

GEOCRES No. 30M11-200DIST. CR REGION W.P. No. 171-00-01CONT. No. W. O. No. STR. SITE No. 37-1519HWY. No. QEWLOCATION BRIDGE NB-1 RAMP FGGE E-
SHERWAY GARDENS RD OVER BROWN'S LINENo. of PAGES -

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. REMARKS:

DS-Lea Associates Ltd.

Transportation and Environmental Consultants

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15 May 2000

Our Ref.: 1950/100

Mr. J. Klowak, P.Eng.
Senior Project Engineer
Highway Engineering, Central Region
Ministry of Transportation
1201 Wilson Avenue
Downsview, Ontario
M3M 1J8

Dear Mr. Klowak:


**Re: Highway 427/QEW Interchange Modifications – Detailed Design
W.P. 166-00-00
Foundation Reports**

Attached are the final foundation reports for the following structures:

- Ramp FGGE E – Sherway Gardens Road over Brown's Line
- Ramp Brown's Line N – FGGE E over Brown's Line
- Highway 427 Ramp N-W over FGGE E – Sherway Gardens Road
- Ramp FGGE E – Brown's Line southbound
- Bridge widening FGGE over the East Mall
- Retaining Wall at N-E/W Ramp to Sherway Gardens Road
- Retaining Wall for Ramp FGGE E – (Brown's Line South) S427N – Queensway Avenue

Yours very truly

DS-LEA ASSOCIATES LTD.



Paul G. Geary, P.Eng.
Head, Roads & Highways

:ecf

Encls:

cc: Betty Bennett, 1 copy
John Lam, 1 copy
Ken Zasitko, 1 copy
Martin Everington, 1 copy for tender review
Mike Chung, City of Toronto (East Mall Structure only)

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**FOUNDATION INVESTIGATION REPORT
PROPOSED BRIDGE NB-1
RAMP FGGE E – SHERWAY GARDENS ROAD
OVER BROWN'S LINE
TORONTO, ONTARIO
W.P. 171-00-01
SITE 37-1519**

Prepared For:

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

SHAHEEN & PEAKER LIMITED

**Project: SP3232B
June 8, 2000**

**250 Galaxy Boulevard
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**FOUNDATION INVESTIGATION REPORT
PROPOSED BRIDGE NB-1
RAMP FGGE E – SHERWAY GARDENS ROAD OVER BROWN'S LINE
TORONTO, ONTARIO
W.P. 171-00-01
SITE 37-1519**

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 the proposed Ramp FGGE E-Sherway Gardens Road over the existing Brown's Line corridor. 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 in between the Queensway and Gardener Expressway (FGGE) at Brown's Line in the City of 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 Wisconsinan 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 between February 11 and 25, 2000 and consisted of drilling and sampling four boreholes (Boreholes B1-1 through B1-4) at the locations shown on Drawing No. SP3232B. The depth of the boreholes ranged from 1.1 m to 9.1 m below the ground surface.

The boreholes were advanced using a track mounted drilling rig, 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 B1-2 and B1-4 were extended until refusal in the bedrock at depths of 5.8 and 1.1 m, respectively, probably on a limestone layer in the shale bedrock. In Boreholes B1-1 and B1-3 augering was stopped at depths of 3.9 and

1.8 m, respectively and the boreholes were extended by diamond drilling (rock coring), using NQ-size core barrel to depths of 9.1 and 5.6 m, respectively.

The soil 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 Limit 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.

Groundwater conditions in the open boreholes were observed during the drilling and at completion of each borehole. In addition, in Boreholes B1-2 and B1-4, piezometers were 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.

The logs of boreholes that were available to us (put down by the Ministry) in 1960's in the vicinity of the proposed bridge site are given in Appendix E.

4. SUBSURFACE CONDITIONS

The subsurface conditions were explored at four borehole locations (Boreholes B1-1 to B1-4). The locations of the boreholes are shown on the Borehole Location and Profile Drawing No. 4 and are also indicated on the individual borehole log sheets. Cross sections of inferred subsurface stratigraphy are given in the same drawing.

Boreholes B1-1 and B1-2 were drilled from the top and B1-4 from the side slope of existing road embankments where the ground elevations were 111.3, 111.7 and 108.9 m, respectively. These boreholes contacted 75 to 175 mm of topsoil, underlain by fill or previously disturbed material to depths ranging between 0.7 and 1.1 m below the ground surface or to Elevations 110.6 to 108.2 m.

Underlying the fill, these boreholes contacted clayey silt till and till/shale complex (except for Borehole B1-4) to the surface of bedrock at approximate depths ranging between 1.0 and 2.9 m (Elevation 108.8 to 107.9 m).

Borehole B1-3 was drilled on the median shoulder (Elevation 105.0 m). This borehole contacted 100 mm of asphalt, underlain by granular base and subbase pavement fill to 0.6 m. Below the granular pavement fill, previously disturbed, completely disintegrated shale mixed with some gravel was contacted to 1.0 m (Elevation 104.0 m) which is underlain by a 0.4 m thick layer of completely disintegrated shale material to the surface of bedrock at 1.4 m or Elevation 103.6 m.

An overall picture of the subsurface conditions is given in Drawing No. 4. For details of the subsurface conditions encountered in the boreholes, reference should be made to the individual Borehole log Sheets in Appendix A. The following paragraphs present a description of the individual strata and groundwater conditions.

4.1 TOPSOIL

All the boreholes, except for Borehole B1-3, which was drilled from the paved shoulder, contacted a 75 to 175 mm thick topsoil layer.

4.2 PAVEMENT

Borehole B1-3 was drilled from the paved median shoulder and this borehole contacted a 100 mm thick asphaltic concrete layer underlain by granular fill to 0.6 m. The grain size distribution of a sample from the granular fill is given in Appendix B.

4.3 FILL

Underlying the topsoil or pavement fill, all four boreholes encountered fill extending to depths ranging from 0.7 to 1.1 m below the ground surface or to Elevations ranging from 110.6 to 104.0 m.

The composition of the fill consisted of clayey silt (Boreholes B1-1 and B1-4), sandy silt (B1-3) and fine sand with silt (B1-2) all with some shale fragments.

We would like to point out that in the thickness of topsoil and the thickness and composition of fill deposits can vary in between and beyond borehole locations.

4.4 CLAYEY SILT TILL

Underlying the fill, the uppermost natural overburden in Boreholes B1-1, B1-2 and B1-4 consists of clayey silt till. The thickness of the deposit at the borehole locations ranges from 0.3 to 0.8 m and the unit extends to depths of 1.0 to 1.8 m below the ground surface, or to Elevations 109.9 to 107.9 m.

This deposit is a cohesive material and it contains shale fragments. The frequency of these shale fragments increases with increasing depth. The grain size distribution of a sample from the deposit is given in Appendix B.

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

Atterberg Limits test performed on a sample from the deposit gave the following values:

Liquid Limit = 25%

Plastic Limit = 18%

Plasticity Index = 7%

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

Standard Penetration tests performed in the deposit yielded N-values ranging from 44 to in excess of 50 blows/0.3 m, indicating a hard material.

