

**FOUNDATION INVESTIGATION AND  
DESIGN REPORTS - PROPOSED  
CULVERT EXTENSION  
MIDTOWN CREEK EAST  
AT STATION 20+310 HIGHWAY 401,  
COBOURG, ONTARIO  
W.P. 205-00-01, GEOCRES NO. 30M16-44**

**AECOM**  
TRANETOB10434AA-AK  
February 6, 2012

February 6, 2012

AECOM  
5080 Commerce Boulevard  
Mississauga Ontario  
L4W 4P2

**Attention: Ms. Peggy Baleka**

Dear Madam,

**RE: Foundation Investigation and Design Reports, Proposed Culvert Extension Midtown Creek  
East at Station 20+310 Highway 401, Cobourg, Ontario W.P. 205-00-011, Geocres No. 30M16-44**

Coffey Geotechnics Inc (Coffey) is pleased to present the Foundation Investigation and Design Reports for the proposed culvert extension on Highway 401, Cobourg, Ontario.

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

Distribution: Original held by Coffey Geotechnics Inc.  
1 copy held by Coffey Geotechnics Inc.  
3 bound copies to AECOM

**FOUNDATION INVESTIGATION REPORT  
MIDTOWN CREEK EAST  
CULVERT EXTENSION  
AT STATION 20+310  
HIGHWAY 401, COBOURG, ONTARIO  
W.P. 205-00-01, GEOCRES NO. 30M16-44**

**AECOM**

TRANETOB010434AA-AK  
February 6, 2012

# CONTENTS

<b>1</b>	<b>INTRODUCTION</b>	<b>1</b>
<b>2</b>	<b>SITE DESCRIPTION AND PHYSIOGRAPHY</b>	<b>1</b>
<b>3</b>	<b>FIELD AND LABORATORY WORK</b>	<b>2</b>
<b>4</b>	<b>SUBSURFACE CONDITIONS</b>	<b>3</b>
<b>4.1</b>	<b>Fill</b>	<b>4</b>
<b>4.1.1</b>	<b>Granular Pavement Fill</b>	<b>4</b>
<b>4.1.2</b>	<b>Embankment Fill – Silty Sand</b>	<b>4</b>
<b>4.2</b>	<b>Topsoil</b>	<b>5</b>
<b>4.3</b>	<b>Clayey Silt To Silty Clay</b>	<b>5</b>
<b>4.4</b>	<b>Silty Sand Till</b>	<b>6</b>
<b>4.5</b>	<b>Groundwater Conditions</b>	<b>7</b>

## Tables

Table 1: Borehole Details

Table 2: Summary of Groundwater Levels

## Drawing

Drawing 1: Borehole Location Plan and Soil Strata

## Appendices

Appendix A: Record of Borehole Sheets

Appendix B: Laboratory Test Results

Figure B1: Grain Size Distribution – Embankment Fill: Silty Sand

Figure B2: Grain Size Distribution – Clayey Silt to Silty Clay

Figure B3: Plasticity Chart – Clayey Silt to Silty Clay

Figure B4: Consolidation Results BH C5: TW7– Clayey Silt To Silty Clay

# CONTENTS

Figure B5: Grain Size Distribution – Silty Sand Till

Appendix C: Site Photographs

Appendix D: Explanation of Terms Used in Report

**FOUNDATION INVESTIGATION REPORT  
MIDTOWN CREEK EAST CULVERT EXTENSION AT STATION 20+310  
HIGHWAY 401, COBOURG, ONTARIO  
G.W.P. 205-00-01**

## **1 INTRODUCTION**

The existing Midtown Creek East culvert will be lengthened as part of the proposed six laning of Highway 401 from Burnham street in Cobourg, Ontario to approximately 2.0 km east of Nagle Road, for a total length of 6.5 km. The existing culvert is located at Station 20+310, under Highway 401 and is a 2.4 m wide, 1.83 m high and approximately 68 m long rigid frame, open footing concrete structure.

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, which will be reported separately as per the project brief, and Midtown Creek East 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 East culvert is located at Station 20+310 under Highway 401, in the Town of Cobourg, as shown in Drawing No. 1. The existing culvert of span 2.4 m, is approximately 68 m long and has a rise of ~1.83 m. See site photographs in Appendix C. It is to be extended by 4 m to 5 m on each side to accommodate the embankment widening. The existing culvert is at a skew to the highway alignment.

Midtown Creek East flows in an approximately north to south direction, is about 3 m wide and at the inlet end, about 1.5 m deep, at the Highway 401 crossing.

The topography in the general area can be described as mildly rolling. According to "The Physiography of Southern Ontario" by L.J. Chapman and D.F. Putnam, 1984, the culvert site is located within the physiographic region known as 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 area is about 5.6 km in width and has a peculiar belted pattern. The land within the project area is almost flat and is covered by glacial/glacial lake deposits overlying a sandy glacial till deposit.

The bedrock within the project area is known to consist 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 and the pavement in the vicinity does not indicate any noticeable distress in terms of settlements/unusual cracking or deformations. However, scouring of the stream bed through the structure had been noted.

### 3 FIELD AND LABORATORY WORK

The fieldwork for this culvert investigation was performed during the period from June 11, 2010 to August 3, 2010, and consisted of drilling and sampling a total of four boreholes (C5 to C 8). 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	Ground Elevation (m)	Drilled Depth (m)	Remarks
C5	20+328	37.0 m Lt C/L (Outside the toe of embankment)	116.0	13.8	Track-mounted rig Solid stem auger
C6	20+315	17.0 m Lt C/L (On top of embankment)	120.9	22.1	Track-mounted rig Solid stem auger Piezometer installed
C7	20+298	16.0 m Rt C/L (On top of embankment)	120.6	16.9	Truck-mounted rig Hollow stem auger
C8	20+289	31.0 m Rt C/L (Outside the toe of embankment)	115.1	10.7	Track-mounted rig Hollow stem auger Piezometer installed

Boreholes C5, C6 and C8 were advanced using a track-mounted drilling rig owned and operated by Eastern Drilling of Chelmsford, Ontario. Borehole C6 was advanced with the track-mounted rig sitting on plywood planks to protect the road surface. Borehole C7 was advanced from the top of the road embankment, using a truck-mounted drilling rig owned and operated by Strong Soil of Chelmsford, Ontario.

No rock coring was undertaken. No special access provisions due to soft ground conditions (e.g. working platform) were required for the drilling. However, due to uneven ground and boulders, the drilling rig had to be moved several times due to uneven slope and a few rock piles before a suitable position was found for Borehole C8.

The borehole locations were established in the field by Coffey engineering staff, utilizing the 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). Several thin walled Shelby tube samples were also obtained in the cohesive soils.

Dynamic cone penetration testing (DCPT) was undertaken in Boreholes C5 and C7 from the bottom of each hole and advanced until refusal (i.e. number of blows to extend the cone by 0.3 m (or a portion thereof) by at least 100). The DCPT consists of driving an uncased 50 mm diameter cone continuously, attached to A-size drill rods, with a driving energy of 475 kJ (63.5 kg hammer free falling for a distance of 0.76 m) per blow, generally, adjacent to the borehole. The number of blows for each 0.3 m of penetration is recorded, providing an indication of the relative changes in the soil density with respect to depth.

In addition to the SPT, where the consistency permitted, in-situ shear vane tests were carried out within the cohesive soils to assess the undrained shear strength of the soil. The field vane shear tests were performed using an MTO 'N' vane.

Groundwater conditions were observed during and on completion of drilling in the open boreholes. Standpipe piezometers were installed in Boreholes C6 and C8 upon their completion. The remaining two 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 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. In addition a consolidation test was carried out on a relatively undisturbed sample recovered with a thin walled open tube sampler (TW) in Borehole C5. The results of the laboratory tests are presented on the appropriate Record of Borehole Sheets in Appendix A, and also in Appendix B, except the results of the consolidation tests are given in Appendix B only. However, the consolidation test location depth is noted on the Record of Borehole Sheet for Borehole C5.

## **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 an 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 1.2 m to 7.2 m thick fill, over natural overburden soils within the depths investigated. Drilling was terminated within the silty sand till encountered in all the boreholes upon reaching either SPT spoon refusal or DCPT refusal.

Fill was intercepted in all the boreholes. In Boreholes C6 and C7, it comprises a pavement fill (~0.6 m thick) and a basically silty sand embankment fill. In Boreholes C5 and C8, beneath some topsoil, this silty



sand embankment fill was also intercepted. Broadly, this silty sand embankment fill has a trace to some clay and gravel and varied in thickness from 1.2 m to 6.6 m.

Underlying the fill, a clayey silt to silty clay layer was intercepted in all the boreholes. with a thickness varying from 1.6 m to 5.9 m. The basal layer, underlying the clayey silt to silty clay deposit, encountered in all the boreholes, within the investigated depths, is a silty sand till in which drilling was terminated. Penetrations into this basal layer ranged from 6.1 to 10.1 m at the termination of the boreholes.

The following paragraphs are presented only to amplify and complement the data 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**

### **4.1.1 Granular Pavement Fill**

A pavement fill was encountered to a depth of 0.6 m (Elevations 120.3 to 120.0 m) from the surface in Boreholes C6 and C7, drilled from the embankment shoulder.

This pavement fill is granular in nature and consists of sand and gravel, some silt, a trace of clay and asphalt and pieces of concrete.

SPT 'N' values of 9 & 16 blows/0.3 m were measured within the pavement fill, indicating a loose to compact relative density.

