

DOCUMENT MICROFILMING IDENTIFICATION

G.I.-30 SEPT. 1976

GEOCRES No. 30M11-203

DIST. CR REGION

W.P. No. 174-00-01

CONT. No.

W. O. No.

STR. SITE No. 37-1522

HWY. No. QEW

LOCATION BRIDGE NB-4 RAMP FGGE E-  
BROWN'S LINE S OVER BROWN'S LINE SouthBound

No. of PAGES -

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OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

**FOUNDATION INVESTIGATION REPORT  
PROPOSED BRIDGE NB-4  
RAMP FGGE E - BROWN'S LINE S  
OVER BROWN'S LINE SOUTHBOUND  
TORONTO, ONTARIO  
W.P. 174-00-01  
SITE 37-1522**

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**Project: SP3232D  
June 8, 2000**

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**FOUNDATION INVESTIGATION REPORT  
PROPOSED BRIDGE NB-4  
RAMP FGGE E BROWN'S LINE S  
OVER BROWN'S LINE SOUTHBOUND  
TORONTO, ONTARIO  
W.P. 174-00-01  
SITE 37-1522**

**1. INTRODUCTION**

As part of the Gardiner Expressway/Q.E.W./Hwy. 427/Brown's Line Interchange modifications, a new bridge is to be constructed which will carry proposed ramp FGGE E, Brown's Line S over Brown's Line South. Shaheen & Peaker Limited (S&P) was retained by DS-Lea Associates Limited, Consulting Engineers, to carry out a foundation investigation for the proposed bridge structure.

The site is located between the Queensway and Gardiner Expressway (FGGE) at the Brown's Line interchange in south-west Toronto.

The purpose of the investigation was to obtain information about the subsurface conditions at the site by means of boreholes.

The findings of the investigation are presented in this report.

**2. PHYSIOGRAPHY**

The site is located in the physiographic region known as the Iroquois Plain, which comprises the lowlands bordering Lake Ontario. During the retreat of the Late Wisconsinian glaciers, this area was occupied by Lake Iroquois. The shoreline of Lake Iroquois corresponded with approximately Dundas Street, which is located about 2 km to the north. During the time Lake Iroquois was present, a large delta formed at the mouth of the former Humber River. This is located about 5 km to the east.

The geological mapping for this area shows that the surficial deposits at this site consist of deltaic and shallow water lacustrine deposits comprised primarily of gravelly sand and silty sand. The stratigraphy for the site indicates that the deltaic and lacustrine deposits were deposited during the Late Wisconsinian

times by glaciers advancing towards the northwest out of the Lake Ontario Basin. The predominant overburden in the area consists of a glacial till deposit known as the Halton till. This deposit contains frequent shale and limestone fragments.

Bedrock consisting of shale with interbedded limestone and sandstone underlies the Halton Till. Available information indicates that the surface of the bedrock of the site can be expected at about Elevation 108.5 m.

### 3. INVESTIGATION PROCEDURES

The fieldwork for the project was performed on February 10 and 26, 2000 and consisted of drilling and sampling four boreholes (Boreholes B6-1 through B6-4) at the locations shown on Drawing No. 2. The depth of the boreholes ranged from 2.4 to 8.7 m below the ground surface.

The boreholes were advanced using track and truck mounted drilling rigs, equipped with solid stem augers and standard testing equipment, under the full time supervision of technical personnel from S&P. Sampling in the boreholes was effected at frequent intervals of depth (i.e. generally at 0.76 m intervals of depth, starting at the ground surface) by the Standard Penetration Test Method (SPT), as specified in ASTM Method D 1586. This consists of freely dropping a 63.5 kg hammer a vertical distance of 0.76 m to drive a 51 mm diameter O.D. split barrel (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 and this gives an indication of the consistency or the compactness condition of the soil deposit.

Boreholes B6-1, B6-3 and B6-4 were extended until refusal in the bedrock at depths of 5.6, 4.0 and 2.4 m, respectively, probably on hard (limestone) layers in the shale bedrock. In Borehole B6-2 augering was stopped at 1.5 m. This borehole and Borehole B6-3 were extended by diamond drilling (rock coring), using NQ-size core barrel to depths of 6.3 and 8.7 m, respectively.

The soil and rock samples and rock cores were shipped to our laboratory in Toronto for further examination and classification. A laboratory testing

programme, consisting of natural moisture content, bulk unit weight and Atterberg Limits tests and grain-size analyses, was performed on selected, representative soil samples. The results of the laboratory tests are presented on the appropriate Borehole Log sheets and also in Appendix B.

Groundwater conditions in the open boreholes were observed during the drilling and at completion of each borehole. In addition, in each of Boreholes B6-1 and B6-4, a piezometer was installed to enable us to monitor the groundwater level over a prolonged period of time, without interference from surface water.

The borehole locations were established in the field by our engineering staff, in relation to the centerline stakes. The borehole geodetic elevations and coordinates were later taken and provided to us by surveyors from Waylaw Technical Services of Paris, Ontario.

#### **4. SUBSURFACE CONDITIONS**

The subsurface conditions were explored at four borehole locations (Boreholes B6-1 through B6-4). The locations of the boreholes are shown on the Borehole Locations and Profile Drawing No. 2 and are also indicated on the individual borehole log sheets (Appendix A). A profile section of inferred subsurface stratigraphy is given in the same drawing.

Boreholes B6-1 and B6-3 were drilled from the existing embankments where ground surface elevations were 112.1 and 109.4 m, respectively. These boreholes contacted 225 and 400 mm topsoil, followed by fill to a depth of 2.1 m (Elevation 110.0 m) in Borehole B6-1. Underlying the topsoil and fill (Borehole B6-1), clayey silt till and till/shale overburden was contacted in both boreholes extending to 3.3 m and 1.6 m or to Elevations 108.8 and 107.8 m, respectively, to the surface of bedrock.

Boreholes B6-2 and B6-4 were drilled from within the road allowance and contacted 216 mm of concrete and 125 mm of asphalt, respectively, followed by granular pavement fill to 0.45 m to the surface of bedrock.

The fill and natural overburden in Boreholes B6-1 and B6-3 and the pavement fill in Boreholes B6-2 and B6-4 are underlain at depths ranging from 0.45 m to 3.3 m or below Elevations 108.8 m to 107.1 m by shale bedrock to the full depth of the exploration.

Details of the subsurface conditions encountered in the boreholes are presented on the Borehole Log Sheets in Appendix A. A description of the various strata encountered in the boreholes is given in the following paragraphs.

#### 4.1 TOPSOIL

Boreholes B6-1 and B6-3 contacted approximately 225 and 400 mm of topsoil, respectively.

#### 4.2 PAVEMENT

Boreholes B6-2 and B6-4 were drilled from the sidewalk and the paved road surface and contacted 216 mm thick concrete and 125 mm of asphaltic concrete, respectively, followed by granular pavement fill to 0.45 m or to Elevations 108.1 and 107.1 m, respectively.

#### 4.3 FILL

Underlying the topsoil, Borehole B6-1 contacted embankment fill consisting of a heterogeneous mixture of sandy silt, clayey silt and silty sand with some rock (mostly shale) fragments. Based on N-values of 3 to 35 blows/0.3 m the fill does not appear to have received a systematic compaction.

It should be pointed out that the thickness of topsoil and the composition and thickness of fill could vary in between and beyond the borehole locations.

#### 4.4 CLAYEY SILT TILL

Underlying the fill (B6-1) and topsoil (B6-3) Boreholes B6-1 and B6-3 contacted a clayey silt to silty clay till deposit at depths of 2.1 m (Elevation 110.0 m) and 0.4 m (Elevation 109.0 m), respectively. The till consists of a heterogeneous mixture of clayey silt to silty clay with some sand and gravel. The grain size

distribution of typical samples from the deposit is given in Appendix B. The deposit contains shale and some limestone fragments, the frequency of which increases with increasing depth. The presence of cobbles and boulders can always be expected in the glacial till deposits.

Atterberg Limits tests performed in the laboratory on two samples from the deposit yielded the following index values.

