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STR. SITE No.

HWY. No. QEW

LOCATION PROPOSED BRIDGE WIDENING
FGGE OVER THE EAST MALL

No. of PAGES -

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OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

**FOUNDATION INVESTIGATION REPORT
PROPOSED BRIDGE WIDENING FGGE
OVER THE EAST MALL
TORONTO, ONTARIO
W.P. 175-00-01**

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**FOUNDATION INVESTIGATION REPORT
PROPOSED BRIDGE STRUCTURE WIDENING FGGE
OVER THE EAST MALL
TORONTO, ONTARIO
W.P. 175-00-01**

1. INTRODUCTION

As part of the Gardiner Expressway/Q.E.W./Hwy. 427/Brown's Line Interchange modifications, the existing Gardiner Expressway (FGGE) bridge over the East Mall will be widened on the south end. Shaheen & Peaker Limited (S&P) was retained by DS-Lea Associates Limited, Consulting Engineers, to carry out a foundation investigation for the proposed bridge widening.

The site is located at FGGE and the East Mall interchange immediately east of Highway 427/Brown's Line in southwest Toronto.

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

The findings of the investigation are presented in this report.

2. PHYSIOGRAPHY

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

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

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

Bedrock consisting of shale with interbedded limestone and sandstone underlies the Halton Till. Available information indicates that the surface of the bedrock of the site can be expected at about Elevation 108 m.

3. INVESTIGATION PROCEDURES

The fieldwork for the project was performed on February 17 and March 21, 2000 and consisted of drilling and sampling two boreholes (Boreholes B5-1 and B5-2) at the locations shown on Drawing No. 1. The depth of the boreholes ranged from 6.2 to 7.7 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.

The boreholes were extended until refusal to augering in the bedrock at depths of 6.2 m (Borehole B5-1) and 7.7 m (Borehole B5-2) probably on limestone layers in the shale bedrock.

The soil and rock samples recovered from the boreholes were shipped to our laboratory in Toronto for further examination and classification. A laboratory-testing programme, consisting of natural moisture content, bulk unit weight and

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

The logs of four of the boreholes drilled for the Ministry in 1966 for the design of the existing bridge structure are given in Appendix D.

Groundwater conditions in the open boreholes were observed during the drilling and at completion of each borehole. In addition, 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. Borehole B5-2 had to be relocated from its staked location and in this case an allowance was made by our field staff for location and elevation differences.

4. SUBSURFACE CONDITIONS

The subsurface conditions were explored at two borehole locations (Boreholes B5-1 and B5-2). The locations of the boreholes are shown on the Borehole Location Drawing No. 1 and are also indicated on the individual borehole log sheets.

Both boreholes were drilled from the top of the existing earth embankments where ground elevations were 113.4 and 113.3 m. They contacted 400 and 125 mm of topsoil underlain by fill to depths of 2.9 and 1.8 m or to Elevations 110.5 and 111.5 m. Underlying the topsoil and the fill, both boreholes encountered a 0.7 to 1.3 m thick fine sand layer, underlain by clayey silt till and till/shale complex to the surface of bedrock at about 5.8 m or Elevations 107.6 and 107.5 m.

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

4.1 TOPSOIL

The boreholes contacted a 125 to 400 mm thick topsoil layer.

4.2 FILL

Material identified as embankment fill was contacted in both boreholes to depths/elevations of 2.9/110.5 m (Borehole B5-1) and 1.8/111.5 m (Borehole B5-2). In B5-1, the fill generally consisted of silt, sand and sandy silt to silty sand with some gravel to a depth of 2.1 m (Elevation 111.3 m). Topsoil inclusions were noted in the lower portion of this fill. This material is considered to be a basically fine grained, granular soil and is underlain by a cohesive fill material (clayey silt and silty clay with shale pieces and traces of gravel and topsoil) to 2.9 m or to Elevation 110.5 m.

In Borehole B5-2, fill was found to extend to a depth of 1.8 m (Elevation 111.5 m). The fill at this location consists of a heterogeneous mixture of fine sand with silty clay and some gravel and shale fragments. Topsoil inclusions were also noted in the fill.

From recorded N-values of between 4 and 20 blows/0.3 m the fill encountered in the boreholes does not appear to have received a systematic compaction.

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

4.3 FINE SAND

Underlying the fill both boreholes contacted a layer of fine sand at depths/elevations of 2.9/110.5 m (Borehole B5-1) and 1.8/111.5 m (Borehole B5-2), respectively. The thickness of this unit ranged from 0.7 to 1.3 m. The grain size distribution of a sample from the deposit is given in Appendix B. The results indicate 88% sand and 12% soil fines. The deposit is brown and was in a wet condition in Borehole B5-1 and moist in Borehole B5-2.

Based on N-values of 12 and 25 blows/0.3 m, the deposit is considered compact. The sand encountered at the east abutment location may have been previously disturbed.

