

**FOUNDATION INVESTIGATION AND DESIGN REPORT
PROPOSED BRIDGE REPLACEMENT
OBJIOU CREEK BRIDGE NEAR HEARST, ONTARIO
SITE NO. 39W-59
G.W.P. 5044-11-00
MTO GEOCRETS NO. 42G-39**

Prepared for:

ONTARIO MINISTRY OF TRANSPORTATION

By:

SPL CONSULTANTS LIMITED

Project: 1067-710 (Objiou 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 Obijou Creek on Grenier Road, west of Highway 583 approximately 28 km south of Hearst, Ontario.

The Terms of Reference (TOR) for this investigation are outlined in the Request for Quotation (RFQ) 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 Grenier Road approximately 300 m east of Highway 583, 28.3 km south of Highway 11 near Hearst, Ontario (see Drawing 1).

The existing structure is a five span timber bridge supported on timber beams. The five spans range from 4.1 m to 8.8 m in length and are supported on a combination of timber and steel piles (two supports on timber and two on steel). The bridge deck is approximately 5.5 m wide. The existing road is 7.5 m wide and gravel surfaced on both sides of the bridge.

The elevation of the road in the general vicinity of the crossing is approximately 99.5 m on the west side of the bridge and 99.2 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 about 98 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.

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.

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

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 for the site was also carried out in North Bay. The files contain a copy of an inspection report carried out in September 2009. The inspection report indicates previous inspections were also carried out in 2004, 2006 and 2007. The inspection report indicates that the bridge was constructed in 1955. Foundations for the bridge comprise the original timber piles and timber bents, as well as steel bents and piles which were installed subsequent to the original construction (the date is not known) to provide additional support to the bridge structure.

No as-built drawings of the bridge structure or foundations, or records of previous foundation investigations were present in the MTO files reviewed.

3.2 Field Investigation

The field investigation included drilling a total of 6 boreholes at the crossing location (BH-1 through BH-6). Due to access restrictions, the investigation was carried out in two stages. The east side of the bridge (BH-4 through BH-6) was completed between March 23rd and March 26th, 2012 while the investigation on the west side (BH-1 through BH-3) was completed on May 1st and May 2nd, 2012.

The boreholes were advanced using truck and track-mounted drill rigs supplied and operated by Landcore Drilling of Chelmsford, ON and Walker Drilling Ltd. of Utopia, ON. The boreholes were drilled using a combination of hollow-stem auger drilling, rotary drilling and rock coring to depths ranging from 7.7 m to 24.6 m below the existing ground surface. During drilling, sampling and in-situ testing including Standard Penetration (SPT) Testing, Dynamic Cone Penetration Testing (DCPT) Testing and field shear vane testing, were carried out at regular intervals.

Standpipe piezometers were installed in Boreholes 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. Upon completion of the field program all piezometers were backfilled and abandoned.

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

3.3 Laboratory Testing

During drilling and in-situ testing, disturbed and relatively undisturbed (Shelby tube) soil samples were obtained at regular intervals. 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, 6 and 9. The results of Atterberg limits (plasticity) testing are presented on the individual borehole logs and summarized in Drawing Nos. 4, 5, 7 and 8. 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

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 a layer of fill was encountered which forms the road structure, as well as the existing embankment. The fill material comprises both granular fill and silty clay and clayey silt fill.

The uppermost portion of the fill is a layer of granular fill which ranges from silty sand to gravelly sand. This layer extended to a depth of 0.8 m in all of the boreholes, with the exception of BH12-1 and BH12-4 where it extended to a depth of 2.3 m and 3.0 m, respectively.

The grain size curves for several samples of the granular 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 some portions of the materials in the field.

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

Borehole No.	Sample No.	Grain Size Distribution		
		% Gravel	% Sand	% Silt & Clay
BH-1	1	10	83	7
BH-1	3	3	83	14
BH-2	1	17	73	10
BH-3	1	26	69	5
BH-4	1	35	49	16
BH-4	2	28	61	11
BH-4	3	14	77	9

Borehole No.	Sample No.	Grain Size Distribution		
		% Gravel	% Sand	% Silt & Clay
BH-4	4	20	69	11
BH-5	1	11	81	8
BH-6	1	26	68	6

The consistency of the fill material (as interpreted based on SPT “N” values which range from 7 to greater than 100, but are typically between 7 and 24) ranged from loose to very dense, but in most places would be considered compact. Very high “N” values may reflect the presence of cobbles and boulders, rather than a very high density of the soil matrix itself.

At some locations, the lower portion of the fill layer comprised a mix of silty clay and clayey silt. This fill was encountered in Boreholes BH-2 and BH-3 on the west side of the bridge and BH-5 and BH-6 on the east side. It was not encountered in Boreholes BH-1 and BH-4.

The consistency of this silty clay and clayey silt fill is described as firm to stiff. Measurements of the soil unit weight yielded values of 19.2 kN/m³ to 20.0 kN/m³ with an average value of 19.6 kN/m³. Atterberg limits (plasticity) testing was carried out on two samples of the cohesive fill. The results (which are presented in Drawing 4) indicate the samples to be medium plasticity silt and high plasticity clay (BH-5 Sample 2 and BH-2 Sample 2, respectively).

The fill extended to a depth of 2.3 m on the west side of the bridge. This corresponds to elevations of 97.1 m to 97.3 m. On the east side of the bridge the fill extended to a depth of 2.3 m to 3.0 m, which corresponds to elevations of 96.2 m to 96.7 m.

4.1.2 Stiff Silty Clay

The uppermost natural soil is a layer of silty clay which was encountered in all of the boreholes drilled at the site.

The results of plasticity testing on nine samples of the silty clay indicate the soil would be classified as low to medium plasticity clay (see Drawing No. 5).

SPT “N” values within the silty clay range from 2 to 12 with an average value of approximately 6. Field vane tests indicate undrained shear strengths ranging from 65 kPa to greater than 100 kPa (with the majority being greater than 100 kPa). Based on the results of the field vane testing the majority of the silty clay is considered to be stiff to very stiff.

Unit weight measurements on samples of the silty clay yielded values of 19.3 kN/m³ to 20.3 kN/m³ with an average value of 19.8 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.

The silty clay is present on both sides of the bridge. On the west side of the bridge the silty clay layer is 5.1 m to 5.4 m thick, with the base of the layer at an elevation of 91.8 to 92.1 m. On the east side of the bridge the silty clay layer is 4.6 m to 5.4 m thick, with the base of the layer at an elevation of 91.3 m to 91.6 m.

The uppermost zone of the silty clay was found to contain a significant fraction of organic material (organic soil, peat, wood debris, etc.) This suggests the area was not stripped prior to construction of the original bridge in 1955. This zone was encountered in Borehole BH-3 (on the west side below the proposed abutment) and BH-4, BH-5 and BH-6 on the east side. This organic zone ranged in thickness from 0.6 m in to 1.8 m at the borehole locations, but could be thicker elsewhere. At the location of the two bridge abutments the organic layer was found to be 1.6 m to 1.8 m in thickness.

4.1.3 Upper Till (Silty Clay and Clayey Silt Till)

The silty clay layer is underlain by a discontinuous layer of clayey silt till (referred to as the “upper” till as distinct from a “lower” till layer discussed in Section 4.1.5 below). The grain size distributions of two of the coarser samples from this soil stratum are presented in Drawing 6, 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	0	41	40	19
BH-3	8	5	44	40	11

Cobbles and boulders were also encountered within the upper till layer and should be anticipated during construction.

The results of plasticity testing on two samples of the upper till indicate the soil would be classified as silt and clay of low plasticity (see Drawing No. 7).

The upper till is hard in consistency (as interpreted based on SPT “N” values which typically range from 40 to greater than 100). 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 Drawing 2.

Unit weight measurements on samples of the silty clay yielded values of 21.0 kN/m³ to 22.4 kN/m³ with an average value of 21.4 kN/m³.

The upper till layer was found to be 1.5 m and 3.0 m thick in Boreholes BH12-2 and BH12-3 on the west side of the existing bridge. This corresponds to elevations of 88.8 m and 90.4 m at the base of the upper

till. BH12-1 was terminated within the upper till layer. On the east side of the bridge the upper till was not encountered in BH12-4. Boreholes BH12-5 and BH12-6 were both terminated within the upper till layer.

4.1.4 Hard Silty Clay & Clayey Silt

Underlying the upper till stratum a layer of heavily over-consolidated silty clay and clayey silt is present.

The results of plasticity testing on one sample indicate the soil would be classified as low plasticity silt/clay (see Drawing No. 8).

SPT “N” values within the silty clay/clayey silt range from 34 to greater than 100 indicating a hard consistency. DCPT testing in BH-4 met refusal in this layer with blow counts in excess of 100 blows per 0.3 m penetration.

Unit weight measurements on samples of the silty clay yielded values of 17.8 kN/m³ to 21.1 kN/m³ with an average value of 20.2 kN/m³.

On the west side of the bridge this stratum was found to be 6.1 m thick in Borehole BH-3 (which completely penetrated the layer). This corresponds to an elevation of 84.3 m at the base of the hard silty clay and clayey silt. Borehole BH-2 was terminated within this layer. On the east side of the existing bridge the hard silty clay and clayey silt was found to be 5.2 m in thickness (extending to 86.4 m elevation).

4.1.5 Lower Till (Very Dense Silty Sand)

The hard silty clay/clayey silt is underlain by silty sand till, which is referred to as “lower” till (in contrast to the “upper” till discussed in Section 4.1.3 above). This till is a heterogeneous mixture of clay, silt sand and gravel, the majority of which would be described as silty sand with varying amounts of gravel and clay. Cobbles and boulders were also encountered during drilling and should be anticipated during construction.

The grain size distributions of two of the coarser samples from this soil stratum are presented in Drawing 9, and are summarized in Table 4 below.

Table 4 – Results of Grain Size Analyses for Lower Till

Borehole No.	Sample No.	Grain Size Distribution			
		% Gravel	% Sand	% Silt	% Clay
BH-3	14	25	44	19	12
BH-3	8	4	49	26	21

SPT “N” values within the lower till strata are consistently greater than 100 indicating a very dense consistency. Some portions of the lower till were recovered intact using rock coring equipment, further suggesting a very dense competent stratum.

Unit weight measurements on samples of the lower till yielded values of 22.9 kN/m³ to 23.5 kN/m³ with an average value of 23.3 kN/m³.

The lower till was encountered in Boreholes BH-3 and BH-4 (all other boreholes met refusal above the lower till layer). At these locations the lower till was found to be 4.9 m and 5.5 m thick, respectively. This corresponds to elevations at the base of the lower till of 79.4 m on the west side of the bridge and 80.9 m on the east side.

4.2 Bedrock

The rock surface was encountered at a depth of 20.1 m on the west side of the bridge (BH-3) and at 18.3 m on the east side of the bridge (BH-4). These depths correspond to elevations of 79.4 m and 80.9 m in Boreholes BH-3 and BH-4, respectively.

At these two locations the rock was cored using “N” size coring equipment. The bedrock in the area includes fresh to moderately weathered, fine to medium grained, strong to very strong, granitic rock. Rock Quality Designation (RQD) values for the rock cored range from 28% to 37 % in BH-2 (indicating poor quality rock based on discontinuity spacing) and 21% to 60 % in BH-4 (indicating very poor to good quality rock).

4.3 Groundwater

The water level in the creek at the time of the investigation was approximately 97.3 m elevation (in the local datum).

Standpipe piezometers were installed in three of the boreholes during drilling (BH-3, BH-4 and BH-5). The piezometers were installed in bedrock in the deeper boreholes and within the overburden soils in one of the shallower boreholes. The groundwater elevations at the site were found to be at 97.2 m to 97.5 m in May 2012.

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 at the time of the investigation, a corresponding increase in groundwater levels should be anticipated.

4.4 Summary

A summary of the soil and groundwater conditions encountered at the crossing location is presented in Table 5 below.

Table 5 – Simplified Stratigraphy and Groundwater Elevations

		Borehole No.					
		BH-1	BH-2	BH-3	BH-4	BH-5	BH-6
Ground Surface Elevation (Local)		99.4	99.4	99.5	99.2	98.9	98.7
Simplified Soil Profiles (Elevation, m)	Fill	99.4 – 97.2	99.4 – 97.1	99.5 – 97.3	99.2 – 96.2	98.9 – 96.7	98.7 – 96.5
	Firm to V. Stiff Silty Clay	97.2 – 92.1	97.1 – 91.8	97.3 – 91.9	96.2 – 91.6	96.7 – 91.3	96.5 – 91.4
	Hard Silty Clay & Clayey Silt Till (Upper Till)	92.1 – 91.0	91.8 – 88.8	91.9 – 90.4	91.6 – 86.4	91.3 – 90.5	91.4 – 91.0
	Hard Silty Clay and Clayey Silt	--	88.8 – 86.5	90.4 – 84.3	--	--	--
	Silty Sand Till (Lower Till)	--	--	84.3 - 79.4	86.4 – 80.9	--	--
	Refusal	91.0	86.5	79.4	80.9	90.5	91.0
	Bedrock (Cored)	--	--	79.4 – 74.9	80.9 – 77.5	--	--
Groundwater Elevation (m)		--	--	97.2	97.5	97.2	--

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.

