

**FOUNDATION INVESTIGATION AND DESIGN REPORT
PRELIMINARY DESIGN AND ENVIRONMENTAL ASSESSMENT
QEW BRIDGE TWINNING OVER CREDIT RIVER
MISSISSAUGA, ONTARIO
W.O. 08-20008, ASSIGNMENT NO. 2008-E-0025**

GEOCRES Number: 30M12-341

Report to

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May 18, 2012
File: 19-1351-174

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Bridge Over Credit River FIDR FINAL Rep may 12.doc

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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 widening of the existing Queen Elizabeth Way (QEW) bridge over the Credit River. This investigation was carried out in support of the preliminary design, environmental assessment and planning for the bridge widening over the river. These works are part of the project involving preliminary design for widening of QEW from West of Hurontario Street to West of Mississauga Road in Mississauga, Ontario.

The purpose of the investigation was to explore the subsurface conditions at selected foundation element locations and, based on the data obtained, to provide a borehole locations and soil strata drawing, records of boreholes, stratigraphic profile and cross-sections, 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 and selected data from previous investigations.

Thurber was retained by McCormick Rankin Corporation (MRC) to carry out the foundation investigation at this site on behalf of the Ministry of Transportation Ontario (MTO) under W.O. 08-20008 and Consultant Assignment No. 2008-E-0025.

2 SITE DESCRIPTION

The existing QEW Credit River bridge is comprised of seven spans and carries six lanes of traffic. These spans include two approach spans over the east valley slope, one approach span over the west valley slope, and four intermediate spans crossing the floodplain and river channel. The bridge twinning is currently proposed to be located immediately to the north of the existing bridge. The existing valley slopes at the bridge location are at an inclination of approximately 1.7H : 1V on the west side and approximately 4H : 1V on the east side, and are moderately vegetated with tall grass, shrubs and some trees.

The river valley is incised up to approximately 19 m below the surrounding plateau. The valley slopes are predominantly formed through shale bedrock. On the plateau, shale underlies overburden soils or fill at shallow depth. The drainage at the site is directed towards Credit River, which flows southward to Lake Ontario.

On the west plateau to the north of QEW, the terrain is relatively flat with residential dwellings located at some distance beyond the proposed bridge footprint. Vegetation is moderate consisting mainly of tall grass, shrubs and occasional small to large trees. An access road cut up to 13 m deep is under construction through the west valley slope to access the west floodplain for bridge rehabilitation purposes. At the east plateau, the vegetation is denser with some residential dwellings located within the proposed bridge footprint. The slope configuration also appears to have been modified by existing fill placed on the original valley slope.

From published geological information, the area of and adjacent to the Credit River valley is situated within the physiographic region known as the Iroquois Plain. In this area, the relatively thin native soil deposits typically consist of cohesive soils (some tills) overlying shale bedrock of the Georgian Bay Formation. The till is known to consist of shale and limestone fragments. Alluvial deposits in the form of clayey silts, silts and fine sands are present within the river floodplain.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project consisted of drilling on the plateau and river floodplain for the proposed QEW bridge twinning. The field work was carried out between May 30 and June 9, 2011 during which time Boreholes 11-01 and 11-02 were drilled and sampled to depths of 7.1 and 8.4 m, respectively. The approximate locations of both boreholes are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix C. Boreholes from previous investigations that are considered relevant to this project are also included on the drawing. The following lists the relevant past investigation reports referenced in this report.

- Thurber report titled “Foundation Investigation Report, Construction Access Road for Bridge Rehabilitation, QEW Bridge Over Credit River, Mississauga, Ontario, W.P. 2186-07-00, GEOCREs No. 30M12-324, File:19-92-92, dated April 8, 2011 (Reference 1).
- Trow, Soderman and Associates letter report titled “Core Drilling to Determine Underside of Existing Footings, Oakville Creek and Credit River Bridges, Queen Elizabeth Highway”, 58-F-264c, Dept. of Highways of Ontario dated August 1958 (Reference 2).
- Dept. of Highways of Ontario archived drawing, D-2241-1, which includes boreholes advanced in 1933 near the existing foundation locations, 1933 (Reference 3).

For the present investigation, the planned borehole locations were staked and/or marked in the field by Thurber. Utility clearance was obtained at all borehole locations by Thurber prior to drilling. Borehole survey data including northings, eastings and ground surface elevations has been provided by MRC to Thurber.

It is noted that two other boreholes had originally been planned for the east plateau and east floodplain, respectively. Due to access constraints and permission to enter restrictions amongst other constraints, it was concluded following discussions with MRC and MTO that the boreholes on the east side of the river will not be drilled at this time. Instead, the available past borehole information will be utilized to provide preliminary foundation recommendations.

OGS Inc. of Almonte, Ontario supplied and operated a Hilti DD-250E tri-pod for advancing Boreholes 11-01 and 11-02 due to the constraints imposed by the overhead power cables and difficult access for conventional drill rigs down to the floodplain, respectively. Wash boring techniques were used to advance the boreholes through soils and highly weathered bedrock. Prior to wash boring, soil and weathered bedrock samples were obtained at selected intervals using a split spoon sampler in conjunction with the Standard Penetration Testing (SPT). Within the floodplain, soil samples in Borehole 11-02 were obtained using a 50-lb hammer instead of a standard 140-lb hammer. Once the top of weathered bedrock was encountered, both boreholes were further advanced into bedrock by NQ size coring equipment to recover core samples.

Groundwater conditions in the open boreholes were observed throughout the drilling operations. Standpipe piezometers were installed in Boreholes 11-01 and 11-02 to permit monitoring of the groundwater level. At this site, 31 mm to 38 mm diameter Schedule 40 PVC pipes with 1.5 m long slotted screens were installed in the boreholes. The sand screen surrounded the pipe and extended above the slotted portion of the pipe. Bentonite holeplug seals were placed above the sand screen in each installation. Borehole completion details are presented in Table 1 following the text.

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

All rock cores were logged, and properties including the Total Core Recovery (TCR), Rock Quality Designation (RQD) and the Fracture Indices (FI) were determined where applicable.

4 LABORATORY TESTING

All recovered soil samples were subjected to visual identification and to natural moisture content determination. The results of this testing are shown on the Record of Borehole sheets in Appendix A. Selected soil samples were subjected to gradation analysis. Atterberg Limits Tests were performed on some of the cohesive samples. The results of this testing program are presented on the Record of Borehole sheets in Appendix A and on the figures in Appendix B.

Suitable rock cores were not available for carrying out the Point Load Test (PLT). Results available from Reference 2 are attached in Table 2 following the text.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets in Appendix A. Details of the encountered soil and rock stratigraphy are presented in these records and on the "Borehole Locations and Soil Strata"

drawing in Appendix C. A summary of bedrock information from current and previous boreholes is shown on an aerial photograph of the site included in Appendix C. A 1933 bridge plan and profile showing the original 1933 boreholes is also included in Appendix C. General description of the stratigraphy based on boreholes advanced during the current and past investigations is given in the following paragraphs. The factual information established at the borehole locations governs any interpretation of site conditions.

In general, the Credit River floodplain is underlain by alluvium and the plateau is underlain by thin deposits of overburden soil. Shale bedrock is present at relatively shallow depths at both locations.

5.1 West Plateau

5.1.1 Topsoil

Topsoil of about 200 mm in thickness was encountered in Borehole 11-01. The access road boreholes (Reference 1) indicated a topsoil thickness ranging from 50 to 150 mm. Although absent in Borehole 11-02, it is anticipated that topsoil is present elsewhere within the floodplain. Topsoil thickness may vary between and beyond the borehole location.

5.1.2 Silty Clay

In Boreholes 11-01 on the plateau, the topsoil is underlain by a deposit of silty clay with trace sand which extends to a depth of 3.3 m below existing ground surface, or approximate Elevation 91.3 m. The silty clay is typically grey in colour at this location. Trace shale fragments were also present within the silty clay.

Standard Penetration Tests (SPT) conducted within the upper portion of this deposit gave 'N' values of 4 and 21 blows per 0.3 m penetration indicating a firm to very stiff consistency. The measured moisture contents of samples recovered from this unit ranged from about 25% to 38%.

Grain size analysis conducted on a sample of this soil is presented in Figures B1. The results indicate that the silty clay contains approximately 2% gravel, 15% sand, 49% silt and 34% clay. Atterberg Limits test was also conducted on a sample from this stratum and the results are presented in Figure B3. The silty clay sample had a measured plasticity index of 17% and a corresponding liquid limit of 38%. These values are indicative of a cohesive soil of intermediate plasticity (group symbol of CI).

It is noted that the overburden stratigraphy in Borehole 11-01 is similar to the overburden encountered in the previous Boreholes 10-01, 10-02 and 10-05 from Reference 1.

5.2 West Floodplain

5.2.1 Alluvium (Clayey Silt and Sand)

Alluvial overburden deposits were encountered in the Credit River floodplain. In Borehole 11-02, the alluvium consists of clayey silt overlying sand deposits. The clayey silt is

approximately 3.5m thick and contains some sand, trace gravel and trace shale fragments. The underside of the clayey silt layer is at Elevation 72.2 m. Based on this borehole and other boreholes in the floodplain from Reference 1, this soil is found to have a firm to very stiff consistency as indicated by SPT 'N' values typically ranging between 5 and 19 blows per 0.3 m penetration, except at locations of shale fragments, inferred cobbles and boulders where the 'N' values are up to the order of 24 to 37 blows. The moisture contents of the clayey silt range from 10% to 18%.

A buried peat layer of 0.6 to 1.0 m thick was encountered in the clayey silt deposit in Boreholes 10-03A and 10-03B from Reference 1 located below the bridge on the floodplain.

Below the clayey silt, sand with trace to some gravel was encountered in Borehole 11-02. Wash boring techniques were used without sampling to advance the boreholes below the clayey silt. Finer soil particles including clay, silt and sand were washed out with the drill water. As such, there were insufficient recovered sand samples to carry out grain size distribution analyses and moisture content determination.

Frequent obstructions were encountered within the lower portion of the clayey silt and throughout the sand. These obstructions may be inferred as cobbles and boulders. The 1933 boreholes from Reference 3 also indicate a mixture of clay, gravel and shale fragments above the bedrock surface.

Grain size analysis conducted on a sample of the clayey silt is presented in Figure B2. The results indicate that the clayey silt contains approximately 0% gravel, 27% sand, 55% silt and 18% clay.

5.3 East Plateau and Valley Slopes

As indicated earlier, no boreholes were drilled on the east plateau or the east valley slope due to access restrictions. The 1933 boreholes indicate the presence of an overburden of a mixture of clay, gravel and shale fragments overlying bedrock at those locations. It must be noted, however, that this information is from 1933 and that the subsurface conditions may have been significantly modified over the years including placement of fill on the valley slopes.

5.4 Shale Bedrock

The overburden soils described above are underlain by shale bedrock. Bedrock was proven by coring below the wash bored depths in Boreholes 11-01 and 10-02. The following table summarizes the depths and elevations of weathered shale encountered at the borehole locations from both the present investigation and boreholes drilled at the existing bridge foundation elements during past investigations.

