

**PRELIMINARY
FOUNDATION INVESTIGATION AND DESIGN REPORT
OGDEN PEDESTRIAN BRIDGE REPLACEMENT
QUEEN ELIZABETH WAY, MISSISSAUGA, ONTARIO
G.W.P. 09-20003**

GEOCRES Number: 30M11-253

Report to

MMM Group Limited

Thurber Engineering Ltd.
2010 Winston Park Drive, Suite 103
Oakville, Ontario
L6H 5R7
Phone: (905) 829 8666
Fax: (905) 829 1166

July 13 2015
File: 19-1351-219

H:\19\1351\219 QEW Cawthra Road\Reports & Memos\Ogden Bridge\Final Report\QEW, Ogden Br - FIDR_Final Jul 14 15.doc

TABLE OF CONTENTS

PART 1 FACTUAL INFORMATION

1	INTRODUCTION.....	1
2	SITE DESCRIPTION.....	1
3	SITE INVESTIGATION AND FIELD TESTING	2
4	LABORATORY TESTING	3
5	DESCRIPTION OF SUBSURFACE CONDITIONS	3
5.1	Pavement	4
5.2	Sand Fill.....	4
5.3	Native Sand.....	4
5.4	Silty Clay	5
5.5	Silty Clay with Sand Till	5
5.6	Bedrock.....	6
5.7	Groundwater Conditions.....	7
6	MISCELLANEOUS.....	8

PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

7	GENERAL	9
8	STRUCTURE FOUNDATION.....	9
8.1	Foundation Alternatives.....	10
8.2	Spread Footings on Native Sand.....	10
8.3	Caisson Foundations	11
8.4	Recommended Foundations.....	14
8.5	Frost Cover	14
9	EXCAVATION, BACKFILLING AND GROUNDWATER CONTROL	14
10	CONSTRUCTION CONCERNS	15
11	INVESTIGATION FOR DETAIL DESIGN	15
12	CLOSURE.....	16

Appendices

Appendix A	Record of Borehole Sheets
Appendix B	Laboratory Test Results, Soil and Rock Samples
Appendix C	Site Photographs
Appendix D	Borehole Location Plan
Appendix E	Comparison of Foundation Alternatives

PRELIMINARY
FOUNDATION INVESTIGATION AND DESIGN REPORT
OGDEN PEDESTRIAN BRIDGE REPLACEMENT
QUEEN ELIZABETH WAY, MISSISSAUGA, ONTARIO
G.W.P. 09-20003

GEOCRES Number: 30M11-253

PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a preliminary foundation investigation carried out at the location of the proposed replacement of the Ogden Pedestrian Bridge over the Queen Elizabeth Way (QEW) and the North and South Service Roads in Mississauga, Ontario. This investigation was carried out in support of the preliminary design, environmental assessment and developing alternatives for QEW improvements from Evans Road to Cawthra Road, approximately 3.5 km in length.

The purpose of the investigation was to explore the subsurface conditions at the site and, based on the data obtained, to provide a borehole location plan, records of boreholes, a 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 from the present investigation.

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

2 SITE DESCRIPTION

The existing Ogden Bridge provides pedestrian access over the QEW from the Applewood Mall on the north side to Ogden Avenue on the south side of QEW. The bridge is located approximately 0.9 km east of the intersection of QEW with Cawthra Road Underpass and approximately 1 km west of the Dixie Road Underpass. The existing bridge is a three span structure with a total length of approximately 66 m and 3.6 m in width. The access to the bridge deck is accommodated by ramps and landings on each side of the bridge. The ramp located on the north side is aligned perpendicular to the bridge and parallel to the North Service Road, with the exit/entrance from Insley Road. The ramp on the south side is aligned parallel to the bridge centreline with the access from Ogden Avenue, near the intersection with the South Service Road.

Visual inspection did not reveal any evidence for performance problems in the bridge or ramp foundations.

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

A residential area is immediately adjacent to the south boundary of the site with residences on both sides of Ogden Avenue. On the north side, the bridge is adjacent to Applewood Mall and Insley Road. The Husky Gas Station is located at the corner of North Service Road and Insley Road in close proximity to the bridge. An overhead hydro line crosses the QEW on the west side of the bridge. The land surrounding the site is sparsely vegetated on the north side; grass, brush and trees are present on the south side.

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

From published geological information, the site is situated within the physiographic region known as the Iroquois Plain. In this area, the soil deposits are relatively thin and typically consist of glaciolacustrine deposits overlying shale bedrock of the Georgian Bay Formation.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project was carried out between December 4 and 10, 2014. The investigation consisted of drilling and sampling of three boreholes denoted as Borehole PB14-01A, PB14-01B and PB14-02. Boreholes PB14-01A and PB14-01B were placed on the north side of the bridge and drilled from the surface of North Service Road. Borehole PB14-02 was drilled on the south side of the bridge, near the existing pedestrian ramps. Borehole PB14-01A was advanced to 7.3 m depth and was terminated without reaching the required depth due to mechanical difficulties with the coring equipment. Borehole PB14-01B was placed approximately 1.5 m to the west of the PB14-01A and advanced through the overburden without sampling, and then cored to recover 3.6 m of bedrock samples.

The borehole locations were marked in the field by Thurber. Utility clearance was obtained for the borehole locations prior to drilling. Borehole location data including northing, easting and surface elevation has been derived based on the preliminary design information provided by MMM Group to Thurber. The approximate borehole locations are shown on the Borehole Locations and Soil Strata drawing enclosed in Appendix D.

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

Groundwater conditions in the open boreholes were observed throughout the drilling operations in soils. Standpipe piezometers were installed in Borehole PB14-01B and PB14-02 to permit monitoring of the groundwater levels. The standpipe piezometer typically consists of 19 mm

diameter Schedule 40 PVC pipes with 1.5 m long slotted screen positioned in the soil strata where groundwater fluctuations are to be monitored. A sand screen surrounded the pipe and extended at least 0.3 m above the slotted screen. Bentonite holeplug seals were placed above the sand screen in each installation to seal the annular space. Following the final water level reading, the piezometers will be decommissioned in general accordance with MOE Regulation 903.

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

Table 3.1 Borehole completion and Piezometer Installation Details

Borehole Number	Ground Elevation	Borehole Termination Depth/Elevation (m)	Borehole Completion Details
PB14-01A	103.8	7.3 / 96.5	Backfilled with bentonite holeplug and cuttings to 0.9 m, concrete to 0.2 m, then asphalt patch to surface.
PB14-01B	103.8	9.4 / 94.4	19 mm diameter piezometer with filter sand from 9.4 to 7.3 m, bentonite holeplug from 7.3 to 0.9 m, concrete to 0.2 m, then asphalt patch to surface.
PB14-02	103.3	9.3 / 94.0	19 mm diameter piezometer with filter sand from 9.3 to 5.8 m, bentonite holeplug from 5.8 to 0.3 m, concrete to 0.15 m, then asphalt patch to surface.

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

4 LABORATORY TESTING

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

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

5 DESCRIPTION OF SUBSURFACE CONDITIONS

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

A general description of the subsurface conditions encountered in the current investigation is given

in the following paragraphs. The factual information established at the borehole locations governs any interpretation of the site conditions.

In general, boreholes drilled at this site encountered pavement structure overlying sand fill and native deposits of sand, silty clay and silty clay till extending to shale bedrock. A grey shale bedrock of Georgian Bay Formation was encountered at 4.9 m depth (Elev. 98.6) on the north side and at 6.1 m depth (Elev. 97.2) on the south side of the bridge.

5.1 Pavement

Borehole PB14-01A was placed on the North Service Road and encountered 175 mm of asphalt.

Borehole PB-02 was placed near the access ramp and pedestrian walkway on the south side and encountered 75 mm of brick pavement underlain by 125 mm of crush stone screening layer.

