

**FOUNDATION INVESTIGATION & DESIGN REPORTS  
PROPOSED CULVERT (C9)  
AT STATION 28+374 ON HIGHWAY 6  
SOUTH OF DURHAM SOUTH TOWN LIMITS AND  
NORTH OF GREY COUNTY ROAD 9, ONTARIO  
G.W.P. 338-97-00  
SITE 8-616/C**

**GEOCRES NO. 41A-194**

**Prepared For:**

**UMA/AECOM ENGINEERING LIMITED**

**Prepared by:**

**SHAHEEN & PEAKER LIMITED**

**Project: SPT1174C  
January 15, 2008**



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## **1. INTRODUCTION**

Shaheen & Peaker Limited (S&P) was retained by UMA/AECOM Engineering Limited (UMA) to conduct a foundation investigation for detail design of the proposed culvert replacements on Highway 6 from 1.1 km south of Grey County Road 9 (North Junction) at Station 21+100 northerly through the Village of Varney to Township of Durham South Limits at Station 11+887 in Grey County, Ontario.

As part of the detail design for the proposed improvements on Highway 6, a foundation investigation was required for the detail design of the proposed replacement of Culvert C9 at Station 28+374 with a new culvert and new retaining/wing walls.

The Terms of Reference (TOR) for this investigation was outlined in the Request for Proposals (RFP) by the Ministry of Transportation (MTO) under Purchase Order Number 3004-E-0042 dated January 2005 and subsequent S&P proposal P07413. The work was performed in accordance with Consultant Agreement No. 3004-E-0042.

The purpose of this investigation was to obtain subsurface information at the site by means of exploratory boreholes. This report presents the findings of the geotechnical investigation at this site.

## **2. PHYSIOGRAPHY**

According to the Physiography of Southern Ontario (by Putnam & Chapman) and the Ontario Geological Survey Map P.2715, the study area lies in the area known as the Horseshoe Moraines. The Horseshoe Moraines has two main distinguishing features; i.e., irregular sand and gravel knobs and ridges (sand plain and kame moraine), and gravel or swamp-covered valleys. These granular deposits constitute aquifers associated primarily with kame deposits at or near the ground surface within a larger more extensive regional till plain. The existing gravel pit in Durham is part of the moraine spillway.

Existing subsurface information from Geocres database indicates that the overburden in this area primarily consisted of sand and gravel. However, south of the CPR Railway (which

runs east-west) and east of CNR Railway limestone bedrock was encountered at about El. 1127 ft (343.7m) during earlier geotechnical investigations.

According to Ontario Department of Mines Map 2039, entitled distribution of Limestone, Dolomite and Precambrian Pebbles in Gravels of Southern Ontario, the overburden (glacial drift), in this general area, is underlain by bedrock of predominately Guelph-Lockport-Amabel Formations with occasional Ancaster Chert beds. The bedrock composition generally consists of 90% dolomite, 3% limestone and 6% Pre-Cambrian rock. However, some shale and occasional gypsum and salt inclusions may also be found in the surrounding area.

Within the project limits, the grade of Highway 6 generally rises from about El. 377.4 m at Station 21+100 to about El. 386.2 m at Station 24+175, then it drops down to El. 383.7 m at Station 24+440 and generally rolls up to about El. 390.2 m at Station 24+700 and down to about El. 348.6 m at Station 10+700, and up to about El. 353.0 m at Station 10+870 (northern limit of contract), and up to El. 356.2 m at Station 11+175.

### **3. INVESTIGATION PROCEDURES**

Based on the scope of work outlined in RFP document and our proposal, the foundation field investigation for the proposed culvert replacement at Station 28+374 (C9) consisted of a total of 3 boreholes to evaluate the subsurface conditions in the areas of the proposed culvert replacement and the associated retaining/wing walls construction.

The field investigation at this site was carried out during several periods from August 16 to November 14, 2006. The field investigation consisted of drilling and sampling of 3 boreholes along the proposed culvert replacement; one at each end of the culvert and one at the crest of the embankment for culvert replacement to a maximum of 6.3 m below the ground surface.

The boreholes were advanced using solid stem, or hollow stem augers run by truck and track mounted drill rigs owned and operated by Walker Drilling Limited. All the boreholes were drilled under the full time supervision of geotechnical engineers from S&P.

Sampling in the boreholes was conducted at frequent intervals of depth by the Standard Penetration Test (SPT) method, as specified in ASTM D1586. This consists of freely dropping a 63.5 kg hammer a vertical distance of 0.76 m to drive a 51 mm 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. Refusal was generally defined by reaching competent material for which the resistance measured by the Standard Penetration Test exceeds 100 blows per 0.3 m of penetration.

Water level observations in the open boreholes were made during drilling and at the completion of each borehole. In addition, one piezometer was installed in a selected borehole. The piezometer allows monitoring of groundwater levels over time without undue interference/impact from surface water.

At the completion of drilling, all boreholes drilled were grouted and sealed using a cement/bentonite mixture. The borehole instrumented with a piezometer was sealed with bentonite seal and grout above the slotted portion of the pipes and at ground surface.

The borehole locations were measured approximately by S&P field staff with reference to the local features, which were converted to station and offset measurements. The corresponding geodetic elevations and coordinates for all the borehole were provided to us by UMA.

A laboratory testing program, consisting of natural moisture content, grain-size analyses (sieve and hydrometer), was performed on selected soil samples.

The results of drilling, in-situ testing and water level measurements, as well as laboratory soil testing are summarized on the Record of Borehole Sheets in Appendix A.

The results of the laboratory tests are also presented separately in Appendix B.

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

The soil conditions at the location of the culvert are discussed in the following sections. Details of the stratigraphy encountered in the boreholes are presented on the Record of Borehole Sheets in Appendix A and on the soil strata drawings in Drawing No. 9B in Appendix B. The following paragraphs are only meant to complement and amplify these data.

