coffey geotechnics
SPECIALISTS MANAGING THE EARTH

FOUNDATION INVESTIGATION AND DESIGN REPORTS PROPOSED REHABILITATION OF MCQUABY CREEK CULVERT AT HIGHWAY 534, TOWNSHIP OF GURD, ONTARIO, G.W.P. 5053-05-00, SITE NO. 44-265/C, GEOCRES NO. 31L-138

D.M. Wills Associates Limited

TRANETOB01238AB
December 17, 2009

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

coffey geotechnics
SPECIALISTS MANAGING THE EARTH

December 17, 2009

D.M. WILLS Associates Limited
452 Charlotte Street
Peterborough, Ontario
K9J 2 W3

Attention: Mr. Michael Lang, P. Eng.

Dear Sir:

RE: Foundation Investigation and Design Report, Proposed Rehabilitation of McQuaby Creek Culvert at Highway 534, Township of Gurd, Ontario, G.W.P. 5053-05-00, Site No. 44-265/C, Geocres No. 31L-138

Please find attached the results of our geotechnical investigation and report relating to the above noted site.

If you have any comments or enquiries please contact the undersigned.

For and on behalf of Coffey Geotechnics Inc.

Ramon Miranda, P. Eng.
Manager, Transportation Division

Attachment A: Attachments

coffey geotechnics
SPECIALISTS MANAGING THE EARTH

FOUNDATION INVESTIGATION REPORT PROPOSED REHABILITATION OF MCQUABY CREEK CULVERT AT HIGHWAY 534, TOWNSHIP OF GURD, ONTARIO G.W.P. 5053-05-00, SITE NO. 44-265/C GEOCRES NO. 31L-138

D.M. Wills Associates Limited

Project: TRANETOB01238AB
December 17, 2009

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

CONTENTS

FOUNDATION INVESTIGATION REPORT
PROPOSED REHABILITAION OF MCQUABY CREEK CULVERT
AT HIGHWAY 534, TOWNSHIP OF GURD, ONTARIO
G.W.P. 5053-05-00 SITE NO. 44-265/C, GEOCRES: 31L-138

1 INTRODUCTION

This project involves the rehabilitation of the McQuaby Creek Culvert under Highway 534 in the Geographic Township of Gurd, Ontario.

Coffey Geotechnics Inc. (Coffey) was retained by D.M. Wills Associate Limited (Wills) to conduct a foundation investigation for the proposed culvert rehabilitation. This report covers the foundation investigation conducted at McQuaby Creek Culvert, about 6.7 km west of Highway 654 under Highway 534.

The existing McQuaby Creek Culvert is an about 25.6 m long structural plate corrugated steel pipe arch (SPCSPA) culvert under Highway 534. This culvert has a skew angle of 82° relative to the centreline of Highway 534.

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

The findings of the investigation are presented in this report.

2 SITE DESCRIPTION AND PHYSIOGRAPHY

The McQuaby Creek Culvert is located at about 6.7 km west of Highway 654 under Highway 534 in the Geographic Township of Gurd. Topography of the project area is rolling in nature.

According to the Physiography of Southern Ontario by L.J. Chapman and D.F. Putnam, 1984, the site is located within the Physiographic Region known as the Algonquin Highlands. The Quaternary deposits found in this area are quite complex, having resulted from a variety of geological processes associated with glacial, glaciofluvial, and glaciolacustrine conditions. A large proportion of the area consists of bare bedrock with thin drift. Much of this region is underlain by Precambrian rocks of the Grenville structural Province.

According to Bedrock Geology of Ontario Map 2544, the bedrock underlying the site consists of Mesoproterozoic Precambrian rocks (i.e. approximately 900 million years old), primarily felsic igneous tonalite, granodionite, monzonite, granite, syenite and derived gneisses.

No significant signs of instability or erosion of the existing embankment at culvert location were identified at the time of our investigation.

3 FIELD AND LABORATORY WORK

The fieldwork for this project was performed on May 1, 4 and 5 and June 1, 2009, and consisted of drilling and sampling of four (4) boreholes (C4, C5, C6 and C7) for culvert rehabilitation and four (4) boreholes for roadway protection on Highway 534 (R1, R2, R3 and R4). Landcore Drilling Inc. of Chelmsford, Ontario carried out the drilling, testing and sampling work of all boreholes. Fieldwork was conducted under the direction and supervision of a technical staff from Coffey. Upon completion, each borehole was backfilled with a mixture of bentonite/cement, as per MTO procedures.

Boreholes C4, C5, C6 and C7 were put down adjacent to the existing McQuaby Creek Culvert to depths between 1.3 to 8.4 m below the existing ground surface. Boreholes R1, R2, R3 and R4 were drilled on the shoulder of Highway 534 to depths between 2.3 and 5.2 m below existing ground level. All boreholes were advanced by continuous flight hollow-stem auger or wash boring in the overburden and by rock coring where bedrock was cored. Borehole C7, which was put down using a tripod and wash boring drilling methods due to difficult access, was relocated twice due to shallow refusal probably on bedrock. The bedrock was proven by diamond drilling and coring in Boreholes C4 and C6, where NW size rock cores were obtained.

The borehole locations were established in the field by Coffey engineering staff, in relation to the existing features. The locations were then tied in and the geodetic elevations of the ground at the borehole locations were determined by the client's surveyors. This survey information was provided to us.

Sampling in the boreholes was effected at frequent intervals of depth by the Standard Penetration Test method (SPT), 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 O.D. 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 (cohesionless) soils (gravels, sands and coarse silts) or the consistency of cohesive soils (clays and clayey silts).

Standard Penetration Tests performed in Borehole C7 were conducted with a 31.5 kg hammer. The numbers of blows of the hammer obtained from these tests were then divided by two, giving approximate, equivalent N-values.

Water level observations in the open boreholes (or casing) were made during the drilling and at completion of each borehole.

A piezometer was installed at the bottom of the Borehole C4 to determine the groundwater levels over a prolonged period of time, without interference from surface water.

The soil and rock core samples were transported to our geotechnical laboratory in Toronto for further examination and classification. A laboratory testing programme, consisting of natural moisture content determinations and grain size analyses, was performed on selected representative samples. Two rock core samples from Boreholes C4 and C6 were forwarded to the laboratory of Golder Associates where the samples were tested for their unconfined compressive strength (UCS), bulk and dry densities. The results of the laboratory tests are presented on the appropriate Record of Borehole Sheets.

4 SUMMARIZED SUBSURFACE CONDITIONS

Boreholes C4, C5, C6 and C7 were advanced at the culvert site. Borehole C4 was put down near the toe of the highway embankment adjacent to the inlet (south) of the culvert at 13.6 m left from the centreline of Highway 534 from the original ground (o.g.) (El. 307.8 m) level while Borehole C7 was put down on the opposite side of the highway embankment near the culvert outlet (north) at 16.2 m right of the centreline of Highway 534 from o.g. level (El. 307.6 m). Boreholes C5 and C6 were advanced at the shoulder, at the top of the highway embankment. Boreholes C5 and C6 were located at 3.0 m left and 3.2 m right from the centreline of Highway 534 at El. 310.8 and 311.1 m, respectively.

Boreholes R1, R2, R3 and R4 were advanced from the shoulder of Highway 534. Boreholes R1 and R2 were put down at 4.5 m right and 5.0 m left of the centreline at Station 12+494, respectively. The ground surface elevations at Boreholes R1 and R2 are 311.0 and 310.4 m, respectively. Boreholes R3 and R4 were put down at Station 12+544, 5.0 m right and 5.0 m left of the centreline at Elevations 311.3 and 310.9 m, respectively.

At the site, Boreholes C5, C6, R1, R2, R3, R4, drilled from the top of the highway embankment, encountered embankment fill to depths of 1.3 to 4.3 m (El. 311.0 to 306.5 m). Underlying the fill, Boreholes R3 and R4 contacted a 0.9 to 2.3 m thick silt layer. Below this silt layer in Boreholes R3 and R4 and below the embankment fill or the o.g. level in most of the remaining boreholes (except for R2), the overburden consists of sand, extending to depths of between 1.3 and 5.4 m (El. 308.1 – 304.7 m). In Borehole R2, refusal to augering was contacted at 2.3 m (El. 308.1 m), immediately below the embankment fill. In the remaining boreholes, refusal to further penetration was contacted at depths of 1.3 to 5.4 m below the ground surface or between El. 308.1 and 304.7 m, probably on the surface of bedrock. In Boreholes C4 and C6, the borehole was further advanced after refusal by rock coring and the presence of bedrock was ascertained. The bedrock was found to consist of a pinkish grey gneiss.

Subsurface conditions at the site are discussed in the following sections. Details of the stratigraphy encountered in the boreholes are presented on the Records of Borehole Sheets and in the soil strata drawings in Appendix A. Photographs of the culvert are included in Appendix C. The following paragraphs are only meant to compliment these data.

4.1 Topsoil

A layer of topsoil ranging from 0.1 to 0.2 m in thickness was contacted in Boreholes C4 and C7 at ground surface.

4.2 Fill

Boreholes C5, C6, R1, R2, R3 and R4 were drilled from the shoulder of Highway 534 and encountered embankment fill to depths ranging from 1.3 m (Borehole R4, El. 309.6 m) to 4.3 m (Borehole C5, El. 306.5 m).

In Boreholes C5 and C6, a 30 to 35 mm thick asphaltic concrete was contacted, underlain by a 0.2 m thick granular base course. In Boreholes R1, R2, R3 and R4, the granular base course was contacted at the ground surface level and its thickness ranged from 0.1 to 0.2 m. Underlying the granular base course, the embankment fill consisted of sand with traces to some gravel. Traces to some silt was also found in the fill material in Boreholes C5, C6 and R2. The presence of asphalt pieces was noted at about 1 m depth in Borehole R3. Traces to some organic matter was also detected at the bottom of the fill layer in Boreholes C5, R1 and R4. Rock pieces were found at the bottom of the fill layer at Borehole R2 where refusal was encountered and the borehole was terminated at 2.3 m depth, probably on bedrock.

The embankment fill is a granular (non-cohesive) soil. The grain-size distribution of four samples recovered from the fill is presented in Figure B-1, in Appendix B which indicate following grain-size distribution:

Gravel: 5 – 9 %

Sand: 73 – 90 %

Silt and Clay: 5 – 20%

This fill is a granular (i.e. non-cohesive) soil. Standard Penetration Tests conducted in the fill yielded N-values between 3 to 64 blows/0.3 m. These results indicate that the relative density of the embankment fill can be described as very loose to very dense, but typically compact in the upper layer to a depth of 2.1 m except for Borehole R2. Embankment fill contacted at Borehole R2 and the remaining lower fill layer can be described as very loose. These results are an indication that the embankment fill did not receive a systematic compaction when it was first placed.

4.3 Silt

A 0.9 to 2.3 m thick layer of silt was found underlying the embankment fill in Boreholes R3 and R4. The silt layer contains clay pockets and traces of organics. Sand seams were also detected within the silt layer in Borehole R3. The deposit extends to depths of 3.7 and 2.2 m (El. 307.6 and 308.7 m) at Boreholes R3 and R4, respectively.

The grain-size distribution of a sample containing sand seams from Borehole R3 gave the following distribution, as shown in Figure B-2, in Appendix B.

Gravel: 0 %

Sand: 42 %

Silt: 53 %

Clay: 5 %

Also shown in the same figure (i.e. Figure B-2, Appendix B) is the grain size analysis on a sample containing clay pockets retrieved from the silt layer in Borehole R4 and the grain-size distribution is as follows:

Gravel: 0%

Sand: 11%

Silt: 67 %

Clay: 22 %

Standard Penetration Tests performed in this mostly non-cohesive unit yielded N-values between 12 and 20 blows/0.3 m. From these results, the compactness condition of the deposit can be described as compact.

In addition, in Borehole C4, a 1.1 m thick peat and silt layer was contacted at a depth of 0.7 m (El. 307.1 m) and the deposit extended to 1.8 m (El. 306.0 m). Due to the interbedded organic (peat) layers, this material is considered very weak and highly compressible. It is a fine grained granular (silt layers) to cohesive (peat layers) material and based on an N-value of 1 blow/0.3 m, it is considered very loose to very soft.

4.4 Sand

Underlying the topsoil in Boreholes C4 and C7, embankment fill in Boreholes C5, C6, and R1, and the silt layer in Boreholes R3 and R4, is a sand deposit with a thickness ranging from about 0.6 to 2.4 m. The sand contains some silt with traces of gravel and clay. This deposit extends to the surface of the proven bedrock (Boreholes C4 and C6) or the inferred bedrock in all boreholes at depths between 1.3 and 5.4 m or at elevations ranging from 308.1 to 304.2 m.