Foundation Element	Borehole Number	Depth to Weathered Shale (m)	Top of Weathered Shale Elevation (m)
West Abutment (proposed bridge)	11-01	3.3	91.3
	10-02	3.2	91.2
West Abutment (existing bridge)	Hole #1	1.5	91.6
	Trow #1	-	87.4
Pier No. 1 (proposed bridge)	11-02	6.3	69.4
Pier No. 1 (existing bridge)	Trow #3	-	69.0
	Hole #3	5.2	70.4
Pier No. 2 (Piers 4 and 5 of existing bridge)	Trow #6	-	68.4
	Trow #7	-	72.2
	Hole #5	-	69.6
	Hole #6	6.1	73.4
East Abutment (existing bridge)	Trow #10	-	88.0
	Trow #9	-	88.4
	Hole #8	6.7	85.6

The shale encountered at this site is fine grained, thinly bedded and grey in colour that is typical of the Georgian Bay Formation. The shale is interbedded with hard grey limestone layers and clay seams. In Borehole 11-01, the shale is in a highly weathered state within the upper 0.3m. Below this zone, the degree of weathering decreases with depth, and the rock becomes moderately to slightly weathered. The clay seams typically range from 20 to 100mm in thickness, with occasional layers up to 350 mm thick. In Borehole 11-02, the shale is highly weathered throughout the depth of investigation. The clay seams are typically 50 mm thick and an occasional sand seam up to 125 mm thick.

In Borehole 11-01, Total Core Recovery (TCR) of the bedrock was generally between 95% and 100%. The Rock Quality Designation (RQD) values were practically zero indicating a very poor rock quality. However, these low RQD values may be partially attributed to the coring equipment used in the tri-pod setup. In Borehole 11-02, Total Core Recovery (TCR) of the bedrock ranged between 30% and 100%. The Rock Quality Designation (RQD) value was 90% indicating an excellent rock quality for the first run. The RQD values decreased to the range of 10 to 35% indicating poor to very poor rock quality in the remaining runs.

Additional investigation and rock coring is recommended during the detailed design stage at each foundation element of the proposed bridge to confirm the depth of rock and the rock quality at each foundation element.

5.5 Groundwater Conditions

Groundwater conditions were observed in the open boreholes upon completion of drilling. Standpipe piezometers were installed and sealed in Boreholes 11-01 and 11-02 to permit longer term groundwater monitoring. The measured groundwater levels in the piezometers are presented in the following table.

Borehole	Date	Ground Surface Elevation (m)	Groundwater	
			Depth (m)	Elevation (m)
11-01 (sealed in bedrock)	September 30, 2010	94.6	3.2	91.4
10-02 (sealed at soil- bedrock interface)	December 17, 2010	94.4	dry	Dry
10-02 (sealed in bedrock)	December 17, 2010	94.4	9.3	85.1
11-02 (sealed at soil- bedrock interface)	June 8, 2011 October 4, 2011	75.7	0.7 1.5	75.0 74.2

It is anticipated that the groundwater level at the floodplain is largely governed by the water level in the Credit River. The water level in the Credit River was noted to be at Elevation 75.03 m on September 16, 1986. The 100-year storm high water level was reported to be at Elevation 77.72 m.

It is noted that all groundwater observations at this site are short term and the levels are expected to fluctuate seasonally and after severe weather events.

6 MISCELLANEOUS

Borehole locations and ground surface elevations were provided to Thurber by MRC.

The drilling and sampling equipment was supplied and operated by OGS Inc. of Almonte, Ontario. The field work was supervised on a full time basis by Ms. Eckie Siu of Thurber Engineering Ltd.

Laboratory testing was carried out at Thurber's Laboratory in Oakville, Ontario.

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

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

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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 to select and design a suitable foundation system for the QEW twinning bridge and to address the stability of the valley slopes.

Based on the preliminary information provided by MRC, it is understood that the proposed twinning bridge will be located immediately north of the existing bridge and will be a steel girder or concrete segmental bridge consisting of three spans. The main span is to have a length of 118 m whereas both the west and east approach spans are to be 68 m in length, for a total 254 m between abutments. The west and east abutments are to be located slightly below the crest of the west and east valley slopes, respectively. Piers 1 (west) and 2 (east) will be located within the west and east floodplains, respectively.

In the west abutment area, the valley slope inclination is estimated to be in the order of 1 to 1.4H : 1V. In the east abutment area, the valley slope is estimated to be in the order of 2.5H : 1V. The height of approach fills is anticipated to be up to the order of 4 m at the west abutment. Approximately 2 m of fill is proposed at the east abutment. The stability of the slopes in their existing conditions and in relation to the proposed abutments will be addressed.

The existing bridge abutments and piers are supported on spread footings founded on the shale bedrock. The footing base elevations of the existing bridge are documented in the Trow (1958) report (Reference 2).

The discussion and recommendations presented in this report are based on the information provided by MRC and on the factual data obtained during the course of this and previous investigations.

8 STRUCTURE FOUNDATION

In general, the stratigraphy along the proposed bridge alignment consists of topsoil and silty clay at the plateaux, and clayey silt and sand alluvium in the floodplain, overlying shale bedrock at relatively shallow depth. The groundwater level ranges from Elevation 91.4 m at the west plateau to Elevation 74.2 m in the floodplain. The groundwater level in the floodplain is largely governed by the water level in the Credit River. The elevations of top of weathered bedrock at the various foundation elements are summarized in Table 8.1 below.

Table 8.1
Bedrock Elevations and Depths at Foundation Locations

Foundation Element	Borehole Number (location)	Elevations (m)		Depth to Bedrock (m)
		Existing Ground Surface	Top of Weathered Bedrock	
West Abutment				
North	10-02* (west of crest)	94.4	91.2	3.2
Centre	11-01 (west of crest)	94.6	91.3	3.3
South	Hole #1*** (east of crest)	93.1	91.6	1.5
	Trow #1** (east of crest)	-	87.4	-
Pier No.1 (West)				
North	11-02 (floodplain)	75.7	69.4	6.3
South	Trow #3** (floodplain)	-	69.0	-
	Hole #3*** (floodplain)	75.6	70.4	5.2
Pier No.2 (East)				
Southwest	Trow#6** (river)	-	68.4	-
Southwest	Hole #5*** (river)	-	69.6	-
Southeast	Trow#7** (east bank)	-	72.2	-
Southeast	Hole #6*** (east bank)	79.6	73.4	6.2
East Abutment				
South	Hole #8*** (west of crest)	-	85.6	-
	Trow #10** (at crest)	-	88.0	-
	Trow #9** (at crest)	-	88.4	-

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- * Thurber report titled “Foundation Investigation Report, Construction Access Road for Bridge Rehabilitation, QEW Bridge Over Credit River, Mississauga, Ontario, W.P. 2186-07-00, GEOCRE No. 30M12-324, File:19-92-92, dated April 8, 2011 (Reference 1).
 - ** Trow, Soderman and Associates letter report titled “Core Drilling to Determine Underside of Existing Footings, Oakville Creek and Credit River Bridges, Queen Elizabeth Highway”, 58-F-264c, Dept. of Highways of Ontario dated August 1958 (Reference 2) – bedrock surface inferred from base of existing bridge footing.
 - *** Dept. of Highways of Ontario archived drawing, D-2241-1, 1933 (Reference 3).

During detailed design, additional boreholes will be required at the foundation elements to confirm the top of bedrock elevations.

9 FOUNDATION DESIGN

9.1 Foundation Alternatives

Consideration was given to various alternate foundation systems, taking into account the site stratigraphy, existing bridge configuration and preliminary design information.

The following lists the various foundation types that were considered:

- Spread footings on weathered shale bedrock
- Augered caissons (drilled shafts) socketted into shale bedrock
- Augered steel H-Piles socketted into shale bedrock.

A comparison of these foundation alternatives based on advantages and disadvantages of each is included in Appendix D.

The choice of foundation types depends on depths to bedrock which will need to be further investigated during detailed design, particularly at the east pier and east abutment. For relatively shallow depths to bedrock, say within the order of 6 m, at the proposed foundation locations, spread footings on bedrock is a feasible and practical option of foundation support for the new bridge. For larger depths to bedrock, deep foundations may be more cost effective. Deep foundations such as augered caissons and augered H-piles may also be considered for providing foundation support. The feasibility of driven H-piles may be assessed once more information on depths to bedrock is available during detailed design.

Due the size and type of the new bridge, an integral abutment design is not considered suitable and is not discussed further in this report.

9.2 Spread Footings on Bedrock

Based on the subsurface stratigraphy available for this site, the new structure can be supported on spread footings founded on weathered shale bedrock. The required depth of excavation for footing construction on bedrock will vary depending on the location. For

planning and preliminary design purposes, the proposed footings should be founded at the same elevations as those for the adjacent existing bridge footings in order not to undermine or exert additional load on the latter.

Table 9.1
Recommended Founding Elevations and Depths of Spread Footings

New Bridge Foundation	Top of Shale Elevation (m)	Highest Founding Elevation (m)	Approximate Founding Depth Below Existing Ground Surface (m)
West Abutment	91.2 to 91.6 (west of crest)	87.4	variable (5 to 7 m)
	87.4 (existing footing east of crest)		
Pier No. 1 (West)	69.0 to 69.4	69.0	6.7
Pier No. 2 (East)	72.2 to 73.5	70.0	Variable (6 to 7 m)
East Abutment	88.0	87.0	variable (6 to 7 m)

The concrete footings may be constructed directly on shale bedrock. In cases where the underside of a footing is higher than the bedrock subgrade due to over-excavation or presence of highly weathered shale, mass concrete fill of the same class as the footing concrete should be used to raise the subgrade to the design footing level.

If further investigation shows that bedrock is present at or below Elevations 70 m and 87 m at the east pier and east abutment locations, respectively, the excavation for footing construction on bedrock would be 8 m or deeper in which case deep foundations such as augered caissons or piles may be more practical.

In most cases, temporary excavations are anticipated to be sloped at 1H : 1V through soils and weathered bedrock (see later section for details) and formed in near vertical open cuts in sound bedrock. At the abutments, temporary groundwater control will likely involve perimeter ditches and pumping from filtered sumps. Surface drainage should be diverted away from the footing excavations at all times. In the floodplain, protection in the form of sheetpiled cofferdams will be required for footing construction. Pumping from filtered sumps will be required inside the cofferdams.

All footing excavations should be inspected prior to placing concrete to confirm that the base has been adequately cleaned. All shattered, loosened rock fragments and any exposed clay seams should be removed from the footprint of the footing and replaced with mass concrete fill as necessary. Shale is prone to rapid deterioration upon exposure to water and air. It is recommended that a working mat of 30 MPa concrete at least 100 mm thick be placed within 24 hours of completion of excavation.

