

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

GEOCRES NO. 41I-200

Prepared For:

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Prepared by:

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**Project: SPT1155
July 14, 2006**



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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, is located immediately downstream of the existing bridge, which will be replaced by the proposed Sturgeon River Replacement Bridge. The Sturgeon River Replacement Bridge will be located at the location of the presently existing bridge.

The purpose of this investigation was to reveal the subsurface conditions at the site of the proposed Sturgeon River Replacement 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 proposed permanent bridge (Sturgeon River Replacement 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 April 6, 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 replacement bridge labeled from south to north is as follows (see Drawing No. 1):

- South Approach
- South Abutment
- South Pier
- North 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 Replacement Bridge will be located in similar alignment to 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 replacement bridge was performed during the period of November 3 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	AP1	1	On land
South Abutment	SA1 to SA4	4	On land
South Pier	SP1 to SP3	3	In river
North Pier	NP1 to NP4	4	In river
North Abutment	NA1 to NA5	5	On land
North Approach	AP2	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 3.3 m and 15 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 NP1 to NP4 and SP1 to SP3), 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, a MTO Field Vane was used to conduct the tests but when the soil became stiffer this was changed to small Field Vane.

Borehole SA4 encountered auger refusal at 5.2 m depth and the split-spoon sampler could not be recovered after SPT testing. The borehole (Borehole SA4-1) was re-drilled at about 2 m south of Borehole SA4 and rock coring commenced at 5.2 m and terminated at 8.2 m depths.

In order to advance the boreholes through cobbles and boulders and to prove bedrock, rotary core drilling was carried out in Boreholes NA1, NA3, NA5, NP1 to NP4, SA1, SA3 SA4, SP1, SP2 and SP3 (13 boreholes) 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 and Atterberg Limits tests, 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 replacement bridge alignment are shown on Drawings 1, 2 and 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 BH1B, BH2B and BH13B) put down by others in September to November 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 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 ASPHALTIC CONCRETE

Boreholes AP1, NA1 to NA4 and SA1 to SA3 were drilled on the asphalt pavement of the roadway. The thickness of the asphalt concrete was found to be about 0.05 m to 0.3 m at the borehole locations.

4.2 TOPSOIL

Topsoil was encountered in boreholes NA1 and SA4 with thickness of about 0.1 m. At Borehole NA1 which was drilled from the existing pavement surface, granular fill materials were encountered overlying the topsoil with thickness of about 2.5 m. The fill materials encountered in this borehole 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.

4.3 FILL

As mentioned before in Borehole NA1, fill was encountered overlying the topsoil. Boreholes AP2, NA2 to NA5 and SA1 to SA4 encountered 0.3 m to 3.6 m thick fill materials. Typically these fill deposits consisted of sandy to silty gravel with occasional cobbles, and extended to Elevations ranging from 225.9.0 m to 223.6 m.

In Boreholes NA1, NA2, NA3, NA4, SA1, SA2 and SA3, which were drilled from the paved road surface, the fill consisted of granular pavement fill while in Borehole AP2 and NA5 which were drilled off the roadway, the fill was found to consist of sand with occasional silt and topsoil pockets and gravelly sand, respectively.

The fills are granular materials (i.e. non-cohesive) and the recorded N-values ranged from 5 to in excess of 50 blows/0.3 m, indicating very loose to very dense relative density, but generally compact. The measured natural moisture contents range from 2 to 15%.

Three grain size distribution analyses were carried out on three samples of the granular pavement fill materials with the results as follows:

Gravel	: 17 – 26%
Sand	: 71 – 79%
Silt & Clay	: 3 – 4%

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

4.4 SANDY SILT, SILTY SAND, SILT AND SAND DEPOSITS

Below the topsoil or fill materials (Boreholes AP1, AP2, NA1, NA2, NA3, NA5, NP3, NP4, SA1 to SA4, SP1 and SP3), cohesionless (i.e. fine-grained granular) deposits with occasional clay seams and organics were encountered. In the boreholes drilled from the land, these deposits were encountered at depths of about 0.5 m to 2.4 m. (Elevations 225.5 m to 223.6 m) and extended to depths of about 1.7 m to 4.4 m (El. 224.1m to El. 220.9m, with thickness ranging from 0.5 m to 2.9 m, while in Boreholes NP4, SP1 and SP3, which were drilled in the river, the deposit was encountered at river bed (El. 220.3 m to 217.5 m) and extended to depths of about 0.7 to 2.0m below the ground level in the River's bottom or to El. 218.3 m to 216.5 m, with thickness ranging from 0.7 m to 2.0 m. In Borehole NP3, which was also drilled in the river, the deposit was encountered at a depth of 3.3m below the water surface in the river and underlying a 0.7m thick layer of cobbles and boulders at El. 218.6m. These gravelly fine grained granular (i.e. non-cohesive) deposits consisted of either sandy silt, silty sand, silty fine sand, sand or silt, or the combination of two or more of these soil types as shown in the Record of Borehole Sheets. In Borehole AP1, 0.6 m thick silty clay was found interlayered with this deposit. Borehole AP1 terminated in these deposit to depths of 3.3 m (El. 222.5 m) upon encountering auger/spoon refusal.

Measured N-values within this deposit range from 3 blow per 0.3 m to 28 blows per 0.3 m indicating a very loose to compact relative density, but in general, a loose to compact 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:

Sand

Natural Moisture Content: 23%
Grain Size (1 sample)
Gravel: 4%
Sand: 93%
Silt & Clay: 3%

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

Fine Sand

Natural Moisture Content: 23%
Grain Size (1 sample)
Gravel: 4%
Sand: 87%
Silt: 5%
Clay: 4%

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

Silty Fine Sand

Natural Moisture Content: 14%
Grain Size (1 sample)
Gravel: 2%
Sand: 69%
Silt: 23%
Clay: 6%

The grain size curve for this material is provided on Figure B4.

Sandy Silt

Natural Moisture Content: 22%
Grain Size (1 sample)
Sand: 45%
Silt: 48%
Clay: 7%

The grain size curve for this material is provided on Figure B5.

4.5 SILTY CLAY

Boreholes AP1, AP2 (drilled at the approaches), SA1 and SA3 (drilled at the south abutment location) encountered a silty clay layer at depths between 1.7 m to 2.1 m below the ground surface to depths of 2.3 m to 3.9 m (0.6 m to 1.8 m thick) below existing ground surface (El. 223.5 m to El. 222.2 m). In Borehole SA3, the presence of cobbles and boulders was noted at the bottom of this cohesive silty clay layer.

Atterberg Limits tests were performed on 3 soil samples in the laboratory and these gave the following index values, as shown in Figure B7.

Liquid Limit: 26 – 37%
Plastic Limit: 15 – 20%
Plasticity Index: 9 – 17

These values are typical of clays of low to medium plasticity (i.e. CL to CI).

The measured natural moisture contents ranged from 29 to 34% and are closer to the measured liquid limit rather than the plastic limits. The calculated Liquidity Index values are between 0.8 and 1.6.

Standard Penetration tests conducted in this silty clay deposit gave N-values of 5 to 26 blows/0.3 m. In Borehole AP1 and AP2 where an N-value of 26 blows/ 0.3m was recorded,

the consistency of the material is described as stiff to very stiff, based on the recorded N-value and other laboratory test result and a visual and tactile examination soil sample. In Borehole SA1 and SA3, N-values of between 5 and 9 blow / 0.3 m were recorded, and undrained shear strength as measured by Field Vane tests were 16 and 21 kPa, indicating a soft to stiff consistency.

4.6 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 SP3, SA1 and SA3. The composition of this basically granular (i.e. non-cohesive) glacial deposit varies from silty sand till to relatively finer silt till. This glacial deposit was encountered at depths ranging from 0.7m (Borehole NP1 and SP2) to 5.2m (Borehole NP4) below the ground surface or between elevations ranging from 223.5m (Borehole AP1) and 214.7m (Borehole SP1). Frequent cobbles and boulders were encountered or inferred at variable depths within the till. Boreholes AP1, AP2, NA2, NA4 and SA2 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 0.3 m (Borehole SP1) to 7.6 m (Borehole NP3). Measured SPT N-values within the glacial till varied from 12 to over 50 blows, indicating that the till is in a compact to very dense relative density.

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

Natural Moisture Content: 5 to 17%
Grain Size (8 samples)
Gravel: 1 – 11%
Sand: 29 – 58%
Silt: 29 – 52%
Clay: 6 – 12%

The grain size curves for this material are provided in an envelope form in Figure B6.

The presence of cobbles and boulders should always be anticipated in the overburden soils, especially in the glacial tills due to the mode of deposition.

4.7 ZONE OF COBBLES AND BOULDERS

Underlying the glacial till and silty clay deposits in Boreholes SA1 and NA3, a basal zone of very coarse granular soil consisting of cobbles and boulders with sand and gravel infill was contacted at 3.2m and 4.8m depth (El. 222.8 m and 220.5 m) respectively, and extended to depths of 3.4 m to 6.0 m (El. 222.6 m and 219.3 m). Thickness of this stratum in these two

boreholes is 0.2m and 1.2m, respectively. In addition some of the boreholes encountered refusal (e.g. Borehole AP1 and AP2, NA2, NA4, SA2) at depth ranging between 3.3 and 6.6 m (EL. 222.5 - 219.4m) possibly on boulder or bedrock.

Borehole NP1 through NP4 and SP1 through SP3, which were drilled in the River, encountered a refusal zone of cobbles and boulders (with some sand and gravel infill) either immediately at the surface of the River bed (Boreholes NP1, NP2, NP3 and SP2) or a short distance below it, underlying a 0.7 to 2.0m thick sand layer (Boreholes NP4, SP1 and SP3). The thickness of this coarse grained granular surficial soil zone at the borehole locations ranged from 0.7m (Boreholes NP1, NP3 and SP2) to 3.2m (Borehole NP4). This upper zone of cobbles and boulders has measured SPT N-values ranging from 25 to over 50 blows per 0.3 m penetration, indicating a compact to very dense relative density, but typically dense. 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.

4.8 BEDROCK

Bedrock was cored and proven in Boreholes NA1, NA3, NA5, NP1 TO NP4, SA1, SA2, SA4, SP1 to SP3. The cored lengths ranged between 0.5 m to 3.2 m. Bedrock or boulders level were inferred by auger/spoon refusal in the other Boreholes. 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)
AP1	225.8	---	---	3.3	222.5
AP2	226.1	---	---	4.1	222.0
NA1	226.0	5.3	220.7	---	---
NA2	226.0	---	---	4.8	221.2
NA3	225.3	6.0	219.3	---	---
NA4	226.0	---	---	6.6	219.4
NA5	226.0	5.2	220.8	---	---
NP1	221.9	8.2	213.7	---	---
NP2	221.8	8.5	213.3	---	---
NP3	221.9	12.2	209.7	---	---
NP4	221.8	8.6	213.2	---	---
SA1	226.0	3.4	222.6	---	---
SA2	226.0	---	---	4.0	222.0
SA3	226.1	3.9	222.2	---	---
SA4	225.7	5.2	220.5	---	---
SP1	222.1	7.7	214.4	---	---
SP2	222.1	11.1	211.0	---	---
SP3	222.0	6.5	215.5	---	---
BH1B	226.2	---	---	4.0	222.2
BH2B	225.9	3.9	222.0	---	---
BH13B	222.1	2.0	220.1	---	---

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 87% to generally 100%. The Rock Quality Designation (RQD) values increase with depth from 33% up 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. Point Load Index Tests on 4 samples from Boreholes NP3 and SP2 recorded corrected $Is(50)$ values of 0.76 to 3.9 MPa. The estimated compressive strength of the rock cores tested ranged from 18 MPa to 94 MPa.

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

4.9 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 observations in these types 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. 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.

