m to 191.6 m and the thickness ranged from about 0.9 m to 1.5 m.

Standard Penetration Testing (SPT) carried out within this deposit measured 'N' values of 0 blows (i.e. weight of hammer) to 19 blows per 0.3 m of penetration indicating a very loose to compact relative density.

The natural water content measured on three (3) samples of this deposit ranged from 17 to 26 percent with an average of about 21 percent.

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

#### **4.2.6 Sand**

A grey to brown sand deposit containing trace to some gravel and trace to some silt was encountered below the silty clay to clay deposit in boreholes SBL-1, SBL-2, 04-73 and 04-75 and below the silt, sandy silt to sand and silt in boreholes SBL-3, SBL-4 and 04-105. The top of this stratum ranged from about Elevation 194.3 m to 190.7 m and the thickness ranged from about 0.2 m to 4.4 m. The bottom of this deposit was defined by refusal to further auger advancement and was confirmed by rock coring in select boreholes.

Standard Penetration Testing (SPT) carried out within this stratum measured 'N' values of 5 blows to 30 blows per 0.3 m of penetration, typically greater than 10 blows per 0.3 m of penetration. The 'N' values indicate a loose to compact relative density within the deposit.

The natural water content measured on samples of this deposit ranged between 8 percent and 23 percent.

Grain size distributions for three (3) samples from this deposit are shown on Figure A-3 of Appendix A.

#### **4.2.7 Bedrock**

Bedrock was encountered and cored in boreholes SBL-2 and SBL-3 located within the footprint of the proposed south and north abutment foundation, respectively. The presence of bedrock was inferred from refusal to further drilling or split spoon sampler / dynamic cone penetration in the other boreholes and DCPTs. At the borehole and DCPT locations, the bedrock surface ranges from as high as Elevation 194.1 m south of Blair (Sly) Creek to as low as Elevation 186.3 m north of the creek.

The bedrock samples are described as fresh to slightly weathered, light grey, pink and black, fine to coarse grained, non-porous to slightly porous granitic gneiss containing near horizontal, distinct foliation. The Total Core Recovery measured on the core samples was 100 percent. The Rock Quality Designation (RQD) measured on the core samples ranged from 47 percent to 100 percent, typically greater than 75 percent, indicating a rock mass of fair 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. Axial point load strength index values ( $Is_{50}$ ) ranged from 7.2 MPa to 10.2 MPa and diametral point load strength index values ranged from 4.7 MPa to 6.5 MPa, indicating a very strong rock mass. A summary of the point load index values on the rock core from the two 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 estimated Unconfined Compressive Strength (UCS) value for each test.

<b>Borehole (Drillhole) No.</b>	<b>Average Axial Point Load Index (MPa)</b>	<b>Average Diametral Point Load Index (MPa)</b>
SBL-2	8.3	5.4
SBL-3	9.6	5.5

#### **4.2.8 Groundwater Conditions**

In general, the samples taken in the overburden boreholes were noted to be moist to wet. The water levels in the open boreholes upon completion of drilling ranged between Elevation 195.1 m to 196.5 m. The groundwater level in the piezometer installed at the soil / bedrock interface in borehole SBL-4 was measured at Elevation 196.6 m (1.1 m depth) on November 14, 2004 and January 4, 2006. The water level of Blair (Sly) Creek was measured at Elevation 196.2 m in July 2004 and at Elevation 196.5 m in December 2004. Details of the piezometer installation, 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.

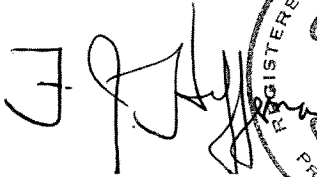
### 4.3 CLOSURE

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.

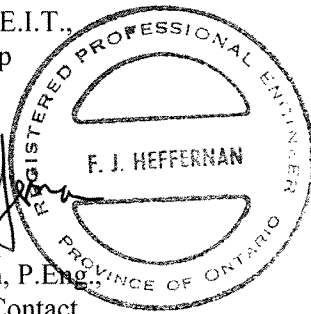
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Chad M. Gilfillan, E.I.T.,  
Geotechnical Group



Fintan J. Heffernan, P.Eng.,  
Designated MTO Contact



J. Paul Dittrich, Ph.D., P.Eng.,  
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**PART B**

**FOUNDATION DESIGN REPORT  
SOUTHBOUND HIGHWAY 69 / BLAIR (SLY) CREEK 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 Southbound Highway 69 bridge 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 Southbound Highway 69 structure will consist of a single-span, slab-on-girder bridge with a 39 m span length and with abutments located north and south of Blair (Sly) Creek.

Based on the information provided on the General Arrangement (GA) Drawing provided by URS on January 11, 2005, the grade of the proposed southbound Highway 69 bridge deck varies between about Elevations 201.0 m and 201.2 m, while the high water level (HWL) for Blair (Sly) Creek has been estimated at Elevation 196.75 m. The proposed approach embankments will be up to about 4.5 m in height relative to the existing ground surface at both the south and north sides of the bridge. The existing ground surface varies from about Elevation 197.8 m to 196.5 m at the borehole and DCPT locations.

It is our understanding that the existing Blair (Sly) Creek, over which the proposed new Southbound Highway 69 structure will cross, is considered to be environmentally sensitive. A retaining wall, approximately 35 m in length and 4.5 m in height, extending beyond the limit of the west wing wall of the south abutment has been proposed as an option to minimize the encroachment of the approach embankment toe into the creek in this area. However, it is understood that consideration is also being given to locally re-aligning the creek into an old channel meander in the area so that a retaining wall may not be required. The recommendations given in the following sections have taken both options of a retaining wall and a local re-alignment of the creek into consideration.

## **5.2 Bridge Foundation Options**

The native soils at the bridge site consist of topsoil, peat and/or organic sandy silt underlain by successive deposits of sandy silt to silty sand (upper); silty clay to clay; silt to sandy silt to sand and silt (lower); and sand over bedrock. The total overburden thickness at the investigated locations near the bridge ranges from about 7 m to 9 m (south of Blair Creek) to about 11 m (north of Blair Creek). The overburden thins out to the south of the bridge (to less than 4 m thick) along the length of the proposed retaining wall area. The native overburden soils are underlain by very strong granitic gneiss bedrock. The bedrock surface at the proposed bridge foundation units, as established at the borehole locations, ranges from about Elevation 187.8 m at the south abutment to about Elevation 186.8 m at the north abutment.

For both the north and south foundation elements, spread footings founded at shallow depth on either the very loose to loose sandy silt to silty sand or on the soft to stiff 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 lower sand or bedrock are also not recommended due to the deep excavation, groundwater control and the temporary shoring that would be required.

Abutment footings perched within the embankment fill (i.e. on well compacted granular) constructed on the native soils is not considered a suitable alternative at either foundation element due to the compressible nature of the underlying foundation soils which would result in post-construction settlement of the footings. Footings constructed on replacement fill (i.e. following sub-excavation and removal of the clayey foundation soils) could be considered. However, a rock fill sub-base would also be subject to post-construction total and differential settlements and is therefore not deemed suitable for support of the bridge foundations. Granular fill placed in compacted lifts would be less susceptible to post-construction settlements, however, this work would need to be carried out in the dry and would require extensive dewatering which may be cost prohibitive.

As such, it is considered that designing the north and south foundation elements as integral abutments supported on piles driven to bedrock is the most feasible option.

The details of the recommendations for this option are presented in the following sections. A summary of the advantages/disadvantages, relative costs and risks/consequences of all of the various alternatives considered for this site is presented in Table 2 following the text of this report.

### 5.3 Steel H-Pile Foundations

As noted in Section 5.2, steel H-piles driven to refusal on the granitic gneiss bedrock may be used for support of an integral abutment at both the north and south sides of the proposed 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 NSSP detailing the gradation of this sand should be included in the Contract Documents (see example in Appendix C).

For design, the following range of pile tip elevations may be assumed for piles terminating on the bedrock surface. The range in elevation has been assessed based on a review of the depth to bedrock as encountered in boreholes put down at, and immediately adjacent to, the area of the north and south abutments. There should be a provision made in the Contract for dealing with varying pile lengths.

<i>Foundation Unit</i>	<i>Design Pile Tip Elevation (m)</i>
South abutment	187 - 188
North abutment	186 - 187

#### 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 an unyielding material; as such, ULS conditions will govern for this foundation type.

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

The loading due to the north and south approach embankment construction will cause consolidation settlement of the underlying soft to stiff silty clay to clay strata if it is not removed as part of the embankment settlement mitigation measures (as discussed in Section 5.7). 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 approximately 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 load acting on the piles. The estimated unfactored downdrag load acting on the HP 310x110 piles for this case are shown in the table below.

<i>Foundation Unit</i>	<i>Unfactored Downdrag Load per Pile (kN)</i>
South abutment	75
North abutment	65

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 either removing and replacing the clayey subsoils or by constructing preload embankments in the abutment area (as discussed in Section 5.7) and allowing the settlement to occur prior to installing the piles.

### **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 310x110 piles embedded in the clayey strata for this case are shown in the table below.

<i>Abutment Location</i>	<i>Soil Unit</i>	<i>Elevation (m)</i>	<i>Unfactored Lateral Load, <math>P_h</math> (kN/m length)</i>
South	Firm Silty Clay	194.8 – 193.5	70
	Soft to Firm Silty Clay	193.5 – 191.9	55
North	Soft Silty Clay	194.5 – 193.5	35
	Firm Silty Clay	193.5 – 192.4	70

Lateral loads on the piles can be reduced or eliminated by either removing and replacing the clayey subsoils or by constructing preload embankments in the abutment area (as discussed in Section 5.7) and allowing the settlement and lateral movement to occur prior to pile installation.

#### 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 potentially 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.

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 potentially 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, potential for horizontal loads on the piles due to lateral soil deformations, 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 integral abutment foundations are being considered for the both the north and south abutments of the bridge. Where very 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. It is our understanding that the installation of CSP liners is not being considered at this site due to the very loose sandy silts and soft to firm clays present below the proposed pile cap levels.

Based on the proposed elevations of the underside of the north and south abutment pile caps as shown on the GA drawing provided by URS (on January 11, 2005) and considering the depth to bedrock encountered in the boreholes at these locations, the total length of the H-piles will be approximately 7.2 m and 9.1 m at the south and north abutment, respectively.

For the relatively long HP 310 x 110 piles driven to bedrock through the soft to firm clays and very loose to compact 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:**

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

**For cohesionless soils:**

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

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:

<i>Abutment Location</i>	<i>Soil Unit</i>	<i>Elevation (m)</i>	<i><math>s_u</math> (kPa)</i>	<i><math>n_h</math> (MPa/m)</i>
South	Very loose Silty Sand	195.0 – 194.8	-	1.3
	Firm Silty Clay	194.8 – 193.5	25	-
	Soft to Firm Silty Clay	193.5 – 191.9	15 (at top) to 25 (at bottom)	-
	Loose to Compact Sand	191.9 – 187.8	-	4.4
North	Very loose Sandy Silt	195.9 – 194.5	-	1.3
	Soft Silty Clay	194.5 – 193.5	15 (at top) to 10 (at bottom)	-
	Firm Silty Clay	193.5 – 192.4	25	-
	Very loose Sandy Silt	192.4 – 190.9	-	1.3
	Compact Sand	190.9 – 186.8	-	4.4

For a single HP 310 x 110 pile embedded about 7.2 m and 9.1 m into the soft to firm silty clay and very loose to compact silt and sand at the south and north abutments respectively, 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 (7.2 m long pile)	120	35	50
North (9.1 m long pile)	140	45	75

It should be noted that the above values are based on the assumption that the pile cap is located at Elevation 195.0 m and 195.9 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.

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 d = Pile Diameter</i>	<i>Subgrade Reaction Reduction Factor (R)</i>
8d	1.00
6d	0.70
4d	0.40
3d	0.25

The proposed pile spacing at the north abutment has not been provided by URS. The subgrade reaction reduction factor should be interpolated for pile spacings in between those listed in the table above.

### 5.3.7 Frost Protection

All pile caps and footings founded on frost susceptible soils 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 give 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 beneath and extending to a distance of 1.8 m beyond the perimeter of each foundation unit. Where insulation is to be placed below spread footings, the insulation product will require an unconfined compressive strength of at least three times the maximum allowable bearing pressure/foundation contact stress in order to minimize potential for long-term creep deformation of the polystyrene insulation.

#### **5.4 Retaining Wall (at South Approach)**

As discussed in Section 5.1, a retaining wall may be required at the southwest corner of the bridge structure. The retaining wall would extend approximately 35 m behind the south abutment wall on the west side of the approach to limit the encroachment of embankment fill into the immediately adjacent Blair Creek. The wall height would range from about 4.0 m to 4.5 m in height at this location.

At this site, the very soft to firm nature of the silty clay subsoils and the presence of the creek, assuming it is not re-aligned, control the global stability of the proposed retaining wall (as well as the approach fills as will be discussed in Section 5.7) and affect the choice of retaining system and associated wall foundation support.

For a retaining wall supported on shallow foundations at the proposed height/geometry, the Factor of Safety (FoS) for global stability is estimated to range from less than 1.0 (at the south end of the wall) to less than 1.2 (at the north end of the wall) for the existing foundation soils. A minimum target FoS of 1.3 is normally used in design for the global stability of retaining walls under static conditions. If the creek could not be re-aligned, even if staged construction and/or preloading of the embankment was possible (i.e. if there were no fill or space restrictions caused by the adjacent creek and large toe berms could be employed to stabilize a preload fill), the strength gain anticipated to occur would not be sufficient to achieve a  $FoS \geq 1.3$  for the global stability of the wall. As such, a conventional retaining wall supported on spread footings is not suitable at this location unless the existing very soft to soft foundation soils are excavated and replaced.

As a result of the weak foundation soils and the global stability issues at this location, the alternatives that can be considered for design of the retaining wall are limited to the options listed

below. Regardless of the design methodology adopted to address the global stability issue, a mechanically-reinforced soil retaining wall system (retained soil system or RSS wall) would not be recommended for this location due to the potential for undermining of the wall and erosion of the backfill by the immediately adjacent Blair Creek and due to the potential for differential settlement between the RSS wall and the pile supported bridge abutment. As such, in all cases, a concrete retaining wall is considered the most suitable option for this site. The type of foundation that could be considered for a concrete wall is dependent on the treatment adopted for the foundation soils as listed below.

**I. Maintain Existing Foundation Soils:**

- 1) Support on near-Continuous Caisson Wall
- 2) Support on H-Piles with Partial Preload and Partial EPS Fill
- 3) Support on H-Pile with Full EPS Backfill

**II. Sub-excavate and Replace Existing Foundation Soils:**

- 4) Support on Shallow Foundations Founded on Rock Fill
- 5) Support on H-Piles Driven through Fill

The following sections discuss the design recommendations for each of the above options. The advantages, disadvantages, relative costs and risks / consequences for the different wall options are summarized in Table 3. It should be noted that the recommendations in these sections are based on the assumption that the existing Blair Creek will not be locally re-aligned.

**5.4.1 Maintain Existing Foundation Soils**

If sub-excavation and removal of the weak foundation soils can not be carried out (i.e. due to the environmental restrictions associated with the immediately adjacent creek) the concrete retaining wall will have to be supported on a deep foundation system (i.e. steel H-piles or concrete caissons) to achieve global stability and limit settlements of the wall. The choice of foundation system depends on the amount of preloading that is carried out and the type of backfill employed behind the wall. The details of these sub-options are described below.

**5.4.1.1 Support on near-Continuous Caisson Wall**

For this option, the retaining wall (and south bridge abutment) could be constructed prior to the placement of any conventional backfill. However, in order to maintain the global stability of the wall/west side of the approach fills, the retaining wall has to be designed to resist the lateral spread of the soft foundation soils and to support the full active pressures from the embankment fill. To satisfy this requirement, 1 m diameter bored piles/concrete caissons installed at close spacing and fixed to the granitic gneiss bedrock with grouted dowels are recommended to support

the wall. Consideration could also be given to the use of caissons to support the bridge abutments and any associated wing walls.

The required caisson lengths/depth to bedrock along the wall can be estimated from the stratigraphic information provided on Drawing 1B. Depending on the depth of the pile cap, the caisson lengths along the wall could range from about 8.5 m (at the north end of the wall) to about 3.5 m (at the south end of the wall). To provide sufficient lateral support of the soft clayey subsoils under embankment fill loading and facilitate the arching of the soils across the caissons (to minimize the chance of rotation of the wall caused by plastic flow of the soil around the foundations), it is recommended that the caissons be spaced (centre-to-centre) at a maximum of two (2) times their diameter.

It is noted that some of the native soils at the site are cohesionless and water-bearing; these soils will flow into the auger hole during bored pile installation if left unsupported. The use of a temporary liner or casing, possibly in conjunction with drilling mud, will be required to advance the caissons with minimal loss of ground.

The granitic gneiss bedrock at the site is classified as very strong with fair to excellent quality. As such, socketting of the caissons into the bedrock (to provide sufficient resistance to lateral forces and overturning moments acting on the wall) will be very difficult and may not be practical even with rock coring and/or churn drilling. It is therefore recommended that the required resistances (to shear forces and bending moments at the base of the caissons) be provided by installing grouted dowels. To provide sufficient space for the required number of dowels, a 1 m diameter caisson is recommended for support of the wall.

The advantage of this option is that there is no requirement to delay the construction of the retaining wall and bridge abutment until the preloading of the embankment fill is completed. The disadvantage is that the pile foundations for the retaining wall and abutment must be designed to support the global stability of the wall and to withstand the full lateral loads due to horizontal soil deformations and full downdrag loads.

#### **5.4.1.1.1 Concrete Caissons - Axial Geotechnical Resistance**

Caissons founded on the surface of the sound granitic gneiss bedrock should be designed based on end-bearing resistance and a factored geotechnical resistance at ULS of 20 MPa should be used. Serviceability Limit State resistances do not apply to caissons founded on the bedrock, since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS. This capacity will have to be reviewed after the caisson configuration is established so that group effects for axial loading may be taken into account.

#### 5.4.1.1.2 Downdrag Load (Negative Skin Friction)

The new loading created by the south approach embankment construction (i.e. the conventional fill behind the retaining wall) will cause consolidation settlement of the underlying soft to firm silty clay strata. Since the caissons would be installed prior to completion of this settlement and because they are end-bearing on bedrock, a small amount of settlement of the clay relative to the stiff caisson will result in the development of negative skin friction. In this case, downdrag loads will need to be taken into account for design of the caissons supporting the retaining wall.

Where the clayey foundation soils remain in place and are not preloaded, the retaining wall caisson structural design should be based on the full downdrag load acting on the piles. The estimated unfactored downdrag load acting on the proposed 1 m diameter caissons for this site are shown in the following table:

<i>Location</i>	<i>Unfactored Downdrag Load per Pile (for 1 m diameter caisson) (kN)</i>
South End of Retaining Wall (assumed 3.5 m long caisson)	150
North End of Retaining Wall (assumed 8.5 m long caisson)	350

Note: downdrag loads on caissons at intermediate locations (with lengths between 3.5 m and 8.5 m) can be interpolated from the above information

The downdrag loads calculated in this manner are unfactored loads. The structural capacity of the caissons must be checked for the factored dead and downdrag loads in accordance with Section 6.8.4 of the *CHBDC* for ULS conditions. The caissons 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.

#### 5.4.1.1.3 Lateral Loads (due to Horizontal Soil Deformations)

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

Where the clayey foundation soils remain in place and are not preloaded prior to caisson installation, the retaining wall caisson structural design should include additional lateral loads acting on the piles. The estimated unfactored lateral load acting on the portion of the 1.0 m diameter caissons embedded in the clayey strata for the caissons installed at close spacing (i.e. centre to centre spacing of 2 times the diameter) 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 End of Wall	Soft Silty Clay	196.0 – 193.5	150
North End of Wall	Firm Silty Clay	196.0 – 195.0	350
	Soft to Firm Silty Clay	195.0 – 193.5	225
	Very Soft to Soft Silty Clay	193.5 – 192.0	150

Note: lateral loads estimated based solutions proposed by Broms (1964) for design of lateral capacity of piles

#### 5.4.1.1.4 Resistance to Lateral Loads

The design of caissons subjected to lateral loads should take into account such factors as those described in Section 5.3.6. For design purposes, both the structural and geotechnical resistances should be evaluated to establish the governing case. Based on the proposed location of the retaining wall shown on the GA drawing provided by URS (on January 11, 2005) and considering the depth to bedrock encountered in the boreholes at this location, the total length of the caissons are estimated to range from approximately 8.5 m to 3.5 m at the north end and south end of the wall, respectively.

For the longer caissons installed at the north end of the wall through the soft to firm clays and very loose to compact silts and sands, the horizontal resistance at Ultimate Limit States (ULS) may 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. For the shorter caissons installed at the south end of the wall the horizontal resistance at Ultimate Limit States (ULS) may be controlled by the lateral capacity of the soil and/or rock adjacent to the pile and grouted dowels. In this case, as described in the Canadian Foundation Engineering Manual (1992), the lateral loading may exceed the capacity of the soil, resulting in large horizontal movements of the piles.