9.2.1 Bearing Resistance

Footings bearing on weathered shale bedrock or on mass concrete fill founded on weathered bedrock, at or below the above recommended elevations, may be designed using a Factored Geotechnical Resistance at ULS of 800 kPa, and a Geotechnical Resistance at SLS (25 mm settlement) of 600 kPa for vertical, concentric loads. Effects of load inclination and eccentricity should be taken into account in accordance with the CHBDC, 2006 Clause 6.7.3 and Clause 6.7.4. The design of the footing may be governed by other considerations such as sliding resistance or overturning moment.

9.2.2 Horizontal Resistance of Footings

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 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.

9.2.3 Frost Cover

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

9.3 Augered Caissons (Drilled Shafts)

Augered caisson (drilled shaft) foundations socketted into shale bedrock may be employed at this site. Table 9.2 presents the recommended founding depths and elevations for caissons at each foundation element. Caisson lengths should be at least 6 m and each caisson should extend at least 4 m into bedrock.

Table 9.2
Founding Elevations for Augered Caissons

Foundation Element	Borehole	Assumed Design Bedrock Elevation (m)	Assumed Founding Elevation (m)
West Abutment	10-02, 11-01 Hole #1, Trow #1	87	83
Pier No.1	11-02, Trow #3, Hole #3	69	65
Pier No.2	Trow #6, Trow #7, Hole #5, Hole #6	70	65
East Abutment	Trow #9, Trow #10 Hole #8	87	83

9.3.1 Axial Resistance

The following Table 9.3 presents factored geotechnical resistances calculated for typical 1.2 and 1.5 m diameter caissons associated with the following minimum socket depths within bedrock.

Table 9.3
Vertical Geotechnical Resistance for Caisson Foundations

Caisson Diameter (m)	Minimum Socket Depth below Bedrock Surface (m)	Factored ULS (kN)	SLS (kN)
1.2	4	5,000	4,000
	5	6,000	4,800
1.5	4	6,000	4,800
	5	8,500	6,800

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 socketted within bedrock.

9.3.2 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}) as follows:

$$k_s = 125 \cdot C_u / D \quad (\text{kN/m}^3)$$

$$p_{ult} = 9 \cdot C_u \text{ (kPa) at and below a depth of } 3 \cdot D \text{ (m) reduced to zero at ground surface}$$

where

$$D = \text{caisson diameter (m)}$$

$$C_u = \text{undrained shear strength (kPa)}$$

The above equations and recommended parameters may be used to analyze the interaction between a caisson and the surrounding soil. The lateral pressures obtained from the analysis should not exceed the ultimate lateral resistance.

Parameter values for lateral caisson resistance are shown in Table 9.4 below.

Table 9.4 – Parameters for Lateral Caisson Resistance

Foundation Element	Elevation (m)	C_u (kPa)	Unit Weight (kN/m^3)	Soil Conditions
West Abutment	G.S. to 87	100	20	Stiff to very stiff silty clay
	Below 87	300	23	Weathered Shale Bedrock
East Abutment	G.S. to 87	50	20	Fill / Overburden
	Below 87	300	23	Weathered Shale Bedrock

Alternatively, for rock sockets formed within the predominantly weathered shale, the ultimate passive force that can be mobilized by the embedded portion of a socket is given by:

$$P_p = 6 \cdot C \cdot D \cdot L$$

where $C = 300 \text{ kPa}$ (equivalent Mohr-Coulomb cohesion based on Hoek and Brown rock mass classification)

$D =$ caisson diameter (m)

$L =$ depth of socket in rock (m)

The structural designer should check whether a 4 m deep socket is sufficient to provide base fixity.

For lateral soil/caisson group interaction analysis, the modulus of subgrade reaction (k_s) may have to be reduced based on caisson spacing.

9.3.3 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. Hard interbedded layers of limestone in the shale bedrock may require the use of coring or rock breaking equipment in addition to the auger equipment.

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 and approved.

9.4 Augered H-Piles

Alternatively, consideration may be given to supporting the foundations on steel H-piles set within sockets that are drilled into bedrock. The sockets should be pre-drilled and the socket base should be cleaned of loose and shattered rock. The pile should then be lowered into the

socket and the remaining space grouted with 30 MPa concrete. Based on the site conditions, it is recommended that a minimum socket depth of 4 m below the top of bedrock be used.

Table 9.5 presents the recommended founding depths and elevations for the augered H-piles at each abutment and pier location. Pile lengths should be at least 6 m and each caisson should extend at least 4 m into bedrock.

Table 9.5
Founding Depths and Elevations for Augered H-Piles

Foundation Element	Borehole	Assumed Design Bedrock Elevation (m)	Assumed Founding Elevation (m)
West Abutment	10-02, 11-01 Hole #1, Trow #1	87	83
Pier No.1	11-02, Trow #3, Hole #3	69	65
Pier No.2	Trow #6, Trow #7, Hole #5, Hole #6	70	65
East Abutment	Trow #9, Trow #10 Hole #8	87	83

9.4.1 Axial Resistance

For a HP 310 x 110 pile grouted within a 600 mm nominal diameter socket extended at least 4 m into shale bedrock, a Factored Geotechnical Resistance at ULS of 2,000 kN per pile, and a Geotechnical Resistance at SLS of 1,800 kN per pile, may be used for design.

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

9.4.2 Lateral Resistance

For preliminary design, lateral soil resistance against an HP 310 x 110 pile can be assessed based on the method outlined in the CHBDC 2006.

Lateral resistance design for augered H-piles may be carried out as per the recommendations in sub-section 9.3.2 by substituting the caisson diameter with augered pile diameter.

9.4.3 Augered Pile Installation

Augered pile installation should be in general accordance with clauses for caissons in OPSS 903 . The pre-drilled holes for forming the pile socket should have a nominal diameter of 600 mm.

General requirements for installation equipment are similar to those for caissons in sub-section 9.3.3.

Subsequent to the seating of a pile in the socket, the remaining space in the pre-drilled hole should be grouted with 30 MPa concrete.

10 SLOPE STABILITY

The existing river valley slopes are moderately vegetated with grass, brushes, shrubs and occasional small trees. Based on preliminary observations, there is no visual evidence of slope instability at the proposed abutment locations. Existing boreholes at the west slope crest indicate that the subsurface conditions consist of 3 to 4 m of silty clay overlying shale bedrock. Available contours indicate that the river slopes at the west and east abutments have overall inclinations of approximately 1 to 1.4H : 1V and 2.5H : 1V, respectively. Both valley slopes are formed in shale bedrock.

New fill will need to be placed at the proposed abutment locations. At the west abutment, preliminary information indicates that the new abutment bearings will be approximately in line with the existing west abutment with up to 3 to 4 m of new fill. Excavation extending up to about 7 m below the existing slope crest is also anticipated. The proposed new east abutment will be located east of the existing abutment and possibly require up to 2 m of new approach fill. It is anticipated that the river slopes will remain stable.

It is noted that the proposed footing at each abutment location is located beyond the zone defined by a 2H : 1V line extending upwards from the toe of the valley slope.

Slope stability analysis will need to be carried out during detailed design to confirm global stability and stability at locations of local steepening at the abutments once more design details on new fills and/or cuts are available.

11 EXCAVATION AND BACKFILL

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

Earth excavation will extend through 3 to 4 m of very stiff silty clay and weathered shale bedrock at the west abutment and in the order of 6.5 m of clayey silt and sand in the west floodplain. At the abutments, excavation through shale could be up to 4 m depth on the slopes. Open cut slopes at a maximum inclination of 1H : 1V may be carried out at the abutments. Alternatively, the abutment footings may be constructed within protective systems (temporary shoring) designed in accordance with OPSS 539.

In order to limit the work area within the floodplain due to environmental reasons and to assist in groundwater control within the excavation, it is recommended that excavation in the floodplain be carried out in protection systems in the form of sheetpiled cofferdams.

The upper 2 m of weathered shale should be considered as soil for the purpose of slope inclination requirements. Vertical sides of temporary excavations formed in the Georgian Bay Shale bedrock below the upper weathered zone should be stable without protection in the short term, i.e. no more

than three weeks. However, the shale will deteriorate upon exposure to air and water, and start to slough into the excavation. If prolonged exposure is inevitable, protection systems should also be used for the shale below the upper weathered zone.

Excavation into the relatively sound shale will encounter hard limestone interbeds. It is anticipated that the shale becomes progressively harder with depth. Heavy excavating equipment, ripping machinery and rock breakers/splitters will be required to break up hard limestone and other intact shale slabs. Any rock excavation should be carried out in accordance with OPSS 902. Blasting is not required nor will it be acceptable at this site.

Protection Systems (temporary shoring) should be designed to satisfy the requirements of OPSS 539.

Granular backfill to the abutment walls should be placed to the extents shown in OPSD 3101.150. All granular materials should meet the extents shown in OPSS 1010.

12 GROUNDWATER CONTROL

The amount of perched water within the soils in the plateau will vary between locations. For excavations through the soils at the abutment locations, groundwater control will likely be limited to diverting surface runoff and sump pumping.

Where sheetpiled cofferdams are used at the pier locations, the sheetpiles will act as a form of groundwater control. Frequent cobbles and boulders may be encountered within the alluvial deposits. The depth of sheetpile penetration within the bedrock will be limited. Therefore, pumping inside the cofferdam will still be required to maintain a reasonably dry base to facilitate foundation construction.

Filtered sumps must be designed properly so that construction drainage water containing eroded soil, weathered rock and fines do not flow into the Credit River.

The design of unwatering systems for the excavations is the responsibility of the Contractor who is expected to retain dewatering specialists for this task.

13 APPROACH EMBANKMENTS

13.1.1 Stability

New granular fill will have to be placed behind new abutment walls as discussed previously. The new fill will be up to 4 m and 2 m in heights at the west and east abutments, respectively, and is not expected to cause instability as discussed in Section 10. At the forward slopes where local steepening is possible, stability analysis will have to be carried out during detailed design to confirm stability.

13.1.2 Settlement

Given the very stiff consistency of the silty clay and the shallow depth of the underlying bedrock, it is anticipated that the loading associated with filling at the new abutment

locations will result in minimal post construction foundation settlements. Elastic settlement within the soil is expected to be completed by the end of fill placement.

14 STATIC EARTH PRESSURE

For cases where backfill to the abutment is placed in accordance with OPSD 3101.150, as recommended, the static lateral earth pressure will be governed by the properties of the material within the backfill limits shown, i.e. a line projected up at 1.5H : 1V for granular backfill.

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

For fully drained backfill, earth pressures acting on the structure should be computed in accordance with Clause 6.9 of the CHBDC, but are generally given by the following expression:

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

where P_h = horizontal pressure on the wall (kPa)
 K = earth pressure coefficient (see below)
 γ = unit weight of retained soil (see below)
 h = depth below top of fill where pressure is computed (m)
 q = value of any surcharge (kPa)

In accordance with Clause 6.9.3 of the CHBDC, a compaction surcharge should be added. The magnitude should be 12 kPa at the top of fill and decreasing to 0 kPa at a depth of 2.0 m for Granular B Type I or at a depth of 1.7 m for Granular A or Granular B Type II.

