

**FOUNDATION INVESTIGATION REPORT
PROPOSED STURGEON RIVER DETOUR BRIDGE
HIGHWAY 64
WEST NIPISSING, ONTARIO
G.W.P. 211-93-00
SITE 43-019**

GEOCRES NO. 41I-200-A

Prepared For:

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Prepared by:

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**Project: SPT1155
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1. INTRODUCTION

Shaheen & Peaker Limited (S&P) was retained by Lea Consulting Limited (LEA) to carry out a foundation investigation at the site of the proposed Sturgeon River Replacement Bridge Project located in the Town of Field, in the Municipality of West Nipissing, approximately 23 km northwest of Highway 17 near Sturgeon Falls. The site falls within MTO District 54 and has MTO Site Number 43-019.

The Sturgeon River Bridge Replacement Project consists of the design and construction of a temporary detour bridge (Sturgeon River Detour Bridge) and a permanent replacement bridge (Sturgeon River Replacement Bridge). The Sturgeon River Detour Bridge, which is covered in this report, is located immediately downstream of the existing bridge, which will be replaced by the proposed Sturgeon River Replacement Bridge.

The purpose of this investigation was to reveal the subsurface conditions at the site of the proposed Sturgeon River Detour Bridge by means of boreholes and to determine the engineering characteristics of the subsurface soils by means of field and laboratory tests.

The findings of the investigation for the temporary detour bridge (Sturgeon River Detour Bridge) are presented in this report.

In 2003, a preliminary geotechnical investigation was carried out by Trow Associates Inc. (ref: Project No. S09329G and dated May 5, 2004) for the Preliminary Design Study of the project. Borehole information relevant to this investigation is presented in Appendix C of this report.

For the purpose of this report, the foundation data presentation of the temporary detour bridge labeled from south to north is as follows (see Drawing No. 1):

- South Approach
- South Abutment
- Centre Pier
- North Abutment
- North Approach.

2. SITE DESCRIPTION AND GEOLOGY

The site of the existing bridge is located where Highway 64 crosses over the Sturgeon River in the former Town of Field, in the Municipality of West Nipissing, Ontario. It is located approximately 23 km northwest of Highway 17 in Sturgeon Falls. The existing bridge is a three-span pony truss and steel stringer structure with a concrete deck, and was constructed in the late 1940's. The bridge is 57.5 m long, with a roadway width of 9.14 m, and has 1.52 m sidewalks along each edge. Some site photographs are attached in Appendix E.

The proposed Sturgeon River Detour Bridge will be located immediately downstream (southeast) of the existing bridge.

Based on available geologic information, the site is in an area of ice-contact sediments. Generally after the last glacial withdrawal, ice-contact sediments (sands and gravels) followed by glaciofluvial sediments (ranging from deltaic and near shore sands and gravels to prodeltaic and lake bottom silts and clays) were deposited on top of the existing sandy glacial till or Precambrian bedrock. The area was then inundated by glacial/erosional activities, depositing sands, silts and clays in low-lying areas. The bedrock generally consists of strongly foliated gneissic to migmatic rocks of the Central Gneiss Belt of the Grenville Province located within the Canadian Shield. It is typical in this type of geology that the bedrock surface is undulating with high variations of overburden soil types and the presence of cobbles or boulders in the overburden.

The elevation of water surface of the Sturgeon River is controlled by the Crystal Falls Dam which is approximately 16 km downstream. Based on information available to us, the water elevation in April 2003 was about 222.1 m and the 1:50 year water level is about 225.7 m.

3. INVESTIGATION PROCEDURES

The fieldwork for the proposed detour bridge was performed during the period of November 9 through November 19, 2005 and as agreed with MTO, it consisted of drilling and sampling eighteen (18) boreholes at the following locations:

LOCATION	BOREHOLE NO.	NO. OF BOREHOLES	DRILLING CONDITION
South Approach	DAP1	1	On land
South Abutment	DSA1 to DSA5	5	On land
	DSA6	1	In river
Centre Pier	DCP1 to DCP6	6	In river
North Abutment	DNA1 to DNA4	4	On land
North Approach	DAP2	1	On land

The plan location of the boreholes is shown on Drawing No. 1. The depth of borehole drilling and rock coring varied between 3.4 m and 13.7 m.

A specialist drilling contractor (Walker Drilling Limited of Utopia, Ontario) carried out the drilling, field testing and sampling work under the direction and supervision of Geotechnical Engineers from S&P. All borehole elevations and locations were determined in the field by S&P.

For the boreholes drilled on land (see above Table), the boreholes were advanced using continuous-flight solid-stem augers powered by a drilling rig, outfitted with tools and equipment for soil sampling and testing. For drilling boreholes in the river (Boreholes DCP1 to DCP6 and DSA6), a skid-mounted drill rig supported by a raft was used. The boreholes were advanced using wash-boring methods. A considerable amount of drilling mud was utilized to counter-balance the hydrostatic uplift due to water table; as well, the sampler and the rods were withdrawn slowly to reduce suction below the groundwater table.

For all the boreholes drilled, samples in the boreholes were taken at frequent intervals of depth by the Standard Penetration Test method (SPT), in general accordance with ASTM D1586. The test consists of freely dropping a 63.5 kg hammer a vertical distance of 0.76 m to drive a 51 mm O.D. split barrel (SS – split-spoon) sampler into the ground. The number of blows of the hammer required to drive the sampler into the relatively undisturbed ground by a vertical distance of 0.30 m is recorded as the Standard Penetration Resistance or the N-value of the soil which is indicative of the compactness condition of granular (or cohesionless) soils (gravels, sands and silts) or the consistency of cohesive soils (clays and clayey soils).

In cohesive (clayey) deposits, where the consistency of the soil permitted, relatively undisturbed samples were taken with thin-walled (TW) Shelby tubes which were pushed into the borehole by the application of static weight by hydraulic pressure. The undrained shear strength of the soil was also measured in-situ by Field Vane tests. Where consistency permitted, MTO Field Vane was used to conduct the tests but when the soil became stiffer this was changed to small Field Vane.

In order to advance the boreholes through cobbles and boulders and to prove bedrock, rotary core drilling was carried out in Boreholes DCP1 to DCP6, DNA1, DNA4, DSA2, DSA5 and DSA6 utilizing NQ size casings and core barrel.

Groundwater conditions in the boreholes were observed during and on completion of drilling in the open boreholes. Upon their completion, the boreholes were grouted using a cement/bentonite mixture as per MTO procedures.

The soil samples and rock cores were transported to our geotechnical laboratory in Toronto for further examination and classification. A laboratory testing programme, consisting of natural moisture content determinations, grain size analyses, Atterberg Limits tests of

selected soil samples and Point Load Strength Index test of rock cores (ASTM D5731-95), was performed on selected representative samples. The results of the laboratory tests are presented on the appropriate Record of Borehole Sheets (Appendix A) and also in Appendix B.

4. SUBSURFACE CONDITIONS

The sub-surface conditions were explored at eighteen (18) boreholes (see Table in Section 3 above) during the current investigation. The plan locations of the boreholes along with the inferred stratigraphic sections along the proposed detour bridge alignment are shown on Drawings 1 to 3. Details of sub-surface conditions encountered at each borehole location for the current investigation, including the results of in-situ testing, groundwater observations and laboratory test results, are presented on the Record of Borehole Sheets in Appendix A. Detailed laboratory test results are enclosed in Appendix B. Relevant borehole information (Records of Boreholes) from previous preliminary investigation near the site (Boreholes BH5B, BH7B and BH8B) put down by others in September 2003, is also provided in Appendix C for reference purposes.

In general, the sub-surface stratigraphy comprises surficial topsoil and/or fill materials overlying very loose to very dense cohesionless silty sand, sandy silt to silt deposits, which are in turn underlain by compact to very dense glacial till with frequent cobbles and boulders, and followed by gneiss bedrock. Difficult augering and sampling spoon bouncing were noted during borehole drilling, indicating the presence of cobbles and boulders in the subsoils. It should be noted that gravel greater than 35 mm in size as well as cobbles and boulders could not be sampled with the standard spoon sampler.

The various soil strata encountered in the boreholes and their geotechnical properties are briefly described in the following subsections of this report. Please note that the following summary is to assist the designers of the project with an understanding of the anticipated soil conditions across the site. Detailed geotechnical information is presented in the Record of Borehole sheets (Appendix A). It should be noted that the soil, bedrock and groundwater conditions may vary in between and beyond borehole locations.

4.1 TOPSOIL

Topsoil was encountered in all boreholes, except those boreholes drilled in the river (i.e. Borehole DCP1 to DCP6 and DSA6), ranging in thickness from 0.1 m to 0.2 m.

At Boreholes DAP1, DAP2 and DNA4, a layer of fill material was encountered overlying the topsoil with thickness ranging from 0.2 m to 0.7 m. The fill materials mainly consist of sand to gravelly sand with occasional cobbles.

It should be noted that the thickness of topsoil may vary in between and beyond the borehole locations.

4.2 FILL

As mentioned before in Boreholes DAP1, DAP2 and DNA4, fill was encountered overlying the topsoil. Typically these fill deposits consisted of sand to gravelly sand with occasional cobbles and the thickness ranged from 0.2 to 0.7 m. In addition, a layer of about 0.8 m to 2.9 m of fill materials was encountered underlying the topsoil in Boreholes DNA1 to DNA4 and Boreholes DSA1 to DSA5. These fill deposits consisted of sand or silty sand to sandy silt or silt with traces to some gravel, some organics and occasional cobbles, and extended to Elevations ranging from 226.0 m to 223.0 m..

The fills are granular materials (i.e. non-cohesive) and the recorded N-values ranged from 3 to 26 blows/0.3 m, indicating very loose to compact relative density. The measured natural moisture contents range from 5 to 32%.

Two grain size distribution analyses were carried out on samples of the relatively finer fill materials with the results as follows:

Gravel: 0 – 2%
Sand: 12 – 47%
Silt: 44 – 75%
Clay: 9 – 11%

The grain size distribution curves are shown in Figure B1 in Appendix B.

4.3 SANDY SILT, SILTY SAND AND SILT DEPOSITS

Below the topsoil (in Boreholes DAP1 and DAP2) or the fill materials (Boreholes DNA1, DNA2 and DNA3, and DSA1 to DSA5), a cohesionless (i.e. fine-grained granular) deposit with occasional clay seam and organics was encountered to depths of about 2.1 m to 10.7 m (Elevations 223.8 m to 214.5 m) with thickness ranging from 1.5 m to 9.8 m. Boreholes DCP2, DCP5, DCP6 and DSA6 which were drilled in the river, encountered this deposit at river bed (2.1 m to 9.4 m below the surface of water in the river) to depths of about 8.0 m to 9.4 m (El. 214.1 m to 212.7 m) below water surface, with thickness ranging from 0.8 m to 5.9 m. These fine grained granular (i.e. non-cohesive) deposits consisted of either sandy silt, silty sand, silty fine sand or silt, or the combination of two or more of these soil types as shown in the Record of Borehole Sheets. In Borehole DAP1, 1.2 m thick silty clay was found interlayered with this deposit. Boreholes DAP1, DNA2 and DSA3 were terminated in this deposit to depths of 5.3 m to 9.2 m (El. 220.7 m to El. 216.0 m), upon encountering auger/spoon refusal.

Measured N-values within this deposit typically range from 1 blow per 0.3 m to over 26 blows per 0.3 m indicating a very loose to compact relative density, but in general, a very loose to loose relative density. Difficult augering conditions which were occasionally encountered,

may be attributed to probable cobbles and/or boulders. It is noted that the cobbles and boulders could not be sampled with the spoon sampler.

Laboratory and field test results from soil samples in this deposit are as follows:

Silty Sand

Natural Moisture Content: 18 to 23%
Grain Size (5 samples)
Sand: 52 – 78%
Silt: 22 – 42%
Clay: 6 – 7%

The grain size curves for this material are presented in an envelope form provided on Figure B2.

Sandy Silt

Natural Moisture Content: 32%
Grain Size (1 samples)
Sand: 40%
Silt: 52%
Clay: 8%

The grain size curve for this material is given on Figure B3.

4.4 SILTY CLAY

Borehole DAP1 encountered a silty clay layer within the above-mentioned cohesionless deposits at depths between 4.8 m to 6.0 m (1.2 m thick) below existing ground surface (El. 220.4 m to El. 219.2 m). The presence of very thin clayey silt, silty clay and clay seams was noted in the fine grained granular immediately above and below this cohesive silty clay layer.

Atterberg Limits test performed in the laboratory on a selected sample from this cohesive material gave the following index values, as shown in Figure B5.

Liquid Limit: 22%
Plastic Limit: 18%
Plasticity Index: 4

These values are generally typical of clays of low to medium plasticity (i.e. CL - ML). However, this sample was obtained from the top zone of the deposit and the deposit attained a more plastic (clayey) nature with increasing depth.

The measured natural moisture contents ranged from 23 to 25% and are closer to the measured liquid limit rather than the plastic limits with Liquidity Index values between 1.25 and 1.75.

Standard Penetration tests conducted in this silty clay deposit gave N-value of 3 blows/0.3 m. Undrained shear strengths as measured by Field Vane tests range from 22 to 28 kPa, indicating a soft to firm consistency.

4.5 SILTY SAND TO SANDY SILT TILL

A stratum of silty sand to sandy silt till (glacial till) consisting of a heterogeneous mixture of sand, silt and gravel with occasional cobbles and boulders was encountered in all boreholes except Boreholes DAP1, DAP2, DNA2, DSA3, DSA5 and DSA6. The composition of this basically granular (i.e. non-cohesive) glacial deposit varies from silty sand till to relatively finer silt till. Frequent cobbles and boulders were encountered or inferred at variable depths within the till. Boreholes DNA3, DNA5 and DSA4 were terminated in this deposit upon auger/spoon refusal on possible boulders or bedrock.

At the borehole locations, the thickness of this unit ranges from 0.8 m (Borehole DCP1) to 4.6 m (Borehole DNA4) and it extends to depths of about 5.4 m to 12 m (El. 220.8 to El. 210.1 m). Measured SPT N-values within the glacial till varied from 10 to in 50 blows, indicating that the till is in a compact to very dense relative density, but in general dense to very dense.

