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STR. SITE No.

HWY. No. QEW

LOCATION PROPOSED RETAINING WALL
FOR RAMP (FGGE) E -(BROWN'S LINE) S
427 - Q ENSWAY AVE

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

**FOUNDATION INVESTIGATION REPORT
PROPOSED RETAINING WALL FOR RAMP (FGGE) E-(BROWN'S LINE) S
427N – QUEENSWAY AVENUE
TORONTO, ONTARIO
W.P. 176-00-01**

Prepared For:

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**FOUNDATION INVESTIGATION REPORT
PROPOSED RETAINING WALL
FOR RAMP (FGGE) E-(BROWNS LINE) S
427N – QUEENSWAY AVENUE
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 Ramp (FGEE) E-Browns Line S, 427N – Queensway Avenue. 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 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.

3. INVESTIGATION PROCEDURES

The fieldwork for the project was performed on February 16, 17, 20 and 21, 2000 and consisted of drilling and sampling four boreholes (Boreholes B9-1 through B9-4) at the locations shown on Drawing No. 1. The depth of the boreholes ranged from 1.2 to 9.0 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 to about 6 m depth and at 1.5 m intervals, thereafter) 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 soil samples were shipped to our laboratory in Toronto for further examination and classification. A laboratory testing programme, consisting of natural moisture content 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 B9-2 and B9-4, 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 four borehole locations (Boreholes B9-1 to B9-4). The locations of the boreholes are shown on the Borehole Location Plan, Drawing No. 1 and are also indicated on the individual borehole log sheets.

Borehole B9-2 was drilled from near top of the existing embankment and Borehole B9-3 was drilled from within the road allowance. The ground surface elevations at the borehole locations ranged from 110.7 and 117.8 m on the west side (i.e. Boreholes B9-1 and 2) to 107.7 and 106.4 m on the east side (Boreholes B9-3 and 4). A veneer of topsoil with thickness of 300 and 250 mm was encountered at the surface in Boreholes B9-1 and B9-2 and a 100 mm thick surficial asphalt pavement was encountered in Borehole B9-3. Borehole B9-3 showed the presence of a granular pavement fill beneath the surficial asphalt extending to the surface of the bedrock. Boreholes B9-2 and B9-4 showed the presence of embankment related fill to depths of 5.5 m and 0.4 m or to Elevations 112.3 and 106.0 m, respectively. Substantial inclusion of topsoil mixed with the fill is present in Borehole B9-4. Underlying the fill in Borehole B9-2, the natural overburden generally consists of dense stratified fine to medium sand. Beneath the topsoil in Borehole B9-1 and sand in B9-2 is a till/shale complex, representing transition to the underlying shale bedrock. The surface of the grey shale bedrock, which underlies the till/shale complex at Boreholes B9-1 and B9-2 and the fill in Boreholes B9-3 and B9-4, was contacted at approximately 0.4 to 7.6 m below the ground surface or between Elevations 110.2 and 106.0 m.

In the piezometers installed in Boreholes B9-2 and B9-4, the groundwater level on March 2, 2000 was measured at 6.9 and 2.2 m, respectively or at Elevations 110.9 and 104.2 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 B9-1 and B9-2 contacted approximately 300 and 250 mm of topsoil, respectively.

4.2 PAVEMENT

Borehole B9-3 was drilled from the paved road surface and showed the presence of 100 mm of asphaltic concrete, underlain by granular pavement fill extending to 0.6 m below the ground surface.

4.3 FILL

Beneath the topsoil in Borehole B9-2 and at the surface in Borehole B9-4 the embankment fill was contacted. At B9-2 location the embankment fill is about 5.5 m deep and at B9-4 location the depth reduces to 0.4 m. At B9-2, the fill was found to consist of a heterogeneous mixture of clayey silt with shale (reworked shale) and limestone fragments/slabs. Brick and asphalt pieces were found at 1.0 m along with topsoil layer below 4.9 m. The fill extends to elevations of 112.3 m and 106.0 m at B9-2 and B9-4 respectively.

