FOUNDATION INVESTIGATION & DESIGN REPORTS
PROPOSED CULVERT (C5) REPLACEMENT
AT STATION 25+145 ON HIGHWAY 6
SOUTH OF DURHAM SOUTH TOWN LIMITS AND
NORTH OF GREY COUNTY ROAD 9, ONTARIO
G.W.P. 338-97-00
GEOCRES NO. 41A-199

Prepared For:

UMA/AECOM ENGINEERING LIMITED

Prepared by:

SHAHEEN & PEAKER LIMITED

Project: SPT1174B
June 6, 2008

A Coffey Geotechnics Company

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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 the proposed Highway 6 vertical realignment and culvert replacements and/or extensions from 1.1 km south of Grey County Road 9 (North Junction) at Station 21+100 through Village of Varney northerly to Township of Durham South Limits at Station 11+870 between the Counties of West Gray and North Wellington in 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 replacement of the culvert structure (C5) at Station 25+145.

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 PO7413. 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 consists 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 (344 ± m) 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 m at Station 24+175, then it drops down to El. 384 m at Station 24+440 and generally rolls up to about El. 390 m at Station 24+700 and down to about El. 349 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. 358 m at Station 11+175.

3. INVESTIGATION PROCEDURES

During the early phases of the projects, the existing Culvert C5 at Sta. 25+145 is a 0.91 x 0.91 m open bottom concrete structure was proposed to be extended to accommodate a new Northbound Passing Lane from Sta. 24+200 to 26+400. Accordingly, an extension on upstream of this culvert east of the existing highway was proposed, as well as the replacement of existing retaining walls. On that basis, four boreholes were drilled (all on the right side of the highway) for the proposed culvert extension and retaining walls. Later, it was decided to replace the structure (as well as extending it). Therefore, another borehole was drilled on the left side of the highway (for the replacement of the culvert).

The field investigation at this site was carried out during several periods from August 22, 2006 to May 16, 2007. The field investigation consisted of drilling and sampling of three boreholes for the culvert replacement/extension (Boreholes C5-1, C5-2 and C5-3) and two boreholes (Boreholes C5-RW1 and C5-RW2) for the associated retaining/wing walls, to a maximum depth of 9.6 m below the ground surface.

The initial four boreholes were drilled using solid stem or hollow stem augers run by truck and track mounted drill rigs owned and operated by Walker Drilling Limited. Borehole C5-3 which was put down in 2007 was advanced by Aardvark Drilling Inc. (for the supplementary investigation). 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, piezometer was installed in Borehole C5-3. 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. Borehole with the installed piezometer (C5-3) was sealed with bentonite above the slotted portion of the pipe 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 the boreholes were provided to us by UMA.

A laboratory testing program, consisting of natural moisture content tests and grain-size analyses, 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

From the information provided to us by UMA, the existing culvert at Sta. 25+145 is an open bottom concrete culvert, about 0.91 m high, 0.91 m wide and 22.9 m long. The invert elevation of the existing culvert is at El. 386.1 m on the upstream side and El. 385.9 m on the downstream side.

The soil conditions encountered at the location of Culvert C5 are discussed in Section 4.1 while the subsurface conditions encountered at the area of the originally proposed retaining walls are discussed in Section 4.2. 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 Drawings 5B and 5C. The following paragraphs are only meant to complement and amplify these data.

4.1 CULVERT C5 SITE

4.1.1 EMBANKMENT FILL

Borehole C5-1, drilled on the right (east) shoulder of the highway from El. 388.2 m, contacted an approximately 3.0 m thick embankment fill. The top of 0.4 m of the fill at the borehole location consists of granular pavement fill (i.e. sand & gravel). Below 0.4 m and extending to 1.7 m below the ground surface or to El. 386.5 m, the embankment fill was found to consist of sand with some gravel and silt. This is a granular (non-cohesive) material.

At 1.7 m depth (El. 386.5 m) an approximately 0.5 m thick organic clayey silt layer/pocket with some sand content was contacted to El. 386.0 m. This is a basically cohesive soil and based on an N-value of 6 blows/0.3 m, its consistency is described as firm. Due to its somewhat organic nature, it can be expected to have a relatively compressible structure.

At 2.2 m below the ground surface or at El. 386.0 m, another granular fill consisting of gravelly sand with traces of silt was contacted. This material was found to extend to 3.0 m or to El. 385.2 m. The grain-size distribution of a sample recovered from this lower fill layer was determined in the laboratory and the test yielded the following results:

Gravel	20%
Sand	72%
Silt and Clay	8%

As shown in Figure B5-1 in Appendix B, the tested material can be described as gravelly sand fill meeting the gradation requirements for Granular 'B' Type I.

A Standard Penetration test performed in the granular fill in Borehole C5-1 below 2.2 m depth yielded an N-value of 16 blows/0.3 m, indicating a compact condition.

4.1.2 TOPSOIL

Boreholes C5-2 and C5-3, which were put down from the bottom of the highway embankment, contacted a topsoil layer extending to a depth of approximately 0.2 m and 0.3m below the ground surface or to El. 386.5 and 385.6 m in Boreholes C5-2 and C5-3, respectively.

4.1.3 SANDY SILT (POSSIBLE FILL)

In Borehole C5-3, below the topsoil, a sandy silt deposit with traces of clay and gravel was encountered at a depth of 0.3 m below the ground surface. This deposit was found to extend to a depth of 3.3 m below the ground surface or to El. 382.6 m. From the presence of organics (e.g. topsoil) at a depth of 3.0 m, the deposit was identified as a possible fill related to the construction activities when the existing culvert was constructed. Alternatively, the presence of organics may be due to its alluvial origin.

Standard Penetration Test performed in this deposit yielded N-values ranging from 4 to 5 blows/0.3 m, indicating a loose to very loose condition.

The measured natural moisture contents of samples recovered from this deposit range from 10% to 38%, mostly about 10%.

4.1.4 ORGANIC SILT

Underneath the topsoil layer in Borehole C5-2 and the embankment fill in Borehole C5-1, a layer of organic silt (mixed with alluvial sand and traces to some gravel in Borehole C5-2) was encountered. The thickness of the deposit was 0.4 m in Borehole C5-1 (i.e. beneath the embankment fill) and 3.5 m in Borehole C5-2. It was found to extend to El. 384.8 m and 383.1 m, respectively. It is possible that at Borehole C5-2 location the deposit was disturbed by the construction activities when the existing culvert was first built.

This is a basically fine-grained granular (i.e. cohesionless) material.

Standard penetration tests performed in this deposit yielded N-values ranging from 7 to 17 blows/0.3m, indicating a loose to compact but generally a loose condition.

The measured moisture contents for this material ranged from 16 to 53%.

4.1.5 SAND AND GRAVEL TO SANDY GRAVEL

Underlying the organic silt in Boreholes C5-2 and C5-3 at depths/elevations of 3.7 m/El. 383.1 m and 3.3 m/El. 382.6 m, a coarse granular deposit was contacted. The composition of the deposit ranges from sand and gravel to sandy gravel with traces to some silt. It was found to extend to depths of about 6.0 m (El. 380.8 m) and 6.9 m (El. 379.0 m) in Boreholes C5-2 and C5-3, respectively.

Grain-size analysis performed on two samples from Boreholes C5-2 and C5-3 yielded the following grain-size distribution (see Figures B5-2 and B5-3 in Appendix B).

Gravel	44 – 57%
Sand,	32 – 38%
Silt & Clay	11 – 18%

Standard Penetration tests performed in this deposit yielded N-values ranging from 3 to 55 blows/0.3 m, indicating a very loose condition near top and very dense condition for rest of this layer in Borehole C5-2 and dense to very dense condition at the top portion and compact condition at the bottom portion of the stratum in Borehole C5-3.

The measured natural moisture contents of this granular deposit range from 5 to 16%.

4.1.6 SILTY FINE SAND

Below the embankment fill and organic silt in Borehole C5-1, a deposit of silty fine sand was encountered at a depth of 3.4 m below the ground surface or at El. 384.8 m. The deposit was found to extend to a depth of 8.6 m below the ground surface or to El. 379.6 m. Its colour was noted to dark brown to brown and from this it is surmised that this is likely to be an alluvial soil containing traces of organic soil.

Standard Penetration tests performed in this deposit yielded N-values ranging from 5 to 7 blows/0.3 m in the upper 4 ± m of the deposit (i.e. to about El. 381 m), indicating a loose condition. Below this depth an N-value of 62 blows/0.3 m was recorded which indicates a very dense relative density.

