



February 22, 2011

FOUNDATION INVESTIGATION AND DESIGN REPORT

**BRULE CREEK BRIDGE REPLACEMENT
HIGHWAY 652, SITE NO. 39E-057
TOWNSHIPS OF LAMARCHE AND GLACKMEYER, ONTARIO
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 133-88-00, AGREEMENT NO. 5008-E-0037**

Submitted to:
LEA Consulting Ltd.
625 Cochrane Drive, Suite 900
Markham, Ontario
L3R 9R9



GEOCRE NO.: 42H-42

Report Number: 09-1191-0022-1

Distribution:

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

REPORT



**A world of
capabilities
delivered locally**





Table of Contents

PART A – FOUNDATION INVESTIGATION REPORT

1.0 INTRODUCTION.....	1
2.0 SITE DESCRIPTION.....	1
3.0 INVESTIGATION PROCEDURES	2
4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS	3
4.1 Regional Geology	3
4.2 Subsurface Conditions.....	4
4.2.1 Topsoil	4
4.2.2 Asphalt.....	4
4.2.3 Fill	4
4.2.4 Silty Clay (Alluvium)	5
4.2.5 Silty Clay.....	6
4.2.6 Clayey Silt to Silty Clay	6
4.2.7 Silty Clay to Clay.....	7
4.2.8 Silt.....	8
4.2.9 Sand to Sandy Silt.....	8
4.2.10 Cobbles and Boulders.....	9
4.2.11 Bedrock.....	9
4.2.12 Groundwater Conditions	9
5.0 CLOSURE.....	11

PART B – FOUNDATION DESIGN REPORT

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS.....	13
6.1 General.....	13
6.2 Shallow Foundations – Detour Bridge	14
6.2.1 Geotechnical Axial Resistance.....	14
6.2.2 Resistance to Lateral Loads.....	15
6.2.3 Frost Protection.....	15
6.3 Deep Foundations – Replacement Bridge	15
6.3.1 Geotechnical Axial Resistance.....	16



6.3.2	Downdrag.....	17
6.3.3	Pile Driving Notes and Set Criteria.....	17
6.3.4	Resistance to Lateral Loads.....	18
6.3.5	Frost Protection.....	20
6.4	Seismic Considerations	21
6.4.1	Site Coefficient.....	21
6.4.2	Seismic Analysis Coefficient	21
6.5	Lateral Earth Pressures for Design.....	21
6.6	Approach Embankment Design	22
6.6.1	Design Assumptions	23
6.6.2	Stability	24
6.6.2.1	Methodology	24
6.6.2.2	Parameter Selection	24
6.6.2.3	Results of Analysis	25
6.6.2.4	Mitigation of Stability (Detour Front Slopes)	26
6.6.3	Liquefaction Potential and Seismic Analysis	27
6.6.4	Settlement.....	27
6.6.4.1	Methodology	27
6.6.4.2	Settlement Criteria	27
6.6.4.3	Parameter Selection	28
6.6.4.4	Settlement of Embankment Fill/Alluvium	29
6.6.4.5	Settlement of Foundation Soils.....	30
6.7	Subgrade Preparation and Embankment Construction.....	31
6.8	Design and Construction Considerations.....	32
6.8.1	Excavations.....	32
6.8.2	Temporary Shoring	33
6.8.3	Groundwater and Surface Water Control	33
6.8.4	Filter Blanket	34
6.8.5	Obstructions.....	34
6.8.6	Existing Structure Monitoring	34
7.0	CLOSURE.....	34



TABLES

Table 1	Evaluation of Foundation Alternatives – Replacement Bridge
Table 2	Evaluation of Foundation Alternatives – Detour Bridge
Table 3	Evaluation of Stability Mitigation Alternatives – Detour Front Slopes

DRAWINGS

Drawing 1	Brule Creek Bridge, Highway 652, Borehole Locations
Drawing 2	Brule Creek Bridge, Highway 652, Soil Strata
Drawing 3	Brule Creek Bridge, Highway 652, Soil Strata
Drawing 4	Brule Creek Temporary Modular Bridge, Highway 652, Borehole Locations and Soil Strata

FIGURES

Figure 1	Typical Detour Abutment on Compacted Fill Core
Figure 2	Undrained Shear Strength vs. Elevation
Figure 3	Stability Analysis, Main Bridge Front Slope (East Side)
Figure 4A	Stability Analysis, Detour Bridge Front Slope (East Side) with 350 kPa Footing Pressure
Figure 4B	Stability Analysis, Detour Bridge Front Slope (East Side) with 110 kPa Footing Pressure



APPENDICES

Appendix A Record of Boreholes and Drillholes

List of Symbols and Abbreviations

Lithological and Geotechnical Rock Description Terminology

Record of Borehole Sheets (BR10-01 to BR10-08)

Record of Drillhole Sheets (BR10-03 and BR10-04)

Appendix B Laboratory Test Results

Figure B-1 Grain Size Distribution – Sand to Sand and Gravel (Fill)

Figure B-2 Grain Size Distribution – Clayey Silt to Silty Clay (Fill)

Figure B-3 Plasticity Chart – Clayey Silt to Silty Clay (Fill)

Figure B-4 Grain Size Distribution – Silty Clay (Alluvium)

Figure B-5 Plasticity Chart – Silty Clay (Alluvium)

Figure B-6 Grain Size Distribution – Silty Clay

Figure B-7 Plasticity Chart – Silty Clay

Figure B-8 Grain Size Distribution – Clayey Silt to Silty Clay

Figure B-9 Plasticity Chart – Clayey Silt to Silty Clay

Figure B-10 Consolidation Test – BR10-04, Sa 8

Figure B-11 Consolidation Test – BR10-07, Sa 8

Figure B-12 Grain Size Distribution – Silty Clay to Clay

Figure B-13 Plasticity Chart – Silty Clay to Clay

Figure B-14 Grain Size Distribution – Silt

Figure B-15 Grain Size Distribution – Sand to Sandy Silt

Appendix C Non-Standard Special Provisions

NSSP CSP for Integral Abutments

NSSP Pile Capacity Verification Procedure



PART A

FOUNDATION INVESTIGATION REPORT
BRULE CREEK BRIDGE REPLACEMENT
HIGHWAY 652, SITE NO. 39E-057
TOWNSHIPS OF LAMARCHE AND GLACKMEYER, ONTARIO
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 133-88-00, AGREEMENT NO. 5008-E-0037



1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by LEA Consulting Ltd. (LEA) on behalf of Ministry of Transportation, Ontario (MTO) to provide preliminary and detail design services for the replacement of the Brule Creek Bridge, and associated detour bridge, located on Highway 652 (east of Cochrane) between the Townships of Lamarche and Glackmeyer. This report addresses the detail design services for the Brule Creek Bridge, Site No. 39E-057.

The terms of reference and scope of work for the foundation investigation are outlined in MTO's Request for Proposal (RFP) dated November 17, 2008. Golder's proposal P81-1685, dated December 2008, for foundation engineering services associated with the replacement and temporary detour bridges is contained in Sections 5.8 and 6.8 of LEA's Technical Proposal that forms part of the Consultant's Agreement Number 5008-E-0037 for this project. Subsequent to the award of the engineering services contract, the Preliminary and Detail Design investigation phases were combined to Detail Design level only. The work was carried out in accordance with Golder's Supplemental Specialty Quality Control Plan for this project dated September 16, 2009. The General Arrangement drawings for the replacement bridge and detour bridge structures were provided to Golder by LEA on May 19, 2010.

Subsurface information for the existing bridge is contained the Department of Highways report available on GEOCRESS (DHO, 1967). The purpose of this investigation is to establish the subsurface conditions at the proposed replacement and detour structure locations by borehole drilling, rock coring, in situ testing and laboratory testing on selected samples. The location of the investigated area is shown on Drawing 1.

2.0 SITE DESCRIPTION

The site is situated in the Townships of Lamarche and Glackmeyer on Highway 652 crossing the Brule Creek, approximately 7 km east of the junction with Highway 11. The surrounding land is generally flat-lying, mainly comprised of grass and tree covered terrain extending beyond the limits of the site and scattered residences. The creek banks adjacent to the existing bridge area are vegetated with landscaped grass and small shrubs. The creek flows in a northerly direction and is less than 6 m wide at the existing bridge location. During the field investigation in June 2010, a small beaver dam was observed immediately south of the bridge. On the return site visit on August 12, 2010, the beaver dam had been breached/broken.

We understand that the Brule Creek channel was relocated in 1968 approximately 30 m to the west of its original location and the existing bridge structure was then constructed. The existing bridge consists of a 27 m long by 8.5 m wide five-span structure founded on approximately 15 m long timber piles. The existing ground surface along the existing highway alignment ranges from Elevation 251.1 m to 251.4 m rising from west to east. The existing embankment front slopes are formed at about 2.5 horizontal to 1 vertical (2.5H:1V) and the side slopes are at about 2H:1V.

The water level in the creek was measured at Elevation 247.0 m upstream (south) and Elevation 246.4 m downstream (north) of the bridge in June 2010, when the beaver dam was still in place. The normal high water level is reported to be Elevation 247.1 m. The existing highway embankment grade is up to approximately 4 m above the creek water level, or about 2.5 m above the surrounding ground surface.



3.0 INVESTIGATION PROCEDURES

The fieldwork at the bridge site was carried out between June 22 and 28, 2010, at which time a total of eight (8) boreholes (BR10-01 to BR10-08) were advanced. Four boreholes (BR10-01 to BR10-04) were advanced for the proposed main bridge abutments and approaches and four (4) boreholes (BR10-05 to BR10-08) were advanced for the proposed detour bridge abutments and approaches. On August 12, 2010, a shallow borehole was drilled immediately adjacent to Borehole BR10-07 to obtain additional Shelby tube samples of the clay stratum for laboratory testing. The locations and elevations of the boreholes are shown on Drawing 1.

All boreholes were drilled using a CME 55 track-mounted drill rig supplied and operated by George Downing Estate Drilling Ltd. (Downing) of Grenville-Sur-La-Rouge, Quebec. The boreholes were advanced using 108 mm inside diameter (I.D.) continuous flight hollow stem augers and/or NW casing with wash boring. In general, soil samples were obtained at intervals of depth of about 0.75 m to 3.0 m, using a 50 mm outer diameter (O.D.) split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586). Samples of the cohesive soils were obtained using 76 mm O.D. thin-walled 'Shelby' tubes (ASTM D1587, Standard Practice for Thin-Walled Tube Sampling) for relatively undisturbed samples. Field vane shear tests were conducted in cohesive soils for determination of undrained shear strengths (ASTM D2573, Standard Test Method for Field Vane Strength Shear Test) using MTO Standard 'N' size vanes. Rock core samples were obtained using an 'NQ' size core barrel in Boreholes BR10-03 and BR10-04. All boreholes were backfilled upon completion in accordance with Ontario Regulation 903 Wells (as amended by Ontario Regulation 372).

The boreholes for the main and detour bridge approaches were advanced to a depth of 15.8 m below ground surface. The boreholes for the detour bridge abutments were advanced to a depth of 20.4 m below ground surface and the boreholes for the main bridge abutments were advanced to casing refusal at depths of 39.5 m and 46.9 m below ground surface. Bedrock core was obtained for lengths of 4.6 m and 3.4 m in Boreholes BR10-03 and BR10-04, respectively, at the main bridge abutments.

The groundwater conditions and water levels in the open boreholes were observed during the drilling operations and are described on the Record of Borehole sheets in Appendix A. Piezometers were installed in Boreholes BR10-01 and BR10-07 to allow monitoring of the stabilized groundwater level at these locations. The piezometers consist of 19 mm O.D. rigid PVC tubing with a 3.0 m long slotted screen sealed within the clayey silt deposit. A flush mounted cap was installed in Borehole BR10-01 which is located on the shoulder of the existing highway. A plastic cap was installed in Borehole BR10-07 which is located on the north shoulder of the proposed detour. Details of the piezometer installations and water level readings are presented on the attached Record of Borehole sheets in Appendix A. The piezometers were decommissioned on August 12, 2010.

Flowing artesian groundwater conditions were encountered in Boreholes BR10-03 and BR10-04 upon encountering the sand to sandy silt deposit underlying the clayey silt to clay deposit at depth. Details of the sealing of the artesian boreholes are given in Section 4.2.12.

Traffic protection was implemented for the boreholes drilled within the roadway in accordance with the Traffic Protection Plan for this project and the MTO Book 7, Temporary Conditions Manual of the Ontario Traffic Manual (2001).

The fieldwork was supervised throughout by a member of our technical staff, who located the boreholes, arranged for the clearance of underground service locations, supervised the drilling and sampling operations, logged the boreholes, and examined and cared for the soil and rock core samples. The samples were identified



in the field, placed in appropriate containers, labelled and transported to our Sudbury geotechnical laboratory where the samples underwent further visual examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate. Classification testing (water content, Atterberg limits and grain size distribution) was carried out on selected samples. One-dimensional consolidation (oedometer) tests were carried out on two Shelby tube samples of the cohesive soil. In addition, uniaxial compressive strength (UCS) testing was carried out on two selected specimens of the bedrock core recovered from the boreholes.

The locations of the boreholes for the proposed main bridge were laid out by Golder relative to the existing bridge features. The locations of the boreholes for the proposed detour bridge were laid out relative to the detour centreline stakes, which were surveyed by Trow Geomatics. Golder surveyed the geodetic ground surface elevation of the boreholes once completed, referencing an existing benchmark located approximately 14 m north and 42 m east of the existing bridge (MTO BM #818171). The northing and easting coordinates (MTM NAD 83) were determined by plotting the boreholes relative to the working points shown on the General Arrangement drawings. The northing and easting coordinates, ground surface elevations and borehole depth for each borehole are presented on the Record of Borehole sheets in Appendix A and summarised below.

Borehole	Borehole Location		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing	Easting		
BR10-01	5435628.2	308645.3	250.8	15.8
BR10-02	5435619.6	308715.2	251.8	15.8
BR10-03	5435626.8	308694.4	251.4	45.0
BR10-04	5435621.1	308666.2	251.1	50.3
BR10-05	5435609.0	308640.0	249.9	15.8
BR10-06	5435606.3	308665.0	248.5	20.4
BR10-07	5435611.6	308695.4	249.6	20.4
BR10-08	5435608.9	308720.4	250.0	15.8

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

Published literature indicates that the site is located in the Western Abitibi Subprovince of the Superior Province (Geology of Ontario; OGS Special Volume 4)¹. The bedrock of this domain consists of metavolcanic and minor metasedimentary rocks.

Based on terrain mapping by the Ontario Geological Survey², the subsurface soils in the vicinity of the site consist of glaciolacustrine plain deposits comprised of peat and clayey silts, overlying bedrock.

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

² Northern Ontario Engineering Geology Terrain Study, OGS Electronic Map



4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions, as encountered in the boreholes advanced during this investigation, together with the results of the laboratory tests carried out on selected soil samples, are presented on the Record of Borehole sheets in Appendix A. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling and observations of drilling progress and cuttings. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Further, subsurface conditions will vary between and beyond the borehole locations. The inferred soil stratigraphy based on the results of the boreholes is shown in profile and cross-section on Drawings 2 to 4.

The existing ground surface encountered at the boreholes along Highway 652 (BR10-01 to BR10-04) ranges from Elevation 250.8 m to 251.8 m sloping up from west to east. The existing ground surface encountered at the boreholes along the detour alignment (BR10-05 to BR10-08) ranges from Elevation 248.5 m to 250.0 m.

In general, the subsoils consist of fill overlying alluvium and a thick deposit of clayey silt to silty clay. Silt and sand to sandy silt deposits underlie the clayey deposit at depth and are in turn underlain by cobbles and boulders over metavolcanic bedrock. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Topsoil

A 30 mm to 200 mm thick layer of topsoil was encountered from ground surface in Boreholes BR10-05 to BR10-08.

4.2.2 Asphalt

A 115 mm and 200 mm thick layer of asphalt was encountered from ground surface in Boreholes BR10-04 and BR10-03, respectively.

4.2.3 Fill

Boreholes BR10-01 to BR10-04, which were advanced within the shoulders or driving lanes of the existing highway, encountered embankment fill consisting of granular fill and/or clayey fill from the ground surface or underlying the asphalt pavement. Silty clay fill was also encountered in Boreholes BR10-07 and BR10-08, located on the east side of the creek (i.e. just west of the original creek location) underlying the topsoil.

Granular Fill

Granular fill consisting of moist, brown sand, sand and gravel or silty sand was encountered in Boreholes BR10-01 to BR10-04. The fill contains trace to some silt and/or trace gravel and/or trace clay. Trace organics was noted in Boreholes BR10-02 and BR10-03. Glass fragments were noted in the silty sand fill in Borehole BR10-03. Borehole BR10-04 encountered wood and cobbles and boulders within the sand fill.



The granular fill is between 1.5 m and 2.9 m thick and was encountered between Elevation 251.8 m and 250.8 m.

The SPT 'N'-values measured in the granular fill range from 10 blows to 38 blows per 0.3 m of penetration indicating a compact to dense relative density.

Grain size distribution tests were carried out on three samples of the granular fill and the results are shown on Figure B-1.

The natural water content measured on samples of the granular fill are between about 2 percent and 10 percent.

A hydrocarbon odour was noted in the sand fill at a depth of 0.8 m in Borehole BR10-03 and at a depth of 1.5 m in Borehole BR10-04.

Clayey Silt to Silty Clay Fill

A 1.4 m to 5.7 m thick layer of moist to wet, brown to grey clayey silt to silty clay fill was encountered below the granular fill in Boreholes BR10-02 and BR10-03 and below the topsoil in Boreholes BR10-07 and BR10-08. The surface of the cohesive fill was encountered between Elevation 250.3 and 248.4 m. The cohesive fill in Boreholes BR10-07 and BR10-08 and the upper 3.5 m in Borehole BR10-02 contains trace to some sand and gravel and trace organics. The cohesive fill in Borehole BR10-03 and the lower 2.2 m in Borehole BR10-02, located on the east side of the existing bridge, consists of brown to black clay, sand, wood, roots and trace to some organics.

SPT 'N'-values recorded in the cohesive fill range from 3 blows to 10 blows per 0.3 m of penetration suggesting a soft to stiff consistency.

Grain size distribution tests were carried out on three samples of the cohesive fill and the results are presented on Figure B-2. Atterberg limits tests were carried out on four samples of the cohesive fill and test results are presented on Figure B-3. The liquid limits range from 33 percent to 47 percent, plastic limits range from 17 percent to 26 percent and the plasticity indices range between 12 percent and 21 percent. These results indicate the deposit is classified as clayey silt of low plasticity to silty clay of intermediate plasticity.

One organic content test carried out on one sample of the cohesive fill indicates 8 percent organics.

The natural water content measured on samples of the cohesive fill ranges from about 15 percent to 46 percent.

A hydrocarbon odour was noted in the silty clay fill samples at a depth of 4.6 m in Borehole BR10-02.

4.2.4 Silty Clay (Alluvium)

A deposit of moist to wet, grey to black silty clay containing trace to some sand, trace gravel and trace to some organics (alluvium) was encountered below the fill materials in Boreholes BR10-01, BR10-04, BR10-07 and BR10-08 and below the topsoil in Borehole BR10-06. The surface of the alluvium deposit was encountered between Elevation 249.0 m and 247.2 m and the thickness ranges from 0.7 m to 1.9 m.



The SPT 'N'-values measured within the silty clay alluvium range from 6 blows to 25 blows per 0.3 m of penetration suggesting a firm to very stiff consistency. Typically, the 'N'-values were less than 14 blows per 0.3 m of penetration suggesting the deposit is firm to stiff.

Grain size distribution tests were carried out on three samples of the silty clay alluvium and the results are presented on Figure B-4. Atterberg limits tests were carried out on two samples of the alluvium deposit and the results are presented on Figure B-5. The liquid limits are 42 percent and 43 percent, the plastic limits are 22 percent and 25 percent and the plasticity indices are 18 percent and 21 percent. The results indicate the deposit is classified as a silty clay of intermediate plasticity.

The natural moisture content measured on several samples of the alluvium range from about 24 percent to 49 percent.

4.2.5 Silty Clay

A deposit of moist to wet, brown to grey, silty clay was encountered below the alluvium in Boreholes BR10-01 and BR10-04 on the west side of the creek, below the topsoil in Borehole BR10-05 and underlying the alluvium in BR10-06 to BR10-08 along the proposed detour. The deposit contains trace to some sand and gravel and trace organics and is considered to be the desiccated/weathered crust of the main clayey silt to clay deposit at the site. The surface of the silty clay deposit was encountered between Elevation 249.9 m and 245.4 m and the thickness ranges from 1.0 m to 2.6 m.

The SPT 'N'-values measured in the silty clay crust range from 4 blows to 20 blows per 0.3 m of penetration. One in situ field vane test carried out within the crust measured an undrained shear strength of 30 kPa. The SPT 'N'-values together with the in situ vane suggest the silty clay crust generally has a firm to very stiff consistency.

Grain size distribution tests were carried out on three samples of this deposit and the results are presented on Figure B-6. Atterberg limits tests were carried out on four samples of the silty clay crust and the test results are presented on Figure B-7. The liquid limits range from 38 percent to 41 percent, the plastic limits range from 17 percent to 19 percent and the plasticity indices range between 21 percent and 24 percent. The results indicate the crust material is classified as a silty clay of intermediate plasticity.

The natural moisture content measured on several samples of the silty clay crust range from 23 percent to 35 percent.

One organic content test was carried out on one sample of the crust material, taken below the alluvium, and indicates 6 percent organics.

4.2.6 Clayey Silt to Silty Clay

The upper portion of the main deposit of cohesive material consists of wet, grey clayey silt to silty clay containing trace to some sand and trace to some gravel. This deposit was encountered below the silty clay crust in Boreholes BR10-01 and BR10-04 to BR10-08 and below the fill material in Boreholes BR10-02 and BR10-03. The surface of this deposit was encountered between Elevation 247.6 m and 244.2 m. In Boreholes BR10-01,



BR10-02, BR10-05 and BR10-08, this deposit extends to the borehole termination depths indicating a thickness between 8.6 m and 13.5 m. In Boreholes BR10-03, BR10-04, BR10-06 and BR10-07, the deposit was proven for a thickness between 9.8 m and 10.8 m.

The SPT 'N'-values measured in the clayey silt to silty clay range from weight of hammer (i.e. 0 blows) to 25 blows per 0.3 m of penetration. Typically, the 'N'-values are between 1 blow and 4 blows per 0.3 m of penetration. In situ field vane testing carried out within this stratum measured undrained shear strengths between 27 kPa and 42 kPa. The SPT 'N'-values together with the in situ vanes suggest the deposit generally has a very soft to firm consistency, with a stiff consistency near the surface of the stratum where it is closer to the overlying crust in some boreholes. An undrained shear strength of 72 kPa was measured in Borehole BR10-05 in a suspected silt seam.

Grain size distribution tests were carried out on several samples of this stratum, including one sample which contained a fine sand seam, and the results are presented on Figure B-8. Atterberg limits tests were carried out on several samples of the clayey silt to silty clay deposit and the test results are presented on Figure B-9. The liquid limits range from 19 percent to 38 percent, the plastic limits range from 10 percent to 17 percent and the plasticity indices range from 8 percent to 22 percent. The results indicate the deposit is classified as a clayey silt of low plasticity to a silty clay of intermediate plasticity.

The natural moisture content measured on several samples of the main clay deposit range from 19 percent to 50 percent.

Two laboratory consolidation (oedometer) tests were carried out on specimens of the clayey silt to silty clay obtained from Boreholes BR10-04 (existing highway alignment) and BR10-07 (detour alignment) and the test results are shown on Figures B-10 and B-11, respectively. The preconsolidation stresses were estimated from the Void Ratio versus logarithmic Pressure plots using the Casagrande method as well as from the Total Work versus Pressure plots. The relevant consolidation test results are summarized below:

Borehole/ Sample Number	Elevation (m)	σ_{vo}' (kPa)	σ_p' (kPa)	$\sigma_p' - \sigma_{vo}'$ (kPa)	OCR	e_o	C_r	C_c	c_v^* (cm ² /s)
BR10-04/8	243.2	95	135	40	1.4	0.713	0.03	0.17	6.7×10^{-4}
BR10-07/8	243.2	65	130	65	2.0	0.775	0.02	0.12	4.7×10^{-3}

Note: *For approximate stress range of $70 \leq \sigma_v' \leq 280$ kPa

where: σ_{vo}' effective overburden stress in kPa

σ_p' preconsolidation stress in kPa

OCR overconsolidation ratio

e_o initial void ratio

C_c compression index (based on void ratio)

C_r recompression index (based on void ratio)

c_v coefficient of consolidation in cm²/s in the normally consolidated range

4.2.7 Silty Clay to Clay

The lower portion of the main deposit of cohesive soil consists of wet, grey silty clay to clay containing trace sand and was encountered below the upper portion of the clayey silt to silty clay stratum in the deepest boreholes, namely BR10-03 and BR10-04 (main bridge abutments) and BR10-06 and BR10-07 (detour bridge abutments). The surface of this deposit was encountered between Elevation 236.0 m and 234.1 m. In



Boreholes BR10-06 and BR10-07, this cohesive stratum was drilled for a thickness of 4.9 m and 7.9 m and extended to the borehole termination depths. In Boreholes BR10-03 and BR10-04, this lower portion of the cohesive deposit was fully penetrated for a thickness of 12.3 m and 15.3 m, respectively.

The SPT 'N'-values measured in the lower portion of the silty clay to clay deposit range from 1 blow to 5 blows per 0.3 m of penetration. In situ field vane testing carried out within this stratum measured undrained shear strengths between 23 kPa and 50 kPa. The SPT 'N'-values together with the in situ vanes suggest the deposit generally has a very soft to firm consistency.

Grain size distribution tests were carried out on two samples of the lower portion of the cohesive deposit and the results are presented on Figure B-12. Atterberg limits tests were carried out on four samples of the silty clay to clay deposit and the results are presented on Figure B-13. The liquid limits range from 47 percent to 59 percent, the plastic limits range from 20 percent to 22 percent and the plasticity indices range from 27 percent to 36 percent. The results indicate that this portion of the deposit is classified as a silty clay of intermediate plasticity to a clay of high plasticity.

The natural moisture content measured on several samples of the main clay deposit range from 40 percent to 50 percent.

4.2.8 Silt

A deposit of wet, grey silt containing trace clay to clayey silt and trace sand was encountered below the silty clay to clay deposit in Boreholes BR10-03 and BR10-04. The surface of the silt deposit was encountered at Elevation 222.1 m and 220.3 m and the thickness of the deposit in both boreholes is 3.0 m.

The SPT 'N'-values measured in the silt deposit range from 11 blows to 17 blows per 0.3 m of penetration indicating a compact relative density.

Grain size distribution tests were carried out on two samples of the silt and the results are shown on Figure B-14.

The natural moisture content measured on two samples of the silt is 24 percent and 28 percent.

4.2.9 Sand to Sandy Silt

A deposit of wet, brown to grey sand to sandy silt containing trace to some clay was encountered below the silt deposit in Boreholes BR10-03 and BR10-04. The surface of the sand to sandy silt deposit in these two boreholes was encountered at Elevation 219.1 m and 217.3 m and the thickness of the deposit is 4.6 m to 12.2 m, respectively.

The SPT 'N'-values measured in the sand to sandy silt range from 11 blows to 35 blows per 0.3 m of penetration indicating a compact to dense relative density.

Grain size distribution tests were carried out on two samples of the sand to sandy silt and the results are shown on Figure B-15.

The natural moisture content measured on two samples of the sand to sandy silt is 23 percent and 24 percent.



4.2.10 Cobbles and Boulders

A 3.7 m and 0.9 m thick deposit of cobbles and boulders was encountered underlying the sand to sandy silt deposit in Boreholes BR10-03 and BR10-04, respectively. The surface of the cobbles and boulders deposit was encountered at Elevation 214.5 m (BR10-03) and 204.2 m (BR10-04). Bedrock coring techniques (in NQ size) were used to advance the boreholes through this deposit.

4.2.11 Bedrock

Bedrock was encountered at Elevation 210.8 m and 204.2 m (i.e. at depths of 40.6 m and 46.9 m below existing grade) in Boreholes BR10-03 and BR10-04 and was cored for 4.4 m and 3.4 m lengths, respectively. The retrieved bedrock core is described as massive, fine grained, dark grey, mafic metavolcanic bedrock with granitic veins and healed and partially healed joints, as presented in the Record of Drillhole sheets in Appendix A.

The Rock Quality Designation (RQD) measured on the core samples ranges from 61 percent to 100 percent, which indicates rock mass of fair to excellent quality as per Table 3.10 of the Canadian Foundation Engineering Manual (CFEM, 2006). The Total Core Recovery (TCR) during bedrock coring was 100 percent.

Laboratory UCS testing was carried out on two core samples of the bedrock. The UCS values are presented below and the test results indicate the bedrock is very strong as per Table 3.5 of the CFEM (2006).

Borehole	Elevation (m)	UCS (MPa)
BR10-03	207.3	175
BR10-04	203.1	121

4.2.12 Groundwater Conditions

Groundwater levels were measured in the open boreholes during and upon completion of drilling. Piezometers were installed in Boreholes BR10-01 and BR10-07 and sealed within the clayey silt to silty clay deposit to monitor the groundwater levels over time. The measured groundwater levels in the open boreholes and piezometers are presented below.



Borehole	Installation	Time and/or Date	Groundwater Depth* (m)	Groundwater Elevation (m)
BR10-01	Open borehole	Upon completion of drilling	4.9	245.9
	Piezometer	June 29, 2010	5.9	244.9
		August 12, 2010	3.4	247.4
BR10-02	Open borehole	Upon completion of drilling	6.1	245.7
BR10-03	Open borehole	Upon completion of drilling	2.0 m above ground surface (7.0 m above creek water level)	253.4
BR10-04	Open borehole	Upon completion of drilling	0.9 m above ground surface (5.6 m above creek water level)	252.0
BR10-05	Open borehole	Upon completion of drilling	14.4	235.5
BR10-06	Open borehole	Upon completion of drilling	16.0	232.5
BR10-07	Piezometer	June 29, 2010	4.8	244.8
		August 12, 2010	0.4	249.2
BR10-08	Open borehole	Upon completion of drilling	Dry to bottom of boreholes at 5.8 m depth	--

*Depth unless otherwise indicated.