5.2 Sand Fill

The fill material comprising brown sand with trace to some gravel, some silt and trace clay underlies the asphalt and brick pavement in the boreholes drilled. Even though not encountered during drilling, cobbles or boulders should be expected in the fill materials. The fill was approximately 1.3 m in thickness with the base at 1.5 m or at Elev. 102.3 to 101.8 in Boreholes PB14-01A and PB14-02, respectively.

Standard Penetration Tests (SPT) conducted within this fill produced 'N' values ranging from 14 to 35 blows per 0.3 m of penetration indicating a compact to dense relative density. The measured natural moisture content of the fill samples ranged from 5% to 24%.

Results of grain size analyses conducted on selected samples of sand fill are presented on the Record of Borehole sheets, on Figure B1 in Appendix B, and are summarized below:

Soil Particles	Percentage (%)
Gravel	0
Sand	72 to 77
Silt	16 to 20
Clay	7 to 8

5.3 Native Sand

The sand fill is underlain by a layer of native brown sand comprising some silt and trace clay. The sand layer is 0.7 m to 1.5 m in thickness with the lower boundary at a depth of 2.2 m (Elev. 101.6) in Borehole PB14-01A, and 3.0 m (Elev. 100.3) in Borehole PB14-02.

Standard Penetration Test (SPT) conducted within the native sand produced 'N' values

ranging from 14 to 24 blows per 0.3 m of penetration indicating a compact relative density. The measured natural moisture content of the sand samples ranged from 18% to 24%.

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

Soil Particles	Percentage (%)
Gravel	0
Sand	86
Silt	11
Clay	3

5.4 Silty Clay

A layer of grey silty clay underlies the sand in the boreholes. This silty clay is probably of glaciolacustrine origin. The deposit was between 0.8 m and 0.9 m thick with the lower boundary encountered at 3.0 m (Elev. 100.8) and 3.9 m depth (Elev. 99.4) in Boreholes PB14-01A and PB14-02, respectively.

SPT 'N' values of 14 and 15 blows per 0.3 m penetration were recorded in the silty clay indicating stiff consistency of the deposit.

The results of grain size analyses conducted on a sample of the silty clay indicate the following fraction contents:

Soil Particles	Percentage (%)
Gravel	0
Sand	6
Silt	45
Clay	49

The results are provided on the Record of Borehole sheets in Appendix A, and illustrated in Figure B3 of Appendix B.

The results of Atterberg Limits test conducted on a sample of the silty clay are provided on the Record of Borehole sheet in Appendix A and are illustrated in Figure B5 of Appendix B. The results indicated that the deposit has liquid limit of 32% and plasticity index of 15%, suggesting low plasticity of the deposit. The measured natural moisture content of the clay samples were 19% and 20%.

5.5 Silty Clay with Sand Till

A layer of till comprising of grey silty clay with sand underlies the silty clay in the

boreholes. Even though not observed during drilling, gravel, cobbles, boulders and fragments of underlying shale and limestone bedrock should be expected in the till deposit. The till deposit was 1.9 m and 2.2 m thick with the lower boundary encountered between a depth of 4.9 m (Elev. 98.9) and 6.1 m (Elev. 97.2) in Boreholes PB14-01A and PB14-02, respectively, and was underlain by shale bedrock.

SPT 'N' values between 25 and 54 blows per 0.3 m penetration were recorded in the till indicating very stiff to hard consistency of the deposit.

The results of Atterberg Limits test conducted on a sample of the silty clay till are provided on the Record of Borehole sheet in Appendix A and are illustrated in Figure B6 of Appendix B. The results indicated that the deposit has liquid limit of 27% and plasticity index of 16%, suggesting low plasticity of the deposit. The measured natural moisture content of the till samples ranged from 10% to 11%.

Results of grain size analyses conducted on selected samples of the till are presented on the Record of Borehole sheets, on Figure B4 in Appendix B, and are summarized below:

Soil Particles	Percentage (%)
Gravel	0
Sand	38 to 39
Silt	36
Clay	25 to 26

5.6 Bedrock

Shale bedrock of the Georgian Bay Formation was encountered below the silty clay till at depths of 4.9 m and 6.1 m (Elev. 98.9 and 97.2) in Boreholes PB14-01A and PB14-02, respectively. Borehole PB14-01B was advanced without sampling to bedrock surface at 4.9 m depth and augered to 5.8 m depth (Elev. 98.0), below which, bedrock was cored to 9.4 m depth (Elev. 94.4). Bedrock was proved by coring between 2.4 m and 3.6 m length in the boreholes. The bedrock was described as moderately weathered to fresh, thinly bedded, weak shale bedrock with occasional strong to very strong limestone interbeds. The limestone interbeds were typically 25 mm to 50 mm thick, however, a limestone layer as thick as 200 mm was encountered at 8.4 m depth in Borehole PB14-01B. Occasional clay seams were noted at various depths in the shale bedrock.

Total Core Recovery (TCR) of the bedrock was 100% in all runs. The Solid Core Recovery (SCR) ranged from 40% to 100%. The Rock Quality Designation (RQD) values ranged typically from 17% to 100% indicating very poor to excellent rock quality. The Fracture Index (FI) indicating a number of natural fractures per 0.3 m of core run, was in the upper 1.5 m to 1.8 m generally between 6 to more than 10. Below that depths, the FI

ranged from 0 to 6.

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

Table 5.1 Depths and Elevations of Bedrock Surface

Borehole	Depth of Bedrock below Ground Surface (m)	Bedrock Elevation (m)
PB14-01A	4.9	98.9
PB14-01B	4.9 ⁽¹⁾	98.9 ⁽¹⁾
PB14-02	6.1	97.2

(1) Depths and elevations of bedrock encountered during drilling. Borehole advanced by coring from 5.8 m depth.

The unconfined compressive strengths (UCS) of the intact shale cores estimated as average value for each run from the results of point load tests (PLT), ranged from 10 MPa to 15 MPa, indicating weak intact rock. The USC values of 86 MPa to 152 MPa were obtained for the samples of the limestone interbeds. The average UCS values for each rock core are included on the Record of Borehole sheets. The results of point load testing are enclosed in Appendix B.

5.7 Groundwater Conditions

Groundwater was observed in the boreholes upon completion of drilling operations. Water was used during drilling and therefore the measured water levels upon completion of drilling may not reflect prevailing water level at this site. Standpipe piezometers were installed in bedrock in Borehole PB14-01B and PB14-04.

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

Table 5.2 Water Level Measurements

Borehole Number	Date	Water Level in Open Borehole (m)		Groundwater Level in Piezometer (m)	
		Depth	Elevation	Depth	Elevation
PB14-01A	Dec. 4, 2014	3.7	100.1	-	-
PB14-01B	Dec.9, 2014	2.2	101.6	-	-
	Jan. 15, 2015			2.8	101.0
PB14-02	Dec. 10, 2014	2.5	100.8	-	-
	Jan. 15, 2015	-	-	2.1	101.2

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

6 MISCELLANEOUS

The drilling and sampling equipment was supplied and operated by Walker Drilling Ltd. of Barrie, Ontario, who supplied a rubber track-mounted Acker drill rig for the duration of the investigation. Traffic protection during the drilling operation was provided by Thurber Engineering.

The field work was supervised on a full time basis by Mr. George Azzopardi of Thurber Engineering Ltd. Overall supervision of the field program was conducted by Weiss Mehdawi, P.Eng.

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

Thurber Engineering Ltd.

Data Compilation and Report Preparation by:
Anna Piascik, P.Eng
Senior Geotechnical Engineer



Report Reviewed by:
Alastair E. Gorman, P.Eng.
Associate, Senior Geotechnical Engineer



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



**PRELIMINARY
FOUNDATION INVESTIGATION AND DESIGN REPORT
OGDEN PEDESTRIAN BRIDGE REPLACEMENT
QUEEN ELIZABETH WAY, MISSISSAUGA, ONTARIO
G.W.P. 09-20003**

GEOCRES Number: 30M11-253

PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 GENERAL

This report provides an interpretation of the geotechnical data in the factual report and presents preliminary foundation design recommendations to assist the design team in selection and design of a suitable foundation system for the replacement of the Ogden Pedestrian Bridge and associated ramps.