4.1.7 SANDY SILT

Underlying the surficial soils mentioned in the preceding paragraphs, all three boreholes contacted a sandy silt material. This deposit was contacted at depths/elevations of 8.6 m/El.379.6 m (Borehole C5-1), 6.0 m/El. 380.8 m (Borehole C5-2) and 6.9 m/El. 379.0 m (Borehole C5-3) and extended to the termination of the boreholes.

This is a fine-grained granular (non-cohesive) material and Standard Penetration tests performed in the deposit yielded N-values ranging from 39 to in excess of 80 blows/0.3 m penetration, indicating a dense to very dense condition.

The measured natural moisture contents of this stratum range from 17 to 21%.

4.1.8 GROUNDWATER CONDITIONS

Groundwater levels in the open boreholes were observed during the drilling and at the completion of each borehole. In addition a piezometer installed in Borehole C5-3 to allow ground water monitoring over a prolonged period of time, without interference from surface water. The observations and recorded values are shown on the individual Record of Boreholes sheets presented in Appendix A.

The results indicate that free-standing water was found in the open boreholes upon their completion at depths/elevations of 7.6 m/El. 380.6 m and 6.3 m/380.4 m, Boreholes C5-1 and C5-2, respectively. These values may, however, not represent the stabilized groundwater levels, due to the fact that the boreholes were backfilled upon their completion (i.e. possible insufficient time for the water level to stabilize). The groundwater level in the piezometer installed in Borehole C5-3 was recorded at a depth of 5.1 m or at El. 380.8 m about two days after the installation. Based on these observations, it is our opinion that the groundwater level at the site was at about El. 381 m at the time of our investigation.

It should also be pointed out that the groundwater is subject to seasonal fluctuations and fluctuations in response to major weather events. In addition, the water table at the site will be influenced by the water level in the water course.

4.2 ORIGINALLY PROPOSED RETAINING WALL BOREHOLES

In addition to the culvert boreholes (described above), two boreholes (C5-RW1 and C5-RW2) were drilled to a maximum of 6.7 m depth on the right side of the highway at the locations shown on the Borehole Location Plan Drawing No. 5A. These boreholes were advanced to evaluate the subsurface conditions in the area of the originally proposed retaining walls.

Boreholes C5-RW1 and C5-RW2, which were located just outside the primary floodplain (El. 387.6 and 386.7 m, respectively), encountered a surficial topsoil underlain by an essentially granular deposit consisting of sandy silt to silty sand till with occasional cobbles and boulders. This deposit is in turn underlain by a gravelly sand till which is in turn underlain by silty fine sand, followed by gravelly sand in Borehole C5-RW1.

4.2.1 TOPSOIL

At the location of Boreholes C5-RW1 and C5-RW2, the ground surface was covered by about 0.15 m of topsoil.

4.2.2 SANDY SILT TO SILTY SAND TILL

Below the topsoil in Boreholes C5-RW1 and C5-RW2, a sandy silt to silty sand till with occasional cobbles and boulders was encountered; extending to a depths of 2.2 m and 4.4 m respectively or to El. 385.4 m to El. 382.4 m.

Grain-size analyses performed on two samples from the deposit yielded the following grain-size distribution.

Gravel	10 - 20%
Sand,	40 - 41%
Silt & Clay	39 - 50%

as shown in Figure B5-4, in Appendix B.

Standard Penetration tests performed in this basically granular (i.e. non-cohesive) deposit yielded N-values ranging from 26 blows/0.3 m to 50 blows/0.08 m penetration, indicating compact to very dense but generally very dense condition.

4.2.3 GRAVELLY SAND TILL

Below topsoil and sandy silt to silty sand till in Boreholes C5-RW1 and C5-RW2, a coarser glacial till deposit was contacted at 2.2 m and 4.4 m below ground surface and extending to depths of about 4.5 m and 6.3 m (end of the borehole) in Boreholes C5-RW1 and C5-RW2 respectively or to El. 383.1 m and El. 380.5 m (end of the borehole), respectively.

This deposit is cohesionless (i.e. granular) soil. During the drilling, the presence of occasional cobbles and boulders was inferred in the deposit.

Standard Penetration tests performed in this coarse grained glacial deposit yielded N-values ranging from 68 blows/0.23 to 50 blows/0.08 m penetration, indicating a very dense condition.

The measured natural moisture contents samples recovered from this deposit generally range from 3 to 8%.

4.2.4 SILTY FINE SAND

Below the glacial deposits, at the location of Borehole C5-RW1, a silty fine sand layer (about 0.8 m in thickness) was contacted at about 4.5 m depth (El. 383.1 m) and extended to 5.3 m or to El. 382.4 m.

A Standard Penetration test performed in this deposit yielded an N-value of 9 blows/0.3 m penetration, indicating loose condition.

4.2.5 GRAVELLY SAND

Below the silty fine sand in Borehole C5-RW1, a gravelly sand deposit was encountered at 5.3 m depth (El. 382.4 m), extending to the termination of this borehole at a depth of 6.7 m (El. 380.9 m).

The measured N-values in this deposit are 33 blows/0.3 m and 50 blows/0.03 m penetration, indicating a compact to very dense condition.

The measured natural moisture contents range from 4 to 8%.

4.2.6 GROUNDWATER CONDITIONS

Groundwater levels in the open boreholes were observed during the drilling and at the completion of each borehole. The observations and recorded values are shown on the individual Record of Borehole sheets.

Both boreholes were dry upon their completion. This may, however, not represent the stabilized groundwater conditions.

It should also be pointed out that the groundwater is subject to seasonal fluctuations and fluctuations in response to major weather events. In addition, the water table at the site will be influenced by the water level in the water course.

SHAHEEN & PEAKER LIMITED

Ramon Miranda, P.Eng.

LICENSED PROFESSIONAL ENGINEER
RAMON MIRANDA
PROVINCE OF ONTARIO

Z.S. Ozden, P.Eng.

