

REPORT

Foundation Investigation and Design Report

*Ogden Avenue Pedestrian Bridge Replacement (Structure Site No. 24X-0192)
QEW Widening from East of Cawthra Road to The East Mall
Cities of Mississauga and Etobicoke
MTO GWP 2102-13-00 & 2432-13-00*

Submitted to:

AECOM

30 Leek Crescent
Richmond Hill, Ontario L4B 4N4

Submitted by:

Golder Associates Ltd.

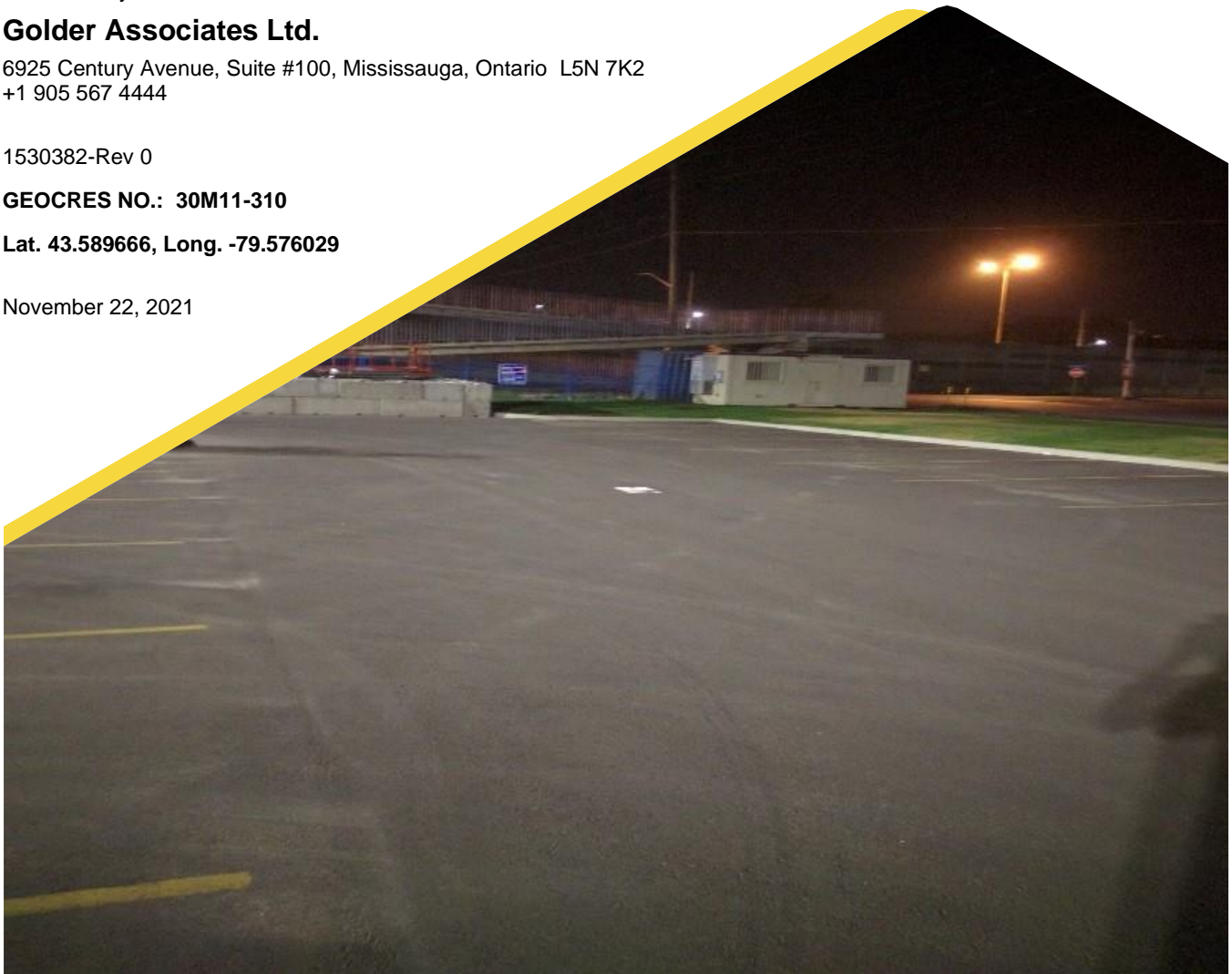
6925 Century Avenue, Suite #100, Mississauga, Ontario L5N 7K2
+1 905 567 4444

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PART A

**FOUNDATION INVESTIGATION REPORT
OGDEN AVENUE PEDESTRIAN BRIDGE REPLACEMENT
(STRUCTURE SITE NO. 24X-0192)
QEW WIDENING FROM EAST OF CAWTHRA ROAD TO THE EAST MALL
CITIES OF MISSISSAUGA AND ETOBICOKE
MTO GWP 2102-13-00 & 2432-13-00**

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by AECOM on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the proposed replacement of the Ogden Pedestrian bridge (Structure Site No. 24X-0192) over the Queen Elizabeth Way (QEW) in the City of Mississauga, Regional Municipality of Peel, Ontario, as part of the widening of the QEW from east of Cawthra Road to The East Mall.

The purpose of this investigation is to establish the subsurface soil, bedrock and groundwater conditions at the proposed bridge structure location by borehole drilling, bedrock coring, geotechnical laboratory testing and analytical chemistry laboratory testing on selected soil and bedrock core samples.

The Terms of Reference (TOR) and the scope of work for the foundation investigation are outlined in MTO's Request for Proposal, dated January 2016, which forms part of the Consultant's Assignment Number 2015-E-0001 for this project. The work has been carried out in accordance with Golder's Project Specific Supplementary Specialty Plan for foundation engineering services for this project, dated June 6, 2016.

2.0 SITE DESCRIPTION

The existing pedestrian bridge structure spans across the QEW, joining the ends of Ogden Avenue and Insley Road, approximately 900 m east of the QEW/Cawthra Road interchange in the City of Mississauga (refer to Key Plan on Drawing 1). The QEW alignment in the project area is oriented generally in a southwest northeast direction; for the purposes of this report, the QEW is taken as being oriented in an east-west direction.

The existing pedestrian bridge structure was constructed in about 1958 and consists of an approximately 70 m long, three-span structure with concrete ramp structures located at both ends. The existing piers are supported on spread footings founded on the near-surface native soil deposits.

Land use is primarily commercial on the north side of the QEW and residential on the south side of the QEW in the vicinity of the pedestrian bridge. The current grade of the QEW at the existing bridge structure is at about Elevation 103.8 m and the existing bridge deck is at about Elevation 109.7 m.

