

**FOUNDATION INVESTIGATION AND
DESIGN REPORTS - BROOK CREEK
CULVERT EXTENSION
AT STATION 21+720, HIGHWAY 401,
COBOURG, ONTARIO
W.P. 205-00-01, GEOCRES NO.30M16- 43**

AECOM

TRANETOB010434AA-AE
February 3, 2012

February 3, 2012

AECOM

5080 Commerce Boulevard
Mississauga, ON L4W 4P2

Attention: Ms. Peggy Baleka

Dear Madam,

**RE: Foundation Investigation and Design Reports - Brook Creek Culvert Extension at Station
21+720, Highway 401, Cobourg, Ontario, W.P. 205-00-01**

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

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

**FOUNDATION INVESTIGATION REPORT
BROOK CREEK CULVERT EXTENSION
AT STATION 21 + 720
HIGHWAY 401, COBOURG, ONTARIO
W.P. 205-00-01, GEOCREC NO.30M16- 43**

AECOM

TRANETOB010434AA-AE
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**FOUNDATION INVESTIGATION REPORT
BROOK CREEK CULVERT EXTENSION AT STATION 21+720
HIGHWAY 401, COBOURG, ONTARIO
W.P. 205-00-01**

1 INTRODUCTION

As part of the expansion (six laning) of Highway 401, from Burham 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 concrete box culvert at Station 21+720.

Coffey Geotechnics Inc. (Coffey) was retained by AECOM to carry out geotechnical investigations for the widening of this 6.5 km long section of the Highway and the foundation investigation reported herein for the above culvert constitutes part of this project. There are four (4) culverts in this project to be addressed and they will be reported separately as per the project brief and the Brook Creek 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 Brook Creek culvert (size: 4.3 m wide by 2.13 m high, 73.7 m long) is located at Station 21+720 under Highway 401, in the Town of Cobourg as shown in Drawing No. 1. The topography is mildly rolling. Photographs of the site are presented in Appendix C.

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

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

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

The existing culvert is skew to the alignment. No noticeable scour was observed at the inlet, outlet, upstream and downstream of the culvert. The approach embankments to the culvert do not show signs of apparent instability or noticeable erosion. The pavement in the vicinity does not indicate any distress in terms of settlements/unusual cracking or deformations.

3 FIELD AND LABORATORY WORK

The fieldwork for this culvert investigation was performed during the period from June 03, 2010 to December 26, 2010, and consisted of drilling and sampling a total of six boreholes (C9 to C12, C12A and F33). 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 (from Highway 401 median C/L)	Ground Elevation (m)	Drilled Depth (m)	Remarks
C9	21+729	38.0 m Lt C/L (Outside the toe of embankment)	128.5	9.3	Track-mounted rig Hollow stem auger Piezometer installed
C10	21+724	14.0 m Lt C/L (On top of embankment)	134.4	15.5	Truck-mounted rig Solid stem auger
C11	21+705	18.0 m Rt C/L (On top of embankment)	133.9	14.0	Track-mounted rig Solid stem auger
C12	21+720	29.0 m Rt C/L (On the slope of embankment)	130.8	13.8	Track-mounted rig Solid stem auger Piezometer installed
F33	21+ 725	18.0 m Rt C/L (On top of embankment)	134.0	16.8	Track-mounted rig Solid stem auger
C12A	21+707	38.0 m Rt C/L (Outside the toe of embankment)	126.6	9.3	Track-mounted rig Solid stem auger

All boreholes listed in Table 1, except Borehole C10, were advanced using a track-mounted drilling rig owned and operated by Eastern Soil Investigation Limited of Ajax, Ontario. Boreholes C11 and F33 were advanced with the track-mounted rig sitting on plywood planks to protect the road surface. Borehole C10 was advanced using a truck-mounted drilling rig owned and operated by Strong Soil Search Inc. of Claremont, Ontario.

Due to access constraints, initially BH C12 could not be placed close to the culvert outlet. In view of the rapidly changing thickness profile of the clayey silt to silty clay layer as observed in between Boreholes C11 and C12, it was considered important to ascertain this layer thickness more reliably close to the outlet end where the southern part of the proposed widening is planned. After resolving the access issues, BH C12A was located in close proximity to the existing culvert outlet end.

No rock coring was undertaken. No special access provisions (e.g. working platform) were required for the drilling.

The borehole locations were established in the field by Coffey engineering staff, using the stations in the field and in relation to the existing features. The ground elevations and co-ordinates at the borehole locations were measured by the client's surveyors and were provided to Coffey. All boreholes were carried under full-time supervision by 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 0.3 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.

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.

The boreholes were terminated upon reaching SPT spoon refusal, i.e. on reaching three consecutive SPT 'N' blow counts/0.3 m in excess of 100 or extrapolated to excess of 100, in compliance with the general practice for structure foundation investigations.

Groundwater conditions were observed during and on completion of drilling in the open boreholes. Standpipe piezometers were installed in Boreholes C9 and C12 upon their completion. The remaining boreholes were grouted using a cement/bentonite mixture as per MTO procedures. Note the piezometers installed have not been decommissioned as they may be useful in monitoring water level prior 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 representative samples. In addition two consolidation tests were carried out on two relatively undisturbed samples recovered with thin walled open tube samplers (TW) in Borehole C12. The results of the laboratory tests are presented on the appropriate Record of Borehole Sheets in Appendix A, and also in Appendix B, except for the results of the consolidation tests which are given in Appendix B only. However, the consolidation test location depths are noted on the Record of Borehole Sheet for Borehole C12.

4 SUBSURFACE CONDITIONS

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

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

In summary, the encountered subsurface conditions at the culvert location consisted of approximately 1.5 m to 7.6 m thick fill, over natural overburden soils within the depths investigated.

The fill intercepted in Boreholes C10, C11 & F33 (drilled from the embankment shoulders) comprises a pavement fill, underlain by silty sand to sandy silt embankment fill. The pavement fill is 0.4 m to 0.7 m thick.

It is underlain by a silty sand to sandy silt embankment fill, 5.9 m to 6.9 m thick, with trace to some clay and trace of gravel. Some organic matter was intercepted towards the bottom of this fill layer. Borehole C12, drilled from the southern slope face, and Borehole C12A, drilled in the vicinity of the culvert outlet, encountered this embankment fill material beneath a veneer of topsoil. At these locations, this silty sand to sandy silt fill is present to depths of 1.5 m to 3.1 m, with evidence of organic matter as well.

Underlying the embankment fill material in Boreholes C10, C11, C12, C12A and F33, below topsoil in Borehole C9, is a clayey silt to silty clay layer, ranging in thickness from 0.7 m to 5.9 m. The underlying basal layer encountered in all the boreholes, within the investigated depths, is a silty sand till. Augering was terminated in this stratum in all the boreholes upon encountering SPT spoon refusal.

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

4.1 Fill

4.1.1 Pavement Fill

A pavement fill was encountered fill to depths of 0.4 m to 0.7 m (Elevations 133.7 m to 133.5 m) from the surface in Boreholes C10, C11 and F33 drilled from the embankment shoulder.

This fill is granular in nature and consists of sand and gravel to sand, some gravel.

An average SPT 'N' value of 17 blows/0.3 m (SPT 'N' ranging from 11 to 23; 3 test results) was measured within the granular fill, indicating a compact relative density. This indicates that the granular fill had received a systematic compaction during the fill placement.

4.1.2 Embankment Fill – Silty Sand To Sandy Silt

Underlying the pavement granular fill, an embankment fill was intercepted that extended to depths of 1.5 m to 7.6 m (Elevations 127.7 m to 126.8 m) in Boreholes C10, C11 and F33. In Boreholes C12 and C12A, this embankment fill was contacted immediately below a surficial topsoil layer (i.e. at a depth of 0.2 m below the ground surface) and was found to extend to depths of 3.5 and 1.5 m below the ground surface or to El. 127.7 and 125.1 m, respectively.

Grain size analysis was carried out on three 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:	2 - 10%
Sand:	42 - 48%
Silt:	30 - 41%
Clay:	11 - 18%

The fill material consisted of silty sand to sandy silt with trace to some clay and trace of gravel. Hence this fill is classified as a granular (i.e. non cohesive) material.

Standard Penetration Tests performed in the embankment fill yielded an average SPT N-value of 17 blows/0.3 m, (SPT 'N' ranging from 0 to 55, showing low values towards the bottom of the fill; 28 test results). This indicates generally a compact relative density, becoming loose to very loose towards the bottom of the fill. The natural water contents measured on samples recovered from this stratum have an average of 11% (ranging from 6.2% to 55.7%; 30 test results) with the water content also showing an increase generally within the bottom 2 m of this layer probably due to the organic material intercepted.

4.2 Topsoil

A layer of topsoil about 0.2 m thick was encountered in Boreholes C9, C12 and C12A. However, it is to be noted, based on our experience, the thickness of organic rich topsoils frequently varies in between and beyond borehole locations.

4.3 Clayey Silt to Silty Clay

A grey brown clayey silt to silty clay deposit was encountered in all boreholes at depths of 0.2 m to 7.6 m (Elevations 128.3 m to 125.1 m), and extended to depths of 4.6 m to 12.2 m (Elevations 126.3 m to 121.8 m).

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

Gravel:	0 - 2%
Sand:	3 - 18%
Silt:	51 - 64%
Clay:	29 - 39%

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

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

Liquid Limit:	20 – 34%
Plastic Limit:	13 – 22%
Plasticity Index:	6 – 17%

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

Two consolidation tests were conducted on 50 mm dia. samples recovered from Borehole C12 with thin-walled Shelby tube samplers. The results are presented in Figures B4 and B5 in Appendix B. The results indicate a probable pre-consolidation pressure, p'_c , of 200 kPa to 300 kPa. The coefficient of volume change, m_v , have values in the range of 0.28 to 0.38 m^2/MN corresponding to the applied field stress range and are considered to be excessive possibly due to sample disturbance. The test results also indicate preconsolidation pressures (i.e. p'_c) in excess of the existing overburden pressures (i.e. p'_o) of the order of 100 to 200 kPa.

Standard Penetration Tests performed in this stratum yielded an average SPT N-value of 5 blows/0.3 m, (SPT N ranging from 0 to 10; 14 test results). Fifteen field shear vane tests were also carried out within this cohesive soil and these tests yielded undrained in-situ vane shear strengths from 30 kPa to in excess of 100 kPa. Natural water contents measured on twenty-two representative samples recovered from this stratum have an average of 24.9% (ranging from 14.3% to 33.0%). This material has a liquidity index varying from 0.2 to 1.6 with an average of 0.9. This average value 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 field and laboratory test results, the clayey silt/silty clay stratum is considered to be typically somewhat over-consolidated and of firm to stiff consistency.

