

**PRELIMINARY FOUNDATION
INVESTIGATION AND DESIGN REPORTS
FOR THE REPLACEMENT OF GLASS'S
BRIDGE OVER THE INNISFIL CREEK
SITE NO. 30-254
MTO CENTRAL REGION, W.P. 2053-11-01
GEOCRES NO. 31D-554**

McCormick Rankin Corporation
Highway 89, Town of Innisfil

Project: TRANETOB20462AA
March 18, 2013

FINAL REPORT

March 18, 2013

McCormick Rankin Corporation
2655 North Sheridan Way, Suite 300
Mississauga, Ontario
L5K 2P8

Attention: Mr. Ben Hui, P.Eng., M.Eng., Senior Project Manager


Dear Mr. Hui:

**RE: Preliminary Foundation Investigation and Design Reports
Replacement of Glass's Bridge over the Innisfil Creek (Site No. 30-254)
Town of Innisfil
MTO Central Region, W.P. 2053-11-01**

Please find attached our final preliminary foundation investigation and design reports relating to the above noted site.

If you have any comments or enquiries please contact the undersigned.

For and on behalf of Coffey Geotechnics Inc.


Zuhtu Ozden, P.Eng.
Senior Principal



PRELIMINARY FOUNDATION INVESTIGATION REPORT

McCormick Rankin Corporation
Replacement of Glass's Bridge Over The Innisfil Creek on
Highway 89, Site No. 30-254
Town of Innisfil
MTO Central Region W.P. 2053-11-01
GEOCRES No. 31D-554

TRANETOB20462AA
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FINAL REPORT

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**FINAL
PRELIMINARY FOUNDATION INVESTIGATION REPORT
REPLACEMENT OF GLASS'S BRIDGE OVER THE INNISFIL CREEK
TOWN OF INNISFIL
MTO CENTRAL REGION, W.P. 2053-11-01**

1 INTRODUCTION

Coffey Geotechnics Inc. (Coffey) was retained by McCormick Rankin Corporation (MRC) to carry out a preliminary foundation investigation for the proposed replacement of the existing Glass' Bridge over the Innisfil Creek in the Town of Innisfil, Ontario. The site is located on Highway 89 about 1 km west of the existing Highway 400. Site number 30-254 is assigned by Ministry of Transportation Ontario (MTO) for this project.

This preliminary study (technical memorandum) was carried out based on the available information only (desktop study) and no borehole investigation was performed. It should be pointed out that this technical memorandum is intended for planning purposes only and that a Detail Design Foundation Investigation and Design Report shall be required for detail design.

Based on the Ontario Bridge Management System (OBMS) report generated on January 2012, the existing Glass' Bridge is an 11 m long and 9 m wide, single span reinforced cast-in-place concrete structure, built circa 1913.

2 SITE DESCRIPTION AND GEOLOGY

The site is located about 1 km west of Highway 400, on Highway 89, as shown on Drawing 1A. The surrounding area is generally flat to gently rolling and Innisfil Creek meanders through the surrounding area (see site photographs in Appendix C).

The project area is located in the southern portion of the Nottawasaga basin which was at one time part of the floor of Lake Algonquin and its surface deposits are therefore generally of deltaic and lacustrine origin. The southern portion of this basin represents a bay, well separated from the main basin by moraine uplands.

According to the Physiography of Southern Ontario by L.J. Chapman and D.F. Putnam, 1984, the project site is located within the Physiographic Region known as the 'Simcoe Lowland'.

Based on the Ontario Geological Survey Bedrock information (<http://www.mndm.gov.on.ca/en/mines-and-minerals/applications/ogsearth/bedrock-geology>), the bedrock in the area comprises shale, limestone, dolostone, arkose and sandstone belongs to Ottawa and Simcoe Groups of the Shadow Lake Formation.

3 SUBSURFACE CONDITIONS

3.1 Past Reports

The existing subsurface information from MTO GEOCRES information system was used to prepare this report. A number of previous geotechnical investigations has been conducted in the surrounding area and below is a brief summary of available GEOCRES information within a 5 km radius of the site.

- ❖ **GEOCRES 31D-31, Foundation Investigation for Proposed Crossing Highway 400, Township of West Gwillimbury, County of Simcoe for Ontario Department of Highways (W.P. 138-60), Universal Geotechnique, 1960.**

The purpose of this investigation was to assess the subsurface conditions for the proposed crossing of Highway 400 over a township road, about 3 km south of Glass's Bridge site. Five boreholes and six penetration tests were carried out for the foundation investigation. A firm to dense grey clayey silty sand till was found at the site underneath a 0.6 to 1.5 m thick fill material. The use of spread footing foundations was recommended, utilizing an allowable bearing pressure of 2.5 tsf (about 240 kPa) on the native clayey silty sand till at Elevation 772 feet (235.3 m) or below. This structure site is located 3.2 km south east of the Glass's Bridge site.

- ❖ **GEOCRES 31D-249, Soil Investigation Cookstown, Ontario, Warnock Hersey Soil Investigation LTD., 1964.**

The purpose of this investigation was to assess the subsurface conditions for a proposed bridge at Highway 89, near Highway 27, immediately west of Cookstown. Two boreholes were put down at the site. Possible fill and surficial granular soils were found, underlain by a massive clay deposit. Due to the observed extremely soft soil below a depth of 12 feet (3.7 m), friction piles were recommended as a bridge foundation option. This bridge site is located 3.5 km west of the Glass's Bridge on Highway 89.

- ❖ **GEOCRES 31D-360, Foundation Investigation Report for Innisfil Creek Replacement Bridge, W.P. 126-95-01, Site 30-249, Highway 27, District 33, Owen Sound, Ministry of Transportation, 1997.**

The purpose of this investigation was to assess the subsurface conditions for the proposed crossing of Highway 27 (over Innisfil Creek). Two boreholes and two dynamic cone penetration tests (DCPT) were carried out to a depth of 46.2 m. Thick soft to firm clayey silt to silty clay was found to overlie compact sandy silt to silty sand at the site. Bearing resistances of 65 kPa at ULS and 50 kPa at SLS were recommended on the native silty clay to clayey silt at Elevation 211.0 m for a box culvert. As an alternative, timber pile option was presented in the report. This site is located 4.6 km south west of the Glass's Bridge site.

❖ **GEOCRES 31D-490, Borehole Soil Investigation Report, New Salt/Sand Storage Structure at Cookstown Patrol Yard, Cookstown, Ontario, G.W.P. 2030-09-00, AMEC Earth and Environmental, 2009.**

The purpose of this investigation was to assess the subsurface conditions for a proposed salt/sand storage structure. Five boreholes and one dynamic cone penetration test (DCPT) were carried out to a maximum depth of 13.7 m. The subsurface profile generally consisted of a surficial pavement, sand and gravel fill, sand fill overlying native sandy silt, which were further underlain by silty clay to clayey silt and clay. This site is located immediately west of the Innisfil Creek on the north side of Highway 89.

Three of these reference sites are 3.2 to 4.6 km away from the project site, while the fourth reference at the Cookstown Patrol Yard is directly across from the site, about ¼ km away. Consequently, this latter reference was used to prepare the following report, due to its close proximity, whilst the remaining three can be considered only as references which indicate that deltaic or glaciolacustrine origin massive cohesive soil deposit is not uncommon in the surrounding area, especially near to the existing watercourses (e.g. Innisfil Creek).

Only GEOCRES 31D-490 was used to prepare the following summary of subsurface conditions in Section 3.2.

3.2 Compiled Subsurface Conditions Based on Previous Investigation

3.2.1 Background

In order to gain a better understanding of the subsurface conditions at the proposed bridge replacement site, the data from the previously completed geotechnical study, immediately west of the Innisfil Creek on the north side of the existing Highway 89 were reviewed.

AMEC Earth and Environmental (AMEC) carried the site investigation in 2009 and their report was available in MTO GEOCRES information system (GEOCRES 31D-490). Five boreholes and one dynamic cone penetration test (DCPT) were carried out at the proposed salt storage structure site.

Based on the findings of the AMEC investigation, the inferred subsurface profiles generally consisted of pavement structure (asphaltic concrete and pavement fill), granular fill overlying a native sandy silt deposit which is underlain by a basically cohesive deposit consisting of silty clay/clayey silt and clay.

The following paragraphs summarize the subsurface conditions (reported by AMEC) at MTO patrol yard, nearby the proposed bridge site.

It should be noted that the soil and groundwater conditions vary between and beyond the borehole coverage.

3.2.2 Pavement Structure and Granular Fill

All boreholes drilled within the existing MTO patrol yard contacted a surficial pavement structure and granular fill to depth of 1.1 to 1.7 m (El. 227.4 m to 226.8 m).

Two grain size analyses were carried out on samples of the granular fill and analyses results are presented in Appendix B. The results of the two grain size analyses indicate the following grain-size distribution:

Gravel:	9 - 10 %
Sand:	76 - 77 %
Silt:	11 %
Clay:	2 - 4 %

Based on the recorded SPT N-values of 5 to 30 blows/0.3 m, this granular fill is in a very loose to compact condition.

3.2.3 Sandy Silt

A brown sandy silt deposit, with trace clay and gravel, was identified below the fill soils in all boreholes except for BH 4 and extended to depths of approximately 2.1 m to 3.7 m (El. 226.4 to 224.8 m).

Three grain size analyses were carried out on samples from this layer. Grain size distributions are presented in Appendix B. The results of the three grain size analyses indicate the following grain-size distribution:

Gravel:	0 - 1 %
Sand:	23 - 39 %
Silt:	53 - 69 %
Clay:	6 - 7 %

Atterberg Limits tests were performed on three soil samples and the results are presented in Appendix B.

Liquid Limit:	14 - 18%
Plastic Limit:	11 - 17%
Natural Moisture Contents:	18.1 - 21.5%

SPT 'N'-values of 5 to 17 blows/0.3 m were recorded within the sandy silt layer, indicating a very loose to compact relative density.

3.2.4 Silty Clay/Clayey Silt/Silt and Clay

A soil deposit consisting of silty clay/clayey silt/silt and clay was encountered underneath the surficial sandy silt layers in Boreholes BH1, BH2, BH3 and BH 5, and underneath the granular fill in BH 4. It extended to the termination depths of all boreholes, which was about 11.1 m (El. 217.4 m to 217.3 m). The silt and clay was brown to grey in color, and contained trace to some sand. DCPT was carried out in BH 4, below the drilled depth, to a depth of about 13.7 m (El. 214.9 m), to confirm consistency of the soil below the drilled depth. In general, the DCPT indicated an increase in the consistency of the soil up to the end of the test (see Record of Borehole in Appendix A).

Ten grain size analyses were carried out on selected samples and the results are presented in Appendix B. The results of the ten grain size analyses indicate the following grain-size distribution:

Gravel:	0 - 1 %
Sand:	0 - 11 %
Silt:	35 - 57 %
Clay:	32 - 61 %

Atterberg Limits tests were performed on ten soil samples and results are included in Appendix B, as follows:

Liquid Limit:	25 - 44%
Plastic Limit:	10 - 18%
Natural Moisture Contents:	19.2-30.6 %

The values that were obtained from the samples are characteristic of a cohesive soil of low to medium plasticity.

