

G.I.-30 SEPT. 1976

GEOCRES No. 30M11-201

DIST. CR REGION

W.P. No. 172-00-01

CONT. No.

W. O. No.

STR. SITE No. 37-1520

HWY. No. QEW

LOCATION BRIDGE NB-2 RAMP BROWN'S
LINE N-FGGE(E) OVER BROWN'S LINE

No. of PAGES -

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OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

**FOUNDATION INVESTIGATION REPORT
PROPOSED BRIDGE NB-2
RAMP BROWN'S LINE N-FGGE(E) OVER BROWN'S LINE
TORONTO, ONTARIO
W.P. 172-00-01
SITE 37-1520**

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**Project: SP3232C
June 8, 2000**

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No. 2

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**FOUNDATION INVESTIGATION REPORT
PROPOSED BRIDGE NB-2
RAMP BROWN'S LINE N – FGGE (E) OVER BROWN'S LINE
TORONTO, ONTARIO
W.P. 172-00-01
SITE 37-1520**

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 Brown's Line N-FGGE (E) over the existing Brown's Line. 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 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 Wisconsinan 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 m.

3. INVESTIGATION PROCEDURES

The fieldwork for the project was performed during the period of February 11 through 25, 2000 and consisted of drilling and sampling five boreholes (Boreholes B2-1 through B2-4 and B2-4A) at the locations shown on Drawing No. 2. The depth of the boreholes ranged from 0.6 to 10.1 m below the ground surface.

The boreholes were advanced using truck and track 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 B2-1, B2-2, B2-4 and B2-4A were extended by augering until refusal in the bedrock at depths of between 0.6 m (Borehole B2-4A) and 5.2 m (Borehole B2-4), probably on limestone layers in the shale bedrock. In Borehole B2-3, augering was stopped at a depth of 5.3 m. This borehole and Borehole B2-1 were extended by diamond drilling (rock coring), using NQ-size core barrel to depths of 10.1 and 7.9 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. Core logs and photographs are given in Appendix C.

The logs of boreholes that were available to us, drilled for the Ministry in 1966 in the vicinity of the proposed bridge site, are given in Appendix E.

Groundwater conditions in the open boreholes were observed during the drilling and at completion of each borehole. In addition, piezometers were installed in Boreholes B2-2 and B2-4 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 determined and provided to us by surveyors from Waylaw Technical Services of Paris, Ontario.

4. SUBSURFACE CONDITIONS

The subsurface conditions were explored at five borehole locations (Boreholes B2-1 to B2-4 and B2-4A). The locations of the boreholes are given in Drawing No. 2 and are also indicated on the individual borehole log sheets in Appendix A. Cross sections of inferred subsurface stratigraphy are given on the same drawing.

Boreholes B2-1, B2-2 and B2-4 were drilled from the top of the earth embankments where ground elevations were 111.2, 110.9 and 112.3 m, respectively. These boreholes contacted 75 to 300 mm of topsoil, underlain by fill and possible fill to depths of 1.4 to 2.7 m, or to Elevation 110.9 to 108.5 m. Beneath the topsoil and the fill, Borehole B2-4 contacted a layer of fine sand, underlain by clayey silt till and till/shale complex to the surface of bedrock at 4.2 m or Elevation 108.1 m. In the remaining two boreholes, underlying the fill, shale bedrock was

contacted at 2.1 and 2.7 m below the ground surface or at Elevations 108.8 and 108.5 m.

Borehole B2-4A was drilled on the slope of an embankment and beneath a 150 mm topsoil layer, it contacted a material resembling a completely disintegrated shale to 0.6 m depth (Elevation 107.3 m), underlain by inferred bedrock.

Borehole B2-3 was located on paved (median) shoulder and contacted 100 mm of asphalt underlain by granular pavement fill to about 0.45 m. The pavement fill is in turn underlain by a material resembling a completely disintegrated shale to 1.1 m or Elevation 103.0 m, to the surface of the bedrock.

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

4.1 TOPSOIL

All the boreholes, except for Borehole B2-3 (drilled from the surface of the paved shoulder) contacted a veneer of topsoil ranging in thickness from 75 to 300 mm.

4.2 PAVEMENT

Borehole B2-3, which was drilled from the paved median shoulder, contacted a 100 mm thick asphaltic concrete layer, underlain by granular base and subbase materials to about 0.45 m.

4.3 FILL

Material identified as fill was contacted in Boreholes B2-1, B2-2 and B2-4 to depths/elevations of 2.3/108.9 m, 2.1/108.8 m and 0.7/111.6 m, respectively. The fill generally consists of clayey silt with shale fragments, or a completely weathered (clayey) shale with some sand and gravel particle sizes.

Standard Penetration tests conducted in these fill materials yielded N-values generally ranging from 6 to 12 blows/0.3 m, indicating that the fill did probably not receive a systematic compaction when it was first placed.

In Borehole B2-1 and B2-4, the materials immediately underlying the fill were identified as 'possible' fill materials, as well as the soil encountered in Borehole B2-4A.

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

4.4 CLAYEY SILT TILL

Clayey silt till was contacted in Boreholes B2-1 and B2-4 at depths of 2.3 m (Elevation 108.9 m) and 0.7 m (Elevation 111.6 m) and extended to depths of 2.7 m (Elevation 108.5 m) and 3.4 m (Elevation 108.9 m), respectively. The deposit contains shale fragments at increasing frequency with depth.

The clayey silt till is a cohesive material and the grain-size distribution of samples from the deposit is given in Appendix B.

It should be pointed out that the presence of cobbles and boulders could always be expected in the glacial till deposits, owing to their mode of deposition.

N-values recorded in the till deposit range from 13 to generally in excess of 50 blows/0.3 m, indicating a stiff to hard but generally hard consistency.

In Borehole B2-4 an approximately 0.3 m thick wet fine sand layer was contacted at 1.4 m (Elevation 110.9 m) within the clayey silt till deposit.

4.5 TILL/SHALE COMPLEX

The lower portions of the overburden often resemble 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. 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.

Till/shale complex was contacted in Borehole B2-4 at a depth of 3.4 m or at Elevation 108.9 m. The grain size distribution of a sample from the deposit is shown in Appendix B.

The till/shale is basically a cohesive material. From a recorded N-value of 56 blows/0.3 m, its consistency is described as hard.

4.6 SHALE BEDROCK

Shale bedrock was encountered or its presence was inferred in all the boreholes at the following approximate depths/elevations.

Estimated Bedrock Depth

Borehole No.	Bedrock Depth (m)	Elevation (m)
B2-1	2.7	108.5
B2-2	2.1	108.8
B2-3	1.1	103.0*
B2-4	4.2	108.1
B2-4A	0.6	107.3

*Borehole B2-3 was located at the road level where the grade was lowered below the surface of the bedrock when the road was first built.

The surface of the shale bedrock should be regarded as approximate only; this is because these depths were often inferred from the observed resistance to augering only 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 also 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 some limestone, siltstone and sandstone seams was noted in the cores obtained from the bedrock in Boreholes B2-1 and B2-3. They were generally 2 to 15 cm thick except for a 225 mm thick seam in Borehole B2-1 near the surface of the bedrock (i.e. about 3.3 m below the ground surface or Elevation 107.9 m) and a 280 mm thick seam in Borehole B2-3 at 7.6 m depth or Elevation 96.5 m. In Boreholes B2-2 and B2-4, where the rock was not cored, the bedrock was penetrated by augering until refusal to augering was encountered at depths of 2.9 m (Elevation 108.0) and 5.2 m (Elevation 107.1), respectively, probably on the surface of a hard layer (e.g. limestone layer/lense). From this, it can be inferred that the bedrock at these two locations does not contain hard layers above these elevations. In Borehole B2-4A, refusal was contacted at 0.6 m depth or Elevation 107.3 m.

As was mentioned in Section 4.5 of this report, excavation methods should take into consideration the sporadic presence of hard shale, siltstone, sandstone and limestone slabs or layers in the till/shale complex and especially in the underlying shale bedrock. The presence of these hard layers can create problems during excavation, especially where site restrictions (e.g. low overhead clearance) would preclude the use of normal, full size equipment.

4.7 GROUNDWATER CONDITIONS

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

The boreholes were dry upon the completion of augering. In Boreholes B2-1 and B2-3, where coring was effected, water was introduced into the boreholes to facilitate diamond drilling, therefore, in these boreholes 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 in each of Boreholes B2-2 and B2-4.

Water levels in the piezometers installed in these boreholes were monitored over a period of approximately two weeks and the recorded values are shown on the borehole logs. The final water levels were recorded at 2.7 m (Elevation 108.2 m) and 1.6 m (Elevation 110.7 m) in Boreholes B2-2 and B2-4, respectively.

