

**FOUNDATION INVESTIGATION & DESIGN REPORTS
PROPOSED CULVERT (C6) REPLACEMENT
AT STATION 25+375 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-200**

Prepared For:

UMA/AECOM ENGINEERING LIMITED

Prepared by:

SHAHEEN & PEAKER LIMITED

**Project: SPT1174F
June 6, 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 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 proposed replacement of Culvert C6 at Station 25+375 with a new culvert and the associated retaining 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 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 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 (343.7 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. 386.2 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 information provided by UMA, existing Culvert C6 at Sta. 25+375 is an open bottom concrete culvert which will be replaced with a new CSP culvert. Furthermore a new Northbound Passing Lane has been proposed from Sta. 24+200 to 26+400, which includes the location of the proposed Culvert C6. On that basis, boreholes were drilled on the granular shoulder of the highway and at the toe of embankment in the vicinity of the proposed culvert replacement; other boreholes were put down for proposed retaining walls on the east (right) side of highway, as well.

The field investigation at this site was carried out during several periods from August 24, 2006 to May 16, 2007. The field investigation consisted of drilling and sampling of 3 boreholes for the culvert replacement and 7 boreholes for the associated retaining walls, to a maximum of 15.7 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 and Aardvark Drilling Inc. All the boreholes were drilled under the full time supervision of geotechnical staff 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. In addition to sampled boreholes Dynamic Cone Penetration Tests (DCPT) were performed from the bottom of Boreholes C6-RW6 and C6-RW7. In this test, a 51 mm diameter, 60-degree apex cone, screw attached to the tip of an A-size rod, is driven into the ground, using the same driving energy as the SPT method. By recording the number of blows of the hammer to drive the cone/rod assembly, into the soil every 0.3 m, a qualitative record of soil compactness condition is obtained. Although the interpretation of the test results is difficult because no samples are obtained by the DCPT and the penetration resistances are not necessarily equal to the N-values, useful information is gained by the continuity of the results and by the elimination of unbalanced hydrostatic force effects which in some cases affect the SPT results.

At the completion of drilling, all boreholes drilled were grouted and sealed using a cement/bentonite mixture. Boreholes installed with piezometer were grouted with bentonite seal above the slotted portion of the pipe and at ground surface.

Water level observations in the open boreholes were made during drilling and at the completion of each borehole. In addition, piezometers were installed in selected boreholes (C6-3, C6-RW4, C6-RW6 and C6-RW7). These piezometers allow monitoring of groundwater levels over time without undue interference/impact from surface water.

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 subsequently provided to us by UMA.

A laboratory testing program, consisting of natural moisture content and 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 vicinity of Culvert C6 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 drawings 6B and 6C. The following paragraphs are only meant to complement and amplify these data.

4.1 CULVERT REPLACEMENT (C6)

From the information provided to us by UMA, the existing culvert at Station 25+375 is an approximately 1.22 m wide and 0.93 m high open bottom concrete culvert. The invert of the existing culvert is at El. 372.82 m on the upstream side (inlet) and El. 372.80 m (outlet) on the downstream side.

Three boreholes were drilled for the culvert replacement. Boreholes C6-2 and C6-3 were put down near the existing culvert ends (from the bottom of the embankment) on the right (east) and left (west) sides of highway respectively, while Borehole C6-1 was advanced on the right shoulder of Highway 6, as shown on Drawing No. 6A.

The subsurface conditions encountered at the borehole locations along the proposed culvert replacement (C6-1, C6-2 and C6-3) are described in the following paragraphs. In brief, below a 5.6 m of embankment fill (Borehole C6-1), 0.7 m of peaty organic silt (Borehole C6-2) and silty clay fill to a depth of 1.5 m (Borehole C6-3), native inter-layered fine to coarse-grained granular deposits were encountered.

The borehole information obtained in the areas of the proposed retaining walls is discussed separately in Section 4.2.

4.1.1 EMBANKMENT FILL

Borehole C6-1 drilled on the gravel shoulder contacted a 0.25 m thick layer of sand & gravel pavement fill underlain by embankment fill material consisting of sand or sand & silt fill with traces to some gravel, extending to 5.6 m depth or to El. 373.0 m. The sand & silt fill layer contacted from 1.5 to 2.6 m below the ground grade, was noted to contain some gravel and occasional asphalt inclusions.

The embankment fill encountered at the borehole location is a basically granular material.

Standard Penetration Tests (SPT) performed in the fill material below 1.5 m depth yielded N-values ranging from 17 to 59 blows/0.3 m penetration, indicating a compact to very dense but generally compact condition. This indicates that a systematic compaction was applied when the fill was first placed.

4.1.2 TOPSOIL AND PEATY ORGANIC SILT

Boreholes C6-2 and C6-3, which were put down from toe of highway embankment, contacted a topsoil and peaty organic silt layer to a depth approximately 0.1 m and 0.7 m in Boreholes C6-3 and C6-2 respectively below the ground surface or to elevations El. 373.0m and El. 372.4 m.

4.1.3 FILL: SILTY CLAY

Below the topsoil in Borehole C6-3, a silty clay fill with traces of organics was contacted at a depth of 0.1 m below ground surface. The fill was found to extend to a depth of 1.5 m below the ground surface or to El. 371.6 m. This maybe original material which was disturbed during the construction of the existing culvert. The natural moisture content of samples recovered from the deposit is 35 and 59%.

Standard Penetration tests performed in this material were 0 and 1 blow/0.3 m. Based on these test results, the consistency of this cohesive soil is described as very soft.

4.1.4 SILTY FINE SAND TO SANDY SILT

Below the topsoil, peaty organic silt and clayey silt fill in Boreholes C6-2 and C6-3, an inter-layered native deposit of silty fine sand to sandy silt was contacted at a depth of 0.7 m and 1.5 m and extended to depths of 3.0 m and 3.8 m below the ground surface or to El. 370.1 m and El. 369.3 m, respectively.

A grain-size analysis test performed on a sample of this deposit from Borehole C6-2/SS2 (from 0.8 to 1.2 m depth), yielded the following grain size distribution, which is also shown in Figure B6-1 in Appendix B.

Gravel	0%
Sand	58%
Silt	34%
Clay	8%

Standard Penetration tests performed in this basically fine-grained granular deposit yielded N-values ranging from 8 to 22 blows/0.3 m, indicating a loose to compact but generally compact condition.

The measured natural moisture contents of recovered samples from this deposit range from 13 to 20%.

4.1.5 SILTY SAND TO SANDY SILT TILL

Below the embankment fill in Borehole C6-1, a 1.9 m thick layer of glacial till was contacted at 5.6 m depth below the ground surface or at El. 373.0 m and was found to extend to 7.5 m depth or El. 371.1 m. This granular till deposit consists of a heterogeneous mixture of silty sand to sandy silt with some gravel and traces of clay-size particles.

A grain-size analysis performed on a sample from the deposit (Borehole C6-1/SS10 at 6.9 to 7.3 m depth) yielded the following grain size distribution, as shown in Figure B6-2 in Appendix B.

Gravel	15%
Sand	47%
Silt	28%
Clay	10%

It should be pointed out that owing to their mode of deposition, the presence of cobbles and boulders should always be anticipated in the glacial till deposits.

Standard Penetration Tests performed in this fine-grained granular deposit yielded N-values of 9 and 19 blows/0.3 m, indicating a compact condition.

The measured natural moisture contents of recovered samples from this deposit range from 12 to 22%.

4.1.6 SAND

Below the till deposit in Borehole C6-1 and silty fine sand in Borehole C6-2 described in the preceding paragraphs, a cohesionless (i.e. granular) deposit of sand with traces of silt and gravel was encountered at 7.5 m depth or El. 371.1 m in Borehole C6-1 and 3.0 m depth or El. 370.1 m in Borehole C6-2. This deposit extended to 8.6 m depth or El. 369.9 m in Borehole C6-1 and 4.5 m depth or El. 368.6 m in Borehole C6-2.

This deposit was generally brown, partly oxidized and wet. The measured natural moisture contents of recovered samples from this material ranged from 14 to 18%.

Standard Penetration tests performed in this granular deposit yielded N-values ranging from 18 to 47 blows/0.3 m, indicating a compact to dense condition.

4.1.7 GRAVELY SAND TO SANDY GRAVEL

Below the soil strata described in the preceding paragraphs, a significant water bearing deposit of gravelly sand to sandy gravel was encountered in all the three boreholes at depths of 3.8 m (Borehole C6-3) to 8.6 m (Borehole C6-1), extending the termination of the investigation or to El. 365.0 m in Boreholes C6-2 and C6-3 and El. 365.9 m in Borehole C6-1.

A grain-size analysis test performed on a sample of this deposit from Borehole C6-3/SS9 (from 6.1 to 6.5 m depth), yielded the following grain size distribution, which is also shown in Figure B6-3 in Appendix B.

Gravel	27%
Sand	68%
Silt & Clay	5%

Standard Penetration Tests performed in this coarse-grained granular deposit yielded N-values ranging from 6 to 46 blows/0.3 m, indicating a loose to dense condition but generally a compact condition.

The measured natural moisture contents of the recovered samples of this coarse-grained deposit generally range from 5 to 19%.

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 was installed in Borehole C6-3 to allow ground water monitoring over a prolonged period of time, without interference of surface water. The observations and recorded values are shown on the individual Record of Borehole sheets in Appendix A.

The results indicate that at the time of our investigation, water was found in the open boreholes at 6.0 m (El. 372.6 m) in Borehole C6-1 and 1.4 m (El. 371.7) in Borehole C6-2 upon completion of drilling. These recorded water levels in open boreholes are unlikely to represent the stabilized groundwater conditions. In the piezometer installed, Borehole C6-3, the water level was measured at the ground surface level or at El. 373.1 m, two days after the installation of the piezometer. This is likely to represent the stabilized groundwater level at the time of the drilling of the borehole (i.e. May 2007).

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 PROPOSED RETAINING WALLS

In addition to the culvert boreholes (described above), seven boreholes (Boreholes C6-RW1, C6-RW2, C6-RW3, C6-RW4, C6-RW5, C6-RW6 and C6-RW7) were drilled at the right (east) side of the highway (on both north and south sides of Culvert C6), as shown on the Borehole Location Plan, Drawing No. 6A. Initially there were only five boreholes (Boreholes C6-RW1 to C6-RW5) but later on two deeper boreholes (C6-RW6 and C6-RW7) were added, in order to obtain additional subsurface information for the proposed walls, as well as due to the fact that the proposed wall heights were increased, from the originally proposed.

Boreholes C6-RW1 through 7 contacted surficial organic silt or topsoil followed by granular deposits of silty fine sand to sandy silt, silty sand to sandy silt till, sand and gravelly sand to sandy gravel.

4.2.1 ORGANIC SILT / PEATY ORGANIC SILT / TOPSOIL

At the location of Boreholes C6-RW1 to C6-RW7, the ground surface was covered by peaty organic silt, organic silt, peaty topsoil or topsoil. The thickness of these organic soil layers range from 0.2 m of topsoil in Borehole C6-RW5 to 1.2 m of organic silt in Borehole C6-RW1 and peaty topsoil in Borehole C6-RW6.

4.2.2 SILTY FINE SAND TO SANDY SILT

The surficial organic silt and topsoil are underlain by silty fine sand to sandy silt in all boreholes (except in Borehole C6-RW1). This surficial deposit was found to extend to depths ranging from 0.8 m (El. 374.6 m) in Borehole C6-RW5 to 6.1 m (El. 367.2 m) in Borehole C6-RW3 (the termination of the borehole). In Borehole C6-RW1, sandy silt to silty sand was contacted at 3.7 m (El. 375.0 m) depth (underlying a surficial sandy silt till deposit) and extended to the termination of the borehole at 6.1 m depth (El. 372.6 m).

Grain-size analysis performed on a silty fine sand sample of this deposit from Borehole C6-RW3 (SS4 from 2.3 to 2.9 m depth) yielded the following grain size distribution, as shown in Figure B6-4 in Appendix B.

Gravel	0%
Sand	68%
Silt	25%
Clay	7%

Standard Penetration Tests performed in this deposit yielded N-values ranging from 5 to in excess of 91 blows/0.3 m, indicating a loose to very dense but generally compact to dense condition.

The measured natural moisture contents of this deposit generally range from 10 to 22%.

4.2.3 SANDY SILT TO SILTY SAND TILL

Below the surficial organic silt in Borehole C6-RW1, at 1.2 m below the ground surface or at El. 377.5 m and also in Boreholes C6-RW2 and Borehole C-6RW5, a deposit of sandy silt to silty sand till was encountered (at 2.2 m/El. 371.5 m and 3.7 m/El. 371.7 m, respectively). This glacial till deposit was found to extend to a depth of 3.7 m (El. 375.0 m in Borehole C6-RW1) to 6.1 m in Boreholes C6-RW2 and C6-RW5 (to the termination of these two boreholes at El. 367.6 m and El. 369.3 m, respectively).

Grain-size analysis performed on two samples of this deposit from Boreholes C6-RW1 and C6-RW2 (SS4 from 2.3 to 2.9 m depth) yielded the following grain size distribution, as shown in Figure B6-5 in Appendix B:

Gravel	7-28%
Sand	39-46%
Silt	20-42%
Clay	6-12%

It should be pointed out that the presence of cobbles and boulders can be expected in glacial till deposits.

Standard Penetration tests performed in this granular glacial till deposit yielded N-values ranging from 21 blows/0.3 m to 83 blows/0.2 m, indicating a compact to very dense condition.

The measured natural moisture contents of samples recovered from this layer was 8 to 22%.

4.2.4 GRAVELLY SAND TO SANDY GRAVEL

A major deposit of gravelly sand to sandy gravel was contacted in Boreholes C6-RW4, C6-RW5, C6-RW6 and C6-RW7 at depths ranging from 0.8 m (El. 374.6 m) in Borehole C6-RW5 to 3.8 m (El. 369.4 m) in Borehole C6-RW6. The deposit was found to extend to depths ranging from 3.7 m (El. 367.7 m) in Borehole C6-RW5 to 15.7 m (El. 357.5 m or to the termination of borehole) in Borehole C6-RW6.

This coarse-grained granular deposit contains cobbles/boulders (e.g., as noted in Borehole C6-RW5 at El. 372.3 m).

Grain-size analyses performed on four samples of this deposit from Boreholes C6-RW4 (SS4), C6-RW5 (SS4), C6-RW6 (SS7) and C6-RW7 (SS10) yielded the following grain size distributions, as shown in Figure B6-6 in Appendix B:

Gravel	39 - 61%
Sand	34 - 57%
Silt and Clay	3 - 8%

This deposit is cohesionless (granular) material and contains cobbles/boulders.

Standard Penetration Tests (SPT) performed in this deposit yielded N-values ranging from 8 blows/0.3 m to 50 blows/0.13 m, indicating a wide range of compactness condition ranging from loose to very dense.

4.2.6 SAND

A sand deposit was encountered in Borehole C6-RW7 at a depth of 4.4 m/El. 370.2 m (in between gravelly sand and gravelly sand to sandy gravel layers) and in Borehole C6-RW4

(below the gravely sand to sandy gravel deposit) at a depth of 5.3 m (El. 367.7 m). The sand deposit was found to extend to a depth of 6.1 m (El. 367.0 m in Borehole C6-RW4 where the borehole was terminated and also to a depth of 6.1 m (El. 368.5 m) in Borehole C6-RW7.

