THURBER ENGINEERING LTD.

FOUNDATION INVESTIGATION AND DESIGN REPORT
BOWEN ROAD UNDERPASS REPLACEMENT
QEW/BOWEN ROAD INTERCHANGE IMPROVEMENT
FORT ERIE, ONTARIO
GWP No. 2482-04-00

GEOCRES No.: 30L15-15

Report to

AECOM

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January 20, 2017
File: 14515
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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation carried out for the proposed replacement of the Queen Elizabeth Way (QEW) Bowen Road underpass structure in the Town of Fort Erie, Niagara Region, Ontario. The bridge replacement is a part of the QEW Bowen Road interchange improvement project.

The purpose of the investigation was to explore the subsurface conditions near the bridge foundation elements and along the high fill alignments on either side of the proposed bridge and, based on the data obtained, to provide borehole locations and soil strata drawings, records of boreholes, stratigraphic profile, 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 site. The titles of these reports are listed as follows:

- Thurber Engineering Ltd. report titled "Preliminary Foundation Investigation and Design Report, Bowen Road I/C Underpass, Queen Elizabeth Way, Fort Erie, Ontario", GEOCRES No. 30L15-13, dated March 11, 2009 (Reference 1).
- Thurber Engineering Ltd. report titled "Foundation Investigation and Design Report, Queen Elizabeth Way Underpass at Bowen Road, Fort Erie, Ontario", GWP No. 2482-04-00, dated January 7, 2015 (Reference 2).

A memorandum containing preliminary foundation recommendations dated September 16, 2016 was issued. Relevant contents in that memo have also been incorporated in this report.

Thurber was retained by AECOM to carry out the foundation investigation at this site under the MTO Assignment Number 2012-E-0007.

2 SITE DESCRIPTION

The existing Bowen Road underpass structure carries the two-lane Bowen Road over the four-lane QEW. The existing structure is a 45 m long, double span rigid frame bridge that runs in an east-west direction, intersecting the QEW at a 37° skew. The immediate approaches to the bridge are in the order of 5 to 6 m in height.

The new bridge centreline will be approximately 7.0 m to 10.0 m south of the existing bridge centreline.

The general surroundings of the new interchange area are largely rural. There are grass covered fields with some shrubs and small trees along the west side of QEW. The land is used for agricultural purposes immediately to the west of QEW, while there is an industrial yard immediately bordering the existing S-EW ramp on the east side.

It is understood that underground services are located on the north and south sides of the existing bridge. These utilities include an Enbridge gas line, a Bell underground line and a hydro line.

From published geological information, the site is situated within the physiographic region known as the Haldimand Clay Plain. This area is typically characterized by glacio-lacustrine silts and clays interbedded with glacial tills at some locations. Dolostone and limestone bedrock of the Paleozoic Era is present across the site at relatively shallow depths.

3 SITE INVESTIGATION AND FIELD TESTING

A preliminary foundation investigation was carried out at the location of the proposed structure in November 2008 (Reference 1). That investigation consisted of drilling and sampling a total of five boreholes (numbered 08-01 to 08-05) terminated upon refusal on probable bedrock, at depths ranging from 1.2 m to 3.1 m (Elevations 180.9 to 184.5). The boreholes were drilled along the Bowen Road alignment near the approaches, abutments and the pier. The Record of Borehole sheets from the previous investigation are attached in Appendix D.

The current site investigation and field testing for the proposed Bowen Road underpass and its approaches was carried out on August 29 and 30, 2016, and consisted of drilling and sampling a total of seven boreholes identified as 16-01 to 16-07. Boreholes 16-02 and 16-04 were drilled near the west and east abutments, and terminated within dolostone bedrock at depths of 10.8 m and 11.6m (Elevations 180.2 and 179.3 m), respectively. Borehole 16-03 was drilled near the pier and also terminated within dolostone bedrock at 5.6 m depth (Elevation 180.0 m). Boreholes 16-01 and 16-05 to 16-07 were drilled at the west and east approaches, and near the high fill areas, to depths ranging from 3.7 m to 7.8 m (Elevations 179.9 to 184.4 m). Rock coring was carried out in Boreholes 16-02 to 16-04 and rock cores of 3.3 m to 4.1 m in length were recovered.

The approximate locations of the current boreholes are shown on the Borehole Locations and Soil Strata Drawing included in Appendix C.

The borehole locations were established in the field by Thurber using a GPS unit. Utility clearance was obtained at all borehole locations prior to drilling. The northing and easting co-ordinates and ground surface elevations of the completed boreholes were provided by AECOM.

A track mounted B57 drill rig was used to conduct the drilling, sampling and in-situ testing. Hollow stem augers were used to advance the boreholes through soils to reach auger refusal on probable bedrock. Soil samples were obtained at selected intervals using a split spoon sampler in conjunction with the Standard Penetration Test (SPT). Boreholes 16-02 to 16-04 were further advanced into bedrock by HQ size rotary coring techniques to recover core samples. All rock cores were logged, and properties including the Total Core Recovery (TCR), Rock Quality Designation (RQD) and the Fracture Indices (FI) were determined where applicable.

The drilling and sampling operations were supervised on a full time basis by a member of Thurber's technical staff. The supervisor logged the boreholes and processed the recovered soil and rock samples for transport to Thurber's laboratory for further examination and testing.

Groundwater conditions were observed in the open boreholes during and upon completion of the drilling operations. Standpipe piezometers each consisting of a 50 mm diameter Schedule 40 PVC pipe with a 1.5 m to 3.0 m long slotted screen were installed within a column of filter sand in Boreholes 16-02 and 16-04 to permit longer term groundwater level monitoring. All boreholes were backfilled in general accordance with O.Reg. 903.

The completion details of the piezometers and boreholes are summarized in Table 3.1. Once the investigation is completed, the piezometers will be decommissioned in accordance with O.Reg. 903.

Table 3.1 – Piezometer and Borehole Completion Details

Foundation Unit	Borehole Number	Piezometer Tip Depth / Elevation (m)	Completion Details
West Approach	16-01	None installed	Backfilled with bentonite holeplug and auger cuttings to 0.1 m, then concrete to ground surface.
West Abutment	16-02	7.0/184.0	Backfilled with bentonite holeplug from 10.8 m to 7.0 m, filter sand from 7.0 m to 4.9 m, bentonite holeplug from 4.9 m to 0.3 m, sand from 0.3 m to 0.15 m, then concrete from 0.15 m to ground surface.
Pier	16-03	None installed	Backfilled with bentonite holeplug to ground surface.
East Abutment	16-04	11.6/179.3	Filter sand from 11.6 m to 8.6 m, bentonite holeplug from 8.6 m to 0.3 m, sand from 0.3 m to 0.15 m, then concrete from 0.15 m to ground surface.
East Approach (15m from East Abutment)	16-05	None installed	Backfilled with bentonite holeplug and auger cuttings to 0.1 m, then concrete to ground surface.
East Approach (55m from East Abutment)	16-06	None installed	Backfilled with bentonite holeplug and auger cuttings to 0.1 m, then concrete to ground surface.
East Approach (96 m from East Abutment)	16-07	None installed	Backfilled with bentonite holeplug and auger cuttings to ground surface.

4 LABORATORY TESTING

All recovered soil samples were subjected to visual identification and to natural moisture content determination. The results of this testing are shown on the Record of Borehole sheets in Appendix A. At least 25% of soil samples were subjected to grain size distribution analysis and/or Atterberg Limits tests where applicable. The results of this testing program are presented on the Record of Borehole sheets in Appendix A and on the figures in Appendix B.

Point Load Tests (PLT) were carried out on selected rock core samples. The rock cores were logged and the results are shown in Appendix B and summarized on the Record of Boreholes sheets in Appendix A.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets for the current and previous investigation in Appendices A and D, respectively. Details of the encountered soil and rock stratigraphy encountered during the present investigation are presented in these records and on the "Borehole Locations and Soil Strata" drawings in Appendix C. General description of the stratigraphy is given in the following paragraphs. The factual information established at the borehole locations governs any interpretation of site conditions.

The soil stratigraphy encountered at the borehole locations typically consists of a pavement structure (asphalt overlying gravelly sand fill) at the road and topsoil at boreholes drilled off the road. The underlying approach embankment consists of silty clay fill. Below and beyond the fill, layers of native firm to hard silty clay and silty clay till were contacted at all locations. The above soil strata are underlain by grey shaley dolostone bedrock across the site. The groundwater level is typically within 5.0 m and 10.0 m below existing ground surface, the at the west and east abutments, respectively.

5.1. Topsoil

A layer of topsoil 25 mm thick was encountered at ground surface in Boreholes 16-03 and 16-07. The topsoil thickness may vary between and beyond the borehole locations.

5.2. Pavement Structure

Pavement structure consisting of asphalt overlying granular fill materials (road base) was encountered in Boreholes 16-01, 16-02, and 16-04 to 16-06, drilled along the Bowen Road platform.

The thickness of the asphalt at these borehole locations was 100 mm. The granular base consisted of gravelly sand and ranged from 200 mm to 600 mm in thickness.

The SPT 'N' values of the granular base typically ranged from 14 to 28 blows per 0.3 m of penetration indicating a compact state. The moisture content of the granular base ranged from 2% to 6%.

5.3. Embankment Fill

Brown to dark brown silty clay fill containing some sand and trace gravel, was contacted below the granular base in Boreholes 16-01, 16-02, and 16-04 to 16-06. The thickness of the silty clay fill varied from 2.5 m to 4.1 m. The depth to the base of the silty clay fill ranged from 3.0 m to 4.6 m (Elevations 185.6 m to 187.4 m).

SPT 'N' values measured in the silty clay fill ranged from 4 to 15 blows per 0.3 m of penetration indicating a typically firm to stiff consistency. The moisture content of the fill ranged from 14% to 25%.

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 Figure B1 of Appendix B. The results of the grain size distribution analyses are summarized below:

Soil Particle	Percentage (%)
Gravel	0 to 3
Sand	17 to 23
Silt	31 to 36
Clay	43 to 50

Atterberg Limits test results are presented in Figure B5 of Appendix B. The results of Atterberg Limits testing are summarized below:

Index Property	Percentage (%)
Plasticity Index	23 to 27
Liquid Limit	41 to 46

The results of the Atterberg Limits testing indicate that the silty clay fill of medium plasticity with a group symbol CI.

5.4. Silty Clay

Deposits of native brown to grey silty clay containing trace to some sand and trace gravel was encountered below the topsoil or silty clay fill in all boreholes, except in Boreholes 16-01 and 16-07. The thickness of the silty clay ranged from 0.6 m to 2.1 m.

The depth to the base of the silty clay varied from 4.1 m to 6.7 m (Elevations 184.3 m to 185.4m) in Boreholes 16-02, 16-04, 16-05 and 16-06. The depth to the base of the silty clay was 0.7 m (Elevation 184.9 m) in Borehole 16-03.

The SPT ‘N’ values of the silty clay typically ranged from 3 to 8 blows per 0.3 m of penetration indicating a soft to firm consistency. In Borehole 16-02 and 16-05, SPT ‘N’ values of 31 and 50 blows per 0.3 m of penetration were measured within the lower part of the deposit near the bedrock surface and immediately above a layer of very dense gravelly sand, respectively. The moisture content of the silty clay typically ranged from 16% to 26%.

The results of grain size distribution analyses and Atterberg Limits testing carried out on selected samples of the silty clay 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	0 to 4
Sand	10 to 19
Silt	34 to 38
Clay	40 to 56

Atterberg Limits test results are presented in Figure B6 of Appendix B. The results of Atterberg Limits testing are summarized below:

Index Property	Percentage (%)
Plasticity Index	28 to 30
Liquid Limit	47 to 49

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

5.5. Gravelly Sand

A 1.1 m thick layer of gravelly sand containing some silt was contacted below the silty clay at 5.0 m depth in Borehole 16-05. Partial blow counts within an SPT attempt, straddling the silty clay and the gravelly sand, indicate that the latter is in a very dense state.

5.6. Silty Clay to Clayey Silt Till

Native brown silty clay to clayey silt till containing trace to some sand and trace to some gravel was encountered below the fill or native silty clay in Boreholes 16-01, 16-03, 16-04, 16-05 and 16-06. In Borehole 16-07, the silty clay till was contacted immediately below the topsoil. The thickness of the glacial till ranged from 1.6 m to 3.6 m. The underside of the till lies immediately above the surface of the proven or inferred bedrock. Where encountered, the depth to the base of the till ranged from 2.3 m to 8.1 m (Elevations 179.9 m to 184.4 m).

In Boreholes 16-01, 16-03, 16-04, 16-05 and 16-06, measured SPT ‘N’ values of the silty clay to clayey silt till typically ranged from 14 to 47 blows per 0.3 m of penetration indicating a firm to hard consistency. In Borehole 16-07, an SPT ‘N’ value of 5 blows per 0.3 m of penetration was measured in the upper 0.5 m indicating a firm consistency. Below the surface, higher ‘N’ values between 63 and 95 blows per 0.3 m penetration were measured indicating a hard consistency. Immediately above the till-bedrock interface in Boreholes 16-01, 16-05 and 16-06, ‘N’ values greater than 50 blows for less than 0.3 m penetration were recorded. These higher values may be attributed to the presence of bedrock fragments, cobbles or boulders. The moisture content of the silty clay to clayey silt till ranged from 8% to 18%.

The results of grain size distribution analyses and Atterberg Limits testing carried out on selected samples of the silty clay till to clayey silt till are presented on the Record of Borehole Sheets included in Appendix A and on Figures B3 to B4 of Appendix B. The results of the grain size distribution analyses are summarized below:

Soil Particle	Percentage (%)
Gravel	0 to 14
Sand	16 to 45
Silt	32 to 43
Clay	12 to 41

Atterberg Limits test results are presented in Figure B7 of Appendix B. The results of Atterberg Limits testing are summarized below:

Index Property	Percentage (%)
Plasticity Index	8 to 25
Liquid Limit	19 to 43

The results of the Atterberg Limits testing indicate that the silty clay till is of low plasticity with a group symbol CL.

5.7. Dolostone Bedrock

Shaley dolostone bedrock was encountered below the silty clay and silty clay to clayey silt till, and proven by coring in Boreholes 16-02 to 16-04. Auger refusal on inferred bedrock was encountered in the remaining boreholes. The depths and elevations of the proven and inferred bedrock surface are summarized in Table 5.1, which includes bedrock information from the current investigation and a previous investigation conducted in 2008 (Reference 1).

Table 5.1 – Bedrock Depths and Elevations

Foundation Unit	Borehole Number	Depth to Proven or Inferred Bedrock (m)	Top of Proven or Inferred Bedrock Elevation (m)
West Approach	16-01	6.2	184.4
	08-01	1.2	184.5
West Abutment	16-02*	6.7*	184.3*
	08-02	1.5	183.7
Pier	16-03*	2.3*	183.3*
	08-03	1.9	183.1
East Abutment	16-04*	8.1*	182.8*
	08-04	3.1	180.9
East Approach (15 m from East Abutment)	16-05	7.8	182.6
East Approach	08-05	2.2	182.8
East Approach (55 m from East Abutment)	16-06	6.4	182.2
East Approach (96 m from East Abutment)	16-07	3.7	179.9

* Bedrock proven by coring

The above values indicate that the bedrock surface generally slopes in a west to east direction across the site.

The bedrock is a grey, thinly bedded and very strong to strong shaley dolostone with shale interbeds. Broken zones were noted in the recovered core samples at varying depths. The recovered rock cores were described as moderately to slightly weathered becoming fresh with depth.

Total Core Recovery (TCR) of the bedrock was 100% in all the core runs. The Rock Quality Designation (RQD) values typically ranged between 17% and 73%, indicating a very poor to fair rock quality. A value of 94% recorded for the third run of Borehole 16-03 indicate an excellent rock quality. The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, is typically less than 5, and occasionally greater than 5 where broken zones were encountered.

The unconfined compressive strength (UCS) of the shaley dolostone, estimated from point load tests conducted on intact rock cores, ranged from an average of 100 to 148 MPa per run, indicating a generally strong to very strong rock.

5.8. Groundwater Conditions

In the boreholes without coring, free standing water was not observed in the open hole upon completion of drilling. Standpipe piezometers were installed in Boreholes 16-02 and 16-04. The piezometric readings obtained on October 22, 2014 are presented in Table 5.2.

Table 5.2 – Groundwater Measurements

Foundation Unit	Borehole	Date	Groundwater		Comment
			Depth (m)	Elevation (m)	
West Abutment	16-02	September 26, 2016	5.1	185.9	piezometer
East Abutment	16-04	September 26, 2016	10.9	180.0	piezometer

These piezometers will continue to be monitored to establish stabilized readings.

The groundwater readings at this site are a short term observation. Seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater level may be at higher elevations after the spring snowmelt or after periods of heavy rainfall.

6 MISCELLANEOUS

Borehole co-ordinates and ground surface elevations were provided to Thurber by AECOM.

The drilling and sampling equipment was supplied and operated by Landshark Drilling of Brantford, Ontario. The field work was supervised on a full time basis by Mr. Omar Ali of Thurber.

Laboratory testing was carried out at Thurber’s MTO approved high complexity Toronto laboratory.

Overall supervision of the field program was conducted by Dr. Sydney Pang, P.Eng. Compilation of data and preparation of the report were carried out by Ms. Rocío Palomeque Reyna, P.Eng.

Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects, reviewed the report.

THURBER ENGINEERING LTD.

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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 QEW Bowen Road replacement bridge and its two approaches.