Earth pressure coefficients for backfill to the abutment walls are dependent on the material used as backfill. Typical unfactored values are shown in Table 13.1.

Table 13.1 – Earth Pressure Coefficients

Wall Condition	Earth Pressure Coefficient (K)			
	OPSS Granular A or OPSS 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 Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (Unrestrained Wall)	0.27	0.40	0.31	0.48
At rest (Restrained Wall)	0.43	-	0.47	-
Passive (Movement Towards Soil)	3.70	-	3.30	-

The factors in the table above 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 C6.9.1 (a) in the Commentary to the CHBDC, 2006.

15 EROSION AND SCOUR PROTECTION

Erosion protection should be provided to the east valley slopes and the river channel at the bridge site. Scour protection should be provided for the pier foundations.

16 CONSTRUCTION CONCERNS

During construction, the Contract Administrator should employ experienced foundation/geotechnical staff to observe foundation construction activities.

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

Spread Footing Construction

- In order to avoid undermining the adjacent existing footings, it is important that all new footing excavations be carried out with care and be adequately supported by protection systems, where required, which will also serve to limit adjacent ground movements. Settlement monitoring of the existing footings will need to be implemented during construction. This issue should be addressed during detailed design.
- All new footings are to be founded on shale bedrock which is prone to rapid deterioration upon exposure to water and air. Once the founding shale is inspected and approved, a lean mix concrete mud slab of at least 100 mm thick should be placed on the subgrade for protection purposes.

Unwatering

- At the pier locations within the floodplain, excavation for footing and/or caisson/pile cap construction will require support systems such as sheetpiled cofferdams. Such systems will provide partial cutoff that is important in minimizing water inflow into the excavations. Unwatering in the form of pumping from filtered sumps inside the cofferdams will be required to maintain a reasonably dry base.

17 INVESTIGATION FOR DETAILED DESIGN

During the detailed design phase of this project, additional site investigations and field testing will be required. The following minimum scope of work is recommended.

Field Investigation

- At least two boreholes must be drilled and sampled at each of the finalized foundation locations. Each of these boreholes should be advanced to reach shale bedrock below which the bedrock should be cored for at least 3 m.

- At each approach, one additional borehole must be drilled within 20 m of the abutment to investigate the nature and extent of the existing approach fills and the underlying native soils and rock. These boreholes must fully penetrate the existing fill into the underlying native materials.

Depending on the final bridge configuration, it is assumed that one or both of the boreholes drilled in this preliminary investigation and selected, previously existing boreholes can be incorporated for detailed design. During the current investigation, both boreholes were advanced using portable tripod equipment due to difficult access conditions for conventional drilling equipment to the floodplain and overhead cables at the slope crest. For the detailed investigation, the access road near the west abutment would be available, the overhead power cables temporarily relocated and the property issues near the east abutment resolved to facilitate access of conventional drill rigs to the desired locations. It is possible that investigation for the east pier would require the use of a raft in the river.

Reporting

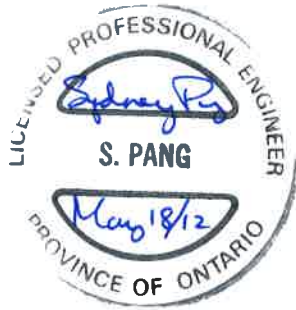
A Foundation Investigation Report and a Foundation Investigation and Design Report must be prepared in accordance with the Ministry's standards. The foundation aspects to be addressed in the reports will include, but not be limited to, all relevant factual subsurface information of the site, foundation alternatives and design recommendations, specific spread footing and/or augered caisson/pile installation requirements, lateral pressure design, excavation, shoring and unwatering, river slope and approach embankment stability assessment, seismic considerations, scour and erosion protection.

18 CLOSURE

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

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

THURBER ENGINEERING LTD.



Engineering Analysis and Report Preparation by:
Sydney Pang, P.Eng.,
Associate, Senior Geotechnical Engineer



Report Reviewed by:
P. K. Chatterji, P.Eng.,
Review Principal, Designated MTO Contact

Appendix A

Record of Borehole Sheets

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

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

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

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

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

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

NOTE: Hierarchy of Soil Strength Prediction

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



4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

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

5. LEGEND FOR RECORDS OF BOREHOLES

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

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


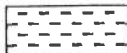



 Water Level
 Shear Strength Determination by Pocket Penetrometer

- (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.

UNIFIED SOILS CLASSIFICATION

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

EXPLANATION OF ROCK LOGGING TERMS

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




DISCONTINUITY SPACING		STRENGTH CLASSIFICATION			Field Estimation of Hardness*
Bedding	Bedding Plane Spacing	Rock Strength	Approximate Uniaxial Compressive Strength		
			(MPa)	(psi)	
Very thickly bedded	Greater than 2m	Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Thickly bedded	0.6 to 2m				
Medium bedded	0.2 to 0.6m	Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Thinly bedded	60mm to 0.2m				
Very thinly bedded	20 to 60mm	Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Laminated	6 to 20mm				
Thinly Laminated	Less than 6mm	Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.
TERMS					
Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.	Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.	Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
Rock Quality Designation: (RQD)	Total length of sound core recovered in pieces 0.1m in length or larger as a percentage of total core run length.	Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen				
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.				

RECORD OF BOREHOLE No 11-01

1 OF 1

METRIC

W.P. W.O. 08-20008 LOCATION N 4 823 959.1 E 295 814.8 QEW Bridge at Credit River ORIGINATED BY SLD
 HWY QEW BOREHOLE TYPE Tripod (Hilt) - Wash Boring and Coring COMPILED BY AN
 DATUM Geodetic DATE 2011.05.30 - 2011.06.02 CHECKED BY SKP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				WATER CONTENT (%)				
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE				w _p w w _L				
94.6							20	40	60	80	100	20	40	60		GR SA SI CL
0.0	TOPSOIL, with roots and organics: (200mm)		1	SS	4											2 15 49 34
0.2	Silty CLAY, trace sand Very Stiff Grey Moist															
			2	SS	21											
	trace shale fragments		3	SS	40/											
			1	RUN	0.075											
			2	RUN												
			3	RUN												
			4	RUN												
			5	RUN												
			4	SS	50/		0.075									
91.3																
3.3	SHALE, weathered, grey															
91.0																
3.6	END OF SPT SAMPLING TO 3.6m AND START CORING. FOR ROCK DETAILS PLEASE REFER TO 11-01R. Piezometer installation consists of 38mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Sep.30/11 3.2 91.4															

RECORD OF BOREHOLE 11-01R

PROJECT : QEW Mississauga Rd. Overpass
 LOCATION : Mississauga, ON
 STARTED : May 30, 2011
 COMPLETED : June 2, 2011

Project No. W.O. 08-20008

INCLINATION: Vertical AZIMUTH:

SHEET 1 OF 1
 DATUM Geodetic

DEPTH SCALE (metres)	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	FIELD/LABORATORY TESTING RESULTS																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																															
				ELEV.		RUN No	PENETRATION RATE (mm/min)	COLOUR FLUSH % RETURN	FR-FRACTURE CL-CLEAVAGE SH-SHEAR VN-VEIN		F-FAULT J-JOINT P-POLISHED S-SLICKENSIDED		SM-SMOOTH R-ROUGH ST-STEPPED PL-PLANAR		FL-FLEXURED UE-UNEVEN W-WAVY C-CURVED		25 Unconfined 50 Compressive Strength 75 (MPa)	● Point Load Test Diametral ▲ Point Load Test Axial ■ Laboratory UCS Test																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
				DEPTH (m)	RECOVERY TOTAL CORE %				SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 3 m	DIP wrt Core Axis	TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY k, cm/sec																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																					
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Bentonite
Seal

Sand

Slotted
Screen

89.24

88.94

87.44

GROUNDWATER ELEVATIONS

▽ SHALLOW/SINGLE INSTALLATION
 WATER LEVEL (date)

▼ DEEP/DUAL INSTALLATION
 WATER LEVEL (date)

LOGGED : SLD
 CHECKED : SKP

RECORD OF BOREHOLE No 11-02

1 OF 1

METRIC

W.P. W.O. 08-20008 LOCATION N 4 824 026 4 E 295 840 4 QEW Bridge at Credit River ORIGINATED BY SLD/ES
 HWY QEW BOREHOLE TYPE Tripod (Hilti) - Wash Boring and Coring COMPILED BY AN
 DATUM Geodetic DATE 2011.06.07 - 2011.06.09 CHECKED BY SKP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)							
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa												
75.7								20	40	60	80	100								
0.0	Clayey SILT, some sand, trace gravel, trace shale fragments Stiff to Very Stiff Brown Moist		1	SS	9*															
			2	SS	10*		75													
			3	SS	12*		74													
			4	SS	5*															
73.3																				
2.4	Frequent obstructions, inferred as cobbles and boulders		5	SS	37*		73													
72.2																				
3.5	SAND, trace to some gravel						72													
71.9	Brown																			
3.8	Frequent obstructions, inferred as cobbles and boulders																			
71.1							71													
4.6																				
70.5																				
70.4	Clayey SILT, some sand, trace gravel																			
5.3	Frequent obstructions, inferred as cobbles and boulders						70													
	Shale fragements																			
69.4																				
6.3	END OF SAMPLING AT 6.3m AND START CORING. FOR ROCK DETAILS PLEASE REFER TO 11-02R. Piezometer installation consists of 31mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Jun.08/11 0.7 75.0 Oct.04/11 1.5 74.2																			

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

RECORD OF BOREHOLE 11-02R

PROJECT : QEW Mississauga Rd. Overpass
 LOCATION : Mississauga, ON
 STARTED : June 7, 2011
 COMPLETED : June 9, 2011

INCLINATION: Vertical AZIMUTH:

Project No. W.O. 08-20008

SHEET 1 OF 1
 DATUM Geodetic

DEPTH SCALE (metres)	DRILLING RECORD			DESCRIPTION	SYMBOLIC LOG	ELEV		RUN No.	PENETRATION RATE (m/min)	COLOUR % RETURN	FLUSH	FR-FRACTURE				F-FAULT				SM-SMOOTH				FL-FLEXURED				Unconfined Compressive Strength (MPa)	FIELD/LABORATORY TESTING RESULTS																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																		
						DEPTH (m)	FLUSH					RECOVERY		R.Q.D. %	FRACT. INDEX PER. 3 m	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY k, cm/sec																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													
	TOTAL CORE %	SOLID CORE %	DIP wrt Core Axis									TYPE AND SURFACE DESCRIPTION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																			
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GROUNDWATER ELEVATIONS

▽ SHALLOW/SINGLE INSTALLATION
 WATER LEVEL (date)

▼ DEEP/DUAL INSTALLATION
 WATER LEVEL (date)