Groundwater levels encountered in the boreholes during and shortly after drilling may not be representative of static groundwater levels since the groundwater levels in the boreholes may not have stabilized on completion of drilling. Further, surface water was noted to be ponded at the toe of the existing highway embankment near Borehole BR10-07 at the time of the groundwater level measurements, which may have resulted in an artificially high groundwater level in Borehole BR10-07 on August 12, 2010.

The water level in Brule Creek was measured at Elevation 247.0 m upstream (south) and 246.4 m downstream (north) of the bridge at the time of the investigation in June 2010. The normal high water level is reported to be Elevation 247.1 m. A beaver dam located on the south side of the bridge resulted in the 0.6 m elevation difference between the upstream and downstream water level measurements.

Groundwater and creek water levels in the area are subject to seasonal fluctuations and to fluctuations after precipitation events and snowmelt. Although perched water was not encountered within the embankment fill during the investigation, it is possible that water is perched within the cohesionless and/or cohesive fill.

Artesian groundwater conditions were encountered in Boreholes BR10-03 and BR10-04 upon penetrating into the sand to sandy silt deposit. The groundwater levels were measured at Elevation 253.4 m and 252.0 m in Boreholes BR10-03 and BR10-04, respectively (corresponding to 7.0 m and 5.6 m above the creek level). The boreholes were sealed at the source as follows, consistent with Ontario Regulation 903 Wells (as amended by Ontario Regulation 372):



- Borehole BR10-03 was sealed using geotextile socks filled with bentonite pellets and pushed down into the borehole to a depth of 39.6 m (Elevation 211.8 m) after removing the core barrel and NW casing from the hole. The bentonite filled geotextile socks were temporarily held down with the drill rods to prevent heaving. Subsequently, alternating layers of bentonite pellets and clay cuttings were dropped into the borehole as the casing was removed followed by a bentonite seal placed in the borehole to the ground surface. Cold patch asphalt was used to restore the ground surface.
- Bentonite slurry drilling mud was used during drilling of Borehole BR10-04 to reduce the uplift effect of the artesian flows so as to facilitate borehole advancement and bedrock coring. Borehole BR10-04 was sealed using bentonite pellets initially from the bottom of the borehole at a depth of 50.3 m (Elevation 200.8 m) followed by the placement of additional bentonite pellets to a depth of 28.0 m (Elevation 243.5 m) after removing the core barrel and NW casing from inside the hollow stem augers. The borehole was then backfilled with alternating layers of clay cuttings and bentonite pellets followed by a near surface seal of bentonite. Cold patch asphalt was used to restore the ground surface.

On August 12, 2010, during the return visit to site to obtain water level readings and decommission the piezometers, it was confirmed visually that Boreholes BR10-03 and BR10-04 did not show artesian flow groundwater conditions.

5.0 CLOSURE

The field drilling program was supervised by Mr. Indulis Dumpis. This report was prepared by Mr. David Muldowney and the technical aspects were reviewed by Ms. Sarah E.M. Coyne, P.Eng., Associate. A quality control review of the report was provided by Mr. Jorge M.A. Costa, P.Eng., Principal and Golder's Designated MTO Contact for this project.



Report Signature Page

GOLDER ASSOCIATES LTD.

D. Muldowney

David Muldowney, B.Eng.
Geotechnical Group

S.E.M. Coyne



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

J.M.A. Costa



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

DAM/SEMC/JMAC/lb

n:\active\2009\1190 sudbury\1191\09-1191-0022 lea brule and wicklows\7000 reporting\brule\final\09-1191-0022 rpt 11feb22 brule creek final fdr.docx



PART B

FOUNDATION DESIGN REPORT
BRULE CREEK BRIDGE REPLACEMENT
HIGHWAY 652, SITE NO. 39E-057
TOWNSHIPS OF LAMARCHE AND GLACKMEYER, ONTARIO
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 133-88-00, AGREEMENT NO. 5008-E-0037



6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides design recommendations on the foundation aspects of the proposed new Highway 652 bridge structure, and associated temporary detour bridge, over Brule Creek. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation at the site.

The interpretation and recommendations presented 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.

6.1 General

We understand that the original Brule Creek Bridge was replaced in 1968. Bridge replacement work included diverting Brule Creek to the west of the then existing structure and relocating the new bridge approximately 30 m west of the pre-1968 location, such that the present east abutment is located at the approximate location of the original west abutment.

The existing bridge consists of a 27.1 m long by 8.5 m wide five-span structure. The existing bridge abutments and piers are reportedly founded on approximately 14 m to 15 m long timber piles driven to about Elevation 236 m. Based on these assumed pile embedment lengths, the timber piles are friction piles terminating in the clayey silt or lower clay deposit.

We understand that the proposed replacement bridge will be a 20 m long by 11.6 m wide integral abutment bridge with 6 m long approach slabs constructed along the current alignment. The proposed grade of the replacement bridge is between Elevation 251.5 m and 251.7 m, up to about 0.4 m above the existing grade. The new bridge will be slightly wider than the existing structure, requiring approximately 1.5 m of embankment widening on each side of the existing embankment.

A 7.4 m wide and 30 m long temporary, single lane modular (ACROW) detour bridge, located to the south (upstream) of the existing bridge, will be required to divert the traffic while the new bridge is being constructed. The detour embankment grade is expected to range between Elevation 251.0 m and 251.2 m, between about 2.5 m and 1.6 m above the existing ground surface at the proposed west and east abutments, respectively.

The subsurface conditions in the vicinity of the proposed main structure and detour bridge generally consist of fill and/or silty clay alluvium underlain by an extensive deposit of grey, clayey silt to clay, of which the upper 1.0 m to 2.6 m, where encountered, is considered to be the desiccated/weathered crust. Bedrock was encountered at depths of about 47 m and 41 m below ground surface (Elevation 204.2 m and 210.8 m) on the west and east sides of the creek, respectively. Artesian conditions were noted during the advancement of Boreholes BR10-03 and BR10-04 after penetrating the sand to sandy silt deposit at about Elevation 219 m and 217 m, respectively.



Given the relatively low geotechnical axial resistance of friction piles at this site, we recommend that the replacement bridge be supported on steel H-piles driven to bedrock. Shallow spread footings are not recommended due to the presence of organics at the abutments and the low geotechnical resistance available. Spread footings on a granular pad are also not recommended due to the potential for settlement of the subsoils due to the grade raise and embankment widening as well as the footing pressure. For the detour, we recommend the abutments be supported on shallow footings placed on a compacted granular pad over the clayey silt to clay deposit.

Tables 1 and 2 summarize the advantages, disadvantages, relative costs and risks/consequences of the foundation alternatives for the replacement and detour structures. Discussion and design recommendations for the various alternatives, where appropriate, are given in the sections below.

The recommendations on the foundation design aspects of the new structure presented in this report take into consideration the impact of the detour bridge foundations and approach embankments on the existing bridge foundations and approach embankments during construction as well as the removal of the existing and detour bridge.

6.2 Shallow Foundations – Detour Bridge

We recommend founding the detour bridge footings on a granular pad extending into the firm to very stiff silty clay crust or the main deposit of clayey silt to silty clay. The surface of the silty clay crust was encountered at about Elevation 247.4 m and 246.6 m at the proposed west and east abutments, respectively. Given a typical proposed underside of footing at Elevation 249.1 m, the granular pad would be between 1.7 m and 2.5 m thick. Given that the required thickness of soil cover for frost protection in this area is 2.5 m (see Section 6.2.3), the granular pads should extend to no higher than Elevation 246.6 m at the proposed west and east abutments. This founding pad elevation is recommended not only to achieve adequate frost protection cover, but to achieve a minimum thickness (typically 2.5 m) of granular material to achieve the bearing resistances given below.

In general, the granular pad should be constructed entirely of Granular 'A' meeting the requirements of Special Provision (SP) 110S13 (Aggregates), provided it is constructed in the dry. However, given stabilized groundwater levels between Elevation 247.4 m and 249.2 m and a normal high creek water level at Elevation 247.1 m (see Section 4.2.12), a portion of the granular pad will be below water, if dewatering is not carried out. Therefore, for this condition, we recommend constructing the lower portion of the pad using Granular 'B' Type II meeting the requirements of SP 110S13 (Aggregates) to 0.6 m above the normal high water level (i.e. to Elevation 247.7 m). It has been observed in the field that Granular 'B' Type II compacts to an adequate degree below the water level without any additional compactive effort, provided it is not placed in more than 2 m of water and provided that the surface above the water is compacted properly. After placement and compaction of the Granular 'B' Type II, Granular 'A' can be placed to the underside of the footing.

6.2.1 Geotechnical Axial Resistance

For spread footings placed on a compacted Granular 'A' core (extending to Elevation 246.6 m) and constructed in the dry, a factored geotechnical axial resistance at Ultimate Limit States (ULS) of 600 kPa may be used. A corresponding Serviceability Limit States (SLS) value of 350 kPa may be used assuming a 2 m to 3 m wide strip or spread footing. These values assume a minimum 2.5 m thick granular pad placed below the base of the footing on or within the silty clay crust or main deposit of clayey silt to silty clay. For footings placed on a



granular pad constructed without dewatering (i.e. Granular 'B' Type II to 0.6 m above the water level overlain by Granular 'A' to the underside of footing), the factored axial geotechnical resistance at SLS to be used for design is 250 kPa.

The granular pad should extend at least 1 m beyond the plan limits of the footing and be sloped down and outwards no steeper than 1H:1V in general accordance with MTO guidelines and Figure 1. The granular pad should be constructed in accordance with OPSS 902 (Excavation and Backfilling) and compacted in accordance with SP 105S10 (Compaction).

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

6.2.2 Resistance to Lateral Loads

Resistance to lateral forces/sliding resistance between the base of the poured concrete (i.e. cast-in-place) footing and the compacted granular pad should be calculated in accordance with Section 6.7.5 of the CHBDC. The coefficient of friction, $\tan \phi'$ may be taken as 0.70 between the base of the cast-in-place concrete footings and the compacted Granular 'A' pad, constructed in-the-dry. For pre-cast concrete footings, $\tan \phi'$ should be taken as 0.56. This value represents an unfactored value.

6.2.3 Frost Protection

All footings should be provided with a minimum of 2.5 m of soil cover for frost protection (Ontario Provincial Standard Drawings (OPSD) 3090.100 Foundation Frost Depths for Northern Ontario). Alternatively, rigid polystyrene insulation could be used to reduce the required thickness of soil cover. As a guideline for design, it is generally adopted by the MTO that a thickness of 25 mm of rigid polystyrene insulation should be assumed to be equivalent to about 300 mm of conventional soil cover. The insulation, if used, should be placed vertically along the face of the foundation (to the base of the footing) and extend horizontally for a distance of 2.5 m beyond the face. A minimum of 1 m of soil cover should be placed over the rigid insulation.

6.3 Deep Foundations – Replacement Bridge

Although artesian conditions were encountered in the cohesionless subsoil deposits penetrated at the abutment locations, in order to achieve the axial resistance to allow for integral abutment design, we recommend that the replacement bridge be supported on steel H-piles driven to bedrock. Alternatively, piles could be driven to a set within the main clayey silt to clay deposit or the sand to sandy silt deposit at depth although the geotechnical axial resistance potentially achievable in these deposits will likely not be sufficient to allow the structural design to proceed.

Due to the presence of cobbles and boulders above the bedrock and due to artesian conditions in the cohesionless deposits, caissons are not considered as a feasible foundations alternative.



6.3.1 Geotechnical Axial Resistance

A factored geotechnical axial resistance at ULS of 1,800 kN may be used for the design of steel HP310X110 piles driven to/into the boulder layer overlying bedrock or to the surface of the mafic metavolcanic bedrock. However, in order to reduce the total number of piles required at the foundation elements and in order to minimize damage to the piles during penetration through the boulder layer overlying the bedrock surface, a heavier pile section (i.e. HP310X132) may be considered. For the design of steel HP310X132 piles driven to/into the boulder layer or to the bedrock surface, a factored geotechnical axial resistance at ULS of 2,100 kN may be used. The resistances for the two pile types given above have been reduced to 90 percent of the bedrock “end-bearing” values as the piles may be “hanging up” on the boulder layer. These values represent a structural limitation for the piles rather than a geotechnical limitation. The geotechnical resistances at SLS for 25 mm of settlement (for the length of piles required at this site) will be greater than the factored axial resistances at ULS. Since the bedrock is considered to be an unyielding material, ULS conditions will govern for this foundation type.

For piles driven to bedrock, the estimated tip elevations are presented below. The elevations are based on the depth to bedrock encountered in the boreholes advanced for the proposed abutments and an underside of pile cap at about Elevation 246 m, as shown on the General Arrangement drawing.

Foundation Unit	Borehole Numbers	Bedrock Surface Elevation	Approximate Design Pile Length*
West Abutment	BR10-04	204.2 m	42 m
East Abutment	BR10-03	210.8 m	35.5 m

*For piles driven to the bedrock surface.

The elevations given above should be assumed to be the design pile tip elevations. However, practically, the piles could “hang up” on the cobbles and boulders deposit overlying the bedrock. The zone where hard driving may be encountered is within 0.9 m and 3.7 m of the bedrock surface at the west and east abutments, respectively.

The length of the replacement bridge is shorter than the existing bridge and, as such, the proposed abutments will be closer to the creek than the existing abutments. Since the boreholes could not be advanced through the existing bridge deck and the ground topography limited access along the existing embankment side slopes, the boreholes were located approximately 3 m to 4 m from the proposed abutments. Therefore, the depths to bedrock at the location of the new bridge abutments may differ from the bedrock depths at the borehole locations given above. Further, due to the location of the proposed abutments relative to the existing timber pile bents, it is not expected that the steel H-piles for the new bridge abutment would intercept the existing timber piles.

For piles driven to bedrock through the artesian deposits or for friction piles, a filter sand blanket (see Section 6.8.4) should be constructed immediately below the pile cap to dissipate artesian groundwater and filter soil fines that may be carried upwards to the surface of the native soils. The potential need for grouting of any space/voids created along the piles should also be considered. In addition, flange reinforcement or driving shoes/rock points should not be used as discussed in Section 6.3.3.



If corrugated steel pipes (CSPs) are installed as part of the integral abutment design for the main bridge (through which the piles will be driven), which we understand is not the case for this site, the CSPs should be backfilled with a loose, fine to medium sand. A Non Standard Special Provision (NSSP) detailing the installation method and gradation of this sand should be included in the Contract Documents; an example is provided in Appendix C.

For friction piles, the geotechnical axial resistance is a function of the shaft resistance and the toe resistance. The factored geotechnical axial resistances at ULS and the geotechnical axial resistances at SLS for various lengths of driven HP310X110 steel piles are presented below, including the recommended option of 22 m long piles terminating above Elevation 224 m.

Embedment Length*	Approximate Tip Elevation	Soil Deposit Pile Terminated Within	Factored Geotechnical Axial Resistance at ULS	Geotechnical Axial Resistance at SLS
22 m	224 m	Silty clay to clay	350 kN	225 kN
25 m	221 m	Silty clay to clay	400 kN	275 kN
30 m	216 m	Sand to sandy silt	600 kN	400 kN

*Below underside of pile cap at Elevation 246.0 m. Piles installed to below Elevation 222 m may encounter artesian conditions.

Clayey silt to silty clay fill containing wood and organics was encountered below the proposed underside of pile cap to Elevation 244.2 m in Borehole BR10-03 at the east abutment. This material should be removed as part of the embankment subgrade preparation as discussed in Section 6.7.

6.3.2 Downdrag

Since the grade raise is less than 0.4 m at the abutments and settlement of the subgrade soils will occur rapidly during construction, downdrag loads need not be considered in the design.

6.3.3 Pile Driving Notes and Set Criteria

All pile installation/driving should be in accordance with OPSS 903 (Deep Foundations). For piles driven to bedrock, the piles should not be fitted with rock points, driving shoes or flange plates (reinforced tips) in order to minimize the width of the gap that may be created as the piles are driven through the silty clay to clay stratum. This will increase the likelihood of the clayey silt to silty clay to “self-seal” against the pile and hence reduce the potential for the creation of a pathway for artesian groundwater along the pile. The heavier pile section, HP 310X132, is recommended to reduce the potential for damage to the pile during driving and penetration through the cobbles and boulders overlying the bedrock. For piles driven to bedrock, Note 5 in Clause 3.3.3 of the Structural Manual (MTO, 2008) should be used on the drawings:

- Piles to be driven to bedrock.

For piles driven to bedrock, set criteria are highly dependent on pile driving hammer type and the selected pile type. 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 need to be set to also avoid overdriving and possibly damaging the piles.

Based on our experience, consideration should be given to the following preliminary criteria for piles driven to bedrock. 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 in stages up to the maximum rated energy over about 40 blows. This procedure is intended to improve the process of seating the pile on the bedrock surface. A final set of no less than 10 blows per 12 mm of penetration should be obtained at the maximum hammer energy.

If friction piles are used, the pile capacity must be verified in the field by the use of the Hiley Formula (Standard Structural Drawing SS 103-11) during the final stages of driving to achieve an ultimate capacity of 875 kN for piles driven to Elevation 224 m. The ultimate geotechnical axial resistance predicted from the Hiley Formula should then be multiplied by a geotechnical resistance factor equal to 0.4 in accordance with Table 6.1 in the CHBDC (2006) to verify the factored ULS design value. The pile driving note to be added to the drawings for this project for the recommended alternative of friction piles driven to Elevation 224 m is Note 3 in Clause 3.3.3 of the Structural Manual:

- Piles to be driven in accordance with Standard Structural Drawing SS 103-11 using an ultimate geotechnical resistance of 875 kN per pile but not below Elevation 224 m without the approval of the engineer.

Assessment of ultimate pile resistance by the Hiley Formula should commence once the pile reaches a depth of not more than 1.5 m above the design pile tip elevation given above and at 0.5 m intervals of depth until the ultimate axial resistance is achieved. If the ultimate capacity as determined by the Hiley Formula is not achieved within the 1.5 m interval down to the design pile tip elevation, the Contractor should stop pile driving and notify the Contract Administrator. At this depth, the pile should be allowed to rest for 48 hours and the Hiley Formula should then be applied immediately upon re-striking the pile. If the ultimate capacity is still not achieved after the 48-hour wait period, the Contract Administrator should be notified and the Contractor must obtain authorization from the Contract Administrator prior to driving the pile below the design pile tip elevation. An NSSP should be included in the Contract to address the pile capacity verification procedure and suggested wording is included in Appendix C.

6.3.4 Resistance to Lateral Loads

Lateral loads can be resisted fully or partially by the use of battered steel H-piles. If vertical piles are used, the resistance to lateral loading will have to be derived from the soil in front of the piles.

The evaluation of the piles subjected to lateral loads should take into account such factors as 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 moment, the soil resistance that can be mobilized, the tolerable lateral deflection at the head of the pile and the pile group effects.



The lateral load response of a single pile may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h , (kPa/m) is determined in accordance with Section C6.8.7 in the Commentary to the CHBDC based on the equation for cohesionless soils given below (CFEM, 1992).

For cohesionless soils:

$$k_h = \frac{n_h z}{B} \quad \text{where} \quad \begin{array}{l} n_h \text{ is the constant of horizontal subgrade reaction (kPa/m)} \\ z \text{ is the depth (m)} \\ B \text{ is the pile diameter or width (m)} \end{array}$$

and for cohesive soils:

$$k_h = \frac{67 s_u}{B} \quad \text{where} \quad \begin{array}{l} s_u \text{ is the undrained shear strength of the soil (kPa)} \\ B \text{ is the pile diameter or width (m)} \end{array}$$

It is understood that an integral abutment foundation design is being considered, however, we understand CSP liners are not required at this site. Where the integral design includes the installation of 3 m long CSP liners (with the annular space between the pile and the liner filled with uniform grained, uncompacted sand), the upper portion of the H-piles will be generally free to flex and move laterally within the limits of the CSP. With this design, the passive lateral resistance over the length of the pile within the CSP liner should be based on the resistance provided by loose sand. The passive lateral resistance on the exterior of the CSP should be based on the resistance provided by the surrounding soil conditions.

The lateral resistance of the piles should be developed primarily from the passive resistance of the soil. The values of n_h and s_u to be used to calculate the coefficient of horizontal subgrade reaction (k_h) to be utilized in the structural analysis for the piles at this location are given below.

Foundation Element Main Bridge (Relevant Borehole)	Soil Unit	Elevation (m)	n_h (kPa/m)	s_u (kPa)
West Abutment (BR10-04)	Loose Sand (Filter Blanket)	246.0 to 245.5	1,300	-
	Soft to Firm Clayey Silt	245.5 to 241.0	-	30
	Soft to Firm Clayey Silt to Clay	241.0 to 220.3	-	35
	Compact Silt, Sand to Sand and Silt, Cobbles and Boulders	220.3 to 204.2	4,400	-
East Abutment (BR10-03)	Loose Sand (Filter Blanket)	246.0 to 245.5	1,300	-
	Firm Clayey Silt (Fill)	245.5 to 244.2*	1,300	-
	Very Soft to Firm Clayey Silt to Clay	244.2 to 222.1	-	35
	Compact to Dense Silt, Sand to Sandy Silt, Cobbles and Boulders	222.1 to 210.8	4,400	-

*Options for removal and replacement of the existing clayey silt fill/alluvium below the abutments are discussed in Sections 6.6.4.4 and 6.7.



For a single HP310X110 or HP310X132 vertical pile extending to the bedrock surface, the estimated factored lateral resistances at ULS and at SLS (for 10 mm of horizontal deflection at the pile cap) are presented below. These values are based on analysis carried out using the Broms' (1964) method as outlined in the CFEM (2006) and the commercially available program LPILE Plus (Version 5.0), produced by Ensoft Inc.

Pile Size	Foundation Location	Lateral Resistance (kN)	
		ULS (Factored)	SLS (10 mm of deflection)
HP310X110	West Abutment	120	40
	East Abutment	120	40
HP310X132	West Abutment	135	45
	East Abutment	135	45

Based on the above discussion, it is considered that both the structural and geotechnical resistances of the piles should be evaluated to establish the governing case at ULS. At SLS, the horizontal resistance of the piles will be controlled by deflections and the horizontal resistance of the pile should be calculated based on the coefficient of horizontal subgrade reaction (k_h) of the soil as discussed above. The SLS resistance should be taken as that corresponding to a horizontal deflection of 10 mm at the underside of the pile cap for units supporting abutments (CHBDC Commentary C6.8.7.1).

The upper zone of soil (down to a depth below the pile cap equal to about $1.5 \times B$ after Broms' (1964), where B = pile diameter) should be neglected in the calculation of lateral resistance of the pile to account for disturbance effects during installation.

Group action for lateral loading should also 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 horizontal subgrade reaction (NAVFAC, 1982) in the direction of loading by a reduction factor, R , as follows:

Pile Spacing in Direction of Loading d = Pile Diameter	Subgrade Reaction Reduction Factor
8d	1.00
6d	0.70
4d	0.40
3d	0.25

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

6.3.5 Frost Protection

At this site, the pile caps should be provided with a minimum of 2.5 m of conventional soil cover for frost protection (as per OPSD 3090.100 Foundation Frost Depths for Northern Ontario). Alternatively, rigid polystyrene insulation could be used to reduce the required thickness of soil cover. As a guideline for design, it is generally adopted by the MTO that a thickness of 25 mm of rigid polystyrene insulation should be assumed to be equivalent to about 300 mm of conventional soil cover. The insulation, if used, should be placed vertically along the face of the foundation (to the base of the pile cap) and extend horizontally for a distance of 2.5 m beyond the face. A minimum of 1 m of soil cover should be placed over the rigid insulation.



6.4 Seismic Considerations

6.4.1 Site Coefficient

For seismic design purposes, the Site Coefficient, S , for this site, based on experience and considering the guidelines in Section 4.4.6 of the CHBDC may be taken as 1.5, consistent with Soil Profile Type III.

6.4.2 Seismic Analysis Coefficient

The potential for seismic (earthquake) loading must also be considered for the design of abutment stems/retaining walls in accordance with Section 4.6 of the CHBDC. According to Table A3.1.1 of the CHBDC, this site is located in Seismic Zone 1. The site-specific zonal acceleration ratio for the Cochrane area is 0.05. Based on experience, for the subsurface conditions at this site, a 50 percent amplification of the ground motion may occur (i.e. Site Coefficient, $S=1.5$ for Soil Profile III from Table 4.4 of CHBDC), resulting in an increase in the Peak Horizontal Acceleration (PHA) from 0.05 g to 0.075 g at the ground surface.

We understand based on Section 4.4.4 of the CHBDC, that this bridge structure is assigned Seismic Performance Zone 1. Given this, and in accordance with Section 4.4.5.1 of the CHBDC, no seismic analysis is required for structures located in Seismic Zone Performance 1.

6.5 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment stems and any associated wing walls/retaining walls will depend on the type and method of placement of the backfill materials, the nature of the soils behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls. As discussed in Section 6.4.2, seismic (earthquake) loading need not be analyzed for this structure.

The following recommendations are made concerning the design of the abutment walls. 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 SP 110S13 Granular 'A' or Granular 'B' Type II but containing less than 5 percent passing the No. 200 sieve size should be used as backfill behind the walls. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with SP 105S10 (Compaction). Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3101.150 (Walls Abutment, Backfill) and OPSD 3121.150 (Walls Retaining, Backfill).
- For structures that are not comprised of integral or semi-integral abutments, rock fill may be used as backfill behind the walls and the material should meet the specifications as outlined in the Northern Region Directive for backfill to structures adjacent to rock fill embankments, dated November 2002. Other aspects of rock backfill requirements should be in accordance with OPSD 3101.200 (Walls Abutment, Backfill Rock).



- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with CHBDC Section 6.9.3 and Figure 6.6. Other surcharge loadings should be accounted for in the design, as required.
- For restrained structures, the granular fill should be placed in a zone with width equal to at least 2.5 m behind the back of the walls (in accordance with Figure C6.20(a) of the Commentary to the CHBDC). For unrestrained structures, granular fill should be placed within the wedge shaped zone defined by a line drawn at no steeper than 1.5H:1V extending up and back from the rear face of the base of the footing (in accordance with Figure C6.20(b), Case II, of the Commentary to the CHBDC).
- For restrained structures, the pressures are based on the proposed embankment fill materials and the existing overburden soils and the following parameters (unfactored) may be used assuming the use of granular fill or rock fill:

	Earth Fill 21 kN/m ³	Rock Fill 19 kN/m ³
Soil unit weight:		
Coefficients of static lateral earth pressure:		
Active, K_a	0.31	0.22
At rest, K_o	0.47	0.35

- For unrestrained structures, 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' 22 kN/m ³	Granular 'B' Type II 21 kN/m ³
Soil unit weight:		
Coefficients of static lateral earth pressure:		
Active, K_a	0.27	0.27
At rest, K_o	0.43	0.43

If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design. The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.9.1 and Table C6.6 of the Commentary to the CHBDC).

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

6.6 Approach Embankment Design

The replacement Brule Creek Bridge will be located along the same alignment as the existing structure. The centreline of the detour will be located about 18 m south of the centreline of the existing Highway 652 alignment. The ground surface along the existing highway ranges from Elevation 251.1 m to 251.4 m, rising from west to east. The proposed grade of the highway is between Elevation 251.5 m and 251.7 m, resulting in a grade raise of about 0.4 m above the existing ground surface. The proposed abutments are about 3 m to 4 m closer to the creek than the current ends of the bridge (i.e. the existing bridge has no actual abutments).



The existing embankment front slopes are formed at about 2.5H:1V and the side slopes are at about 2H:1V. The new bridge will be slightly wider than the existing, requiring approximately 1.5 m of embankment widening on each side of the existing embankment and 0.4 m grade raise.

The ground surface along the proposed detour alignment ranges from Elevation 248.5 m to 250.0 m, rising from west to east. The proposed grade of the detour embankment ranges between Elevation 251.0 m and 251.2 m, resulting in filling between about 2.5 m and 1.6 m above the existing ground surface at the proposed west and east abutments, respectively.

The subsurface conditions in the vicinity of the proposed replacement and detour approach embankments generally consist of fill and/or silty clay alluvium (containing organics) underlain by extensive deposit of grey, clayey silt to clay, of which the upper 1.0 m to 2.6 m, where encountered, is considered to be the desiccated/weathered crust. Thin deposits of silt, sand to sandy silt and cobbles and boulders are present between the overlying clay deposits and underlying bedrock which was encountered at about 47 m and 41 m below the ground surface on the west and east sides of the creek, respectively.

The east side of the creek contains more fill and alluvium, to approximately Elevation 244.2 m, being lowest in Borehole BR10-03, located at the proposed east abutment of the main bridge. The east side of the existing bridge correlates to the west side of the pre-1968 creek location, which could explain the greater depth of material containing wood and organics. The fill and/or alluvium extends to Elevation 247.4 m on the west side of the existing creek (Boreholes BR10-04 and BR10-06), about 3 m higher than on the east side.

Unstabilized groundwater levels were measured in the open boreholes and range between 2.0 m above ground surface (or about 7.0 m above the creek water level) to 16.0 m below ground surface, between Elevation 253.4 m and 232.5 m, respectively. Groundwater levels measured in the piezometers approximately six weeks after installation are Elevation 247.4 m and 249.2 m; however, the higher groundwater level could have been influenced by nearby surface water.

The water level in the river was measured at Elevation 247.0 m upstream (south) and 246.4 m and downstream (north) of the bridge at the time of the investigation in June 2010; a beaver dam located on the south side of the bridge resulted in the 0.6 m elevation difference. The normal high water level is reported to be Elevation 247.1 m.

The following sections present the design assumptions and methodology, parameter selection and results of stability and settlement analysis for the new approach embankments (main highway and detour alignments), including recommendations for stability and settlement mitigation measures, as required. The proximity of the existing bridge embankments to the detour embankments have been considered as the existing bridge will continue to be used for traffic flow during construction of the detour bridge.

