



FOUNDATION INVESTIGATION AND DESIGN REPORT  
THE REPLACEMENT OF WEST DUFFINS CREEK BRIDGE ON HIGHWAY 7,  
CITY OF PICKERING, REGIONAL MUNICIPALITY OF DURHAM, ONTARIO  
W.P. 280-91-00  
AGREEMENT NUMBER 2006-E-0075

Submitted to:

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## TABLE OF CONTENTS

	Page
1.0 INTRODUCTION.....	1
2.0 SITE DESCRIPTION.....	1
3.0 QUATERNARY GEOLOGY.....	2
4.0 INVESTIGATION PROCEDURES.....	2
4.1 Field Investigation.....	2
4.2 Laboratory Tests.....	3
5.0 SUB-SURFACE CONDITIONS.....	3
5.1 Topsoil.....	4
5.2 Asphaltic Concrete.....	4
5.3 Sand and Gravel / Sand / Silty Sand Fill .....	4
5.4 Sandy Silty Clay Fill .....	5
5.5 Silty Sand / Sandy Silt / Clayey Sandy Silt .....	5
5.6 Clayey Silt / Silt.....	6
5.7 Silty Clay / Clayey Silt Till.....	7
5.8 Silty Sand / Sandy Silt Till .....	7
5.9 Sand / Silty Sand / Silt .....	8
5.10 Groundwater .....	8
6.0 DISCUSSIONS AND RECOMMENDATIONS.....	9
6.1 Piled Foundation .....	13
6.1.1 Resistance to Axial Loads .....	13
6.1.2 Resistance to Lateral Loads .....	15
6.2 Retaining / Wing Walls.....	18
6.2.1 Retaining Wall Type .....	18
6.2.2 Lateral Earth Pressures.....	18
6.3 Approach Embankment.....	19
6.4 Construction Staging and Detour .....	21
6.5 Construction Comments.....	22
6.6 Construction Inspection.....	23
7.0 LIMITED ENVIRONMENTAL INVESTIGATION .....	23
8.0 SOIL CORROSIVITY.....	23
9.0 CLOSURE.....	25

REPORT LIMITATIONS

DRAWING Nos. 1, 2A to 2C

## **APPENDICES**

- APPENDIX A: Record of Boreholes
- APPENDIX B: Laboratory Test Results
- APPENDIX C: Drawings for Existing and New Bridge Foundations
- APPENDIX D: Borehole Logs extracted from MTO report dated December 1965  
(W.J. 65-F-122 / W.P. 570-56, GEOCREs No. 30M14-41)
- APPENDIX E: Results of Slope Stability Analyses
- APPENDIX F: Site Photographs
- APPENDIX G: Non Standard Special Provision

## **1.0 INTRODUCTION**

AMEC Earth & Environmental, a division of AMEC Americas Limited (AMEC), Consulting Geotechnical, Construction Quality Control and Environmental Engineers, was retained by the Ministry of Transportation (Central Region) to conduct a foundation and pavement investigation for the replacement of West Duffins Creek bridge on Highway 7, City of Pickering, Regional Municipality of Durham in Ontario.

Nine (9) borehole and four (4) core locations and depths for foundation and pavement investigation in the vicinity of the existing bridge area were specified by MTO in the Terms of Reference as outlined in the Request for Quotation (Agreement Number: 5006-E-0075 dated March 2007). Authorization to proceed with this investigation was signed by the Regional / Branch Director of MTO dated 18 May 2007. The work was carried out by AMEC according to the MTO Terms of Reference and AMEC's Proposal No. P27132 dated 1 May 2007.

A previous geotechnical report at the existing bridge site was provided by MTO (W.J. 65-F-122 / W.P. 570-56, GEOCRETS No. 30M14-41), describing five (5) boreholes drilled at the site at the end of 1965 (Appendix D). Additional information (i.e., drawings for the existing and new bridges) was also forwarded to AMEC. Such information was used in preparing this report wherever applicable.

The investigation was carried out by means of a limited number of boreholes, in-situ tests and laboratory tests on selected samples. Based on AMEC's interpretation of the data obtained, recommendations for the new bridge foundation are provided. The results of the pavement investigation are presented in a separate report (i.e., Pavement Field Investigation Report prepared by AMEC – Reference No. TT73061 dated 7 September 2007).

## **2.0 SITE DESCRIPTION**

The site for the foundation investigation was the existing bridge located on Highway 7, crossing West Duffins Creek in the City of Pickering, Ontario, as shown in the site location plan (Drawing No. 1). At the time of investigation, the existing two-lane bridge was about 40 m in length and 13 m in width. The bridge was approximately 6 m to 7 m above the creek. The creek banks at the bridge location were covered by rip-rap, trees and bushes (site photographs in Appendix F).

The existing bridge was planned to be replaced by a new bridge with a length of about 43 m and a width of 14 m with a possible 7.5 m wide future widening along both sides of the new bridge. The new bridge would be supported by an integral abutment – pile system to be constructed behind the existing piled foundations, away from the creek banks.

### **3.0 QUATERNARY GEOLOGY**

According to the Quaternary Geology – Toronto and Surrounding Area - Southern Ontario, the vicinity of site is covered with Young Tillis comprising clayey silt till and sandy silt till.

### **4.0 INVESTIGATION PROCEDURES**

#### **4.1 Field Investigation**

In accordance with the Terms of Reference for this investigation, seven (7) borehole locations (BH 6 to BH 12) in the vicinity of the existing bridge were to be investigated for foundation investigation and two (2) boreholes (BH 13 and BH 14) with seven (7) cores (Core 1 to Core 7) for pavement investigation. The additional cores (Core 5 to Core 7) were requested by MTO. The boreholes (i.e., BH 6 to BH 14) were numbered after the existing five boreholes (BH 1 to BH 5) already drilled in 1965 for the design and construction of the existing bridge.

The fieldwork was performed from 3 July 2007 to 25 July 2007, starting with staking out the borehole locations, followed by clearing underground utilities, acquiring all necessary permits for road occupancy, and drilling the boreholes using trunk/track-mounted drilling rigs. The drilled borehole locations were surveyed in the field by an Ontario Land Surveyor firm (Avantis Surveying Inc.).

The borehole locations are presented on Drawing Nos. 2A to 2C. The coordinates and the geodetic ground surface elevations at the specified borehole locations were surveyed by the surveyor after drilling.

