

**FOUNDATION INVESTIGATION AND DESIGN REPORT  
PRELIMINARY DESIGN AND ENVIRONMENTAL ASSESSMENT  
QEW MISSISSAUGA ROAD OVERPASS  
MISSISSAUGA, ONTARIO  
W.O. 08-20008, ASSIGNMENT NO. 2008-E-0025**

**GEOCRES Number: 30M12-342**

**Report to**

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**PART 1: FACTUAL INFORMATION**

**1 INTRODUCTION**

This report presents the factual findings obtained from a preliminary foundation investigation carried out for the proposed replacement of the QEW/Mississauga Road overpass. This investigation was carried out to support the preliminary design of the 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 near the foundation elements and, based on the data obtained, to provide a borehole locations and soil strata drawing, records of boreholes, stratigraphic profile, laboratory test results and a written description of the subsurface conditions. A model of the subsurface conditions was developed from the data obtained during the course of the present investigation.

Information contained in the following report for a previous investigation was reviewed as part of the current investigation.

- Racey, MacCallum and Associates Limited report titled “Soils Investigation, Queen Elizabeth Way Overpass at Mississauga Road”, GEOCRES No. 30M12-116, Report No.: S-500-516/55/T-103-1, dated June 20, 1955 (Reference 1).

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/Mississauga Road overpass is located 500 m south-west of the QEW/Credit River crossing on a flat landscaped area of small trees and shrubs close to residential housings.

The existing QEW/Mississauga Road interchange structure is a single span overpass that is currently proposed to be replaced by a new structure of sufficient width to accommodate extra lanes on the

QEW as a result of widening works at Credit River Bridge, located north-east of the interchange structure.

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

### 3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project was carried on September 17, 2011. Two boreholes were drilled and sampled. The boreholes were drilled within the Mississauga Road cut and were numbered MR11-01 and MR11-02. Borehole MR11-01 was drilled to the west and borehole MR11-02 was drilled to the east of the overpass. Details of borehole depth are summarised below:

<b>Borehole Number</b>	<b>Ground Elevation</b>	<b>Borehole Termination Elevation</b>	<b>Borehole Termination Depth (m)</b>
MR11-01	94.3	90.7	3.6
MR11-02	93.3	85.6	7.7

The borehole locations were staked and/or marked in the field by Thurber. Utility clearance was obtained at all borehole locations prior to drilling. The northing and easting co-ordinates and ground surface elevations of the completed boreholes were provided by MRC.

A track mounted CME 55 drill rig was used to conduct the drilling, sampling and in-situ testing. Auger drilling techniques were used to advance the boreholes through soils and weathered bedrock. Soil and weathered bedrock samples were obtained at selected intervals using a split spoon sampler in conjunction with the Standard Penetration Testing (SPT). These boreholes were further advanced into bedrock by NQ size rotary coring techniques to recover core samples. All rock cores were logged, and properties including the Total Core Recovery (TCR), Rock Quality Designation (RQD) and the Fracture Indices (FI) were determined where applicable.

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

Standpipe piezometer was installed in borehole MR11-02 to monitor groundwater level. The piezometer comprises a 25 mm diameter Schedule 40 PVC pipes with 1.52 m long slotted screens. The sand screen surrounding the pipe extends above the slotted portion of the pipe. Bentonite holeplug seals were placed above the sand screen in the piezometer installation. The completion details of the boreholes and piezometer installation are summarised in Table 1 attached after the text

of the report. On completion of drilling and sampling, boreholes without piezometer installation were backfilled with bentonite holeplug and then concreted to surface.

#### **4 LABORATORY TESTING**

All recovered soil samples were subjected to visual identification and to natural moisture content determination. The results of this testing are shown on the Record of Borehole sheets in Appendix A. 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.

Point Load Tests (PLT) were carried out on selected rock core samples and the results are shown in Table 2 attached after the text of the report and on the Records of Boreholes 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 and rock stratigraphy are presented in these records and on the “Borehole Locations and Soil Strata” drawing in Appendix C. General description of the stratigraphy is given in the following paragraphs. The factual information established at the borehole locations governs any interpretation of site conditions.

The stratigraphy at the Mississauga Road Interchange overpass typically consists of pavement structure overlying fill. Shale bedrock is encountered below the fill.

##### **5.1 Pavement Structure**

Pavement structure typically consists of 100 to 250 mm of asphalt overlying 400 mm of brown sand with some gravel. Moisture content between 3% and 3.5% was measured in the sand.

A SPT ‘N’ value of 53 blows per 0.3 m penetration was recorded within the granular layer of the pavement structure.

##### **5.2 Gravelly Sand and Sand Fill**

Fill was encountered below the pavement structure in both boreholes. In Borehole MR11-01, the fill comprises brown sand containing some gravel, some silt and trace of clay. In Borehole MR11-02, the fill consists of brown gravelly sand overlying sand with trace silt. Trace of organics was noted in the gravelly sand fill, whereas limestone and shale fragments were noted in the underlying sand fill.

The fill is 1.0 m thick in Borehole 11-01 and the underside of the fill is at 1.1 m depth (Elevation 93.2 m). In Borehole MR11-02, the fill below the road granular is 2.1 m thick and extends to 2.8 m depth (Elevation 90.5 m). Within the upper 1.1 to 1.3 m, the SPT ‘N’ values of the fill ranged from 47 to 63 blows per 0.3 m penetration indicating a dense to

very dense condition. Below 1.3 m depth in Borehole 11-02, 'N' values of 12 and 18 blows indicate a compact condition. The moisture content of the fill ranged from about 4% to 10%.

Grain size analysis was conducted on a sand fill sample from Borehole MR11-01. Results of the test is plotted in Figure B1 in Appendix B and summarized below.

Gravel %	17
Sand %	62
Silt %	16
Clay %	5

Grain size analysis was conducted on another sand fill sample from Borehole MR 11-02. Results of the test are presented in Figures B2 and summarised below.

Gravel %	49
Sand %	37
Silt & Clay %	14

Visual inspection of the sample used for the above test indicates high content of shale fragments and occasional limestone fragments in the sand fill above the bedrock in Borehole MR11-02. The 49% of gravel indicated by the grain size analysis test results comprised mainly of shale fragments.

### 5.3 Shale Bedrock

Weathered shale bedrock was encountered below the fill described above and proven by coring in boreholes MR11-01 and MR11-02. The depths and elevations of the bedrock surface are summarised below:

<b>Borehole Number</b>	<b>Depth to Weathered Shale (m)</b>	<b>Top of Weathered Shale Elevation (m)</b>
MR11-01	1.1	93.2
MR11-02	2.8	90.5

The bedrock was described as grey, fine grained, thinly bedded shale with hard limestone interbeds throughout. The shale is completely to highly weathered in Borehole MR11-01 and highly to moderately weathered in Borehole MR11-02. The degree of weathering generally decreases with depth. Clay seams were noted at depths of 1.9 to 2.2 m within the shale in borehole MR11-01. A hard limestone interbed at depths of 5.7 to 5.8 m and vertical joints at depths of 6.6 to 7 m were noted in borehole MR11-02.

Total Core Recovery (TCR) of the bedrock was generally between 55% and 100%. The Rock Quality Designation (RQD) values typically ranged between 30% and 70%, indicating

rock quality to be poor to fair. In borehole MR11-01, the first run of the core returned a RQD value of zero. The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, is typically greater than 20 and occasionally ranged from 0 to 15.

The estimated unconfined compressive strength (UCS) of the shale, interpreted from point load tests conducted on intact rock cores, ranged from 2 to 16 MPa, indicating a very weak to weak rock. A point load test on a limestone interbed in Borehole MR11-02 gives a UCS value of 190 MPa indicating a very strong rock.

#### 5.4 Groundwater Conditions

A standpipe piezometer was installed in Borehole MR11-02. The piezometric reading obtained on September 30, 2011 is presented below.

Borehole	Date	Ground Surface Elevation (m)	Groundwater	
			Depth (m)	Elevation (m)
MR11-02	September 30, 2011	93.3	2.3	91.0

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

## 6 MISCELLANEOUS

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

The drilling and sampling equipment was supplied and operated by DBW Drilling Limited of Ajax, Ontario. The field work was supervised on a full time basis by Mr. David Ametrano 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 Miss Mei Cheong. The report was reviewed by Dr. Pang.

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 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 proposed replacement of the QEW/Mississauga Road Overpass.

The existing QEW/Mississauga Road overpass is a single-span structure supported on two abutments. The overpass sits within the Mississauga Road cut and carries the QEW over the road. Design drawings for the existing bridge provided by MRC indicate that existing abutment footings (near the mid-point of the bridge) are founded on bedrock at Elevation 91.6 m. Based on Borehole MR11-01, the footings are founded within the rock cut.