3.0 INVESTIGATION PROCEDURES

3.1 Previous Investigation

In December 2014, a preliminary foundation investigation for the Ogden Pedestrian Bridge replacement was carried out at the site by Thurber Engineering Ltd. (Thurber) during which time a total of three boreholes, designated as Boreholes PB 14-01 A/B and PB 14-02, were advanced. The results of the Thurber investigation are contained in their report titled "Preliminary Foundation Investigation Report, Ogden Pedestrian Bridge Replacement, Queen Elizabeth Way, Mississauga, Ontario", Report No. 19-1351-219, dated July 15, 2015 (GEOCRE 30M11-253).

The locations of the boreholes advanced by Thurber are shown on Drawing 1, Borehole Locations and Soil Strata, drawing, and the borehole records and the summary of the laboratory testing results from this investigation are presented in Appendix A. Borehole PB14-01B was advanced approximately 1.5 m west of Borehole PB 14-01A through the overburden without sampling in order to recover additional bedrock core samples and to allow for the installation of a standpipe piezometer for monitoring of the groundwater level in the bedrock. While the coordinate system of the Thurber borehole locations is not noted, it is inferred that they are referenced to the UTM coordinate

system, and coordinates have been converted to MTM NAD 83 CSRS CBNV6-2010.0 for this report. The northing and eastings of the boreholes, advanced during the previous investigation are presented below and tabulated on Drawing 1.

Borehole No.	Location (MTM NAD 83, Zone 10)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (Latitude, °)	Easting (Longitude, °)		
PB 14-01A	4,827,775.4 (43.555579)	298,610.0 (-79.611422)	103.8	7.3 (including 2.4 m of bedrock core)
PB 14-01B	4,827,775.4 (43.555514)	298,608.5 (-79.611214)	103.8	9.4 (including 3.6 m of bedrock core)
PB 14-02	4,827,723.6 (43.556210)	298,674.4 (-79.610898)	103.3	9.3 (including 3.2 m of bedrock core)

3.2 2016 and 2020 Investigation

The field work for this investigation was carried out in September and October 2016 and February 2020 during which time four sampled boreholes (designated as Boreholes OHS-1, OPB-1, OPB-2, and OPB-3) were advanced in the area of the structure, at the locations shown on Drawing 1.

The field borehole investigation was carried out using a truck mounted CME 75 drill rig, supplied and operated by Davis Drilling of Milton, Ontario and a track mounted Diedrich D120 drill rig, supplied and operated by Walker Drilling of Utopia, Ontario. The boreholes were advanced through the overburden using 108 mm outside diameter (O.D.) solid stem augers and 200 mm O.D. hollow stem augers. Soil samples were obtained at 0.75 m and 1.5 m intervals of depth, using a 50 mm outer diameter (O.D.) split spoon sampler driven by an automatic hammer in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586-08)¹. Samples of the bedrock in Boreholes OPB 1 and OPB 2 were obtained using a 'HQ' size rock core barrel, and Borehole OPB-3 were obtained using an 'NQ' size rock core barrel and coring techniques.

The boreholes were typically advanced to sampler refusal and bedrock was confirmed by augering and sampling in Borehole OHS-1 and by coring in Boreholes OPB-1, OPB-2 and OPB-3. The boreholes were advanced to depths ranging from about 6.2 m to 10.6 m below existing ground surface, including coring of bedrock for core lengths ranging between about 3.0 m to 4.4 m in Boreholes OPB-1, OPB-2 and OPB-3.

The water levels in the open boreholes and field moisture content of the recovered soil samples were observed during the drilling operations and are noted on the borehole records in Appendix B. All boreholes were backfilled with bentonite upon completion in accordance with Ontario Regulation 903, Wells (as amended).

The field work was observed by members of Golder's engineering and technical staff, who located the boreholes, arranged for the clearance of underground services and private locates, observed the drilling, sampling and in-situ testing operations, and logged the boreholes. The samples were identified in the field, placed in appropriate

¹ ASTM D1586-08a – Standard Test Method for Standard Penetration Tests and Split Barrel Sampling of the soil.

containers, labelled and transported to Golder's Mississauga geotechnical laboratory where the samples underwent further visual examination and laboratory testing. Classification testing (water content, Atterberg limits and grain size distribution) was carried out on selected samples in accordance with MTO and/or ASTM Standards, as applicable. Selected rock core samples were submitted to Geomechanica Inc. of Toronto, Ontario for uniaxial compression testing, assessment of Young's modulus and bulk density.

The as-drilled borehole locations and the ground surface elevations were obtained using a GPS (Trimble XH 3.5G), having an accuracy of 0.1 m in the vertical and 0.1 m in the horizontal directions. The locations given in the Record of Borehole/Drillhole sheets and shown on Drawing 1 are positioned relative to MTM NAD 83 (Zone 10) CSRS CBNV6-2010.0 northing and easting coordinates and the ground surface elevations are referenced to Geodetic datum (CGVD 2013). The borehole locations, ground surface elevations and drilled depths are summarized below.

Borehole No.	Location (MTM NAD 83)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (Latitude, °)	Easting (Longitude, °)		
OHS-1	4,827,742.0 (43.589586)	298,624.3 (-79.576480)	103.5	6.2
OPB-1	4,827,795.8 (43.596105)	298,595.9 (-79.569451)	103.1	9.1 (including 3.0 m of bedrock core)
OPB-2	4,827,798.8 (43.599621)	298,577.3 (-79.565717)	103.2	8.6 (including 3.1 m of bedrock core)
OPB-3	4,827,751.0 (43.589666)	298,660.7 (-79.576029)	103.5	10.6 (including 4.4 m of bedrock core)

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

The project area is located within the Iroquois Plain physiographic region, as delineated in The Physiography of Southern Ontario (Chapman and Putman, 1984)².

The glacial Iroquois Plain stretches along the northern shoreline of Lake Ontario, extending from the Niagara Escarpment in the west to the Scarborough Bluffs in the east. The Iroquois Plain soils consist of glaciolacustrine sediments deposited in Lake Iroquois, primarily sands, silts and gravels, with a shallow cover of till remaining over the bedrock.

The bedrock underlying the Greater Toronto Area consists of three shale dominated units: from oldest to youngest, they are the Blue Mountain, Georgian Bay and Queenston Formations. These bedrock formations are essentially horizontally bedded, although on a regional scale, they dip gently to the south. The Georgian Bay Formation which

² Chapman, L.J. and Putman, D.F., 1984, The Physiography of Southern Ontario, Ontario Geological Society, Special Volume 2, Third Edition. Accompanied by Map p. 2715, Scale 1:600,000.)

underlies the study area consists mainly of blue-grey shale, containing siltstone, sandstone and limestone interbeds. Outcrops of this formation are commonly found along water courses on the west side of Toronto and in Mississauga, notably in the Humber River, Mimico Creek, Etobicoke Creek and Credit River valleys.

