

**FOUNDATION INVESTIGATION AND DESIGN REPORTS
PROPOSED JOHNSON CREEK CULVERT REPLACEMENT
HIGHWAY 17 WEST OF HIGHWAY 638, ONTARIO
WP 5271-08-01 SITE NO. 38S-404/C
G.W.P. 5271-08-00
MTO GEOCRETS NO. 41K-92**

Prepared for:

MCINTOSH PERRY CONSULTING ENGINEERS

By:

SPL CONSULTANTS LIMITED

Project: 750-1001 (Johnson Creek)
January 2013



SPL Consultants Limited
Geotechnical Environmental Materials Hydrogeology

146 Colonnade Road
Ottawa, Ontario K2E 7Y1
Tel: 613.228.0065 Fax: 613.228.0045

PART A
FOUNDATION INVESTIGATION REPORT
PROPOSED JOHNSON CREEK CULVERT REPLACEMENT
HIGHWAY 17 WEST OF HIGHWAY 638, ONTARIO
WP 5271-08-01 SITE NO. 38S-404/C
G.W.P. 5271-08-00
MTO GEOCRES NO. 41K-92

Prepared for:

MCINTOSH PERRY CONSULTING ENGINEERS

By:

SPL CONSULTANTS LIMITED

Project: 750-1001 (Johnson Creek)
January 2013



SPL Consultants Limited
Geotechnical Environmental Materials Hydrogeology

146 Colonnade Road
Ottawa, Ontario K2E 7Y1
Tel: 613.228.0065 Fax: 613.228.0045

Table of Contents

PART A: FOUNDATION INVESTIGATION REPORT

1. INTRODUCTION	1
2. SITE DESCRIPTION.....	1
3. INVESTIGATION PROCEDURES	1
3.1 Desktop Study	1
3.2 Field Investigation	2
3.3 Laboratory Testing	2
4. SUBSURFACE CONDITIONS.....	2
4.1 Soil Conditions	3
4.1.1 Granular Fill.....	3
4.1.2 Sand	3
4.1.3 Silty Clay	4
4.1.4 Sandy Silt.....	5
4.1.5 Auger Refusal	5
4.2 Groundwater Conditions	5
4.3 Summary	6
5. CLOSURE.....	6

Drawings

No.

Site Plan	1
Borehole Location Plan and Soil Strata	2
Grain Size Distribution - Fill	3
Grain Size Distribution - Sand	4
Grain Size Distribution – Silty Clay	5
Grain Size Distribution – Sandy Silt	6

Appendices

Appendix A:	Borehole Logs (Record of Borehole Sheets)
Appendix B:	Oedometer Test Results
Appendix C:	Chemical Test Results
Appendix D:	Explanation of Terms used in Report

1. INTRODUCTION

SPL Consultants Limited (SPL) was retained by McIntosh Perry Consulting Engineers to conduct a foundation investigation as part of the proposed culvert replacement at Johnson Creek on Highway 17 approximately 8 km west of Highway 638, near Portlock, Ontario.

The Terms of Reference (TOR) for this investigation are outlined in the Request for Quotation (RFQ) issued by the Ministry of Transportation (MTO) under Agreement No. 5010-E-0001 dated April, 2010 and SPL's subsequent proposal No. P10.06.018 dated June, 2010.

The purpose of the foundation investigation was to obtain subsurface information at the site by means of exploratory boreholes. This report presents the findings of the foundation investigation carried out at the site, as well as general comments and recommendations for the design and construction of the proposed culvert replacement.

As part of this project a geotechnical (pavement) investigation was also carried out at the site concurrent with the foundation investigation. The results of the pavement investigation are presented under separate cover.

2. SITE DESCRIPTION

The site is located on Highway 17 approximately 8 km west of Highway 638, near Portlock, Ontario. The elevation of the natural ground in the immediate vicinity of the Johnson Creek crossing is approximately 178.5 m to 179 m. The elevation of the highway (top of pavement) at the crossing is approximately 183 m (the embankment is approximately 4 m to 4.5 m high at the crossing). Johnson Creek itself flows within a small stream course approximately 3.5 m to 5 m wide. At the time of the site survey the water level in the creek was at an elevation of approximately 177.7 m.

The existing culvert is a single span 3.2 m wide CSP culvert with a fill cover of approximately 2.5 m. The approximate length of the existing culvert is 35 m.

3. INVESTIGATION PROCEDURES

The foundation investigation was carried out in July, 2011. The scope of work for this assignment included a desk study, field investigations, laboratory testing, analysis and preparation of this report.

3.1 Desktop Study

Surficial geology in the area comprises glaciolacustrine deposits (silt and clay with minor sand) as well as glacial till (undifferentiated predominantly sand to silty sand matrix with cobbles and boulders). Bedrock geology maps of the general area indicate the bedrock to be conglomerate, sandstone, siltstone and argillite of the Gowganda formation.

3.2 Field Investigation

Field investigations were carried out on July 14, 2011 and included drilling two boreholes at the crossing location (BH-1 and BH-2). As mentioned previously, additional shallow boreholes were advanced at the same time for the geotechnical (pavement) portion of the work; the results of these boreholes are submitted with the geotechnical (pavement) investigation report under separate cover.

The boreholes were advanced using a truck-mounted drill rig supplied and operated by Groundwork Drilling Inc. of Etobicoke, ON. The boreholes were advanced using hollow-stem auger drilling as well as Dynamic Cone Penetration Testing (DCPT) to the depth of 12.8 m below the existing ground surface. During drilling, sampling and in-situ testing [including Standard Penetration (SPT) Testing, and DCPT testing] were carried out at regular intervals.

A standpipe piezometer was installed in Borehole BH-1 to allow for subsequent measurement of stabilized groundwater levels at the site. All boreholes were backfilled with bentonite and soil cuttings and were sealed at the ground surface. All boreholes were drilled and abandoned in accordance with Ontario Regulation 903.

Borehole locations are shown in Drawing 2. Borehole records are included in Appendix A.

3.3 Laboratory Testing

During drilling and in-situ testing, soil samples were obtained for further examination and classification. A laboratory testing program, including determination of natural water content, Atterberg limits, grain size distribution (sieve and hydrometer) and chemical analyses, was carried out on selected representative soil samples. A single relatively undisturbed (Shelby tube) sample was subjected to oedometer testing in order to determine consolidation properties.

The results of natural water content tests and Atterberg limits testing are included on the relevant borehole logs in Appendix A. The results of determination of grain size distribution tests are summarized on the individual borehole logs and are presented in Drawings 3 through 6. The results of Oedometer testing are included in Appendix B

Chemical testing to determine sulphate content, chloride content, pH and soil resistivity and electrical conductivity was also carried out on selected soil samples obtained during drilling. The results of these tests are included in Appendix C.

4. SUBSURFACE CONDITIONS

The subsurface conditions at the site are discussed in the following sections. Detailed descriptions of the soil and groundwater conditions encountered at each of the borehole locations are included in the individual borehole logs in Appendix A.

4.1 Soil Conditions

Detailed descriptions of each of the soils encountered during drilling are included in the borehole records included in Appendix A.

4.1.1 Granular Fill

All of the boreholes drilled as part of this investigation were drilled on the unpaved shoulder of existing highway; asphalt was not encountered.

The uppermost soil encountered in the boreholes was the granular fill which forms the existing embankment, pavement structure and culvert backfill.

The uppermost portion of the fill was found to be dense to very dense (as inferred from SPT “N” values) sand and gravel which extended to approximately 0.8 m below the existing pavement surface, and is assumed to be the part of the pavement structure. Underlying this material the lower fill was found to be gravelly sand. The sand was found to be in a compact state (again based on SPT “N” values). The grain size curve for a selected sample of this fill is presented in Drawing 3. A summary of the grain size distribution is also presented in Table 1 below. It should be noted that this grain size distribution test was carried out on a sample obtained through SPT testing, which does not recover coarse gravel, cobble and boulder sized particles. Because of this it is possible that the grain size distributions measured may be finer overall than portions of the materials in the field. Although the 200 mm boreholes advanced during the field investigation did not encounter significant numbers of cobbles and boulders, they may be present in the fill material and could be encountered during construction.

Table 1 – Results of Grain Size Analyses for Fill Material

Borehole No.	Sample No.	Grain Size Distribution		
		% Gravel	% Sand	% Silt & Clay
BH-2	SS-3	31	61	8

The fill material extended to a depth of 4.1 m to 4.3 m below the existing road surface in the boreholes drilled as part of this investigation. This corresponds to elevations of 178.8 m to 178.6 m.

4.1.2 Sand

A layer of coarse sand was encountered in Borehole BH-1 immediately below the granular fill. This sand extended from 4.1 m to 6.3 m below existing pavement surface (2.1 m in thickness). The grain size distribution of a sample of this layer is analyzed and presented in Drawing 4. It is also summarized in Table 2 below.

Table 2 – Results of Grain Size Analyses for Native Sand

Borehole No.	Sample No.	Grain Size Distribution		
		% Gravel	% Sand	% Silt & Clay
BH-1	SS-6	15	82	3

The consistency of the sand based on SPT “N” values would be described as loose. DCPT “N” values indicate a compact consistency, which may suggest some disturbance of the sand during SPT testing, possibly due to drilling disturbance in the base of the hole due to inflow of water into the augers.

This sand layer was not encountered in BH-2 drilled on the south side of the culvert, and therefore may or may not be present below the culvert itself.

4.1.3 Silty Clay

The sand layer (in BH-1) and granular fill (in BH-2) are underlain by a layer of silty clay which was encountered in both boreholes. The thickness of the silty clay layer was found to be approximately 3.9 m and 7.9 m in boreholes BH-1 and BH-2, respectively

Grain size analyses were performed on two samples of the silty clay deposit. The results of these tests are presented on respective borehole logs as well as Drawing 5, and are summarized in Table 3 below.

Table 3 – Results of Grain Size Analyses for Native Silty Clay

Borehole No.	Sample No.	Grain Size Distribution			
		% Gravel	% Sand	% Silt	% Clay
BH-1	SS-8	0	3	35	62
BH-2	SS-6	0	4	60	36

The results of Atterberg limit testing on two samples of the silty clay indicate a medium to high plasticity (see borehole records included in Appendix A).