Grain-size analysis conducted on three samples retrieved from this deposit gave the following grain size distribution (see Figure B-3 in Appendix B):

Gravel: 5 – 12 %

Sand: 65 – 73 %

Silt: 15 – 26 %

Clay: 2 – 4 %

N-values obtained from Standard Penetration Tests performed in this granular (i.e. non-cohesive) deposit are between 3 and in excess of 50 blows/0.3 m. This indicates that the deposit is in a very loose to very dense in condition, typically dense near the bedrock surface.

4.5 Bedrock

A pinkish grey gneiss bedrock was encountered in Boreholes C4 and C6 and was proven by NQ coring.

Table 4.5.1 - Bedrock elevation and condition

Borehole	Ground Elevation (m)	Bedrock Depth/Elevation (m)	T.C.R (%) *	R.Q.D. (%) **
C4	307.8	3.7 / 304.2	92 – 100	86 – 87
C5	310.8	4.9 / 305.9***		
C6	311.1	5.4 / 305.8	100	100
C7	307.6	1.3 / 306.3***		
R1	311.0	3.7 / 307.3***		
R2	310.4	2.3 / 308.1***		
R3	311.3	5.2 / 306.1***		
R4	310.9	2.8 / 308.1***		

*T.C.R. = Total Core Recovery
**R.Q.D = Rock Quality Designation
***Inferred bedrock depth/elevation

The Boreholes were advanced into the bedrock for a vertical distance of about 3.0 m by NQ coring. The percentage of recovery was 92 to 100% while the RQD values vary from 86% to 100%. These results indicate rock quality from good to excellent.

Two unconfined compression tests were performed on selected intact rock core samples from Borehole C4 and C6 and the tests yielded unconfined compression strengths (UCS) of 130.5 MPa and 128.2 MPa, as shown on Table 4.5.2. The laboratory testing results for rock core samples are attached in Appendix B.

Table 4.5.2 – Unconfined Compression Test Data

Borehole & Sample No.	Approximate Depth (m)	Approximate Elevation (m)	Bulk Density (kN/m3)	Dry Density (kN/m3)	Unconfined Compressive Strength (MPa)
C4-RC6	3.7	304.1	26.16	26.14	130.5
C6-RC8	5.5	305.6	27.72	26.71	128.2

At the borehole locations the surface of the bedrock was contacted at Elevations 305.8 m (Borehole C6) and 304.2 m (Borehole C4). The inferred bedrock surface in the remaining boreholes ranges from 1.3 to 5.4 m from the ground surface (El. 306.3 m and 305.8 m). From these results, the surface of the bedrock appears to dip mildly towards the creek bed.

4.6 Groundwater Conditions

Groundwater levels were observed in open boreholes while drilling and upon completion of each borehole. In boreholes where NQ coring and/or wash boring were used (i.e. water introduced into the boreholes) the on-completion water levels may not be reliable. The observations made in the boreholes are summarized in Table 4.6.1 and presented on the Record of Borehole Sheets in Appendix A.

Table 4.6.1 - Groundwater conditions

Borehole	Ground Elevation (m)	Depth / Elevation of the Tip of Piezometer (m)	Date	Water Level Depth / Elevation (m)	Sample wet during drilling Depth / Elevation (m)
C4*	307.8	6.7 / 301.1	May 4, 2009 May 6, 2009	0.5 / 307.3 0.3 / 307.5	0.7 / 307.1
C5	310.8		May 5, 2009 (completion)	3.4 / 307.4	3.3 / 307.5
C6*	311.1		May 5, 2009 (completion)	3.1 / 308.0	4.8 / 306.3
C7**	307.6		June 1, 2009 (completion)	0.3 / 307.3	0.6 / 307.1
R1	311.0		May 4, 2009 (completion)	dry	2.5 / 308.5
R2	310.4		May 4, 2009 (completion)	dry	2.0 / 308.4
R3	311.3		May 4, 2009 (completion)	3.1 / 308.2	3.1 / 308.2

Borehole	Ground Elevation (m)	Depth / Elevation of the Tip of Piezometer (m)	Date	Water Level Depth / Elevation (m)	Sample wet during drilling Depth / Elevation (m)
R4	310.9		May 4, 2009 (completion)	2.3 / 308.6	2.3 / 308.6

*water used for coring
*water used for wash boring

Based on these results, the ground water level at the time of our investigation was between El. 308.5 and 308.0 m at the location of boreholes R1 through R4 and generally between 307.5 and 307.0 m at Boreholes C4 through C7 (i.e. closer to the creek location).

It should be noted that the observed groundwater levels represent the condition at the time of our investigation and they are subject to fluctuations, both seasonally and in response to major weather events.

For and on behalf of Coffey Geotechnics Inc.

Winnie Chan, E.I.T.

LICENSED PROFESSIONAL ENGINEER
RAMON MIRANDA
PROVINCE OF ONTARIO

Ramon Miranda, P.Eng.
Manager, Transportation Division

REGISTERED PROFESSIONAL ENGINEER
Z. S. OZDEN
PROVINCE OF ONTARIO

Zuhtu Ozden, P.Eng.
Senior Principal

Drawings

METRIC

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

NOTES:

FOR DETAILED SUBSURFACE CONDITIONS REFER TO RECORD OF BOREHOLE SHEETS.

CONT No.
GWP: 5053-05-00

SHEET

HIGHWAY 534, MCQUABY CREEK
CULVERT REHABILITATION
BOREHOLE LOCATION PLAN

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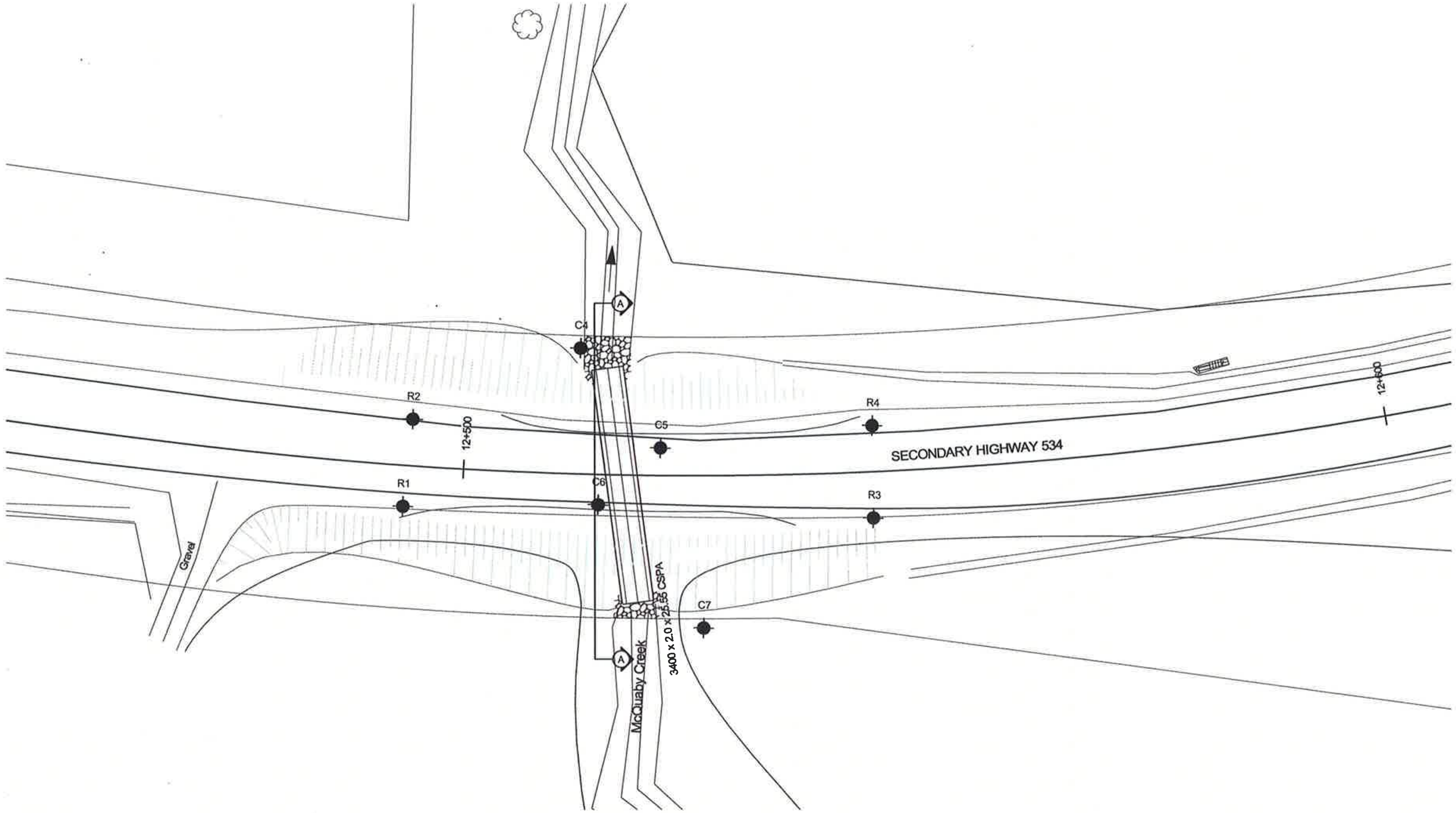
SPECIALISTS MANAGING THE EARTH



PLAN
SCALE
12.5m 0 12.5 25m

KEY PLAN
N.T.S.

LEGEND

Borehole

No.	ELEVATION	STATION	OFFSET
C4	307.8	12+512	13.6m Lt C/L
C5	310.8	12+521	3.0m Lt C/L
C6	311.1	12+517	3.2m Rt C/L
C7	307.6	12+525	16.2m Rt C/L
R1	311.0	12+494	4.5m Rt C/L
R2	310.4	12+494	5.0m Lt C/L
R3	311.3	12+544	5.0m Rt C/L
R4	310.9	12+544	5.0m Lt C/L

-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.

LICENSED PROFESSIONAL ENGINEER
RAMON MIRANDA
Dec 16/09
PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER
Z. S. OZDEN
Dec 16/09
PROVINCE OF ONTARIO

REVISIONS		
DATE	BY	DESCRIPTION

Geocres No 31L-138

TRANETOB01238AB			DIST 54
SUBM'D	CHECKED	DATE Dec. 17, 2009	SITE 44-265/C
DRAWN PHK	CHECKED RM	APPROVED ZO	DWG 1

CONT No.
GWP: 5053-05-00

HIGHWAY 534, MCQUABY CREEK
CULVERT REHABILITATION
SOIL STRATA (SECTION)

SHEET

METRIC

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

NOTES:

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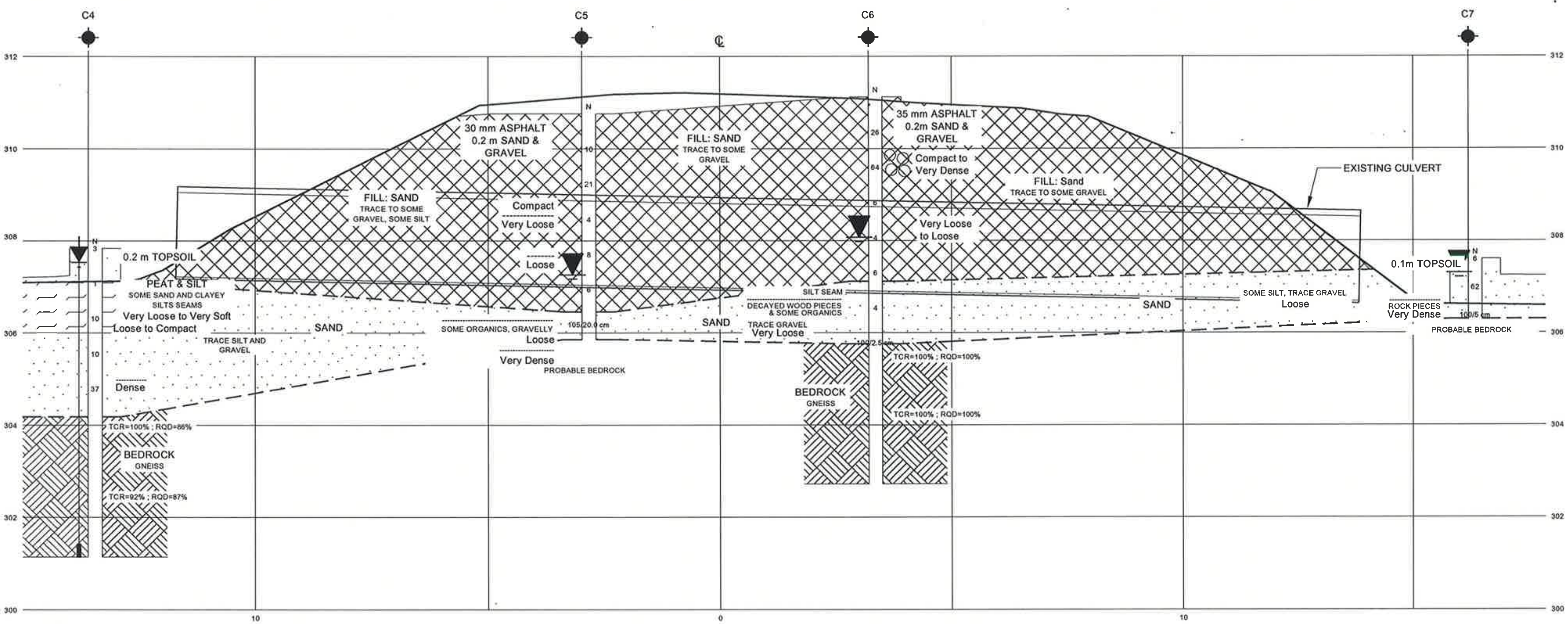
SPECIALISTS MANAGING THE EARTH



SECTION A-A
SCALE
2.5m 0 2.5 5m

KEY PLAN
N.T.S.