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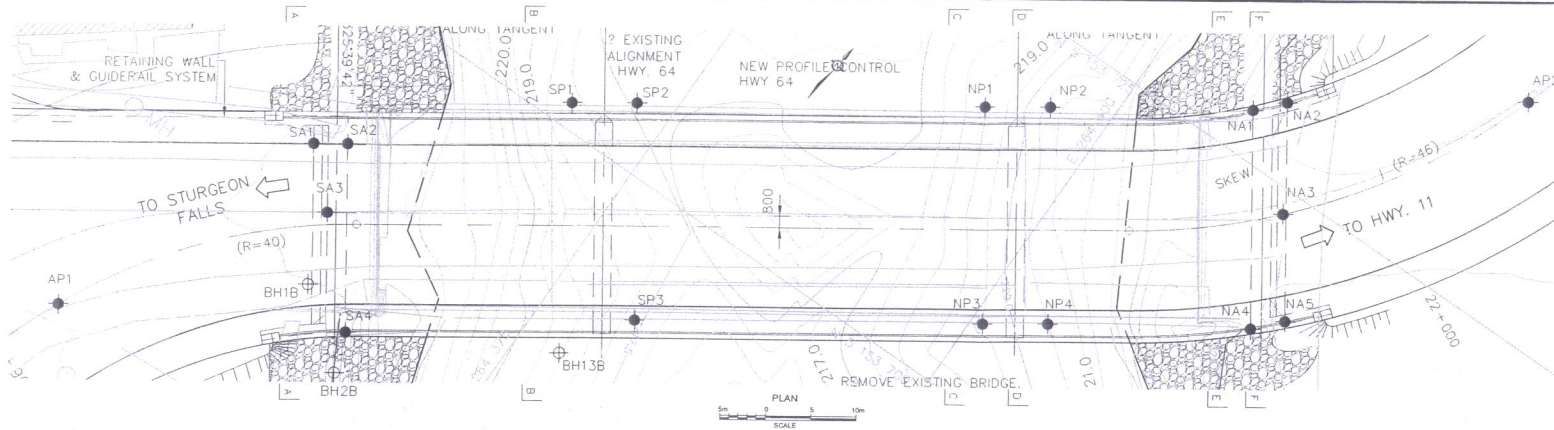
Zuhtu Ozden, P.Eng.



K. R. Peaker, Ph.D., P.Eng.

ZO:tr/hd

Drawings



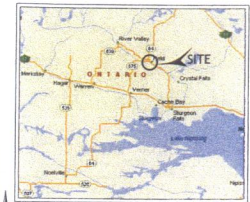
METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
ARE IN KILOMETRES + METRES.

CONT No.
WP No 211-93-01

STURGEON RIVER BRIDGE (NEW)
BORE HOLE LOCATIONS & SOIL STRATA

SHAHEEN & PEAKER LIMITED



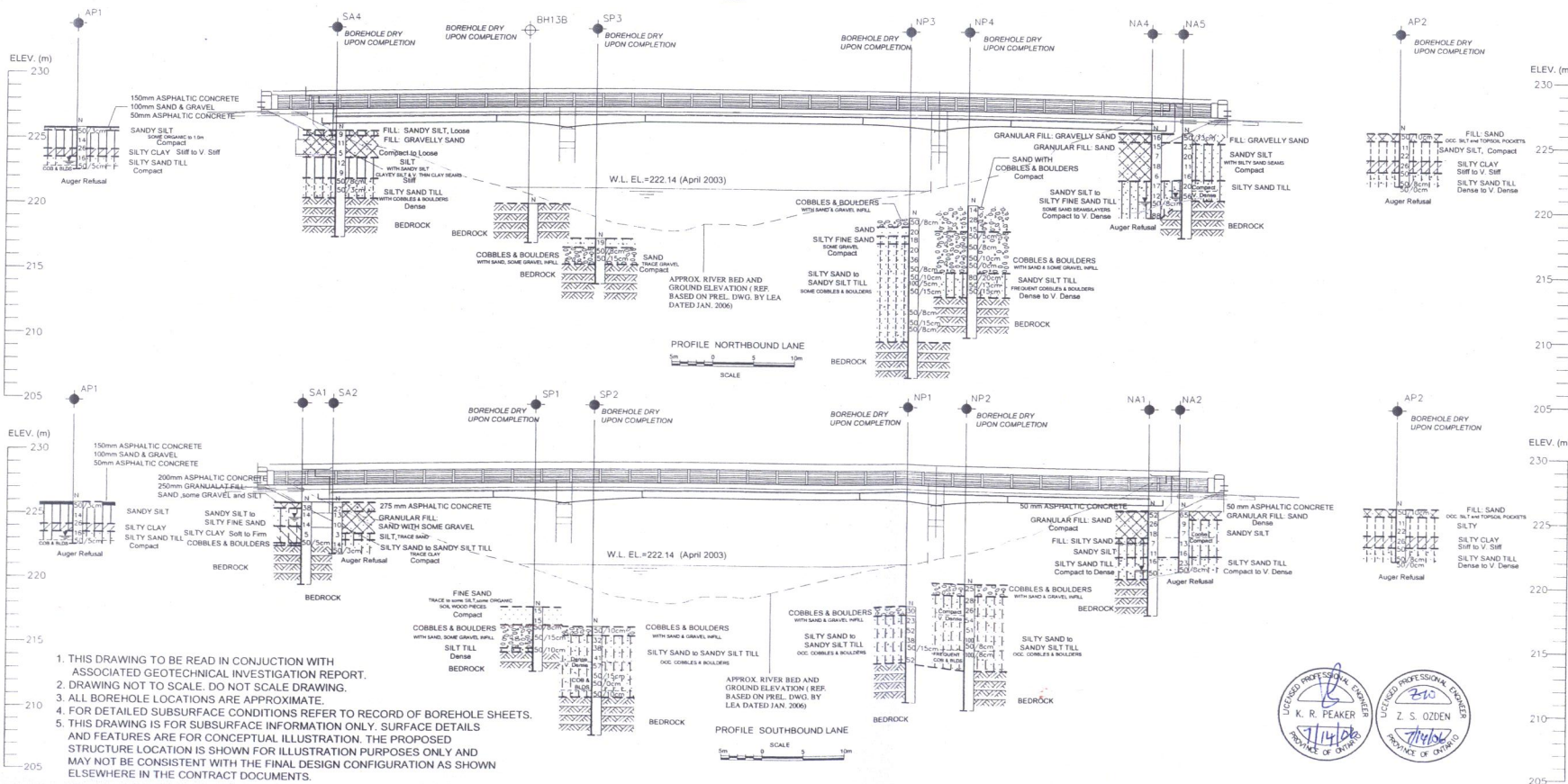
LEGEND

- Approx. Borehole Location (THIS INVESTIGATION)
- ⊕ Approx. Borehole Location (by OTHERS, 2003)
- N Blows/0.3m (Std. Pen. Test, 475 Jblow)
- ⬇ Recorded Water Level at Time of Investigation Nov., 2005

BH No.	ELEV.	CO-ORDINATES	
		NORTH	EAST
SA4	225.7	5 153 679.8	264 365.0
SP3	186.2	5 153 691.8	264 381.0
NP4	221.9	5 153 705.4	264 400.9
NA4	226.0	5 153 715.6	264 415.2
NA5	226.0	5 153 717.4	264 417.9
AP1	225.8	5 153 669.7	264 347.6
AP2	226.1	5 153 739.4	264 430.0
BH13B	222.1	5 153 687.0	264 378.0
BH1B	226.2	5 153 681.0	264 361.0
BH2B	225.9	5 153 677.0	264 366.0
SA1	226.0	5 153 689.2	264 356.7
SA2	226.0	5 153 690.5	264 357.7
SP1	187.3	5 153 701.7	264 368.8
SP2	182.5	5 153 704.2	264 372.5
NP1	221.9	5 153 717.8	264 392.3
NP2	221.8	5 153 720.3	264 395.1
NA1	226.0	5 153 728.1	264 407.7
NA2	226.0	5 153 729.9	264 409.3

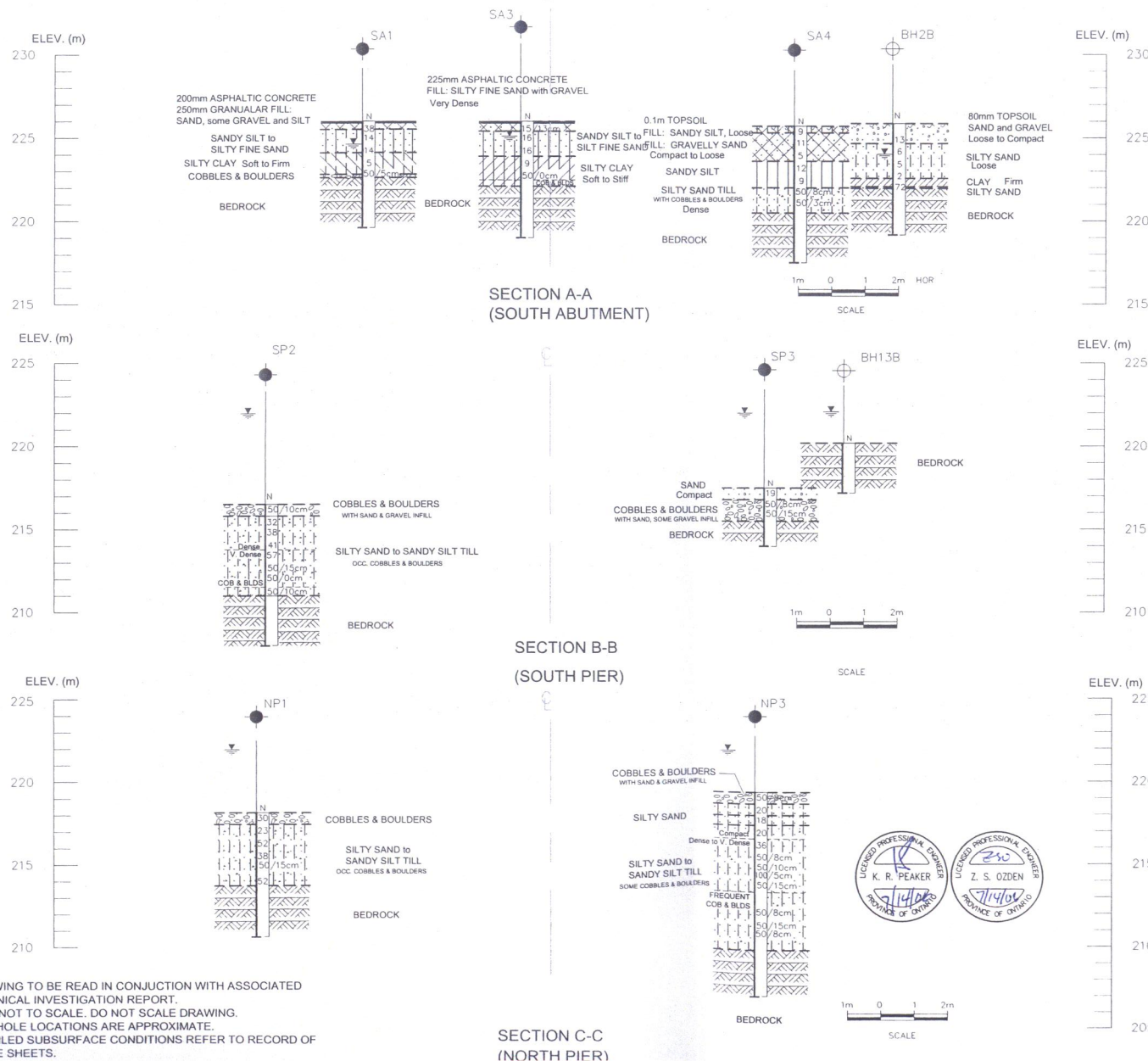
NOTE
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

DATE	BY	DESCRIPTION
Geocres No. 411 - 200		
HWY No. 64		DIST 54
SUBMITD 20	CHECKED KSH	DATE Feb. 2006 SITE 43-019



- THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ASSOCIATED GEOTECHNICAL INVESTIGATION REPORT.
- DRAWING NOT TO SCALE. DO NOT SCALE DRAWING.
- ALL BOREHOLE LOCATIONS ARE APPROXIMATE.
- FOR DETAILED SUBSURFACE CONDITIONS REFER TO RECORD OF BOREHOLE SHEETS.
- THIS DRAWING IS FOR CONCEPTUAL ILLUSTRATION. THE PROPOSED STRUCTURE LOCATION IS SHOWN FOR ILLUSTRATION PURPOSES ONLY AND MAY NOT BE CONSISTENT WITH THE FINAL DESIGN CONFIGURATION AS SHOWN ELSEWHERE IN THE CONTRACT DOCUMENTS.





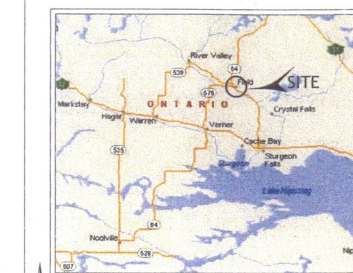
CONT No.

WP No 211-93-01

STURGEON RIVER BRIDGE (NEW)
BORE HOLE CROSS SECTIONS
& SOIL STRATA



SHAHEEN & PEAKER LIMITED



KEY PLAN
N.T.S.

