

GEORE'S No:  
30M11-208

**FOUNDATION INVESTIGATION REPORT  
RAMP FGGE E – SHERWAY GARDENS ROAD RETAINING STRUCTURE  
AT NORTH ABUTMENT OF EXISTING BRIDGE NO. 4  
TORONTO, ONTARIO  
W.P. 176-00-01**

NR1

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**Project: SP3232K  
June 30, 2000**

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**1. INTRODUCTION**

Gardiner Expressway/QEW/Highway 427/Brown's Line Interchange modifications will involve the construction of a new ramp which will carry FGGE E-Sherway Gardens Road. The new ramp will pass under the existing Bridge #4 and will come close to the existing pile cap of the north abutment, especially at the east side. This will necessitate permanent shoring in the form of a retaining structure.

Shaheen & Peaker Limited (S&P) was retained by DS-Lea Associates Limited, Consulting Engineers, to carry out a foundation investigation consisting of one borehole.

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

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. FIELD AND LABORATORY WORK**

The fieldwork for the project was performed on February 15 and March 17, 2000 and consisted of drilling and sampling one borehole (Boreholes B3-1) at the location shown on Drawing No. 1.

The borehole was advanced using 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 borehole 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.

Augering was stopped at a depth of 8.8 m and the borehole was extended by diamond drilling and NQ rock coring to a depth of 17.3 m.



The soil samples and rock cores were shipped to our laboratory in Toronto for further examination and classification. A laboratory testing programme, consisting of grain size analyses, natural moisture content, Atterberg Limits and bulk unit tests, 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.

The geodetic elevation and coordinates of the borehole were provided to us by Waylaw Technical Services of Paris, Ontario.

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

Details of the subsurface conditions encountered in the borehole drilled for this investigation (i.e. Borehole B3-1) are given on the Borehole Log Sheets in Appendix A. The Record of Borehole Sheets for boreholes drilled by MTO in the general area in 1965 and 1966 (for the existing interchange structure) are given in Appendix E.

A description of the various strata contacted in Borehole B3-1 is given in the following paragraphs.

##### **4.1 TOPSOIL**

The borehole encountered a 175 mm thick topsoil layer.

##### **4.2 FILL**

Underlying the topsoil, the borehole contacted embankment fill extending to a depth of 7.5 m or to Elevation 110.8 m.

The composition of the fill encountered in the borehole was quite variable and ranged from gravelly sand with some silt to sandy silt and clayey silt with some gravel and shale fragments. The presence of traces of topsoil and occasional pieces of metal, asphalt and concrete mixed with the fill was noted.

The grain size distribution of two samples from the gravelly sand zones of the fill is given in Figure 1 in Appendix B.

The presence of organic clayey silt and topsoil was noted below about Elevation 111.5± m and this may represent the original ground surface before the existing interchange was built.

It should be pointed out that the thickness of topsoil and the thickness and composition of man-made fill deposits can vary beyond the borehole location.

#### 4.3 CLAYEY SILT TILL

The uppermost inorganic natural soil, underlying the fill at the borehole location, is a glacial till deposit. It consists of a heterogeneous mixture of clayey silt with some sand and gravel. Embedded shale and limestone fragments are also common. The presence of silty clay till and clayey silt seams was also noted. The grain size distribution of a sample with silty clay till zones (seams) is given in Figure No. 2 in Appendix B. The plasticity chart of the same sample is given in Figure No. 3 (Appendix B).

The till was contacted at 7.5 m below the ground surface or at Elevation 110.8 m and extended to 8.8 m or to Elevation 109.5 m.

The clayey silt till is a cohesive material and, from the recorded N-values of 19 and in excess of 95 blows/0.3 m, its consistency is described as very stiff to hard.

The frequency of the shale and limestone fragments in the glacial till increases with increasing depth and the lower portion of the till resembles a highly weathered shale which is sometimes referred to as till/shale complex. It represents a transition zone into the underlying bedrock. This material is described as a residual soil or a completely weathered shale bedrock. Shale and limestone slabs (layers) may remain. Excavations methods in the till, till/shale and in the underlying shale bedrock should, therefore, take into account the possible presence of hard shale or limestone slabs/layers.

#### 4.4 SHALE BEDROCK

The borehole contacted shale bedrock at about Elevation 109.5 m.

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 occasional limestone and siltstone seams was noted in the cores obtained from the bedrock. They were generally 2 to 15 cm thick except for a 215 mm thick seam at about Elevation 106.0 m. It should also be pointed out that up to 50 mm thick clay seams were contacted at various elevations.

The percentage of recovery of the cores obtained from the bedrock ranged from 93 to 100%. The measured R.Q.D. (Rock Quality Designation) values ranged from 25 to 76%, indicating a poor to fair quality.

#### 4.5 GROUNDWATER CONDITIONS

Water conditions in the borehole were observed while augering and the borehole was dry during this period and at the completion of augering. However, water was introduced in the borehole to expedite diamond drilling below about Elevation 110 m. For this reason, stabilized water level could not be observed.

The boreholes, drilled in the vicinity of the site for the present and the previous boreholes, generally show a groundwater table between 111 and 110 m.

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

**Shaheen & Peaker Limited**

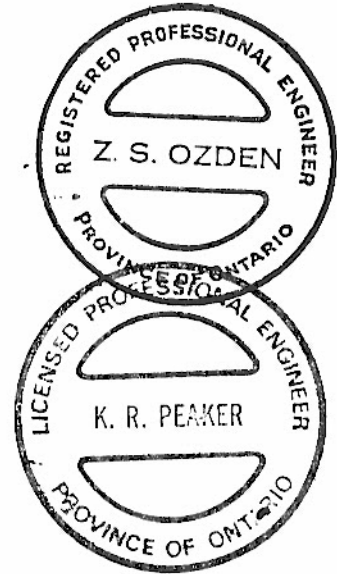


Zuhtu Ozden, P.Eng.



