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LOCATION PROPOSED RETAINING WALL AT
N-E/W RAMP TO SHERWAY GARDENS

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OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

**FOUNDATION INVESTIGATION REPORT
PROPOSED RETAINING WALL AT N – E/W RAMP
TO SHERWAY GARDENS ROAD
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: SP3232G
June 12, 2000**

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

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**FOUNDATION INVESTIGATION REPORT
PROPOSED RETAINING WALL
AT N-E/W RAMP TO SHERWAY GARDENS ROAD
TORONTO, ONTARIO
W.P. 176-00-01**

1. INTRODUCTION

As part of the Gardiner Expressway/Q.E.W./Hwy. 427/Browns Line Interchange modifications, a new retaining wall is to be constructed at the proposed N-E/W Ramp to Sherway Gardens Road. Shaheen & Peaker Limited (S&P) was retained by DS-Lea Associates Limited, Consulting Engineers, to carry out a foundation investigation for the proposed retaining wall structure.

The site is located near the interchange of Q.E.W. and Highway 427 in 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.

3. INVESTIGATION PROCEDURES

The fieldwork for the project was performed on February 1 and 4, 2000 and consisted of drilling and sampling five boreholes (Boreholes 7-1 through 7-5) at the locations shown on Drawing No. 1. The depth of the boreholes ranged from 3.3 to 6.8 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 into the shale bedrock at depths ranging from 3.4 to 6.8 m.

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

Groundwater conditions in the open boreholes were observed during the drilling and at completion of each borehole. In addition, in each of Boreholes 7-2 and 7-3, a piezometer was installed to enable us to monitor the groundwater level over a prolonged period of time, without interference from surface water.

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

4. SUBSURFACE CONDITIONS

The subsurface conditions were explored at five borehole locations (Boreholes 7-1 to 7-5). The locations of the boreholes are shown on Drawing No. 1 and are also indicated on the individual borehole log sheets.

The boreholes were drilled from the toe of the existing retaining wall and the ground surface elevations at the borehole locations ranged from 114.3 to 115.5 m. Boreholes 7-1 and 7-2 contacted 200 and 180 mm of asphalt, respectively, followed by 500 and 430 mm granular pavement fill. Borehole 7-3 contacted 500 mm of granular fill at the surface. The granular (pavement) fill generally consists of 250 to 300 mm of crusher run limestone overlying 180 to 250 mm of sand and gravel. A veneer of topsoil with a thickness of 500 mm was encountered at the surface in Borehole 7-4. Buried topsoil with thickness of 300 mm was contacted below the pavement structure in Borehole 7-2.

Fill was contacted below the granular fill or topsoil in Boreholes 7-1, 7-3 and 7-4 and at the surface in Borehole 7-5. The fill consists of a mixture of silty sand to sandy silt to clayey silt with some organics and extends to depths ranging between 0.9 and 1.5 m or to between Elevations 112.8 and 114.3 m.

Underlying the topsoil or fill, the natural overburden generally consists of compact to very dense sand to silty sand or sandy silt. The thickness of this granular deposit varies from 0.3 to 2.6 m and the deposit extends to the surface of the shale bedrock in Boreholes 7-2 and 7-3.

Beneath the granular deposit, Boreholes 7-1, 7-4 and 7-5 contacted a glacial till deposit consisting of a heterogeneous mixture of silt to silt and sand to clay and silt with shale and limestone. The glacial till extends to the surface of the shale bedrock, except at Borehole 7-5 where a 1.3 m thick silty sand deposit with clayey silt layers and shale fragments was contacted below 4.0 m; this deposit extends to the shale bedrock. The surface of the shale bedrock was contacted at depths ranging between 2.1 and 5.3 m or between Elevations 110.2 and 112.2 m in all of the boreholes.

In the piezometers installed in Boreholes 7-2 and 7-3, the groundwater level on March 2, 2000 was measured at 1.3 and 1.5 m, respectively or at Elevation 113.0 and 113.2 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

Boreholes 7-4 contacted approximately 500 mm of topsoil at the surface.

4.2 PAVEMENT

Borehole 7-1 and 7-2 were drilled from the paved road surface and showed the presence of 200 and 180 mm of asphaltic concrete, underlain by 500 and 430 mm of granular pavement fill, respectively. Borehole 7-3 contacted 500 mm of granular fill at the surface. The pavement fill is underlain by a 300 mm thick topsoil layer in Borehole 7-2.

4.3 FILL

Fill was contacted below the granular fill or topsoil in Boreholes 7-1, 7-3 and 7-4 and at the surface in Borehole 7-5. The fill consists of a heterogeneous mixture of silty sand to sandy silt to clayey silt with some organics. The fill extends to depths ranging between 0.9 and 1.5 m below the ground surface or between Elevation 112.8 and 114.3 m.

N-values recorded within the fill generally ranged from 15 to 43 blows/0.3 m indicating that some reasonable degree of compaction was applied when the fill was placed.