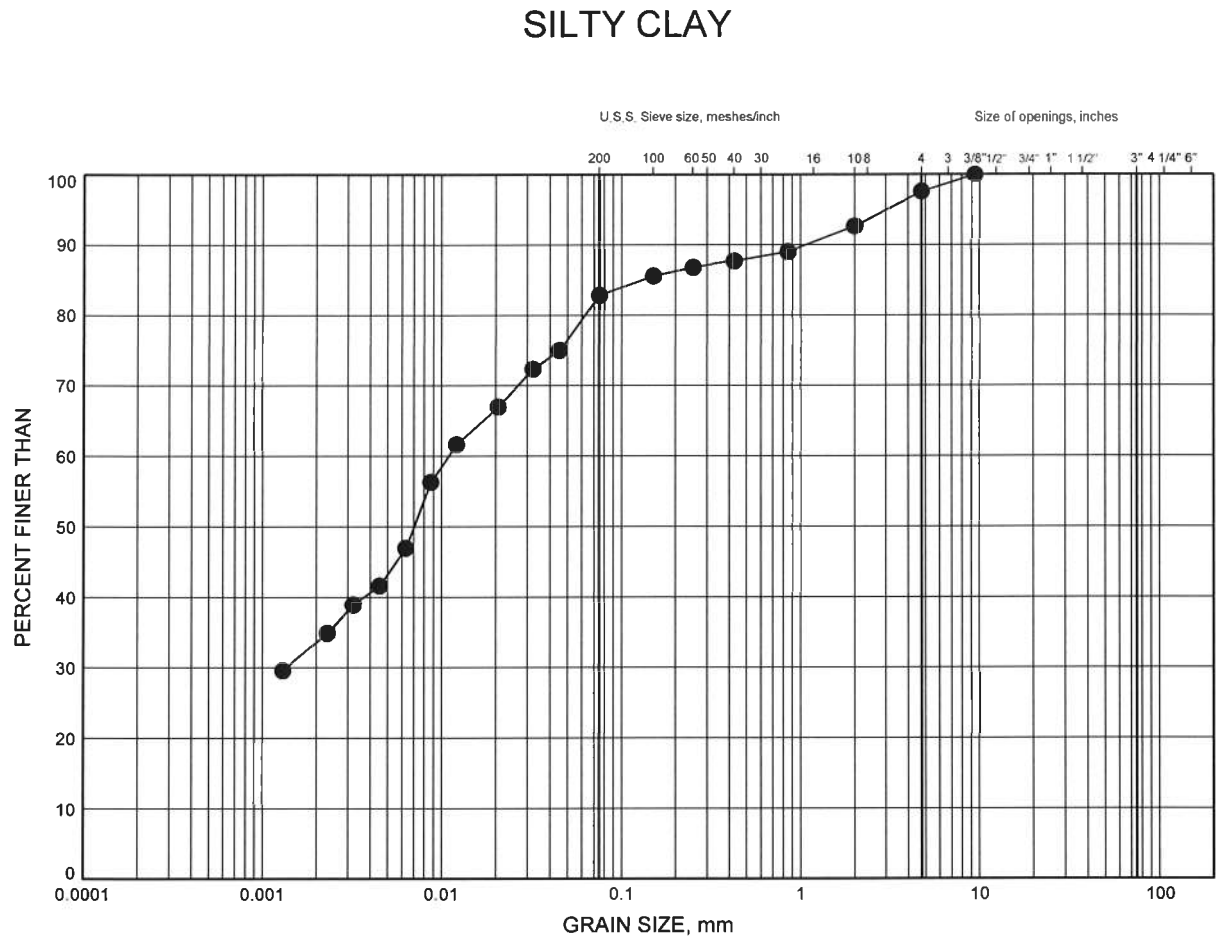
LOGGED : SLD
 CHECKED : SKP

Appendix B

Laboratory Test Results

QEW Bridge at Credit River
GRAIN SIZE DISTRIBUTION

FIGURE B1



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	11-01	1.07	93.50

GRAIN SIZE DISTRIBUTION - THURBER 1174 GPJ 5/17/12

Date May 2012
W.P.# W.O. 08-20008

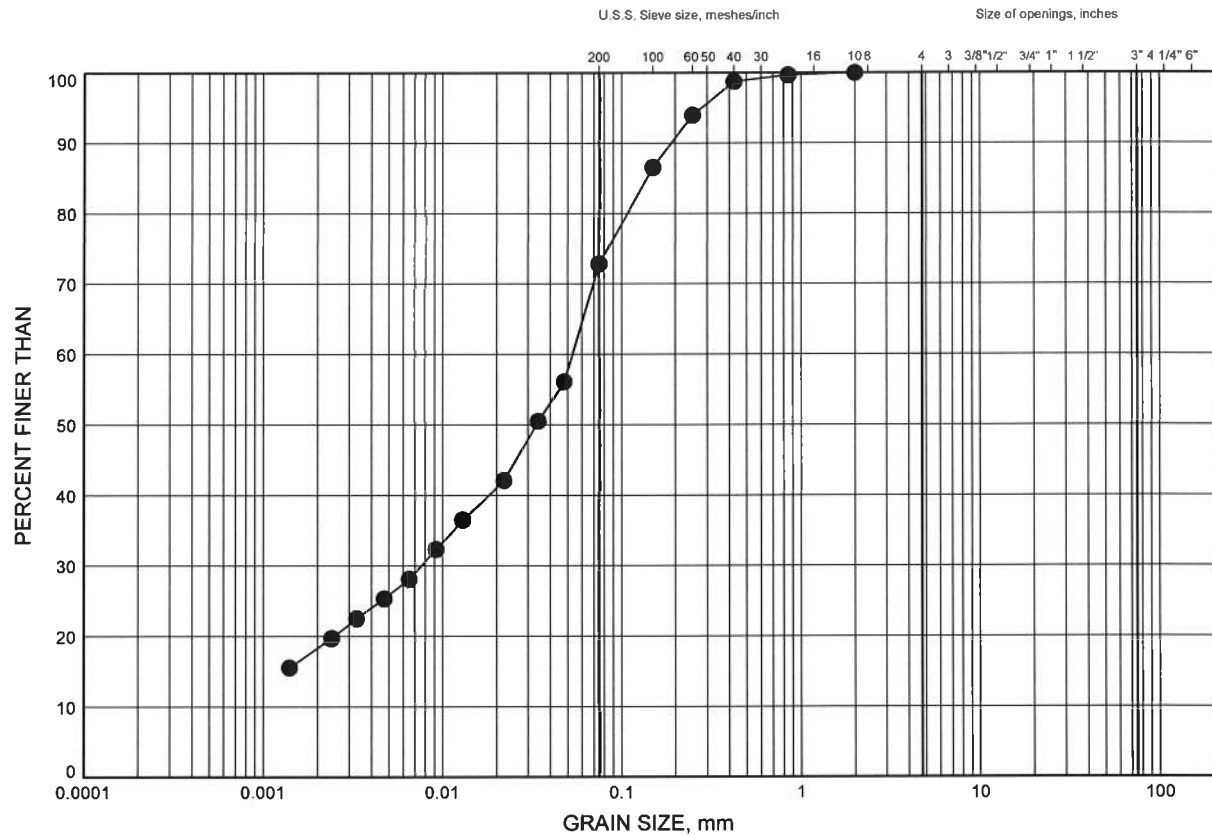


Prep'd MFA
Chkd. SKP

QEW Bridge at Credit River
GRAIN SIZE DISTRIBUTION

FIGURE B2

CLAYEY SILT



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	11-02	0.91	74.83

GRAIN SIZE DISTRIBUTION - THURBER 1174.GPJ 5/17/12

Date May 2012
W.P.# W.O. 08-20008

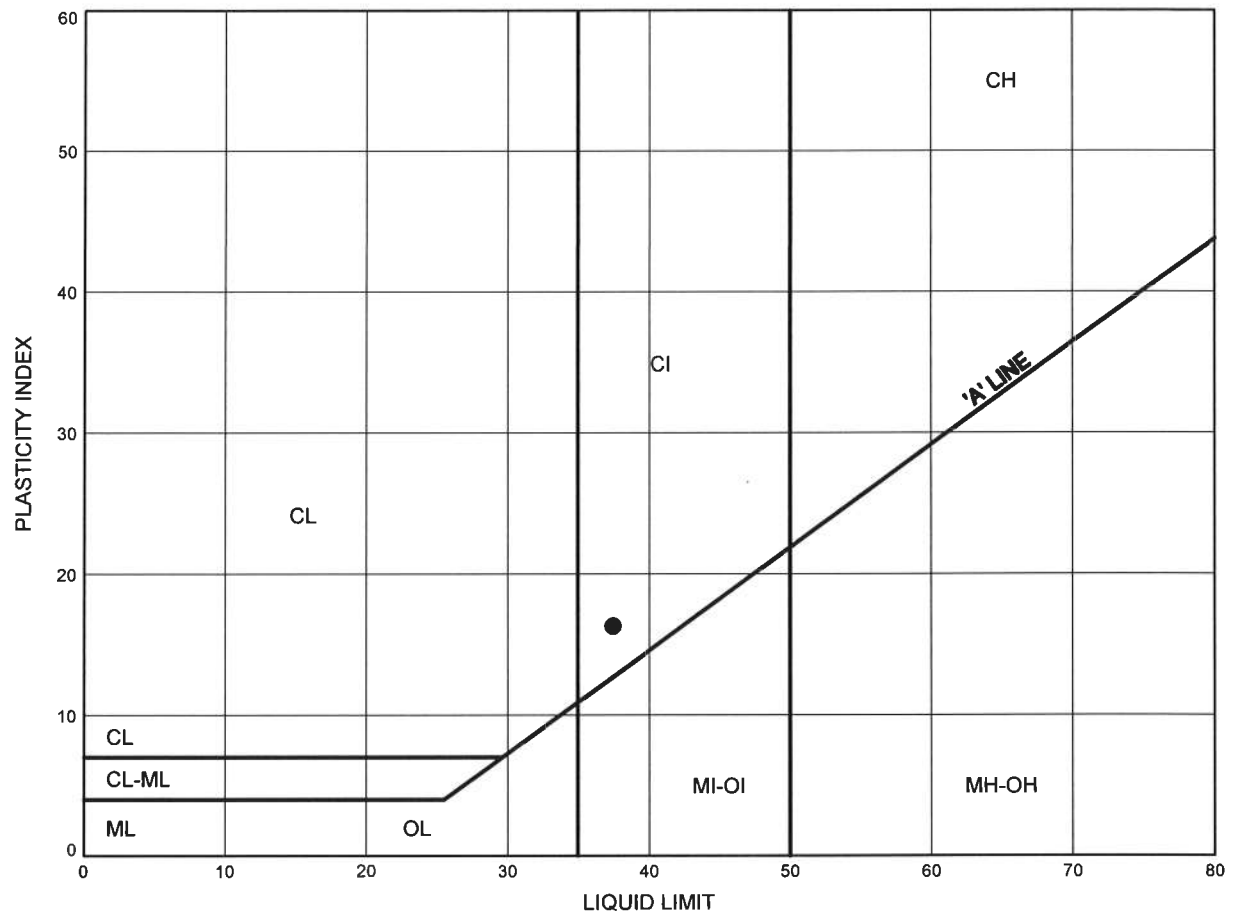


Prep'd MFA
Chkd. SKP

QEW Bridge at Credit River
ATTERBERG LIMITS TEST RESULTS

FIGURE B3

SILTY CLAY



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	11-01	1.07	93.50

THURBALT 1174.GPJ 5/17/12

Date May 2012
W.P.# W.O. 08-20008



Prep'd MFA
Chkd. SKP

TABLE 2 - Point Load and Unconfined Compressive Strength Test Results
Credit River Access Road

