

**PRELIMINARY**  
**FOUNDATION INVESTIGATION AND DESIGN REPORT**  
**ETOBICOKE CREEK BRIDGE REPLACEMENT**  
**QUEEN ELIZABETH WAY, ETOBICOKE, ONTARIO**  
**G.W.P. 09-20003, SITE No. 37-784**

**GEOCRES Number: 30M11-252**

**Report to**

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July 14, 2015  
File: 19-1351-219

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**PART 1: FACTUAL INFORMATION**

**1 INTRODUCTION**

This report presents the factual findings obtained from a preliminary foundation investigation carried out at the location of the proposed replacement of the structure carrying the Queen Elizabeth Way (QEW) over Etobicoke Creek in Etobicoke, Ontario. This investigation was carried out in support of the preliminary design, environmental assessment and developing alternatives for QEW improvements from Evans Avenue to Cawthra Road, approximately 3.5 km in length.

The purpose of the investigation was to explore the subsurface conditions at the site and, based on the data obtained, to provide borehole location and soil strata drawings, records of boreholes, laboratory test results and a written description of the subsurface conditions. A model of the subsurface conditions was developed from the data obtained from the present investigation.

The existing subsurface information contained in the MTO Foundation Investigation Reports dated 1966 prepared for the widening of the Etobicoke Creek Bridge (then designated as Bridge #21, Geocres No. 31M11-20) and for the Ramp W-Evans Ave. (Bridge #22, Geocres No. 31M11-26) have been reviewed and incorporated in this report to supplement the subsurface information obtained during the present investigation.

Thurber was retained by MMM Group to carry out the preliminary foundation investigation at this site on behalf of the Ministry of Transportation Ontario (MTO) under Consultant Assignment No. 2008-E-0075.

**2 SITE DESCRIPTION**

The existing Etobicoke Creek Bridge carrying Eastbound and Westbound lanes of the QEW is located approximately 1.3 km west of the intersection of QEW with Highway 427 and 1.2 km east of the existing Dixie Road Underpass. The original bridge was built in 1936, and was widened twice; first an arch centre span, then with slab on girders on both sides. The length of the bridge of 75.1 m (end to end wingwalls) and width of 49.6 m are indicated on the General Arrangement drawing dated January 1967, prepared for the bridge widening. The bridge is a three span structure with the approach spans on each side. The existing approach embankments are approximately 7.5

m in height. At this location, QEW travels in the northeast to southwest direction; for the purpose of this report, the direction of east – west has been assumed.

The Etobicoke Creek flows southerly through a broad valley into Lake Ontario. The Etobicoke Creek valley is mostly treed on the west side of the creek. The area east of the creek and south of QEW, is occupied by parkland with a number of baseball diamonds. Beyond the valley crest in the NW, SW and SE quadrants, the land has been developed in residential subdivisions. In the NE quadrant, there is an area of vacant land that is believed to contain a closed landfill site. Beyond this area, there are institutional (health care), commercial and retail developments.

Photographs of the bridge and surrounding area are presented in Appendix C.

From published geological information, the site is situated within the physiographic region known as the Iroquois Plain. In this area, the soil deposits are relatively thin and typically comprising from sands to cohesive soils overlying shale bedrock of the Georgian Bay Formation. The shallow soils are known to contain fragments of shale and limestone.

### **3 SITE INVESTIGATION AND FIELD TESTING**

The site investigation and field testing comprised of four boreholes denoted EC14-01 to EC14-04. The field work was carried out on August 7, 2014 when boreholes EC14-01 and EC14-02 were drilled on each side of the east abutment from the ground level, and between December 14 and 17, 2014, when Boreholes EC14-03 and EC14-04 were drilled near the west abutment from the QEW level. The approximate borehole locations are shown on the Borehole Locations and Soil Strata drawing enclosed in Appendix D.

The borehole locations were staked and/or marked in the field by Thurber. Utility clearance was obtained for the borehole locations prior to drilling. Borehole location data including northing, easting and surface elevation has been derived based on the preliminary design information provided by MMM Group to Thurber.

A rubber track-mounted drill rig was used in combination with continuous flight hollow stem augers to advance the boreholes through embankment fill into bedrock, and a track mounted drill rig was used for drilling boreholes at the east abutment. Soil samples were obtained at selected intervals using a split spoon sampler in conjunction with the Standard Penetration Testing (SPT). Boreholes were advanced to bedrock and the bedrock was proved by a minimum of 3 m coring using NQ-sized coring equipment. All rock cores were logged, and properties including Total Core Recovery (TCR), Solid Core Recovery (SCR), Rock Quality Designation (RQD) and Fracture Indices (FI) were determined, where applicable.

Groundwater conditions in the boreholes were observed throughout the drilling operations. Standpipe piezometers were installed in three out of four boreholes to permit monitoring of the groundwater levels. The standpipe piezometer typically consists of 19 mm diameter Schedule 40 PVC pipe with 1.5 m long slotted screen positioned in the stratum where groundwater fluctuations are to be monitored. The sand screen surrounded the pipe and extended at least 0.3 m above the

slotted screen. Bentonite holeplug seals were placed above the sand screen in each installation to seal the annular space. Following the final water level reading, the piezometers were decommissioned in general accordance with MOE Regulation 903.

The details of borehole completion and piezometer installation are summarized in Table 3.1.

**Table 3.1 Borehole completion and Piezometer Installation Details**

<b>Borehole Number</b>	<b>Ground Elevation</b>	<b>Borehole Termination Depth/Elevation (m)</b>	<b>Borehole Completion Details</b>
EC14-01	93.8	7.9 / 85.9	19 mm diameter piezometer with filter sand from 7.9 to 5.5 m, bentonite holeplug from 5.5 to 0.6 m, then 0.3 m of cuttings and 0.3 m of concrete to surface.
EC14-02	93.2	5.0 / 88.2	Backfilled with bentonite holeplug to 0.6 m, then cuttings to surface.
EC14-03	100.0	11.7 / 88.4	19 mm diameter piezometer with filter sand from 11.7 m to 7.9 m, bentonite holeplug from 7.9 m to 0.2, then concrete to surface.
EC14-04	100.0	11.6 / 88.5	19 mm diameter piezometer with filter sand from 11.6 m to 7.9 m, bentonite holeplug from 7.9 m to 0.3, then concrete to surface.

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 samples for transport to Thurber's laboratory for further examination and testing.

#### **4 LABORATORY TESTING**

All recovered soil samples were subjected to visual identification and natural moisture content determination. The results of the testing are shown on the Record of Borehole sheets attached in Appendix A. Selected soil samples were subjected to grain size distribution analysis. The results of this testing program are presented on the Record of Borehole sheets in Appendix A and on the Figures in Appendix B.

Selected rock cores were subjected to Point Load Testing (PLT). The results of the PLT are included in Appendix B. Unconfined compressive strengths (UCS) of the rock cores correlated from the PLT results are enclosed in the Record of Borehole sheets in Appendix A.

#### **5 DESCRIPTION OF SUBSURFACE CONDITIONS**

Reference is made to the Record of Borehole sheets in Appendix A. Details of the encountered soil stratigraphy are presented in these records and on the Borehole Locations and Soil Strata drawing in Appendix D. The existing subsurface information contained in the MTO Foundation Investigation Reports prepared for the widening of the Etobicoke Creek Bridge (then Bridge #21,

Geocres No. 31M11-20) and for the Ramp W-Evans Ave. (Bridge #22, Geocres No. 31M11-26) both dated 1966, have been reviewed and the Record of Borehole sheets and drawings showing borehole locations are enclosed in Appendix E for reference. The results presented in those reports are in general agreement with the results of the present investigation.

A general description of the subsurface conditions encountered in the present investigation is given in the following paragraphs. The factual information established at the borehole locations governs any interpretation of the site conditions.

In general, all boreholes drilled at this site encountered fill materials extending to bedrock surface or concrete foundations. The embankment fill was encountered extending to as much as 7.6 m depth at the west abutment. The fill materials in close proximity to the east abutment but outside of the embankment varied from 1.0 m to 1.5 m in thickness. A grey shale bedrock of Georgian Bay Formation was encountered at approximately Elev. 92 in the vicinity of the east abutment. On the west side, each borehole encountered concrete, which was believed to be the west wingwall footing placed on shale bedrock between Elev. 91.5 and Elev. 91.7.

### **5.1 Topsoil**

Boreholes EC14-01 and EC14-02 were advanced from the ground surface on both sides of the east abutment and encountered 175 mm and 150 mm of topsoil.

### **5.2 General Fill/Reworked Native Soils**

Fill materials were encountered underlying the topsoil in Boreholes EC14-01 and EC14-02 drilled in close proximity to the east bridge abutment. The fill materials were 1 m to 1.5 m in thickness and varied from sand to clayey silt with sand. It is believed that these fills are native soil deposits reworked during the construction carried out at the bridge site.

The upper 0.7 m of fill in Borehole EC14-01 consisted of sand with some silt, trace clay and trace topsoil. The sand fill was moist, compact and extended to 0.9 m depth (Elev. 92.9).

Underlying the sand fill in Borehole EC14-01 and topsoil in Borehole EC14-02 was clayey silt and sand fill. Trace gravel, occasional concrete and limestone fragments were noted in this fill. This fill was 0.8 m and 1 m thick and was stiff to hard. The fill extended to the bedrock surface at 1.7 m depth (Elev. 92.1) and 1.2 m depth (Elev. 92.0) in Boreholes EC14-01 and EC14-02, respectively. The results of grain size analysis conducted on a clayey silt and sand fill sample is presented on Figure B1 in Appendix B, indicating that the fill contains:

Soil Particles	Percentage (%)
Gravel	2
Sand	46
Silt	34
Clay	18

The measured natural moisture contents of the samples of the clayey silt with sand fill ranged from 9% to 15%.

### 5.3 Asphalt Pavement and Concrete Slab

Boreholes EC14-03 and EC14-04 were advanced through the west approach slab of the QEW. The boreholes encountered 150 mm and 113 mm of asphalt at the surface. Asphalt pavement was underlain by 300 mm of concrete in both boreholes.

### 5.4 Embankment Fill

Brown gravelly sand to sand and gravel with some silt and trace clay, underlies the asphalt and approach slab in Boreholes EC14-03 and EC14-04. Occasional cobbles were noted in this fill. Thickness of the gravelly sand/sand and gravel fill varied from 5.0 m to 5.7 m, and the fill base was encountered between depths of 5.5 m and 6.1 m (Elevation 94.6 and 93.9 m) in Boreholes EC14-03 and EC14-04.

Standard Penetration Test (SPT) conducted within this fill produced 'N' values of 16 to 63 blows per 0.3 m of penetration indicating a compact to very dense relative density. The measured natural moisture content of the fill samples ranged from 5% to 9%.

Results of grain size analyses conducted on selected samples of gravelly sand/sand and gravel fill are presented on the Record of Borehole sheets, on Figure B2 in Appendix B, and are summarized below:

Soil Particles	Percentage (%)
Gravel	24 to 45
Sand	42 to 58
Silt and Clay	11 to 18

The gravelly sand fill in Borehole EC14-03 is underlain by 1.8 m of sand fill. The sand fill contains some silt and trace gravel and is very loose. The results of a grain size analysis conducted on this fill sample is presented on Figure B2 in Appendix B, indicating that the sand fill contains:



Soil Particles	Percentage (%)
Gravel	9
Sand	66
Silt and Clay	25

The sand and gravel fill in Borehole EC14-04 is underlain by 1.5 m of clayey silt fill, which contains some sand and trace gravel. Frequent limestone fragments were encountered in the lower 0.5 m zone of this fill. Standard Penetration Tests (SPT) conducted within the clayey silt fill resulted in 'N' value of 10 blows per 0.3 m penetration, indicating stiff consistency. The measured natural moisture contents of the samples were 6% and 15%.

The results of a grain size analysis conducted on a clayey silt fill sample is presented on Figure B1 in Appendix B, indicating that the fill contains:

Soil Particles	Percentage (%)
Gravel	6
Sand	13
Silt	59
Clay	22

The embankment fill extended to the top of the concrete foundations at 7.3 m depth (Elev. 92.7) in Borehole EC14-03 and 7.6 m depth (Elev. 92.4) in Borehole EC14-04.

### 5.5 Concrete Footings

Beneath the approach slab and embankment fill, Boreholes EC14-03 and EC14-04 encountered concrete, which is believed to be part of the footings of the bridge wingwalls. The boreholes were advanced through the concrete by coring for 1 m and 0.9 m depth, respectively.

The underside of the footing was encountered at 8.3 m depth (Elev. 91.7) and 8.5 m depth (Elev. 91.5), in Borehole EC14-03 and EC14-04, respectively.

### 5.6 Bedrock

Shale bedrock of the Georgian Bay Formation was encountered below the general fill materials at depths of 1.7 m and 1.2 m (Elev. 92.1 and 92.0) in Borehole EC14-01 and EC14-02, and underlying the wingwall footings at 8.3 m and 8.5 m depth (Elev. 91.7 and 91.5) in Borehole EC14-03 and EC14-04. Bedrock was proved by coring for length of between 3.1 m to 5.2 m. The bedrock was described as highly to slightly weathered, thinly

bedded, weak grey shale with occasional strong to very strong limestone interbeds up to 125 mm in thickness. The shale bedrock was highly weathered in the upper 1 m zone in Borehole EC14-01, where the borehole was advanced by augering and SPT testing. The highly weathered zone in Borehole EC14-01 extended to Elev. 91.1. With the exception of the highly weathered 1 m zone in EC14-01, the bedrock in the remaining boreholes was described as moderately to slightly weathered. Occasional clay seams were noted at various depths in the shale bedrock.

Total Core Recovery (TCR) of the bedrock ranged from 58% to 100%. The Solid Core Recovery (SCR) ranged from 48% to 100%. The Rock Quality Designation (RQD) values ranged typically from 33% to 93% indicating poor to excellent rock quality. One RQD value of 13% was obtained for Run 2 in Borehole EC14-01 indicating a very poor rock quality.

Fracture Index (FI) of the rock cores indicating frequency of natural fractures per 0.3 m of core run, ranged from 0 to more than 10, typically being less than 7. Photographs of the recovered rock cores and the results of the Point Load tests are enclosed in Appendix B.

The depths to bedrock and the bedrock surface elevations encountered in the boreholes are summarized in the following table.