Detailed soil descriptions are included in the borehole records included in Appendix A. SPT and DCPT “N” values are included in the borehole records as well as on Drawing 2.

A single sample of the silty clay soil was subjected to oedometer testing to determine the consolidation properties of the soil. The results of this test are included in Appendix B, and are summarized in Table 4 below.

Table 4 – Summary of Consolidation Properties for Silty Clay

Borehole/ Sample No.	Depth	Preconsolidation Pressure σ_p' (kPa)	Existing Effective Stress σ_{v0}' (kPa)	Compression Index C_c	Recompression Index C_r	Initial Void Ratio e_0	Over Consolidation Ratio OCR
BH-1 Sample 9	9.1 m	150	95	1.1	0.34	1.86	1.58

4.1.4 Sandy Silt

The native silty clay is underlain by a layer of sandy silt. This deposit was encountered at a depth of 10.2 m and 12.2 m below the existing road surface (at an elevation of 172.2 m to 170.7 m) in Boreholes BH-1 and BH-2, respectively, and extends to the depth of drilling at both locations.

The grain size distribution of a representative sample from this layer is presented with the borehole logs and also on Drawing 6. The grain size distribution is also summarized in Table 5 below.

Table 5 – Results of Grain Size Analyses for Sandy Silt

Borehole No.	Sample No.	Grain Size Distribution			
		% Gravel	% Sand	% Silt	% Clay
BH-1	SS-10	0	20	69	11

The sandy silt is in a compact to dense state based on SPT and DCPT “N” values.

4.1.5 Auger Refusal

Auger refusal was not encountered in either of the boreholes drilled as part of this.

4.2 Groundwater Conditions

Groundwater was noted below a depth of approximately 4 m in both of the boreholes drilled at the site. A standpipe piezometer was installed in BH-1 near the bottom of the borehole in the sandy silt layer.

The groundwater level at the site was measured the day after completion of drilling and found to be at an elevation of 181.4 m. This is approximately 3.7 m higher than the level of the creek at the time of investigation, as well as the existing natural ground, which would indicate the presence of artesian water pressures within the silty sand layer encountered at 10 m to 12 m depth. It is possible that the silty clay layer acts as a low-permeability confining layer causing increased pore pressures within the underlying sandy silt.

Groundwater levels in the upper granular materials (both the fill as well as the coarse sand layer encountered in Borehole BH-1) would be expected to be similar to the water level in the creek, as these materials are relatively permeable and are likely to be hydraulically connected to the creek.

It should be noted that the groundwater levels can vary and are subject to seasonal fluctuations as well as fluctuations in response to major weather events, and in particular for this site, in response to changes in the level of the creek. If construction is carried out at a time when the creek level is higher than the level in July, 2011, a corresponding increase in groundwater levels should be anticipated, particularly in the fill and sand layers.

4.3 Summary

A summary of the soil and groundwater conditions encountered at the Johnson Creek crossing location is presented in Table 6 below.

Table 6 – Simplified Stratigraphy and Groundwater Elevations

Borehole No.	Ground Surface Elevation	Simplified Stratigraphy (Depth)				Measured Groundwater Elevation
		Granular Fill	Native Coarse Sand	Silty Clay	Sandy Silt	
BH-1	182.9	0.0– 4.1 m	4.1 – 6.3 m	6.3 – 10.2 m	10.2 – 12.8 m	El. 181.4 m
BH-2	182.9	0.0– 4.3 m	--	4.3 – 12.2 m	12.2 – 12.8 m	--

5. CLOSURE

Field investigations for this project were supervised by Naeem Ehsan, P.Eng. This report was prepared by Mr. Chris Hendry, P.Eng. Mr. Fanyu Zhu, P.Eng., SPL's project manager and designated MTO Contact, and Mr. Shaheen Ahmad, P.Eng., SPL's quality control auditor provided independent review and quality control.

SPL CONSULTANTS LIMITED



Chris Hendry, M.Eng., P.Eng.

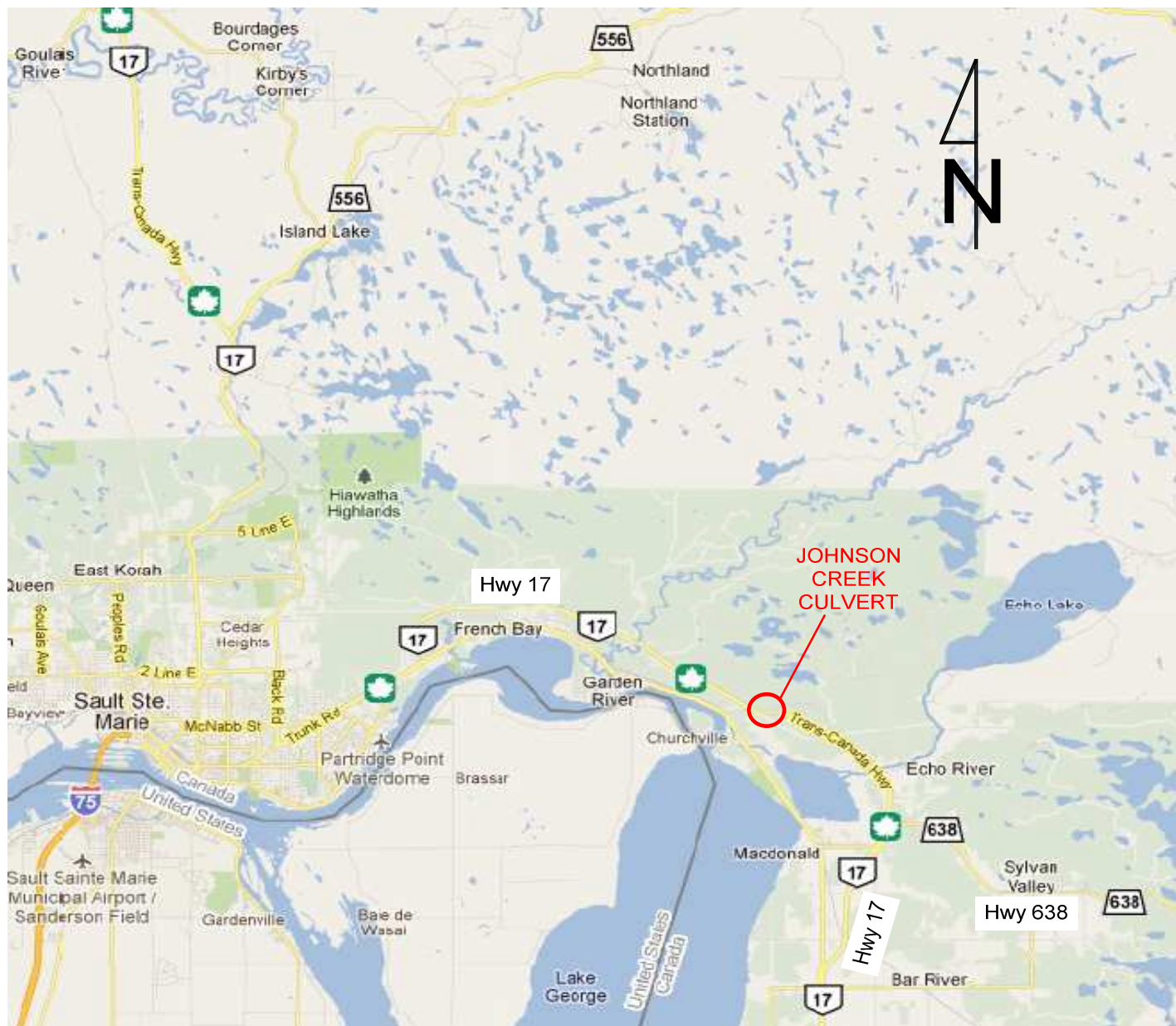




Fanyu Zhu, Ph.D., P.Eng.




Shaheen Ahmad, M.A.Sc., P.Eng.

Drawings



Client: McIntosh Perry Consulting Engineers		Title: SITE PLAN	
Project#:	750-1001	DWG #:	1
Drawn:	NT	Approved:	CH
Date:	AUG 12-2011	Scale:	N. T. S.
Size:	Letter	Rev:	0
 SPL Consultants Limited Geotechnical Environmental Materials Hydrogeology			

METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT No
WP No 5271-08-01



JOHNSON CREEK CULVERT
HIGHWAY 17
BORE HOLE LOCATIONS & SOIL STRATA

SHEET
15

SPL Consultants Limited
Geotechnical • Environmental • Materials • Hydrogeology



LEGEND

- ⊕ Bore Hole & Cone
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- ↓ WL at time of Investigation July 2011
- ⊕ WL in Piezometer
- ⊕ Piezometer