N-values recorded within the fill generally ranged from 8 to 55 blows/0.3 m indicating that some reasonable degree of compaction was applied when the embankment was built.

4.4 SAND

Underlying the fill in Borehole B9-2, a 1.5 m thick sand layer was contacted at a depth of 5.5 m or at Elevation 112.3 m. This is a basically fine to medium grained granular material and from recorded N-values of 52 and 53 blows/0.3 m it is described as very dense.

4.5 TILL/SHALE COMPLEX

Underlying the topsoil in Borehole B9-1 and the sand layer in Borehole B9-2, a clayey till deposit was contacted which contains very frequent shale fragments. This grey till often resembles a highly weathered shale and it is sometimes referred to as a till/shale complex. It represents a transition zone into the underlying shale bedrock. This material is often described as a residual soil or a completely weathered shale bedrock. Limestone layers may remain. For this reason, contractors are advised that excavation methods should take into consideration the possible presence of hard shale or limestone slabs/layers in the till, till/shale and in the underlying shale bedrock.

The till/shale complex is a basically cohesive material and based on the resistance to augering and the recorded N-values its consistency is described as hard.

4.6 SHALE BEDROCK

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

Estimated Bedrock Depth

Borehole No.	Bedrock Depth (m)	Elevation (m)
B9-1	0.6	110.1
B9-2	7.6	110.2
B9-3	0.6	107.1
B9-4	0.4	106.0

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 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 seams was noted from resistance to the auger and from the SPT samples in all of the boreholes.

4.7 GROUNDWATER CONDITIONS

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

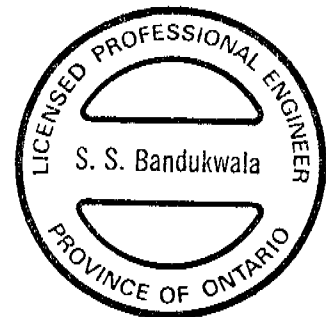
To enable us to monitor the groundwater level over a prolonged period of time without interference from surface water, a piezometer was installed in each of Boreholes B9-2 and B9-4. The water level in the piezometers was monitored over a period of time and the recorded values are shown on the borehole logs. The final water levels were recorded at 6.9 m (Elevation 110.9 m) and 2.2 m (Elevation 104.2 m) in Boreholes B9-2 and B9-4, 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