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

Shaheen & Peaker Limited

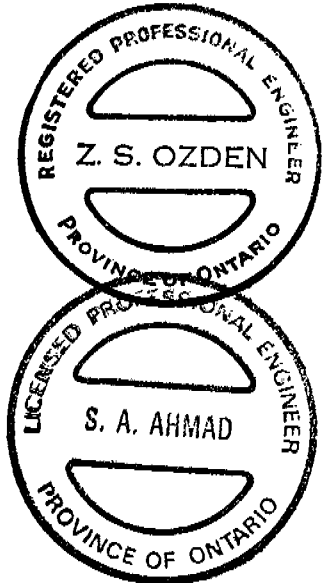


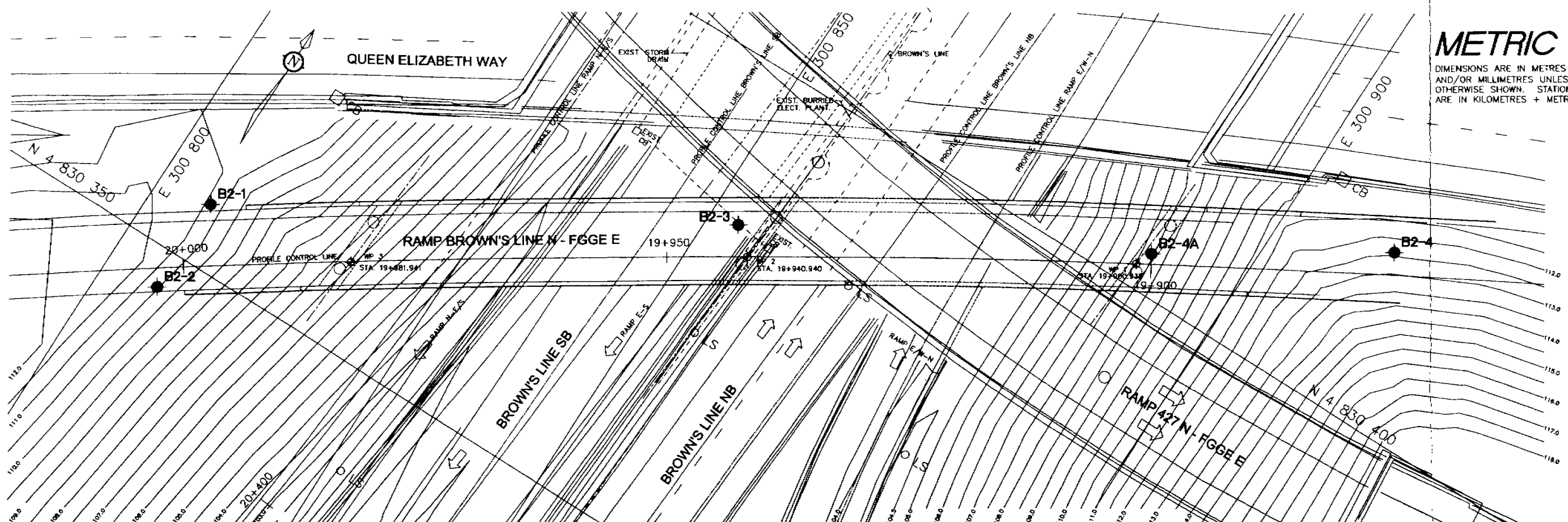
Zuhtu Ozden, P.Eng.



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

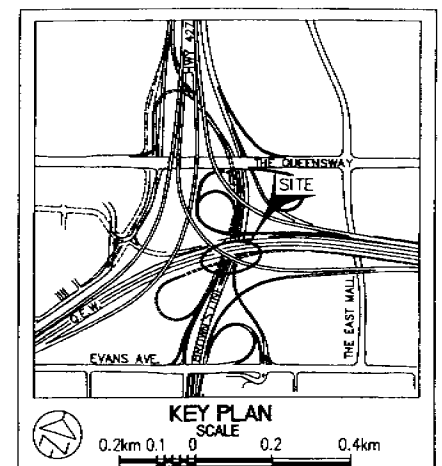
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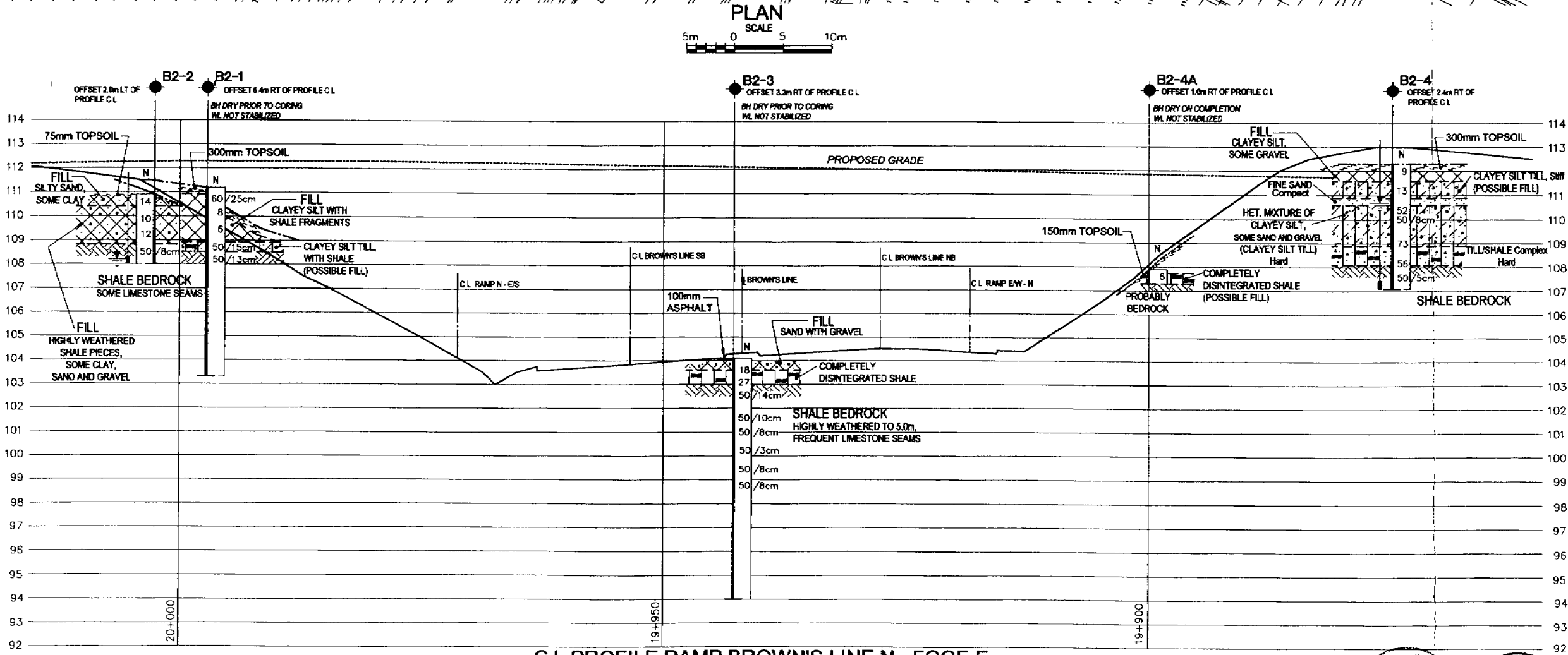


METRIC
 DIMENSIONS ARE IN METRES
 AND/OR MILLIMETRES UNLESS
 OTHERWISE SHOWN. STATIONS
 ARE IN KILOMETRES + METRES.