Liquid limit = 25%

Plastic limit = 18-19%

Plasticity Index = 6-7%

Natural moisture content = 7-15%

These values are characteristic of clayey soils of low plasticity. The fact that the measured natural moisture contents are below the measured plastic limit value indicate that the material is over consolidated.

Standard Penetration tests performed in the deposit yielded N-values which ranged from 19 to in excess of 75 blows/0.3 m, indicating a very stiff to hard (generally hard) consistency.

The till is a cohesive material.

#### 4.5 TILL/SHALE COMPLEX

With increasing depth, the frequency of the shale and limestone fragments in the till increases and the lower portions of the till often resembles a highly weathered shale, a material which is sometimes referred to as a till/shale complex. It represents a transition zone into the underlying shale bedrock. This unit is often described as a residual soil or a completely, weathered shale bedrock. Shale and limestone slabs (layers) may remain. Contractors are advised that excavation methods should take into account the possible presence of hard shale or limestone slabs/layers in the till, till/shale and in the underlying shale bedrock.

The till/shale complex was contacted in Borehole B6-1 and B6-3 at approximate depths of 2.6 m (Elevation 109.5 m) and 1.3 m (Elevation 108.1 m) and



extended to the bedrock surface at about 3.3 m (Elevation 108.8 m) and 1.6 m (Elevation 107.8 m), respectively.

The till/shale complex is a cohesive material and based on the resistance to augering during drilling and the recorded N-values, its consistency is described as hard.

#### 4.6 SHALE BEDROCK

Shale bedrock was encountered in all the four boreholes at the following approximate depths/elevations.

**Estimated Bedrock Depth/Elevation**

Borehole No.	Bedrock Depth (m)	Elevation (m)
B6-1	3.3	108.8
B6-2	0.5	108.1
B6-3	1.6	107.8
B6-4	0.5	107.1

The surface of the shale bedrock should be regarded as approximate only; this is because the depth to the surface of the bedrock was often inferred from the observed resistance to augering and, where possible, from split-spoon samples and auger cuttings.

The bedrock underlying the site belongs to the Georgian Bay Formation (also known as the Dundas-Meaford Formation) of the Upper Ordovician Period of the Paleozoic Era. The Georgian Bay Formation is approximately 450 million years old and consists of grey shale with interbeds of relatively more competent siltstone and sandstone and harder limestone. It is known to contain occasional thin clay seams. The hard layers/seams are usually less than about 100 to 150 mm thick but some layers are much thicker. These are actually lenses and they can vary significantly in thickness over short distances. Stress relief features, such as folds and faults are also found in the Georgian Bay Formation. In these features, the rock is heavily fractured and sheared, and contains layers of shale rubble and clay.

The presence of occasional limestone, siltstone and sandstone seams was noted in the cores obtained from the bedrock in Boreholes B6-2 and B6-3. They were generally 2 to 16 cm thick except for a 400 mm thick layer in Borehole B6-3 at 4.0 m depth or Elevation 105.4 m. In Boreholes B6-1 and B6-4, where the rock was not cored, the bedrock was penetrated by augering until refusal to augering was encountered at depths of 5.6 m (Elevation 106.5) and 2.4 m (Elevation 105.2), respectively, probably on the surface of hard layers (e.g. limestone layers/lenses). From this, it can also be inferred that the bedrock at these two locations does not contain significantly hard layers above these elevations.

#### 4.7 GROUNDWATER CONDITIONS

Groundwater conditions in the open boreholes were observed during the drilling and at the completion of augering in each borehole.

In Borehole B6-3, water seepage into the borehole was noted at 3.9 m or at Elevation 105.5 m.

In Boreholes B6-2 and B6-3, where rock coring was effected, water was introduced into the boreholes to facilitate diamond drilling. In these boreholes, therefore, stabilized water levels could not be measured.

To enable us to monitor the groundwater level over a prolonged period of time without interference from surface water, a piezometer was installed on each of Boreholes B6-1 and B6-4. Water levels in the piezometers were monitored over a period of time and the recorded values are shown on the borehole logs. The final water levels were recorded at 4.0 m (Elevation 108.1 m) and 0.8 m (Elevation 106.8 m) in Boreholes B6-1 and B6-4, respectively.

It should be pointed out that the groundwater table would be subject to seasonal fluctuations and in response to major weather events.

**Shaheen & Peaker Limited**

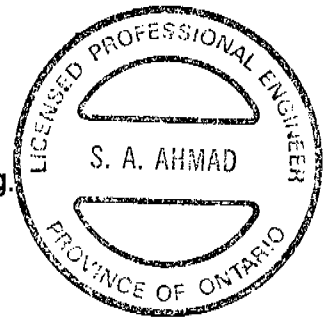


Zuhtu Ozden, P.Eng.



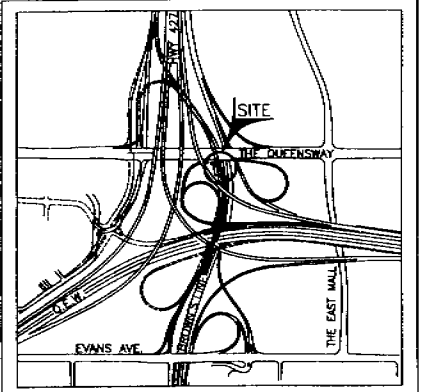
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for Shaheen A. Ahmad, M.A.Sc., P. Eng.





Shaheen & Peaker Limited



LEGEND

- Bore Hole
- ⊕ Dynamic Cone Penetration Test (Cone)
- ⊕ Bore Hole & Cone
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60' Cone, 475 J/blow)
- W L at time of investigation Feb. 2000
- W L in Piezometer
- Piezometer

No	ELEVATION	CO-ORDINATES	
		NORTH	EAST
B6-1	112.1	4 830 598.7	300 782.4
B6-2	108.6	4 830 618.0	300 796.7
B6-3	109.4	4 830 577.4	300 794.8
B6-4	107.6	4 830 583.0	300 811.8

NOTE

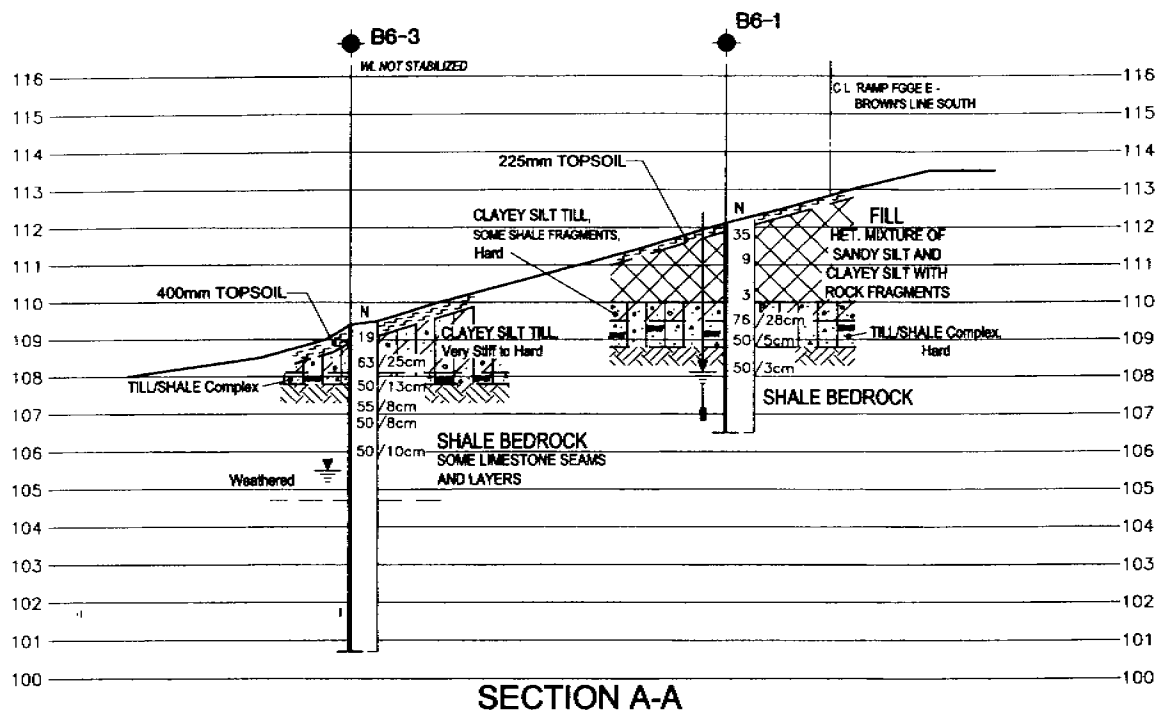
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section GC 2.01 of OPS Gen. Cond.