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

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# APPENDIX A

## Borehole Log Sheets

# RECORD OF BOREHOLE No B3-1

1 OF 2

METRIC

W.P. 176-00-01

LOCATION

Ramp FGGE E- Under N Span of Existing Bridge # 4; 4 830 391.8 N, 300 732.3 E ORIGINATED BY G.I

DIST

HWY 427 & QEW

BOREHOLE TYPE

Solid Stem Augers, NQ Rock Core

COMPILED BY G.I

DATUM Geodetic

DATE

15.02.00 17.03.00

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 <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	TEST VALUES			20	40	60	80	100					
118.3	Ground surface																
118.1	175mm Topsoil																
0.2	MIXED FILL:  Gravelly Sand, some silt with Sandy Silt & Clayey Silt zones, some gravel, shale fragments, trace of topsoil, occasional pieces of metal, asphalt, concrete, brown/dark brown/grey		1	SS	18		118										27 55 (18)
			2	SS	12		117									21.0	
			3	SS	24		116									20.1	
			4	SS	7		115									19.6	
			5	SS	14		114									20.1	
			6	SS	81/20		113										38 45 14 3
			7	SS	43		112										
			8	SS	19		111										
			9	SS	50/8		110										
			10	SS	25		109										
110.8	organic clayey silt & topsoil						108										
7.5	CLAYEY SILT TILL with Silty Clay Till and clayey silt seams, brown, very stiff to hard		11	SS	19		107										4 18 45 33
			12	SS	95/18		106										
109.5	TILL/SHALE complex						105										
8.8	SHALE BEDROCK occasional limestone seams grey		13	NQ RC	Rec. 93%		104										RQD= 25%
			14	NQ RC	Rec. 97%		103										RQD=32%
			15	NQ RC	Rec. 100%		102										RQD=76%
	weathered 215 mm limestone seam		16	NQ RC	Rec. 100%		101										no water return while drilling RQD=36%
			17	NQ RC	Rec. 100%		100										RQD=65%

Continued Next Page

+ 3 . x 3 : Numbers refer to Sensitivity

20  
15 10 5 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No B3-1

2 OF 2

METRIC

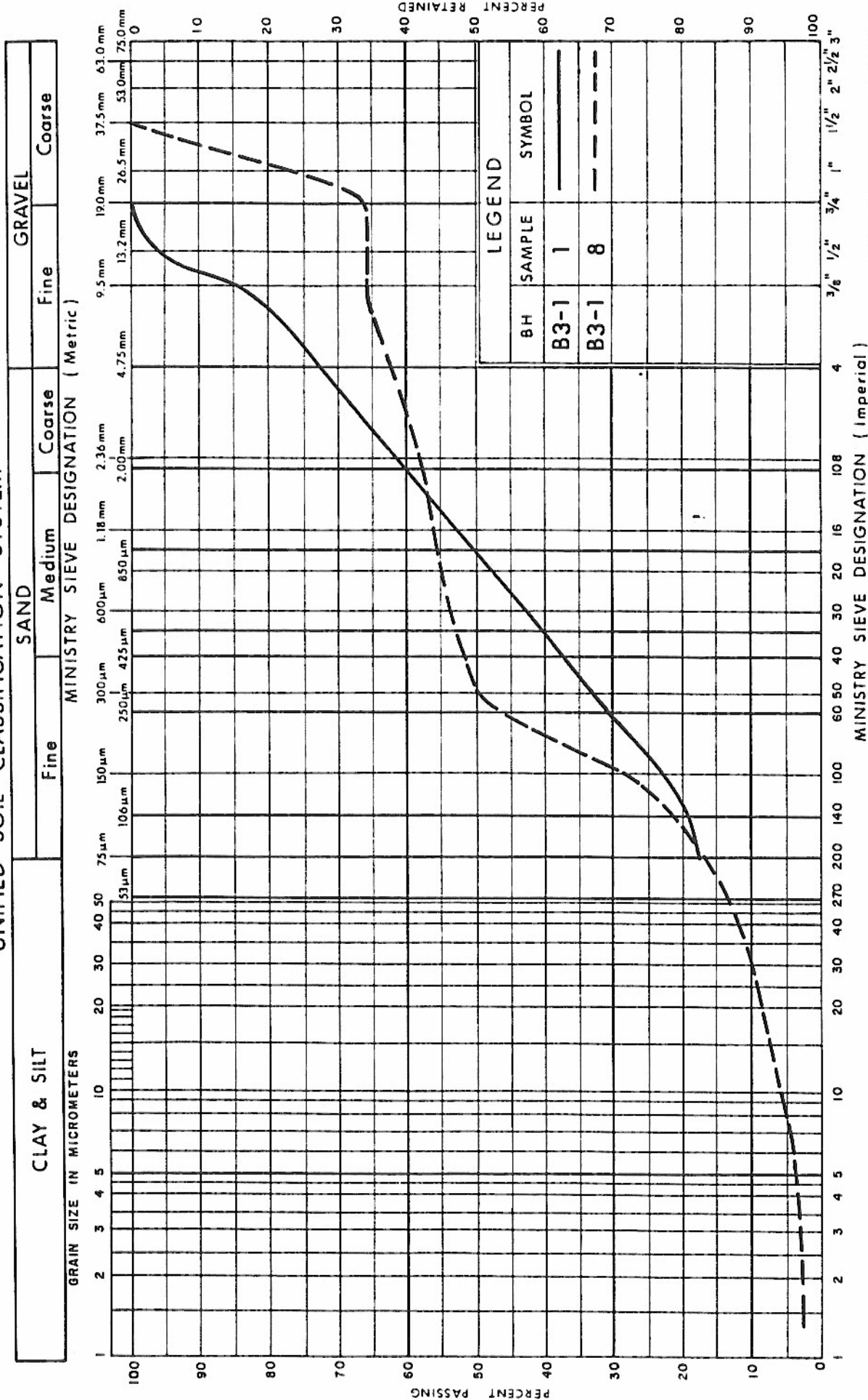
W.P. 176-00-01 LOCATION Ramp FGGE E- Under North Span of Existing Bridge # 4; 4 830 391.8N; 300 732.1E ORIGINATED BY G.I.  
DIST HWY 427 & QEW BOREHOLE TYPE Solid Stem Augers, NQ Rock Core COMPILED BY G.I.  
DATUM Geodetic DATE 15.02.00 17.03.00 CHECKED BY S.B.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			20	40	60	80	100		
							SHEAR STRENGTH kPa						
							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE						
							20	40	60	80	100		
							WATER CONTENT (%)						
							20	40	60				
							PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT						
							W <sub>p</sub>	W		W <sub>L</sub>			
101.0	SHALE BEDROCK occasional limestone seams grey		17	NQ RC	Rec. 100%	103							RQD=65%
			18	NQ RC	Rec. 100%	102							RQD=48%
17.3	End of borehole Borehole dry prior to coring Water level not stabilized												

