

**FOUNDATION INVESTIGATION REPORT  
PROPOSED CULVERT EXTENSION AT COBOURG CREEK  
(FORMERLY DYE WORKS CREEK)  
HIGHWAY 401 AT BURNHAM STREET  
NEW S-E RAMP  
COBOURG, ONTARIO  
G.W.P. 377-01-00**

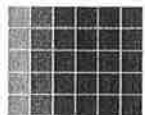
**Prepared For:**

**UMA ENGINEERING LIMITED**

**Prepared by:**

**SHAHEEN & PEAKER LIMITED**

**Project: SPT1078  
March 17, 2003**



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1

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**1. INTRODUCTION**

Part of the planned intersection improvements at the Highway 401/Burnham Street interchange requires the construction of a new S-E ramp and a new N-E loop ramp which will allow traffic from Burnham Street to enter the east bound lanes of Highway 401. The inclusion of these proposed ramps will require the widening of Highway 401 from four to six lanes. Hence, the existing Cobourg Creek culvert will require extensions at both the north and south sides of the highway.

An investigation was carried out in June 1957 by MTO for the existing culvert. The purpose of the present investigation was to obtain additional information at the site by means of boreholes.

Shaheen & Peaker Limited (S&P) was retained by UMA Engineering Limited to carry out a foundation investigation for the proposed extensions of this existing concrete arch culvert. This report presents the factual foundation investigation results for the extension of this culvert. The work was performed in accordance with Consultant Assignment Agreement No. 2005-A-000487.

The findings of the investigation are presented in this report.

**2. SITE DESCRIPTION AND GEOLOGY**

The site is located approximately 250 m east of the intersection of Highway 401 and Burnham Street (County Road 18) and 565 m west of the Highway 401/Ontario Street underpass, about 3 km north of the Town of Cobourg (south side of Highway 401) and the Township of Hamilton (north side of Highway 401), County of Northumberland.

Along Highway 401 the grade rises gently to the west by approximately 17 m over a distance of 645 m and reaches a high point of Elev. 108 m about 390 m west of the Burnham Street underpass structure. To the east the grade also gently rises by about 12 m to Elev. 103 m some 420 m east of the existing culvert. Cobourg Creek (formerly Dye Works Creek) generally flows northeast to southwest towards Lake Ontario some 4 km

south of Highway 401. Small shrubs and grass exist in the immediate vicinity of the inlet and outlet openings of the culvert. However, further to the north and south of the culvert large deciduous trees exist.

The existing culvert at Cobourg Creek is an open bottom 12.2 m wide concrete arch with a rise of 5.8 m and an approximate length of 65.5 m. It crosses under the existing four lanes of Highway 401.

Highway 401 has a median storm sewer system that outlets to a detention pond at the northeast quadrant of the Highway 401/Burnham St. interchange. Drainage along Burnham Street and the Interchange ramps are conveyed to Cobourg Creek via vegetated ditches.

The study area is located in the physiographic region known as the "Iroquois Plain." The plain consists of drumlins and sand plains (Ref: Chapman and Putnam, 1984).

The lowermost bedrock in the general area (i.e. Northumberland County) consists of Precambrian rock, with upper layers of limestone. These limestone layers are made up of the Trenton Group bedrock formations and were deposited during the Middle Ordovician Period, during the Paleozoic seas, some 480 million years ago.

Glacio-lacustrine lake plain deposits of silt and clay with gently rolling terrain characterize the soils of the area.

The majority of the interchange is located on Schomberg soils<sup>1</sup>. These soils are identified as part of the Gray Brown Podzolic Group, composed of silt, loam and clay. Characteristics of these soils include good drainage, irregular/moderately sloping topography and stone free.

At the interchange site the soil type is Smithfield, a silty clay loam of the Gray Brown Podzolic Group. Characteristics of this soil type are imperfect drainage, smooth to gently sloping topography, free of stones.

The third soil category is found to the northeast side of Cobourg Creek. Here the soils belong to the Percy type, a fine sandy loam of the Gray Brown Podzolic Group. The characteristics of this soil type are good drainage, smooth/ gently sloping topography free of stones.

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<sup>1</sup> Hoffman, D. W and C. J. Acton, Soils of Northumberland County, Report No. 42 of the Ontario Soil Survey, Ontario Ministry of Agriculture and Food Agriculture Canada, 1974.

### **3. INVESTIGATION PROCEDURES**

The fieldwork for this project was conducted on November 5 and November 6, 2002 and consisted of drilling and sampling three boreholes (Boreholes SP1, SP2 and SP3). The plan locations of the boreholes, along with the stratigraphic sections, are shown on Drawing No. 1.

The boreholes were advanced with solid stem continuous flight augers using a track mounted drilling rig owned and operated by Groundworks Drilling Inc., under the full time supervision of geotechnical personnel from S&P.

The depths of the boreholes ranged from 9.2 to 12.2 m. Sampling in the boreholes was conducted at frequent intervals of depth 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.

Auger cuttings were used to backfill the boreholes and the upper portion was then grouted and sealed using a cement/bentonite mixture.

The borehole locations were established in the field by our engineering staff, in relation to the existing features. The geodetic elevations and coordinates for Boreholes SP1 and SP3 were determined and provided to us by surveyors from J. D. Barnes Limited. The geodetic elevation and coordinates for Borehole SP2 were determined by S&P personnel referring to Stake 3003 whose coordinates and geodetic elevation were provided by surveyors from J. D. Barnes Limited.

Water level observations in the open boreholes were made during the drilling and at the completion of each borehole. Piezometers were also installed in Boreholes SP1 and SP2 to enable us to monitor groundwater levels over a prolonged period of time without interference from surface water. The water levels in these piezometers were measured during a subsequent site visit made on November 14, 2002.

During our visit on November 14, 2002 a steel probe was advanced adjacent to the inside concrete face of the existing culvert approximately 1.8 m from the southwest corner of the existing culvert. The purpose of probing was to confirm the elevation of the top of the existing footing (Elev. 296 ft) as indicated in the 1957 construction drawings for the existing culvert. The probe encountered refusal at Elevation 90.2 m (296 ft) probably on the top of the footing of the culvert.

The results of drilling, in-situ testing and water level measurements are summarized on the Record of Borehole Sheets in Appendix A. The Record of Borehole Sheets (M.T.O 1957) are shown in Appendix B.

A laboratory testing programme consisting of natural moisture content, bulk unit weight, Atterberg Limits tests and grain-size analyses, was performed on selected soil samples. The results of the laboratory tests are presented on the appropriate Borehole Log Sheets and also in Appendix C.

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

Details of the subsurface conditions encountered in the borehole are presented on the Record of Borehole sheets in Appendix A. The conditions are further described in the following paragraphs.

##### **4.1 TOPSOIL**

Topsoil was encountered in Boreholes SP1 and SP2 extending to depths ranging from 0.1 m to 0.4 m below ground surface. The topsoil encountered in Borehole SP1 was found to have a high sand content.

##### **4.2 FILL**

###### **4.2.1 SAND & GRAVEL (GRANULAR FILL)**

Borehole SP3 was drilled on the gravel shoulder of Highway 401 Eastbound lanes (EBL) approximately 11 m east of the centre line of the existing concrete arch culvert. This borehole contacted a surficial granular fill layer of brown sand and gravel that extends to a depth of 0.5 m (Elev. 97.1 m) below ground surface.

A Standard Penetration test conducted in this granular deposit yielded an N-value of 19 blows for 0.3 m penetration. From this value the relative density of this deposit is described as compact. The measured moisture content of a sample from this deposit was 7%.

###### **4.2.2 SILTY CLAY (EMBANKMENT FILL)**

The sand and gravel (granular) fill at Borehole SP3 is underlain by a 6.2 m thick layer of silty clay fill that extends to a depth of 6.7 m (Elev. 90.9 m) below ground surface.

This fill generally consists of silty clay mixed with some sand, traces of gravel and occasional topsoil inclusions. It is classified as a basically cohesive soil. Some topsoil was

encountered within this deposit at depths of about 1.8 m and also again at 4.8 m below existing grade. There was also an observed colour change from brown to grey in this fill layer at 5.9 m (Elev. 91.7 m) below ground surface.

N-values recorded in this fill material ranged from 9 to 22 blows/0.3 m, indicating a stiff to very stiff consistency. These recorded N-values also indicate that the fill received some degree of systematic compaction when the embankment was built.

The results of grain-size distribution analyses carried out on four selected samples are given in Figure 1 of Appendix C. The results indicate the following grain size distribution:

Gravel:	1 – 2 %
Sand:	12 – 29 %
Silt:	38 – 46 %
Clay:	32 – 40 %

Atterberg Limits tests performed on two samples from the embankment fill gave the following index values:

Liquid Limit:	28-33%
Plastic Limit:	15-19%
Plasticity Index:	12-13%

As shown in Figure 2 in Appendix C, these values indicate that this fill is essentially composed of silty clay material and the deposit is described as cohesive.

The measured natural moisture content of samples recovered from this fill material ranged from 14 to 28% and the bulk unit weight of samples from this deposit ranged between 19.5 and 21.5 kN/m<sup>3</sup>.

#### 4.3 SILTY SAND

The topsoil deposit in Boreholes SP1 and SP2 is underlain by a native deposit of brown to dark brown silty sand that extends to depths ranging from 1.0 m (Elev. 90.6 m) to 1.4 m (Elev. 90.6 m) below ground surface. This deposit contains some gravel and some clay at Borehole SP1. At Borehole SP2 the deposit was found to contain traces of topsoil and rootlets and some organics were encountered to a depth of 0.7 m below ground surface.

Standard Penetration tests carried out in this fine grained granular deposit yielded N-values of 4 to 23 blows/0.3 m, indicating a loose to compact relative density.

A grain-size distribution analysis was carried out on a selected sample from this deposit retrieved from Borehole SP1. The results, illustrated in Figure 3 of Appendix C indicate the following grain size distribution:

Gravel:	19 %
Sand:	48 %
Silt:	20 %
Clay:	13 %

The measured natural moisture content of representative samples from this deposit ranged from 7% at Borehole SP1 to 29% at Borehole SP2. The relatively high moisture content (29%) encountered in Borehole SP2 is most likely due to the presence of organics found in this deposit.