REGISTERED PROFESSIONAL ENGINEER
Z. S. OZDEN
PROVINCE OF ONTARIO

Drawings

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

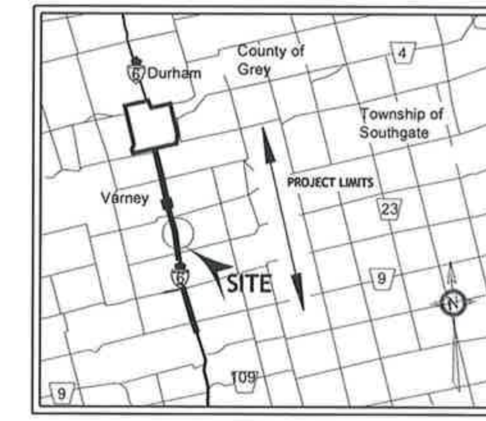
METRIC

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

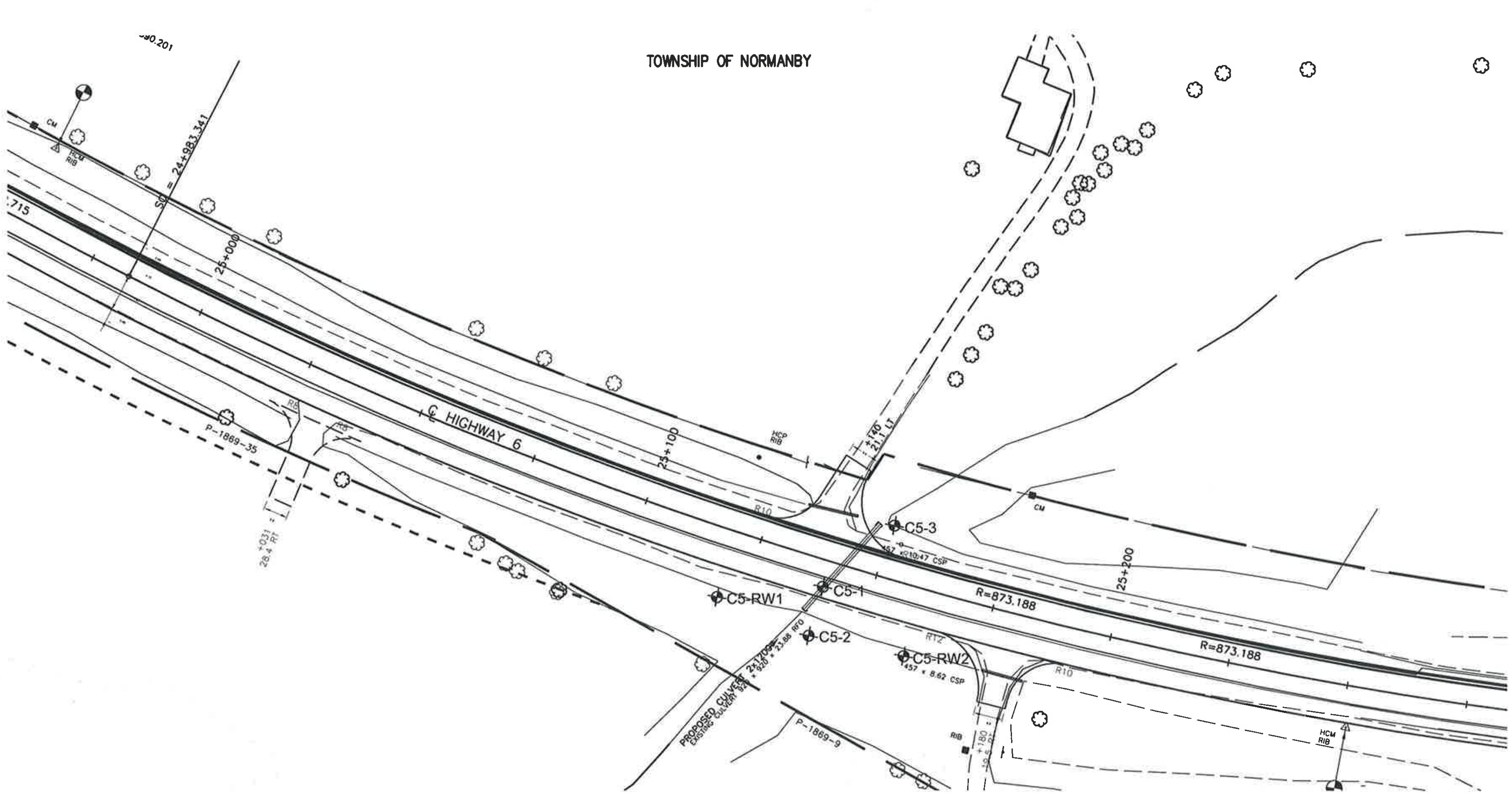
CONT No.
GWP: 338-97-00

Highway 6, Durham
Culvert C5 @ Sta. 25+145
Borehole Location Plan

SHAHEEN & PEAKER LIMITED



KEY PLAN
N.T.S



SCALE
10m 0 10 20m

BOREHOLE LOCATION PLAN

LEGEND

Borehole

No.	ELEV.	CO-ORDINATES NORTH	CO-ORDINATES EAST
C5-1	388.2	4 886 284.1	200 505.5
C5-2	386.7	4 886 283.0	200 516.0
C5-RW1	387.6	4 886 263.0	200 511.5
C5-RW2	386.7	4 886 303.1	200 516.5
C5-3	385.9	4 886 296.3	200 490.4

NOTES

- The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries assumed from geological evidence.
- This drawing is for subsurface information only. Surface details and features are for conceptual illustration.
- 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-199

SPT 1174			DIST
SUBM'D	CHECKED	DATE Jan., 2008	SITE
DRAWN SM	CHECKED RM	APPROVED ZO	DWG 5A

REGISTERED PROFESSIONAL ENGINEER
Z. S. OZDEN
June 6/08
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
RAMON MIRANDA
June 6/08
PROVINCE OF ONTARIO

METRIC

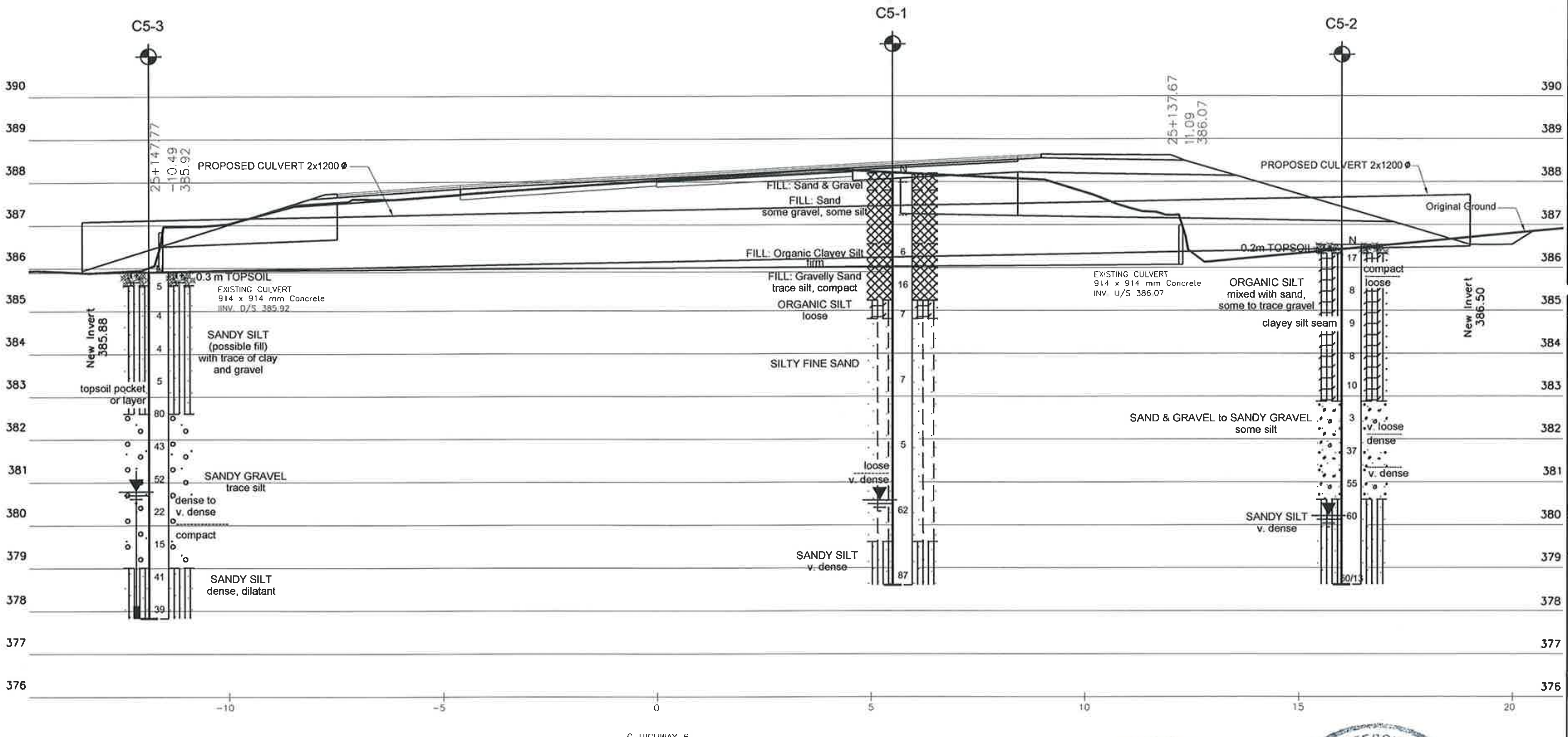
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES + METRES.

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

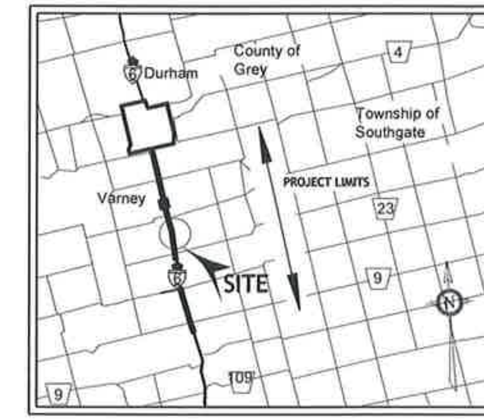
CONT No.
GWP: 338-97-00

Highway 6, Durham
Culvert C5 @ Sta. 25+145
SOIL STRATIGRAPHY

SHAHEEN & PEAKER LIMITED



PROFILE ALONG CULVERT C5 @ STA. 25+140



KEY PLAN
N.T.S

LEGEND

- Borehole
- N Blows/0.3m (Std. Pen. Test, 475 J/blow)
- Water Level at Time of Investigation (W. L. NOT STABILIZED)
- Water Level in Piezometer
- Piezometer

No.	ELEV.	CO-ORDINATES NORTH	EAST
C5-1	388.2	4 886 284.0	200 505.6
C5-2	386.7	4 886 283.0	200 516.0
C5-3	385.9	4 886 296.3	200 490.4