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 formulae presented previously in Section 5.3.6 depending on the soil type supporting the pile. For the horizontal reaction provided by the bedrock, the values of  $k_h$  provided in the table below may be used directly. The following ranges for the values of  $s_u$  and  $n_h$  may be used to estimate the values of  $k_h$

for the different soil types as required for the structural analysis to estimate the factored lateral resistances at SLS. The range in values reflects the variability in the subsurface conditions:

<i>Abutment Location</i>	<i>Soil Unit</i>	<i>Elevation (m)</i>	<i>s<sub>u</sub> (kPa)</i>	<i>n<sub>h</sub> (MPa/m)</i>
South End of Wall	Very loose Sandy Silt	197.0 – 196.0	-	1.3
	Soft Silty Clay	196.0 – 193.5	15	-
	Loose to Compact Sand	193.5 – 192.5	-	4.4
	Bedrock	Below 192.5	k <sub>h</sub> = 11.5 GPa/m	
North End of Wall	Very loose Silty Sand	197.0 – 196.0	-	1.3
	Firm Silty Clay	196.0 – 195.0	40	-
	Very Soft to Soft Silty Clay	195.0 – 193.5	25	-
	Soft to Firm Silty Clay	193.5 – 192.0	18	-
	Compact Sand	192.0 – 188.0	-	4.4
	Bedrock	Below 188	k <sub>h</sub> = 11.5 GPa/m	

Upon completion of the preliminary structural design of the caissons for the retaining wall, it is recommended that Golder check the design by carrying out a soil/structure interaction analysis to estimate the lateral performance of the wall/caissons and check if there is sufficient number of, and length of, rock dowels for the caissons (i.e. considering the field/mass strength of the bedrock). It should be noted that the preliminary analysis carried out to date to provide an estimate of the minimum required rock dowel length (i.e. minimum 1 m long) was based on the assumption that a 3 m wide by 2.4 m high triangular rock fill berm is constructed on the west side of the proposed retaining wall along its full length (i.e. as indicated on the General Arrangement drawing provided by URS). The passive resistance provided by this berm was found to be a key factor in the design.

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 described in Section 5.3.6.

#### **5.4.1.1.5 Lateral Earth Pressures for Design**

The recommendations on lateral earth pressures provided in Section 5.6 can be employed for the design of the retaining wall.

The passive earth pressure available from the rock fill berm to be constructed on the west side of the wall can be calculated based on the following parameters:

	Rock Fill
Soil unit weight:	19 kN/m <sup>3</sup>
Coefficients of static lateral earth pressure:	
Passive, $K_p$	0.4

It should be noted that the value of  $K_p$  is low for this situation as a result of the geometry of the proposed rock fill in this area (i.e. a small triangular berm 3 m wide by 2.4 m high). It is also noted that the given value of  $K_p$  includes the effects of wall friction and has been factored down (by 1.5) to account for the fact that a large passive strain would be required for full mobilization of the passive resistance.

#### **5.4.1.1.6 Settlements and Preload Period**

Following completion of construction of the retaining wall (and abutment wall), the backfill / embankment fill should be placed up to the final grade (i.e. top of pavement) required for the approach. Based on the results of settlement analysis (described in Section 5.4.2.1.1) for the up to 4.5 m high backfill behind the wall, the total long-term (i.e. post-construction) consolidation settlement of the cohesive foundation soils in the area of the proposed wall is estimated to range from about 210 mm at the south end of the wall to about 125 mm at the north end of the wall. In addition, up to about 20 mm of immediate settlement is expected due to compression of the cohesionless strata. It is estimated that the preload fills will have to remain in place for about 16 months to achieve 90 percent consolidation of the up to 4.5 m thick silty clay strata in this area. Following completion of the preload period, the embankment fills can be regraded (and removed as required) to allow construction of the final pavement structure. Creep settlement of the clayey strata (on the order of about 20 mm per log-cycle of time) should be expected following completion of construction.

#### **5.4.1.2 Support on H-Piles with Partial Preload and Partial EPS Fill**

For this option, prior to the retaining wall (or bridge abutment) construction, a portion of the wall backfill/approach embankment would initially be constructed with conventional fill to preload the foundation soils thereby limiting the post-construction movements of the subsoils and settlement of the backfill. Since, as discussed previously, a full preload is not possible at this location (as a result of stability issues and the proximity of the creek), following the partial preload period and construction of the H-pile supported wall, light-weight expanded polystyrene (EPS) fill would be employed to backfill the remainder of the wall.

The advantages of this option are that the piles for the support of the retaining wall and abutment could be designed neglecting lateral loads due to horizontal soil deformations and downdrag loads. In addition, the retaining wall can be designed neglecting active pressures over the height of the EPS fill (if a gap is maintained) and the post-construction settlement of the approach embankment will be small as a result of the preloading and partial replacement with light-weight fill. The disadvantages of this option include a 16 month delay in the construction of the retaining wall and bridge abutment and the additional costs associated with the EPS fill material.

#### **5.4.1.2.1 Preload Extent and Period**

The area of the preload would extend from about the south limit of the retaining wall to the south abutment area. The retaining wall backfill and/or south approach could be constructed to full height along the east side of the embankment in this area, however, to satisfy stability requirements (i.e. maintain a  $FoS \geq 1.3$ ) at all locations, the west side (adjacent to the creek) would have to be stepped down and only filled to about half of the required embankment height. The area of the half height portion of the preload embankment would be necessary along the full length of the proposed wall extending from the edge of the creek (on the west side) to about 4.5 m east of the wall location at the south end of the wall and to about 2.5 m east from the wall location at the north end of the wall.

For the silty clay strata ranging in thickness from about 2.5 m to 4.5 m at this location, it is estimated that the preload fill would be required to remain in place for up to about 16 months to achieve 90 percent consolidation. It should be noted that the installation of wick drains could be considered to accelerate the rate of consolidation at this location, however, given the relatively thin nature of the compressible soils (i.e. less than or equal to 4 m thick), wick drains would not be very efficient at this location. It is estimated that wick drains installed at 1.5 m or 1.0 m triangular spacing would reduce the time to reach 90 percent consolidation to about 8.5 months or 5.5 months, respectively.

#### **5.4.1.2.2 Steel H-Pile Foundations**

Following the end of the preload period, a portion of the preload fills (i.e. the western half) would be removed to a distance of 2.75 m back from (i.e. east of) the proposed retaining wall location. Following this, steel H-piles could be installed for support of the concrete retaining wall (and bridge abutment) foundations.

The general pile design recommendations for the steel H-pile supported wall should be as per Section 5.3 above. The required pile lengths/depth to bedrock along the wall can be estimated from the stratigraphic information provided on Drawing 1B. Depending on the depth of the pile

cap, the pile lengths along the wall could range from about 8.5 m (at the north end of the wall) to about 3.5 m (at the south end of the wall).

#### **5.4.1.2.3 Partial EPS Fill**

Following installation of the H-piles and the construction of the retaining wall and abutment, the area remaining between the new wall and the preload fill should be backfilled with light-weight EPS fill.

The required EPS fill would have approximate dimensions of 2.75 m (base width), 6.25 m (top width), 2.8 m (height), 35 m (long) for a total volume of about 450 m<sup>3</sup>. The EPS fill will have to be covered with at least 1 m of conventional fill/pavement structure to prevent differential pavement icing.

It should be noted that there is a risk of differential settlement occurring longitudinally along the pavement structure with this alternative at about the boundary between the two dissimilar (i.e. conventional versus EPS) fill types.

#### **5.4.1.3 Concrete Retaining Wall with Full EPS Fill**

For this option, the loading imposed by the 4.0 m to 4.5 m high retaining wall/approach embankment fills on the soft and compressible foundation soils could be minimized by using light weight EPS fill for the full length behind the wall and abutment. The use of this material would eliminate the need for partial preloading with a stepped geometry (acting as a stabilizing berm), reduce/minimize downdrag and lateral loads on piles due to vertical and horizontal soils deformations and eliminate the requirement to support the wall on closely spaced caissons.

This option would require that a zero net loading approach be adopted whereby the placement of the conventional fill for the pavement structure (minimum 1 m thick) on top of the EPS would have to be offset by an equivalent thickness of additional EPS installed at the base of the embankment. In addition, due to the low bearing capacity afforded by the weak foundation soils, the concrete retaining wall and abutment would still have to be supported on conventional H-pile foundations as per the recommendations described previously.

It should be noted that preliminary calculations indicate that the Factor of Safety against potential uplift/buoyancy of the EPS is greater than 1.5 at the HWL condition, assuming that the EPS is installed up to 1 m below the existing ground surface. However, if this option is adopted, the FoS would need to be re-checked once the depth/geometries of the EPS have been finalized.

Although this option has many advantages (as described above) and will not delay the construction of the retaining wall and abutment, the volume of EPS fill required (i.e. approximately 3 m high x 18 m wide x 35 m long) could make the cost of this alternative higher than that of the other options.

#### **5.4.2 Sub-excavate and Replace Existing Foundation Soils**

If sub-excavation and removal of the weak foundation soils is carried out and the area is backfilled with new competent fill, the concrete retaining wall can be supported on a shallow foundation system (i.e. spread footings) while maintaining global stability ( $FoS > 1.3$ ) and limiting settlements of the wall. Alternatively, the retaining wall could be supported on H-piles driven through the new fill (to bedrock) so long as the maximum particle size of the fill material is restricted so that it does not interfere with/obstruct the installation of the piles. The details of these sub-options are described below.

##### **5.4.2.1 Support on Shallow Foundations Founded on Rock Fill**

The bottom of the silty clay strata is located at a depth ranging from about 3 m to 4.5 m below existing ground surface at the south and north ends of the proposed wall, respectively. Sub-excavation and removal of the clayey strata to these depths is considered feasible, will allow the wall to be supported on conventional shallow foundations, avoid the need for backfill construction using partial preloading and partial or full EPS fill, and will provide the best technical solution in terms of the stability and long-term performance of the roadway.

However, the proposed wall is located close to the surveyed edge of the adjacent Blair (Sly) Creek (i.e. about 5 m away). Sub-excavation to a depth of up to 4.5 m over the area required for placement of fill at depth (i.e. at up to 4.5 m below the base of the wall) will extend up to about 5 m away (west) from the edge of proposed wall. As such, the sub-excavation could potentially undermine Blair Creek which we understand is considered an environmentally sensitive area.

To avoid having the excavation impact the creek, it is recommended that a sheet pile support system be installed at the excavation limit to allow the work to be carried out while minimizing disturbance to the creek. Recommendations with respect to the sheet pile support are given in Section 5.10.

Employing this approach will increase the effective thickness of new embankment height/wall backfill by up to 4.5 m because of the additional fill required below the existing ground surface. The additional fill below fill grade should be constructed with a side slope profile of 1.25H:1V where possible, but in no place should be steeper than 1H:1V (assuming that rock fill is utilized as the replacement fill). The use of rock fill will cause post-construction settlement of the wall

(as discussed below) and the increase in effective rock fill embankment height will result in additional post-construction settlement of the roadway.

#### **5.4.2.1.1 Settlement**

The settlement of the wall is governed by the embankment/wall backfill loading on the underlying fill and the native sands overlying the bedrock that are not removed as part of the sub-excavation. The immediate settlement of the native sands is expected to be up to 25 mm and will occur during construction. The long-term settlement of the rock fill foundation will be up to about 45 mm and will occur post-construction with up to about 30 mm occurring during the first year following completion of the wall and placement of the backfill.

The settlements associated with this options are acceptable from a foundation perspective and considered to provide the most effective design in terms of limiting the total settlement. Additional measures such as delaying the construction of the pavement/asphalt on the top of wall backfill for as long as is possible in the schedule could be adopted to further minimize the differential settlement between the barrier on the abutment and the barrier on the retaining wall.

#### **5.4.2.1.2 Geotechnical Resistances**

Spread footings constructed on the rock fill foundation material (that has been placed in accordance with Special Provision SP 206S03 dated January 2004) may be designed based on a factored geotechnical resistance at Ultimate Limit States (ULS) of 500 kPa.

The geotechnical resistance at Serviceability Limit States (SLS) for 25 mm of settlement under footing load only is estimated to be 350 kPa. However, this value does not consider that settlement of the retaining wall footing will occur due to compression of the rock fill itself under its own weight as noted above.

The resistance to lateral forces / sliding resistance between the base of the wall and the rock fill subgrade should be calculated in accordance with Section 6.7.5 of the CHBDC. The coefficient of friction,  $\tan \phi'$ , between the concrete wall footing and the rock fill foundation that replaces the sub-excavated material may be taken as 0.60. This represents an unfactored value; in accordance with the CHBDC, a factor of 0.8 is to be applied in calculating the horizontal resistance.

The retaining wall footing should be provided with a minimum of 1.8 m of soil cover for frost protection.

#### **5.4.2.2 Support on H-Piles Driven through Fill**

Steel H-piles could be installed for support of the concrete retaining wall, driven through the new fill material. This would minimize the potential for differential settlements between the wall and the bridge wingwall / abutment, especially if the bridge abutment is also founded on H-Piles.

The general pile design recommendations for the steel H-pile supported wall should be as per Section 5.3 above. The required pile lengths/depth to bedrock along the wall can be estimated from the stratigraphic information provided on Drawing 1B. Depending on the depth of the pile cap, the pile lengths along the wall could range from about 8.5 m (at the north end of the wall) to about 3.5 m (at the south end of the wall).

To accommodate pile driving, the fill material in the area of the pile foundations should be granular, not exceeding 75 mm particle size (not rock fill) so that pile driving to bedrock can be carried out without encountering obstructions.

This option will be more costly, but will minimize undesirable effects of differential settlement between potentially two different foundation types on the retaining wall and on the bridge wingwall / abutment.

### **5.5 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.

### **5.6 Lateral Earth Pressures for Design**

The lateral earth pressures acting on the retaining wall, abutment stems and any associated wing 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' 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:

	Granular 'A'	Granular 'B' Type II
Soil unit weight:	22 kN/m <sup>3</sup>	21 kN/m <sup>3</sup>
Coefficients of static lateral earth pressure:		
Active, $K_a$	0.27	0.31
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 zonal 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 ( $\text{kN/m}^3$ )
    - taken as soil unit weights given above for fill materials
    - taken as  $9.2 \text{ kN/m}^3$  for the native materials below Elevation 196.75 m at the north and south abutment
  - $d$  is the depth below the top of the wall (m); and
  - $H$  is the height of the wall above the toe (m).

## **5.7 Approach Embankment Design**

The construction of the Southbound Highway 69 structure over Blair (Sly) Creek will require placement of up to about 4.5 m of fill within the limits of both the south and north approach embankments. As discussed in Section 5.4, a retaining wall is proposed as one option to support the required fills and minimize encroachment of the creek at the southwest corner of the south approach. It is understood that consideration is also being given to locally re-aligning Blair (Sly) Creek into an old channel meander in this area so that a retaining wall would not be required. This section considers both options of the use of a retaining wall and the re-alignment of the creek (i.e. no retaining wall).

Based on the investigated locations at this site, the approach embankments will be founded on a thin surficial layer of sandy silt or silty sand underlain by silty clay to clay over a lower sandy silt to silty sand and/or sand, overlying bedrock. At the south approach, the total thickness of the overburden is between about 7 m and 8.5 m (with the silty clay to clay being up to about 4.5 m thick) while at the north approach, the total thickness of the overburden is about 11 m (with the silty clay to clay being up to about 5 m thick). All topsoil and organic matter should be stripped from below the approach embankment areas, and all subgrade soils should be proof-rolled prior to fill placement.

The results of stability and settlement analysis for the new approach embankments are presented in the following sections. It should be noted that the proposed southbound Highway 69 alignment located immediately north and south of the proposed structure have been investigated as swamp crossings and are reported under separate cover.

### **5.7.1 Stability**

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 (Version 5.20), 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 sites 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 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.

At all areas, the analyses assume that organic soils (encountered at or below the ground surface during field investigation operations) have been removed prior to construction of the new embankments. The piezometric conditions required in the analyses were based on the water level in Blair (Sly) Creek and on the groundwater levels noted during drilling and measured in piezometers in and immediately adjacent to this area. In general, the groundwater level is located at about the elevation of the water level in Blair (Sly) Creek.

The following table summarizes the simplified stratigraphy and the associated strengths and unit weights employed for the different soil types in the approach areas as well as at the south end of the retaining wall. For the purposes of analysis, both earth fill and rock fill have been considered for the construction of the approach embankments as indicated in the table below. Rock fill is assumed to have side slopes at 1.25H:1V and the earth fill is assumed to have side slopes at 2H:1V. A discussion on the different fill types, with respect to stability, is provided in Section 5.7.1.3.

**South Approach Embankment – North End of Retaining Wall Area**

<i>Soil Type</i>	<i>Unit Weight (kN/m<sup>3</sup>)</i>	<i>Strength Parameters</i>
Rock Fill	19	$c' = 0 \text{ kPa}, \phi' = 38^\circ$
Earth Fill (Sand and Gravel)	21	$c' = 0 \text{ kPa}, \phi' = 35^\circ$
Very loose to loose Sandy Silt to Silty Sand	19	$c' = 0 \text{ kPa}, \phi' = 30^\circ$
Soft to Firm Silty Clay to Clay	16.5	$s_u = 40 \text{ kPa (El.196m-195m)}$ $s_u = 30 - 20 \text{ kPa (El.195m-193.5m)}$ $s_u = 15 - 25 \text{ kPa (El.193.5m-192m)}$
Loose to compact Sand	20	$c' = 0 \text{ kPa}, \phi' = 33^\circ$

**South Approach Embankment – South End of Retaining Wall Area**

<i>Soil Type</i>	<i>Unit Weight (kN/m<sup>3</sup>)</i>	<i>Strength Parameters</i>
Rock Fill	19	$c' = 0 \text{ kPa}, \phi' = 38^\circ$
Earth Fill (Sand and Gravel)	21	$c' = 0 \text{ kPa}, \phi' = 35^\circ$
Very loose to loose Sandy Silt to Silty Sand	19	$c' = 0 \text{ kPa}, \phi' = 30^\circ$
Soft Silty Clay to Clay	16.5	$s_u = 17.5 - 15 \text{ kPa (El.196m-194.5m)}$ $s_u = 12.5 - 17.5 \text{ kPa (El.194.5m-193.5m)}$
Loose to compact Sand	20	$c' = 0 \text{ kPa}, \phi' = 33^\circ$

**North Approach Embankment**

<i>Soil Type</i>	<i>Unit Weight (kN/m<sup>3</sup>)</i>	<i>Strength Parameters</i>
Rock Fill	19	$c' = 0 \text{ kPa}, \phi' = 38^\circ$
Earth Fill (Sand and Gravel)	21	$c' = 0 \text{ kPa}, \phi' = 35^\circ$
Very loose to loose Sandy Silt to Silty Sand	19	$c' = 0 \text{ kPa}, \phi' = 30^\circ$
Very loose Organic Sandy Silt	18	$c' = 0 \text{ kPa}, \phi' = 28^\circ$
Very Soft to Soft Silty Clay to Clay	16.5	$s_u = 20 \text{ kPa (El.196m-195m)}$ $s_u = 17.5 - 10 \text{ kPa (El.195m-193.5m)}$ $s_u = 20 \text{ kPa (El.193.5m-192m)}$
Loose to compact Sand	20	$c' = 0 \text{ kPa}, \phi' = 33^\circ$

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>Location</i>	<i>Embankment Height at Critical Section (m)</i>	<i>Earth Fill Option</i>		<i>Rock Fill Option</i>	
		<i>Recommended Side Slope Profile</i>	<i>Minimum Factor of Safety</i>	<i>Recommended Side Slope Profile</i>	<i>Minimum Factor of Safety</i>
South Approach (South End of Retaining Wall Area – East Side)	4	2H : 1V	$\geq 1.3$	1.25H : 1V	$\geq 1.3$
South Approach (South End of Retaining Wall Area – West Side at Creek)	4	(see Section 5.7.1.1)			
South Approach (North End of Retaining Wall Area – East Side)	4.5	2H : 1V	$\geq 1.3$	1.25H : 1V	$\geq 1.3$
South Approach (North End of Retaining Wall Area – West Side at Creek)	4.5	(see Section 5.7.1.1)			
South Approach (Front Slope)	4.5	2H : 1V	$\geq 1.3$	1.25H : 1V	$\geq 1.3$
North Approach (East Side at Creek)	4.5	(see Section 5.7.1.2)			
North Approach (West Side)	4.5				
North Approach (Front Slope at Creek)	6				

The incorporation of a 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. However, as discussed in the following section, stabilizing berms may be required in some of the approach fill/proposed retaining wall areas.

#### **5.7.1.1 South Approach – West Side Stability at Creek**

The west side of the proposed south approach embankment is located immediately adjacent to the existing Blair Creek. In this area, the top of a soft to firm layer of silty clay to clay was encountered in the boreholes between about Elevation 196 m and 192 m (about 1 m below ground surface on average). The clayey stratum has a total thickness ranging from about 2.5 m to 4.5 m and undrained shear strengths as low as 12.5 kPa. Given the subsoil conditions, if the creek is to be locally re-aligned away from this area, the approach embankment could be constructed without a retaining wall however, berms up to about 1.5 m high and 1.5 m wide (extending beyond the toe

of the embankment fill (as shown on Figure 10) would be required to achieve an adequate Factor of Safety for global stability in this area. After re-alignment, the existing creek should be backfilled with Granular 'B' Type II or rock fill. If rock fill is used, an upstream cutoff at the junction with the newly re-aligned creek would be required.

