

**FOUNDATION INVESTIGATION AND
DESIGN REPORTS, RESTOULE RIVER
BRIDGE AT HAWTHORNE DRIVE,
TOWNSHIP OF PATTERSON,
DISTRICT 54, SUDBURY, ONTARIO
G.W.P. 5112-06-00, GEOCRE 31L-145**

Giffels Associates Ltd / IBI Group

TRANETOB10388AA-AA
June 22, 2011

June 22, 2011

Giffels Associates Ltd / IBI Group
30 International Boulevard
Toronto, Ontario M9W5P3

Attention: Mr. Ted Brumfitt, P.Eng.


Dear Mr. Brumfitt,

**RE: Foundation Investigation and Design Reports - Restoule River Bridge at Hawthorne Drive,
Township of Patterson, District 54, Sudbury, Ontario, G.W.P. No. 5112-06-00**

Coffey Geotechnics Inc (Coffey) is pleased to present the Foundation Investigation and Design Reports for the proposed replacement of Restoule River bridge located at Site 44-012, Hawthorne Drive, Township of Patterson, Sudbury Area, Ontario.

Please call us on 416 213 1255 should you require further clarification on any aspects of the reports.

For and on behalf of Coffey Geotechnics Inc.


Delfa Sarabia, M.Eng.
Senior Geotechnical Engineer


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Principal Engineer



Zuhtu Ozden, P.Eng.

Senior Principal

Distribution: Original held by Coffey Geotechnics Inc.
1 hard copy to Giffels Associates Ltd / IBI Group
1 hard copy to MTO Project Manager
1 hard copy to MTO Pavements and Foundation Section

**FOUNDATION INVESTIGATION REPORT
RESTOULE RIVER BRIDGE AT
HAWTHORNE DRIVE, TOWNSHIP OF
PATTERSON, DISTRICT 54,
SUDBURY, ONTARIO
G.W.P. 5112-06-00, GEOCRE 31L-145**

Giffels Associates Ltd / IBI Group

TRANETOB10388AA-AA
June 22, 2011

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FOUNDATION INVESTIGATION REPORT
RESTOULE RIVER BRIDGE AT HAWTHORNE DRIVE
TOWNSHIP OF PATTERSON, DISTRICT 54, SUDBURY, ONTARIO
G.W.P. 5112-06-00

1 INTRODUCTION

At the request of Giffels Associates Ltd / IBI Group, Coffey Geotechnics Inc. (Coffey) has prepared this foundation investigation report for the proposed replacement of Restoule River bridge (Bridge Site 44-012), at Hawthorne Drive in Restoule in the Township of Patterson. The site is located about 30 m west of Hawthorne Drive and Highway 534 intersection, about 10 km south east of Restoule Provincial Park and about some 40 km west of Highway 11, Powassan, Ontario. The foundation investigation was generally carried out in accordance with Coffey proposal (Reference PO9124, dated January 30, 2009) and the requirements of the RFP.

We understand that prior to replacement of the bridge, a temporary detour bridge will be constructed to the north of the existing bridge.

The purpose of the investigation was to obtain information about the subsurface conditions at the site by means of boreholes, and to assess the engineering characteristics of the subsurface soils by means of field and laboratory tests.

This report provides factual information concerning subsurface conditions, in situ test results and laboratory test results, based on the foundation investigation undertaken.

2 SITE DESCRIPTION AND PHYSIOGRAPHY

2.1 Site Description

The Site is located at Site 44-012 on Hawthorne Drive, about 30 m west of Highway 534 intersection in the Township of Patterson, Sudbury Area.

The existing bridge over Restoule River, constructed in 1921, is a single span structure containing concrete 'T' beam deck embedded into abutments. The total length of the bridge is 13.4 m and the total width is 4.88 m (4.06 m width from curb to curb). Presently the concrete beams and deck soffit are cracked and delaminated.

The existing approach embankments, which are approximately 1 to 2 m high, exhibit neither apparent signs of instability nor excessive erosion.

Rock outcrops were observed on the river banks.

Photographs of the Site are presented in Appendix C.

2.2 Physiography

According to the Physiography of Southern Ontario by L.J. Chapman and D.F. Putnam, 1984, the project site is located within the Physiographic Region known as the Algonquin Highlands. The Quaternary overburden deposits found in this area are generally shallow but thickness over the bedrock varies greatly over short distances, resulting from geological processes associated with glacial, glaciofluvial, and glaciolacustrine conditions. Much of this region is underlain by Precambrian rocks.

According to Bedrock Geology of Ontario Map 2544, the bedrock underlying this area consists of Mesoproterozoic Precambrian rocks (i.e. approximately 0.9 to 1.6 billion years old) of the Central Gneiss Belt that mainly consist of migmatitic rocks and gneisses of undetermined protolith. These rocks are described as commonly layered biotite gneisses and migmatites; but quartzofeldspathic gneisses, orthogneisses, and paragneisses are also locally contacted.

3 METHOD OF INVESTIGATION

3.1 Fieldwork

The fieldwork for the investigation was carried out in September 2010 and comprised of drilling 14 boreholes (BH1 to BH12, BHC1 and BHC2) at the locations shown on the Borehole Location Plan, Drawing 1. Table 1 below presents a summary of the borehole details.

Table 1: Borehole Details

Borehole No.	Station	Offset from Hawthorne Drive C/L	Existing Ground Elevation (m)	Drilled Depth (m)
BH1	11+818	3.5 m Left of C/L	227.2	2.3
BH2	11+828	3.4 m Left of C/L	226.6	6.4
BH3	11+862	2.5 m Right of C/L	226.4	6.1
BH4	11+871	4.1 m Right of C/L	226.6	2.4
BH5	11+818	11.1 m Left of C/L	225.0	2.6
BH6	11+834	10.3 m Left of C/L	223.3	4.5
BH7	11+863	7.2 m Left of C/L	226.2	2.4
BH8	11+866	10.4 m Left of C/L	226.2	2.6
BH9	11+831	8.9 m Left of C/L	223.8	1.7

Borehole No.	Station	Offset from Hawthorne Drive C/L	Existing Ground Elevation (m)	Drilled Depth (m)
BH10	11+861	7.5 m Left of C/L	226.1	5.9
BH11	11+823	3.0 m Right of C/L	226.8	2.4
BH12	11+862	5.4 m Left of C/L	226.3	2.4
BHC1	11+836	7.1 m Left of C/L	223.2	0.8
BHC2	11+851	5.6 m Left of C/L	223.0	1.0

Boreholes BH1 to BH12 were drilled using a Bombardier, a track mounted drilling rig. The borehole drilling was carried out by Landcore, a drilling subcontractor. Each borehole was advanced using hollow stem augers within the overburden, to depths of about 1.1 to 2.7 m below the ground surface. Standard Penetration Tests (SPTs) were carried out in the overburden at selected depth intervals, to assess the soil strength and obtain samples for logging and testing purposes. SPTs were carried out in general accordance with ASTM D1586. The test consists of freely dropping a 63.5 kg hammer a vertical distance of 0.76 m to drive a 51 mm outside diameter (OD) split-barrel (SS-split-spoon) sampler into the ground. The number of blows of the hammer required to drive the sampler into the relatively undisturbed ground by a vertical distance of 0.30 m is recorded as the Standard Penetration Resistance or the N-value of the soil which is indicative of the compactness condition of granular (or cohesionless) soils (gravels, sands and silts) or the consistency of cohesive soils (clays and clayey soils). In BH2, BH3, BH6 and BH10, rock was cored to 3.4 to 3.9 m below the overburden or to depths of 4.5 to 6.4 m below the ground surface, using NQ coring technique.

Boreholes BHC1 and BHC2 were drilled primarily for rock coring purposes as these boreholes are located near the edge of the river where rock outcrops are present. Rock coring in these boreholes was carried out to 0.8 to 1.0 m below the ground surface, using a portable coring drill.

Rock core samples were boxed and colour photographed.

The soil and rock samples were described in the field, placed in appropriate containers, labelled and transported to our Etobicoke geotechnical laboratory where the samples underwent further detailed visual examination and samples were selected for geotechnical laboratory testing.

Groundwater levels and inflows observed in the open boreholes during drilling were recorded. The boreholes were grouted upon their completion using a cement/bentonite mixture, as per MTO procedures.

The borehole locations were located using existing site features. The borehole location coordinates and ground elevations were subsequently measured by Client's surveyors and were provided to Coffey.

A Coffey representative was present during the drilling operations to direct sampling and testing, record test results and log materials encountered.

Appendix A presents the Record of Borehole Sheets and rock core photographs.

3.2 Laboratory Testing

Soil and rock samples obtained during the investigation were taken to our Etobicoke laboratory. The following tests were performed on selected soil samples:

- Natural moisture content tests;
- Grain size analyses (sieve);
- Grain size analyses (sieve and hydrometer tests); and
- Atterberg Limits tests.

Point Load Strength Index (11 tests) and Uniaxial Compressive Strength (UCS) tests (2 tests) were performed on selected rock core samples.

Appendix B presents laboratory test results sheets for all the tests carried out except the natural moisture content results as they are presented on the Record of Borehole Sheets in Appendix A.

4 SUBSURFACE CONDITIONS

Detailed descriptions of the materials encountered in the boreholes are presented on the Record of Borehole Sheets in Appendix A, which also includes rock core photographs. Explanation of Terms Used in Report is presented in Appendix D.

Drawings 1 and 2 present generalized subsurface profiles along the proposed bridge replacement and temporary detour bridge.

In general, below topsoil and fill (including pavement fill and embankment fill), the site is underlain by native soils consisting of mainly silty sand to sandy silt and sand and gravel. Below the native soils, bedrock was encountered in the boreholes, which was described as gneiss. The surface of the bedrock was inferred or proven at depths ranging from 1.1 to 2.7 m below the existing ground surface for boreholes drilled in the vicinity of the Hawthorne Drive (BH1 to BH12) and at the ground surface for boreholes drilled on the edge of the river (BHC1 and BHC2) or at Elevations 224.9 to 222.1 m.

The Record of Borehole Sheets and sections indicate the subsurface conditions only at the borehole locations. Note that the material boundaries indicated on the logs are approximate and based on visual observations. These boundaries typically represent a transition from one material type to another and should not be regarded as an exact plane of geological change. It should be pointed out that the subsurface conditions may vary across this Site.

The following summarizes the surface conditions encountered in the boreholes.

4.1 Topsoil

Topsoil was encountered at the ground surface in BH5, BH6, BH9 and BH10 which were drilled generally outside of Hawthorne Drive roadway. The thickness of topsoil at these borehole locations ranged from 0.15 to 0.30 m.

Note that in our experience, the thickness of organic rich soils frequently varies in between and beyond borehole locations.

4.2 Fill

The fill encountered in the boreholes can be classified into two categories, namely pavement fill (asphalt and granular fill) and embankment fill.

4.2.1 Pavement Fill

For boreholes drilled on Hawthorne Drive paved surface (BH1, BH7, BH8, BH11 and BH12), asphalt was encountered at the surface. The recorded thickness of asphaltic concrete at the borehole locations ranged from about 35 to 80 mm.

Below the asphalt or the ground surface, a 0.2 to 0.8 m thick granular pavement fill was encountered in all the boreholes except for BH5, BH6, BH9, BH10, BHC1 and BHC2 and was described as sand and gravel.

Grain size distribution analyses carried out on two samples taken from the granular pavement fill indicate the following distribution, as shown in Figure B-1, in Appendix B.

Gravel:	40 %
Sand:	57 – 58 %
Silt and Clay:	2 – 3 %

Standard Penetration Tests yielded SPT N-values of 7 to 12 blows/0.3 m within the granular pavement fill layer, indicating a loose to compact condition.

4.2.2 Embankment Fill

Below the pavement fill (BH2, BH3, BH4, BH7, BH8, BH11 and BH12) or topsoil (BH10), embankment fill was encountered. The embankment fill was found to consist of silty sand to sandy silt with traces to some clay content. The embankment fill was found to extend to depths of 1.2 to 2.1 m below the existing road surface or to Elevations ranging between 225.6 m and 224.5 m (probably representing the original ground surface level before the existing bridge was constructed minus the stripped material thickness).

Grain size distribution analyses carried out on nine samples taken from the embankment fill indicate the following distribution, as shown in Figure B-2, in Appendix B.

Gravel:	0 – 11 %
Sand:	24 – 62 %
Silt:	24 – 57 %
Clay:	6 – 20 %

Atterberg Limits tests conducted on three selected samples from the finer embankment fill soil (i.e. sandy silt with some clay content), where clay content found to be higher based on tactile and visual examinations, indicated the following results, also shown in Figure B-3, in Appendix B.

Liquid Limit:	20 – 25 %
Plastic Limit:	16 – 19 %
Plasticity Index:	4 – 6 %

Based on the test results above, the embankment fill is considered to be generally granular (non-cohesive) material with some finer zones (with a relatively higher clay content) exhibiting a low plasticity (i.e. a CL-ML material). It should be noted that Atterberg limits test for this type of granular material is not reliable, if clay content is low.

SPT N-values of 6 to 11 blows/0.3 m were recorded within this embankment fill indicating loose to compact but generally a loose condition.

4.3 Natural Overburden

Underneath the embankment fill or topsoil, granular (non-cohesive) natural overburden soils were encountered at Elevations 226.4 to 223.0 m with thicknesses of about 0.4 to 2.3 m. The overburden soils typically consist, at most borehole locations, fine grained granular soils comprising silty sand to sandy silt but at some borehole locations, the overburden was found to consist of coarser granular soils (i.e. sand & gravel at BH8 and BH10). The basically granular overburden soils were found to extend to rock or inferred rock (i.e. auger refusal) at Elevations 224.9 to 222.1 m.

The following are the grain size distributions of the selected six samples taken from silty sand and sand deposits, as presented in Figure B-4, in Appendix B.

Gravel:	3 – 12 %
Sand:	40 – 60 %
Silt:	18 – 41 %
Clay:	6 – 15 %

Coarser granular materials (sand & gravel) were encountered in BH8 and BH10. The following are the grain size distributions of the sand & gravel samples from BH8 and BH10, as presented in Figure B-5, in Appendix B.

Gravel:	36 – 46 %
Sand:	38 – 48 %
Silt and Clay:	16 %

Standard Penetration Tests yielded SPT N-values of 3 to in excess of 100 blows/0.3 m within these granular soils, indicating varying relative densities from very loose to very dense. SPT blow counts corresponding to very loose to loose conditions were taken within the just below the embankment fill in BH3 and BH12 and within the upper 1 m in Borehole BH9 drilled outside the Hawthorne Drive. Generally, these native granular soils are considered to have a compact relative density and becoming dense to very dense near the surface of the bedrock.