4.5 TILL/SHALE COMPLEX

The lower portions of the overburden in the general area often resemble a highly weathered shale. This material is sometimes referred to as till/shale complex, which represents a transition zone into the underlying shale bedrock. This unit may often be described as a residual soil or a completely

weathered shale bedrock. Shale and limestone slabs or layers may remain. Excavation methods should take into consideration the possible presence of hard shale or limestone slabs/layers in the till, till/shale and in the underlying bedrock.

Till/shale complex was contacted in Boreholes B1-1 and B1-2 at depths of 1.6 and 1.8 m or Elevations 109.7 and 109.9 m, respectively and extended to 2.8 and 2.9 m or to Elevations 108.5 and 108.8, respectively to the surface of the bedrock.

The grain size distribution curve for a sample from this unit is given in Appendix B.

The following index values were obtained on a sample recovered from this deposit:

- Liquid Limit = 28%
- Plastic Limit = 21%
- Plasticity Index = 7%

The till/shale is a cohesive material. Based on N-values ranging from 44 to in excess of 50 blows/0.3 m and on the basis of resistance to augering during drilling, the consistency of this deposit is considered hard.

4.6 SHALE BEDROCK

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

Estimated Bedrock Depth

Borehole No.	Bedrock Depth (m)	Elevation (m)
B1-1	2.8	108.5
B1-2	2.9	108.8
B1-3	1.4	103.6*
B1-4	1.0	107.9

*This borehole was drilled from the road surface where the grade was lowered below the bedrock surface when the road was first built.

In most cases, 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 is known to consist 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 B1-1 and B1-3. They were generally 2 to 15 cm thick, except in Borehole B1-1 where the cores showed the presence of an approximately 0.3 m thick limestone layer at Elevation 103.6 m. In Boreholes B1-2 and B1-4, where the rock was not cored, the bedrock was penetrated by augering until refusal to augering was encountered at depths of 5.0 m (Elevation 106.7) and 1.1 m (Elevation 107.8), respectively, probably on the surface of a hard layer (e.g. limestone layer). From this, it can also be inferred that the bedrock at these two locations does not contain hard layers above these elevations.

As mentioned in Section 4.5, excavation methods should take into consideration the rather sporadic presence of hard shale, siltstone, sandstone and limestone slabs or layers in the till, till/shale complex and especially in the underlying shale bedrock. If present, these hard layers can create difficulties during excavation, especially where site restrictions, such as low overhead clearance, would preclude the use of full size equipment for this purpose.

4.7 GROUNDWATER CONDITIONS

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

All four boreholes were dry upon the completion of augering.

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 B1-2 and B1-4. The water levels in the piezometers were monitored over a period time when the water level was recorded at depths of 5.1 m and 0.9 m or at Elevation 106.6 and 108.0 m, respectively.

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

Shaheen & Peaker Limited

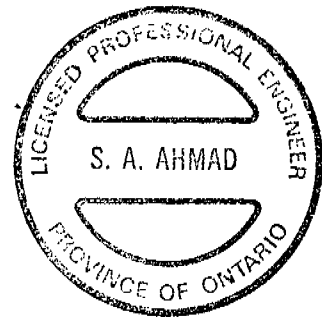


Zuhtu Ozden, P.Eng.



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

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APPENDIX A

Borehole Log Sheets

RECORD OF BOREHOLE No B1-1

1 OF 1

METRIC

W.P. 171-00-01 LOCATION Ramp FGGE(E)-Sherway Gardens Road, 4 830 431.2 N; 300 814.9 E ORIGINATED BY G.I.
DIST HWY 427 & QEW BOREHOLE TYPE Solid Stem Augers, NQ Rock Core COMPILED BY G.T.
DATUM Geodetic DATE 12.02.00 14.02.00 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		
111.3	Ground surface													
111.1	175 mm Topsoil		1	SS	12		111							
0.2	FILL : Clayey Silt, some sand and gravel & shale fragments, grey													
110.5	CLAYEY SILT TILL		2	SS	58		110							12 22 49 17
0.8	grey, hard													
109.7	TILL/SHALE Complex		3	SS	50/8		109							
1.6	grey, hard													
108.5			4	SS	50/13		108							
2.8	SHALE BEDROCK*													
	occasional limestone seams, grey													
	Weathered													
			5	SS	50/8		107							*part of rock core not recovered Auger from 3.9 to 5.3 m
			6	SS	50/8		106							
			7	NQ RC	Rec. 49%*		105							R.Q.D. =30%
			8	NQ RC	Rec. 92%		104							R.Q.D.=67%
			9	NQ RC	Rec. 100%		103							R.Q.D.=51%
	0.3 m thick limestone layer @ 7.7 m		10	NQ RC	Rec. 100%									
102.2														
9.1	End of borehole Borehole dry before coring Water level not stabilized													N=50/8 denotes 50 blows for 8 cm penetration

RECORD OF BOREHOLE No B1-2

1 OF 1

METRIC

W.P. 171-00-01 LOCATION Ramp FGGE(E)-Sherway Gardens Road, 4 830 426.5 N, 300 813.9 E ORIGINATED BY G.I
DIST HWY 427 & QEW BOREHOLE TYPE Solid Stem Augers 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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
111.7	Ground surface													
111.6	150 mm Topsoil		1	SS	12									
0.1	FILL : Fine Sand with silt, some gravel & shale pieces		2	SS	27								20.3	
110.6	CLAYEY SILT TILL													
1.1	very stiff to hard, brown to 1.5 m, grey below		3	SS	44									
109.9	(possible fill)													
1.8	TILL/SHALE Complex		4	SS	75/28								21.2	16 24 46 14
108.8	grey, hard													
2.9	SHALE BEDROCK grey		5	SS	50/6									
			6	SS	60/3									
			7	SS	50/5									
105.9			8	SS	50/NP*									
5.8	Auger refusal at 5.8 m probably on a limestone layer Borehole dry on completion Piezometer installed to 5.8 m Water level in piezometer Feb.20 - 5.3 m Feb.24 - 5.3 March. 2 - 5.1 m													N.P denotes no penetration

+ 3 x 3 : Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No B1-3

1 OF 1

METRIC

W.P. 171-00-01 LOCATION Ramp FGGE(E)-Sherway Gardens Road, 4 830 446.6 N; 300 853.4 E ORIGINATED BY G.I
DIST HWY 427 & QEW BOREHOLE TYPE Solid Stem Augers, NQ Rock Core COMPILED BY G.T
DATUM Geodetic DATE 20.02.00 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 (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
105.0	Ground surface													
104.8	100 mm Asphalt		1	SS	50/15									34 52 (14)
104.4	FILL : Sand with Gravel, some silt													
0.6 104.0	FILL : Sandy Silt with shale fragments & Gravel		2	SS	50/8									
1.0 103.6	Completely disintegrated													
1.4	SHALE grey		3	SS	50/5									
			4	SS	50/8									
			5	NQ RC	Rec. 100%									R.Q.D.=70%
	SHALE BEDROCK occasional limestone seams, grey		6	NQ RC	Rec. 98%									R.Q.D.=70%
			7	NQ RC	Rec. 100%									R.Q.D.=83%
99.4														
5.6	End of borehole Borehole dry before coring Water level not stabilized													