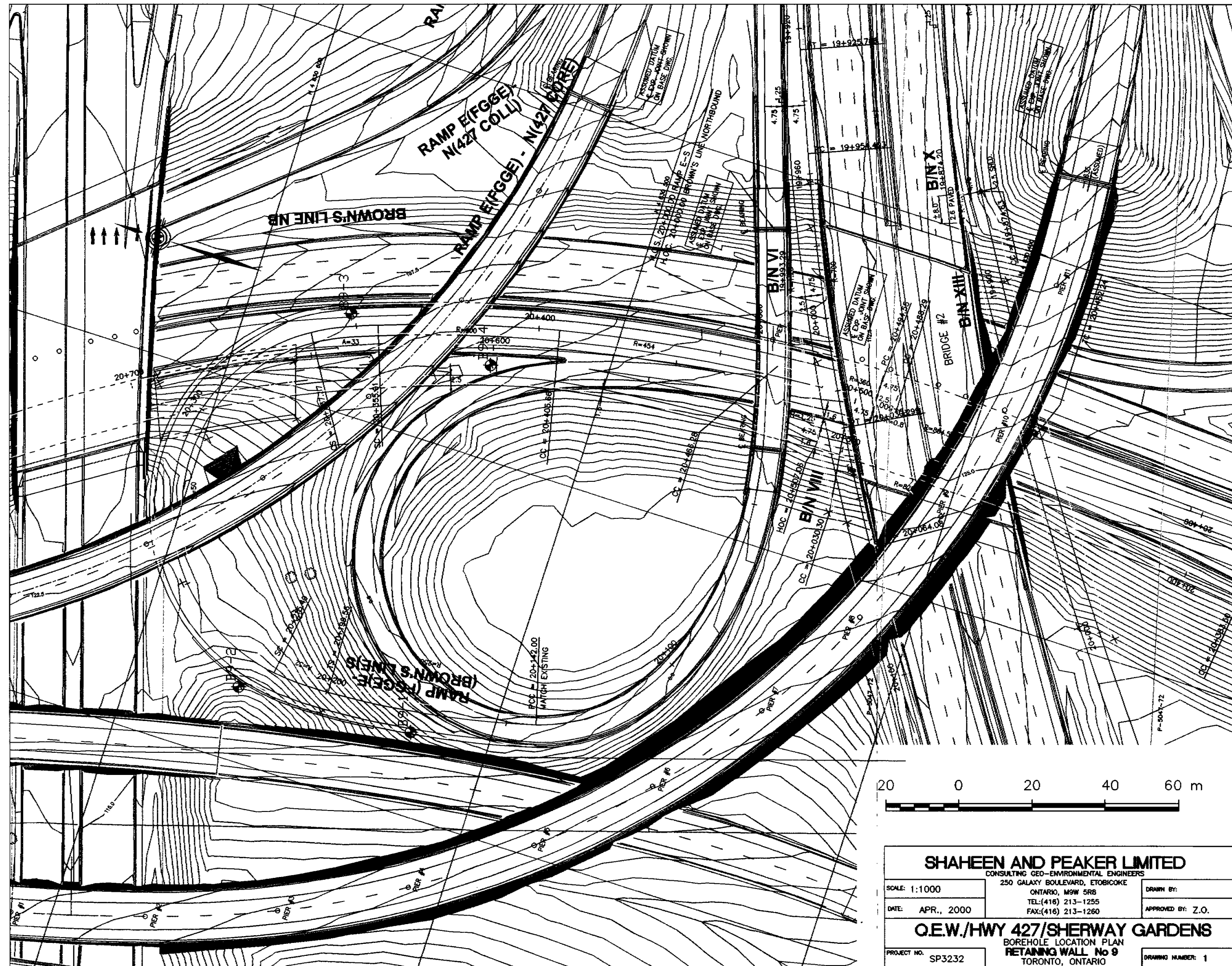


Shabbir Bandukwala, P. Eng.



Zuhtu Ozden, P.Eng.





APPENDIX A

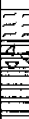
Borehole Log Sheets

RECORD OF BOREHOLE No B9-1

1 OF 1

METRIC

W.P. 176-00-01 LOCATION Ramp FGGE(E)-Brown's Line S; 4 830 521.5 N; 300 722.8 E ORIGINATED BY E.P.
 DIST HWY 427 & QEW BOREHOLE TYPE Solid Stem Augers COMPILED BY G.T.
 DATUM Geodetic DATE 16.02.00 CHECKED BY Z.O.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
110.7	Ground surface							20	40	60	80	100					
110.4	300 mm TOPSOIL		1	SS	14												
110.1	TILL/SHALE Complex																
0.6	SHALE BEDROCK		2	SS	50/13		110										
109.5	grey, limestone layers		3	AS													
1.2	End of borehole Dry on completion Water level no stabilized																N=50/13 denotes 50 blows for 13 cm penetration

3 x 3: Numbers refer to
Sensitivity

20
15-5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No B9-2

1 OF 1

METRIC

W.P. 176-00-01 LOCATION Ramp FGGE(E)-Brown's Line S; 4 830 570.1 N; 300 720.5 E ORIGINATED BY G.I
DIST HWY 427 & QEW BOREHOLE TYPE Solid Stem Augers COMPILED BY G.T
DATUM Geodetic DATE 16.02.00 & 17.02.00 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 (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	N VALUES			SHEAR STRENGTH kPa						WATER CONTENT (%)		
								20 40 60 80 100		20 40 60						
117.8	Ground surface															
117.6	250 mm TOPSOIL		1	SS	16											
0.3	FILL Clayey SIL, with shale and limestone fragments/slabs, pieces of brick and asphalt at 1.0 m		2	SS	25											
			3	SS	34											
			4	SS	8											
			5	SS	32											
			7	SS	38											
112.9	FILL		8	SS	53											
4.9	Sand overlying SIL topsoil, compact		9	SS	52											
112.3	SAND															
5.5	fine to medium, stratified, very dense, wet at 6.4 m															
110.8	TILL/SHALE complex		10	SS	50/5											
7.0	grey, hard															
110.2	SHALE BEDROCK															
7.6	grey, occasional limestone seams															
108.8	End of borehole															
9.0	Water level in piezometer Feb.20 - 6.9 m Feb.22 - 6.9 m Feb.24 - 6.9 m March.2 - 6.9 m															