4.4 SAND TO SILTY SAND TO SANDY SILT

Immediately underlying the topsoil or fill, the natural overburden generally consists of fine to medium grained sand to silty sand to sandy silt. The thickness of this granular deposit varies from 0.3 to 2.6 m and the deposit extends to the surface of the shale bedrock at depths of 2.4 and 4.1 m in Boreholes 7-2 and 7-3, respectively, or Elevations 111.9 and 110.6 m. N-values recorded in the granular deposit ranged between 20 and 55 blows/0.3 m indicating that it is in compact to very dense.

Typical grain size distribution curves from the granular deposit are shown in Appendix B.

4.5 TILL

Beneath the granular deposits in Boreholes 7-1, 7-4 and 7-5, a glacial till deposit was contacted. The till consists of a heterogeneous mixture of silt to silt and sand to clay and silt with shale and limestone fragments. The frequency of shale and limestone increases with increasing depth. The presence of cobbles and boulders can always be expected in the glacial till deposits. The till extends to the surface of the shale bedrock, except at Borehole 7-5 where a 1.3 m thick silty sand layer with clayey silt seams and shale fragments was contacted below 4.0 m and extended to the surface of the shale bedrock at 5.3 m below the ground surface or Elevation 110.2 m. The surface of the shale bedrock below the till and the underlying silty sand deposit (Borehole 7-5) was contacted at depths ranging from 2.1 to 5.3 m or between Elevation 110.2 to 112.2 m.

N-values recorded in the till ranged from 42 blows/0.3 m to 50 blows/0.08 m, indicating a dense to very dense relative density.

4.7 SHALE BEDROCK

Shale bedrock was encountered in all the five boreholes at the following approximate depths/elevations:

Estimated Bedrock Depth

Borehole No.	Bedrock Depth (m)	Elevation (m)
7-1	2.1	112.2
7-2	2.4	111.9
7-3	4.1	110.6
7-4	4.6	110.3
7-5	5.3	110.2

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 and, where possible, from split-spoon samples and auger cuttings.

The bedrock underlying the site is known to belong 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 seams was inferred from resistance to the auger and from the SPT samples in Boreholes 7-1 to 7-3 and silt seams were present in Borehole 7-3.

4.8 GROUNDWATER CONDITIONS

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

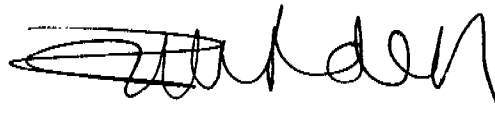
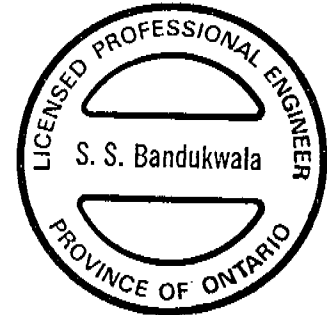
To enable us to monitor the groundwater level over a prolonged period of time without interference from surface water, a piezometer was installed in each of Boreholes 7-2 and 7-3. The water level in the piezometers was monitored over a period of time and the recorded values are shown on the borehole logs. The final water levels were recorded at 1.3 m (Elevation 113.0 m) and 1.5 m (Elevation 113.2 m) in Boreholes 7-2 and 7-3, 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