# APPENDIX B

## Laboratory Test Results



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## GRAIN SIZE DISTRIBUTION

GRAVELLY SAND, SOME SILT (FILL)

FIG No 1

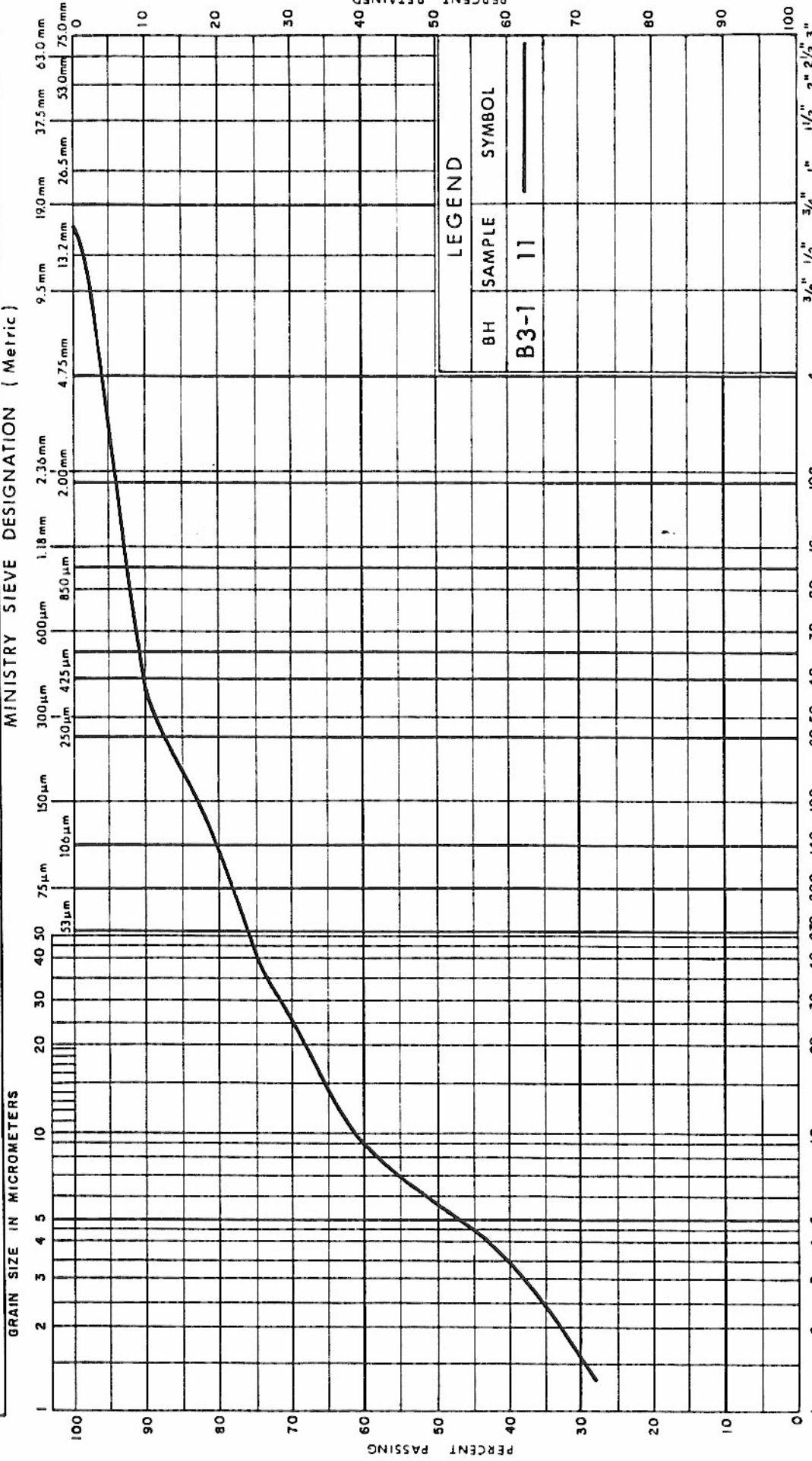
WP 176-00-01

SP 3232K

# UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT		SAND			GRAVEL	
		Fine	Medium	Coarse	Fine	Coarse

GRAIN SIZE IN MICROMETERS



## LEGEND

BH	SAMPLE	SYMBOL
B3-1	11	—

Ministry of  
Transportation



## GRAIN SIZE DISTRIBUTION

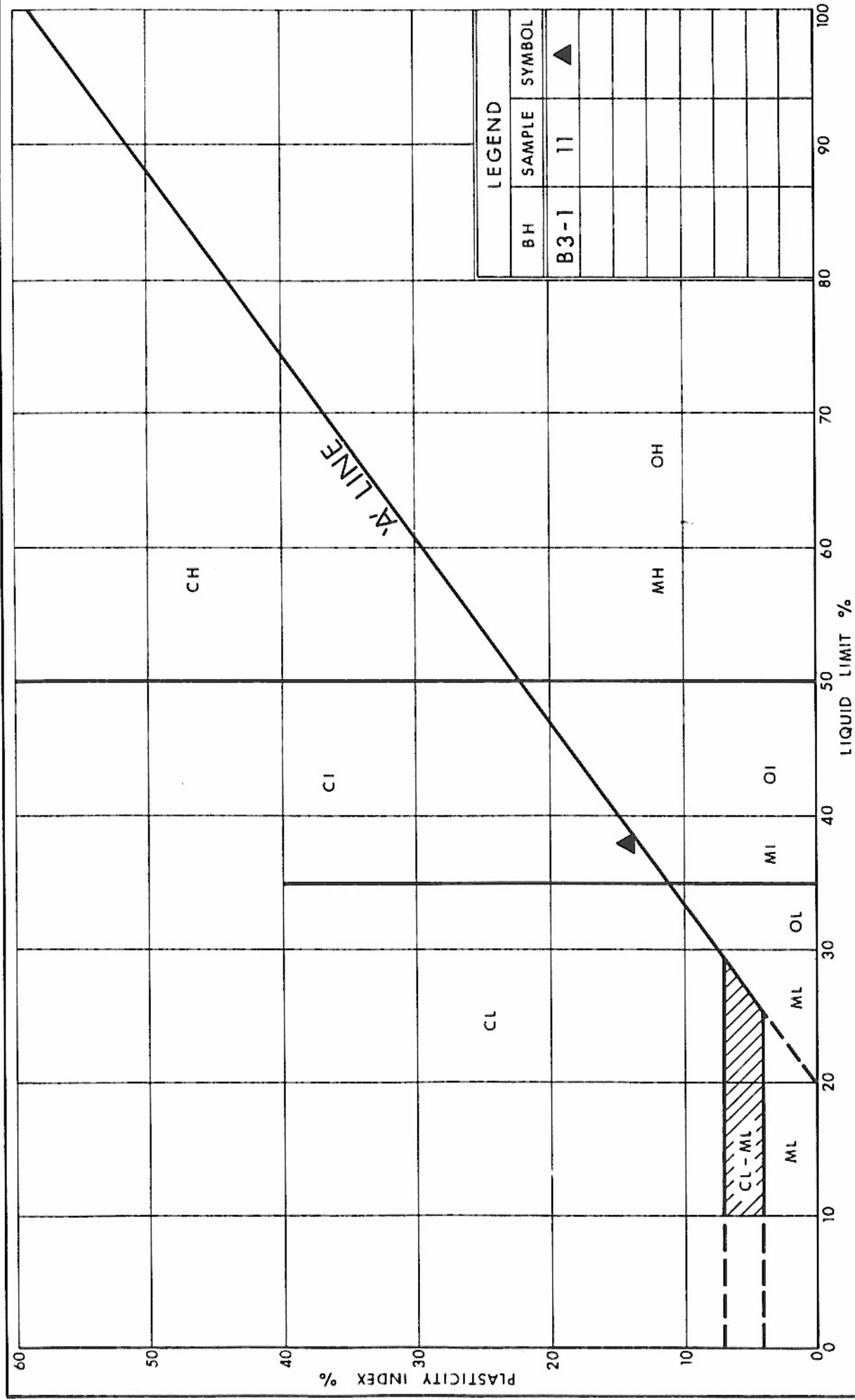
CLAYEY SILT TILL (WITH SILTY CLAY TILL SEAMS)

FIG No 2

W P 176-00-01

SP 3232K

Oct 75, FF-S-21



# PLASTICITY CHART

CLAYEY SILT TILL (WITH SILTY CLAY TILL SEAMS)

Ministry of  
Transportation



FIG No 3

W P 176-00-01

SP 3232K

## APPENDIX C

### Core Log

# CORE LOG

BH NO B3-1

PROJECT Foundation Investigation	ORIENTATION Vertical	ELEVATION (m) 118.3	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
118.3			OVERBURDEN: see soils log for description																
	1																		
	2																		
	3																		
	4																		
	5																		
	6																		
	7																		
	8																		
109.5	9		GEORGIAN BAY FORMATION:	2	B	F	C	SP	T	0					1	93	25	100	gr
