

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
NORTHBOUND 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-53

DISTRIBUTION

- 3 Copies - Ministry of Transportation, Ontario,
North Bay, Ontario (Northeastern Region)
- 1 Copy - Ministry of Transportation, Ontario,
Downsview, Ontario (Pavement and Foundation Section)
- 2 Copies - URS Canada Inc.
Markham, Ontario
- 2 Copies - Golder Associates Ltd.,
Mississauga, Ontario



January 2006



03-1111-028-2

TABLE OF CONTENTS

<u>SECTION</u>	<u>PAGE</u>
PART A - FOUNDATION INVESTIGATION REPORT	
1.0 INTRODUCTION	1
2.0 SITE DESCRIPTION	2
3.0 INVESTIGATION PROCEDURES	3
3.1 Foundation Investigation	3
4.0 GENERAL SITE GEOLOGY AND SUBSURFACE CONDITIONS	5
4.1 Geology	5
4.2 Subsurface Conditions and General Overview	5
4.2.1 Topsoil	6
4.2.2 Silty Sand to Sandy Silt	6
4.2.3 Silty Clay to Clay	6
4.2.4 Silt to Clayey Silt	7
4.2.5 Silty Sand to Gravelly Sand	7
4.2.6 Bedrock	8
4.2.7 Groundwater Conditions	8
4.3 CLOSURE	9
PART B - FOUNDATION DESIGN REPORT	
5.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS	10
5.1 General	10
5.2 Bridge Foundation Options	10
5.3 Spread Footings (South Abutment)	12
5.3.1 Geotechnical Resistance	13
5.3.2 Resistance to Lateral Loads	14
5.3.3 Frost Protection	15
5.4 Steel H-Pile Foundations (North Abutment)	15
5.4.1 Axial Geotechnical Resistance	15
5.4.2 Downdrag Load (Negative Skin Friction)	15
5.4.3 Lateral Loads (due to Horizontal Soil Deformations)	16
5.4.4 Set Criteria	17
5.4.5 Pile Driving Note	17
5.4.6 Resistance to Lateral Loads	17
5.4.7 Frost Protection	20
5.5 Site Coefficient	20
5.6 Lateral Earth Pressures for Design	20
5.7 Approach Embankment Design	23

5.7.1	Stability.....	23
5.7.1.1	North Approach – Front Slope Stability	26
5.7.1.2	Embankment Fill Types and Berm Requirements	27
5.7.1.2.1	Earth Fill	27
5.7.1.2.2	Rock Fill.....	27
5.7.2	Liquefaction Potential	28
5.7.3	Settlement	28
5.7.3.1	Settlement of Foundation Soils (South Approach)	30
5.7.3.2	Settlement of Foundation Soils (North Approach)	30
5.7.3.3	Settlement of Rock Fill	32
5.7.3.4	Settlement of Earth Fill	32
5.8	Mitigation of Stability Issues / Time Dependent Settlements	32
5.8.1	Full Sub-excavation	33
5.8.2	Toe Berms and Preloading	34
5.8.3	Wick Drains / Staged Construction	35
5.8.4	Light Weight (EPS) Fill	36
5.8.5	Surcharging	36
5.9	Subgrade Preparation and Embankment Construction.....	37
5.9.1	Removal of Organics	37
5.9.2	Embankment Fill Placement	38
5.10	Design and Construction Considerations	38
5.10.1	Excavations	38
5.10.1.1	Temporary (or Permanent) Shoring	39
5.10.1.2	Staged Excavation.....	40
5.10.2	Groundwater and Surface Water Control.....	40
5.11	CLOSURE	41

In Order
Following
Page 41

References

Lists of Abbreviations and Symbols
Lithological and Geotechnical Rock Description Terminology
Record of Borehole Sheets (NBL-1 to NBL-6)

LIST OF TABLES

Table 1	Summary of Point Load Tests on Rock Core Samples
Table 2-A	Evaluation of Foundation Alternatives – South Abutment
Table 2-B	Evaluation of Foundation Alternatives – North Abutment
Table 3	Summary of Recommendations at Structure Approach Embankments (incl. Platform Widening)
Table 4	Evaluation of Settlement / Stability Mitigation Alternatives – North Approach
Table 5	Evaluation of Preload Option for Settlement / Stability Mitigation – North Approach

LIST OF FIGURES

- Figure 1 Site Location Map
Figure 2 North Approach Embankment – Front Slope Stability

LIST OF DRAWINGS

- Drawing 1A Highway 69 – Proposed NBL Bridge over Blair (Sly) Creek – Borehole Locations and Soil Strata
Drawing 1B Highway 69 – Proposed NBL Bridge over Blair (Sly) Creek – Cross Section

LIST OF APPENDICES

- Appendix A Laboratory Test Data
Figure A-1 Plasticity Chart – Silty Clay to Clay
Figure A-2 Grain Size Distribution (NBL-2 Sa#3) – Clayey Silt, some sand
Figure A-3 Grain Size Distribution (NBL-1 Sa#4 / NBL-2 Sa#4) – Silty Sand to Gravelly Sand
Appendix B Sample Non-Standard Special Provisions

PART A

**FOUNDATION INVESTIGATION REPORT
NORTHBOUND 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**

PAGE

1.0	INTRODUCTION	1
2.0	SITE DESCRIPTION	2
3.0	INVESTIGATION PROCEDURES	3
3.1	Foundation Investigation	3
4.0	GENERAL SITE GEOLOGY AND SUBSURFACE CONDITIONS	5
4.1	Geology	5
4.2	Subsurface Conditions and General Overview	5
4.2.1	Topsoil.....	6
4.2.2	Silty Sand to Sandy Silt	6
4.2.3	Silty Clay to Clay	6
4.2.4	Silt to Clayey Silt	7
4.2.5	Silty Sand to Gravelly Sand	7
4.2.6	Bedrock	8
4.2.7	Groundwater Conditions	8
4.3	CLOSURE	9

In Order
Following
Page 41

Record of Borehole Sheets (NBL-1 to NBL-6)

Table 1 Summary of Point Load Tests on Rock Core Samples

Figure 1 Site Location Map

Drawing 1A	Highway 69 – Proposed NBL Bridge over Blair (Sly) Creek – Borehole Locations and Soil Strata
Drawing 1B	Highway 69 – Proposed NBL Bridge over Blair (Sly) Creek – Cross Section

Appendix A Laboratory Test Data
Figure A-1 Plasticity Chart – Silty Clay to Clay
Figure A-2 Grain Size Distribution (NBL-2 Sa#3) – Clayey Silt, some sand
Figure A-3 Grain Size Distribution (NBL-1 Sa#4 / NBL-2 Sa#4) – Silty Sand to Gravelly Sand

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 Northbound 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 northbound structure at Blair Creek was provided to Golder by URS on January 11, 2005.

This report addresses the investigation for the proposed Northbound Highway 69 structure over Blair (Sly) Creek and the associated approach embankments only. 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 120 m east of existing Highway 69 (north of Nobel, Ontario) at the proposed northbound Highway 69 crossing of Blair (Sly) Creek, approximately 300 m north of existing Highway 559 (as shown on Figure 1).

In general, the topography in the area of the overall project site consists of rolling terrain including densely treed areas and numerous bedrock outcrops separated by low-lying swamp areas. The proposed northbound structure is to be constructed in a relatively low-lying, grassland adjacent to the floodplain of Blair (Sly) Creek. South of the creek, moderately spaced trees and small shrubs dominate the area. North of the creek, a low-lying wet grassy area transitions into a wooded area consisting of mature trees. 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 proposed Northbound Highway 69 bridge structure investigation was carried out between October 25 and December 4, 2004 during which time a total of six (6) sampled boreholes (NBL-1 to NBL-6) were put down at the site. Three (3) boreholes were drilled at the proposed south foundation element location, one borehole was drilled at the proposed north foundation element location and one borehole was advanced to refusal within the limits of each of the proposed south and north approach embankments. All of the boreholes were advanced to refusal on inferred bedrock. In one borehole at each of the abutment locations, bedrock coring was carried out to a minimum depth of 3 m.

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.

The boreholes were advanced to auger and/or sampler refusal (i.e. inferred bedrock) which occurred at depths ranging from about 2.1 m to 6.8 m below the existing ground surface (not including rock coring). At boreholes NBL-2 and NBL-3, located within the footprints of the proposed foundation units, the drilling was further advanced into the bedrock by coring 3.2 m and 3.7 m respectively. The groundwater level in the open boreholes was observed throughout the drilling operations and a piezometer was installed in NBL-3 to permit monitoring of the groundwater level at this location. The piezometer consisted of 38 mm O.D. threaded PVC tubing with a slotted screen at depth and was backfilled with a sand filter and sealed with bentonite within the borehole. The installation details and water level readings are described on the Record of Borehole sheets that follow the text of this report. All boreholes and the piezometer were abandoned in accordance with O.Reg. 128 (amendment to O.Reg. 903).

The field work was supervised throughout by members of our engineering and technical staff, who confirmed the locations of the boreholes, arranged for the clearance of underground service locations, supervised the drilling, sampling and in-situ testing operations, logged the boreholes, and examined and cared for the soil and rock samples. The samples were identified in the field,

placed in appropriate containers, labelled and transported to our Mississauga geotechnical laboratory where the samples underwent further detailed visual examination and appropriate laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards as appropriate. Classification testing such as water content, grain size distribution and Atterberg limits were carried out on samples of the overburden soils. In addition, the results of a consolidation (oedometer) test carried out for the investigation and design of the immediately adjacent swamp crossing (i.e. on a Shelby tube sample from a borehole located approximately 35 m north of Blair (Sly) Creek) was also considered. Strength testing such as point load index were carried out on specimens from the rock core.

All investigated borehole locations were located in the field by Callon Dietz prior to drilling operations. The surveying of the elevations 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 as encountered from the boreholes at the proposed Northbound Highway 69 structure over of Blair (Sly) Creek is shown on Drawings 1A and 1B.

In general, the subsoils at the structure site consist of topsoil underlain by successive deposits of silty sand to sandy silt, silty clay to clay, silt to clayey silt and silty to gravelly sand over bedrock. The total overburden thickness at the investigated locations ranges from about 2.1 m (south of Blair Creek) to about 6.8 m (north of Blair Creek). All of the boreholes were terminated at the inferred bedrock surface; with the exception of two (2) boreholes which were cored at least three metres into the bedrock.

A more detailed description of the subsurface conditions encountered in the boreholes (NBL-1 to NBL-6) is provided in the following sections.

4.2.1 Topsoil

A layer of topsoil was encountered at ground surface in all boreholes. The surface of the topsoil (i.e. ground surface) ranged between Elevations 197.6 m and 196.6 m and the thickness ranged between about 0.1 m and 0.3 m at the borehole locations.

4.2.2 Silty Sand to Sandy Silt

A light brown to grey, oxidized silty sand to sandy silt deposit containing trace gravel and organics was encountered below the topsoil in all boreholes. The top of this deposit ranged from about Elevation 197.5 m to 196.4 m and the thickness ranged from about 0.2 m to 0.4 m.

Standard Penetration Testing (SPT) carried out within this stratum measured 'N' values ranging from 2 blows to 8 blows per 0.3 m of penetration indicating a very loose to loose relative density within the deposit.

4.2.3 Silty Clay to Clay

A deposit of reddish brown to grey silty clay to clay containing trace sand and organics was encountered below the silty sand to sandy silt in all boreholes for the northbound structure. The soil structure of this deposit was noted to be mottled and/or varved with thin sand seams observed in certain boreholes. The top of this layer varied between Elevation 197.1 m and 196.0 m and the thickness ranged from about 1.1 m south of Blair (Sly) Creek to 4.0 m north of the creek. The bottom of this deposit was defined by refusal to further auger advancement in borehole NBL-6.

Standard Penetration Testing (SPT) carried out within this stratum measured 'N' values ranging from 1 blow to 14 blows per 0.3 m of penetration. 'N' values measured in the boreholes were the highest in those south of Blair Creek where the deposit was found to be thinner.

In situ field vane testing carried out in borehole NBL-3 and NBL-4 (north of Blair (Sly) Creek) measured undrained shear strengths ranging from 16 kPa to 52 kPa, with an average of about 29 kPa. Sensitivity was found to range from 4.7 to 9.0, typically less than 6.5. In general, the field vane test results together with the SPT 'N' values suggest the silty clay to clay stratum has a soft to stiff consistency.

The natural water content measured on samples of this deposit ranged between 25 percent and 51 percent with an average of 36 percent.

Atterberg limits testing was carried out on three (3) samples of the silty clay to clay. The liquid limit ranged from about 39 to 65 percent and the plastic limit ranged from about 17 to 25 percent yielding a plasticity index ranging from about 21 to 40 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.

4.2.4 Silt to Clayey Silt

A reddish grey to grey silt to clayey silt deposit containing trace to some clay and sand was encountered below the silty clay to clay in NBL-1, NBL-2 and NBL-5, all located south of the Blair Creek. The top of this deposit ranged from Elevation 196.0 m to 195.1 m and the thickness ranged from about 0.5 m to 0.8 m.

Standard Penetration Testing (SPT) carried out within this stratum measured 'N' values of 1 blow to 6 blows per 0.3 m of penetration indicating a very loose to loose relative density within the deposit.

The natural water content measured on two (2) samples of this deposit was 36 percent and 37 percent.

A grain size distribution for a clayey silt sample from this deposit is shown on Figure A-2 of Appendix A.

4.2.5 Silty Sand to Gravelly Sand

A grey silty to gravelly sand deposit was encountered below the silt to clayey silt deposit in boreholes south of the creek (excluding NBL-6) and directly below the silty clay to clay deposit in boreholes north of the creek. The top of this deposit ranged from Elevation 195.5 m to 192.2 m and the thickness ranged from about 0.5 m south of the creek to about 2.4 m north of the creek. 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 2 blows to 20 blows per 0.3 m of penetration, typically greater than 5 blows per 0.3 m of penetration. The 'N' values indicate a very loose to compact relative density within the deposit.

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

Grain size distributions for two (2) samples from this deposit are shown on Figure A-3 of Appendix A.

4.2.6 Bedrock

Bedrock was encountered and cored in boreholes NBL-2 and NBL-3 located within the footprint of the south and north abutment foundations, respectively. The presence of bedrock was inferred from auger or spoon refusal in the other boreholes. At the borehole locations, the bedrock surface (as confirmed by coring and/or inferred from auger refusal) ranges from as high as Elevation 195.5 m south of Blair (Sly) Creek to as low as Elevation 190.1 m north of the creek.

The bedrock samples are described as fresh, 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 82 percent to 100 percent indicating a rock mass of good 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 ranged from 6.4 MPa to 10.3 MPa and diametral point load strength index values ranged from 2.5 MPa to 6.2 MPa, indicating a strong to very strong rock mass. A summary of the point load index values on the rock core from the two (2) boreholes where coring was carried out is shown in the following table. Table 1 following the text of this report presents a detailed list of all point load index testing results performed for this investigation along with the estimated Unconfined Compressive Strength (UCS) value for each test.