A preliminary general arrangement drawing provided by MRC indicates that the proposed replacement structure will be in the order of 55 m wide measured parallel to Mississauga Road. The new structure will be designed to carry up to four travelled lanes in each direction of the QEW, and include median lanes and a median barrier wall. The Mississauga Road cut at the structure location is approximately 7 to 8 m deep. Depending on the staging arrangements, construction of the replacement structure may require temporary detour and/or stoppage of traffic on Mississauga Road.

Information provided by MRC indicates that the new bridge will be constructed in three stages as outlined in the following. Stage one consists of constructing the new structure within the bridge widening footprint to the north of the existing bridge. After completion, the westbound traffic for QEW will be shifted to this new structure. The demolishing of the northerly portion of the existing bridge will be carried out after which the central portion of the new structure will be constructed. After its completion, the eastbound QEW traffic will be shifted to this new central portion. The existing southerly portion of the bridge will then be demolished and replaced by the southerly portion of the new structure.

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

## **8 NEW ABUTMENT FOUNDATIONS**

The proposed bridge will be a single span structure supported on the east and west abutments that are parallel to the Mississauga Road alignment. At Mississauga Road grade (Elevation 94.3 m) near the northerly limit of the proposed structure, the stratigraphy consists of a pavement structure overlying weathered shale bedrock at a shallow depth of 1.1 m (Elevation 93.2 m). At road grade (Elevation 93.3 m) near the southerly limit of the existing structure, the stratigraphy consists of a pavement structure overlying gravelly sand fill, grading into a compact sand fill (possibly footing trench backfill) which is underlain by weathered shale bedrock at 2.8 m depth (Elevation 90.5 m). Information from Reference 1 indicates that the original ground prior to construction of the Mississauga Road cut sloped from Elevation 99.2 m at the northwest to Elevation 98.4 m at the southeast. The original top of bedrock was noted to be within the range of Elevations 96.6 and 97.0m. The lower portion of the existing Mississauga Road cut is formed through shale bedrock.

### **8.1 Foundation Alternatives**

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

- Spread footings founded on shale bedrock
- Augered steel H-piles socketted into shale bedrock

Spread footings on native soil were not considered as native soil is absent within the Mississauga Road cut. Augered H-piles are required if an integral abutment design is considered. Augered caissons and driven piles are not considered feasible at this site due to the shallow bedrock. Advantages and disadvantages of feasible foundation alternatives are presented in the table in Appendix D.

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

Spread footings founded on shale bedrock are considered feasible to support the new abutments. The required depth of excavation for footing construction on bedrock will vary depending on the location. Within the footprint of the structure widening beyond the cut, excavations could extend through 2 to 2.5 m of fill and native silty clay overburden to reach bedrock. Within the existing bridge footprint in the cut, excavations to bedrock would involve removing an existing sandy road fill.

For planning and preliminary design purposes, the new footings should be founded on bedrock, at or below the frost penetration depth, and below the base of the existing cut. At locations where new footings overlap existing footing trenches, the space not occupied by the new footings should be backfilled with concrete fill.

### **8.2.1 Bearing Resistance**

The spread footings for the new bridge should be founded on weathered shale bedrock or mass concrete fill on bedrock. These footings should be founded at or below the elevation of the adjacent existing footings. The design drawings show that the existing footings are founded on bedrock at Elevation 91.6 m near the mid-point of the bridge. The boreholes drilled for the current investigation indicate that bedrock is at Elevations 93 m and 90 m near the northerly and southerly limits of the cut, respectively. The actual founding elevation at specific locations will have to be determined during detailed design.

The footings founded on weathered shale bedrock, or concrete fill of similar class as that of the footings placed on bedrock, should be designed using a Factored Geotechnical Resistance at ULS of 1,000 kPa, and a Geotechnical Resistance at SLS (25 mm settlement) of 800 kPa.

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

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

### **8.2.2 Lateral Resistance**

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.

### **8.2.3 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.

### **8.2.4 Footing Subgrade Preparation**

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

Shale is prone to softening upon exposure to water and air. Concrete or mud slab should be placed within 24 hours following completion of excavation to prevent deterioration of the shale. The mud slab should be at least 100 mm thick and formed with the same class of concrete as that of the footings. Where sub-excavation is required to remove the existing footings, backfill and other unsuitable material from below the design founding level, the founding surface should be re-established using concrete fill of the same class of concrete as that used for the footing.

### 8.3 Augered H-Piles

If an integral abutment design is being considered, the structure will need to be supported 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.

#### 8.3.1 Geotechnical Axial Resistance

For HP 310x110 steel H-piles placed in rock sockets, an axial Factored Geotechnical Resistance at ULS of 2,000 kN per pile, and a Geotechnical Resistance at SLS of 1,800 kN per pile is recommended.

A minimum pile length of 6 m should be used. The structural resistance of the pile must be checked by the structural designer.

The axial resistance will not be significantly affected by pile spacing for piles ending on bedrock.

#### 8.3.2 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.

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

##### Sand Fill

$$k_s = n_h \cdot z / D \text{ (kN/m}^3\text{)}$$

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

##### Shale

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

$$p_{ult} = \begin{aligned} &0 \text{ between ground surface and } 1.5D \\ &9 C_u \text{ (kPa) at and below a depth of } 1.5D \end{aligned}$$

where  $n_h$  = coefficient related to soil density, kN/m<sup>3</sup>

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D	=	diameter of augered hole (m)
C <sub>u</sub>	=	undrained shear strength (kPa).
z	=	depth of caisson embedment (m)

At this site,  $n_h$  may be taken as 4,000 kN/m<sup>3</sup> for the sand fill and an equivalent C<sub>u</sub> of the bedrock may be taken as 300 kPa.

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

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:

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

where C<sub>u</sub> = 300 kPa (equivalent Mohr-Coulomb cohesion based on Hoek and Brown rock mass classification)

D = diameter of socket (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-pile interaction analysis, the modulus of subgrade reaction ( $k_s$ ) may have to be reduced based on pile spacing.

### 8.3.3 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.

The augered pile installation equipment should be capable of dislodging and removing any obstructions such as cobbles, boulders and other obstructions in the fill and till deposits. 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. Temporary steel liners may be required to support the hole sidewalls and minimize groundwater inflow.

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

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.

## 9 ABUTMENT WALL BACKFILL AND LATERAL PRESSURES

Backfill to abutment walls where required, should be in accordance with OPSS 902, and placed to the extents shown in OPSD 3101.150. Any backfill to the abutments should consist of Granular A or Granular B Type II material meeting the requirements of Special Provision 110S13 “Amendment to OPSS 1010, April 2004”.

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

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

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

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

$K$  = earth pressure coefficient (see Table 9.1)

$\gamma$  = unit weight of retained soil (see Table 9.1)

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

$q$  = value of any surcharge (kPa)

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

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

**Table 9.1 – Earth Pressure Coefficients**

Wall Condition	Earth Pressure Coefficient (K)					
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$		Native Silty Clay and Shale Bedrock $\phi = 30^\circ, \gamma = 25 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)
Active (Unrestrained Wall)	0.27	0.40	0.31	0.48	0.33	0.54
At rest (Restrained Wall)	0.43	-	0.47	-	0.50	-
Passive (Movement Towards Soil Mass)	3.7	-	3.3	-	3.0	-

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

The factors in Table 9.1 are “ultimate” values and require certain movements for the respective conditions to be mobilized. The values to be used in design can be estimated from Figure C6.9.1 (a) in the Commentary to the Canadian Highway Bridge Design Code.

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

## **10 EXCAVATION AND GROUNDWATER CONTROL**

### **10.1 Temporary Conditions**

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

Excavation for foundation construction will extend through the pavement structure and road fill into the shale bedrock. Excavation into the slopes of the Mississauga Road cut, including the area to the north of the existing bridge for its widening, may encounter native clayey silt and silty clay, and shale bedrock.

All excavations must be carried out in a manner that avoids destabilising the existing overpass structure and the existing slopes along the Mississauga Road cut.

The groundwater table near the base of the cut is at a depth of 2.3 m, which is above the base of the proposed footing excavations. It should be noted that the site is within an existing cut situated in a local, topographically low, area. The Contractor should make provision to remove any groundwater, seepage water or surface water collecting in the excavations.

Groundwater control at this site will likely involve diverting surface runoff away from the excavations and sump pumping. Filtered sumps must be designed properly so that construction drainage water containing eroded soil, weathered rock and fines do not flow downslope along Mississauga Road.

#### **10.1.1 Temporary Cut through Soils**

Excavation at the base of the Mississauga Road cut will extend through 1.1 m to 2.8 m of pavement structure and fill. Excavation into the existing slopes of the Mississauga Road cutting may encounter clayey silt and silty clay. Cobbles, boulders and shale fragments may be encountered in these soils. Where space permits, temporary unsupported excavations through a majority of soils and weathered shale (upper 2 m) at this site may be formed with side slopes not exceeding 1H : 1V. Flatter slopes may be required at locations where the soils are less competent than that assumed or where water seepage affects surficial stability.