			Shale with Interbedded Limestone and Sandstone		C	D	VC	RP	SO	40									
	10		Shale (86.5%) thinly bedded or laminated, dark grey, generally unweathered, low strength						SO	50					2	97	32	100	grey
					C	V			SO	40									
	11		Limestone (3.8%) fine grained, fossiliferous, unweathered, moderate strength						SO	30									
									SO	20									
	12		Sandstone and Siltstone (9.7%) stratified, brownish grey to medium grey unweathered, moderate strength	1	C	V	C	SP	T	0					3	100	76	100	grey
					B	F	VC	RP	SO	20									
	13		Discontinuities: bedding joints are at close to very close intervals, the maximum thickness of limestone or sand/siltstone layers was about 215 mm, joint surfaces are smooth planar to rough planar		C	V			SO	25									
									SO	20									
	14		Clay seams noted at various depths: 8.8 m (40 mm), 9.47 m (50 mm), 9.81 m (40 mm), 10.21 m (30 mm), 11.05 m (20 mm), 11.10 m (20 mm), 12.25 m (25 mm), 13.17 m (20 mm), 13.34 m (25 mm), 14.3 m (15 mm), 14.35 m (30 mm), 16.55 m (75 mm)						SO	25					4	100	36	0	
			Rubble zone of 10 mm thickness noted at 11.15 m depth.						SO	15									
	15								SO	30									
	16														5	100	65	100	grey
	17																		
101.0	18		End of Borehole												6	100	48	100	grey
	19																		

