THURBER ENGINEERING LTD.

DRAFT
FOUNDATION INVESTIGATION AND DESIGN REPORT
HIGHWAY 401 CROSSING OF ETOBICOKE CREEK
WIDENING OF THE EASTBOUND COLLECTOR
MISSISSAUGA, ONTARIO
G.W.P. 2147-10-00, CONTRACT 2

GEOCRES NO.

Report

to

AECOM

Date: November 11, 2016
File: 12669

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings from a foundation investigation carried out by Thurber Engineering Ltd. (Thurber) for the design and construction of the proposed widening of the eastbound collector of Highway 401 crossing at Etobicoke Creek in Mississauga, Ontario. Thurber was retained by AECOM to carry out the foundation investigation at this site on behalf of the Ministry of Transportation Ontario (MTO) under Assignment No. 2012-E-0036.

The purpose of the investigation was to explore the subsurface conditions at the site and, based on the data obtained, provide borehole location and soil strata drawings with stratigraphic profile and cross-section(s), records of boreholes, laboratory test results and a written description of the subsurface conditions. A model of the subsurface conditions was developed from the data obtained during the course of the present investigation.

During the preparation of this report and in addition to the boreholes drilled, reference has been made to information on subsurface conditions contained in previous foundation reports for the existing bridges. A technical memorandum on desktop evaluation dated September 14, 2016 has been prepared by Thurber as part of the current investigation.

2 SITE AND PROJECT DESCRIPTION

The site is located on Highway 401 at Etobicoke Creek, Mississauga, Ontario between two existing bridges (Highway 401 eastbound express bridge and Highway 401 eastbound collector bridge). Based on the available information provided by AECOM and historical drawings, each of the two existing bridges is comprised of a three-span pre-stressed concrete girder structure supported on two abutments and two piers. Each abutment is supported on H-piles driven to bedrock and/or refusal. Each pier is supported on concrete spread footing founded on bedrock. The height between the highway grade and the creek water level is in the order of 12 m. The forward slopes and adjacent side slopes were designed to have inclinations of 2H : 1V with mid-height benches, and 3H : 1V, respectively. The lower portion of the west and east forward slopes were covered by gabion baskets for slope and scour protection.

The area surrounding the creek floodplain is generally moderately vegetated. The northeast quadrant of the crossings is occupied by Pearson airport lands; whereas, the remaining three quadrants are largely of commercial and light industrial usage. Beyond the bridge footprints, the natural terrain adjacent to the creek banks is generally sloping towards the creek channel. The vegetation consists of grass with some trees and shrubs. In addition, a recreational trail runs between the existing east abutments and east piers.

At the location of the existing bridges, the Etobicoke Creek flows in a southeast direction towards Lake Ontario.

Photographs of the site showing the general lay of the surrounding land are included in Appendix D.

Based on a preliminary General Arrangement (GA) drawing provided by AECOM dated May 2016, this project involves the widening of the EB collector bridge using pre-stressed concrete girders, which are to be located in the air gap between the EB collector and express bridges. Two new piers and two new abutments will be constructed adjacent to and aligned with the existing EB collector piers and abutments to support the new widening of the bridge deck.

In addition, an existing sanitary sewer runs in close proximity to the proposed west abutment foundations. For H-piles that may be used to support the new west abutment, information from AECOM indicates that the clearance at the closest point could be only about 1.8 m between the sewer line and the new piles.

From published geological information in *The Physiography of Southern Ontario* by Chapman and Putnam (1984), the site lies within the physiographic region known as the Peel Plain. The Peel Plain contains deep river and stream valleys, and is characterized by cohesive glacial till with shale and limestone fragments. The soil deposit is underlain at relatively shallow depths by grey shale bedrock of the Georgian Bay Formation.

3 INVESTIGATION PROCEDURES

The site investigation for this project was carried out between August 22 and September 13, 2016 during which time a total of six (6) boreholes denoted as Boreholes EC 16-01 to EC 16-04, CA 16-01, and CA 16-02 were advanced to depths ranging from 8.0 m to 21.1 m (see Table 3.1). It is noted that Boreholes CA-01 and CA-02 were drilled for the design of the proposed construction access routes.

Boreholes EC16-01 and CA 16-01 were advanced near the proposed east abutment; whereas, boreholes EC 16-02 and CA 16-02 were advanced near the proposed west abutment. Boreholes EC 16-03 and EC 16-04 were drilled near the proposed west and east piers, respectively. The approximate locations of the boreholes are shown on the Borehole Locations and Soil Strata Drawing provided in Appendix C. Borehole details are provided in Table 3.1 below.

Table 3.1 – Borehole Details

Foundation Unit	Borehole Number	Approximate Ground Elevation (m)*	Borehole Termination Depth (m)	Borehole Termination Elevation (m)
West Abutment	CA 16-02	155	8.2	146.8
	EC 16-02	155	21.1	133.9
West Pier	EC 16-03	144	8.8	135.2
East Pier	EC 16-04	144	8.1	135.9
East Abutment	EC 16-01	155	20.9	134.1
	CA 16-01	155	8.0	147.0

* Approximate elevation obtained from AECOM Drawing No. 2205-13-00 prepared in July 2016

A track-mounted Diedrich D-50 drill rig was used throughout. The boreholes were initially advanced using hollow stem augers and then switched to NQ or HQ coring with water to obtain rock core samples, where required. In all boreholes, soil samples were obtained at selected intervals with a 50 mm outside diameter split spoon sampler driven in conjunction with the Standard Penetration Test (SPT).

Groundwater conditions were observed in the open boreholes throughout the drilling operations. Standpipe piezometers were installed in Boreholes EC 16-01 and EC 16-02 to permit monitoring of the groundwater levels at the site. Each piezometer consisted of a 19 mm diameter PVC pipe with a slotted screen sealed at a selected depth within the borehole. The boreholes in which no piezometer was installed were backfilled in general accordance with Ontario Regulation 903. Piezometer installation and borehole completion details are summarized in Table 3.2 below.

Table 3.2 – Piezometer and Borehole Completion Details

Foundation Unit	Borehole Number	Piezometer Tip Depth / Elevation (m)	Completion Details
West Abutment	CA 16-02	None installed	Backfilled with bentonite holeplug and auger cuttings to surface.
	EC 16-02	21.0/134.0	Backfilled with filter sand from 21.1 m to 16.5 m, then bentonite holeplug from 16.5 m to 7.3 m, then bentonite and auger cuttings to surface.
West Pier	EC 16-03	None installed	Backfilled with bentonite holeplug and auger cuttings, and reinstated gabion baskets to surface.
East Pier	EC 16-04	None installed	Backfilled with bentonite holeplug and auger cuttings to surface.
East Abutment	EC 16-01	20.9/134.1	Backfilled with filter sand from 20.9 m to 17.4 m, then bentonite holeplug from 17.4 m to 16.2 m, then bentonite holeplug and auger cuttings to surface.
	CA 16-01	None installed	Backfilled with bentonite holeplug and auger cuttings to surface.

The field work was supervised on a full time basis by a member of Thurber's technical staff who marked/staked the boreholes in the field, arranged for the clearance of buried utilities, directed the drilling, sampling and in-situ testing operations, logged the boreholes and processed the recovered soil and rock samples for transport to Thurber's laboratory for further examination and testing.

4 LABORATORY TESTING

All recovered soil samples were subjected to visual identification and to natural moisture content determination. Selected samples were also subjected to grain size distribution analysis (hydrometer and/or sieve analysis) and Atterberg Limits testing, where appropriate. Laboratory testing results are summarized on the Record of Borehole sheets included in Appendix A and are presented on the figures included in Appendix B.

All recovered cores of the shale bedrock were visually examined in the laboratory to confirm and supplement the field description. Selected core samples were also subjected to laboratory point load tests in the axial and diametral directions, where possible. The results are presented on the Record of Borehole sheets in Appendices A and B.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Details of the encountered soil stratigraphy are presented on the Record of Borehole sheets included in Appendix A and on the Borehole Location and Soil Strata drawings in Appendix C. A general description of the stratigraphy, based on the conditions encountered in the boreholes, is given in the following paragraphs. However, the factual data presented on the Record of Borehole sheets takes precedence over this general description and must be used for interpretation of the site conditions. It should be recognized and expected that soil conditions may vary between and beyond borehole locations.

In general, the subsurface stratigraphy encountered in the boreholes consists of surficial topsoil and/or fill deposit overlying silty sand to sandy silt in the vicinity of the proposed east abutment, and silty clay in the vicinity of the proposed west abutment. These upper native soil deposits are underlain by a deposit of silty clay to clayey silt till which is in turn underlain by weathered shale which grades into a moderately to slightly weathered shale bedrock. The groundwater level at the abutments is in the order of 6 m below highway grade, and at the piers coincides approximately with the creek level.

5.1 Topsoil

A layer of surficial topsoil with a thickness ranging from 13 mm to 100 mm was encountered in Boreholes EC 16-02, EC 16-04, CA 16-01 and CA 16-02.

5.2 Gabion Baskets

Gabion baskets cover the lower portion of the west and east forward slopes. Prior to drilling at Borehole EC 16-03, the rock fill inside the gabions in the immediate vicinity of the borehole location was temporarily removed to expose the underlying soil. The gabions at this location are approximately 0.9 m in thickness.

5.3 Silty Clay Fill

Embankment fill consisting of silty clay with sand and trace gravel was encountered in boreholes located near the abutments. The thickness of the silty clay fill varied between 2.3 m and 4.9 m (base elevations from approximately 152.7 m to 150.1 m). A very dense 0.7 m to 0.8 m thick surficial layer of silt and sand fill was encountered at Boreholes EC 16-02 and CA 16-02.

SPT ‘N’ values recorded in the silty clay fill at the location of the east abutment in Boreholes EC 16-01 and CA 16-01 ranged from 4 to 9 blows per 0.3 m of penetration, indicating a firm to stiff consistency. SPT ‘N’ values recorded in the silty clay fill at the west abutment in Boreholes EC 16-02 and CA 16-02 typically ranged from 8 to 22 blows per 0.3 m of penetration, indicating a stiff to very stiff consistency. An occasional ‘N’ value of greater than 100 blows for less than 0.3 m penetration indicated the possible presence of cobbles in Borehole CA 16-02. Measured moisture contents within the cohesive fill varied between 5% and 20%.

The results of grain size distribution analyses and Atterberg Limits testing carried out on selected samples of the silty clay fill are presented on the Record of Borehole Sheets included in Appendix A and on Figures B1 and B6 of Appendix B. The results of the grain size distribution analyses are summarized below:

Soil Particle	Percentage (%)
Gravel	2 to 7
Sand	24 to 34
Silt	33 to 41
Clay	26 to 33

The results of Atterberg Limits testing are summarized below:

Index Property	Percentage (%)
Plasticity Index	14 to 16
Liquid Limit	32 to 34

The results of the Atterberg Limits testing indicate the layer to be of low plasticity with group symbol CL.

5.4 Silty Sand to Sandy Silt

A layer of silty sand to sandy silt with some clay and some to trace gravel was encountered below the fill in Boreholes EC 16-01 and CA 16-01. This layer has a thickness of 8.0 m (base elevation at 144.7 m) in Borehole EC 16-01. Borehole CA 16-01 was terminated within the sands and silts at 8.0 m depth (Elevation 147.0 m).

SPT 'N' values recorded in this layer ranged from 48 blows per 0.3 m penetration to greater than 100 blows for less than 0.3 m of penetration, indicating a dense to very dense condition. Measured moisture contents within this layer varied between 6% and 21%.

The results of grain size distribution analyses carried out on selected samples of this layer are presented on the Record of Borehole Sheets included in Appendix A and on Figure B2 of Appendix B. The results of the grain size distribution analyses are summarized below:

Soil Particle	Percentage (%)
Gravel	6 to 16
Sand	35 to 45
Silt	33 to 44
Clay	14 to 15

5.5 Silty Clay

A deposit of silty clay with some sand and trace gravel was encountered underlying the fill at Boreholes EC 16-02 and CA 16-02 and underneath the topsoil at Borehole EC 16-04. The thickness of the silty clay varied from 1.4 to 5.3 m (base elevation from 144.8 to 142.6 m) in Boreholes EC 16-02 and EC 16-04. A loose 0.8 m thick interlayer of silty sand with trace to some clay and trace organics was encountered immediately below the silty clay in Boreholes EC 16-04. Borehole CA 16-02 was terminated at 8.2 m depth (Elevation 146.8 m) within the silty clay.

SPT 'N' values recorded in the silty clay layer at the location of the west abutment boreholes (EC 16-02 and CA 16-02) typically ranged from 12 to 24 blows per 0.3 m of penetration indicating a stiff to very stiff consistency. An occasional 'N' value of 45 blows indicates the presence of a hard zone in Borehole EC 16-02. SPT 'N' values recorded in the silty clay layer at the east pier borehole (EC 16-04) ranged from 5 to 10 blows per 0.3 m of penetration indicating a firm to stiff consistency. Measured moisture contents within the cohesive layer varied between 15% and 20%.

The results of grain size distribution analyses and Atterberg Limits testing carried out on selected samples of the silty clay layer are presented on the Record of Borehole Sheets included in Appendix A and on Figures B3 and B7 of Appendix B. The results of the grain size distribution analyses are summarized below:

Soil Particle	Percentage (%)
Gravel	3 to 6
Sand	18 to 26
Silt	40 to 44
Clay	28 to 36

The results of Atterberg Limits testing are summarized below:

Index Property	Percentage (%)
Plasticity Index	16 to 17
Liquid Limit	36 to 37

The results of the Atterberg Limits testing indicate that this deposit is of medium plasticity with a group symbol of CI.

5.6 Gravelly Sand

A layer of gravelly sand with some silt and clay and trace of weathered shale was encountered below the gabion baskets at Borehole EC 16-03. The layer has a thickness of 1.3 m with a base elevation at 141.8 m.

SPT 'N' values recorded in this layer ranged from 17 to 20 blows per 0.3 m of penetration indicating a compact condition. Measured moisture contents within this layer varied between 5% and 10%.

The results of grain size distribution analyses carried out on selected samples of this layer are presented on the Record of Borehole Sheets included in Appendix A and on Figure B4 of Appendix B. The results of the grain size distribution analyses are summarized below:

Soil Particle	Percentage (%)
Gravel	35
Sand	51
Silt and Clay	14

5.7 Clayey Silt to Silty Clay Till

A till deposit consisting of clayey silt to silty clay with trace to some sand and trace gravel was found underlying the above soils in Boreholes EC 16-01, EC 16-02 and EC 16-04. The thickness of the cohesive till deposit ranged from 1.3 to 4.9 m (base elevations from 139.8 m to 140.5 m). In Borehole CA 16-01, an upper 1.3 m thick layer of clayey silt till was encountered immediately below the fill. The base of this layer is at Elevation 151.3 m.

SPT 'N' values recorded in this till deposit ranged from 11 blows per 0.3 m penetration to greater than 80 blows for less than 0.3 m of penetration, indicating a stiff to hard consistency. Measured moisture contents within the till layer varied between 9% and 20%.

