FOUNDATION INVESTIGATION AND DESIGN REPORTS – SPENCER CREEK BRIDGE REPLACEMENT
G.W.P. NO. 2174-08-00
MTO GEOCRES NO. 30M5-282

Morrison Hershfield Limited
Highway 5, Township of Beverly, Ontario

TRANETOB11558AA
September 06, 2012

Coffey Geotechnics Inc.
20 Meteor Drive Etobicoke Ontario M9W 1A4 Canada

coffey geotechnics
SPECIALISTS MANAGING THE EARTH

September 06, 2012

Morrison Hershfield
Suite 600, 235 Yorkland Boulevard
Toronto, ON M2J 1T1

Attention: Mr. Joe Ostrowski, P.Eng.

Dear Mr. Ostrowski,

RE: Foundation Investigation and Design Reports
Spencer Creek Bridge Replacement, Highway 5, Township of Beverley, Ontario
G.W.P. No. 2174-08-00 MTO GEOCRES No. 30M5-282

Coffey Geotechnics Inc (Coffey) is pleased to present the Final Foundation Investigation and Design Reports for the proposed Spencer Creek Bridge Replacement on Highway 5, Beverly Township, 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.

for **Ramon Miranda, P.Eng.**

Principal Engineer

Distribution: Original held by Coffey Geotechnics Inc.
1 hard copy to Morrison Hershfield
1 hard copy to MTO Project Manager
1 hard copy to MTO Pavements and Foundation Section

coffey geotechnics
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CONTENTS

CONTENTS

FOUNDATION INVESTIGATION REPORT

SPENCER CREEK BRIDGE REPLACEMENT, HIGHWAY 5,

TOWNSHIP OF BEVERLEY, ONTARIO,

G.W.P. NO. 2174-08-00 - MTO GEOCRES NO. 30M5-282

1 INTRODUCTION

At the request of Morrison Hershfield, Coffey Geotechnics Inc. (Coffey) has prepared this foundation investigation report for the proposed Spencer Creek Bridge Replacement on Highway 5, in the Township of Beverly, Ontario. The foundation investigation was carried out in general accordance with Coffey proposal, dated June 15, 2011) and the requirements of the RFP.

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 Bridge Site # 36–81 (Spencer Creek Bridge) is located at approximately 1.2 kilometres east of intersection of Highway 5 with Westover Road (Peters Corner), at about Station 23+800 on Highway 5, northwest of Hamilton, Ontario.

The existing structure over the Spencer Creek is a single-span (15 m) bridge with a concrete deck and asphalt wearing surface.

The direction of flow in Spencer Creek is southerly. The width of the creek is about 12 m and at the time of our investigation the water depth in the creek was about 0.7 m.

Photographs of the Site are presented in Appendix C.

2.2 Physiography

The site is located in a valley incised by the Spencer Creek near (west of) the Niagara Escarpment and according to “The Physiography of Southern Ontario” by L.J. Chapman and D.F. Putnam (1984), the general area is at confluence of the physiographic regions known as the Flamborough Plain and the Norfolk Sand Plain. The Flamborough Plain spans from Flamborough Township and extends north to Acton (in the Town of Halton Hills). It is bounded on the northwest by the Galt Moraine, and on the south by the silts and sands of glacial Lake Warren. A few drumlins are found scattered over limestone plains and swamps. The plain slopes to the south and carries little overburden like boulder glacial till or sand and gravel. Spencer Creek serves the Beverly swamp and the area north of it and flows into the Dundas Valley.

The Norfolk Sand Plain is wedge shaped and includes the western half of Regional Municipality of Haldimand-Norfolk, the eastern end of Elgin County, southern Brant, and a small corner of Oxford. In general, the plain declines from north to south in a very gentle slope (only about 0.3 m in a kilometer), while a noticeable break in the slope occurs eight to fifteen kilometers from the shore of Lake Erie. Except for the tributary of the Grand River in a small area, the drainage of the plain is through small rivers (e.g. Otter Creek and Big Creek) flowing directly to Lake Erie. This region is characterized by its sand and silt overburden (coarse-textured glaciolacustrine deposits), and usually silt and clay strata or beds of boulder clay occur within 9 m from the surface. The overburden is underlying by the bedrock of Guelph Formation, which typically consists of tan to brown, fine to medium crystalline dolostone. Ontario Geotechnical Borehole database and regional drift thickness mapping indicate that the depth to bedrock in the area of this site may be less than 5 to 12 m.

Being close to the Niagara Escarpment, the bedrock underlying the project area presents a complex picture at the interface of Guelph, Amabel and Lockport Formations, which belong to mainly the Middle Ordovician Period (i.e. approximately 430 million years old). These formations generally consist of dolostone/ limestone with shale, siltstone and sandstone interbeds.

3 METHOD OF INVESTIGATION

3.1 Fieldwork

The fieldwork for the proposed bridge replacement was carried out on December 13, 14 and 15, 2011 and comprised of drilling eight boreholes (A1, A2, and F1 through F6) at the locations shown on the Borehole Location Plan and Soil Strata, Drawing 1. Table 1 below presents a summary of the borehole details.

Table 3.1 – Borehole Details

Borehole No./Location	**Approximate Station**	**Offset from Hwy 5 C/L**	**Existing Ground Elevation (m)**	**Drilled/Tested Depth (m)**
A1 West Approach	23+765	5.1 m Left of C/L	240.4	5.6
A2 East Approach	23+840	5.0 m Right of C/L	238.8	3.5
F1 West Abutment (Northern End)	23+784	5.2 m Left of C/L	239.8	4.9

Borehole No./Location	**Approximate Station**	**Offset from Hwy 5 C/L**	**Existing Ground Elevation (m)**	**Drilled/Tested Depth (m)**
F2 West Abutment (Southern End)	23+786	5.2 m Right of C/L	239.8	8.8
F3 East Abutment (Northern End)	23+820.5	5.2 m Left of C/L	239.1	7.1
F4 East Abutment (Southern End)	23+822.5	5.2 m Right of C/L	239.1	4.4
F5 West Abutment (Northern End)	23+781	5.2 m Left of C/L	239.9	8.7
F6 East Abutment (Southern End)	23+825.5	5.2 m Right of C/L	239.0	7.4

Davis Drilling Limited of Milton, Ontario carried out the drilling operation under the direction and supervision of Coffey geotechnical personnel. As shown on Table 3.1, the depths of the boreholes varied from 3.5 to 8.8 m. The boreholes were drilled using a track mounted (Bombardier) drill rig. Each borehole was advanced using a solid stem flight auger or hollow stem augers within the overburden materials, to depths of about 3.5 m to 5.6 m below the ground surface. Standard Penetration Tests (SPTs) were carried out at frequent 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 over 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).

Boreholes F5 and F6 were straight augered without sampling through the overburden to refusal on presumed bedrock. In Boreholes F2, F3, F5 and F6, rock was cored by at least 3 m to a maximum depth of

8.8 m below the ground surface, using NQ coring technique. Rock core samples were stored in wooden boxes 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. In Boreholes F1 and F4, a piezometer was installed in each borehole to enable long term groundwater level monitoring. The remaining boreholes were grouted upon their completion using a cement/bentonite mixture as per MTO procedures.

The borehole locations were determined in the field, based on the existing site features. The borehole location coordinates and ground elevations were subsequently measured by the client's surveyors and were provided to Coffey.

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;
- Unit weight tests;
- Grain size analyses (sieve);
- Grain size analyses (sieve and hydrometer tests);
- Atterberg Limits tests.

Selected rock core samples were tested for Unconfined Compression Strength Test at Golder Associates Material Testing Laboratory in Mississauga.

The results of all laboratory tests are presented on the Record of Borehole Sheets in Appendix A. Appendix B presents laboratory test results sheets.

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.

Drawing 1 presents the borehole location plan as well as the generalized subsurface profile along the proposed Spencer Creek Bridge.

All the boreholes were advanced from paved shoulder and encountered asphalt and pavement granular fill. Below the pavement granular fill, an embankment fill, consisting generally of sandy silt mixed with clayey silt, was encountered. In some of the boreholes, the embankment fill is underlain by native soils, consisting of clayey silt, to the surface of the bedrock. The bedrock encountered in the boreholes consists of dolostone/limestone and the surface of the bedrock was inferred or proven at depths ranging from 3.5 to 5.6 m below the existing road surface level or at Elevations 235.3 to 234.7 m.

The Record of Borehole Sheets and soil strata indicate the subsurface conditions at the borehole locations. However, the material boundaries indicated on the logs are approximate, 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.

The following description of the individual soil strata is to assist the designers of the project with an understanding of the anticipated subsurface conditions underlying the site.

4.1 Asphalt

All the boreholes drilled from paved shoulder of Highway 5 contacted an asphaltic concrete surface layer ranging in thickness from 100 to 240 mm.

4.2 Pavement Granular Fill

All the boreholes contacted pavement granular fill under the asphalt. The pavement fill consisting of a 0.15 to 0.2 m thick layer of granular base over 0.4 to 0.7 m thick granular sub-base. The base course consists of sand and gravel to gravelly sand with traces to some silt, while the granular sub-base consists of sand with some gravel to sand with traces to some gravel and some silt.

Five grain size analyses were carried out on representative samples from the granular soils that make up this granular fill. The results are presented on the Record of Borehole Sheets in Appendix A, and the grain size distribution curves are presented in Figure B1 in Appendix B. The results indicate the following grain size distribution.

Gravel	7 – 42 %
Sand	45 – 79 %
Silt and Clay	10 – 14 %

Standard Penetration Tests performed in this pavement fill yielded N-values ranging from 11 to 36 blows/0.3 m, indicating compact to dense compactness condition.

4.3 Embankment Fill

Below the pavement fill, embankment fill was encountered in all the sampled boreholes. The embankment fill, contacted in the boreholes, generally consists of sandy silt mixed with clayey silt. The embankment fill was found to extend to depths of 1.7 to 4.4 m below the existing road surface or to Elevations ranging between 238.7 and 234.7 m (probably representing the original ground surface level before the existing bridge was constructed minus the stripped material thickness). The fill contains of traces of gravel, rootlets and organics. In Borehole F3, the bottom portion of fill was found to be comprised of organic silt with some sand and some peat (fibrous black organic matter) with a natural moisture content 52%. The presence of some organic rich soils was also noted in most of the remaining boreholes. In fact, in general, at the bottom of this embankment fill, the presence of some organic contaminated soils is typical, exhibiting a dark brown/greyish black to black colour. This is probably due to the mixing of the existing organic soils with the fill when the embankment was first constructed (i.e. inadequate stripping).