4.2 Subsurface Conditions

The subsurface soil, bedrock and groundwater conditions as encountered in the boreholes advanced during the 2016 and 2020 investigation and the results of the laboratory tests carried out on selected soil and bedrock core samples are presented on the borehole and drillhole records, which are provided in Appendix B along with photographs of the bedrock core samples. The results of the in-situ field tests (i.e., SPT “N” values) as presented on the Record of Borehole sheets and in sub sections of Section 4.2 are uncorrected. Lists of abbreviations and symbols and lithological, geotechnical rock description terminology, field estimation of rock hardness and rock weathering classification are also included in Appendix B to assist in the interpretation of the borehole and drillhole records. The results of the geotechnical laboratory testing carried out on selected soil and bedrock core samples are also presented in Appendix C. The analytical laboratory test report is included in Appendix D and the test results are summarized in Section 4.2.9.

Stratigraphic boundaries shown on the Record of Borehole sheets and on the stratigraphic profile on Drawing 1 are inferred from non continuous sampling, observations of drilling progress and the results of Standard Penetration Tests. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Furthermore, subsurface conditions will vary between and beyond the borehole locations; however, the factual data presented in the record of Borehole and Drillhole sheets governs any interpretation of the site conditions. It should be noted that the interpreted stratigraphy shown on Drawing 1 is a simplification of the subsurface conditions.

In general, the subsurface conditions in the area of the proposed replacement structure consist of a layer of asphalt underlain by a deposit of granular fill associated with the construction of the existing highway and parking lot on the north side of the North Service Road. The fill is underlain by a deposit of sandy silt to silty sand to sand, underlain by deposits of clayey silt and clayey silt till or residual soil. The native deposits are underlain by shale bedrock.

A detailed description of the subsurface conditions encountered in the boreholes from the previous and 2016 to 2020 investigations are provided in the following sections. Where relatively significant thicknesses of overburden were encountered, the various soil types are described in detail for each main deposit.

4.2.1 Asphalt and Brick/Crushed Stone Fill

All of the boreholes, with the exception of Borehole PB 14-02, were advanced through the surface of the QEW highway or the Applewood Plaza parking lot and penetrated a layer of asphalt varying in thickness from about 80 mm to 230 mm.

Borehole PB 14-02 was advanced in the vicinity of the south end of the existing bridge structure at the pedestrian ramp and encountered a 75 mm thick layer of brick pavement at the existing ground surface, underlain by a 125 mm thick layer of crushed stone.

4.2.2 Fill

A 0.3 m to 1.6 m thick layer of fill was encountered underlying the asphalt and brick bedding in all the boreholes advanced at the site. The fill is variable in composition and generally consists of an upper layer of sand and gravel

underlain by a lower layer of silty sand to sand. The surface of the fill was encountered between about Elevations 103.6 m and 103.0 m and extends to depths of between about 0.4 m and 1.7 m below ground surface.

The SPT “N” values measured within the non-cohesive fill range from 4 blows to 35 blows per 0.3 m of penetration, indicating that the fill layer has a loose to dense compactness condition.

Grain size distribution tests were carried out on two samples of the fill material sampled in the boreholes from the previous investigation and the results are included in Appendix A. The water content measured on five samples of the granular fill ranges between about 5% and 22%.

4.2.3 Sandy Silt to Silty Sand to Sand

In Boreholes OPB-1, OPB-2, OPB-3, PB 14-01 and PB 14-02 a 0.7 m to 2.3 m thick non-cohesive deposit consisting of sandy silt to silty sand to sand, trace clay, was encountered underlying the fill deposit at depths between 0.4 m and 1.7 m below ground surface (between Elevations 103.1 m and 101.5 m). A 0.1 m thick deposit of gravel was encountered in Borehole OPB-3 at the interface of the sand deposit and the underlying clayey silt till deposit (described below).

The SPT “N” values measured within the sand deposit range between 12 to 26 blows per 0.3 m of penetration, indicating that the sand deposit has a compact compactness condition.

The results of a grain size distribution test carried out on one sample of the sand deposit from the previous investigation are included in Appendix A. Grain size distribution testing was carried out on three selected samples of the silty sand to sand deposit from Boreholes OPB-1, OPB-2 and OPB-3 and the results are shown on Figure C-1 in Appendix C.

An Atterberg limits test was carried out on a select sample of this deposit and measured a liquid limit of about 17%, a plastic limit of about 15%, and a plasticity index of about 2%. The result of the Atterberg Limit test is plotted on the plasticity chart on Figure C-2 in Appendix C. The result indicates that the deposit consists of silt of low plasticity. The natural water content measured on four samples of the granular deposit are between 18% and 23%.

4.2.4 Silty Clay to Clayey Silt

In Boreholes OHS-1, PB 14-01 and PB 14-02 a 0.8 m to 1.5 m thick cohesive deposit ranging in composition from silty clay to clayey silt containing trace to some sand, was encountered underlying the fill and sandy silt to sand deposits at depths between 1.4 m and 3.0 m below ground surface (between Elevations 102.1 m and 100.3 m).

The SPT “N” values measured within the cohesive deposit range between 4 blows and 15 blows per 0.3 m of penetration, suggesting that the cohesive deposit has a firm to stiff consistency.

The result of a grain size distribution test carried out on one sample of the cohesive deposit from the previous investigation is included in Appendix A. A grain size distribution test was carried out on a selected sample of the clayey silt deposit from Borehole OHS-1 and the results are shown on Figure C-3 in Appendix C. Atterberg limits tests were carried out on two samples of this deposit (one sample from each investigation) and measured liquid limits of 29% and 32%, plastic limits of 15% and 17%, and plasticity indices of 14% and 15%. The result of the Atterberg limits test on a sample of this deposit from the previous investigation are included in Appendix A and the result of the Atterberg limits test on a sample of this deposit from the 2016 to 2020 investigation is shown on Figure C-4 in Appendix C. These results indicate that the deposit consists of clayey silt of low plasticity. The natural water content measured on two samples of the cohesive deposit are 19% and 20%.

4.2.5 Clayey Silt with Sand (Till)

In Boreholes OPB-1, OPB-2, OPB-3, PB 14-01A and PB 14-02 a 1.9 m to 3.4 m thick cohesive till deposit consisting of clayey silt with sand, containing trace to some gravel, was encountered underlying the sand and cohesive deposits (where present) at depths between 2.7 m and 3.9 m below ground surface (between Elevations 100.8 m and 99.4 m).

The SPT “N” values measured within the cohesive till deposit range between 11 blows and 54 blows per 0.3 m of penetration, suggesting that the cohesive till deposit has a stiff to hard consistency.