Run	Depth (m)	UCS (MPa)	Rock Type	Test			
Borehole		10-01		Total Rock Core			
2	3.4	0.7	Shale	Rock Type	Shale	Limestone	
2	3.6	0.6	Shale	Average (MPa)	3.3	49.6	
2	3.9	2.7	Shale	Minimum (MPa)	0.6		
2	4.4	3.4	Shale	Maximum (MPa)	10.7		
3	4.7	10.7	Shale				
3	5.1	3.0	Shale	Run #	Average (MPa)		
3	5.4	0.7	Shale	2	1.9		
3	5.7	6.8	Shale	3	4.7		
3	5.8	3.4	Shale	4	2.0		
3	6.2	3.4	Shale	5	2.8		
4	6.5	2.0	Shale	6	4.7		
4	6.9	0.7	Shale	7	14.6		
4	7.3	0.7	Shale				
4	7.6	4.8	Shale				
5	7.9	0.7	Shale				
5	8.3	2.7	Shale				
5	8.7	5.4	Shale				
5	8.9	2.9	Shale				
5	9.2	2.0	Shale				
6	9.6	9.4	Shale				
6	10.0	1.3	Shale				
6	10.2	6.8	Shale				
6	10.7	1.4	Shale				
7	11.0	4.8	Shale				
7	11.6	0.7	Shale				
7	11.7	3.4	Shale				
7	12.2	49.6	Limestone				
Borehole		10-02		Total Rock Core			
1	4.4	0.5	Shale	Rock Type	Shale	Limestone	Shale/Limestone
1	4.4	0.5	Shale	Average (MPa)	4.2	109.5	26.6
2	5.0	0.7	Shale	Minimum (MPa)	0.5	95.2	5.1
2	5.0	0.5	Shale	Maximum (MPa)	13.8	120.4	48.1
2	5.7	0.5	Shale				
2	6.0	0.5	Shale	Run #	Average (Mpa)		
2	6.0	8.9	Shale	2	2.2		
3	6.4	1.7	Shale	3	3.6		
3	6.4	3.1	Shale	4	2.8		
3	6.9	4.1	Shale	5	1.9		
3	7.4	0.7	Shale	6	5.0		
3	7.4	8.6	Shale	7	19.7		

TABLE 2 - Point Load and Unconfined Compressive Strength Test Results
Credit River Access Road

Run	Depth (m)	UCS (MPa)	Rock Type	Test		
4	7.8	0.7	Shale		8	6.1
4	7.8	4.5	Shale		9	25.2
4	8.7	1.7	Shale		10	2.0
4	8.7	4.4	Shale		11	6.6
5	9.5	0.7	Shale		12	20.0
5	9.5	5.4	Shale		13	3.3
5	10.1	0.5	Shale		14	7.2
5	10.1	3.2	Shale		15	9.2
5	10.5	0.5	Shale			
5	10.5	1.4	Shale			
6	11.0	0.5	Shale			
6	11.0	13.8	Shale			
6	11.8	0.7	Shale			
6	11.8	7.6	Shale			
6	12.4	2.6	Shale			
7	12.5	1.7	Shale			
7	12.5	6.5	Shale			
7	12.9	112.9	Limestone			
7	13.2	1.7	Shale			
7	13.2	4.1	Shale			
7	13.6	10.7	Shale	UC		
7	13.7	0.7	Shale			
8	14.0	0.7	Shale			
8	14.0	8.5	Shale			
8	14.8	1.7	Shale			
8	14.8	9.5	Shale			
8	15.1	3.4	Shale			
8	15.1	13.0	Shale			
9	15.5	0.7	Shale			
9	15.5	2.6	Shale			
9	16.0	0.7	Shale			
9	16.0	1.5	Shale			
9	16.4	120.4	Limestone			
10	17.0	0.7	Shale			
10	17.0	3.9	Shale			
10	17.6	0.7	Shale			
10	17.6	2.5	Shale			
10	18.0	0.7	Shale			
10	18.0	3.8	Shale			
11	18.6	10.1	Shale			
11	18.6	12.9	Shale			

TABLE 2 - Point Load and Unconfined Compressive Strength Test Results
Credit River Access Road

Run	Depth (m)	UCS (MPa)	Rock Type	Test				
11	19.1	0.7	Shale	UC				
11	19.1	7.3	Shale					
11	19.3	11.9	Shale					
11	19.6	0.7	Shale					
11	19.6	2.7	Shale					
12	19.9	3.4	Shale					
12	19.9	11.5	Shale					
12	20.3	1.7	Shale					
12	20.3	7.2	Shale					
12	21.0	95.2	Limestone					
12	21.1	1.7	Shale					
12	21.1	6.3	Shale					
12	21.3	5.1	Shale/Lime					
12	21.3	48.1	Shale/Lime					
13	21.9	0.7	Shale					
13	21.9	5.9	Shale					
14	22.4	5.0	Shale					
14	22.4	9.4	Shale					
15	23.0	5.1	Shale					
15	23.0	8.7	Shale					
15	23.9	6.7	Shale					
15	23.9	12.4	Shale					
15	24.3	13.4	Shale					
Borehole		10-05		Total Rock Core				
1	3.2	1.4	Shale	Rock Type		Shale	Limestone	Shale/Limestone
2	3.5	0.9	Shale	Average (MPa)		3.0	73.1	3.7
2	4.0	0.8	Shale	Minimum (MPa)		0.7	27.9	2.0
2	4.2	1.1	Shale	Maximum (MPa)		10.8	200.5	5.4
2	4.8	4.1	Shale					
3	5.0	0.7	Shale	Run #		Average (Mpa)		
3	5.3	8.4	Shale	1		1.4		
3	5.6	1.5	Shale	2		1.7		
3	6.0	1.3	Shale	3		2.6		
3	6.3	1.3	Shale	4		4.6		
4	6.4	1.3	Shale	5		9.6		
4	6.7	8.5	Shale	6		3.8		
4	7.1	2.7	Shale	7		7.7		
4	7.4	10.8	Shale	8		4.7		
4	7.5	2.0	Shale	9		61.2		
4	7.8	2.0	Shale/Lime					
5	8.1	1.4	Shale					

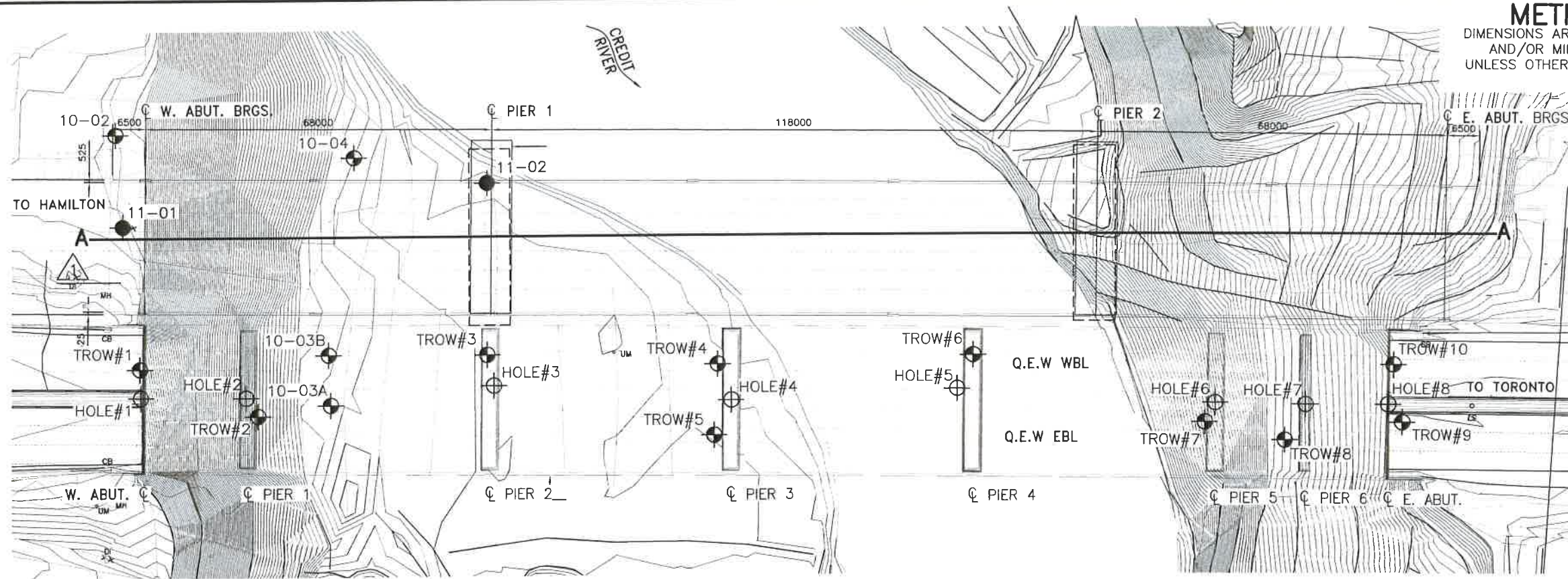
TABLE 2 - Point Load and Unconfined Compressive Strength Test Results
Credit River Access Road

Run	Depth (m)	UCS (MPa)	Rock Type	Test
5	8.5	1.4	Shale	
5	8.9	0.7	Shale	
5	9.0	35.0	Limestone	
6	9.4	3.4	Shale	
6	9.8	0.7	Shale	
6	10.0	2.9	Shale	
6	10.3	5.4	Shale	
6	10.7	6.8	Shale	
7	11.0	2.0	Shale	
7	11.3	27.9	Limestone	
7	11.6	1.3	Shale	
7	11.9	0.7	Shale	
7	12.2	6.8	Shale	
8	12.5	4.7	Shale	
8	13.0	2.7	Shale	
8	13.5	5.9	Shale	
8	13.7	5.4	Shale/Lime	
9	14.1	37.7	Limestone	
9	14.4	200.5	Limestone	
9	14.6	64.3	Limestone	
9	15.0	1.3	Shale	
9	15.2	2.0	Shale	

Appendix C

Borehole Locations and Soil Strata Drawing

19-1351-174



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

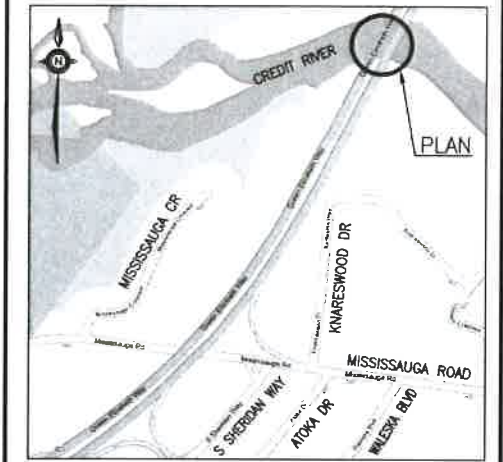
CONT No
WP No 08-20008

BRIDGE TWINNING OVER
CREDIT RIVER
QUEEN ELIZABETH WAY
BOREHOLE LOCATIONS AND SOIL STRATA

MRC MCCORMICK RANKIN
A MEMBER OF THE MCM GROUP



THURBER ENGINEERING LTD.