+³ ×³: Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No B1-4

1 OF 1

METRIC

W.P. 171-00-01 LOCATION Ramp FGGE(E)-Sherway Gardens Road, 4 830 459.5 N; 300 887.8 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		NATURAL MOISTURE CONTENT		LIQUID LIMIT		UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)							
108.9	Ground surface																
108.8	75 mm Topsoil																
0.1	FILL : Clayey Silt with shale/some organics, grey		1	SS	21												
108.2																	
107.9	CLAYEY SILT TILL with Silt &		2	SS	85/20												
107.8	Clayey Silt seams, brown, hard																
1.1	SHALE : grey																
	End of borehole																
	Refusal to further augering @ 1.1 m, probably on a limestone layer																
	Water level in piezometer																
	Feb. 28 - 0.8 m																
	March. 2 - 0.9 m																

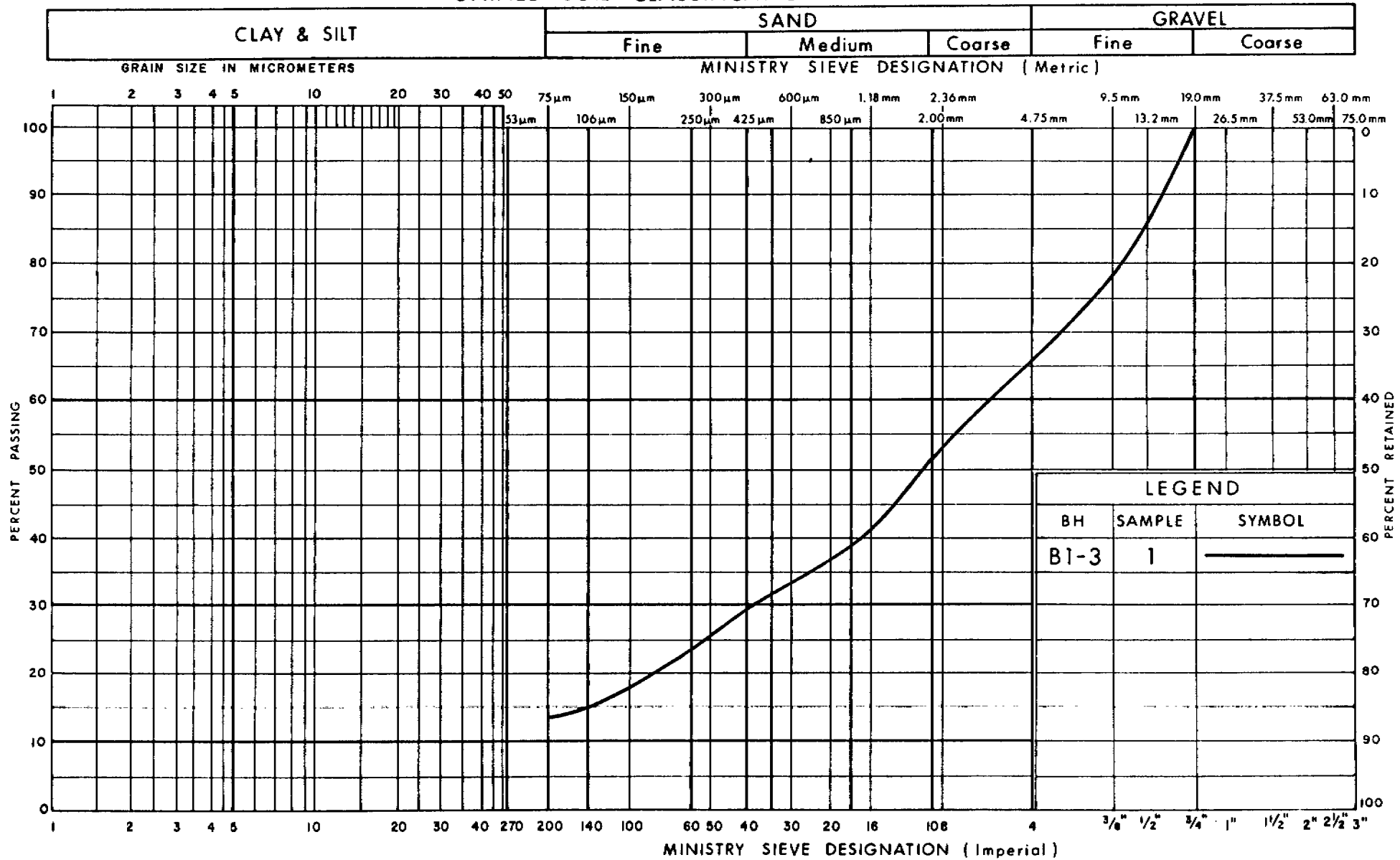
+ 3, x 3 : Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE

APPENDIX B

Laboratory Test Results

UNIFIED SOIL CLASSIFICATION SYSTEM



Ontario

Ministry of
Transportation

GRAIN SIZE DISTRIBUTION

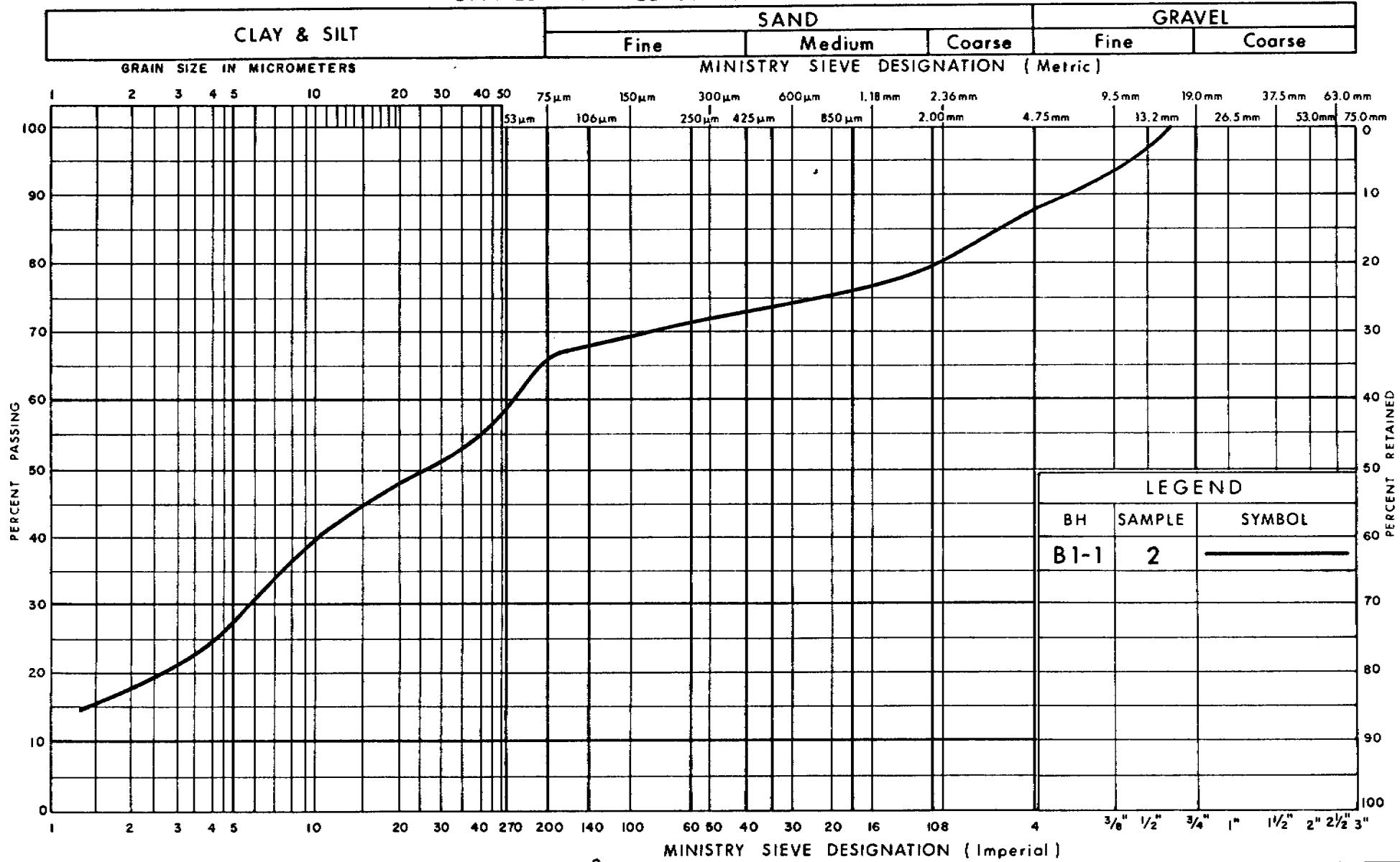
SAND WITH GRAVEL, SOME SILT (GRANULAR FILL)