No	ELEVATION	STATION	OFFSET
BH-1	182.9	17+958.8	8.0m E
BH-2	182.9	17+966.3	6.0m W

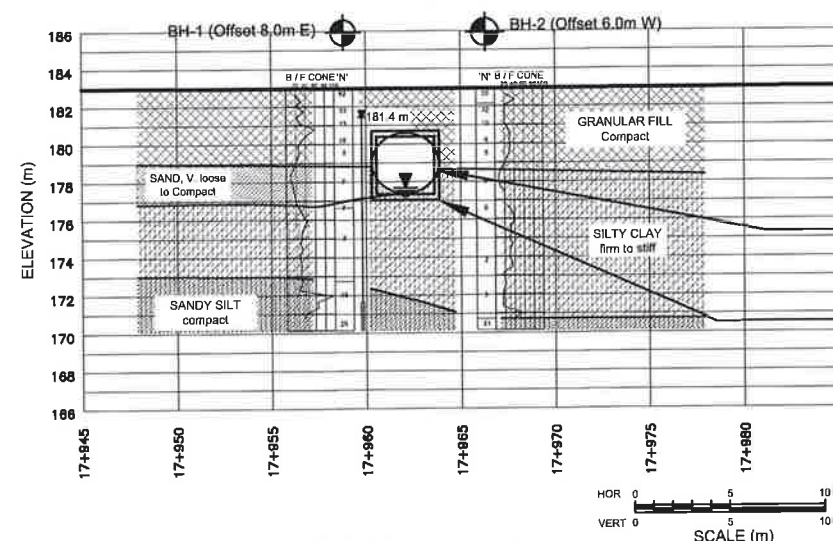
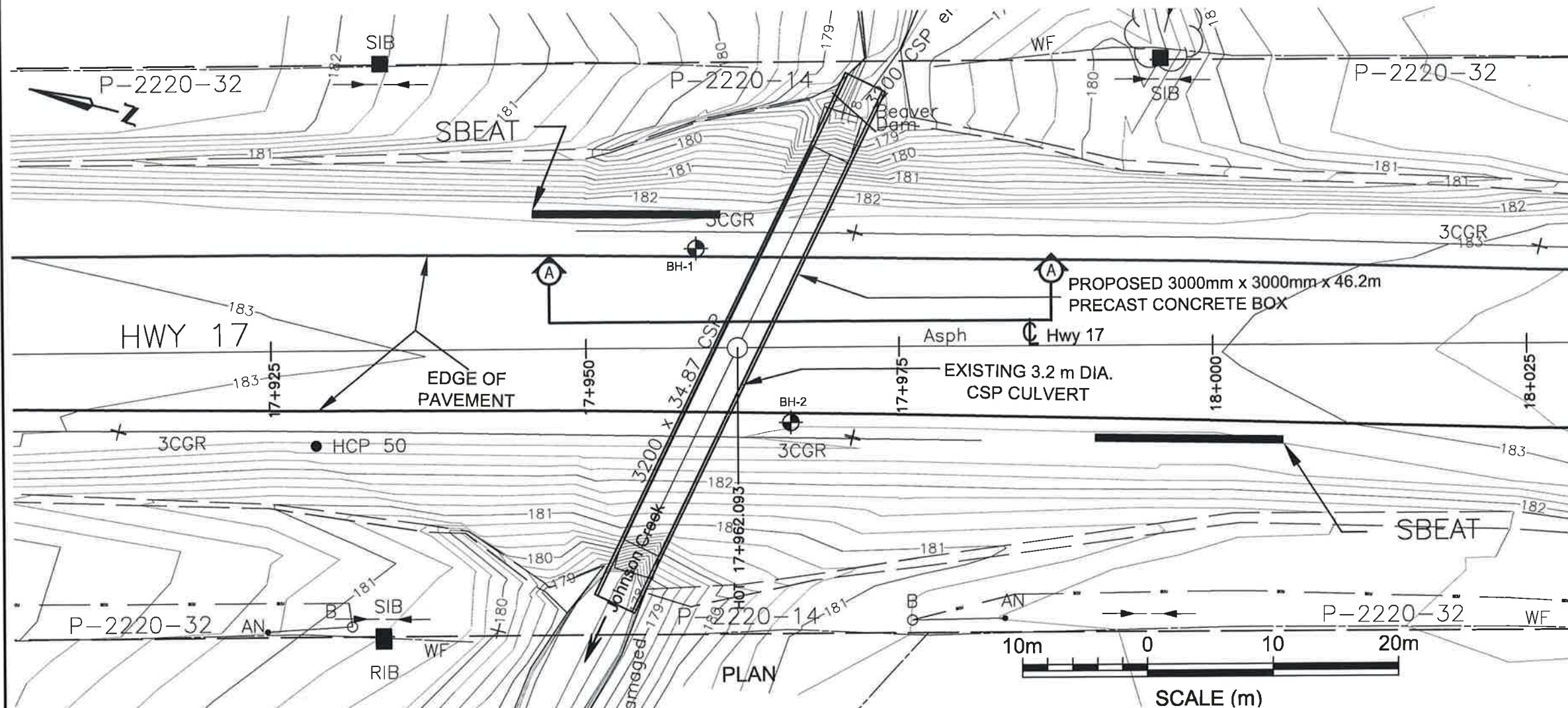
NOTES

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore holes the boundaries are assumed from geological evidence.

REVISIONS	DATE	BY	DESCRIPTION
Nov19/12	TJC		Final Revision
Feb18/12	TJC		Revision 1
DATE	BY		DESCRIPTION

GEORES No 41K-92

HWY No 17	CHECKED CH	DATE Nov19, 2012	DIST Algoma
SUBM'D CH	CHECKED CH	APPROVED FZ	SITE 385-404/C
DRAWN TJC	CHECKED CH		DWG 2

