

**FOUNDATION INVESTIGATION AND
DESIGN REPORTS – PROPOSED
CULVERT EXTENSION AND PARTIAL
REPLACEMENT, MIDTOWN CREEK WEST
AT STATION 19+560, HIGHWAY 401,
COBOURG, ONTARIO
W.P. 205-00-01, GEOCRES NO. 30M16- 45**

AECOM

TRANETOB010434AA-AF
February 6, 2012

February 6, 2012

AECOM
5080 Commerce Boulevard
Mississauga, ON L4W 4P2

Attention: Ms. Peggy Baleka

Dear Ms. Baleka;

RE: Foundation Investigation and Design Reports-Proposed Culvert Extension and Partial Replacement, Midtown Creek West at Station 19+560, Highway 401, Cobourg, Ontario, W.P 205-00-01.

Please find attached the Foundation Investigation and Design Reports relating to the above noted site.

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

For and on behalf of Coffey Geotechnics Inc.


Ramon Miranda, P. Eng.

Principal

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3 bound copies to AECOM

**FOUNDATION INVESTIGATION REPORT
PROPOSED CULVERT EXTENSION AND
PARTIAL REPLACEMENT, MIDTOWN
CREEK WEST AT STATION 19+560,
HIGHWAY 401, COBOURG, ONTARIO
W.P. 205-00-01, GEOCRES NO. 30M16- 45**

AECOM

TRANETOB010434AA-AF
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**FOUNDATION INVESTIGATION REPORT
MIDTOWN CREEK WEST CULVERT EXTENSION AND PARTIAL REPLACEMENT
AT STATION 19+560, HIGHWAY 401, COBOURG, ONTARIO
W.P. 205-00-01**

1 INTRODUCTION

As part of the expansion (six laning) of Highway 401, from Burnham Street in Cobourg, Ontario, to approximately 2.0 km east of Nagle Road (a total length of 6.5 km), it is proposed to extend the existing Midtown Creek West non-rigid frame open (NRFO) culvert at Station 19+560. Partial replacement of the structure may also be necessary.

Coffey Geotechnics Inc. (Coffey) was retained by AECOM to carry out geotechnical investigations for the widening of this 6.5 km long section of the Highway and the foundation investigation reported herein for the above culvert constitutes part of this project. There are four (4) culverts in this project to be addressed and they will be reported separately as per the project brief and Midtown Creek West culvert forms one of them.

The purpose of the geotechnical investigation was to obtain information about the Site and the subsurface conditions by means of boreholes, field and laboratory tests. Based on the information obtained, the engineering characteristics of the subsurface soils are assessed and site conditions are described.

This report presents factual information concerning the subsurface conditions, based on the foundation investigation undertaken.

2 SITE DESCRIPTION AND PHYSIOGRAPHY

The Midtown Creek West culvert is located at Station 19+560 under Highway 401, in the Town of Cobourg, as shown in Drawing No. 1. The existing culvert (variable span: 2.12 m to 2.44 m from inlet to outlet; rise: ~1.83 m and approximately 64 m long). This culvert is to be extended by 4 m to 5 m on each side to accommodate the embankment widening and a length of 8 m from the inlet end (northern end) may be replaced. The existing culvert is at a skew to the highway alignment.

Midtown Creek West flows in an approximately north to south direction, is 4 to 5 m wide and at the inlet end, up to 2 m deep, at the Highway 401 crossing.

The topography is mildly rolling. Photographs of the site are presented in Appendix C. According to "The Physiography of Southern Ontario" by L.J. Chapman and D.F. Putnam, 1984, the culvert site is located within the Iroquois Plain. The Iroquois Plain was previously inundated by a body of water known as Lake Iroquois, the forerunner to the present Lake Ontario. Iroquois Plain at Cobourg is about 5.6 km in width and has peculiar belted pattern. The land within the project area is almost flat and is covered by glacial and/or glacial lake deposits overlying a sandy glacial till deposit.

The bedrock within the project area consists of limestone, dolostone, shale, arkose and sandstone of the Simcoe Group from the Middle Ordovician (Bedrock Geology of Ontario, Southern Sheet, Map 2544).

The approach embankments to the culvert do not show signs of apparent instability or noticeable erosion. However, significant scour erosion has been reported at the existing culvert. See site photographs in

Appendix C. The pavement in the vicinity does not indicate any distress in terms of settlements/unusual cracking or deformations.

3 FIELD AND LABORATORY WORK

The fieldwork for this culvert investigation was performed during the period from July 15, 2010 to November 29, 2010, and consisted of drilling and sampling a total of five boreholes (C1 to C4A). Table 1 presents the borehole details and the borehole locations at the site are shown on Borehole Location Plan, Drawing No. 1.

Table 1: Borehole Details

Borehole Number	Station	Offset (approximate)	Ground Elevation (m)	Drilled Depth (m)	Remarks
C1	19+560	37.0 m Lt C/L (Outside the toe of embankment)	110.6	13.9	Track-mounted rig Solid stem auger Piezometer installed
C2	19+560	18.0 m Lt C/L (From the embankment shoulder)	113.4	12.2	Track-mounted rig Solid stem auger
C3	19+556	25.0 m Rt C/L (Outside the toe of embankment)	110.9	4.0	Track-mounted rig Hollow stem auger Piezometer installed
C4	19+559	36.0 m Rt C/L (Outside the toe of embankment)	108.9	2.5	Track-mounted rig Hollow stem auger
C4A	19+560	34.0 m Rt C/L (Outside the toe of embankment)	109.0	7.8	Track-mounted rig Solid stem auger

Boreholes C1, C2 and C4A were advanced using a track-mounted drilling rig owned and operated by Eastern Soil Investigation Limited of Ajax, Ontario. Borehole C2 was advanced with the track-mounted rig sitting on plywood planks to protect the road surface. Boreholes C3 and C4 were advanced using a track-mounted drilling rig owned and operated by Strong Soil Search Inc. of Claremont, Ontario.

No rock coring was undertaken.

The borehole locations were established in the field by Coffey engineering staff, utilizing existing station markings in the field and in relation to existing features. The ground elevations and co-ordinates at the borehole locations were determined by the client's surveyors and were provided to Coffey. All boreholes were carried under full-time supervision of technical personnel from Coffey.

Soil samples in the boreholes were taken at regular depth intervals by the Standard Penetration Test method (SPT) carried out in general accordance with ASTM D1586. The test consists of freely dropping a 63.5 kg hammer a vertical distance of 0.76 m to drive a 51 mm outside diameter (OD) split-barrel (SS-split-spoon) sampler into the ground. The number of blows of the hammer required to drive the sampler into the relatively undisturbed ground by a vertical distance of 0.30 m is recorded as the Standard Penetration Resistance or the N-value of the soil which is indicative of the compactness condition of

granular (or cohesionless) soils (gravels, sands and silts) or the consistency of cohesive soils (clays and clayey soils).

An in-situ shear vane test was attempted within the cohesive stratum intercepted in Borehole C2 to assess the undrained shear strength of the soil.

Borehole C4A, advanced adjacent to Borehole C4, was a confirmatory borehole which was undertaken solely to investigate the basal silty sand to sandy silt till in the vicinity of Borehole C4. Borehole C4 had to be terminated at shallow depth due to auger refusal and it was necessary to establish whether this early refusal was due to large boulders in the general vicinity or if it was due to an isolated boulder. Therefore Borehole C4A was augered straight from the ground surface to the top of the till layer and SPT testing and soil description were undertaken only within the till layer.

Groundwater conditions were observed during and on completion of drilling in the open boreholes. Standpipe piezometers were installed in Boreholes C1 and C3 upon their completion. The remaining boreholes were grouted using a cement/bentonite mixture as per MTO procedures. Note the piezometers installed have not been decommissioned as they may be useful in monitoring water level prior to or during the construction. As part of the construction, the piezometers need to be decommissioned in accordance with Ontario Regulation 903 (amended to Ontario Regulation 372/07).

A laboratory testing program, consisting of natural water content tests, Atterberg Limits tests and grain size analyses including hydrometer testing, was performed on selected samples. The results of the laboratory tests are presented on the appropriate Record of Borehole Sheets in Appendix A, and also in Appendix B.

4 SUBSURFACE CONDITIONS

Details of the subsurface conditions encountered in the boreholes are presented on the Record of Borehole Sheets in Appendix A. Appendix D presents the Explanation of Terms Used in Report.

The subsurface conditions encountered at the culvert location are described in the following sections. The following descriptions of the individual strata are to assist the designers of the project with an understanding of the anticipated subsurface conditions underlying the site. It should be noted that the subsurface conditions may vary in between and beyond the borehole locations. Drawing 1 presents the inferred stratigraphic section at the Culvert location based on the borehole data.

In summary, the encountered subsurface conditions at the culvert location consisted of approximately 0.3 m to 4.6 m thick fill, over natural overburden soils within the depths investigated. Drilling was terminated within the silty sand to sandy silt till encountered in all the boreholes upon reaching either SPT spoon refusal or auger refusal.

Fill was intercepted in all the boreholes. In Borehole C2, the fill comprises a pavement fill (~0.5 m thick), underlain by a silty sand to sandy silt embankment fill. In Boreholes C1, C3 and C4, the silty sand to sandy silt embankment fill was intercepted beneath a veneer of topsoil. Broadly, this silty sand to sandy silt embankment fill has a trace to some clay and gravel and varied in thickness from 0.3 m to 4.1 m, at the borehole locations.

Underlying the fill, a clayey silt layer was intercepted in all the boreholes, except in Borehole C3, with a thickness varying from 0.7 m to 1.5 m. The basal layer, encountered in all the boreholes, within the

investigated depths, is a silty sand to sandy silt till in which drilling was terminated. Penetrations into this basal layer ranging in thickness from 1.0 m to 10.1 m were achieved at the time of termination.

The following paragraphs are presented only to amplify and complement the data presented on the Record of Borehole Sheets. It is to be noted that the elevations reported for strata boundaries are from the shallowest occurrence to the deepest occurrence based on the borehole data.

4.1 Fill

4.2 Pavement Fill

A pavement fill was encountered to a depth of 0.5 m (Elevation 112.9 m) from the surface in Borehole C2, drilled from the embankment shoulder.

This pavement fill is granular in nature and consists of sand with some silt and gravel.

A SPT 'N' values of 10 blows/0.3 m was measured within the pavement fill, indicating a loose relative density, however a single result is insufficient to make a general assessment of the state of compaction.

4.3 Embankment Fill – Silty Sand to Sandy Silt

Underlying the pavement fill, an embankment fill was intercepted that extended to a depth of 4.6 m (Elevation 108.8 m) in Borehole C2, and in the remaining boreholes, this embankment fill was present beneath a veneer of topsoil to depths of 0.6 m to 3.0 m (Elevations 108.3 m to 107.9 m).

Grain size analysis was carried out on two representative samples of this fill material. The results are presented on the Record of Borehole Sheets in Appendix A and the grain size curves are presented in Figure B1, Appendix B. The curves show the following grain-size distributions:

Gravel:	0 - 5%
Sand:	22 - 40%
Silt:	40 - 51%
Clay:	15 - 27%

Based on the grain size analyses and visual description of the SPT samples, the fill material consisted of silty sand to sandy silt with trace to some clay and gravel. The presence of organic inclusions was also noted. This embankment fill is classified as a basically granular (i.e. non cohesive) material.