Grain size distribution analyses were conducted on selected soil samples from this stratum, giving the following grain size measurements:

Natural Moisture Content: 9 to 17%
Grain Size (5 samples)
Gravel: 8 – 14%
Sand: 49 – 65%
Silt: 16 – 35%
Clay: 5 – 8%

The grain size curves for this material are provided on Figure B4.

4.6 ZONE OF COBBLES AND BOULDERS

Underlying the glacial till deposit in Boreholes DCP1, DCP2 and DCP5, and underlying the sandy deposit in Borehole DAP1, a very coarse granular soil consisting of cobbles and boulders with sand and gravel infill was contacted at 8.9 m to 11.0 m depth (El. 216.3 m to El. 211.1 m), and extended to depths of 9.2 m to 11.9 m (El. 216.0 m to El. 210.2 m). Thickness

of this stratum varies from 0.3 m to 1.0 m. Measured SPT N-values for this deposit are generally over 50 blows per 0.3 m penetration. Due to the coarse nature of the soil, these values may not be reliable nevertheless it is believed that they indicate a dense to very dense relative density.

Boreholes DCP1, DCP3 and DCP4 (boreholes drilled in the river) encountered this deposit at river bed to depths of about 8.2 m to 8.6 m (El. 213.9 m to 213.5 m) below water surface and overlying the glacial till deposit, with thickness ranging from 0.7 m to 1 m. This upper zone of cobbles and boulders has measured SPT Nvalues ranging from 12 to over 50 blows per 0.3 m penetration, indicating a compact to very dense relative density, but typically compact. During the time of investigation, no evidence of artesian pressure in this deposit was noted.

Due to the extremely coarse nature of this deposit, representative samples could not be obtained for grain-size analysis. In some boreholes, owing to the coarse nature of this deposit, washboring and diamond coring had to be utilized to penetrate it.

It should be pointed out that the presence of cobbles and boulders can be expected at other locations at the site.

4.7 BEDROCK

Bedrock was cored and proven in Boreholes DCP1 to DCP6, DNA1, DNA4, DSA2, DSA5 and DSA6. The cored lengths ranged between 0.5 m to 3.2 m. Bedrock or boulders level were inferred by auger/spoon refusal in Boreholes DAP1, DAP2, DNA2, DNA3, DSA1, DSA3 and DSA4. Photographs of some rock cores samples are included in Appendix D. The following table summarizes the approximate bedrock surface elevations:

Borehole No.	Ground/Water Surface Elevation (m)	Depth to Bedrock Surface (proven by rock coring) (m)	Elevation Of Bedrock Surface (proven by rock coring) (m)	Depth to Inferred Bedrock / Boulder Surface (Auger Refusal) (m)	Elevation of Inferred Bedrock / Boulder Surface (Auger Refusal) (m)
DAP1	225.2	---	---	9.2	216.0
DAP2	225.9	---	---	3.4**	222.5**
DCP1	222.0	10.2	211.8	---	---
DCP2	222.1	10.5	211.6	---	---
DCP3	222.1	10.6	211.5	---	---
DCP4	222.1	12.0	210.1	---	---
DCP5	222.1	11.9	210.2	---	---
DCP6	222.1	11.7	210.4	---	---
DNA1	225.7	6.6	219.1	---	---
DNA2	226.0	---	---	5.3	220.7
DNA3	226.0	---	---	5.4	220.6
DNA4	226.0	6.7	219.3	---	---
DSA1	225.3	---	---	9.9	215.4
DSA2	225.3	9.8	215.5	---	---
DSA3	225.3	---	---	8.5	216.8
DSA4	225.3	---	---	10.4	214.9
DSA5	225.2	10.7	214.5	---	---
DSA6	222.1	8.0	214.1	---	---
BH5B	226.1	6.4	219.7	---	---
BH7B	225.5	---	---	5.7	219.8
BH8B	225.7	6.3	219.4	---	---

Note: **possible coarse grained materials or boulders in the glacial till.

The recovered rock core samples show that the Precambrian bedrock consists of a slightly weathered massive, moderately closely to closely jointed pinkish grey to greenish grey gneiss with occasional micaceous layers. The joints are largely sub-vertical. The percentage of Total Core Recovery varies from 60% to 100%. The Rock Quality Designation (RQD) values increase with depth from 17% to 100%. Occasional values of lower rock core recovery or RQD values may be attributed to the coring operations and/or the presence of mica zones.

Based on these values and visual examination of the cores, the rock is considered to be of very poor to excellent quality, but in general fair to excellent quality.

4.8 WATER SURFACE AND GROUNDWATER CONDITIONS

Groundwater conditions were observed in the open boreholes during the drilling and upon completion of each borehole. However, because wash-boring methods were used in some boreholes, particularly in rock coring operations, water level conditions in these type of boreholes may not be useful.

It is believed that the water level at the site would be largely controlled by the water level in the water course and can be expected on both sides of the river valley, to be somewhat higher than the water level in the Sturgeon River. The elevation of water surface of the Sturgeon River is controlled by the Crystal Falls Dam which is approximately 16 km downstream. Based on information available to us, the water elevation in April 2003 was about 222.1 m and the 1:50 year water level is about 225.7 m.

The groundwater table would be subject to seasonal fluctuations and in response to major weather events.

SHAHEEN & PEAKER LIMITED

Zuhtu Ozden, P.Eng.

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ZO:tr/ldrive



Drawings

Appendix A

Record of Borehole Sheets (Present Investigation)

SPT 1155

RECORD OF BOREHOLE No DAP 1

1 OF 1

METRIC

GWP 211-93-01 LOCATION Highway 64, Sturgeon River Detour Bridge, ON Coords: N 5153625.6; E 264363.6 ORIGINATED BY G.I.
DIST 54 HWY 64 BOREHOLE TYPE Solid Stem Augers COMPILED BY J.Z.
DATUM Geodetic DATE 11/19/2005 CHECKED BY Z.O.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					
								○ UNCONFINED	+ FIELD VANE	● POCKET PENETR.			
225.2	Ground Surface						20	40	60	80	100		
0.0	FILL: GRAVELLY SAND greyish brown, dense, moist		1	SS	45								
224.5													
0.7	PEATY TOPSOIL, black		2	SS	26								
224.3													
0.9	some organics to 1.2 m		3	SS	4								
	SILT very loose to compact, wet, dilatant		4	SS	23								
	some sandy silt & clayey silt seams		5	SS	17								
	some clayey silt & clay seams stiff		6	SS	10								
220.4			7	TW	PH								
4.8	SILTY CLAY grey, soft to firm		8	SS	3								
219.2			9	SS	8								
6.0	some clayey silt & clay seams												
	SILTY FINE SAND with sandy silt seams		10	SS	12								
216.0	cobbles & boulders		11	SS	50/8								
9.2	End of Borehole. Auger refusal @ 9.2m, possible bedrock. *Water level at 3.7 m (not stabilized) and hole open to 4.6 m on completion.												

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



SPT 1155

RECORD OF BOREHOLE No DAP 2

1 OF 1

METRIC

GWP 211-93-01 LOCATION Highway 64, Sturgeon River Detour Bridge, ON Coords: N 5153725.5; E 264432.5 ORIGINATED BY G.I.
DIST 54 HWY 64 BOREHOLE TYPE Solid Stem Augers COMPILED BY J.Z.
DATUM Geodetic DATE 11/19/2005 CHECKED BY Z.O.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)							
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE							WATER CONTENT (%) PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT w _p w w _L						
225.9	Ground Surface		1	SS	50/8		225	224	223	20	40	60	80	100	20	40	60	GR	SA	SI	CL
226.0	FILL: GRAVELLY SAND 0.2 m TOPSOIL																				
224.5	SANDY SILT to SILTY SAND yellowish brown, compact, moist		2	SS	17																
224.5	SILT with clayey silt seams yellowish brown, very stiff, wet, dilatant		3	SS	22																
223.8	SILTY SAND (possible till) greyish brown, very dense, wet		4	SS	56																
222.5			5	SS	50/10																
3.4	End of Borehole. Spoon refusal at 3.4 m. *Water level at 3.1 m (not stabilized) and hole open to full depth on completion.		6	SS	100/3																

+ 3, x 3: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

SPT 1155

1 OF 1

METRIC

GWP	211-93-01	LOCATION	Highway 64, Sturgeon River Detour Bridge, ON	Coords: N 5153683.3; E 264392.3	ORIGINATED BY	G.I.	
DIST	54	HWY	64	BOREHOLE TYPE	Solid Stem Augers & Wash Boring & Diamond Coring	COMPILED BY	J.Z.
DATUM	Geodetic	DATE	11/9/2005		CHECKED BY	Z.O.	

[illegible]

+³, ×³: Numbers refer to Sensitivity

SPT 1155

RECORD OF BOREHOLE No DCP 2

1 OF 1

METRIC

GWP 211-93-01 LOCATION Highway 64, Sturgeon River Detour Bridge, ON Coords: N 5153679.3; E 264393.7 ORIGINATED BY G.I.
DIST 54 HWY 64 BOREHOLE TYPE Solid Stem Augers & Wash Boring & Diamond Coring COMPILED BY J.Z.
DATUM Geodetic DATE 11/10/2005 CHECKED BY Z.O.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
222.1 0.0	Water Surface						222						
							221						
							220						
							219						
							218						
							217						
							216						
							215						
214.5 7.6	SILTY SAND to SANDY SILT traces of gravel, occ. cobbles & boulders grey, compact, wet		1	SS	12		214						
213.7 8.4	SILTY SAND to SANDY SILT TILL occ. cobbles & boulders grey, compact, wet		2	SS	20		213						
			3	SS	11		212						
212.2 9.9	COBBLES & BOULDERS with sand, some gravel infill		4	SS	50/0		211						
211.6 10.5	GNEISS BEDROCK pinkish grey, fine to medium grained, slightly weathered in fractures, moderately closely to closely jointed, occasional micaceous layers		5	NQ RC	Rec. 97%								RQD=65%
210.1 12.0	End of Borehole. Auger refusal at 10.5 m, switch to casing & rock coring. Wash boring used to facilitate coring. Borehole open to full depth on completion.												

+ 3, x 3: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

SPT 1155

RECORD OF BOREHOLE No DCP 3

1 OF 1

METRIC

GWP 211-93-01 LOCATION Highway 64, Sturgeon River Detour Bridge, ON Coords: N 5153682.1; E 264396.1 ORIGINATED BY G.I.
DIST 54 HWY 64 BOREHOLE TYPE Solid Stem Augers & Wash Boring & Diamond Coring COMPILED BY J.Z.
DATUM Geodetic DATE 11/10/2005 CHECKED BY Z.O.

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			20 40 60 80 100	20 40 60 80 100	W P W W L	20 40 60			
222.1 0.0	Water Surface													
	WATER													
214.6 7.5	COBBLES & BOULDERS sand & gravel infill		1	SS	22									
213.9 8.2	SILTY SAND TILL with gravelly sand layers occ. cobbles & boulders grey, compact to dense, wet		2	SS	37									
			3	SS	20									
			4	SS	60/20									
211.5 10.6	cobbles & boulders		5	NQ										
				RC										
210.0 12.1	GNEISS BEDROCK with granite inclusion, pinkish grey, fine to medium grained, slightly weathered, moderately closely to closely jointed, occasional micaceous layers		6	NQ RC	Rec. 98%									RQD=82%
	End of Borehole. Auger refusal at 10.2 m, switch to casing & rock coring. Wash boring used to facilitate coring. Borehole open to 0.6 m on completion.													

+ 3, x 3: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

SPT 1155

RECORD OF BOREHOLE No DCP 4

1 OF 1

METRIC

GWP 211-93-01 LOCATION Highway 64, Sturgeon River Detour Bridge, ON Coords: N 5153676.6; E 264401.7 ORIGINATED BY J.Z.
DIST 54 HWY 64 BOREHOLE TYPE Solid Stem Augers & Wash Boring & Diamond Coring COMPILED BY J.Z.
DATUM Geodetic DATE 11/16/2005 CHECKED BY Z.O.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
222.1 0.0	Water Surface						222							
							221							
							220							
							219							
							218							
							217							
							216							
							215							
214.2 7.9	COBBLES & BOULDERS with sand infill		1	SS	12		214							
213.5 8.6			2	SS	16		213							
			3	SS	47		212							
			4	SS	59		211							
			5	SS	46		210							
210.1 12.0			6	SS	50/10		209							
			7	NQ	Rec.									
208.8 13.3	End of Borehole. Auger refusal at 12 m, switch to casing & rock coring. Wash boring used to facilitate coring. Borehole open to 2.1 m on completion.													

+ 3, x 3: Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE

SPT 1155

1 OF 1

METRIC

+³, ×³: Numbers refer to Sensitivity

SPT 1155

1 OF 2

METRIC

GWP	211-93-01	LOCATION	Highway 64, Sturgeon River Detour Bridge, ON	Coords: N 5153674.3; E 264402.8	ORIGINATED BY	G.I.	
DIST	54	HWY	64	BOREHOLE TYPE	Solid Stem Augers & Wash Boring & Diamond Coring	COMPILED BY	J.Z.
DATUM	Geodetic	DATE	11/11/2005		CHECKED BY	Z.O.	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	
222.1 0.0	Water Surface					
	WATER					
214.3 7.8	SILTY FINE SAND trace gravel & organics occ. cobbles & boulders grey, compact, wet		1	SS	19	
			2	SS	29	
212.8 9.3	SANDY SILT TILL occ. cobbles & boulders grey, dense, wet		3	SS	42	
			4	SS	44	
			5	SS	47	
210.4 11.7	GNEISS BEDROCK pinkish grey, fine grained, slightly weathered, moderately closely to closely jointed, occasional micaceous layers		6	NQ RC	Rec. 92%	
			7	NQ RC	Rec. 92%	
207.3 14.8						

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
20	40	60	80	100	W _p	W	W _L		
SHEAR STRENGTH kPa									
○ UNCONFINED			+ FIELD VANE						
● POCKET PENETR.			× LAB VANE						
20	40	60	80	100	WATER CONTENT (%)				
					20	40	60		

222									
221									
220									
219									
218									
217									
216									
215									
214						○			
213									
212						○			
211						○			
210									
209									
208									

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Continued Next Page

+³, ×³: Numbers refer to Sensitivity

SPT 1155

RECORD OF BOREHOLE No DCP 6

2 OF 2

METRIC

GWP 211-93-01 LOCATION Highway 64, Sturgeon River Detour Bridge, ON Coords: N 5153674.3; E 264402.8 ORIGINATED BY G.I.
 DIST 54 HWY 64 BOREHOLE TYPE Solid Stem Augers & Wash Boring & Diamond Coring COMPILED BY J.Z.
 DATUM Geodetic DATE 11/11/2005 CHECKED BY Z.O.