FIG No 1

W P 171-00-01

SP 3232B

UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of
Transportation

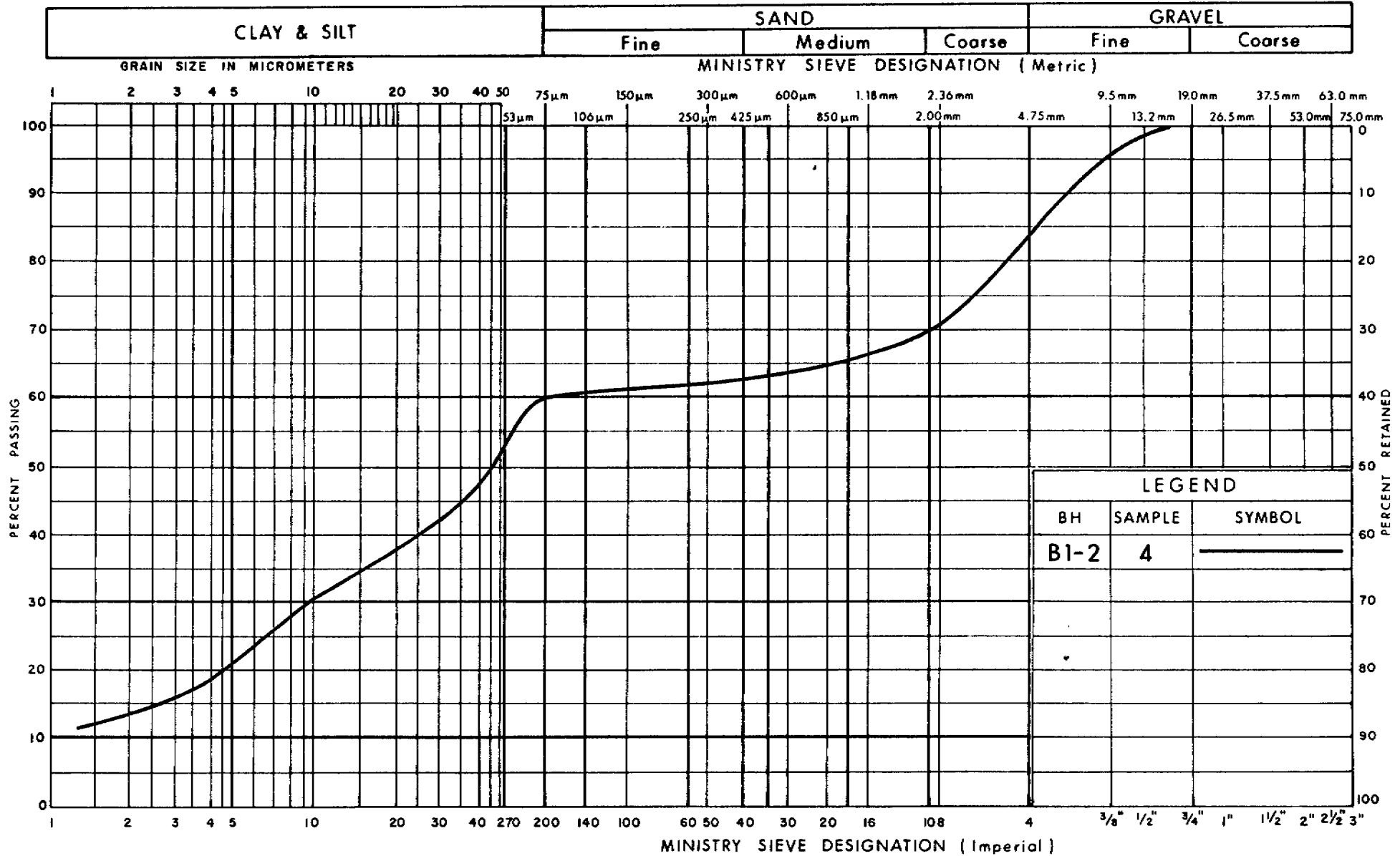
GRAIN SIZE DISTRIBUTION CLAYEY SILT TILL