Five grain size analyses were performed on representative samples from this fill and results are presented on the Record of Borehole Sheets in Appendix A, and the grain size distribution curves are presented in Figure B2 in Appendix B. The results indicate the following grain size distribution.

Gravel	1 – 15 %
Sand	20 – 34 %
Silt	38 – 51 %
Clay	14 – 19 %

The soil is basically fine grained granular (i.e. non-cohesive) soil, but at some locations where clay content is high, it attains a basically cohesive character.

The recorded N-values range from 1 to 11 blows/0.3 m, showing very loose to compact relative density, or a very soft to stiff consistency.

The measured bulk unit weight of one selected sample is 20.5 kN/m3.

4.4 Clayey Silt

Underneath the embankment fill, in Boreholes A1, A2, F1 and F2, a clayey silt deposit was encountered at depths ranging from 1.7 to 4.3 m or Elevations 238.7 to 235.5 m, with thicknesses ranging from 0.6 to 3.9 m. The lower extent of the deposit extends to the bedrock surface. This cohesive deposit generally consists of clayey silt with traces of sand and gravel. In Borehole A1, a thin layer of sandy silt with some clay was contacted within this cohesive deposit.

The following are the grain size distributions of the selected three samples retrieved from this deposit, as shown in Figure B3 in Appendix B.

Gravel:	0 – 18 %
Sand:	2 – 23 %
Silt:	41 – 73 %
Clay:	18 – 27 %

Atterberg Limits tests conducted on two representative samples from this deposit indicated the following test results, also shown in Figure B4 in Appendix B.

Liquid Limit:	22 – 24 %
Plastic Limit:	14 %
Plasticity Index:	8 – 10 %

It is noted that in Borehole A2, the top layer of this deposit (0.4 m thick) is described as silty clay and Atterberg Limits tests performed on this sample show the following results, also shown in Figure B4 in Appendix B.

Liquid Limit: 33 %

Plastic Limit: 20 %

Plasticity Index: 13 %

Based on the above results (see Figure B4 in Appendix B), the material is considered to have a low plasticity (i.e. a CL material).

Standard Penetration Tests carried out in this cohesive deposit yielded N-values ranging from 2 to in excess of 24 blows/0.3 m. Based on the SPT results, the clayey silt deposit is considered to have a soft to very stiff consistency, but generally firm to stiff.

4.5 Bedrock

The fill and/or the clayey silt are underlain by bedrock. Bedrock was encountered or inferred at all borehole locations at Elevations ranging between 235.3 and 234.7 m. As presented on the individual Record of Borehole Sheets in Appendix A and also in Table 4.1 below, the presence of bedrock was inferred from refusal to augering in Boreholes F1, F4, A1 and A2, while in Boreholes F2, F3, F5 and F6, the rock was proven, after auger refusal, by diamond drilling and obtaining cores of the rock by NQ coring.

Table 4.1 – Bedrock Level Observations

Borehole No.	**Ground (Road) Surface Elevation (m)**	**Depth below Ground Surface/Elevation of the Bedrock Surface (m)**	**TCR * (%)**	**RQD ** (%)**
F1	239.8	4.9 / 234.9 ***	N/A	N/A
F2	239.8	5.1 / 234.7	94 - 100	83 - 92
F3	239.1	4.0 / 235.1	89	64
F4	239.1	4.4 / 234.7 ***	N/A	N/A
F5	239.9	5.2 / 234.7	93 - 100	91 - 100
F6	239.0	4.3 / 234.7	97	90
A1	240.4	5.6 / 234.8 ***	N/A	N/A

Borehole No.	Ground (Road) Surface Elevation (m)	Depth below Ground Surface/Elevation of the Bedrock Surface (m)	TCR * (%)	RQD ** (%)
A2	238.8	3.5 / 235.3 ***	N/A	N/A

* TCR = Total Core Recovery

** RQD = Rock Quality Designation

*** Inferred bedrock depth/elevation (no coring)

From the table presented above, the surface of the bedrock appears to be relatively flat, exhibiting an elevation difference of 0.6 m (i.e. between Elevations 235.3 and 234.7 m) at the borehole locations. It should however be pointed out that the surface of the bedrock may be different at other locations than at the boreholes, especially within the creek bed where it may be lower due to scour. From the rock cores, the bedrock is described as a light grey, slightly weathered, fine grained dolostone/limestone. As mentioned before, from the published information, the bedrock in this area is known to consist of dolostones, limestones with sandstone and shale and some siltstone gypsum and salt inclusions, and belongs to the Middle Ordovician Period (i.e. approximately 430 million years old).

The Total Core Recovery (TCR) measured in the rock cores ranged from 89 to 100% and Rock Quality Designation (RQD) values of 64 to 100% were recorded. Based on these values, the rock mass quality can be described as fair to excellent but typically good to excellent. Unconfined Compression tests were performed on two selected rock core samples from Boreholes F2 and F3 and the tests yielded Unconfined Compression Strength (UCS) values of 50 MPa (Borehole F2) and 148 MPa (Borehole F3). These results indicate that the rock can be classified as being generally strong to very strong, according to the International Society of Rock Mechanics (ISRM) classification.

4.6 Groundwater Conditions

Groundwater levels were observed in the open boreholes while drilling and upon completion of each borehole. Standpipe piezometers were installed in each of Boreholes F1 and F4. The observations made in the boreholes are shown on the individual Record of Borehole Sheets in Appendix A, and are summarized in the following Table 4.2.

Table 4.2 – Groundwater Level Observations

Borehole No.	Depth/Elevation of the Tip of Piezometer (m)	Date of Water Level Measurement	Measured Water Level Depth/Elevation (m)	Comments
F1	4.9 / 234.9 piezometer installed	Dec. 14, 2011 (completion) Dec. 15, 2011	No water observed * 2.4 / 237.4	First reading measured just after installing piezometer; the second reading one day thereafter.
F2	-	Dec. 14, 2011 (completion)	5.1 / 234.7 *	Measured upon borehole completion of overburden drilling, before coring.
F3	-	Dec. 14, 2011 (completion)	4.0 / 235.1 **	Wet cave-in depth measured upon completion of overburden drilling, before coring.
F4	4.4 / 234.7 piezometer installed	Dec. 14, 2011 (completion) Dec. 15, 2011	No water observed * 2.5 / 236.6	First reading measured just after installing piezometer; the second reading one day thereafter.
A1	-	Dec. 13, 2011 (completion)	No water observed *	Measured upon borehole completion.
A2	-	Dec. 13, 2011 (completion)	No water observed *	Measured upon borehole completion.

* Groundwater level measured not stabilized
** Cave-in depth measured

Based on the above observations and in particular the measurements conducted in the standpipe piezometers installed in Boreholes F1 and F4, which are believed to represent the stabilized groundwater conditions at the time of our investigation, in general, the groundwater table at the site at the time of our investigation was about 2 to 3.5 m below the existing highway surface level or between about Elevations 238 and 236 m. The water level measured in the Spencer Creek on November 01, 2011 was at Elevation 235.5 m.

It should be noted that groundwater levels are subject to variations due to the influence of rainfall, temperature, local drainage, seasons and other factors. There may also be potential for the development of perched groundwater following periods of rainfall and groundwater may rise up to or within pavement granular fill. In addition, the water level in the watercourse would influence the groundwater level at the site.

For and on behalf of Coffey Geotechnics Inc.

Sanket Shah, B.Eng., E.I.T.

Engineer-in-Training

LICENSED PROFESSIONAL ENGINEER
RAMON MIRANDA
PROVINCE OF ONTARIO

for **Ramon Miranda, P.Eng.**

Principal

REGISTERED PROFESSIONAL ENGINEER
Z. S. OZDEN
PROVINCE OF ONTARIO

Zuhtu Ozden, P.Eng.

Senior Principal

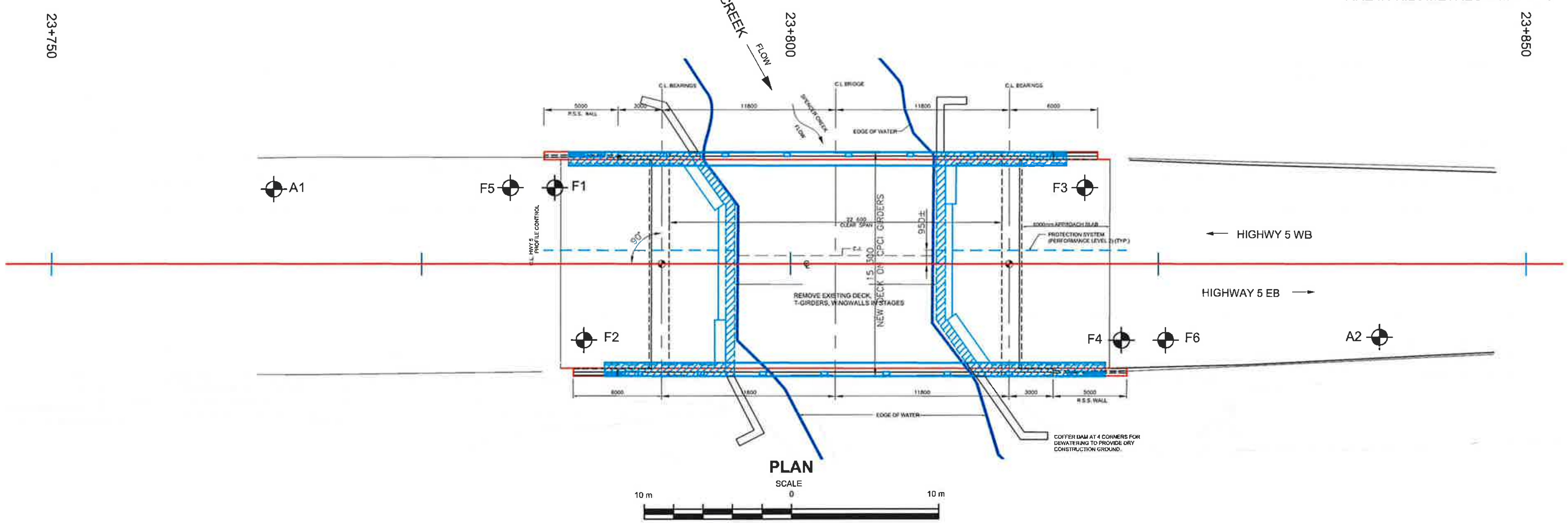
Drawings

NOTES:

FOR DETAILED SUBSURFACE CONDITIONS REFER TO RECORD OF BOREHOLE SHEETS.