=NOTE=

- The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries assumed from geological evidence.
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REGISTERED PROFESSIONAL ENGINEER Z. S. OZDEN PROVINCE OF ONTARIO Jun 6/08

LICENSED PROFESSIONAL ENGINEER RAMON MIRANDA PROVINCE OF ONTARIO June 6/08

REV.	DATE	BY	DESCRIPTION

Geocres No. 41A-199

SPT 1174			DIST
SUBM'D	CHECKED	DATE Jan., 2008	SITE
DRAWN SM	CHECKED RM	APPROVED ZO	DWG 5B

Appendix A

Record of Borehole Sheets

RECORD OF BOREHOLE No C5-1

METRIC

PROJECT: SPT1174
LOCATION: Hwy 6, Durham - Sta. 25+145, 5.5m Rt C/L
ORIGINATED BY: JL
BOREHOLE TYPE: Hollow Stem Augers
DATE: 8/22/2006
COMPILED BY: XS
DATUM: Geodetic
CHECKED BY: FS

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
388.2	Ground Surface						
0.0 / 387.8 (0.4)	**FILL: Sand & Gravel** brown, damp	1	AS	---		388	
0.4 / 386.5 (1.7)	**FILL: Sand** some gravel, trace to some silt, damp	2	AS	---		387	
1.7 / 386.0 (2.2)	**FILL: Organic Clayey Silt** some fine sand dark brown, moist, firm	3	SS	6		386	
2.2 / 385.2 (3.0)	**FILL: Gravelly Sand** trace silt brown, damp, compact	4	SS	16			20 72 (8)
3.0 / 384.8 (3.4)	**ORGANIC SILT** black, moist, loose	5	SS	7		385	
3.4 / 379.6 (8.6)	**SILTY FINE SAND** dark brown; moist, loose; ---; very dense wet	6	SS	7		384, 383	
		7	SS	5		382, 381	
		8	SS	62	▼*	380	
8.6 / 378.6 (9.6)	**SANDY SILT** brown, wet very dense	9	SS	87		379	
9.6	End of borehole.						

* Water level in open borehole at 7.6m (El. 380.6m) upon completion (not stabilized).

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100
PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30
UNIT WEIGHT γ kN/m^3

$+^3, \times^3$: Numbers refer to Sensitivity   ○ $^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No C5-2

METRIC

PROJECT SPT1174
LOCATION Hwy 6, Durham - Sta. 25+140, 16m Rt, C/L
ORIGINATED BY JL
BOREHOLE TYPE Hollow Stem Augers
DATE 10/3/2006
COMPILED BY HL
DATUM Geodetic
CHECKED BY FS

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
386.7	Ground Surface						
386.5 / 0.2	0.2m **TOPSOIL**						
	compact	1	SS	17			
	loose					386	
	ORGANIC SILT mixed with sand, some to trace gravel, black to dark brown, moist	2	SS	8			
	clayey silt seam	3	SS	9		385	
		4	SS	8		384	
	compact	5	SS	10			
383.1 / 3.7	**SAND & GRAVEL TO SANDY GRAVEL** some silt, brown, moist to wet					383	
	very loose	6	SS	3			combined SS6 + SS7
	dense	7	SS	37		382	44 38 (18)
	very dense	8	SS	55		381	
380.8 / 6.0	**SANDY SILT** oxidised brown, moist to wet very dense	9	SS	60	▼*		
						380	
378.8 / 7.9		10	SS	50/13		379	

End of borehole.

* Water level in open borehole at 6.3m (El. 380.4 m) upon completion (not stabilized).

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30 (sample 5 value: 53)

UNIT WEIGHT γ kN/m^3

$+^3, \times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No C5-3

METRIC

PROJECT SPT1174

LOCATION Hwy 6, Durham - Sta. 25+151, 10.7 m Lt C/L

ORIGINATED BY NE

BOREHOLE TYPE Hollow Stem Augers

DATE 5/16/2007

COMPILED BY XS

DATUM Geodetic

CHECKED BY NE

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
385.9	Ground Surface					
385.6 / 0.0	0.3 m **TOPSOIL**, trace gravel	1	SS	5		
0.3	**SANDY SILT** (possible fill) with trace of clay and gravel brown, loose to very loose	2	SS	4	385	
		3	SS	4	384	
	topsoil pocket or layer	4	SS	5	383	
382.6 / 3.3	**SANDY GRAVEL** trace silt, brown; damp, dense to very dense	5	SS	80	382	
		6	SS	43		57 32 9 2
		7	SS	52	381	
	compact, wet	8	SS	22	380	spoon wet @ 5.5 m
		9	SS	15		
379.0 / 6.9	**SANDY SILT** brown, dense, wet, dilatant	10	SS	41	379	
377.8		11	SS	39	378	
8.1	End of borehole. Monitoring well installed to depth of 8.1 m. Water level inwell: May 18, 2007 --- 5.1 m (El. 380.8 m).					

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

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

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

WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

+3, ×3: Numbers refer to Sensitivity

O 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No C5-RW1

METRIC

PROJECT: SPT1174
LOCATION: Hwy 6, Durham - Sta. 25+120, 14m Rt C/L
ORIGINATED BY: JL
BOREHOLE TYPE: Solid Stem Augers
DATE: 10/3/2006
COMPILED BY: XS
DATUM: Geodetic
CHECKED BY: FS

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
387.6	Ground Surface					
387.5 / 0.2	0.15m **TOPSOIL**					
	SANDY SILT to SILTY SAND TILL brown, moist; compact / very dense	1	SS	28	387	
		2	SS	27	386	
		3	SS	63		10 40 (50) possible boulder
385.4 / 2.2	**GRAVELLY SAND TILL** occasional cobble, brown, moist to damp, very dense	4	SS	50/13	385	
		5	SS	72/23	384	
		6	SS	68/23		
383.1 / 4.5	**SILTY FINE SAND** trace gravel, moist, loose	7	SS	9	383	
382.4 / 5.3	**GRAVELLY SAND** trace to some silt, brown, moist, very dense to dense	8	SS	50/3	382	
		9	SS	33		
380.9 / 6.7	End of borehole. Borehole dry upon completion.				381	

Column headings also include: STRAT PLOT; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L, WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³.

+ 3, × 3: Numbers refer to Sensitivity   ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No C5-RW2

METRIC

PROJECT SPT1174

LOCATION Hwy 6, Durham - Sta. 25+160, 14.5m Rt C/L

ORIGINATED BY JL

BOREHOLE TYPE Solid Stem Augers

DATE 10/3/2006

COMPILED BY HL

DATUM Geodetic

CHECKED BY FS

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
386.7	Ground Surface					
386.6 / 0.0 / 0.2	0.15m **TOPSOIL**					
	SANDY SILT to SILTY SAND TILL occasional cobble / boulder brown, moist; compact / very dense	1	SS	26		
		2	SS	86/23cm	386	
		3	SS	50/13cm	385	combined SS3 + SS4
		4	SS	50/13cm	384	20 41 (39)
		5	SS	50/8cm	383	
		6	SS	50/13cm		
382.4 / 4.4	**GRAVELLY SAND TILL** some pebbles, occasional cobble / boulder brown, oxidised, moist very dense	7	SS	50/13cm	382	
		8	SS	50/15cm	381	
380.5 / 6.3		9	SS	50/10cm		
	End of borehole. Borehole dry upon completion.					

Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³.