<i>Borehole (Drillhole) No.</i>	<i>Average Axial Point Load Index (MPa)</i>	<i>Average Diametral Point Load Index (MPa)</i>
NBL-2	9.3	4.6
NBL-3	6.9	5.4


4.2.7 Groundwater Conditions

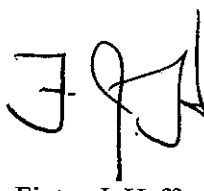
In general, the samples taken in the overburden boreholes were noted to be moist to wet. The water levels in the open boreholes ranged between Elevation 196.3 m and Elevation 195.7 m upon completion of drilling. The groundwater level in the piezometer installed at the soil / bedrock interface in borehole NBL-3 was measured at Elevation 196.6 m (0.4 m depth) on December 21, 2004 and at Elevation 196.5 m (0.5 m depth) on January 4, 2006. The water level of Blair (Sly) Creek was measured at Elevation 196.2 m in July 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.

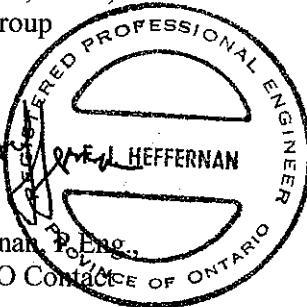
4.3 CLOSURE


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.

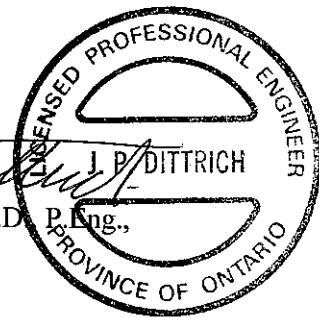
GOLDER ASSOCIATES LTD.


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


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




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



CMG/JPD/FJH/cmg/sm

n:\active\2003\1111\03-1111-028 urs hwy 69 parry sound\reporting\final\2 - blair creek - nbl\03-1111-028-2 final rpt jan06 hwy69nbl-blaircreek structure.doc

January 2006

03-1111-028-2

PART B

**FOUNDATION DESIGN REPORT
NORTHBOUND 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 Northbound 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 Northbound 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 northbound Highway 69 bridge deck varies between about Elevation 201.3 m and 201.6 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 m and 5 m in height at the south and north sides of the bridge, respectively. The existing ground surface varies from about Elevation 197.6 m to 196.6 m at the borehole locations.

It is our understanding that the existing Blair (Sly) Creek, over which the proposed new Northbound Highway 69 structure will cross, is considered to be an environmentally sensitive area. The recommendations given in the following sections have taken this into account as it pertains to foundation design and construction, excavation, drainage and other considerations.

5.2 Bridge Foundation Options

The native soils at the bridge site consist of topsoil underlain by successive deposits of silty sand to sandy silt, silty clay to clay, silt to clayey silt and silty to gravelly sand over bedrock. The total overburden thickness at the investigated locations ranges from about 2 m to 3.3 m (south of Blair Creek) to nearly 7 m (north of Blair Creek). The native overburden soils are underlain by strong to very strong granitic gneiss bedrock. The bedrock surface at the proposed foundations, as

established at the borehole locations, ranges from about Elevation 195.5 m to 193.9 m at the south foundation element to about Elevation 190.2 m in the one borehole advanced within the north foundation element. It should be noted that the average bedrock surface elevation at the south foundation area is approximately 4.4 m higher than at the north foundation area.

At the south foundation element, due to the shallow nature of the overburden deposits, an integral abutment (which typically requires a minimum pile length of 5 m) is not considered practical since this option would necessitate significant excavation/trenching into the very strong bedrock and would likely be cost prohibitive. However, the granitic gneiss bedrock is suitable for the support of this abutment on shallow foundations.

At the north foundation element, spread footings founded at shallow depth on either the very loose to loose silty sand or on the soft to firm silty clay to clay are not recommended due to the low axial resistance and expected settlement of these strata. Spread footings founded on the underlying sand or bedrock are also not recommended due to the deep excavation, groundwater control and the temporary shoring that would be required for construction. It is considered that supporting the north abutment on piles driven to bedrock is the most feasible option at this location.

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 settlement of the footings. In addition, sub-excavation and removal of the clayey foundation soils followed by placement of granular in compacted lifts would need to be carried out in the dry and would require extensive dewatering (especially at the north abutment) which may be cost prohibitive.

As such, the following summarizes the foundation alternatives that could be considered for this site:

- South Abutment – semi-integral abutment supported on spread footings on bedrock
- North Abutment – integral abutment supported on piles driven to bedrock

The details of the recommendations for these options 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 Tables 2-A (south abutment) and 2-B (north abutment) following the text of this report.

5.3 Spread Footings (South Abutment)

The south bridge abutment may be supported on spread footings placed on the properly prepared granitic gneiss bedrock. The details of the bedrock surface elevation as encountered in the boreholes at the south foundation element is summarized in the following table.

<i>Foundation Element</i>	<i>Borehole Numbers</i>	<i>Depth to Bedrock</i>	<i>Bedrock Surface Elevation</i>
South abutment	NBL-2, NBL-5 and NBL-6	2.1 m to 3.0 m	193.9 m to 195.5 m

Based on the borehole results, there is variability in the bedrock surface within the limits of south foundation element. In addition, although the RQD values for the rock core obtained from borehole NBL-2 are high, it may be necessary to subexcavate loose or fractured portions of the upper bedrock from within some areas of the foundation footprint. For design, the following options for founding levels at the south abutment may be considered :

1. A founding elevation of about 195.5 m may be assumed:

In this case, following the removal of the overburden, the bedrock surface would have to be cleaned and then mass concrete would be placed to raise the grade to the founding level. A Non-Standard Special Provision (NSSP) should be made in the Contract Documents for additional mass concrete placement to accommodate variations in the bedrock surface (an example is provided in Appendix B) The benefit of this approach is that excavation into the strong to extremely strong bedrock is avoided. This approach is considered to be the preferred option for this site.

2. Alternatively, a founding elevation of about 193.9 m could be assumed:

In this case, following the removal of the overburden, excavation of the higher portions of the bedrock would be required within the foundation footprint. Based on the borehole results, subexcavation of up to about 1.6 m of bedrock would be required. It is noted that the bedrock is classified as strong to very strong (i.e. estimated unconfined compressive strengths in the range of about 60 MPa to 240 MPa) and it is probable that the level of fracturing in the upper portions of the rock is variable. This will make excavation difficult, particularly in areas where only small depths and narrow zones of removal are needed. As such, bedrock excavation would have to be carried out using line drilling and pre-shearing techniques in order to minimize shattering and over-break and to provide better control over the configuration of the founding surface. However, even with these special techniques, excavation of the bedrock will likely be difficult and as such this approach is not considered favourable for this site.

3. As a third option, an intermediate founding level may be assumed for design. In this case, a combination of bedrock sub-excavation and mass concrete placement will be required.

In all areas where mass concreting is to be employed, it will be necessary to clean, scale and remove any loose debris to ensure a proper bond to the bedrock. In addition, a check on the sliding resistance between the mass concrete and the sloped bedrock should be carried out (in accordance with the recommendations provided in Section 5.3.2).

As an alternative to the use of mass concreting, consideration could also be given to employing an engineered fill (i.e. compacted granular fill) to raise the grade to the founding level. However, as described below, dewatering will be necessary to ensure proper placement and compaction of the fill. In addition, as discussed in the following section, footings founded on compacted granular fill will have lower values of geotechnical resistance at ULS and SLS.

It is noted that the excavations to expose the bedrock surface at the south abutment will, in some places, extend through relatively thin (i.e. about 1.2 m thick) water-bearing sand and clayey silt to silt overburden soils. A suitable dewatering scheme (possibly including the construction of a sheet pile cofferdam as discussed in Section 5.10) is recommended to maintain a dry and stable excavation especially during periods of high groundwater levels.

5.3.1 Geotechnical Resistance

Spread footings placed on the surface of the properly prepared granitic gneiss bedrock may be designed based on a factored geotechnical resistance at Ultimate Limit States (ULS) of 10,000 kPa. For footings placed on a mass concrete pad (constructed in the dry), the factored geotechnical resistance at Ultimate Limit States (ULS) is as given above for bedrock assuming that the strength of the concrete used to form the pad is at least 25 MPa. The geotechnical resistance at Serviceability Limit States (SLS) for 25 mm of settlement will be greater than the factored axial resistance at ULS, since the granitic gneiss bedrock and the mass concrete (placed directly on the bedrock) are considered to be unyielding materials; as such, ULS conditions will govern for this foundation type.

For spread footings placed on a compacted Granular 'A' engineered fill, a factored geotechnical resistance at ULS of 900 kPa may be assumed for preliminary design. The geotechnical resistance at SLS (for 25 mm of settlement) will depend on the thickness of the engineered fill; a value of 350 kPa may be assumed for preliminary design.

All loose, shattered and/or fractured rock within the footprint of the footings and at the footing level should be removed and replaced with concrete. A provision should be included in the

Contract Documents to address the requirements for field inspection of the exposed bedrock. Groundwater control measures would be required in order to carry out this inspection in the dry. MTO Special Provision 902S01 – Excavation and Backfilling – should be included in the Contract Documents (Appendix B).

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

5.3.2 Resistance to Lateral Loads

Resistance to lateral forces/sliding resistance between the base of the concrete footings and the granitic gneiss bedrock or between the base of the concrete footings and the compacted Granular 'A' fill should be calculated in accordance with Section 6.7.5 of the *CHBDC*. In the case of mass concrete or compacted Granular 'A' placed on the bedrock surface (in the dry), the design must also check the sliding resistance between the base of the mass concrete or the base of the granular and the bedrock.

The coefficient of friction, $\tan \delta$, may be taken as 0.70 between the base of the concrete footings and/or mass concrete and the bedrock for construction in the dry. The coefficient of friction, $\tan \delta$, may be taken as 0.55 between the base of the concrete footing and the compacted Granular 'A' fill and 0.45 between the compacted Granular 'A' fill and the bedrock for construction in the dry. These values represent unfactored values; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating the horizontal resistance.

If necessary, the sliding resistance between the concrete footing and/or mass concrete and the bedrock can be supplemented by dowelling into the bedrock. The horizontal resistance of the dowels is dependent on the strength of the bedrock, grout and steel. For this site, where the rock mass is essentially as strong as or stronger than concrete, the design of the dowels in the rock may be handled in the same way as the dowel embedment into the concrete. This assumes that the unconfined compressive strength of the grout will be similar to that of the concrete. The dowels should have a minimum embedded length within the sound bedrock of 1 m, and the structural strength of the dowel and compressive strength of the grout should not be exceeded. If dowelling into bedrock is adopted at this site, a NSSP should be included in the Contract Documents to specify the installation, materials and testing of the dowels (and example is provided in Appendix B).

For the compacted Granular 'A' fill option, it may be necessary to construct the fill pad with 2H:1V side slopes in order to achieve sufficient lateral sliding resistance.

5.3.3 Frost Protection

For spread footings or mass concrete constructed in the dry and founded directly on the properly prepared granitic gneiss bedrock at this site, frost susceptibility is not an issue. Otherwise, footings should be provided with a minimum of 1.8 m of soil cover for frost protection.

5.4 Steel H-Pile Foundations (North Abutment)

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 the north side 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 B).

For design, the following pile tip elevation may be assumed for piles terminating on the bedrock surface. The 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 abutment. There should be a provision made in the Contract for dealing with varying pile lengths.

<i>Foundation Unit</i>	<i>Approximate Design Pile Tip Elevation (m)</i>
North abutment	190

5.4.1 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.

5.4.2 Downdrag Load (Negative Skin Friction)

The loading from north approach embankment construction will cause consolidation and 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 5.8). 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 relative to the stiff pile will result in the development of negative skin friction on the piles. In this case, downdrag loads will need to be taken into account for design of the piles supporting the abutments.

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

The load calculated in this manner is an unfactored load. 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 5.8) and allowing the settlement to occur prior to installing the piles.

5.4.3 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, P_h (kN/m length)</i>
North	Firm silty clay to clay	196.6 – 194.5	110
	Soft silty clay to clay	194.5 – 192.6	55 (at top) to 65 (at bottom)

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 5.8) and allowing the settlement and lateral movement to occur prior to pile installation.

5.4.4 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 B).

5.4.5 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”.

5.4.6 Resistance to Lateral Loads

The design of piles subjected to lateral loads should take into account such factors as the batter of the piles (if any), the relative rigidity of the pile to the surrounding soil, the fixity condition at the head of the pile (pile cap level), the structural capacity of the pile to withstand bending moments, the soil resistance that can be mobilized, the tolerable lateral deflections at the head of the pile

and pile group effects. For a longer, more flexible pile, the maximum yield moment of the pile may be reached prior to mobilization of the lateral geotechnical resistance. For design purposes, both the structural and geotechnical resistances should be evaluated to establish the governing case.

It is understood that an integral abutment foundation is being considered for the north abutment 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 considering the soft to firm clays present below the proposed pile cap level.

Based on the proposed elevation of the underside of the north abutment pile cap as shown on the GA drawing provided by URS (on January 11, 2005) and considering the depth to bedrock encountered in borehole NBL-3 at this location, the total length of the H-piles will be approximately 5.3 m.

For short HP 310 x 110 piles driven to bedrock through the soft to firm clays and very loose to compact sands at the north abutment, the horizontal resistance at Ultimate Limit States (ULS) will be controlled by the lateral capacity of the soil adjacent to the pile. 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.

For a single HP 310 x 110 pile embedded about 5.3 m into the soft to firm silty clay to clay and underlying very loose to compact sand, a factored lateral resistance at ULS of 75 kN is recommended 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.

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

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

For cohesive soils:

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

For cohesionless soils:

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

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

<i>Soil Unit</i>	<i>Elevation (m)</i>	<i>s_u (kPa)</i>	<i>n_h (MPa/m)</i>
Firm silty clay to clay	196.6 – 194.5	40	-
Soft silty clay to clay	194.5 – 192.6	20 (at top) to 25 (at bottom)	
Very loose to compact sand	192.6 – 190.2	-	1.3

For a single, 5.3 m long HP 310 x 110 pile that is driven through the soft to firm silty clay to clay and very loose to compact sands, a lateral resistance at SLS of 30 kN is recommended for a horizontal deformation of 10 mm at the pile cap (assuming the pile cap is at Elevation 195.5 m) based on analyses carried out using LPILE Plus (Version 5.0). For a horizontal deformation of 20 mm at the pile cap, a lateral resistance at SLS of 45 kN is recommended. It should be noted that 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.

5.4.7 Frost Protection

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

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 1.2, consistent with Soil Profile Type II.