6.6.1 Design Assumptions

The results of the current subsurface investigation and the results of the original investigation (DHO, 1967) have been considered together to determine the degree of consolidation of the clayey silt to clay deposit in the area of the proposed structures.



For the main bridge approach embankments within 20 m of the abutments, the analyses assume that all existing fill and organic soils (i.e. alluvium) will be removed from below the reconstructed embankment footprint (to the base of the pile cap only at the east approach) prior to construction of the widened/raised embankments. For the detour approaches, the surficial fill and alluvium (where encountered) have been left in place, for purposes of the analyses given the temporary nature of the structure.

The creek water level used for design is Elevation 246.4 m for the low water level case and Elevation 247.9 m for the high water level case. Groundwater levels beyond the creek are between Elevation 247.4 m and 249.2 m based on the water levels measured in the piezometers.

Both granular fill and rock fill have been considered for the construction of the approach embankments. Granular fill embankments are assumed to have side slopes at 2H:1V and rock fill slopes are typically formed at 1.25H:1V.

6.6.2 Stability

Analyses were performed on the critical sections of the proposed approach embankments for conditions during and after construction to assess the stability and liquefaction potential for the proposed embankment height, existing geometry and soil stratigraphy. The critical embankment sections at this site are the front slopes (towards the creek) and the side slopes, for both the permanent structure and the detour. The geometry of the proposed approach embankments, existing ground surface and existing creek bed included in the analyses are based on the information from the GA drawing provided by LEA.

6.6.2.1 Methodology

Limit equilibrium slope stability analyses were performed using the commercially available program GeoStudio 2007 (Version 7.13), 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 was 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 adopted for the design of embankment slopes under static conditions at the end of construction. 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 design embankment height and geometries. In general, circular slip surfaces were analysed in the design. In addition, effective stress (drained) analyses were conducted to assess long-term conditions applying a Factor of Safety of 1.3.

6.6.2.2 Parameter Selection

For the cohesionless deposits and granular fill, effective stress parameters were employed in the analysis assuming drained conditions and the parameters were estimated from empirical correlations using the results of the in situ SPT 'N'-values. The correlations proposed by Peck et al. (1974), Schmertmann (1975) and NAVFAC (1982) were employed and the results were tempered by engineering judgment based on precedent experience in similar soils.



For the cohesive fill and native soils, 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 in situ field vane tests and estimated from correlations with the SPT 'N'-values results and other laboratory test data. Where appropriate, Bjerrum's correction factor (1973) as a function of the plasticity index of the soil was employed to estimate the average mobilized undrained shear strength from the results of the in situ field vane tests.

The effective stress parameters for the cohesive strata (effective friction angle and cohesion) for evaluating long-term drained conditions were estimated using empirical correlations with plasticity index (PI) proposed by Mitchell (1993), Ladd et al. (1977) and Kulhawy and Mayne (1990).

Summarized below are the simplified stratigraphy and the associated strengths and unit weights employed for the different soil types in the proposed approach areas. The parameters used in the analysis for the clayey silt to clay deposits under the main and detour alignments are shown graphically on Figure 2. The slope stability analyses model geometry and stratigraphy are shown on Figures 3 and 4A for the critical sections identified above.

Soil Type	Unit Weight (kN/m³)	Undrained Shear Strength (kPa)	Angle of Internal Friction
New Granular* Fill	21	--	35°
Existing Sand, Sand and Gravel or Silty Sand Fill	20	--	30°
Existing Clayey Silt to Silty Clay Fill (including wood/organics)	16	30 kPa	30°
Silty Clay Alluvium	17	30 kPa	30°
Clayey Silt to Clay (under existing embankment)	19	30 kPa to Elev. 241.0 m 35 kPa below Elev. 241.0 m (See Figure 2)	30°
Clayey Silt to Clay (under detour embankment)	19	28.5 kPa to Elev. 241.0 m 35 kPa below Elev. 241.0 m (See Figure 2)	27°

*Granular 'B' Type I or II.

6.6.2.3 Results of Analysis

The results of the stability analyses are summarized below for the critical slope sections of the approach embankments. The minimum Factor of Safety is based on a deep-seated, global trial failure surface that would impact the operation of the roadway.

Structure	Maximum New Fill Height	Location	Factor of Safety*	Reference Figure
Main Bridge	0.4 m	Front slopes	> 1.3	Figure 3
		North side slope	> 1.3	n/a



Structure	Maximum New Fill Height	Location	Factor of Safety*	Reference Figure
Detour Bridge**	2.6 m	Front slopes	< 1.3	Figure 4A
		South side slope	> 1.3	n/a

*Assumes granular fill with side slopes of 2H:1V.

**Includes the equivalent footing pressure of 350 kPa.

The minimum Factors of Safety for deep seated failure surfaces for each section for the undrained case are presented above. The results of the analyses for the front slopes typically also indicate shallow surficial slip surfaces (in front of the abutments themselves) with Factor of Safety less than 1.3. However, these surfaces are not representative of true deep seated failure conditions. Selection of the minimum Factor of Safety involves engineering judgement on the results generated by the computer program and selection of a realistic failure surface that would impact the operation of the roadway. Proper erosion protection as described in Section 6.7 should alleviate these potential surficial slip surfaces.

Limit equilibrium analysis indicates a Factor of Safety greater than 1.3, for the undrained case, for all of the critical embankment front and side slope sections for the main bridge front and side slopes and the detour bridge side slopes. The drained analyses also give Factor of Safety greater than 1.3 for deep seated failure surfaces, however, erosion protection measures should be provided to alleviate potential near surface sloughs. Therefore, mitigation measures are not required at these locations.

For the detour bridge front slopes, measures to mitigate stability will be required to achieve a Factor of Safety of greater than 1.3 as discussed in Section 6.6.2.4 (see also Figure 4B).

Analysis was also carried out using rock fill and 1.25H:1V side slopes and the Factor of Safety was greater than the same case for granular fill. For the detour bridge front slopes, the use of rock fill did not increase the Factor of Safety to greater than 1.3 and mitigation is still required even if rock fill were to be used.

6.6.2.4 Mitigation of Stability (Detour Front Slopes)

As discussed above, the detour embankment front slopes Factor of Safety for stability is less than 1.3 for the combination of the removal of fill and placement of the granular pad and embankment fill plus the footing pressure from the detour bridge on the granular pad. To mitigate the front slope stability issues associated with the detour bridge and increase the Factor of Safety to greater than 1.3, a mitigation measure is required. Several options to mitigate stability of the detour front slopes have been considered and are as follows:

- Reduce the footing pressure from 350 kPa to 110 kPa;
- Lengthen the bridge from 30 m to 45 m (7.5 m equally on each side of the creek) and reduce footing pressure to 150 kPa;
- Utilize pile foundations instead of a spread footing on a granular pad; or
- Utilize a temporary culvert rather than a bridge.



The preferred and recommended option from a foundations perspective is to remove the existing fill and replace with granular material, construct a granular pad (2.5 m thick) and reduce the footing pressure to a maximum of 110 kPa, which will result in a Factor of Safety against deep seated failure in the undrained state greater than 1.3 (see Figure 4B). The other options are less desirable from a settlement and/or cost perspective and/or may not be feasible based on other non-foundations criteria. The advantages, disadvantages, relative costs and risks/consequences for the stability mitigation options for the detour front slopes are summarized and ranked in Table 3.

6.6.3 Liquefaction Potential and Seismic Analysis

As noted in Section 6.4.2, this site is located in Seismic Zone 1 with a $PHA < 0.08$. Further, the bridge structure is not a lifeline structure. As such, based on Section 4.4.4 of the CHBDC, the site is assigned a Seismic Performance of 1 and, therefore, in accordance with Section 4.4.5.1 of the CHBDC, liquefaction analysis is not required.

6.6.4 Settlement

Settlement of the approach embankments can be expected as a result of the loading from the new fills on the existing fill and 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.

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

6.6.4.1 Methodology

To estimate the magnitude of the expected settlements, analyses were carried out on the critical sections of the proposed approach embankments using the commercially available program Settle3D (Version 2.003) produced by Rocscience Inc. as well as hand/spreadsheet calculations. The rate of settlement of the cohesive foundation soils was assessed using Terzaghi's one-dimensional consolidation theory. The model geometry and stratigraphy are shown on Figures 3 and 4A (as used for the stability analyses).

Consolidation settlement of the clayey silt to clay deposit is expected. Time-dependent settlement of the cohesive fill materials may also occur if left in place below the embankments. Settlement of the cohesionless deposits is expected to be elastic, occurring during or shortly after construction.

6.6.4.2 Settlement Criteria

Based on information obtained from MTO Foundations, a settlement of 10 mm to 25 mm from the abutment to 25 m behind the abutment is considered acceptable within 10 years post-paving for the approach embankments of bridges. This criterion has been used for determining whether mitigation measures are required to limit post-construction settlement of the approach embankments for both the replacement bridge and the detour bridge.



6.6.4.3 Parameter Selection

The immediate compression of the existing fill and native cohesionless deposits 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 very soft to firm clayey silt to clay deposit was assessed using the results of the in situ field vane tests and the 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), Britto and Gunn (1987) and Koppula (1986).

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, the comparison to the earlier data (DHO 1967) and the following correlations relating mobilized undrained shear strength to preconsolidation pressure:

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

where: $s_{u(mob)}$ = average mobilized undrained shear strength (kPa)

σ_p' = preconsolidation stress (kPa)

and

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

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 (i.e. ranging from 0.88 to 1 for this site as Plasticity Indices range from about 10 percent to about 35 percent)

The following simplified stratigraphy, unit weights and deformation parameters have been employed in the settlement analysis of the proposed approach embankments. The thickness of the materials varies across the site and the maximum thicknesses have been used in the settlement estimates. Given the relatively low embankment heights, the full thickness of the clayey silt to clay deposit (up to about 27 m) will not be impacted by the fill loading and the estimated effective compressible thickness has been used in the analyses as noted below and has been confirmed by computer analysis. Further, the lower cohesionless soils are not anticipated to settle under the imposed loading due to the great depth that these strata are present below ground surface.

Deposit	Location	Maximum Thickness (m)	Unit Weight (kN/m ³)	Deformation Properties
New Granular* Fill	Main Bridge Detour Bridge	0.4 4.8**	21	See Section 6.6.4.4
Existing Sand, Sand and Gravel or Silty Sand Fill	Main Bridge Detour Bridge	3.0*** n/a	20	E' = 10 MPa
Existing Clayey Silt to Silty Clay Fill (including wood/organics)	Main Bridge Detour Bridge	5.7*** 2.8	16	E' = 5 MPa



Deposit	Location	Maximum Thickness (m)	Unit Weight (kN/m ³)	Deformation Properties
Silty Clay Alluvium	Main Bridge Detour Bridge	1.2*** 1.8	17	E' = 5 MPa
Silty Clay (crust)	Main Bridge Detour Bridge	2.6 2.3	19	E' = 10 MPa
Clayey Silt to Clay**** (under existing embankment)	Main Bridge-West Main Bridge-East Detour Bridge-West Detour Bridge-East	27 m but effective thickness is less than 10 m	19	(see below)

* Granular 'B' Type I or II.

** Includes 2.5 m thick Granular pad.

*** To be removed prior to embankment construction for the main alignment as per Section 6.6. For the detour, does not require removal except under granular pad).

**** Maximum thickness of main deposit that will be influenced by embankment loading (approximately two times the height of the embankment below the base of the fill) is estimated to be less than 10 m of the total 27 m thick stratum.

n/a Indicates deposit not encountered.

The following consolidation parameters were estimated for the clayey silt to clay deposit based on the results of laboratory consolidation tests performed on specimens of the clayey silt to clay obtained from Boreholes BR10-04 and BR10-07 (as shown below) and compared with values estimated from empirical correlations using the results of the in situ tests and laboratory index testing as described above.

Location	Borehole/ Sample No./Elevation	σ_{vo}' (kPa)	σ_p' (kPa)	OCR	e_o	C_r	C_c	C_y (cm ² /s)
West Abutment Main bridge	BR10-04/8 243.2 m	95	135	1.4	0.713	0.03	0.17	6.7×10^{-4}
East Abutment Detour bridge	BR10-07/8 243.2 m	65	130	2.0	0.775	0.02	0.12	4.7×10^{-3}

6.6.4.4 Settlement of Embankment Fill/Alluvium

Existing Fill/Alluvium – Detour Bridge Approach Embankments

In order to limit excavation depths for the detour adjacent to the existing approach embankments, the existing fill and alluvium, excluding that to be removed for construction of the 2.5 m thick granular pad, may be left in place under the detour embankments. Settlement of the existing fill will occur under the new embankment and granular pad loading. Fill and alluvium were not encountered in the west detour approach borehole (BR10-05) and 4.8 m of silty clay fill and alluvium were encountered in the east detour approach borehole (BR10-08). The estimated settlement of the fill and alluvium at the east detour approach is estimated to be less than about 10 mm under the up to 1.4 m of new embankment fill placement. This settlement is expected to occur during construction, although given the cohesive nature of the fill and alluvium, some settlement may occur over the life of the detour. Maintenance of the detour roadway during this time period may be required.



If dewatering is not carried out for the granular pad construction at the detour, then Granular 'B' Type II would have to be used for placement under water and some settlement of the Granular 'B' Type II below the water level could occur. However, given the limited thickness of placement below the water level, the settlement is expected to be minimal.

Existing Fill/Alluvium – Main Bridge Approach Embankments

Given the low grade raise (less than 0.4 m) and limited widening (about 1.5 m on each side) of the existing embankment, we recommend partial removal of the existing fill and alluvium. The partial removal should consist of sub-excavating the existing fill/alluvium to the underside of the pile cap and filter blanket (i.e. Elev. 245.5 m) from the abutment to the end of the approach slab. The estimated settlement of the remaining clayey silt fill and alluvium materials is expected to be less than 50 mm at the approach embankments. The majority of this settlement is expected to occur during construction; however, due to the cohesive nature of the fill and alluvium, some post embankment construction may occur, likely to be completed within about six months after construction. Although this quantity of settlement is greater than the settlement criteria (Section 6.6.4.2), it is considered acceptable for a secondary highway. Further, since the anticipated settlement will occur at a distance back from the abutment and approach slab, it should not result in a "bump" at the abutments.

Alternatively, to remove the uncertainty in the estimated magnitude of settlement associated with the variability in the thickness and characteristics of these materials, specifically, the presence of organic material, wood, etc., consideration could be given to removal of all the fill and alluvium soils below the main bridge embankments.

New Granular Fill

We recommend the use of granular fill for the new embankment construction at this site. Granular fill could consist of Granular 'B' Type I or Type II or Select Subgrade Material (SSM) in accordance with SP 110S13 (Aggregates). The settlement of the properly compacted granular fills is expected to be less than about 25 mm and will occur during construction.

If rock fill is used for main bridge approach embankment construction, the settlement of the less than 5 m thick of properly placed and compacted rock fill is expected to be less than about 25 mm and will occur during construction. Post-construction settlement of rock fill at this site is expected to be negligible.

6.6.4.5 Settlement of Foundation Soils

Based on the embankment geometry at the critical sections indicated above, settlement of the subsoils should be anticipated. Due to relatively low embankment heights at this site and given the level of over-consolidation of the clayey silt to clay deposit, it is anticipated that the settlement of the clayey silt to clay deposit will be relatively small and will occur during construction. As such, post-construction settlement is not anticipated and, therefore, mitigation of settlement is not required. The immediate settlement of clayey silt to clay subsoils, within the zone of influence of the embankments, is presented below for the critical sections (i.e. at/near the abutments and the approaches 20 m behind the abutments).



Location	Critical Section	Relevant Borehole	Estimated Immediate Settlement		
			New Granular Fill*	Existing Fill/Alluvium (partial removal only)	Clayey Silt to Clay
Main Bridge	West Approach	BR10-01	<25 mm	~ 50**	15
	West Abutment	BR10-04		n/a	25
	East Abutment	BR10-03		n/a	25
	East Approach	BR10-02		~ 50**	20
Detour Bridge	West Approach	BR10-05	~25 mm***	n/a	15
	West Abutment	BR10-06		n/a	35****
	East Abutment	BR10-07		n/a	30****
	East Approach	BR10-08		10	20

*Granular 'B' Type I, II or SSM.

**If the existing fill/alluvium is only partially removed below the embankment, some settlement could occur post-construction.

***Due to placement of Granular 'B' Type II below the water level without dewatering.

****Includes influence of shallow footing (equivalent mitigated pressure of 110 kPa, as per Section 6.6.2.4).

The total estimated immediate settlement of the fills and native subsoils at both the main and detour sites is estimated to be less than 70 mm. Post-construction settlement is not anticipated and, therefore, mitigation of settlement is not required.

6.7 Subgrade Preparation and Embankment Construction

For the main bridge approach embankments within 20 m of the abutments, partial removal of the fill and organic soils (i.e. alluvium) is recommended below the reconstructed embankment footprint prior to construction of the embankments. For the detour approaches, the surficial fill and alluvium (where encountered) may be left in place below the new fill. Topsoil and near surface organics should be removed below the footprint of the proposed main and detour bridge embankments. All softened/loosened soils should be stripped from below the approach embankments, prior to placement of new fill.

At the main east approach embankment (Boreholes BR10-02 and BR10-03), the existing fill and the alluvium should be removed to Elevation 245.5 m (the underside of the filter blanket) between STA 10+670 (abutment) and STA 10+677 and then slope upwards at 1H:1V to the frost penetration depth with a frost taper installed to meet the pavement subgrade in accordance with OPSD 3101.150 (Walls Abutment, Backfill). This will result in excavations up to 5.9 m deep below the existing ground surface.

At the main west approach embankment (Boreholes BR10-01 and BR10-04), the base of the existing fill and the alluvium is at Elevation 247.4 m at STA 10+650 (abutment) and Elevation 247.8 m at STA 10+690 (20 m behind the abutment). Given an underside of pile cap and filter blanket at Elevation 245.5 m, the native soils require removal for pile cap construction. Beyond the abutment backfilling zone, the base of the sub-excavation can increase at 1H:1V as per the OPSD. This will result in excavations up to 5.1 m deep below the existing ground surface.



The fill and alluvium below the detour embankments, where present, are only required to be sub-excavated from the area below the granular pad to Elevation 246.6 m. This will result in excavations up to 3.0 m deep below the existing ground surface.

Granular fill materials and placement should be carried out in accordance with the requirements as outlined in SP 206S03 (Earth Excavation, Grading) above the water level. If filling below the water level is carried out at the detour for construction of the footing pads, then Granular 'B' Type II should be used to backfill the excavation to Elevation 241.7 m, as discussed in Section 6.2 and shown on Figure 1, in accordance with Ontario Provincial Standard Specification (OPSS) 209 (Embankments Over Swamps and Compressible Soils). All granular fill should be placed in lifts with loose thickness not exceeding 300 mm and compacted to at least 95 percent of the standard Proctor maximum dry density. Side slopes for granular fill embankments should be no steeper than 2H:1V. If rock fill is used for embankment construction, side slopes can be constructed at 1.25H:1V. Rock fill could be used for granular pad construction, however, the upper surface should be "chinked" and a minimum of 300 mm of Granular 'B' Type II and 150 mm of Granular 'A' should be placed below the underside of the footings.

Prior to placement of the granular subbase and base courses, the final lift of embankment fill should be compacted to 100 percent of the Standard Proctor maximum dry density. Inspection and field density testing should be carried out by qualified personnel during fill placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

In order to minimize differential settlement between the existing embankment slopes and the newly placed embankment fill, the new fill should be keyed into the existing embankment side slope per the requirements of OPSD 208.010 (Benching of Earth Slopes).

The abutment front slopes and side slopes adjacent to the creek require erosion protection in accordance with SP 511S01 (Rip Rap, Gravel Sheetting). Erosion protection should be placed on the slopes to at least 0.5 m above the design high water level. Erosion protection could consist of a minimum 0.6 m thick layer of rip rap (300 mm diameter), rock protection or concrete slope paving. The designer should address the potential for scour below the pile caps in the design of the bridge foundations.

To reduce surface water erosion on the embankment side slopes, topsoil and seeding should be carried out as soon as possible after construction where earth fill is used. If this slope protection is not in place before winter, then alternate protection measures, such as covering the slope with straw or gravel sheetting to prevent erosion, will be required to reduce the potential for remedial works on the side slopes in the spring prior to topsoil dressing and seeding.

6.8 Design and Construction Considerations

6.8.1 Excavations

At the west abutment, the excavation for pile cap construction will extend to Elevation 246.0 m. The excavations for sub-excavation of the fill and pile cap construction at the east abutment will extend to about 0.5 m below the base of the proposed pile cap, to Elevation 245.5 m. A cofferdam will likely be required at the abutments, due to the depth of sub-excavation below the groundwater table (i.e. up to about 3.7 m) and the proximity to the creek.



For construction of the detour granular pads, the excavations for the abutments will extend to Elevation 246.6 m and can be carried out in open cut. Shoring will likely not be required at these locations as the fill for the granular pad can be placed below the water level.

Excavations can be carried out in open cut except where temporary roadway protection will be required between the existing and new embankments and at the main bridge abutments as discussed above. Open cut slopes within the fill materials should be maintained at no steeper than 2H:1V both above and below the water level.

All excavations must be carried out in accordance with the latest edition of the Ontario Occupational Health and Safety Act (OHSa) and Regulations for Construction Projects and good construction practice. The existing fill materials and the native soils should be classified as Type 3 soil, according to the OHSa.

6.8.2 Temporary Shoring

A temporary excavation support system, if required, should be designed and constructed in accordance with OPSS 539 (Temporary Protection Systems). The lateral movement of the temporary shoring system should meet Performance Level 2, as specified in OPSS 539 (Temporary Protection Systems).

Temporary roadway protection may be required between the existing bridge and the excavation for the granular pads for the detour bridge. Further, temporary roadway protection will be required between the detour abutments/approach embankments and the new abutment pile caps and for pile installation and abutment/approach embankment construction.

Given the proximity to the creek and the depth of excavation required for the main bridge abutments, particularly at the east abutment, consideration should be given to the use of temporary shoring such as a sheet-pile cut-off wall or other cofferdam type construction. Above the water level, other types of temporary shoring could be considered including braced excavations such as sheet piling, soldier pile and lagging or tied-back walls.

6.8.3 Groundwater and Surface Water Control

Although perched water was not encountered within the fill during the latest investigation, it is possible that water is perched within the fill materials. The Contractor should therefore anticipate perched water within the embankment fill sub-excavation for the main bridge approach embankments.

Surface water should be directed away from the excavation at all times.

The excavation for the abutments (main and detour) will be located adjacent to Brule Creek. In order to construct the pile caps and detour granular pads, groundwater inflow should be expected and controlled dewatering within an open cut excavation or temporary shoring. Shoring, if utilized, should be advanced to an appropriate depth to control groundwater inflow from the creek. The excavations will be advanced through mainly cohesive fills, wood and organics at some locations and through soft to stiff native cohesive soils.



6.8.4 Filter Blanket

For the main bridge, given the proximity of the pile tips to the artesian groundwater for the recommended friction pile alternative or for the case of piles driven to bedrock, we recommend that a drainage/filter blanket consisting of a minimum 0.5 m thick layer of concrete fine aggregate (OPSS 1002, Aggregates, Concrete) be placed below the underside of the pile caps encasing all the piles. At the west abutment, this will extend to Elevation 245.5 m, or to 0.5 m below the underside of pile cap. At the east abutment, unsuitable fill must be removed to Elevation 244.2 m and therefore the filter blanket material should be placed from this elevation to the underside of pile cap, about 1.8 m thick. The concrete fine aggregate layer should extend a minimum of 0.5 m horizontally beyond each of the pile caps. Further, the excavation at the front of the abutment (towards the creek) should be backfilled with free draining material extending at least 0.5 m horizontally from the front face of the abutment.

6.8.5 Obstructions

As part of the design and construction of the new abutment foundations, careful consideration should be given to the location of the existing piles relative to the new construction of the detour footings, temporary shoring and cofferdams and replacement bridge piles. Specifically, the designer should check that the new piles (batter and orientation) and temporary shoring do not interfere with the existing piles. This should be checked to the full extent of the pile/shoring length.

The existing timber piles extend to depths between 12 m and 15 m below the ground surface, which is above the elevation where artesian pressures were encountered during the field investigation. However, if the timber piles are removed, there would be only about half the thickness of the silty clay stratum resisting the artesian pressure at the base of the extraction hole and, therefore, there would be a risk of creating a pathway for artesian groundwater and for potential ground loss. Also, given that the boreholes remained open after completion of drilling, the cohesive soils are not likely to fully squeeze-in and seal the hole. Further, backfilling the pile holes, where removed within the creek channel, may not be feasible or practical below the water level. We recommend that the existing timber piles be left in place and “cut off” at the creek bed level and not be pulled out nor partially removed.

6.8.6 Existing Structure Monitoring

Given the age of the existing structure, the close proximity of construction activities for the detour abutments and the requirement for the existing structure to remain in operation during construction of the detour, it is recommended that the existing bridge be monitored for settlement and lateral movement while in operation during the detour construction, especially during installation of temporary roadway protection (if required), excavation for the granular pad and new abutments and during pile driving. This could be carried out using survey points (lateral and vertical deformation) and/or settlement points. We understand that LEA has an NSSP for this purpose and it should be included in the Contract Documents.

7.0 CLOSURE

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



Report Signature Page

GOLDER ASSOCIATES LTD.

David Muldowney, B.Eng.
Geotechnical Group



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



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

DAM/SEMC/JMAC/lb

n:\active\2009\1190 sudbury\1191\09-1191-0022 lea brule and wicklows\7000 reporting\brule\final\09-1191-0022 rpt 11feb22 brule creek final fdr.docx



REFERENCES

- Azzouz, A.S., Krizek, R.J., and Corotis, R.B., 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, Second 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., 1964. Lateral Resistance of Piles in Cohesive Soils; Journal for Soil Mechanics and Foundation Engineering., ASCE, Vol. 90, SM2, pp. 27-64.
- Canadian Geotechnical Society, 2006. Canadian Foundation Engineering Manual, Fourth Edition.
- Canadian Geotechnical Society, 1992. Canadian Foundation Engineering Manual 1992, Third Edition.
- Canadian Highway Bridge Design Code (CHBDC) and Commentary on CAN/CSA S6-06, 2006. CSA Special Publication, S6.1-06. Canadian Standard Association.
- Department of Highways Ontario, 1967. Foundation Investigation Report for the Proposed New Brule Creek Bridge at Hwy. #574, Site 39E-57, WP 307-66, District No. 16 (Cochrane, Ont.), Department of Highways Ontario, June 22, 1967, including "Bore Hole Locations & Soil Strata" Drawing 67-F-39A, dated July 5, 1967, GEOCRE. No. 42H-001.
- Koppula, S.D., 1986. Discussion: Statistical Estimation of Compression Index, Geotechnical Testing Journal, ASTM, Vol. 4, No. 2, pp. 68-73.
- 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.
- Ladd, C.C., and Foott, R., Ishihara, K., Schlosser, F., and Poulos, H.G., 1977. Stress-deformation and strength characteristics. Proceedings of the 9th International Conference on Soil Mechanics and Foundation Engineering, Tokyo, Vol. 2, pp.421-494.
- Mesri, G., 1975. Discussion on new design procedure for stability of soft clays. ASCE Journal of the Geotechnical Engineering Division, Vol. 101, GT4, pp. 409-412.
- Ministry of Transportation, Ontario, 2008. Structural Manual. Provincial Highways Management Division, Highway Standards Branch, Bridge Office.
- Ministry of Transportation, Ontario, 2001. Ontario Traffic Manual, Book 7. Temporary Conditions, Field Edition.
- Mitchell, J.K., 1993. Fundamentals of Soil Behaviour. 2nd Edition, John Wiley and Sons, Inc., New York.



NAVFAC Design Manual DM 7.2. Soil Mechanics, Foundation and Earth Structures. U.S. Navy, 1982 (Revalidated 1986). Alexandria, Virginia.

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

Peck, R.B., Hanson, W.E., and Thornburn, T.H., 1974. Foundation Engineering, Second 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.

ASTM International

ASTM D1586 Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils

ASTM D1587 Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes

ASTM D2573 Standard Test Method for Field Vane Shear Test in Cohesive Soil

Commercial Software

LPile (Version 5.0) by Ensoft Inc.

GeoStudio (Version 7.13) by Geo-Slope International Ltd.

Settle 3D (Version 2.003) by Rocscience Inc.