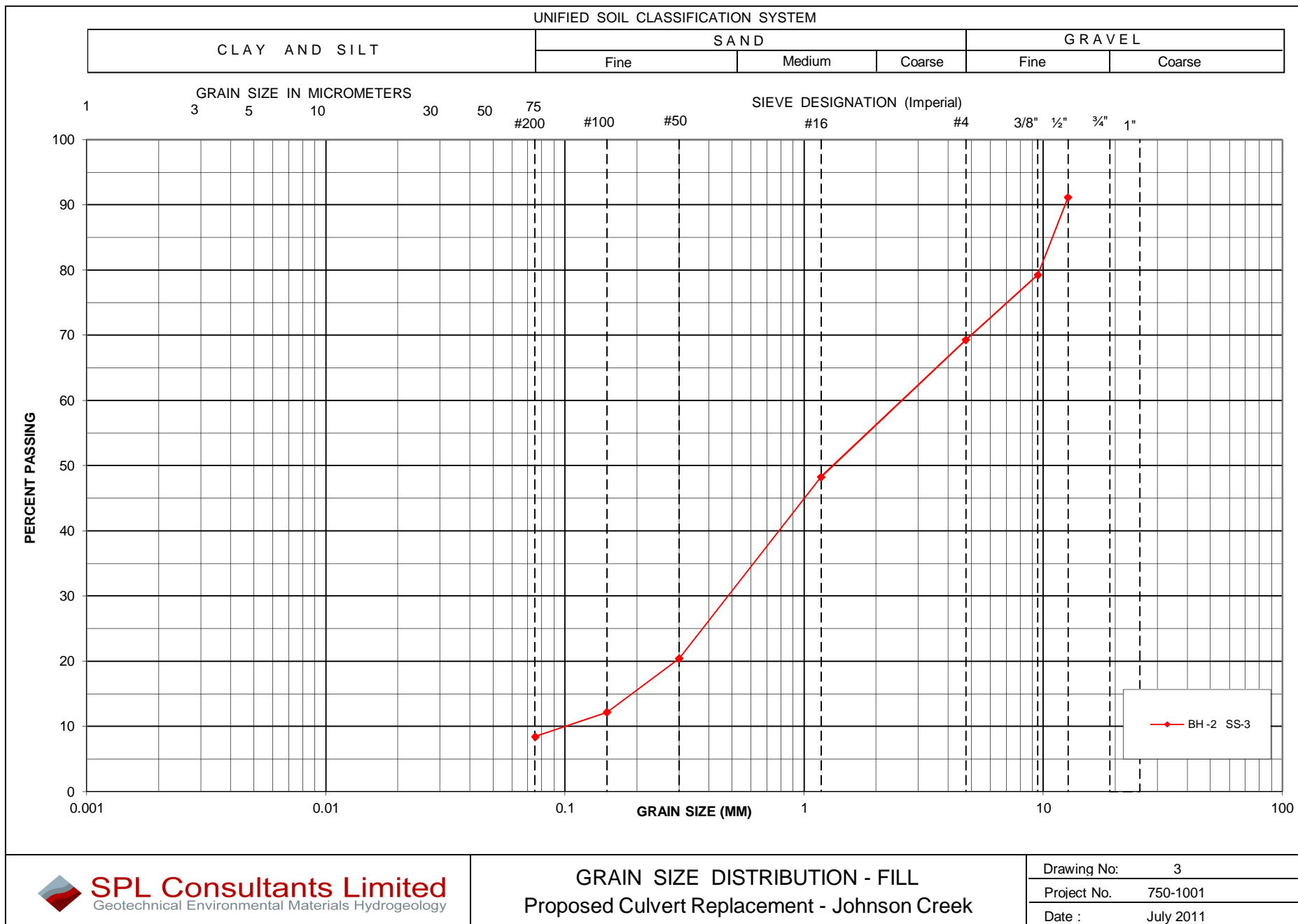


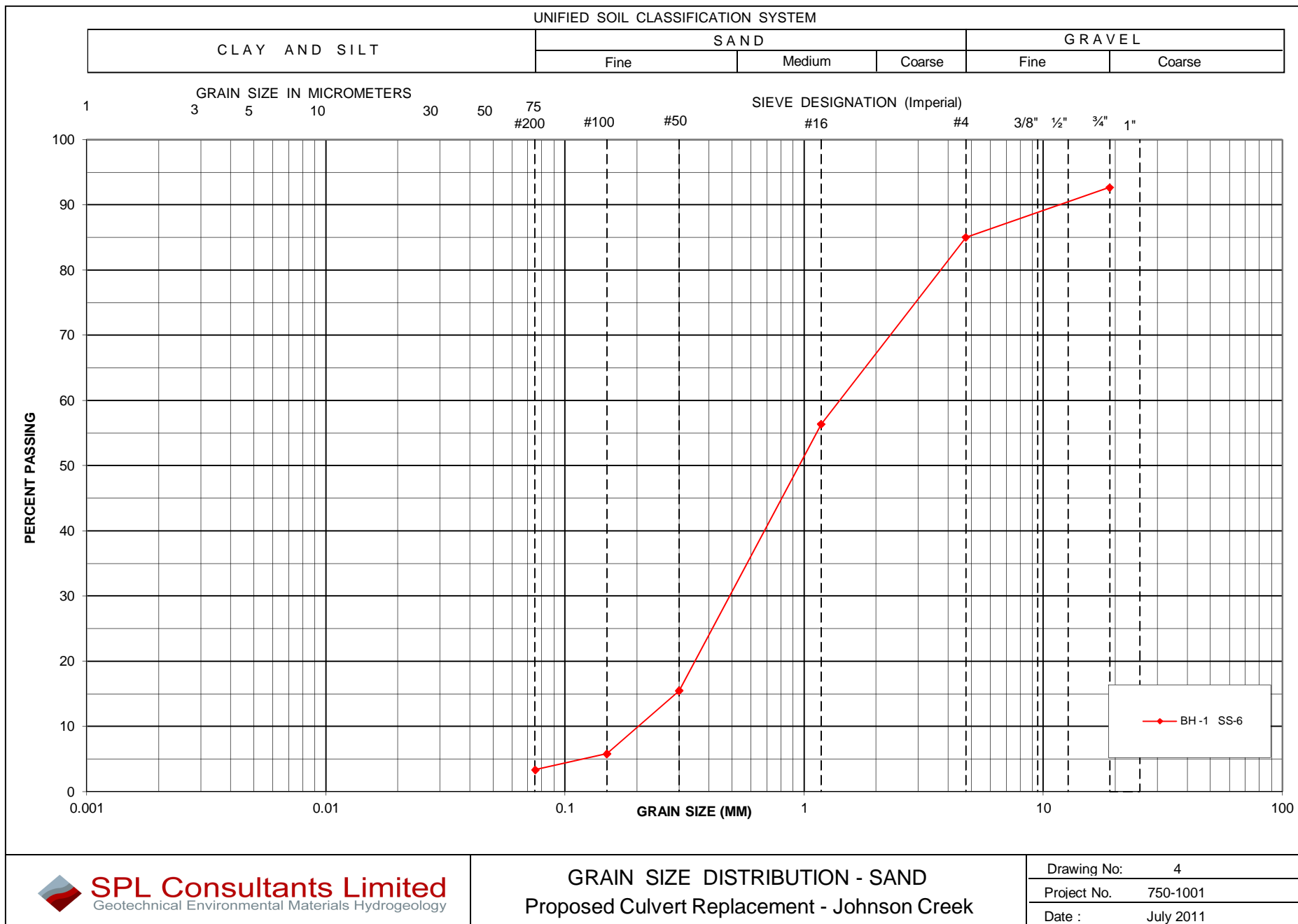
SOIL STRATA SYMBOLS

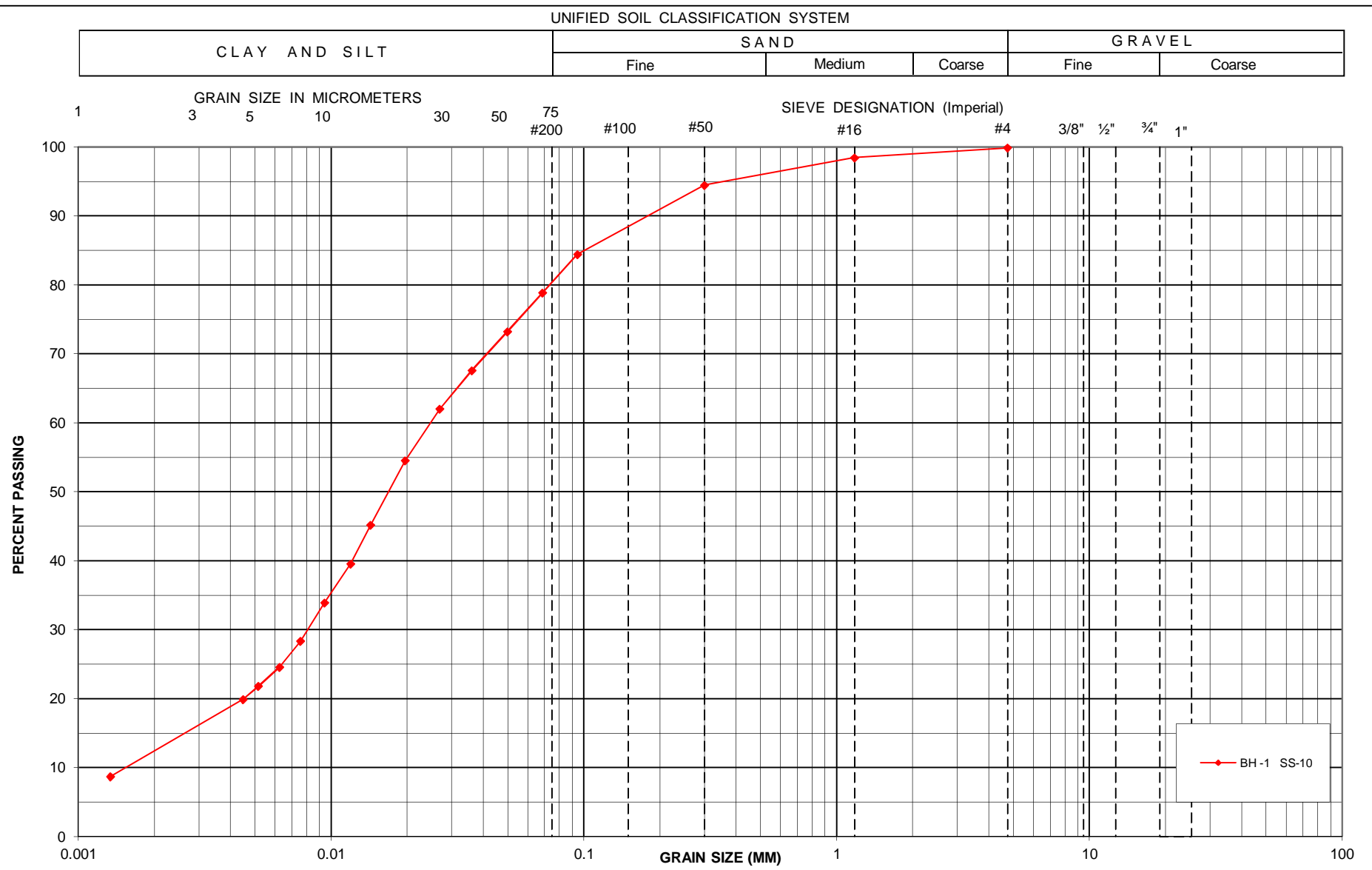
	FILL		SAND
	SILTY CLAY		SANDY SILT

CROSS SECTION A-A'









Appendix A

Borehole Logs (Record of Borehole Sheets)

RECORD OF BOREHOLE No BH-1

1 OF 1

METRIC

W.P. 5271-08-01 LOCATION See Borehole Location Plan ORIGINATED BY NE
DIST Algoma HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY NE
DATUM Geodetic DATE 14/07/2011 - 14/07/2011 CHECKED BY CH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
182.9 0.0	Fill: Sand and Gravel trace silt greyish brown, moist very dense		1	SS	62									
182.1 0.8														
	Fill: Sand some gravel to gravelly, trace silt brown to brownish grey, moist loose to compact		2	SS	50/ 25mm									
			3	SS	13									
			4	SS	16									
			5	NR	9									
178.8 4.1	Coarse Sand trace to some gravel, trace silt brown, wet very loose to loose		6	SS	2									
176.6 6.3	Silty Clay trace sand greyish brown to reddish brown, moist firm to stiff		7	SS	7									
			8	SS	2									
			9	TW										
172.7 10.2	Sandy Silt some clay, grey, wet compact		10	SS	25									
	some gravel, trace clay		11	SS	26									
170.1 12.8	End of Borehole Notes: 1. Water level at 4.3m during drilling. 2. Water level at 3.6m upon completion. 3. 19 mm dia. piezometer was installed to a depth of 12.8 m. 4. Water level in piezometer Date Depth (m) Elevation (m) July 15, 2011 1.5 181.4													

+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

ONL_MOT SPL-M-MTO-750-JUNE-JOHNSON CREEK.GPJ ON_MOT.GDT 14/12/11

RECORD OF BOREHOLE No BH-2

1 OF 1

METRIC

W.P. 5271-08-01 LOCATION See Borehole Location Plan ORIGINATED BY NE
DIST Algoma HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY NE
DATUM Geodetic DATE 14/07/2011 - 14/07/2011 CHECKED BY CH

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		
182.9 0.0	Fill: Sand and Gravel trace silt greyish brown, moist dense		1	SS	30									
182.1 0.8	Fill: Sand some gravel to gravelly, trace silt brown to greyish brown, moist loose to dense		2	SS	32		182							
			3	SS	10		181							31 61 8
			4	SS	8		180							
			5	SS	5		179							auger grinding
178.6 4.3	Silty Clay trace sand brownish grey to reddish brown, moist firm to stiff		6	SS	8		178							wet spoon 0 4 60 36
			7	SS	6		177							
			8	TW			176							
			9	SS	2		175							
			10	SS	3		174							
							173							
							172							
							171							
170.7 12.2	Sandy Silt: some gravel to gravelly, some clay grey, wet compact.		11	SS	21									
170.1 12.8	End of Borehole Notes: 1. Water level at 4.6m during drilling. 2. Water level at 5.5m and borehole caved to 11.9m upon completion.													

+ 3, X 3: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

ONL MOT SPL-M-MTO-750-JUNE-JOHNSON CREEK.GPJ ON MOT.GDT 14/12/11

Appendix B

Oedometer Test Results

October 13, 2011

Project No. 11-1183-0054

PO#750-1001

Fanyu Zhu
SPL Consultants Limited
16 - 6221 Highway 7 West
Vaughan, Ontario
L4H 0K8

GEOTECHNICAL LABORATORY TESTING

Dear Sir

This letter reports the results of laboratory testing carried out on the sample received at our office in Mississauga. The results of the tests are summarized in the attached table and figures.

The testing services reported herein have been performed in accordance with the indicated recognized standard, unless noted otherwise. This report is for the sole use of the designated client. This report constitutes a testing service only and does not represent any results interpretation or opinion regarding specification compliance or material suitability.

We trust that the results are sufficient for your current requirements. If you have any questions, please do not hesitate to call us.

GOLDER ASSOCIATES LTD.



Marijana Manojlovic
Laboratory Manager

MM/lg



Golder Associates Ltd.

2390 Argentia Road, Mississauga, Ontario, Canada L5N 5Z7
Tel: +1 (905) 567 4444 Fax: +1 (905) 567 6561 www.golder.com

Golder Associates: Operations in Africa, Asia, Australasia, Europe, North America and South America

Golder, Golder Associates and the GA globe design are trademarks of Golder Associates Corporation.