The existing bridge is a three span structure with a total length of approximately 66 m and 3.6 m width. The access to the bridge deck is accommodated by ramps and landings on each side of the bridge.

Based on the preliminary General Arrangement Drawings (GA) dated December 2014 provided by MMM Group, it is understood that the replacement structure will be located on the same alignment as the existing bridge with structural pedestrian access ramps aligned parallel to the bridge centreline. The replacement bridge will be a three span structure with spans 14 m, 53.9 m and 11.0 m. The north access ramps will have 11.3 m and 11.5 m spans. A single span of 23 m in length is proposed for the south access ramp. The Preliminary General Arrangement drawing indicates that the proposed bridge and access ramps will be supported on spread footings.

The discussion and recommendations presented in this report are based on the information provided by MMM Group and on the factual data obtained during the course of the current investigation.

8 STRUCTURE FOUNDATION

In general, boreholes drilled at this site encountered pavement structure overlying sand fill and native deposits of sand, silty clay and silty clay with sand till extending to shale bedrock. A grey shale bedrock of Georgian Bay Formation was encountered at 4.9 m depth (Elev. 98.9) on the north side and at 6.1 m depth (Elev. 97.2) on the south side of the bridge.

The groundwater levels in the standpipe piezometers were measured at 2.8 m depth (Elev. 101.0) on the north side and at 2.1 m (Elev. 101.2) on the south side of the bridge. The groundwater level

at the bridge site will fluctuate seasonally.

8.1 Foundation Alternatives

Based on subsurface conditions encountered in the boreholes at this site, both shallow and deep foundation options are considered feasible for the support of the replacement structure. Consideration was given to the following foundation types to support the new piers:

- Spread footings founded on native sand deposit, and
- Caissons extended to shale bedrock or sealed/socketed into bedrock.

Spread footings constructed on native compact sand could offer cost effectiveness and relatively easy construction. Spread footing placed on native sand would require approximately 1.5 m deep excavation and, generally, will be located above the groundwater level.

Caissons advanced to bedrock surface or sealed into bedrock could be considered for this site as they would offer constructability advantage at the bridge piers.

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

Recommendations for design of the feasible foundation alternatives are presented in the following sections together with the corresponding geotechnical design parameters.

A preferred foundation scheme from a geotechnical perspective is recommended.

8.2 Spread Footings on Native Sand

Based on subsurface conditions encountered at this site, the use of spread footings placed on native compact sand to support the piers is considered feasible.

For preliminary design of spread footings to support the bridge piers the following founding levels could be assumed:

North Piers – Elevation 102.3
South Piers – Elevation 101.8.

The above founding elevations are provided for the planning/preliminary design; the actual founding elevations at specific foundation unit locations will have to be determined during detailed design stage.

For preliminary design the following values of factored Geotechnical Resistance at ULS and Geotechnical Reaction may be used:

	Footing Width (m)	
	1.0	1.5
Factored Geotechnical Resistance at ULS (kPa)	270	300
Geotechnical Reaction at SLS (kPa)	180	200

The geotechnical resistance at SLS quoted above corresponds to 25 mm of settlement of an individual footing and are for concentric, vertical loads only. In the case of eccentric or inclined loading, the geotechnical resistance should be calculated as illustrated in the CHBDC 2006 Clause 6.7.3 and Clause 6.7.4.

Resistance to lateral forces / sliding resistance between the footing concrete and the sand at the founding level should be evaluated in accordance with the CHBDC, 2006 assuming an ultimate coefficient of friction of 0.4.

The exposed sand at the founding level should be protected from disturbance due to construction traffic.

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

8.3 Caisson Foundations

Augered caisson foundations advanced to the top of bedrock could be designed for end-bearing, provided the bedrock surface is cleaned. The bedrock surface indicated in Table 5.1, i.e. Elevation 98.9 at the north end and Elevation 97.2 at the south end, may be used for the preliminary design of caissons, if this option is selected.

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

Higher capacity could be obtained for caissons socketed into shale bedrock. The factored axial geotechnical resistances for 0.9 m and 1.2 m diameter caissons associated with the following minimum socket depths within bedrock may be used in design.

Factored Axial Geotechnical Resistance for Caisson Foundations

Caisson Diameter (m)	Minimum Socket Depth below Bedrock Surface (m)	Factored Geotechnical Resistance at ULS (kN)
0.9	2.0	2,600
	2.5	3,300
1.2	2.0	3,500
	2.5	4,300

The SLS condition will not govern for caissons founded on rock.

The geotechnical lateral resistance of a caisson in soils may be calculated using values of the coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) presented below.

The lateral resistance acting on a caisson in cohesionless soils may be calculated using the following correlations:

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

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

where z = depth of embedment of caisson (m)
 D = caisson width or diameter (m)
 n_h = coefficient of horizontal subgrade reaction (kN/m^3)
 γ' = effective unit weight (kN/m^3)
 K_p = passive earth pressure coefficient

The geotechnical lateral resistance acting on a caisson in cohesive soils may be calculated using correlations, as follows:

$$k_s = 67 s_u / D \quad (\text{kN/m}^3)$$

$$p_{ult} = 9 s_u \quad (\text{kPa})$$

where s_u = undrained shear strength (kPa)
 D = caisson width or diameter in metres

The above correlations and parameters presented in Table 8.1, below, may be used to analyse the interaction between a caisson and the surrounding soil. The lateral pressures obtained from the analysis should not exceed the ultimate lateral resistance.

The spring constant, K_s , for analysis may be obtained from the expression, $K_s = k_s L D$ (kN/m), where k_s is the coefficient of horizontal subgrade reaction (kN/m^3), D is the caisson width (m) and L is the length (m) of the caisson segment or element used in the analysis.

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

Table 8.1 – Soil Parameters for Lateral Resistance Estimate

Foundation Element	Soil Strata	Elevation (m)		$\gamma^{(2)}$ (kN/m ³)	n_h (kN/m ³)	K_p	S_u (kPa)
		Top	Bottom				
North Piers	Sand Fill	GS ¹⁾	102.3	20	5,000	3.0	-
	Native Sand (above GWT)	102.3	101.6	20	5,000	3.3	-
	Silty Clay	101.6	100.8	17	-	-	100
	Silty Clay and Sand Till	100.8	98.9	18	-	-	200
South Piers	Sand Fill	GS ¹⁾	101.8	20	5,000	3.0	-
	Native Sand (above GWT)	101.8	101.2 ³⁾	20	5,000	3.3	-
	Native Sand (below GWT)	101.2 ³⁾	100.3	20	3,000	3.3	-
	Silty Clay	100.3	99.4	17	-	-	100
	Silty Clay with Sand Till	99.4	97.2	18	-	-	200

Notes: ¹⁾ GS – ground surface or elevations of the proposed roadway grade.
²⁾ Total unit weight; the groundwater level should be considered, where applicable, to obtain effective unit weight.
³⁾ Groundwater levels measured during this investigation; the levels to be confirmed during detailed design.

The lateral resistance of caissons can be supplemented, if required, by socketing into shale bedrock. The lateral resistance that can be mobilized in front of a pile socket in shale, assuming a clear spacing of at least one socket diameter between the sockets, may be determined using the coefficient of horizontal subgrade reaction k_s and ultimate lateral resistance p_{ult} estimated as follows:

$k_s =$ 10 MN/m³ at the bedrock surface, increasing linearly to
100 MN/m³ at a depth of 3 caisson socket diameters and below.

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

The structural designer should determine a required depth of socket to provide base fixity.

Caisson installation should be in accordance with OPSS 903. The caisson installation equipment should be capable of dislodging and removing any obstructions such as cobbles, boulders and rock slabs in the soil deposit. Layers of limestone in the shale bedrock may require the use of coring or rock breaking equipment in addition to the auger equipment, if caissons are to be socketed in the bedrock.