The borehole investigation was under the full-time supervision of experienced geotechnical personnel from AMEC.

The boreholes were advanced at least to the anticipated depths specified in the Terms of Reference. The deepest borehole was drilled to a depth of about 15.7 m.

Soil samples were normally taken at 1.5 m intervals during the performance of Standard Penetration Test (SPT) in accordance with ASTM D1586. This consisted of freely dropping a 63.5 kg (140 lbs.) hammer for a vertical distance of 0.76 m (30 inches) to drive a 51 mm (2 inches) diameter O.D. split-barrel (split spoon) sampler into the ground. The number of blows of the hammer required to drive the sampler into the relatively undisturbed ground by a vertical distance of 0.30 m (12 inches) was recorded as SPT 'N' value of the soil which indicated the consistency of cohesive soils or the relative density of non-cohesive soils. The 63.5 kg (140 lbs.) hammer was used in both the truck/track-mounted drilling rigs. The majority of the boreholes were terminated due to SPT 'N' value in excess of 100 blows per 0.3 m.

At one borehole (BH 11) which encountered relatively-soft clayey soil, a MTO Field Vane Test was carried out. However, the soil shear strength was too high to be measured by the Vane Test.

Soil samples were normally collected for each soil layer exposed in the test holes for laboratory inspection and testing.

The collected soil samples were screened on site by a gas detector for the total organic vapours (TOV), the results of which are shown in the Record of Boreholes.

Upon completion of drilling, the boreholes were backfilled with bentonite in accordance with the general requirements of Ministry of the Environment Regulation 903 as indicated in the Record of Boreholes.

The soil samples were transported to AMEC's Advanced Soil Laboratory in Scarborough (Toronto) for further examination and laboratory soil testing. The program of laboratory testing included, where applicable, grain size analysis, Liquid and Plastic Limits, in-situ water content determination, and soil corrosivity analysis.

The results of the in-situ and laboratory tests are presented in the corresponding Record of Boreholes (Appendix A) and Laboratory Test Results - Appendix B.

## **4.2 Laboratory Tests**

Representative soil samples were subject to laboratory testing in AMEC's Advanced Soil Laboratory in Scarborough (Toronto) for soil classification. The following tests were conducted:

- Natural water content determination (91);
- Grain size distribution analysis (25); and
- Liquid and Plastic Limits (21).

The results of the laboratory tests are included in the Record of Boreholes in Appendix A. The grain size distribution curves and Liquid / Plastic Limits are shown in Appendix B.

## **5.0 SUB-SURFACE CONDITIONS**

In brief, the soil profile consisted of asphaltic concrete or topsoil underlain by fill soils overlying native soils and / or till deposits. The till deposits were underlain by native sand / silty sand / silt at two borehole locations. Groundwater was encountered in all boreholes (drilled below 2 m depth) from a depth of about 0.9 m to 6.9 m below the existing ground surface. The stratigraphic units and groundwater conditions at the borehole locations are discussed in the

following sections. Detailed information is provided in the Record of Boreholes (Appendix A).

The following summary is to assist the designers of the project with an understanding of the anticipated soil conditions across the site. However, it should be noted that the soil and groundwater conditions may vary between the borehole locations.

## 5.1 Topsoil

The boreholes drilled outside the paved road surface (i.e., BH 6, BH 8 to BH 10 and BH 12) were covered by topsoil with a thickness ranging from approximately 150 mm to 300 mm, except at the location of Borehole BH 12 where the thickness of topsoil was about 910 mm. **It should be noted that the thickness of topsoil varied considerably within the site.**

## 5.2 Asphaltic Concrete

The boreholes drilled within the paved road surface (i.e., BH 7, BH 11, BH 13 and BH 14) encountered asphaltic concrete with a thickness varying from about 190 mm to 230 mm.

## 5.3 Sand and Gravel / Sand / Silty Sand Fill

The asphaltic concrete was underlain by sand and gravel fill with a thickness ranging from about 0.2 m to 0.6 m, except at BH 11 where the thickness of the sand and gravel fill which was underlain by 0.5 m thick sand fill was about 0.1 m.

The topsoil and the sand and gravel fill underlying the asphaltic concrete were underlain by sand and gravel / sand / silty sand fill that extended to a depth of about 1.4 m (Elevation 182.2 m) to 8.2 m (Elevation 176.7 m) below the existing ground surface.

The SPT 'N' values of the sand and gravel / sand / silty sand fill varied considerably from 4 to 49 blows per 0.3 m, indicating very loose to dense relative density. **The fill soils could contain some cobbles / boulders as some stone pieces were noticed in the recovered soil samples.**

The results of laboratory tests conducted on soil samples are as follows:

Natural moisture content (%):	1 to 25
Plastic Limit:	10 to 14
Liquid Limit:	18 to 20

Grain size (8 samples):	Gravel (%):	0 to 58
	Sand (%):	31 to 74

.../...

Silt (%):	9 and 41
Clay (%):	2 and 21

The plasticity indices are plotted in Figure No. B1 in Appendix B and the grain size distribution curves are presented in Figure No. B4 in Appendix B.

## 5.4 Sandy Silty Clay Fill

At the location of BH 7, sandy silty clay fill, possibly pocket, was encountered underlying the silty sand fill from a depth of about 7.6 m to 8.2 m (Elevation 177.4 m to 176.7 m). The SPT 'N' value of the sandy silty clay fill was 5 blows per 0.3 m (firm consistency).

The result of laboratory test conducted on one sample is as follows:

Natural moisture content (%):	17 to 28
Plastic Limit:	13
Liquid Limit:	24
Grain size (1 sample):	Gravel (%): 7
	Sand (%): 21
	Silt (%): 31
	Clay (%): 41

The plasticity index is plotted in Figure No. B1 in Appendix B and the grain size distribution curve is presented in Figure No. B5 in Appendix B.

## 5.5 Silty Sand / Sandy Silt / Clayey Sandy Silt

Native clayey sandy silt was encountered in BH 8 underlying the sand and silty sand fill from a depth of about 2.7 m to 4.0 m (Elevation 176.7 to 175.4 m). The SPT 'N' value was 9 blows per 0.3 m (loose relative density).