The results of two grain size distribution test carried out on samples of the cohesive till deposit from the previous investigation are included in Appendix A. Grain size distribution testing was carried out on three selected samples of the till deposit from Boreholes OPB-1, OPB-2 and OPB-3 and the results are shown on Figure C-5 in Appendix C. Atterberg limits tests were carried out on four samples of this deposit (one from the previous investigation and three samples from the 2016 to 2020 investigation) and measured liquid limits ranging from about 24% to 26%, plastic limits ranging from 14% to 16%, and plasticity indices from about 10% to 12%. The result of the Atterberg limits test on the sample of this deposit from the previous investigation is included in Appendix A and the results of the Atterberg limits tests on samples of this cohesive till deposit from the 2016 to 2020 investigation are shown on Figure C-6 in Appendix C. These results indicate that the cohesive till deposit consists of clayey silt of low plasticity. The natural water content measured on samples of the cohesive till deposit range from 9% to 16%.

4.2.6 Clayey Silt (Residual Soil)

Underlying the clayey silt deposit in Borehole OHS-1 and the clayey silt till deposit in Borehole OPB-1, a 1.3 m and 1.5 m thick deposit of residual soil composed of clayey silt, trace to some gravel and trace shale fragments was encountered at a depths of 2.9 m and 4.5 m below ground surface (Elevation 100.6 m and 98.6 m), respectively.

Residual soil is a heterogeneous mix of fully weathered bedrock that is disintegrated into a soil like material that no longer retains the structure of parent bedrock.

The SPT “N” values measured within the residual soil deposit range from 20 blows to 40 blows per 0.3 m of penetration, suggesting a very stiff to hard consistency.

An Atterberg limits test was carried out on a select sample of the clayey silt residual soil deposit and measured a liquid limit of 23%, a plastic limit of 14% and a corresponding plasticity index of 9%. This result, which is plotted on a plasticity chart on Figure C-7 in Appendix C, indicates that the clayey silt residual soil deposit can be classified as a clayey silt of low plasticity. The water content measured on a sample of the residual soil deposit is 13%.

4.2.7 Shale Bedrock

The upper portion of the bedrock was sampled by split-spoon and the bedrock was confirmed by rock coring in Boreholes OPB-1, OPB-2, OPB-3, PB 14-01A/B and PB 14-02; and the bedrock surface was inferred from split spoon sampling in Borehole OHS-1. The depths to bedrock below ground surface, and the corresponding bedrock surface elevation are summarized below.

Foundation Element	Borehole No.	Depth to Bedrock Surface (m)	Bedrock Surface Elevation (m)	Comments
Piers 1, 6 and 7	OPB-1	5.8	97.3	Bedrock cored
Piers 3 and 4	OPB-2	4.9	98.3	Bedrock cored
Pier 8	PB 14-01A	4.9	98.9	Bedrock cored
Pier 8	PB 14-01B	5.8	98.0	Bedrock cored
Between Piers 8 and 9	OHS-1	4.4	99.1	Split-spoon samples
Pier 9	OPB-3	6.1	97.4	Bedrock cored
Piers 13 to 15	PB 14-02	6.1	97.2	Bedrock cored

In general, the bedrock surface as encountered or inferred in the area of the proposed bridge replacement structure slopes down towards the north-west.

Based on a review of the bedrock core samples from the 2016 to 2020 investigation and descriptions of the bedrock from the previous investigation, the bedrock consists of shale of the Georgian Bay Formation. In general, the bedrock samples are described as moderately to slightly weathered to fresh, thinly laminated to thinly bedded, fine grained, faintly porous, very weak to weak, grey, with strong to very strong limestone interbeds at varying intervals, as presented in the borehole records from the previous investigation in Appendix A and the drillhole records from the 2016 and 2020 investigation in Appendix B, and as shown on the photographs of the recovered core samples on Figures B-1 and B-2 in Appendix B. The degree of weathering of the bedrock samples (i.e., fresh to moderately weathered – W1 to W3), and the strength classification of the intact rock mass based on field identification (i.e., strong to very strong – R4 to R5) are described in accordance with the International Society for Rock Mechanics (ISRM)³ standard classification system.

The Rock Quality Designation (RQD) measured on the core samples ranges from about 17% to 100%, indicating a rock mass of very poor to excellent quality as per Table 3.10 of CFEM (2006)⁴. The Total Core Recovery (TCR) and Solid Core Recovery (SCR) of samples recovered are between 54% and 100% and between 25% and 100%, respectively.

Unconfined Compression (UC) tests (ASTM D7012)⁵ were carried out on selected core samples of the shale bedrock obtained in Boreholes OPB-1 and OPB-2. The uniaxial compressive strength (UCS), bulk density and tangent Young's modulus of the intact samples are summarized below and the details of the tests are presented on the Rock Laboratory Test Results report from Geomechanica included in Appendix C.

³ International Society for Rock Mechanics Commission on Test Methods, 1985. Int. J. Rock Mech. Min. Sci. & Geomech. Abstr. Vol 22, No. 2, pp. 51-60.

⁴ Canadian Geotechnical Society, 2006. Canadian Foundation Engineering Manual (CFEM), 4th Edition. The Canadian Geotechnical Society, BiTech Published Ltd., British Columbia.

⁵ ASTM D7012 – Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens

Borehole No.	Sample Depth Interval (m)	Sample Elevation Interval (m)	Uniaxial Compressive Strength (UCS) (MPa)	Bulk Density (g/cm ³)	Tangent Young's Modulus (GPa)
OPB-1	7.13 – 7.29	95.97 – 95.81	21.1	2.588	1.1
OPB-2	6.31 – 6.41	96.89 – 96.79	16.2	2.582	0.7

Point load index tests carried out on shale and limestone sections of the bedrock core during the previous investigation suggest uniaxial compressive strengths between about 11 MPa and 16 MPa on shale samples and between about 64 MPa and 115 MPa on limestone samples (refer to Appendix A). These values suggest that the shale bedrock is weak ($5 \text{ MPa} < R_2 < 25 \text{ MPa}$) and the limestone layers are strong to very strong ($50 \text{ MPa} < R_4 < 100 \text{ MPa}$ to $100 \text{ MPa} < R_4 < 250 \text{ MPa}$) as per Table 3.5 in CFEM (2006).

Based on the laboratory UCS from the 2016 to 2020 investigation, in accordance with Table 3.5 in CFEM (2006)⁴, the shale bedrock is generally classified as weak (R_2 , $5 \text{ MPa} < \text{UCS} < 25 \text{ MPa}$).

4.2.8 Groundwater Conditions

The overburden samples obtained from the borehole investigations were generally moist to wet. The depths to the water level observed in the boreholes upon completion of drilling and prior to rock coring varied between about 1.1 m and 3.7 m below ground surface (between Elevations 102.4 m and 100.1 m); Borehole OPB-2 was noted to be dry prior to bedrock coring operations.

A summary of the water levels recorded in the piezometers about one month after the 2014 borehole drilling for the preliminary design investigation is presented below:

Borehole No.	Stratum Sealed Into	Water Level Depth (m)	Water Elevation (m)	Date
PB 14-01B	Bedrock	2.8	101.0	January 15, 2015
PB 14-02	Bedrock	2.1	101.2	January 15, 2015

It should be noted that the groundwater level in the area is subject to seasonal fluctuations and precipitation events, and should be expected to be higher during wet periods of the year.

