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DIST. CR REGION

W.P. No. 173-00-01

CONT. No.

W. O. No.

STR. SITE No. 37-1521

HWY. No. QEW

LOCATION BRIDGE NB-3 Hwy 427
RAMP N-W OVER FGGEE - SHERWAY GARDENS RD.

No OF PAGES -

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

GEOCRES No: 30M11-202

FOUNDATION INVESTIGATION REPORT
PROPOSED BRIDGE NB-3
HIGHWAY 427 RAMP N-W
OVER FGGE E-SHERWAY GARDENS ROAD
TORONTO, ONTARIO
W.P. 173-00-01
SITE 37-1521

Prepared For:

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FOUNDATION INVESTIGATION REPORT

FOUNDATION INVESTIGATION REPORT
PROPOSED BRIDGE NB-3
HIGHWAY 427 RAMP N-W
OVER RAMP (FGEE) E-SHERWAY GARDENS ROAD
TORONTO, ONTARIO
W.P. 173-00-01
SITE 37-1521

1. INTRODUCTION

As part of the Gardiner Expressway/Q.E.W./Hwy. 427/Brown's Line Interchange modifications, a new bridge is to be constructed which will carry the existing Highway 427 Ramp N-W over the proposed Ramp (FGEE) E-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 bridge 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. 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 7, 10 and 17, 2000 and consisted of drilling and sampling four boreholes (Boreholes B4-1 through B4-4) at the locations shown on Drawing No. 2. The depth of the boreholes ranged from 10.2 to 12.4 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.

Boreholes B4-2 and B4-4 were extended until refusal in the bedrock at depths of 11.7 and 10.2 m, respectively, probably on a limestone layer in the shale bedrock. In Boreholes B4-1 and B4-3 augering was stopped at depths of 9.4 and

9.1 m, respectively and the boreholes were extended by diamond drilling (rock coring), using NQ-size core barrel to depths of 12.1 and 12.4 m, respectively.

The soil samples and rock cores were shipped to our laboratory in Toronto for further examination and classification. A laboratory testing programme, consisting of natural moisture content, bulk unit weight and Atterberg Limit tests and grain-size analyses, was performed on selected, representative soil samples. The results of the laboratory tests are presented on the appropriate Borehole Log sheets and also in Appendix B.

Groundwater conditions in the open boreholes were observed during the drilling and at completion of each borehole. In addition, in Borehole B4-2, 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 B4-1 to B4-4). The locations of the boreholes are shown on the Borehole Locations and Profile Drawing No. 2 and are also indicated on the individual borehole log sheets. Cross sections of inferred subsurface stratigraphy are given in the same drawing.

The boreholes were drilled from the top of the existing ramp embankment and the ground surface elevations at the borehole locations ranged from 118.7 and 118.0 m on the north side (i.e. Boreholes B4-1 and 2) to 116.8 and 116.2 m on the south side (Boreholes B4-3 and 4). The boreholes showed the presence of embankment related fill to depths ranging between 5.9 and 7.5 m or to between Elevations 111.3 and 110.3 m. The fill generally consists of a mixture of clayey silt with frequent shale fragments and some sand. Underlying the fill, the natural overburden generally consists of brown clayey silt till with some shale fragments changing to grey till with frequent shale fragments with increasing depth. With further increase in depth the till attains a till/shale complex character, representing transition to the underlying shale bedrock. The surface of the grey shale bedrock at the borehole locations was contacted at approximately 7.9 to 10.3 m below the top of the embankment surface or between Elevations 108.9 and 108.0 m.

In a piezometer installed in Borehole B4-2, the groundwater level at the time of the investigation was measured at 7.9 m or at Elevation 110.8 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 B4-2 and B4-4 contacted approximately 400 and 500 mm of topsoil, respectively. A lower layer of topsoil was also contacted in Borehole B4-1 at 6.7 m depth, immediately below the embankment fill. This probably represents original topsoil which was present before the existing embankment was built.

4.2 PAVEMENT

Boreholes B4-1 and B4-3 were drilled from the paved shoulder surface and they showed the presence of 125 mm of asphaltic concrete, underlain by granular pavement fill extending to 0.9 m and 0.45 m below the ground surface, respectively. The grain size distribution of a sample from the granular fill is shown in Figure 1 in Appendix B.

4.3 EMBANKMENT FILL

The embankment fill was found to extend to depths ranging between 5.9 m (Boreholes B4-3 and B4-4) and 7.5 m (Borehole B4-1) below the ground surface at the borehole locations or to elevations ranging from 111.3 and 110.3 m.

The fill generally consists of clayey silt with frequent shale fragments and some sand. In most cases, it appeared to be similar to the indigenous till or till/shale complex materials. In some of the boreholes, trace amounts of topsoil and organics were found mixed with the fill. Some portions of the fill consisted of silty sand (e.g. Borehole B4-4 below about 3 m) with some silt zones/pockets. From the borehole results, the fill can be classified as a basically cohesive soil with some non-cohesive (granular) zones.

N-values recorded within the fill generally ranged from 10 to 50 blows/0.3 m indicating that some reasonable degree of compaction was applied when the embankment was built.

It should be pointed out that the thickness of topsoil and the thickness and composition of fill can vary in between and beyond borehole locations.

4.4 SANDY SILT

Underlying the fill and the original topsoil in Borehole B4-1, a 0.3 m thick sandy silt layer was contacted at a depth of 7.0 m or at Elevation 111.0 m. This basically fine grained granular material contains some organics and from a recorded N-value of 5 blows/0.3 m it is described as loose.

4.5 CLAYEY SILT TILL

Below the fill and the sandy silt deposit (Borehole B4-1), the boreholes contacted a clayey silt to silty clay till deposit at depths ranging from 5.9 m (Elevation 110.3 and 110.9 m at Boreholes B4-3 and BH4-4, respectively) to 7.5 m (Elevation 111.2 m) in Borehole B4-2. This unit consists of a heterogeneous, unsorted mixture of clayey silt and silty clay with some sand and gravel. The grain size distribution of samples from the upper zones of the deposit is shown in Figure 2 in Appendix B. The deposit also contains some shale fragments, the frequency of which increases with increasing depth. 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 upper zones of the deposit yielded the following index values:

Liquid Limit = 30-32%
Plastic Limit = 19-22%
Plasticity Index = 10-11%

These values are characteristic of clayey soils of low plasticity. The measured natural moisture contents generally range from 14 to 19%. The fact that these values are generally at or below the measured plastic limit values indicate that the material is over consolidated.

The till is a cohesive material.

N-values recorded in the upper, brown coloured zones of the deposit range from 31 to 54 blows/0.3 m. Based on these values together with a visual and tactile examination of the recovered soil samples, the consistency of the material is described as hard, except in the very upper zone, immediately below the sandy silt, in Borehole B4-1, where the material is considered very stiff.

In Boreholes B4-2 and B4-3, the till attains at greater depths a grey colour and contains more frequent shale and limestone fragments. From the recorded N-values and resistance to augering during drilling, the consistency of the grey till is described as hard.

4.6 TILL/SHALE COMPLEX

As mentioned before, with increasing depth, the colour of the clayey silt to silty clay till changes to grey and the material contains very frequent shale fragments. The lower portions of this grey till often resemble 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. Shale or limestone slabs/layers may remain. Excavation methods in the till, till/shale and in the underlying shale bedrock should take into account the possible presence of hard shale or limestone slabs/layers.

The till/shale complex is a basically cohesive material and based on the resistance to augering and the recorded N-values (which are in excess of 50 blows/0.3 m) its consistency is described as hard.

4.7 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)
B4-1	9.8	108.2
B4-2	10.3	108.4
B4-3	8.2	108.0
B4-4	7.9	108.9

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, siltstone and sandstone seams was noted in the cores obtained from the bedrock in Boreholes B4-2 and B4-3. They were generally 2 to 12 cm thick. In Boreholes B4-2 and B4-4, where the rock was not cored, the bedrock was penetrated by augering until refusal to augering was encountered at depths of 11.7 m (Elevation 107.0) and 10.2 m (Elevation 106.6), respectively, probably on the surface of a hard layer (e.g. limestone layer). From this, it can also be inferred that the bedrock at these two locations do not contain hard layers above these elevations.

4.8 GROUNDWATER CONDITIONS

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

All the boreholes were dry at the completion of augering, except for Borehole B4-4 in which a water level was recorded at 10.1 m (Elevation 106.7 m). These observations are, however, not considered to represent stabilized groundwater levels.

To enable us to monitor the groundwater level over a prolonged period of time without interference from surface water, a piezometer was installed in Borehole B4-2. The water level in the piezometer was monitored over a twelve day period when the water level was recorded at a depth of 7.9 m or at Elevation 110.8 m (i.e. immediately below the original ground surface level before the site was filled). This is believed to represent the groundwater level at the site.