LEGEND

- Approx. Borehole Location (THIS INVESTIGATION)
- Approx. Borehole Location (by OTHERS, 2003)
- N Blows/0.3m (Std. Pen. Test, 475 J/blow)
- Recorded Water Level at Time of Investigation Nov., 2005

BH No.	ELEV.	CO-ORDINATES	
		NORTH	EAST
SA1	226.0	5 153 689.2	264 355.7
SA3	226.1	5 153 685.8	264 359.2
SA4	225.7	5 153 679.8	264 365.0
BH2B	225.9	5 153 677.0	264 366.0
BH13B	222.1	5 153 687.0	264 378.0
SP2	182.5	5 153 704.2	264 372.5
SP3	186.2	5 153 691.8	264 381.0
NP1	221.9	5 153 717.8	264 392.3
NP3	221.9	5 153 705.4	264 400.8

NOTE

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

REV	DATE	BY	DESCRIPTION

Geocres No. 411-200

HWY No. 64

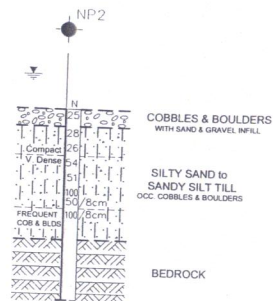
DIST 54

SUBMIT 20

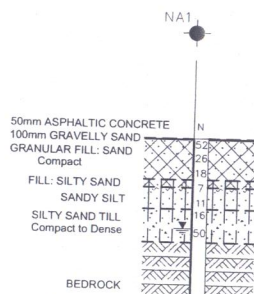
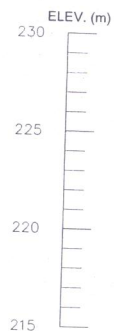
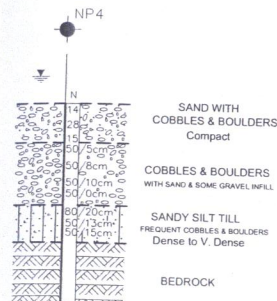
CHECKED KSH

DATE Feb 2006

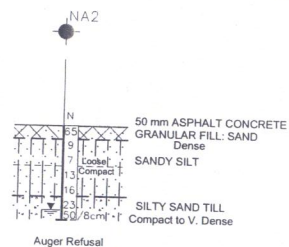
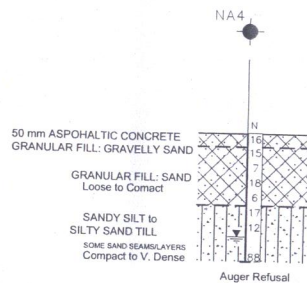
SITE 43-019



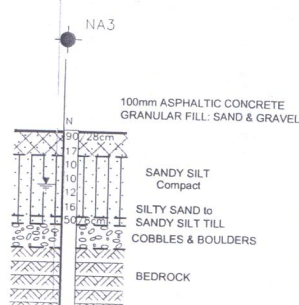
SECTION D-D
(NORTH PIER)



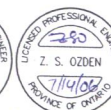
SECTION E-E
(NORTH ABUTMENT)



SECTION F-F
(NORTH ABUTMENT)



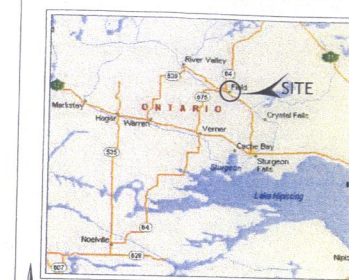
1. THIS DRAWING TO BE READ IN CONJUNCTION WITH ASSOCIATED GEOTECHNICAL INVESTIGATION REPORT.
2. DRAWING NOT TO SCALE. DO NOT SCALE DRAWING.
3. ALL BOREHOLE LOCATIONS ARE APPROXIMATE.
4. FOR DETAILED SUBSURFACE CONDITIONS REFER TO RECORD OF BOREHOLE SHEETS.
5. FOR SECTION LOCATIONS REFER TO DRAWING 1.



CONT No.
WP No 211-93-01

STURGEON RIVER BRIDGE (NEW)
BORE HOLE CROSS SECTIONS
& SOIL STRATA

SHAHEEN & PEAKER LIMITED



KEY PLAN
N.T.S

LEGEND

- Approx. Borehole Location (THIS INVESTIGATION)
- ⊕ Approx. Borehole Location (by OTHERS, 2003)
- N Blows/0.3m (Std. Pen. Test, 475 J/blow)
- ☼ Recorded Water Level at Time of Investigation Nov., 2005

BH No.	ELEV.	CO-ORDINATES	
		NORTH	EAST
NA1	226.0	5 153 728.1	264 407.7
NA2	226.0	5 153 729.9	264 409.3
NA3	226.1	5 153 723.4	264 413.5
NA4	226.0	5 153 715.6	264 416.2
NA5	226.0	5 153 717.4	264 417.9
NP2	221.8	5 153 720.3	264 396.1
NP4	221.8	5 153 707.9	264 404.5

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

REV.	DATE	BY	DESCRIPTION

Geocres No. 411-200

HWY No. 64		DIST 54
SUBMD ZO	CHECKED KSH	DATE Feb. 2006
DRAWN JZ	CHECKED	APPROVED

SITE 43-019
DWG 3

Appendix A

Record of Borehole Sheets (Present Investigation)

SPT 1155

RECORD OF BOREHOLE No AP 1

1 OF 1

METRIC

GWP 211-93-00 LOCATION Highway 64, Sturgeon River Bridge, ON Coords: N 5153669.7; E 264347.6
 DIST 54 HWY 64 BOREHOLE TYPE Solid Stem Augers
 DATUM Geodetic DATE 11/18/2005
 ORIGINATED BY G.I.
 COMPILED BY J.Z.
 CHECKED BY Z.O.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT LIMIT			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		W P W W L				
225.8	Ground Surface													
225.5	150 mm Asphaltic Concrete		1	SS	60/3									
0.3	100 mm Sand & Gravel		2	SS	14		225							
	50 mm Asphaltic Concrete		3	SS	26		224							
	SANDY SILT dark grey and some organics to 1.0 m brown below, compact, moist		4	SS	16		223							
224.1	SILTY CLAY brown, stiff to v. stiff													
1.7														
223.5														
2.3	SILTY SAND TILL brown, compact, wet cobbles and boulders													
222.5	End of Borehole. Auger refusal at 3.3 m.													
3.3														
	*Water level at 2.8 m (not stabilized) and hole open to 2.9 m on completion.													

SPT 1155

RECORD OF BOREHOLE No AP 2

1 OF 1

METRIC

GWP 211-93-00 LOCATION Highway 64, Sturgeon River Bridge, ON Coords: N 5153739.4; E 264430.0 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		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI C
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				
226.1	Ground Surface		1	SS	50/10		226							
0.0	FILL: SAND, occasional silt and topsoil pockets, brown, wet													
225.5														
0.6	SANDY SILT yellowish brown, compact		2	SS	11		225							
224.0			3	SS	22		224							
2.1	SILTY CLAY brown, stiff to v. stiff		4	SS	26									
223.1														
3.0	SILTY SAND TILL dense to very dense, wet		5	SS	50		223							
222.0			6	SS	50/8									
4.1	End of Borehole. Auger refusal at 4.1 m. *Borehole dry(not stabilized) and open to 3.9 m on completion.						222							

SPT 1155

RECORD OF BOREHOLE No NA 1

1 OF 1

METRIC

GWP 211-93-00 LOCATION Highway 64, Sturgeon River Bridge, ON Coords: N 5153728.1; E 264407.7 ORIGINATED BY J.Z.
DIST 54 HWY 64 BOREHOLE TYPE Solid Stem Augers & NQ Rock 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			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE						PLASTIC LIMIT w _p NATURAL MOISTURE CONTENT w LIQUID LIMIT w _L				
226.0	Ground Surface						226	20	40	60	80	100						
0.0	50 mm Asphaltic Concrete 100 mm Gravelly Sand		1	SS	52													
	GRANULAR FILL: SAND with trace to some gravel brown, compact, damp		2	SS	26		225										17	79 (4)
			3	SS	18		224											
223.9	FILL: Silty Sand trace gravel, brown, loose		4	SS	7		223											
223.5	100 mm TOPSOIL																	
2.5	SANDY SILT greyish brown, loose to compact, moist		5	SS	11		222											
222.4			6	SS	16		221										6	58 30 6
3.6	SILTY SAND TILL grey, wet		7	SS	50		220											RQD=90%
		compact ----- dense	8	NQ			219											RQD=89%
220.7		cobbles & boulders		RC														
5.3	GNEISS BEDROCK grey, fine to medium grained, slightly weathered, moderately closely to closely jointed, occasional micaceous layers		9	NQ RC	Rec. 100%		218											
			10	NQ RC	Rec. 100%													
217.9																		
8.2	End of Borehole. Auger refusal at 5.1 m, switch to casing & rock coring. Water used to facilitate coring. *Water level at 4.6 m (not stabilized) and open to 4.6 m on completion.																	

SPT 1155

RECORD OF BOREHOLE No NA 2

1 OF 1

METRIC

GWP 211-93-00 LOCATION Highway 64, Sturgeon River Bridge, ON Coords: N 5153729.9; E 264409.3 ORIGINATED BY J.Z.
 DIST 54 HWY 64 BOREHOLE TYPE Solid Stem Augers 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				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 ○ UNCONFINED + FIELD VANE ● POCKET PENETR. X LAB VANE								
							20	40	60	80	100	20	40	60		
226.0	Ground Surface															
0.0	50 mm Asphaltic Concrete		1	SS	65											
225.3	100 mm Gravelly Sand															
0.7	GRANULAR FILL: Sand, trace gravel, brown, dense, damp															
	SANDY SILT		2	SS	9											
	yellowish brown to 3 m greyish brown below															
			3	SS	7											
	loose															
	compact		4	SS	13											
	wet		5	SS	16											
222.4																
3.6	SILTY SAND TILL		6	SS	23											
	wet															
	brown, compact															
	grey, very dense		7	SS	50/8											
221.2																
4.8	End of Borehole. Split spoon and auger refusal at 4.8 m.															
	*Water level at 4.6 m (not stabilized) and 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 NA 3

1 OF 1

METRIC

GWP 211-93-00 LOCATION Highway 64, Sturgeon River Bridge, ON Coords: N 5153723.4; E 264413.5 ORIGINATED BY G.I.
DIST 54 HWY 64 BOREHOLE TYPE Solid Stem Augers & NQ Rock Coring COMPILED BY J.Z.
DATUM Geodetic DATE 11/17/2005 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. X LAB VANE					WATER CONTENT (%) W P W L		
225.3 0.0	Ground Surface														
224.0 1.3	100 mm Asphaltic Concrete		1	SS	90/28		225								
	GRANULAR FILL: 150 mm Sand & Gravel SAND with some gravel & silt brown, compact, moist		2	SS	17		224								
SANDY SILT brown, compact (possible fill)		3	SS	10	223										
		4	SS	10	222										
		5	SS	12	221										
		6	SS	16	220										
220.9 4.4	SILTY SAND to SANDY SILT TILL brown, compact to dense, wet		7	SS	50/8		219								
220.5 4.8	COBBLES & BOULDERS with sand and some gravel infill		8	NQ RC			218								
219.3 6.0	GNEISS BEDROCK with granite inclusion, pinkish grey, medium grained, slightly weathered in fractures, moderately closely to closely jointed, occasional micaceous layers		9	NQ RC	Rec. 100%		217								
216.2 9.1			10	NQ RC	Rec. 100%										
	End of Borehole. Auger refusal at 4.8 m, probably a boulder. *Water level at 3.1 m (not stabilized) and hole open to 3.1 m on completion.														

+³, ×³: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE



SPT 1155

RECORD OF BOREHOLE No NA 4

1 OF 1

METRIC

GWP 211-93-00 LOCATION Highway 64, Sturgeon River Bridge, ON Coords: N 5153715.6; E 264416.2 ORIGINATED BY J.Z.
DIST 54 HWY 64 BOREHOLE TYPE Solid Stem Augers 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			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE					PLASTIC LIMIT W _P NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L
226.0	Ground Surface						20	40	60	80	100		
0.0	50 mm Asphaltic Concrete		1	SS	16								
225.3	GRANULAR FILL: Gravelly Sand grey, compact, damp												
0.7	GRANULAR FILL: SAND with trace to some gravel grey, loose to compact, damp		2	SS	15								
			3	SS	7								20 77 (3)
			4	SS	18								
			5	SS	6								
222.3	SANDY SILT to SILTY SAND TILL some sand seams/layers brown to 4.3 m, grey below compact to very dense, wet		6	SS	17								
3.7													
			7	SS	12								
			8	SS	50/8								no recovery sample taken from augers
219.4	some clayey silt pockets		9	SS	88								
6.6	End of Borehole. Auger refusal at 6.6 m. *Water level at 5.3 m (not stabilized) and hole open to 5.3 m on completion.												