Based on the measured N-values from 6 to in excess of 50 blows/0.3m, consistency of this soil stratum can be described as firm to hard.

It is our opinion that the thickness of clayey soil deposit may possibly increase towards the creek and that the consistency of the natural soil may vary across the surrounding area.

3.2.5 Groundwater Conditions

Free-standing water was reported in the open boreholes immediately upon their completion. The measured groundwater levels as reported are shown on the Record of Boreholes (Appendix A) and are summarized in the following table:

Table 3.2.5.1
Groundwater Conditions

Borehole	Ground Surface Elevation (m)	Water Level Measurement Depth (m)*	Water Level Measurement Elevation (m)*	Remarks
1	228.5	2.7	225.8	-
2	228.5	2.7	225.8	-
3	228.6	1.8	226.8	-
4	228.6	3.1	225.5	-
5	228.5	1.8	226.7	-

* Not stabilized

It should be pointed out that the groundwater would be subject to seasonal fluctuations and fluctuations in response to major weather events. The groundwater may also be influenced by the water level in the watercourse, known to be Innisfil Creek.

Groundwater condition at the proposed bridge site may be controlled by creek water.

For and on behalf of Coffey Geotechnics Inc.

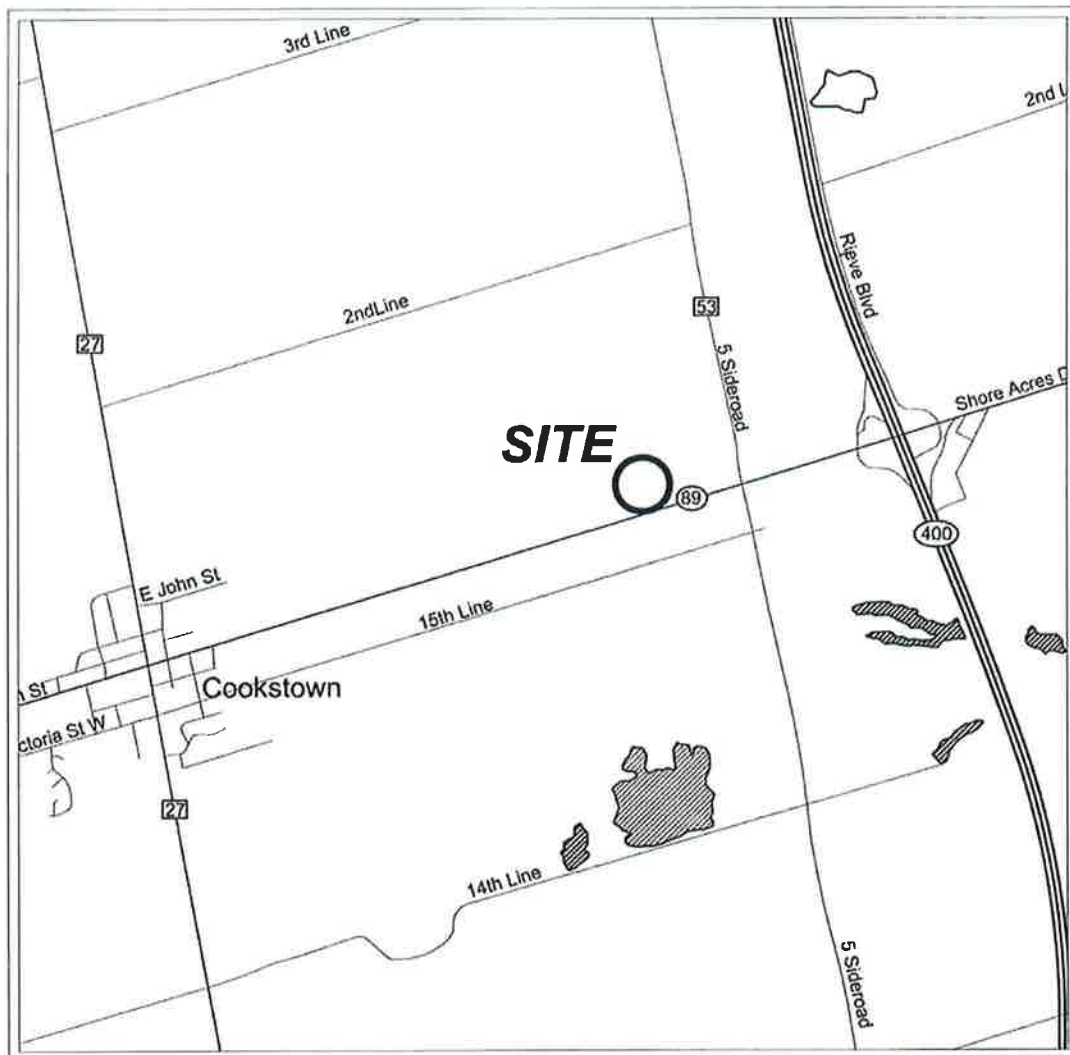

Gwangha Roh, Ph.D., P.Eng.
Senior Geotechnical Engineer




Zuhtu Ozden, P.Eng.
Senior Principal



Drawings



KEY PLAN

500 0 500 1000 1500 2000m

Approximate Scale

AMEC Earth & Environmental,
a Division of AMEC Americas Limited



CLIENT LOGO



CLIENT

MINISTRY OF
TRANSPORTATION ONTARIO

TITLE

SITE PLAN

DWN BY:

KW

DATUM:

-

DATE:

November 2009

PROJECT

BOREHOLE SOIL INVESTIGATION FOR
NEW SALT/SAND STORAGE STRUCTURE AT COOKSTOWN PATROL YARD
AGREEMENT NUMBER 2009-C-0082, G.W.P. No. 2030-09-00

CHKD BY:

PB

REV. NO.:

A

PROJECT NO:

TT93045

PROJECTION:

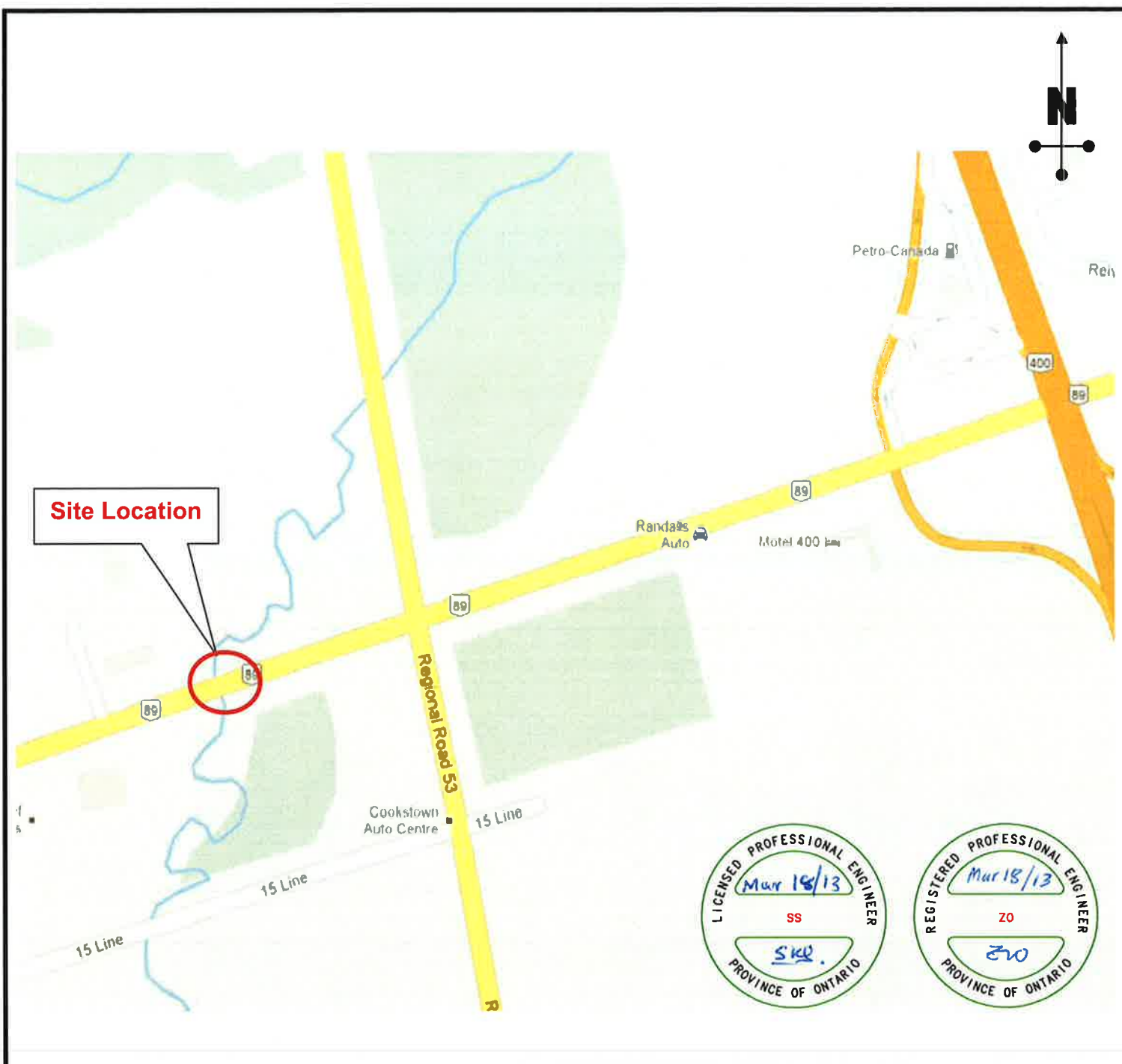
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SCALE:

AS SHOWN

DRAWING No.