$+^3, \times^3$: Numbers refer to Sensitivity ○ $^{3\%}$ STRAIN AT FAILURE

Appendix B

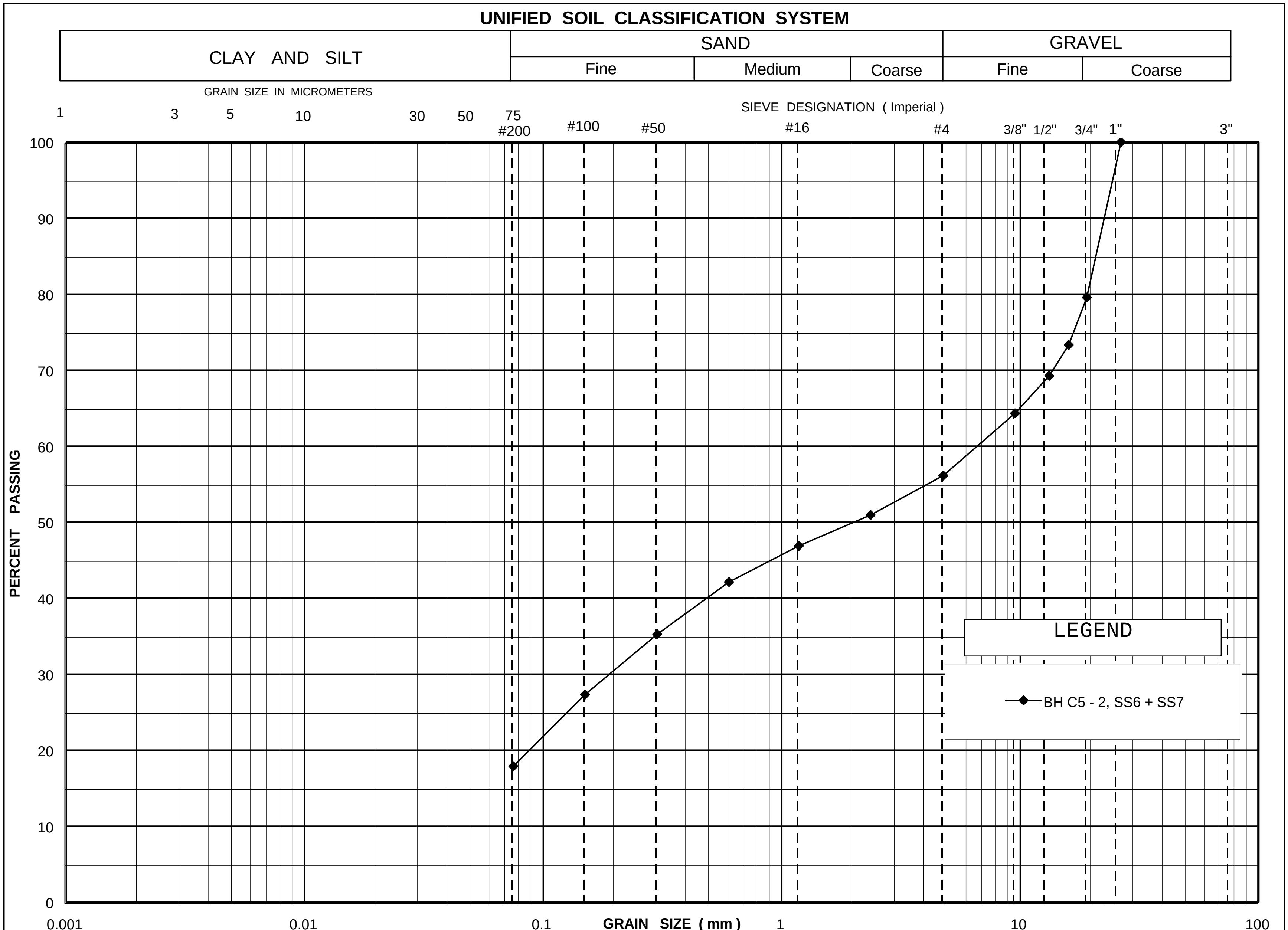
Laboratory Test Results

SHAHEEN & PEAKER LIMITED

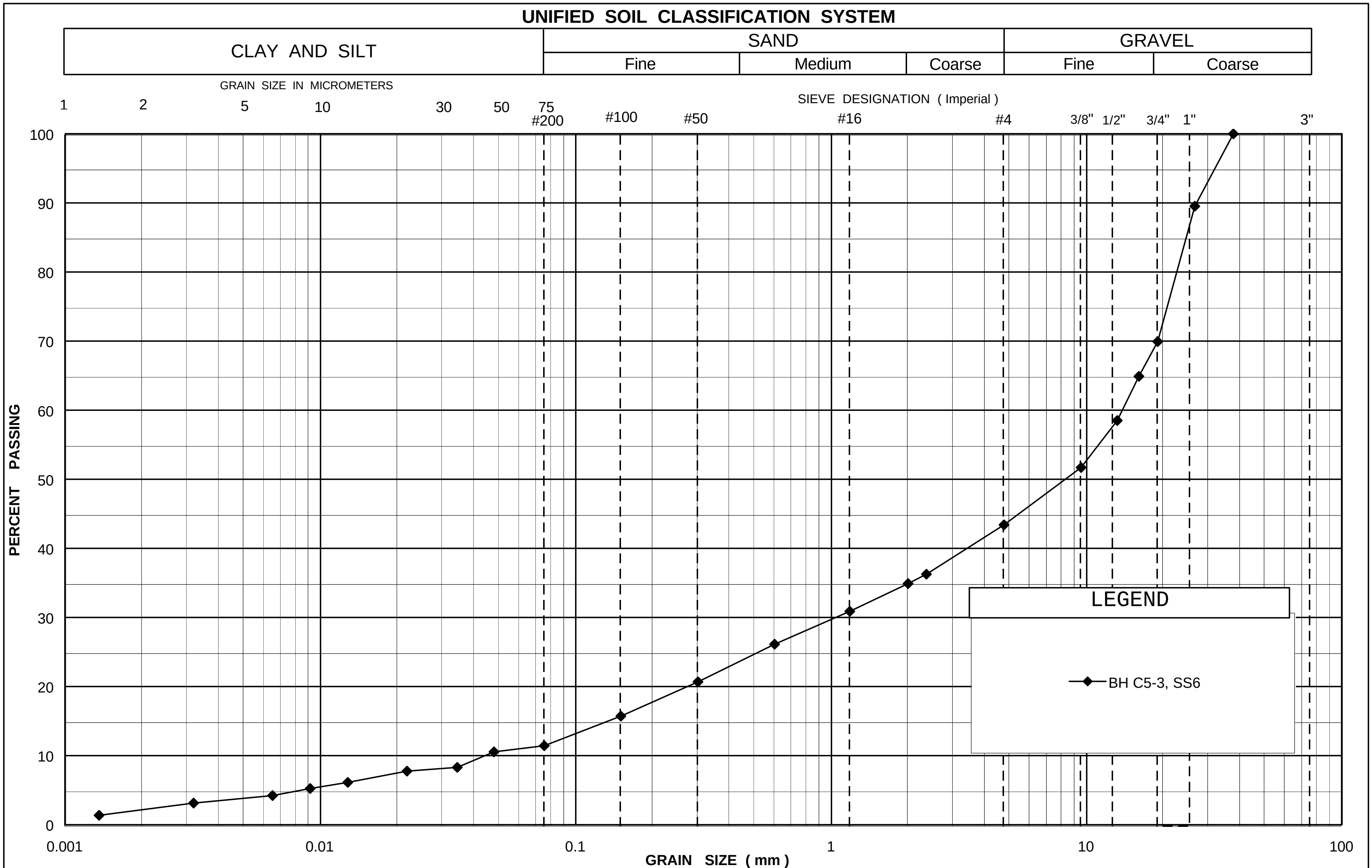
GRAIN SIZE DISTRIBUTION

GRAVELLY SAND FILL meeting the gradation requirements for Granular 'B'