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			20	40					
	End of Borehole. Auger refusal at 11.7 m, switch to casing & rock coring. Wash boring used to facilitate coring. Borehole open to full depth on completion.													

SPT 1155

RECORD OF BOREHOLE No DNA 1

1 OF 1

METRIC

GWP 211-93-01 LOCATION Highway 64, Sturgeon River Detour Bridge, ON Coords: N 5153712.3; E 264417.1 ORIGINATED BY J.Z.
DIST 54 HWY 64 BOREHOLE TYPE Solid Stem Augers & NQ Rock Coring COMPILED BY J.Z.
DATUM Geodetic DATE 11/11/2005 CHECKED BY Z.O.



SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																							
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● POCKET PENETR. X LAB VANE					WATER CONTENT (%) W _P W W _L																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																						
225.7	Ground Surface							20	40	60	80	100																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																							

SPT 1155

1 OF 1

METRIC

GWP	211-93-01	LOCATION	Highway 64, Sturgeon River Detour Bridge, ON	Coords: N 5153705.6; E 264425.1	ORIGINATED BY	J.Z.	
DIST	54	HWY	64	BOREHOLE TYPE	Solid Stem Augers	COMPILED BY	J.Z.
DATUM	Geodetic	DATE	11/10/2005		CHECKED BY	Z.O.	

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			20 40 60 80 100	20 40 60 80 100	20 40 60						
226.0 0.0	Ground Surface															
223.0 3.0	0.1 m TOPSOIL FILL: SANDY SILT trace gravel, some organics, occ. cobbles compact to loose, yellowish brown to 1.5 m, brown to 3 m		1	SS	7											
			2	SS	15											
			3	SS	13											
			4	SS	11											
220.7 5.3	SANDY SILT grey, wet compact to loose ----- dense		5	SS	13											
			6	SS	8											
			7	SS	41											
			8	SS	50/0											
End of Borehole.																
Spoon and auger refusal at 5.3 m.																
*Borehole dry (not stabilized) and open to full depth on completion.																

+³, ×³: Numbers refer to Sensitivity

SPT 1155

RECORD OF BOREHOLE No DNA 3

1 OF 1

METRIC

GWP 211-93-01 LOCATION Highway 64, Sturgeon River Detour Bridge, ON Coords: N 5153704.8; E 264427.2 ORIGINATED BY J.Z.
DIST 54 HWY 64 BOREHOLE TYPE Solid Stem Augers COMPILED BY J.Z.
DATUM Geodetic DATE 11/11/2005 CHECKED BY Z.O.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L		
226.0	Ground Surface												
0.0	0.1 m TOPSOIL		1	SS	6		226						
	FILL: SAND trace gravel, some organics dark brown, loose to compact												
225.0	1.0		2	SS	13		225						
	dark brown to yellowish brown yellowish brown												
	SANDY SILT compact, damp to moist		3	SS	15		224						
	occ. thin clay seams		4	SS	11		223						
223.2	2.8		5	SS	8		222						
	SILTY SAND to SANDY SILT occasional thin clay seams grey, loose, wet		6	SS	10		221						
221.9	4.1		7	SS	37								
	SILTY SAND TILL with silty sand layers grey, compact to dense, wet												
220.8	5.2		8	SS	50/5								
	SILT TILL, grey, very dense												
220.6	5.4												
	End of Borehole. Spoon refusal at 5.4 m. *Borehole dry (not stabilized) and open to full depth on completion.												

SPT 1155

RECORD OF BOREHOLE No DNA 4

1 OF 1

METRIC

GWP 211-93-01 LOCATION Highway 64, Sturgeon River Detour Bridge, ON Coords: N 5153704.3; E 264429.6 ORIGINATED BY J.Z.
DIST 54 HWY 64 BOREHOLE TYPE Solid Stem Augers & NQ Rock Coring COMPILED BY J.Z.
DATUM Geodetic DATE 11/10/2005 CHECKED BY Z.O.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				
								20	40	60		
226.0	Ground Surface		1A	SS	26							GR SA SI CL
0.0	FILL: SAND some gravel & silt, occ. cobbles brown, compact											
225.3												
0.7	0.15 m TOPSOIL		1	SS	14							2 12 75 1
	FILL: SILT with slightly organic zones, yellowish brown compact to loose		2	SS	9							
223.9												
2.1	SILTY SAND TILL with some sand & silt seams brown		3	SS	26							
			4	SS	24							
221.6			5	SS	53							
4.4	SILT TILL some cobbles & boulders grey, very dense		6	SS	50/15							
			7	SS	50/13							
			8	SS	50/3							
219.3			9	NQ								
6.7			10	NQ	Rec. RC 60%							RQD=50%
			11	NQ	Rec. RC 100%							RQD=95%
216.2			12	NQ	Rec. RC 100%							RQD=86%
9.8	End of Borehole. Auger refusal at 6.1 m, switch to casing & rock coring. Water used to facilitate coring. Borehole open to full depth on completion.											

SPT 1155

RECORD OF BOREHOLE No DSA 1

1 OF 1

METRIC

GWP 211-93-01 LOCATION Highway 64, Sturgeon River Detour Bridge, ON Coords: N 5153645.7; E 264363.0 ORIGINATED BY J.Z.
DIST 54 HWY 64 BOREHOLE TYPE Solid Stem Augers COMPILED BY J.Z.
DATUM Geodetic DATE 11/14/2005 CHECKED BY Z.O.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
225.3 0.0	Ground Surface											
	0.1 m TOPSOIL		1	SS	7		225					
	FILL: Silty Sand to Sandy Silt some topsoil, traces of organics brown/dark brown, loose, damp		2	SS	9		224					
223.5 1.8			3	SS	7		223					
	SANDY SILT to SILTY FINE SAND occasional very thin clay seams greyish brown, very loose moist to 3 m, wet below		4	SS	5		222					
			5	SS	4		221					
			6	SS	4		220					
220.9 4.4			7	SS	7		219					
			8	SS	12		218					
			9	SS	11		217					
	SILTY SAND grey, wet		10	SS	20		216					
217.4 7.9			11	SS	63							
	SILTY SAND TILL grey, compact to very dense, wet											
215.4 9.9	End of Borehole. Auger refusal at 9.9 m. *Water level at 3.1 m (not stabilized) and hole open to 3.1 m on completion.											

+ 3, x 3: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

SPT 1155

RECORD OF BOREHOLE No DSA 2

1 OF 1

METRIC

GWP 211-93-01 LOCATION Highway 64, Sturgeon River Detour Bridge, ON Coords: N 5153651.5; E 264366.4 ORIGINATED BY J.Z.
DIST 54 HWY 64 BOREHOLE TYPE Solid Stem Augers & NQ Rock Coring COMPILED BY J.Z.
DATUM Geodetic DATE 11/12/2005 CHECKED BY Z.O.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)		
225.3 0.0	Ground Surface						20 40 60 80 100	PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L					
	0.1 m TOPSOIL		1	SS	11										
	FILL: Sandy Silt some organics brown/dark brown, compact to loose, damp		2	SS	7										
223.9 1.4															
	brown, damp		3	SS	5										
	SANDY SILT with some silty sand layer grey, very loose, wet		4	SS	3										
			5	SS	2										
			6	SS	2										
	occ. very thin clay seams		7	SS	2										
220.2 5.1															
	SILTY SAND grey, compact to loose, wet		8	SS	12										
			9	SS	8										
218.0 7.3															
	SILTY SAND TILL grey, compact, wet		10	SS	23										
	very dense		11	SS	68/23										
215.5 9.8															
	GNEISS BEDROCK greenish grey to grey, fine to medium grained, slightly weathered, moderately closely to closely jointed, occasional micaceous layers		12	NQ RC	Rec. 90%										
			13	NQ RC	Rec. 98%										
			14	NQ RC	Rec. 66%										
			15	NQ RC	Rec. 71%										
212.3 13.0	End of Borehole.														
	Auger refusal at 9.8 m, switch to casing & rock coring. Water used to facilitate coring.														
	*Water level at 5.5 m (not stabilized) and hole open to 5.5 m on completion.														

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

SPT 1155

RECORD OF BOREHOLE No DSA 3

1 OF 1

METRIC

GWP 211-93-01 LOCATION Highway 64, Sturgeon River Detour Bridge, ON Coords: N 5153647.3; E 264368.5 ORIGINATED BY J.Z.
DIST 54 HWY 64 BOREHOLE TYPE Solid Stem Augers COMPILED BY J.Z.
DATUM Geodetic DATE 11/13/2005 CHECKED BY Z.O.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● POCKET PENETR. X LAB VANE		WATER CONTENT (%)				
225.3	Ground Surface													
0.0	0.1 m TOPSOIL FILL: Sandy Silt occ. topsoil layers/pockets brown, very loose to compact, moist		1	SS	4		225							
			2	SS	15		224							
223.9			3	SS	6		223							
1.4			4	SS	4		222							
			5	SS	2		221							
			6	SS	4		220							
			7	SS	9		219							
			8	SS	6		218							
			9	SS	10		217							
			10	SS	3									
216.8	End of Borehole. Auger refusal at 8.5 m, probably a boulder. *Water level at 2.8 m (not stabilized) and hole open to 2.8 m on completion.													
8.5														

SPT 1155

1 OF 1

METRIC

GWP	211-93-01	LOCATION	Highway 64, Sturgeon River Detour Bridge, ON	Coords: N 5153643.4; E 264373.5	ORIGINATED BY	J.Z.	
DIST	54	HWY	64	BOREHOLE TYPE	Solid Stem Augers	COMPILED BY	J.Z.
DATUM	Geodetic	DATE	11/14/2005		CHECKED BY	Z.O.	

[illegible]

+³, ×³: Numbers refer to Sensitivity

SPT 1155

RECORD OF BOREHOLE No DSA 6

1 OF 1

METRIC

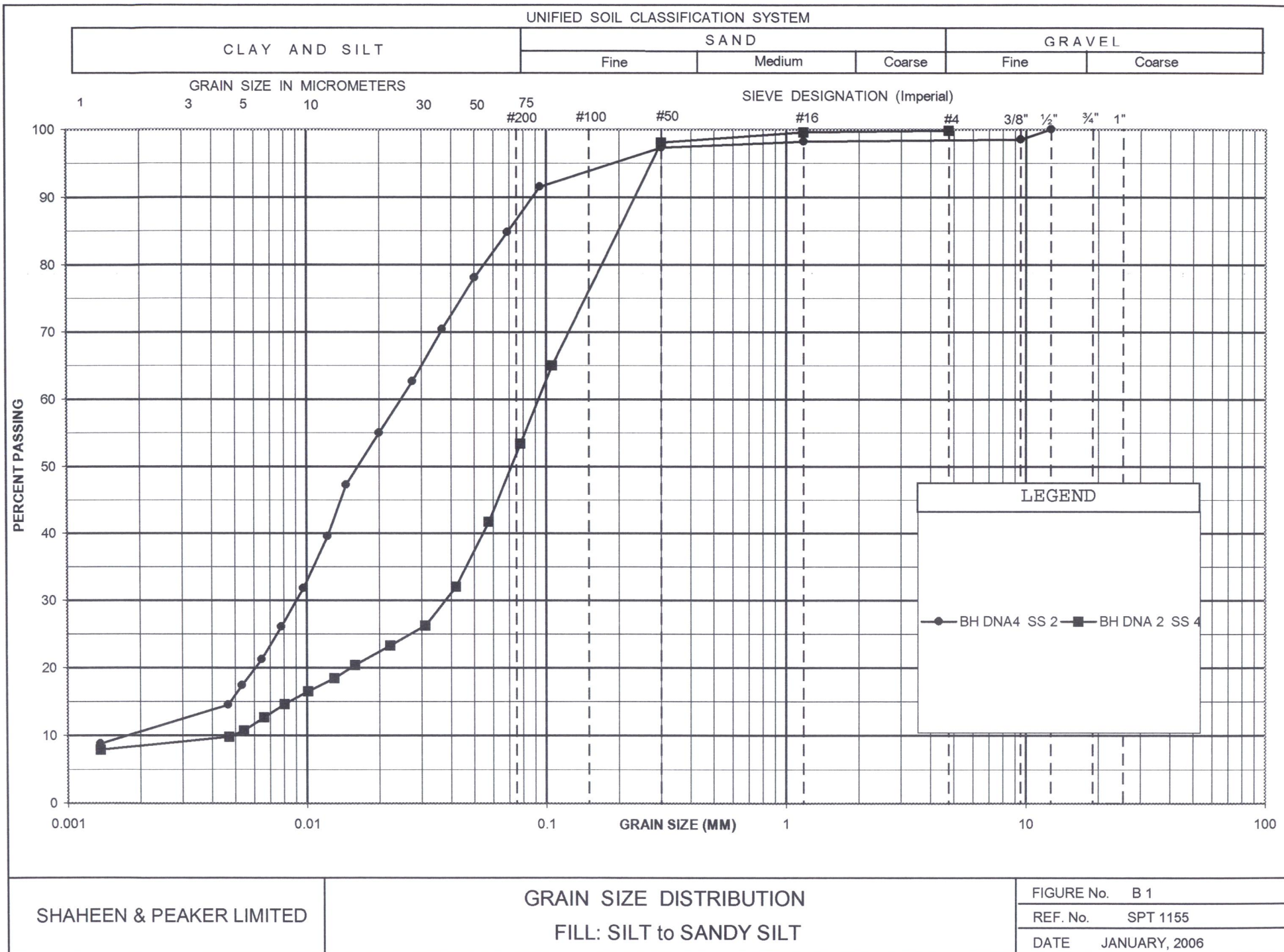
GWP 211-93-01 LOCATION Highway 64, Sturgeon River Detour Bridge, ON Coords: N 5153641.9; E 264378.0 ORIGINATED BY G.I.
DIST 54 HWY 64 BOREHOLE TYPE Solid Stem Augers & Wash Boring & Diamond Coring COMPILED BY J.Z.
DATUM Geodetic DATE 11/15/2005 CHECKED BY Z.O.