LEGEND

- Borehole
- 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	STATION	OFFSET
C4	307.8	12+512	13.6m Lt C/L
C5	310.8	12+521	3.0m Lt C/L
C6	311.1	12+517	3.2m Rt C/L
C7	307.6	12+525	16.2m Rt C/L

-NOTE-
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RAMON MIRANDA
Dec 16/09
PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER
Z. S. OZDEN
Dec 16/09
PROVINCE OF ONTARIO

REVISIONS	DATE	BY	DESCRIPTION

Geocres No 31L-138

TRANETOB01238AB			DIST 54
SUBM'D	CHECKED	DATE Dec. 17, 2009	SITE 44-265/C
DRAWN PHK	CHECKED RM	APPROVED ZO	DWG 2

METRIC

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

NOTES:

FOR DETAILED SUBSURFACE CONDITIONS REFER TO RECORD OF BOREHOLE SHEETS.

CONT No.

GWP: 5053-05-00

SHEET

HIGHWAY 534, MCQUABY CREEK
CULVERT REHABILITATION
SOIL STRATA (PROFILES)

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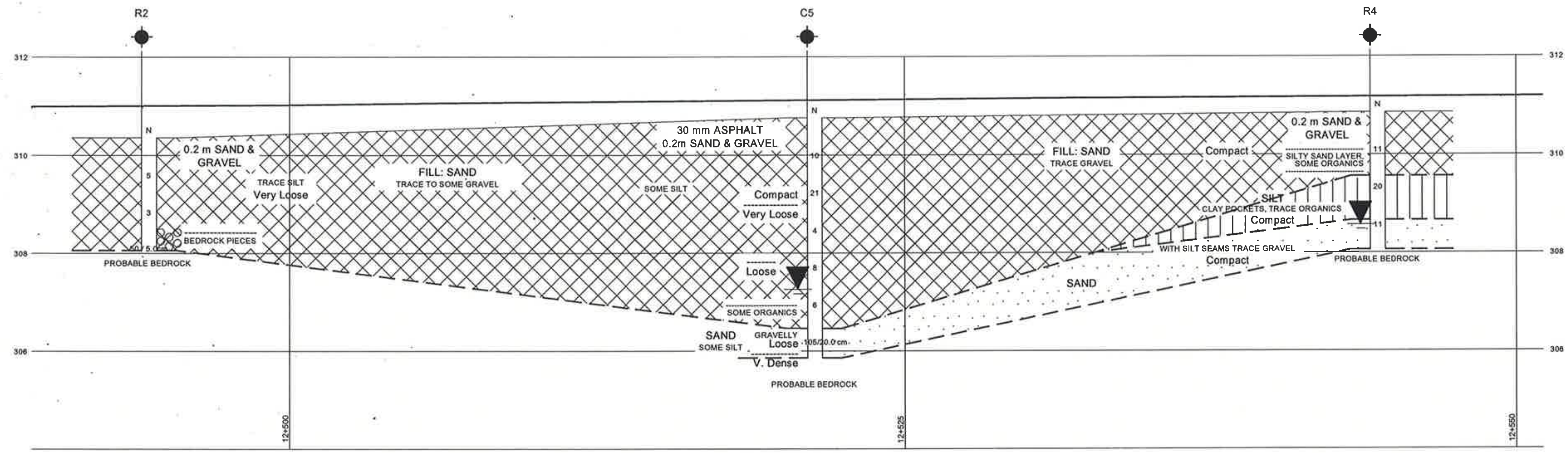
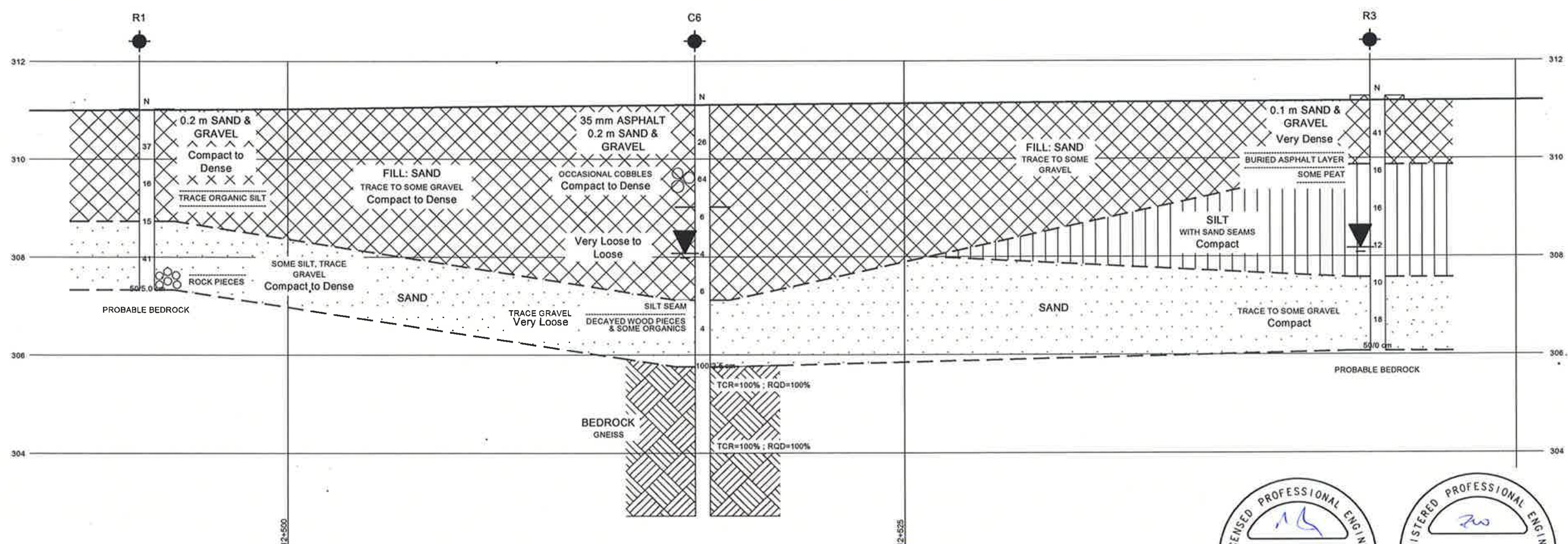


LT PROFILE



RT PROFILE

HORIZONTAL SCALE

5m 0 5 10m

KEY PLAN
N.T.S.

LEGEND

Borehole

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	STATION	OFFSET
C5	310.8	12+521	3.0m Lt C/L
C6	311.1	12+517	3.2m Rt C/L
R1	311.0	12+494	4.5m Rt C/L
R2	310.4	12+494	5.0m Lt C/L
R3	311.3	12+544	5.0m Rt C/L
R4	310.9	12+544	5.0m Lt C/L

-NOTE-

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

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LICENSED PROFESSIONAL ENGINEER
RAMON MIRANDA
Dec16/09
PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER
Z. S. OZDEN
Dec16/09
PROVINCE OF ONTARIO

REVISIONS	DATE	BY	DESCRIPTION

Geocres No 31L-136

TRANETOB01238AB				DIST	54
SUBM'D		CHECKED	DATE Dec. 17, 2009	SITE	44-265/C
DRAWN PHK		CHECKED RM	APPROVED ZO	DWG	3

Appendix A

Borehole Logs

RECORD OF BOREHOLE No C4

1 OF 1 **METRIC**

GWP 5053-05-00 LOCATION (McQuaby Creek Culvert) Sta: 12+512 ; 13.6 m Lt C/L Hwy 534 ORIGINATED BY ZI

DIST 54 HWY HWY 534 BOREHOLE TYPE Hollow Stem Auger, Wash Boring & NQ Coring COMPILED BY WC

DATUM Geodetic DATE 5/1/2009 & 5/4/2009 CHECKED BY ZO

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
307.8	GROUND SURFACE					
0.0	0.2 m **TOPSOIL** **SAND** tr. silt and gravel grey, wet	1	SS	3		
307.1 / 0.7	**PEAT & SILT** some sand and clayey silt seams dark brown, v. loose to v. soft, wet	2	SS	1	307	spoon wet @ 0.7 m
306.0 / 1.8	**SAND** some silt and gravel greyish brown, wet; loose to compact	3	SS	10	306	12 71 15 2 auger grinding from 1.8 to 3.7 m
		4	SS	10	305	
	dense	5	SS	37		
304.2 / 3.7	**BEDROCK** pinkish grey gneiss	6	RC	TCR=100% ; RQD=86%	304, 303	wash boring from 3.6 m NQ coring from 3.7 m
		7	RC	TCR=92% ; RQD=87%	302	
301.1 / 6.7	End of borehole Water level @ 0.5 m (not stabilized)* upon completion Piezometer installed to 6.7 m Water level records in Piezometer: 0.5 m May 04, 2009 0.3 m May 06, 2009					

Plot columns: STRAT PLOT; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH (kPa) O UNCONFINED + FIELD VANE ● POCKET PENETR. X 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, X³: Numbers refer to Sensitivity

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

Ministry of Transportation Ontario

Foundation Design

TRANETOB01238AB

RECORD OF BOREHOLE No C5

METRIC

GWP 5053-05-00	LOCATION (McQuaby Creek Culvert) Sta: 12+521 ; 3.0 m Lt C/L (EP) Hwy 534		ORIGINATED BY ZI
DIST 54 HWY HWY 534	BOREHOLE TYPE Hollow Stem Auger		COMPILED BY WC
DATUM Geodetic	DATE 5/5/2009		CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%) (approx.)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
310.8	GROUND SURFACE							
0.0	30 mm **ASPHALT** 0.2 m **SAND & GRAVEL** **FILL:** Sand some silt, tr. to some gravel brown, wet					310		
		1	SS	10			~13	9 81 (10) auger grinding
	compact	2	SS	21		309	~16	auger grinding from 1.7 to 2.1 m
	v. loose	3	SS	4		308	~10	7 73 16 4
	loose	4	SS	8	▼*	307	~18	spoon wet @ 3.3 m
	some organics	5	SS	6			~32	
306.5 4.3	**SAND** some silt, greyish brown, wet gravelly, loose						~19	auger grinding
305.9	v. dense	6	SS	105/20.0 cm		306	~18	spoon bouncing
4.9	End of Borehole Auger refusal @ 4.9 m on bedrock Water level @ 3.4 m upon completion (not stabilized)* Hole caved-in @ 4.1 m upon completion							

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/m^3

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

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

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TRANETOB01238AB

RECORD OF BOREHOLE No C6

METRIC

GWP 5053-05-00 LOCATION (McQuaby Creek Culvert) Sta: 12+517 ; 3.2 m Rt C/L (EP) Hwy 534 ORIGINATED BY ZI