Standard Penetration Tests performed in this layer yielded an average SPT 'N' -value of 13 blows/0.3 m, (SPT 'N' ranging from 3 to 30; 14 test results). This indicates generally a loose to compact relative density, with SPT 'N' values indicating a loose state towards the bottom of the fill. Natural water contents measured on samples recovered from this stratum have an average of 18.0% (ranging from 5.0% to 42.0%; 14 test results). Increased moisture contents noted in this layer with depth are probably due to the organic material intercepted.

4.4 Topsoil

A veneer of topsoil, 0.2 m to 0.3 m thick, was encountered in Boreholes C1, C3 and C4. However, it is to be noted, based on our experience, the thickness of organic rich top soils frequently varies in between and beyond borehole locations, especially near water courses.

4.5 Clayey Silt

A clayey silt deposit was encountered in all the boreholes except in Borehole C3, at depths of 0.6 m to 4.6 m (Elevations 108.8 m to 108.3 m), and extended to depths of 1.5 m to 5.3 m (Elevations 108.1 m to 106.8 m). The thickness of this deposit at the locations of the boreholes was found to range from 0.7 m to 1.5 m.

Grain size analyses were carried out on two representative samples of this deposit. The results are presented on the Record of Borehole Sheets in Appendix A and the grain size curves are presented in Figure B2, Appendix B. The curves show the following grain-size distributions:

Gravel:	0 - 1%
Sand:	5 - 6%
Silt:	59 - 60%
Clay:	34 - 35%

Based on these results, the clayey silt layer is considered to be of low permeability.

Atterberg Limit tests were performed on two representative samples from the stratum. As shown in Figure B3 in Appendix B, the tests indicated the following index values:

Liquid Limit:	26 – 28%
Plastic Limit:	17 – 19%
Plasticity Index:	9%

The above values are characteristic of a cohesive soil of low plasticity (CL).

Standard Penetration Tests performed in this deposit yielded an average SPT 'N'-value of 12 blows/0.3 m, (SPT 'N' ranging from 5 to 20; 4 test results), indicating a firm to very stiff consistency. Natural water contents measured on four samples recovered from this stratum yielded an average of 18.8% (ranging from 14.3% to 20.8%; 4 test results).

Based on these field and laboratory test results, the clayey silt deposit is considered to be somewhat over-consolidated and of firm to very stiff consistency.

4.6 Silty Sand to Sandy Silt Till

Underlying the strata discussed in the preceding paragraphs, encountered in all the boreholes as the basal layer within the depths investigated, is a silty sand to sandy silt till deposit at depths of 1.5 m to 5.3 m (Elevations 108.1 m to 104.4 m), and extended to the termination of the boreholes at depths of 2.5 m to 13.9 m (Elevations 106.9 m to 96.7 m).

The following are results of grain size distribution tests conducted on eight representative samples as shown in Figure B4 in Appendix B.

The curves show the following grain-size distributions:

Gravel:	0 - 23%
Sand:	9 - 54%
Silt:	25 - 72%
Clay:	8 - 19%

Natural water contents measured on representative samples recovered from this stratum have an average of 8.6% (ranging from 4.5% to 17.7%; 26 test results).

Based on visual & tactile observations and grading and moisture information, this deposit can be described as a moist to wet, grey, heterogeneous mixture of silty sand to sandy silt, with trace to some gravel and clay. Hence the deposit is classified as a basically granular (non-cohesive) soil. Due to the clay content (where it occurs) some cementation can be expected. As encountered in the boreholes drilled, the presence of cobbles and boulders should always be anticipated in glacial till deposits, due to their mode of deposition.

Standard Penetration Tests performed in this stratum yielded SPT 'N'-values ranging from 32 blows/0.3 m, to in excess of 100 (24 test results). These SPT 'N' results indicate typically a dense to generally very dense relative density.

All boreholes were terminated within this deposit at depths of 2.5 m to 13.9 m (Elevations 106.9 m to 96.7 m) due to high penetration resistance encountered by the SPT spoon or auger refusal.

4.7 Groundwater Conditions

Groundwater levels were observed in the open boreholes while drilling and upon completion of each borehole. In addition, two piezometers were installed in Boreholes C1 and C3 to monitor the groundwater levels with time, without interference from surface water. The tips of piezometers were installed within the silty sand till.

All the observations are shown on the individual Record of Borehole sheets in Appendix A and the most current observations are summarized below in Table 2.

Table 2: Summary of Groundwater Levels

Borehole No	Ground Surface Elevation (m)	Depth/Elevation of the Tip of Piezometer (m)	Water Level Measurement (most current)		Highest Water Level Measured Elevation (m)
			Depth/Elevation (m)	Date Measured	
C1	110.6	6.1/104.5	1.2/109.4	15 Oct 2010 (92 days after completion)	109.4 15 Oct 2010 (92 days after completion)
C2	113.4	Not installed	5.9/107.5* Cave-in @ 6.3 m	15 Jul 2010	—
C3	110.9	3.8/107.1	1.9/109.0	15 Oct 2010 (80 days after completion)	109.0 15 Oct 2010 (80 days after completion)
C4	108.9	Not installed	Dry on completion	27 July 2010	—
C4A	109.0	Not installed	3.4/105.6* Cave-in @ 7.4 m	29 Nov 2010	—

Note: * Groundwater level measured not stabilized.

In general, groundwater levels measured on completion, are not considered to have stabilised, therefore, may not represent the groundwater table at the site. Further, the highest groundwater levels measured at the site in Boreholes C1 and C3 were both about 1.1 m above the existing original ground surface (o.g.) level or approximately at Elevations 109.4 m and 109.0 m respectively.

These observations in Table 2 therefore suggest the existence of an artesian head in the silty sand to sandy silt till.

It should be pointed out that the groundwater would be subject to seasonal fluctuations and fluctuations in response to major weather events. The water table at the site may be influenced by the water level in the watercourse. A perched water table may also occur due to the accumulation of the surface water in the surficial fills overlying the low permeable clayey silt.

For and on behalf of Coffey Geotechnics Inc.



Vasantha Wijeyakulasuriya, M.Eng.

Principal



Ramon Miranda, P.Eng.

Principal



Zuhtu Ozden, P.Eng.

Senior Principal



Drawing

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.

coffey geotechnics
SPECIALISTS MANAGING THE EARTH



LEGEND

- Borehole
- N Blows/0.3m (Std. Pen. Test, 475 J/blow)
- Water Level in Piezometer
- Piezometer
- Section

No.	ELEVATION	EASTING	NORTHING
C1	110.6	412009.4	4873263.2
C2	113.4	412021.9	4873246.6
C3	110.9	412045.7	4873211.0
C4	109.9	412053.1	4873205.6
C4A	109.0	412053.6	4873206.6

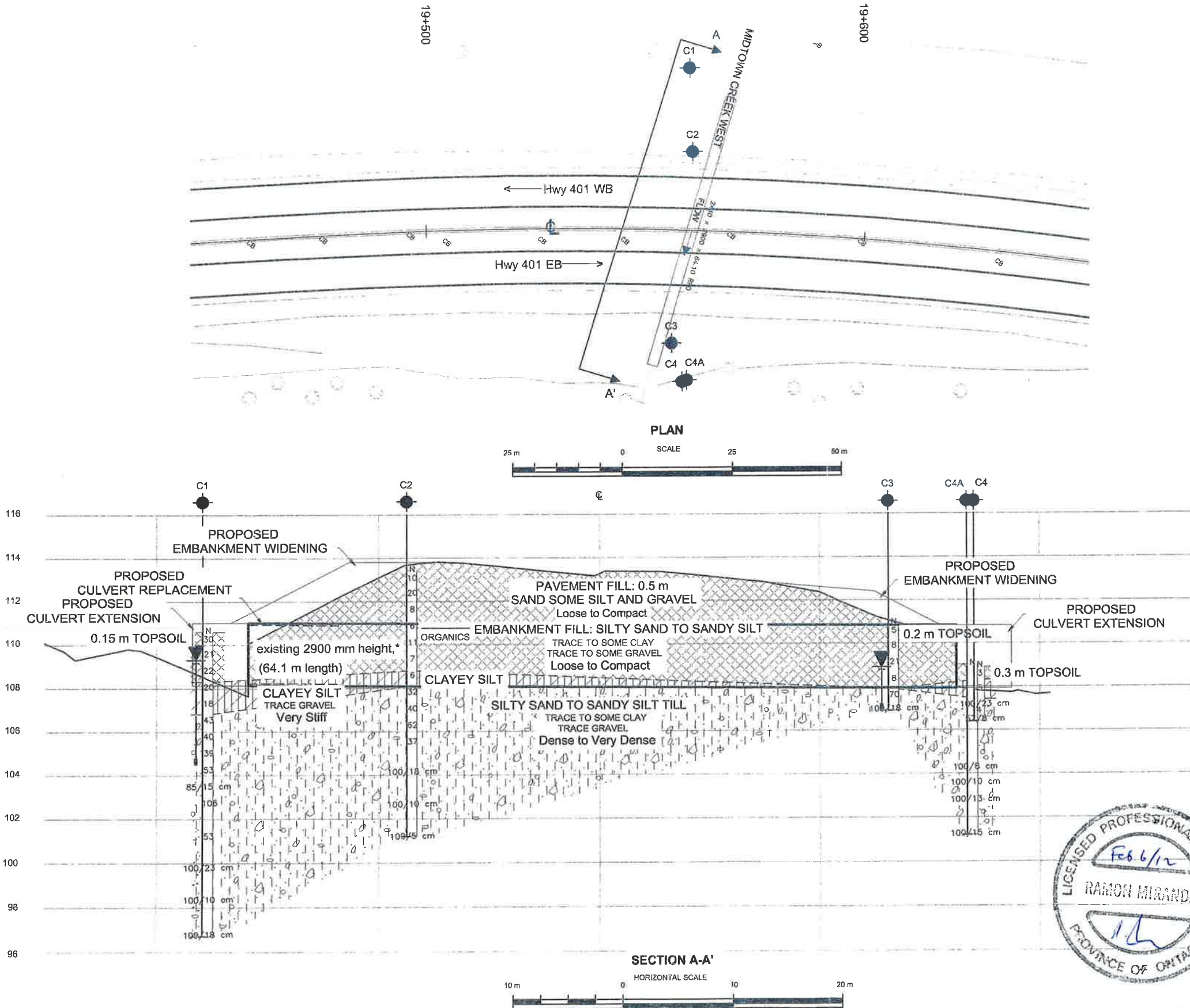
* The height between the top of footings & the underside of the soffit is 1.83 m.