Ministry of Transportation Ontario Special Provisions

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

SP 206S03 Earth Excavation, Grading; Rock Embankment

SP 511S01 Rip Rap; Rock Protection

Ontario Provincial Standard Drawings

OPSD 208.010 Benching of Earth Slopes

OPSD 3090.100 Foundation Frost Depths for Northern Ontario

OPSD 3101.150 Walls Abutment, Backfill Minimum Granular Requirement

OPSD 3101.200 Walls Abutment, Backfill Rock

OPSD 3121.150 Walls Retaining, Backfill Minimum Granular Requirement



Ontario Provincial Standard Specifications

OPSS 209	Construction Specification for Embankments Over Swamps and Compressible Soils
OPSS 501	Construction Specification for Compacting
OPSS 539	Construction Specification for Temporary Protection Systems
OPSS 902	Construction Specification for Excavating and Backfilling Structures
OPSS 903	Construction Specification for Deep Foundations
OPSS 1002	Material Specification for Aggregates - Concrete

Ontario Provincial Standard Structural Drawings

SS 103-11	Pile Driving Control, 2002
-----------	----------------------------

Ontario Water Resources Act

Ontario Regulation 372/97	Amendment to Ontario Regulation 903
Ontario Regulation 903/90	Wells



FOUNDATION REPORT, BRULE CREEK BRIDGE REPLACEMENT
SITE 39E-057, HIGHWAY 652, GWP 133-88-00, AGREEMENT NO. 5008-E-0037

Table 1: Evaluation of Foundation Alternatives - Replacement Bridge

Foundation Type	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Piles driven to bedrock at 35.5 m to 42 m depth below the underside of pile cap	1	<ul style="list-style-type: none"> ■ Straightforward construction. ■ Allows for integral abutment design. ■ Avoids risk of not achieving sufficient resistance. ■ Higher axial resistance than for shallow foundations. ■ Alternative pile sections (310X110 or 310X132) available to satisfy both axial resistance and driving into/through boulder layer overlying bedrock, although factored ULS reduced to 90%. 	<ul style="list-style-type: none"> ■ Requires filter sand blanket under pile cap to prevent ground loss and embankment instability associated with driving piles through artesian layer and seating on bedrock. ■ Heavier pile section recommended to avoid using flange reinforcement and/or rock points to minimize gap between pile and silty clay/clay deposit. ■ Although still feasible, piles longer than 35.5 m could be problematic in terms of driving and seating process. ■ Dewatered excavation (cofferdam) required adjacent to the creek to allow pile cap construction in-the-dry. 	<ul style="list-style-type: none"> ■ Relative costs lower than caissons. 	<ul style="list-style-type: none"> ■ High risk of artesian groundwater emanating to the ground surface during pile driving, including impacting embankment stability.
Friction piles terminated above Elevation 224 m	2	<ul style="list-style-type: none"> ■ Straightforward construction. ■ Allows for integral abutment design. ■ Avoids potential problems associated with artesian groundwater conditions below Elevation 220 m. 	<ul style="list-style-type: none"> ■ Lower geotechnical resistance with friction piles and therefore more piles may be required and potential inability to use integral abutment design. ■ Risk of not achieving design resistance above termination depth. ■ Possible risk of problems if artesian deposit is accidentally penetrated, such as pile settlement or ground loss. ■ Dewatered excavation (cofferdam) required adjacent to the creek to allow pile cap construction in the dry. 	<ul style="list-style-type: none"> ■ Potential higher cost if extra piles are required compared to piles driven to bedrock. 	<ul style="list-style-type: none"> ■ Low risk of problems associated with artesian groundwater during pile driving.
Shallow foundations on native soil or granular pad	3	<ul style="list-style-type: none"> ■ Straightforward construction. ■ Removes potential complications associated with pile installation in terms of not achieving resistance or artesian groundwater. 	<ul style="list-style-type: none"> ■ Lower geotechnical axial resistance. ■ Dewatered excavation (cofferdam) required adjacent to the creek to allow footing construction in-the-dry. ■ Settlement of subsoils, as a result of replacement fill and minor grade raise, will occur. ■ Front slope may not be stable with extra load from footings. 	<ul style="list-style-type: none"> ■ Typically lower cost than deep foundations. 	<ul style="list-style-type: none"> ■ Low risk of complications associated with cofferdam construction. ■ Risk of settlement of subsoils due to embankment construction



**FOUNDATION REPORT, BRULE CREEK BRIDGE REPLACEMENT
SITE 39E-057, HIGHWAY 652, GWP 133-88-00, AGREEMENT NO. 5008-E-0037**

Table 2: Evaluation of Foundation Alternatives - Detour Bridge

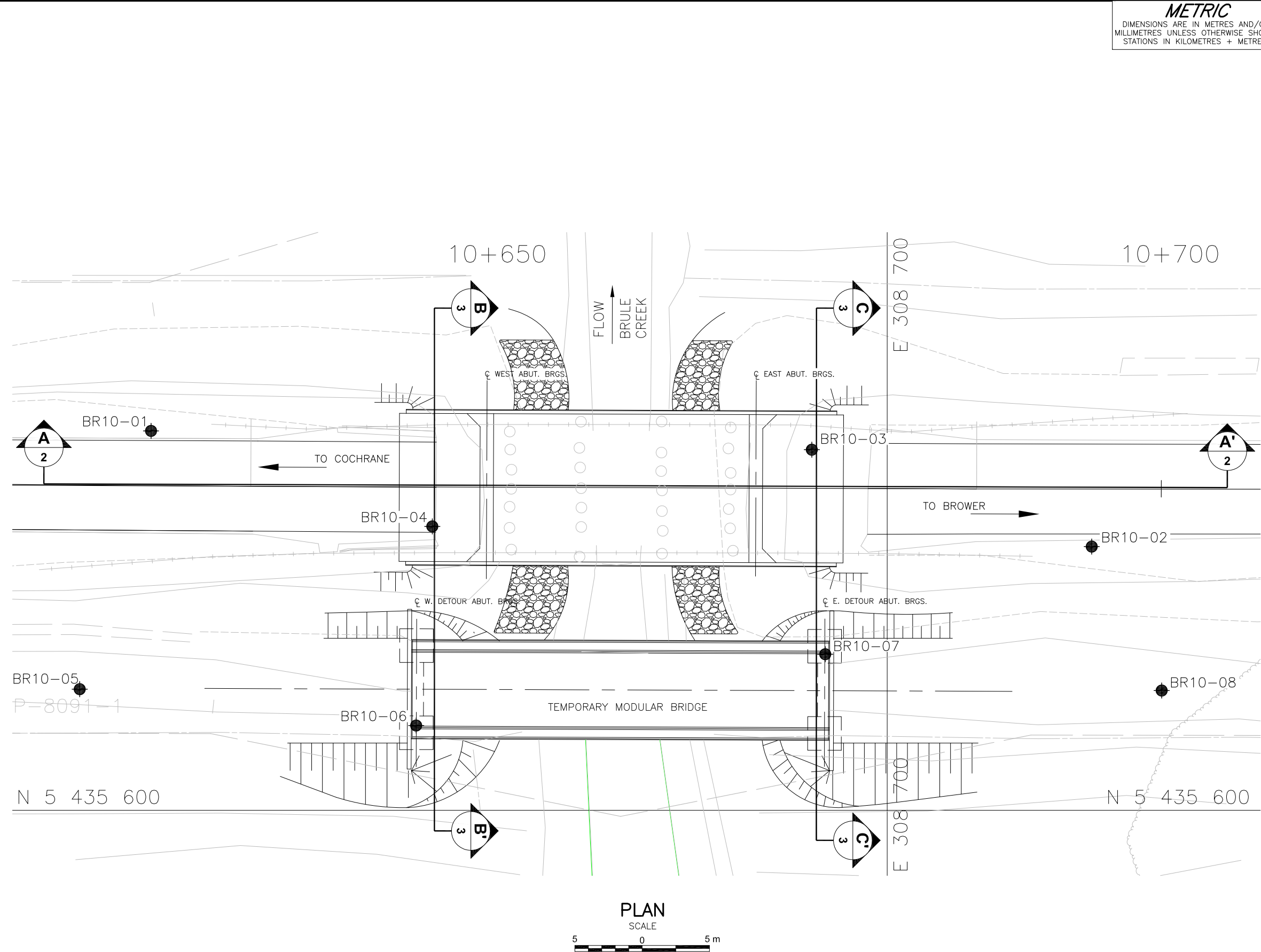
Foundation Type	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Shallow foundations on granular pad	1	<ul style="list-style-type: none"> ■ Straightforward construction. ■ Higher axial resistance than spread footings directly on native soils. ■ Granular pad below the water can be placed without dewatering to eliminate cofferdam requirement. 	<ul style="list-style-type: none"> ■ Settlement of subsoils will occur as a result of granular pad and embankment construction but will occur rapidly. ■ Some settlement of the granular pad is expected if the fill is placed sub-aqueously. ■ Stability may be decreased due to applied footing pressure immediately after pad/embankment construction. ■ Does not allow for integral abutment design. 	<ul style="list-style-type: none"> ■ Typically lower cost than deep foundations. ■ Cost of granular fill must be taken into account 	<ul style="list-style-type: none"> ■ Some risk of settlement of the granular pad if sub-aqueous filling is carried out (without compaction). ■ Settlement of subsoils will occur under the abutment as a result of pad and embankment construction.
Deep Foundations	2	<ul style="list-style-type: none"> ■ Straightforward construction. ■ Higher axial resistance than for shallow foundations. ■ Allows for integral abutment design if a pile foundation is used, however, this is not required for a temporary structure. 	<ul style="list-style-type: none"> ■ Possible risk of problems if artesian deposit is accidentally penetrated, such as pile settlement or ground loss. ■ Dewatered excavation (cofferdam) required adjacent to the creek to allow pile cap construction in-the-dry. 	<ul style="list-style-type: none"> ■ Relative costs much higher than shallow foundations. 	<ul style="list-style-type: none"> ■ High risk of problems associated with artesian groundwater during pile driving, including impacting embankment stability.
Shallow foundations on native soil	3	<ul style="list-style-type: none"> ■ Straightforward construction. 	<ul style="list-style-type: none"> ■ Dewatered excavation (cofferdam) required adjacent to the creek to allow footing construction in-the-dry. ■ Lower axial resistance compared to spread footings on a granular pad. ■ Settlement of subsoils will occur as a result of embankment construction but will occur rapidly. ■ Does not allow for integral abutment design. 	<ul style="list-style-type: none"> ■ Higher cost compared to footings on a granular pad due to need for cofferdam. 	<ul style="list-style-type: none"> ■ Low risk of complications associated with cofferdam construction. ■ Settlement of subsoils will occur under the abutment as a result of embankment construction.



FOUNDATION REPORT, BRULE CREEK BRIDGE REPLACEMENT
SITE 39E-057, HIGHWAY 652, GWP 133-88-00, AGREEMENT NO. 5008-E-0037

Table 3: Evaluation of Stability Mitigation Alternatives – Detour Front Slopes

Mitigation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Reduce Footing Pressure (in combination with removal of fill and construction of a granular pad)	1	<ul style="list-style-type: none"> ■ Lowers the driving force and increases the Factor of Safety. ■ Most straightforward if design allows. 	<ul style="list-style-type: none"> ■ Design has to be able to accommodate lower (110 kPa) footing pressure as an option. 	<ul style="list-style-type: none"> ■ Low impact compared to proposed design. 	<ul style="list-style-type: none"> ■ Low risk.
Lengthen Bridge (places foundation loads further away from creek on stronger foundation soils conditions)	2	<ul style="list-style-type: none"> ■ Decreases overall front slope angle and increases the Factor of Safety. 	<ul style="list-style-type: none"> ■ Other non-foundations criteria may eliminate this as an option such as road grade elevation needs. ■ Footing pressure may still need to be decreased, but not as low as 110 kPa. 	<ul style="list-style-type: none"> ■ Higher cost of longer bridge. 	<ul style="list-style-type: none"> ■ Low risk.
Piled Foundation	3	<ul style="list-style-type: none"> ■ Eliminate footing load from driving force, slope is stable. ■ Accommodates higher axial resistances from the detour structure. 	<ul style="list-style-type: none"> ■ Piling may impact existing bridge foundations which are in poor condition. 	<ul style="list-style-type: none"> ■ Higher cost of deep foundations. 	<ul style="list-style-type: none"> ■ High risk of impacting existing bridge foundations.
Temporary Culvert	4	<ul style="list-style-type: none"> ■ Eliminates stability as a concern. 	<ul style="list-style-type: none"> ■ Settlement of the subsoils will become a concern due to embankment construction on creek banks. ■ Not sufficient soils information to permit design and satisfy contractual issues. ■ Excavation in creek channel could be problematic (dewatering). 	<ul style="list-style-type: none"> ■ Complete re-design would be required. ■ Costs vary depending on culvert type, etc. 	<ul style="list-style-type: none"> ■ High risk of post-construction settlement; however, this may be tolerated since this is a temporary structure.

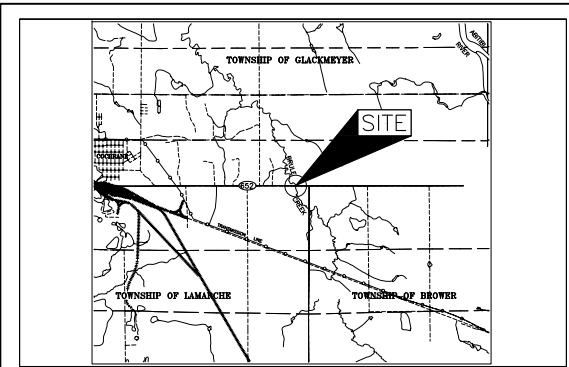


METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No. WP No.133-88-00		
BRULE CREEK BRIDGE HIGHWAY 652		
BOREHOLE LOCATIONS		SHEET



Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA



KEY PLAN

LEGEND	
	Borehole

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
BR10-01	250.8	5435628.2	308645.3
BR10-02	251.8	5435619.6	308715.2
BR10-03	251.4	5435626.8	308694.4
BR10-04	251.1	5435621.1	308666.2
BR10-05	249.9	5435609.0	308640.0
BR10-06	248.5	5435606.3	308665.0
BR10-07	249.6	5435611.6	308695.4
BR10-08	250.0	5435608.9	308720.4

NOTES

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

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

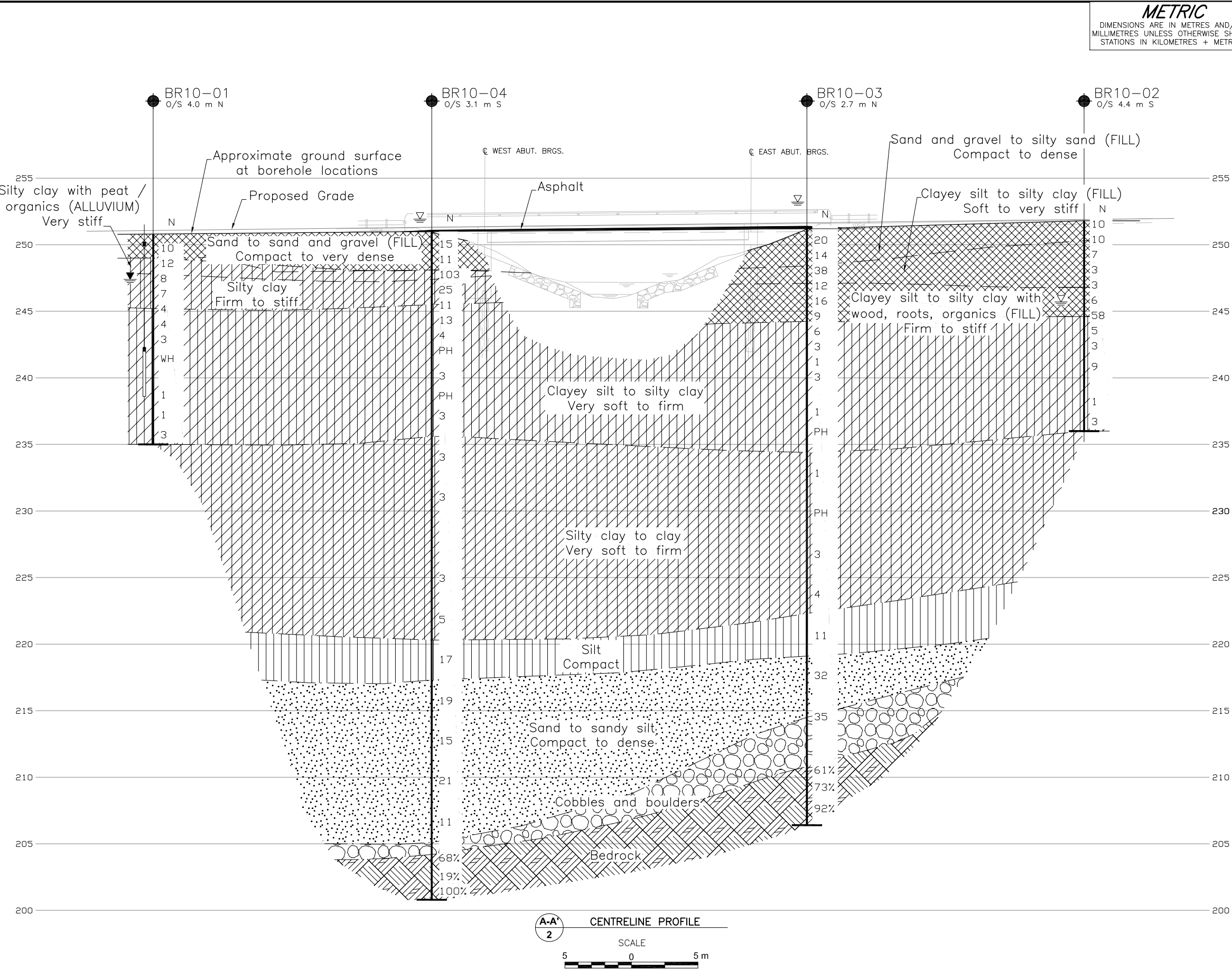
The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS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

REFERENCE

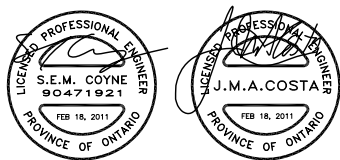
Base plans provided in digital format by LEA Consulting Ltd., drawing file BRULE-GA.dwg and BRULE-DET-GA.dwg, dated APR, 2010, received May 20, 2010.



NO.	DATE	BY	REVISION
Geocres No. 42H-42			
HWY. 652	PROJECT NO. 0911910022		DIST.
SUBM'D. DAM	CHKD. SEMC	DATE: FEB 2011	SITE: 39E-057
DRAWN: JJJ	CHKD.	APPD. JMAC	DWG. 1



A-A' 2 CENTRELINE PROFILE
SCALE
5 0 5 m



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

CONT No.
WP No.133-88-00

BRULE CREEK BRIDGE
HIGHWAY 652
SOIL STRATA

Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA

SHEET

LEGEND

Borehole
 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 AUG. 12, 2010
 WL upon completion of drilling

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
BR10-01	250.8	5435628.2	308645.3
BR10-02	251.8	5435619.6	308715.2
BR10-03	251.4	5435626.8	308694.4
BR10-04	251.1	5435621.1	308666.2
BR10-05	249.9	5435609.0	308640.0
BR10-06	248.5	5435606.3	308665.0
BR10-07	249.6	5435611.6	308695.4
BR10-08	250.0	5435608.9	308720.4

NOTES

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

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

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

REFERENCE

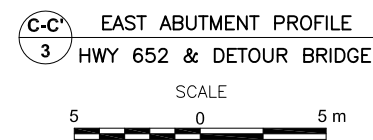
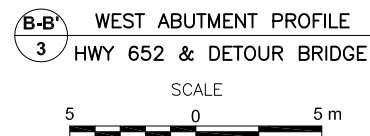
Base plans provided in digital format by LEA Consulting Ltd., drawing file BRULE-GA.dwg and BRULE-DET-GA.dwg, dated APR, 2010, received May 20, 2010.

NO.	DATE	BY	REVISION
1			

Geocres No. 42H-42

HWY. 652	PROJECT NO. 0911910022	DIST.
SUBM'D. DAM	CHKD. SEMC	DATE: FEB 2011
DRAWN: J.J.L.	CHKD.	APPD. J.M.A.C.
		SITE: 39E-057
		DWG. 2


SHEET








BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
BR10-01	250.8	5435628.2	308645.3
BR10-02	251.8	5435619.6	308715.2
BR10-03	251.4	5435626.8	308694.4
BR10-04	251.1	5435621.1	308666.2
BR10-05	249.9	5435609.0	308640.0
BR10-06	248.5	5435606.3	308665.0
BR10-07	249.6	5435611.6	308695.4
BR10-08	250.0	5435608.9	308720.4

NO.	DATE	BY	REVISION	
Geocres No. 42H-42				
HWY. 652		PROJECT NO. 0911910022		DIST.
SUBM'D. DAM	CHKD. SEMC	DATE: FEB 2011		SITE: 39E-057
DRAWN: JJJ	CHKD.	APPD. JMJC		DWG. 3



CONT No. WP No.133-88-00	
BRULE CREEK TEMPORARY MODULAR BRIDGE HIGHWAY 652	SHEET
BOREHOLE LOCATIONS AND SOIL STRATA	

LEGEND

- | | |
|---|--|
|  | Borehole |
|  | Seal |
|  | Piezometer |
| N | Standard Penetration Test Value |
| 16 | Blows/0.3m unless otherwise stated
(Std. Pen. Test, 475 j/blow) |
|  | WL in piezometer, measured on AUG. 12, 2010 |
|  | WL upon completion of drilling |

No.	ELEVATION	NORTHING	EASTING
BR10-01	250.8	5435628.2	308645.3
BR10-02	251.8	5435619.6	308715.2
BR10-03	251.4	5435626.8	308694.4
BR10-04	251.1	5435621.1	308666.2
BR10-05	249.9	5435609.0	308640.0
BR10-06	248.5	5435606.3	308665.0
BR10-07	249.6	5435611.6	308695.4
BR10-08	250.0	5435608.9	308720.4

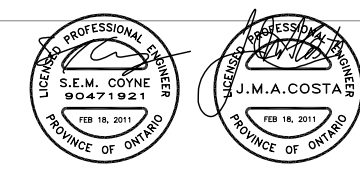
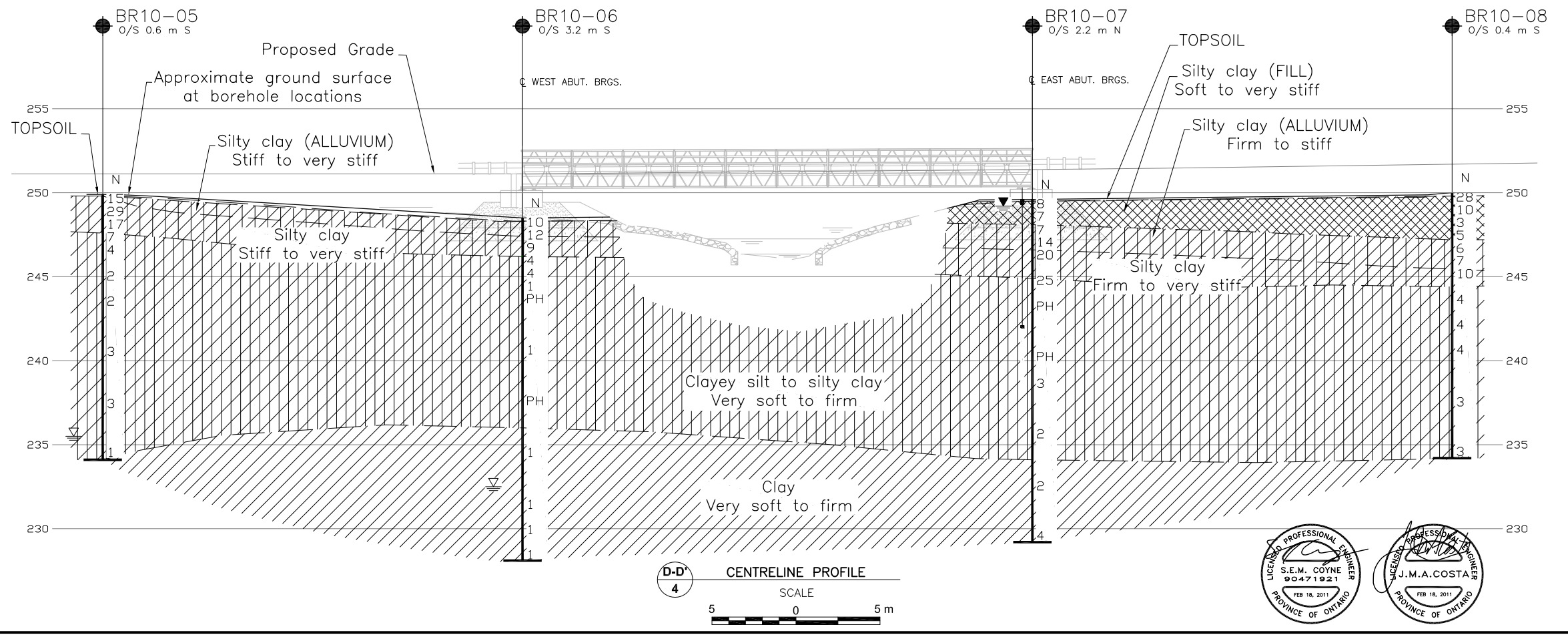
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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

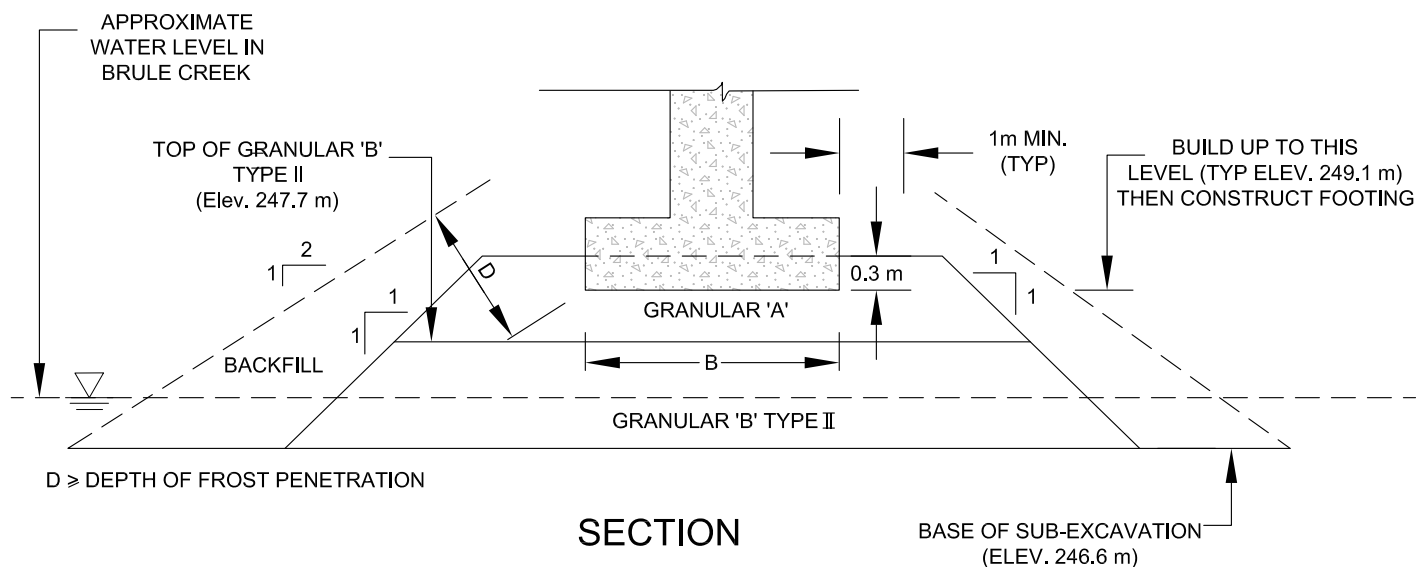
The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS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

Base plans provided in digital format by LEA Consulting Ltd., drawing file BRULE-GA.dwg and BRULE-DET-GA.dwg, dated APR, 2010, received May 20, 2010

NO.	DATE	BY	REVISION				
Geocres No. 42H-42							
HWY. 652			PROJECT NO. 0911910022		DIST.		
SUBM'D. DAM		CHKD. SEMC	DATE: FEB 2011		SITE: 39E-057		
DRAWN: JJL		CHKD.	APPD. JMAC		DWG. 4		



PLOT DATE: February 2, 2011
FILENAME: \\sud1-s-cadgis\Golder\CAD\Projects\09-1191-0022 Brule Wicklow Bridges\Brule River Bridge\09-1191-0022FIG1.dwg



CONSTRUCTION SEQUENCE:

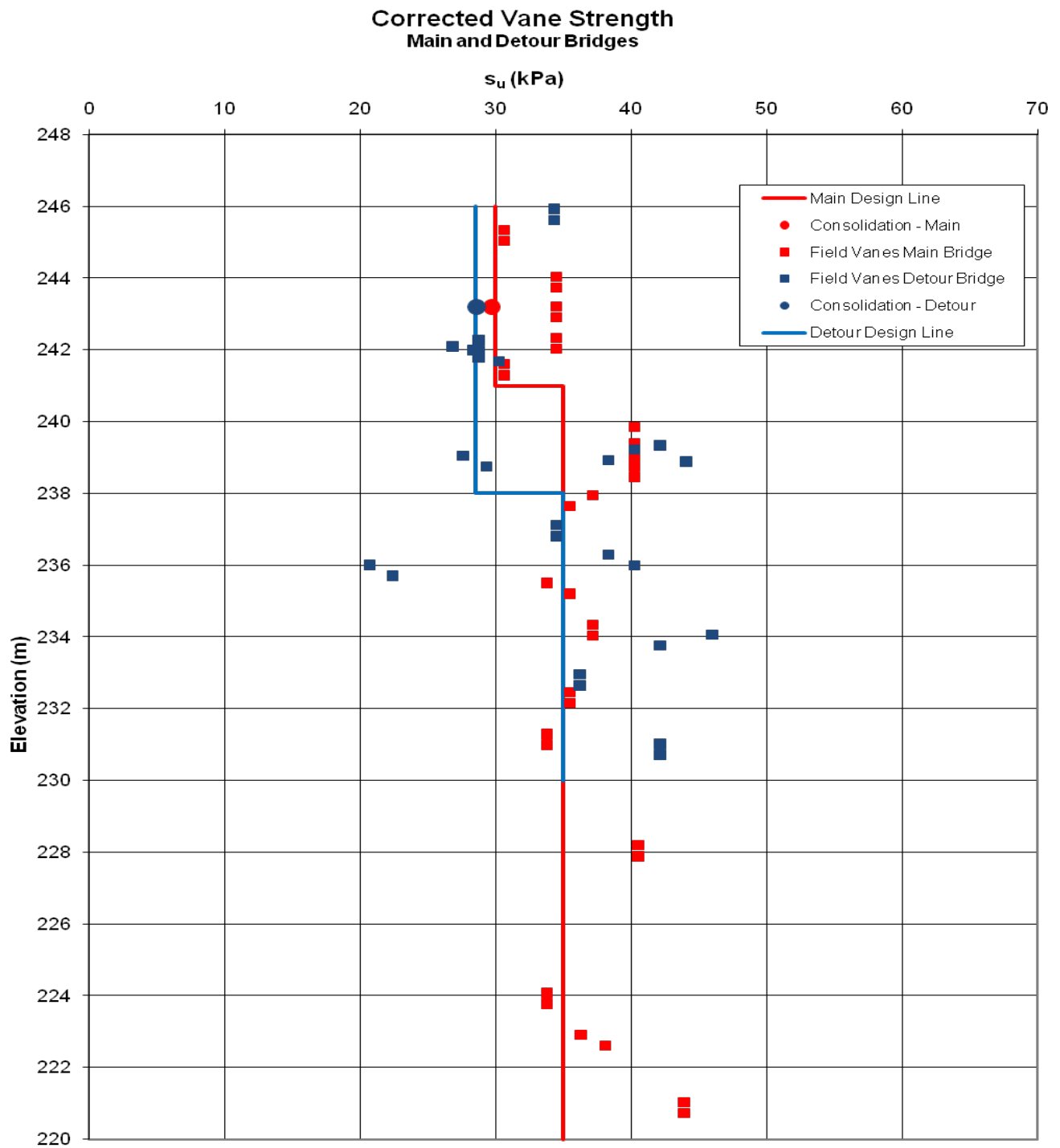
1. REFER TO ACCOMPANYING FOUNDATION DESIGN REPORT, SECTION 6.2.
2. REMOVE SUBSOILS UNDER FOOTPRINT OF COMPACTED GRANULAR CORE TO ELEVATION SPECIFIED.
3. PLACE GRANULAR 'B' TYPE II BELOW THE GROUND WATER LEVEL ELEVATION 247.7 m IN ACCORDANCE WITH OPSS 209.
4. COMPACT GRANULAR 'B' TYPE II IN ACCORDANCE WITH SP105S10.
5. PLACE AND COMPACT GRANULAR 'A' TO UNDER SIDE OF FOOTING LEVEL IN ACCORDANCE WITH SP105S10.
6. CONSTRUCT CONCRETE FOOTING.
7. PLACE REMAINDER OF GRANULAR 'A' AND BACKFILL AS REQUIRED.
8. SOURCE M.T.C 1982.