##### **4.1 CULVERT C9**

The existing culvert at Station 28+375 is a 2134 x 1524 x 24.7 m CSPA culvert with an invert at about El. 334.1 to 334.5 m.

For the proposed culvert replacement at this location, three boreholes were drilled. Boreholes C9-1 and C9-3 were put down on the west (left) and east (right) sides of the highway close to the ends of the existing culvert, respectively. Borehole C9-2 was placed on the west side (left) shoulder of the highway, as shown on Site Plan and Soil Profile on Drawing Nos. 9A and 9B.

Borehole C9-2 which was drilled from the shoulder of the highway, immediately adjacent to the existing culvert location showed the presence of a silty sand fill with traces to some gravel, to a depth of 2.2 m or El. 334.3 m.

Borehole C9-1 which was put down beyond the toe of the embankment, contacted topsoil and organic silt to a depth of 0.7 m below the ground surface or to El. 333.5 m. Similarly, Borehole C9-3, which was located beyond the toe of the embankment contacted a 0.3 m thick topsoil layer, underlain by alluvial sand and gravel to 0.7 m or El. 333.9 m.

These surficial soils in all three boreholes are underlain, below El. 334.2 to 333.5 m, by a major deposit of silty sand to sandy silt till with sand & gravel layers.

Details of the stratigraphy encountered in the boreholes are given on the individual Record of Borehole Sheets in Appendix A. Engineering properties of the soil strata and the groundwater conditions are discussed in the following paragraphs.

#### 4.1.1 FILL

Borehole C9-2 which was drilled from the top of the embankment immediately adjacent to the existing culvert contacted a basically granular (non-cohesive) fill deposit consisting of silty sand with traces to some gravel. This deposit was found to extend to a depth of 2.3 m below the shoulder of the road or to El. 334.1 m and based on N-values which range from 13 to 27 blows/0.3 m, the material appears to have received systematic compaction when it was first placed. From the N-values, the relative density of the deposit at the borehole location can be described as compact.

#### 4.1.2 TOPSOIL/ORGANIC CLAYEY SILT

Borehole C9-1 contacted a 0.7 m thick layer of alluvium consisting of topsoil and organic clayey silt. Borehole C9-3 contacted 0.3 m thick topsoil. Some organic mixture was found in Borehole C9-2 immediately below the embankment fill. The thickness of the organic soils at or near courses can however be expected to be variable.

#### 4.1.3 SILTY SAND TO SANDY SILT TILL

At depths between 0.3 m and 2.3 m the boreholes contacted (below Elevations 334.2 and 333.5 m) a relatively fine-grained glacial till deposit. The deposit consists of a heterogeneous mixture of sand and silt size particles with some gravel and traces of clay size particles. The presence of cobbles and boulders was also noted during drilling and testing of the soil. The till, which is the predominant soil type underlying the site within the depths explored, also contains some sand or sand & gravel interbeds. In Borehole C9-3, the till was found to extend to the fill depth of the boreholes at 4.6 m (El. 330.0 m) while in the other two

boreholes, it was found to be underlain by a sand & gravel deposit at depths of 3.7 m (El. 330.5 m) and 4.4 m (El. 332.0 m).

The grain-size distribution of two samples from the deposit was determined in the laboratory. As shown in Figure B9-1, the following grain-size distribution is indicated:

Gravel:	12-26%
Sand:	36-54%
Silt:	20-43%
Clay:	0- 9%

As was mentioned before, the presence of cobbles and boulders was inferred in the deposit.

The measured natural moisture contents of samples from the material ranged from typically 10 to 15%.

N-values recorded in the deposit ranged from 6 blows/0.3 m to in excess of 50 blows/0.15 m. From the test results, the deposit is considered to be compact to very dense with possible occasional loose zones (i.e. the recorded low N-value= 6 may be due to disturbance during drilling).

#### 4.1.4 SAND & GRAVEL

A sand & gravel deposit with some cobbles and occasional boulders was contacted underlying the glacial till in Boreholes C9-1 and C9-2 at depths/elevations of 3.7 m/330.5 and 4.4 m/332.0 m, respectively. The boreholes were terminated in this coarse grained granular deposit after penetrating it for a vertical distance of about 2 m at elevations 328.2 m and 330.2 m, respectively.

Standard Penetration tests performed in this deposit encountered refusal after little penetration (N-values of 50 blows/0.13 m to 100 blows/0.03 m) due to the presence of frequent cobbles and boulders. Based on these results and on the observations made during drilling, the relative density of the granular deposit is described as very dense.

The deposit was wet and water-bearing and based on a visual examination of the soil samples it is considered to be considerably more pervious than the overlying glacial till deposit.

#### 4.1.5 GROUNDWATER CONDITIONS

The groundwater conditions were observed during the drilling, upon completion of each borehole and in the piezometer installed in Borehole C9-1. The observations are given on the Record of Borehole Sheets.

During drilling the samples became wet at 0.8 to 1.5 m depth in Boreholes C9-1 and C9-3, respectively and upon completion of the boreholes, free-standing water was recorded in the boreholes at the following depths/elevations BH C9-1 at 0.6 m/333.5 m, BH C9-2 at 2.1 m/334.3 m and BH C9-3 at 1.5 m/333.0 m. In the piezometer installed in Borehole C9-1 the recorded water level rose from an initial reading of 1.4 m six days after completion to 0.2 m (El. 333.9 m) seven days thereafter. From these observations and the moisture contents of the recovered soil samples, the groundwater level at the time of our investigation was close to the o.g. level (i.e. typically within 1 m). The groundwater level would, however, be subject to seasonal fluctuations and in response to major weather events as well as the water level in the watercourse.

**SHAHEEN & PEAKER LIMITED**

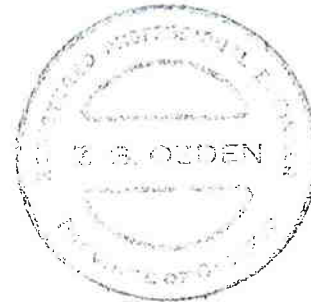
  
Ramon Miranda, P.Eng.