Temporary steel liners will be required to support the caisson sidewalls and to minimize groundwater inflow. The caisson excavation should be dewatered to allow cleaning of the base and walls prior to placing concrete. Concrete should be placed with minimum delay after the socket is drilled, cleaned, inspected and approved.

8.4 Recommended Foundations

Spread footings founded on the native sand deposit is the preferred option for the bridge foundations, considering the cost effectiveness and loading requirements.

From the geotechnical point of view, caisson foundations are also feasible for this bridge. They could offer constructability advantage when working at or near the operational highway.

A comparison of the foundation options is presented in Appendix E.

8.5 Frost Cover

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

9 EXCAVATION, BACKFILLING AND GROUNDWATER CONTROL

All excavations and backfilling should be carried out in accordance with OPSS 902 and the requirements of the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the existing fill and native sand within the depth of excavation may be classified as Type 3 soils above the water table and Type 4 soils below the water table. If excavations extend below the sand base, the encountered silty clay and till deposits could be classified as Type 2 soils. Flatter slopes may be required at locations where water seepage affects stability of an excavation.

Excavations for the proposed spread footings construction are expected to extend through the general and roadway fill to reach the native sand. The groundwater levels as measured in the piezometers were below the recommended founding levels at the time of this investigation. The groundwater levels are expected to fluctuate seasonally and after severe weather events. Moreover, perched water may be present in the fill at shallow depth. Some water control, such as pumping from sumps, may be required.

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

Roadway protection, if required, should be provided in accordance with OPSS 539 and designed for Performance Level 2.

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

10 CONSTRUCTION CONCERNS

Based on the information available at this time (Preliminary Design) potential construction concerns include, but are not necessarily limited to the excavation and dewatering for the spread footings, namely:

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

11 INVESTIGATION FOR DETAIL DESIGN

During the detailed design phase of this project, additional site investigations and field testing will be required. The existing subsurface information should be reviewed and supplemented to comply with the MTO Foundation Engineering Terms of Reference.

It is anticipated that additional boreholes will be required. These boreholes should sample the overburden and bedrock and should include piezometers or observation wells to monitor the groundwater level.

12 CLOSURE

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

Thurber Engineering Ltd.

Anna M. Piascik, P.Eng.
Senior Geotechnical Engineer



Alastair E. Gorman, P.Eng.
Associate, Senior Foundation Engineer



P.K. Chatterji, P.Eng.
MTO Review Principal



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.

EXPLANATION OF ROCK LOGGING TERMS


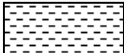



ROCK WEATHERING CLASSIFICATION

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

DISCONTINUITY SPACING

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

SYMBOLS

	CLAYSTONE
	SILTSTONE
	SANDSTONE
	COAL
	BEDROCK

STRENGTH CLASSIFICATION

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

TERMS

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

UNIFIED SOILS CLASSIFICATION

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

RECORD OF BOREHOLE No PB 14-01A

1 OF 1

METRIC

W.P. 09-20003 LOCATION Ogden Pedestrian Bridge N 4 827 308.2 E 614 898.9 ORIGINATED BY GA
 HWY QEW BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2014.12.04 - 2014.12.04 CHECKED BY AMP

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					
103.8	GROUND SURFACE												
0.0	ASPHALT: (175mm)												
0.2	SAND, some silt, trace clay, occasional gravel Compact to Dense Brown Dry (FILL)		1	SS	35								
			2	SS	14								
102.3													
1.5	SAND, some silt, trace clay Compact Brown Wet		3	SS	20								
101.6													
2.2	Silty CLAY, trace sand Stiff Grey Wet		4	SS	15								
100.8													
3.0	Silty CLAY with SAND, trace gravel Very Stiff to Hard Grey Wet (TILL)		5	SS	25								
			6	SS	54								
98.9													
4.9	SHALE BEDROCK, slightly weathered, thinly bedded, very weak, strong to very strong limestone interbeds, grey (Georgian Bay Formation)												
	Very strong limestone interbeds (50mm) at 6.5m and 6.7m		1	RUN									
	Horizontal joint (25mm) at 7.2m												
	Clay seam (25mm) at 7.2m		2	RUN									
96.5													
7.3	END OF BOREHOLE AT 7.3m. BOREHOLE OPEN TO 7.3m AND WATER LEVEL AT 3.7m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND CUTTINGS TO 0.9m, CONCRETE TO 0.2m, THEN ASPHALT PATCH TO SURFACE.												

ONTMT4S 1219.GPJ 2012TEMPLATE(MTO).GDT 1/27/15

+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No PB 14-01B

1 OF 2

METRIC

W.P. 09-20003 LOCATION Ogden Pedestrian Bridge N 4 827 308.2 E 614 898.9 ORIGINATED BY GA
 HWY QEW BOREHOLE TYPE NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2014.12.09 - 2014.12.09 CHECKED BY AMP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa			W _p	W	W _L			
								○ UNCONFINED + FIELD VANE								
								● QUICK TRIAXIAL × LAB VANE								
103.8	GROUND SURFACE							20	40	60	80	100				
0.0	Augered to 5.8m, then start coring For soil stratigraphy refer to Borehole PB14-01A							20	40	60	80	100				

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

20
15
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No PB 14-01B

2 OF 2

METRIC

W.P. 09-20003 LOCATION Ogden Pedestrian Bridge N 4 827 308.2 E 614 898.9 ORIGINATED BY GA
 HWY QEW BOREHOLE TYPE NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2014.12.09 - 2014.12.09 CHECKED BY AMP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
							20	40	60	80	100	W _p	W	W _L			
	Continued From Previous Page 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) 2015.01.15 2.8 101.0																

RECORD OF BOREHOLE No PB 14-02

1 OF 2

METRIC

W.P. 09-20003 LOCATION Ogden Pedestrian Bridge N 4 827 257.6 E 614 964.2 ORIGINATED BY GA
 HWY QEW BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2014.12.10 - 2014.12.10 CHECKED BY AMP

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					
103.3	GROUND SURFACE												
0.0	BRICK: (75mm)												
0.1													
0.2	Crushed Stone: (125mm)		1	SS	16		103						
	SAND, trace gravel, some silt, trace clay Compact Dark Brown Dry (FILL)		2	SS	14		102						0 72 20 8
101.8													
1.5	SAND, some silt, trace clay Compact Brown Wet		3	SS	14		101						
			4	SS	24		100						0 86 11 3
100.3													
3.0	Silty CLAY, trace sand Stiff Grey Wet		5	SS	14		100						
99.4													
3.9	Silty CLAY with SAND, trace gravel Hard Grey Moist (TILL)		6	SS	34		99						0 38 36 26
							98						
97.2			7	SS	50/		97						
6.1	SHALE BEDROCK, slightly weathered to fresh, thinly bedded, weak, occasional very strong limestone interbeds, grey (Georgian Bay Formation) Limestone interbeds (50mm) at 6.7m, 6.8m and 7.1m Horizontal joints from 6.3m to 7.7m Clay seam (25mm) at 7.7m Limestone interbeds at 7.9m, 8.3m, 8.5m, 8.6m Horizontal joints from 7.8m to 8.9m		1	RUN	0.075		96						RUN #1 TCR=100% SCR=57% RQD=32% UCS=9.7MPa Shale UCS=152MPa Limestone
			2	RUN			95						RUN #2 TCR=100% SCR=97% RQD=77% UCS=12.4MPa Shale
94.0													
9.3	END OF BOREHOLE AT 9.3m. BOREHOLE OPEN TO 9.3m AND WATER LEVEL AT 2.5m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe												