The results of grain size distribution analyses and Atterberg Limits testing carried out on selected samples of the clayey silt till are presented on the Record of Borehole Sheets included in Appendix A, and on Figures B5 and B8 of Appendix B. The results of the grain size distribution analyses are summarized below:

Soil Particle	Percentage (%)
Gravel	1 to 6
Sand	7 to 27
Silt	46 to 72
Clay	20 to 22

5.8 Shale Bedrock

The soils described above are underlain by grey shale bedrock of the Georgian Bay Formation. The bedrock is typically weathered within the upper 1.4 to 3.5 m (bottom elevation of 138.2m to 138.8 m), becoming moderately to slightly weathered below these depths. Zones of broken cores and horizontal fractures were noted throughout the bedrock cores. The shale encountered in the boreholes is described as fine grained, thinly bedded and contains very strong limestone interbeds which are generally 25 to 100 mm thick. Vertical and horizontal fractures were observed in the recovered cores.

Within the upper portion of the shale (the weathered zone) where rock coring was not carried out, SPT N-values obtained in the shale bedrock were greater than 100 blows for less than 0.3 m penetration. Moisture contents in the SPT weathered shale samples ranged from 5% to 17%.

Bedrock was proven by coring in Boreholes EC 16-01 to EC 16-04. Table 5.1 summarizes depths and elevations to the top of bedrock.

Table 5.1 – Depths and Elevations of Top of Shale

Foundation Unit	Borehole Number	Top of Weathered Shale		Top of Moderately Weathered Shale	
		Depth (m)	Elevation (m)	Depth (m)	Elevation (m)
West Abutment	EC 16-02	14.9	140.1	16.8	138.2
West Pier	EC 16-03	2.2	141.8	5.7	138.3
East Pier	EC 16-04	3.5	140.5	5.2	138.8
East Abutment	EC 16-01	15.2	139.8	16.6	138.4

Total core recovery in the bedrock ranged from 62% to 100%. The RQD values at the location of east and west piers (boreholes EC 16-03 and EC 16-04) ranged from 37% to 80% indicating poor to good rock quality. The RQD values at the location of east and west abutments were 0% indicating a very poor quality, except a RQD value of 80% was obtained for Run#3 in Borehole EC 16-01.

The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, ranged between 3 and 8 in Boreholes EC 16-03 and EC 16-04, whereas most values in Boreholes EC 16-01 and 16-02 were greater than 20.

From correlation with point load test results, the estimated unconfined compressive strength of intact shale rock cores generally ranged from about 2 to 45 MPa indicating a very weak to medium strong rock. Correlation results for the limestone interbeds ranged from 64 to 245 MPa indicating a typically strong to very strong rock. These estimated rock strength values are interpreted from point load tests that were conducted on rock cores recovered from the boreholes. The results of these tests are summarized on the Record of Borehole sheets included in Appendix A and on the Point Load Test Sheets in Appendix B.

5.9 Groundwater Conditions

Groundwater conditions were observed during drilling operations and groundwater levels were measured in the open boreholes upon completion of drilling. Standpipe piezometers were installed in boreholes EC 16-01 and EC 16-02 to monitor the groundwater level at the site. The groundwater levels measured in the open boreholes and in the piezometers are summarized below.

Table 5.2 – Groundwater Levels and Observations

Foundation Element	Borehole	Date	Water Level (m)		Comment
			Depth	Elevation	
West Abutment	EC 16-02	Oct. 5, 2016	4.3* 6.0	150.7* 149.0	Open hole* Piezometer
West Pier	EC 16-03	Sept. 13, 2016	2.1	141.9	Open hole*
East Pier	EC 16-04	Sept. 14, 2016	2.4	141.6	Open hole*
East Abutment	EC 16-01	Oct. 5, 2016	5.8	149.2	Piezometer

* Water has been added into the hole as part of the drilling operations.

The groundwater levels above are short-term readings and seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater levels may be at a higher elevation after periods of significant or prolonged precipitation. It is anticipated that the groundwater level at the pier locations is largely influenced by the water level in the creek.

6 MISCELLANEOUS

Thurber marked and/or staked the borehole locations in the field and obtained buried utility clearances prior to drilling.

Geotechnical laboratory testing was carried out at Thurber's MTO approved high complexity Toronto area laboratory.

Altech Drilling Ltd. supplied and operated the drilling, sampling and in-situ testing equipment for the field investigation.

The field investigation was supervised on a full time basis by a member of Thurber's technical staff. Compilation of data and preparation of the report was carried out by Messrs. Mohamad Hosney, P.Eng. and Sydney Pang, P.Eng. The report was reviewed by Dr. P.K. Chatterji, P.Eng.

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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 GENERAL

This report provides interpretation of the geotechnical data in the factual report and presents foundation design recommendations to assist the design team to select and design a suitable foundation system for the proposed widening of the Highway 401 Eastbound Collector bridge over Etobicoke Creek in the City of Mississauga, Ontario.

This foundation investigation and design report with the interpretation and recommendations are intended for the use of the Ministry of Transportation, and shall not be used or relied upon for any other purposes or by any other parties including the construction or design-build contractor. The contractor must make their own interpretation based on the factual data in Part 1 of the report. Where comments are made on construction, they are provided only in order to highlight those aspects which could affect the design of the project. Contractors must make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods and scheduling.

The bridge widening is to be located within the air gap between the existing EB Collectors and EB Express bridges. A General arrangement (GA) drawing provided by AECOM (Drawing No. 2205-13-00 dated July 2016) indicates that the widening bridge is a 79.3 m long (between abutment bearings) and 6.9m wide three span structure supported by two piers and two abutments. A 7.6 m wide approach slab with sleeper slab is to be located behind each abutment.

A 2 m diameter sanitary trunk sewer line is located in close proximity to the proposed west abutment. Information from AECOM indicates that the clearance between the pipe and the proposed battered piles for supporting the west abutment could be in the order of 4.5 m. It is recommended that the exact location of this sewer be confirmed during the design stage and pertinent measures be taken to protect the sewer from damage during construction of the new bridge foundations.

It is understood that consideration is being given to supporting the east and west abutments on H-piles; whereas, the east and west piers are to be supported on spread footings. The forward slopes are to have an inclination of 2H : 1V with mid-height benches. Rock protection is to be provided for the lower portion of the slopes. The height between the highway grade and the river is up to the order of 12 m.

The discussion and recommendations presented in this report are based on design information provided by AECOM to date, and the factual data obtained during the course of this investigation. Selected data from previous investigations for the existing bridges is also referenced and as summarized in the desktop study memorandum below:

- Draft Memorandum for "Foundation Assessment for Four Structures, Highway 401 Crossing of Etobicoke Creek, Mississauga, Ontario, G.W.P. 2147-10-00, Contract 2" dated September 14, 2016.

8 BRIDGE FOUNDATIONS

The subsurface conditions generally consist of the existing silty clay fill overlying stiff to very stiff silty clay and dense to very dense silty sand to sandy silt at the west and east abutments, respectively. A typically stiff to hard silty clay to clayey silt till underlies the above deposits. At the pier locations adjacent to the creek, the subsurface conditions immediately below the surface consist of gravelly sand at the west pier and silty clay at the east pier. The site is underlain by stiff to hard clayey silt to silty clay till overlying grey shale bedrock. The groundwater level at the west and east abutments is in the order of 6 m below existing highway grade. The groundwater level at the west and east piers general coincides with the creek level.

8.1 Structure Classification

In accordance with the currently applicable CHBDC CSA S6-14 (2014), the analysis and design of structures depend on its importance category and consequence classification. Such designations are defined by the Regulatory Authority which, in this case, is the Ministry of Transportation (MTO).

Information provided by AECOM indicates that this bridge has been classified as a Major Route Bridge in accordance with MTO Policy 2016-03 dated July 7, 2016, and with Typical Consequence based on CHBDC S6-14 Sections 4.4.2 and 6.5.2, respectively.

Based on the above classification and Table 6.1 in Section 6.5.2 in the CHBDC, a consequence factor, ψ, of 1.0 has been used for assessing ULS and SLS geotechnical resistances. Should the consequence classification changes, the geotechnical assessment and recommendations will need to be reviewed and revised as necessary.

8.2 Foundation Alternatives

<u>Piers</u>

Spread footings founded on bedrock is a feasible and cost effective option to support the east and west piers due to the relatively shallow depths to bedrock. It is understood that spread footings were proposed for supporting the existing east and west piers of the two adjacent bridges.

<u>Abutments</u>

The use of spread footings at the abutments is possible but not preferable. Due to the close proximity of the proposed foundation to the final slope surface, the footings would have to be founded at lower elevations in order to achieve higher bearing resistance. In addition, the existing bridge is supported on H-piles, therefore the widening portion is also recommended to be founded on H-piles (deep foundations) to minimize differential settlement. The resulting higher abutment walls, which also require deeper excavations, are unlikely to be cost effective.

For supporting the east and west abutments, consideration was given to the following foundation types for the abutments:

- Driven steel H-piles.
- Augered steel H-piles.

- Augered caissons.

Steel H-piles driven to bedrock are technically feasible for both abutments. It is understood that driven H-piles are being used for supporting the existing east and west abutments of the two adjacent bridges. However, driven piles are not recommended for the west abutment due to the presence of a sanitary trunk sewer in close proximity to the proposed west abutment footprint since there is potential for damage to the sewer pipe due to vibration during pile driving operations. Augered H-piles or caissons socketted into the bedrock are feasible for providing foundation support to both abutments. These types of foundation construction are associated with relatively small vibration and will, therefore, minimize the risks of damaging the existing sewer. Since caissons are not being considered, no foundation recommendations have been developed.

From a foundations technical, constructability and cost-effectiveness perspective, the recommended foundation options for the bridge foundations are as follows:

- Spread footings founded on or within bedrock for both piers;
- Augered H-piles socketted into bedrock at the west abutment;
- Augered H-piles socketted into bedrock or H-piles driven to bedrock at the east abutment.

More detailed comparisons of the technical advantages and disadvantages of the alternative foundation schemes is presented in Appendix E.

8.3 Foundations for East and West Piers

8.3.1 Spread Footings on Bedrock

Spread footings founded on weathered shale bedrock are considered feasible to support the new piers. The required depth of excavation for footing construction on weathered bedrock will vary depending on the location. Within the footprint of the structure widening and below the surficial gabion baskets, excavation for new footing construction at the west pier could extend through gravelly sand to reach the weathered shale at a minimum depth of 2.2 m. At the east pier where gabion baskets do not exist at some locations, the excavation could extend through firm to stiff silty clay and loose silty sand underlain by hard silty clay till to reach weathered shale at a minimum depth of 3.5 m.

For planning and preliminary design purposes, the new footings should be founded on weathered shale at or below the frost penetration depth.

8.3.2 Axial Bearing Resistance

The spread footings for the new widening bridge should be founded on weathered shale bedrock or mass concrete fill on bedrock should sub-excavation be required below the design founding level. The desktop study indicates that the existing pier footings of the adjacent EB Collector bridge was designed to be founded at approximate Elevation 140.2 m. The new footings should be founded at the elevation of the adjacent existing footings.

The footings founded on the weathered shale bedrock, or mass concrete fill of similar class as that of the footings placed on bedrock, may be designed using a Factored Geotechnical Resistance at ULS of 1,000 kPa and a Geotechnical Resistance at SLS (up to 25 mm settlement) of 700 kPa. These geotechnical resistances are applicable for a minimum footing width of 2.0 m.

Where the bedrock slopes within the foundation footprint, the footing subgrade should be prepared either by excavating a horizontal surface into the bedrock, or stepping the footing base.

The geotechnical resistances quoted above are for concentric, vertical loads only. In the case of eccentric or inclined loading, the geotechnical resistance must be calculated as illustrated in the CHBDC 2014 Clauses 6.7.3 and 6.7.4.

8.3.3 Lateral Resistance

Resistance to lateral forces / sliding resistance between the footing concrete and the bedrock surface should be evaluated in accordance with the CHBDC 2014 assuming an ultimate coefficient of friction of 0.5.

If the frictional component is insufficient to resist lateral forces, the horizontal resistance may be increased by dowelling the footing into the rock mass. Dowels are considered to be comparatively short steel bars that may be assumed to provide shear resistance.

8.3.4 Frost Cover

Although the shale is geologically defined as bedrock, it is susceptible to frost action. Therefore, all footings founded on shale must be provided with a minimum 1.2 m of earth cover as frost protection.

8.3.5 Footing Subgrade Preparation

The bases of the foundation excavations must be inspected by a geotechnical engineer to confirm that the exposed surface conforms to the design requirements and has been adequately prepared to receive concrete. Any loose or shattered rock must be removed and the footing be founded on undisturbed shale bedrock or mass concrete on bedrock.

Shale is prone to softening upon exposure to water and air. Mass concrete or working (mud) slab should be placed within 24 hours following completion of excavation to prevent deterioration of the shale. The working slab should be at least 100 mm thick and formed with the same class of concrete as that of the footings. An allowance should be made for sub-excavation to remove disturbed shale and unsuitable materials, amongst other reasons, from below the design founding level, the founding surface should be re-established using mass concrete fill of the same class of concrete as that used for the footing. All pier construction work should be carried out in the dry.

8.4 Foundations for the East and West Abutments

8.4.1 Driven Steel H-Piles

The east abutment foundations for the new widening bridge may be supported on steel H-piles driven to shale bedrock. Steel H-piles installed in augered holes are recommended for the west abutment and are discussed in the following sub-section 8.4.2. A standard HP 310 x 110 section may be used. The elevation at which the piles are expected to develop the required resistance may vary between 138 m and 139 m. These pile tip elevations should be used for estimating purposes only. The actual pile tip elevations will be controlled during pile driving as described in sub-section 8.4.1.3.

It is noted that the results of Borehole EC 16-01 at the east abutment indicate the presence of cobbles and/or very dense zones (e.g. 'N' > 100 blows) within the sandy silt to silty sand. Such obstructions induce risks of impeding pile penetration to shale bedrock during driving. Accordingly, it is recommended that augered piles also be used for supporting the east abutment.

8.4.1.1 Axial Resistance

The axial, Factored Geotechnical Resistance at Ultimate Limit States (ULS) and Geotechnical Resistance at Serviceability Limit States (SLS) for an HP 310 x 110 driven to refusal in shale bedrock, at the estimated elevations given above, are 2,000 kN per pile. The SLS condition does not govern design of piles driven to refusal in bedrock.

The structural capacity of a pile must not be exceeded and should be confirmed by the structural designer.

8.4.1.2 Lateral Resistance

Lateral bridge loadings can be geotechnically resisted by battered piles.

The lateral resistance in the cohesionless soils may be calculated using the coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) as follows:

$$k_s = n_h\, z / B \qquad (\mathrm{kN/m^3})$$
$$p_{ult} = 3\, \gamma'\, z\, K_p \qquad (\mathrm{kPa})$$

where:

z	=	depth of embedment of pile	(m)
B	=	pile width in metres (0.310 m for HP 310 x 110)	
n_h	=	coefficient related to soil density (kN/m^3)	
γ'	=	effective unit weight (kN/m^3) (equals to total unit weight for cohesionless soils above groundwater level and for cohesive soils)	
K_p	=	passive earth pressure coefficient	

The geotechnical lateral resistance acting on a pile in cohesive soils may be calculated using the coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) as follows:

$$k_s = 67\, S_u / B \qquad (\mathrm{kN/m^3})$$
$$p_{ult} = 9\, S_u \qquad (\mathrm{kPa})$$

where:

S_u	=	undrained shear strength	(kPa)
B	=	pile width in metres (0.310 m for HP 310 x 110)	

The geotechnical parameters provided in Table 8.1 below are for analyzing the interaction between a pile and the surrounding soil. Lateral pressures obtained from the soil-pile interaction analysis must not exceed the ultimate lateral resistance.