ZO:tr/idrive



Z.S. Ozden, P.Eng.



# Drawings

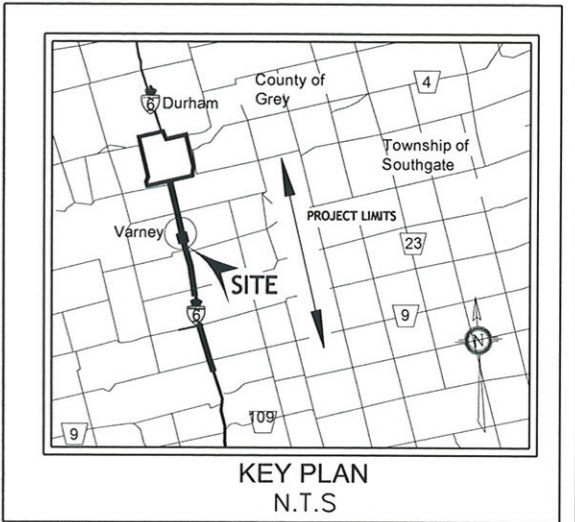
METRIC  
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES UNLESS  
OTHERWISE SHOWN. STATIONS  
ARE IN KILOMETRES + METRES.

NOTES:  
FOR DETAILED SUBSURFACE CONDITIONS  
REFER TO RECORD OF BOREHOLE SHEETS.

CONT No.  
GWP: 338-97-00

Highway 6, Durham  
Culvert C9 @ Sta. 28+375  
BOREHOLE LOCATIONS

SHAHEEN & PEAKER LIMITED



KEY PLAN  
N.T.S

LEGEND

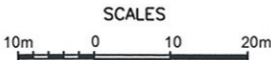
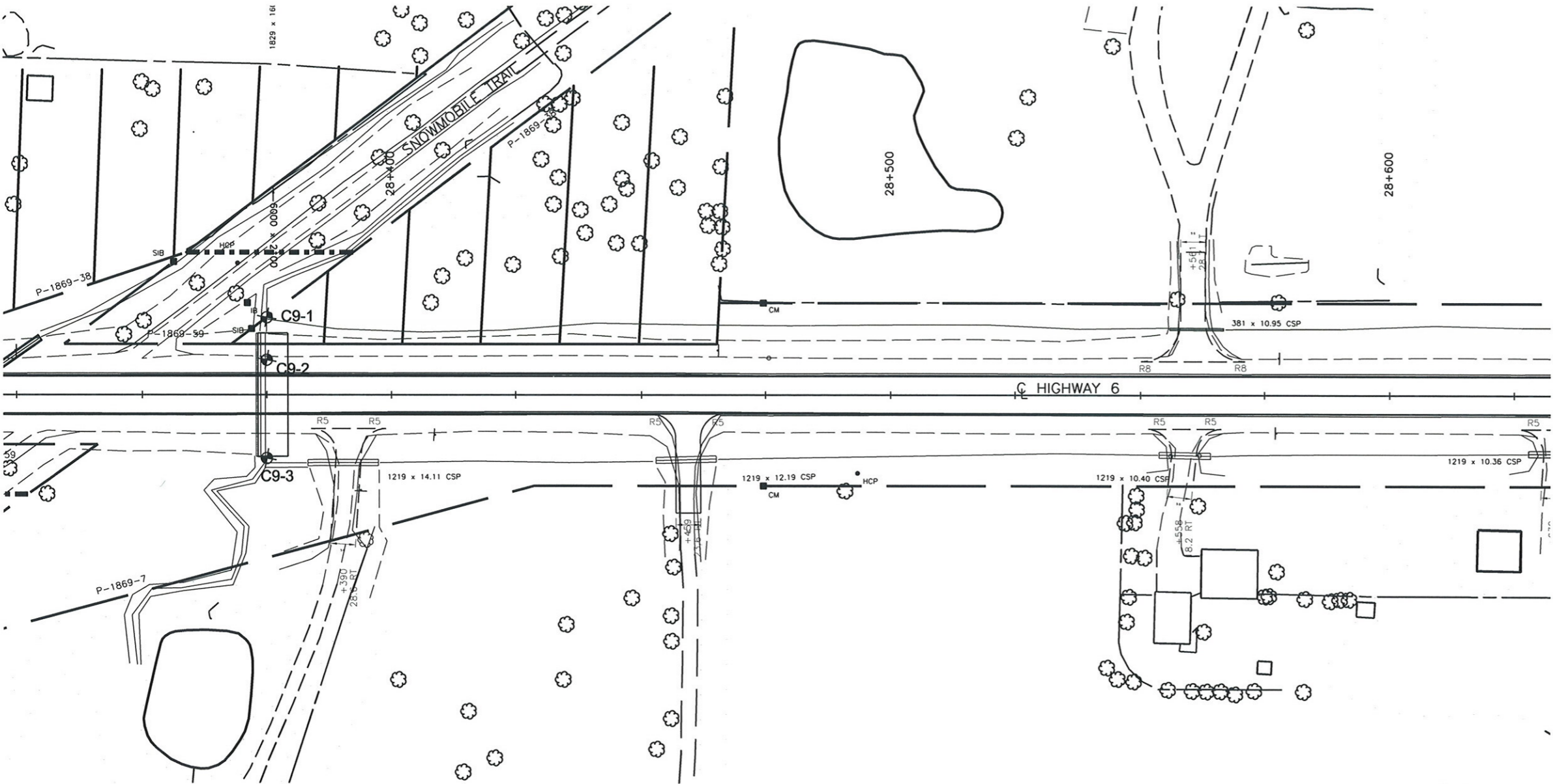
No.	ELEV.	CO-ORDINATES	
		NORTH	EAST
C9-1	334.1	4,889,464.8	199,928.9
C9-2	336.4	4,889,466.4	199,937.3
C9-3	334.5	4,889,470.0	199,956.6