#### 4.4 SANDY GRAVEL

In Borehole SP1 the topsoil and silty sand strata were further underlain by a deposit of sandy gravel. This granular deposit was contacted at 1.4 m (Elev. 90.6m) and it extended to 2.1 m (Elev. 89.9 m) below ground surface. There was an observed colour change from brown to grey in this stratum at a depth of 1.8 m (Elev. 90.2 m).

A Standard Penetration test conducted in this stratum gave an N-value of 14 blows/0.3 m indicating a compact condition. The natural moisture content of a representative sample of this deposit was of the order of 13%.

A representative sample of this material was subjected to a grain-size distribution analyses and the results are presented in Figure 4 of Appendix C. The results show 60% gravel, 34% sand and 6% silt and clay size particles.

#### 4.5 GLACIAL TILL (SANDY SILT TILL)

The sandy gravel in Borehole SP1, silty sand in Borehole SP2 and fill in Borehole SP3 are underlain by a major deposit of glacial till. This stratum was encountered at depths ranging between 1.0 m (Elev. 90.6 m) to 6.7 m (Elev. 90.9 m) below ground surface and it extended to borehole termination depths of between 9.3 m (Elev. 82.3 m) and 12.2 m (Elev. 85.4 m) below ground surface.

This deposit was grey in colour and generally consisted of a heterogenous mixture of silt, sand and gravel with trace to some clay. It is basically a sandy silt till with some clayey silt to silty sand till layers/zones. Being of glacial origin, the till can be expected to contain random cobbles and boulders due to its mode of deposition.



The results of grain-size distribution analyses carried out on ten representative samples of this till deposit are given in Figure 5 in Appendix C in an envelope form. These results show the following grain-size distribution:

Gravel:	4 – 39 %
Sand:	32 – 52 %
Silt:	16 – 32 %
Clay:	9 – 29 %

Atterberg Limits tests performed on two samples gave the following index values:

Liquid Limit:	14 – 16 %
Plastic Limit:	10 – 11 %
Plasticity Index:	4 – 5 %
Natural Moisture Content:	8 – 9%

As shown in Figure 6 in Appendix C, these values indicate soils of low plasticity typical of silts and silty sands with trace to some clay. Based on these test results together with a visual and tactile examination of the recovered soil samples, this deposit is described as a granular (non cohesive) material with occasional cohesive zones.

Standard Penetration tests conducted in this deposit yielded N-values of 22 blows to more than 100 blows for 0.3 m penetration. From these values the relative density of this deposit is described as dense to very dense.

The natural moisture content of samples recovered from this deposit ranged from 6 to 12% and the unit weight ranged from 22.7 to 24.2 kN/m<sup>3</sup>.

#### 4.6 GROUNDWATER CONDITIONS

Groundwater conditions in the open boreholes were observed while drilling and at the completion of each borehole. In addition, piezometers were installed in two of the boreholes (i.e. Boreholes SP1 and SP2) to enable prolonged groundwater level measurements, without interference from surface water. The observations and recorded values are shown on the individual borehole log sheets.

Stabilized water levels in the piezometers were measured at elevations of 90.3 and 91.6 m or at depths of 0.4 and 1.3 m below the ground surface at the borehole locations. Based on these values and the change of the colour of the soil from brown to grey, which is generally at Elevations 90 m - 92 m, the permanent groundwater table at the site can be expected between Elevations 90 m and 92 m. This groundwater level is expected to be controlled by the water level in the existing creek.

It should also be pointed out that the groundwater is subject to seasonal fluctuations and fluctuations in response to major weather events.

**SHAHEEN & PEAKER LIMITED**



Rehman Abdul, P.Eng.



Ramon Miranda, P.Eng.



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ZO:tr/hdi

# DRAWINGS

# **APPENDIX A**

## **Records of Boreholes** **(Shaheen & Peaker Limited)**

SPT 1078

# RECORD OF BOREHOLE No SP2

1 OF 1

METRIC

GWP 377-01-00 LOCATION Highway 401; Town of Cobourg, ON - Coords: N 4 872 043.1; E 409 624.4 ORIGINATED BY G.I  
DIST 21 HWY 401 BOREHOLE TYPE Solid Stem Augers COMPILED BY G.T  
DATUM Geodetic DATE 11/6/2002 CHECKED BY R.A.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT      NATURAL LIMIT                  MOISTURE CONTENT			UNIT WEIGHT  γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa			WATER CONTENT (%)				
91.6 0.0	Ground Surface							20 40 60 80 100				W <sub>p</sub> W      W <sub>L</sub>	kN/m <sup>3</sup>	GR SA SI CL	
	100 mm <b>Topsoil</b> <b>SILTY SAND:</b> trace topsoil and rootlets, some organics below 0.7 m, brown to dark brown, loose to compact, moist		1	SS	4		91					○		23.6	18 41 21 20
90.6 1.0			2	SS	22							○			
			3	SS	31		90					41		23.2	15 37 29 19
			4	SS	50/13							○		23.0	11 39 30 20
			5	SS	50/13		89					○			
			6	SS	50/14		88					○		24.2	21 47 23 9
			7	SS	50/13		87					○			
			8	SS	50/8		86					○		23.6	13 52 22 13
			9	SS	50/8		85					○			
			10	SS	50/10		84					○			
							83								
82.3 9.3	End of borehole		11	SS	50/15							○			
	* Water level at 3.2 m (not stabilized) and hole open to 7.8 m on completion.  ** Piezometer Installed at 9.1 m. Water level on: Nov. 06/2002 - Elev 84.9 m ( 6.7 m) Nov. 14/2002 - Elev 90.3 m ( 1.3 m)														

\* Water level at 9.2 m (not stabilized) and  
hole open to 7.8 m on completion.

\*\* Piezometer Installed at 9.1 m.  
Water level on:  
Nov. 06/2002 - Elev 84.9 m ( 6.7 m)  
Nov. 14/2002 - Elev 90.3 m ( 1.3 m)

RECORD OF BOREHOLE No SP3

1 OF 1

METRIC

WP 377- 01- 00 LOCATION Highway 401; Town of Cobourg, ON - Coords: N 4 871 995.5; E 409 659.9 ORIGINATED BY G.I  
DIST 21 HWY 401 BOREHOLE TYPE Solid Stem Augers COMPILED BY G.T  
DATUM Geodetic DATE 11/8/2002 CHECKED BY R.A.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT			UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED      + FIELD VANE ● POCKET PENETR.      × LAB VANE		WATER CONTENT (%) w <sub>p</sub> w      w <sub>L</sub>				GR	SA	SI	CL
97.6	Ground Surface							20 40 60 80 100									
0.0	FILL: Sand and Gravel, compact, brown, damp		1	SS	19			40 80 120 160 200									
97.1																	
0.5			2	SS	22												
	some topsoil		3	SS	9												
			4	SS	16												
	FILL: Silty Clay, trace gravel, some sand, occasional topsoil inclusions. stiff to very stiff, damp to moist		5	SS	21												
			6	SS	10												
	some sandy silt with topsoil, dark brown		7	SS	15												
			8	SS	10												
	brown		9	SS	18												
	grey																
90.9			10	SS	91												
6.7			11	SS	50/13												
	Heterogeneous mixture of Silt, sand and gravel, some clay, (GLACIAL TILL) very dense, grey, moist		12	SS	50/8												
			13	SS	50/8												
85.4			14	SS	50/5												
12.2	End of borehole																
	* Water level at 6.3 m (not stabilized) and hole open to 7.9 m on completion.																

+ <sup>3</sup>, × <sup>3</sup>: Numbers refer to Sensitivity

20  
15-φ-5  
10 (%) STRAIN AT FAILURE

# APPENDIX B

## Records of Boreholes (MTO 1957)

DEPARTMENT OF HIGHWAYS - ONTARIO  
MATERIALS & RESEARCH BRANCH - FOUNDATIONS SECTION - DOWNSVIEW  
**OFFICE REPORT ON SOIL EXPLORATION**

DRILL RIG 54-1 OPERATION BORE & PENET N JOB F-57-17 WP. 50-57 BORING 1 STA. 692+80 (50 ft)  
CASING ØX (standard samplers to fit unless noted) DATUM GEODETIC DATE REPORT JULY 1957  
SAMPLER HAMMER WT. 250 LBS. DROP 19 INCHES COMPILED BY H.S. CHECKED BY A.L. DATE BORING 5 JUNE 1957

**ABBREVIATIONS**

V - INSITU VANE SHEAR TEST Q - TRIAXIAL QUICK K - PERMIABILITY  
M - MECHANICAL ANALYSIS S - TRIAXIAL SLOW C - CONSOLIDATION  
U - UNCONFINED COMPRESSION WL - WATER LEVEL IN CASING CA - CASING  
TC - TRIAXIAL CONSOLIDATED QUICK WT - WATER TABLE IN SOIL γ - UNIT WEIGHT