5.6 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment stems and any associated wing walls / retaining walls will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the magnitude of surcharge including construction loadings, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls. Seismic (earthquake) loading must also be taken into account in the design.

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

- Select free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) Granular 'A' or Granular 'B' 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(I) of the *Commentary to the CHBDC*) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case II in Figure C6.9.1(I) 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 ³	19 kN/m ³
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 ³	21 kN/m ³
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 20 per cent amplification of the ground motion may occur (i.e. Site Coefficient, $S = 1.2$), resulting in an increase in the ground surface acceleration from 0.05g to 0.06g. The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio of $A = 0.06$.
- In accordance with Sections 4.6.4 and C.4.6.4 of the CHBDC and its *Commentary*, for structures which allow lateral yielding, the horizontal seismic coefficient, k_h , used in the calculation of the seismic active pressure coefficient, is taken as 0.5 times the zonal acceleration ratio (i.e. $k_h = 0.03$). For structures that do not allow lateral yielding, k_h is taken as 1.5 times the zonal acceleration ratio (i.e. $k_h = 0.09$). The seismic active 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	0.32	0.26	0.30
Non-yielding wall	0.37	0.30	0.34

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

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

- The earthquake-induced dynamic 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;
γ'	is the effective unit weight of the soil (kN/m^3)
	<ul style="list-style-type: none"> • taken as soil unit weights given above for fill materials • taken as 19 kN/m^3 for the native materials above Elevation 192 m at the north abutment and above Elevation 195 m at the 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 Northbound Highway 69 structure over Blair (Sly) Creek will require placement of up to about 4 m and 5 m of fill within the limits of the south and north approach embankments, respectively.

Based on the investigated locations at this site, the approach embankments will be founded on a thin surficial layer of silty sand to sandy silt underlain by silty clay to clay over silt and/or silty to gravelly sand, overlying bedrock. At the south approach, the total thickness of the overburden is about 3.3 m (with the silty clay to clay being about 1.5 m thick) while at the north approach, the total thickness of the overburden is about 7 m (with the silty clay to clay being about 4 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 northbound 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 site considering the design requirements and the field data available. The stability analyses were performed to check that the target minimum factor of safety was achieved for the proposed embankment heights and geometries.

As noted above, the subsoils encountered below the 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 Creek and on the groundwater levels noted during drilling of the boreholes in and immediately adjacent to this area. In general, the groundwater level is located at about the elevation of the existing ground surface at the north approach and about 1.5 m below ground surface at the south approach.

The following table summarizes the simplified stratigraphy and the associated strengths and unit weights employed for the different soil types in the approach areas. For the purpose of analysis, 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 5.7.1.2.

South Approach Embankment

<i>Soil Type</i>	<i>Unit Weight (kN/m³)</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 Silty Sand to Sandy Silt	19	$c' = 0 \text{ kPa}, \phi' = 30^\circ$
Stiff Silty Clay to Clay	20	$s_u = 50 \text{ kPa}$
Very loose to loose Silt	18	$c' = 0 \text{ kPa}, \phi' = 28^\circ$
Compact Silty Sand to Gravelly Sand	20	$c' = 0 \text{ kPa}, \phi' = 33^\circ$

North Approach Embankment

<i>Soil Type</i>	<i>Unit Weight (kN/m³)</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 Silty Sand	19	$c' = 0 \text{ kPa}, \phi' = 30^\circ$
Soft to Firm Silty Clay to Clay	19	Top of Layer: $s_u = 35 \text{ kPa}$ Bottom of Layer: $s_u = 15 \text{ kPa}$ to $s_u = 25 \text{ kPa}$
Very loose to compact Sand	19	$c' = 0 \text{ kPa}, \phi' = 32^\circ$

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.

Location	Embankment Height at Critical Section (m)	Earth Fill Option		Rock Fill Option	
		Recommended Side Slope Profile	Minimum Factor of Safety	Recommended Side Slope Profile	Minimum Factor of Safety
South Approach	4	2H : 1V	≥ 1.3	1.25H : 1V	≥ 1.3
North Approach (side slope)	5				
North Approach (front slope)	>5	(see Section 5.7.1.1)			

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, a stabilizing berm may be required at the front slope of the north approach embankment.

5.7.1.1 North Approach – Front Slope Stability

The front slope of the proposed north approach embankment is located within the flood plain (i.e. at high water level) of the adjacent 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 196.0 m and 196.6 m (about 0.5 m below ground surface). The clayey stratum has a total thickness of about 4.0 m and undrained shear strengths as low as 15 kPa.

To achieve a Factor of Safety (FoS) = 1.3 for the greater than 5 m high embankment fill in this area, it would be necessary to construct a small rock fill berm at the toe of the front slope of the embankment. Stability analysis indicate that a toe berm approximately 1 m high by 4 m wide is necessary to achieve an adequate FoS at this location (as shown on Figure 2).

Although the required berm is not very large, environmental or hydraulic considerations associated with the creek and/or flood plain could make this option unfeasible or impractical. As such, other stability mitigation options should be considered including full sub-excavation and removal of the weak/soft soils, staged construction with wick drain installation 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 will be 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 4. The full sub-excavation option has been ranked as the preferred alternative for this area.

5.7.1.2 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.

5.7.1.2.1 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.

5.7.1.2.2 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 required wherever the embankment will exceed a height of 6 m such that the uninterrupted rock fill slope never exceeds a height of 6 m (as per MTO Northeastern Region guidelines). We understand that the Northeastern Region requirements for berms have recently changed from 6 m to 10 m height. However, we have been instructed to maintain the original guidelines for this project.

5.7.2 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 7.0 earthquake events under the approach embankment. Pseudo-static methods of embankment stability analysis indicate that a yield acceleration of approximately 0.10 g results in a factor of safety against side slope instability of 1.0. Based on this yield acceleration and the correlation proposed by Makdisi and Seed (1978), it is estimated that very little additional deformations (i.e. less than about 5 mm) of the embankment could result under the design earthquake event. Localized failures at the embankment toe, resulting in steepening of the embankment side slopes, could occur. Since deep-seated global instability is not anticipated under the design earthquake event, localized toe failures would be mainly a maintenance issue. This should be considered in the life-cycle costing when assessing the relative costs of the works. Alternatively, consideration could be given to sub-excavation and removal of these silty subsoils prior to construction of the approach embankments in order to eliminate the potential for seismically induced instability at the embankment toes.

5.7.3 Settlement

Settlement of the approach embankments 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 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 m and 5 m for the south and north approaches, respectively. 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 silty sand to sandy silt, silt and sand layers was assessed by estimating an elastic modulus of deformation based on the SPT 'N'-values and empirical correlations found in literature by Bowles (1984) and Kulhawy and Mayne (1990).

The consolidation settlement of the soft to 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)
 σ_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)
 μ = 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 the ground surface.

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 a discussion on the rate of settlement is included.

5.7.3.1 Settlement of Foundation Soils (South Approach)

The following simplified stratigraphy and deformation parameters have been developed for and employed in the settlement analysis of the proposed 4 m high rock fill embankment at the south approach.

<i>Soil</i>	<i>Thickness (m)</i>	<i>Unit Weight (kN/m³)</i>	<i>Estimated Deformation Properties</i>
Rock fill (4 m embankment + removal of 0.3 m organics)	4.3 (high)	19	-
Silty Sand to Sandy Silt	0.2	19	$E' = 4 \text{ MPa}$
Silty Clay to Clay	1.6	20	$m_v = 3.4 \times 10^{-4} \text{ kPa}^{-1}$
Silt	0.7	18	$E' = 1.5 \text{ MPa}$
Silty Sand to Gravelly Sand	0.6	20	$E' = 18 \text{ MPa}$

Based on the results of the settlement analysis, the maximum total settlement of the foundation soils in the area of the south approach is estimated to be about 90 mm. This total settlement is estimated to be comprised of about 45 mm of immediate settlement due to compression of the cohesionless soil layers and about 45 mm of time dependent settlement of the cohesive soil layers.

Assuming a coefficient of consolidation (c_v) of about $4.1 \times 10^{-3} \text{ cm}^2/\text{s}$ (based on empirical correlations with liquid limit using US Navy (1971) for an over-consolidated soil) and assuming two-way drainage of the approximately 1.6 m thick silty clay to clay layer, it is estimated that the about 90 percent of the consolidation settlement will be completed in less than about 1 month.

The magnitude of creep settlement for the silty clay to clay strata is expected to be negligible (i.e. less than 5 mm per log-cycle of time) at this location.

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 5 m high rock fill embankment at the north approach.

<i>Soil</i>	<i>Thickness (m)</i>	<i>Unit Weight (kN/m³)</i>	<i>Estimated Deformation Properties</i>
Rock fill (5 m embankment + removal of 0.2 m organics)	5.2 (high)	19	-
Silty Sand	0.3	19	E' = 3 MPa
Silty Clay to Clay	4.0	19	(see below)
Sand	2.3	20	E' = 8 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 immediately adjacent to the area of the north approach embankment.

<i>Location</i>	<i>Elevation (m)</i>	<i>σ_{vo}' (kPa)</i>	<i>σ_p' (kPa)</i>	<i>OCR</i>	<i>e_o</i>	<i>C_r</i>	<i>C_c</i>	<i>c_v (cm²/s)</i>
NBL - North Approach	196.3 to 192.2	20	75	3.8	1.3	0.06	0.65	1.0 x 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 195 mm. This total settlement is estimated to be comprised of about 35 mm of immediate settlement due to compression of the cohesionless soil layers and about 160 mm of time dependent settlement of the cohesive soil layers.

Assuming a coefficient of consolidation (c_v) of about 1.0x10⁻³ cm²/s (based on empirical correlations with liquid limit using US Navy (1971) and on results of laboratory consolidation tests on samples from the adjacent swamp area) and assuming two-way drainage of the approximately 4.0 m thick silty clay to clay layer, it is estimated that the about 90 percent of the consolidation settlement will be completed in about 13 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 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 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+675 to 10+695	4+0.3 = 4.3	45
North Approach	10+735 to 10+755	5+0.2 = 5.2	50

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

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 5.7.1 and 5.7.3, the embankment stability and estimated magnitude of post-construction (i.e. time dependent) settlements differ at the proposed south and north approaches.

At the south approach, the lower embankment height in conjunction with the relatively thinner and stiffer consistency of the silty clay layer results in an improved stability condition and smaller consolidation settlements. In addition, the over-consolidated nature of the silty clay in this area results in a relatively higher coefficient of consolidation (c_v) and in-turn, a faster rate of settlement. As such, for the current embankment height proposed, no stability issues are

anticipated so long as all organic layers are removed prior to construction. Similarly, the settlements of the foundation soils in this area are expected to occur during or shortly after construction (i.e. within about 1 month) assuming that all of the surface and near surface layers of organic layers have been removed within the footprint of the approach prior to filling. Therefore, in this area, no special construction procedures are considered necessary to maintain stability or mitigate foundation soil settlements so long as the new embankment fill is placed as early as possible in the construction schedule. The embankment fill at the south approach should be constructed to full height at the beginning of the construction contract and allowed to consolidate for at least 6 weeks prior to the construction of the final pavement structure. Following this recommendation will reduce the post-construction settlements and the need for maintenance of the roadway surface.

At the north approach however, the higher embankment height combined with the thicker and softer consistency of the silty clay to clay stratum creates a stability problem for the front slope of the proposed embankment. In addition, larger time-dependent consolidation settlements are anticipated to occur over a period of up to 13 months following completion of construction. In this area, 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 height and to limit the post-construction settlements and subsequent maintenance on the new roadway surface. The following sections outline the options and recommendations for achieving the target factor of safety for the required embankment geometry and for minimizing post-construction settlements that could effect roadway performance. The advantages, disadvantages, relative costs and risks/consequences for the mitigation options at these areas are summarized and also presented and ranked in Table 4.

5.8.1 Full Sub-excavation

The bottom of the silty clay to clay layer at the north approach is located approximately 4.5 m below the ground surface. Sub-excavation and removal of the clayey strata to this depth is considered feasible, would avoid the need for toe berms on the front slope and will provide the best technical solution in terms of the stability and long-term performance of the roadway.

However, the embankment toe near the southeast corner of the proposed north approach is close to the surveyed edge of the adjacent Blair (Sly) Creek (i.e. less than about 4.5 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 more than 5.5 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 avoid having the excavation impact the creek, the following options could be considered:

- Construct a low-permeability earth fill berm to temporarily 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 diversion of the creek may not be possible due to environmental restrictions imposed by the Ministry of Natural Resources (MNR) and as such this will need to be taken into account when considering the feasibility of this alternative. 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 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.

5.8.2 Toe Berms and Preloading

For the approximately 4 m thick silty clay to clay strata at this site, it is estimated that 90 percent of the post-construction foundation soil settlements will be completed in about 13 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. For this alternative, sub-excavation and the temporary diversion or sheet pile support at Blair (Sly) Creek would not be required. However, depending on the sequence of construction and the height of preload fill, a toe berm

approximately 4 m wide by 1 m high may be required to maintain the stability of the front slope of the embankment (i.e. towards the creek).

If environmental constraints will not allow the toe berm to remain in place permanently, the toe berm could be considered a temporary requirement and removed after a minimum of 4 months of consolidation and strength gain has occurred in the silty clay to clay strata.

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.

A number of alternatives associated with the sequence of construction for the preloading option can be considered. Each alternative has its own advantages and disadvantages in terms of the length of preload period; the need for a toe berm; the timing for bridge construction; and the requirements to include downdrag and lateral loads in the abutment piles design. The advantages, disadvantages and risks/consequences for these sub-options are summarized in Table 5.

5.8.3 Wick Drains / Staged Construction

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 months following completion of embankment construction. However, preliminary calculations indicate that installing wick drains on a triangular grid at a 1.5 m spacing to a depth of about 4.5 m would accelerate the consolidation process such that about 90 percent of the primary consolidation could be completed in about 7.5 months after completion of embankment construction.

With the wick drains, the embankment could be constructed in two stages, without the need for a toe berm on the front slope so long as the first stage (approximately 4 m high) was allowed to consolidate for a minimum period of 4 months prior to completing the embankment. However, monitoring of the settlement and dissipation of the excess porewater pressures would be required to check that adequate consolidation (and strength gain in the clayey strata) had occurred prior to proceeding with the final construction stage.

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.

5.8.4 Light Weight (EPS) Fill

The loading imposed by the 5 m high north approach embankment on the soft and compressible foundation soils in this area could be reduced by using ultra-light weight (i.e. expanded polystyrene) fill. The use of this material for the embankment fill would eliminate 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 (i.e. approximately 4 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. about half) could be employed if the construction schedule will allow a combination of preloading followed by partial excavation and replacement with limited EPS fill. Preliminary analyses indicate that the following sequence could be adopted:

- i) construct a 4 m high preload embankment and allow it to remain in place for about 8 months (no toe berms required);
- ii) remove 2.5 m of the embankment fill
- iii) install 2 m of EPS fill over the length of the approach
- iv) construct 1.5 m of 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 20 mm per log-cycle of time) should be expected with this option.