**Table 5.1      Depths and Elevations of Bedrock Surface**

<b>Borehole</b>	<b>Depth to Bedrock below Ground Surface (m)</b>	<b>Bedrock Elevation (m)</b>
EC14-01	1.7	92.1
EC14-02	1.2	92.0
EC14-03	8.3 <sup>1</sup>	91.7 <sup>1</sup>
EC14-04	8.5 <sup>1</sup>	91.5 <sup>1</sup>

<sup>(1)</sup> Depths and elevations of bedrock below the concrete footings; boreholes drilled from the top of the embankment.

Unconfined compressive strengths (UCS) of the intact rock cores, estimated from the results of point load tests (PLT) conducted on the shale samples, ranged from 4.4 MPa to 18.5 MPa, indicating very weak to weak intact rock. The upper bound UCS value suggests the presence of limestone interbeds in the tested samples of shale. The USC values of 25.6 MPa to 112.1 MPa were obtained for the samples of the limestone interbeds, indicating medium strong to very strong rock. The average values of UCS for selected runs of rock cores are included on the Record of Borehole sheets.

## 5.7 Groundwater Conditions

Groundwater was observed in the boreholes during drilling in the overburden, however, water was used during drilling operations, and therefore the measured water levels may not reflect prevailing water level at this site. Standpipe piezometers were installed and sealed in bedrock at three borehole locations.

The measured groundwater levels in the piezometers are presented in the following table.

**Table 5.2 Water Level Measurements**

Borehole	Date	Ground Surface Elevation (m)	Groundwater	
			Depth (m)	Elevation (m)
EC14-01	September 29, 2014	93.8	2.1	91.7
	January 15, 2015		2.0	91.8
EC14-03	March 31, 2015	100.0	6.9	93.1
EC14-04	March 31, 2015	100.0	7.1	92.9

The groundwater levels are expected to fluctuate seasonally and after severe weather events.

The following water levels in Etobicoke Creek were indicated on the archive drawing (No. D6091-1, Contract 67-40, W.P. 174-65-3, dated January 1967) prepared for the then-bridge widening:

H.W.L 313.9 ft / 95.7 m	North (East) side (Hurricane Hazel)
310.0 ft/ 94.5 m	South (West) side (Hurricane Hazel)
N.W.L 303.0 ft / 92.4 m	Average - April 1966.

The Preliminary General Arrangement drawing dated January 2015 indicates water level in the Etobicoke Creek at Elev. 92.4 m.

## 6 MISCELLANEOUS

Walker Drilling Ltd. of Barrie, Ontario, supplied and operated a track-mounted drill rig for duration of the August 2014 investigation at the east bridge abutment. The drilling and sampling at the west bridge abutment was carried out by Determination Drilling based in Hamilton, Ontario, who supplied a rubber track-mounted drill rig for duration of the December 2014 investigation at the west bridge abutment.

Traffic protection during the drilling operation at the west abutment was provided by Direct Traffic Management Inc. of Hamilton, Ontario.

The field work was supervised on a full time basis by Mr. Stephan Loranger in August 2014 and in December 2014 by Ms. Eckie Siu of Thurber Engineering Ltd. Overall supervision of the field program was conducted by Mr. Weiss Mehdawi, P.Eng.

The report was prepared by Ms. Anna Piascik, P. Eng., and reviewed by Mr. Alastair Gorman, P.Eng. and Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

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**PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS**

**7 GENERAL**

This report provides an interpretation of the geotechnical data in the factual report and presents preliminary foundation design recommendations to assist the design team in the selection and design of a suitable foundation system for the Etobicoke Creek Bridge Replacement.

The existing Etobicoke Creek Bridge is a three span structure with approach spans on each side. The original bridge was built in 1936, and widened twice. The length of the bridge of 75.1 m (end to end wingwalls) and width of 49.6 m are indicated on the General Arrangement drawing dated January 1967, prepared for the bridge widening. Based on this drawing, the existing foundation units (abutments and piers) are founded on spread footings placed on shale bedrock. The existing approach embankments are approximately 7.5 m in height.

Based on the preliminary General Arrangement Drawings (GA) dated January 2015 provided by MMM Group, it is understood that the replacement structure will be located on the same alignment as the existing bridge with a nominal shift of the proposed bridge centreline to the north. The replacement bridge will be a three span structure with spans 17.5 m, 28.7 m and 17.5m. A total length of 78.4 m and width of 56.6 m are indicated on the Preliminary General Arrangement drawing. The proposed abutments will be located behind the existing abutments and the approaches will be 7.4 m in length. The Preliminary General Arrangement drawing indicates that the proposed bridge foundations will be supported on spread footings, which will be placed on shale bedrock at the piers and on granular pads constructed on bedrock at the abutments. The heights of the approach embankments will be consistent with the heights of the existing embankments of approximately 7.5 m.

The discussion and recommendations presented in this report are based on the information provided by MMM Group and on the factual data obtained during the course of the current investigation. The subsurface information from the existing MTO reports available for this site was reviewed and incorporated, as appropriate.

## **8 STRUCTURE FOUNDATIONS**

In general, all boreholes drilled at this site encountered fill materials extending to bedrock surface or concrete foundations. The embankment fill extended to as much as 7.6 m depth at the west abutment and the general fill considered to be native soils reworked during the bridge construction was encountered to 1.7 m depth at the east abutment. The grey shale bedrock of Georgian Bay Formation was encountered at approximate Elev. 92 m in the vicinity of the east abutment. The west wingwalls footing was placed on shale bedrock between Elev. 91.5 and 91.7 m.

The groundwater level in the standpipe piezometer installed in Borehole EC14-01 and sealed in the shale was measured at Elev. 91.8, which was slightly below the bedrock surface at that location. The water level in Etobicoke Creek was indicated at Elev. 92.4 on the General Arrangement Plan.

The groundwater level at the bridge site will fluctuate and, in general, it will be controlled by the water level in Etobicoke Creek.

### **8.1 Foundation Alternatives**

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

- Spread footings placed on shale bedrock
- Spread footings placed on the granular pad
- Steel H-piles socketed into shale bedrock, and
- Caissons socketed into shale bedrock.

Shale bedrock underlies the bridge site at shallow depths. Spread footings constructed directly on bedrock surface or on granular pads placed on bedrock is expected to offer cost effectiveness and relatively easy construction.

A foundation consisting of H-piles socketed into bedrock is feasible and would permit design of integral or false-integral abutments. Due to shallow depth to bedrock, socketing piles in the bedrock would be required to develop sufficient lateral resistance.

Caissons socketed into bedrock could be also considered at this site.

Advantages and disadvantages of feasible foundation alternatives are presented in the table in Appendix F.

Recommendations for design of the feasible foundation alternatives are presented in the following sections together with the corresponding geotechnical design parameters. A preferred foundation scheme from a geotechnical perspective is recommended.

### **8.2 Spread Footings on Bedrock**

Based on the subsurface conditions encountered at this site, the use of spread footings

founded on bedrock to support the abutments and piers is considered feasible from a geotechnical perspective.

The depths to bedrock and the bedrock surface elevations encountered in the boreholes advanced during the present investigation are indicated in Table 5.1. The existing subsurface information from the MTO reports prepared in 1966 (enclosed in Appendix E) has been utilized to supplement information on the bedrock surface in the general bridge area. Table 8.1, below, provides anticipated founding elevations for spread footings utilizing the available bedrock information.

**Table 8.1 - Anticipated Founding Elevations for Spread Footings on Bedrock**

Foundation Element	Borehole Number	Depth to Bedrock / Elevation of Bedrock Surface (m)	Thickness of Highly Weathered Zone (m)	Anticipated Founding Elevation (m)
East Abutment	EC14-01 EC14-02	1.7 / 92.1 1.2 / 92.0	1.0 0.5	91.1
East Pier	164 <sup>2)</sup> 166 <sup>2)</sup>	0.0 / 91.9 0.7 / 92.2	0.3 1.6	90.6
West Pier	163 <sup>2)</sup> 165 <sup>2)</sup>	0.3 / 92.5 0.1 / 91.3	0.6 0.3	91.0
West Abutment	EC14-03 <sup>1)</sup> EC14-04 <sup>1)</sup>	8.3 / 91.7 8.5 / 91.5	N/A N/A	91.5

<sup>1)</sup> Boreholes drilled from the top of the QEW embankment; depths and elevations of bedrock below the concrete footings. Thickness of highly weathered zone is not available.

<sup>2)</sup> Data obtained from the MTO Report Geocres #30M11-20 (enclosed in Appendix E); bedrock elevations are derived from boreholes drilled in the vicinity of the foundation units and are provided for planning purposes, only.

The actual founding elevation at specific foundation locations will have to be confirmed during detailed design stage.

The footings founded on weathered shale bedrock could be designed using a Factored Geotechnical Resistance at ULS of 1,000 kPa. The Geotechnical Reaction at SLS will not govern the design.

The geotechnical resistances quoted above are for concentric, vertical loads only. In the case of eccentric or inclined loading, the geotechnical resistance should be calculated as illustrated in the CHBDC 2006 Clause 6.7.3 and Clause 6.7.4.

Resistance to lateral forces / sliding resistance between the footing concrete and the

bedrock surface should be evaluated in accordance with the CHBDC, 2006 assuming an ultimate (unfactored) coefficient of friction of 0.5.

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

The exposed shale bedrock at the subgrade level should be protected from disturbance such as due to construction traffic and weathering. For protection, a 100 mm concrete working slab should be placed at the footing subgrade within short time following exposing the founding strata.

Excavation and backfilling for the footings should be in accordance with OPSS 902.

### **8.3 Spread Footings on Engineered Fill**

The foundations consisting of spread footing founded on engineered fill pads bearing on bedrock can be considered at this site. The founding levels on bedrock indicated in Table 8.1 can be used in design. This foundation option is feasible at the abutments, in conjunction with adequate scour protection. It is neither recommended nor beneficial at the piers.

The engineered fill should consist of OPSS Granular “A” or Granular B Type II placed in 150 mm lifts and compacted to 100% of its SPMD at  $\pm 2\%$  of optimum moisture content. The top of the founding pad should be at least 1 m wider than the footprint of the spread footing. The side slopes of the engineered fill pad should be inclined not steeper than 1H:1V.

The base of the engineered fill should extend down to the anticipated founding elevations given in Table 8.1.

A 2 m wide footing bearing on a minimum 1 m thick pad of engineered fill placed on bedrock may be designed for the following capacities:

Factored Geotechnical Resistance at ULS	- 900 kPa
Geotechnical Resistance at SLS	- 350 kPa

The geotechnical resistance at SLS quoted above corresponds to 25 mm of settlement of an individual footing.

The geotechnical resistances are for vertical, concentric loads. Where eccentric or inclined loads are applied, the resistance used in design must be reduced in accordance with the CHBDC Clause 6.7.3 and Clause 6.7.4.

The lateral resistance of the footings founded on engineered fill may be computed using an unfactored friction coefficient of 0.6. This is an “ultimate” value and requires a degree of sliding movement to occur to fully mobilize the resistance.



## 8.4 Augered H-Piles/Integral Abutment Considerations

A foundation consisting of H-piles socketed into bedrock is feasible and would permit design of integral abutments.

The bedrock surface elevations to be used in the preliminary design of pile foundations are indicated in Table 8.1, above. The integral abutment design requires that the piles possess flexibility in the upper 3 m of the pile length. To provide the required flexibility for piles to be installed through the compacted fill, the upper 3 m of the piles should be surrounded by a 600 mm diameter CSP as specified by the integral abutment design procedures. After the pile is installed, the space between the pile and the CSP should be filled with sand. An NSSP should be included in the contract documents specifying the gradation of the sand according to Table 8.2.

**Table 8.2 Integral Abutment Sand Backfill Grading**

MTO Sieve Designation		Percentage Passing
2 mm	#10	100%
600 µm	#30	80%-100%
425 µm	#40	40%-80%
250 µm	#60	5%-25%
150 µm	#100	0%-6%

### 8.4.1 Axial Resistance

Steel H-piles should be considered as founded entirely in bedrock, as the resistance developed in the relatively shallow overburden soils (fill and sand) will be relatively low. For an HP 310 x 110 pile grouted within a 600 mm diameter socket extended at least 3 m into shale bedrock below the levels indicated in Table 8.1, a factored axial structural resistance at ULS of 2,000 kN per pile may be used for design.

The SLS condition does not apply to piles founded in bedrock.

### 8.4.2 Lateral Resistance

The lateral resistance that can be mobilized in front of a pile socket/caisson in shale, assuming a clear spacing of at least one socket diameter between the sockets, may be computed using the coefficient of horizontal subgrade reaction  $k_s$  and ultimate lateral resistance  $p_{ult}$  estimated as follows:

$k_s =$  10 MN/m<sup>3</sup> at the bedrock surface, increasing linearly to  
100 MN/m<sup>3</sup> at a depth of 3 pile socket diameters and below.

$p_{ult} =$  300 kPa at the bedrock surface, increasing linearly to  
3000 kPa at the depth of 3 pile socket diameters and below.

The spring constant,  $K_s$ , for analysis may be obtained from the expression,  $K_s = k_s L D$  (kN/m), where  $k_s$  is the coefficient of horizontal subgrade reaction (kN/m<sup>3</sup>),  $D$  is the pile

width (m) and L is the length (m) of the pile segment or element used in the analysis.

The ultimate lateral resistance,  $P_{ult}$ , may be obtained from the expression,  $P_{ult} = p_{ult} L D$ ; this represents the ultimate load, at which the pile fails and will not support any additional load at greater displacements.

The structural designer should check whether a 3.0 m deep socket is sufficient to provide base fixity.

The reduction factors due to pile spacing are provided in Table 8.3, below.

**Table 8.3 – Reduction Factors for Subgrade Reaction due to Pile Spacing**

Condition	Pile Spacing (Centre to Centre)	Reduction Factor
Pile group oriented <i>perpendicular</i> to direction of loading	4D	1.0
	1D	0.5
Pile group oriented <i>parallel</i> to direction of loading	8D	1.0
	6D	0.7
	4D	0.4
	3D	0.25

Consideration may be given to the use of battered piles if lateral pile capacities higher than the available geotechnical lateral resistances are required.

#### 8.4.3 Augered Pile Installation

Pile installation should be in general accordance with OPSS 903. The pre-drilled holes for forming the H-pile socket should have a diameter of 600 mm minimum.

The pile installation equipment should be capable of dislodging and removing any obstructions such as cobbles, boulders and other obstructions in the fill and native soils. Hard layers of limestone interbedded in the shale bedrock may require the use of coring or rock breaking equipment in addition to the augering equipment. Temporary steel liners should be used to support the socket sidewalls of the predrilled holes and minimize groundwater inflow.

The pile socket excavation should be dewatered to allow cleaning of the base and walls prior to placing concrete. Concrete should be placed with minimum delay after the socket is drilled, cleaned, inspected and approved.