4.4 Bedrock

Bedrock was proven by coring in BH2, BH3, BH6, BH10, BHC1 and BHC2, where the top of bedrock was encountered at depths of 0 to 2.7 m below the existing ground surface or at Elevations 224.1 (BH2) to 222.2 m (BH6). The rock was cored to 0.8 m to 3.9 m depths below the top of bedrock surface. The remaining boreholes (i.e. BH1, BH4, BH5, BH7, BH8, BH9, BH11 and BH12) encountered auger refusal on possible bedrock at depths of 1.7 to 2.6 m below the existing ground surface or at Elevations 224.9 to 222.1m. From these observations, the surface of the bedrock along the existing Hawthorne Drive seems to be dipping down mildly towards the River (about 5-10%). Bedrock sloping perpendicular to the existing road on the east side of the existing bridge seems to be mild (about 10% dipping), while bedrock sloping on the west side of the bridge seems to be dipping relatively sharp towards north (about 20 to 30%). This should be confirmed during the construction.

Based on the rock cores recovered, the bedrock was described as a grey gneiss with granitic mixture. The recorded total core recovery (TCR) ranged from 63 to 100 %. Rock Quality Designation (RQD) values of 45 to 98 % were recorded. The recorded lower values of the RQD were affected by the near vertical joints present within the bedrock. Based on these values, the rock mass quality can be described as poor to excellent but typically fair to excellent.

Point Load Strength Index and Uniaxial Compressive Strength (UCS) tests were performed on the selected rock cores. The test results are presented in Appendix B. $I_{s(50)}$ values ranging from 1.4 to 12.3 MPa and UCS values of 92.3 to 166.7 MPa were recorded. Based on these results, the rock encountered at the site is classified as typically R4 to R6 (strong to extremely strong) with R3 (medium strong) recorded in BHC2 at 0.5 m. The ratio between UCS and $I_{s(50)}$ values were calculated on samples tested at adjacent depths. $UCS/I_{s(50)}$ (axial) ratio of 14.3 to 20.3 and $UCS/I_{s(50)}$ (diametral) ratio of 12.6 to 17.0 were calculated.


4.5 Groundwater Conditions

Groundwater conditions in the open boreholes were observed during drilling operations within the overburden and no groundwater was observed. Based on these observations, the groundwater level at the time of our investigations was below the bedrock surface. In selected boreholes, the rock was cored and as a result water was introduced in the boreholes during the coring operations which prevented reliable groundwater observations.


We measured the water level in the River from the existing bridge on October 18, 2010 and it was found to be 4.2 m below the crown of road surface or at about Elevation 222.4 m, which is below the bedrock surface level, recorded at most borehole locations.

It should be noted that groundwater levels are subject to variations due to the influence of rainfall, temperature, local drainage, seasons and other factors. In addition, the groundwater level at the site could also be influenced by the water level in the River.

For and on behalf of Coffey Geotechnics Inc.


Delfa Sarabia, M.Eng.
Senior Geotechnical Engineer


Ramon Miranda, P.Eng.
Manager, Transportation


Zuhtu Ozden, P.Eng.
Senior Principal

Drawings

NOTES:

FOR DETAILED SUBSURFACE CONDITIONS
REFER TO RECORD OF BOREHOLE SHEETS.

METRIC

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

CONT No.

GWP: 5112-06-01

RESTOULE RIVER BRIDGE AT
HAWTHORNE DRIVE
BOREHOLE LOCATION PLAN
AND SOIL STRATA 1 OF 2



SHEET

coffey geotechnics
SPECIALISTS MANAGING THE EARTH



KEY PLAN
N.T.S.

LEGEND

- Borehole
- Corehole
- N Blows/0.3m (Std. Pen. Test, 475 J/blow)

No.	ELEVATION	EASTING	NORTHING
BH1	227.2	5075.7	5009.0
BH2	226.6	5085.4	5009.4
BH3	226.4	5119.4	5001.7
BH4	226.6	5127.9	4998.8
BH5	225.0	5075.6	5016.7
BH6	223.3	5091.7	5016.4
BH7	226.2	5122.3	5011.1
BH8	228.2	5125.8	5013.8
BH9	223.8	5089.0	5014.7
BH10	226.1	5011.7	5120.3
BH11	226.8	5081.0	5002.8
BH12	226.3	5120.4	5009.6
BHC1	223.2	5094.0	5013.0
BHC2	223.0	5109.1	5011.0

-NOTE-

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

NOTE: This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

REVISIONS	DATE	BY	DESCRIPTION

Geocres No. 31L-145

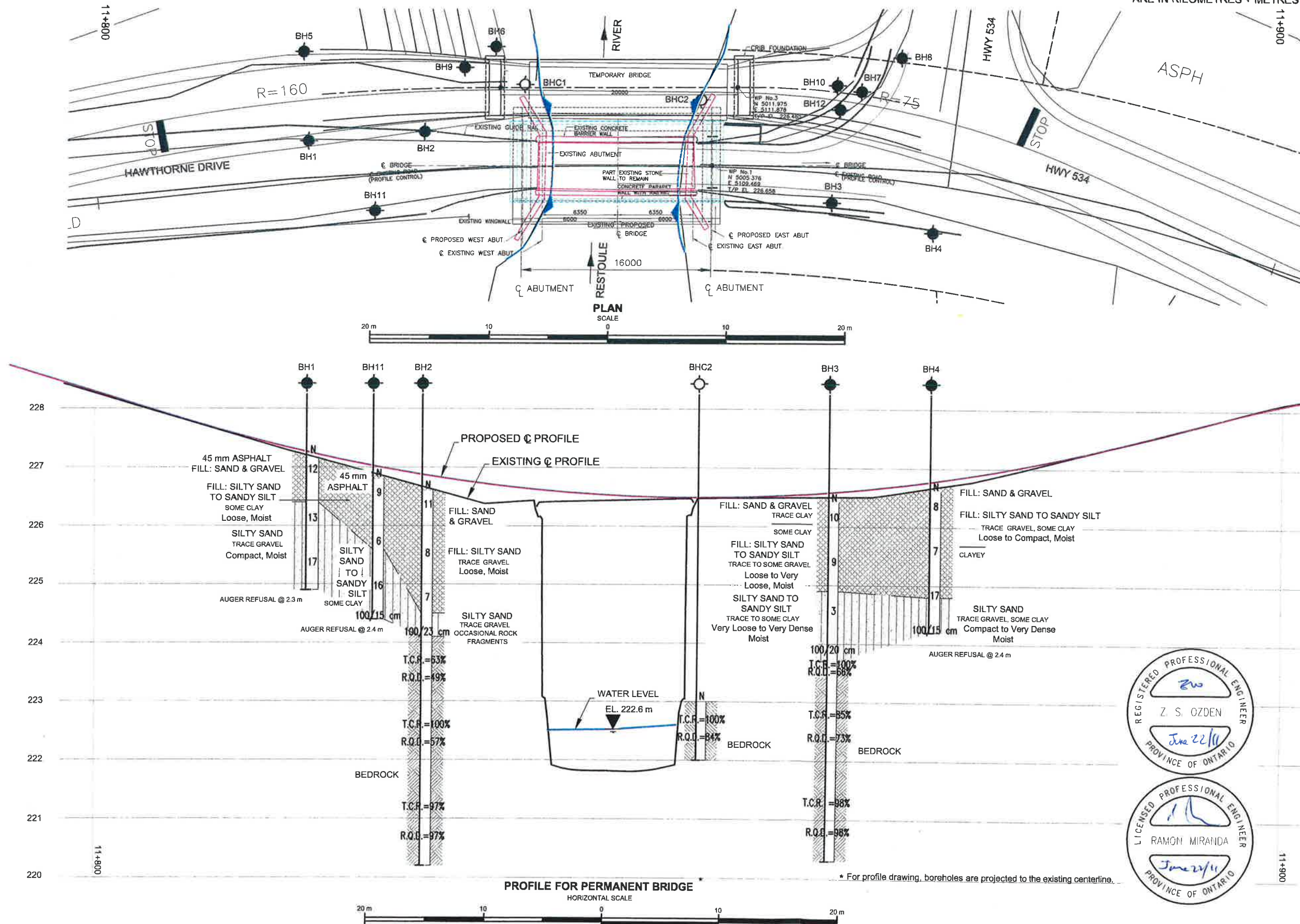
SUBMD	CHECKED	DATE	June 16, 2011	SITE
DRAWN	SH	CHECKED	RM	APPROVED



PROFILE FOR PERMANENT BRIDGE

HORIZONTAL SCALE

* For profile drawing, boreholes are projected to the existing centerline.



NOTES:
FOR DETAILED SUBSURFACE CONDITIONS
REFER TO RECORD OF BOREHOLE SHEETS.

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

CONT No.
GWP: 5112-06-01

RESTOULE RIVER BRIDGE AT
HAWTHORNE DRIVE
BOREHOLE LOCATION PLAN
AND SOIL STRATA 2 OF 2

SHEET

coffey geotechnics
SPECIALISTS MANAGING THE EARTH



LEGEND

- Borehole
- Corehole
- N Blows/0.3m (Std. Pen. Test, 475 J/blow)

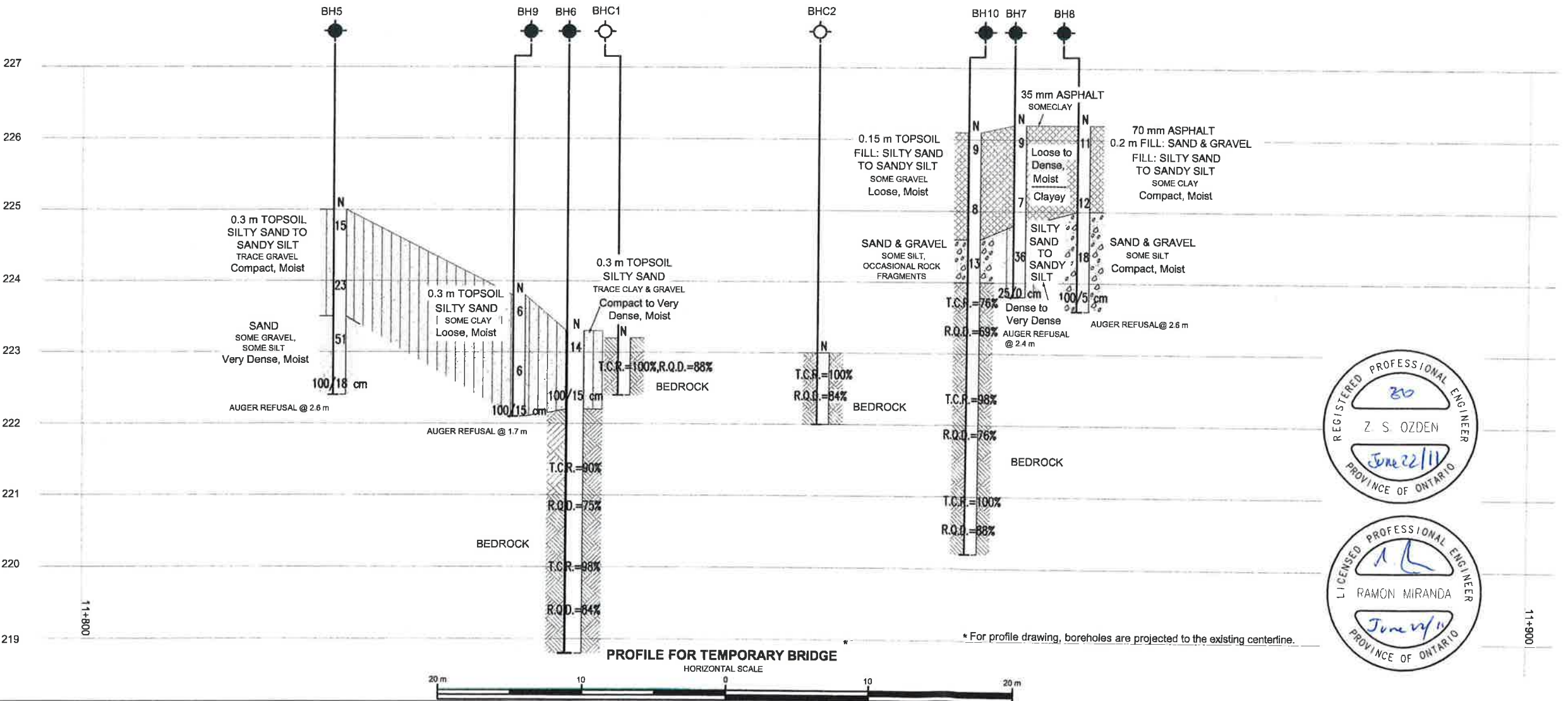
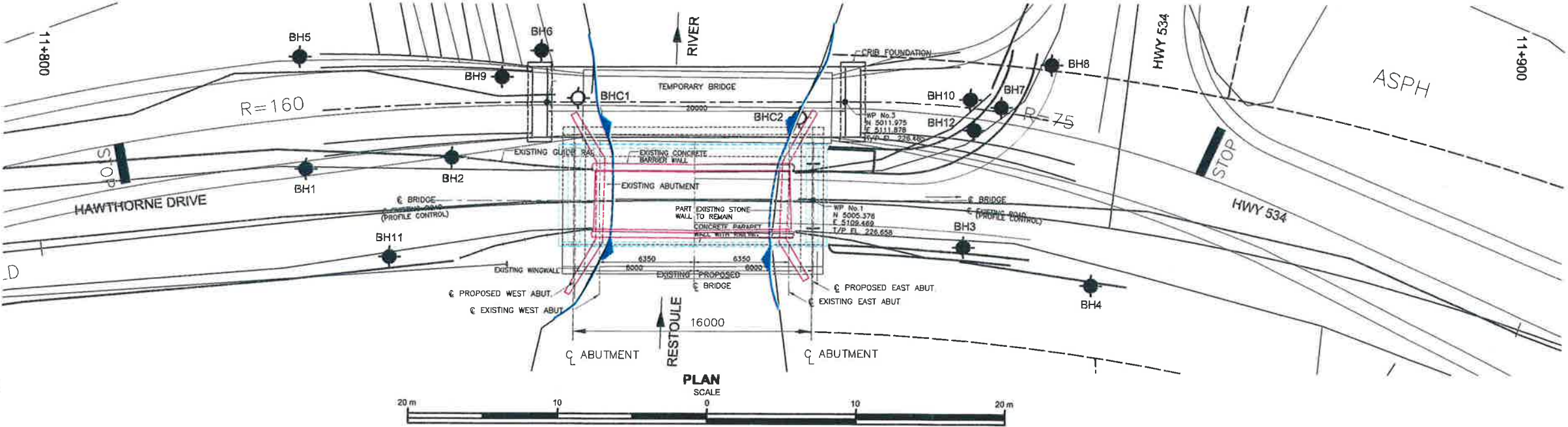
No	ELEVATION	EASTING	NORTHING
BH1	227.2	5075.7	5009.0
BH2	226.8	5085.4	5009.4
BH3	226.4	5119.4	5001.7
BH4	226.6	5127.9	4998.8
BH5	225.0	5075.6	5016.7
BH6	223.3	5091.7	5016.4
BH7	226.2	5122.3	5011.1
BH8	226.2	5125.8	5013.8
BH9	223.8	5089.0	5014.7
BH10	226.1	5011.7	5120.3
BH11	226.8	5081.0	5002.8
BH12	226.3	5120.4	5009.6
BHC1	223.2	5094.0	5013.0
BHC2	223.0	5109.1	5011.0

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

NOTE: This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

REVISIONS	DATE	BY	DESCRIPTION

Geocres No. 31L - 145			
TRANET010388AA			
SUBMIT	CHECKED	DATE	SITE
DRAWN	SH	CHECKED RM	APPROVED ZO
DWG		2	



* For profile drawing, boreholes are projected to the existing centerline.