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: This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

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

REV.	DATE	BY	DESCRIPTION
Geocres No. 41A-194			
SPT 1174			DIST
SUBM'D	CHECKED	DATE Jan., 2008	SITE 8-616/C
DRAWN SM	CHECKED RM	APPROVED ZO	DWG 9A



PLAN



METRIC

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

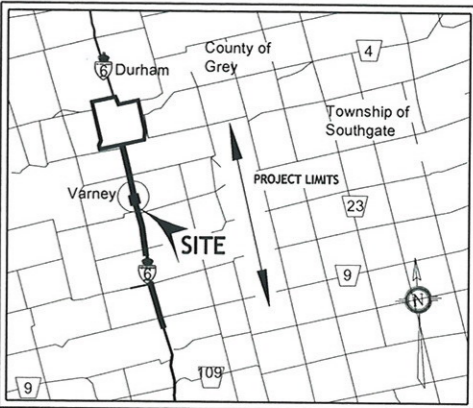
NOTES:  
FOR DETAILED SUBSURFACE CONDITIONS  
REFER TO RECORD OF BOREHOLE SHEETS.

CONT No.  
GWP: 338-97-00

Highway 6, Durham  
Culvert C9 @ Sta. 28+375  
PROFILE & SOIL STRATIGRAPHY



SHAHEEN & PEAKER LIMITED



KEY PLAN  
N.T.S

LEGEND

- Borehole
- Blows/0.3m (Std. Pen. Test, 475 J/blow)
- Water Level at Time of Investigation  
Aug./ Nov., 2006 (Not stabilized)
- Water Level in Piezometer
- Piezometer

No.	ELEV.	CO-ORDINATES	
		NORTH	EAST
C9-1	334.1	4 889 464.8	199 928.9
C9-2	336.4	4 889 466.4	199 937.3
C9-3	334.5	4 889 470.0	199 956.6

NOTE

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

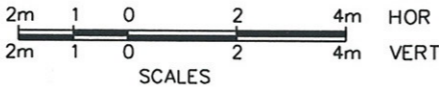
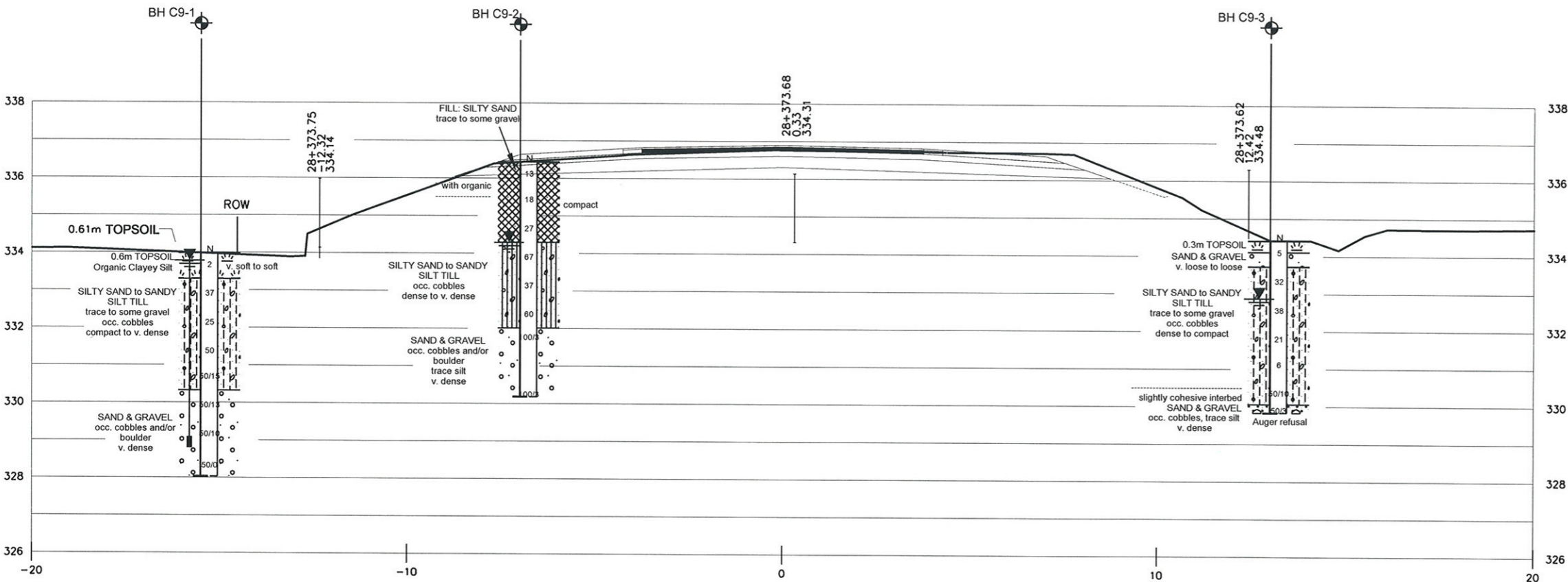
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REV.	DATE	BY	DESCRIPTION
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Geocres No. 41A-194

SPT 1174			DIST
SUBM'D ZO	CHECKED RM	DATE Jan., 2008	SITE 8-616/C
DRAWN SM	CHECKED RM	APPROVED ZO	DWG 9B



STRATIGRAPHY ALONG CULVERT C9 @STA. 28+373.68



# Appendix A

## Record of Borehole Sheets

SPT1174

# RECORD OF BOREHOLE No C9-1

1 OF 1

METRIC

GWP 338-97-00 LOCATION Hwy 6, Durham - Sta. 28+375, 15.5m Lt C/L ORIGINATED BY ZI  
 DIST HWY 6 BOREHOLE TYPE Solid Stem Augers COMPILED BY XS  
 DATUM Geodetic DATE 11/8/2006 CHECKED BY FS