Subsequent to the seating of a pile in the socket, the socket should be grouted with 30 MPa concrete.

If an integral abutment design is selected, the holes augered through the overburden and for the temporary liner should be of sufficient diameter to permit placement of the CSPs required in the integral abutment design.

## 8.5 Augered Caissons (Drilled Shafts)

Augered caisson foundations socketed into shale bedrock may be considered at this site. The bedrock surface indicated in Table 8.1 could be used for the preliminary design of the socketed caissons, if this option is selected.

### 8.5.1 Axial Resistance

Table 8.4 below presents factored geotechnical resistances calculated for 1.2 m and 1.5 m diameter caissons associated with the following minimum socket depths within bedrock.

**Table 8.4 - Vertical Geotechnical Resistance for Caisson Foundations**

<b>Caisson Diameter (m)</b>	<b>Minimum Socket Depth below Bedrock Surface (m)</b>	<b>Factored ULS (kN)</b>
1.2	2	3,500
	2.5	4,250
	3.0	5,100
1.5	2.0	4,250
	2.5	5,300
	3.0	6,350

The SLS conditions do not apply to caissons founded on rock.

The above values were obtained for the shaft resistance in the rock sockets only considering difficulties to properly clean the bases of the relatively small diameter sockets.

The minimum spacing between adjacent caissons should be as per the CHBDC 2006. The vertical resistance will not be significantly affected by the caisson spacing for caissons socketed within bedrock.

### 8.5.2 Lateral Resistance

The lateral resistance of a caisson may be calculated using a value for the coefficient of horizontal subgrade reaction ( $k_s$ ) and ultimate lateral resistance ( $p_{ult}$ ) presented in Section 8.4.2 of this report.

### 8.5.3 Caisson Installation

Caisson installation should be in accordance with OPSS 903.

The caisson installation equipment should be capable of dislodging and removing any obstructions such as cobbles, boulders and rock slabs in the soil deposit. Layers of limestone in the shale bedrock may require the use of coring or rock breaking equipment in addition to the auger equipment.

Temporary steel liners should be used to support the caisson sidewalls to minimize sloughing and groundwater inflow.

The caisson excavation should be dewatered to allow cleaning of the base and walls prior to placing concrete. Concrete should be placed with minimum delay after the socket is drilled, cleaned, inspected and approved.

### **8.6 Recommended Foundations**

The spread footings founded on shale bedrock and/or on granular pads constructed on bedrock are the preferred foundation options for this bridge foundations, considering the cost effectiveness and constructability aspects.

From a geotechnical point of view, all foundation options presented above are feasible for this bridge site.

The comparison of the foundation options are presented in Appendix F.

### **8.7 Scour Protection and Erosion Control**

Scour and erosion protection must be provided for the footings located in the creek floodplain. Design of the scour and erosion protection measures should be carried out by specialists experienced in this field. Typically, rock protection should be provided over all earth surfaces with which creek water is likely to be in contact.

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

### **8.8 Frost Cover**

The depth of frost penetration at this site is approximately 1.2 m. The base of footings or pile caps should be provided with a minimum of 1.2 m of earth cover as protection against frost action.

## **9 EXCAVATION AND GROUNDWATER CONTROL**

All excavations and backfilling should be carried out in accordance with OPSS 902 and the requirements of the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the existing fill and native soils, if encountered within the depth of excavation may be classified as Type 3 soils above the water table and Type 4 soils below the water table. Flatter slopes may be required at locations where water seepage affects stability of an excavation.

Excavations for abutment footings construction are expected to extend through the embankment fill and probably through shallow native deposits overlying the bedrock. The groundwater levels as measured in the piezometer and indicated on the archive drawings, follow the bedrock surface or are slightly above the bedrock surface. The groundwater will be governed by the water level in the creek. Perched water may be present in the embankment fill. Some water control, such as pumping

from sumps, may be required.

The selection of the method of excavation is the responsibility of the Contractor and should be based on his equipment availability, experience and interpretation of the site conditions. It is anticipated that a hydraulic excavator will be suitable. Provision should be made for the handling of pavement materials and potential obstructions in the fill, and cobbles, boulders and rock slabs above the bedrock.

It is understood that the new structure will be constructed in stages and that the traffic on QEW will be maintained at all times, which will require roadway protection in the existing approach fills. Roadway protection should be provided in accordance with OPSS 539 and designed for Performance Level 2.

The design of any roadway protection or dewatering system that may be required is the responsibility of the Contractor. All shoring systems should be designed by a professional engineer experienced in such design.

## **10 APPROACH EMBANKMENTS**

The approach embankments will be as much as 7.5 m in height. The proposed approach embankments with permanent embankment slopes inclined at 2H: 1V or flatter for earth fill are considered to be stable.

Settlement induced by the proposed fill placement is anticipated to be essentially completed at the end of construction.

When placing new fill against the existing embankment, benching will be required for the existing embankment slopes in accordance with OPSD 208.010. If the embankment height is planned to exceed 8 m, a mid-height berm comprising a 2 m wide outside bench should be incorporated into the embankment design.

## **11 LATERAL EARTH PRESSURES**

Backfill to the abutment walls should be in accordance with OPSS 902 and should consist of Granular A or Granular B Type II material. All granular material should meet the specifications of OPSS.PROV 1010. Compaction equipment to be used adjacent to retaining structures should be restricted in accordance with OPSS 902.

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

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

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

$K$  = coefficient of lateral earth pressure (see Table 11.1)

$\gamma$  = unit weight of retained soil (see Table 11.1)

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

$q$  = value of any surcharge (kPa)

Earth pressure coefficients for backfill to the abutment wall are dependent on the material used as backfill. Typical values are given in Table 11.1.

The coefficients provided in Table 11.1 are “ultimate” values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.16 in the Commentary to the Canadian Highway Bridge Design Code (CHBDC).

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 at a depth of 1.7 m for Granular A or Granular B Type II.

**Table 11.1 – Coefficients of Lateral Earth Pressure**

Loading Condition	Earth Pressure Coefficient (K)			
	OPSS Granular A or Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$	
	Horizontal Backfill	Sloping Backfill (2H:1V)	Horizontal Backfill	Sloping Backfill (2H:1V)
Active (Unrestrained Wall)	0.27	0.39*	0.31	0.47*
At-rest (Restrained Wall)	0.43	-	0.47	-
Passive (Movement Towards Soil Mass)	3.7	-	3.3	-

\* For wing walls.

## 12 SEISMIC CONSIDERATIONS

The following seismic parameters should be used for design in accordance with the CHBDC for a design earthquake with 475-year return period:

- Velocity Related Seismic Zone 0
- Zonal Velocity Ratio 0.05
- Acceleration Related Seismic Zone 1
- Zonal Acceleration Ratio 0.05
- Peak Ground Acceleration 0.04 g

The soil profile type at this site has been classified as Type I. Therefore, according to Clause 4.4.6.1 of the CHBDC, a Site Coefficient “S” (ground motion amplification factor) of 1.0 should be used in seismic design. In accordance with Clause 4.6.4 of the CHBDC, retaining structures should be designed using active ( $K_{AE}$ ) and passive ( $K_{PE}$ ) earth pressure coefficients that incorporate the effects of earthquake loading. For the design of retaining walls, the coefficients of lateral earth

pressure in Table 12.1 may be used.

**Table 12.1 – Coefficient of Lateral Earth Pressure for Seismic Loading**

Loading Condition	Earth Pressure Coefficient ( $K_E$ ) for Seismic Loading			
	OPSS Granular A or Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$	
	Horizontal Backfill	Sloping Backfill (2H:1V)	Horizontal Backfill	Sloping Backfill (2H:1V)
Active ( $K_{AE}$ )*	0.29	0.42	0.32	0.51
At-rest ( $K_{OE}$ )**	0.46	-	0.51	-
Passive ( $K_{PE}$ )*	3.5	-	3.1	-

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

\*\* After Woods (1973).

Based on review on the SPT data, seismically-induced liquefaction of foundation soils is not anticipated under the design earthquake.

### 13 CONSTRUCTION CONCERNS

During construction, the Contract Administrator (CA) should retain an experienced geotechnical engineer to observe foundation construction activities and to provide advice to the CA regarding any issues that need to be referred to the design team.

Potential construction concerns include, but are not necessarily limited to the excavation and dewatering for the spread footings, namely:

- The water level in the creek may fluctuate and may impact the groundwater level. Any excavation carried out below the groundwater level runs a risk of being destabilized due to the inflow of water. Adequate groundwater control measures should be in place to maintain the stability of the excavation and to prevent loss of ground.
- Cobbles or other buried obstruction may be encountered and interfere during excavation in the existing embankment fill.
- If deep foundations are selected for this structure, the pile/caisson installation equipment should be capable of dislodging and removing any obstructions such as cobbles, boulders and other obstructions in the fill and native soils, if encountered. Hard layers of limestone interbedded in the shale bedrock may require the use of coring or rock breaking equipment in addition to the augering equipment. Contractor should be prepared to advance holes to specified elevations.
- If the existing foundations are to be removed and this removal disturbs the ground within the zone of influence of the new foundations, the disturbance must be made good, possibly by backfilling with concrete

#### 14 INVESTIGATION FOR DETAIL DESIGN

During the detailed design phase of this project, additional site investigations and field testing will be required. Depending on the final configuration/location of the foundation units, the existing subsurface information should be reviewed and supplemented, as required, to comply with the MTO Foundation Engineering Terms of Reference.

It is envisioned that as a minimum two boreholes will be required at each foundation element to refine the bedrock surface profile and the thickness of the highly weathered zone within the bedrock.

#### 15 CLOSURE

Engineering analysis and preparation of the foundation design report were carried out by Ms. Anna Piascik, P.Eng. The report was reviewed by Mr. Alastair E. Gorman, P.Eng. and Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

#### THURBER ENGINEERING LTD.

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**Appendix A**  
**Record of Borehole Sheets**

## SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

### 1. TEXTURAL CLASSIFICATION OF SOILS

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

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

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

### 3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

DESCRIPTIVE TERM	UNDRAINED SHEAR STRENGTH (kPa)	APPROXIMATE SPT <sup>(1)</sup> '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


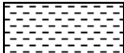



### ROCK WEATHERING CLASSIFICATION

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

### DISCONTINUITY SPACING

<b>Bedding</b>	<b>Bedding Plane Spacing</b>
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

<b>Rock Strength</b>	<b>Approximate Uniaxial Compressive Strength (MPa)</b>	<b>Approximate Uniaxial Compressive Strength (psi)</b>	<b>Field Estimation of Hardness*</b>
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 <sub>L</sub> < 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 <sub>L</sub> < 30%).
		CI	Inorganic clays of medium plasticity, silty clays. (30% < W <sub>L</sub> < 50%).
		OL	Organic silts and organic silty-clays of low plasticity.
	SILTS AND CLAYS W <sub>L</sub> > 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			

# RECORD OF BOREHOLE No EC 14-01

1 OF 1

METRIC

W.P. 09-20003 LOCATION Etobicoke Creek Bridge N 4 829 007.0 E 616 272.0 ORIGINATED BY SLL  
 HWY QEW BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN  
 DATUM Geodetic DATE 2014.08.07 - 2014.08.07 CHECKED BY AMP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT  $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
93.8	GROUND SURFACE							20	40	60	80	100		
0.0	TOPSOIL, frequent roots and rootlets: (175mm)		1	SS	13									
0.2	SAND, some silt, trace clay, trace topsoil Compact Dark Brown Moist (FILL)													
92.9	Clayey SILT and SAND, trace gravel, occasional limestone fragments		2	SS	22									
0.9	Hard Brown Moist (FILL)													
92.1	SHALE BEDROCK, highly weathered, occasional limestone interbeds		3	SS	100/									
1.7	Hard Grey Moist (Georgian Bay Formation)				0.175									
91.1	SHALE BEDROCK, moderately to slightly weathered, thinly bedded, weak to strong, occasional strong limestone interbeds, grey (Georgian Bay Formation)		4	SS	100/									
2.7	Highly weathered zones 50mm to 75mm thick to 3.4m depth.		1	RUN										RUN #1 TCR=100% SCR=76% RQD=37% UCS=12MPa
			2	RUN										RUN #2 TCR=58% SCR=48% RQD=13% UCS=14.6MPa (Average)
			3	RUN										RUN #3 TCR=100% SCR=100% RQD=37% UCS=18.5MPa (Average)
			4	RUN										RUN #4 TCR=100% SCR=100% RQD=57% UCS=77.4MPa (Average)
85.9	END OF BOREHOLE AT 7.9m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.													
7.9	WATER LEVEL READINGS: DATE      DEPTH (m)      ELEV. (m) 2014.09.29      2.1      91.7 2015.01.15      2.0      91.8													

ONTMT4S 1219.GPJ 2012TEMPLATE(MTO).GDT 1/22/15



+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity 20 15 10 5 0 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No EC 14-02

1 OF 1

METRIC

W.P. 09-20003 LOCATION Etobicoke Creek Bridge N 4 829 040.7 E 616 224.1 ORIGINATED BY SLL  
 HWY QEW BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN  
 DATUM Geodetic DATE 2014.08.07 - 2014.08.07 CHECKED BY AMP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT						UNIT WEIGHT  $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa											
93.2	GROUND SURFACE							20	40	60	80	100							
0.0	<b>TOPSOIL</b> , frequent roots and rootlets: (150mm)  Clayey <b>SILT</b> , trace sand and gravel, occasional concrete fragments Stiff to Hard (FILL)		1	SS	9		93												
0.2																			
			2	SS	50/														
92.0					0.150														
1.2	<b>SHALE BEDROCK</b> , grey, moderately to slightly weathered, thinly bedded, very weak to weak, occasional medium to very strong limestone interbeds (Georgian Bay Formation)  100mm clay seam at 1.3m 50mm vertical joint at 1.6m 38mm clay seam at 1.7m		1	RUN			92												
			2	RUN					91										
									90										
			3	RUN			89												
88.2																			
5.0	END OF BOREHOLE AT 5.0m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 0.6m, THEN CUTTINGS TO SURFACE.																		