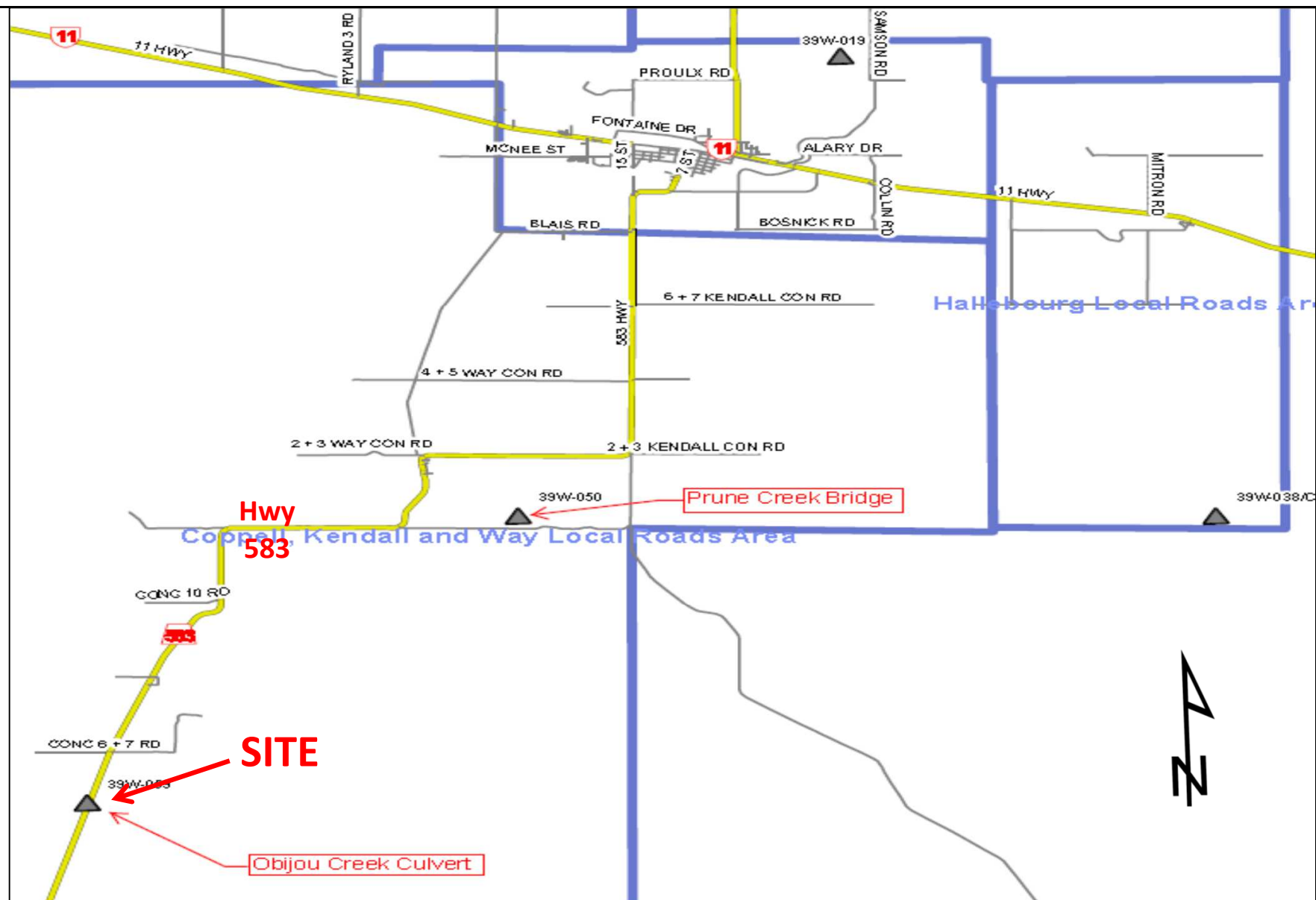



Fanyu Zhu, Ph.D., P.Eng.

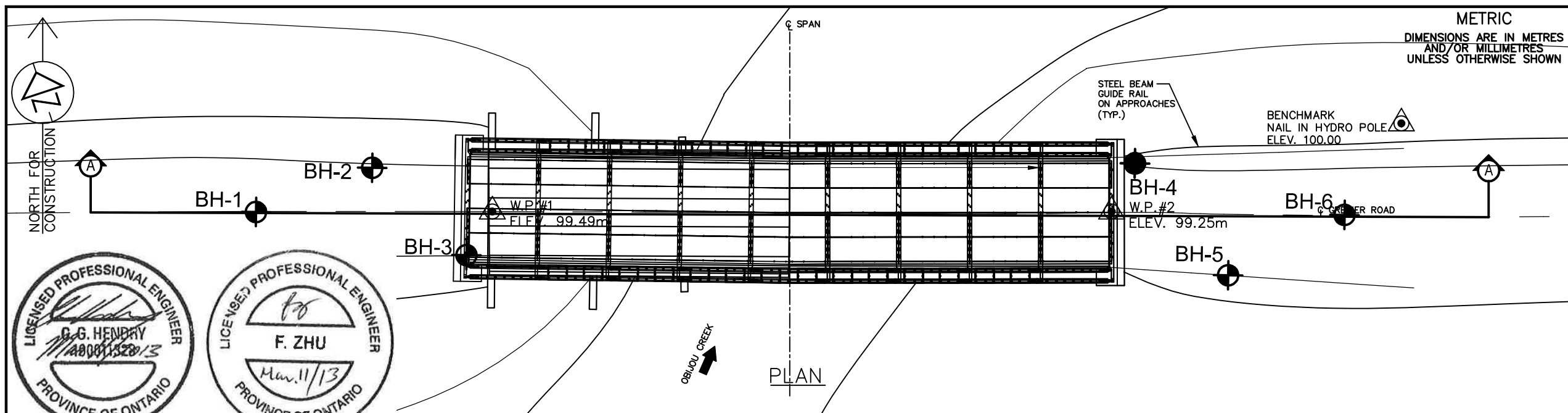


Shaheen A. Ahmad, M.Sc., P.Eng.

Drawings



Client: Ministry of Transportation Ontario			Title: SITE PLAN	
Project#:	1067-710	DWG #:	1	Project: Geotechnical Investigation Objou Creek Bridge Replacement
Drawn:	NT	Approved:	CH	
Date:	May 2012	Scale:	N. T. S.	
Size:	Letter	Rev:	0	
				 SPL Consultants Limited Geotechnical Environmental Materials Hydrogeology

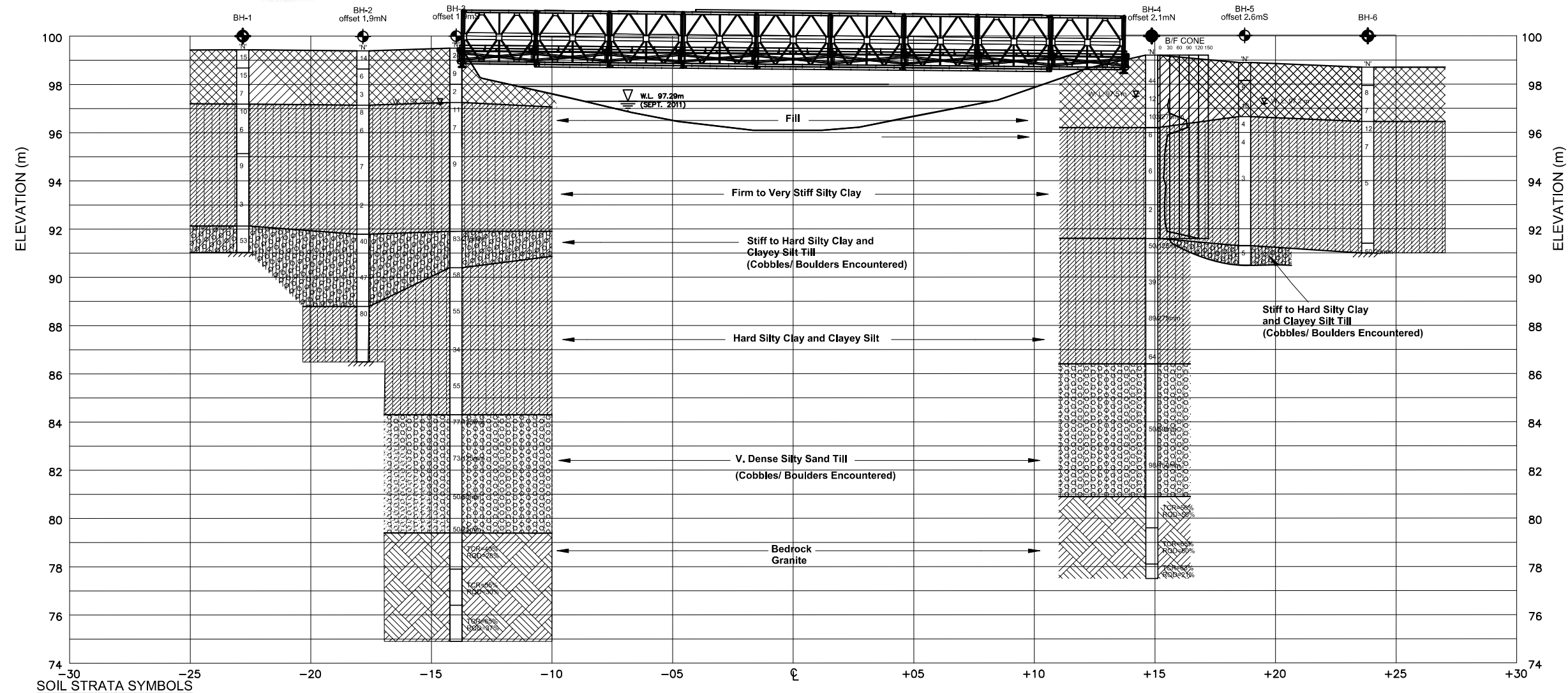
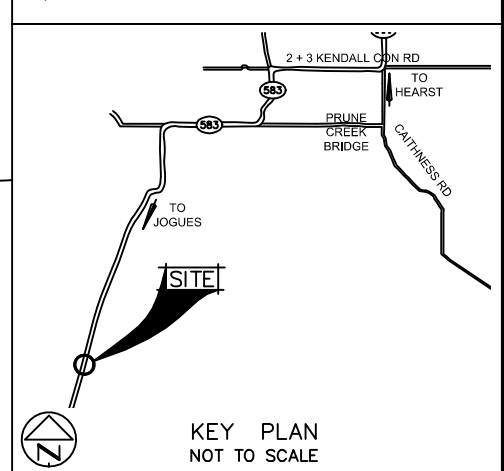


CONT No 2013-5602
WP No 5079-11-01

OBIJOU CREEK BRIDGE
BORE HOLE LOCATIONS
AND GEOSECTION

SHEET
9

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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)
- W.L. at time of investigation (March 2012)
- W.L. at time of survey (Sept. 2012)
- W.L. in Piezometer
- Piezometer
- Auger Refusal

No	ELEVATION	NORTHING	EASTING
BH-1	99.43	5488583	293674
BH-2	99.39	5488583	293680
BH-3	99.50	5488580	293683
BH-4	99.20	5488577	293712
BH-5	98.90	5488574	2913715
BH-6	98.70	5488578	293718