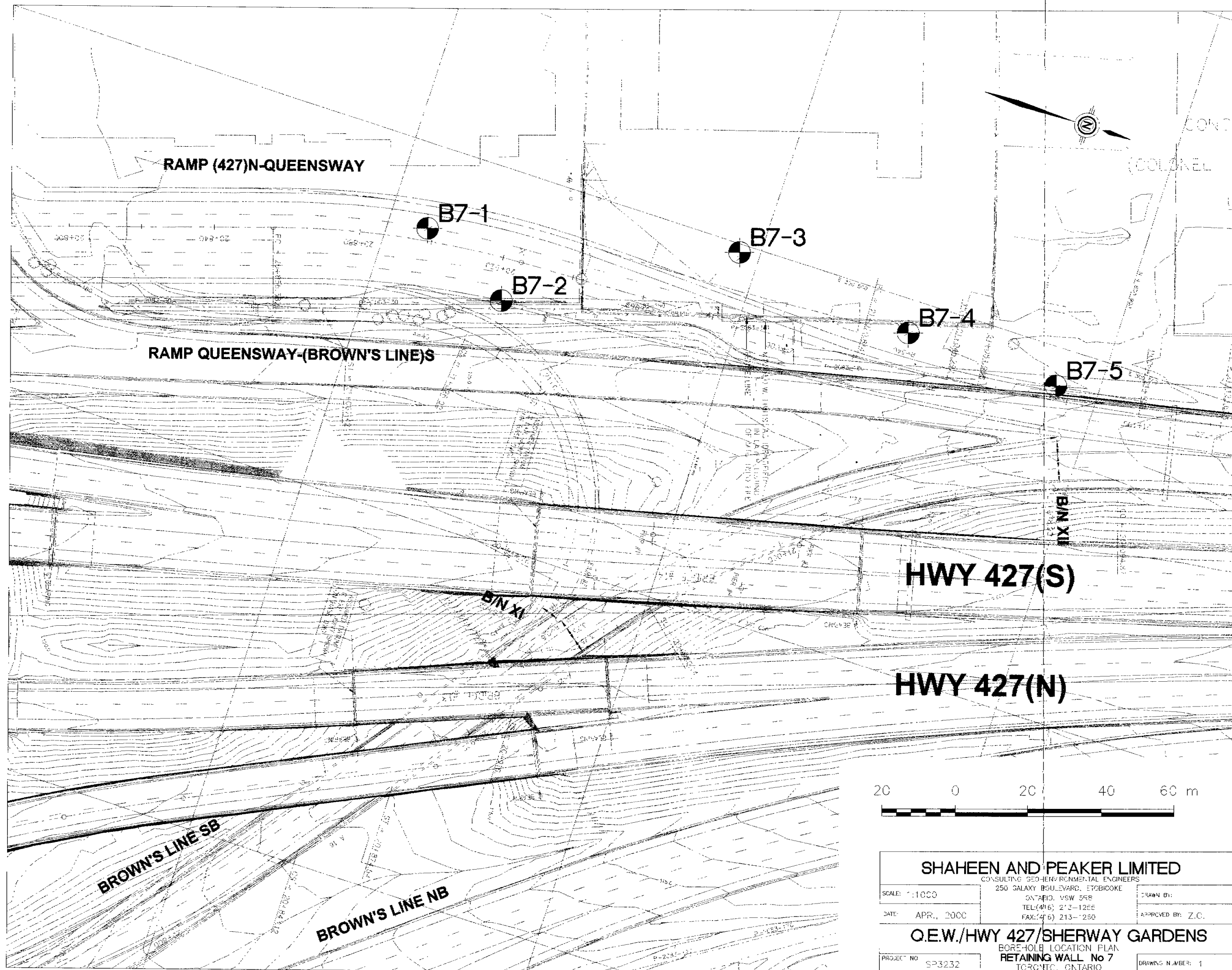


Shabbir Bandukwala, P. Eng.



Zuhtu Ozden, P.Eng.





APPENDIX A

Borehole Log Sheets

RECORD OF BOREHOLE No 7-1

1 OF 1

METRIC

W.P. 176-00-01 LOCATION Ramp N (427) - E/W (Queensway); 4 830 711 N; 300 522.7 E ORIGINATED BY G.I.
 DIST HWY 427 & QEW BOREHOLE TYPE Solid Stem Augers COMPILED BY G.I.
 DATUM Geodetic DATE 01.02.00 CHECKED BY S.B.

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	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
114.3	Ground surface													
114.1	200 mm Asphalt													
113.8	300 mm Crusher run limestone		1	AS			114							
113.6	200 mm Sand and Gravel													
0.7	FILL: Silty Sand, organic seams, brown, dense		2	SS	35		113							
112.8														
112.5	SAND		3	SS	42									
112.2	fine to medium grey, wet, dense						112							
2.1	SILT TILL: weathered, grey, Shale fragments, dense		4	SS	49									
	SHALE BEDROCK													
	highly weathered in the upper													
110.9	0.5 m, occasional limestone layers, grey		5	SS	50/14		111							
3.4	End of borehole Dry on completion Water level not stabilized													N=50/14 denotes 50 blows for 14 cm penetration

+ 3, x 3: Numbers refer to Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 7-2

1 OF 1

METRIC

W.P. 178-00-01 LOCATION Ramp N (427) - E/W (Queensway); 4 830 736.4 N; 300 535.3E ORIGINATED BY G.I.
 DIST HWY 427 & QEW BOREHOLE TYPE Solid Stem Augers COMPILED BY G.T.
 DATUM Geodetic DATE 01.02.00 CHECKED BY S.B.

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		
114.3	Ground surface													
114.1	180 mm Asphalt													
113.9	250 mm Crusher run Limestone		1	AS			114							
113.7	180 mm Sand and Gravel													
113.4	TOPSOIL: Sandy silt, trace gravel, black		2	SS	20		113							1 89 (10)
0.9	SAND: fine to medium, brown, grey below 1.6 m, moist to wet, compact to very dense		3	SS	55		112							
112.6	SANDY SILT layered, grey, wet, very dense		4	SS	71		111							
111.9	SHALE BEDROCK weathered, occasional limestone layers, grey, very dense		5	SS	90/25		110							
110.9	End of borehole Water level in piezometer Feb. 02 = 1.7 m Feb. 03 = 1.7 m Feb. 04 = 1.7 m Feb. 05 = 1.7 m Feb. 06 = 1.7 m Feb. 07 = 1.7 m Feb. 12 = 1.7 m Feb. 22 = 1.8 m March 02 = 1.3 m													
3.4														

+ 3, x 3: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 7-3

1 OF 1

METRIC

W.P. 176-00-01 LOCATION Ramp N (427) - E/W (Queensway); 4 830 794.8 N; 300 504 E ORIGINATED BY G.I
DIST HWY 427 & QEW BOREHOLE TYPE Solid Stem Augers COMPILED BY G.T
DATUM Geodetic DATE 01.02.00 CHECKED BY S.B