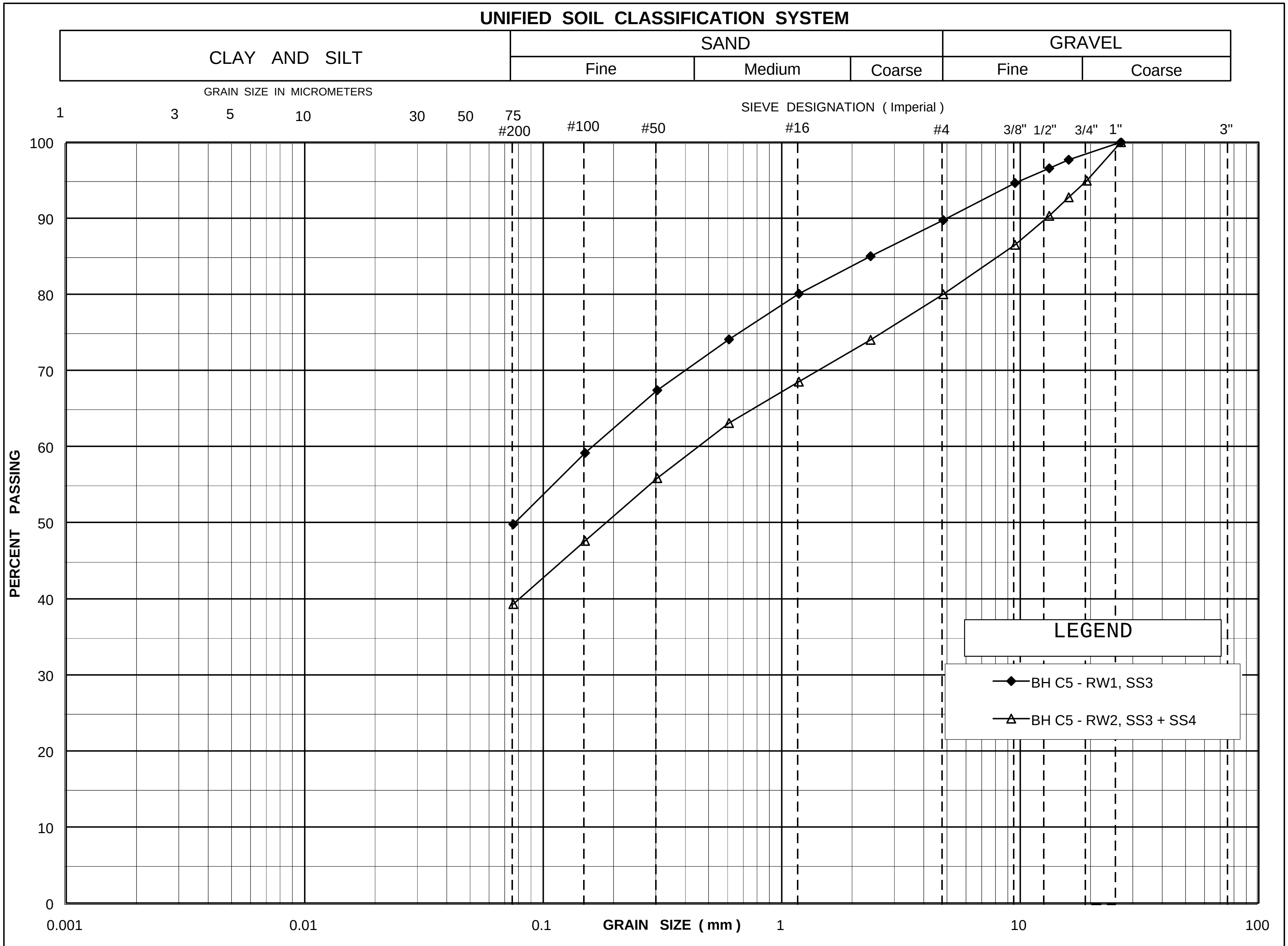
FIGURE No.	B5-1
G. W. P. 338-97-00	
REF. No.	SPT 1174



SHAHEEN & PEAKER LIMITED	GRAIN SIZE DISTRIBUTION SAND & GRAVEL	FIGURE No. B5-2 G. W. P. 338-97-00 REF. No. SPT 1174



SHAHEEN & PEAKER LIMITED	GRAIN SIZE DISTRIBUTION Sandy Gravel Trace Silt, Trace Clay	FIGURE No. B5-3 G. W. P. 338-97-00 REF. No. SPT 1174



SHAHEEN & PEAKER LIMITED

GRAIN SIZE DISTRIBUTION

SANDY SILT to SILTY SAND TILL

FIGURE No. B5-3A

G. W. P. 338-97-00

REF. No. SPT 1174

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 $\overline{N}$.

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 COMPOSION 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
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_l, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ε	%	LINEAR STRAIN
$\varepsilon_1, \varepsilon_2, \varepsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICALL PROPERTIES OF SOIL

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

PHYSICAL PROPERTIES OF SOIL

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

FOUNDATION DESIGN REPORT
PROPOSED CULVERT (C5) REPLACEMENT
AT STATION 25+145 ON HIGHWAY 6
SOUTH OF DURHAM SOUTH TOWN LIMITS AND
NORTH OF GREY COUNTY ROAD 9, ONTARIO
G.W.P. 338-97-00
GEOCRES NO. 41A-199

Prepared For:

UMA/AECOM ENGINEERING LIMITED

Prepared by:

SHAHEEN & PEAKER LIMITED

Project: SPT1174B
June 6, 2008

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FOUNDATION DESIGN REPORT
PROPOSED CULVERT (C5) REPLACEMENT
AT STATION 25+145 ON HIGHWAY 6
SOUTH OF DURHAM SOUTH TOWN LIMITS AND
NORTH OF GREY COUNTY ROAD 9, ONTARIO
G.W.P. 338-97-00

5. DISCUSSION AND RECOMMENDATIONS

5.1 CULVERT REPLACEMENT

Based on the latest information provided by UMA, the existing 0.91 x 0.91 m x 22.9 m long open bottom concrete culvert at Sta. 25+145 will be replaced with twin 1.2 m diameter, 33 m long, CSP culverts (no new headwall/retaining wall). The invert elevation of the new culverts will be at about El. 386.5 m on the upstream (which is at the extension side) and El. 385.9 m on the downstream side (which will match the existing).

One of the new culverts will be located adjacent to the existing culvert and the other one will be at the existing culvert location. It is anticipated that the existing culvert will provide drainage during the construction of one of the culverts adjacent to it, after which the existing culvert will be removed and replaced with the second of the new twin culverts. Virtually no grade changes are anticipated in the roadway over the culvert, except in the area of the proposed embankment widening on the right side of the highway where a new northbound passing lane will be constructed. Here, the grade will be raised by up to about 2.5 m, above the existing o.g. (original grade) level.

Three boreholes were drilled at the site, namely Boreholes C5-2 and C5-3 on the east and west ends (near the toe of the embankment), while the third borehole (Borehole C5-1) was put down from the top of the embankment on the east shoulder of the highway, immediately adjacent to the existing culvert.

Below some 3.0 m thick embankment fill and an underlying 0.4 m thick layer of organic silt in Borehole C5-1, and topsoil and/or alluvial soils in Boreholes C5-2 and C5-3, the boreholes show the presence of silty fine sand (Borehole C5-1), sand & gravel to sandy gravel in Boreholes C5-2 and C5-3. The surface of the silty fine sand was contacted at about El. 384.8 m in Borehole C5-1 and the surface of sand & gravel to sandy gravel was contacted at about El. 383.1 m and 382.6 m in Boreholes C5-2 and C5-3, respectively. These deposits are underlain by sandy silt in all three boreholes, below El. 380.8 – 379.0 m.

The groundwater level at the time of our investigation was found at about El. 381 m upon completion of drilling. These recorded water levels in open boreholes may not present the stabilized groundwater levels and, as well, the water level at the site could be subject to fluctuations due to major weather events and seasonal variations. Depending on the water level in the existing watercourse, the groundwater table may rise close to the ground surface (o.g.) level.

The boreholes show that at the proposed invert elevation of 386.5 to 385.9 m, the subsoil conditions at the borehole locations are unsuitable to support the proposed twin culverts. As well at greater depths the soil conditions and their compactness condition are quite variable. For these reasons, either the use of deep foundations or the removal and replacement of the unsuitable soils with engineered fill will be required.

In our opinion, the engineered fill option would provide a more cost-effective solution. In this case, the use of CSP culverts (as proposed by UMA) would be a better choice in comparison with more rigid concrete structures, since CSP's would be more flexible and therefore less sensitive to total and differential settlements.

5.1.1 CULVERT FOUNDATION SUPPORT

The native inorganic silty fine sand and sand & gravel to sandy gravel, in their undisturbed state, are suitable to support the proposed CSP culverts. Geotechnical resistances of the order of Factored Bearing Resistance at ULS of 100 to 800 kPa and Bearing Resistances SLS equal to 50 to 500 kPa are available in these deposits, depending on the selected foundation level. The foundation resistances required for the proposed CSPs are, however, expected to be relatively small (presumably less than 50 kPa), under the existing embankment. However, somewhat higher resistances (i.e. somewhat higher than 50 kPa would be required) on the extension right side, since the grade raise will be up to about 2.5 m above the o.g. level, due to the proposed widening (i.e. o.g. El. 386 ± vs. finished pavement El. 388.5± m).

A possible foundation alternative would be the removal of all unsuitable soils to the surface of the sufficiently competent inorganic deposits and to replace them with engineered fill.

This alternative would involve significant sub-excavation of organic silt and the sandy silt fill material (up to about 3 to 4 m depth at the location of Boreholes C5-2 and C5-3) and subsequent backfilling with engineered fill up to the desired invert elevation for the new culverts. The anticipated soil removal/replacement elevations at the borehole locations are 384.8 m, 383.1 m, and 382.6 m at Borehole C5-1, C5-2 and C5-3, respectively (i.e. to the surface of the undisturbed natural inorganic soil).