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 (%) GR SA SI CL
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>			
						20 40 60 80 100										
334.1																
0.0	0.6 m <b>TOPSOIL</b> Organic Clayey Silt with rootlets dark brown, moist very soft to soft		1	SS	2											
333.5																
0.7	<b>SILTY SAND to SANDY SILT TILL</b> trace to some gravel occasional cobbles greyish brown to grey, wet compact to very dense		2	SS	37											
			3	SS	25											
			4	SS	50											
			5	SS	50/15											
330.5																
3.7	<b>SAND &amp; GRAVEL</b> some cobbles and/or boulders grey, wet very dense		6	SS	50/13											
			7	SS	50/10											
			8	SS	50/0											
328.2																
6.0	End of borehole.  Water level at 0.6 m upon completion, cave at 4.9 m. Piezometer installed to depth of 5.2 m.  Water level in piezometer: Nov. 14, 2006 ---1.4 m (El. 332.7 m) Nov. 21, 2006 ---0.2 m (El. 333.9 m)															

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity 20 15 10 5 0 (%) STRAIN AT FAILURE

SPT1174

# RECORD OF BOREHOLE No C9-2

1 OF 1

METRIC

GWP 338-97-00 LOCATION Hwy 6, Durham - Sta. 28+375, 7m Lt C/L ORIGINATED BY NH  
 DIST          HWY 6 BOREHOLE TYPE Hollow Stem Augers COMPILED BY XS  
 DATUM Geodetic DATE 8/16/2006 CHECKED BY FS

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 (%)  GR   SA   SI   CL
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				WATER CONTENT (%)				
								○ UNCONFINED      + FIELD VANE ● POCKET PENETR.      × LAB VANE				W <sub>P</sub> W      W <sub>L</sub>				
336.4						20	40	60	80	100	10	20	30			
0.0	<b>FILL: Silty Sand</b> trace to some gravel  with organic  brown, moist compact		1	SS	13									○		
			2	SS	18									○		
			3	SS	27											
334.1																
2.3	          some organics		4	SS	67									○		
	<b>SILTY SAND to SANDY SILT TILL</b> occasional cobbles greyish brown to grey, wet dense to very dense		5	SS	37									○		
			6	SS	60									○		
332.0																
4.4	<b>SAND &amp; GRAVEL</b> trace silt some cobbles and / or boulders brown, wet very dense		7	SS	100/3**									○		

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity 20 15 10 5 0 (%) STRAIN AT FAILURE


SPT1174

# RECORD OF BOREHOLE No C9-3

1 OF 1

METRIC

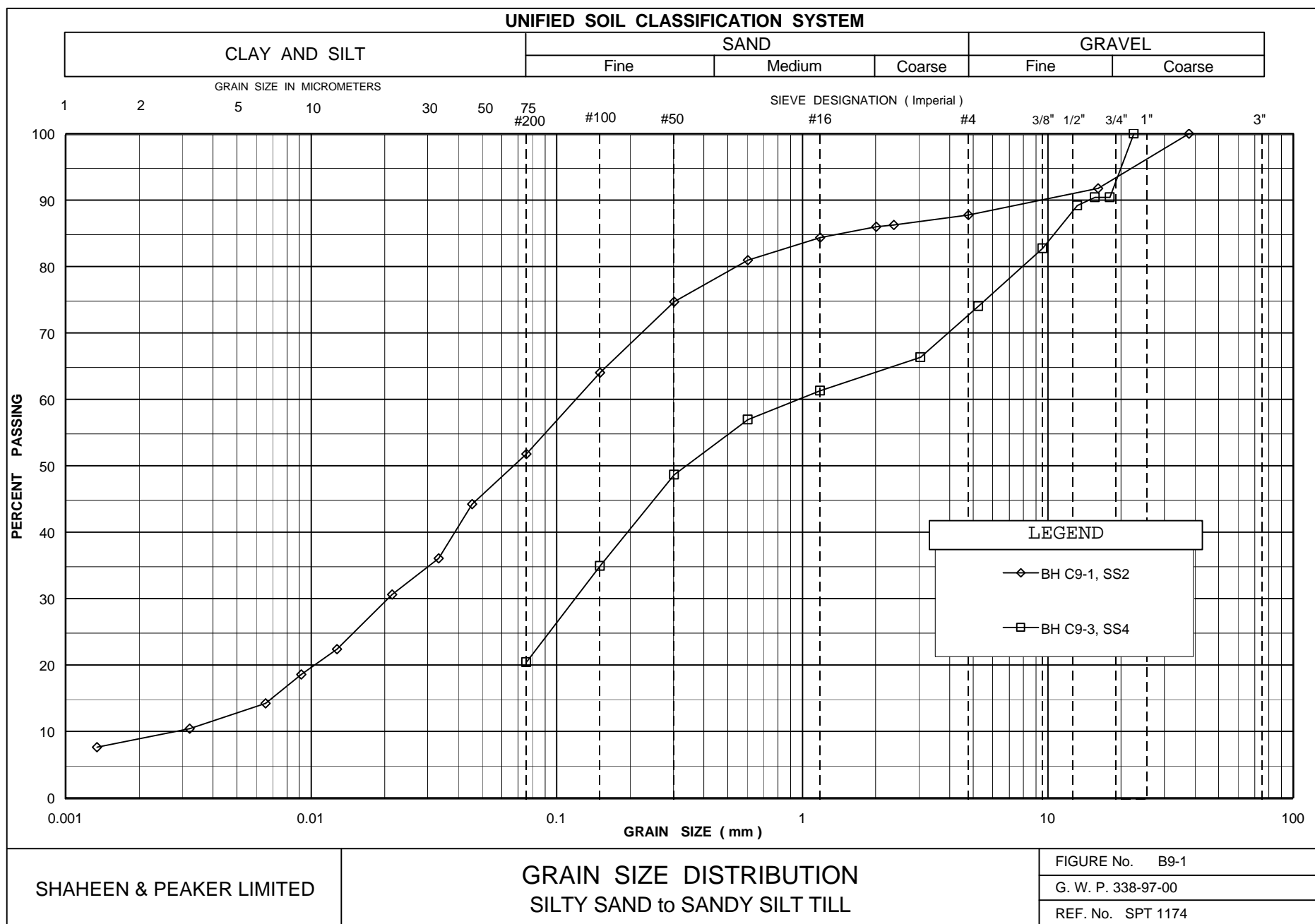
GWP 338-97-00 LOCATION Hwy 6, Durham - Sta. 28+375, 12.7m Rt. C/L ORIGINATED BY ZI  
 DIST          HWY 6 BOREHOLE TYPE Hollow Stem Augers COMPILED BY XS  
 DATUM Geodetic DATE 11/14/2006 CHECKED BY FS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL				
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						WATER CONTENT (%)			
								○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE						W P — W — W L			
334.5							20	40	60	80	100	10	20	30			
334.2	<b>SILTY SAND to SANDY SILT</b>		1	SS	5												
334.2	some organics																
0.3	<b>SILTY SAND to SANDY SILT TILL</b> trace to some gravel with frequent cobbles grey, wet		2	SS	32												
				3	SS		38										
				4	SS	21											
	dense to compact																
	loose		5	SS	6												
	very dense & bouldery		6	SS	50/10												
330.0			7	SS	50/3**												
4.6	End of borehole.  Auger refusal at 4.4 m, hole caved-in @ 4.3 m upon completion.  * Water level in open borehole at 1.5 m (El. 333.0 m) upon completion (not stabilized).																