4.4 Silty Sand Till

Underlying the clayey silt to silty clay layer, a silty sand till deposit was encountered in all boreholes, at depths of 4.6 m to 12.2 m (Elevations 126.3 m to 121.8 m), and extended to the termination of the boreholes at depths of 9.3 m to 16.8 m (Elevations 119.9 m to 117.0 m).

The results of grain size distribution tests conducted on four representative samples and one sample from a gravelly zone intercepted in Borehole C11, are shown in Figure B6 in Appendix B.

Results of representative samples:

Gravel:	7 - 12%
Sand:	45 - 56%
Silt:	26 - 37%
Clay:	10 - 12%

Result of the sample from the gravelly zone:

Gravel:	27%
Sand:	66%
Silt and Clay:	7%

Natural water contents measured on twenty six samples recovered from this stratum have an average of 9.6% (ranging from 5.0% to 23.5%).

Based on visual and tactile observations and grading and moisture information, this stratum is described as a moist to wet, grey, heterogeneous mixture of silty sand with trace to some gravel and clay with occasional gravelly zones, as encountered in Borehole C11. Hence the stratum is classified as a basically granular (non-cohesive) soil. Due to the clay content some cementation can be expected. As encountered in Borehole C9, the presence of cobbles and boulders should always be anticipated in glacial till deposits, due to their mode of deposition.

Standard Penetration Tests performed in this stratum yielded an average SPT N-value of 80 blows/0.3 m, (SPT N ranging from 10 to in excess of 100; 24 test results). Except for the upper 1.0 m of the stratum in Borehole C12, which was found to be loose (a single result), all the remaining SPT 'N' results indicate a compact to very dense condition.

All boreholes were terminated within this stratum at depths ranging 9.3 m to 16.8 m below the ground surface or at Elevations 119.9 m to 117.0 m upon encountering SPT spoon 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 C9 and C12 to monitor the groundwater levels with time, without interference from surface water. The tips of piezometers were installed within the silty sand till.

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

Table 2: Groundwater Level Observations

Borehole No	Ground Surface Elevation (m)	Depth/Elevation of the Tip of Piezometer (m)	Water Level Measurement (most current)		Highest Water Level Measured Elevation (m)
			Depth/Elevation (m)	Date Measured	
C9	128.5	9.3/119.2	+0.8/129.3 (artesian above ground surface)	Oct 15, 2010 (134 days after completion)	130.6 (2.1 m Artesian on June 3, 2010)
C10	134.4	Not installed	5.8/128.6 Cave-in @ 6.1 m	Jul 26, 2010 (on completion)	—
C11	133.9	Not installed	5.8/128.1 Cave-in @ 6.1 m	Jul 8, 2010 (on completion)	—
C12	130.8	9.1/121.7	0.9/129.9	Oct 15, 2010 (101 days after completion)	130.4 (July 22, 2010)
F33	134.0	Not installed	5.8/128.2* Cave-in @ 6.4 m	Jul 8, 2010 (on completion)	—
C12A	126.6	Not installed	0.9/125.7*	Dec 7 2010 (completion)	—

Note: * Groundwater level measured not stabilized.

In general, groundwater levels measured on completion, are not considered to have stabilised, therefore, may not represent the groundwater table at the site. Further, based on the most current observations made in the piezometers in Boreholes C9 and C12, the groundwater level at the site, at the time of the most recent groundwater monitoring, was about 0.8 m to 2.2 m above the existing original ground surface (o.g.) level or approximately at Elevations 129.3 m to 129.9 m respectively.

These observations in Table 2 therefore suggest the existence of an artesian head in the silty sand till underlying the low permeable, clayey silt to silty clay stratum. The highest water level measured in Borehole C9 was about 2 m above the original existing ground level (o.g.).

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 surface water in the surficial 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

BROOK CREEK CULVERT EXTENSION
BOREHOLE LOCATION PLAN
AND SOIL STRATA



SHEET

coffey geotechnics
SPECIALISTS MANAGING THE EARTH



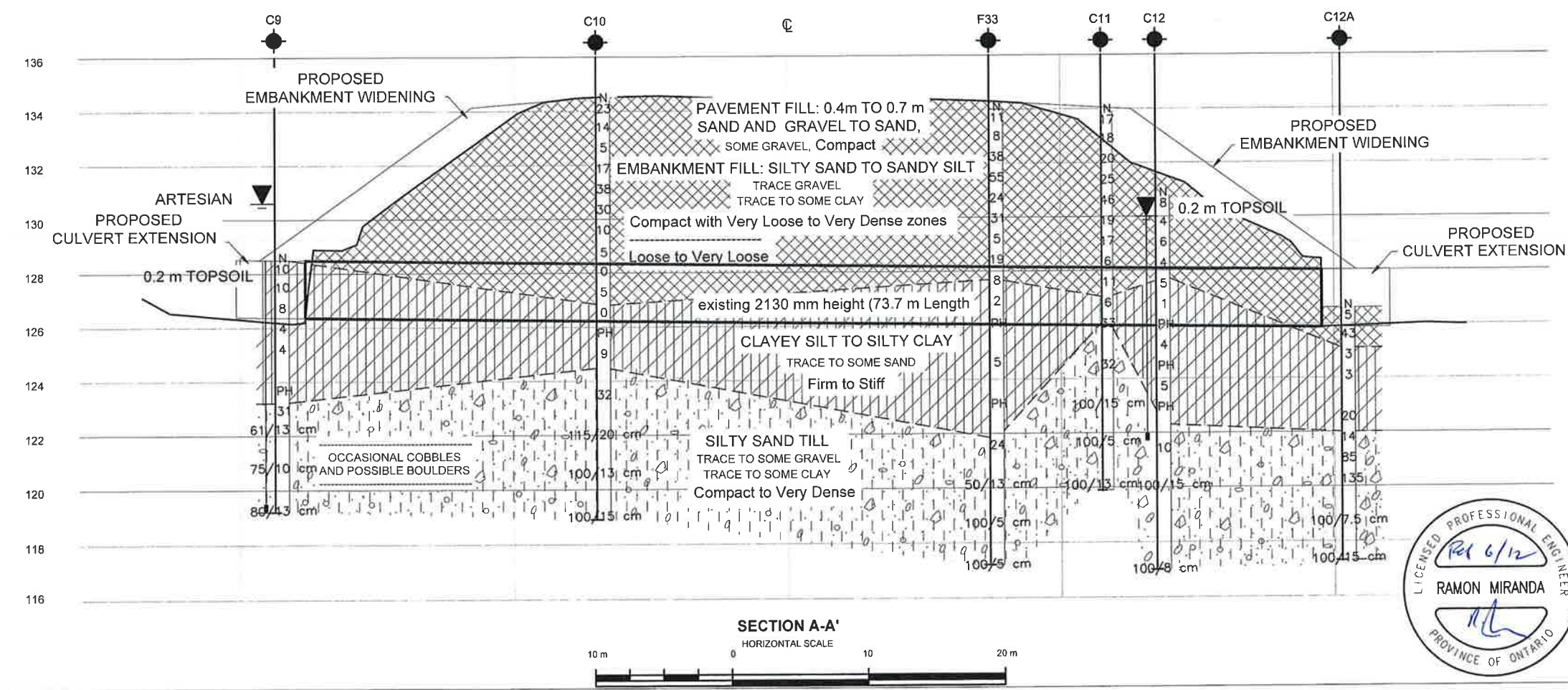
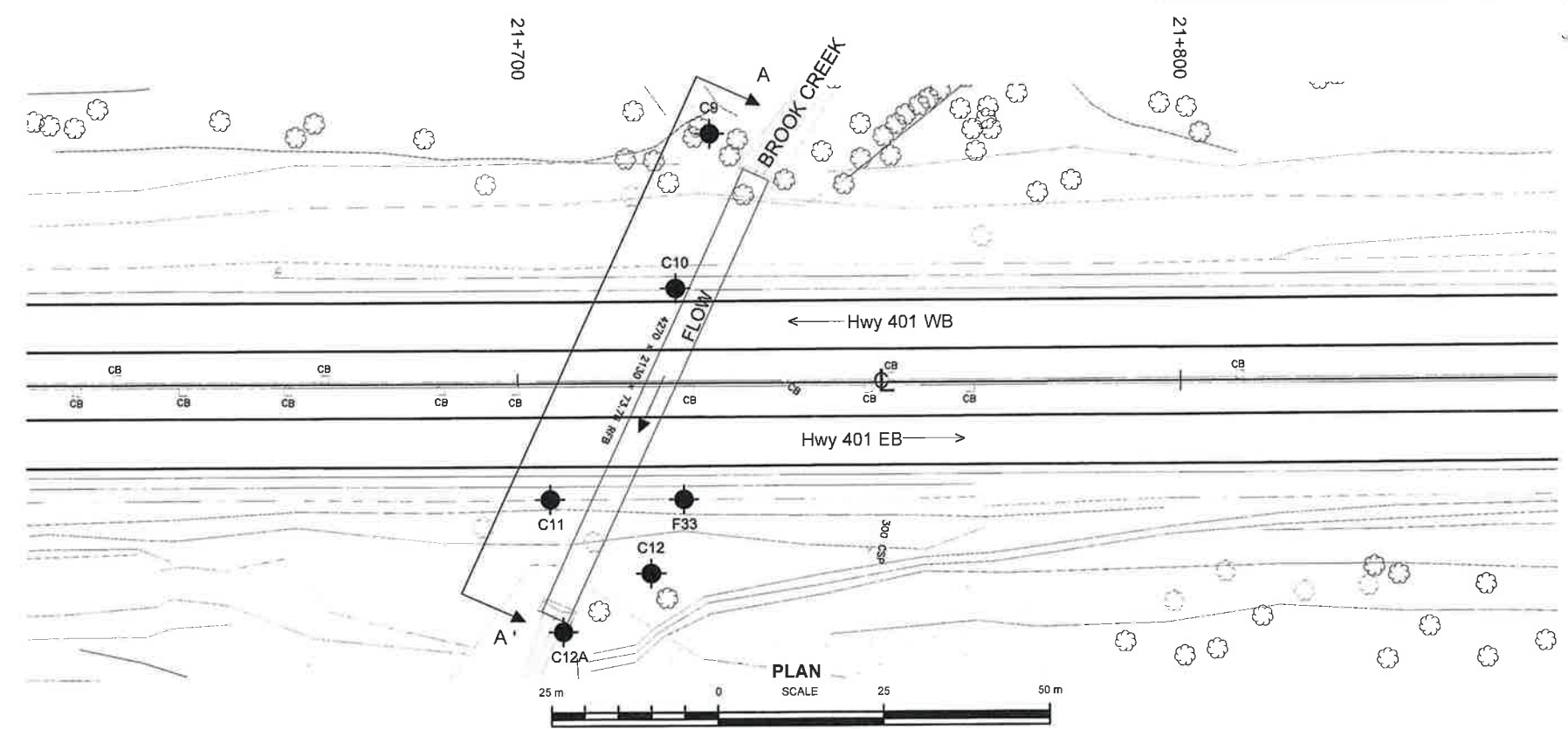
LEGEND			
	Borehole		
	Blows/0.3m (Std. Pen. Test, 475 J/blow)		
	Water Level in Piezometer		
	Piezometer		
	Section		
No.	ELEVATION	EASTING	NORTHING
C9	128.5	414022.5	4873886.6
C10	134.4	414032.7	4873886.9
C11	133.9	414037.3	4873830.0
C12	130.8	414056.2	4873830.3
C12A	128.6	414051.1	4873815.2
F33	134.0	414053.2	4873842.2