4.2.9 Analytical Testing Results

Two specimens of the clayey silt till deposit were submitted for analysis of parameters used to assess the potential corrosivity of the site soil to steel and concrete. The Bureau Veritas Laboratories report is provided in Appendix D and the test results are summarized below.

Parameter	Borehole OPB-1 / SA 4B (Elev. 99.6 m)	Borehole OPB-2 / SA 5 (Elev. 99.0 m)
pH	7.71	7.84
Resistivity (ohm-cm)	1,400	2,000
Electrical Conductivity (umho/cm)	699	508
Soluble Chlorides (ug/g)	250	130
Soluble Sulphates (ug/g)	200	190

5.0 CLOSURE

This report was prepared by Mr. Matthew Kelly, P.Eng., a geotechnical engineer with Golder. Ms. Lisa Coyne, P.Eng., Principal and MTO Foundations Designated Contact for Golder, conducted a technical and quality control review of the report.

Golder Associates Ltd.



Matthew Kelly, P.Eng.
Geotechnical Engineer



Lisa Coyne, P.Eng.
MTO Foundations Designated Contact, Principal

MWK/WC/LCC/ml

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PART B

FOUNDATION DESIGN REPORT
OGDEN AVENUE PEDESTRIAN BRIDGE REPLACEMENT
(STRUCTURE SITE NO. 24X-0192)
QEW WIDENING FROM EAST OF CAWTHRA ROAD TO THE EAST MALL
CITIES OF MISSISSAUGA AND ETOBICOKE
MTO GWP 2102-13-00 & 2432-13-00

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides detail foundation engineering recommendations for design of the proposed Ogden Pedestrian Bridge replacement (Structure Site No. 24X-0192) spanning over the Queen Elizabeth Way (QEW) as part of the widening of the QEW from Cawthra Road in Mississauga to the Highway 427 area in Etobicoke. These recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the 2014 and the 2016 - 2020 foundation investigations. The discussion and recommendations presented are intended to provide the designer with sufficient information to assess the feasible foundation alternatives and carry out design of the bridge foundations. The foundation investigation report, discussion and recommendations are intended for the use of MTO and its designers and shall not be used or relied upon for any other purpose or by any other parties, including the construction or design-build contractor. The contractor must make their own interpretation based on the factual data in Part A (Foundation Investigation) of the report. Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project and for which special provisions may be required in the Contract Documents. Those requiring information on the aspects of construction must make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling, and the like.

6.1 General

Based on the General Arrangement (GA) drawing provided by AECOM in November 2021, the proposed 79.9 m long, three-span replacement structure will consist of a 54.0 m long central span, a 13.8 m long northern span and a 12.1 m long southern span. A pedestrian ramp structure will be located at both the north and south ends of the bridge, each supported on seven piers, as shown on Drawing 1.

6.2 Consequence and Site Understanding Classification

The proposed bridge crosses over the QEW and has the potential to impact alternative transportation corridors. In accordance with Section 6.5 of the 2019 Canadian Highway Bridge Design Code and its Commentary (CHBDC 2019), the proposed bridge and its foundation system is considered to be classified as having a “typical consequence level” associated with exceeding limits states design. In addition, given the level of foundation investigation completed to date at this location in comparison to the degree of site understanding in Section 6.5 of the CHBDC (2019), the level of confidence for design is considered to be a “typical degree of site and prediction model understanding.” Accordingly, the appropriate corresponding ULS and SLS consequence factor, Ψ , from Table 6.1 and geotechnical resistance factors, ϕ_{gu} and ϕ_{gs} , from Table 6.2 of the CHBDC (2019) have been used for design.

6.3 Foundations Options

Given the presence of near-surface competent native soil deposits and presence of bedrock at relatively shallow depths below ground surface, the bridge piers may be founded on shallow foundations on native soil or bedrock. Based on the foundation locations and the subsurface conditions, the piers could be supported on spread footings founded at about Elevation 100 m along the northern half of the bridge at Piers 1 to 8 and at about Elevation 99 m along the southern half of the bridge at Piers 9 to 16, to extend below looser/weaker fill materials and compressible cohesive layers, where present. At these elevations, the spread footings would be founded on the native till deposit. Should higher bearing resistance be required beyond those achievable for spread footings founded on the till deposit, the spread footings could also be founded on bedrock at about Elevations 97 m to 96 m, although this would require significant excavation.

Alternatively, to minimize excavation during construction together with associated plan footprints and protection system requirements, the piers could be supported on “deep” foundation elements, such as H-piles, pipe piles or caissons (drilled shafts).

The preferred foundations option for this bridge will depend on the site limitations. The areas at the pier locations are very confined, with little working room, particularly at piers 8 and 9, which are located between the active lanes of the highway and service roads. In addition, the north and south ends of the bridge are bordered by private properties, consisting of a commercial parking lot at the north end and a private residence at the south end, and by municipal roads. Excavations below grade, of at least 3 m depth for shallow foundations, will likely require temporary protection systems on all sides. The preferred foundation alternative will need to consider the additional costs associated with the constructability constraints due to the very limited working room and the potential impacts (e.g., vibrations at private residences) to adjacent properties.

A comparison of the foundation options based on advantages, disadvantages, risks/consequences and relative costs is provided in Table 1 following the text of this report and is summarized below.

- **Shallow foundations** – spread/strip footings: Shallow foundations consisting of strip or spread footings founded on the native soil or bedrock are feasible for support of the piers. Temporary protection systems will be required for all the excavations. Assuming the piers are founded at or below Elevation 100 m, the pier excavations will penetrate below the groundwater level and groundwater control to maintain dry conditions at the base of the footings will be required.
- **Deep foundations** – driven steel H piles or Pipe Piles: Driven steel H piles are considered feasible for support of the bridge structure. Such deep foundations would achieve greater geotechnical resistances than shallow foundations but requires specialized equipment for installation. Excavations (and temporary protection systems) for construction of the pile caps would also be required, although these would extend to shallower depths than required for the shallow foundations. Impacts to adjacent private property owners from vibrations and noise during driving may be difficult to manage or mitigate.
- **Deep foundations** – drilled shafts (caissons): Drilled shafts (caissons) are considered feasible for the support of the piers but may be more expensive than either shallow foundations or driven pile foundations, although fewer caisson elements would be required in comparison to the number of steel H-piles that would be required. The caisson socket may have to penetrate hard limestone layers increasing the cost but such foundations could still be more cost-effective than H-piles or pipe piles, considering that increased resistances would be achieved, and there would be cost savings related to protection systems as compared with the shallow foundation or H-pile options. If caissons are adopted for support of the piers, temporary liners within the overburden and into the upper portion of rock will be required during construction to control potential groundwater seepage and ground losses.