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity 20 15 10 5 0 5 10 15 20 (%) STRAIN AT FAILURE

# Appendix B

## Laboratory Test Results



# Appendix C

## Explanation of Terms Used in Report

## EXPLANATION OF TERMS USED IN REPORT

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

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 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 STRUCUTRAL 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

**JOINT 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
$S_t$	1	SENSITIVITY = $c_u / \tau_r$

## PHYSICAL PROPERTIES OF SOIL

$P_s$	kg/m <sup>3</sup>	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	$e_{min}$	1, %	VOID RATIO IN DENSEST STATE
$j_s$	kN/m <sup>3</sup>	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	$I_D$	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
$P_w$	kg/m <sup>3</sup>	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
$j_w$	kN/m <sup>3</sup>	UNIT WEIGHT OF WATER	$s_r$	%	DEGREE OF SATURATION	$D_n$	mm	N PERCENT – DIAMETER
$P$	kg/m <sup>3</sup>	DENSITY OF SOIL	$w_L$	%	LIQUID LIMIT	$C_u$	1	UNIFORMITY COEFFICIENT
$j$	kN/m <sup>3</sup>	UNIT WEIGHT OF SOIL	$w_p$	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
$P_d$	kg/m <sup>3</sup>	DENSITY OF DRY SOIL	$w_s$	%	SHRINKAGE LIMIT	q	m <sup>3</sup> /s	RATE OF DISCHARGE
$j_d$	kN/m <sup>3</sup>	UNIT WEIGHT OF DRY SOIL	$I_p$	%	PLASTICITY INDEX = $(W_L - W_p) / I_p$	v	m/s	DISCHARGE VELOCITY
$P_{sat}$	kg/m <sup>3</sup>	DENSITY OF SATURATED SOIL	$I_L$	1	LIQUIDITY INDEX = $(W - W_p) / I_p$	i	1	HYDAULIC GRADIENT
$j_{sat}$	kN/m <sup>3</sup>	UNIT WEIGHT OF SATURATED SOIL	$I_c$	1	CONSISTENCY INDEX = $(W_L - W) / I_p$	k	m/s	HYDRAULIC CONDUCTIVITY
$P'$	kg/m <sup>3</sup>	DENSITY OF SUBMERED SOIL	$e_{max}$	1, %	VOID RATIO IN LOOSEST STATE	j	kN/m <sup>3</sup>	SEEPAGE FORCE
$j'$	kN/m <sup>3</sup>	UNIT WEIGHT OF SUBMERGED SOIL						

# Appendix D

## Site Photographs

**Foundation Investigation Report of Culvert C11 on Highway 6: GWP 338-97-00**



Photo (1): Culvert C11 at Station 11+736 on Highway 6, East End



Photo (2): Highway 6 at Station 11+736 (Culvert C11), Facing North

**FOUNDATION DESIGN REPORT  
PROPOSED CULVERT (C9)  
AT STATION 28+374 ON HIGHWAY 6  
SOUTH OF DURHAM SOUTH TOWN LIMITS AND  
NORTH OF GREY COUNTY ROAD 9, ONTARIO  
G.W.P. 338-97-00  
SITE 8-616/C**

**GEOCRES NO. 41A-194**

**Prepared For:**

**UMA/AECOM ENGINEERING LIMITED**

**Prepared by:**

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**Project: SPT1174C  
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### APPENDIX E: LIMITATIONS OF REPORT

**FOUNDATION DESIGN REPORT  
PROPOSED CULVERT (C9)  
AT STATION 28+374 ON HIGHWAY 6  
SOUTH OF DURHAM SOUTH TOWN LIMITS AND  
NORTH OF GREY COUNTY ROAD 9, ONTARIO  
G.W.P. 338-97-00  
SITE 8-616/C**

## **5. DISCUSSION AND RECOMMENDATIONS**

As part of the rehabilitation of Highway 6, Culvert (C9) at Station 28+375 will be replaced. The existing culvert is 2134 x 1524 x 24.7 m CSPC culvert. The new culvert will also be 24.7 m long with the same invert elevation as the existing culvert (i.e. El. 334.5 m at its east end and 334.1 at its west end). The new structure will however consist of a precast concrete box culvert and its inside dimensions will be 6.0 m (width) by 2.1 m (height). The top of the culvert will be at about El. 336.7 m on the east side and 336.3 m on the west side (see Drawing No. 9B).