FIG No 2

W P 171-00-01

SP 3232B

UNIFIED SOIL CLASSIFICATION SYSTEM



Ontario

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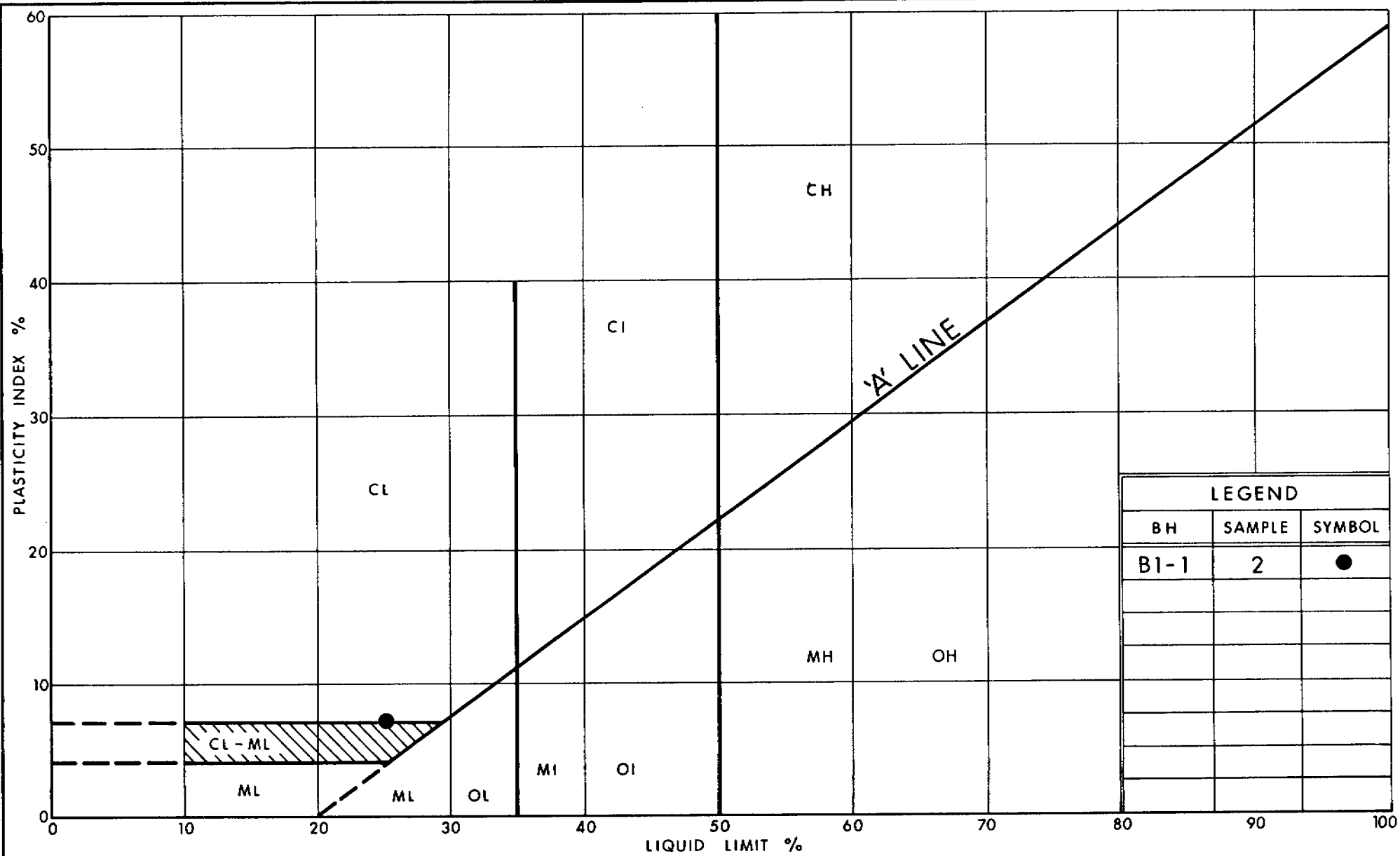
GRAIN SIZE DISTRIBUTION

TILL/SHALE Complex

FIG No 3

W P 171-00-01

SP 3232B



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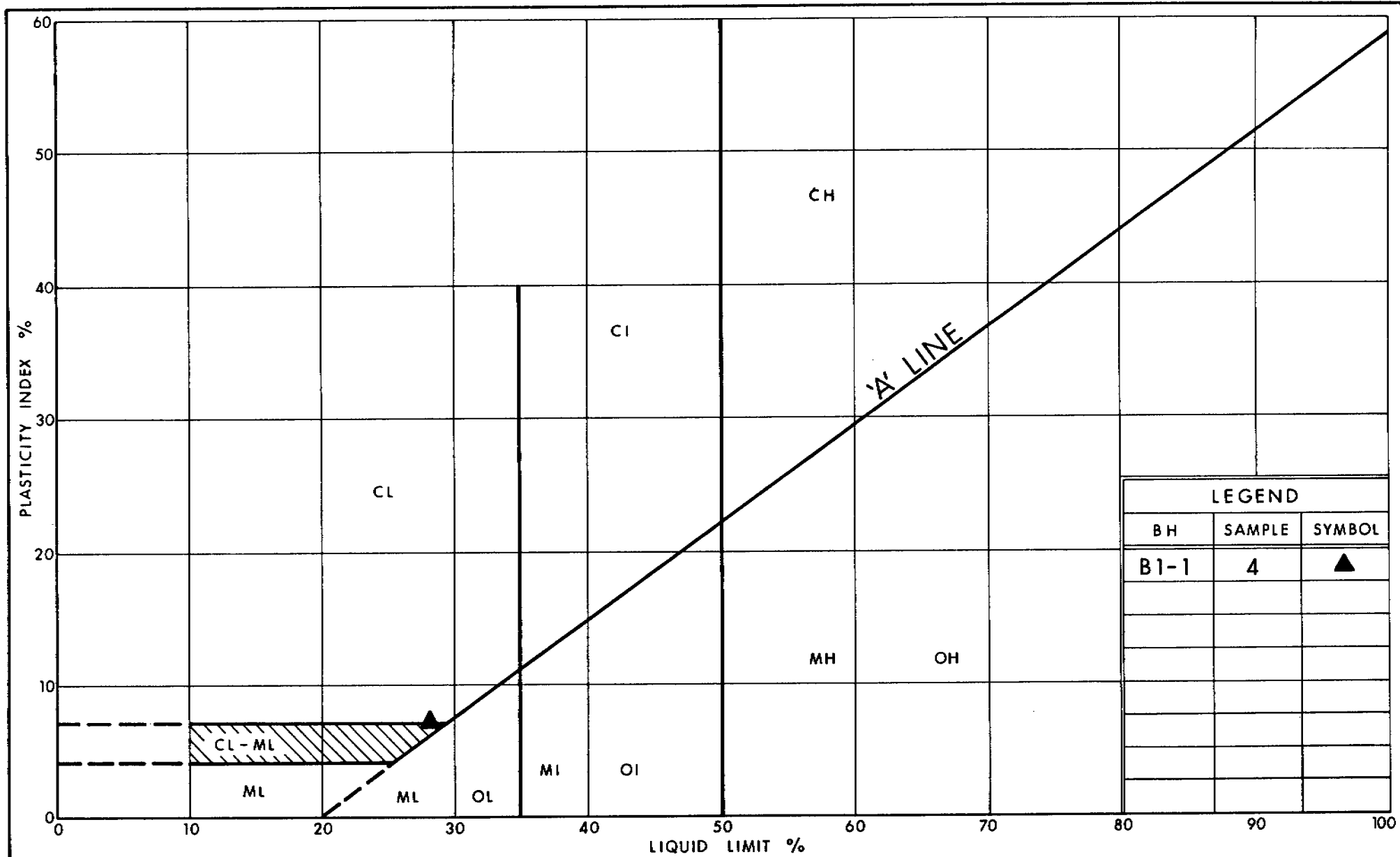
Ontario

PLASTICITY CHART CLAYEY SILT TILL

FIG No 4

W P 171-00-01

SP 3232B



Ministry of
Transportation

PLASTICITY CHART TILL/SHALE Complex

FIG No 5

W P 171-00-01

SP 3232B



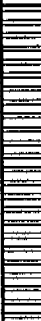
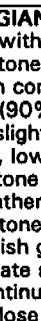
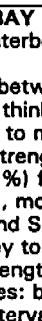
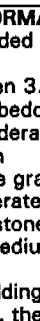
APPENDIX C