DIST 54 HWY HWY 534 BOREHOLE TYPE Hollow Stem Auger& NQ coring COMPILED BY WC

DATUM Geodetic DATE 5/5/2009 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
311.1	GROUND SURFACE					311		
0.0	35 mm **ASPHALT** 0.2 m **SAND & GRAVEL** **FILL:** Sand tr. to some gravel, occ. cobbles brown/ dark brown, compact to v. dense damp to moist	1	SS	26		310		
		2	SS	64				auger grinding @ 1.2 m; auger grinding from 1.7 to 2.1 m
309.0 / 2.1	**FILL:** Sand tr. gravel, brown v. loose to loose, moist	3	SS	6		309		5 90 (5)
		4	SS	4	▼*	308		
307.1 / 4.0	silt seam, loose; decayed wood pieces & some organics **SAND** tr. gravel, grey, v. loose, wet	5	SS	6		307		
		6	SS	4		306		spoon wet @ 4.8 m
305.8 / 5.4		~~7~~	~~SS~~	~~100/2.5 cm~~				~~auger grinding~~
	BEDROCK pinkish grey gneiss	8	RC	TCR=100% ; RQD=100%		305		auger refusal and advanced by wash boring from 5.3 m; NQ coring from 5.4 m
		9	RC	TCR=100% ; RQD=100%		304, 303		
302.7 / 8.4	End of Borehole Water level @ 3.1 m (not stabilized)* upon completion Hole Caved-in @ 3.7 m upon completion							

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH (kPa) O UNCONFINED + FIELD VANE ● POCKET PENETR. X 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 20 15 5 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No C7

METRIC

GWP 5053-05-00 LOCATION (McQuaby Creek Culvert) Sta: 12+525 ; 16.2 m Rt C/L (Ditch) Hwy 534 ORIGINATED BY ZI

DIST 54 HWY HWY 534 BOREHOLE TYPE Wash Boring COMPILED BY WC

DATUM Geodetic DATE 6/1/2009 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. X 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
307.6	GROUND SURFACE										
0.0	0.1 m **TOPSOIL** **SAND** some silt, tr. gravel, brown, loose, wet		1	SS	6	▼*					
							307				
	rock pieces, v. dense		2	SS	62				o		5 73 20 2
306.3			3	SS	100/5 cm						spoon bouncing
1.3	End of Borehole										

End of Borehole
Refusal @ 1.3 m on possible bedrock.

Water level @ 0.3 m (not stabilized)* upon completion
Hole caved-in @ 0.6 m upon completion.

Modified N values are obtained by dividing the number of blows by 31.75 kg hammer by two.

Borehole moved twice adjacent to its original location due to shallow refusal.

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

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RECORD OF BOREHOLE No R1

METRIC

GWP 5053-05-00 LOCATION (McQuaby Creek Culvert) Sta: 12+494 ; 4.5 m Rt C/L (Sh) Hwy 534 ORIGINATED BY ZI

DIST 54 HWY HWY 534 BOREHOLE TYPE Hollow Stem Auger COMPILED BY WC

DATUM Geodetic DATE 5/4/2009 CHECKED BY ZO

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
311.0	GROUND SURFACE				311	
0.0	0.2 m **SAND & GRAVEL** **FILL:** Sand tr. to some gravel, brown compact to dense, moist	1	SS	37	310	cobble/stone @ spoon tip
	tr. organic silt	2	SS	16	309	stone @ spoon tip
308.7 2.3	**SAND** some silt, tr. gravel greyish brown, compact to dense, wet	3	SS	15		5 65 26 4 spoon wet below @ 2.5 m
		4	SS	41	308	auger grinding
	rock pieces	~~5~~	~~SS~~	~~50/0.0 cm~~		~~spoon bouncing~~
307.3 3.7	End of Borehole Auger refusal @ 3.7 m probably on bedrock Water level dry (not stabilized) and open upon completion					

Water content (%) plotted: Sample 1 ≈ 10; Sample 2 ≈ 15; Sample 3 ≈ 15; Sample 4 ≈ 12.

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

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

Ministry of Transportation Ontario

Foundation Design

TRANETOB01238AB

RECORD OF BOREHOLE No R2

METRIC

GWP	5053-05-00	LOCATION	(McQuaby Creek Culvert) Sta: 12+494 ; 5.0 m Lt C/L (Sh) Hwy 534	ORIGINATED BY	ZI
DIST	54 HWY HWY 534	BOREHOLE TYPE	Hollow Stem Auger	COMPILED BY	WC
DATUM	Geodetic	DATE	5/4/2009	CHECKED BY	ZO

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
310.4	GROUND SURFACE									
0.0	0.2 m **SAND & GRAVEL** **FILL:** Sand tr. gravel, tr. silt, brown v. loose						310			
	moist		1	SS	5			o		5 89 (6)
	wet						309			
			2	SS	3			o		spoon wet @ 2.0 m
	rock pieces		3	SS	50/5.0 cm			o		auger grinding spoon wet spoon bouncing
308.1 2.3	End of Borehole Auger refusal @ 2.3 m on probably on bedrock Water level dry (not stabilized) upon completion Hole caved-in @ 2.0 m upon completion									

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 10 20 30

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

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

RECORD OF BOREHOLE No R3

METRIC

GWP	5053-05-00	LOCATION	(McQueby Creek Culvert) Sta: 12+544 ; 5.0 m Rt C/L (Sh) Hwy 534	ORIGINATED BY	ZI
DIST	54 HWY HWY 534	BOREHOLE TYPE	Hollow Stem Auger	COMPILED BY	WC
DATUM	Geodetic	DATE	5/4/2009	CHECKED BY	ZO

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
311.3	GROUND SURFACE							
0.0	0.1 m **SAND & GRAVEL** **FILL:** Sand tr. gravel, brown damp to moist, v. dense						311	
	buried asphalt layer		1	SS	41			
309.9 / 1.4	some peat		2	SS	16		310	
	SILT with sand seams, greyish brown compact		3	SS	16		309	
	moist							
	dilatant, wet		4	SS	12	▼*	308	spoon wet @ 3.1 m; 0 42 53 5
307.6 / 3.7	**SAND** tr. to some gravel, greyish brown compact, wet		5	SS	10		307	
			6	SS	18			
306.1 / 5.2			7	SS	50/0 cm			spoon bouncing

End of Borehole
Auger refusal @ 5.2 m on bedrock
Water level @ 3.1 m (not stabilized)* upon completion
Hole caved-in @ 3.4 m upon completion

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH (kPa): O UNCONFINED + FIELD VANE ● POCKET PENETR. X 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, X ³: Numbers refer to Sensitivity — 20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

TRANETOB01238AB

RECORD OF BOREHOLE No R4

METRIC

GWP 5053-05-00 LOCATION (McQuaby Creek Culvert) Sta: 12+544 ; 5.0 m Lt C/L (Sh) Hwy 534 ORIGINATED BY ZI

DIST 54 HWY HWY 534 BOREHOLE TYPE Hollow Stem Auger COMPILED BY WC

DATUM Geodetic DATE 5/4/2009 CHECKED BY ZO

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
310.9	GROUND SURFACE							
0.0	0.2 m **SAND & GRAVEL** **FILL:** Sand tr. gravel, brown compact, moist; silty sand layer, some organics		1	SS	11		310	
309.6 / 1.3	**SILT** clay pockets, tr. organics greyish brown, compact, moist		2	SS	20		309	0 11 67 22
308.7 / 2.2	**SAND** with silt seams (dilatant), tr. gravel brown, compact, wet		3	SS	11	▽*		Spoon wet @ 2.3 m
308.1 / 2.8	End of Borehole Auger refusal @ 2.8 m on bedrcok Water level @ 2.3 m (not stabilized)* upon completion Hole caved-in @ 2.4 m upon completion							Auger grinding

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

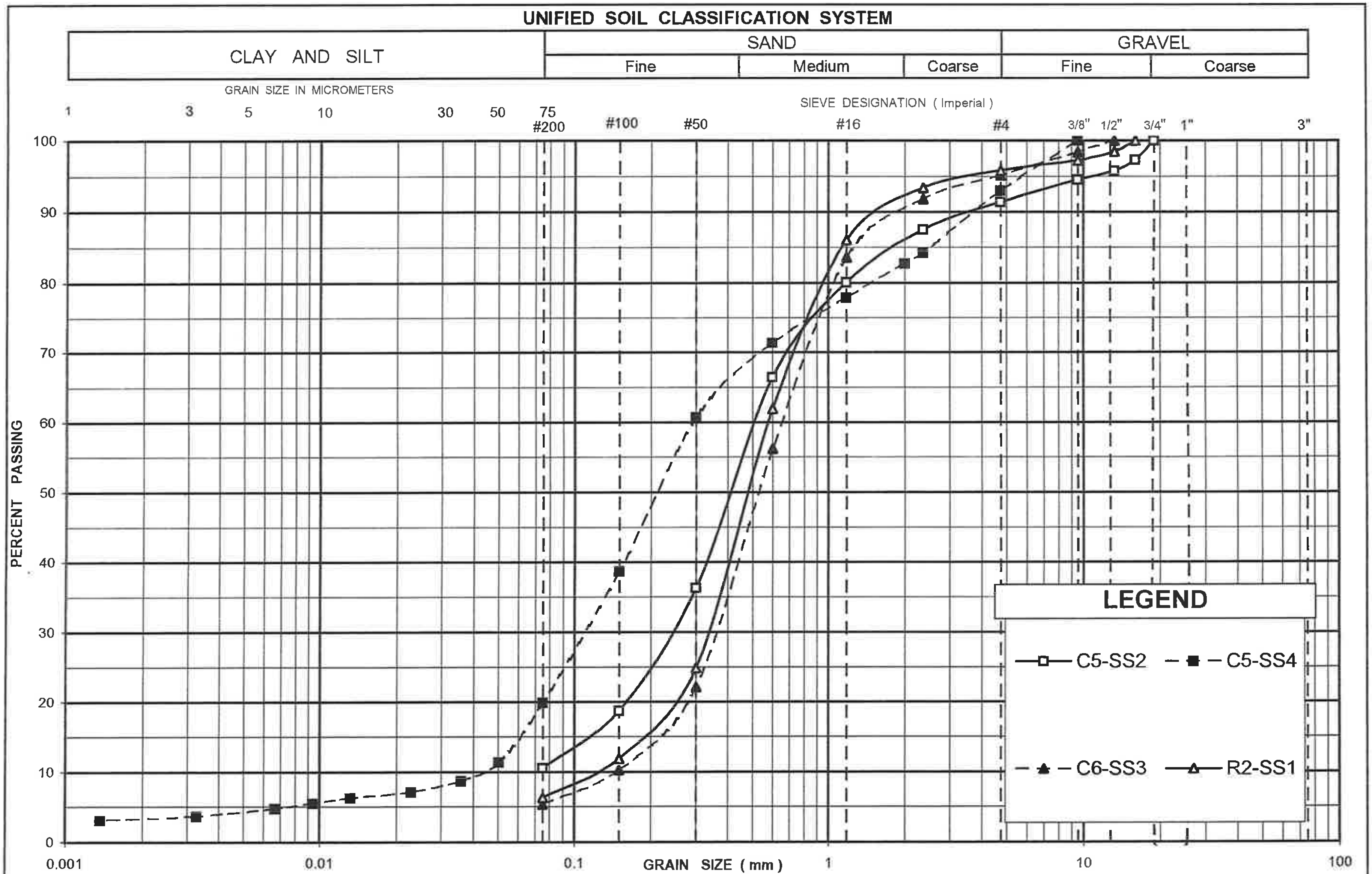
PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