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

REGISTERED PROFESSIONAL ENGINEER
Z. S. OZDEN
PROVINCE OF ONTARIO

Zuhtu Ozden, P.Eng.

trzip#hd

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

LICENSED PROFESSIONAL ENGINEER
S. A. AHMAD
PROVINCE OF ONTARIO

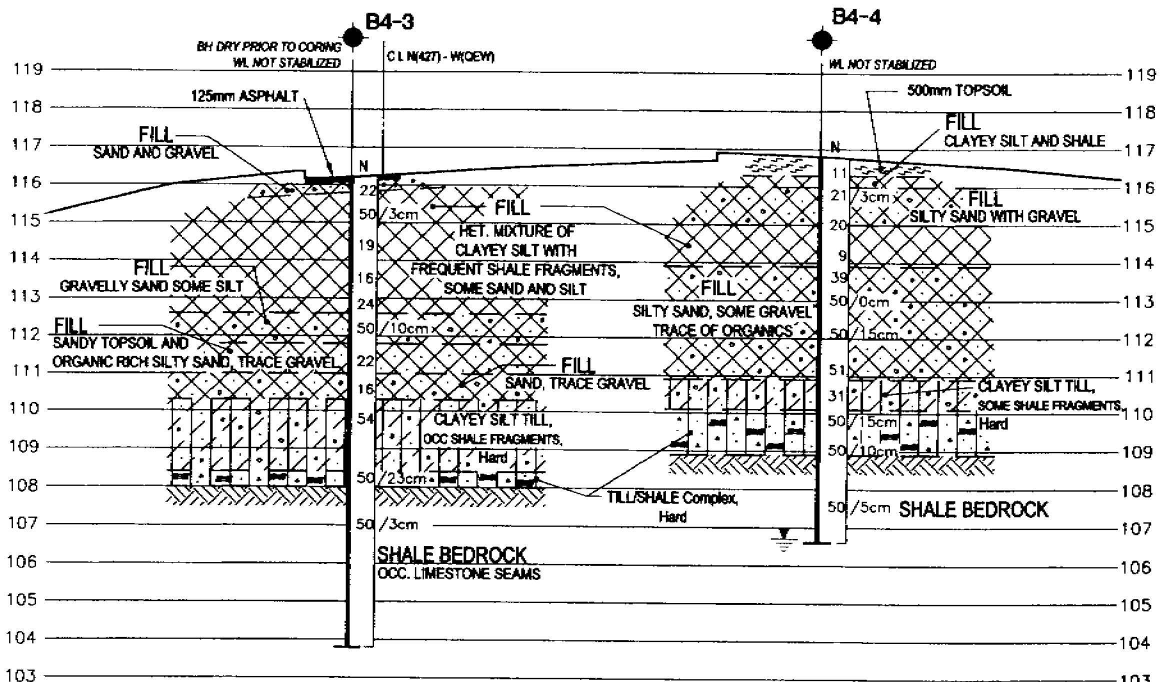
CONT No 2000-0080
WP No 173-00-01

BRIDGE NB-3
HWY 427 N-W Q.E.W. OVER
FGGE E-SHERWAY GARDENS ROAD
BORE HOLE LOCATIONS & SOIL STRATA

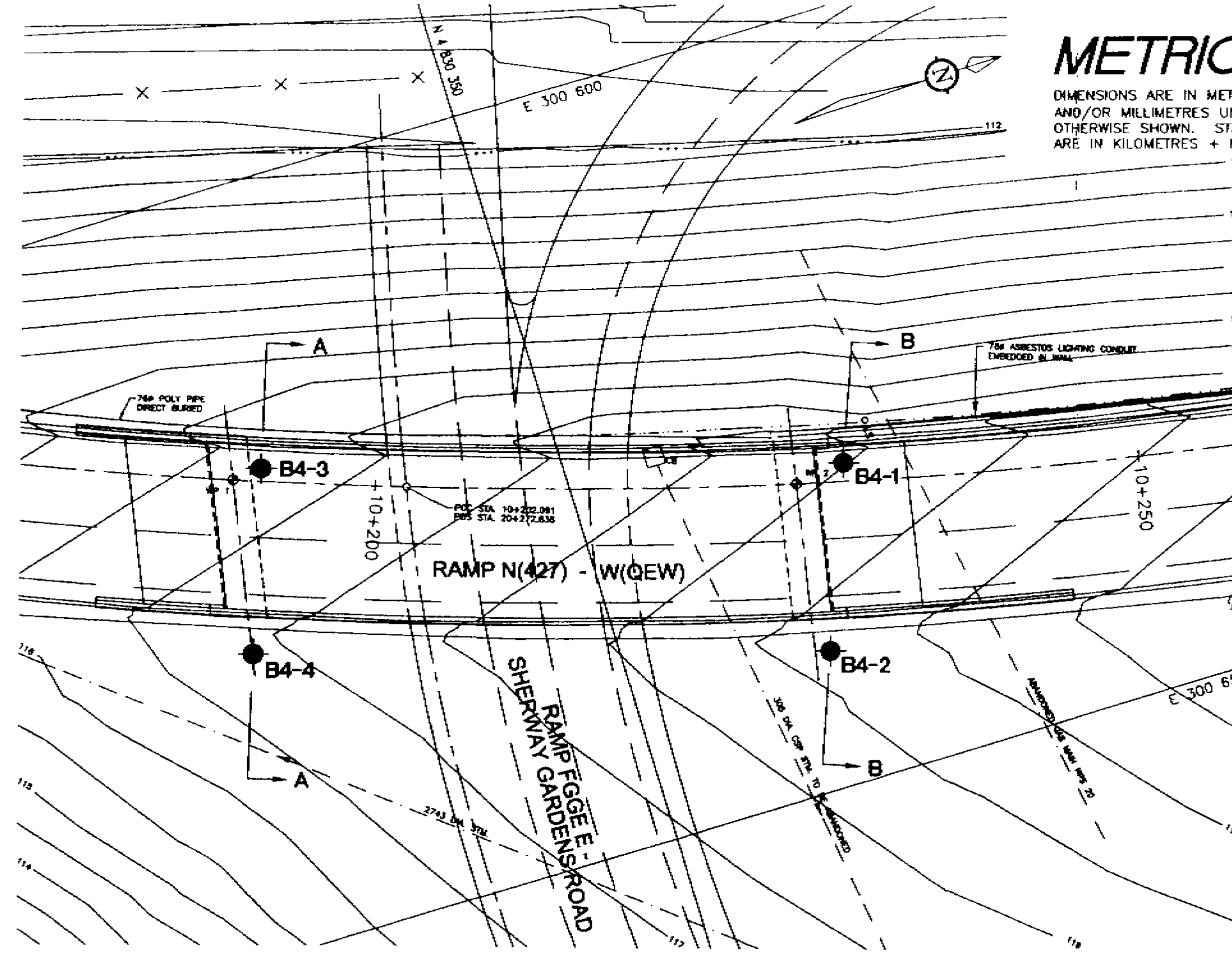
SHEET 524

METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES + METRES.