Table 8.1 – Geotechnical Parameters for Lateral Pile Analysis

Soil Unit	Elevation (m)		γ' (kN/m^3)	n_h (kN/m^3)	K_p	S_u (kPa)
	Top	Bottom				
West Abutment (reference Borehole EC 16-02)						
Silty Clay Fill stiff to very stiff	155	150	18	-	-	100
Silty Clay stiff to hard	150	145	19	-	-	150
Clayey Silt Till stiff to very stiff	145	140	19	-	-	100
East Abutment (reference Borehole EC16-01)						
Silty Clay Fill firm	155	152	17	-	-	40
Silty Sand to Sandy Silt dense to very dense (above groundwater)	152	149	21	15,000	4.2	-
Silty Sand to Sandy Silt dense to very dense (below groundwater)	149	145	11*	10,000	4.2	-
Clayey Silt Till hard	145	140	20	-	-	200

Note (*): Submerged unit weight

The spring constant, K, for analysis may be obtained by the expression, $K = k_s \times d_z \times B$ (kN/m), where k_s is the coefficient of horizontal subgrade reaction (kN/m^3), B is the pile width (m), d_z is the length (m) of the pile segment or element used in the analysis. The ultimate lateral resistance on any one segment of pile, P_{ult}, may be obtained from the expression, $P_{ult} = p_{ult} \times d_z \times B$. This represents the ultimate load at the contact between the soil and the pile above which additional load cannot be supported at greater displacements.

For lateral soil-pile group interaction analysis, the values for k_s should be reduced based on pile spacing.

Where a pile group is oriented ***perpendicular*** to the direction of loading, group action may be considered by reducing values of k_s using a reduction factor R as follows:

Pile Spacing Perpendicular to Direction of Loading	Horizontal Subgrade Reaction Reduction Factor, R
4 B	1.00
1 B	0.50

where B is the diameter of the pile, and spacing is measured centre to centre.

Where a pile group is oriented ***parallel*** to the direction of loading, group action may be considered by reducing values of k_S using a reduction factor R as follows:

Pile Spacing Parallel to Direction of Loading	Horizontal Subgrade Reaction Reduction Factor, R
8 B	1.00
6 B	0.70
4 B	0.40
3 B	0.25

8.4.1.3 Pile Installation

All piles shall be installed in accordance with OPSS 903.

Pile driving must be controlled in accordance with Standard SS 103-11 (Hiley Formula) and an ultimate pile resistance must be specified by the designer. The appropriate pile driving note to be shown on the contract drawing is "Piles to be driven to bedrock and in accordance with Standard SS103-11 using an ultimate geotechnical resistance equal to two times the maximum factored design load at ULS".

Glacially derived soils contain cobbles and boulders as evident in some of the boreholes. In order to prevent pile damage while driving through boulders, cobbles and harder/denser zones to achieve the required tip elevations and soil resistance, it is recommended that the pile tips be reinforced with driving shoes such as the Titus Steel Standard Points for H Piles or approved equivalent. It is noted that the geotechnical behaviour of weathered shale resembles that of soils and, as such, rock points are not required.

8.4.1.4 Frost Protection

Frost protection should be provided to all the pile caps and may take the form of 1.2 m of earth cover in any direction, or equivalent thermal insulation, over the underside of the pile cap.

8.4.2 Augered H-Piles

As discussed above, consideration should be given to supporting the west abutment on steel H-piles set within sockets that are drilled into bedrock. Augered piles may also be used at the east abutment. The sockets should be pre-drilled and the socket base should be cleaned of loose and shattered rock. The pile should then be lowered into the socket and the remaining space grouted with 30 MPa concrete. Based on the site conditions, it is recommended that a minimum socket depth of 3 m below the top of weathered shale be used.

Table 8.2 presents the recommended founding elevations for the augered H-piles at each abutment location.

Table 8.2
Founding Depths and Elevations for Augered H-Piles

Foundation Element	Borehole	Assumed Design Shale Elevation (m)	Assumed Founding Elevation (m)
West Abutment	EC 16-02	140	138
East Abutment	EC 16-01	140	138

8.4.2.1 Axial Resistance

For a HP 310 x 110 pile grouted within a 600 mm nominal diameter socket extended at least 3 m into weathered shale, a Factored Geotechnical Resistance at ULS of 2,000 kN per pile may be used for design. The SLS condition does not apply to piles socketted within bedrock.

The structural resistance of the pile should be reviewed by the structural designer to confirm that the value given above is not exceeded.

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8.4.2.2 Lateral Resistance

For design purposes, lateral soil resistance against an HP 310 x 110 pile can be assessed based on the method outlined in the CHBDC 2014.

Lateral resistance design for augered H-piles may be carried out as per the recommendations in sub-section 8.4.1.2 by substituting the H-pile width with the augered pile diameter.

It is understood that the current design incorporates augered piles battered at an inclination of 1H : 8V to provide additional resistance to lateral loadings at the abutments.

8.4.2.3 Augered Pile Installation

Augered pile installation should be in general accordance with clauses for caissons in OPSS 903. The pre-drilled holes for forming the pile socket should have a nominal diameter of 600 mm to be able to accommodate an HP 310 x 110 pile.

The augered holes would extend through existing embankment fill, and deposits of cohesionless and cohesive soils containing cobbles, boulders and shale fragments, to found within the weathered shale. The installation (augering) equipment must be capable of dislodging and removing any obstructions such as cobbles, boulders, shale/limestone slabs and to penetrate very dense/hard layers within the glacial till. Soil sloughing and water seepage will occur in unsupported holes primarily from the fill and water-bearing sands and silts. Construction of augered piles will require the use of temporary steel liners to support the sidewalls and to provide seepage cut-off where required. The liners must not be installed by vibratory means at the west abutment to avoid adverse effects on the existing sewer.

After each rock socket is drilled, cleaned and inspected, and subsequent to the seating of an H-pile in the socket, the annular space between the pile and the pre-drilled hole should be filled with 30 MPa concrete. Any accumulated water should be pumped out from the hole prior to placing concrete. Concrete should be placed with a minimum delay after the pile is set in place. The tremie technique should be employed to place concrete inside the caisson hole. Suggested wordings for an NSSP on augered pile installation are included in Appendix F.

9 ABUTMENT WALL BACKFILL AND LATERAL PRESSURES

The backfill to abutment walls should be in accordance with OPSS 902 and placed to the extents shown in OPSD 3101.150 where applicable. Any backfill to the walls should consist of Granular A or Granular B Type II material meeting the requirements of OPSS.PROV 1010.

If the support system allows yielding of the wall (unrestrained system), active horizontal earth pressure may be used in the geotechnical design of the structure. If the support system does not allow yielding (restrained system), at-rest horizontal earth pressures should be used.

Earth pressures acting on the structure may be assumed to be triangular and to be governed by the characteristics of the abutment backfill. For a fully drained condition, the pressures should be computed in accordance with the CHBDC but are generally given by the expression:

$$p_h = K(\gamma h + q)$$

where: p_h = horizontal pressure on the wall at depth h (kPa)

K = earth pressure coefficient (see Table 9.1)

γ = unit weight of retained soil (see Table 9.1)

h = depth below top of fill where pressure is computed (m)

q = value of any surcharge (kPa).

In accordance with Clause 6.9.3 of the CHBDC, a compaction surcharge should be added. The magnitude should be 12 kPa at the top of fill and decreasing to 0 kPa at a depth of 1.7 m for Granular A or Granular B Type II. Compaction equipment to be used adjacent to retaining structures should be restricted in accordance with OPSS.PROV 501.

Earth pressure coefficients for backfill to the abutment wall are dependent on the material used as backfill. Typical values are shown in Table 9.1.

Table 9.1 – Earth Pressure Coefficients

Wall Condition	Earth Pressure Coefficient (K)					
	OPSS Granular A and Granular B Type II $\phi = 35°$, $\gamma = 22.8$ kN/m^3		OPSS Granular B Type I $\phi = 32°$, $\gamma = 21.2$ kN/m^3		Existing Embankment Fill $\phi = 30°$, $\gamma = 20.0$ kN/m^3	
	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)
Active (Unrestrained Wall)	0.27	0.40	0.31	0.48	0.33	0.54

At rest (Restrained Wall)	0.43	-	0.47	-	0.50	-
Passive (Movement towards soil mass)	3.7	-	3.3	-	3.0	-

In conventional design, the use of a material with a high friction angle and low active pressure coefficient (e.g. Granular A, Granular B Type II) might be preferred as it results in lower earth pressures acting on the wall.

The factors in Table 9.1 are “ultimate” values and require certain movements for the respective conditions to be mobilized. The values to be used in design can be estimated from Figure C6.16 in the Commentary to the CHBDC 2014.

It is recommended that perforated sub-drains and weep holes be installed, where applicable, to provide positive drainage of the granular backfill behind the abutment walls. Reference should be made to OPSD 3102.100.

10 APPROACH EMBANKMENTS

Existing approach fills within the air gap have been constructed to just below the highway grade. Archived GA drawings indicate that the forward slopes have a design inclination of 2H : 1V with intermediate benches. Gabion baskets are present at the lower slopes for surficial slope stabilization and scour protection.

As part of the bridge widening, a small amount of new fill will need to be placed immediately behind the new abutment walls. Provided that the new fill is placed as recommended below and 2H : 1V slopes with benches to match the existing slope configurations are maintained, the resulting forward slopes will remain stable. Given the stress distribution within the approach fill and the competent ground conditions below, the foundation settlement that will be induced by the placement of the new fill should be considered negligible.

Prior to fill placement, the subgrade must be adequately prepared to receive the fill. Within widening areas, all topsoil, organics, soft/loosened or wet soils should be sub-excavated. All subgrade should be inspected and approved prior to placing fill. In areas where new fill is to be placed on existing fill, the existing fill surface should be benched in accordance with OPSD 208.01.

All approach fill must be constructed with adequate quality control in accordance with OPSS.PROV 206 and OPSS.PROV 501 requirements. It is recommended that OPSS.PROV.1010 Granular A or B Type II materials be used as new fill.

Vegetation cover should be established on all exposed earth slopes for protection against surficial erosion. Reference should be made to OPSS.PROV 804.

11 EXCAVATION

Temporary excavations will be required during construction at this site. All temporary excavations must be carried out in accordance with the Occupational Health and Safety Act (OHSA).

Excavation for foundation construction will extend predominantly through the existing embankment fill and the underlying native sands and silts, silty clay and clayey silt till. For the purpose of OHSA, the existing fills and native soils above the groundwater level may be classified as Type 3 soils. Any cohesionless soil below the water level may be classified as a Type 4 soil.

All excavations must be carried out in a manner that avoids undermining or destabilising the foundations of the existing bridges, existing slopes and the sanitary trunk sewer.

Pier footing construction will need to be carried out inside sheetpile cofferdams which are discussed in more details in the sections below.

Roadway protection (temporary shoring) will likely be required to retain the existing abutments, approaches and piers during new footing and pile cap construction. If space permits at some locations, temporary excavation may be formed with temporary sideslopes not steeper than 1H : 1V. Flatter slopes may be required at locations where the soils are less competent than what is assumed during design or where water seepage affects surficial stability.

Excavation and backfilling for foundation construction should be carried out with reference to the requirements in OPSS 902.

12 GROUNDWATER AND SURFACE WATER CONTROL

The piezometric readings in the east and west abutment areas indicate that the water level is at approximate Elevations 149.0 m and 149.2 m. The water levels measured during borehole augering at the location of the piers vary between Elevations 141.9 m and 141.6 m. Within the creek valley, the GA drawing shows that the water level in Etobicoke creek was at Elevation 142.5 in 1973.

The piers will be constructed adjacent to the creek flow channel. Excavations for footing construction will extend below the creek water level. It will, therefore, be necessary for the Contractor to construct a cofferdam (enclosure) capable of excluding the creek flow and supporting the native soils through which the excavation must be formed. One type of cofferdam that may be considered consists of interlocking steel sheet piles. For providing a partial groundwater cut-off below the base of the excavation, the sheet piles should be installed within the clayey silt to silty clay till and/or the underlying weathered shale. Pre-drilling of pilot holes along the cofferdam alignment may be required prior to sheetpile installation. The sheetpiles must be supplemented by sump pumping within the enclosure. It is anticipated that the following procedures may be required for pier construction.

- Install sheetpile cofferdam (enclosure)
- Dewater within the cofferdam
- Excavate to underside of the footing
- Construct the footing.

Suggested wordings for an NSSP on cofferdams for pier construction are included in Appendix F.

Responsibility for design of the cofferdam and dewatering system must remain the responsibility of the contractor.

The design of cofferdams must take into account the maximum creek level likely to occur during construction. It is recommended that the Contract Documents identify a creek level against which the cofferdam must provide protection and prevent flooding of the work area. The appropriate creek level should be determined by a hydrologist, and should probably incorporate the level that can be reached due to a storm of an appropriate return period.

The following geotechnical parameters may be used for design of the temporary cofferdam (sheet pile enclosure):

γ = 20 kN/m^3 (bulk unit weight)
γ' = 10 kN/m^3 (submerged unit weight below water level)
K_a = 0.33 (existing fill and native soils)

K_p = 3.0 (existing fill and native soils)

The actual pressure distribution acting on the shoring system is a function of the construction sequence and the relative flexibility of the wall and these factors must be considered when designing the shoring system.

12.1 Environmental Concerns

The volumes of water to be removed by unwatering, including the displacement of water during tremie concrete placement, should be estimated. A Permit To Take Water (PTTW) may be required.

Water discharged from unwatering operations or displaced during concrete placement may not be suitable for direct discharge to the creek. The contract documents must alert the contractor to this fact and include an item for treatment of the water to the satisfaction of TRCA, MOE, MNR, DFO or other agencies having jurisdiction prior to discharge into the creek.

13 ROADWAY PROTECTION

It is understood that the existing EB Collector and Express bridges will remain operational during construction of the new widening on the EC collector. Roadway protection is likely required during the widening of the EB collector beyond the east and west abutments. An item titled “Protection System” as per OPSS.PROV 539 (Level 2) should be included in the contract documents. The design of roadway protection should be the responsibility of the Contractor.

A feasible roadway protection system such as a temporary soldier pile and lagging wall may be designed using the geotechnical parameters given in section 12 above.

The designer of the roadway protection system should check whether the depth of piles is adequate for providing lateral resistance and base fixity.

The actual pressure distribution acting on the shoring system is a function of the construction sequence and the relative flexibility of the wall and these factors must be considered when designing the shoring system. All shoring systems should be designed by a Professional Engineer experienced in such designs.

14 EROSION AND SCOUR PROTECTION

Erosion/rock protection must be provided along the toe of any slopes and over all surfaces that may be in contact with the creek flow. These measures may include reconstructing and/or reinstating the gabion baskets on the lower slopes where applicable.

Scour protection measures should be provided to the footings at the piers. The underside of the footings must be at an elevation where they are protected against undermining by scour.

A vegetation cover should be established on all other exposed earth surfaces to protect against surficial erosion, in general accordance with OPSS.PROV 804.

15 SEISMIC CONSIDERATIONS

According to Clause 4.4.4 of the CHBDC, an earthquake with a 2475-year return period or 2% probability of exceedance in 50 years should be used for seismic design.