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			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								○ UNCONFINED	+ FIELD VANE	● POCKET PENETR.						× LAB VANE		
222.1 0.0	Water Surface																	
	WATER																	
220.0 2.1		SANDY SILT to SILTY SAND some organic silt content occ. wood pieces & rootlets dark grey to grey, very loose to loose, wet		1	SS	1												
				2	SS	3												
				3	SS	6												
				4	SS	5												
			5	SS	4													
			6	SS	2													
			7	SS	7													
		compact	8	SS	20													
214.1 8.0	GNEISS BEDROCK with granite inclusion, pinkish grey to grey, fine to medium grained, slightly weathered, moderately closely to closely jointed, occasional micaceous layers		9	NQ RC	Rec. 95%										RQD=73%			
			10	NQ RC	Rec. 100%										RQD=100%			
211.1 11.0	End of Borehole. Auger refusal at 9.0 m, switch to casing & rock coring. Water used to facilitate coring. Borehole open to 0.6 m on completion.																	

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

Appendix B

Laboratory Test Results



SHAHEEN & PEAKER LIMITED

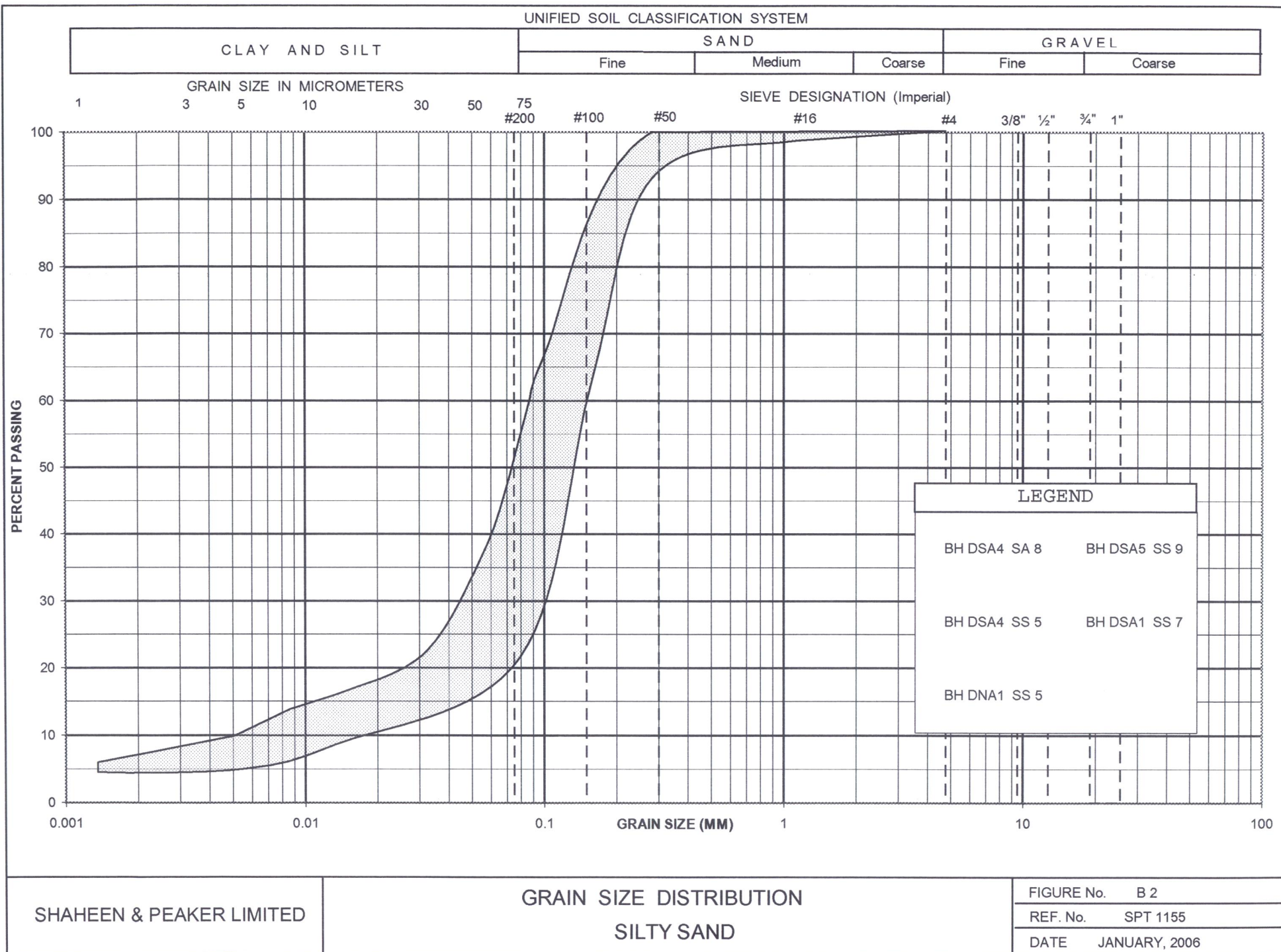
GRAIN SIZE DISTRIBUTION

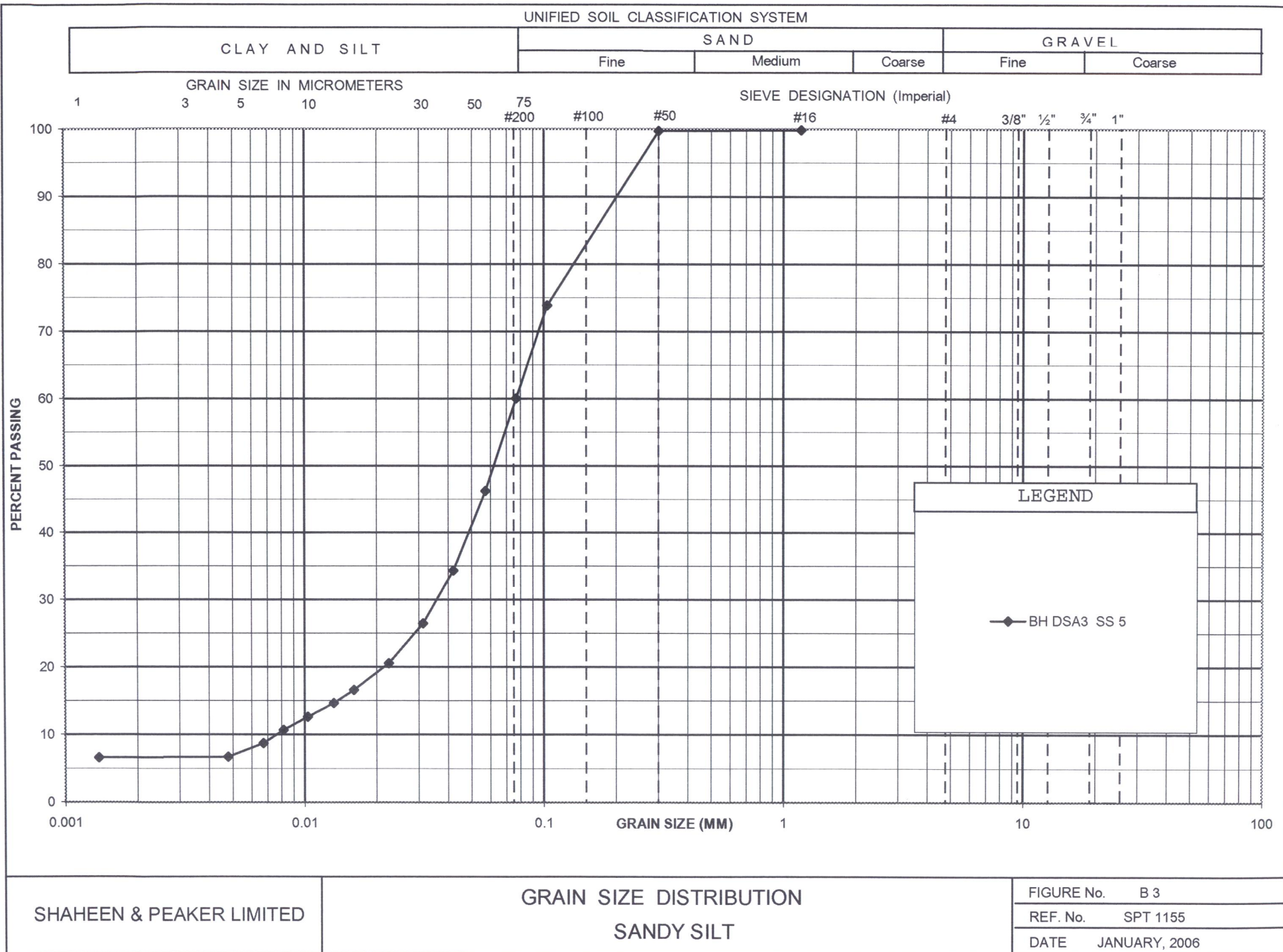
FILL: SILT to SANDY SILT

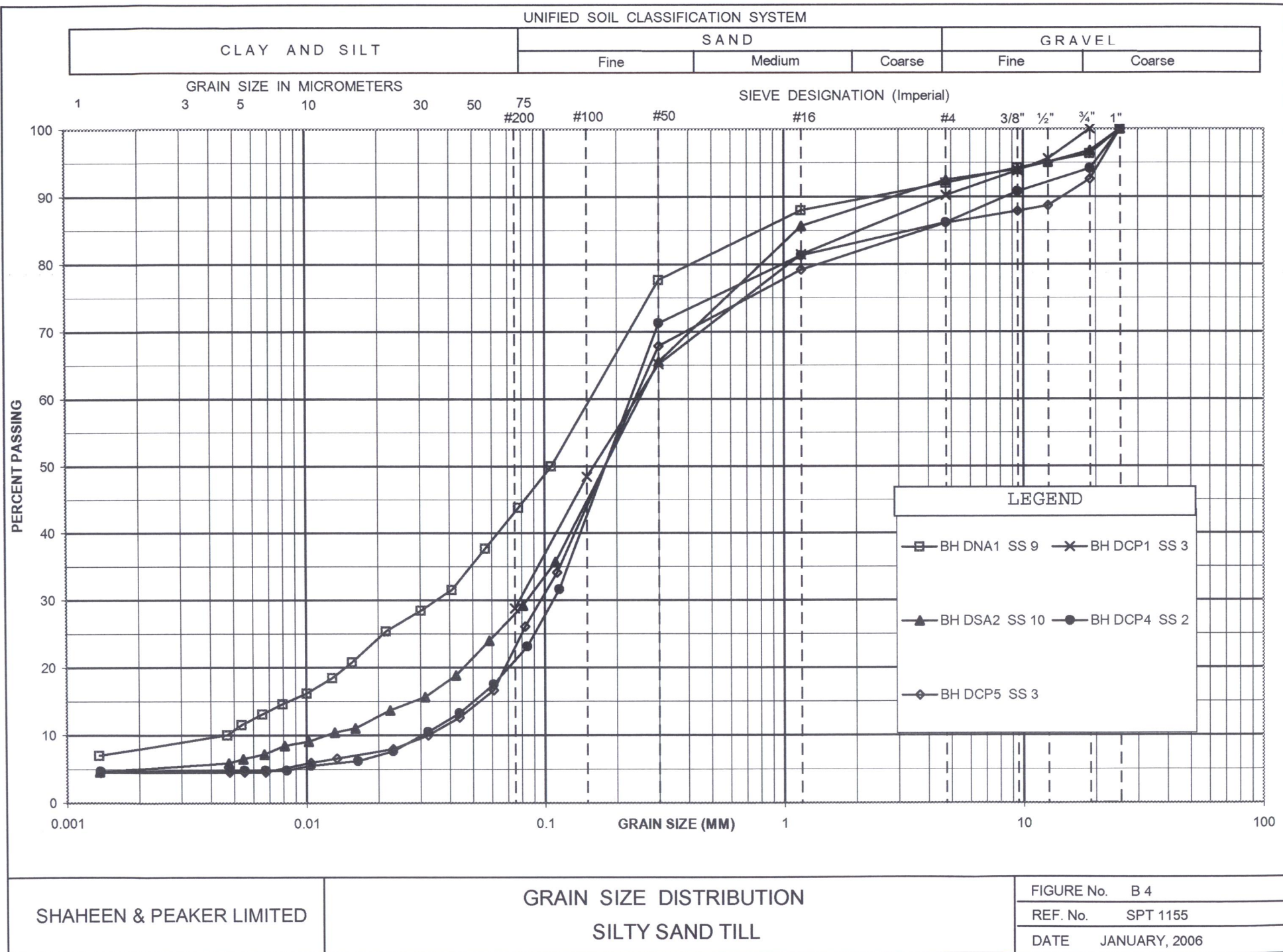
FIGURE No. B 1

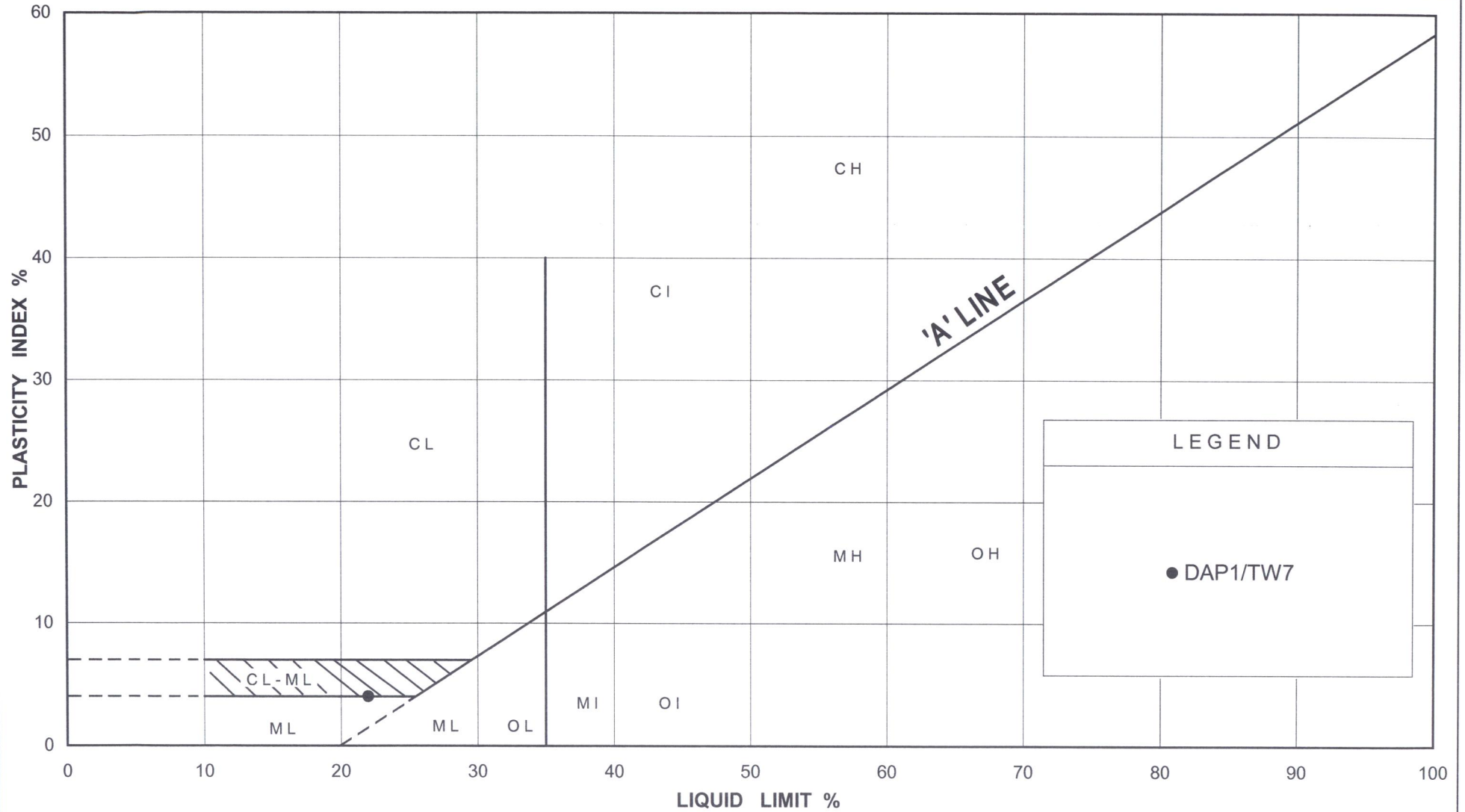
REF. No. SPT 1155

DATE JANUARY, 2006









SHAHEEN & PEAKER LIMITED

PLASTICITY CHART

FIGURE B 5

G.W.P. 211-93-00

REF No SPT 1155

Appendix C
Record of Borehole Sheets for BH-5B, BH-7B
and BH-8B
(Previous Preliminary Investigation by Others)
Geocres No. 41I-170

RECORD OF BOREHOLE BH7B

1 OF 1

METRIC

WP No. 211-93-01 LOCATION FIELD, ONTARIO, N 5153708, E 264420 ORIGINATED BY P.C.
 DIST Nipissing HWY 64 BOREHOLE TYPE 200 mm diameter HOLLOW STEM AUGER / CME-55 DRILL RIG COMPILED BY A.Q.
 DATUM Geodetic DATE September 18, 2003 CHECKED BY T.C.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20					
225.5	GROUND SURFACE												
0.0	SAND AND GRAVEL, brown, damp, very loose, poorly graded, medium to coarse grained sand, fine to coarse grained gravel, some organics.		1	SS	3								
			2	SS	3								
223.1			3	SS	3								
2.4	SILTY SAND, light brown, wet, very loose, poorly graded, fine grained, some silt. - compact to dense, some clay, trace to some rootlets below ~3.05 m depth.		4	SS	30								
220.8													
4.7	SILTY CLAY, grey, wet, soft to firm, medium plasticity, trace fine grained sand.		5	SS	3								
220.1													
5.3	SILTY SAND, grey, wet, very dense, poorly graded, fine grained, interlayered coarse grained sand ~40 mm apart and 13 mm thick.		6	SS	68								
219.7													
5.7	AUGER REFUSAL ON BEDROCK OR BOULDER AT ~5.74 m DEPTH.												



RECORD OF BOREHOLE BH8B

1 OF 1

METRIC

WP No. 211-93-01

LOCATION FIELD, ONTARIO, N 5153712, E 264422

ORIGINATED BY P.C.

DIST Nipissing HWY 64

BOREHOLE TYPE 200 mm diameter HOLLOW STEM AUGER / CME-55 DRILL RIG

COMPILED BY A.Q.

DATUM Geodetic

DATE September 24, 2003

CHECKED BY T.C.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION			
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			SHEAR STRENGTH: Cu, KPa				WATER CONTENT (%)							
								0 ● UNCONFINED QUICK TRIAXIAL	20 40 60 80	+FIELD VANE LAB SHEAR	80	wp	w	wl					
225.7	GROUND SURFACE														GR	SA	SI	CL	
0.0	SANDY ORGANICS						225												
225.0																			
0.8	SILTY SAND, reddish brown, damp, loose, poorly graded, fine grained, trace silt. - brown, trace fine grained gravel below ~1.52 m depth. - brown to grey, compact to very dense, well graded, fine to coarse grained, trace organics below ~2.29 m depth. - well graded, trace fine to medium grained gravel below ~3.05 m depth. - grey, wet, fine grained, trace coarse sand, trace medium grained gravel below ~3.81 m depth.		1	SS	5		224	X											
			2	SS	8														
			3	SS	10		223	X											
			4	SS	54					X									
			5	SS	26		222		X								5%	41%	54%
			6	SS	50		221			X									
220.4																			
5.3	SILTY SAND TILL, grey, compact, poorly graded, fine grained, some medium to coarse grained sand, some fine to medium grained gravel.		7	SS	13		220	X											
219.4			8	SS	100							X/Bounding					16%	47%	37%
6.3	GRANITIC GNEISS pinkish grey to greenish grey, slightly weathered, coarse grained, limited fracturing along mica planes, some subvertical joints, foliation dips at ~20°, very hard.		9	NQ			219												
			10	NQ			218												
							217												
216.5																			
9.2	END OF BOREHOLE AT ~9.22 m DEPTH. ROCK CORE: At ~6.30-7.77 m depth: Rec=100%, RQD=100% At ~7.77-9.22 m depth: Rec=100%, RQD=90%																		



RECORD OF BOREHOLE BH5B

1 OF 1

METRIC

WP No. 211-93-01 LOCATION FIELD, ONTARIO, N 5153712, E 264416 ORIGINATED BY P.C.
 DIST Nipissing HWY 64 BOREHOLE TYPE 200 mm diameter HOLLOW STEM AUGER / CME-55 DRILL RIG COMPILED BY A.Q.
 DATUM Geodetic DATE September 18, 2003 CHECKED BY T.C.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20 40 60 80				wp wl				
								SHEAR STRENGTH: Cu, KPa				WATER CONTENT (%)				
								UNCONFINED QUICK TRIAXIAL FIELD VANE LAB SHEAR								
						20 40 60 80				10 20 30 40			kN/m ³ GR SA SI CL			
226.1	GROUND SURFACE															
225.4	ASPHALT, ~80 mm thick FILL: SAND AND GRAVEL, brown, damp, well graded, medium to coarse grained.		1	SS	11		X									
224.6	SAND, brown, damp, compact, well graded, medium to coarse grained.		2	SS	5		X									
221.6	SAND AND GRAVEL, brown, damp, loose, well graded, medium to coarse grained sand, coarse grained gravel. - wet, compact below ~3.81 m depth.		3	SS	5		X									
			4	SS	6		X									
			5	SS	10		X									
			6	SS	13		X									
221.6	SILTY SAND, brown, wet, compact, poorly graded, fine grained, some silt.		7	SS	18		X									
219.7	- brown, very dense below ~6.1 m depth.		8	SS	100											
215.5	GRANITIC GNEISS pinkish grey to greenish grey, slightly weathered, medium to coarse grained, limited fracturing, some subvertical joints, foliation dips at ~20°, very hard.		9	NQ												
9.6	END OF BOREHOLE AT ~9.6 m DEPTH.		10	NQ												
ROCK CORE: At ~6.4-8.0 m depth: Rec=100%, RQD=90% At ~8.0-9.6 m depth: Rec=100%, RQD=100%																



Appendix D

Photographs of Rock Cores



Photograph D-1 Borehole DCP6



Photograph D-2 Borehole DNA1

Appendix E

Site Photographs



Photograph E-1 South Side (Looking North), August 2005



Photograph E-2 West Side (Looking East) , August 2005



Photograph E-3 North Side (Looking South), August 2005



Photograph E-4 Boulders under the Bridge, August 2005

Appendix F

Explanation of Terms Used in Report

EXPLANATION OF TERMS USED IN REPORT

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

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND 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_s	1	SWELLING INDEX
c_a	1	RATE OF SECONDARY CONSOLIDATION
c_v	m ² /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_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
j_s	kN/m ³	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
P_w	kg/m ³	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