A preliminary General Arrangement (GA) drawing, dated September 2016 prepared by MTO and provided by AECOM indicates that the new underpass consists of a two-span structure supported by one pier and two abutments. This GA drawing illustrates that each proposed abutment is supported on two rows of H-piles driven to bedrock, and the pier is supported on spread footings founded on bedrock. The new bridge will be 73.0 m in length between abutment bearings and 11.6 m in width, and will intersect the QEW at a 40° skew. Both immediate approaches to the bridge and the high fills at the east embankment are up to the order of 8 m in height above the QEW. The Bowen Road Underpass structure will be designed to carry two lanes of traffic with two shoulders. The new bridge centreline will be in the order of 7 m to 10 m to the south of the existing bridge centreline. The finished grade of the realigned Bowen Road crossing will be up to the order of 3 m above the existing Bowen Road grade.

The Thurber September 2016 memo was prepared based on a previous GA drawing dated September 2008, which has since been superseded by the 2016 GA. Subsequent information also indicated that MTO concurred that the foundation option of driven H-piles to bedrock, as shown the 2016 GA, is not applicable due to shallow bedrock at this site.

Underground utilities are located on the north and south sides of the existing structure. It is understood that some utilities will be relocated for the new bridge construction. Recent information from MTO indicates that the Enbridge gas line currently located on the south side of Bowen Road is to be relocated further south by up to 50 m prior to commencement of construction.

The discussion and recommendations presented in this report are based on preliminary design information provided by AECOM to date, and the factual data obtained during the course of this investigation. A previous preliminary investigation carried out along the existing Bowen Road, documented in Reference 1, has been reviewed and relevant information has been incorporated in this report.

8 BRIDGE FOUNDATIONS

In general, the subsurface conditions at this site consists of topsoil or pavement structure (asphalt and granular base), and typically stiff to firm silty clay fill overlying firm to hard silty clay and silty clay till to clayey silt till. Dolostone bedrock underlies the above soil deposits. At the west approach and west abutment, bedrock was encountered at 6.2 m to 6.7 m depths (Elevations 184.4 m to 183.7 m) below the existing Bowen Road grade. Bedrock was encountered at 1.9 m to 2.3 m (Elevations 183.1 to 183.3 m) below the QEW median grade. At the east approach and east abutment, bedrock was encountered at 6.4 m to 8.1 m depths (Elevations 182.2 m to 182.8 m) below the existing Bowen Road grade.

8.1. Foundation Alternatives

Information from MTO indicates that the bridge has a skew of 40° and, therefore a semi-integral abutment design is selected instead of an integral abutment design.

Consideration was given to the following foundation types for the new abutments and pier:

- H-piles driven to bedrock
- Augered H-piles socketted into bedrock
- Augered pipe piles socketted into bedrock
- Augered caissons (drilled shafts) socketted into bedrock
- Spread footings on bedrock
- Spread footings on engineered fill and/or native soil.

A comparison of the foundation alternatives based on their respective advantages and disadvantages is included in Appendix E.

Abutments

From a foundations perspective, a semi-integral abutment design is feasible for this site as indicated on the preliminary 2016 GA drawing. The required pile length below the abutment stem should be determined by the structural designer for satisfying base fixity and/or other structural requirements. It is understood that a minimum 3 m of pile embedment is required for structural considerations.

The preliminary 2016 GA drawing indicates that the underside of the abutment stems is at approximate Elevations 185 m and 186 m at the west and east abutments, respectively. Based on the bedrock elevations established at the boreholes, there is insufficient depth to accommodate the minimum pile length. Driven piles are, therefore, not suitable for this site due to the shallow bedrock. Deep foundations including steel H-piles, steel pipe piles and augered caissons will therefore need to be socketted into bedrock by installing and grouting them within pre-drilled holes.

Construction of spread footings on bedrock or spread footings on engineered pad resting on bedrock requires large excavations within roadway protection systems and dewatering. Although technically feasible, the cost effectiveness of this option should be assessed especially for the east abutment where the bedrock elevation is lower i.e. deeper excavation.

Pier

From a foundations perspective, spread footings founded on bedrock is a feasible option as indicated on the preliminary 2016 GA drawing. Open excavation in conjunction with roadway protection will be required within the narrow QEW median.

The depth to bedrock is in the order of 2 m to 3 m below the QEW grade. As such, spread footings founded on engineered fill or native soils are not practical for the pier.

Another feasible foundation option at the pier is augered caissons (drilled shafts) which can be designed to be structurally connected to the superstructure without a cap. This would eliminate the need for open excavation in conjunction with roadway protection and better cope with space restriction at the narrow QEW median.

Driven steel H-piles or pipe piles are not suitable for the pier due to the shallow bedrock. Steel H-piles or pipe piles socketted within bedrock may be feasible only if there is sufficient space and if it is cost effective to construct the pile cap.

Recommended Foundations

From a foundations technical and constructability perspective, it is recommended that spread footings on engineered fill founded on bedrock be used for providing foundation support to the abutments. Alternatively, consideration should also be given to using augered steel H-piles or pipe piles socketted into bedrock. At the pier, both spread footings founded on bedrock and augered caissons socketted into bedrock are feasible foundation options.

The relative cost effectiveness between the options should be assessed; spread footings require excavation within roadway protection, whereas piles and caissons require coring and socketting into bedrock.

8.2. Steel H-Piles

The steel H-piles for supporting the abutments will need to be socketted into bedrock. 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. A pre-drilled hole for commonly used H-piles (e.g. HP 310 x 110) are typically 600mm in nominal diameter. The actual depth of sockets should be designed to satisfy structural requirements for pile embedment depths, and to provide the required lateral resistances and base fixity. It is recommended that a minimum 2 m deep socket in the dolostone bedrock be used.

The recommended design highest founding elevations are as follows:

Table 8.1 – Design Socketted Pile Tip Elevations (2 m long sockets)

Foundation Element	Borehole	Highest Pile Tip Elevation (m)
West abutment	16-02 (north) 08-02 (south)	182.3 181.7
East abutment	16-04 (north) 08-04 (south)	180.8 178.9

8.2.1 Axial Resistance

For steel H-piles grouted in rock sockets, the following axial design geotechnical resistances per pile may be used.

Table 8.2 – Design Pile Resistances

Pile Type	Factored Geotechnical Resistance at ULS (kN)
HP 310 x 110	2,000
HP 360 x 132	2,400

The geotechnical SLS condition does not govern pile design in rock.

It is noted that the theoretical factored geotechnical resistances are up to the order of 2,700 kN and 3,600 kN for HP 310 x 110 and HP 360 x 132, respectively. The values given in Table 8.2 above are customarily provided as per MTO structural considerations and directive. It is anticipated that the designer would carry out the foundation design based on the values in Table 8.2 and recommendations provided elsewhere in this report.

The structural resistance of the pile must be checked by the structural designer. The piles shall also be designed to provide the required lateral resistance and base fixity.

Downdrag on piles is not considered to be a design issue at this site.

8.2.2 Lateral Resistance

For pile lateral resistance design, soil-pile interaction analyses may be carried out using the coefficient of horizontal subgrade reaction values provided in Table 8.3 and in conjunction with the equations below.

The lateral resistance of a pile may be calculated using a value for the coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) as follows:

Cohesionless soils

k_s = $n_h . z / D$ (kN/m^3)

p_{ult} = $3 . \gamma . z . K_p$ (kPa)

where
p_{ult} = ultimate lateral resistance mobilized by a pile, kPa
z = depth of embedment of pile, m
D = augered pile diameter, m
n_h = coefficient related to soil density, kN/m^3
γ = total unit weight of fill, γ_T (above groundwater level), kN/m^3
γ = submerged unit weight of cohesionless soils, γ' (below groundwater level), kN/m^3
K_p = passive earth pressure coefficient

Cohesive Soils

k_s = $67\, C_u / D$ (kN/m^3)

p_{ult} = $9\, C_u$ (kPa)

where

C_u = undrained shear strength of native soil (kPa)

The above equations and recommended parameters may be used to analyze the interaction between a pile and the surrounding soil. The lateral pressures obtained from the analysis should not exceed the ultimate lateral resistance. Soil parameter values for lateral pile resistance are provided in Table 8.3 below.

Table 8.3 – Soil Parameters for Lateral Pile Resistance

Foundation Element	Elevation (m)	C_u (kPa)	Unit Weight (kN/m^3)	Soil Conditions
West Abutment	Underside of abutment to bedrock	75	20	Silty Clay to Silty Clay Till (very stiff to hard)
East Abutment	Underside of abutment to 183	75	19	Silty Clay to Silty Clay Till (stiff to hard)
	183 to bedrock	150	20	Silty Clay Till (hard)

The spring constant, K, for analysis may be obtained by the expression, $K = k_s \times d_z \times D$ (kN/m), where k_s is the coefficient of horizontal subgrade reaction (kN/m^3), D is the augered pile (socket) diameter (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, $P_{ult} = p_{ult} \times d_z \times D$. 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 the 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 by a reduction factor R as follows:

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

where D is the diameter of the augered 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 by a reduction factor R as follows:

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

Intermediate values may be obtained by interpolation.

For sockets formed within the dolostone bedrock, the ultimate passive force that can be mobilized by the embedded portion of a rock socket is given by:

$$P_p = 6 \,.\, C \,.\, D \,.\, L$$

where C = 2,000 kPa (equivalent Mohr-Coulomb cohesion based on Hoek and Brown rock mass classification)

D = augered pile (socket) diameter (m)

L = depth of socket in rock (m)

The structural designer should check if a 2 m long socket is sufficient to provide base fixity.

8.2.3 Augered H-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 minimum diameter of 600 mm.

The augered pile installation equipment should be capable of dislodging and removing any obstructions such as cobbles, boulders, rock slabs or fragments and other obstructions in the till deposits. Hard dolostone will require the use of coring and/or rock breaking equipment in addition to the auger equipment. Temporary steel liners may be required to support the hole sidewalls and minimize groundwater inflow.

The augered pile socket excavation should be dewatered to allow cleaning of the base and walls prior to seating the pile in the socket. The remaining space in the pre-drilled hole should be grouted with 30 MPa concrete.

The Contract Documents should contain an NSSP alerting the Bidders to the possible presence of cobbles, boulders and rock slabs in the native silty clay and silty clay to clayey silt till immediately above the bedrock. Suggested texts for NSSP's are included in Appendix F.

8.3. Pipe Piles

Steel pipe piles can be used at this site if semi-integral abutments are considered. The steel pipe piles for supporting the abutments will need to be socketted into bedrock. 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. The actual depth of sockets shall be designed to provide the required lateral resistances and base fixity. It is recommended that a minimum 2 m deep socket in the dolostone bedrock be used. The recommended design founding elevations are presented in Table 8.1.

8.3.1 Axial Resistance

For steel pipe piles grouted in minimum 2 m deep rock sockets, the following axial design geotechnical resistances per pile may be used.

Table 8.5 – Design Pile Resistances

Pile Type	Factored Geotechnical Resistance at ULS (kN)
Pipe 324 mm dia. x 12.7 mm thick wall	2,000
Pipe 406 mm dia. x 12.7 mm thick wall	2,800

The geotechnical SLS condition does not govern pile design in rock.

The structural resistance of the pile must be checked by the structural designer. The piles shall also be designed to provide the required lateral resistance and base fixity.

8.3.2 Lateral Resistance

For lateral resistance design of pipe piles, soil-pile interaction analyses may be carried out using the coefficient of horizontal subgrade reaction values provided in Table 8.3, and in conjunction with the equations and method outlined in section 8.2.2 above.

8.3.3 Augered Pile Installation

Augered pipe pile installation should be carried out as discussed in section 8.2.4 of this report for augered H-pile installation.

8.4. Augered Caissons (Drilled Shafts)

Augered caissons (drilled shafts) foundations formed through the silty clay and silty clay to clayey silt till and socketted into dolostone bedrock may be employed at the pier and the abutments. Table 8.6 below presents the recommended founding depths and elevations for caissons at the pier and abutments. It is recommended that each rock socket be extended to at least 2 times the socket diameter below the top of bedrock.

Table 8.6 - Founding Elevations for Augered Caissons

Foundation Element	Borehole	Caisson Diameter (m)	Assumed Design Bedrock Elevation (m)	Assumed Founding Elevation (m)
West Abutment	16-02 (north)	1.2	184.3	181.9
		1.5		181.3
	08-02 (south)	1.2	183.7	181.3
		1.5		180.7
Pier	08-03 (north)	1.2	183.1	180.7
		1.5		180.1
	16-03 (south)	1.2	183.3	180.9
		1.5		180.3
East Abutment	16-04 (north)	1.2	182.8	180.4
		1.5		179.8
	08-04 (south)	1.2	180.9	178.5
		1.5		177.9

8.4.1 Axial Resistance

The following Table 8.7 presents factored geotechnical resistances calculated for typical 1.2 and 1.5 m diameter caissons associated with the following minimum socket depths within bedrock.

Table 8.7 - Vertical Geotechnical Resistance for Caisson Foundations

Caisson Diameter (m)	Minimum Socket Depth below Bedrock Surface (m)	Factored ULS (kN)
1.2	2.4 (2D)	4,000
1.5	3.0 (2D)	7,500

* D = caisson diameter

The SLS condition does not govern design of caisson socketted in bedrock.

The minimum spacing between adjacent caissons should be as per the CHBDC 2014.

8.4.2 Lateral Resistance

For lateral resistance design of caissons, soil-caisson interaction analyses may be carried out using the coefficient of horizontal subgrade reaction values provided in Table 8.3 for the silty clay and silty clay till to clayey silt till and in conjunction with the equations and method outlined in section 8.2.2 above. For caisson analysis, D which denotes the caisson diameter.

8.5 Spread Footings on Bedrock

Spread footings founded on the dolostone bedrock may be considered for supporting the new pier. It is also technically feasible to use spread footings founded on bedrock to support the abutments. The top of bedrock elevations encountered at Boreholes 16-03 and 08-03 are summarized in Table 8.8.

Table 8.8 - Founding Elevation for Footing on Bedrock

Foundation Element	Borehole	Approx. Depth to Bedrock Below Existing Grade (m)	Top of Bedrock Elevation (m)
Pier	16-03	2.3 (from QEW)	183.3
	08-03	1.9 (from QEW)	183.1
West Abutment	16-02	6.7 (from Bowen Road)	184.3
	08-02	1.5 (from QEW)	183.7
East Abutment	16-04	8.1 (from Bowen Road)	182.8
	08-04	3.1 (from QEW)	180.9

Footings founded on dolostone bedrock at or below the elevation quoted above may be designed using a Factored Geotechnical Resistance at Ultimate Limit States (ULS) of 3,000 kPa. The SLS condition does not govern footing design on bedrock.

The geotechnical resistance quoted above is based on a minimum 2.0 m wide footing and for concentric, vertical loads only. In the case of eccentric or inclined loading, the geotechnical resistance must be reduced in accordance with the CHBDC 2014.

Resistance to lateral forces / sliding resistance between the cast-in-place concrete footings and dolostone bedrock should be calculated in accordance with the CHBDC 2014 assuming an unfactored coefficient of friction, tan δ, of 0.7.

The concrete footings may be constructed directly on the surface of the dolostone bedrock. In cases where the underside of a footing is higher than the bedrock subgrade due to a localized dip of top of bedrock or otherwise, mass concrete fill of the same class as the footing should be used to raise the subgrade to the design footing level. The top surface of the bedrock should be stripped of all overburden and be cleaned. All shattered and loosened rock fragments must be removed from the footprint of the footing or mass concrete fill.

All footing excavations must be inspected prior to placing concrete to confirm that the base has been adequately cleaned. Hand cleaning may be required to remove loose rock.

8.6 Spread Footings on Engineered Fill on Bedrock

Alternatively, each of the two abutments may be supported on spread footings founded on a compacted Granular A pad resting on the underlying dolostone bedrock. The bedrock elevations in Table 8.8 above should be used for footing design.

The required depth of excavation for fill pad and footing construction on bedrock will vary depending on the location. It is anticipated that approximately 2 to 4 m of the native silty clay and silty clay to clayey silt till will be excavated and removed to expose the dolostone bedrock. The exposed bedrock subgrade must be inspected by a geotechnical engineer to confirm that the exposed surface conforms with the design requirements. Any loose or shattered rock must be removed and the engineered granular pad be founded on undisturbed dolostone bedrock. For uneven bedrock surfaces, mass concrete with an unconfined compressive strength of at least 30MPa may be used to form a level working platform. Suggested wordings for an NSSP on bedrock subgrade preparation is included in Appendix G.

Placement and compaction of the Granular A pad must be carried out in the dry. The MTO standard “abutment on compacted fill showing Granular A core” should be followed (see Appendix H). The Granular A should comply with OPSS.PROV 1010 requirements and compacted to 100% of its Standard Proctor Maximum Dry Density as per OPSS.PROV 501. Suggested wordings for an NSSP on engineered fill pad construction is included in Appendix G.

It is recommended that footings founded on a compacted Granular A pad with a minimum thickness of 2 m be designed for a Factored Geotechnical Resistance at Ultimate Limit States (ULS) of 1,000 kPa and a Geotechnical Resistance at Serviceability Limit States (SLS) (up to 25mm settlement) of 500 kPa be used for design. These values are for vertical concentric loads only. Effects of load inclination and eccentricity need to be taken into account as per the Canadian Highway Bridge Design Code (CHBDC, 2014).

At the east abutment, the engineered Granular A pad could be up to 4 m in thickness. Provided that the pad is constructed as outlined above on well prepared bedrock subgrade, it is anticipated that the total settlement of the footing, due to bridge loads and fill self-weight induced compression, will not exceed 25 mm.

Resistance to lateral forces / sliding resistance between the concrete footings and compacted Granular A subgrade should be calculated in accordance with the CHBDC 2014 assuming an unfactored coefficient of friction, tan δ, of 0.55.

8.7 Frost Cover

All footings and pile caps founded on native soil or engineered fill must be provided with a minimum 1.2 m of earth cover, or its thermal equivalent, as frost protection. Frost cover is not required for footings on bedrock.