### **4.1.2 Embankment Fill – Silty Sand**

Underlying the pavement fill, an embankment fill was intercepted that extended to depths of 6.6 to 7.2 m (Elevations 114.0 to 113.7 m) in Boreholes C6 and C7. In Boreholes C5 and C8, a fill deposit of similar composition was present beneath a 0.6 to 0.8 m thick layer of topsoil, to depths of 1.8 to 3.0 m (Elevations 114.2 to 112.1 m). This fill was probably placed at the time of the construction of the existing culvert, to facilitate construction. From the elevations recorded for the bottom of the fill in the boreholes, the original grade (o.g.) at the culvert site appears to be about 112 to 114 m.

Grain size analyses were carried out on five 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 - 13%

Sand: 40 - 69%

Silt: 21 - 38%

Clay: 10 - 20%

Based on the grain size analyses, the fill material consisted of silty sand with trace to some clay and gravel. The presence of organic inclusions was also noted. In borehole C6, asphalt fragments were found at mid-height of the embankment fill. The fill is classified as a basically granular (i.e. non cohesive) material.

Standard Penetration Tests performed in this material yielded an average SPT 'N'-value of 17 blows/0.3 m, (SPT 'N' range 2 – 38; 22 test results; In Borehole C6 at a depth of about 3 m, asphalt pieces were encountered (as mentioned before) giving a SPT 'N' = 100, and this value was disregarded in calculating the average). The SPT 'N' values indicate generally a very loose to dense (i.e. variable) compactness condition but typically a compact relative density.

Natural water contents measured on twenty seven samples recovered from this stratum has an average of 13.0% (ranging from 3.3% to 52.1%). Increased moisture contents noted in this layer with depth are probably due to the organic materials intercepted.

## **4.2 Topsoil**

As mentioned before, a surficial layer of topsoil, 0.6 to 0.8 m thick, was encountered in Boreholes C5 and C8, overlying the fill. It should be pointed out that, based on our experience, the thickness of organic rich topsoil frequently varies in between and beyond borehole locations, especially in depressed areas and immediately near watercourses.

## **4.3 Clayey Silt To Silty Clay**

A clayey silt to silty clay deposit was encountered in all boreholes at depths of 1.8 to 7.2 m (Elevations 114.2 to 112.1 m), and extended to depths of 4.6 to 12.0 m (Elevations 112.1 – 108.3 m).

Grain size analyses were carried out on four representative samples and on one sample from a sandy zone intercepted in Borehole C8, 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 for the representative samples:

Gravel: 0%  
Sand: 2 - 4%  
Silt: 47 - 57%  
Clay: 39 - 50%

Results of the sample from the sandy zone yielded:

Gravel: 3%  
Sand: 23%  
Silt: 48%  
Clay: 26%

Based on these results, the clayey silt to silty clay layer is considered to be of low permeability, with occasional sandy zones as contacted in Boreholes C5, C6 and C8.

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

Liquid Limit: 25 – 32 %

Plastic Limit: 14 – 17%

Plasticity Index: 10 – 15%

The above values are characteristic of a cohesive soil of low to medium plasticity. As shown on the Record of Borehole Sheets, the measured natural moisture contents of the samples tested are closer to the measured Liquid Limits rather than the measured Plastic Limit values.

A consolidation test was conducted on a 50 mm dia. undisturbed sample recovered with a thin-walled Shelby tube from Borehole C5. The coefficient of volume decrease,  $m_v$ , for this sample yielded values in the range 0.1 to 0.2  $\text{m}^2/\text{MN}$  corresponding to the applied field stress range. The results indicate a stress history with a possible pre-consolidation pressure,  $p_c$  of 200 kPa, or a pressure increment over the in-situ vertical effective stress,  $p_o$ , of about 130 kPa as shown in Figure B4 in Appendix B. As well, a  $C_c$  value of the order of 0.17 and a  $C_r$  value of about 0.035 are obtained.

Standard Penetration Tests performed in this deposit yielded an average SPT 'N' of 9 blows/0.3 m, (SPT 'N' ranging from 4 to 14; 8 test results). Eleven field shear vane tests were also carried out within this cohesive stratum and the undrained in-situ vane shear strength was measured between approximately 40 kPa to in excess of 100 kPa, but typically about 60 kPa, with the higher values possibly influenced by the sandy zones. Average natural water content measured on samples recovered from this stratum was 24% (ranging from 12 % (the more sandy zones) to 33%; 12 test results). This material has a liquidity index varying from 0.7 to 1.1 with an average of 0.9 which is indicative of a normally consolidated condition, which appears to be at variance with other parameters such as the preconsolidation pressures estimated from the oedometer testing.

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

#### **4.4 Silty Sand Till**

Underlying the clayey silt to silty clay stratum, a silty sand till deposit was encountered in all the boreholes as the basal layer, within the depths investigated, at depths of 4.6 to 12.0 m (Elevations 112.1 to 108.3 m), and extended to the termination of the sampled portion of the boreholes, at depths of 10.7 to 22.1 m (Elevations 104.9 to 98.8 m).

The following are results of grain size distribution tests conducted on six representative samples from this deposit, as shown on Figure B5 in Appendix B.

Gravel: 8 - 27%

Sand: 40 - 62%

Silt 13 - 32%

Clay 6 - 16%

Natural water contents measured on samples recovered from this stratum has an average of 8.9% (ranging from 6.2% to 15.5%; 22 test results).

Based on visual and tactile observations, and grading and moisture information, this deposit is described as a moist to wet heterogeneous mixture of silty sand 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, e.g. see the grain size distribution for Sample SS11 from Borehole C7) some cementation can be expected.

Standard Penetration Tests performed in this stratum yielded an average SPT 'N' value of about 51 blows/0.3 m, (SPT 'N' ranging from 3 to in excess of 100; 20 test results). The recorded SPT 'N' values indicate a generally very loose to compact zone within the upper 2 to 3 m becoming dense to very dense below the upper relatively weak zone.

Although not specifically encountered in the boreholes drilled (except for auger grinding on cobbles in Borehole C5, below El. 104 m), the presence of cobbles and boulders should always be anticipated in glacial till deposits, due to their mode of deposition.

All boreholes were terminated within this deposit at depths ranging from 10.7 to 22.1 m (Elevations 104.4 to 98.8 m) due to high resistance encountered by the SPT spoon, or the DCPT or the auger refusal.

#### **4.5 Groundwater Conditions**

Groundwater levels were observed in the open boreholes while drilling and upon completion of each borehole. In addition, piezometers were installed in Boreholes C6 and C8 to monitor the groundwater levels with time, without interference from surface water. The tips of the 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)		High Water Level Measured
			Depth/Elevation (m)	Date Measured	Depth/Elevation (m) / Date
C5	116.0	Not installed	1.0/115.0* Cave-in @ 3.2 m (El. 112.8 m) Spoon wet 2.5 m (El. 113.5 m)	11 Jun 2010 (completion)	1.0/115.0
C6	120.9	12.2/108.7 (artesian)	5.8/115.1 Cave-in @ 12.2 m	15 Oct 2010 (122 days after completion)	5.5/115.4 16 July 2010 (1 day after completion)
C7	120.6	Not installed	4.6/116.0* Cave-in @ 9.1 m	30 Jul 2010 (completion)	4.6/116.0
C8	115.1	9.1/106.0 (artesian)	1.9/113.2 Cave-in @ 9.1 m Spoon wet @ 3.1 m (El. 117.5 m)	15 Oct 2010 (73 days after completion)	1.9/113.2 15 Oct 2010 (73 days after completion)


Note: \* Groundwater level measured not stabilized.

In general, groundwater levels measured on completion are not considered to have stabilised, and therefore, may not represent the groundwater table at the site. Further, the highest groundwater levels measured at the site in Boreholes C6 and C8 were 1.7 m and 1.1 m respectively above the existing original ground surface (o.g.) level or approximately at Elevations 115.4 m and 113.2 m respectively.

These observations in Table 2 suggest the existence of an artesian head in the silty sand till underlying the low permeable, clayey silt to silty clay stratum.

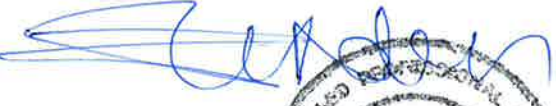
It should be pointed out that the groundwater would be subject to seasonal fluctuations and fluctuations in response to major weather events. The water table at the site may be influenced by the water level in the watercourse. A perched water table may also occur due to the accumulation of the surface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


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



METRIC

NOTES:

FOR DETAILED SUBSURFACE CONDITIONS  
REFER TO RECORD OF BOREHOLE SHEETS.

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

CONT No.