However, if as a result of environmental restrictions the creek cannot be locally re-aligned, a retaining wall (as discussed in Section 5.4) would be required to limit the encroachment of embankment fill into the creek at this location. Depending on the foundation type and/or treatment adopted for the wall, conventional backfill may only be employed for about half the required height of fill behind the retaining wall (if the fill is constructed on the existing soft foundation soils) to achieve a Factor of Safety (FoS) of 1.3. In addition, if the existing foundation soils are maintained and preloading with conventional fills is employed, a rock fill berm will be required at the toe of the west slope of the fills over the full length of the proposed retaining wall area up to the edge of the creek. Stability analyses indicate that the required toe berm should have dimensions that transition from approximately 2 m high by 4.75 m wide (at the south end of the wall) to approximately 2.5 m high by 2.75 m wide (at the north end of the wall) to achieve an adequate FoS at this location (as shown on Figure 2 and 3, respectively). EPS fill would then have to be used to fill the remainder of the embankment to the required grade.

This option (i.e. the use of a toe berm for preloading up to the edge of the creek combined with restricting the amount of conventional fill immediately behind the wall) may be considered unfeasible or impractical due to environmental or hydraulic implications associated with the berm. The use of a combination of conventional fill and EPS fill over a portion of the approach (i.e. on the west side) could also potentially cause differential settlements on the roadway. As such, other stability mitigation options should be considered including full sub-excavation and removal of the weak/soft soils, wick drain installation to accelerate settlements, or the use of light weight fill to reduce driving forces. A discussion of the advantages, disadvantages, relative costs, risks/consequences for the mitigation options at this area are discussed further in Section 5.8 and presented in Table 4.

As discussed in Section 5.7.3, the soft clay strata in this area will also cause time dependent (consolidation) settlements of the new embankment. The sub-excavation, wick drains and the light weight fill options would also mitigate the long-term (i.e. post-construction) settlements. These and other mitigative alternatives are discussed in Section 5.8 and are included in Table 4. The full sub-excavation option has been ranked as the preferred alternative for this area.

#### **5.7.1.2 North Approach**

The north approach embankment is located within the floodplain of and immediately adjacent to Blair Creek. In this area, the top of a very soft to soft layer of silty clay to clay was encountered

in the boreholes between about Elevation 196 m and 192 m (about 1 m below ground surface on average). The clayey stratum has a total thickness of up to about 5 m and undrained shear strengths as low as 10 kPa. Given the subsoil conditions, if in-filling of the creek was permissible, berms (extending beyond the toe of the embankment fill) up to about 8.5 m wide (on the front slope) and up to about 5.5 m wide (on the east side slope) would be required to achieve an adequate Factor of Safety for global stability. On the west side of the approach, a berm approximately 3 m wide is required for global stability.

Given the restrictions on fill placement imposed by the vicinity of the creek, to achieve a Factor of Safety (FoS) = 1.3 for the 4.5 m high (east side of approach) to 6 m high (front slope of approach) embankment fill in this area, conventional backfill can only be employed for about half the required height of fill (EPS fill would have to be used to fill the remainder of the embankment to the required grade) and it will be necessary to construct a rock fill berm at the toe of the east and front slope of the embankment (extending back from the edge of the creek) over the full length of the approach. Stability analyses indicate that on the east side, the required toe berm should have dimensions of approximately 2 m high by 5.5 m wide to achieve an adequate FoS at this location (as shown on Figure 4). On the front slope, the analysis indicates that the toe berm should have dimensions of approximately 2 m high by 8.5 m wide (see Figure 5). On the west side of the approach, there are no restrictions on fill placement and conventional fill can be used to construct the full height of approach in this area. However, to achieve a FoS = 1.3 for the required up to 4.5 m high fill, a toe berm that is 2 m high and 3 m wide is required (see Figure 6).

This option (i.e. the use of toe berms up to the edge of the creek combined with restricting the amount of conventional fill immediately behind the abutment wall and on the east side of the embankment and using EPS to fill to the required grade) may be considered unfeasible or impractical due to environmental or hydraulic implications associated with the berms. The use of a combination of conventional fill and EPS fill over portions of the approach (i.e. behind abutment wall and on the east side) could also potentially cause differential settlements on the roadway. As such, other stability mitigation options should be considered including full sub-excavation and removal of the weak/soft soils, wick drain installation to accelerate settlements or the use of light weight fill to reduce driving forces. A discussion of the advantages, disadvantages, relative costs, risks/consequences for the mitigation options at this area are discussed further in Section 5.8 and presented in Table 5.

As discussed in Section 5.7.3, the soft clay strata in this area will also cause time dependent (consolidation) settlements of the new embankment. The sub-excavation, wick drains and the light weight fill options would also mitigate the long-term (i.e. post-construction) settlements. These and other alternatives to mitigate settlements are discussed in Section 5.8 and are included in Table 5. The full sub-excavation option has been ranked as the preferred alternative for this area.

### **5.7.1.3 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 / bedrock), construction cost and time, and ease of construction / availability.

#### **5.7.1.3.1 Earth Fill**

The main advantage of using earth fill (i.e. sand and gravel) is the ease of construction and the lack of post-construction settlements within the fill embankment itself. However, this option will require a larger volume of fill and wider right-of-way because the side slopes will be flatter than rock fill slopes. For this project, acceptable earth fill is considered to be suitable locally available and/or imported, granular material.

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

#### **5.7.1.3.2 Rock Fill**

The main advantage of using rock fill is the ability to achieve steeper embankment side slopes. This is useful in areas with limited right-of-ways. In addition, rock fill will likely be available from the rock cuts proposed for the 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 only required where 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.

### **5.7.2 Liquefaction Potential**

The liquefaction potential of the soils below the approach embankments under seismic loading has been considered using the empirical method outlined in Section C.4.6.2 of the *CHBDC* Commentary, which correlates the cyclic resistance ratio of the soils with their normalized penetration resistance and fines content. Based on this assessment, and assuming a ground surface acceleration of 0.06 g, a factor of safety of greater than 1.0 against liquefaction is

obtained for magnitude 6.2 earthquake events under the approach embankment. Pseudo-static methods of embankment stability analysis indicate that a yield acceleration of approximately 0.20 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 sandy silt subsoils prior to construction of the approach embankments in order to eliminate the potential for seismically induced liquefaction at the embankment toes.

### **5.7.3 Settlement**

Settlement of the approach embankments (including the fill behind the retaining wall) can be expected as a result of the loading from the new fills 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 and retaining wall using either the commercially available program UNISSETTLE (Version 3.2) or hand calculations where the subsoils consist of thin deposits of cohesive strata and/or cohesionless soils. The rate of settlement of the cohesive foundation soils was assessed by spreadsheet calculations using Terzaghi's one-dimensional consolidation theory.

For these analyses, the critical sections are assumed to correspond to the greatest new embankment heights, approximately 4.5 m for both the south and north approaches. In addition, an analysis of the settlement of the 4 m high fills at the south end of the retaining wall was also carried out. The unit weights and slope profiles for the embankment fill described in Section 5.7.1 were employed in the analyses. The analyses performed assume that the organic soils/topsoil have been removed prior to construction and that rock fill has been used for the embankment construction.

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 / silty sand / 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 stiff silty clay to clay layers was assessed using the results of the in situ field vane and SPT tests and/or laboratory consolidation 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).

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 settlement analyses for both approach embankments assume that any surficial or near surface organic soils have been removed prior to construction of the new embankments. The piezometric conditions required in the analyses were based on the groundwater levels noted during drilling and measured in the piezometer installation. In general, the groundwater level was assumed to be located at about the elevation of Blair (Sly) Creek (or about 0.25 m below the ground surface in the vicinity of the creek and about 1.25 m below ground surface at the south end of the wall).

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 foundation soils in these areas (due to the loading imposed by the new approach embankment fills) is presented and the rate of settlement is discussed.

### **5.7.3.1 Settlement of Foundation Soils (South Approach + Retaining Wall)**

The following simplified stratigraphy and deformation parameters have been developed for and employed in the settlement analysis of the proposed 4 m to 4.5 m high south approach rock fill embankment at the south end and north end of the proposed retaining wall area, respectively.

Note that the results of the settlement analyses discussed in this section are applicable regardless if the proposed retaining wall is employed or not.

#### North End of Retaining Wall

<i>Soil</i>	<i>Thickness (m)</i>	<i>Unit Weight (kN/m<sup>3</sup>)</i>	<i>Estimated Deformation Properties</i>
Rock fill (4.5 m embankment + removal of 0.3 m organics)	4.8 (high)	19	-
Sandy Silt to Silty Sand	1.0	19	E' = 5 MPa
Silty Clay to Clay	4.0	16.5	(see below)
Sand	4.0	20	E' = 15 MPa

#### South End of Retaining Wall

<i>Soil</i>	<i>Thickness (m)</i>	<i>Unit Weight (kN/m<sup>3</sup>)</i>	<i>Estimated Deformation Properties</i>
Rock fill (4.0 m embankment + removal of 0.3 m organics)	4.3 (high)	19	-
Sandy Silt to Silty Sand	1.0	19	E' = 5 MPa
Silty Clay to Clay	2.5	16.5	(see below)
Sand	1.0	20	E' = 15 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 the silty clay to clay obtained adjacent to the area of the south approach embankment.

<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 End of Wall	196 to 192	25	75	3	1.9	0.10	0.90	1.0 x 10 <sup>-3</sup>
South End of Wall	196 to 193.5	30	55	2				

Note: values of  $\sigma_{vo}'$  and  $\sigma_p'$  above are estimated at the middle of the silty clay to clay layer and vary above and below this location.

Based on the results of the settlement analysis, the maximum total settlement of the foundation soils in the area of the south approach/retaining wall is estimated to range from about 155 mm (at south abutment) to about 240 mm (at south end of wall). This total settlement is estimated to be comprised of about 20 mm to 30 mm of immediate settlement due to compression of the cohesionless soil layers and about 125 mm to 210 mm of time dependent settlement of the cohesive soil layers.

Assuming a coefficient of consolidation ( $c_v$ ) of about  $1.0 \times 10^{-3} \text{ cm}^2/\text{s}$  (based on results of laboratory consolidation tests as well as empirical correlations with liquid limit using US Navy (1971)) and assuming two-way drainage of the up to 4 m thick (near the south abutment) silty clay to clay layer, it is estimated that the about 90 percent of the consolidation settlement will be completed in about 13 months. It should be noted that the clay stratum was found to be locally up to about 4.5 m thick in some areas of the south approach. In this area, it may take up to about 16 months to achieve 90 percent of the consolidation settlement.

The magnitude of creep settlement for the silty clay to clay strata is expected to be about 25 mm per log-cycle of time at this site.

### 5.7.3.2 Settlement of Foundation Soils (North Approach)

The following simplified stratigraphy and deformation parameters have been developed for and employed in the settlement analysis of the proposed 4.5 m high rock fill 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 (4.5 m embankment + removal of 0.3 m organics*)	4.8 (high)	19	-
Silty Sand	1.0	19	$E' = 5 \text{ MPa}$
Silty Clay to Clay	4.5	16.5	(see below)
Silty Sand to Sand	5	20	$E' = 15 \text{ MPa}$

\*Note: removal of organics up to 2.9 m thick is required in the area at and behind the north abutment. However, the clay stratum is on average thinner at this location and as such the settlements calculated based on the above simplified stratigraphy are considered representative for the north approach area.

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 the silty clay to clay obtained adjacent to the area of the north approach embankment.

<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>
SBL - North Approach	196 to 191.5	25	50	2	1.4	0.07	0.69	$1.0 \times 10^{-3}$

Note: values above are estimated at the middle of the silty clay to clay layer

Based on the results of the settlement analysis, the maximum total settlement of the foundation soils in the area of the north approach is estimated to be about 490 mm. This total settlement is estimated to be comprised of about 50 mm of immediate settlement due to compression of the cohesionless soil layers and about 440 mm of time dependent settlement of the cohesive soil layers.

Assuming a coefficient of consolidation ( $c_v$ ) of about  $1.0 \times 10^{-3}$  cm<sup>2</sup>/s (based on empirical correlations with liquid limit using US Navy (1971)) and assuming two-way drainage of the approximately 4.5 m thick silty clay to clay layer, it is estimated that the about 90 percent of the consolidation settlement will be completed in about 16 months.

The magnitude of creep settlement for the silty clay to clay strata is expected to be about 20 mm per log-cycle of time at this site.

### 5.7.3.3 Settlement of Rock Fill

If rock fill is used for the construction of the embankments, in addition to the settlement due to compression of the foundation soils described above, there will be settlement due to compression of the rock fill itself. Settlement of the rock fill depends on the type of rock and on the method and sequence of placement and compaction of the fill. Assuming that the rock fill is not end dumped in its final position and is placed in accordance with the requirements as outlined in the Special Provision 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, for the up to 4.5 m high approach embankments, the settlement of the rock fill will be about 1% of the new effective height of rock fill.

<i>Location of Embankment</i>	<i>Approximate Chainage</i>	<i>Maximum New Embankment Height* (m)</i>	<i>Estimated Settlement of Embankment Soils (mm)</i>
South Approach	10+619 to 10+639	$4.5+0.3 = 4.8$	45
North Approach	10+678 to 10+688	$3.8+2.9 = 6.7$	70
	10+688 to 10+698	$4.5+0.3 = 4.8$	45

Notes : \*includes additional fill required after removal of maximum depth of topsoil/organics

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

#### 5.7.3.4 Settlement of Earth Fill

If granular fill is used for embankment construction, settlement of the properly compacted embankment fills are expected to be less than 25 mm and will occur during construction. It is recommended that the fines content of the earth fill used for embankment construction be minimized to avoid long-term settlement and maintenance issues.

### 5.8 Mitigation of Stability Issues / Time Dependent Settlements

As discussed in Section 5.4 and 5.7.1 the presence of the very soft to firm silty clay stratum at both the south and north approaches creates stability problems for the potential retaining wall and the proposed heights of new embankment fill in these areas. In addition, as discussed in Section 5.7.3, these soft and compressible soils will also result in time dependent settlements of the new embankments. In these areas, consideration needs to be given to adopting a design and/or following a construction sequence to achieve the minimum target factor of safety of 1.3 for the proposed new embankment heights and geometry and to limit the post-construction settlements and subsequent maintenance on the new roadway pavement structure.

For the south and north approach areas, the following sections outline the options and recommendations for achieving the target factor of safety for the proposed embankment geometries and for minimizing the time dependent, post-construction settlements that could affect 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 4 and 5, respectively.

#### 5.8.1 South Approach

As discussed previously, a soft to firm silty clay to clay stratum underlies a surficial layer of very loose to loose silty sand in this area of the Blair (Sly) Creek floodplain. The presence of this soft

layer influences both the stability and magnitude of post-construction settlement of the proposed 4.5 m high approach embankment and/or potential retaining wall. In order to achieve the MTO's objective of producing a highway embankment design that will minimize post-construction settlements, the following alternatives can be considered. The alternatives described below have been evaluated and ranked on the basis of the advantages, disadvantages, relative costs and risks/consequences and are summarized in Table 4.

#### **5.8.1.1 Full Sub-excavation**

The bottom of the silty clay to clay stratum is located at depths ranging from about 3 m to 5.5 m (averaging about 4.5 m) below existing ground surface at the south approach and south end of the proposed retaining wall area, respectively. Sub-excavation and removal of the clayey strata to this depth is considered feasible, would avoid the need for toe berms on the west slope of the approach embankment/potential retaining wall and will provide the best technical solution in terms of the stability and long-term performance of the roadway.

However, the embankment toe at the west side of the potential retaining wall and near the northwest corner of the proposed south approach is close to the surveyed edge of the adjacent existing Blair Creek (i.e. less than about 3 m away). Sub-excavation to a depth of 4.5 m over the area required for placement of fill at depth (i.e. at 4.5 m below the base of the embankment) will extend up to 5.5 m away from the proposed embankment toe at ground surface. If the creek is locally re-aligned into the apparent old/original channel meander (which is farther away from the west edge of the approach as shown on Figure B-1), the majority of the sub-excavation required in this area could be carried out without any additional special measures and with minimal impact to the creek environment. A small amount of sheet pile support may still be locally required around the northwest corner of the embankment depending on the location of the abutment/approach in relation to the new creek alignment in this area. If however the creek is not re-aligned, the sub-excavation would potentially undermine Blair (Sly) Creek which we understand is considered an environmentally sensitive area.

As such, if the creek is to remain on its current alignment, to minimize the impact of the excavation on the creek, the following options could be considered:

- Construct a low-permeability earth fill berm to temporarily (or permanently) divert the creek around the area of encroachment; or
- Install a sheet pile support system 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 5.10. 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 (considering the high water levels in the sand strata underlying the clay). In addition, excavation ‘in-the-wet’ would be required 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 5.10.1. This methodology for excavation in strips (and ‘in-the-wet’) should be adopted in areas adjacent to the proposed 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 would have to be supported by tie-backs (i.e. anchors to 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 approximately 4.5 m because of the additional fill required below the existing ground surface. The additional below fill grade 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 45 mm) post-construction settlement of the embankment rock fill.

#### **5.8.1.2 Toe Berms and Preloading**

For the approximately 4 m thick silty clay to clay strata at this location (locally up to 4.5 m thick), it is estimated that 90 percent of the post-construction foundation soil settlements will be completed in about 13 to 16 months. If the construction schedule can accommodate this period, pre-loading the foundation soils by building the embankment as early as possible can be considered.

If the creek can be re-aligned (and if a retaining wall structure is not employed), berms up to about 2 m high and 5 m wide (extending beyond the toe of the embankment fill) will be required along the west side of the approach to achieve an adequate Factor of Safety for global stability in this area.

If the creek cannot be re-aligned, based on the embankment height/proposed retaining wall geometry and the soft subsoil conditions, a toe berm and preloading may be required depending on the foundation type adopted for the wall (as discussed in Section 5.4). In addition, conventional backfill may only be employed for about half the required height of fill behind the retaining wall to achieve a Factor of Safety (FoS) of 1.3 (as shown on Figure 2). If the existing foundation soils are maintained and preloading with conventional fills is employed, a rock fill

berm would be required at the toe of the west slope of the fills over the full length of the proposed retaining wall area up to the edge of the existing creek. Stability analyses indicate that the required toe berm should have dimensions that transition from approximately 2 m high by 4.75 m wide (at the south end of the wall) to approximately 2.5 m high by 2.75 m wide (at the north end of the wall) to achieve an adequate FoS at this location (as shown on Figure 2 and 3, respectively). Lightweight EPS fill would then have to be used to fill the remainder of the embankment to the required grade. For this alternative, sub-excavation and the temporary diversion or sheet pile support at Blair (Sly) Creek would not be required, however the use of a toe berm with preloading up to the edge of the creek combined with restricting the amount of conventional fill immediately behind the wall may be considered not feasible or impractical due to environmental or hydraulic implications associated with the berm. The use of a combination of conventional fill and EPS fill over a portion of the approach (i.e. on the west side) could also potentially cause differential settlements on the roadway.

It should be noted that some additional long-term settlements due to secondary consolidation (i.e. creep) of the silty clay to clay strata should be expected with this option. It is estimated that creep settlements of about 25 mm over each log-cycle of time after the substantial completion of primary consolidation will occur. Therefore, following the substantial completion of the primary consolidation, about 40 mm of additional creep settlement is expected to occur within about 50 years.

#### **5.8.1.3 Wick Drains**

As noted above, it is estimated that without any foundation treatment, about 90% of the primary consolidation settlement of the silty clay to clay strata would be completed in about 13 to 16 months following completion of embankment construction. However, preliminary calculations indicate that installing wick drains on a triangular grid at a 1.5 m or 1.0 m spacing to a depth of up to about 5 m would accelerate the consolidation process such that about 90 percent of the primary consolidation could be completed in about 8.5 or 5.5 months, respectively after completion of embankment construction.

Due to the soft/weak subsoils, even with the use of wick drains, the embankment construction would still require a toe berm on the west slope to maintain the target factor of safety in combination with either not re-aligning the creek and having to restrict the amount of conventional fill (plus employing partial EPS fill immediately behind the wall) or re-aligning the creek and using conventional fill for the entire embankment. Monitoring of the settlement and dissipation of the excess porewater pressures would be required to check that adequate consolidation/settlement had occurred prior to proceeding with the final pavement construction.

It should be noted that some additional long-term settlements due to secondary consolidation (i.e. creep) of the clayey strata (on the order of about 25 mm per log-cycle of time) should be expected with this option.

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

The loading imposed by the 4.5 m high south approach embankment on the soft and compressible foundation soils in this area could be reduced by using light weight (i.e. expanded polystyrene) fill. The use of this material for the embankment fill would reduce the need for stabilizing toe berms and would result in minor time-dependent (consolidation) settlement of the clayey strata. The following alternatives that make use of EPS fill could be considered:

- Construct the entire approach with EPS fill; or
- Preload the embankment to a nominal height with conventional fill, followed by partial excavation and replacement with EPS fill.

The volume of EPS fill required to construct the first alternative to the end of the retaining wall area (i.e. approximately 3.5 m high x 18 m wide x 40 m long) could make the cost of this alternative higher than that of some of the other options discussed above.

However, it is estimated that less EPS fill (i.e. less than half) could be employed if the construction schedule will allow a combination of preloading (with or without wick drains) followed by partial excavation and replacement with limited EPS fill as discussed in the previous sections (and as shown on Figure 7). Preliminary analyses indicate that the following sequence could be adopted:

- i) construct preload embankment with a toe berm on the west side that transition from approximately 2 m high by 4.75 m wide (at the south end of the wall) to approximately 2.5 m high by 2.75 m wide (at the north end of the wall);
- ii) allow preload to remain in place for about 13 to 16 months (no wick drains) or about 6 to 9 months (with wick drains);
- iii) remove west side of embankment fill to a distance of about 2.75 m east of proposed wall location;
- iv) install EPS fill (2.75 m wide (at base) x 6.25 m wide (at top) x 2.8 m high) over the length of the approach/retaining;
- v) construct pavement structure on top of the EPS.