-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.	TRANET0810434AA	DIST	
SUBMD	CHECKED	DATE	Feb 14, 2011
DRAWN	SH	CHECKED	RM
APPROVED	ZO	DWG	



Appendix A

Record of Borehole Sheets

TRANETOB10434AA: Highway 401

RECORD OF BOREHOLE No C1

1 OF 2

METRIC

GWP G.W.P. 205-00-01 LOCATION Station 19+559, 37 m Lt of C/L (E 412009.4, N 4873263.2) ORIGINATED BY GJ
 DIST HWY 401 BOREHOLE TYPE Solid Stem Auger COMPILED BY WC
 DATUM Geodetic DATE 7/15/2010 CHECKED BY ZO

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)						
							○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE						
							20 40 60 80 100						
110.6	GROUND SURFACE												
0.0	0.2 m TOPSOIL												
	EMBANKMENT FILL: Silty Sand tr. gravel, tr. to some clay, some organics compact, moist	1	SS	30		110							
		2	SS	21									
		3	SS	22		109							spoon wet below 1.5 m
108.3		4	SS	20		108							low recovery due to presence of gravel
2.3	CLAYEY SILT tr. gravel, v. stiff, moist to wet	5	SS	18		107							0 5 60 35
106.8		6	SS	43		106							
3.8		7	SS	40		105							
	SILTY SAND TO SANDY SILT TILL tr. gravel, tr. to some clay dense to v. dense moist to wet	8	SS	39		104							6 23 59 12
		9	SS	53		103							
		10	SS	85 / 15		102							
		11	SS	106		101							
		12	SS	53		100							0 9 72 19
		13	SS	100 / 23		99							
		14	SS	100 / 10		98							
		15	SS	100 / 18		97							11 54 25 10
96.7	End of Borehole.												
13.9	Piezometer installed at 6.1 m. Date / Water level measurement: July 15, 2010 / 1.8 m July 16, 2010 / 1.5 m												

Continued Next Page

+ 3, x 3

Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

TRANETO10434AA: Highway 401

RECORD OF BOREHOLE No C1

2 OF 2

METRIC

GWP G.W.P 205-00-01 LOCATION Station 19+559, 37 m Lt of C/L (E 412009.4, N 4873263.2) ORIGINATED BY GJ
 DIST HWY 401 BOREHOLE TYPE Solid Stem Auger COMPILED BY WC
 DATUM Geodetic DATE 7/15/2010 CHECKED BY ZO

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			N-VALUES	SHEAR STRENGTH (kPa)					
95.6	August 19, 2010 / 1.3 m October 15, 2010 / 1.2 m												

METRIC

+ 3, × 3 Numbers refer to Sensitivity

TRANETO10434AA: Highway 401

RECORD OF BOREHOLE No C3

1 OF 1

METRIC

GWP G.W.P 205-00-01 LOCATION Station 19+557, 26 m Rt of C/L (E 412045.7, N 4873211.0) ORIGINATED BY GJ
 DIST HWY 401 BOREHOLE TYPE Hollow Stem Auger COMPILED BY WC
 DATUM Geodetic DATE 7/27/2010 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE							
110.9 0.0	GROUND SURFACE														
	0.2 m TOPSOIL		1	SS	5										
	EMBANKMENT FILL: Silty Sand to Sandy Silt tr. to some clay loose to compact, moist to wet tr. organics		2	SS	8										
			3	SS	21										
			4	SS	8									5 40 40 15	
107.9 3.0			5	SS	70									4 41 43 12	
	SILTY SAND TILL some clay v. dense, moist		6	SS	100 / 18										
106.9 4.0	End of Borehole. Borehole caved-in @ 3.8 m upon completion. Piezometer installed @ 3.8 m upon completion. Date / Measured water level July 27, 2010 / dry - on completion August 19, 2010 / 2.0 m October 15, 2010 / 1.9 m														

TRANETO10434AA: Highway 401

RECORD OF BOREHOLE No C4

1 OF 1

METRIC

GWP G.W.P 205-00-01 LOCATION Station 19+559, 35 m Rt of C/L (E 412053.0, N 4873205.6) ORIGINATED BY SK
 DIST HWY 401 BOREHOLE TYPE Hollow Stem Auger COMPILED BY WC
 DATUM Geodetic DATE 7/27/2010 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)							
							20	40	60	80	100				
							○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE			WATER CONTENT (%)					
							20	40	60	80	100	10	20	30	
							PLASTIC LIMIT			NATURAL MOISTURE CONTENT			LIQUID LIMIT		
							w _P			w			w _L		
108.9	GROUND SURFACE														
0.0	0.3 m TOPSOIL		1	SS	3										
108.3	EMBANKMENT FILL: Silty Sand to Sandy Silt tr. to some clay, v. loose, wet		2	SS	5										
0.6	CLAYEY SILT some sand, firm, wet														
107.4			3	SS	100 / 23 cm										
1.5	SILTY SAND TILL some gravel, tr. clay v. dense, moist to wet		4	SS	57 / 8 cm										
106.4	End of Borehole. Borehole was dry upon completion. Another borehole drilled @ 3.5 m east to confirm refusal. (See Record of Borehole C4A)														
2.5															

TRANETO10434AA: Highway 401

RECORD OF BOREHOLE No C4A

1 OF 1

METRIC

GWP G.W.P 205-00-01 LOCATION Station 19+560, 34 m Rt of C/L (E 412053.6, N 4873206.6) ORIGINATED BY LG
 DIST HWY 401 BOREHOLE TYPE Solid Stem Auger COMPILED BY WC
 DATUM Geodetic DATE 11/29/2010 CHECKED BY ZO

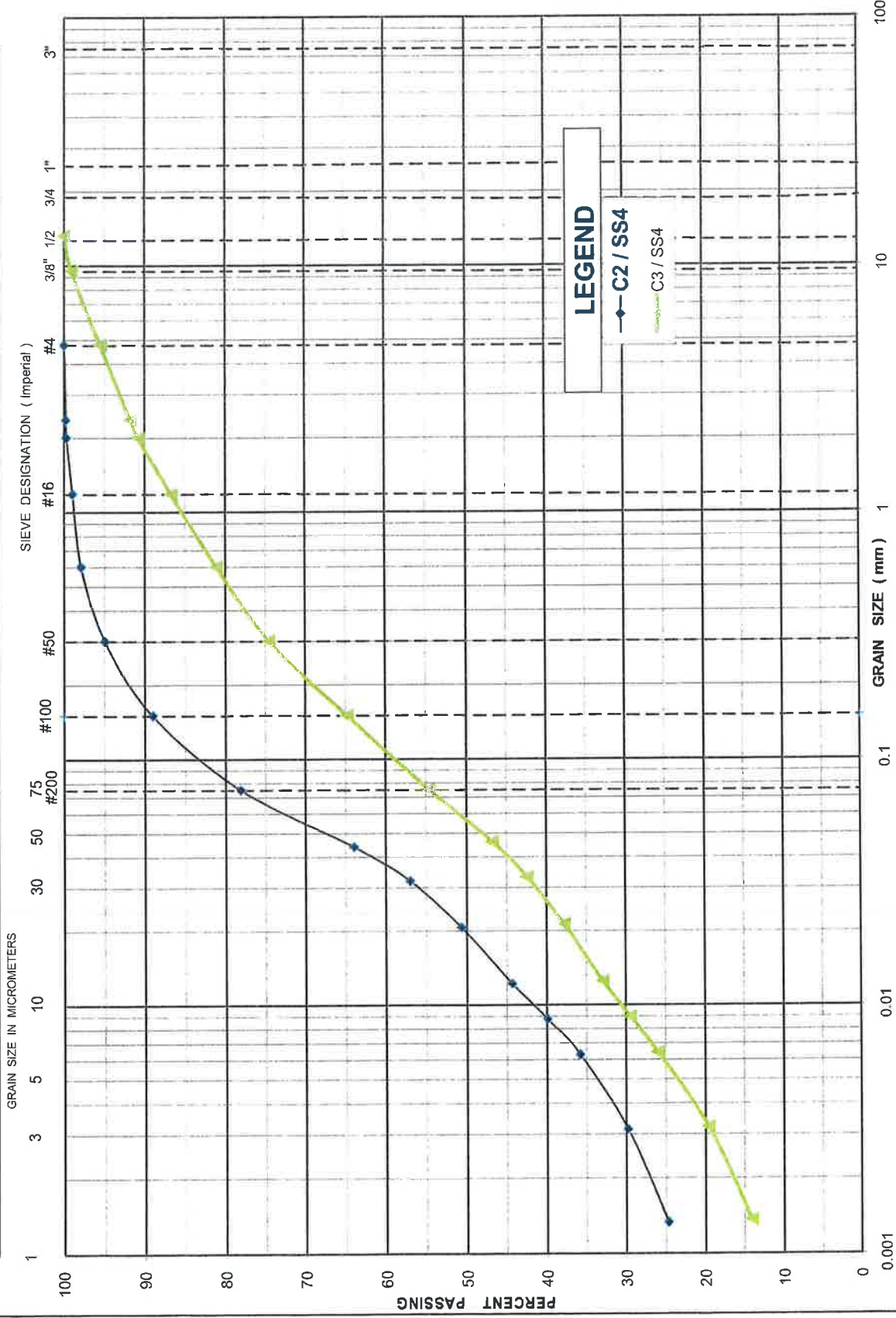
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20					
109.0 0.0	GROUND SURFACE												
	Straight Auger See Record of Borehole C4												
104.4 4.6	SILTY SAND TILL some gravel v. dense, moist		1	SS 100 / 8 cm									Auger grinding @ 4.6 m
			2	SS 100 / 10 cm									
			3	SS 100 / 13 cm									
101.2 7.8	End of Borehole. Water level @ 3.4 m (not stabilized)* upon completion. Borehole caved-in @ 7.4 m upon completion.		4	SS 100 / 15 cm									