NOTES

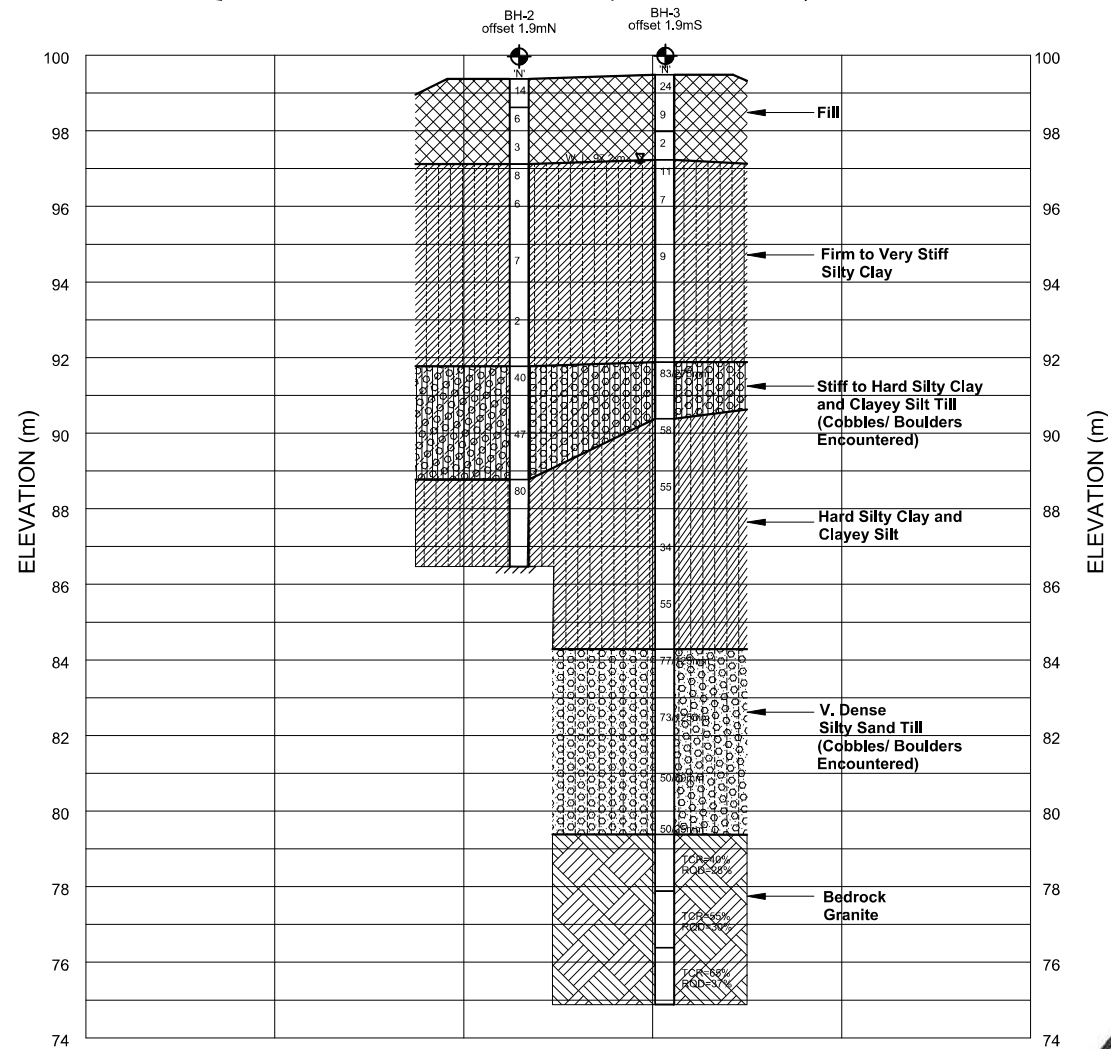
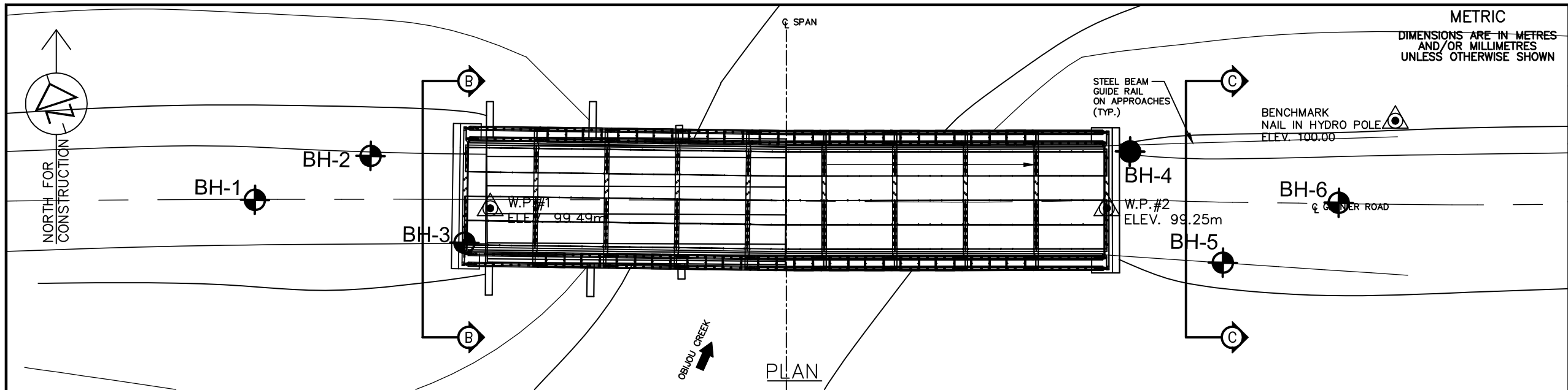
Borehole elevations are based on local datum.

REVISIONS

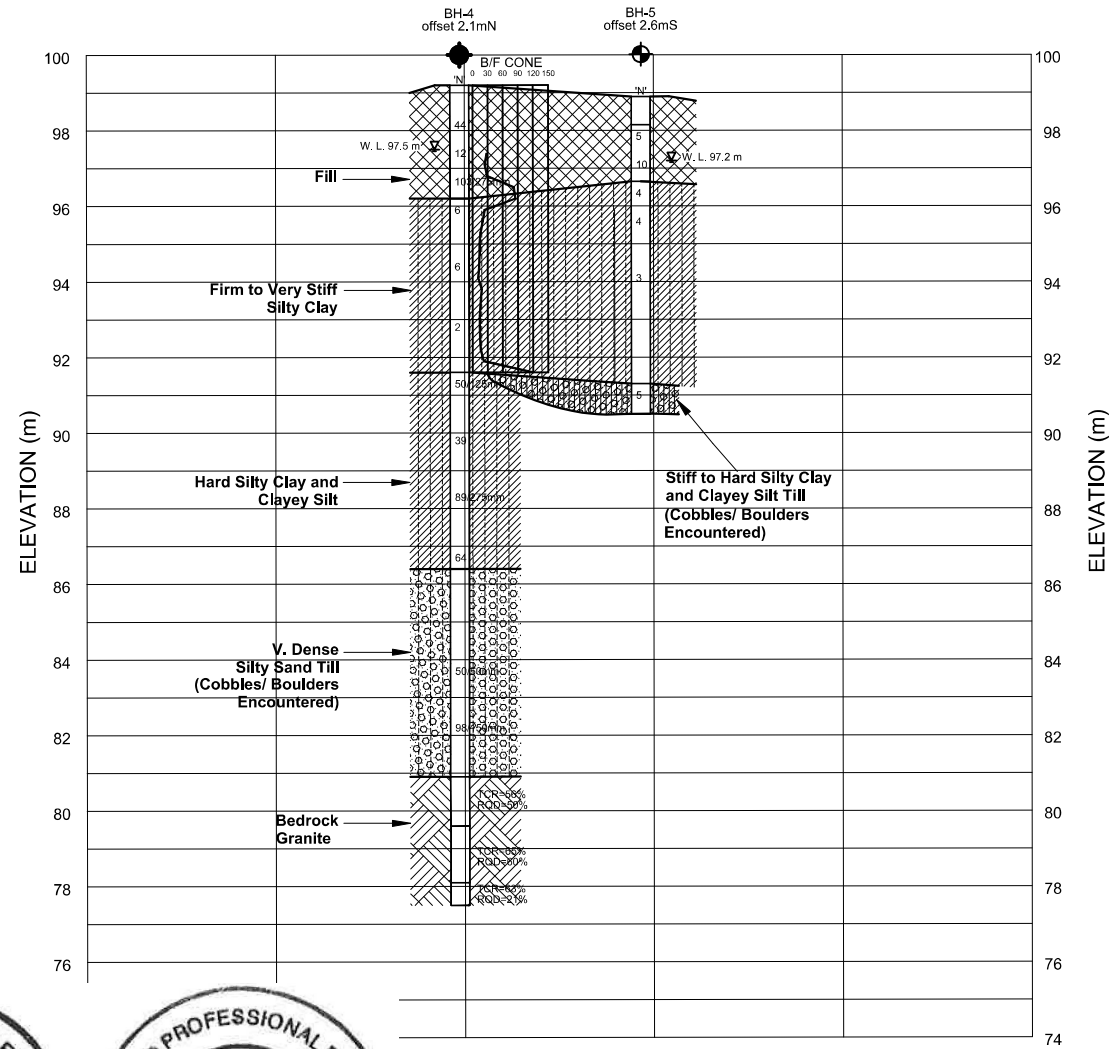
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DEC 2012	TJC	Revision 02 - Revised Draft - Issued for Exec. Review
MAY 2012	TJ	Revision 01 - Draft Reprt

GEORES No. 42G-39

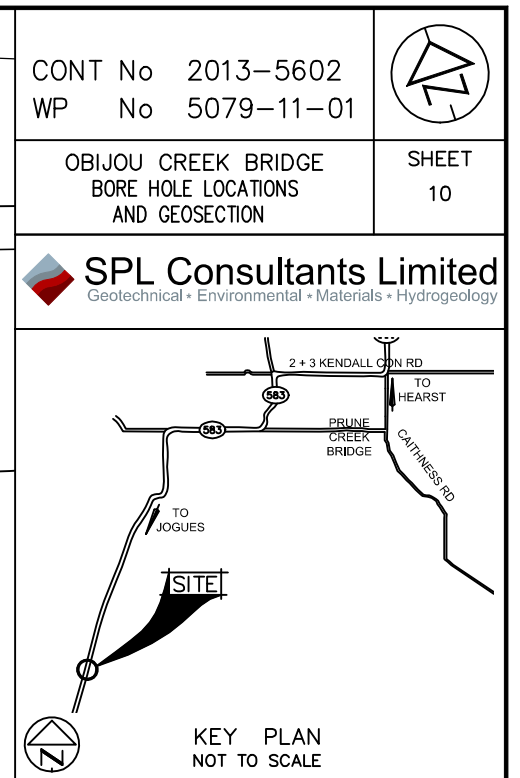
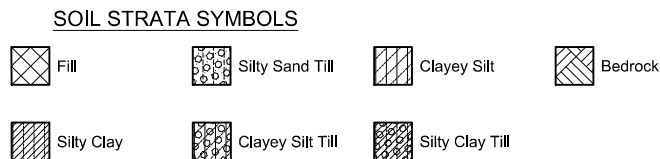
HWY No	CHECKED CH	DATE	DIST
SUBM'D	CHECKED CH	APPROVED CH	SITE 39W-59
DRAWN	CHECKED CH	APPROVED CH	DWG 2A



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 (Sept. 2012)
- WL in Piezometer
- Piezometer
- Auger Refusal

No	ELEVATION	NORTHING	EASTING
BH-1	99.43	5488583	293674
BH-2	99.39	5488583	293680
BH-3	99.50	5488580	293683
BH-4	99.20	5488577	293712
BH-5	98.90	5488574	2913715
BH-6	98.70	5488578	293718

NOTES

Borehole elevations are based on local datum.

Note: Ground surface has not been surveyed.
Ground surface shown on sections B-B and C-C
is based on BH elevations and visual observations
and is not to scale.

REVISIONS

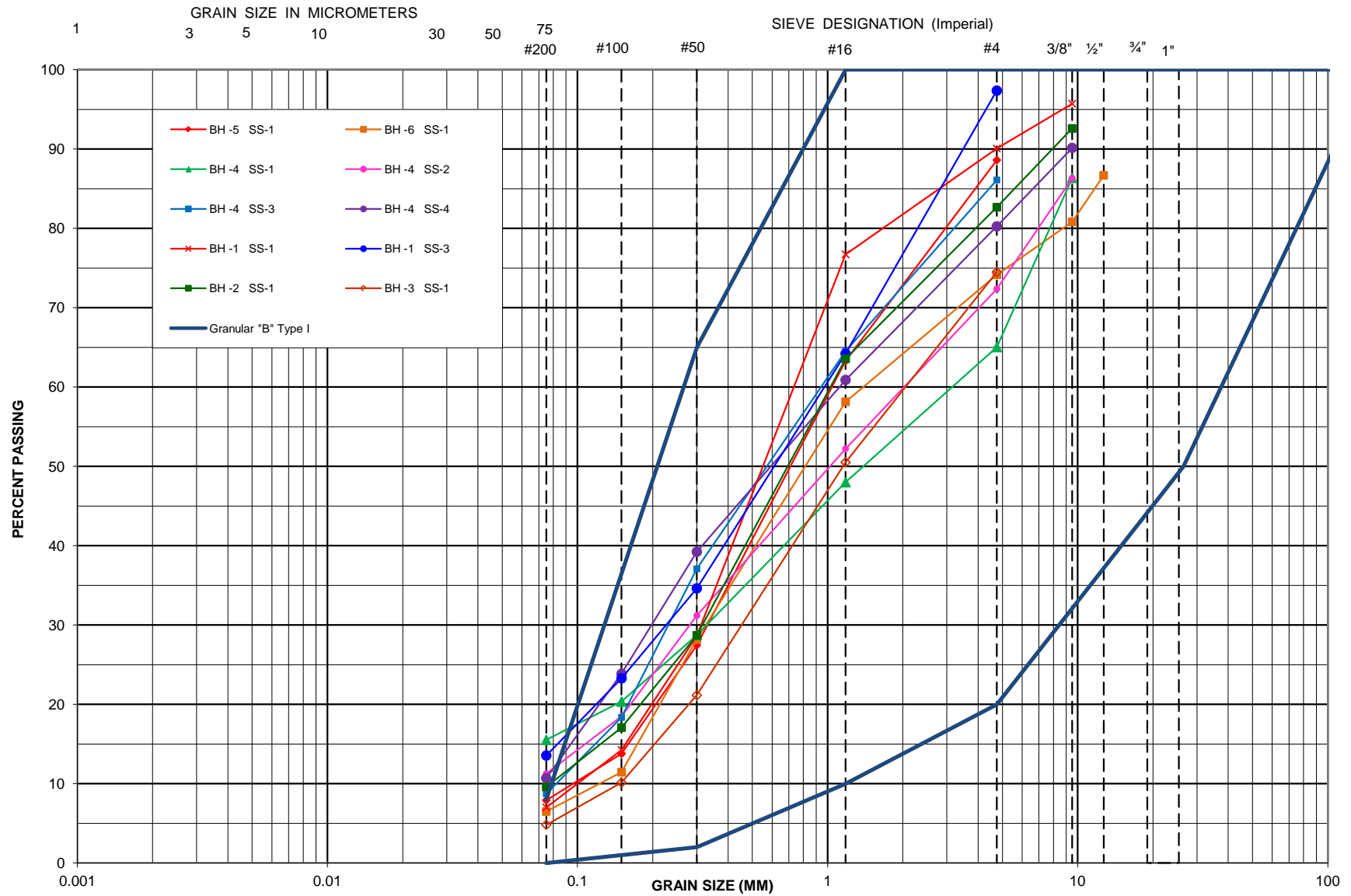
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DEC 2012	TJC	Revision 02 - Revised Draft - Issued for Exec. Review
MAY 2012	TJ	Revision 01 - Draft Reprt