SPT 1155

RECORD OF BOREHOLE No NA 5

1 OF 1

METRIC

GWP 211-93-00 LOCATION Highway 64, Sturgeon River Bridge, ON Coords: N 5153717.4; E 264417.9 ORIGINATED BY G.I.
DIST 54 HWY 64 BOREHOLE TYPE Solid Stem Augers & NQ Rock Coring 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			20 40 60 80 100	PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	
226.0 0.0	Ground Surface		1	SS	50/13		226					
225.2 0.8	FILL: GRAVELLY SAND greyish brown, wet		2	SS	23		225					
	dark brown some organics		3	SS	20		224					
	occ. organics		4	SS	11		223					
	SANDY SILT with silty sand seams brown, compact, wet, dilatant		5	SS	16		222					
222.4 3.6	clayey silt/clay seams below 3.2 m		6	SS	20		221					
	SILTY SAND TILL brown, wet		7	SS	56		220					
220.8 5.2	compact very dense		8	NQ RC	Rec. 98%		219					
	GNEISS BEDROCK with granite inclusion, pinkish grey, fine to medium grained, slightly weathered in fractures, moderately closely to closely jointed, occasional micaceous layers		9	NQ RC	Rec. 100%		218					
217.9 8.1	End of Borehole. Auger refusal at 5.2 m, switch to casing & rock coring. Water used to facilitate coring. *Water level at 4.9 m (not stabilized) and open to 4.9 m on completion.											

+³, ×³: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

SPT 1155

RECORD OF BOREHOLE No NP 1

1 OF 1

METRIC

GWP 211-93-00 LOCATION Highway 64, Sturgeon River Bridge, ON Coords: N 5153717.8; 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 11/4/2005 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 60 80 100	20 40 60 80 100					
221.9 0.0	Water Surface													
218.1 3.8	WATER													
217.4 4.5	COBBLES & BOULDERS with sand & gravel infill		1	SS	30									
	compact dense to very dense		2	SS	23									
	SILTY SAND to SANDY SILT TILL occ. cobbles & boulders grey, wet		3	SS	52									11 47 36 6
			4	SS	38									
			5	SS	50/15									
213.7 8.2			6	SS	52									
	GNEISS BEDROCK with granite inclusion, pinkish grey, fine to medium grained, slightly weathered, moderately closely to closely jointed, occasional micaceous layers		7	RC	Rec. 100%									RQD=80%
			8	RC	Rec. 100%									RQD=92%
210.6 11.3	End of Borehole. Auger refusal at 8.2 m, switch to casing & rock coring. Wash boring used to facilitate coring. *Borehole open to 1.5 m on completion.													

+³, ×³: Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE

SPT 1155

RECORD OF BOREHOLE No NP 2

1 OF 1

METRIC

GWP 211-93-00 LOCATION Highway 64, Sturgeon River Bridge, ON Coords: N 5153720.3; 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 11/3/2005 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		
221.8 0.0	Water Surface												
	WATER												
220.0 1.8													
219.1 2.7	COBBLES & BOULDERS with sand & gravel infill		1	SS	25								
			2	RC									
			3	SS	28								
			4	SS	26								
			5	SS	54								
			6	SS	51								
			7	SS	100								
			8	SS	50/8								
			9	SS	100/8								
			10	RC									
			11	RC									
213.3 8.5	GNEISS BEDROCK with granite inclusion, pinkish grey, fine to medium grained, slightly weathered, moderately closely to closely jointed, occasional micaceous layers		12	NQ RC	Rec. 98%								
			13	NQ RC	Rec. 100%								
210.2 11.6	End of Borehole. Spoon bouncing at 7.7 m, switch to casing & rock coring. Wash boring used to facilitate coring. *Borehole open to 1.1 m on completion.												

SPT 1155

RECORD OF BOREHOLE No NP 3

1 OF 2

METRIC

GWP 211-93-00 LOCATION Highway 64, Sturgeon River Bridge, ON Coords: N 5153705.4; E 264400.8 ORIGINATED BY G.I.
DIST 54 HWY 64 BOREHOLE TYPE Solid Stem Augers & Wash Boring & Diamond Coring COMPILED BY J.Z.
DATUM Geodetic DATE 11/6/2005 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	PLASTIC LIMIT W P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W L	
221.9 0.0	Water Surface											
	WATER											
219.3 2.6	COBBLES & BOULDERS with sand & gravel infill		1	SS	50/8		221					
218.6 3.3	SILTY SAND some gravel & cobbles, compact, wet brown grey		2	SS	20		220					
			3	SS	18		219					
217.3 4.6			4	SS	20		218					
			5	SS	36		217					
	SILTY SAND to SANDY SILT TILL some cobbles & boulders grey, wet		6	SS	50/8		216					2 42 48 8
			7	SS	50/10		215					
			8	SS	100/5		214					
			9	SS	50/15		213					
			10	RC			212					
			11	SS	50/8		211					
			12	SS	50/15		210					
			13	SS	50/8		209					RQD=56% Is(50)=3.92MPa
209.7 12.2			14	NQ RC			208					RQD=73% Is(50)=2.30MPa
	gneiss bedrock with granite and veins inclusion, pinkish grey to grey, fine to medium grained, slightly weathered, moderately closely to closely jointed, occasional micaceous layers		15	NQ RC	Rec. 96%		207					
			16	NQ RC	Rec. 97%							

Continued Next Page

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

SPT 1155

RECORD OF BOREHOLE No NP 3

2 OF 2

METRIC

GWP 211-93-00 LOCATION Highway 64, Sturgeon River Bridge, ON Coords: N 5153705.4; E 264400.8 ORIGINATED BY G.I.
DIST 54 HWY 64 BOREHOLE TYPE Solid Stem Augers & Wash Boring & Diamond Coring COMPILED BY J.Z.
DATUM Geodetic DATE 11/6/2005 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 (%)					
15.0	End of Borehole. Auger refusal at 11.5 m, switch to casing & rock coring. Wash boring used to facilitate coring. *Borehole open to 3.7 m on completion.													

+³, ×³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

SPT 1155

RECORD OF BOREHOLE No NP 4

1 OF 1

METRIC

GWP 211-93-00 LOCATION Highway 64, Sturgeon River Bridge, ON Coords: N 5153707.9; E 264404.5 ORIGINATED BY J.Z.
DIST 54 HWY 64 BOREHOLE TYPE Solid Stem Augers & Wash Boring & Diamond Coring COMPILED BY J.Z.
DATUM Geodetic DATE 11/5/2005 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	
221.8 0.0	Water Surface											
220.3 1.5	SAND with cobbles & boulders some gravel & silt grey, compact, wet		1	SS	14		221					
			2	SS	28		220					
218.3 3.5	COBBLES & BOULDERS with sand and some gravel infill		3	SS	15		219					
			4	SS	50/5		218					
			5	SS	50/8		217					
			6	RC			216					
			7	SS	50/10		215					
			8	SS	50/10		214					
215.1 6.7	SANDY SILT TILL frequent cobbles & boulders grey, dense to very dense, wet		9	SS	80/20		213					
			10	SS	50/13		212					
			11	SS	50/15		211					
213.2 8.6	GNEISS BEDROCK with granite inclusion, pinkish grey, fine to medium grained, slightly weathered, moderately closely to closely jointed, occasional micaceous layers		12	RC								
			13	NQ RC	Rec. 100%							
			14	NQ RC	Rec. 97%							
210.1 11.7	End of Borehole. Auger refusal at 6.6 m, switch to casing & rock coring. Wash boring used to facilitate coring. *Borehole open to 2.8 m on completion.											

+³, ×³: Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE

SPT 1155

RECORD OF BOREHOLE No SA 1

1 OF 1

METRIC

GWP 211-93-00 LOCATION Highway 64, Sturgeon River Bridge, ON Coords: N 5153689.2; E 264355.7 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		PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40					
226.0	Ground Surface													
0.0	200 mm Asphaltic Concrete		1	SS	38									
225.5	250 mm GRANULAR FILL: sand, some gravel and silt		2	SS	14									
0.5	SANDY SILT to SILTY FINE SAND dark brown with organics to 0.9 m, brown below, compact, moist to 1.0 m, wet below		3	SS	14									
224.1	SILTY CLAY brown, soft to firm		4	SS	5									
1.9			5	SS	50/5									
222.8	COBBLES & BOULDERS		6	NQ RC	Rec. 100%									hit a stone
3.2			7	NQ RC	Rec. 100%									RQD=85%
222.6	GNEISS BEDROCK with granite inclusion, pinkish grey, fine to medium grained, slightly weathered, moderately closely to closely jointed, occasional micaceous layers		8	NQ RC	Rec. 89%									RQD=90%
3.4														RQD=78%
219.6	End of Borehole. Auger refusal at 3.4 m, switch to casing & rock coring. Water used to facilitate coring. *Water level at 1.5 m (not stabilized) and hole open to 5.5 m on completion.													

+³, ×³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

SPT 1155

RECORD OF BOREHOLE No SA 2

1 OF 1

METRIC

GWP 211-93-00 LOCATION Highway 64, Sturgeon River Bridge, ON Coords: N 5153690.5 E 264357.7 ORIGINATED BY G.I.
DIST 54 HWY 64 BOREHOLE TYPE Solid Stem Augers COMPILED BY J.Z.
DATUM Geodetic DATE 11/18/2005 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		
226.0	Ground Surface						226							
0.0	275 mm Asphaltic Concrete													
225.3	GRANULAR FILL: Gravelly Sand		1	SS	27									
0.7	greyish brown, compact, moist													
	GRANULAR FILL:		2	SS	13		225							
	sand with some gravel													
	brown, compact, moist		3	SS	10		224							
223.6														
2.4	SILT, trace sand (possible fill)		4	SS	3		223							
223.1	brown, very loose, wet													
2.9	SILTY SAND to SANDY SILT TILL		5	SS	14									
	trace clay, greyish brown													
	compact, wet, dilatant		6	SS	56/3		222							
222.0	cobbles & boulders													
4.0	End of Borehole. Auger refusal at 4.0 m. Possible bedrock.													
	*Water level at 3.2 m (not stabilized) and hole open to 3.2 m on completion.													

+³, ×³: Numbers refer to
Sensitivity

20
15 5
10
(%) STRAIN AT FAILURE

SPT 1155

RECORD OF BOREHOLE No SA 3

1 OF 1

METRIC

GWP 211-93-00 LOCATION Highway 64, Sturgeon River Bridge, ON Coords: N 5153685.8 E 264359.2 ORIGINATED BY G.I.
DIST 54 HWY 64 BOREHOLE TYPE Solid Stem Augers & NQ Rock Coring COMPILED BY J.Z.
DATUM Geodetic DATE 11/17/2005 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					
								20 40 60 80 100		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			
226.1	Ground Surface												
0.0	225 mm Asphaltic Concrete												
225.5	FILL: SILTY FINE SAND with GRAVEL greyish brown, very dense		1	SS	50/13								
0.6	dark brown, somewhat organic		2	SS	16								
	yellowish brown												
	SANDY SILT to SILTY FINE SAND compact, moist		3	SS	16								
224.0													
2.1	SILTY CLAY brown, soft to stiff		4	SS	6								
			5	SS	9								
222.2	cobbles & boulders below 3.6 m		6	SS	50/0								
3.9													
	GNEISS BEDROCK with granite inclusion, pinkish grey, fine to medium grained, slightly weathered, moderately closely to closely jointed, occasional micaceous layers		7	NQ RC	Rec. 100%								
			8	NQ RC	Rec. 100%								
219.1	End of Borehole.												
7.0	Auger refusal at 3.9 m, switch to casing & rock coring. Wash boring used to facilitate coring. *Water level at 0.9 m (not stabilized) and hole open to 2.4 m on completion.												

SPT 1155

RECORD OF BOREHOLE No SA 4

1 OF 1

METRIC

GWP 211-93-00 LOCATION Highway 64, Sturgeon River Bridge, ON Coords: N 5 153 679.8; E 264 365.0 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				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			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● POCKET PENETR. x LAB VANE				WATER CONTENT (%) w _p w w _L				
225.7	Ground Surface															
0.0	0.1 m TOPSOIL															
225.3	FILL: Sand Silt, brown, loose		1	SS	9											
0.4	FILL: Gravelly Sand dark brown, compact to loose damp to moist		2	SS	11											
223.6		----- silty	3	SS	5											
2.1	SANDY SILT with clayey silt & thin clay seams greyish brown, stiff, wet		4	SS	12											
			5	SS	9											
222.0																
3.7	SILTY SAND TILL with cobbles & boulders grey, very dense, wet		6	SS	50/8											
			7	SS	50/8											
220.5																
5.2	End of Borehole. Auger refusal at 5.2 m, move 2 m south to redrill, see Borehole SA 4-1 for detail. *Borehole dry (not stabilized) and hole open to 1.5 m on completion.															