+ 3, X 3: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No B9-3

1 OF 1

METRIC

W.P. 176-00-01 LOCATION Ramp FGGE(E)-Brown's Line S; 4 830 571.7 N; 300 827.1 E ORIGINATED BY G.I.
 DIST HWY 427 & QEW BOREHOLE TYPE Solid Stem Augers COMPILED BY G.T.
 DATUM Geodetic DATE 20.02.00 CHECKED BY S.B.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					
107.7	Ground surface												
107.6	100 mm ASPHALT												
107.1	FILL: Sand and Gravel, brown		1	AS									
0.6	SHALE BEDROCK grey, occasional limestone seams					107							
106.1													
1.6	End of borehole Dry on completion Water level not stabilized												

+ 3, x 3: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No B9-4

1 OF 1

METRIC

W.P. 176-00-01 LOCATION Ramp FGGE(E)-Brown's Line S; 4 830 531.2 N; 300 825.6 E ORIGINATED BY G.I.
 DIST HWY 427 & QEW BOREHOLE TYPE Solid Stem Augers COMPILED BY G.T.
 DATUM Geodetic DATE 21.02.00 CHECKED BY S.B.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa				WATER CONTENT (%)				
						20	40	60	80	100	20	40	60			
106.4	Ground surface															
0.0	FILL		1	SS	55											
106.0	Clayey Silt with topsoil		2	SS	50/13											
0.4	SHALE BEDROCK grey, occasional limestone seams		3	SS	50/3											
			4	SS	50/3											
			5	SS	50/3											
102.5			6	SS	50/3											
3.9	End of borehole Water level in piezometer Feb.22 = 2.3 Feb.24 = 2.3 March 2 = 2.2		7	SS	50/3											