j_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
j	kN/m ³	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
P_d	kg/m ³	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m ³ /s	RATE OF DISCHARGE
j_d	kN/m ³	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $(W_L - W_p) / I_p$	v	m/s	DISCHARGE VELOCITY
P_{sat}	kg/m ³	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $(W - W_p) / I_p$	i	1	HYDAULIC GRADIENT
j_{sat}	kN/m ³	UNIT WEIGHT OF SATURATED SOIL	I_c	1	CONSISTENCY INDEX = $(W_L - W) / 1_p$	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
j'	kN/m ³	UNIT WEIGHT OF SUBMERGED SOIL						

**FOUNDATION DESIGN REPORT
PROPOSED STURGEON RIVER DETOUR BRIDGE
HIGHWAY 64
WEST NIPISSING, ONTARIO
G.W.P. 211-93-00
SITE 43-019**

GEOCRES NO. 41I-200-A

Prepared For:

LEA CONSULTING LIMITED

Prepared by:

SHAHEEN & PEAKER LIMITED

**Project: SPT1155
July 14, 2006**



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APPENDIX G LIMITATIONS OF REPORT

**FOUNDATION DESIGN REPORT
PROPOSED STURGEON RIVER DETOUR BRIDGE
HIGHWAY 64
WEST NIPISSING, ONTARIO
G.W.P. 211-93-00
GEOCRES NO. 41I-200-A**

5. DISCUSSION AND RECOMMENDATIONS

The Sturgeon River Bridge Replacement Project consists of the design and construction of a temporary detour bridge (Sturgeon River Detour Bridge) and a permanent replacement bridge (Sturgeon River Replacement Bridge). The Sturgeon River Detour Bridge, which is covered in this report, is located immediately downstream of the existing bridge, which will be replaced by the proposed Sturgeon River Replacement Bridge.

The site of the existing bridge is located where Highway 64 crosses over the Sturgeon River in the former Town of Field in the Municipality of West Nipissing, Ontario, and approximately 23 km northwest of Highway 17 in Sturgeon Falls. The existing bridge is a three-span pony truss and steel stringer structure with a concrete deck, and was constructed in the late 1940's. The bridge is 57.5 m long, with a roadway width of 9.14 m, and has 1.52 m wide sidewalks along each side of the bridge.

The proposed Sturgeon River Detour Bridge consists of a two-span modular bridge with 9.14 m in width (two-lane) and a standard footwalk on the east side of the bridge. The length of the bridge is about 82 m. The proposed elevation of the top of the detour bridge structure is approximately 226.2 m, and the height of the approach embankment fill will be in the order of 1 m (or less) above the existing grade.

The sub-surface conditions were explored at eighteen (18) boreholes (see Table in Section 3 of the Foundation Investigation Section of this report) during the current investigation. In general, the sub-surface stratigraphy comprises surficial topsoil and/or fill materials overlying very loose to very dense cohesionless silty sand, sandy silt to silt deposits, which are in turn underlain by compact to very dense glacial till with frequent cobbles and boulders, and followed by gneiss bedrock. Difficult augering and sampling spoon bouncing were noted during borehole drilling, indicating the presence of cobbles and boulders in the subsoils.

The water level at the site would be largely controlled by the water level in the water course. The elevation of water surface of the Sturgeon River is controlled by the Crystal Falls Dam which is approximately 16 km downstream. Based on information available to us, the water elevation was about 222.1 m in April 2003 and the 1:50 year water level is about 225.7 m.

The groundwater level at the abutment locations can be expected to be somewhat higher (about 0.5 m to 1.0 m higher) than the water level in the river.

The groundwater table can be expected to be subject to seasonal fluctuations and in response to major weather events.

5.1 FOUNDATIONS

Based on the results of geotechnical investigation and consideration of the rather complex subsurface conditions at the site, we have studied a number of foundation options varying from normal spread footings to deep foundations which include drilled caissons, driven piles (steel H-piles, steel tube piles, pre-cast concrete piles and timber piles) and auger-press piles.

The drilled caisson foundation option socketed into the very dense till or in the underlying bedrock was considered. The presence of very dense glacial till together with frequent cobbles and boulders overlying the bedrock and the prevailing high water table may render the use of caisson foundations at some support element locations an uneconomical solution, in particular at the pier location. Auger-press concrete pile may be relatively advantageous for use in water-bearing granular deposits, but this pile type could be costly and offer low resistance to lateral loading. In addition, they may not be able to extend to desired depths due to the presence of boulders. Auger-press piles are therefore considered to be unsuitable for this project based on cost and reliability.

Driven piles at this site will suffer from the disadvantage that it is difficult to determine to what depths the piles will drive. It is considered that some piles will reach the bedrock while others will terminate in the cobbles and boulders zone. The use of driven H-piles to the surface of the gneiss bedrock can be considered at the abutment, however the piles may be too short (i.e. the lack of pile embedment in the overburden (minimum 4 to 5 m) may have adverse effect to the stability of the pile foundation), as will be discussed later.

The relative merits and disadvantages of various foundation support types are summarized in Table 5.1.1.

Table 5.1.1
Summary of Foundation Alternatives

Foundation Type	Comments	Recommendations
○ Spread footings on bedrock.	Feasible foundation option but extensive excavation and dewatering required, especially at the pier locations	Recommended at abutment locations, but may not be economical due to deep excavations and dewatering requirements.
○ Drilled concrete caissons extending into bedrock.	Technically feasible but may be difficult to reach bedrock due to the presence of cobbles and boulders in the overburden. Also difficult to install due to water-bearing granular overburden.	Technically feasible but may be costly, especially at the pier location.
○ Driven Piles (steel H-piles, steel tube piles, pre-cast concrete piles and timber piles)	Only H-piles are feasible due to presence of cobbles and boulders within the overburden which could damage the piles. Also limited by sloping bedrock surface.	H-Piles can be considered at the abutment locations provided that adequate pile length can be achieved (i.e. if pile lengths are not too short).
○ Heavy wall steel pipe pile socketed into bedrock with grout anchors. Surficial cobbles and boulders at river bed to be grouted to provide a stable base when advancing the steel pipe pile. The hollow sections of the steel pipe pile will be filled with concrete/grout.	Pile diameters limited to 12" to 18" (300 mm to 450 mm). Down-the-Hole Hammer may be required to break and remove the boulders and/or rock to allow construction of rock socket of the pile.	Recommended for Centre Pier but should be checked for cost-effectiveness.