Continued Next Page

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

METRIC

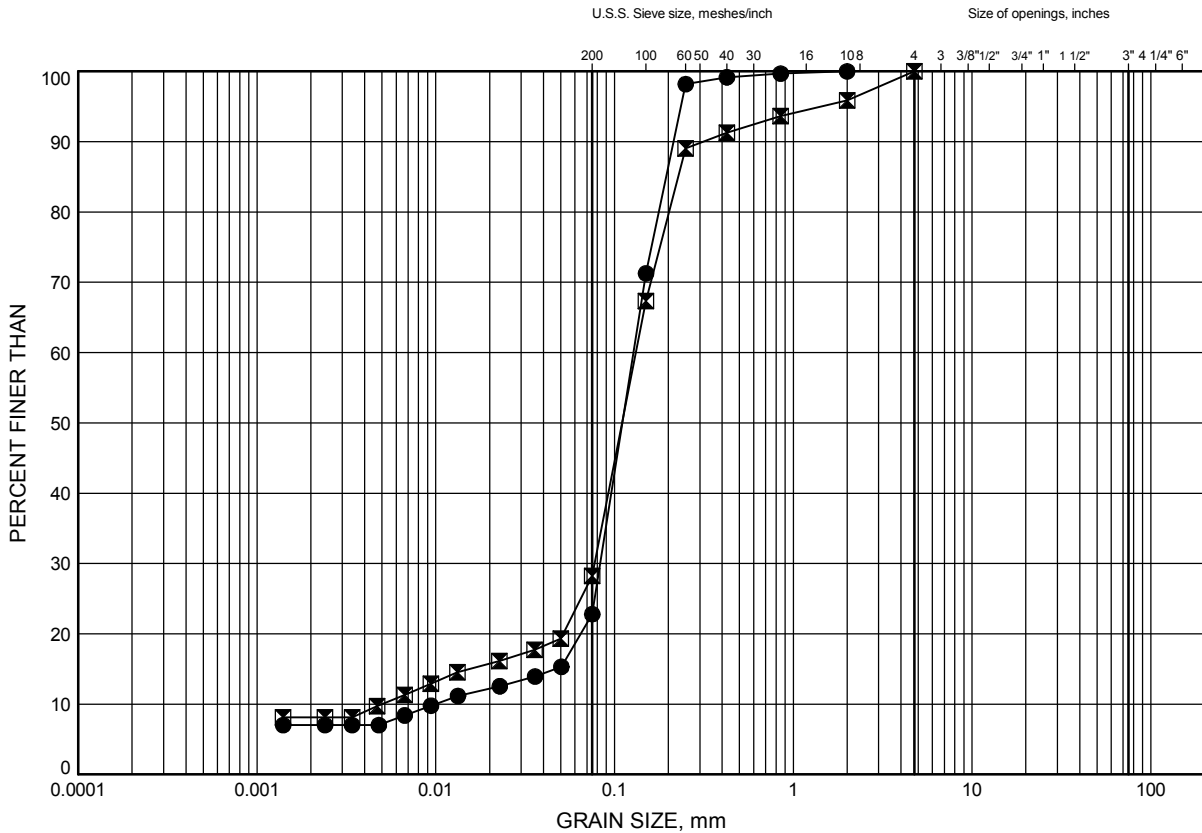
[illegible]

Appendix B
Laboratory Test Results
Soil and Rock Samples

QEW Cawthra Road 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)
●	PB 14-01A	1.07	102.73
⊠	PB 14-02	1.07	102.23