METRIC

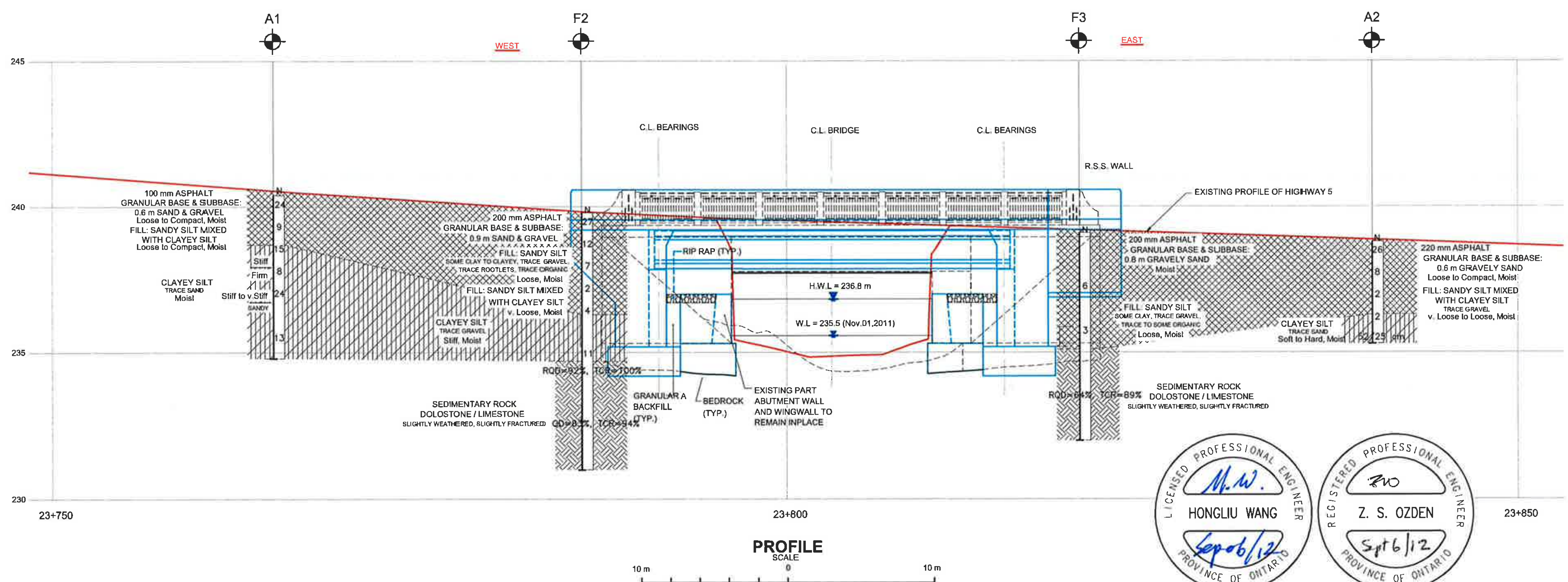
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES + METRES.



PLAN

SCALE

10 m 0 10 m



PROFILE

SCALE

10 m 0 10 m

LICENSED PROFESSIONAL ENGINEER
HONGLIU WANG
Sep 06/12
PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER
Z. S. OZDEN
Sept 6/12
PROVINCE OF ONTARIO

CONT No. -

GWP: 2174-08-00

SPENCER CREEK
BRIDGE REPLACEMENT HIGHWAY 5
BOREHOLE LOCATION PLAN
AND SOIL STRATA

SHEET

coffey geotechnics
SPECIALISTS MANAGING THE EARTH

KEY PLAN
N.T.S.

LEGEND

Borehole & Cone

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

Water Level at Time of Investigation (W. L. NOT STABILIZED)

Water Level in Piezometer

Piezometer

No.	ELEVATION	EASTING	NORTHING
A1	240.4	259867.3	4793823.1
A2	238.8	259942.8	4793828.5
F1	239.8	259885.9	4793827.1
F2	239.8	259889.9	4793817.3
F3	239.1	259921.1	4793834.4
F4	239.1	259925.7	4793824.8
F5	239.9	259882.9	4793826.5
F6	239.0	259928.6	4793825.4

-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. 30M5-282

	TRANETOB11558AA		DIST
SUBM'D	CHECKED	DATE Feb.10, 2012	SITE -
DRAWN SSH	CHECKED RM	APPROVED ZO	DWG 1

Appendix A

Record of Borehole Sheets and Rock Core Photographs

RECORD OF BOREHOLE No A1

METRIC

GWP	G.W.P 2174-08-00	LOCATION	Station 23+765, 5.1 m Lt C/L of Highway 5 (N 4793823.1, E 259867.3)
DIST	HWY 5	BOREHOLE TYPE	Solid Stem Auger
DATUM	Geodetic	DATE	12/13/2011

ORIGINATED BY AS
COMPILED BY SK
CHECKED BY ZO

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
240.4	GROUND SURFACE						
0.0	100 mm **ASPHALT** **GRANULAR BASE:** 0.2 m Sand and Gravel some silt, brown, loose to compact, moist	1	SS	24	240	o	40 47 (13)
239.7 / 0.7	**GRANULAR SUBBASE:** 0.4 m Sand, some gravel, some silt brown, loose to compact, moist **FILL:** Sandy Silt mixed with clayey silt brown, loose to compact, moist	2	SS	9	239	o	
238.7 / 1.7	**CLAYEY SILT** tr. sand, brown, moist — stiff	3	SS	15		o	
	firm	4	SS	8	238	o	1 8 64 27
	stiff to v. stiff; sandy	5	SS	24	237	o	
	brown to dk. brown	6	SS	13	236	├─o─┤	
234.8 / 5.6	End of Borehole Auger refusal @ 5.6 m probably on bedrock Borehole dry (not stabilized)* and open upon completion				235		

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH (kPa) O UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L, WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

$+^3, \times^3$: Numbers refer to Sensitivity

15-⊕-5 (20 above, 10 below) (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

Foundation Design

TRANETOB11558AA: Spencer Creek Bridge

RECORD OF BOREHOLE No A2

METRIC

GWP G.W.P 2174-08-00 LOCATION Station 23+840, 5.0 m Rt C/L of Highway 5 (N 4793828.5, E 259942.8) ORIGINATED BY AS

DIST HWY 5 BOREHOLE TYPE Solid Stem Auger COMPILED BY SK

DATUM Geodetic DATE 12/13/2011 CHECKED BY ZO

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
238.8	GROUND SURFACE						
0.0	220 mm **ASPHALT** **GRANULAR BASE:** 0.15 m Gravelly Sand brown, loose to compact, moist	1	SS	26			
238.0 0.8	**GRANULAR SUBBASE:** 0.4 m Sand, some gravel, tr. to some silt brown, loose to compact, moist				238		22 68 (10)
	FILL: Sandy Silt mixed with clayey silt tr. gravel brown, v. loose to loose, moist	2	SS	8			
		3	SS	2	237		
236.3 2.5	some org. greyish black						9 32 45 14
	CLAYEY SILT tr. sand greyish black to grey, soft to hard, moist; silty clay	4	SS	2	236		
235.3		5	SS	52/25.0 cm			spoon bouncing
3.5	End of Borehole Auger refusal @ 3.5 m probably on bedrock Borehole dry (not stabilized)* and open upon completion						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH (kPa): O UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE: 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m^3

GROUND WATER CONDITIONS; STRAT PLOT

$+^3, \times^3$: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No F1

METRIC

GWP: G.W.P 2174-08-00 | LOCATION: Station 23+784, 5.2 m Lt C/L of Highway 5 (N 4793827.1, E 259885.9) | ORIGINATED BY: AS

DIST: | HWY: 5 | BOREHOLE TYPE: Solid Stem Auger | COMPILED BY: SK

DATUM: Geodetic | DATE: 12/14/2011 | CHECKED BY: ZO

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL	REMARKS
239.8	GROUND SURFACE										
0.0	240 mm **ASPHALT** **GRANULAR BASE:** 0.15 m Sand and Gravel some silt, brown, loose to compact, moist **GRANULAR SUBBASE:** 0.7 m Sand, tr. to some gravel, some silt brown, loose to compact, moist	1	SS	33			42	45	(13)		
238.7 / 1.1		2	SS	15	239						
	FILL: Sandy Silt mixed with clayey silt tr. gravel, tr. org. brown, v. loose to compact, moist	3	SS	5	238		1	30	51	18	
	org. silt some rootlets greyish black	4	SS	1	237						
		5	SS	1	236						
235.5 / 4.3	**CLAYEY SILT** some sand to sandy, tr. gravel brown, stiff to hard, moist	6	SS	59/15 cm	235		18	23	41	18	Spoon bouncing @ 4.9 m
234.9 / 4.9	End of Borehole										

End of Borehole
Auger refusal @ 4.9 m on bedrock
Piezometer installed to 4.9 m
Water level Records:
Dec 14, 2011 dry
Dec 15, 2011 2.4 m

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH (kPa): O UNCONFINED, + FIELD VANE, ● POCKET PENETR., × LAB VANE (20, 40, 60, 80, 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10, 20, 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

+³, ×³: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No F2

METRIC

GWP G.W.P 2174-08-00 | LOCATION Station 23+786, 5.2 m Rt C/L of Highway 5 (N 4793817.32, E 259889.93) | ORIGINATED BY AS

DIST | HWY 5 | BOREHOLE TYPE Hollow Stem Auger, NQ Coring, NW Casing | COMPILED BY SK

DATUM Geodetic | DATE 12/14/2011 | CHECKED BY ZO

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
239.8	GROUND SURFACE							
0.0	200 mm **ASPHALT** **GRANULAR BASE:** 0.2 m Sand and Gravel brown, loose to compact, moist **GRANULAR SUBBASE:** 0.7 m Sand, tr. to some gravel, some silt brown, loose to compact, moist	1	SS	27				
238.7 / 1.1		2	SS	12		239		7 79 (14)
	FILL: Sandy Silt some clay to clayey tr. gravel, tr. rootlets, tr. org. brown, loose, moist	3	SS	7		238		5 28 48 19
237.5 / 2.3	**FILL:** Sandy Silt mixed with clayey silt tr. gravel, tr. rootlets, tr. org. brown, v. loose, moist	4	SS	2		237		
	black, some org.	5	SS	4				
236.3 / 3.5	sandy brown **CLAYEY SILT** tr. gravel, grey, stiff, moist					236		
		6	SS	11	▼*	235		0 2 73 25
234.7 / 5.1	**SEDIMENTARY BEDROCK** dolostone / limestone light grey, slightly weathered slightly fractured	7	RC	RQD=92% TCR=100%		234		NQ Coring started @ 5.1 m UCS = 50.5 MPa
		8	RC	RQD=83% TCR=94%		233, 232		
231.0 / 8.8	End of Borehole Water level @ 5.1 m (not stabilized)* upon completion.					231		