PROJECT		GWP 133-88-00 BRULE CREEK DETOUR BRIDGE	
TITLE		TYPICAL DETOUR ABUTMENT ON COMPACTED FILL CORE	
PROJECT No.		09-1191-0022	FILE No. 09-1191-0022FIG1.dwg
DESIGN			SCALE NTS REV.
CAD	JJL	FEB 2011	FIGURE No.
CHECK	SEMC	FEB 2011	1
REVIEW	JMAC	FEB 2011	



UNDRAINED SHEAR STRENGTH VS. ELEVATION

FIGURE 2



Date: February 2011
Project: 09-1191-0022-1

Golder Associates

Drawn: AMW
Checked: SEMC

Stability Analysis Main Bridge Front Slope (East Side)

FIGURE 3

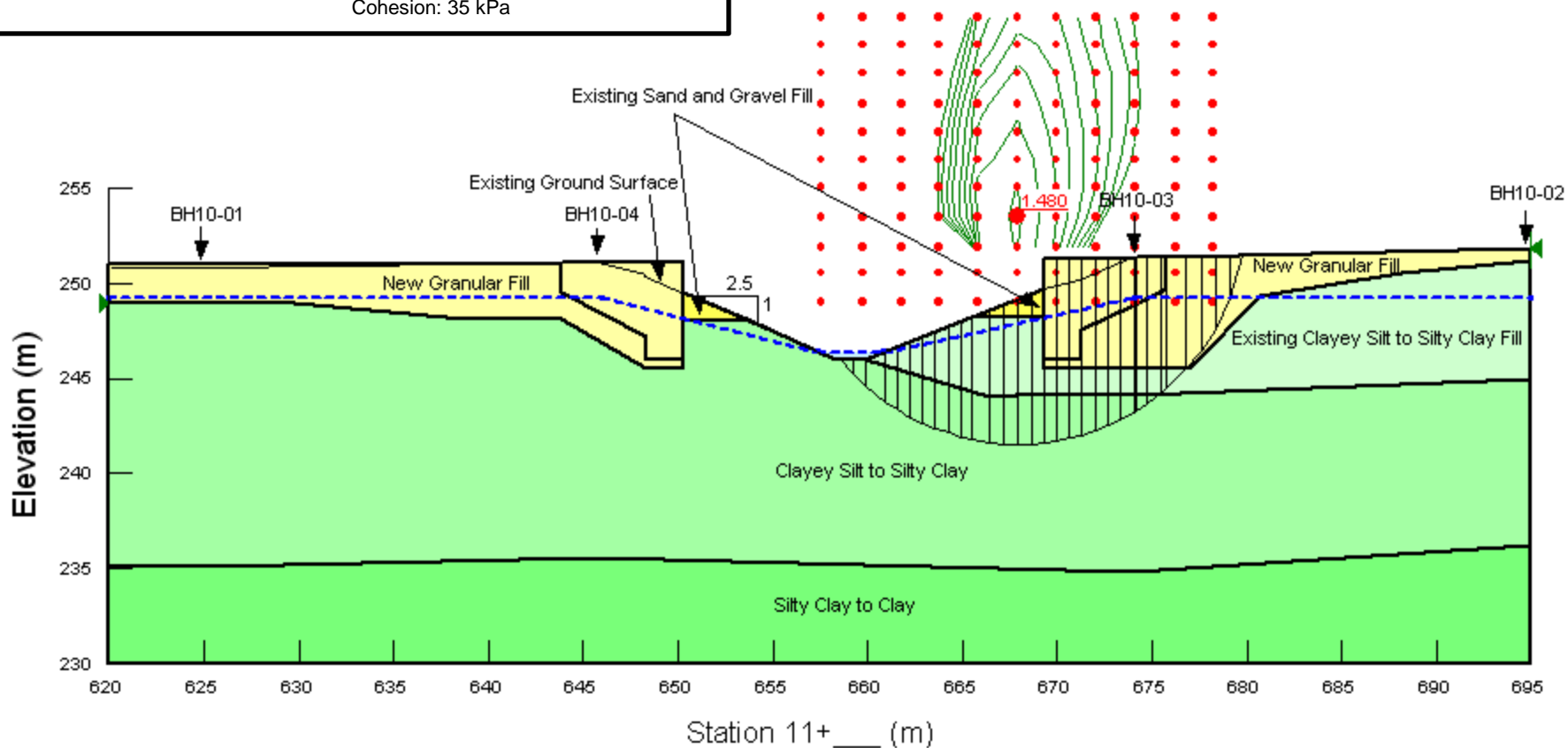
New Granular Fill
Unit Weight: 21 kN/m³
Phi: 35°

Existing Clayey Silt to Silty Clay Fill
Unit Weight: 16 kN/m³
Cohesion: 30 kPa

Existing Sand and Gravel Fill
Unit Weight: 20 kN/m³
Phi: 30°

Clayey Silt to Silty Clay
Unit Weight: 19 kN/m³
Cohesion: 30 kPa

Silty Clay to Clay
Unit Weight: 19 kN/m³
Cohesion: 35 kPa



DATE: February 2011

PROJECT: 09-1191-0022-1



Drawn by: DAM Checked by: SEMC

Stability Analysis **Detour Bridge Front Slope (East Side) with 350 kPa Footing Pressure**

FIGURE 4A

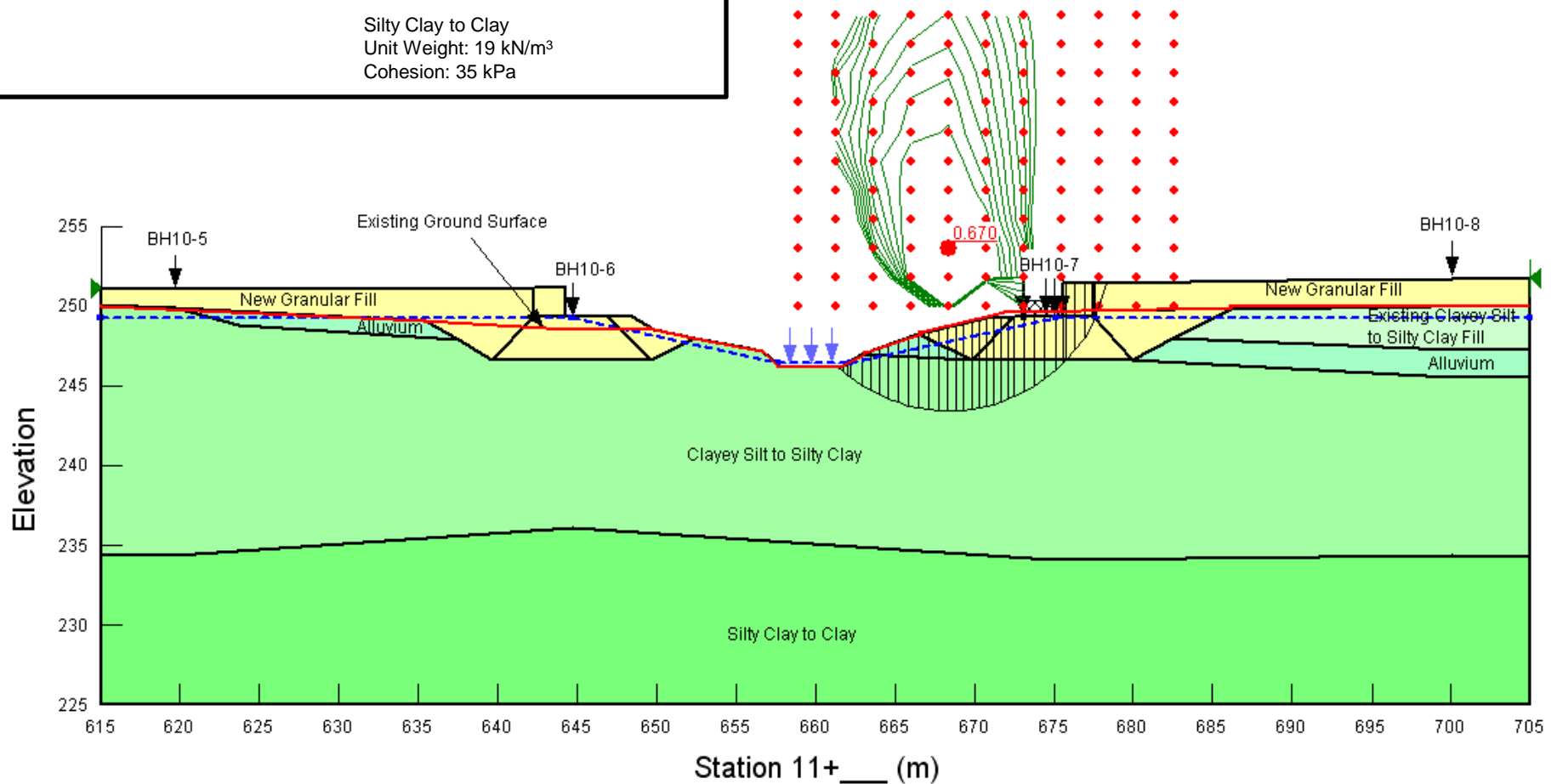
New Granular Fill
 Unit Weight: 21 kN/m³
 Phi: 35°

Existing Clayey Silt to Silty Clay Fill
 Unit Weight: 16 kN/m³
 Cohesion: 30 kPa

Silty Clay Alluvium
 Unit Weight: 17 kN/m³
 Cohesion: 30 kPa

Clayey Silt to Silty Clay
 Unit Weight: 19 kN/m³
 Cohesion: 28.5 kPa

Silty Clay to Clay
 Unit Weight: 19 kN/m³
 Cohesion: 35 kPa



DATE: February 2011

PROJECT: 09-1191-0022-1



Drawn by: DAM Checked by: SEMC

Stability Analysis **Detour Bridge Front Slope (East Side) with 110 kPa Footing Pressure**

FIGURE 4B

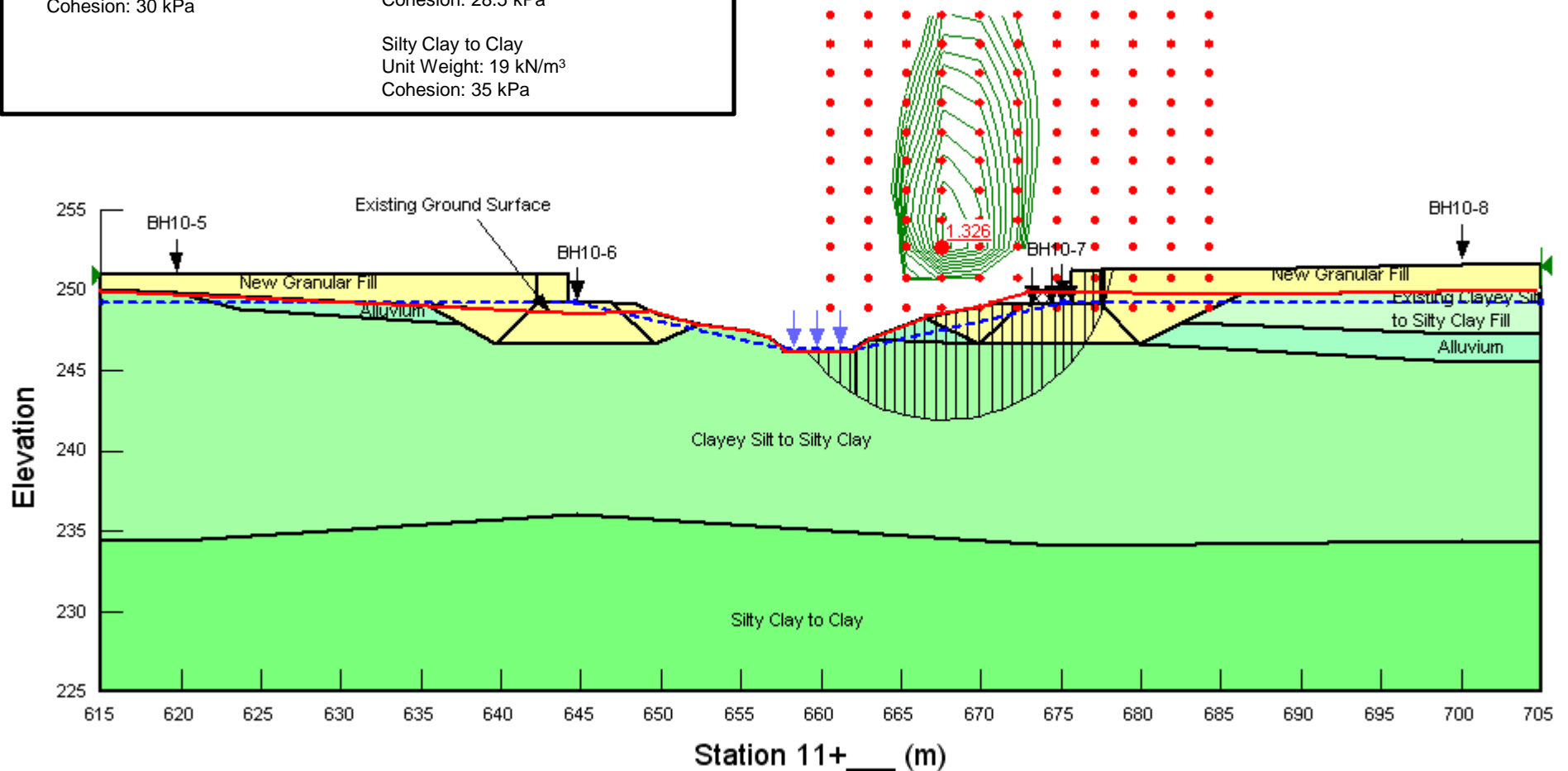
New Granular Fill
 Unit Weight: 21 kN/m³
 Phi: 35°

Existing Clayey Silt to Silty Clay Fill
 Unit Weight: 16 kN/m³
 Cohesion: 30 kPa

Silty Clay Alluvium
 Unit Weight: 17 kN/m³
 Cohesion: 30 kPa

Clayey Silt to Silty Clay
 Unit Weight: 19 kN/m³
 Cohesion: 28.5 kPa

Silty Clay to Clay
 Unit Weight: 19 kN/m³
 Cohesion: 35 kPa



DATE: February 2011
 PROJECT: 09-1191-0022-1



Drawn by: DAM Checked by: SEMC



APPENDIX A

Record of Boreholes and Drillholes



LIST OF SYMBOLS

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

1. 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
FoS	Factor of Safety
V	volume
W	weight

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

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



LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
SS	Split-spoon
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.).

Dynamic Cone Penetration Resistance, N_d :

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

PH:	Sampler advanced by hydraulic pressure
PM:	Sampler advanced by manual pressure
WH:	Sampler advanced by static weight of hammer
WR:	Sampler advanced by weight of sampler and rod

Piezocone Penetration Test (CPT)

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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Density Index (Relative Density)	N Blows/300 mm or Blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils

Consistency	C_u, S_u
	<u>kPa</u> <u>psf</u>
Very soft	0 to 12 0 to 250
Soft	12 to 25 250 to 500
Firm	25 to 50 500 to 1,000
Stiff	50 to 100 1,000 to 2,000
Very stiff	100 to 200 2,000 to 4,000
Hard	over 200 over 4,000

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO ₄	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

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



LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE

Fresh: no visible sign of weathering

Faintly weathered: weathering limited to the surface of Major discontinuities

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

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

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

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

BEDDING THICKNESS

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

JOINT OR FOLIATION SPACING

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

GRAIN SIZE

<u>Terms</u>	<u>Size*</u>
Very Coarse Grained	> 60 mm
Coarse Grained	2 – 60 mm
Medium Grained	60 microns – 2 mm
Fine Grained	2 – 60 microns
Very Fine Grained	< 2 microns

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

CORE CONDITION

Total Core Recovery (TCR)

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

Solid Core Recovery (SCR)

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

Rock Quality Designation (RQD)

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

DISCONTINUITY DATA

Fracture Index

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

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

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

Description and Notes

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

Abbreviations

B - Bedding	⊥ - Perpendicular To
FO - Foliation / Schistosity	- Parallel To
CL - Cleavage	P - Polished
SH - Shear Plane / Zone	K - Slickensided
VN - Vein	SM - Smooth
F - Fault	R - Rough
CO - Contact	ST - Stepped
J - Joint	PL - Planar
FR - Fracture	U - Undulating
MF - Mechanical Fracture	C - Curved



PROJECT 09-1191-0022		RECORD OF BOREHOLE No BR10-01		1 OF 2		METRIC	
W.P. 133-88-00		LOCATION N 5435628.2; E 308645.3		ORIGINATED BY		ID	
DIST HWY 652		BOREHOLE TYPE 108mm I.D. Continuous Flight Hollow Stem Augers		COMPILED BY		JJL	
DATUM Geodetic		DATE June 22, 2010		CHECKED BY		DAM	

[illegible]

Continued Next Page


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

MIS-MTO001 09-1191-0022 BRULE WICKLOW.GPJ GAL-MISS.GDT 02/02/11 DATA INPUT:

PROJECT 09-1191-0022		RECORD OF BOREHOLE No BR10-01				2 OF 2 METRIC											
W.P. 133-88-00		LOCATION N 5435628.2; E 308645.3				ORIGINATED BY ID											
DIST _____ HWY 652		BOREHOLE TYPE 108mm I.D. Continuous Flight Hollow Stem Augers				COMPILED BY JJL											
DATUM Geodetic		DATE June 22, 2010				CHECKED BY DAM											
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					WATER CONTENT (%)				
	--- CONTINUED FROM PREVIOUS PAGE ---						20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W _p W W _L 20 40 60					
235.0 15.8	CLAYEY SILT, trace to with sand Very soft to firm Grey Wet End of Borehole Note: 1. Water level at a depth of 4.9 m below ground surface (Elev. 245.9 m) upon completion of drilling. 2. Water level in piezometer at 5.9 m depth (Elev. 244.9 m) and 3.4 m depth (Elev. 247.4 m) on June 29, 2010 and August 12, 2010 respectively.		11	SS	3		235										

MIS-MTO001 09-1191-0022 BRULE WICKLOW.GPJ GAL-MISS.GDT 02/02/11 DATA INPUT:

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

PROJECT 09-1191-0022		RECORD OF BOREHOLE No BR10-02				2 OF 2 METRIC											
W.P. 133-88-00		LOCATION N 5435619.6; E 308715.2				ORIGINATED BY ID											
DIST _____ HWY 652		BOREHOLE TYPE 108mm I.D. Continuous Flight Hollow Stem Augers				COMPILED BY JJL											
DATUM Geodetic		DATE June 22, 2010				CHECKED BY DAM											
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					WATER CONTENT (%)				
	--- CONTINUED FROM PREVIOUS PAGE ---						<div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 20 40 60 80 100 </div> <div style="display: flex; justify-content: space-between;"> ○ UNCONFINED + FIELD VANE </div> <div style="display: flex; justify-content: space-between;"> ● QUICK TRIAXIAL × REMOULDED </div>					<div style="display: flex; justify-content: space-between;"> W_p W W_L </div>					
236.0 15.8	End of Borehole Note: 1. Water level at a depth of 6.1 m below ground surface (Elev. 245.7 m) upon completion of drilling. 2. Limited sample recovery (50 mm) in Sample 8. Small amount of sand, gravel and wood.		12	SS	3		236										

PROJECT		09-1191-0022		RECORD OF BOREHOLE No BR10-03		1 OF 4 METRIC											
W.P.		133-88-00		LOCATION		N 5435626.8; E 308694.4											
DIST		HWY 652		BOREHOLE TYPE		108mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, NQ Coring											
DATUM		Geodetic		DATE		June 23 and 24, 2010											
				ORIGINATED BY		ID											
				COMPILED BY		JJL											
				CHECKED BY		DAM											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	20 40 60 80 100	W _p W W _L	WATER CONTENT (%)	20 40 60	γ	GR SA SI CL			
251.4	GROUND SURFACE																
0.0	ASPHALT																
0.2	Sand and gravel, trace silt, trace clay (FILL) Compact Brown Moist		1	SS	20		251										
			2	SS	14		250							44 51 3 2			
249.1																	
2.3	Silty sand, trace gravel, trace glass fragments (FILL) Dense Brown Moist to wet		3	SS	38		249										
248.4																	
3.0	Silty clay, with sand, trace organics (FILL) Stiff to very stiff Brown Moist		4	SS	12		248										
			5	SS	16		247										
247.1	Sand and gravel layer between 3.4 m and 3.5 m depth.																
4.3	Clayey silt to silty clay, trace to with sand, trace gravel, wood / roots, trace to some organics (FILL) Firm to stiff Brown to black Moist to wet		6	SS	9		246							OC=8.1%			
			7	SS	6		245							6 42 32 20			
244.2																	
7.2	CLAYEY SILT, trace to some sand Very soft to firm Grey Wet		8	SS	3		244										
	Switched to NW casing at 8.2 m depth.						243										
			9	SS	1		242										
							241										
			10	SS	3		240										
							239										
							238										
			11	SS	1		237										

Continued Next Page

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


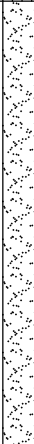


MIS-MTO001 09-1191-0022 BRULE WICKLOW.GPJ GAL-MISS.GDT 02/02/11 DATA INPUT:

PROJECT 09-1191-0022				RECORD OF BOREHOLE No BR10-03				2 OF 4 METRIC					
W.P. 133-88-00				LOCATION N 5435626.8; E 308694.4				ORIGINATED BY ID					
DIST _____ HWY 652				BOREHOLE TYPE 108mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, NQ Coring				COMPILED BY JJL					
DATUM Geodetic				DATE June 23 and 24, 2010				CHECKED BY DAM					
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT		UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)			
	--- CONTINUED FROM PREVIOUS PAGE ---												
234.4	CLAYEY SILT, trace to some sand Very soft to firm Grey Wet		12	TO	PH		236						
17.0	SILTY CLAY to CLAY Very soft to firm Grey Wet						235						
							234	4 +					
			13	SS	1		233						0 0 15 85
							232						
							231	3 +					
			14	TO	PH		230						
							229						
							228						
			15	SS	3		227						
							226						
							225						
			16	SS	4		224						
							223	3 +					
222.1	SILT, some to with clay Compact Grey Wet						222						

MIS-MTO 001 09-1191-0022 BRULE WICKLOW.GPJ GAL-MISS.GDT 02/02/11 DATA INPUT:

Continued Next Page

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

PROJECT		133-88-00		LOCATION		N 5435626.8; E 308694.4		ORIGINATED BY		ID										
DIST		HWY 652		BOREHOLE TYPE		108mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, NQ Coring		COMPILED BY		JUL										
DATUM		Geodetic		DATE		June 23 and 24, 2010		CHECKED BY		DAM										
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 (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	SHEAR STRENGTH kPa					WATER CONTENT (%)									
--- CONTINUED FROM PREVIOUS PAGE ---																				
219.1	SILT, some to with clay Compact Grey Wet		17	SS	11		221										0	0	77	23
220																				
32.3	SAND, some silt to Sandy SILT, trace to some clay Dense Brown to grey Wet Artesian conditions noted below 33.0 m depth.		18	SS	32		219										0	82	13	5
218																				
217																				
216																				
214.5	COBBLES and BOULDERS, with gravel		19	SS	35		215													
36.9							214													
							213													
	Artesian conditions (water flowing out of casing) between 38.9 m and 39.5 m depth. Switched to NQ coring at 39.5 m depth.						212													
							211													
210.8	MAFIC METAVOLCANIC (BEDROCK)		1	RC	REC 100%		210										RQD = 61%			
40.6	Bedrock cored from 40.6 m to 45.0 m depth. For details of bedrock coring refer to Record of Drillhole BR10-03.		2	RC	REC 100%		209													
			3	RC	REC 100%		208													
							207													
206.4																				

MIS-MTO001 09-1191-0022 BRULE WICKLOW.GPJ GAL-MISS.GDT 02/02/11 DATA INPUT:

Continued Next Page

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

PROJECT <u>09-1191-0022</u>	RECORD OF BOREHOLE No BR10-03	4 OF 4 METRIC
W.P. <u>133-88-00</u>	LOCATION <u>N 5435626.8; E 308694.4</u>	ORIGINATED BY <u>ID</u>
DIST <u></u> HWY <u>652</u>	BOREHOLE TYPE <u>108mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, NQ Coring</u>	COMPILED BY <u>JJL</u>
DATUM <u>Geodetic</u>	DATE <u>June 23 and 24, 2010</u>	CHECKED BY <u>DAM</u>

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL	
								20	40	60	80	100	W _p	W	W _L						
	--- CONTINUED FROM PREVIOUS PAGE ---																				
45.0	End of Borehole Note: 1. Water level at 2.0 m above ground surface (Elev. 253.4 m), corresponds to approximately 7.0 m above the creek level, upon completion of drilling. 2. On August 12, 2010, visual inspection at the ground surface indicated no artesian flow condition.																				