5.8.5 Surcharging

As noted above, a toe berm approximately 4 m wide by 1 m high is necessary to maintain the stability of the front slope if the embankment is built up to the required final grade height of about 5 m without staging or control on the rate of construction. An even larger front slope berm in addition to side berms would be required if a surcharge was to be placed on top of the required 5 m high embankment.

It is estimated that a front slope berm 8.5 m wide by 3 m high, and side slope berms 8 m wide by 2 m high would be needed to add a 2 m surcharge to the top of the embankment and maintain a FoS of 1.3. However, under the influence of a 2 m surcharge, it is expected that the majority (i.e. >95%) of the primary consolidation settlement of the final/design embankment geometry would be completed in about 6 months. This could potentially save about 7 months in the construction schedule.

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. In addition, the size of the toe berms required to maintain stability of the embankment under surcharge loading will (in places) encroach into the adjacent creek. As such, this option may not be feasible due to environmental restrictions.

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 3 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 at the north approach embankment, it will be necessary to backfill the excavation in the area of the north abutment foundation 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) of up to about 0.2 m to 0.3 m deep can be expected in some areas of the new approach embankments. These organic layers should be stripped from the plan limits of the approach areas prior to fill placement.

5.9.2 Embankment Fill Placement

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

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.

5.10 Design and Construction Considerations

5.10.1 Excavations

As noted in Section 5.3, excavations for construction of the spread footings for the south bridge abutment will extend up to about 3.3 m deep. As noted in Section 5.8, excavations for the settlement and stability mitigation measures at the north approach embankment (if sub-excavation and removal is adopted) will extend up to 4.5 m deep. In addition, as noted in Section 5.7, 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 prior to fill placement.

At the south abutment, temporary excavations to a depth of up to about 3.3 m (i.e. to the bedrock surface for footing construction) could be carried out 'in-the-dry' by open cut with side slopes of about 1H:1V. 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.

At the north approach, where space permits, temporary excavations (i.e. those that are open only for a relatively short period) to a depth of about 4.5 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 sheet pile shoring and/or staged excavation described in the sections below could be considered.

At the 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 1H:1V. Assuming this excavation is carried out 'in-the-dry' to a maximum depth of 1.5 m and the groundwater conditions at this area as described in Section 4, it is calculated that the Factor of Safety against base heave is approximately 1.2. 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.

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 (or Permanent) Shoring

Where space, the location of the creek and/or high groundwater levels restrict the use of open cuts, a temporary (or permanent) sheet pile support system could be constructed to support excavations and to provide protection to the environmentally sensitive creek. As discussed above, the use of temporary shoring could be considered for construction at the south abutment foundation. In areas where excavation is required immediately adjacent to Blair (Sly) Creek, a sheet pile shoring system could be installed to minimize the disturbance to the creek. For this case, the sheet piling should remain in place after construction to reduce the chance of the creek migrating into the more permeable materials employed for backfilling the excavations. The excavation support system (temporary or permanent at the creek) should be in accordance with OPSS 539 (Performance Level 3).

For the up to 4.5 m deep excavations required at the north abutment and approach, the sheet pile support system will have to include tie-back anchors (to bedrock) to support the wall if the adjacent excavations are carried out without restrictions on extent or width. However, if the excavation (and backfilling operations) adjacent to the sheet piling are carried out simultaneously in short sections of limited width (as discussed below), the sheet piling could be supported by the edges of the narrow excavation and tie-back anchors may not be required.

If temporary shoring is employed for construction of the footing at the south abutment, some groundwater inflow should be expected considering that up to 1.5 m of water bearing sand and/or silt exists immediately above the bedrock at this location. In addition, considering the strong to very strong nature of the bedrock, it will not be possible to toe the sheet piling into the rock to cut-off inflows.

Ground water inflow into the shored excavation at the south abutment should be controlled by a dewatering system (as discussed in Section 5.10.2) so that the footing construction is carried out in the dry.

5.10.1.2 Staged Excavation

At the north approach, 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 north abutment and north approach, the groundwater level is generally at or within about 0.5 m of the ground surface. If sub-excavation is adopted as the settlement and stability mitigation measure at this location, groundwater flow into the approximately 4.5 m 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 north abutment foundation (i.e. for construction of the pile cap), the relatively shallow depth of excavation required (about 1.5 m deep) is not anticipated to require groundwater control so long as surface water is directed away from the excavation at all times.

At the south abutment, the groundwater level is at about Elevation 196 m or about 1.5 m to 2 m below ground surface. As discussed previously, an excavation ranging from about 1.7 m to 3.3 m in depth through silty clay to clay, silt to clayey silt, and sand to silty sand strata is required to construct the abutment footing on the bedrock surface and, as such, this excavation may require groundwater control. However, based on grain size distributions for the silt and sand strata from borehole NBL-2 (at the south abutment), 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.


If temporary shoring (i.e. sheet piling) is installed to facilitate the excavation at the south abutment, some measures such as the placement of sand bags between the tips of the sheet piles and the sloping bedrock surface could be employed to cut-off seepage inflows, if necessary.

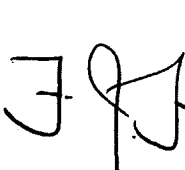
In all cases, surface water should be directed away from the excavations at all times.


5.11 CLOSURE

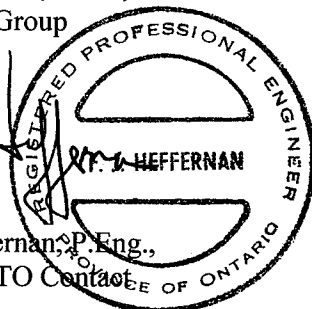
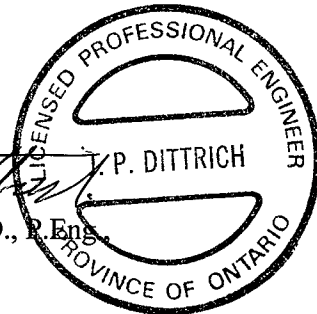
This Foundation Design Report was prepared by Mr. Chad Gilfillan and reviewed by Dr. J. Paul Dittrich, Ph.D., P.Eng., an Associate with Golder. Mr. Fintan Heffernan, P.Eng., Golder's Designated MTO Contact for this project, conducted an independent quality review of the report.

GOLDER ASSOCIATES LTD.


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


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


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



CMG/JPD/FJH/cmg/sm

n:\active\2003\1111\03-1111-028 urs hwy 69 parry sound\reporting\final\2 - blair creek - nbl\03-1111-028-2 final rpt jan06 hwy69nbl-blaircreek structure.doc

REFERENCES

Azzouz, A.S., R.J. Krizek, and R.B. Corotis. 1976. Regression Analysis of Soil Compressibility. *Soils and Foundations*, Tokyo, Vol. 16, no.2, pp.19-29.

Bjerrum, L. 1973. Problems of Soil Mechanics and Construction of Soft Clays and Structurally Unstable Soils. State-of-the-art Report, Session 4. Proceedings, 8th International Conference on Soil Mechanics and Foundation Engineering, Moscow, Vol. 3, pp. 111-159.

Bowles, J.E. 1984. *Physical and Geotechnical Properties of Soils*, 2nd Edition. McGraw-Hill Book Company, New York.

Britto, A.M. and Gunn, M.J. 1987. *Critical State Soil Mechanics via Finite Elements*. Ellis Horwood Ltd., Chichester, England.

Broms, B.B. (1964a). Lateral Resistance of Piles in Cohesive Soils; *Journal for Soil Mechanics and Foundation Engineering*, ASCE, Vol. 90, SM2, pp. 27-64.

Broms, B.B. (1964b). Lateral Resistance of Piles in Cohesionless Soils; *Journal for Soil Mechanics and Foundation Engineering*, ASCE, Vol. 90, SM3, pp. 123-156.

Canadian Foundation Engineering Manual. 1992. Third Edition. Canadian Geotechnical Society, Technical Committee on Foundations.

Canadian Highway Bridge Design Code (CHBDC) and Commentary on CAN/CSA-S6-00. 2001. CSA Special Publication, S6.1-00. Canadian Standards Association.

Chapman, L.J. and Putnam, D.F. 1984. *The Physiography of Southern Ontario*, 3rd Edition (Ontario Geological Survey, Special Volume 2). Ontario Ministry of Natural Resources.

Kulhawy, F.H. and Mayne, P.W. 1990. *Manual on Estimating Soil Properties for Foundation Design*. EL-6800, Research Project 1493-6. Prepared for Electric Power Research Institute, Palo Alto, California.

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

Makdisis, F.I., and Seed, H.B. 1978. Simplified Procedure for Estimating Dam and Embankment Earthquake-Induced Deformations. ASCE Journal of the Geotechnical Engineering Division, V. 104, GT7, pp 849-867.

Mesri, G. 1975. Discussion on New Design Procedure for Stability of Soft Clays. ASCE Journal of the Geotechnical Engineering Division, V. 101, GT4, pp. 409-412.

Peck, R.B., Hanson, W.E., and Thornburn, T.H. 1974. Foundation Engineering, 2nd Edition, John Wiley and Sons, New York.

Powers, J.P. 1992. Construction Dewatering – New Methods and Applications, 2nd Edition, John Wiley and Sons, New York.

Schmertmann, J.H. 1975. Measurement of In-Situ Shear Strength. In Proceedings, ASCE Specialty Conference on In-Situ Measurement of Soil Properties, Vol. 2, Raleigh, pp. 57-138.

Terzaghi, K. and Peck, R.B. 1967. Soil Mechanics in Engineering Practice, 2nd Edition, John Wiley and Sons, New York, pp.72

Tokimatsu, K., and Seed, H. 1987. Evaluation of Settlements in Sands Due to Earthquake Shaking. ASCE Journal of Geotechnical Engineering, V.113, N.8.

U.S. Navy. 1971. Soil Mechanics, Foundations and Earth Structures. NAVFAC Design Manual DM-7, Washington, D.C.

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

	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

(b) Cohesive Soils

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 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

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 test¹
CIU consolidated isotropically undrained triaxial test with porewater pressure measurement¹
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
SO₄ concentration of water-soluble sulphates
UC unconfined compression test
UU unconsolidated undrained triaxial test
V field vane (LV-laboratory vane test)
γ unit weight

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

SA\FINAL\DATA\ABBREV\2000\LOFA-D00.DOC

LIST OF SYMBOLS

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

I. General

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation
*	Density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_a	coefficient of secondary consolidation
m_v	coefficient of volume change
c_v	coefficient of consolidation
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation pressure
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction $= \tan \delta$
c'	effective cohesion
c_{u, s_u}	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 + \sigma_3)/2$ or $(\sigma'_1 + \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 + \sigma_3)$
S_t	sensitivity

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

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 03-1111-028		RECORD OF BOREHOLE No NBL-1		1 OF 1 METRIC	
G.W.P. 335-00-00		LOCATION N 5032902.5 E 256523.0		ORIGINATED BY EHS	
DIST 52 HWY 69		BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger		COMPILED BY KG	
DATUM Geodetic		DATE October 25, 2004		CHECKED BY CG	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								20 40 60 80 100						
								○ UNCONFINED + FIELD VANE						
								● QUICK TRIAXIAL × REMOULDED						
197.6	GROUND SURFACE							20 40 60 80 100	20 40 60					GR SA SI CL
0.0	Topsoil													
197.1	Silty Sand to Sandy Silt		1	SS	8									
0.5	Loose													
	Brown, oxidized													
	Moist													
	Clay, some silt, trace sand and organics		2	SS	14									
	Stiff													
196.0	Mottled reddish brown and grey													
1.6	Moist to wet													
	Silt, some clay, trace sand, occasional clay seams		3	SS	6									
195.5	Loose													
2.1	Brownish grey, layered													
	Wet													
	Gravelly Sand, trace silt		4	SS	20									26 61 (13)
194.7	Compact													
2.9	Grey													
	Wet													
	End of Borehole													
	Auger refusal													
	Notes:													
	1. Water level in open borehole at 1.7 m depth upon completion of drilling.													

PROJECT 03-1111-028		RECORD OF BOREHOLE No NBL-2		1 OF 1 METRIC	
G.W.P. 335-00-00		LOCATION N 5032906.2 ; E 256514.2		ORIGINATED BY EHS	
DIST 52 HWY 69		BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger		COMPILED BY KG	
DATUM Geodetic		DATE October 25, 2004		CHECKED BY CG	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)			
								○ UNCONFINED	+ FIELD VANE							● QUICK TRIAXIAL	× REMOULDED	
197.1	GROUND SURFACE						20	40	60	80	100	20	40	60	GR SA SI CL			
0.0	Topsoil																	
0.3	Silty Sand Very loose to loose Light brown and oxidized Moist		1	SS	4	▽												
	Silty Clay, trace sand and organics Stiff Mottled reddish brown Moist		2	SS	12													
195.6																		
1.5	Silt, some clay, to Clayey Silt, trace to some sand Very loose Mottled reddish brown and grey Wet		3	SS	1											0 14 58 28		
194.8																		
2.3	Silty Sand to Sand, some silt, trace gravel Compact Grey Wet		4	SS	16										30 60 (37)			
194.3																		
2.8	Bedrock																	
	Refer to Record of Drillhole NBL-2 for details																	
191.1	End of Borehole																	
6.0	Notes: 1. Auger refusal at 2.8 m depth. 2. Water level in open borehole at 1.0 m depth upon completion of drilling.																	

MISS MTO 031111028AAMTO.GPJ ON MOT.GDT 20/12/05

PROJECT: 03-1111-028

RECORD OF DRILLHOLE: NBL-2

SHEET 1 OF 1

LOCATION: N 5032906.2 ; E 256514.2

DRILLING DATE: October 25, 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	COLOR % RETURN	JN - Joint FLT - Fault SHR - Shear VN - Vein CJ - Conjugate	BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage	PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular	PO - Polished K - Slickensided SM - Smooth Ro - Rough MB - Mechanical Break	BR - Broken Rock NOTE: For additional abbreviations refer to list of abbreviations & symbols.	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec	DIP w.r.t. CORE AXIS	FRACT. INDEX PER .3m	RECOVERY TOTAL CORE %	SOLID CORE %	R.O.D. %	WATER LEVELS INSTRUMENTATION
		- continued from Record of Borehole -		194.31																	
3		Granitic Gneiss Fine to coarse grained Fresh Faintly porous Light grey, pink and black Near horizontal, distinct foliation		2.79																	
4					1									MB,,							
														MB,,							
														MB,,							
														MB,,							
5					2									JN,,							
														JN,,							
														MB,,							
														MB,,							
														MB,,							
6		END OF DRILLHOLE		191.31										MB,,							
				5.79																	
7																					
8																					
9																					
10																					
11																					
12																					

Refer to Record of
Borehole NBL-2

DEPTH SCALE

1:50



LOGGED: EHS

CHECKED: CG

MISS-ROCK-2 03111028AARCK.GPJ GAL-CANADA.GDT 15/05 JFC

PROJECT 03-1111-028			RECORD OF BOREHOLE No NBL-3			1 OF 1 METRIC												
G.W.P. 335-00-00			LOCATION N 5032921.3; E 256478.7			ORIGINATED BY PH												
DIST 52 HWY 69			BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger			COMPILED BY KG												
DATUM Geodetic			DATE November 16, 2004			CHECKED BY CG												
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40						60	80	100	20
197.0	GROUND SURFACE																	
0.0	Topsoil		1	SS	3													
0.4	Silty Sand Greyish brown Silty Clay to Clay Soft to firm Reddish brown to grey Wet		2	SS	6													
			3	SS	3													
			4	TO	PH													
192.6			5	SS	2													
4.4	Fine to coarse Sand, trace gravel, trace silt Very loose to compact Grey Wet		6	SS	13													
			7	SS	8													
190.2																		
6.8	Bedrock Refer to Record of Drillhole NBL-3 for details																	
186.5																		
10.5	End of Borehole Notes: 1. Auger refusal at 6.8 m depth. 2. Water level in piezometer at 0.4 m depth on December 21, 2004.																	