**SAMPLE TYPES**

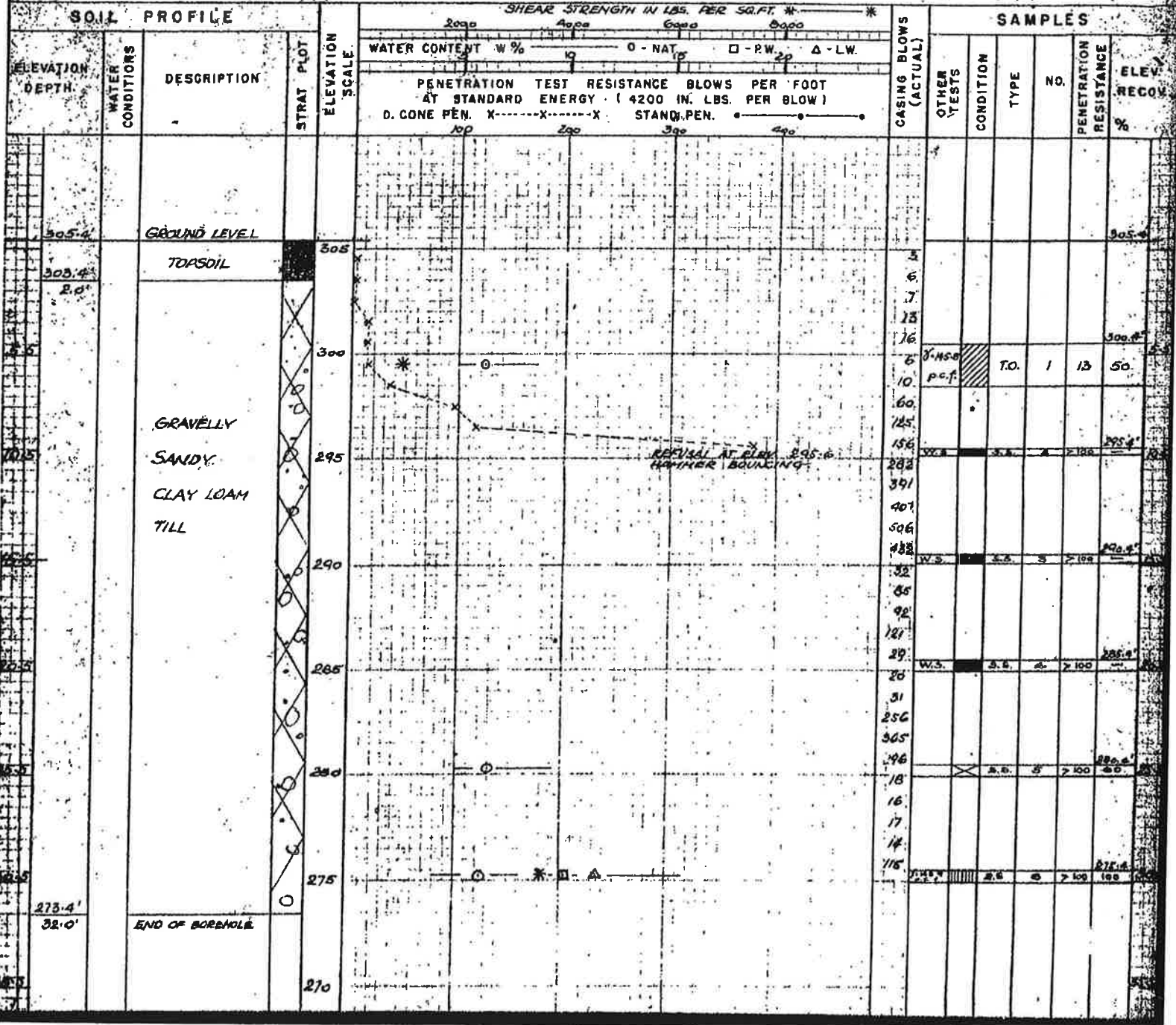
C.S. - CHUNK S.S. - SLEEVE SAMPLE  
D.O. - DRIVE OPEN P.S. - PISTON SAMPLE  
D.F. - DRIVE FOOT VALVE W.S. - WASHED SAMPLE  
T.O. - THIN WALLED OPEN R.C. - ROCK CORE

**SAMPLE CONDITION**



- DISTURBED  
- FAIR  
- GOOD  
- LOST

**SOIL PROFILE**





DEPARTMENT OF HIGHWAYS - ONTARIO  
 MATERIALS & RESEARCH BRANCH - FOUNDATIONS SECTION - DOWNSVIEW  
**OFFICE REPORT ON SOIL EXPLORATION**

DRILL RIG 54-1 OPERATION BORE & PENET. JOB F-57-17 WP. 50-57 BORING 2 STA. 691+75 (48' R)  
 CASING Bx (standard samplers to fit unless noted) DATUM GEODETIC DATE REPORT JULY 1957  
 SAMPLER HAMMER WT. 250 LBS. DROP 12 INCHES COMPILED BY H.S. CHECKED BY AL DATE BORING JUNE 1957

## ABBREVIATIONS

V - INSITU VANE SHEAR TEST Q - TRIAXIAL QUICK K - PERMIABILITY  
 M - MECHANICAL ANALYSIS S - TRIAXIAL SLOW C - CONSOLIDATION  
 U - UNCONFINED COMPRESSION WL - WATER LEVEL IN CASING CA - CASING  
 Q<sub>c</sub> - TRIAXIAL CONSOLIDATED QUICK W.T. - WATER TABLE IN SOIL γ - UNIT WEIGHT

## SAMPLE TYPES

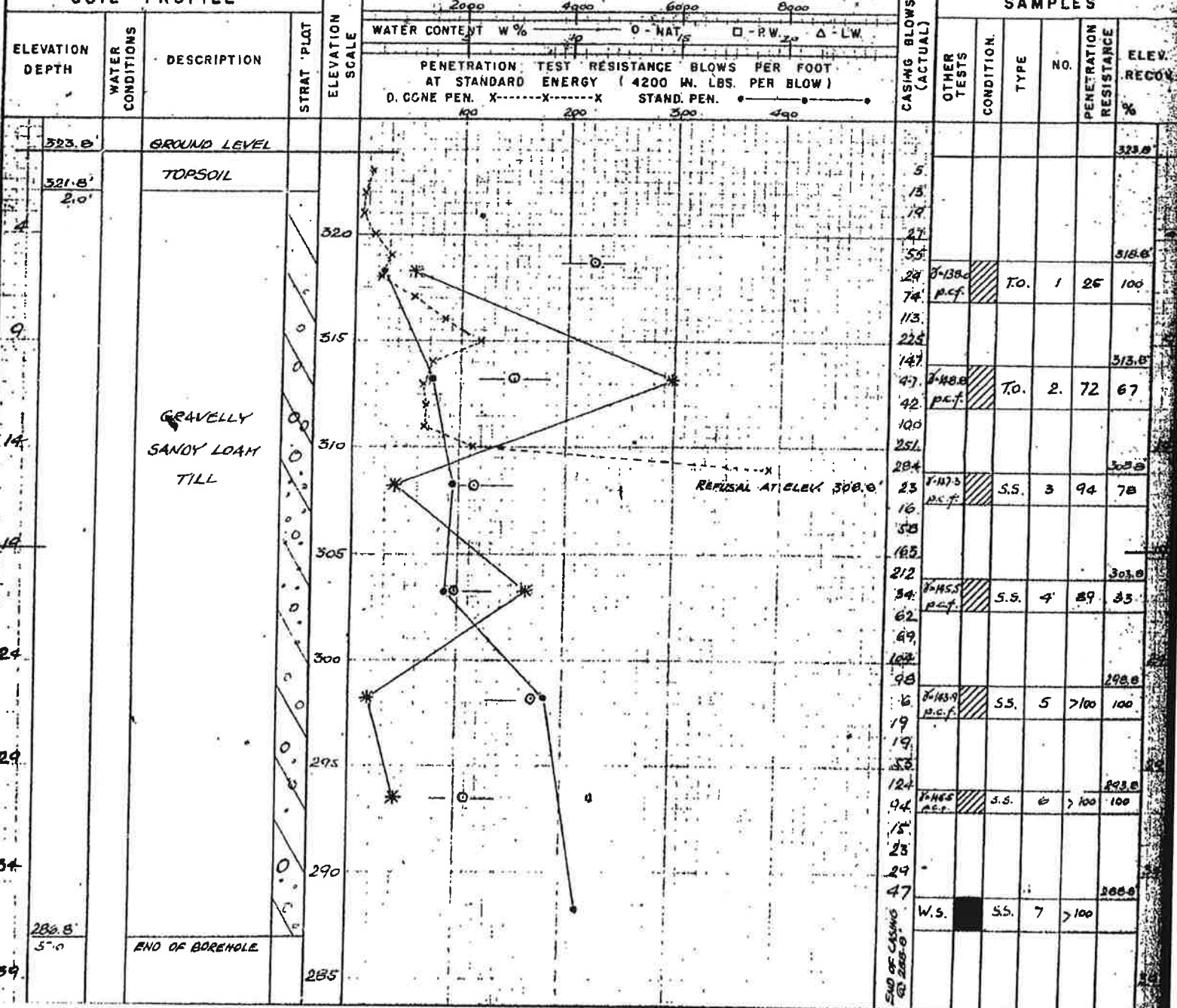
C.S. - CHUNK S.S. - SLEEVE SAMPLE  
 D.O. - DRIVE OPEN P.S. - PISTON SAMPLE  
 D.F. - DRIVE FOOT VALVE W.S. - WASHED SAMPLE  
 T.O. - THIN WALLED OPEN R.C. - ROCK CORE

## SAMPLE CONDITION



- DISTURBED  
 - FAIR  
 - GOOD  
 - LOST

## SOIL PROFILE



DEPARTMENT OF HIGHWAYS - ONTARIO  
MATERIALS & RESEARCH BRANCH - FOUNDATIONS SECTION - DOWNSVIEW  
**OFFICE REPORT ON SOIL EXPLORATION**

DRILL RIG 54-1 OPERATION PENETRATION JOB F-52-17 WP 50-57 BORING 3 STA. 691+45 (45' LT)  
CASING BK (standard samplers to fit unless noted) DATUM GEODETIC DATE REPORT JULY 1957  
SAMPLER HAMMER WT. 250 LBS. DROP 12 INCHES COMPILED BY H.S. CHECKED BY A.L. DATE BORING 14 JUNE 1957

**ABBREVIATIONS**

V - INSITU VANE SHEAR TEST Q - TRIAXIAL QUICK K - PERMIABILITY  
M - MECHANICAL ANALYSIS S - TRIAXIAL SLOW C - CONSOLIDATION  
U - UNCONFINED COMPRESSION WL - WATER LEVEL IN CASING CA - CASING  
QC - TRIAXIAL CONSOLIDATED QUICK WT - WATER TABLE IN SOIL γ - UNIT WEIGHT

**SAMPLE TYPES**

C.S. - CHUNK S.S. - SLEEVE SAMPLE  
D.O. - DRIVE OPEN P.S. - PISTON SAMPLE  
D.F. - DRIVE FOOT VALVE W.S. - WASHED SAMPLE  
T.O. - THIN WALLED OPEN R.C. - ROCK CORE