Based on the encountered soil conditions, this site is assessed to be Site Class C for seismic site response according to Table 4.1 of the CHBDC. The peak ground acceleration (PGA) associated with the design earthquake is 0.067g for Site Class C. The above PGA value should be assigned a site coefficient of 1.00 based on Table 4.8 of the CHBDC.

In accordance with Clause 4.6.5 of the CHBDC, retaining structures should be designed using active (K_{AE}) and passive (K_{PE}) earth pressure coefficients that incorporate the effects of earthquake loading.

For the design of retaining walls under seismic loading, the coefficients of horizontal earth pressure in Table 15.1 may be used:

Table 15.1 – Earth Pressure Coefficient for Earthquake Loading

Loading Condition	Granular A or Granular B Type II ϕ = 35°; γ = 22.8 kN/m^3	OPSS Granular B Type I or Type III ϕ = 32°; γ = 21.2 kN/m^3
Active (K_{AE})*	0.30	0.34
At Rest (K_{0E})**	0.54	0.59

Loading Condition	Granular A or Granular B Type II ϕ = 35°; γ = 22.8 kN/m^3	OPSS Granular B Type I or Type III ϕ = 32°; γ = 21.2 kN/m^3
Passive (K_{PE})	3.6	3.1

* After Mononobe and Okabe, passive case assumes a horizontal surface in front of the wall.

** After Woods (1973).

The very dense sandy silt to silty sand and the stiff to hard silty clay to clayey silt at this site are not considered susceptible to liquefaction under seismic loading.

16 EXISTING UTILITIES AND STRUCTURES

A 2 m diameter sanitary trunk sewer is located in close proximity to the proposed west abutment. It is important to confirm the exact location and existing conditions of this active sewer prior to completing the design, and that this sewer will not interfere with the new foundations, or will not be adversely affected by construction of the new foundations. It should also be prudent to confirm that there is no other buried utilities in the general vicinity of this site. These utilities must not be damaged during construction of the new structure. If necessary, relocation of, and/or special protective measures for affected utilities may be required. Consideration should also be given to requiring the Contractor to reconfirm the location of the 2 m diameter sewer and any other buried utilities in the area prior to commencing construction.

As discussed above, augered piles have been proposed at the west abutment to minimize vibration associated with pile installation in close proximity to the sewer. Pile driving at the east abutment (if used) will also induce vibration that must be monitored to confirm that it will not result in adverse effects on the existing bridge.

It is recommended that the following be carried out prior to the commencement of construction:

- Carry out pre-construction condition survey including documentation of any existing distress associated with the existing bridge structures and utilities. Any distress should be reported to and discussed with the structure/utility owner.

- Implement an instrumentation and monitoring program to include vibration and settlement monitoring during installation of roadway protection (shoring), cofferdam, excavation and new foundation construction (piles and footings). Establish review and alert level criteria for allowable settlement and lateral movement following discussions with the owner of the structure/utility.

Establish review and alert level criteria for vibration levels (in terms of peak particle velocity, ppv) during pile augering (at the west abutment) or pile driving. Establish and agree on remedial action, if required, prior to start of construction.

- Establish reference points over each abutment and piers of the existing structures and to monitor movement of these points relative to known, fixed reference points on a regular basis during foundation construction.

Suggested wordings for an NSSP on vibration/settlement monitoring are included in Appendix F.

Carry out post-construction condition survey of the existing structures/utilities.

17 CONSTRUCTION CONCERNS

Potential construction concerns include, but are not necessarily limited to, the following:

1. Foundation construction in close proximity to existing sewer and bridge foundations.

Settlement and vibration monitoring of the existing sewer and bridge foundations should be conducted before and during construction. Settlement monitoring should continue after construction.

2. Presence of cobbles and boulders within the glacial deposits.

Glacial deposits inherently contain cobbles and boulders, which may affect installation of H-piles. The Contractor shall be prepared to remove, drill through and/or penetrate these obstructions and extend the piles to the design foundation level.

3. Pile refusal at higher elevation.

The glacial till deposits at this site may contain cobbles, boulders and shale fragments. It is possible that some driven piles, where used, at the east abutment would achieve refusal at a higher elevation than anticipated due to encountering obstructions. A pile must not be damaged by overdriving and driving must not terminate prematurely without the approval of the design team.

4. Augered Pile installation

Cohesionless soils could be susceptible to disturbance under conditions of unbalanced hydrostatic head. Where augered piles are employed, temporary steel liners should be used to support the hole sidewalls and provide seepage cut-off where required. The liners must not be installed by vibratory means at the west abutment to avoid adverse effects on the existing sewer. The augering equipment must also be able to dislodge, remove or otherwise handle obstructions and hard/very dense zones within the soils.

5. Excavations

Excavations for pier footing construction must not undermine the adjacent existing footings. Care must be exercised during excavation to avoid disturbing the founding subgrade. The exposed subgrade soils should be expeditiously inspected, approved and protected from disturbance.

6. Underground utilities

Any information on the location of the sanitary trunk sewer and other buried utilities should be carefully reviewed. All new foundation footprints should be clear of any buried utility. Vibration (and settlement, if possible) monitoring for buried utilities, should be provided by qualified personnel prior during and upon completion of construction.

7. Creek zone excavation and sediments

At the pier locations, excavation for footing construction adjacent to the creek will require removal of the excavated material from the site. The creek should be protected from excess sediment loading at all times.

18 CLOSURE

Engineering analysis and preparation of this foundation design report was carried out by Messrs. Mohamad Hosney, P.Eng. and Dr. Sydney Pang, P.Eng.

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Appendix A

Record of Borehole Sheets

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

CLASSIFICATION	PARTICLE SIZE	VISUAL IDENTIFICATION
Boulders	Greater than 200mm	same
Cobbles	75 to 200mm	same
Gravel	4.75 to 75mm	5 to 75mm
Sand	0.075 to 4.75mm	Not visible particles to 5mm
Silt	0.002 to 0.075mm	Non-plastic particles, not visible to the naked eye
Clay	Less than 0.002mm	Plastic particles, not visible to the naked eye

2. COARSE GRAIN SOIL DESCRIPTION (50% greater than 0.075mm)

TERMINOLOGY	PROPORTION
Trace or Occasional	Less than 10%
Some	10 to 20%
Adjective (e.g. silty or sandy)	20 to 35%
And (e.g. sand and gravel)	35 to 50%

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

DESCRIPTIVE TERM	UNDRAINED SHEAR STRENGTH (kPa)	APPROXIMATE SPT(1) 'N' VALUE
Very Soft	12 or less	Less than 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	Greater than 200	Greater than 30

NOTE: Hierarchy of Soil Strength Prediction

1) Laboratory Triaxial Testing
2) Field Insitu Vane Testing
3) Laboratory Vane Testing
4) SPT value
5) Pocket Penetrometer

4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

DESCRIPTIVE TERM	SPT "N" VALUE
Very Loose	Less than 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	Greater than 50

5. LEGEND FOR RECORDS OF BOREHOLES

SYMBOLS AND ABBREVIATIONS FOR SAMPLE TYPE

SS Split Spoon Sample WS Wash Sample AS Auger (Grab) Sample
TW Thin Wall Shelby Tube Sample TP Thin Wall Piston Sample
PH Sampler Advanced by Hydraulic Pressure PM Sampler Advanced by Manual Pressure
WH Sampler Advanced by Self Static Weight RC Rock Core SC Soil Core

$$\text{Sensitivity} = \frac{\text{Undisturbed Shear Strength}}{\text{Remoulded Shear Strength}}$$

Water Level

C_{pen} Shear Strength Determination by Pocket Penetrometer

(1) SPT 'N' Value Standard Penetration Test 'N' Value – refers to the number of blows from a 63.5kg hammer free falling a height of 0.76m to advance a standard 50 mm outside diameter split spoon sampler for 0.3 m depth into undisturbed ground.

(2) DCPT Dynamic Cone Penetration Test – Continuous penetration of a 50 mm outside diameter, 60° conical steel point attached to "A" size rods driven by a 63.5 kg hammer free falling a height of 0.76 m. The resistance to cone penetration is the number of hammer blows required for each 0.3 m advance of the conical point into undisturbed ground.

EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION

Fresh (FR)	No visible signs of weathering.
Fresh Jointed (FJ)	Weathering limited to the surface of major discontinuities.
Slightly Weathered (SW)	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock material.
Moderately Weathered (MW)	Weathering extends throughout the rock mass, but the rock material is not friable.
Highly Weathered (HW)	Weathering extends throughout the rock mass and the rock is partly friable.
Completely Weathered (CW)	Rock is wholly decomposed and in a friable condition, but the rock texture and structure are preserved.

DISCONTINUITY SPACING

Bedding	**Bedding Plane Spacing**
Very thickly bedded	Greater than 2m
Thickly bedded	0.6 to 2m
Medium bedded	0.2 to 0.6m
Thinly bedded	60mm to 0.2m
Very thinly bedded	20 to 60mm
Laminated	6 to 20mm
Thinly Laminated	Less than 6mm

SYMBOLS

CLAYSTONE

SILTSTONE

SANDSTONE

COAL

BEDROCK

STRENGTH CLASSIFICATION

Rock Strength	**Approximate Uniaxial Compressive Strength**		**Field Estimation of Hardness***
	(MPa)	**(psi)**	
Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.
Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail

TERMS

Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length
Solid Core Recovery:(SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run
Rock Quality Designation:(RQD)	Total length of sound core recovered in pieces 0.1m in length or larger as a % of total core run length.
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen
Fracture Index:(FI)	Frequency of natural fractures per 0.3m of core run.

UNIFIED SOILS CLASSIFICATION

MAJOR DIVISIONS		GROUP SYMBOL	TYPICAL DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW	Well-graded gravels or gravel-sand mixtures, little or no fines.
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines.
		GM	Silty gravels, gravel-sand-silt mixtures.
		GC	Clayey gravels, gravel-sand-clay mixtures.
	SAND AND SANDY SOILS	SW	Well-graded sands or gravelly sands, little or no fines.
		SP	Poorly-graded sands or gravelly sands, little or no fines.
		SM	Silty sands, sand-silt mixtures.
		SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS	SILTS AND CLAYS $W_L < 50\%$	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays. ($W_L < 30\%$).
		CI	Inorganic clays of medium plasticity, silty clays. ($30\% < W_L < 50\%$).
		OL	Organic silts and organic silty-clays of low plasticity.
	SILTS AND CLAYS $W_L > 50\%$	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
		OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils.
CLAY SHALE			
SANDSTONE			
SILTSTONE			
CLAYSTONE			
COAL			

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No CA 16-01

METRIC

GWP# 2147-10-00 | LOCATION Construction Access N 4 834 962.2 E 294 912.8 | ORIGINATED BY ES

HWY 401 | BOREHOLE TYPE Solid Stem Augers | COMPILED BY AN

DATUM Geodetic | DATE 2016.08.27 - 2016.08.27 | CHECKED BY SKP

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
155.0	GROUND SURFACE									
0.0	**TOPSOIL:** (100mm)									
0.1	Silty **CLAY**, some sand, trace gravel Firm to Stiff Dark Brown to Brown Moist (FILL)	1	SS	5						
		2	SS	6	154					
		3	SS	9	153					
152.6 2.4	Clayey **SILT**, some sand, trace gravel Very Stiff Grey Moist (TILL)	4	SS	22	152					
		5	SS	30			6	27	46	21
151.3 3.7	Silty **SAND** to Sandy **SILT**, some clay, some gravel Dense to Very Dense Brown Moist	6	SS	49	151					
	Occasional cobbles	7	SS	82	150					
		8	SS	100/ 0.275	149 / 148		16	35	35	14
147.0		9	SS	100/ 0.250	147					
8.0	END OF BOREHOLE AT 8.0m. BOREHOLE OPEN AND DRY UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND CUTTINGS TO SURFACE.									

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

$+^3, \times^3$: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No CA 16-02

METRIC

GWP# 2147-10-00	LOCATION Construction Access N 4 834 891.0 E 294 811.4	ORIGINATED BY ES
HWY 401	BOREHOLE TYPE Hollow Stem Augers	COMPILED BY AN
DATUM Geodetic	DATE 2016.08.24 - 2016.08.24	CHECKED BY SKP

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) | SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE (20 40 60 80 100) | PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L — WATER CONTENT (%) (20 40 60) | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%)

ELEV DEPTH	DESCRIPTION	STRAT PLOT
155.0	GROUND SURFACE	
0.0	**TOPSOIL:** (13mm)	
	SILT to **SAND**, some clay, some gravel Very Dense Brown Moist (FILL)	
154.2 0.8	Silty **CLAY**, with sand, trace gravel Very Stiff to Hard Brown Moist (FILL) Possible cobbles	
150.9 4.1	Silty **CLAY**, some sand, trace gravel Stiff to Very Stiff Brown Moist	
146.8 8.2	END OF BOREHOLE AT 8.2m. BOREHOLE OPEN AND DRY UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND CUTTINGS TO SURFACE.	

NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
1	SS	54				
2	SS	20				
3	SS	17	7	34	33	26
4	SS	12				
5	SS	73/ 0.200				
6	SS	12				
7	SS	22	3	21	42	34
8	SS	24				
9	SS	18	6	26	40	28

Elevation scale: 154, 153, 152, 151, 150, 149, 148, 147

+³, ×³: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No EC 16-01

METRIC

GWP#	2147-10-00	LOCATION	Etobicoke Creek N 4 834 954.4 E 294 902.6	ORIGINATED BY	ES/TM
HWY	401	BOREHOLE TYPE	Hollow Stem Augers/NQ Coring	COMPILED BY	AN
DATUM	Geodetic	DATE	2016.08.26 - 2016.08.26	CHECKED BY	SKP

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
155.0	GROUND SURFACE								
0.0	Silty **CLAY**, some sand, trace gravel, trace organics, trace rootlets Firm Brown Moist (FILL)	1	SS	6					
		2	SS	6	154	2	24	41	33
		3	SS	4	153				
152.7 2.3	Silty **SAND** to Sandy **SILT**, some clay, trace gravel Dense to Very Dense Brown Moist	4	SS	49	152				
		5	SS	48					
		6	SS	50	151	8	45	33	14
		7	SS	91/ 0.225	150				
		8	SS	50/ 0.075	149				
	Grey	9	SS	88	148 147	6	35	44	15
		10	SS		146				

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

GROUND WATER CONDITIONS

STRAT PLOT

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No EC 16-01

METRIC

GWP# 2147-10-00 | LOCATION Etobicoke Creek N 4 834 954.4 E 294 902.6 | ORIGINATED BY ES/TM
HWY 401 | BOREHOLE TYPE Hollow Stem Augers/NQ Coring | COMPILED BY AN
DATUM Geodetic | DATE 2016.08.26 - 2016.08.26 | CHECKED BY SKP

Continued From Previous Page

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT kN/m3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
144.7								
10.3	Clayey **SILT**, trace to some sand, trace gravel. Hard. Grey. Moist. (TILL)	11	SS	80/ 0.275	144	o		1 7 72 20
					143			
		12	SS	78/ 0.250		o		
					142			
		13	SS	47	141	o		
139.8					140			
15.2	**SHALE**, highly weathered. Grey	14	SS	119/ 0.150		o		
					139			
138.4	Rock coring started at 16.6m depth						FI	
16.6	**SHALE**, moderately weathered, thinly bedded, horizontal laminated, grey, weak with strong to very strong limestone interbeds						>20	RUN #1 TCR=92% SCR=13% RQD=0%
		1	RUN		138		>20	UCS = 244.9MPa
	Limestone interbeds (25mm to 50mm) at 16.9m, 17.0m, 17.4m, 17.5m and 17.7m						>20	
							>20	UCS = 64.4MPa
	Highly broken zone (100mm) at 17.6m				137		>20	RUN #2 TCR=62% SCR=17% RQD=0%
	Limestone interbeds (25mm to 75mm) at 17.8m, 17.9m, 18.5m and 18.7m						>20	
		2	RUN				>20	
	Highly broken zone (25mm to 50mm) at 18.1m, 18.2m and 18.4m				136		>20	
	Vertical fracture (50mm) at 18.7m						>10	RUN #3 TCR=100% SCR=88% RQD=80%
							9	UCS = 198.7MPa
	Vertical fracture (150mm) at 19.3m						>15	

Continued Next Page

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE — 20 40 60 80 100
PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L — WATER CONTENT (%) 20 40 60

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No EC 16-01

METRIC

GWP#	2147-10-00	LOCATION	Etobicoke Creek N 4 834 954.4 E 294 902.6	ORIGINATED BY	ES/TM
HWY	401	BOREHOLE TYPE	Hollow Stem Augers/NQ Coring	COMPILED BY	AN
DATUM	Geodetic	DATE	2016.08.26 - 2016.08.26	CHECKED BY	SKP

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 | SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100 | PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60 | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DCPT / SHEAR STRENGTH	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS GR SA SI CL
	Continued From Previous Page										
	Limestone interbeds (25mm to 100mm) at 19.3m, 19.5m, 20.3m, 20.4m, 20.5m and 20.8m		3	RUN						7 5 8	
134.1 20.9	END OF BOREHOLE AT 20.9m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen.										