MISS_MTO 031111029AAMTO.GPJ ON MOT.GDT 20/12/05

PROJECT: 03-1111-028

RECORD OF DRILLHOLE: NBL-3

SHEET 1 OF 1

LOCATION: N 5032921.3 ; E 256478.7

DRILLING DATE: November 16, 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.		RUN No.	PENETRATION RATE (mm/min)	FLUSH	COLOUR % RETURN	RECOVERY				R.Q.D. %	FRACT. INDEX PER .3m	DISCONTINUITY DATA				HYDRAULIC CONDUCTIVITY K, cm/sec	Diameter Point Load Index (MPa)	RMC -Q AVG.	NOTES WATER LEVELS INSTRUMENTATION	
				DEPTH (m)	ELEV. (m)					TOTAL CORE %	SOLID CORE %	B Angle	DIP w.r.t. CORE AXIS			TYPE AND SURFACE DESCRIPTION	10 ⁻⁹	10 ⁻⁴	10 ⁻³					10 ⁻²
		- continued from Record of Borehole -		190.20 6.80																				
7		Granitic Gneiss Fine to coarse grained Fresh Faintly porous Light grey, pink and black Near horizontal, distinct foliation			1										JN.,									
8					2										JN., JN., JN.,									
9					3										JN., JN., JN., JN., JN., JN.,									
10																								
		END OF DRILLHOLE		186.46 10.54																			Refer to Record of Borehole NBL-3	
11																								
12																								
13																								
14																								
15																								
16																								

DEPTH SCALE

1 : 50



LOGGED: PH

CHECKED: CG

MISS-ROCK-2 03111028AARCK.GPJ GAL-CANADA.GDT 15/2/05 JFC

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

PROJECT 03-1111-028

RECORD OF BOREHOLE No NBL-5

1 OF 1 **METRIC**

G.W.P. 335-00-00

LOCATION N 5032899.8 : E 256513.1

ORIGINATED BY EHS

DIST 52 HWY 69

BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger

COMPILED BY KG

DATUM Geodetic

DATE December 4, 2004

CHECKED BY CG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE x REMOULDED						
196.9	GROUND SURFACE							20 40 60 80 100							
0.0	Topsoil														
0.4	Silty Sand, trace organics Very loose Light brown		1	SS	2	▽	196								
	Silty Clay, trace fine sand, trace organics Soft to stiff Reddish brown and grey Moist to wet		2	SS	10										
195.1															
1.8	Silt, trace to some gravel, trace fine sand, occasional reddish brown clay seams Very loose Grey and reddish brown Wet		3	SS	3										
194.3							195								
	Fine Sand, trace silt Compact Grey Wet		4	SS	17										
193.9							194								
3.0	End of Borehole Auger Refusal														
Note: 1. Water level in open borehole at 1.2 m depth upon completion of drilling.															

+³, X³

Numbers refer to Sensitivity

○³%

STRAIN AT FAILURE

PROJECT 03-1111-028		RECORD OF BOREHOLE No NBL-6		1 OF 1 METRIC	
G.W.P. 335-00-00		LOCATION N 5032912.0 ; E 256516.6		ORIGINATED BY EHS	
DIST 52 HWY 69		BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger		COMPILED BY KG	
DATUM Geodetic		DATE December 4, 2004		CHECKED BY CG	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× REMOULDED							
197.6	GROUND SURFACE																	
0.0 197.3	Topsoil		1	SS	2													
0.5	Silty fine Sand, trace gravel, trace organics Very loose Light brown, oxidized Wet		2	SS	10													
	Silty Clay, trace fine sand, trace organics, occasional fine sand seams Firm to stiff																	
	Mottled reddish brown Moist to wet		3	SS	6													
195.5 2.1	End of Borehole Auger Refusal																	
Note: 1. Open borehole dry upon completion of drilling.																		

MISS_MTO 031111028AAMTO.GPJ ON MOT.GDT 20/12/05

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

PROJECT NO. 03-1111-028-2							
LOCATION: Proposed Highway 69 NBL / Blair (Sly) Creek Bridge							
DATE: January 05, 2005							
Borehole Number	Sample Number	Rock Type	Sample Depth (ft)	Sample Depth (m)	Test Type	Is (50mm) (MPa)	Approx. UCS ¹ (Is ₅₀ ×23)(MPa)
NBL-2	1	Granitic Gneiss	9.3	2.8	D	2.470	57
NBL-2	2	Granitic Gneiss	11.7	3.6	A	10.320	237
NBL-2	3	Granitic Gneiss	13.4	4.1	D	5.288	122
NBL-2	4	Granitic Gneiss	15.9	4.8	A	8.209	189
NBL-2	5	Granitic Gneiss	16.5	5.0	D	5.535	127
NBL-2	6	Granitic Gneiss	18.3	5.6	D	5.016	115
NBL-3	1	Granitic Gneiss	22.3	6.8	D	5.143	118
NBL-3	2	Granitic Gneiss	23.8	7.3	A	7.351	169
NBL-3	3	Granitic Gneiss	26.8	8.2	D	4.862	112
NBL-3	4	Granitic Gneiss	28.8	8.8	A	6.411	147
NBL-3	5	Granitic Gneiss	31.9	9.7	D	5.169	119
NBL-3	6	Granitic Gneiss	33.6	10.2	D	6.233	143
SUMMARY					Average Axial	8.073	186
					Average Diametral	4.965	114
					St. Dev. Axial	1.668	38
					St. Dev. Diametral	1.092	25
					Number of Axial Tests	4	
					Number of Diametral Tests	8	

¹ 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 Hwy 69 Parry Sound\Reporting\Final\2 - Blair Creek - NBL\Tables\03-1111-028 Table 1 Hwy69 NBL-Blair Creek PLT.xls\POINT LOAD

TABLE 2-A
EVALUATION OF FOUNDATION ALTERNATIVES – SOUTH ABUTMENT
Highway 69 NBL Structure at Blair 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>
Spread Footings on mass concrete pad (Founding Level of about 195.5 m)		Can minimize or eliminate bedrock excavation depending on design footing level.	<p>Variable bedrock surface will require soil excavation followed by mass concrete placement to achieve level footing.</p> <p>Sand and silt strata overlying bedrock combined with relatively high groundwater level may result in seepage into excavation required to expose bedrock. Temporary sheeting system may have to be installed.</p>	<p>Much lower relative costs than piled foundations since less bedrock excavation required.</p> <p>Additional costs may be required for installation of temporary sheeting and dewatering.</p>	<p>If bedrock is higher than anticipated, bedrock removal is required.</p> <p>Variability in bedrock surface will impact mass concrete quantities and excavation depths.</p> <p>Dewatering system may be required if seepage volumes are greater than those anticipated.</p>
Spread Footings on bedrock and compacted Granular 'A' fill (Founding Level of about 195.5 m)		Can minimize or eliminate bedrock excavation depending on design footing level. Eliminates the need for mass concrete placement.	<p>Variable bedrock surface will require soil excavation followed by compacted Granular 'A' engineered fill placement to achieve level footing.</p> <p>Sand and silt strata overlying bedrock combined with relatively high groundwater level may result in seepage into excavation required to expose bedrock. Temporary sheeting system may have to be installed</p> <p>Compacted Granular 'A' pad may require 2H:1V side slopes for sliding resistance resulting in larger area of dewatering required for construction.</p> <p>Lower ULS and SLS values for footings constructed on compacted Granular 'A' fill as compared with values for footings on bedrock.</p>	<p>Much lower relative costs than piled foundations since less bedrock excavation required.</p> <p>Additional costs may be required for installation of larger temporary sheeting and dewatering.</p> <p>Additional costs associated with potential requirement for larger footings due to lower ULS and SLS values.</p>	<p>If bedrock is higher than anticipated, bedrock removal is required.</p> <p>Variability in bedrock surface will impact Granular 'A' quantities and excavation depths.</p> <p>Dewatering system may be required if seepage volumes are greater than those anticipated.</p>

TABLE 2-A
EVALUATION OF FOUNDATION ALTERNATIVES – SOUTH ABUTMENT
Highway 69 NBL Structure at Blair 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>
Spread Footings on bedrock (Founding Level of about 193.9 m)		Can eliminate mass concrete and/or compacted Granular 'A' fill.	Soil excavation required and variable bedrock surface may require bedrock excavation to achieve level footing. Bedrock excavation will be difficult and will have to be blasted using controlled blasting techniques to minimize shattering and over-break. Sand and silt strata overlying bedrock combined with relatively high groundwater level may result in seepage into excavation required to expose bedrock. Temporary sheeting system may have to be installed.	Much lower relative costs than piled foundations since less bedrock excavation required. Additional costs for difficult bedrock excavation. Additional costs may be required for installation of temporary sheeting and dewatering.	Dewatering system may be required if seepage volumes are greater than those anticipated.
Spread Footings perched within embankment fill		Can eliminate bedrock removal and/or mass concrete placement at south abutment.	Potential for differential settlement between south abutment (due to compression of embankment fill) and north abutment (founded on piles driven to bedrock).	Lower relative costs than piled foundations since less bedrock excavation required. Possible higher relative costs than spread footing on bedrock since lower allowable bearing capacity will require larger footing size.	Different footing design required at south and north abutment and potential for differential settlements.
Steel H Piles			Due to shallow depth of bedrock, bedrock excavation to form trench will be required to achieve minimum required piles lengths.	Significant bedrock trench for H-piles will increase costs for blasting and backfilling as compared to costs for bedrock excavation for spread footing alternative.	Not recommended due to significant depth of excavation required in strong bedrock.

n:\active\2003\1111\03-1111-028 urs hwy 69 party sound\reporting\final\2 - blair creek - nbl\tables\table2_evaluation foundation alternatives.doc

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

TABLE 2-B
EVALUATION OF FOUNDATION ALTERNATIVES – NORTH ABUTMENT
Highway 69 NBL Structure at Blair Creek
G.W.P. 335-00-00

<i>Footings 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.	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 sheeting 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 (7 m) excavation required. Temporary sheeting and groundwater control required to expose bedrock surface.</p> <p>Variable bedrock surface will require bedrock and soil excavation with mass concrete placement to achieve level footing. Bedrock will have to be blasted using controlled blasting techniques to minimize shattering and over-break.</p>	Increased cost for groundwater control and temporary sheeting as compared with shallower footings.	Not recommended due to significant depth of excavations.
Shallow Spread Footings on loose silty sand or soft silty clay	X		Low geotechnical resistance. Differential settlements between north and south abutment due to consolidation of underlying soft to firm silty clay under footing and approach embankment loading.	Lower relative costs than piled foundations.	Not recommended due to potential for differential settlements anticipated between north and south abutment.

n:\active\2003\1111\03-1111-028 urs hwy 69 parry sound\reporting\final\2 - blair creek - nbl\tables\table2_evaluation foundation alternatives.doc

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

TABLE 3
Summary of Recommendations at Structure Approach Embankments (incl. Platform Widening)
Highway 69 NBL Structure at Blair Creek
G.W.P. 335-00-00

Highway	Approx. Station	Proposed Works	Surface Conditions	Recommended Embankment Fill Type	Organics Encountered Along Alignment	Recommended Side Slope	Side Berm Recommended	Estimated Post-Construction* Settlement (δ) and Platform** Widening (w) (mm)	Swamp Excavation / Organic Removal OPSD
Highway 69 NBL at Blair Creek Structure	10+675 to 10+695	South Approach Swamp crossing (fill up to 4.3 m high)	Swamp area adjacent to Blair Creek. Bedrock at about 3 m depth.	Rock fill	Yes. Up to 0.3 m below ground surface.	1.25H : 1V	No.	$\delta = 45 + 45 = 90$ $w = 1000$	Remove all organics within footprint of embankment.
	10+735 to 10+755	North Approach Swamp crossing (fill up to 5.2 m high)	Swamp area in flood plain adjacent to Blair Creek. Bedrock at about 7 m depth.	Rock fill	Yes. Up to 0.2 m below ground surface.	1.25H: 1V	No.	$\delta = 45 + 50 = 95$ $w = 1000$	203.010 (new embankment construction over swamp)

n:\active\2003\1111\03-1111-028 urs hwy 69 parry sound\reporting\final\2 - blair creek - nbl\tables\table3_summaryapproachembankmentrecommendations (incl nre 98-200).doc

Note : * Settlements include compression of rockfill plus compression of cohesive layers below embankment (where encountered). Estimate at north approach 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.