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity 20 15 10 5 0 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No EC 14-03

1 OF 2

METRIC

W.P. 09-20003 LOCATION Etobicoke Creek Bridge N 4 828 998.0 E 616 200.0 ORIGINATED BY ES  
 HWY QEW BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN  
 DATUM Geodetic DATE 2014.12.16 - 2014.12.17 CHECKED BY AMP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT  γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)									
100.0	GROUND SURFACE							20	40	60	80	100						GR	SA	SI	CL	
0.0	ASPHALT:(150mm)							20	40	60	80	100										
0.2	CONCRETE:(300mm)							20	40	60	80	100										
99.5								20	40	60	80	100										
0.5	Gravelly SAND, some silt, trace clay Compact to Very Dense Brown Moist (FILL)		1	SS	16		99															
			2	SS	17		98													24	58	18 (SI+CL)
	Occasional cobbles		3	SS	39		97															
			4	SS	63		96													32	53	15 (SI+CL)
			5	SS	59		95															
94.5							94															
5.5	SAND, some silt, trace gravel Loose Brown Wet (FILL)		6	SS	3		93													9	66	25 (SI+CL)
92.7	Concrete with rebars		1	RUN			92															
91.7	SHALE BEDROCK moderately to slightly weathered, thinly bedded, weak, occasional strong limestone interbeds, grey (Georgian Bay Formation)		2	RUN			91															

Continued Next Page

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

ONTMT4S 1219.GPJ 2015TEMPLATE(MTO).GDT 4/2/15

RECORD OF BOREHOLE No EC 14-03

2 OF 2

METRIC

W.P. 09-20003 LOCATION Etobicoke Creek Bridge N 4 828 998.0 E 616 200.0 ORIGINATED BY ES  
 HWY QEW BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN  
 DATUM Geodetic DATE 2014.12.16 - 2014.12.17 CHECKED BY AMP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					WATER CONTENT (%)				
							20	40	60	80	100	W <sub>p</sub>	W	W <sub>L</sub>			
88.3	Continued From Previous Page <b>SHALE BEDROCK</b> moderately to slightly weathered, thinly bedded, weak, occasional strong limestone interbeds, grey (Georgian Bay Formation)		3	RUN			89									1	RUN #3 TCR=59% SCR=48% RQD=33% UCS=13MPa (Average)
11.7	END OF BOREHOLE AT 11.7m. Piezometer installation consists of 25mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen.  WATER LEVEL READINGS: DATE      DEPTH (m)      ELEV. (m)  2015.03.31      6.9      93.1															4	
																>5	
																>5	
																>5	



# RECORD OF BOREHOLE No EC 14-04

1 OF 2

METRIC

W.P. 09-20003 LOCATION Etobicoke Creek Bridge N 4 828 974.0 E 616 233.0 ORIGINATED BY ES  
 HWY QEW BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN  
 DATUM Geodetic DATE 2014.12.14 - 2014.12.15 CHECKED BY AMP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT  $\gamma$  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
100.0	GROUND SURFACE							<div><div>20406080100</div><div>0 UNCONFINED + FIELD VANE</div><div>● QUICK TRIAXIAL × LAB VANE</div></div>						
0.0	ASPHALT:(113mm)							<div><div>20406080100</div><div>WATER CONTENT (%)</div><div>PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT</div><div>W<sub>P</sub> W W<sub>L</sub></div></div>						
0.1	CONCRETE:(300mm)													
99.6														
0.4	SAND and GRAVEL, some silt Compact to Very Dense Brown Moist (FILL)  Occasional cobbles		1	SS	23		99							
			2	SS	35		98							
			3	SS	55		97						45 42 13 (SI+CL)	
			4	SS	25		96							
			5	SS	58		95						34 55 11 (SI+CL)	
93.9							94							
6.1	Clayey SILT, some sand, trace gravel  Stiff Grey (FILL)  Frequent limestone fragments below 7.1m depth		6	SS	10		93						6 13 59 22	
92.4			7	SS	50/									
7.6	CONCRETE		1	RUN	0.075		92							
91.5							91							
8.5	SHALE BEDROCK moderately to slightly weathered, thinly bedded, weak, occasional medium strong limestone interbeds, grey (Georgian Bay Formation) Clay seams (125mm) at 9.1m and (25mm) at 9.9m  Highly broken zone (25mm to 75mm) at 9.1m, 9.2m and 9.8m		2	RUN									RUN #2 TCR=97% SCR=85% RQD=65% UCS=9.3MPa (Average)	

Continued Next Page

+ 3, x 3: Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No EC 14-04

2 OF 2

METRIC

W.P. 09-20003 LOCATION Etobicoke Creek Bridge N 4 828 974.0 E 616 233.0 ORIGINATED BY ES  
 HWY QEW BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN  
 DATUM Geodetic DATE 2014.12.14 - 2014.12.15 CHECKED BY AMP

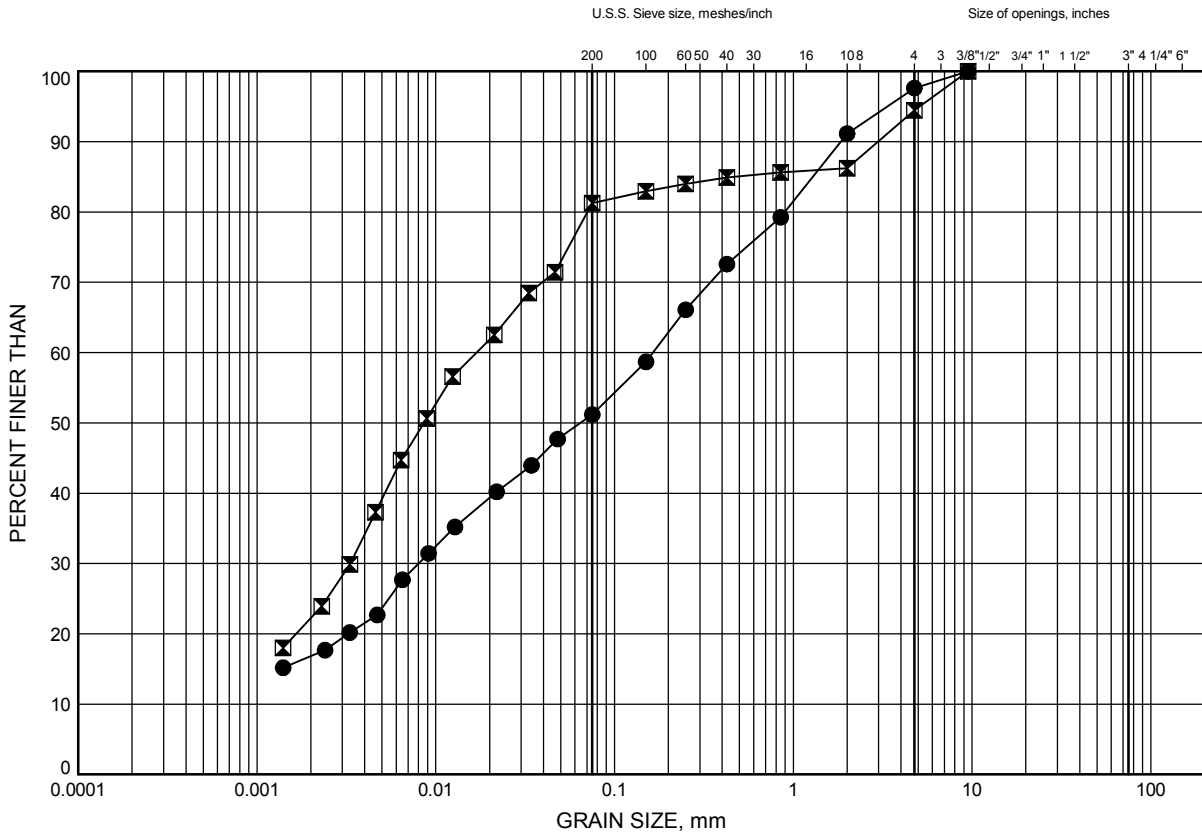
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
	Continued From Previous Page																
88.4	<b>SHALE BEDROCK</b> moderately to slightly weathered, thinly bedded, weak to strong, occasional strong limestone interbeds, grey (Georgian Bay Formation) Clay seams (25mm) at 10.4m, 10.6m, 10.7m and 11.4m Highly broken zone (100mm) at 11.0m		3	RUN			89									>10 5 6 >10 2	RUN #3 TCR=88% SCR=82% RQD=53% UCS=14.3MPa (Average)
11.6	END OF BOREHOLE AT 11.6m. Piezometer installation consists of 25mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen.  WATER LEVEL READINGS: DATE      DEPTH (m)      ELEV. (m)  2015.03.31      7.1      92.9																

**Appendix B**  
**Laboratory Test Results for**  
**Soil and Rock Samples**

QEW Cawthra Road  
**GRAIN SIZE DISTRIBUTION**

FIGURE B1

**CLAYEY SILT TO CLAYEY SILT WITH SAND FILL**



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

**LEGEND**

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	EC 14-01	1.07	92.74
⊠	EC 14-04	6.40	93.64

Date January 2015  
W.P. 09-20003

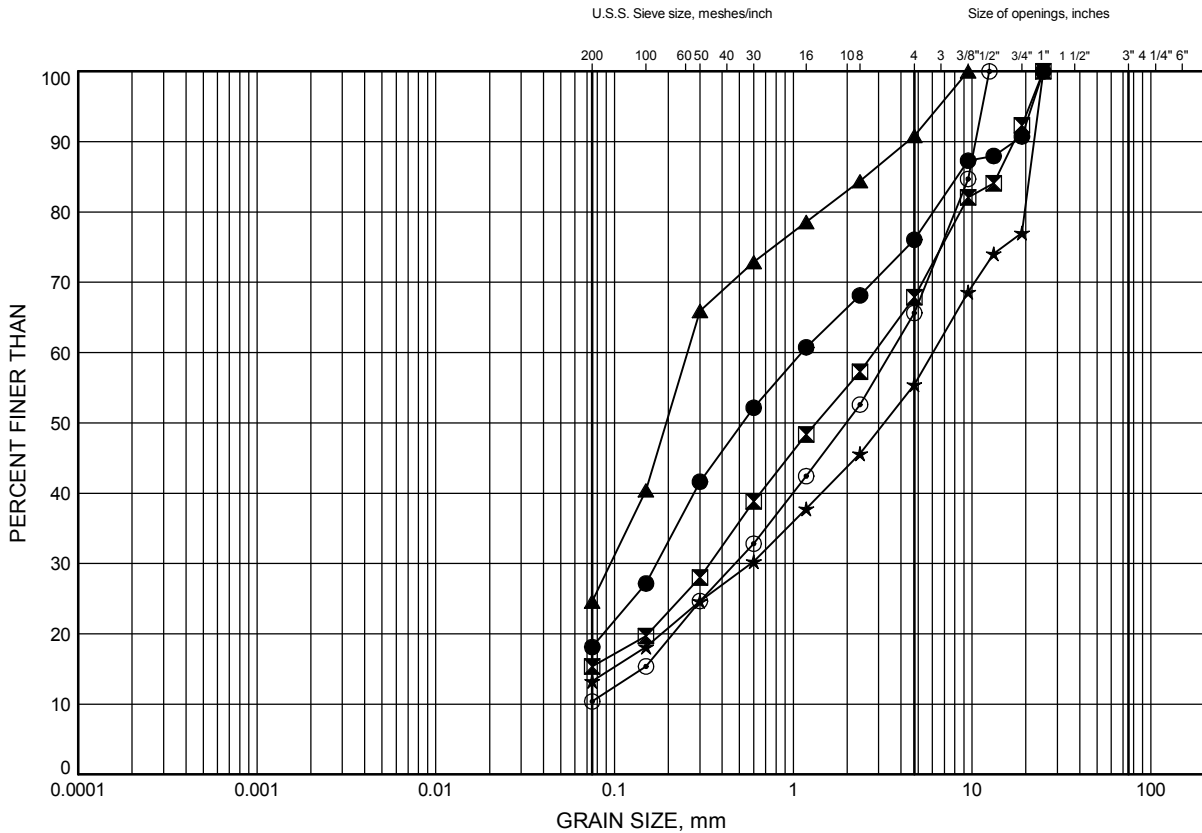


Prep'd MFA  
Chkd. AMP

QEW Cawthra Road  
GRAIN SIZE DISTRIBUTION

FIGURE B2

CLAYEY SILT TO CLAYEY SILT WITH SAND FILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	EC 14-03	1.83	98.21
⊠	EC 14-03	3.35	96.69
▲	EC 14-03	6.40	93.64
★	EC 14-04	2.59	97.45
⊙	EC 14-04	4.88	95.16

Date January 2015  
W.P. 09-20003



Prep'd MFA  
Chkd. AMP



THURBER ENGINEERING LTD.

## POINT LOAD TEST SHEET

Job No : 19-1351-219

Client : MMM

Project Name : QEW CAWTHRA ROAD  
ETOBICOKE CREEK BRIDGE

Date Drilled : 7 Aug, 2014

Core Size : NQ BH No : EC14-01

Date Tested : 8 Aug, 2014

Tester : GAZ

Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (MPa)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	1	3.3	A	1.9	47.0	65.1	12.3	Shale	Weak
2	2	4.2	A	2.2	46.8	59.8	15.2	Shale	Weak
3	2	4.6	D	1.4	45.1	95.0	15.4	Shale	Weak
4	2	4.9	A	1.9	47.3	58.8	13.2	Shale	Weak
5	3	5.4	A	2.4	47.3	50.7	18.5	Shale	Weak
6	3	6.4	D	1.9	47.2	53.9	18.5	Shale	Weak
7	4	7.5	A	11.8	47.4	51.8	90.7	Shale/Limestone	Strong
8	4	7.9	D	6.4	47.1	100.1	64.0	Shale	Strong
9									
10									
11									
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13									
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35									

\* It is ideal to perform axial test on core specimens with D/L ratio of  $1.1 \pm 0.1$

Long pieces of core can be tested diametrically to produce suitable lengths for axial testing

\* Diametral Test should have  $0.7 \times D$  on either side of test point.

Last Modified: August 15, 2013



**THURBER** ENGINEERING LTD.

## POINT LOAD TEST SHEET

Job No : 19-1351-219

Client : MMM

Project Name : QEW CAWTHRA ROAD  
ETOBICOKE CREEK BRIDGE

Date Drilled : 7 Aug, 2014

Core Size : NQ BH No : EC14-02

Date Tested : 8 Aug, 2014

Tester : GAZ

Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (MPa)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	1	1.5	D	2.9	47.5	117.4	29.0	Limestone	Medium Strong
2	1	1.9	A	2.3	47.3	65.0	14.9	Shale	Weak
3	2	2.3	A	17.7	47.4	66.1	112.1	Shale/Limestone	Very Strong
4	2	3.3	D	0.4	47.3	long	4.4	Shale	Very Weak
5	3	3.6	A	1.3	47.3	64.6	8.7	Shale	Weak
6	3	4.6	D	3.2	47.3	40.9	31.5	Shale/Limestone	Medium Strong
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\* It is ideal to perform axial test on core specimens with D/L ratio of  $1.1 \pm 0.1$

Long pieces of core can be tested diametrically to produce suitable lengths for axial testing

\* Diametral Test should have  $0.7 \times D$  on either side of test point.

