

**FOUNDATION INVESTIGATION AND DESIGN REPORT
PROPOSED BRIDGE REPLACEMENT
PRUNE CREEK BRIDGE NEAR HEARST, ONTARIO
SITE NO. 39W-50
G.W.P. 5033-07-00
MTO GEOCRES NO. 42G-41**

Prepared for:

ONTARIO MINISTRY OF TRANSPORTATION

By:

SPL CONSULTANTS LIMITED

Project: 1067-710 (Prune Creek Bridge)
March 2013



SPL Consultants Limited
Geotechnical Environmental Materials Hydrogeology

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1. INTRODUCTION

SPL Consultants Limited (SPL) was retained by the Ontario Ministry of Transportation (MTO) to conduct a foundation investigation as part of a proposed bridge replacement at Prune Creek on Concession Road 12, 2.6 km east of Highway 583 near Hearst, Ontario.

The terms of reference (TOR) for this investigation are outlined in the Request for Proposal (RFP) issued by the MTO under Agreement No. 5011-E-0023 dated November 2011 and SPL's subsequent Proposal No. P11.12.011 dated December 2011.

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 bridge replacement.

2. SITE DESCRIPTION

The site is located on Concession Road 12 approximately 2.6 km east of Highway 583, 17.5 km south of Highway 11 near Hearst, Ontario (see Drawing 1).

The existing structure is a three span timber bridge supported on timber trusses and girders. The main span is approximately 29 m in length with two smaller approach spans of 5 m to 6 m. The intermediate piers are supported on timber and steel bents. The deck is approximately 5.3 m wide and is also constructed of timber. The existing road is gravel surfaced on either side of the bridge.

The elevation of the road in the general vicinity of the crossing is approximately 99.2 m on the west side of the bridge and 98.8 m on the east side of the bridge.¹ The existing approach embankments are approximately 2 m to 3 m high at the crossing.

At the time of the investigation the water level in the creek was at approximately 3 m below the existing deck elevation, at about 96 m elevation.

3. INVESTIGATION PROCEDURES

The foundation investigation was carried out in March 2012. The scope of work for this assignment included a desk study, field investigations, laboratory testing, analysis and preparation of this report.

¹ A local datum was assigned to this project by the MTO prior to beginning the foundation investigations. All elevations referenced in this report are with respect to the local project datum unless otherwise indicated.

3.1 Desk Study

Surficial geology in the area comprises glacio-lacustrine deposits (silt and clay with minor sand), as well as fine-grained glacial till and, in some areas, exposed bedrock.

Bedrock geology maps of the general area indicate the bedrock to be muscovite-bearing granitic rock.

As part of this investigation a review of the MTO's files in North Bay was also carried out for the site. The files contain a drawing of the bridge dated 1936 which includes an interpretation of the sub-surface conditions at the site.

The drawing indicates the native soil below the approach embankments includes a layer of soft clay, overlying a layer of stiff clay and gravel, which in turn overlies bedrock. The west abutment and both intermediate piers are supported on piles which range from 6.1 m (20') to 7.6 m (25'). At the east abutment a "concrete pedestal" is indicated placed on the bedrock, with the structure above. It is possible that the intention was to excavate to shallow bedrock on the east side; the findings of the current investigation suggest the bedrock is deeper than shown in the drawings on the east side, and the foundation may not be as indicated on the original drawings (the drawings are from 14 years before the bridge was apparently constructed – in 1950 – and it is possible they do not represent the final design). The general arrangement of the bridge is, however, consistent with the drawings and the overall stratigraphy of the site is consistent with the findings of the current investigation (though the depths of the various soil types differ at some locations).

3.2 Field Investigation

The field investigation was carried out between March 20 and March 22, 2012. The field investigation included drilling a total of 6 boreholes at the crossing location (BH-1 through BH-6).

The boreholes were advanced using a truck-mounted drill rig supplied and operated by Landcore Drilling of Chelmsford, ON. The boreholes were drilled using a combination of hollow-stem auger drilling and rock coring to depths ranging from 7.2 m to 10.7 m below the existing ground surface. During drilling, sampling and in-situ testing including Standard Penetration (SPT) Testing and Dynamic Cone Penetration Testing (DCPT) Testing, were carried out at regular intervals.

Standpipe piezometers were installed in Boreholes BH-2, BH-3, BH-4 and BH-5 to allow for measurement of groundwater levels at the site. All boreholes were backfilled with bentonite and soil cuttings and were sealed at the ground surface.

Borehole locations are shown in Drawing 2. Borehole logs are included in Appendix A of this report.

3.3 Laboratory Testing

Upon completion of drilling and in-situ testing, soil samples were returned to SPL's laboratory for further examination and classification. A laboratory testing program, including determination of natural water

content, unit weight, Atterberg limits (plasticity), grain size distribution (sieve and hydrometer) and chemical analyses, was carried out on selected representative soil samples. In addition, consolidation testing was carried out on a sample of silty clay to determine the soil load-settlement characteristics.

The results of natural water content testing are included on the relevant borehole logs in Appendix A. The results of grain size distribution testing are summarized on the individual borehole logs and are presented in Drawings 3 and 4. The results of Atterberg limits (plasticity) testing are presented on the individual borehole logs and summarized in Drawing No. 5. The results of consolidation testing are included in Appendix B.

Chemical testing to determine sulphate content, chloride content, pH and soil resistivity 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

4.1.1 Fill

All of the boreholes drilled as part of this investigation were located on the existing gravel-surfaced (unpaved) road. At all locations granular fill was encountered which forms the road structure, as well as the existing embankment.

The granular fill is primarily gravelly sand, with a trace of silt. The grain size curves for several samples of the fill are presented in Drawing 3. A summary of the grain size distribution of these samples is also presented in Table 1 below. It should be noted that these grain size distribution tests were carried out on samples obtained through SPT testing which does not recover coarse gravel, cobble and boulder sized particles. Because of this the grain size distributions shown on Drawing 3 and Table 1 may be finer than portions of the materials in the field.

Table 1 – Results of Grain Size Analyses for Sandy Fill Material

Borehole No.	Sample No.	Grain Size Distribution		
		% Gravel	% Sand	% Silt & Clay
BH-1	2	41	55	4
BH-2	2	17	76	7
BH-3	2	25	70	5
BH-3	3	19	75	6
BH-4	1	23	73	4

Borehole No.	Sample No.	Grain Size Distribution		
		% Gravel	% Sand	% Silt & Clay
BH-4	3	8	87	5
BH-5	1	25	71	4
BH-5	2	34	63	3
BH-6	1	25	71	4

The density of the fill material (as interpreted based on SPT “N” values) ranged from dense to very dense. Very high “N” values may reflect the presence of cobbles and boulders, rather than a very high density of the soil matrix itself.

On the east side of the bridge (BH-4 and BH-6) a layer of silty clay fill (approximately 0.7 m and 1.2 m in thickness) was also encountered between the upper granular fill and the native silty clay below. The consistency of this silty clay fill would be described as stiff to very stiff (based on SPT “N” values). A single measurement of the soil unit weight yielded a value of 18.8 kN/m³. This silty clay fill was not encountered on the west side of the bridge.

The fill material extended to a depth of 1.5 m to 2.3 m below the existing road surface in the boreholes drilled on the west side of the proposed bridge (BH1 through BH-3) and to a depth of 1.5 m to 3.0 m on the east side of the bridge (BH-4 through BH-6). This corresponds to local elevations of 95.9 m to 97.7 m.

4.1.2 Silty Clay

The fill layer is underlain by a layer of silty clay which was encountered in all of the boreholes drilled at the site.

The results of plasticity testing on six samples of the silty clay indicate the soils would be classified as low to high plasticity clay, with two samples being near the boundary between medium plasticity clay and medium plasticity silt. For simplicity the deposit is referred to as silty clay as that is the predominant soil type.

Based on the results of SPT, DCPT and field vane testing the silty clay would be considered stiff to very stiff near the interface with the fill above, becoming firm to stiff with depth. This corresponds to undrained shear strengths of greater than 100 kPa near the top of the layer and as low as 30 kPa with depth.

Unit weight measurements on samples of the silty clay yielded values of 16.5 kN/m³ to 19.7 m³ with an average value of 18.3 kN/m³.

One sample of the silty clay soil was subjected to consolidation testing to determine the consolidation properties of the soil. The results of this test are included in Appendix B, and are summarized in Table 2 below.

Table 2 – 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
BH2-TW8	6.4 m	110	80	0.52	0.13	1.382	1.38

The silty clay is present on both sides of the bridge, but is significantly thicker on the west side. On the west side of the bridge (BH-1 through BH-3) the silty clay layer was found to be 4.3 m to 5.0 m in thickness. On the east side of the bridge (BH-4 through BH-6) the layer was found to be 0.9 m to 2.5 m in thickness.

4.1.3 Silty Sand and Sandy Silt

The silty clay layer is underlain by a layer of silty sand and sandy silt with trace clay and trace to some gravel. The grain size distributions of several samples from this soil stratum are presented in Drawing 4, and are summarized in Table 3 below.

Table 3 – Results of Grain Size Analyses for Silty Sand and Sandy Silt

Borehole No.	Sample No.	Grain Size Distribution			
		% Gravel	% Sand	% Silt	% Clay
BH-1	8	9	27	55	9
BH-3	10	0	27	71	2
BH-4	7	0	51	47	2
BH-5	6	15	51	31	3
BH-6	5	2	62	34	2
BH-6	7	17	50	29	4

The consistency of the silty sand and sandy silt ranges from compact to very dense (as interpreted based on SPT “N” values). SPT “N” values and DCPT resistance values are presented on the borehole logs included in Appendix A as well as on the cross-sections presented as Drawings 2A and 2B.

4.1.4 Bedrock/Auger Refusal

Auger refusal was encountered in all six of the boreholes drilled as part of this investigation. The depth to refusal ranged from 7.2 m to 8.1 m below the existing road surface, which corresponds to an elevation of 91.1 m to 92.2 m (in the local datum).

At two locations (BH-2 on the west side of the bridge and BH-4 on the east side) the rock was cored using “N” size coring equipment. The bedrock in the area includes fresh to slightly weathered, fine to medium grained granite. Rock Quality Designation (RQD) values for the rock cored range from 22% to 51 % in BH-2 (indicating poor to fair quality rock) and 64% to 100 % in BH-4 (indicating fair to good quality rock). The rock surface was confirmed at the location of the proposed bridge abutments at

approximately 91.5 m elevation on the west side of the bridge (BH-2) and 92.2 m elevation on the east side (BH-4).