It is anticipated that protection systems (temporary shoring) will be required at some locations including areas immediately adjacent to the existing structure.

#### **10.1.2 Temporary Cut through Rock**

Excavation into the existing slopes of the Mississauga Road cut and in other areas may encounter shale with hard limestone interbeds. Temporary unsupported excavations through shale bedrock below the upper 2 m may be formed with near vertical side slopes. 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 for this site.

#### **10.1.3 Protection Systems**

Temporary cuts along Mississauga Road will require protection systems (temporary shoring). Protection systems should be the responsibility of the Contractor and designed by a licensed Professional Engineer experienced in such designs with consideration of adjacent traffic loads and any sloping surfaces to satisfy the requirements of OPSS 539.

#### **10.2 Permanent Conditions**

Permanent cut slopes through overburden soils and shale bedrock should be formed at an inclination not steeper than 2H : 1V. Vegetation cover should be established on all exposed soil and shale slopes to protect against surficial erosion with reference to OPSS 572.

Continued seepage originating from perched water in the fill, sand and silt interlayers in the silty clay to clayey silt and joints within the shale, is expected to occur in the long term. A permanent drainage system must be redesigned and constructed within the enlarged cut.

### **11 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:

- In order to avoid undermining the adjacent existing footings and the surrounding slopes, 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 and temporary decking, where used, 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;

- Undulation in bedrock surface or sloping bedrock surface may require the use of concrete fill to prepare the design founding elevation;
- The fill and glacial tills may contain cobbles and boulders, and the underlying shale bedrock may contain hard limestone interbeds. Equipment selected for excavation or to install augered piles must be capable of penetrating, handling and/or removing these obstructions;
- Steel liner should be used to advance pile sockets into the bedrock to support the sidewalls, minimise groundwater inflow and enable machine cleaning of the socket base.

## **12 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 three boreholes must be drilled and sampled at each of the two finalized abutment locations. Each borehole should be advanced to reach shale bedrock below which the bedrock should be cored for at least 3 m.
- At each approach to the widening portion of the bridge, one borehole must be drilled within 20 m of each abutment to investigate the nature and extent of the native soils and rock at the approaches. These boreholes must fully penetrate any fill and native soils into the underlying shale bedrock.
- At least two additional boreholes must be drilled near the crest of the existing slopes on both sides of the existing cut to investigate the nature of the soils and rock that are to be excavated for foundation construction.

Depending on the final bridge configuration, it is assumed that one or both of the boreholes drilled in this preliminary investigation can be incorporated for detailed design.

### *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 for the site, foundation alternatives and design recommendations, specific spread footing and/or augered pile installation requirements, temporary foundations for traffic detour, lateral pressure design, excavation, shoring and unwatering, approach embankment stability assessment and seismic considerations.

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### **13 CLOSURE**

Engineering analysis and preparation of this foundation design report was carried out by Miss Mei Cheong, M.Phil. The report was reviewed and updated by Dr. Sydney Pang, P.Eng.

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

Engineering Analysis and Report Preparation by:  
Miss Mei Cheong, M.Phil.  
Geotechnical Specialist



Report Reviewed 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 <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  
 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

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

<u>DISCONTINUITY SPACING</u>		<u>STRENGTH CLASSIFICATION</u>			
Bedding	Bedding Plane Spacing	Rock Strength	Approximate Uniaxial Compressive Strength		Field Estimation of Hardness*
			(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.
<u>TERMS</u>		Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.	Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.	Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail
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.				
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 MR11-01

1 OF 1

METRIC

W.P. W.O. 08-20008 LOCATION N 4 823 600.3 E 295 564.2 QEW Mississauga Road Overpass ORIGINATED BY DA  
 HWY QEW BOREHOLE TYPE Solid Stem Augers/NQ Core Barrel COMPILED BY AN  
 DATUM Geodetic DATE 2011.09.17 - 2011.09.17 CHECKED BY MEF

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				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						
94.3								20	40	60	80	100		
0.0	ASPHALT: (100mm)		1	AS										
0.1	SAND, some gravel, some silt, trace clay Dense Brown Moist (FILL)		1	SS	47		94							17 62 16 5
93.3	Some limestone fragments		2	SS	62/ 0.28									
1.1	SHALE, weathered, grey						93							
92.8														
1.5	END OF SPT SAMPLING AT 1.2m. AUGER TO 1.5m AND START CORING. FOR ROCK DETAILS PLEASE REFER TO MR11-01R.													

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

# RECORD OF BOREHOLE MR11-01R

PROJECT : QEW Mississauga Rd. Overpass  
LOCATION : Mississauga, ON  
STARTED : September 17, 2011  
COMPLETED : September 17, 2011

Project No. W.O. 08-20008


INCLINATION: Vertical AZIMUTH:

SHEET 1 OF 1  
DATUM Geodetic

[illegible]

## GROUNDWATER ELEVATIONS

 SHALLOW/SINGLE INSTALLATION  
WATER LEVEL (date)

 DEEP/DUAL INSTALLATION  
WATER LEVEL (date)

LOGGED : DA  
CHECKED : MEF

RECORD OF BOREHOLE No MR11-02

1 OF 1

METRIC

W.P. W.O. 08-20008 LOCATION N 4 823 601.1 E 295 645.2 QEW Mississauga Road Overpass ORIGINATED BY DA  
 HWY QEW BOREHOLE TYPE Solid Stem Augers/NQ Core Barrel COMPILED BY AN  
 DATUM Geodetic DATE 2011.09.17 - 2011.09.17 CHECKED BY MEF

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			20	40	60	80	100		
93.3														
0.0 93.1	ASPHALT: (250mm)													
0.3 92.6	SAND, some gravel Very Dense Brown Moist (FILL)		1	SS	53		93							
0.7 92.0	Gravelly SAND, trace organics Very Dense Brown Moist (FILL)		2	SS	63		92							
1.3 90.5	SAND, fine grained, trace silt, some limestone fragments, trace shale fragments Compact Moist (FILL)		3	SS	12		91							
2.8 90.2	SHALE, weathered, grey		4	SS	18									49 37 14 (SI+CL)
3.1	END OF SPT SAMPLING AT 3.1m AND START CORING. FOR ROCK DETAILS PLEASE REFER TO MR11-02R.  Piezometer installation consists of 25mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.  WATER LEVEL READINGS: DATE      DEPTH (m)      ELEV. (m) Sep.30/11      2.3      91.0		5	SS	50/	0.100								

+ <sup>3</sup>, × <sup>3</sup>: Numbers refer to  
Sensitivity

20  
15  
10  
(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE MR11-02R

PROJECT : QEW Mississauga Rd. Overpass  
LOCATION : Mississauga, ON  
STARTED : September 17, 2011  
COMPLETED : September 17, 2011

Project No. W.O. 08-20008


INCLINATION: Vertical      AZIMUTH:

SHEET 1 OF 1  
DATUM Geodetic

[illegible]

## GROUNDWATER ELEVATIONS

▽ SHALLOW/SINGLE INSTALLATION  
WATER LEVEL (date)

 DEEP/DUAL INSTALLATION  
WATER LEVEL (date)