N-values recorded in this granular deposit range from 23 to 32 blows/0.3 m. From the test results, the deposit is considered to be compact to dense.

4.2.7 GROUNDWATER CONDITIONS


Groundwater levels in the open boreholes were observed during the drilling and at the completion of each borehole. In addition, piezometers were installed in Boreholes C6-RW4, C6-RW6 and C6-RW7 to enable us to monitor the groundwater levels over prolonged periods of time without undue interference from surface water. The observations and recorded values are shown on the individual Record of Borehole sheets.

Based on the recorded values, it is our opinion that at the time of our investigations, the groundwater level at the site was typically between the ground surface (o.g.) level and 0.3 m below the ground surface in the primary valley of the watercourse i.e. at El. 372-373 m; about 1 m below the ground surface (o.g.) a short distance beyond the primary valley i.e. at El. 373.5± m (e.g. Borehole C6-RW7) and to about 2 m below the ground surface further beyond, i.e. at about El. 373.5-374.0 m (e.g. at Borehole C6-RW1 location).

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.


Z.S. Ozden, P.Eng.



Drawings

METRIC

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

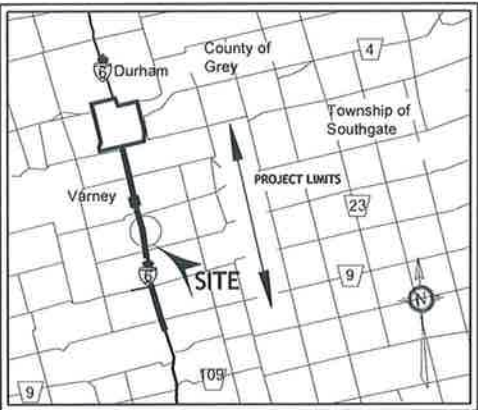
NOTES:
FOR DETAILED SUBSURFACE CONDITIONS
REFER TO RECORD OF BOREHOLE SHEETS.

CONT No.
GWP: 338-97-00

Highway 6, Durham
Culvert C6 @ Sta. 25+375
Borehole Location Plan



SHAHEEN & PEAKER LIMITED



KEY PLAN
N.T.S

LEGEND

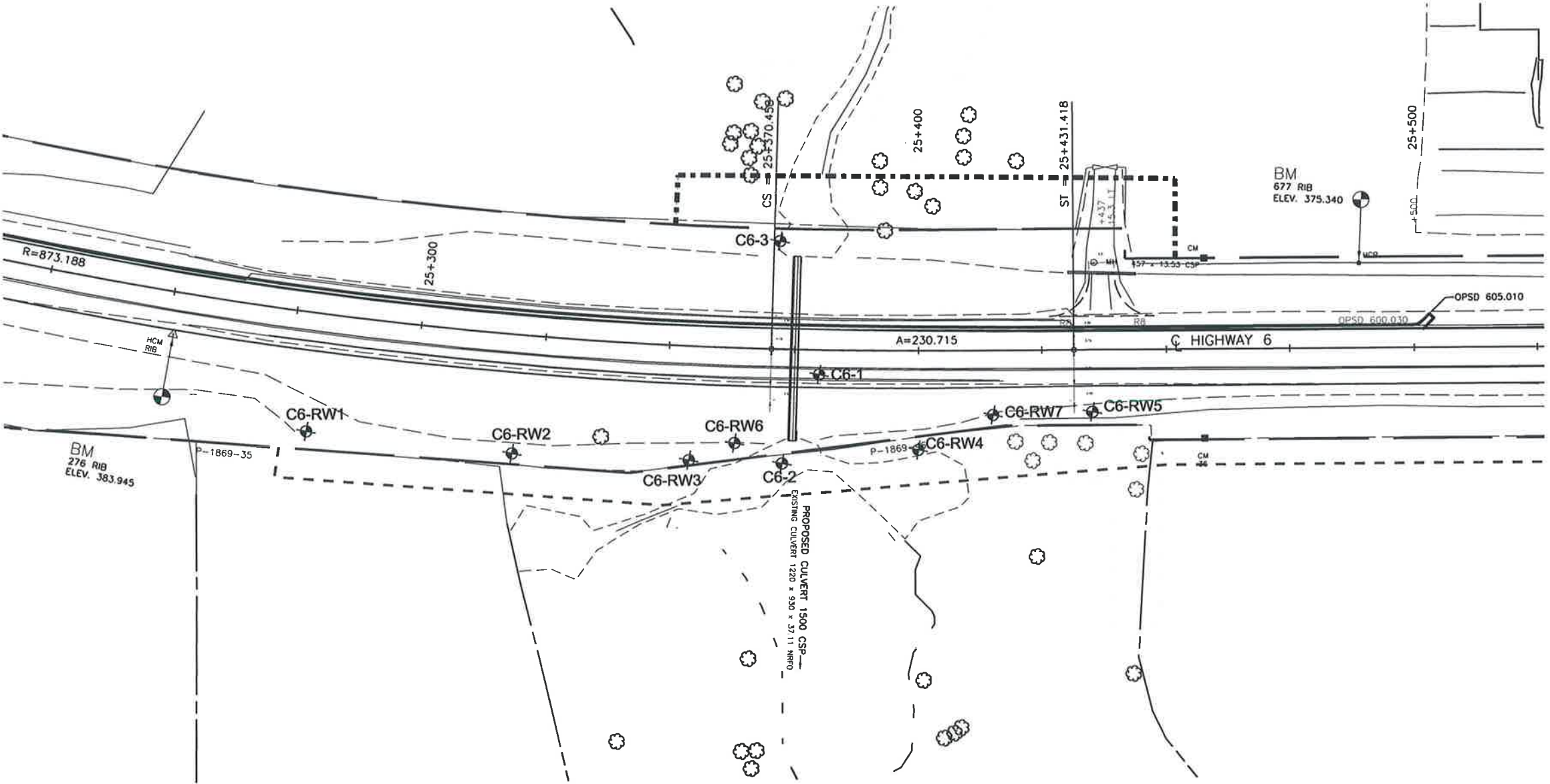
Borehole

No.	ELEV.	CO-ORDINATES	
		NORTH	EAST
C6-1	378.6	4 886 524.6	200 499.0
C6-2	373.1	4 886 520.5	200 517.9
C6-3	373.1	4 886 512.2	200 473.9
C6-RW1	378.7	4 886 425.4	200 528.5
C6-RW2	373.7	4 886 466.7	200 525.6
C6-RW3	373.3	4 886 502.0	200 520.6
C6-RW4	373.1	4 886 547.1	200 510.4
C6-RW5	375.4	4 886 580.3	200 496.6
C6-RW6	373.2	4 886 510.5	200 515.6
C6-RW7	374.6	4 886 560.7	200 500.8

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-200			
SPT 1174 F			DIST
SUBM'D	CHECKED	DATE Jan., 2008	SITE
DRAWN SM	CHECKED RM	APPROVED ZO	DWG 6A



PLAN



METRIC

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

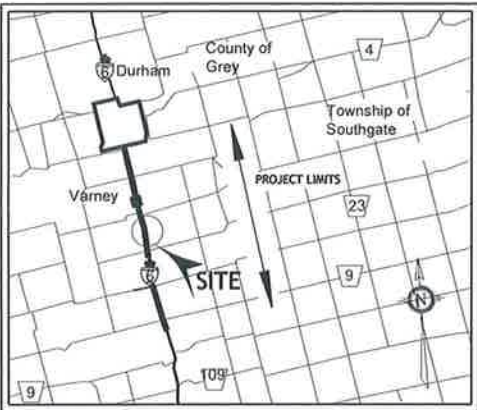
NOTES:
FOR DETAILED SUBSURFACE CONDITIONS
REFER TO RECORD OF BOREHOLE SHEETS.

CONT No.
GWP: 338-97-00

Highway 6, Durham
Culvert C6 @ Sta. 25+375
SOIL STRATIGRAPHY



SHAHEEN & PEAKER LIMITED



KEY PLAN
N.T.S

LEGEND

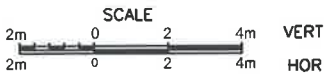
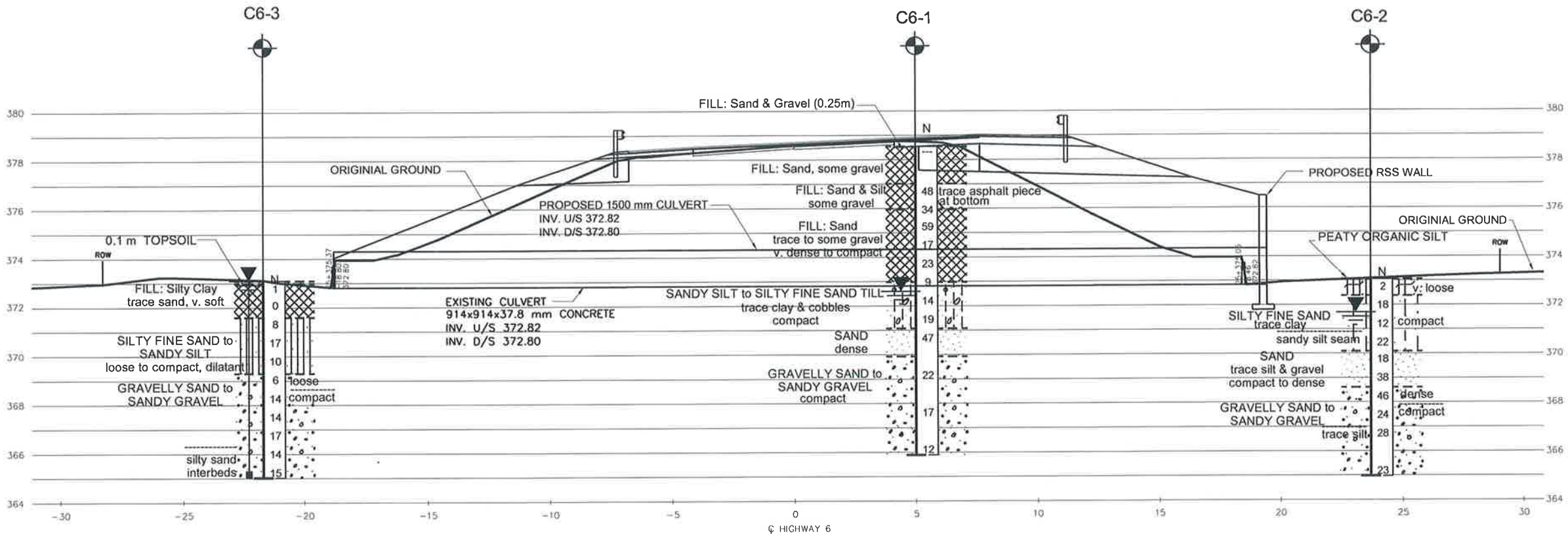
- Borehole
- Blows/0.3m (Std. Pen. Test, 475 J/blow)
- Water Level at Time of Investigation (W. L. NOT STABILIZED)
- Water Level in Piezometer
- Piezometer

No.	ELEV.	CO-ORDINATES	
		NORTH	EAST
C6-1	378.6	4 886 524.6	200 499.0
C6-2	373.1	4 886 520.5	200 517.9
C6-3	373.1	4 886 512.2	200 473.9

=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-200			
SPT 1174			DIST
SUBM'D	CHECKED	DATE Jan., 2008	SITE
DRAWN SM	CHECKED RM	APPROVED ZO	DWG 6B



STRATIGRAPHIC SECTION ALONG CULVERT C6 @ STA. 25+375



METRIC

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

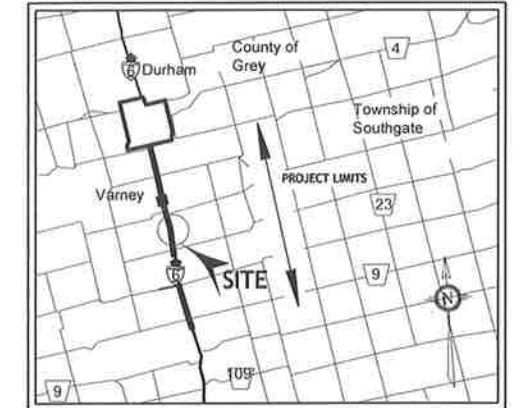
NOTES:
FOR DETAILED SUBSURFACE CONDITIONS
REFER TO RECORD OF BOREHOLE SHEETS.

CONT No.
GWP: 338-97-00

Highway 6, Durham
Culvert C6 @ Sta. 25+375
SOIL STRATIGRAPHY



SHAHEEN & PEAKER LIMITED



KEY PLAN
N.T.S

LEGEND

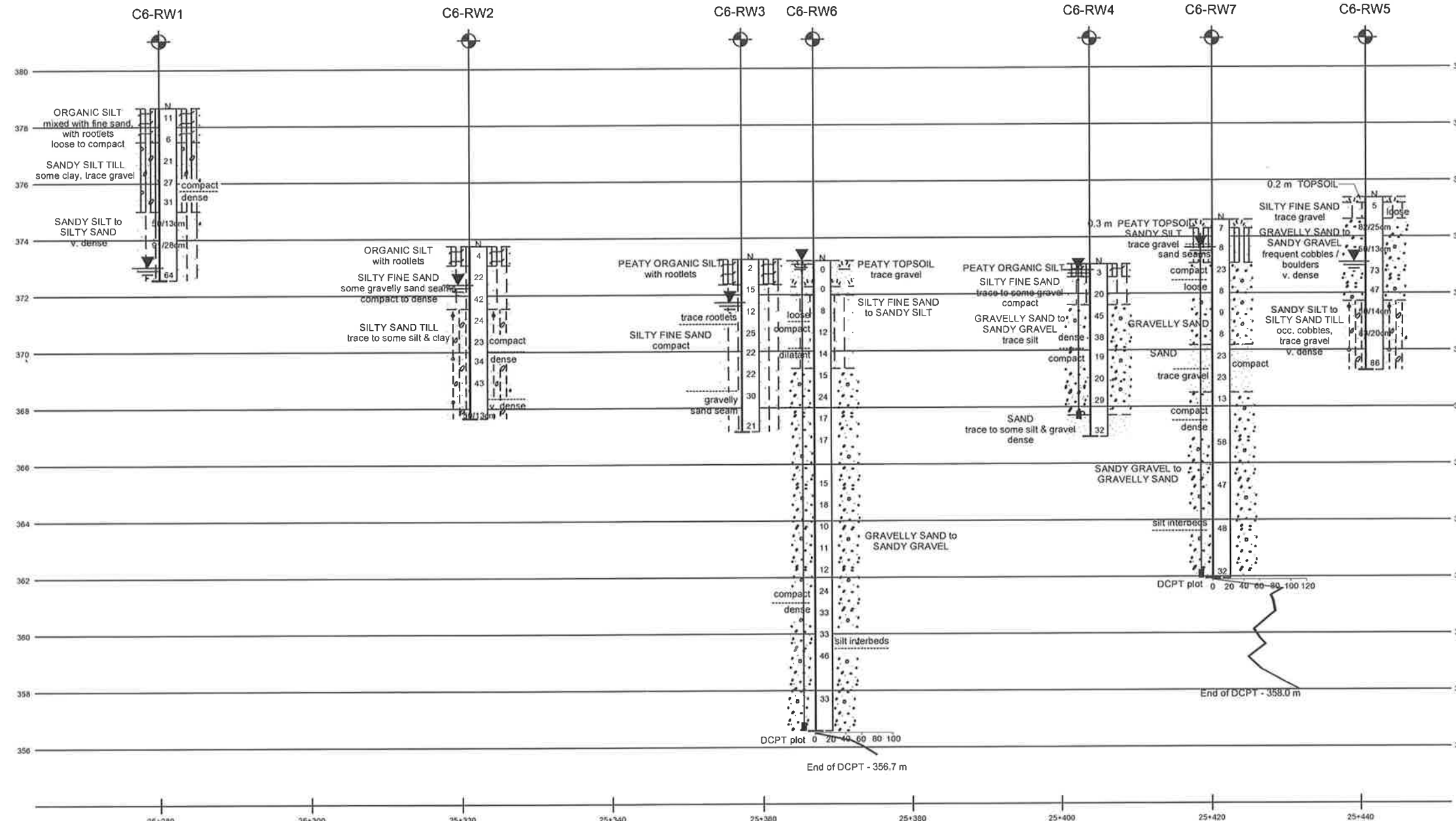
- ⊕ Borehole ⊕ Dynamic Cone Penetration Test
N Blows/0.3m (Std. Pen. Test, 475 J/blow)
▽ Water Level at Time of Investigation
(W. L. NOT STABILIZED)
▽ Water Level in Piezometer
| Piezometer