4.2 Groundwater Conditions

The water level in the creek at the time of the investigation was approximately 96 m elevation (in the local datum). Seepage was noted within the sandy and silty soils (which are below that elevation) encountered during drilling.

Standpipe piezometers were installed in four of the boreholes during drilling; in the silty clay layer (one) the silt and sand layer (two) and the underlying bedrock (one). The groundwater levels at the site were found to be at 96.6 m to 96.9 m elevation (or slightly above the level of the creek at the time of drilling).

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 March 2012, a corresponding increase in groundwater levels should be anticipated.

4.3 Summary

A summary of the soil and groundwater conditions encountered at the crossing location is presented in Table 4 below.

Table 4 – Simplified Stratigraphy and Groundwater Elevations

BH No.	Ground Surface Elev. (local)	Simplified Soil Stratigraphy (local Elev.)					Measured Groundwater Elevation (local)
		Fill	Silty Clay	Sandy Silt & Silty Sand	Auger Refusal	Bedrock (Cored)	
BH-1	99.2 m	99.2 – 97.7	97.7 – 93.1	93.1 – 91.1	91.1	--	--
BH-2	99.1 m	99.1 – 96.9	96.9 – 91.5	--	91.5	91.5 – 88.4	96.9
BH-3	99.1 m	99.1 – 96.9	96.9 – 91.8	91.8 – 91.0	91.0	--	96.9
BH-4	98.9 m	98.8 – 95.9	95.9 – 94.7	94.7 – 92.2	92.2	92.2 – 89.5	96.6
BH-5	98.9 m	98.9 – 97.4	97.4 – 94.9	94.9 – 91.7	91.7	--	96.7
BH-6	98.9 m	98.9 – 96.8	96.8 – 95.9	95.9 – 91.6	91.6	--	--

5. CLOSURE

The field investigations were supervised by Mr. Naeem Ehsan, P.Eng. This report was prepared by Mr. Chris Hendry, P.Eng. Mr. Fanyu Zhu, P.Eng., who is the project manager and SPL's designated MTO contact and Mr. Shaheen Ahmad, P.Eng., who is the project quality control auditor, provided quality control and independent review of the technical aspects of this report.

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.

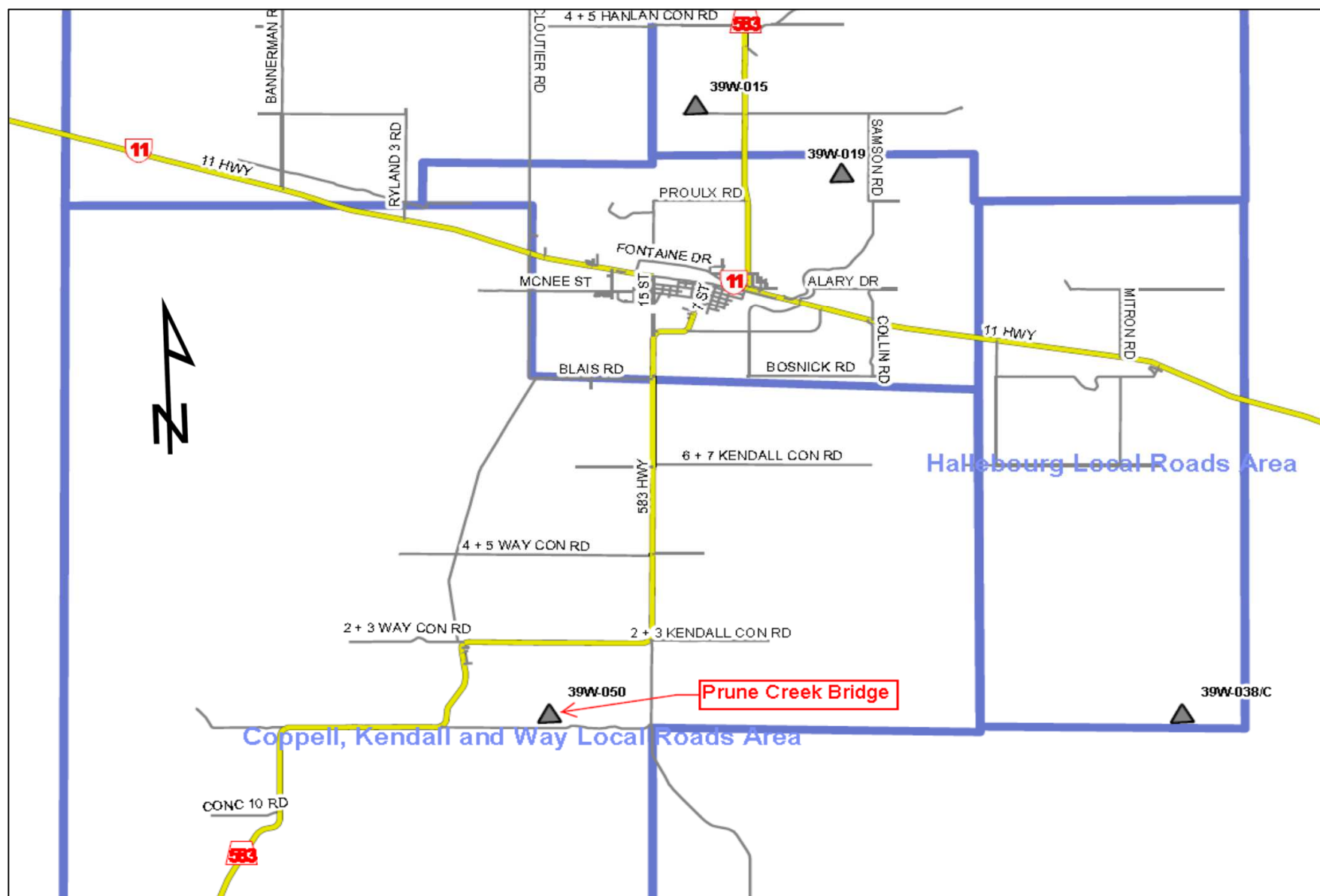


Fanyu Zhu, Ph.D., P.Eng.



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

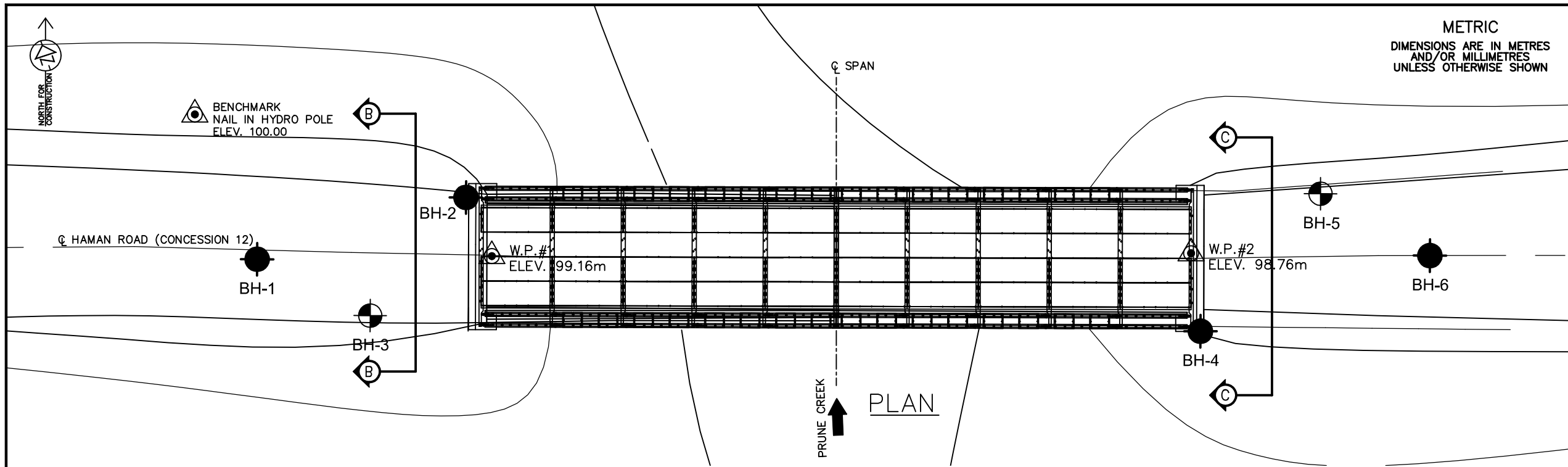
Drawings



Client: Ministry of Transportation Ontario			Title: SITE PLAN	
Project#:	1067-710	DWG #:	1	Project: Geotechnical Investigation Prune Creek Bridge Replacement
Drawn:	NT	Approved:	CH	
Date:	April 2012	Scale:	N. T. S.	
Size:	Letter	Rev:	0	



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

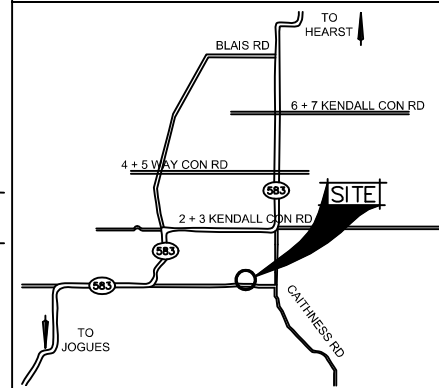
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WP No 5079-11-01