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE 20 40 60 80 100
PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30
UNIT WEIGHT γ kN/m³

$+^3, \times^3$: Numbers refer to Sensitivity — 15-⊕-5 (20 above, 10 below) (%) STRAIN AT FAILURE

TRANETOB11558AA: Spencer Creek Bridge

RECORD OF BOREHOLE No F3

METRIC

GWP G.W.P 2174-08-00 | LOCATION Station 23+820.5, 5.2 m Lt C/L of Highway 5 (N 4793834.44, E 259921.1) | ORIGINATED BY AS

DIST ___ HWY 5 | BOREHOLE TYPE Hollow Stem Auger, NQ Coring, NW Casing | COMPILED BY SK

DATUM Geodetic | DATE 12/14/2011 | CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
239.1	GROUND SURFACE								
0.0	200 mm **ASPHALT** **GRANULAR BASE:** 0.2 m Gravelly Sand brown, moist **GRANULAR SUBBASE:** 0.6 m Sand, some gravel, some silt brown, moist					239			
238.1 / 1.0	**FILL:** Sandy Silt some clay, tr. gravel tr. to some org. brown, loose, moist	1	SS	6		238 / 237			12 34 38 16
	org. silt some sand tr. rootlets black, some peat, v. loose	2	SS	3		236		52	
235.1 / 4.0	**SEDIMENTARY BEDROCK** dolostone / limestone light grey, slightly weathered moderately fractured	3	RC	RQD=64% TCR=89%		235 / 234 / 233			NQ Coring started @ 4.0 m; UCS = 148 MPa
232.0 / 7.1	End of Borehole Wet cave @ 4.0 m upon completion.					232			

+ 3, × 3: Numbers refer to Sensitivity

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

RECORD OF BOREHOLE No F4

METRIC

GWP G.W.P 2174-08-00	LOCATION Station 23+822.5, 5.2 m Rt C/L of Highway 5 (N 4793824.8, E 259925.7)		ORIGINATED BY AS
DIST	HWY 5	BOREHOLE TYPE Solid Stem Auger	COMPILED BY SK
DATUM Geodetic	DATE 12/14/2011		CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	UNIT WEIGHT (kN/m³)	GR	SA	SI	CL
239.1	GROUND SURFACE									
0.0	210 mm **ASPHALT** **GRANULAR BASE:** 0.2 m Sand and Gravel some silt, brown, compact to dense, moist **GRANULAR SUBBASE:** 0.6 m Sand, some gravel, some silt brown, compact to dense, moist	1	SS	36	239		41	46	(13)	
238.1 / 1.0	**FILL:** Sandy Silt mixed with clayey silt tr. gravel, tr. org. brown, soft to firm / v. loose to loose, moist	2	SS	11	238					
		3	SS	6	237	20.5				
		4	SS	3			15	20	51	14
	some org. black tr. rootlets	5	SS	6	236					
234.7 / 4.4	End of Borehole Auger refusal @ 4.4 m on bedrock Piezometer installed to 4.4 m Water level Records: Dec 14, 2011 dry Dec 15, 2011 2.5 m				235					

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH (kPa): O UNCONFINED, + FIELD VANE, ● POCKET PENETR., × LAB VANE (20, 40, 60, 80, 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (10, 20, 30); REMARKS & GRAIN SIZE DISTRIBUTION (%)

$+^3, \times^3$: Numbers refer to Sensitivity

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

RECORD OF BOREHOLE No F5

METRIC

GWP: G.W.P 2174-08-00
LOCATION: Station 23+781, 5.2 m Lt C/L of Highway 5 (N 4793826.49, E 259882.92)
ORIGINATED BY: AS
DIST:
HWY: 5
BOREHOLE TYPE: Hollow Stem Auger, NQ Coring, NW Casing
COMPILED BY: SK
DATUM: Geodetic
DATE: 12/14/2011
CHECKED BY: ZO

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	STRAT PLOT	SAMPLES – NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH (kPa) (O UNCONFINED, + FIELD VANE, ● POCKET PENETR., × LAB VANE) 20 40 60 80 100	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
239.9 / 0.0	GROUND SURFACE										
	Straight auger up to 5.2 m						239, 238, 237, 236, 235				
234.7 / 5.2	**SEDIMENTARY BEDROCK** dolostone / limestone light grey, slightly weathered slightly fractured		1	RC	RQD=100% TCR=100%		234				
			2	RC	RQD=91% TCR=93%		233, 232				
231.2 / 8.7	End of Borehole Water used to core bedrock Water level unreliable										

$+^3$, $\times^3$: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No F6

METRIC

GWP G.W.P 2174-08-00 | LOCATION Station 23+825.5, 5.2 m Rt C/L of Highway 5 (N 4793825.38, E 259928.6) | ORIGINATED BY AS

DIST ____ HWY 5 | BOREHOLE TYPE Hollow Stem Auger, NQ Coring, NW Casing | COMPILED BY SK

DATUM Geodetic | DATE 12/14/2011 | CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH (kPa) O UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE (20 40 60 80 100)	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
239.0	GROUND SURFACE						239				
0.0	Straight auger up to 5.2 m						238 237 236 235				
234.7 4.3	**SEDIMENTARY BEDROCK** dolostone / limestone light grey, slightly weathered slightly fractured		1	RC	RQD=90% TCR=97%		234 233 232				
231.6 7.4	End of Borehole Water used to core bedrock Water level unreliable										