Last Modified: August 15, 2013



**THURBER** ENGINEERING LTD.

## POINT LOAD TEST SHEET

Job No : 19-1351-219

Client : MMM

Project Name : QEW CAWTHRA ROAD  
ETOBICOKE CREEK BRIDGE

Date Drilled : 12 Dec, 2014

Core Size : NQ BH No : EC14-03

Date Tested : 18 Dec, 2014

Tester : JW

Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (MPa)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	1	8.4	A	1.1	46.8	28.1	14.2	Shale	Weak
2	1	8.6	D	0.9	47.8	99.5	9.2	Shale	Weak
3	2	9.0	D	1.7	47.8	120.0	16.2	Shale	Weak
4	2	9.7	D	1.4	47.7	102.6	14.1	Shale	Weak
5	3	10.5	A	2.0	47.0	49.7	16.1	Shale	Weak
6	3	11.0	D	1.1	47.8	100.6	10.5	Shale	Weak
7									
8									
9									
10									
11									
12									
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34									
35									

\* It is ideal to perform axial test on core specimens with D/L ratio of  $1.1 \pm 0.1$

Long pieces of core can be tested diametrically to produce suitable lengths for axial testing

\* Diametral Test should have  $0.7 \times D$  on either side of test point.

Last Modified: August 15, 2013





THURBER ENGINEERING LTD.

## POINT LOAD TEST SHEET

Job No : 19-1351-219

Client : MMM

Project Name : QEW CAWTHRA ROAD  
ETOBICOKE CREEK BRIDGE

Date Drilled : 15 Dec, 2014

Core Size : NQ BH No : EC14-04

Date Tested : 18 Dec, 2014

Tester : JW

Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (MPa)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	2	9.0	D	0.6	47.1	85.6	5.6	Shale	Weak
2	2	9.6	A	2.2	47.3	53.5	16.8	Shale	Weak
3	2	9.9	D	0.6	47.3	49.8	5.6	Shale	Weak
4	3	10.2	D	1.1	47.0	87.0	11.2	Shale	Weak
5	3	10.9	D	0.6	47.1	70.2	6.2	Shale	Weak
6	3	11.3	D	2.6	47.1	115.5	25.6	Shale	Medium Strong
7									
8									
9									
10									
11									
12									
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35									

\* It is ideal to perform axial test on core specimens with D/L ratio of  $1.1 \pm 0.1$

Long pieces of core can be tested diametrically to produce suitable lengths for axial testing

\* Diametral Test should have  $0.7 \times D$  on either side of test point.

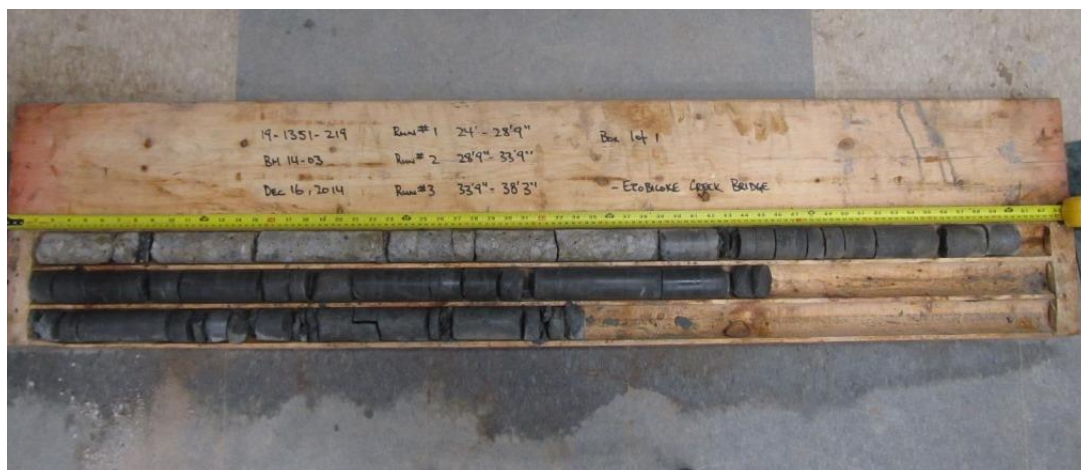
Last Modified: August 15, 2013



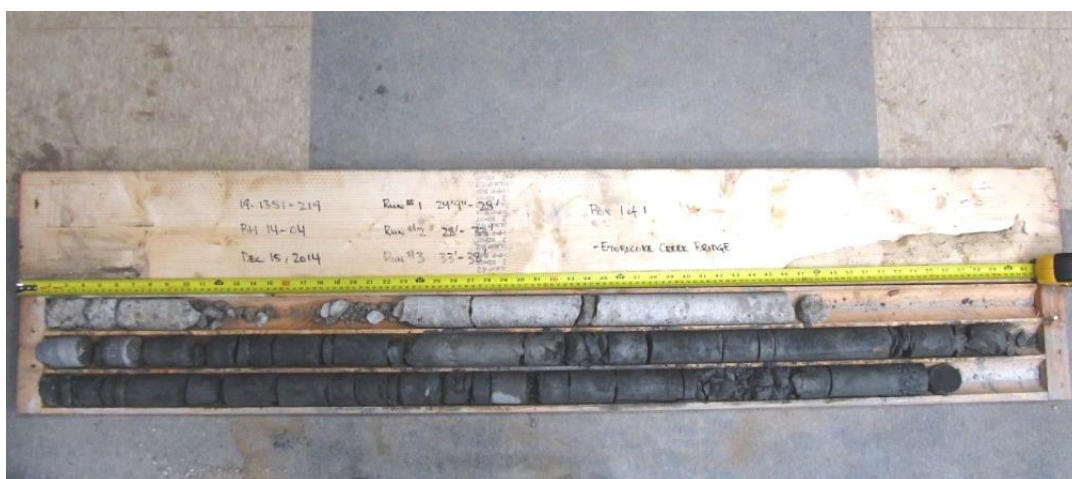
Photograph 1 – Rock cores from Borehole EC14-01



Photograph 2 – Rock cores from Borehole EC14-02



Photograph 3 – Rock Core from Borehole EC14-03



Photograph 4 – Rock Core from Borehole EC14-04

**Appendix C**  
**Site Photographs**





**Photograph 1 – View from the east side of Etobicoke Creek; looking southwest towards the west pier**



**Photograph 2 – View from the east side of Etobicoke Creek; looking northwest towards the west pier**



**Photograph 3 – Looking at the west creek bank between Etobicoke Bridge (on the right) and the ramp**



**Photograph 4 – Looking west at the North Bridge Elevation**





**Photograph 5 - Location of Borehole EC14-01; near the south corner of the east abutment.**





**Photograph 6 – Location of Borehole EC14-02; near the north corner of the east abutment**



**Photograph 7 – Looking at the northwest side of the west abutment**

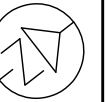




**Photograph 8 – View from the northwest corner of the bridge; looking southeast.**

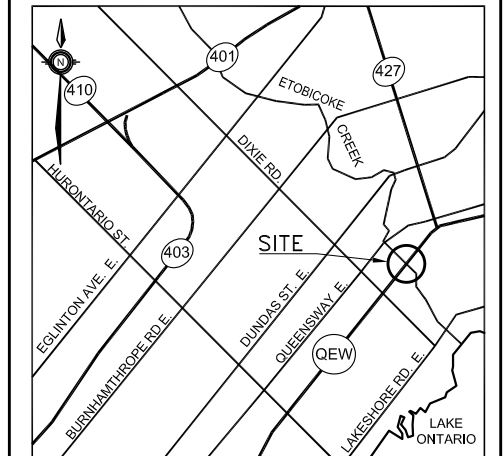
**Appendix D**  
**Borehole Locations and Soil Strata Drawings**

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








CONT No  
WP No 09-20003

QEW  
ETOBICOKE CREEK  
BRIDGE REPLACEMENT  
BOREHOLE LOCATIONS PLAN



## KEYPLAN

## LEGEND

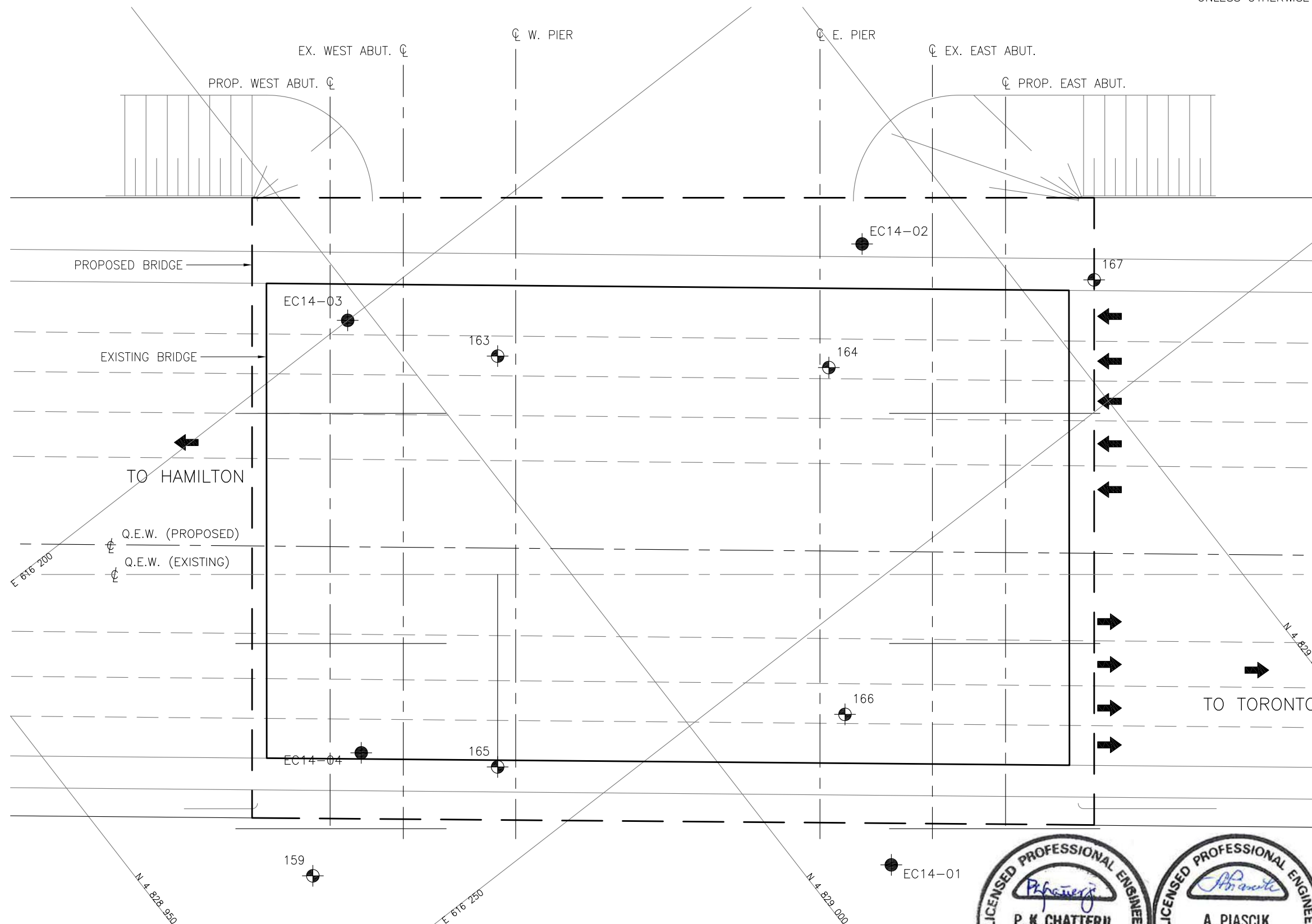
	Borehole (Present Investigation)
	Borehole (1966 Investigation)
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
	Water Level
	Head Artesian Water
	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

[illegible]

-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. 30M11-252**

[illegible]