-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. 30M16-43				DIST	
TRANETO010434AA				SITE	
SUBMD	CHECKED	DATE	Feb 09, 2011	DWG	
DRAWN	SH	CHECKED	RM	APPROVED	ZO



Appendix A

Record of Borehole Sheets

TRANETOB10431AA Page 28 of 401

RECORD OF BOREHOLE No C9

1 OF 1

METRIC

GWP GWP 205-00-01

LOCATION

Station 21+729 35 m. E of C/L (E 414022.5 N 4673888.6)

ORIGINATED BY RK

DIST HWY 401

BOREHOLE TYPE

Hold on Stern Auge!

COMPILED BY SK

DATUM Geodetic

DATE _____

5/3/2010

CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
REF. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	TEST VALUES			SHEAR STRENGTH (kPa)			
							<div><div>0 20 40 60 80 100</div><div>UNCONFINED + FIELD VANE</div><div>POCKET PENETR x LAB VANE</div><div>20 40 60 80 100</div></div>	<div><div>0 10 20 30</div><div>WATER CONTENT (%)</div></div>			
128.6 0.0	GROUND SURFACE 0.2 m TOPSOIL CLAYEY SILT TO SILTY CLAY In sand, firm to stiff		1	SS	10						
			2	SS	10						
	brown, moist grey, wet		3	SS	2						0 6 62 32
			4	SS	4						
			5	SS	4					19.6	0 3 62 35
	In gravel		6	TW	PH						hardly pushed shelby tube
123.3 5.3	SILTY SAND TILL In gravel, In clay grey, dense to v. dense, wet		7	SS	31					23.5	7 45 37 11 spongy wet
	occasional cobbles and possible boulders		8	SS	61 / 130						auger grinding
			9	SS	7 / 11						auger grinding
119.2 9.3	End of Borehole @ 9.3 m Borehole was open upon completion Water Level @ 4.3 m (not stabilized) upon completion Piezometer installed to 9.3 m Date / Measured Water Level June 03, 2010 / (+) 2.1 m (Artesian Condition) August 19, 2010 / (+) 1.4 m (Artesian Condition) October 13, 2010 / (+) 0.8 m (Artesian Condition)		10	SS	20 / 15						8 68 26 10

$$+^3 \times^3$$

Numbers refer to
Sensitivity

(%) STRAIN AT FAILURE

TRANETCB10434AA Highway 401

RECORD OF BOREHOLE No C10

1 OF 2

METRIC

GWP G.W.P 205-00-01

LOCATION Station 2'-724, 14 m L of CUL (E 414032 / N 4873866 g)

ORIGINATED BY GJ

DIST HWY 401

BOREHOLE TYPE Solid Stem Auger

COMPILED BY WC

DATUM Geodetic

DATE 7/26/2010

CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		UNIT WEIGHT Y kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)				WATER CONTENT (%)
								○ UNCONFINED ● POCKET PENETR	+ FIELD VANE x LAB VANE			
							20 40 60 80 100	10 20 30				
134.4	GROUND SURFACE											
0.0	GRANULAR FILL : Sand and Gravel tr asphalt and concrete pieces brown compact, moist		1	SS	23							
133.7	EMBANKMENT FILL : Silty Sand tr gravel tr to some clay compact with loose to v loose zones		2	SS	14							
0.7			3	SS	5						10 42 30 18	
			4	SS	17							
			5	SS	38							
			6	SS	30							
			7	SS	10							
			8	SS	5							
			9	SS	0							
			10	SS	5							
			11	SS	0							
128.8	CLAYEY SILT TO SILTY CLAY gray, firm, wet		12	TW	PH						spoon wet below 7.6 m G 3 83 34	
7.5	with fine sand lenses brown, stiff		13	SS	9							
			14	SS	32						failed to push vane deeper than 8.9 m due to dense material undisturbed low recovery due to presence of gravel	
124.5	SILTY SAND TILL tr clay tr to some gravel gray, dense to v dense, wet		15	SS	15 / 20							
9.0			16	SS	10 / 13							

Continued Next Page

4 3 X 3

Numbers refer to
Sensitivity

30
15
10

(%) STRAIN AT FAILURE


TRANET081043AAA Highway 401

RECORD OF BOREHOLE No C10

2 OF 2

METRIC

GWP G.W.P. 205-00-01 LOCATION Station 21+724, 14 m LI of C/L (E 414032 7, N 3273455 3) ORIGINATED BY GJ
 DIST HWY 401 BOREHOLE TYPE Solid Stem Auger COMPILED BY WC
 DATUM Geodetic DATE 7/28/2010 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT Y kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
DEPTH DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)					WATER CONTENT (%)
								○ UNCONFINED 20 40 60 80 100	+ FIELD VANE x LAB VANE				
110.4	SILTY SAND TILL tr clay, tr to some gravel	0'	17	SS	119	119			10 20 30				
118.2													
End of Borehole Water level @ 3.8 m (not stabilized)* upon completion Borehole cased in @ 8.1 m upon completion													

TRANET081043AA Highway 401

RECORD OF BOREHOLE No C11

1 OF 1

METRIC

GWP G.W.P. 225-00-01

LOCATION

Station 2+705, 16 m R of C/L (E 44037 3 N 4572830 0)

ORIGINATED BY GJ

DIST HWY 401

BOREHOLE TYPE

Solid Stem Auger

COMPILED BY WC

DATUM Geodetic

DATE

7/8/2010

CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	10 20 30	WATER CONTENT (%)	GR SA SI CL		
133.0	GROUND SURFACE												
130.0	GRANULAR FILL: Sand, some gravel and silt tr. rootlets, brown, compact, moist		1	SS	17								
129.0	EMBANKMENT FILL: Silty Sand to Sandy Silty tr. to some clay tr. gravel gray to grayish brown compact to dense, moist		2	SS	18		133						
			3	SS	20		132						
			4	SS	25		131						
			5	SS	46		130						
			6	SS	19		129						
	with organics		7	SS	17		128						
	loose		8	SS	6		128						2 45 41 12
	some organics, blackish gray tr. wood pieces		9	SS	11		127						
127.0	CLAYEY SILT TO SILTY CLAY brown, stiff, wet		10	SS	6		127						0 4 57 39
125.3	SILTY SAND TILL tr. to some gravel tr. to some clay gray dense to v. dense, wet		11	SS	33		126						12 48 28 12
			12	SS	32		125						spoon wet below 9.1 m
			13	SS	100 / 13 cm		123						27 66 (7)
			14	SS	100 / 13 cm		122						
			15	SS	100 / 13 cm		120						
119.9	End of Borehole Water level @ 5.8 m (not stabilized)* upon completion. Borehole caved-in @ 6.1 m upon completion												