Appendix B

Laboratory Test Results

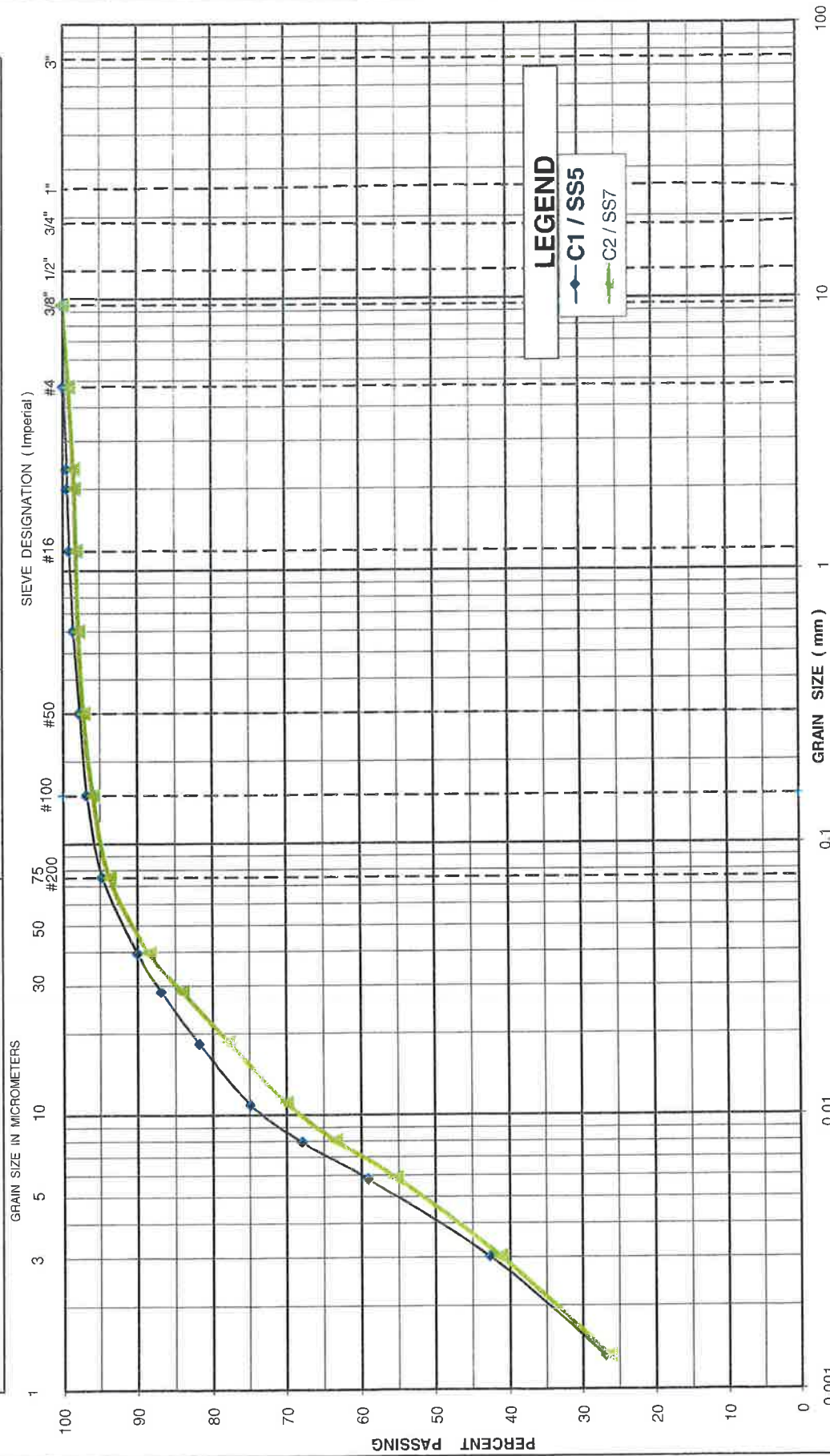
UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT		SAND			GRAVEL		
		Fine	Medium	Coarse	Fine	Coarse	Coarse



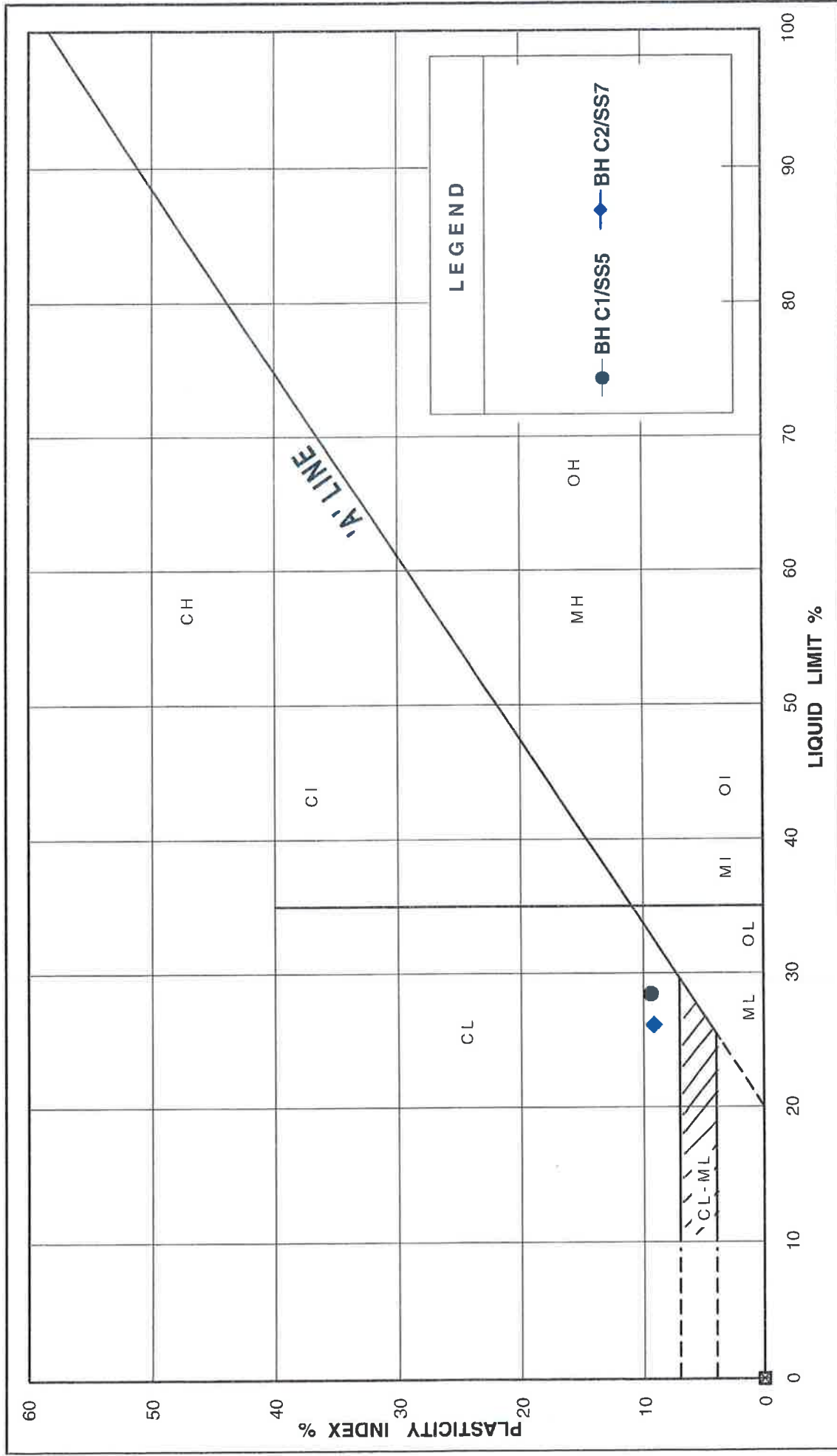
UNIFIED SOIL CLASSIFICATION SYSTEM

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



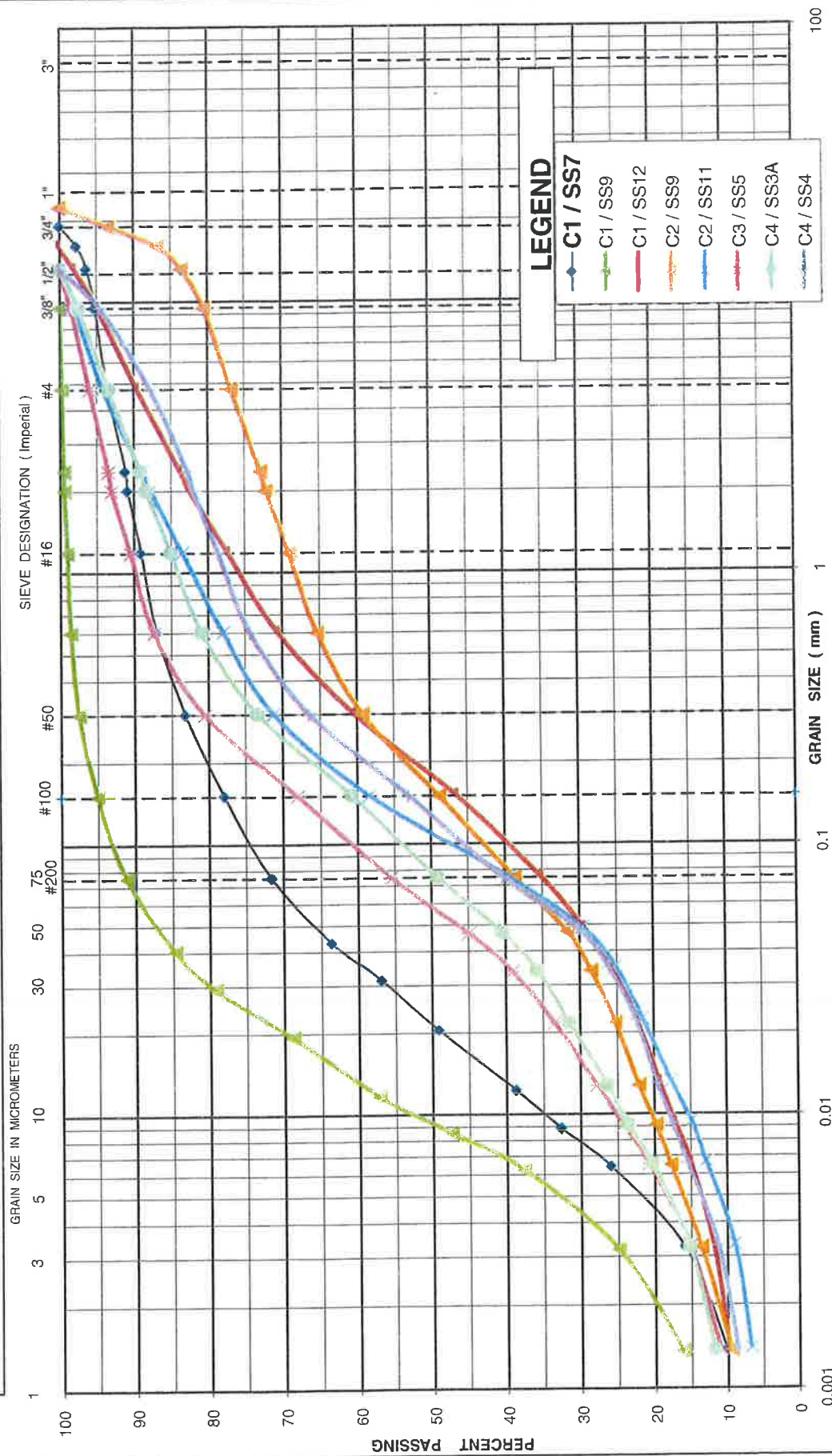
GRAIN SIZE DISTRIBUTION CLAYEY SILT

FIGURE NO.: B2
PROJECT NO: TRANETOB10434AA
DATE: OCT. 2010



UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT		SAND			GRAVEL		
		Fine	Medium	Coarse	Fine	Coarse	