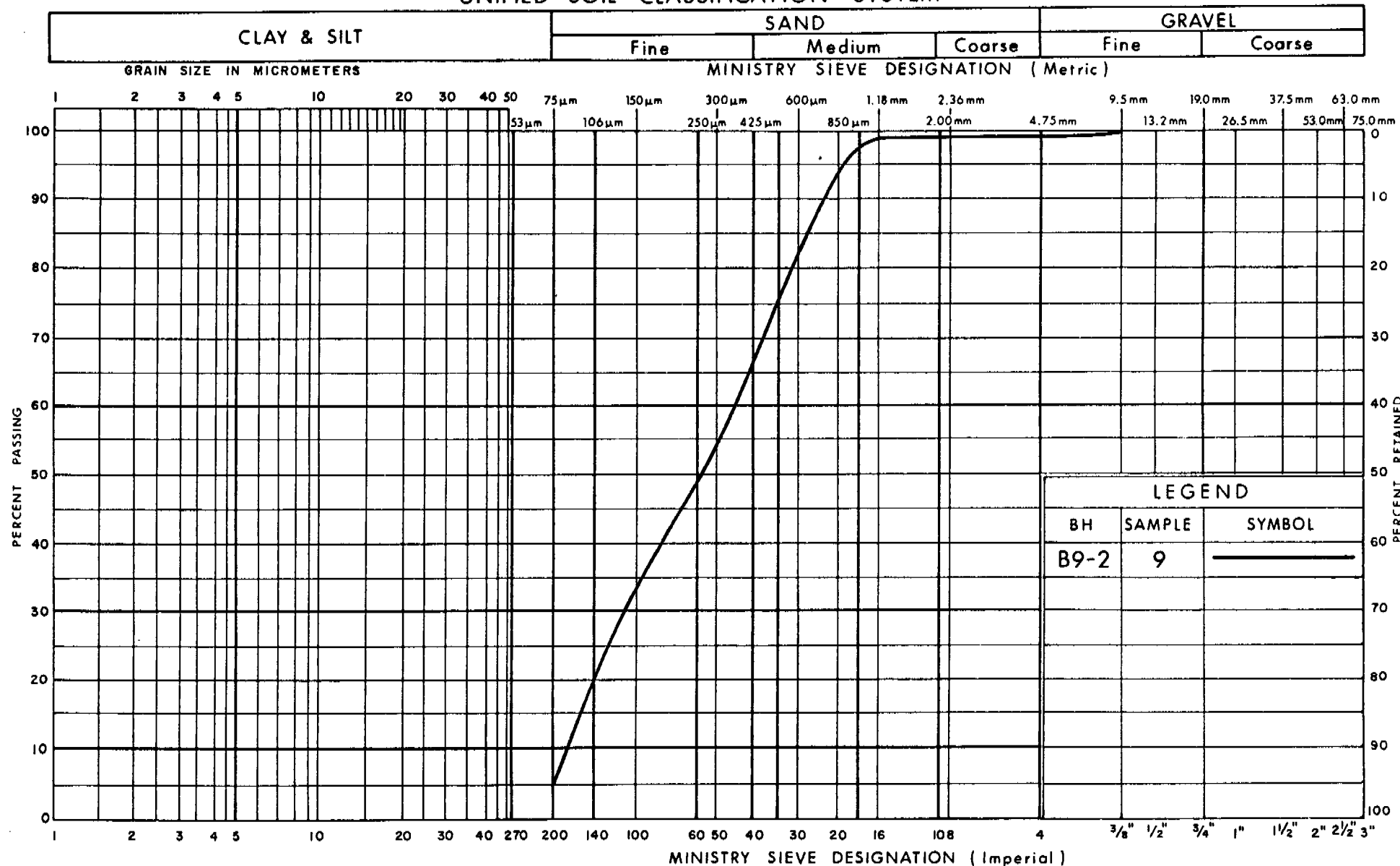
+ 3, x 3: Numbers refer to
Sensitivity

20
15 5
10 (%) 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 3232H

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 (RQD), FOR MODIFIED RECOVERY, IS:

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

JOINTING AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

MECHANICAL PROPERTIES OF SOIL

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

STRESS AND STRAIN

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

PHYSICAL PROPERTIES OF SOIL

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

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**FOUNDATION DESIGN REPORT
PROPOSED RETAINING WALL FOR RAMP (FGGE) E – (BROWN'S LINE) S
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5. DISCUSSION AND RECOMMENDATIONS

We understand that part of the existing ramp (FGGE) E-(Brown's Line) S will be demolished and replaced with a new ramp to the north of the existing location. The new ramp will incorporate retaining walls. The west leg of the ramp will be constructed in cut and the height of the retaining wall will vary from 1.4 to 3.7 m. The east leg of the ramp will be in fill and the height of the retaining wall, which will retain the ramp, will be maximum 6.5 m and will eventually meet the existing grade.

At Boreholes B9-1, B9-3 and B9-4, the existing ground and road surface elevation ranges from 106.4 to 110.7 m and in these boreholes the surface of the shale bedrock was encountered, immediately below the surficial topsoil, pavement fill or shallow embankment related fill, at about 0.4 to 0.6 m below the ground surface or between Elevations 106.0 and 110.1 m.

At Borehole B9-2, drilled near top of the existing embankment, the existing ground surface is at Elevation 117.8 m, that is higher than the general bedrock elevation in the area. At this borehole location, a veneer of topsoil is followed by embankment related fill to a depth of about 5.5 m below the existing ground surface or to Elevation 112.3 m. The natural overburden consists of a shallow layer of granular sand deposit over a till/shale complex transition zone to the shale bedrock. The surface of the shale bedrock was encountered at approximate elevation of 110.2 m.