+³, ×³: Numbers refer to Sensitivity 15-⊕-5 (20 / 10) (%) STRAIN AT FAILURE

Appendix B

Test Results

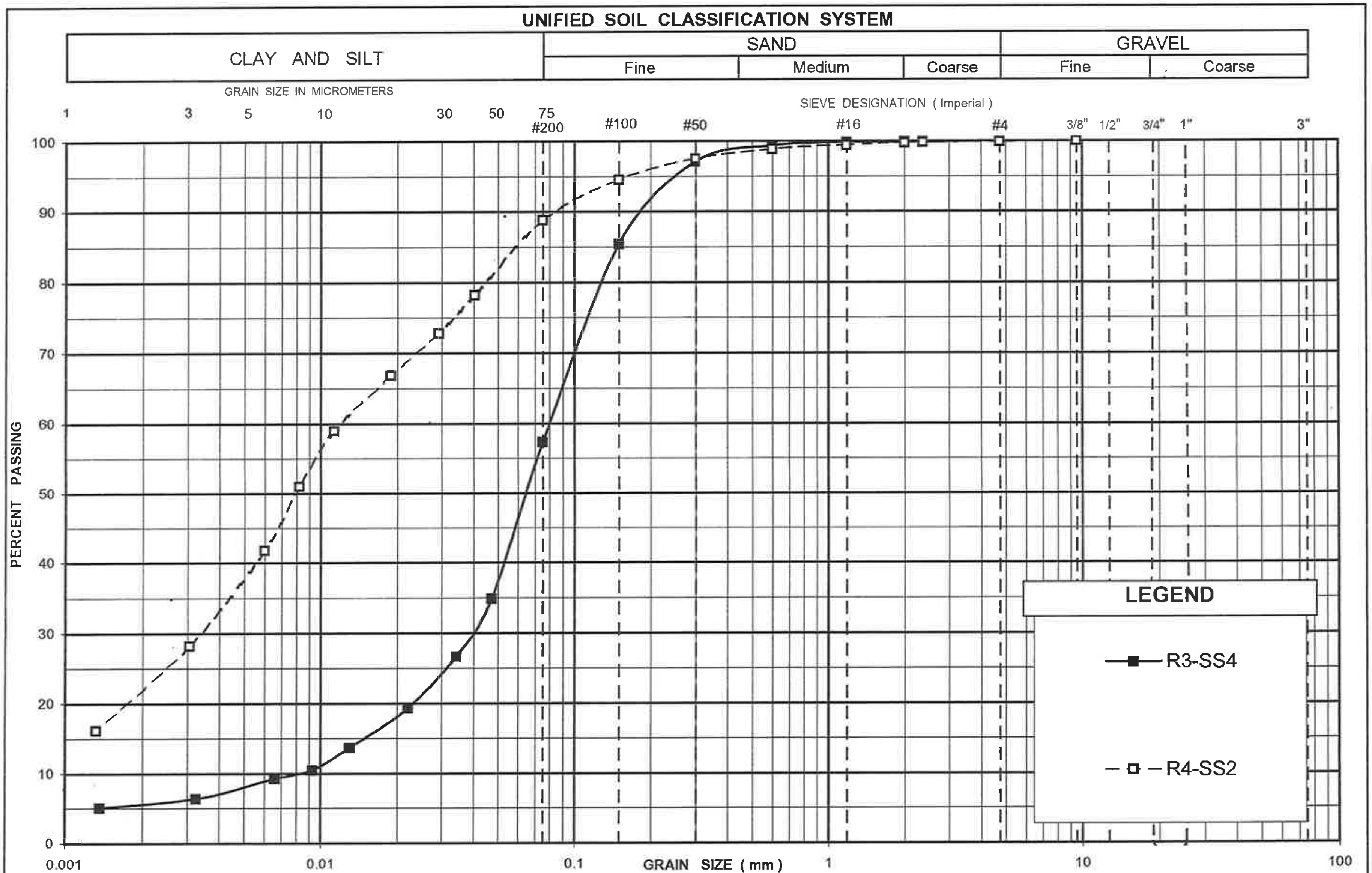


coffey geotechnics
SPECIALISTS MANAGING THE EARTH

GRAIN SIZE DISTRIBUTION

EMBANKMENT FILL: Sand, trace to some gravel and silt, trace clay

FIGURE NO.:	B-1
PROJECT NO:	TRANETOB01238AB
DATE:	AUGUST, 2009

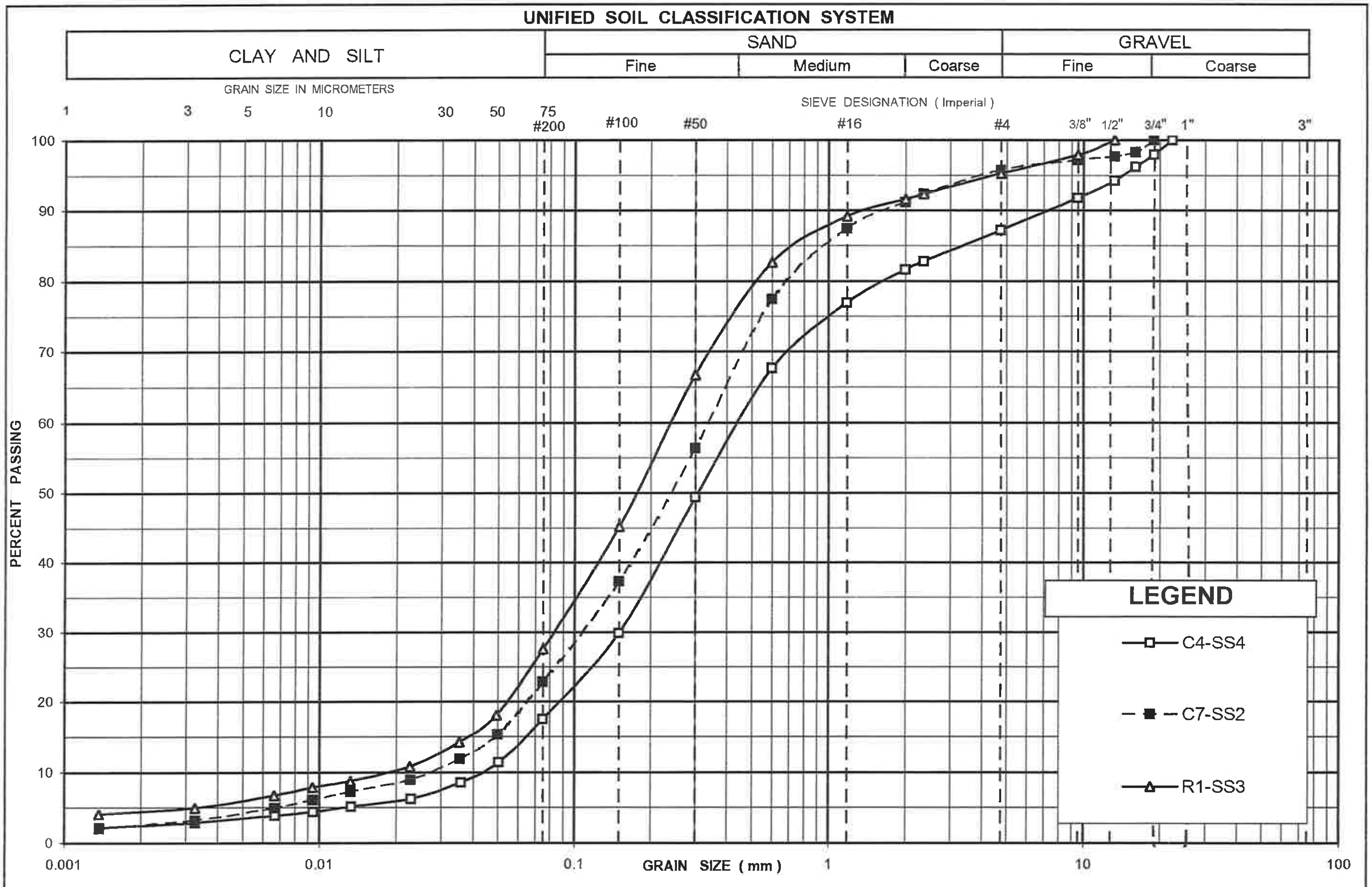


coffey geotechnics
SPECIALISTS MANAGING THE EARTH

GRAIN SIZE DISTRIBUTION

SILT, with some clay pockets and sand seams

FIGURE NO.:	B-2
PROJECT NO:	TRANETOB01238AB
DATE:	AUGUST, 2009



coffey geotechnics
SPECIALISTS MANAGING THE EARTH

GRAIN SIZE DISTRIBUTION

SAND, some gravel, trace to some silt, trace clay

FIGURE NO.:	B-3
PROJECT NO:	TRANETOB01238AB
DATE:	AUGUST, 2009

UNCONFINED COMPRESSION TEST (UC)

ASTM D 7012-04

SAMPLE IDENTIFICATION

PROJECT NUMBER	09-1183-6003 / SPT1238	SAMPLE NUMBER	RC6
BOREHOLE NUMBER	C4	SAMPLE DEPTH, m	3.7

TEST CONDITIONS

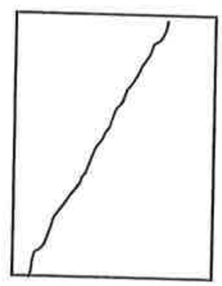
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.29

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.90	WATER CONTENT, (specimen) %	0.08
SAMPLE DIAMETER, cm	4.75	UNIT WEIGHT, kN/m^3	26.16
SAMPLE AREA, cm^2	17.72	DRY UNIT WT., kN/m^3	26.14
SAMPLE VOLUME, cm^3	193.15	SPECIFIC GRAVITY, assumed	0.00
WET WEIGHT, g	515.49	VOID RATIO	#DIV/0!
DRY WEIGHT, g	515.08		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	130.5

REMARKS:

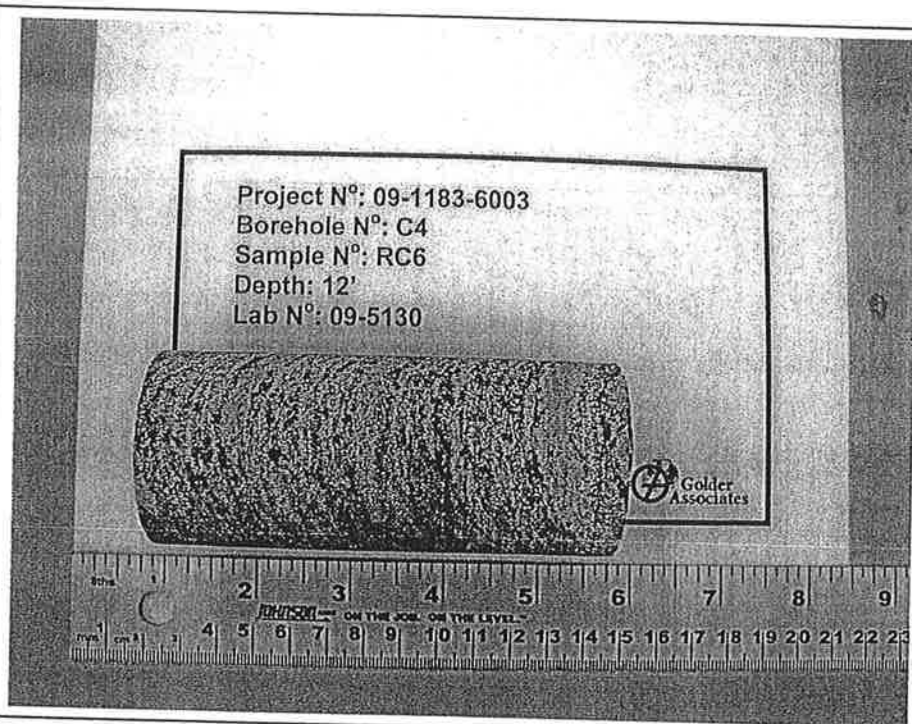
DATE: 8/24/2009

Checked By: [signature]

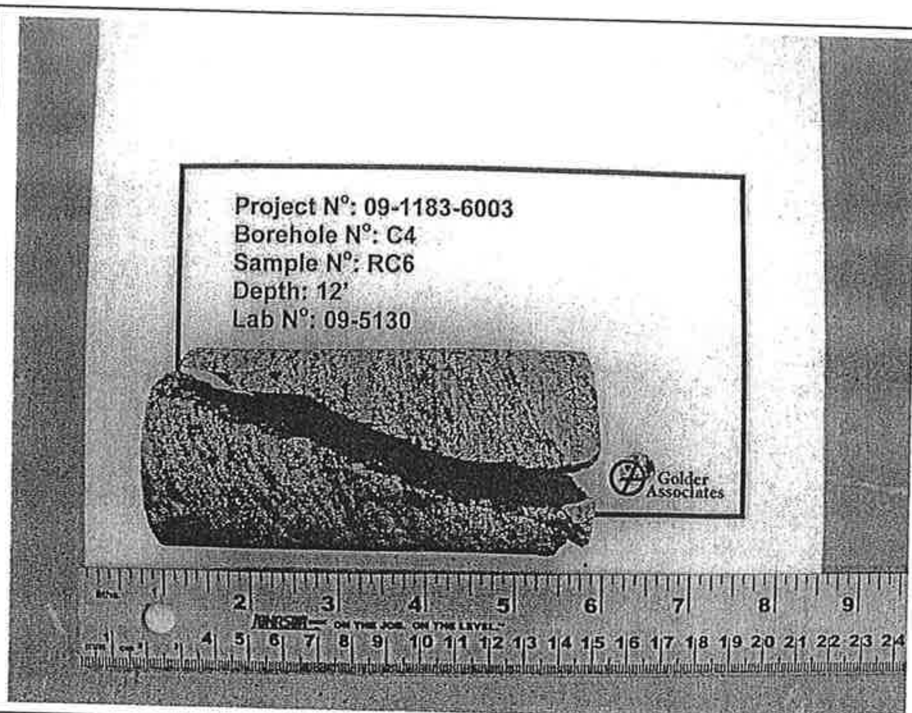
Golder Associates

UNCONFINED COMPRESSION TEST
ASTM D7012-04

FIGURE



BEFORE COMPRESSION



AFTER COMPRESSION

Date 8/25/2009

Project 09-1183-6003/SPT1238

Golder Associates

Drawn AH

Chkd. [signature]