9 LATERAL PRESSURES

A false abutment design is proposed in the preliminary 2016 GA drawings. Lateral pressures for the Retained Soil Systems (RSS) wall design may be addressed as discussed below. Any backfill adjacent to the abutments should consist of Granular A or Granular B Type II material meeting the requirements of OPSS 1010.

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 2014 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 2.0 m for Granular B Type I or 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 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 or OPSS Granular B Type II $\phi = 35°, \gamma = 22.8\ kN/m^3$		OPSS Granular B Type I $\phi = 32°, \gamma = 21.2\ kN/m^3$		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	-

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.

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/or weep holes be installed, where applicable, to provide positive drainage of the granular backfill behind the abutment walls. Reference may be made to OPSD 3102.100 where appropriate.

10 EMBANKMENT DESIGN AND CONSTRUCTION

10.1. Global Stability

New fill will be placed to widen the east and west approaches. The preliminary GA drawing indicates a grade raise in the order of 3 m to 4 m at both approaches, and that the forward slopes have a design inclination of 2H : 1V. The new fill will be up to 8 m in height within the widening areas.

Given the firm to very stiff silty clay to silty clay till overlying shallow bedrock, and provided that the new fill is placed as recommended in this report at a 2H : 1V slope inclination or flatter, the forward slopes and side slopes will remain stable. Figures F1 and F2 in Appendix F show that the Factors of Safety (F.S.) for long term (drained) conditions at the forward slopes are greater than 1.3.

As per MTO requirements, mid-height benches with a minimum width of 2 m are required for maintenance purposes should the fill reaches 8 m or higher. The berms should maintain a 2% grade to positively drain surface runoff.

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 embankment fill must be constructed with adequate quality control in accordance with OPSS.PROV 206 and 501 requirements. It is recommended that the new fill material should consist of OPSS.PROV.1010 Select Subgrade Material (SSM), or Granular A or B Type II materials.

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

10.2. Settlement

Foundation settlement at this site due to new fill placement will result in elastic compression of the over-consolidated silty clay and silty clay to clayey silt till. This subgrade settlement is expected to be complete by the end of fill placement.

Post construction settlement up to the order of 25 mm could occur due to compression of the SSM or granular fills, and is anticipated to be completed within one to two years after the end of embankment construction.

11 RETAINED SOIL SYSTEMS

Retained soil system (RSS) walls may be used at both abutments provided that subject to the requirements presented in this section. RSS walls should be specified to be “High Performance” and “High Appearance”. The contract drawings should include information on the longitudinal alignment of the wall in plan, the top and base elevations of the wall in profile, cross-sectional space constraints and an NSSP for the RSS wall. The concrete levelling pads supporting the front panels may be stepped to accommodate specific geometric configurations.

The performance of a RSS is dependent on, among other factors, the characteristics of its foundation. Failure to provide an adequate foundation may lead to settlement and distortion of the RSS and, in severe cases, to possible failure of the system. The foundation of the entire RSS mass must be considered, i.e. from the face of the wall to the furthest extent of the reinforcement.

To provide an acceptable foundation performance, the RSS mass may be founded on the native firm to very stiff silty clay or silty clay till, or on a pad of engineered fill. The highest founding elevations shown in Table 12.1 are recommended.

Table 12.1 - Founding Elevation for RSS Wall

Foundation Element	Borehole	Highest Founding Level (m)
West Abutment	16-02	185.0
East Abutment	16-04	185.0

A Factored Geotechnical Resistance at ULS of 350 kPa and a Geotechnical Resistance at SLS of 225 kPa may be used for preliminary design.

The fill pad under the RSS mass must be placed and compacted as engineered fill consisting of OPSS Granular A or B Type II compacted to 100% of its Standard Proctor Maximum Dry Density (SPMDD) at a moisture content within 2% of optimum. The engineered pad must extend at least 500 mm beyond the limits of the RSS mass and levelling strip.

The entire block of reinforced earth must be designed against various modes of failure including sliding and overturning. Sliding resistance along the base of the wall on engineered granular fill or native silty clay/silty clay till may be estimated using an ultimate friction coefficient of 0.45.

Topsoil, organics, loose fill and any soft/wet native material should be stripped from the footprint of the RSS. The soil subgrade under the RSS foundation should be inspected, and any incompetent materials should be sub-excavated and backfilled with approved, compacted granular fill.

The proprietary RSS system must meet the Ministry's specifications for performance and appearance. The RSS supplier/designer may specify more stringent criteria or other requirements related to the particular design. The internal stability of the RSS wall should be analyzed by the supplier/designer of the proprietary product selected for this site.

RSS walls founded on the undisturbed native soils or engineered fill at this site, and having a maximum height of 8 m, will satisfy global stability requirements. Figures F1 to F4 in Appendix F show that the Factors of Safety (F.S.) for long term (drained) conditions at the forward and side slopes with RSS walls are greater than 1.3.

12 EXCAVATION AND GROUNDWATER CONTROL

All temporary excavations must be carried out in accordance with the Occupational Health and Safety Act (OHSA). The excavation and backfilling for foundations must be carried out in accordance with OPSS 902.

For the purposes of the OHSA, the fill and the native soils at this site may be classified as Type 3 materials.

Excavation for foundation construction will extend through topsoil, pavement structure, silty clay fill into the native silty clay and silty clay till to clayey silt till.

Native soils at this site are relatively impermeable. There could be seepage from perched water within the surficial fill. Groundwater control during construction will likely involve diverting surface runoff away from the excavations and sump pumping. Filtered sumps must be designed properly so that construction drainage water containing eroded soil particles does not flow towards the QEW and the surroundings.

Where space permits, temporary unsupported excavations through the cohesive soils at this site may be formed with side slopes not exceeding 1H : 1V.

Dewatering, where required, is the responsibility of the Contractor who should retain specialists in this field for design and implementation of any dewatering systems.

13 ROADWAY PROTECTION

Roadway protection will be required during construction of the abutments and the piers. An item titled "Protection System" as per OPSS.PROV 539 should be included in the contract documents. It is recommended that Performance Level 2 as per Clause 539.04.01.01 and the alignment of the roadway protection be specified on the contract drawings.

The design of roadway protection should be the responsibility of the Contractor. However, one option that is considered to be suitable for use as temporary shoring at this site is a soldier pile and lagging wall. It is anticipated that the protection system will need to be extended predominantly through the existing silty clay embankment fill into the underlying native silty clay to clayey silt till to develop the required toe resistance.

A soldier pile and lagging wall may be designed using the parameters given below:

Soil Bulk Unit Weight	γ	=	20 kN/m^3
Submerged Unit Weight	γ'	=	9.8 kN/m^3
Coefficient of Active Pressure	K_a	=	0.33 (approach fills)
Coefficient of Passive Pressure	K_p	=	3.0 (approach fills)
		=	3.4 (native glacial tills)

The surcharge should include soil loadings above the top of the pile and other loadings adjacent to the wall. A soldier pile and lagging wall will be permeable and therefore water pressure acting on the retained height may be set to zero. 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 roadway protection system.

The designer of the roadway protection system should check whether the depth of the soldier piles is sufficient to provide base fixity.

All roadway protection systems should be designed by a Professional Engineer experienced in such designs.

14 SEISMIC CONSIDERATIONS

According to Clause 4.4.4 of the CHBDC 2014, 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 2014. The peak ground acceleration, PGA, associated with the design earthquake for Site Class C is 0.067g.

The above PGA value will be modified with the site coefficient is 1.00 based on Table 4.8 of the CHBDC 2014.

In accordance with Clause 4.6.5 of the CHBDC 2014, retaining structures should be designed using active (K_{AE}) and passive (K_{PE}) earth pressure coefficients that incorporate the effects of earthquake loading. The coefficients of horizontal earth pressure for seismic loading presented in Table 14.1 may be used:

Table 14.1 – Earth Pressure Coefficients for Earthquake Loading

Condition	Earth Pressure Coefficient (K)	
	OPSS Granular A or Granular B Type II $\phi = 35°, \gamma = 22.8$ kN/m^3	OPSS Granular B Type I (modified) $\phi = 32°, \gamma = 21.2$ kN/m^3
Active (K_{AE})*	0.30	0.34
Passive (K_{PE})	3.6	3.1
At Rest (K_{OE})**	0.54	0.59

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

The firm to very stiff silty clay, silty clay to clayey silt till and bedrock at this site are not considered susceptible to liquefaction under seismic loading.

15 ADJACENT STRUCTURES AND BURIED UTILITIES

It is understood that an existing Bell Canada, Enbridge Gas line and Hydro utilities are present on the north and south sides of the existing bridge. Other buried utilities might also be present in the new foundation construction areas. In particular, AECOM advises that an Enbridge gas line located on the south side of the existing bridge and approaches is to be relocated up to 50 m further south, which is well beyond the influence zone that will be imposed by the new embankment fills. As long as this relocation is completed prior to commencing bridge construction, it is anticipated that the new construction will have negligible effects on the integrity of the gas line.

It is important to confirm the exact locations and exact conditions of all these utilities, and compare with the extent of the potential work zones related to the foundations of the proposed widening structures and associated works. These utilities must not be adversely affected by construction of the new foundations. If necessary, relocation of, and/or special protective measures for affected utilities may be required.

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 utilities. Any distress should be reported to and discussed with the structure/utility owner.
- Implement an instrumentation and monitoring program to include settlement monitoring during installation of shoring, excavation and new foundation construction. Establish review and alert level criteria for allowable settlement and lateral movement following discussions with the owner of the structure/utility. Establish and agree on remedial action, if required, prior to start of construction.

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

Utility relocation that may be required after bridge construction should be carried out with care such that the new bridge foundations and other utilities will not be undermined.

16 CONSTRUCTION CONCERNS

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

- Foundation construction in close proximity to the existing gas line, other utilities and the existing bridge foundations. Settlement and vibration monitoring should be conducted before and during construction. Settlement monitoring should continue after construction.
- Variation in bedrock surface or sloping bedrock surface may require the use of concrete fill to prepare the design founding subgrade; or pose difficulties in confirming the pile or caisson socket depths.
- The fill and silty clay till may contain cobbles and boulders. Equipment selected for excavation or to install augered piles and caissons must be capable of penetrating, handling and/or removing these obstructions;
- Steel liner should be used to support the caisson sidewalls, minimise groundwater inflow and enable machine cleaning of the socket base when advancing pile sockets into the bedrock.
- Excavation will be required through predominantly cohesive soils to expose the dolostone bedrock. The exposed bedrock subgrade must be inspected by a geotechnical engineer to confirm that the exposed surface conforms with the design requirements. Any loose or shattered rock must be removed and the engineered granular pad be formed on undisturbed dolostone bedrock. Mass concrete may be used to form a working platform on uneven bedrock surfaces.
- The forward and side embankment slopes should be inspected after construction for surficial disturbance. Where necessary, remedial measures such as re-vegetation and/or placement of gravel sheeting may be required.

17 CLOSURE

Engineering analysis and preparation of this foundation design report was carried out by Ms. Rocío Palomeque Reyna, P.Eng and Dr. Sydney Pang, P.Eng.

Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects, reviewed the report.

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Appendix A

Record of Borehole Sheets

(current investigation)

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

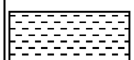
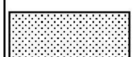
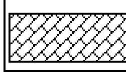
ROCK WEATHERING CLASSIFICATION

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 16-01

METRIC

W.P. 2482-04-00 LOCATION QEW/Bowen Rd. Underpass N 4 754 992.3 E 346 919.1 ORIGINATED BY OA

HWY QEW BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN

DATUM Geodetic DATE 2016.08.29 - 2016.08.29 CHECKED BY RPR

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
190.7	GROUND SURFACE					
0.0	**ASPHALT:** (100mm)					
0.1	Gravelly **SAND** Compact Grey Moist (FILL)	1	SS	28	○	
190.0 / 0.7	Silty **CLAY**, some sand, trace gravel Stiff to Firm Brown to Dark Brown Moist (FILL)	2	SS	9	○	
		3	SS	6	○	
		4	SS	5	○	
		5	SS	6	○	3 23 31 43
186.2 / 4.4	Silty **CLAY**, some sand to sandy, trace gravel Very Stiff Dark Brown Moist (TILL)	6	SS	19	○	8 40 35 17
184.4	Hard	7	SS	50/ 0.150	○	
6.2	END OF BOREHOLE AT 6.2m UPON REFUSAL ON BEDROCK. NO FREE WATER IN BOREHOLE UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND AUGER CUTTINGS TO 0.1m, THEN CONCRETE TO SURFACE.					

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (190, 189, 188, 187, 186, 185); 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

+ 3, × 3: Numbers refer to Sensitivity

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 16-02

METRIC

W.P.	2482-04-00	LOCATION	QEW/Bowen Rd. Underpass N 4 754 993.0 E 346 932.4	ORIGINATED BY	OA
HWY	QEW	BOREHOLE TYPE	Hollow Stem Augers/HQ Coring	COMPILED BY	AN
DATUM	Geodetic	DATE	2016.08.29 - 2016.08.29	CHECKED BY	RPR

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	NUMBER	TYPE	"N" VALUES	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
191.0	GROUND SURFACE					
0.0 / 0.1	**ASPHALT:** (100mm)					
190.6 / 0.5	Gravelly **SAND**, Compact, Grey, Moist (FILL)	1	SS	15		
	Silty **CLAY**, some sand, trace gravel, Stiff to Firm, Brown, Moist (FILL)	2	SS	8		
		3	SS	5		0 17 33 50
		4	SS	5		
		5	SS	7		
186.5 / 4.6	Silty **CLAY**, some sand, trace gravel, Firm, Brown, Moist	6	SS	7		
	Hard	7	SS	31		3 19 38 40
	Coring started at 6.7m					
184.3 / 6.7	Shaley **DOLOSTONE**, slightly to moderately weathered, thinly bedded, grey, with shale interbeds				FI	
	Horizontal joints (25mm) at 7.1m, 7.2m, 7.3m, 7.4m and 7.5m	1	RUN		6, 6, 1	RUN #1 TCR=100% SCR=100% RQD=17% UCS=145MPa (Dolostone)
	Horizontal joints (25mm) at 7.8m, 7.9m, 8.0m, 8.1m, 8.3m, 8.7m, 8.8m, 8.9m and 9.2m	2	RUN		8, 5, 4, 4	RUN #2 TCR=100% SCR=100% RQD=32% UCS=130MPa (Dolostone)
	Horizontal joints (25mm) at 9.4m, 9.5m, 9.6m, 9.7m, 9.8m, 9.9m, 10.3m and 10.7m				3, 5	
	Rubble zone (50mm) at 9.9m				5	

Elevation scale: 190, 189, 188, 187, 186 (groundwater level ▼ at approx. 186), 185, 184, 183, 182

Continued Next Page

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

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

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 16-02

METRIC

W.P. 2482-04-00 | LOCATION QEW/Bowen Rd. Underpass N 4 754 993.0 E 346 932.4 | ORIGINATED BY OA

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

DATUM Geodetic | DATE 2016.08.29 - 2016.08.29 | CHECKED BY RPR

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										
	Shaley **DOLOSTONE**, thinly bedded, fresh, grey Vertical joint (175mm) at 10.3m		3	RUN						3 3 1	RUN #3 TCR=100% SCR=100% RQD=50% UCS=85MPa (Dolostone)
180.2 10.8	END OF BOREHOLE AT 10.8m. NO FREE WATER IN BOREHOLE UPON COMPLETION OF DRILLING. Piezometer installation consists of 50mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Sep26/ 16 5.1 185.9										

+ ³, × ³: Numbers refer to Sensitivity

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

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 16-03

1 OF 1

METRIC

W.P. 2482-04-00

LOCATION QEW/Bowen Rd. Underpass N 4 754 966.0 E 346 983.5

ORIGINATED BY OA

HWY QEW

BOREHOLE TYPE Hollow Stem Augers/HQ Coring

COMPILED BY AN

DATUM Geodetic

DATE 2016.08.30 - 2016.08.30

CHECKED BY RPR

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	UNIT WEIGHT / FI	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
185.6	GROUND SURFACE					
0.0	**TOPSOIL:** (25mm)					
	Silty **CLAY**, some sand, trace gravel, occasional organics, roots and rootlets; Firm; Brown; Moist	1	SS	8		
184.9 / 0.7	Silty **CLAY**, some sand to sandy, trace gravel; Very Stiff to Hard; Brown; (TILL)	2	SS	27		
		3	SS	47		4 39 37 20
183.3 / 2.3	Coring started at 2.3m. Shaley **DOLOSTONE**, slightly weathered, thinly bedded, grey, with shale interbeds. Horizontal joints (25mm) at 3.0m, 3.2m, 3.6m, 3.7m and 4.3m	1	RUN		FI: 5, 4	RUN #1 TCR=100% SCR=100% RQD=17%
		2	RUN		2, 3, 2, 1, 3	RUN #2 TCR=100% SCR=100% RQD=70% UCS=148MPa (Dolostone)
	Fresh. Horizontal joints (25mm) at 4.9m, 5.0m, 5.3m and 5.4m	3	RUN		0, 1, 2, 3	RUN #3 TCR=100% SCR=100% RQD=94% UCS=126MPa (Dolostone)
180.0 / 5.6	END OF BOREHOLE AT 5.6m. NO FREE WATER IN BOREHOLE UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO SURFACE.				1	

Chart headers: GROUND WATER CONDITIONS; ELEVATION SCALE (185, 184, 183, 182, 181, 180); 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