Based on the above considerations and as detailed in the Table 1, from a foundations perspective, it is considered that piers supported on drilled shafts, socketed nominally into bedrock, is the most technically (including constructability concerns) feasible, cost effective and preferred foundation alternative.

6.4 Spread Footings

6.4.1 Founding Elevations on Native Soil or Bedrock

Detailed below, for each foundation element are recommended founding elevation for spread footings on native soil or bedrock.

Pier No.	Reference Borehole(s)	Founding Elevation in Cohesive Till Deposit (m)	Bedrock Surface Elevation (m)	Founding Elevation on Bedrock (m)
1 to 7	OPB-1 OPB-2	99.5	98.4 to 97.3	97.0
8	PB 14-01A/B	100.5	98.9	98.0
9	OPB-3	99.7	97.4	96.0
10 to 16	OPB-3 PB 14-02	100.8 to 99.0	97.4 to 97.2	96.0

Geotechnical resistance values for footings founded at the elevations recommended above are provided in Section 6.4.2.

The GA drawing indicates that the proposed structure is to be constructed approximately along the same alignment as the existing structure and that Piers 8, 9, 10 and 11 will be located in close proximity to the existing foundations. In this regard, any new footings located within an area (influence zone) bounded by a line drawn up and away from the existing footings at 1 horizontal:1 vertical (1H:1V), should be constructed with the underside of the new footings either at or below the elevation of the existing footings.

Based on the description of the recovered bedrock core samples, the upper approximately 2 m of the shale bedrock is weak, but containing strong to very strong limestone layers, and where excavations for the foundations extends into this formation, appropriate construction equipment and procedures (such as hoe-ramming) will be required. It is recommended that a Non-Standard Special Provision (NSSP) be included in the Contract Documents to warn the contractor of such obstructions; this is discussed further in Section 6.9.2. Further, as discussed in Section 6.9.6, vibration monitoring should be carried out at the nearest residences located immediately adjacent to the south pedestrian ramp during hoe-ramming activities.

The footing subgrade should be inspected, in accordance with OPSS 902 (Excavating and Backfilling Structures) to check that all existing fill, organics and any other deleterious materials have been removed. Furthermore, if the excavations extend into bedrock, it will be necessary to clean, scale and remove all loose, shattered and/or fractured rock within the area of the footing to ensure proper concrete bond to the bedrock.

The native soil or bedrock foundation subgrade will be susceptible to disturbance from ponded water, precipitation from inclement weather and/or construction traffic. If the concrete for the footings cannot be poured immediately after excavation and inspection, it is recommended that a concrete working slab having a minimum thickness of 100 mm and a minimum of 28-day compressive strength of 20 MPa be placed in the excavation within four hours of exposure of the founding level to protect the integrity of the subgrade. An NSSP to address this item is included in Appendix E, which should be included in the Contract Documents.

6.4.2 Geotechnical Resistance

Strip or spread footings, placed on the properly prepared till deposit, at or below the design elevations given in Section 6.4.1, should be designed based on the factored ultimate geotechnical resistances at ULS and the factored serviceability geotechnical resistances at SLS (for 25 mm of settlement) given below for the footing sizes noted.

Geotechnical Resistances for Strip/Spread Footings Founded on Very Stiff to Hard Cohesive Till

Assumed Approximate Footing Size	Factored Ultimate Geotechnical Resistance (kPa)	Factored Serviceability Geotechnical Resistance (for 25 mm of settlement) (kPa)
6.0 m x 5.0 m	750 kPa	250 kPa
5.0 m x 5.0 m	700 kPa	300 kPa
4.0 m x 5.0 m	700 kPa	300 kPa
3.0 m x 5.0 m	600 kPa	400 kPa

The settlement will be dependent on the footing size, configuration and applied loads; the geotechnical resistance should, therefore, be reviewed if the selected footing width or founding elevation differs from those given in Section 6.4.1.

Spread footings founded on the properly prepared bedrock at or below the design elevations given in Section 6.4.1 may be designed based on a factored ultimate geotechnical resistance at ULS of 4 MPa. The resistance at SLS does not need to be considered since the bearing pressure for 15 mm of settlement will exceed the ULS resistance.

The factored geotechnical resistances provided above are given for loads that will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the footing, inclination of the load should be taken into account in accordance with Section 6.10.2 of the CHBDC (2019) using an undrained shear strength of 100 kPa for the very stiff to hard cohesive till deposit.

6.4.3 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the new concrete footings and the subgrade should be calculated in accordance with Section 6.10.4 of the CHBDC (2019). For cast-in-place concrete footings constructed directly on the native soils, or bedrock, or on a concrete working slab, the sliding resistance may be calculated based on the unfactored coefficient of friction, $\tan \phi'$, which can be taken as follows:

- Cast-in-place footing or concrete working slab to cohesive till $\tan \phi' = 0.62$
- Cast-in-place footing or concrete working slab shale bedrock $\tan \phi' = 0.62$

If necessary, the sliding resistance between the concrete footing and/or working slab and the native soils, or bedrock at the piers can be supplemented by construction of a shear key or dowelling into the bedrock.

The horizontal resistance of dowels is dependent on the strength of the bedrock, grout and steel. The dowels should have a minimum length within the fair quality bedrock (having an RQD > 50%) of 1 m, and the structural strength of the dowels and compressive strength of the grout should not be exceeded. For uplift of the dowels, a factored

resistance of 200 kPa may be assumed for the grout-to-rock bond stress for ULS design. The actual bond stress along the rock-grout interface may vary from the design value given and it should therefore be verified in the field by pull-out testing. If dowelling into bedrock is adopted at this site, an NSSP should be included in the Contract Documents to specify the installation, materials and testing of the dowels; an example is provided in Appendix E. These values assume that construction is carried out in dry conditions.

6.4.4 Frost Protection

Strip or spread footings should be founded at a minimum depth of 1.2 m below the lowest surrounding grade to provide adequate protection against frost penetration as interpreted from OPSD 3090.101 (Foundation Frost Depths for Southern Ontario).

6.5 Steel H-Pile Foundations

6.5.1 Founding Elevations

Consideration can be given to supporting the piers on steel H-piles (or pipe piles), driven to bedrock. Provided below are the design pile tip elevations for pile foundations driven to refusal on or into the shale bedrock; based on the strength of the shale bedrock and weathering observed in the recovered core samples, the recommended design tip elevations assume that only nominal penetration into the bedrock will occur. There should be a provision made in the Contract for dealing with varying pile lengths due to the variability in depth to the bedrock surface and degree of weathering and the lengths given below should be considered minimum lengths.

The structural designers should assess whether the pile lengths are sufficient from a structural perspective. If steel H-piles are adopted, it is recommended that an NSSP be included in the Contract Documents to describe to the Contractor the strength and character of the bedrock; an NSSP is included in Appendix E for this purpose.