UNCONFINED COMPRESSION TEST (UC)

ASTM D 7012-04

SAMPLE IDENTIFICATION

PROJECT NUMBER	09-1183-6003 / SPT1238	SAMPLE NUMBER	RC8
BOREHOLE NUMBER	C6	SAMPLE DEPTH, m	5.5

TEST CONDITIONS

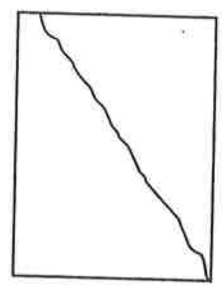
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.27

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.65	WATER CONTENT, (specimen) %	0.06
SAMPLE DIAMETER, cm	4.70	UNIT WEIGHT, kN/m^3	26.72
SAMPLE AREA, cm^2	17.35	DRY UNIT WT., kN/m^3	26.71
SAMPLE VOLUME, cm^3	184.77	SPECIFIC GRAVITY, assumed	0.00
WET WEIGHT, g	503.72	VOID RATIO	#DIV/0!
DRY WEIGHT, g	503.42		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	128.2

REMARKS:

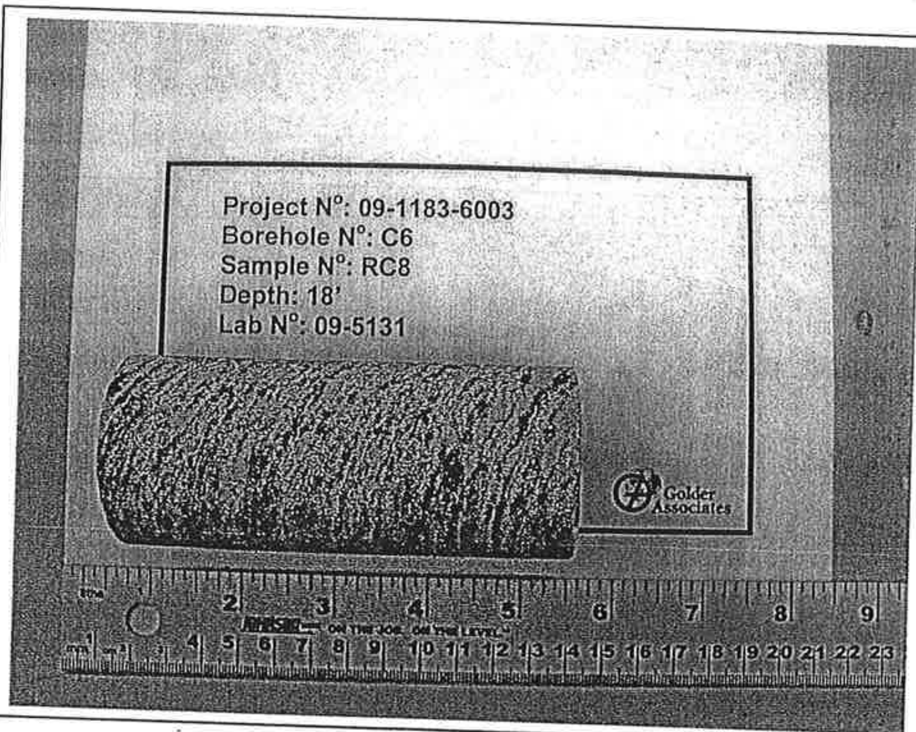
DATE: 8/24/2009

Checked By: [signature]

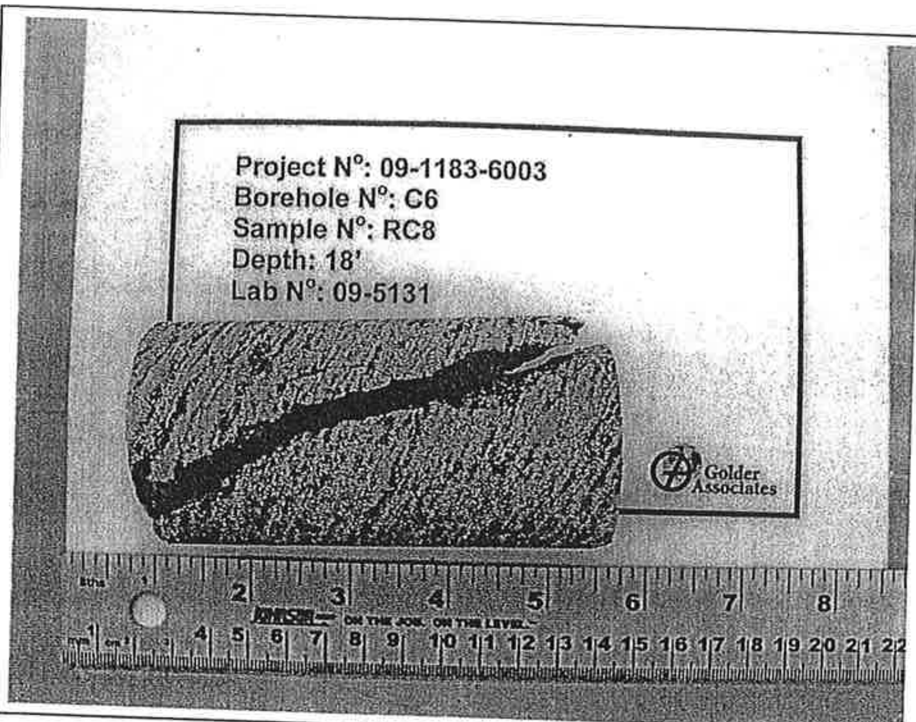
Golder Associates

UNCONFINED COMPRESSION TEST
ASTM D7012-04

FIGURE



BEFORE COMPRESSION



AFTER COMPRESSION

Date 8/25/2009

Project 09-1183-6003/SPT1238

Golder Associates

Drawn AH

Chkd. [signature]

Appendix C

Site Photographs


Figure C-1 – View of the inlet of McQuaby Creek Culvert


Figure C-2 - View of the outlet of McQuaby Creek Culvert

Appendix D

Rock Core Photographs

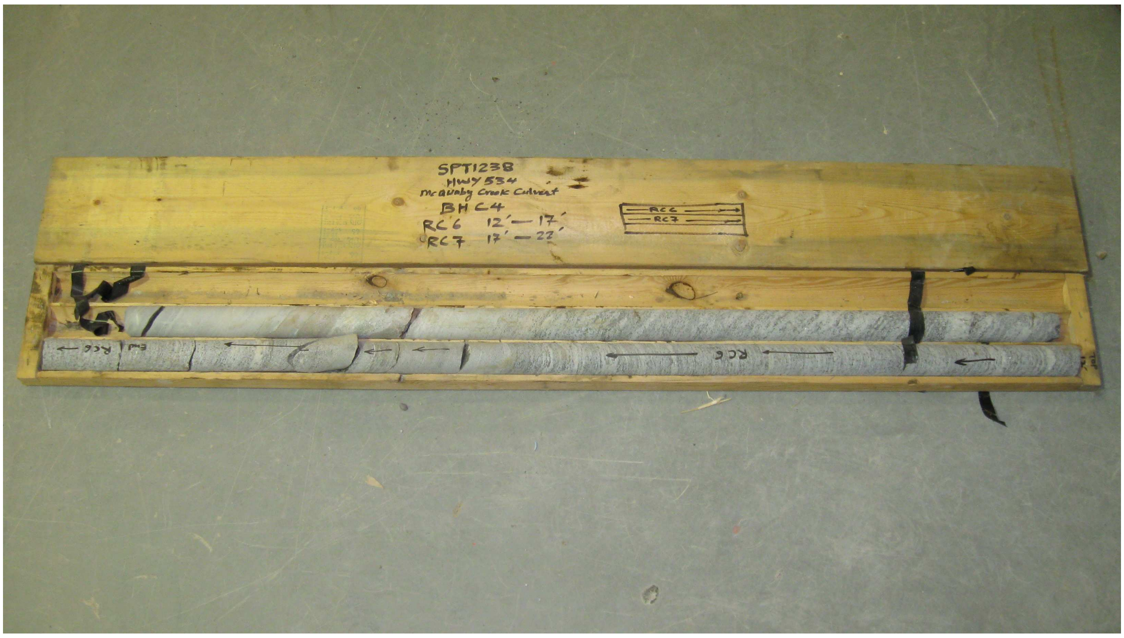


Figure D-1 – Rock core retrieved from Borehole C4

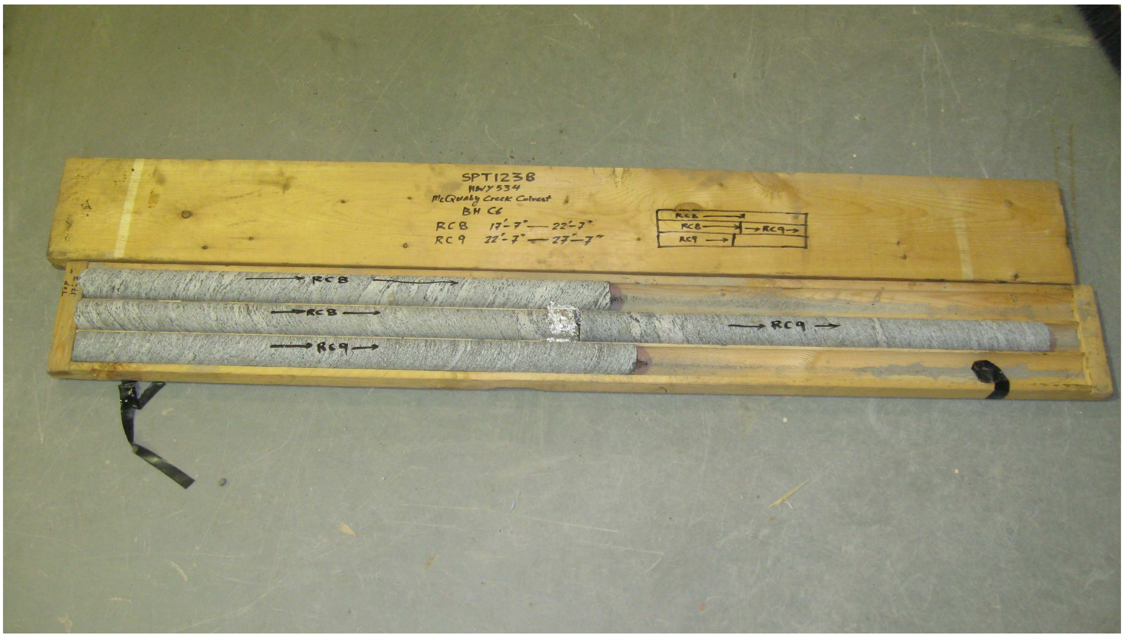


Figure D-2 – Rock core retrieved from Borehole C6

Appendix E

Explanation of Terms Used in the 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.

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 COMPOSION AND STRUCUTRAL 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
$\sigma_l, \sigma_2, \sigma_3$	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						

coffey geotechnics
SPECIALISTS MANAGING THE EARTH

FOUNDATION DESIGN REPORT
PROPOSED REHABILITATION OF MCQUABY CREEK CULVERT AT HIGHWAY 534, TOWNSHIP OF GURD, ONTARIO
G.W.P. 5053-05-00, SITE NO. 44-265/C
GEOCRES NO. 31L-138

D.M. Wills Associates Limited

Project: TRANETOB01238AB
December 17, 2009

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

CONTENTS

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PROPOSED REHABILITAION OF MCQUABY CREEK CULVERT
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5 DISCUSSION AND RECOMMENDATIONS

The McQuaby Creek, which flows northerly through the existing culvert under Highway 534, is located 6.7 km west of Highway 654 in the geographic Township of Gurd, MTO Sudbury Area. The existing McQuaby Creek culvert is a 25.6 m long structural plate corrugated steel pipe arch culvert (i.e. SPCSPA) which has 3.4 m span and 2.0 m rise. The existing culvert has a skew angle of 82° relative to the centerline of Highway 534.