GWP: 205-00-01

MIDTOWN CREEK EAST CULVERT  
EXTENSION  
BOREHOLE LOCATION PLAN  
AND SOIL STRATA



SHEET

coffey geotechnics  
SPECIALISTS MANAGING THE EARTH



LEGEND

- Borehole
- N Blows/0.3m (Std. Pen. Test, 475 J/blow)
- Water Level in Piezometer
- Water Level at Time of Investigation (W.L. NOT STABILIZED)
- Piezometer

No	ELEVATION	EASTING	NORTHING
C5	116.0	412762.4	4873451.0
C6	120.9	412750.1	4873430.6
C7	120.6	412734.2	4873397.0
C8	115.1	412725.2	4873381.3

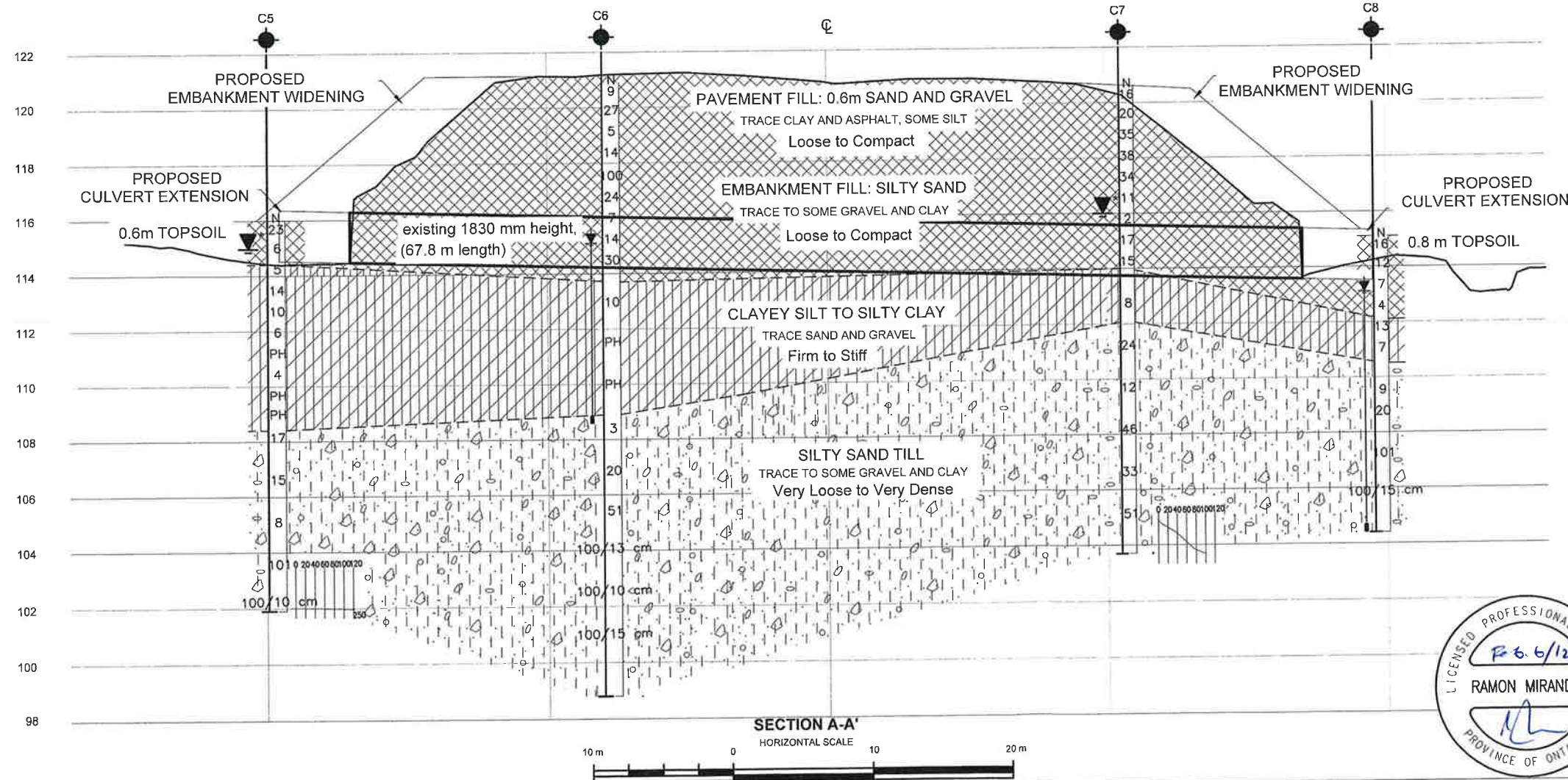
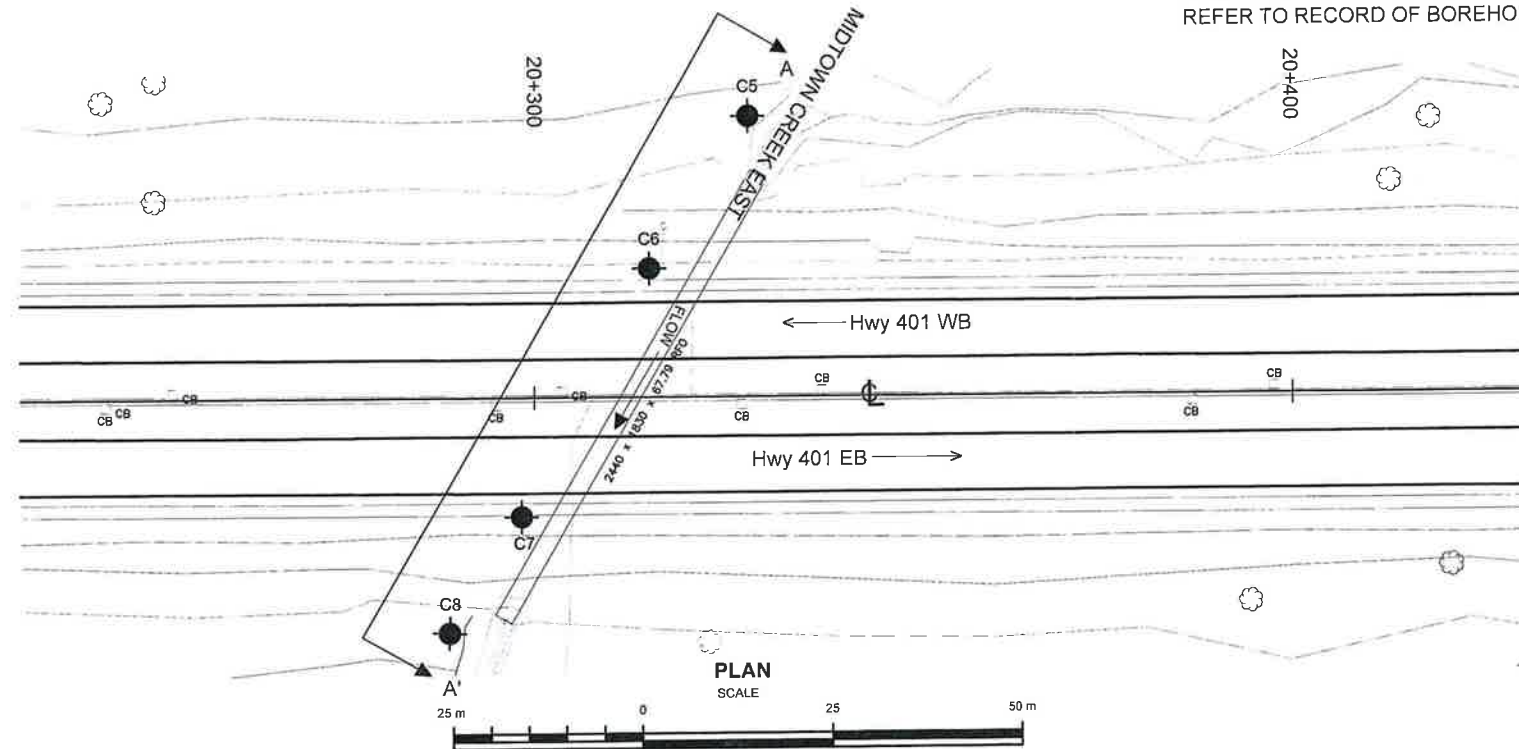
-NOTE-

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. 30M16-44	TRANET0104345AA	DIST
SUBMD	CHECKED	DATE Aug 25, 2011
DRAWN	SH	CHECKED RM
APPROVED	ZO	DWG
1		