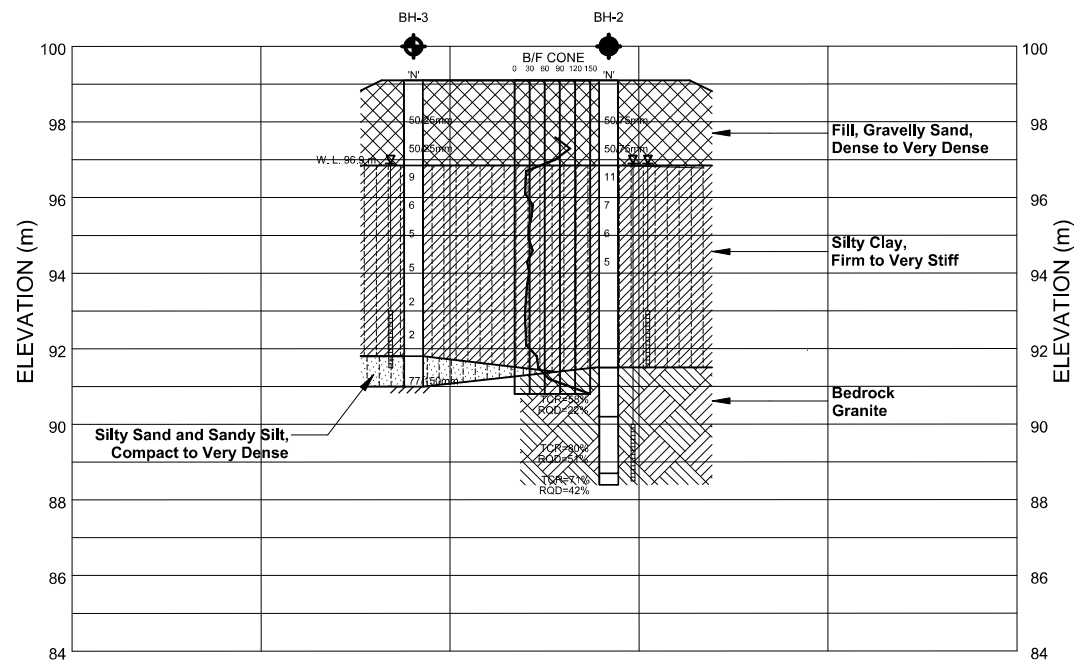
PRUNE CREEK BRIDGE
AND GEOSECTION

SHEET
4

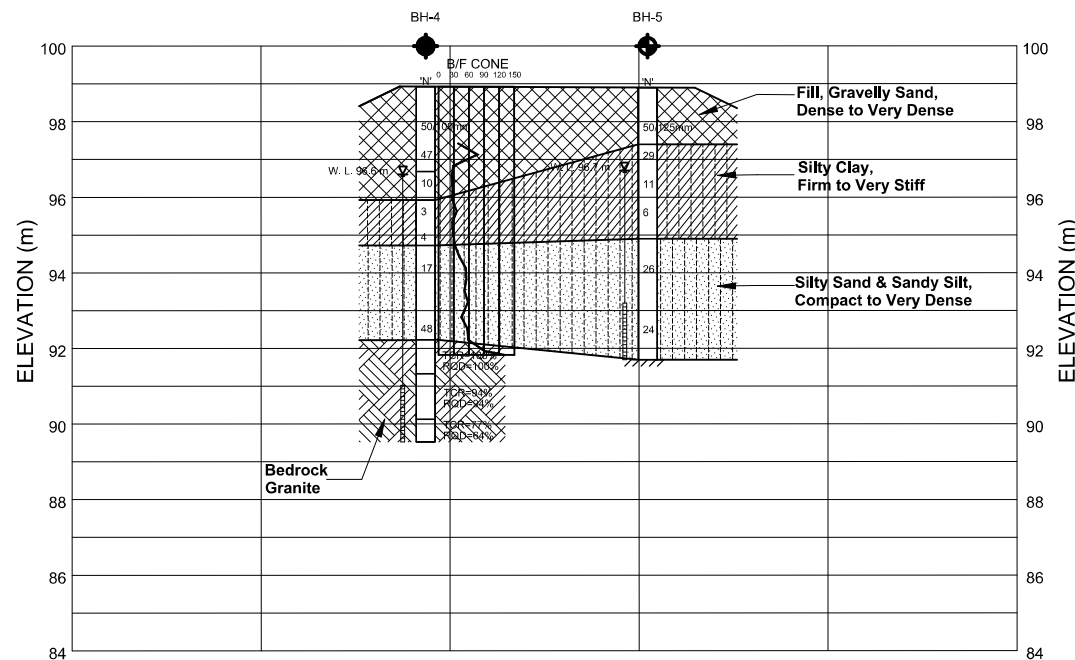
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KEY PLAN
NOT TO SCALE



SECTION B-B



SECTION C-C

LEGEND

- Bore Hole
- Bore Hole & Cone
- Benchmark Elev.=100.00
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- WL at time of investigation (March 2012)
- WL at time of survey
- WL in Piezometer
- Piezometer
- Auger Refusal

No	ELEVATION	NORTHING	EASTING
BH-1	99.160	5495448	303634
BH-2	99.108	5495449	303640
BH-3	99.145	5495447	303638
BH-4	98.890	5495445	303674
BH-5	98.850	5495448	303677
BH-6	98.880	5495447	303682

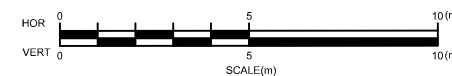
NOTES

Borehole elevations are based on local datum.

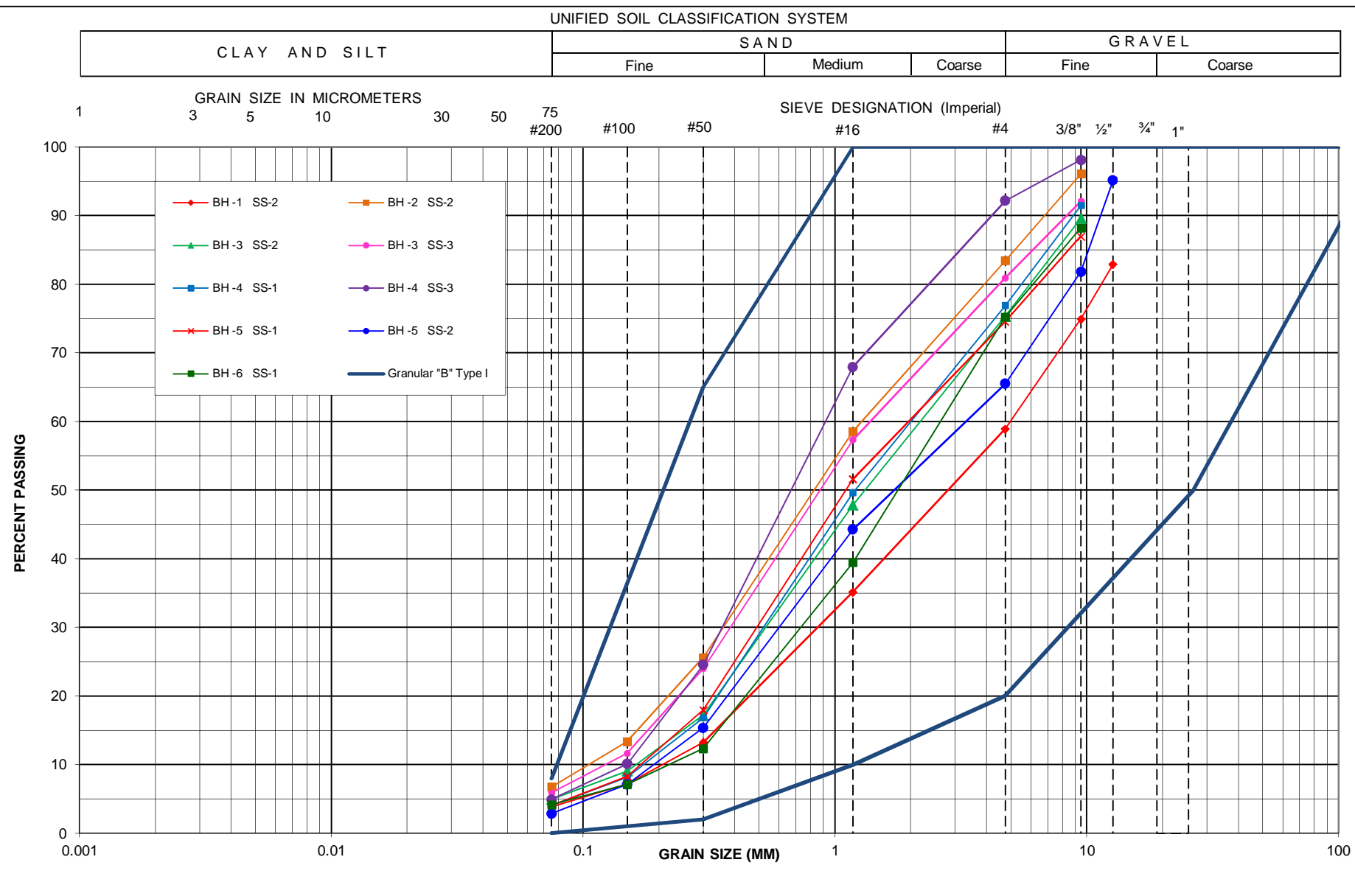
Note: Ground surface has not been surveyed.
Ground surface shown is based on BH elevations
and visual observation and is not to scale.