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GRAIN SIZE DISTRIBUTION SILTY SAND TO SANDY SILT TILL

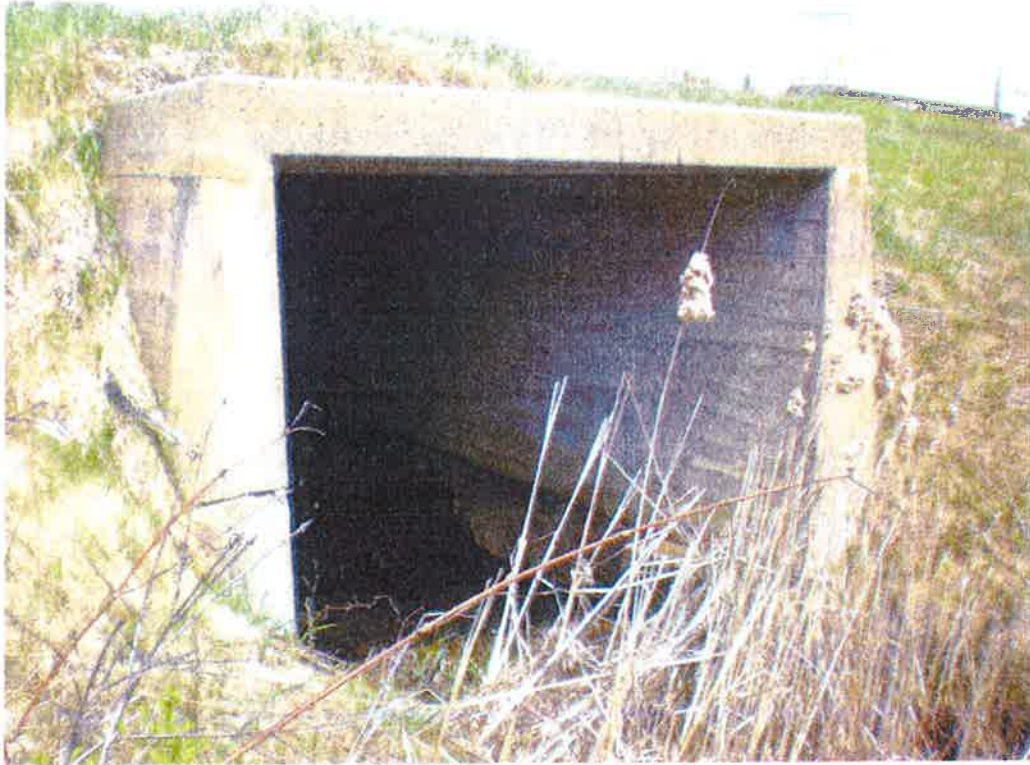
FIGURE NO.: B4

PROJECT NO: TRANETOB10434AA

DATE: OCT. 2010

Appendix C

Site Photographs



Midtown Creek West Station 19+560 EBL South End



Midtown Creek West EBL Station 19+560



Midtown Creek West Station 19+560 EBL South End



Midtown Creek West Station 19+560 Looking South

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 \bar{N} .

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

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

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

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

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

JOINT AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

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

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}		COEFFICIENT OF VOLUME CHANGE
c_c	1		COMPRESSION INDEX
c_s	1		SWELLING INDEX
c_a	1		RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s		COEFFICIENT OF CONSOLIDATION
H	m		DRAINAGE PATH
T_v	1		TIME FACTOR
U	%		DEGREE OF CONSOLIDATION
σ'_{vo}	kPa		EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa		PRECONSOLIDATION PRESSURE
τ_f	kPa		SHEAR STRENGTH
c'	kPa		EFFECTIVE COHESION INTERCEPT
ϕ'	-°		EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa		APPARENT COHESION INTERCEPT
ϕ_u	-°		APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa		RESIDUAL SHEAR STRENGTH
τ_r	kPa		REMOULDED SHEAR STRENGTH
S_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
γ_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
γ_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
γ	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_n	%	SHRINKAGE LIMIT	q	m^3/s	RATE OF DISCHARGE
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $(w_L - w_p) / I_p$	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
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL	I_c	1	CONSISTENCY INDEX = $(w_L - w) / I_p$	k	m/s	HYDRAULIC CONDUCTIVITY
P'	kg/m^3	DENSITY OF SUBMERGED SOIL	e_{max}	1, %	VOID RATIO IN LOOSEST STATE	j	kN/m^2	SEEPAGE FORCE
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL						

**FOUNDATION DESIGN REPORT –
PROPOSED CULVERT EXTENSION AND
PARTIAL REPLACEMENT, MIDTOWN
CREEK WEST AT STATION 19+560,
HIGHWAY 401, COBOURG, ONTARIO
W.P. 205-00-01, GEOCRES NO. 30M16- 45**

AECOM
TRANETOB010434AA-AF
February 6, 2012

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Appendices

Appendix E: List of SPs, OPSSs, OPSDs and NSSP

Appendix F: Limitations of Report

**FOUNDATION DESIGN REPORT
MIDTOWN CREEK WEST CULVERT EXTENSION AND PARTIAL REPLACEMENT
AT STATION 19+560 HIGHWAY 401, COBOURG, ONTARIO
W.P. 205-00-01**

5 DISCUSSION AND RECOMMENDATIONS

As part of the expansion (six laning) of Highway 401, from Burnham Street in Cobourg, Ontario to approximately 2.0 km east of Nagle Road (a total length of 6.5 km), it is proposed to extend the existing non-rigid frame open (NRFO) culvert at Station 19+560.

The existing culvert is a non-rigid frame, open (NRFO) footing, cast-in-place culvert (variable span: 2.12 m to 2.44 m, inlet to outlet ; rise: ~1.83 m and approximately 64 m long, with about 900 mm deep footings). It is our understanding that this culvert is to be extended by 4 m to 5 m on each side to facilitate the embankment widening. At the northern end, i.e. the inlet end, however, the existing culvert may need to be replaced, for structural reasons, by up to about 8 m. Five boreholes (C1, C2, C3, C4 and C4A) were advanced along or in close proximity to the alignment of the existing culvert, as presented in Drawing No.1.

The foundation recommendations discussed in this report are based on the findings of our geotechnical investigation, which included field and laboratory testing, to characterise the subsurface conditions. It will address the culvert foundations and the approach embankment issues i.e. within about 20 m of each side of the culvert as well as general design recommendations. Further, our assessment is based on the following assumptions:

- Marginal or no road elevation change in the vicinity of the culvert.

The new structure extension and the part replacement will be matched to the existing culvert which has an invert elevation of 108.1 m at the inlet and 107.9 m at the outlet.

We have received the cross sections of the embankment at the culvert location (as at December 2010). Based on the cross sections provided the existing side slopes are approximately 2H:1V.

5.1 Geotechnical Characterisation

In summary, the encountered subsurface conditions at the culvert location generally consisted of approximately 0.3 m to 4.6 m thick fill, over natural overburden soils (consisting of clayey silt underlain by silty sand to sandy silt till) within the depths investigated, as presented in the Stratigraphic Section in Drawing No. 1.

The fill intercepted in Borehole C2 comprises surficial pavement fill, underlain by a silty sand to sandy silt embankment fill to a depth of 4.6 m. Beneath a veneer of topsoil in Boreholes C1, C3 and C4, embankment fill was also encountered. In Borehole C1 it is of silty sand nature whilst in the other two boreholes, the embankment fill is a silty sand to sandy silt material. The embankment fill, with a thickness ranging from 0.3 m to 4.1 m, was noted with a trace to some clay and gravel,

Broadly, based on the recorded N-values, this silty sand to sandy silt embankment fill is generally loose to compact.

Underlying the embankment fill, a firm to very stiff clayey silt was intercepted in all the boreholes except in Borehole C3, with a thickness varying from 0.7 m to 1.5 m. The basal layer, generally underlying the clayey silt, is a silty sand till, in which drilling was terminated. Based on the recorded N-values, this glacial deposit has typically a dense to generally very dense relative density. The till contains cobbles and boulders.

Groundwater levels were observed in the open boreholes while drilling and upon completion of each borehole. In addition, piezometers were installed in Boreholes C1 and C3 to monitor the groundwater levels. The tips of piezometers were installed within the silty sand till. In these two boreholes, the highest water levels recorded in the piezometers were at Elevations 109.4 m and 109.0 m respectively. Therefore the groundwater levels would suggest the presence of an artesian head emanating from the silty sand till. The highest measured water levels in the piezometers installed in the boreholes indicate a hydrostatic head of about 1.1 m above the existing o.g. (original ground) elevations.

Quaternary Geology Map of Ontario Southern Sheet depicts the Oak Ridge Moraine deposit north of Cobourg. This deposit is a major source of groundwater and is an extensive deposit about 160 km long and 5 to 20 km wide, generally in a east-west direction, across Southern Ontario and its south-east tip extends into the Cobourg area. The major hydrogeological axis of groundwater flow is in a southerly direction towards the Lake (as the ground elevation drops towards the Lake). Artesian groundwater conditions south of the Oak Ridge Moraine deposit are generally observed when water conducting bodies such as the silty sand till, which are likely fed by the moraine deposit or other water bearing bodies, are overlain by relatively impervious cohesive strata, such as the clayey silt to silty clay stratum as found at this culvert location. Artesian groundwater conditions are likely to manifest as a result of such flows, when a water bearing layer which usually has some spatial connectivity to a natural or artificial source of water which controls the water pressure in the layer, is suitably confined, especially at low elevations such as in the vicinity of creeks.

It should be pointed out that the groundwater would be subject to seasonal fluctuations and fluctuations in response to major weather events. The water table at the Site would also be influenced by the water level in the watercourse. A perched water table may also occur due to the accumulation of surface water in the surficial fills overlying the low permeable, clayey silt.

Significant scour erosion has been reported at the existing culvert. See site photographs in Appendix C.

5.2 Culvert Foundations

We understand that the proposed extensions and the inlet end replacement (if required) are planned to match the existing culvert which consists of a non-rigid frame, open (NRFO) structure on strip footing foundations which are about 0.9 m deep. In general, the replacement/extension of the existing NRFO (open bottom concrete) culvert can be carried out with a matching NRFO or a concrete box culvert or a CSP. However, given the need for spread footings for open bottom culverts, which in turn would require deeper excavations below the groundwater table, as well as some extensive shoring, the option of an open bottom culvert is not a recommended option, from a geotechnical point of view.

With the prevailing Site conditions, from a geotechnical viewpoint, the use of a flexible pipe such as a CSP (corrugated steel pipe) was considered for the project. However, MTO may be reluctant to use a CSP culvert under a major highway, such as Highway 401. If this is the case then the use of a precast concrete box culvert is the recommended option. It is our opinion however the latter should be installed in short sections (e.g. 2.4 m) for flexibility purposes. A precast concrete structure is preferred to a cast-in-place concrete structure in view of the former having better durability, ease of installation, faster implementation and especially greater flexibility. We understand, however, that AECOM's preferred option is for a rigid frame open footing culvert due to concerns that it would be difficult to maintain temporary flow passage through the existing culvert whilst installing box sections.

Based on the borehole data and the invert levels of the existing culvert, the founding layer of the existing culvert is considered to be the dense to very dense silty sand till within the majority length of the culvert except perhaps towards the ends where a veneer of clayey silt may overlie the glacial till.

As mentioned before, the existing embankment side slopes at the culvert location are presently standing at approximately 2H:1V, based on the cross sections provided to us by AECOM. In our opinion, matching the existing slopes (i.e. 2H:1V) is feasible from a geotechnical point of view, as well as from practicability. In our analysis therefore, we have assumed 2H:1V side slopes. However flatter side slopes would also be acceptable, if desired.

5.2.1 Frost Protection

Design frost protection depth for the Site is about 1.5 m. A minimum 1.5 m thick permanent soil cover or equivalent thermal insulation is required for frost protection of foundations. In case of rip-rap (rock fill), only one-half of the rock fill thickness should be assumed to be effective in providing frost protection.