This approach would effectively eliminate the majority of the primary consolidation settlement. However, some additional long-term settlements due to secondary consolidation (i.e. creep) of the clayey strata (on the order of about 25 mm per log-cycle of time) should be expected with this option.

It should be noted that preliminary calculations indicate that the Factor of Safety against potential uplift/buoyancy of the EPS is greater than 1.5 at the HWL condition, assuming that the EPS is installed up to 1 m below the existing ground surface. However, if this option is adopted, the FoS would need to be re-checked once the depth/geometries of the EPS have been finalized.

#### **5.8.1.5     Surcharging**

As noted above, if the creek is not re-aligned (and a combination of conventional fill and EPS is employed), a toe berm approximately 2.75 m to 4.75 m wide by about 2 m to 2.5 m high, constructed up to the edge of the creek, is necessary to maintain the stability of the west slope if the embankment is built rapidly up to the required final grade height of about 4.5 m. If the creek is re-aligned (and the embankment is to be constructed with only conventional fill), a toe berm approximately 2 m high by 5 m wide is required to maintain stability of the full height of the west side proposed embankment within the approach and retaining wall area.

Even larger toe berms on the west slope, in addition to a berm on the front slope, would be required if a surcharge was to be placed on top of the required 4.5 m high embankment.

Given the environmental restrictions associated with encroaching on Blair Creek, it will likely not be possible to construct the required larger berms to support the surcharging and as such, this mitigation alternative is not considered practical at this location.

#### **5.8.2        North Approach**

As discussed previously, a very soft to soft silty clay to clay stratum underlies a surficial layer of very loose silty sand to organic sandy silt in this area of the Blair Creek floodplain. The presence of this very soft layer influences both the stability and magnitude of post-construction settlement of the proposed 4.5 m high approach embankment. In order to achieve the MTO's objective of producing a highway embankment design that will minimize post-construction settlements, the following alternatives can be considered. The alternatives described below have been evaluated and ranked on the basis of the advantages, disadvantages, relative costs and risks/consequences and are summarized in Table 5.

##### **5.8.2.1     Full Sub-excavation**

The bottom of the silty clay to clay stratum is located at depths ranging from about 5 m to 6 m below existing ground surface at the north approach. Sub-excavation and removal of the clayey strata to this depth is considered feasible, would avoid the need for toe berms on the east, west and front slope of the approach embankment and will provide the best technical solution in terms of the stability and long-term performance of the roadway.

However, the embankment toe along the east side and at the southeast corner of the approach is close to the surveyed edge of the adjacent Blair Creek (i.e. about 4 m away). Sub-excavation to a depth of 6 m over the area required for placement of fill at depth (i.e. at 5 m below the base of the embankment) will extend up to about 7.5 m away from the proposed embankment toe at ground surface. As such, the sub-excavation could potentially undermine Blair (Sly) Creek which we understand is considered an environmentally sensitive area.

To minimize the impact of the excavation on the creek, the following options could be considered:

- Construct a low-permeability earth fill berm to temporarily (or permanently) divert the creek around the area of encroachment; or
- Install a sheet pile support system at the excavation limit to allow the work to be carried out while minimizing the encroachment on the creek.

We understand that a local diversion of the creek is being considered; however the proposed diversion (which is in the area of the south approach) will not improve the proximity of the creek to the north approach embankment. As such depth/geometries, the installation of sheet piling should be considered in this area to minimize the impact of excavation. Recommendations with respect to the sheet pile support are given in Section 5.10. In addition, since the groundwater table is located at about the level of the ground surface, the sub-excavation will 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 (considering the high water levels in the sand strata underlying the clay). In addition, excavation ‘in-the-wet’ will be required 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 5.10.1. This methodology for excavation in strips (and ‘in-the-wet’) should be adopted in areas adjacent to the proposed 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 will have to be supported by tie-backs (i.e. anchors to 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 approximately 6 m because of the additional fill required below the existing ground surface. The additional below fill grade 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 60 mm) post-construction settlement of the embankment rock fill.

### **5.8.2.2 Toe Berms and Preloading**

For the approximately 4.5 m thick silty clay to clay strata at this location, it is estimated that 90 percent of the post-construction foundation soil settlements will be completed in about 16 months. If the construction schedule can accommodate this period, pre-loading the foundation soils by building the embankment as early as possible can be considered.

If in-filling of the creek in this area was permissible, or if a re-alignment of the creek could move the creek away from the toe of the embankment in this area, berms (extending beyond the toe of the embankment fill or abutment wall) up to about 8.5 m wide (on the front slope) and up to about 5.5 m wide (on the east side slope) would be required to achieve an adequate Factor of Safety for global stability. On the west side of the approach, a berm approximately 3 m wide is required for global stability.

Since re-alignment of the creek in the area of the north approach is considered unlikely, to avoid the placement of fill into the creek and to achieve a Factor of Safety of 1.3 for the 4.5 m high (east side of approach) to 6 m high (front slope of approach) embankment in this area, conventional backfill can only be employed for about half the required height of fill. Lightweight EPS fill would have to be used to build the remainder of the embankment to the required grade. In addition, it will be necessary to construct a rock fill berm at the toe of the east and front slope of the embankment (extending back from the edge of the creek) over the full length of the approach. Stability analyses indicate that on the east side, the toe berm should have dimensions of approximately 2 m high by 5.5 m wide to achieve the target FoS at this location (as shown on Figure 4). On the front slope, the analysis indicates that the toe berm should have dimensions of approximately 2 m high by 8.5 m wide (as shown on Figure 5). On the west side of the approach, there are no restrictions on fill placement and conventional fill can be used to construct the full height of approach in this area. However, to achieve a  $FoS = 1.3$  for the required up to 4.5 m high fill, a toe berm that is 2 m high and 3 m wide is required (as shown on Figure 6).

For this alternative, sub-excavation and the temporary diversion or sheet pile support at Blair (Sly) Creek would not be required, however the use of a toe berm with preloading up to the edge of the creek combined with restricting the amount of conventional fill immediately behind the wall may be considered unfeasible or impractical due to environmental or hydraulic implications associated with the berm. The use of a combination of conventional fill and EPS fill over a portion of the approach (i.e. on the east side and behind the abutment) could also potentially cause differential settlements on the roadway.

It should be noted that some additional long-term settlements due to secondary consolidation (i.e. creep) of the silty clay to clay strata should be expected with this option. It is estimated that creep settlements of about 20 mm over each log-cycle of time after the substantial completion of primary consolidation will occur. Therefore, following the substantial completion of the primary

consolidation, about 30 mm of additional creep settlement is expected to occur within about 50 years.

### **5.8.2.3 Wick Drains**

As noted above, it is estimated that without any foundation treatment, about 90% of the primary consolidation settlement of the silty clay to clay strata would be completed in about 16 months following completion of embankment construction. However, preliminary calculations indicate that installing wick drains on a triangular grid at a 1.5 m or 1.0 m spacing to a depth of up to about 6 m would accelerate the consolidation process such that about 90 percent of the primary consolidation could be completed in about 8.5 or 5.5 months, respectively after completion of embankment construction.

Due to the soft/weak subsoils, even with the use of wick drains, the embankment construction would require toe berms on the east, west and front slope in combination with restricting the amount of conventional fill and requiring partial EPS fill in order to maintain the target factor of safety. Monitoring of the settlement and dissipation of the excess porewater pressures would be required to check that adequate consolidation/settlement had occurred prior to proceeding with the final pavement construction.

It should be noted that some additional long-term settlements due to secondary consolidation (i.e. creep) of the clayey strata (on the order of about 20 mm per log-cycle of time) should be expected with this option.

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

The loading imposed by the 4.5 m high south approach embankment on the soft and compressible foundation soils in this area could be reduced by using light weight (i.e. expanded polystyrene) fill. The use of this material for the embankment fill would reduce the need for stabilizing toe berms and would result in minor time-dependent (consolidation) settlement of the clayey strata. The following alternatives that make use of EPS fill could be considered:

- Construct the entire approach with EPS fill; or
- Preload the embankment to a nominal height with conventional fill, followed by partial excavation and replacement with EPS fill.

The volume of EPS fill required to construct the first alternative to the end of the retaining wall (i.e. approximately 3.5 m high x 18 m wide x 20 m long) could make the cost of this alternative higher than that of some of the other options discussed above.

However, it is estimated that less EPS fill (i.e. less than half) could be employed if the construction schedule will allow a combination of preloading (with or without wick drains) followed by partial excavation and replacement with limited EPS fill on the front slope and east side slope as discussed in the previous sections (and as shown on Figures 8 and 9). Preliminary analyses indicate that the following sequence could be adopted:

- i) construct preload embankment with a toe berm 2 m high by 5.5 m wide on the east slope, 2 m high by 8.5 m wide on the front slope and 2 m high by 3 m wide on the west slope;
- ii) allow preload to remain in place for about 13 to 16 months (no wick drains) or about 6 to 9 months (with wick drains);
- iii) remove embankment fill to a distance of about 6 m back from the abutment wall and 1.5 m back from the east side of the preload embankment;
- iv) install EPS fill (6 m wide (at base) x 8 m wide (at top) x 1.5 m high) over the width of the embankment behind the abutment;
- v) install EPS fill (4.5 m wide (at base) x 4.5 m wide (at top) x 1.2 m high) over the full length of the east side of the approach embankment;
- vi) construct pavement structure on top of the EPS.

This approach would effectively eliminate the majority of the primary consolidation settlement. However, some additional long-term settlements due to secondary consolidation (i.e. creep) of the clayey strata (on the order of about 25 mm per log-cycle of time) should be expected with this option.

It should be noted that preliminary calculations indicate that the Factor of Safety against potential uplift/buoyancy of the EPS is greater than 1.5 at the HWL condition, assuming that the EPS is installed up to 1 m below the existing ground surface. However, if this option is adopted, the FoS would need to be re-checked once the depth/geometries of the EPS have been finalized.

#### **5.8.2.5 Surcharging**

As noted above, toe berms approximately 2 m high x 5.5 m wide (on the east slope) and 2 m high x 8.5 m wide (on the front slope) constructed up to the edge of the creek, are necessary to maintain the stability of the embankment if it is built rapidly up to the required final grade height of about 4.5 m. In addition, a toe berm 2 m high x 3 m wide is required on the west slope. Even larger toe berms would be required if a surcharge was to be placed on top of the required 4.5 m high embankment.

Given the environmental restrictions associated with encroaching on of Blair Creek, it will likely not be possible construct the required larger berms to support the surcharging and as such, this mitigation alternative is not considered feasible at this location.

## **5.9 Subgrade Preparation and Embankment Construction**

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

Table 6 summarizes the recommended fill type to be placed for the widenings, 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.

If sub-excavation and removal is adopted as the method to mitigate the time-dependent settlements and stability problems at the south (incl. the retaining wall) and north approach embankment, it will be necessary to backfill the excavation in the area of the south and north abutment foundations and retaining wall 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.

### **5.9.1 Removal of Organics**

Based on the information from the borings obtained during the field investigation, organic deposits (i.e. topsoil / organic sandy silt) of up to about 2.9 m deep can be expected in the area of the north approach embankment. At the south approach, the organic deposits are typically about 0.3 m deep. These organic layers should be stripped from the plan limits of the approach areas prior to fill placement.

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

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 the requirements as outlined in the Special Provision SP 206S03 dated January 2004, in regular lifts with loose thickness not exceeding 300 mm, and be compacted to at least 95 percent of the Standard Proctor maximum dry density. The final lift prior to placement of the granular sub-base or base course should be placed and compacted to current MTO requirements for pavements. Inspection and field density

testing should be carried out by qualified geotechnical personnel during all earth fill placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved. Side slopes for earth fill embankments should be no steeper than 2H:1V.

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

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

Erosion protection (i.e. rip-rap) of a suitable size and thickness should be placed (as required) on the front and adjacent side slopes and at the toe of the potential retaining wall in order to protect the approach embankment, abutment foundations and retaining wall foundation areas 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.

## **5.10 Design and Construction Considerations**

### **5.10.1 Excavations**

As noted in Section 5.8, excavations as part of the settlement and stability mitigation measures at the south (incl. retaining wall, if required) and north approach embankments (if sub-excavation and removal is adopted) would extend up to 8.5 m deep and 6 m deep, respectively.

In addition, as noted in Section 5.9, excavation within the plan limits of the approach embankments will be required in order to remove topsoil / organic deposits up to about 0.3 m deep and up to 2.9 m deep at the south and north approaches, respectively, prior to fill placement.

At the south and north approach, where space permits, temporary excavations (i.e. those that are open only for a relatively short period) to a depth of up to about 6 m for removal of the silty clay to clay 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, the recommendations for temporary shoring or staged excavation described in the sections below could be considered.

At the south and north abutment, temporary excavation to a depth of about 1.5 m for pile installation and construction of the pile cap should be carried out with side slopes about 3H:1V. Assuming the excavations are carried out 'in-the-dry' to a maximum depth of 1.5 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 1.1 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.

Groundwater inflows will have to be controlled as discussed in Section 5.10.2. 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.

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.

#### **5.10.1.1 Temporary Shoring**

Where space and/or high groundwater levels restrict the use of open cuts, a temporary support system could be constructed to support excavations. As discussed above, the use of temporary shoring could 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 MTO Special Provision (SP) 539S01. The temporary support system should be designed to Performance Level 3 as defined in SP 539S01. Roadway protection, if required, should be as per current MTO Special Provision 539S01.

If temporary shoring is employed for pile installation and pile cap construction at the abutments, groundwater inflows should be expected considering that up to 1.5 m of sand and/or silt exists immediately below the ground surface and water table at these location. Ground water inflow into the shored excavation should be controlled by a dewatering system (as discussed in Section 5.10.2) so that the pile cap construction can be carried out in the dry.

#### **5.10.1.2 Staged Excavation**

At the south (incl. retaining wall, if required) and north approaches, if excavation 'in-the-wet' for removal of the silty clay to clay 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 silty clay to clay 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.

### **5.10.2 Groundwater and Surface Water Control**

In the area of the south and north abutments, the groundwater level is generally at or within about 0.5 m of the ground surface. Behind the abutments in the areas of the approaches, the groundwater level is up to 1.5 m below 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 6 m deep excavations can be expected to occur due to the high groundwater levels and permeable nature of the sand strata underlying the silty clay to clay. 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 pile caps), the approximately 1.5 deep excavations may require groundwater control 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 the abutment areas, and considering the limits of dewatering proposed by Powers (1992), it is considered that the volume of seepage through the sands and silts may be slow 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.

## 5.11 CLOSURE

This Foundation Design Report was prepared by Mr. Chad Gilfillan, E.I.T. and Dr. J. Paul Dittrich, Ph.D., P.Eng., an Associate with Golder. Mr. Fintan J. Heffernan, P.Eng., Golder's Designated MTO Contact for this project, conducted an independent audit review of the report.

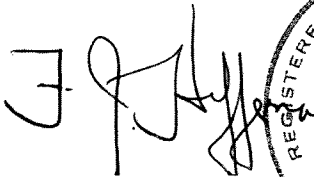
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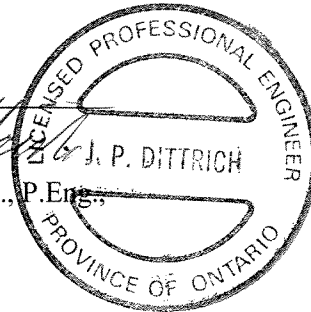
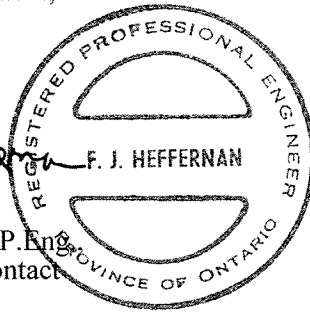
Chad M. Gilfillan, E.I.T.,  
Geotechnical Group



J. Paul Dittrich, Ph.D., P.Eng.,  
Associate



Fintan J. Heffernan, P.Eng.  
Designated MTO Contact



CMG/JPD/FJH/cmg/jpd/sm

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## **RECORD OF BOREHOLE SHEETS**

## 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  $= (\text{compressive strength})/2$
  - \* 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	

PROJECT		335-00-00		LOCATION		Hwy 69 SBL Sta. 10+621 Centre-line		ORIGINATED BY		EHS						
DIST		52		HWY		69		BOREHOLE TYPE		Power Auger 108 mm I.D. Hollow Stem Auger						
DATE		Geodetic		DATE		December 4, 2004		CHECKED BY		CG						
<b>RECORD OF BOREHOLE No SBL-1</b> <span style="float: right;">1 OF 1 <b>METRIC</b></span>																
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
197.8	GROUND SURFACE															
0.0	Topsoil, some sand		1	SS	3											
197.5																
0.4	Sandy Silt, trace clay		2	SS	10											
197.0	Loose															
0.8	Light brown, Oxidized Moist															
	Silty Clay to Clay, some silt, trace sand															
	Stiff to soft															
	Reddish brown to grey		3	SS	4											
	Moist to wet															
			4	SS	WH											
			5	SS	WH											
192.2																
5.6	Sand, some gravel, trace silt															
	Loose															
	Grey		6	SS	8											
	Wet															
190.7																
7.1	End of Borehole Auger Refusal															
	Note:															
	1. Water level in open borehole at 1.8 m depth upon completion of drilling.															

PROJECT <u>03-1111-028</u>		<b>RECORD OF BOREHOLE No SBL-2</b>		1 OF 1 <b>METRIC</b>										
W.P. <u>335-00-00</u>		LOCATION <u>Hwy 69 SBL Sta. 10+639 Centre-line</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>December 2, 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						
196.5	GROUND SURFACE													
0.0	Water													
0.2	Silty Sand, some organics (fibrous), trace gravel Very loose Light brown to brown, oxidized Wet		1	SS	1									
194.8			2	SS	1									
1.7	Silty Clay to Clay, trace sand Very soft to soft Mottled/varved reddish brown and grey Wet		3	SS	WH									
			4	SS	WH									
191.9			5	SS	25									
4.6	Sand, trace to some gravel, trace silt Loose to compact Grey Wet		6	SS	7									
			7	SS	20									
			8	SS	30									
187.8	Bedrock													
8.6	Refer to Record of Drillhole SBL-2 for details													
184.6														
11.9	End of Borehole													
	Notes:  1. Auger refusal at 8.6 m depth.  2. Borehole advanced within floodplain of Blair (Sly) Creek. Water level at borehole location 0.2 m above creek bed.													

PROJECT: 03-1111-028

**RECORD OF DRILLHOLE: SBL-2**

SHEET 1 OF 1

LOCATION: Hwy 69 SBL Sta. 10+639 Centre-line

DRILLING DATE: December 3, 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	RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.3 m	B Angle	DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY				Diametral Point Load Index (MPa)	RMC -Q AVG.	NOTES WATER LEVELS INSTRUMENTATION
								TOTAL CORE %	SOLID CORE %						K <sub>1</sub> cm/sec	K <sub>2</sub> cm/sec	K <sub>3</sub> cm/sec	K <sub>4</sub> cm/sec			
								88 													

DEPTH SCALE

1 : 50



LOGGED: EHS

CHECKED: CG

MIS-RCK 002 031111028AARCK.GPJ GAL-MISS.GDT 22/6/06 JFC

PROJECT 03-1111-028		<b>RECORD OF BOREHOLE No SBL-3</b>				1 OF 1 <b>METRIC</b>								
W.P. 335-00-00		LOCATION Hwy 69 SBL Sta. 10+678 Centre-line				ORIGINATED BY BL								
DIST 52 HWY 69		BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger				COMPILED BY KG								
DATUM Geodetic		DATE November 3, 2004				CHECKED BY CG								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
197.4	GROUND SURFACE													
0.0	Topsoil		1	SS	3									
196.8	Silty Sand, trace organics and rootlets (Fill)		2	SS	2									
0.6	Loose to very loose Brown Moist		3	SS	2									
	Organic Sandy Silt to Silty Sand, some wood fibres and rootlets		4	SS	WH									
	Very loose Dark brown to black Moist to wet													
194.5	Silty Clay to Clay, trace sand		5	SS	WH									
2.9	Soft to firm Grey Wet		6	SS	WH									
192.4	Sandy Silt to Sand and Silt		7	SS	WH									
5.1	Very loose to compact Grey Wet		8	SS	13									
190.9	Sand, trace to some silt, trace gravel		9	SS	13									
6.5	Compact Brown Wet		10	SS	16									
186.8	Bedrock		11	SS	3/15									
10.7	Refer to Record of Drillhole SBL-3 for details													
183.3	End of Borehole													
14.1	Notes:													
	1. Spoon Refusal at 10.7 m depth.													

SHEET 1 OF 1

DATUM: Geodetic

DRILLING CONTRACTOR: MARATHON DRILLING LTD.