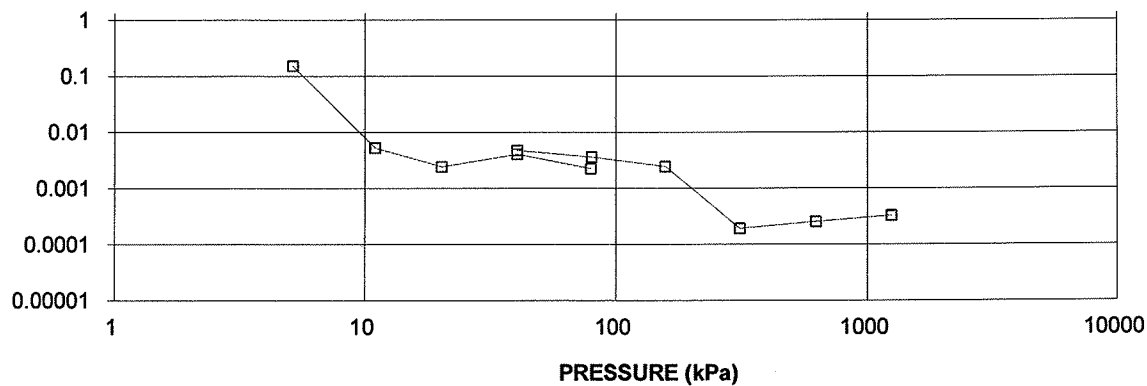
CONSOLIDATION TEST SUMMARY					FIGURE		
SAMPLE IDENTIFICATION							
Project Number	11-1183-0054			Sample Number	TW-9		
Borehole Number	1			Sample Depth, m	9.14-9.75		
TEST CONDITIONS							
Test Type	Standard			Load Duration, hr	24		
Oedometer Number	5						
Date Started	8/17/2011						
Date Completed	9/05/2011						
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL							
Sample Height, cm	1.90			Unit Weight, kN/m ³	15.94		
Sample Diameter, cm	6.33			Dry Unit Weight, kN/m ³	9.44		
Area, cm ²	31.47			Specific Gravity, measured	2.75		
Volume, cm ³	59.79			Solids Height, cm	0.665		
Water Content, %	68.86			Volume of Solids, cm ³	20.92		
Wet Mass, g	97.16			Volume of Voids, cm ³	38.87		
Dry Mass, g	57.54			Degree of Saturation, %	101.9		
TEST COMPUTATIONS							
Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t ₉₀ sec	cv, cm ² /s	mv m ² /kN	k cm/s
0.00	1.900	1.858	1.900				
5.20	1.900	1.858	1.900	5	1.53E-01	1.01E-05	1.52E-07
10.96	1.896	1.852	1.898	144	5.30E-03	3.65E-04	1.90E-07
20.02	1.893	1.847	1.894	311	2.45E-03	1.80E-04	4.32E-08
40.02	1.882	1.830	1.887	184	4.10E-03	2.89E-04	1.16E-07
79.16	1.861	1.799	1.871	327	2.27E-03	2.84E-04	6.31E-08
20.07	1.877	1.822	1.869				
10.76	1.884	1.833	1.880				
40.02	1.872	1.815	1.878	156	4.79E-03	2.16E-04	1.01E-07
79.69	1.859	1.796	1.865	202	3.65E-03	1.74E-04	6.22E-08
156.91	1.818	1.734	1.838	290	2.47E-03	2.79E-04	6.75E-08
312.46	1.604	1.412	1.711	3197	1.94E-04	7.24E-04	1.38E-08
624.14	1.388	1.087	1.496	1852	2.56E-04	3.65E-04	9.15E-09
1246.79	1.252	0.883	1.320	1127	3.28E-04	1.15E-04	3.69E-09
312.84	1.290	0.940	1.271				
79.16	1.358	1.043	1.324				
20.52	1.435	1.158	1.396				
5.81	1.492	1.244	1.463				
Note: k calculated using cv based on t ₉₀ values. Specimen taken 13cm from the bottom of the Shelby tube.							
SAMPLE DIMENSIONS AND PROPERTIES - FINAL							
Sample Height, cm	1.49			Unit Weight, kN/m ³	17.77		
Sample Diameter, cm	6.33			Dry Unit Weight, kN/m ³	12.02		
Area, cm ²	31.47			Specific Gravity, measured	2.75		
Volume, cm ³	46.95			Solids Height, cm	0.665		
Water Content, %	47.88			Volume of Solids, cm ³	20.92		
Wet Mass, g	85.09			Volume of Voids, cm ³	26.03		
Dry Mass, g	57.54						
<div style="display: flex; justify-content: space-between;"> <div>Prepared By: LH</div> <div>Golder Associates</div> <div>Checked By: </div> </div>							

CONSOLIDATION TEST SUMMARY

FIGURE

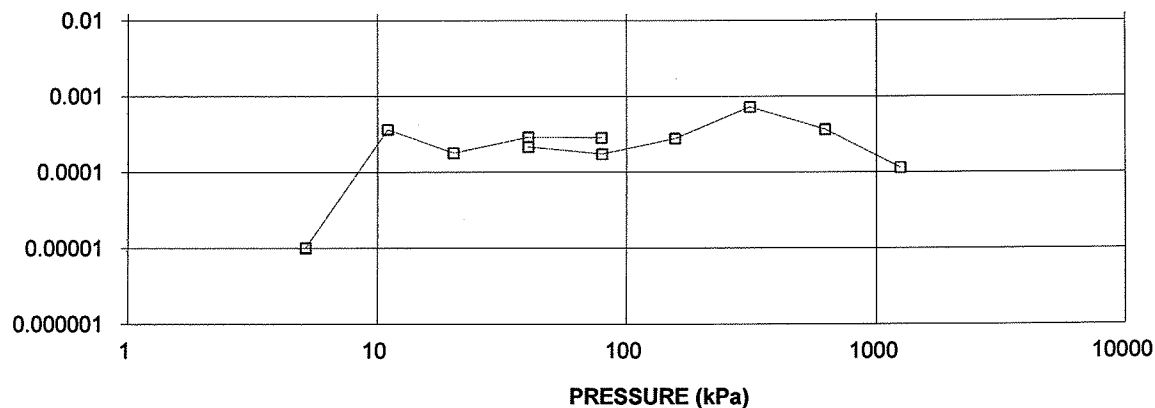
COEFFICIENT OF CONSOLIDATION,
cm²/s

CONSOLIDATION TEST
CV cm²/s VS P IRESSURE (kPa)
BH 1 SA TW-9



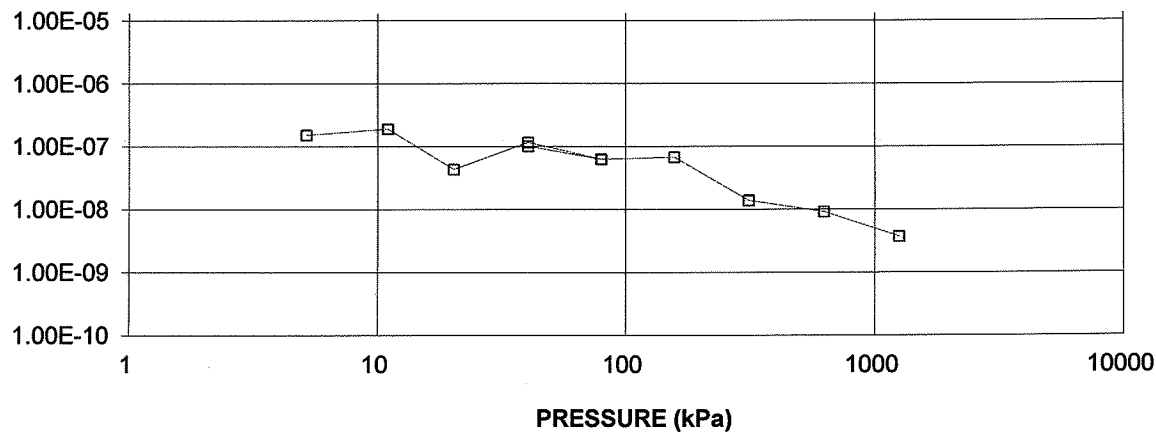
VOLUME COMPRESSIBILITY, m²/kN

CONSOLIDATION TEST
MV m²/kN vs PRESSURE (kPa)
BH 1 SA TW-9



HYDRAULIC CONDUCTIVITY, cm/s

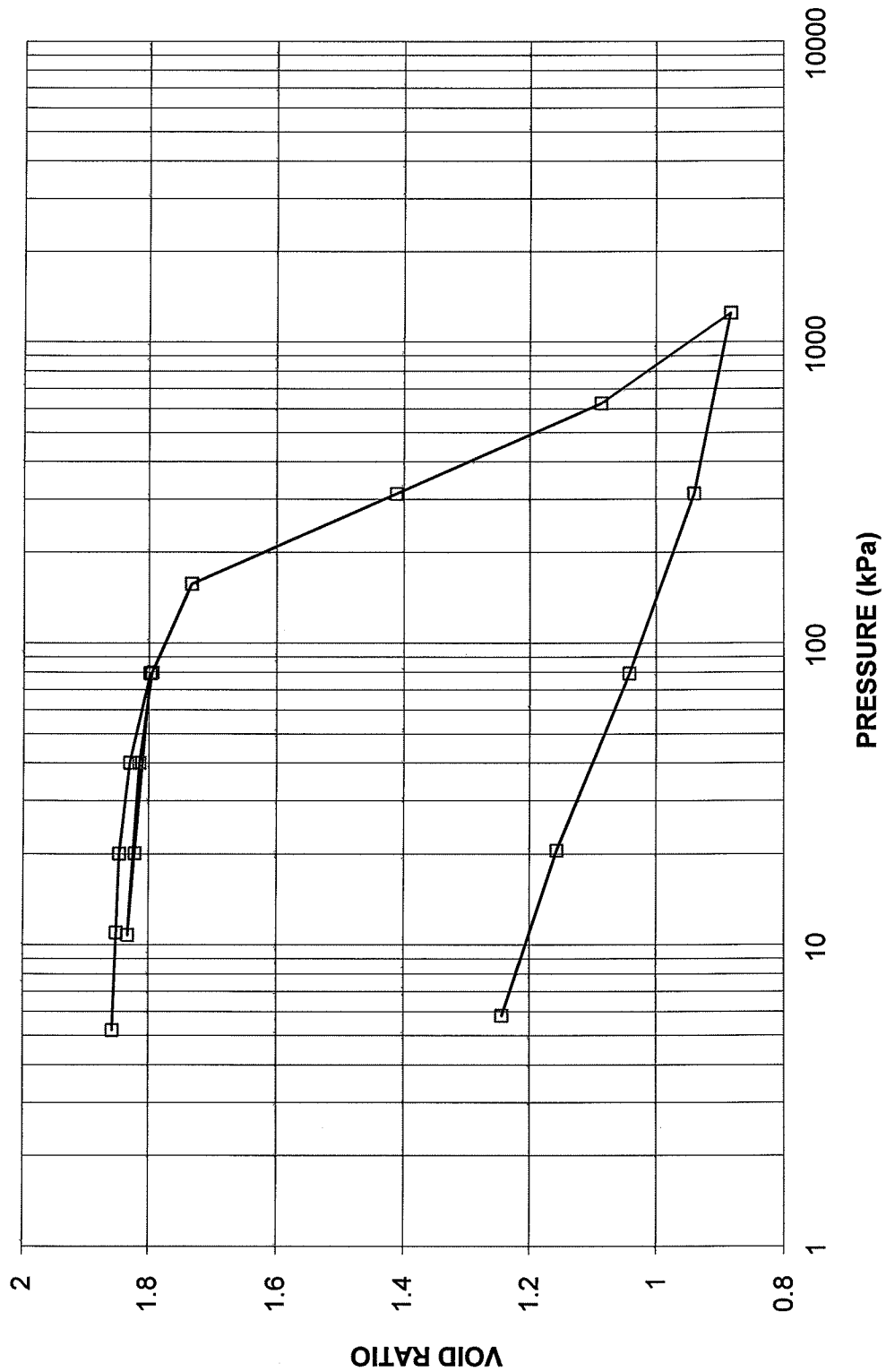
CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BH 1 SA TW-9