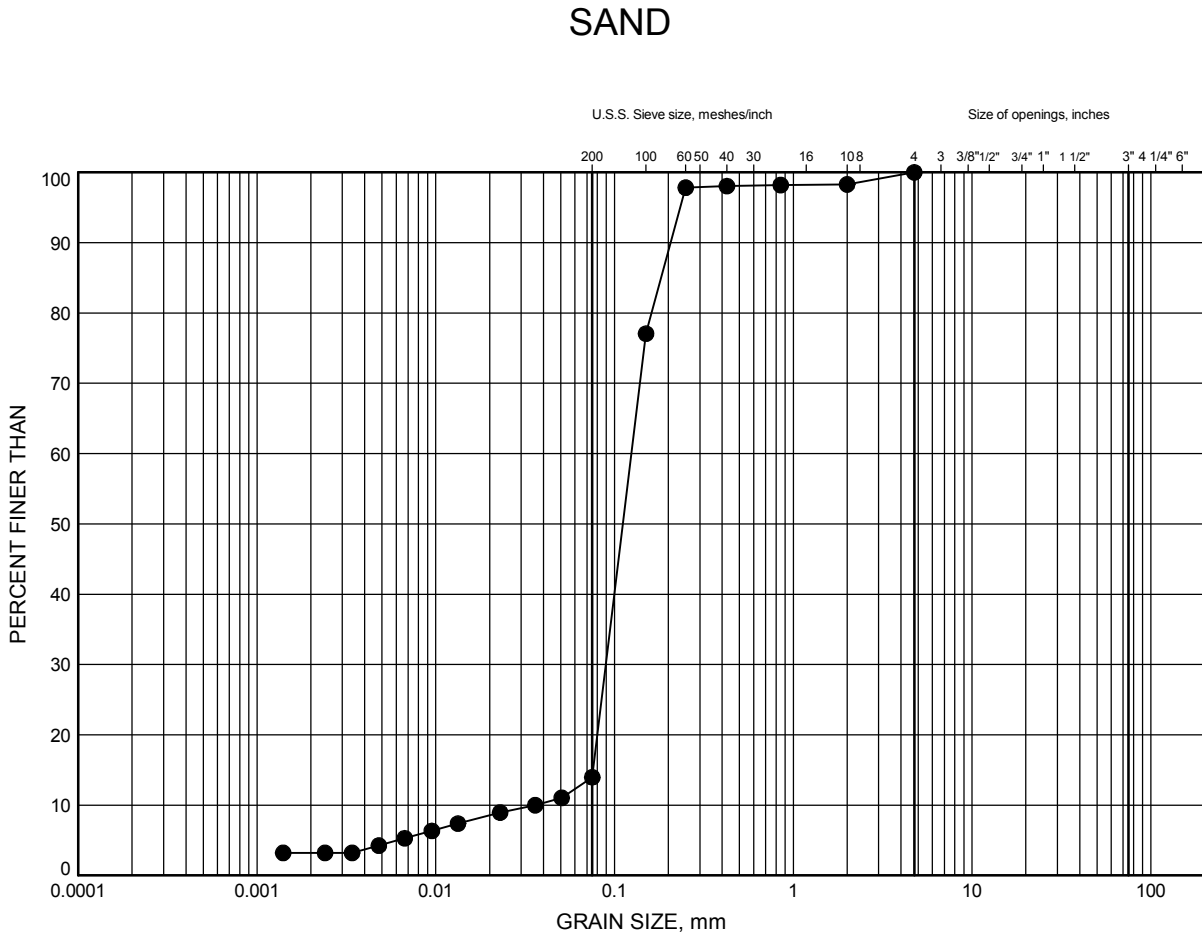
Date January 2015
W.P. 09-20003



Prep'd MFA
Chkd. AMP

QEW Cawthra Road
GRAIN SIZE DISTRIBUTION

FIGURE B2



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	PB 14-02	2.59	100.71

Date January 2015
W.P. 09-20003

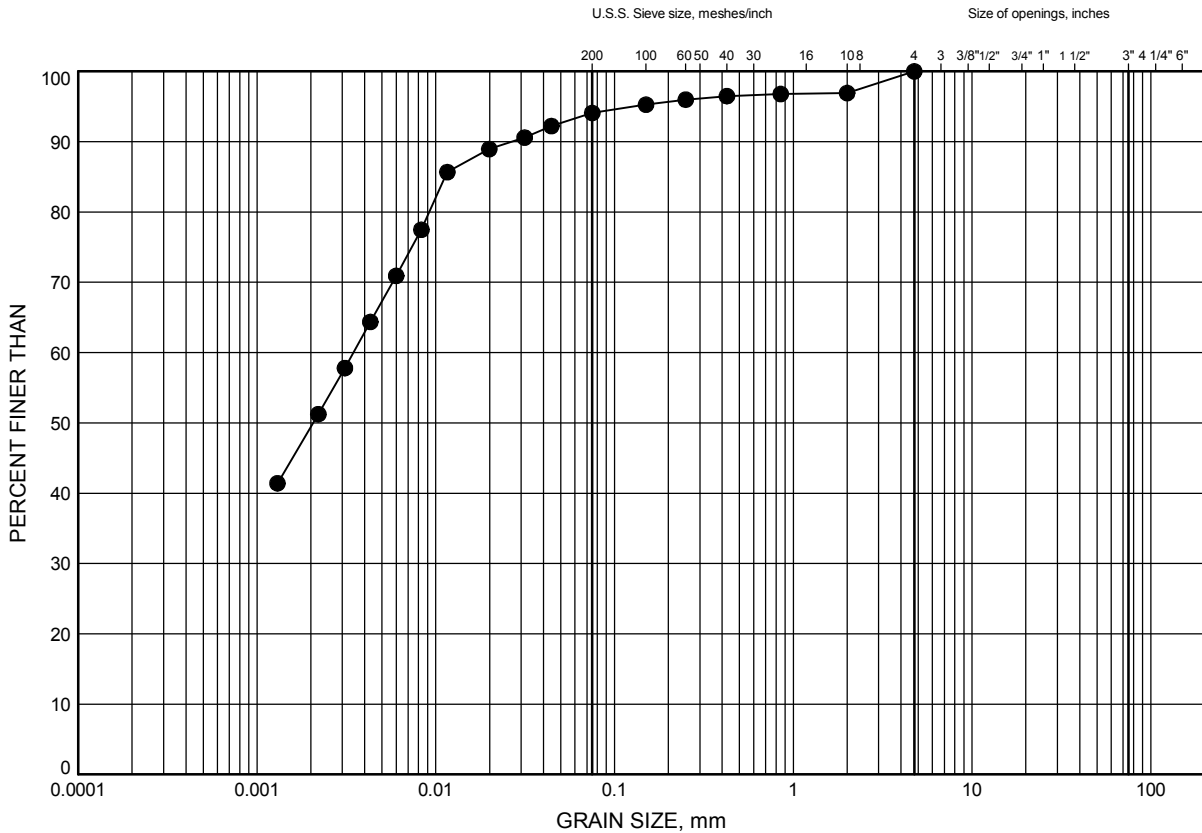


Prep'd MFA
Chkd. AMP

QEW Cawthra Road
GRAIN SIZE DISTRIBUTION

FIGURE B3

SILTY CLAY



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	PB 14-01A	2.59	101.21

Date January 2015
W.P. 09-20003

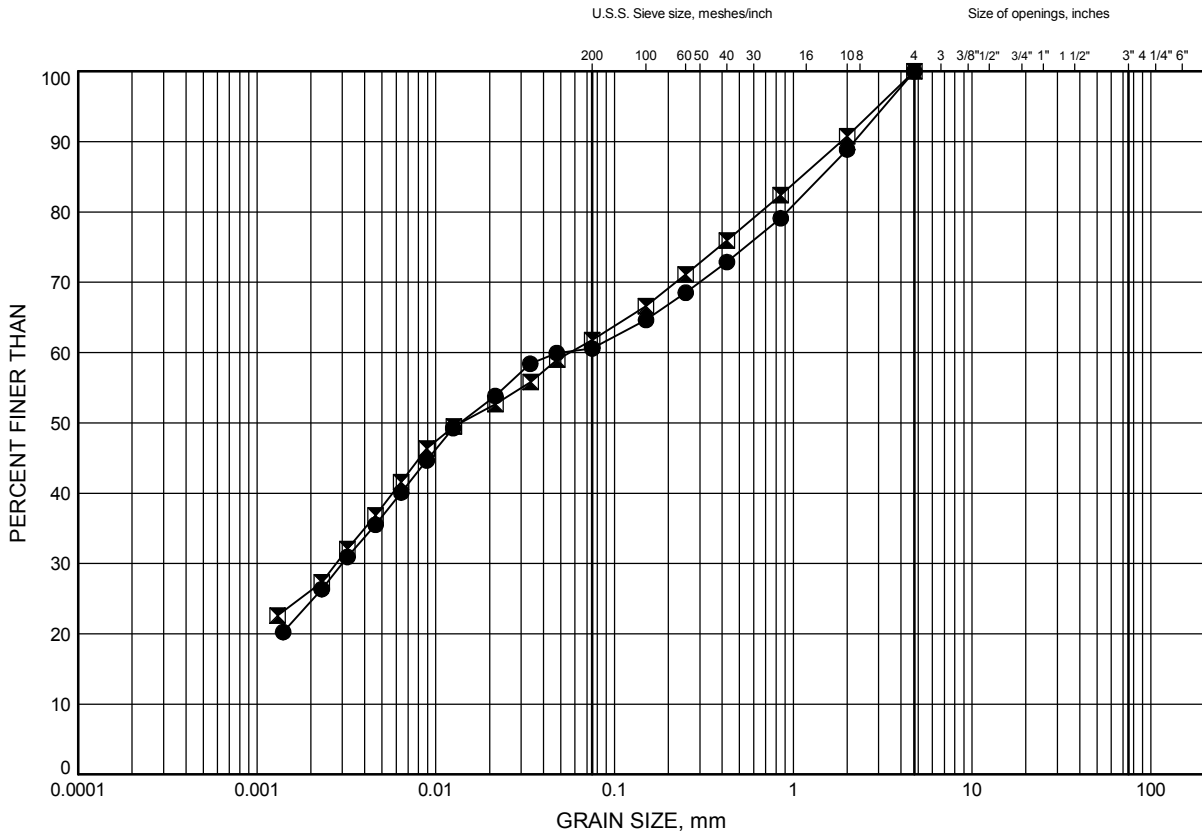


Prep'd MFA
Chkd. AMP

QEW Cawthra Road
GRAIN SIZE DISTRIBUTION

FIGURE B4

SILTY CLAY WITH SAND TILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	PB 14-01A	4.42	99.38
⊠	PB 14-02	4.88	98.42