CHECKED: CG

MIS-RCK 002 031111028AARCK.GPJ GAL-MISS.GDT 22/6/06 JFC

PROJECT		03-1111-028		<b>RECORD OF BOREHOLE No SBL-4</b>		1 OF 1 <b>METRIC</b>														
W.P.		335-00-00		LOCATION		Hwy 69 SBL Sta. 10+685 Centre-line														
DIST		52 HWY 69		BOREHOLE TYPE		Power Auger 108 mm I.D. Hollow Stem Auger														
DATUM		Geodetic		DATE		November 3, 2004														
						ORIGINATED BY BL														
						COMPILED BY KG														
						CHECKED BY CG														
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					
197.7	0.0	GROUND SURFACE							20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED			20 40 60 W <sub>p</sub> W W <sub>L</sub>			20 40 60 W <sub>p</sub> W W <sub>L</sub>			GR SA SI CL		
		Organic Sandy Silt to Silty Sand, some wood fibres and rootlets Dark brown to black Very loose Moist		1	SS	2		197												
				2	SS	2														
196.1		Silty Sand Very loose Grey Wet		3	SS	2		196												
195.7	2.0	Silty Clay Very soft to soft Mottled brownish grey to grey Moist to wet		4	SS	WH		195												
				5	SS	TO														
								194												
				6	SS	WH		193												
		Becoming Clayey Silt below 5.1 m depth																		
								192												
191.6	6.1	Sandy Silt Compact Grey Wet		7	SS	19		191												
190.7	7.0	Sand, trace to some silt, trace gravel Loose to compact Brown to grey Wet						190												
				8	SS	5														
								189												
				9	SS	19		188												
								187												
				10	SS	25														
186.3	11.4	End of Borehole Auger refusal																		
		Notes:  1. Water level in piezometer measured at 1.1 m depth on November 14, 2004.																		

MIS-MTO 001 031111028AAMTO.GPJ GAL-MISS.GDT 22/6/06

PROJECT		335-00-00		LOCATION		Hwy 69 SBL Sta. 10+588 Centre-line		ORIGINATED BY		SB						
DIST		52		HWY		69		BOREHOLE TYPE		Power Auger 108 mm I.D. Hollow Stem Auger						
COMPILED BY		LG		DATE		March 25, 2004		CHECKED BY		CG						
<div style="display: flex; justify-content: space-between;"> <span>PROJECT 03-1111-028</span> <span><b>RECORD OF BOREHOLE No 04-73</b></span> <span>1 OF 1 <b>METRIC</b></span> </div>																
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
197.2	GROUND SURFACE															
196.9	Fibrous Peat		1	SS	2											
196.6	Silty Sand, trace clay															
196.0	Loose Grey Moist															
	Silty Clay, trace sand															
	Very soft Brown and grey Moist		2	SS	WH											
194.3	Sand															
3.1	End of BH Auger refusal Spoon refusal		3	SS	60/0											
Note: Water level in open borehole at 1.8m depth upon completion of drilling.																

PROJECT 03-1111-028			RECORD OF BOREHOLE No 04-75			1 OF 1 METRIC		
W.P. 335-00-00			LOCATION Hwy 69 SBL Sta. 10+612 Centre-line			ORIGINATED BY SB		
DIST 52 HWY 69			BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger			COMPILED BY LG		
DATUM Geodetic			DATE March 25, 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 Wp NATURAL MOISTURE CONTENT W LIQUID LIMIT Wl WATER CONTENT (%)
197.3	GROUND SURFACE							
0.0	Fibrous Peat							
0.2	Sandy Silt, trace clay Loose to very loose Grey Moist		1	SS	3		197	
196.2			2	SS	1		196	
1.1	Silty Clay to Clay, trace sand and rootlets Very soft to soft Brown and grey Moist to wet		3	TO	PH		195	
			4	SS	WH		194	
193.5			5	SS	15		193	
3.8	Fine to Medium Sand, trace gravel Compact Grey Wet		6	SS	4/15			
192.5								
4.8	End of BH Auger refusal Spoon refusal  Note: Water level in open borehole at 1.8m depth upon completion of drilling.							

PROJECT 03-1111-028			RECORD OF BOREHOLE No 04-105			1 OF 1 METRIC		
W.P. 335-00-00			LOCATION Hwy 69 SBL Sta. 10+620 o/s L 10.8 m			ORIGINATED BY EHS		
DIST 52 HWY 69			BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Augers			COMPILED BY DD		
DATUM Geodetic			DATE Oct. 27, 2004			CHECKED BY BL		
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 (%)
197.1	GROUND SURFACE						197	
0.0	Topsoil		1	SS	10		196	
0.3	Silty fine Sand, trace organics						195	
0.8	Compact Light brown, oxidized Moist		2	SS	10		194	
196.3	Clay, some silt, trace fine sand, trace organics Soft to firm Mottled grey and reddish brown Wet		3	SS	3		193	
194.5	Silty Clay to Clay, trace fine sand Soft Grey Wet		4	SS	WH		192	
192.5	Silt, some fine sand, trace clay, trace organics Very loose Grey Wet		5	SS	WH		191	
191.0	Fine to medium Sand, some gravel, trace silt Loose to dense Grey Wet		6	SS	6		190	
189.3	Augers grinding		7	SS	46/10			
7.8	End of BH Spoon Refusal Auger Refusal  Note: 1. Water level at 2.0 m depth upon completion of drilling							

PROJECT <u>03-1111-028</u>		<b>RECORD OF PENETRATION TEST No DC04-32</b>				1 OF 1 <b>METRIC</b>										
W.P. <u>335-00-00</u>		LOCATION <u>Hwy 69 SBL Sta. 10+599.5 o/s L 9.6m</u>				ORIGINATED BY <u>SB</u>										
DIST <u>52</u> HWY <u>69</u>		BOREHOLE TYPE <u>Dynamic Cone Penetration Test</u>				COMPILED BY <u>LG</u>										
DATUM <u>Geodetic</u>		DATE <u>March 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 $\gamma$	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa								
197.3 0.0	GROUND SURFACE						20	40	60	80	100					
193.7 3.6	End of DCPT Refusal to penetration Hammer rebounding						20	40	60	80	100					

PROJECT <u>03-1111-028</u>		<b>RECORD OF PENETRATION TEST No DC04-45</b>				1 OF 1 <b>METRIC</b>										
W.P. <u>335-00-00</u>		LOCATION <u>Hwy 69 SBL Sta. 10+622.5 o/s R 10.8m</u>				ORIGINATED BY <u>EHS</u>										
DIST <u>52</u> HWY <u>69</u>		BOREHOLE TYPE <u>Dynamic Cone Penetration Test</u>				COMPILED BY <u>DD</u>										
DATUM <u>Geodetic</u>		DATE <u>October 27, 2004</u>				CHECKED BY <u>BML</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	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
197.5 0.0	GROUND SURFACE					20	40	60	80	100						
197																
196																
195																
194																
193																
192																
191.4 6.1	End of DCPT Refusal to penetration Hammer rebounding															

## **TABLES**

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

PROJECT NO 03-1111-028							
LOCATION: Proposed Highway 69 SBL / Blair (Sly) Creek Bridge							
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)
SBL-2	1	Granitic Gneiss	28.9	8.8	D	5.109	118
SBL-2	2	Granitic Gneiss	30.4	9.3	A	9.383	216
SBL-2	3	Granitic Gneiss	31.5	9.6	D	4.982	115
SBL-2	4	Granitic Gneiss	33.9	10.3	A	7.192	165
SBL-2	5	Granitic Gneiss	34.5	10.5	D	5.237	120
SBL-2	6	Granitic Gneiss	36.2	11.0	D	6.531	150
SBL-3	1	Granitic Gneiss	35.2	10.7	D	6.293	145
SBL-3	2	Granitic Gneiss	36.3	11.1	A	10.225	235
SBL-3	3	Granitic Gneiss	39.2	12.0	D	5.680	131
SBL-3	4	Granitic Gneiss	39.8	12.1	A	9.050	208
SBL-3	5	Granitic Gneiss	41.7	12.7	D	4.735	109
SBL-3	6	Granitic Gneiss	44.4	13.5	D	5.314	122
<b>SUMMARY</b>					<b>Average Axial</b>	<b>8.962</b>	<b>206</b>
					<b>Average Diametral</b>	<b>5.485</b>	<b>126</b>
					<b>St. Dev. Axial</b>	<b>1.280</b>	<b>29</b>
					<b>St. Dev. Diametral</b>	<b>1.168</b>	<b>27</b>
					<b>Number of Axial Tests</b>	<b>4</b>	
					<b>Number of Diametral Tests</b>	<b>8</b>	

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

Note: Specimens tend to be anisotropic in nature (ie. stronger axial than diametral).

N:\Active\2003\1111\03-1111-028 URS Rev 69 Parry Sound\Reporting\Final\3 - Blair Creek  
- SBL Tables\03-1111-028-3 Table 1 Hwy69 SBL-Blair Creek P.T.X19\POINT LOAD

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

<i>Footing Option</i>	<i>NF</i>	<i>Advantages</i>	<i>Disadvantages</i>	<i>Relative Costs</i>	<i>Risks/Consequences</i>
Steel H Piles		Relatively straight forward construction. Allows for integral abutment design.	<p>Downdrag 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.	Granular fill (less than 75 mm sizes) must be placed in abutment area for pile driving considerations if sub-excavation and removal is adopted as approach embankment stability/settlement mitigation alternative.
Spread Footings perched within embankment fill		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 differential settlements anticipated between north and south abutment.
Spread Footings on bedrock or mass concrete pad	X		<p>Deep (8 m to 11 m) excavations required. Temporary shoring and groundwater control required to expose bedrock surface.</p> <p>Variable bedrock surface will require mass concrete placement to achieve level footing.</p>	Increased cost for groundwater control and temporary shoring as compared with shallower footings.	Not recommended due to significant depth of excavations.
Shallow Spread Footings on loose silty sand, soft silty clay, or compact sand at depth	X		<p>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.</p> <p>Deep (5 m) excavations required to found on compact sand. Temporary shoring and groundwater control required to expose compact sand at depth.</p>	<p>Lower relative costs than piled foundations for shallow foundations.</p> <p>Increased cost for groundwater control and temporary shoring for footings on compact sand at depth.</p>	Not recommended due to potential for differential settlements anticipated between north and south abutment.

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**NF:** Indicates that the founding option is considered not feasible.

**TABLE 3**  
**EVALUATION OF RETAINING WALL ALTERNATIVES**  
**Highway 69 SBL Structure at Blair (Sly) Creek**  
**G.W.P. 335-00-00**

<i>Footing Option</i>	<i>NF</i>	<i>Advantages</i>	<i>Disadvantages</i>	<i>Relative Costs</i>	<i>Risks/Consequences</i>
Concrete Cantilever Wall founded on rock fill after sub-excavation and replacement of soft clays.		Settlement of foundation soils reduced. Adequate factor of safety for global stability achieved. Potential for undermining of wall backfill minimized by concrete cantilever design. No preloading period required.	Settlement of rock fill foundation of up to about 45 mm.  Permanent sheet pile shoring required to support and minimize impact on the adjacent Blair Creek. High groundwater table and permeable layers below clay strata will require sub-excavation and replacement 'in-the-wet'. Staged excavation in short strips required to maintain stability of sheet pile shoring.	Increased costs for sub-excavation of soft soils and replacement with rock fill. Additional costs for sheet pile shoring along Blair Creek.	Differential settlement with respect to adjacent piled abutment. Potential for some impact on creek still exists.
Concrete Cantilever Wall (with granular backfill) supported on near continuous caisson wall installed through native soils.		No sub-excavation and replacement or preloading of foundation soils required prior to wall construction. No sheet pile shoring required at Blair Creek. Potential for undermining of wall backfill minimized by concrete cantilever design.	Caisson foundations for supporting wall must be designed to maintain global stability of backfill and to resist the lateral spread of the soft foundation soils. Settlements of backfill ranging from 125 mm to 210 mm will still occur following completion of backfilling. Delay of about 16 months required after backfilling wall and before constructing pavement structure.	High costs for near continuous caisson wall (i.e. 1 m diameter concrete caissons installed at 2 m spacings).	Socketting of caissons very difficult into strong to very strong granitic gneiss bedrock. As such, caissons must be fixed to bedrock with grouted dowels.
Concrete Cantilever Wall (with granular and EPS backfill) supported on steel H-piles installed through partially preloaded native soils.		No sub-excavation and replacement of foundation soils required. No sheet pile shoring required at Blair Creek. Potential for undermining of wall backfill minimized by concrete cantilever design. Piles can be designed neglecting lateral loads and downdrag loads.	Partial preloading of wall backfill / approach embankment required for 16 months prior to wall construction to reduce settlements and minimize lateral loading on H-pile wall foundations. After preloading, EPS fill required to raise backfill to grade in order to maintain stability and to minimize settlements and lateral movements in foundation soils after wall construction.	Increased costs for preload construction and partial fill removal. Increased costs for H-piles and for partial EPS backfill behind wall.	16 month delay for preload period prior to wall construction.
Concrete Cantilever Wall (with full EPS backfill) supported on steel H-piles installed through native soils.		Minimal sub-excavation and replacement of foundation soils required. No sheet pile shoring required at Blair Creek. Potential for undermining of wall backfill minimized by concrete cantilever design. Piles can be designed neglecting	Placement of conventional fill for pavement structure on top of EPS must be offset by partial sub-excavation and replacement with additional EPS fill below existing grade to achieve a zero net loading condition.  Retaining wall must still be supported on H-piles due to low bearing capacity of weak foundation	Increased costs for H-piles plus high costs for large volume of EPS fill required.	

**TABLE 3**  
**EVALUATION OF RETAINING WALL ALTERNATIVES**  
**Highway 69 SBL Structure at Blair (Sly) Creek**  
**G.W.P. 335-00-00**

<i>Footing Option</i>	<i>NF</i>	<i>Advantages</i>	<i>Disadvantages</i>	<i>Relative Costs</i>	<i>Risks/Consequences</i>
		lateral loads and downdrag loads. EPS reduces load on compressible soils thereby increasing global stability, minimizing settlements and avoiding requirement for preloading period.	soils.  Very high material costs.		
Retained Soil System (RSS) Wall founded on native soils	X	Minimal excavation required. No sheet pile shoring required at Blair Creek.	Settlements of native silty clay ranging from about 125 mm to 210 mm. Magnitude of settlement could be reduced if partial preloading carried out. Full preloading not possible due to global stability problems and/or requirements for large berms in-filling Blair Creek.  Factor of Safety < 1.3 for global stability of wall on soft clay foundation with and without preloading.	Less expensive than having to sub-excavate and replace soft strata prior to construction.	Not recommended due to problems with global stability and large magnitudes of settlement. In addition, erosion/undermining of wall backfill (causing potential internal instability of RSS system) possible due to close proximity to Blair Creek.
Retained Soil System (RSS) Wall founded on rock fill after sub-excavation and replacement of soft clays.	X	Settlement of foundation soils reduced. Global stability improved. No preloading period required.	Settlement of rock fill foundation of up to about 45 mm.  Permanent sheet pile shoring required to support the adjacent Blair Creek. High groundwater table and permeable layers below clay strata will require sub-excavation and replacement 'in-the-wet'. Staged excavation in short strips required to maintain stability of sheet pile shoring.	Increased costs for sub-excavation of soft soils and replacement with rock fill. Additional costs for sheet pile shoring along Blair Creek.	Not recommended due to potential for erosion/undermining of wall backfill (causing internal instability of RSS system) due to close proximity to Blair Creek. Potential for some impact on creek still exists.

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**NF:** Indicates that the founding option is considered not feasible.

**TABLE 4**  
**EVALUATION OF SETTLEMENT / STABILITY MITIGATION ALTERNATIVES**  
**South Approach Embankment - Highway 69 SBL 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 5.5 m deep)	1	Stability and long-term settlement issues minimized since all or nearly all weak, soft and compressible materials are removed. Stability berms not required.	If Blair Creek cannot be re-aligned, permanent sheet pile shoring will be required to support the adjacent creek on the west side of the embankment. Permanent sheet pile shoring may be required locally at the northwest corner of the embankment whether the creek is re-aligned or not. High groundwater table and permeable layers below clay strata will 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 permanent sheet pile shoring along the edge of the adjacent Blair Creek.	Low 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.
Pre-Loading and Stability Berms (1.5 m high x 1.5 m wide) assuming Blair Creek can be re-aligned.	2	Relatively simple operation; no deep subexcavation or temporary shoring required.	Lengthened construction time required. Settlement of foundation soil may take 13 to 16 months to reach 90% consolidation.	Low cost. However some additional costs required for berm construction.	Settlement of embankment/ foundation soils will occur. Secondary consolidation (creep) will occur.
Partial Pre-Loading and Partial Filling / Stability Berms (4.75 m wide x 2 m high at south end, 2.75 m wide x 2.5 m high at north end) on West Side. Partial EPS Fill.	3	Relatively simple operation; no deep sub-excavation or temporary shoring. Toe berms / partial embankment filling required on west slope to provide stability.	Lengthened construction time required. Settlement of foundation soil may take 13 to 16 months to reach 90% consolidation. EPS fill required to raise embankment to required grade (after preloading) to maintain stability if Blair Creek cannot be re-aligned.	Additional costs due to preload construction and partial fill removal. Increased costs for partial EPS fill required.	Settlement of embankment/ foundation soils reduced. Secondary consolidation (creep) will occur. Risk of differential settlements along roadway due to combination of conventional fill and EPS fill.

**TABLE 4**  
**EVALUATION OF SETTLEMENT / STABILITY MITIGATION ALTERNATIVES**  
**South Approach Embankment - Highway 69 SBL 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 (1 m or 1.5 m triangular spacing and 5.5 months or 8.5 months preload time) with Toe Berms and/or Partial EPS Fill	4	Reduce the partial preload duration. 90 % consolidation settlement complete within 5.5 months or 8.5 months of partial embankment construction.	Increased time for installation of wicks. Monitoring of settlements and pore pressures required.  Toe berms still required for stability on west slope (as indicated above). EPS fill still required to raise embankment to required grade (after preloading) to maintain stability if Blair Creek cannot be re-aligned.	Cost savings in sub-excavation and replacement offset by drain installation and monitoring costs. Increased costs for partial EPS fill required.	Settlement of embankment/ foundation soils will occur; lower risk that 90 percent consolidation settlement will not occur during preload period. Secondary consolidation (creep) will occur. Risk of differential settlements along roadway due to combination of conventional fill and EPS fill.
Light Weight Fill (EPS) Full Embankment Construction (40 m long x 18 m wide x 3.5 m high)	5	Reduces load on compressible soils thereby increasing stability and reducing settlement of foundation soils. Settlement of embankment fill minimized.	Very high material costs.	Cost savings in berm fill or sub-excavation and replacement, but relative cost of 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	NF		If Blair Creek cannot be re-aligned, surcharging cannot be carried out without constructing large toe berms on the front, east and west embankment slopes, which would encroach into Blair Creek. If Blair Creek can be re-aligned, large toe berms would still be required at the front and northwest embankment slopes.	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.

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**NF:** Indicates that the founding option is considered not feasible.

**TABLE 5**  
**EVALUATION OF SETTLEMENT / STABILITY MITIGATION ALTERNATIVES**  
**North Approach Embankment - Highway 69 SBL 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 6 m deep)	1	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 will be required to support the adjacent Blair Creek on the east side and at the southeast corner of the embankment. High groundwater table and permeable layers below clay strata will 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 permanent sheet pile shoring along the edge of the adjacent Blair Creek.	Low 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.
Partial Pre-Loading and Partial Filling / Stability Berms (5.5 m wide x 2 m high on east side, 3 m wide x 2 m high on west side, 8.5 m wide x 2 m high on front slope) combined with Partial EPS Fill.	2	Relatively simple operation; no deep sub-excavation or temporary shoring. Toe berms / partial embankment filling required on east, west and front slopes to provide stability.	Lengthened construction time required. Settlement of foundation soil may take 16 months to reach 90% consolidation. EPS fill required on east side and front slope to raise embankment to required grade (after preloading) to maintain stability.	Additional costs due to preload construction and partial fill removal. Increased costs for partial EPS fill required.	Settlement of embankment/ foundation soils reduced. Secondary consolidation (creep) will occur. Risk of differential settlements along roadway due to combination of conventional fill and EPS fill.
Wick Drains (1 m or 1.5 m triangular spacing and 5.5 months or 8.5 months preload time) with Toe Berms and Partial EPS Fill	3	Reduce the partial preload duration. 90 % consolidation settlement complete within 5.5 months or 8.5 months of partial embankment construction.	Increased time for installation of wicks. Monitoring of settlements and pore pressures required.  Toe berms still required for stability on east, west and front slopes (as indicated above). EPS fill required on east side and front slope to raise embankment to required grade (after preloading) to maintain stability.	Cost savings in sub-excavation and replacement offset by drain installation and monitoring costs. Increased costs for partial EPS fill required.	Settlement of embankment/ foundation soils will occur; lower risk that 90 percent consolidation settlement will not occur during preload period. Secondary consolidation (creep) will occur. Risk of differential settlements along roadway due to combination of conventional fill and EPS fill.

**TABLE 5**  
**EVALUATION OF SETTLEMENT / STABILITY MITIGATION ALTERNATIVES**  
**North Approach Embankment - Highway 69 SBL 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>
Light Weight Fill (EPS) Full Embankment Construction (20 m long x 18 m wide x 3.5 m high)	4	Reduces load on compressible soils thereby increasing stability and reducing settlement of foundation soils. Settlement of embankment fill minimized.	Very high material costs.	Cost savings in berm fill or sub-excavation and replacement, but relative cost of 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	NF		Surcharging cannot be carried out without constructing large toe berms on the east and front embankment slopes that would encroach into Blair Creek. Larger toe berm also required on west slope	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.

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**NF:** Indicates that the founding option is considered not feasible.