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. and Mar. 2000			
↓	W L in Piezometer			
⊥	Piezometer			
No	ELEVATION	CO-ORDINATES NORTH	EAST	
B2-1	111.2	4 830 356.4	300 805.9	
B2-2	110.9	4 830 346.1	300 805.9	
B2-3	104.1	4 830 383.9	300 853.1	
B2-4	112.3	4 830 417.8	300 911.5	
B2-4A	107.9	4 830 404.0	300 890.5	

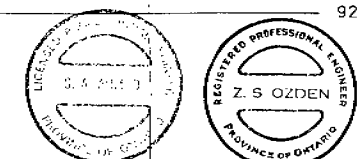


C L PROFILE RAMP BROWN'S LINE N - FGGE E

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			
HWY No	427 & QEW	DIST	
SUBM'D ZO	CHECKED ZO	DATE	May, 2000
DRAWN	JTW	CHECKED	JP
SITE			37-1520
DWG			2



APPENDIX A

Borehole Log Sheets

RECORD OF BOREHOLE No B2-1

1 OF 1

METRIC

W.P. 172-00-01 LOCATION Ramp Brown's Line N - FGGE(E) 4 830 356.4 N; 300 805.9 E ORIGINATED BY G.I
DIST HWY 427 & QEW BOREHOLE TYPE Solid Stem Augers - NQ Rock Core COMPILED BY G.T
DATUM Geodetic DATE 11.02.00 CHECKED BY Z.O

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	
111.2	Ground surface											
110.9	300 mm Topsoil		1	AS			111					
0.3	FILL		2	SS	60/25							
	Clayey Silt with shale fragments, grey, damp		3	SS	8		110					
			4	SS	6							
108.9							109					
108.5	CLAYEY SILT TILL with shale (possible fill)		5	SS	50/15							
2.7			6	SS	50/13		108					
	225 mm limestone seam		7	NQ RC	Rec. 100%		107					
			8	NQ RC	Rec. 100%		106					
	SHALE BEDROCK grey some limestone seams		9	NQ RC	Rec. 100%		105					
							104					
103.3												
7.9	End of borehole Borehole dry prior to coring Water level not stabilized											AS- Auger sample

RECORD OF BOREHOLE No B2-2

1 OF 1

METRIC

W.P. 172-00-01 LOCATION Ramp Brown's Line N - FGGE(E) 4 830 346.1 N; 300 805.9 E ORIGINATED BY M.J.
 DIST HWY 427 & QEW BOREHOLE TYPE Solid Stem Augers COMPILED BY G.T.
 DATUM Geodetic DATE 15.02.00 CHECKED BY Z.O.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								20	40	60	80	100					
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE									
								20	40	60	80	100					
110.9	Ground surface																
110.8	75 mm Topsoil		1	SS	14											21.0	Refusal to further augering @ 2.1m probably on a limestone layer
110.4	FILL: Silty Sand, some clay																
0.5	FILL highly weathered Shale pieces, some clay, sand and gravel		2	SS	10		110									21.1	
			3	SS	12												
108.8							109										
2.1	SHALE BEDROCK some limestone seams, grey		4	SS	50/8												
108.0			5	AS													
2.9	End of borehole Refusal to augering @ 2.9 m Piezometer installed to 2.9 m Water level in piezometer Feb. 20 - dry Feb. 22 - dry Feb. 24 - 2.8 m March. 2 - 2.7 m																Moved 1.0 m SE and re-drilled Refusal to augering again @ 2.9 m probably on a limestone layer

RECORD OF BOREHOLE No B2-3

1 of 1

METRIC

W.P. 172-00-01 LOCATION Ramp Brown's Line N. - FGGE(E) 4 830 383.9 N, 300 853 1 E ORIGINATED BY G.I.
DIST HWY 427 & QEW BOREHOLE TYPE Solid Stem Augers, NQ Rock Core COMPILED BY G.T.
DATUM Geodetic DATE 25.02.00 CHECKED BY Z.O.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				
104.1	Ground surface							20 40 60 80 100				
104.0	100 mm Asphalt							20 40 60 80 100				
103.6	FILL : Sand with gravel, brown, Completely disintegrated shale		1	SS	18			20 40 60 80 100				
103.0			2	SS	27			20 40 60 80 100				
1.1	SHALE BEDROCK grey highly weathered to 5.0 m frequent limestone seams 280 mm limestone seam		3	SS	50/14			20 40 60 80 100				
			4	SS	50/10			20 40 60 80 100				
			5	SS	50/8			20 40 60 80 100				
			6	SS	60/8			20 40 60 80 100				
			7	SS	50/8			20 40 60 80 100				
			8	SS	50/8			20 40 60 80 100				
			9	NQ RC	Rec. 100%			20 40 60 80 100				
			10	NQ RC	Rec. 100%			20 40 60 80 100				
			11	NQ RC	Rec. 100%			20 40 60 80 100				
			12	NQ RC	Rec. 100%			20 40 60 80 100				
		94.0										
		10.1	End of borehole Borehole dry prior coring Water level not stabilized									

+ 3, x 3; Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No B2-4

1 OF 1

METRIC

W.P. 172-00-01 LOCATION Ramp Brown's Line N - FGGE(E) 4 830 417.8 N; 300 911.5 E ORIGINATED BY E.P.
DIST HWY 427 & QEW BOREHOLE TYPE Solid Stem Augers COMPILED BY G.T.
DATUM Geodetic DATE 17.02.00 CHECKED BY Z.O.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					
112.3	Ground surface							20 40 60 80 100	PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT		
112.0	300 mm Topsoil		1	SS	9		112						
0.3	FILL : Clayey Silt, some gravel												
111.6													
0.7	CLAYEY SILT TILL		2	SS	13							21.5	14 37 32 17
110.9	grey, stiff, (possible fill)						111						
110.6	Fine SAND, brown, compact, wet		3	SS	52							20.2	
1.7	Heterogeneous mixture of Clayey Silt, some sand and gravel (CLAYEY SILT TILL)		4	SS	50/8		110						
108.9	grey, hard		5	SS	73		109					22.1	18 17 49 16
3.4	TILL/SHALE Complex grey, hard												
108.1			6	SS	56		108						6 53 28 13
4.2	SHALE BEDROCK grey		7	SS	50/5								
107.1													
5.2	End of borehole Refusal to further augering @ 5.2 m possibly on a limestone layer Piezometer installed to 5.2 m Water level in piezometer Feb.20 = 2.4 m Feb.24 = 1.8 m Feb.29 = 1.6 m March. 2 = 1.6 m												

+ 3 . x 3 : Numbers refer to
Sensitivity

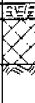
20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No B2-4A

1 OF 1

METRIC

W.P. 172-00-01 LOCATION Ramp Brown's Line N - FGGE(E) 4 830 404.0 N, 300 890.5 E ORIGINATED BY J.W.
DIST HWY 427 & QEW BOREHOLE TYPE Solid Stem Augers COMPILED BY G.T.
DATUM Geodetic DATE 24.02.00 CHECKED BY S.B.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
107.9	Ground surface		1	SS	6												
107.8	150 mm Topsoil																
107.3	Completely disintegrated Shale (possible fill)																
0.6	End of borehole Refusal to further augering probably on bedrock Borehole dry on completion Water level not stabilized																