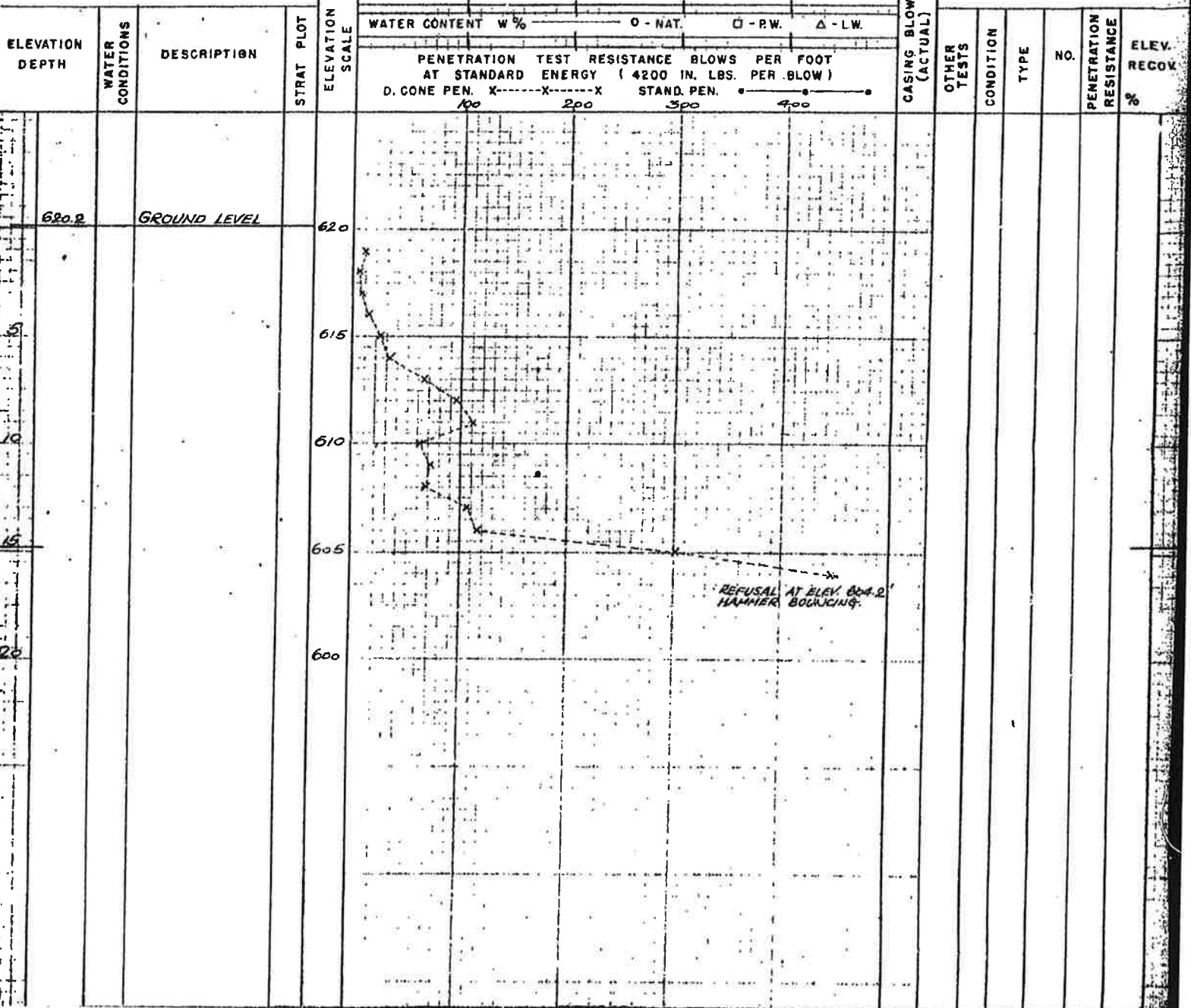
**SAMPLE CONDITION**



- DISTURBED  
- FAIR  
- GOOD  
- LOST

**SOIL PROFILE**

**SAMPLES**

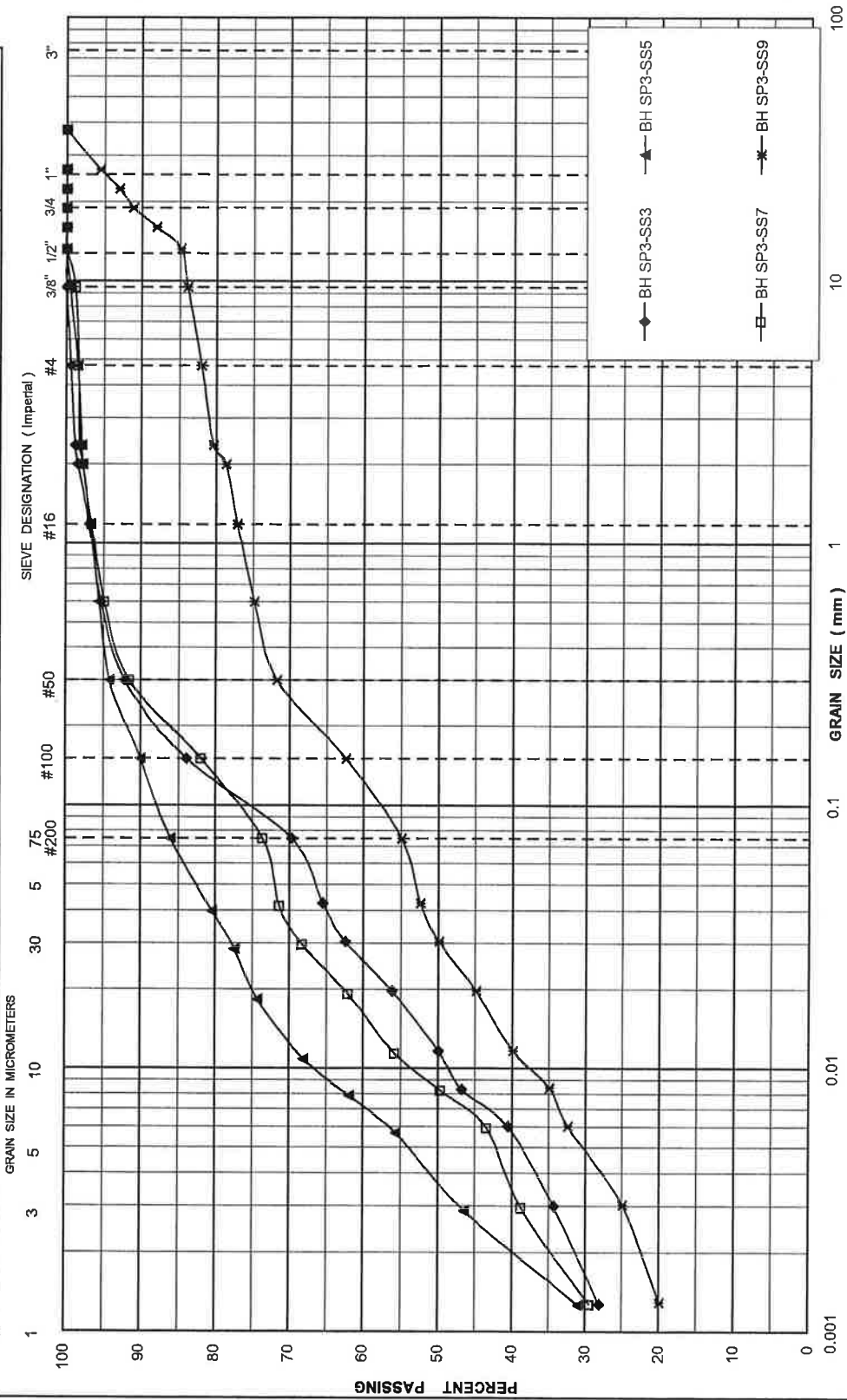


# APPENDIX C

## Laboratory Test Results

# UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT			SAND			GRAVEL		
			Fine	Medium	Coarse	Fine	Coarse	



## GRAIN SIZE DISTRIBUTION Silty Clay (Embankment Fill)

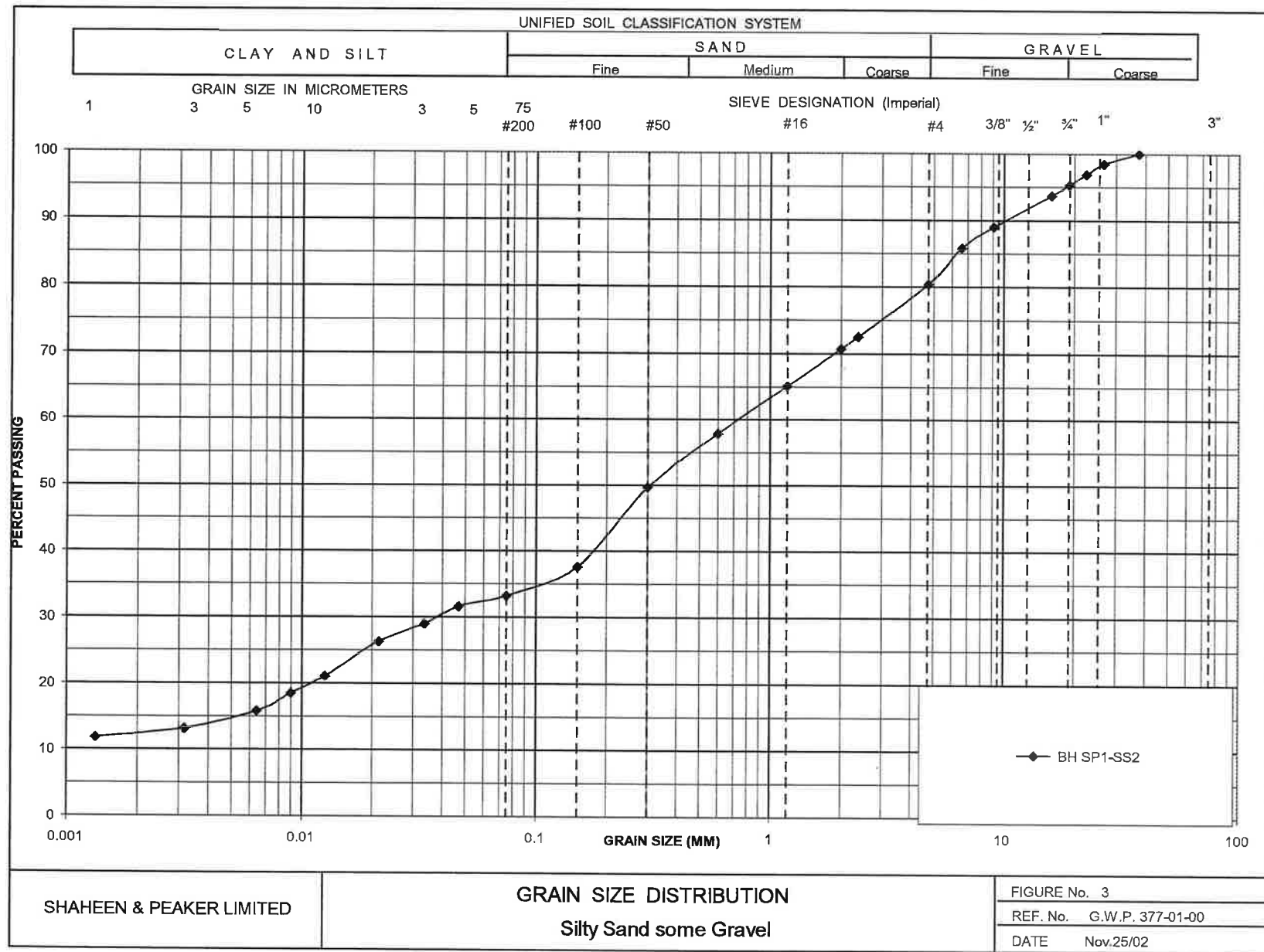
SHAHEEN & PEAKER LIMITED

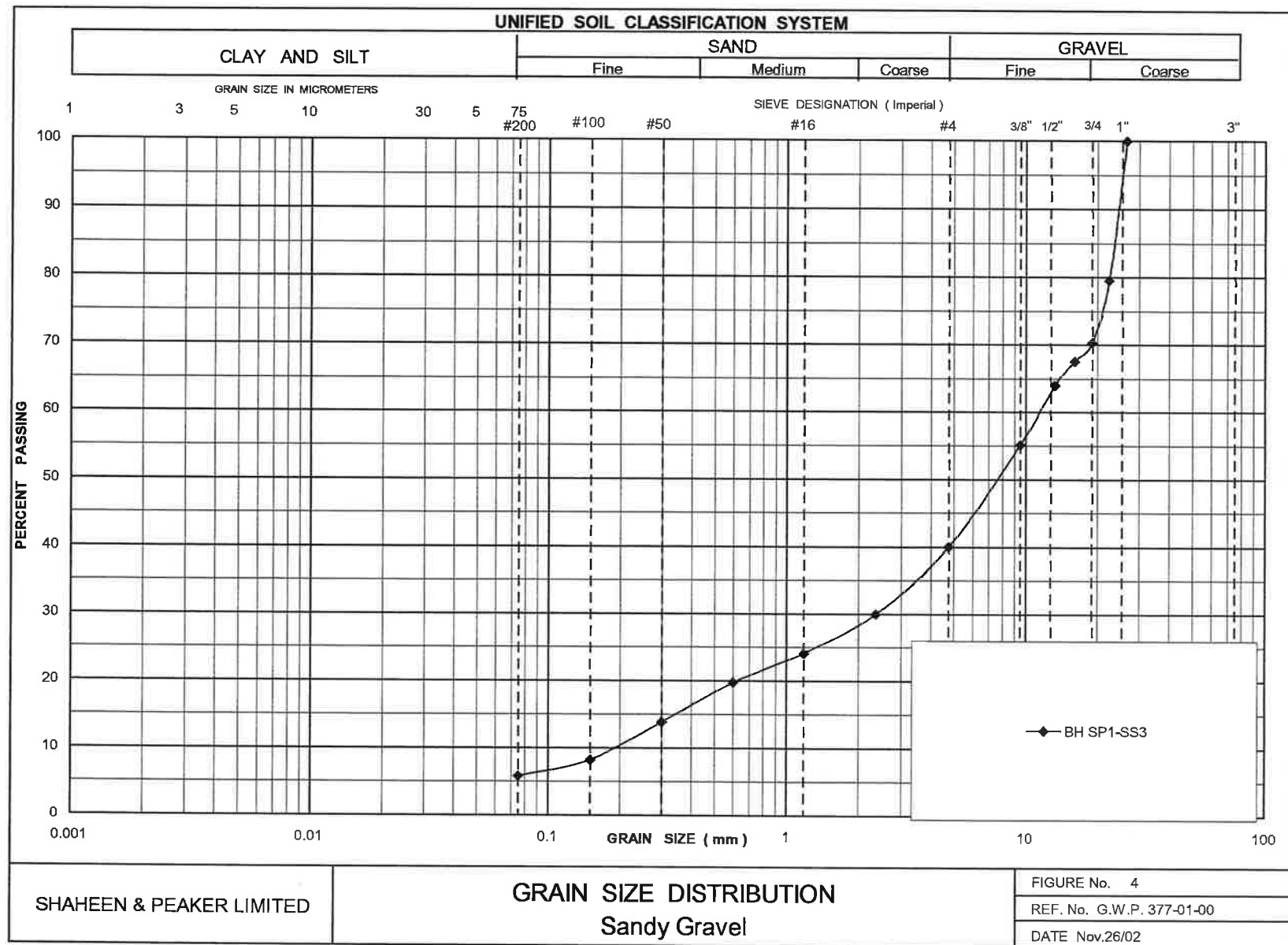
FIGURE No. 1

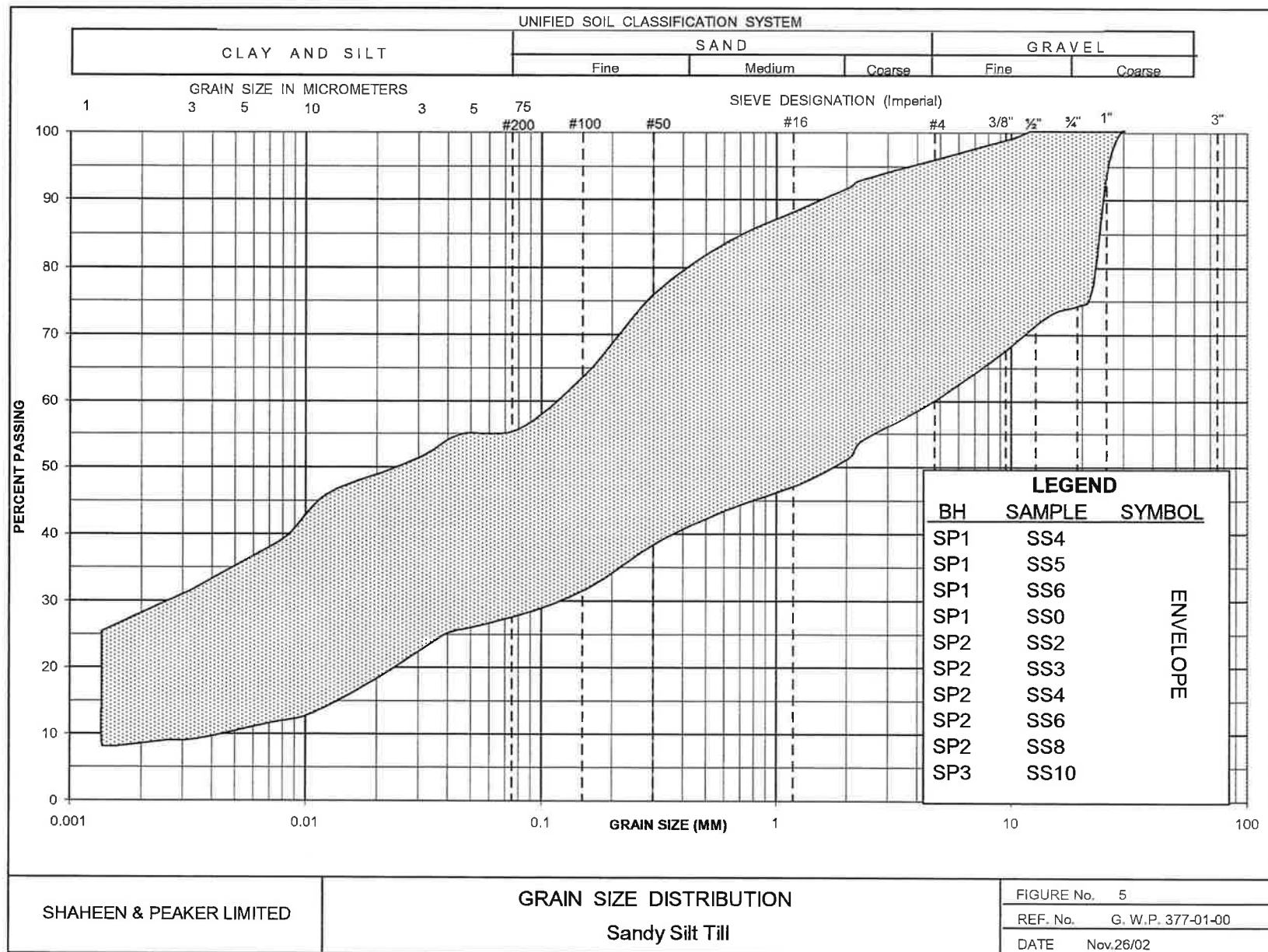
REF. No. G.W.P.377-01-00

DATE Nov.26/02

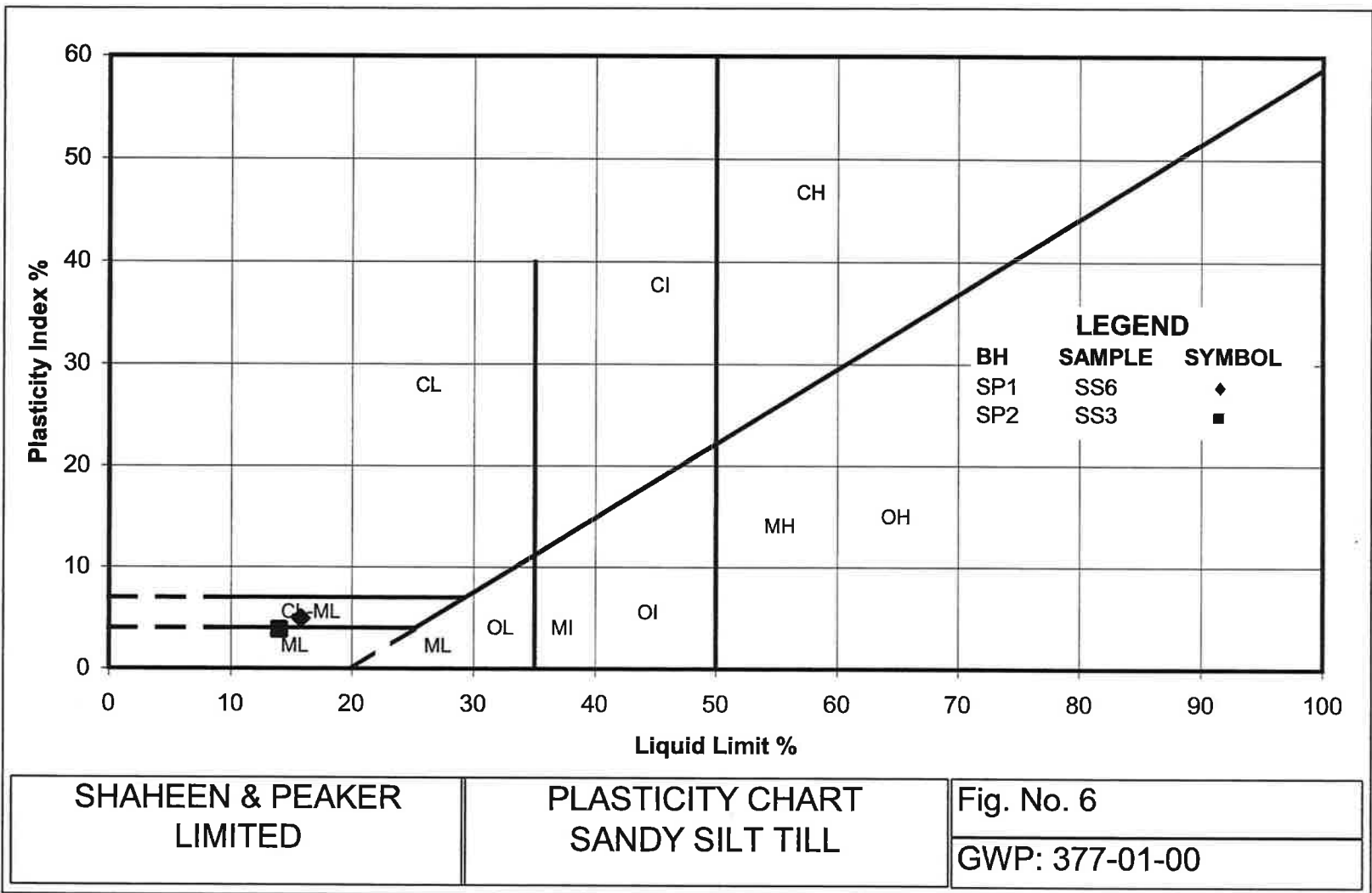












## APPENDIX D

### 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 N.

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

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

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

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

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

**JOINTING AND BEDDING:**

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

## ABBREVIATIONS AND SYMBOLS

### FIELD SAMPLING

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

### STRESS AND STRAIN

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

### MECHANICAL PROPERTIES OF SOIL

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

## PHYSICAL PROPERTIES OF SOIL

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

**FOUNDATION DESIGN REPORT  
PROPOSED CULVERT EXTENSION AT COBOURG CREEK  
(FORMERLY DYE WORKS CREEK)  
HIGHWAY 401 AT BURNHAM STREET  
NEW S-E RAMP  
COBOURG, ONTARIO  
G.W.P. 377-01-00**

**Prepared For:**

**UMA ENGINEERING LIMITED**

**Prepared by:**

**SHAHEEN & PEAKER LIMITED**

**Project: SPT1078  
March 17, 2003**



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### APPENDICES

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**APPENDIX F: Limitations of Report**

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**5. DISCUSSION AND RECOMMENDATIONS**

**5.1 GENERAL**

As part of the planned intersection improvements at the Highway 401/ Burnham Street interchange, Highway 401 will be widened from four to six lanes. Consequently, the present culvert (through which Cobourg Creek flows) will require approximately 19 m and 28 m extensions at the north and south ends respectively.

The existing culvert is an open bottom concrete arch culvert measuring 12.2 m in width with a rise of 5.8 m. The culvert is 65.5 m long and is covered with approximately 1.2 m of fill. The new extensions to both ends of the culvert are expected to be precast concrete arches and the northern end will be constructed with concrete wing walls.