TABLE 4
EVALUATION OF SETTLEMENT / STABILITY MITIGATION ALTERNATIVES
North Approach Embankment - Highway 69 NBL Structure at Blair Creek
G.W.P. 335-00-00

<i>Stability/ Settlement Mitigation Option</i>	<i>Rank</i>	<i>Advantages</i>	<i>Disadvantages</i>	<i>Relative Costs</i>	<i>Risks/Consequences</i>
Full Subexcavation and Replacement (up to 4.5 m deep)	1	Stability and long-term settlement issues minimized since all or nearly all weak, soft and compressible material are removed. Stability berms not required.	Permanent sheet pile shoring will be required to support the adjacent Blair Creek near 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.	Additional costs for sub-excavation, disposal and for permanent sheet pile shoring near 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.
Pre-Loading (about 13 months) and Stability Berms (4 m wide x 1 m high on front slope) (see Table 5 for other alternatives for this option)	2	Relatively simple operation; no deep sub-excavation or temporary shoring. Toe berms required on front slope to provide stability.	May lengthen construction time required. Settlement of foundation soil takes 13 months to reach 90% consolidation. Berm construction may have to be temporary due to environmental and hydrology considerations in flood plain of creek.	Low cost. However some additional costs required for berm construction and possibly for berm removal.	Settlement of embankment/ foundation soils will occur. Secondary consolidation (creep) will occur.
Wick Drains / Staged Construction (1.5 m triangular spacing and 7.5 months preload time)	3	Reduce the preload duration. 90 % consolidation settlement complete within 7.5 months of completion of embankment construction. Potential to eliminate the need for toe berm on front slope if construction is staged.	Increased time for installation of wicks. Monitoring of settlements and pore pressures required. Staged construction to eliminate berm requirement would add 4 months to the time to construct embankment to full height.	Cost savings in berm fill and/or sub-excavation and replacement offset by drain installation and monitoring costs.	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.
Light Weight Fill (EPS) Preload plus Partial Unloading / EPS Fill in Embankment Construction (20 m long x 18 m wide x 2 m high)	4	Reduces load on compressible soils thereby increasing stability and reducing settlement of foundation soils. Combined preload plus EPS fill reduces volume of expensive light weight fill required. No toe berms required.	In order to minimize EPS fill volume, a 4 m high preload embankment still required to be constructed and remain in place for 8 months prior to partial unloading and replacement with 2 m high of EPS fill.	Higher cost of fill materials plus increased costs due to preload construction and partial fill removal.	Still require preloading unless large quantity of EPS used (see below). Factor of safety for stability increased. Settlements of foundation soils and embankment fill minimized.

TABLE 4
EVALUATION OF SETTLEMENT / STABILITY MITIGATION ALTERNATIVES
North Approach Embankment - Highway 69 NBL Structure at Blair 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>
Surcharging (2 m high and 6 months surcharge time)	5	Time for 90% of final embankment geometry consolidation settlement will be reduced to about 6 months. Post construction consolidation settlement reduced.	Toe berms now required on side slopes and toe berm on front slope must be increased in size to maintain stability of higher embankment. Front berm: 8.5 m wide x 3 m high Side berms: 8 m wide x 2 m high	Increased cost of construction and material for surcharge and wider toe berms, however, surcharge fill can be reused elsewhere at site.	Still expect some settlement of embankment fill. Secondary consolidation (creep) will occur.
Light Weight Fill (EPS) Full Embankment Construction (20 m long x 18 m wide x 4 m high)	6	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.

n:\active\2003\1111\03-1111-028 urs hwy 69 parry sound\reporting\final\2 - blair creek - nbl\tables\table4_evaluation settlement_stability mitigation alternatives.doc

TABLE 5
EVALUATION OF PRELOAD OPTION FOR SETTLEMENT / STABILITY MITIGATION
North Approach Embankment - Highway 69 NBL Structure at Blair Creek
G.W.P. 335-00-00

<i>Stability/ Settlement Mitigation Option</i>	<i>Rank</i>	<i>Advantages</i>	<i>Disadvantages</i>	<i>Risks/Consequences</i>
Option #2A <ul style="list-style-type: none"> - build embankment to full height (H=5 m) up to abutment with a front slope toe berm (4 m wide x 1 m high) - preload for 13 months (min.) to reach U=90% - remove berm and excavate embankment back so front slope toe is min. 4.5 m back from abutment wall location (for stability during subexcavation/abutment construction) - locally excavate at abutment for pile installation and construction - reconstruct embankment to full height (H=5 m) up to abutment wall (no front slope toe berm required) - no additional preload time required before paving 	1	Abutment piles design can neglect downdrag loads and lateral bending due to vertical and lateral soil movements.	Bridge construction delayed for at least 13 months. Toe berm for front slope stability required for 4 months (min).	Total time for embankment preloading = 13 months. Settlement of embankment/foundation soils will occur. Secondary consolidation (creep) will occur.
Option #2B <ul style="list-style-type: none"> - build embankment to full height (H=5 m) with toe of front slope located a minimum of 8 m back from abutment wall location (for stability during subexcavation / abutment construction) - locally excavate at abutment for pile installation and abutment construction - construct toe berm (4 m wide x 1 m high) in front of abutment wall - construct remainder of embankment to full height (H=5 m) up to abutment wall - preload for 13 months (min.) before paving to reach U=90% - toe berm can be removed after 4 months (min.) partial preload time 	2	No delay in bridge construction.	Abutment piles must be designed for full downdrag and full lateral bending due to vertical and lateral soil movements. Toe berm for front slope stability required for 4 months (min).	Total time for embankment preloading = 13 months. Settlement of embankment/foundation soils will occur. Secondary consolidation (creep) will occur.
Option #2C <ul style="list-style-type: none"> - build embankment to full height (H=5 m) up to abutment with a front slope toe berm (4 m wide x 1 m high) - partial preload for 4 months (min.) - remove berm and excavate embankment back so front slope toe is min. 6 m back from abutment wall location (for stability during subexcavation/abutment construction) - locally excavate at abutment for pile installation and construction - reconstruct embankment to full height (H=5 m) up to abutment wall (no front slope toe berm required) - preload for an additional 10 months (min.) before paving to reach U=90% 	3	Flexibility in start of bridge construction.	Bridge construction delayed for at least 4 months. Abutment piles must be designed for full downdrag and partial lateral bending due to vertical and lateral soil movements. Toe berm for front slope stability required for 4 months (min).	Total time for embankment preloading = 14 months. Settlement of embankment/foundation soils will occur. Secondary consolidation (creep) will occur.

TABLE 5
EVALUATION OF PRELOAD OPTION FOR SETTLEMENT / STABILITY MITIGATION
North Approach Embankment - Highway 69 NBL Structure at Blair Creek
G.W.P. 335-00-00

<i>Stability/ Settlement Mitigation Option</i>	<i>Rank</i>	<i>Advantages</i>	<i>Disadvantages</i>	<i>Risks/Consequences</i>
Option #2D - build embankment to less than full height (H=4 m) up to abutment (no front slope toe berm required) - partial preload for 8 months (min.) - excavate embankment back so front slope toe is min. 6 m back from abutment wall location (for stability during subexcavation/abutment construction) - locally excavate at abutment for pile installation and construction - reconstruct embankment to full height (H=5 m) up to abutment wall (no front slope toe berm required) - preload for an additional 11 months (min.) before paving to reach U=90%	4	No toe berm required on front slope of embankment.	Bridge construction delayed for at least 8 months. Abutment piles must be designed for full downdrag and partial lateral bending due to vertical and lateral soil movements.	Total time for embankment preloading = 19 months. Settlement of embankment/ foundation soils will occur. Secondary consolidation (creep) will occur.

n:\active\2003\1111\03-1111-028 urs hwy 69 parry sound\reporting\final\2 - blair creek - nbl\tables\table5_details of preload option for settlement_stability mitigation.doc

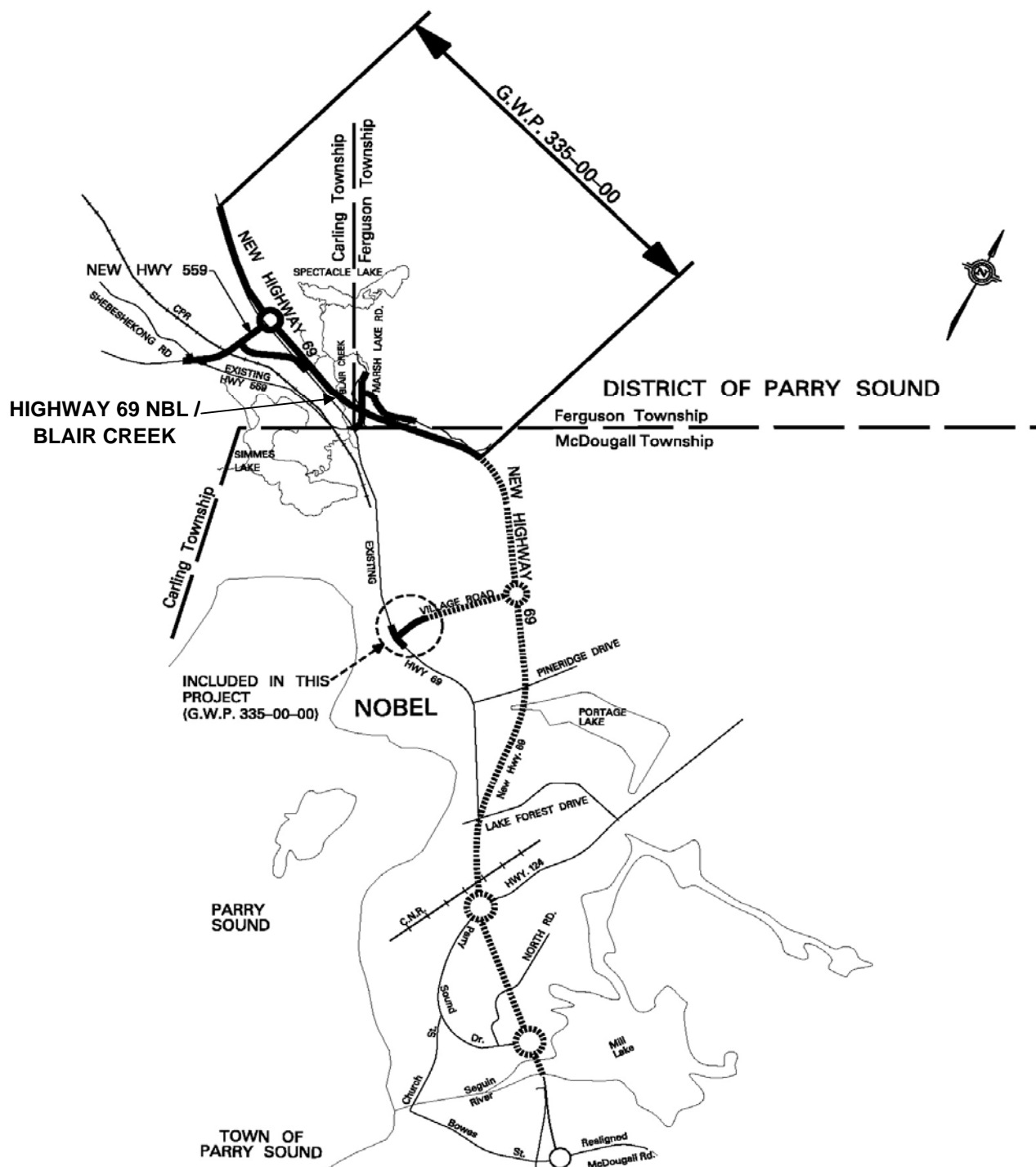
TABLES

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: January 2006
Project: 03-1111-028-2