Depending on the water level in the watercourse, the groundwater level at the site may be at or near the o.g. level and therefore, the sub-excavation and placement of the engineered fill may be below the groundwater level. If this happens, (i.e. if excavation is to be carried out below the groundwater level), extensive dewatering will be required to facilitate the excavation and the subsequent placement of the engineered fill. This would be further complicated by the fact that, if the existing culvert is to be used to maintain the flow in the watercourse, one-half of the excavation and the engineered fill will need to be carried out first. After the construction of the first of the twin culvert CSPs, the existing culvert would be removed and sub-excavation would be carried out adjacent to the recently completed new CSP structure. In this event, it must be ensured that during the sub-excavation of the second culvert site and subsequent fill placement, no lateral yielding of the first culvert (i.e. the completed culvert) and the soils supporting the culvert would occur. If lateral yield is allowed to take place, excessive settlements may ensue.

We recommend that a granular fill be used for the construction of the engineered fill. It is also recommended that the granular fill consist of a well-graded soil such as Granular ‘A’ or Granular ‘B’ Type I or II. This is because fine sand and silt size particles may infiltrate into the poorly graded granular fill (such as clear limestone) thus causing settlements. If it is necessary to use a poorly graded fill, the exposed subgrade (bottom) and the sides as well as the top must be protected with a suitable geotextile. The placement of a geotextile may, however, in practice, be impractical to implement.

After the excavation and removal of all the unsuitable soils to a sufficient depth, the granular fill should be placed in shallow lifts not exceeding 300 mm before compaction. An exception to this may be the bottom layer immediately above the exposed and approved subgrade. The thickness of this layer may need to be increased to 500 mm, if the bottom of the excavation is not sufficiently dry and stable to effect proper compaction. Each layer of the engineered fill should be uniformly compacted to not less than 95% of the Standard Proctor Maximum Dry Density (SPMDD) of the material. The degree of compaction of the upper 0.3 m of the granular fill (which would receive the bedding material) should be increased to at least 97% of the SPMDD. A Bearing Resistance at ULS equal to 240 kPa can be assigned to the engineered fill prepared in this manner.

From the information provided to us, there will be no grade raise over and above the existing embankment. In fact, since the proposed culverts are larger than the existing culvert, there would be a net stress decrease. This is likely to compensate the placing of heavier fill after replacing the unsuitable soils. For this reason, with this approach, no foundation support problems are anticipated and the settlements under the existing embankment should be within reasonable limits (i.e. less than 30 mm) for a CSP type culvert. However, beyond the edge of the shoulder of the embankment of the grade raise will cause additional settlement under the extension (i.e. new passing lane). Here, the anticipated grade raise can be expected to cause another 20 mm settlement bringing the total anticipated settlements to about 50 mm. In our opinion, a settlement of this magnitude would be tolerable for a CSP. However, an allowance can be made to reduce the effects of settlements by overbuilding the bedding level by about 25 mm (i.e. make allowance for an approximately 25 mm settlement) from the inlet of the proposed culvert to the right edge of the present (existing) pavement edge, gradually reducing to zero some four meters beyond (i.e. towards the centerline of the present roadway).

Another (compromise) approach would be to reduce the depth of sub-excavation. This would reduce difficulties related to a deep excavation, as well as the severity of possible associated dewatering, in case of a high water level. With this approach, however, increased settlements may occur. This approach would be as follows: The existing soils would be sub-excavated to about El. 384.8 m. The exposed subgrade would then be inspected by the Geotechnical Engineer and evaluated. This would include digging shallow test pits to probe the subgrade. Based on the evaluation results, if necessary, the subgrade would be further lowered by removing any unsuitable soils. Once the approved subgrade level is reached, the grade would be raised using engineering fill, as discussed earlier in this section of the report. In this case, an ULS value of 180 kPa can be assigned to the subsoil prepared on this manner. Assuming an at least 1.0 m of good quality engineering fill is utilized beneath the pipe (i.e. beneath the bottom of bedding level), the settlements should not exceed 75 mm. This is believed to be within tolerable limits for the twin CSP's provided that no lateral yield occurs. Some of these settlements can be compensated by providing an 'overbuild' at the bedding level on the right side as discussed previously (i.e. about 25 mm). On the right side (beyond about the edge of the shoulder of the existing road) where the grade is to be raised, the excavation should be taken down to not less than El. 384.2 m (i.e. an additional 0.6 m, in a gradual manner) after which the exposed subgrade should be inspected, evaluated and approved, as discussed above.

Table 5.1.1.1 summarizes the pros and cons of the two options related to engineered fill approach.

Table 5.1.1.1
Comparison of Engineered Fill Options

Construction Method	Comments	Recommendations
Full removal of all unsuitable soils to competent subgrade and replacement with engineered fill.	Greater depths of excavation of existing soils and soil replacement, increased potential difficulties for dewatering and greater possibility of lateral yield of the first of the twin new culverts to be built. However, if executed properly, presents reduced risk of settlements of the twin culverts.	More reliable from engineering point of view but more time consuming to construct and more costly in comparison with reduced depth of excavation method.
Reduced depth of excavation and replacement with engineered fill.	Reduced depth of excavation and reduced potential problems due to dewatering, speedier construction, reduces the possibility of lateral yield of the first of the twin culverts during the construction of the second one, but increases the possibility of excessive settlements.	More cost-effective but less reliable from engineering point of view (i.e. more potential for settlements).

We recommend that in order to reduce the risk of potential problems due to dewatering, the construction be scheduled to be carried out during a dry season.

5.1.2 BEDDING

The bedding material should be placed as soon as practicable after the preparation of the subgrade, as discussed, its inspection and approval. The bedding should be in accordance with the appropriate standards (e.g. OPSD-802.010 and 802.014) and should consist of not less than 200 mm thick layer (after compaction) of approved granular material, such as Granular ‘A.’ The thickness of the bedding material may need to be increased depending on the site conditions at the time of construction. The bedding material should be compacted to at least 98% of the material’s SPMDD. If the bedding is to consist of a poorly graded material such as clear crushed stone, a suitable geotextile should be placed as a separator at the bottom and sides of the excavation, as well as the top.

5.1.3 BACKFILLING

The bedding and embedment material should be extended along the sides to cover the pipe. The selection and placing of the backfill should be in accordance with OPSD-802.010 and OPSD-802.014. The backfill should consist of free-draining, non-frost susceptible granular materials such as Granular ‘A’ or ‘B’ (OPSS-1010). All granular backfill materials should be placed in thin lifts (i.e. not exceeding 300 mm before compaction) and should be compacted to at least 96% of the material’s SPMDD. The Granular ‘A’ and Granular ‘B’ sub-base courses should be compacted to 100% of the SPMDD.

We would like to point out that the performance of flexible pipe culverts is largely dependent on the side support provided by the backfill and the adjacent soils. The use of proper backfill material and especially good compaction are, therefore, necessary for proper side support. For the twin culverts greater care may be required when compacting the soils in between the two culverts, depending on the distance between the culverts. The use of heavy compaction equipment should, however, be avoided immediately adjacent and above the pipes, as per MTO practice. During backfill placement, the height of the backfill should be maintained at approximately same level on both sides of the pipe, to avoid lateral displacement of the pipe.

Proper frost treatment is required in accordance with OPSD-803.030 or 803.031, whichever is applicable.

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

Computation of earth pressures acting against any rigid culvert walls and any wing walls should be in accordance with the Canadian Highway Bridge Design Code, (CHBDC) S6-06. For design purposes, the following properties can be assumed for backfill.

Compacted Granular 'A' and Granular 'B' Type 2

Angle of Internal Friction ϕ=35° (unfactored)

Unit weight = 22 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_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 1

Angle of Internal Friction ϕ=30° (unfactored)

Unit Weight = 21 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_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.

In accordance with the Province's Safety Regulation, the following soil classification would be applicable.

Granular Pavement Fill	Type 2 soil
Granular Embankment Fill	Type 3 soil above water level
Topsoil and Organic Silt	Type 3 soil above water level
	Type 4 soil below water level
Sandy Silt (possible fill) Sandy Silt (natural) and Silty Fine Sand	
	Type 3 soil above water level
	Type 4 soil below water level
Sandy Gravel	Type 2 soil above water level
	Type 4 soil below water level

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 5.1.4.1 can be used for the design of the temporary shoring system, based on the borehole results.