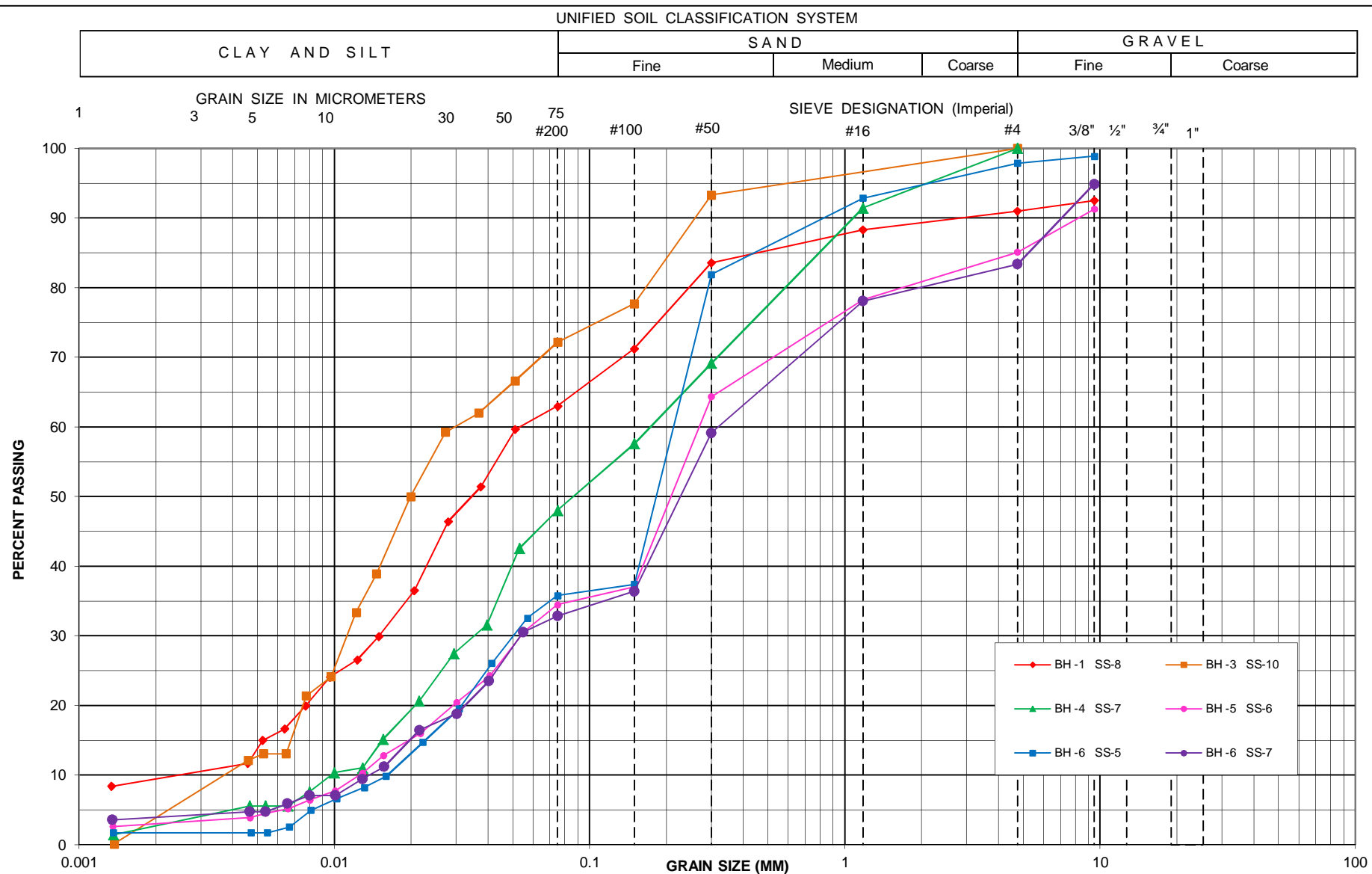
SOIL STRATA SYMBOLS

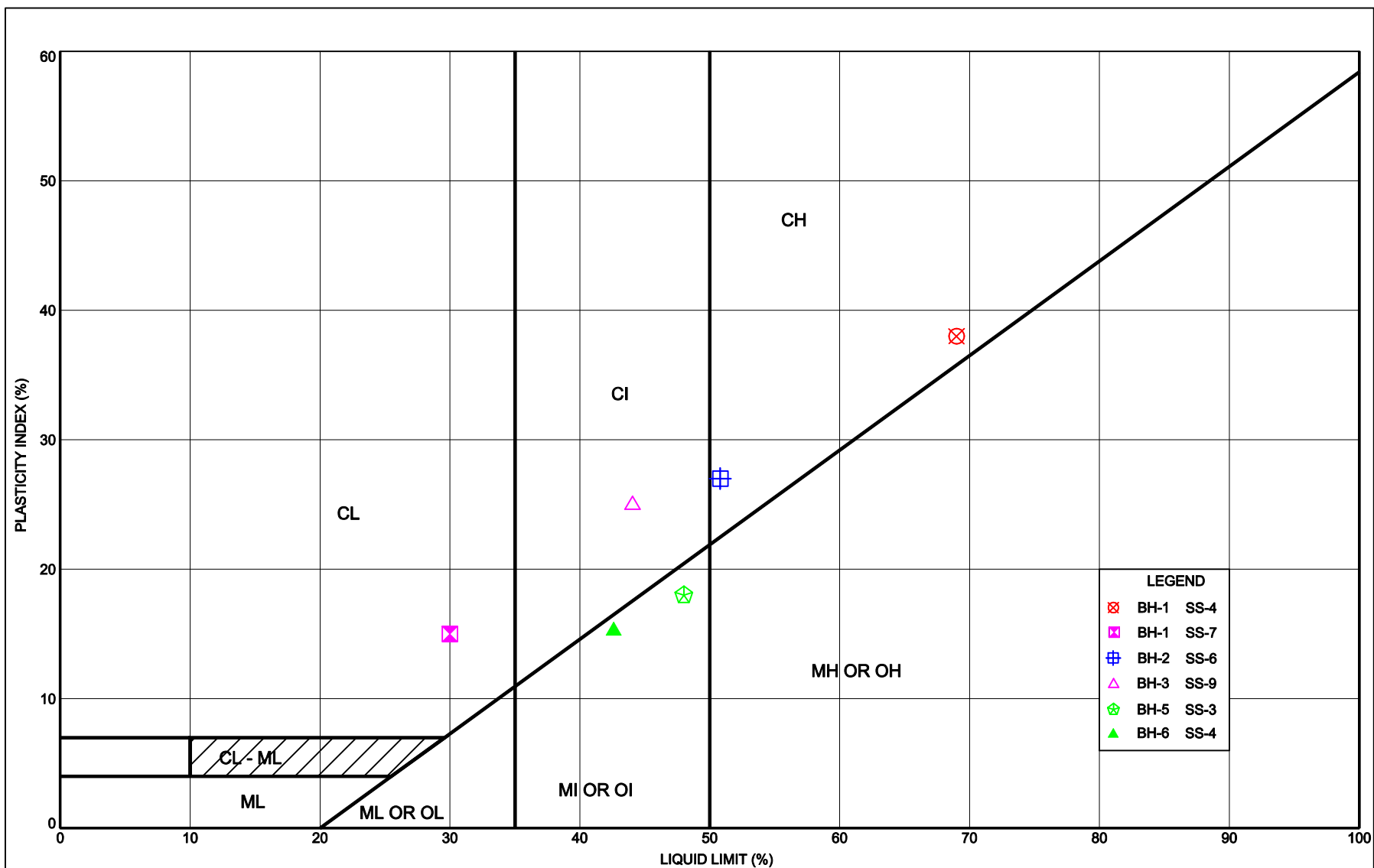
- Fill
- Silty Sand and Sandy Silt
- Silty Clay
- Bedrock




REVISIONS	FEB 2013	ZMO	Revision 03 – Final Report	
	DEC 2012	TJC	Revision 02 – Revised Draft – Issued for Exec. Review	
	July 2012	TJ	Revision 01 – Draft Reprt	
	DATE	BY	DESCRIPTION	
GEOCRES No 42G-41				
HWY No				DIST
SUBM'D	CHECKED CH	DATE	SITE 39W-50	
DRAWN	CHECKED CH	APPROVED	DWG 2B	





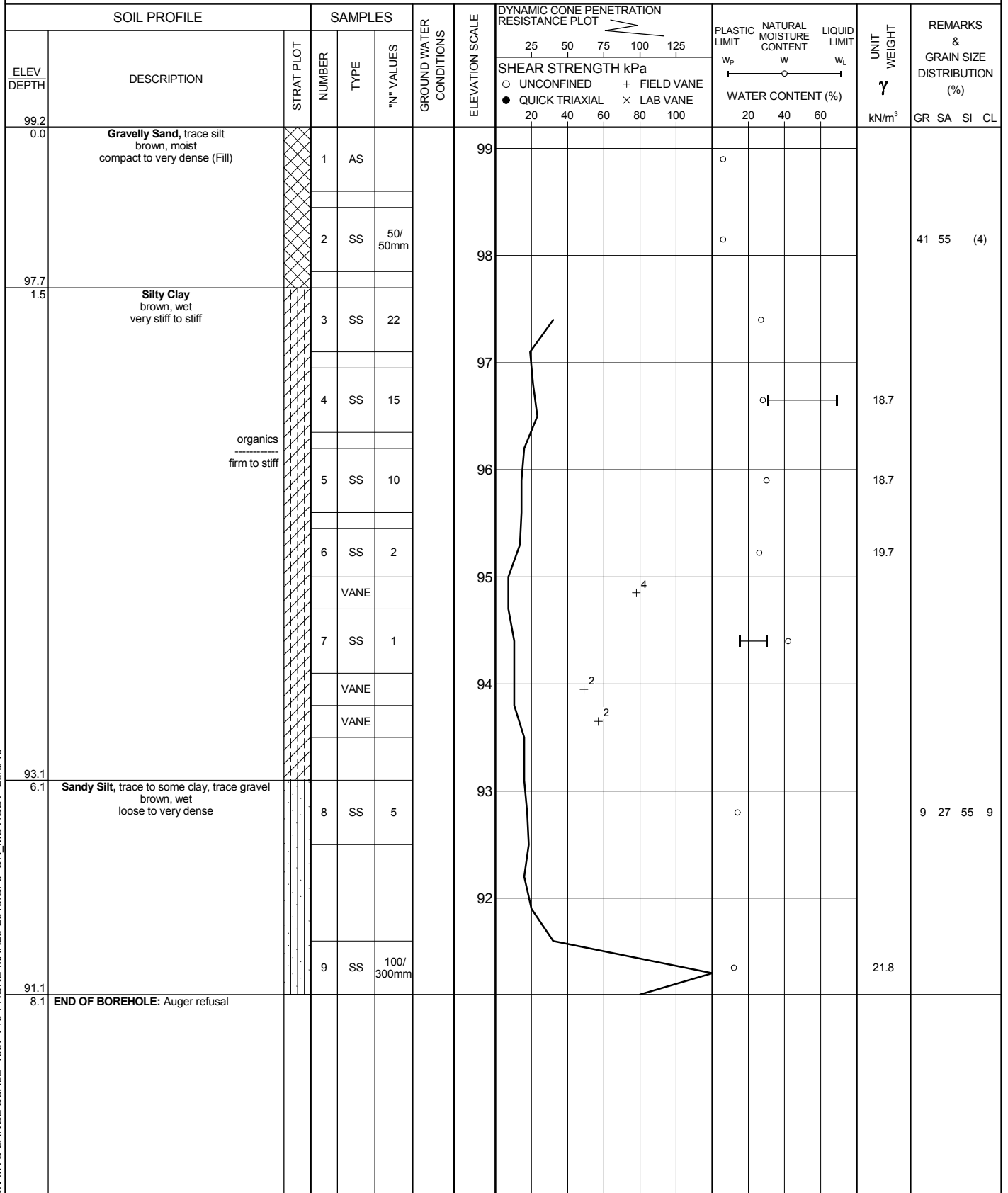


Client: Ministry of Transportation Ontario		Title: PLASTICITY CHART - Clayey Silt & Silty Clay		Project: Geotechnical Investigation Prune Creek Bridge Replacement	
Date: April 2012	Drawn: NT	Scale: N/A	Project No.: 1067-710	 SPL Consultants Limited Geotechnical • Environmental • Materials • Hydrogeology	
Original Size: LETTER	Approved: CH	Rev: 0	Drawing No.: 5		