+ ³, × ³: Numbers refer to Sensitivity

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

ONTMT4S MTO-14515.GPJ 2015TEMPLATE(MTO).GDT 10/24/16

Ministry of Transportation Ontario

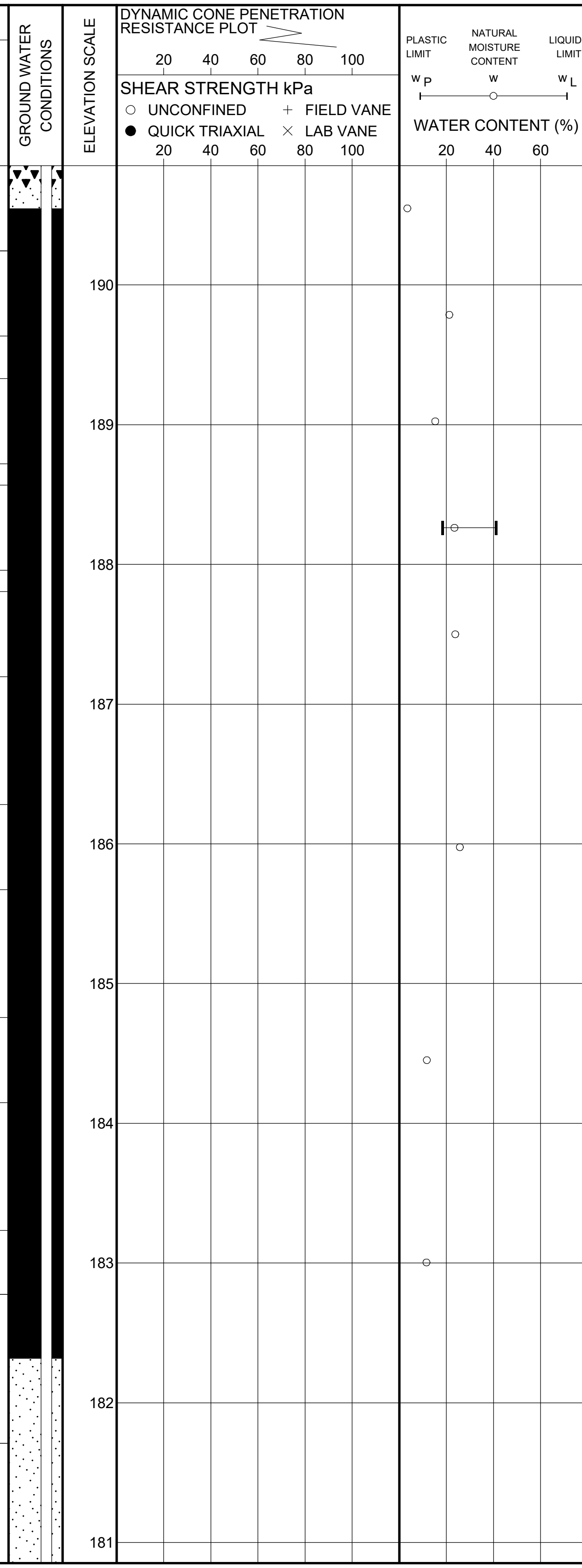
THURBER

RECORD OF BOREHOLE No 16-04

METRIC

W.P.	2482-04-00	LOCATION	QEW/Bowen Rd. Underpass N 4 754 993.8 E 346 992.1	ORIGINATED BY	OA
HWY	QEW	BOREHOLE TYPE	Hollow Stem Augers/HQ Coring	COMPILED BY	AN
DATUM	Geodetic	DATE	2016.08.30 - 2016.08.30	CHECKED BY	RPR

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	UNIT WEIGHT γ kN/m3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
190.9	GROUND SURFACE						
0.0	**ASPHALT:** (100mm)						
0.1	Gravelly **SAND** Compact Grey Moist (FILL)	1	SS	14			
190.2 / 0.7	Silty **CLAY**, some sand, trace gravel Firm Brown Moist (FILL)	2	SS	7	190		
		3	SS	5	189		
		4	SS	4			0 18 36 46
		5	SS	5	188		
186.9 / 4.0	Silty **CLAY**, some sand, trace gravel Soft Brown Moist				187		
		6	SS	3	186		
184.8 / 6.1	Silty **CLAY**, some sand to sandy, some gravel Very Stiff Brown Moist (TILL)	7	SS	20	185		
					184		
	Coring started at 8.1m	8	SS	39	183		14 38 32 16
182.8 / 8.1	Shaley **DOLOSTONE**, slightly weathered, thinly bedded, grey, with shale interbeds					FI	RUN #1 TCR=100% SCR=100% RQD=33% UCS=105MPa (Dolostone)
	Horizontal joints (25mm) at 8.2m, 8.3m, 8.4m, 8.5m, 8.8m and 9.0m	1	RUN		182	4, 3, 2	
	Sub-horizontal joint (25mm) at 9.2m					1	RUN #2 TCR=100% SCR=100% RQD=73% UCS=100MPa (Dolostone)
	Horizontal joints (25mm) at 9.5m, 9.6m, 9.8m, 9.9m, 10.1m, 10.2m and 10.6m	2	RUN		181	1	



Continued Next Page

+3, ×3: Numbers refer to Sensitivity

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

RECORD OF BOREHOLE No 16-04

METRIC

W.P.	2482-04-00	LOCATION	QEW/Bowen Rd. Underpass N 4 754 993.8 E 346 992.1	ORIGINATED BY	OA
HWY	QEW	BOREHOLE TYPE	Hollow Stem Augers/HQ Coring	COMPILED BY	AN
DATUM	Geodetic	DATE	2016.08.30 - 2016.08.30	CHECKED BY	RPR

ELEV / DEPTH	DESCRIPTION	SAMPLES: NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page						
	Shaley **DOLOSTONE**, thinly bedded, fresh, grey					3	
	Rubble zone (25mm) at 10.5m					2	
	Horizontal joints (25mm) at 10.9m and 11.1m	3	RUN		▼ 180	4 1 0	RUN #3 TCR=100% SCR=100% RQD=94% UCS=110MPa (Dolostone)
179.3 11.6	END OF BOREHOLE AT 11.6m. Piezometer installation consists of 50mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen.						

WATER LEVEL READINGS:

DATE	DEPTH (m)	ELEV. (m)
Sep26/ 16	10.9	180.0

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

+3, ×3: Numbers refer to Sensitivity

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

Ministry of Transportation Ontario

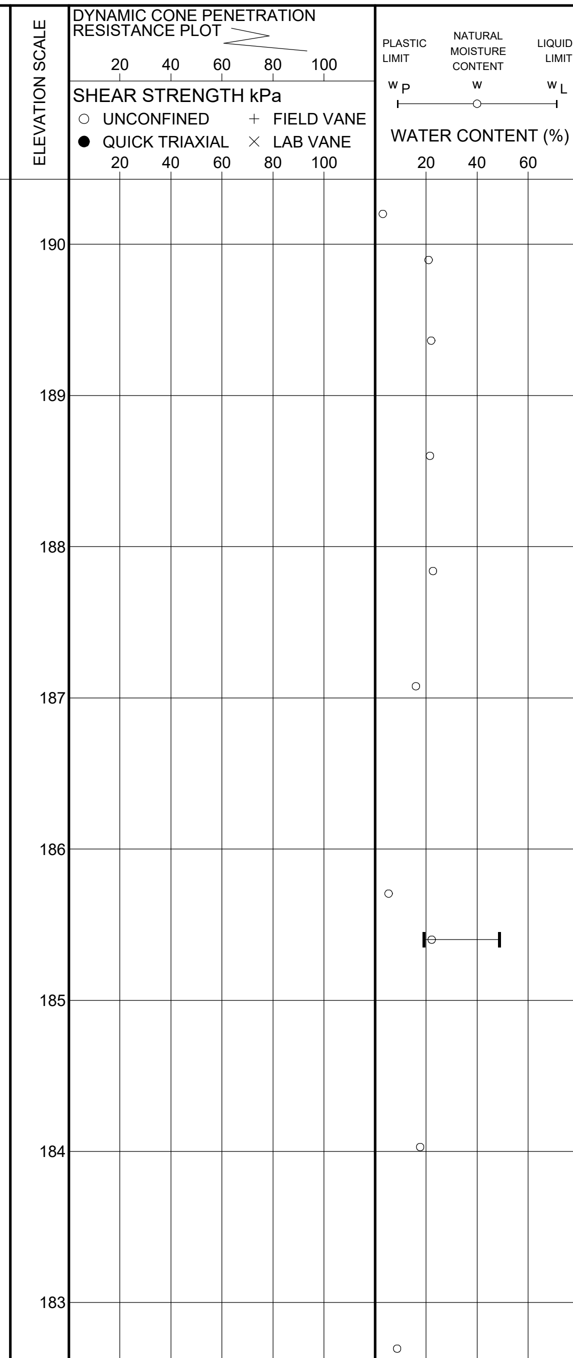
THURBER

RECORD OF BOREHOLE No 16-05

METRIC

W.P.	2482-04-00	LOCATION	QEW/Bowen Rd. Underpass N 4 754 994.3 E 347 005.8	ORIGINATED BY	OA
HWY	QEW	BOREHOLE TYPE	Hollow Stem Augers	COMPILED BY	AN
DATUM	Geodetic	DATE	2016.08.29 - 2016.08.29	CHECKED BY	RPR

ELEV / DEPTH (m)	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
190.4	GROUND SURFACE					
0.0	**ASPHALT:** (100mm)					
0.1	Gravelly **SAND** Compact Grey Moist (FILL)	1	SS	21		
190.0 / 0.5	Silty **CLAY**, trace sand, trace gravel Stiff to Firm Brown Moist (FILL)	2	SS	9		
		3	SS	13		
		4	SS	6		
187.4 / 3.0	Silty **CLAY**, some sand, trace gravel Firm Brown Moist	5	SS	6		
		6	SS	50		4 18 35 43
185.4 / 5.0	Gravelly **SAND**, some silt Very Dense Grey Moist					
184.3 / 6.1	Clayey **SILT**, some sand to sandy, trace gravel Very Stiff Brown Wet (TILL)	7	SS	17		
	Hard	8	SS	50/ 0.075		6 45 36 13
182.6 / 7.8	END OF BOREHOLE AT 7.8m UPON REFUSAL ON BEDROCK. NO FREE WATER IN BOREHOLE UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND AUGER CUTTINGS TO 0.1m, THEN CONCRETE TO SURFACE.					



+3, ×3: Numbers refer to Sensitivity

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 16-06

METRIC

W.P. 2482-04-00 | LOCATION QEW/Bowen Rd. Underpass N 4 754 995.3 E 347 046.7 | ORIGINATED BY OA
HWY QEW | BOREHOLE TYPE Hollow Stem Augers | COMPILED BY AN
DATUM Geodetic | DATE 2016.08.29 - 2016.08.29 | CHECKED BY RPR

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR SA SI CL
188.6	GROUND SURFACE				
0.0	**ASPHALT:** (100mm)				
188.5 / 0.1 (188.3 / 0.3)	Gravelly **SAND** Compact Grey Moist (FILL)	1	SS	22	
	Silty **CLAY**, trace sand, trace gravel Very Stiff to Firm Brown Moist (FILL)	2	SS	7	
		3	SS	15	
		4	SS	7	
185.6 / 3.0	Silty **CLAY**, trace sand, trace gravel Firm Brown to Grey Moist	5	SS	7	0 10 34 56
184.5 / 4.1	Silty **CLAY**, some sand to sandy, some gravel Very Stiff Brown Moist (TILL)	6	SS	14	
	Hard	7	SS	50/ 0.150	12 33 38 17
182.2 / 6.4	END OF BOREHOLE AT 6.4m UPON REFUSAL ON BEDROCK. NO FREE WATER IN BOREHOLE UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND AUGER CUTTINGS TO 0.1m, THEN CONCRETE TO SURFACE.				

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (188, 187, 186, 185, 184, 183); 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; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

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

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 16-07

METRIC

W.P.	2482-04-00	LOCATION	QEW/Bowen Rd. Underpass N 4 754 977.4 E 347 090.6	ORIGINATED BY	OA
HWY	QEW	BOREHOLE TYPE	Hollow Stem Augers	COMPILED BY	AN
DATUM	Geodetic	DATE	2016.08.29 - 2016.08.29	CHECKED BY	RPR

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	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	GR	SA	SI	CL
183.6	GROUND SURFACE										
0.0	**TOPSOIL:** (25mm)										
	Silty **CLAY**, some sand to sandy, trace gravel Firm to Hard Brown to Grey Moist (TILL)		1	SS	5						
							183				
			2	SS	63			0	16	43	41
							182				
			3	SS	74						
			4	SS	95						
							181				
			5	SS	58			9	38	41	12
179.9							180				
3.7	END OF BOREHOLE AT 6.2m UPON REFUSAL ON BEDROCK. NO FREE WATER IN BOREHOLE UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND AUGER CUTTINGS TO SURFACE.										

+ ³, × ³: Numbers refer to Sensitivity

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

Appendix B

Laboratory Test Results

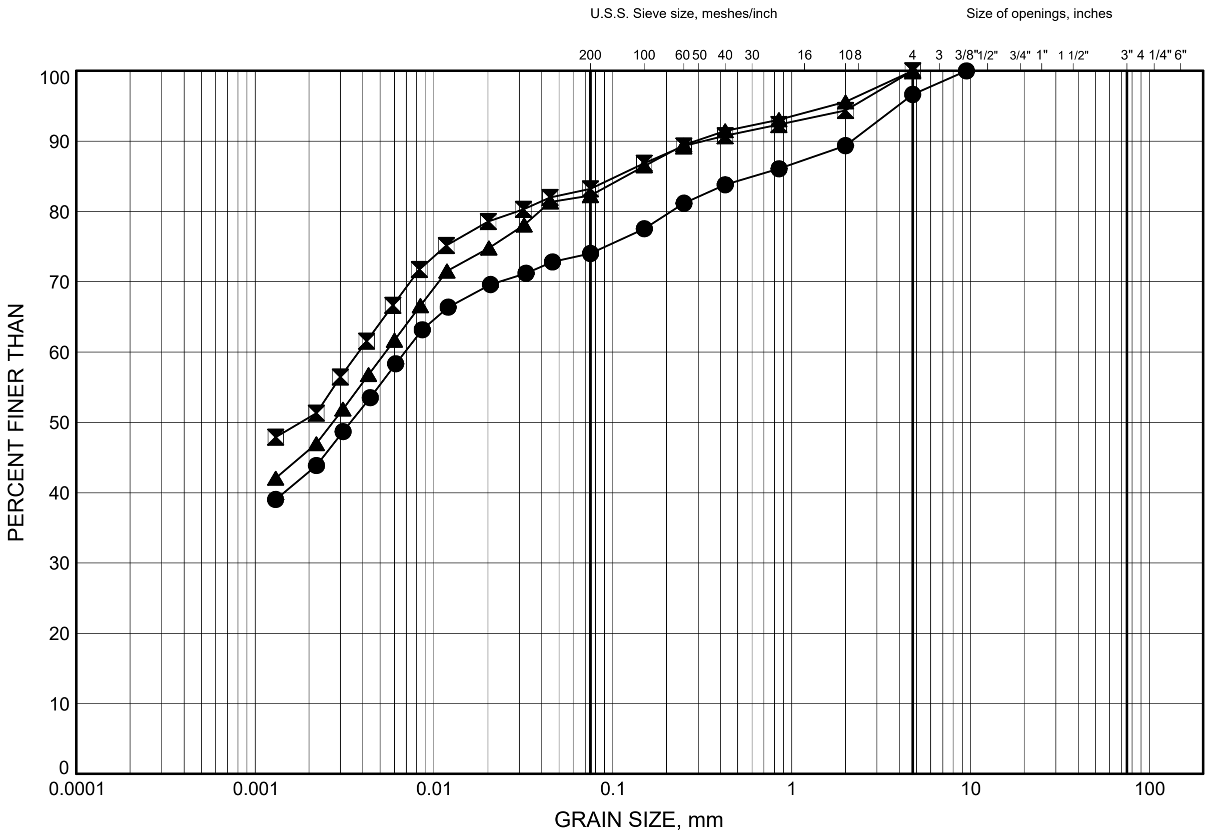
(current investigation)

QEW/Bowen Rd. Underpass

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)
●	16-01	3.35	187.31
⧗	16-02	1.83	189.19
▲	16-04	2.59	188.26