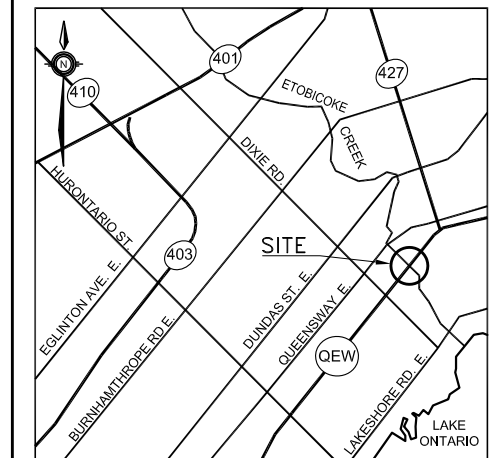
## PLAN



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






CONT No  
WP No 09-20003

QEW  
ETOBICOKE CREEK  
BRIDGE REPLACEMENT  
BOREHOLE LOCATIONS AND SOIL STRATA



## KEYPLAN

## LEGEND

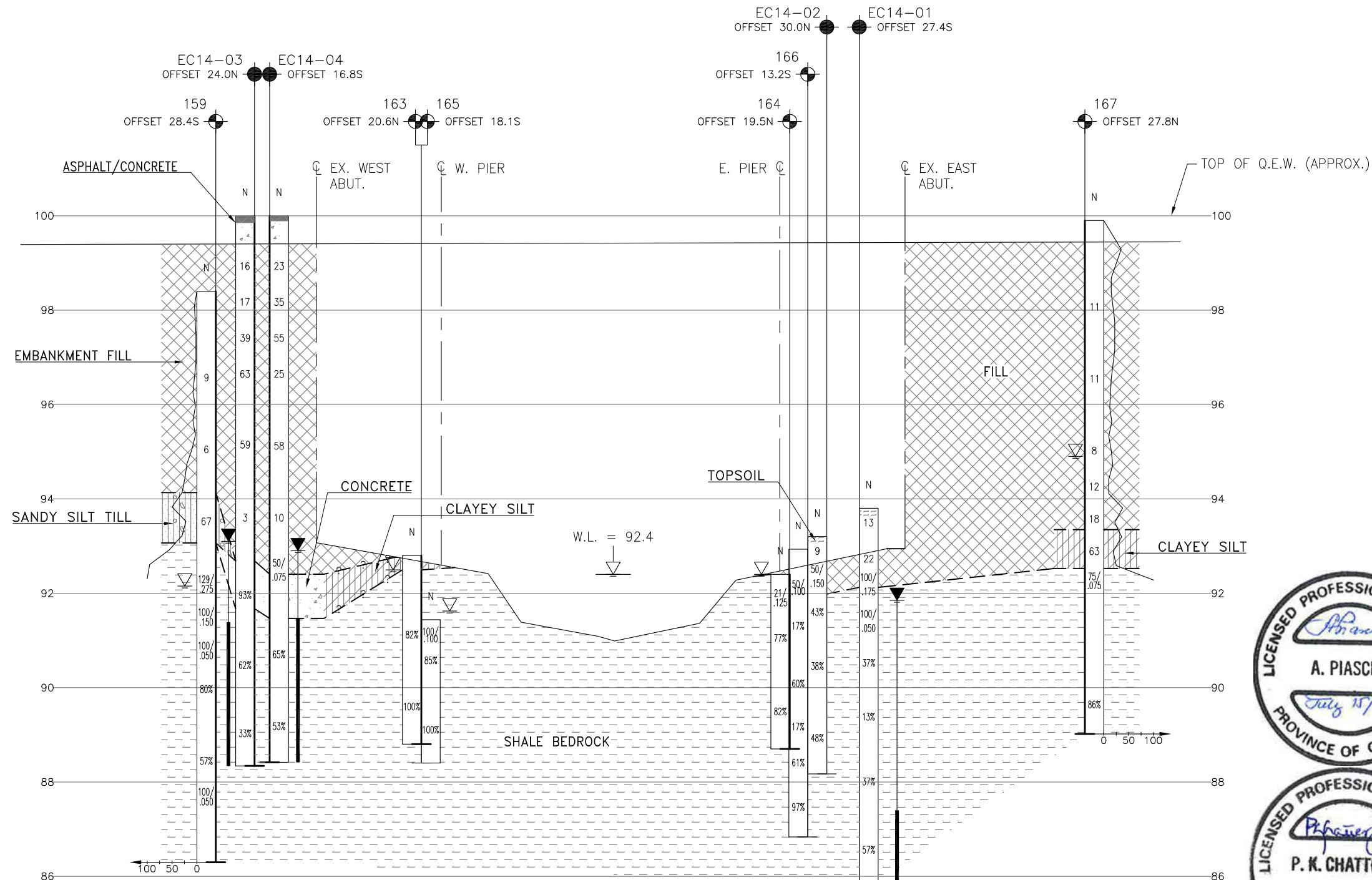
- |   |                                       |
|---|---------------------------------------|
|  | Borehole (Present Investigation)      |
|  | Borehole (1966 Investigation)         |
| N   | Blows /0.3m (Std Pen Test, 475J/blow) |
| CONE  | Blows /0.3m (60' Cone, 475J/blow)     |
| PH  | Pressure, Hydraulic                   |
|  | Water Level                           |
|  | Head Artesian Water                   |
|  | Piezometer                            |
| 90%   | Rock Quality Designation (RQD)        |
| A/R   | Auger Refusal                         |

[illegible]

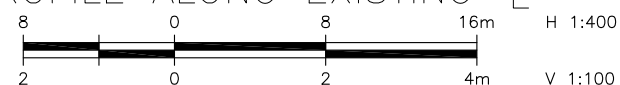
-NOTES-

- 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. 30M11-252**



### PROFILE ALONG EXISTING C



REVISIONS										
	DATE	BY	DESCRIPTION							
DESIGN	AMP	CHK	AEG	CODE	LOAD	DATE	JUL 2015			
DRAWN	MFA	CHK	AMP	SITE	37-784	STRUCT	DWG	2		

## **Appendix E**

### **Record of Borehole Sheets and Borehole Location Drawings MTO Reports, Geocres Nos: 31M11-20 and 30M11-26**

D O M I N I O N   S O I L   I N V E S T I G A T I O N   L I M I T E D

77 CROCKFORD BOULEVARD - SCARBOROUGH ONTARIO CANADA - TELEPHONE 751-6563

RANCH  
69 QUEENS AVENUE  
LONDON, ONTARIO  
TELEPHONE GC. 3-3251



FOUNDATION ENGINEERS

ASSOCIATED COMPANY  
SOIL TESTING AND ENGINEERING LTD.  
34 BRENTFORD ROAD,  
KINGSTON 5, JAMAICA, WEST INDIES  
TELEPHONE: 66896

September 14, 1966.

Our Ref: 6-7-15  
Your Ref: W.P. 174-65

Mr. A. G. Stermac,  
Principal Foundation Engineer,  
Materials & Testing Division,  
Department of Highways, Ontario,  
Downsview Avenue,  
Downsview, Ontario.

Attention: Mr. K. Selby, P. Eng.

Re: Soil Investigation for Q.E.W. & Highway #27  
Interchange Bridge #21

Dear Sirs:

Enclosed are the records of boreholes Nos. 1 to 13 inclusive which were put down at your request at the above proposed structure. The boreholes were located in the field and their elevations taken by a survey crew of the Department. The borehole locations indicated on the borehole logs are given as offsets from stations along the centre line of the Q. E. W. as shown on Drawing No. D5860-IP of your Bridge Division.

Also enclosed are copies of the grain size distribution curves showing the particle size distribution of the overburden in this area.

We trust you will find the forwarded information satisfactory.

Yours very truly,

DOMINION SOIL INVESTIGATION LIMITED,

*I. P. Lieszkowsky*  
I. P. Lieszkowsky, P. Eng.,  
Project Engineer.

IPL/jvm  
Encls.





# GEOTECHNICAL DATA SHEET FOR BOREHOLE . 155 .

OUR REFERENCE NO. 6 - 7 - 15

W.P. 174 - 65

CLIENT: D. H. O.

PROJECT: Q. E. W. & HWY NO 27 INTERCHANGE, BRIDGE NO 21

METHOD OF BORING: WASHBORING

DIAMETER OF BOREHOLE 2 3/8"

ENCLOSURE NO.

LOCATION: 85' RT of STA. 258 + 97

DATE AUG. 17 & 18, 1966

DATUM ELEVATION: G. S. C.

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE blows per foot		CONSISTENCY water content % PL W LI	REMARKS
				NUMBER	TYPE	N or Advancement of Sampler	2.0 4.0 6.0 8.0 10.0	SHEAR STRENGTH lbs/sq ft		
315.4	0	GROUND SURFACE								
		Loose to Compact Brown SANDY, CLAYEY SILT (FILL)		1	SS	16				
310.0	5									
307.9	7.5	Very Dense, Brown SANDY SILT								
306.0	9.4									
305.0	10	Grey Extremely Weathered SHALE		2	S.S.	100/5				
				3	W.S.					
				4	S.S.	100/NP				
				5	R.C.	30%				
				6	S.S.	100/1				
300.0	15									
		Grey SHALE BEDROCK		7	R.C.	65%				
295.0	20			8	R.C.	50%				
290.0	25			9	R.C.	98%				
287.5	28.0	END OF BOREHOLE								
285.0	30									

W.L. El. 305.6'  
AUG. 25, 1966



# GEOTECHNICAL DATA SHEET FOR BOREHOLE 156..

OUR REFERENCE NO 6-7-15

W.P. 174-65-1

CLIENT: D. H. O.

PROJECT: Q.E.W. & HWY. No. 27 INTERCHANGE, BRIDGE No 21

LOCATION: 85' RT. of STA. 258 + 67

DATUM ELEVATION: G. S. C.


METHOD OF BORING: WASHBORING

DIAMETER OF BOREHOLE: 3"

DATE: AUG. 17, 1966.

ENCLOSURE NO

ELEVATION ft.	DEPTH ft	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE blows per foot					CONSISTENCY water content %			REMARKS
				NUMBER	TYPE	N- or Advancement of Sampler	20	40	60	80	100	PL	W	LI	
307.8	0	GROUND SURFACE													
		Dense, Brown SANDY SILT													
305.0	2.6			1	CS										
303.8	4.0	Grey, Extremely Weathered SHALE		2	SS	100/NP									
	5	Grey SHALE													
300.0	7.5	Weathered Sound		3	RC 50 %										
	10	BEDROCK													
295.0				4	RC 100 %										
	15														
290.0				5	RC 100 %										
288.5															
	20	END OF BOREHOLE													



W.L. El. 304.0 ft  
Aug. 25, 1968

W.L. El. 304.0 ft  
Aug. 25, 1966

# GEOTECHNICAL DATA SHEET FOR BOREHOLE 157

OUR REFERENCE NO. 6-7-15  
W. P. 174-65-1

CLIENT: D. H. O.

PROJECT: Q.E.W. & HWY. No 27. INTERCHANGE, BRIDGE No 21

LOCATION: 85' RT. of STA. 257+03

DATUM ELEVATION: G. S. C.

METHOD OF BORING: WASHBORING

DIAMETER OF BOREHOLE: 3"

DATE: AUG. 11, 1966

ENCLOSURE NO.

ELEVATION ft	DEPTH ft	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE blows per foot					CONSISTENCY water content %		REMARKS
				NUMBER	TYPE	No. of or Advancement of Sampler	20	40	60	80	100	PL	W	
305.6	0	GROUND SURFACE												
		5" ORGANIC TOPSOIL		1	SS	75								
		Grey		2	SS	50/1/2"								
		Extremely Weathered												
		Broken SHALE												
305.6	5			3	SS	50/NP								
300.0				4	RC	70 %								
		Grey												
		SHALE												
		BEDROCK		5	RC	83 %								
295.0	10													
293.4	12.2													
		END OF BOREHOLE												
290.0	15													

W.L. El. 305.1  
AUG. 25, 1966

Gr. 34 % ; Sa. 21 %  
Sl. 27 % ; Cl. 18 %

# GEOTECHNICAL DATA SHEET FOR BOREHOLE 158.

OUR REFERENCE NO. 6-7-15

W.P. 174-65-1

CLIENT: D. H. O.

PROJECT: Q. E. W. & HWY. No. 27 INTERCHANGE, BRIDGE No 21

LOCATION: 85' RT. of STA. 256 + 71

DATUM ELEVATION: G. S. C.

METHOD OF BORING: WASHBORING

DIAMETER OF BOREHOLE 3"

DATE: AUG. 10. 1966.

ENCLOSURE NO

ELEVATION ft	DEPTH ft	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE blows per foot		CONSISTENCY water content %		REMARKS
				NUMBER	TYPE	N Advancement of Sampler	20	40	60	80	
317.0	0	GROUND SURFACE									
315.0		Generally Compact to Dense Brown SANDY SILT with some gravel and a trace of clay.									
310.0	5	(FILL)		1	SS	13					
307.5	9.5			2	SS	40					
305.0	10	Grey Extremely Weathered SHALE									
303.7	13.3										
300.0	15	Grey SHALE with layers of LIMESTONE BEDROCK		3	RC 56 %						
				4	RC 87 %						
295.0	20			5	RC 79 %						
290.0	23	END OF BOREHOLE									

W.L. 307.0 ft.  
AUG. 25, 1966

# GEOTECHNICAL DATA SHEET FOR BOREHOLE 159...

OUR REFERENCE NO. 6-7-15

W.P. 174-65-1

CLIENT: D.H.O.

PROJECT: Q.E.W. & HWY. NO 27 INTERCHANGE, BRIDGE NO 21 DIAMETER OF BOREHOLE: 2 3/8"

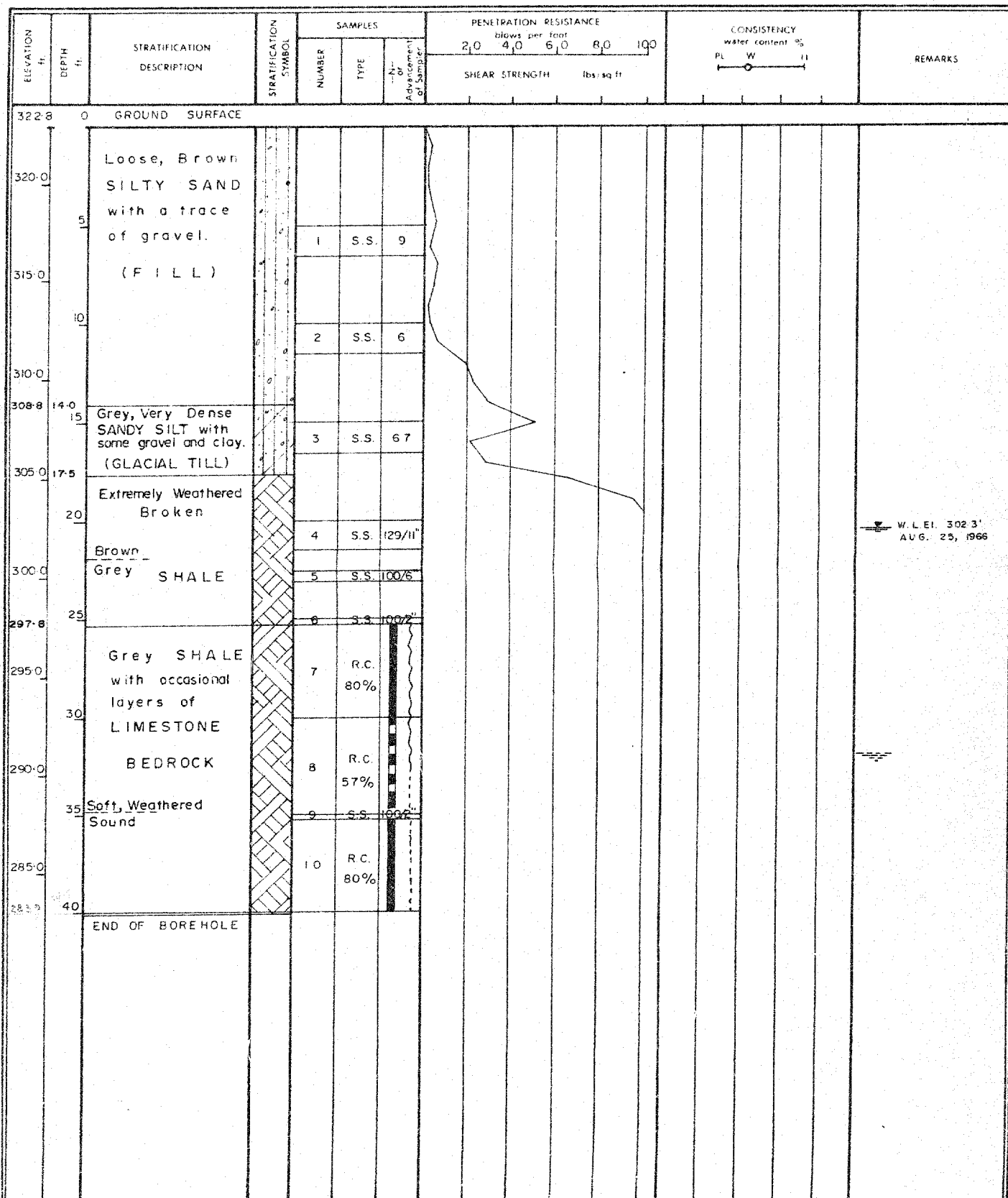
LOCATION: 73' LT. of STA. 258 + 97

DATUM ELEVATION: G.S.C.