Appendix A

Borehole Logs (Record of Borehole Sheets)

METRIC



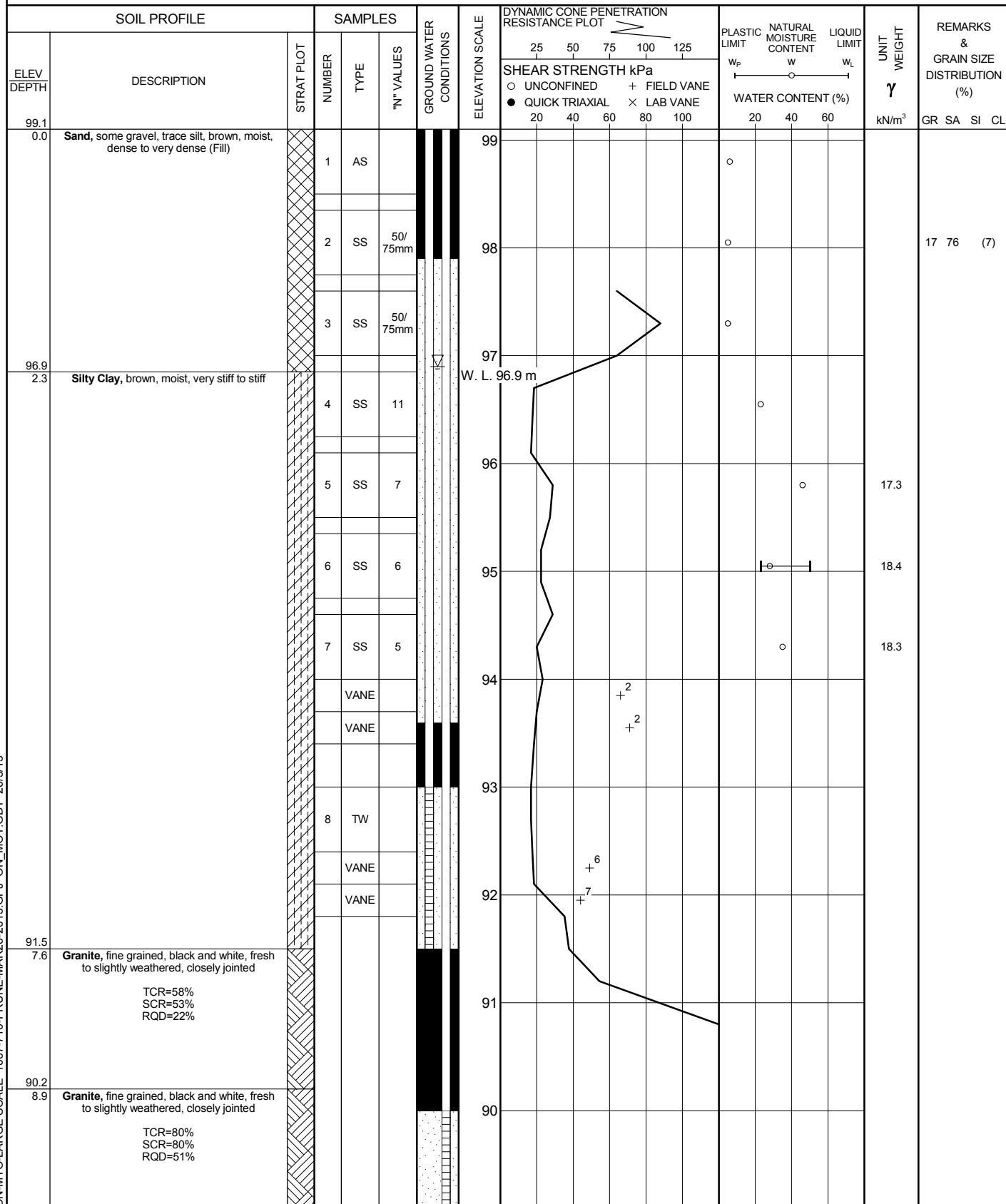
+³, ×³: Numbers refer to Sensitivity ○^{3%} STRAIN AT FAILURE

RECORD OF BOREHOLE No BH-2

1 OF 2

METRIC

W.P. 5033-07-00 LOCATION Prune Creek Bridge, E 303640, N 5495449 (See Borehole Location Plan) ORIGINATED BY NE
DIST _____ HWY Concession 12 BOREHOLE TYPE Hollow Stem Auger and N Size Core COMPILED BY NT
DATUM Local DATE Mar/20/2012 CHECKED BY CH



Continued Next Page

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

RECORD OF BOREHOLE No BH-2

2 OF 2

METRIC

W.P. 5033-07-00 LOCATION Prune Creek Bridge, E 303640, N 5495449 (See Borehole Location Plan) ORIGINATED BY NE
DIST HWY Concession 12 BOREHOLE TYPE Hollow Stem Auger and N Size Core COMPILED BY NT
DATUM Local DATE Mar/20/2012 CHECKED BY CH

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	25	50	75	100	125	W _p	W		
88.7	Granite , fine grained, black and white, fresh to slightly weathered, closely jointed TCR=71% SCR=67% RQD=42% END OF BOREHOLE Notes: 1) Water at 6.8m below surface before coring. 2) Two 19mm standpipe piezometers installed to 7.6m depth and 10.6m depth upon completion of drilling. 3) Water level in both piezometers at 2.8m					89										
10.4																
88.4																
10.7																

RECORD OF BOREHOLE No BH-3

1 OF 1

METRIC

W.P. 5033-07-00 LOCATION Prune Creek Bridge, E 303638, N 5495447 (See Borehole Location Plan) ORIGINATED BY NE
DIST _____ HWY Concession 12 BOREHOLE TYPE Hollow Stem Auger COMPILED BY NT
DATUM Local DATE Mar/21/2012 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	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			25	50	75	100	125					
99.1 0.0	Gravelly Sand , trace silt brown, moist dense to very dense (Fill)		1	AS			99										
			2	SS	50/ 25mm		98										25 70 (5)
			3	SS	50/ 25mm												19 75 (6)
96.9 2.3	Silty Clay brown, wet very stiff to stiff		4	SS	9		97										
			5	SS	6		96										
			6	SS	5		95										
			VANE														
			7	SS	5		94										
			VANE														
			8	SS	2		93										
			VANE														
			9	SS	2		92										
			VANE														
91.8 7.3	Sandy Silt , trace clay brown, wet very dense																
91.0 8.1	END OF BOREHOLE: Auger refusal		10	SS	77/ 150mm		91										0 27 71 2
	Notes: 1) 19mm standpipe piezometer installed upon completion. 2) Water level at 2.2m below road surface 2 days after installation.																

ON-MTO-LARGE SCALE 1067-710-PRUNE-MAR26-2013.GPJ ON_MOT.GDT 26/3/13

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

RECORD OF BOREHOLE No BH-4

1 OF 2

METRIC

W.P. 5033-07-00 LOCATION Prune Creek Bridge, E 303674, N 5495445 (See Borehole Location Plan) ORIGINATED BY NE
DIST _____ HWY Concession 12 BOREHOLE TYPE Hollow Stem Auger and N Size Core COMPILED BY NT
DATUM Local DATE Mar/22/2012 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			25	50	75	100	125					
98.9 0.0	Gravelly Sand, trace silt brown, moist very dense to dense (Fill)		1	AS													23 73 (4)
			2	SS	50/ 100mm		98										
			3	SS	47		97										8 87 (5)
96.7 2.3	Silty Clay, trace sand brown, moist stiff (Fill)		4	SS	10		96									18.8	
95.9 3.0	Silty Clay, brown, wet firm		5	SS	3		95									13.6	
			6	SS	4		94									16.5	
94.7 4.2	Silty Sand, trace clay brown, wet compact to dense		7	SS	17		93										0 51 47 2
			8	SS	48		92										
92.2 6.7	Granite, fine to medium grained, grey and white, fresh to slightly weathered, closely jointed TCR=100% SCR=100% RQD=100%						91										
91.3 7.6	Granite, fine to medium grained, grey and white, fresh to slightly weathered, closely jointed TCR=94% SCR=94% RQD=94%						90										
90.1 8.8	Granite, fine to medium grained, grey and white, fresh to slightly weathered, closely jointed																
89.5 9.4	TCR=77% SCR=77% RQD=64%																
	END OF BOREHOLE																

Continued Next Page

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

ON-MTO-LARGE SCALE 1067-710-PRUNE-MAR26-2013.GPJ ON_MOT.GDT 26/3/13

RECORD OF BOREHOLE No BH-4

2 OF 2

METRIC

W.P. 5033-07-00 LOCATION Prune Creek Bridge, E 303674, N 5495445 (See Borehole Location Plan) ORIGINATED BY NE
 DIST HWY Concession 12 BOREHOLE TYPE Hollow Stem Auger and N Size Core COMPILED BY NT
 DATUM Local DATE Mar/22/2012 CHECKED BY CH

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	25	50	75	100	125	W _p	W		
	Notes: 1) 19mm standpipe piezometer installed upon completion. 2) Water level in piezometer at 2.3m depth below road surface 1 day after installation.															

ON-MTO-LARGE SCALE 1067-710-PRUNE-MAR26-2013.GPJ ON_MOT.GDT 26/3/13

RECORD OF BOREHOLE No BH-5

1 OF 1

METRIC

W.P. 5033-07-00 LOCATION Prune Creek Bridge, E 303677, N 5495448 (See Borehole Location Plan) ORIGINATED BY NE
DIST _____ HWY Concession 12 BOREHOLE TYPE Hollow Stem Auger COMPILED BY NT
DATUM Local DATE Mar/22/2012 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			25	50	75	100	125					
98.9 0.0	Gravelly Sand, trace silt brown, moist dense to very dense (Fill)		1	AS													25 71 (4)
			2	SS	50/ 125mm		98										34 63 (3)
97.4 1.5	Silty Clay, with organic material dark brown, wet very stiff		3	SS	29		97										
			4	SS	11		96									18.8	
			5	SS	6		95										
94.9 4.0	Silty Sand, some gravel, trace clay brown, wet compact		6	SS	26		94										15 51 31 3
			7	SS	24		93										
							92										
91.7 7.2	END OF BOREHOLE: Auger refusal																
	Notes: 1) 19mm standpipe piezometer installed upon completion. 2) Water level at 2.2m depth below road surface 1 day after installation.																