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		
114.7	Ground surface													
114.5	250 mm Crusher run limestone	Δ												
114.2	250 mm Sand and Gravel	□	1	AS										
0.5	FILL	⊗												
	Silty Sand, brown, moist to wet, dense		2	SS	43									
113.2														
1.5	SILTY SAND	—												
	fine to medium, brown, wet, compact to dense		3	SS	26									
			4	SS	43									
111.7														
3.0	SAND	—												
	fine to medium, some silt, grey, wet, very dense		5	SS	54									
			6	SS	68									
110.6														
4.1	SHALE BEDROCK	—												
	highly weathered in the upper 0.5 m, silt seams, occasional limestone layers, grey		7	SS	50/8									
109.3														
5.4	End of borehole													
	Water level in piezometer													
	Feb.02 = 1.8 m													
	Feb.03 = 1.8 m													
	Feb.04 = 1.9 m													
	Feb.05 = 1.9 m													
	Feb.06 = 1.9 m													
	Feb.07 = 1.9 m													
	Feb.22 = 1.9 m													
	March 02 = 1.5 m													

+ 3, x 3: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 7-4

1 OF 1

METRIC

W.P. 176-00-01 LOCATION Ramp N (427) - EW (Queensway); 4 830 845.4 N; 300 509.1 E ORIGINATED BY M.J.
DIST HWY 427 & QEW BOREHOLE TYPE Solid Stem Augers COMPILED BY G.T.
DATUM Geodetic DATE 04.02.00 CHECKED BY S.B.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT Y kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
114.9	Ground surface													
0.0	TOPSOIL		1	AS										
114.4			2	SS	13									
114.0	FILL: clayey silt to silty sand, some organics, compact.		3	SS	32		114							
0.9	SILTY SAND fine to medium, occasional till layers, brown, compact		4	SS	30		113							
112.5			5	SS	44		112							
2.4	SAND medium, brown, wet, dense		6	SS	44		111							
111.5			7	SS	50/8		110							
3.4	Heterogeneous mixture of Silt and Sand some gravel (SILTY SAND TILL)		8	SS	50/15		109							
110.3	brown, wet, very dense													
4.6	SHALE BEDROCK highly weathered to weathered, grey													
108.1	rubble seam at 6.7 m													
6.8	End of borehole Water level on completion 2.4 m (not stabilized)													

+ 3, x 3; Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 7-5

1 OF 1

METRIC

W.P. 176-00-01 LOCATION Ramp N (427) - E/W (Queensway); 4 830 888.5 N; 300 510.6 E ORIGINATED BY M.J.
 DIST HWY 427 & QEW BOREHOLE TYPE Solid Stem Augers COMPILED BY G.I.
 DATUM Geodetic DATE 04.02.00 CHECKED BY S.B.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
115.5	Ground surface																
0.0	FILL Sandy silt, some organics, trace roots, dark brown to black, compact		1	AS			115										
114.3			2	SS	15												
114.0	SAND : brown, moist		3	SS	29		114										
1.5	CLAYEY SILT TILL		4	SS	38												
113.4	Shale fragments, grey, hard																
2.1	SILT TILL numerous limestone/shale fragments, grey, very dense		5	SS	50		113										
			6	SS	50/13		112										
111.5																	
4.0	SILTY SAND with Clayey Silt seams, Shale pieces, grey, wet, very dense		7	SS	50/15		111										
110.2																	
5.3	SHALE weathered, grey						110										
109.3			8	SS	50/8												
6.2	End of borehole Water level on completion 3.0 m (not stabilized)																