**S & P**

Shaheen & Peaker Limited  
Consulting Geo-Environmental Engineers

# EXPLANATORY SHEET TO CORE LOG

Column No.	Description	Column No.	Description	Approx. Uniaxial Compressive Strength
1	Elevation of geotechnical boundary.	13	Strength of rock material: Very high strength = specimen can only be chipped by geological hammer High strength = specimen requires a number of blows of geological hammer to fracture it; cannot be scraped with pocket knife Medium strength = specimen can be fractured by single firm blow of geological hammer; can be scraped with pocket knife, not peeled Low strength = shallow indentations made by firm blow with point of geological hammer; can be peeled by pocket knife with difficulty Very low strength = crumbles under firm blow with point of geological hammer; can be peeled by pocket knife	200 MPa
2	Depth of geotechnical boundary in borehole.	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.	50 - 200 MPa
3	Geological symbol for rock or soil material.		Fracture frequency 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 - 50 MPa
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.		Joint spacing Very wide = 3 m Wide = 1 - 3 m Moderate = 30 cm - 1 m Close = 5 - 30 cm Very close = 5 cm	4 - 15 MPa
5-11	Joint (discontinuity) characteristics		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.	1 - 4 MPa
5	Number of joint sets: a rock mass can be intersected by a number of joint sets of varying orientations.	15	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.	
6	Joint type: B = Bedding Joint F = Fault C = Cross Joint S = Shear Plane		RQD 0 - 25 % very poor 25 - 50 % poor 50 - 75 % fair 75 - 90 % good 90 - 100 % excellent	
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°	16	Core and Casing Sizes: changes of core and casing sizes are indicated. Water recovery, level and tests.	
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	17		
9	Roughness: RU = Rough Undulating RP = Rough Planar SU = Smooth Undulating SP = Smooth Planar LU = Slickensided Undulating LP = Slickensided Planar	18		
10	Filling: T = Tight, hard, non-softening O = Oxidation surface staining only SA = Slightly altered; clay-free S = Sandy particles, clay-free Si = Sandy and silty, minor clay NC = Non softening clays ( 5 mm) SO = Softening clays ( 5 mm) SC = Swelling clay fillings ( 5 mm)			
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			

## 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 <sup>1</sup>	SPLIT SPOON	T P	THINWALL PISTON
WS	WASH SAMPLE	OS	OSTERBERG SAMPLE
ST	SLOTTED TUBE SAMPLE	RC	ROCK CORE
BS	BLOCK SAMPLE	PH	TW ADVANCED HYDRAULICALLY
CS	CHUNK SAMPLE	PM	TW ADVANCED MANUALLY
TW	THINWALL OPEN	FS	FOIL SAMPLE

### STRESS AND STRAIN

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

### MECHANICAL PROPERTIES OF SOIL

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

### PHYSICAL PROPERTIES OF SOIL

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



## APPENDIX E

### Logs Boreholes Drilled in 1960's

# OUR REFERENCE NO. 6-6-15 GEOTECHNICAL DATA SHEET FOR BOREHOLE . 97 .

CLIENT: D. H. O.

PROJECT: Q.E.W. & HWY. No. 27. INTERCHANGE

LOCATION: 178,259 N ; 208,972 E

DATUM ELEVATION: 0. S.C.

METHOD OF BORING: AUGERING

DIAMETER OF BOREHOLE: 4 1/2"

DATE: JUNE 29. 1966.

W.P. 238 - 61 - 3

ENCLOSURE NO.

ELEVATION #	DEPTH #	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE blows per foot		CONSISTENCY water content %	REMARKS			
				NUMBER	TYPE	Advancement of sampler 1/2"	20	40	60	80	100		
							SHEAR STRENGTH lbs/sq ft						
355.8	0	GROUND SURFACE											
		12" TOPSOIL											
		Greenish Brown Compact to Very Dense SANDY SILT with some GRAVEL and a trace of CLAY (GLACIAL TILL)		1	AS								
	5			2	SS	27							
	10			3	SS	85							
356.0 355.0		Grey Soft weathered Sound SHALE with intermittent bands of hard LIMESTONE BEDROCK		4	SS	27/3							
				5	RC	90.1%							
350.0	15			6	RC	90%							
345.0	20	END OF BOREHOLE											

W.L. 363.7 Ft.  
JULY 4. 1966.

Gr. 10% ; Sa. 15 %  
Sl. 65% ; Cl. 10 %



**FOUNDATION DESIGN REPORT  
RAMP FGGE E – SHERWAY GARDENS ROAD RETAINING STRUCTURE  
AT NORTH ABUTMENT OF EXISTING BRIDGE NO. 4  
TORONTO, ONTARIO  
W.P. 176-00-01**

**Prepared For:**

**DS-LEA ASSOCIATES LIMITED  
251 Consumers Road, Suite 1200  
North York, Ontario  
M2J 4R3**

**Prepared by:**

**SHAHEEN & PEAKER LIMITED**

**Project: SP3232K  
June 30, 2000**

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

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**FOUNDATION DESIGN REPORT  
RAMP FGGE E –SHERWAY GARDENS ROAD RETAINING STRUCTURE  
AT NORTH ABUTMENT OF EXISTING BRIDGE NO. 4  
TORONTO, ONTARIO  
W.P. 176-00-01**

**5. DISCUSSION AND RECOMMENDATIONS**

The new FGGE E-Sherway Gardens Road will pass under the existing Bridge No. 4. To accommodate a sufficient vertical clearance under the bridge between the new roadway and the existing bridge structure, the existing grade under the bridge will be lowered. Where the new roadway approaches the north abutment of the existing bridge, permanent shoring will be required in order to effect the proposed excavation. As the new roadway approaches the existing bridge at a skew, the excavation comes closest to the north abutment at its east corner.