+ 3 x 3

Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

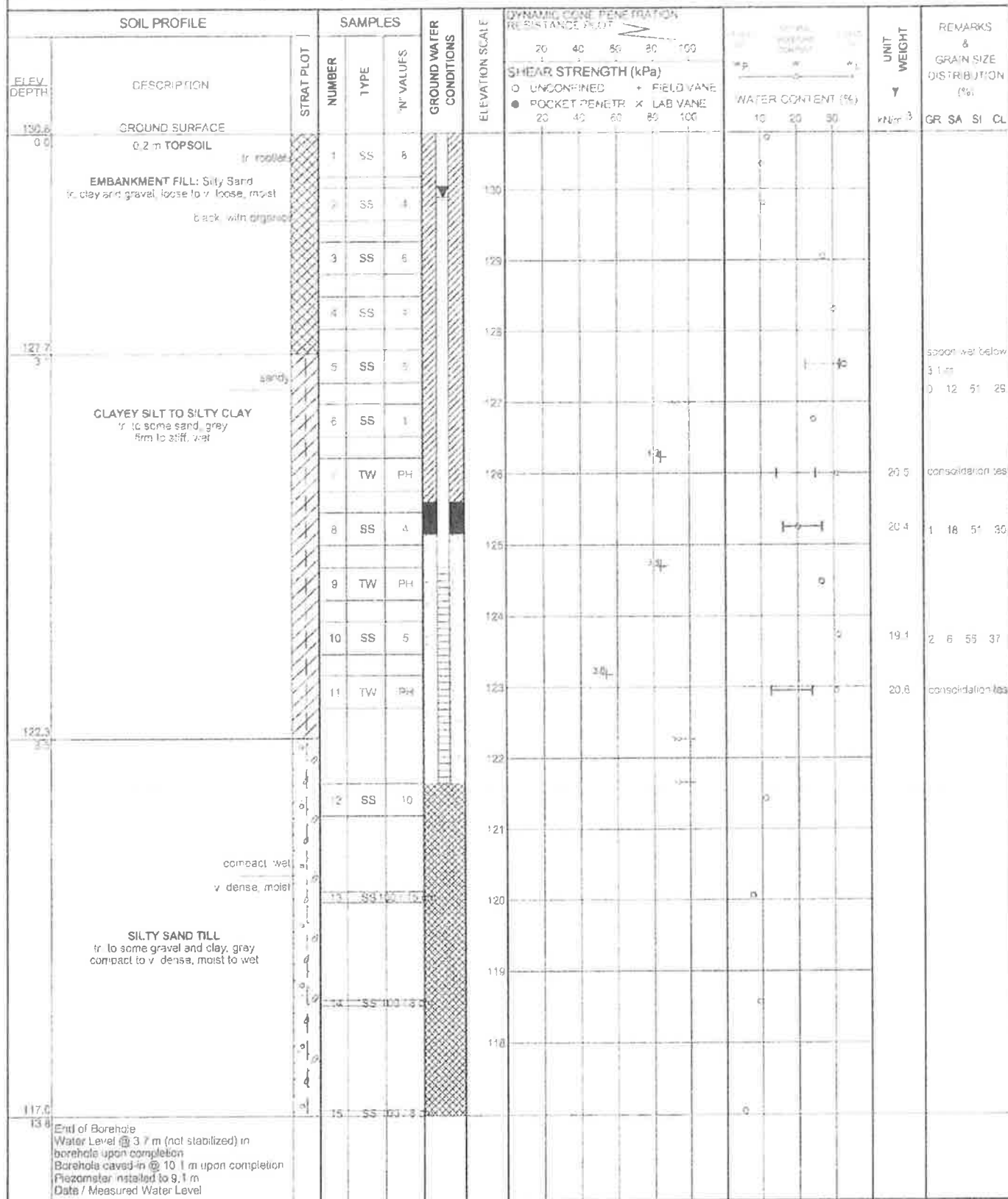
TRANETOBI0434AA Highway 401

RECORD OF BOREHOLE No C12

1 OF 2

METRIC

GWP G.W.P. 205-00-01 LOCATION Station 21+720, 29 m Rt of C/L (E 414056.2, N 4873630.3)
 DIST HWY 401 BOREHOLE TYPE Solid Stem Auger
 DATUM Geodetic DATE 7/9/2010 7/7/2013
 ORIGINATED BY GJ
 COMPILED BY WC
 CHECKED BY ZO



Continued Next Page

+ 3 × 3

Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

TRANETOB10434AA: Highway 401

RECORD OF BOREHOLE No C12

2 OF 2

METRIC

GWP G.W.P 205-00-01 LOCATION Station 21+720, 29 m Rt of C/L (E 414056 2 N 4873630 3)
 DIST HWY 401 BOREHOLE TYPE Solid Stem Auger
 DATUM Geodetic DATE 7/6/2010 7/7/2010

ORIGINATED BY GJ
 COMPILED BY WC
 CHECKED BY ZG

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT Y kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			20 40 60 80 100	10 20 30	10 20 30		
115.5	July 6, 2010 / 3.7 m July 22, 2010 / 0.4 m August 19, 2010 / 0.8 m October 15, 2010 / 0.9 m						SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ● POCKET PENETR x LAB VANE 20 40 60 80 100	WATER CONTENT (%) 10 20 30			

± 3. x 3

Numbers refer to
Sensitivity

20
16
10
(%) STRAIN AT FAILURE

TRANETO810434AA Highway 401

RECORD OF BOREHOLE No C12A

1 OF 1

METRIC

GWP GWP 205.05-01 LOCATION Station 21+707, 38 m Rt of C/L (E 414051.1 N 4873815.2) ORIGINATED BY LG
DIST HWY 401 BOREHOLE TYPE Self Stem Auger COMPILED BY WC
DATUM Geodetic DATE 12/7/2010 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE (kPa)		UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)				WATER CONTENT (%)
								○ UNCONFINED ● POCKET PENETR	× FIELD VANE × LAB VANE			
							20 40 60 80 100	10 20 30				
126.6 0.0	GROUND SURFACE											
	0.2 m TOPSOIL		1	SS	5							
	EMBANKMENT FILL: Silty Sand some organics		2	SS	43*						* A piece of gravel in sample	
125.1 1.1			3	SS	3							
	CLAYEY SILT TO SILTY CLAY											
	grey, soft to firm, moist		4	SS	3							
			5	SS	6							
			6	SS	20							
	some gravel		7	SS	14							
122.0 4.6	SILTY SAND TILL											
	some gravel		8	SS	85							
	grey, compact to v. dense, moist to wet		9	SS	135							
			10	SS	100 / 7.5 cm							
117.3 9.3			11	SS	100 / 15 cm							
	End of Borehole Water level @ 0.9 m (not stabilized)* upon completion Borehole caved-in @ 1.6 m upon completion											

TRANETO10434AA: Highway 401

RECORD OF BOREHOLE No F33

1 OF 2

METRIC

GWP G.W.P 205-00-01 LOCATION Station 21+725, 18 m Rt of C/L (E 414053.2, N 4873842.2) ORIGINATED BY GJ
 DIST HWY 401 BOREHOLE TYPE Solid Stem Auger COMPILED BY WC
 DATUM Geodetic DATE 7/8/2010 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
FLEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	
134.0	GROUND SURFACE						134					
0.0	0.4 m GRANULAR FILL: Sand, some gravel brown, compact, moist		1	SS	11							
	EMBANKMENT FILL: Silty Sand to Sandy Silt tr. gravel, tr. to some clay, brown to grey loose to v. dense, moist		2	SS	8		133					
			3	SS	38		132					4 48 37 11
			4	SS	55							
	tr. organics		5	SS	24		131					
			6	SS	31		130					
	dilatant		7	SS	5		129					
	wet		8	SS	19							
			9	SS	8		128					
127.7 6.3	black with organics		10	SS	2		127					0 5 64 31
	CLAYEY SILT TO SILTY CLAY with silty sand lenses grey, firm to v. stiff, wet		11	TW	PH		126					Shelby tube wet
			12	SS	5		125					
			13	TW	PH		124					
			14	SS	24		123					
			15	SS 50 / 13 cm			122					Failed to push vane further beyond 11.7 m
121.8 12.2	SILTY SAND TILL tr. gravel, tr. to some clay grey						121					9 45 34 12
	compact, wet v. dense, moist						120					

Continued Next Page

+ 3 x 3

Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE



TRANETOB10434AA: Highway 401

RECORD OF BOREHOLE No F33

2 OF 2

METRIC

GWP G.W.P 205-00-01 LOCATION Station 21+725, 18 m Rt of C/L (E 414053.2, N 4873842.2) ORIGINATED BY GJ
DIST HWY 401 BOREHOLE TYPE Solid Stem Auger COMPILED BY WC
DATUM Geodetic DATE 7/8/2010 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
119.0	SILTY SAND TILL tr. gravel, tr. to some clay grey, v. dense		16	SS	100 / 5 cm		119							
117.2							118							
16.8	End of Borehole. Water level @ 5.8 m (not stabilized)* upon completion. Borehole caved-in @ 6.4 m upon completion.		17	SS	100 / 5 cm									