No.	ELEV.	CO-ORDINATES	
		NORTH	EAST
C6-RW1	378.7	4 886 425.4	200 528.5
C6-RW2	373.7	4 886 466.7	200 525.6
C6-RW3	373.3	4 886 502.0	200 520.6
C6-RW4	373.1	4 886 547.1	200 510.4
C6-RW5	375.4	4 886 580.3	200 496.6
C6-RW6	373.2	4 886 510.5	200 515.6
C6-RW7	374.6	4 886 560.7	200 500.8

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.
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REV.			
	DATE	BY	DESCRIPTION
Geocres No. 41A-200			
SPT 1174			DIST
SUBM'D	CHECKED	DATE Jan., 2008	SITE
DRAWN SM	CHECKED FS	APPROVED ZO	DWG 6C



SCALE
1m 0 2m VERT
5m 0 5 10m HOR

PROFILE ALONG PROPOSED RETAINING WALL

Appendix A

Record of Borehole Sheets

SPT1174

RECORD OF BOREHOLE No C6-1

1 OF 1

METRIC

GWP 338-97-00 LOCATION Hwy 6, Durham - Sta. 25+380, 5m Rt C/L ORIGINATED BY JL
DIST HWY 6 BOREHOLE TYPE Hollow Stem Augers COMPILED BY XS
DATUM Geodetic DATE 8/24/2006 CHECKED BY FS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L		
378.6 0.0	GROUND SURFACE												
	FILL: Sand & Gravel (0.25 m)		1	AS	11		378						
	FILL: Sand some gravel brown to dark brown, moist to damp		2	AS	---								
377.0 1.5							377						
	FILL: Sand & Silt some gravel trace asphalt piece at bottom brown, moist		3	SS	48								
376.0 2.8			4	SS	34		376						
	FILL: Sand trace to some gravel brown, moist very dense to compact		5	SS	59		375						
			6	SS	17		374						
			7	SS	23								
373.0 5.6			8	SS	9		373						
	SILTY SAND to SANDY SILT TILL brown to greyish brown, wet compact		9	SS	14		372						
			10	SS	19								
371.1 7.5			11	SS	47		371						
	SAND brown, oxidised dense						370						
369.9 8.6			12	SS	22		369						
	GRAVELLY SAND to SANDY GRAVEL brown to grey wet, compact		13	SS	17		368						
							367						
365.9 12.7			14	SS	12		366						
	End of borehole.												
	* Water level in open borehole at 6.0 m (El. 372.5 m) upon completion (not stabilized).												

+³ . X³ : Numbers refer to Sensitivity 20 15 10 5 (%) STRAIN AT FAILURE

SPT1174

RECORD OF BOREHOLE No C6-2

1 OF 1

METRIC

GWP 338-97-00 LOCATION Hwy 6, Durham - Sta. 25+375, 23m Rt C/L ORIGINATED BY JL
DIST HWY 6 BOREHOLE TYPE Hollow Stem Augers COMPILED BY XS
DATUM Geodetic DATE 10/4/2006 CHECKED BY FS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
373.1 0.0	GROUND SURFACE						373					
372.4 0.7	PEATY ORGANIC SILT with rootlets, black, moist very loose		1	SS	2		372				147	
			2	SS	18		371					0 58 34 8
	SILTY FINE SAND trace clay moist to wet, compact		3	SS	12		370					
	grey oxidised brown sandy silt seam		4	SS	22		369					
370.1 3.0	SAND trace silt & gravel brown, wet compact to dense		5	SS	18		368					
			6	SS	38		367					
368.6 4.5			7	SS	46		366					
	GRAVELLY SAND to SANDY GRAVEL brown, wet		8	SS	24							
	dense compact trace silt		9	SS	28							
365.0 8.1	End of borehole.		10	SS	23							
	* Water level in open borehole at 1.4 m (El. 371.7 m) upon completion (not stabilized).											

+³ . X³ . Numbers refer to
Sensitivity

20
15 10 5
10 (%) STRAIN AT FAILURE

SPT1174

RECORD OF BOREHOLE No C6-3

1 OF 1

METRIC

GWP 338-97-00 LOCATION Hwy 6, Durham - Sta. 25+372, 21.7 m Lt C/L ORIGINATED BY NE
 DIST HWY 6 BOREHOLE TYPE Hollow Stem Augers COMPILED BY XS
 DATUM Geodetic DATE 5/16/2007 CHECKED BY NE

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
FLEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80					
373.1 0.0	GROUND SURFACE 0.1 m TOPSOIL, trace gravel FILL: Silty Clay trace sand and organics grey, wet, very soft		1	SS	1											
371.6 1.5	SILTY FINE SAND to SANDY SILT brown, loose to compact, wet, dilatant		2	SS	0											
			3	SS	8											
			4	SS	17											
			5	SS	10											
369.3 3.8	GRAVELLY SAND to SANDY GRAVEL brown, wet		6	SS	6											
			7	SS	14											
			8	SS	14											
			9	SS	17											
			10	SS	14											
365.0 8.1	End of borehole. Monitoring well installed to depth of 8.1 m. Water level in well: May 18, 2007 --- 0 m (El. 373.1 m).		11	SS	15											

RECORD OF BOREHOLE No C6-RW1

1 OF 6

METRIC

GWP 338-97-00 LOCATION Hwy 6, Durham - Sta. 25+280, 24m Rt C/L ORIGINATED BY JL
 DIST HWY 6 BOREHOLE TYPE Hollow Stem Augers COMPILED BY XS
 DATUM Geodetic DATE 10/5/2006 CHECKED BY FS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● POCKET PENETR. X LAB VANE					W _P W W _L		
							20 40 60 80 100				10 20 30				
378.7 0.0	GROUND SURFACE														
377.5 1.2	ORGANIC SILT mixed with fine sand, with rootlets dark brown, loose to compact		1	SS	11		378								
			2	SS	6										
	SANDY SILT to SILTY SAND TILL some clay, trace gravel brown to brownish grey, moist to wet		3	SS	21		377								
			4	SS	27		376						28 46 20 6		
375.0 3.7		compact dense	5	SS	31		375								
	SANDY SILT to SILTY SAND brown, moist to wet very dense		6	SS	50/13cm										
			7	SS	91/28cm		374								
372.6 6.1	End of borehole.		8	SS	64		373								
	* Water level in open borehole at 5.6 m (El. 373.0 m) upon completion (not stabilized).														

Continued Next Page

Continued Next Page

+ 3 . X 3 : Numbers refer to
Sensitivity

20
15 10 5
10 (%) STRAIN AT FAILURE

SPT1174

RECORD OF BOREHOLE No C6-RW2

1 OF 1

METRIC

GWP 338-97-00 LOCATION Hwy 6, Durham - Sta. 25+320, 24m Rt C/L ORIGINATED BY JL
 DIST HWY 6 BOREHOLE TYPE Solid Stem Augers COMPILED BY XS
 DATUM Geodetic DATE 10/5/2006 CHECKED BY FS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80					
373.7 0.0	GROUND SURFACE															
373.0 0.7	ORGANIC SILT with rootlets, dark brown, moist		1	SS	4											
	SILTY FINE SAND some gravelly sand seams brown, compact to dense		2	SS	22											
371.5 2.2			3	SS	42											
			4	SS	24											
			5	SS	23											
			6	SS	34											
			7	SS	43											
367.6 6.1			8	SS	50/13cm											
	End of borehole. * Water level in open borehole at 1.4 m (El. 372.4 m) upon completion (not stabilized).															

SPT1174

RECORD OF BOREHOLE No C6-RW3

1 OF 1

METRIC

GWP 338-97-00 LOCATION Hwy 6, Durham - Sta. 25+355, 23m Rt, C/L ORIGINATED BY JL
DIST HWY 6 BOREHOLE TYPE Solid Stem Augers COMPILED BY XS
DATUM Geodetic DATE 10/5/2006 CHECKED BY FS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT Y kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			20	40						60	80
373.3 0.0	GROUND SURFACE		1	SS	2		373									
372.4 0.9	PEATY ORGANIC SILT with rootlets, black, moist to wet		2	SS	15		372									
			3	SS	12		371									
	SILTY FINE SAND grey to brown, moist compact		4	SS	25		370									
			5	SS	22		369									
			6	SS	22		368									
	gravelly sand seam		7	SS	30											
367.2 6.1	End of borehole.		8	SS	21											
	* Water level in open borehole at 1.5 m (El. 371.8 m) upon completion (not stabilized).															

+ 3 . X 3 . Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE

SPT1174

RECORD OF BOREHOLE No C6-RW4

1 OF 1

METRIC

GWP 338-97-00 LOCATION Hwy 6, Durham - Sta. 25+400, 20m Rt. C/L ORIGINATED BY NH
DIST HWY 6 BOREHOLE TYPE Solid Stem Augers COMPILED BY XS
DATUM Geodetic DATE 10/6/2006 CHECKED BY FS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● POCKET PENETR. X LAB VANE						
373.1	GROUND SURFACE							20 40 60 80 100						
0.0	PEATY ORGANIC SILT		1	SS	3		373							
372.6	black, with rootlets & decayed wood inclusions													
0.5	SILTY FINE SAND		2	SS	20		372							
	trace to some gravel													
371.6	brown, moist, compact		3	SS	45		371							
1.5	GRAVELLY SAND to SANDY GRAVEL		4	SS	38		370							
	brown, wet		5	SS	19		369							
			6	SS	20		368							
			7	SS	29									
367.7	SAND		8	SS	32		367							
5.3	trace to some silt & gravel													
367.0	brown, wet, dense													
6.1	End of borehole.													
	Piezometer installed to depth of 5.5 m. Water level in piezometer: Oct. 10, 2006 ---1.5 m (El. 371.5 m) Nov. 22, 2006 ---0.2 m (El. 372.9 m)													

SPT1174

RECORD OF BOREHOLE No C6-RW5

1 OF 1

METRIC

GWP 338-97-00 LOCATION Hwy 6, Durham - Sta. 25+435, 12.5m Rt, C/L ORIGINATED BY JL
 DIST HWY 6 BOREHOLE TYPE Solid Stem Augers & Hollow Stem Augers COMPILED BY XS
 DATUM Geodetic DATE 10/6/2006 CHECKED BY FS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
375.4 0.0	GROUND SURFACE																
374.6 0.8	0.2 m TOPSOIL, with rootlets SILTY FINE SAND trace gravel brown, moist, loose		1	SS	5		375										
			2	SS	32/25cm		374										
	GRAVELLY SAND to SANDY GRAVEL occasional cobbles / boulders brown, very dense		3	SS	50/13cm		373										boulder
			4	SS	73		372										wet spoon 48 44 (8)
371.7 3.7			5	SS	47		371										difficult augering from 3.1m depth and below
	SANDY SILT to SILTY SAND TILL brown, moist very dense		6	SS	50/14cm		370										Solid Stem Auger changed to Hollow Stem Auger due to caving
			7	SS	33/20cm												
369.3 6.1	End of borehole.		8	SS	86												
	Water level in open borehole at 2.3 m (El. 373.1m) upon completion (not stabilized).																

SPT1174

RECORD OF BOREHOLE No C6-RW6

1 OF 2

METRIC

GWP 338-97-00

LOCATION Hwy 6, Durham - Sta. 25+364, 19.2m Rt. C/L

ORIGINATED BY NE

DIST HWY 6

BOREHOLE TYPE Hollow Stem Augers

COMPILED BY XS

DATUM Geodetic

DATE 5/14/2007

CHECKED BY NE

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	W _p W W _L	WATER CONTENT (%)			
373.2 0.0	GROUND SURFACE		1	SS	0									
372.0 1.2	PEATY TOPSOIL trace gravel, wet		2	SS	0									
	SILTY FINE SAND to SANDY SILT wet, dilatant	grey loose	3	SS	8									
		compact brown	4	SS	12									
			5	SS	14									
369.4 3.8			6	SS	15									
			7	SS	24									
			8	SS	17									
			9	SS	17									
			10	SS	15									
			11	SS	18									
	GRAVELLY SAND to SANDY GRAVEL brown, wet, compact		12	SS	10									
			13	SS	11									
			14	SS	12									
			15	SS	24									
		compact dense	16	SS	33									
			17	SS	33									
		silt interbeds	18	SS	46									
358.2														

Continued Next Page

+ 3 × 3 Numbers refer to Sensitivity 20 15 10 5 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No C6-RW6

2 OF 2

METRIC

GWP 338-97-00

LOCATION Hwy 6, Durham - Sta. 25+364, 19.2m Rt, C/L

ORIGINATED BY NE

DIST HWY 6

BOREHOLE TYPE Hollow Stem Augers

COMPILED BY XS

DATUM Geodetic

DATE 5/14/2007

CHECKED BY NE

SOIL PROFILE

SAMPLES

GROUND WATER

CONDITIONS

ELEVATION SCALE

DYNAMIC CONE PENETRATION

RESISTANCE PLOT

20 40 60 80 100

SHEAR STRENGTH kPa

○ UNCONFINED + FIELD VANE

● POCKET PENETR. X LAB VANE

20 40 60 80 100

PLASTIC

LIMIT

NATURAL

MOISTURE

CONTENT

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ELEV

DEPTH

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DESCRIPTION

GRAVELLY SAND to SANDY GRAVEL
brown, wet, compact

End of borehole @ 15.7 m.

Dynamic Cone Penetration test performed from
15.7 to 16.6 m.

End of Dynamic Cone Penetration test.

Monitoring well installed to depth of 16.6 m.
Water level in well:
May 18, 2007 --- 0 m (El. 373.2 m).