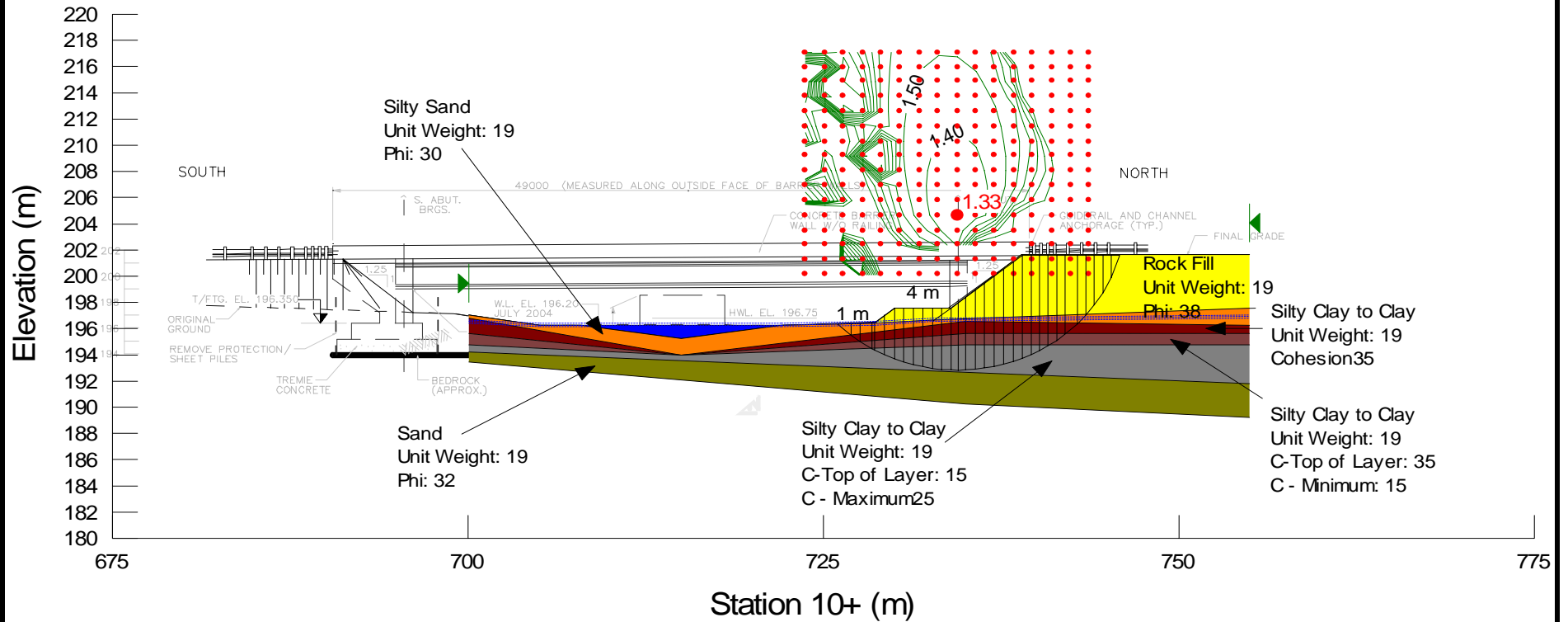
Drawn by: CMG
Checked by: JPD

Golder Associates

Provided in digital format by URS on January 7, 2005

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

FIGURE 2



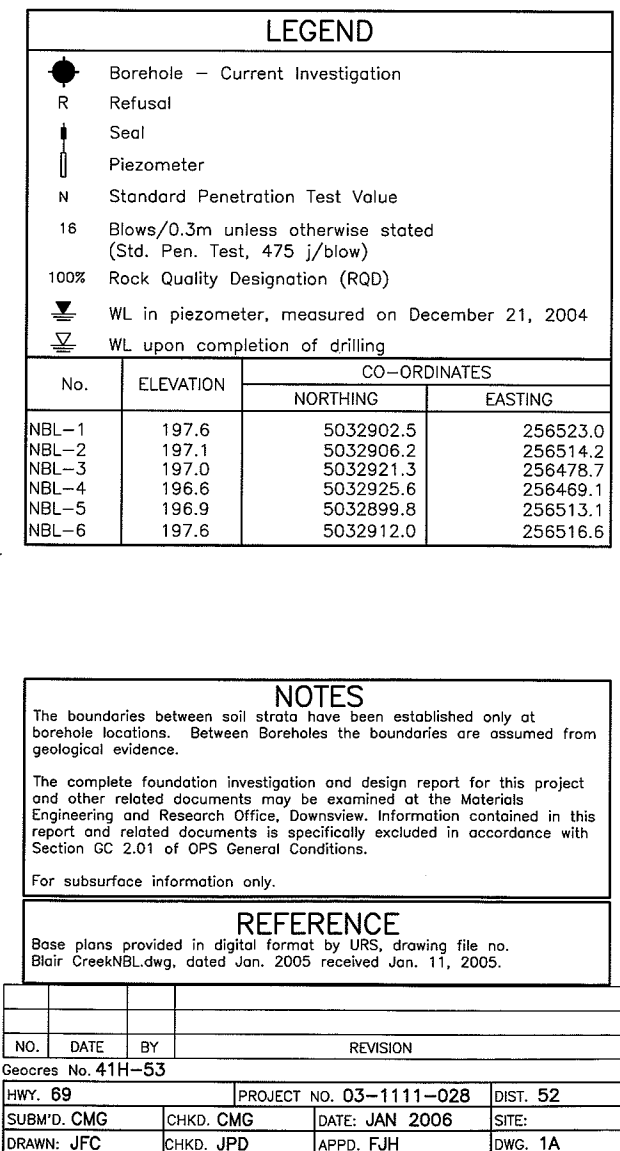
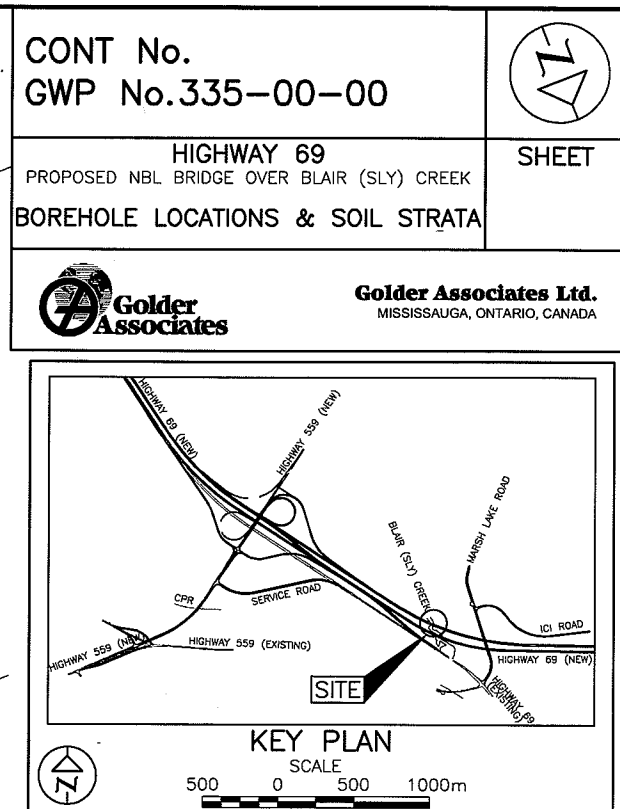
File Name: 03-1111-028 NBL Bridge North Approach Profile_rockfill_berm H=5m b&w.slz
 Last Saved Date: 2/17/2005

Date: January 2006
 Project: 03-1111-028-2

Golder Associates

Drawn: CMG
 Checked: JPD

DRAWINGS



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

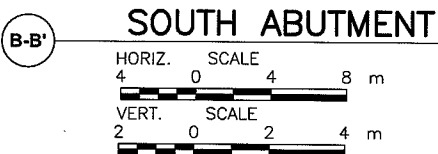
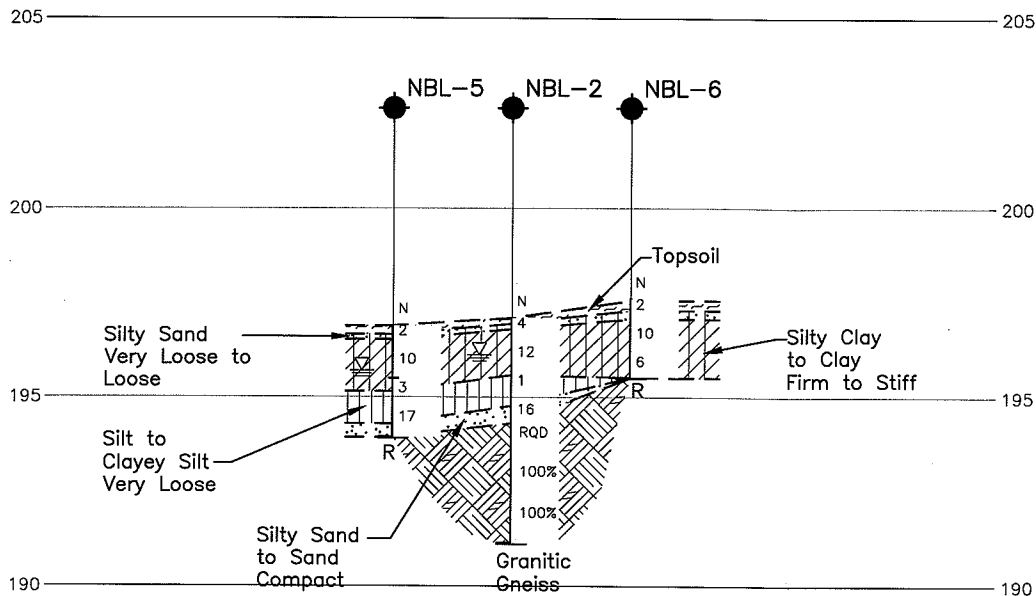
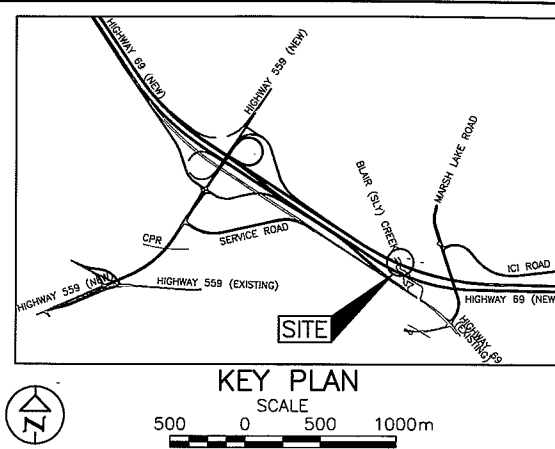
CONT No.
GWP No.335-00-00

HIGHWAY 69
PROPOSED NBL BRIDGE OVER BLAIR (SLY) CREEK
CROSS SECTION

SHEET



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



LEGEND

- Borehole - Current Investigation
- R Refusal
- ⊥ Seal
- ⊥ Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- ≡ WL in piezometer, measured on December 21, 2004
- ≡ WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
NBL-1	197.6	5032902.5	256523.0
NBL-2	197.1	5032906.2	256514.2
NBL-3	197.0	5032921.3	256478.7
NBL-4	196.6	5032925.6	256469.1
NBL-5	196.9	5032899.8	256513.1
NBL-6	197.6	5032912.0	256516.6

NOTES

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

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

For subsurface information only.

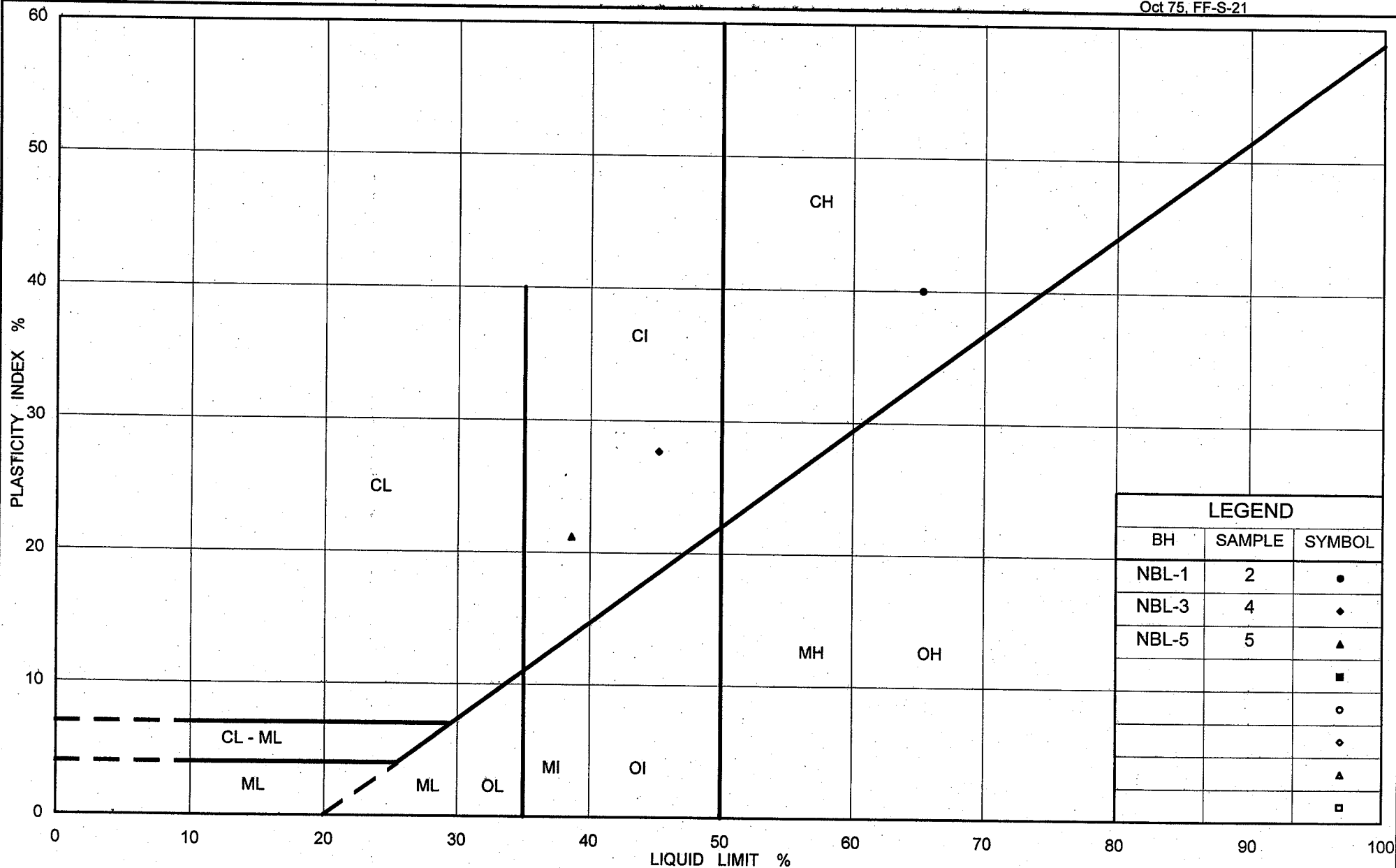
REFERENCE

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



NO.	DATE	BY	REVISION
Geocres No. 41H-53			
HWY. 69	PROJECT NO. 03-1111-028		DIST. 52
SUBM'D. CMG	CHKD. CMG	DATE: JAN 2006	SITE:
DRAWN: JFC	CHKD. JPD	APPD. FJH	DWG. 1B