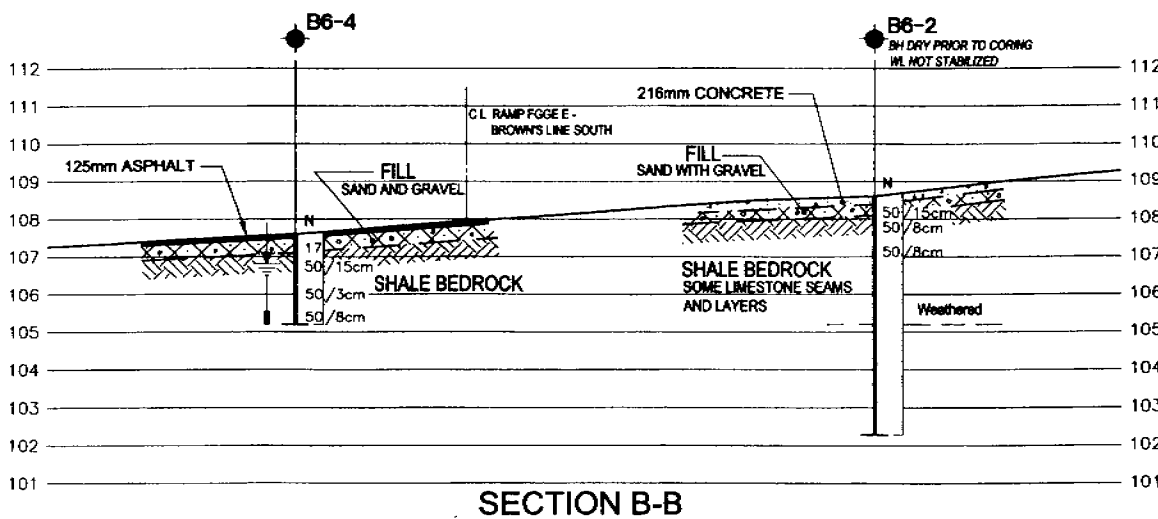
REV.	DATE	BY	DESCRIPTION
Geocres No			
PRJY No	427 & QEW	DIST	
SUBM'D ZO	CHECKED ZO	DATE	May, 2000
DRAWN JTW	CHECKED JP	APPROVED	SITE 37-1522
			DWG 2

METRIC

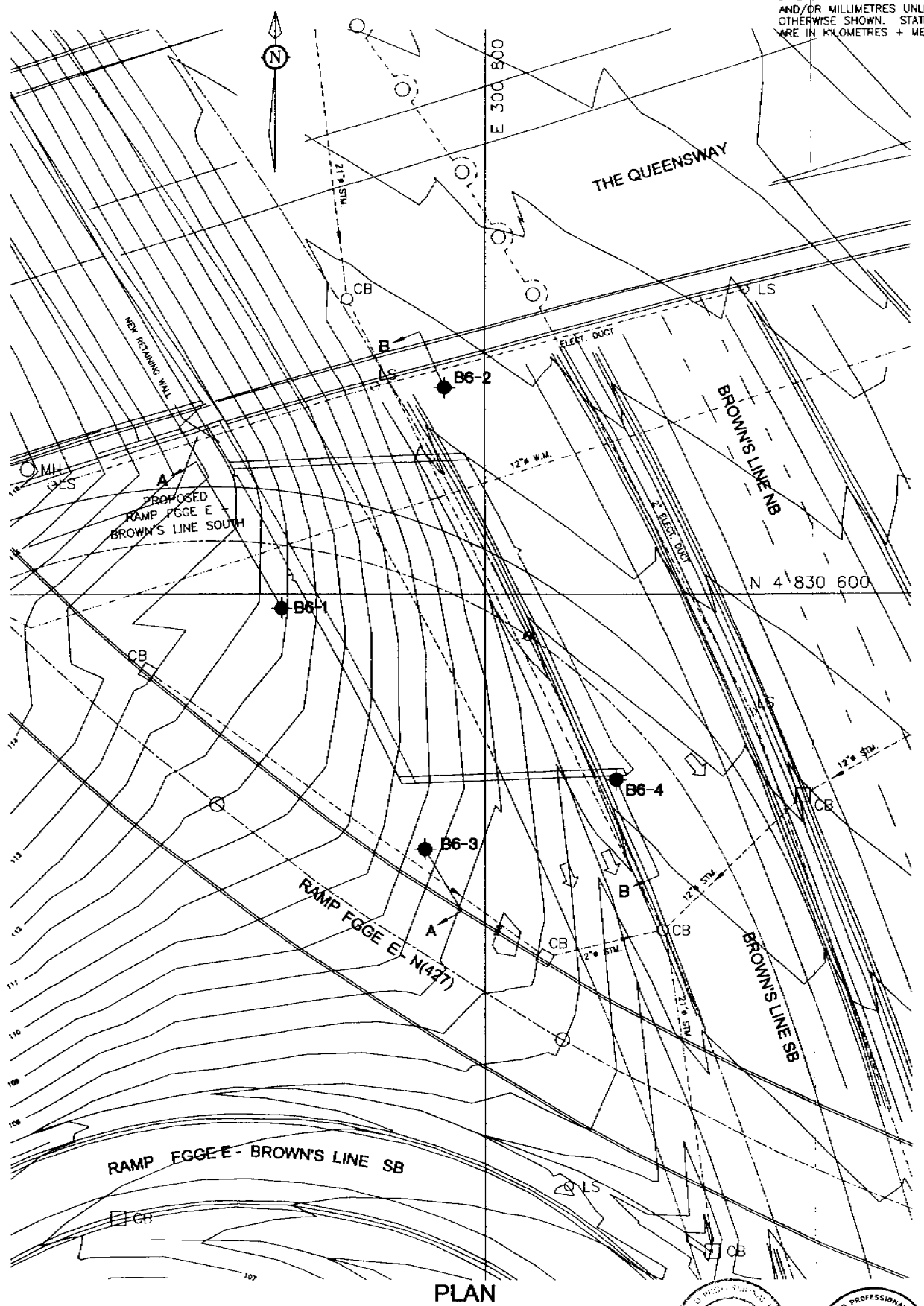
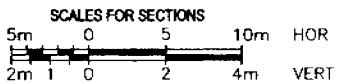
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES + METRES.



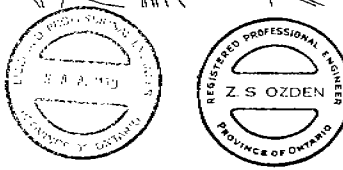
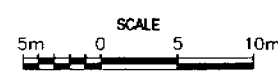
SECTION A-A



SECTION B-B



PLAN



# APPENDIX A

## Borehole Log Sheets

# RECORD OF BOREHOLE No B6-1

1 OF 1

METRIC

W.P. 174-00-01 LOCATION Ramp FGGE(E)-Brown's Line S; 4 830 598.7 N; 300 782.4 E ORIGINATED BY G.W  
DIST HWY 427 & QEW BOREHOLE TYPE Solid Stem Augers COMPILED BY G.T  
DATUM Geodetic DATE 18.02.00 CHECKED BY Z.O

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT  w <sub>p</sub>	NATURAL MOISTURE CONTENT  w	LIQUID LIMIT  w <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED    + FIELD VANE ● QUICK TRIAXIAL    × LAB VANE									
112.1	Ground surface							20	40	60	80	100					
111.9	225 mm Topsoil		1	SS	35		112							○			
0.2	FILL: Heterogeneous mixture of Sandy Silt & Clayey Silt with rock fragments, brown & grey		2	SS	9		111							○			21.0
			3	SS	3		110							○			
110.0			4	SS	76/28		109							○	H		21.7
2.1	CLAYEY SILT TILL: some shale fragments, grey, hard						108										
109.5	TILL/SHALE Complex grey, hard						107										
2.6																	
108.8																	
3.3	SHALE BEDROCK grey																
106.5																	
5.6	End of borehole Refusal to further augering @ 5.6 m probably on a limestone layer Borehole dry on completion Piezometer installed to 5.3 m Water level in piezometer Feb.20 - 4.2 m Feb.24 - 4.0 m March.2 - 4.0 m																