+³, ×³: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE


SPT 1155

RECORD OF BOREHOLE No SA 4-1

1 OF 1

METRIC

GWP 211-93-00 LOCATION Highway 64, Sturgeon River Bridge, ON ORIGINATED BY J.Z.
 DIST 54 HWY 64 BOREHOLE TYPE Solid Stem Augers & NQ Rock Coring 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		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 60 80 100	20 40 60 80 100						20 40 60 80 100
225.7 0.0	Ground Surface														
	Augered to 5.2 m without sampling See Record of Borehole No. SA 4.														
220.5 5.2	GNEISS BEDROCK with granite inclusion, pinkish grey, fine to medium grained, slightly weathered, moderately closely to closely jointed, occasional micaceous layers		8	NQ RC	Rec. 87%									RQD=86%	
			9	NQ RC	Rec. 92%										RQD=90%
			10	NQ RC	Rec. 100%										RQD=95%
217.5 8.2	End of Borehole.														
	Casing & rock coring at 5.2 m. Water used to facilitate coring.														

SPT 1155

RECORD OF BOREHOLE No SP 1

1 OF 1

METRIC

GWP 211-93-00 LOCATION Highway 64, Sturgeon River Bridge, ON Coords: N 5153701.7; E 264368.8 ORIGINATED BY G.I.
DIST 54 HWY 64 BOREHOLE TYPE Solid Stem Augers & Wash Boring & Diamond Coring COMPILED BY J.Z.
DATUM Geodetic DATE 11/8/2005 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 60 80 100	20 40 60 80 100	20 40 60 80 100					
222.1 0.0	Water Surface						222								
							221								
							220								
							219								
217.9 4.2	FINE SAND trace to some silt, some organic soil and wood pieces dark grey, compact, wet		1	SS	15		218								4 87 5 4
			2	SS	15		217								
216.5 5.6	COBBLES & BOULDERS with sand and some gravel infill		3	SS	50/8		216								
			4	SS	50/15		215								
214.7 7.4	SILT TILL , grey, very dense		5	RC			214								
214.4 7.7	GNEISS BEDROCK with granite inclusion, pinkish grey to grey, fine to medium grained, slightly weathered, moderately closely to closely jointed, occasional micaceous layers		6	SS	50/10		213								RQD=52%
212.9 9.2	End of Borehole. Auger refusal at 3.5 m, switch to casing & rock coring. Wash boring used to facilitate coring. *Borehole dry(not stabilized) and open to 2.4 m on completion.		7	NQ RC	Rec. 100%										

+³, ×³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

SPT 1155

RECORD OF BOREHOLE No SP 2

1 OF 1

METRIC

GWP 211-93-00 LOCATION Highway 64, Sturgeon River Bridge, ON Coords: N 5153704.2; E 264372.5 ORIGINATED BY G.I.
DIST 54 HWY 64 BOREHOLE TYPE Solid Stem Augers & Wash Boring & Diamond Coring COMPILED BY J.Z.
DATUM Geodetic DATE 11/8/2005 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	60					
222.1 0.0	Water Surface														
	WATER														
216.5 5.6	COBBLES & BOULDERS with sand & gravel		1	SS	50/10										
215.8 6.3	SILTY SAND to SANDY SILT TILL occasional cobbles & boulders grey, wet		2	SS	32									1	52 39 8
			3	SS	38										
			4	SS	41										
			5	SS	57									1	40 47 12
			6	SS	50/15										
			7	SS	50/0										
			8	RC											
211.0 11.1	GNEISS BEDROCK with granite and vein inclusion, pinkish grey, fine to medium grained, slightly weathered, moderately closely to very closely jointed, weathering in fractures, occasional micaceous layers, highly weathered with some sand and rock fragments at top 1m of bedrock		9	SS	50/10										
			10	NQ RC	Rec. 100%										RQD=33% Is(50)=0.76MPa
			11	NQ RC	Rec. 100%										RQD=72% Is(50)=0.96MPa
208.0 14.1	End of Borehole. Auger refusal at 5.5 m, switch to casing & rock coring. Wash boring used to facilitate coring. Borehole open to 0.6 m on completion														

+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

SPT 1155

RECORD OF BOREHOLE No SP 3

1 OF 1

METRIC

GWP 211-93-00 LOCATION Highway 64, Sturgeon River Bridge, ON Coords: N 5153691.8; E 264381.0 ORIGINATED BY G.I.
DIST 54 HWY 64 BOREHOLE TYPE Solid Stem Augers & Wash Boring & Diamond Coring COMPILED BY J.Z.
DATUM Geodetic DATE 11/7/2005 CHECKED BY Z.O.

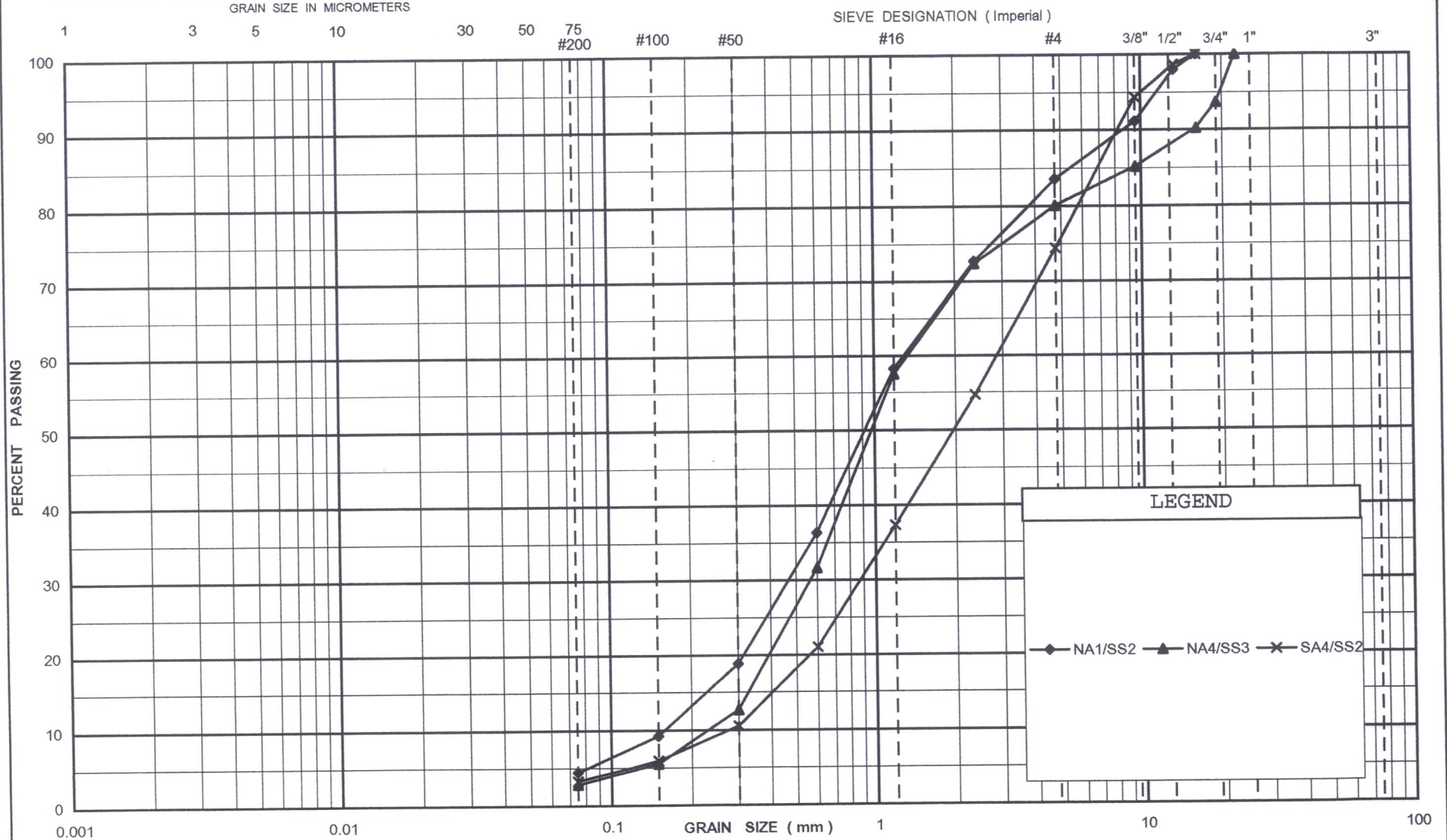
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)
								○ UNCONFINED	+ FIELD VANE	● POCKET PENETR.						
222.0 0.0	Water Surface						222	20	40	60	80	100	20	40	60	GR SA SI CL
	WATER						221									
							220									
							219									
							218									
217.5 4.5		SAND, trace gravel grey, compact, wet	○	1	SS	19		217						○		
216.8 5.2	COBBLES & BOULDERS with sand, some gravel infill	○	2	SS	50/8		216									
			3	RC												
			4	SS	50/15											
215.5 6.5	GNEISS BEDROCK with granite and vein inclusion, pinkish grey, medium to coarse grained, slightly weathered, moderately closely to very closely jointed, weathered in fractures, occasional micaceous layers		5	RC			215									
			6	NQ RC	Rec. 100%											
214.0 8.0	End of Borehole. Spoon refusal at 6.0 m, switch to casing & rock coring. Wash boring used to facilitate coring. *Borehole dry (not stabilized) and hole open to 0.3 m on completion.						214									

Appendix B

Laboratory Test Results

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse



SHAHEEN & PEAKER LIMITED

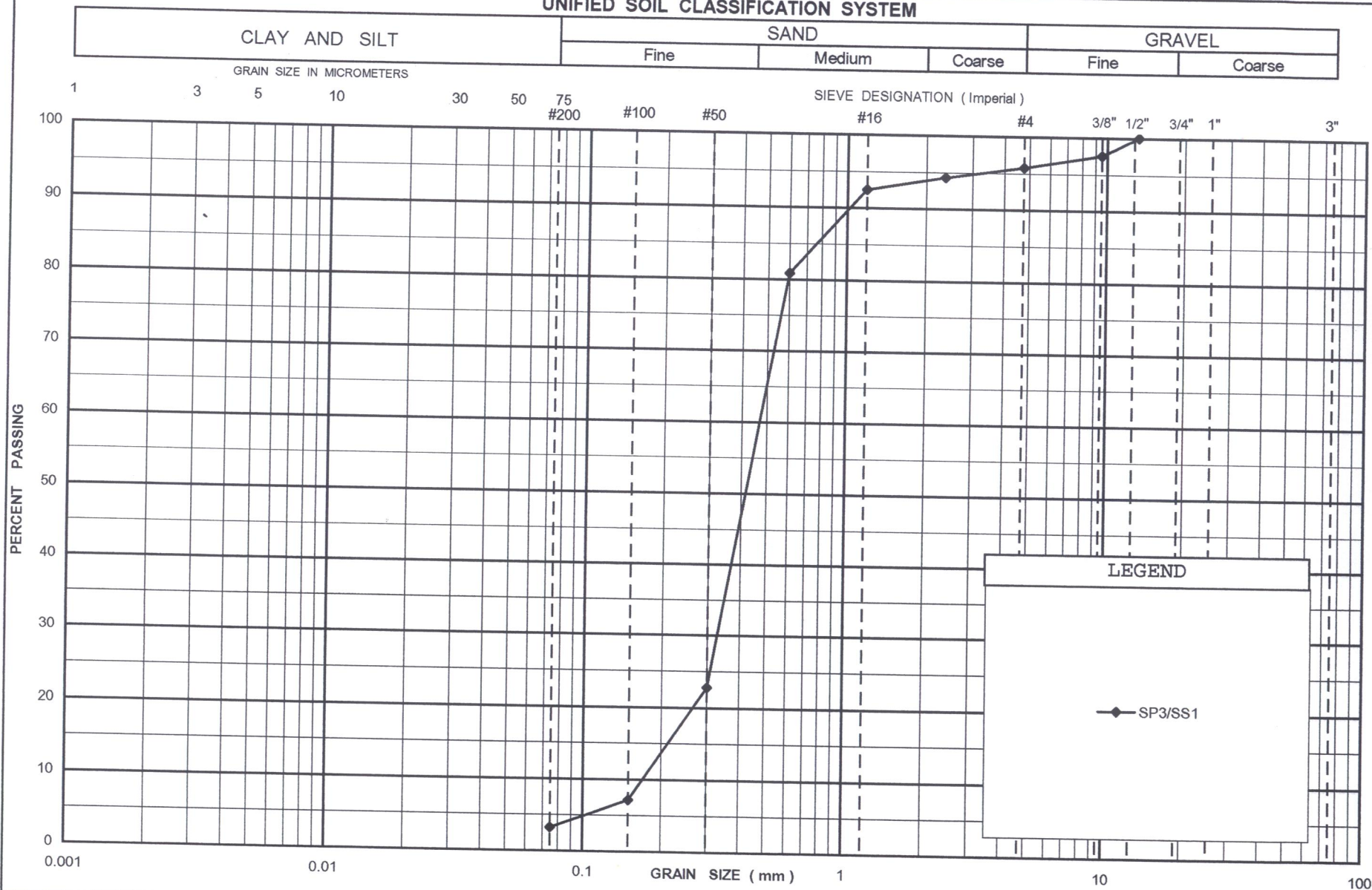
GRAIN SIZE DISTRIBUTION
FILL: Sand with some Gravel