STRAT PLOT

19

SS

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NUMBER

19

SPT1174

RECORD OF BOREHOLE No C6-RW7

1 OF 2

METRIC

GWP 338-97-00

LOCATION Hwy 6, Durham - Sta. 25+415, 13m Rt C/L

ORIGINATED BY NE

DIST HWY 6

BOREHOLE TYPE Hollow Stem Augers

COMPILED BY XS

DATUM Geodetic

DATE 5/15/2007

CHECKED BY NE

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			20	40						60	80
374.6 0.0	GROUND SURFACE															
	0.3 m PEATY TOPSOIL SANDY SILT trace gravel, brown, loose, wet		1	SS	7											
		sand seams	2	SS	8											
373.1 1.5			3	SS	23											
		compac loose	4	SS	8											
	GRAVELLY SAND brown, wet		5	SS	9											
			6	SS	8											
370.2 4.4			7	SS	23											
	SAND brown, wet, compact		8	SS	23											
		trace grave														
368.5 6.1			9	SS	13											
		compac dense														
			10	SS	58											
	SANDY GRAVEL to GRAVELLY SAND brown, wet		11	SS	47											
		silt interbeds	12	SS	48											
361.9 12.7	End of borehole @ 12.7 m.		13	SS	32											
	Dynamic Cone Penetration test performed from 12.7 to 16.8 m.															

Continued Next Page

+ 3 . X 3 : Numbers refer to
Sensitivity

20
15 10 5
10 (%) STRAIN AT FAILURE

SPT1174

RECORD OF BOREHOLE No C6-RW7

2 OF 2

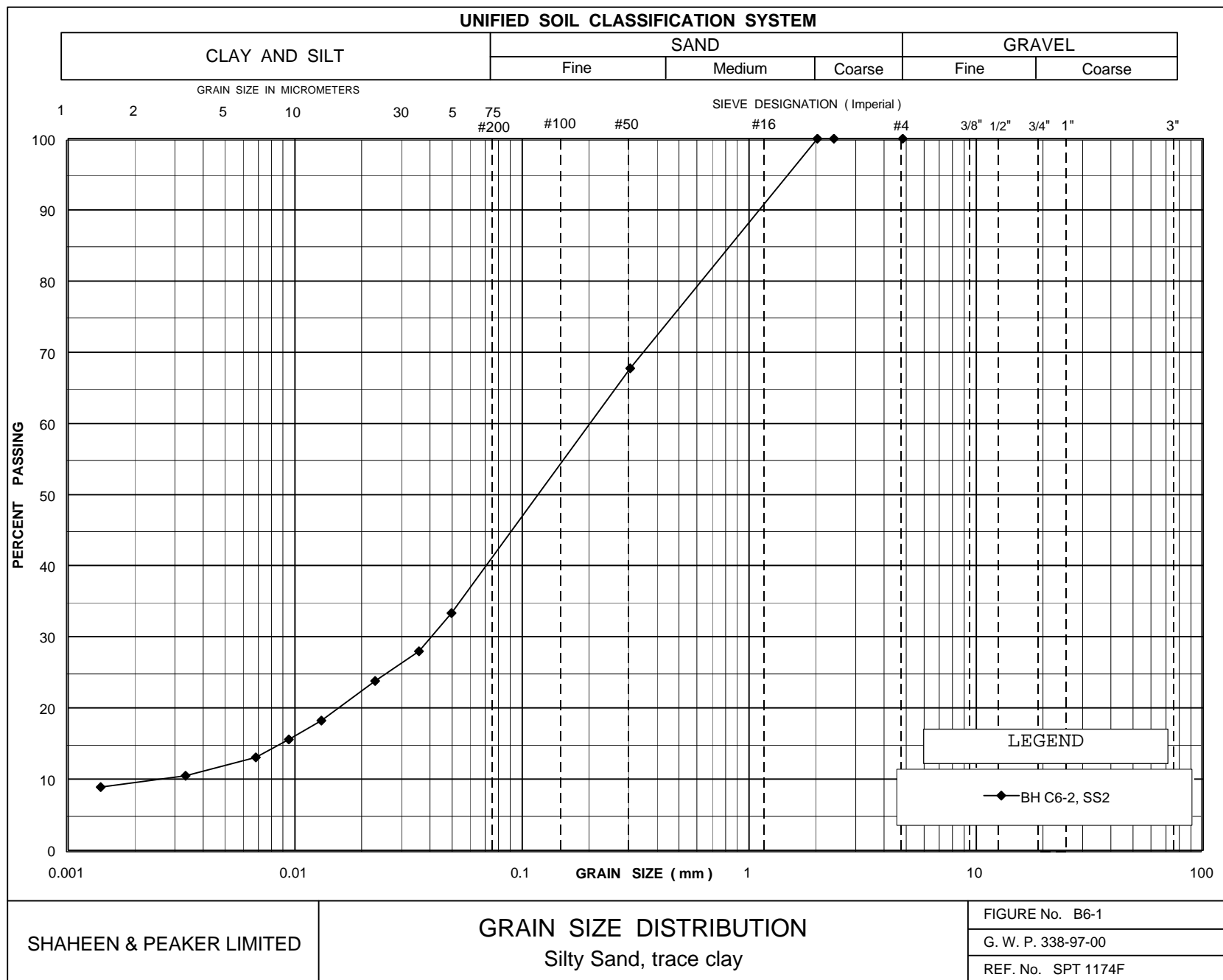
METRIC

GWP 338-97-00 LOCATION Hwy 6, Durham - Sta. 25+415, 13m Rt C/L ORIGINATED BY NE
DIST HWY 6 BOREHOLE TYPE Hollow Stem Augers COMPILED BY XS
DATUM Geodetic DATE 5/15/2007 CHECKED BY NE

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100					
359.6														
358.0														
16.8	End of Dynamic Cone Penetration test. Monitoring well installed to depth of 12.7 m. Water level in well: May 18, 2007 --- 1.0 m (El. 373.6 m).													

Appendix B

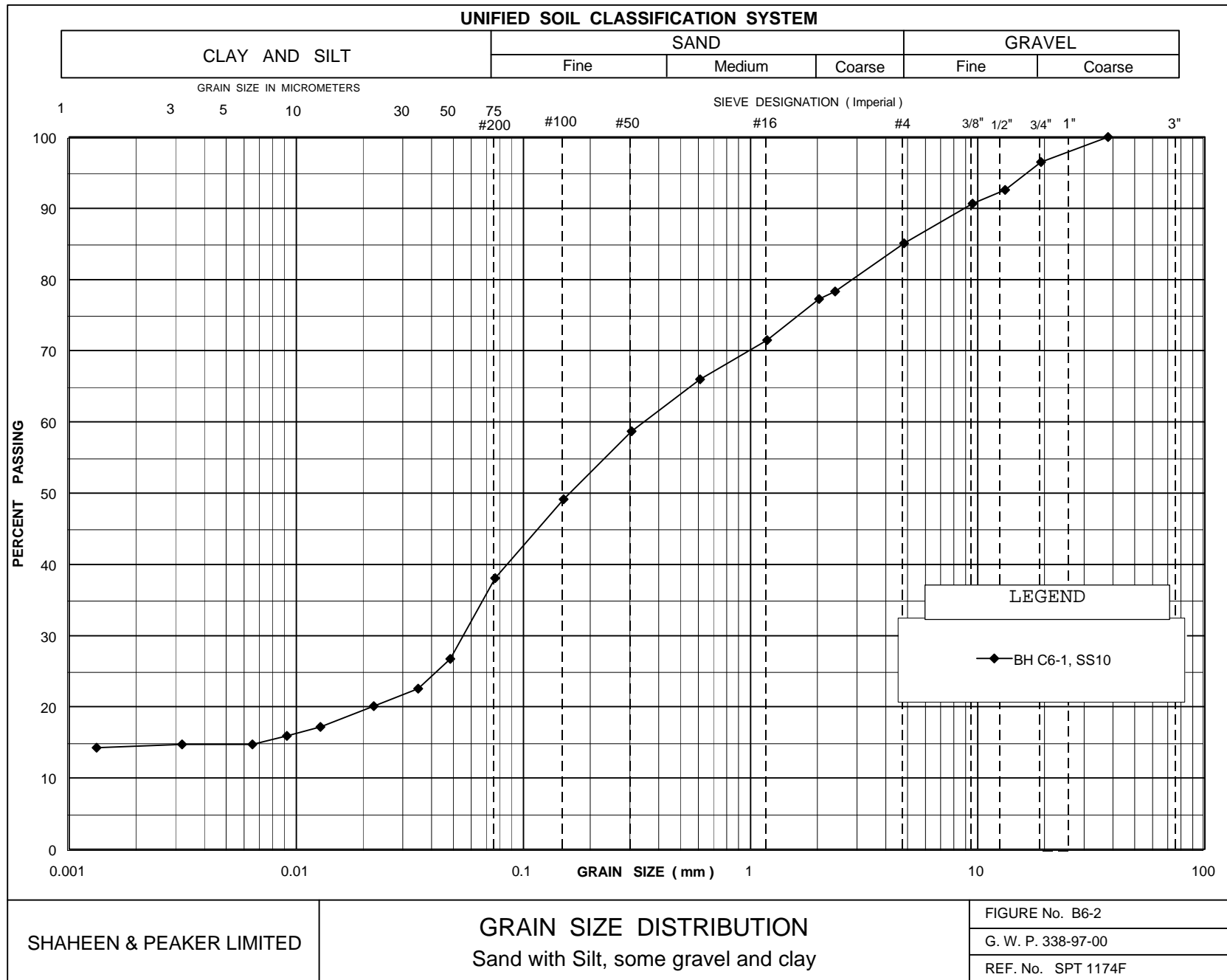
Laboratory Test Results



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GRAIN SIZE DISTRIBUTION
Silty Sand, trace clay

FIGURE No. B6-1
G. W. P. 338-97-00
REF. No. SPT 1174F



0.001 0.01 0.1 1 10 100

GRAIN SIZE (mm)