+ 3 . x 3 : Numbers refer to  
Sensitivity

20  
15 5  
10 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No B6-2

1 OF 1

METRIC

W.P. 174-00-01 LOCATION Ramp FGGE(E)-Brown's Line S: 4 530 618.0 N: 300 796.7 E ORIGINATED BY G.I  
DIST HWY 427 & QEW BOREHOLE TYPE Solid Stem Augers, NQ Rock Core COMPILED BY G.T  
DATUM Geodetic DATE 10.02.00 26.02.00 CHECKED BY Z.O

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		20	40	60	80	100				
108.6	Ground surface														
108.4	216 mm Concrete		1	AS											
108.1	FILL: Sand with gravel		2	SS	50/15										
0.5			3	SS	50/8										
			4	SS	50/8										
			5	NQ RC	Rec. 80%										
	SHALE BEDROCK grey some limestone seams and layers weathered		6	NQ RC	Rec. 90%										R.Q.D=23%
			7	NQ RC	Rec. 100%										R.Q.D=25%
102.3															R.Q.D=85%
6.3	End of borehole Water level not stabilized														

+ 3, x 3: Numbers refer to  
Sensitivity

20  
15 5  
10 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No B6-3

1 OF 1

METRIC

W.P. 174-00-01 LOCATION Ramp FGGE E - Brown's Line S; 4 830 577.4 N; 300 794.8 E ORIGINATED BY G.I.  
DIST HWY 427 & QEW BOREHOLE TYPE Solid Stem Augers & NQ Rock Core COMPILED BY G.T.  
DATUM Geodetic DATE 16.02.00 CHECKED BY Z.O.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
109.4	Ground surface													
0.0 109.0	400 mm Topsoil		1	SS	19		109							
0.4	CLAYEY SILT TILL: grey, brown, very stiff to hard		2	SS	63/25		108							11 45 27 17
108.1														
107.8	TILL/SHALE Complex		3	SS	50/13		107							
1.6			4	SS	55/8									
			5	SS	50/8		106							
			6	SS	50/10									
	SHALE BEDROCK grey some limestone seams and layers weathered		7	RC	100%		105							Refusal to further augering @ 4.0 m on a 400 mm limestone layer R.Q.D.=0%
			8	NQ RC	Rec. 93%									R.Q.D.=50%
			9	NQ RC	Rec. 85%		104							R.Q.D.=46%
			10	NQ RC	Rec. 100%		103							R.Q.D.=72%
			11	NQ RC	Rec. 98%		102							R.Q.D.=51%
100.7							101							
8.7	End of borehole Water encountered @ 3.9 m while drilling Water level not stabilized													

+ 3 . x 3 : Numbers refer to  
Sensitivity

20  
15  
10  
(%) STRAIN AT FAILURE



# RECORD OF BOREHOLE No B6-4

1 OF 1

METRIC

W.P. 174-00-01 LOCATION Ramp FGGE(E)-Brown's Line S; 4 830 583.0 N; 300 811.8 E ORIGINATED BY M.A.  
DIST HWY 427 & QEW BOREHOLE TYPE Solid Stem Augers COMPILED BY G.T.  
DATUM Geodetic DATE 26.02.00 CHECKED BY Z.O.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
107.6	Ground surface																
107.5	125 mm Asphalt			AS													
107.1	FILL; Sand and gravel		1	SS	17												
0.5	SHALE BEDROCK grey		2	SS	50/15		107										
			3	SS	50/3		106										
105.2			4	SS	50/8												
2.4	End of borehole Refusal to augering @ 2.4 m, probably on a limestone layer Water level @ 1.4 m upon completion Piezometer installed to 2.4m Water level in piezometer Feb. 28 - 0.8 m March 2 - 0.8 m																

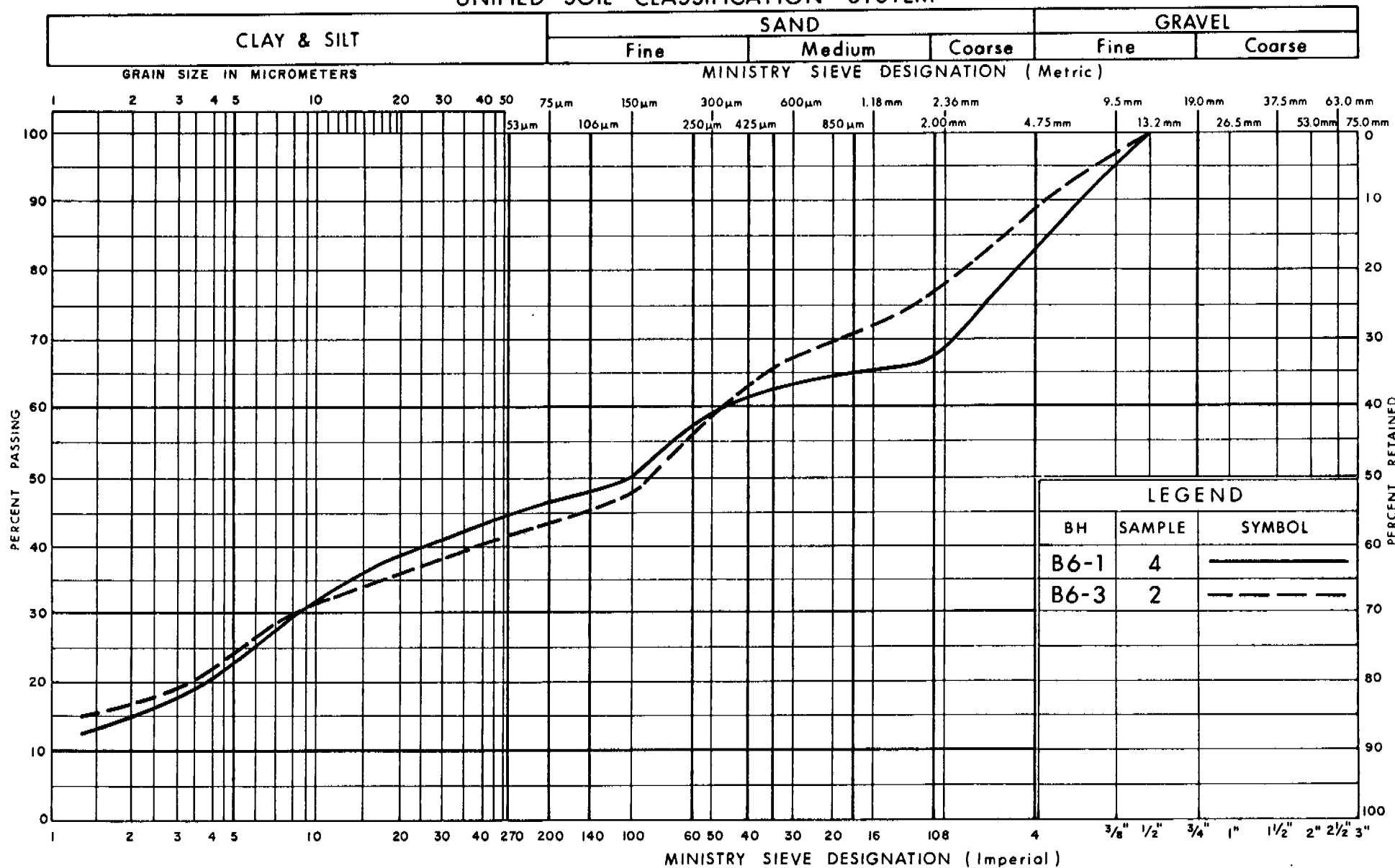
+ 3, x 3 : Numbers refer to  
Sensitivity