**CONSOLIDATION TEST
VOID RATIO VS LOG PRESSURE**

FIGURE

**CONSOLIDATION TEST
VOID RATIO vs PRESSURE
BH 1 SA TW-9**



Project No.11-1183-0054

Prepared By:LH

Golder Associates

Checked By: *bl*

SPECIFIC GRAVITY TEST RESULTS

ASTM D 854-06 TEST METHOD A

PROJECT NUMBER	11-1183-0054
PROJECT NAME	SPL Consultants / Testing / PO#750-1001
DATE TESTED	September, 2011

Sample	Specific Gravity
1	2.75

Note: Test carried out on soil particles <4.75mm using distilled water.

Checked By: 

Golder Associates

Appendix C

Chemical Test Results

**CLIENT NAME: SPL BEATTY
6221 HIGHWAY 7 WEST UNIT 16
VAUGHAN, ON L4H0K8**

ATTENTION TO: Naeem Ehsan

PROJECT NO: 750-1001

AGAT WORK ORDER: 11T512909

SOIL ANALYSIS REVIEWED BY: Anthony Dapaah, PhD (Chem), Inorganic Lab Manager

DATE REPORTED: Jul 29, 2011

PAGES (INCLUDING COVER): 4

VERSION*: 1

Should you require any information regarding this analysis please contact your client services representative at (905) 712 5100, or at 1-800-856-6261

***NOTES**

All samples will be disposed of within 30 days following analysis. Please contact the lab if you require additional sample storage time.



AGAT Laboratories

Certificate of Analysis

AGAT WORK ORDER: 11T512909

PROJECT NO: 750-1001

5835 COOPERS AVENUE
MISSISSAUGA, ONTARIO
CANADA L4Z 1Y2
TEL (905)712-5100
FAX (905)712-5122
<http://www.agatlabs.com>

CLIENT NAME: SPL BEATTY

ATTENTION TO: Naeem Ehsan

Chloride,EC,Resistivity, Sulphate, pH

DATE SAMPLED: Jul 08, 2011

DATE RECEIVED: Jul 22, 2011

DATE REPORTED: Jul 29, 2011

SAMPLE TYPE: Soil

Parameter	Unit	G / S	RDL	BH11-WCC-1	BH11-WCC-2	BH11-JC-1 2-2.5	BH11-JC-2 7-20	BH11-PL-4 2-2.5	BH11-PL-1 8-25
				3-5	8-2.5				
				2565072	2565074	2565080	2565082	2565095	2565096
Chloride (2:1)	µg/g		2	43	5	13	47	86	119
pH, 2:1 CaCl ₂ Extraction	pH Units			5.54	6.68	12.2	7.28	8.51	7.96
Electrical Conductivity (2:1)	mS/cm		0.002	0.110	0.026	4.76	0.219	0.230	0.341
Resistivity (2:1)	ohm.cm		1	9090	38500	210	4570	4350	2930
Sulfate (2:1)	µg/g		2.0	3.6	2.2	18.1	21.6	25.4	12.2

Comments: RDL - Reported Detection Limit; G / S - Guideline / Standard

Certified By:

Quality Assurance

CLIENT NAME: SPL BEATTY

AGAT WORK ORDER: 11T512909

PROJECT NO: 750-1001

ATTENTION TO: Naeem Ehsan

Soil Analysis

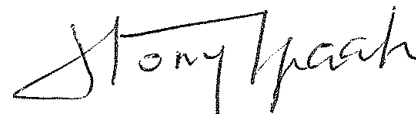
RPT Date: Jul 29, 2011

			DUPLICATE				REFERENCE MATERIAL			METHOD BLANK SPIKE			MATRIX SPIKE		
PARAMETER	Batch	Sample Id	Dup #1	Dup #2	RPD	Method Blank	Measured Value	Acceptable Limits		Recovery	Acceptable Limits		Recovery	Acceptable Limits	
								Lower	Upper		Lower	Upper		Lower	Upper

Chloride, EC, Resistivity, Sulphate, pH

Chloride (2:1)	1	2565072	43	41	4.8%	< 2	99%	90%	110%	93%	80%	120%	95%	80%	120%
pH, 2:1 CaCl ₂ Extraction	1		7.10	7.01	1.3%	<	102%	80%	120%						
Electrical Conductivity (2:1)	1	2565072	0.110	0.110	0.0%	< 0.002	100%	90%	110%						
Sulfate (2:1)	1	2565072	3.6	3.0	18.2%	< 2.0	90%	90%	110%	93%	90%	110%	100%	80%	120%

Certified By:



Method Summary

CLIENT NAME: SPL BEATTY

AGAT WORK ORDER: 11T512909

PROJECT NO: 750-1001

ATTENTION TO: Naeem Ehsan

PARAMETER	AGAT S.O.P	LITERATURE REFERENCE	ANALYTICAL TECHNIQUE
Soil Analysis			
Chloride (2:1)	INOR 1005	McKeague 4.12 & SM 4110 B	ION CHROMATOGRAPH
pH, 2:1 CaCl ₂ Extraction	INOR-93-6031	MSA part 3 & SM 4500-H+ B	PH METER
Electrical Conductivity (2:1)	INOR 1036	McKeague 4.12, SM 2510 B	EC METER
Resistivity (2:1)	INOR 1036		EC METER
Sulfate (2:1)	INOR 1005	McKeague 4.12 & SM 4110 B	ION CHROMATOGRAPH



AGAT Laboratories

CHAIN OF CUSTODY RECORD

213

SR6

5835 Coopers Avenue
Mississauga, Ontario; L4Z 1Y2
Phone: 905-712-5100; Fax: 905-712-5122
Toll free: 800-856-6261
www.agatlabs.com
<http://webearth.agatlabs.com>

LABORATORY USE ONLY

Arrival Condition: ☐ Good ☐ Poor (complete "notes")
Arrival Temperature: 17.9 AGAT WO #: 11T512909
Notes: _____

Client Information

Company: SPL Consultants Limited
Contact: NAEEM EHSAN
Address: 6221, Hwy 7, Unit-16,
Vaughan, ON
Phone: 905-856-065 Fax: 905-856-0025
Project: 750-1001 PO: _____
AGAT Quotation #: _____

Please note, if quotation number is not provided, client will be billed full price for analysis.

Invoice To Same as Above? Yes/No (circle)

Company: _____
Contact: _____
Address: _____
Phone: _____ Fax: _____

Report Information - reports to be sent to:

1. Name: NAEEM EHSAN
Email: nehshan@splconsultants.ca
2. Name: _____
Email: _____

Regulatory Requirements

☐ Regulation 153 Table (Indicate one)
☐ Ind/Com
☐ Res/Park
☐ Agriculture
Soil Texture (check one)
☐ Coarse ☐ Med/Fine
☐ Sewer Use Region (Indicate one)
☐ Sanitary
☐ Storm
☐ Regulation 558
☐ CCME
☐ Other (Indicate) _____
☐ Prov. Water Quality Objectives (PWQO)
☐ Nutrient Management Act (NMA)

Is this a drinking water sample (potable water intended for human consumption)?
☐ Yes ☐ No (If "Yes" please use the Drinking Water Chain of Custody Record)

Report Format

☐ Single Sample per page
☒ Multiple Samples per page
☐ Results by fax

Turnaround Time (TAT) Required* Regular TAT:

☒ 5 to 7 Working Days
Rush TAT: (please provide prior notification)
Rush Surcharges Apply
☐ 3 to 5 Working Days
☐ 2 Working Days
☐ 1 Working Day
OR
DATE REQUIRED (Rush surcharges may apply): _____

*TAT is exclusive of weekends and statutory holidays

Sample Identification	Date Sampled	Time Sampled	Sample Matrix	# of Containers	Comments Site/ Sample Information	Metals and Inorganics	Metal Scan (except Hg, B, Cr)	GC/MS Fractions 1 to 4	VOCs	PAHs	PCBs	TCLP Metals/Inorganics	TCLP	Storm Sewer Use	Sanitary Sewer Use	Chloride	Electrical Conductivity	Resistivity	PH	Sulfate	LABORATORY USE ONLY
BH 11-WCC-1(S ₂ -5)	08/07		Soil	1 bag	Wharmeliffa Creek																
BH 11-WCC-2(S ₂ -25)	09/07		Soil	1 bag	Wharmeliffa Creek																
BH 11-JC-1(S ₂ -205)	July 14		Soil	1 bag	Johnson Creek																
BH 11-JC-2(S ₇ -20)	14/07		Soil	1 bag	Johnson Creek																
BH 11-DL-4(S ₂ -25)			Soil	1 bag	Picnic Lake																
BH 11-DL-1(S ₂ -25)			Soil	1 bag	Picnic Lake																
Samples Relinquished By (print name & sign) <u>NAEEM EHSAN</u> NAEEM EHSAN						Date/Time <u>July 20, 2011</u>		Samples Received By (print name & sign) <u>Roy R</u> July 22/						Date/Time <u>9:00</u>		Pink Copy - Client		PAGE _____ of _____			
Samples Relinquished By (print name & sign)						Date/Time		Samples Received By (print name & sign)						Date/Time <u>4:44</u>		Yellow + Golden Copy - AGAT		NO: 155356			
																White Copy - AGAT					

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 63.5 kg, 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 STRUCUTRAL FEATURES AND/OR STRENGTH.