Appendix A

Record of Borehole Sheets and Rock Core Photographs

TRANETOB10388AA: Restoule River Bridge

RECORD OF BOREHOLE No 1

1 OF 1

METRIC

GWP G.W.P 5112-06-00 LOCATION 11+818, 3.5 m Lt C/L of Hawthorne Drive (E 5075.7, N 5009.0) ORIGINATED BY LG
 DIST HWY 534 BOREHOLE TYPE Hollow Stern Auger COMPILED BY SK
 DATUM Geodetic DATE 9/8/2010 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L		
227.2 0.0	GROUND SURFACE													
	45 mm ASPHALT		1	SS	12		227							
	FILL: Sand and Gravel, compact													
226.4 0.8			2	SS	13		226							7 60 25 8
	SILTY SAND trace gravel brown, compact, moist		3	SS	17									
224.9 2.3	End of Borehole Auger refusal @ 2.3 m probably on bedrock Borehole dry (not stabilized)* and caved-in @ 1.6 m upon completion						225							

TRANETOB10388AA: Restoule River Bridge

RECORD OF BOREHOLE No 2

1 OF 1

METRIC

GWP G.W.P 5112-06-00 LOCATION 11+828, 3.4 m Lt C/L of Hawthorne Drive (E 5085.4, N 5009.4) ORIGINATED BY LG
 DIST HWY 534 BOREHOLE TYPE Hollow Stem Auger, NQ Coring, Wash Boring COMPILED BY SK
 DATUM Geodetic DATE 9/8/2010 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
226.6	GROUND SURFACE													
0.0	FILL: Sand and Gravel, compact		1	SS	11		226							40 58 (2)
225.8														
0.8	FILL: Silty Sand trace gravel brown, loose, moist		2	SS	8		225							5 56 30 9
			3	SS	7									
224.5														
2.1	SILTY SAND													
224.1	trace gravel, occ. rock fragments, v. dense		4	SS	100 / 23 cm		224							Auger refusal and Start of NQ coring at 2.6 m
2.5														
			5	RC	T.C.R.=63% R.Q.D.=49%		223							
	BEDROCK grey, gneiss		6	RC	T.C.R.=100% R.Q.D.=57%		222							
			7	RC	T.C.R.=97% R.Q.D.=97%		221							
220.2														
6.4	End of Borehole @ 6.4 m Borehole dry (not stabilized)* before coring and caved-in @ 2.6 m													

TRANETOB10388AA: Restoule River Bridge

RECORD OF BOREHOLE No 3

1 OF 1

METRIC

GWP G.W.P 5112-06-00 LOCATION 11+862, 2.5 m Rt C/L of Hawthorne Drive (E 5119.4, N 5001.7) ORIGINATED BY LG
 DIST HWY 534 BOREHOLE TYPE Hollow Stem Auger, NQ Coring, Wash Boring COMPILED BY SK
 DATUM Geodetic DATE 9/9/2010 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
226.4	GROUND SURFACE													
226.0	FILL: Sand and Gravel, compact		1	SS	10		226							11 54 29 6
0.2	tr. clay													
	some clay		2	SS	9									1 32 50 17
224.9	FILL: Silty Sand to Sandy Silt tr. to some gravel brown, loose, moist						225							
1.5	SILTY SAND TO SANDY SILT tr. to some clay, brown, wet		3	SS	3									4 40 41 15
	v. loose													Auger refusal and
	v. dense		4	SS	100 / 20 cm		224							Start of NQ coring
223.7														at 2.7 m
2.7	BEDROCK grey, gneiss		5	RC	T.C.R.=100% R.Q.D.=66%		223							UCS=92.3 MPa
			6	RC	T.C.R.=85% R.Q.D.=73%		222							
			7	RC	T.C.R.=98% R.Q.D.=98%		221							
220.3														
6.1	End of Borehole @ 6.1 m Borehole dry (not stabilized)* before coring and caved-in @ 2.4 m													

+ 3, x 3 Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

TRANETOB10388AA: Restoule River Bridge

RECORD OF BOREHOLE No 4

1 OF 1

METRIC

GWP G.W.P 5112-06-00 LOCATION 11+871, 4.1 m Rt C/L of Hawthorne Drive (E 5127.9, N 4998.8) ORIGINATED BY LG
 DIST HWY 534 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SK
 DATUM Geodetic DATE 9/8/2010 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
226.6	GROUND SURFACE													
226.0	FILL: Sand and Gravel		1	SS	8		226							
0.2														
	FILL: Silty Sand to Sandy Silt trace gravel, some clay brown, loose to compact, moist		2	SS	7		226							0 24 57 19
224.8							225							
1.8	SILTY SAND tr. gravel, some clay		3	SS	17									
224.2	brown, compact to v. dense, moist													
2.4	End of Borehole Auger Refusal @ 2.4 m probably on bedrock Borehole dry (not stabilized)* and caved-in @ 2.2 m upon completion		4	SS	100 / 15 cm									

TRANETOB10388AA: Restoule River Bridge

RECORD OF BOREHOLE No 5

1 OF 1

METRIC

GWP G.W.P 5112-06-00 LOCATION 11+818, 11.1 m Lt C/L of Hawthorne Drive (E 5075.6, N 5016.7) ORIGINATED BY LG
DIST HWY 534 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SK
DATUM Geodetic DATE 9/8/2010 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
225.0	GROUND SURFACE													
0.0	0.3 m Topsoil		1	SS	15									
	SILTY SAND TO SANDY SILT trace gravel brown, compact, moist		2	SS	23									3 45 41 11
223.5														
1.5	SAND some gravel, some silt brown, v. dense, moist		3	SS	51									17 59 18 6
222.4			4	SS	100 / 18 cm									
2.6	End of Borehole Auger Refusal @ 2.6 m probably on bedrock Borehole dry (not stabilized)* and caved-in @ 1.8 m upon completion													

+³, ×³: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE



TRANETO10388AA: Restoule River Bridge

RECORD OF BOREHOLE No 6

1 OF 1

METRIC

GWP G.W.P 5112-06-00 LOCATION 11+834, 10.3 m Lt C/L of Hawthorne Drive (E 5091.7, N 5016.4) ORIGINATED BY LG
DIST HWY 534 BOREHOLE TYPE Hollow Stem Auger, NQ Coring, Wash Boring COMPILED BY SK
DATUM Geodetic DATE 9/8/2010 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
223.3 0.0	GROUND SURFACE		1	SS	14		223							10 57 23 10
222.2 1.1	0.3 m Topsoil SILTY SAND trace clay and gravel brown, compact to v. dense, moist		2	SS	100 / 15 km		222							
	BEDROCK grey, gneiss		3	RC	T.C.R. = 90% R.Q.D. = 75%		221							UCS=166.7 MPa
			4	RC	T.C.R. = 98% R.Q.D. = 84%		220							
218.8 4.5							219							
	End of Borehole @ 4.5 m Borehole dry (not stabilized)* before coring and caved-in @ 1.2 m													

TRANETOB10388AA: Restoule River Bridge

RECORD OF BOREHOLE No 7

1 OF 1

METRIC

GWP G.W.P 5112-06-00 LOCATION 11+863, 7.2 m Lt C/L of Hawthorne Drive (E 5122.3, N 5011.1) ORIGINATED BY LG
DIST HWY 534 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SK
DATUM Geodetic DATE 9/8/2010 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
226.2 0.0	GROUND SURFACE		1	SS	9		226							GR 8 SA 62 SI 24 CL 6
224.8 1.4	35 mm ASPHALT 0.2 m FILL: Sand and Gravel FILL: Silty Sand to Sandy Silt some clay, grey, loose, moist		2	SS	7		225							1 25 54 20
223.8 2.4	SILTY SAND TO SANDY SILT occ. rock fragments dense to v. dense		3	SS	36		224							
	End of Borehole Auger Refusal @ 2.4 m probably on bedrock Borehole dry (not stabilized)* and caved-in @ 2.0 m upon completion		4	SS	25 / 0 cm									

+ 3 X 3

Numbers refer to
Sensitivity

20
15 5
10

(%) STRAIN AT FAILURE

TRANETOB10388AA: Restoule River Bridge

RECORD OF BOREHOLE No 8

1 OF 1

METRIC

GWP G.W.P 5112-06-00 LOCATION 11+866, 10.4 m Lt C/L of Hawthorne Drive (E 5125.8, N 5013.8) ORIGINATED BY LG
 DIST HWY 534 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SK
 DATUM Geodetic DATE 9/9/2010 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
FLEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
226.2 0.0	GROUND SURFACE		1	SS	11		226							0 39 47 14
225.0 1.2	70 mm ASPHALT 0.2 m FILL: Sand and Gravel FILL: Silty Sand to Sandy Silt some clay grey, compact, moist		2	SS	12		225							
	SAND AND GRAVEL some silt brown, moist		3	SS	18									36 48 (16)
223.6 2.6	compact v. dense		4	SS	100 / 5 cm		224							spoon bouncing at 2.5 m
	End of Borehole Auger Refusal @ 2.6 m probably on bedrock Borehole dry (not stabilized)* and caved-in @ 2.0 m upon completion													

+ 3 x 3 : Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

TRANETOB10388AA: Restoule River Bridge

RECORD OF BOREHOLE No 9

1 OF 1

METRIC

GWP G.W.P 5112-06-00 LOCATION 11+831, 8.9 m Lt C/L of Hawthorne Drive (E 5089.0, N 5014.7) ORIGINATED BY LG
 DIST HWY 534 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SK
 DATUM Geodetic DATE 9/8/2010 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
FLEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
223.8 0.0	GROUND SURFACE													
	0.3 m Topsoil		1	SS	6									
	SILTY SAND some clay, brown, moist													
	loose		2	SS	6		223							7 54 28 11
222.1 1.7	v. dense		3	SS	100 / 15 cm									
	End of Borehole Auger Refusal @ 1.7 m probably on bedrock Borehole dry (not stabilized)* and caved-in @ 1.4 m upon completion													

+³ ×³ Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

TRANETOB10388AA: Restoule River Bridge

RECORD OF BOREHOLE No 10

1 OF 1

METRIC

GWP G.W.P 5112-06-00 LOCATION 11+861, 7.5 m Lt C/L of Hawthorne Drive (E 5120.3, N 5011.7) ORIGINATED BY LG
 DIST HWY 534 BOREHOLE TYPE Hollow Stem Auger, NQ Coring, Wash Boring COMPILED BY SK
 DATUM Geodetic DATE 9/9/2010 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
226.1 0.0	GROUND SURFACE		1	SS	9		226							
	0.15 m Topsoil FILL: Silty Sand to Sandy Silt some gravel brown, loose, moist		2	SS	8		225							
224.6 1.5	SAND AND GRAVEL some silt, occ. rock fragments compact		3	SS	13		224							
224.0 2.1	BEDROCK grey, gneiss		4	RC	T.C.R.=76% R.Q.D.=69%		223							
			5	RC	T.C.R.=98% R.Q.D.=76%		222							
			6	RC	T.C.R.=100% R.Q.D.=88%		221							
220.2 5.9	End of Borehole @ 5.9 m Borehole dry (not stabilized)* before coring and caved-in @ 0.9 m													

+³, ×³: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE



TRANETO10388AA: Restoule River Bridge

RECORD OF BOREHOLE No 11

1 OF 1

METRIC

GWP G.W.P 5112-06-00 LOCATION 11+823, 3.0 m Rt C/L of Hawthorne Drive (E 5081.0, N 5002.8) ORIGINATED BY LG
DIST HWY 534 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SK
DATUM Geodetic DATE 9/9/2010 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
226.8 0.0	GROUND SURFACE		1	SS	9									GR SA SI CL
226.2 0.6	35 mm ASPHALT FILL: Sand and Gravel													40 57 (3)
225.6 1.2	FILL: Silty Sand to Sandy Silt some clay brown, loose, moist		2	SS	6		226							0 30 51 19
	SILTY SAND TO SANDY SILT some clay, brown, moist													
	compact		3	SS	16		225							
224.4 2.4	v. dense		4	SS	100 15 cm									
	End of Borehole Auger Refusal @ 2.4 m probably on bedrock Borehole dry (not stabilized)* and caved-in @ 2.0 m upon completion													

+ 3, X 3 : Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE

TRANETO10388AA: Restoule River Bridge

RECORD OF BOREHOLE No 12

1 OF 1

METRIC

GWP G.W.P 5112-06-00 LOCATION 11+862, 5.4 m Lt C/L of Hawthorne Drive (E 5120.4, N 5009.6) ORIGINATED BY LG
 DIST HWY 534 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SK
 DATUM Geodetic DATE 9/9/2010 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
226.3 0.0	GROUND SURFACE													
	80 mm ASPHALT		1	SS	7		226							
	0.2 m FILL: Sand and Gravel													
	FILL: Sandy Silt some clay brown, loose, moist		2	SS	8		225							0 25 57 18
224.6 1.7	SILTY SAND		3	SS	9									
223.9 2.4	tr. clay, occ. rock fragments, loose		4	SS	10		224							
	End of Borehole Auger Refusal @ 2.4 m probably on bedrock Borehole dry (not stabilized)* and caved-in @ 2.1 m upon completion													