WATER LEVEL READINGS:

DATE	DEPTH (m)	ELEV. (m)
Oct05/2016	5.8	149.2

+ 3, × 3: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No EC 16-02

METRIC

GWP# 2147-10-00 LOCATION Etobicoke Creek N 4 834 895.8 E 294 820.5 ORIGINATED BY ES

HWY 401 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN

DATUM Geodetic DATE 2016.08.22 - 2016.08.24 CHECKED BY SKP

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	GR	SA	SI	CL
155.0	GROUND SURFACE								
0.0	**TOPSOIL:** (50mm)								
	SILT to **SAND**, trace silt, some gravel, Very Dense, Brown, Moist (FILL)	1	SS	62					
154.3 / 0.7	Silty **CLAY**, with sand, trace gravel, occasional wood fibres, Brown, Moist (FILL)	2	SS	18					
		3	SS	11					
	Stiff to Very Stiff	4	SS	8		3	30	37	30
		5	SS	16					
	Grey	6	SS	22					
150.1 / 4.9	Silty **CLAY**, some sand, trace gravel, Stiff to Hard, Grey, Moist	7	SS	14					
		8	SS	12		3	19	44	34
		9	SS	22					
		10	SS	45		4	18	42	36

Elevation scale: 154, 153, 152, 151, 150, 149, 148, 147, 146

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

ONTMT4S MTO-12669.GPJ 2015TEMPLATE(MTO).GDT 10/14/16

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No EC 16-02

METRIC

GWP# 2147-10-00 LOCATION Etobicoke Creek N 4 834 895.8 E 294 820.5 ORIGINATED BY ES

HWY 401 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN

DATUM Geodetic DATE 2016.08.22 - 2016.08.24 CHECKED BY SKP

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page							
144.8	Occasional cobbles							
10.2	Clayey **SILT**, some sand, trace gravel, occasional sand seams Very Stiff to Stiff Grey Moist (TILL)							
		11	SS	19	144	○		
					143			
		12	SS	17		○		
					142			
		13	SS	11	141	○		5 20 53 22
140.1 14.9	**SHALE**, highly weathered, occasional limestone layers Grey				140			
		14	SS	101/ 0.200		○		
					139			
138.2	Rock coring started at 16.8m depth	15	SS	100/ 0.025		○	FI	
16.8	**SHALE**, moderately weathered, thinly bedded, grey, weak to medium strong with medium strong limestone interbeds Limestone interbed (25mm) at 16.9m and 17.6m	1	RUN		138		>20 >20 >20	RUN #1 TCR=100% SCR=0% RQD=0%
	Limestone interbed (25mm to 75mm) at 17.8m, 18.0m, 18.1m and 18.7m Vertical fracture (25mm) at 18.1m Soft zone (250mm) at 18.2m	2	RUN		137 136		>20 >20 >20 >20 >20	RUN #2 TCR=75% SCR=0% RQD=0%
	Clay seam (50mm) at 19.2m Limestone interbed (25mm) at 19.3m	3	RUN				>10 >20	RUN #3 TCR=100% SCR=0% RQD=0% UCS = 26.8MPa

Continued Next Page

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100
PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No EC 16-02

METRIC

GWP# 2147-10-00 | LOCATION Etobicoke Creek N 4 834 895.8 E 294 820.5 | ORIGINATED BY ES
HWY 401 | BOREHOLE TYPE Hollow Stem Augers/NQ Coring | COMPILED BY AN
DATUM Geodetic | DATE 2016.08.22 - 2016.08.24 | CHECKED BY SKP

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page										
	Limestone layer (25mm) at 19.9m Vertical fracture (175mm) at 19.9m		4	RUN						>20 >10 >10 >10	UCS = 12.4MPa UCS = 22.9MPa UCS = 27.3MPa RUN #4 TCR=87% SCR=63% RQD=0%
133.9 21.1	END OF BOREHOLE AT 21.1m. WATER LEVEL IN OPEN BOREHOLE AT 4.3m UPON COMPLETION. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Oct05/2016 6.0 149.0						134				

+ ³, × ³: Numbers refer to Sensitivity

20 / 15 / 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No EC 16-03

METRIC

GWP# 2147-10-00 LOCATION Etobicoke Creek N 4 834 915.2 E 294 854.9 ORIGINATED BY MH
HWY 401 BOREHOLE TYPE Hollow Stem Augers/HQ Coring COMPILED BY AN
DATUM Geodetic DATE 2016.09.13 - 2016.09.13 CHECKED BY SKP

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
144.0	GROUND SURFACE								
0.0	Gabion baskets with cobbles								
143.1 / 0.9	Gravelly **SAND**, trace silt, pieces of weathered shale Compact Brown Moist	1	SS	20		143	○		
		2	SS	17	▽	142	○		35 51 14 (SI+CL)
141.8 / 2.2	**SHALE**, highly weathered Grey	3	SS	26		141	○		
		4	SS	31		140	○		
		5	SS	67		139	○		
138.3	Rock coring started at 5.7m depth	6	SS	50/0.075			○	FI	
5.7	**SHALE**, moderately to slightly weathered, thinly bedded, grey, weak to medium strong with strong to very strong limestone interbeds Rubble zone (50mm) at 5.7m Limestone layer (75mm) at 5.9m Horizontal joint (25mm) from 5.9m to 6.7m	1	RUN			138		6, 3, 3	UCS = 157.2MPa RUN #1 TCR=100% SCR=93% RQD=37%
	Rubble zone (150mm) at 6.8m Horizontal joint (25mm) from 7.0m to 8.1m	2	RUN			137		8, >20, 5, 5	UCS = 27.6MPa RUN #2 TCR=100% SCR=82% RQD=50% UCS = 15.9MPa
	Horizontal joint (25mm) at 8.3m, 8.5m and 8.7m Clay layer (25mm) at 8.4m	3	RUN			136		4, 8, 2	UCS = 7.8MPa UCS = 20.5MPa UCS = 44.8MPa RUN #3 TCR=100% SCR=79% RQD=63%
135.2 / 8.8	END OF BOREHOLE AT 8.8m. WATER LEVEL AT 2.1m UPON COMPLETION. BOREHOLE BACKFILLED WITH MIXTURE OF BENTONITE HOLEPLUG AND CUTTINGS, THEN GABION WALL REINSTATED TO SURFACE.								

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE: 20 40 60 80 100
PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%): 20 40 60

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No EC 16-04

METRIC

GWP# 2147-10-00 | LOCATION Etobicoke Creek N 4 834 930.9 E 294 876.2 | ORIGINATED BY MH

HWY 401 | BOREHOLE TYPE Hollow Stem Augers/HQ Coring | COMPILED BY AN

DATUM Geodetic | DATE 2016.09.12 - 2016.09.12 | CHECKED BY SKP

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT / FI	REMARKS & GRAIN SIZE DISTRIBUTION (%)
144.0	GROUND SURFACE								
0.0	**TOPSOIL:** (50mm)								
	Silty **CLAY**, trace to some sand and gravel, trace roots and organics Firm to Stiff Grey Moist	1	SS	5					
		2	SS	10		143			
142.6 / 1.4	Silty **SAND**, trace to some clay, trace organics Loose Grey Wet	3	SS	4		142			
141.8 / 2.2	Silty **CLAY**, trace weathered shale Hard Grey Wet (TILL)	4	SS	33	∇	141			
		5	SS	28					
140.5 / 3.5	**SHALE**, highly weathered Grey	6	SS	100/ 0.275		140			
		7	SS	100/ 0.200		139			
138.8	Rock coring started at 5.2m depth							FI	RUN #1 TCR=100% SCR=65% RQD=48%
5.2	**SHALE**, moderately weathered, thinly bedded, weak to very weak with strong to very strong limestone interbeds, grey Rubble zone (150mm) at 5.2m and (100mm) at 5.6m Horizontal joint (25mm) at 5.5m, 5.7m and 5.8m Limestone interbed (25mm) at 5.8m	1	RUN			138		>7 4 4 2 2	UCS = 10.1MPa UCS = 20.7MPa UCS = 17.2MPa
	Slightly weathered to fresh Horizontal joint (25mm) at 6.7m, 6.9m, 7.2m 7.9m and 8.0m Limestone interbed (25mm) at 7.0m and 7.9m	2	RUN			137 136		1 2 4 1 4	RUN #2 TCR=100% SCR=97% RQD=80% UCS = 17.0MPa UCS = 1.7MPa UCS = 24.3MPa
135.9 / 8.1	END OF BOREHOLE AT 8.1m. WATER LEVEL AT 2.4m UPON COMPLETION. BOREHOLE BACKFILLED WITH MIXTURE OF BENTONITE HOLEPLUG AND CUTTINGS TO SURFACE.								

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

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

THURBER

Appendix B

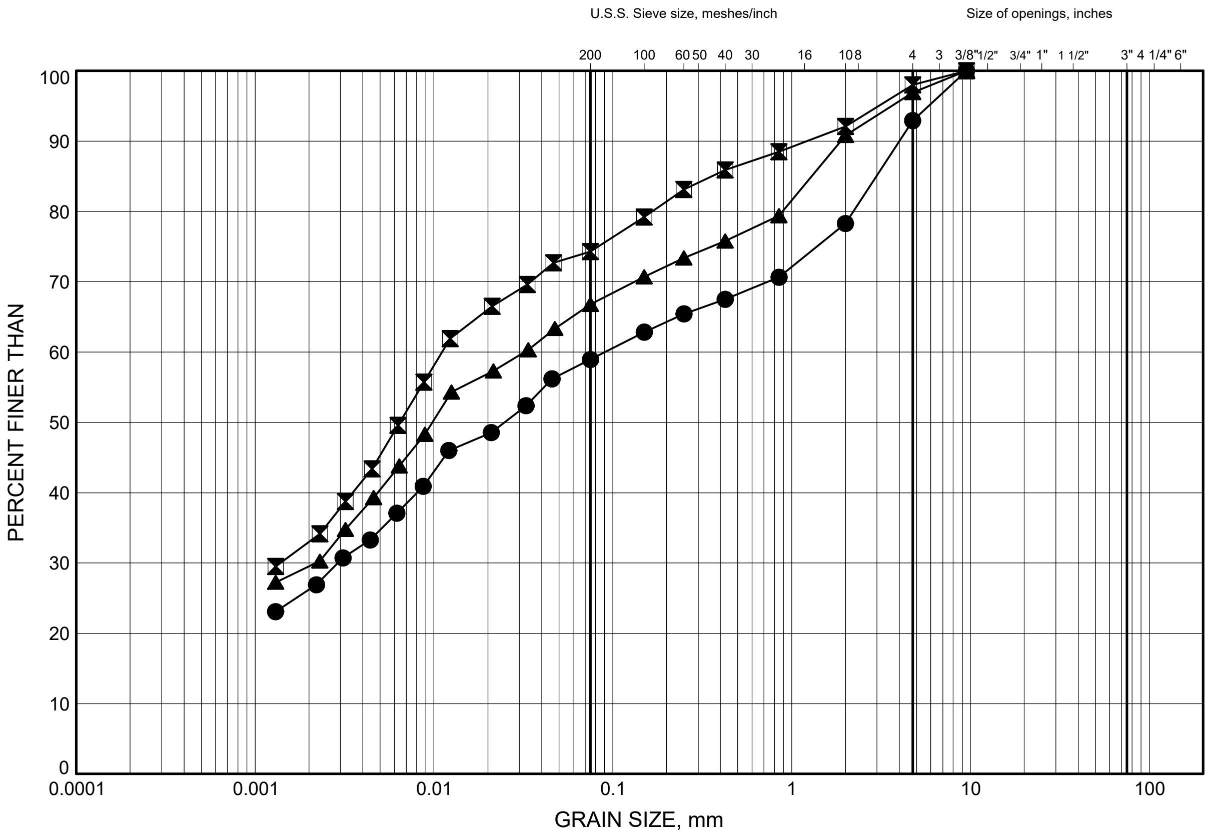
Laboratory Test Results Including Point Load Tests

Etobicoke Creek

GRAIN SIZE DISTRIBUTION

FIGURE B1

Silty CLAY FILL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	CA 16-02	1.83	153.17
⧗	EC 16-01	1.07	153.93
▲	EC 16-02	2.59	152.41