Pier No.	Borehole No.	Recommended Underside of Pile Cap Elevation	Bedrock Surface Elevation (m)	Design Pile Tip Elevation (m)	Estimated Approx. Pile Length below Pile Cap
1 to 7	OPB-1 OPB-2	102.3	98.3 to 97.3	97.7 to 97.0	4.3 to 4.9
8	PB 14-01 A/B	102.3	98.9 to 98.0	98.0	4.6
9	OPB-3	102.1	97.4	97.3	5.2
10 to 16	OPB-3 PB 14-02	101.8	97.4 to 97.2	97.0	5.3 to 5.1

For steel H-piles driven to / into bedrock consideration must be given to the potential for damage to the pile tips during seating on the bedrock. The piles should be reinforced at the tip with MTO flange plates, driving shoes or rock points to reduce the potential for damage to the piles during driving, in accordance with OPSS 903 (*Deep Foundations*). Based on local experience and given the soil and shallow bedrock conditions at this site, driving shoes in accordance with OPSD 3000.100 (steel H-pile driving shoes) are recommended over flange plates.

If steel pipe piles, such as 324 mm diameter by 6.4 mm wall thickness (12 ¾ inch by ¼ inch wall thickness), are used in lieu of H-piles, driving shoes should be in accordance with OPSP 3001.100 Type II (*Steel Tube Pile Driving Shoe*) and driven to the design pile tip noted above, recognizing that it will likely not be possible to achieve these tip elevations.

As discussed further in Section 6.9.6, vibration monitoring at the nearest residences which are located immediately adjacent to the south ramp structure should be carried out during deep foundation construction activities.

6.5.2 Geotechnical Axial Resistances

For HP310x110 steel H-piles driven to bedrock, and similarly for pipe piles as noted above, a factored ultimate geotechnical resistance of 1,600 kN per pile may be used for design. The factored serviceability geotechnical resistance at SLS for 25 mm of settlement (for the length of piles required at this site) will be greater than the factored ultimate geotechnical resistance at ULS, as such, the factored ultimate geotechnical resistance at ULS will govern for this foundation type.

A heavier steel H-pile section, such as HP310x132 could also be used in order to assist with seating the pile. A factored ultimate geotechnical resistance at ULS of 1,850 kN may be used for design for this larger pile section and the factored ultimate geotechnical resistance will govern the design, rather than the factored serviceability geotechnical resistance at SLS.

6.5.3 Set Criteria and Pile Driving Note

Pile installation should be carried out in accordance with OPSS 903 (Deep Foundations). For piles driven to bedrock, set criteria are highly dependent on pile driving hammer type and the selected pile. The set criteria can be established through a variety of methods, including empirical correlations and wave equation analyses, at the time of construction once the hammer and pile types are known. The criteria need to be set to also avoid overdriving and possibly damaging the pile.

The pile driving note that should be added to the drawings for this project is Note 5 in Clause 3.3.3 of the MTO's Structural Manual (MTO 2016), as follows:

- "Piles to be driven to bedrock."

6.5.4 Resistance to Lateral Loads

Resistance to lateral loading may be derived using vertical piles, with enhanced support offered by inclined (battered) piles, if required. For vertical piles, the resistance to lateral loading will be derived solely from the soil in front of the piles, whereas inclined piles derive lateral resistance from the soil in front of the piles as well as the horizontal component of the axial load present in the inclined pile.

Where ground conditions are generally competent and the lateral loads on piles are relatively small such that the maximum lateral pile deflections will be relatively small, the resistance to lateral loading in front of a single pile can be estimated using subgrade reaction theory (as outlined below). However, if it is noted that the response of a pile to lateral loads is highly nonlinear and methods that assume linear behavior (such as subgrade reaction theory) are only appropriate where the maximum pile deflections are less than 1% of the pile diameter, where the loading is static (no cycling) and where the pile material is linear (CFEM, 2006). Where these conditions are not met, the non-linear lateral behavior of the soil should be considered by the use of P-y curves.

The factored serviceability geotechnical response of the soil in front of the piles under lateral loading at this site may be calculated using subgrade reaction theory suggested in CHBDC (2019) Commentary (Section C6.11.2.2), where the coefficient of horizontal subgrade reaction, k_h , (kPa/m) is based on the equation given below, as described by Terzaghi (1955) and the Canadian Foundation Engineering Manual (CFEM 1992).

For cohesionless soils:

$$k_h = \frac{n_h z}{B}$$

Where: n_h is the constant of horizontal subgrade reaction (kPa/m), as given below;
 z is the depth (m); and,
 B is the pile diameter/width (m).

For cohesive soils:

$$k_h = \frac{67s_u}{B}$$

Where: s_u is the undrained shear strength of the soil (kPa); and,
 B is the pile diameter/width (m).

The following values of n_h and s_u (Terzaghi, 1995) may be incorporated into the calculations of horizontal subgrade reaction (k_h) for structural analyses for a single vertical pile. The ranges in values reflect the variability in the subsurface conditions, the soil properties and the approximate nature of the analysis and the non-linear nature of the soil behaviour (such that k_h is a function of deflection).

Pier No.	Soil Deposit	Elevation (m)	N_h (kPa/m)	S_u (kPa)
1 to 7	Compact to dense silty sand to sand fill	103.1 to 101.5	5,000	--
	Compact silty sand to sandy silt	101.6 to 99.9	4,000	--
	Very stiff to hard clayey silt with sand till	99.9 to 98.4	--	100
8	Compact to dense sand fill	103.8 to 102.3	5,000	--
	Compact sand	102.3 to 101.6	5,000	--
	Stiff silty clay	101.6 to 100.8	--	75
	Very stiff to hard silty clay with sand till	100.8 to 98.9	--	100
9	Sand and gravel fill	103.5 to 103.1	5,000	--
	Compact sand	103.1 to 100.8	5,000	--
	Stiff to very stiff clayey silt with sand till	100.8 to 97.4	--	100

Pier No.	Soil Deposit	Elevation (m)	N_h (kPa/m)	S_u (kPa)
10 to 16	Sand fill	103.3 to 101.8	5,000	--
	Compact sand	101.8 to 100.3	5,000	--
	Stiff silty clay	100.3 to 99.4	--	50
	Stiff to hard clayey silt to silty clay with sand till	99.4 to 97.2	--	100

Both the structural and geotechnical resistances of the piles should be evaluated to establish the governing case at ULS. At SLS, the horizontal reaction of the piles will be controlled by deflections and the horizontal resistance of the pile should be calculated based on the coefficient of horizontal subgrade reaction (k_h) of the soil as discussed above. The SLS resistance should be taken as that corresponding to a horizontal deflection of 10 mm at the underside of the pile cap for units supporting the abutments (CHBDC (2019) Commentary Section 6.11.2.2).