+ ³, x ³: Numbers refer to Sensitivity
 20
15 10 5 0 (%) STRAIN AT FAILURE

TRANETOB10388AA: Restoule River Bridge

RECORD OF BOREHOLE No C1

1 OF 1

METRIC

GWP	G.W.P 5112-06-00	LOCATION	11+836, 7.1 m Lt C/L of Hawthorne Drive (Under Bridge) (E 5094.0, N 5013.1)	ORIGINATED BY	LG
DIST	HWY 534	BOREHOLE TYPE	Portable Core Drill	COMPILED BY	SK
DATUM	Geodetic	DATE	9/15/2010	CHECKED BY	ZO

[illegible]

+ 3, × 3 Numbers refer to Sensitivity

(%) STRAIN AT FAILURE



TRANETOB10388AA: Restoule River Bridge

RECORD OF BOREHOLE No C2

1 OF 1

METRIC

GWP G.W.P 5112-06-00 LOCATION 11+851, 5.6 m Lt C/L of Hawthorne Drive (Under Bridge) (E 5108.8, N 5011.1) ORIGINATED BY LG
DIST HWY 534 BOREHOLE TYPE Portable Core Drill COMPILED BY SK
DATUM Geodetic DATE 9/16/2010 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL					
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			20	40						60	80	100	20	40
223.0 0.0	GROUND SURFACE						223												
222.0 1.0	BEDROCK grey, gneiss		1	RC	T.C.R. = 100% R.Q.D. = 84%		222												
	End of Corehole @ 1.0 m																		

+³, ×³ : Numbers refer to
Sensitivity

20
15 10 5 0
10 (%) STRAIN AT FAILURE

Start Coring at 2.6m

RC5
RC6 3.4m
RC7 4.9m



End of BH2 at 6.4m

BOREHOLE BH2

Start Coring at 2.7m

RC6 3.1m

RC5



RC7 4.6m

End of BH3 at 6.1m

BOREHOLE BH3



BOREHOLE BH6



BOREHOLE BH10

Start Coring at 0m



BOREHOLE BHC1

Start Coring at 0m



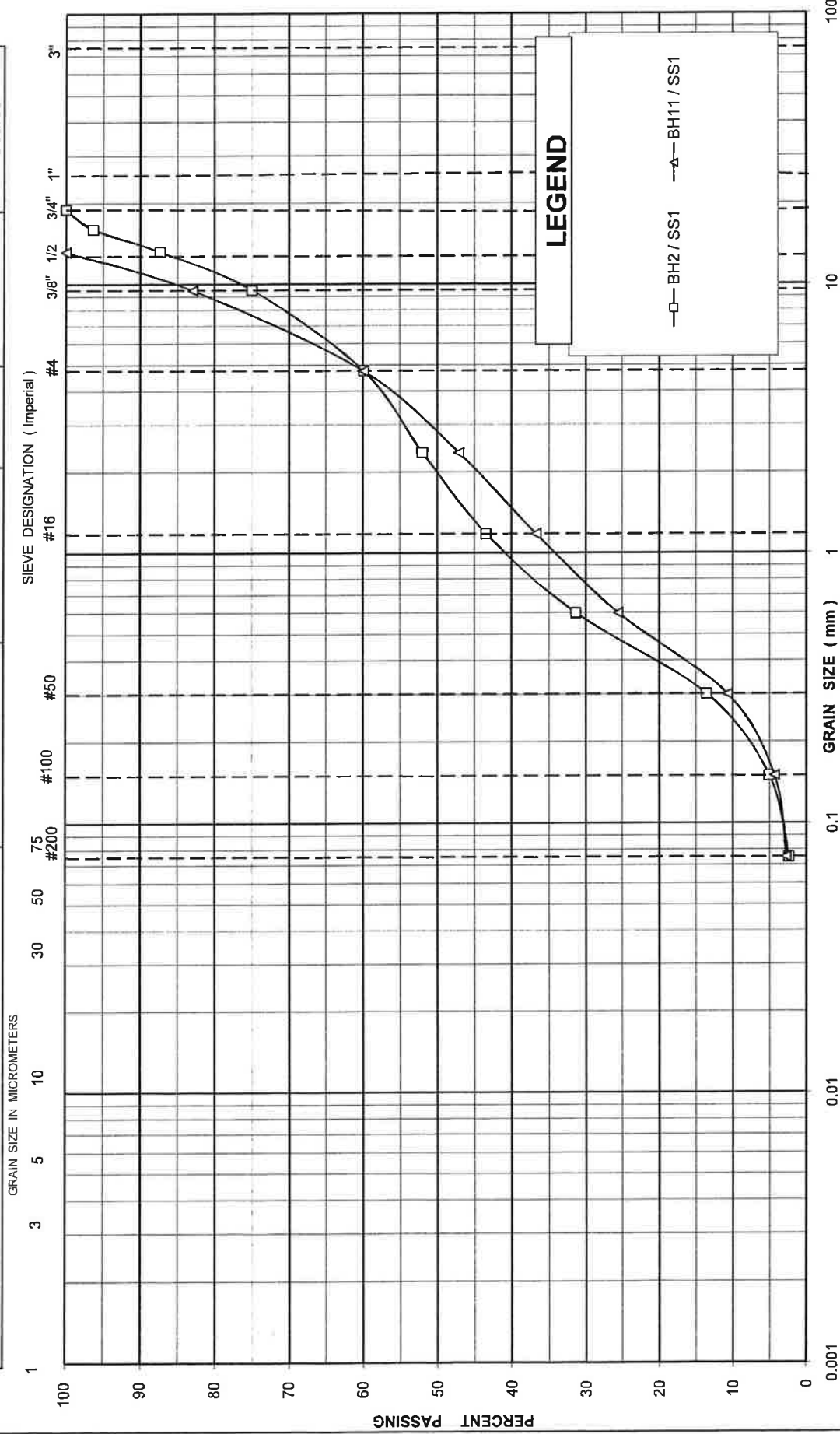
BOREHOLE BHC2

Appendix B

Laboratory Test Results

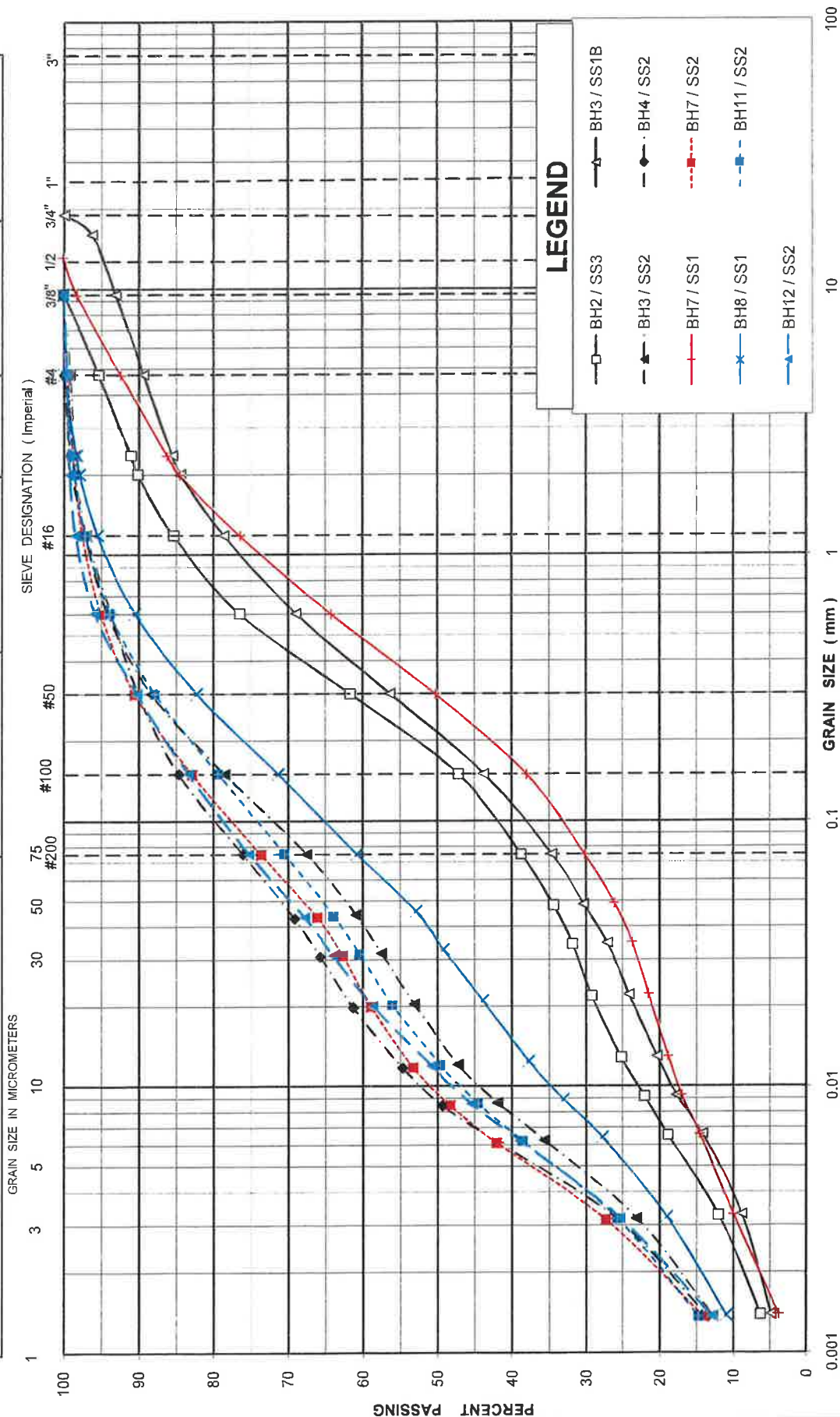
UNIFIED SOIL CLASSIFICATION SYSTEM

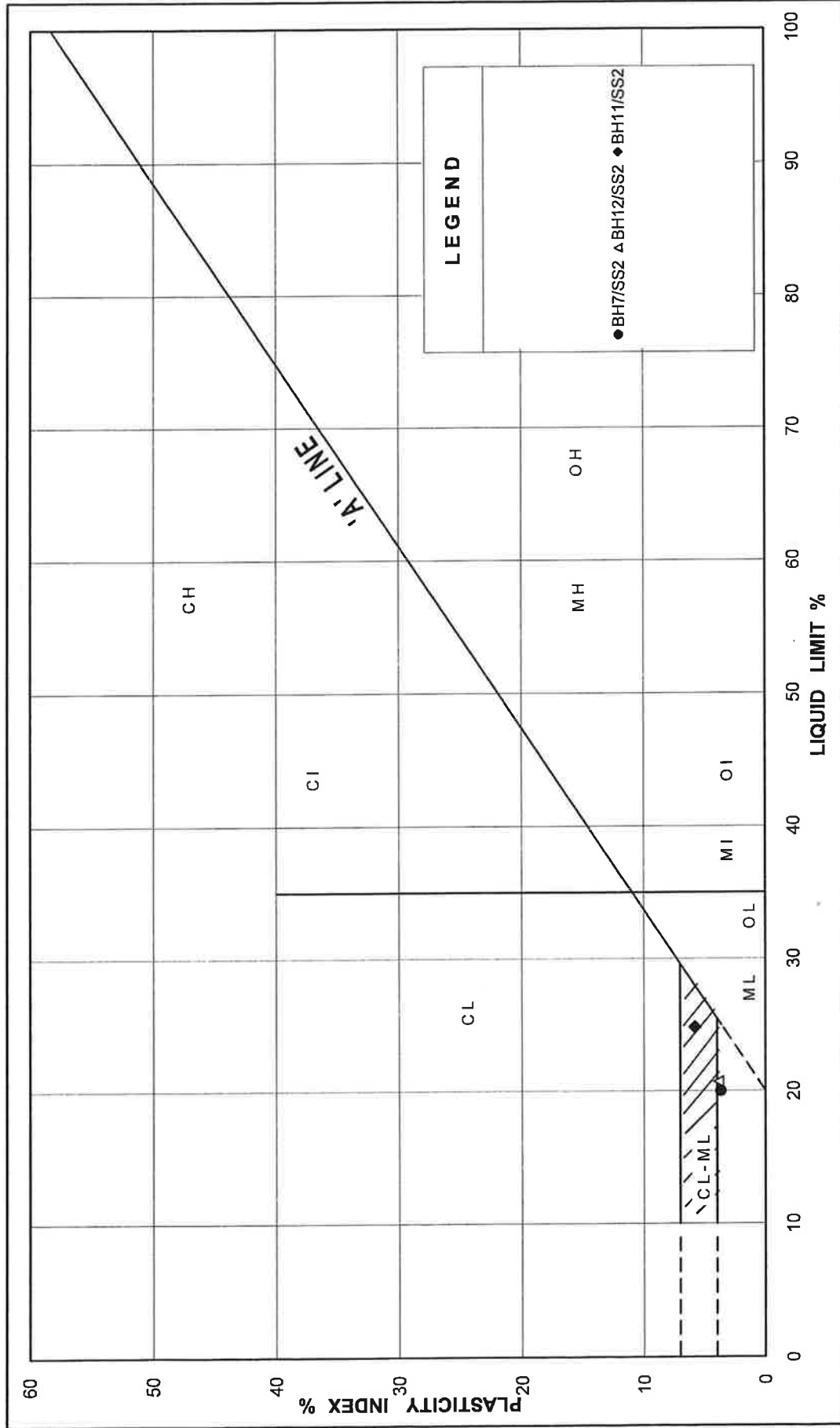
CLAY AND SILT			SAND			GRAVEL		
			Fine	Medium	Coarse	Fine	Coarse	



UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT		SAND			GRAVEL		
GRAIN SIZE IN MICROMETERS		Fine	Medium	Coarse	Fine	Coarse	