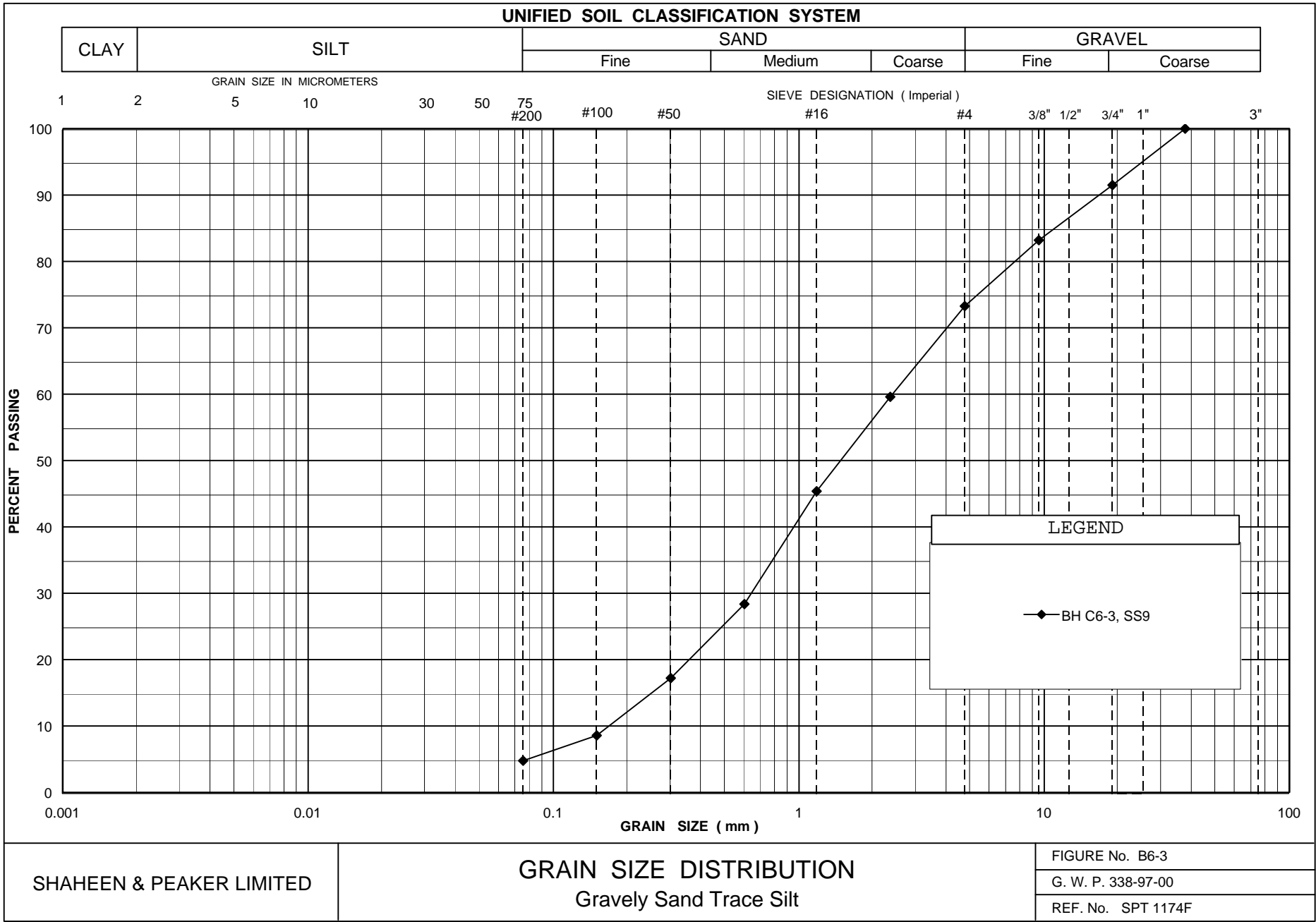
SHAHEEN & PEAKER LIMITED

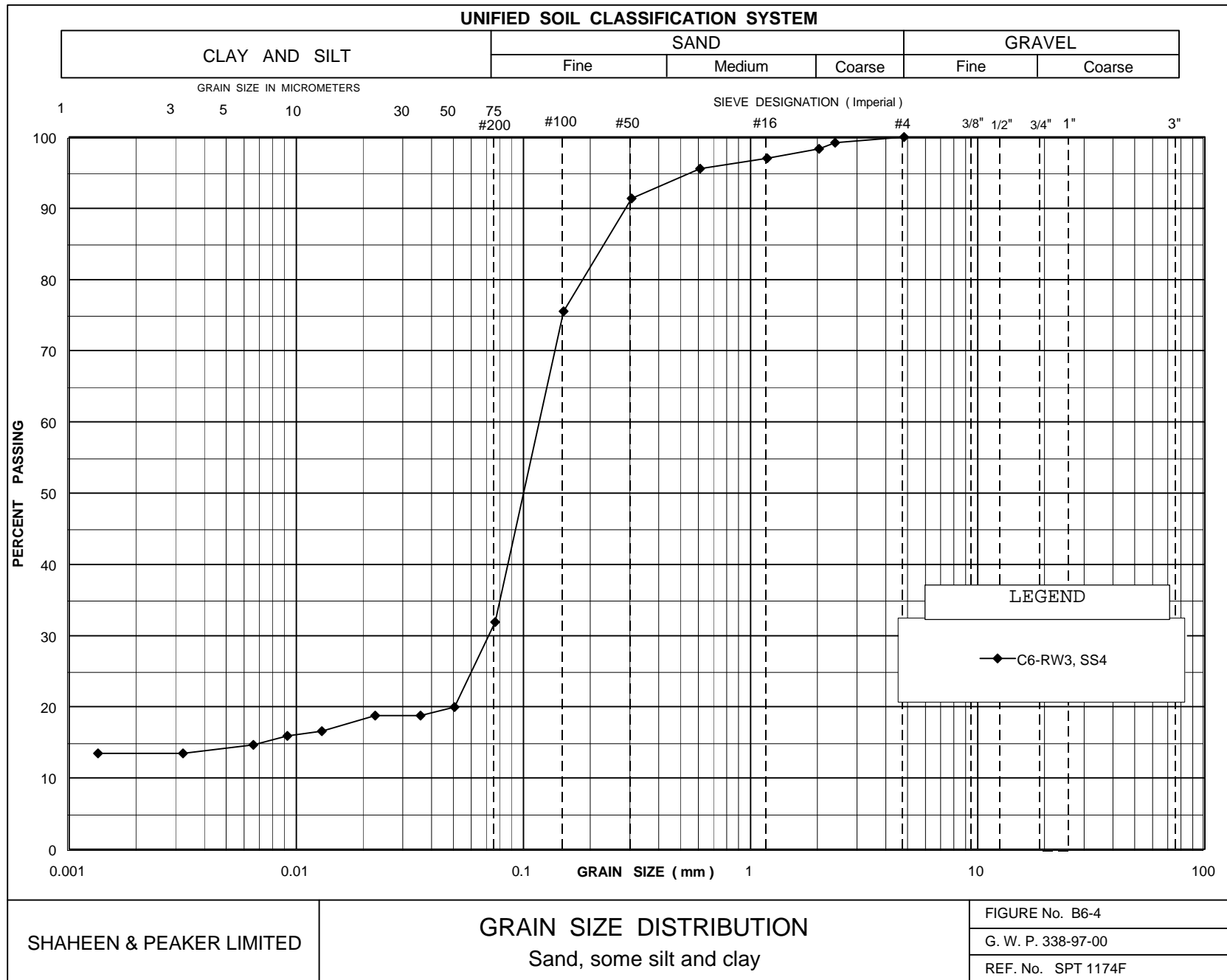
GRAIN SIZE DISTRIBUTION
Sand with Silt, some gravel and clay

FIGURE No. B6-2

G. W. P. 338-97-00

REF. No. SPT 1174F





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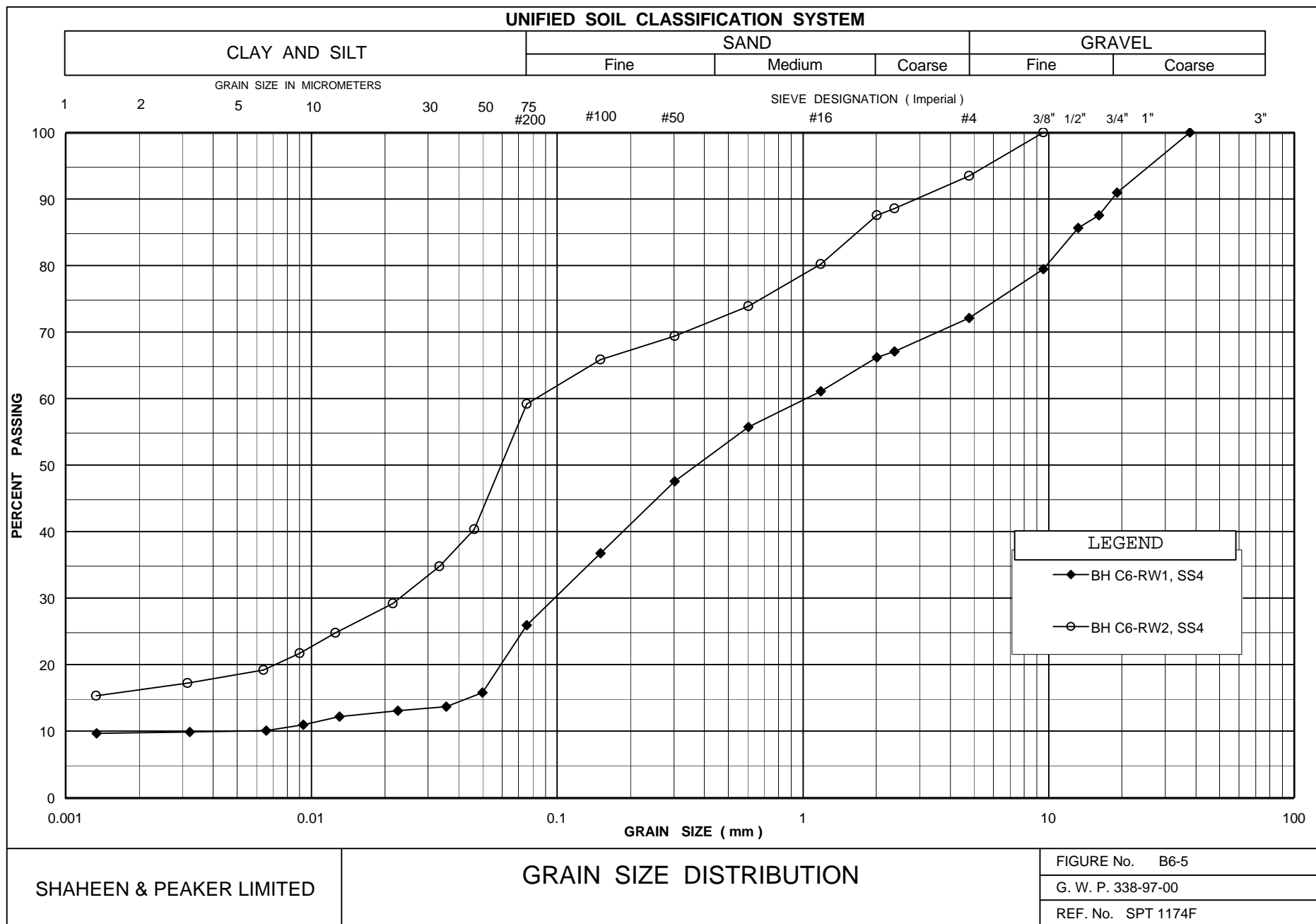
GRAIN SIZE DISTRIBUTION

Sand, some silt and clay

FIGURE No. B6-4

G. W. P. 338-97-00

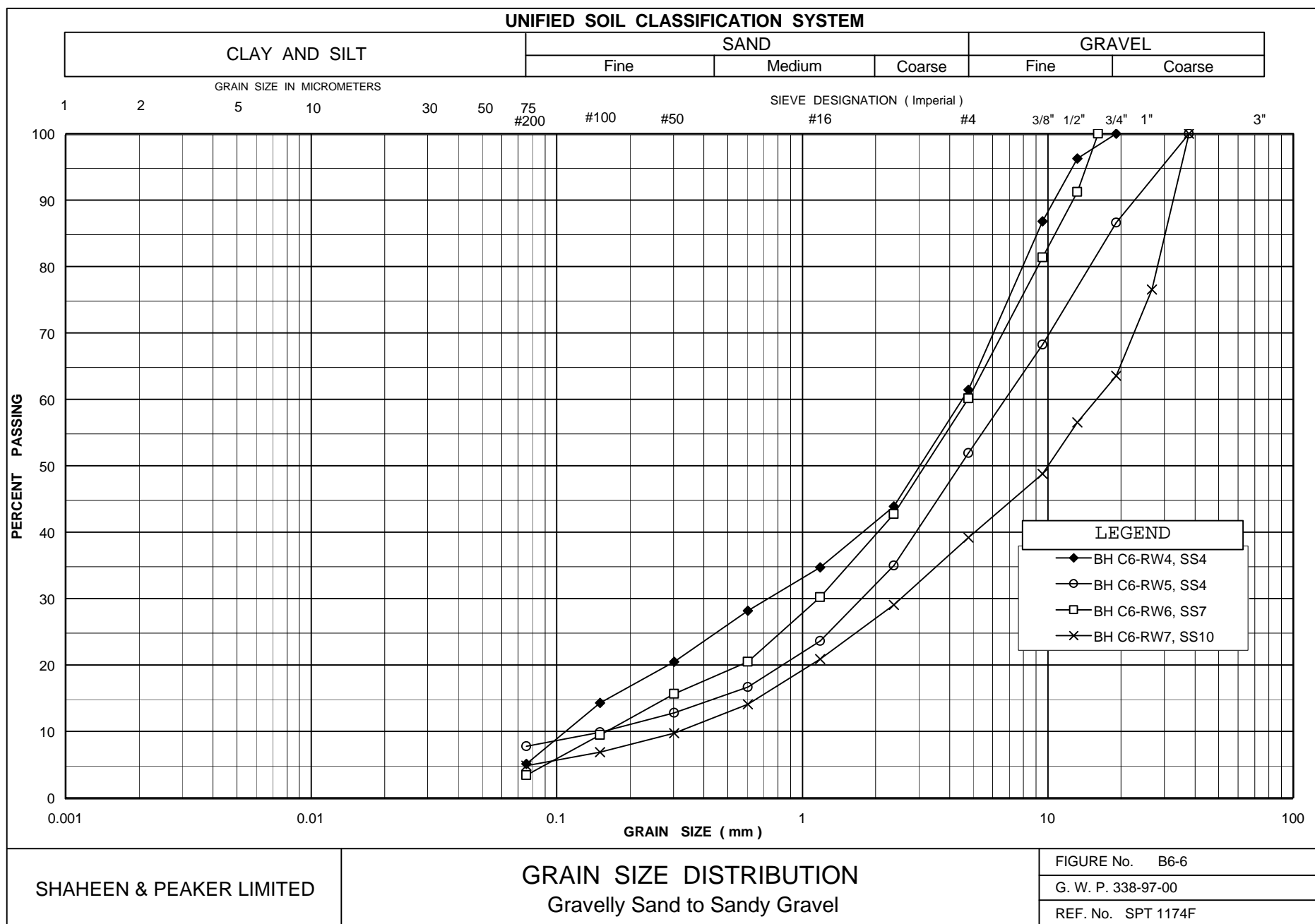
REF. No. SPT 1174F



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GRAIN SIZE DISTRIBUTION

FIGURE No. B6-5
G. W. P. 338-97-00
REF. No. SPT 1174F



Appendix C

Site Photographs



Photograph C-1 Location of Culvert (C 6) at Highway 6



Photograph C-2 Existing Culvert (C6) at Highway 6



Photograph C-3 BH C6-1



Photograph C-4 Borehole C6-2



Photograph C-5 Borehole C6-3



Photograph C-6 BH C-6-RW3



Photograph C-1 Location of Culvert (C 6) at Highway 6



Photograph C-2 Existing Culvert (C6) at Highway 6



Photograph C-3 BH C6-1



Photograph C-4 Borehole C6-2



Photograph C-5 Borehole C6-3



Photograph C-6 BH C-6-RW3

Appendix D

Explanation of Terms Used in Report

EXPLANATION OF TERMS USED IN REPORT

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

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 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
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

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'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = 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
j_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
j_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
j	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
j_d	kN/m ³	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $(W_L - W_p) / I_p$	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
j_{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
j'	kN/m ³	UNIT WEIGHT OF SUBMERGED SOIL						

**FOUNDATION DESIGN REPORT
PROPOSED CULVERT (C6) REPLACEMENT
AT STATION 25+375 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-200**

Prepared For:

UMA/AECOM ENGINEERING LIMITED

Prepared by:

SHAHEEN & PEAKER LIMITED

**Project: SPT1174F
June 6, 2008**



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APPENDICES

APPENDIX E TUNNELMAN'S GROUND CLASSIFICATION AND PROBABLE WORKING CONDITIONS

APPENDIX F: SLOPE STABILITY ANALYSIS RESULTS

APPENDIX G: LIMITATIONS OF REPORT

**FOUNDATION DESIGN REPORT
PROPOSED CULVERT (C6) REPLACEMENT
AT STATION 25+375 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

The culvert replacement and retaining wall construction options and recommendations at the vicinity of culvert C6 at Station 25+375 are discussed in the following sections.

5.1 CULVERT REPLACEMENT

We understand that the existing culvert at Station 25+375 is a 1.22 m wide, 0.93 m high and 37.8 m long open bottom concrete box culvert. The invert of the culvert is at El. 372.82 m on the upstream (east) side and El. 372.80 m on the downstream (west) side.

Based on the information provided to us by UMA, the new culvert will be a 1500 mm diameter, 38 m long CSP. The inverts will be the same as existing (El. 372.82 m on its upstream and El. 372.80 m on the downstream side; as shown in Drawing 6B). As also shown in this drawing, the height of the embankment at the culvert location is up to about 6 m above the o.g. level (also see Site Photographs in Appendix C).

Three boreholes were drilled at the site, namely Boreholes C6-2 and C6-3 on the east and west ends (near the toe of the embankment), while the third borehole (Borehole C6-1) was put down at the crest of the embankment on the east shoulder of Highway 6, immediately adjacent to the existing culvert. Below some 5.6 m of embankment fill in Borehole C6-1 to El. 373.0 m, a 0.7 m thick peaty organic silt in Borehole C6-2 to El. 372.4 m and silty clay fill to 1.5 m or to El. 371.6 m in Borehole C6-3, the natural inorganic soil strata were found to consist of silty fine sand to sandy silt in Boreholes C6-2 and C6-3 and silty sand to sandy silt till in Borehole C6-1. These deposits are in turn underlain by sand and even coarser grained gravelly sand to sandy gravel deposits.

The groundwater table at the time of our investigation was at or immediately below the o.g. level, but can be expected to be subject to fluctuations.

Due to the presence of variable soil conditions and high water table, the use of a CSP type culvert is preferred at this site. If a concrete culvert must be used, a pre-cast closed bottom culvert would be better suited in comparison with a rigid open bottom concrete type culvert, especially since the soil at both ends appear to be highly erodible. As the proposed design by UMA is a CSP type culvert (which is the preferred option from geotechnical point of

view), this type of culvert is discussed in the following paragraphs. If, however, you need more information on other type of culverts, we will be pleased to discuss this.

The proposed 1500 mm diameter CSP culvert can be installed using an open cut construction or alternatively by tunneling methods, as summarized in the following table.

Table 5.1.1
Comparison of Installation Methods

Construction Method	Comments	Recommendations
Open Cut Construction	Owing to variable foundation soil types as well as a possible variation in the embankment fill, the presence of high water table and possible presence of cobbles and boulders, open cut construction is considered to be more reliable than tunneling, as well as being less costly.	Recommended based on reliability, and cost factors. However, it will cause some traffic disruptions during construction, which may not be acceptable to MTO.
Tunnelling	Variable foundation soil conditions, possible variations in the embankment fill, a lack of a proven stable cover above pipe obvert, along with the recorded high water table and possible presence of boulders render tunneling a more risky and probably more costly method other than open cut. However, it will not cause traffic disruptions during the construction.	More risky and less cost-effective than open cut but can be considered if some traffic disruption during the construction would be unacceptable.