Date October 2016

W.P. 2482-04-00

THURBER

Prep'd AN

Chkd. ROR

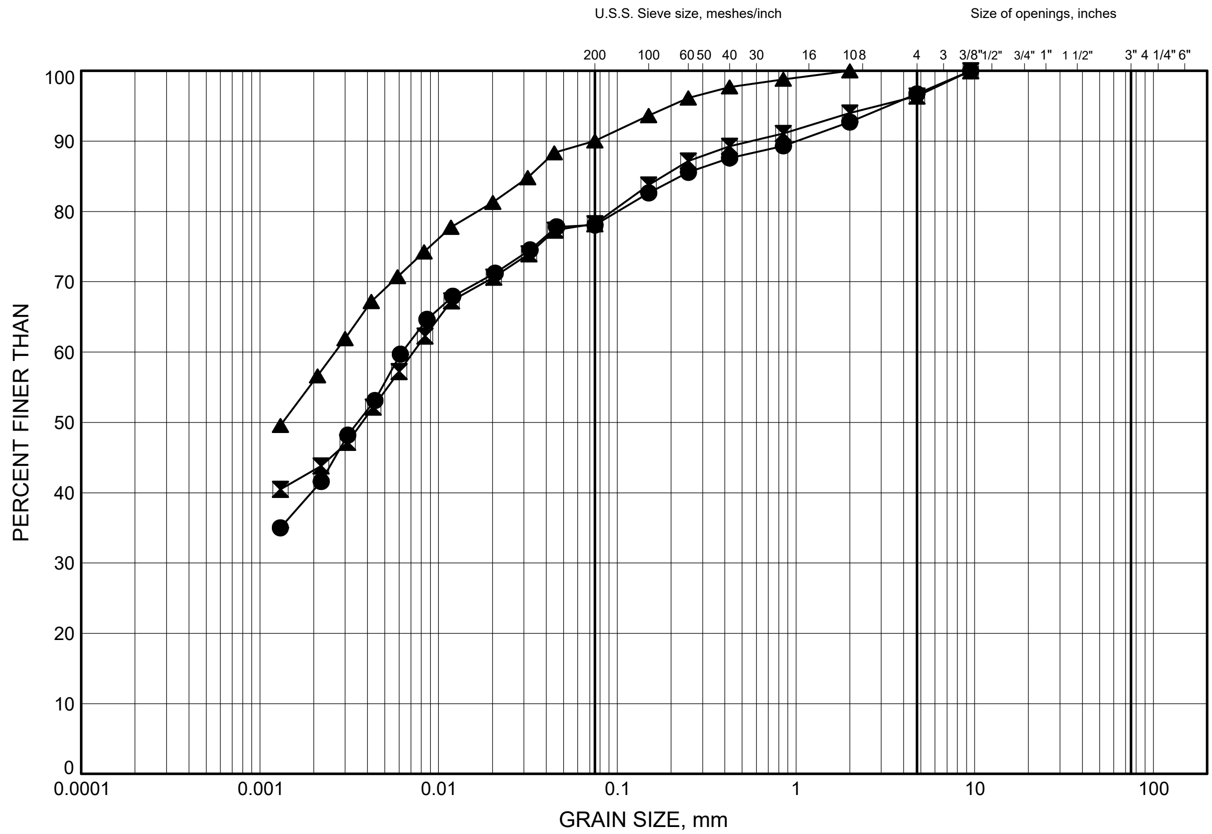
GRAIN SIZE DISTRIBUTION - THURBER MTO-14515.GPJ 10/24/16

QEW/Bowen Rd. Underpass

GRAIN SIZE DISTRIBUTION

FIGURE B2

Silty CLAY



<table>
<tr><td rowspan="1">SILT and CLAY</td><td>FINE</td><td>MEDIUM</td><td>COARSE</td><td>FINE</td><td>COARSE</td><td rowspan="2">COBBLE SIZE</td></tr>
<tr><td>FINE GRAINED</td><td colspan="3">SAND</td><td colspan="2">GRAVEL</td></tr>
</table>

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	16-02	6.40	184.62
⧗	16-05	5.03	185.40
▲	16-06	3.35	185.30

Date October 2016

W.P. 2482-04-00

THURBER

Prep'd AN

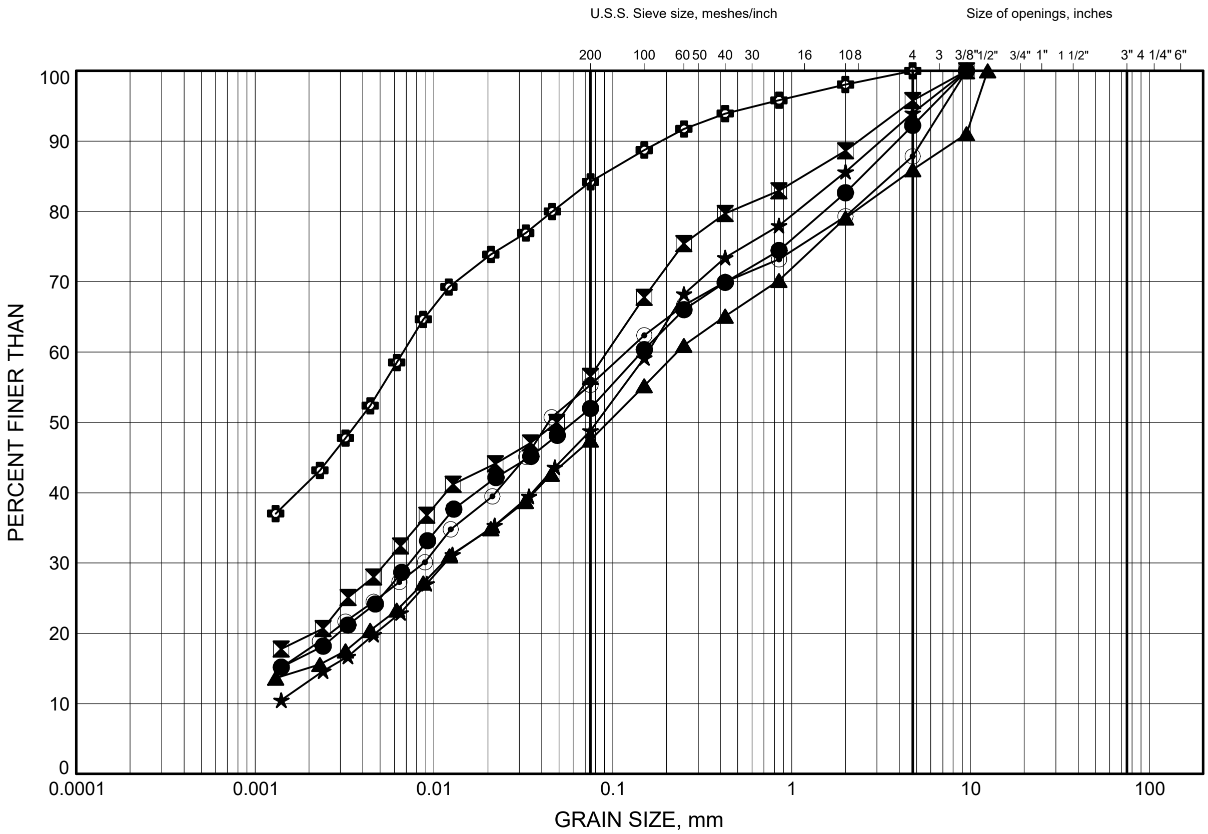
Chkd. RPR

QEW/Bowen Rd. Underpass

GRAIN SIZE DISTRIBUTION

FIGURE B3

Silty CLAY to Clayey SILT TILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	16-01	4.88	185.78
⧗	16-03	1.83	183.77
▲	16-04	7.92	182.93
★	16-05	7.73	182.70
⊙	16-06	6.25	182.40
✪	16-07	1.07	182.53

Date October 2016

W.P. 2482-04-00

THURBER

Prep'd AN

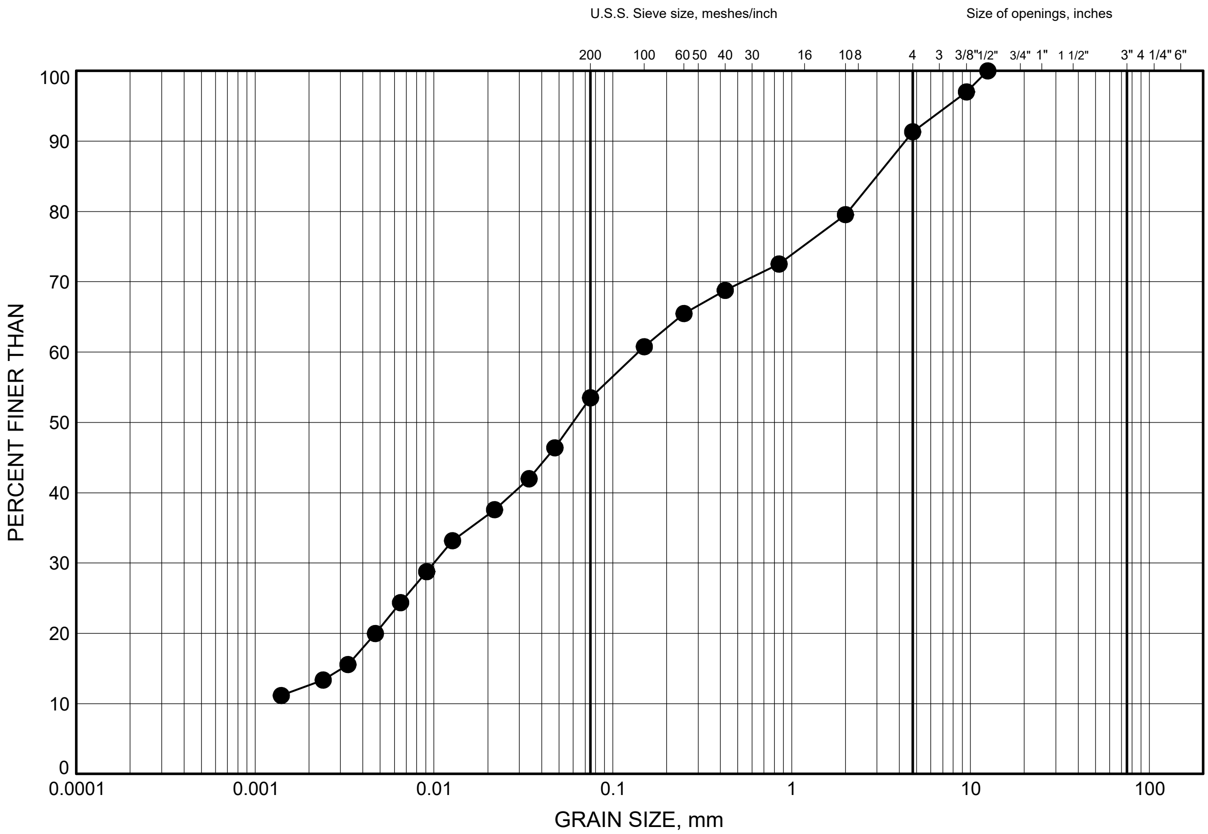
Chkd. RPR

QEW/Bowen Rd. Underpass

GRAIN SIZE DISTRIBUTION

FIGURE B4

Silty CLAY to Clayey SILT TILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	16-07	3.35	180.24

Date October 2016

W.P. 2482-04-00

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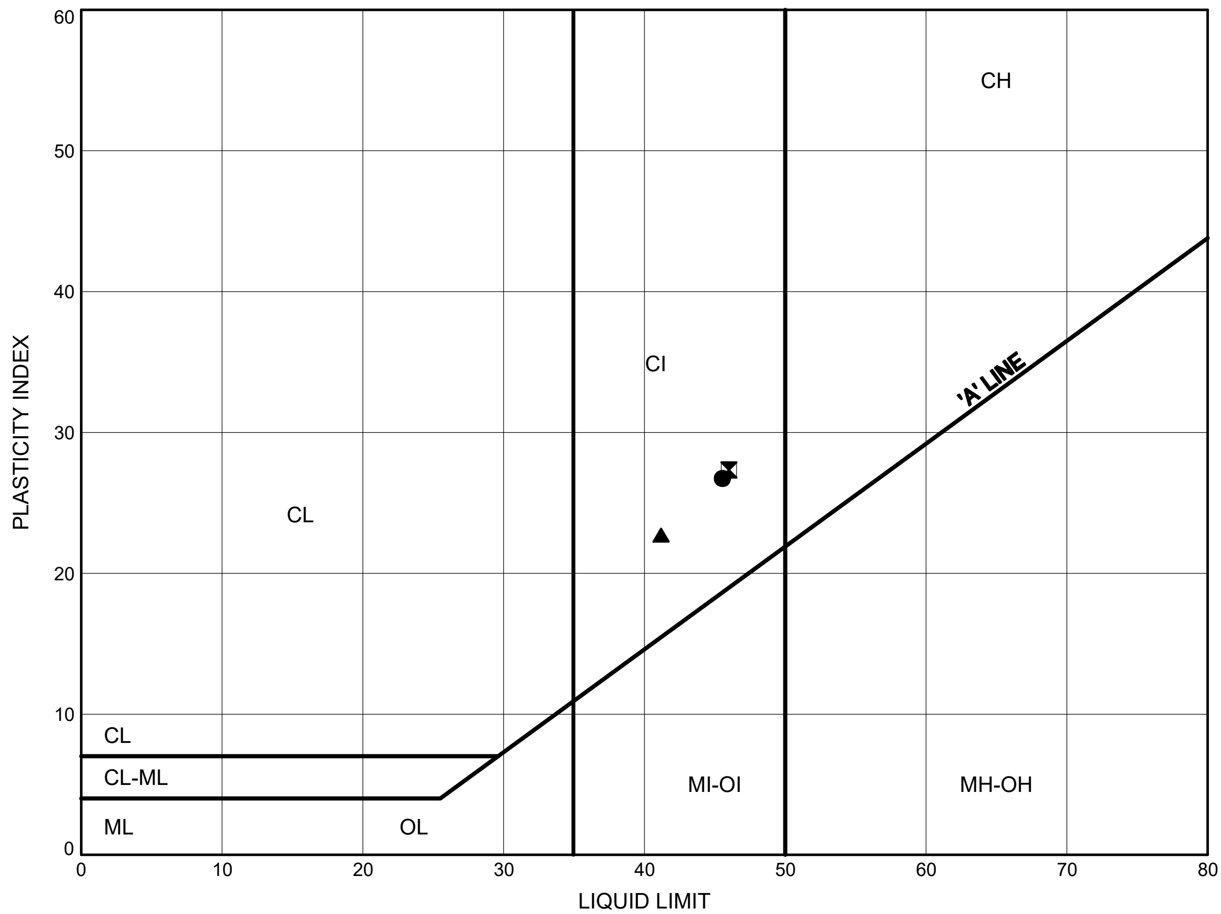
GRAIN SIZE DISTRIBUTION - THURBER MTO-14515.GPJ 10/20/16

QEW/Bowen Rd. Underpass

ATTERBERG LIMITS TEST RESULTS

FIGURE B5

Silty CLAY FILL



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	16-01	3.35	187.31
⧗	16-02	1.83	189.19
▲	16-04	2.59	188.26