APPENDIX A
LABORATORY TEST DATA



Ministry of Transportation

Ontario

PLASTICITY CHART Silty Clay to Clay

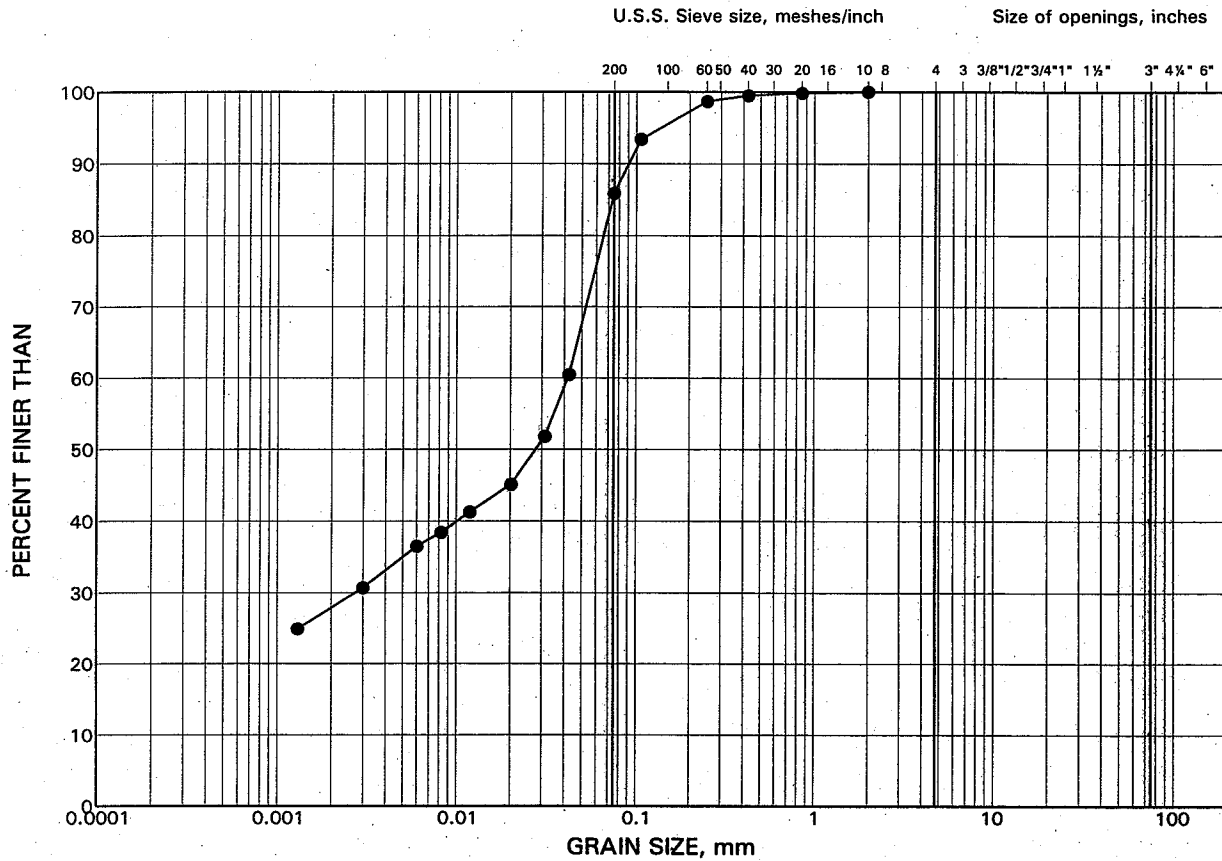
FIG No. A-1

Project No. 03-1111-028-2

GRAIN SIZE DISTRIBUTION

Clayey Silt, some sand

FIGURE A-2



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
•	NBL-2	3	195.3

Date 2/16/2005

Project 03-1111-028-2

Golder Associates

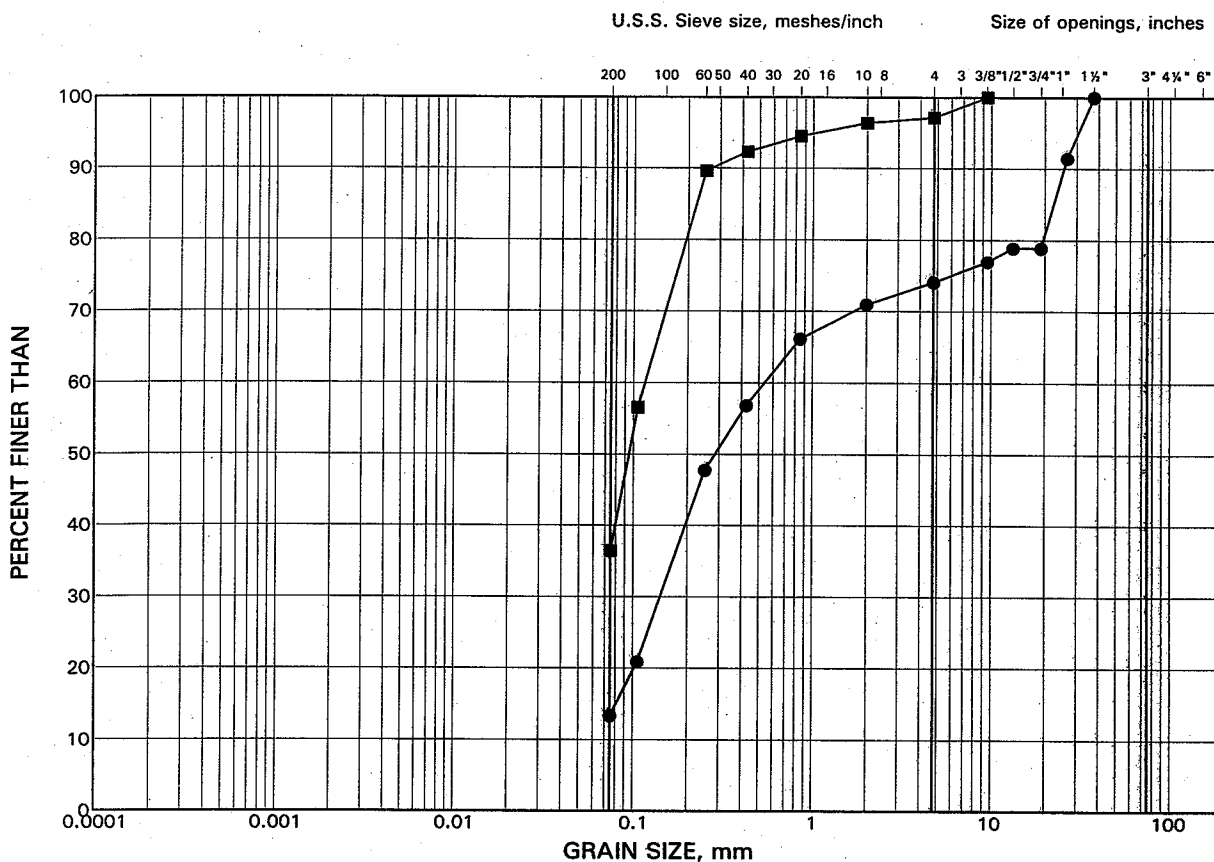
Prepared by LG

Checked by *GA*

GRAIN SIZE DISTRIBUTION

Silty Sand to Gravelly 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)
●	NBL-1	4	195.0
■	NBL-2	4	194.5

APPENDIX B

SAMPLE NON-STANDARD SPECIAL PROVISIONS

MASS CONCRETE – Item No.

Special Provision

Scope of Work

The scope of work for the above noted tender item includes the mass concrete under the South abutment footing.

Construction

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

Basis of Payment

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

EARTH EXCAVATION FOR STRUCTURE – Item No.

ROCK EXCAVATION FOR STRUCTURE – Item No.

UNWATERING STRUCTURE EXCAVATION – Item No.

Special Provision No. 902S01M

Excavation and Backfilling-Structures

902.02 REFERENCES

Section 902.02 of OPSS 902, December, 1983, is amended by the addition of the following:

OPSS 510

902.03 DEFINITIONS

Section 902.03 of OPSS 902, December, 1983, is amended by the addition of the following:

Quality Verification Engineer: means an Engineer with a minimum of five (5) years experience related to excavation and backfilling of structures, or alternatively had demonstrated expertise by providing satisfactory quality verification services for the work at a minimum of two (2) projects of similar scope to the Contract. The Quality Verification Engineer shall be retained by the Contractor to certify that the work is in general conformance with the contract documents and issue of certificate(s) of conformance.

902.04 SUBMISSION AND DESIGN REQUIREMENTS

Section 902.04 of OPSS 902, December, 1983, is deleted and replaced with the following:

902.04.01 Site Survey

Prior to commencing the work, the Contractor shall submit to the Contract Administrator a condition survey of property and structures that may be affected by the work. The survey shall include, but not be limited to, locations and conditions of adjacent properties, buildings, underground structures, utility services and structures such as walls abutting the site.

902.04.02 Working Drawings

Working Drawings for protection systems shall be according to OPSS 539.

Where unwatering is required, the Contractor shall be responsible for the design of the unwatering scheme for the intended purpose. The design of temporary structures or protection system for unwatering shall be according to OPSS 539.

902.04.03 Submission of Certificate of Conformance

The Contractor shall submit to the contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer upon completion of each of the following operations and prior to commencement of each subsequent operations:

EARTH EXCAVATION FOR STRUCTURE – Item No.
ROCK EXCAVATION FOR STRUCTURE – Item No.
UNWATERING STRUCTURE EXCAVATION – Item No.

Special Provision No. 902S01M

- Excavation for Foundation
- Excavation for Backfill and Frost Tapers
- Use of Excavation Material
- Backfilling

The Certificate of Conformance shall state that the work has been carried out in general conformance with the contract documents, specifications and/or stamped working drawings.

902.05.03 Backfill

Subsection 902.05.03 is amended by the addition of the following:

The Contractor shall be responsible for ensuring the quality of the material used for backfill. The quality of the material shall be verified by test results from a qualified and recognized testing laboratory. The frequency of sampling and testing shall be according to ASTM D75-87 and D3665.

902.05.04 Protection System

Section 902.05 of OPSS 902, December, 1983, is amended by the addition of the following:

Protection systems shall be according OPSS 539.

902.07.01 Protection Schemes

Subsection 902.07.01 of OPSS 902, December, 1983, is amended by replacing the word "Engineer" in the last paragraph with the words "Contract Administrator".

902.07.02 Excavation

Subsection 902.07.02 of OPSS 902, December, 1983, is deleted and replaced with the following:

902.07.02.01 General

For excavation, the Contractor shall be responsible for preventing any deterioration of the foundation soil or rock, surface water from entering and eroding the face of the excavation, and build up of hydrostatic pressures which may have harmful effects upon the temporary or permanent structures.

902.07.02.02 Excavation for Foundation

The excavation for foundation shall be inspected and approved by the Quality Verification Engineer prior to construction of the footing. Immediately after the inspection and prior to

EARTH EXCAVATION FOR STRUCTURE – Item No.

ROCK EXCAVATION FOR STRUCTURE – Item No.

UNWATERING STRUCTURE EXCAVATION – Item No.

Special Provision No. 902S01M

commencement of subsequent activity, a certificate of conformance shall be submitted to the Contract administrator.

The Contractor shall be responsible for maintaining the stability of the excavation if any excavation below a stream or channel bed is carried out.

902.08 Measurement for Payment

902.09.01 Structures

Subsection 902.09.01 of OPSS 902, is amended by deleting the first five paragraphs and replacing them with the following:

“Earth Excavation for Structure” and “Rock Excavation for Structure” applies to the specific structure(s) designated, i.e., Bridge, Retaining Wall or Culvert, and is measured by Plan Quantity, as may be revised by Adjusted Plan Quantity, of the volume in cubic metres below the designated payment surface.

The above measurement also includes, where applicable, the excavation quantities, below the designated payment surface, for placing granular backfill and for placing the granular frost tapers.

For open footing culverts, the above measurement also includes the excavation quantities below the designated payment surface but between the plan areas of the footings and above a stream bed or the top of the footings, whichever is higher.

Where the structure excavation overlaps excavation required for other work, deductions will not be made to the structure excavation measurement.

902.10 Basis of Payment

902.10.01 Excavation and Backfill

Subsection 902.10.01 of OPSS 902 is amended by deleting the first paragraphs and replacing it with the following:

Payment at the contract price(s) for the tender item(s) “Earth Excavation for Structure” and “Rock Excavation for Structure” shall be full compensation for all labour, equipment and material for all excavation required, for removal of pavement, curb and gutter and sidewalk except where there is a separate item for removal of pavement, curb and gutter and sidewalk which overlaps pavement, curb and gutter and sidewalk removal required for structure excavation, protection of adjacent works, unwatering backfilling and compacting around the footing according to subsection 902.07.04, placing and compacting of suitable material infill in accordance with OPSS 206 and management of any surplus or unsuitable excavated material, including the cost of disposal areas, all according to the requirements of this specification.

EARTH EXCAVATION FOR STRUCTURE – Item No.
ROCK EXCAVATION FOR STRUCTURE – Item No.
UNWATERING STRUCTURE EXCAVATION – Item No.

Special Provision No. 902S01M

The Contractor shall be responsible for all additional costs due to excavation beyond the required tolerance limits, including but not limited to additional structure design, granular materials, concrete, reinforcing steel and retention of the services of a blasting consultant.

The Contractor shall be responsible for restoring the over excavated area to its original conditions. For over excavation in earth, the backfill materials shall be granular material such as Granular A or B compacted according to OPSS 501. For over excavation in rock, concrete shall be placed to achieve the original excavation limits. Te concrete shall be of the same class concrete as the element it supports.

902.07.02.03 Excavation for Backfill and Frost Tapers

Excavation for backfill and frost tapers shall be carried out according to the specifications and details shown on the contract drawings. The Contractor shall be responsible for restoring the over excavated portion with backfill and shall be compacted to OPSS 501.

The excavation for backfill and frost tapers shall be inspected and approved by the Quality Verification Engineer prior to placement of fill material. Immediately after the inspection and prior to commencement of subsequent activity, a certificate of conformance shall be submitted to the Contract Administrator.

902-07.02.04 Preservation of Channel

Where applicable, the Contractor shall be responsible for restoring a channel back to its original conditions unless other wise specified in the contract.

902.07.02.04 Removals

Where applicable, removal of pavement, curb and gutter, and sidewalks shall be according to OPSS 510.

902.07.03 Unwatering Structure Excavation

Subsection 902.07.03 of OPSS 902, December, 1983, is amended by replacing the first paragraph with the follows:

The Contractor shall carry out all work necessary to prevent disturbance to the founding material. Concrete shall be placed in the dry, unless otherwise specified in the contract.

After the unwatering, the excavation shall be inspected and approved by the Quality Verification Engineer prior to construction of the footing. Immediately after the inspection and prior to commencement of subsequent activity, a certificate of conformance shall be submitted to the Contract Administrator.

EARTH EXCAVATION FOR STRUCTURE – Item No.
ROCK EXCAVATION FOR STRUCTURE – Item No.
UNWATERING STRUCTURE EXCAVATION – Item No.

Special Provision No. 902S01M

902.07.04 Backfilling

Subsection 902.07.04 of OPSS 902, December, 1983, is deleted and replaced with the following:

The Contractor shall ensure that the concrete has reached at least 70 percent of its design strength before placing the backfill against an abutment, wingwall, retaining wall or concrete culvert.

Backfilling shall be according to OPSS 501.

The backfilling shall be according to OPSS 501.

The backfilling operation shall be inspected and approved by the Quality Verification Engineer. Immediately after the inspection and prior to commencement of subsequent activity, a certificate of conformance shall be submitted to the Contract Administrator.

DOWELS Into Rock – Item No.

Special Provision

Scope of Work

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

Construction

Dowels into rock shall be constructed in accordance with OPSS 904. All reinforcing steel supplied shall be in accordance with OPSS 1440 (dowel bars conforming to CSA Standard CSAG30.18, Grade 400).

Where dowels are to be placed in rock, holes shall be drilled to the required depth and size. Hole diameter shall be two times the nominal diameter of the dowel. Each hole shall be cleaned out, grouted and the dowel set in place. Grout shall be of the same strength as the footing concrete (or at least 25 MPa at 28 days).

If the hole contains water, the contractor shall remove the water otherwise a tremie procedure shall be used to completely fill the hole with grout. The dowel shall be forced into the hole after the grout has been placed and while it is still fresh.

Rock Dowel Testing

All proposed testing procedures shall be in general conformance with ASTM D 3689-90 and ASTM D 114381 (Re-approved 1994). Field testing must be carried out in the presence of, and the results reviewed and approved by, the Contract Administrator.

Performance Tests

The following table summarizes the number of rock dowels where performance testing shall be carried out to confirm that the design load of the rock dowels can be achieved. The Contract Administrator will select the rock dowels to be tested.

Bridge	Foundation	Number of Dowels for Performance Testing
Highway 69 NBL / Blair Creek	South Abutment	2

Performance test shall be by axial tensioning using a hydraulic jack with a capacity of at least 1.5 times the ultimate strength of the dowels.

Rock dowels shall be loaded and unloaded in 3 cycles and measurements of the displacement of the dowel shall be carried out at each load increment (step) in accordance with the following schedule:

Cycle-Step	1-1	1-2	1-3	2-1	2-2	2-3	2-4
% Design Load	50	75	25	50	75	100	25

DOWELS Into Rock – Item No.

Special Provision

Cycle-Step	3-1	3-2	3-3	3-4	3-5
% Design Load	50	75	100	110	25

The design load shall be taken as 360 kN for 35M dowels, 252 kN for 30M dowels, 180 kN, for 25M dowels, and 108 kN for 20M dowels.

Displacement measurements shall be carried out at each load increment using calibrated displacement gauges capable of measuring movements of 0.0025 cm. Measurements shall be referenced to an independent fixed referenced pint.

Rock dowels which fail to meet the acceptance criteria shall be replaced at the Contractor's expense and re-tested. If a rock dowel fails, 3 additional rock dowels shall be tested at the same abutment and pier footing as directed by the Contract Administrator.

Acceptance criteria for the rock dowels will be in accordance with the Post-tensioning Institute (1985) as follows:

The dowels are acceptable if the total elastic movement is greater than 80% of the theoretical elastic elongation of the free stressing and is less than the theoretical elongation of the free stressing length plus 50% of the bond length.

Basis of Payment

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

CSP FOR INTEGRAL ABUTMENTS – Item No.

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:

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

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:

<u>Product</u>	<u>Manufacturer</u>
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.