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

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

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

JOINT AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

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

MECHANICAL PROPERTIES OF SOIL

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

PHYSICAL PROPERTIES OF SOIL

P_s	kg/m ³	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	e_{min}	1, %	VOID RATIO IN DENSEST STATE
γ_s	kN/m ³	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
ρ_w	kg/m ³	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kN/m ³	UNIT WEIGHT OF WATER	s_r	%	DEGREE OF SATURATION	D_n	mm	N PERCENT – DIAMETER
P	kg/m ³	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ	kN/m ³	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
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
P'	kg/m ³	DENSITY OF SUBMERED SOIL	e_{max}	1, %	VOID RATIO IN LOOSEST STATE	j	kN/m ³	SEEPAGE FORCE
γ'	kN/m ³	UNIT WEIGHT OF SUBMERGED SOIL						

**PART B
FOUNDATION DESIGN REPORT
PROPOSED JOHNSON CREEK CULVERT REPLACEMENT
HIGHWAY 17 WEST OF HIGHWAY 638, ONTARIO
WP 5271-08-01 SITE NO. 38S-404/C
G.W.P. 5271-08-00
MTO GEOCRES NO. 41K-92**

Prepared for:

MCINTOSH PERRY CONSULTING ENGINEERS

By:

SPL CONSULTANTS LIMITED

Project: 750-1001 (Johnson Creek)
January 2013



SPL Consultants Limited
Geotechnical Environmental Materials Hydrogeology

146 Colonnade Road
Ottawa, Ontario K2E 7Y1
Tel: 613.228.0065 Fax: 613.228.0045

Table of Contents

PART B: FOUNDATION DESIGN REPORT

6. DISCUSSION AND RECOMMENDATIONS	7
6.1 General	7
6.2 Frost Protection	7
6.3 Seismic Performance.....	7
6.4 Foundations and Bearing Capacity	8
6.4.1 Foundation Options	8
6.4.2 Bearing Resistance	8
6.4.3 Sliding Resistance.....	8
6.5 Bedding, Cover and Backfill.....	9
6.6 Earth Pressures	9
6.7 Embankment Widening	11
6.8 Erosion Protection	11
6.9 Construction Considerations.....	12
6.10 Corrosion and Cement Type	13
7. CLOSURE	14
8. REFERENCES	15

6. DISCUSSION AND RECOMMENDATIONS

6.1 General

The subsurface conditions encountered in the boreholes drilled at the site included a layer of granular fill ranging from 4.1 m to 4.3 m in thickness at the culvert location which forms the road structure, embankment and backfill around the existing culvert.

The fill layer is underlain by native soils which include loose sand (on the north side of the culvert), firm to stiff silty clay and compact to dense sandy silt.

Groundwater was encountered at approximately 4 m below the ground surface, at a similar elevation as the water in the creek. This would be expected given the granular nature of the fill material which forms the fill embankment, as well as the uppermost sandy native soils.

The groundwater level measured at within the deep sandy silt layer was found to be at an elevation of 181.4 m, which is approximately 3.7 m higher than the level of the creek at the time of investigation, suggesting that an artesian condition may exist below the silty clay layer.

The proposed culvert structure is a 3 m wide by 3 m high pre-cast concrete box culvert. The invert of the new culvert will be 177.3 m to 177.4 m (which is slightly lower than the existing culvert). The replacement culvert will be in the same location as the existing culvert. The current design does not include any changes to the final embankment height.

Based on the borehole information the culvert and bedding will be founded on firm to stiff silty clay and possibly loose to compact sand in some areas. Either are expected to be adequate to support the proposed structure.

6.2 Frost Protection

The depth of frost penetration for the Johnson Creek site is 1.8 m. The existing fill material within the frost depth is predominantly sand and gravel and is considered to have a low susceptibility to frost heaving. As such, frost tapers are not required for the new construction.

6.3 Seismic Performance

The site is located in an area of relatively low seismic activity. The Peak Horizontal Ground Acceleration (PHA) for an earthquake with a 10% chance of exceedance in 50 years (475 year return period event) is 0.011 g. Based on the Canadian Highway Bridge Design Code (CHBDC) this corresponds to a Seismic Performance Zone 1 (assuming the crossing would be classified as an Emergency Route or Other Bridge; Performance Zone 2 if classified as an Lifeline), and Zonal Acceleration Ratio of $A = 0$ (CHBDC Section 4.4).

For the purposes of assessing the effects of site conditions under seismic conditions, the site may be assumed to be Soil Profile Type III, which corresponds to a Site Coefficient $S = 1.5$ (CHBDC Section 4.4.6).

6.4 Foundations and Bearing Capacity

6.4.1 Foundation Options

MTO has selected a closed-bottom, pre-cast concrete box culvert as the preferred replacement option. The sub-surface conditions at the site are considered to be adequate for the founding of the preferred replacement structure (pre-cast box culvert) on normal foundations (granular bedding placed over native granular soils or existing granular fill).

Deep foundations are technically feasible, but are not required as conventional shallow foundations will provide sufficient bearing resistance and settlement performance for the proposed culvert.

6.4.2 Bearing Resistance

The bedding for the new culvert structures will be placed on the sand and/or silty clay soils (both are present in the borehole logs at approximately the elevation of the founding invert).

For the new culvert, which will be 3 m wide and will be founded at approximately 177.3 m elevation, the unfactored geotechnical bearing resistance at Ultimate Limit State (ULS) can be taken as 400 kPa. A resistance factor of 0.5 should be applied to this value, yielding a factored bearing resistance of 200 kPa at ULS (Ultimate Limit States). This value is for a concentrically loaded foundation. Eccentric loads (if present) should be accounted for by considering an effective bearing area as outlined in Section 6.7.2 of the CHBDC.

The geotechnical bearing resistance at the Serviceability Limit State (SLS) can be taken as 100 kPa.

Provided that the subgrade is not unduly disturbed during construction the total and differential settlements associated with the above SLS resistance values are expected to be less than 25 mm and 20 mm, respectively. It is expected that for this level of settlement the new culverts will not require a camber.

6.4.3 Sliding Resistance

For the purposes of evaluating the sliding resistance (Section 6.7.5 of the CHBDC) of either the native soils or the granular fill below the foundation the effective cohesion, c' , should be assumed to be zero. The effective friction angle (ϕ') for the native soils may be assumed to be 28° . These values are unfactored values. A resistance factor of 0.8 should be applied to the resulting resistance to obtain the factored sliding resistance as per the CHBDC.

6.5 Bedding, Cover and Backfill

Bedding, Cover and Backfill details for the new culverts should be as per MTOD 803.021. Bedding for the new culverts may consist of either:

- 500 mm of compacted Granular A or Granular B Type II; or
- 300 mm of compacted Granular A or Granular B Type II placed over a lean concrete working slab.

If constructed properly, either bedding treatment is considered adequate from a foundations perspective.

A 75 mm levelling course of additional Granular A or fine aggregate should also be provided between the bedding and the culvert. In order to minimize the potential for piping and undermining of the culvert foundations the bedding should be wrapped in a non-woven geotextile which meets the requirements of OPSS 1860.

Cover for the new culverts should be a minimum of 300 mm thick as per MTOD 803.021 and may include either Granular A or Granular B with a maximum particle size of 75 mm (as per OPSS 422 and Special Provision 422S01).

Granular Backfill may consist of either imported Granular A or B material, or salvageable portions of the existing soils (Granular A and B is preferred for fills below the water table as well as immediately below the pavement structure). Portions of the fill which forms the embankment may meet the requirements of OPSS 1010 for Granular B Type I. Other portions of the existing fill embankment may meet the requirements of OPSS 1010 for SSM. The excavated soils should be reviewed as excavated and suitable portions may be stockpiled for re-use as backfill (if a cost-effective stockpile location is available). Material from below the water table, as well as the native soil, is unlikely to be suitable for re-use as backfill and there will be a net import of granular fill required for construction.

All bedding, cover and backfill should be placed in lifts not exceeding 200 mm and in accordance with OPSS 206. All fill material should be compacted in accordance with OPSS 422 (as amended by SP422S01), OPSS 501 and OPSS 902.

Heavy vibratory equipment should not be used behind the culvert and retaining walls within the restricted zone as outlined in OPSS 501.

6.6 Earth Pressures

Computation of earth pressures acting against culvert walls and retaining structures should be in accordance with the Canadian Highway Bridge Design Code (CHBDC). For design purposes, the following properties can be assumed for the backfill:

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

Angle of Internal Friction (ϕ) = 35 degrees (unfactored)

Unit Weight = 22 kN/m³

Coefficients of Lateral Earth Pressure:

Earth Pressure Coefficient	Level Backfill	Sloping Backfill 3H:1V	Sloping Backfill 2H:1V
K_a	0.27	0.34	0.40
K_b	0.35	0.44	0.50
K_0	0.43	0.56	0.62
K^*	0.45	0.60	0.66

Compacted Granular 'B' Type I

Angle of Internal Friction (ϕ) = 32 degrees (unfactored)

Unit Weight = 21 kN/m³

Coefficients of Lateral Earth Pressure:

Earth Pressure Coefficient	Level Backfill	Sloping Backfill 3H:1V	Sloping Backfill 2H:1V
K_a	0.30	0.38	0.47
K_b	0.38	0.48	0.57
K_0	0.47	0.61	0.69
K^*	0.51	0.67	0.76

Notes:

K_a is the coefficient of active earth pressure;

K_b is the coefficient of active earth pressure for an unrestrained structure including compaction efforts;

K_0 is the coefficient of earth pressure at rest;

K^* is the coefficient of earth pressure at rest for a fully restrained structure including compaction efforts.