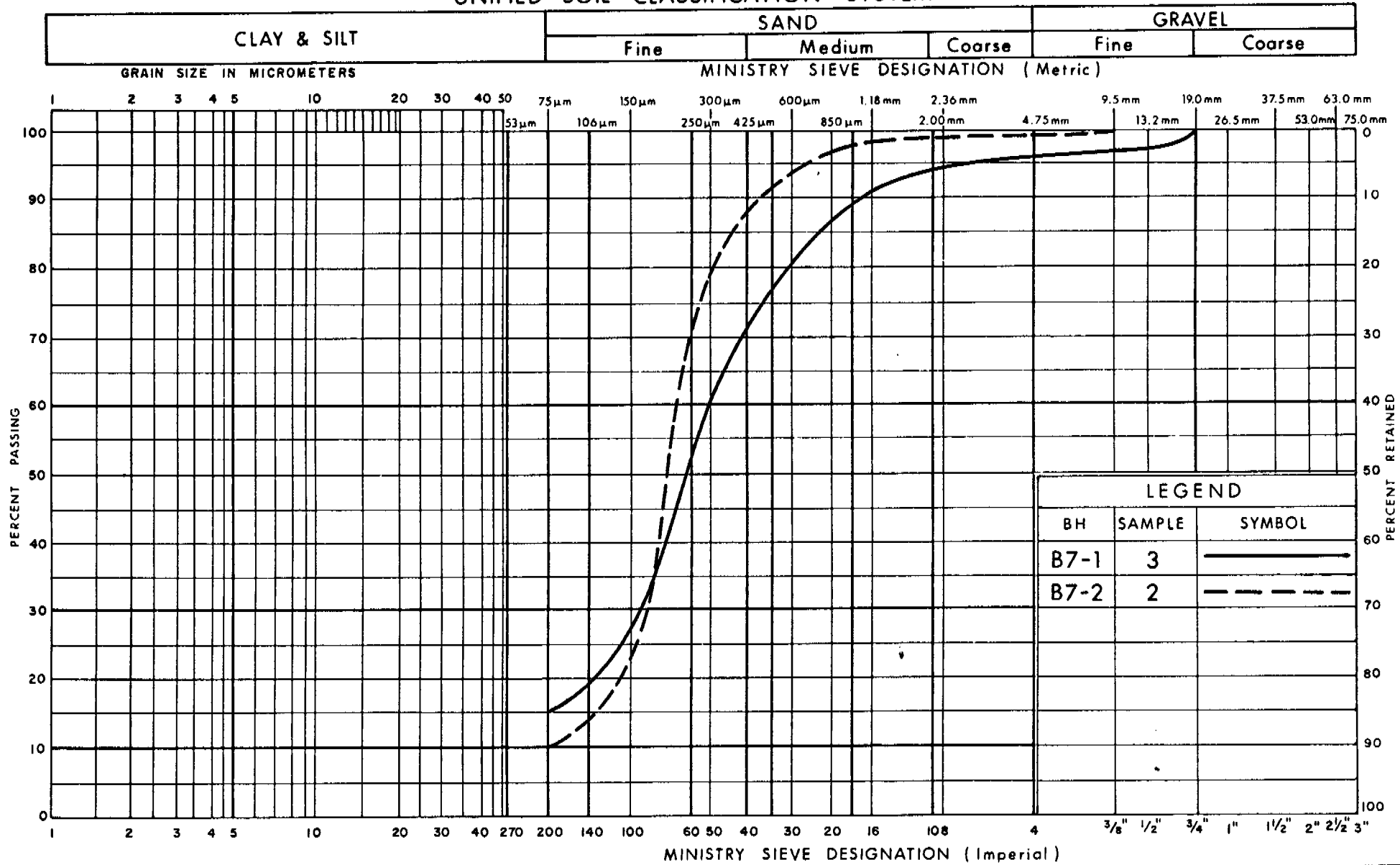
+ 3, x 3: Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE

APPENDIX B

Laboratory Test Results

UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of
Transportation

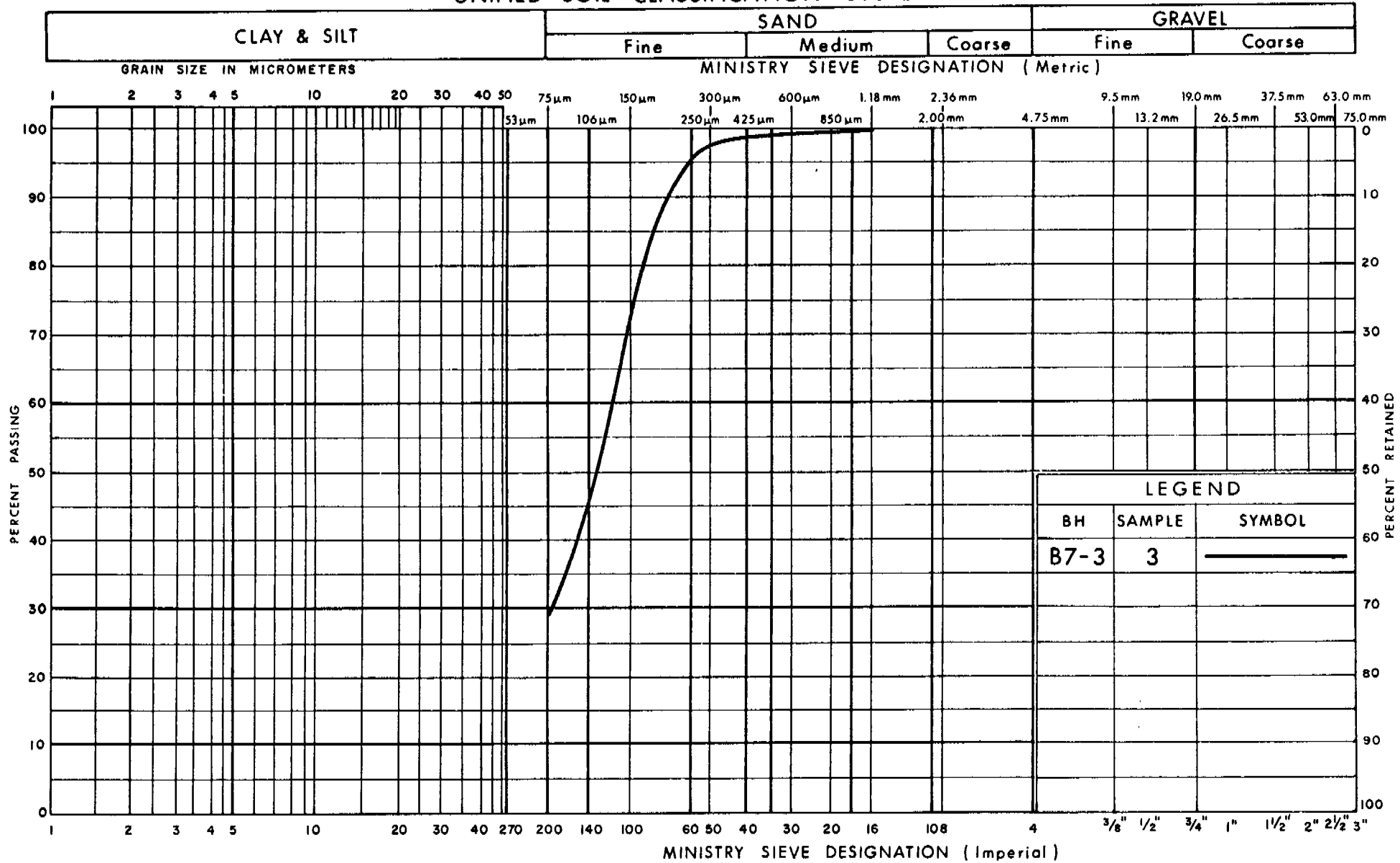
GRAIN SIZE DISTRIBUTION SAND

FIG No 1

W P 176-00-01

SP 3232G

UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of
Transportation

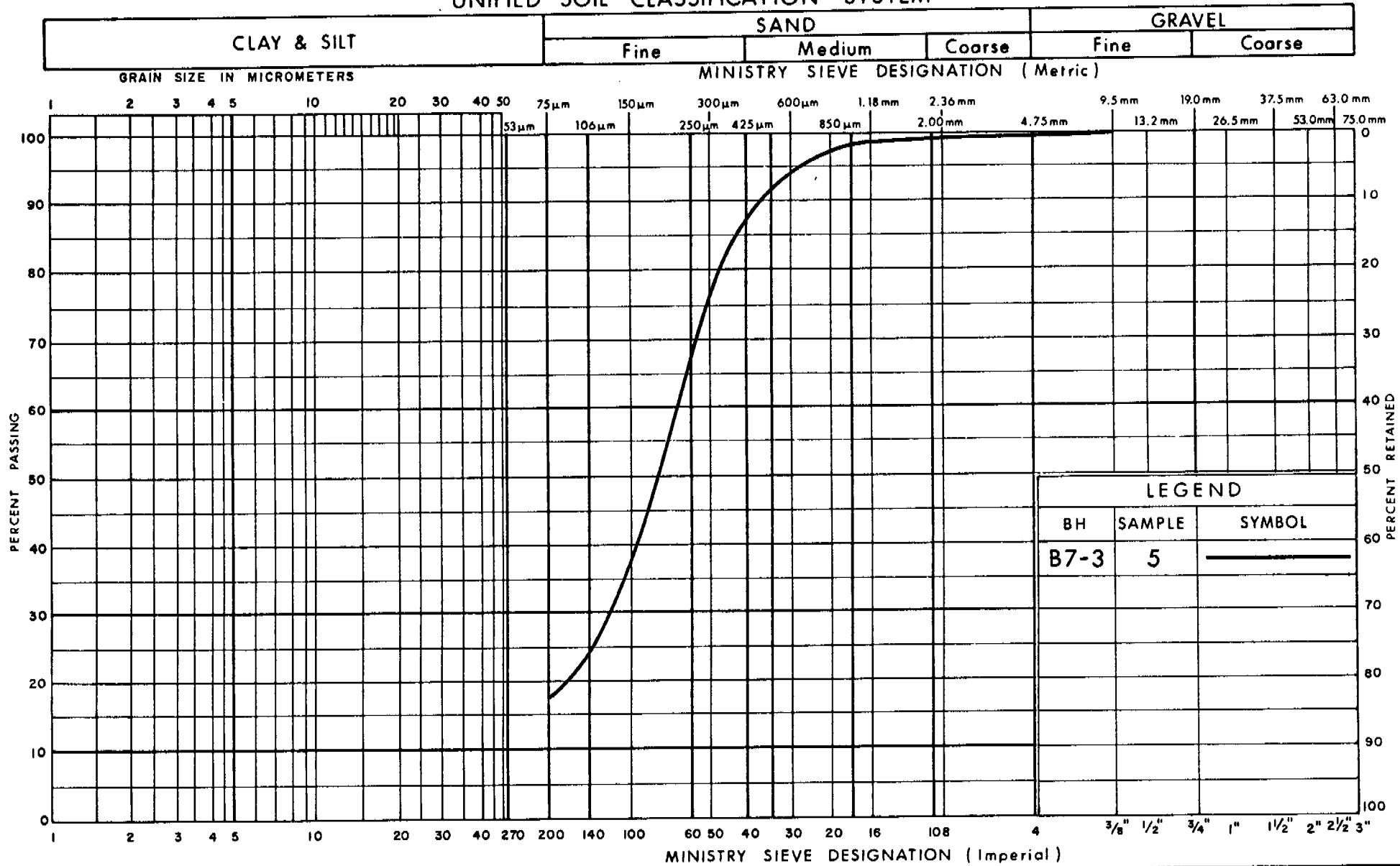
GRAIN SIZE DISTRIBUTION
SILTY SAND

FIG No 2

W P 176-00-01

SP 3232G

UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of
Transportation

Ontario

GRAIN SIZE DISTRIBUTION
SAND, SOME SILT

FIG No 3

W P 176 -00-01

SP 3232 G

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 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS \bar{N} .