+ 3, x 3

Numbers refer to
Sensitivity

20
15
10

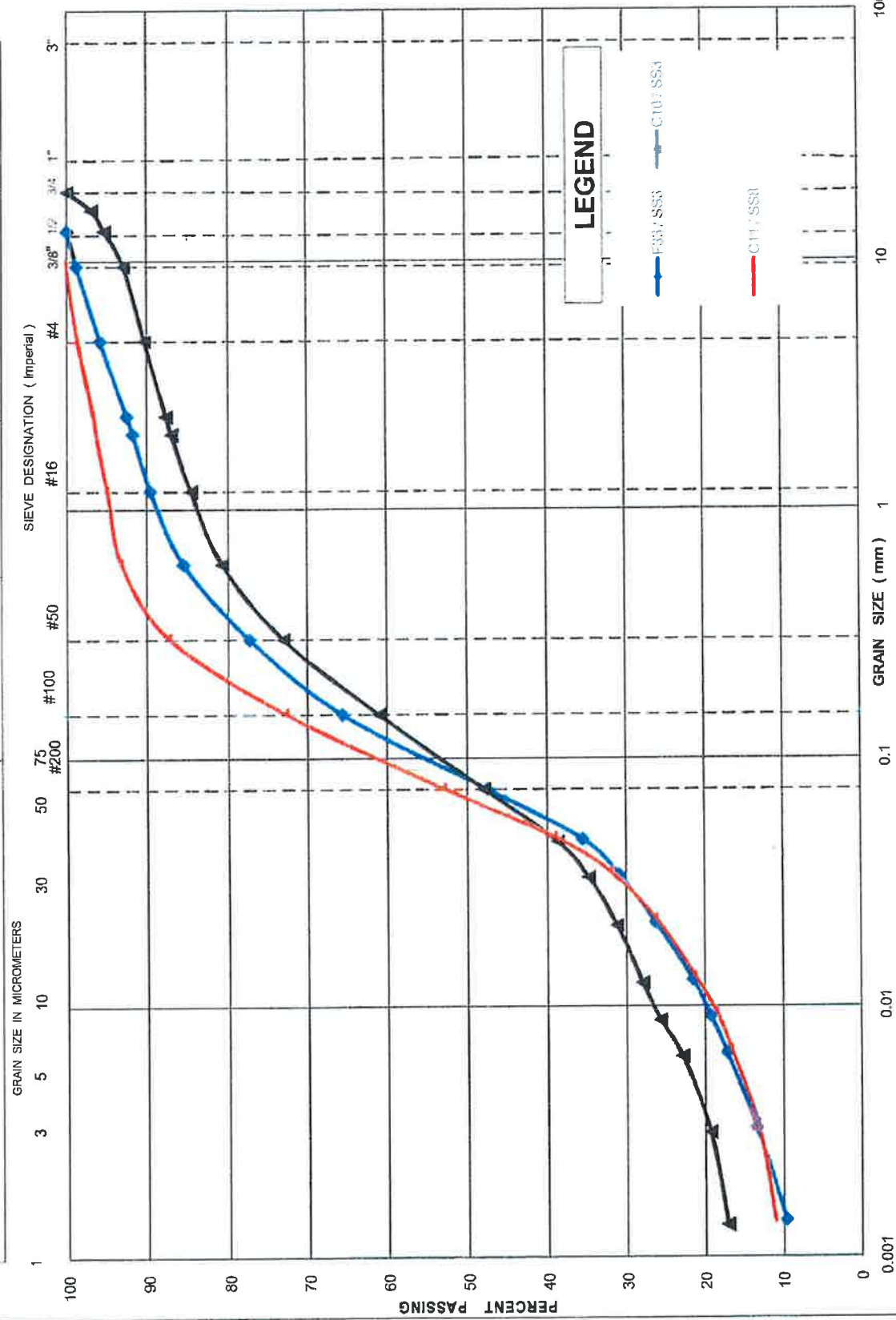
(%) STRAIN AT FAILURE

Appendix B

Laboratory Test Results

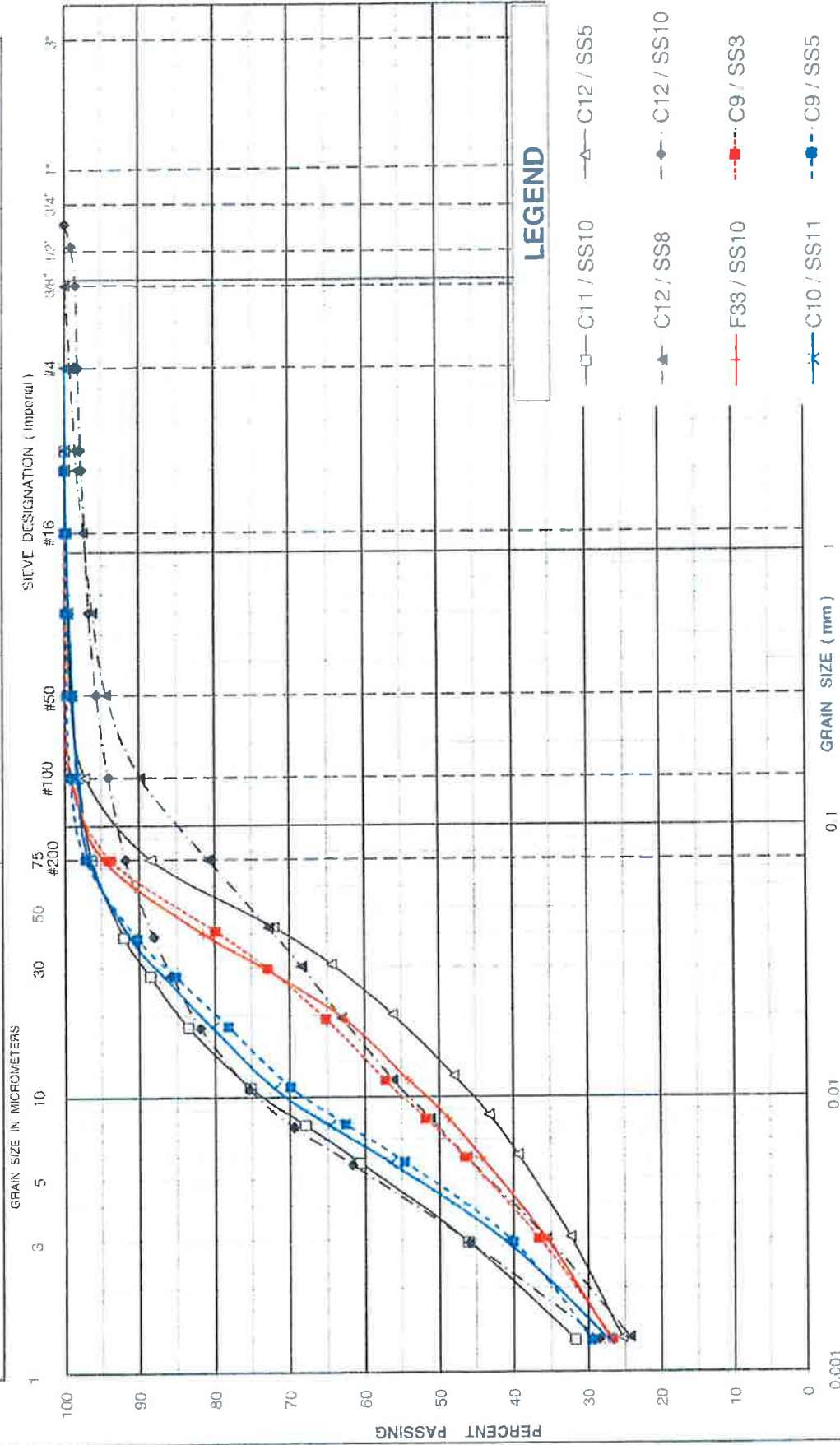
UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT				SAND			GRAVEL		
				Fine	Medium	Coarse	Fine	Coarse	

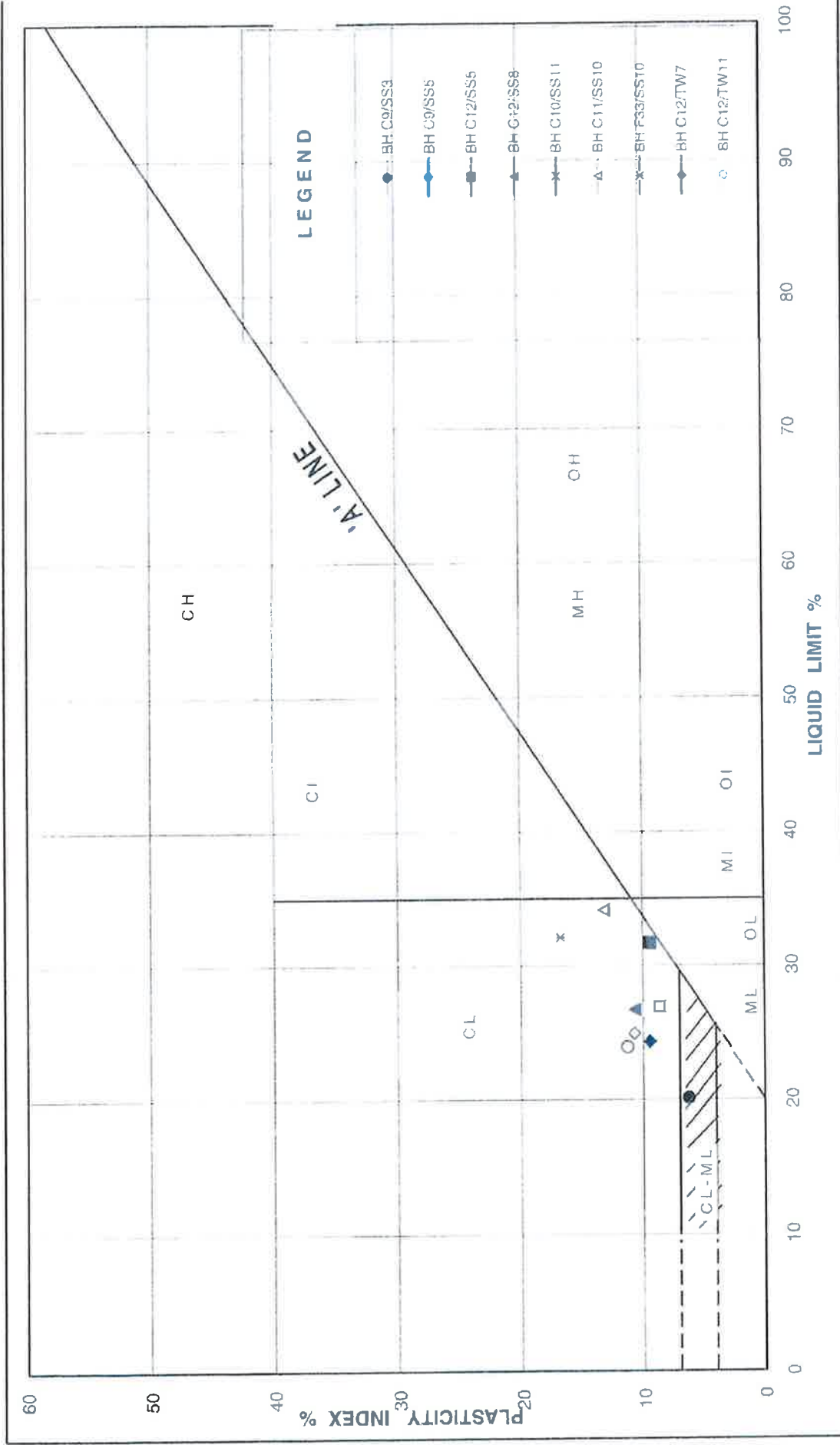


UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT		SAND			GRAVEL		
		Fine	Medium	Coarse	Fine	Coarse	