LOGGED : DA  
CHECKED : MEF

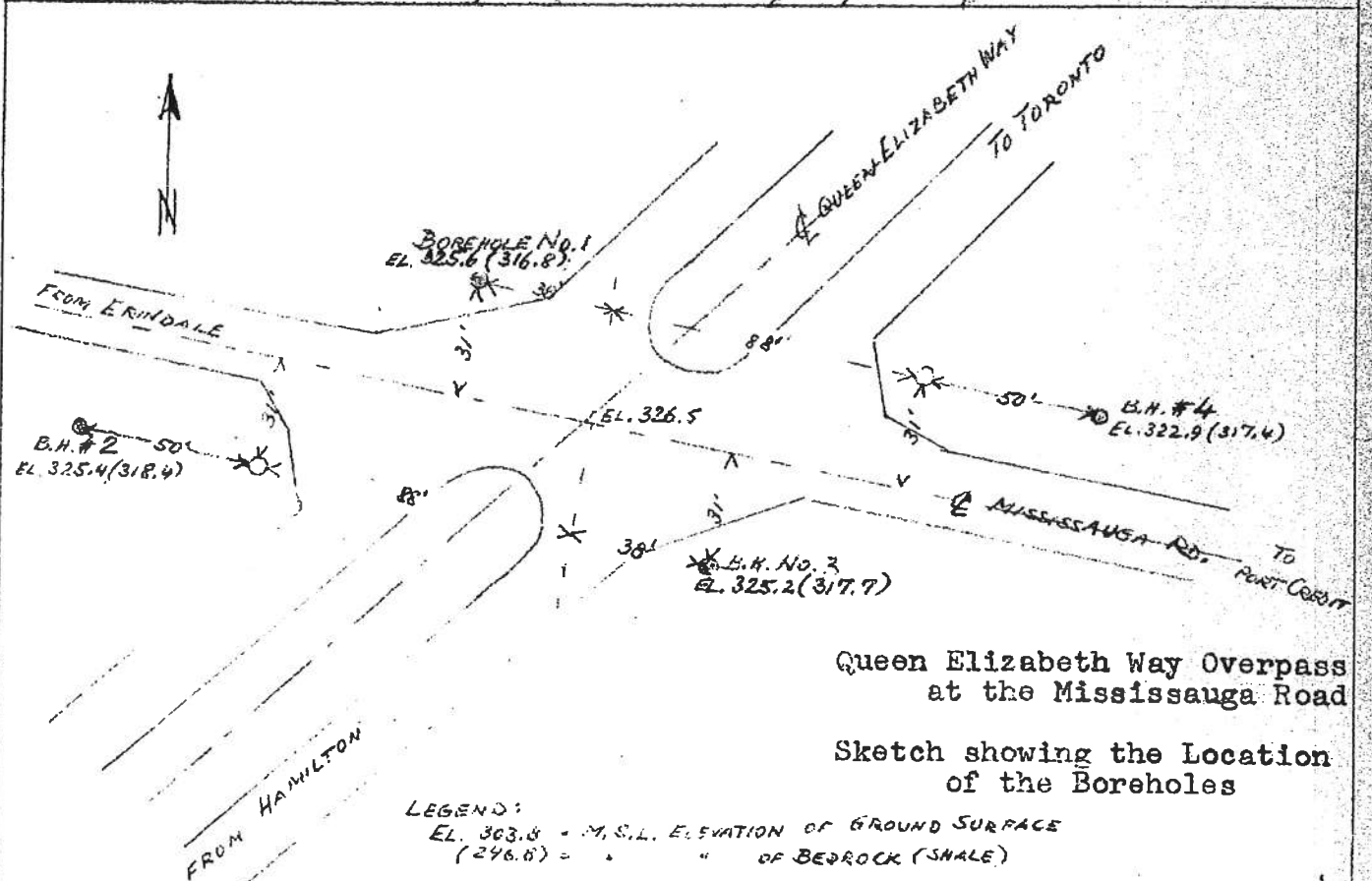
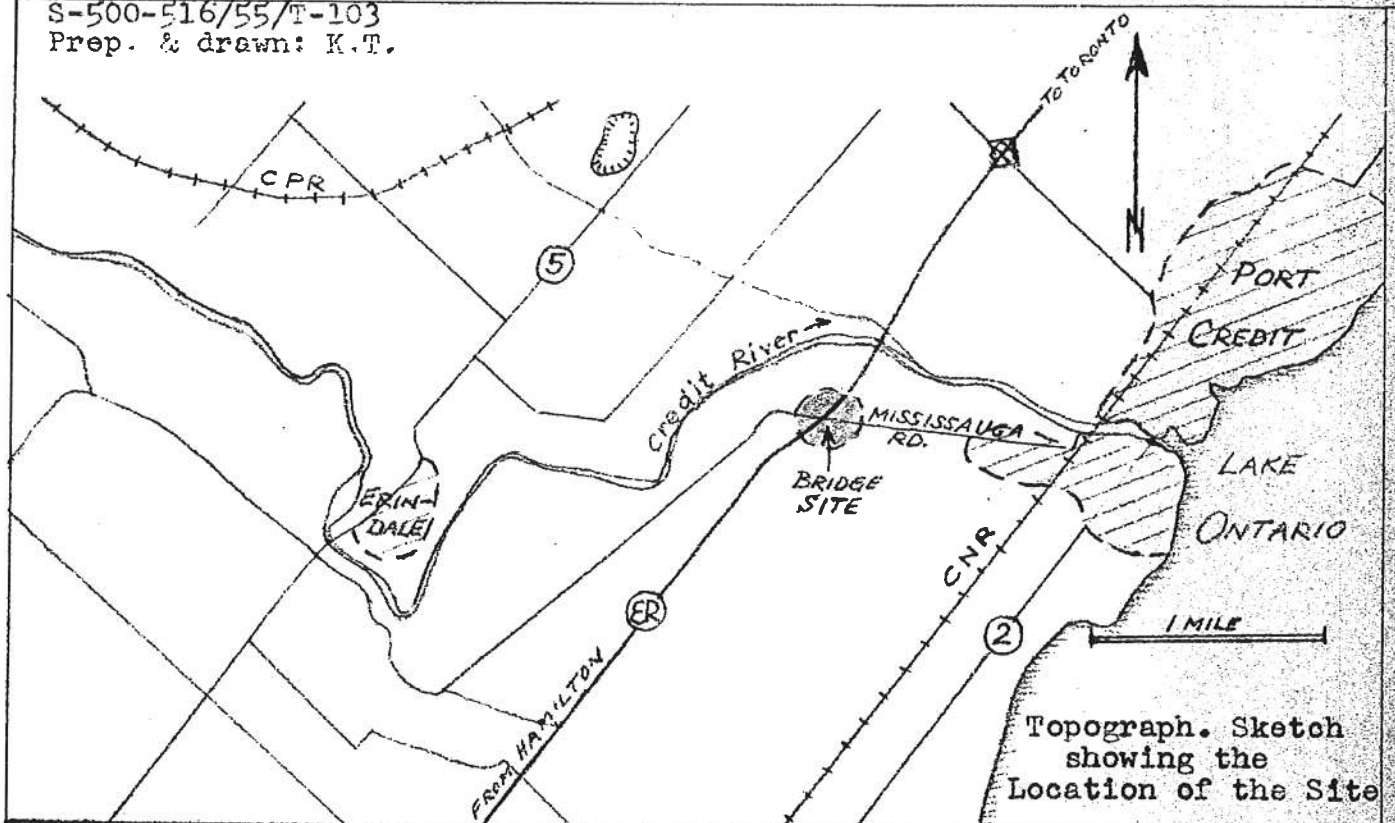
**Table A1**

**Borehole Completion Details**

<b>Borehole</b>	<b>Tip Position (m)</b>		<b>Completion Details</b>
	<b>Depth</b>	<b>Elev.</b>	
MR11-01	None installed		Bentonite seal from 3.6 to 0.3 m, then concrete to surface.
MR11-02	4.6	88.7	Sand filter from 7.7 to 2.4 m, drill cuttings and bentonite holeplug from 2.4 to 0.3 m, then concrete to surface.

ENCLOSURE No. 1

S-500-516/55/T-103  
Prep. & drawn: K.T.



Order No.: S-500-576/577-103 RACEY, MACCALLUM AND ASSOCIATES  
LIMITED

A. McADDERN  
Driller

Hole Begun 12/5/55

*Foundation Engineering Division*

Hole Ended 19/5/55

**Engineering Data Sheet for Borehole:** \_\_\_\_\_

## Helper

Job Name: Queen Elizabeth Way Overpass at Mississauga Road

B.F.W. & K.T.