METHOD OF BORING: WASHBORING

ENCLOSURE NO.

DATE: AUG. 17-19, 1966



VERTICAL SCALE: 1 IN TO 5 FT.

DOMINION SOIL INVESTIGATION LIMITED

MADE: D. A. M. CHD.

# GEOTECHNICAL DATA SHEET FOR BOREHOLE . 160 . .

OUR REFERENCE NO. 6-7-15  
W.P. 174-65-1

CLIENT: D. H. O.

PROJECT: Q.E.W. & HWY. No. 27. INTERCHANGE, BRIDGE No 21

LOCATION: 73' LT. of STA. 258+67

DATUM ELEVATION: G. S. C.

METHOD OF BORING: WASHBORING

DIAMETER OF BOREHOLE: 3"

DATE: AUG. 17, 1966.

ENCLOSURE NO.

ELEVATION ft	DEPTH ft	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE blows per foot				CONSISTENCY water content %				REMARKS
				NUMBER	TYPE	N- or Advancement of Sampler	20	40	60	80	100	PL	W	LI	
304.5	0	GROUND SURFACE													
		Compact to Dense Brown SILTY SAND & GRAV.		1	A B	SS									Gr. 39% ; Sa. 33 % Si - Cl. 28 %
301.6	2.9	V. Hard, Grey, CLAYEY SILT with some sand and gravel		2		SS									
300.0	4.2	Grey, Weathered Broken SHALE		3		RC 33 %									Gr. 8 % ; Sa. 20 % Si. 53 %; Cl. 19 %
298.0	6.5			4		SS									
		Grey SHALE with layers of LIMESTONE BEDROCK.		5		RC 86 %									
295.0	10			6		RC 86 %									
290.0	15	END OF BOREHOLE													
285.0	20														

W.L. El. 301.7'  
AUG. 25, 1966

OUR REFERENCE NO. 6-7-15  
W.P. 174-65-1

PROJECT: Q. E. W. & HWY. No. 27. INTERCHANGE, BRIDGE No. 21

## METHOD OF BORING WASHBORING

DIAMETER OF BOREHOLE 3"

DATE: AUG. 10, 1966.

DATUM ELEVATION: G. S. C.

ENCLOSURE NO.

[illegible]

# GEOTECHNICAL DATA SHEET FOR BOREHOLE 162..

OUR REFERENCE NO. 6-7-15

W.P. 174-55-1  
CLIENT: D. H. O.

PROJECT: Q.E.W. & HWY No. 27 INTERCHANGE, BRIDGE No. 2; DIAMETER OF BOREHOLE 3"  
METHOD OF BORING WASHBORING

ENCLOSURE NO.

LOCATION 73' LT. of STA. 256+71

DATE: AUG. 11, 1966

DATUM ELEVATION: G. S. C.

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE blows per foot					CONSISTENCY water content %			REMARKS
				NUMBER	TYPE	N or Advance of Sampler	20	40	60	80	100	PL	W	LI	
323.1	0	GROUND SURFACE													
320.0	5	Compact to Dense Brown SILTY SAND with a trace of clay and gravel.		1	SS	39									Gr. 5% ; S <sub>a</sub> 40% Si. - Cl. 55%
315.0	10	(FILL)		2	SS	13									Gr. 4% ; S <sub>a</sub> 62% Si. - Cl. 34%
310.0	15			3	SS	25									Gr. 3% ; S <sub>a</sub> 52% Si. 30% ; Cl. 15%
305.7	17.4	Grey, Weathered SHALE													Refusal in Cone Test at El. 305.7 ft.
304.8	18.5														
300.0	20	Grey CALCAREOUS SHALE		4	RC	73%									W.L. El. 303.1' AUG. 12, 1966
295.0	25	BEDROCK		5	RC	60%									
290.0	30			6	RC	81%									
285.0	35	END OF BOREHOLE													

VERTICAL SCALE: 1 IN. TO 5 FT.

DOMINION SOIL INVESTIGATION LIMITED

MADE: V. G. H. CHD.

OUR REFERENCE NO. 6-7-15

W.P. 174 - 65 - 1

CLIENT: D. H. O.

PROJECT: Q.E.W. & HWY. No. 27 INTERCHANGE, BRIDGE No 21

LOCATION: 67' RT. of STA. 258+32

DATUM ELEVATION: G. S. C.

METHOD OF BORING: WASHBORING

DIAMETER OF BOREHOLE 3"

DATE: AUG. 16. 1966.

ENCLOSURE NO.

[illegible]





# GEOTECHNICAL DATA SHEET FOR BOREHOLE . . 165 .

OUR REFERENCE NO. 6-7-15

W. P. 174-65-1

CLIENT: D. H. O.

PROJECT: Q. E. W. & HWY. No. 27. INTERCHANGE, BRIDGE No 21

LOCATION 60' LT. of STA. 258+32

DATUM ELEVATION: G. S. C.

METHOD OF BORING: WASHBORING

DIAMETER OF BOREHOLE 3"

DATE: AUG. 16. 1966.

ENCLOSURE NO

ELEVATION ft	DEPTH ft	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE blows per foot					CONSISTENCY water content %			REMARKS
				NUMBER	TYPE	N or Advancement of Sampler	20	40	60	80	100	P <sub>1</sub>	W	LI	
300.0	0	CREEK BOTTOM													
		4" SAND & GRAVEL		1	SS	100/4"									
		Grey SHALE with layers of LIMESTONE		2	RC	85 %									
295.0	5	BEDROCK		3	RC	100 %									
290.0	10	END OF BOREHOLE													
285.0	15														

W. L. El. 300.6 ft.

OUR REFERENCE NO. 6-7-15  
W. P. 174 - 65 - 1  
CLIENT: D. H. O.

# GEOTECHNICAL DATA SHEET FOR BOREHOLE 166.

PROJECT: Q. E. W. & HWY. No. 27. INTERCHANGE, BRIDGE No 21  
LOCATION: 60' LT. of STA. 257 + 38  
DATUM ELEVATION: G. S. C.

METHOD OF BORING WASHBORING  
DIAMETER OF BOREHOLE 3"  
DATE: AUG. 9. 1966.

ENCLOSURE NO.

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE blows per foot					CONSISTENCY water content %			REMARKS
				NUMBER	TYPE	N- or Advancement of Sampler	20	40	60	80	100	PL	W	LI	
304.9	0	GROUND SURFACE													
302.6	2.3	SAND, GRAVEL and BOULDERS													
300.0	5	Grey, Weathered Broken SHALE		1	SS	50/4"									
297.4	7.5			2	RC	17 %									
295.0	10	Grey SHALE with layers of LIMESTONE.		3	RC	60 %									
				4	RC	17 %									
290.0	15	BEDROCK		5	RC	61 %									
				6	RC	97 %									
285.0	20	END OF BOREHOLE													

W.L. EL. 302.4'  
AUG. 10, 1966

Gr. 29% ; So. 14%  
Sl. 39% ; Cl. 18%

# GEOTECHNICAL DATA SHEET FOR BOREHOLE 167.

OUR REFERENCE NO. 6-7-15

W. P. 174-65-1

CLIENT: D. H. O.

PROJECT: Q.E.W. & HWY. No. 27. INTERCHANGE, BRIDGE No. 21

METHOD OF BORING: WASHBORING

DIAMETER OF BOREHOLE: 3"

ENCLOSURE NO.

LOCATION: 44' RT. of STA. 256+60

DATE: AUG. 9, 1966.

DATUM ELEVATION: G. S. C.

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE blows per foot					CONSISTENCY water content %			REMARKS
				NUMBER	TYPE	N or Advance of Sample	20	40	60	80	100	PL	W	LI	
327.7	0	GROUND SURFACE													
325.0	5	Compact, Brown GRAVELLY SAND with some silt.		1	SS	11									
320.0	10	( F I L L )		2	SS	11									
315.0	15			3	SS	8									
310.0	20			4	SS	12									
308.2	21.5	Very Hard CLAYEY SILT with a trace of sand and gravel.		5	SS	18									
305.0	24.2			6	SS	63									
303.5	25	Grey Extremely Weathered SHALE		7	SS	75/3"									
300.0	27.5														
295.0	30	Grey CALCAREOUS SHALE BEDROCK		8	RC	71%									
290.0	35			9	RC	86%									
290.0	40	END OF BOREHOLE													

Gr. 37% ; Sa. 55%  
Si. 8%

Cave - In  
El. 311.2 ft.  
Aug. 25, 1966

Gr. 3% ; Sa. 9%  
Si. 57% ; Cl. 31%

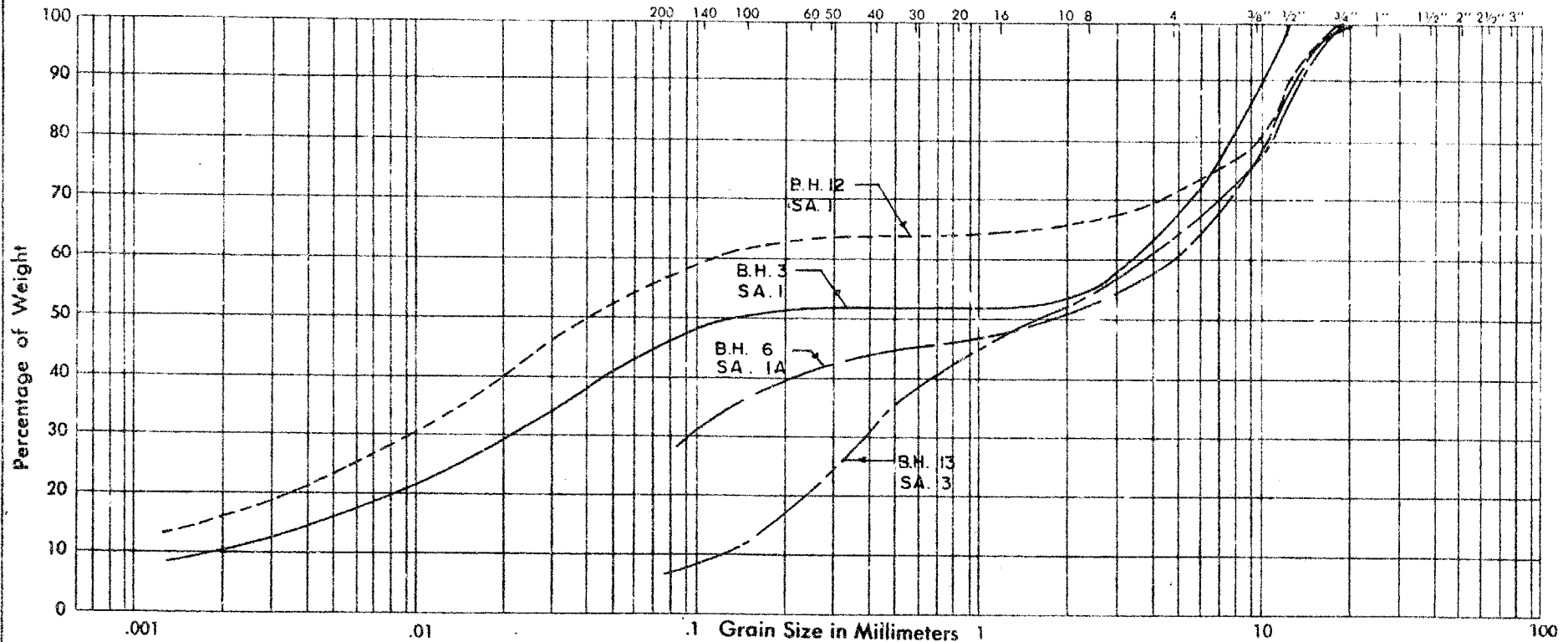
# DOMINION SOIL INVESTIGATION LIMITED

## GRAIN SIZE DISTRIBUTION

OUR REFERENCE NO 6-7-15

UNIFIED SOIL CLASSIFICATION  
SYSTEM

SILT AND CLAY	SAND			GRAVEL	
	FINE	MEDIUM	COARSE	FINE	COARSE



PROJECT: Q.E.W. & HWY. NO 27 INTERCHANGE

LOCATION: BRIDGE NO 21  
157, 160, 166, 167

BOREHOLE NO.: 3, 6, 12, 13

SAMPLE NO.: 1, 1A, 1, 3

DEPTH OF SAMPLE:

ELEVATION OF SAMPLE:

COEFFICIENT OF UNIFORMITY

COEFFICIENT OF CURVATURE

**Classification of Sample and Group Symbol:**

GRAVELLY SAND & SILT  
with a trace of clay.

PLASTIC PROPERTIES:

LIQUID LIMIT % =

PLASTIC LIMIT % =

PLASTICITY INDEX % =

MOISTURE CONTENT % =

ACTIVITY =

Enclosure No.

DOMINION SOIL INVESTIGATION LIMITED  
77 CROCKFORD BOULEVARD - SCARBOROUGH ONTARIO CANADA - TELEPHONE 751-6565

BRANCH  
389 QUEENS AVENUE  
LONDON, ONTARIO  
TELEPHONE GE. 8-8851



FOUNDATION ENGINEERS

ASSOCIATED COMPANY  
SOIL TESTING AND ENGINEERING LTD.  
34 BRENTFORD ROAD,  
KINGSTON 5, JAMAICA, WEST INDIES  
TELEPHONE: 66886

September 21, 1966.

Our Ref: 6-8-15  
Your Ref: W.P. 174-65-2

Mr. A. G. Stermac,  
Principal Foundation Engineer,  
Materials & Testing Division,  
Department of Highways, Ontario,  
Downsview Avenue,  
Downsview, Ontario.

Attention: Mr. K. Selby, P. Eng.

Re: Soil Investigation for Q.E.W. and Highway #27  
Interchange, Bridge #22

Dear Sirs:

Enclosed are eleven copies of the records of  
Boreholes No. 133 to 140 inclusive.