+ 3, x 3: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

APPENDIX B

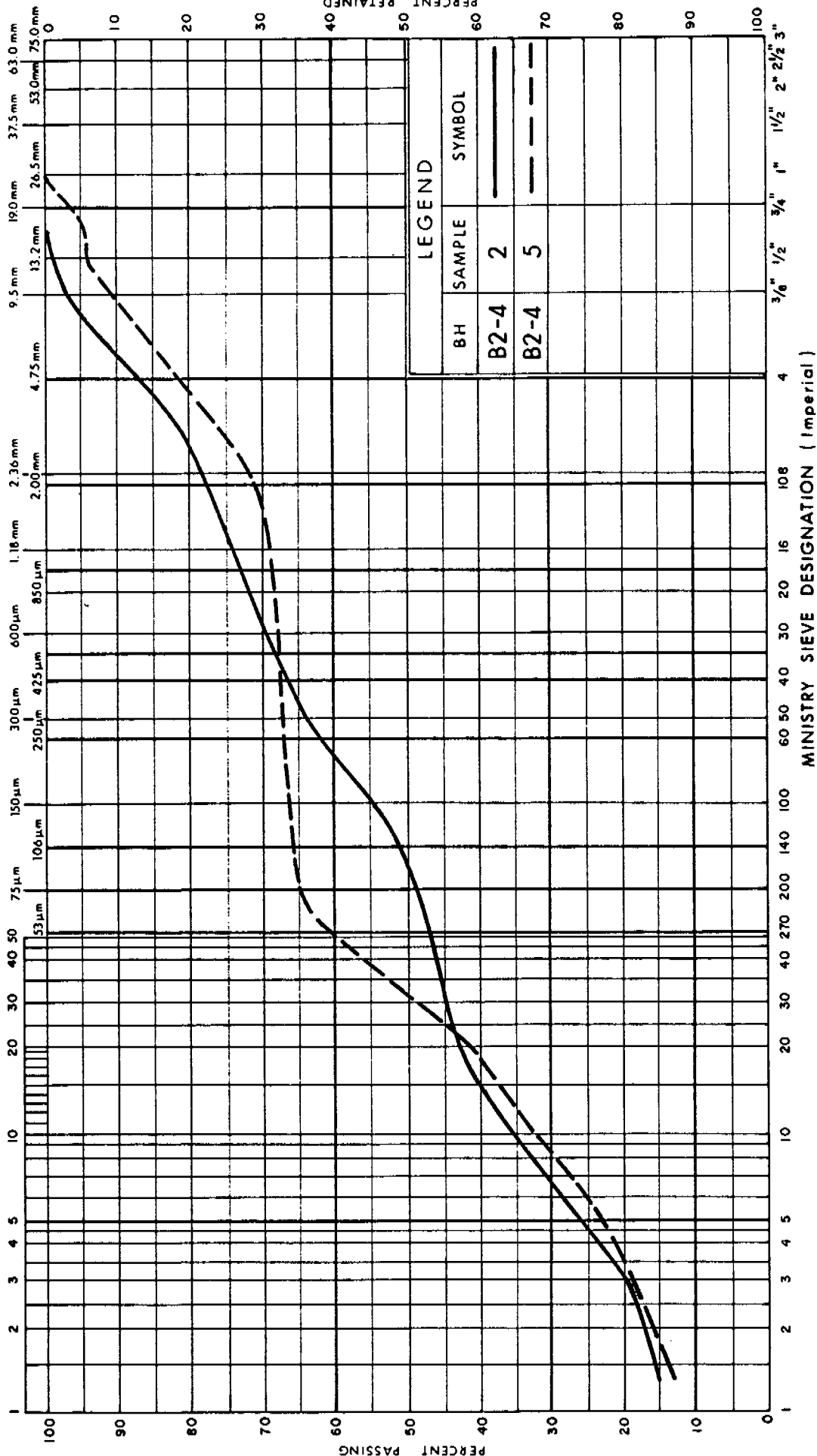
Laboratory Test Results

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT		SAND			GRAVEL		
		Fine	Medium	Coarse	Fine	Coarse	

GRAIN SIZE IN MICROMETERS

MINISTRY SIEVE DESIGNATION (Metric)



Ministry of
Transportation



FIG No 1

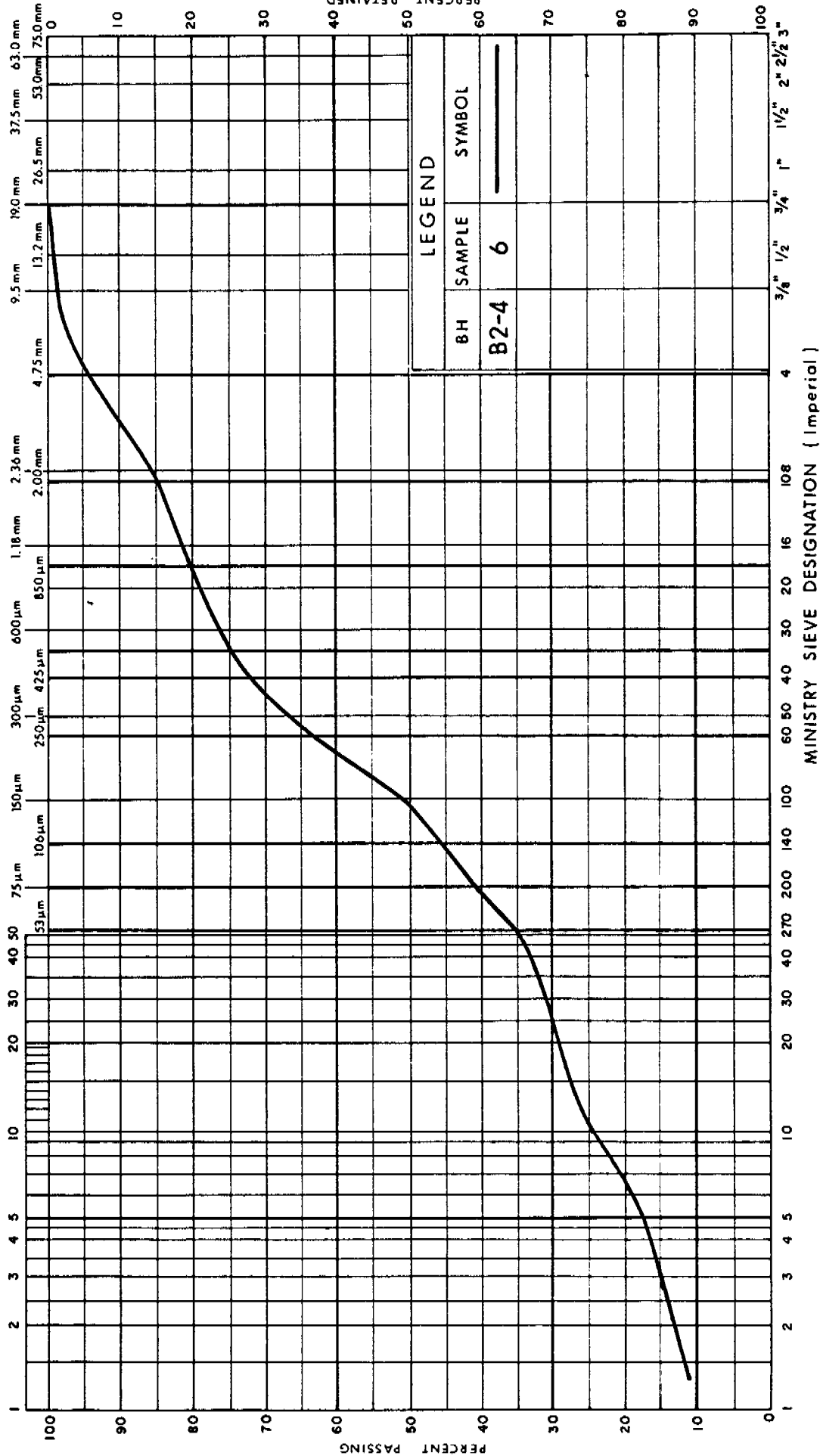
W P 172-00-01

SP 3232C

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT		SAND			GRAVEL	
		Fine	Medium	Coarse	Fine	Coarse

MINISTRY SIEVE DESIGNATION (Metric)