+ 3, × 3: Numbers refer to Sensitivity

15-⌖-5 (20 above, 10 below) (%) STRAIN AT FAILURE

BOREHOLE F2 RC7

BOREHOLE F2 RC8

BOREHOLE F3 RC3

BOREHOLE F5 RC1

BOREHOLE F5 RC2

BOREHOLE F6 RC1

Appendix B

Laboratory Test Results

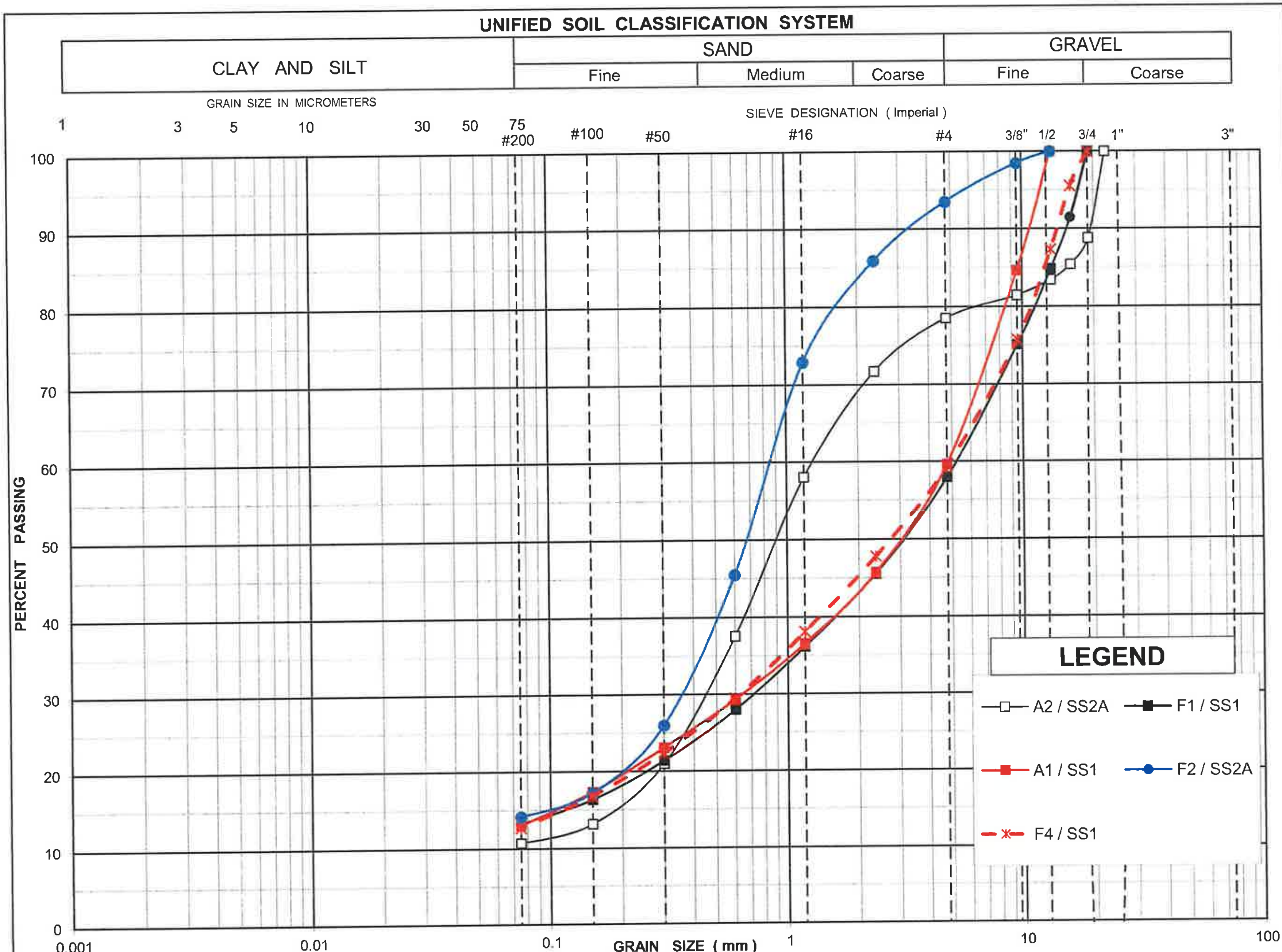


FIGURE NO.:	B1
PROJECT NO:	TRANETOB11558AA
DATE:	Jan, 2012

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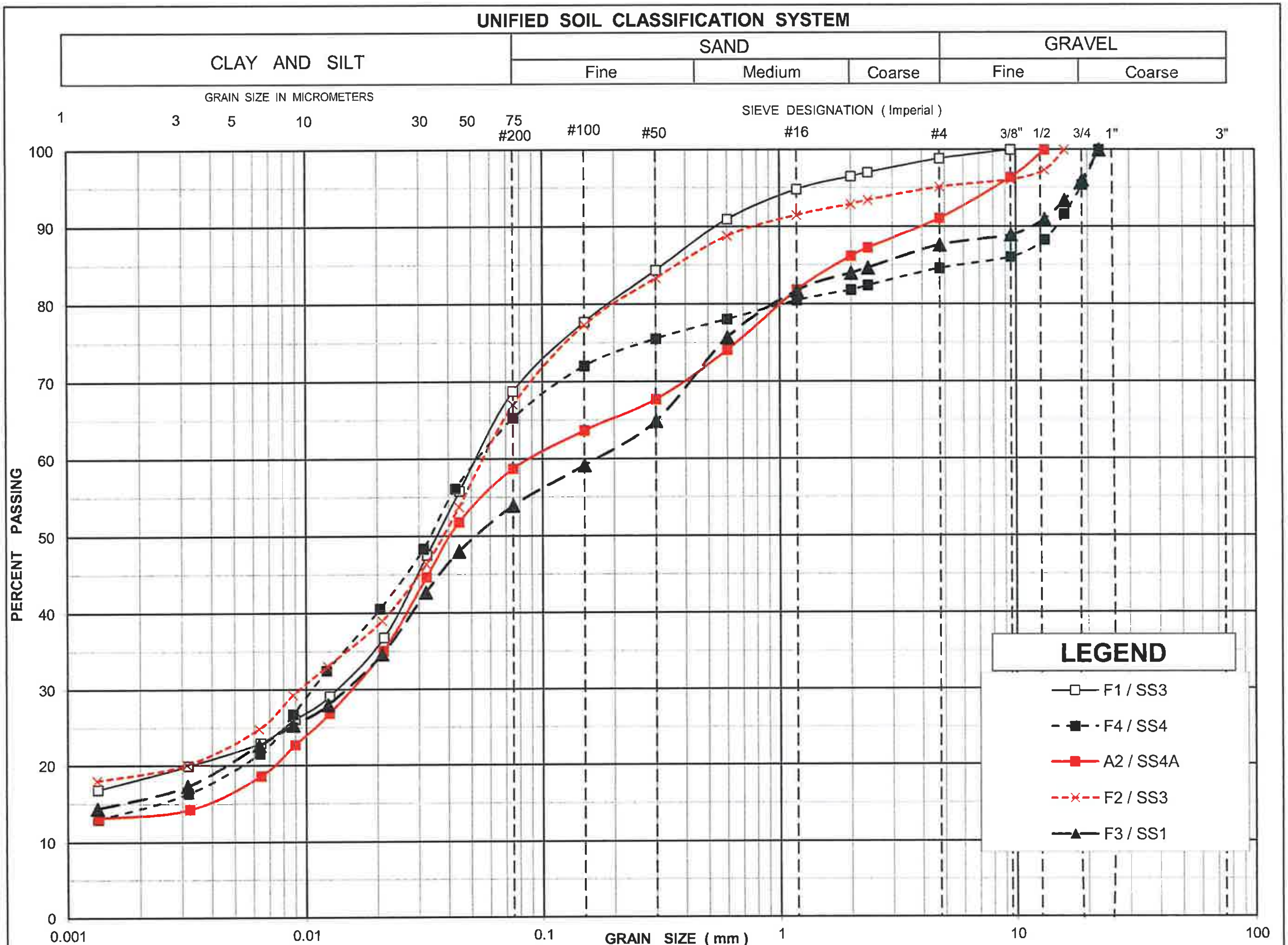
GRAIN SIZE DISTRIBUTION

PAVEMENT GRANULAR FILL



coffey geotechnics
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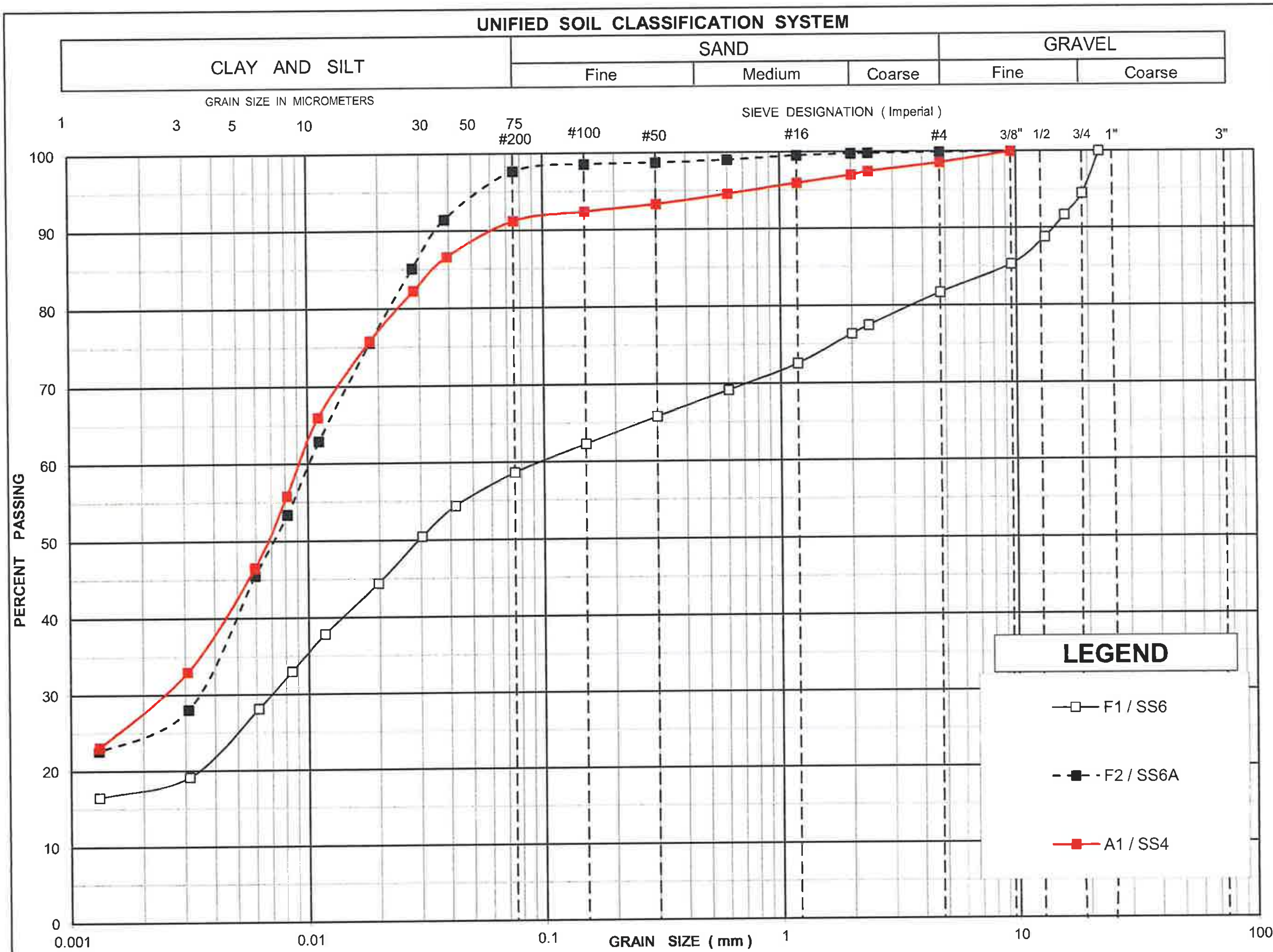
GRAIN SIZE DISTRIBUTION

EMBANKMENT FILL: Sandy Silt mixed with clayey silt

FIGURE NO.: B2

PROJECT NO: TRANETOB11558AA

DATE: Jan, 2012



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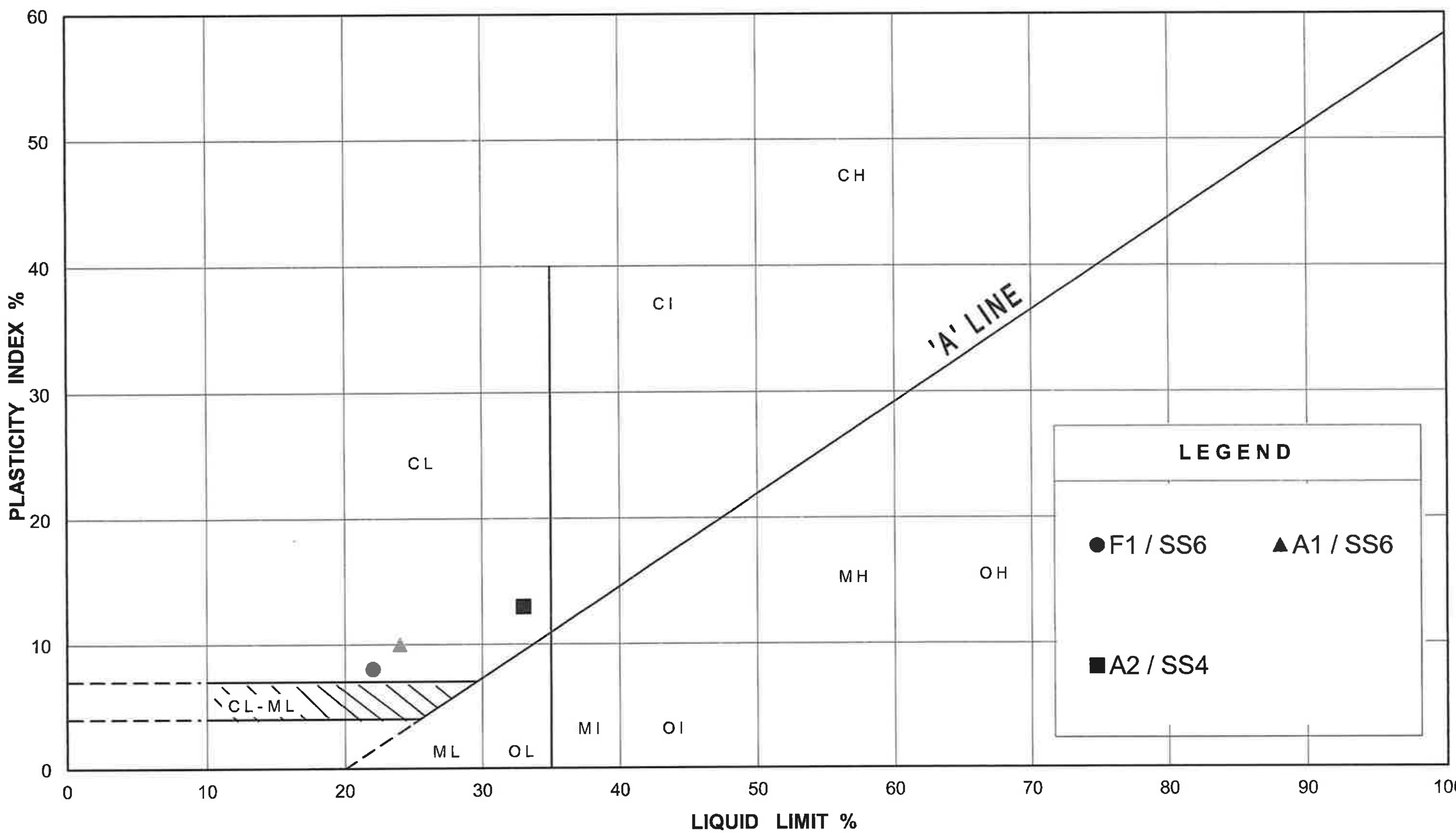
SPECIALISTS MANAGING THE EARTH