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

ON-MTO-LARGE SCALE 1067-710-PRUNE-MAR26-2013.GPJ ON_MOT.GDT 26/3/13

RECORD OF BOREHOLE No BH-6

1 OF 1

METRIC

W.P. 5033-07-00 LOCATION Prune Creek Bridge, E 303682, N 5495447 (See Borehole Location Plan) ORIGINATED BY NE
DIST _____ HWY Concession 12 BOREHOLE TYPE Hollow Stem Auger COMPILED BY NT
DATUM Local DATE Mar/21/2012 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	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			25	50	75	100	125					
98.9 0.0	Gravelly Sand, trace silt brown, moist (Fill)		1	AS													25 71 (4)
98.0 0.9	Silty Clay, some sand brown, moist very stiff (Fill)		2	SS	31		98										
			3	SS	29		97										
96.8 2.1	Silty Clay brown, moist stiff		4	SS	11		96									18.7	
95.9 3.0	Silty Sand, trace clay, trace to some gravel brown, wet compact		5	SS	15		95										2 62 34 2
			6	SS	16		94										17 50 29 4
			7	SS	22		93										
92.8 6.1	Sandy Silt brown, wet compact		8	SS	32		92										
91.6 7.3	END OF BOREHOLE: Auger refusal																

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

ON-MTO-LARGE SCALE 1067-710-PRUNE-MAR26-2013.GPJ ON_MOT.GDT 26/3/13

Appendix B

Consolidation Test Results

CONSOLIDATION TEST SUMMARY**FIGURE****SAMPLE IDENTIFICATION**

Project Number	12-1183-0040	Sample Number	TW8
Borehole Number	12-2	Sample Depth, m	6.1

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	1		
Date Started	4/11/2012		
Date Completed	4/23/2012		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.54	Unit Weight, kN/m ³	17.08
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m ³	11.40
Area, cm ²	31.61	Specific Gravity, measured	2.77
Volume, cm ³	80.22	Solids Height, cm	1.065
Water Content, %	49.77	Volume of Solids, cm ³	33.68
Wet Mass, g	139.71	Volume of Voids, cm ³	46.55
Dry Mass, g	93.28	Degree of Saturation, %	99.7

TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t ₉₀ sec	c _v cm ² /s	m _v m ² /kN	k cm/s
0.00	2.538	1.382	2.538				
6.16	2.535	1.379	2.536	1109	1.23E-03	1.98E-04	2.39E-08
10.96	2.529	1.373	2.532	974	1.40E-03	5.17E-04	7.07E-08
20.83	2.516	1.361	2.522	1245	1.08E-03	5.15E-04	5.47E-08
40.10	2.490	1.337	2.503	1434	9.26E-04	5.34E-04	4.84E-08
78.96	2.436	1.286	2.463	1382	9.30E-04	5.45E-04	4.97E-08
156.27	2.282	1.142	2.359	3894	3.03E-04	7.84E-04	2.33E-08
311.14	2.105	0.976	2.193	2667	3.82E-04	4.51E-04	1.69E-08
620.72	1.969	0.848	2.037	1949	4.51E-04	1.73E-04	7.67E-09
1240.30	1.849	0.736	1.909	1109	6.96E-04	7.60E-05	5.19E-09
2440.06	1.744	0.637	1.796	866	7.90E-04	3.46E-05	2.68E-09
1240.30	1.751	0.644	1.747				
311.14	1.801	0.690	1.776				
78.96	1.859	0.745	1.830				
20.83	1.917	0.799	1.888				
6.16	1.959	0.839	1.938				

Note:

k calculated using c_v based on t₉₀ values.

Specimen taken 20cm from bottom of the tube.

Loading stages assigned by the client.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.96	Unit Weight, kN/m ³	19.38
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m ³	14.77
Area, cm ²	31.61	Specific Gravity, measured	2.77
Volume, cm ³	61.93	Solids Height, cm	1.065
Water Content, %	31.23	Volume of Solids, cm ³	33.68
Wet Mass, g	122.41	Volume of Voids, cm ³	28.26
Dry Mass, g	93.28		

Prepared By: LFG

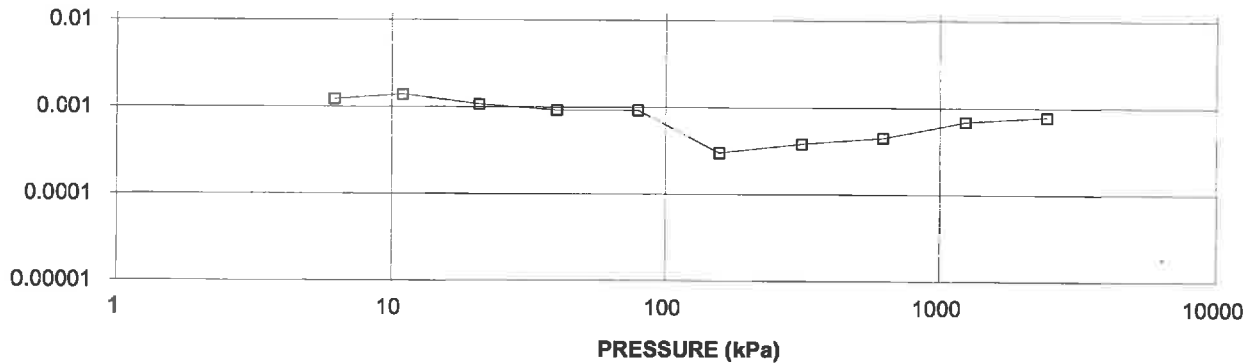
Golder AssociatesChecked By: 