Three boreholes were drilled at the site, namely Boreholes C9-1 and C9-3 on the west and east ends (below the embankment), while the third borehole (Borehole C9-2) was put down from the top of the embankment on the west shoulder of the highway, immediately adjacent to the existing culvert. Below some embankment fill (Borehole C9-2) and surficial organic alluvium, the boreholes show the presence of a major glacial deposit consisting of silty sand to sandy silt till with sand layers. The surface of the inorganic soil (below the embankment fill and the alluvial and organic soils) was contacted at about El. 334.0 m at Boreholes C9-2 and C9-3 and at about 333.5 m in Borehole C9-1. The groundwater table at the time of our investigation was recorded at or close to these elevations.

The undisturbed till encountered in the boreholes is considered to support both concrete box and CSP type culverts but since the crossing has been designated as a precast concrete box culvert this option will be discussed in the following sections.

### **5.1 CULVERT FOUNDATION SUPPORT**

Reference to boreholes indicates that the proposed culvert can be supported on the undisturbed competent sandy silt to silty sand till.

The recommended highest founding depths/elevation at each borehole location is tabulated below.

Table 5.1.1

Borehole No.	Existing Ground Surface Elevation (m)	Recommended Highest Founding Level (i.e. bottom of concrete slab) (m)	Elevation (m)	Subgrade Material
C9-1	334.1	0.8	333.2	compact to dense silty sand to sandy silt till
C9-2	336.4	2.5	333.9	dense to very dense silty sand to sandy silt till
C9-3	334.5	0.9	333.6	compact to dense silty sand to sandy silt till

The following geotechnical resistances are recommended for the culvert to be placed on undisturbed competent sandy silt to silty sand till.

Bearing Resistance at U.L.S. = 300 kPa  
Factored Geotechnical Resistance at S.L.S. = 200 kPa

Since the imposed loads due to the structure would be less than the existing, there should be no problems. However, an allowance of 25 mm of possible total settlement should be made due to possible rebound during construction due to stress relief. If this amount of differential settlement is acceptable between individual precast segments these settlements should not present problems, as well cambering should not be necessary.

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. Frost and scour depths may also need to be considered.

We recommend that all bearing surfaces should be inspected and approved by a qualified Geotechnical Engineer (QVE).

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

## 5.2 BEDDING

We recommend that a minimum 250 mm thick bedding material be placed beneath the concrete box culvert slab to provide uniform support. This can consist of a well-graded material such as Granular 'A'. For ease of construction, consideration may also be given to the use of 20 mm clear stone or preferably an HL4 type material. In this case (i.e. if a well-graded bedding material is not used) however, the bedding should be protected against the

migration of the silt subgrade by placing a suitable geotextile against the subgrade soil. The geotextile (OPSS 1860) should be a Class II non woven type of filter cloth with Filtering Opening Size (F.O.S.) not larger than 115 micron (such as Terraxfix 400R, or approved equivalent). We also recommend that the compatibility of the geotextile with the exposed silty subgrade be reviewed and approved during the construction.

The unfactored horizontal resistance against sliding between approved till surface and the bedding can be calculated using a friction angle of 26 degrees. The same value can be used if a geotextile is utilized in conjunction with the bedding (i.e. if a poorly grade material is used as a bedding material). It is, however, believed that sliding will not present a problem.

### 5.3 BACKFILLING

Backfilling for the culvert construction should consist of select, suitable materials, compacted in accordance with the MTO standards and conform to OPSD-803.01. For fills below the groundwater level or immediately below the roadway, it is recommended that Granular 'A' or 'B' aggregates be used. Where necessary, proper tapering as per standards should be provided. Below a depth of about 1.5 from the finished road grade, approved compactable fill, such as select subgrade materials (SSM) can be used. It is recommended that in order to minimize damage to pavement due to frost heave, materials with similar frost heave characteristics with the existing adjacent soils be used for backfill in this intermediate zone.

In any case, 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 it, care should be exercised when compacting fill adjacent to and immediately on top of the culvert structure and compaction equipment should be restricted in size as per MTO convention. The backfilling operation should be carried out simultaneously on both sides of the culvert as per MTO specifications.

Backfilling behind any retaining (wing) walls should consist of granular materials in accordance with the Ontario Ministry of Transportation Standards. Free draining backfill materials, weepholes, etc. should be provided in order to prevent hydrostatic pressure build-up.

Computation of earth pressures acting against rigid culvert walls and any wing walls should be in accordance with the Canadian Highway Bridge Design Code, (CHBDC) 2006. For design purposes, the following properties can be assumed for backfill.

**Compacted Granular 'A' or Granular 'B' Type II**

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

Unit weight = 22 kN/m<sup>3</sup>

Coefficient of Lateral Earth Pressure:

Level Backfill	Backfill Sloping at 3H:1V	Backfill Sloping at 2H:1V
$K_a=0.27$	$K_a=0.34$	$K_a=0.40$
$K_b=0.35$	$K_b=0.44$	$K_b=0.50$
$K_o=0.43$	$K_o=0.56$	$K_o=0.62$
$K^*=0.45$	$K^*=0.60$	$K^*=0.66$

**Compacted Granular 'B' Type I**

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

Unit Weight = 21 kN/m<sup>3</sup>

Coefficient of Lateral Earth Pressure:

Level Backfill	Backfill Sloping at 3H:1V	Backfill Sloping at 2H:1V
$K_a=0.33$	$K_a=0.42$	$K_a=0.54$
$K_b=0.41$	$K_b=0.52$	$K_b=0.64$
$K_o=0.50$	$K_o=0.66$	$K_o=0.76$
$K^*=0.57$	$K^*=0.74$	$K^*=0.86$

Note:

- $K_a$  is the coefficient of active earth pressure
- $K_b$  is the backfill earth pressure coefficient for an unrestrained structure including compaction efforts
- $K_o$  is the coefficient of earth pressure at rest
- $K^*$  is the earth pressure coefficient for a soil loading a fully restrained structure and includes compaction effects

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

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. The effect of compaction 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 the retaining walls should be restricted in size as per current MTO practice.