Date January 2015
W.P. 09-20003



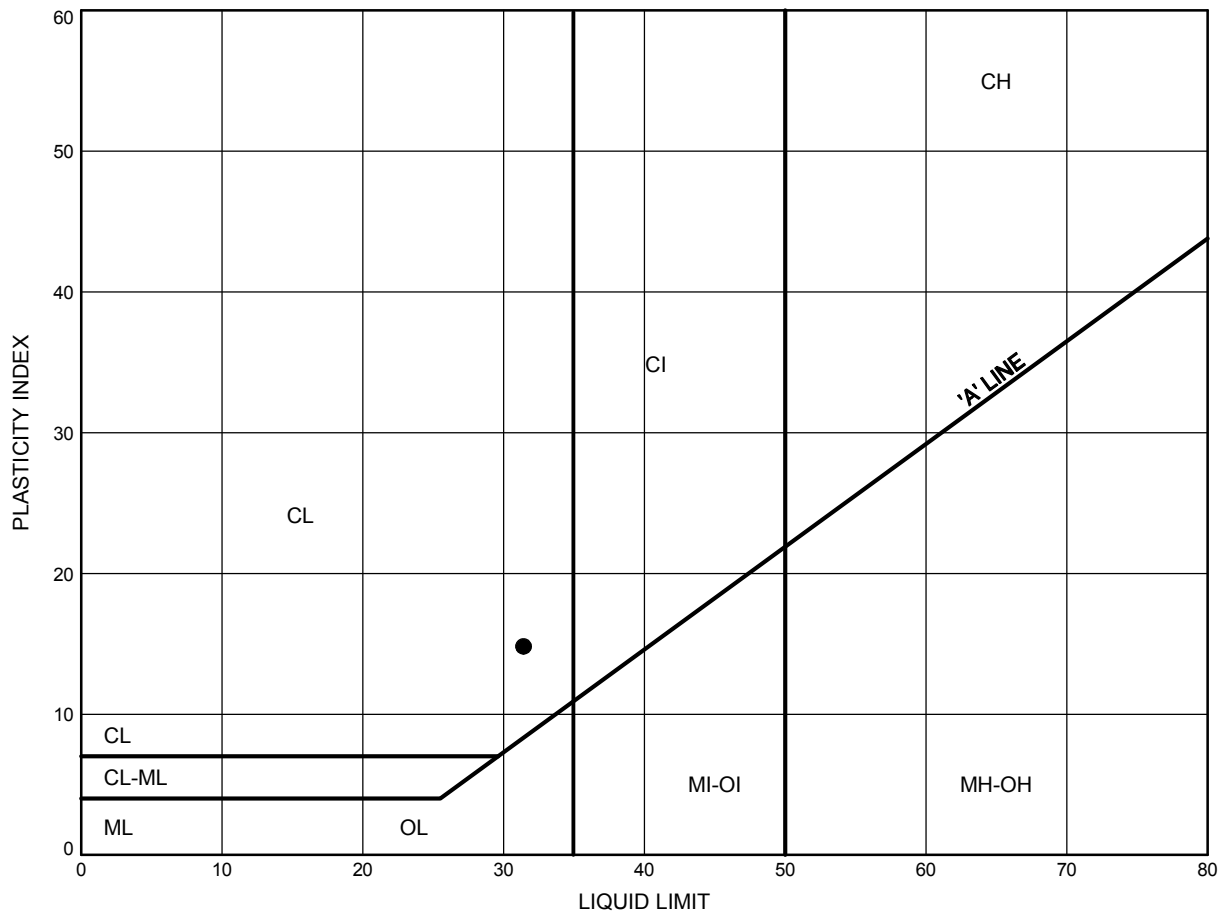
Prep'd MFA
Chkd. AMP

QEW Cawthra Road

ATTERBERG LIMITS TEST RESULTS

FIGURE B5

SILTY CLAY



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	PB 14-01A	2.59	101.21

Date January 2015
W.P. 09-20003



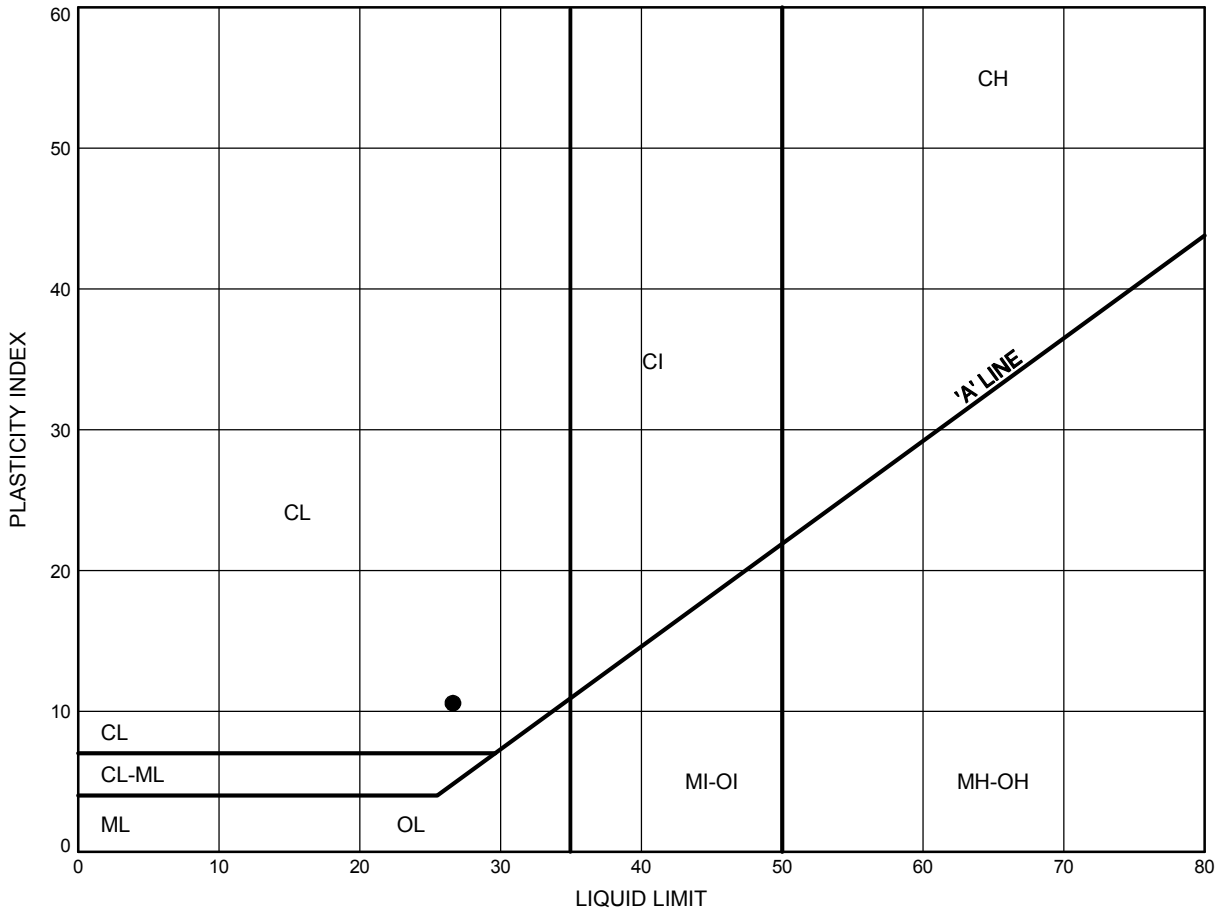
Prep'd MFA
Chkd. AMP

QEW Cawthra Road

ATTERBERG LIMITS TEST RESULTS

FIGURE B6

SILTY CLAY WITH SAND TILL



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	PB 14-02	4.88	98.42

Date January 2015
W.P. 09-20003



Prep'd MFA
Chkd. AMP



THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

Job No : 19-1351-219

Client : MMM

QEW CAWTHRA ROAD

Date Drilled : 9 Dec, 2014

PEDESTRIAN BRIDGE

Date Tested : 15 Dec, 2014

Project Name :
Core Size : NQ BH No : PB14-01B

Tester : ISP

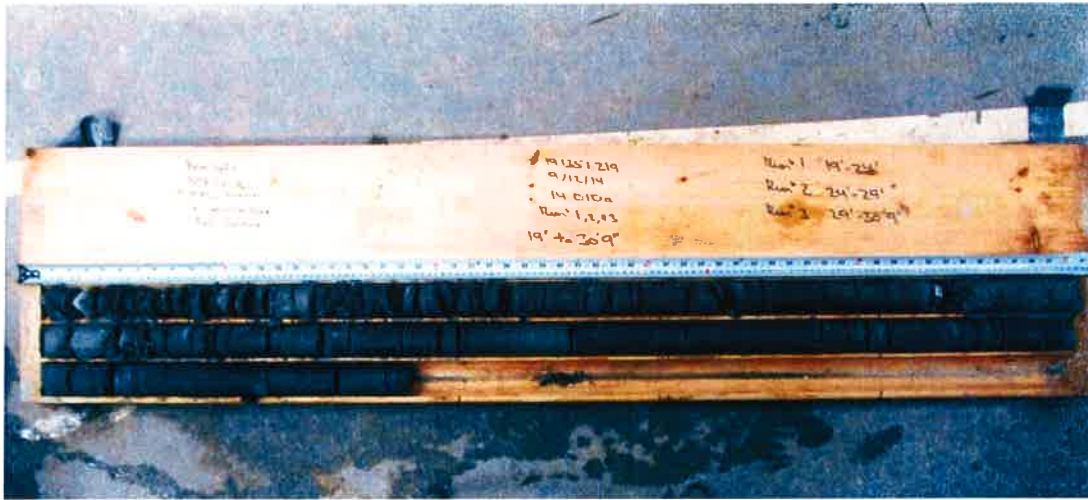
Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (MPa)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	1	6.7	A	15.8	46.8	56.1	115.1	Limestone	Very Strong
2	1	6.9	A	1.9	47.1	50.5	14.9	Shale	Weak
3	2	7.8	A	2.1	46.8	55.2	15.7	Shale	Weak
4	2	8.1	D	10.0	46.8	152.9	100.6	Limestone	Very Strong
5	2	8.2	A	2.3	46.9	73.0	13.8	Shale	Weak
6	2	8.4	D	6.4	46.8	138.5	64.1	Limestone	Strong
7	2	8.4	A	14.9	46.8	69.3	92.2	Limestone	Strong
8	3	9.1	A	1.5	46.9	59.9	10.6	Shale	Weak
9									
10									
11									
12									
13									
14									
15									
16									
17									
18									
19									
20									
21									
22									
23									
24									
25									
26									
27									
28									
29									
30									
31									
32									
33									
34									
35									