FIGURE No. B 1

REF. No. SPT 1155

DATE JANUARY, 2006

UNIFIED SOIL CLASSIFICATION SYSTEM



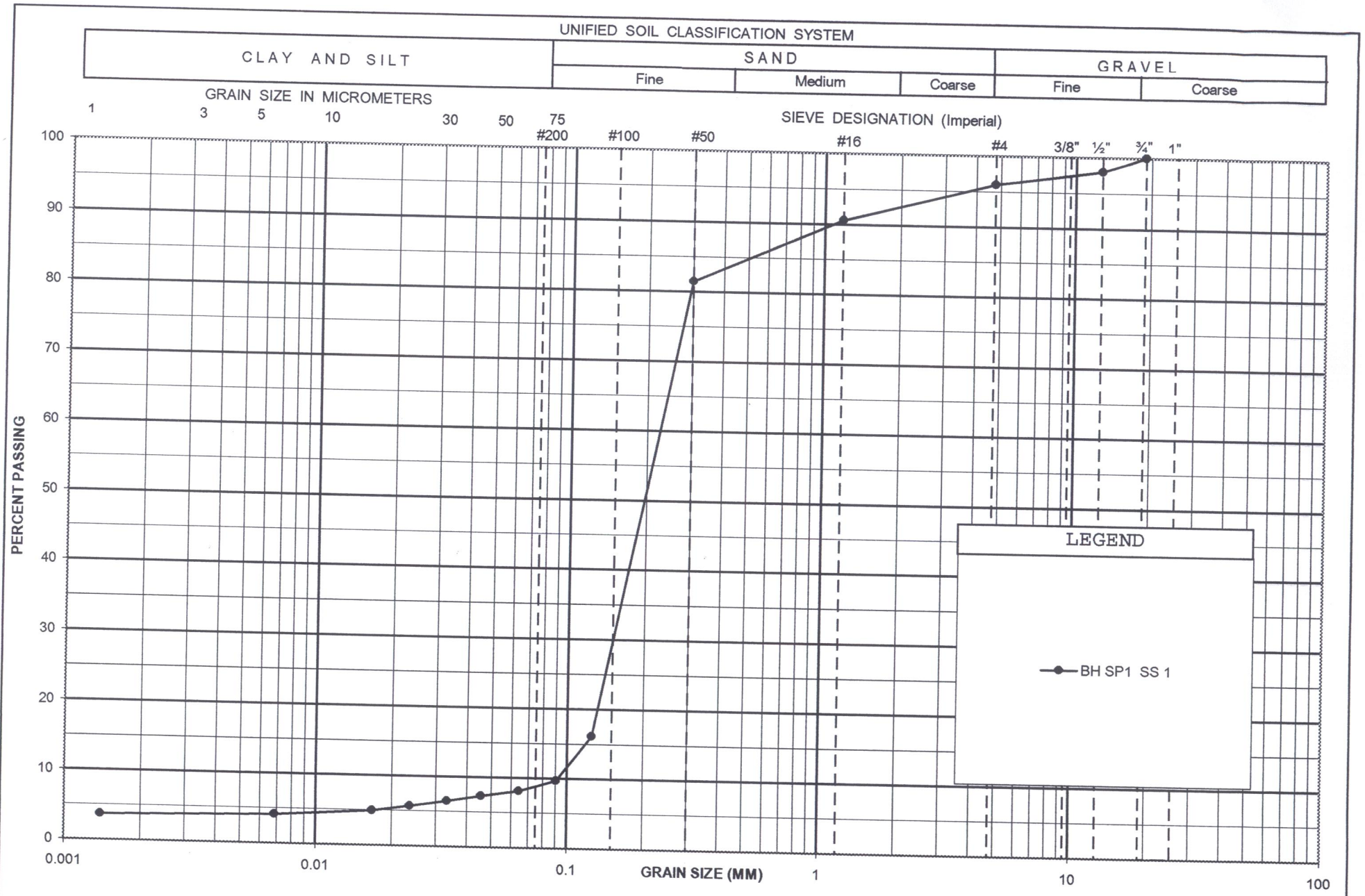
SHAHEEN & PEAKER LIMITED

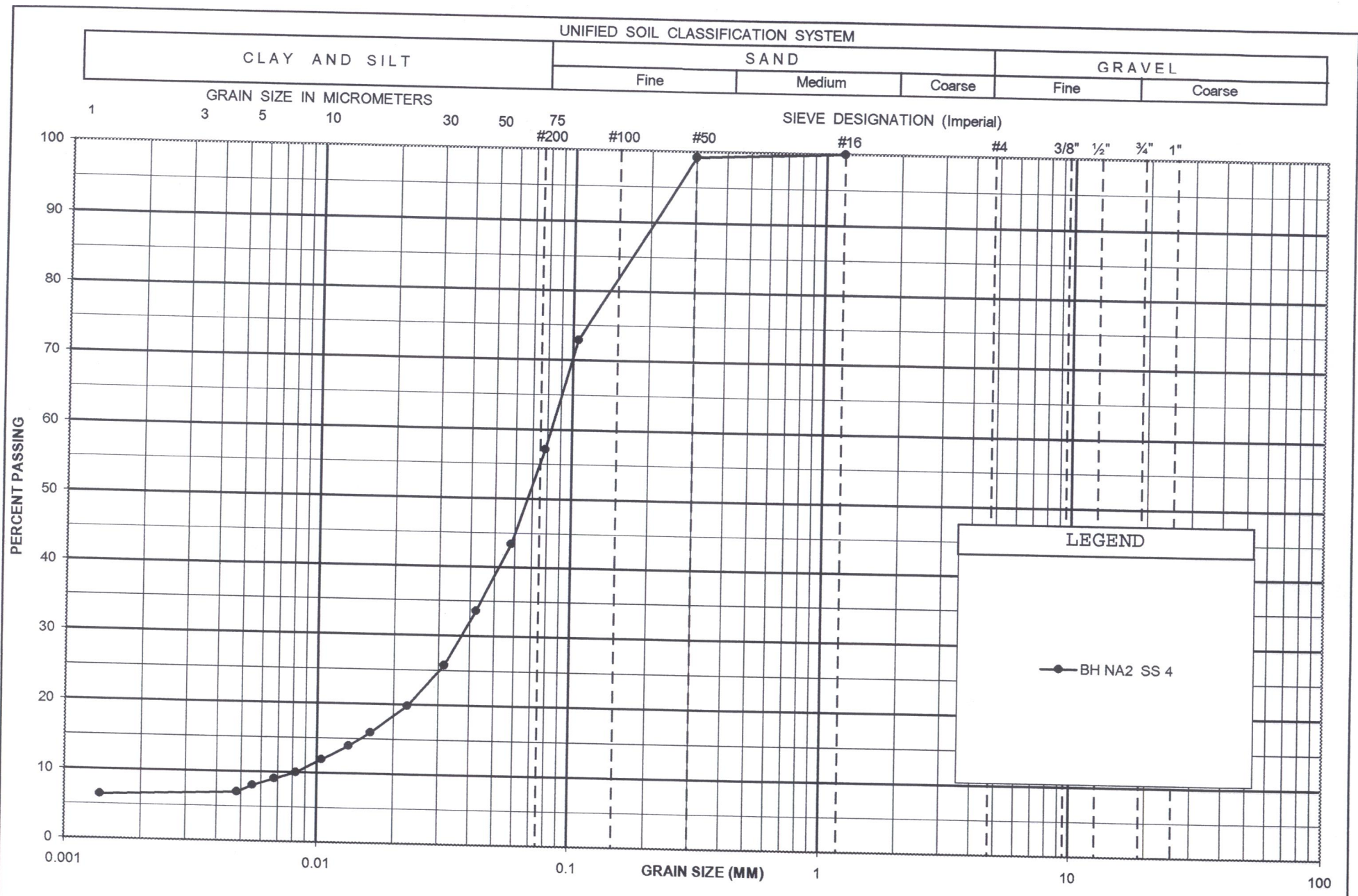
GRAIN SIZE DISTRIBUTION
SAND

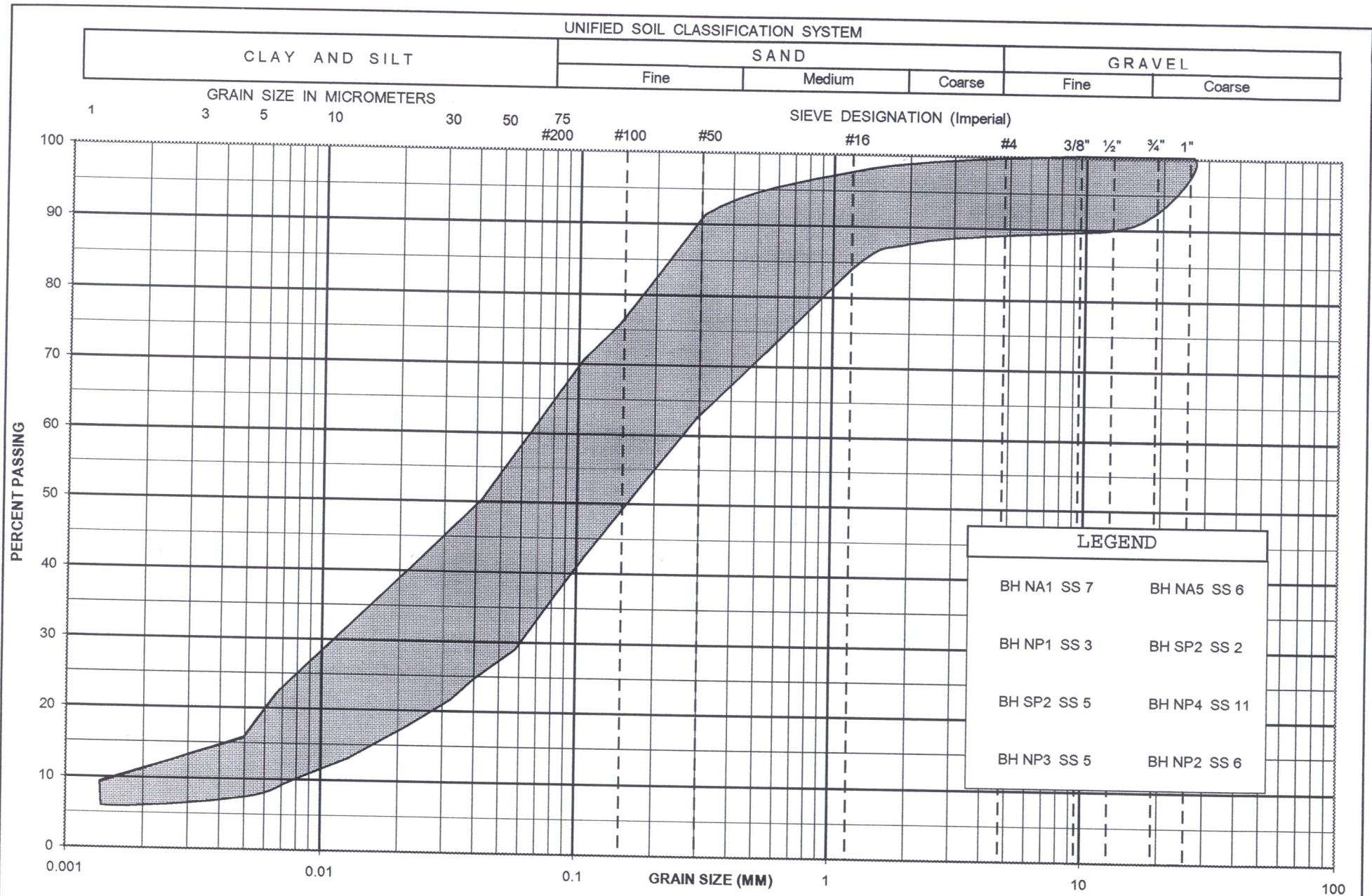
FIGURE No. B 2

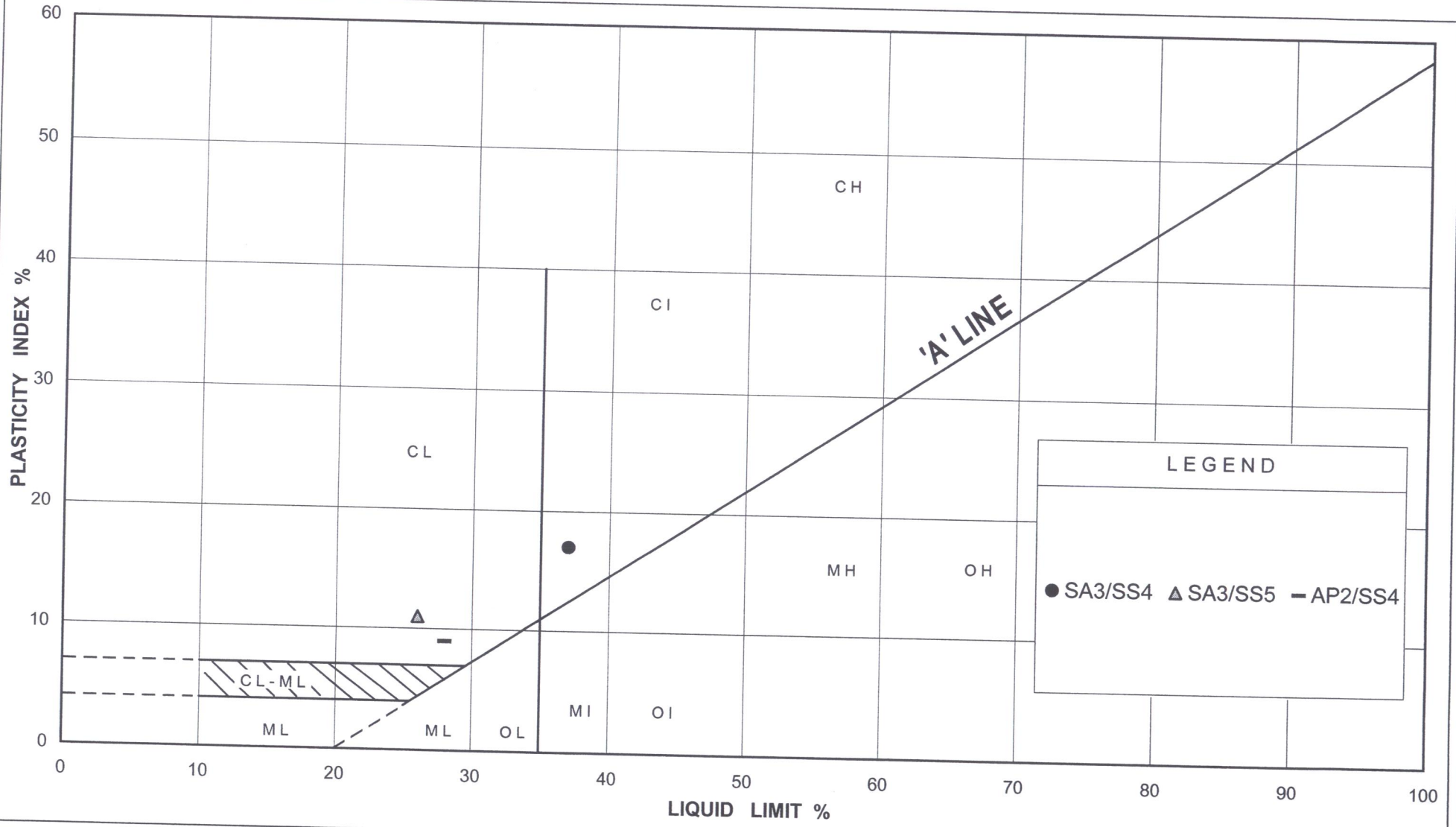
REF. No. SPT 1155

DATE JANUARY, 2006









SHAHEEN & PEAKER LIMITED

PLASTICITY CHART

FIGURE B 7

G.W.P. 211-93-00

REF No SPT 1155

Appendix C
Record of Borehole Sheets for BH-1B, BH-2B
and BH-13B
(Previous Preliminary Investigation by Others)
Geocres No. 41I-171

RECORD OF BOREHOLE BH1B

1 OF 1

METRIC

WP No. 211-93-01 LOCATION FIELD, ONTARIO, N 5153681, E 264361 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					
226.2	GROUND SURFACE														
226.1	ASPHALT - 60 mm thick														
225.9	FILL: GRANULAR 'A'														
225.9	FILL: GRANULAR 'C'														
225.4 0.8	SILTY SAND, light to dark brown, damp, loose, poorly graded, fine to coarse grained, trace gravel, some to with silt.		1	SS	5		X								
			2	SS	5		X								
223.6 2.5	- brown, wet, very loose, poorly graded, fine grained below ~2.36 m depth.		3	SS	4		X								
223.1 3.1	SILTY CLAY, brown, moist, firm to stiff, medium plasticity.		4	SS	11		X								
222.8 3.3	Pocket Pen = 75 kPa at ~2.59 m depth SAND, brown, wet, compact, poorly graded, fine to medium grained.		5	SS	75		+			X					
222.3 3.8	SANDY SILTY CLAY, brown, wet, soft, low plasticity, coarse grained sand, some medium grained gravel.														
222.2 4.0	SAND, dark brown, wet, medium to coarse grained, some fine grained gravel. AUGER REFUSAL ON BEDROCK OR BOULDER AT ~3.96 m DEPTH.														



RECORD OF BOREHOLE BH2B

1 OF 1

METRIC

WP No. 211-93-01 LOCATION FIELD, ONTARIO, N 5153677, E 264366 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			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 (%)										
								UNCONFINED QUICK TRIAXIAL		FIELD VANE LAB SHEAR		wp	w	wl								
						20	40	60	80	20	40	60	80	10	20	30	40	GR	SA	SI	CL	
225.9	GROUND SURFACE																					
225.9 0.1	TOPSOIL, ~80 mm thick																					
224.7	SAND AND GRAVEL, brown, damp, loose to compact, poorly graded, coarse grained sand, medium grained gravel, trace organics.		1	SS	13																	
224.7 1.2	SILTY SAND, brown, damp, loose, poorly graded, fine grained, with silt, trace medium grained, gravel. - wet, trace clay below ~2.21 m depth.		2	SS	6																	
			3	SS	5																	
222.6			4	SS	2																	
222.6 3.3	CLAY, brown to grey, wet, firm, medium plasticity.		5	SS	72																	
222.1			6	NQ																		
222.0 3.8	SILTY SAND, brown, wet, very dense, poorly graded, fine grained, trace to some clay.		7	NQ																		
222.0 3.9	GRANITIC GNEISS pinkish grey to greenish grey, slightly weathered, coarse grained, limited fracturing, some subvertical joints, foliation dips at ~20°, very hard.																					
219.2	END OF BOREHOLE AT ~6.71 m DEPTH.																					
219.2 6.7	ROCK CORE: At ~3.89-5.13 m depth: Rec=100%. RQD=90% At ~5.13-6.71 m depth: Rec=100%. RQD=90%																					



RECORD OF BOREHOLE BH13B

1 OF 1

METRIC

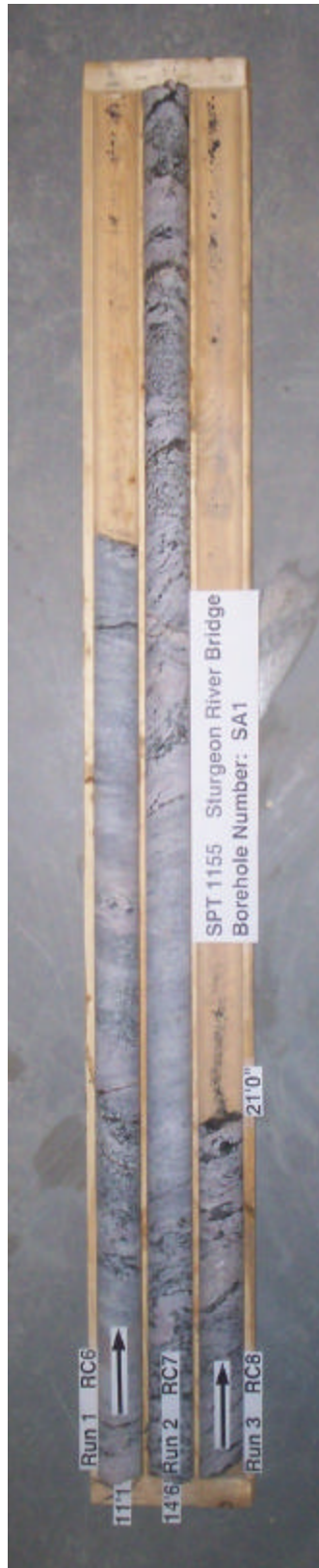
WP No. 211-93-01 LOCATION FIELD, ONTARIO, N 5153687, E 264378
 DIST Nipissing HWY 64 BOREHOLE TYPE 200 mm diameter HOLLOW STEM AUGER / CME-55 DRILL RIG
 DATUM Geodetic DATE November 05, 2003
 ORIGINATED BY S.M.
 COMPILED BY A.Q.
 CHECKED BY T.C.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) X CONE PENETRATION TEST				PLASTIC LIMIT wp	NATURAL MOISTURE CONTENT w	LIQUID LIMIT wl	UNIT WEIGHT kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20	40	60	80					