4.4 CLAYEY SILT TILL

Immediately underlying the fine sand, clayey silt till was contacted in Boreholes B5-1 and B5-2 at depths of 3.6 m (Elevation 109.8) and 3.1 m (Elevation 110.2 m), respectively. The till consists of a heterogeneous mixture of clayey silt with some sand and gravel sizes. It contains shale and limestone fragments and the frequency of these increases with increasing depth. The grain size distribution of typical samples from the deposit is given on the grain-size distribution curves presented in Appendix B.

The presence of cobbles and boulders can always be expected in the glacial till deposits, due to their mode of deposition.

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

Liquid Limit = 21-25%

Plastic Limit = 17-21%

Plasticity Index = 4%

These values are representative 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.

The till is a cohesive material and based on recorded N-values which range from 27 to in excess of 57 blows/0.3 m, its consistency is described as very stiff to hard but generally hard.

4.5 TILL/SHALE COMPLEX

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

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

The till/shale complex was contacted at depths of 5.3 m and 4.2 m in Boreholes B5-1 and B5-2 or at Elevations 108.1 and 109.1 m, respectively and extended to the surface of the inferred bedrock at a depth of 5.8 m or Elevations 107.6 and 107.5 m.

The till/shale is a cohesive material. Based on N-values in excess of 57 blows/0.3 m together with the observed resistance to augering while drilling, its consistency is described as hard.

4.6 SHALE BEDROCK

Shale bedrock was encountered at the following approximate elevations at the borehole locations:

Estimated Bedrock Depth

Borehole No.	Bedrock Depth (m)	Elevation (m)
B5-1	5.8	107.6
B5-2	5.8	107.5

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

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

Formation. In these features, the rock is heavily fractured and sheared, and contains layers of shale rubble and clay.

In Boreholes B5-1 and B5-2, the bedrock was penetrated by augering to depth/elevations of 6.2 m/107.2 and 7.7 m/105.6, or by 0.4 and 1.9 m, respectively, until refusal was encountered, probably on the surface of a hard layer (e.g. limestone layer). From this, it can be inferred that the bedrock at these two locations does not contain hard layers above these elevations.

4.7 GROUNDWATER CONDITIONS

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

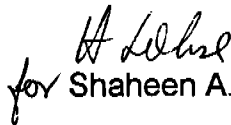
To enable us to monitor the groundwater level over a prolonged period of time without interference from surface water, a piezometer was installed in each of the two boreholes. The water levels in the piezometers were monitored over a period of time when the water level was recorded at depths of 3.1 m and 6.2 m or at Elevations 110.3 m and 107.1 m in Boreholes B5-1 and B5-2, respectively.

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



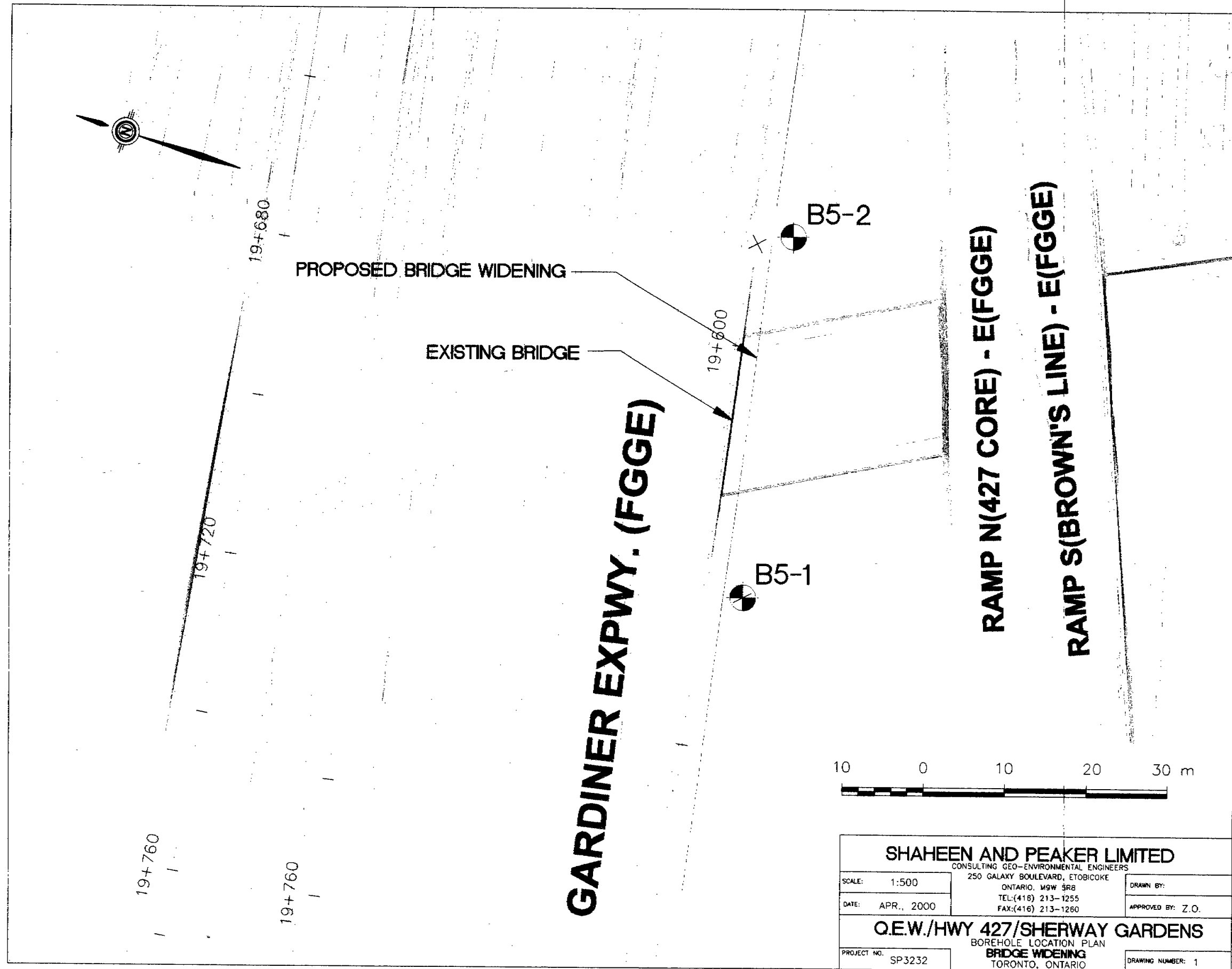
Zuhtu Ozden, P.Eng.