As an alternative to conventional retaining walls, consideration could be given to MTO's Retained Soil System in which case the designer will have to include the geometric, performance and appearance requirements (i.e: medium performance and low to medium appearance).

## 5.4 CONSTRUCTION

The excavation should be carried out in accordance with the Safety Regulation of the Province (i.e. Occupational Health and Safety Act O. Reg 213/91), as well as the following specifications:

SP105 S19 – Protection Systems

SP902 S01 – Excavation and Backfilling to Structures

The boreholes show that the excavations for the construction of the culvert can be expected to extend through basically granular embankment fill, underlying topsoil or other organic mixed layer into the silty sand to sandy silt till with sand or sand & gravel layers. These soils can be classified as follows:

Granular Pavement Fill	Type 2 soil (above water level)
Topsoil and Organic Rich Soil	Type 3 above water level
	Type 4 below water level
Silty sand to sandy silt till	Type 2 above water level
	Type 3 below water level
Sand or sand & gravel	Type 2 above water level

Depending on the site conditions at the time of construction, dewatering will likely be required to stabilize the soil and to prevent its dilatation. It is our opinion that the groundwater level can be lowered by up to about 0.6 m by means of gravity drainage and pumping from strategically located filtered sumps. Closely spaced deep filtered sumps may be required if deeper water level lowering is required. For more than about 0.6 to 0.8 m water lowering well points or deep wells may be required. For this reason, we recommend that, if possible, the construction be carried out during a dry period. As well, care should be taken to avoid disturbing the foundation soils by minimizing construction traffic (including foot traffic) and minimizing vibrations.

We recommend that the water flow in the existing watercourse be diverted away from the culvert so that the construction can be carried out in sufficiently dry conditions. Alternatively, the new culvert can be constructed adjacent to the existing one. The existing culvert can be used to provide site drainage during the construction and can be removed after the completion of the new culvert. Groundwater can be given to an NSSP for proper diversion of the creek water flow and the dewatering of the foundation excavations, with the responsibility assigned to the Contractor. Contractor should also be warned of the presence of cobbles and boulders encountered during the field investigation.

It is expected that temporary shoring will be required to support the excavations. Shoring system should be designed so that the lateral movement of any portion of the roadway protection system will not exceed the established criterion for the structural performance

level. In this case, the required performance level is considered 2. The coefficient of lateral earth pressures given in Table 5.1.4 can be used for the design of the temporary shoring system.

Table 5.1.4  
Recommended Unfactored Parameters for Temporary Shoring Design

Soil Type	$K_a$	$K_o$	$K_p$	Unit Weight ( $\text{kN/m}^3$ )
Granular Embankment Fill	0.30	0.45	3.3	21.5
Organic Topsoil	0.41	0.58	2.4	15.0
Silty sand to sandy silt till	0.30	0.45	3.3	21.0
Sand & gravel	0.28	0.44	3.6	21.0

## 5.5 EROSION PROTECTION

Erosion scour protection should be provided at the culvert inlet and outlet (including the side slopes). The erosion/scour protection should be designed by a specialist River Engineer/Scientist (as erosion and scour largely depend on the velocity of water in the watercourse and its regime) who is familiar with the findings of this report. The following are some general suggestions, considering that the boreholes indicate that below some surficial alluvial deposits, the main soil type consists of silty sand to sandy silt till.

We recommend that concrete cut-off (apron) and head walls be constructed both at the inlet and outlet to prevent seepage beneath and around the culvert, especially through the granular bedding and granular backfill around the culvert. Beneath the culvert, the concrete cut-off wall should extend to a suitable depth (e.g. below any possible scour depth).

In addition to cut-off and head walls, consideration may be given to erosion/scour protection at the inlet and the outlet.

At the inlet, consideration may also be given, as an alternative to concrete head walls, to the use 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 from beneath 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 0.5 m above the high water level in the watercourse down to the channel bed and up the other side in a continuous manner. 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. The clay seal should be extended at least 6 m beyond the inlet.

At the outlet as well as at the inlet (if clay seal is not used), in addition to the concrete cut-off and head walls or in conjunction with, a 0.6 m thick rock protection consisting of 300 mm size rock can be considered, overlying a 200 mm thick layer of granular filter material. This should extend at least 6 m along the channel and the sides (to at least 0.3 m above the high water level). The granular filter material underlying the rock protection should consist of a suitable granular material such as Granular 'A'. Alternatively, a suitable geotextile can be used from the rock fill, in lieu of the granular filter material.

Another reference for consideration is OPSD 810.010 Rip-Rap Treatment for Concrete Culvert Outlets.

## 5.6 FROST PROTECTION

Design frost protection for the general area is 1.6 m. Therefore, a permanent soil cover of 1.6 m or its thermal equivalent of artificial insulation is required for frost protection of foundations. In case of riprap (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 culverts and retaining walls are finalized, our recommendations be reviewed for their specific availability. The Limitations of Report, as quoted in Appendix E, are an integral part of this report.

### SHAHEEN & PEAKER LIMITED

  
Ramon Miranda, P.Eng.





Z.S. Ozden, P.Eng.



ZO:tr/idrive

# Appendix E

## Limitations of Report

## **LIMITATIONS OF REPORT**

This report is intended solely for the Client named. The material in it reflects our best judgment in light of the information available to Shaheen & Peaker Limited at the time of preparation. Unless otherwise agreed in writing by Shaheen & Peaker Limited, it shall not be used to express or imply warranty as to the fitness of the property for a particular purpose. No portion of this report may be used as a separate entity, it is written to be read in its entirety.

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.

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