Based on the data obtained from the boreholes, the proposed Detour Bridge can be founded on spread footing foundations supported on the undisturbed, dewatered dense to very dense glacial till or on the underlying gneiss bedrock. However at some borehole locations under any one support element, the footing need to be extended right to the surface of the bedrock or very close to it (i.e. the overburden is not competent enough). Since it is not desirable to support any one foundation partly on overburden and partly on bedrock (which may lead to possible differential movements within the footing), placing the footing on the overburden is not considered to be a good choice (i.e. it is recommended that in such cases all spread footing be carried down to the surface of the bedrock). The foundation design will need to take into consideration of dewatering requirements for excavations extending below the water table and the fact that the construction and dewatering will be carried out adjacent to a water

course (Sturgeon River) or in the River itself (i.e. the pier). With this background the following are our recommendations at each support location.

5.1.1 NORTH ABUTMENT

The following options can be considered for the foundation of the north abutment

5.1.1.1 SPREAD FOOTING FOUNDATIONS

Based on the borehole data (from this investigation and previous investigation) the north abutment can be founded on spread footing foundations placed on the undisturbed very dense glacial till stratum at about 4 m to 7 m below the existing ground or on the underlying bedrock as tabulated below.

Table 5.1.1.1.1

Foundation Location	Reference Borehole	Existing Ground Surface Elevation (m)	Recommended Highest Footing Base (Bottom) Level Below Existing Ground Surface (m)	Recommended Highest Footing Base (Bottom) Elevation (m)	Subgrade Material
West side	DNA1	225.7	5.6 6.6	220.1 219.1	Glacial Till Gneiss Bedrock
West side	5B	226.1	6.1 6.4	220.0 219.7	Silty Sand Gneiss Bedrock
West side	7B	225.5	5.4 5.7	220.1 219.8	Silty Sand Inferred Possible Bedrock
West side	8B	225.7	6.1 6.3	219.6 219.4	Glacial Till Gneiss Bedrock
East side	DNA2	226.0	4.9 5.3	221.1 220.7	Glacial Till/Boulders Inferred Possible Bedrock
East side	DNA3	226.0	5.0 5.4	221.0 220.6	Glacial Till Inferred Possible Bedrock
East side	DNA4	226.0	4.4 6.7	221.6 219.3	Glacial Till/Boulders Gneiss Bedrock

It should be noted that in between and beyond the borehole locations, the bedrock surface and the depth to the surface of the competent till may vary considerably.

For design purposes the following bearing resistances (with reference to the founding elevations as shown in the above table) may be used:

Table 5.1.1.1.2

Soil / Rock Type	Factored Bearing Resistance at ULS (kPa)	Bearing Resistance at SLS (kPa)
Glacial Till/Boulders	800	450
Gneiss Bedrock	5000	---

The factored bearing resistance at ULS given in the above table incorporates a resistance factor of 0.5 as per Canadian Highway Bridge Design Code (CHBDC), CAN/CSA-S6-00. The SLS will not govern the design for footings founded on bedrock.

The serviceability condition is based on the premise that the maximum total and differential settlements will not exceed 25 mm and 20 mm, respectively. This can be achieved provided that the founding subgrade is undisturbed during the construction.

The actual founding elevation should also be chosen with due consideration for frost and scour depths, as well as dewatering requirements.

As can be seen from the above table, deep excavation below water table will be required. As the footings should be constructed in the dry conditions, dewatering as well as a temporary shoring system will be required.

Due to the nature of the subsoil materials, unless proper dewatering is effected, seepage may occur from the cohesionless layers (sandy to silty materials) and glacial till deposits, and tight interlocking sheet piling may be required. It should be noted that, the presence of random cobbles and boulders in the till may render the installation of sheet piles difficult and these aspects should be taken into consideration in the design of sheet piles and the dewatering scheme.

Allowance should be made to place a 100 mm thick concrete mud mat (i.e. skim coat) in all footing excavations when the bearing surface consists of overburden as soon as possible after excavation. All footing excavations should be inspected and approved by the geotechnical engineer prior to pouring the concrete mud mat.

Steel dowels or rock anchors into bedrock, as discussed in Section 5.1.4, may be required to provide sufficient sliding and uplift resistance.

At some borehole locations (e.g. Boreholes 5B, 7B, 8B, DNA2 and DNA4), the bearing stratum in the overburden is very close to the surface of the bedrock/inferred possible bedrock (i.e. 0.2 m to 0.4 m). For ease of construction as well as more favourable

geotechnical resistance values, it is better to extend the footing right down to the surface of bedrock. At Borehole DNA1 and DNA4, on the other hand, there is 1.0 m to 2.3 m vertical distance to the surface of the bedrock. It is therefore more economical to place the footing on the overburden at these two locations (i.e. west and east ends). However, as was mentioned before, it is not good engineering practice to place the footing on two different type of bearing media. It is therefore recommended that the footing be extended to the surface of the bedrock along its entire width and length.

5.1.1.2 CAISSON FOUNDATIONS

Augered and cast-in-place concrete foundations (drilled caissons) can be considered. Caisson foundations socketed at least 0.5 m into the gneiss bedrock can be designed using a factored Bearing Resistance at ULS of 8000 KPa. Bearing Resistance at SLS will not govern. Socketing the caissons deeper into the bedrock may be required to aid increase the horizontal resistances. This design value applies to commonly used caisson sizes in Ontario (i.e. typically between 0.76 m and 2.1 m diameter). We recommend, however, the use of smaller size caissons (i.e. between 0.76 and 1.2 m) be adopted for practical purposes. This is because difficulties may be encountered during the installation of the caissons due to the presence of granular overburden below water table. As well, cobbles and boulders may be present in the glacial till and in the zone immediately above the bedrock surface. As well, the cost of socketing into bedrock will increase significantly for larger diameter caissons. This can be discussed with a specialist contractor, particularly with relation to cost effectiveness of available caisson drilling equipment with relation to diameter, etc. The minimum caisson diameter should be 0.76 to enable the inspection and clean-out of the base, if necessary.

5.1.1.3 DRIVEN H-PILES

Another alternative which may be considered is the use of short H-piles to support the abutment.

The following table summarizes the approximate anticipated pile tip elevation at each borehole location for HP 310 x 110 steel H-pile.

Borehole No.	Estimated Pile Tip Refusal Depth Below Existing Ground (m)	Estimated Pile Refusal Elevation (m)	Refusal Stratum
DNA1	6.6	219.1	Gneiss Bedrock (occasional boulders above bedrock surface)
DNA2	5.3*	220.7*	Inferred possible bedrock or boulders
DNA3	5.4*	220.6*	Inferred possible bedrock or boulders
DNA4	6.7	219.3	Gneiss Bedrock (occasional boulders above bedrock surface)
5B	6.4	219.7	Gneiss Bedrock
7B	5.7*	219.7*	Inferred Possible Bedrock or Boulders
8B	6.3	219.4	Gneiss Bedrock

Note: *Inferred possible bedrock or boulders

The following axial resistances are estimated for HP 310 x 110 steel H-piles driven to practical refusal as documented above.

- Factored Axial Resistance at U.L.S.= 1500 kN/pile
- Axial Resistance at S.L.S. = 1000 kN/pile

These values were chosen in consideration of the fact that the piles will be rather short and that some piles may not reach the surface of the bedrock.

The piles will need to be driven using a suitably heavy hammer capable of delivering a rated energy of at least 50 kJ/blow, but not more than 65 kJ/blow. All pile driving should be carried out in accordance with SP903S01.

The horizontal loads will need to be taken by means of battered piles. In this instance, we recommend that the batter be limited to no more than 4 vertical :1 horizontal, as in practice, greater batter is difficult to install.

In addition to the spread footing foundation option, steel H-pile foundation is a good alternative for the North Abutment, provided the piles will not be too short (i.e. at least 4 to 5 m beneath the bottom of the pile cap). Consideration may need to be given to the fact that some of the piles may encounter refusal before reaching the surface of the bedrock.

5.1.2 SOUTH ABUTMENT

At the South Abutment location of the Detour Bridge the thickness of the overburden is somewhat thicker in comparison with the findings of boreholes at the north abutment location and the bedrock elevations are slightly lower (between elevations 214 m and 216 m). The overburden mainly consists of wet, very loose to compact sandy to silty cohesionless materials with thickness up to 8 to 10.5 m. For this reason, the use of normal spread footing foundations may be uneconomical and consideration may need to be given to driven steel H-piles and to caissons socketed in the bedrock.

5.1.2.1 DRIVEN H-PILES

The results of investigation showed that with the prevailing subsurface conditions the use of a low displacement pile, such as a steel H-pile with a heavy section (e.g. HP 310 x 110), would be feasible to support the South Abutment. The use of steel H-pile for the Centre Pier is not considered feasible because of the insufficient thickness of overburden.

The piles would preferably be driven to the surface of the bedrock where uniformly high resistances can be utilized. At some locations the driven piles could penetrate through the very dense glacial till deposit and found on the underlying bedrock. Experience in the general area, however, shows that in many cases the surface of the bedrock can frequently be uneven and unpredictable. It is therefore likely that some of the piles may not reach the surface of the bedrock and may terminate on boulders above the bedrock surface.

In order to adequately penetrate the very dense glacial till with frequent cobbles and boulders, and to seat into bedrock, a heavier section such as HP 310 x 110 equipped with rock points (Titus Standard H-Bearing Pile Point or equivalent) should be used.

Based on the results of the boreholes, the following Table 5.1.2.1.1 summarizes the estimated average pile tip elevations that may be assumed for design purposes.

Table 5.1.2.1.1

Borehole	Existing Ground/Water Surface Elevation (m)	Approx. Water Depth (m)	Estimated Approximate Depth of Pile Tip Level below Existing Ground/Water Surface (m)	Estimated Approximate Elevation of Pile Tip Level below Existing Ground Surface (m)	Founding Stratum
DSA1	225.3	---	9.9*	215.4*	Inferred Possible Bedrock
DSA2	225.3	---	9.8	215.5	Bedrock
DSA3	225.3	---	8.5*	216.8*	Inferred Possible Bedrock
DSA4	225.3	---	10.4*	214.9*	Inferred Possible Bedrock
DSA5	225.2	---	10.7	214.5	Bedrock
DSA6	222.1	2.1	8.0 (measured from water surface)	214.1	Bedrock

Note: *Inferred possible bedrock or boulders.

The above estimated pile tip elevations are based on the assumption that the piles would penetrate through the very dense glacial till, cobbles or boulders within the overburden. However, some piles could reach refusal on boulders before reaching the bedrock.

For HP 310 x 110 steel H-piles driven to practical refusal within the very dense cohesionless deposit or bedrock at or below the elevations shown in Table 5.1.2.1.1 above, the following axial resistances may be assumed for design.

- Factored Axial Resistance at Ultimate Limit States = 1,500 kN
- Geotechnical Resistance at Serviceability Limit States = 1,000 kN

The above values were selected in view of the fact that some premature pile refusals may be encountered at elevations higher than those shown in Table 5.1.2.1.1, and the piles will likely be rather short.

If Hiley Formula is used, the estimated ultimate resistance of the piles driven to practical refusal at the elevations quoted in Table 5.1.2.1.1 is approximately 3,000 kN. The piles should be driven with a suitably heavy hammer capable of delivering a rated capacity of at least 50 kJ per blow. The energy should, however, be restricted to not more than 65 kJ per blow.

Cobbles and / or boulders were inferred or encountered within the subsoils in some boreholes. In view of the presence of occasional cobbles or boulders in the overburden, and a possible bouldery layer immediately above the surface of the bedrock, and the anticipated

hard driving conditions, as mentioned before, the piles should be equipped with rock points (Titus Standard H-Bearing Pile Point or equivalent).

Oversize materials (e.g. greater than 75 millimetre nominal diameter) should not be used in the fills through which piles would be driven.

In accordance with Ministry of Transportation of Ontario's standard practice, the piles should be driven to about 2.0 metres to 3.0 metres above the design elevations as given in Table 5.1.2.1.1 and the driving should then be monitored and controlled by the Hiley Formula in accordance with MTO Standards SS103-10 and SS103-11. The Contractor should be aware that a Quality Verification Engineer should be retained to provide piling inspections and a certificate of conformance.

During the driving process, piles which have already been driven should be monitored to determine if they are heaving due to effects of driving adjacent piles. If this phenomenon occurs, the affected piles should be re-driven. It is recommended that not less than 10% of the piles and at least two piles in each foundation support element be re-struck no sooner than 24 hours after initial installation, as a precaution against relaxation, as per SP903S01. If relaxation occurs, then all piles in that foundation element should be re-tapped.

It is possible that, in particular reference to the geology of Northern Ontario, due to the undulations in the surface of the bedrock and the variations in the overburden soils, some of the piles may penetrate to several metres below the estimated tip elevations given above. We recommend that this aspect be taken into consideration when ordering and preparing piles.

The geotechnical resistance at Serviceability Limit States is dependent on the settlement of the pile group and, therefore, is governed by the size of the pile group. The pile group configuration is currently not available to us. Provided that the piles are designed and installed as recommended above, it is considered that the quoted Serviceability Limit States value corresponds to no more than 25 millimetres of settlement for the pile group.

For frost protection, all pile caps should have a permanent earth cover of at least 2.2 m, or equivalent artificial insulation.

5.1.2.2 CAISSON FOUNDATIONS

Augered and cast-in-place concrete foundations (drilled caissons) can be considered. Caisson foundations socketed at least 0.5 m into the gneiss bedrock can be designed using a factored Bearing Resistance at ULS of 8000 kPa and SLS will not govern. Deeper socket depths may be considered for augmenting horizontal caisson resistances. Commonly used caisson sizes (i.e. between 0.76 and 2.1 m in diameter) can be selected to utilize this resistance value. However, we recommend the use of smaller diameter caissons (i.e.

between 0.76 m and 1.2 m) for practical purposes in view of possible difficulties that may be experienced during installation. The minimum caisson diameter should be 0.76 m to enable inspection and clean-out of the base, if necessary, during the installation.