LEGEND

BH	SAMPLE	SYMBOL
B2-4	6	—

Ministry of Transportation

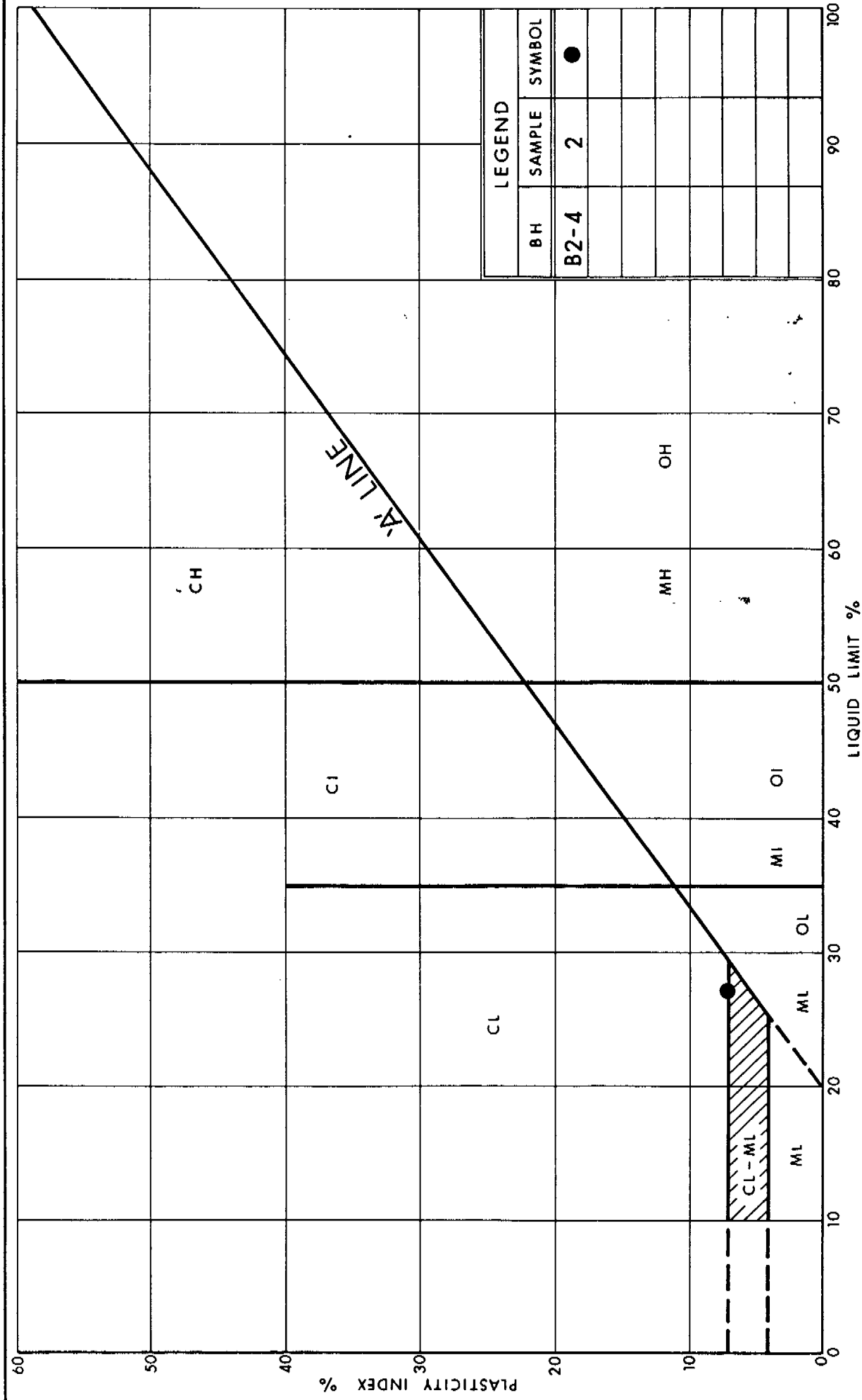


GRAIN SIZE DISTRIBUTION TILL / SHALE Complex

FIG No 2

W P 172-00-01

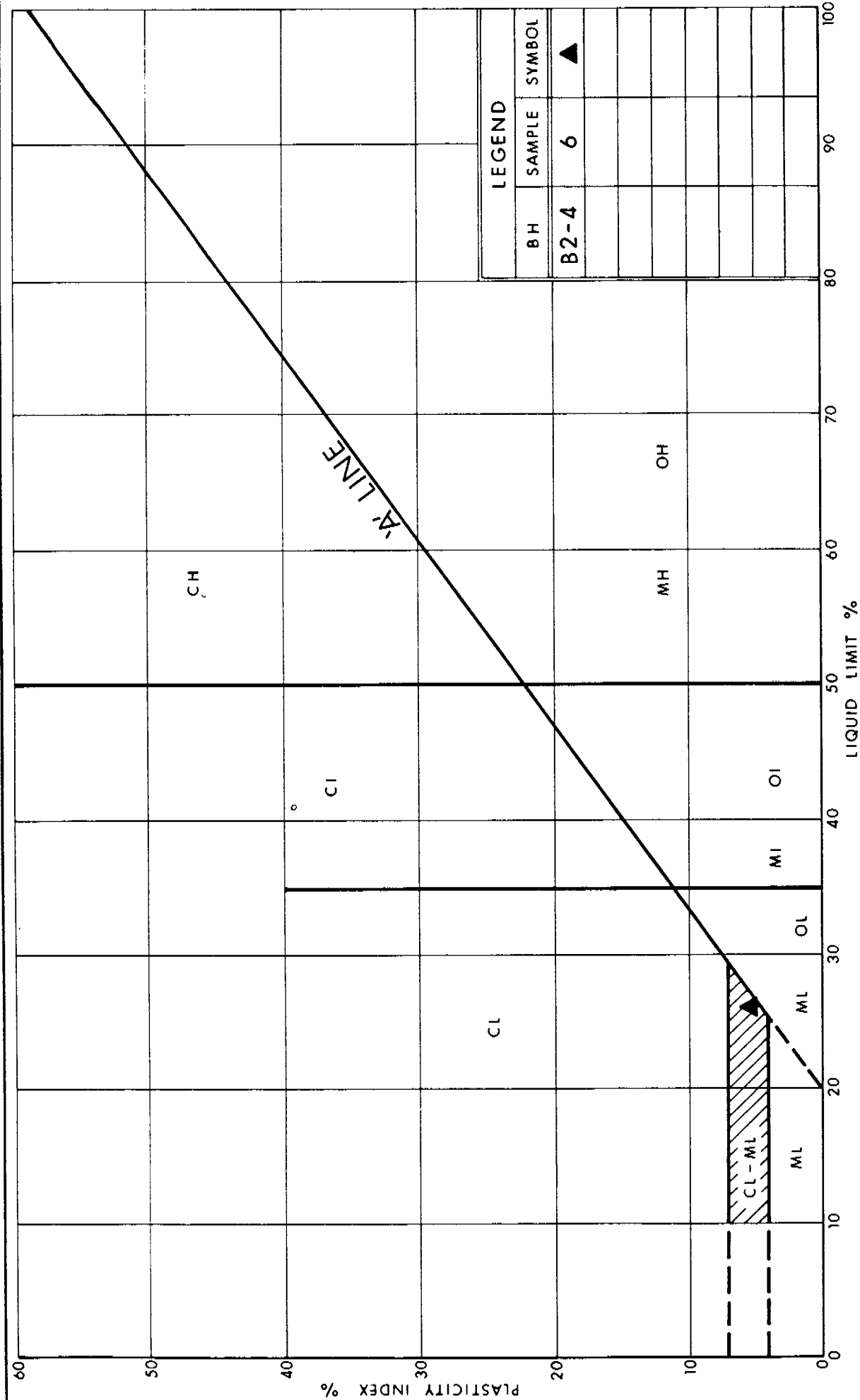
SP 3232C



LEGEND		
BH	SAMPLE	SYMBOL
B2-4	2	●

PLASTICITY CHART
CLAYEY SILT TILL

FIG No 3
WP 172-00-01
SP 3232C

Ministry of
Transportation

PLASTICITY CHART TILL / SHALE Complex

FIG No 4

W P 172-00-01

SP 3232C

APPENDIX C

Core Logs and Photographs

CORE LOG

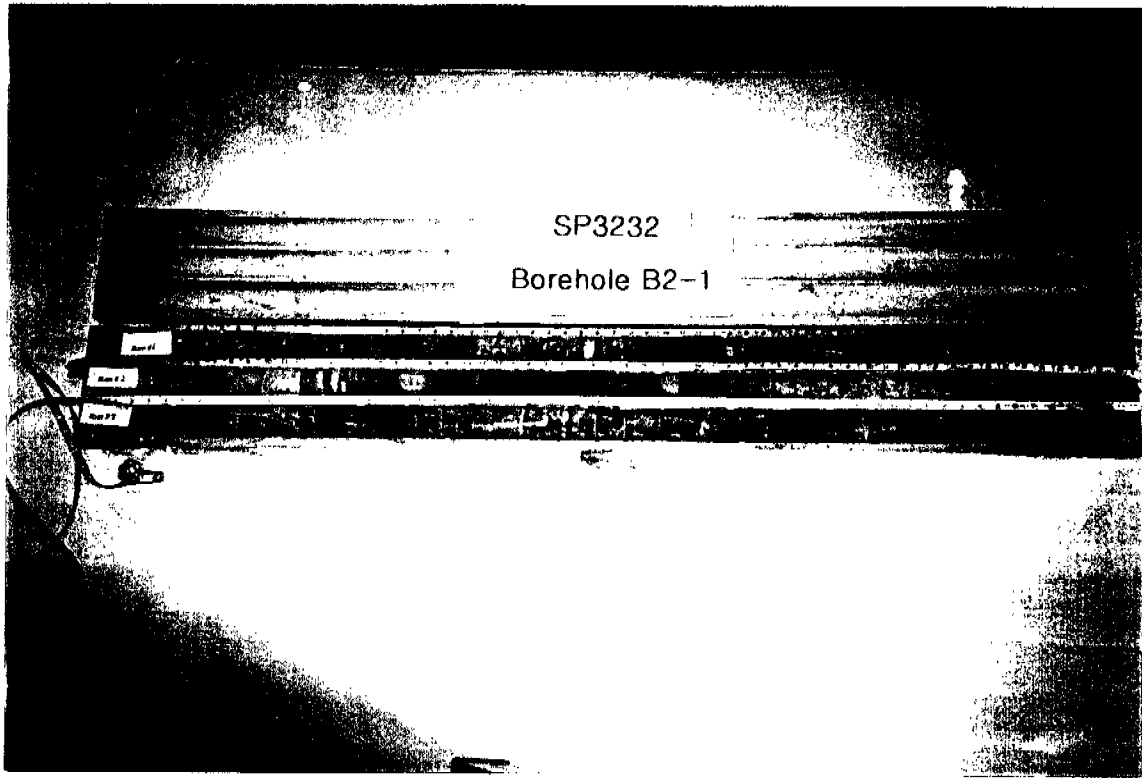
BH NO B2-1

PROJECT Foundation Investigation	ORIENTATION Vertical	ELEVATION (m) 111.2	DATUM Geodetic	PROJECT NO. SP3232
LOCATION Q.E.W./Hwy 427/Brown's Line Interchange Modification	DATE STARTED 02/11/00	COMPLETED 02/11/00	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)								
111.2	1		OVERBURDEN: see soils log for description															
108.5	2																	
107.9	3		SHALE: sampled with split-barrel sampler; see soils log for description															
	4		GEORGIAN BAY FORMATION: Shale with Interbedded Limestone and Sandstone	1	B	F D	C VC	SP RP	T	0				1	100	65	100	grey
	5		Shale (79%) thinly bedded or laminated, dark grey, slightly to moderately weathered to 4.8 m, low strength	2	B	V												
	6		Limestone (9%) fine grained, fossiliferous, unweathered, moderate strength		C				R R	5 5				2	100	45	100	grey
	7		Sandstone and Siltstone (12%) stratified, brownish grey to medium grey, unweathered, moderate strength											3	100	75	100	grey
103.3	8		Discontinuities: bedding joints are at close to very close intervals, the maximum thickness of limestone or sand/siltstone layers was about 225 mm, occasional random vertical joints, joint surfaces are smooth planar to rough planar															
	9		Rubble seams about 5 mm thick noted at 5.57 and 5.65 m depth.															
	10		End of Borehole															
	11																	
	12																	
	13																	
	14																	
	15																	
	16																	
	17																	
	18																	
	19																	