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

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

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

Last Modified: August 15, 2013



Photograph 1 – Rock Core from Drillhole 14-01a



Photograph 2 – Rock Core from Drillhole 14-02

Appendix C
Site Photographs



Photograph 1 – Ogden Bridge, East Elevation; looking south



Photograph 2 – North end of Ogden Bridge and access ramps; looking east along North Service Road



Photograph 3 – North ramps and landings; also visible overhead hydro lines - view from the west side of Insley Road



Photograph 4 – Southern section of the Ogden Bridge: looking west along South Service Road.

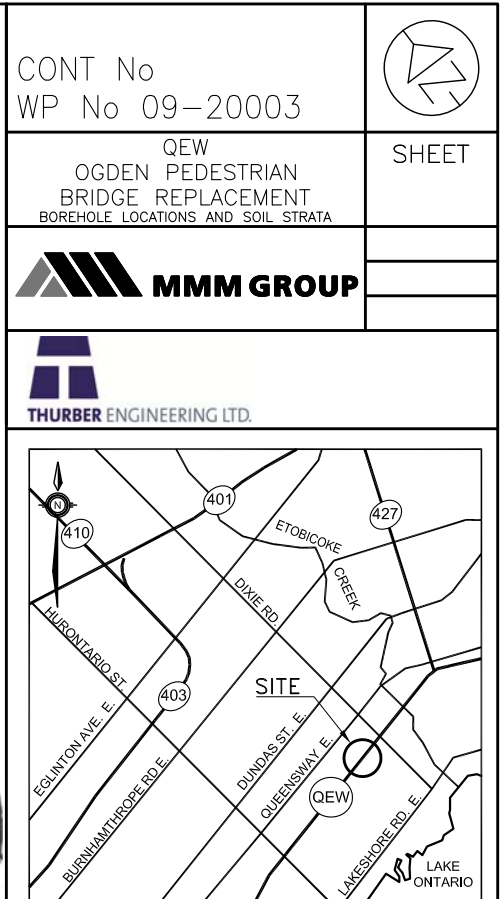
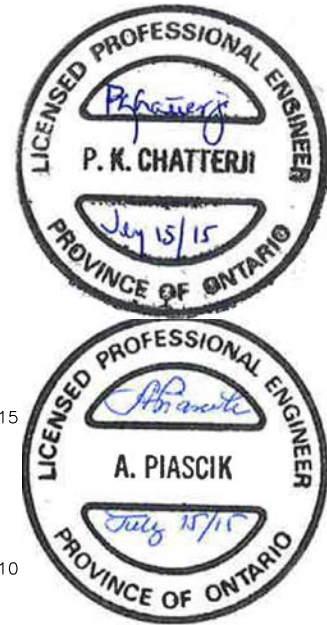
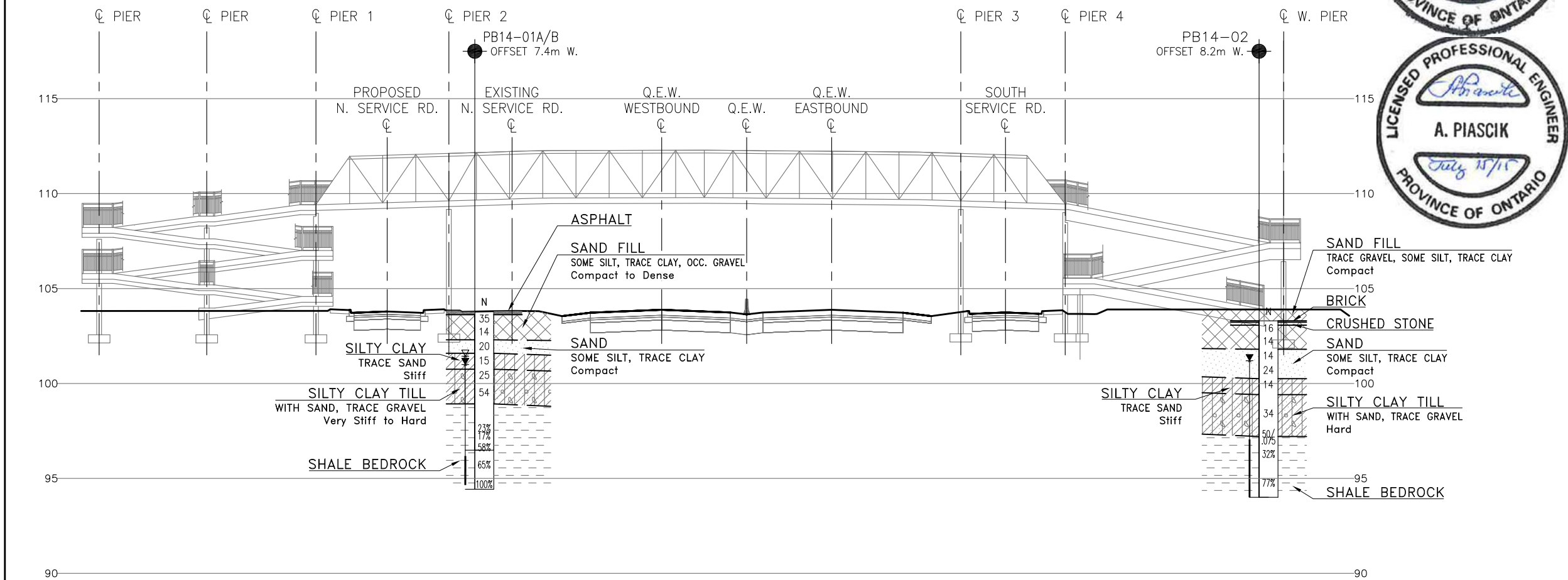


Photograph 5 – South ramps and landings; looking east from Ogden Avenue



Photograph 6 – Drill rig lined up in front of south ramps at Borehole PB14-02; looking southeast from Ogden Avenue

Appendix D
Borehole Location Plan



Appendix E
Comparison of Foundation Alternatives

COMPARISON OF FOUNDATION ALTERNATIVES

Spread Footings on Native Sand	Caissons / Drilled Shafts
<p>Advantages:</p> <ul style="list-style-type: none"> i. Relative ease of construction. ii. Relatively shallow excavation iii. More cost effective than deep foundations. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. Higher geotechnical resistances available ii. Can be installed more expeditiously than spread footings, shortening duration of the foundation construction iii. Construction of caissons may continue in freezing weather.
<p>Disadvantages:</p> <ul style="list-style-type: none"> i. May require groundwater control. ii. May require temporary protection system. 	<p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher unit cost than for spread footings. ii. Temporary liners will be required to install caissons through cohesionless soils. iii. Difficulty in sealing liners at base. iv. Potential difficulties with augering and penetrating hard limestone layers, if socketing required. v. Potential difficulties in cleaning and inspecting bases.
Low risk of encountering problems during construction.	High risk of encountering obstruction in soils and harder layers within bedrock that would require additional procedures to advance the augers to the desired elevation.
RECOMMENDED	FEASIBLE