5.1.1 OPEN CUT CONSTRUCTION

For an open cut construction, the existing embankment fill will be excavated. As vertical excavations will likely be required, temporary shoring will probably be necessary for ground support due to limited space on the roadway. Locally, temporary shoring typically consists of soldier piles and lagging, while sometimes driven interlocking steel sheet piling is also used. Open cut construction will thus require lane closures which may be undesirable (i.e. a disadvantage of open cut construction) as well since the embankment height is in excess of 5 m, temporary shoring may be costly.

5.1.1.1 CULVERT FOUNDATION SUPPORT

The proposed culvert can be supported on the native, undisturbed silty sand to sandy silt till (Borehole C6-1), silty fine sand to sandy silt (Borehole C6-3) and the silty fine sand (Borehole C6-2). The recommended highest founding depths/elevations at each borehole location are given in Table 5.1.1.1.1 below.

Table 5.1.1.1.1
Top of Suitable Bearing Stratum at Borehole Locations

Borehole No.	Existing Ground Elevation (m)	Recommended Highest Founding Level (m)	Elevation (m)	Subgrade Material
C6-1	378.6	5.8	372.8	Compact Sandy silt to silty sand till
C6-2	373.1	0.8	372.3	Compact silty fine sand
C6-3	373.1	1.6	371.5	Loose to compact silty fine sand to sandy silt

Based on the borehole data, the following geotechnical resistances are available for a CSP culvert to be placed on approved (by the QVE), inorganic competent natural soil strata:

$$\begin{aligned}\text{Bearing Resistance at ULS} &= 280 \text{ kPa} \\ \text{Factored Geotechnical Resistance at SLS} &= 150 \text{ kPa}\end{aligned}$$

Under the existing embankment, where there is no grade raise proposed, in theory there should be no settlements, since the loads imposed will be less than the existing (i.e. the removal of soil for the proposed 1500 mm diameter culvert will result in a net stress decrease). It is, however, recommended that an allowance of about 20 mm settlement be made for possible rebound and settlement.

As shown on Drawing 6B, up to about 0.3 to 0.8 m of grade raise will be effected for a minor widening on the west side of the existing embankment. Based on the results of Borehole C6-3 which was put down on this side of the embankment, provided the weak silty clay fill is removed, this grade raise can be expected to cause a further settlement of between 6 and 8 mm, bringing the total maximum settlement on this side to about 28 mm. This is considered to be within acceptable limits for the proposed 1500 mm diameter CSP. On the opposite (i.e. east) side considerable widening and hence grade raise are proposed. From Drawing 6B the widening is about 5 m, with up to about 4 m additional embankment fill to be placed. Based on the results of Boreholes C6-1 and C6-2, assuming the organic peaty topsoil is removed, the additional settlement due the grade raise on the east side can be expected to be about 30 mm, bringing the total settlement to 50 mm. This magnitude of settlement is considered to be within tolerable limits. However, we recommend that grade under the culvert be overbuilt by about 20 mm in anticipation of this settlement. The overbuilding of the invert would be about 20 mm between the inlet of the pipe and the edge of the proposed embankment (e.g. proposed guide-rail location) gradually diminishing to zero at the middle point of the new passing lane.

It should be pointed out that at the time of our investigation, the groundwater table at the site was near the ground surface. If the subgrade is unduly disturbed during the construction, excessive settlements can occur (e.g. if the site was not properly dewatered). We recommend, therefore, 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.

In consideration of alternative culvert types, the subsurface conditions are not well-suited for the use of an open bottom concrete culvert. A closed bottom concrete culvert can be considered (pre-cast concrete culvert preferred) using the geotechnical resistances provided earlier in this section of the report. However, conditions are better suited for the use of a CSP culvert. If however, a pre-cast concrete culvert is to be used, we will be pleased to further discuss this alternative.

5.1.1.2 BEDDING

After the completion of excavation to the subgrade level (i.e. to the bottom of bedding material elevation beneath the pipe), the founding subgrade should be inspected, evaluated and approved by qualified personnel. The minimum excavation elevations to the suitable subgrade level at the borehole locations were given in Table 5.1.1.1.1 in Section 5.1.1.1 of this report. For example, at Borehole C6-3 the unsuitable soils extend to 1.6 m below o.g. or to El. 371.5 m, i.e. the existing soils at this borehole location is expected to be sub-excavated to this grade, as directed by the engineer. After the removal of all organic, weak or otherwise unsuitable soils to the acceptable approved subgrade level, where feasible the subgrade should be rolled from the surface and the grade should be raised, where necessary, using Granular 'A' or Granular 'B' – Type II materials or their equivalent. The fill should be placed in layers not exceeding 300 mm when loosely placed and each layer should be uniformly compacted to not less than 95% of the material's Standard Proctor Maximum Dry Density (SPMDD). Depending on the site conditions at the time of construction, the thickness of the first lift immediately above the approved subgrade level may need to be increased to 400 or even 500 mm, if the base is not sufficiently dry and stable to effect proper compaction.

The excavation, approval of the subgrade and the placement of the fill should be done as quickly as possible to avoid the loosening and disturbance of the subgrade soils.

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 a 300 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, approved geotextile should be placed as a separator at the bottom and sides of the excavation, as well as the top.

5.1.1.3 BACKFILLING

The bedding and embedment material should be extended along the sides and to cover the top of 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' base 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. 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.

Reference should be made to SP105S10 for compaction behind the culvert and any retaining walls.

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 II

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

Unit weight = 22 kN/m³

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/m³

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.

5.1.1.4 CONSTRUCTION

The construction of the pipe should be in accordance with SP421S01 and 421S04, for construction of pipe culverts.

The flow of water in the existing watercourse will need to be maintained during the construction. This can be achieved by placing a temporary pipe for the construction period or using the existing culvert for this purpose until the new culvert is built. The flow would then be diverted to the completed new culvert and the existing culvert would be removed or plugged.

Depending on the groundwater level encountered at the time of the construction, dewatering will likely be required to facilitate the construction and to preserve the load

carrying capability of the founding soils. The groundwater, where necessary, can be depressed by means of closely spaced filtered sumps. However, in this manner the groundwater will probably be depressed by not more than about 0.5 m. To depress the groundwater level further deeper, other methods such as deep wells and/or well points would be required. It should, however, be pointed out it may not be feasible to place well points across the highway unless traffic is totally diverted.

We recommend that the contractor be made aware of possible dewatering requirements to facilitate the construction. In this respect, the contractor may choose to dig some test pits to investigate conditions at the time of construction and the necessity for dewatering, and the methods that may be required for this purpose.

The contractor should also be made aware of the possible presence of cobbles and boulders in the embankment fill and also on the underlying natural soils, especially in the coarse grained soils (i.e. gravelly sand to sandy gravel deposits) and the glacial till.

We recommend that an NSSP be included in the Contract Documents to warn the contractor of possible dewatering requirements and the possible presence of cobbles and boulders in the founding soils.

All excavations should be carried out in accordance with the Province's Occupational Health and Safety Act (OHSA), O. Reg. 213/91, as well as the following:

- SP 105 S19 – Protection Systems
- SP 902 S01 – Excavation and Backfilling - Structures

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

Granular Pavement Fill	Type 3 soil
Granular Embankment Fill	Type 3 soil above water level
Topsoil and Organic Silt, Silty Clay Fill (very soft)	Type 4 soil
Silty Fine Sand, Silty Fine Sand to Sandy Silt	Type 3 soil above water level
	Type 4 soil below water level
Silty Sand to Sandy Silt Till	Type 3 soil above water level
	Type 4 soil below water level
Gravelly Sand to Sandy Gravel	Type 3 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 5.1.1.4.1 can be used for the design of the temporary shoring system, based on the borehole results of Borehole C6-1.

Table 5.1.1.4.1
Recommended Unfactored Parameters for Temporary Shoring Design

Soil Type	K_a	K_o	K_p	Unit Weight (kN/m^3)
Pavement Fill	0.28	0.42	3.5	21.5
Embankment Fill	0.33	0.5	3.0	20.5
Silty Sand to Sandy Silt Till, compact	0.33	0.5	3.0	21.0
Sand, dense	0.30	0.45	3.3	21.0
Gravelly sandy gravel, compact	0.30	0.45	3.3	21.0

The shoring system will need to be designed by a Professional Engineer experienced in this type of work. The shoring system may require the use of temporary tiebacks due to the height of fill to be supported (i.e. in excess of 4 m). We will be pleased to provide information on this aspect, if necessary.

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

5.1.2 TUNNELLING

We understand that the invert of the pipe culvert will be at about El. 372.8 m. Boreholes C6-2 and C6-3, located near the inlet and the outlet, were both drilled from o.g. elevation of 373.1 m, which is only approximately 0.3 m above the proposed invert level. These boreholes show the presence of very weak and/or organic soils to Elevations of 372.4 and 371.6 m, respectively. Below this, alluvial deposits of silty fine sand and silty fine sand to sandy silt were encountered to Elevations of 370.1 and 369.3 m, respectively.

Borehole C6-1, which was drilled from the right (east) shoulder of the roadway, contacted embankment fill to El. 373.0 m followed by a compact glacial deposit consisting of silty sand to sandy silt till to El. 371.1 m.

These show that near the culvert entrance the tunneling will proceed through, near the bottom, within organic soils and possibly in the underlying compact alluvial silty fine sand. Under the embankment (based on Borehole C6-1) sand fill would be encountered throughout much of the tunnel cross section, with till deposit at or near the invert elevation. However, the composition of the embankment fill and the underlying natural soil may change across the road. At the outlet, a very weak silty clay fill with some organics was

contacted in Borehole C6-3 followed by loose to compact silty fine sand to sandy silt, similar to inlet conditions.

From this information, it can be surmised that the tunnel can be expected to possibly proceed on mixed face conditions.

A classification of soils for tunneling purposes, commonly used in Ontario, is given in Appendix E. According to this, the above described soils fall into the following categories.

- Peat/organic silt (Borehole C6-2) and silty clay fill (Borehole C6-3): "Very Soft Squeezing".
- Sand with traces to some gravel (embankment fill) and Silty Fine Sand (Borehole C6-2) and Silty Fine Sand to Sandy Silt (Borehole C6-3): "Cohesive Running" to "Running" above groundwater level (GWL) and "Flowing" below GWL.
- Silty Sand to Sandy Silt Till (Borehole C6-1): "Slow Ravelling" above GWL and "Fast Ravelling" below GWL.

These are not favourable soil conditions for tunneling, especially below the groundwater level. At the time of our investigation, the groundwater level was recorded at about El. 373.1 m. From this, it can be concluded that the groundwater level will likely need to be somewhat lowered, since the invert of the pipe at about 0.3 m below this elevation.

In addition, cobbles and boulders may be present in the embankment fill, as well as in the glacial till deposits and also possibly within the alluvial deposits.

Another problem that may arise with regards to tunneling operations is shaft construction at the entrance and possibly at the exit points. The site and especially the granular deposits will need to be dewatered for this purpose, which will be difficult to implement. As well neither the surficial organic soils nor the very weak silty clay fill will provide any thrust block support to speak of. The underlying fine grained granular soils will provide some support but after some yield (i.e. lateral movement).

5.1.2.1 TUNNELLING OPTIONS

The following are possible typical options for tunneling which are commonly used in Ontario.

- Jack and Bore
- Tunnelling with Hand Mining methods
- Pipe Jacking with TBM
- Micro-Tunneling
- Pipe Ramming
- Horizontal Directional Drilling (HDD)

The selection of a preferred option will depend, among other factors, the construction cost, practicability of construction, risk of ground subsidence, scheduling, etc.

The following table represents an overview of the six aforementioned methods and is intended only to assist the designers in their choice of most suitable method. However, contractors may come up with more suitable options or variations of such methods.

Table 5.1.2.1.1 Summary of Tunnelling Options

Construction option	Comments	Recommendations
Jack and Bore	Dewatering is required before and during the construction	May be costly due to dewatering, possible boulders in the embankment fill and thrust block construction as well as due to rather large pipe diameter.
Hand Mining	Some dewatering is required before and during the construction.	The sand and gravel layer/lense encountered in the embankment fill may create problems (i.e. running conditions) during hand mining. Hand mining can be expected to be a relatively slow operation.
Pipe Jacking with TBM using earth pressure balance	Considered uneconomical	May be objectionable based on cost
Micro-Tunnelling	Considered uneconomical	May be objectionable based on cost.
Pipe Ramming	Dewatering may be required before and during the construction. The proposed diameter is in the upper limits of presently available equipment in Ontario	May be objectionable based on the need for dewatering and vibrations induced during construction. Boulders may create problems, if encountered.
Horizontal Directional Drilling (HDD)	The proposed diameter (i.e. 1500 mm) is in the upper limits of presently available equipment in Ontario. May require dewatering.	May be suitable if entrance and exit points can be established some distance beyond the embankment (i.e. property availability). Presence of cobbles and boulders may create problems, if encountered.

Details of each option are briefly discussed below.

5.1.2.1.1 JACK AND BORE METHOD

Jack and bore method forms a borehole from a drive shaft to a reception shaft by means of rotating cutting head. Spoil is transported back to the drive shaft by helical auger flights rotating inside a steel casing. The casing is jacked in place simultaneously with the augering operation. After the installation of the steel casing, the utility pipe is installed inside the casing and the gap between the casing and the pipe is grouted. The maximum

casing diameter used in this operation is typically limited to about 1.5 m for most contractors in Ontario.

From the findings of the boreholes, the tunneling operation can be expected to go through a mixed face, consisting of embankment fill consisting of sand with traces to some gravel at Borehole C6-1 location about El. 373.0 m and silty sand to sandy silt till, or silty fine sand or silty fine sand to sandy silt. The groundwater level at the time of our investigation was recorded at El. 373.1 m (i.e. about 0.3 m above the invert level).

The sand fill can be expected to be unstable (i.e. running soil) during the construction in the upper mid portions of the face and will require special precautions. The underlying natural soils can be classified as flowing or running or fast raveling soils below the water table and will thus require dewatering to stabilize. Dewatering was discussed in Section 5.1.1.4 of this report and will be further discussed later on in Section 5.1.3.

Another aspect of tunneling with this method is that the construction of shafts in the water bearing silty sand, silty sand to sandy silt and especially the underlying sand/gravel soils (which were encountered below about El. 369-371 m), can be expected to be difficult. In addition, the surficial soils encountered in Boreholes C6-2 and C6-3 will provide little or no passive resistance for a thrust block to facilitate jacking operations (i.e. deep foundations may be required) and these aspects can be expected to increase the cost of tunneling by this method. In addition, allowance may need to be made for the possibility of the presence of boulders.

We recommend that a specialist contractor(s) be consulted for tunnelling by jack and bore method, with the anticipated soil conditions. Consideration can also be given to drill additional borehole(s) to further investigate the composition and condition of the existing embankment, to reduce risk factors.

5.1.2.1.2 TUNNELLING BY HAND MINING

In a hand mining operation, the excavation of the tunnel is accomplished manually and a temporary ground support system is required during the operation. The temporary ground support system can be steel or concrete segmental liner or steel ribs with wood lagging. Groundwater control may be required to minimize water leakage into the tunnel. Workers are required inside the tunnel to perform the excavation and/or spoil removal. The excavation will be accomplished by hand mining with the assistance of small excavation tools.

With this method, control of alignment and grade is accomplished by overmining in the direction of the change and the pipe will move into the overmined area as it is pushed forward.