KEYPLAN

LEGEND

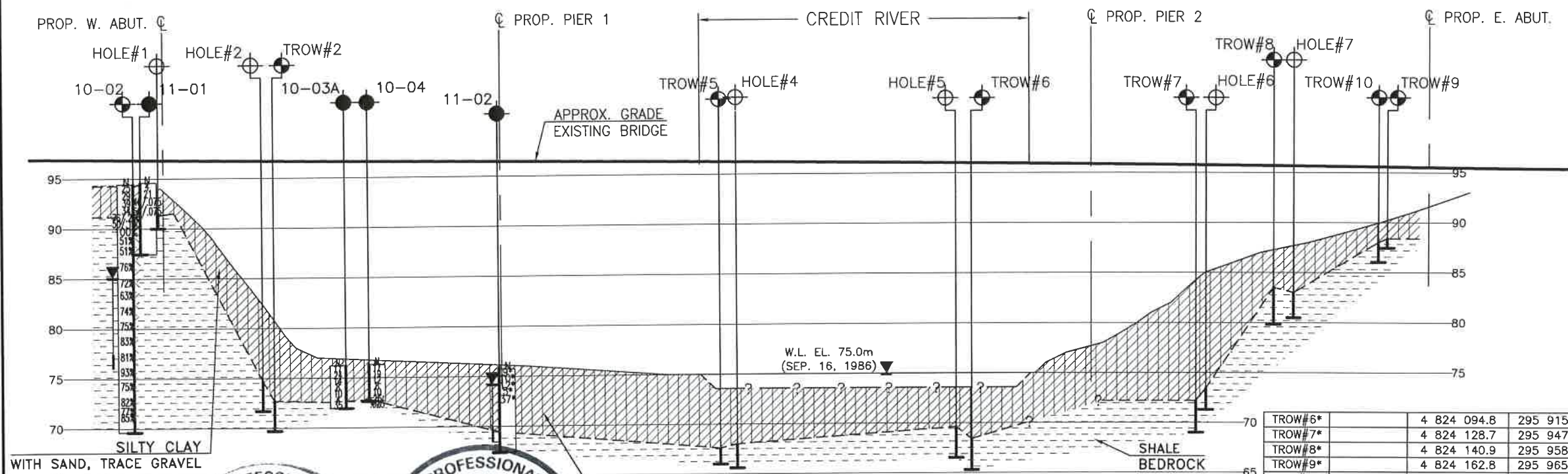
- Borehole (Current Investigation)
- ⊙ Borehole (Trow 1958))
- ⊙ Borehole (Dept. of Highway 1933)
- ⊙ Approx. Borehole Location (Survey Data Not Available)
- Blows /0.3m (Std Pen Test, 475J/blow)
- N North Arrow
- ▽ Water Level
- ⊕ Head Artesian Water
- ⊕ Piezometer
- 90% Rock Quality Designation (RQD)

NO	ELEVATION	NORTHING	EASTING
10-02	94.4	4 823 966.7	295 797.9
10-03A	76.2	4 823 979.0	295 865.1
10-03B	76.3	4 823 983.2	295 856.2
10-04	76.3	4 824 005.7	295 823.9
11-01	94.6	4 823 959.1	295 814.8
11-02	75.7	4 824 026.4	295 840.5
TROW#1*		4 823 949.1	295 841.3
TROW#2*		4 823 965.4	295 860.3
TROW#3*		4 824 010.7	295 870.6
TROW#4*		4 824 049.8	295 893.3
TROW#5*		4 824 042.7	295 905.4

NOTES

- The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.
- This drawing is for subsurface information only. Surface details and features are for conceptual illustration.
- Boreholes from Trow & Dept. of Highways are located on the alignment of the existing bridge, and are offset from the alignment of the proposed bridge.

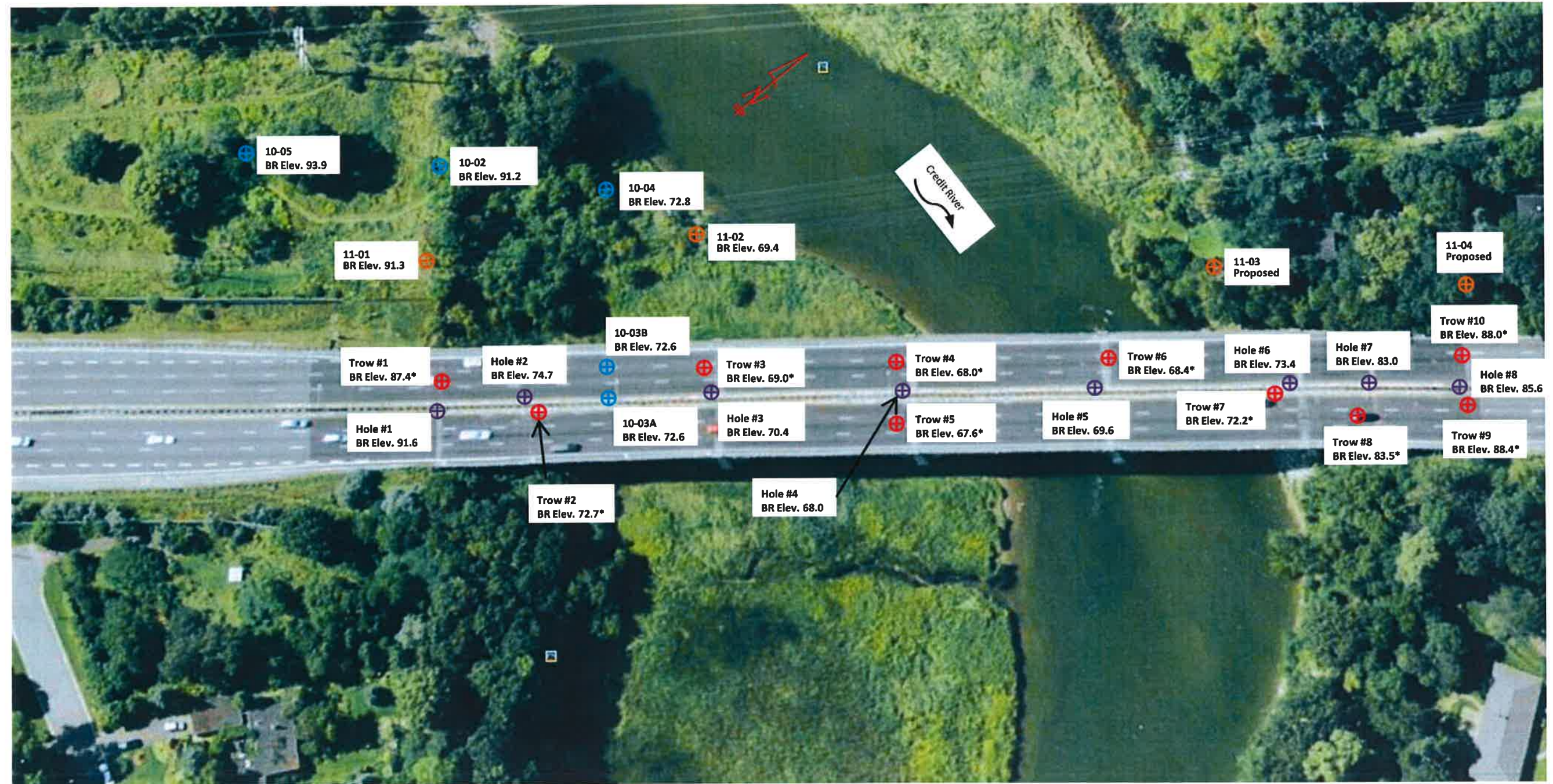
GEOCRES No. 30M12-341



TROW#6*		4 824 094.8	295 915.0
TROW#7*		4 824 128.7	295 947.6
TROW#8*		4 824 140.9	295 958.0
TROW#9*		4 824 162.8	295 965.4
TROW#10*		4 824 166.5	295 954.5
HOLE#1*	93.1	4 824 946.7	295 846.3
HOLE#2*	79.3	4 824 965.0	295 856.0
HOLE#3*	75.6	4 824 009.1	295 876.5
HOLE#4*	75.6	4 824 048.9	295 900.8
HOLE#5*	74.2	4 824 088.9	295 919.4
HOLE#6*	79.5	4 824 132.3	295 945.1
HOLE#7*	87.5	4 824 147.7	295 147.7
HOLE#8*	92.3	4 824 961.0	295 162.0



DATE	BY	DESCRIPTION
DESIGN	SKP	CHK SKP
DRAWN	AN	CHK PKC
		CODE
		LOAD
		DATE MAY 2012
		STRUCT
		DWG 19-1351-174-1



- ⊕ Existing BH – Dept of Highways 1933
- ⊕ Existing BH – Trow 1958
- ⊕ Existing BH – Thurber 2010 (Access Road)
- ⊕ Existing/Proposed BH – Thurber 2011 (Current Investigation)

Notes

1. * Indicates base of footing/top of bedrock interface elevation.