West of Station 1+360 about 1 m of cut to the proposed finished grade is proposed. From Station 1+360 the existing ground surface drops towards Cobourg Creek and up to about 8 m of fill is expected at the west approach. East of the proposed south culvert extension, between Station 1+418 and Station 1+450 embankment fill heights up to 7 m high are expected. The southern crown elevation of the existing culvert is approximately 96.4 m and the grade of Highway 401 directly above the culvert is about Elevation 97.5 m. The new S-E Ramp over the proposed southern culvert extension will have a finished grade of Elevation 99 m. The top of the embankment fills at the southern culvert extension are therefore expected to be about 2 to 3 m above the crown of the proposed extension of the culvert.

Based on drawings provided by UMA Engineering Limited, the top of footing of the existing culvert is at Elevation 90.2 m. The bottom of the existing culvert footing is understood to be at about Elevation 88.4 m.

Presently, no information is available for the centre line profile of the proposed E-N/S ramp to be constructed on the north side of Highway 401. For the purposes of this report it is assumed that the elevation of the centre line profile of this ramp over the proposed northern culvert extension will likely be similar to the existing elevation of the north shoulder of

Highway 401 at this location. In this case no more than 1.2 m of fill is expected above the crown of the proposed culvert extension.

Boreholes SP1 and SP2 revealed the presence of approximately 0.4 m of topsoil to Elev. 91.7 m followed by silty sand to Elev. 90.6 m. These deposits were further underlain by a 0.7 m thick deposit of sandy gravel encountered only at Borehole SP1. These surficial deposits were further underlain by a major till sheet (sandy silt till), which consists of a heterogeneous mixture of silt, sand and gravel with trace to some clay. This stratum extended to borehole termination depths of 9.2 m (Elev. 82.8 m) and 9.3 m (Elev. 82.3 m) and has a dense to very dense state of compactness.

Borehole SP3 was advanced from the top of the embankment (Elev. 97.6 m) on the gravel south shoulder of Highway 401. This borehole revealed the presence of a 0.5 m thick layer of sand and gravel (granular fill) to Elev. 97.1 m underlain by a stiff to very stiff silty clay embankment fill to a depth of 6.7 m or Elev. 90.9 m. These strata were further underlain by a major till sheet that extended to the termination depth of the borehole i.e. 12.2 m (Elev. 85.4 m). The glacial till has a very dense relative density.

The groundwater level at the site is believed to be at elevations ranging between 90 and 92 m and is expected to be controlled by the water level in the existing creek.

#### 5.1.1 AVAILABLE OPTIONS

The existing culvert is an open bottom concrete arch culvert with an approximate width of 12.2 m and a rise of 5.8 m.

In general, the choice of culvert type depends on such parameters as the initial cost, service life, maintenance costs, hydraulic performance, ease of construction, salvageability and structural strength of material used, fish passage, etc. The material choice includes corrugated steel, concrete and plastic, while the Ministry may entertain alternative materials, as well. Some of the factors considered in making an appropriate choice of material type include:

- Steel and plastic have the advantage of simpler and quicker construction, especially in remote areas, while steel has the added advantage of often being at least partly salvageable after being washed out.
- A well designed concrete culvert is extremely durable under a wide range of conditions.
- Precast concrete and smooth walled plastic pipes provide more efficient inlets than sharp edged inlets on metal culverts.

- The greater roughness of corrugated interiors may be an advantage for fish passage and for other situations where barrel or outlet velocities must be reduced.
- Flexible pipe culverts may have an advantage over concrete box culverts in certain unfavourable foundation soil conditions.

As a material choice in the present case, a concrete culvert is considered more appropriate under a major highway, such as Highway 401, for the reasons cited above (i.e. its durability) as well as the existing good foundation conditions as revealed by the boreholes. The following concrete culvert options may be considered.

- Replacing the existing culvert with a closed bottom box culvert.
- Replacing the existing culvert with an open bottom box culvert.
- Extending the existing arch culvert with a closed end arch culvert of similar geometry as the existing culvert.
- Extending the existing arch culvert with an open end arch culvert of similar geometry as the existing culvert.

All of the above options are feasible.

A disadvantage of open bottom culverts is that they are vulnerable to failure caused by scour, degradation or artificial deepening. In the present case, however, we understand that the existing open bottom culvert has performed satisfactorily (e.g. no problems due to scour, etc. have been reported); it is obvious that the use of the same type culvert for the extensions will by far be the most cost-effective choice. In addition, replacing the existing culvert with a new box culvert may create new environmental impacts during its construction. For these reasons, it is our opinion that matching the existing culvert for the extensions is the better choice.

## 5.2 STRUCTURE FOUNDATIONS

The proposed culvert extension can be founded on normal strip foundations. The use of deep foundations is neither necessary nor recommended for the following reasons. The soils underlying the site, below the foundation levels, are competent materials. Therefore, the use of deep foundations will be much more costly in comparison to normal strip footing foundations. Driven piles will require pre-boring to attain sufficient penetration depths, increasing their cost. Furthermore, vibrations induced while driving the piles may be detrimental to the integrity of the existing structure. While this latter aspect can be remedied by using drilled and cast-in-place concrete caissons, this type of foundations are also costly



especially in view of the prevailing high groundwater table at the site. The use of deep foundations at this site is, therefore, not recommended.