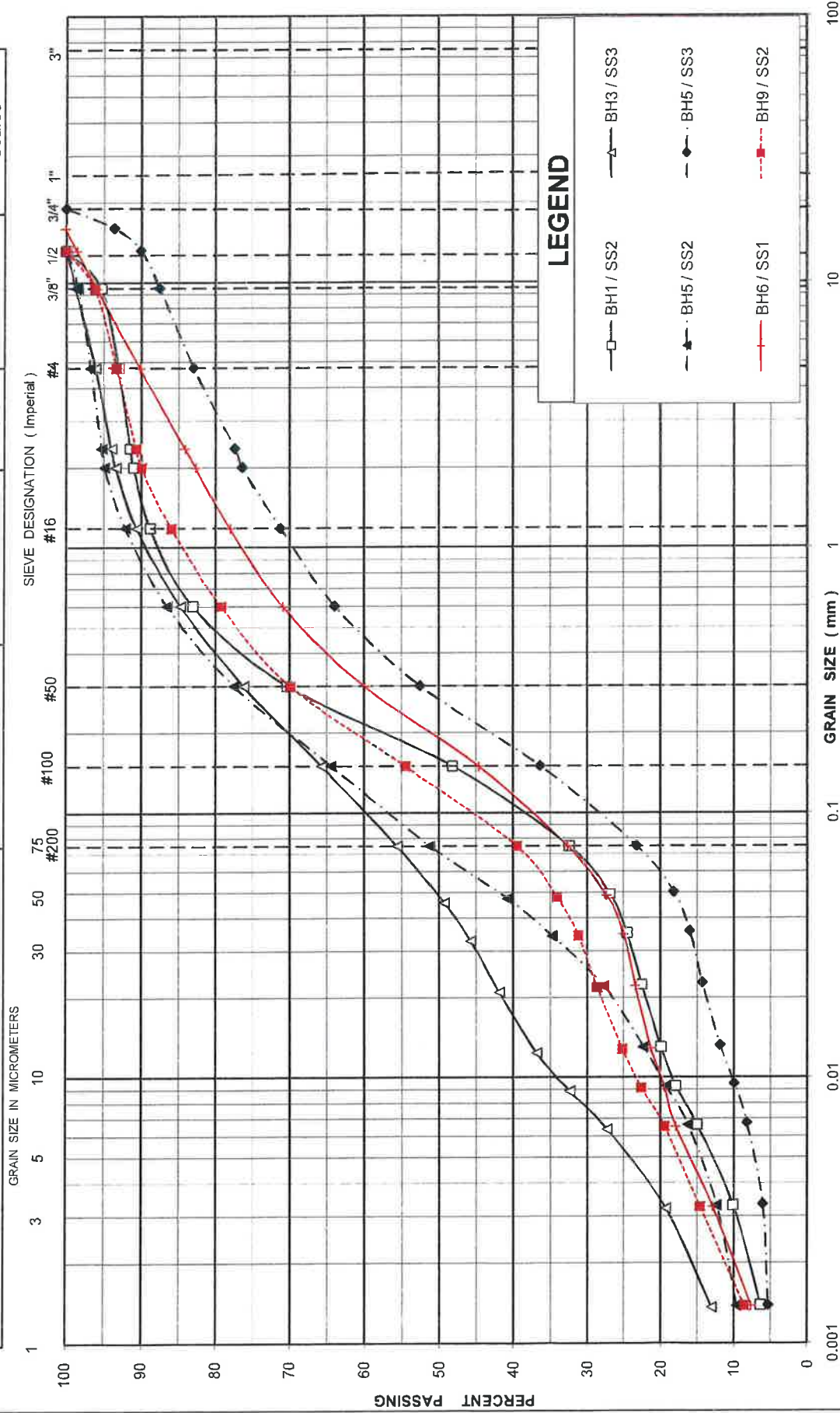




 SPECIALISTS MANAGING THE EARTH	PLASTICITY CHART		FIGURE No. B-3
	EMBANKMENT FILL - Sandy Silt, some clay		REF. No. TRANETOB10388AA
			DATE March, 2011

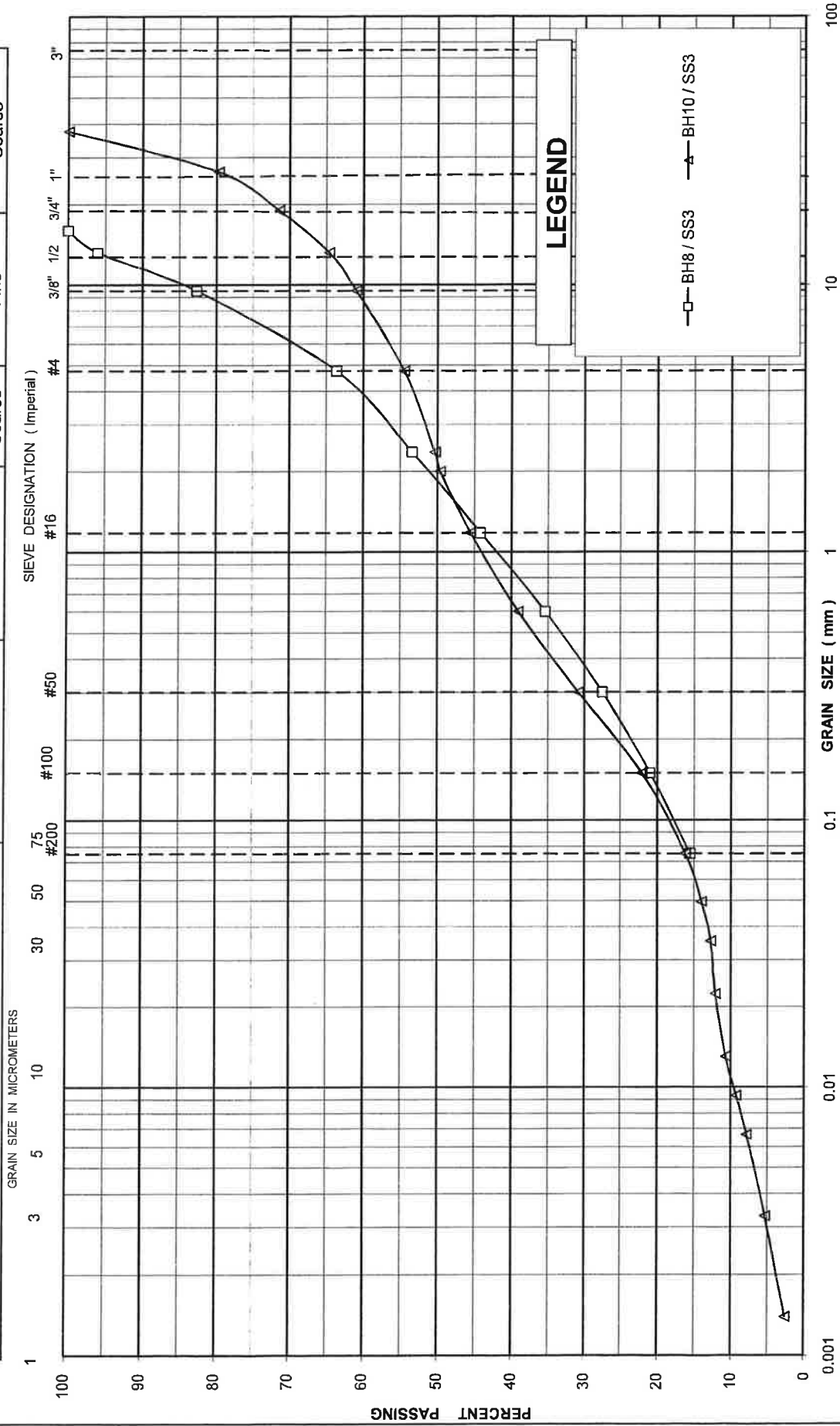
UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT		SAND			GRAVEL		
		Fine	Medium	Coarse	Fine	Coarse	



UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT			SAND			GRAVEL		
GRAIN SIZE IN MICROMETERS			Fine	Medium	Coarse	Fine	Coarse	



Project No **TRANETOB10388AA**

Sheet **1** of **1**

Point Load Strength Index Test Results

Client	Giffels Associates Limited / IBI Group		
Principal	ETOBICOKE		
Project	Restoule River Bridge Replacement		
Location	Patterson, ON		
Test Method	ASTM D5731-08 Standard Test Method for Determination of the Point Load Strength Index of Rock		
Test Machine	ELE		
Calibration Date -			
Sampling Technique	Storage History		
Moisture Condition	Natural		
Loading Rate	>30 seconds - 3 minutes		
Sampling Date	Nov 16 2010		
Testing Date	Nov 16 2010		
Tested By	DS		
Checked	ZO		

Test Method	ASTM D5731-08 Standard Test Method for Determination of the Point Load Strength Index of Rock			Sampling Technique		Sampling Date								
	Test Machine	ELE	Calibration Date -	Storage History	Moisture Condition	Testing Date	Nov 16 2010							
				Loading Rate		Tested By								
				Natural		DS/SS								
				>30 seconds - 3 minutes										
Rock Type	Location	Depth (m)	Diametral Tests					Axial, Block, and Irregular Lump Tests					Strength Classification	
			D (mm)	L (mm)	P (kN)	I _{s(50)} (MPa)	Failure Mode	W (mm)	D (mm)	L (mm)	P (kN)	I _s (MPa)		I _{s(50)} (MPa)
Gneiss	BH2	3.33	47	60	6.54	2.88	Through substance	47	27	9	5.57	5.05	Through substance	R5
Gneiss	BH2	4.67	47	50	16.7	7.35	Through substance	47	43	12.86	5	5.03	Bad break	R4/R5
Gneiss	BH3	2.89	47	50	9.51	4.19	Parallel to bedding	47	42	11.4	4.54	4.54	no break	R5
Gneiss	BH3	4.64	47	30	27.88	12.27	Through substance	47	32	9.09	4.75	4.47	Through substance	R5
Gneiss	BH6	2.97	47	50	22.21	9.78	Through substance	47	43	12.62	4.9	4.94	no break	R5/R6
Gneiss	BH6	4.46	47	100	5.07	2.23	Through substance	47	44	30.41	11.55	11.68	Through substance	R5/R6
Gneiss	BH10	2.17	47	90	8.22	3.62	Parallel to bedding	47	40	16.82	7.03	6.96	Through substance	R5
Gneiss	BH10	2.28	47	80	12.43	5.47	Parallel to bedding	47	30	8.36	4.66	4.32	Through substance	R4
Gneiss	BHC1	0.28	47	70	18.63	8.2	Through substance	47	23	9.86	7.16	6.26	Through substance	R4/R5
Gneiss	BHC2	0.46	47					47	35	3.14	1.5	1.44	Parallel to bedding	R5
														R3/R5

UNCONFINED COMPRESSION TEST (UC)

ASTM D 7012-07

SAMPLE IDENTIFICATION

PROJECT NUMBER	10-1183-0103	SAMPLE NUMBER	RC5
BOREHOLE NUMBER	BH3	SAMPLE DEPTH, m	2.95-3.08

TEST CONDITIONS

MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST, min	>2 <15	L/D	2.35

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	11.05	WATER CONTENT, (specimen) %	0.14
SAMPLE DIAMETER, cm	4.70	UNIT WEIGHT, kN/m ³	26.19
SAMPLE AREA, cm ²	17.35	DRY UNIT WT., kN/m ³	26.16
SAMPLE VOLUME, cm ³	191.71	SPECIFIC GRAVITY, assumed	2.70
WET WEIGHT, g	512.25	VOID RATIO	0.01
DRY WEIGHT, g	511.53		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	92.3
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REMARKS:

DATE:

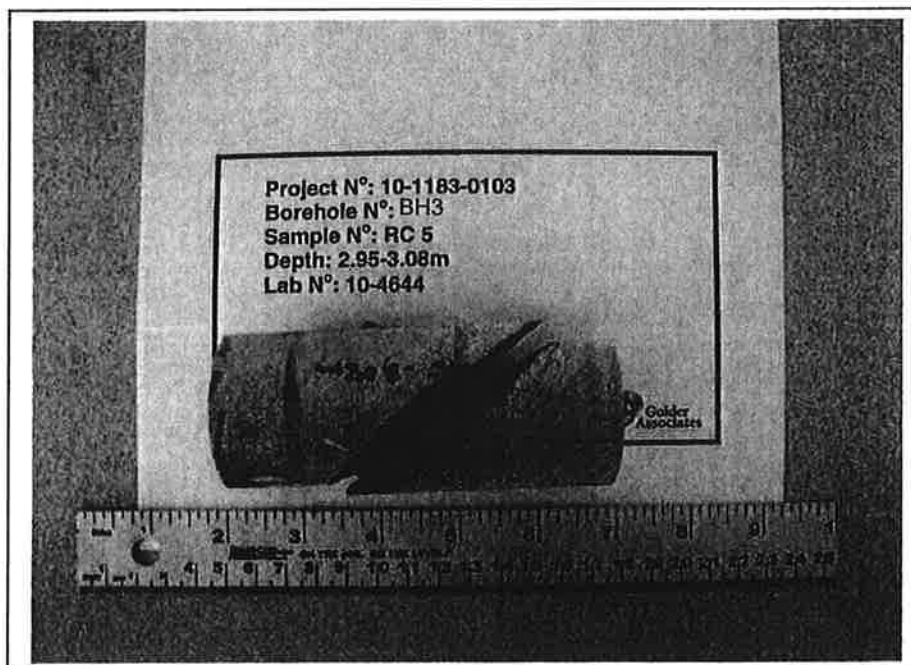
11/30/2010

UNCONFINED COMPRESSION TEST
ASTM D7012-07

FIGURE



BEFORE COMPRESSION



AFTER COMPRESSION

Date 12/1/2010
Project 10-1183-0103

Golder Associates

Drawn AH
Chkd. *[Signature]*

UNCONFINED COMPRESSION TEST (UC)

ASTM D 7012-07

SAMPLE IDENTIFICATION

PROJECT NUMBER	10-1183-0103	SAMPLE NUMBER	RC3
BOREHOLE NUMBER	BH6	SAMPLE DEPTH, m	4.34-4.50

TEST CONDITIONS

MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST, min	>2 <15	L/D	2.44

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	11.45	WATER CONTENT, (specimen) %	0.07
SAMPLE DIAMETER, cm	4.70	UNIT WEIGHT, kN/m ³	26.68
SAMPLE AREA, cm ²	17.35	DRY UNIT WT., kN/m ³	26.66
SAMPLE VOLUME, cm ³	198.65	SPECIFIC GRAVITY, assumed	2.70
WET WEIGHT, g	540.60	VOID RATIO	-0.01
DRY WEIGHT, g	540.22		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	166.7
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REMARKS:

DATE:

11/30/2010

UNCONFINED COMPRESSION TEST
ASTM D7012-07

FIGURE



BEFORE COMPRESSION



AFTER COMPRESSION

Date 12/1/2010
Project 10-1183-0103

Golder Associates

Drawn AH
Chkd. *[Signature]*

Appendix C

Site Photographs



Photograph 1. Existing Restoule River Bridge (looking west)



Photograph 2. Existing Restoule River Bridge - north side (looking west)



Photograph 3. North east abutment (looking east)



Photograph 4. South west abutment (looking west)

Appendix D

Explanation of Terms Used in Report

EXPLANATION OF TERMS USED IN REPORT

N-VALUE: THE STANDARD PENETRATION TEST (SPT) N-VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N-VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N-VALUE IS DENOTED THUS N.