222.1 0.0	GROUND SURFACE															
220.2 2.0	WATER															
217.2 5.0	GRANITIC GNEISS pinkish to greenish grey, slightly weathered, coarse grained, limited fracturing along mica planes, very hard.		1	BQ												
			2	BQ												
			3	BQ												
	END OF BOREHOLE AT ~4.95 m DEPTH. ROCK CORE: At ~1.98-2.69 m depth: Rec=100%, RQD=66% At ~2.69-3.71 m depth: Rec=98%, RQD=90% At ~3.71-4.95 m depth: Rec=100%, RQD=91%															



Appendix D

Photographs of Rock Cores



Photograph D-1 Borehole SA1



Photograph D-2 Borehole SP2



Photograph D-3 Borehole NA3



Photograph D-4 Borehole NP3

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

MECHANICALL 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 REPLACEMENT BRIDGE
HIGHWAY 64
WEST NIPISSING, ONTARIO
G.W.P. 211-93-00
SITE 43-019**

GEOCRES NO. 41I-200

Prepared For:

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Prepared by:

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**Project: SPT1155
July 14, 2006**



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APPENDIX G LIMITATIONS OF REPORT

**FOUNDATION DESIGN REPORT
PROPOSED STURGEON RIVER REPLACEMENT BRIDGE
HIGHWAY 64
WEST NIPISSING, ONTARIO
G.W.P. 211-93-00
GEOCRES NO. 411-200**

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 Replacement Bridge, which is covered in this report, is located at location of the presently existing 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 existing bridge is 57.5 m long, with a roadway width of 9.14 m, and has 1.52 m sidewalks along each edge.

Based on the information provided to us, the proposed Sturgeon River Replacement Bridge is a two lane three-span bridge of 64 m long and 11 m in roadway width. A 1.8 m wide sidewalk with railing will be provided along each side of the bridge. The proposed elevation of the bridge deck (road surface) of the Replacement Bridge structure is approximately 227.1 m to 227.5 m, and the height of the approach embankment fill will be about 1 m above the existing grade.

The sub-surface conditions were explored at eighteen (18) boreholes (see Table in Section 3 of the foundation 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, sand and silt deposits, which are in turn underlain by compact to very dense glacial till with frequent cobbles and boulders, and followed by gneiss bedrock. The elevations of the bedrock surface were found to be variable and sloping between borehole locations. 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

level was about 222.1 m in April 2003 and the 1:50 year water level is about 225.7 m. It should be noted that the groundwater table can expected to be subject to seasonal fluctuations and in response to major weather events.

5.1 FOUNDATIONS

We understand that the existing foundations for the existing bridge will be removed prior to the construction of new foundations for the replacement bridge. Based on the results of geotechnical investigation and consideration of the rather complex subsurface conditions at the site, we have considered 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, render the use of caisson foundations at some support element locations an uneconomical solution. Auger-press concrete piles 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 or possibly in the very dense glacial till deposit. The lack of pile embedment in the overburden (minimum 4 to 5 m) will have adverse effect to the stability of the pile foundation.