As was mentioned in Section 5.1.1.2 of this report, difficulties may be encountered during the installation of the caissons due to the presence of water bearing granular overburden. As well, cobbles and boulders may be present in the glacial till and in the zone immediately above the surface of the bedrock. This can be discussed with a specialist contractor, particularly with regard to cost effectiveness of locally available caisson drilling equipment with relation to caisson diameter.

5.1.2.3 SPREAD FOOTING FOUNDATIONS

Based on the findings of the boreholes, feasible highest bearing surface depths and elevations for spread footing foundations at each borehole location are tabulated below.

Table 5.1.2.3.1

Foundation Location	Reference Borehole	Existing Ground/Water Surface Elevation (m)	Approx. Water Depth (m)	Recommended Highest Footing Base (Bottom) Level Below Existing Ground/Water Surface* (m)	Recommended Highest Footing Base (Bottom) Elevation (m)	Subgrade Material
West side	DSA1	225.3	---	9.3 9.9	216.0 215.4	Glacial till Inferred possible bedrock
West side	DSA2	225.3	---	9.3 9.8	216.0 215.5	Glacial till Gneiss bedrock
West side/centre	DSA3	225.3	---	8.5 10.2	216.8 215.1	Inferred possible boulder Interpolated possible bedrock
West side/centre	DSA4	225.3	---	9.5 10.4	215.8 214.9	Glacial till Inferred possible bedrock
East side	DSA5	225.2	---	10.7	214.5	Gneiss bedrock
East side	DSA6	222.1	2.1	8.0	214.1	Gneiss bedrock

It should be noted that in between and beyond the borehole locations, the bedrock surface and the depth to the surface of the competent till may vary considerably.

As was mentioned before, since it is undesirable to place the footing partially on bedrock and partially on overburden, it is recommended that the footing be extended to the surface of the gneiss bedrock.

For footings placed on gneiss bedrock a factored Bearing Resistance at ULS of 5000 kPa can be used while SLS will not govern on the surface of the relatively sound rock.

As can be seen from the above table, deep excavation (8 to 11 m deep) will be required, and considerable portion of which will be below the groundwater table. For this reason, spread footing foundations are unlikely to present an economical solution.

The use of compacted granular fill can be considered. This will however require the removal of soils to the surface of the bedrock or close to it. Because of this and the fact that compaction of granular engineered fill below the ground surface will be very difficult, supporting the footing on granular engineered fill is unlikely to be a suitable solution.

In view of these, more feasible options for the South Abutment foundations appear to be the use of caissons or driven piles.

5.1.3 CENTRE PIER

The results of Boreholes DCP1 through DCP6 which were put down from a raft in the river show below 7.4 to 8.3 m of water, the presence of between 2.8 and 4.1 m thick overburden, underlain by gneiss bedrock. Zones of cobbles and boulders were found in the generally fine grained granular overburden soils. These conditions lead us to believe that the construction will be very difficult. Normal spread footings supported in the glacial till or on the underlying bedrock will necessitate deep excavations below the surface of the water in the river, and expensive dewatering. Normal caissons socketed in the bedrock will likely be costly as equipment will need to be supported on a barge and as drilling through cobbles and boulders as well as socketing into the bedrock will require special equipment, which may not be locally available at reasonable prices. With this background the following are a discussion of available options.

5.1.3.1 SPREAD FOOTING FOUNDATIONS

In view of the high water depth of the river and shallow overburden materials, the costs of dewatering, sheet piling enclosure or cofferdam works for the construction of spread footing foundations will be prohibitively high and therefore this option will unlikely be economical.

Based on the findings of the boreholes, feasible highest bearing surface depths and elevations for spread footing foundations at each borehole location are tabulated below.

Table 5.1.3.1.1

Foundation Location	Reference Borehole	Existing Approx Water Surface Elevation (m)	Approx. Water Depth (m)	Recommended Highest Footing Base (Bottom) Level Below Existing Ground/Water Surface* (m)	Recommended Highest Footing Base (Bottom) Elevation (m)	Subgrade Material
West side	DCP1	222.0	7.4	10.2 (2.8)	211.8	Gneiss Bedrock
West side	DCP2	222.1	7.6	10.5 (2.9)	211.6	Gneiss Bedrock
West side	DCP3	222.1	7.5	10.6 (3.1)	211.5	Gneiss Bedrock
East side	DCP4	222.1	7.9	10.3 (2.4) 12.0 (4.1)	211.8 210.1	Glacial Till Gneiss Bedrock
East side	DCP5	222.1	8.3	10.4 (2.1) 11.9 (3.6)	211.7 210.2	Glacial Till Gneiss Bedrock
East side	DCP6	222.1	7.8	10.4 (2.6) 11.7 (3.9)	211.7 210.4	Glacial Till Gneiss Bedrock

***Note: values in brackets are depths measured from the river bed.**

It should be noted that in between and beyond the borehole locations, the bedrock surface and the depth to the surface of the competent till may vary considerably.

For design purposes the following bearing resistance (with reference to the founding elevations as shown in the above Table 5.1.3.1.1) may be used:

Table 5.1.3.1.2

Soil / Rock Type	Factored Bearing Resistance at ULS (kPa)	Bearing Resistance at SLS (kPa)
Glacial Till/Boulders	800	450
Gneiss Bedrock	5000	---

The factored bearing resistance at ULS given in the above table (Table 5.1.3.1.2) incorporated a resistance factor of 0.5 as per Canadian Highway Bridge Design Code

(CHBDC), CAN/CSA-S6-00. The SLS will not govern the design for footings founded on bedrock.

The actual founding elevation should be chosen with due consideration for scour depths, as well as dewatering requirements.

In the overburden, the serviceability condition is based on the premise that the maximum total and differential settlements will not exceed 25 mm and 20 mm, respectively. To achieve this, the founding subgrade must be properly dewatered. Otherwise, it may be disturbed and dilate, leading to excessive settlements when structural loads are applied.

As can be seen from the above table, deep excavation below water table will be encountered. As the footings should be constructed in the dry conditions, an extensive temporary shoring system using sheetpile and/or cofferdam with dewatering will be required.

If the footings are founded on a layer of cobbles and boulders, they must be properly grouted below and at least 2 m beyond the perimeter of the foundation to create a stable mass. If tremie base will be applied for sealing the cofferdam, such grouting may not be required.

For these reasons, we recommend that the footing be extended to the surface of the bedrock and that the use of spread footing foundations may not be a good choice.

5.1.3.2 CAISSON FOUNDATIONS

As was discussed in the previous sections, caissons socketed at least 0.5 m into the relatively sound gneiss bedrock can be designed using a Factored Bearing Resistance at ULS of 8000 kPa. Bearing Resistance at SLS need not be considered. Deeper sockets may be considered for lateral stability, including ice impact.

The above quoted design value applies to commonly utilized caisson diameter sizes in Ontario (i.e. typically between 0.76 and 2.1 m in diameter). We recommend, however, the use of smaller diameter caissons (i.e. between 0.76 and 1.2 m) for practical purposes. In addition, the minimum caisson size should be 0.76 m to enable the inspection of the base of the caisson, although this may be difficult within the river.

In general there appears to be a zone of cobbles and boulders immediately at ground surface at the bottom of the river, as well as intermediate zones, including immediately above the bedrock surface. As well the presence of cobbles and boulders can be expected in the glacial till deposit. As mentioned before with these conditions, standard caisson foundations are unlikely to be economical.

5.1.3.3 DRIVEN H-PILE FOUNDATIONS

The use of driven piles is considered to be a poor choice due to the fact that overburden is relatively shallow at all borehole locations (between 2.8 and 4.1 m), as well zones of cobbles and boulders were encountered at or near the ground surface at the River's bottom.

5.1.3.4 HEAVY WALLED STEEL TUBING WITH GROUTED ANCHOR

As normal diameters (i.e. 0.76 m and larger) caisson construction will be very expensive, a small diameter caisson-like approach with a permanent steel casing can be considered. The use of a small diameter or 0.30 m (12 inches) to 0.45 m (18 inches) heavy-walled permanent steel tube is one such approach which has been successfully used in bridge construction over water courses, in similar circumstances. This method reduces the difficulty and cost associated with advancing the casing through cobbles and boulders as well as socketing into the bedrock.

The following is a brief explanation of this method.

- If necessary, grout the surface or near surface zone of cobbles and boulders to achieve a stable base condition.
- Drive the heavy gage steel tube (0.3 m to 0.45 m diameter).
- Clean out the overburden soils inside the casing using a down-hole hammer to break apart and chop the boulders and cobbles to manageable sizes for removal.
- Advance into the bedrock in a similar fashion
- Using a separate drill rig, install a steel anchor from inside the permanent casing into the bedrock for uplift resistance as well as filling with concrete/grout inside the permanent steel casing.
- Install as many such piles as necessary, all connected with a pile cap for external stability. Bracing below the pile cap level and/or between the free lengths of the piles below the pile cap levels can be considered, if required.

It is recommended that the permanent steel tube be inserted in the sound bedrock by at least 0.4 m. However, the actual depth of bedrock socketing will depend on other considerations, such as structural adequacy requirements. As well, deeper penetration may be required for axial resistance. In this case assuming that the steel rim area at the tip of the casing is in full contact with the penetrated rock, the steel pile can be treated as a steel tube pile resting on bedrock, for design purposes (i.e. to determine axial resistance). The capacity of the pile will therefore depend largely on the steel wall thickness and the diameter (or contact area between the steel pipe and rock) of the pile (steel tube), as well as depth of embedment. But

for preliminary estimation purposes, an axial resistance at ULS of 100 MPa of steel area can be assumed. For example, with this approach a 324 mm (12 inches) diameter steel tube pile with a wall thickness of 12.7 mm (0.5 inch) will have a steel area of 12410 mm² and will thus provide an axial resistance of about 1240 kN/pile. Axial resistance at SLS need not be considered.

The structural strength and structural slenderness stability of the piles should be checked by an experienced Structural Engineer.

5.1.4 GENERAL COMMENTS

The factored bearing resistance at ULS incorporates a resistance factor of 0.5 as per Canadian Highway Bridge Design Code (CHBDC), CAN/CSA-S6-00. The SLS will not govern the design for footings founded on bedrock.

The serviceability condition is based on the premise that the maximum total and differential settlements will not exceed 25 mm and 20 mm, respectively. This can be achieved provided that the founding subgrade is undisturbed during the construction.

The actual founding elevation should also be chosen with due consideration for frost and scour depths, as well as dewatering requirements.

It is recommended that same type of foundation be used for each support element (abutments and piers), and the foundations (footings or piles) be founded on same type of soil deposit or bedrock.

As can be seen from the prior discussions, deep excavation below water table (particularly for the South Abutment) will be encountered. As the footings should be constructed in the dry conditions, temporary shoring system using sheetpile and/or cofferdam with well point dewatering system will be required. Shoring system could be a combination of sheet piles, caisson wall, or soldier piles and lagging, rakers, struts and soil anchors. For the Centre Pier, due to the shallow depth of the overburden and the presence of boulders, the shoring/cofferdam would need to be constructed within a template frame.

Due to the nature of the subsoil materials, seepage may occur from the cohesionless layers (sandy to silty materials) and glacial till deposits, and tight interlocking sheet piling may be required. It should be noted that the presence of random cobbles and boulders in the till may render the installation of sheet piles difficult and these aspects should be taken into consideration in the design of sheet piles and the dewatering scheme.

Allowance should be made to place a 100 mm thick concrete mud mat (i.e. skim coat) in all footing excavations as soon as possible after excavation, where the bearing surface consists of overburden. All footing excavations should be inspected and approved by the geotechnical engineer prior to pouring the concrete mud mat.

Following the construction of the abutment footings, backfill should be placed to a sufficient height above the footing (i.e. at least 2.2 m) to prevent disturbance and frost penetration.

Under inclined loading conditions, the bearing resistance at U.L.S. should be reduced in accordance with the Canadian Highway Bridge Design Code (C.H.B.D.C.). For the evaluation of sliding resistance of the foundations, the ultimate angle of friction between the underside of the concrete foundation and the undisturbed bearing stratum is given below:

- Dense Glacial Till : $\phi = 30$ degrees
- Gneiss Bedrock : $\phi = 30$ degrees

Footings should have a permanent earth cover of at least 2.2 m or equivalent artificial insulation for frost protection. Scour should also be considered in selecting footing depths.

If the footing is to be supported on the bedrock, all the overburden and shattered or otherwise unsuitable rock should be removed, exposing the acceptable, clean bedrock surface. The grade may be raised to the required footing elevation using mass concrete. The operation should ensure that the surface of the rock and/or mass concrete are sufficiently clean and roughened to facilitate proper bonding between rock-concrete or between concrete-concrete.

As noted earlier, the bedrock surface may vary considerably in between and beyond borehole locations. If the footings are likely to be founded on steeply sloping bedrock (steeper than 10H : 1V), the rock surface should be cut and leveled to provide a horizontal step-like base. The grade may then be raised to the required elevations using mass concrete. In case excessive irregularities in the bedrock surface are encountered, any rock knobs should be cut/removed and localized dents/holes be cleaned and filled up with mass concrete.

Bedrock would be prone to deterioration due to the opening of existing joints or fractures in the bedrock as a result of frost action. Provided that surface water is diverted away from the footings, frost protection need not be provided for footings placed on massive, sound bedrock, although for added protection an earth cover of at least 0.3 m is recommended.

If the bedrock is not massive and water can accumulate in the joints or fractures of the rock (thus causing deterioration of the founding medium by expansion due to freezing) then there may be a requirement to provide up to full frost protection (i.e. 2.2 m). For this purpose, the proposed bearing surface should be inspected by qualified engineering personnel. If the rock is not massive, then the excavation can be extended deeper until acceptable rock is found or to the full frost protection depth of 2.2 m, whichever comes first.

Sliding resistance can be provided by penetrating into the bedrock (i.e. keying-in and utilizing passive rock resistance), utilizing the sliding resistance between the concrete and the bedrock, shear in grouted dowels and/or rock anchors. For the evaluation of the sliding resistance of the foundation the value of the ultimate angle of friction between the underside of the foundations and the clean, intact bedrock surface (or between concrete surfaces) as discussed above can be used.