Native sandy silt was found underlying clayey silt (Section 5.6) in BH 6 from a depth of about 2.1 m to 4.0 (Elevation 176.7 to 174.8 m), in BH 9 from a depth of about 2.1 m to 4.0 m (Elevation 176.6 to 174.8 m) and in BH 10 from a depth of about 2.1 m to 5.5 m (Elevation 178.0 to 174.7 m). The SPT 'N' values of the sandy silt ranged 1 to 5 blows per 0.3 m (very loose relative density) except in BH 6 at a depth of about 2.5 m where the SPT 'N' value was 12 blows per 0.3 m (compact relative density) and BH 10 at a depth of about 4.8 m where the SPT 'N' value was 31 blows per 0.3 m (dense relative density).

**Some cobbles / boulders were encountered in the boreholes.**

The results of laboratory tests conducted on soil samples are as follows:

.../...



Natural moisture content (%):	10 to 14
Plastic Limit:	10 to 12
Liquid Limit:	16 to 18

Grain size (5 samples):	Gravel (%):	7 to 32
	Sand (%):	26 to 38
	Silt (%):	23 and 42
	Clay (%):	7 and 27

The plasticity indices are plotted in Figure Nos. B1 and B2 in Appendix B and the grain size distribution curves are presented in Figure No. B7 in Appendix B.

## 5.6 Clayey Silt / Silt

Native clayey silt was also encountered underlying the sand fill in BH 9 from a depth of about 1.5 m to 2.1 m (Elevation 177.2 to 176.6 m), and underlying the sand and gravel fill in BH 6 from a depth of about 1.4 m to 2.1 m (Elevation 177.4 to 176.7 m) and in BH 10 from a depth of about 1.4 m to 2.1 m (Elevation 178.8 to 178.0 m). The SPT 'N' values of the clayey silt ranged from 2 to 8 blows per 0.3 m (soft to stiff consistency).

At the location of BH 12, native silt was encountered underlying the sand fill from a depth of about 1.4 m to 2.9 m (Elevation 182.2 to 180.7 m). The SPT 'N' value of the silt varied from 12 to 28 blows per 0.3 m (stiff to very stiff consistency).

The results of laboratory tests conducted on soil samples are as follows:

Natural moisture content (%):	14 to 27
Plastic Limit:	12 to 14
Liquid Limit:	22 to 24

Grain size (1 sample):	Gravel (%):	3
	Sand (%):	28
	Silt (%):	45
	Clay (%):	24

The plasticity index is plotted in Figure No. B1 in Appendix B and the grain size distribution curve is presented in Figure No. B6 in Appendix B.

## 5.7 Silty Clay / Clayey Silt Till

The fill soils in BH 11 was underlain by silty clay / clayey silt till deposits from a depth of about 4.9 m to 8.2 m (Elevation 180.7 to 177.3 m). The SPT 'N' values of the silty clay / clayey silt till varied from 10 to 22 blows per 0.3 m, indicating stiff to very stiff consistency.

The results of laboratory tests conducted on soil samples are as follows:

Natural moisture content (%):	10 to 18
Plastic Limit:	14
Liquid Limit:	25
Grain size (1 sample):	Gravel (%): 4
	Sand (%): 25
	Silt (%): 47
	Clay (%): 34

The plasticity indices are plotted in Figure No. B1 in Appendix B and the grain size distribution curves are presented in Figure No. B6 in Appendix B.

## 5.8 Silty Sand / Sandy Silt Till

Silty sand / sandy silt till was encountered underlying the fill, native soils or silty clay / clayey silt till in all the boreholes drilled below 2 m depth and extended to the borehole termination depths (except in BH 11 and BH 12). The depth to the top of the silty sand / sandy silt till deposits below the existing ground surface varied from about 2.9 m to 5.5 m (Elevation 180.7 to 174.7 m).

The SPT 'N' values of the silty sand / sandy silt till varied widely from 6 to more than 50 blows per 0.3 m. However, the SPT 'N' values at lower depths of the silty sand / sandy silt till deposits were in excess of 50 blows per 0.3 m, indicating very dense conditions.

The till deposits contained some cobbles / boulders as encountered in the boreholes during drilling.

The results of laboratory tests conducted on soil samples are as follows:

Natural moisture content (%):	7 to 28
Plastic Limit:	8 to 12
Liquid Limit:	15
Grain size (6 samples):	Gravel (%): 0 to 9
	Sand (%): 5 to 80

Silt (%):	14 and 85
Clay (%):	3 and 21

The plasticity indices are plotted in Figure Nos. B2 and B3 in Appendix B and the grain size distribution curves are presented in Figure Nos. B6 and B8 in Appendix B.

## 5.9 Sand / Silty Sand / Silt

Native sand / silty sand / silt was found underlying the silty sand / sandy silt till deposits in BH 12 and BH 11, respectively. The sand / silty sand / silt extended from a depth of about 10.1 m (Elevation 173.5 m) in BH 12 and 14.6 m (Elevation 170.9 m) in BH 11 to the borehole termination depths.

The SPT 'N' values of the sand / silty sand / silt varied from 46 to more than 50 blows per 0.3 m, indicating dense to very dense conditions.

The results of laboratory tests conducted on soil samples are as follows:

Natural moisture content (%):	12 to 16
Grain size (3 samples):	Gravel (%): 0 to 11
	Sand (%): 8 to 77
	Silt (%): 5 and 82
	Clay (%): 1 and 7

The grain size distribution curves are presented in Figure Nos. B6 and B8 in Appendix B.

## 5.10 Groundwater

The groundwater level in each open borehole was observed during drilling and measured upon completion of drilling. The measured groundwater levels are shown in the Record of Boreholes (Appendix A).

Groundwater levels were encountered in all the boreholes drilled (i.e., BH 6 to BH 12) as follows:

Borehole No.	Groundwater Depth below Existing Ground Surface, m	Elevation, m
6	1.8	177.0
7	6.7	178.3
8	1.2	178.2
9	2.4	176.3
10	3.1	177.1
11	6.7	178.9
12	0.9	182.7

It should be noted that the groundwater at the site would fluctuate seasonally and can be expected to be somewhat higher during the spring months and in response to major weather events / water levels in the creek.

## 6.0 DISCUSSIONS AND RECOMMENDATIONS

Based on the information provided by MTO (Appendix C), the existing bridge would be demolished and replaced by a new bridge which would be slightly wider and longer than the existing one (from the existing deck width of about 12.8 m to 14.2 m, and from the existing length of about 40.0 m to 42.6 m). Future widening of the new bridge has also been considered. The existing centre line and the profile grades would not be significantly changed. In essence, the existing bridge deck and road width would be widened by about 0.7 m on each side of the existing road, while the bridge deck length would be lengthened by about 1.3 m on each side.