#### 5.2.1 SPREAD FOOTING FOUNDATIONS

Reference to Boreholes SP1, SP2 and SP3 indicates that the culvert extensions can be founded on strip footing foundations in the undisturbed competent sandy silt till. The choice of foundation elevation should be determined based on the elevation of the existing footings and the expected frost and scour depths, which is lower. It is understood that the existing culvert foundations are at Elevation 88.4 m. The footings of the proposed culvert extensions should be founded at the same elevation as the existing footings (i.e. 88.4 m) provided that this elevation satisfies scour and frost protection criteria. If the proposed footing elevation for the extension is expected to be more than 0.4 m lower than 88.4 m (i.e. the existing arch culvert footing elevation) then shoring will be required to prevent undermining during construction and to ensure that the load bearing capacity of the existing footings is maintained.

For frost protection, the footings should have a permanent minimum earth cover of at least 1.5 m or its thermal equivalent. The recommended founding depths and bearing resistances on undisturbed, competent sandy silt till are tabulated below.

Table 5.1

Borehole No	Existing Ground Surface Elevation (m)	Recommended Highest (bottom) of Footing Below Existing Ground Surface (m)	Recommended Highest (bottom) of Footing Elevation (m)	Factored Bearing Resistance at U.L.S. (kPa)	Bearing Resistance at S.L.S. (kPa)	Subgrade Material
SP1	92.0	2.5 3.0	89.5 89.0	700 800	350 450	Sandy Silt Till Sandy Silt Till
SP2	91.6	2.0 2.6	89.6 89.0	700 800	350 450	Sandy Silt Till Sandy Silt Till
SP3	97.6	7.0 8.6	90.6 89.0	700 800	350 450	Sandy Silt Till Sandy Silt Till

Higher bearing resistance values are available at greater depths but are not believed to be necessary for the light structure proposed, as well as to keep the foundations as high as possible due to dewatering requirements, as discussed later in this report.

With the recommended serviceability values, the total and differential settlements are expected not to exceed 20 mm provided that the founding subgrade is undisturbed during the construction. The settlements will be relatively short term and should take place within a few weeks of construction. It should however be remembered that these settlements will translate into differential settlements between the existing culvert footings and those of the extensions. This aspect should be taken into consideration by designing construction joints (at the interface between the existing culvert and the proposed extensions) to accommodate estimated differential settlements of up to 20 mm.

Where weak, organic or otherwise unsuitable soils are encountered at the foundation subgrade level, they should be removed and replaced with lean concrete or Granular 'A' type material compacted to not less than 100% of the material's Standard Proctor Maximum Dry Density (SPMDD). In this instance, because of the high water table, the use of lean concrete is recommended to raise the foundation grades. All founding subgrades should be inspected and evaluated by the Quality Verification Engineer (QVE), at the time of construction.

The unfactored horizontal resistance against sliding between poured concrete and approved till surface can be calculated using a friction angle of 31° in the gravelly sandy silt till. This value can be increased to 35° for poured concrete and well compacted Granular 'A' type material, although lateral resistance is unlikely to be a problem for culvert foundations.

### 5.3 BACKFILLING AND LATERAL EARTH PRESSURES

Backfilling for the culvert extensions should consist of suitable free-draining granular materials, compacted in accordance with the MTO standards and should conform to the applicable OPSD. For fills below the groundwater level or immediately below the roadway, it is recommended that Granular 'A' or 'B' materials be used. Where necessary, proper tapering as per MTO standards should be provided. The fill should be compacted in shallow lifts, not exceeding 200 mm loose thickness, to at least 98% of the material's Standard Proctor Maximum Dry Density (SPMDD). The Granular 'A' or 'B' materials should be compacted to 100% of their SPMDD's. To avoid damaging or laterally dislocating the structure, care should be exercised when compacting fill adjacent to and immediately on top of the culvert structure. Compaction equipment should be restricted in size as per MTO convention to prevent structural damage to the culvert. The backfilling operation should be carried out simultaneously on both sides of the culvert as per MTO specifications.

Backfill behind any retaining (wing) walls should consist of Granular 'B' type materials in accordance with the Ontario Ministry of Transportation Standards. Free draining backfill materials, weepholes, etc. should be provided in order to prevent hydrostatic build-up, as shown on OPSD-3501.000.

Computation of earth pressures acting against rigid culvert walls should be in accordance with the Canadian Highway Bridge Design Code (CHBDC) 2000. For design purposes, the following properties can be assumed for backfill:

**Compacted Granular 'A'**

Angle of Internal Friction  $\Phi = 35^\circ$  (unfactored)

Unit Weight =  $22 \text{ kN/m}^3$

Coefficient of Lateral Earth Pressure:

Level Backfill	Backfill Sloping at 3H:1V	Backfill Sloping at 2H:1V
$K_a = 0.27$	$K_a = 0.34$	$K_a = 0.40$
$K_o = 0.43$	$K_o = 0.56$	$K_o = 0.62$

**Compacted Granular 'B'**

Angle of Internal Friction  $\Phi = 30^\circ$  (unfactored)

Unit Weight =  $21 \text{ kN/m}^3$

Coefficient of Lateral Earth Pressure:

Level Backfill	Backfill Sloping at 3H:1V	Backfill Sloping at 2H:1V
$K_a = 0.33$	$K_a = 0.42$	$K_a = 0.54$
$K_o = 0.50$	$K_o = 0.66$	$K_o = 0.76$

NOTE:  $K_a$  is the coefficient of active earth pressure.

$K_o$  is the coefficient of earth pressure at rest.

These values are based on the assumption that the backfill behind the retaining structure is free-draining granular material and adequate drainage is provided.

For rigid culvert walls which will not yield sufficiently to develop active earth pressures, at rest earth pressures should be used. Further, the effects of compaction will cause additional pressures which should be taken into consideration as per Clause 6.9.3 of the Canadian Highway Bridge Design (CHBDC) 2000.

The earth pressure coefficient adopted will depend on whether the retaining structure is restrained or some movement can occur such that the active state of earth pressure can develop. For a restrained structure where the rotational or horizontal movement is not sufficient to mobilize an active earth pressure condition, an at-rest pressure plus any compaction surcharge should be used for design. The movements required to activate active and passive earth pressure conditions can be determined by referring to Figure C 6.9.1(a) and Table C 6.9.1(a) of the Canadian Highway Bridge Design Code 2000.

The effect of compaction during construction and beyond should also be taken into account in the selection of the appropriate earth pressure coefficients. The use of vibratory compaction equipment behind the culvert and wing walls should be restricted in size, as per MTO convention.

#### 5.4 TEMPORARY SHORING DESIGN

At this culvert location, it may be necessary to construct shoring especially in the case where excavation is adjacent to the granular shoulder of Highway 401. In such cases where shoring is required, shoring by soldier piles and lagging can be considered. The shoring should be designed by a Professional Engineer experienced in this type of work. Shoring should be designed so that the lateral movement of any portion of the roadway protection system will not exceed the established criterion for the structure performance level. In this case, the performance level should be Level 2 - Angular Distortion 1:200 but shall not be more than 25 mm.

Locally, temporary shoring systems generally consist of support provided by conventional soldier piles and timber lagging. The soldier piles can be designed as cantilever structures or supported by employing a soil anchor system depending upon the depth of soil to be retained and the required performance criteria. Where necessary, soldier piles can be installed either by driving or pre-augering. The vibratory effects caused by driving may adversely affect the existing culvert. Further, control of the alignment of the piles, as well as required penetration depths are likely to be problematic, due to the presence of cobbles and boulders which could be encountered in the sandy silt till and the very dense nature of the soil. Consequently, pre-augering is considered to be a more feasible construction option. It should however be pointed out that high water table encountered at the site may necessitate measures to prevent the collapse of the auger holes, especially if the holes cannot be immediately concreted. Control of groundwater will also be required for proper lagging procedures. Proper fitting will be required to prevent the loss of soil between the lagging elements especially where cohesionless soils (i.e. the sandy silt till, sandy gravel and silty sand) are encountered below the water table. The use of geotextile behind the lagging is recommended to prevent the loss of soil fines.

Adjacent to the existing Highway 401 the excavations could be up to 7 to 8 m deep. If the deflection is acceptable, then for shallower excavations a cantilever type of shoring may be considered and the coefficient of lateral earth pressure parameters given in Table 5.2 can be used for the design. If a soil anchor tieback system is required (e.g. deep excavation support), then for preliminary design of temporary soil anchors, a tentative bond resistance (soil to concrete bond value) at U.L.S. of 60 kPa can be used in the dense to very dense sandy silt till and S.L.S will not govern. This value must be confirmed by load testing. Bond resistances in the embankment fill and in the relatively weak silty sand and sandy gravel overlying the sandy silt till should not be relied upon.

**Table 5.2**  
**Recommended Unfactored Parameters for Temporary Shoring Design**

Soil Type	Ka	Ko	Kp	$\gamma$ (kN/m <sup>3</sup> )
Granular Fill	0.30	0.45	3.3	21.5
Silty Clay Fill	0.33	0.50	3.0	20.5
Sandy Silt till	0.30	0.45	3.3	23.5

## 5.5 EMBANKMENT STABILITY

Provided that all organic soils, fill, weak and otherwise unsuitable materials are removed as per MTO standards before placing the fill, no instability problems are anticipated due to foundation conditions for the anticipated embankment heights of up to 7 m. The existing slopes should be benched prior to placing fill for the new embankments in accordance with OPSD-208.010.

Provided that the embankment subgrade is properly prepared, conventional embankment slopes of 2 horizontal in 1 vertical (2H:1V) would be stable. Loading generated from embankments of up to 7 m in height is expected to result in a foundation soil settlement of approximately 20 mm, which should take place within a few weeks of construction.

All organic and other unsuitable soils should be removed within an envelope given by an imaginary slope no steeper than 1:1 from the toe of the proposed embankment as depicted by the sketch presented in Appendix E. After stripping, the exposed subgrade should be inspected and approved. It should then be compacted from the surface using a suitably heavy compactor, under the supervision of the QVE.