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

C_u (kPa)	0 – 12	12 – 25	25 – 50	50 – 100	100 – 200	>200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 – 5	5 – 10	10 – 30	30 – 50	>50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND/OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY IS:

RQD (%)	0 – 25	25 – 50	50 – 75	75 – 90	90 – 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

JOINT AND BEDDING:

SPACING	50mm	50 – 300mm	0.3m – 1m	1m – 3m	>3m
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

SS	SPLIT SPOON	TP	THINWALL PISTON
WS	WASH SAMPLE	OS	OSTERBERG SAMPLE
ST	SLOTTED TUBE SAMPLE	RC	ROCK CORE
BS	BLOCK SAMPLE	PH	TW ADVANCED HYDRAULICALLY
CS	CHUNK SAMPLE	PM	TW ADVANCED MANUALLY
TW	THINWALL OPEN	FS	FOIL SAMPLE

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
c_c	1	COMPRESSION INDEX
c_s	1	SWELLING INDEX
c_a	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_i	1	SENSITIVITY = c_u / τ_r

PHYSICAL PROPERTIES OF SOIL

P_s	kg/m^3	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	e_{min}	1, %	VOID RATIO IN DENSEST STATE
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{\text{max}} - e}{e_{\text{max}} - e_{\text{min}}}$
ρ_w	kg/m^3	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kN/m^3	UNIT WEIGHT OF WATER	s_r	%	DEGREE OF SATURATION	D_n	mm	N PERCENT – DIAMETER
ρ	kg/m^3	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ	kN/m^3	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
ρ_d	kg/m^3	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m^3/s	RATE OF DISCHARGE
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $(w_L - w_p)$	v	m/s	DISCHARGE VELOCITY
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $(w - w_p) / I_p$	i	1	HYDRAULIC GRADIENT
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $(w_L - w) / I_p$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL	e_{max}	1, %	VOID RATIO IN LOOSEST STATE	j	kN/m^2	SEEPAGE FORCE
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL						

**FOUNDATION DESIGN REPORT
RESTOULE RIVER BRIDGE AT
HAWTHORNE DRIVE, TOWNSHIP OF
PATTERSON, DISTRICT 54, SUDBURY,
ONTARIO, G.W.P. 5112-06-00,
GEOCRES 31L-145**

Giffels Associates Ltd / IBI Group
TRANETOB10388AA-AA
June 22, 2011

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Appendices

Appendix E: List of SPs, OPSSs and OPSDs

Appendix F: GA drawing

Appendix G: Limitations of Report

FOUNDATION DESIGN REPORT
RESTOULE RIVER BRIDGE AT HAWTHORNE DRIVE
TOWNSHIP OF PATTERSON, DISTRICT 54, SUDBURY, ONTARIO
G.W.P. 5112-06-00

5 DISCUSSION AND RECOMMENDATIONS

5.1 General

The existing bridge which carries Hawthorne Drive over Restoule River in the Township of Patterson is to be replaced with a new, wider structure. The new bridge will be constructed on the existing bridge alignment. As such, a temporary bridge will need to be constructed to the immediate north of the existing bridge prior to the construction of the proposed new bridge.

The fourteen boreholes drilled indicate that the site is generally underlain by topsoil, fill (including pavement fill and embankment fill), and native soils consisting of silty sand to sandy silt and sand & gravel, which are in turn underlain by bedrock. The pavement fill consist of 35 to 80 mm thick asphalt over 0.2 to 0.8 m thick loose to compact sand and gravel fill. The embankment fill was found to extend to depths of 1.2 to 2.1 m below the ground surface or to Elevations 225.6 to 224.5 m and generally consists of silty sand to sandy silt in a loose to compact condition. Below the embankment fill and topsoil, native soils were encountered at 0.3 to 2.1 m below the ground surface or at Elevations 226.4 to 223.0 m with thicknesses of about 0.4 to 2.3 m. Native soils are mainly granular soils namely, silty sand to sandy silt with sand and sand & gravel in some areas, and are typically compact to very dense, where the very dense condition occurs near the surface of the bedrock. Some very loose to loose zones were found near the surface at the interface with existing fills or near the ground surface. The surface of the bedrock was found or inferred (i.e. auger refusal) to be at 0 to 2.7 m below the existing ground surface or at Elevations 224.9 to 222.1 m. Based on the rock cores recovered, the bedrock was described as grey gneiss with granitic mixture and was classified as medium strong to extremely strong rock (R3 to R6 Grade). The groundwater level at the time of our investigation was below the surface of the bedrock at the borehole locations and the water level in the River was at about Elevation 222.4 m. The groundwater level would however be subject to seasonal variations and variations in response to major weather events, as well as being influenced by the water level in the River.

5.2 Bridge Foundations

We understand that the new permanent bridge will be constructed on the existing bridge alignment and it would be a single span bridge, about 16 m length and 7 m wide (3.5 m longer and 2 m wider than the existing bridge).

Based on the borehole information, the subsurface conditions at the site are favourable for the use of normal spread footings bearing on the bedrock to support the proposed new bridge. It is our opinion that deep foundations are impractical and cost ineffective for this project from geotechnical engineering point of view. We understand, however, that the existing bridge abutment and wing walls may need to be

maintained because of their historical significance based on GA drawing provided to us by IBI group (See Appendix F). Excavation for new spread footing foundations near to the existing structure may not be acceptable. In this case, the proposed bridge would need to be supported on deep foundations.

Shallow and deep foundations are discussed in the following sections

5.2.1 Shallow Foundations

The bedrock surface level at the borehole locations was found (i.e. proven by coring) or inferred (auger refusal) to vary from the ground surface at the riverbank areas to a maximum of 2.7 m below the existing ground surface in other areas. The following geotechnical resistances are available for footings bearing on relatively level, sound bedrock:

- Factored Bearing Resistance at U.L.S. = 10,000 kPa
- Bearing Resistance at S.L.S. will not govern

The quoted U.L.S. value may be impractical (i.e. too high) for use when designing the foundations of a rather small bridge structure, such as the present case, but is given here for the sake of completion.

When making these recommendations it is assumed that the foundations will extend below the adjacent sloping ground. If the foundations are to be constructed adjacent to sloping ground, stability must be assured by socketing/keying-in the foundations sufficiently into the bedrock and/or dowelling/anchoring into the bedrock. For example, if the footing is placed immediately adjacent to the steeply sloping rock, the rock near the edge beneath the footing may break-off (e.g. spalling), thus undermining the footing. For this reason, the footing must be placed sufficiently away from the edge of the steeply sloping rock River bank. In addition, the footing must be placed on sufficiently level rock surface. If necessary, the bedrock surface can be flattened by levelling or making benches or the problem may be alleviated by providing dowels. We recommend that this aspect be discussed with us when some of the details are known such as the position of the footing in relation to existing River banks. As well, it should be ensured that rock beneath the footing level will not be subject to detrimental scour or frost effects which might jeopardize the footings.

For inclined loading conditions, the bearing resistance should be reduced in accordance with the Canadian Highway Bridge Design Code (CHBDC CAN/CSA, S6-06).

For the evaluation of the sliding resistance of the foundation, the interface friction angle between the underside of the concrete footing and the clean and sufficiently roughened bedrock surface can be taken as 35 degrees. Horizontal shear resistance can be supplemented, if required, by keying-in to the bedrock and utilizing the passive rock resistance and/or shear in grouted dowels and/or rock anchors. We recommend a minimum dowel length of 1.5 m.

If there are uplift forces which are to be resisted by rock anchors, the factored rock/bond resistance at ULS can be taken as 600 kPa and SLS will not govern. The upper 0.5 m of rock should, however, not be included in calculating the resistance and the minimum embedment depth should be 1.5 m into the rock, below the underside of the footing. The bond resistance depends on anchor installation methods, grouting procedures, etc. and must be confirmed by field load testing. The anchors should also be checked for rock wedge pull out assuming a 60 degree apex cone/wedge and the anchor group resistance should also be checked.

The bearing surfaces should be inspected, evaluated and approved by the Geotechnical Engineer/Geologist appointed by the QVE. This is very important for this project for the following reason.

Normally for frost protection in this geographic area, the footings should have a permanent earth cover of at least 2.0 m. However, this is likely to be very expensive and based on our experience, if the footings are placed on sufficiently massive rock (i.e. no jointing, cracks, fissures, etc,) it may be possible to reduce this depth or even eliminate it. For this purpose the following approach can be taken. The surface of the bedrock on which the footing is to be supported should be made level and carefully inspected by a competent Geologist or a Geotechnical Engineer. The surface of the rock to receive the footing must be free of fractures, jointing, cracks, fissures or bedding planes, or any other defects which water can get into and cause problems due to frost. This is also true for rock surrounding the footing footprint. These areas must also be defect free or made so, such that water could not enter to cause problems with the rock supporting the footing (i.e. further opening the existing defects or causing heave due to frost action). In other words, water must be prevented from entering the rock beneath and immediately surrounding the rock. We recommend that unless normal frost protection (e.g. up to 2.0 m) is provided, an NSSP be prepared to ensure that the rock must be competent enough to support the bearing resistance required and that it must be massive enough to prevent frost action both beneath the footing and in the adjacent areas surrounding the footing. If necessary, measures must be taken to seal any cracks to prevent any water seepage and penetration (e.g. grouting and/or placing an erosion resistant layer of concrete over the surface, etc., depending on the conditions). In addition, the geometry must be checked for stability purposes, as mentioned before, by the evaluator. The rock must also be checked for any bedding planes or other defects which may cause the footings to slide towards the River. These are standard field features which are normally evaluated by the Geologist or the Geotechnical Engineer, provided they are experienced enough. We recommend however that this aspect be also mentioned in the NSSP. In addition, the bearing surface should be cleaned and free from any loose debris prior to concreting of footings.

5.2.2 Deep Foundations

Based on the findings of boreholes, the thickness of the overburden overlying bedrock at the proposed bridge support locations appears to be less than 3 m. For this reason, it would be impractical and cost ineffective to support a rather short span bridge on deep foundations. If, however, deep foundations must be used for this particular bridge, frost depth of 2 m also should be taken into account for deep foundation design because pile cap bottom needs to be placed below the frost depth. The actual lengths of deep foundations would also depend on the socket length of deep foundation into the bedrock; however, difficulties may arise to install deep foundations due to the presence of gneiss bedrock at the site (i.e. a strong rock type). Sloping of bedrock also needs to be taken into account for deep foundation design and construction. With this preamble, the followings are our recommendations regarding deep foundations.

5.2.2.1 Timber piles

Pile length will be extremely short (less than order of 1 m). As well based on the prevailing subsurface conditions, timber piles are not feasible for this project due to the anticipated hard driving conditions. This option is therefore considered impractical and not recommended, based on reliability.

5.2.2.2 Driven Steel Piles

Pile length will be extremely short (less than order of 1 m). It is our opinion that during driving the piles may 'walk' on the bedrock, even if rock injector (i.e. Oslo point, Titus rock injector or equivalent) tips are adopted for pile driving. This is mainly because the remaining overburden (about 1 m thick or less) below the frost depth to the bedrock is not competent enough to provide sufficient lateral support (confinement) for pile driving.

As well, If the existing structure needs to be maintained, driving piles near the existing structure may not be acceptable due to the anticipated vibrations.

This option is therefore considered impractical and not suited for the prevailing surface and subsurface conditions.

5.2.2.3 Cast-in-place Concrete Piles

Cast-in-place concrete piles (drilled caissons) may be a feasible option and piles socketed into the bedrock would be required to resist the axial and lateral loads. Vibrations and noise will be less in comparison with driven pile foundation option. While excavating, rock adjacent to caisson should not be shattered (damage to the bedrock should be minimized). As well, vibrations will need to be minimized to prevent damage to the existing structures while excavating the bedrock. This foundation option can be expected to be costly wherever the bedrock is fairly sound.

Geotechnical resistances of cast-in-place concrete piles increase with socket depth into the bedrock. For caissons which extend at least 1 m below the surface of the bedrock, 10,000 kPa can be used (end bearing). For example, a 0.76 m (30 inch) diameter caisson will have a based area of $(0.76)^2 \pi / 4 = 0.43 \text{ m}^2$. When designed for a value of 10,000 kPa, a resistance of $10,000 \text{ kPa} \times 0.43 \text{ m}^2 = 4536 \text{ kN}$ is obtained.

For caissons socketed more than 1.0 m below the rock surface, an additional resistance of 600 kPa can be utilized (owing to adhesion). For example for a 0.76 m diameter caisson extending 2.5 m below the surface of bedrock the additional resistance would be $0.76 \text{ m} \times \pi \times (2.5 \text{ m} - 1.0 \text{ m}) \times 600 \text{ kN/m}^2 = 2149 \text{ kN}$ bringing the total factored resistance at U.L.S. to $4536 + 2149 = 6685 \text{ kN}$.

The minimum spacing of the caissons centre to centre should not be less than three diameters.

As was mentioned before, if the rock surface in front of the caisson is sloping and the caisson is located close to the sloping surface, this geometry may adversely affect the resistance, in particular the horizontal resistance. As well, if the rock around the caisson is shattered during the construction, this too will adversely affect the resistances and as such excessive shattering of the rock in the vicinity of the caissons must be avoided.

Due to the presence of granular fill materials overlying the bedrock at the site, temporary casing may required for this option. This temporary casing needs to be advanced to the bedrock surface (or deeper into bedrock depending on the bedrock condition) to maintain the hole stability during excavation and concrete placement.

The rock socket portion (bottom and side wall) should be cleaned before the start of concrete placement to ensure intimate contact of the concrete with bedrock. All loose or broken rock pieces need to be removed. Where the bottom of excavations need to be visually inspected, a minimum caisson diameter of 0.76 m would be required, to enable the leaning and inspection of the base of the caisson.

The casing/liner would be withdrawn as the concrete is poured, ensuring a sufficient head of concrete in the casing to prevent 'necking'.

5.2.2.4 Micropiles

Consideration can also be given to the use of micropile foundations for the proposed bridge. This option may be a feasible option because disturbance to the existing wing wall and abutment wall structures will be minimal and the installation is feasible with relatively smaller equipment in comparison with a caisson installation rig.

A micropile is constructed by drilling a borehole, placing reinforcement, and grouting the hole. Micropiles can withstand axial and/or lateral loads, and may be considered a substitute for conventional piles or as one component in a composite soil/pile mass, depending upon the design concept employed. Micropiles can be installed in most soil and rock types and ground conditions. Micropile structural capacities, by comparison, rely on high capacity steel elements to resist most or all of the applied loads. These steel elements have been reported to occupy as much as one-half of the whole volume. The special drilling and grouting methods used in micropile installation allow for high grout/ground bond values along the grout and ground interface. The grout transfers the load through friction from the reinforcement to the ground in the micropile bond zone in a manner similar to that of ground anchors. Due to the small pile diameter (typically 160 to 260 mm), any end-bearing contribution in micropiles is generally neglected. The grout/ground bond strength achieved is influenced primarily by the ground type and grouting method used, i.e., pressure grouting or gravity feed. The role of the drilling method is also influential, although less well quantified.