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



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APPENDIX A

Borehole Log Sheets

RECORD OF BOREHOLE No B5-1

1 OF 1

METRIC

W.P. 175-00-01 LOCATION FGGE over the East Mall; 4 830 486.3 N; 301 143.7 E ORIGINATED BY E.P.
 DIST HWY 427 & QEW BOREHOLE TYPE Solid Stem Augers COMPILED BY G.T.
 DATUM Geodetic DATE 17.02.00 CHECKED BY Z.O.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					
113.4	Ground surface							20 40 60 80 100					
0.0	400 mm Topsoil		1	SS	7		113						
113.0													
0.4	FILL: Silt & Sand , some gravel, brown		2	SS	7								
112.1							112						
1.3	FILL : Silty Sand and Sandy Silt, some gravel and topsoil, brown, black		3	SS	19								
111.3													
2.1	FILL: Clayey Silt & Silty Clay with shale pieces, trace gravel & topsoil		4	SS	20		111						
110.5													
2.9	FINE SAND brown, compact, wet		5	SS	25		110						
109.8													
3.6	Heterogeneous mixture of Clayey silt, some sand and gravel (CLAYEY SILT TILL) frequent shale fragments		6	SS	45		109						
108.1													
5.3	TILL/SHALE: complex, grey, hard		8	SS	88/28		108						
107.6													
5.8	SHALE BEDROCK, grey		9	SS	50/5								
107.2													
6.2	End of borehole Refusal to further augering @ 6.2 m possibly on a limestone layer Water level @ 4.4 m upon completion (not stabilized) Piezometer installed to 5.7 m Water level in piezometer Feb. 20 = 3.3 m Feb. 29 = 3.1 m March. 2 = 3.1 m												N=50/13 denotes 50 blows for 13 cm penetration

RECORD OF BOREHOLE No B5-2

1 OF 1

METRIC

W.P. 175-00-01 LOCATION FGGE over the East Mall; 4 830 493.0 N; 301 184.6 E ORIGINATED BY J.W.
DIST HWY 427 & QEW BOREHOLE TYPE Solid Stem Augers COMPILED BY G.T.
DATUM Geodetic DATE 21.03.00 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		
113.3	Ground surface													
113.2	125 mm Topsoil													
0.1	FILL: Fine Sand with Silty Clay, some gravel & shale fragments & topsoil, brown, grey, black		1	SS	4		113							
			2	SS	8		112							
111.5			3	SS	32									
1.8	FINE SAND brown, compact, moist		4	SS	12		111							0 88 8 4
110.2														
3.1	CLAYEY SILT TILL grey, very stiff to hard		5	SS	27		110							4 26 47 23
109.1			6	SS	53									19 21 47 13
4.2	TILL/SHALE Complex grey, hard						109							
			7	SS	57/15									
107.5			8	SS	59/23		108							
5.8	SHALE BEDROCK grey													
			9	SS	50/3		107							
			10	SS	50/3									
105.6			11	SS	50/3		106							
7.7	End of borehole Refusal to further augering @ 7.7m possibly on a limestone layer Piezometer installed Water level in piezometer @ 6.2 m on March 28/2000													