Shaheen & Peaker Limited

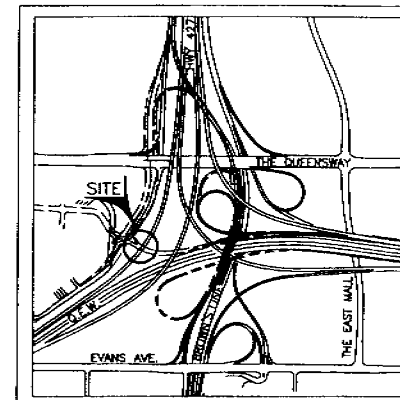


SECTION A-A



PLAN

SCALE 5m 0 5 10m



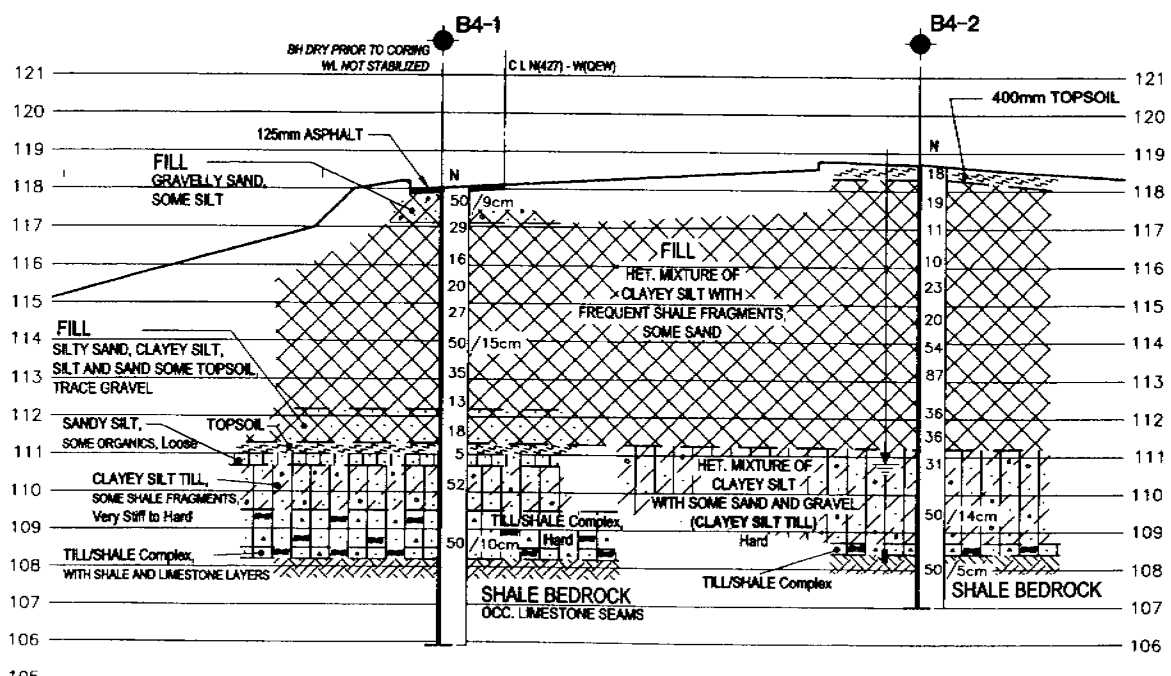
KEY PLAN

SCALE 0.2km 0.1 0 0.2 0.4km

LEGEND

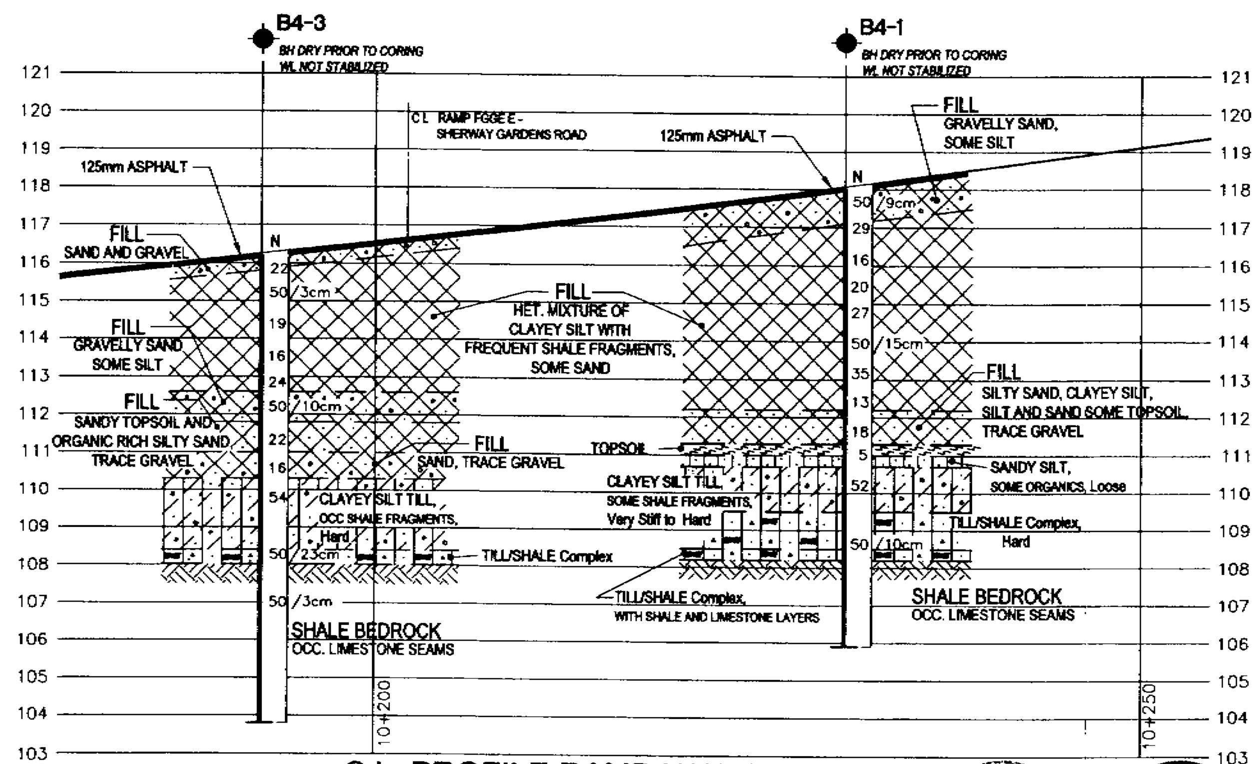
- Bore Hole
- Dynamic Cone Penetration Test (Cone)
- Bore Hole & Cone
- N — Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE — Blows/0.3m (60° Cone, 475 J/blow)
- W L at time of investigation Feb. 2000
- W L in Piezometer
- Piezometer

No	ELEVATION	CO-ORDINATES NORTH	EAST
B4-1	118.0	4 830 367.7	300 629.4
B4-2	118.7	4 830 363.3	300 641.1
B4-3	116.2	4 830 331.2	300 618.7
B4-4	116.8	4 830 327.2	300 630.3



SECTION B-B

SCALE FOR SECTIONS 2m 1 0 2 4m



C L PROFILE RAMP N(427) - W(Q.E.W.)

SCALES 5m 0 5 10m HOR
2m 1 0 2 4m VERT

=NOTE=
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section GC 2.01 of OPS Gen. Cond.

S. A. AMAN
Z. S. OZDEN
REGISTERED PROFESSIONAL ENGINEER PROVINCE OF ONTARIO

REV.	DATE	BY	DESCRIPTION

Geocres No

HWY No 427 & QEW			DIST
SUBM'D ZO	CHECKED ZO	DATE May, 2000	SITE 37-1521
DRAWN JTW	CHECKED JP	APPROVED	DWG 2

APPENDIX A

Borehole Log Sheets

RECORD OF BOREHOLE No B4-1

METRIC

W.P. 173-00-01 LOCATION Hwy 427 Ramp N-W 4 830 367.7 N; 300 629.4 E ORIGINATED BY M.J
DIST HWY 427 & QEW BOREHOLE TYPE Solid Stem Augers, NQ Rock Core COMPILED BY G.T
DATUM Geodetic DATE 07.02.00 CHECKED BY Z.O

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
118.0	Ground surface					
117.9	125 mm Asphalt					
0.1	FILL Gravelly Sand, some silt	1	SS	50/9		
117.1 / 0.9	FILL Heterogeneous mixture of Clayey Silt with frequent shale fragments some sand grey and brown moist to wet	2	SS	29		29 59 (12)
		3	SS	16		
		4	SS	20		
		5	SS	27		
		6	SS	50/15		
		7	SS	35		
		8	SS	13		
112.2 / 5.8	FILL: Silty Sand, Clayey Silt, Silt and Sand, some topsoil, trace gravel, moist to wet	9	SS	18		
111.3	TOPSOIL: Sandy, black, moist					
111.0	SANDY SILT: some organics, dark brown, loose, wet	10	SS	5	19.7	
110.7 / 7.3	CLAYEY SILT TILL: some shale fragments, brown, very stiff to hard	11	SS	52	20.3	Sand seam @ 7.5 m
109.5 / 8.5	TILL/SHALE Complex, grey, hard	12	SS	50/10		
108.5		~~13~~	~~RC~~			
108.2 / 9.8	TILL/SHALE Complex, with Shale and limestone layers	14	RC	100%		RQD=0%
	SHALE BEDROCK: occasional limestone seams, grey	15	NQ RC	Rec. 100%		RQD=47%
		16	NQ RC	Rec. 100%		RQD=62%
105.9 / 12.1	End of borehole Borehole dry prior to coring Water level not stabilized					'N'=50/9denotes 50 blows for 9 cm penetration

Columns also printed: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (117, 116, 115, 114, 113, 112, 111, 110, 109, 108, 107, 106); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60.

+³, ×³: Numbers refer to Sensitivity

15 ⊕ 5 (20 above, 10 below) (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No B4-2

METRIC

W.P. 173-00-01 LOCATION Hwy 427 Ramp N-W 4 830 363.3 N; 300 641.1 E ORIGINATED BY G.I

DIST HWY 427 & QEW BOREHOLE TYPE Solid Stem Augers COMPILED BY G.T

DATUM Geodetic DATE 07.02.00 10.02.00 CHECKED BY Z.O

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
118.7	Ground surface										
0.0 / 118.3	400 mm Topsoil	1	SS	18							
0.4	mostly shale				118						
	FILL Heterogeneous mixture of Clayey Silt with frequent shale fragments some sand grey and brown	2	SS	19							
		3	SS	11	117						
		4	SS	10	116						
		5	SS	23	115						
	some organics	6	SS	20							
		7	SS	54	114						
		8	SS	87	113						
	topsoil seam	9	SS	36	112						
	some organics	10	SS	36							
111.2 / 7.5	Heterogeneous mixture of Clayey Silt with some sand and gravel (CLAYEY SILT TILL) hard brown	11	SS	31	111		21.2	5	19	47	29
	----- grey frequent shale fragments	12	SS	50/14	110 / 109						
108.7	TILL/ SHALE complex										
108.4 / 10.3	SHALE BEDROCK grey	~~13~~	~~SS~~	~~50/5~~	108						
107.0 / 11.7	End of borehole Refusal to further augering at 11.7 m, probably on a limestone layer in the shale bedrock Piezometer installed to 10.5 m Water level in piezometer Feb. 10 - 7.5 m Feb. 12 - 7.7 m Feb. 20 - 7.8 m Feb. 22 - 7.9 m				107						

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100

PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

REMARKS & GRAIN SIZE DISTRIBUTION (%)

$+^3, \times^3$: Numbers refer to Sensitivity; 20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No B4-3