The main advantage of this technique is that it can be economical, and that large boulders can be removed.

This technique is limited by the difficulty of controlling the grouting quality and its impact to the environment. Higher risk of ground subsidence may also be encountered with anticipated settlements could be in the order of 20 to 40 mm. In addition, with this method, the tunnel project will probably take a longer time to complete, in comparison with many of the other methods mentioned.

Similar to jack and bore method, the presence of sand fill of the advancing face can create problems, as well dewatering will be required to lower the groundwater level to below the invert level. This method will likely be suitable for this project and it too should be discussed with a specialist contractor(s).

5.1.2.1.3 PIPE JACKING WITH TBM

The method of pipe jacking with TBM installs a prefabricated pipe through the ground from a drive shaft to a reception shaft. The pipe is pushed by jacks located in the drive shaft and the jacking force is transmitted through the pipe to the face of the excavation. The excavation with this method is accomplished by a TBM (Tunnel Boring Machine) and the spoil is transported out of the jacking pipe and shaft manually or mechanically. Typically, pipe jacking with TBM is applicable to tunnels with relatively larger diameter (e.g. 1 to 3 m).

This tunneling method is so versatile that it can be executed with virtually any ground conditions (except large boulders) with adequate precautions. In unstable soil conditions (i.e. silty sand to sandy silt below the groundwater level), an Earth Pressure Balance Machine (EPBM) or slurry shield TBM is required to counterbalance the ground and hydrostatic pressures and to minimize ground subsidence. With EPBM, dewatering of the wet silty sand to sandy silt and/or the glacial till would be less stringent.

The main disadvantage of this method is its high capital and set-up costs. This technique also requires good operator skill and experience. In addition, this method has a very tight alignment and grade tolerance since the permanent lining (the pipe segments) is being installed during the tunnel operation. If large boulders are encountered, hand-mining may have to be employed which could lead to project delay and extra costs.

With pipe jacking and the use of EPBM, if operated properly, the maximum ground settlement is expected to be minimal in the order of 10 to 20 mm, which is considered acceptable under the highway.

In our opinion this method will not be cost effective in the present case, and is therefore not recommended based on cost factor.

5.1.2.1.4 MICRO-TUNNELLING

This technique is the improvement of the pipe jacking technique with TBM. It is a remotely controlled, guided pipe-jacking process that provides continuous support to the excavation face. The guidance system usually consists of a laser mount in the drive shaft, communicating a reference line to a target mounted inside the tunnelling machine. This technique provides ability to control excavation face stability by applying mechanical or fluid pressure to counterbalance the earth and hydrostatic pressures.

The main advantage of this technique is that it is sophisticated and will most likely complete the project in shorter time. It will also complete the tunneling operation with even less ground subsidence, if operated properly, which is estimated to be less than 12 mm.

The main disadvantage of this method is its very high cost and it is therefore not recommended based on economics.

5.1.2.1.5 PIPE RAMMING

In a pipe ramming operation, a pneumatic ramming tool attached to the rear of a steel casing drives the casing into the ground with repeated percussive blows. The installed pipe usually has an open end that allows the soil to enter the casing during the installation. The spoils inside the casing can be removed either during or after the installation, by auger, compressed air or water jetting. After completing the installation, the culvert pipe is installed inside the casing and the gap between the casing and the pipe is grouted.

This method of installation is mostly used with pipes less than 1.5 m but up to 1.8 m in diameter may be feasible and for pipe installation over relatively short distances (i.e. less than 45 m). Although longer distance (up to 100 m) has been achieved in favorable ground condition (i.e. firm to stiff clayey soil), the length of the tunnel installed by pipe ramming is limited by the ground conditions. We recommend that the feasibility of this method be discussed with a specialist contractor. In particular, the presence of mixed face conditions, the presence of potentially unstable sand, the effect of the water level in the underlying sandy silt (i.e. whether dewatering is required) as well as possible presence of boulders. Another disadvantage of pipe-ramming is the vibrations created and noise generated, which may be objectionable while the traffic is maintained on the highway. As well vibrations may cause settlements due to pipe ramming, including the possibility of the liquefaction of the fine grained granular natural soils. We recommend that this too should be discussed with a specialist contractor(s), as well as the need for additional borehole(s).

5.1.2.1.6 HORIZONTAL DIRECTIONAL DRILLING (HDD)

This method consists of pilot boring, back reaming and pipe pulling. Drilling begins with a small diameter pilot hole along a designated alignment, using flexible drill rods with remote controlled steering system. After the pilot boring, a back reamer is installed and drilled back

through the pilot hole to achieve the required diameter for the pipe to be installed. Typical ratio of diameter of reamer to pipe is 1.3 to 1.5. Special drilling fluid is used to prevent the collapse of borehole as well as providing a lubricant for the drilling and flushing spoils. Selection of reamer and drilling fluid highly depends on the ground conditions.

The feasible diameter of pipe or tunnel in this method is 0.5 to 1.5 m and drive length can be up to 2000 m for smaller diameter pipes. The subsurface conditions encountered in the boreholes may be feasible for a horizontal directional drilling operation if the drilling mud would support the soil and prevent a 'flowing' or 'ravelling' ground condition into the hole during the drilling.

Reference should be made to OPSS 450 for this method, as well for related specifications.

Often the HDD installation could be impeded by obstructions in the ground, such as boulders. In the present case, no boulders were encountered in the zone where tunneling would be carried out. If the embankment fill was placed under proper engineering supervision, normally boulders should not exist. However, the contractor would need to be prepared to deal with such a condition, should it occur. In addition, boulders may be present in the glacial till and even in the alluvial soils depending on the water velocity in the watercourse. These and other details such as the need for dewatering and property requirements should be discussed with a contractor specializing in this method of trenchless construction. Proper distance for entry and exit points and property requirements during the construction will need to be sorted out to determine the feasibility of this method.

5.1.3 DEWATERING FOR TUNNELLING

The design of the dewatering system for tunneling will depend on the required draw-down for the selected construction (tunnelling) method. Since the draw-down will need to be effected by pumping from the toe of the embankment (i.e. dewatering from the surface of the traveled portion of the road will unlikely be possible), the dewatering system will need to be carefully designed and executed to effect the desired draw-down, especially at a point near the centerline of the road (i.e. the farthest away from the dewatering location). The gravelly sand to sandy gravel deposit underlying the site below about El. 369-370 m is a coarse grained soil (i.e. pervious material) and as such a significant amount of water may need to be pumped from within this deposit to effect the required draw-down. Consideration can also be given to well pointing for dewatering purposes. As was mentioned before, we recommend that this responsibility be assigned to the contractor via an NSSP. In addition, although this is unlikely, any effects of dewatering on the nearby structures, services and wells may need to be studied.

5.1.4 DESIGN PARAMETERS

The bulk units for the soil types were provided in Section 5.1.1 of the report. For soils surrounding the tunnel, the ratio of the horizontal to vertical earth pressure K , can be taken as 0.5 (i.e. $K=0.5$), for design purposes.

5.1.5 SETTLEMENT DUE TO TUNNELLING

Settlements caused by tunneling in the overburden soils are the aggregate of settlement caused by ground loss due to over-excavation or ground relaxation in response to excavation, and settlement due to the deformation of flexible primary tunnel support.

The first type of settlement is the direct result of the movement of ground into the tunnel heading. The factors which influence the magnitude of this settlement include soil strength and stiffness, the method of tunnelling and the quality of tunnel operations (including the method of handling localized conditions such as removing boulders). Even when tunneling is carried out apparently through homogenous soils with the same equipment and crews, ground settlements typically vary by a factor of 2 or 3. This variation can be ascribed to items such as ploughing of the shield or use of overcutters, quality and speed of ring grouting and localized variations in soil type, strength or stiffness.

In summary settlement (or heave) of the road surface due to tunnel construction is difficult to estimate but generally with good workmanship settlements over the tunnel centerline should not exceed 25 mm. Beyond the centerline the settlements would gradually decrease to zero, typically at a distance of about two diameters.

In addition to these settlements, settlements due to dewatering (if any) will need to be taken into consideration. The magnitude of settlement due to dewatering will depend on the depth of the lowering the water table. In this case, since the depth of dewatering is expected to be less than about 1.0 m (except at shaft location(s)) and the soil below about El. 371 m is relatively competent (i.e. relatively incompressible), the amount of settlements due to dewatering should not exceed 10 mm.

The term 'ground loss' as discussed above does not include unexpected, uncontrolled loss of ground resulting from face instability. Much greater settlements than the estimates given above will occur where there is instability at the face.

To ensure that ground settlements are limited to acceptable values, it is recommended that ground movements along the tunnel in critical areas be monitored during and following the tunneling operation.

Settlement monitoring could consist of paint mark points on the pavement along the centerline of the culvert and beyond the culvert. Surface settlement points should also be installed beyond the paved portion (i.e. in the shoulder). In addition, we recommend that

consideration be given to deep settlement points (e.g. placed about 0.8 m above the tunnel's crown) in the shoulders. This is because deep settlement points will react to any ground loss settlement during tunneling much faster than surface settlement points, since time-dependent arching effect is less pronounced immediately above the crown. The settlements will need to be monitored with reference to reliable, frost free benchmark(s).

We recommend that a minimum of three sets of repeatable baseline readings be taken on all of the settlement points well in advance of the start of tunneling. Settlement monitoring should be conducted at least three times daily during the installation of the tunnel under the road embankment. The frequency of readings can then be further reduced to once daily for ten days, weekly for a period of one month and then once monthly for five months following grouting of the tunnel annular space.

Should settlement monitoring indicate excessive ground movement prior to the tunnel reaching the traveled lanes, immediate changes to the tunneling and ground support procedures must be adopted. Table 5.1.5.1 details the recommended monitoring 'Alert Levels.'

Table 5.1.5.1 Recommended Settlement 'Alert Levels'

Ground Movement	Notes
<6 mm	Proceed. No action required.
6-15 mm	Immediately notify the MTO and the geotechnical engineer for further assessment. Proceed with caution.
>15 mm	Halt mining and bulkhead the tunnel face until further assessment is carried out by the MTO and the geotechnical engineer; Carry out immediate remedial work to the settlement zone as approved by the MTO and geotechnical engineer.

In addition to settlement monitoring, during tunneling, the quantity of the excavated soils should also be monitored and compared with the theoretical volume of excavation in order to assess the risk of excessive over-excavation.

5.2 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 below some organic deposits, very weak silty clay fill and possible other alluvial deposits at the watercourse level, the boreholes indicate that the native soils can be expected to consist of silty sand to sandy silt. The silty sand to sandy silt is considered to be a highly erodible and frost-susceptible soil.

We recommend that concrete cut-off (apron) be constructed both at the inlet and outlet to prevent seepage beneath and around the culvert, especially through the granular bedding, granular backfill around the culvert and the underlying pervious silty sand to sandy silt. Beneath the culvert, the concrete cut-off wall should extend to a suitable depth (e.g. below any possible scour depth). Consideration may also be given to an impervious seal at the inlet, outlet and at an intermediate location.

At the inlet, consideration may also be given to the use of a clay seal. The purpose of the clay seal is to ensure that the 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 is typically 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. Typically, the clay seal is protected by laying a 0.6 m thick rock protection over it. The clay seal would generally be extended at about 6 to 8 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/or impervious seal or in conjunction with these, a 0.6 m thick rock protection, consisting typically of 300 mm size rock can be considered. If and where the subgrade is found to consist of sandy silt or organic soils, a layer of granular filter material should be used. This would generally be extended about 8 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 (where necessary) can 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 Culvert Outlets.

5.3 PROPOSED RETAINING WALLS

We understand that the widening of the embankment to provide a north bound passing lane on the east side of the highway will entail the construction of retaining walls due to property restrictions. As shown on the stratigraphic section, Drawing No. 6B, the top of the retaining wall will be at El. 376.5 \pm m. As the o.g. level within the primary watercourse valley is about 373.0 \pm m, the height of the retaining walls within the primary valley will be about 3.5 m. Immediately behind the wall, the ground surface will be sloped up at approximately 3H:1V configuration as the road level elevation is about 379.0 m, on the east side. The height of the retaining wall can be expected to reduce beyond the primary valley, as the o.g. level gradually rises by 1 to 2 m.

Boreholes C6-RW1 through C6-RW7 drilled for the retaining walls, along with Boreholes C6-1 and C6-2 (which were put down for the culvert) show below some surficial organic soils the presence of granular deposits ranging from silty fine sand to sandy silt, silty sand to sandy silt till and sand, gravelly sand/sandy gravel. The groundwater level was found at the time of our investigations at or immediately below the ground surface within the primary valley (lowlands) and typically one to two meters immediately outside the primary valley lands.

5.3.1 FOUNDATION CONDITIONS FOR RETAINING WALLS

We understand that the construction of an RSS (Reinforced Soil System) is proposed but the following geotechnical design information for conventional concrete retaining walls are included for completeness.

The geotechnical resistances which would be available at the borehole location for footings placed on natural competent, approved subgrade are presented below.

Table 5.3.1.1
Recommended Geotechnical Resistances

Borehole No.	Existing Ground Surface Elevation (m)	Re-commended Footing Base (Bottom) Level Below Existing Ground Surface (m)	Re-commended Footing Base (Bottom) Elevation (m)	Factored Bearing Resistance at ULS (kPa)	Bearing Resistance at SLS (kPa)	Subgrade Material
C6-RW1	378.7	1.7 or below	377.0 or below	320	200	Sandy silt to silty sand till
C6-RW2	373.7	0.9 or below	372.8 or below	320	200	Silty fine sand
C6-RW3	373.3	1.0-2.3 2.4 or below	372.3-371.0 370.9 or below	200 320	120 200	Silty fine sand Silty fine sand
C6-RW6	373.2	1.4 or below	371.8 or below	180	100	Silty fine sand to sandy silt
C6-2	373.1	0.9 or below	372.2 or below	200	130	Silty fine sand
C6-1	378.6*	5.8-6.7 6.8 and below	372.8-371.9 371.8 and below	180 230	100 150	Silty sand to sandy silt till Silty sand to sandy silt till
C6-RW4	373.1	0.8 and below	372.3	240	160	Silty fine sand
C6-RW7	374.6	1.5-1.6	373.1-373.0	180	100	Gravelly sand
C6-RW5	375.4	0.8	374.6 or below	320**	200**	Gravelly sand to sandy gravel

*Borehole drilled from the top of road embankment.

** Higher resistance would be available depending on the design. We will be pleased to provide additional information, if required.

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 retaining 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 subgrade surface can be calculated using a friction angle of 28 degrees.

The lateral earth pressures acting on retaining walls will depend on the type and the method of placement of the backfill materials, slope of soil immediately behind the wall and the subsequent lateral movements of the retaining structure. The backfill properties given in Section 5.1.1.3 can be used for design purposes. In addition, traffic loads and vibrations should be taken into consideration.

In addition to reinforced concrete retaining walls, consideration can be given to gravity structures such as gabion or crib type walls, if some lateral yielding would be acceptable.

Consideration can also be given to a permanent soldier pile or a contiguous caisson wall construction. This may be economical since shoring will likely be required (i.e. equipment mobilization) at the site for the construction of the culvert itself. It should, however, be pointed out the coarse grained granular soils as well as cobbles and boulders, if encountered, would slow down the progress of drilling (augering) operations for the caissons, and especially tieback installation (if and where required), thus increasing the cost.