According to the documents on the existing bridge foundation provided by MTO (Appendix C), the existing bridge foundation for each abutment consisted of two rows of driven steel piles (12 B.P. 53). There were nine (9), 1:3.5 battered piles on the front row with a spacing of 4' 9" (1.45 m) and five (5), 1:6 battered piles in the back row at a spacing of 9' 6" (2.90 m). The pile length was estimated to be 30' (9.1 m) each. The top of the existing piles was at approximately Elevation 180.0 m while the tip of the existing piles was at about Elevation 170.9 m. The design load per pile was 60 tons. **It should be noted that the drawings of the existing bridge as provided by MTO that are included in Appendix C of this report have not been clearly indicated that they are "as-built" drawings.**

Based on the Interim Foundation Recommendations provided by MTO (date of meeting – 29 May 2007), the design pile loads for the new bridge are as follows:

Factored axial resistance at Ultimate Limit State	= 1,200 kN
Axial resistance at Serviceability Limit State	= 900 kN

The above design pile loads and the pile type will be confirmed by the results of the present geotechnical investigation.

The two existing bridge abutments would be demolished, leaving a portion of the existing pile caps and piles in place. The new piles would be driven behind the back rows of the existing piles. **If the new piles were driven behind the existing battered piles in the back row, there could be a potential obstruction to the advance and the performance of the new piles. It is therefore imperative that the as-installed battered piles be verified and the new piles be driven at a sufficient distance away from the existing battered piles.**

The new bridge foundation would be an integral abutment design (Appendix C). The new abutment would be partially covered by a 1.5H:1V slope creek bank. The underside of the abutment wall would be at Elevation 181.3 m. The top of the new piles would be at an elevation slightly higher Elevation 181.3 m and the tip of the new piles would be at an elevation that provides the design pile axial resistance.

The bridge approach embankment would be at a slope of 2H:1V with a maximum height of about 5 m. The existing bridge approach embankments would be slightly widened, possibly by about 0.7 m on each side, in order to accommodate the new bridge width.

Based on the soil conditions encountered in the boreholes drilled for this investigation and the existing bridge foundation, deep foundations (i.e., driven steel piles) should be capable for supporting the new bridge structure. Other potential foundation types typically used in Ontario, i.e., shallow foundation (spread/strip footings) and deep foundation (cast-in-place caissons) are possible alternatives. However, based on the comparison of the three foundation types (Table 1), the driven steel piles should be the preferred foundation type.

For piled foundations, two sizes of HP piles (HP 310 x 110 or HP 356 x 132) may be considered with the axial geotechnical reaction at Serviceability Limit State of 900 kN for HP 310 x 110 and 1,100 kN for HP 356 x 132.

All new bank slopes under the new bridge and new bridge approach embankments, if required, should be constructed by compacted engineered fill or equivalent.

The details of the piled foundation and embankment are discussed in the following sections.

**TABLE 1**  
**COMPARISON OF ALTERNATIVE FOUNDATION TYPES FOR BRIDGE REPLACEMENT**

Foundation Type	Description	Advantages	Disadvantages	Risks / Consequences	Cost Comparison
Piled Foundation	Driving steel H piles behind / between the existing steel H piles	<ul style="list-style-type: none"> <li>- Based on the existing piled foundation, driven steel H piles have shown to be constructible and capable of supporting the existing bridge.</li> <li>- Steel H piles are stronger and require less soil displacement during pile driving than other driven pile types (steel pipe, timber, and concrete).</li> <li>- No dewatering required during pile driving.</li> <li>- Pile resistance can be confirmed by pile driving records or pile driving analyzer.</li> <li>- Low post-construction settlement.</li> </ul>	<ul style="list-style-type: none"> <li>- New piles may have to be driven close the existing piles which may obstruct the advance of the new piles, unless the as-built pile position details are known.</li> <li>- Driving piles in glacial till deposits may encounter cobbles/boulders which may obstruct pile advance and/or damage piles.</li> <li>- Variation in pile lengths could be high if the main pile driving criterion is the design pile resistance.</li> </ul>	<ul style="list-style-type: none"> <li>- Risks of encountering existing piles and possible cobbles/boulders in till deposits, which may require relocating piles, adding more piles, preaugering prior to driving piles, etc.</li> <li>- Risks of accurate pile length due to variation in soil conditions and possible presence of cobbles/boulders in till deposits, which may require considerable pile cutting or splicing an additional pile section during pile driving.</li> </ul>	<ul style="list-style-type: none"> <li>- Medium to high. Steel price could change significantly in a short period.</li> </ul>
Augered, cast-in-place caissons	Caissons installed by augering soil, installing temporary steel casing, and filling the augered hole with re-bars and concrete to form caissons.	<ul style="list-style-type: none"> <li>- Augering allows for removing cobbles/boulders if encountered.</li> <li>- If the existing piles are encountered during augering, the piles could be cut (large steel casing will be required).</li> </ul>	<ul style="list-style-type: none"> <li>- For the same axial resistance, caisson cross-sectional areas would typically be larger than those of driven steel piles.</li> <li>- Temporary steel casing will be required to stabilize the sidewalls of the augered hole.</li> </ul>	<ul style="list-style-type: none"> <li>- Risks of bottom heave / disturbance due to groundwater, which would result in reducing the end bearing resistance.</li> <li>- Risks of "necking" in caissons during pulling the steel casing while pouring concrete, resulting in a weak point in the</li> </ul>	<ul style="list-style-type: none"> <li>- Medium to high.</li> </ul>

Foundation Type	Description	Advantages	Disadvantages	Risks / Consequences	Cost Comparison
		<ul style="list-style-type: none"> <li>- Caisson lengths can be specified.</li> </ul>	<ul style="list-style-type: none"> <li>- Dewatering may be required, otherwise tremie concrete under groundwater is necessary.</li> <li>- Difficult to clean the bottom of the caisson, prior to pouring concrete.</li> <li>- Caisson resistance can not be confirmed during construction, although the exposed soil conditions during augering can confirm the soil conditions used in the design.</li> </ul>	caisson.	
Spread / strip footings	Support the bridge structure by spreading its load onto ground surface.	<ul style="list-style-type: none"> <li>- Use only an excavator to excavate the existing soil to the footing founding depth for footing construction.</li> <li>- The soil conditions at the founding depth can be confirmed.</li> </ul>	<ul style="list-style-type: none"> <li>- The founding depth of spread / strip footings should be close to the creek bed in order to enhance bearing capacity and slope stability. Thus, a relative-deep foundation (5 to 7 m depth below the existing bridge deck level) may be required.</li> <li>- Significant dewatering during construction may be required.</li> <li>- The base of the footing may have to be relatively large to provide sufficient bearing surface.</li> </ul>	<ul style="list-style-type: none"> <li>- Risks of relatively large excavation and significant dewatering, thereby increasing construction cost and time.</li> <li>- Risks of high groundwater level due to creek water level which could affect the performance of the footings (particularly settlement).</li> </ul>	- Low to medium.