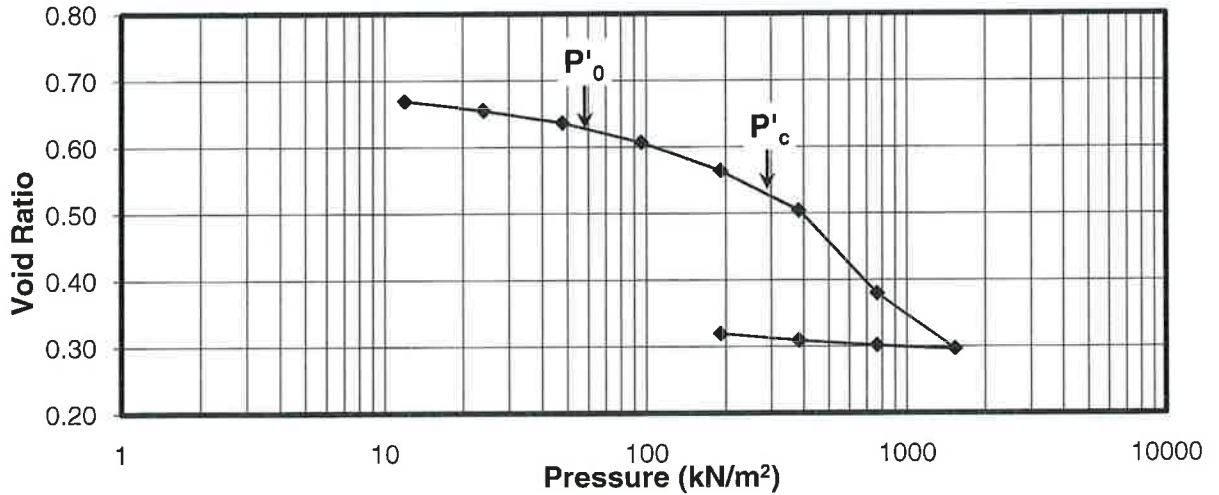


GRAIN SIZE DISTRIBUTION
CLAYEY SILT TO SILTY CLAY

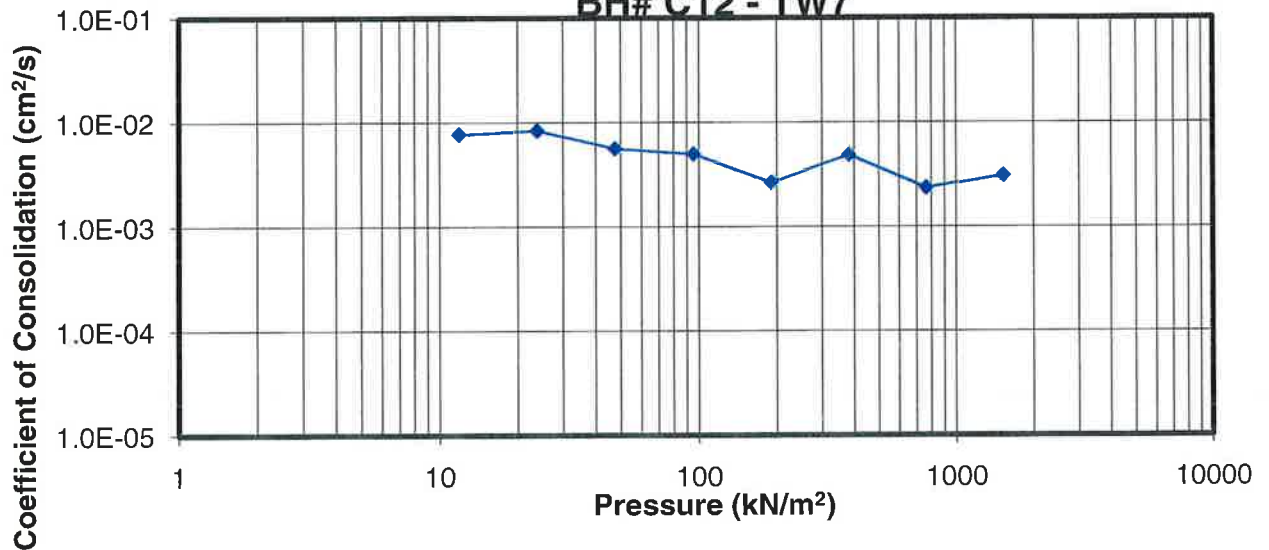



 SPECIALISTS MANAGING THE EARTH	PLASTICITY CHART CLAYEY SILT TO SILTY CLAY	
	FIGURE No.	B3
	REF. No.	TRANETO10434AA
	DATE	OCTOBER, 2010

Void Ratio versus Pressure BH# C12 - TW7



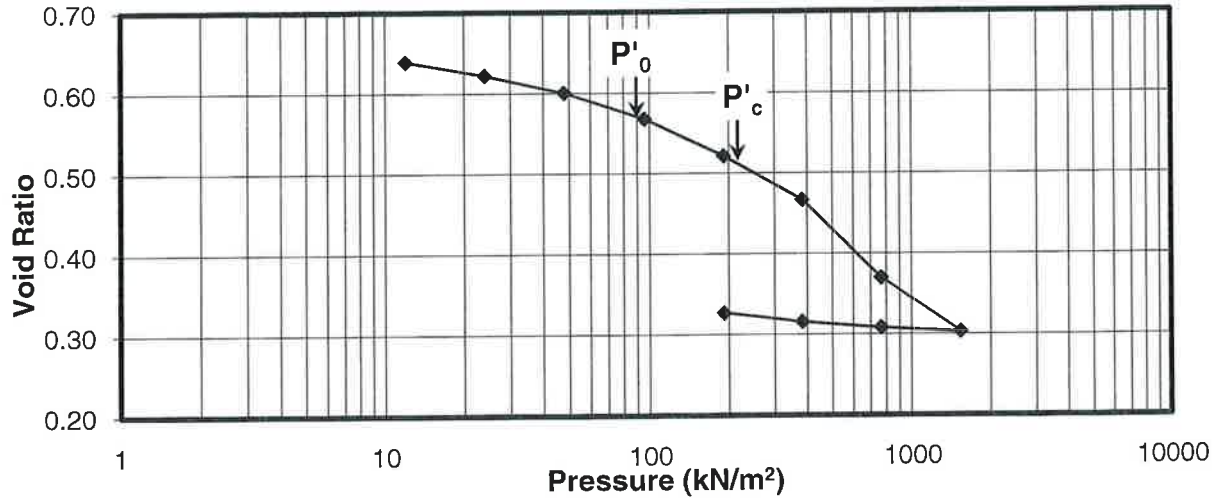
Coefficient of Consolidation vs. Pressure BH# C12 - TW7



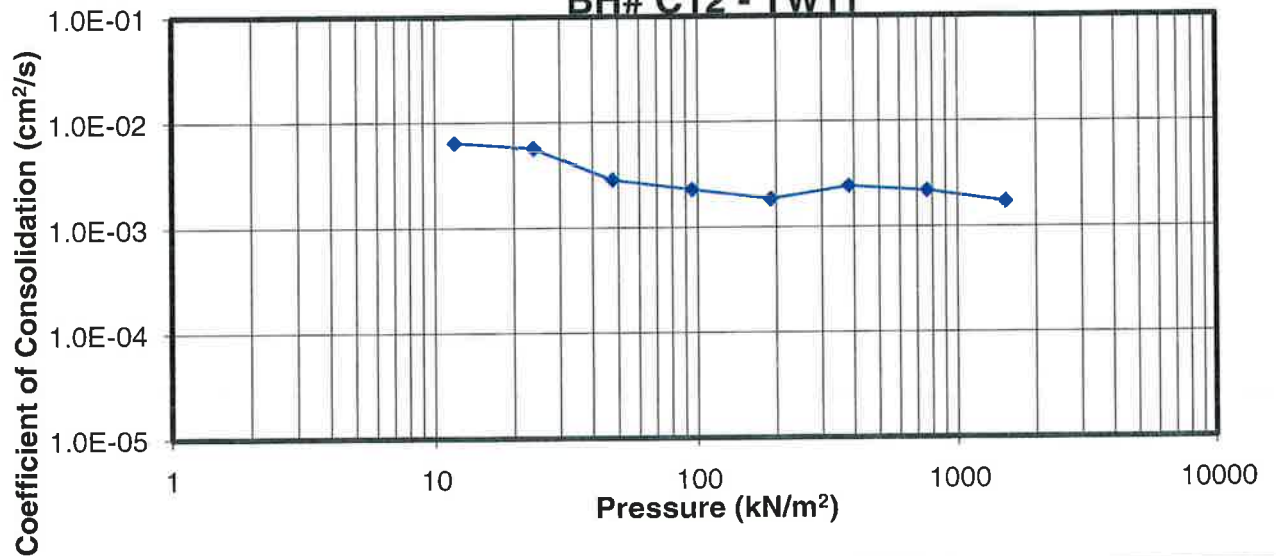
drawn	VW	 coffey geotechnics SPECIALISTS MANAGING THE EARTH	client:	AECOM	
approved	ZO		project:	HIGHWAY 401 EXPANSION	
date	Jan-11			BROOK CREEK CULVERT EXTENSION	
scale	as shown		title:	CONSOLIDATION TEST RESULT - C12 TW7	
original size	Letter		project no:	TRANETOB10434AA	figure no: B4


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Void Ratio versus Pressure BH# C12 - TW11



Coefficient of Consolidation vs. Pressure BH# C12 - TW11

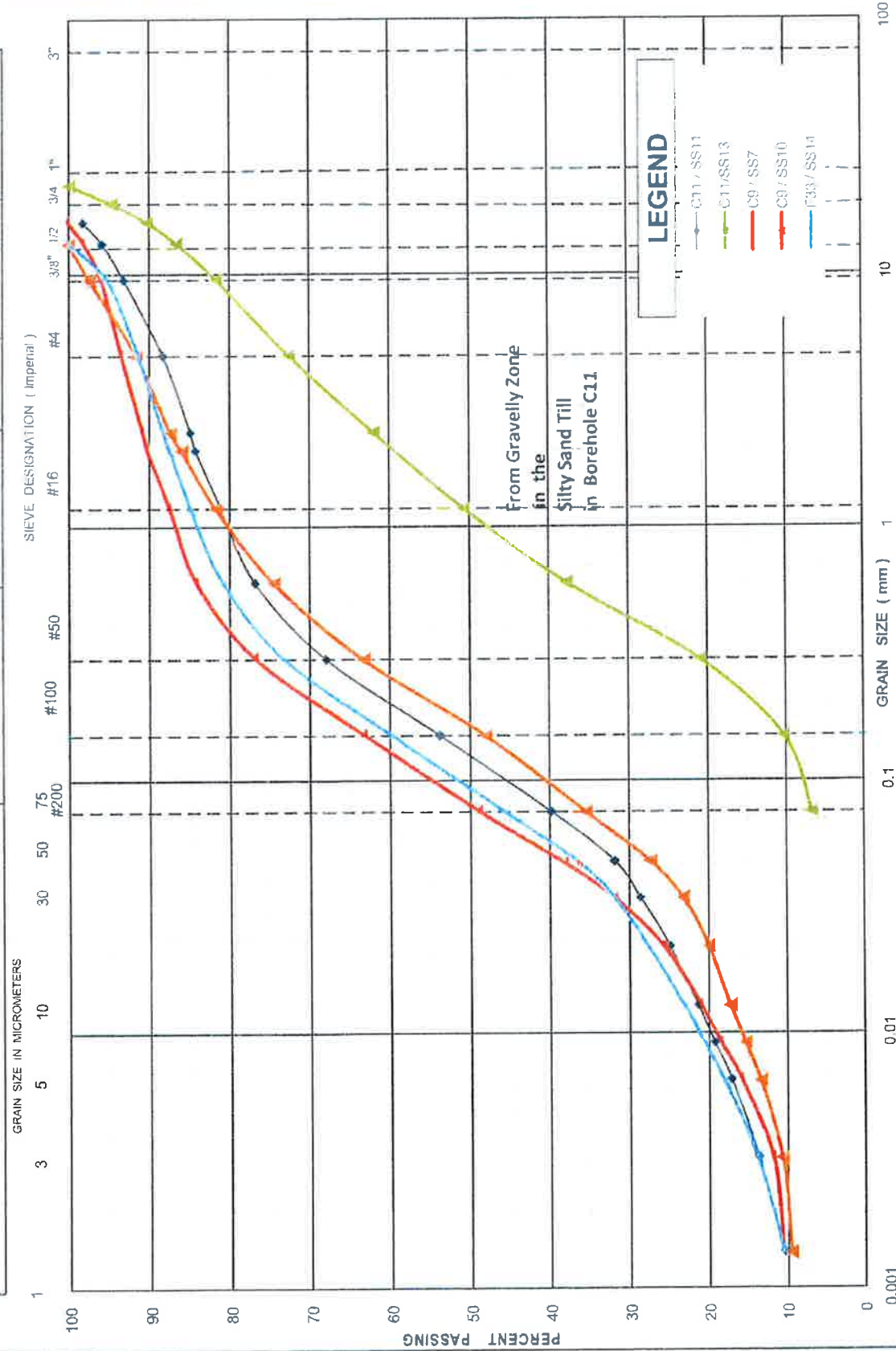


drawn	VW	 SPECIALISTS MANAGING THE EARTH	client:	AECOM	
approved	ZO		project:	HIGHWAY 401 EXPANSION	
date	Jan-11			BROOK CREEK CULVERT EXTENSION	
scale	as shown		title:	CONSOLIDATION TEST RESULT - C12 TW11	
original size	Letter		project no:	TRANETOB10434AA	figure no: B5