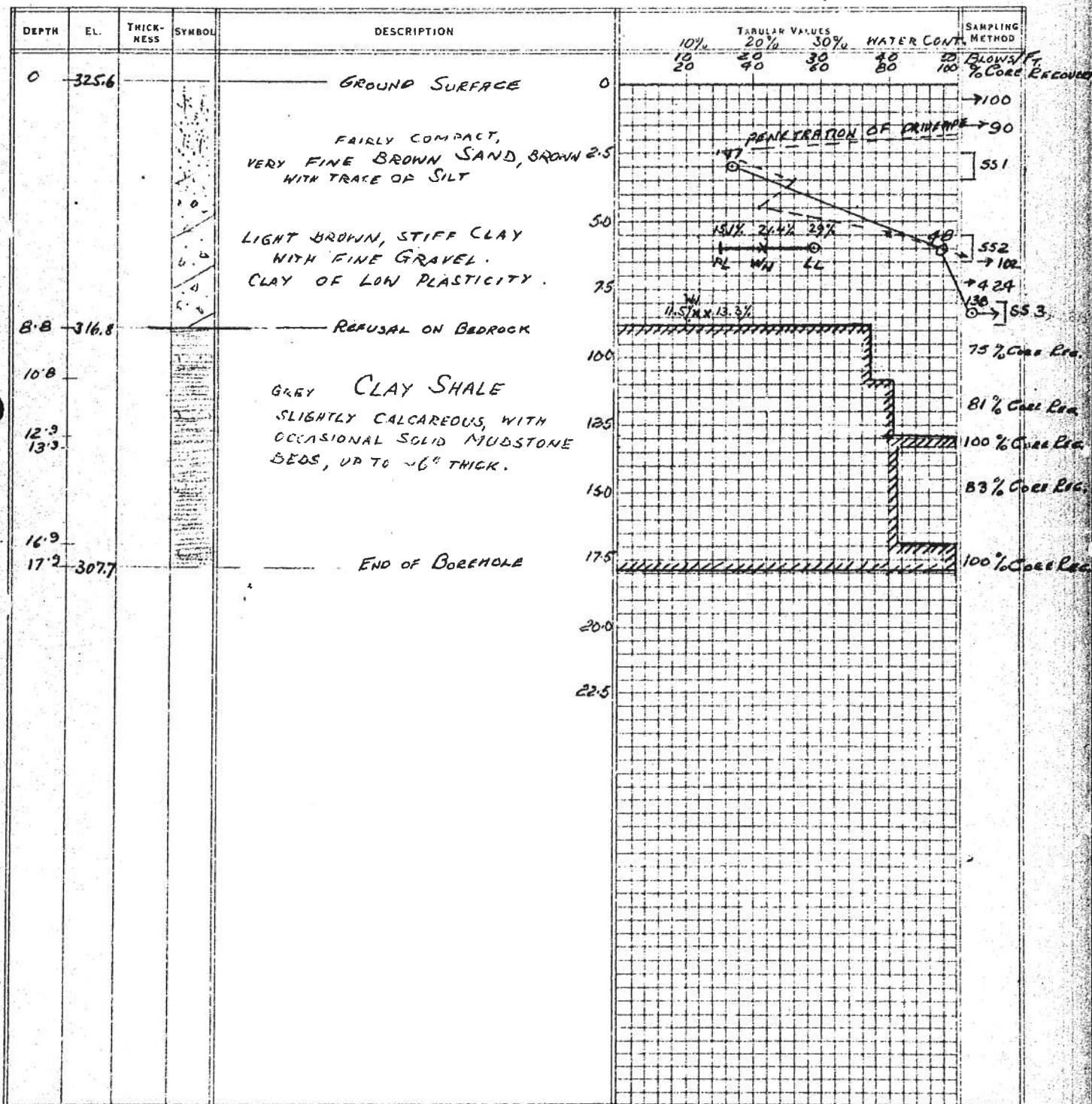
Job Located: ON QUEEN ELIZABETH HWY AT MISSISSAUGA RD.

Checked by

Hole Located: AS SHOWN ON ATTACHED SKETCH PLAN

Hole Elevation: 325.16 Datum: M. S.L.

Day 6 / Month 6 / Year 55



Order No.: S-500-716/55/7-105 RACEY, MACCALLUM AND ASSOCIATES

LIMITED

A. M. CADDEN  
Driller

Hole Begun 14/5/55

Foundation Engineering Division

Hole Ended 17/5/55 Engineering Data Sheet for Borehole: 2

Helper

Job Name: Queen Elizabeth Way Overpass at Mississauga Road

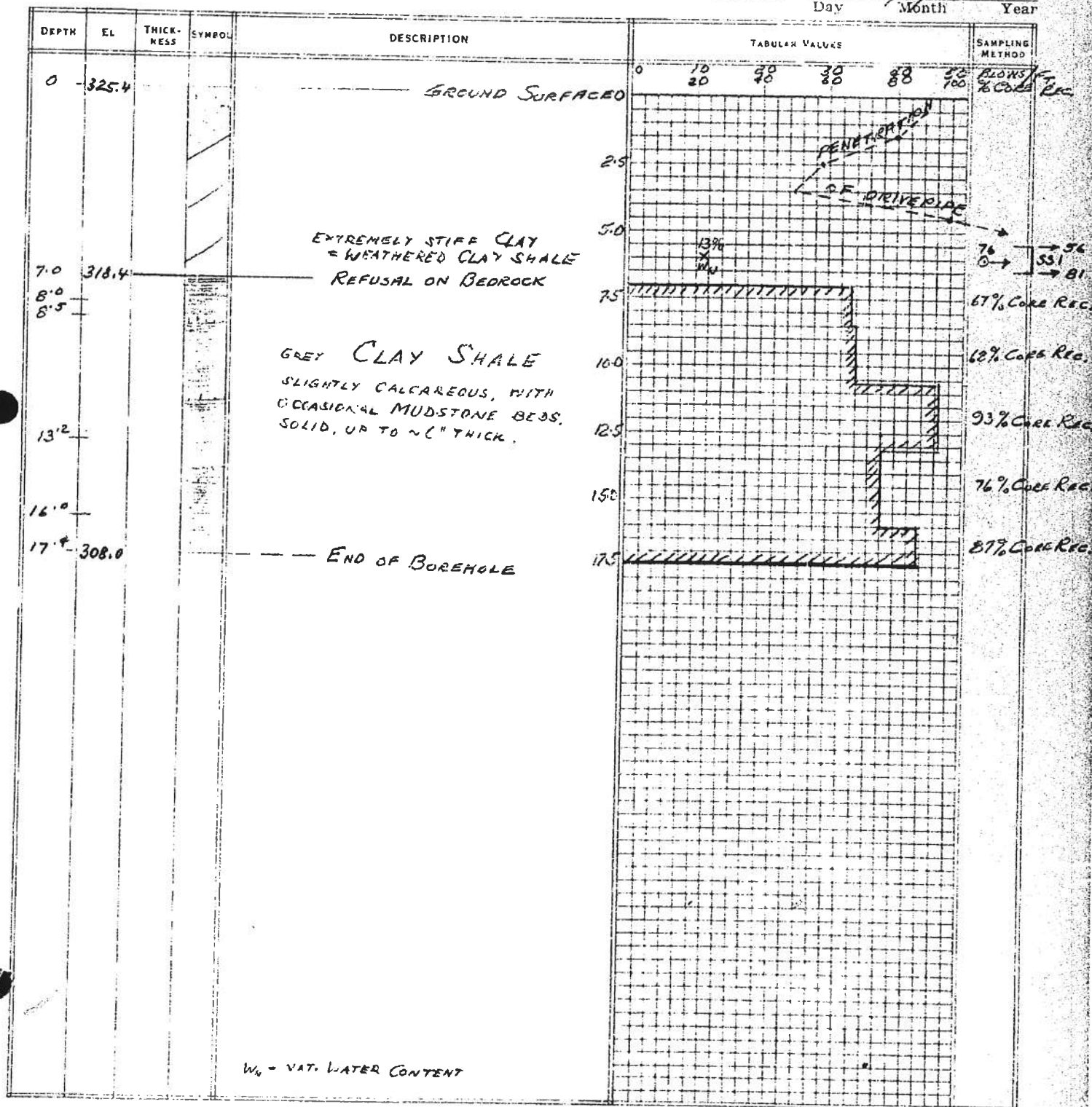
Job Located: ON QUEEN ELIZABETH HWY. AT MISSISSAUGA RD.

Hole Located: AS SHOWN ON ATTACHED SKETCH PLAN

Hole Elevation: 325.4 Datum: M.S.L.

B.F.W. & K.T.  
Checked by

Day 6/6/55 Month Year



A. McCARPEN  
Driller

Hole Begun 17/5/55

Foundation Engineering Division

Hole Ended 18/5/55

**Engineering Data Sheet for Borehole: 3**

## Helper

Job Name: Queen Elizabeth Way Overpass at Mississauga Road

B.F.W. & K.T.