## 6.1 Piled Foundation

Deep foundations, typically driven steel H-piles, should be used to support the new bridge structures. The piles should be driven into the competent native soil / till strata. In order to adequately penetrate the competent soil strata that contain cobbles / boulders, a heavy section such as **HP 310 x 110 or HP 356 x 132 equipped with reinforced driving shoes (OPSD 3000.100 Nov. 2005 Rev. 1) as per Ministry of Transportation of Ontario's Standard requirements would be suitable for use.**

### 6.1.1 Resistance to Axial Loads

Based on the results of the boreholes drilled in the vicinity of the new bridge abutments, Table 2 summarizes the approximate pile tip elevations that should be used for design purposes.

**TABLE 2  
APPROXIMATE PILE TIP LEVELS**

Support Location	Reference Borehole	Approximate Pile Tip Level (m)		Founding Stratum
		Elevation (m)	Depth Below Existing Grade (m)	
East Abutment	BH 11	172.6 to 170.6	13.0 to 15.0	Very dense silty sand /sandy silt till and/or sand/silty sand
	BH 12	171.6 to 169.6	12.0 to 14.0	Dense to very dense sand
West Abutment	BH 7	172.0 to 170.0	13.0 to 15.0	Very dense silty sand / sandy silt till
	BH 8	171.4 to 169.4	8.0 to 10.0	Very dense silty sand / sandy silt till

**Due to the presence of the existing battered piles which would remain on site, the new piles should not be positioned within a distance of 2 m immediately behind the existing piles (i.e., 2 m from the centre of the top of the existing battered piles in the back row to the centre of the new vertical piles). The as-installed existing pile positions should be**



**confirmed by MTO, possibly by obtaining the as-built drawings (if different from those shown in Appendix C.** The 2 m distance mentioned would provide a minimum distance of about 0.6 m between the tip of the existing battered piles and the end portion of the new piles.

Based on the drawings provided by MTO (Appendix C), the tip of the existing piles should be at about Elevation 170.9 m. If the existing piles were driven to the design load of 60 tons per pile, the new piles with a higher pile capacity (900 to 1,100 kN – SLS) would likely have to be installed slightly below the existing pile tip elevations. **It is therefore recommended that the minimum pile tip elevation of the new piles should be specified at Elevation 171 m.**

If installing the new piles between the existing piles can not be avoided, the new piles should be located between the existing battered piles at a minimum horizontal distance of 0.6 m (centre to centre) from the existing battered piles. The new vertical piles for the integral abutment design should be located at a minimum spacing of 1.2 m from each other. **The actually-driven pile tip elevations should be installed at or below Elevation 171 m and should likely be close to the lower values of the estimated pile tip elevations shown in Table 2.**

The approximate pile tip levels are based on the assumption that the piles would penetrate a minimum of 1.0 m into the competent soil stratum. The elevations shown in Table 2 are approximated from the results of a limited number of boreholes drilled and should not be considered as accurate pile tip elevations.

From the design of the new bridge abutment shown in the drawing provided by MTO (Appendix C), the top of the new piles would be slightly above Elevation 181.3 m. The new pile tip elevations should be at Elevation 171 m or lower. As such, the new vertical pile lengths would be at least 10 m. **The minimum length of the new piles in an integral bridge abutment should be specified by the design engineers after considering the structure-foundation interaction. If the estimated 10 m pile length is considered insufficient and longer piles are required, the new piles may have to be installed by preaugering the new pile locations and subsequently drive the piles.**

For HP 310 x 110 and HP 356 x 132 steel H-piles driven to practical refusal within competent native soil / till strata, the following axial resistances should be used for design:

- Factored Geotechnical Axial Resistance (Compression) at Ultimate Limit States = 1,200 kN for HP 310 x 110 and 1,500 kN for HP 356 x 132 in competent soil / till stratum, with an applied resistance factor of 0.5.
- Geotechnical Reaction at Serviceability Limit States = 900 kN for HP 310 x 110 and 1,100 kN for HP 356 x 132 in competent soil / till stratum.

If Hiley Formula is used during pile driving, the estimated ultimate resistance of the piles driven to practical refusal within the competent soil / till strata, at about the elevations shown in

Table 2, are approximately 2,700 kN for HP 310 x 110 and 3,300 kN for HP 356 x 132. The piles should be driven with a suitably heavy hammer capable of delivering a rated capacity of at least 50 kJ per blow. The energy should, however, be restricted to not more than 60 kJ per blow in order to prevent damage to the piles.

If necessary, the pile capacity may be verified by conducting field tests, i.e., Pile Driving Analyzer and/or static pile load testing in accordance with ASTM procedures.

**Cobbles and/or boulders will likely be encountered within the native soil / till strata. Hard pile driving conditions should therefore be anticipated. The piles should be equipped with reinforced driving shoes as per Ministry of Transportation of Ontario's Standard requirements in order to adequately penetrate the native soil / till strata.**

It is possible that, due to the variations in the soil / till strata, some piles may penetrate a few metres below the approximate tip elevations shown in Table 2. This aspect should be taken into consideration during design and construction.

The Geotechnical Reaction at Serviceability Limit States is dependent on the settlement of the pile group and, therefore, is governed by the size of the pile group. Provided that the piles are designed and installed as recommended in this report, the Serviceability Limit States value provided should correspond to not more than 25 mm of settlement of the pile group. If necessary, the pile group settlement should be calculated using the design pile group configuration.

Oversize materials (e.g., greater than 75 mm nominal diameter) should not be used in any new fill soils through which piles would be driven.