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

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

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

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

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

JOINTING AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

MECHANICAL PROPERTIES OF SOIL

m_v	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'_{v0}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

STRESS AND STRAIN

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

PHYSICAL PROPERTIES OF SOIL

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

**FOUNDATION DESIGN REPORT
PROPOSED RETAINING WALL AT N – E/W RAMP
TO SHERWAY GARDENS
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: SP3232G
June 12, 2000**

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

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Limitations of Report

Appendix D

**FOUNDATION DESIGN REPORT
PROPOSED RETAINING WALL
AT N – E/W RAMP TO SHERWAY GARDENS ROAD
TORONTO, ONTARIO
W.P. 176-00-01**

5. DISCUSSION AND RECOMMENDATIONS

We understand that part of the existing ramp will be demolished and replaced with a new ramp to the west of the existing location. The approach embankment will be about 180 m long and will incorporate a retaining wall structure. At its highest location the retaining wall structure will be approximately 5 m deep (at the north end) and the embankment will eventually meet the existing grade.

At the borehole locations, the existing ground surface elevation ranges from 114.3 to 115.5 m, which is higher than the general bedrock elevation in the area. Fill consisting of a mixture of silty sand to sandy silt to clayey silt with organics and extending to depths ranging between 0.9 to 1.5 m was encountered in the boreholes. Underlying the topsoil or fill materials, the native material consists of compact to very dense sand to silty sand or sandy silt. The thickness of this granular deposit varies from 0.3 to 2.6 m. Glacial till consisting of a heterogeneous mixture of silt to silt and sand with shale and limestone fragments/slabs was encountered below the granular deposit at Boreholes 7-1, 7-4 and 7-5. The glacial till or granular deposit (i.e. sandy silt or sand) overlies the shale bedrock. The surface of the shale bedrock was encountered at approximate elevations ranging between 110.2 and 112.2 m.

5.1 FOUNDATIONS

The boreholes show that the proposed retaining wall structure can be supported on conventional strip footing foundations founded on the natural competent, compact to dense sands, below any fill materials, at depths of 1.2 to 1.5 m below existing grade. The recommended founding elevations and soil bearing resistances at the borehole locations are presented in the following Table. The footings should have a minimum width of 1.2 m.

TABLE 1

Borehole No.	Existing Ground Surface Elevation (m)	Recommended Highest Footing Base (m)		Geotechnical Resistance (kPa)		Subgrade Materials	Remarks
		Depth Below Existing Ground Surface (m)	Elevation (m)	At S.L.S.	At U.L.S.*		
7-1	114.3	1.5	112.8	250	500	Sand Silt till	
		1.8	112.5	300	600		
7-2	114.3	1.2	113.1	250	500	Sand Sandy Silt	Dewatering required**
		1.7	112.6	400	800		
7-3	114.7	1.5	113.2	250	500	Silty Sand Silty Sand	Dewatering required**
		2.2	112.5	400	800		
7-4	114.9	1.2	113.7	250	500	Silty Sand Sand	Dewatering Required**
		2.4	112.5	400	800		
7-5	115.5	1.5	114.0	400	700	Clayey Silt Till Silt till	
		2.3	113.2	500	800		

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

** Based on water levels recorded at the time of our investigation.

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

Due to the shallow depth of the recorded water table, it is recommended that the footings be founded at the highest possible elevation given in the above Table 1. Lowering the footing elevations at Borehole 7-2, 7-3 and 7-4 will require positive dewatering prior to excavation. If proper dewatering is not applied, loss of ground and sloughing would occur. Any excavation carried out to a depth of 0.5 m or more below the water table will require the use of well points to

lower the water table to not less than 0.8 m below the base of the footing excavations.

All footing excavations should be carefully inspected by the geotechnical engineer to ensure that the footings are founded on natural undisturbed soils, which are capable of supporting the design pressures.

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.

Based on the subgrade conditions and the height of the retaining wall, global stability is not considered a problem. The unfactored horizontal resistance against sliding between concrete and approved undisturbed, competent, native founding soil can be calculated using a friction angle of 28 degrees.

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

The lateral earth pressures acting on the retaining wall will depend on the type and method of placement of the backfill materials and on the subsequent lateral movement of the structure. The lateral earth pressures to be used in the design should be computed in accordance with Section 6-7 of the OHBDC.

Granular backfill should be placed behind the retaining walls to conform to the minimum requirements illustrated in OPSD 3504.00. The granular backfill should conform to OPSS Form 1010 for either Granular 'A' or 'B' Type 1. To maintain free draining characteristics in these granular fill materials, the maximum percentage passing the No. 200 sieve (75 μ m) should be limited to 5%.