5.2.2 Rigid Frame Open Footing Culvert on Strip Foundations Option

Proposed Founding Levels: As stated earlier, the existing culvert is a NRFO footing culvert. If a rigid frame open footing culvert must be used, then strip footing foundations for the proposed works should extend below the scour/frost depths. The elevations of the underside of the strip footings of the existing open footing culvert are not known. Assuming a footing depth of 1.4 m for the strip footings of the proposed open bottom culvert extensions/replacement, founded on 100 mm thick lean mix concrete mud mat, the required minimum frost protection cover could be achieved. However, at the immediate interface the difference in elevation of the bottom of the existing footings should not be more than 0.1 m in order to prevent undermining of the existing footings and/or imposing additional stresses.

The requirement for the frost protection depth may however not necessarily be sufficient for scour purposes.

As was mentioned before, the site is characterised by a high groundwater table and an artesian condition emanating from the silty sand to sandy silt till deposit. It is important to ensure that no hydraulic blow-out occurs under the artesian head in the silty sand to sandy silt till underlying the clayey silt, with the progress of excavation. Based on the ground conditions encountered in Boreholes C1 and C4, the frost protection depth requirement of 1.5 m, would place the underside of the strip footings for both the culvert extensions and replacement, generally within the till deposit.

For example, the highest groundwater level measured in Borehole C1 was at Elevation 109.4 m, i.e. about 1.1 m above the existing original ground surface (o.g.) or 2.6 m above the top of the till deposit (El.

106.8 m). This would generate hydraulic gradients in excess of unity within the top 2.6 m of the till deposit. Such excessive hydraulic gradients are known to have the potential for significant piping within the upper till depths and consequent loosening of the bearing subgrade. Therefore, in order to ensure stability against piping and also to provide sufficiently dry excavations, depressurising of the artesian head is required. For this purpose, the dewatering must be capable of reducing the hydraulic head in the silty sand till stratum underlying the clayey silt deposit to at least 1.0 m below the intended excavation elevation and should be maintained during the culvert installation.

For the culvert replacement commencing at the existing inlet end, depending on the extent of replacement, a minimum 1.5 m frost protection depth should be provided. However, the new strip footings for the replacement, at the immediate interface with the existing, should not be founded more than 0.1 m above or below the level of the adjacent (existing) strip footings to minimise the influence on the existing footings and/or to prevent undermining of the existing footings.

With the 1.5 m frost protection criterion, the founding level for the culvert extension at the inlet end would be at Elevation 106.6 m (assuming the existing invert Elevation is 108.1 m), and the founding level for the culvert extension at the outlet end would be at Elevation 106.4 m (assuming the existing outlet Elevation is 107.9 m), which would satisfy the minimum 1.5 m permanent soil cover for frost depth.

Estimate of Applied Stress Increment: Based on soil-structure interaction considerations, the estimated increment of applied maximum vertical stress (due to the culvert extension and associated embankment widening) on top of the proposed culvert extensions is about 55 kPa. This applied stress increment estimate is based on a consideration of the height of the maximum soil column over the culvert extension and soil arching effects (which manifests when two dissimilar bodies, i.e. a culvert – high stiffness concrete and earthfill – of relatively low stiffness, interact) as stipulated in Table 7.13 of the Canadian Highway Bridge Design Code (S6-06). Further, this stress increment estimate assumes that the existing side slopes are at 2H:1V configuration and the proposed 4 m to 5 m widening will follow the same side slope configuration. This will cause a settlement of the existing culvert in the vicinity of the interface with the proposed culvert, as well as transferring stresses onto the new extension.

There will not be any additional stress increment at the interface between the culvert replacement section and the adjoining existing culvert section, as no grade raise is anticipated at the paved portion of the existing highway.

Assessment of Bearing Resistances: The following geotechnical resistances can be used for the silty sand to sandy silt till founding stratum, in its undisturbed state, to facilitate the structural design of the culvert extensions/replacement.

Factored Geotechnical Resistance at ULS = 300 kPa.

Geotechnical Resistance at SLS = 200 kPa.

Higher bearing resistances are available but unlikely to be required.

These resistances are valid for footings placed at El. 106.6 m or below at the inlet end and at El. 107.7 m or below at the outlet end (applicable for the till deposit). The actual footing depth elevations within the till deposit, however, are subject to meeting the frost and scour depth requirements. A minimum footing width of 0.6 m is required.

The recommended geotechnical reaction at SLS is for a standalone construction without any structural interaction effects to be accommodated. In the present case, the interaction between the existing culvert and the extension would impose a lower geotechnical reaction at SLS. This is not considered an issue, as the applied maximum vertical stress increment on top of the proposed culvert extensions is about 55 kPa. This order of applied stress increment is not expected to cause adverse structural issues from resulting settlements, as discussed next.

Estimate of Settlements: Based on soil-structure interaction considerations, the vertical stress, induced by the embankment widening on the culvert extensions is estimated to cause total settlements not exceeding 10 mm and differential settlements not exceeding 5 mm, provided that the bearing subgrade is not disturbed during construction. As will be discussed later in this report, good construction techniques, including dewatering, will be required to achieve this. An allowance should be made to accommodate a differential settlement of about 10 mm between the existing and the new culvert.

The stresses due to embankment widening can also be expected to cause a settlement under the widened portion of the embankment of the existing culvert. This settlement is not expected to be more than 10 mm. This may need to be taken into consideration in assessing the integrity of the existing culvert.

If, as discussed earlier, part of the existing culvert is to be replaced, settlements of lesser magnitude can be expected to occur, provided that the founding subgrade is undisturbed during the construction.

Assessment of Sliding Resistance: The unfactored horizontal resistance against sliding of the open bottom culvert footings on the underlying till stratum can be calculated using an interface friction angle of 29°. Sliding, however, is unlikely to pose a problem since the major horizontal thrust on the culvert walls is along the road axis which is more or less counterbalanced from either side.

Construction Considerations: It should be noted that foundation bearing soils near the water table are susceptible to disturbance from construction activity. Care should be taken during the excavation and construction of the footings to minimize disturbance of the bearing soils. Stabilization of wet subgrades should be anticipated. Disturbance of the underlying soils during construction of the structure, in proximity to the groundwater table, could influence the settlement of the structure. In this regard, the placing of a 100 mm thick layer of lean concrete (mud mat), as soon as possible (within 3 hours), on the foundation bearing surface supporting the open bottom concrete culvert structure, after the excavation and the approval of the subgrade by a QVE, is recommended as per OPSS 902.

Care should be exercised during construction of the foundations adjacent to the existing culvert to avoid possible undermining and/or influence on the existing structure. Proper dewatering will be required for this purpose. As mentioned before, the dewatering system should be capable of reducing the hydrostatic head in the till deposit to at least 1.0 m below the intended excavation levels.

Cobbles and boulders within the till may be encountered during construction. Where oversized materials are encountered to protrude from the exposed subgrade, these materials should be removed and replaced with mass concrete.

A Geotechnical Engineer who is familiar with the findings of this investigation should evaluate all bearing surfaces prior to placement of reinforcement and concrete to confirm that the founding conditions are consistent with the recommendations given in this report. All organic, very loose/soft/firm or otherwise unsuitable soils should be removed prior to pouring the concrete.

Further construction considerations including dewatering and surface drainage issues are discussed in Section 5.6.

5.2.3 Box Culvert Option

A precast concrete box culvert is preferred to a cast-in-place concrete structure in view of the former having better durability, ease of installation, faster implementation and especially greater flexibility. It is our opinion however the latter should be installed in short sections (e.g. 2.4 m) for flexibility purposes.

Proposed Founding Levels: For a box culvert type of extension/replacement, it is recommended that the excavation of the upper portion of the founding stratum within the extended/replaced culvert lengths should be carried out to allow for the bedding material. In this instance, the recommended maximum bedding thickness is 300 mm, but may need to be thicker depending on the OPSD requirements. The founding level for the culvert extension at the inlet end, assuming the existing invert El. is 108.1 m, a base concrete thickness of 0.4 m and a bedding layer thickness of 0.3 m, is El. 107.4 m or below. Based on the ground conditions encountered in Borehole C1 (near the proposed inlet end), the underlying clayey silt thickness, below the closed bottom culvert base, is around 0.6 m. Based on these conditions, the Factor of Safety (FOS) against blow-out is less than 1, which would imply blow-out of the overlying thin clayey layer. The minimum FOS required against hydraulic blow-out as per the Canadian Foundation Engineering Manual (CFEM) is about 1.2. Therefore, in order to ensure stability against blow-out and also to provide sufficiently dry excavations, depressurising of the artesian head is required. It must be capable of reducing the hydraulic head on the silty sand till stratum underlying the clayey stratum to at least 1.0 m below the intended excavation elevation and should be maintained during culvert installation.

For the culvert extension at the southern end, i.e. existing outlet end, for ease of trafficability, the remaining (if any) thin sliver of firm clayey silt may soften under construction disturbance below the proposed culvert bedding. If this happens, the thin veneer of softened clayey silt may need to be replaced with granular bedding material extending to the surface of the till deposit at El. 107.4 m or below. Due to the artesian head in the till deposit, the founding stratum would be subject to piping issues and consequent loosening of the bearing subgrade and similar depressurising of the artesian head, as at the inlet end, is required.

At the interface, the invert level of the box culvert extension/replacement should match that of the existing open footing culvert.

Estimate of Applied Stress Increment: Based on soil-structure interaction considerations, the estimated increment of applied vertical stress (due to the culvert extension and associated embankment widening) on top of the proposed culvert extensions is about 55 kPa. This estimate assumes that the existing side slopes are at 2H:1V configuration and the proposed 4 m to 5 m widening will follow the same side slope configuration.

Assessment of Bearing Resistance: The following geotechnical resistances would be available for a closed bottom concrete culvert design at Elevation 107.4 m or below at the existing inlet end, and Elevation 107.0 m or below at the existing outlet end, provided that the founding subgrade is undisturbed during construction:

Factored Geotechnical Resistance at ULS = 250 kPa;

Geotechnical Resistance at SLS = 150 kPa;

Higher resistances would be available but are unlikely to be required.

It is to be noted that the bearing resistances for the open bottom culvert were higher as the founding level of the strip footings of the open bottom culvert are within the more competent, i.e. dense to very dense, silty sand till than is the situation with the box culvert. As well, the bottom slab for the box culvert is wider than the strip footing foundations and as such unit bearing pressure is lower but the zone of influence is greater (i.e. deeper), compared with strip footings.

The above geotechnical resistances for the cohesive founding stratum, in its undisturbed state. The underlying silty sand till is more competent for bearing resistance purposes, in comparison with the clayey silt stratum.

The recommended geotechnical resistance at SLS is for a standalone construction without any structural interaction effects to be accommodated. In the present case, the interaction between the existing culvert and the extension would impose a lower geotechnical resistance at SLS. This is not considered an issue, as discussed earlier, as the imposed maximum vertical stress on top of the culvert extensions, is estimated to be about 55 kPa. This order of applied stress increment is not expected to cause adverse structural issues from resulting settlements, as discussed next.

Estimate of Settlements: Based on the estimated applied vertical stress increment and provided that the bearing subgrade is not unduly disturbed during the construction, under the estimated applied stresses the total and differential settlements should not exceed 12 mm and 18 mm, respectively. As will be discussed later in this report, good construction techniques, including dewatering, will be required to achieve this.