**TABLE 6**  
**Summary of Recommendations at Structure Approach Embankments (incl. Platform Widening)**  
**Highway 69 SBL Structure at Blair (Sly) 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 ( $\delta$ ) <sup>*</sup> and Platform Widening (w) <sup>**</sup> (mm)	Swamp Excavation / Organic Removal OPSD
Highway 69 SBL at Blair (Sly) Creek Structure	10+600 to 10+639	South Approach Swamp crossing (fill up to 4.5 m high)	Swamp area adjacent to Blair Creek. Soft clay to a depth of up to 5.5 m.	Rock fill	Yes. Up to 0.3 m below ground surface.	1.25H : 1V	No.	$\delta = 45 + 45 = 90$ w = 1000	203.010 (new embankment construction over swamp)
	10+678 to 10+698	North Approach Swamp crossing (fill up to 4.5 m high)	Swamp area in flood plain adjacent to Blair Creek. Soft Clay to a depth of up to 6 m.	Rock fill	Yes. Up to 2.9 m below ground surface.	1.25H: 1V	No.	$\delta = 45 + 55 = 100$ w = 1000	203.010 (new embankment construction over swamp)

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**Note :** \* Settlements include compression of rockfill plus compression of cohesive layers below embankment (where encountered). Estimates assumes that sub-excavation and removal of soft clay has been adopted as mitigation measure.

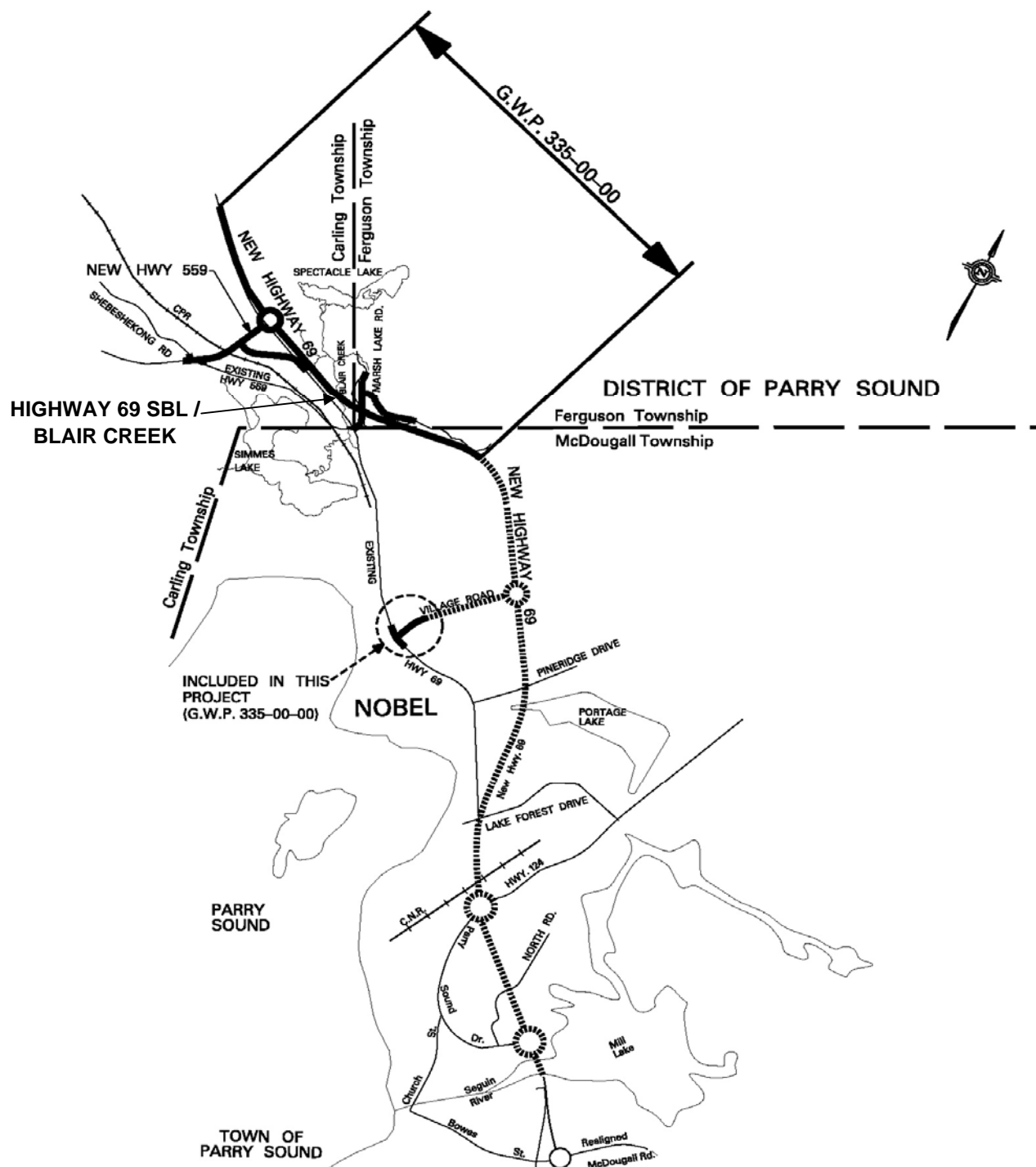
\*\* Recommended embankment platform widening (per embankment side) based on guidelines in NRE 98-200.

## **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: February 2006  
Project: 03-1111-028-3

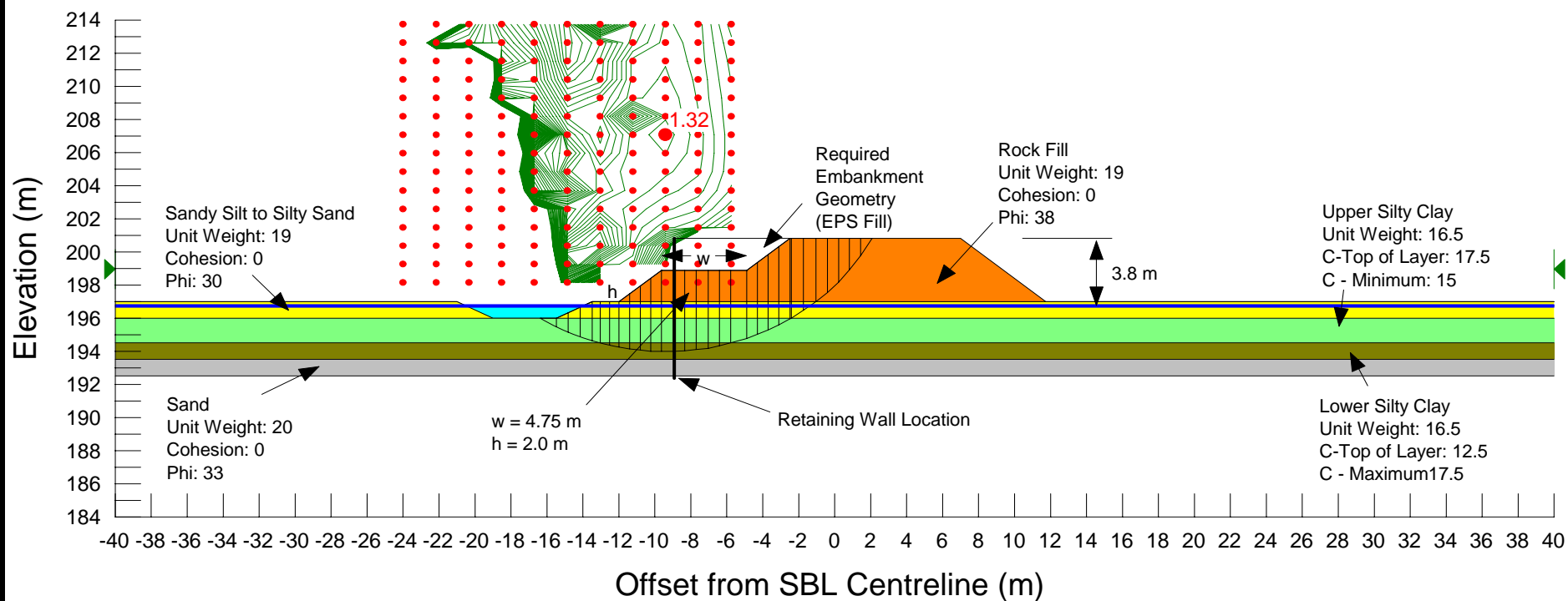
Drawn by: CMG  
Checked by: JPD

Golder Associates

Provided in digital format by URS on January 7, 2005

**SOUTHBOUND HIGHWAY 69 STRUCTURE OVER BLAIR (SLY) CREEK**  
**SOUTH APPROACH EMBANKMENT - SOUTH END**

**FIGURE 2**



File Name: 03-1111-028 SBL Bridge South Approach\_RetainingWall\_SouthEnd\_MaxFill.siz  
 Last Saved Date: 6/30/2005

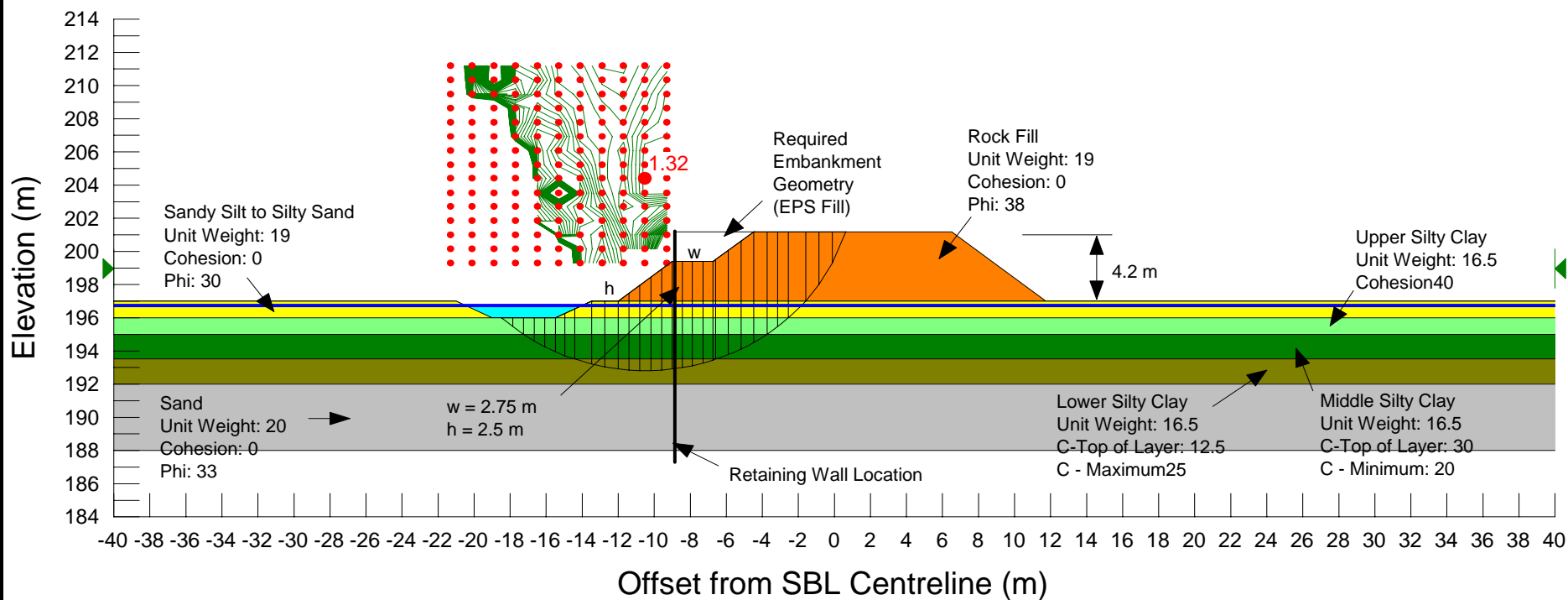
Date: February 2006  
 Project: 03-1111-028-3

**Golder Associates**

Drawn: CMG  
 Checked: JPD

# **SOUTHBOUND HIGHWAY 69 STRUCTURE OVER BLAIR (SLY) CREEK** **SOUTH APPROACH EMBANKMENT - NORTH END**

**FIGURE 3**



File Name: 03-1111-028 SBL Bridge South Approach\_RetainingWall\_NorthEnd\_MaxFill.slz  
 Last Saved Date: 6/28/2005

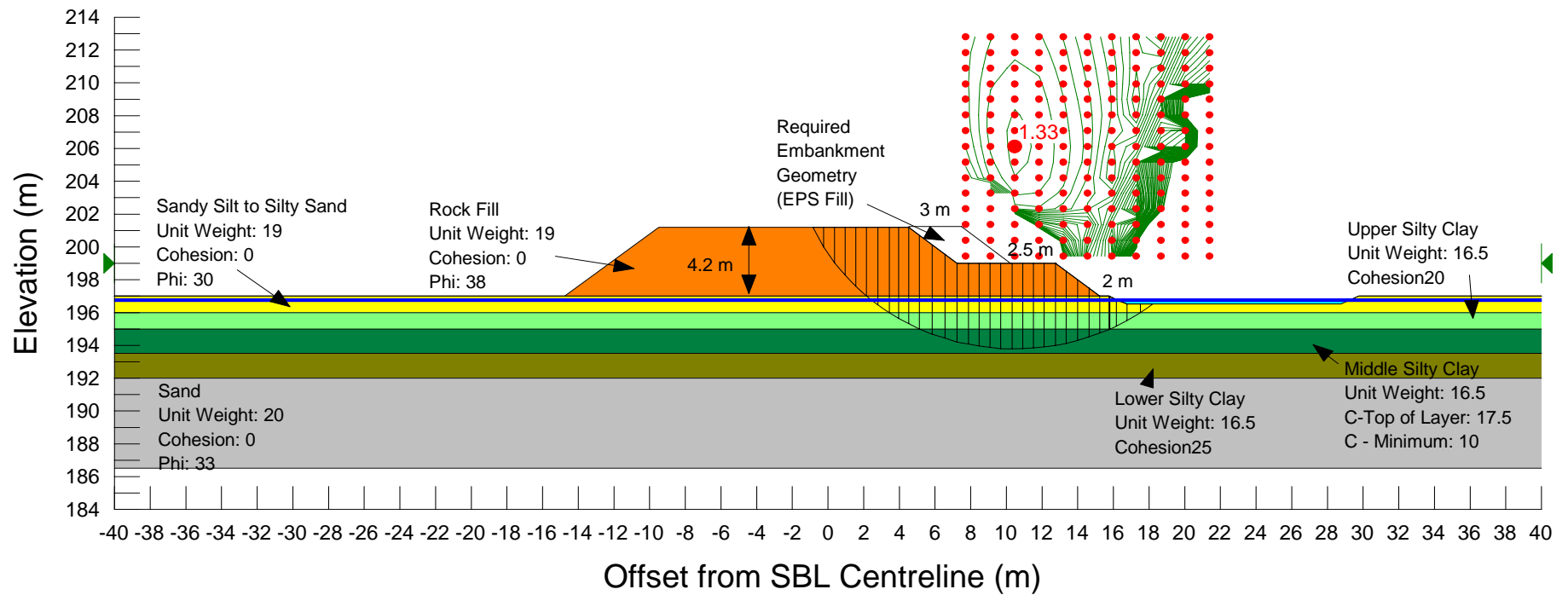
Date: February 2006  
 Project: 03-1111-028-3

**Golder Associates**

Drawn: CMG  
 Checked: JPD

# **SOUTHBOUND HIGHWAY 69 STRUCTURE OVER BLAIR (SLY) CREEK** **NORTH APPROACH EMBANKMENT - EAST SIDE**

**FIGURE 4**



File Name: 03-1111-028 SBL Bridge North Approach\_MaxFill.slz  
 Last Saved Date: 6/27/2005

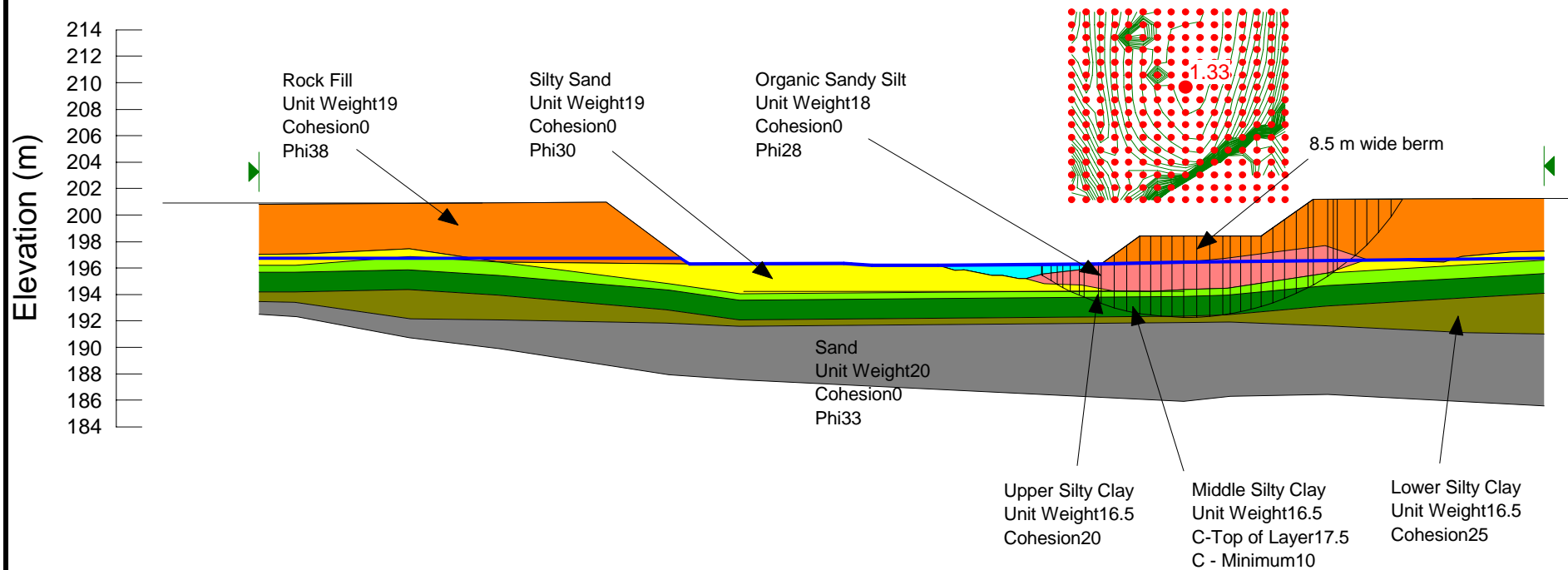
Date: February 2006  
 Project: 03-1111-028-3

**Golder Associates**

Drawn: CMG  
 Checked: JPD

**SOUTHBOUND HIGHWAY 69 STRUCTURE OVER BLAIR (SLY) CREEK**  
**NORTH APPROACH EMBANKMENT FRONT SLOPE STABILITY**

**FIGURE 5**



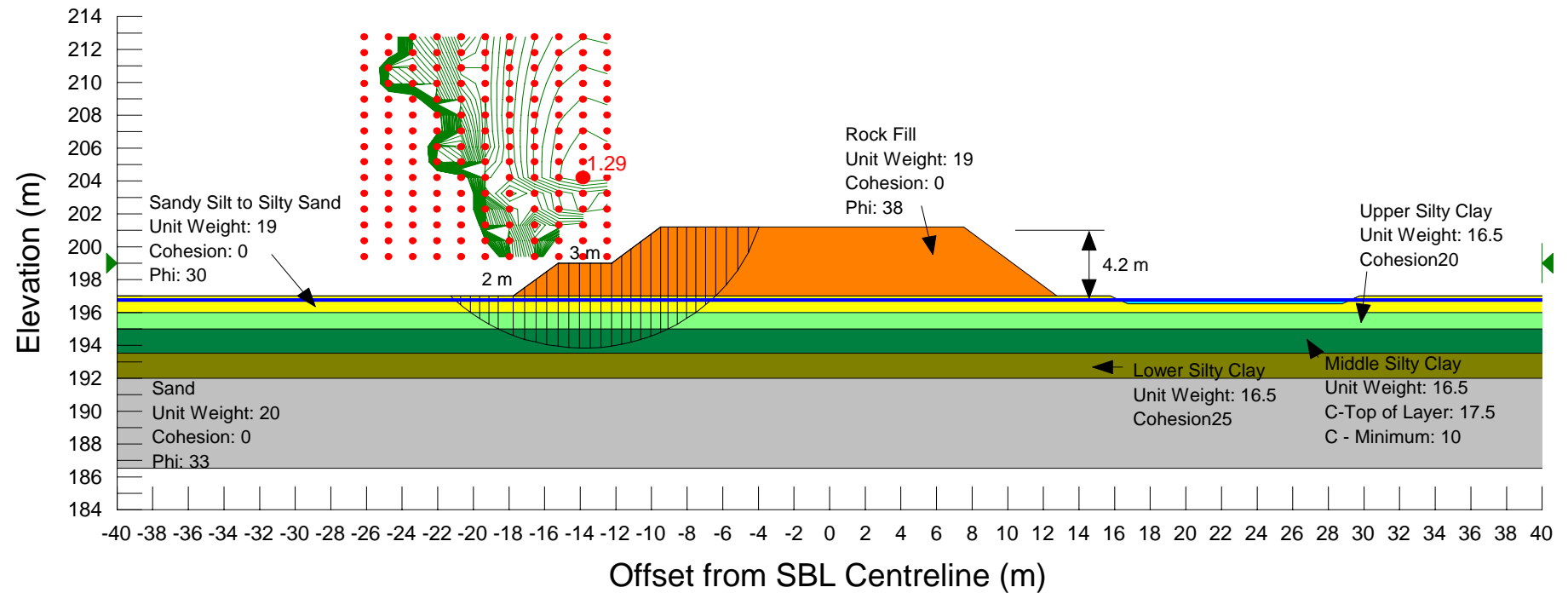
Date: February 2006  
 Project: 03-1111-028-3