5.1 FOUNDATIONS

The boreholes show that the proposed retaining wall structure can be supported on conventional strip footing foundations founded on shale bedrock at Boreholes B9-1, B9-3 and B9-4. At Borehole B9-2, the footings should be lowered to the undisturbed native very dense sand, present below the fill materials. The

recommended founding elevations and soil bearing resistances at the borehole locations are presented in the following Table. Footings founded in sand 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 Material
		Depth Below Existing Ground	Elevation (m)	At S.L.S.	At U.L.S.*	
9.1	110.7	0.6	110.1	**	1000	Shale Bedrock
9-2	117.8	5.5	112.3	400	800	Sand
9-3	107.7	0.6	107.1	**	1000	Shale Bedrock
9-4	106.4	0.6	105.8	**	1000	Shale Bedrock

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

** The design of foundations will not be governed by settlements, since the pressures required to produce detrimental settlements will be greater than factored capacity at U.L.S. Total and differential settlements will not exceed 12 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 18 mm, respectively. This can be achieved provided that the founding subgrade is undisturbed during the construction.

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.

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.

The unfactored horizontal resistance against sliding between concrete and approved undisturbed, competent, native founding sand can be calculated using a friction angle of 28 degrees.

Based on the subgrade conditions and the height of the retaining wall, global stability is not considered a problem.

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.

The lateral earth pressure, P_h may be computed using the equivalent fluid pressures presented in Section 6-7.4 of the OHBDC, or employing the following expression based on unfactored earth pressure distribution:

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

Where	=	
K	=	Earth pressure coefficient, use values from table below
γ	=	unit weight of soil, = 21.2 kN/m ³ for Granular 'B' = 22.8 kN/m ³ for Granular 'A'
H	=	Depth below top of wall, m
Q	=	unit surcharge pressure

TABLE 2

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

The above parameters are based on level (horizontal) ground behind the retaining walls. Allowance should be made for traffic loads.

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

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.

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)

At Borehole B9-2 location, the RSS will be founded in the lower portion of the compact fill. This area should be thoroughly proofrolled to detect any loose areas in the fill material. All loose areas detected should be sub-excavated and replaced by compacted engineered fill. Should any buried topsoil be present within 0.9 m from the base of the RSS, it should be sub-excavated and replaced by engineered fill compacted to not less than 98% of Standard Proctor Maximum Dry Density (SPMDD). In order to limit the total deflections within 25 mm, it is further recommended that the fill in the upper 0.5 m below the base of RSS be engineered to not less than 98% SPMDD, using suitable sub-excavated subsoil.

5.4 CONSTRUCTION COMMENTS

Water levels in the piezometers installed in Boreholes B9-2 and B9-4 were recorded at Elevation 110.9 and 106.0 m, respectively. At Borehole B9-4, the groundwater seepage is within the shale bedrock and at Borehole B9-2, the groundwater is perched in the wet sand overlying the shale bedrock. The general excavations for the foundations will be above the measured water levels and therefore no major groundwater problems are anticipated. However, any seepage from a groundwater or perched water source can be handled by means of gravity drainage and where necessary, by pumping from open sumps. Should excavations

be extended below the water level in the sand deposit (Borehole B9-2), then more elaborate methods such as deep filtered sumps may be required.

Temporary excavations more than 1.2 m deep should be sloped no steeper than 1:1. Depending on conditions, however, flatter side slopes may be required in the sand deposit while 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. The shale is prone to weathering and disturbance and this recommendation applies to shale bedrock as well as the overburden soils. 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 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 75 mm (including the settlement of the embankment fill under its own weight), 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

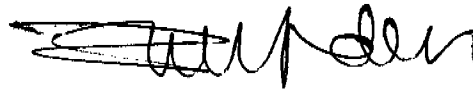
We recommend that once the details of the structure are finalized, our recommendations be reviewed for their specific applicability.

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

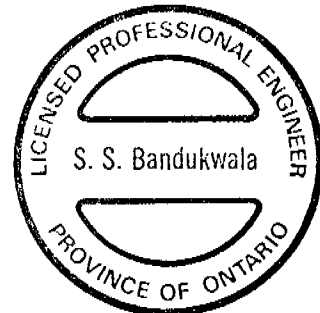
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APPENDIX D

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.