We understand that the existing north abutment is supported on steel piles driven into the bedrock (to about Elevation  $108.1 \pm$  m) as shown in Appendix F. The elevation of the top of the pile supported footing (i.e. the pile cap) is  $115.1 \pm$  m and the adjacent finished road elevation will be about 111.5 to 111.0 m that is about 4 m below.

Borehole B3-1, which was drilled for this investigation at about 14 m north east of the east corner of the existing bridge abutment, showed the presence of fill to about Elevation 111 m followed by very stiff to hard clayey silt till and till/shale complex to the surface of shale bedrock at about Elevation 109.5 m. The surface of the bedrock in Borehole 97 drilled by MTO in the 1960's to the south of this location is indicated at Elevation 108.5 m and in Borehole 4, located further south it is at 107.4 m. From this it appears that the surface of the bedrock in the general area dips mildly from north to south.

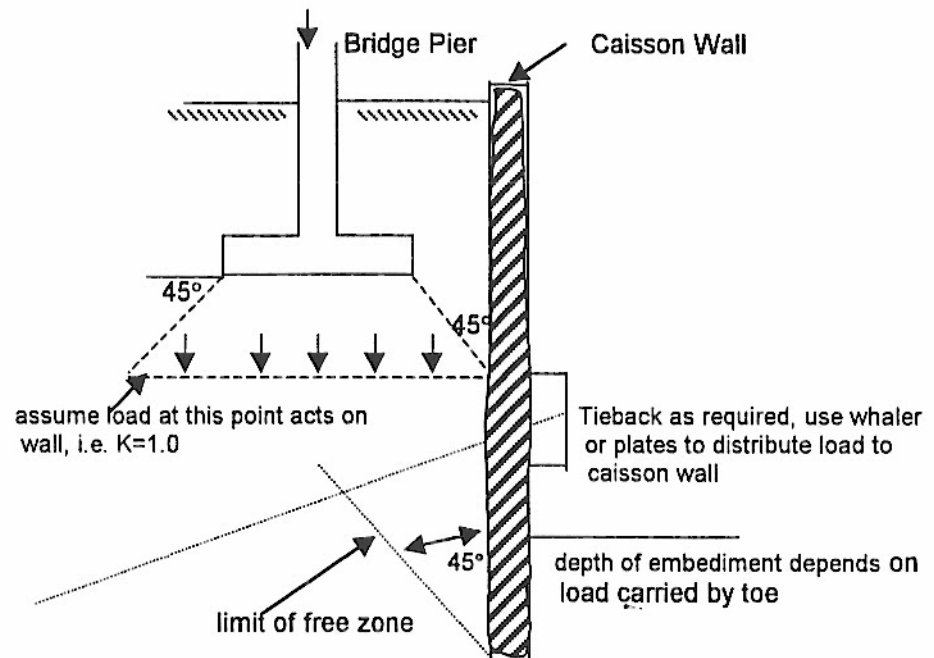
As presently proposed, the permanent retaining structure will be an approximately 32 m long contiguous caisson wall which will incorporate permanent tieback (rock anchors) and precast concrete facing. At its closest location, the back

of the wall (i.e. caisson) will come to about 7 m from the east edge of the existing bridge foundation. Because of their battered nature, the pile tips may be as close as 2 m to the caisson wall (at about Elevation 108 m – see sketch in Appendix F). Permanent tieback (rock anchors) will be provided in order to minimize the yielding.

In our opinion, with the prevailing subsurface conditions, the proposed scheme is a suitable solution. The following values can be used for the design of the caissons.

$K_0$	=	0.45 all soils, do not use $K_a$
$K$	=	1.0 when retaining horizontal loads from adjacent footings, piles, etc.
$K$	=	0.15 in the shale bedrock
$\gamma$	=	21 kN/m <sup>3</sup> all soils above watertable
$\gamma$	=	23 kN/m <sup>3</sup> shale bedrock
$Q$	=	surcharge loads as required

Note that  $K$  values assume a horizontal backfill. If the backfill is inclined  $K_0$  will increase. Where footings or pile tips are within a 45° line drawn from the edge of the footing or pile tip and this line intersects the wall, the effect of this load must be incorporated into the wall design. In this case,  $K=1$  is applicable to this portion of the load on the wall.



**SKETCH OF CAISSON/RETAINING WALL**

The following unfactored values can be used for the calculation of passive earth pressure in front of the caisson wall.

$K_p = 4.0$  for clayey silt till and till/shale complex

$K_p = 4.5$  for shale bedrock

The resistance to lateral movement is provided by the passive resistance developed on the face of a single 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.

Based on results of Borehole B3-1, the unfactored  $c_u$  (i.e. undrained shear strength) for the material can be taken as 300 kPa between Elevation 111 and 110 m; 500 kPa between 110 and 108 m; 1200 kPa from 108 to 106 m 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:

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

Where

$k_s$  = coefficient of subgrade reaction

$c_u$  = undrained shear strength of the material

$d$  = pile diameter

The expression and the values given are applicable to single caisson units only.