A factored bonding resistance between 600 and 1000 kPa (between the fresh gneiss bedrock and the grout) can be used (at ULS and SLS will typically not govern) for preliminary design purposes. The lateral resistances would also depend on the diameter, as well as extent of the socket length into the bedrock. Typically factored resistances of the order 700 to 1000 kN/micropile (at ULS) are available and SLS will not govern.

The use of micropiles may be less economical than caissons due to the fact that the installation requires a more specialized installer for the micropiles than the many contractors who are able to routinely install caissons. But as mentioned before, this may represent an attractive option as it will minimize disturbance to the site.

The axial and horizontal resistances of micropiles and other details regarding the design of micropiles can be discussed with specialist contractor and will be pleased to expand on this further should you wish to pursue this option.

5.2.3 Temporary Bridge

We understand that a temporary bridge, 4.8 m wide, will be constructed about 6.7 m (centreline to centreline) from existing bridge, prior to the construction of the proposed new bridge. We also understand that the temporary bridge deck is to be supported by 'crib' type wall on each end of the bridge.

The bedrock appears to be relatively shallow at the location of the proposed cribs; at the surface on the eastern end (BHC2) and at 1.1 m depth on the western end (BH6). We recommend that the cribs to be founded on level bedrock surface. If crib foundations are to be constructed on a sloping ground, stability must be assured by keying-in the foundations sufficiently into the bedrock and/or dowelled (anchored) into the bedrock. It should be ensured that the crib wall is sufficiently away from the edge of the sloping River bank, so that an instability condition will not result, as discussed above. Bearing resistances presented above in Section 5.2.1 can be used. The cribs can be anchored into the bedrock or embedded into the bedrock, if sliding or horizontal resistance is required. Recommendations for sliding or horizontal resistance are also presented in Section 5.2.1.

The bearing surfaces for the temporary bridge should be inspected, evaluated and approved by the Geotechnical Engineer appointed by the QVE. The strict recommendations to prevent water entering the supporting bedrock which would create problems due to frost, etc., are unlikely to be necessary for the temporary bridge, especially if it is not likely to be used over the winter period. Nevertheless, the supporting surface must be checked and evaluated to ensure its competence as well as stability due to geometry, i.e. sufficiently away from the edge of the sloping rock, as well as for any bedding planes which may slide towards the River, etc.

5.3 Backfill behind the abutments

Backfill behind the abutments should consist of non-frost susceptible, free-draining granular materials in accordance with the MTO Standards and the requirements of OPSD 3101.150.

Free-draining backfill materials such as Granular 'A' or Granular 'B' Type I, II or III can be used. To maintain free draining characteristics in these granular till materials, the maximum percentage passing the No. 200 sieve (76 μm) should be limited to 5 %. Drains pipes, weep holes and the like should be incorporated to reduce hydrostatic pressure build-up.

5.3.1 Lateral Earth Pressures

Computation of earth pressures should be in accordance with CHBDC. For design purposes, the following parameters (unfactored) can be used.

Compacted Granular 'A' and Granular 'B' Type II

Angle of Internal Friction, $\phi = 35^\circ$ (unfactored)

Unit Weight = 22 kN/m³

Coefficient of Lateral Earth Pressure:

$K_a = 0.27$

$K_b = 0.35$

$K_o = 0.43$

$K^* = 0.45$

Compacted Granular 'B' Type I and Type III

Angle of Internal Friction, $\phi = 32^\circ$ (unfactored)

Unit Weight = 21 kN/m^3

Coefficient of Lateral Earth Pressure:

$K_a = 0.31$

$K_b = 0.41$

$K_o = 0.47$

$K^* = 0.57$

Where K_b is the 'intermediate' earth pressure coefficient for a partially restrained structure. This case occurs when some movement (yield) of the structure takes place but not in a sufficient magnitude to fully mobilize an active condition (as such it is an intermediate condition between K_o and K_a).

K^* is the earth pressure coefficient for a soil loading a fully-restrained structure, including compaction surcharge effects.

These values are based on the assumption that the backfill behind the retaining structure is free-draining and adequate drainage is provided. It is also assumed that the ground behind the retaining structure is level.

The earth pressure coefficient adopted will depend on whether the retaining structure is restrained or movements can be allowed such that the active state of earth pressure can develop. If the abutment is restrained and does not allow lateral yielding, then at rest pressures should be used in accordance with CHBDC CAN/CSA-S6-06. This is the case for this project as a rigid frame structure resting on bedrock (i.e. non-yielding) is under consideration. Vibrations generated by the highway traffic should also be taken into consideration in the selection of appropriate earth pressure coefficients. The effect of compaction should also be taken into account in the selection of the appropriate earth pressure coefficients in accordance with Section 6.9 of CHBDC CAN/CSA-S6-06.

For unrestrained wing walls (if any), the intermediate earth pressure coefficient K_b may be adopted. In the determination of degree of wall displacement or rotation to mobilize the fully active earth pressure state, Section C6.9 of the CHBDC CAN/CSA-S6-06 Commentary should be referenced.

Vibratory equipment for use behind abutments and retaining walls should be restricted in size as per current MTO practice.

Seismic analysis is not required for single span bridge regardless of seismic performance zone except for single span truss bridges as per Section 4.4.5.2 of CHBDC CAN/CSA-S6-06.

5.4 Embankments

The existing approach embankments have a maximum height of 2.5 m. These approach embankments are expected to be excavated during the construction of the footings for the proposed new bridge and reconstructed afterwards.

For the temporary bridge, approach embankments of up to 2.5 m high are anticipated to be constructed, if the height of the temporary bridge is similar to the existing bridge.

The following are recommended for site preparation:

- Strip surface vegetation, tree roots, topsoil, other organics, and other unsuitable and/or loose/soft materials.
- Where feasible, proof roll the exposed surface.
- If localized soft or loose spots or excessive heave occurs during proof rolling, further excavate and replace with suitable fill.

The removal of the organic soils should be carried within an envelope given by an imaginary slope no steeper than 1H:1V from the toe of the proposed embankment, as per MTO standard procedures. After stripping, the exposed subgrade should be inspected and approved. It should be compacted from the surface using a suitable compactor.

For preliminary estimating purposes, the thickness of topsoil to be removed can be taken as 0.3 m. However, the thickness of the topsoil and/or other organic soils could vary at the o.g. level.

Proper benching of the existing slopes should be implemented during the construction of the approach embankments, as per MTO procedures and in accordance with OPSD 208.010.

Foundation failures are not anticipated for the approach embankments of this height (i.e. approximately 2.5 m high from the original ground surface) constructed with normal 2H:1V side slopes or flatter, assuming that the soil conditions are similar to those encountered in the boreholes. It should however be ensured that any fill placed over the rock surface will not slide down, thus causing instability.

The materials used for the construction of the embankment fills should consist of approved, acceptable earth fill (e.g. Select Subgrade Materials - OPSS1010). Fill used for construction of the embankments should be in accordance with OPSS 212 and fill placement should meet or exceed the requirements of SP 105S10 and OPSS 206. Construction should be in accordance with SP 206S03. In general, the fills should be placed in suitable lift thicknesses not exceeding 300 mm when loose placed and each lift should be uniformly compacted to at least 95% of the material's Standard Proctor Maximum Dry Density (SPMDD). We recommend that the fill within 0.5 m of the pavement subgrade level should be compacted to 100% SPMDD. Although this is not a requirement with MTO procedures, in our experience it enhances the performance of the road.

For maximum 2.5 m high embankments, based on the least favourable borehole data, a settlement of the order of 15 mm was calculated due to the settlement of foundation soils. As the founding soils are generally granular soils, the settlements should primarily take place within three to six weeks. In addition, the settlement of the new embankment fills under their own weight can be expected to occur. If the embankment is constructed to MTO standards, however, this should not exceed 15 mm. The settlement due to the own weight of the new embankment will depend on the type of soil used to build the embankment (e.g. the settlement of granular soils will be relatively rapid while clayey soils will settle more slowly). Assuming an average SSM type soil, the settlement of the new embankment under its own weight should also be substantially completed within about one month.

As these calculated settlements are not excessive, neither surcharging nor preloading is considered necessary for the approach embankments. We recommend that, if feasible, the paving of the road (i.e.

placing of asphalt) be delayed (say by at least two weeks) after raising the grade to the final base level, to allow some of the anticipated settlements to take place.

Proper erosion control measures should be implemented both during the construction and permanently. This can be achieved by prompt seed and cover (OPSS 572) or sodding (OPSS 571).

5.5 Construction Comments

All excavations, shoring and backfilling should be carried out in conformance with the Occupational Health and Safety Act (OHSA), as well as the following specifications.

- SP 105S19 – Protection Schemes
- SP 902S01 – Excavation and Backfilling – Structures

In accordance with OHSA, the soils can be classified as follows

Existing Fill	Type 3 soil above water level;	Type 4 soil below water level
Native soils – v loose to loose	Type 4 soil above water level;	Type 4 soil below water level
– compact/dense/v.dense	Type 3 soil above water level;	Type 4 soil below water level

Excavations within the existing fill and native soils should be possible using heavy equipment such as a hydraulic excavator. Where the excavations will be extended into the bedrock, this will require ripping and/or rock breaking equipment. Rock saw may be required to reduce the risks of overbreak. Bedrock encountered at this site was classified as medium strong near the surface, becoming extremely strong with depth. Contractors should be encouraged to examine the engineering logs and rock cores as well as exposed bedrock at the site to make their own assessment of anticipated excavation plant and production rates/difficulties.

During the investigation, groundwater was encountered below the rock surface and the water level in the River was at Elevation 222.4 m (i.e. generally below the bedrock surface encountered in the boreholes). However, a perched water condition may occur due to the accumulation of surface water in the overburden. This may need dewatering. As well, inflows may occur at soil/bedrock interface and from structural defects in the rock, such as joints, fissures, bedding planes and weathered seams in the bedrock. It is however believed that for site excavations, excessive seepage into open excavations is not anticipated and the seepage can be handled by gravity drainage and pumping from open sumps. For excavations extending below the water level in the River, more aggressive measures will be required.

The excavated soils free from topsoil and organics can be used as general construction backfill where they can be compacted with smooth drum or pad-foot type rollers. Loose lifts of soil, which are to be compacted, should not exceed 300 mm. On site verification of the excavated fill for re-use as backfill by a suitably qualified personnel during construction would be required. The excavated soils are not considered to be free draining. Where free draining backfill is required, imported granular fill such as OPSS Granular B should be used. Note that the excavated soils are subject to moisture content increase during wet weather which would make these materials too wet for adequate compaction. Stockpiles should therefore be compacted at the surface or be covered with tarpaulins to help minimize moisture uptake. Excavated rock

can also be re-used provided that it is broken into sufficiently small sizes (i.e. typically less than 0.3 m nominal diameter).

For the construction of crib foundation for the temporary bridge, less extensive excavations are anticipated and in general a temporary excavation slope of 1H:1V to 2H:1V can be used.

For the proposed new bridge, temporary support may be necessary to retain the existing embankment fills during construction of the new bridge foundations. This will require shoring. However, it would probably be more convenient for the Contractor to slope the ground rather than shore it for the duration of the construction where feasible, as a temporary bridge will be constructed prior to construction of the proposed new bridge. Temporary slope of 1H:1V to 2H:1V (for very loose to loose conditions) can be used provided no relatively high loadings (e.g. heavy machinery, stockpiles etc.) are present at the crest of the slope.

5.6 Frost Protection

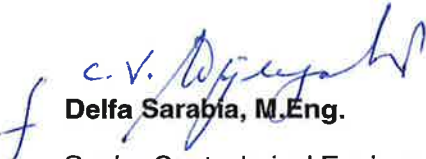
Design frost protection depth for the site is 2.0 m. A minimum 2.0 m thick permanent soil cover or equivalent thermal insulation is required for all of footings. However, where footings are placed on massive rock (i.e. free of fissures, etc. where water can get into) and the recommended measures presented Section 5.2.2 are carried out, the frost depth requirements can be reduced or eliminated, depending on the geometry, etc.

In case of rip-rap (rock fill), only one-half of the rock fill thickness should be assumed to be effective in providing frost protection.

6 CLOSURE

The "Limitations of Report" as presented in Appendix G are integral part of the report.

For and on behalf of Coffey Geotechnics Inc.