1 OF 1 **METRIC**

W.P. 173-00-01 LOCATION Hwy 427 Ramp N-W 4 830 331.2 N; 300 618.7 E ORIGINATED BY G.I

DIST HWY 427 & QEW BOREHOLE TYPE Solid Stem Augers, NQ Rock Core COMPILED BY G.T

DATUM Geodetic DATE 17.02.00 CHECKED BY Z.O

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
116.2	Ground surface						
116.1	125 mm Asphalt				116		
115.8 0.4	FILL: Sand and Gravel, brown	1	SS	22		○	
	FILL Heterogeneous mixture of Clayey Silt with frequent shale fragments, some sand, grey & brown, damp	2	SS	50/3	115		
		3	SS	19	114	○	
		4	SS	16		○	
112.6 3.6		5	SS	24	113	○	
111.8 4.4	FILL: Gravelly Sand, some silt, dark brown, damp	6	SS	50/10	112	○	
111.0 5.2	FILL: Sandy Topsoil & organic rich Silty Sand, trace gravel, dark brown, brown, black	7	SS	22	111	○	
110.3 5.9	FILL: Sand, trace gravel, dark brown	8	SS	16		○	
	CLAYEY SILT TILL occasional shale fragments, hard brown	9	SS	54	110	○	5 7 63 25
108.4	- - - - - grey frequent Shale fragments				109		
108.0 8.2	TILL/SHALE complex	10	SS	50/23	108	○	
	SHALE BEDROCK occasional limestone seams, grey	~~11~~	~~SS~~	~~50/3~~	107		
		12	NQ RC	Rec. 100%	106		RQD=51%
103.8 12.4		13	NQ RC	Rec. 100%	105 104		RQD=25%
	End of borehole Borehole dry prior to coring Water level not stabilized						

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L — WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

$+^3, \times^3$: Numbers refer to Sensitivity   15 ⊕ 5 (20 above, 10 below) (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No B4-4

METRIC

W.P. 173-00-01 LOCATION Hwy 427 Ramp N-W 4 830 327.2 N; 300 630.3 E ORIGINATED BY G.I

DIST ______ HWY 427 & QEW BOREHOLE TYPE Solid Stem Augers COMPILED BY G.T

DATUM Geodetic DATE 07.02.00 CHECKED BY Z.O

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
116.8	Ground surface						
0.0 / 116.3	500 mm Topsoil	1	SS	11			
116.0	FILL: Clayey Silt & Shale						
0.8 / 115.0	FILL Silty Sand with Gravel brown, moist	2	SS	21			
		3	SS	20			
1.8 / 113.9	FILL Heterogeneous mixture of Clayey Silt, Sand & Silt with frequent shale fragments grey, damp	4	SS	9			
2.9	FILL Silty Sand, some gravel trace of organics, brown, damp to wet	5	SS	39			
		~~6~~	~~SS~~	~~50/00~~			SS6 no recovery
		7	SS	50/15			
110.9		8	SS	51		20.6	
5.9 / 110.1	CLAYEY SILT TILL: some shale fragments, brown, hard	9	SS	31		20.7	
6.7 / 108.9	TILL/SHALE Complex grey, hard	10	SS	50/15		21.3	
		11	SS	50/10			
7.9 / 106.6	SHALE BEDROCK grey	~~12~~	~~SS~~	~~50/5~~	▼		
10.2	End of borehole Refusal to further augering at 10.2 m, probably on a limestone layer on the Shale bedrock Water level at 10.1 m on completion (not stabilized)						

Elevation scale: 116, 115, 114, 113, 112, 111, 110, 109, 108, 107

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

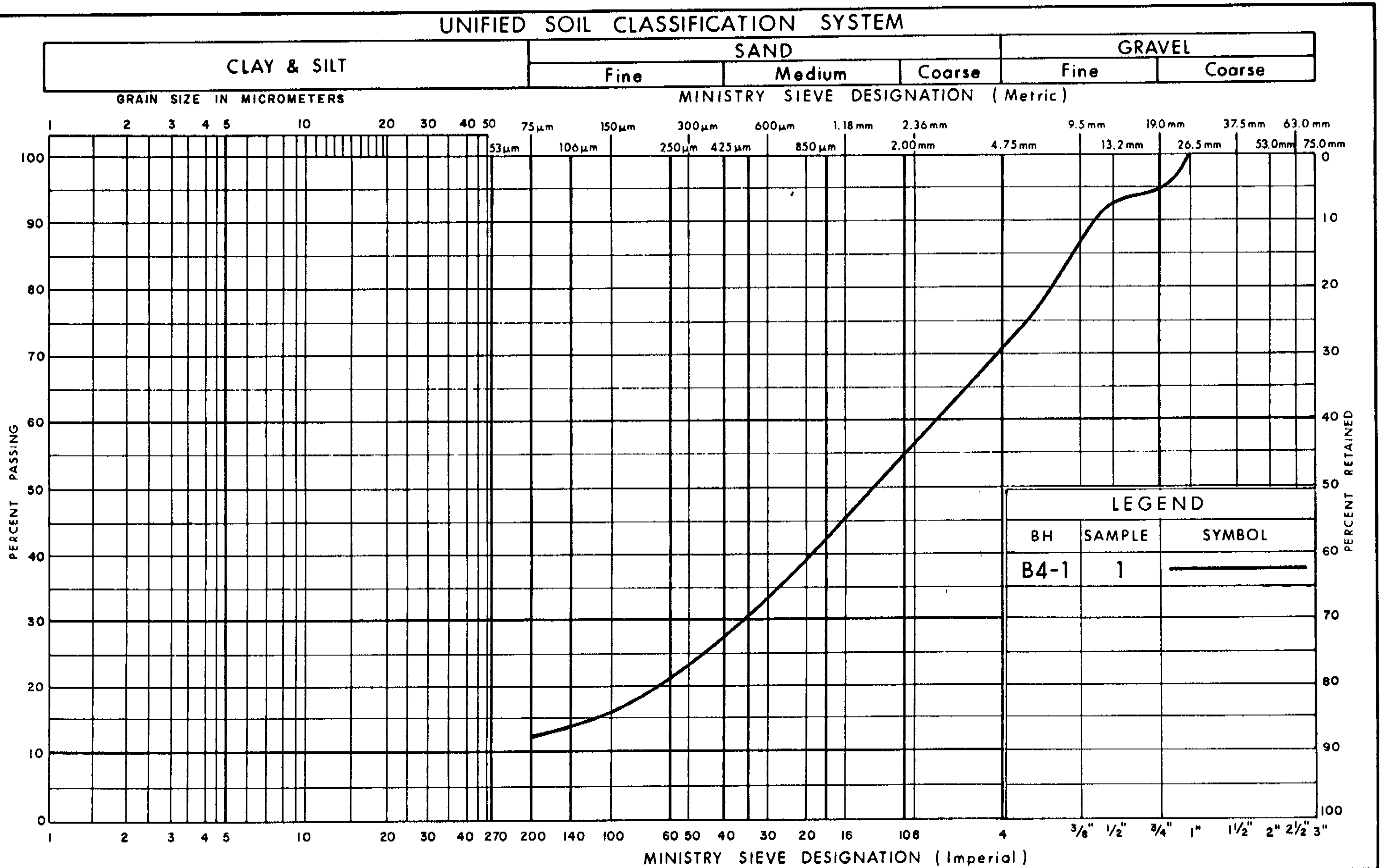
WATER CONTENT (%): 20 40 60

$+^3, \times^3$: Numbers refer to Sensitivity   15 ⊕ 5 (20 above, 10 below) (%) STRAIN AT FAILURE

APPENDIX B

Laboratory Test Results

78 12 M



Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION

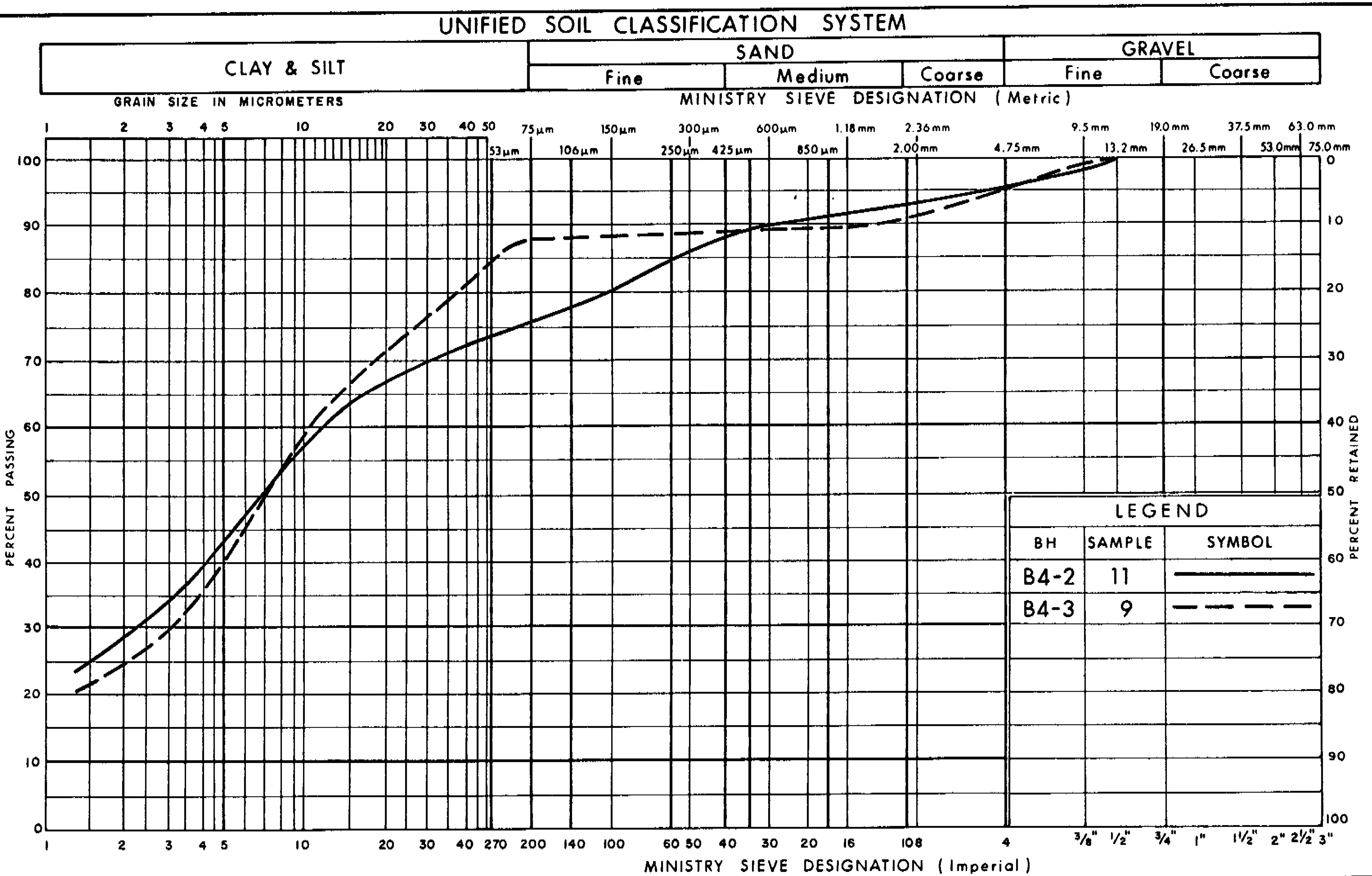
GRAVELLY SAND, SOME SILT (GRANULAR FILL)