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UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT			SAND			GRAVEL		
			Fine	Medium	Coarse	Fine	Coarse	



Appendix C

Site Photographs



Photograph 1. Existing Brook Creek culvert at Station 21+720 EB, south end



Photograph 2. Brook Creek at Station 21+720 EB, looking south



Photograph 3. Station 21+720 EB, looking east



Photograph 4. Station 21+720 EB, looking west

Appendix D

Explanation of Terms Used in Report

EXPLANATION OF TERMS USED IN REPORT

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

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS

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

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

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

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

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

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

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

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

JOINT AND BEDDING

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_{1L}, \epsilon_{2L}, \epsilon_{3L}$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

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

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES	e	1.0	VOID RATIO	e_{min}	1.0	VOID RATIO IN DENSEST STATE
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES	n	1.0	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
ρ_w	kg/m^3	DENSITY OF WATER	w	1.0	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kN/m^3	UNIT WEIGHT OF WATER	S_r	%	DEGREE OF SATURATION	D_n	mm	N PERCENT – DIAMETER
ρ	kg/m^3	DENSITY OF SOIL	W_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ'	kN/m^3	UNIT WEIGHT OF SOIL	W_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
ρ_d	kg/m^3	DENSITY OF DRY SOIL	W_s	%	SHRINKAGE LIMIT	q	m^3/s	RATE OF DISCHARGE
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $(W_L - W_p)$	v	m/s	DISCHARGE VELOCITY
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $(W - W_p) / I_p$	i	1	HYDAULIC GRADIENT
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL	I_c	1	CONSISTENCY INDEX = $(W_L - W) / I_p$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL	e_{max}	1.0	VOID RATIO IN LOOSEST STATE	j	kN/m^2	SEEPAGE FORCE
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL						

**FOUNDATION DESIGN REPORT
BROOK CREEK CULVERT EXTENSION
AT STATION 21+720, HIGHWAY 401
COBOURG, ONTARIO
W.P. 205-00-01, GEOCREC NO. 30M16- 43**

AECOM

TRANETOB010434AA-AE
February 3, 2012

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Appendices

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

Appendix F: Limitations of Report

**FOUNDATION DESIGN REPORT
BROOK CREEK CULVERT EXTENSION AT STATION 21 + 720
HIGHWAY 401, COBOURG, ONTARIO
W.P. 205-00-01**

5 DISCUSSIONS AND RECOMMENDATIONS

As part of the expansion (six laning) of Highway 401, from Burham 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 concrete box culvert at Station 21+720.

The existing culvert is a concrete box culvert (4.3 m wide by 2.13 m high and approximately 74 m long). It is our understanding that this culvert is to be extended by 4.5 m on each side. Six boreholes (C9, C10, C11, C12, C12A & F33) were advanced along or in close proximity to the alignment of the existing culvert, including three boreholes from the embankment shoulders, a northern borehole drilled from the embankment toe area and two southern boreholes, one from the slope face and the other from outside the toe. The borehole layout is 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 culvert foundations, approach embankment issues i.e. within about 20 m of each side of the culvert as well as general design recommendations. Further, our recommendations are based on the following assumptions:

- The new structure extension will be similar and matched to the existing box culvert which has an invert elevation of 126.35 m at the inlet and 125.87 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.
- Marginal or no road elevation change in the vicinity of the culvert.
- The construction of the new lanes will likely coincide with the construction of the culvert extensions on each side and that detour lanes will not be necessary. However, placement of fill for the proposed embankment widenings will not be carried out until the completion of the culvert extensions.

5.1 Geotechnical Characterisation

In summary, the encountered subsurface conditions at the culvert location consisted of approximately 1.5 m to 7.6 m of fill, over natural overburden soils (consisting of silty clay to clayey silt underlain by silty sand till) within the depths investigated, as presented in the Stratigraphic Section in Drawing No. 1.

The embankment fill intercepted in Boreholes C10, C11 & F33 (located on the embankment shoulders) comprises surficial pavement fill underlain by a silty sand to sandy silt embankment fill. Some organic matter was intercepted within the bottom 2 m of the embankment fill, with an associated increase in natural

moisture contents, and the fill displaying a tendency to become very loose to loose. Borehole C12, drilled from the southern slope face and Borehole C12A drilled from the vicinity of the culvert outlet, encountered this fill material beneath topsoil. At these locations, this silty sand to sandy silt fill is present to depths of 1.5 m to 3.1 m. However, it is in a very loose to loose state, with evidence of organic matter as well.

Underlying the embankment fill material in Boreholes C10, C11, C12, C12A and F33 and below the topsoil in Borehole C9, a firm to stiff, clayey silt to silty clay layer, up to about 6 m thick was encountered. This cohesive deposit is considered to be of low permeability, low plasticity and of medium compressibility. The thickness of the silty clay to clayey silt deposit at the borehole locations, varies from 0.7 m at Borehole C11 to 3.1 m, 5.4 m and 5.9 m at Boreholes C12A, C12 and F33 respectively. From this it appears that the thickness of this cohesive deposit diminishes very rapidly from east to west (i.e. significant thickness changes are seen from BH C11 to BH F33 as depicted in Drawing No. 1). This aspect may have some significance in the performance of the culvert and of the embankment, as well as the construction aspects of the extensions. The underlying basal layer encountered is a compact to very dense, wet, silty sand till and appears to be water bearing.

Groundwater levels were observed in the open boreholes while drilling and upon completion of each borehole. In addition, piezometers were installed in Boreholes C9 and C12 to monitor the groundwater levels. The tips of piezometers were installed within the silty sand till. The groundwater observations suggest the presence of an artesian head in the silty sand till, confined under the low permeable, clayey silt to silty clay stratum. In these two boreholes, the highest water levels in the piezometers recorded were at Elevations 130.6 to 130.4 m respectively or about 2 m above the existing o.g. (original ground) level as in the case of Borehole C9.

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

It should be pointed out that the groundwater and surface water levels are 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 surface water in the surficial fills overlying the low permeable, clayey silt to silty clay stratum.

5.2 Culvert Foundations

We understand that the proposed culvert extensions are planned to match the existing box culvert. In general, the extension of the existing box culvert can be carried out with a concrete box culvert or a

matching Rigid Frame Open Bottom 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.

Given the site conditions, from a geotechnical viewpoint, the use of a flexible pipe such as a CSP (corrugated steel pipe) was considered for the project. However, MTO may be reluctant to use a CSP culvert under a major highway, such as Highway 401. If this is the case then the use of a precast concrete box culvert is the recommended option. We understand, however, that AECOM's preferred option is for a rigid frame open footing culvert due to concerns that it would be difficult to maintain temporary flow passage through the existing culvert whilst installing box sections.

Based on the borehole data and the invert levels of the existing culvert, the founding layer of the existing culvert is considered to be the firm to stiff, clayey silt to silty clay layer within the majority length of the culvert except perhaps towards the outlet end where a veneer of fill may overlie the clayey stratum as shown on Drawing No.1.

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

5.2.1 Frost Protection

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

5.2.2 Rigid Frame Open Footing Culvert on Strip Foundations Option

Proposed Founding Levels: As stated earlier, the existing culvert is a box culvert. If a rigid frame open footing culvert must be used, then strip footing foundations for the proposed works should extend below scour/frost depths. The elevation of the underside of the existing box culvert is unknown. It is likely to be founded on bedding material typically 300 mm thick. The top of the proposed strip footings, however, must match the invert elevations at the inlet and at the outlet, 126.35 m and 125.87 m respectively. Assuming a footing depth of 1.4 m for the strip footing of the proposed open bottom culvert extensions, the required minimum frost protection cover could be achieved, founded on 100 mm thick lean mix concrete mud mat. This depth may however not necessarily be sufficient for scour purposes.

As was mentioned before, the site is characterised by high groundwater table and an artesian condition emanating from the silty sand till deposit. It is therefore 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 progress of

excavation. For example, the highest groundwater level measured in Borehole C9 (in close proximity to the extension at the inlet end) was at Elevation 130.6 m, i.e. about 2 m above the existing original ground surface (o.g.). Assuming that the footing excavation extends to El. 124.8 m, the thickness of the remaining clayey silt to silty clay, at the inlet location after excavating 1.5 m to provide for frost protection (i.e. assuming that the footing excavation would extend to El. 124.9 m) would be about 1.6 m. Based on these conditions, the Factor of Safety (FOS) against blow-out is about 0.5, which would imply blow-out of the overlying thin clayey layer. The minimum FOS required against hydraulic blow-out as per the Canadian Foundation Engineering Manual (CFEM) is about 1.2. Therefore in order to ensure stability against blow-out and also to provide sufficiently dry excavations, depressurising of the artesian head is required. It must be capable of reducing the hydraulic head on the silty sand till stratum underlying the clayey stratum to at least 1.0 m below the intended excavation elevation and should be maintained during culvert installation. It should also be pointed out that the thickness of the clayey silt to silty clay layer appears to be highly variable, and in some areas the footings may possibly extend right into the underlying silty sand till layer as evidenced by the findings of Borehole C11.

Estimate of Applied Stress Increment: Based on soil-structure interaction considerations, the estimated maximum increment of applied vertical stress (due to the culvert extension and associated embankment widening) on top of the proposed culvert is 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, the proposed 4.5 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 Resistance: The cohesive stratum which is up to about 6 m thick in the general vicinity would generally constitute the bearing stratum for the open footing culvert extensions. The undrained shear strength of this deposit in the immediate vicinity of the culvert (Boreholes C9, C10, C11, C12, C12A and F33), was measured in-situ with the field vane and found to be between 30 kPa and in excess of 100 kPa (perhaps due to silty sand lenses) but typically about 60 kPa at a depth after providing for frost protection. The underlying silty sand till is generally more competent for bearing resistance purposes, in comparison with the clayey silt to silty clay stratum.

The following geotechnical resistances for this cohesive founding stratum, in its undisturbed state, are recommended for the structural design of the open footing culvert extensions.

- Factored Geotechnical Resistance at ULS = 180 kPa
- Geotechnical Resistance at SLS = 120 kPa.