# Appendix A

## Record of Borehole Sheets

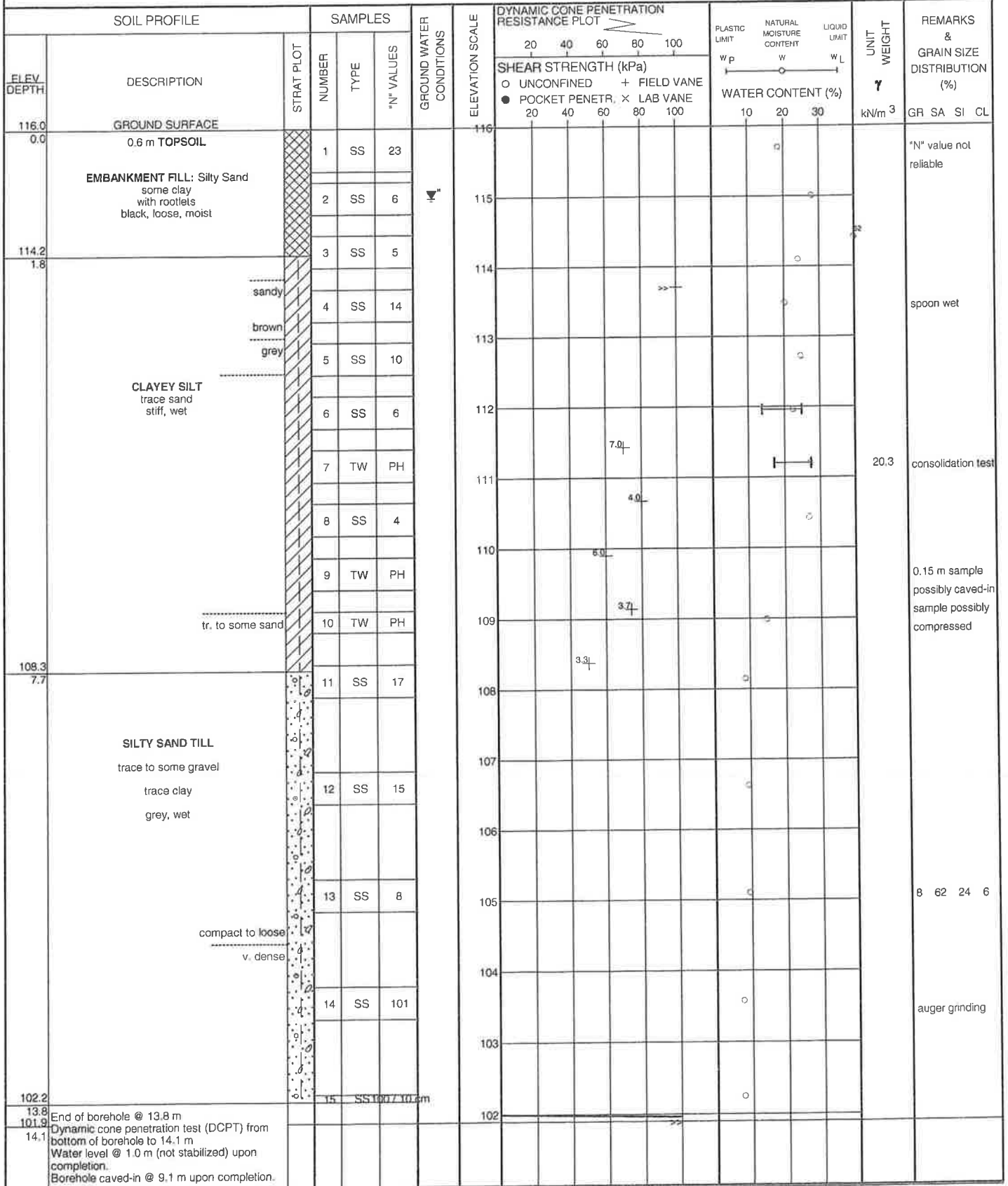
TRANETOB10434AA: Highway 401

# RECORD OF BOREHOLE No C5

1 OF 1

METRIC

GWP G.W.P 205-00-01 LOCATION Station 20+328, 37 m Lt of C/L (E 412762.4, N 4873451.0) ORIGINATED BY GJ  
 DIST HWY 401 BOREHOLE TYPE Solid Stem Auger, DCPT COMPILED BY SK  
 DATUM Geodetic DATE 6/11/2010 CHECKED BY ZO



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

TRANETOB10434AA: Highway 401

# RECORD OF BOREHOLE No C6

1 OF 2

METRIC

GWP G.W.P 205-00-01 LOCATION Station 20+315, 17 m Lt of C/L (E 412750.1, N 4873430.6) ORIGINATED BY GJ  
 DIST HWY 401 BOREHOLE TYPE Solid Stem Auger COMPILED BY SK  
 DATUM Geodetic DATE 6/15/2010 CHECKED BY ZO

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT	NATURAL MOISTURE CONTENT		
120.9	GROUND SURFACE										
0.0	0.3 m GRANULAR FILL: Sand and gravel 0.3 m GRANULAR FILL: Sand some gravel and silt, trace clay brown, loose to compact, moist	1	SS	9		120					
		2	SS	27							
	EMBANKMENT FILL: Silty Sand trace to some gravel, trace clay brown, loose to compact, moist	3	SS	5		119					
		4	SS	14							
		5	SS	100		118					13 44 33 10
	asphalt fragments	6	SS	24		117					
		7	SS	7		116					
		8	SS	14		115					
	black, some org. grey compact to dense	9	SS	30		114					spoon wet 0 69 21 10
113.7											
7.2											
	CLAYEY SILT TO SILTY CLAY trace sand brown, stiff, wet	10	SS	10		113					0 3 56 41
		11	TW	PH		112					
		12	TW	PH		111	1.8				
						110	2.1				0 3 47 50
						109	5.5				
108.9											
12.0	SILTY SAND TILL trace to some clay, trace to some gravel grey, wet	13	SS	3		108					11 58 21 10
	v. loose compact to dense	14	AS			107					
		15	SS	20		106					

Continued Next Page

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



TRANETO10434AA: Highway 401

## RECORD OF BOREHOLE No C6

2 OF 2

METRIC

GWP G.W.P 205-00-01 LOCATION Station 20+315, 17 m Lt of C/L (E 412750.1, N 4873430.6) ORIGINATED BY GJ  
DIST HWY 401 BOREHOLE TYPE Solid Stem Auger COMPILED BY SK  
DATUM Geodetic DATE 6/15/2010 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			FLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100						WATER CONTENT (%)
105.9	SILTY SAND TILL trace clay, trace to some gravel grey, v. dense, wet		16	SS	51		105									
								104								
			17	SS100 / 13.2m				103								
			18	AS				102								
			19	SS100 / 10.2m				101								
			20	SS100 / 15.2m				100								
98.8	End of Borehole sampling @ 20.0 m Water Level @ 5.8 m (not stabilized) upon completion Auger drilled down to 22.1 m for piezometer Borehole caved-in @ 12.2 m upon completion. Piezometer installed to 12.2 m Piezometer water level records : June 16, 2010 5.6 m July 16, 2010 5.5 m Aug 19, 2010 5.9 m Oct 15, 2010 5.8 m						99									
22.1																

TRANETOB10434AA: Highway 401

# RECORD OF BOREHOLE No C7

1 OF 2

METRIC

GWP G.W.P 205-00-01 LOCATION Station 20+298, 16 m Rt of C/L (E 412734.3, N 4873397.0) ORIGINATED BY GJ  
DIST HWY 401 BOREHOLE TYPE Hollow Stem Auger, DCPT COMPILED BY WC  
DATUM Geodetic DATE 7/29/2010 7/30/2010 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	
120.6 0.0	GROUND SURFACE											
	0.6 m GRANULAR FILL: sand and gravel, tr. asphalt, concrete pieces brown, compact, moist		1	SS	16		120					
	EMBANKMENT FILL: Silty Sand tr. to some clay, tr. to some gravel brown, v. loose to dense, moist to wet		2	SS	20		119					
			3	SS	35		118					
			4	SS	38		117					
			5	SS	34		116					
			6	SS	11		115					
			7	SS	2		114					
		grey	8	SS	17		113					
		black, some organics	9	SS	15		112					
114.0 6.6	CLAYEY SILT TO SILTY CLAY brown, firm to stiff, moist		10	SS	8		111					
112.1 8.5	SILTY SAND TILL tr. clay, tr. to some gravel grey, compact to v. dense, wet		11	SS	24		110					
			12	SS	12		109					
			13	SS	46		108					
			14	SS	33		107					
							106					

Continued Next Page

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



TRANETOB10434AA: Highway 401

# RECORD OF BOREHOLE No C7

2 OF 2

METRIC

GWP G.W.P 205-00-01 LOCATION Station 20+298, 16 m Rt of C/L (E 412734.3, N 4873397.0) ORIGINATED BY GJ  
DIST            HWY 401 BOREHOLE TYPE Hollow Stem Auger, DCPT COMPILED BY WC  
DATUM Geodetic DATE 7/29/2010 7/30/2010 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT	UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
105.6											
104.9			15	SS	51						
15.7	End of borehole Dynamic cone penetration test (DCPT) from 15.8 m to 16.9 m										
103.7											
16.9	Water level @ 4.6 m (not stabilized) upon completion. Borehole caved-in @ 9.1 m upon completion.										

+ 3 x 3

Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE



TRANETOB10434AA: Highway 401

# RECORD OF BOREHOLE No C8

1 OF 1

METRIC

GWP G.W.P. 205-00-01 LOCATION Station 20+289, 31 m Rt of C/L (E 412725.2, N 4873381.3) ORIGINATED BY LG  
DIST            HWY 401 BOREHOLE TYPE Hollow Stem Auger COMPILED BY WC  
DATUM Geodetic DATE 8/3/2010 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100					
115.1 0.0	GROUND SURFACE						115								
	0.8 m TOPSOIL		1	SS	16										
	EMBANKMENT FILL: Silty Sand grey, v. loose to compact, moist		2	SS	12		114								
			3	SS	7		113								0 48 34 18
			4	SS	4		112								
112.1 3.0	CLAYEY SILT tr. gravel grey, firm to stiff, wet		5	SS	13		111								spoon wet below 3.1 m
	sandy		6	SS	7		110								0 4 57 39
110.5 4.6	SILTY SAND TILL tr. to some gravel grey, compact to v. dense, wet						109								3 23 48 26
			7	SS	9		108								
			8	SS	20		107								
			9	SS	101		106								2.1 m of soil back-up @ 7.6 m
			10	SS	100 / 15.8 m		105								23 56 13 8
104.4 10.7	End of Borehole Auger refusal @ 10.7 m Piezometer installed @ 9.1 m. Borehole caved-in @ 9.1 m upon completion. Date / Measured Water Level August 19, 2010 / 2.0 m October 15, 2010 / 1.9 m														1.5 m of soil back-up @ 10.7 m