FIG No 1

W P 173-00-01

SP 3232A

78 12 M



Ministry of Transportation
Ontario

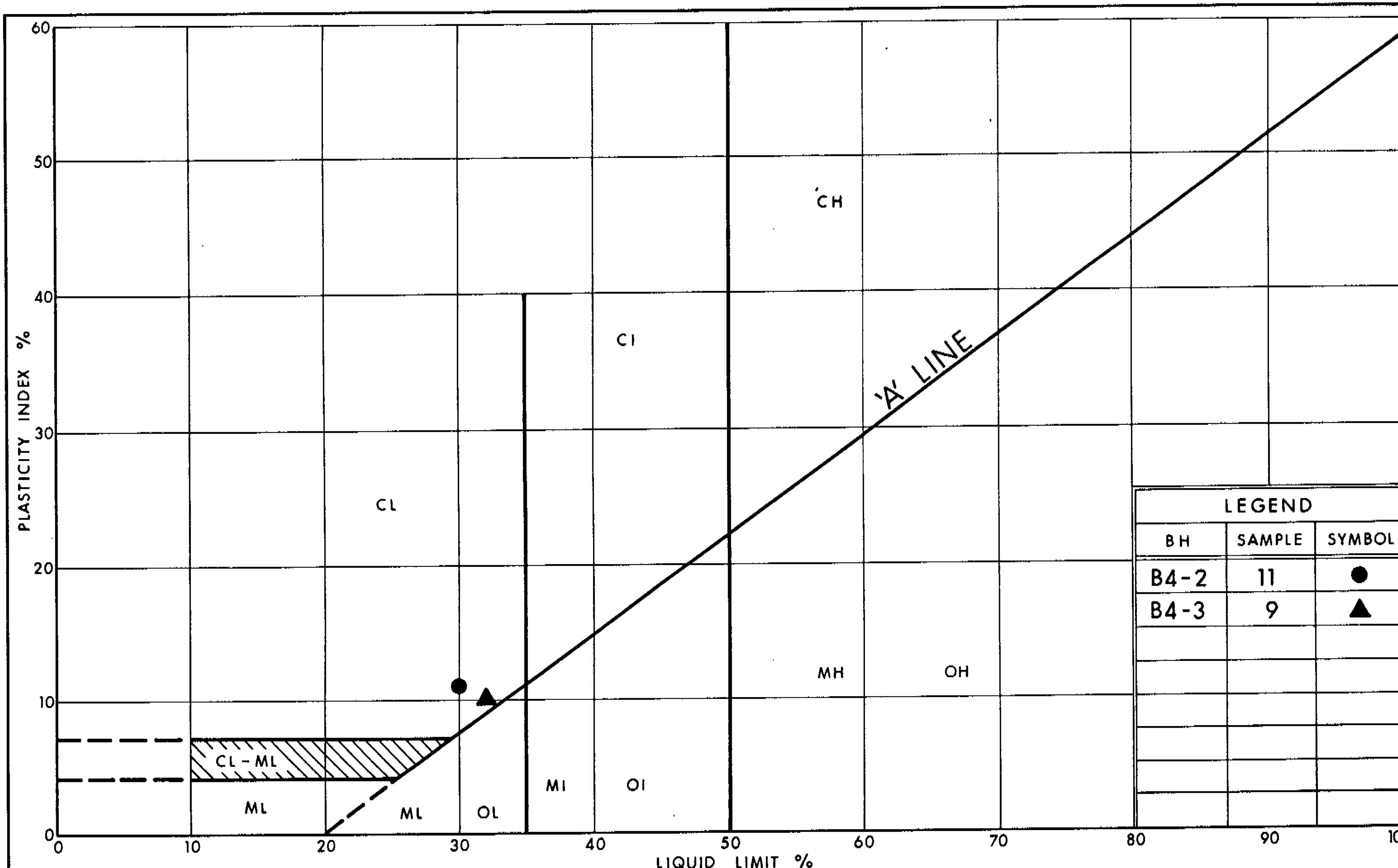
GRAIN SIZE DISTRIBUTION
CLAYEY SILT TILL

FIG No 2

W P 173-00-01

SP 3232A

Oct 75, FF-S-21



Ministry of Transportation
Ontario

PLASTICITY CHART
CLAYEY SILT TILL

FIG No 3
W P 173-00-01
SP 3232A

APPENDIX C

Core Logs and Photographs

CORE LOG

BH NO B4-1

PROJECT	ORIENTATION	ELEVATION (m)	DATUM	PROJECT NO.
Foundation Investigation	Vertical	118.0	Geodetic	SP3232
LOCATION	**DATE STARTED**	**COMPLETED**	**LOGGED BY**	**DRAWING NO.**
Q.E.W./Hwy 427/Brown's Line Interchange Modification	02/07/00	02/07/00	E.P.	
CLIENT	**DRILLER**	**DRILL TYPE**	**CORE BARREL**	**SHEET**
	Groundworks	CME 75	NQ	1 of 1

ELEV. (m)	DEPTH (m)	SYMBOL	GENERAL DESCRIPTION	JOINT CHARACTERISTICS: No. OF SETS	JOINT TYPE	ORIENTATION	SPACING	ROUGHNESS	FILLING	APERATURE (mm)	WEATHERING	STRENGTH	FRACTURE FREQUENCY	RUN No.	RECOVERY %	RQD	WATER RECOVERY %	WATER COLOUR
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
118.0			OVERBURDEN: see soils log for description															
108.9			**CLAYEY SILT TILL**											1	100	0	100	gr
														2	100	0	100	gr
108.2			**GEORGIAN BAY FORMATION:** Shale with Interbedded Limestone and Sandstone Shale (91%) thinly bedded or laminated, dark grey, slightly weathered up to 10.54 m, low strength Sandstone and Siltstone (9%) stratified, brownish grey to medium grey, unweathered, moderate strength Discontinuities: bedding joints are at close to very close intervals, the maximum thickness of sand/siltstone layers was about 120 mm, joint surfaces are smooth planar to rough planar 5 mm rubble seam noted at 10.57 m	1	B	F	C VC	SP RP	T R	0 5				3	100	47	100	gr
														4	100	62	100	grey
105.9			End of Borehole															