1



SITE LOCATION PLAN

coffey
geotechnics
SPECIALISTS MANAGING
THE EARTH

Date:
MARCH 6, 2013

Project:
TRANETOB20462AA

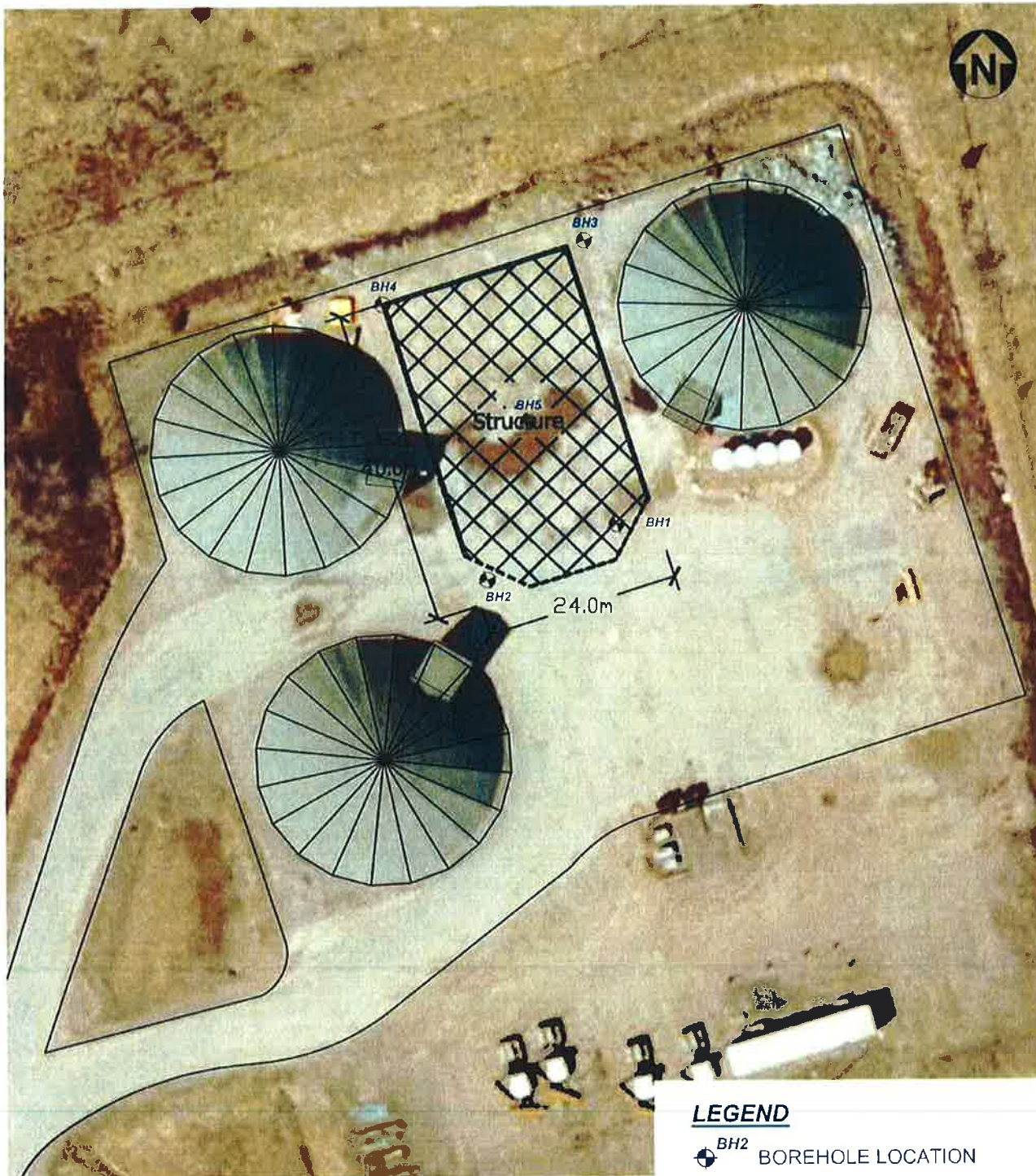
**INNISFIL CREEK BRIDGE,
HWY 89, ONTARIO**

Prepared for: Ministry of Transportation, Ontario

Prepared By:
SSH

Reviewed By:
SS

Drawing No **1A**



AMEC Earth & Environmental,
a Division of AMEC Americas Limited



CLIENT LOGO



Ontario

CLIENT

MINISTRY OF
TRANSPORTATION ONTARIO

TITLE BOREHOLE LOCATION PLAN

DWN BY:
KW

DATUM:

DATE: November 2009

PROJECT BOREHOLE SOIL INVESTIGATION FOR
NEW SALT/SAND STORAGE STRUCTURE AT COOKSTOWN PATROL YARD
AGREEMENT NUMBER 2009-C-0092, G.W.P. No. 2030-09-00

CHK'D BY:
PB

REV. NO.: A

PROJECT NO:
TT93045

PROJECTION:

SCALE:
N.T.S.

DRAWING No.
2

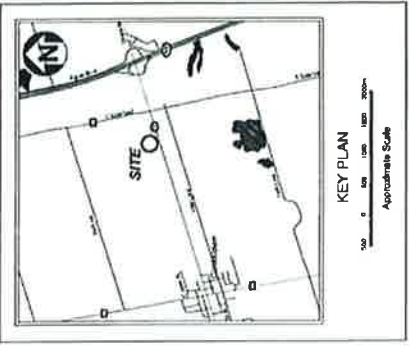
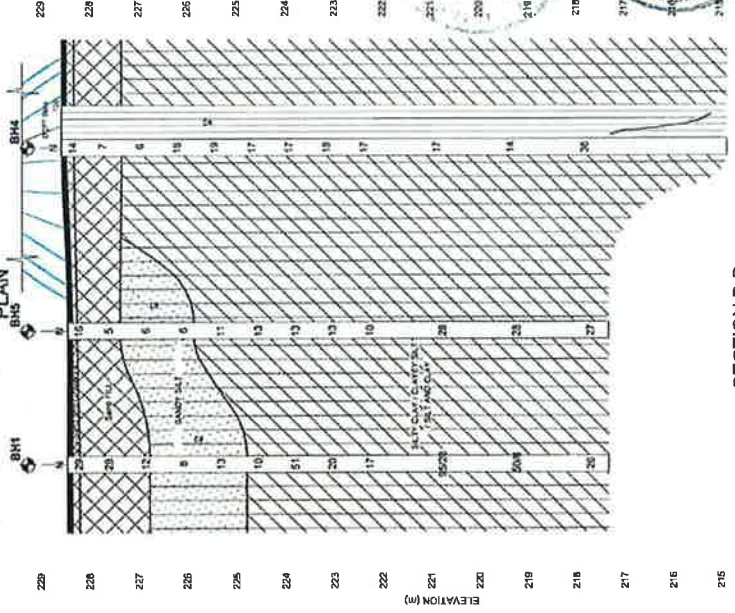
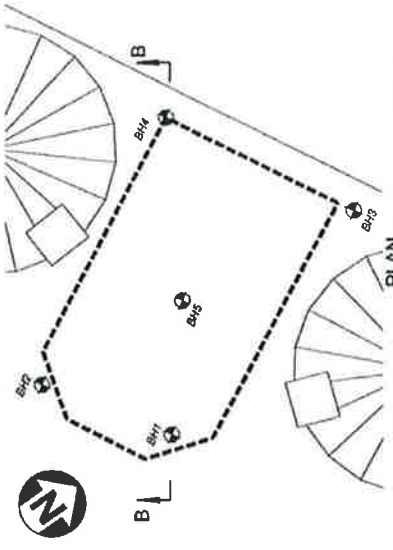
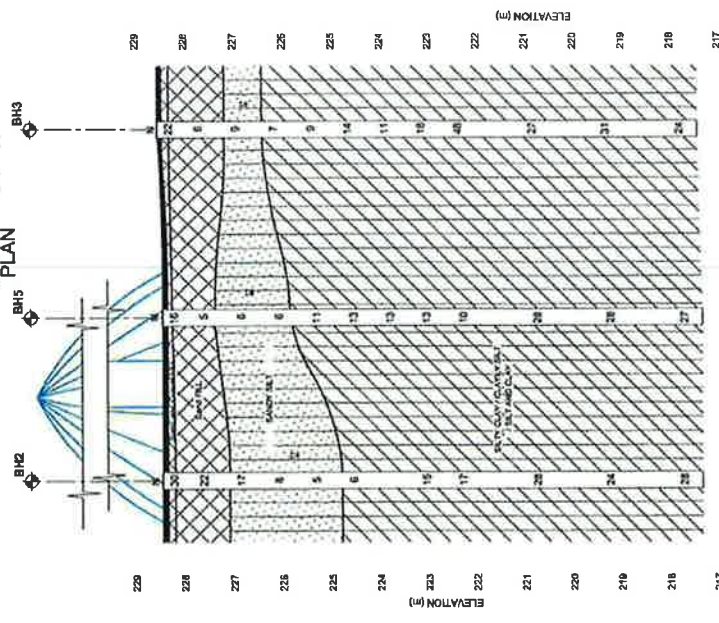
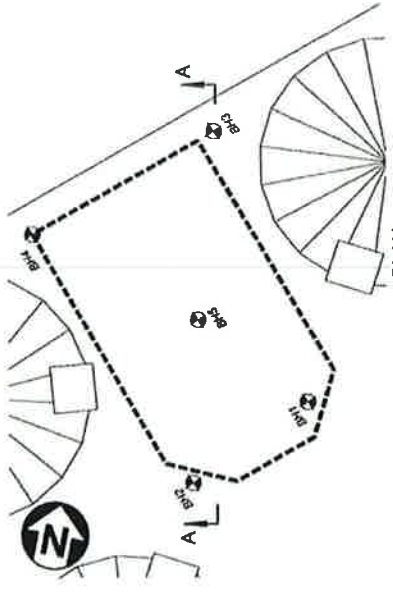
METRIC
DIMENSIONS AND MEASURES
AND/OR ALTIMETRIES
UNLESS OTHERWISE SHOWN

AGREEMENT No.
2009-C-0092
G.W.P. No.
2030-09-00

BOREHOLE SOIL INVESTIGATION FOR NEW
SALT/ SAND STORAGE STRUCTURE AT
COOMSTOWN PATROL YARD

amec
AMEC Earth & Environmental
a Division of AMEC Americas Limited

SHEET



LEGEND
BOREHOLE LOCATION
GROUNDWATER LEVEL IN BOREHOLE AT TIME OF INVESTIGATION

NOTES
1. The boundaries between soil strata have been established only at borehole locations. Between boreholes, the boundaries are assumed from geological evidence and may be subject to considerable error.

SOIL STRATIGRAPHY
ASPHALTIC CONCRETE
FILL
SILTY CLAY / CLAYEY SILT / SILT AND CLAY
SANDY SILT



0 6 12 18m
SCALE

SECTION B-B

SECTION A-A

Appendix A

Record of Borehole Sheets

RECORD OF BOREHOLE No BH 1

1 OF 1

G.W.P. 2030-09-00 LOCATION Cookstown Patrol Yard, Cookstown, Ontario (N:4894828 E:606382) ORIGINATED BY SAL
 DIST HWY BOREHOLE TYPE Solid Stem Augering COMPILED BY SN
 DATUM Geodetic DATE 15 October 2009 - 15 October 2009 CHECKED BY PB
 PROJECT Borehole Soil Investigation for New Sand/Salt Storage Structures JOB NO. TT93045

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DEPTH m	ELEVATION SCALE m	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	SOIL VAPOUR READING	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES				SHEAR STRENGTH kPa		WATER CONTENT (%)		W _p		
									20 40 60 80 100	20 40 60 80 100					
									20 40 60 80 100	20 40 60 80 100					
228.5															
228.4	about 100 mm ASPHALT														
228.3	brown		1	SS	29		228							0	
	Sand and Gravel FILL		2	SS	28		1							0	10 77 11 2
	trace to some silt														
	compact														
	moist														
	brown														
	Sand FILL						227							5	
226.8	some silt, trace clay and gravel		3	SS	12		2								
1.7	compact														
	moist														
	some asphalt fragments and organics														
	brown		4	SS	8		226							15	
	SANDY SILT														
	trace clay						3								
	loose to compact														
	moist		5	SS	13		225							5	
	trace gravel														
224.8	wet														
3.7	grey		8	SS	10		4				10	25	24	0	11 57 32
	SILTY CLAY / CLAYEY SILT / SILT AND CLAY														
	trace to some sand														
	trace gravel		7	SS	51		224				16	27	34	0	1 2 55 42
	stiff to hard														
	moist		8	SS	20		223							0	
			9	SS	17		222							0	
							221								
			10	SS	95/28		220							15	
							219								
			11	SS	50/8		218							10	
							217								
217.3			12	SS	26		218							10	
11.1	End of Borehole														
	Groundwater in open borehole on completion: 2.7 m														
	Borehole was backfilled with bentonite at the completion of drilling.														