Date October 2016
W.P. 2482-04-00

THURBER

Prep'd AN
Chkd. RPR

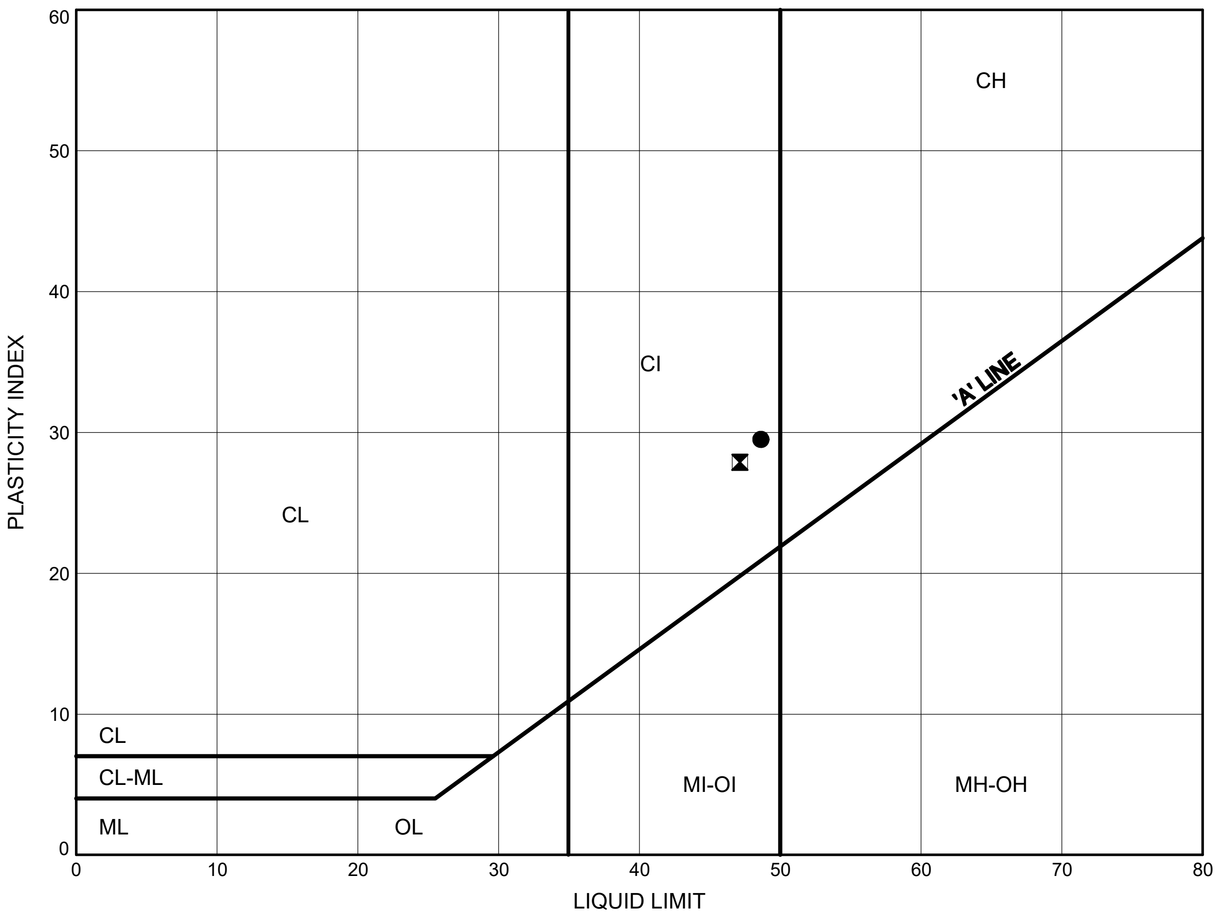
THURBALT MTO-14515.GPJ 10/20/16

QEW/Bowen Rd. Underpass

ATTERBERG LIMITS TEST RESULTS

FIGURE B6

Silty CLAY



LEGEND

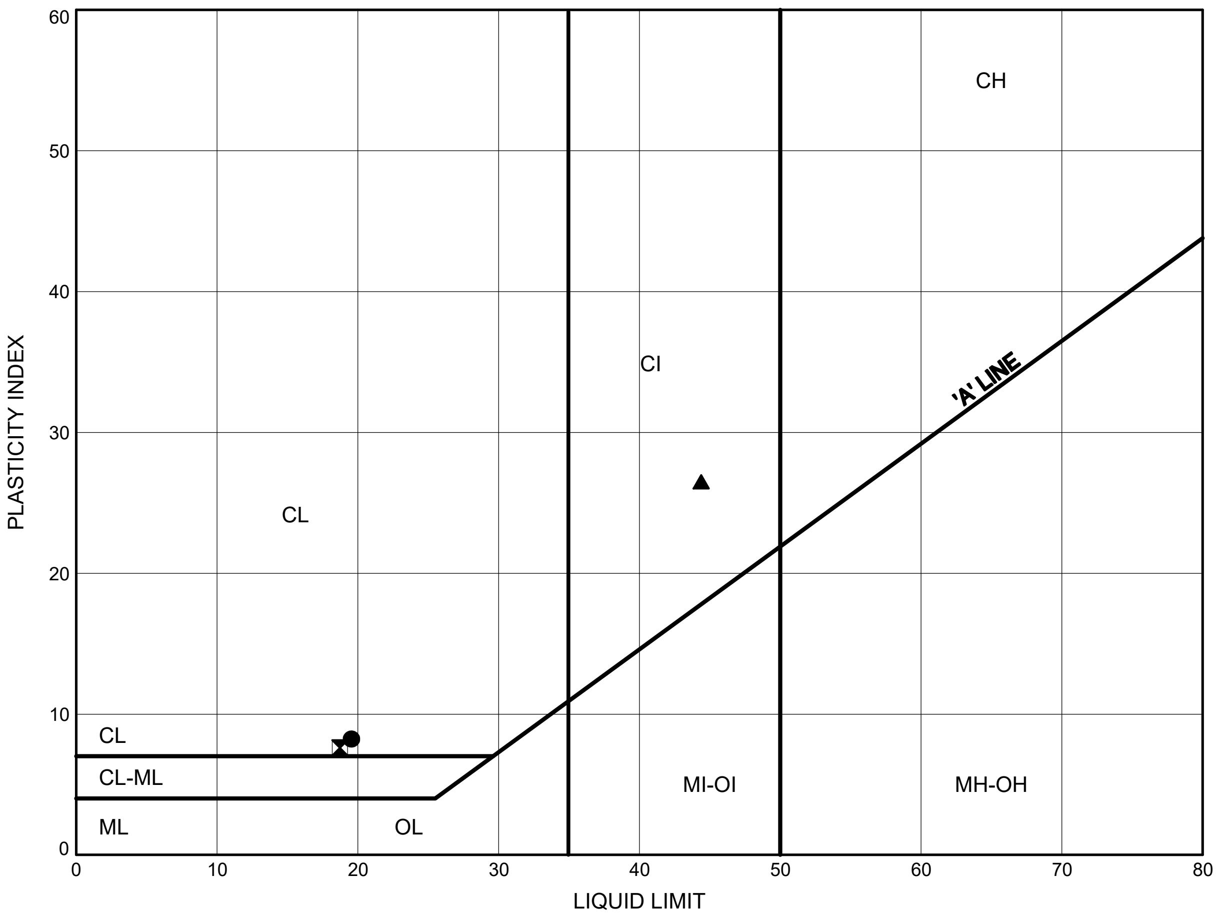
SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	16-05	5.03	185.40
⧗	16-06	3.35	185.30

QEW/Bowen Rd. Underpass

ATTERBERG LIMITS TEST RESULTS

FIGURE B7

Silty CLAY TILL



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	16-03	1.83	183.77
⧗	16-06	6.25	182.40
▲	16-07	1.07	182.53

THURBALT MTO-14515.GPJ 10/20/16

Date October 2016

W.P. 2482-04-00

THURBER

Prep'd AN

Chkd. RPR

Appendix C

Drawings titled “Borehole Locations and Soil Strata”

METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

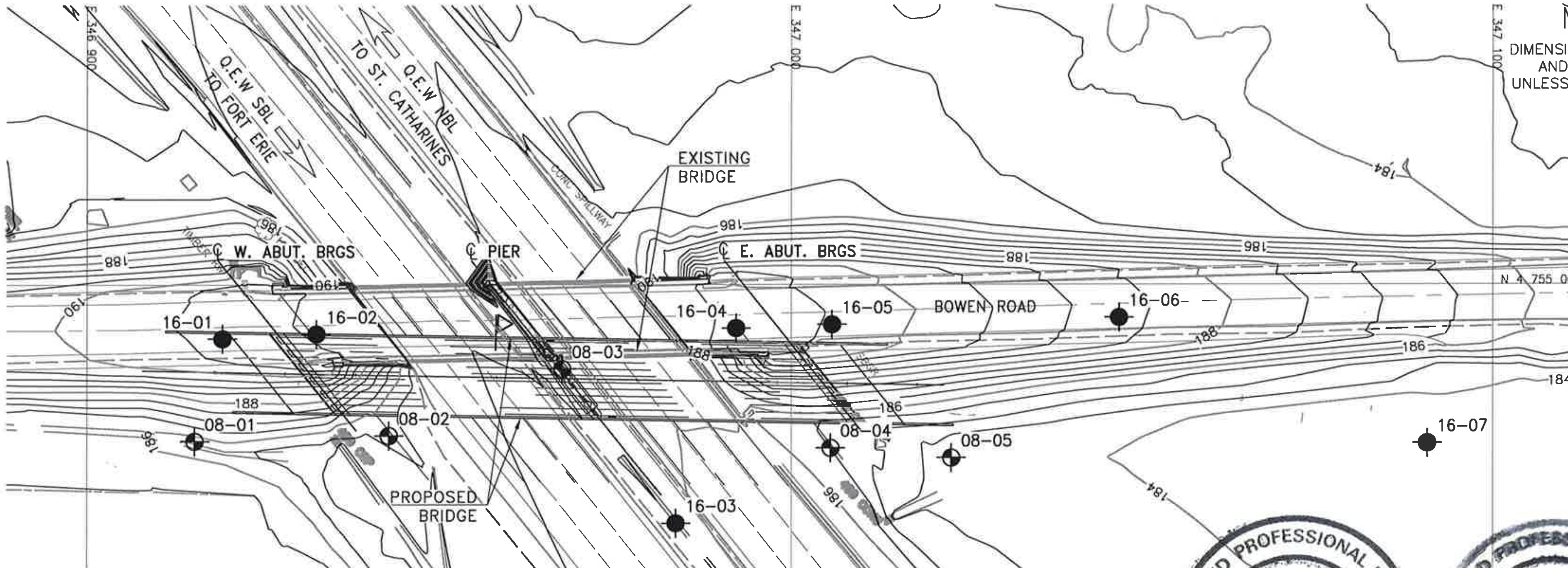
CONT No
WP No 2482-04-00

BOWEN ROAD UNDERPASS AT QEW BRIDGE REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

AECOM

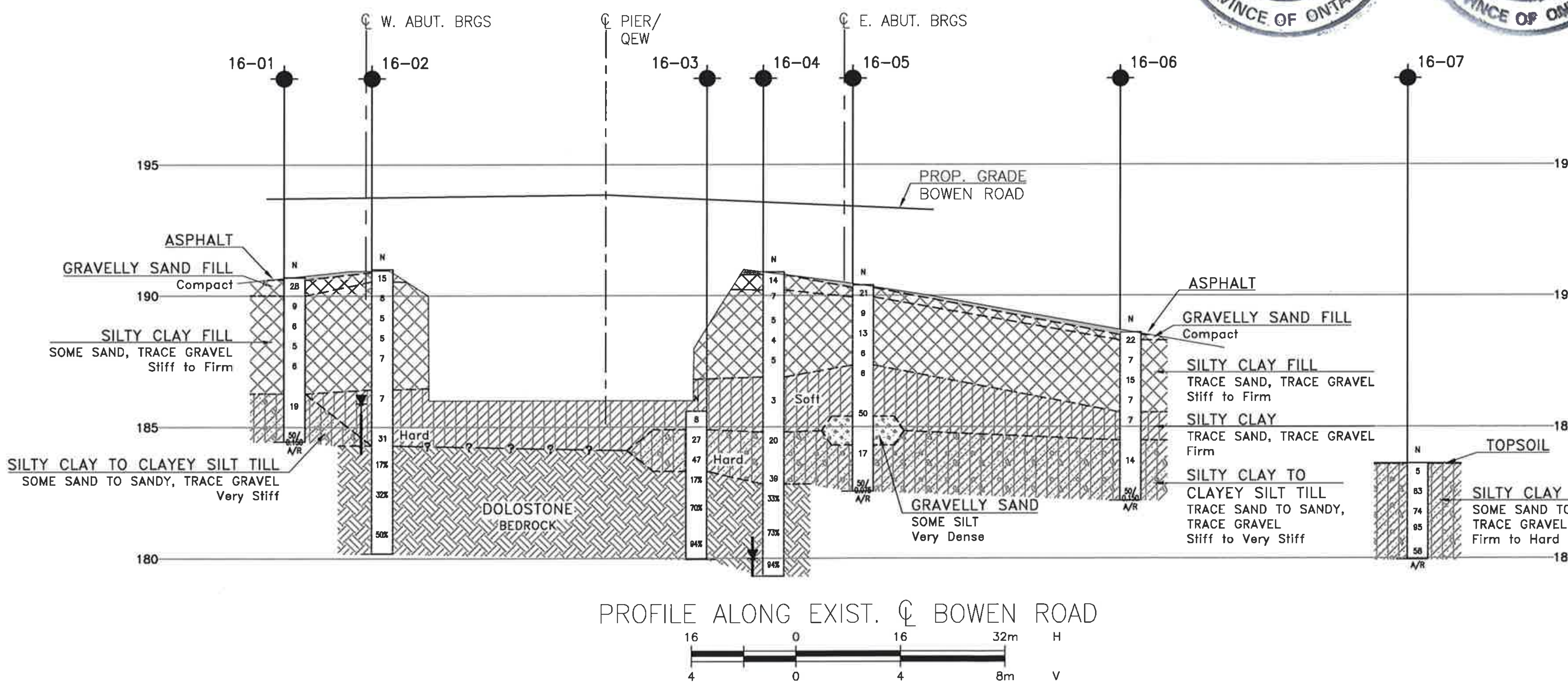
THURBER ENGINEERING LTD.



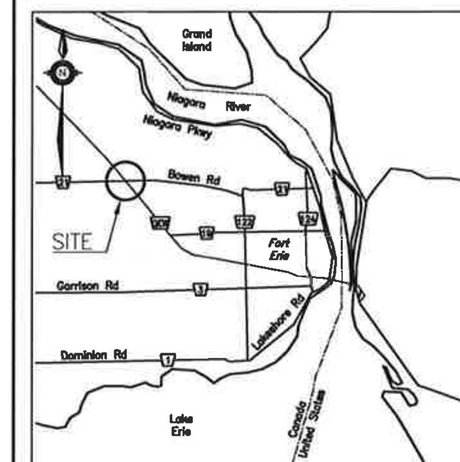
PLAN

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
Jan 20/17
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
S. PANG
Jan 20/2017
PROVINCE OF ONTARIO



PROFILE ALONG EXIST. ℄ BOWEN ROAD



KEYPLAN

LEGEND

	Borehole (Current Investigation)
	Borehole (Previous Investigation)
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
08-01	185.7	4 754 977.7	346 915.2
08-02	185.3	4 754 978.5	346 942.7
08-03	185.0	4 754 988.0	346 967.3
08-04	184.0	4 754 976.7	347 005.6
08-05	185.0	4 754 975.3	347 022.8
16-01	190.7	4 754 992.3	346 919.1
16-02	191.0	4 754 993.0	346 932.4
16-03	185.6	4 754 966.0	346 983.5
16-04	190.9	4 754 993.8	346 992.1
16-05	190.4	4 754 994.3	347 005.8
16-06	188.6	4 754 995.3	347 046.7
16-07	183.6	4 754 977.4	347 090.6

-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. 30L15-15

REVISIONS	DATE	BY	DESCRIPTION		
DESIGN RPR	CHK SKP	CODE	LOAD		DATE JAN 2017
DRAWN AN	CHK RPR	SITE	STRUCT	DWG 1	

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PLOTDATE: 1/17/2017 5:20 PM

MINISTRY OF TRANSPORTATION, ONTARIO

METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

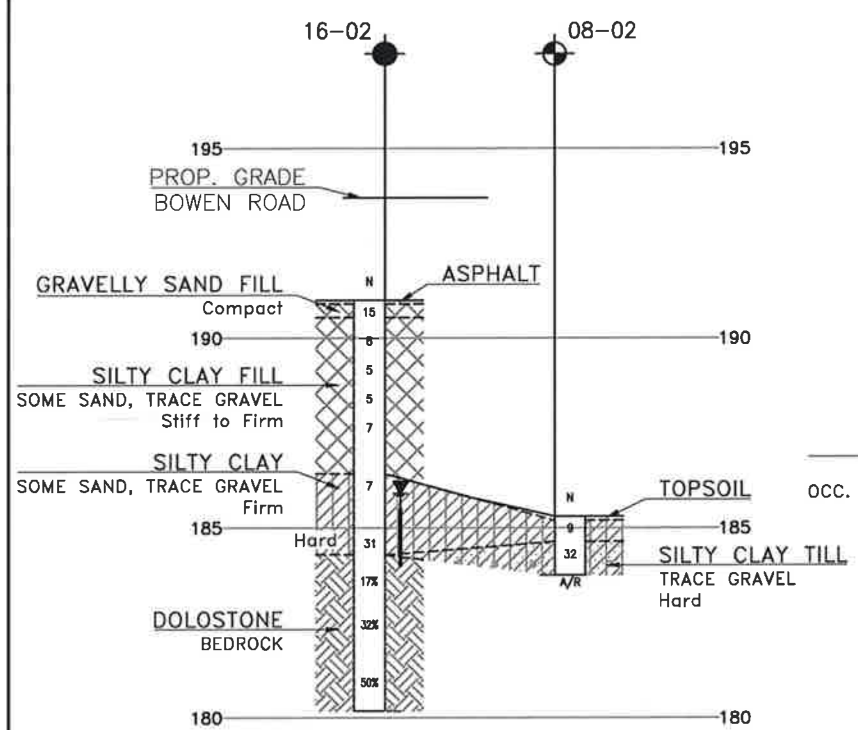
CONT No
WP No 2482-04-00

BOWEN ROAD UNDERPASS AT QEW
BRIDGE REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA

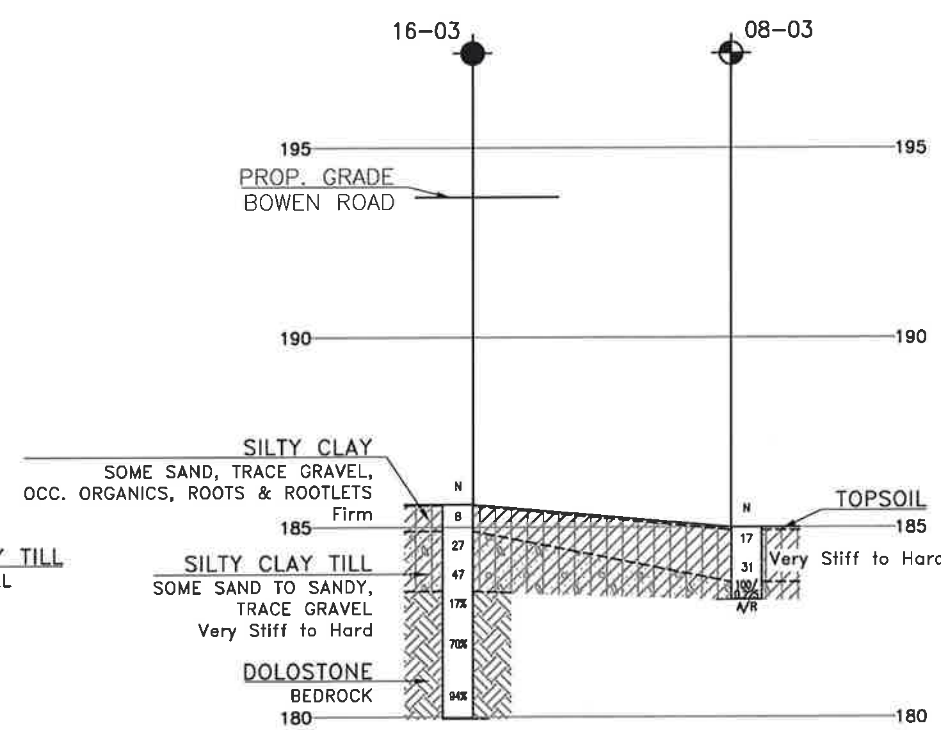
SHEET

AECOM

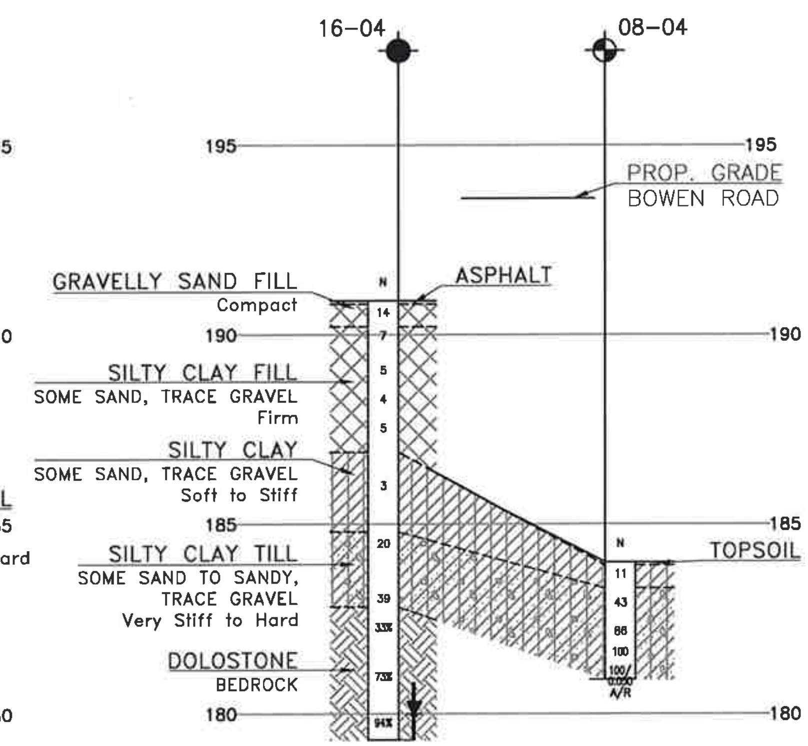
THURBER ENGINEERING LTD.