MIS-MTO001 09-1191-0022 BRULE WICKLOW.GPJ GAL-MISS.GDT 02/02/11 DATA INPUT:

PROJECT: 09-1191-0022

RECORD OF DRILLHOLE: BR10-03

SHEET 1 OF 1

LOCATION: N 5435626.8 ; E 308694.4

DRILLING DATE: June 23 and 24, 2010

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55

DRILLING CONTRACTOR: George Downing Estate Drilling Ltd.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH	RECOVERY TOTAL CORE %	SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3 m	B Angle °	DIP w.r.t. CORE AXIS °	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	HYDRAULIC CONDUCTIVITY k, cm/s	Diametral Point Load Index (MPa)	RMC -Q AVG.	
		Refer To Previous Page		210.8																
	NW	MAFIC METAVOLCANIC, with granitic veins Fine grained Healed to partially healed joints Massive Dark Grey		40.6	1	Grey / 100%							JIR JIR JIR JIR JIR							
41																				
42					2	Grey / 100%							BR JIR JIR JIR JIR							
43	June 24, 2010 NQ Coring												JIRx4 BR JIR JIR							
44					3	Grey / 100%							JIR JIR JIR JIR JIR							UCS=175 MPa
45		End Of Drillhole		206.4 45.0									JIR JIR JIR							
46																				
47																				
48																				
49																				
50																				

DEPTH SCALE

1 : 50



LOGGED: ID

CHECKED: DAM

SUD-RCK 09-1191-0022 BRULE WICKLOW.GPJ GAL-MISS.GDT 02/02/11 DATA INPUT:

PROJECT 09-1191-0022			RECORD OF BOREHOLE No BR10-04			1 OF 4 METRIC															
W.P. 133-88-00			LOCATION N 5435621.1; E 308666.2			ORIGINATED BY ID															
DIST HWY 652			BOREHOLE TYPE 108mm I.D. Continuous Flight Hollow Stem Augers, NW Casing			COMPILED BY JJL															
DATUM Geodetic			DATE June 25 and 26, 2010			CHECKED BY DAM															
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ			GR SA SI CL		
251.1	GROUND SURFACE							20 40 60 80 100	20 40 60	20 40 60	20 40 60	20 40 60	20 40 60	20 40 60	20 40 60	20 40 60	20 40 60	20 40 60	20 40 60	20 40 60	20 40 60
0.0	ASPHALT						251														
0.3	Sand and gravel (FILL) Sand, some gravel, trace to some silt (FILL) Compact to very dense Brown Moist		1	SS	15		250														
	50 mm thick layer of WOOD at 1.5 m depth.		2	SS	11		249														
	Cobbles and boulders between 2.4 m and 3.0 m depth.		3	SS	103		248														
248.1	SILTY CLAY with peat layers and organics (ALLUVIUM) Very stiff Grey to black and brown Moist		4	SS	25		247														
247.4	SILTY CLAY, trace sand Stiff Brown Moist		5	SS	11		246														
3.7			6	SS	13		245														
245.5	CLAYEY SILT, trace sand Soft to firm Grey Wet		7	SS	4		244														
5.6			8	TO	PH		243														
	Switched to NW casing at 8.2 m depth.		9	SS	3		242														
			10	TO	PH		241														
			11	SS	3		240														
							239														
							238														
							237														

Continued Next Page

+ 3, X 3: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

MIS-MTO 001 09-1191-0022 BRULE WICKLOW.GPJ GAL-MISS.GDT 02/02/11 DATA INPUT:


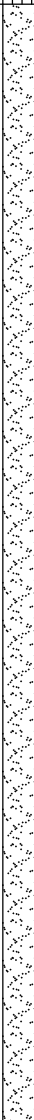
PROJECT <u>09-1191-0022</u>		RECORD OF BOREHOLE No BR10-04		2 OF 4 METRIC	
W.P. <u>133-88-00</u>		LOCATION <u>N 5435621.1; E 308666.2</u>		ORIGINATED BY <u>ID</u>	
DIST <u> </u> HWY <u>652</u>		BOREHOLE TYPE <u>108mm I.D. Continuous Flight Hollow Stem Augers, NW Casing</u>		COMPILED BY <u>JJL</u>	
DATUM <u>Geodetic</u>		DATE <u>June 25 and 26, 2010</u>		CHECKED BY <u>DAM</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)								
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED										
	--- CONTINUED FROM PREVIOUS PAGE ---							20 40 60 80 100		20 40 60								
235.6							236											
15.5	SILTY CLAY to CLAY Soft to firm Grey Wet							3 + 4 +										
							235											
			12	SS	3		234			40	50							
							233											
							232	4 + 4 +										
			13	SS	3		231											
							230											
							229											
							228	3 + 4 +										
							227											
							226											
			14	SS	3		225			40								
							224	3 + 3 +										
							223											
			15	SS	5		222											
								3										

Continued Next Page

+ 3, X 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE


MIS-MTO001 09-1191-0022 BRULE WICKLOW.GPJ GAL-MISS.GDT 02/02/11 DATA INPUT:

PROJECT 09-1191-0022			RECORD OF BOREHOLE No BR10-04			3 OF 4 METRIC						
W.P. 133-88-00			LOCATION N 5435621.1; E 308666.2			ORIGINATED BY ID						
DIST HWY 652			BOREHOLE TYPE 108mm I.D. Continuous Flight Hollow Stem Augers, NW Casing			COMPILED BY JJL						
DATUM Geodetic			DATE June 25 and 26, 2010			CHECKED BY DAM						
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W _p — W — W _L WATER CONTENT (%)	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
220.3	SILTY CLAY to CLAY Soft to firm Grey Wet						221	3				
30.8	SILT, trace to some sand, trace to some clay Compact Grey Wet		16	SS	17		220					
							219		o			0 6 86 8
217.3	SAND to SAND and SILT, trace clay Compact Grey Wet Artesian condition noted below 33.8 m depth.		17	SS	19		218					
33.8							217					
							216					
							215					
							214					
			18	SS	15		213		o			0 63 33 4
							212					
							211					
		19	SS	21		210						
						209						
						208						
						207						
			20	SS	11							

Continued Next Page

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

MIS-MTO 001 09-1191-0022 BRULE WICKLOW.GPJ GAL-MISS.GDT 02/02/11 DATA INPUT:

PROJECT 09-1191-0022		RECORD OF BOREHOLE No BR10-04				4 OF 4 METRIC												
W.P. 133-88-00		LOCATION N 5435621.1; E 308666.2				ORIGINATED BY ID												
DIST HWY 652		BOREHOLE TYPE 108mm I.D. Continuous Flight Hollow Stem Augers, NW Casing				COMPILED BY JJL												
DATUM Geodetic		DATE June 25 and 26, 2010				CHECKED BY DAM												
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					WATER CONTENT (%)					
	--- CONTINUED FROM PREVIOUS PAGE ---						<div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 20 40 60 80 100 </div> <div style="display: flex; justify-content: space-between;"> ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED </div>					<div style="display: flex; justify-content: space-between;"> W_p W W_L </div>						
205.1	COBBLES and BOULDERS					206											RQD = 68%	
46.0						205												
204.2			MAFIC METAVOLCANIC (BEDROCK) Bedrock cored from 46.9 m to 50.3 m depth. For details of bedrock coring refer to Record of Drillhole BR10-04.	1	RC	REC 100%	204											
46.9				2	RC	REC 100%	203											RQD = 19%
	3	RC		REC 100%	202										RQD = 100%			
200.8	50.3																	
End of Borehole Note: 1. Water level at 0.9 m above ground surface (Elev. 252.0 m) corresponds to approximately 5.5 m above the creek level, upon completion of drilling. 2. Bentonite powder added to wash water to advance borehole below 32 m. 3. On August 12, 2010 visual inspection at the ground surface indicated no artesian flow condition.																		

MIS-MTO 001 09-1191-0022 BRULE WICKLOW.GPJ GAL-MISS.GDT 02/02/11 DATA INPUT:

SHEET 1 OF 1

DATUM: Geodetic

DRILLING CONTRACTOR: George Downing Estate Drilling Ltd.


LOGGED: ID
CHECKED: DAM

PROJECT 09-1191-0022			RECORD OF BOREHOLE No BR10-05			1 OF 2 METRIC																
W.P. 133-88-00			LOCATION N 5435609.0; E 308640.0			ORIGINATED BY ID																
DIST _____ HWY 652			BOREHOLE TYPE 108mm I.D. Continuous Flight Hollow Stem Augers			COMPILED BY JJL																
DATUM Geodetic			DATE June 27, 2010			CHECKED BY DAM																
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)							
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ			GR SA SI CL			
249.9	GROUND SURFACE							20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W _p — W — W _L 20 40 60			kN/m ³						
0.1	ORGANICS (TOPSOIL)		1	SS	15		249															
	SILTY CLAY, trace to some sand, trace organics Very stiff Brown Wet		2	SS	29		248															0 5 46 49
			3	SS	17		247															
247.6	CLAYEY SILT to SILTY CLAY, trace to some sand, trace gravel Very soft to firm Grey Wet		4	SS	7		246															1 6 46 47
2.3			5	SS	4		245															
			6	SS	2		244															1 20 34 45
			7	SS	2		243															
			8	SS	3		242															
							241															
							240															
							239															
							238															
			9	SS	3		237															
							236															
	SILT layer 300 mm thick at 13.7 m depth.						235															

Continued Next Page

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

MIS-MTO 001 09-1191-0022 BRULE WICKLOW.GPJ GAL-MISS.GDT 02/02/11 DATA INPUT:

PROJECT <u>09-1191-0022</u>		RECORD OF BOREHOLE No BR10-05				2 OF 2 METRIC											
W.P. <u>133-88-00</u>		LOCATION <u>N 5435609.0; E 308640.0</u>				ORIGINATED BY <u>ID</u>											
DIST <u> </u> HWY <u>652</u>		BOREHOLE TYPE <u>108mm I.D. Continuous Flight Hollow Stem Augers</u>				COMPILED BY <u>JJL</u>											
DATUM <u>Geodetic</u>		DATE <u>June 27, 2010</u>				CHECKED BY <u>DAM</u>											
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					W _p W W _L				
	--- CONTINUED FROM PREVIOUS PAGE ---						20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100					WATER CONTENT (%) 20 40 60					
234.1 15.8	End of Borehole Note: 1. Water level at a depth of 14.4 m below ground surface (Elev. 235.5 m) and rising, upon completion of drilling.		10	SS	1		234								○		

PROJECT 09-1191-0022			RECORD OF BOREHOLE No BR10-06			1 OF 2 METRIC						
W.P. 133-88-00			LOCATION N 5435606.3; E 308665.0			ORIGINATED BY ID						
DIST _____ HWY 652			BOREHOLE TYPE 108mm I.D. Continuous Flight Hollow Stem Augers			COMPILED BY JJL						
DATUM Geodetic			DATE June 27, 2010			CHECKED BY DAM						
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC NATURAL LIQUID UNIT REMARKS			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	20 40 60	W _p W W _L	γ	GR SA SI CL
248.5	GROUND SURFACE											
0.0	ORGANICS (TOPSOIL)											
0.2	SILTY CLAY, some sand, trace gravel, trace to some organics (ALLUVIUM) Stiff Dark brown Moist		1	SS	10		248					3 13 41 43
247.4			2	SS	12							
1.1	SILTY CLAY trace to some sand, trace gravel Stiff Brown Moist to wet		3	SS	9		247					
246.2												
2.3	CLAYEY SILT to SILTY CLAY Very soft to firm Grey Wet		4	SS	4		246					
			5	SS	4		245					0 17 38 45
			6	SS	1		244					
			7	TO	PH		243					
							242					
							241					
			8	SS	1		240					
							239					
							238					
			9	TO	PH		237					
							236					
236.0	CLAY some silt, trace sand Very soft to firm Grey Wet						235					
12.5			10	SS	1		234					0 1 18 81

MIS-MTO 001 09-1191-0022 BRULE WICKLOW.GPJ GAL-MISS.GDT 02/02/11 DATA INPUT:

Continued Next Page


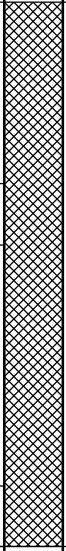
+³, ×³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

PROJECT <u>09-1191-0022</u>		RECORD OF BOREHOLE No BR10-06				2 OF 2 METRIC	
W.P. <u>133-88-00</u>		LOCATION <u>N 5435606.3; E 308665.0</u>				ORIGINATED BY <u>ID</u>	
DIST <u> </u> HWY <u>652</u>		BOREHOLE TYPE <u>108mm I.D. Continuous Flight Hollow Stem Augers</u>				COMPILED BY <u>JJL</u>	
DATUM <u>Geodetic</u>		DATE <u>June 27, 2010</u>				CHECKED BY <u>DAM</u>	

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					WATER CONTENT (%)				
							<div style="display: flex; justify-content: space-between; font-size: small;"> 20 40 60 80 100 20 40 60 80 100 </div> <div style="display: flex; justify-content: space-between; font-size: x-small;"> ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED </div>					<div style="display: flex; justify-content: space-between; font-size: x-small;"> W_p W W_L </div>					
	CLAY some silt, trace sand Very soft to firm Grey Wet						233										
						▽											
			11	SS			232										
							231										
			12	SS			230						○				
							229										
228.1 20.4	End of Borehole Note: 1. Water level at a depth of 16.0 m below ground surface (Elev. 232.5 m) and rising, upon completion of drilling.		13	SS													

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

MIS-MTO001 09-1191-0022 BRULE WICKLOW.GPJ GAL-MISS.GDT 02/02/11 DATA INPUT:

PROJECT 09-1191-0022		RECORD OF BOREHOLE No BR10-07				2 OF 2 METRIC												
W.P. 133-88-00		LOCATION N 5435611.6; E 308695.4				ORIGINATED BY ID												
DIST _____ HWY 652		BOREHOLE TYPE 108mm I.D. Continuous Flight Hollow Stem Augers				COMPILED BY JJL												
DATUM Geodetic		DATE June 28, 2010				CHECKED BY DAM												
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa		WATER CONTENT (%)								
--- CONTINUED FROM PREVIOUS PAGE ---																		
234.1 15.5	CLAY, some silt, trace sand Soft to firm		12	SS	2		234	6										
							233											
							232											
							231											
							230											
229.2 20.4	End of Borehole																	
	Note: 1. Water level in piezometer at 4.8 m depth (Elev. 244.8 m) and 0.4 m (Elev. 249.2 m) depth on June 29, 2010 and August 12, 2010 respectively 2. On August 12, 2010, additional borehole advanced 1.5 m south and Shelby tube samples obtained at 4.6 m, 6.1 m, and 7.6 m depths.																	

MIS-MTO 001 09-1191-0022 BRULE WICKLOW.GPJ GAL-MISS.GDT 02/02/11 DATA INPUT:

+ 3, X 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

MIS-MTO001 09-1191-0022 BRULE WICKLOW.GPJ GAL-MISS.GDT 02/02/11 DATA INPUT:

Continued Next Page

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



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



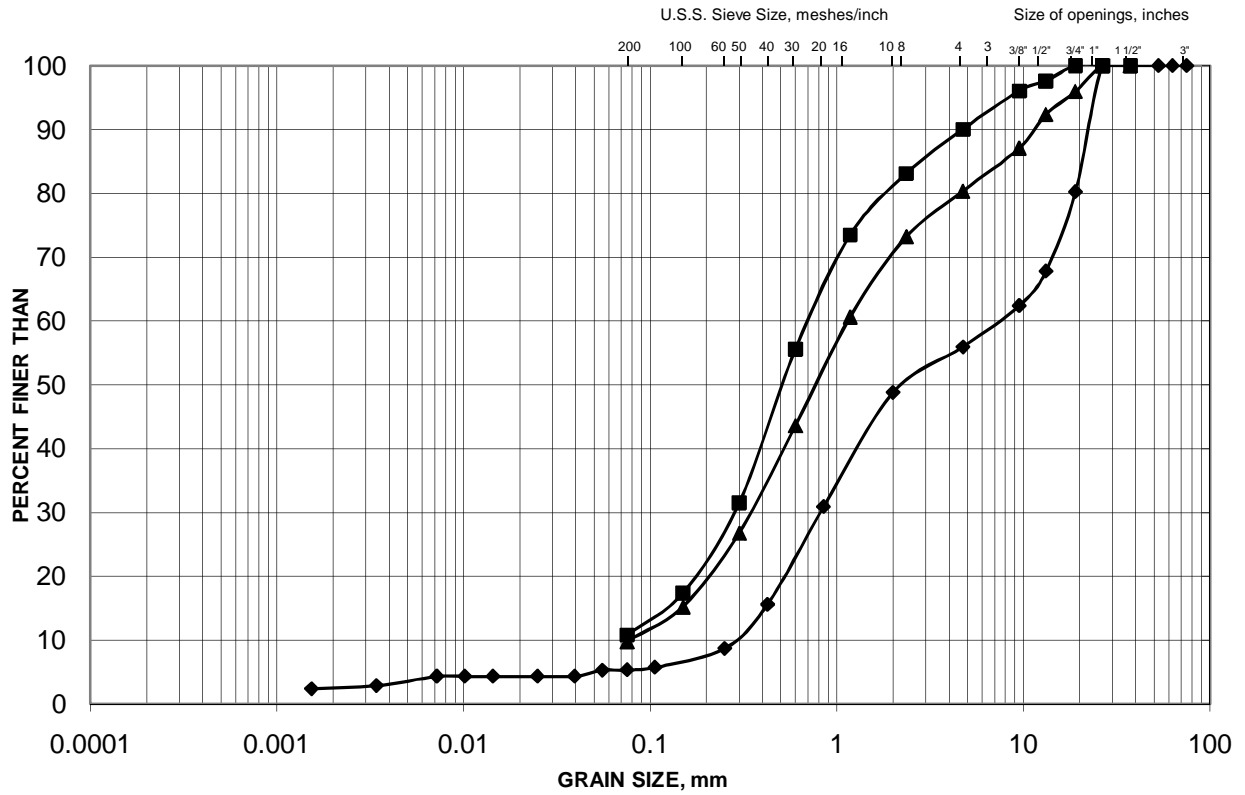
APPENDIX B

Laboratory Test Results

GRAIN SIZE DISTRIBUTION

Sand to Sand and Gravel (Fill)

FIGURE
B-1



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
■	BR10-02	1	251.5
◆	BR10-03	2	249.6
▲	BR10-04	3	248.5

Project Number: 09-1191-0022-1

Checked By: SEMC

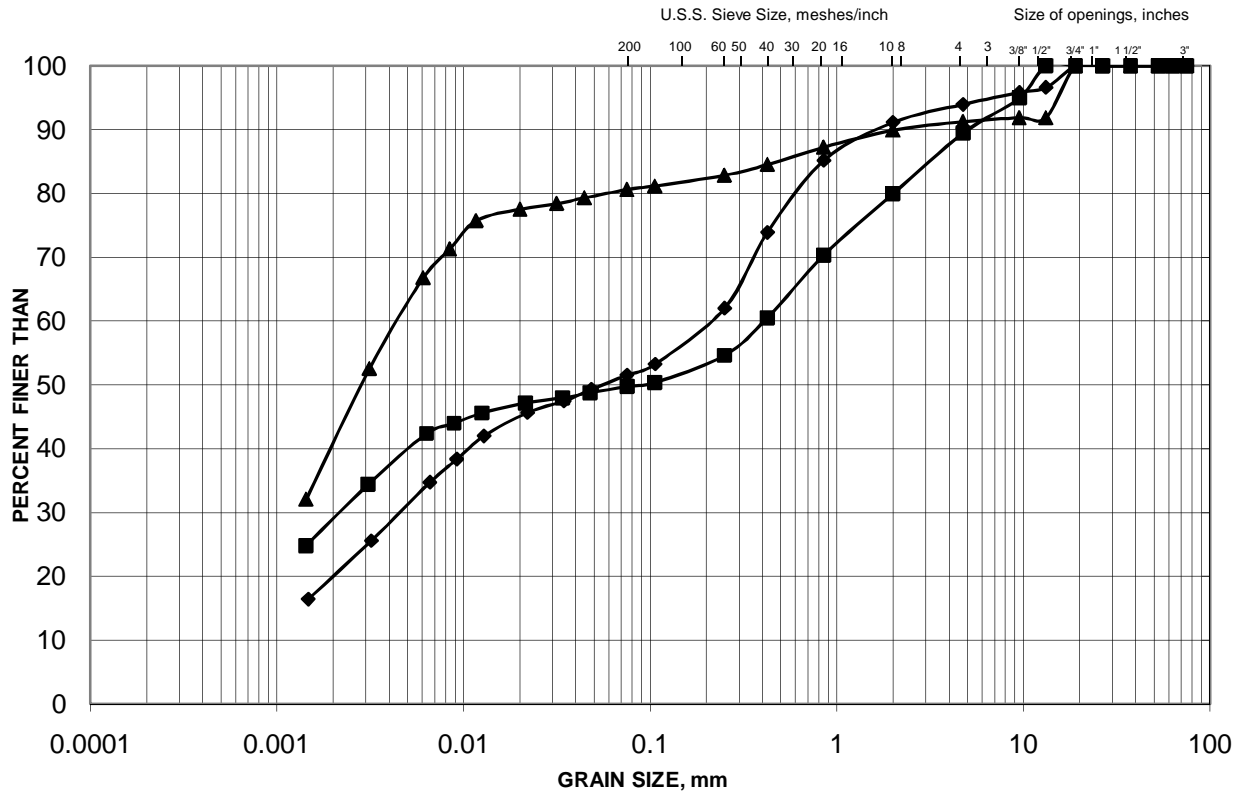
Golder Associates

Date: February 2011

GRAIN SIZE DISTRIBUTION

Clayey Silt to Silty Clay (Fill)

FIGURE
B-2



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

LEGEND

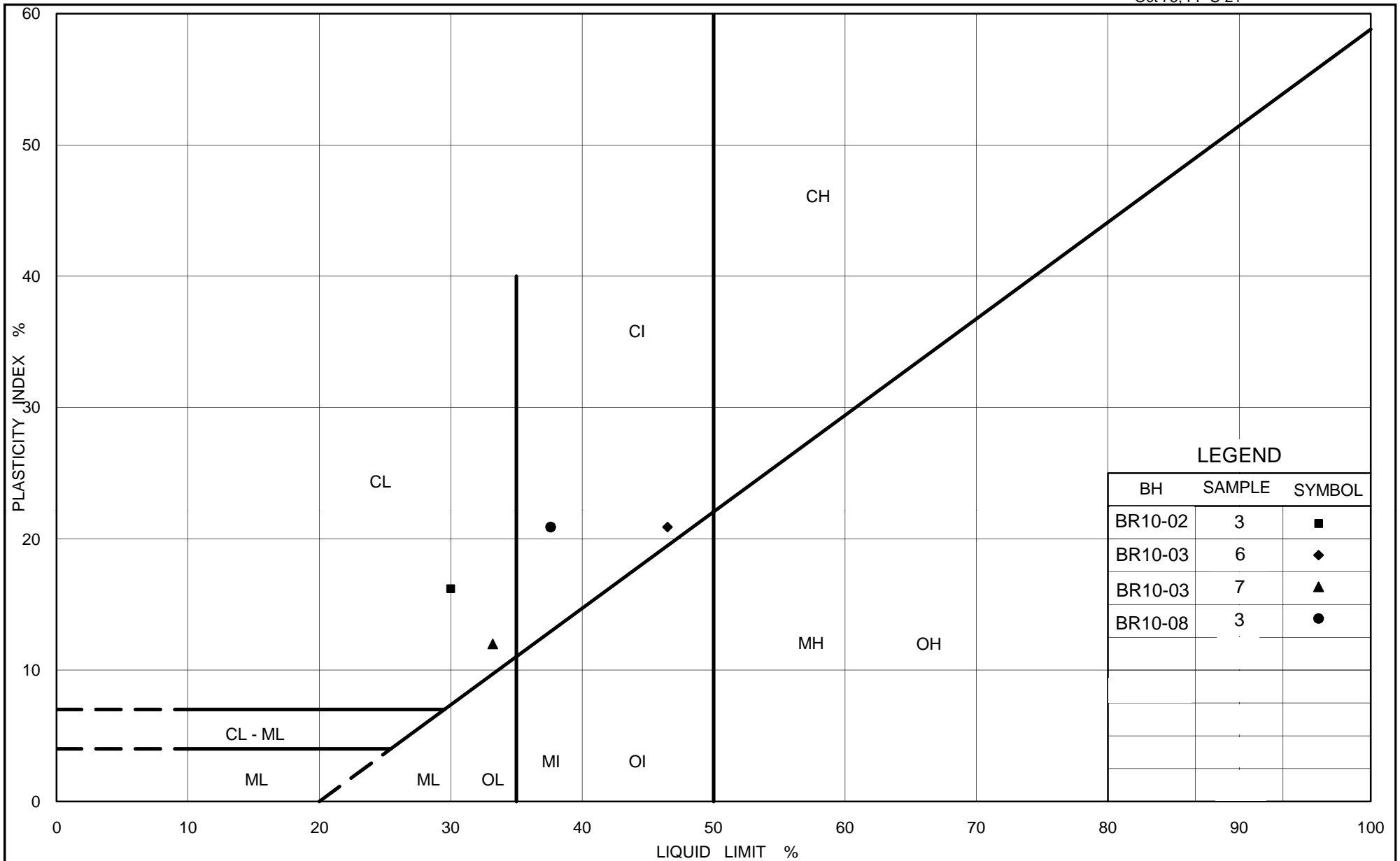
SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
■	BR10-02	3	250.0
◆	BR10-03	7	245.0
▲	BR10-08	3	248.2

Project Number: 09-1191-0022-1

Checked By: SEMC

Golder Associates

Date: February 2011



Ministry of Transportation
Ontario

PLASTICITY CHART

Clayey Silt to Silty Clay (Fill)

Figure B-3

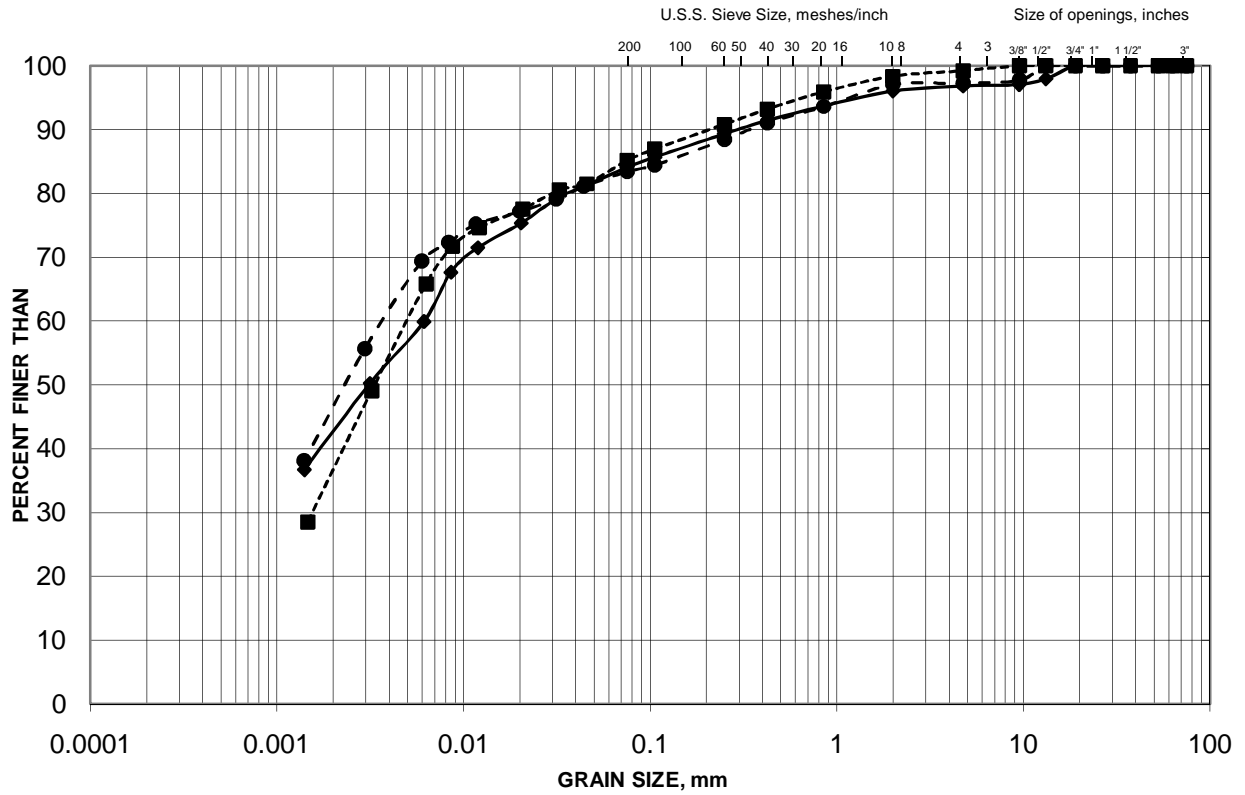
Project No. 09-1191-0022-1

Checked By: SEMC

GRAIN SIZE DISTRIBUTION

Silty Clay (Alluvium)

FIGURE
B-4



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

LEGEND

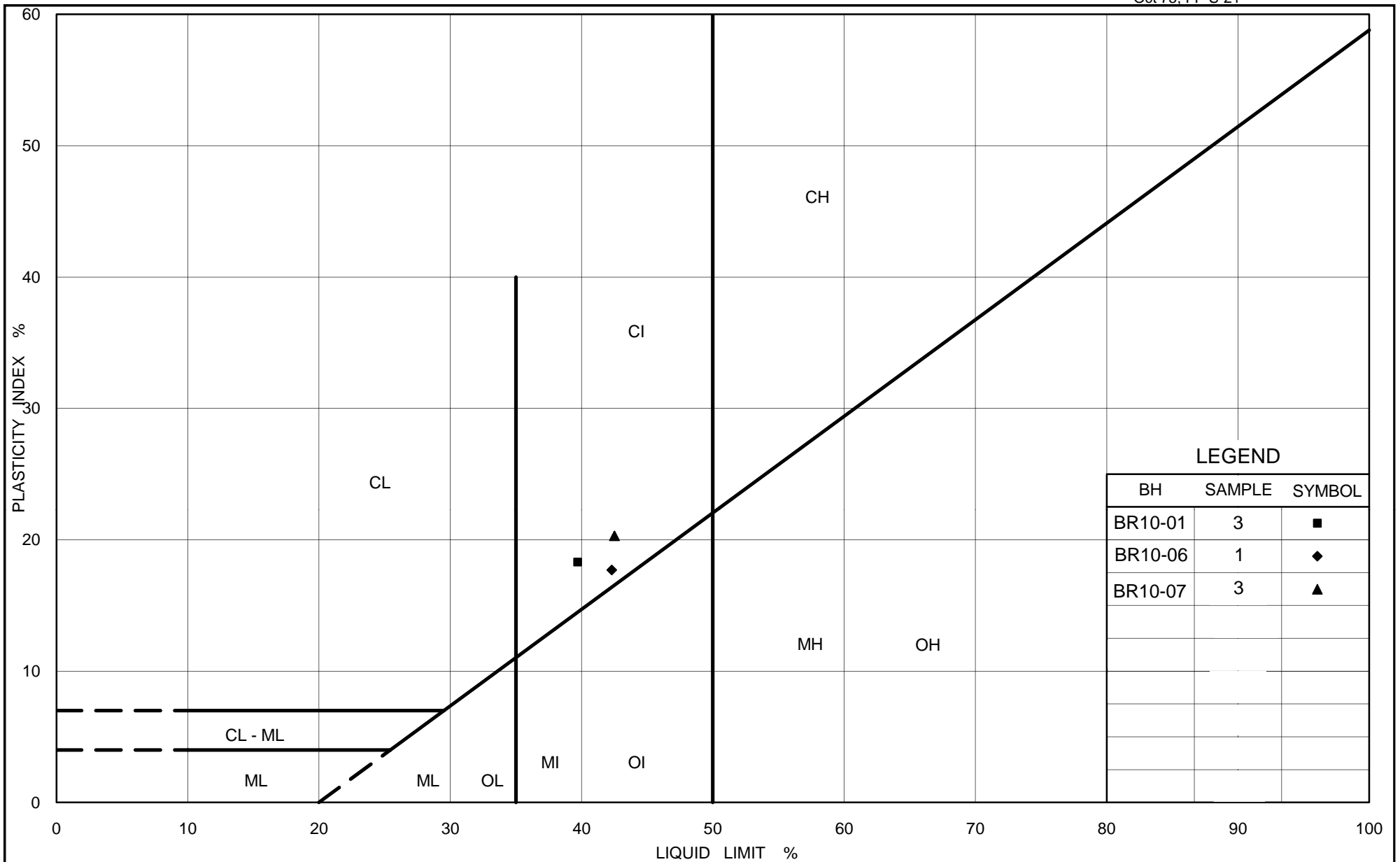
SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
-●-	BR10-01	3	248.2
-◆-	BR10-06	1	248.2
-■-	BR10-07	3	247.8

Project Number: 09-1191-0022-1

Checked By: SEMC

Golder Associates

Date: February 2011



Ministry of Transportation
Ontario

PLASTICITY CHART

Silty Clay (Alluvium)