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

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

RECORD OF BOREHOLE No BH 3

1 OF 1

G.W.P. 2030-09-00 LOCATION Cookstown Patrol Yard, Cookstown, Ontario (N:4894864 E:606378) ORIGINATED BY SAL
DIST HWY BOREHOLE TYPE Solid Stem Augering COMPILED BY SN
DATUM Geodetic DATE 16 October 2009 - 16 October 2009 CHECKED BY PB
PROJECT Borehole Soil Investigation for New Sand/Salt Storage Structures JOB NO. TT93045

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DEPTH m	B ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	SOIL VAPOUR READING PPM	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES				20	40	60	80	100					
228.6																		
228.3	about 80 mm ASPHALT																	
228.2	brown																	
	Sand and Gravel FILL		1	SS	22		228										0	
	trace to some silt																	
	compact		2	SS	8		1										0	9 76 11 4
	moist																	
227.2	brown																	
1.4	Sand FILL						227										15	31 63 6
	some silt, trace clay and gravel		3	SS	9													
	loose to compact																	
	moist						2											
226.4	brown																	
2.1	SANDY SILT						226										0	
	trace clay, trace organics		4	SS	7													
	loose																	
	moist to wet						3										0	
	brown																	
	SILTY CLAY / CLAYEY SILT /		5	SS	9		225										0	
	SILT AND CLAY																	
	trace sand and gravel		6	SS	14		4										0	
	firm to hard																	
	moist						224											
			7	SS	11												0	
			8	SS	18		223										5	
			9	SS	48		6										5	48 52
							222											
							7											
			10	SS	27		221										20	
							220											
							9											
			11	SS	31		219										35	
							10											
							218										15	
			12	SS	24		11											
217.4																		
11.1	End of Borehole																	
	Groundwater in open borehole on completion: 1.8 m																	
	Borehole was backfilled with bentonite at the completion of drilling.																	

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

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

RECORD OF BOREHOLE No BH 4

2 OF 2

G.W.P. 2030-09-00	LOCATION Cookstown Patrol Yard, Cookstown, Ontario (N:4894856 E:606353)	ORIGINATED BY SAL
DIST _____ HWY _____	BOREHOLE TYPE Solid Stem Augering and Dynamic Cone Penetration	COMPILED BY SN
DATUM Geodetic	DATE 15 October 2009 - 15 October 2009	CHECKED BY PB
PROJECT Borehole Soil Investigation for New Sand/Salt Storage Structures		JOB NO. TT93045

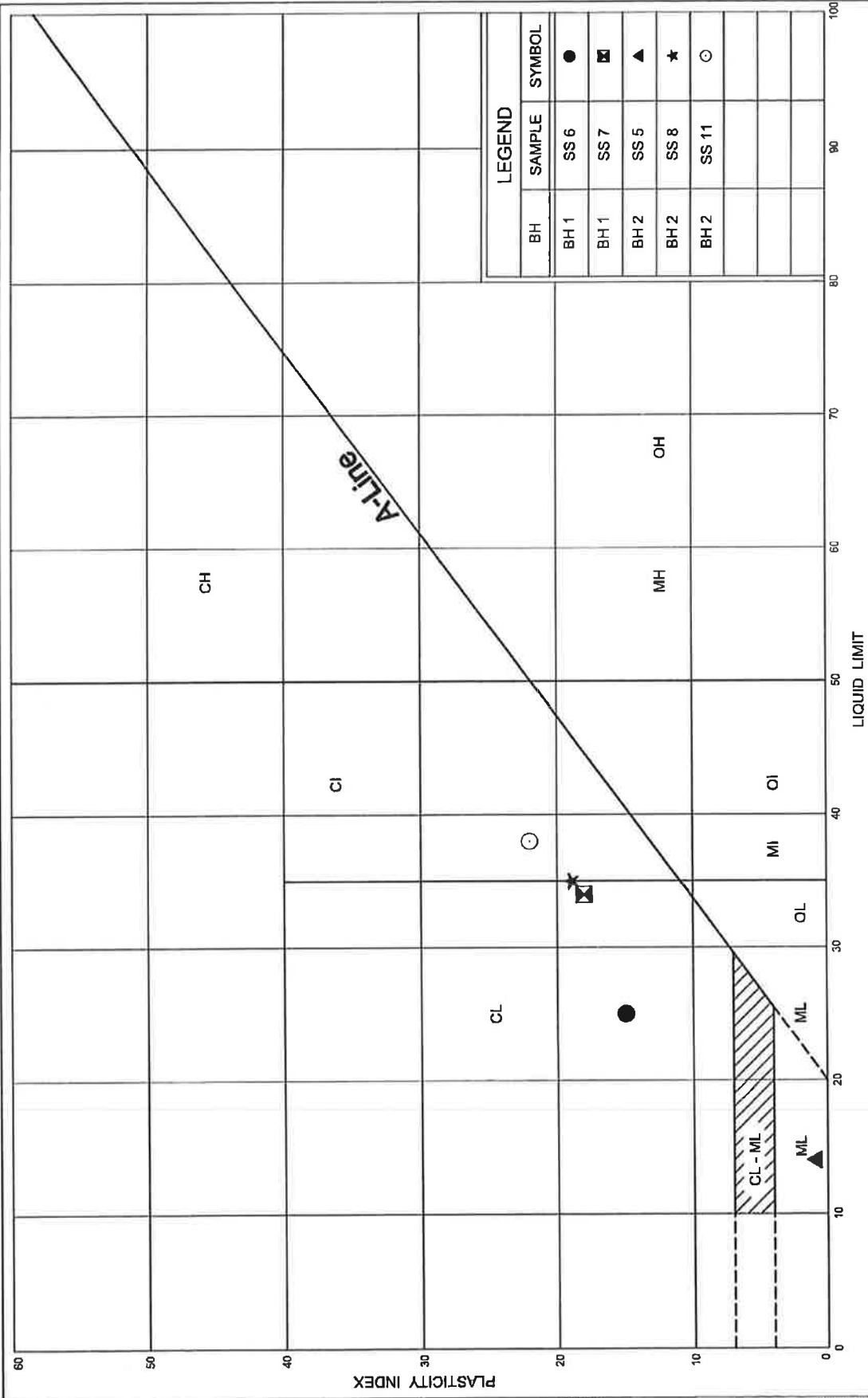
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DEPTH	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	SOIL VAPOUR READING	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES				SHEAR STRENGTH kPa									WATER CONTENT (%)		
									<div><div></div><div>20406080100</div></div>				<div><div>W_p</div><div>W</div><div>W_L</div></div>							
									<div><div>○ UNCONFINED</div><div>■ QUICK TRIAXIAL</div></div> <div><div>+ FIELD VANE</div><div>× LAB VANE</div></div>				<div><div></div><div>102030</div></div>							
									20	40	60	80	100	10	20	30	PPM	GR SA SI CL		
214.9								215												
13.7	End of DCPT Groundwater in open borehole on completion: 3.1 m Borehole was backfilled with bentonite at the completion of drilling.																			

RECORD OF BOREHOLE No BH 5

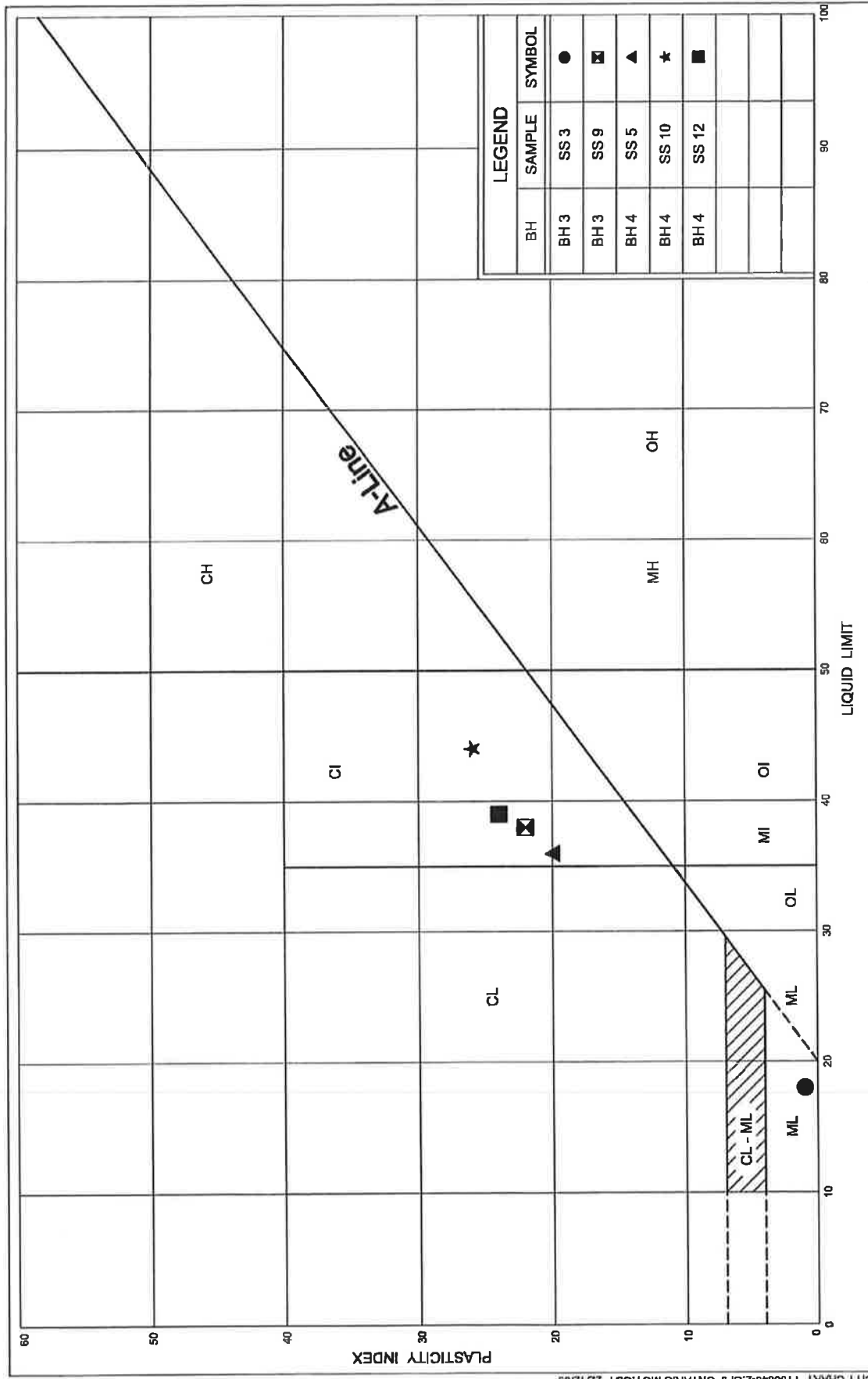
G.W.P. 2030-09-00		LOCATION Cookstown Patrol Yard, Cookstown, Ontario (N:4894841 E:606371)		1 OF 1		ORIGINATED BY SAL													
DIST _____ HWY _____		BOREHOLE TYPE Solid Stem Augering				COMPILED BY SN													
DATUM Geodetic		DATE 16 October 2009 - 16 October 2009				CHECKED BY PB													
PROJECT Borehole Soil Investigation for New Sand/Salt Storage Structures						JOB NO. TT83045													
SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	DEPTH m	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	SOIL VAPOUR READING PPM	REMARKS & GRAIN SIZE DISTRIBUTION (%)							
ELEV. (m)	DEPTH (m)	DESCRIPTION	STRAT. PLOT										NUMBER	TYPE	"N" VALUES	20	40	60	80
228.5	0.0	about 100 mm ASPHALT																	
228.4	0.1	brown Sand and Gravel FILL		1	SS	16													
228.3	0.2	trace to some silt compact moist		2	SS	5													
227.4	1.1	brown Sand FILL																	
		some silt, trace clay and gravel loose to compact moist		3	SS	6													
225.9	2.6	brown SANDY SILT		4	SS	8													
		trace clay and gravel loose moist to wet																	
		brown SILTY CLAY / CLAYEY SILT / SILT AND CLAY		5	SS	11													
		trace sand firm to very stiff moist		6	SS	13													
		grey		7	SS	13													
				8	SS	13													
				9	SS	10													
				10	SS	28													
				11	SS	28													
				12	SS	27													
217.3	11.1	End of Borehole																	
		Groundwater in open borehole on completion: 1.8 m																	
		Borehole was backfilled with bentonite at the completion of drilling.																	