As relatively low vertical forces can be expected, high rock resistances are unlikely be necessary but based on Borehole B3-1 results, a value of up to 2500 kPa can be used for U.L.S. below Elevation 106.0 m, for the design of the caissons; S.L.S will not govern.

Some general comments for the design and construction are as follows:

- (a) Use double corrosion protection on the tiebacks;
- (b) Test all tiebacks to a proof load of 133% of design load, but load test one tieback to 200% of design;
- (c) Monitor all tieback installations;
- (d) Do not load the tieback so that movement during stressing (into the soil or rock) exceeds 4 mm;
- (e) Keep the free (unloaded) zone in the area defined by a 45° line drawn up from the toe of the pile, but the minimum free (unbonded) length should be 6 m;

For the calculation of anchor resistance for tie back design, the bond resistance at U.L.S. can be taken as 500 kPa in the bedrock and S.L.S. will not govern. The minimum bond length should be 5 m.

- (f) Use 30 MPa concrete for the toe and 4 MPa concrete above;
- (g) The walers will be welded to the H piles in the caissons and the face of all caissons will be cut back to the face of the H piles.
- (h) If any water is expected, use a drainage blanket between the caisson wall and the precast facing.
- (i) The only significant vertical load may be from the tiebacks, hence, the caisson base is not normally inspected and caissons smaller than 760 mm diameter can be considered.
- (j) Rock deterioration due to weathering and its consequences should be considered, if and where applicable.

The limited headroom available when working underneath the existing bridge deck will require special consideration. In addition, hard shale, siltstone, sandstone and particularly limestone seams and layers, which are known to be present in the shale bedrock, may be difficult to penetrate during the caisson excavations, especially if small equipment is to be used for this purpose due to limited headroom.

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, including the shale bedrock.

As an alternative to the presently proposed system of a tied back contiguous caisson wall, the use of conventional 'soldier pile and lagging' (with tiebacks) type of shoring can also be considered but, based on our experience, in general a rigid, non-yielding support structure is needed when an excavation extends close to existing significant structures, such as existing buildings and bridges. In such cases, the permanent retaining structure must be designed to resist the horizontal and vertical pressures applied by these foundations. Traditionally, in the Toronto area, contiguous caisson type rigid structures (as proposed in this case) have been utilized when excavating immediately adjacent to existing buildings, etc., in order to minimize lateral yield and surface settlements. It is, therefore, our opinion that conventional 'soldier pile and lagging' is not a suitable alternative.

Other types of support such as 'nailing' have been considered but these are not believed to be sufficiently rigid for the intended use.

A structural diaphragm (slurry) wall can be considered. Because of limited space available, only a narrow wall can likely be constructed and therefore, in this case, anchoring (i.e. tie backs) will be necessary. In addition, from a stability point of view, opening a continuous trench in front of slopes and existing foundations may endanger the stability and, therefore, it must be constructed with this aspect in mind (i.e. constructed in short sections). In this respect, a contiguous caisson wall is a safer approach. In addition, the subsurface conditions are not well suited for the use of a slurry wall (i.e. bedrock at shallow depth) as the availability of equipment capable of operating in a restricted area, especially due to low overhead, may present problems. Another disadvantage of this type of support is that its future performance is highly dependent on workmanship.

## 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 G, are an integral part of this report.

### Shaheen & Peaker Limited



Zuhtu Ozden, P.Eng.



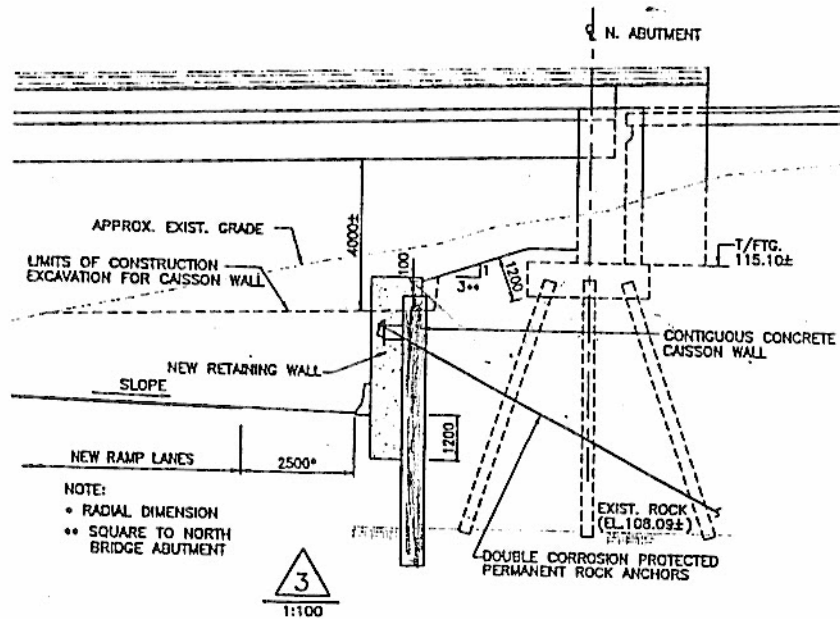
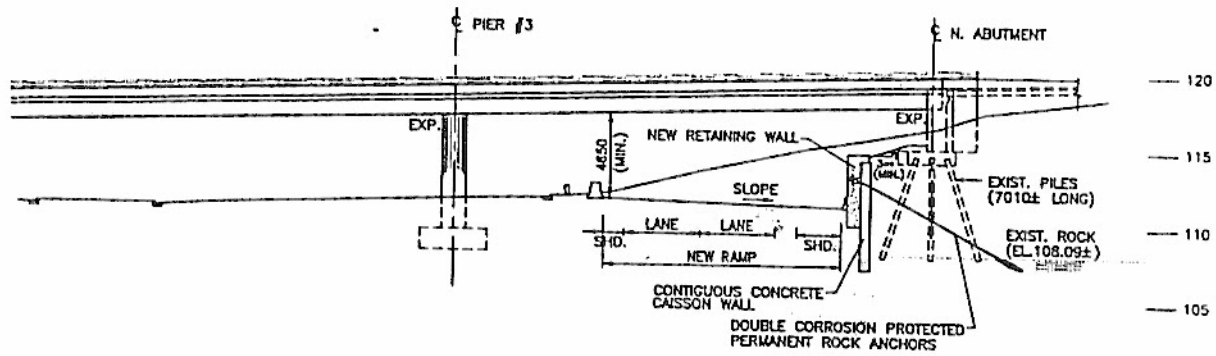
K. R. Peaker, Ph.D., P. Eng.