An allowance should be made to accommodate a differential settlement of about 12 mm between the existing and the new culvert. If, as discussed earlier, part of the existing culvert is to be replaced, settlements of lesser magnitude can be expected to occur (provided that the founding subgrade is undisturbed during the construction), since there will be no grade raise above that portion of the existing embankment.

Assessment of Sliding Resistance: The unfactored horizontal resistance against sliding of the open bottom culvert footings on the clayey silt stratum can be calculated using an interface friction angle of 24° and for sliding considerations on the till layer, an interface friction angle of 28° is appropriate. Sliding, however, is unlikely to pose a problem since the major horizontal thrust on the culvert walls is along the road axis which is more or less counterbalanced from either side.

Construction Considerations: Care should be exercised during construction of foundations adjacent to the existing culvert to avoid possible undermining and/or influence on the existing structure.

The transportation and placement of the precast concrete box culvert segments will need to proceed with caution such that the weight of the adjacent embankment and that of the construction equipment including the loaded crane will not cause disturbance and/or failure of the newly constructed bedding and/or the underlying subgrade. The suitability of the existing embankment to carry the loaded crane will need to be determined. This is the contractor's responsibility. The crane should, however, not operate on the subgrade prepared to receive the precast segments.

Further construction considerations including dewatering and surface drainage issues are discussed in Section 5.6.

5.2.4 CSP Type Culvert

As was mentioned before, because of its flexibility, a CSP type culvert would be a preferable option for this project. This type of culvert can be placed on a minimum 300 mm thick granular bedding, but the thickness may need to be increased depending on the size of the pipe, as per OPSD requirements. This type of pipe will need good side support for proper performance (especially an elliptical pipe). This may be difficult to provide, especially if a portion of the existing pipe is to be replaced, in the narrow confines of a shored excavation. Further, given the observed scour erosion at the existing culvert, good side support can be compromised. As well, MTO will unlikely allow the use of a CSP type culvert under Highway 401.

5.2.5 Recommended Culvert Option

A CSP type of culvert is not particularly suited for the prevailing subsurface conditions as given the observed scour erosion at the existing culvert, good side support can be compromised. Also, MTO will unlikely allow the use of a CSP type culvert under Highway 401 and as well it does not match the configuration of the existing culvert.

A rigid frame open bottom culvert can also be considered but due to the fact that more extensive dewatering and de-pressurization are required, it is considered less suitable from a geotechnical engineering point of view.

A box culvert, especially a precast concrete box culvert, is also suitable for the existing site conditions. Some dewatering and de-pressurization will be required (as discussed in Section 5.6) but not nearly to the same extent as an open bottom culvert supported on strip footings. Therefore from a foundation perspective, this is the recommended option. Therefore from a foundation perspective, this is the recommended option. In summary from a geotechnical engineering point of view, a box culvert is the recommended option for the project.

5.3 Embankment Widening

Embankment widening slopes, as proposed by AECOM, are at 2H:1V as shown on Drawing 1, since the existing embankment will be widened with side slopes similar to the existing at 2H:1V, in order to accommodate the six laning. The crest of the widened embankment will match that of the existing embankment. The above are recommended for adoption for construction.

In the general vicinity of the existing culvert, as discussed above, the existing embankment is up to about 4.6 m high applying a maximum vertical stress of the order of 90 kPa on the foundation soils. The maximum additional vertical stress that will be imposed on the founding layer due to the embankment widening is estimated about 50 kPa. From the borehole data at this Site and from knowledge of the cohesive stratum along this alignment in other areas, a conservative estimate of the undrained strength of this cohesive layer is about 50 kPa (reflects the lower strength of the cohesive layer in the vicinity of the proposed outlet). For stability purposes, the worst scenario is that the embankment widening is overlying the cohesive layer to some depth and not on the dense to very dense till. Even under such a scenario, the stability of the approach widening is not considered an issue given that only a marginal increase in the grade level is expected. However, this necessitates that all organic, weak or otherwise unsuitable materials will be removed as per MTO standards prior to placing the embankment fills.

The resulting maximum foundation settlement under the widened approaches is estimated to be less than 12 mm, including the settlement of the existing embankment under the widened portion. We do not

envisage this order of settlement under the widening to have an adverse impact on the existing traffic lanes, but the paving should be delayed for about four weeks after end of construction to effect majority of the settlements before paving. In other words, we recommend that asphalt paving be implemented no earlier than four weeks after the grade reaches the bottom of asphalt elevation. If necessary, an operational constraint should be included to implement this aspect.

All organic and other unsuitable materials should be removed within an envelope and given by an imaginary slope no steeper than 1:1 from the toe of the proposed embankment. After stripping, the exposed subgrade should be inspected, approved and properly rolled from the surface, using a suitably heavy compactor. The existing Site conditions (e.g. high water table) could influence the choice of compaction equipment. Dewatering and surface drainage measures mentioned earlier and more fully discussed in Section 5.6 should facilitate the achievement of proper compaction under wet conditions and the first lift of the fill may need to consist of free-draining granular materials.

Proper benching of the embankment slope should be implemented during widening of the embankment, as per MTO procedures and in accordance with OPSD 208.010.

The materials used for the construction of the embankment fills should consist of approved, acceptable earth fill. Fill used for the construction of the widening should be in accordance with OPSS 212 and fill placement should meet or exceed the requirements of OPSS 501 and OPSS 206. In general, the fills should be placed in lifts not exceeding 300 mm before compaction and each fill should be uniformly compacted to at least 95% of the material's Standard Proctor Maximum Dry Density. In as much as possible, the fill used should match the existing embankment fill, within the frost zone.

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

5.4 Bedding

For precast box culverts, bedding should be in accordance with OPSD803.010 but the bedding should have a minimum thickness of 300 mm. For CSP type culverts, the bedding thickness should be at least 300 mm but may need to be increased as per OPSD, depending on the pipe diameter. Any requirement for thicker bedding due to prevailing subgrade conditions at the time of construction should be managed through stabilising the subgrade by lowering the artesian head and control of surface water flow. This is necessitated in view of the potential for blow out/piping under the artesian head as discussed earlier in the report. Such issues, at the time of construction, should be decided by the Geotechnical Engineer retained as part of the QVE.

It should be noted that at the interface between the existing and the extension culvert, the excavation for the placement of bedding should be constructed so as not to undermine the existing structure.

The bedding should consist of a well-graded granular material such as a Granular 'A' or a Granular 'B' Type II (OPSS 1010).

The bedding material should be placed as soon as practicable after the preparation of the subgrade, its inspection and approval, as was discussed in the previous sections of this report. The bedding material should be in accordance with the appropriate standards for culverts (e.g. for box culverts, OPSD

803.010). The bedding material should be compacted to MTO standards (OPSS 501 or SP 105S10) whichever is applicable).

5.5 Backfilling

The bedding and embedment material should be extended along the sides and the top to cover the culvert. The selection and placing of the backfill should be in accordance with OPSS-803.010 for concrete culverts. The backfill should consist of free-draining, non-frost susceptible granular materials such as Granular 'A' or 'B' (OPSS-1010). All granular backfill materials should be placed in thin lifts (i.e. not exceeding 300 mm before compaction) and each lift should be compacted to at least 95% of the material's SPMDD (Standard Proctor Maximum Dry Density). Although this is not an MTO requirement, we recommend that, if feasible, the degree of compaction of the fill materials within 1.5 m of pavement subgrade be increased to not less than 98% of the material's SPMDD. In our experience, we found that this enhances the performance of the road by providing superior support to the overlying pavement. The Granular 'A' base and Granular 'B' sub-base courses (OPSS 1010) should be compacted to not less than 100% of the material's SPMDD.

The use of proper backfill material and especially good compaction are necessary for proper side support and successful performance of the culvert. For the same reason, the organic soils or other unsuitable materials should be removed within a distance of at least 0.5 m beyond the footprint of the culvert. The use of heavy compaction equipment should be avoided immediately adjacent and above the culvert, as per MTO practice. During backfill placement, the height of the backfill should be maintained at approximately the same level on both sides of the structure, to avoid lateral displacement (dislodging) and/or damage of the structure.

For fills immediately below the roadway, we recommend that Granular 'A' or 'B' aggregates (OPSS 1010) be used. Where necessary, proper tapering as per standards should be provided. Below a depth of about 1.5 m to 2.0 m from the finished road grade, an approved compactable fill, such as select subgrade materials (SSM) can be used.

Proper frost treatment is required in accordance with OPSS-803.030 or 803.031, whichever is applicable.

Backfilling behind retaining (wing) walls, if any, should consist of granular materials in accordance with the MTO standards. Free draining backfill materials, weepholes, etc. should be provided in order to prevent hydrostatic pressure build-up.

Computation of earth pressures acting against rigid culvert walls and any wing walls should be in accordance with the Canadian Highway Bridge Design Code, S6-06:(CHBDC) 2006. For design purposes, the following properties can be assumed for backfill.

Compacted Granular 'A' or Granular 'B' Type II

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

Unit weight = 22 kN/m³

Coefficient of Lateral Earth Pressure:

Level Backfill	Backfill Sloping at 3H:1V	Backfill Sloping at 2H:1V
Ka=0.27	Ka=0.34	Ka=0.40
Ko=0.43	Ko=0.56	Ko=0.62

Compacted Granular 'B' Type I

Angle of Internal Friction $\phi=30^\circ$ (unfactored)

Unit Weight = 21 kN/m³

Coefficient of Lateral Earth Pressure:

Level Backfill	Backfill Sloping at 3H:1V	Backfill Sloping at 2H:1V
Ka=0.33	Ka=0.42	Ka=0.54
Ko=0.50	Ko=0.66	Ko=0.76

Note: Ka is the coefficient of active earth pressure

Ko is the coefficient of earth pressure at rest

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

The earth pressure coefficient adopted will depend on whether the retaining structure is restrained or some movement can occur such that the active state of earth pressure can develop. Allowance should be made for compaction induced stresses in the selection of the appropriate earth pressure coefficients, and reference should be made to Clause 6.9.3 of CHBDC (S6-06). The use of vibratory compaction equipment behind the culvert and the retaining walls should be restricted in size as per current MTO practice.

A culvert is akin to a single span bridge and CHBDC exempts single span bridges of the need to undertake earthquake dynamic analyses and hence dynamic earth pressures are not required to be considered for culvert designs for earthquake analyses.

5.6 Construction Considerations

It is anticipated that the excavations for the construction comprise excavating the existing embankments and stripping unsuitable soils from beneath the embankment widening areas. Relatively deep excavations can be anticipated to extend the excavation to founding levels in the case of an open bottom concrete culvert structure, which is however not our recommended option. Excavations should be

possible using heavy equipment such as a hydraulic excavator. Boulders and cobbles can be expected within the embankment fill, and till layers if intercepted.

The settlement due to reducing the hydraulic head on the silty sand till stratum underlying the clayey silt deposit to at least 1.0 m below the intended excavation elevation is estimated to be not more than 5 mm (for an open footing culvert) and this settlement is expected to taper off within a radius of influence less than 50 m. This is not considered an issue. When groundwater recharge takes place after the short period of construction of the culvert extensions (say ~ 4 to 8 weeks) part of this settlement would recover due to the local ground water regime returning to its prior equilibrium levels on cessation of pumping.