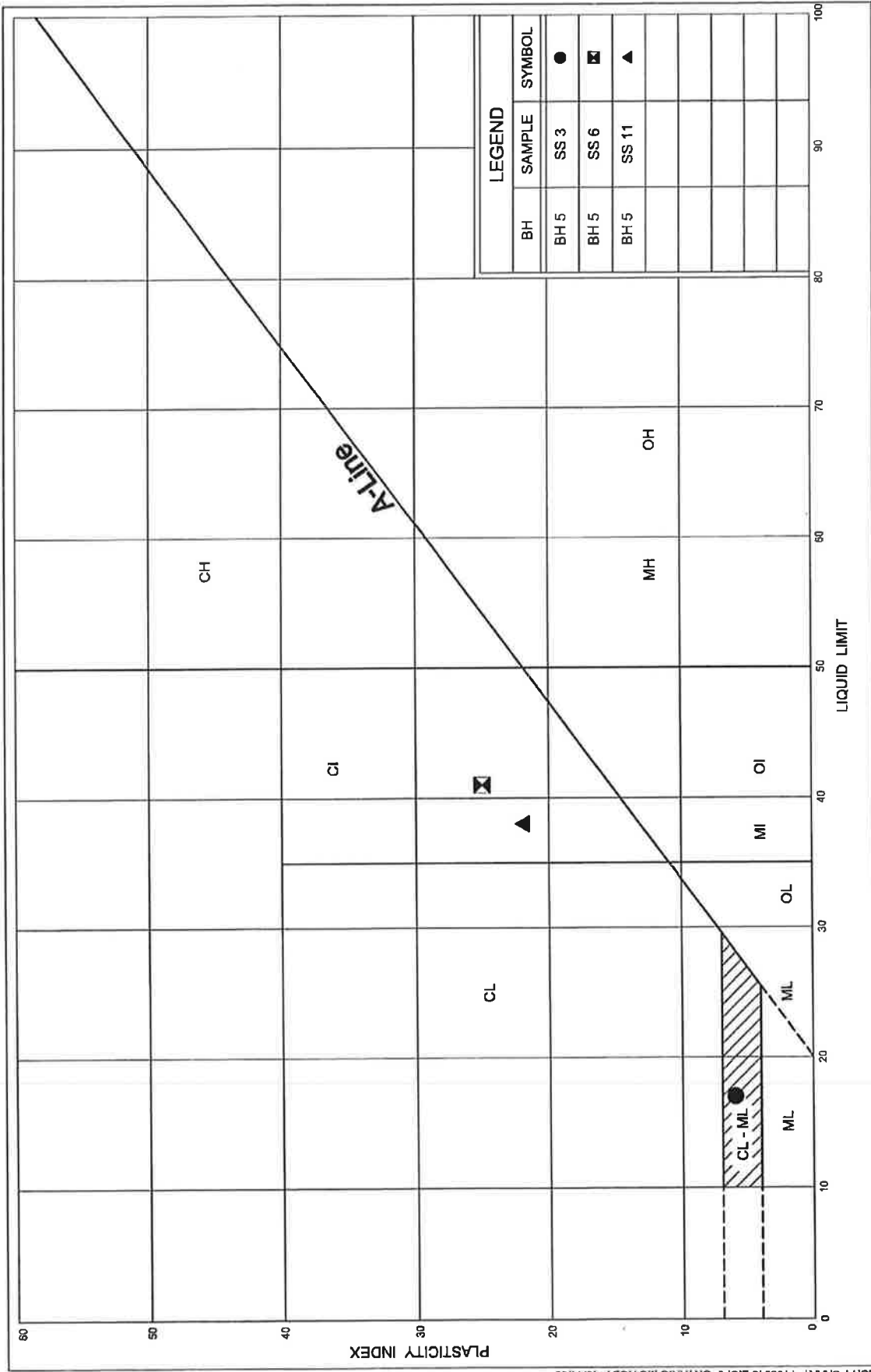
Appendix B

Test Results



LEGEND		
BH	SAMPLE	SYMBOL
BH 1	SS 6	●
BH 1	SS 7	⊠
BH 2	SS 5	▲
BH 2	SS 8	★
BH 2	SS 11	○





UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT		SAND			GRAVEL		
		Fine		Medium	Coarse	Fine	Coarse

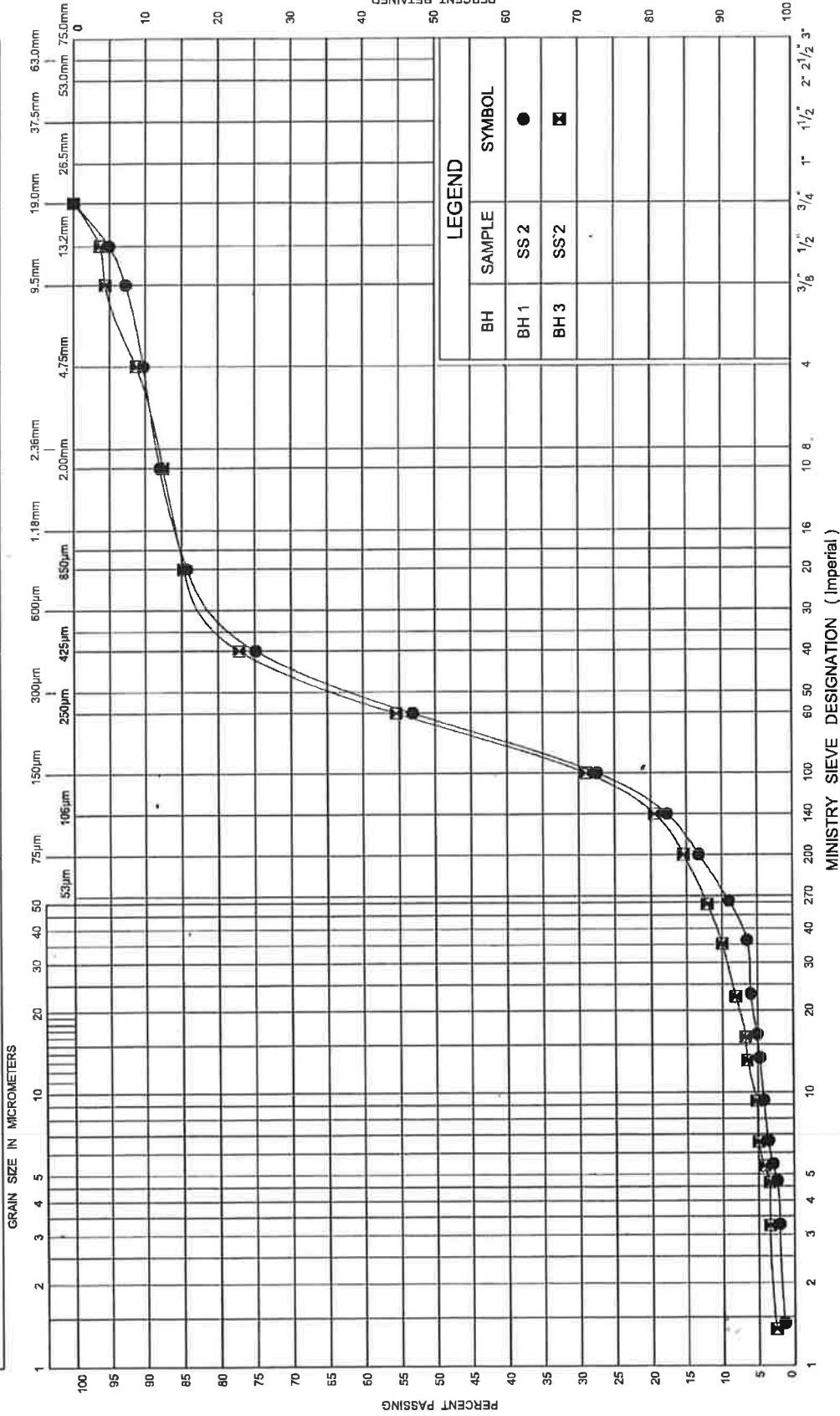


Figure No. B4

GRAIN SIZE DISTRIBUTION

SAND (FILL)