The above values assume that the backfill behind the structure is free-draining granular fill, and that proper drainage is provided. Water pressures must also be accounted for in areas below the water table.

The appropriate earth pressure coefficient for design will depend upon whether the retaining structure is restrained or some movement can occur such that the active earth pressure state can develop. The effect of compaction should also be taken into account when selecting the appropriate earth pressure coefficients.

In accordance with the method outlined in the CHBDC and Commentaries Section 4.6.4, for a Zonal Acceleration Ratio of $A = 0$ the earth pressure under the design seismic event is equal to the earth pressure under static conditions (the horizontal seismic coefficient, k_h is 0.5 or 1.5 times the Zonal Acceleration Ratio, and for the design earthquake $A = 0$).

6.7 Embankment Widening

It is understood that the existing roadway embankment will be widened slightly to facilitate a detour around the construction site. Based on the conditions encountered in the boreholes, foundation failures are not anticipated for the proposed embankment widening with normal (2H:1V or flatter) slopes, assuming that all organic or unsuitable materials are removed as per MTO standards and procedures for stripping and benching prior to placing the embankment fills.

All unsuitable materials should be removed and the approved embankment subgrade should be proofrolled. The construction of the new embankment widening may require dewatering and/or groundwater control as discussed in Section 6.9 below where the base of the embankment is below the water table.

The sides of the existing embankment should be benched prior to placing fill material for the embankment widening, as per OPSS OPSD 208.01. Fill material should be placed in lifts not exceeding 300 mm in thickness and compacted to 95% SPMDD as per OPSS 206 and OPSS 501. Borrow material should consist of select suitable inorganic earth, free of objectionable inclusions such as cobbles, boulders, frozen materials, organic soils, etc. The existing fill material may be suitable for this purpose. Borrow material for the proposed embankment widening should be approved prior to installation from both a geotechnical and environmental standpoint.

Based on the subsurface conditions present, it is expected that the settlement at the surface of the embankment will be less than 50 mm (including settlement of the fill itself as well as the underlying soils) most of which will occur within approximately 6 weeks of construction (assuming granular fill is used for the embankment). These estimated settlements are typical of this type of construction and considered within acceptable limits. The embankment widening will be removed at the end of construction and the embankment returned approximately to its original geometry. The potential for long-term settlement of the clay layer is therefore minimal.

All embankment construction (including review of exposed subgrade, approval of fill materials, etc.) should be carried out under the review and supervision of a qualified person.

6.8 Erosion Protection

The native soils at the site are expected to be susceptible to erosion (both the sand and the silty clay). Erosion and scour protection (such as rip rap treatment in similar to OPSD 810.919) will be required at the culvert inlets and outlets. The sizing of the erosion protection should be carried out by a specialist who is familiar with the site hydraulics and the findings of this investigation.

The current culvert design includes upstream and downstream cut-off walls on the new culverts. These walls should extend to below the base of the bedding and levelling course to prevent flow of water below the walls through the permeable bedding layer. It is also recommended that the bedding be enclosed in a non-woven geotextile (OPSS 1860) in order to reduce the potential for piping and erosion of the culvert bedding.

6.9 Construction Considerations

Construction Dewatering

Seepage was encountered in the upper granular fill was found to be approximately equal to the level of the water in the river at the time of the investigation. The groundwater level is expected to be sensitive to changes in the water level in the river. For this reason it is recommended that if possible construction be carried out in a dry period when the river water level would be expected to be at its lowest level. It is also understood that a bypass will be in place during construction in order to convey the water around the culvert under construction.

The replacement will involve excavations below the groundwater table, and even with the above measures, dewatering will likely be required to stabilize the native soils, to maintain a dry working area and to minimize disturbance of the native soils during construction.

The creek water level was approximately 177.5 m elevation in July 2011 when the investigation was completed. Excavation to an elevation of approximately 176 m (or at least 1.5 m below the water table) will be required to accommodate the culvert, bedding, levelling course, etc. These deeper excavations will likely require an active dewatering system (such as well points) to maintain a dry excavation.

Where the excavation is in silty clay it is likely that groundwater inflow can be handled by pumping from properly filtered sumps. Where sand (such as was encountered in BH-1) is exposed an underground impervious barrier (such as a sheet pile wall; the choice of protection systems and cut-off walls will ultimately be the responsibility of the contractor) may also need to be constructed to control groundwater flows into the excavation.

Temporary Excavations

All excavations should be carried out in accordance with the most recent Occupational Health and Safety Act (OHSA). Part III of Ontario Regulation 213/91 deals with excavations. In addition, the following Ontario Provincial Standard Specifications (OPSS) also deal with temporary excavations:

OPSS 539 – Construction Specification for Temporary Protection Systems

OPSS 902 – Construction Specification for Excavating and Backfilling - Structures

The soils at the site include granular fill in the pavement structure and embankment, underlain by very loose to loose sand and firm to stiff silty clay. The granular fill can be classified as Type 3 soil above the

water table and Type 4 below the water table. The native soils can be classified as Type 3 soil above the water table and Type 4 soil below the water table.

Temporary excavations above the water table are likely feasible using sloped excavations in the granular fill. Excavations below the water table will require some form of protection system. It is also noted that the preliminary staging will require excavation in close proximity to the travelled lanes of the highway, which will preclude the use of sloped excavations in some areas (as there is not sufficient space).

If temporary shoring is required it would typically consist of soldier piles and timber lagging or interlocking sheet piles. It should be noted that cobbles and boulders were inferred during drilling within the lower portions of the fill material and could be encountered during construction. This should be considered when selecting protection systems and installation methods.

Foundation Excavations

The bearing capacities provided in Section 6.4 above assume that the subgrade is not excessively disturbed during construction. Given the fact that the foundations for any new structures will be below the groundwater table in loose to compact sand and silt, it will require careful construction control to achieve this condition. Installation and operation of an adequate dewatering system, as discussed above, will be critical to the construction of the foundations.

A layer of lean concrete working slab (mud slab) on foundation bearing surfaces can also be included in the design (see Section 6.5 above). If used, the working slab should be placed immediately after excavation and inspection (before placement of bedding and levelling layers) to minimize foundation disturbance. If excavation conditions are found to be better than anticipated then the requirement for the lean concrete mud slab may be waved at the discretion of the CA and QVE. All excavated surfaces should be kept free of frost, water, etc., during the course of construction.

All excavated surfaces should be inspected prior to foundation construction by a qualified individual who is familiar with the findings of this investigation and the design and construction of similar structures.

6.10 Corrosion and Cement Type

Two soil samples were submitted to AGAT Laboratories for testing related to soil corrosivity and potential exposure of concrete elements to sulphate attack. The results of these tests are included in Appendix C of this report.

The test results indicate that the sulphate content of the native soils is relatively low, and sulphate-resistant Portland cement is not required.

The test results (soil resistivity, pH and chloride content) indicate potential for corrosion of steel elements. Appropriate care should be taken in designing the corrosion protection system for any buried steel structures.

7. CLOSURE

Field investigations for this project were supervised by Naeem Ehsan, P.Eng. This report was prepared by Mr. Chris Hendry, P.Eng. Mr. Fanyu Zhu, P.Eng., SPL's project manager and designated MTO Contact, and Mr. Shaheen Ahmad, P.Eng., SPL's quality control auditor provided independent review and quality control.

We trust that the information contained in this report is satisfactory. Should you have any questions, please do not hesitate to contact this office.

SPL CONSULTANTS LIMITED

Chris Hendry, M.Eng., P.Eng.



Fanyu Zhu, Ph.D., P.Eng.



Shaheen Ahmad, M.A.Sc., P.Eng.

8. REFERENCES

The following section provides a general list of references, as well as a list of Ontario Provincial Standard Specifications which are expected to be relevant to the Foundations portion of the proposed work.

General References

CAN/CSA-S6-06 Canadian Highway Bridge Design Code, 2011

Canadian Foundation Engineering Manual, 2006. 4th Edition. Canadian Geotechnical Society

Relevant Ontario Provincial Standard Specifications

OPSS NO.	TITLE
128	Supply of Pre-Qualified Materials and Products
182	Environmental Protection for Construction in Waterbodies and on Waterbody banks.
201	Clearing, Close Cut Clearing, Grubbing, and Removal of Surface and Piled Boulders
206	Grading
401	Trenching, Backfilling, and Compacting
404	Support Systems
422	Precast Reinforced Concrete Box Culverts and Box Sewers in Open Cut
501	Compacting
504	Preservation, Protection and Reconstruction of Existing Facilities
506	Dust Suppressants
510	Removals
511	Rip-Rap, Rock Protection, and Granular Sheeting
514	Trenching, Backfilling, and Compacting
518	Control of Water from Dewatering Operations
539	Temporary Protection Systems
805	Temporary Erosion and Sediment Control Measures
902	Excavating and Backfilling – Structures
1001	Aggregates - General
1010	Aggregates – Base, Subbase, Select Subgrade, and Backfill Material
1860	Geotextiles

Relevant CDED Special Provisions

Provision No.	Title
100S60	Amendment to MTO General Conditions of Contract, April 2010 – use of unlicensed vehicles...
104S04	Amendment to OPSS 401, November 2010
105S21	Amendment to OPSS 501, November 2010
110S13	Amendment to OPSS 1010, April 2004
199S55	Record Drawings for Structures and Foundations
422S01	Precast Concrete Box Culvert
511S01	Rip Rap
539S02	Protection System – Amendment to OPSS 512, April 2011
805F01	Light-Duty Sediment Barriers, etc.

Relevant OPSD's

OPSD No.	Title
803.010	Backfill and Cover for Concrete Culverts with Spans Less Than or Equal to 3 m
810.010	Rip-Rap Treatment for Sewer and Culvert Inlets
810.020	Rip-Rap Treatment for Ditch Inlets
3090.100	Foundation, Frost Penetration Depths for Northern Ontario

Relevant MTOD's

MTOD No.	Title
803.021	Bedding and Backfill for Precast Concrete Box Culverts