20  
15 5  
10 (%) STRAIN AT FAILURE

# APPENDIX B

## Laboratory Test Results

## UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of  
Transportation

## GRAIN SIZE DISTRIBUTION CLAYEY SILT TILL

FIG No 1

W P 174 - 00 - 01

SP 3232D

## APPENDIX C

# Rock Core Logs and Photographs

# CORE LOG

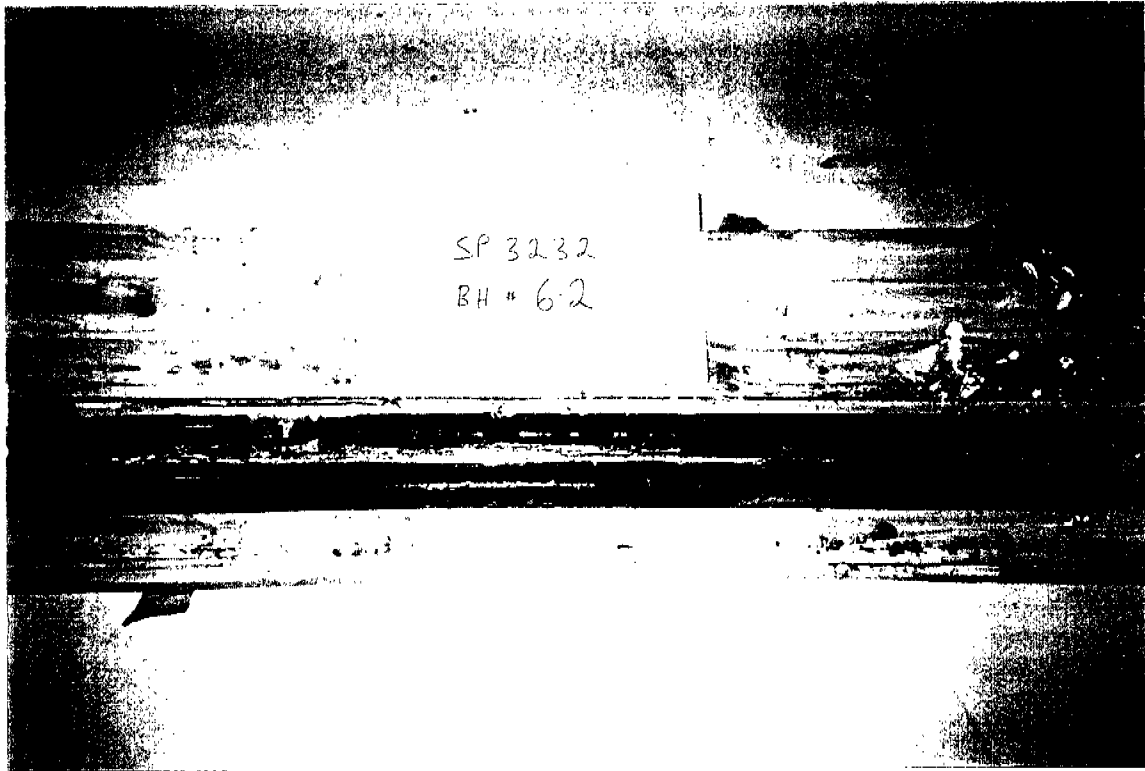
## BH NO B6-2

PROJECT Foundation Investigation	ORIENTATION Vertical	ELEVATION (m) 108.6	DATUM Geodetic	PROJECT NO. SP3232
LOCATION Q.E.W./Hwy 427/Brown's Line Interchange Modification	DATE STARTED 02/10/00	COMPLETED 02/26/00	LOGGED BY E.P.	DRAWING NO.
CLIENT	DRILLER Groundworks	DRILL TYPE CME 75	CORE BARREL NQ	SHEET 1 of 1

ELEV. (m)	DEPTH (m)	SYMBOL	GENERAL DESCRIPTION	JOINT CHARACTERISTICS							WEATHERING	STRENGTH	FRACTURE FREQUENCY	RUN No.	RECOVERY %	ROD	WATER RECOVERY %	WATER COLOUR
				No. OF SETS	JOINT TYPE	ORIENTATION	SPACING	ROUGHNESS	FILLING	APERATURE (mm)								
108.6	108.1	1	OVERBURDEN: see soils log for description															
106.9		2	SHALE, sampled with split-barrel sampler; see soils log for description															
		3	GEORGIAN BAY FORMATION: Shale with Interbedded Limestone and Sandstone	2	B C	F V	C VC	SP RP	T	0				1	80	23	100	grey
		4	Shale (75.5%) thinly bedded or laminated, dark grey, slightly to moderately weathered in sections, low strength															
		5	Limestone (11%) fine grained, fossiliferous, shaley in some parts, unweathered, high strength			V								2	90	25	100	grey
		6	Sandstone and Siltstone (13.5%) stratified, brownish grey to medium grey, unweathered, moderate strength											3	100	85	100	grey
102.3		7	Discontinuities: bedding joints are at close to very close intervals, the maximum thickness of limestone or sand/siltstone layers was about 180 mm, joint surfaces are smooth planar to rough planar, occasional vertical joints															
		8	Core broken up to 2.1 m															
		9	Rubble seams noted at 2.58 m (10 mm), 2.68 m (10 mm), 3.09 m (20 mm), 3.13 m (50 mm), 3.64 m (20 mm), and 5.03 m (25 mm)															
		10	Clay seams noted at 2.54 m (10 mm) and 2.74 m (10 mm)															
		11	End of Borehole															
		12																
		13																
		14																
		15																
		16																
		17																
		18																
		19																

### S & P

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Consulting Geo-Environmental Engineers



**Photograph of Core  
Borehole B6-2  
Elev. 106.9 m – 102.3 m**

# CORE LOG

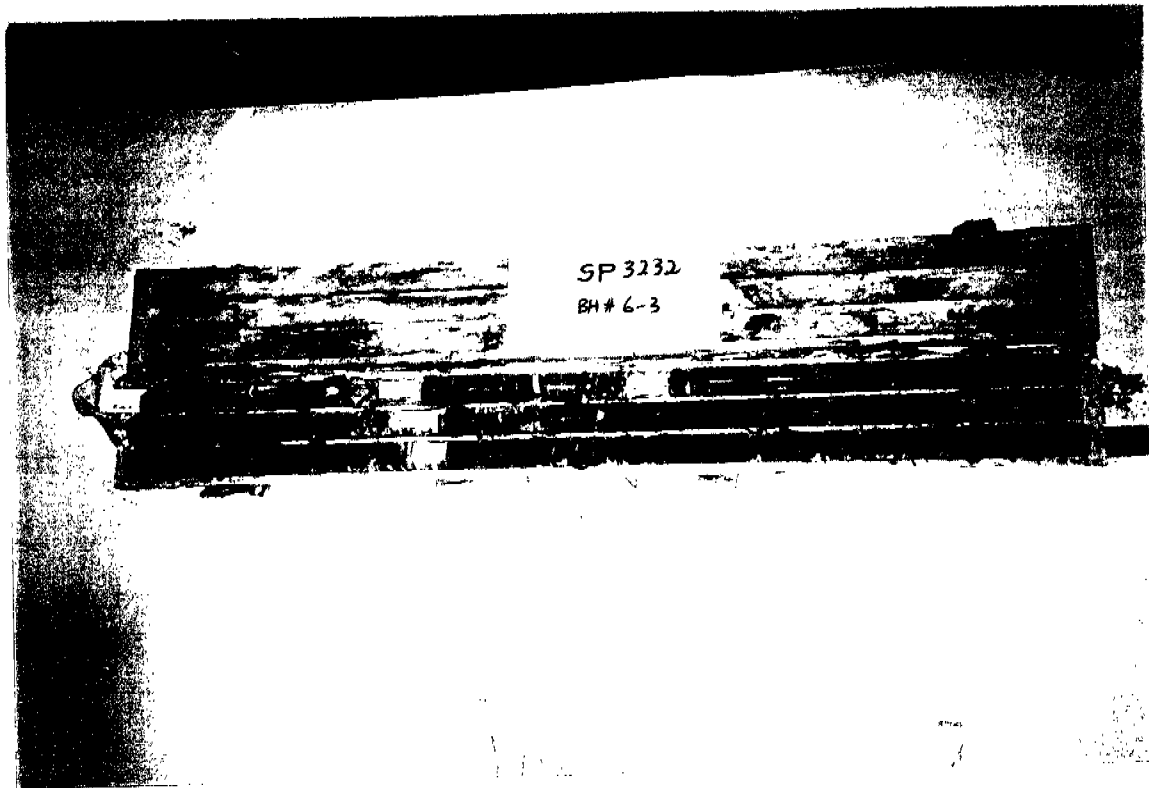
## BH NO B6-3

PROJECT Foundation Investigation	ORIENTATION Vertical	ELEVATION (m) 109.4	DATUM Geodetic	PROJECT NO. SP3232
LOCATION Q.E.W./Hwy 427/Brown's Line Interchange Modification	DATE STARTED 02/16/00	COMPLETED 02/16/00	LOGGED BY E.P.	DRAWING NO.
CLIENT	DRILLER Groundworks	DRILL TYPE CME 75	CORE BARREL NQ	SHEET 1 of 1