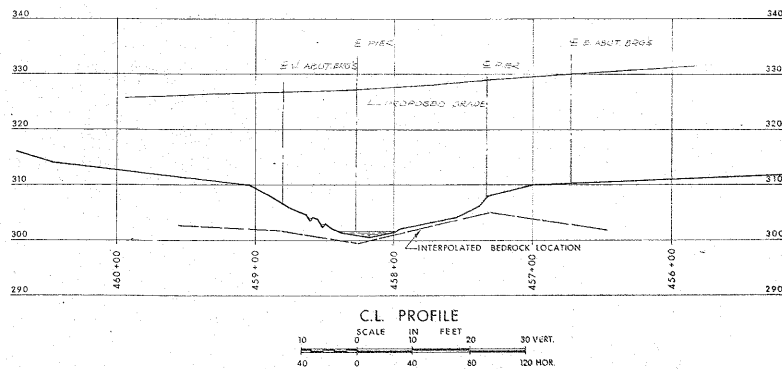
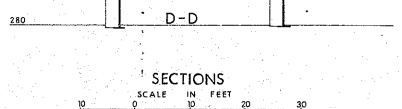
We trust that the information contained in the  
Borehole Logs is sufficient for your requirements.

Yours very truly,

DOMINION SOIL INVESTIGATION LIMITED,

I. P. Lieszkowszky, P. Eng.

IPL/jvm  
Encls.



DEPARTMENT OF HIGHWAYS - ONTARIO			
MATERIALS & TESTING DIVISION - FOUNDATION SECTION			
BRIDGE No.22			
RAMP W-Ev. OVER ETOBICOKE CREEK			
KING'S HIGHWAY NO. <u>Q.E.W. &amp; HWY. No.27</u> INTER. DIST. NO. <u>1</u>			
C.O. YORK		METRO TORONTO	
TWP. ETOBICOKE		LOT <u>      </u> CON. <u>      </u>	
BORE HOLE LOCATIONS & SOIL STRATA			
SUBM'D K.S.	CHECKED	WR NO. 174-05-2	M.S.T. DRAWING NO.
DRAWN ON	CHECKED	JOB NO. 65-P-104U	65-P-104U
DATE	6 OCT 64	SITE NO.	BRIDGE DRAWING NO.
APPROVED <i>(Signature)</i> CONTRACT NO.			

[illegible]

# GEOTECHNICAL DATA SHEET FOR BOREHOLE 133.

OUR REFERENCE NO. 6-8-15

CLIENT: D.H.O.

PROJECT: Q.E.W. & HWY. NO 27 INTERCHANGE - BRIDGE #22

METHOD OF BORING: WASH BORING

ENCLOSURE NO.

LOCATION: 175, 167 N; 206, 590 E.

DIAMETER OF BOREHOLE 2 3/8"  
DATE: AUG. 19-23, 1966

DATUM ELEVATION: G.S.C.

W. P. 174-65-2

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE blows per foot					CONSISTENCY water content %			REMARKS
				NUMBER	TYPE	N- or Advance- ment of Sampler	20	40	60	80	100	PL	W	LI	
307.4	0	GROUND SURFACE													
		Compact, Brown GRAVELLY SAND													
305.0	2.0	Compact to Dense Brown		1	S.S.	37									
	5	GRAVEL, some sand													
		CLAYEY SILT LAYER		2	S.S.	115/10"									
300.4	7.0	Grey		3	S.S.	100/2"									
300.0		extremely weathered		4	S.S.	100/2"									
		SHALE		5	R.C.	86%									
295.0		BEDROCK		6	S.S.	100/2"									
	15	intermittent layers of LIMESTONE		7	R.C.	68%									
290.0				8	S.S.	100/2"									
	20			9	R.C.	47%									
285.0															
	25			10	R.C.	84%									
280.0	27.5	END OF BOREHOLE													
279.3															
	30														

W. L. E.I. 303.1'  
AUG. 25, 1966

VERTICAL SCALE: 1 IN TO 5 FT.

DOMINION SOIL INVESTIGATION LIMITED

MADE: D. A. M. CH'D.



# GEOTECHNICAL DATA SHEET FOR BOREHOLE . . 134.

OUR REFERENCE NO. 6 - 8 - 15

CLIENT: D. H. O.

PROJECT: Q. E. W. 8 HWY. NO 27 INTERCHANGE - BRIDGE NO 22

LOCATION: 175,146 N; 206,611 E

DATUM ELEVATION: G. S. C.

METHOD OF BORING: WASH BORING

DIAMETER OF BOREHOLE: 2 3/8"

DATE: AUG. 23 & 24, 1966

W. P. 174 - 65 - 2

ENCLOSURE NO.

ELEVATION ft	DEPTH ft	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE blows per foot					CONSISTENCY water content %			REMARKS
				NUMBER	TYPE	N- or Advancement of Sampler	2.0	4.0	6.0	8.0	10.0	PL	W	LI	
304.7	0	GROUND SURFACE													
303.7	1.0	Soft ORGANIC SILT													
302.7	2.0	CLAYEY SILT													
300.0	5	Grey SHALE BED ROCK		1	S.S.	100/1"									
				2	S.S.	100/3"									
				3	S.S.	100/4"									
295.0	10	extremely weathered broken		4	S.S.	100/1"									
		weathered		5	R.C.	48%									
290.0	15			6	R.C.	78%									
		sound		7	R.C.	80%									
285.0	20														
280.0	25														
		END OF BOREHOLE													

W.L. El. 302.7'  
AUG. 25, 1966

OUR REFERENCE NO 6-8-15

## GEOTECHNICAL DATA SHEET FOR BOREHOLE . 135 .

CLIENT: D. H. O.

METHOD OF BORING: WASH BORING

PROJECT: Q. E. W. & HWY. NO 27 INTERCHANGE-BRIDGE NO 22 DIAMETER OF BOREHOLE: 2  $\frac{3}{8}$  "

ENCLOSURE NO

LOCATION: 175, 202 N; 206, 625 E.

DATE: AUG. 25, 1966

DATUM ELEVATION: G. S. C.

W. P. 174 - 65 - 2

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE blows per foot		CONSISTENCY water content %		REMARKS	
				NUMBER	TYPE	N or Advancement of Sampler	2.0 4.0 6.0 8.0 10.0	SHEAR STRENGTH lbs/sq ft	PL W LI			
299.8	0	GROUND SURFACE (CREEK BOTTOM)										
		weathered broken		1	S.S.	100/NP						
297.5		soft		2	R.C.	27%						
295.0	5			3	S.S.	100/3"						
294.5		sound		4	R.C.	100%						
290.0	10	BEDROCK		5	R.C.	82%						
287.5	12.5	END OF BOREHOLE										
285.0	15											

W.L. E1301.0 ft.



OUR REFERENCE NO. 6 - 8 - 15

## GEOTECHNICAL DATA SHEET FOR BOREHOLE . . 137.

CLIENT: D.H.O.

PROJECT: Q.E.W. &amp; HWY. No 27 INTERCHANGE - BRIDGE No 22

METHOD OF BORING: WASH BORING

DIAMETER OF BOREHOLE: 2 3/8"

ENCLOSURE NO.

LOCATION: 175,273 N; 206,698 E.

DATE: AUG 22 &amp; 23, 19

DATUM ELEVATION: G.S.C.

WP 174 - 65 2

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE blows per foot					CONSISTENCY water content %	REMARKS
				NUMBER	TYPE	1 2 or Advancement of Sampler	2,0	4,0	6,0	8,0	10,0		
							SHEAR STRENGTH lbs/sq ft						
307.6	0	GROUND SURFACE											
305.0		Very Dense Brown GRAVEL some sand and silt											
304.8	3.0			1	S.S. 100%								
	5	Grey SHALE		2	R.C. 23%								
		BEDROCK		3	S.S. 100%								
		layers of		4	R.C. 46%								
300.0		LIMESTONE		5	S.S. 100%								
	10			6	R.C. 73%								
				7	S.S. 100%								
295.0		weathered sound		8	R.C. 53%								
290.0	15			9	R.C. 90%								
288.6	19.0	END OF BOREHOLE											

W.L. El 302.0'  
AUG 25, 1960

# GEOTECHNICAL DATA SHEET FOR BOREHOLE .13.8.

OUR REFERENCE NO 6-8-15

CLIENT: D. H. O.

PROJECT Q.E.W. & HWY. NO 27 INTERCHANGE - BRIDGE NO 22

METHOD OF BORING WASH BORING

DIAMETER OF BOREHOLE 2 3/8"

ENCLOSURE NO

LOCATION 175, 252 N; 206, 719 E.

DATE AUG 23 & 24, 1966

DATUM ELEVATION G.S.C.

WP 174 - 65 - 2

ELEVATION ft	DEPTH ft	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE blows per foot					CONSISTENCY water content %			REMARKS
				NUMBER	TYPE	N & Advance of Sample	2.0	4.0	6.0	8.0	10.0	PL	W	LI	
308.1	0	GROUND SURFACE													
305.0		Compact, Brown CLAYEY SILT with some gravel (FILL)		1	S.S.	38									
303.6	4.5			2	S.S.	50/3									
	5	Grey SHALE BEDROCK		3	R.C.	67%									
300.0				4	S.S.	100/NP									
	10			5	R.C.	37%									
				6	S.S.	100/4									
295.0		weathered sand		7	R.C.	17%									
	15			8	S.S.	100/1									
				9	R.C.	90%									
290.0				10	S.S.	100/NP									
	20			11	R.C.	70%									
285.0				12	R.C.	90%									
283.1	25	END OF BOREHOLE													

W.L. El. 302.1'  
AUG. 25, 1966

OUR REFERENCE NO. 6 - 8 - 15

## GEOTECHNICAL DATA SHEET FOR BOREHOLE . . 139 .

CLIENT: D. H. O.

PROJECT: Q. E. W. &amp; HWY. No 27 INTERCHANGE - BRIDGE No 22

LOCATION: 175, 307 N; 206,733 E.

DATUM ELEVATION: G.S.C.

METHOD OF BORING: WASHBORING


DIAMETER OF BOREHOLE: 2 3/8"

DATE: AUG. 19, 1966

W. P. 174 - 65 - 2

ENCLOSURE NO.

ELEVATION ft	DEPTH ft	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE					CONSISTENCY water content %  PL      W      LI	REMARKS
				NUMBER	TYPE	N 0 Advancement of Sampler	2.0	4.0	6.0	8.0	100		
306.9	0	GROUND SURFACE											
305.9	1.0	SANDY SILT (FILL)											
305.0		TOP SOIL											
304.4	2.5	ORGANIC SANDY SILT		1	S.S.	13							
303.2	3.7	Dense, Brown											
	5	SANDY SILT (TILL)		2	S.S.	85/5"							
301.4	5.5	Grey		3	R.C.	32%							
300.0		SHALE		4	S.S.	100/NP							
	10	BEDROCK		5	R.C.	37%							
295.0		weathered											
	15	sound		6	R.C.	80%							
290.0													
286.9	18.0	END OF BOREHOLE											
	20												
285.0													

 W.L. El. 302.9'  
AUG. 25, 1966

OUR REFERENCE NO 6 - 8 - 15

## GEOTECHNICAL DATA SHEET FOR BOREHOLE . . . 140 . . .

CLIENT: D.H.O.

PROJECT: Q.E.W. &amp; HWY. No 27 INTERCHANGE-BRIDGE No 22

LOCATION 175, 285 N; 206, 754 E.

DATUM ELEVATION: G.S.C.

METHOD OF BORING WAC 1 BORING

DIAMETER OF BOREHOLE 2 3/8"

DATE AUG. 18 &amp; 19, 1966


W.P. 174 - 65 - 2

ENCLOSURE NO

ELEVATION ft	DEPTH ft	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE blows per foot					CONSISTENCY water content %			REMARKS
				NUMBER	TYPE	N- or Advancement of Sampler	20	40	60	80	100	PL	W	LI	

310.8	0	GROUND SURFACE													
310.0		Dense, Brown GRAVELLY to CLAYEY SILT (FILL)		1	S.S.	35									
306.3	4.5	Very Dense SAND and GRAVEL		2	S.S.	90/6									
305.0															
303.5	7.3	Grey SHALE		3	S.S.	75/5									
				4	R.C.	68%									
300.8	10	weathered sound		5	S.S.	100/5									
300.0				6	R.C.	96%									
				7	S.S.	100/1									
		BEDROCK													
		intermittent LIMESTONE layers		8	R.C.	83%									
295.0	15														
293.2	17.5	END OF BOREHOLE													
	20														
290.0															

W.L. EL. 306.4'  
AUG. 25, 1966

 W.L. El. 306.4'  
AUG. 25, 1966

**Appendix F**  
**Comparison of Foundation Alternatives**



### COMPARISON OF FOUNDATION ALTERNATIVES

Spread Footings on Bedrock	Spread Footings on Granular Pad Constructed on Bedrock	Augered H-piles Socketed in Shale Bedrock	Caissons / Drilled Shafts
<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. Relative ease of construction.</li> <li>ii. More cost effective than deep foundations.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. Relative ease of construction.</li> <li>ii. More cost effective than deep foundations.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. Higher geotechnical resistance comparing to spread footings</li> <li>ii. Installation less influenced by weather and groundwater compared to spread footing</li> <li>iii. Facilitate the integral abutment design.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. Higher geotechnical resistance is available for caissons socketed into bedrock than for socketed piles.</li> <li>ii. Construction of caissons could continue in freezing weather.</li> </ul>
<p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Potentially deep excavation at abutments</li> <li>ii. May require groundwater control.</li> <li>iii. Not feasible for integral abutment design.</li> </ul>	<p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Lower bearing capacity than for footings placed directly on bedrock.</li> <li>ii. Potentially deep excavation at abutments</li> <li>iii. May require groundwater control.</li> <li>iv. Not feasible for integral abutment design.</li> </ul>	<p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Higher unit costs than spread footings.</li> <li>ii. Piles will require sockets.</li> <li>iii. Potential difficulties penetrating hard limestone layers during augering for pile sockets.</li> </ul>	<p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Higher unit cost than for spread footings or H-piles.</li> <li>ii. Temporary liners will be required to install caissons through cohesionless soils.</li> <li>iii. Difficulty in sealing liners at base.</li> <li>iv. Potential difficulties penetrating hard limestone layers during augering.</li> <li>v. Difficulties in cleaning and inspecting bases.</li> </ul>
Low risk of encountering problems during construction.	Low risk of encountering problems during construction.	High risk of encountering harder layers within bedrock that would require additional procedures to advance the augers to the desired elevation.	High risk of encountering harder layers within bedrock that would require additional procedures to advance the augers to the desired elevation.
<b>RECOMMENDED</b>	<b>RECOMMENDED</b>	<b>FEASIBLE</b>	<b>FEASIBLE</b>