During the pile driving process, piles which have already been driven should be monitored to determine if they are heaving due to the effects of driving adjacent piles. If this phenomenon occurs, the heaved piles should be re-driven. It is recommended that not less than 10 % of the piles and at least two piles in each abutment be re-struck no sooner than 24 hours after initial installation, as a precaution against relaxation. If relaxation occurs, all piles in that abutment should be re-tapped.

### 6.1.2 Resistance to Lateral Loads

Laterally-applied loads on piles can be resisted, from the geotechnical consideration, by the driven piles through passive earth pressures developed in the soils in which the piles are embedded. The pile tip elevations recommended in Table 2 indicate that the piles will be approximately 10 m in length, depending on the design of the bridge abutment and the actual pile penetration into the competent soil / till strata. Lateral pile resistance may be considered in accordance with Section 6.4 of the Canadian Highway Bridge Design Code (CAN/CSA-S6-00).

The recommended horizontal resistances for a HP 310 x 110 pile and a HP 356 x 132 pile at this site are as follows:

Factored Horizontal Resistance at Ultimate Limit States	=	100 kN for HP 310 x 110
	=	120 kN for HP 356 x 132
Horizontal Reaction at Serviceability Limit States	=	40 kN for HP 310 x 110
	=	50 kN for HP 356 x 132

In accordance with the Ministry of Transportation of Ontario's requirements (Ministry of Transportation of Ontario Structural Office Standard), piles for integral abutments require a 3.0 m long flexible zone. The typical Ministry of Transportation of Ontario's standards for the flexible zone consist of a space in between the driven pile and a concentric corrugated steel pipe. The corrugated steel pipe surrounding the H-pile has a typical diameter of 600 mm. After the pile is driven, the space between the H-pile and the corrugated steel pipe is filled with coarse sand.

If conventional abutments on pile groups are to be built instead of integral abutments, the unbalanced horizontal forces may be partially resisted by battered piles.

For lateral soil-pile interaction analysis, the horizontal subgrade reaction to the pile in cohesionless soils, i.e., the silty sand / sandy silt soils encountered at the site, may be calculated from the following expression:

$$k_s = n_h \times z / d$$

where  $k_s$  = coefficient of horizontal subgrade reaction,  
 $n_h$  = coefficient related to soil density,  
 $d$  = pile width,  
 $z$  = depth.

Based on the results of the boreholes drilled in the vicinity of the new bridge abutments, Table 3 summarizes the estimated values of internal friction angle, bulk unit weight and coefficient related to soil density.

**TABLE 3**  
**SOIL PARAMETERS RECOMMENDED FOR DESIGN**

REFERENCE BOREHOLE	APPLICABLE DEPTH FROM EXISTING GROUND SURFACE (m)	APPROX. ELEVATION (m)	SIMPLIFIED SOIL TYPE	ESTIMATED BULK UNIT WEIGHT (kN/m <sup>3</sup> )	ESTIMATED ANGLE OF INTERNAL FRICTION $\phi$ (DEGREES)	ESTIMATED UNDRAINED SHEAR STRENGTH (kPa)	ESTIMATED $n_h$ VALUE (MN/m <sup>2</sup> )
West Abutment							
BH 7	0.2 to 8.2	184.8 to 176.7	Silty sand fill	20	32	-	5
	8.2 to 10.0	176.7 to 175.0	Compact silty sand / sandy silt till	20	34	-	5
	8.2 to 15.4	175.0 to 169.6	Very dense silty sand / sandy silt till	22	37	-	10
East Abutment							
BH 11	0.2 to 4.9	185.4 to 180.7	Silty sand / sand and gravel / sandy silt fill	20	32	-	5
	4.9 to 8.2	180.7 to 177.3	Very stiff to stiff silty clay / clayey silt till	22	-	100	-
	8.2 to 12.0	177.3 to 173.6	Compact to loose silty sand / sandy silt till	20	33	-	5
	12.0 to 14.6	173.6 to 170.9	Very dense silty sand / sandy silt till	22	37	-	10
	14.6 to 15.7	170.9 to 169.9	Very dense sand	22	37	-	10

## **6.2 Retaining / Wing Walls**

### **6.2.1 Retaining Wall Type**

To support the approach embankment, stable slopes and/or retaining wall structures can be used. Several soil retaining wall structures typically used are integral abutment with wing walls and conventional abutment with retaining wall structure (e.g., reinforced-concrete retaining wall, gabion retaining wall, retained soil system (RSS), etc.). The retaining wall structures are required to support the approach embankment typically in two directions – one where the retaining wall structure is underneath the bridge deck and the other where the retaining wall structure length is parallel to the road alignment. Where space is available, slopes would be preferred to retaining wall structures due generally to the lower construction costs of slopes.

In the case of the West Duffins Creek Bridge Replacement where the integral bridge abutment is the preferred foundation type and space is available for constructing slopes, it is preferable to construct a slope along the creek bank underneath the new bridge deck to support the approach embankment similar to the existing bank slope underneath the existing bridge. The existing piled foundation and part of the existing bridge abutment would remain in place, thereby enhancing the slope stability. The approach embankment would therefore be partially supported by the integral abutment and bank slope, resulting in reducing construction cost of a retaining wall structure although the bridge span has to be slightly longer.

As for the support of the approach embankment in the direction parallel to the road alignment, wing walls supported by the integral abutment or retaining wall structures may be used. Due to the available space, wing walls supported by the integral abutment should be used, similar to the existing wing walls of the existing bridge. The reinforced-concrete wing walls to be extended from the integral abutment would support the approach embankment immediately adjacent to the bridge deck, together with embankment slopes.

The wing walls or retaining structures should be designed using the soil parameters and relevant considerations described in the following section.

### **6.2.2 Lateral Earth Pressures**

Backfill materials behind abutments and wing (retaining) walls should consist of non-frost susceptible, free-draining granular materials in accordance with the Ministry of Transportation of Ontario's standards.

Free-draining backfill materials (i.e., Granular 'A' or Granular 'B') and the provision of drain pipes and weep holes, etc., should be provided to prevent hydrostatic pressure build-up. Computation of earth pressures should be in accordance with Section 6.9.2 – Lateral Pressures of Canadian Highway Bridge Design Code (CAN/CSA-S6-00). For design purposes, the following parameters (unfactored) should be used.

.../...