ELEV. (m)	DEPTH (m)	SYMBOL	GENERAL DESCRIPTION	JOINT CHARACTERISTICS							WEATHERING	STRENGTH	FRACTURE FREQUENCY	RUN No.	RECOVERY %	RQD	WATER RECOVERY %	WATER COLOUR
				No. OF SETS	JOINT TYPE	ORIENTATION	SPACING	ROUGHNESS	FILLING	APERATURE (mm)								
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
109.4			OVERBURDEN: see soils log for description															
107.8	1																	
	2		SHALE, sampled with split-barrel sampler; see soils log for description															
	3																	
105.5	4		GEORGIAN BAY FORMATION:	1	B	F	C	SP	T	0				1	100	0	100	gr
	5		Shale with Interbedded Limestone and Sandstone				VC	RP						2	93	50	100	gr
	6		Shale (88%) thinly bedded or laminated, dark grey, generally unweathered, low strength											3	85	46	100	grey
	7		Limestone (3%) fine grained, fossiliferous, unweathered, high strength											4	100	72	100	grey
	8		Sandstone and Siltstone (9%) stratified, brownish grey to medium grey, unweathered, moderate strength											5	98	51	100	grey
100.7	9		Discontinuities: bedding joints are at close to very close intervals, the maximum thickness of limestone or sand/siltstone layers was about 400 mm, joint surfaces are smooth planar to rough planar															
	10		Core broken up to 4.37 m															
	11		Clay seams noted at 7.77 m (20 mm), 7.85 m (20 mm), and 8.12 m (45 mm)															
	12		End of Borehole															
	13																	
	14																	
	15																	
	16																	
	17																	
	18																	
	19																	

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Photograph of Core  
Borehole B6-3  
Elev. 105.5 m – 100.7 m



## EXPLANATORY SHEET TO CORE LOG

Column No.	Description
1	Elevation of geotechnical boundary.
2	Depth of geotechnical boundary in borehole.
3	Geological symbol for rock or soil material.
4	General description of geotechnical unit - qualitative description including rock type(s), percentage rock types, frequency and sizes of interbeds, colour, texture, weathering, strength, general joint spacing.
5-11	Joint (discontinuity) characteristics
5	Number of joint sets: a rock mass can be intersected by a number of joint sets of varying orientations.
6	Joint type:      B = Bedding Joint      F = Fault C = Cross Joint      S = Shear Plane
7	Orientation: only variations in dip can be identified in core; dip direction is obtained from field mapping or oriented core. F = Flat      = 0 - 20° D = Dipping      = 20 - 50° V = Vertical      = 50 - 90°
8	Joint spacing: this is an approximate measure of spacing between joints in specific joint sets VW = Very Wide      = 3 m W = Wide      = 1 - 3 m M = Moderate      = 30 cm - 1 m C = Close      = 5 - 30 cm VC = Very Close      = 5 cm
9	Roughness:      RU = Rough Undulating RP = Rough Planar SU = Smooth Undulating SP = Smooth Planar LU = Slickensided Undulating LP = Slickensided Planar
10	Filling:      Approx. Ø <sub>r</sub> T = Tight, hard, non-softening      - O = Oxidation surface staining only      25 - 35 SA = Slightly altered; clay-free      25 - 30 S = Sandy particles, clay-free      25 - 30 Si = Sandy and silty, minor clay      20 - 25 NC = Non softening clays ( 5 mm)      16 - 24 SO = Softening clays ( 5 mm)      12 - 16 SC = Swelling clay fillings ( 5 mm)      6 - 12
11	Aperture:      estimated sizes of joint opening
12	Degree of weathering of rock material: Unweathered      = no signs of discolouration or oxidation Slightly weathered      = partial discolouration; fractures (joints) typically oxidized Moderately weathered      = total discolouration Highly weathered      = total discolouration; typically friable & pitted Completely weathered      = resembles a soil; rock structure usually preserved

Column No.	Description	Approx. Uniaxial Compressive Strength												
13	<p><b>Strength of rock material:</b></p> <p><b>Very high strength</b> = specimen can only be chipped by geological hammer</p> <p><b>High strength</b> = specimen requires a number of blows of geological hammer to fracture it; cannot be scraped with pocket knife</p> <p><b>Medium strength</b> = specimen can be fractured by single firm blow of geological hammer; can be scraped with pocket knife, not peeled</p> <p><b>Low strength</b> = shallow indentations made by firm blow with point of geological hammer; can be peeled by pocket knife with difficulty</p> <p><b>Very low strength</b> = crumbles under firm blow with point of geological hammer; can be peeled by pocket knife</p>	<p>200 MPa</p> <p>50 - 200 MPa</p> <p>15 - 50 MPa</p> <p>4 - 15 MPa</p> <p>1 - 4 MPa</p>												
14	<p><b>Fracture Frequency:</b> Number of natural joints occurring over a metre length of core. All natural joints are counted irrespective of the number of joint sets.</p> <table><tr><th>Fracture frequency</th><th>Joint spacing</th></tr><tr><td>0.3/m</td><td>= Very wide = 3 m</td></tr><tr><td>0.3 - 1/m</td><td>= Wide = 1 - 3 m</td></tr><tr><td>1 - 3/m</td><td>= Moderate = 30 cm - 1 m</td></tr><tr><td>3 - 20/m</td><td>= Close = 5 - 30 cm</td></tr><tr><td>20/m</td><td>= Very close = 5 cm</td></tr></table>	Fracture frequency	Joint spacing	0.3/m	= Very wide = 3 m	0.3 - 1/m	= Wide = 1 - 3 m	1 - 3/m	= Moderate = 30 cm - 1 m	3 - 20/m	= Close = 5 - 30 cm	20/m	= Very close = 5 cm	
Fracture frequency	Joint spacing													
0.3/m	= Very wide = 3 m													
0.3 - 1/m	= Wide = 1 - 3 m													
1 - 3/m	= Moderate = 30 cm - 1 m													
3 - 20/m	= Close = 5 - 30 cm													
20/m	= Very close = 5 cm													
15	<p><b>Run Number and Core Recovery:</b></p> <p>(i) Drill run number;</p> <p>(ii) Core Recovery is the total length of core pieces, irrespective of their individual lengths, obtained in a core run and expressed as a percentage of the length of that core run.</p>													
16	<p><b>Rock Quality Designation (RQD):</b> The total length of those pieces of sound core which are 10 cm or greater in length in a core run expressed as a percentage of the total length of that core run. Sound pieces of rock are those pieces separated by natural breaks and not machine breaks or subsequent artificial breaks.</p> <table><tr><th>RQD</th><th>Rock Mass Classification (After Deere)</th></tr><tr><td>0 - 25 %</td><td>very poor</td></tr><tr><td>25 - 50 %</td><td>poor</td></tr><tr><td>50 - 75 %</td><td>fair</td></tr><tr><td>75 - 90 %</td><td>good</td></tr><tr><td>90 - 100 %</td><td>excellent</td></tr></table>	RQD	Rock Mass Classification (After Deere)	0 - 25 %	very poor	25 - 50 %	poor	50 - 75 %	fair	75 - 90 %	good	90 - 100 %	excellent	
RQD	Rock Mass Classification (After Deere)													
0 - 25 %	very poor													
25 - 50 %	poor													
50 - 75 %	fair													
75 - 90 %	good													
90 - 100 %	excellent													
17	<p><b>Core and Casing Sizes:</b> changes of core and casing sizes are indicated.</p>													
18	<p><b>Water recovery, level and tests.</b></p>													