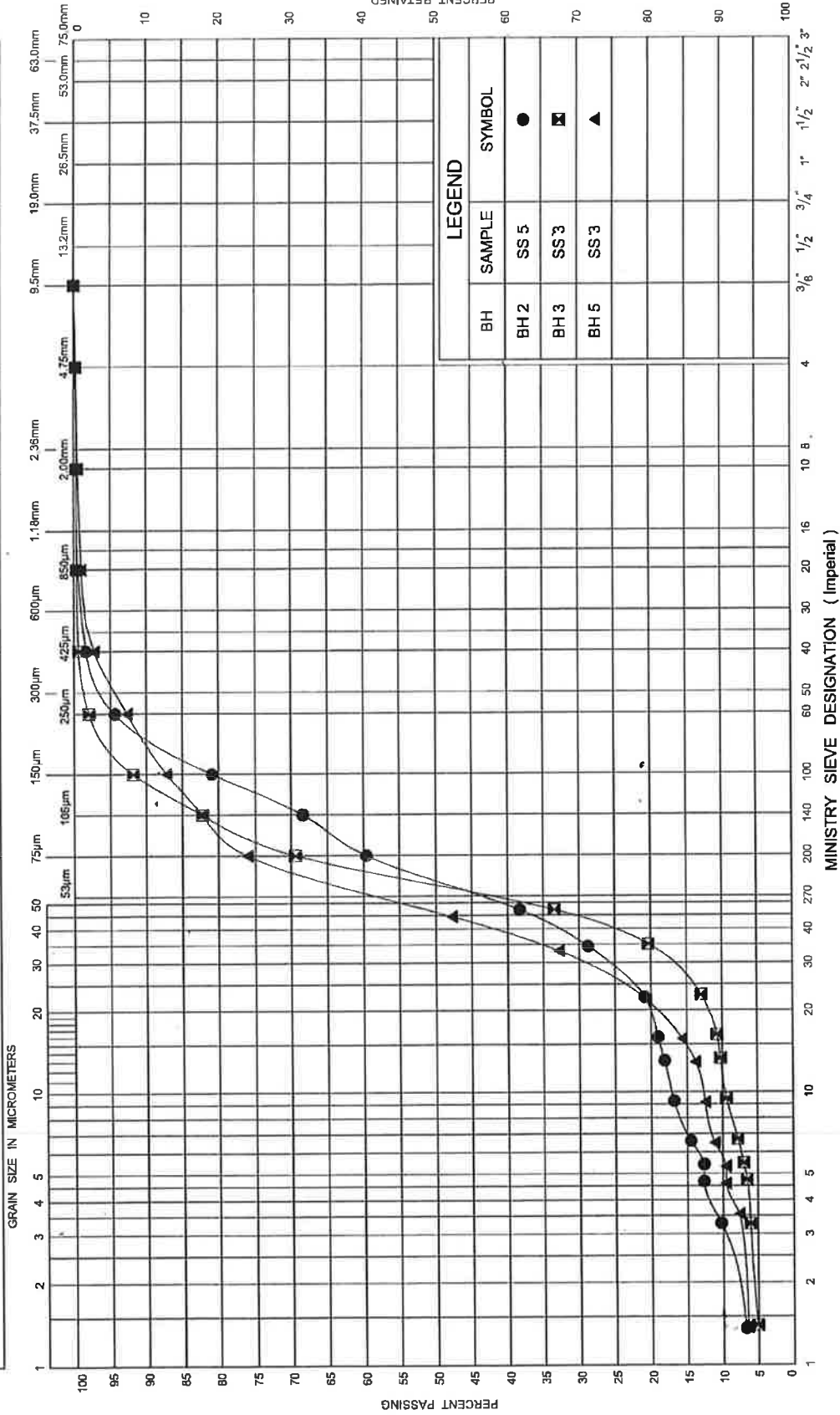
some silt, trace clay and gravel

G.W.P. 2030-09-00

Cookstown Patrol Yard

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT		SAND			GRAVEL		
		Fine		Medium	Coarse	Fine	Coarse



GRAIN SIZE DISTRIBUTION

SANDY SILT

trace clay and gravel

Figure No. B5

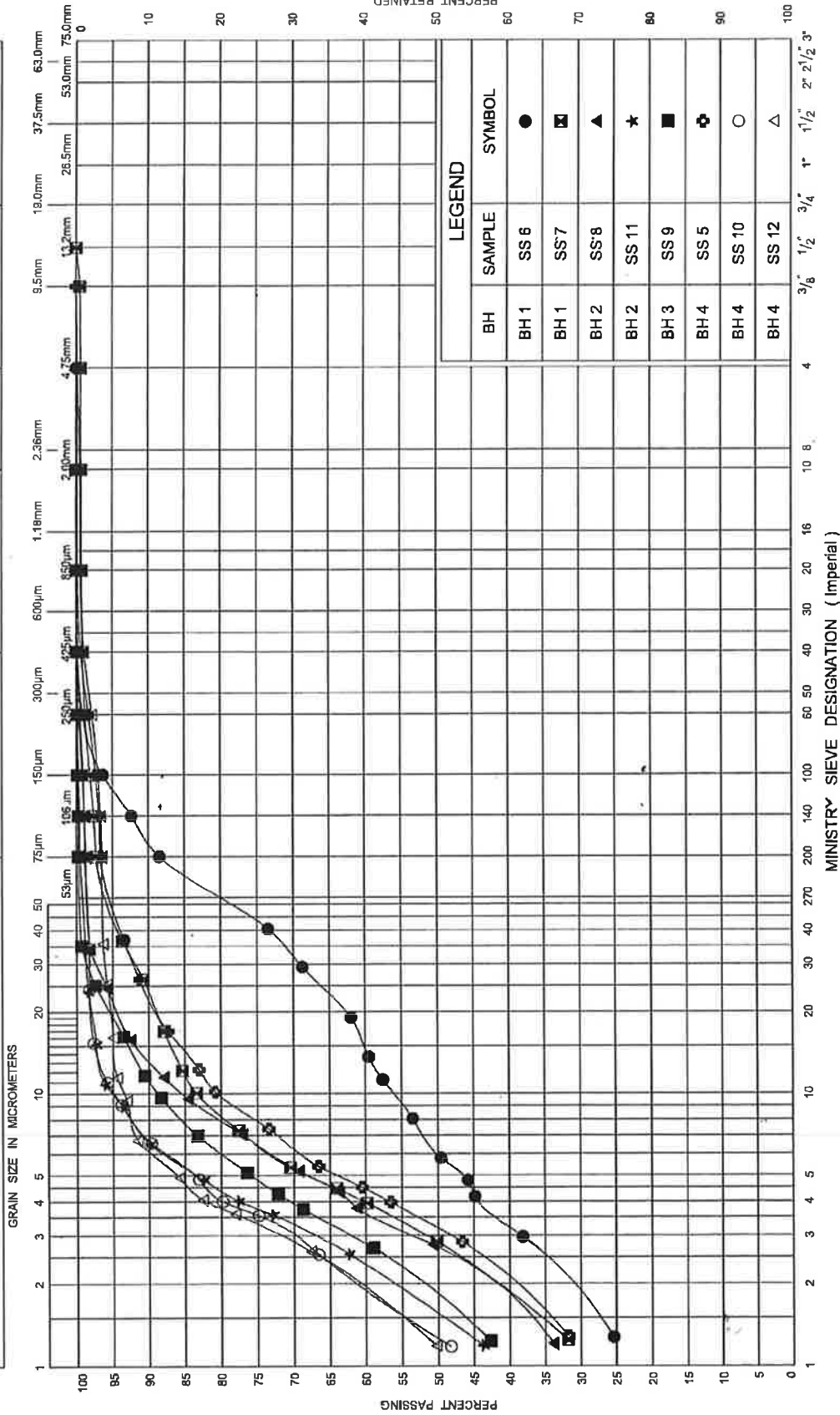
G.W.P. 2030-09-00

Cookstown Patrol Yard



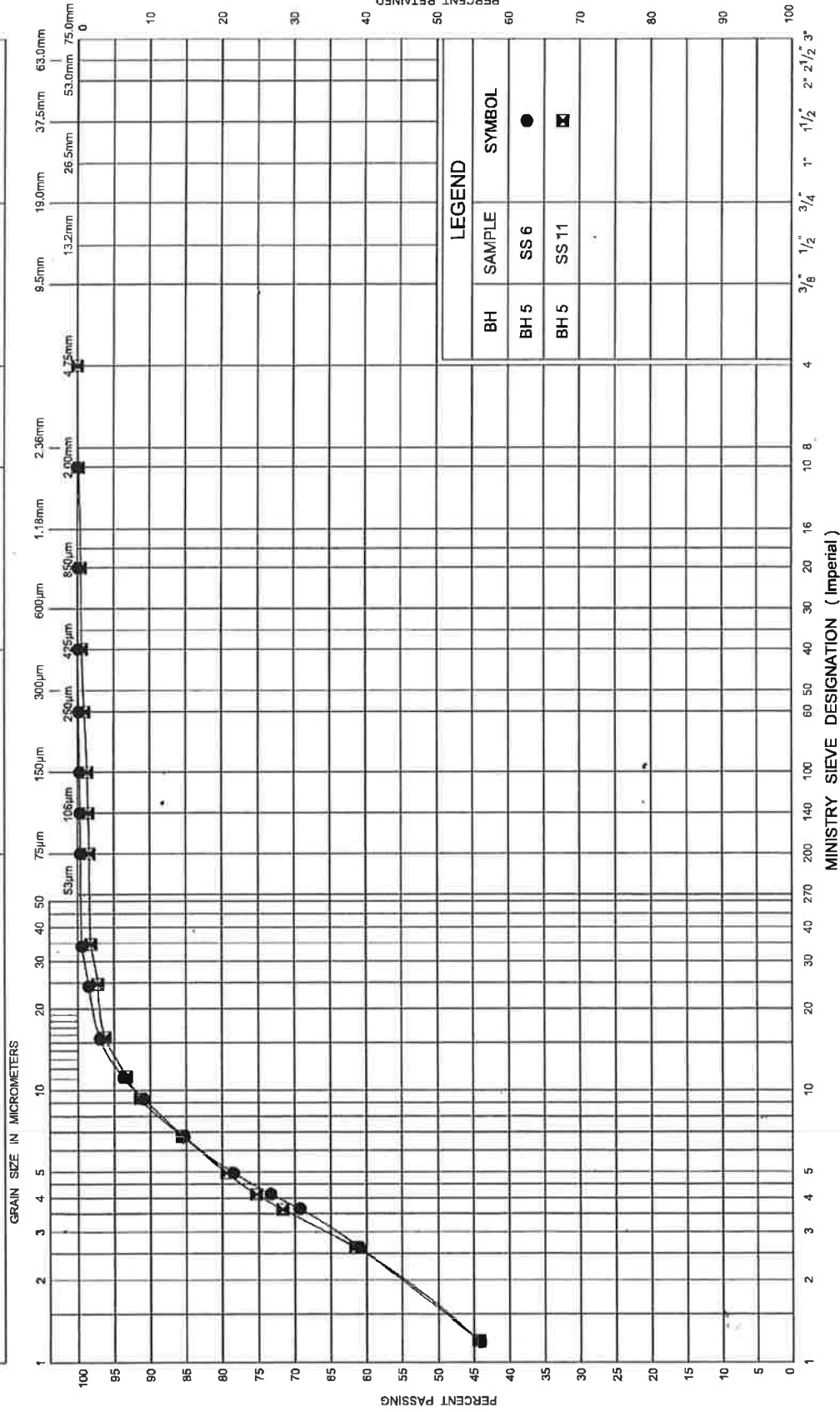
UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT		SAND			GRAVEL		
		Fine		Medium	Coarse	Fine	Coarse



UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT		SAND			GRAVEL		
		Fine	Medium	Coarse	Fine	Coarse	



GRAIN SIZE DISTRIBUTION

SILT AND CLAY
trace sand

Figure No. B7

G.W.P. 2030-09-00

Cookstown Patrol Yard



Appendix C

Site Photographs



Photograph 1. Site Setting (looking west)



Photograph 2. South side of Bridge



Photograph 3. Looking East



Photograph 4. MTO patrol yard on north-west side of the bridge



Photograph 5. Innisfil Creek

Appendix D

Explanation of Terms Used in the 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 STRUCTURAL FEATURES AND/OR STRENGTH.