S & P

Shaheen & Peaker Limited
Consulting Geo-Environmental Engineers

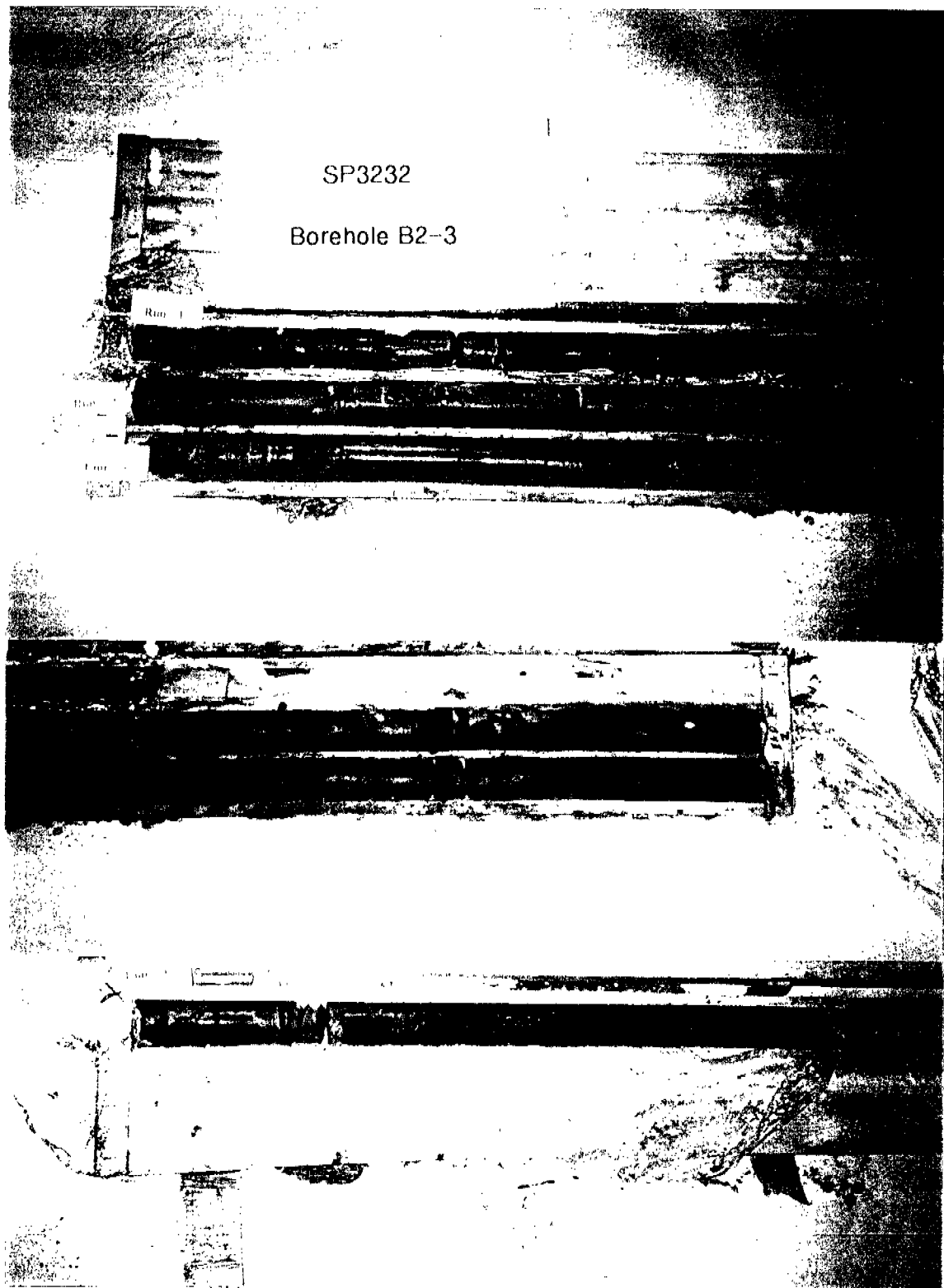


Photograph of Core
Borehole B2-1
Elev. 107.9 m – 103.3 m

CORE LOG											BH NO B2-3							
PROJECT Foundation Investigation					ORIENTATION Vertical		ELEVATION (m) 104.1		DATUM Geodetic		PROJECT NO. SP3232							
LOCATION Q.E.W./Hwy 427/Brown's Line Interchange Modification					DATE STARTED 02/25/00		COMPLETED 02/25/00		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
104.1			OVERBURDEN: see soils log for description															
103.0	1		SHALE: sampled with split-barrel sampler; see soils log for description															
	2																	
	3																	
	4																	
98.8	5																	
	6		GEORGIAN BAY FORMATION: Shale with Interbedded Limestone and Sandstone Shale (76%) thinly bedded or laminated, dark grey, slightly to moderately weathered to 6.0 m, low strength Limestone (11%) fine grained, fossiliferous, unweathered, moderate strength Sandstone and Siltstone (13%) stratified, brownish grey to medium grey, unweathered, moderate strength Discontinuities: bedding joints are at close to very close intervals, the maximum thickness of limestone or sand/siltstone layers was about 280 mm, occasional random vertical joints, joint surfaces are smooth planar to rough planar Clay seams of 5 mm noted at 7.82 m and 8.71 m, and 50 mm at 8.62 m	1	B	F	C	SP	T	0				1	100	32	100	grey
	7													2	100	92	100	grey
	8								SO	5								
	9								SO	50				3	100	75	100	grey
	10								SO	5				4	100	93	100	grey
94.0	11																	
	12																	
	13																	
	14																	
	15																	
	16																	
	17																	
	18																	
	19																	

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Photograph of Core
Borehole B2-3
Elev. 98.8 m – 94.0 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 _r 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>Strength of rock material:</p> <p>Very high strength * specimen can only be chipped by geological hammer</p> <p>High strength * specimen requires a number of blows of geological hammer to fracture it; cannot be scraped with pocket knife</p> <p>Medium strength * specimen can be fractured by single firm blow of geological hammer; can be scraped with pocket knife, not peeled</p> <p>Low strength = shallow indentations made by firm blow with point of geological hammer; can be peeled by pocket knife with difficulty</p> <p>Very low strength = 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>Fracture Frequency: 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>Run Number and Core Recovery:</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>Rock Quality Designation (RQD): 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>Core and Casing Sizes: changes of core and casing sizes are indicated.</p>													
18	<p>Water recovery, level and tests.</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 (RQD), 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

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

MECHANICAL PROPERTIES OF SOIL

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

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

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	e_{min}	1, %	VOID RATIO IN DENSEST STATE
γ_s	KN/m^3	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
ρ_w	kg/m^3	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	KN/m^3	UNIT WEIGHT OF WATER	S_r	%	DEGREE OF SATURATION	D_n	mm	n PERCENT - DIAMETER
ρ	kg/m^3	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ	KN/m^3	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
ρ_d	kg/m^3	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m^3/s	RATE OF DISCHARGE
γ_d	KN/m^3	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	i	1	HYDRAULIC GRADIENT
γ_{sat}	KN/m^3	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL	e_{max}	1, %	VOID RATIO IN LOOSEST STATE	j	KN/m^2	SEEPAGE FORCE
γ'	KN/m^3	UNIT WEIGHT OF SUBMERGED SOIL						

APPENDIX E

Logs of Boreholes Drilled in 1966

GEOTECHNICAL DATA SHEET FOR BOREHOLE . . 88 . .