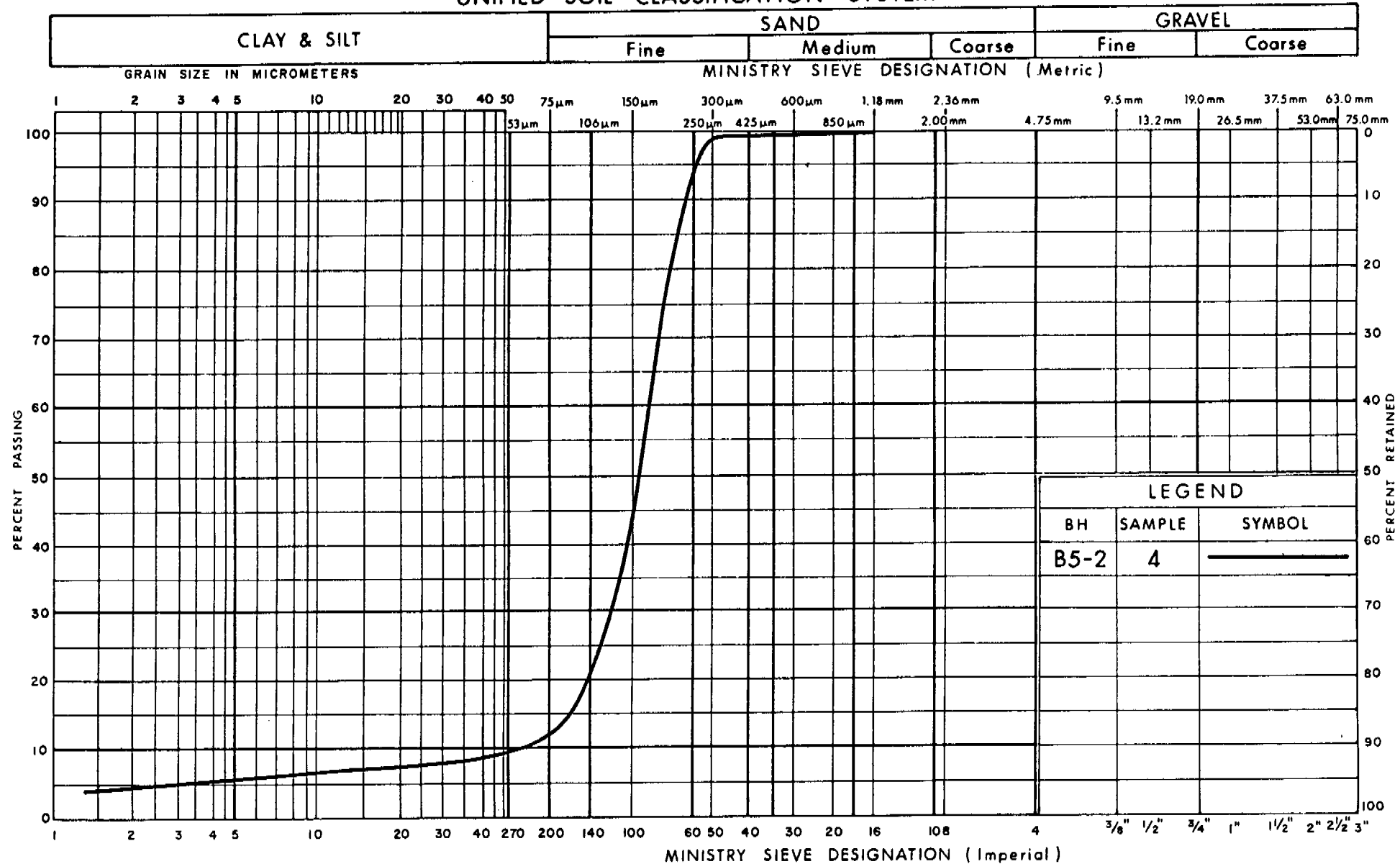
+ 3, x 3: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