GRAIN SIZE DISTRIBUTION - THURBER MTO-12669.GPJ 10/12/16

Date October 2016

GWP# 2147-10-00

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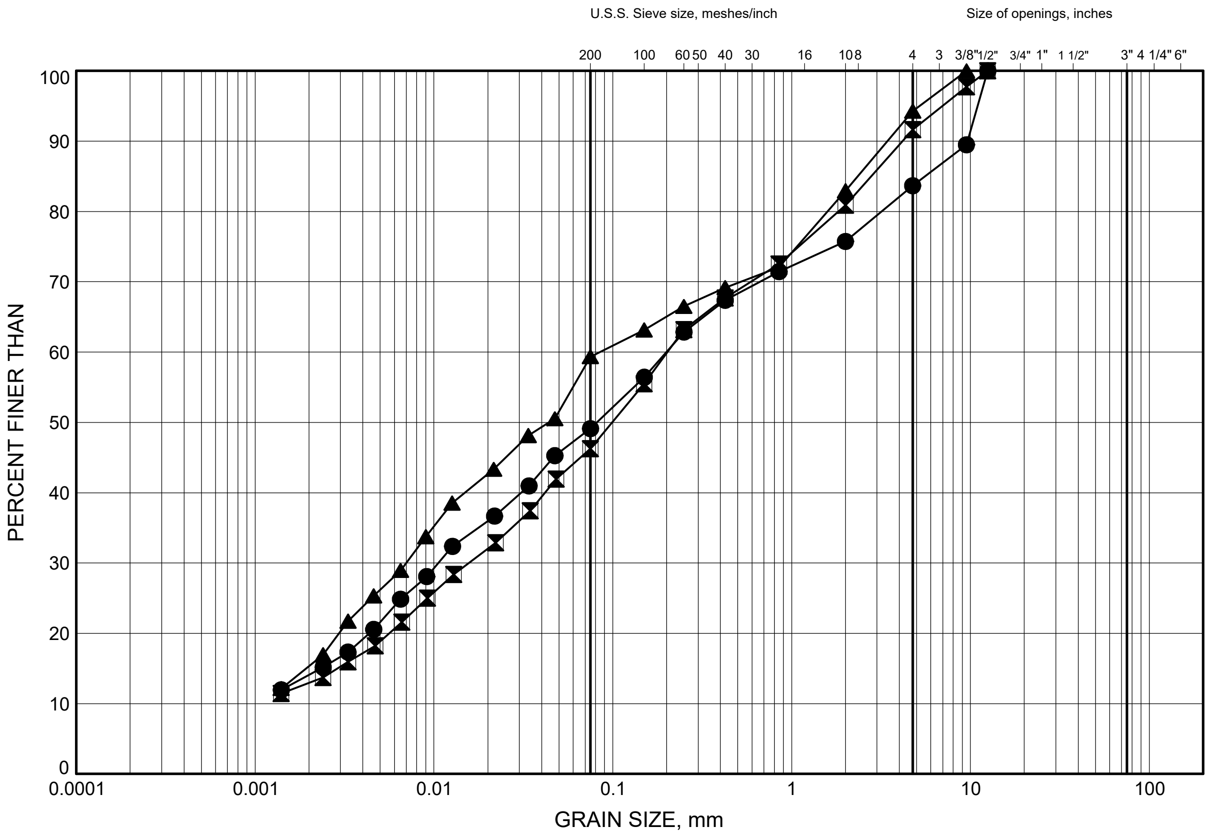
Etobicoke Creek

GRAIN SIZE DISTRIBUTION

FIGURE B2

Silty SAND to Sandy SILT



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	CA 16-01	6.40	148.60
⧗	EC 16-01	4.11	150.89
▲	EC 16-01	7.85	147.15

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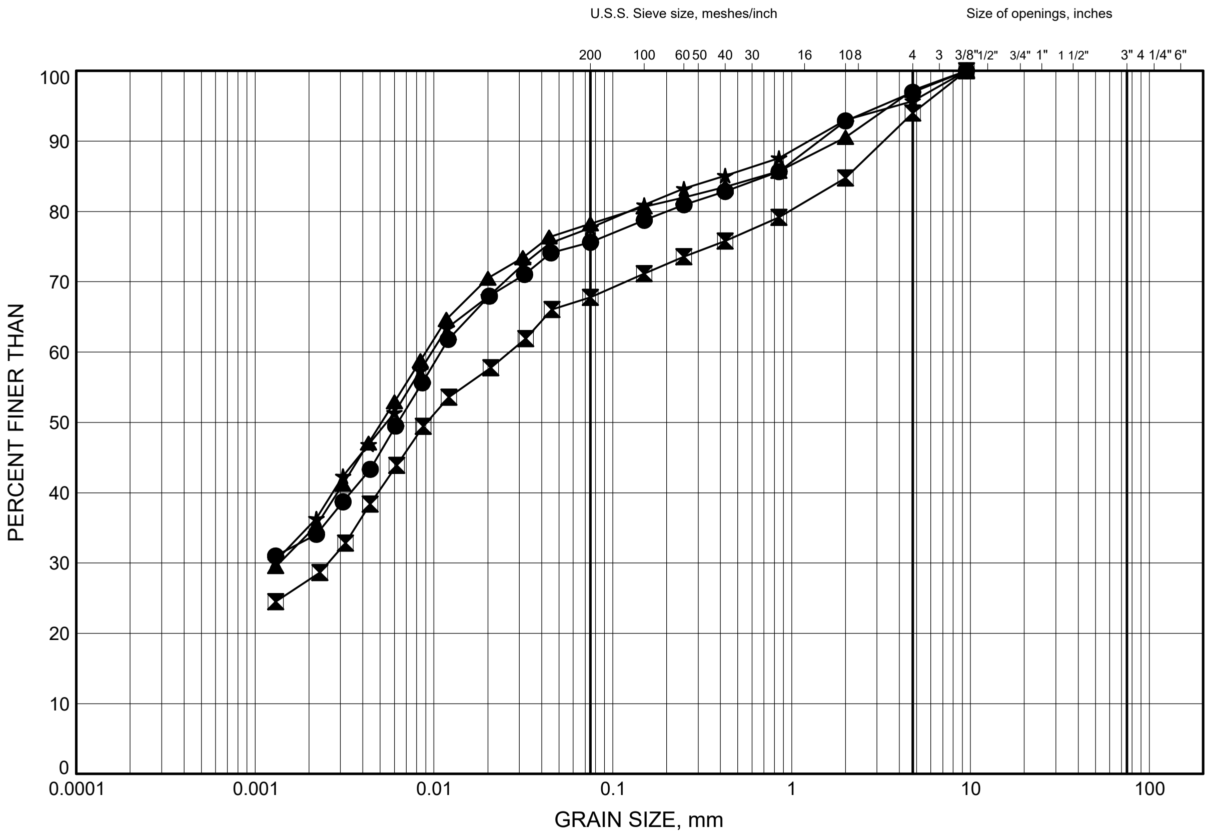
GRAIN SIZE DISTRIBUTION

FIGURE B3

Silty CLAY



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	CA 16-02	4.88	150.12
⧗	CA 16-02	7.92	147.08
▲	EC 16-02	6.40	148.60
★	EC 16-02	9.45	145.55

Date October 2016

GWP# 2147-10-00

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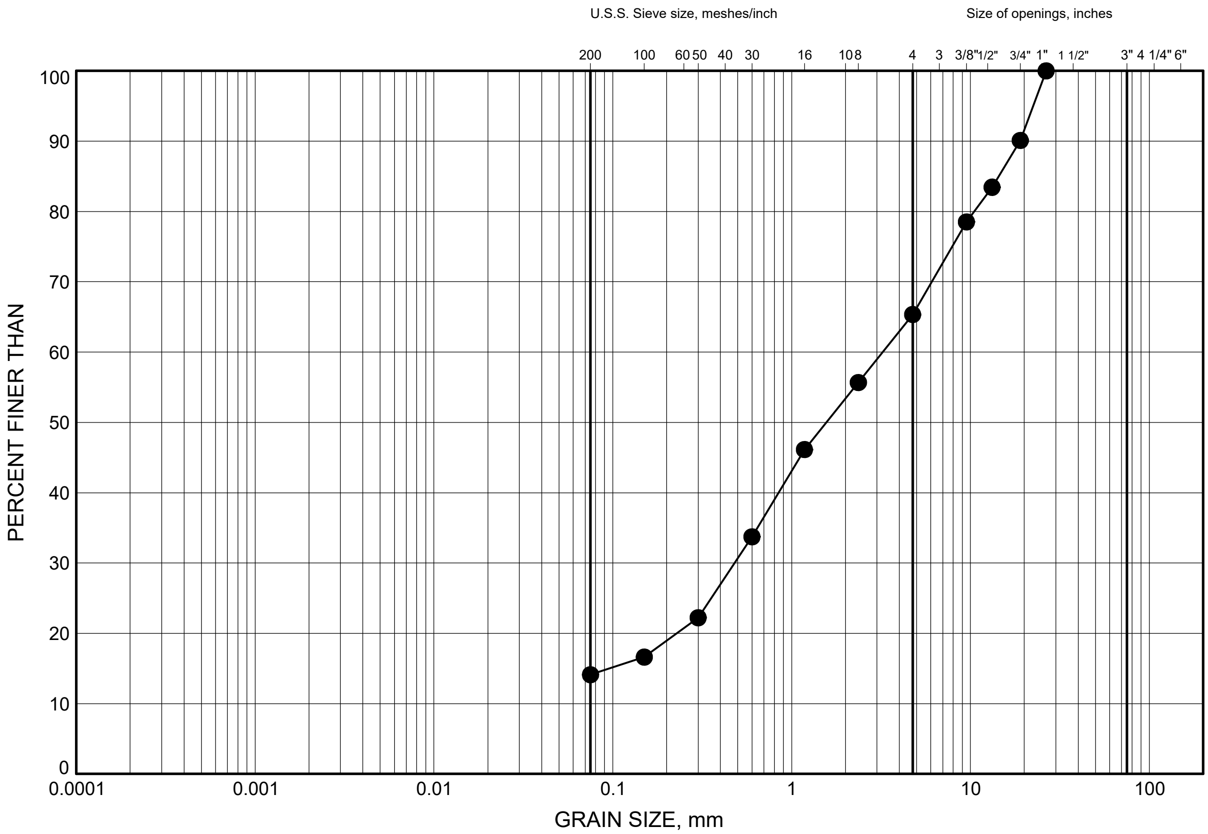
Etobicoke Creek

GRAIN SIZE DISTRIBUTION

FIGURE B4

Gravelly SAND



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	EC 16-03	1.83	142.17

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Date October 2016

GWP# 2147-10-00

THURBER

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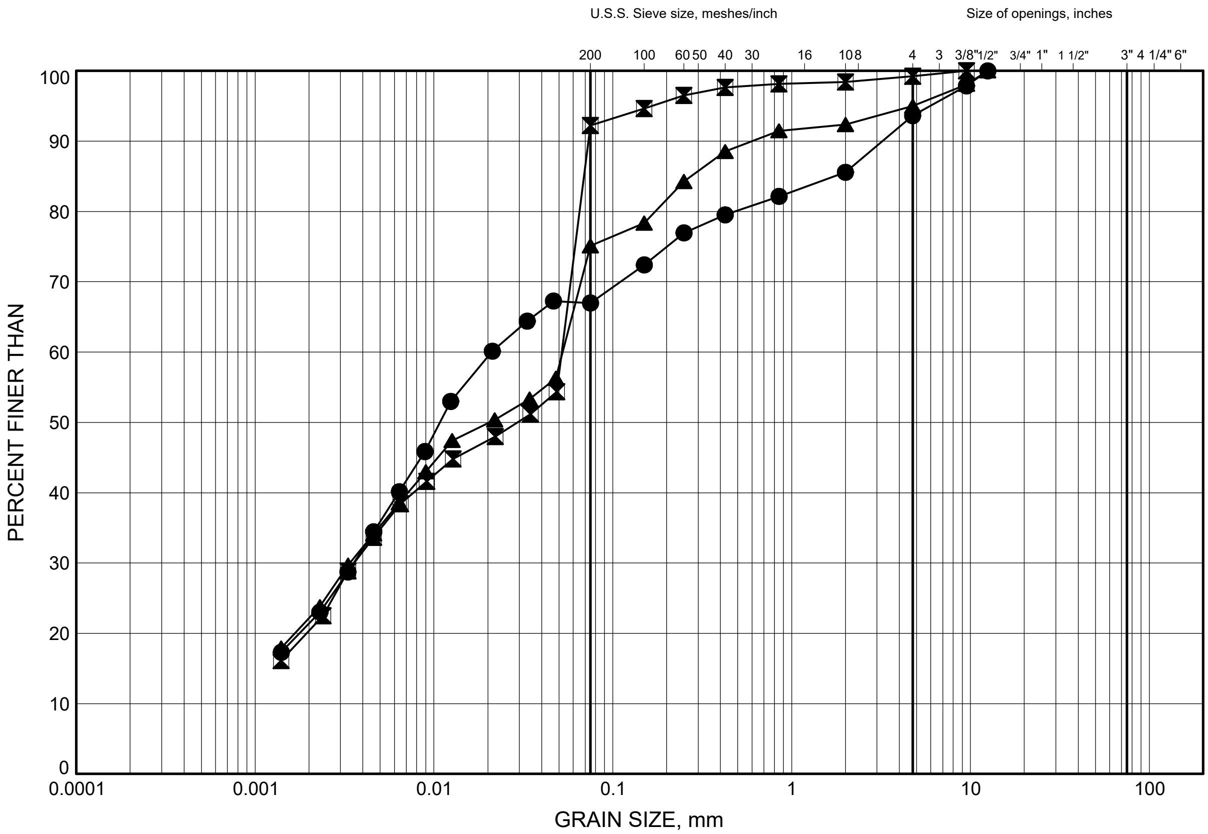
Chkd. MH

Etobicoke Creek

GRAIN SIZE DISTRIBUTION

FIGURE B5

Clayey SILT to Silty CLAY TILL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	CA 16-01	3.35	151.65
⧗	EC 16-01	10.90	144.10
▲	EC 16-02	14.02	140.98

GRAIN SIZE DISTRIBUTION - THURBER MTO-12669.GPJ 10/12/16

Date October 2016

GWP# 2147-10-00

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Prep'd AN

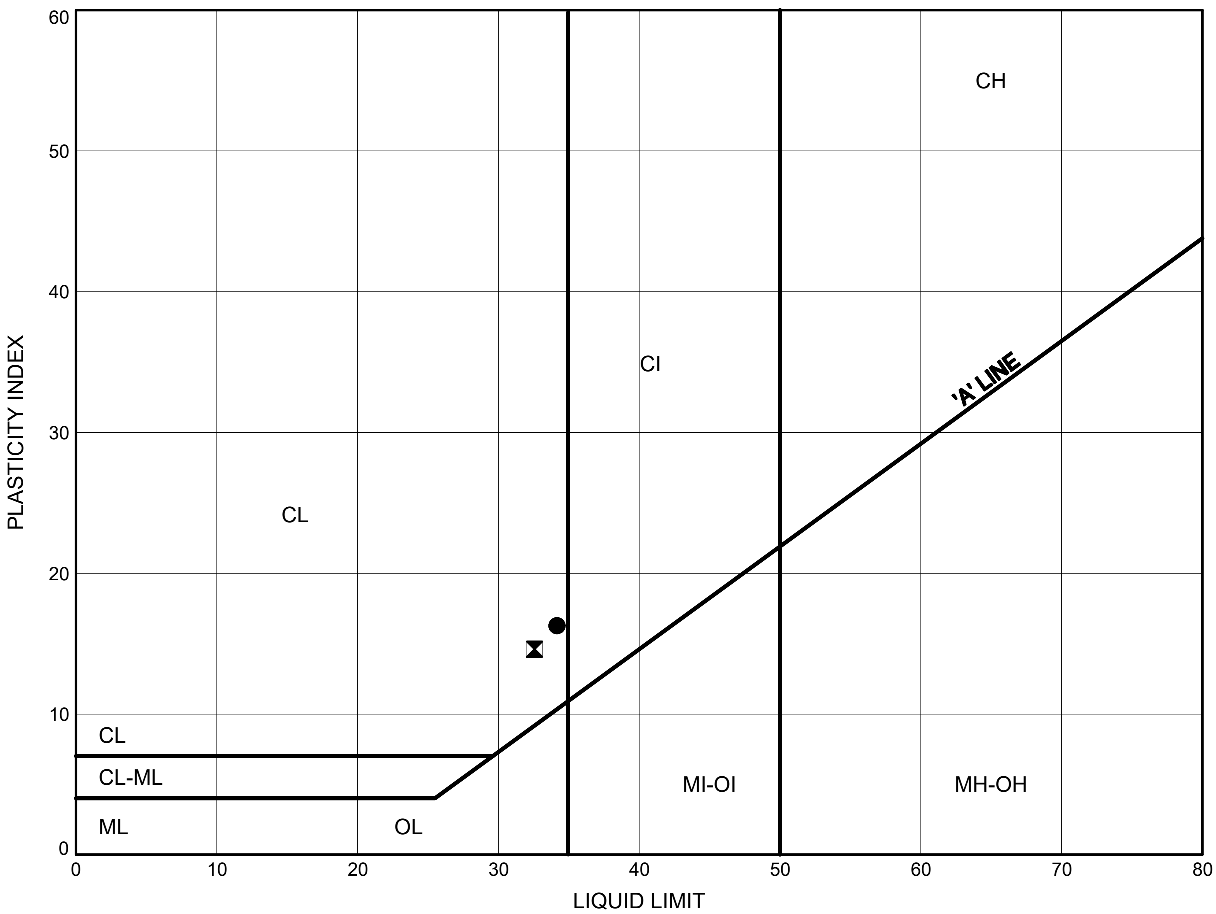
Chkd. MH

Etobicoke Creek

ATTERBERG LIMITS TEST RESULTS

FIGURE B6

Silty CLAY FILL



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	EC 16-01	1.07	153.93
⧗	EC 16-02	2.59	152.41