Core Logs and Photographs

CORE LOG

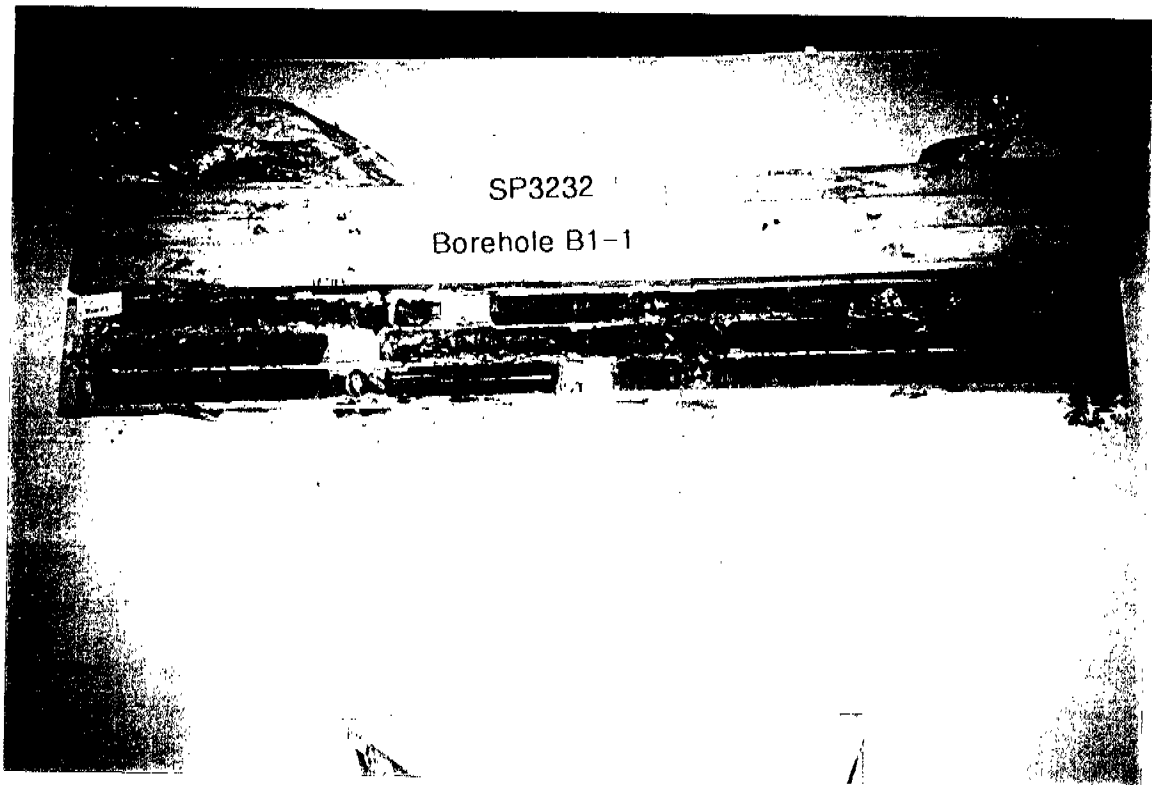
BH NO B1-1

PROJECT Foundation Investigation	ORIENTATION Vertical	ELEVATION (m) 111.3	DATUM Geodetic	PROJECT NO. SP3232
LOCATION Q.E.W./Hwy 427/Brown's Line Interchange Modification	DATE STARTED 02/12/00	COMPLETED 02/12/00	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	
111.3			OVERBURDEN: see soils log for description																
	1																		
	2																		
108.5			SHALE: sampled with split-barrel sampler; see soils log for description																
107.4			GEORGIAN BAY FORMATION: Shale with Interbedded Limestone and Sandstone Broken core between 3.91 m and 5.18 m Shale (90%) thinly bedded or laminated, dark grey, slightly to moderately weathered to 5.4 m, low strength Limestone (7%) fine grained, fossiliferous, unweathered, moderate strength Sandstone and Siltstone (3%) 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 300 mm (at 7.7 m), joint surfaces are smooth planar to rough planar Rubble seams of 20 mm noted at 6.29 m, 6.70 m, and 7.73 m	1	F	F	C VC	SP RP	T	0					1	49	0	100	grey
	5																		
	6													2	92	30	100	grey	
	7								R										
	8								R										
	9								R					3	100	67	100	grey	
102.2																			
	10																		
	11																		
	12																		
	13																		
	14																		
	15																		
	16																		
	17																		
	18																		
	19																		
			End of Borehole																