CONSOLIDATION TEST SUMMARY

FIGURE

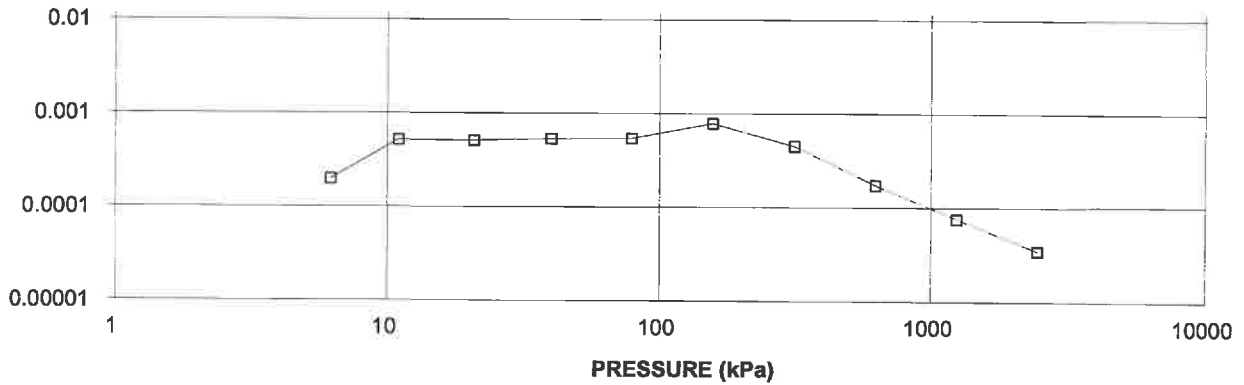
COEFFICIENT OF CONSOLIDATION,
cm²/s

CONSOLIDATION TEST
C_v cm²/s VS PRESSURE (kPa)
BH 12-2 SA TW8



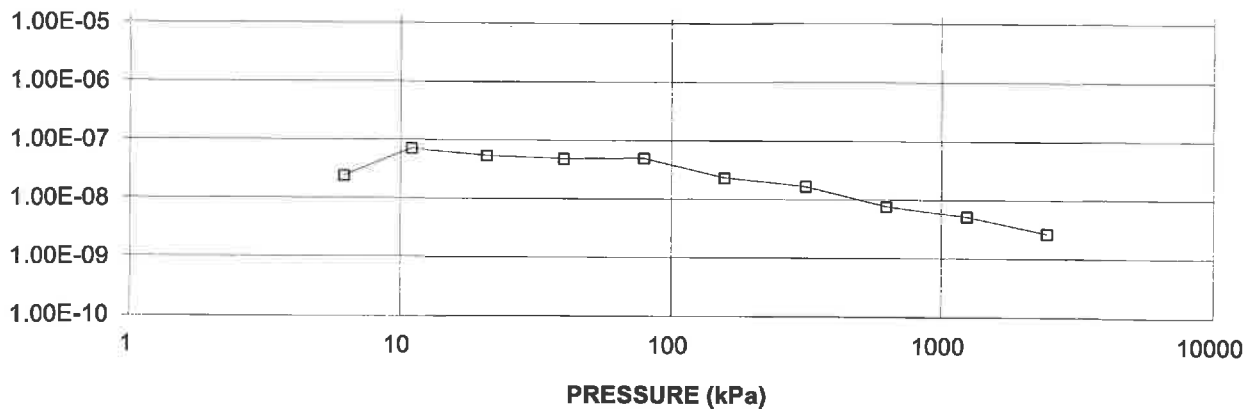
VOLUME COMPRESSIBILITY, m²/kN

CONSOLIDATION TEST
M_v m²/kN vs PRESSURE (kPa)
BH 12-2 SA TW8



HYDRAULIC CONDUCTIVITY, cm/s

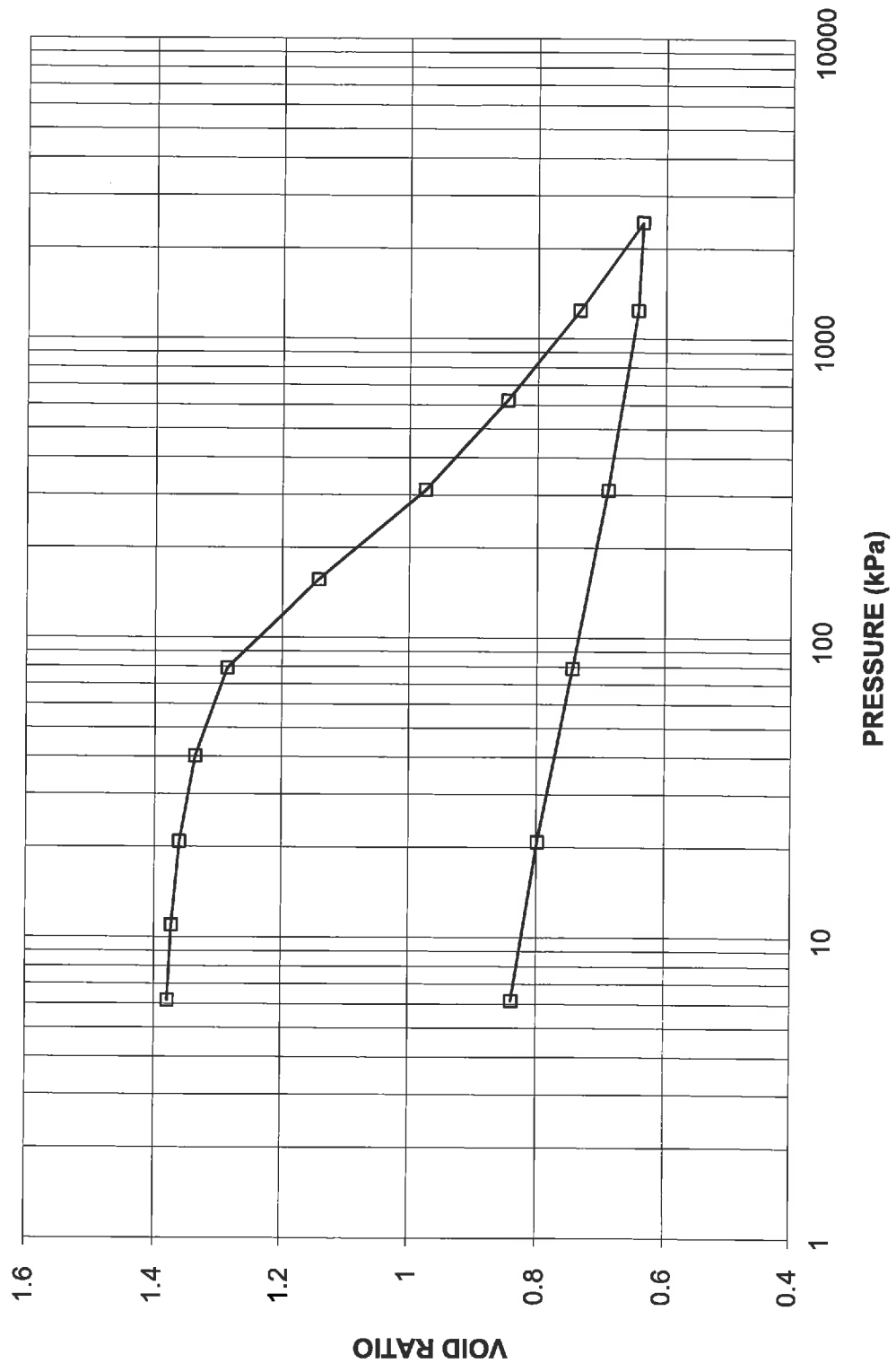
CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BH 12-2 SA TW8



**CONSOLIDATION TEST
VOID RATIO VS LOG PRESSURE**

FIGURE

**CONSOLIDATION TEST
VOID RATIO vs PRESSURE
BH 12-2 SA TW8**



Appendix C

Chemical Test Results

Client: SPL Consultants Ltd.
146 Colonnade Rd., Unit 17
Ottawa, ON
K2E 7Y1
Attention: Mr. Neem Tavakkoli
PO#: Visa
Invoice to: SPL Consultants Ltd.

Report Number: 1207144
Date Submitted: 2012-04-18
Date Reported: 2012-04-24
Project:
COC #: 156562

Group	Analyte	MRL	Units	Guideline	Lab I.D.	Sample Matrix	Sample Type	Sampling Date	Sample I.D.
					952021	952022	952023		
Agri. - Soil	Electrical Conductivity	0.05	mS/cm		Soil	Soil	Soil		
	pH	2.0			2012-03-20	2012-03-20	2012-03-20		
General Chemistry	Cl	0.002	%		BH-2/SS-3	BH-3/SS-5	BH-5/SS-6		
	Resistivity	1	ohm-cm						
	SO4	0.01	%						

Guideline =*** = Guideline Exceedence**

Results relate only to the parameters tested on the samples submitted.

Methods references and/or additional QA/QC information available on request.

MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial Water Quality Guideline, IPWQO = Interim Provincial Water Quality Objective.

Client: SPL Consultants Ltd.
146 Colonnade Rd., Unit 17
Ottawa, ON
K2E 7Y1
Attention: Mr. Neem Tavakkoli
PO#: Visa
Invoice to: SPL Consultants Ltd.

Report Number: 1207144
Date Submitted: 2012-04-18
Date Reported: 2012-04-24
Project:
COC #: 156562

QC Summary

Analyte	Blank	QC % Rec	QC Limits
Run No 230492 Analysis Date 2012-04-19 Method C CSA A23.2-4B			
Cl	<0.002 %	105	90-110
Run No 230655 Analysis Date 2012-04-24 Method Ag Soil			
Electrical Conductivity	<0.05 mS/cm	112	80-120
pH		101	90-110
Resistivity			
SO4	<0.01 %	92	70-130

Guideline =*** = Guideline Exceedence**

Results relate only to the parameters tested on the samples submitted.
Methods references and/or additional QA/QC information available on request.

MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial Water Quality Guideline, IPWQO = Interim Provincial Water Quality Objective.

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 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						

**FOUNDATION DESIGN REPORT
PROPOSED BRIDGE REPLACEMENT
PRUNE CREEK BRIDGE NEAR HEARST, ONTARIO
SITE NO. 39W-50
G.W.P. 5033-07-00
MTO GEOCRES NO. 42G-41**

Prepared for:

ONTARIO MINISTRY OF TRANSPORTATION

By:

SPL CONSULTANTS LIMITED

Project: 1067-710 (Prune Creek Bridge)
March 2013



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6. DISCUSSION AND RECOMMENDATIONS

6.1 General

The subsurface conditions encountered in the boreholes drilled at the site include a layer of fill which forms the existing approach embankments. This fill material is underlain by native soils which include a layer of firm to stiff silty clay followed by a layer of compact to very dense silty sand and sandy silt. The silty clay is generally thicker and the underlying silty sand/sandy silt layer thinner on the west side of the bridge than on the east. The soils at the site are underlain by bedrock at a depth of 7 m to 8 m below the existing ground surface.

Groundwater levels in both the native soils and the rock were found to be at an elevation of approximately 96.7 m to 96.9 m (in the local datum), which is slightly above the water level in the creek at the time of the investigation.

The proposed bridge structure (based on preliminary General Arrangement drawings provided to us) is a pre-fabricated bridge with two abutments and no intermediate piers (see Drawing 2). It is understood the location, alignment and elevation of the new bridge will be identical to the old one, and that no re-grading or widening of the road and existing embankments will be required.

6.2 Frost Protection

The depth of frost penetration for the Prune Creek Bridge site may be assumed to be 2.6 m. All foundation elements should therefore have a permanent soil cover of at least 2.6 m (or its thermal equivalent if artificial insulation is used). Riprap or rock fill should be assumed to be only 50% effective in providing frost protection (i.e. an equivalent thickness of 50% of the actual layer thickness should be used to assess frost protection).

The uppermost fill within the frost zone is sand and gravel which would be considered to have a low susceptibility to frost heave. The depth to more frost susceptible silty clay soils was found to be 1.5 m to 2.3 m on the west side of the bridge and 0.9 m to 2.3 m on the east side, which is within the frost zone.

The existing road is a gravel-surfaced rural road, and it is assumed that there are no plans to pave the road in the future. It is therefore assumed that seasonal maintenance of the road will be carried out such that some differential frost heaving and settlement is acceptable. If that is the case then the installation of frost tapers between the structure backfill and the silty soils within the frost zone is not required. If seasonal maintenance of the road cannot be tolerated then frost tapers should be added between the granular backfill and the existing silty soils.

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 Bridge; Performance Zone 2 if the crossing is considered a Lifeline), and Zonal Acceleration Ratio of $A = 0$ (CHBDC Section 4.4).

For the purposes of assessing the effects of site conditions under seismic loading, 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

6.4.1 Foundation Options

A layer of relatively soft compressible clay is present across the site and is approximately 5 m thick in the area below the proposed west abutment. If shallow foundations are used to support the bridge then this clay layer would pose a risk of foundation settlement.

Deep foundations are therefore considered the most suitable foundation type for this structure. It is also noted that the existing bridge is supported on piled foundations.

Table 5 presents a comparison of typical foundation options for the site.

6.4.2 Deep Foundations

It is recommended that deep foundations be used to support the bridge structure. Several types of piles could potentially be used, however given the relatively remote location it is likely that the most economical foundation system would be steel piles driven to rock. Rock was encountered at 91.5 m elevation on the west side of the bridge and 92.2 m elevation on the east side. Similar tip elevations would therefore be expected for driven pile foundations.

All pile foundations should be designed and constructed in accordance with the CHBDC and OPSS 903.

6.4.2.1 Axial Capacity of Piled Foundations

Steel piles driven to granite bedrock typically generate relatively high ultimate geotechnical capacities, equal to or in excess of the structural capacity of the steel section. For the purposes of preliminary design, and HP310x110 (as one example) could be assumed to generate a factored geotechnical resistance on the order of 1,975 kN at ULS (based on the cross-sectional area of the steel, $f_y = 350$ MPa and a resistance factor of 0.4).

Settlements for piles driven to rock are typically negligible, and the geotechnical resistance mobilized at the SLS condition would be expected to exceed the factored ULS value. SLS conditions therefore do not generally govern the design of piles driven to rock.

Table 5: Summary of Foundation Alternatives
Foundation Investigation and Design Report
Proposed Prune Creek Bridge Replacement, Site No. 39W-50

Foundation Option	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Shallow Foundations	<ul style="list-style-type: none"> Routine excavation and construction procedure Usually the cheapest construction method 	<ul style="list-style-type: none"> Possibility of poor performance due to settlement of underlying clay soils. Weather dependant 	<ul style="list-style-type: none"> Typically cheaper than deep foundations 	<ul style="list-style-type: none"> Risk of long-term settlement of foundations Risk of difficulties with dewatering and excavation and disturbance of subgrade Risk of requiring removal of unsuitable soils and replacement with additional engineered fill
Driven Pile Foundations	<ul style="list-style-type: none"> Improved foundation performance, particularly with respect to settlement Foundation capacity can be tested/confirmed during construction Usually faster installation than cast-in-place concrete 	<ul style="list-style-type: none"> Usually more expensive than shallow foundations Requires specialist contractor 	<ul style="list-style-type: none"> Usually more expensive than shallow foundations 	<ul style="list-style-type: none"> Risk of difficulties during pile driving

6.4.2.2 Uplift Capacity of Piled Foundations

The uplift capacity of piled foundations is developed as a result of skin friction along the surface of the pile. The average unfactored ultimate shaft friction along the pile shaft may be taken as 50 kPa for the purposes of calculating the uplift resistance. A resistance factor of 0.3 should be applied to the total resistance as per the CHBDC. The dead weight of the pile itself (with an appropriate resistance factor for dead weight) may also be included in the uplift resistance.