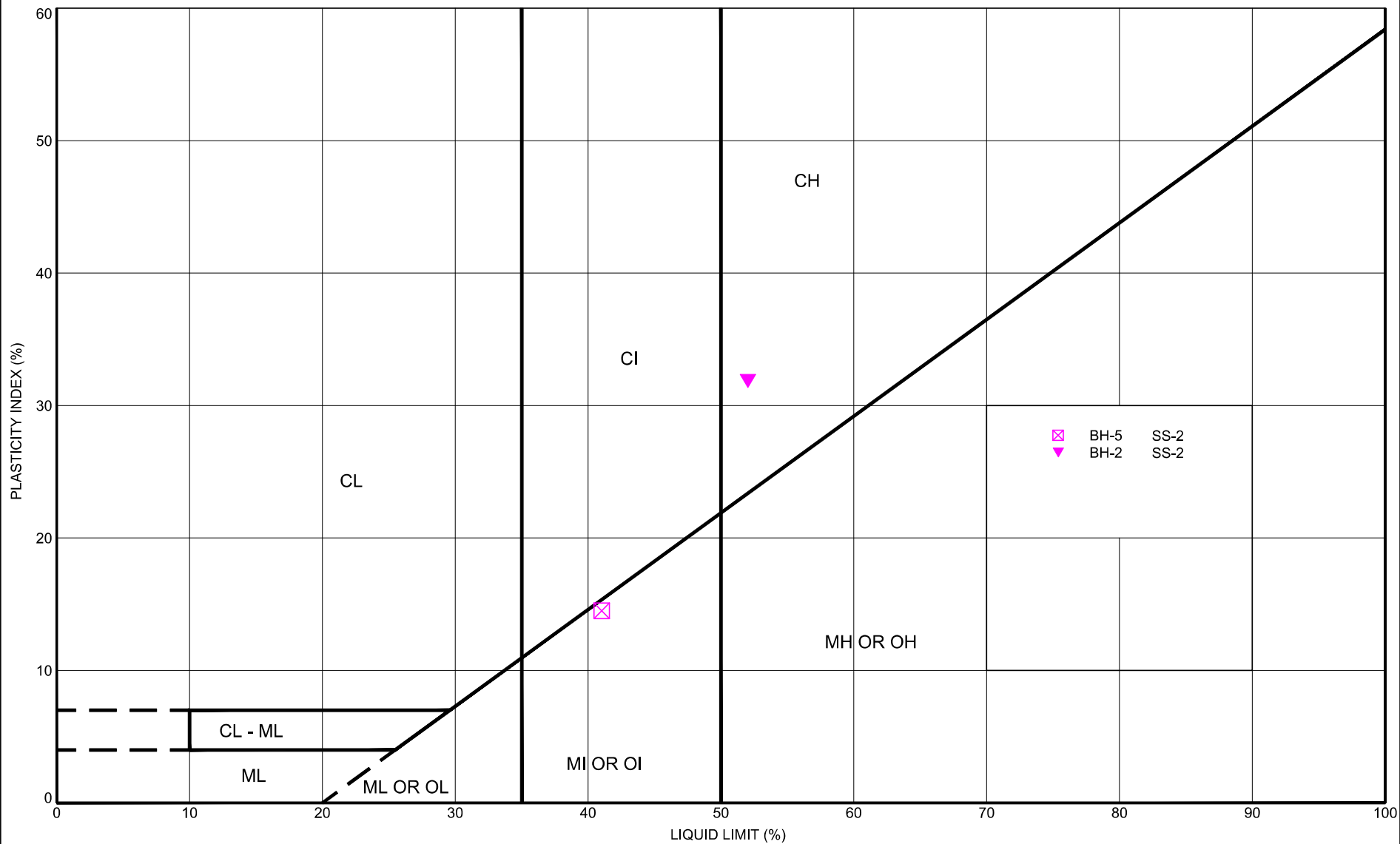
GEORES No. 42G-39


HWY No	CHECKED CH	DATE	DIST
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DRAWN	CHECKED CH	APPROVED CH	DWG 2B

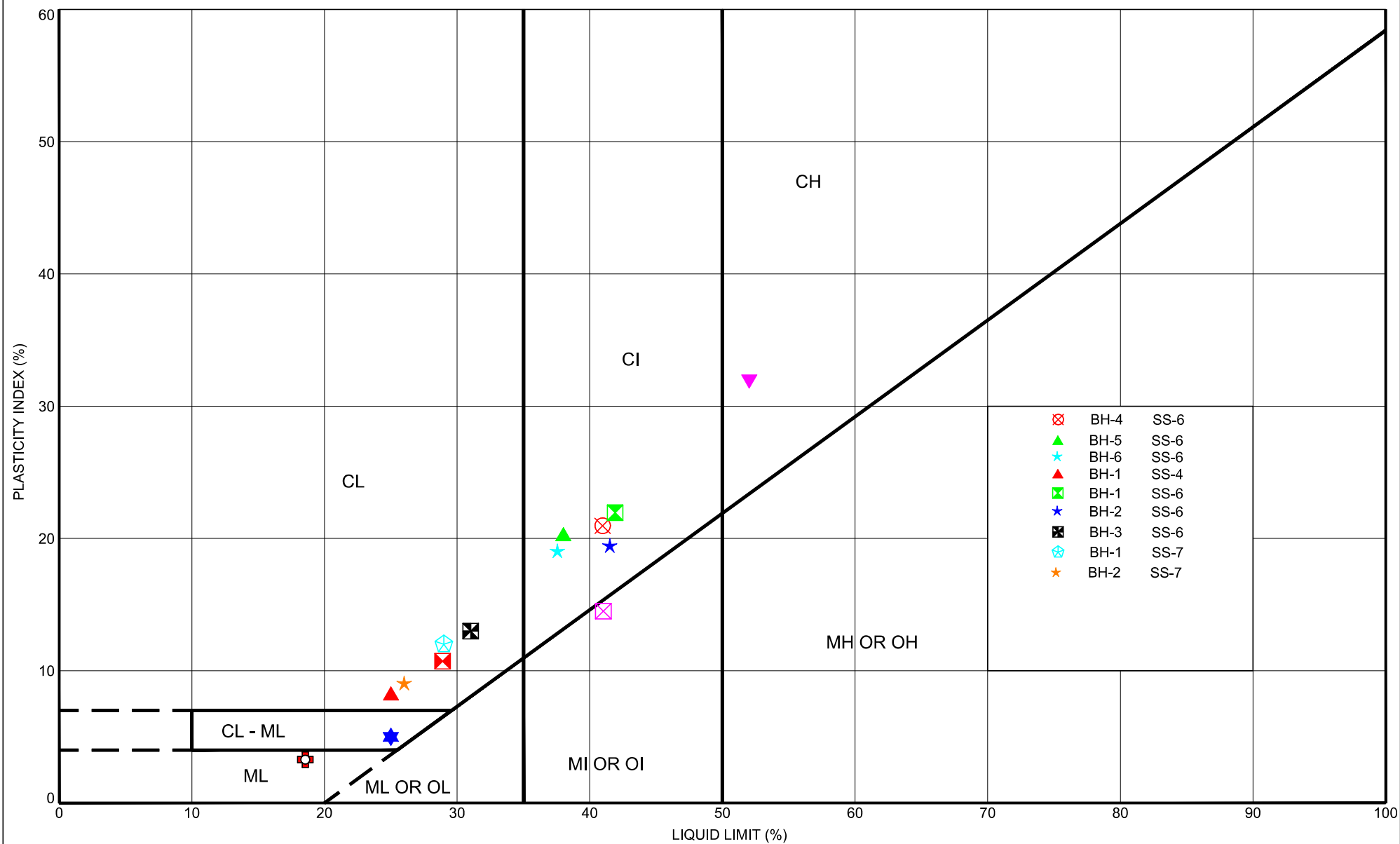
UNIFIED SOIL CLASSIFICATION SYSTEM


CLAY AND SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse





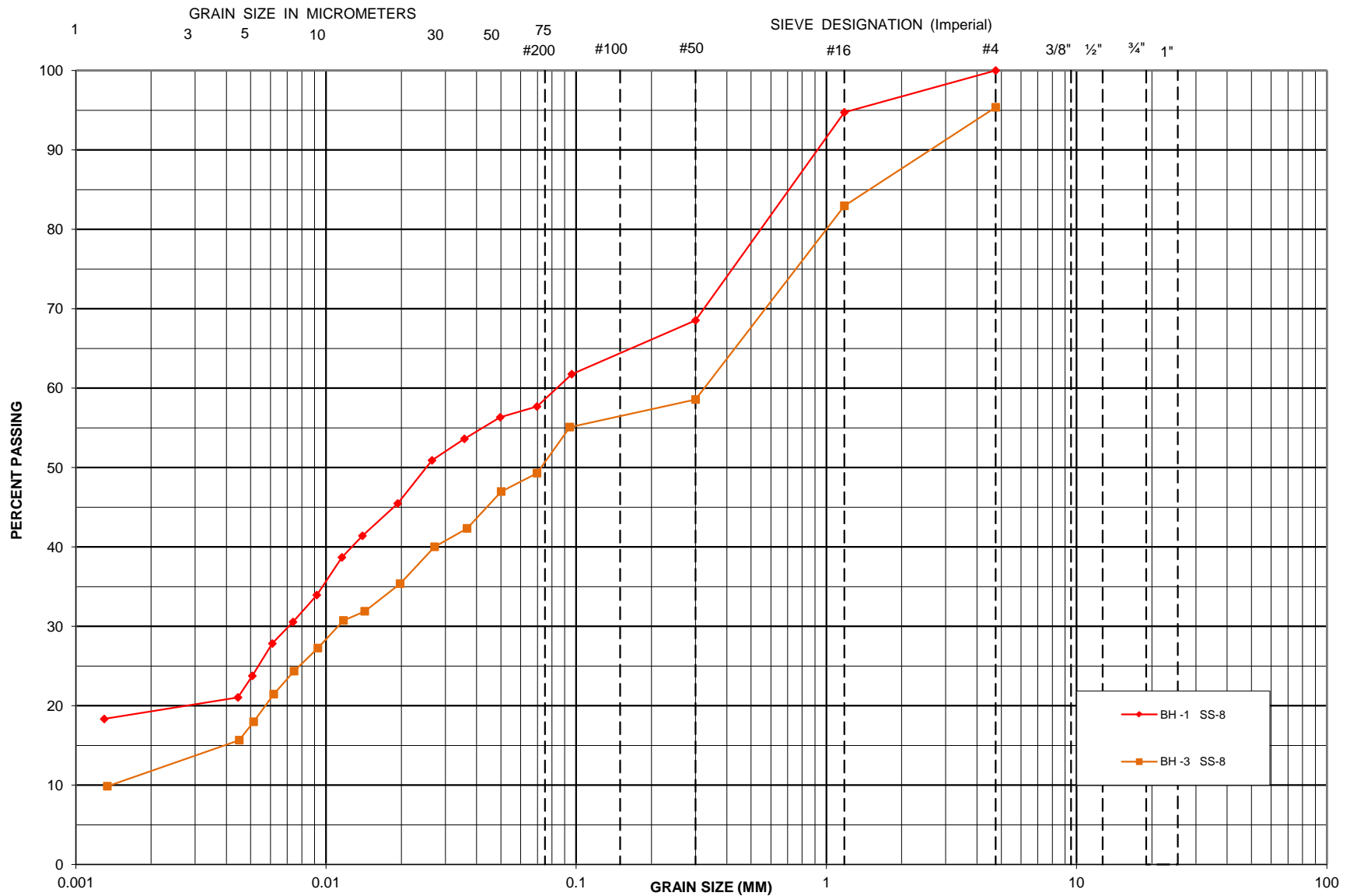
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Date: May 2012	Drawn: NT	Scale: N/A	Project No.: 1067-710	 SPL Consultants Limited Geotechnical • Environmental • Materials • Hydrogeology	
Original Size: LETTER	Approved: CH	Rev: N/A	Drawing No.: 4		