If there are net uplift forces which are to be resisted by rock anchors, or for increasing sliding resistance, the factored rock/grout bond capacity at U.L.S. can be taken as 750 kPa (assuming a non-shrink grout of minimum strength of 30 MPa) and S.L.S. will not govern. The upper 0.2 m of the rock should, however, not be included in calculating the resistance

and the minimum embedment depth should be 1.5 m into sound rock. The anchors should also be checked for rock wedge pull-out assuming a 60 degree apex cone/wedge and the anchor group resistance should also be checked. The structural resistance of the rock anchor should be checked by an experienced Structural Engineer.

Potential impacts of construction on the presently existing bridge will need to be considered. For example, if driven piles are to be utilized this would be an important aspect. Depending on the distance of the existing foundation units from the pile driving location, the vibrations generated by the pile driving may need to be monitored and/or a vibration specialist may need to be consulted. An NSSP may need to be included for this purpose.

Possible effects of pile driving on slope stability may also need to be considered.

5.2 LATERAL EARTH PRESSURES

Backfill behind abutments and retaining walls should consist of non-frost susceptible, free-draining granular materials in accordance with the Ontario Ministry of Transportation Standards and the requirements of OPSD 3101.150 and OPSD 3101.200.

Free-draining backfill materials (i.e. Granular 'A' or Granular 'B') and the provision of drain pipes and weep holes, etc., should prevent hydrostatic pressure build-up. Computation of earth pressures should be in accordance with C.H.B.D.C.. For design purposes, the following parameters (unfactored) can be used.

Compacted Granular 'A' and Granular 'B' Type II

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

Unit Weight = 22 kN/m^3

Coefficient of Lateral Earth Pressure:

$K_a = 0.27$

$K_b = 0.35$

$K_o = 0.43$

$K^* = 0.45$

Compacted Granular 'B' Type I

Angle of Internal Friction, $\phi = 32^\circ$ (unfactored)

Unit Weight = 21 kN/m^3

Coefficient of Lateral Earth Pressure:

$K_a = 0.31$

$K_b = 0.41$

$K_o = 0.47$

$K^* = 0.57$

Where K_b is the 'intermediate' earth pressure coefficient for a partially restrained structure.

K^* is the earth pressure coefficient for a soil loading a fully-restrained structure, including compaction surcharge effects.

These values are based on the assumption that the backfill behind the retaining structure is free-draining and adequate drainage is provided. As well, it is assumed that the ground behind the retaining structure is level. For sloping ground behind the retaining structure, reference can be made to C.H.B.D.C. Figure C6.9.1(e).

The earth pressure coefficient adopted will depend on whether the retaining structure is restrained or movements can be allowed such that the active state of earth pressure can develop. If the abutment is restrained and does not allow lateral yielding, then at rest pressures should be used in accordance with C.H.B.D.C.. The effect of compaction should also be taken into account in the selection of the appropriate earth pressure coefficients in accordance with Section 6.9 of C.H.B.D.C..

For unrestrained wing walls (if any), the intermediate earth pressure coefficient K_0 may be adopted. In the determination of degree of wall displacement or rotation to mobilize the fully active earth pressure state, Section C6.9 of the C.H.B.D.C. Commentary can be consulted.

Vibratory equipment for use behind abutments and retaining walls should be restricted in size as per current MTO practice.

If rock fill is used for backfill, special care is required to prevent damage to the retaining structures. In such a case, a cushion of Granular 'A' material or finely-graded rock fill (e.g. less than 200 mm normal diameter) should be placed between the structure and the rock fill. This cushion should be at least 0.45 m wide and if Granular 'A' is used, proper filtering should be provided to prevent the loss of finer particles from the Granular 'A' cushion into the coarse rock fill.

When temporary shoring measures are required, the shoring system should be properly designed and executed to minimize induced settlements in the adjoining ground surface and/or structures. The method used for this purpose will depend on the details of the project as well as permissible yield movements (e.g. sensitivity of existing services to settlement). We recommend that settlement markers be placed close to the adjacent structures to monitor the movements induced due to construction. A pre-excavation condition survey of the existing surrounding structures prior to the commencement of any excavation work is recommended. If temporary shoring is to be used, the design and analysis should be carried out in accordance with the recommendations of the (CFEM). The following soil parameters could be used for the design of temporary shoring system:

- Coefficient of lateral earth pressure coefficient for temporary flexible wall (e.g. sheet pile wall) = 0.3
- Coefficient of lateral earth pressure coefficient for temporary rigid wall (e.g. caisson wall) = 0.5

5.3 APPROACH EMBANKMENTS

Based on the available information, the existing grades will be raised by less than 1 m above the existing grades. Based on this no foundation stability problems are anticipated.

All organic and other unsuitable soils should be removed within an envelope area given by an imaginary slope not steeper than 1:1 from the toe of the proposed embankment. Based on the available borehole data, for preliminary estimating purposes, the average thickness of unsuitable soils to be stripped can be assumed to be about 1 m. However, the thickness of organic or otherwise unsuitable soils can be variable, especially near watercourses. We have no geotechnical/foundation information within the existing river bed but for preliminary estimating purposes, allowance should be made to remove about 1 m of unsuitable sediments.

The groundwater table in the subsoils will likely necessitate some drainage and/or surficial dewatering during stripping, subsequent proofrolling (where practical) and fill placement. For this reason, it is our opinion that a granular fill will likely be necessary in the low-lying areas, until the fill reaches the existing ground surface level or even slightly higher, depending on the construction season and site conditions. The dewatering will likely consist of gravity drainage and pumping from strategically placed filtered sumps.

Assuming properly compacted, acceptable inorganic earth fill materials are utilized, 2 horizontal to 1 vertical side slopes can be used for the construction of the approach fills. However, local flattening will be required depending on the geometry and especially where surcharge is required, as will be discussed later. Proper erosion control measures should be implemented by seed and cover (OPSS 572) or sodding (OPSS 571).

The materials used for the construction of the embankment fills should consist of approved, acceptable earth fill. Fill used for construction of the embankments should be in accordance with OPSS 212 and fill placement should meet or exceed the requirements of OPSS 501 and OPSS 206. Construction should be in accordance with Special Provision 206S03. In general, the fills should be placed in lifts not exceeding 300 mm before compaction and each lift should be uniformly compacted to at least 95% of the material's Standard Proctor Maximum Dry Density.

Based on the findings of Borehole DAP1 on the South Approach, the anticipated foundation settlement under the stresses generated by the approximately 1 m grade raise is approximately 20 mm, while another 10 mm of settlement can occur due to settlement of the new embankment fill under its own weight, bringing the estimated total settlement along this approach to about 30 mm. The foundation settlement can be expected to be substantially completed within a period of about nine months. On the North Approach, based on the findings of Borehole DAP2, the anticipated foundation settlement under the weight of the approximately 1 m high embankment fill is about 10 mm, and another 10 mm of settlement

can occur due to settlement of the new embankment fill under its own weight, bringing the total estimated settlement to 20 mm. Based on the borehole results, the foundation settlements can be expected to be completed within a period of about two months. The settlement of the embankment under its own weight will depend on the type of soil used for construction but in general should not exceed a period of about two to three months. Such settlements are considered to be within tolerable limits; especially since the roadway will be of a temporary nature.

5.4 EXCAVATION AND GROUNDWATER CONTROL

All excavations should be carried out in accordance with the latest edition of the Ontario Occupational Health and Safety Act and its regulations (i.e. Occupational Health and Safety Act O.Reg. 213/91).

Excavation for foundations at the abutments would be extended through fill into the native silts, sands, gravel and glacial till below the groundwater table. Temporary unsupported excavation side slopes not steeper than 1 horizontal to 1 vertical (1H:1V) through the fill would be stable provided that the excavation base is above the groundwater table. Slopes forming through the surficial fill will require flatter inclinations, say 2H:1V or flatter. Pumping from properly filtered sumps will be required to control water seepage due to perched water and surface runoff. Below the groundwater table, substantial inflow into the temporary excavations through cohesionless soils should be expected, and the side slopes will be very flat, unless dewatering is effected. Sump pumping alone may not be sufficient to maintain a reasonably dry excavation to facilitate foundation construction. Local dewatering by means of filtered wells or well points may be required. The Pier will be constructed below the river level and therefore substantial dewatering would be required. At both the pier and abutment locations, shallow penetration/embedment of sheet piles in the dense till, the sheet piles will require pins or dowels at the toe (with a template) to provide temporary support until more rigid bracing can be installed.

At the abutment locations, if spread footings are to rest on overburden, allowance should be made to place an approximately 100 mm thick layer of lean concrete on the subgrade surface, i.e. excavation base, within four hours of preparation and acceptance of the bearing soil. It should be pointed out that if the foundation soil is disturbed, excessive settlements could occur after structural loads are applied. Care should also be exercised to minimize disturbance to the silty subgrade during excavation.

During the construction, temporary runoff controls such as sediment trap, interceptor drain, dike and / or silt fence should be provided and installed to prevent uncontrolled water / sediment flow down slope towards the water course. The effluent from dewatering operations should also be filtered or passed through sediment traps to prevent turbidity.

5.5 FROST PROTECTION

Design frost protection depth for the general area is 2.2 m. Therefore, a permanent soil cover of 2.2 m or its thermal equivalent of artificial insulation is required for frost protection of foundations, including pile caps. In case of rip-rap (rock fill), only one-half of the rock fill thickness should be assumed to be effective in providing frost protection.

5.6 SCOUR PROTECTION

In order to minimize erosion and scour at the bridge abutment locations, scour protection using rockfill may be required to prevent further erosion and scour of the river bank near the bridge foundations. The rockfill should be placed to at least 1 m above the high water level. A geotextile separator, or granular filter layer, will be required between rockfill and native soils, in order to prevent infiltration of fine soils into the rockfill and subsequent settlement. The geotextile separator should comprise a Class II non-woven geotextile with a Filtration Opening Size (F.O.S.) of 105 to 210 micrometres. The scour protection should extend for a distance of at least 20 m from each side of the bridge abutment. MTO Drainage Management Manual should be referred for detailed design of scour protection.

All the side slopes should be protected against erosion during construction and permanently, including rock protection placed to the high water level, as per hydrological considerations. The rock protection should be separated from the native soils or embankment material with a geotextile filter fabric or a filter zone of granular material. The filter fabric should have a filtering opening size (F.O.S.) not larger than 120 microns.

Care should be taken during construction to ensure there is no risk of undermining the bridge structure.

5.7 SEISMIC DESIGN DATA

5.7.1 SITE COEFFICIENT

The subsurface conditions encountered at the site are represented by Soil Profile Type II (see Clause 4.4.6.2 of CHBDC CAN/CSA-S6-00, Dec. 2000 Ed). For seismic design, therefore, in accordance with Clause 4.4.6.1 site coefficient, S , for the site is 1.2.

5.7.2 SEISMIC ZONE AND ZONAL ACCELERATION RATIO (A)

Table A3.1.7 of the CHBDC provides a zonal Acceleration Ratio (A) of 0.05 for Sturgeon Falls which is closest listed locality in Table A3.1.7. Based on this, a Zonal Acceleration Ratio of 0.05 can be assigned to the site along with a Velocity Related Seismic Zone (Z_v) of 1.

As site coefficient (S) is 1.2, and the zonal acceleration is 0.05, the design zonal acceleration ratio for the site can be taken as $A=0.06$.

5.7.3 SEISMIC EARTH PRESSURES

Seismic (earthquake) loading should be taken into account in the design in accordance with Section 4.6 of the CHBDC.

In accordance with Sections 4.6.4 and C.4.6.4 of the CHBDC and its Commentary, the horizontal seismic coefficient, k_h , used in the calculation of the seismic active pressure coefficient, is taken as $k_h=0.06$. The seismic active earth pressure coefficient is also dependent on the vertical component of the earthquake acceleration, k_v . Three discrete values of vertical acceleration are typically selected analysis, corresponding to $k_v = +2/3 k_h$, $k_v = 0$, and $k_v = -2/3 k_h$.

The following seismic active pressure coefficients (K_{AE}) may be used in design; these coefficients reflect the maximum K_{AE} obtained using the k_h , and three values of k_v as described above. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is flat.

Seismic Active Pressure Coefficients

Active Earth Pressure Coefficient	Granular 'A' ($\phi = 35^\circ$ - unfactored)	Granular 'B' Type II ($\phi = 32^\circ$ - unfactored)
Non-Seismic, K_a	0.27	0.31
Seismic, K_{AE}	0.28	0.32

In the calculation of K_{AE} , the effect of the friction between the wall and the soil is not considered (i.e. $\delta=0$).

5.7.4 LIQUEFACTION POTENTIAL AND SLOPE STABILITY

The proposed structures will be supported by footings or deep foundations (driven piles or caissons) founded in/on dense tills and/or bedrock. The founding soils are considered not liquefiable.

The liquefaction potential of the soils below the approach embankments under seismic loading has been considered using the empirical method outlined in Section C4.6.2 of the CHBDC Commentary, which correlates the cyclic resistance ratio of the soils with their normalized penetration resistance and fines content. Based on this assessment, and assuming a ground surface acceleration of 0.06 g, a factor of safety of greater than 1.0

against liquefaction is obtained for magnitude 7.0 earthquake events under the approach embankment.

6. CLOSURE

We recommend that during finalizing of the details of the Detour bridge, close liaison be maintained with the foundation (geotechnical) consultant to select optimum solutions regarding settlement, fill stability, surcharging, etc. issues, as well as reviewing recommendations contained in this report for their specific applicability.

The Limitations of Report, as quoted in Appendix G, are an integral part of this report.

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Appendix G

Limitations of Report

LIMITATIONS OF REPORT

This report is intended solely for the Client named. The material in it reflects our best judgment in light of the information available to Shaheen & Peaker Limited at the time of preparation. Unless otherwise agreed in writing by Shaheen & Peaker Limited, it shall not be used to express or imply warranty as to the fitness of the property for a particular purpose. No portion of this report may be used as a separate entity, it is written to be read in its entirety.

The conclusions and recommendations given in this report are based on information determined at the testhole locations. The information contained herein in no way reflects on the environment aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the testholes may differ from those encountered at the testhole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the testhole locations and should not be used for other purposes, such as grading, excavating, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report.

The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work. This work has been undertaken in accordance with normally accepted geotechnical engineering practices.

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*Project: SPT1155
Lea Consulting Limited*

*Foundation Investigation Report
Proposed Sturgeon River Detour Bridge
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