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

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

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

JOINT AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

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

MECHANICAL PROPERTIES OF SOIL

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

PHYSICAL PROPERTIES OF SOIL

P_s	kg/m^3	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	e_{\min}	1, %	VOID RATIO IN DENSEST STATE
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{\max} - e}{e_{\max} - e_{\min}}$
P_w	kg/m^3	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kN/m^3	UNIT WEIGHT OF WATER	s_r	%	DEGREE OF SATURATION	D_n	mm	N PERCENT – DIAMETER
P	kg/m^3	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ	kN/m^3	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
P_d	kg/m^3	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m^3/s	RATE OF DISCHARGE
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $(W_L - W_p)$	v	m/s	DISCHARGE VELOCITY
P_{sat}	kg/m^3	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $(W - W_p) / I_p$	i	1	HYDRAULIC GRADIENT
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $(W_L - W) / I_p$	k	m/s	HYDRAULIC CONDUCTIVITY
P'	kg/m^3	DENSITY OF SUBMERGED SOIL	e_{\max}	1, %	VOID RATIO IN LOOSEST STATE	j	kN/m^3	SEEPAGE FORCE
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL						

PRELIMINARY FOUNDATION DESIGN REPORT

McCormick Rankin Corporation
Replacement of Glass's Bridge over The Innisfil Creek,
Site No. 30-254
Highway 89, Town of Innisfil
MTO Central Region, W.P. 2053-11-01
GEOCRES No. 31D-554

TRANETOB20462AA
March 18, 2013

FINAL REPORT

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Appendices

Appendix E: GA Drawing

Appendix F: Advantages, Disadvantages, Costs and Risks/Consequences of Foundation Alternatives

Appendix G: Limitations of Report

**FINAL
PRELIMINARY FOUNDATION DESIGN REPORT
REPLACEMENT OF GLASS'S BRIDGE OVER THE INNISFIL CREEK
TOWN OF INNISFIL
MTO CENTRAL REGION, W.P. 2053-11-01**

4 DISCUSSION AND RECOMMENDATIONS

Based on the Ontario Bridge Management System (OBMS) report generated on January 2012, the existing Glass's bridge is a 11 m long and 9 m wide single span, reinforced cast-in-place concrete bridge built in or around 1913*.

The existing Glass's Bridge general arrangement (GA) drawing obtained from MTO Geocres is given in Appendix E. Based on the drawing, the existing bridge appears to have been founded on spread footings placed about 4 feet (about 1.2 m) below the stream bed and height of the bridge (from the top of the foundations to the top of abutments) is about 13 feet (3.96 m).

AMEC Earth and Environmental (AMEC) carried out a site investigation in 2009 directly across from the site, ¼ km away, and their report was available in MTO GEOCRES information system (GEOCRES 31D-490). Based on the findings of the AMEC investigation, estimated subsurface profiles nearby the bridge location generally consisted of up to 1.7 m deep fill overlying a surficial native sandy silt layer, which is underlain at depths ranging from 2.1 to 3.7 m below the ground surface by a silty clay/clayey silt and clay deposit.

Based on the topography at the site, the existing bridge may possibly be founded on the native silty clay/clayey silt/clay deposit underlying the surficial sandy silt deposit.

The existing Glass's Bridge will be replaced with a wider and longer bridge at the site.

**It should be noted that the drawings prepared by W.A. Spellman of Barrie, also indicate that the bridge may have been built after 1933 or alternatively these drawings may have been prepared for a proposed widening of the structure.*

4.1 New Bridge Construction

Based on the preliminary GA drawing provided to us by MRC (see Appendix E), one scenario is to replace the existing bridge with a wider and longer bridge.

Assuming that the subsurface conditions at the site are similar to those of the new salt/sand storage building location at the Cookstown Patrol Yard, the site would be underlain by a silty clay/clayey silt/clay deposit (referred to as silty clay hereafter for brevity). No field vane testing was performed to measure the in-situ undrained shear strength of the silty clay deposit, however, from the available data, it appears that in general, the material has a firm to stiff consistency near the surface, becoming more competent with increased depth. From these findings, the following tentative conclusions emerge:

- The use of normal spread footings to support the structure is unlikely to be feasible;
- The widening will inevitably involve grade raises over and above the original grades (o.g.). However as any grade raise will cause settlements it is prudent to keep the grade raise to a practical minimum in anticipation of impending consolidation settlements; and,

- Moderate grade raises of 2 to 3 m should not cause foundation failures for the side slopes.

With this preamble, the following is a discussion, based on the available data.

While the existing bridge may possibly have been supported on normal spread footing foundations on the silty clay deposit, the use of normal spread footings for a longer span structure will unlikely be feasible and for preliminary design and estimating purposes, it may be prudent to assume that deep foundations will be needed. In addition, MTO will likely prefer an integral abutment type structure and, in that event, driven steel H-piles will need to be utilized.

The boreholes drilled by AMEC did not penetrate a sufficient depth for the purposes of deep foundations. The boreholes were generally terminated at about El. 217 m while a Dynamic Core Penetration Test (DCPT) reached El. 215 m. While these data are not sufficient for making specific recommendations for deep foundations, there is some evidence that the soil is becoming more competent at or below about El. 215 m.

From the available GEOCRE data, there is no information on the overburden depth to the surface of bedrock but there is some evidence that the depth to the surface of the bedrock may be quite deep. One of the references quoted in this report namely, the investigation carried out for Innisfil Creek Replacement Bridge (GEOCRE 31D-360), which is located about 4.6 km to the south of this project site, extended to a depth of 46 m or to El. 170 m, without reaching the surface of the bedrock.

4.1.1 Foundation Alternatives for the Bridge Replacement

The clayey soil at the site is considered unsuitable to support normal shallow spread footing foundations and, therefore, deep foundations will probably need to be utilized.

Auger press (auger cast) piles and driven concrete piles are not considered to present cost effective, reliable solutions.

Timber piles may be a cost effective solution, but the variable consistency of the silty clay deposit and possible time dependent deterioration of such piles due to environmental conditions would render timber piles an undesirable choice. In addition, MTO will unlikely to permit such a structure to be supported on timber piles.

The use of drilled and cast-in-place concrete (caisson) foundations to support the structure can be feasible. The silty clay is considered to be impervious enough to permit the installation of the caissons without undue difficulties. However, if the surficial sandy silt is present at the site and/or pervious seams/layers within the silty clay, these may render the installation of the caissons somewhat difficult due to water seepage, especially adjacent to a creek. Tentatively, the following resistances can be assigned for caissons extending to about El. 215 m.

Factored Geotechnical Resistance at U.L.S.	=	1,600 kPa
Recommended Axial Resistance at S.L.S.	=	1,100 kPa*

Higher values may be available at greater depths.

*The S.L.S. value is based on an anticipated settlement of about 25 mm.

The use of steel tube piles can also be considered, driven to practical refusal on the very hard/very dense soil (if such deposits exist at greater depths). Under normal circumstances, closed-end steel tube piles filled with concrete are used. For example, a 324 mm x 9.4 mm size steel tube pile can be expected to provide a Factored Axial Resistance at U.L.S. of about 1,400 kN and on Axial Resistance S.L.S. equal to 900 kN. The quoted axial resistance at S.L.S. is based on a maximum settlement value of 25 mm. However, tube piles are not suitable for integral abutment type bridges and as well, since they are higher displacement piles compared to steel H-piles, the use of steel H-piles is considered a somewhat better alternative.

4.1.2 Favourable Foundation Alternative Details

With due consideration of the site subsurface conditions, the use of driven H-piles may be a favourable foundation option for the new bridge. All piles should be installed as per OPSS 903.

The driving of the piles in the field should be monitored by a recognized pile driving formula such as the Hiley Formula. The estimated ultimate resistance of the piles by the Hiley Formula can be calculated by dividing the recommend axial resistance at U.L.S. by a resistance factor of 0.4 as per current MTO practice.

For preliminary design purposes, a geotechnical reaction of 600kN/pile at SLS and a factored geotechnical resistance of 900kN/pile at ULS can be used for HP310x110 steel H-piles driven to about El. 214 m. These resistances include both end bearing and frictional (adhesion) components. If piles are driven to more competent soil at greater depths, greater resistances such as a Factored Geotechnical Resistance at ULS of up to 1,800 kN/pile and a Geotechnical Reaction of 1,200 kN/pile may possible to utilize for HP 310 x 110 steel H-piles. The S.L.S. resistances are based on a maximum settlement of 25 mm. However, with the present information no prediction can be made as to how deep the piles will have to be driven for higher resistances. These resistances should be re-assessed once detail geotechnical investigation at the bridge location is carried out. Inducing additional stress on the massive clayey soil at the site (e.g. widening or grade raise, etc.) will cause downdrag forces on the piles and these downdrag forces will need to be considered in the design of the piles. Such forces will need to be considered as an external load as per Canadian Highway Bridge Design Code (CHBDC) S6-06.

During the driving process, piles which have already been driven will need to be monitored to determine if they are heaving due to the effects of driving of adjacent piles. If this phenomenon occurs, the affected piles will need to be re-driven. Retapping, to check that relaxation has not occurred, will be necessary in accordance with MTO procedures. Furthermore, it may be necessary to stagger the driving of the piles.

The minimum spacing between the piles should be chosen with due consideration of pile lengths.

The lateral resistance of the piles can be assessed once detail subsoil conditions at the site become available. If required, lateral loading can be supported by horizontal components of battered piles. In this instance, we recommend that the batter be limited to no more than 4:1, as in practice greater batter would be difficult to install. However, since the use of integral abutments is likely to be preferred, the use of battered piles to resist horizontal forces will unlikely be feasible.

If widening of the embankment at the site is proposed, oversize materials (e.g. greater than 75 mm nominal diameter) should not be used in the embankment fills through which piles would be driven.