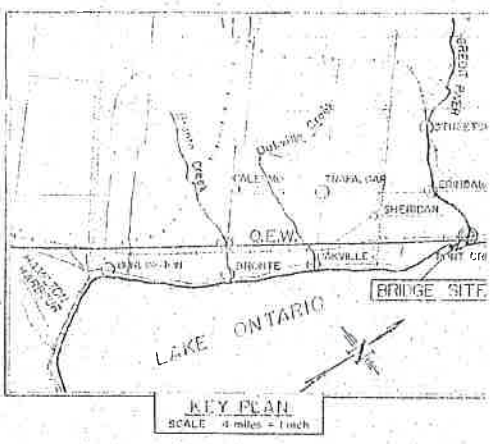
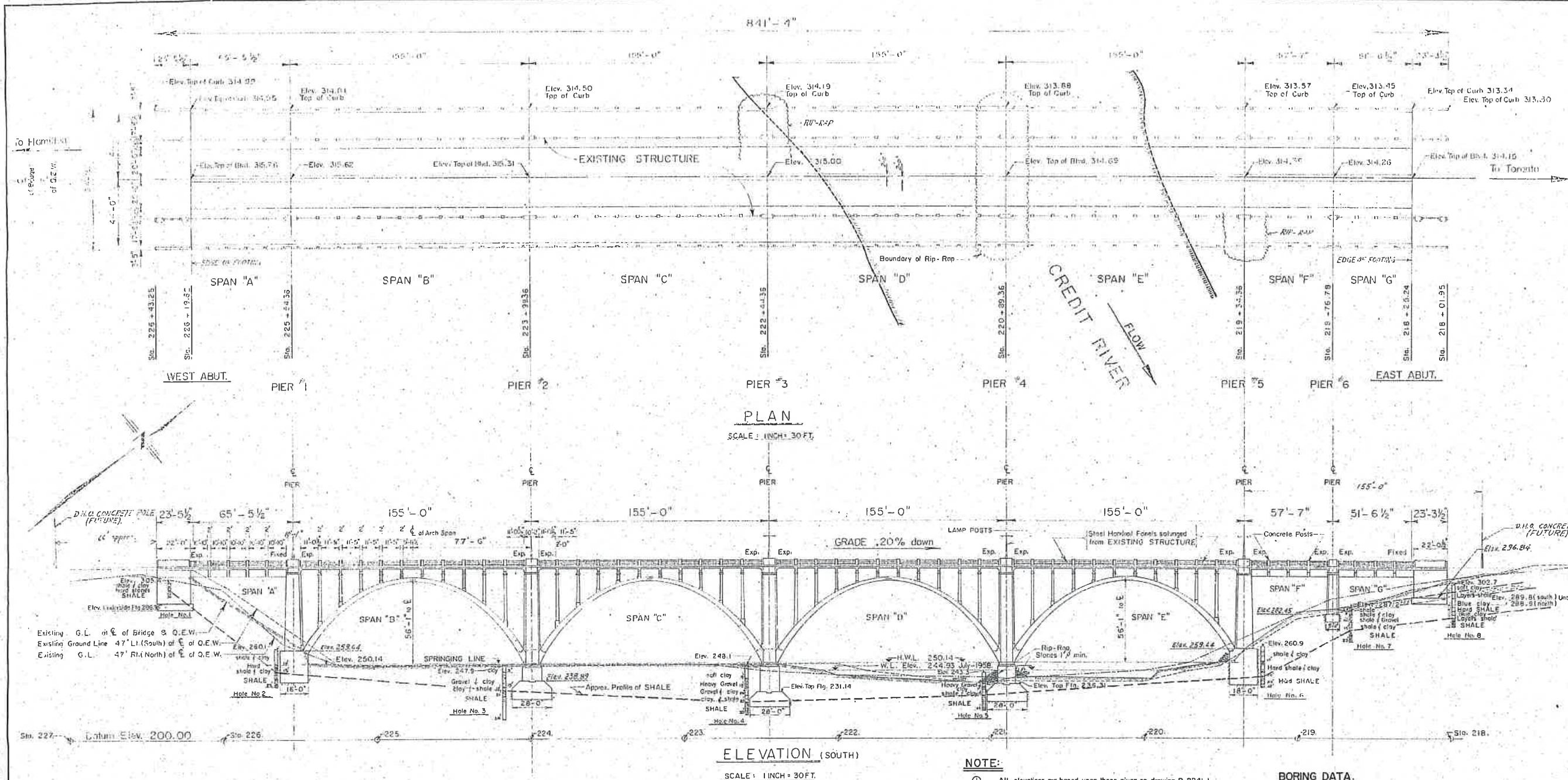
BRIDGE TWINNING OVER
CREDIT RIVER
QUEEN ELIZABETH WAY
BOREHOLE LOCATION PLAN

JOB# 19-1351-174



THURBER ENGINEERING LTD.
GEOTECHNICAL • ENVIRONMENTAL • MATERIALS

ENGINEER:	DRAWN:	APPROVED:
SKP	AN	PKC
DATE:	SCALE:	DRAWING No.:
MAY 2012	NTS	19-1351-174-2



Note to District Engineer.

Concrete work on this structure must not be commenced until monuments to fix control points have been erected and checked by the District Engineer.

Note to General Contractor.

Structure to be built in accordance with Form No. 9 revised March 1957 and the Special Provisions, both copies of which may be obtained from the District Engineer.

All construction joints must be approved by the Bridge Engineer.

Top of curb to be lined up by instrument.

Concrete Mix.

Footings - 2500 p.s.i., Minimum Strength Q
All other - 3000 p.s.i.
Add 1/4 lb. Pozzolith "S" per bag of cement.

Maximum size aggregate - Footings 1 1/2"
Remainder 3/4"

SEQUENCE OF CONSTRUCTION.

- 1 CONTRACTOR TO CONSTRUCT ALL FOOTINGS ON BOTH SIDES OF BRIDGE. EACH FOOTING TO BE BACKFILLED BEFORE PROCEEDING TO NEXT FOOTING. ALSO FOOTINGS ON NORTH AND SOUTH SIDES OF EACH PIER OR ABUTMENT TO BE COMPLETED BEFORE PROCEEDING TO NEXT PIER OR ABUTMENT.
- 2 CONSTRUCT NORTH SIDE OF BRIDGE TO BEARING LEVEL AT PIERS AND ABUTMENTS AND TO TOP OF ARCH COLUMNS.
- 3 CONSTRUCT TRAFFIC BARRIER ON NORTH SIDE AND REMOVE CONCRETE FROM EXISTING STRUCTURE ON NORTH SIDE ACCORDING TO PLANS. (CONTRACTOR TO TAKE CARE NOT TO DAMAGE NEW STRUCTURE FROM FALLING DEMOLISHED CONCRETE).
- 4 COMPLETE CONSTRUCTION OF NORTH PORTION AND OPEN THIS PORTION TO TRAFFIC WHEN CONCRETE HAS CURED.
- 5 CONSTRUCT TRAFFIC BARRIER ON SOUTH SIDE OF EXISTING BRIDGE AND COMPLETE BRIDGE CONSTRUCTION.
- 6 CONTRACTOR TO DIVERT TRAFFIC TO NEW PORTIONS OF BRIDGE AND REPAIR EXISTING BRIDGE AS SHOWN ON DWG. D-4141-12.

LIST OF DRAWINGS:

- D-4141-1 PLAN & ELEVATION
 2 PLAN AND ELEVATION OF ARCH SPANS
 3 ELEVATIONS & SECTIONS OF PIERS No.2,3,4
 4 APPROACH SPANS - EAST.
 5 SECTIONS AND DETAILS
 6 ELEVATIONS & SECTIONS OF PIER No.6
 7 PIER No.5
 8 PIER No.1
 9 APPROACH SPAN - WEST
 10 FOOTINGS
 11 BEARING PLATES
 12 METHODS FOR REPAIRING EXISTING BRIDGE
 13 HANDRAIL PANELS
 14 REINFORCING STEEL
 15 REINFORCING STEEL
 16 REINFORCING STEEL
 17 REINFORCING STEEL
 18 REINFORCING STEEL
 19 REINFORCING STEEL
 20 REINFORCING STEEL
 21 REINFORCING STEEL
 22 PROPOSED ARCH FALSEWORK.

NOTE:

- 1 All elevations are based upon those given on drawing D-2241-1 of the original MIDDLE ROAD BRIDGE over the CREDIT RIVER Contract No. 33-48.
- 2 General Contractor shall be responsible for finishing the bridge bearing seats to the specified elevations to a tolerance plus or minus 1/8 inch. If bridge bearing seats are cast too low general contractor shall provide full bearing steel shim plates to bring bearing seats up to the correct elevations. If bridge bearing seats are cast too high these shall be bush hammered down.
- 3 Deck to be treated with a 5% SILICONE solution (see SPECS).
- 4 Foundation material is considered to be similar to that shown on drawing D-2241-1.
- 5 All exposed edges to be chamfered 1/2 inch except as noted.
- 6 Where new concrete is to be poured against existing concrete surface the surface of the existing concrete shall be roughened by the contractor with or without hammer and then soaked with water before the new concrete is poured.
- 7 Footings below shale line to be poured against rock without the use of forms.
- 8 No explosives to be used for removing old concrete.
- 9 Old handrail panels, lamp posts, and catch basin tops to be salvaged for new structure.
- 10 Clear concrete cover: footings 3 inches
superstructure 2 inches
deck 1 inch, except as noted.
- 11 Original bridge plans during construction to be read in conjunction with these plans.
List of original MIDDLE ROAD BRIDGE over the CREDIT RIVER contract No. 33-48.
Drawings No. D-2241-1 to 7 inclusive.

BORING DATA.

The complete soil investigation report, BA 782, may be examined at the Bridge Office, 280 Davenport Rd. Toronto. THE DEPARTMENT OF HIGHWAYS does not guarantee the accuracy of this report nor the boring data shown on this plan.

Elevations of footings shown on original Middle Road Bridge over the Credit River Contract No. 33-48 were assumed to be correct.

Note:-

Stations shown taken from PLAN D-2241-1
 Station 218 + 01.95 = Sta. 470 + 01.04
 Existing Structure

W.P. 65-

DEPARTMENT OF HIGHWAYS-ONTARIO
 BRIDGE OFFICE-TORONTO

CREDIT RIVER BRIDGE
 WIDENING

THE KING'S HIGHWAY No. Q.E.W. DIST. CO. PEEL TWP. TORONTO LOT 5-8 CON. VAN

PLAN & ELEVATION

APPROVED

BRIDGE ENGINEER

DESIGN ENGINEER

REVISIONS

NO.	DATE	BY	DESCRIPTION
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DATE: MARCH, 1959

Twop# 81-263-1-C

Appendix D

Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES

Spread Footings on Shale Bedrock	Augered H-Piles Socketted into Shale	Augered Caissons Socketted into Shale
<p>Advantages:</p> <ul style="list-style-type: none"> i. Consistent with foundations of existing bridge. ii. Relatively high geotechnical resistance is available on the shale. iii. Lower unit costs than deep foundations. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Subexcavation within sheetpiled cofferdams will be required in the floodplain. ii. Shale is prone to rapid deterioration upon exposure to water and air and, therefore, has to be adequately protected prior to footing construction. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. Augered H-Piles will develop geotechnical resistance comparable to driven piles to refusal. ii. Installation of piles could continue in freezing weather. iii. Foundation construction requires less volume of excavation than footings <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher unit costs than spread footings. ii. Potential difficulties penetrating hard limestone/shale slabs layers during augering. iii. Space between H-pile and hole sidewall requires grouting with cementitious materials. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. High resistance is available for caissons founded within shale bedrock. ii. Construction of caissons could continue in freezing weather. iii. Foundation construction requires less volume of excavation than footings. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher unit costs than spread footings ii. Potential difficulties penetrating hard limestone/shale slabs layers during augering. iii. More likely to encounter groundwater problems during installation due to the large hole sizes. iv. Potential difficulties in cleaning excavation base and sidewall.
<p>Medium risk of encountering groundwater seepage during footing construction within sheetpiling in the floodplain that would require groundwater control measures.</p>	<p>Medium risk of encountering harder layers within rock that would require additional procedures to advance the augers to the desired elevation.</p>	<p>Medium risk of encountering harder layers within rock that would require additional procedures to advance the augers to the desired elevation. Medium risk of encountering groundwater seepage that would require balancing water head and tremie concrete.</p>
FEASIBLE	FEASIBLE	FEASIBLE