Figure B-5

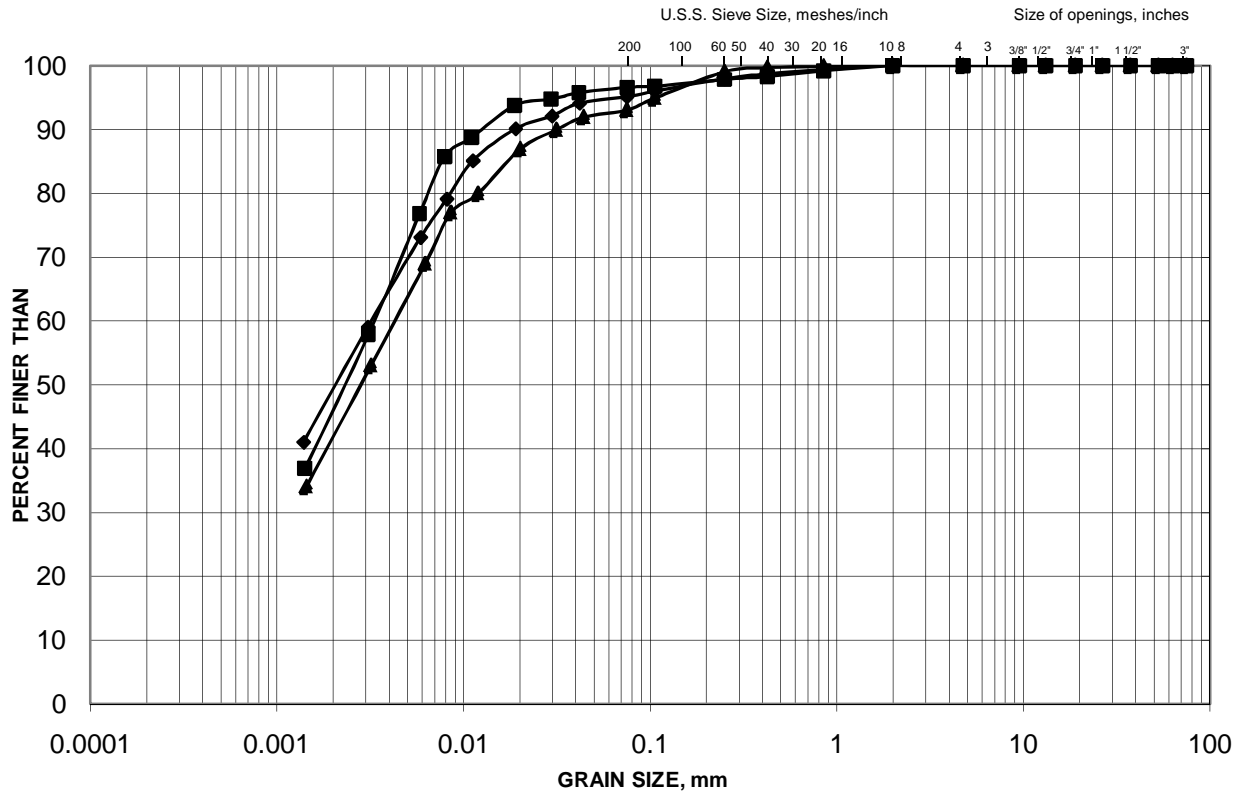
Project No. 09-1191-0022-1

Checked By: SEMC

GRAIN SIZE DISTRIBUTION

Silty Clay

FIGURE
B-6



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

LEGEND

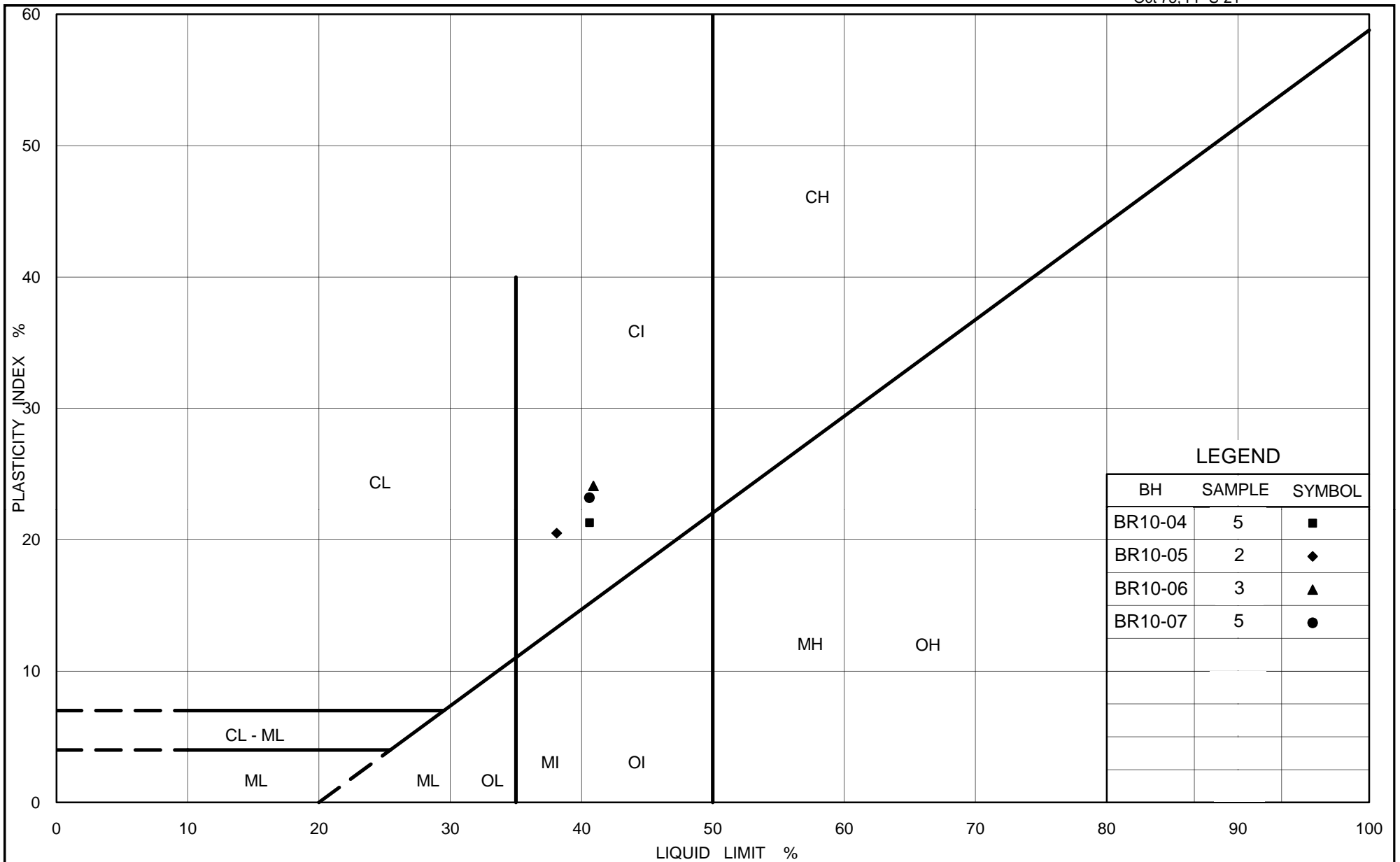
SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
■	BR10-04	5	246.9
◆	BR10-05	2	248.8
▲	BR10-07	5	246.3

Project Number: 09-1191-0022-1

Checked By: SEMC

Golder Associates

Date: February 2011



Ministry of Transportation
Ontario

PLASTICITY CHART

Silty Clay

Figure B-7

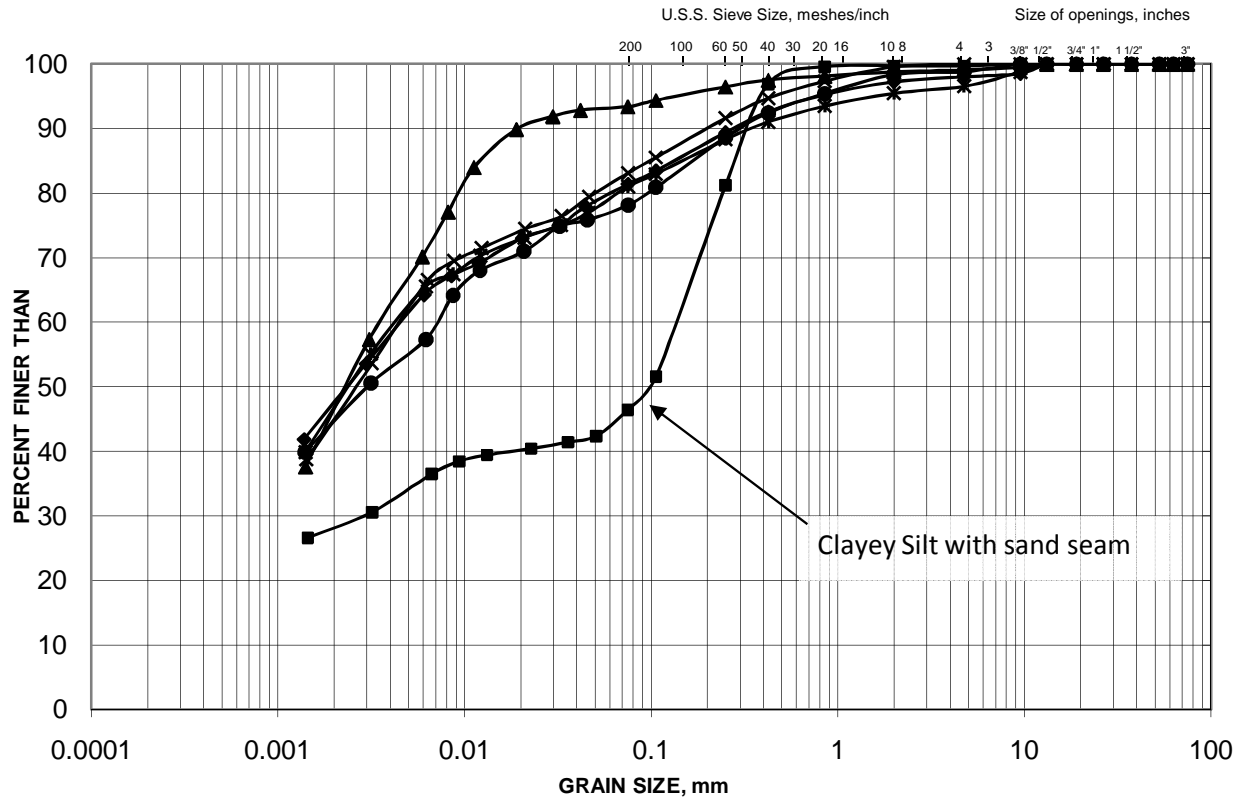
Project No. 09-1191-0022-1

Checked By: SEMC

GRAIN SIZE DISTRIBUTION

Clayey Silt to Silty Clay

FIGURE
B-8



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

LEGEND

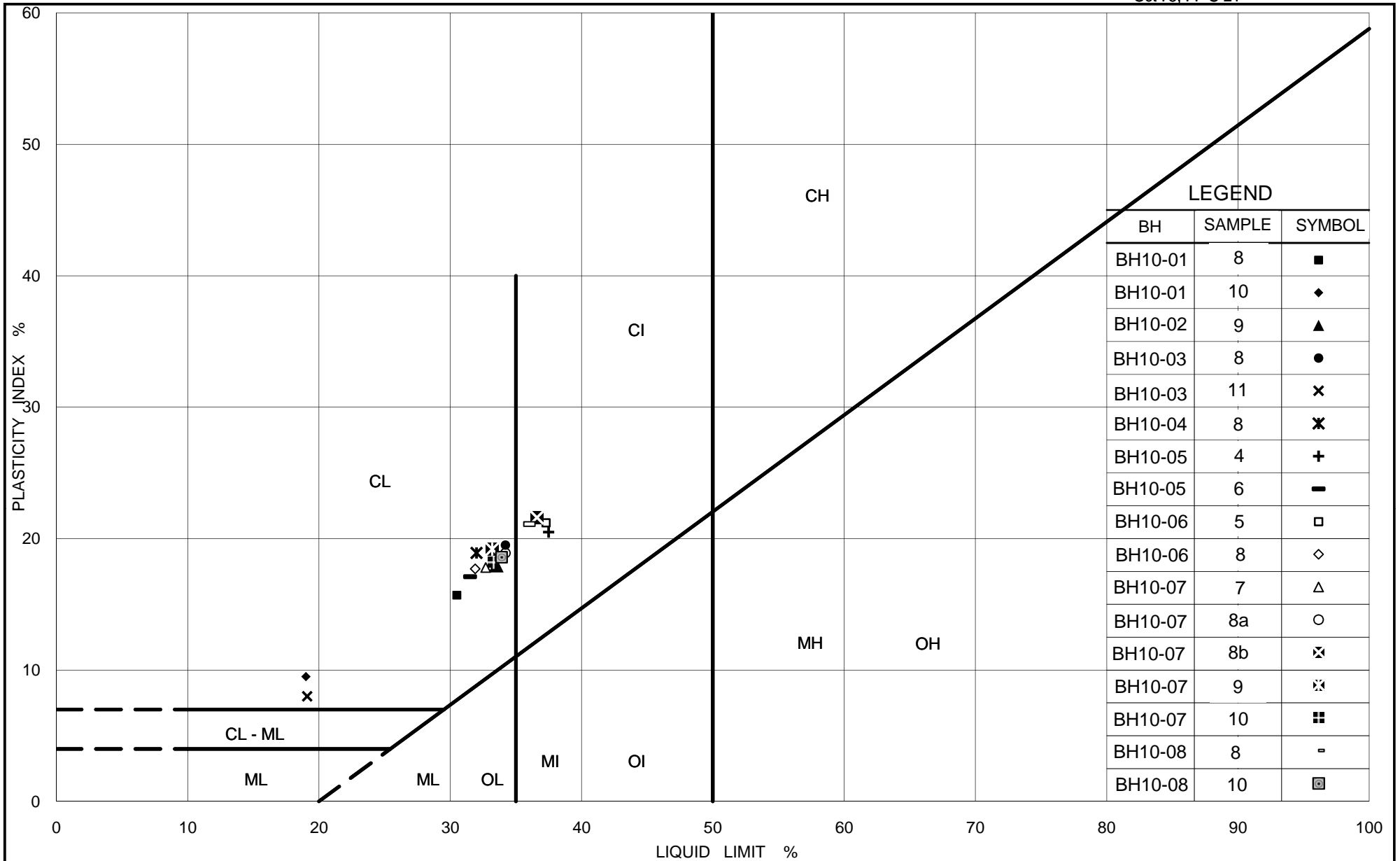
SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
■	BR10-01	10	236.8
◆	BR10-02	9	243.9
▲	BR10-05	4	247.3
●	BR10-05	6	245.0
×	BR10-06	5	245.1
✱	BR10-08	8	243.6

Project Number: 09-1191-0022-1

Checked By: SEMC

Golder Associates

Date: February 2011



Ministry of Transportation
Ontario

PLASTICITY CHART

Clayey Silt to Silty Clay

Figure B-9

Project No. 09-1191-0022-1

Checked By: SEMC

CONSOLIDATION TEST SUMMARY**FIGURE B-10**

Page 1 of 4

SAMPLE IDENTIFICATION

Project Number 09-1191-022-1
Borehole Number BR10-04

Sample Number 8
Sample Depth, m 7.9

TEST CONDITIONS

Test Type Standard Load Duration, hr 24
Date Started 7/22/10
Date Completed 8/6/10

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.55	Unit Weight, kN/m ³	19.39
Sample Diameter, cm	6.36	Dry Unit Weight, kN/m ³	15.21
Area, cm ²	31.77	Specific Gravity, measured	2.66
Volume, cm ³	80.95	Solids Height, cm	1.487
Water Content, %	27.54	Volume of Solids, cm ³	47.26
Wet Mass, g	160.08	Volume of Voids, cm ³	33.69
Dry Mass, g	125.51	Degree of Saturation, %	102.6

TEST COMPUTATIONS

Pressure	Primary Consolidation	Corr. Height	Void Ratio	Average Height	t ₉₀	cv.	mv	k	Total Work
kPa	mm	cm		cm	sec	cm ² /s	m ² /kN	cm/s	kJ/m ³
0.0	0.00	2.548	0.713	2.548					
8.9	0.04	2.544	0.710	2.546	960	0.0014	1.64E-04	2.31E-08	0.007
17.9	0.04	2.540	0.708	2.542	900	0.0015	1.76E-04	2.62E-08	0.028
35.1	0.09	2.531	0.702	2.536	1560	0.0009	2.01E-04	1.72E-08	0.119
69.2	0.14	2.517	0.692	2.524	1320	0.0010	1.66E-04	1.66E-08	0.416
142.6	0.31	2.487	0.672	2.502	2520	0.0005	1.63E-04	8.41E-09	1.699
284.9	0.75	2.412	0.621	2.449	2700	0.0005	2.06E-04	9.52E-09	8.129
570.5	0.78	2.334	0.569	2.373	1920	0.0006	1.07E-04	6.54E-09	21.979
1139.7	0.75	2.259	0.518	2.296	1680	0.0007	5.17E-05	3.37E-09	49.460
2279.0	0.56	2.203	0.481	2.231	1200	0.0009	1.93E-05	1.66E-09	91.916
570.5	-0.15	2.217	0.491	2.210					
142.6	-0.21	2.238	0.505	2.228					
35.1	-0.34	2.272	0.527	2.255					
8.9	-0.36	2.308	0.552	2.290					

Note:

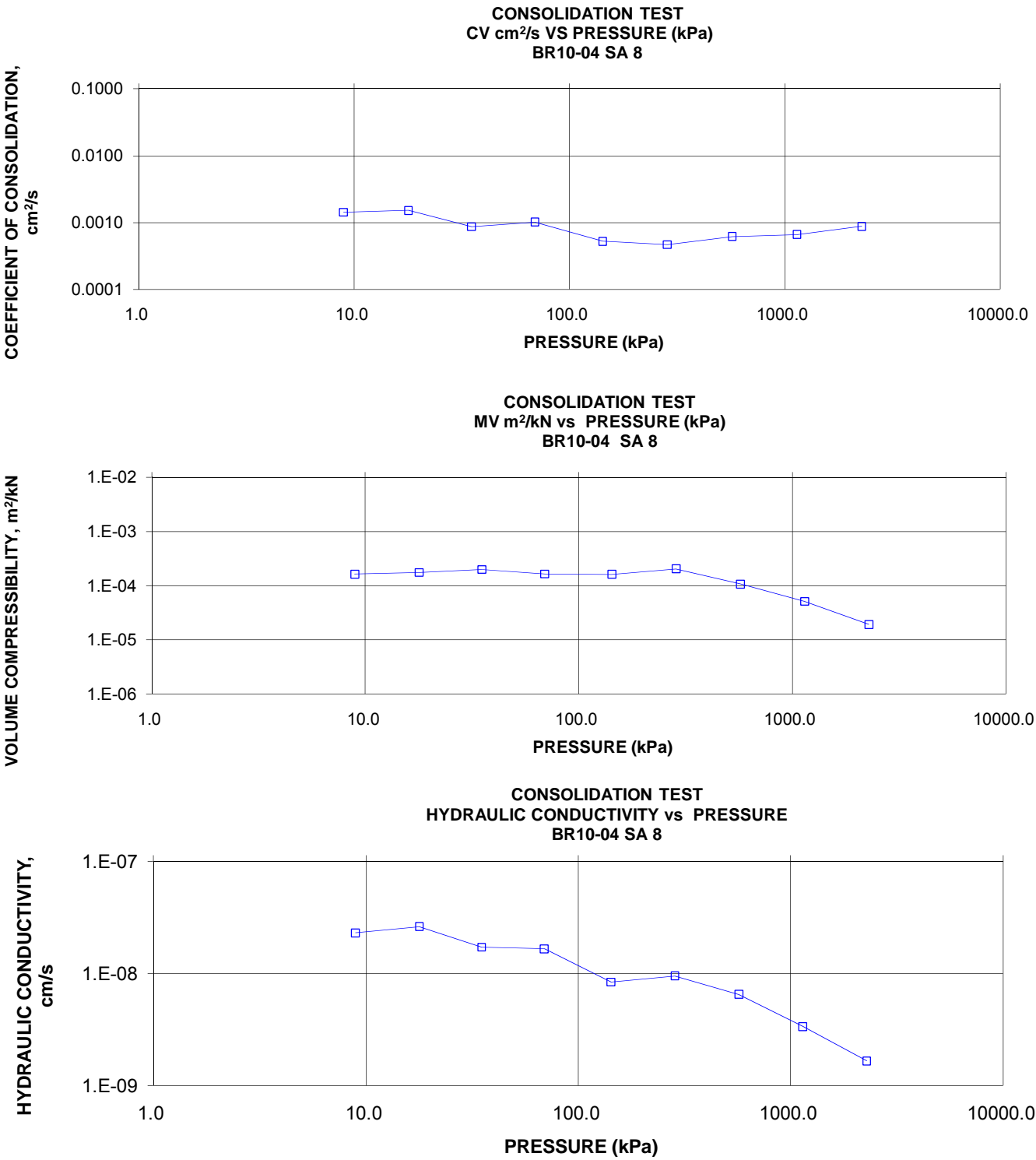
k calculated using cv based on t₉₀ values.**SAMPLE DIMENSIONS AND PROPERTIES - FINAL**

Sample Height, cm	2.31	Unit Weight, kN/m ³	19.96
Sample Diameter, cm	6.36	Dry Unit Weight, kN/m ³	16.79
Area, cm ²	31.77	Specific Gravity, measured	2.66
Volume, cm ³	73.33	Solids Height, cm	1.487
Water Content, %	18.90	Volume of Solids, cm ³	47.26
Wet Mass, g	149.23	Volume of Voids, cm ³	26.07
Dry Mass, g	125.51		

Prepared By: TG

Golder Associates

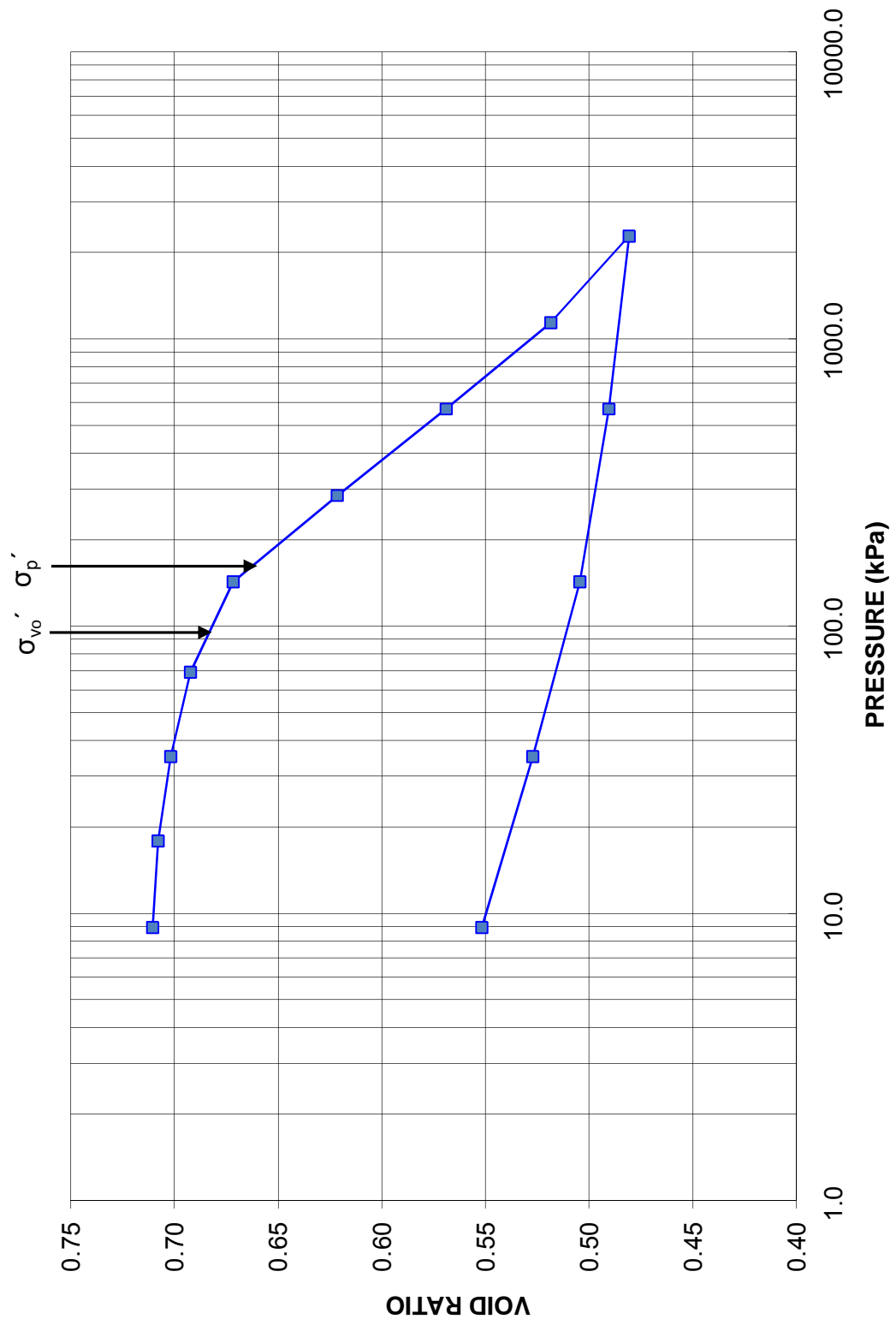
Checked By: AB



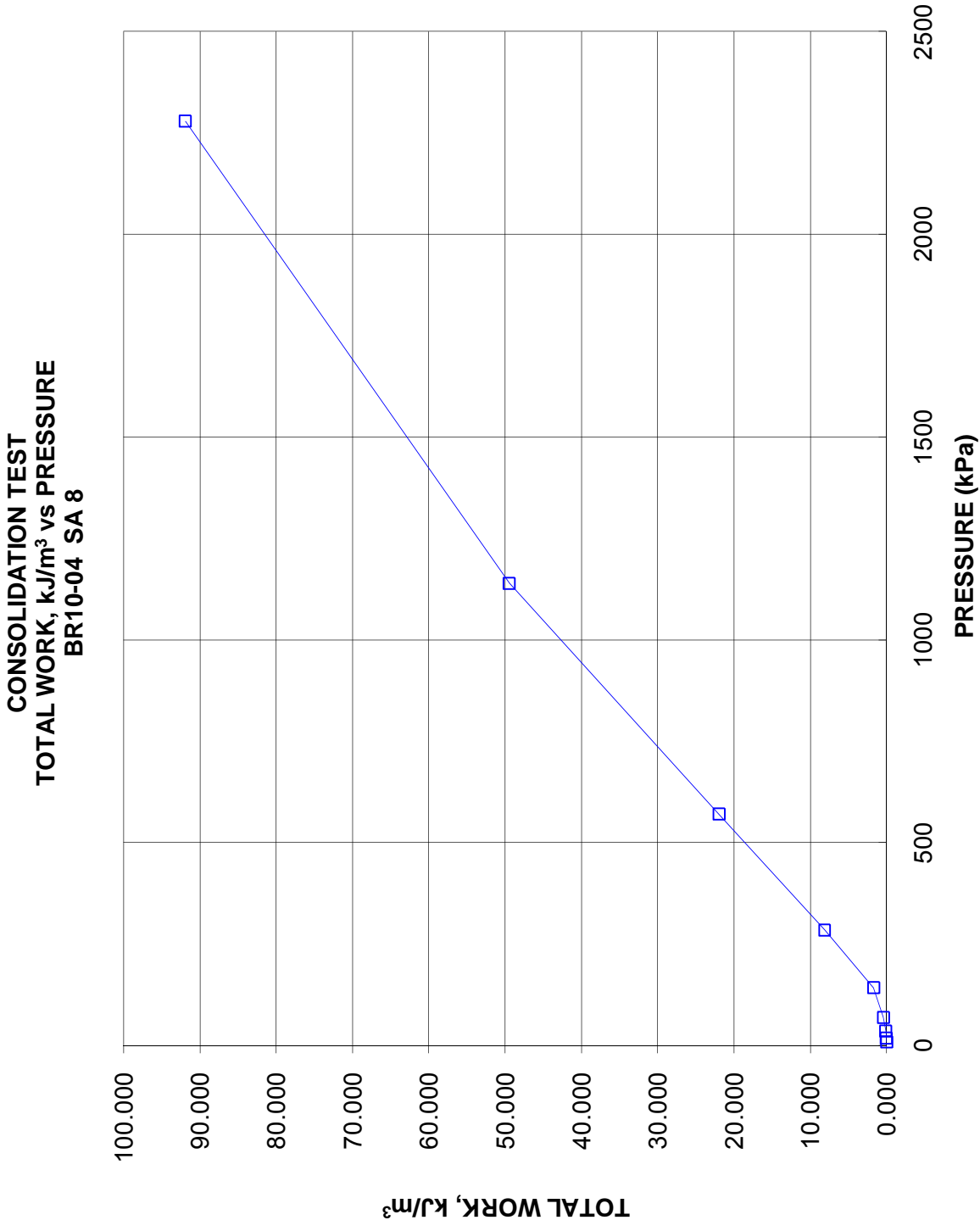
CONSOLIDATION TEST
VOID RATIO VS LOG PRESSURE

FIGURE B-10
Page 3 of 4

CONSOLIDATION TEST
VOID RATIO VS PRESSURE
BR10-04 SA 8



CONSOLIDATION TEST
TOTAL WORK VS PRESSURE



CONSOLIDATION TEST SUMMARY**FIGURE B-11**

Page 1 of 4

SAMPLE IDENTIFICATION

Project Number **09-1191-022-1**
 Borehole Number **BR10-07**

Sample Number **8**
 Sample Depth, m **6.4**

TEST CONDITIONS

Test Type Standard Load Duration, hr 24
 Date Started **8/16/10**
 Date Completed **8/30/10**

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.55	Unit Weight, kN/m ³	19.13
Sample Diameter, cm	6.36	Dry Unit Weight, kN/m ³	14.86
Area, cm ²	31.77	Specific Gravity, measured	2.69
Volume, cm ³	80.95	Solids Height, cm	1.436
Water Content, %	28.69	Volume of Solids, cm ³	45.61
Wet Mass, g	157.89	Volume of Voids, cm ³	35.34
Dry Mass, g	122.69	Degree of Saturation, %	99.6