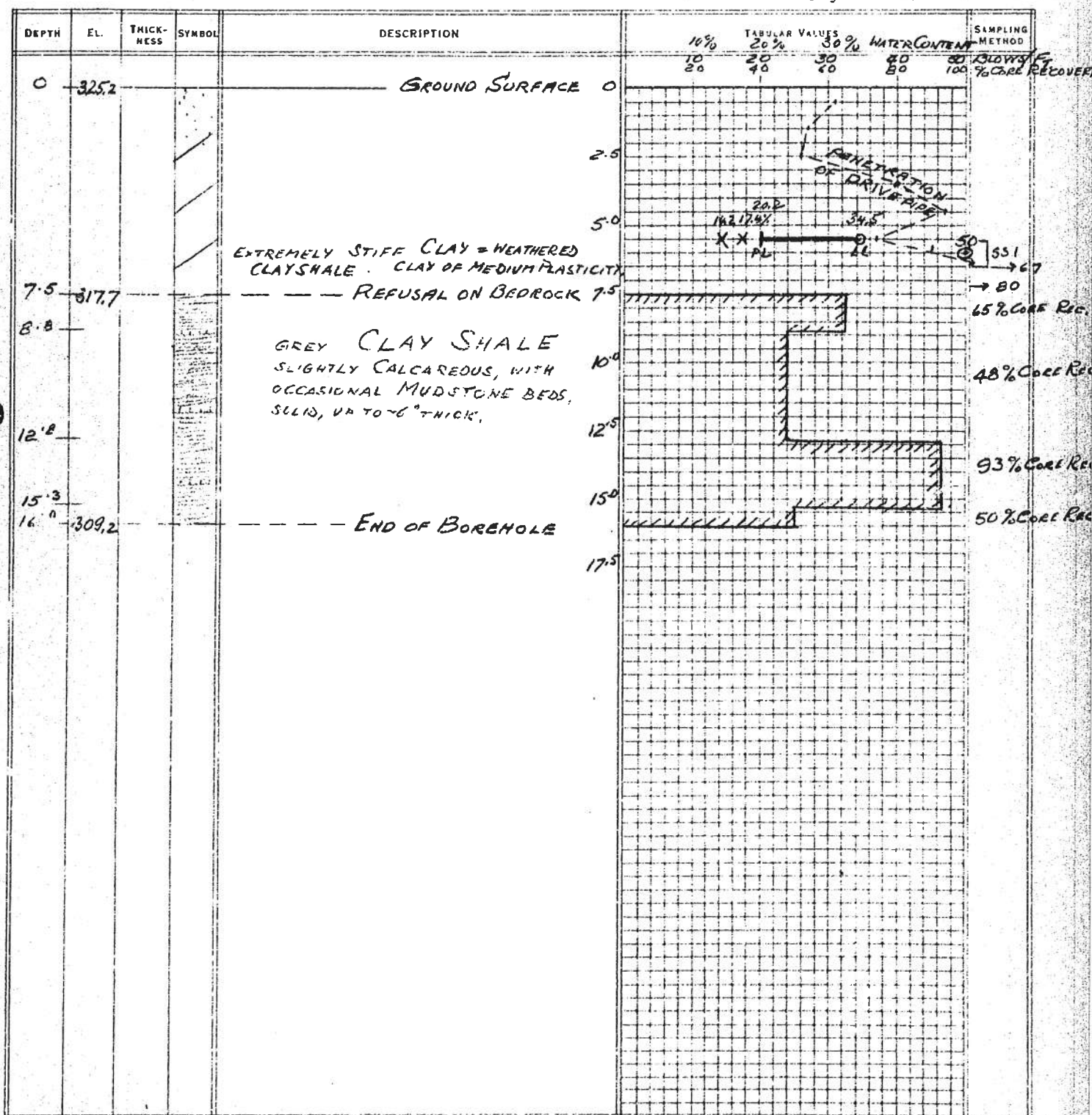
Job Located: ON QUEEN ELIZABETH HWY AT MISSISSAUGA RD.

Checked by

Hole Located: As SHOWN ON ATTACHED SKETCH PLAN

Hole Elevation: 325.2 Datum: M.S.L.

Day 6 Month 6 Year 55



Order No.: S-500-54/55/1-103 RACEY, MACCALLUM AND ASSOCIATES  
LIMITED

A. M. CADDEN  
Driller

Hole Begun 18/5/55

Foundation Engineering Division

Hole Ended 19/5/55

Engineering Data Sheet for Borehole: 4

Helper

Job Name: Queen Elizabeth Way Overpass at Mississauga Road

B. F. W. E. K. T.

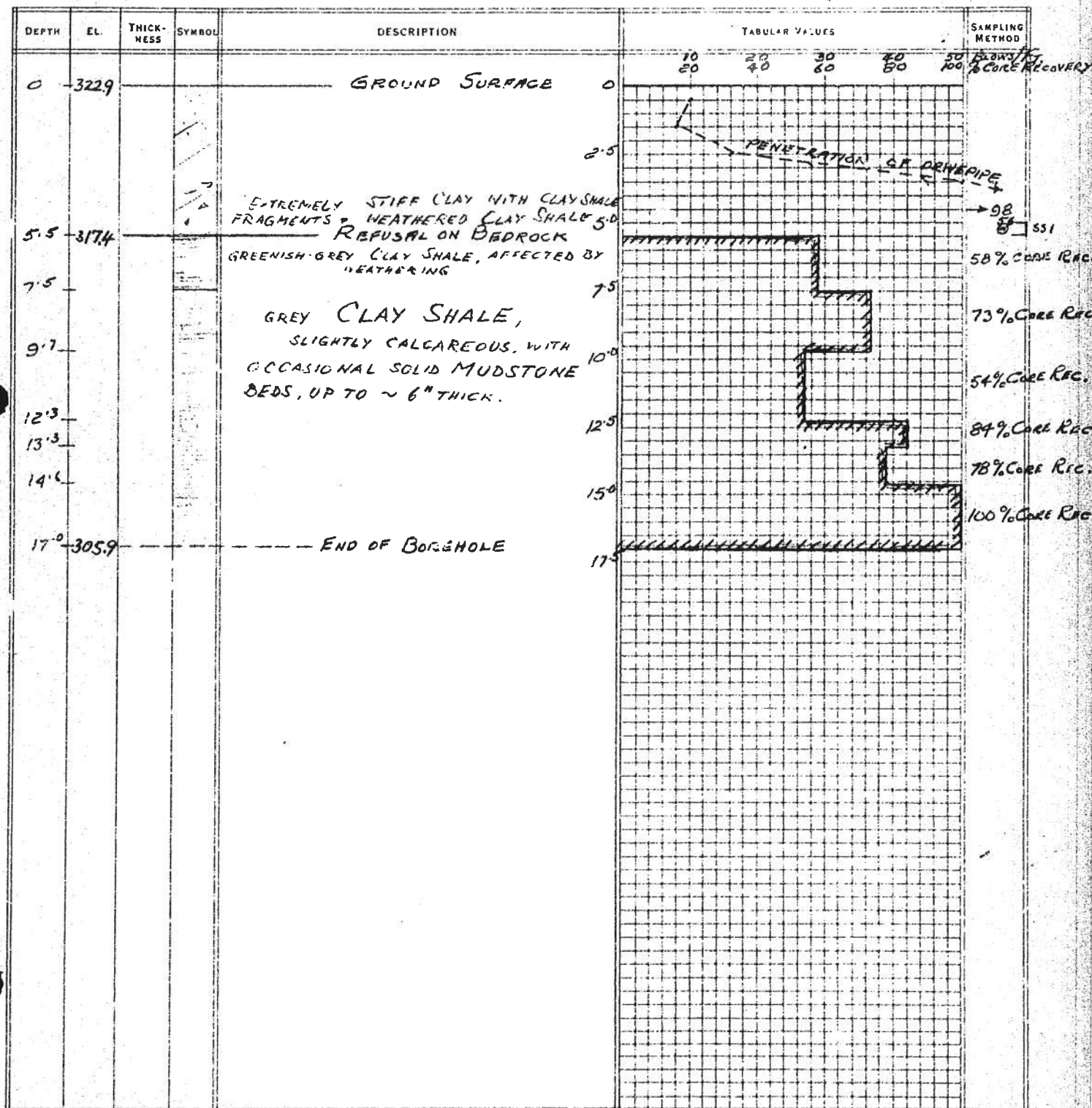
Job Located: ON QUEEN ELIZABETH HWY AT MISSISSAUGA RD.

Checked by

Hole Located: AS SHOWN ON ATTACHED SKETCH PLAN

Hole Elevation: 322.9 Datum: M.S.L.