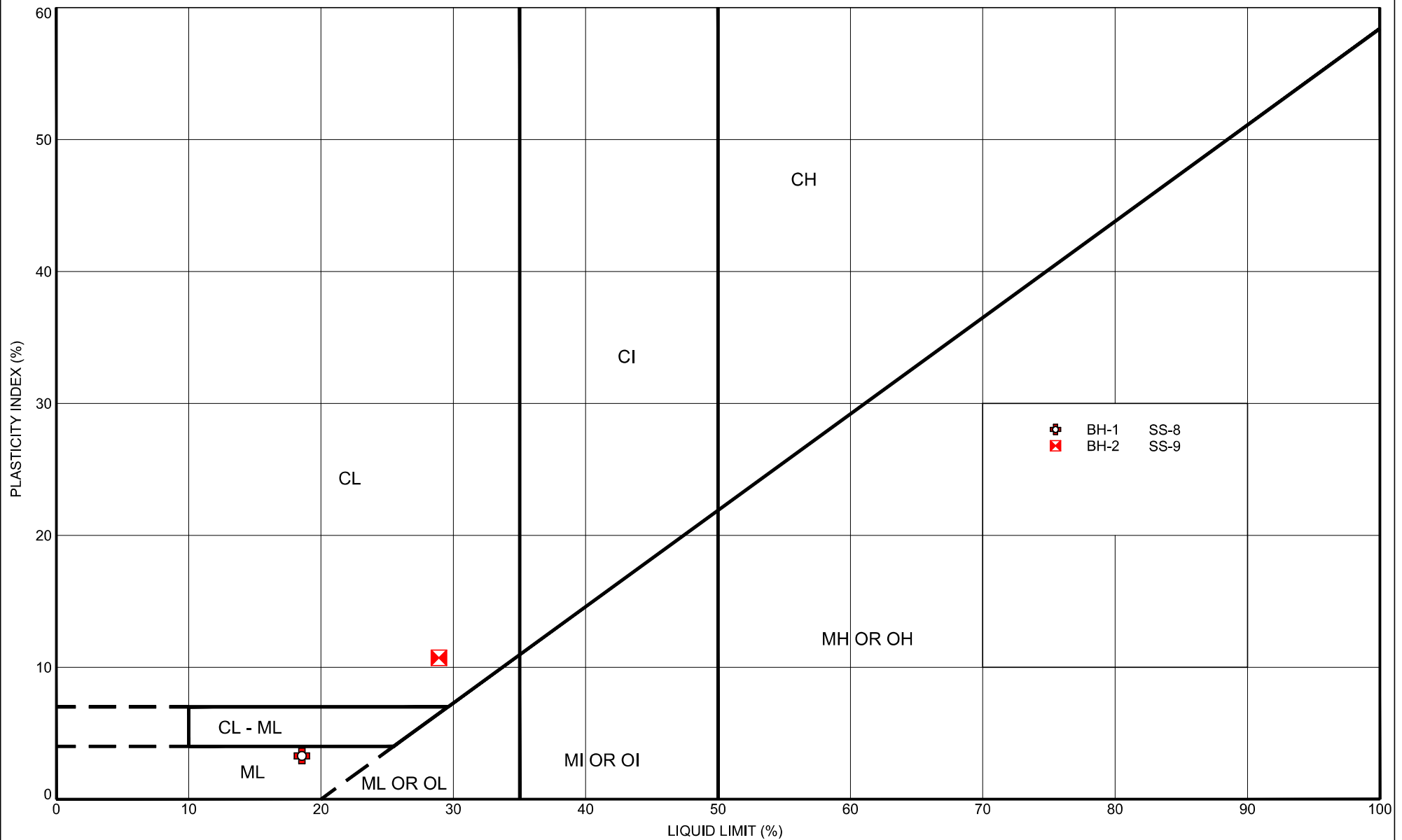



Client: City of Ottawa		Title: PLASTICITY CHART FIRM TO STIFF SILTY CLAY		Project: Foundation Investigation Obijou Creek Bridge Replacement	
Date: May 2012	Drawn: NT	Scale: N/A	Project No.: 1067-710	 SPL Consultants Limited Geotechnical • Environmental • Materials • Hydrogeology	
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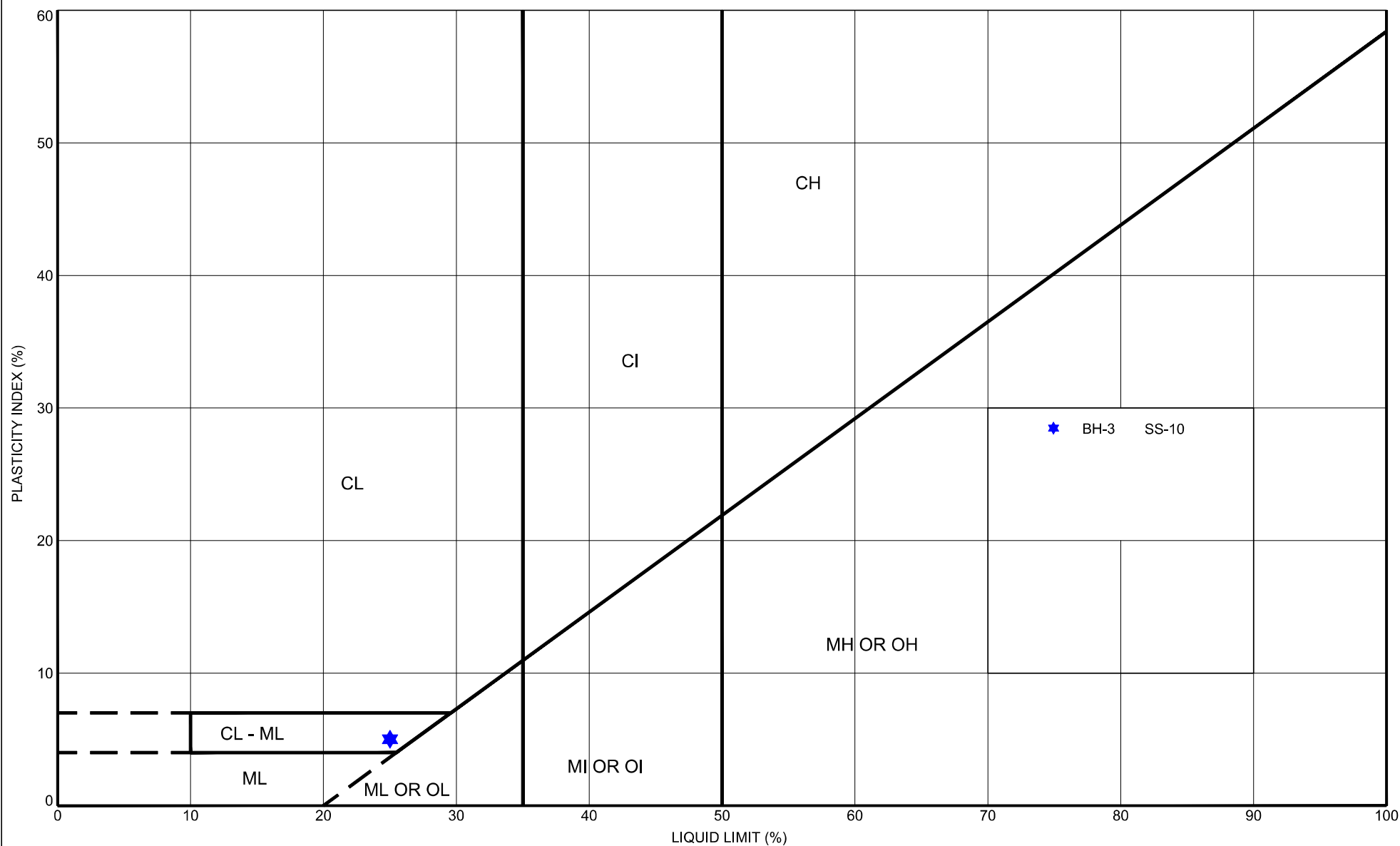
UNIFIED SOIL CLASSIFICATION SYSTEM


CLAY AND SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse





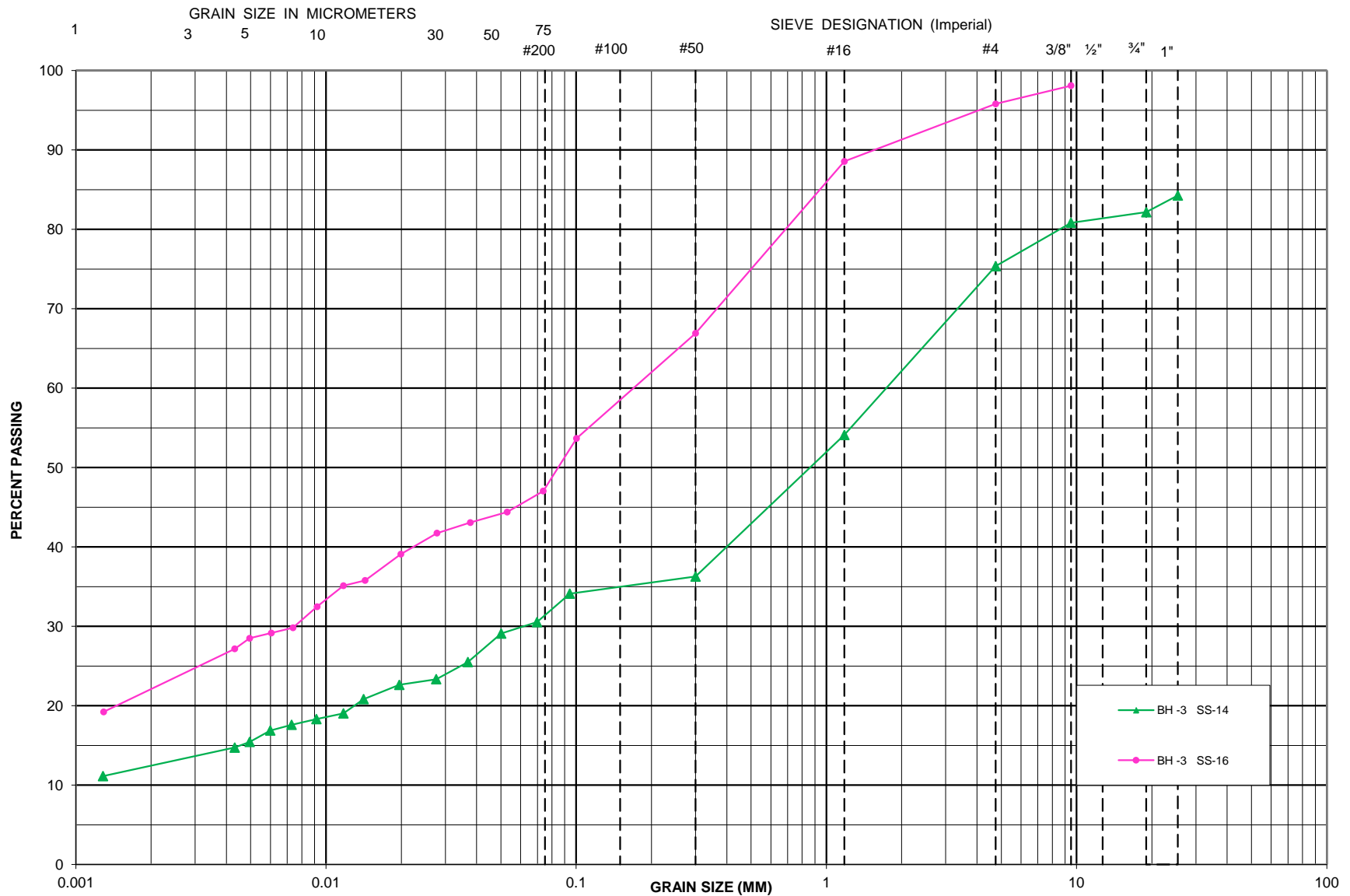
Client: City of Ottawa		Title: PLASTICITY CHART - UPPER TILL		Project: Foundation Investigation Obijou Creek Bridge Replacement	
Date: May 2012	Drawn: NT	Scale: N/A	Project No.: 1067-710	 SPL Consultants Limited Geotechnical • Environmental • Materials • Hydrogeology	
Original Size: LETTER	Approved: CH	Rev: N/A	Drawing No.: 7		