OUR REFERENCE NO. 6-6-13

CLIENT: D.H.O.

PROJECT: Q.E.W. & HWY. 27 INTERCHANGE, BRIDGE NO. 2

LOCATION: 178,488 N 209,594 E

DATUM ELEVATION: G.S.C.

METHOD OF BORING: AUGERING & WASHBORING

DIAMETER OF BOREHOLE: 4 1/2" - 2 1/2"

DATE: JULY 5, 1968

W.P. 238-61-1

ENCLOSURE NO.

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE blows per foot					CONSISTENCY water content %			REMARKS
				NUMBER	TYPE	TEST	20	40	60	80	100	PI	W	LI	
367.8	0	GROUND SURFACE													
366.8	1.0	TOPSOIL & FILL													
365.0		Compact to Dense Brown FINE SAND		1	AS	-									W.L. El. 364.8 ft July 5, 1968
361.7	5.3	Very Dense Grey SANDY SILT with some gravel and shale fragments (GLACIAL TILL)		2	AS	71/7									
360.0															
355.0	10.0			3	SS	74/9									
354.0															
350.0	12.0	Grey SHALE with bands of limestone		4	R.C.	44%									
345.0	20.0	BEDROCK		5	R.C.	48%									
340.0	25.0			6	R.C.	60%									
340.0	30.0	END OF BOREHOLE													

VERTICAL SCALE: 1 IN TO 5 FT.

DOMINION SOIL INVESTIGATION LIMITED

MADE: C.K. CND:

GEOTECHNICAL DATA SHEET FOR BOREHOLE . . 89 . .

OUR REFERENCE NO. 6-6-18

CLIENT: D.H.O.
PROJECT: BRIDGE No. 5, Q.E.W. & HWY. 27.
LOCATION: 178,446 N ; 209,348 E
DATUM ELEVATION: G.S.C.

METHOD OF BORING: WASHBORING
DIAMETER OF BOREHOLE: 2 3/8"
DATE: JUNE 30, 1966.
W.P. 238-61-4

ENCLOSURE NO.

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE					CONSISTENCY			REMARKS
				NUMBER	TYPE	N- or Advance of Sample	blows per foot					water content %			
							SHEAR STRENGTH					PL W LI			
368.9	0	GROUND SURFACE													
		Brown SAND (FILL)													
367.4	1.5	Dark Brown CLAYEY SILT with a trace of SAND and GRAVEL (FILL)													
365.0	5	Organic TOPSOIL													
363.9	5.9	Dense FINE SAND with some SILT		1	SS	SS									
361.4	7.8	Very Dense Grey SAND and SILT with numerous SHALE fragment and a trace of embedded fine GRAVEL (GLACIAL TILL)		2	SS	50/4"									
360.0	10			3	SS	65/4"									
355.0				4	SS	100/4"									
353.9	15			5	SS	100/4"									
350.0		Grey SHALE BEDROCK		6	RC	89 %									
348.2	20	END OF BOREHOLE													

W.L. 364.6 Ft. JULY 6, 1966.
Sa - 83% ; Si. - 17%
Sa - 56% , Si. - 44%

GEOTECHNICAL DATA SHEET FOR BOREHOLE . . . 115.

OUR REFERENCE NO. 6-6-13

CLIENT: D.H.O.

PROJECT: Q.E.W. & HWY. No. 27 INTERCHANGE, BRIDGE No. 2

LOCATION: 178,488 N 209,454 E

DATUM ELEVATION: G.S.C.

METHOD OF BORING: WASHBORING.

DIAMETER OF BOREHOLE: 3 1/4"

DATE: JUNE 30 - JULY 5, 1966

W.P. 238-61-1

ENCLOSURE NO.

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE blows per foot					CONSISTENCY water content %			REMARKS
				NUMBER	TYPE	TEST	20	40	60	80	100	PL	W	LI	
367.6	0	GROUND SURFACE													
		6" TOPSOIL													
365.0		SANDY, CLAYEY SILT (FILL)													
363.8	4.0	Dense, Brown FINE SAND		1A	SS	60									
360.0	8.0	Dense, Grey SANDY SILT with some gravel (GLACIAL TILL)		1B	SS	60									
359.8	10														
355.0	12	Grey EXTREMELY WEATHERED SHALE		3	R.C.	80 %									
350.0	20			4	SS	60									
		BEDROCK		5	R.C.	80 %									
345.0	25			6	SS	60									
				7	R.C.	10 %									
340.0	30	Sound		8	R.C.	0 %									
335.0	32			9	R.C.	80 %									

W.L. El. 364.2 ft.
July 6, 1966

VERTICAL SCALE 1 IN 10 5 11

DOMINION SOIL INVESTIGATION LIMITED

MADE: C.K. CHD

GEOTECHNICAL DATA SHEET FOR BOREHOLE . 119.

OUR REFERENCE NO 6-6-13

CLIENT: D.H.O.
 PROJECT: Q.E.W. & HWY. No. 27 INTERCHANGE, BRIDGE No 2 DIAMETER OF BOREHOLE: 3 1/2"
 LOCATION: 178,360N 209,326E
 DATUM ELEVATION: G.S.C.

METHOD OF BORING: WASHBORING.

DATE: JULY 6, 1966

W.P. 238-61-1

ENCLOSURE NO

ELEVATION	DEPTH	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES		PENETRATION RESISTANCE		CONSISTENCY	REMARKS
				NUMBER	TYPE	Blows per foot	SHEAR STRENGTH	PL W LI	
368.0	0	GROUND SURFACE							
365.0	3	1" ASPHALT SILTY SAND with some gravel FILL							
364.0	4.0	SILTY FINE SAND		1	SS	35			
363.0		Ha, Grey CLAYEY SILT to SANDY SILT with some gravel and shale fragments (GLACIAL TILL)		2	SS	100/2			
360.0				3	SS	100/2			
355.0				4	WS	-			
352.2	14.8	Grey SHALE with layers of limestone		5	RC.	77%			
350.0				6	RC.	80%			
345.0		BED ROCK							
	25	END OF BOREHOLE							

W.L. El. 362.8 ft.
 July 6, 1966

**FOUNDATION DESIGN REPORT
PROPOSED BRIDGE NB-2
RAMP BROWN'S LINE N-FGGE(E) OVER BROWN'S LINE
TORONTO, ONTARIO
W.P. 172-00-01
SITE 37-1520**

Prepared For:

**DS-LEA ASSOCIATES LIMITED
251 Consumers Road, Suite 1200
North York, Ontario**

Prepared by:

SHAHEEN & PEAKER LIMITED

**Project: SP3232C
June 8, 2000**

**250 Galaxy Boulevard
Etobicoke, Ontario
M9W 5R8
Tel: (416) 213-1255
Fax: (416) 213-1260**

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APPENDICES

Limitations of Report

Appendix F

**FOUNDATION DESIGN REPORT
PROPOSED BRIDGE NB-2
RAMP BROWN'S LINE N-FGGE(E) OVER BROWN'S LINE
TORONTO, ONTARIO
W.P. 172-00-01
SITE 37-1520**

5. DISCUSSION AND RECOMMENDATIONS

The new bridge will carry proposed Ramp Brown's Line N-FGGE(E) over the existing Brown's Line corridor and associated ramps. The structure will be an approximately 9 m wide, 2 span pre-stressed concrete girder structure, each span measuring approximately 41 m in length. The elevation of the existing road beneath the bridge structure is approximately 104 m and finished bridge elevation will be 112± m.

Five boreholes were drilled for the investigation. At Boreholes B2-1, B2-2 and B2-4, where the existing ground elevations range from 112.3 to 110.9 m, that is higher than the general bedrock elevation in the area, the natural overburden generally consists of a shallow layer of clayey silt till and till/shale complex and in these boreholes the surface of the bedrock was contacted at approximate elevations ranging from 108.8 to 108.1 m. In boreholes put down by others in 1966 in the general area for the Ministry before the construction of the present interchanges, the surface of the bedrock is reported at Elevations ranging from 109.0 to 107.7 m (Appendix E).

Borehole B2-3 was drilled from the road (Elevation 104.1 m) where the grade was lowered below the original bedrock surface elevation when the road was first built.