GRAIN SIZE DISTRIBUTION

CLAYEY SILT

FIGURE NO.: B3
PROJECT NO: TRANETOB11558AA
DATE: Jan, 2012



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PLASTICITY CHART

CLAYEY SILT

FIGURE No.	B4
REF. No.	TRANETOB11558AA
DATE	Jan 2012

UNCONFINED COMPRESSION TEST (UC)

ASTM D 7012-07

SAMPLE IDENTIFICATION

PROJECT NUMBER	12-1183-0004	SAMPLE NUMBER	8
BOREHOLE NUMBER	F2	SAMPLE DEPTH, m	6.14-6.33

TEST CONDITIONS

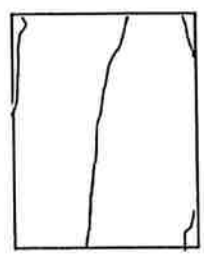
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.46

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	11.64	WATER CONTENT, (specimen) %	0.15
SAMPLE DIAMETER, cm	4.74	UNIT WEIGHT, kN/m^3	26.64
SAMPLE AREA, cm^2	17.63	DRY UNIT WT., kN/m^3	26.60
SAMPLE VOLUME, cm^3	205.23	SPECIFIC GRAVITY, assumed	2.80
WET WEIGHT, g	557.75	VOID RATIO	0.03
DRY WEIGHT, g	556.90		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	50.5

REMARKS:

DATE: 1/10/2012

Checked By: MM

Golder Associates

FIGURE NO.: B5

UNCONFINED COMPRESSION TEST (UC)

ASTM D 7012-07

SAMPLE IDENTIFICATION

PROJECT NUMBER	12-1183-0004	SAMPLE NUMBER	3
BOREHOLE NUMBER	F3	SAMPLE DEPTH, m	5.13-5.31

TEST CONDITIONS

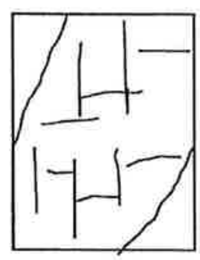
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.46

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	11.75	WATER CONTENT, (specimen) %	0.11
SAMPLE DIAMETER, cm	4.78	UNIT WEIGHT, kN/m^3	26.94
SAMPLE AREA, cm^2	17.93	DRY UNIT WT., kN/m^3	26.91
SAMPLE VOLUME, cm^3	210.61	SPECIFIC GRAVITY, assumed	2.80
WET WEIGHT, g	578.70	VOID RATIO	0.02
DRY WEIGHT, g	578.04		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	148.1

REMARKS:

DATE: 1/10/2012

Checked By: [signature]

Golder Associates

FIGURE NO.: B6

Appendix C

Site Photographs

Photograph 1. Existing Spencer Creek Bridge (looking east)

Photograph 2. Existing Spencer Creek Bridge (looking west)

Photograph 3. Existing Spencer Creek Bridge (looking northwest from south side)

Photograph 4. Existing Spencer Creek Bridge (East Abutment - looking north from south side)

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 $\overline{N}$.

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.

<u>CONSISTENCY</u>: 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*

<u>DENSENESS</u>: 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 COMPOSION AND STRUCUTRAL FEATURES AND/OR STRENGTH.

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

<u>MODIFIED RECOVERY</u>: 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*

<u>JOINT AND BEDDING</u>:

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
$\sigma_l, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ε	%	LINEAR STRAIN
$\varepsilon_1, \varepsilon_2, \varepsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICALL 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'	$-^o$	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	$-^o$	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	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
i_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES	n	1,%	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
P_w	kg/m^3	DENSITY OF WATER	w	1,%	WATER CONTENT	D	mm	GRAIN DIAMETER
i_w	kN/m^3	UNIT WEIGHT OF WATER	s_r	%	DEGREE OF SATURATION	D_n	mm	N PERCENT – DIAMETER
P	kg/m^3	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
i	kN/m^3	UNIT WEIGHT OF SOIL	w_P	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
P_d	kg/m^3	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m^3/s	RATE OF DISCHARGE
i_d	kN/m^3	UNIT WEIGHT OF DRY SOIL	I_P	%	PLASTICITY INDEX = $(W_L – W_L)$	v	m/s	DISCHARGE VELOCITY
P_{sat}	kg/m^3	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $(W – W_P)/ I_P$	i	1	HYDAULIC GRADIENT
i_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $(W_L – W) / 1_P$	k	m/s	HYDRAULIC CONDUCTIVITY
P'	kg/m^3	DENSITY OF SUBMERED SOIL	e_{max}	1,%	VOID RATIO IN LOOSEST STATE	j	kN/m^3	SEEPAGE FORCE
i'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL						

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SPECIALISTS MANAGING THE EARTH

FOUNDATION DESIGN REPORT, SPENCER CREEK BRIDGE REPLACEMENT G.W.P. NO. 2174-08-00 MTO GEOCRES NO. 30M5-282

Morrison Hershfield Limited
Highway 5, Township Of Beverley, Ontario

TRANETOB11558AA
September 06, 2012

Coffey Geotechnics Inc.
20 Meteor Drive Etobicoke Ontario M9W 1A4 Canada

CONTENTS

CONTENTS

Tables

Appendices

FOUNDATION DESIGN REPORT

SPENCER CREEK BRIDGE REPLACEMENT, HIGHWAY 5,

TOWNSHIP OF BEVERLEY, ONTARIO,

G.W.P. NO. 2174-08-00 - MTO GEOCRES NO. 30M5-282

5 DISCUSSIONS AND RECOMMENDATIONS

5.1 General

The existing bridge, which carries Highway 5 over the Spencer Creek, north-west of the Town of Dundas in the Township of Beverley, is to be replaced with a longer structure. The existing structure is 14.6 m long (center to center) and 15.3 m in width. The replacement bridge will be a 23.6 m long and 15.3 m wide structure (i.e. the same width as the existing bridge), along the same alignment centerline as the existing. We understand that the existing bridge is supported on normal spread footing foundations bearing on the bedrock. We also understand that the existing footings and the lower portions of the existing abutments will remain in place, while the new footings and abutments will be constructed immediately behind the existing.

Eight boreholes were drilled from the top of the existing road (i.e. Highway 5) embankment and these boreholes indicate below some asphalt, pavement fill and embankment fill, the presence of a native clayey silt in some of the boreholes. These overburden soils are underlain by dolostone/limestone bedrock at depths ranging from 3.5 to 5.6 m below the road level. The pavement fill consists of 100 to 240 mm thick layer of asphalt over a 0.6 to 0.9 m thick layer of typically compact to dense sand and gravel pavement fill. The underlying embankment fill was found to extend to depths of 1.7 to 4.4 m below the ground surface or to Elevations 238.7 to 234.7 m and generally consists of sandy silt mixed with clayey silt and is in a very loose to compact or very soft to stiff condition. Below the embankment fill, native overburden was encountered in Boreholes F1, F2, A1 and A2 at 1.7 to 4.3 m below the ground surface or at Elevations 238.7 to 235.5 m with thicknesses of about 0.6 to 3.9 m. The native overburden consists of a cohesive clayey silt deposit, and has a typically firm to stiff consistency. It is noted that at the east approach of the proposed bridge (i.e. Borehole A2), a soft layer of silty clay zone was encountered within the clayey silt deposit.

The surface of the bedrock was found (i.e. cored) or inferred (i.e. auger refusal) to be at 3.5 to 5.6 m below the existing ground surface or at Elevations 235.3 to 234.7 m. Based on the available subsurface conditions, the surface of bedrock appears to be relatively flat. However, it should be noted that in between and beyond borehole locations, the bedrock surface and the depth to the suitable bedrock surface may vary.

Based on the rock cores recovered, the bedrock was described as light grey, slightly weathered, slightly fractured dolostone/limestone. According to the Rock Quality Designation (RQD) of the rock cores, ranging from 64% to 100%, the dolostone/limestone can be classified as fair to excellent rock mass quality. The unconfined compressive strengths of two rock cores were 50.5 and 148 MPa, respectively, indicating strong to very strong rock strength (R4 to R5 Grade).

The groundwater level at the time of our investigation was estimated to range between Elevations 238 and 236 m, but would be subject to seasonal variations and variations in response to major weather events, as well as being influenced by the water level in the creek.

5.2 Bridge Foundations

We understand that the new bridge will be constructed on the existing bridge alignment and it will be a single span bridge, about 23.6 m in length (i.e. about 9.0 m longer than the existing bridge). The width will remain the same, at 15.3 m. At present, the following sequence of events is envisaged. One half of the existing bridge will be cut and demolished, while maintaining the traffic on the remaining half. After completion of the construction of one half of the new bridge, it will carry the traffic while the other half is being demolished and reconstructed.

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 (similar to the existing bridge). The overburden soils are unsuitable to support the new bridge. We understand that the existing abutment footings (which rest on the bedrock) will not be removed except for the top portions of the abutment walls which will be cut-off, based on preliminary GA drawing provided to us by Morrison Hershfield. As well, we understand that the new footings will be constructed at a distance of about 4.5 m (center to center) behind the existing abutment footings. If, however, excavation for new spread footing foundations near the existing structure is unacceptable for strategic reasons, the proposed bridge may be supported on deep foundations, if desired, as discussed later on in this report.

Shallow and deep foundations alternatives are discussed in the following sections, and a summary of foundation alternatives is given in a tabular form in Appendix E.

5.2.1 Shallow Foundations

The bedrock surface level at the borehole locations was found (proven by coring) or inferred (auger refusal) to vary in depth from 3.5 to a maximum of 5.6 m below the existing ground surface, in the F-series boreholes. The abutment footings can be founded on the dolostone/limestone bedrock, as presented below.