Depth scale (m): 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19

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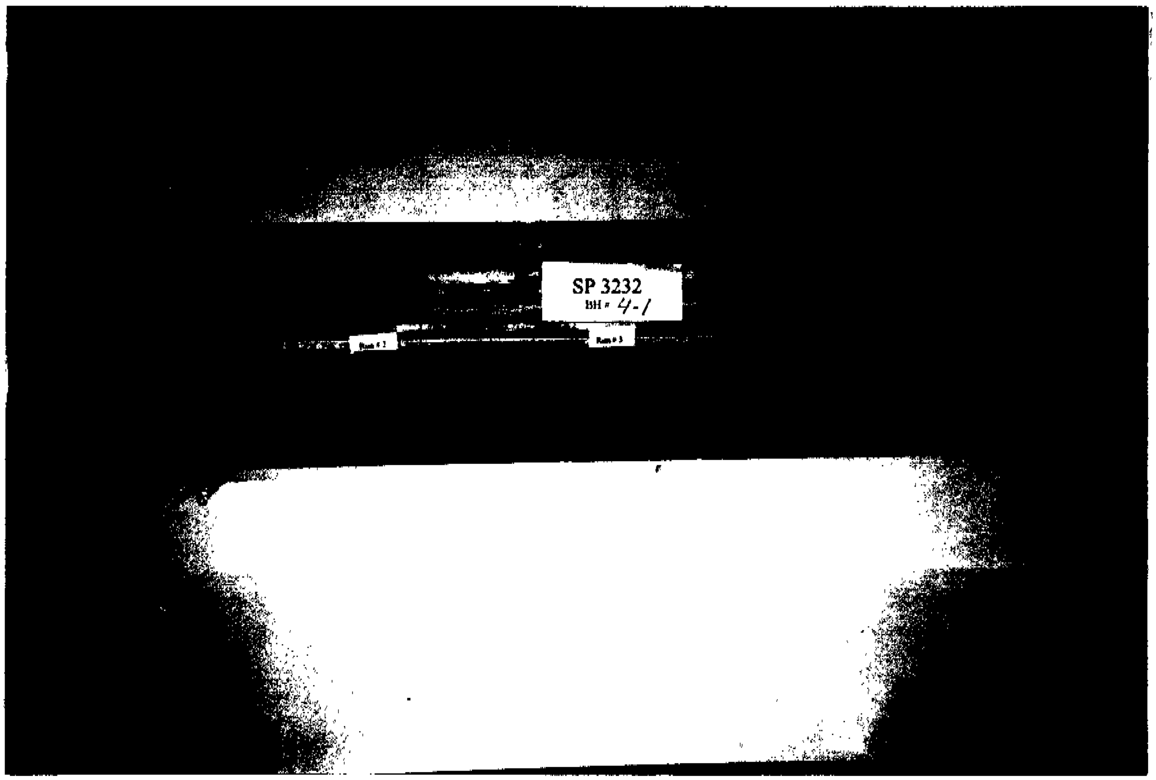
Consulting Geo-Environmental Engineers



Photograph of Core
Borehole B4-1
Elev. 108.9 m – 105.9 m

CORE LOG

BH NO B4-3

PROJECT	ORIENTATION	ELEVATION (m)	DATUM	PROJECT NO.
Foundation Investigation	Vertical	116.2	Geodetic	SP3232
LOCATION	**DATE STARTED**	**COMPLETED**	**LOGGED BY**	**DRAWING NO.**
Q.E.W./Hwy 427/Brown's Line Interchange Modification	02/17/00	02/17/00	E.P.	
CLIENT	**DRILLER**	**DRILL TYPE**	**CORE BARREL**	**SHEET**
	Groundworks	CME 75	NQ	1 of 1

ELEV. (m)	DEPTH (m)	SYMBOL	GENERAL DESCRIPTION	No. OF SETS	JOINT TYPE	ORIENTATION	SPACING	ROUGHNESS	FILLING	APERATURE (mm)	WEATHERING	STRENGTH	FRACTURE FREQUENCY	RUN No.	RECOVERY %	RQD	WATER RECOVERY %	WATER COLOUR
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
116.2			**OVERBURDEN**: see soils log for description															
108.0	8		**SHALE**: sampled with split-barrel sampler; see soils log for description															
107.1	9		**GEORGIAN BAY FORMATION:** Shale with Interbedded Limestone and Sandstone Shale (82%) thinly bedded or laminated, dark grey, slightly to moderately weathered up to 11.9 m, low strength Limestone (8%) fine grained, fossiliferous, shaley in some parts, unweathered, high strength Sandstone and Siltstone (10%) stratified, brownish grey to medium grey, unweathered, moderate strength Discontinuities: bedding joints are at close to very close intervals, the maximum thickness of limestone or sand/siltstone layers was about 90 mm, joint surfaces are smooth planar to rough planar, occasional vertical joints Rubble seams noted at 9.3 m (5 mm), 11.89 m (5 mm), 11.98 m (20 mm)	2	B C	F V	C VC	SP RP	T R	0 25				1	98	51	100	grey
									R R	5 20				2	96	25	100	grey
103.8			End of Borehole															

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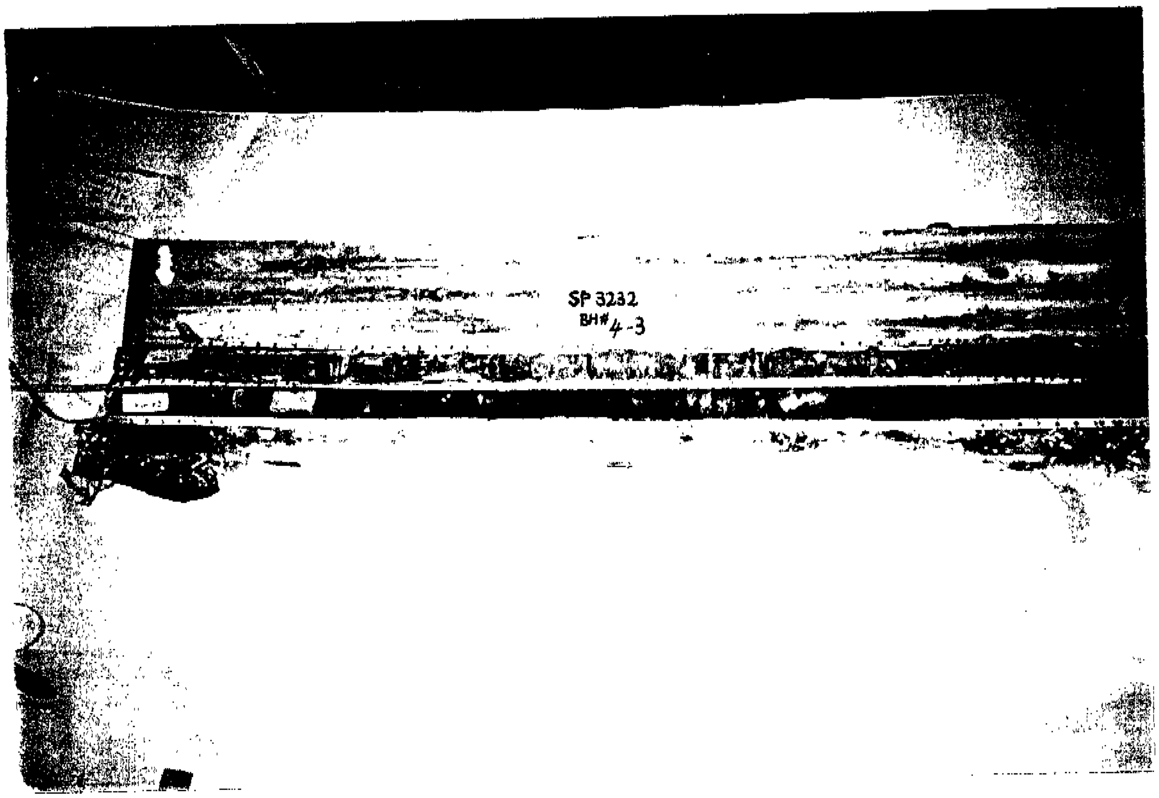

Photograph of Core
Borehole B4-3
Elev. 107.1 m – 103.8 m

EXPLANATORY SHEET TO CORE LOG

Column No.	Description
1	Elevation of geotechnical boundary.
2	Depth of geotechnical boundary in borehole.
3	Geological symbol for rock or soil material.
4	General description of geotechnical unit - qualitative description including rock type(s), percentage rock types, frequency and sizes of interbeds, colour, texture, weathering, strength, general joint spacing.
5-11	Joint (discontinuity) characteristics
5	Number of joint sets: a rock mass can be intersected by a number of joint sets of varying orientations.
6	Joint type: B = Bedding Joint; C = Cross Joint; F = Fault; S = Shear Plane
7	Orientation: only variations in dip can be identified in core; dip direction is obtained from field mapping or oriented core.

F	= Flat	= 0 - 20°
D	= Dipping	= 20 - 50°
V	= Vertical	= 50 - 90°

8 Joint spacing: this is an approximate measure of spacing between joints in specific joint sets

VW	= Very Wide	= 3 m
W	= Wide	= 1 - 3 m
M	= Moderate	= 30 cm - 1 m
C	= Close	= 5 - 30 cm
VC	= Very Close	= 5 cm

9 Roughness:

RU	= Rough Undulating
RP	= Rough Planar
SU	= Smooth Undulating
SP	= Smooth Planar
LU	= Slickensided Undulating
LP	= Slickensided Planar

10 Filling:

		Approx. ϕ_r
T	= Tight, hard, non-softening	-
O	= Oxidation surface staining only	25 - 35
SA	= Slightly altered; clay-free	25 - 30
S	= Sandy particles, clay-free	25 - 30
Si	= Sandy and silty, minor clay	20 - 25
NC	= Non softening clays (5 mm)	16 - 24
SO	= Softening clays (5 mm)	12 - 16
SC	= Swelling clay fillings (5 mm)	6 - 12

11 Aperture: estimated sizes of joint opening

12 Degree of weathering of rock material:

Unweathered	= no signs of discolouration or oxidation
Slightly weathered	= partial discolouration; fractures (joints) typically oxidized
Moderately weathered	= total discolouration
Highly weathered	= total discolouration; typically friable & pitted
Completely weathered	= resembles a soil; rock structure usually preserved

Column No.	Description

13 Strength of rock material:

		Approx. Uniaxial Compressive Strength
Very high strength	= specimen can only be chipped by geological hammer	200 MPa
High strength	= specimen requires a number of blows of geological hammer to fracture it; cannot be scraped with pocket knife	50 - 200 MPa
Medium strength	= specimen can be fractured by single firm blow of geological hammer; can be scraped with pocket knife, not peeled	15 - 50 MPa
Low strength	= shallow indentations made by firm blow with point of geological hammer; can be peeled by pocket knife with difficulty	4 - 15 MPa
Very low strength	= crumbles under firm blow with point of geological hammer; can be peeled by pocket knife	1 - 4 MPa

14 Fracture Frequency: Number of natural joints occurring over a metre length of core. All natural joints are counted irrespective of the number of joint sets.