APPENDIX B

Laboratory Test Results

UNIFIED SOIL CLASSIFICATION SYSTEM



Ontario

Ministry of
Transportation

GRAIN SIZE DISTRIBUTION

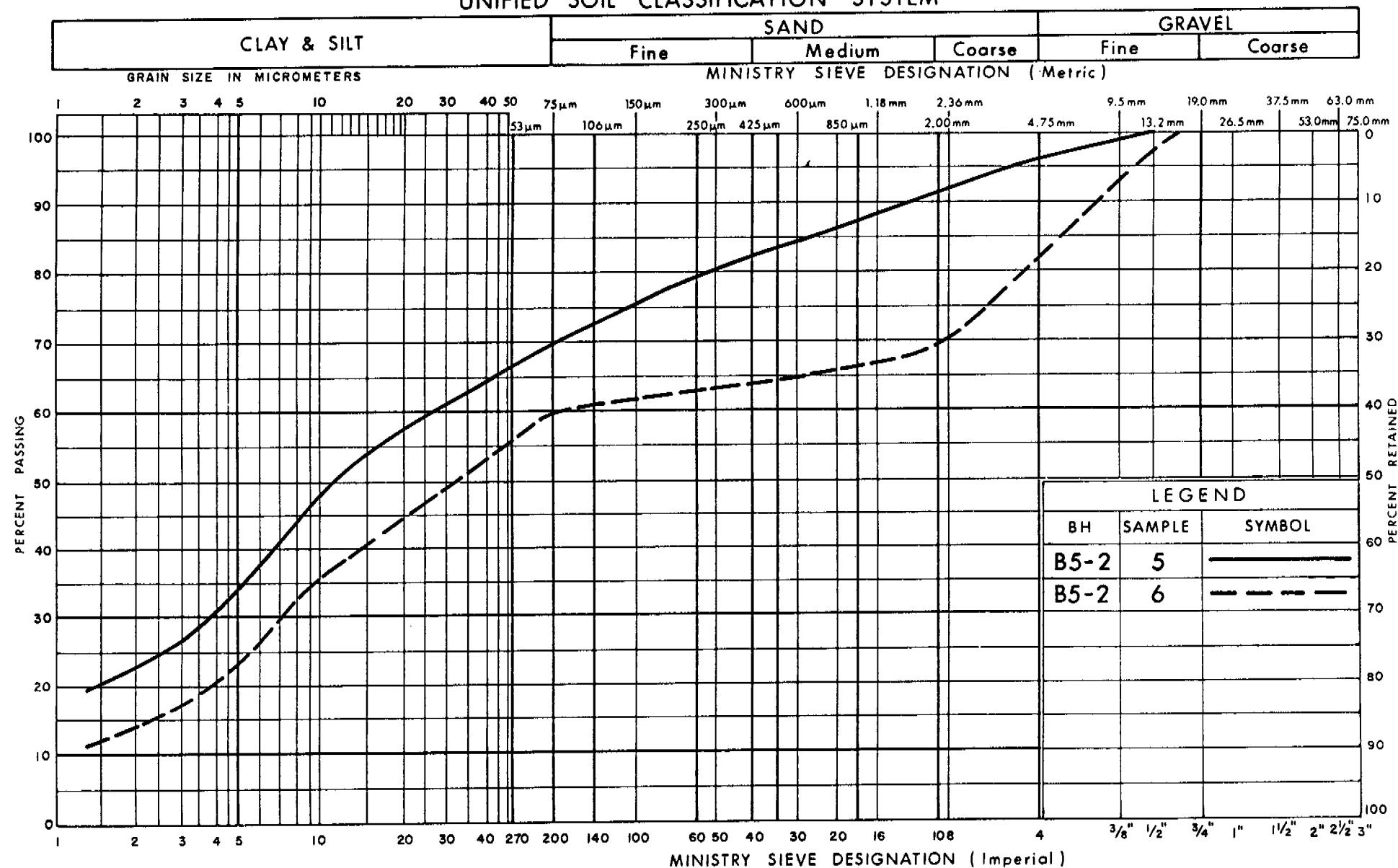
FINE SAND

FIG No 1

W P 175-00-01

SP 3232E

UNIFIED SOIL CLASSIFICATION SYSTEM



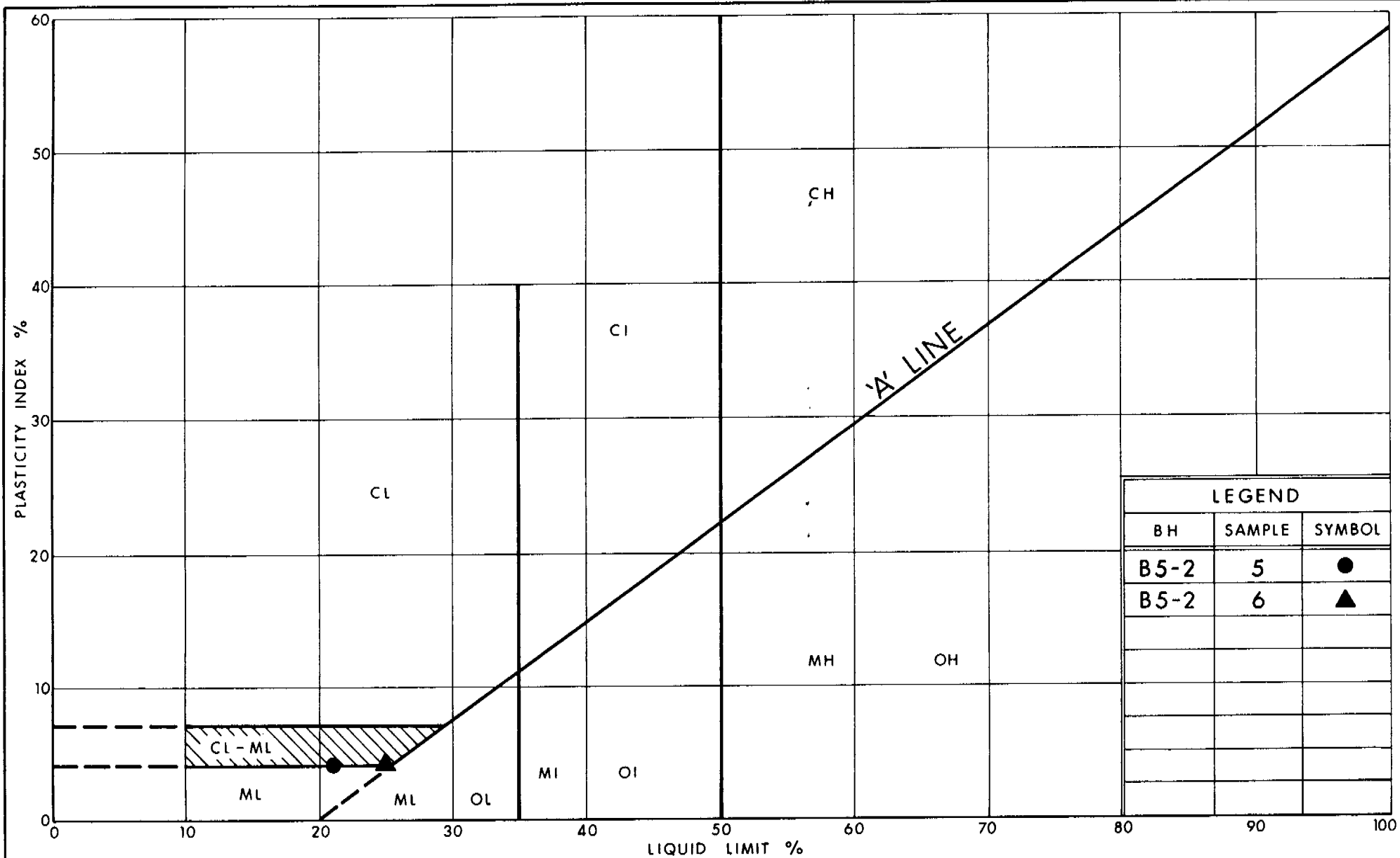
Ministry of
Transportation

GRAIN SIZE DISTRIBUTION CLAYEY SILT TILL

FIG No 2

W P 175-00-01

SP 3232E



Ministry of
Transportation
Ontario

PLASTICITY CHART CLAYEY SILT TILL

FIG No 3

W P 175-00-01

SP 3232E

APPENDIX C

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 31mm 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 (31mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

c_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	> 200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS / 0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	> 50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

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

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

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

JOINTING AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

MECHANICAL PROPERTIES OF SOIL

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

STRESS AND STRAIN

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

PHYSICAL PROPERTIES OF SOIL

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

APPENDIX D

Logs of Boreholes Drilled in 1966

OUR REFERENCE NO. - 6 - 9 - 41

CLIENT: D. H. O.