Day 6/6/55 Month Year



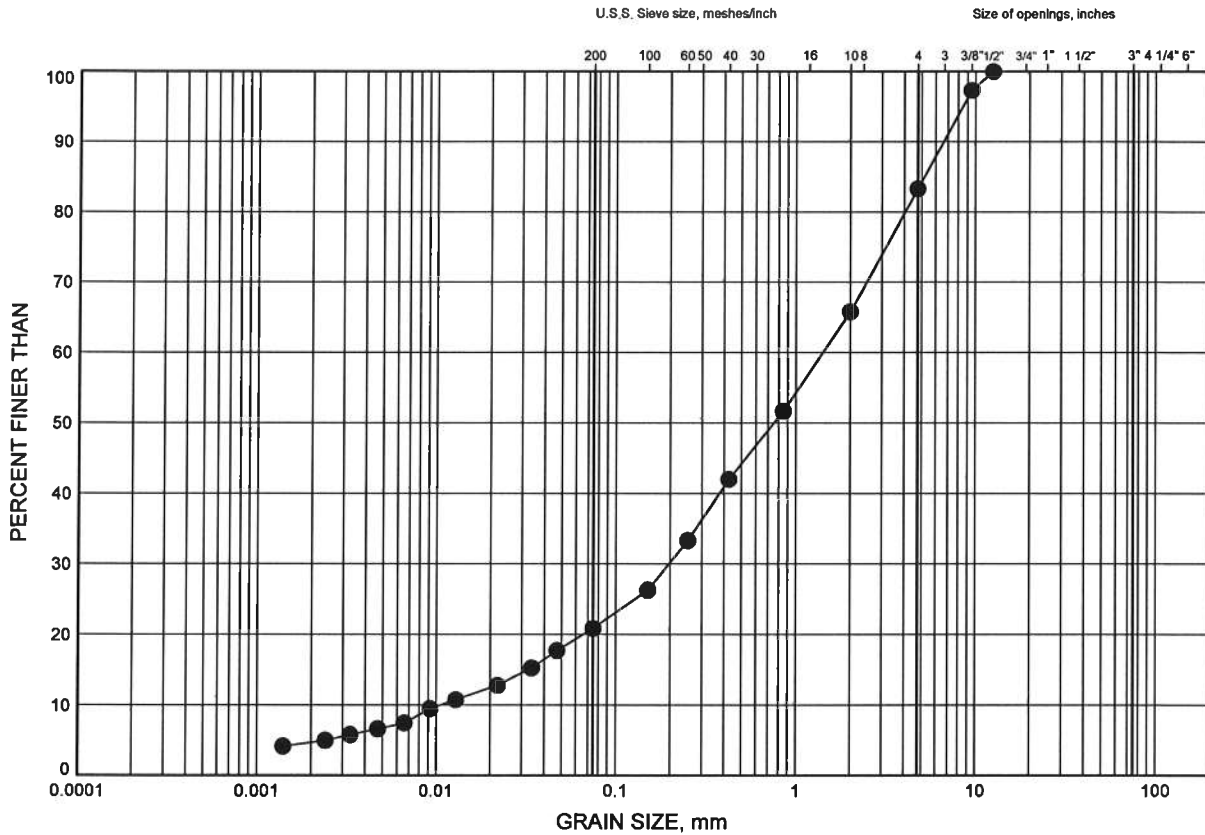
## **Appendix B**

### **Laboratory Test Results**

QEW Mississauga Rd. Overpass  
GRAIN SIZE DISTRIBUTION

FIGURE B1

SAND FILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	MR11-01	0.35	93.98

GRAIN SIZE DISTRIBUTION - THURBER 1174.GPJ 11/18/11

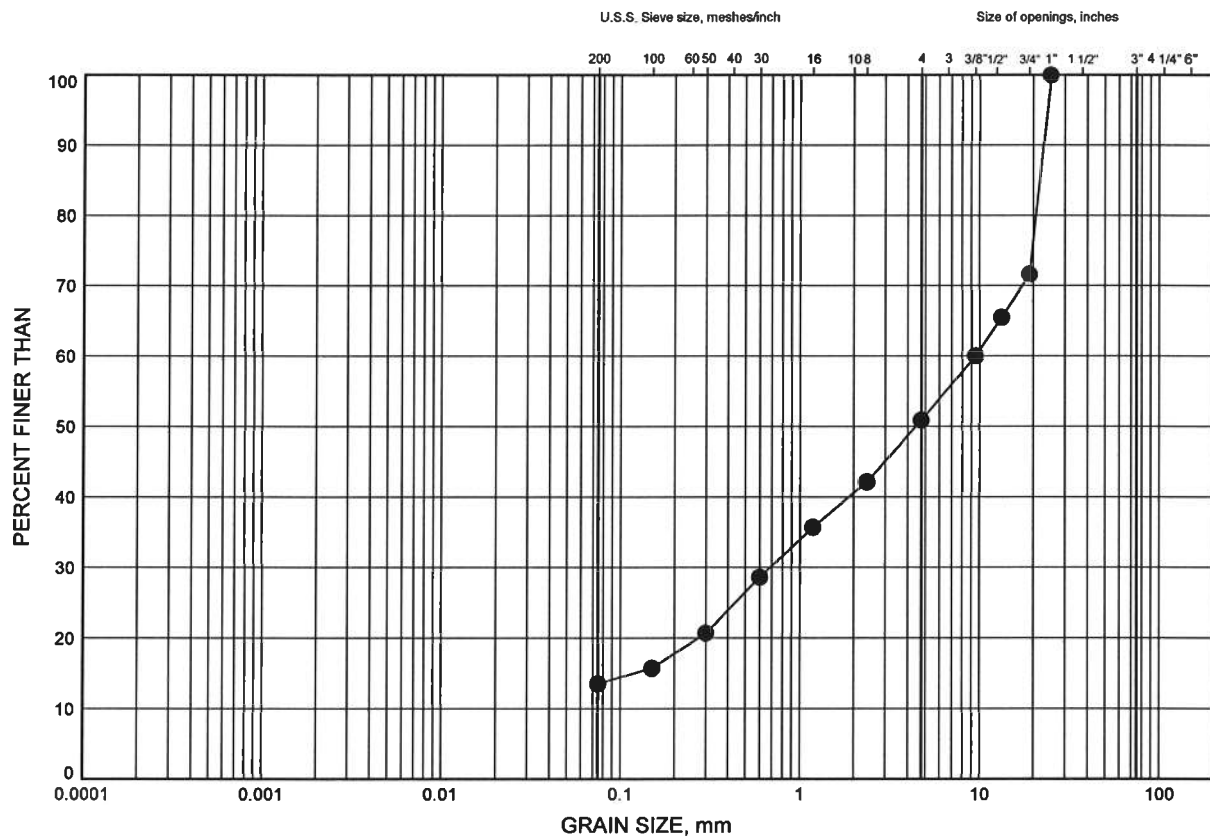
W.P.# W.O. 08-20008...  
Prepared By AN.....  
Checked By SKP.....



QEW Mississauga Rd. Overpass  
GRAIN SIZE DISTRIBUTION

FIGURE B2

SAND FILL, With SHALE Fragments



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	MR11-02	2.57	90.74

GRAIN SIZE DISTRIBUTION - THURBER 1174.GPJ 11/18/11

W.P.# W.O.08-20008  
Prepared By AN  
Checked By SKP



**Table B1**  
**Point Load Test Results**

POINT LOAD TEST SHEET									
Job No :		19-1351-174				Client :		MRC	
Project Name :		Mississauga Road / QEW Bridge				Date Drilled :		17/09/2011	
Core Size :		NQ	BH No :		MR11-01		Date Tested :		19/09/2011
							Tester :		DA
Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (kPa)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	2	3.3	A	2240	47.0	56.2	16.2	shale	Weak
2	2	3.5	D	1560	47.0	72.5	15.6	shale	Weak
3									
4									
5									
6									
7									
8									
9									
10									
11									
12									
13									
14									
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25									
26									
27									
28									
29									
30									
* 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.									