# Appendix B

## Laboratory Test Results



# UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT		SAND			GRAVEL		
		Fine	Medium	Coarse	Fine	Coarse	

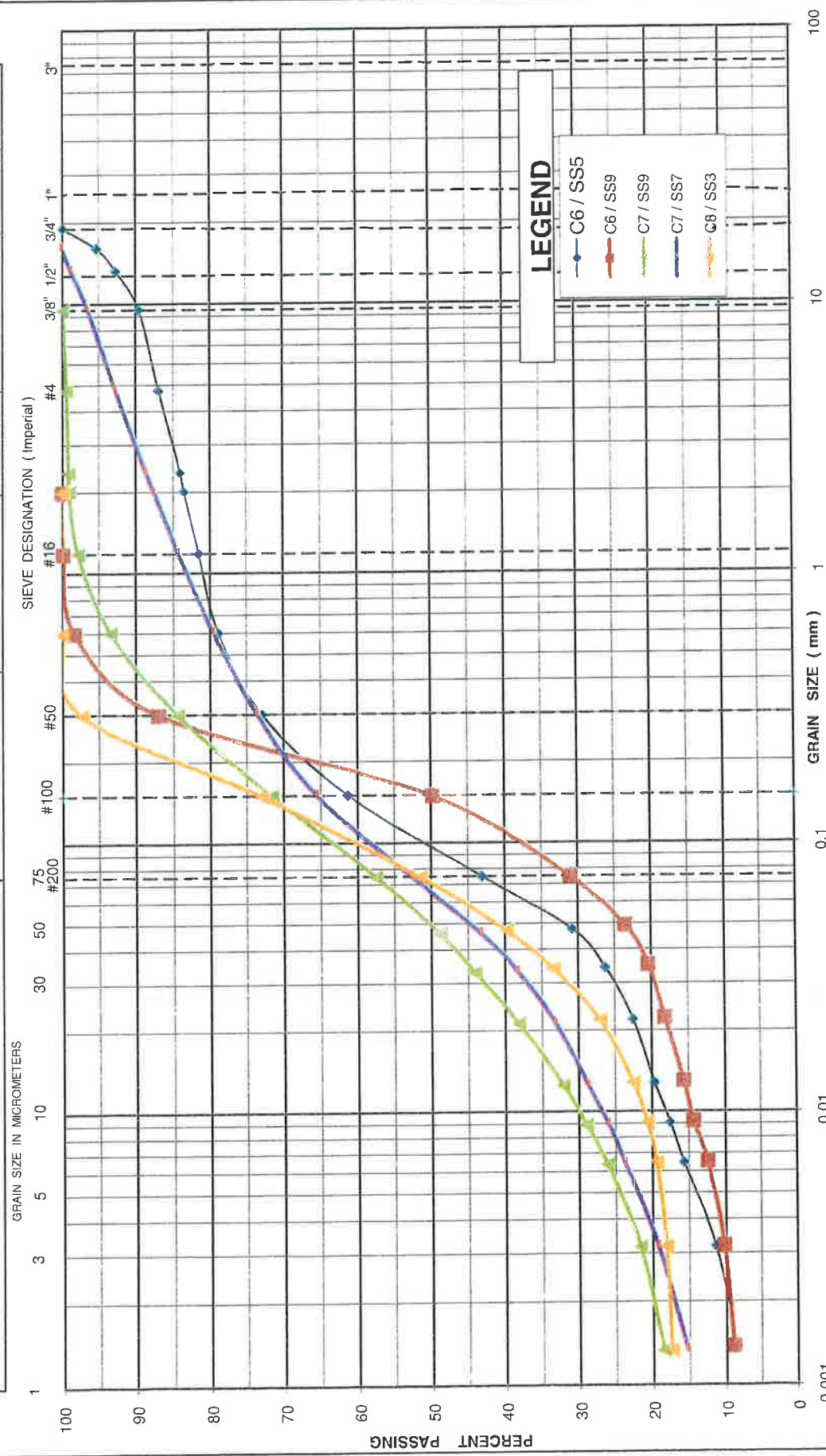
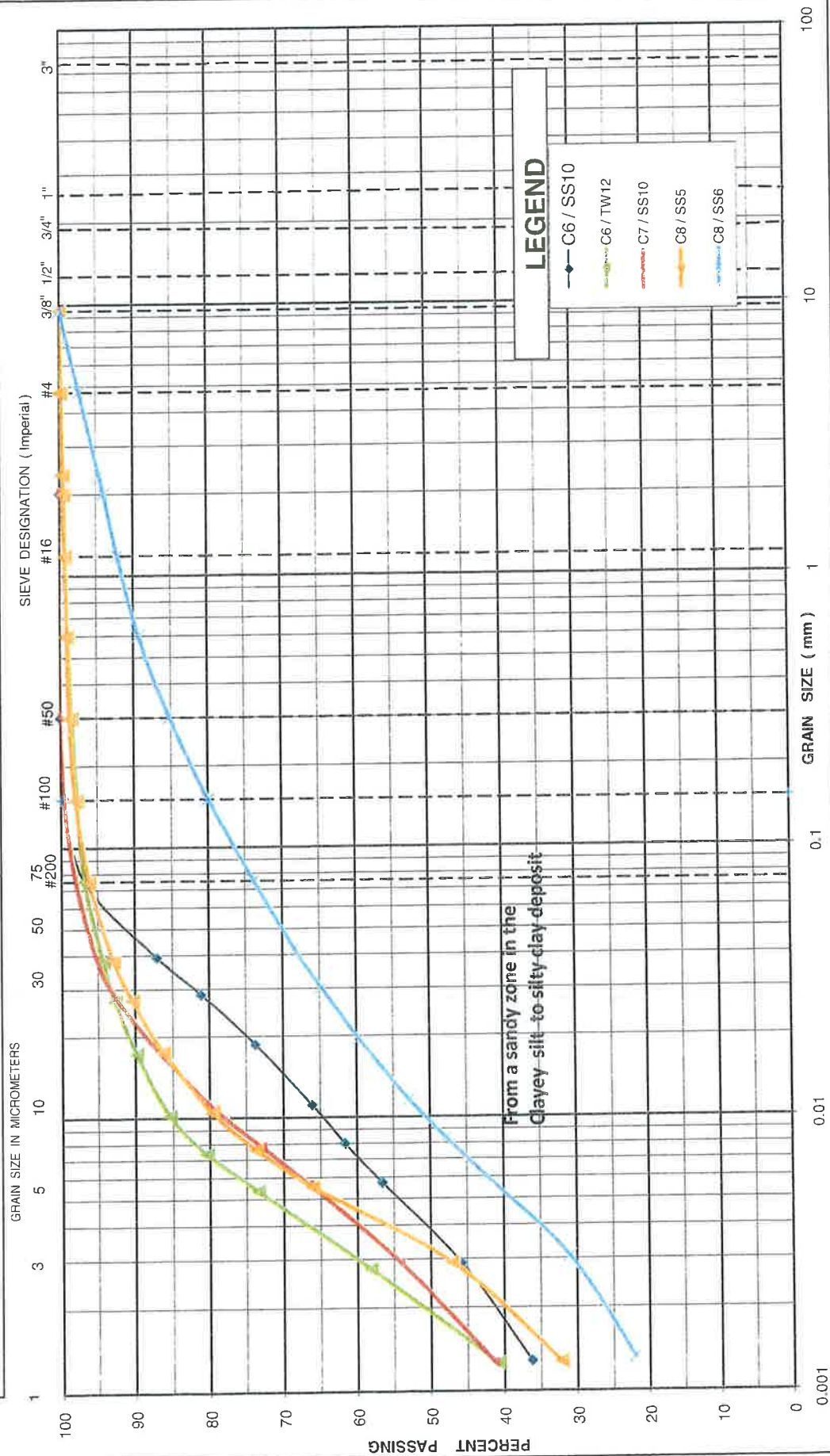


FIGURE NO.: B1  
PROJECT NO: TRANETOB10434AA  
DATE: AUG. 2010

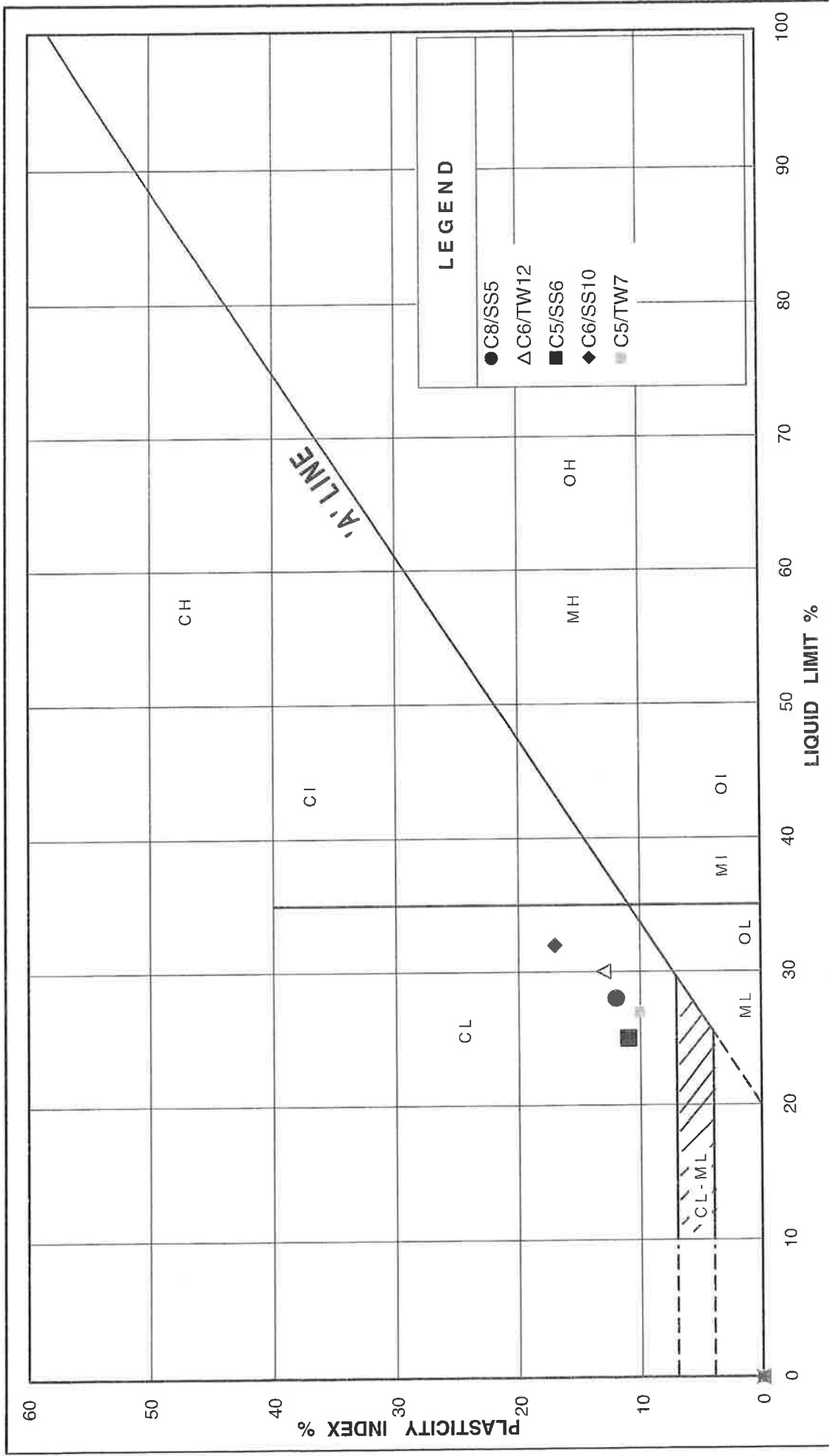
GRAIN SIZE DISTRIBUTION  
Embankment Fill: Silty Sand


# UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT		SAND			GRAVEL		
		Fine	Medium	Coarse	Fine	Coarse	



## GRAIN SIZE DISTRIBUTION Clayey Silt To Silty Clay





**coffey** geotechnics  
SPECIALISTS MANAGING THE EARTH

**PLASTICITY CHART**

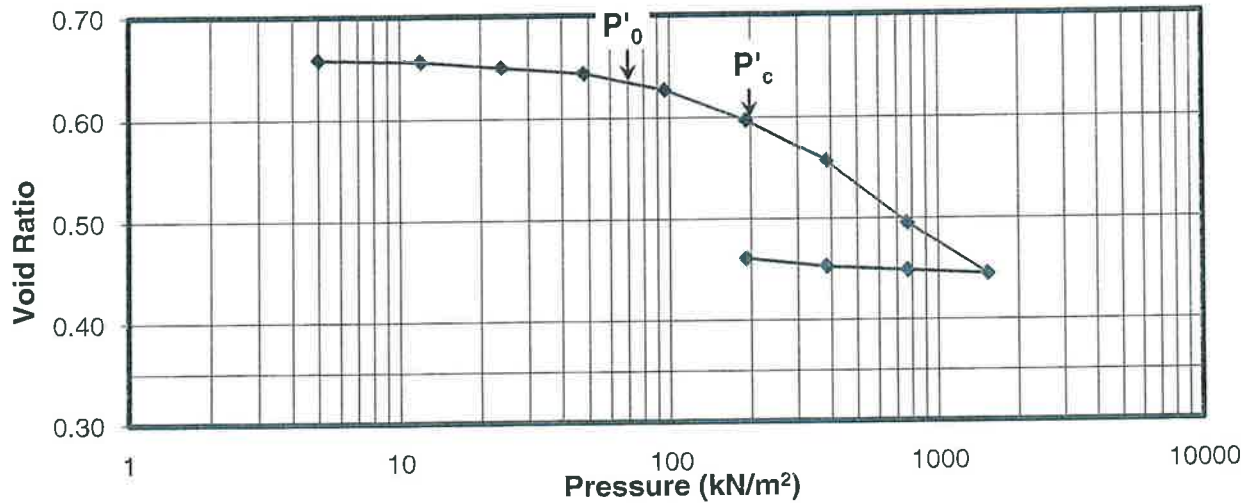
**Clayey Silt to Silty Clay**

FIGURE No. B3

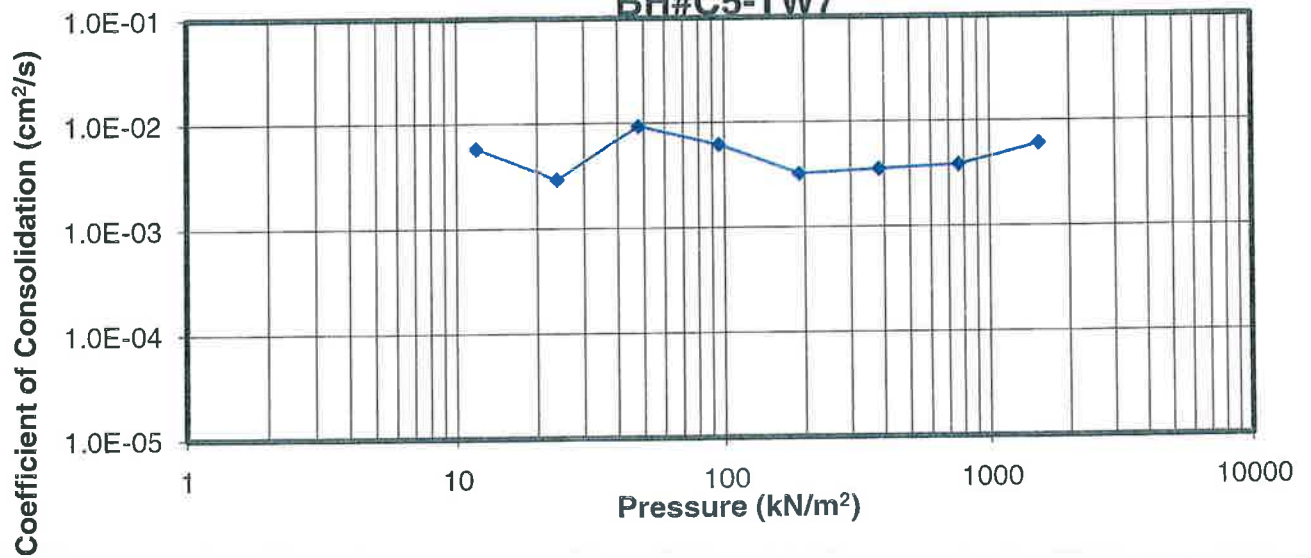
REF. No. TRANETOB10434AA


DATE Sept, 2010

### Void Ratio versus Pressure BH#C5-TW7



### Coefficient of Consolidation vs. Pressure BH#C5-TW7



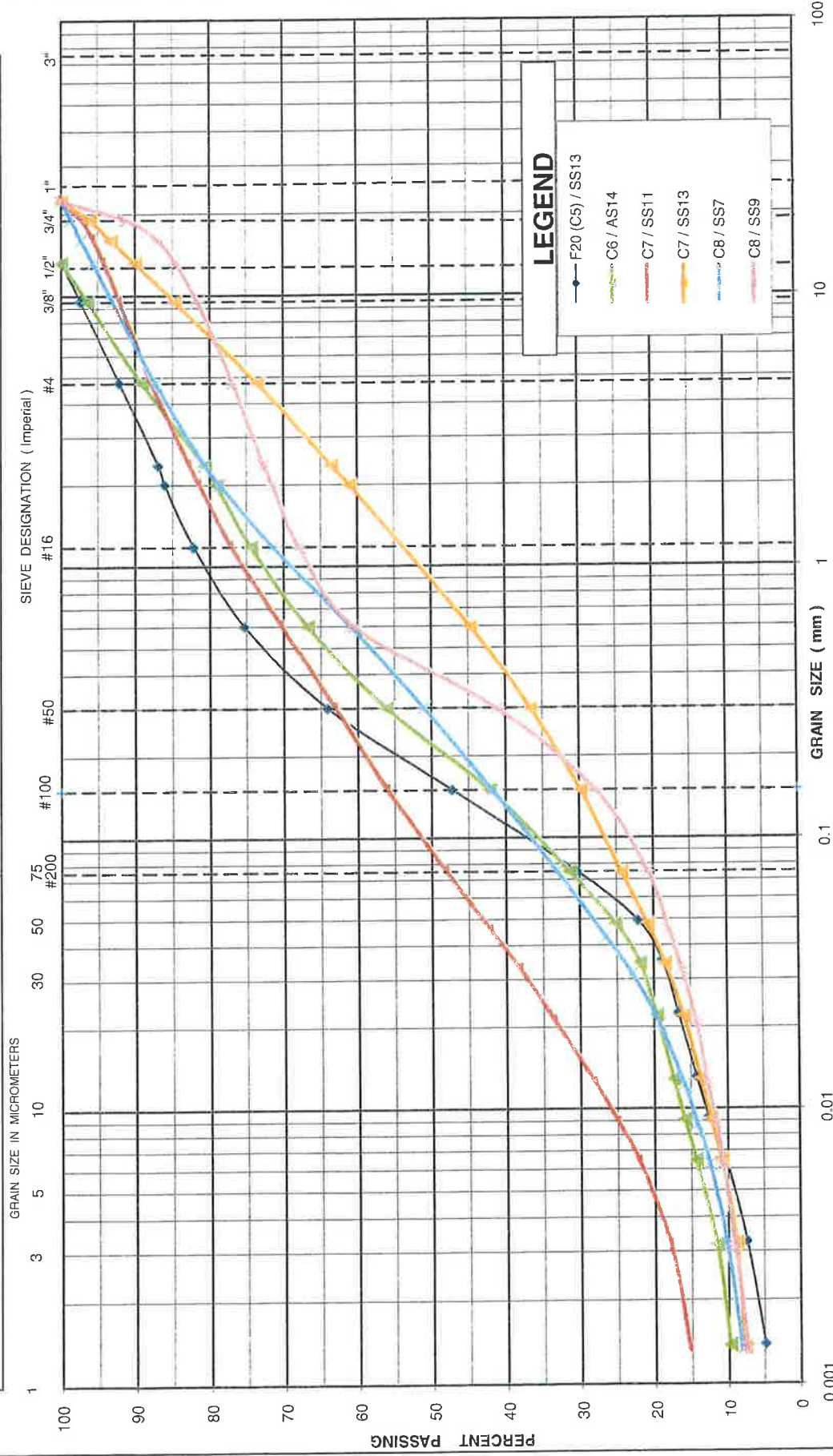
drawn	SS	 <b>coffey</b> <b>geotechnics</b> <small>SPECIALISTS MANAGING THE EARTH</small>	client:	AECOM	
approved	ZO		project:	HIGHWAY 401 EXPANSION	
date	Jan-11			Midtown Creek East	
scale	as shown		title:	CONSOLIDATION TEST RESULT - C5 TW7	
original size	Letter		project no:	TRANETOB10434AA	figure no: B4

F:\GEOT\transprt\ACTIVE\PROJECT2010\10434 - TRANETOB10434AA - Hwy 401, Burnham to Nagle\foundation reports\Culverts\Midtown Creek East\BH C5-TW 7- consolidation.



# UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT			SAND			GRAVEL		
			Fine	Medium	Coarse	Fine	Coarse	



 SPECIALISTS MANAGING THE EARTH	GRAIN SIZE DISTRIBUTION		FIGURE NO.: B5
	Silty Sand Till		PROJECT NO: TRANETOB10434AA
			DATE: JULY, 2010

# Appendix C

## Site Photographs





Station 20+330 WBL, Looking East



Midtown Creek East Culvert, Looking at North End of Culvert

# 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
$\sigma$	kPa	TOTAL NORMAL STRESS
$\sigma'$	kPa	EFFECTIVE NORMAL STRESS
$\tau$	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
$\epsilon$	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
$\mu$	1	COEFFICIENT OF FRICTION

### MECHANICAL PROPERTIES OF SOIL

$m_v$	kPa <sup>-1</sup>	COEFFICIENT OF VOLUME CHANGE
$c_c$	1	COMPRESSION INDEX
$c_s$	1	SWELLING INDEX
$c_a$	1	RATE OF SECONDARY CONSOLIDATION
$c_v$	m <sup>2</sup> /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
$T_v$	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
$\sigma'_{vm}$	kPa	EFFECTIVE OVERBURDEN PRESSURE
$\sigma'_p$	kPa	PRECONSOLIDATION PRESSURE
$\tau_f$	kPa	SHEAR STRENGTH
$c'$	kPa	EFFECTIVE COHESION INTERCEPT
$\phi'$	°	EFFECTIVE ANGLE OF INTERNAL FRICTION
$c_u$	kPa	APPARENT COHESION INTERCEPT
$\phi_u$	°	APPARENT ANGLE OF INTERNAL FRICTION
$\tau_R$	kPa	RESIDUAL SHEAR STRENGTH
$\tau_r$	kPa	REMOULDED SHEAR STRENGTH
$S_r$	1	SENSITIVITY = $c_u / \tau_r$

### PHYSICAL PROPERTIES OF SOIL

$P_s$	kg/m <sup>3</sup>	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	$e_{min}$	1, %	VOID RATIO IN DENSEST STATE
$\gamma_s$	kN/m <sup>3</sup>	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 <sup>3</sup>	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
$\gamma_w$	kN/m <sup>3</sup>	UNIT WEIGHT OF WATER	$S_r$	%	DEGREE OF SATURATION	$D_n$	mm	N PERCENT – DIAMETER
$\rho$	kg/m <sup>3</sup>	DENSITY OF SOIL	$W_L$	%	LIQUID LIMIT	$C_u$	1	UNIFORMITY COEFFICIENT
$\gamma'$	kN/m <sup>3</sup>	UNIT WEIGHT OF SOIL	$W_p$	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
$\rho_d$	kg/m <sup>3</sup>	DENSITY OF DRY SOIL	$W_s$	%	SHRINKAGE LIMIT	q	m <sup>3</sup> /s	RATE OF DISCHARGE
$\gamma_d$	kN/m <sup>3</sup>	UNIT WEIGHT OF DRY SOIL	$I_p$	%	PLASTICITY INDEX = $(W_L - W_p)$	v	m/s	DISCHARGE VELOCITY
$P_{sat}$	kg/m <sup>3</sup>	DENSITY OF SATURATED SOIL	$I_L$	1	LIQUIDITY INDEX = $(W - W_p) / I_p$	i	1	HYDRAULIC GRADIENT
$\gamma_{sat}$	kN/m <sup>3</sup>	UNIT WEIGHT OF SATURATED SOIL	$I_c$	1	CONSISTENCY INDEX = $(W_L - W) / I_p$	k	m/s	HYDRAULIC CONDUCTIVITY
$\rho'$	kg/m <sup>3</sup>	DENSITY OF SUBMERGED SOIL	$e_{max}$	1, %	VOID RATIO IN LOOSEST STATE	j	kN/m <sup>3</sup>	SEEPAGE FORCE
$\gamma'$	kN/m <sup>3</sup>	UNIT WEIGHT OF SUBMERGED SOIL						

**FOUNDATION DESIGN REPORT -  
PROPOSED CULVERT EXTENSION  
MIDTOWN CREEK EAST  
AT STATION 20+310 HIGHWAY 401,  
COBOURG, ONTARIO  
W.P. 205-00-01, GEOCRES NO. 30M16-44**

**AECOM**  
TRANETOB010434AA-AK  
February 6, 2012

# CONTENTS

<b>5</b>	<b>DISCUSSION AND RECOMMENDATIONS</b>	<b>10</b>
<b>5.1</b>	<b>Geotechnical Characterisation</b>	<b>10</b>
<b>5.2</b>	<b>Culvert Foundations</b>	<b>11</b>
<b>5.2.1</b>	<b>Frost Protection</b>	<b>12</b>
<b>5.2.2</b>	<b>Rigid Frame Open Footing Culvert on Strip Foundations Option</b>	<b>12</b>
<b>5.2.3</b>	<b>Box Culvert Option</b>	<b>14</b>
<b>5.2.4</b>	<b>CSP Type Culvert</b>	<b>16</b>
<b>5.2.5</b>	<b>Recommended Culvert Option</b>	<b>16</b>
<b>5.3</b>	<b>Embankment Widening</b>	<b>17</b>
<b>5.4</b>	<b>Bedding</b>	<b>18</b>
<b>5.5</b>	<b>Backfilling</b>	<b>18</b>
<b>5.6</b>	<b>Construction Considerations</b>	<b>20</b>
<b>5.7</b>	<b>Erosion Protection</b>	<b>23</b>
<b>6</b>	<b>CLOSURE</b>	<b>24</b>

## Appendices

Appendix E: SPs, OPSSs, OPSDs and NSSP

Appendix F: Limitations of Report

**FOUNDATION DESIGN REPORT  
MIDTOWN CREEK EAST CULVERT EXTENSION  
AT STATION 20+310 HIGHWAY 401, COBOURG, ONTARIO  
G.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 rigid frame open footing culvert at Station 20+310.

The existing culvert is a rigid frame, open footing, cast-in-place culvert (span: 2.4 m, inlet to outlet rise: ~1.83 m and approximately 68 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. Four boreholes (C5, C6, C7, and C8) 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 will be matched to the existing culvert which has an invert elevation of approximately 114.5 m at the inlet and 113.6 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 1.2 m to 7.2 m thick fill, over natural overburden soils (consisting of clayey silt to silty clay underlain by silty sand till) within the depths investigated, as presented in the Stratigraphic Section in Drawing No. 1.

The fill intercepted in Boreholes C6 and C7 comprise surficial pavement fill, underlain by a silty sand embankment fill to depths of 6.6 to 7.2 m. In Boreholes C5 and C8, beneath some topsoil (0.6 to 0.8 m thick), a silty sand fill was contacted ranging in thickness from 1.2 to 2.2 m. This fill is similar in composition to the embankment fill encountered in Boreholes C6 and C7 and is believed to have been placed to facilitate the construction of the existing culvert.

Broadly, based on the recorded N-values, the silty sand embankment fill is considered to be generally loose to compact.

Underlying the embankment fill, a firm to very stiff clayey silt to silty clay was intercepted in all the boreholes, with a thickness varying from 1.6 to 5.9 m. Based on visual and tactile observations and

grading results, the clayey silt to silty clay has been identified with occasional sandy zones as contacted in Boreholes C5, C6 and C8 (e.g. Borehole C8, grading result for sample SS6 as shown on Figure B2). At the borehole locations, this cohesive deposit extends to El. 112.1 m (BH C7) to El. 108.3 m (BH C5) and it appears to be thicker at the inlet side.

The basal layer, generally underlying the clayey silt to silty clay, is a silty sand till, in which drilling was terminated. The recorded SPT 'N' values indicate a generally very loose to compact zone within the upper 2 to 3 m becoming dense to very dense below the upper relatively weak zone. Although not specifically encountered in the boreholes drilled (except for auger grinding on cobbles in Borehole C5, below El. 104 m), the presence of cobbles and boulders should always be anticipated in glacial till deposits, due to their mode of deposition.

Groundwater levels were observed in the open boreholes while drilling and upon completion of each borehole. In addition, piezometers were installed in Boreholes C6 and C8 to monitor the groundwater levels. The tips of the piezometers were installed within the silty sand till. In these two boreholes, the highest water levels recorded in the piezometers were at Elevations 113.2 m and 115.4 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 C6 and C8 indicate hydrostatic heads of 1.7 m and 1.1 m respectively, 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 southern 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 to silty clay.