Table 5.1 – Spread Footing Foundations on Bedrock

Location / Borehole Number	Existing Ground Surface Elevation (m)	Recommended Highest Founding Depth / Elevation (m)	Bearing Stratum	Recommended Factored Bearing Resistance at ULS (kPa)	Recommended Bearing Resistance at SLS (kPa)
West Abutment					
BH F1	239.8	5.1 / 234.7 *	bedrock *	6,000	not govern
BH F2	239.8	5.3 / 234.5	bedrock	6,000	not govern
BH F5	239.9	5.4 / 234.5	bedrock	6,000	not govern
East Abutment					
BH F3	239.1	4.2 / 234.9	bedrock	6,000	not govern
BH F4	239.1	4.6 / 234.5 *	bedrock *	6,000	not govern
BH F6	239.0	4.5 / 234.5	bedrock	6,000	not govern

* inferred

The extent of the rock excavation will depend on the actual founding level. For this purpose, all loose, fractured or weathered bedrock under the footprint of the footing should be removed and replaced with concrete. Mass concrete may be placed to raise the grade to the founding level, where necessary. All excavations should be carried out in conformance with Excavation and Backfilling - Structures (OPSS 902).

The quoted ULS value above 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 completeness.

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.

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 sloping rock, the rock near the edge beneath the footing may break-off (e.g. spalling), thus undermining the footing. For this reason, the foundation must be placed sufficiently away from the edge of the steeply sloping rock. With the presently proposed scheme, the new footings are to be placed behind the existing ones, and as such, this would unlikely present problems, but this aspect is mentioned for the sake of completeness (or if the design is changed).

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^o. 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^o 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 Quality Verification Engineering (QVE). This is very important for this project for the following reasons.

According to OPSD 3090.101, normally for frost protection in this geographic area, the footings should have a permanent earth cover of at least 1.2 m. 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 to ensure its suitability for supporting design resistance values. If the minimum required frost depth cannot be provided, although it is not likely, the surface of the rock to receive the footings 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 and/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 understand, however, that the existing footings will not be removed and only the top portion will be cut-off. In that case, if the space between the new and the existing footings will be backfilled after the construction of the new footings, this will probably provide sufficient frost protection provided that the height of the fill is not less than 1.2 m and that adequate drainage to prevent the accumulation of water is maintained. If, however, frost protection and drainage cannot be provided, we recommend an NSSP be prepared to ensure that the rock which will receive the footings 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 water seepage and penetration (e.g. grouting and/or placing a suitable 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. 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 made free from any loose debris prior to concreting of footings. For frictional horizontal resistance, the surface of the rock must be sufficiently cleaned prior to pouring the concrete.

5.2.2 Deep Foundations

Based on the findings of boreholes, the depth from the top of the existing (and proposed) embankment to the surface at the bedrock at the proposed bridge support locations appears to be generally 4 to 5 m. For this reason, it may 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 1.2 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 strong sedimentary bedrock at the site. Sloping of bedrock also needs to be taken into account for deep foundation design and construction, if necessary. We understand that a rigid frame structure is proposed. If, however, an integral abutment option needs to be considered, then this is likely to be an expensive option from foundation point of view, as it will require drilling rather deep holes in the bedrock to provide the required flexing opportunity for the steel H-piles. In this instance, a semi-integral abutment may be a better choice.

With these preambles, the following are our recommendations regarding deep foundations.

5.2.2.1 Timber Piles

Pile length may be extremely short (less than 3 m). As well, based on the prevailing subsurface conditions, timber piles are not feasible for this project due to the anticipated hard driving conditions. When the pile reaches the bedrock rather abruptly, the pile may be damaged (i.e. split). This option is therefore considered impractical and not recommended, based on reliability.

5.2.2.2 Driven Steel Piles

Pile length may be extremely short (less than 3 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 rather thin overburden below the anticipated pile cap is not competent enough to provide sufficient lateral support (confinement) for pile driving.

As well, since part of 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

The use of augered and cast-in-place concrete pile foundations (drilled caissons) can be a feasible option, and piles socketed into the bedrock would be required to resist both axial and lateral loads. For this purpose, 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 structure while excavating the bedrock. This foundation option may be costly however due to the strong nature of the underlying bedrock (i.e. difficult to penetrate).

Geotechnical resistances of cast-in-place concrete piles increase with socket depth into the bedrock. Caisson extended at least 0.7 m into the sound dolostone/limestone bedrock can be designed for an axial geotechnical resistance of 7,000 kPa at ULS (factored) and bearing resistance at SLS need not to be considered. For example, a 0.76 m (30 inch) diameter caisson will have a based area of π x (0.76 m)2 / 4 = 0.45 m2. When designed with the value of 7,000 kPa, a resistance of 7,000 kPa x 0.45 m2 = 3,150 kN is obtained. These design values are applicable to commonly used caisson sizes in Ontario (i.e. between 0.76 and 1.5 m diameter), but large diameter caissons (i.e. greater than 1.2 m in diameter) may be difficult to install.

For caissons socketed more than 0.7 m below the rock surface, an additional adhesional resistance of 600 kPa can be utilized (owing to adhesion). For example, for a 0.76 m diameter caisson extending 1.5 m below the surface of bedrock, the additional resistance would be π x 0.76 m x (1.5 m – 0.7 m) x 600 kN/m2 = 1,146 kN bringing the total factored resistance at ULS to 3,150 kN + 1,146 kN = 4,296 kN/caisson.

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

If the rock around the caisson is shattered during the construction, this 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 native cohesive soils and bedrock at the site, temporary steel casing may required for this option. This temporary casing may need 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.

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 structure 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 most 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 1,000 kPa (between the fresh sedimentary 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 the extent of the socket length into the bedrock. Typically, factored resistances of the order 700 to 1,000 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 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 Recommended Foundation Option

With the present design, the most suitable foundation option from geotechnical point of view is, in our opinion, spread footing foundations supported on bedrock, immediately behind the existing bridge foundations, both in term of economics and reliability with the prevailing subsurface conditions.

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 and OPSD 3101.200. Free-draining backfill materials such as Granular ‘A’ or Granular ‘B’ Type I or II can be used. To maintain free draining characteristics in these granular materials, the maximum percentage passing the Sieve No. 200 (75 μm) should be limited to 5 %. Drain pipes, weep holes and the like should be incorporated to reduce hydrostatic pressure build-up.

Computation of earth pressures should be in accordance with CHBDC CAN/CSA, S6-06. For design purposes, the following parameters (unfactored) can be used.

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

Unit Weight, γ = 22 kN/m^3

Internal Friction Angle, ϕ = 35° (unfactored)

Coefficient of Lateral Earth Pressure		
Level Backfill	**Backfill Sloping at 3H:1V**	**Backfill Sloping at 2H:1V**
K_a = 0.27	K_a = 0.34	K_a = 0.40
K_o = 0.43	K_o = 0.56	K_o = 0.62

Compacted Granular 'B' Type I

Unit Weight, γ = 21 kN/m^3

Internal Friction Angle, ϕ = 32° (unfactored)

Coefficient of Lateral Earth Pressure		
Level Backfill	**Backfill Sloping at 3H:1V**	**Backfill Sloping at 2H:1V**
K_a = 0.31	K_a = 0.42	K_a = 0.54
K_o = 0.47	K_o = 0.66	K_o = 0.76

where Ka is the active earth pressure coefficient; Ko is the lateral earth pressure coefficient at rest.

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

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 Clause 6.9 of CHBDC CAN/CSA-S6-06.

For unrestrained wing walls (if any), the intermediate earth pressure coefficient Kb may be adopted. In the determination of degree of wall displacement or rotation to mobilize the fully active earth pressure state, Clause 6.9 of CHBDC CAN/CSA-S6-06 Commentary can be consulted.

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

5.4 Retaining Walls

We understand that new retaining walls are proposed on the northwest and southeast corners of the proposed bridge. The height of the proposed retaining walls is expected to be less than 4 m.

The choice of retaining walls is often based on multiple factors such as cost, aesthetics and environmental considerations. There is a wide variety choices available, but the more common are cantilevered concrete and gravity type retaining walls and retained soil system (RSS) walls. The use of a RSS wall often presents an attractive option. Some of these are discussed in the following paragraphs. It is anticipated that the position of the groundwater table at the site and dewatering requirements will play a role in the selection process.

5.4.1 Concrete Retaining Walls

Strip footing foundations may be used for the support of conventional cantilever or gravity type retaining walls. The footings of the proposed concrete retaining walls can be founded on the dolostone/limestone bedrock, as presented below.

Table 5.2 – Recommended Footing Foundations on Bedrock – Concrete Retaining Walls

Location / Borehole Number	Existing Ground Surface Elevation (m)	Recommended Highest Founding Depth / Elevation (m)	Bearing Stratum	Recommended Factored Bearing Resistance at ULS (kPa)	Recommended Bearing Resistance at SLS (kPa)
Northwest					
BH F1	239.8	5.1 / 234.7 *	bedrock *	6,000	not govern
BH F5	239.9	5.4 / 234.5	bedrock	6,000	not govern
Southeast					
BH F4	239.1	4.6 / 234.5 *	bedrock *	6,000	not govern
BH F6	239.0	4.5 / 234.5	bedrock	6,000	not govern

* inferred

All loose and highly fractured or weathered bedrock under the footprint of the footing should be removed and replaced with concrete. Mass concrete may be placed to raise the grade to the founding level, where necessary.

The quoted ULS value above may be impractical (i.e. too high) for use when designing the foundations of the proposed retaining walls of relatively low height, but is given here for the sake of completeness.

Under inclined loading conditions, the geotechnical resistance at ULS should be reduced in accordance with CHBDC.

The sliding resistance of the footings should be checked. The unfactored horizontal resistance against sliding between concrete and the clean and sufficiently roughened bedrock surface can be taken as 35o. The footings should also be checked against overturning. Following the construction of the footings, backfill should be placed to a sufficient height above the footing (e.g. 1.2 m) to prevent disturbance and/or frost penetration.

Computation of lateral earth pressures acting against concrete retaining walls should be in accordance with CHBDC. The properties of backfill should be referred to Section 5.3 of this report. It should be emphasized that for rigid frame structures supported on bedrock (i.e. unyielding), at rest lateral earth pressures are more appropriate.

5.4.2 Retained Soil System (RSS) Wall

In principle, a RSS consists of tying vertical facing units into a soil mass, with their tensile strips. This can present an attractive solution for this project. The system incorporates four elements:

- a soil backfill
- tensile reinforcing strips
- facing elements at boundaries
- mechanical connections between reinforcing elements

The soil backfill is generally a granular material with not more than 10 to 15% by weight passing the #200 mesh size sieve. It should not contain materials corrosive to reinforcing strips. Within the reinforced zone, the soil is able to stand at much steeper slopes than possible without reinforcing. In the past, up to 45º slopes were used, but over the past several decades, the system has evolved to provide steeper slopes (e.g. nearly vertical). In general, the reinforcing strips extend beyond the facing a horizontal distance of 70 – 80 % of the height of the wall retained. This is a patented method and the provider of the system normally guarantees its internal stability. Provided that the granular pad supporting the wall is placed on the bedrock, there should be no problem with external stability. The surface elevations of the bedrock contacted in the boreholes are given in Table 5.3.