## APPENDIX D

### Explanation of Terms Used in Report

## EXPLANATION OF TERMS USED IN REPORT

**N VALUE:** THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, 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 475 J 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 (R Q D), FOR MODIFIED RECOVERY, IS:

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

**JOINTING 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

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE

### MECHANICAL PROPERTIES OF SOIL

$m_v$	$\text{kPa}^{-1}$	COEFFICIENT OF VOLUME CHANGE
$C_c$	1	COMPRESSION INDEX
$C_s$	1	SWELLING INDEX
$C_\alpha$	1	RATE OF SECONDARY CONSOLIDATION
$c_v$	$\text{m}^2/\text{s}$	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
$T_v$	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
$\sigma'_{v0}$	kPa	EFFECTIVE OVERBURDEN PRESSURE
$\sigma'_p$	kPa	PRECONSOLIDATION PRESSURE
$\tau_f$	kPa	SHEAR STRENGTH
$c'$	kPa	EFFECTIVE COHESION INTERCEPT
$\phi'$	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
$c_u$	kPa	APPARENT COHESION INTERCEPT
$\phi_u$	-°	APPARENT ANGLE OF INTERNAL FRICTION
$\tau_R$	kPa	RESIDUAL SHEAR STRENGTH
$\tau_r$	kPa	REMOULDED SHEAR STRENGTH
$S_i$	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

### STRESS AND STRAIN

$u_w$	kPa	PORE WATER PRESSURE
$u$	1	PORE PRESSURE RATIO
$\sigma$	kPa	TOTAL NORMAL STRESS
$\sigma'$	kPa	EFFECTIVE NORMAL STRESS
$\tau$	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
$\epsilon$	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
$\mu$	1	COEFFICIENT OF FRICTION

### PHYSICAL PROPERTIES OF SOIL

$\rho_s$	$\text{kg}/\text{m}^3$	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	$e_{\min}$	1, %	VOID RATIO IN DENSEST STATE
$\gamma_s$	$\text{kN}/\text{m}^3$	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	$I_D$	1	DENSITY INDEX = $\frac{e_{\max} - e}{e_{\max} - e_{\min}}$
$\rho_w$	$\text{kg}/\text{m}^3$	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
$\gamma_w$	$\text{kN}/\text{m}^3$	UNIT WEIGHT OF WATER	$S_r$	%	DEGREE OF SATURATION	$D_n$	mm	n PERCENT - DIAMETER
$\rho$	$\text{kg}/\text{m}^3$	DENSITY OF SOIL	$w_L$	%	LIQUID LIMIT	$C_u$	1	UNIFORMITY COEFFICIENT
$\gamma$	$\text{kN}/\text{m}^3$	UNIT WEIGHT OF SOIL	$w_p$	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
$\rho_d$	$\text{kg}/\text{m}^3$	DENSITY OF DRY SOIL	$w_s$	%	SHRINKAGE LIMIT	q	$\text{m}^3/\text{s}$	RATE OF DISCHARGE
$\gamma_d$	$\text{kN}/\text{m}^3$	UNIT WEIGHT OF DRY SOIL	$I_p$	%	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
$\rho_{sat}$	$\text{kg}/\text{m}^3$	DENSITY OF SATURATED SOIL	$I_L$	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	i	1	HYDRAULIC GRADIENT
$\gamma_{sat}$	$\text{kN}/\text{m}^3$	UNIT WEIGHT OF SATURATED SOIL	$I_C$	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
$\rho'$	$\text{kg}/\text{m}^3$	DENSITY OF SUBMERGED SOIL	$e_{\max}$	1, %	VOID RATIO IN LOOSEST STATE	j	$\text{kN}/\text{m}^3$	SEEPAGE FORCE
$\gamma'$	$\text{kN}/\text{m}^3$	UNIT WEIGHT OF SUBMERGED SOIL						

**FOUNDATION DESIGN REPORT  
PROPOSED BRIDGE NB-4  
RAMP FGGE E – BROWN'S LINE S  
OVER BROWN'S LINE SOUTHBOUND  
TORONTO, ONTARIO  
W.P. 174-00-01  
SITE 37-1522**

**Prepared For:**

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**Prepared by:**

**SHAHEEN & PEAKER LIMITED**

**Project: SP3232D  
June 8, 2000**

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FOUNDATION DESIGN REPORT

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APPENDICES

Limitations of Report

Appendix E

**FOUNDATION DESIGN REPORT  
PROPOSED BRIDGE NB-4  
RAMP FGGE E – BROWN'S LINE S  
OVER BROWN'S LINE SOUTHBOUND  
TORONTO, ONTARIO  
W.P. 174-00-01  
SITE 37-1522**

## **5. DISCUSSION AND RECOMMENDATIONS**

The proposed bridge will carry the re-aligned Ramp FGGE E – Brown's Line S over the Brown's Line Southbound Lanes. It will be an approximately 18 m wide, 34 m long rigid frame structure. We understand that the finished elevation for Ramp FGGE – Brown's Line S over the rigid frame deck will be approximately 113 m while the elevation for Brown's Line will be about 6 m below or at about Elevation 107 m, providing a vertical clearance of about 5 m.

At Boreholes B6-1 and B6-3, where the existing grades are 112.1 and 109.4 m, that is higher than the general bedrock elevation in the area, the natural overburden consists of a shallow layer of clayey silt and till/shale complex and the surface of the shale bedrock was encountered at approximate elevations of 108.8 and 107.8 m, respectively.

At Boreholes B6-2 and B6-4, the existing ground surface at the existing road level are 108.6 and 107.6 m, respectively and in these boreholes the surface of the shale bedrock was encountered immediately underlying the pavement fill at about 0.5 m below the ground surface or Elevations 108.1 and 107.1 m.

### **5.1 FOUNDATIONS**

The boreholes show that the proposed bridge structure can be supported on normal spread footing foundations founded on the natural competent glacial till and till/shale complex or the shale bedrock. The recommended soil and rock resistances at each borehole location are presented in the following table.

Table 1

Borehole Number	Existing Ground Surface Elevation at Borehole Location (m)	Recommended Footing Base (Bottom) Depth Below Existing Ground Surface at Borehole Location (m)	Recommended Footing Base (Bottom) Elevation (m)	Factored Bearing Resistance at U.L.S. (kPa)*	Bearing Resistance at S.L.S. (kPa)	Subgrade Material
B6-1	112.1	2.4	109.7	800	400	clayey silt till
		3.1	109.0	1000	600	till/shale
		3.6	108.5	1000	n/a**	shale bedrock
		4.3	107.8	1200	n/a**	shale bedrock
B6-2	108.6	1.2	107.4	1000	600	shale bedrock
		2.0	106.6	1000	n/a**	shale bedrock
		2.6	106.0	1200	n/a**	shale bedrock
B6-3	109.4	1.2	108.2	800	400	clayey silt till
		2.0	107.4	1000	600	shale bedrock
		2.6	106.8	1000	n/a**	shale bedrock
		3.2	106.2	1200	n/a**	shale bedrock
B6-4	107.6	1.0	106.6	1000	600	shale bedrock
		1.6	106.0	1000	n/a**	shale bedrock
		2.1	105.5	1200	n/a**	shale bedrock

\* incorporating a resistance factor of 0.5 as per Ontario Highway Bridge Design Code (OHBDC), 3<sup>rd</sup> Edition.

\*\* Total and differential settlements will not exceed 12 mm for foundations placed on properly prepared bedrock surface, under the direction of geotechnical engineer.

The serviceability condition is based on the premise that the maximum total and differential settlements will not exceed 25 mm and 18 mm, respectively. This can be achieved provided that the founding subgrade is undisturbed during the construction.

As the elevation for Brown's Line Southbound under the bridge will be about 107 m, the footing elevations will be about 106 m or lower. The borehole results indicate that at these elevations the footings will be founded on the shale bedrock where Factored Bearing Resistance values at U.L.S. of up to 1200 kPa can be utilized (although resistances of this magnitude are unlikely to be necessary for the proposed structure) and settlements can be expected not to exceed 12 mm provided that the rock surface is undisturbed.