According to data supplied by D.M. Wills Associates Limited (Wills), the invert of the existing SPCSPA is estimated at about El. 307.2 m at the inlet on the south side of the Highway, dropping to about El. 306.6 m at the outlet on the north side.

The existing culvert exhibits deterioration due to corrosion at critical locations (especially near the bottom) in the pipe and the rate of the associated loss of structural strength could accelerate as corrosion increases. In addition, there is a minor deformation within the upper portion of the culvert at the middle of the culvert length.

In accordance with general practice, the following repair methodologies were considered by Wills.

- Do nothing
- Concrete paving on culvert invert
- Reline culvert with corrugated steel pipe
- Replacement of culvert

Do nothing and delay the rehabilitation schedule is not recommended due to the existing culvert condition, and especially since a corrugated metal culvert is a feasible steel structure which requires interaction with circumferential soil pressure for stability. Loss of the invert portion could result in severe distortion/collapse of the culvert.

Invert paving with reinforced (i.e. wire mesh) concrete is cost effective solution to rehabilitate culverts that are not deteriorated above the springline and the extent of corrosion on the invert of the pipe is minimal. The McQuaby Creek Culvert however exhibits heavy corrosion throughout the invert with full perforation in various places. This complete section loss together with the deformation of the obvert in various places creates uncertainty in the effectiveness of a concrete invert paving option.

Relining is the most effective way to renew the culvert and provide the required service. This process is comprised of lining the existing culvert with either another CSPA or other type of construction material suited to the situation. Annular space between liner and the existing culvert is grouted using concrete. This option will not cause any adverse effects on the Highway 534 traffic.

Replacement of a deteriorated culvert with a new culvert is typically favorable option. A CSP type culvert can be considered as an alternative but it will have the same corrosion problem as the exiting one in the future. Precast concrete culvert has been favored in the northern Ontario given their potential to provide enhanced durability against aggressive environments. A staged construction using roadway protection system needs to be implemented to maintain the traffic flow during the replacement of the culvert. If staged construction is not applicable to this site, full road closure or detour option need be considered. Based on the above, as well as construction costs (i.e. approximately $282,000 for relining vs. $407,000 for replacement with a precast concrete culvert), relining option will be adopted for this project.

At the site, Boreholes C4, C5, C6 and C7, drilled in the immediate vicinity of the existing culvert show, underlying the culvert, the presence of very loose to very dense sand overlying gneiss bedrock. A natural silt layer was also found in Borehole C4 between El. 307.1 and 306.0 m. The surface of the inferred and proven bedrock ranges from El. 306.3 to 304.2 m.

Boreholes R1, R2, R3 and R4 were drilled at the site for roadway protection (i.e. to the west and east of the culvert location) from the shoulder of the existing Highway 534. Boreholes contacted embankment fill which is underlain in Borehole R3 and R4 by a 0.9 to 2.3 m thick silt deposit. The embankment fill in Borehole R1 and the silt in Boreholes R3 and R4 are underlain by a deposit of sand to refusal at El. 308.1 – 306.1 m probably on the surface of the bedrock. In Borehole R2, refusal to augering was encountered at El. 308.1 m, also on the surface of the bedrock.

At the time of our investigation, the groundwater level was encountered at about El. 307.5 to 307.0 m at Boreholes C4 and C7 in the close vicinity of the McQuaby Creek, and about 1 m higher at Boreholes R1 through R4 where the ground surface is also higher. The groundwater level would be subject to seasonal fluctuations and can also be expected to be controlled by the water level in the McQuaby Creek. Measured water level of McQuaby Creek, based on the drawing provided to us by Wills, is about El. 307.3 m (June, 2009).

5.1 Rehabilitation of Existing Culvert

To rehabilitate the existing culvert, it is proposed to reline it. We understand that this will be accomplished by placing a 2500 x 1830 mm CSPA culvert inside the existing culvert (3400 x 2010 mm CSP). A clear space of minimum 50 mm will be provided between the inside (liner) culvert and the outside (existing) culvert along the invert. The annular space between the outside and the liner culvert pipes will then be grouted. Adequate bracing will be provided within the liner and against the existing pipe to maintain line, grade and pipe liner during grouting operations.

We understand that the unit weight of the grout to be used will be about 2160 kg/m^3 and the liner pipe weight will be about 270 kg/m. According to the drawing provided to us by Wills, 2 layers of polyethylene sheet will be installed before grouting where the existing culvert bottom is fully perforated. Based on the above, the estimated increase in weight per metre is about 42 kN. Assuming a granular bedding material of about 0.2 m thick and at least 3.4 m wide, the additional stresses on the surface of the sand subgrade would be about 10 to 12 kPa.

Based on these assumptions and the information obtained from the boreholes, field and laboratory tests, the foundation settlement should not exceed 10 mm. In our opinion, this amount of settlement would not cause an undue concern neither for the performance of the road nor the culvert itself.

We recommend that during the construction the amount of grout pumped be checked and compared with calculated volumes and in the event of a discrepancy, the construction will need to be halted and the reason(s) for the discrepancy will need to be investigated.

We also recommend that the settlement (or heave) of the road surface be monitored before during and after the construction. Settlement monitoring could consist of paint mark points on the pavement along the centerline and edges of the culvert. Surface settlement points could also be installed beyond the paved portion (i.e. in the shoulder). The settlements will need to be monitored with reference to reliable, frost-free benchmark(s).

We recommend that a minimum of three sets of repeatable baseline readings be taken on all of the settlement points in advance of the start of construction. Settlement points should be conducted at least once daily during construction and twice daily during the grouting operations. After the construction, the frequency of the readings can be reduced to once weekly for two weeks and a further one more reading one month later.

5.1.1 Erosion Protection

We understand that no cut-off wall was found at both inlet and outlet during the investigation.

We recommend that the existing culvert be evaluated for the sufficiency of the existing erosion and scour measures and if observations show that they are deficient or if the relining is expected to adversely affect erosion and scour potential, further measures may be necessary. The following is a discussion of possible erosion measures.

Erosion and scour protection should be provided at the culvert inlet and outlet (including the slopes and sides). The erosion/scour protection should be designed by a specialist River Engineer/Scientist (as erosion and scour largely depend on the velocity of water in the watercourse and its regime) who is familiar with the findings of this report. The following are some general suggestions, considering sand overlying the bedrock. The sand is considered to be a highly erodible soil.

We recommend that concrete cut-off (apron) be constructed both at the inlet and outlet to prevent seepage beneath and around the culvert, especially through the granular bedding and granular backfill around the culvert. Beneath the culvert, the concrete cut-off wall should extend to a suitable depth (e.g. below any possible scour depth or to the bedrock surface if it is shallow). Consideration may also be given to an impervious seal at the inlet and outlet.

At the inlet, consideration may also be given to the use of a clay seal. The purpose of the clay seal is to ensure that water flow is channeled through the culvert and does not seep through the backfill around the structure and from beneath the structure. The clay seal should therefore be continuous and is typically 0.6 m thick. It should comply with the material specifications given in OPSS 1205. It should be extended around the culvert from at least 0.5 m above the high water level in the watercourse down to the channel bed and up the other side in a continuous manner. It should be ensured that it extends to cover all the granular backfill materials to prevent any seepage through them. Typically, the clay seal is protected by laying a 0.6 m thick rock protection over it. The clay seal would generally be extended at about 8 m beyond the inlet.

At the outlet as well as at the inlet (if clay seal is not used), in addition to the concrete cut-off and/or impervious seal or in conjunction with these, a 0.6 m thick rock protection, consisting typically of 300 mm size rock can be considered. As the subgrade can be expected to consist of granular soils, a layer of granular or man-made filter material should be used. This would generally be extended about 8 m along the channel and the sides (to at least 0.3 m above the high water). The granular filter material underlying the rock protection can consist of a suitable granular material such as Granular ‘A’. Alternatively, a suitable geotextile can be used underneath the rock fill, in lieu of the granular filter material. Another reference for consideration is OPSD 810.010 Rip-Rap Treatment for Culvert Outlets.

5.2 Wing Walls

It is unlikely that new wing walls will be required for the proposed method of rehabilitation but the following are provided for completeness.

Backfilling for any retaining (wing) walls should consist of suitable free-draining granular materials, compacted in accordance with the MTO standards and should conform to the applicable standards such as OPSD-3101.150 and SP 105S10. For fills below the groundwater level or immediately below the roadway, it is recommended that Granular ‘A’ or ‘B’ materials be used. Where necessary, proper tapering as per MTO standards should be provided. The fill should be compacted in shallow lifts, not exceeding 200 mm loose thickness, to at least 98% of the material’s Standard Proctor Maximum Dry Density (SPMDD). To avoid damaging or laterally dislocating the structure, care should be exercised when compacting fill adjacent to and immediately on top of the retaining wall structures. Compaction equipment should be restricted in size as per Ontario Ministry of Transportation (MTO) convention to prevent structural damage to the culvert.

Backfill behind any retaining (wing) walls should consist of Granular ‘B’ type materials in accordance with the MTO Standards. Free draining backfill materials, weepholes, etc. should be provided in order to prevent hydrostatic build-up, as shown on OPSD-3101.150.

Computation of earth pressures acting against rigid culvert walls and any wing walls should be in accordance with CHDBC. For design purposes, the following properties can be assumed for backfill.

Compacted Granular ‘A’ or Granular ‘B’ Type II

Angle of Internal Friction ϕ=35° (unfactored)

Unit weight = 22 kN/m^3

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_b=0.35	K_b=0.44	K_b=0.50

Level Backfill	Backfill Sloping at 3H:1V	Backfill Sloping at 2H:1V
K_o=0.43	K_o=0.56	K_o=0.62
K*=0.45	K*=0.60	K*=0.66

Compacted Granular 'B' Type I

Angle of Internal Friction ϕ=30° (unfactored)

Unit Weight = 21 kN/m^3

Coefficient of Lateral Earth Pressure:

Level Backfill	Backfill Sloping at 3H:1V	Backfill Sloping at 2H:1V
K_a=0.33	K_a=0.42	K_a=0.54
K_b=0.41	K_b=0.52	K_b=0.64
K_o=0.50	K_o=0.66	K_o=0.76
K*=0.57	K*=0.74	K*=0.86

Note: K_a is the coefficient of active earth pressure

K_b is the backfill earth pressure coefficient for an unrestrained structure including compaction efforts

K_o is the coefficient of earth pressure at rest

K* is the earth pressure coefficient for a soil loading a fully restrained structure and includes compaction effects

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

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. As well, it is 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 (e.g. supported on bedrock), then at rest pressures should be used in accordance with CHBDC S6-06. 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 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 S6-06 Commentary can be consulted.

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

As an alternative to conventional retaining walls, consideration could be given to MTO's Retained Soil System in which case the designer will have to include the geometric, performance and appearance requirements (i.e: medium performance and low to medium appearance).

Based on the findings of our investigation as revealed by Boreholes C4 through C7 and particularly Boreholes C4 and C7, strip footing foundations to support reinforced concrete retaining walls can be designed for the following tentative geotechnical resistances, provided the footings are placed on the sand deposit above the bedrock at about El. 306.0 m.

Factored Bearing Resistance at U.L.S.	=	200 kPa
Bearing Resistance at S.L.S.	=	100 kPa

These values are based on a footing width of 1.8 m and are based on the findings of Borehole C4. It appears that from the findings of Borehole C7 (of the opposite end at the inlet area) bedrock may be encountered at the quoted elevation. Therefore if the project involves strip footing foundations, the foundation design and resistances should be discussed with Coffey. In addition, frost and possibly scour will need to be taken into consideration when choosing the appropriate bearing elevation.

All footing excavations should be carefully inspected, evaluated and approved by the Geotechnical Engineer appointed by the QVE, who is familiar with the findings of this investigation.

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

The structure will need to be checked against overturning and sliding, with an appropriate factor of safety. The unfactored horizontal resistance against sliding between poured concrete and approved sand subgrade surface can be calculated using a friction angle of 28 degrees

Consideration can be given to other wall types including RSS (Reinforced Soil System), etc. Gabion type or crib type walls may also be suitable if some lateral yielding would not be objectionable. These aspects can be discussed with us, if desired, once the details of the site project are known.