PROJECT: Q.E.W. & HWY. N^o 27 INTERCHANGE - BRIDGE N^o 17

DIAMETER OF BOREHOLE: 2 $\frac{3}{8}$ "

DATE: OCT. 14, 1966

LOCATION: 178,754 N., 210,495 E.

DATUM ELEVATION: G. S. C.

W.P. 36-65-1

[illegible]

GEOTECHNICAL DATA SHEET FOR BOREHOLE . 175 .

OUR REFERENCE NO. 6-9-41

CLIENT: D. H. O.

PROJECT: Q.E.W. & HWY. N° 27 INTERCHANGE - BRIDGE N° 17

LOCATION: 178,690 N., 210,538 E.

DATUM ELEVATION: G.S.C.

METHOD OF BORING: WASHBORING

DIAMETER OF BOREHOLE: 2 3/8"

DATE: OCT. 7, 1966

W.P. 36-65-1

ENCLOSURE NO.

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE blows per foot					CONSISTENCY water content %		REMARKS
				NUMBER	TYPE	N° Advancement of Sampler	2,0	4,0	6,0	8,0	10,0	PL	W	
367-1	0	GROUND SURFACE												
		SANDY TOPSOIL												
365-0	2-0	Compact, Brown SAND and SILT (FILL)		1	C.S.									
361-6	5-8	BLACK TOPSOIL		2	S.S.	11								
361-1	6-0	FINE SAND, some silt												
360-0	7-1	Dense, Grey SANDY SILT with some clay and gravel. boulder		3	S.S.	56								
	10			4	S.S.	200/2"								
358-6	11-8			5	R.C.									
355-0				6	S.S.	85/6"								
	15	Grey SHALE		7	S.S.	63/6"								
				8	S.S.	116/7"								
350-0		weathered sound		9	R.C.	100%								
	20	BEDROCK		10	R.C.	87%								
345-0														
344-1	23-0	END OF BOREHOLE												

W.L. El. 362.8'
Oct. 12, 1966

100/T

GEOTECHNICAL DATA SHEET FOR BOREHOLE 178.

OUR REFERENCE NO. 6 - 9 - 41

CLIENT: D. H. O.
PROJECT: Q.E.W. & HWY. N° 27 INTERCHANGE - BRIDGE N° 17
LOCATION: 178, 672 N., 210, 468 E.
DATUM ELEVATION: G. S. C.