Client: City of Ottawa		Title: PLASTICITY CHART HARD SILTY CLAY AND CLAYEY SILT		Project: Foundation Investigation Obijou Creek Bridge Replacement	
Date: May 2012	Drawn: NT	Scale: N/A	Project No.: 1067-710	 SPL Consultants Limited Geotechnical • Environmental • Materials • Hydrogeology	
Original Size: LETTER	Approved: CH	Rev: N/A	Drawing No.: 8		

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse








Appendix A

Borehole Logs (Record of Borehole Sheets)

METRIC

W.P.	5044-11-00	LOCATION	Obijou Creek Bridge (see borehole location plan), E 293674, N 5488583	ORIGINATED BY	NE
DIST	New Liskeard HWY Grenier Road	BOREHOLE TYPE	Hollow Stem Auger	COMPILED BY	TJ
DATUM	Local	DATE	May/02/2012	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			SHEAR STRENGTH kPa						WATER CONTENT (%)		GR	SA	SI	CL													
								25	50					75	100	125	20	40	60													
99.4	0.0		1	SS	15		99											10	83	(7)												
98.7			0.8	2	SS			15	98																							
	3	SS								7	97																					
97.2			2.3		4			SS				10	96																			
	5	SS			6			95																								
															94																	
	4.3		6	SS	9					93																						
			VANE									92																				
																		91														
	7	SS	3	90																												
					VANE												89															
																						88										
92.1	7.3															87																
			8		SS									53					86													
																					85											
91.0	8.4						84																									
			END OF BOREHOLE: auger refusal								83																					
																							82									
						81																										
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									-33																							
																	-34															
																					-35											

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

RECORD OF BOREHOLE No BH-2

1 OF 1

METRIC

W.P. 5044-11-00 LOCATION Obijou Creek Bridge (see borehole location plan), E 293680, N 5488583 ORIGINATED BY NE
DIST New Liskeard HWY Grenier Road BOREHOLE TYPE Hollow Stem Auger COMPILED BY TJ
DATUM Local DATE May/02/2012 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					
99.4 0.0	GRAVELLY SAND brown, moist compact (Fill)		1	SS	14		99										17 73 (10)
98.6 0.8	SILTY CLAY some sand, trace gravel, trace organics brown, moist stiff (Fill)		2	SS	6		98									19.6	
			3	SS	3												
97.1 2.3	SILTY CLAY grey, moist stiff to very stiff		4	SS	8		97										
			5	SS	6		96										
			6	SS	7		95										
			VANE				94										Su>100 kPa
			7	SS	2		93										
			VANE				92										Su>100 kPa
91.8 7.6	SILTY CLAY some sand, trace gravel brown, moist hard (TILL) - cobbles/boulders noted during augering		8	SS	40		91									21.0	
			9	SS	47		90									20.8	
88.8 10.6	CLAYEY SILT some sand, trace gravel brown, moist hard		10	SS	80		89									20.9	
							88										
							87										
86.5 12.9	END OF BOREHOLE: auger refusal																

ON-MTO-LARGE SCALE 1067-710-OBJOU-MAR27-2013.GPJ ON_MOT.GDT 27/3/13

RECORD OF BOREHOLE No BH-3

1 OF 2

METRIC

W.P. 5044-11-00 LOCATION Obijou Creek Bridge (see borehole location plan), E 293683, N 5488580 ORIGINATED BY NE
DIST New Liskeard HWY Grenier Road BOREHOLE TYPE Hollow Stem Auger and N Size Core COMPILED BY TJ
DATUM Local DATE May/01/2012 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					
99.5 0.0	GRAVELLY SAND brown, moist compact (Fill)		1	SS	24		99										26 69 (5)
98.7 0.8	SILTY CLAY some sand, trace gravel, trace organics brown, moist firm (Fill)		2	SS	9		98										
97.3 2.3	SILTY CLAY some organics, pieces of wood dark brown, moist stiff		3	SS	2		97										
			4	SS	11		96										
			5	SS	7		95										
			6	SS	9		94										
			VANE				93										
			VANE				92										
			7	TW			91										
			VANE				90										
91.9 7.6	CLAYEY SILT some sand, trace gravel grey, moist hard (TILL)		8	SS	83/ 275mm		89										
90.4 9.1	SILTY CLAY grey, moist hard		9	SS	58		88										
			10	SS	55		87										
			11	SS	34		86										
			12	SS	55		85										

Continued Next Page

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

ON-MTO-LARGE SCALE 1067-710-OBJOU-MAR27-2013.GPJ ON_MOT.GDT 27/3/13

RECORD OF BOREHOLE No BH-3

2 OF 2

METRIC

W.P. 5044-11-00 LOCATION Obijou Creek Bridge (see borehole location plan), E 293683, N 5488580 ORIGINATED BY NE
DIST New Liskeard HWY Grenier Road BOREHOLE TYPE Hollow Stem Auger and N Size Core COMPILED BY TJ
DATUM Local DATE May/01/2012 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					
84.3 15.2	SILTY SAND some gravel, trace to some clay brown, wet very dense (TILL) - cobbles/ boulders noted during drilling		13	SS	77/ 125mm		84									23.5	
			14	SS	73/ 125mm		83									23.2	25 44 19 12
			15	SS	50/ 50mm		81									22.9	
79.4 20.1	GRANITE strong to very strong, slightly to moderately weathered grey-brown, fine grained TCR=48% SCR=38% RQD=28%		16	SS	50/ 25mm		79									23.4	4 49 26 21
77.9 21.6	GRANITE strong to very strong, slightly to moderately weathered grey-brown, fine grained TCR=53% SCR=45% RQD=30%						78										
76.4 23.1	GRANITE strong to very strong, slightly to moderately weathered grey-brown, fine grained TCR=65% SCR=58% RQD=37%						77										
74.9 24.6	END OF BOREHOLE Notes: 1) A 19mm standpipe piezometer installed to 24.6m depth upon completion. 2) Water level found at 2.3m below surface (El. 97.2m) on May 03, 2012.						76										
							75										

RECORD OF BOREHOLE No BH-4

1 OF 2

METRIC

W.P. 5044-11-00 LOCATION Obijou Creek Bridge (see borehole location plan), E 293712, N 5488577 ORIGINATED BY NE
DIST New Liskeard HWY Grenier Road BOREHOLE TYPE Hollow Stem Auger and N Size Core COMPILED BY TJ
DATUM Local DATE Mar/23/2012 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					
99.2 0.0	GRAVELLY SAND brown, moist compact (Fill)		1	AS			99										35 49 (16)
			2	SS	44		98										28 61 (11)
			3	SS	12		97										14 77 (9)
			4	SS	103/ 275mm		96										20 69 (11)
96.2 3.0	SILTY CLAY some organics dark brown, moist firm		5	SS	6		95										
	--- less organics grey below 4.6m		6	SS	6		94									20.2	>100 kPa
				VANE			93										
			7	SS	2		92										>100 kPa
				VANE			91										
				VANE			90										
91.6 7.6	SILTY CLAY grey, moist hard		8	SS	50/ 125mm		89										
			9	SS	39		88									20.9	
			10	SS	89/ 275mm		87										
			11	SS	64		86										
86.4 12.8	SILTY SAND trace gravel brown, moist very dense (TILL) - cobbles/ boulders noted during drilling		12	SS			85										

Continued Next Page

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

ON-MTO-LARGE SCALE 1067-710-OBJOU-MAR27-2013.GPJ ON_MOT.GDT 27/3/13

RECORD OF BOREHOLE No BH-4

2 OF 2

METRIC

W.P. 5044-11-00 LOCATION Obijou Creek Bridge (see borehole location plan), E 293712, N 5488577 ORIGINATED BY NE
DIST New Liskeard HWY Grenier Road BOREHOLE TYPE Hollow Stem Auger and N Size Core COMPILED BY TJ
DATUM Local DATE Mar/23/2012 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 (%) GR SA SI CL						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			25	50	75	100	125						SHEAR STRENGTH kPa			WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE															
	SILTY SAND trace gravel brown, moist very dense (TILL) - cobbles/ boulders noted during drilling (continued)		13	SS	50/ 50mm		84																
							83																
			14	SS	98/ 150mm		82									23.3							
80.9							81																
18.3	GRANITE strong to very strong, slightly to moderately weathered grey-brown, fine grained						80																
79.6	TCR=56% SCR=50% RQD=50%						79																
19.6	GRANITE strong to very strong, slightly to moderately weathered grey-brown, fine grained						78																
78.1	TCR=65% SCR=65% RQD=60%																						
21.1	GRANITE strong to very strong, slightly to moderately weathered grey-brown, fine grained																						
77.5	TCR=63% SCR=38% RQD=21%																						
21.7	END OF BOREHOLE Notes: 1) A 19mm standpipe piezometer installed to 19.8m depth upon completion. 2) Water level found at 1.7m below surface (El. 97.5m) on Mar 26, 2012.																						

ON-MTO-LARGE SCALE 1067-710-OBUJOU-MAR27-2013.GPJ ON_MOT.GDT 27/3/13

RECORD OF BOREHOLE No BH-5

1 OF 1

METRIC

W.P. 5044-11-00 LOCATION Obijou Creek Bridge, (see borehole location plan), E 2913715, N 5488574 ORIGINATED BY NE
DIST New Liskeard HWY Grenier Road BOREHOLE TYPE Hollow Stem Auger COMPILED BY TJ
DATUM Local DATE Mar/26/2012 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 brown, moist compact (Fill)		1	AS													11 81 (8)
98.2 0.8	CLAYEY SILT some sand, trace organics brown, moist firm to stiff (Fill)		2	SS	5		98									20.0	
			3	SS	10		97.2 m									19.2	
96.7 2.3	SILTY CLAY some organics dark brown, moist firm --- less organics grey below 3.0m		4	SS	4		96										
			5	SS	4		95										
			6	SS	3		94									19.3	
				VANE													>100 kPa
				VANE													>100 kPa
			7	TW			93										
							92										
91.3 7.6	CLAYEY SILT trace sand, trace gravel brown, moist stiff (TILL)		8	SS	5		91										
90.5 8.4	END OF BOREHOLE: auger refusal																
Notes: 1) A 19mm standpipe piezometer installed to 8.4m depth upon completion. 2) Water level found at 1.7m below surface (El. 97.0m) on May 03, 2012.																	

ON-MTO-LARGE SCALE 1067-710-OBJOU-MAR27-2013.GPJ ON_MOT.GDT 27/3/13

RECORD OF BOREHOLE No BH-6

1 OF 1

METRIC

W.P. 5044-11-00 LOCATION Obijou Creek Bridge (see borehole location plan), E 293718, N 5488578 ORIGINATED BY NE
DIST New Liskeard HWY Grenier Road BOREHOLE TYPE Hollow Stem Auger COMPILED BY TJ
DATUM Local DATE Mar/25/2012 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					
98.7 0.0	GRAVELLY SAND brown, moist compact (Fill)		1	AS			98										26 68 (6)
98.0 0.8	CLAYEY SILT some sand, trace gravel brown, moist stiff (Fill)		2A	SS	8												12 13 14 15
			2B														1 23 3 4
			3	SS	7		97										
96.5 2.3	SILTY CLAY some organics grey, moist stiff		4	SS	12		96									143	20.3
			5	SS	7												
	--- less organics brown below 3.5m						95										
			6	SS	5		94										
				VANE													
				VANE			93										
			7	TW			92										
91.4 7.3	SILTY CLAY trace sand, trace gravel brown, moist hard (TILL)		8	SS	50/ 25mm		91										
91.0 7.7	END OF BOREHOLE: auger refusal																

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

Appendix B

Consolidation Test Results

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

Project Number	12-1183-0040	Sample Number	TW7
Borehole Number	12-5	Sample Depth, m	6.1

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	2		
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	21.91
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m ³	18.81
Area, cm ²	31.58	Specific Gravity, measured	2.77
Volume, cm ³	80.09	Solids Height, cm	1.756
Water Content, %	16.49	Volume of Solids, cm ³	55.45
Wet Mass, g	178.92	Volume of Voids, cm ³	24.64
Dry Mass, g	153.59	Degree of Saturation, %	102.8

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	2.536	0.444	2.536				
6.27	2.472	0.408	2.504	9707	1.37E-04	4.02E-03	5.40E-08
11.04	2.464	0.403	2.468	3425	3.77E-04	6.94E-04	2.57E-08
20.86	2.449	0.395	2.456	2365	5.41E-04	6.06E-04	3.21E-08
40.15	2.433	0.386	2.441	1215	1.04E-03	3.23E-04	3.29E-08
79.05	2.414	0.375	2.423	623	2.00E-03	1.95E-04	3.81E-08
158.14	2.390	0.361	2.402	577	2.12E-03	1.17E-04	2.43E-08
311.31	2.368	0.348	2.379	290	4.14E-03	5.79E-05	2.35E-08
621.56	2.340	0.333	2.354	228	5.15E-03	3.44E-05	1.74E-08
1244.80	2.313	0.317	2.326	194	5.91E-03	1.77E-05	1.02E-08
2489.61	2.274	0.295	2.293	116	9.61E-03	1.21E-05	1.14E-08
1244.80	2.281	0.299	2.277				
311.31	2.298	0.309	2.289				
79.05	2.312	0.317	2.305				
20.86	2.325	0.324	2.319				
6.27	2.336	0.330	2.331				

Note:

k calculated using cv based on t₉₀ values.