Settlements of this magnitude are unlikely to cause any cracking in flexible (i.e. asphaltic concrete) pavements. However, combined with settlements due to grade raise on the extension, there may be some effect. For this reason, it is recommended that paving be implemented in as delayed a fashion as possible (for about four weeks).

Surface drainage measures are also required to ensure sufficiently dry conditions during the proposed works. This will likely require deep wells possibly in conjunction with well points. When designing the dewatering system, the presence of boulders should be taken into consideration. It is also likely to consist of gravity drainage in shallow perimeter ditches and pumping from strategically placed deep filtered sumps. It should also be kept in mind that more aggressive dewatering will be required for an open bottom excavation (as deeper excavations will be required) in comparison with a closed bottom culvert.

In addition to lowering the artesian heads and provide sufficiently dry conditions during construction, it will be necessary to divert the water flowing in the watercourse. This could consist of the construction of a temporary cofferdam, such as pre-cast concrete barrier (e.g. jersey barrier), low permeability soil cofferdam barrier, sand bags, etc. to divert the water away from the culvert extension, or the construction of a temporary culvert, although the latter would be impractical and not cost-effective. In the case of diversion, consideration should be given to temporary diversion and storage of the surface water and promptly pumping the water downstream of the new construction area, into the existing watercourse channel near the outlet. In this regard, measures would need to be taken within the existing culvert to prevent back flow of water into the construction area and installation of suitable sedimentation control.

For these reasons, we recommend that, in order to reduce the severity of dewatering, the construction be carried out during a dry period, if possible. It is, however, normally up to the Contractor to come up with a plan to achieve a suitable diversion and dewatering.

We recommend that the requirement for dewatering be 'red-flagged' to the Contractor and that the Contractor be asked to submit their diversion and dewatering method to the CA for information purposes, prior to construction. We also recommend that NSSP be included, alerting the Contractor (including the dewatering contractor) of the subsurface and groundwater conditions which may cause the disturbance of the new and the existing foundations during construction as well as the need to avert undermining of the existing footing.

For the southern extension, we recommend that the excavated temporary side slopes at 2H:1V but if necessary no steeper than 1.5H:1V for cuts within the existing embankment fill. A minimum of 1.8 m clearance should be maintained between the moving traffic and the edge of the slopes for 2H:1V side slopes and 2.5 m for 1.5H:1V side slopes, with appropriate speed limits. We also recommend that these

slopes be visually monitored for any movement especially if workers are present at the toe of the slopes. These temporary slopes should only be utilized for a short duration. Side slopes of 2H:1V are considered stable both for the short and the long term, and are recommended.

For the northern end culvert extension, and replacement of up to 8 m length, we recommend that, as for the southern end, the temporary side slopes be maintained at 2H:1V but if necessary no steeper than 1.5H:1V for cuts within the existing embankment fill. However, this will likely cause lane closures since a minimum 1.8 m to 2.5 m clearance should be maintained between the moving traffic and the edge of the slopes, as before. If however, vertical excavation support using shoring is used, lane closure(s) could possibly be averted. The dense to very dense nature of the underlying till with the possible occurrence of boulders should be taken into consideration when contemplating the installation of shoring. However, this is the responsibility of the Contractor. If construction sequencing is programmed for the southern widening to be implemented first, then with possible traffic routing, the northern culvert replacement and extension may be undertaken with temporary side slopes without disruption to traffic. Proper dewatering measures would be critical for these operations.

Foundation bearing soils near the water table and in wet weather are susceptible to disturbance from construction activity. Care should be taken during excavation and construction to minimize disturbance of the bearing soil. Stabilization of wet subgrades should be anticipated. Disturbance of the underlying soils during construction of the structures, in the proximity to the groundwater table, could influence the future settlements of the proposed structures.

Stockpiles should be placed well away from the edge of the excavation and their height should be controlled so they do not surcharge the sides of the excavation. Surface drainage should be controlled to prevent flow of surface water into the excavations.

Excavation safety, stability of temporary construction slopes and lateral support systems are the Contractor's responsibility.

Discussions regarding groundwater issues during excavation were provided in the previous sections.

All excavations must be carried out in accordance with the Safety Regulation of the Province (i.e. Occupational Health and Safety Act (OHSA) O. Reg. 213/91), as well as the following specifications:

OPSS 539 – Construction Specification for Temporary Protection Systems;

OPSS 902 – Construction Specification for Excavating and Backfilling - Structure.

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

Embankment Fill	Type 3 soil above water level;
Clayey Silt	Type 3 soil;
Sandy Silt to Silty Sand Till	Type 3 soil above water level; Type 4 soil below water level.

The excavated soils, free from topsoil and organics can be used as general construction backfill where they can be compacted using suitable compactors, provided as-placed fill soils have suitable water contents and that the underlying soils are reasonably firm (i.e. provide suitable support). Loose lifts of soil, which are to be compacted, should not exceed 300 mm. During construction, on-Site verification of

the excavated fill or natural soils for re-use as backfill by suitably qualified personnel would be required. In addition, during wet periods, the embankment fill and the silty sand till may be unsuitable for reuse. Selective stockpiling and double handling may be required for reuse of these materials.

The on-Site excavated soils are not considered to be free draining. Where free draining backfill is required, imported granular fill such as Granular 'B' (OPSS 1010) should be used.

Note that the excavated soils are subject to moisture content increase during wet weather which would make these materials too wet for adequate compaction. Stockpiles should therefore be compacted at the surface or be covered with tarpaulins to help minimize moisture ingress.

Where excavation support is necessary, the shoring should be designed so that the lateral movement of any portion of the 'roadway 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 5.6.1 Recommended Unfactored Parameters for Temporary Shoring Design

Soil Type	Ka	Ko	Kp	γ (kN/m ³)
Pavement Fill (Gravelly Sand).	0.28	0.44	3.5	21.5
Embankment Fill (Silty Sand to Sandy Silt).	0.33	0.50	3.0	19.0
Topsoil.	0.49	0.70	1.4	15.0
Clayey Silt.	0.40	0.55	2.4	18.0
Silty Sand to Sandy Silt Till (compact).	0.32	0.49	3.1	20.5
Silty Sand to Sandy Silt Till (dense to v. Dense).	0.29	0.45	3.4	22.0

In Ontario, temporary shoring systems typically consist of soldier piles and timber lagging or steel sheet piles. In this case, the use of soldier piles and lagging is considered more suitable than interlocking steel sheet piling. As was mentioned before, the presence of cobbles and boulders in the till should be taken into consideration. As well, the recorded artesian pressures should also be considered (especially for the steel sheet pile case). The consequences of any induced vibrations on the existing structure and the embankment, as well as the pavement, should be taken into consideration.

5.7 Erosion Protection

According to condition assessments of the culvert carried out recently by AECOM and previously by others, erosion of the creek bed had been noted along the culvert structure, more pronounced at the northern end i.e. inlet end, exposing the footings as presented in the photographs in Appendix C.

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 hydraulic energy, i.e. velocity of water in the watercourse and its

regime and the erodible nature of stream bed material) who is familiar with the findings of this report. The existing conditions at the culvert Site should be examined for the selection of appropriate scour and erosion protection schemes.

To minimise under-seepage beneath and around the culvert structure and its surrounds, the hydraulic gradient of the flow path should be reduced. This could be achieved through the use of a cut-off wall/apron guided by the existing conditions, soil erodability and watercourse dynamics. Based on our experience, a cut-off is recommended for box culverts. Consideration may also be given to a low permeability clay seal at the inlet and outlet. Given the significant scour erosion observed at the culvert Site, a cut-off wall in conjunction with a clay seal may need to be considered for mitigation of scour erosion.

The following are some general suggestions for erosion and scour protection.

We recommend that concrete cut-off (apron) be constructed at the inlet to prevent seepage beneath and around the culvert, especially through the granular bedding and granular backfill around the culvert. Consideration can be given to providing a concrete apron at the outlet, especially if there is none. Beneath the culvert, the concrete cut-off wall should extend to a suitable depth (e.g. below any possible scour depth).

At the inlet, consideration may also be given to the use of a clay seal. The purpose of the clay seal is to allow water flow to be channelled through the culvert and minimise seepage 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. At the culvert Site, the existing cohesive stratum may be suitable for this purpose. In any event, the clay seal should be extended around the culvert from at least 0.3 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. The clay seal would generally be extended to about 6 to 8 m beyond the inlet.

At the outlet as well as at the inlet, in addition to the concrete cut-off and/or low permeability seal, a 0.6 m thick rock protection, consisting typically of 300 mm size rock can be considered. Another reference for consideration is OPSD 810.010 Rip-Rap Treatment for Culvert Outlets.

At the culvert outlet, a filter diaphragm could also be considered to minimize the risk of migration of fines.

A concrete headwall may be considered to reduce the potential for embankment slope erosion at the culvert location.

6 CLOSURE

We recommend that once the details of the culverts are finalized, our recommendations be reviewed for their specific availability. The "Limitations of Report" presented in Appendix G, are an integral part of this report.

For and on behalf of Coffey Geotechnics Inc.


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Appendix E

List of SP, OPSSs, OPSDs and NSSP

List of SPs, OPSSs, OPSDs and NSSP referenced in the report

SP 105S10 Construction Specification for Compaction

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 Systems

OPSS 571 Construction Specification for Sodding

OPSS 572 Construction Specification for Seed and Cover

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

OPSS 1205 Material Specification for Clay Seal

OPSD 208.010 Benching of Earth Slopes

OPSD 803.010 Backfill and Cover for Concrete Culverts

OPSD 803.030 Frost Treatment – Pipe Culverts, Frost Penetration Line Below Bedding Grade

OPSD 803.031 Frost Treatment – Pipe Culverts, Frost Penetration Line Between Top of Pie and Bedding Grade

OPSS 902 – Construction Specification for Excavating and Backfilling-Structures.

NSSP for Dewatering Culvert Excavation

DEWATERING STRUCTURE EXCAVATIONS - Item No. 119

Special Provision

The requirements of OPSS 902, November 2009 shall govern this specification with the following amendments:

902.07 CONSTRUCTION **902.07.04 Dewatering Culvert Excavation**

Subsection 902.07.04 of OPSS 902, November 2009, is amended by addition of the following paragraphs:

The contractor shall be alerted that based on groundwater observations artesian groundwater conditions exist at the Project Site.

The potential for hydraulic blow-out should be minimised and sufficiently dry conditions for excavations should be provided by reducing the hydraulic head in the glacial till stratum under artesian pressure, to at least 1.0 m below the lowest intended excavation elevation and should be maintained within the foundation excavation during culvert installation. Perched water and surface drainage measures and control of water flowing in the watercourse are also required to ensure sufficiently dry conditions during construction within the foundation excavation.

When designing the dewatering system and, if required, shoring support for the support of excavation slopes, the presence of cobbles and boulders within the underlying glacial deposits should be taken into consideration.

902.10 BASIS FOR PAYMENT **902.10.02 Dewatering Culvert Excavation - Item**

Subsection 902.10.02 of OPSS 902, November 2009, is amended by addition of the following paragraph:

Payment at the contract price for the tender Item "Dewatering Culvert Excavation" shall also include full compensation for all labour, equipment and material to do the work as specified under subsection 902.07.04 as amended.

Appendix F

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.