The total uplift resistance of a pile group is the lesser of the sum of the individual pile resistances as described above, or the resistance of a single “block” of soil with a perimeter equal to the perimeter of the pile group. The mass of the soil inside the “block” may be included in the calculation; use 18 kN/m³ for the soil unit weight.

6.4.2.3 Pile Lateral Resistance

Lateral loads may be resisted by battered piles, or by the lateral resistance of vertical piles.

The lateral resistance of piles is typically governed by limiting the deflection which will occur under loading to some acceptable level. The geotechnical parameter most commonly used to determine

lateral deflection of piles is the coefficient of horizontal subgrade reaction (k_h). For this site k_h may be assumed to be:

$$k_h = 67 S_u/B$$

Where:

S_u = the undrained shear strength (use 50 kPa);

B = the pile diameter or width in the direction of loading (m)

This parameter is associated with acceptable deflections, and therefore represents an SLS value.

The value above is for a single pile. Group interaction must be considered when piles are spaced closely together. Group effects may be accounted for by reducing the coefficient of horizontal subgrade reaction (k_h) by an appropriate factor as follows:

Table 6 – Coefficient of Horizontal Subgrade Reaction Reduction Factors

Pile Spacing in Direction of Loading (d = pile diameter)	Reduction Factor
6d	1.0
3d	0.25

Values for other pile spacings may be interpolated from the above. No reduction is required for the first row of piles (i.e. the row which bears against undisturbed soil with no piles in front).

Other more rigorous (and typically less conservative) methods of evaluating the lateral resistance of piles exist, and may be used. These methods, however, require knowledge of the proposed loading and pile properties, and are therefore best addressed in detailed design.

For reference, Table 6.8.7.1 (a) included with the commentary to the CHBDC also provides values of assessed lateral resistances for typical pile sections at ULS and SLS conditions for preliminary design (assume stiff cohesive material).

6.4.2.4 Negative Friction and Downdrag

The new bridge will be constructed in the same location as the existing structure, and there are no plans to raise the embankment. If this is the case then the magnitude of potential downdrag forces on the piles would be expected to be minimal for the relatively short piles.

6.4.2.5 Construction Considerations

Based on the results of the drilling program at the site it is expected that the piles will need to be driven to a tip elevation of approximately 91.5 m on the west side, and to 92 m on the east side (in the local

datum). Piles will be driven to relatively strong bedrock; significant pile penetration into the rock is not expected.

It should be noted that the existing bridge abutments are founded on timber piles. The locations of the new piles will need to ensure that they are not unintentionally driven into the old foundation piles (which presumably will simply be cut off below grade and left in place).

Battered piles should be driven with rock points to avoid sliding of piles along the rock surface. Historical drawings of the bridge show the rock surface to be dipping into the creek. This may cause construction problems for any battered piles which also slope towards the creek.

Pile driving criteria are dependent upon the pile type, length, design load and driving equipment used. A preliminary pile driving criteria should be established prior to construction and confirmed through a program of dynamic testing (PDA testing) carried out during installation.

6.5 Earth Pressures and Backfilling

Backfill for pile caps, retaining walls and other below-grade structures should consist of suitable free-draining granular fill material placed and compacted in accordance with MTO standards, and should conform to the applicable OPSDs. For fills below the groundwater table, or immediately below the roadway, it is recommended that Granular 'A' or Granular 'B' (OPSS 1010) fill should be used.

Fill material should be placed in shallow lifts, not exceeding 300 mm loose thickness. Granular 'A' and Granular 'B' materials immediately below the roadway should be compacted to 100% of their Standard Proctor Maximum Dry Density (SPMDD). Remaining fills may be compacted to a minimum of 98% of SPMDD. All compaction should be carried out in accordance with OPSS 501.

To avoid damaging or laterally displacing the structures, care should be exercised when compacting fill adjacent to structures. In these locations compaction equipment should be restricted in size to prevent damage to the structures. Backfilling should be carried out on both sides of buried structures simultaneously.

Computation of earth pressures acting against buried 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) = 30 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	0.33	0.42	0.54
K_b	0.41	0.52	0.64
K_0	0.50	0.66	0.76
K^*	0.57	0.74	0.86

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.

According to 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 essentially equal to the earth pressure under static conditions (because the horizontal seismic coefficient, k_h is 0.5 or 1.5 times the Zonal Acceleration Ratio, and for the design earthquake $A = 0$).

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.

6.6 Embankment Stability and Settlement

It is understood that the new bridge will be constructed in the same location as the existing structure, and that the existing embankment will not be raised or widened. It is further understood that the performance of the existing embankment slopes has been adequate in the past. No history of instability is indicated in the maintenance files for the site.

6.6.1 Embankment Stability

Based on the conditions encountered in the boreholes, foundation failures are not anticipated for the existing embankments with normal (2H:1V or flatter) slopes. The slope into the existing creek is currently constructed at an angle of 2:1 to 3:1 and there is no history of instability indicated in the historical files for the site.

The new bridge abutments will be supported on deep foundations and will not provide any additional loading or surcharge to the creek bank, and therefore would not be expected to adversely impact the stability of the existing slopes.

6.6.2 Embankment Settlement

It is understood the new bridge will be constructed in the same location as the old bridge, such that the existing embankments will be incorporated into the design (and no additional fill is required). The existing bridge was constructed in 1950, and so the majority of the settlement due to the embankment loading will have already occurred over the life of the existing structure. In this case, future settlement is expected to be minimal (less than 25 mm) and should be accommodated by the routine re-grading and maintenance of the existing gravel road.

6.7 Erosion Protection

The foundation recommendations in Section 6.4 and the above discussion regarding the stability of the slopes at the abutments assume no undermining due to erosion or scour of the native soils by the creek. The soils in the creek bed would be expected to include relatively soft silty clay and silty sand/sandy silt, and would be susceptible to erosion given high enough flow velocities. Erosion and scour protection or adequate set-backs from the water-course will be required at the crossing location to ensure the creek does not negatively impact the foundations. The existing bridge abutments incorporate rip-rap erosion protection. The design of erosion protection should be carried out by a specialist who is familiar with the site and the findings of this investigation.

6.9 Construction Staging

The contract drawings provided to SPL indicate that no construction staging is required. Traffic will be diverted around the site on existing roads and the bridge replaced in one operation.

6.9 Additional Construction Considerations

Construction Dewatering

The groundwater level within the glacial till at the site was found to be at approximately 96.6 m to 96.9 m local elevation, or 2 m to 2.5 m depth below the existing road surface.

The groundwater level would be expected to be sensitive to changes in the water level in the creek, and for this reason it is recommended that construction be carried out when the creek is at as low a level as possible. If construction is carried out at a time when the creek is at a higher level than in March 2012 then a corresponding increase in groundwater levels should be expected. In particular, if the level of the creek is above the existing granular fill embankment, it should be expected that the embankment will be saturated to the level of the creek.

For shallow excavations in the silty clay to a depth of around 2.6 m (such as those for frost protection of abutments, pile caps, retaining walls, etc.) it is anticipated that seepage will be manageable using properly filtered sumps and ditches.

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

For preliminary planning, the soils at the site include primarily granular fill in the embankment and pavement structure, firm cohesive fill material which forms the existing embankment and native stiff silty clay. The fill and native silty clay soil may be may be classified as Type 3 Soil above the water table and as Type 4 Soil below the water table. These soil classifications should be confirmed in detailed design based on conditions identified during construction by a competent person based on the soil conditions exposed in the excavations.

If temporary shoring is required it would generally consist of soldier piles and timber lagging or interlocking sheet piles. It should be noted that cobbles and boulders could be encountered in the granular silty sand and sandy silt soils which are relatively thick on the east side of the bridge. In addition rock is expected to be present at relatively shallow depth which should also be considered when selecting shoring systems and installation methods.

6.10 Corrosion and Cement Type

Two soil samples were submitted to Exova Accutest for testing related to soil corrosivity and potential exposure of concrete elements to sulphate attack. The results of these tests are included in Appendix B and are summarized in Table 7 below.

Table 7 – Results of Soil Corrosivity Testing

Sample No.	Soil Type	Soil Parameter			
		Resistivity (ohm-cm)	pH	Chloride (%)	Sulphate (%)
BH-2 SS3	Fill	7690	8.4	0.003	0.05
BH-3 SS5	Silty Clay	2500	8.0	0.007	0.03
BH-5 SS6	Silty Sand	8330	8.4	0.003	0.04

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

The test results (soil resistivity) indicate that there is a moderate potential for corrosion of buried steel elements within the silty clay (which is the uppermost native soil present below the fill embankment). Appropriate care should be taken in designing the corrosion protection requirements for any buried steel structures.

7. CLOSURE

The field investigations were supervised by Mr. Naeem Ehsan, P.Eng. This report was prepared by Mr. Chris Hendry, P.Eng. Mr. Fanyu Zhu, P.Eng., who is the project manager and SPL's designated MTO contact and Mr. Shaheen Ahmad, P.Eng., who is the project quality control auditor, provided quality control and independent review of the technical aspects of this report.

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

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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-S6-00 Canadian Highway Bridge Design Code, December 2011

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

Relevant Ontario Provincial Standard Specifications

OPSS NO.	TITLE
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
301	Restoring Unpaved Roadway Surfaces
501	Compacting
504	Preservation, Protection and Reconstruction of Existing Facilities
506	Dust Suppressants
510	Removals
518	Control of Water from Dewatering Operations
539	Temporary Protection Systems
805	Temporary Erosion and Sediment Control Measures
902	Excavating and Backfilling – Structures
903	Deep Foundations
942	Prestressed Soil and Rock Anchors
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...
105S21	Amendment to OPSS 501, November 2010
110S13	Amendment to OPSS 1010, April 2004
199S55	Record Drawings for Structures and Foundations
539S02	Protection System – Amendment to OPSS 512, April 2011
805F01	Light-Duty Sediment Barriers, etc.