METHOD OF BORING: WASH BORING
DIAMETER OF BOREHOLE: 2 3/8"
DATE: OCT. 6, 1966
W.P. 36 - 65 - 1

ENCLOSURE NO. .

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE blows per foot				CONSISTENCY water content %				REMARKS
				NUMBER	TYPE	N- 8 Advancement of Sampler	2.0	4.0	6.0	8.0	10.0	PL	W	LI	
							SHEAR STRENGTH				lbs/sq ft				

W.L. El. 362.7
Oct. 12, 1966

Gr. 1% ; Sa. 84%
CL - CL. 15%

Gr. 14% ; Sa. 42%
Si. 32% ; CL. 12%

Gr. 6% ; Sa. 39%
Si. 37% ; CL. 18%

GEOTECHNICAL DATA SHEET FOR BOREHOLE .179.

OUR REFERENCE NO. 6 - 9 - 41

CLIENT: D.H.O.
PROJECT: Q.E.W. & HWY. N° 27 INTERCHANGE — BRIDGE N° 17
LOCATION: 178,755 N., 210,410 E.
DATUM ELEVATION: G.S.C.

METHOD OF BORING: WASHBORING
DIAMETER OF BOREHOLE: 2 3/8
DATE: OCT. 12, 1966
W.P. 36 - 65 - 1

ENCLOSURE NO.

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE blows per foot					CONSISTENCY water content %					REMARKS
				NUMBER	TYPE	N- or Advancement of Sampler	20	40	60	80	100	PL	W	LI			
362.7	0	GROUND SURFACE															
361.2	1.5	Black, ORGANIC SANDY SILT															
360.0		Dense, Brown FINE SAND with some silt.		1	C.S.												
358.8	3.9	Very Dense Grey SAND and SILT with some clay gravel and shale fragments.		2	S.S.	82											
355.0				3	S.S.	77											
352.7	10	Grey SHALE		4	S.S.	88											
350.0		weathered sound		5	S.S.	100%											
349.5				6	S.S.	100%											
	15	BEDROCK		7	R.C.	88%											
				8	R.C.	83%											
345.0	18.0	END OF BOREHOLE															
344.5	20																

100 F/10'

Gr. 16% ; Sa. 43%
Si. 31% ; Cl. 10%

C.I. El. 360.7'
Oct. 19, 1966

**FOUNDATION DESIGN REPORT
PROPOSED WIDENING OF FGGE BRIDGE
OVER EAST MALL
TORONTO, ONTARIO
W.P. 175-00-01**

Prepared For:

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M2J 4R3**

Prepared by:

SHAHEEN & PEAKER LIMITED

**Project: SP3232E
June 8, 2000**

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APPENDICES

Limitations of Report

Appendix E

**FOUNDATION DESIGN REPORT
PROPOSED WIDENING OF FGGE BRIDGE
OVER THE EAST MALL
TORONTO, ONTARIO
W.P. 175-00-01**

5. DISCUSSION AND RECOMMENDATIONS

The existing Gardiner Expressway (FGGE) bridge over the East Mall will be widened on the south side by about 3.5 m to accommodate a new lane, as part of the FGGE/QEW/Hwy 427/Brown's Line Interchange improvements project. The proposed widening will match the existing bridge and will be an approximately 20 m span, reinforced concrete rigid frame structure. The bridge elevation is $113\pm$ m while the pavement elevation of the East Mall beneath is approximately 107.5 m.

Boreholes B5-1 and B5-2 drilled near the west and east abutment locations, respectively contacted, below some pavement fill and moist to wet sand deposits, very stiff to hard clayey silt till at about Elevation 110 m. The till contains some shale and limestone fragments, the frequency of which increases with increasing depth and the material attains a till/shale character, which is a transition zone into the underlying bedrock. The surface of the bedrock was encountered at about $107.5\pm$ m. In Boreholes 174, 175, 178 and 179, drilled (by others) in 1966 for the existing bridge (at or near the south end of the existing structure), the surface of the bedrock was contacted at Elevations ranging between 107.5 and 108.4 m (Appendix D).

5.1 FOUNDATIONS

The boreholes show that the foundations for the widened bridge section can be supported on shallow footings extended below the fill and the upper less competent zones of the natural stratum into the competent overburden or the underlying bedrock.

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
B5-1	113.4	4.4	109.0	800	400	clayey silt till shale bedrock shale bedrock
West Abutment		5.9	107.5	1000	600	
		6.2	107.2	1000	1000**	
		or below	or below			
B5-2	113.3	4.3	109.0	800	400	till/shale complex shale bedrock shale bedrock
East Abutment		5.9	107.4	1000	600	
		6.8	106.5	1000	1000**	
		or below	or below			

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

** Total and differential settlements will not exceed 15 mm for foundations placed on properly prepared bedrock surface, under the direction of geotechnical engineer.