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 Location	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
B2-1	111.2	2.7	108.5	1000	600	shale bedrock
West Abutment		3.2	108.0	1000	1000**	shale bedrock
		3.9	107.3	1200	1200**	shale bedrock
B2-2	110.9	2.2	108.7	1000	600	shale bedrock
West Abutment		2.9	108.0	1000	1000**	shale bedrock
		3.6	107.3	1200	1200**	shale bedrock
B2-3	104.1	1.5	102.6	1000	600	shale bedrock
Central Pier		2.1	102.0	1000	1000**	shale bedrock
		2.5	101.6	1200	1200**	shale bedrock
B2-4A	107.9	0.6	107.3	1000	600	shale bedrock
East Abutment***		1.4	106.5	1000	1000**	shale bedrock
		2.0	105.9	1200	1200**	shale bedrock
B2-4	112.3	3.8	108.5	1000	600	till/shale complex
East Abutment		4.6	107.7	1000	1000**	shale bedrock
		5.2	107.1	1200	1200**	shale bedrock

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

** Total and differential settlements should be less than 10 mm for foundations placed on properly prepared bedrock surface.

*** Other boreholes drilled in the general area, including previously drilled MTO boreholes, indicate that where the surface of the bedrock was not previously excavated, bedrock may be encountered at a higher elevation (e.g. Elevation 108.5 m)

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

Where the surface of the rock is lower than the proposed founding level or where the rock is shattered and/or highly weathered, any 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.

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

As presently proposed, the abutments of the bridge are expected to be supported on normal spread footings founded on shale bedrock at about Elevation $105\pm$ m and $104\pm$ m for the east and west abutments, respectively and the central pier will be supported on drilled and cast-in-place concrete piles (caissons). Table 1 shows that for the design of the spread footings (i.e. abutment foundations) at or below Elevation $105\pm$ m a factored rock bearing resistance at U.L.S. of 1200 kPa can be used and the settlements should be less than 10 mm for this resistance value.

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. We recommend that the minimum dowel length below the underside of the footing should be 2 m.

If there are net uplift forces which are to be resisted by rock anchors, or for increasing the sliding resistance, the factored rock/grout bond capacity at U.L.S. in the bedrock can be taken as 500 kPa (incorporating a safety factor of 2 against an ultimate failure condition) 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.

The abutments can also be founded on engineered fill consisting of Granular A type material compacted in thin layers to at least 100% of the material's Standard Proctor Maximum Dry Density. We understand however that because of the congested nature of the proposed and existing bridges (i.e. lack of space) and logistics involved, the use of abutments founded on engineered fill is not a viable alternative. If, however, you require us to further comment on this alternative we will be pleased to do so.

Based on the Borehole B2-3 results, for caisson design a value of up to 5000 kPa can be used for vertical rock resistance at U.L.S. at or below Elevation 98.2 m (i.e. at least 5.9 m below the existing grade at the borehole location). The minimum caisson length below the finished grade should be 5.5 m.

The resistance to lateral movement is provided by the passive resistance developed on the face of the caisson. The unfactored passive resistance developed over the width of the caisson can be computed assuming $2 c_u$ at the surface, increasing linearly to $9 c_u$ at a depth of 3 diameter and beyond. This pressure can be converted into a passive resistance by using a bearing width equal to the caisson diameter.

The unfactored c_u (i.e. undrained shear strength) for the shale can be taken as 1200 kPa within a depth of 5 m of the ground surface and 1600 kPa below.

Passive resistance developed within 1.2 m of the ground surface (i.e. within the frost zone) should be ignored.

Coefficient of horizontal subgrade reaction (force per volume) can be calculated from the following expression:

$$k_s = \frac{67 c_u}{d}$$

Where

k_s = coefficient of subgrade reaction

c_u = undrained shear strength of the material

d = pile diameter

As mentioned before, the c_u value (unfactored) can be assumed to be 1200 kPa within 5 m of the ground surface and 1600 kPa below.

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/m³

Coefficient of Lateral Earth Pressures:

$$K_a = 0.27$$

$$K_o = 0.43$$

Compacted Granular 'B'

Unit Weight = 21 kN/m³

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., 3rd 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., 3rd 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.

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 APPROACH EMBANKMENTS

We understand that only minor grade adjustments can be expected for the construction of the approach embankments. For this purpose, all the surficial topsoil and otherwise unsuitable soils should be removed. Where the new embankment fill will abut into the existing embankment ramp slopes, proper benching should be applied, as per MTO and Ontario Provincial 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. 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).

Water levels in the piezometers were recorded below the anticipated excavation depths required for the construction of the approach embankments. Therefore, we do not anticipate major problems due to groundwater seepage during the stripping of the unsuitable soils from the top of the existing embankments and during backfilling. It is believed that any seepage from a perched water source and/or from surface water should be controllable by gravity drainage and where necessary by pumping from open sumps.

5.4 CONSTRUCTION COMMENTS

At the time of our investigation, the groundwater level in the piezometers was encountered at Elevations 110.7 (Borehole B2-4) and 108.2 m (Borehole B2-2). The strata below these elevations are, however, generally relatively impervious. For this reason, during construction major problems due to groundwater seepage are not anticipated, even below the recorded water levels. It is believed that where necessary groundwater seepage can be handled by means of gravity drainage and pumping from sumps. Occasional coarse zones (e.g. wet sand layer encountered in Borehole B2-4, between Elevations 110.9 m and 110.6 m) may create local problems but in our opinion, these too can be handled by providing local drainage and vigorous pumping.

Allowance should be made to place a 150 mm thick concrete mud mat in all footing excavations within about four hours of excavation to minimize disturbance. The shale is prone to weathering and disturbance and a similar mud mat would be required on founding surfaces placed on shale. All footing

excavations should be inspected and approved, prior to pouring the concrete, by the geotechnical engineer, at the time of construction.

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.

Based on the borehole results, temporary slopes in the fill can be maintained, for the duration of construction, at side slopes no steeper than 1½ Horizontal in 1 Vertical. Where the fill is weak or where wet granular soils are encountered (including the zone between the bottom of the fill and the competent natural clayey silt till) the slopes may have to be locally flattened (e.g. Borehole B2-4). In the competent clayey silt till and in the till/shale complex 1:1 side slopes should be stable for temporary excavations. In the underlying shale somewhat steeper side slopes may be permissible, as directed by the geotechnical engineer.

All slope faces should be protected against erosion.

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.

A minimum caisson diameter of 0.76 m will be necessary and temporary steel liners will have to be employed so that the bases of the caisson holes can be cleaned, inspected and approved. It should also be pointed out that the walls of the caissons in the bedrock will have to be cleared of any overburden smear and we recommend that an allowance be made for this purpose. The temporary liners can be withdrawn after inspection and approval of the caisson base and walls by the geotechnical engineer and while pouring concrete. We recommend that the concrete be poured without undue delay after the completion of the caisson hole to prevent the deterioration of shale. The presence of hard layers (e.g. limestone layers) will present problems during the installation of the caissons

since these hard limestone layers will significantly increase the time required to auger the caissons. Problems due to excessive groundwater seepage are known to occasionally occur in this formation, which will necessitate dewatering measures such as vigorous pumping.

All excavation and shoring should be carried out in accordance with the Safety Regulations of the Province. If shoring is constructed using soldier piles and conventional lagging, the soldier piles will have to be socketed into the bedrock, which was contacted at most borehole locations below Elevations ranging from about 109 to 108 m.

For shoring design, the following unfactored parameters can be used:

Table 2
Recommended Unfactored Parameters for Temporary Shoring Design

Stratum	Ka	Ko	Kp	γ (kN/m ³)
Fill	0.5	0.55	3.0	20.0
Clayey Silt Till and Till/Shale Complex	0.25	0.45	4.0	21.5
Shale Bedrock	0.15	0.45	4.5	23.0

For the design of raker footings placed in the hard clayey silt till and till/shale complex and extending at least 0.6 m into the general excavation level, the following soil resistance values can be used:

U.L.S. = 400 kPa

S.L.S. = 200 kPa

These values can be increased to 800 kPa and 400 kPa for relatively sound bedrock.

For calculating anchor resistance for tie back design, the bond resistance at U.L.S. can be taken as 70 kPa in the clayey silt till and the till/shale complex and 500 kPa in the bedrock and S.L.S. will not govern.

By means of good construction techniques, undermining of existing bridge footings should be prevented.

We understand that permanent roadway protection may be required beyond the abutments and that this will likely be achieved by means of contiguous caisson wall construction. Our recommendations for caisson design and construction were given in Section 5.1 and in the earlier paragraphs of this section. As the caissons will be rigid (unyielding) structures, their design should be based on K_0 condition of the retained material.

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 structures. This aspect should be taken into consideration of structure walls are to be placed directly against vertical bedrock face.

5.5 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

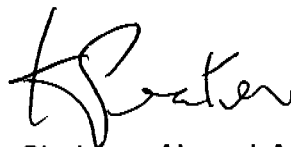
We recommend that once the details of the structure are finalized, our recommendations be reviewed for their specific applicability.

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

Shaheen & Peaker Limited



Zuhtu Ozden, P.Eng.



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

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APPENDIX F

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.