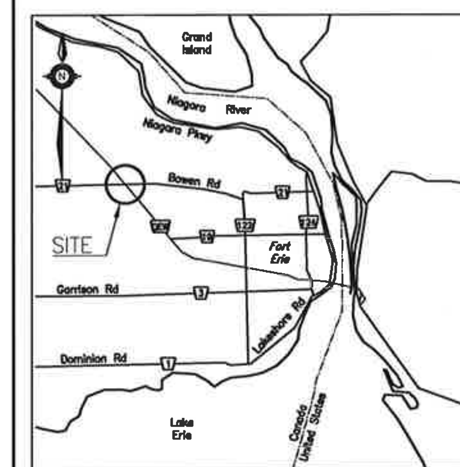
SECTION ALONG W. ABUT BRGS



SECTION ALONG PIER



SECTION ALONG E. ABUT BRGS



KEYPLAN

LEGEND

	Borehole (Current Investigation)
	Borehole (Previous Investigation)
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
08-01	185.7	4 754 977.7	346 915.2
08-02	185.3	4 754 978.5	346 942.7
08-03	185.0	4 754 988.0	346 967.3
08-04	184.0	4 754 976.7	347 005.6
08-05	185.0	4 754 975.3	347 022.8
16-01	190.7	4 754 992.3	346 919.1
16-02	191.0	4 754 993.0	346 932.4
16-03	185.6	4 754 966.0	346 983.5
16-04	190.9	4 754 993.8	346 992.1
16-05	190.4	4 754 994.3	347 005.8
16-06	188.6	4 754 995.3	347 046.7
16-07	183.6	4 754 977.4	347 090.6

-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. 30L15-15

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
Jan 20/17
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
S. PANG
Jan 20/2017
PROVINCE OF ONTARIO

REVISIONS

DATE	BY	DESCRIPTION

DESIGN RPR	CHK SKP	CODE	LOAD	DATE JAN 2017
DRAWN AN	CHK RPR	SITE	STRUCT	DWG 2

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Appendix D

Record of Borehole Sheets
(previous investigation)

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 08-01

METRIC

G.W.P.	2482-04-00	LOCATION	N 4 754 977.7 E 346 915.2	ORIGINATED BY	SLL
HWY	Q.E.W.	BOREHOLE TYPE	Solid Stem Auger	COMPILED BY	AN
DATUM	Geodetic	DATE	2008.11.21 - 2008.11.21	CHECKED BY	DEE

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 (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X 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
185.7	Geodetic										
0.0	**TOPSOIL** (50mm)										
0.0	Silty **CLAY**, trace roots, trace rootlets Stiff Brown (CL)		1	SS	10						3 27 42 27
185.0 / 0.7	Silty **CLAY**, trace gravel, trace shale fragments Very Hard Reddish Brown (TILL) (CL)		2	SS	57/ 0.200		185				
184.5 / 1.2	END OF BOREHOLE (SAMPLER BOUNCING) AT 1.2m ON POSSIBLE BEDROCK. BOREHOLE OPEN AND DRY UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH HOLEPLUG BENTONITE, MIXED WITH CUTTINGS TO SURFACE.										

ONTMT4S 7455.GPJ 11/3/09

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

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

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 08-02

METRIC

G.W.P.	2482-04-00	LOCATION	N 4 754 978.5 E 346 942.7	ORIGINATED BY	SLL
HWY	Q.E.W.	BOREHOLE TYPE	Solid Stem Auger	COMPILED BY	AN
DATUM	Geodetic	DATE	2008.11.17 - 2008.11.17	CHECKED BY	DEE

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	NATURAL MOISTURE CONTENT (%)	UNIT WEIGHT kN/m3	GR	SA	SI	CL
185.3	Geodetic											
0.0	**TOPSOIL** (100mm)											
0.1	Silty **CLAY**, trace roots and rootlets Stiff Brown (CL)	1	SS	9		185						
184.6 / 0.7	Silty **CLAY**, trace gravel Hard Brown (TILL) (CL)	2	SS	32		184			2	21	47	29
183.7 / 1.5	END OF BOREHOLE AT 1.5m UPON AUGER REFUSAL ON POSSIBLE BEDROCK. BOREHOLE OPEN AND DRY UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH AUGER CUTTINGS AND HOLEPLUG TO SURFACE.											

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

REMARKS & GRAIN SIZE DISTRIBUTION (%)

+ 3, × 3: Numbers refer to Sensitivity

15-⊕-5 (20 above, 10 below) (%) STRAIN AT FAILURE

ONTMT4S 7455.GPJ 11/3/09

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 08-03

METRIC

G.W.P. 2482-04-00	LOCATION N 4 754 988.0 E 346 967.3		ORIGINATED BY SLL
HWY Q.E.W.	BOREHOLE TYPE Soild Stem Auger		COMPILED BY AN
DATUM Geodetic	DATE 2008.11.21 - 2008.11.21		CHECKED BY DEE

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 (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X 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
185.0	Geodetic										
0.0	**TOPSOIL** (75mm)						185				
0.1	Silty **CLAY**, trace gravel, trace roots and roollets Very Stiff Brown (CL)		1	SS	17				o		
			2	SS	31		184		o		
183.5 1.4	Silty **CLAY**, some sand, trace gravel Very Hard Brown (TILL) (CL)		3	SS	100/ 0.225				o—I		6 42 38 14
183.1 1.9	END OF BOREHOLE AT 1.9m UPON AUGER REFUSAL ON POSSIBLE BEDROCK. BOREHOLE OPEN AND DRY UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG, MIXED WITH CUTTINGS TO SURFACE.										

ONTMT4S 7455.GPJ 11/3/09

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

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 08-04

METRIC

G.W.P. 2482-04-00	LOCATION N 4 754 976.7 E 347 005.6		ORIGINATED BY SLL
HWY Q.E.W.	BOREHOLE TYPE Soild Stem Auger		COMPILED BY AN
DATUM Geodetic	DATE 2008.11.21 - 2008.11.21		CHECKED BY DEE

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
184.0	Geodetic				184		
0.0	**TOPSOIL** (75mm)						
0.1	Silty **CLAY**, trace gravel, trace roots and rootlets Stiff Brown (CL)	1	SS	11		o	
183.3 / 0.7	Silty **CLAY**, trace gravel, shale fragments Hard Reddish Brown (TILL) (CL)	2	SS	43	183	o	
		3	SS	86		o ├─┤	4 26 42 28
					182		
		4	SS	100		o	
					181		
180.9		5	SS	100/ 0.050		o	
3.1	END OF BOREHOLE AT 3.1m UPON AUGER REFUSAL ON POSSIBLE BEDROCK. BOREHOLE OPEN TO 2.9m AND DRY UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG, MIXED WITH AUGER CUTTINGS TO SURFACE.						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X 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

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

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

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 08-05

METRIC

G.W.P. 2482-04-00 LOCATION N 4 754 975.3 E 347 022.8 ORIGINATED BY SLL

HWY Q.E.W. BOREHOLE TYPE Soild Stem Auger COMPILED BY AN

DATUM Geodetic DATE 2008.11.21 - 2008.11.21 CHECKED BY DEE

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
185.0	Geodetic				185		
0.0 / 0.0	**TOPSOIL** (50mm)						
	Silty **CLAY**, trace gravel, shale fragments, Very Stiff, Brown (CL)	1	SS	24		o	2 24 43 31
184.3 / 0.7	Silty **CLAY**, trace gravel, shale fragments, Very Hard, Reddish Brown (TILL) (CL)	2	SS	50	▼ 184	o	
		3	SS	80	183	o	
182.8 / 2.2	END OF BOREHOLE AT 2.2m UPON AUGER REFUSAL. BOREHOLE OPEN AND DRY UPON COMPLETION OF DRILLING. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.						

WATER LEVEL READINGS:

DATE	DEPTH (m)	ELEV. (m)
11/28/08	1.1	183.9

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

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

Appendix E

Comparison of Foundation Alternatives

COMPARISON OF FOUNDATION ALTERNATIVES FOR FOUNDATION ELEMENTS

Foundation Element	Steel Piles socketted in bedrock	Augered Caissons	Spread Footing on bedrock Spread Footing on Engineered Fill on bedrock
	Advantages: • High capacity for piles seating on bedrock • Relatively straightforward installation • Construction of piles could continue in freezing weather. • Foundation construction requires less excavation than footings. ***Disadvantages:*** • Higher cost than spread footings • Piles must be socketed to sufficient depth within bedrock. • Concreting or grouting of the annular space within the pile socket is required.	***Advantages:*** • High bearing capacity on bedrock • Reduced requirements for excavation and roadway protection • Less disruption to traffic particularly at the piers. • Sub-excavation of fill and variable material not required. ***Disadvantages:*** • Higher cost than spread footings • Installation through bedrock; difficult installation if socketted into bedrock. • Dewatering may be required • Potential difficulty in cleaning and inspecting bases.	***Advantages:*** • High bearing capacity on bedrock • Relatively straightforward installation • Frost protection not required for footings on rock ***Disadvantages:*** • Requires excavation and roadway protection during construction • Mass concrete fill may be required to provide a level working platform.
West Abutment	**Recommended**	**Feasible**	**Recommended**
Pier	**Not Recommended**	**Recommended**	**Recommended**
East Abutment	**Recommended**	**Feasible**	**Feasible**

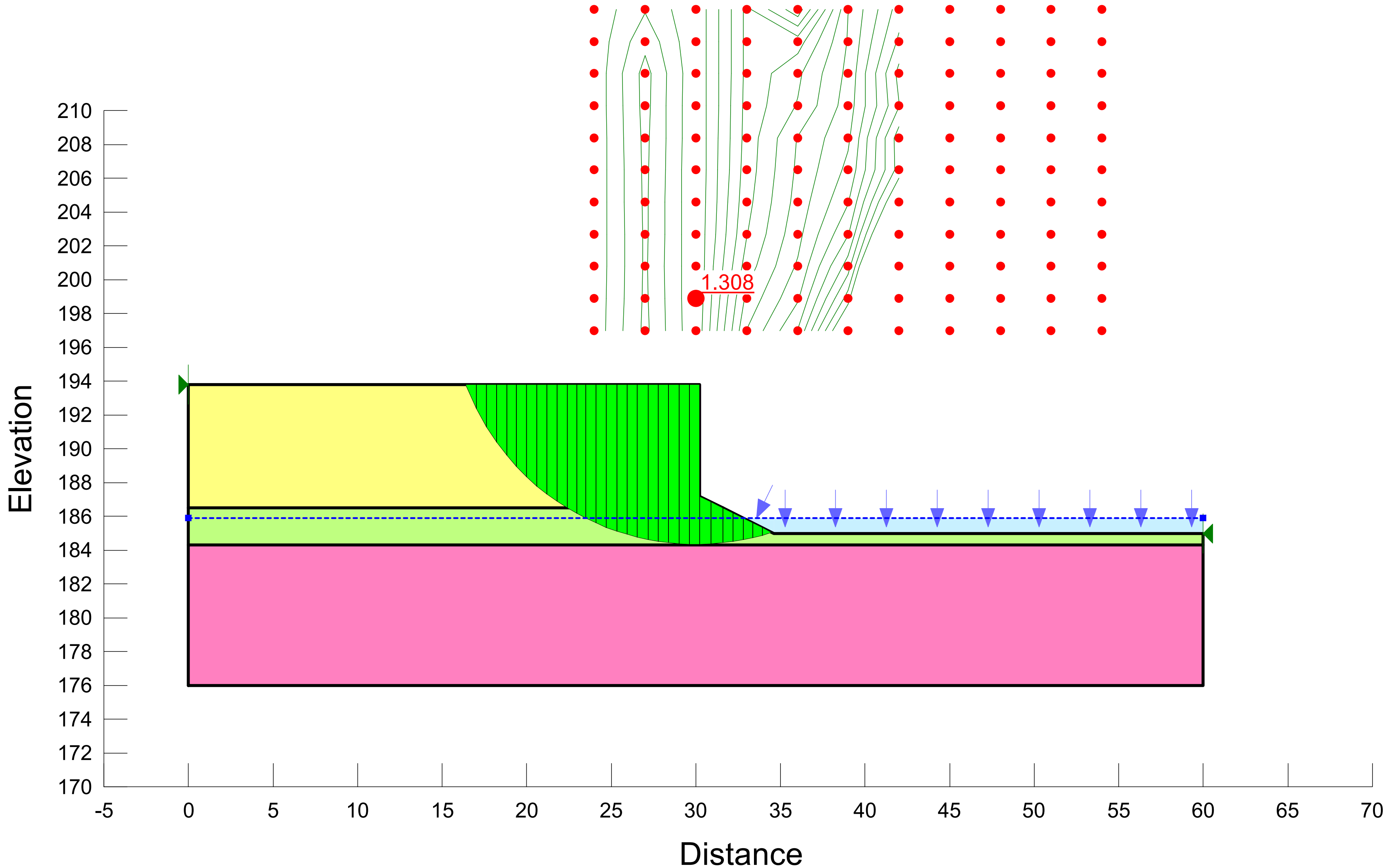
THURBER

Appendix F

Selected Slope Stability Analysis Results

14515
Bowen Road Underpass at QEW, Replacement Bridge
RSS Wall, 6.6 m high
West abutment, Forward Slope
Drained Analysis

Name: Stiff to firm silty clay FILL Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 30 ° Phi-B: 0 ° Piezometric Line: 1
Name: Firm Silty Clay Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 28 ° Phi-B: 0 ° Piezometric Line: 1
Name: Dolostone Bedrock Unit Weight: 24 kN/m³ Cohesion: 2000 kPa Phi: 0 ° Phi-B: 0 ° Piezometric Line: 1
Name: Retaining wall Unit Weight: 0.01 kN/m³ Cohesion: 200 kPa Phi: 45 ° Phi-B: 0 ° Piezometric Line: 1
Name: Structture Unit Weight: 25 kN/m³ Cohesion: 60000 kPa Phi: 0 ° Phi-B: 0 ° Piezometric Line: 1



2016-10-26
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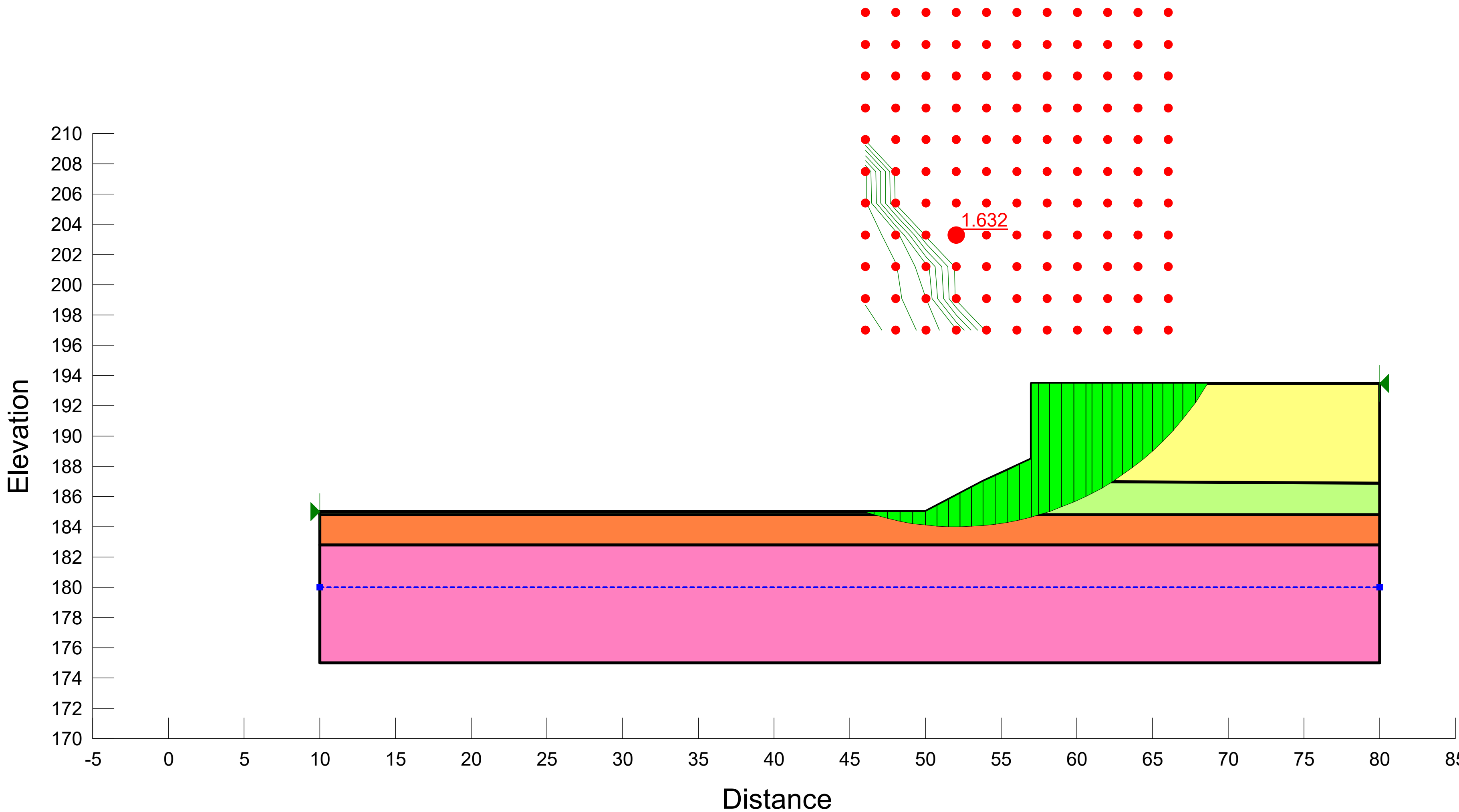
Figure F1