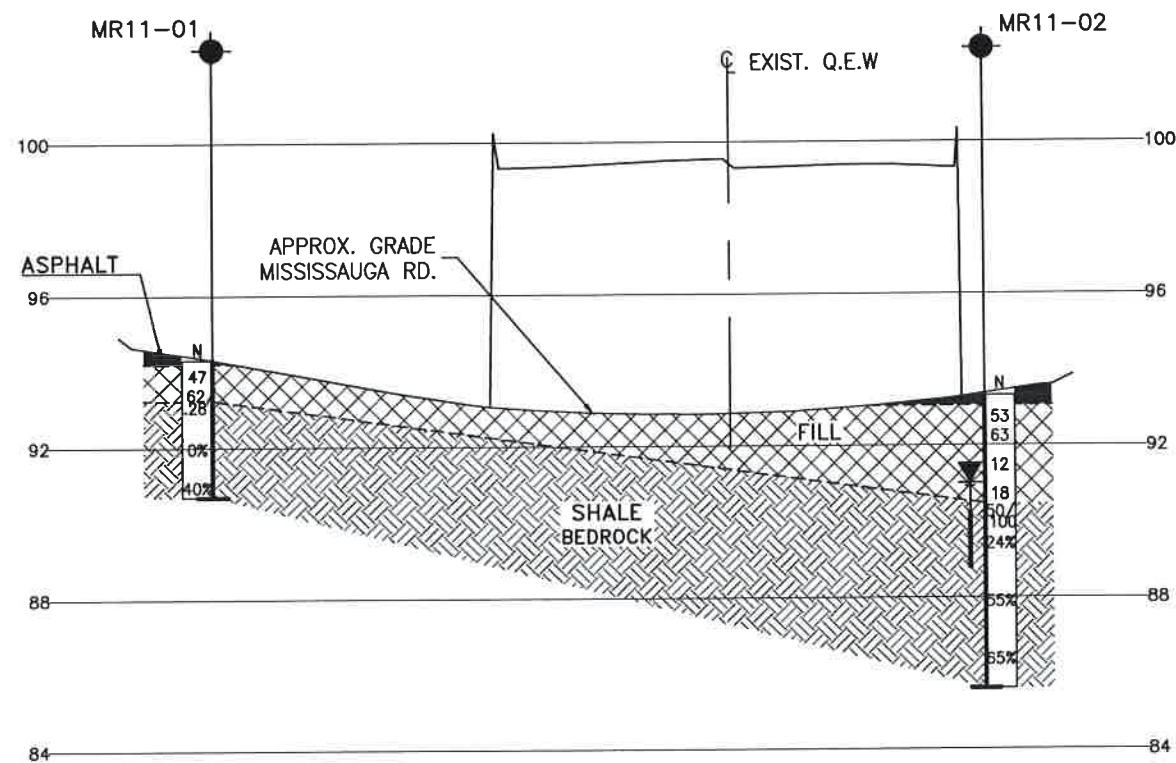
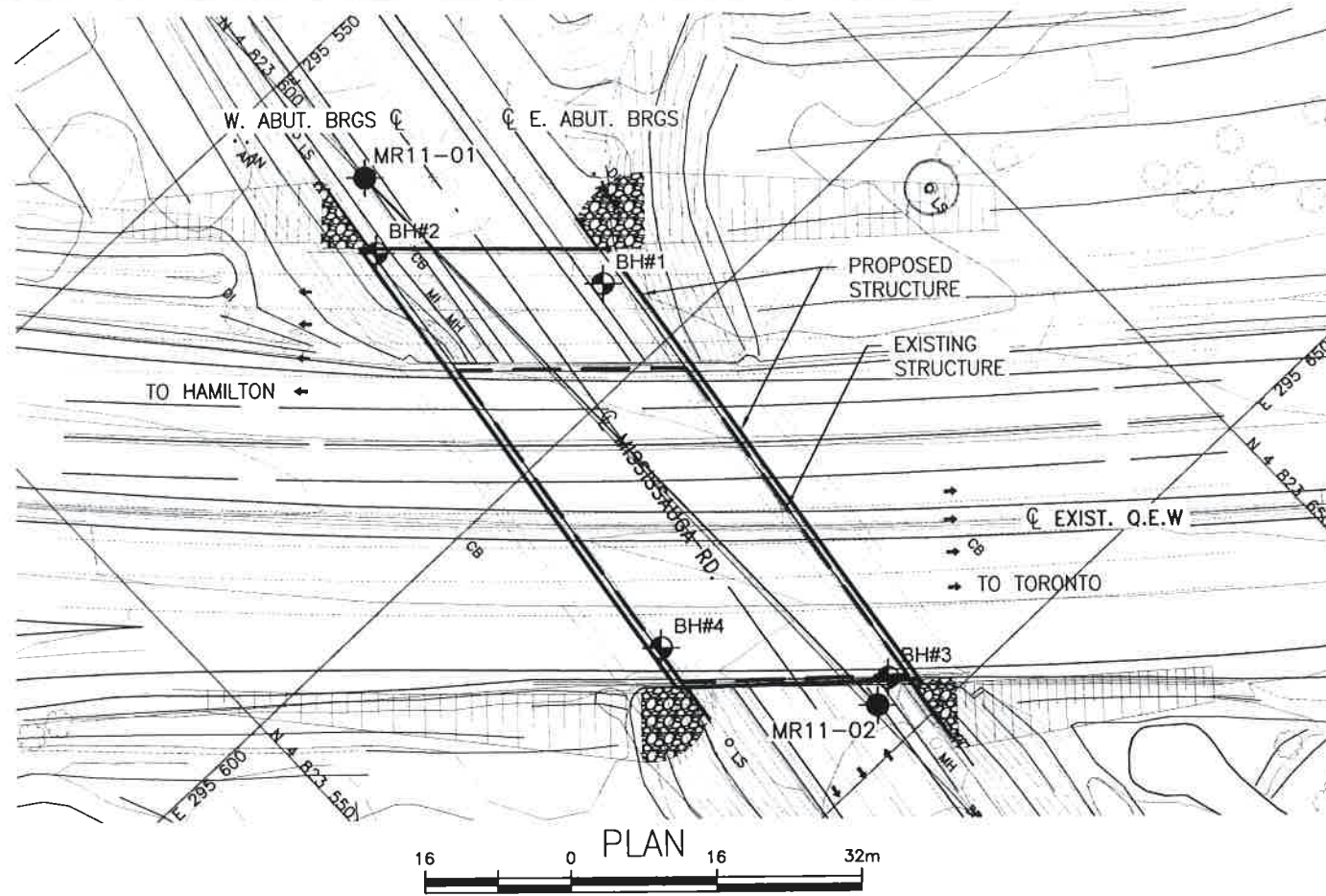
QEW Mississauga Road Overpass  
Mississauga

POINT LOAD TEST SHEET										
Job No :		19-1351-174					Client :		MRC	
Project Name :		Mississauga Road / QEW Bridge					Date Drilled :		17/09/2011	
Core Size :		NQ	BH No :		MR11-02		Date Tested :		19/09/2011	
							Tester :		DA	
Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (kPa)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes	
1	1	3.4	A	780	47.0	39.9	7.4	shale	Weak	
2	1	3.5	D	180	47.0	60.6	1.8	shale	Very Weak	
3	1	3.7	D	480	47.0	76.2	4.8	shale	Very Weak	
4	1	3.9	A	1340	47.0	53.1	10.1	shale	Weak	
5	2	4.8	D	580	47.0	79.9	5.8	shale	Weak	
6	2	5.0	A	2240	47.0	63.4	14.8	shale	Weak	
7	2	5.2	A	980	47.0	58.7	6.8	shale	Weak	
8	2	5.6	D	1480	47.0	65.1	14.8	shale	Weak	
9	2	5.7	D	19000	47.0	89.7	190.3	limestone	Very Strong	
10	2	6.4	A	1500	47.0	55.3	11.0	shale	Weak	
11	2	6.5	D	1400	47.0	80.9	14.0	shale	Weak	
12	2	7.1	D	1640	47.0	70.4	16.4	shale	Weak	
13	2	7.5	A	1740	47.0	59.9	12.0	shale	Weak	
14										
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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.										

## **Appendix C**

### **Drawings**

19-1351-174



PROFILE ALONG Q.E.W. MISSISSAUGA RD.

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

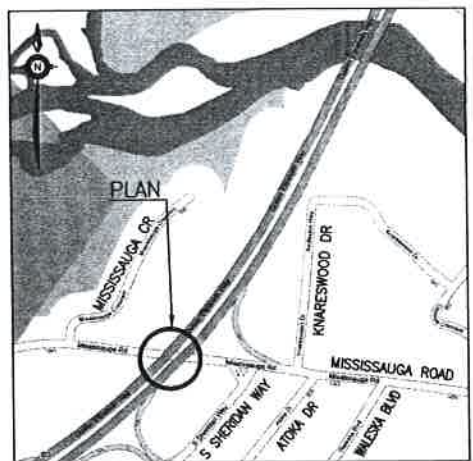


CONT No  
WP No 08-20008

QUEEN ELIZABETH WAY  
MISSISSAUGA ROAD  
INTERCHANGE OVERPASS  
BOREHOLE LOCATIONS AND SOIL STRATA

**MRC** McCORMICK RANKIN  
A MEMBER OF MAM GROUP

**THURBER ENGINEERING LTD.**



**LEGEND**

- ◆ Borehole (Current Investigation)
- ◆ Borehole (Previous Investigation)
- N Blows /0.3m (Std Pen Test, 475J/blow)
- \* Approx. Borehole Location (Survey Data Not Available)
- PH Pressure, Hydraulic
- W Water Level
- HA Head Artesian Water
- PZ Piezometer
- 90% Rock Quality Designation (RQD)
- A/R Auger Refusal

NO	ELEVATION	NORTHING	EASTING
MR11-01	94.3	4 823 600.3	295 564.1
MR11-02	93.3	4 823 601.1	295 645.2
BH#1*		4 823 612.6	295 587.7
BH#2*		4 823 595.3	295 571.1
BH#3*		4 823 603.8	295 643.8
BH#4*		4 823 588.7	295 630.2

**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.

**GEOCRES No. 30M12-342**

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	SKP	CHK	SKP
DRAWN	AN	CHK	
CODE	LOAD	DATE	MAY 2012
SITE	STRUCT	DWG	1

## **Appendix D**

### **Foundation Comparison**

**COMPARISON OF FOUNDATION ALTERNATIVES**

<b>Option 1</b> <b>Spread footing on Shale Bedrock</b>	<b>Option 2</b> <b>Augered H-Piles Socketted into Shale</b>
<b><i>Advantages:</i></b> i. Consistent with foundations of existing bridge. ii. Relative ease of construction. iii. More cost effective than deep foundations.	<b><i>Advantages:</i></b> i. Will develop geotechnical resistance comparable to driven piles to refusal. ii. Installation less influenced by weather and groundwater compared to spread footing iii. Required for integral abutment design
<b><i>Disadvantages:</i></b> i. Potentially up to 2.8 m deep excavation which may require dewatering. ii. Not feasible for integral abutment design.	<b><i>Disadvantages:</i></b> i. Higher unit costs than spread footings. ii. Potential difficulties penetrating hard limestone layers/shale slabs during augering. iv. Space between H-pile and hole sidewall requires grouting with cementitious materials.
Low risk of encountering problems during construction.	Medium risk of encountering harder layers within rock that would require additional procedures to advance the augers to the desired elevation.
<b>RECOMMENDED</b>	<b>FEASIBLE</b>