TEST COMPUTATIONS

Pressure	Primary Consolidation	Corr. Height	Void Ratio	Average Height	t ₉₀	cv.	mv	k	Total Work
kPa	mm	cm		cm	sec	cm ² /s	m ² /kN	cm/s	kJ/m ³
0.0	0.00	2.548	0.775	2.548					
8.9	0.03	2.545	0.773	2.547	135	0.0102	1.26E-04	1.25E-07	0.005
17.9	0.03	2.543	0.771	2.544	118	0.0117	1.11E-04	1.27E-07	0.018
35.1	0.05	2.538	0.768	2.540	101	0.0135	1.08E-04	1.43E-07	0.068
69.2	0.08	2.530	0.762	2.534	194	0.0070	9.37E-05	6.43E-08	0.235
142.6	0.15	2.515	0.751	2.522	375	0.0036	8.11E-05	2.86E-08	0.870
284.9	0.63	2.452	0.708	2.483	375	0.0035	1.73E-04	5.91E-08	6.200
570.5	0.55	2.397	0.670	2.424	240	0.0052	7.56E-05	3.85E-08	15.794
1139.7	0.50	2.347	0.635	2.372	86	0.0138	3.41E-05	4.62E-08	33.453
2279.0	0.50	2.298	0.601	2.323	60	0.0191	1.71E-05	3.19E-08	69.499
570.5	-0.16	2.314	0.612	2.306					
142.6	-0.28	2.343	0.632	2.328					
35.1	-0.36	2.379	0.657	2.361					
8.9	-0.32	2.410	0.679	2.395					

Note:

k calculated using cv based on t₉₀ values.**SAMPLE DIMENSIONS AND PROPERTIES - FINAL**

Sample Height, cm	2.41	Unit Weight, kN/m ³	18.90
Sample Diameter, cm	6.36	Dry Unit Weight, kN/m ³	15.71
Area, cm ²	31.77	Specific Gravity, measured	2.69
Volume, cm ³	76.58	Solids Height, cm	1.436
Water Content, %	20.28	Volume of Solids, cm ³	45.61
Wet Mass, g	147.57	Volume of Voids, cm ³	30.97
Dry Mass, g	122.69		

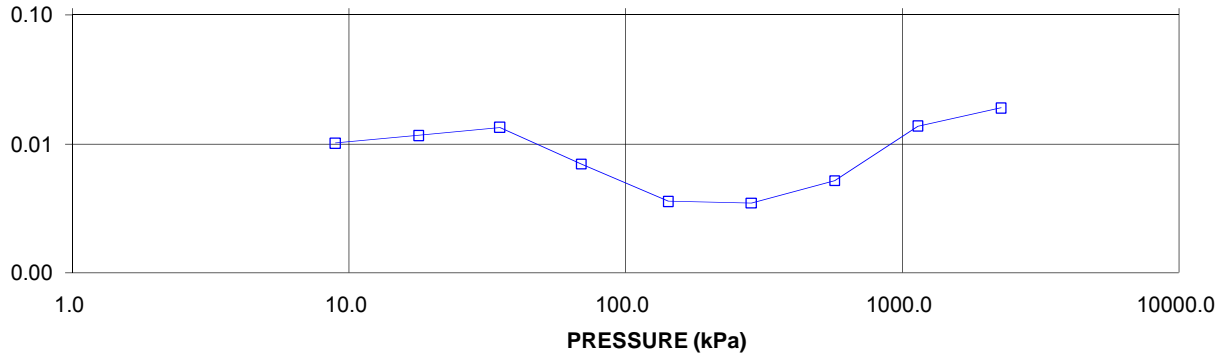
Prepared By: TG

Golder Associates

Checked By: AB

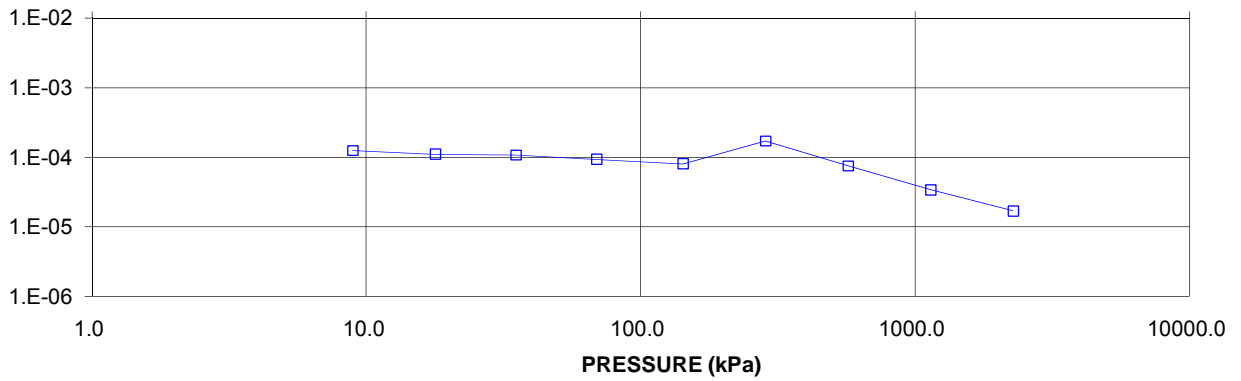
COEFFICIENT OF CONSOLIDATION,
cm²/s

CONSOLIDATION TEST
CV cm²/s VS PRESSURE (kPa)
BR10-07 SA 8



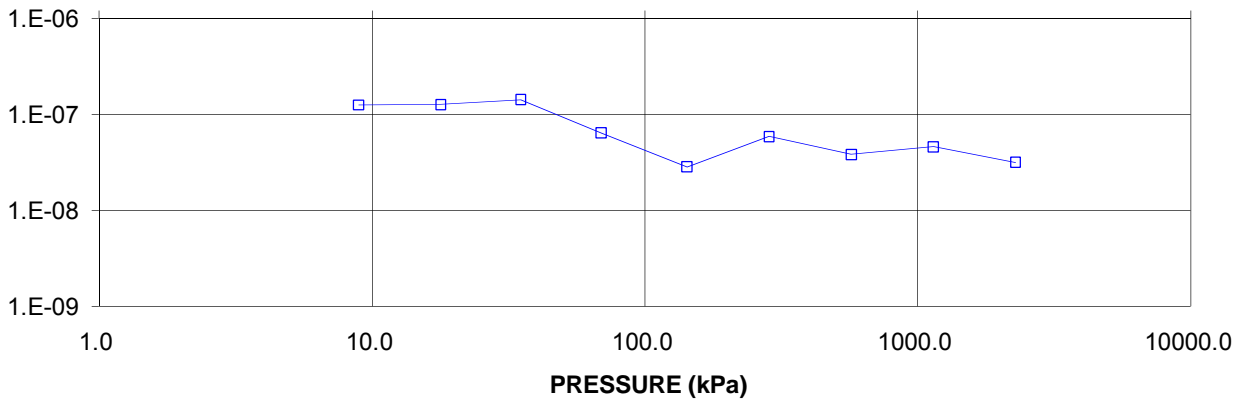
VOLUME COMPRESSIBILITY, m²/kN

CONSOLIDATION TEST
MV m²/kN vs PRESSURE (kPa)
BR10-07 SA 8



HYDRAULIC CONDUCTIVITY,
cm/s

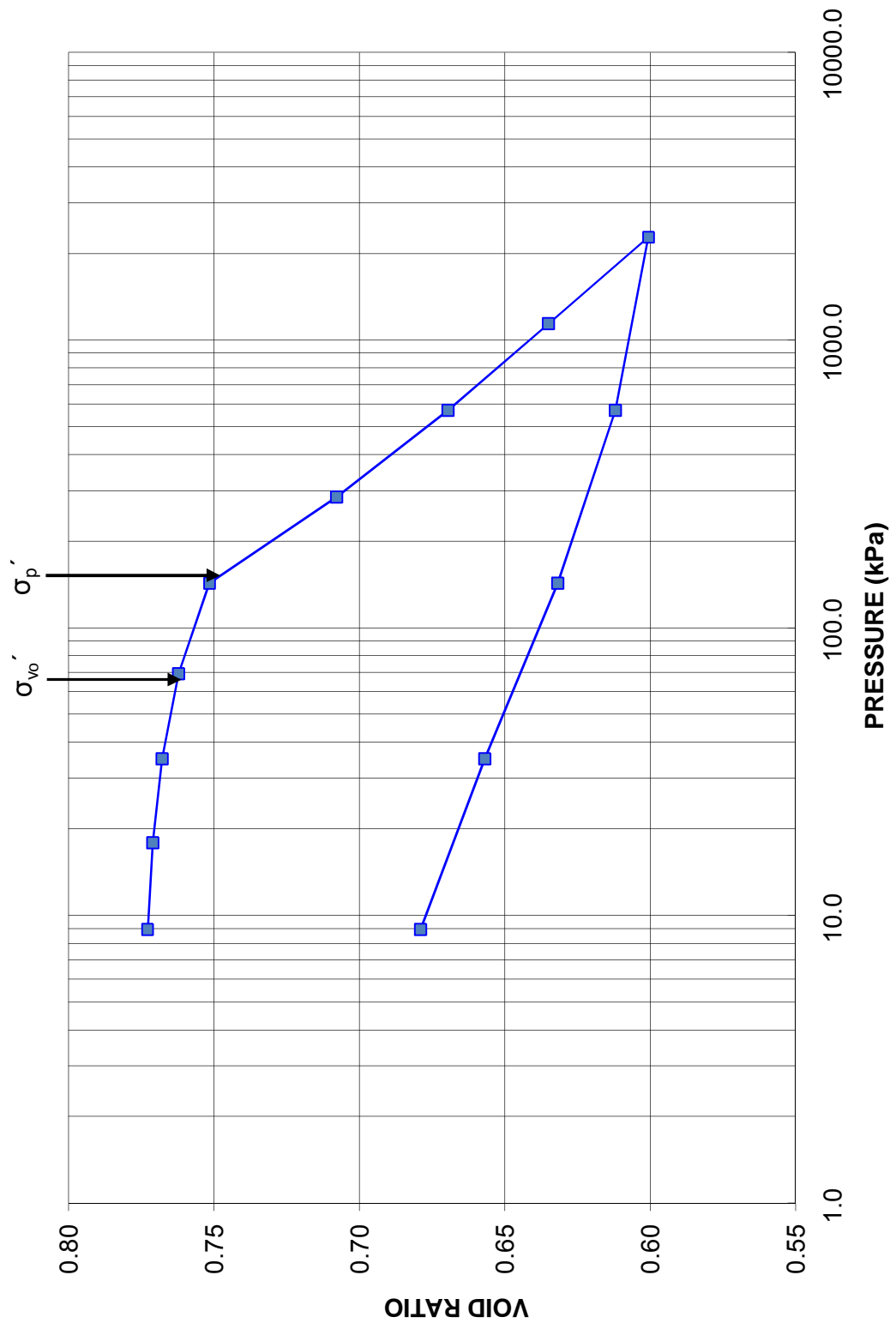
CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BR10-07 SA 8



**CONSOLIDATION TEST
VOID RATIO VS LOG PRESSURE**

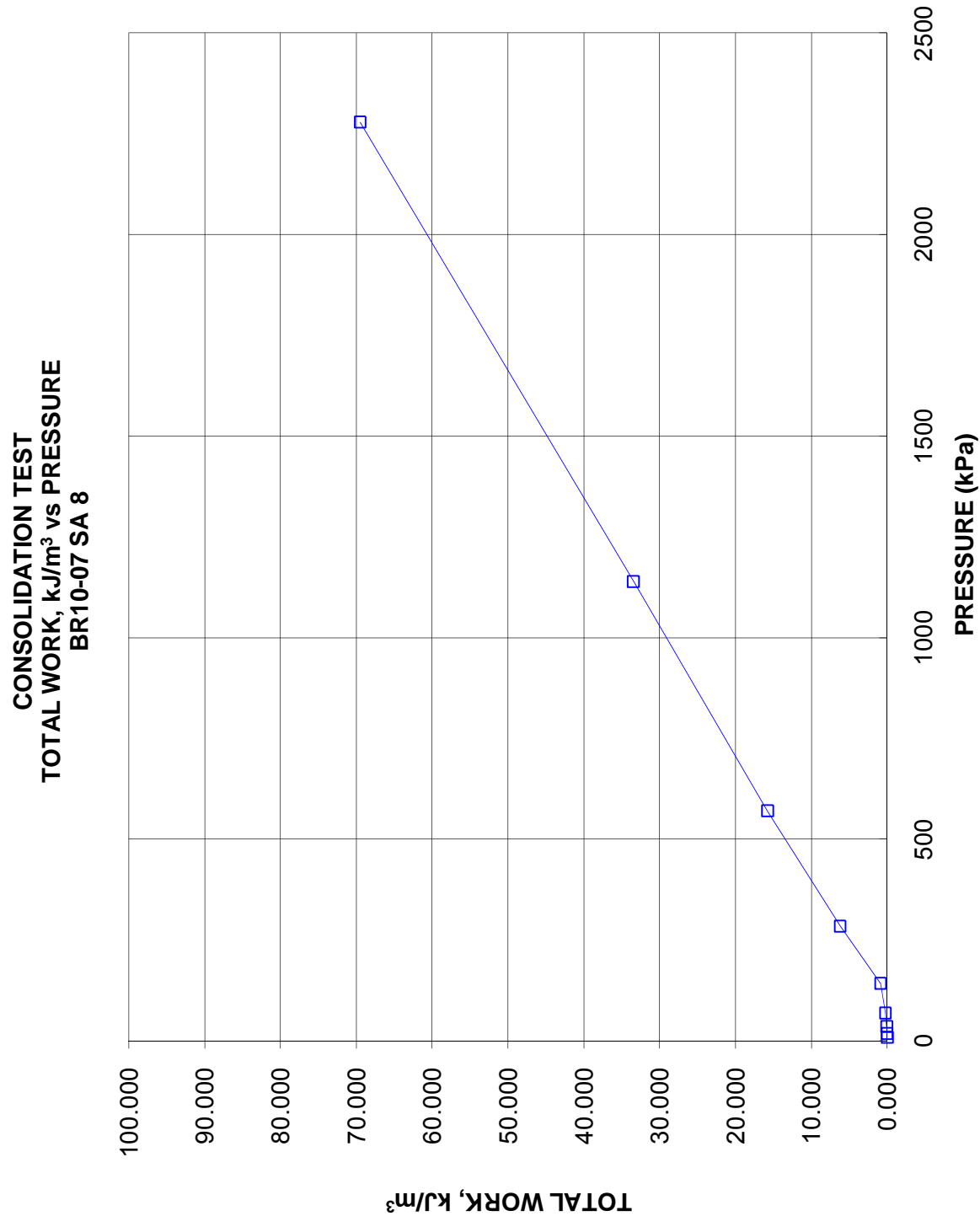
FIGURE B-11
Page 3 of 4

**CONSOLIDATION TEST
VOID RATIO vs PRESSURE
BR10-07 SA 8**



CONSOLIDATION TEST
TOTAL WORK VS PRESSURE

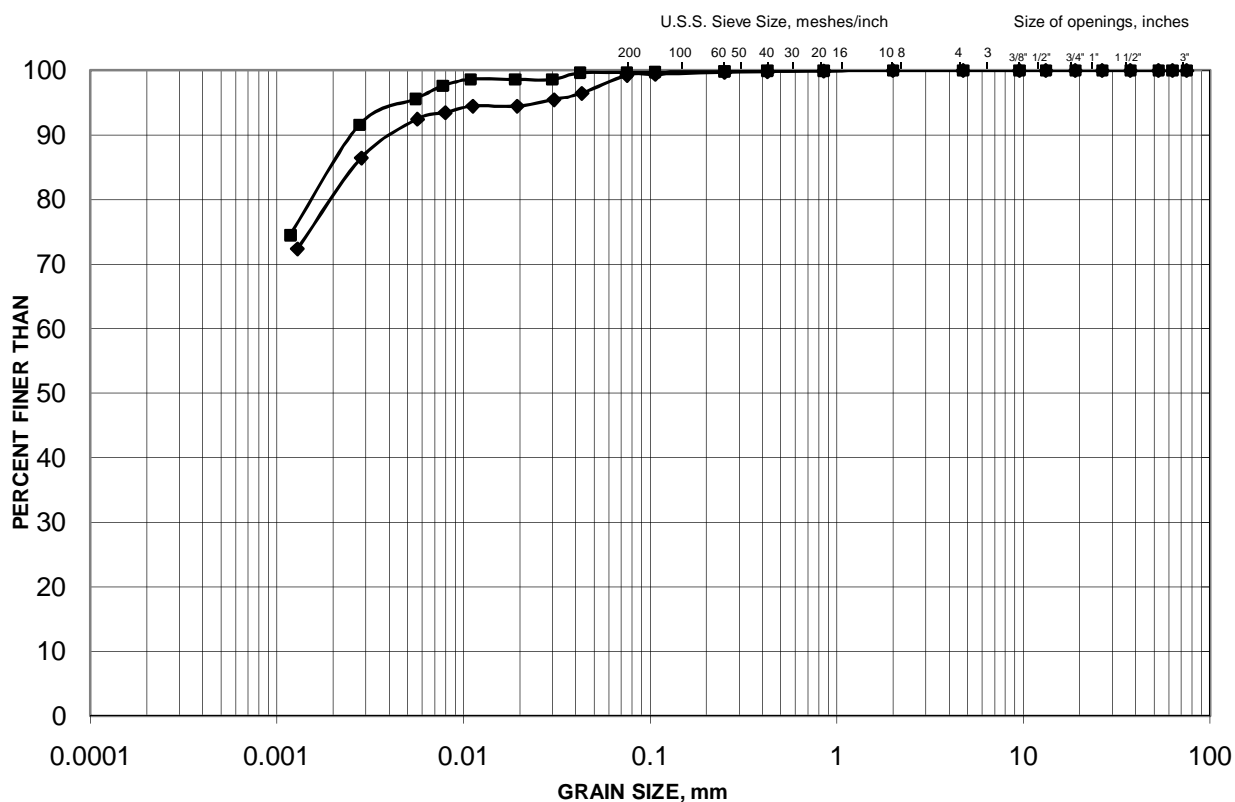
FIGURE B-11
Page 4 of 4



GRAIN SIZE DISTRIBUTION

Silty Clay to Clay

FIGURE
B-12



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

LEGEND

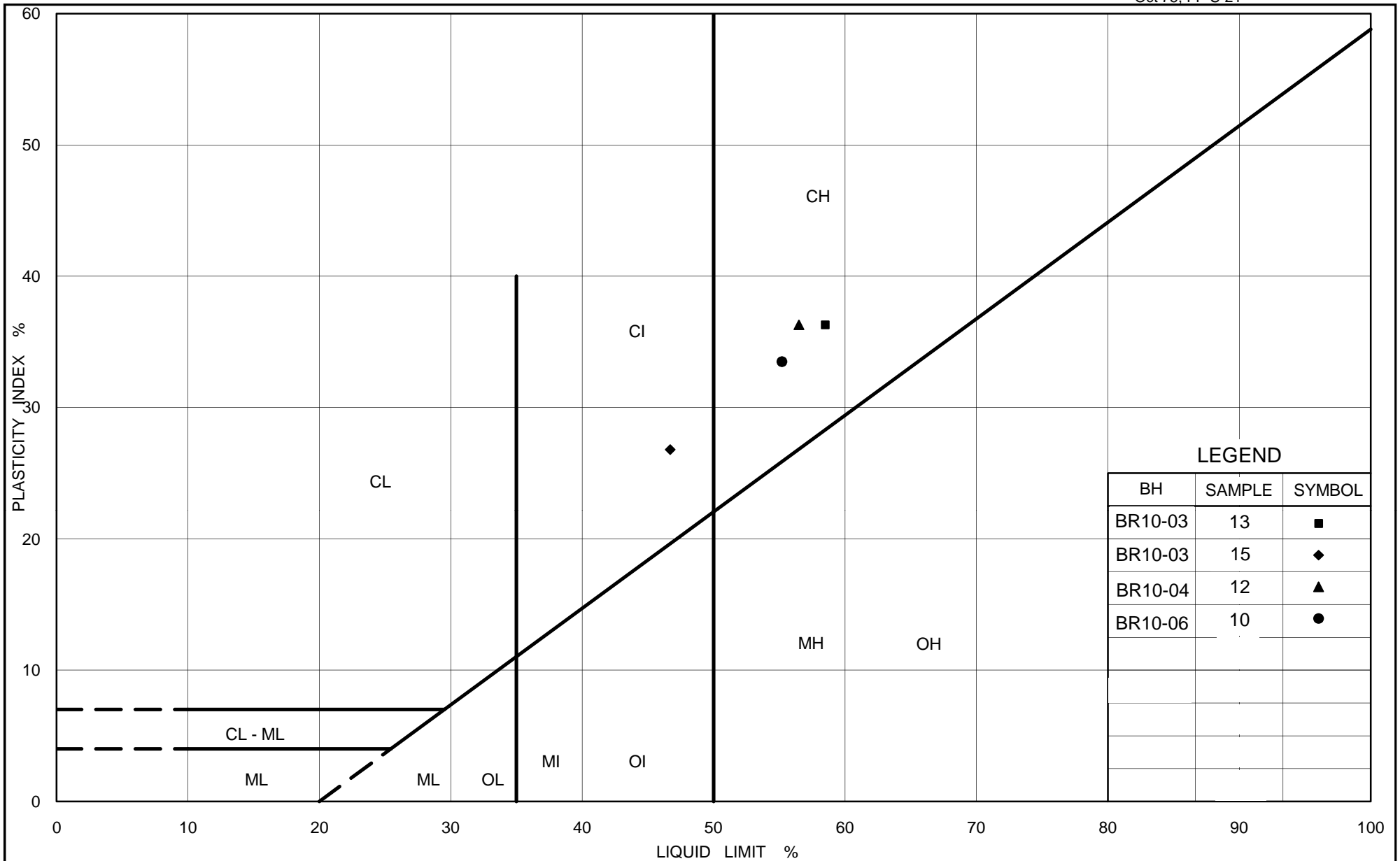
SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
■	BR10-03	13	232.8
◆	BR10-06	10	234.5

Project Number: 09-1191-0022-1

Checked By: SEMC

Golder Associates

Date: February 2011



Ministry of Transportation
Ontario

PLASTICITY CHART

Silty Clay to Clay

Figure B-13

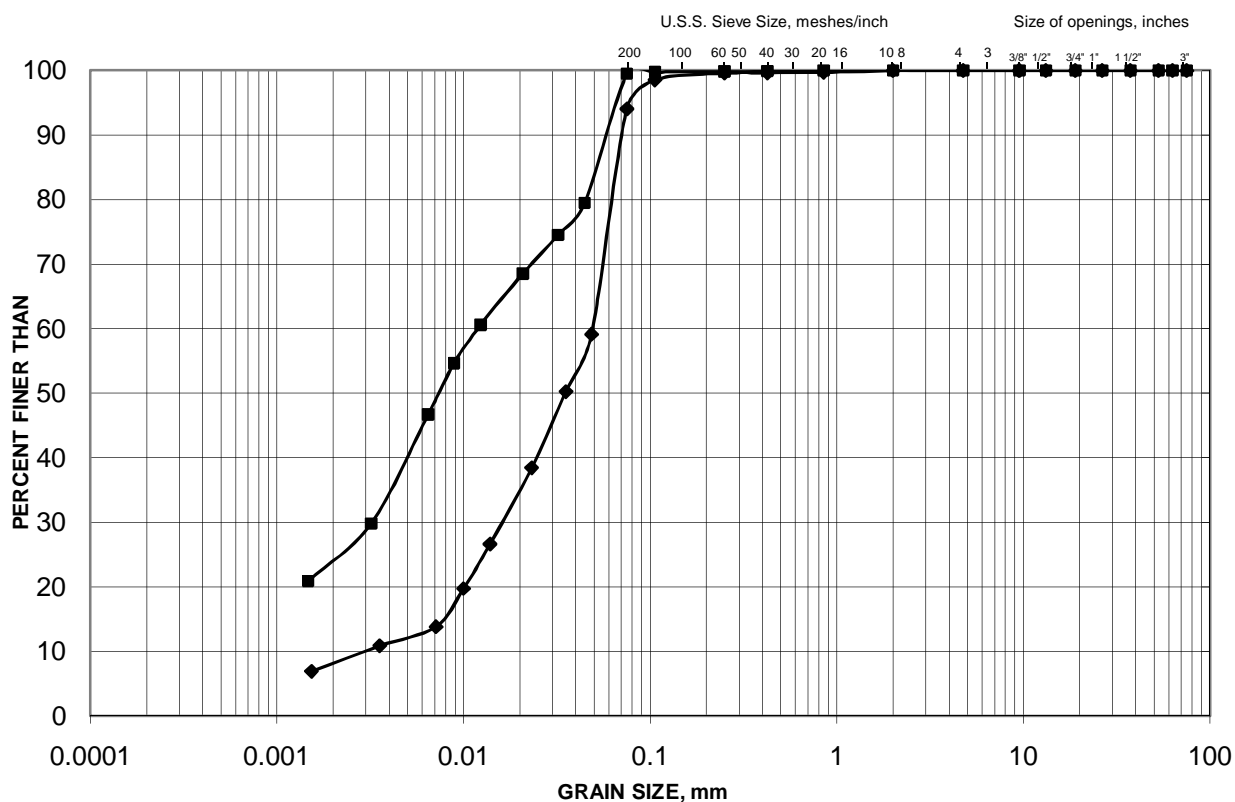
Project No. 09-1191-0022-1

Checked By: SEMC

GRAIN SIZE DISTRIBUTION

Silt

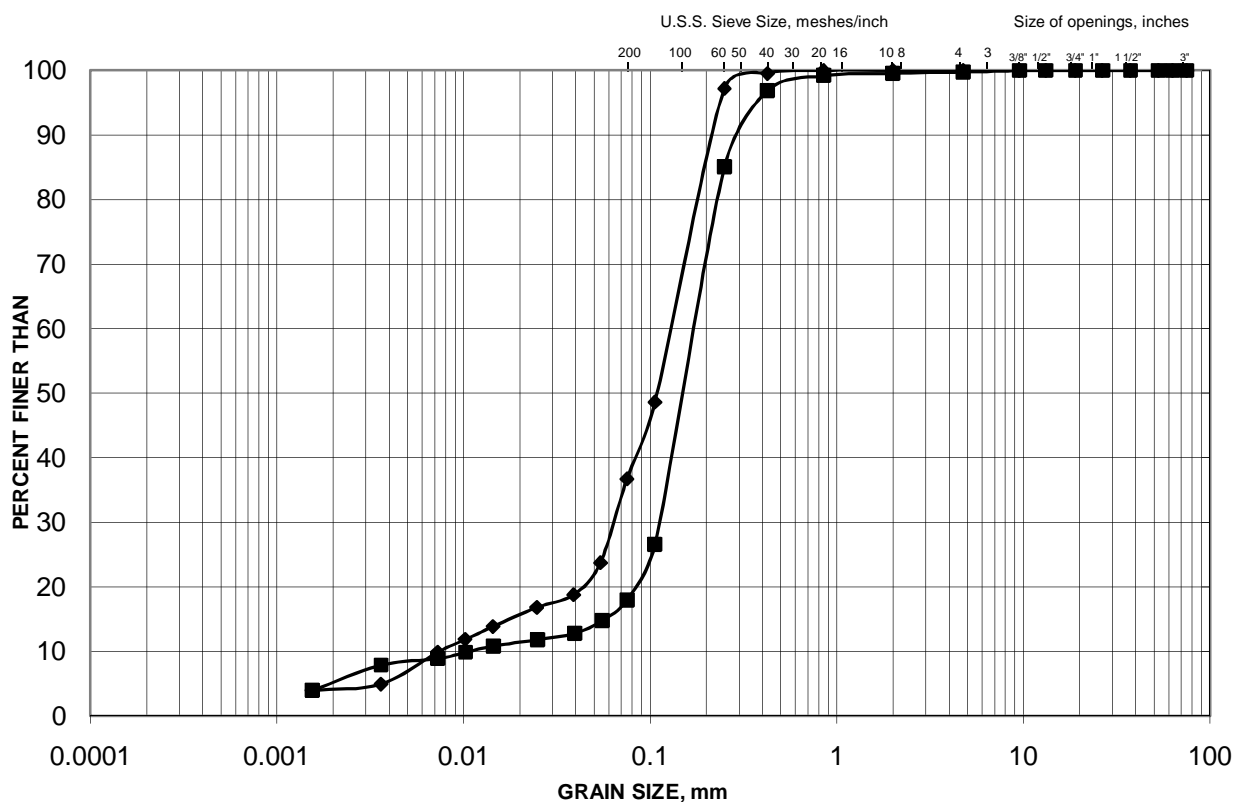
FIGURE
B-14



GRAIN SIZE DISTRIBUTION

Sand to Sandy Silt

FIGURE
B-15



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
■	BR10-03	18	217.6
◆	BR10-04	18	212.7

Project Number: 09-1191-0022-1

Checked By: SEMC

Golder Associates

Date: February 2011



APPENDIX C

Non-Standard Special Provisions

CSP FOR INTEGRAL ABUTMENTS – Item No.

Non-Standard 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 Administrator, 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.

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 Weight
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. Install piles by driving to the design tip elevation or bedrock if end-bearing piles are selected.
4. Place loose sand into the CSP.
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 perimeters of the top of all CSPs at each abutment are at the elevation and orientation shown on the working drawings.

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.

PILE CAPACITY VERIFICATION PROCEDURE - Item No.

Non-Standard Special Provision

Scope

The Contractor shall commence assessment of the pile capacity by the Hiley Formula (Standard Structural Drawing SS-103-11) once the pile reaches a depth of 1.5 m above the design pile tip elevation shown in the Contract Drawings and assess the ultimate axial resistance of the pile using the Hiley Formula at 0.5 m intervals of depth until the ultimate axial resistance is achieved. If the ultimate capacity as determined by the Hiley Formula is not achieved within the 1.5 m interval down to the design pile tip elevation the Contractor shall stop pile driving and notify the Contract Administrator. At this depth the pile should be allowed to rest for 48 hours, and the Hiley Formula should then be applied immediately upon re-striking of the pile. If the ultimate capacity is still not achieved after the 48 hour wait period, the Contract Administrator shall be notified and authorization given prior to driving the pile below the design pile tip elevation.

References

OPSS 903

Basis of Payment

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

END OF SECTION

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

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

solutions@golder.com
www.golder.com

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