#### Compacted Granular 'A'

- Unit weight =  $22 \text{ kN/m}^3$
- Coefficient of Lateral Earth Pressures:
  - $K_a = 0.27$  (active condition)
  - $K_o = 0.43$  (at-rest condition)
  - $K_p = 1.20$  (passive condition)

#### Compacted Granular 'B'

- Unit weight =  $21 \text{ kN/m}^3$
- Coefficient of Lateral Earth Pressures:
  - $K_a = 0.31$  (active condition)
  - $K_o = 0.47$  (at-rest condition)
  - $K_p = 1.10$  (passive condition)

It should be noted that the above design parameters assume relatively-level ground surface and backfill behind the retaining structure. The coefficient of Lateral Earth Pressure for the passive condition has been reduced in order to limit the wall movement that is necessary to mobilize the passive resistance.

The earth pressure coefficient to be used in the design will depend on whether the retaining structure is restrained or movements are allowed such that the active and passive states of earth pressure can develop. If the abutment is restrained and does not allow lateral yielding, the at-rest pressures should be used for design. Similarly, the effects of compaction should also be taken into account in the selection of the appropriate earth pressure coefficients.

Vibratory equipment for use behind abutments and retaining walls should be restricted in size as per current Ministry of Transportation of Ontario's practice.

### 6.3 Approach Embankment

The slopes of the existing bridge approach embankments are approximately 2H:1V with a maximum height of about 5 m. The existing bridge approach embankments may be slightly widened for the new bridge by about 0.7 m or more if the new bridge is further widened. The existing bridge approach embankments would likely be excavated, in whole or in part, during the excavation for the installation of piles and the construction of the new bridge abutments.

New approach embankments should be constructed with compacted engineered fill with 2H:1V side slopes or rock fill with 1.25H:1V side slopes. For widening the existing embankments, the new fill should be constructed with benching according to OPSD – 208.010 (Benching of Earth Slopes).

According to the drawings provided by MTO (Appendix C), the existing road width would be widened by about 0.7 m on each side. By properly benching the additional slope to the existing slope (if not removed and reconstructed) and properly compacting the new fill, settlement of the new extended slope should not be significant and should be limited to the new shoulder. The existing approach embankment would form the main part of the road surface. As such, the settlement of the new road surface should be negligible. If the existing bridge approach embankment is removed and a new bridge approach embankment is constructed to the same elevation as the existing one, settlement of the new approach embankment should be minimal.

If the existing bank slope below the bridge is totally reconstructed for the new bridge, its slope stability should be investigated and analysed as part of the detail design. Based on the drawings provided by MTO for the new bridge, a preliminary slope stability analysis was carried out and the results yield a minimum calculated factor of safety against slope instability of 1.17 for the existing bridge as shown in Figure Nos. E1 and E2.1 (Appendix E) and 1.21 for the new bridge as shown in Figure No. E2.2 (Appendix E). The calculated factors of safety should be higher when considering the three-dimensional effect and the existing piles that would remain on site. The bank slope underneath the new bridge should therefore be stable. Furthermore, it should be noted that the new bridge abutment to be located behind the existing abutment would reduce the applied load on the bank slope, thereby enhancing the slope stability of the new bank slope.

If the new bridge embankments are significantly different in dimensions from the existing embankments, the stability and settlement of the new embankments should be analysed during the detail design. If 2H:1V slopes are used to construct the widened approach embankment without increasing the existing embankment height, the calculated minimum factor of safety against slope instability would be about 1.9, as shown in Figure No. E3 in Appendix E, which should be stable. The slope stability analysis was based on the highest approach slope and soil parameters as shown in Figure No. E3 in Appendix E.

All organic matters and other unsuitable soils should be removed, as per Ministry of Transportation of Ontario's current practice, with an envelope given by a gradient not steeper than 1H:1V away from the toe of the embankment. For above water construction, the exposed subgrade should be inspected, approved and properly compacted from the surface, to a minimum of 95 % Standard Proctor Maximum Dry Density (SPMDD), under the supervision of qualified geotechnical personnel. Any soft spots identified during stripping and/or recompacting should be sub-excavated and replaced with compacted engineered fill. Care should be exercised to minimize disturbance to the subgrade during preparation and the construction of embankment. For under water construction, all organic matters and /or soft / loose soils should be removed and backfilled with rock / granular soils until the backfill level is above the water level. Otherwise, dewatering is required for placing an engineered fill.

For an engineered fill, the fill materials used for construction of the conventional earth fill embankment, or for the purposes of backfilling, should consist of approved, clean earth fill (e.g. .../...



Select Subgrade Materials – Ontario Provincial Standards Specifications Number: 1010). The fill may be imported for this purpose or the excavated soils may be reused provided that they do not contain organic matters and can be compacted to the specifications. The fill materials should be placed in accordance with Ontario Provincial Standards Specifications Number: 501. Each lift should not exceed 300 mm before compaction and each lift should be uniformly compacted to at least 95 % of the Standard Proctor Maximum Dry Density (SPMDD) of the materials. The degree of compaction within the top 0.6 m of the fill (i.e., the subgrade immediately beneath the granular sub-base) should be increased to 98 % SPMDD. The selection, placement and compaction of the fill should be carried out under a geotechnical control program.

For the fill embankment at this site, using properly compacted and acceptable inorganic fill material, the side slopes should not be steeper than 2H:1V for earth fill embankment and 1.25H:1V for rock fill embankment. Proper erosion control measures should be implemented both during construction and on a permanent basis. This can be achieved by immediate seeding or sodding (Ontario Provincial Standards Specification Number: 572) or equivalent.

#### **6.4 Construction Staging and Detour**

To construct the new bridge, the existing bridge structure and part of the existing bridge approach embankment will have to be demolished. Due to right-of-way limitation within the site, a detour, if necessary, may be constructed through the creek. This will however necessitate temporary crossing of the creek and working in-stream, which should be avoided due to the stringent requirements for environmental protection.

Unless a detour through the creek is provided, half of the existing bridge structure will have to be demolished while the remaining half will still be open for traffic. This will reduce the traffic lane to be a single lane and all necessary traffic protection will have to be provided. A roadway protection system (i.e., temporary shoring system) will be required along the bridge approach alignment and adjacent to the existing bridge to support the partially-excavated existing bridge approach embankment.