Integral abutments can be considered for the new bridge. In accordance with MTO requirements (MTO Structural Office Standard), piles for integral abutments require a 3 m long flex zone.

MTO Structural Office requirements (Report SO-96-01) indicate that the flex zone can be provided by augering a 600 mm diameter hole 3000 mm deep and filling with uniform sand. A special provision should be included in the contract specifying the supply and installation of the CSP's, including the gradation of the sand. The required gradation of the uniform sand is presented in the following Table.

Table 4.1.1
Sand Gradation required for the Flex Zone

Sieve Size	Percentage Passing
2 mm	100 %
600 μ m	80-100 %
425 μ m	40-80 %
250 μ m	4-25 %
150 μ m	0-6 %

4.2 Lateral Earth Pressures

Backfill behind abutments 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' Type I or Type II, with minus 0.075 mm sieve size material not exceeding 5%) 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 static 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/m³

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/m³

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.

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 Canadian Highway Bridge Design Code (CHBDC S6-06). 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 CHBDC.

For unrestrained wing walls (if any), the intermediate earth pressure coefficient K_b 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 CHBDC Commentary can be consulted. K^* is typically used when the retaining structure is supported on unyielding foundations, such as spread footings on bedrock. This is very unlikely to be the case for this project. We recommend however that where the lateral yield of the retaining structure may render the use of active soil pressure (i.e. the use of K_a may be possible), the intermediate pressure coefficient K_b be adopted to allow for future changes in the pressure distribution due to vibrations induced by the highway traffic.

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

4.3 Approach Embankments

If additional stress on the massive clayey soil at the site (e.g. widening or grade raise etc.) is to be induced, settlement of the approach embankment will need to be assessed. If calculated settlements are excessive, either surcharging or preloading will need to be considered with or without wick drains for the approach embankments. In the case of embankment widening, new fill should be properly placed on the existing embankment as per OPSD 208.01. All organics and softened material should be stripped from the plan limits of the immediate approach embankment prior to placement of any fill.

With the present site conditions, it is unlikely that the road grade at the bridge site will be raised by more than 0.3 m. However, existing embankments will likely be widened, thus causing settlements.

For a 2 to 3 m grade raise over and above the o.g. levels, a settlement of the order of 50 mm can be expected, based on currently available data. Such settlements will also cause a downdrag on the piles, which will need to be carefully considered and this may render preloading/surcharging with or without wick drains necessary.

While Record of Borehole Sheets by AMEC for the adjacent site does not indicate such a condition, it is our experience that the presence of rather thick organic and/or otherwise unsuitable soils can be expected in

the low areas, especially adjacent to water courses. This may require stripping beyond normal embankment construction. If however all organic and otherwise unsuitable soils are removed (stripped) as per MTO standards, we do not anticipate embankment slope instability due to foundation conditions, as was mentioned before, for grade raise not exceeding 3 m over the o.g. grades.

4.4 Construction

All excavations, shoring and backfilling should be carried out in conformance with the Occupational Health and Safety Act (OHSA), as well as the following specifications.

- OPSS 539 Construction Specification for Temporary Protection System
- OPSS 902 Construction Specification for Excavation and Backfilling – Structures

Excavations within the existing fill and native soils should be possible using heavy equipment such as a hydraulic excavator.

Extent of dewatering and unwatering should be assessed carefully with consideration of groundwater and creek water conditions at the time of construction. If required, a cofferdam should be implemented to maintain a relatively dry work platform for the proposed bridge construction.

In the case of staged construction (if full road closure is not permitted), temporary shoring will be required to support the excavations. In Ontario, shoring typically consists of soldier pile and timber lagging or sheet piling (with or without bracing / rakers). The shoring system should be designed so that the lateral movement of any portion of the roadway protection system will not exceed the established criterion for the structural performance level. In this instance, the anticipated performance level is 2. The shoring system should be designed by a Professional Engineer, experienced in this type of work. We will be pleased to assist temporary support design once the details are known.

4.5 Scour Protection

We recommend that creek and bridge scour protection and erosion control be designed by an experienced Hydraulic Engineer. Based on our visual observations, the Creek appears to be undergoing meandering and causing some significant erosion adjacent to the existing bridge (hence the existing sheet pile wall on the north-west corner area of the existing bridge).

4.6 Frost Protection

Design frost protection depth for the general area is about 1.5 m. Therefore, a permanent soil cover of about 1.5 m or its thermal equivalent of artificial insulation is required for frost protection of foundations, including pile caps. In case of rockfill, only one-half of the rockfill thickness should be assumed to be effective in providing frost protection.

4.7 Seismic Design

Seismic analysis is not required for single span bridges regardless of seismic performance zone except for single span truss bridges as per Clause 4.4.5.2 of CHBDC CAN/CSA-S6-06.

5 RECOMMENDATIONS FOR DETAILED FOUNDATION INVESTIGATION

Our proposal, (dated May 4, 2012) for the detail investigation entails eight boreholes for the structure and two boreholes for the approaches. This Technical Memorandum is intended for planning purposes only and that a Detail Design Foundations investigation and Design Report will be required for Detail Design.

Some of the salient features for the detailed foundation investigation are as follows;

- Field vane tests (MTO N vane or smaller vane for higher shear strength than 200 kPa) to be carried out during borehole investigation;
- Consolidation characteristics of the clayey soil to be investigated; and,
- Consideration can be also given to the use of CPT (Cone Penetration Test), especially in the weak clayey soil (with allowance of dissipation tests).

6 CLOSURE

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

For and on behalf of Coffey Geotechnics Inc.


Gwangha Roh, Ph.D., P.Eng.
Senior Geotechnical Engineer

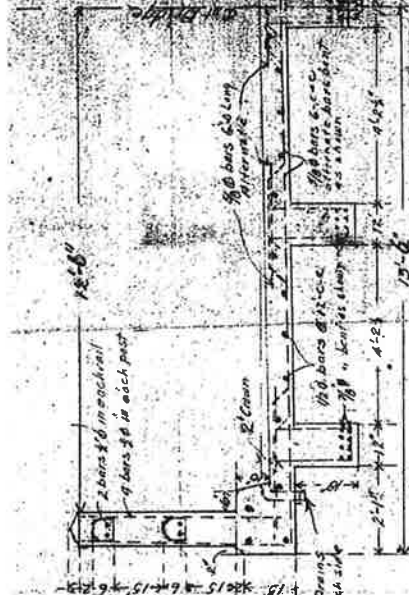



Zuhtu Ozden, P.Eng.
Senior Principal



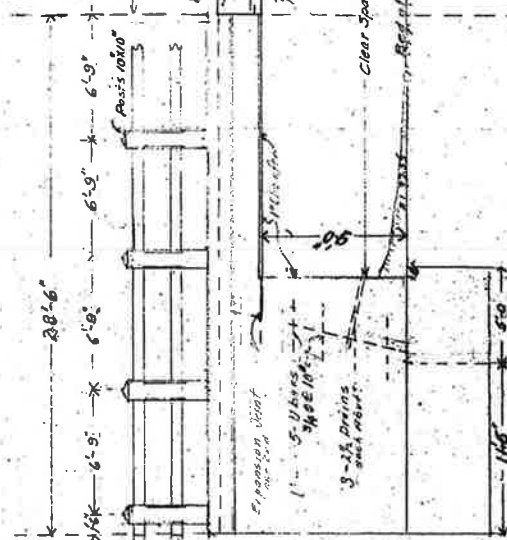
Appendix E

GA Drawings

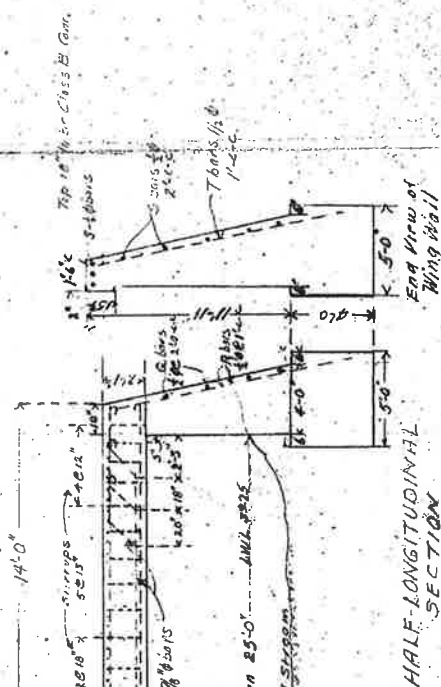


HALF CROSS-SECTION
SCALE - 1" = 1'

NOTE - Structure built in accordance with the D.H.O. Specifications for Highway Bridges 1933.
Depth of Features subject to revision by the Engineer.
Concrete bridge to be placed at all bearings between piers, in mass concrete, to be finished.
Only washed, screened gravel shall be used.
Piers and abutts shall be poured continuously.



HALF-LONGITUDINAL SECTION
SCALE 1"=1'



End View of
Wing Mail

COUNTY OF SIMCOE
GLASS'S BRIDGE
BRIDGE ON ROAD NO. 3
LOT 51-5 CON. 1
T⁹ OF INNISFIL
Longing Claims "H-15"

W.A. Spellman, B.A.S.
Consulting Eng.
June 1963. Barrie

Appendix F

Advantages, Disadvantages, Costs and Risks/Consequences of Foundation Alternatives

Table F-1

Foundation Options for New Bridge Over Innisfil Creek

Foundation Type	Advantage/ Disadvantage	Risks/Consequences	Relative Costs	Recommendations
Shallow foundations	-Low cost/not suitable for integral abutments	-Prone to settlements	Low cost	-Unlikely to be feasible due to the prevailing subsurface conditions
Driven steel H-piles foundations	-Low displacement piles and as such more suitable than other types of driven piles such as precast concrete or steel tube piles -Suitable for integral abutments.	-Vibration and noise	Moderate cost	-Favourable
Driven Steel Tube Pile Foundations	-Similar to driven steel H-piles, except they are higher displacement piles in comparison with H-piles and are unsuitable for integral abutments.	-Similar to driven steel H-piles except possible greater vibration generation	Moderate Cost	-Less favourable than H-piles
Timber Piles	-Low cost but less reliable than steel piles due to environmental reasons.	-May be subject to deterioration in time.	Low	-Unlikely to be acceptable to MTO, not recommended.
Drilled and cast-in-place Concrete piles (drilled caissons) foundations	-Less vibrations and noise created than driven piles. -Not suitable for integral abutments	-Possible hole instability issues	More expensive than driven piles	-Feasible option but less suitable for the prevailing subsurface conditions from Geotechnical point of view in comparison with steel H-piles
CFA (continuous flight auger pile)	-Not suitable with the available information -Does not present an advantage	-May be considered if a water bearing soil is encountered underlying the silty clay deposit	High cost	-Not recommended

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 Coffey Geotechnics Inc. (Coffey) at the time of preparation. Unless otherwise agreed in writing by Coffey, 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. Coffey accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.