The fill materials used for the construction of the embankment fills should consist of approved acceptable indigenous earth fill or select subgrade materials (SSM as per OPSS 1010). If the indigenous silty sand or the sandy silt till material are used, their use should be restricted to below 2 m depth from the finished pavement surface, due to their frost susceptible nature (i.e. greater than 25% soil fines or combined silt and clay content). The embankment fill should be placed on the approved and properly prepared subgrade in suitably shallow lifts and compacted in accordance with OPSS 501. The selection, placement and compaction of the fill should be carried out under the supervision of the Quality Verification Engineer.

The settlement of embankment fills, prepared as described above, due to its own consolidation, should not exceed 40 mm. The time rate of settlement will depend on the material used for construction. For granular fills, the settlement should be substantially completed during the construction and within a few weeks thereafter, while clayey fills will

consolidate over a longer period of time. These quoted settlements would be in addition to the foundation settlements of up to 20 mm, as discussed earlier. As the total of the two is only 60 mm, preloading or surcharging is not considered necessary especially since some of the settlements will take place during construction.

## 5.6 EROSION PROTECTION

Erosion protection should be provided at the culvert inlet and outlet (including the slopes and sides)

At the inlet this could consist of a clay seal. The purpose of the clay seal is to ensure that water flow is channeled through the culvert and does not seep through the backfill around the structure and underneath the structure. The clay seal should therefore be continuous and at least 0.6 m thick. It should comply with the material specifications given in OPSS 1205. It should be extended around the culvert from at least 0.3 m above the high water level down to the channel bed. It should be ensured that it extends to cover all the granular backfill materials to prevent any seepage through them. The clay seal should be protected by laying a 0.6 m thick rock protection over it.

Alternatively, concrete cut-off and head walls can be constructed to protect the granular backfill and to prevent seepage around the culvert.

At the outlet, a 0.6 m thick layer of rock protection consisting of 300 mm size rock should be used, overlaying a 300 mm thick layer of filter material. The filter material should consist of a granular material such as Granular 'A' or equivalent. Alternatively a suitable geotextile could be used in lieu of the granular filter (i.e. Granular 'A'). This filtered rock protection should extend at least 6 m along the channel from the outlet. It should also extend at least 6 m adjacent to the culvert outlet to the high water level and should protect the granular fill around the culvert. A head wall can also be used to protect the granular fill around the culvert outlet against erosion. In this case, however, filtered erosion protection such as rip-rap should be provided along the channel and the sides beyond the concrete cut-off, head walls at the outlet.

## 5.7 CONSTRUCTION COMMENTS

All excavations, shoring and backfilling should be carried out in conformance with the safety regulations of the province (i.e. Occupational Health and Safety Act, O.Reg. 213/91), as well as the following specifications.

SP 539S01 – Protection Schemes

SP 902S01 – Excavation and Backfilling to Structures

The boreholes show that the temporary excavations will extend through surficial topsoil, silty sand, sandy gravel, into competent sandy silt till. Excavations close to the shoulder of Highway 401 will extend through surficial sand and gravel, silty clay embankment fill, into competent sandy silt till. At Boreholes SP1 and SP2 the relatively high groundwater table will most likely require the construction of side slopes flatter than 1:1. At Borehole SP3 the stiff to very stiff silty clay embankment fill is expected to stand temporarily at 45° to the horizontal, provided that the height of excavation is less than 2 m. For excavations exceeding 2 m in height, flatter side slopes are recommended. Based on the result of Borehole SP3, excavations exceeding 2 m in height in the fill should be temporarily stable at 1 ½H:1V side slopes, with local flattening to 2H:1V, if and where required, depending on the degree of compaction and quality of the existing fill, as required by the QVE.

Provided that the existing footings are not disturbed during the construction, negligible settlements of the existing fully loaded culvert footings can be expected due to the relatively long time period since their initial construction. Therefore, settlements of the proposed extensions are expected to translate into differential settlements between the existing and proposed sections. Consequently, suitably designed construction joints will be required at the interfaces between the existing culvert and proposed extensions to accommodate estimated settlements of 20 mm.

It should be pointed out that it may be possible to use the existing footings of the culvert to accommodate the proposed extensions. However, the geometry of these existing footings, their integrity and the adequacy of the founding soils will require confirmation by test pit explorations at the time of construction. If this is not possible, we recommend removal of this section of the footings and replacement.

By means of good construction techniques, undermining of the existing culvert footings should be prevented. If there is a difference in elevation between the existing and new footings, shoring may be required.

The settlements of the existing footings which will be utilized to support the new structure will depend on such factors as the degree of additional loading, construction procedures followed when the footings were initially constructed, etc. Assuming however, that proper subgrade preparation techniques were followed, settlements should be less than 20 mm.

We recommend that the flow of water through the existing culvert be diverted away from the extension area so that the construction will proceed in sufficiently dry conditions. Depending on the flow conditions, it may be possible to accomplish this by containing the flow and diverting it via a solid pipe.

A cofferdam can also be considered for diverting the water, provided it is sufficiently impervious and affords sufficient work space.

After the flow of water in the creek is properly diverted, groundwater seepage into footing excavations through the sandy silt till deposit which has a reasonably high clay content (see Grain Size Distribution curve, Figure 5, Appendix C) should be moderate and it is believed that this seepage can be controlled by gravity drainage and pumping from strategically placed filtered sumps. An interceptor perimeter trench may also be required. It should be pointed out that the sandy silt till subgrade can be expected to be dilatant, especially in the presence of water. If the structure is placed in disturbed, dilated soil, excessive settlements could occur after the application of structural loads and backfilling. For this reason, we recommend that once dewatering becomes effective, the footing excavations be carried out in short sections (approximately 3 m in length) and concrete poured as soon as possible.

As mentioned before, disturbance and dilation of the sandy silty till bearing subgrade should be prevented. A short trial section of footing excavation at the beginning may be prudent, depending on the site conditions. It should be pointed out that relatively higher seepage could be expected to occur through the sandy gravel deposit encountered at Borehole SP1. This should be intercepted via an interceptor trench and pumping before the water emanating from this pervious soil can enter the foundation excavations.

As the new footings are expected to match the elevation of the existing footings, (i.e. they will not require deep excavations and dewatering), dewatering implemented for the construction of the new footings are not expected to cause settlements of the dense to very dense foundation soils under the existing footings.

If, however, dewatering by gravity drainage and pumping from filtered sumps does not produce adequate dewatering of the footing excavations, then vacuum well pointing may need to be resorted to. It should, however, be pointed out that the dense to very dense nature of the glacial till deposit, and the likelihood of the presence of random cobbles and boulders in the till can be expected to render the installation of well points difficult and these aspects should be taken into consideration in the design of well points.

Consideration can be given to a NSSP for proper diversion of the creek water flow and the dewatering of the foundation excavations, with responsibility assigned to the Contractor.

All bearing surfaces should be inspected and approved by a qualified Geotechnical Engineer (QVE).

It is recommended that allowance be made to pour, as directed by the Geotechnical Engineer (QVE), a 100 mm thick layer of lean concrete (mud mat) on the foundation bearing surfaces as soon as possible after excavation and approval.



## 5.8 FROST PROTECTION

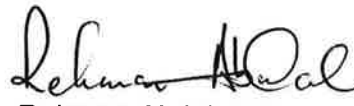
Design frost protection for the general area is 1.5 m. Therefore a permanent soil cover of 1.5 m or its thermal equivalent is required for frost protection of foundations. In case of rip rap (rock fill) only one-half of the rock fill thickness should be assumed to be effective in providing frost protection.

## 6. CLOSURE

We recommend that once the details of the proposed culvert extensions 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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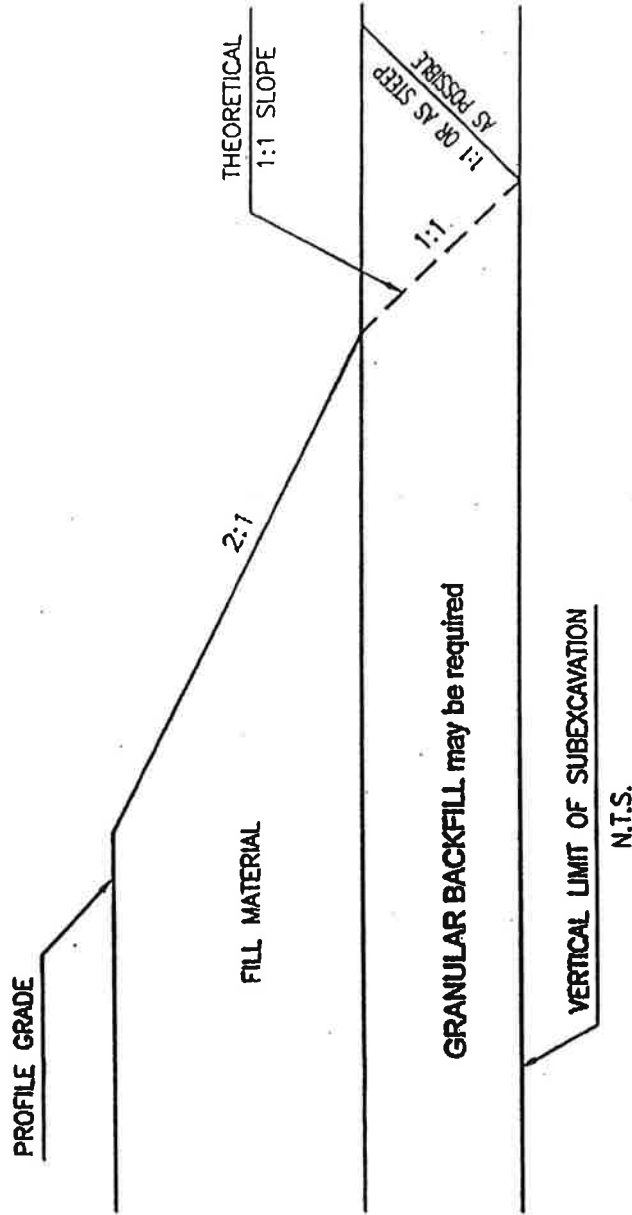
Z.S. Ozden, P.Eng.



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## Appendix E

### Removal of Unsuitable Soils From Beneath Approach Fills



REMOVAL OF UNSUITABLE SOILS  
FROM BENEATH APPROACH FILLS  
N.T.S.

# 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.