THURBALT MTO-12669.GPJ 10/12/16

Date October 2016

GWP# 2147-10-00

THURBER

Prep'd AN

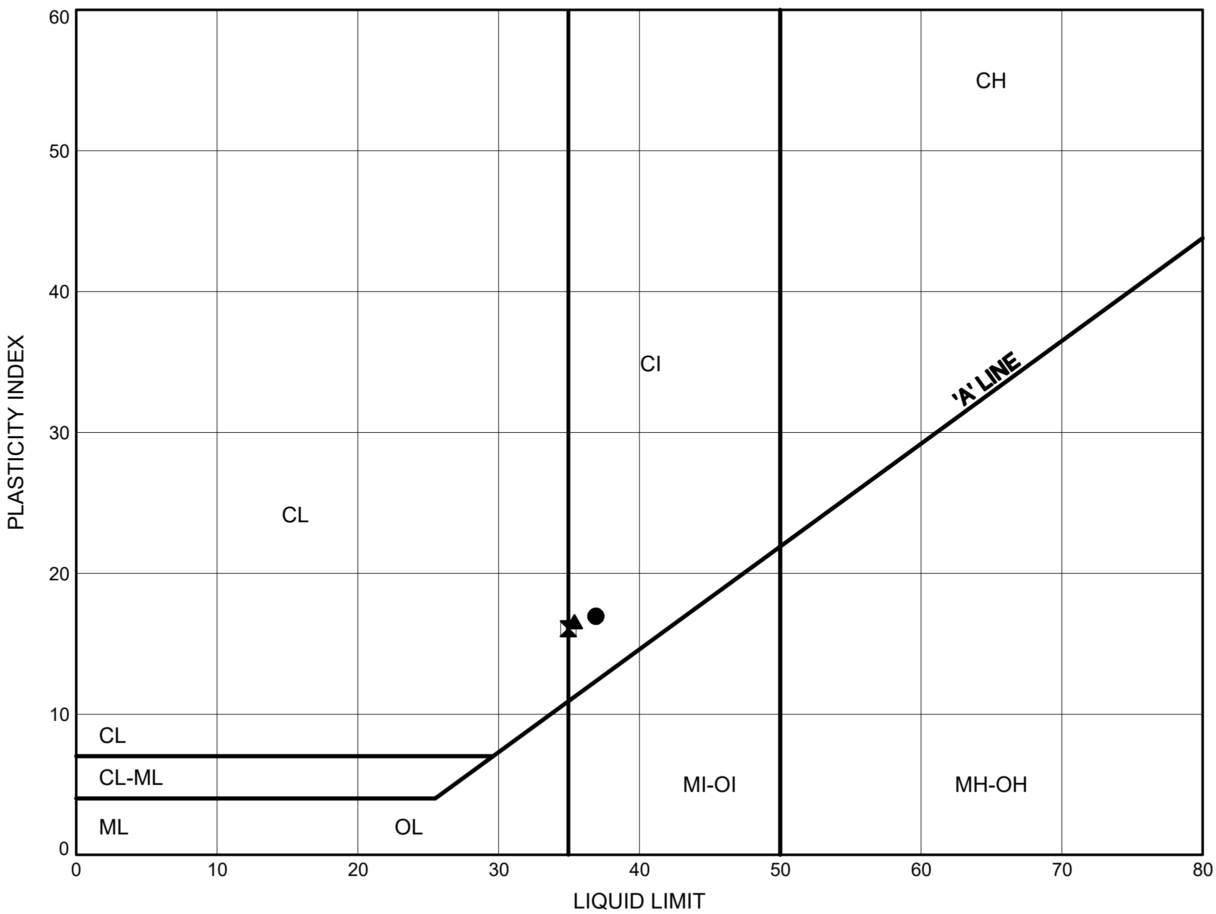
Chkd. MH

Etobicoke Creek

ATTERBERG LIMITS TEST RESULTS

FIGURE B7

Silty CLAY



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	CA 16-02	4.88	150.12
⧗	CA 16-02	7.92	147.08
▲	EC 16-02	9.45	145.55

THURBALT MTO-12669.GPJ 10/12/16

Date October 2016

GWP# 2147-10-00

THURBER

Prep'd AN

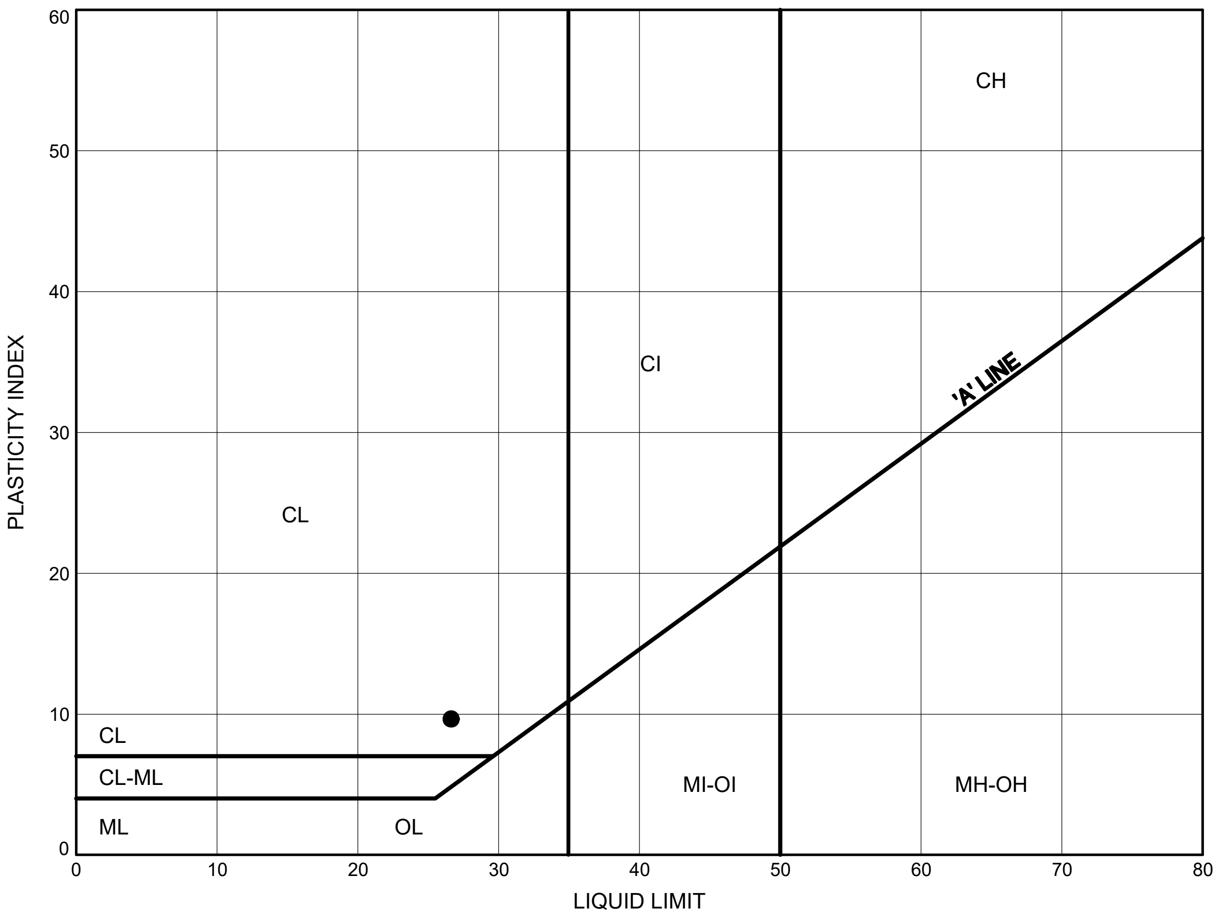
Chkd. MH

Construction Access

ATTERBERG LIMITS TEST RESULTS

FIGURE B8

Clayey SILT TILL



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	CA 16-01	3.35	151.65

Date October 2016

GWP# 2147-10-00

THURBER

Prep'd AN

Chkd. MH

THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

Job No :	12669	**Client :**	AECOM
		Date Drilled :	26-Aug-16
Project Name :	HWY 401 Etobicoke Creek	**Date Tested :**	28-Aug-16
Core Size :	NQ **BH No :** BH EC 16-01	**Tester :**	BT

Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (MPa)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	1	17.1	A	29.5	46.6	47.5	244.9	Limestone	Very Strong
2	1	17.7	D	6.4	46.9	38.1	64.4	Limestone	Strong
3	3	19.6	A	27.2	47.1	55.4	198.7	Limestone	Very Strong

* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1
Long pieces of core can be tested diametrically to produce suitable lengths for axial testing
* Diametral Test should have 0.7 x D on either side of test point.

Last Modified: August 15, 2013

THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

Job No :	12669	**Client :**	AECOM
		Date Drilled :	23-Aug-16
Project Name :	HWY 401 Etobicoke Creek	**Date Tested :**	24-Aug-16
Core Size :	NQ **BH No :** BH EC 16-02	**Tester :**	CG

Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (MPa)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	3	19.4	A	2.9	45.7	41.3	26.8	Shale (grey)	Medium Strong
2	4	20.0	A	1.6	45.7	51.8	12.4	Shale (grey)	Weak
3	4	20.3	A	2.7	45.7	47.5	22.9	Shale (grey)	Weak
4	4	20.7	A	2.8	45.7	40.0	27.3	Shale (grey)	Medium Strong

* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1
Long pieces of core can be tested diametrically to produce suitable lengths for axial testing
* Diametral Test should have 0.7 x D on either side of test point.

Last Modified: August 15, 2013

THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

Job No :	12669	**Client :**	AECOM
		Date Drilled :	13-Sep-16
Project Name :	HWY 401 Etobicoke Creek	**Date Tested :**	20-Sep-16
Core Size :	HQ **BH No :** BH EC 16-03	**Tester :**	BT

Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (MPa)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	1	5.9	A	29.3	63.3	61.4	157.2	Limestone	Very Strong
2	1	6.6	A	5.7	62.9	70.8	27.6	Shale (grey)	Medium Strong
3	2	7.6	A	2.5	63.2	49.7	15.9	Shale (grey)	Weak
4	2	8.0	A	1.3	63.0	54.8	7.8	Shale (grey)	Weak
5	3	8.3	A	3.2	63.2	48.9	20.5	Shale (grey)	Weak
6	3	8.6	A	8.4	63.1	62.0	44.8	Shale (grey)	Medium Strong

* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1
Long pieces of core can be tested diametrically to produce suitable lengths for axial testing
* Diametral Test should have 0.7 x D on either side of test point.

Last Modified: August 15, 2013

THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

Job No :	12669	**Client :**	AECOM
		Date Drilled :	12-Sep-16
Project Name :	HWY 401 Etobicoke Creek	**Date Tested :**	20-Sep-16
Core Size :	HQ **BH No :** BH EC 16-04	**Tester :**	BT

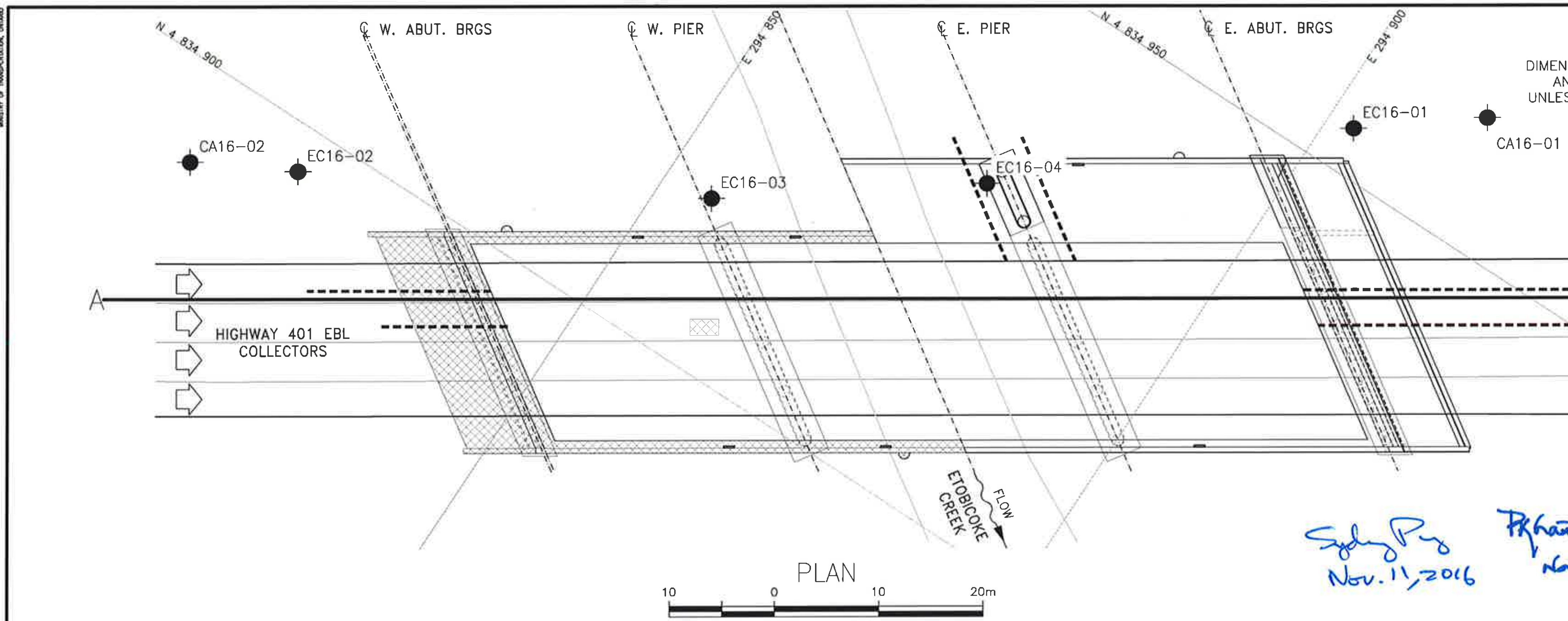
Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (MPa)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	1	5.4	A	1.9	63.2	61.6	10.1	Shale (grey)	Weak
2	1	6.0	A	3.7	63.1	58.4	20.7	Shale (grey)	Weak
3	1	6.4	A	3.2	63.1	60.3	17.2	Shale (grey)	Weak
4	2	7.0	A	3.3	63.0	66.1	17.0	Shale (grey)	Weak
5	2	7.2	A	0.3	63.0	59.4	1.7	Shale (grey)	Very Weak
6	2	7.7	A	5.0	63.0	70.3	24.3	Shale (grey)	Weak

* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1
Long pieces of core can be tested diametrically to produce suitable lengths for axial testing
* Diametral Test should have 0.7 x D on either side of test point.