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## APPENDIX F

### North Abutment of Existing Bridge No. 4 and Proposed Retaining Works



DRAWING NOT TO BE SCALED  
100 mm ON ORIGINAL DRAWING

## NORTH ABUTMENT OF EXISTING BRIDGE NO. 4 AND PROPOSED RETAINING WORKS

# APPENDIX G

## Limitations of Report

## LIMITATIONS OF REPORT

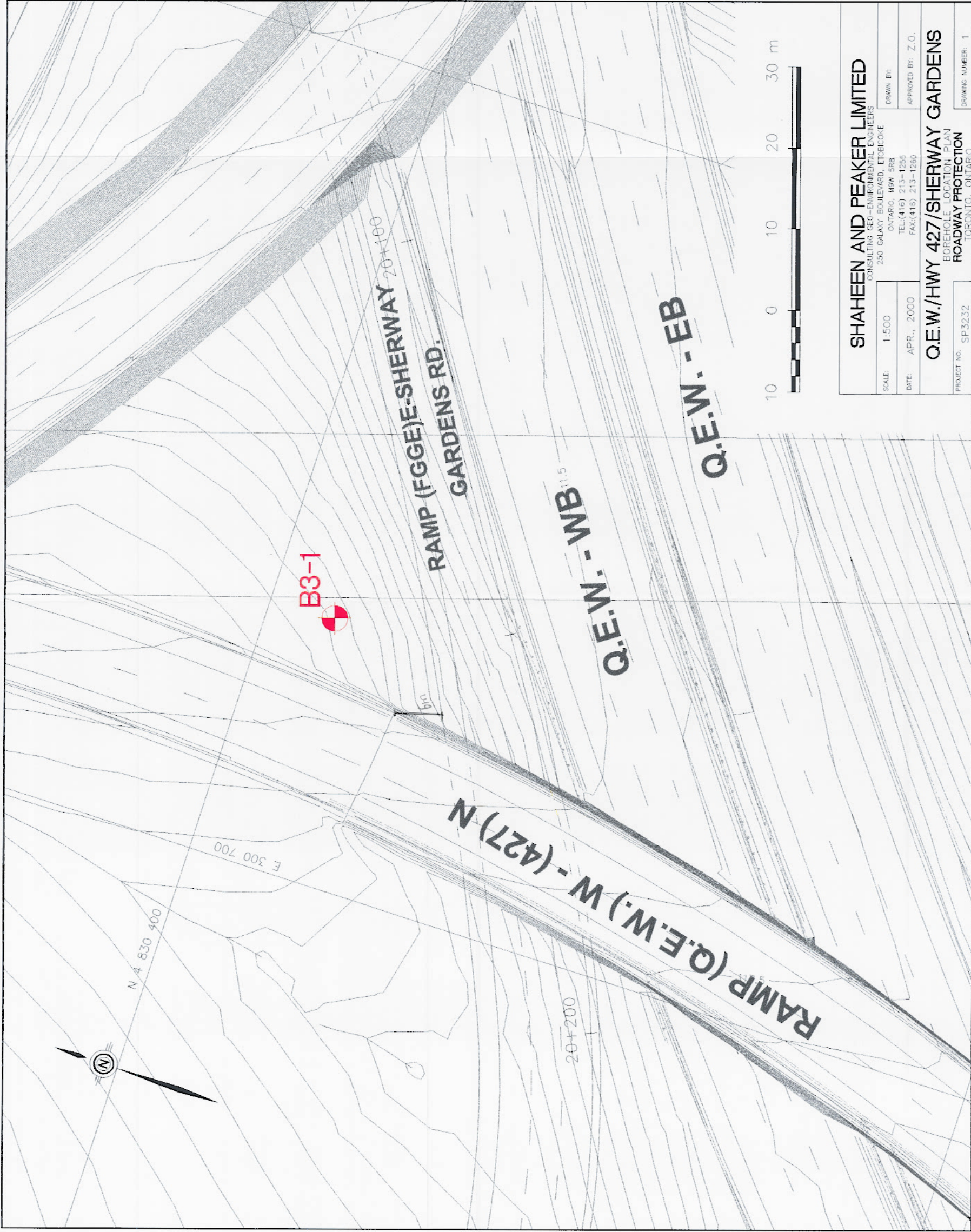
The conclusions and recommendations given in this report are based on information determined at the testhole locations. The information contained herein in no way reflects on the environment aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the 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.





**SHAHEEN AND PEAKER LIMITED**

CONSULTING GEO-ENVIRONMENTAL ENGINEERS  
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ONTARIO, M9W 5R8  
SCALE: 1:500  
DATE: APR., 2000  
DRAWN BY:  
APPROVED BY: Z.O.

**Q.E.W./HWY 427/SHERWAY GARDENS**

BOREHOLE LOCATION PLAN  
ROADWAY PROTECTION

DRAWING NUMBER: 1

PROJECT NO: SP3232

TORONTO, ONTARIO