Specimen taken 13cm from bottom of the tube.

Loading stages assigned by the client.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	2.34	Unit Weight, kN/m ³	23.43
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m ³	20.42
Area, cm ²	31.58	Specific Gravity, measured	2.77
Volume, cm ³	73.77	Solids Height, cm	1.756
Water Content, %	14.76	Volume of Solids, cm ³	55.45
Wet Mass, g	176.26	Volume of Voids, cm ³	18.33
Dry Mass, g	153.59		

Prepared By: LFG

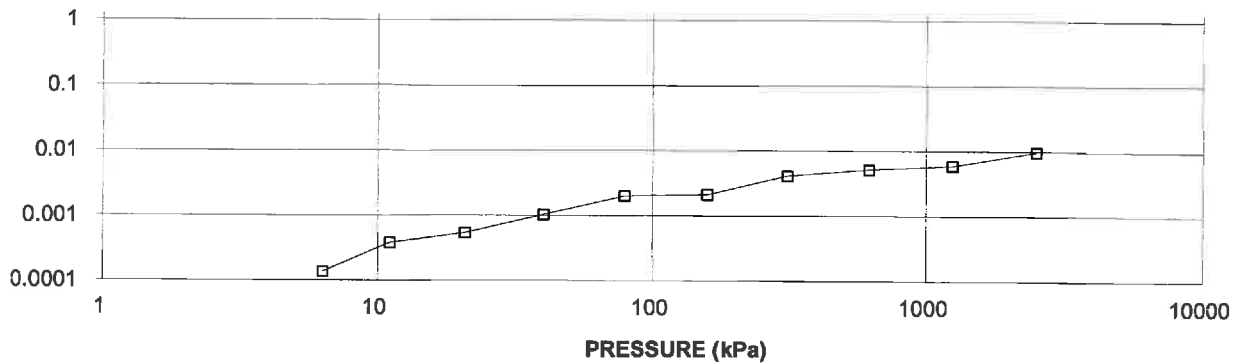
Golder AssociatesChecked By: 

CONSOLIDATION TEST SUMMARY

FIGURE

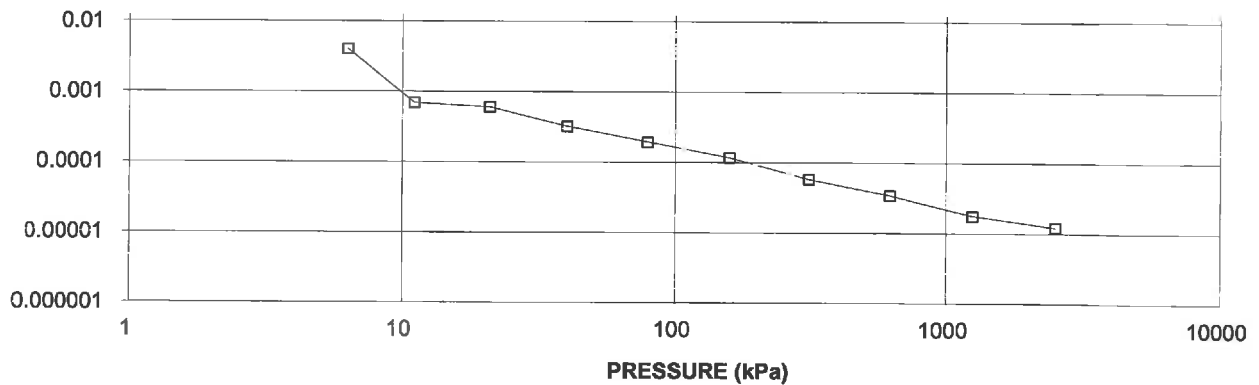
COEFFICIENT OF CONSOLIDATION,
cm²/s

CONSOLIDATION TEST
C_v cm²/s VS PRESSURE (kPa)
BH 12-5 SA TW7



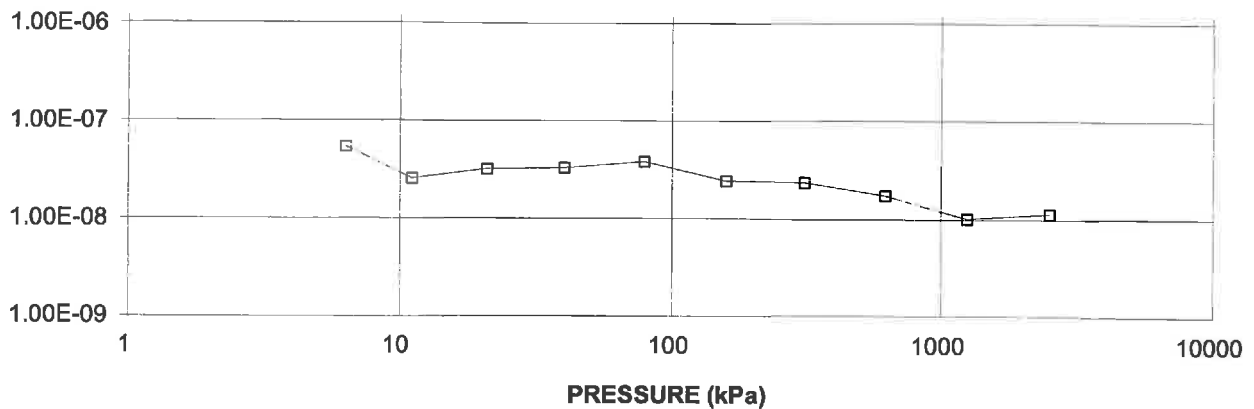
VOLUME COMPRESSIBILITY, m²/kN

CONSOLIDATION TEST
M_v m²/kN vs PRESSURE (kPa)
BH 12-5 SA TW7



HYDRAULIC CONDUCTIVITY, cm/s

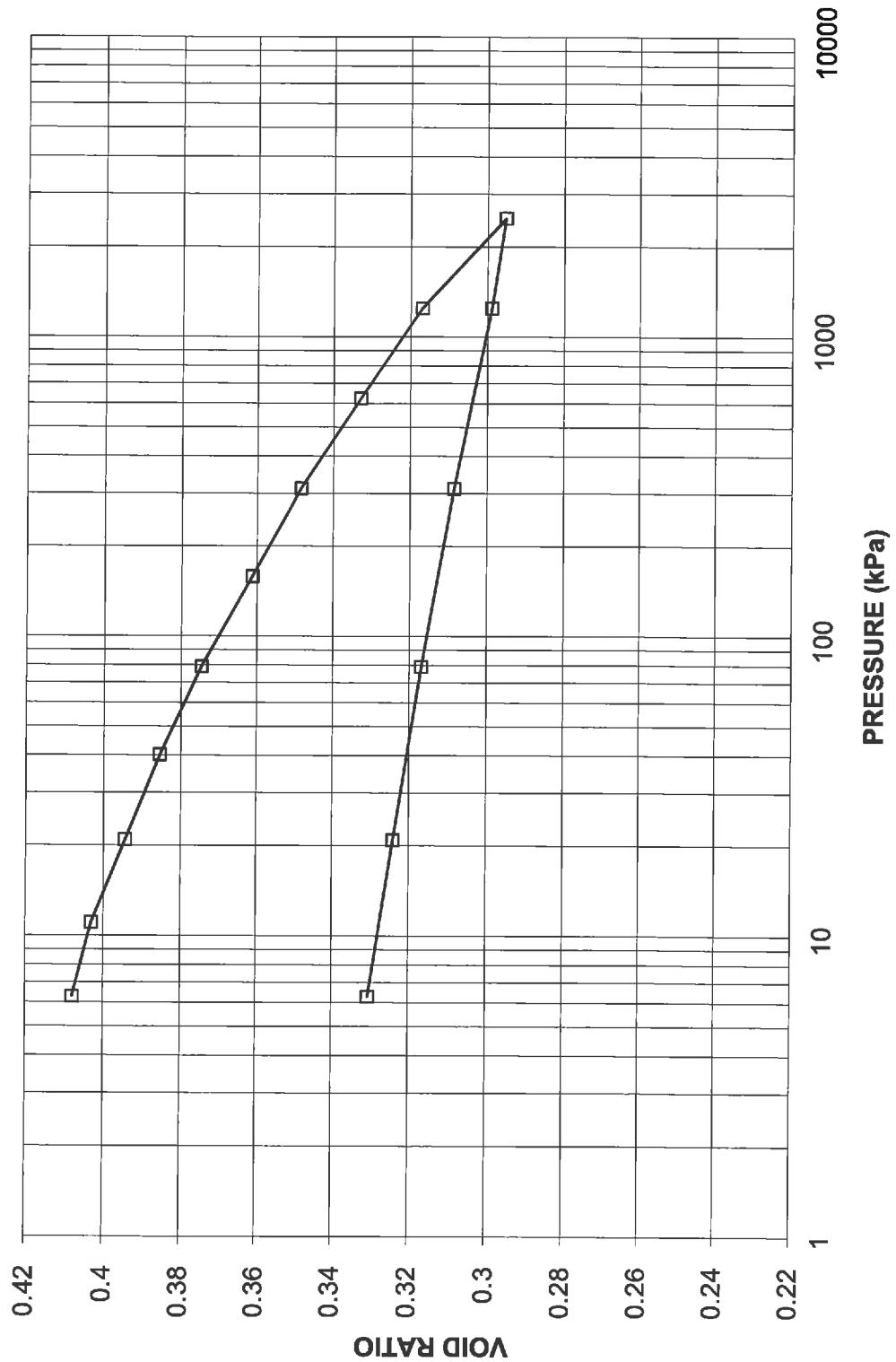
CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BH 12-5 SA TW7



**CONSOLIDATION TEST
VOID RATIO VS LOG PRESSURE**

FIGURE

**CONSOLIDATION TEST
VOID RATIO vs PRESSURE
BH 12-5 SA TW7**



SPECIFIC GRAVITY TEST RESULTS

ASTM D 854-06 TEST METHOD A

PROJECT NUMBER	12-1183-0040	
PROJECT NAME	SPL Consultants / Testing / 1067-710	
DATE	May, 2012	
Borehole	Sample	Specific
No.	No.	Gravity
12-2	TW8	2.77
12-5	TW7	2.77

Note: Test carried out on soil particles <2.00mm using distilled water.

Checked By: 

Golder Associates

Appendix C

Chemical Test Results

Client: SPL Consultants Ltd.
146 Colonnade Rd., Unit 17
Ottawa, ON
K2E 7Y1
Attention: Ms. Katie Linton
PO#: Visa
Invoice to: SPL Consultants Ltd.

Report Number: 1219287
Date Submitted: 2012-09-06
Date Reported: 2012-09-11
Project: 1067-710 Obijou
COC #: 156566

Group	Analyte	MRL	Units	Guideline	Lab I.D.	Sample Matrix	Sample Type	Sampling Date	Sample I.D.
					983162	983163	983164	983165	
					Soil	Soil	Soil	Soil	
					2012-05-01	2012-05-01	2012-05-01	2012-05-01	
					BH3/SS2	BH3/SS5	BH4/SS3	BH4/SS6	
Agri. - Soil	Electrical Conductivity	0.05	mS/cm		0.17	0.20	0.12	7.80	
	pH	2.0			7.5	6.5	7.9	7.8	
General Chemistry	Cl	0.002	%		0.004	0.002	<0.002	<0.002	
	Resistivity	1	ohm-cm		5880	5000	8330	3120	
	SO4	0.01	%		0.21	<0.01	<0.01	<0.01	

Guideline = * = **Guideline Exceedence**

** = Analysis completed at Mississauga, Ontario.

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.

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QC Summary

Analyte	Blank	QC % Rec	QC Limits
Run No 237957 Analysis Date 2012-09-07 Method C CSA A23.2-4B			
Cl	<0.002 %	104	90-110
Run No 238110 Analysis Date 2012-09-11 Method Ag Soil			
Electrical Conductivity	<0.05 mS/cm	100	80-120
pH		100	90-110
Resistivity			
SO4	<0.01 %	104	70-130

Guideline = * = Guideline Exceedence

** = Analysis completed at Mississauga, Ontario.

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

MECHANICALL 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
OBJIOU CREEK BRIDGE NEAR HEARST, ONTARIO
SITE NO. 39W-59
G.W.P. 5044-11-00
MTO GEOCRETS NO. 42G-39**

Prepared for:

ONTARIO MINISTRY OF TRANSPORTATION

By:

SPL CONSULTANTS LIMITED

Project: 1067-710 (Objiou 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 a series of native soil and rock strata which include:

- a layer of stiff silty clay;
- Stiff to hard silty clay and clayey silt till;
- Hard silty clay and clayey silt;
- Very dense silty sand till;
- Granitic bedrock

With the exception of the upper stiff silty clay the soils are generally very competent, with very high “N” values evident during drilling.