The backfill should be placed in accordance with OPSS 501. A perforated subdrain should be installed behind the base of the walls as shown in OPSD 3504.00 to maintain the granular fill in a drained condition. The subdrain should be directed through a positive outlet into a municipal sewer or highway drainage system.

$$P_h = K(\gamma H + Q)$$

Where

$$\gamma = \text{Unit weight of soil, } = 21.2 \text{ kN/m}^3 \text{ for Granular 'B'}$$
$$= 22.8 \text{ kN/m}^3 \text{ for Granular 'A'}$$

H = Depth below top of wall, m

Q = Unit surcharge pressure

WALL TYPE	Earth pressure Coefficient (K)	
	GRANULAR 'A' $\phi = 35^\circ$	GRANULAR 'B' $\phi = 30^\circ$
Restrained Wall (K_0)	0.43	0.50
Unrestrained Wall (K_a)	0.27	0.33

At some locations, the new retaining wall will be constructed in front of the existing retaining wall adjacent to the existing roadway. After the new wall is constructed (up to 5 m high), the upper portion (i.e. upper 1.2 m±) of the existing concrete retaining wall will be removed. The new construction will create a void space in between the two walls. This space will increase from nil to a maximum of about 2.5 m. The earth pressures generated by the backfill in between the two structures are as follows:

- (a) Use of lean concrete = lean concrete will act as a thick liquid until the concrete sets and therefore, during this period, a coefficient of earth pressure (K) equal to 1.0 would apply.
- (b) Use of earth fill = the backfill in the gap between the retaining structures will transform full earth pressure from the existing structure into the new (front) wall. In addition, because of the vibrations induced by the traffic, earth pressure coefficient of at rest should be used (i.e. K_0) for the calculation of earth pressures. If a clear crushed stone is used for backfill (i.e. less prone to settlements when placed from the top with less than adequate compaction) then a K_0 value of 0.4 can be used for this purpose, along with a unit weight of 21 kN/m^3 . This assumes that proper drainage is provided to relieve any hydrostatic pressures.
- (c) The design values for a K_0 value for Granular 'A' and Granular 'B' materials are $K_0=0.43$ for Granular 'A' and 0.50 for Granular 'B' materials, respectively (see Table 2). Needless to say, allowance should also be made for traffic loads. These values can be used for the calculation of earth pressure behind the existing retaining wall (which will be transferred to the new wall as described above) provided that these materials are present. A K_0 value of 0.5 can be used to calculate the earth pressure behind the existing wall for acceptable ordinary backfill such as SSM materials.

A compaction surcharge equal to 16 kPa should be included in the lateral earth pressures for the structural design of the retaining walls in accordance with OHBDC 6-7.4.3.

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

5.3 RETAINED SOIL SYSTEM

A Retained Soil System (RSS) may be used for the retaining walls. The supplier of the RSS should be responsible for design of the structure such as backfill, reinforcement, and internal and external stability. The following information should be included in the contract drawing:

- The length and location
- Height and space constraints
- Elevation of top and bottom of RSS
- Performance requirement (High Performance)
- Appearance requirement (High Appearance)

5.4 CONSTRUCTION COMMENTS

Water levels in the piezometers installed in Boreholes 7-2 and 7-3 were recorded at shallow depths of 1.3 to 1.5 m, i.e. Elevations 113.0 and 113.2 m, respectively. Due to the shallow depth of the water table, it is recommended that the footings be founded at the highest possible elevation given in the Section 5.1, Table 1. Lowering the footing elevations at Borehole 7-2, 7-3 and 7-4 will require positive dewatering prior to excavation, otherwise it will result in loss of ground and sloughing. The groundwater can be lowered by up to 0.5 m by pumping from over-sized excavations and using flatter side slopes of about 2.5 horizontal to 1.0 vertical. Any excavation carried out deeper than 0.5 m below the watertable, will require the use of well-points. If well points are used, then the watertable must be lowered to not less than 0.8 m below the base of the footing excavations.

Above the water table, temporary excavations more than 1.2 m deep should be sloped no steeper than 1:1. In the shale, somewhat steeper side slopes may be permissible, as directed by the geotechnical engineer. Alternatively, deeper excavations should be supported in accordance with the Provincial Safety Regulations. All slope faces should be protected against surface erosion.

It is recommended that an approximately 150 mm thick layer of lean concrete be placed on bearing surfaces within four hours of excavation. Following the construction of the footings, backfill should be placed to a sufficient height

above the footing (i.e. at least 1.2 m) to prevent disturbance and frost penetration. All foundation excavations and bearing surfaces should be inspected and approved by the geotechnical engineer appointed by the contract administrator.

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

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

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

The materials used for the construction of the embankment fills should consist of approved, acceptable fill, free of cobbles and boulders, frozen materials, organic soils, etc. (e.g. select subgrade materials – OPSS 1010). The fill should be placed in thin lifts not exceeding 300 mm before compaction and each lift should be uniformly compacted to at least 95% of the material's Standard Proctor Maximum Dry Density. The degree of compaction within the top 0.6 m of the fill (i.e. subgrade immediately beneath the granular subbase) should be increased to 98%. The selection, placement and compaction of the fill should be carried out under the supervision of a geotechnical engineer. The settlement of embankment fills

prepared as described above should not exceed 50 mm, based on the borehole results and assuming that any existing fills not removed were properly compacted when they were first placed. Surcharging or pre-loading is therefore considered not to be necessary.

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

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

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 D, are an integral part of this report.

Shaheen & Peaker Limited


Shabbir Bandukwala, P.Eng.




Zuhtu Ozden, P. Eng.



APPENDIX D

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.