**Golder Associates**

Drawn: CMG  
 Checked: JPD

**SOUTHBOUND HIGHWAY 69 STRUCTURE OVER BLAIR (SLY) CREEK**  
**NORTH APPROACH EMBANKMENT - WEST SIDE**

**FIGURE 6**



File Name: 03-1111-028 SBL Bridge North Approach\_WestSide\_Berm.siz  
 Last Saved Date: 6/27/2005

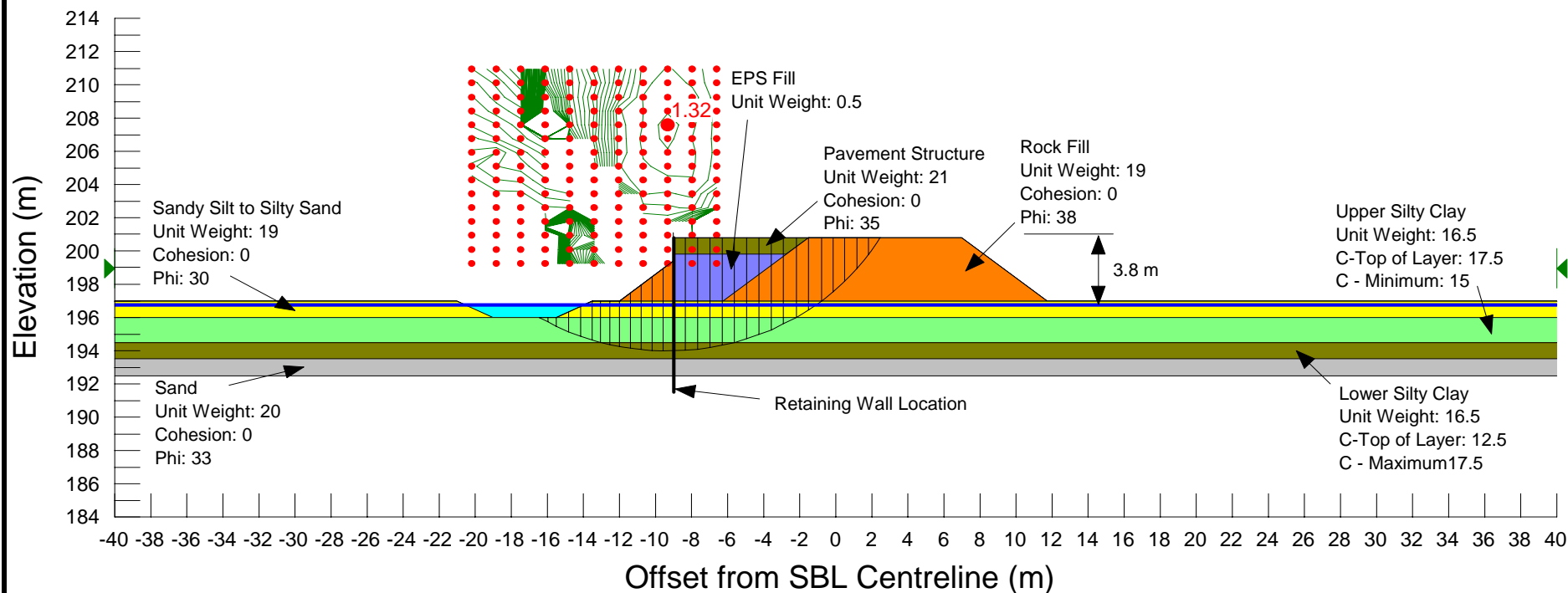
Date: February 2006  
 Project: 03-1111-028-3

**Golder Associates**

Drawn: CMG  
 Checked: JPD

**SOUTHBOUND HIGHWAY 69 STRUCTURE OVER BLAIR (SLY) CREEK**  
**SOUTH APPROACH EMBANKMENT - SOUTH END - RETAINING WALL WITH EPS**

**FIGURE 7**



File Name: 03-1111-028 SBL Bridge South Approach\_RetainingWall\_SouthEnd\_EPS.slz  
 Last Saved Date: 4/24/2005

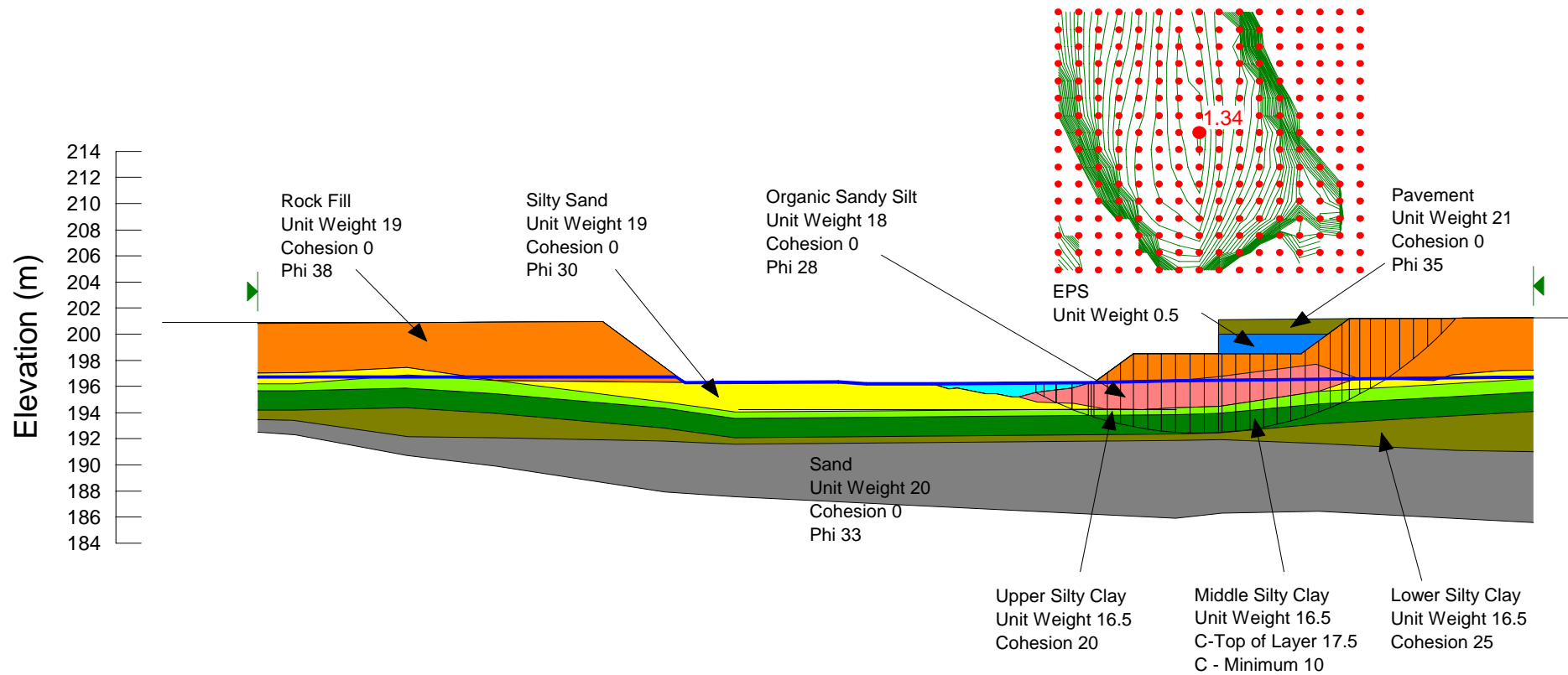
Date: February 2006  
 Project: 03-1111-028-3

**Golder Associates**

Drawn: CMG  
 Checked: JPD

**SOUTHBOUND HIGHWAY 69 STRUCTURE OVER BLAIR (SLY) CREEK**  
**NORTH APPROACH EMBANKMENT FRONT SLOPE STABILITY WITH EPS FILL**

**FIGURE 8**



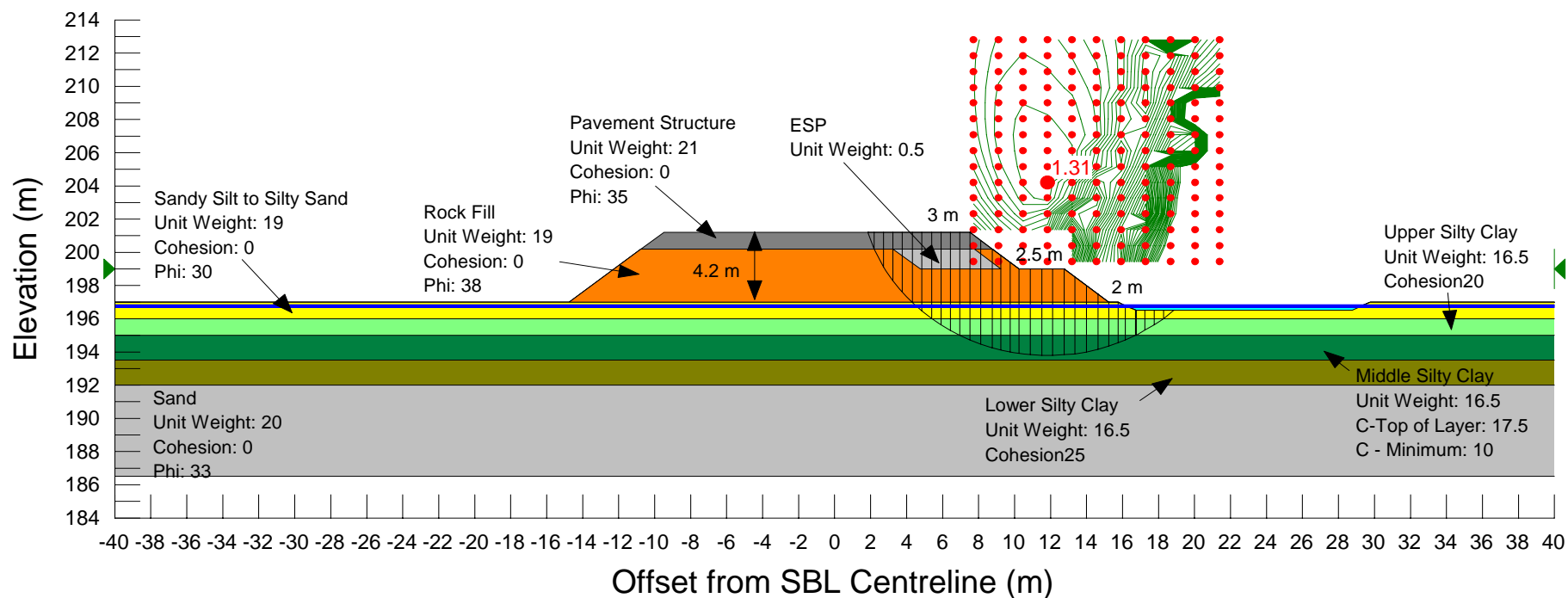
Date: February 2006  
 Project: 03-1111-028-3

**Golder Associates**

Drawn: CMG  
 Checked: JPD

**SOUTHBOUND HIGHWAY 69 STRUCTURE OVER BLAIR (SLY) CREEK**  
**NORTH APPROACH EMBANKMENT - EAST SIDE - WITH EPS**

**FIGURE 9**



File Name: 03-1111-028 SBL Bridge North Approach\_MaxFill\_EPS(4.5m).slz  
 Last Saved Date: 6/28/2005

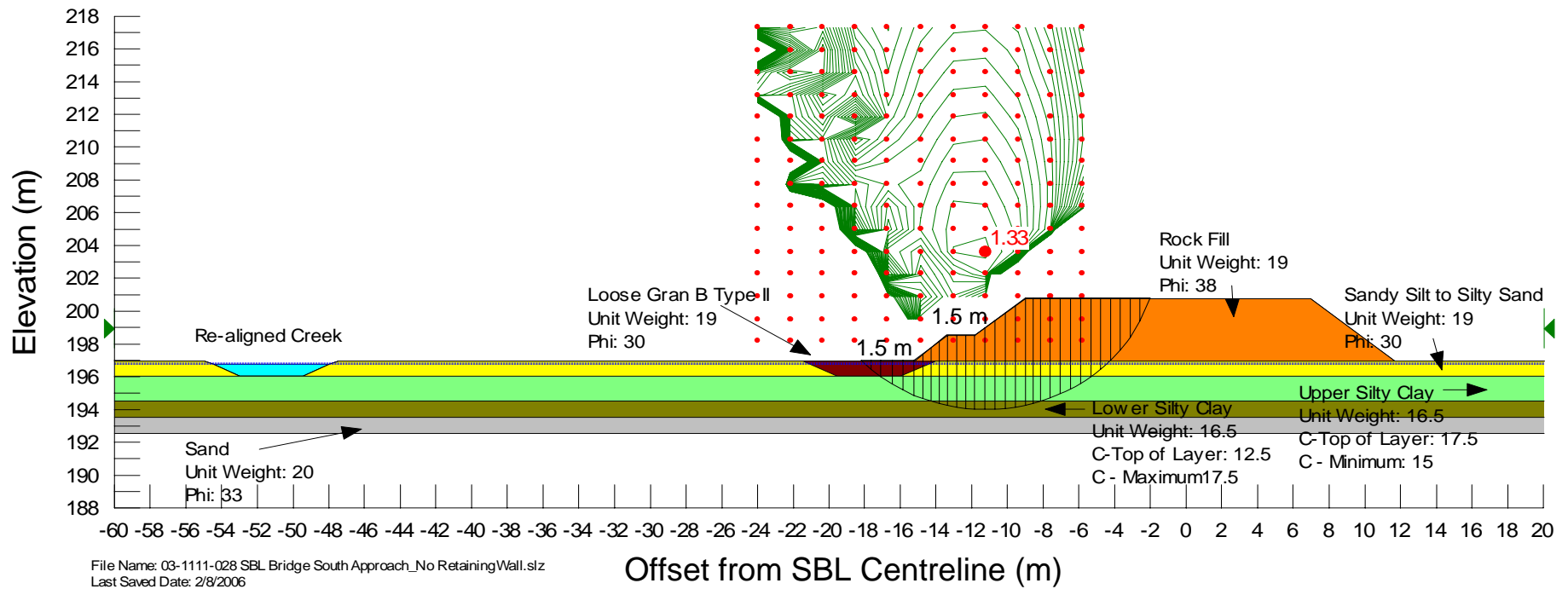
Date: February 2006  
 Project: 03-1111-028-3

**Golder Associates**

Drawn: CMG  
 Checked: JPD

**SOUTHBOUND HIGHWAY 69 STRUCTURE OVER BLAIR (SLY) CREEK**  
**SOUTH APPROACH EMBANKMENT - RE-ALIGNED BLAIR CREEK**

**FIGURE 10**

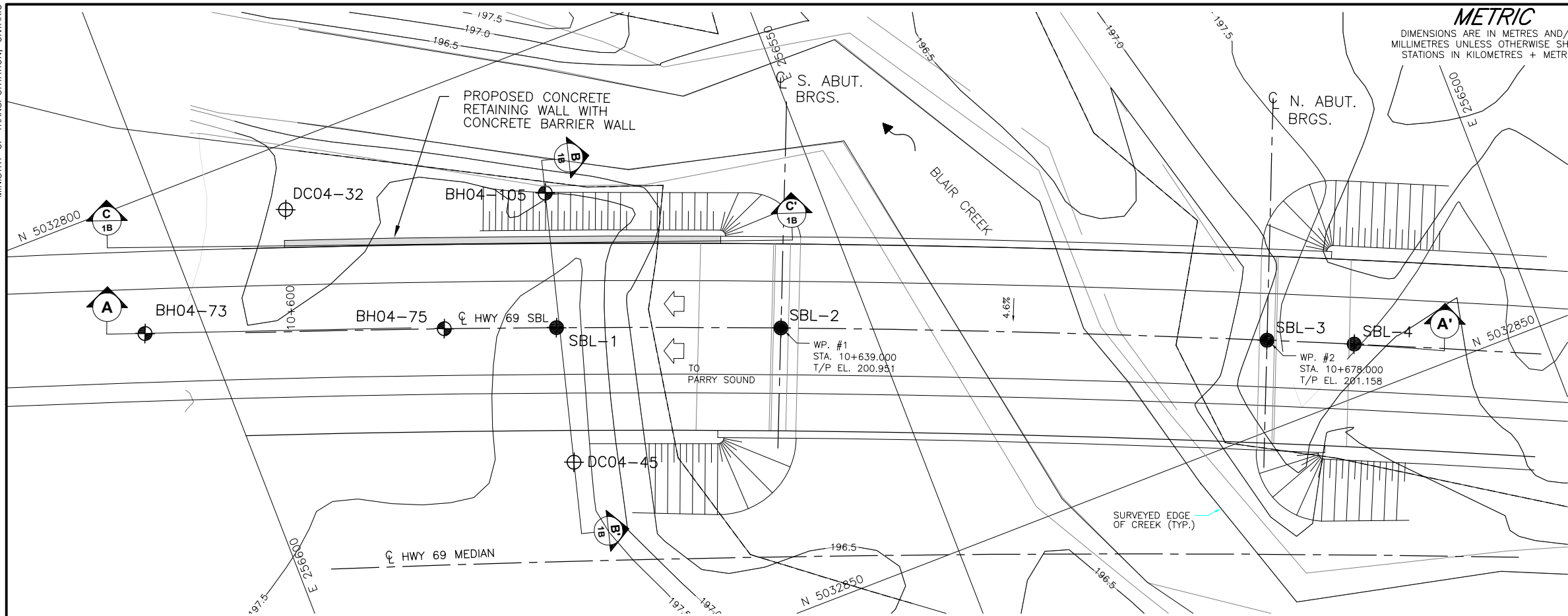


Date: February 2006  
 Project: 03-1111-028-3

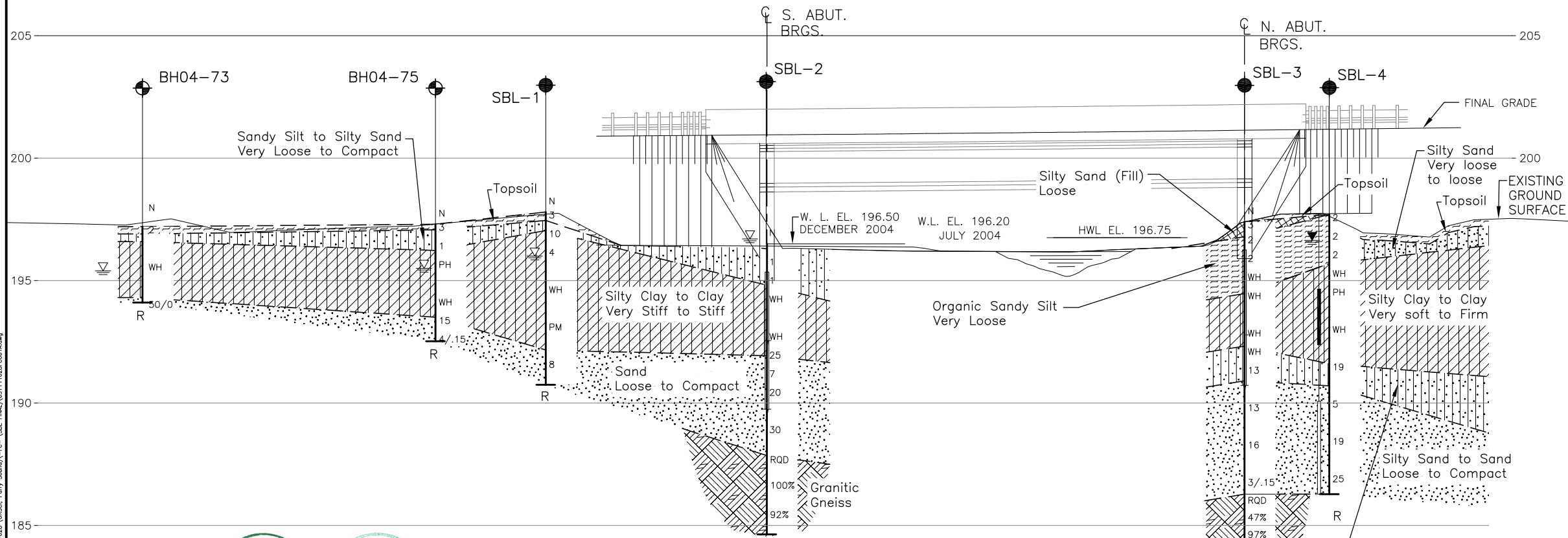
**Golder Associates**

Drawn: CMG  
 Checked: JPD

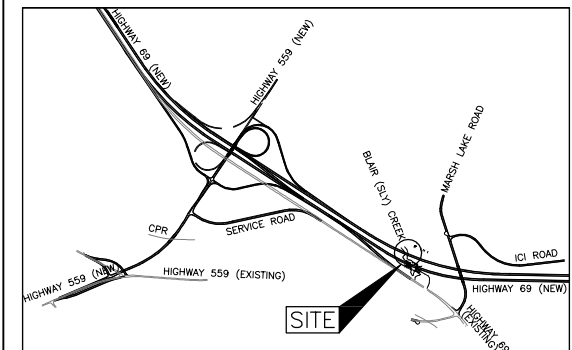
## **DRAWINGS**



PLAN

SCALE  
4 0 4 8 m

CENTRELINE PROFILE

HORIZ. SCALE  
4 0 4 8 m  
VERT. SCALE  
2 0 2 4 m**METRIC**  
DIMENSIONS ARE IN METRES AND/OR  
MILLIMETRES UNLESS OTHERWISE SHOWN.  
STATIONS IN KILOMETRES + METRES.CONT No.  
GWP No. 335-00-00HIGHWAY 69  
PROPOSED SBL BRIDGE OVER BLAIR (SLY) CREEK  
BOREHOLE LOCATIONS & SOIL STRATA**Golder Associates Ltd.**  
MISSISSAUGA, ONTARIO, CANADA