Last Modified: August 15, 2013

Appendix C

Drawings titled "Borehole Locations and Soil Strata"



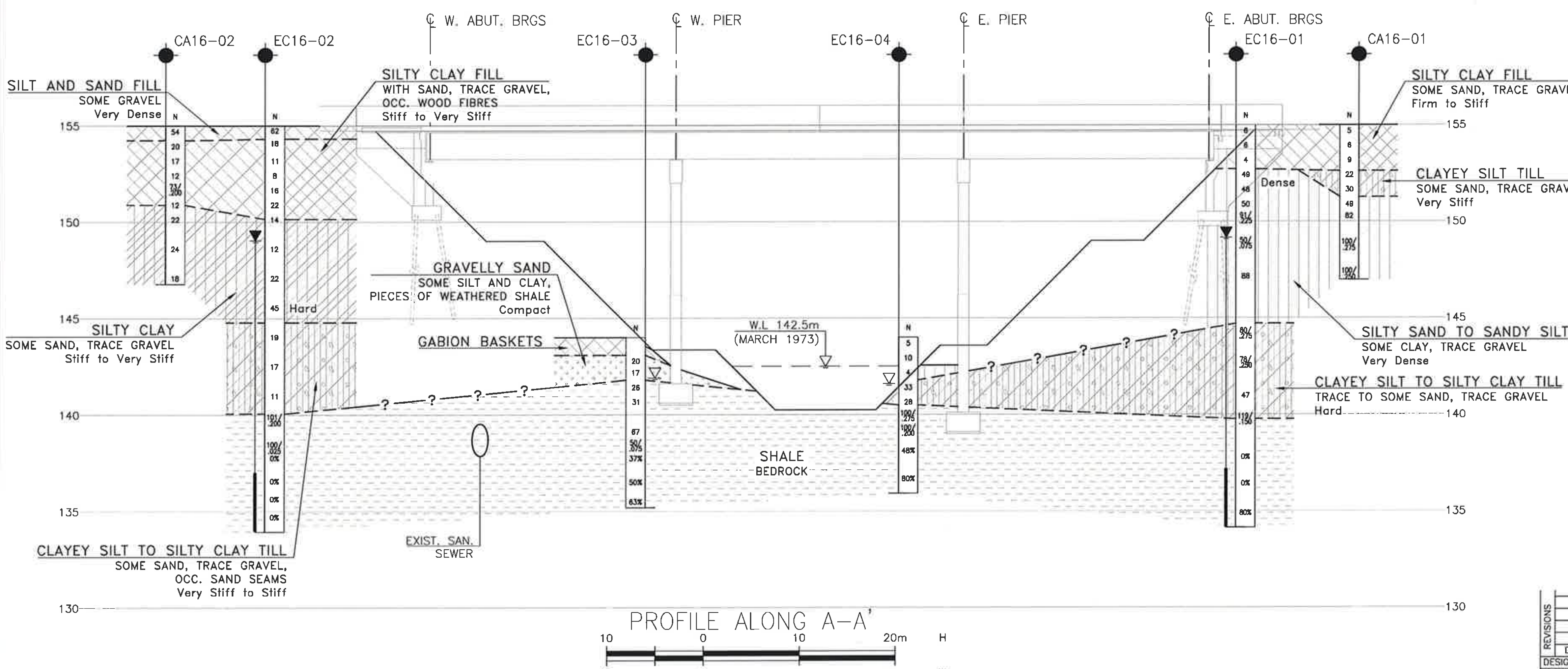
PLAN

Sydney Pry Nov. 11, 2016

PKhanna Nov 11/16

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN



PROFILE ALONG A–A'

CONT No

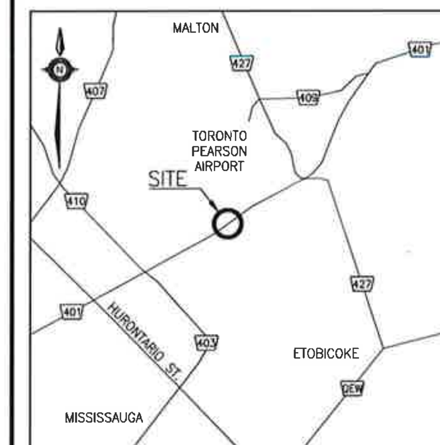
GWP No 2147-10-00

HIGHWAY 401
ETOBICOKE CREEK BRIDGE
REHABILITATION
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

AECOM

THURBER ENGINEERING LTD.



KEYPLAN

LEGEND

	Borehole
	Borehole and Cone
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
	Water Level
	Head Artesian Water
	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
CA16-01	155.0	4 834 962.2	294 912.8
CA16-02	155.0	4 834 891.0	294 811.4
EC16-01	155.0	4 834 954.4	294 902.6
EC16-02	155.0	4 834 895.8	294 820.5
EC16-03	144.0	4 834 915.2	294 854.9
EC16-04	144.0	4 834 930.8	294 876.2

-NOTES-

1) The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
2) This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

GEOCRES No.

REVISIONS	DATE	BY	DESCRIPTION

DESIGN MH	CHK MH	CODE	LOAD	DATE OCT 2016
DRAWN AN	CHK SKP	SITE	STRUCT	DWG 1

MINISTRY OF TRANSPORTATION, ONTARIO

FILENAME: H:\Drafting\12000\12669\ED12669-PLPR-Hwy401 EtobicokeCreek.dwg
PLOTDATE: 10/12/2016 11:18 AM

Appendix D

Site Photographs

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Photos 1 and 2: Existing EB collector bridge (left hand side) and EB express bridge (right hand side), west side of Etobicoke Creek; looking west.

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Photo 3: Existing EB collector bridge (right hand side), east side of Etobicoke Creek; looking east.

Photo 4: Gabion baskets at the lower portion of the west forward slopes between the existing EB collector bridge and EB express bridge, west side of Etobicoke Creek; looking south.

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Appendix E

Comparison of Foundation Alternatives

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COMPARISON OF FOUNDATION ALTERNATIVES

Spread Footings on Bedrock	Driven Steel H-Pile to Bedrock	Augered Steel H-Pile socketed into Bedrock	Augered Caissons socketed into Bedrock
Advantages: i. Ease of construction. ii. Lower cost than deep foundations. ***Disadvantages:*** i. Relatively large excavations required (cofferdams required at the piers). ii. Dewatering will be required; extent will depend on depth of excavation and groundwater level at time of construction. iii. May increase requirements for roadway protection at the abutments.	***Advantages:*** i. Ease of construction. ii. Higher vertical resistance than spread footings at abutments. ***Disadvantages:*** i. Higher unit cost than footings. ii. Cobbles and boulders may be encountered in glacially derived soils that could impede pile penetration to required depths. iii. Vibration due to pile driving could have adverse effects on existing bridges, sewer line, and any other adjacent utilities.	***Advantages:*** i. Minimize vibration impact on adjacent utilities and structures. ii. Higher vertical resistance than spread footings at abutments. iii. Same vertical resistance as driven H-piles to bedrock. iv. Higher lateral resistance than driven H-piles due to larger diameter. ***Disadvantages:*** i. Steel liners will be required during installation to minimize sidewall sloughing and water seepage. ii. Tremie concrete may need to be used.	***Advantages:*** i. Minimize vibration impact on adjacent utilities and structures. ii. Higher lateral resistance is available due to larger diameter. iii. Less number of caissons is required for each foundation element than if steel piles were used. ***Disadvantages:*** i. Higher unit cost than driven piles. ii. Steel liners will be required during installation to minimize sidewall sloughing and water seepage. ii. Tremie concrete may need to be used. iii. Potential basal instability if water-bearing soils are exposed at the base.
NOT RECOMMENDED FOR WEST ABUTMENT	**NOT RECOMMENDED FOR WEST ABUTMENT**	**RECOMMENDED FOR WEST ABUTMENT**	**FEASIBLE FOR WEST ABUTMENT**
NOT RECOMMENDED FOR EAST ABUTMENT	**FEASIBLE FOR EAST ABUTMENT**	**FEASIBLE FOR EAST ABUTMENT**	**FEASIBLE FOR EAST ABUTMENT**
RECOMMENDED FOR PIERS	**NOT FEASIBLE FOR PIERS**	**NOT FEASIBLE FOR PIERS**	**NOT FEASIBLE FOR PIERS**

THURBER

Appendix F

List of OPSS and Suggested Wordings for NSSPs

1. List of OPSS Documents Referenced in this Report

- OPSS.PROV 903
- OPSS.PROV 206
- OPSS.PROV 804
- OPSS.PROV 501
- OPSS.PROV 539
- OPSS 902
- OPSS.PROV 1010
- OPSD 3102.100
- OPSD 208.010

2. Suggested Text for NSSP on "Footing Construction at Piers"

All footing construction procedures should follow the guidelines provided in OPSS 902.

The bases of the foundation excavations must be inspected by a geotechnical engineer to confirm that the exposed surface conforms to the design requirements and has been adequately prepared to receive concrete. Any loose or shattered rock must be removed and the footing be founded on undisturbed shale bedrock or mass concrete on bedrock.

Shale is prone to softening upon exposure to water and air. Mass concrete or working (mud) slab should be placed within 24 hours following completion of excavation to prevent deterioration of the shale. The working slab should be at least 100 mm thick and formed with the same class of concrete as that of the footings. An allowance should be made for sub-excavation to remove disturbed shale and unsuitable materials, amongst other reasons, from below the design founding level, the founding surface should be re-established using mass concrete fill of the same class of concrete as that used for the footing.

All pier construction work should be carried out in the dry. This can be achieved by carrying out the work within a cofferdam supplemented by sump pumping.

3. Suggested Text for NSSP on "Augered Pile Installation at Abutments"

Augered pile installation should be in general accordance with clauses for caissons in OPSS.PROV 903. The pre-drilled holes for forming the pile socket should have a nominal diameter of 600 mm to be able to accommodate an HP 310 x 110 pile.

The augered holes are expected to extend through existing embankment fill, and deposits of cohesionless and cohesive soils containing cobbles, boulders and shale fragments, to found within the weathered shale. The installation (augering) equipment must be capable of dislodging and removing any obstructions such as cobbles, boulders, shale/limestone slabs and to penetrate very dense/hard layers within the glacial till. Soil sloughing and water seepage will occur in unsupported holes primarily from the fill and water-bearing sands and silts. Construction of augered piles will require the use of temporary steel liners to support the sidewalls and to provide seepage cut-off where required. The liners must not be installed by vibratory means at the west abutment to avoid adverse effects on the existing sewer.

After each rock socket is drilled, cleaned and inspected, and subsequent to the seating of an H-pile in the socket, the annular space between the pile and the pre-drilled hole should be filled with 30 MPa concrete. Any accumulated water should be pumped out from the hole prior to placing concrete. Concrete should be placed with a minimum delay after the pile is set in place. The tremie technique should be employed to place concrete inside the hole.

4. Suggested Text for NSSP on "Cofferdam and Dewatering at Piers"

Construction for both piers shall be carried out within dewatered cofferdams (enclosures). The Contractor shall retain an experienced Professional Engineer to design the cofferdams and a dewatering specialist to design and implement an effective dewatering system. Construction of the pier footings shall be carried out in the dry.

The piers will be constructed adjacent to the creek flow channel. Excavations for footing construction will extend below the creek water level. It will, therefore, be necessary to construct a cofferdam (enclosure) capable of excluding the creek flow and supporting the native soils through which the excavation must be formed. One type of cofferdam that may be considered consists of interlocking steel sheet piles. For providing a partial groundwater cut-off below the base of the excavation, the sheet piles should be installed within the clayey silt to silty clay till and/or the underlying weathered shale. Allowance should be made to pre-drilling some pilot holes along the cofferdam alignment prior to sheetpile installation. The sheetpiles must be supplemented by sump pumping within the enclosure.

5. Suggested Text for NSSP on "Management of Construction Water"

Water discharged from dewatering operations or displaced during concrete placement may not be suitable for direct discharge to the Etobicoke Creek. The Contractor shall provide the necessary means for treatment of the water to the satisfaction of TRCA, MOE, MNR or other agencies having jurisdiction prior to discharge into the creek. Prior to commencing the construction operations, the Contractor shall submit its proposed methodology on construction water treatment to the Contract Administrator for distribution to all concerned parties for review and comments.

6. Suggested Text for NSSP on "Vibration and Settlement Monitoring"

The Contractor shall monitor vibration levels at ground surface where pile augering will be carried out for the new west abutment foundations. The vibration monitoring equipment shall be placed on the ground above the existing 2 m diameter sanitary trunk sewer alignment in close proximity to the new abutment. The monitoring locations should be strategically selected to characterize vibration propagation. Vibration levels due to pile augering (including temporary liner installation where required) are measured in peak particle velocity (ppv). The monitoring criteria that have been established for this project are as follows:

a) For a vibration frequency of 30 Hz or less (typical of construction activities including augering holes), a review ppv level of 10 mm/sec and an alert ppv level of 12 mm/sec shall be used.

b) Survey markers consisting of a fluorescent paint park on the top surface of any maintenance hole adjacent to the existing west abutment and up to four (4) surveyors' pins hammered into the ground and slope at selected locations along the centreline of the section of sewer alignment closest to the new west abutment. The vertical and lateral positions of these points must be surveyed relative to known, fixed reference datum points on a regular basis. The suggested monitoring frequency is:

   1) Three readings on separate days prior to construction to establish a baseline;
   2) Twice daily while any foundation construction is in progress;
   3) Daily for one week after completion of foundation construction; and
   4) Twice weekly for the following week.

c) The vertical and horizontal precision readings should be ± 2 mm. All readings must be reported to the Contract Administrator within 24 hours and immediately if there is any movement. The Contract Administrator must then advise the owner/operator of the trunk sewer and MTO as necessary. It is noted that the augered pile installation operation is not expected to cause movements of the maintenance hole(s) and the surrounding ground surface.

d) Vibration monitoring shall be carried out by the Contractor, or his representatives, using vibration monitoring equipment such as the Instantel Blast Mate Monitors, or equal. These monitors shall be deployed at selected locations along the sewer alignment.

e) Any exceedance of the review or alert levels must be reported to the Contract Administrator immediately. Should the vibration level reach or exceed the review level as specified in Clause a), but less than the alert level, and provided that other forms of distress are not evident, the augered pile installation operations may proceed with caution and in conjunction with precautionary measures. If the vibration monitoring readings are not acceptable, the Contractor must alter the caisson installation procedures until the measured vibrations are within acceptable limits.

f) Should there be any sign of potential adverse effect on the maintenance hole(s) and surrounding ground as a result of visual inspections, or if the measured vibration level approaches the alert level, or if there is a change in the elevations of the survey marks that indicate settlement or the development of a trend of settlement, the Contractor shall immediately stop the augering work. The Contract Administrator will then review the situation and in conjunction with the Contractor, come up with a plan for re-commencing any augering operation in the area.

g) All vibration and settlement monitoring results must be submitted to the Contract Administrator at the end of each day.