Groundwater levels in both the native soils and the rock were found to be at an elevation of approximately 97.2 m to 97.5 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) will be a pre-fabricated bridge, 25 m to 30 m in length, with no intermediate piers (see Drawings 2A and 2B). It is understood the alignment and elevation of the new bridge will be nearly identical to the old one, and that no re-grading of the road and existing embankments will be required.

6.2 Frost Protection

The depth of frost penetration for the Obijou 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 granular fill which meets the requirements of OPSS 1010 for Granular B type I, and which would therefore be considered to have a low susceptibility to frost heave. The depth to more frost susceptible silty soils ranges from 0.8 m to 3.0 m in the various boreholes. Based on the conditions encountered at the borehole locations it is likely that there are frost-susceptible soils within the 2.6 m frost depth at the bridge location.

The existing road is a gravel-surfaced rural road, and it is understood that there are no plans to pave the road in the future. It is also understood the road is seasonal, and is not maintained through the winter. For these reasons, it is expected 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 some frost heaving and maintenance cannot be tolerated then frost tapers should be added installed.

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 I, which corresponds to a Site Coefficient $S = 1.0$ (CHBDC Section 4.4.6).

6.4 Foundations

The uppermost portion of the silty clay in Boreholes BH-3 and BH-4 contains significant amounts of organic soils, wood debris, etc. and is not considered suitable for shallow foundations. Based on the geometry of the bridge, shallow foundations would require excavations to significant depth (below the level of the watercourse) and would require temporary protection systems, dewatering, etc., which would complicate construction, impact the watercourse and eliminate much of the cost savings typically associated with shallow foundation construction.

For these reasons, shallow foundations are not recommended. It is recommended the proposed bridge be supported on deep foundations (piles).

6.4.1 Deep Foundations

The existing bridge is founded on a combination of steel and timber piles, and piles would be an acceptable foundation system for the new bridge. Several types of piles could potentially be used, however it is likely that the most economical foundation system would be driven steel piles (due to the relatively difficult drilling conditions, and the remote location which may impact concrete supply) for drilled piles.

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

6.4.1.1 Axial Capacity of Piled Foundations

Piles Driven to Rock

Steel piles driven to granite bedrock will generate relatively high ultimate geotechnical capacities, equal to or in excess of the structural capacity of the steel section. For the purposes of example, and

HP310x110 would generate a factored geotechnical resistance at Ultimate Limit State (ULS) on the order of 1,975 kN (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 Serviceability Limit State (SLS) condition would be expected to exceed the factored ULS value of the pile.

Combined Friction and End-Bearing Piles

As an alternative to end-bearing piles driven to rock, shorter piles could be designed on the basis of a combination of friction and end-bearing resistance. Shorter piles would have a lower capacity, but would not require driving to bedrock. This would have some advantages:

- The piles can be shorter and smaller, which will typically reduce the size of the equipment required for driving;
- There is less risk of damage to piles due to heavy driving (particularly through the lower till which is very competent and will be difficult to penetrate);
- There is less potential for construction delays due to piles which meet early refusal, become damaged and need to be replaced, cannot penetrate the lower till, etc.

Based on the results of the foundation investigation it is considered that there is a high probability of problems during driving if the foundation piles are designed using very high capacities on the assumption they will be driven to rock.

At the Obijou creek site the piles could (for example) be driven to the lower till strata which is 13 m to 15 m below the existing road surface (84.3 m elevation on the west side and 86.4 m elevation on the east side) but not to the rock surface.

It is likely that the existing pile foundations (particularly the timber piles) are driven to a shallow depth and are supported by a combination of friction and end bearing (rather than on rock).

The ultimate geotechnical resistance of a pile driven to the lower till layer (but not to rock) which is present at an elevation of approximately 84.3 m on the west side of the bridge and 86.4 m elevation on the east side may be calculated as:

$$R_u = A_s r_s + A_t r_t$$

Where:

R_u = the unfactored geotechnical resistance of the pile (kN);

A_s = the pile shaft area;

r_s = the unfactored shaft resistance (see Table 6 below);

A_t = the pile toe area;

r_t = the pile toe resistance (see Table 6 below).

Table 6 – Ultimate Pile Unit Resistances

Soil Strata	Unfactored Shaft Resistance (r_s)	Unfactored Toe Resistance (r_t)
Fill	50 kPa	--
Stiff Silty Clay	50 kPa	--
Silty Clay Upper Till	50 kPa	--
Hard Silty Clay & Clayey Silt	75 kPa	--
Very Dense/Hard Lower Till	150 kPa	9,000 kPa

Using the above values, the unfactored geotechnical capacity of the previous example HP310x110 driven to the lower till at approximately 15 m on the west abutment would be approximately 1,900 kN. This value is clearly lower than the capacity of a similar pile driven to rock. The design capacity will, however, be much easier to achieve in the soil conditions present at the site.

The total resistance of a pile group should be taken as the lesser of the sum of the individual pile capacities or the capacity of an equivalent block of soil with the same dimensions as the pile group.

The appropriate geotechnical resistance factor depends upon the method of confirming the actual pile capacity. For static analysis only, using the above values, the resistance factor should be taken as 0.5. For static analysis with confirmation through dynamic testing during construction the resistance factor may be taken as 0.5. A resistance factor of 0.6 may be used if the pile capacities are confirmed through static load testing (though for the small number of piles this is unlikely to be cost-effective). For the example pile this would yield estimated factored resistances of 760 kN to 950 kN. Actual pile capacities should be confirmed using the Hiley formula and/or dynamic (PDA) testing.

The settlement of foundations supported on piles will be dependent upon the dimensions of the pile group, the length of the piles and the pile loading. However, a properly designed pile foundation installed in the very stiff soils present at the Obijou creek site would be expected to experience relatively small deflections (less than 25 mm).

6.4.1.2 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. For the purposes of calculating the uplift resistance, the unfactored ultimate shaft friction along the pile shaft may be taken as outlined in Table 6 above. 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 19 kN/m³ for the soil unit weight.

6.4.1.3 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 most of the lateral resistance will be derived from the silty clay stratum (assuming the pile cap is buried for frost protection) k_h may be assumed to be:

$$k_h = 67 S_u/B$$

Where:

S_u = the undrained shear strength (use 75 kPa);

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

This parameter is associated with acceptable deflections, and therefore represents an unfactored 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 7 – 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).

6.4.1.4 Negative Friction and Downdrag

It is understood that 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 minimal.

6.4.1.5 Construction Considerations

Based on the results of the drilling program at the site it is expected that the bedrock surface is at an elevation of approximately 79.4 m on the west side of the bridge (BH-3) and 80.9 m on the east side of the bridge (BH-4). If piles are to be driven to rock they would need to be driven 18 m to 20 m through very dense/hard soils. Both the upper and lower till strata are expected to contain cobbles and boulders and piles should be fitted with appropriate driving shoes in order to protect the tip during driving. Any battered piles should be driven with rock points to avoid sliding along the rock surface (if piles are to be driven to rock). In addition, a relatively heavy pile section should be used to resist driving stresses and minimise damage during installation. Even with these measures, allowance should be made for wasting of piles which become damaged and for reduced design capacities for piles which cannot be driven to rock.

Shorter piles which would be designed for lower capacities and would derive their resistance from a combination of shaft friction as well as end bearing in the lower till. Although driving will still be through the hard silt/clay, the problems associated with penetrating the lower till will be eliminated, and there is more chance that a pile which meets early refusal will still achieve the required ultimate capacity (or close to it). Although this approach will require more piles it is less likely to result in cost overruns and delays during construction.

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 based on the contractor's proposed equipment and confirmed through a program of dynamic testing (PDA testing) carried out during installation.

It should be noted that the existing bridge abutments and piers are founded on 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).

6.4.3 Comparison of Foundation Options

A comparison of foundation options is presented in Table 8 below.

Table 8: Summary of Foundation Alternatives
Foundation Investigation and Design Report
Proposed Obijou Creek Bridge Replacement, Grenier Rd., Ontario Site No. 39-W59

Foundation Option	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Shallow Foundations	Not Recommended			
Driven Steel Piles (Driven to Rock)	<ul style="list-style-type: none"> Piles can achieve high capacities if driven to rock H-Piles are commonly available and provide a reliable foundation Pile capacities can be confirmed during construction through driving criteria and dynamic testing Does not require deep temporary excavations, dewatering, etc. Easy to isolate foundation construction from watercourse 	<ul style="list-style-type: none"> Requires heavy pile sections and large equipment to drive piles to deep rock surface through very dense/hard soils 	<ul style="list-style-type: none"> Higher than spread footings Similar to other deep foundation options 	<ul style="list-style-type: none"> Relatively low risk for completed works Some risk of pile damage and driving difficulties due to heavy driving and deep pile lengths, as well as delays for design changes and reviews
Driven Steel Piles (Friction and End Bearing in Soil)	<ul style="list-style-type: none"> Piles would be shorter and could potentially be driven with smaller equipment H-Piles are commonly available and provide a reliable foundation Pile capacities can be confirmed during construction through driving criteria and dynamic testing 	<ul style="list-style-type: none"> Smaller individual pile capacities (as compared to piles driven to rock) and therefore more piles 	<ul style="list-style-type: none"> Higher than spread footings Similar to other deep foundation options 	<ul style="list-style-type: none"> Relatively low risk Piles do not have to be driven through lower till stratum May be possible to avoid the need for field splicing of piles

Essentially the choice of foundation system depends on the relative importance of construction risk. Piles driven to rock will generate very high capacities, and therefore require fewer piles to support the bridge. They will, however, have to be driven through very dense/hard soils and installation problems (and associated delays) should be anticipated.

A foundation consisting of shorter piles will clearly require more piles, but the installation will be easier and less likely to result in delays during construction as well as large numbers of wasted or damaged piles. Given the relatively small size of the job (and therefore the high proportion of cost for mob/demob and setting up to drive the piles vs. the relatively small cost for the actual piles themselves) the additional cost for a larger number of shorter piles may not be a significant amount of the overall cost.

Based on considerations of potential construction problems and delays a foundation system consisting of shorter piles driven to the lower till is the preferred option, from a geotechnical perspective.

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 and in accordance with OPSS 206 and 902. All backfill material should be compacted in accordance with OPSS 501 and OPSS 902.

Heavy vibratory equipment should not be used adjacent to structures within the restricted zone as outlined in OPSS 501. 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. The coefficient of passive earth pressure (K_p) may be taken as 3.0 (unfactored).

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.

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 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 $A = 0$).

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 slopes into the existing creek are currently at an angle of 3:1 to 4:1 and will not be steepened as part of the current bridge replacement.

New bridge abutments supported on deep foundations will not provide any additional loading or surcharge to 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 majority of the settlement due to the embankment loading will have already occurred over the life of the existing structure. 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 (Section 6.6) assume no undermining will due to erosion or scour of the native soils. The soils in the creek bed would be expected to include silty clay and organic soils, 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 sizing of the erosion protection will depend upon the anticipated velocities and flood levels in the water course, and should be carried out by a specialist who is familiar with the hydraulics of the creek and the findings of this investigation.

6.8 Additional Construction Considerations

Construction Dewatering

The groundwater level within the glacial till at the site was found to be at approximately 97.2 m to 97.5 m local elevation, or 1.7 m to 2.3 m depth below the existing road surface. This elevation is approximately coincident with the level of the watercourse at the time of the investigation.

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 early May 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 fill embankment will be saturated to the level of the creek.

Excavations for pile caps will likely be below the groundwater table. For excavations only slightly below the water table (less than 1 m) it may be feasible to manage seepage using properly filtered sumps and ditches. For deeper excavations an active excavation de-watering system as well as some form of cut-off (such as sheet pile walls) will be required to reduce the flow of water into the excavation, in particular through the fill and the buried organic layer between the fill and the underlying silty clay.

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 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.

Temporary shoring would generally consist of interlocking sheet piles or a similar structural support (which for this site would need to be relatively water tight). It should be noted that cobbles and boulders were encountered in the till layers and should be anticipated during construction.

6.9 Corrosion and Cement Type

Three 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 C and are summarized in Table 9 below.

Table 9 – Results of Soil Corrosivity Testing

Sample No.	Soil Type	Soil Parameter			
		Resistivity (ohm-cm)	pH	Chloride (%)	Sulphate (%)
BH-3 SS2	Silty Clay Fill	5880	7.5	0.004	0.21
BH-3 SS5	Silty Clay	5000	6.5	0.002	< 0.01
BH-4 SS3	Granular Fill	8330	7.9	< 0.002	< 0.01
BH-4 SS6	Silty Clay	3120	7.8	< 0.002	< 0.01

The test results indicate that the sulphate content of the existing fill in BH-3 Sample 2 is 0.21%. This corresponds to a moderate to severe degree of sulphate exposure. Sulphate resistant Portland cement is recommended.

The test results indicate that there is a moderate potential for corrosion of buried steel elements within the silty clay fill and the native silty clay soils. In addition, the pH of the silty clay at Borehole BH-3 is acidic, which further promotes corrosion of steel. 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-S6-06 Canadian Highway Bridge Design Code, Reprinted October 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.