Fracture frequency		Joint spacing	
0.3/m	=	Very wide	= 3 m
0.3 - 1/m	=	Wide	= 1 - 3 m
1 - 3/m	=	Moderate	= 30 cm - 1 m
3 - 20/m	=	Close	= 5 - 30 cm
20/m	=	Very close	= 5 cm

15 Run Number and Core Recovery:

(i) Drill run number;

(ii) Core Recovery is the total length of core pieces, irrespective of their individual lengths, obtianed in a core run and expressed as a percentage of the length of that core run.

16 Rock Quality Designation (RQD): The total length of those pieces of sound core which are 10 cm or greater in length in a core run expressed as a percentage of the total length of that core run. Sound pieces of rock are those pieces separated by natural breaks and not machine breaks or subsequent artificial breaks.

RQD	Rock Mass Classification (After Deere)
0 - 25 %	very poor
25 - 50 %	poor
50 - 75 %	fair
75 - 90 %	good
90 - 100 %	excellent

17 Core and Casing Sizes: changes of core and casing sizes are indicated.

18 Water recovery, level and tests.

APPENDIX D

Explanation of Terms Used in Report

EXPLANATION OF TERMS USED IN REPORT

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

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

N (BLOWS/0.3 m)	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

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

MECHANICAL PROPERTIES OF SOIL

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

PHYSICAL PROPERTIES OF SOIL

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

FOUNDATION DESIGN REPORT
PROPOSED BRIDGE NB-3
HIGHWAY 427 RAMP N-W
OVER FGGE E-SHERWAY GARDENS ROAD
TORONTO, ONTARIO
W.P. 173-00-01
SITE 37-1521

Prepared For:

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June 8, 2000

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FOUNDATION DESIGN REPORT

FOUNDATION DESIGN REPORT
PROPOSED BRIDGE NB-3
HIGHWAY 427 RAMP N-W
OVER RAMP (FGEE) E-SHERWAY GARDENS ROAD
TORONTO, ONTARIO
W.P. 173-00-01
SITE 37-1521

5. DISCUSSION AND RECOMMENDATIONS

The proposed bridge will carry the existing Highway 427 N-W Ramp over the proposed Ramp (FGEE) E-Sherway Gardens Road. It will be an approximately 12 m wide, 38 m long, single span structure. The existing N-W Ramp is located on a fill embankment and, at the proposed bridge location, the embankment elevation ranges from about 116.5 m on the south abutment location to 118 m at the north abutment area. The finished grade of Ramp (FGEE) E-Sherway Gardens Road will be about 109.5 m and will therefore involve lowering the grade. The finished bridge elevation will range from about 116.5 m on the south to about 118 m at the north end. The construction of the bridge requires the construction of a detour ramp immediately east of the existing N-W Ramp. The existing ramp would then be cut into in order to build the new bridge, after the construction of the detour road.

In the four boreholes drilled at the site, fill was found to extend to Elevations ranging between about 110.3 m and 111.3 m and from this, it can be inferred that the approximate ground surface elevations before the site was developed generally ranged between 110.5 and 111.5 m. The clayey silt till overburden gradually blends into the bedrock surface and therefore it is difficult to establish a very accurate definition of the bedrock surface but based on the drilling results, the bedrock surface elevation at the borehole locations ranges from about 109 to 108 m.

5.1 FOUNDATIONS

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

As the finished roadway grade beneath the bridge will be at Elevation 109.5 ± m, the founding grades for the abutments can be expected to be about 108.5 ± m. The following table presents the recommended soil and rock resistances at various depths and elevations.

Table 1

Borehole Location	Existing Ground Surface Elevation (m)	Recommended Footing Base (bottom) Depth Below Existing Ground Surface (m)	Recommended Footing Base (bottom) Elevation (m)	Factored Bearing Resistance at U.L.S.* (kPa)	Bearing Resistance at S.L.S. (kPa)	Subgrade Material
B4-1	118.0	8.0	110.0	800	400	Clayey silt till
		9.0	109.0	1000	500	Till/shale complex
		9.5	108.5	1000	600	Till/shale complex
North		10.0	108.0	1000	n/a**	Bedrock
abutment		10.5	107.5	2000	n/a**	Bedrock
B4-2	118.7	8.7	110.0	800	400	Clayey silt till
		9.7	109.0	1000	500	Clayey silt till
North		10.2	108.5	1000	n/a**	Till/shale complex
abutment		10.7	108.0	2000	n/a**	Bedrock
B4-3	116.2	6.2	110.0	800	400	Clayey silt till
		7.7	108.5	1000	500	Clayey silt till
South		8.7	107.5	1000	n/a**	Bedrock
abutment		9.2	107.0	2000	n/a**	Bedrock
B4-4	116.8	6.3	110.5	800	400	Clayey silt till
		6.8	110.0	1000	500	Till/shale complex
South		8.3	108.5	1000	n/a**	Bedrock
abutment		8.8	108.0	2000	n/a**	Bedrock

*Incorporating a resistance factor of 0.5 as per Ontario Highway Bridge Design Code (OHBDC), 3rd Edition.
**S.L.S. will not govern.

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

Where the surface of the rock is lower than the proposed founding level or where the rock is shattered and/or highly weathered, the unsuitable materials should be removed and replaced with mass concrete.

Under 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. We recommend that the minimum dowel length below the underside of the footing should be 2 m.

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

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 quoted value 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 pull-out assuming a 60 degree apex cone/wedge and the anchor group resistance should also be checked.

For frost protection, the footings should have a permanent earth cover of at least 1.2 m, or equivalent artificial insulation, including footings founded on shale bedrock.

The use of integral abutments would also be possible, in which case the abutments would be supported on driven steel H-piles (e.g. HP310 x 310). In this instance, however, augering into bedrock will likely be required to provide sufficient fixity, along with the required upper 3 m flex zones. We will be pleased to provide details of this approach if you want us to do so.

The abutments can also be founded on engineered fill consisting of Granular A type material compacted in thin layers to at least 100% of the material's Standard Proctor Maximum Dry Density. Prior to the placement of the engineered fill, the upper variable, weak and otherwise unsuitable zones of the existing subgrade should be stripped to the surface of the competent stratum. The Granular A pad supporting the spread footing foundations should be at least 1.5 m thick. The suggested highest founding subgrade elevations at the borehole locations are given in Table 2.

Table 2
Stripping Depths for Engineered Fill

General Area	Borehole No.	Existing Ground Elevation (m)	Recommended Stripping Depth (m)	Recommended Stripping Elevation (m)	Anticipated Soil Type After Stripping
North Abutment	BH4-1	118.0	7.6	110.4	Clayey silt till
	BH4-2	118.7	7.7	111.0	Clayey silt till
North Abutment	BH4-3	116.2	6.0	110.2	Clayey silt till
	BH4-4	116.8	6.2	110.6	Clayey silt till

The construction of the Granular A pad and of the earth fill should meet the minimum requirements as per Ontario Ministry of Transportation, as shown on Figure No. E1 in Appendix E. The Granular A pad supporting the spread footing foundations should be at least 1.5 m thick.

For footings satisfying these requirements a factored vertical bearing resistance at U.L.S. equal to 900 kPa and a bearing resistance at S.L.S. of 350 kPa can be utilized.

The unfactored horizontal resistance against sliding between concrete and properly compacted Granular A fill can be calculated using an angle of friction of 35 degrees.

For frost protection, the footings should have a permanent earth cover of at least 1.2 m.

5.2 LATERAL EARTH PRESSURES

Backfill behind abutments and retaining walls should consist of non-frost susceptible, free draining granular materials in accordance with the Ontario Ministry of Transportation Standards.

Free-draining backfill materials (i.e. Granular A or Granular B) and the provision of drain pipes and weep holes, etc., should prevent hydrostatic pressure build-up. Computation of earth pressures should be in accordance with O.H.B.D.C. For design purposes, the following parameters (unfactored) can be used.

Compacted Granular 'A'
Unit Weight = 22 kN/m^3
Coefficient of Lateral Earth Pressures:
$K_a = 0.27$
$K_o = 0.43$

Compacted Granular 'B'
Unit Weight = 21 kN/m^3
Coefficient of Lateral Earth Pressures:
$K_a = 0.31$
$K_o = 0.47$

These values are based on the assumption that the backfill behind the retaining structure is free-draining and adequate drainage is provided. As well, it is assumed that the ground behind the retaining structure is level.

The earth pressure coefficient adopted will depend on whether the retaining structure is restrained or movements can be allowed such that the active state of earth pressure can develop. If the abutment is restrained and does not allow lateral yielding, then at rest pressures should be used as per Clause C6-7.1 of the O.H.B.D.C., 3rd Edition. The effect of compaction should also be taken into account in the selection of the appropriate earth pressure coefficients in accordance with Clause 6-7.4.3, O.H.B.D.C., 3rd Edition.