S & P

Shaheen & Peaker Limited
Consulting Geo-Environmental Engineers



Photograph of Core
Borehole B1-1
Elev. 107.4 m – 102.2 m

CORE LOG

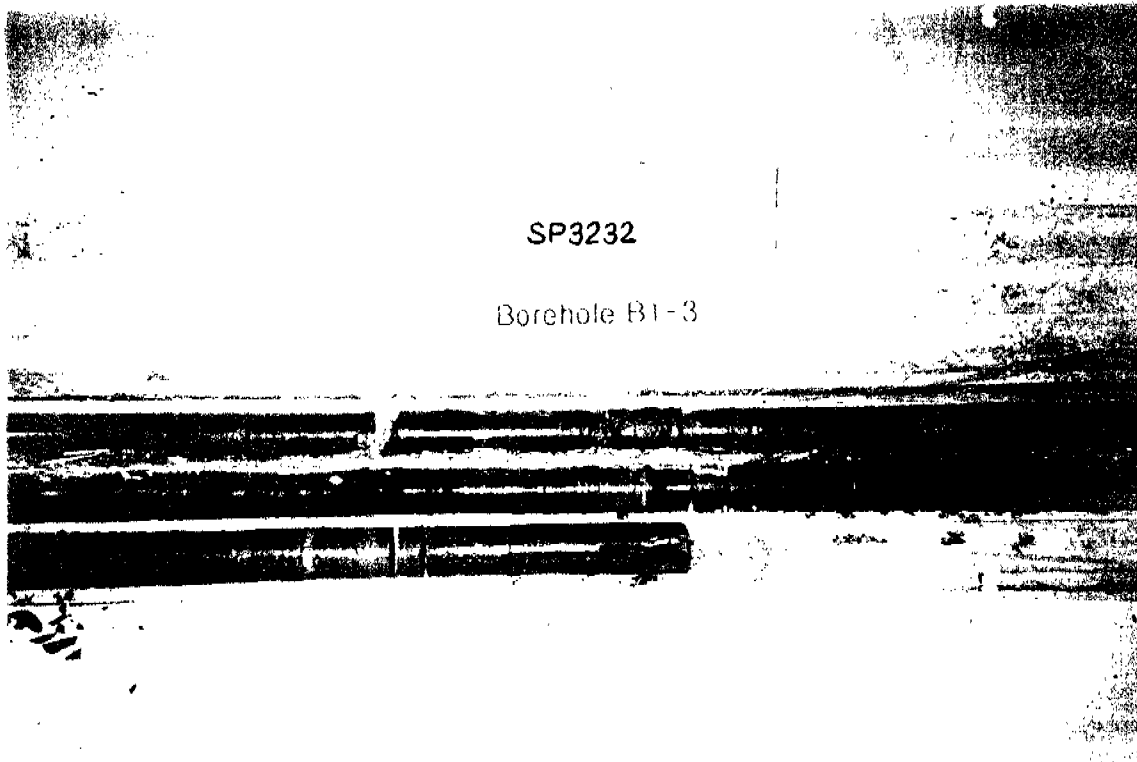
BH NO B1-3

PROJECT Foundation Investigation	ORIENTATION Vertical	ELEVATION (m) 105.0	DATUM Geodetic	PROJECT NO. SP3232
LOCATION Q.E.W./Hwy 427/Brown's Line Interchange Modification	DATE STARTED 02/15/00	COMPLETED 02/15/00	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
105.0			OVERBURDEN: see soils log for description															
103.6	1																	
	2		GEORGIAN BAY FORMATION: Shale with Interbedded Limestone and Sandstone	1	B	F	C VC	SP RP	T R R	0 5 5				1	100	70	100	grey
	3		Shale (96%) thinly bedded or laminated, dark grey, slightly to moderately weathered to 2.8 m, low strength															
	4		Limestone (1%) fine grained, fossiliferous, unweathered, moderate strength						SO	5				2	98	70	100	grey
	5		Sandstone and Siltstone (3%) stratified, brownish grey to medium grey, unweathered, moderate strength															
99.4	6		Discontinuities: bedding joints are at close to very close intervals, occasional vertical joints, the maximum thickness of limestone or sand/siltstone layers was about 90 mm, joint surfaces are smooth planar to rough planar	C	V									3	100	93	100	grey
	7		Rubble seams of 5 mm noted at 2.68 m and 3.0 m															
	8		Clay seam of 5 mm noted at 4.1 m															
	9		End of Borehole															
	10																	
	11																	
	12																	
	13																	
	14																	
	15																	
	16																	
	17																	
	18																	
	19																	


S & P


Shaheen & Peaker Limited
Consulting Geo-Environmental Engineers



Photograph of Core
Borehole B1-3
Elev. 103.6 m – 99.4 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	Orientations: 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	Fillings: Approx. ϕ_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	Strength of rock material:														
	Very high strength	= specimen can only be chipped by geological hammer	200 MPa												
	High strength	= specimen requires a number of blows of geological hammer to fracture it; cannot be scraped with pocket knife	50 - 200 MPa												
	Medium strength	= specimen can be fractured by single firm blow of geological hammer; can be scraped with pocket knife, not peeled	15 - 50 MPa												
	Low strength	= shallow indentations made by firm blow with point of geological hammer; can be peeled by pocket knife with difficulty	4 - 15 MPa												
	Very low strength	= crumbles under firm blow with point of geological hammer; can be peeled by pocket knife	1 - 4 MPa												
14	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.														
		<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	Run Number and Core Recovery: (i) Drill run number; (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.														
16	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.														
	<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	Core and Casing Sizes: changes of core and casing sizes are indicated.														
18	Water recovery, level and tests.														

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

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

STRESS AND STRAIN

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

MECHANICAL PROPERTIES OF SOIL

m_v	kPa ⁻¹	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_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 = $\frac{c_u}{\tau_r}$

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 1960's

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

JOB 65-P-104W.P. 275-64-1DATUM G.S.C.

RECORD OF BOREHOLE NO. 8

FOUNDATION SECTION

LOCATION 178,658 N 209,540 EORIGINATED BY P.McBORING DATE Oct. 1, 1965COMPILED BY H.S.BOREHOLE TYPE Washboring - EX Casing.CHECKED BY all

SOIL PROFILE		SAMPLES		ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE		LIQUID LIMIT — W _L PLASTIC LIMIT — W _P WATER CONTENT — W _P		BULK DENSITY P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	NUMBER	TYPE		BLOWS / FOOT	SHEAR STRENGTH P.S.F.	W _L	W _P		
369.0	Groundlevel									
0.0	Silty sand. Loose.	1	SS	5						
5.0	Clayey silt with some sand & gravel. (Glacial Till) Very dense.	2	SS	88 for 9"						
356.0		3	SS	50 for 3"						
13.0	Shaley limestone with intermittent limestone.	4	HC	59%						
349.0		5	HC	94%						
20.0	End of borehole.									

Refusal at 5.4'

Blocked dry
4.0'

GEOTECHNICAL DATA SHEET FOR BOREHOLE . . 88 . .

OUR REFERENCE NO. 6-6-13

CLIENT: D.H.O.

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

LOCATION: 178,488 N 209,884 E

DATUM ELEVATION: G.S.C.

METHOD OF BORING: AUGERING & WASHBORING

DIAMETER OF BOREHOLE: 4 1/2" - 2 3/4"

DATE: JULY 5, 1966

W.P. 238-81-1

ENCLOSURE NO.

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES		PENETRATION RESISTANCE Blows per foot					CONSISTENCY water content %		REMARKS
				NUMBER	TYPE	20	40	60	80	100	PL	W	
347.5	0	GROUND SURFACE											
346.8	1.0	TOPSOIL & FILL											
345.0		Compact to Dense Brown FINE SAND		1	AS								W.L. El. 364.8 ft. July 6, 1966
341.7	5.8	Very Dense Grey SANDY SILT with some gravel and shale fragments (GLACIAL TILL)		2	SS	71/77							
340.0				3	SS	74/79							
335.0	15.0	Grey SHALE with bands of limestone		4	R.C.	44%							
330.0				5	R.C.	48%							
325.0		BEDROCK		6	R.C.	60%							
340.0	30	END OF BOREHOLE											

VERTICAL SCALE: 1 IN TO 5 FT.

DOMINION SOIL INVESTIGATION LIMITED

MADE: C.K. CHD.

OUR REFERENCE NO. 6-6-16 GEOTECHNICAL DATA SHEET FOR BOREHOLE 89

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 #	DEPTH #	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES		PENETRATION RESISTANCE		CONSISTENCY water content % PL W LI	REMARKS
				NUMBER	TYPE	blows per foot	SHEAR STRENGTH lbs/sq ft		
368.9	0	GROUND SURFACE							
367.4	1.5	Brown SAND (FILL)							
365.0	5	Dark Brown CLAYEY SILT with a trace of SAND and GRAVEL (FILL)							
363.9	5.9	Organic TOPSOIL		1A	SS	55			
363.0		Dense FINE SAND with some SILT		1B					
361.4	7.5	Very Dense Grey SAND and SILT		2	SS	50/4"			
360.0	10	with numerous SHALE fragment and a trace of embedded fine GRAVEL (GLACIAL TILL)		3	SS	65/4"			
				4	SS	100/4"			
355.0	15			5	SS	100AMP			
353.9		Grey SHALE BEDROCK		6	RC	85%			
350.0	20								
348.7		END OF BOREHOLE							

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

100/5"

GEOTECHNICAL DATA SHEET FOR BOREHOLE . . 90 . .

OUR REFERENCE NO. 6-6-14

CLIENT: D.H.O.
PROJECT: BRIDGE No. 3, Q.E.W. & HWY. 27.
LOCATION: 178,590 N ; 209,356 E
DATUM ELEVATION: G.S.C.

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

ENCLOSURE NO.

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES		PENETRATION RESISTANCE		CONSISTENCY		REMARKS
				NUMBER	TYPE	blows per foot	lb./sq. ft.	PI	W	
369.7	0	GROUND SURFACE								
		Compact to Dense Brown SILTY FINE SAND with a trace of CLAY		1	AS					Gr. 67 % Si. 30 % ; Cl. - 3 % W.L. 365.2 Ft. JULY 6, 1966.
365.0	5			2	SS	39				
368.7	6.0	Very Dense Grey SILTY SAND with a trace of GRAVEL and CLAY		3	SS	71/5"				Gr. 8 % ; Si. 59 % Si. 25 % ; Cl. 8 %
360.0	10			4	SS	75/5"				
367.2	12.5	Grey SHALE with intermittent layers of LIMESTONE		5	RC	84 %				
359.0	15			6	RC	40 %				
350.0	20	BEDROCK		7	RC	18 %				
346.0	25			8	RC	76 %				
340.0	30	END OF BOREHOLE								
335.0	35									

VERTICAL SCALE: 1 IN TO 5 FT

DOMINION SOIL INVESTIGATION LIMITED

MADE: V.G.H. CHD.