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

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

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

We understand that the existing bridge is supported on footing foundations resting on the shale bedrock at about Elevation 106.4 m. The foundation for the new section (i.e. widening) of the bridge will therefore probably be founded at the same elevation to avoid excessive stress superimposition and to minimize disturbance (e.g. undermining) of the existing foundations. The results of the boreholes drilled for this investigation and of the boreholes previously drilled in 1966 by others for the existing bridge structure indicate that at this elevation the foundation will generally be extended about 1.2 m below the surface of the bedrock. Reference to Table 1 shows that at this elevation a rock resistance value of up to 1000 kPa can be used for both the U.L.S. and S.L.S. It is our opinion that with this

value the settlements should not exceed 15 mm for foundations placed under the direction of the geotechnical engineer on properly prepared bedrock surface.

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.

For the evaluation of the sliding resistance of the foundation (O.H.B.D.C. 6-8.4.3), the ultimate angle of friction between the underside of the foundation and the clean shale bedrock surface (or between concrete surfaces) can be taken as 25 degrees. Horizontal shear resistance can be supplemented, if required, by penetrating in the bedrock (i.e. keying-in and utilizing passive rock resistance) and/or shear in grouted dowels and/or rock anchors. We recommend that the minimum dowel length below the underside of the footing should be 2 m.

If there are net uplift forces which are to be resisted by rock anchors, or for increasing the sliding resistance, the factored rock/grout bond capacity at U.L.S. in the bedrock can be taken as 500 kPa and S.L.S. will not govern. The value quoted at U.L.S. incorporates a safety factor of 2 against an ultimate failure condition. 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 pullout 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.

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 drainpipes 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.

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.

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

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

Only minor grade adjustments are expected for the construction of the approach embankments. For this purpose, all the surficial topsoil and otherwise unsuitable soils should be removed. For the widening of the existing embankment, proper benching of the existing side slopes 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 the 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).

A moist to wet sand layer was contacted below about Elevations 110.5 and 111.5 m in Boreholes B5-1 and B5-2, respectively. Since the road elevation for the East Mall is lower than this elevation (i.e. Elevation $107.5 \pm$ m, that is below the original ground level when the existing interchange was built in 1960's), the original water table is likely to have been depressed below the original water levels in the vicinity of the existing bridge structure and the wet condition encountered in the sand layer is likely to represent a perched condition. Nonetheless, depending on the season of the year and the weather conditions during the time of the construction, wet and unstable soil conditions as well as water seepage can be encountered when this deposit is intercepted in the excavation.

- Should this happen, dewatering would be required to stabilize the soil. In addition, the side slopes will have to be stabilized by gravel sheeting during the construction. Regardless of the conditions encountered during the construction, it is recommended that where the sand is intercepted, properly filtered gravel sheeting be applied for permanent slope stability both in cuts and fills.

5.4 CONSTRUCTION COMMENTS

As mentioned in the preceding section, the water level in Borehole B5-1 was recorded in the sand layer immediately below the fill at about Elevation 111 m. The strata underlying the sand deposit are, however, relatively impervious and therefore, during the construction major dewatering problems are not anticipated once the sand layer is dewatered. In the clayey deposits underlying the sand and in the shale bedrock, groundwater seepage should be moderate and can be handled by means of gravity drainage and pumping from sumps. Occasional coarser, fissured or fractured zones may create local problems but in our opinion, these too can be handled by providing local drainage and 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 both in the overburden and in the shale bedrock (the shale is prone to rapid weathering and disintegration). At the time of construction, all footing

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

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 (e.g. the wet sand layer underlying the fill). 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 will be required (e.g. the sand layer encountered in the boreholes).

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

All excavations and shoring should be carried out in accordance with the Safety Regulations of the Province. If shoring is constructed using soldier piles and conventional lagging, the soldier piles will have to be socketed into the bedrock, which was contacted at most borehole locations below about Elevation 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 below the general excavation level, the following soil resistance values can be used:

U.L.S. = 400 kPa

S.L.S. = 200 kPa

These values can be increased to 800 kPa and 400 kPa for relatively sound bedrock. For calculating anchor resistance for tieback 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.

We recommend that by means of good construction techniques, the undermining of the existing bridge foundations should be avoided; otherwise detrimental settlements of the existing structure could occur after the construction.

As mentioned before, for foundations placed at or below about Elevation 106.4 m, settlements of up to 15 mm could occur. While some of these settlements will take place during the construction, the remainder will translate into differential settlements between the existing (old) and the proposed (new) widened section. If this settlement is objectionable, then it is recommended that the existing and the new bridge sections be separated by construction joints to allow independent vertical movements of the old and the new sections.

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



Zuhtu Ozden, P.Eng.



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



ZO:tr/d#hd

APPENDIX E

Limitations of Report

LIMITATIONS OF REPORT

The conclusions and recommendations given in this report are based on information determined at the testhole locations. The information contained herein in no way reflects on the environment aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the testholes may differ from those encountered at the testhole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the testhole locations and should not be used for other purposes, such as grading, excavating, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report.

The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work. This work has been undertaken in accordance with normally accepted geotechnical engineering practices.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. Shaheen & Peaker Limited accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.