Table 5.1.4.1
Recommended Unfactored Parameters for Temporary Shoring Design

Soil Type	**K_a**	**K_o**	**K_p**	**Unit Weight (kN/m^3)**
Granular Embankment Fill	0.30	0.45	3.3	21.5
Sandy Silt (possible fill)	0.45	0.55	2.0	18.5
Organic Silt	0.55	0.70	1.2	15.0
Silty Fine Sand	0.40	0.53	2.5	18.0
Sand & Gravel to Gravelly Sand	0.30	0.45	3.3	21.5
Lower Sandy Silt	0.45	0.53	3.0	19.0

For widening of the embankment proper benching of the existing slopes should be implemented as per OPSD-208.010.

5.1.5 EROSION PROTECTION

Erosion and 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 at the invert level the soils are primarily alluvial sandy silt, silt and silty fine sand soils which are considered to be highly erodible soil types.

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 (i.e. below any possible scour depth).

In addition to cut-off and head walls, consideration may be given to other erosion and scour protection at the inlet and the outlet and possibly at an intermediate location.

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 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 (if clay seal is not used), in addition to the concrete cut-off and head walls, a 0.6 m thick rock protection consisting of 300 mm size rock can be considered, overlying a 300 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 beneath 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.

5.2 RETAINING WALLS

Boreholes C5-RW1 and C5-RW2 were put down for possible retaining walls on the right side of the highway. The recent design by UMA does not include retaining walls with the new dual CSP culverts; instead the length of the culvert has been increased to accommodate the proposed passing lane. Recommendations for the retaining walls are therefore not required. The following section has however been included for the sake of completeness.

Boreholes C5-RW1 and C5-RW2 appear to be located beyond the immediate watercourse valley and therefore organic or other alluvial deposits were not encountered. Beneath a 0.15 m thick topsoil, the subsoil at these borehole locations consists of glacial till deposits, along with a layer of silty fine sand at 4.5 m depth below the ground surface in Borehole C5-RW1, underlain by gravelly sand. Standard Penetration tests performed in these two boreholes indicate that at these borehole locations the glacial till deposits are in a compact condition in the upper 0.8 to 1.5 m depth and very dense below. The silty fine sand contacted in Borehole C5-RW1 is loose and the underlying gravelly sand is very dense to dense.

The following geotechnical resistance would be available for footings placed on natural competent glacial till deposits for about 2 or 3 m high retaining walls.

Reference Borehole	Existing Ground Elevation (m)	Re-commended Footing Base (Bottom) Level Below Existing Ground Surface (m)	Re-commended Footing Base (Bottom) Elevation (m)	ULS (kPa)	SLS (kPa)	Subgrade Material
C5-RW1	387.6	0.6-1.5 1.6-2.2 1.6-2.6	387.0-386.1 386.0-385.4 385.4-385.0	300 400 450	180 260 300	Sandy silt to silty sand till Sandy silt to silty sand till Gravelly sand till
C5-RW2	386.7	0.5-0.9 1.0-2.6	386.2-385.8 385.7-384.1	300 500	180 350	Sandy silt to silty sand till Sandy silt to silty sand till

It should be pointed out that the resistances provided in the above table are from boreholes drilled outside the primary flood plain. If retaining walls are required inside the primary flood plain (e.g. Borehole C5-2), poor soil conditions can be anticipated and this aspect would be further discussed with us.

Under inclined loading conditions, the bearing resistance at ULS should be reduced in accordance with CHBDC.

All footing excavations will need to be inspected, evaluated and approved by the Geotechnical Engineer appointed by the QVE.

The structure will need to be checked against overturning and sliding, with an appropriate factor of safety. The unfactored horizontal resistance against sliding between poured concrete and approved till subgrade surface can be calculated using a friction angle of 28 degrees.

Consideration can be given to other wall types including RSS (Reinforced Soil System), etc. Gabion type or crib type walls may also be suitable if some lateral yielding would not be objectionable. These aspects can be discussed with us, if desired, once the details of the site project are known.

5.3 EMBANKMENT WIDENING

Based on the latest design information provided by UMA, a new Northbound Passing Lane is proposed from Sta. 24+200 to 25+400, which includes the location of the proposed Culvert C5 at Sta. 25+145.

On the right side of the existing highway, three boreholes were put down namely, Boreholes C5-RW1, C5-RW-2 and C5-2. Boreholes C5-RW1 and C5-RW2 (located just outside the primary floodplain) encountered surficial topsoil underlain by native sandy silt to silty sand till followed by gravelly sand till, silty fine sand and gravelly sand; whereas Borehole C5-2 (located within the primary floodplain) encountered an extensive organic silt mixed with alluvial soils, followed by deposits of sand and gravel to sandy gravel and sandy silt.

We understand that the proposed grade raise above the original grades (o.g.) for the new Northbound Passing Lane on the right side of the existing highway will generally be about 2.5 m or less within the primary floodplain of the watercourse and typically less than 2.0 m beyond. Based on the conditions encountered in the exploratory boreholes put down around Culvert C5, no foundation failures are anticipated for the proposed embankment widening with normal (2H:1V) side slopes, provided all the organic weak or otherwise unsuitable materials are removed as per MTO standards and replaced with engineered fill prior to placing the new embankment fills.

The following table summarizes the anticipated stripping depths/elevations at the borehole locations.

Table 5.3.1
Anticipated Stripping Depths/Elevations

Borehole No./Ground Surface Elevations (m)	Anticipated Stripping Depth (m)	Elevation (m)
C5-RW1/El. 387.6 m	0.2	387.4
C5-2/El. 386.7 m	3.6*	383.1
C5-RW2/El. 386.7	0.2	386.5

*At this location, fill depth removal to 3.6 m may not be necessary depending on the inspection results by a qualified person.

It should be pointed out that the above table is for preliminary estimating purposes only and actual stripping depths must be verified and approved in the field by proper inspection by a qualified Geotechnical Engineer, as part of QVE tasks.

All organic and other unsuitable soils should be removed within an envelope area given by an imaginary slope not steeper than 1:1 from the toe of the proposed embankment widening, as per normal MTO procedures.

After stripping and inspection, the approved subgrade should be proof-rolled from the surface using a suitably heavy compactor. Application of compaction below the water table may be difficult and may require some dewatering.

Where deep excavations are required, stripping and backfilling may need to be performed in short sections in order not to cause instability of the existing embankment. This aspect should be looked into, after the details are known.

The sides of the existing embankment should be properly benched prior to placing the fill for the widening of the approach embankments, as per Ontario Provincial Standards Drawing OPSD 208.010.

The fills should be placed in lifts not exceeding 300 mm before compaction and each lift should be uniformly compacted to at least 95% of the material's Standard Proctor Maximum Dry Density. The selection, placement and compaction of the fill should be carried out under geotechnical supervision.

All borrow materials for proposed embankment widening should be approved by the geotechnical consultant from both geotechnical and environmental standpoints. The borrow materials should consist of select suitable inorganic earth borrow, free of objectionable inclusions such as cobbles and boulders, frozen materials, organic soils, etc., at or near the optimum moisture content.

Based on the available borehole data, assuming that properly compacted, acceptable inorganic earth fill materials are used for the approach slopes, 2H:1V side slopes can be used for embankment widening, provided that the subgrade is prepared in the manner described, including the removal of the unsuitable soils. The side slopes should protected from erosion during construction. Proper erosion control measures should be implemented by prompt seed and cover (OPSS 572).

The anticipated settlements depend on the height of embankments. For a typical embankment height of 2.5 m above o.g., the anticipated foundation settlements within the primary flood plain should not exceed 50 mm, about 60% of which should take place within a period of about eight weeks of the placement of embankment fills to their full height. Outside the primary flood plain, Boreholes C5-RW1 and C5-RW2 show much more competent soils and the height of embankment is expected to be less than 2.0 m. On this basis, the anticipated foundation settlements would be less than 12 mm. In addition to the foundation settlements discussed above, the embankment will settle under its own weight. This would depend on the materials used and compaction achieved but should typically not exceed 20 mm for an embankment height of 2.5 m and less than that for shallower embankment heights. The total of the two settlements would typically be about 70 mm within the primary flood plain and about 20 mm beyond, in the area of Boreholes C5-RW1 and C5-RW2, leading to a differential settlement of the order of 50 mm. Total settlements of this magnitude (provided the site is properly stripped to be free of organic soils) are normally considered to be within acceptable limits. However, consideration may be given to delaying the paving of the roadway for a period of at least about eight weeks, particularly in view of expected differential settlements between the section of the new widening within the primary valley of the watercourse and the sections beyond.

If and where the suggested side slopes can not be accommodated due to space limitations, consideration may be given to the use of temporary shoring, toe wall and/or reinforced earth slopes, as appropriate. In that case, further consultations are recommended.

5.9 BEARING SURFACES

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

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

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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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