GEOTECHNICAL DATA SHEET FOR BOREHOLE . . . 115.

OUR REFERENCE NO. G-6-13

CLIENT: D.H.O.

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

METHOD OF BORING: WASHBORING.

ENCLOSURE NO.

LOCATION: 178,468 N 209,454 E

DIAMETER OF BOREHOLE: 3 1/2"

DATE: JUNE 30 - JULY 8, 1966

DATUM ELEVATION: G.S.C.

W.P. 238-BI-1

ELEVATION + DEPTH	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE blows per foot					CONSISTENCY water content %		REMARKS
			NUMBER	TYPE	TEST	2.0	4.0	6.0	8.0	10.0	PL	W	
367.5	0 GROUND SURFACE												
365.0	6" TOP SOIL												
363.5	SANDY, CLAYEY SILT (FILL)												
362.5	Dense, Brown FINE SAND		1	SS	60								
360.0	Dense, Grey SANDY SILT with some gravel (GLACIAL TILL)												
357.5	10		2	SS	100/2								
355.0	Grey EXTREMELY WEATHERED SHALE		3	R.C.	80 %								
350.0	18		4	SS	100/2								
345.0	20		5	R.C.	10 %								
340.0	BEDROCK		6	SS	100/2								
335.0	25		7	R.C.	10 %								
330.0	30		8	R.C.	0 %								
325.0	35		9	R.C.	80 %								
	Sound												

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

VERTICAL SCALE 1 IN = 10 FT

DOMINION SOIL INVESTIGATION LIMITED

MADE: C.K. CHD

**FOUNDATION DESIGN REPORT
PROPOSED BRIDGE NB-1
RAMP FGGE E-SHERWAY GARDENS ROAD
OVER BROWN'S LINE
TORONTO, ONTARIO
W.P. 171-00-01
SITE 37-1519**

Prepared For:

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

Prepared by:

SHAHEEN & PEAKER LIMITED

**Project: SP3232B
June 8, 2000**

**250 Galaxy Boulevard
Etobicoke, Ontario
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APPENDICES

Limitations of Report

Appendix F

**FOUNDATION DESIGN REPORT
PROPOSED BRIDGE NB-1
RAMP FGGE E – SHERWAY GARDENS ROAD
OVER BROWN'S LINE
TORONTO, ONTARIO
W.P. 171-00-01
SITE 37-1519**

5. DISCUSSION AND RECOMMENDATIONS

The proposed bridge which will carry proposed Ramp FGGE E – Sherway Gardens Road over the existing Brown's Line corridor and associated ramps, will be an approximately 10 m wide, two span structure. As presently proposed, the west span is 34 m long while east span is 32 m, for a total length of about 66 m. The approximate proposed elevation for the proposed bridge is 111 m while the elevation of the existing road beneath is 105 m.

At two of the four boreholes drilled for this investigation, the ground surface elevation was somewhat higher than the general bedrock surface elevation in the area before the present interchange was constructed in the 1960's. These two boreholes (i.e. Boreholes B1-1 and B1-2) were drilled from the top of the existing ramp embankments at Elevations 111.3 and 111.7 m, respectively and they contacted about 1 m of fill underlain by clayey silt till and till/shale complex overburden to about 3 m below the ground surface to the surface of the bedrock at about Elevations 108.5 and 108.8 m, respectively. Available borehole information from MTO (Borehole Log Sheets presented in Appendix E) indicate that the surface of the bedrock in the general area was contacted at Elevations ranging from about 109 to 108 m, in the boreholes drilled before the construction of the existing interchange.

5.1 FOUNDATIONS

The boreholes show that the proposed bridge structure can be supported on normal spread footing foundations extended below the fill and the upper weak zones of the natural stratum.

The following table presents the recommended soil and bedrock resistances at various depths and elevations at the borehole locations.

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
B1-1 West Abutment	111.3	2.0	109.3	800	400	till/shale
		2.7	108.6	1000	600	till/shale
		3.3	108.0	1000	1000**	shale
		5.3	106.0	1500	1500**	shale
B1-2 West Abutment	111.7	2.1	109.6	800	400	till/shale
		2.7	109.0	1000	600	till/shale
		3.2	108.5	1000	1000**	shale
		5.3	106.4	1500	1500**	shale
B1-3 Central Pier	105.0	1.3	103.7	1000	600	weathered shale
		1.8	103.2	1000	1000**	shale
		2.6	102.4	1500	1500**	shale
B1-4 East Abutment ***	108.9	0.7	108.2	800	400	clayey silt till
		1.0	107.9	1000	600	weathered shale
		1.7	107.2	1000	1000**	shale
		2.5	106.4	1500	1500**	shale

* 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, approved bedrock surface.

*** Previously drilled MTO boreholes indicate that where the surface of the bedrock was not previously excavated, bedrock may be encountered at a higher elevation (i.e. 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, the unsuitable materials should be removed and replaced with mass concrete, under the direction of the geotechnical engineer.

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 $106\pm$ m and $105\pm$ m for the east and west abutments, respectively. Table 1 shows that for the design of the spread footings for the abutment foundations a factored rock bearing resistance at U.L.S. of up to 1500 kPa can be used at or below Elevation 106.0 m 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 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. It is however believed that because of the very close proximity of the existing bridges, the use of abutments founded on engineered fill will not be feasible from a constructability point of view.

We understand that as presently proposed the central pier will be supported on drilled and cast-in-place concrete pile (caisson) foundations. Based on the Borehole B2-3 results, for caisson design a value of up to 5000 kPa can be used for U.L.S. at or below Elevation 101.4 m (i.e. at least 3.6 m below the existing grade at the borehole location). The minimum caisson length below the finished grade should be 3.6 m.

For the evaluation of horizontal resistance of caisson installations, 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 bedrock can be taken as 1200 kPa within a depth of 2 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 unfactored c_u value can be assumed to be 1200 kPa within 2 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 Elevation 108.0 (Borehole B1-4) and 106.6 m (Borehole B1-2). 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 may create local problems but in our opinion, if they occur, 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 mud mat would also 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 the slopes may have to be locally flattened. 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 as these hard layers will significantly increase the time required to auger into the bedrock. Problems due to excessive groundwater seepage into caisson excavations have been known to occur occasionally and should it happen, this will necessitate measures such as vigorous pumping.

All 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 the calculation of 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.

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_o condition of the retained material.

We also recommend that by means of good construction techniques, the undermining of the existing bridge foundations should be avoided.

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

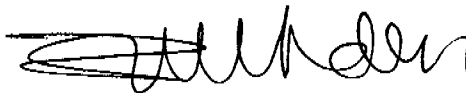
Design frost penetration for the general area is 1.2 m. Therefore, a permanent soil cover of 1.2 m or its thermal equivalent is required for frost protection of foundations, including those founded on the shale bedrock.

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 E, are an integral part of this report.

Shaheen & Peaker Limited



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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.