5.3 Construction Comments

All excavations should be carried out in accordance with the Occupational Health and Safety Act (OHSA), Regulation 213/91, as well as well as the following specifications:

SP 105 S19 – Protection Systems

SP 902 S01 – Excavation and Backfilling to Structures

Although this is not expected, if excavations are required, the following soil classifications can be expected for temporary excavations in accordance with OHSA.

Fill and Topsoil : Type 3 soil above groundwater level

Type 4 soil below groundwater level.

Sand : Type 3 soil above groundwater level (or if the soil is dewatered)

Type 4 soil below groundwater level

Silt : Type 3 soil above groundwater level (or if the soil is dewatered)

Type 4 soil below groundwater level

Dewatering will be required to stabilize the soil and to facilitate construction if and where excavations are required. It is our opinion that in the silt and sand deposits the groundwater can be controlled and depressed by about 0.5 m by means of strategically spaced and located filtered sumps. If further drawdown is necessary then deep wells and/or well points may be required. In this instance, the position of the bedrock should be carefully considered.

In addition, the flow of water in the existing culvert will need to be diverted to facilitate the construction.

If excavations are anticipated, all bearing surfaces should be carefully evaluated and approved by the Geotechnical Engineer, appointed by the QEV. Consideration can also be given to an NSSP for proper diversion of the creek flow inside the culvert and the dewatering of excavations (especially foundation excavations), with the responsibility assigned to the Contractor.

Allowance should be made to place a skim-coat of concrete (mud-slab) once the excavation is completed, inspected and approved, without any delay.

With the proposed method, roadway protection is unlikely be required but following brief comments are provided for the sake of completeness.

Locally temporary shoring systems generally consist of support provided by conventional soldier piles and timber lagging. For shallow excavations, the system can be designed as cantilever structures or supported by raker footings. They can also employ a soil anchor system, depending upon the depth of soil required to be supported and the performance criteria used. A tight interlocking steel sheet piling system is sometimes also used.

The suggested coefficient of lateral earth pressures based on the findings of boreholes is given in Table 5.3.1, for the design of the shoring system. The shoring system should be designed by a professional engineer, experienced in this type of work.

Table 5.3.1: Recommended Unfactored Parameters for Temporary Shoring Design

Soil Type	**K_a**	**K_o**	**K_p**	**γ(kN/m^3)**
Granular Embankment Fill	0.30	0.45	3.3	21.5
Embankment Fill	0.36	0.53	2.8	20.5

Soil Type	K_a	K_o	K_p	$\gamma (kN/m^3)$
Silt	0.38	0.55	2.7	18.0
Peat and Silt	0.50	0.68	1.8	15.0
Sand (loose to v. loose)	0.36	0.53	2.8	19.5
Sand (compact to dense)	0.32	0.49	3.1	20.5

It should be pointed out that the position of the bedrock at the site will greatly influence the type and cost of the shoring system. This aspect should be taken into consideration in the design, in which case additional boreholes may need to be put down to explore undulations in the bedrock surface.

5.4 Frost Protection

Design frost protection for the general area is 1.9 m. A permanent soil cover of at least 1.9 m or its thermal equivalent is therefore required for frost protection. In case of riprap (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 quoted in Appendix G, are an integral part of this report.

For and on behalf of Coffey Geotechnics Inc.

Gwangha Roh, Ph.D.

LICENSED PROFESSIONAL ENGINEER
RAMON MIRANDA
PROVINCE OF ONTARIO

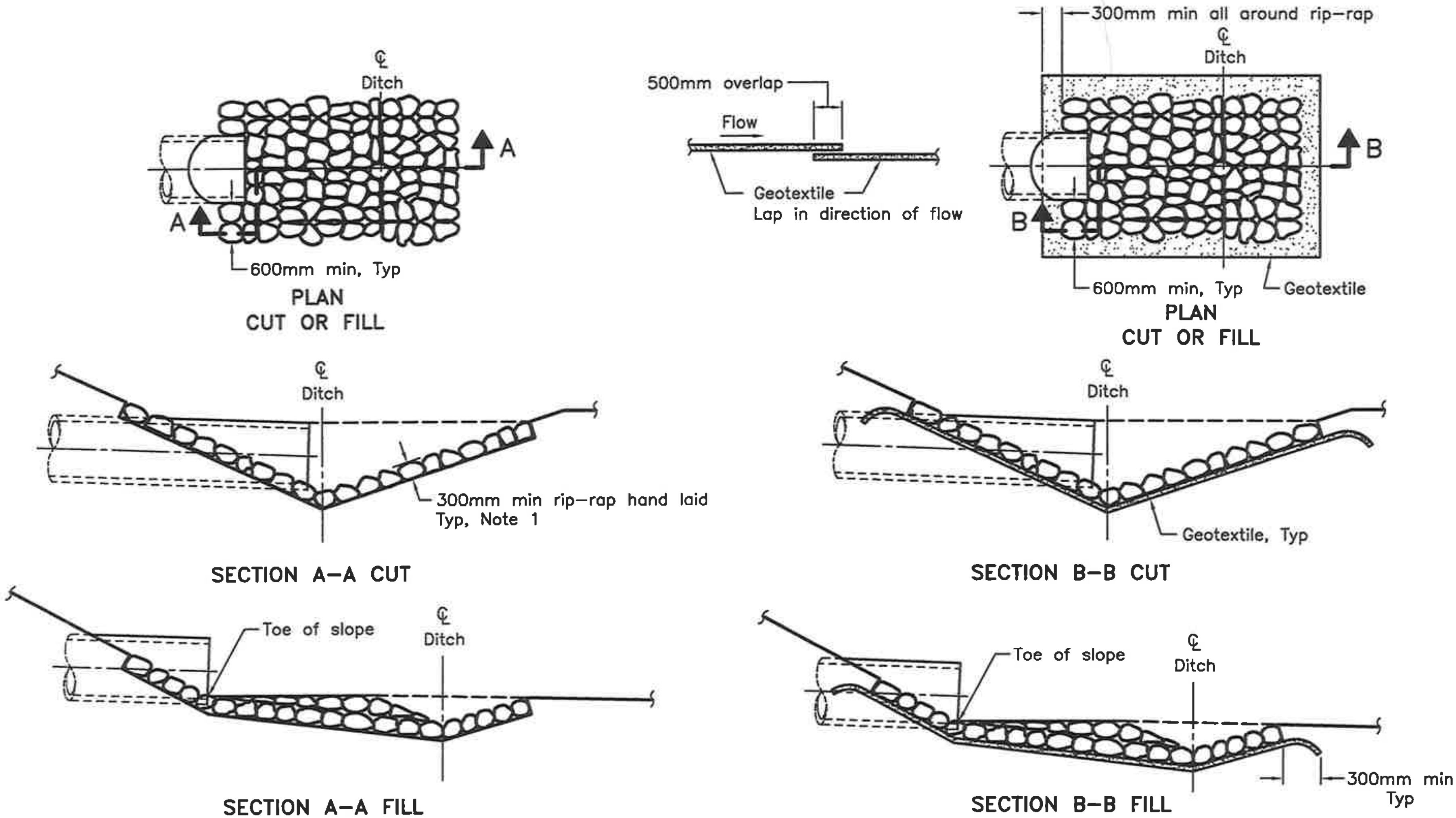
Ramon Miranda, P.Eng.

Zuhtu Ozden, P.Eng.

REGISTERED PROFESSIONAL ENGINEER
Z. S. OZDEN
PROVINCE OF ONTARIO

Appendix F

OPSD



TYPE A – WITHOUT GEOTEXTILE

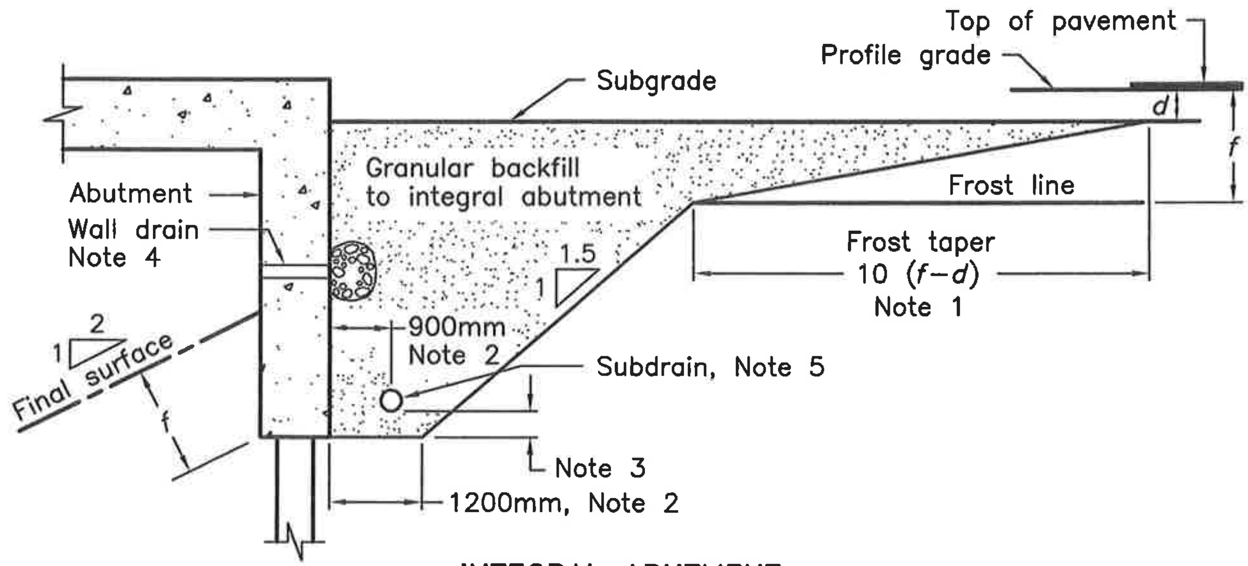
TYPE B – WITH GEOTEXTILE

NOTES:

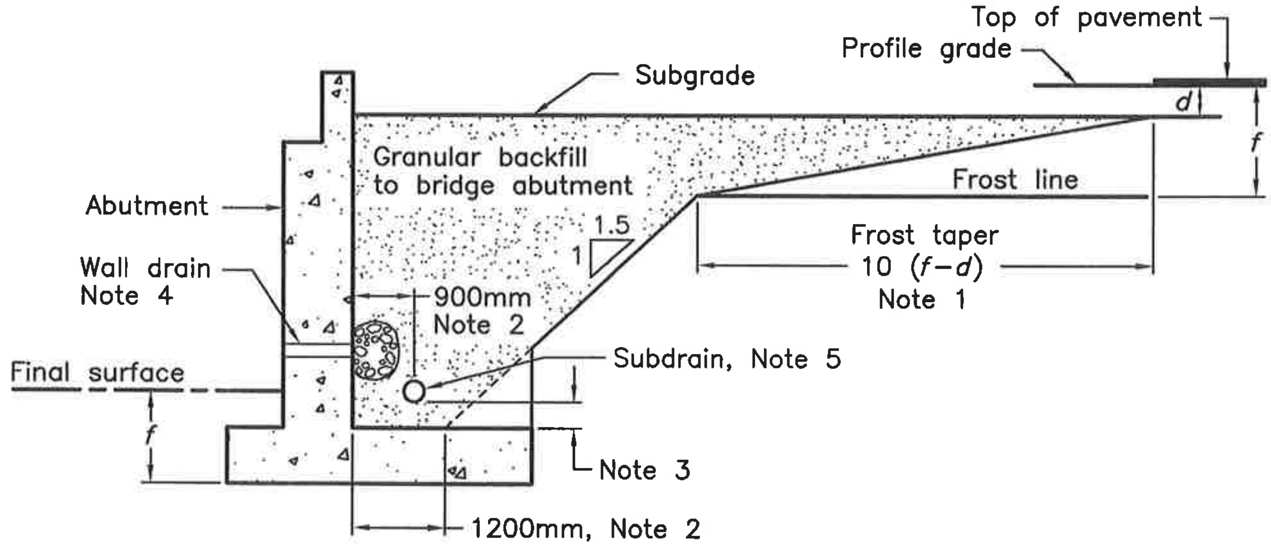
1 The thickness of the rip–rap layer shall be at least 1.5 times the rip–rap mean diameter.

A All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING	Nov 2007	Rev 1
RIP–RAP TREATMENT FOR SEWER AND CULVERT OUTLETS		
	OPSD 810.010	



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 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.