Where the surface of the rock is lower than the proposed founding level or where the rock is shattered and/or highly weathered, the unsuitable materials should be removed and replaced with mass concrete.

Under inclined loading conditions the Bearing Resistance at U.L.S. should be reduced in accordance with Clause 6-8.4.2 of O.H.B.D.C.

For the evaluation of the sliding resistance of the foundation (O.H.B.D.C. 6-8.4.3), the ultimate angle of friction between the underside of the foundation and the clean shale bedrock surface (or between concrete surfaces) can be taken as 25 degrees. Horizontal shear resistance can be supplemented, if required, by penetrating in the bedrock (i.e. keying-in and utilizing passive rock resistance) and/or shear in grouted dowels and/or rock anchors.

The unfactored horizontal resistance against sliding between concrete and approved till surface can be calculated using a friction angle of 29 degrees.

If there are net uplift forces which are to be resisted by rock anchors, the factored rock/grout bond capacity at U.L.S. in the bedrock can be taken as 500 kPa and S.L.S. will not govern. The upper 0.3 m of the rock should, however, not be included in calculating the resistance and the minimum embedment depth should be 1.5 m into the rock (below the underside of the footing). 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.

For frost protection, the footings should have a permanent earth cover of at least 1.2 m, or equivalent artificial insulation, including footings founded on shale bedrock.

## 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.

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 O.H.B.D.C. For design purposes, the following parameters (unfactored) can be used.



**Compacted Granular 'A'**Unit Weight = 22 kN/m<sup>3</sup>

Coefficient of Lateral Earth Pressures:

$$K_a = 0.27$$

$$K_o = 0.43$$

**Compacted Granular 'B'**Unit Weight = 21 kN/m<sup>3</sup>

Coefficient of Lateral Earth Pressures:

$$K_a = 0.31$$

$$K_o = 0.47$$

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 as per Clause C6-7.1 of the O.H.B.D.C., 3<sup>rd</sup> Edition. The effect of compaction should also be taken into account in the selection of the appropriate earth pressure coefficients in accordance with Clause 6-7.4.3, O.H.B.D.C., 3<sup>rd</sup> Edition.

Foundations on bedrock will be unyielding and in that case the at-rest condition will govern the earth pressure.

Vibratory equipment for use behind abutments and retaining walls should be restricted in size as per current MTO practice and as specified in OPSS 501.

For retaining walls abutting into the rigid structure as an alternative to conventional retaining walls, MTO's Retained Soil System may be used. The following should be included in the Contract Documents:

- identify longitudinal extent in plan of the Retained Soil System
- identify in plan transverse space constraints (top of wall and bottom of wall)
- identify elevation of top of wall and bottom of wall
- include NSSP for Retained Soil Systems in Contract Documents

The Retained Soil System should be of high performance and high appearance.

### 5.3 CONSTRUCTION COMMENTS

Water levels in the piezometers installed in Boreholes B6-1 and B6-4 were recorded at Elevations 108.1 and 106.8 m, respectively. In addition, the natural overburden overlying the bedrock is relatively impervious. For these reasons, major problems due to groundwater seepage into footing excavations during construction are not envisaged. It is our opinion that seepage from a groundwater or perched water source can be handled by means of gravity drainage and where necessary, by pumping from open sumps.

Above the water table, temporary excavations more than 1.2 m deep should be sloped no steeper than 1:1. In the shale, somewhat steeper side slopes may be permissible, as directed by the geotechnical engineer. Alternatively, excavations should be supported in accordance with the Provincial Safety Regulations. It should also be pointed out that the fill encountered in Borehole B6-1 appeared to be poorly compacted and may therefore require temporary side slopes of between 1 ½ - 2 horizontal in 1 vertical. All slope faces should be protected against surface erosion.

It is recommended that an approximately 150 mm thick layer of lean concrete be placed on bearing surfaces within four hours of excavation. The shale is prone to weathering and disturbance and this recommendation applies to shale bedrock as well as the overburden soils. Following the construction of the footings, backfill should be placed to a sufficient height above the footing (i.e. at least 1.2 m) to prevent disturbance and frost penetration. All foundation excavations and bearing surfaces should be inspected and approved by the geotechnical engineer appointed by the contract administrator.

The backfilling behind the walls of the rigid frame structure should be carried out in such a manner that the height of the backfill on either side remains approximately equal.

The Paleozoic sedimentary rocks of southern Ontario, including the Georgian Bay shales, exhibit an expansion in the horizontal direction, due to a phenomenon known as residual stress relief, which can result in damage to buried structures. This aspect should be taken into consideration if structure walls are to be placed directly against vertical bedrock face.

The hard overburden and the highly weathered shale materials can be excavated using conventional excavation equipment with buckets provided with hardened teeth. As mentioned before, the presence of cobbles, boulders and rock slabs can also be expected in the overburden. The difficulty in excavating can be expected to increase with depth of excavation. The removal of hard layers in the shale including sandstone and siltstone and especially limestone layers, may create difficulties in excavating and the use of hoe ramming, rippers or jack hammering may be necessary.

Based on the borehole results, the strength of the natural foundation materials is such that deep seated failure under the weight of the approach fills (up to 6 m high) is not anticipated, provided that all organic soils, weak or otherwise unsuitable materials are removed as per MTO standards, before placing the fill.

For the construction of the embankment fills, all the surficial topsoil and otherwise unsuitable soils should be removed. Where the new embankment fill will abut into the existing ramp embankment slopes, proper benching should be applied, as per MTO and OPSS Standards (OPSD-208.01). After the removal of all unsuitable soils, the exposed subgrade should be inspected, approved and properly compacted from the surface, using a suitably heavy compactor, under the supervision of a geotechnical engineer who is familiar with the findings of this report and appointed by the contract administrator.

The materials used for the construction of the embankment fills should consist of approved, acceptable fill, free of cobbles and boulders, frozen materials, organic soils, etc. (e.g. select subgrade materials -- OPSS 1010). The fill should be placed in thin 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. The degree of compaction within the top 0.6 m of the fill (i.e. subgrade immediately beneath the granular subbase) should be increased to 98%. The selection, placement and compaction of the fill should be carried out under the supervision of a geotechnical engineer. The settlement of embankment fills prepared as described above should not exceed 50 mm, based on the borehole results and assuming that any existing fills not removed were properly compacted when they were first placed. Surcharging or pre-loading is therefore considered not to be necessary.

Assuming properly compacted, acceptable inorganic earth fill materials constructed as described above, 2 horizontal in 1 vertical side slopes can be used for fills as well as for cuts. Proper erosion control measures should be implemented. This can be achieved by immediate seeding or sodding (OPSS 572).

Groundwater level was recorded at about Elevation 107-108 m, that is, below the anticipated excavation depths required for the construction of the embankment fills. Therefore, we do not anticipate major problems due to groundwater seepage during the stripping of the unsuitable soils. However, since the water level at the borehole locations was recorded at or above the road level, permanent drainage may need to be implemented to enhance the performance of the road.

As mentioned before, permanent cut faces can be maintained at 2H:1V slopes. But they should be inspected during the construction for possible local instabilities and, where necessary, remedial measures, such as gravel sheeting may be required.

Vegetation should be established on all slope faces to protect against surficial erosion as per OPSS 572.

#### 5.4 FROST PROTECTION

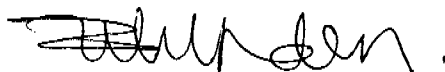
Design frost penetration for the general area is 1.2 m. Therefore, a permanent soil cover of 1.2 m or its thermal equivalent is required for frost protection of foundations, including those founded on the shale bedrock.

## 6. CLOSURE

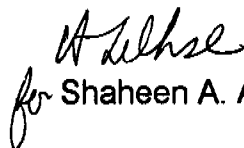
We recommend that once the details of the structure are finalized, our recommendations be reviewed for their specific applicability.

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

### Shaheen & Peaker Limited



Zuhtu Ozden, P.Eng.



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



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# APPENDIX E

## Limitations of Report

## **LIMITATIONS OF REPORT**

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.