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 glacial till or bedrock.	Feasible foundation option but extensive excavation and dewatering required, especially at the pier locations	Recommended at abutment locations. For the pier locations, if cofferdam will be constructed for the removal of existing foundation, footing foundation may be used.
○ Drilled concrete caissons extending into bedrock.	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.	May not be economical.
○ Driven Piles (steel H-pile, steel tube pile, pre-cast concrete pile and timber pile)	Not recommended due to presence of cobbles and boulders within the overburden which could damage the piles. Also limited by sloping bedrock surface and insufficient pile embedment depth in the overburden.	Not recommended based on reliability at most support elements. However it is possible to use steel H- piles at the north abutment location where the overburden is deeper.
○ Heavy wall steel pipe pile socketed into bedrock with grouted anchors. If needed, provide grout or tremie concrete on the surface of cobbles and boulders. The hollow sections of the steel pipe pile will be filled with concrete/grout. Refer to Section 5.1.3.4 for details.	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 concrete pile.	Recommended.

Based on the data obtained from the boreholes, the proposed Replacement 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. 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. piers). 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 the north abutment can be founded on spread footing foundations placed on the undisturbed very dense glacial till stratum at about 4.4 to 5.4m below the existing ground or on the underlying bed rock as tabulated below.

Table 5.1.1.1.1

Foundation Location	Reference BH	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	NA1	226.0	---	4.5 5.3	221.5 220.7	Glacial Till/Boulders Gneiss Bedrock
West side	NA2	226.0	---	4.5 5.0	221.5 221.0	Glacial Till/Boulders Inferred Possible Bedrock
Centre	NA3	225.3	---	4.4 6.0	220.9 219.3	Glacial Till/Boulders Gneiss Bedrock
East side	NA4	226.0	---	5.4 6.6	220.6 219.4	Glacial Till/Boulders Inferred Possible Bedrock
East side	NA5	226.0	---	4.5 5.2	221.5 220.8	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. If tremie base will be applied on the bearing surface for sealing the cofferdam placed underwater, the mud mat will not be required. 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 5.1.5, may be required to provide sufficient sliding and uplift resistance.

5.1.1.2 CAISSON FOUNDATION

Augered and cast-in-place concrete foundations (drilled caissons) can be considered. Caisson foundations socketed at least 0.5m into the gneiss bedrock can be designed using a factored Bearing Resistance at ULS of 8000 KPa. Bearing Resistance at SLS will not govern. This design value applies to commonly used caisson sizes in Ontario (i.e. between 0.76 and 2.1 m diameter). However, the use of smaller size caissons (i.e. between 0.76 and 1.2 m) is recommended. Difficulties may be encountered during the installation of the caissons due to the presence of granular overburden below water table (e.g. sandy silt deposit in Borehole NA3). As well, cobbles and boulders present in the glacial till and particularly zone immediately along the bedrock surface (e.g. Borehole NA3). 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 size should be 0.76 m to enable the inspection of the base of the caisson, 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

Foundation Location	Borehole No.	Estimated Pile Tip Refusal Depth Below Existing Ground (m)	Estimated Pile Refusal Elevation (m)	Refusal Stratum
West Side	NA1	5.2m	220.8	Cobble & boulders
West Side	NA2	4.8m	221.2	Cobble & boulders
Centre	NA3	4.8m	220.5	Cobble & boulders
East Side	NA4	6.6m	219.4	Cobble & boulders
East Side	NA5	5.2m	220.8	Gneiss & Bedrock

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 with the applied Resistance Factor of 0.5.
- Axial Resistance at S.L.S. = 1000 kN/pile

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, a greater batter is difficult to install.

The relatively low values recommended are due to anticipated short pile lengths. The minimum pile length should be at least 4 m. Another consideration with such short piles is uplift capacity. A further consideration is that driven piles will unlikely be suitable at other support elements. For these reasons, driven piles are unlikely to be a good choice. If, however, consideration is to be given to this option, we will be pleased to further discuss it.

5.1.2 SOUTH ABUTMENT

At the South Abutment location the thickness of the overburden is somewhat less in comparison with the findings of boreholes at the north abutment location. For this reason, the use of normal spread footing foundations appear to be more feasible, while short drilled caisson foundation can also be considered. Driven piles will unlikely be feasible since their lengths will be very short.

5.1.2.1 SPREAD FOOTING FOUNDATIONS

Based on the borehole data the South Abutment can be founded on spread footing foundations on the bedrock as detailed below.

Table 5.1.2.1.1

Foundation Location	Reference BH	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	SA1	226.0	---	3.4	222.6	Gneiss Bedrock
West side	SA2	226.0	---	4.0	222.0	Inferred Possible Bedrock
Centre	SA3	226.1	---	3.9	222.2	Gneiss Bedrock
East side	SA4	225.7	---	5.2	220.5	Gneiss Bedrock

For design purposes, the Factored Bearing Resistance at ULS can be taken as 5000 kPa and Bearing Resistance SLS will not govern.

Since the rock surface is uneven, mass concrete may be applied to provide a level surface.

5.1.2.2 CAISSON FOUNDATION

As discussed for the North Abutment, drilled caissons may be a solution. 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 and SLS need not be considered.

As mentioned before, there appears to be a zone of cobbles and boulders immediately above the bedrock surface and there may cause difficulties during the construction of the caissons. Cobbles and boulders can also be expected in the overburden soils, especially in the glacial tills due to their mode of deposition.

Based on our discussions with a deep foundation contractor, caisson diameters between 760 mm and 1200 mm are commonly used in Ontario.

5.1.3 NORTH PIER

The results of Boreholes NP1 through NP4 which were put down from a raft in the river show below 1.5 to 3.8 m of water, the presence of between 4.4 and 9.6 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.

In the selection of suitable foundation option, other factors such as the provision of scour protection, application of tremie base to seal the cofferdam, excavation/removal of existing foundations, should be considered.

5.1.3.1 SPREAD FOOTING FOUNDATIONS

In view of the dewatering requirements and the cost of erection of sheet pile enclosure or other cofferdam requirements, consideration may be given to supporting the foundations on the undisturbed glacial till at elevations summarized below (Table 5.1.3.1.1). In the same

table, the surface of bedrock elevations are shown, in case it is desired to extend the footings on the surface of the bedrock (although this will likely be impractical).

Table 5.1.3.1.1

Foundation Location	Reference BH	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	NP1	221.9	3.8	5.4 (1.6) 8.2 (4.4)	216.5 213.7	Glacial Till Gneiss Bedrock
West side	NP2	221.8	1.8	4.3 (2.5) 8.5 (6.7)	217.5 213.3	Glacial Till Gneiss Bedrock
East side	NP3	221.9	2.6	5.9 (3.3) 12.2 (9.6)	216.0 209.7	Glacial Till Gneiss Bedrock
East side	NP4	221.8	1.5	4.8 (3.3) 6.7 (4.2) 8.6 (7.1)	217.0 215.1 213.2	Boulders Glacial Till Gneiss Bedrock

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

For design purposes the following bearing resistance (with reference to the founding elevations as shown in the above Table 5.1.3.1.2) 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.

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.

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 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.

We understand that the existing foundation will be removed prior to the construction of the new foundation, and cofferdam and tremie base will be applied to seal the cofferdam to minimize water seepage. In this case, same cofferdam may be used for the construction of new spread footing foundation.

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.

The footing elevation should be chosen with due regard to scour considerations. It is likely that cobbles and boulders are a result of former scour action and as such placing the footings on a zone of cobbles and boulders may not be appropriate (e.g. Borehole NP4) and these footings may need to be extended to the surface of the till deposit.

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.

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 Borehole NP1 location (at other boreholes the overburden thickness is between 6.1 and 9.6 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 0.30m (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, apply grout or tremie base on 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, 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. The capacity of the pile will 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 horizontal loads may be taken by means of battered piles and it is recommended that the batter be limited to no more than 4 vertical to 1 horizontal.

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

5.1.4 SOUTH PIER

Boreholes SP1, SP2 and SP3 were drilled from a raft (in the River) for this investigation, as well as Borehole 13B (drilled by others in 2003) during a previous investigation. Depth of water in the River at the time of investigation ranged from 2.0 m (BH13B) to 5.5 m (Borehole SP2). The depth of overburden below the bottom of the River to the bedrock surface ranges from zero (at Borehole 13B, rock was contacted right at river bed) to 5.5 m at Borehole SP2.

5.1.4.1 SPREAD FOOTING FOUNDATIONS

The footing for the South Pier can be founded on the clean, intact and massive bedrock. The bedrock surface elevations at the Borehole locations are summarized in the table below (Table 5.1.4.1.1). Due to the sloping bedrock surface, the rock surface should be cut and leveled to provide a horizontal step-like base. If necessary, the grade may then be raised to the required elevations using concrete.

Table 5.1.4.1.1

Foundation Location	Reference BH	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	SP1	222.1	4.2	5.7 (1.5) 7.7 (3.5)	216.4 214.4	Glacial Till/Boulders Gneiss Bedrock
West side	SP2	222.1	5.6	8.3 (2.7) 11.1 (5.5)	213.8 211.0	Glacial Till/Boulders Gneiss Bedrock
East side	SP3	222.0	4.5	5.4 (0.9) 6.5 (2.0)	216.6 215.5	Glacial Till/Boulders Gneiss Bedrock
East side	13B	222.1	2.0	2.0 (0.0)	220.2	Gneiss Bedrock

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

From the above table it is evident that at some borehole locations, the footing cannot be placed on the overburden (i.e. it will need to be extended down to the surface of the bedrock) due to scour considerations, etc. Since it is not good design/construction practice to place footing partially on overburden and partially on bedrock, it is recommended that the South Pier foundation be extended below the overburden to the surface of the bedrock.

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

It should be noted that in between and beyond the borehole locations, the bedrock surface may vary considerably.

The recommended Factored Bearing Resistance at ULS for footings placed on the surface of relatively sound bedrock is 5,000 kPa, while SLS will not govern.

Dewatering and sheet pile cofferdam will be required in the excavation/construction of the footings. In view of the insufficient thickness of overburden and the bouldery conditions, tremie concrete seal at the bottom of the cofferdam and anchoring the supports in the bedrock would be required. The sheeting for the cofferdam would be constructed in a template frame before stronger lateral supports are implemented. Tight interlocking sheet piling and extensive dewatering may be required to allow the construction of the footings in the dry conditions. 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.

Consideration may also be given to supporting the founding element by spread footing on bedrock on the east side where the surface of the bedrock appears to be high to caissons socketed into the bedrock towards the west where the surface of bedrock is relatively deeper.

5.1.4.2 HEAVY-WALLED STEEL TUBING WITH GROUT ANCHOR

As was discussed in the previous section of this report for the north pier, the use of heavy-walled steel tube piles with grout anchor present a realistic solution. This method will not be repeated here for the sake of brevity. However, attention is drawn to the high rock surface elevation as recorded at Borehole 13B location.

5.1.5 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 South 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) or tremie concrete 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 protection should also be considered (refer to Section 5.6).

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 found 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.

The staging of construction with respect to the removal of the existing bridge foundations will need to be determined.

Potential effects of construction on the detour bridge structure should be taken into consideration. If driven piles are to be utilized then, depending on the distance from the foundation of the detour bridge to the pile driving location, the vibration generated by pile driving may need to be monitored and/or a vibrations specialist may need to be consulted. An NSSP may need to be included with regards to this aspect. The effects of pile driving on slope stability may also need to be looked into.

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 should be made to CHBDC 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 about 1.1 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 organic 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 AP1 And AP2, the anticipated foundation settlement under the stresses generated by the approximately 1 m grade raise is approximately 10 mm, while another 10 to 15 mm of settlement can occur due to settlement of the new embankment fill under its own weight. The anticipated total settlements are therefore not more than 25 mm, which necessitate neither surcharging nor preloading. The foundation settlements should be substantially completed within a period of about 6 months while the settlement due to the own weight of the embankment will depend on the type of soil used to build the embankment (e.g. the settlement of granular soils will be relatively rapid while clayey soils will settle more

slowly). Assuming an average SSM type soil, the settlement of the embankment under its own weight should also be substantially completed within about 3 months.

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 pier and 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. 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. Due to the 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.

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. A 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

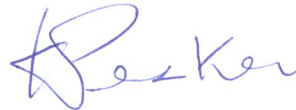
We recommend that during finalizing of the details of the Replacement bridge, close liaison be maintained with the foundation (geotechnical) consultant to select optimum solutions regarding settlement, fill stability, surcharging, etc. issues, as well as reviewing recommendations contained in this report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

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

SHAHEEN & PEAKER LIMITED



Zuhtu Ozden, P.Eng.



K. R. Peaker, Ph.D., P.Eng.



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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.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. Shaheen & Peaker Limited accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.