14515
Bowen Road Underpass at QEW, Replacement Bridge
East Abutment, Forward Slope
RSS Wall - 5 m high
Drained Analysis

Name	Unit Weight	Cohesion	Phi	Phi-B	Piezometric Line
Stiff to firm silty clay FILL	20 kN/m³	0 kPa	30 °	0 °	1
Firm Silty Clay	20 kN/m³	0 kPa	28 °	0 °	1
Dolostone Bedrock	24 kN/m³	2000 kPa	0 °	0 °	1
Retaining wall	0.01 kN/m³	300 kPa	45 °	0 °	1
Silty Clay Till	21 kN/m³	0 kPa	32 °	0 °	1
Structure	25 kN/m³	60000 kPa	0 °	0 °	1



2016-10-26
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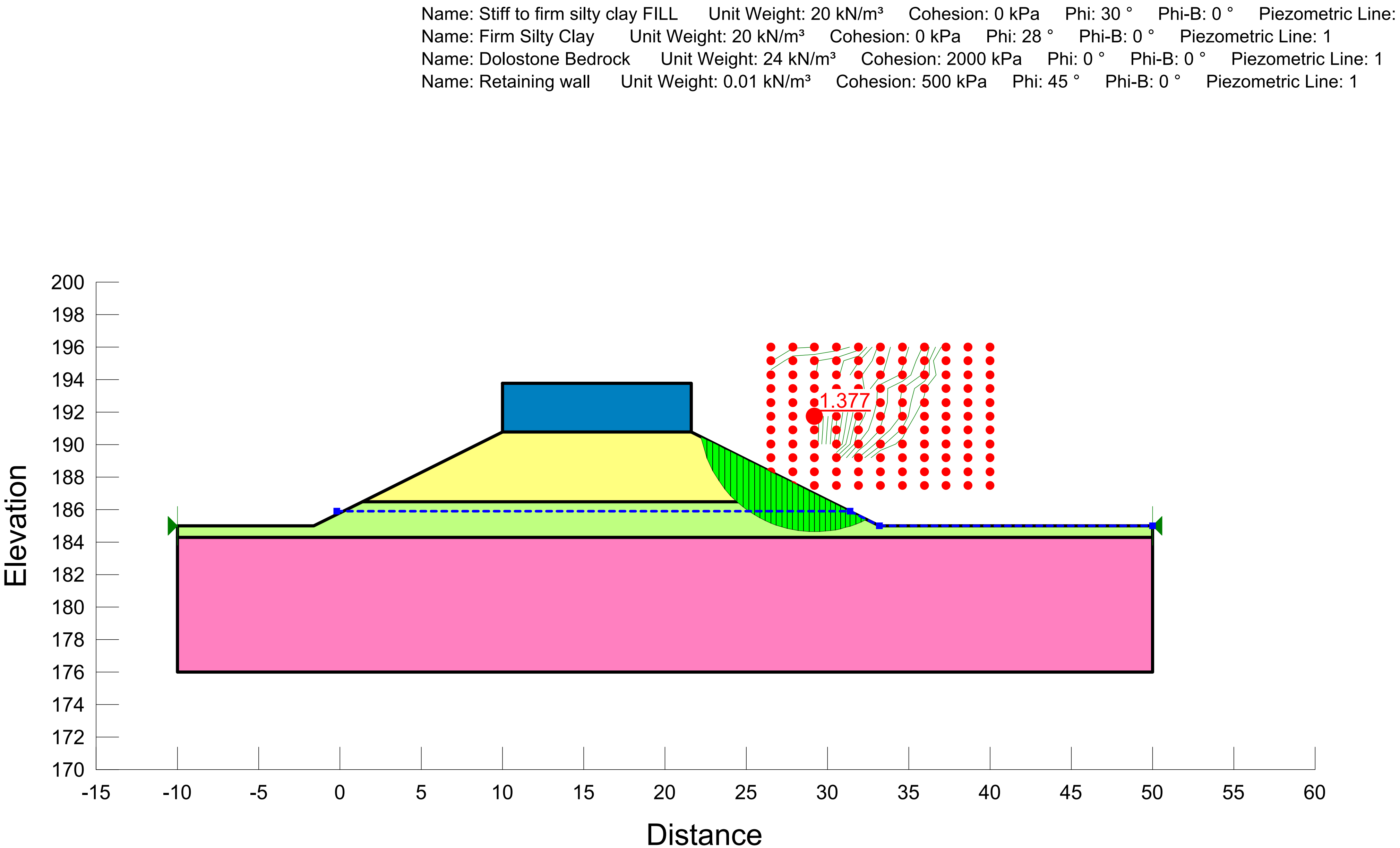
Figure F2

14515
Bowen Road Underpass at QEW, Replacement Bridge
RSS Wall, 3.0 m high
West Approach
Drained Analysis

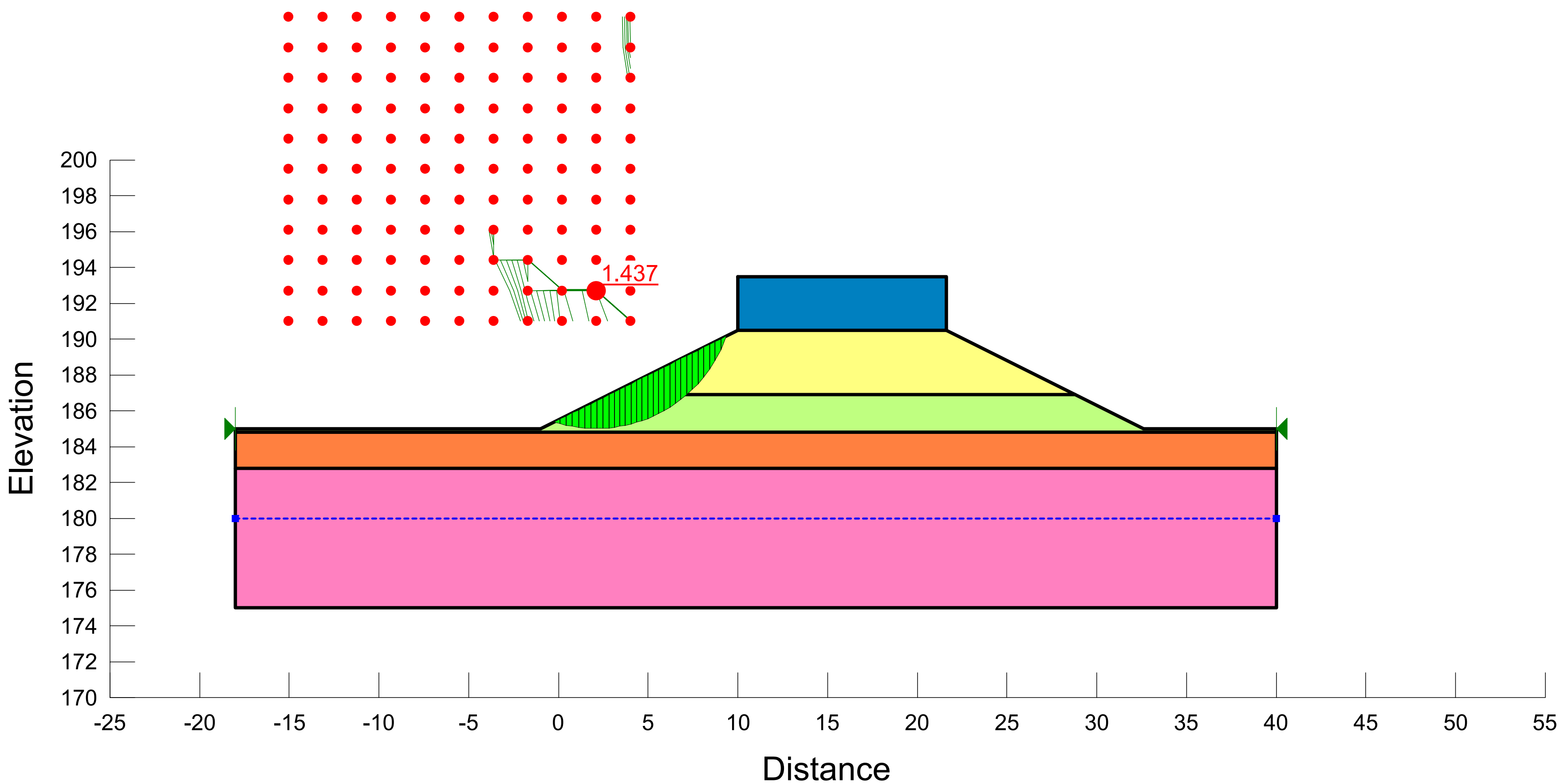


2016-10-26
H:\14000-14999\14515 QEW Bowen Road Interchange\Analysis\Bowen Rd\slope stability\Bowen Rd. - West Approach - drained.gsz

Figure F3

14515
Bowen Road Underpass at QEW, Replacement Bridge
RSS Wall, 3.0 m high
East Approach
Drained Analysis

Name: Stiff to firm silty clay FILL Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 30 ° Phi-B: 0 ° Piezometric Line: 1
Name: Firm Silty Clay Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 28 ° Phi-B: 0 ° Piezometric Line: 1
Name: Dolostone Bedrock Unit Weight: 24 kN/m³ Cohesion: 2000 kPa Phi: 0 ° Phi-B: 0 ° Piezometric Line: 1
Name: Retaining wall Unit Weight: 0.01 kN/m³ Cohesion: 300 kPa Phi: 45 ° Phi-B: 0 ° Piezometric Line: 1
Name: Silty Clay Till Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 32 ° Phi-B: 0 ° Piezometric Line: 1



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H:\14000-14999\14515 QEW Bowen Road Interchange\Analysis\Bowen Rd\slope stability\Bowen Rd. - slope - RSS Wall - East Approach.gsz

Figure F4

Appendix G

List of SPs and OPSS, and Suggested Text for NSSP

1. **List of Special Provisions and OPSS Documents Referenced in this Report**

- OPSS.PROV 903
- OPSS.PROV 501
- OPSS.PROV 804
- OPSS 902
- OPSS.PROV 539
- OPSS.PROV 206
- OPSD 3102.100
- OPSD 3101.150
- OPSD 208.010

2. Suggested text for a NSSP on "Construction of Socketed Piles"

Installation of socketed H-piles and caissons shall be in accordance with OPSS 903.

Socketted H-piles installation at this site will require excavation through cohesive soils below the groundwater table and construction of sockets in the underlying bedrock. Bedrock is present at Elevations 180.7 and 179.8 at the west and east abutments. The Contractor is advised of the following:

- The installation methods and equipment must be capable of dislodging, removing or otherwise penetrating cobbles and boulders in the soils overlying the bedrock.
- The bedrock consists of strong to very strong dolostone bedrock. The strength and hardness of this rock must be taken into account when selecting equipment to advance the pile into rock. Equipment supplied to install the pile in rock must be capable of excavating the bedrock to the specified dimensions without disturbing or fracturing the bedrock forming the sidewalls and base of the socket. Blasting to facilitate the removal of bedrock is not permitted.
- The rock socket must be formed entirely within the bedrock below the level of any cobbles and boulders. Any length of pile above the bedrock surface will not be considered part of the specified length of rock socket.

- The annular space between the rock socket wall and pile shall be filled with 30 MPa concrete or grout to the top of the bedrock surface. The plumbness and alignment of the pile shall be maintained during concreting.

3. Suggested text for a NSSP on "Construction of Caissons"

Caisson installation shall be in accordance with OPSS 903.

Caisson installation at this site will require excavation through cohesive soils below the groundwater table and construction of sockets in the underlying bedrock. The Contractor is advised of the following:

- Measures must be employed to maintain sidewall stability during installation of the caissons and prevent collapse soils into the rock socket. Selection of the methods and equipment employed to achieve this is the responsibility of the Contractor.
- The installation methods and equipment must be capable of dislodging, removing or otherwise penetrating cobbles and boulders in the soils overlying the bedrock.
- The bedrock consists of strong to very strong dolostone bedrock. The strength and hardness of this rock must be taken into account when selecting equipment to advance the pile into rock. Equipment supplied to install the pile in rock must be capable of excavating the bedrock to the specified dimensions without disturbing or fracturing the bedrock forming the sidewalls and base of the socket. Blasting to facilitate the removal of bedrock is not permitted.
- The rock socket must be formed entirely within the bedrock below the level of any cobbles and boulders. Any length of caisson above the bedrock surface will not be considered part of the specified length of rock socket.

4. Suggested text for a NSSP on "Bedrock Subgrade Preparation"

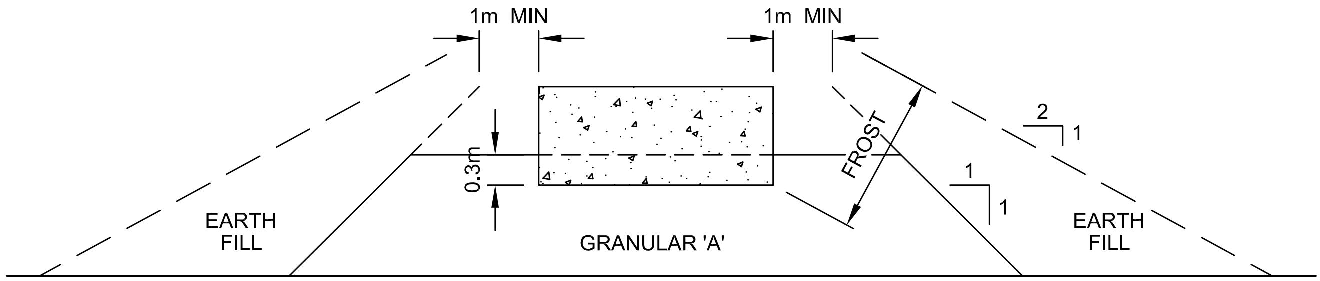
At the abutments, it is anticipated that approximately 2 to 4 m of the native silty clay and silty clay to clayey silt till will be excavated and removed to expose the dolostone bedrock. The exposed bedrock subgrade shall be inspected by a geotechnical engineer to confirm that the exposed surface conforms with the design requirements. Any loose or shattered rock must be removed and the engineered granular pad be founded on undisturbed dolostone bedrock. For uneven bedrock surfaces, mass concrete with an unconfined compressive strength of at least 30 MPa may be used to form a level working platform.

5. Suggested text for a NSSP on "Engineered Fill Pad Construction"

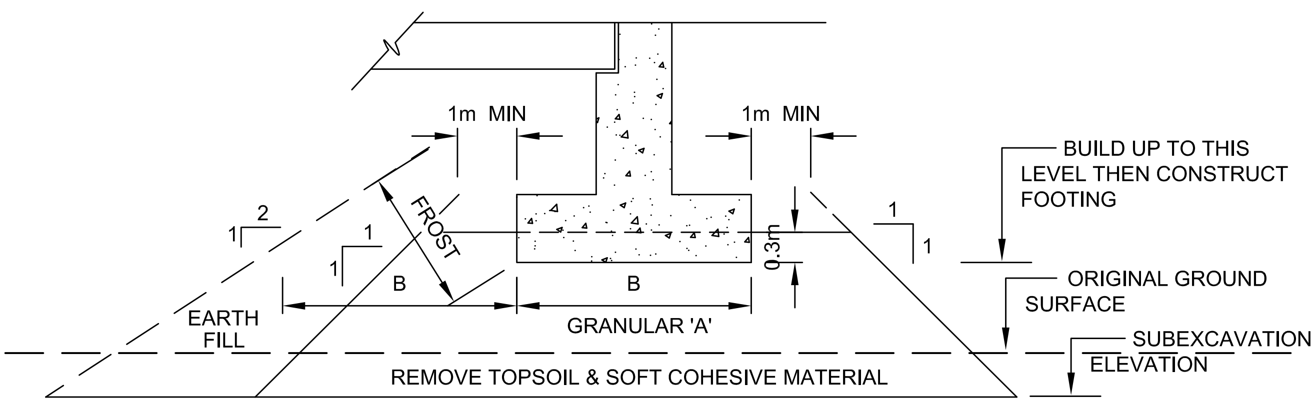
Placement and compaction of the Granular A pad must be carried out in the dry. The MTO standard "Abutment on Compacted Fill Showing Granular A Core" shall be followed. The Granular A shall comply with OPSS.PROV 1010 requirements, be placed in thin lifts and each lift be compacted to 100% of its Standard Proctor Maximum Dry Density within ±2% of its optimum moisture content as per OPSS.PROV 501. The entire fill construction operation shall be witnessed, tested and approved by experienced geotechnical personnel on a full time basis.

Appendix H

Abutment on Compacted Fill Showing Granular A Core



CROSS-SECTION



LONGITUDINAL SECTION

NOT TO SCALE

NOTES:

1. REMOVE TOPSOIL AND SOFT SILTY CLAY SUBSOIL UNDER FOOTPRINT OF COMPACTED GRANULAR 'A' AND EARTH FILL.
2. PLACE GRANULAR 'A' AND EARTH FILL TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO O.P.S.S. 501.
3. CONSTRUCT CONCRETE FOOTING.
4. PLACE REMAINDER OF GRANULAR 'A' AND EARTH FILL AS REQUIRED.
5. SOURCE M.T.C. 1982.

ABUTMENT ON COMPACTED FILL SHOWING GRANULAR A CORE

THURBER ENGINEERING LTD.

ENGINEER :	DRAWN :	APPROVED :
SKP	AN	PKC
DATE :	SCALE :	DRAWING No.
JANUARY 2017	N.T.S	FIGURE H1

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