Based on the boreholes drilled at the existing bridge approach embankments (BH 7, BH 8, BH 11 and BH 12), temporary cantilever sheetpiles or soldier piles with wood laggings may be used to support the partially-removed approach embankment. The temporary shoring system should be designed using the following soil parameters and surcharge:

Soil bulk unit weight	=	21 kN/m <sup>3</sup>
Coefficient of lateral soil earth pressure	=	0.4
Traffic load as surcharge	=	12 kPa



Groundwater levels shown in the Record of Boreholes (Appendix A) should be considered in the design of the temporary shoring system.

## **6.5 Construction Comments**

All excavations should be carried out in accordance with the latest edition of the Ontario Occupational Health and Safety Act and its regulations (i.e. Occupational Health and Safety Act O.Reg. 213/91).

The boreholes show that the excavation for the structure foundations (including abutment and wing walls) should encounter mainly the fill soils (silty sand, sand and gravel, and clayey silt) and possibly some till deposits. These soils should be classified as Type 3 and according to the Occupational Health and Safety Act, temporary open cut slopes should not be steeper than 1H:1V.

Based on MTO's drawings (Appendix C), the bottom of the excavation required for the construction of the abutment and wing walls should be at about Elevation 181 m. The groundwater levels encountered in the boreholes were below Elevation 181 m, except in Borehole BH 12 located outside the approach embankment. As such, groundwater seepage, if any, should not be significant. Other water sources could be perched groundwater and/or surface runoff. Dewatering could be carried out by gravity drainage and/or pumping from a properly filtered sump. The base of the excavation should be graded towards a sump pump in order to drain any surface water inflow into the excavation and from severe weather events in order to avoid excessive softening of the base.

**No major excavation difficulties are foreseen but allowance should be made for boulders and cobbles which could occur randomly in fill soils and glacial till deposits.**

**The demolition of the existing bridge abutment should be limited such that the remaining existing bridge abutment and its piled foundation would not interfere with the performance of the new bridge and enhance the slope stability of the approach slope underneath the bridge.**

Allowance should be made to place an approximately 150 mm thick layer of lean concrete on the subgrade surface, i.e. excavation base for the new bridge abutment, within four hours of preparation and acceptance of the bearing soil. It should be pointed out that if the foundation soil is disturbed, excessive settlements could occur after structural loads are applied.

Compacting the granular backfill soils against the new bridge abutment should be carried out with a small compacting equipment in order to prevent potential damage to the new bridge abutment. Both sides of the bridge abutment should be backfilled to approximately the same level at the same time in order to minimize the lateral load due to compaction.

## 6.6 Construction Inspection

It is recommended that a quality control programme of inspection and testing be carried out during the construction phase of the project to confirm that the conditions encountered are consistent with design assumptions; and to confirm that the various project specifications and material requirements and handling are followed.

## 7.0 LIMITED ENVIRONMENTAL INVESTIGATION

In accordance with the Terms of Reference, soil samples obtained during the geotechnical field drilling program were field screened for evidence of environmental impact. The field screening activities included measuring the total organic vapours (TOV) in the headspace of samples with a portable hydrocarbon surveyor instrument (Thermo Gastechtor 1238ME). The borehole locations were positioned at the site as required by the geotechnical investigation.

Based on the soil conditions encountered in all the boreholes, the soil profile at the site comprised predominantly existing asphaltic concrete and/or topsoil overlying gravelly/sandy/silty fill soils which were underlain by native soils and / or till deposits. No visual or olfactory evidence of environmental impact was observed in the fill and native soil / till samples recovered from the boreholes. The measured TOV concentrations in all soil samples were non-detect as shown in the Record of Boreholes in Appendix A. The TOV results are semi-quantitative at best and are generally only used for relative sample comparison purposes when selecting samples for laboratory analysis.

## 8.0 SOIL CORROSIVITY

Two soil samples (BH 9 - SS 9 and BH 12 – SS 6) were analysed by AMEC's chemical laboratory in Mississauga to determine the soil corrosivity with respect to concrete and steel. The results (Appendix B) are presented as follows.

Soil Sample No.	pH	Resistivity (ohms-cm)	Chloride (µg/g)	Sulphate (µg/g)
BH 9 – SS 9	7.2	5380	7	126
BH 12 – SS 6	7.1	4030	36	145

The tests have shown sulphate values of 126 and 145 ppm (µg/g) and according to Table 10 - "requirements for Concrete Subject to Sulphate Attack", Clause 15.5.2, of CSA Standard Specification A23. 1-94, any soil which has a sulphate content below 1000 ppm is not considered corrosive with respect to concrete. As such, the general-use Cement Type 10 can be used. The soil resistivity measured is "mild" at BH 9 and "severe" at BH 12, while the pH .../...

values are close to neutral. The soil corrosivity on concrete and steel should be studied, if necessary, by a corrosivity expert.

## 9.0 CLOSURE

The sub-soil information and recommendations contained in this report should be used solely for the purpose of foundation assessment of this site.

Non Standard Special Provision (NSSP) is included in Appendix G which provides samples of wordings that MTO may consider using them in the construction contract.

AMEC should be retained to review the recommendations provided in this report, once the details of the project are finalized and prior to the final design stage of the project.

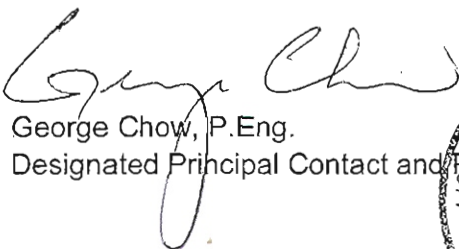
The attached Report Limitations is an integral part of this report.

Sincerely,

AMEC Earth & Environmental,  
A division of AMEC Americas Limited



Prapote Boonsinsuk, Ph.D., P.Eng.  
Project Manager



George Chow, P.Eng.  
Designated Principal Contact and Project Reviewer

**AMEC Earth & Environmental, a division of AMEC Americas Limited**

**REPORT LIMITATIONS**

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 environmental 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. It is recommended practice that the Geotechnical Engineer be retained during the construction to confirm that the subsurface conditions across the site do not deviate materially from those encountered in the testholes.

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. Since all details of the design may not be known, we recommend that we be retained during the final design stage to verify that the design is consistent with our recommendations, and that assumptions made in our analysis are valid.

The comments made in this report relating to potential construction problems and possible methods of construction 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. No other warranty is expressed or implied.

The benchmark and elevations mentioned in this report were obtained strictly for use by this office in the geotechnical design of the project. They should not be used by any other party for any other purpose.

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. AMEC Earth & Environmental accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.