5.3.2 REINFORCED SOIL SYSTEM (RSS) TYPE WALLS

As was mentioned before RSS type walls are proposed by UMA for this project. In principle, a reinforced soil system consists of tying vertical facing units into a soil mass, with their tensile strips. It consists of four elements:

- A soil backfill
- Tensile reinforcing strips
- Facing elements at boundaries
- Mechanical connections between reinforcing elements

The soil backfill is generally a granular material with not more 10 to 12% by weight passing #200 mesh size sieve. It should not contain materials corrosive to reinforcing strips. Within the reinforced zone, the soil is able to stand at much steeper slopes than possible without reinforcing.

RSS type walls can be utilized after the removal of existing fills and all organic or organic rich soils and say underlying weak or otherwise unsuitable soils (to the surface of sufficiently competent natural, inorganic soils) and their replacement with properly compacted

acceptable engineered fills. The recommended strip depths/elevations as well as the highest founding depths/elevations for the foundations of the facing at the borehole locations are tabulated below:

Table 5.3.2.1
 Recommended RSS Stripping Depths and Highest Facing Wall Foundation Structures

Borehole No./ Existing Ground surface elevation(m)	Recommended Stripping Depth/Elevation (m)	Recommended Founding Depth/Elevation for Footing of Facing Wall (m)	Subgrade soil
C6-RW1/378.7	1.3/377.4	1.6/377.1	Silty sand to sandy silt till
C6-RW2/373.7	0.7/373.0	0.8/372.9	Silty fine sand
C6-RW3/373.3	0.9/372.4	1.0/372.3	Silty fine sand
C6-RW4/373.1	0.6/372.5	0.8/372.3	Silty fine sand
C6-RW5/375.4	0.5/374.9	0.7/374.7	Silty fine sand
C6-RW6/373.2	1.4/371.8	1.6/371.6	Silty fine sand to sandy silt
C6-RW7/374.6	0.5/374.1	1.2/373.4	Sandy silt
C6-2/373.1	0.8/372.3	0.9/372.2	Silty fine sand

The following geotechnical bearing resistances are available at the borehole locations at the elevations given in Table 5.3.2.1, for a minimum footing width of 1.0 m placed on approved undisturbed inorganic natural soil.

Factored Bearing Resistance at ULS	=	130 kPa
Geotechnical Resistance at SLS	=	100 kPa

Provided that the bearing subgrade is not unduly disturbed during the construction, with the recommended serviceability resistance value, the total and differential settlements should not exceed 25 mm. Good construction techniques, including proper dewatering will be required to achieve this. As well, all bearing surfaces must be inspected, evaluated and approved by a Geotechnical Engineer who is familiar with the findings of this investigation.

RSS is a patented procedure and the provider is responsible for external and internal stability of the system. We have, however, carried out a preliminary global stability analysis, using the following soil parameters.

Table 5.3.2.2
Soil Parameters Used for Preliminary Global Stability Analysis

Soil type	Bulk Unit Weight (kN/m ³)	Angle of Shearing Resistance (degrees)	Shear Strength
RSS Wall Backfill	21.0	Assumed to be solid block	Assumed to be solid block
Existing Embankment fill	20.5	31	0
Silty fine sand and silty fine sand/silty sand	18.0	31	0

In our analysis, the RSS was assumed to be a minimum 3.0 m wide solid block (i.e. the failure surfaces do not intersect it due to its very high strength). The stability analysis results are presented in Appendix F. The calculated factor of safety is about 1.46 (assuming water level to be at about El. 373.0 m) which indicates that an RSS wall with at least 3 m long reinforcing strips is feasible (See Figure F-1 in Appendix F) but this should be verified by the supplier/contractor when details are known. Figure F-2 shows an assumed higher water level, in which case the safety factor is reduced to about 1.40.

After stripping and inspection of the exposed subgrade and its approval, where feasible it should be proofrolled from the surface before placing engineered fill. This will require proper dewatering.

RSS wall should be constructed in accordance with Special Provision No. 599S22 and 599S23.

5.4 EMBANKMENT WIDENING

Based on the latest design information provided by UMA (as discussed before), a new northbound passing lane is to be constructed, which includes the location of Culvert C6.

The anticipated stripping depth for the placement of the embankment fill was given in Table 5.3.2.1 of the previous section of this report. It should be pointed out that the table presented is for 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 (i.e. about 3 m widths perpendicular to the existing embankment) in order not to cause instability of the existing embankment.

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

The fills used to raise the grade and to construct the embankment 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 be 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 and the width of the widenings. For a typical embankment widening width of 5 m and grade raise of 4 m (see Drawing 6B), 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, the height of embankment is expected to be less and on this basis, the anticipated foundation settlements would be less. 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 25 mm for an embankment height of 3-4 m. The total of the two settlements would typically be about 75 mm within the primary flood plain becoming less beyond. 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. This will also reduce differential settlements between the section of the new widening within the primary valley of the watercourse and the sections beyond. Alternatively, a surcharging of about 1 m for a period of six to eight weeks can also be considered.

5.5 BEARING SURFACES

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

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 G, are an integral part of this report.

SHAHEEN & PEAKER LIMITED


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Appendix E

Tunnelman's Ground Classification and Probable Working Conditions

Tunnelman's Ground Classification and Probable Working Conditions

Soil Classification	Representative Soil Types	Tunnel Working Conditions
Hard	Very hard calcareous clay; cemented sand and gravel	Tunnel heading may be advanced without roof support
Firm	Loess above GWT; Various calcareous clay with low plasticity	Tunnel heading may be advanced without roof support. Permanent support can be constructed before the ground will start to move
Slow Ravelling and Fast Ravelling	Fast raveling occurs in residual soils or in sand with clay binder below the GWT. Above the GWT, the same soils may be Slow Ravelling or even Firm	Chunks of material may drop out of the crown or the sides some time after the ground has been exposed, in Fast Ravelling ground, the process starts within a few minutes; otherwise it is classed as Slow Ravelling
Squeezing	Soft or medium-soft clay	Ground slowly advances into tunnel without fracturing and without perceptible increase of water content in ground surrounding the tunnel.
Swelling	Heavily pre-compressed clays with a plasticity index greater than 30. Sedimentary formations containing layers of anhydrite.	Like squeezing ground, moves slowly into tunnel, but the movements is associated with a very considerable volume increase in the ground surrounding the tunnel.
Cohesive Running and Running	Occurs in clean, fine moist sand Occurs in clean, coarse or medium sand above the GWT	Removal of the lateral support of any surface rising at an angle of more than about 34° to the horizontal is followed by a 'run,' whereby the material flows like granulated sugar until the slope angle is approx. 34°. If the 'run' is preceded by a brief period of raveling, the ground is called Cohesive Running
Very soft Squeezing	Clays and silts with high plasticity indices	Ground advances rapidly into the tunnel in a plastic flow
Flowing	Any ground below the GWT that has an effective grain size in excess of about 0.005 mm	Flowing ground moves like a viscous liquid. It can invade the tunnel not only through the roof and the sides but also through the invert. If the flow is not stopped, it will eventually completely fill the tunnel
Bouldery	Boulder glacial till; riprap fill; some land slide deposits, some residual soils. The matrix between boulders may be gravel, sand, silt, clay and in any combination	Problems incurred in advancing shield or in forepoling; blasting or hand mining ahead of machine may become necessary.

Appendix F

Slope Stability Analysis Results

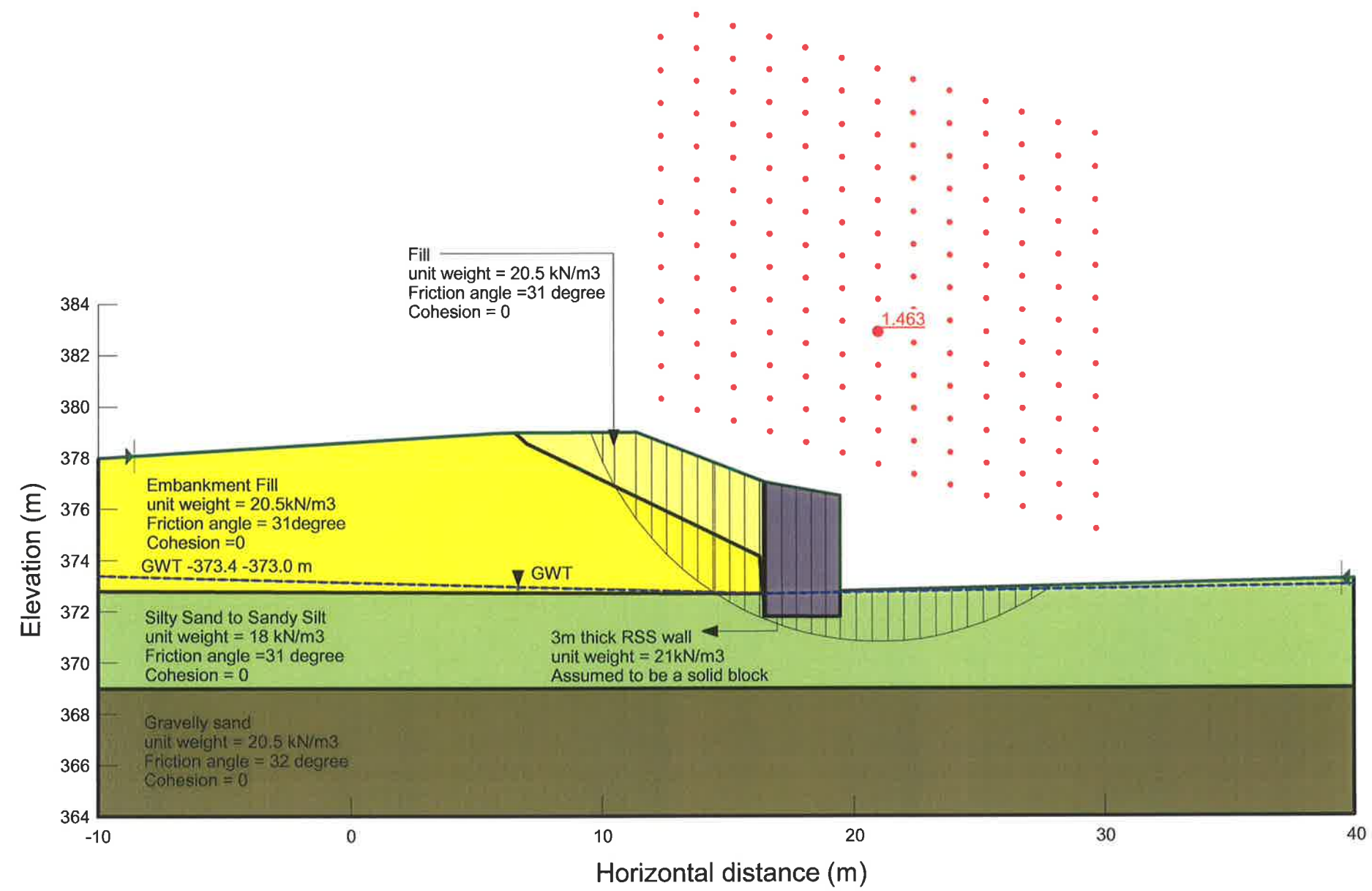


Figure F-1

Partially submerged case

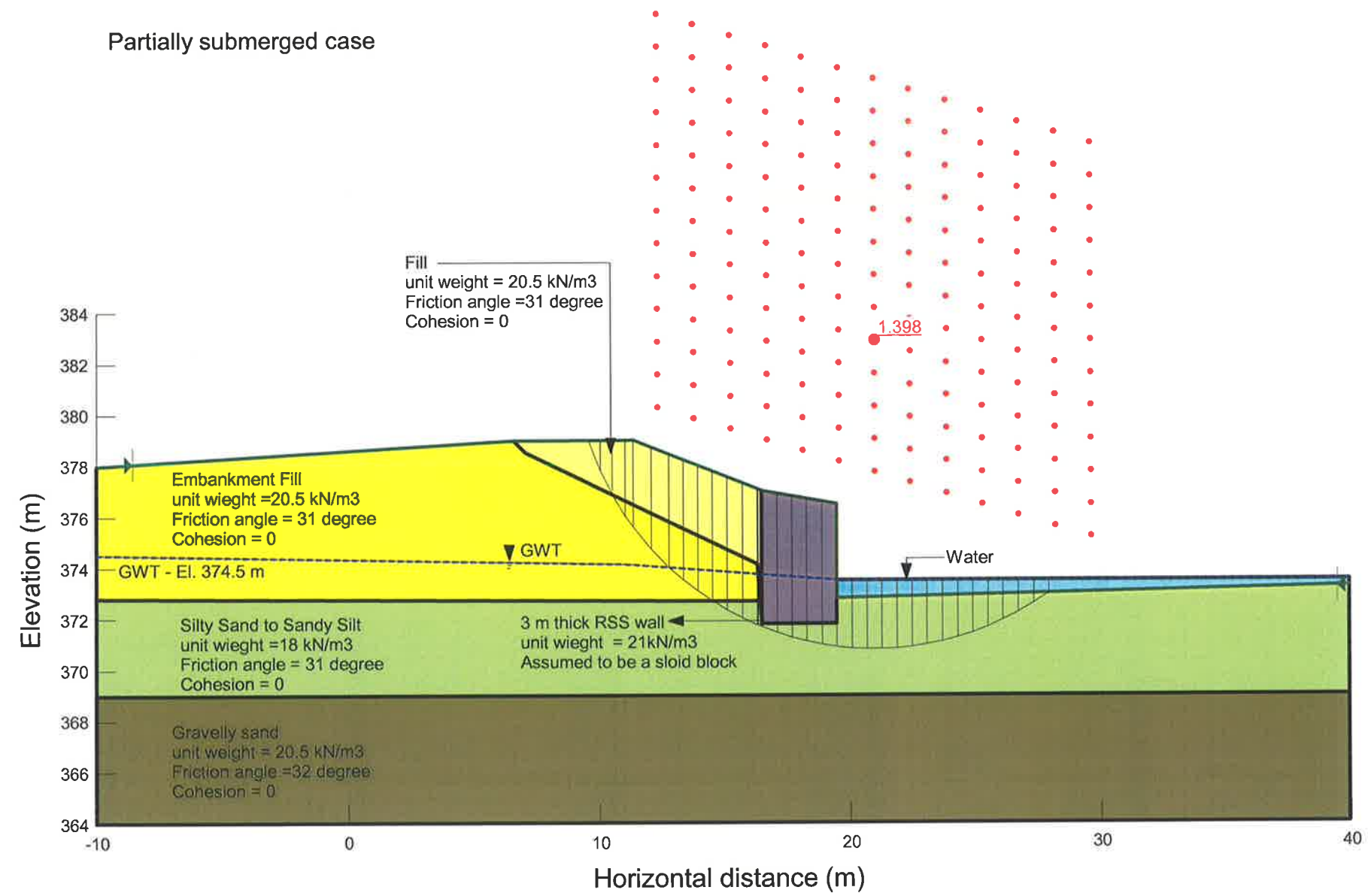


Figure F-2

Appendix G

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.

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