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

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

As an alternative to conventional retaining walls, MTO's Retained Soil System may be used. The following should be included in the Contract Documents:

- identify longitudinal extent in plan of the Retained Soil System
- identify in plan transverse space constraints (top of wall and bottom of wall)

- identify elevation of top of wall and bottom of wall
- include NSSP for Retained Soil Systems in Contract Documents

The Retained Soil System should be of high performance and high appearance.

5.3 RAMP SUPPORT FOR TEMPORARY ROADWAY PROTECTION

The construction of the new bridge will be started after the construction of the detour ramp to the east. To facilitate the construction of the bridge, an approximately 7 to 8 m deep vertical cut will have to be made along the western limits of the detour ramp at the proposed bridge location, in order to provide space for the new bridge. It is understood that the detour ramp (i.e. the vertical cut) will be supported during the construction of the bridge by means of soldier piles and conventional lagging, which will be removed with the detour after the construction of the bridge structure.

In this case, the soldier piles will be socketed into the bedrock, the surface of which was encountered at approximately Elevation 108.4 and 108.9 at Boreholes B4-2 and B4-4, respectively.

For the design, the following unfactored soil parameters can be used:

Recommended Unfactored Parameters for Temporary Shoring Design

Elevation (m)		Stratum	Ka	Ko	Kp	γ (kN/m³)
From	To					
118	110.0	Fill	0.5	0.55	3.0	20.0
110.9	108.4	Clayey Silt Till and Till Shale Complex	0.25	0.45	4.0	21.5
108.4		Shale bedrock	0.15	0.45	4.5	23.0

For the design of raker footings placed in the hard clayey silt till below about Elevation 110.9 m and extending at least 0.6 m into the general excavation level, the following soil resistance values can be used:

U.L.S. = 400 kPa
S.L.S. = 200 kPa

For calculating anchor resistance for tie back design, the bond resistance at U.L.S. can be taken as 70 kPa in the clayey silt till and the till/shale complex and 500 kPa in the bedrock and S.L.S. will not govern.

5.4 APPROACH EMBANKMENTS FOR DETOUR ROAD

A detour road will be constructed on the east side of the existing N-W ramp. A second ramp (Ramp FGGE-E, Browns Line-S) is located within the close proximity (to the east) of this ramp. Because of this, the entire space between these two ramps was more or less filled, when these two ramps were originally constructed. Therefore, in the vicinity of the proposed bridge, the construction of the detour ramp will involve minor adjustments only (i.e. less than 2 m fills or cuts).

For the construction of the ramp, all the surficial topsoil and otherwise unsuitable soils should be removed. Where the new embankment fill will abut into the existing ramp 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 the lifts not exceeding 300 mm before compaction and each lift should be uniformly compacted to at least 95% of the material's Standard Proctor Maximum Dry Density. The degree of compaction within the top 0.6 m of the fill (i.e. subgrade immediately beneath the granular subbase) should be increased to 98%. The selection, placement and compaction of the fill should be carried out under the supervision of a geotechnical engineer. The settlement of embankment fills prepared as described above should not exceed 50 mm, based on the borehole results. Surcharging or pre-loading is therefore considered not to be necessary.

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

Groundwater level was recorded at about Elevation 111 m, that is, below the anticipated excavation depths required for the construction of the temporary road ramp. Therefore, we do not anticipate major problems due to groundwater seepage during the stripping of the unsuitable soils from the top of the existing embankments and during backfilling. It is believed that any seepage from a perched water source and/or from surface water should be controllable by gravity drainage and where necessary by pumping from open sumps.

5.5 CONSTRUCTION COMMENTS

Groundwater table at the time of our investigation was encountered at Elevation 111 m and the strata below this elevation were found to be relatively impervious. For this reason, during construction major problems due to groundwater seepage are not anticipated. It is believed that where necessary groundwater seepage can be handled by means of gravity drainage and pumping from sumps.

Allowance should be made to place a 150 mm thick concrete mud mat in all footing excavations within about four hours of excavation to minimize disturbance. The shale is prone to weathering and disturbance and a similar 100 to 150 mm thick mud mat would be required on founding surfaces placed on shale.

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 excavations and shoring should be carried out in accordance with the safety regulations of the Province.

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 (including the zone between the bottom of the fill and the competent natural clayey silt till) the slopes may have to be locally flattened. In the competent clayey silt till and in the till/shale complex 1:1 side slopes should be stable for temporary excavations. In the underlying shale somewhat steeper side slopes may be permissible, as directed by the geotechnical engineer.

All slope faces should be protected against erosion.

As mentioned before, permanent cut faces can be maintained at 2H:1V slopes. But they should be inspected during the construction for possible local instabilities and, where necessary, remedial measures, such as gravel sheeting may be required.

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

5.6 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 F, are an integral part of this report.

Shaheen & Peaker Limited

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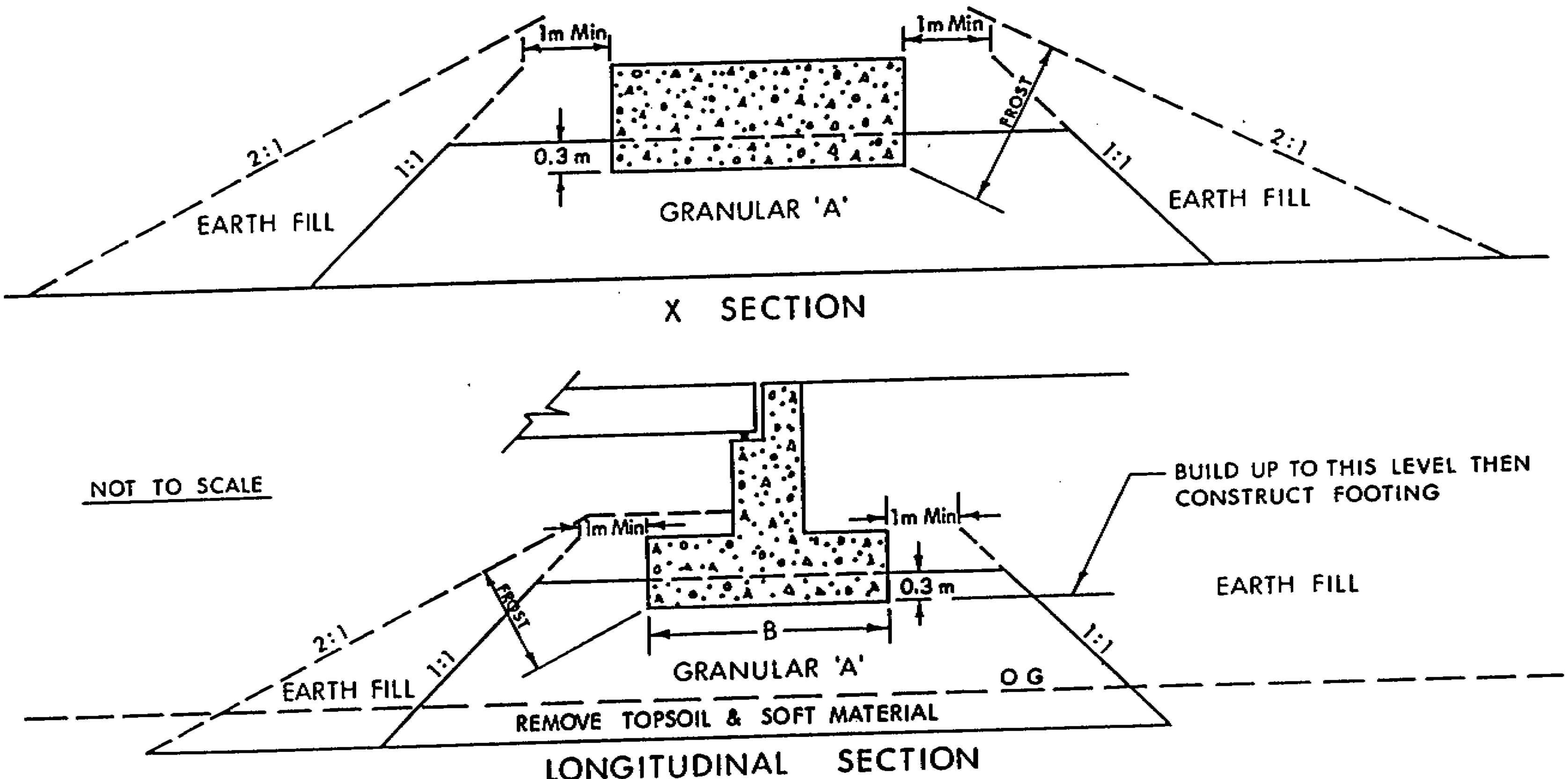
APPENDIX E
Abutment on Compacted Fill Showing Granular 'A' Core



NOTES:

1 - REMOVE TOPSOIL &/OR SOFT SUBSOIL UNDER AREA OF COMPACTED GRANULAR 'A' & EARTH FILL.

2 - PLACE GRANULAR 'A' & EARTH FILL TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO CURRENT M T O STANDARDS.

3 - CONSTRUCT CONCRETE FOOTING.

4 - PLACE REMAINDER OF GRANULAR 'A' & EARTH FILL AS REQUIRED.

Ministry of Transportation Ontario	ABUTMENT ON COMPACTED FILL SHOWING GRANULAR 'A' CORE	FIG No E-1 W P 173-00-01

APPENDIX F

Limitations of Report

LIMITATIONS OF REPORT

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

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

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

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