These resistances are valid for footings placed at El. 125.0 m and below.

The recommended geotechnical resistance at SLS is for a standalone construction without any structural interaction effects to be accommodated. In general, the interaction between the existing culvert and the extension should impose a lower geotechnical resistance at SLS. However, in the present case, the footings of the culvert extension will be founded below the existing box culvert base level. This lower placement, subject to careful construction control, will largely minimise any adverse settlement effect.

Estimate of Settlements: Based on soil-structure interaction considerations, the vertical stress induced by the embankment widening on the culvert extensions is estimated to cause total settlements not exceeding 15 mm and differential settlements not exceeding 10 mm, provided that the bearing subgrade is not disturbed during construction. As will be discussed later in this report, good construction techniques, including dewatering, will be required to achieve this. An allowance should be made to accommodate a differential settlement of about 10 mm between the existing and the new culvert.

The stresses due to embankment widening can also be expected to cause a settlement under the widened portion of the embankment of the existing culvert. This settlement is not expected to be more than 12 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 culvert base on the underlying cohesive stratum can be calculated using an interface friction angle of 24° . 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 and the existing box culvert foundations. 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.

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 or placing the bedding materials. Where unsuitable bearing conditions are observed, remedial procedures can be established in the field to avoid construction delays.

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

5.2.3 Box Culvert Option

The extension of the existing box culvert can be carried out with a similar concrete box culvert. A precast concrete structure is preferred to a cast-in-place concrete structure in view of the former having better durability, ease of installation, faster implementation and especially greater flexibility. 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. For the culvert extension at the southern end, i.e. existing outlet end, the loose silty sand fill with some organics should be replaced with granular bedding to about Elevation 125.1 m, i.e. the top of the clayey silt to silty clay layer. The founding levels based on the reported invert levels at the inlet and the outlet as stated earlier, with provision for 300 mm of granular bedding has adequate Factor of Safety (FOS) against hydraulic blow-out.

At the interface, the invert level of the box culvert extension should match that of the existing box culvert.

Estimate of Applied Stress Increment: The estimated increment of applied vertical stress (due to the culvert extension and associated embankment widening) on top of the native soil is about 50 kPa. This estimate assumes that the existing side slopes are at 2H:1V configuration and the proposed 4.5 m widening will follow the same side slope configuration.

Assessment of Bearing Resistance: The following geotechnical resistances would be available for a closed bottom concrete culvert design at Elevation 126 m or below at the inlet end and at Elevation 125 m or below at the outlet end, provided that the founding subgrade is undisturbed during construction:

- Factored Geotechnical Resistance at ULS = 150 kPa;
- Geotechnical Resistance at SLS = 100 kPa;

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 applied vertical stress increment under the culvert extensions, is estimated to be about 50 kPa. This order of applied stress increment is not expected to cause adverse structural issues from resulting settlements, as discussed next.

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

Estimate of Settlements: Based on the estimated applied vertical stress and provided that the bearing subgrade is not unduly disturbed during the construction, the total and differential settlements should not exceed 10 mm and 5 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 5 mm between the existing and the new culvert.

As was mentioned before the widening of the embankment will cause a settlement of the existing culvert and this was discussed in Section 5.2.2.

Construction Considerations: As stated earlier, at the interface, the invert level of the box culvert extension should match that of the existing box culvert. 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 Option

As was mentioned before, because of its flexibility, a CSP type culvert would be a preferable option for this project. This type of culvert can be placed on a minimum 300 mm thick granular bedding, but the thickness may need to be increased depending on the size of the pipe, as per OPSD requirements. This type of pipe will need good side support for proper performance (especially an elliptical pipe). This type of culvert is well suited for the prevailing subsurface conditions, however, 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 well suited for the prevailing subsurface conditions, however, MTO will unlikely allow the use of a CSP type culvert under Highway 401 and as well it does not match the configuration of the existing culvert.

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

A box culvert, especially a precast concrete box culvert, is also suitable for the existing site conditions 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.

The stability of the approach widenings is not considered an issue given the undrained strengths of the founding cohesive layer discussed earlier and supplemented by borehole data (Borehole F33) in the vicinity of the culvert obtained for the fills. This assumes that all organic, weak or otherwise unsuitable materials will be removed as per MTO standards prior to placing the embankment fills.

The maximum additional vertical stress that will be imposed on the founding layer due to the embankment widening is estimated about 50 kPa. The resulting maximum settlement under the widened approaches is estimated less than 5 mm at Borehole C11 (provided the silty sand till is not unduly disturbed during construction) to about 15 mm at Boreholes C12 and F33 where the clayey deposit is thicker. We do not envisage this order of settlement under the widening to have an adverse impact on the existing traffic lanes, but the paving should be delayed for about four weeks after end of construction to effect majority of the settlements before paving. In other words, we recommend that asphalt paving be implemented no earlier than four weeks after the grade reaches the bottom of asphalt elevation. If necessary, an operational constraint should be included to implement this aspect.

All organic and other unsuitable soils should be removed within an envelope and given by an imaginary slope not 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 existing embankment slope should be implemented during the widening of the embankment, as per MTO procedures and in accordance with OPSD 208.010.

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

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

5.4 Bedding

For precast box culverts, bedding should be in accordance with OPSD803.010 but the bedding should have a minimum thickness of 300 mm. Any requirement for thicker bedding due to prevailing subgrade conditions at the time of construction should be managed through stabilising the subgrade by lowering the artesian head and control of surface water flow. This is necessitated in view of the potential for the

cohesive stratum to blow out under the artesian head as discussed earlier in the report, with deepening excavation and resulting thinning of the cohesive stratum. Subsurface conditions such as in the vicinity of Borehole C11, are susceptible to such construction issues.

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 box culverts (e.g. OPSD 803.010). The bedding material should be compacted to MTO standards (OPSS 501 or SP 105S10) whichever is applicable). Alternatively, the granular bedding could be replaced with unshrinkable fill, which is a self-compacted cementitious material, and should have a minimum of 1 MPa compressive strength. The site should be maintained in a reasonably dry condition for this material to attain sufficient strength.

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 at least 98% of the material's SPMDD. 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 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 to and above the culvert, as per MTO practice. During backfill placement, the height of the backfill should be maintained at approximately 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, weep-holes, etc. should be provided in order to prevent hydrostatic pressure build-up.

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

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

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

Unit weight = 22 kN/m³

Coefficient of Lateral Earth Pressure:

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

Compacted Granular 'B' Type I

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

Unit Weight = 21 kN/m³

Coefficient of Lateral Earth Pressure:

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

Note: Ka is the coefficient of active earth pressure

Ko is the coefficient of earth pressure at rest

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

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

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

5.6 Construction Considerations

It is anticipated that the excavations for the construction comprise the excavations of the existing embankments and stripping of unsuitable soils from beneath the embankment widening areas. 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.

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. As discussed earlier, for the open

bottom culvert option to minimise the potential for hydraulic blow-out, depressurising of the artesian head is required whilst for both options, i.e. the open bottom and the closed bottom options, in order to provide sufficiently dry excavations, the hydraulic head on the silty sand till stratum underlying the cohesive deposit needs to be reduced to at least 1.0 m below the lowest intended excavation elevation and should be maintained during culvert installation.

The settlement due to reducing the hydraulic head on the silty sand till stratum underlying the cohesive deposit to at least 1.0 m below the lowest intended excavation elevation during the construction period is estimated to be not more than 15 mm (for an open footing culvert) and this settlement is expected to taper off within a radius of influence less than 50 m.

Also, on cessation of pumping, when groundwater recharge takes place after the short period of construction of the culvert extensions (say ~ 4 to 6 weeks), part of this settlement would recover due to the local ground water regime returning to its prior equilibrium levels.

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 six weeks).

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

In addition to lowering the artesian head and any perched water, to provide sufficiently dry conditions during the extension of the culvert, it will be necessary to divert the water flowing in the watercourse. This could consist of a 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 could 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 suitable diversion and dewatering.

We recommend that the requirement for dewatering be 'red-flagged' to the Contractor and that the Contractor be asked to submit their diversion and dewatering method including the method of depressurization of the possible artesian head in the till deposit underlying the silty clay to clayey silt layer, 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.

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 proximity to the groundwater table, could influence future settlements of the proposed structures.

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

For both ends, i.e. inlet and outlet extensions, we recommend that the 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.

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-Structures.

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

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

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.

The excavated soils free from topsoil and organics can be used as general construction backfill where it can be compacted with smooth drum type rollers. Loose lifts of soil, which are to be compacted, should not exceed 300 mm. On-site verification of the excavated fill for re-use as backfill by suitably qualified personnel, during construction, would be required. The as excavated moisture contents of the clayey silt to silty clay deposit could be near or even exceed its Liquid Limit and therefore in most cases adequate compaction of this material should not be anticipated. In addition, during wet periods, the embankment fill

and the silty sand till will likely 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 and/or be covered with tarpaulins to help minimize moisture ingress.

For the box culvert option, 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 cohesive layer. The suitability of the existing embankment to carry the loaded crane will need to be determined. This is the contractor's responsibility.

Where excavation support is necessary, the shoring should be designed so that the lateral movement of any portion of the 'roadway protection system' will not exceed the established criterion for the structure performance level. In this case, the Performance Level is considered to be 2. The shoring system should be designed by a Professional Engineer experienced in this type of Work.

Table 5.6.1 Recommended Unfactored Parameters for Temporary Shoring Design

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

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

5.7 Erosion Protection

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 and their performance at the culvert site should be examined for the selection of appropriate scour and erosion protection schemes.

Consideration may be given to the construction of a headwall at both inlet and outlet locations to reduce the potential for erosion.

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 seal at the inlet and outlet.

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

We recommend that a concrete cut-off (apron) wall 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 also be given to providing a concrete apron at the outlet. Beneath the culvert, the concrete cut-off wall should extend to a suitable depth (e.g. below any possible scour depth).

In addition to the cut-off wall, at the inlet, consideration may 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. 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 m beyond the inlet.

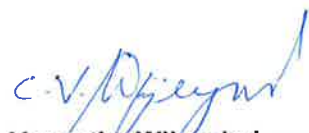
At the outlet as well as at the inlet, in addition to the concrete cut-off and/or low permeability seal, a 0.6 m thick rock protection, consisting typically of 300 mm size rock can be considered.

Another reference for consideration is OPSD 810.010 Rip-Rap Treatment for Culvert Outlets.

6 CLOSURE

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

For and on behalf of Coffey Geotechnics Inc.



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Appendix E

List of SP, OPSSs 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.