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Senior Geotechnical Engineer


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Manager, Transportation


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Senior Principal

Appendix E

List of SPs, OPSSs and OPSDs

List of SPs, OPSSs and OPSDs referenced in the report

SP 105S10 Construction Specification for Compaction

SP 105S19 Protection Systems

SP 206S03 Grading, Earth and Rock Excavation, Excavation for Pavement Widening

SP 902S01 Excavation and Backfilling of Structures

OPSS 206 Construction Specification for Grading

OPSS 212 Construction Specification for Borrow

OPSS 571 Construction Specification for Sodding

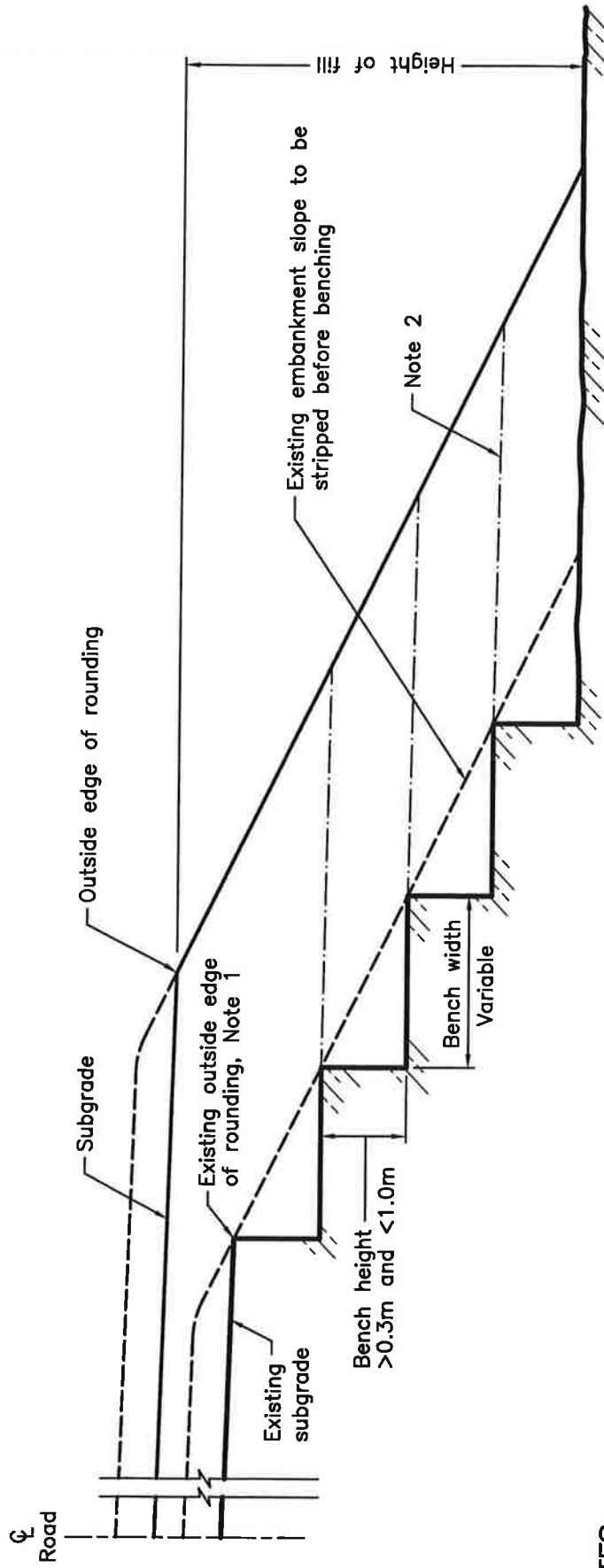
OPSS 572 Construction Specification for Seed and Cover

OPSS 903 Construction Specification for Deep Foundations

OPSS 1010 Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

OPSD 208.010 Benching of Earth Slopes

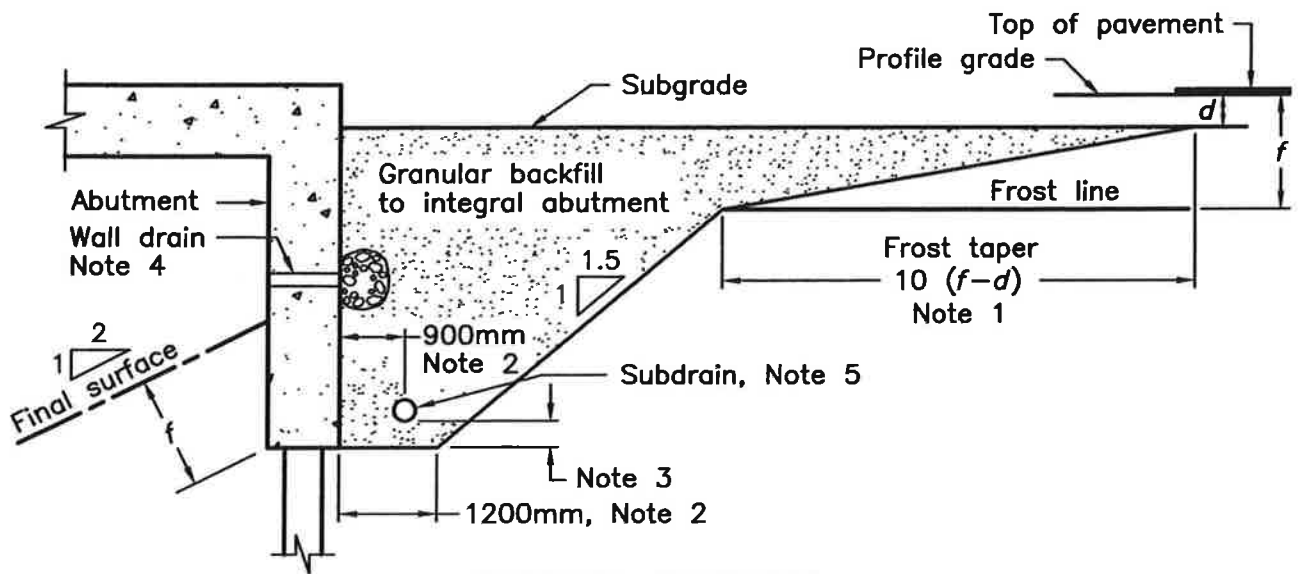
OPSD 3101.150 Walls, Abutment, Backfill, Minimum Granular Requirement



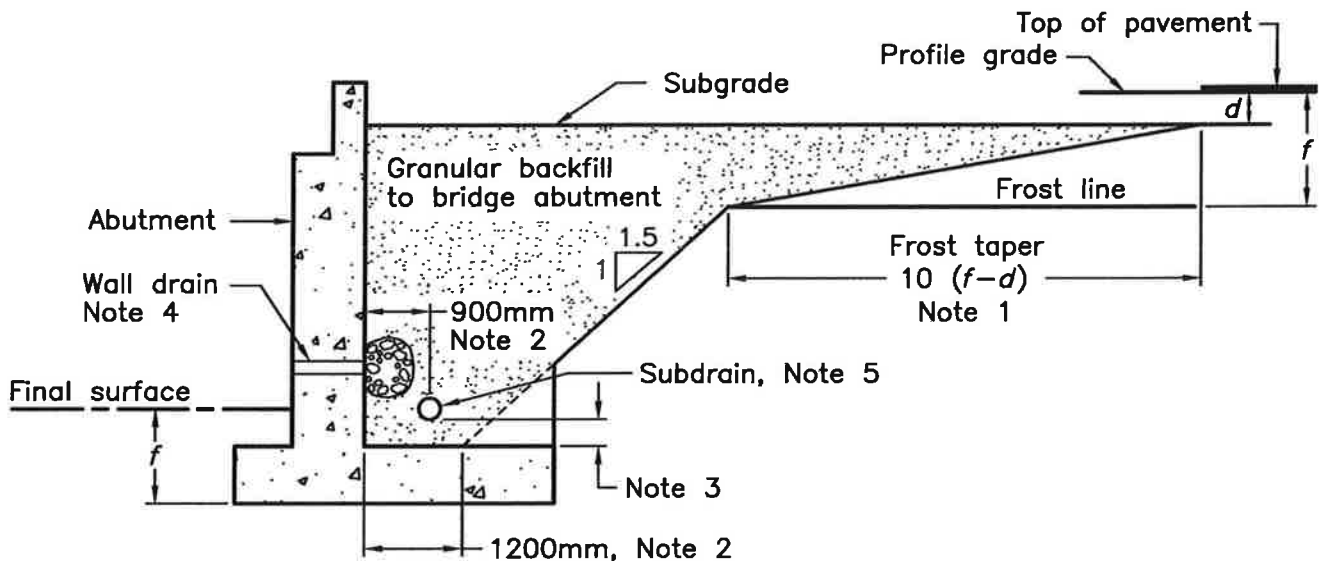
NOTES:

- 1 When the subgrade is below the existing outside edge of rounding, benching shall be carried out below the point where the subgrade intersects the existing slope.
 - 2 Benches are to be excavated one level at a time and the fill placed and compacted before the next bench is excavated.
- A Benching is not required on existing slopes flatter than 3H:1V.
- B All dimensions are in metres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING		Nov 2008	Rev 2
BENCHING OF EARTH SLOPES			
			OPSD 208.010



INTEGRAL ABUTMENT



ABUTMENT

NOTES:

- 1 d = depth of combined base and subbase courses.
 f = roadbed depth of frost penetration as specified.
- 2 Dimensions perpendicular to back face of abutment.
- 3 Height to be consistent with positive drainage of subdrain as specified.
- 4 Where specified, wall drains shall be installed according to OPSD-3190.100.
- 5 150mm dia perforated pipe subdrain wrapped with geotextile.
- A Lateral limits of granular backfill to bridge abutment to be inside face to inside face of retaining wall or wingwall. Frost taper shall extend the full width of the fill unless interrupted by the retaining wall or wingwall.
- B Sections shown are parallel to centreline of roadway.
- C Subdrain to be installed with a 2% gradient behind wall.
- D All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2005

Rev 0

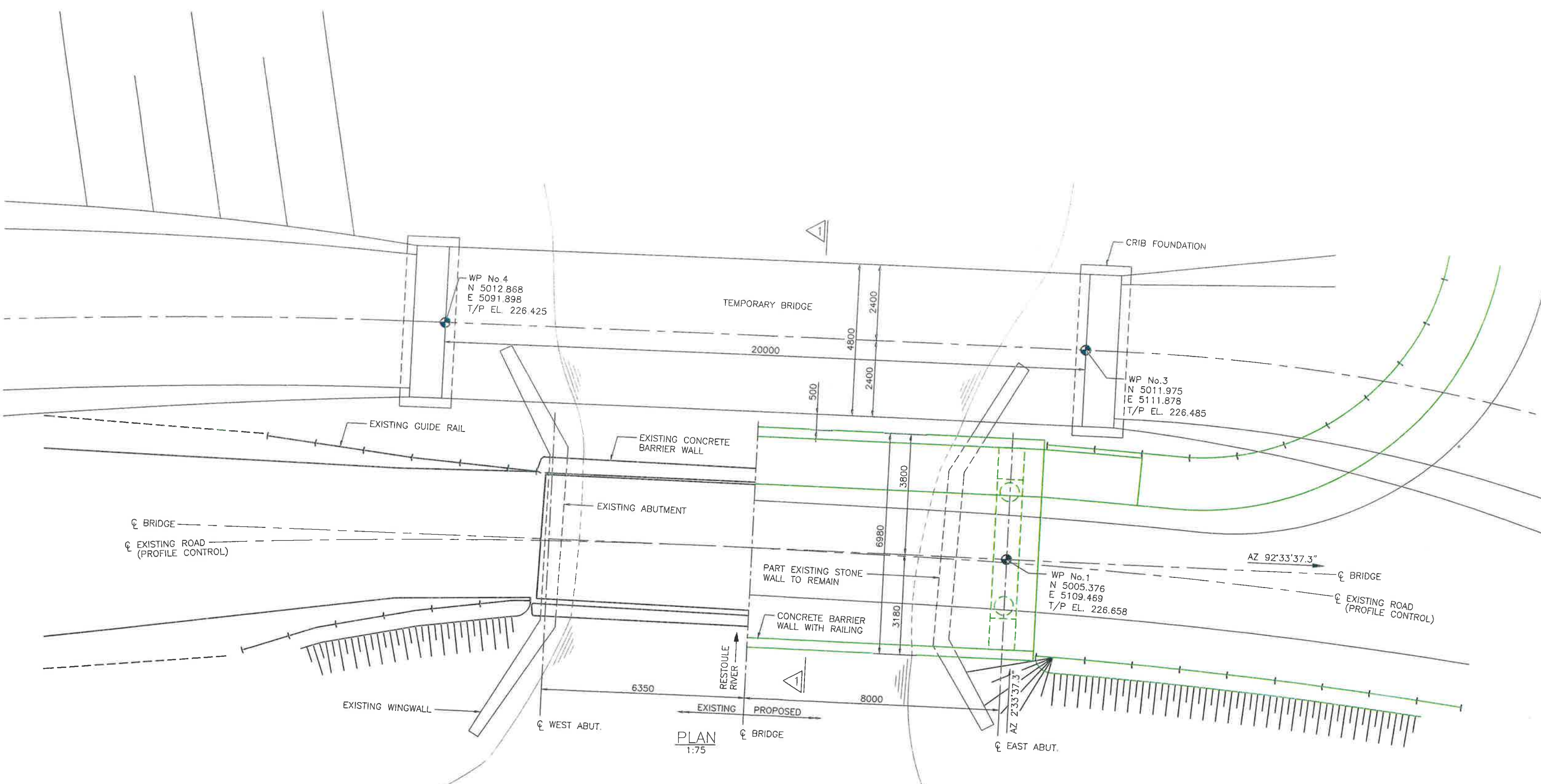
WALLS
ABUTMENT, BACKFILL
MINIMUM GRANULAR REQUIREMENT

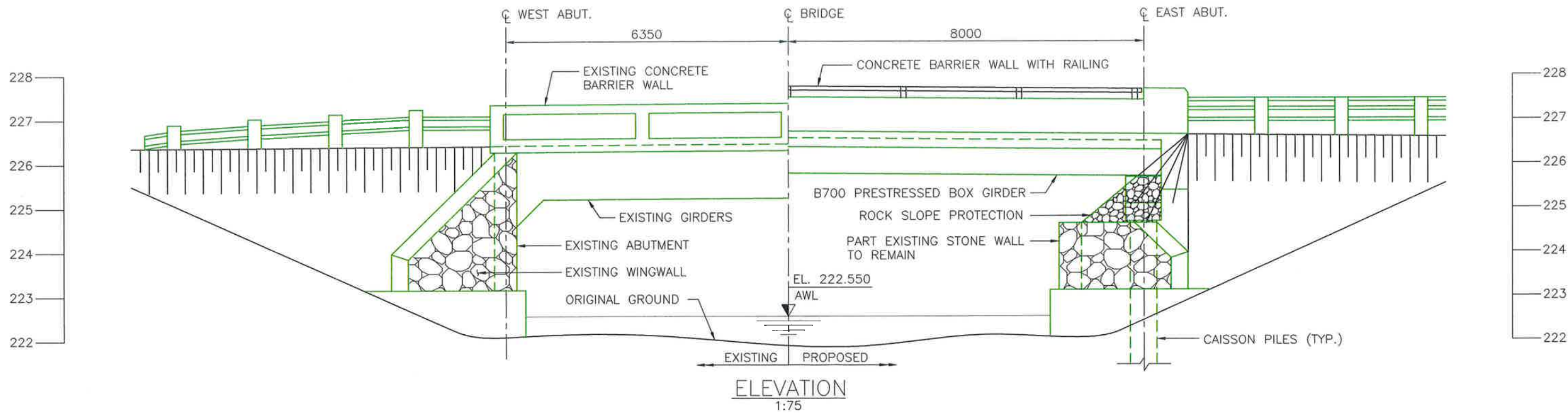


OPSD - 3101.150

Appendix F

GA Drawing





Appendix G

Limitations of Report

LIMITATIONS OF REPORT

This report is intended solely for the Client named. The material in it reflects our best judgment in light of the information available to Coffey Geotechnics Inc. (Coffey) at the time of preparation. Unless otherwise agreed in writing by Coffey, it shall not be used to express or imply warranty as to the fitness of the property for a particular purpose. No portion of this report may be used as a separate entity, it is written to be read in its entirety.

The conclusions and recommendations given in this report are based on information determined at the testhole locations. The information contained herein in no way reflects on the environment aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the testholes may differ from those encountered at the testhole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the testhole locations and should not be used for other purposes, such as grading, excavating, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report.

The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work. This work has been undertaken in accordance with normally accepted geotechnical engineering practices.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. Coffey accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.