KEY PLAN

SCALE  
500 0 500 1000m

LEGEND

- Borehole - Current Investigation
- ⊙ Borehole - Investigated for swamp crossing
- ⊕ Dynamic Cone Penetration Test - Investigated for swamp crossing
- 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
SBL-1	197.8	5032821.7	256573.6
SBL-2	196.5	5032828.2	256556.9
SBL-3	197.4	5032843.2	256520.9
SBL-4	197.7	5032846.0	256514.5
BH04-73	197.2	5032810.2	256604.6
BH04-75	197.3	5032818.5	256582.1
BH04-105	197.1	5032811.3	256570.6
DC04-32	197.3	5032805.0	256590.5
DC04-45	197.5	5032832.2	256576.2

NOTES

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

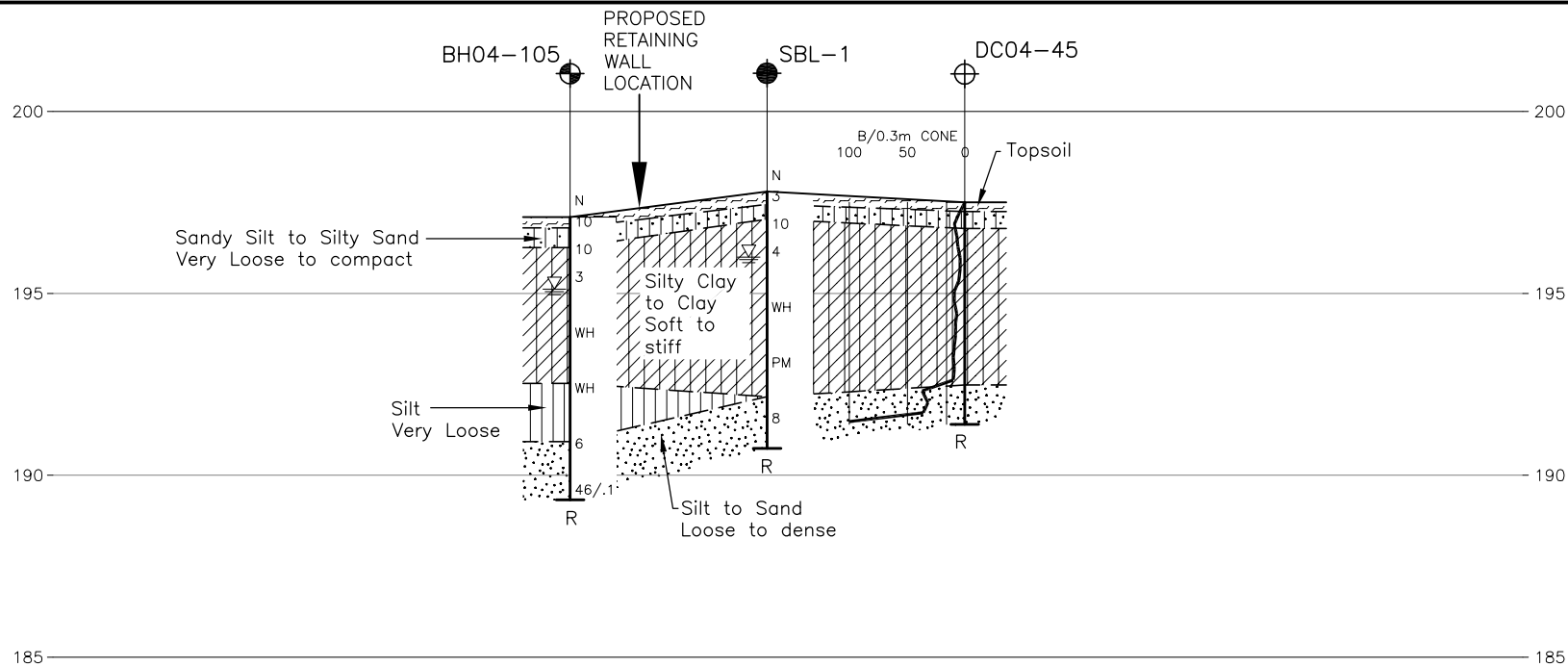
The boundaries between soil strata have been established only at borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

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

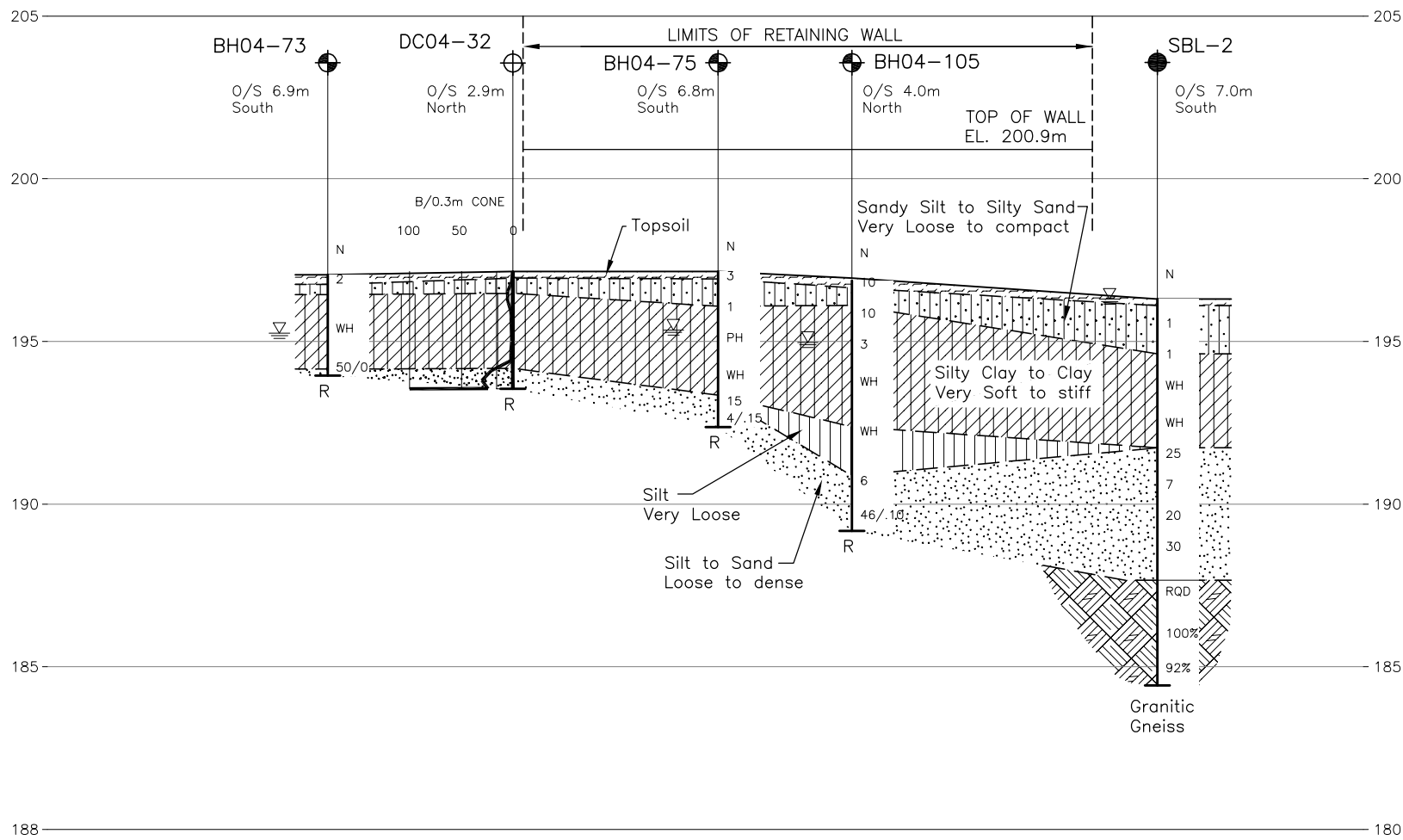
REFERENCE

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

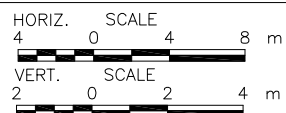
NO.	DATE	BY	REVISION
Geocres No. 41H-54			
HWY. 69	PROJECT NO. 03-1111-028		DIST. 52
SUBM'D. CMG	CHKD. CMG	DATE: FEB 2006	SITE:
DRAWN: JFC	CHKD. JPD	APPD. FJH	DWG. 1A



**B-B' 1A SOUTH APPROACH**



**C-C' 1A RETAINING WALL PROFILE**



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

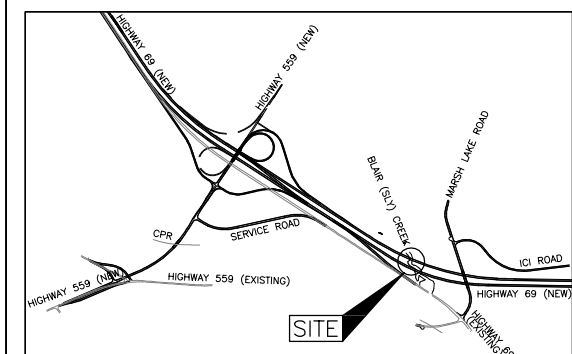
CONT No.  
GWP No. 335-00-00

HIGHWAY 69  
PROPOSED SBL BRIDGE OVER BLAIR (SLY) CREEK  
CROSS SECTIONS

SHEET



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



**KEY PLAN**



**LEGEND**

- Borehole - Current Investigation
- Borehole - Investigated for swamp crossing
- Dynamic Cone Penetration Test - Investigated for swamp crossing
- 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
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BH04-73	197.2	5032810.2	256604.6
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BH04-105	197.1	5032811.3	256570.6
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**NOTES**

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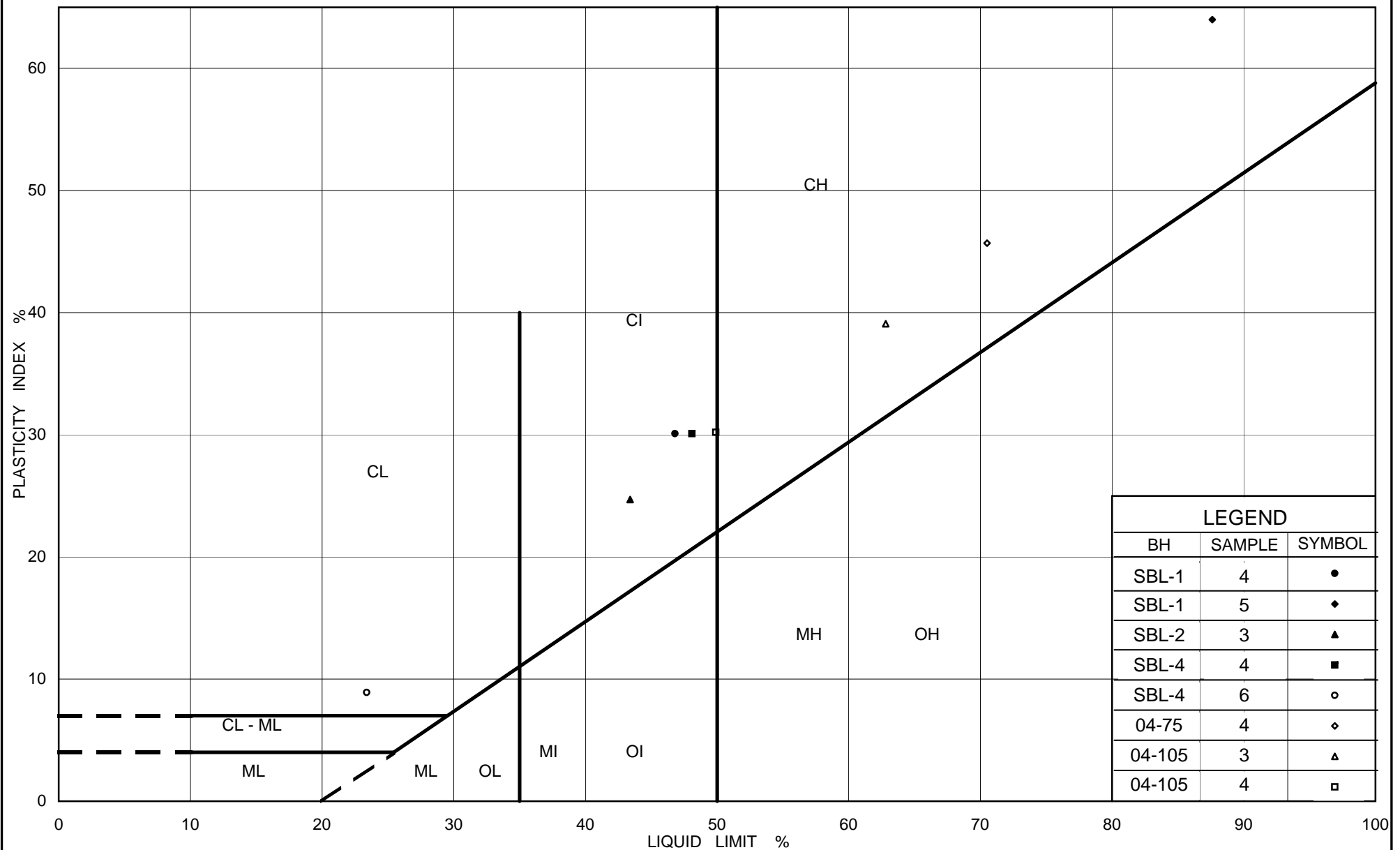
For subsurface information only.

**REFERENCE**

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

NO.	DATE	BY	REVISION
Geocres No. 41H-54			
HWY. 69	PROJECT NO. 03-1111-028		DIST. 52
SUBM'D. CMG	CHKD. CMG	DATE: FEB 2006	SITE:
DRAWN: JFC	CHKD. JPD	APPD. FJH	DWG. 1B

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



Ministry of Transportation

Ontario

# **PLASTICITY CHART** Clayey Silt to Clay

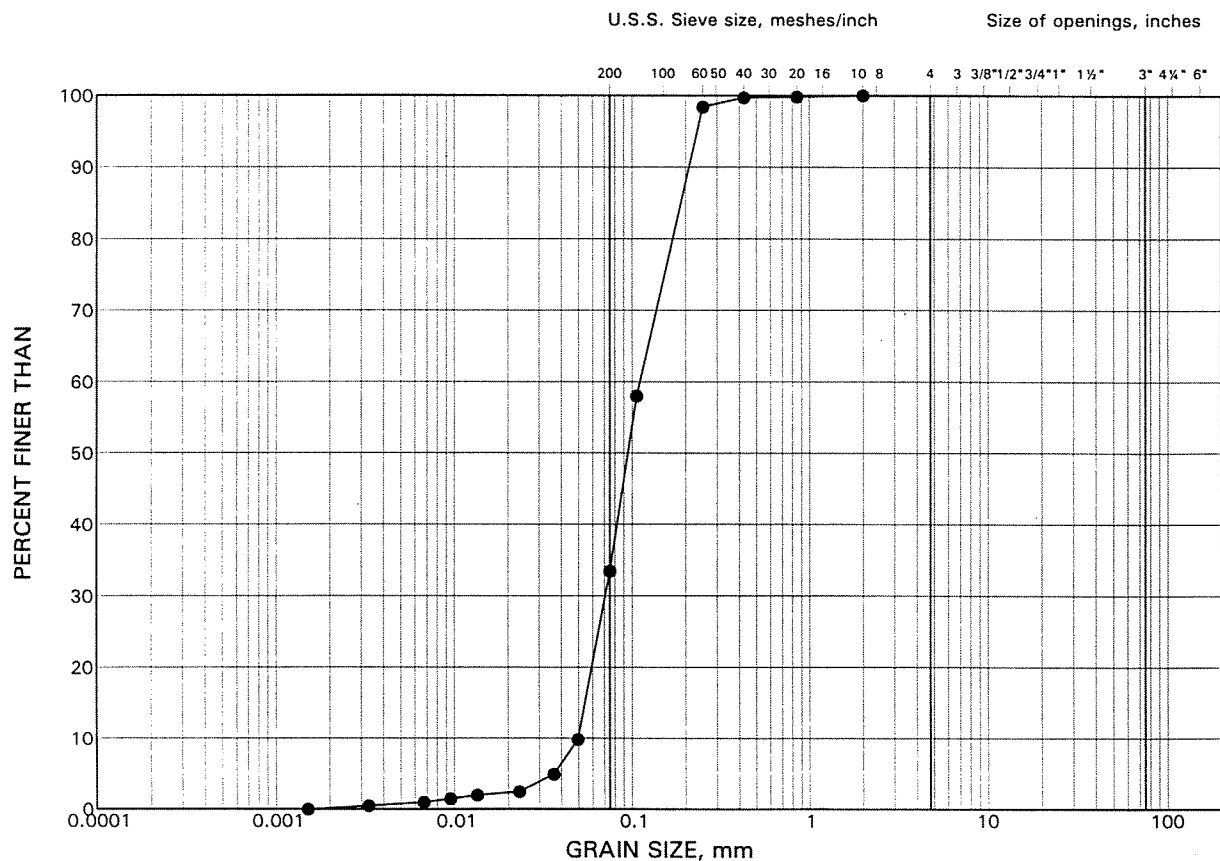
FIG No. A-1

Project No. 03-1111-028-3

# GRAIN SIZE DISTRIBUTION

## Sand and Silt

FIGURE A-2



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

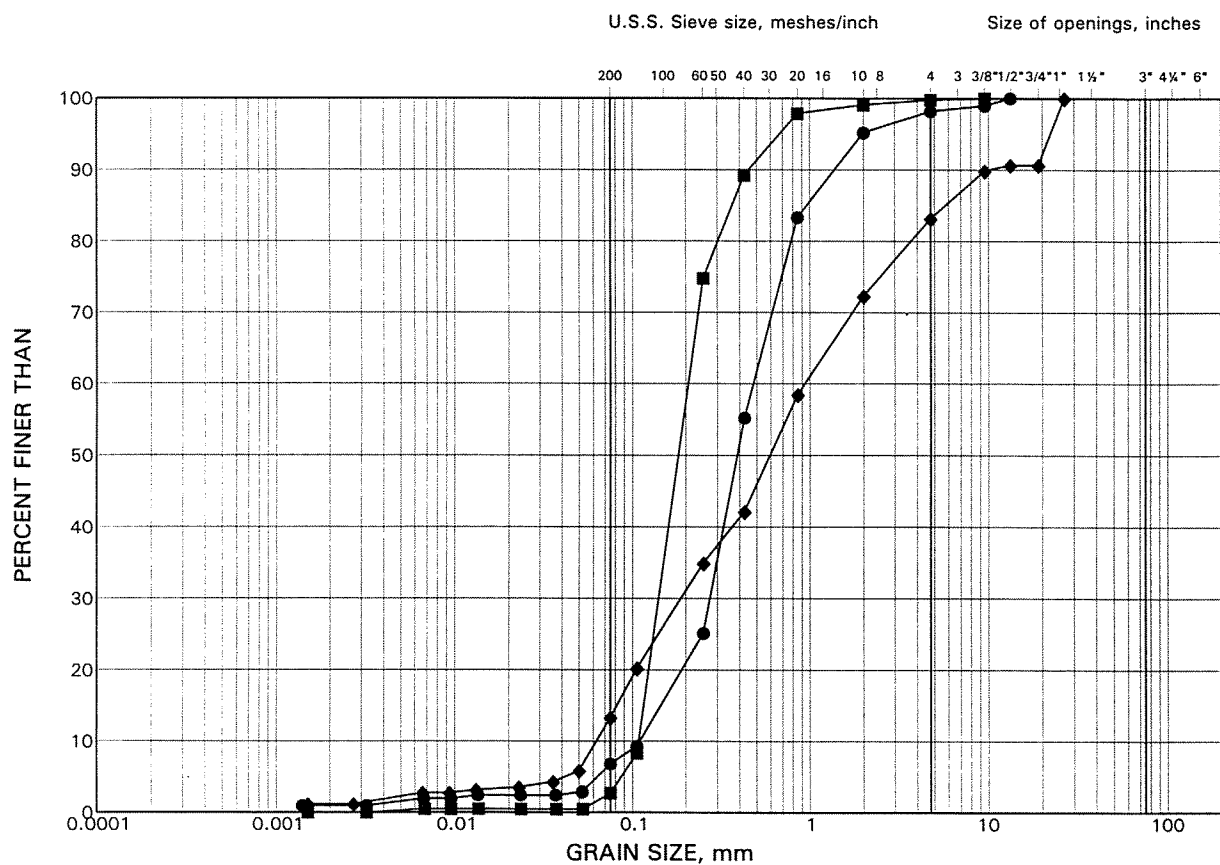
### LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
•	SBL-3	8	191.1

# GRAIN SIZE DISTRIBUTION

## Sand

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)
●	SBL-2	7	190.3
■	SBL-4	9	188.3
◆	04-105	6	190.7

**APPENDIX B**

**OBLIQUE AERIAL PHOTOGRAPH OF BLAIR (SLY) CREEK**

Oblique Aerial Photograph  
Blair (Sly) Creek

FIGURE B-1



**APPENDIX C**

**SAMPLE NON-STANDARD SPECIAL PROVISIONS**

Special Provision

---

**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.**

---

### **Special Provision**

---

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**

---

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.**

---

### **Special Provision**

---

#### **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.