## **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 rigid frame, open footing structure on strip footing foundations. In general, the extension of the existing (open bottom concrete) culvert can be carried out with a matching open bottom concrete structure 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 which is favourably suitable for the prevailing site conditions. It will however be difficult to match a circular (or elliptical) corrugated steel structure to the existing rectangular shaped concrete structure. As well, 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. In this instance 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 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 constructing the extensions. In the following sections, therefore, we will discuss both options.

Based on the borehole data and the invert levels of the existing culvert, the founding layer of the existing culvert (assuming a footing depth of 0.9 m) is considered to be the firm to stiff clayey silty to silty clay layer.

#### **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 rigid frame open footing culvert. If a rigid frame open footing culvert must be used, then strip footing foundations for the proposed structure 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, 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. Beyond this point the footings can be extended deeper at a slope of 2.5H:1V.

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 along with an artesian condition emanating from the silty sand till deposit. It is important to ensure that no hydraulic blow-out occurs under the artesian head in the silty sand till underlying the clayey silt to silty clay, with the progress of excavation. Based on the ground conditions encountered in Boreholes C5 and C8, the frost protection depth requirement of 1.5 m, would place the underside of the strip footings for the culvert extensions generally within the clayey silt to silty clay deposit.

At the outlet, the groundwater level measured in Borehole C8 (where the underlying cohesive layer above the till deposit is thinner compared to that of Borehole C5, and hence the potential for blow-out is greater), was at Elevation 113.2 m, i.e. about 1.1 m above the existing original ground surface (o.g.) or 2.7 m above the top of the till deposit (El. 110.5 m). At the existing outlet, based on the results of Boreholes C7 and C8, the elevation of the bottom of the clayey silt to silty clay, or the top of the till deposit appears to

be El. 111.0 m. Allowing for a minimum footing depth of 1.5 m to provide for frost depth, this would result in an excavation bottom Elevation of 112.1 m for the proposed strip footings at the existing outlet location. This results in a FoS against hydraulic heave (base blow-out) of less than 1.0 (i.e. not meeting the minimum FOS required against hydraulic blow-out as per the Canadian Foundation Engineering Manual (CFEM) of about 1.2). Therefore, in order to ensure sufficiently dry excavations, depressurising of the artesian head is required at culvert extension at this end.

At the inlet, the clayey silt to silty clay extends deeper, and based on Boreholes C5 and C6 findings, it appears to extend to about El. 108.5 m but compared to the outlet end (i.e. BH C8) a higher piezometric head was recorded. The piezometric level recorded in Borehole C6 was El. 115.4 m. The culvert invert elevation at this end was given to us as being 114.5 m. Assuming that the footing excavation will extend 1.5 m below this level (i.e. El. 113.0 m), a FoS of 1.1 is obtained by the equilibrium method (ignoring the contribution of shear strength of the clayey silt to silty clay deposit). This safety factor is also below the recommended safety factor of 1.2. In addition, the silty and sandy interbeds may create problems.

In view of the above considerations, dewatering must be capable of reducing the hydraulic head in the silty sand till stratum underlying the clayey silt to silty clay deposit to at least 1.0 m below the intended excavation elevation and should be maintained during the culvert installation.

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.

Assessment of Bearing Resistances: The following geotechnical resistances can be used for the natural clayey silt to silty clay founding stratum, in its undisturbed state, to facilitate the structural design of the culvert extensions.

Factored Geotechnical Resistance at ULS = 180 kPa.

Geotechnical Resistance at SLS = 120 kPa.

These resistances are valid for footings placed on the clayey silt to silty clay deposit with a minimum undrained shear strength of 60 kPa. This should be assured during the construction. 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: The settlements will depend on the actual width of the footings and the actual bearing elevations. However, at this time, 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 18 mm and differential settlements not exceeding 10 mm, provided that the bearing subgrade is not disturbed during the construction. We will be pleased to further elaborate on this when the footings details are known. 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 possible differential settlement of about 15 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 15 mm. This may need to be taken into consideration in assessing the integrity of the existing culvert.

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 25°. 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.

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.

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, it is recommended that the excavation of the upper portion of the founding stratum within the extended 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 114.5 m, a base concrete thickness of 0.4 m and a bedding layer thickness of 0.3 m, is El. 113.8 m or below and based on similar considerations, the founding level for the culvert extension at the outlet end is El. 112.9 m.

Based on the ground conditions encountered in the boreholes, the calculated Factor of Safety (FOS) against blow-out is somewhat more than 1.2, meeting the minimum FOS required against hydraulic blow-out as per the Canadian Foundation Engineering Manual (CFEM) of about 1.2. However, in view of the possibility of silt and sand interbeds under excess piezometric pressure and in order to provide sufficiently dry excavations, depressurising of the artesian head is recommended at both culvert ends. The dewatering 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 loose embankment fill may need to be replaced with granular bedding material extending to the surface of the clayey silt to silty clay deposit at El. 112.1 m or below.

At the interface, the invert level of the box culvert extension 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 can be used for a closed bottom concrete culvert design at Elevation 113.8 m or below at the existing inlet end, and Elevation 112.9 m or below at the existing outlet end, for the clayey silt to silty clay founding stratum, provided that the founding subgrade is undisturbed during construction:

Factored Geotechnical Resistance at ULS = 180 kPa;

Geotechnical Resistance at SLS = 120 kPa;

The above geotechnical resistances are for the cohesive founding stratum, in its undisturbed state. The lower zones of the underlying silty sand till is more competent for bearing resistance and compressibility 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 20 mm and 15 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 15 mm between the existing and the new culvert.

Assessment of Sliding Resistance: The unfactored horizontal resistance against sliding of the close bottom culvert on the clayey stratum can be calculated using an interface friction angle of 25 degrees. 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 OPSS requirements. This type of pipe will need good side support for proper performance (especially an elliptical pipe). This may be difficult to provide in the narrow confines of a shored excavation. 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 suited for the prevailing subsurface conditions at the existing culvert, providing good side support can be ensured. However, a circular or elliptical cross section will be difficult to match with the existing configuration and as well, MTO will unlikely allow the use of a CSP type culvert under Highway 401.

A rigid frame open bottom culvert can also be considered but due to the fact that somewhat more extensive de-pressurization is required in view of the deeper founding levels for the footings, 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 and as well it matches the existing culvert. 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. 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. We recommend that the design side slopes for the widened portion be no steeper than 2H:1V.

In the general vicinity of the existing culvert, as discussed above, the existing embankment is up to about 7.2 m high applying a maximum vertical stress of the order of 140 to 150 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 60 kPa. For foundation stability purposes, this order of shear strength is sufficient for foundation stability in the clayey silt to silty clay deposit and the underlying silty sand till is generally more competent. Therefore no problems with the stability of foundation soils are anticipated provided that all organic, weak or otherwise unsuitable materials are 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 about 20 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 traffic lanes, however, 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 soils 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. As was mentioned before, depending on the site conditions, the first lift at the o.g. level may need to be granular soil and the thickness of the lift may need to be somewhat increased. 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. We recommend however that the degree of compaction within 0.6 m of the final subgrade level be increased to 98%. This is because, based on our experience, this marginally increased compaction effort improves the future performance of the pavement. 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 OPSS, 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 excess piezometric 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) 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 OPSD-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, organic soils or otherwise unsuitable materials soils 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 OPSD-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/m<sup>3</sup>

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/m<sup>3</sup>

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**

Excavatability: 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.

Dewatering and Drainage Issues: 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 and this settlement is expected to taper off within a radius of influence less than 25 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 (up to about four weeks).

Surface drainage measures are also required to ensure sufficiently dry conditions during the proposed works. This is likely to consist of gravity drainage in shallow perimeter ditches and pumping from strategically placed deep filtered sumps. The depressurizing measures to prevent a blow out condition, on the other hand, will likely require deep wells and/or vacuum well points. 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 and de-pressurization 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 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 footings.

Side Slope Excavation Issues: For the inlet and outlet culvert extensions, 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. However, the design of the temporary construction slopes is the responsibility of the Contractor.

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 Type 4 soil below 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.

Shoring will unlikely be required for this project during construction. However, the following parameters are provided for soil types encountered in the boreholes, for the sake of completeness.

**Table 5.6.1 Recommended Unfactored Parameters for Temporary Shoring Design**

Soil Type	Ka	Ko	Kp	$\gamma$ (kN/m <sup>3</sup> )
Granular Pavement Fill (Sand and Gravel).	0.30	0.45	3.3	21.5
Embankment Fill (Silty Sand, some gravel and clay).	0.33	0.50	3.0	20.0
Clayey Silt to Silty Clay.	0.40	0.58	2.4	17.5
Silty Sand Till (top 2 m).	0.32	0.48	3.1	20.5
Silty Sand Till (below 2 m).	0.28	0.43	3.6	22.0

Shoring system, if required, should be designed so that the lateral movement of the portion of the 'roadway protection system' will not exceed the established criterion for the structure performance level. In this case, the required Performance Level is considered to be 2.

The shoring design should be carried out by a Professional Engineer, experienced in this type of Work.

## 5.7 Erosion Protection

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 F, are an integral part of this report.

For and on behalf of Coffey Geotechnics Inc.

  
**Vasantha Wijeyakulasuriya, M.Eng.**  
Principal

  
**Ramon Miranda, P.Eng.**  
Principal



  
**Zuhtu Ozden, P.Eng.**  
Senior Principal



# Appendix E

**List of SP, OPSSs and 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.