Table 5.3 – Inferred Bedrock Surface at Borehole Locations

Location / Borehole Number	Existing Ground Surface Elevation (m)	Inferred Bedrock Elevation (m)
Northwest		
BH F1	239.8	234.9
BH F5	239.9	234.7
Southeast		
BH F4	239.1	234.7
BH F6	239.0	234.7

A MESA type wall (provided by Tensar) or a similar system by other suppliers (preferably on MTO's approved list) would also likely be suitable. Depending on the details, this type of wall would likely be placed on a reinforced granular pad.

These types of walls (i.e. RSS) would require less rigorous construction dewatering than if spread footing foundations are used to support a reinforced concrete retaining wall, and the facing elements can be made attractive from an aesthetics point of view, if desired. The follow information should be included in the contract drawings:

- the length and location
- height and space constraints
- elevation of top and bottom of RSS
- performance requirement (Medium Performance)
- Appearance requirement (Medium Appearance)

A disadvantage of this system is that future excavations in the vicinity of the wall can damage the reinforcing strips.

We will be pleased to discuss RSS type walls in more detail, if required.

5.4.3 Recommended Retaining Wall Option

In our opinion, reinforced concrete retaining wall is the preferred option for this project.

5.5 Embankment Fills

Based on the preliminary GA and profile drawings provided to us by Morrison Hershfield, the existing approach embankments have an approximate maximum height of 4.4 m over the original ground (o.g.) levels. The existing side slopes of the embankment fills appear to be at approximately 2H:1V. We understand that the proposed embankments will have a grade increase of up to 0.23 m, which occurs near the structure location. The existing embankments will not be widened. This and the fact that grade raise will be limited to less than 0.3 m are appropriate, as the existing embankment fills appear to be of inferior quality and could undergo excessive settlements if vertical grade is raised substantially and/or embankments are widened. Visual observation of the existing embankment slopes indicated no apparent signs of instability. The existing approach embankments immediately adjacent to the existing abutments are expected to be excavated during the construction of the footings for the proposed new bridge and reconstructed afterwards.

The following are recommended for site preparation:

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

Besides the above recommendations, site preparation should be in accordance with SP 206S03 and OPSS501.

Based on drawings provided to us by Morrison Hershfield, the maximum height of the proposed embankment fills will be of the order of 4.6 m. According to the results of the current site investigation, provided that all organic soils, unsuitable fill, weak and other unsuitable materials are removed under the footprint of the embankment before placing the fill, no instability problems are anticipated due to foundation conditions with the proposed height of embankments (i.e. up to approximately 4.6 m high). Conventional embankment slopes of 2H:1V or flatter would be stable, assuming that the subsurface 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.

All organic and otherwise unsuitable soils should be removed within an envelope given by an imaginary slope no steeper than 1H:1V from the toe of the proposed embankment. After stripping, the exposed subgrade should be inspected and approved. It should then be compacted, where feasible, from the surface using a suitable compactor.

The materials used for the construction of the embankment fills should consist of approved, acceptable earth fill (e.g. Select Subgrade Materials or Granular ‘B’ - OPSS1010). Fill used for the construction of the embankments should be in accordance with OPSS 212 and fill placement should meet or exceed the requirements of OPSS 206/SP 206S03 and OPSS 501. 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).

Excavation of the existing embankment fills immediately adjacent to the existing abutments will be required to facilitate the construction of the new abutment footings and abutment walls. As the new footings will probably be placed on the bedrock (see Section 5.2.3), excavation to bedrock or to near to it will be required in the general area of footing construction. In that event, for the construction of the embankment immediately beyond the foundation excavation areas, it is recommended that the overburden be removed to the surface of the bedrock or to near the surface of the bedrock (making sure only competent overburden is left in place). With this approach, there will be no foundation settlement (i.e. bedrock is unyielding) or nearly no settlement (i.e. very shallow competent overburden will be left in place), and this will minimize differential settlements that can be expected between the existing and the new portions of the newly rehabilitated road. However, there will be some settlements due to the settlement of the new embankment fill. For maximum 5.8 m high embankment (from the proposed road grade to bedrock), the settlement of the new embankment fills under their own weight can be expected to occur. If the embankment is constructed to MTO standards, this settlement should not exceed 30 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 or two months. It is however envisaged that in the confined space between the shoring of the existing embankment and the new abutment and wing walls, the use of suitably heavy compaction equipment will not be possible. For this reason, and to expedite the work, it is recommended that a granular fill be used, such as a Granular 'B' type fill with no more than 100 mm particle size. This will minimize the settlements, will make construction easier for the Contractor, as well as having the added benefit of reducing earth pressures on the abutment and wing walls.

Beyond this construction area, at pavement rehabilitation locations, it is expected that only the existing base and possibly the subbase will be removed, and new pavement fills will be constructed on top of the existing embankment fills. In addition, there will be no widening of the existing embankments. With this situation, the maximum anticipated grade raise is less than 0.3 m, and the settlement caused by this grade raise is expected to be less than 10 mm.

As these calculated settlements are not excessive, neither surcharging nor preloading is considered necessary for the approach embankments.

The effects of potential embankment loading on any existing underground services should be evaluated.

Proper erosion control measures should be implemented both during the construction and permanently. This can be achieved by prompt seed and cover (OPSS 804) or sodding (OPSS 803).

5.6 Seismic Design

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

5.7 Frost Protection

Design frost protection depth for the site is 1.2 m. A minimum 1.2 m thick permanent soil cover or equivalent thermal insulation is required for all of footings, including pile caps. 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) is used, only one half of the rock fill thickness should be accounted to be effective in providing frost protection.

5.8 Construction Considerations

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.

- OPSS 539 Construction Specification for Temporary Protection System
- OPSS 902 Construction Specification for Excavation and Backfilling – Structures

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

Existing Fill (very loose to compact)	Type 4 soil above water level;	Type 4 soil below water level
Clayey Silt (stiff to hard)	Type 3 soil above water level;	Type 4 soil below water level
Clayey Silt (soft to firm)	Type 4 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 strong to very strong. You may wish to ‘red flag’ this aspect. 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 between Elevations 238 and 236 m. A perched water condition may occur due to the presence of pervious granular fill over the less pervious clayey silt, as well, there are sandy silt interbeds in the clayey silt deposit. This may need dewatering. In addition, 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 creek (reported at

Elevation 235.5 on November 1, 2011 and at Elevation 236.8 m for high water level), more aggressive measures may be required.

The onsite excavated overburden, and especially the existing embankment fills are not considered to be suitable for re-use where engineered fill (e.g. embankment construction) is required. If necessary, they can be used to flatten the existing embankments to flatter slopes than 2H:1V. 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 be re-used provided that it is broken into sufficiently small sizes (i.e. typically less than 0.2 m nominal diameter).

The staged construction (i.e. close half of road for construction while open the other half for traffic) will be carried out for the replacement of the existing bridge. For the proposed new bridge, temporary support will be necessary to retain the existing embankment fills during construction of the new bridge foundations. The temporary shoring should be designed so that the lateral movement of any portion of the 'road protection system' will not exceed the established criterion for the structure performance level. In this case, the Performance Level is considered to be 2. The shoring system should be designed by a Professional Engineer experienced in this type of work.

Table 5.4 – Recommended Unfactored Parameters for Temporary Shoring Design

Soil Type	**Ka**	**Ko**	**Kp**	**γ (kN/m^3)**
Pavement Fill – Gravel and Sand	0.28	0.44	3.5	21.5
Fill - Sandy Silt mixed with clayey silt, very loose to compact	0.36	0.53	2.8	18.5
Clayey Silt, firm to stiff	0.36	0.53	2.8	19.0

In Ontario, temporary shoring systems typically consist of soldier piles & timber lagging or steel sheet piles. However, as mentioned before, the driven steel piles may 'walk' on the surface of the bedrock, and therefore the use of steel sheet piling and/or driven pile options are not considered practical. Soldier piles & timber lagging, using caisson type excavations extending into the bedrock (to support the steel soldier piles) is considered a feasible option for this project. Typically, the installation of soldier piles can be done by augering the existing fill, native soils and top portion of bedrock, installing H-piles and casting concrete between voids of bedrock and H-piles. However, this may be costly, as the caisson excavations will have to extended into the bedrock and this may present difficulties. This (and also if caisson type deep foundations are to be used) may need to be 'red flagged' to the Contractor as discussed earlier in this section of the report, to reduce the risk of the possible claims that rock is very difficult to excavate. The temporary shoring system should be designed by a Professional Engineer experienced in this type of work.

6 CLOSURE

The Limitations of Report, as quoted in Appendix G, are an integral part of this report.

For and on behalf of Coffey Geotechnics Inc.

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Appendices

Appendix E

Comparison of Foundation Alternatives

Summary of Foundation Alternatives

Foundations Type	Advantage/ Disadvantage	Risks/ Consequences	Relative Costs	Recommendations
Spread Footings on Bedrock	May require temporary shoring and dewatering effort	Dewatering may be required depending on the groundwater conditions of the time of construction	Low to medium	Probably the best option with the presently planned structure
Timber Piles	May not penetrate a sufficient depth below the pile caps, with the prevailing subsurface conditions, and may be damaged during installation	Not a good option with the prevailing subsurface conditions, not reliable	Low	Impractical for this project, not recommended
Driven Steel Piles	Will not penetrate a sufficient depth below the pile caps May have adverse impact upon the existing structures due to vibration	May 'walk' on the bedrock due to insufficient lateral support	Moderate	Impractical for this project, not recommended
Drilled Caissons	Less vibrations than driven piles May require dewatering	Installing caissons through the water bearing granular soils will present problems. Sound bedrock may cause problems during the construction of drilled caisson foundations.	Moderate to high	A feasible option, but more expensive than spread footing foundation option
Micropile Foundations	Minimizes vibrations, noise and dewatering	Cost effectiveness is a main concern	Expensive due to special equipment/ material and specialist contractor	Unlikely to be cost effective for this project

Appendix F

List of SPs, OPSSs and OPSDs

List of OPSSs and OPSDs referenced in the report

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 501	Construction Specification for Compacting
OPSS 539	Construction Specification for Temporary Protection System
OPSS 803	Construction Specification for Sodding
OPSS 804	Construction Specification for Seed and Cover
OPSS 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material
OPSD 3090.101	Ontario Provincial Standard Drawing - Foundation / Frost Penetration Depths for South Ontario
OPSD 3101.150	Ontario Provincial Standard Drawing - Walls / Abutment, Backfill / Minimum Granular Requirement
OPSD 3101.200	Ontario Provincial Standard Drawing - Walls / Abutment, Backfill / Rock

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.