The upper zone of the soil (down to a depth below the pile cap equal to about $1.5 \times B$ (where B is the pile diameter) should be neglected in the calculation of lateral resistance of the pile to account for disturbance effects during installation.

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters between rows of driven steel H piles. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading by a reduction factor, R (NAVFAC DM 7.2, 1986) as follows:

Pile Spacing in Direction of Loading (D = Pile Diameter)	Subgrade Reaction Reduction Factor, R
8D	1.00
6D	0.70
4D	0.40
3D	0.25

The subgrade reaction reduction factor should be interpolated for pile spacings in between those provided in the above summary. Reduction for group effects is negligible when the centre-to-centre pile spacing exceeds three pile diameters measured in the direction perpendicular to loading.

6.5.5 Frost Protection

All pile caps should be provided with a minimum of 1.2 m of soil cover for frost protection as interpreted from OPSD 3090.101 (Foundation, Frost Penetration Depths for Southern Ontario).

6.6 Caissons (Drilled Shafts)

6.6.1 Founding Elevations and Geotechnical Axial Resistances

Caisson foundations could also be considered for support of the piers. A temporary liner and placement of concrete by the tremie method will be required to install caissons at the site as the groundwater table is above the bedrock surface. The following founding levels are recommended to penetrate through the weathered/fractured zone and into good quality bedrock, together with factored ultimate geotechnical resistances corresponding to the given founding level:

Pier No.	Borehole No.	Bedrock Surface Elevation (m)	"1 m Socket"			"1.5 m Socket"		"2 m Socket"	
			Maximum (Highest) Founding Elev. (m)	900 mm Caisson f-ULS (kN)	1200 mm Caisson f-ULS (kN)	Maximum (Highest) Founding Elev. (m)	900 mm Caisson f-ULS (kN)	Maximum (Highest) Founding Elev. (m)	900 mm Caisson f-ULS (kN)
1 to 7	OPB-1 OPB-2	98.4	96.0	2,000	3,000	95.5	2,500	95.0	3,000
8	PB 14-01A/B	98.9	96.5			96.0		95.5	
9	OPB-3	97.4	96.0			95.5		95.0	
10 to 16	OPB-3 PB 14-02	97.2	95.5			95.0		94.5	

The factored serviceability geotechnical resistance (for 25 mm of settlement) is greater than that given for the factored ultimate geotechnical resistance, and therefore the serviceability condition does not apply.

The centre-to-centre spacing between proposed caissons within a group founded in bedrock for the pedestrian bridge foundations should be greater than 2.5 times the caisson diameter to limit interaction between caissons. So long as this minimum caisson spacing within a group is maintained, the efficiency factor for the pile group is expected to be 1.0 (i.e., no reduction for group effects is required).

The shale bedrock is generally weak, but contains strong to very strong limestone interbeds and therefore the sockets will likely need to be advanced into the bedrock by churn drilling or down-the-hole hammer. If caissons are adopted as the foundation alternative, it is recommended that an NSSP be included in the Contract Documents to describe to the Contractor the strength and character of the bedrock; an NSSP is included in Appendix E for this purpose.

For caissons designed for end bearing, the performance of the caissons in compression will depend to a large degree upon the final cleaning and verification of the condition of the subgrade rock at the base of the caisson. For caissons acting in compression, the base of each caisson excavation must be cleaned to remove all loose cuttings to ensure that the tremied concrete is in intimate contact with the competent shale bedrock. The inspection of the base of the rock sockets can be accomplished after flushing and cleaning of the base by means of a Shaft Inspection Device (SID) such as a video camera. Should the camera inspection indicate that loosened/unacceptable soil or rock is present at the base the caisson, the socket base would need to be re-cleaned and re-inspected.

A geotechnical engineer must confirm that the conditions encountered are consistent with the information obtained from the boreholes and that the required minimum socket geometry and cleanliness has been obtained.

6.6.2 Resistance to Lateral Loads

The geotechnical resistance to lateral loading for the caissons should be calculated in accordance with Section 6.5.4, using the horizontal subgrade formulas and parameter values presented herein.

6.7 Seismic Design

6.7.1 Seismic Site Classification

Subsurface ground conditions for seismic site characterization were established based on the results of the borehole investigations. Based on the anticipated foundation levels on or near to the surface of the bedrock, the site may be classified as Site Class D for spread footings founded on the native site soils, and Site Class C for spread footings or deep foundations founded on/in bedrock, in accordance with Table 4.1 of the CHBDC (2019) in the absence of any geophysical testing. Geophysics testing, if carried out, could provide a more favourable Site Class designation, but would also depend on the elevation of the foundations. For example, Table 4.1 of the CHBDC (2019) indicates that Site Class A and B are not to be used if there is more than 3 m of soils between the rock and the underside of the bridge foundations (i.e., footings or pile caps).

6.7.2 Spectral Response Values and Seismic Performance Category

In accordance with Section 4.4.3.4 of the CHBDC (2019), the peak ground acceleration (PGA) and design spectral acceleration (Sa) values for Site Class C and Site Class D are presented below.

Seismic Design Values for Footings or Deep Foundations on Bedrock (Site Class C)			
Seismic Hazard Values	10% Exceedance in 50 years (475-year return period)	5% Exceedance in 50 years (975-year return period)	2% Exceedance in 50 years (2,475 return period)
PGA (g)	0.042	0.077	0.148
PGV (m/s)	0.031	0.052	0.094
Sa (0.2) (g)	0.070	0.122	0.230
Sa (0.5) (g)	0.043	0.068	0.119
Sa (1.0) (g)	0.023	0.036	0.060
Sa (2.0) (g)	0.011	0.017	0.028
Sa (5.0) (g)	0.0023	0.0039	0.0068
Sa (10.0) (g)	0.0010	0.0016	0.0028

Seismic Design Values for Footings on Native Soil (Site Class D)			
Seismic Hazard Values	10% Exceedance in 50 years (475-year return period)	5% Exceedance in 50 years (975-year return period)	2% Exceedance in 50 years (2,475 return period)
PGA (g)	0.0542	0.0993	0.1857
PGV (m/s)	0.0456	0.0764	0.1352
Sa (0.2) (g)	0.0868	0.1513	0.2789
Sa (0.5) (g)	0.0632	0.1000	0.1712
Sa (1.0) (g)	0.0357	0.0558	0.0912
Sa (2.0) (g)	0.0173	0.0267	0.0433
Sa (5.0) (g)	0.0036	0.0062	0.0106
Sa (10.0) (g)	0.0015	0.0024	0.0041

In accordance with Table 4.10 of the *CHBDC (2019)*, the bridge structure (Importance Category of “Major-Route or Other”), falls within Seismic Performance Category 1 and therefore, analysis for seismic loads is not required as per Section 4.4.5.1 of the *CHBDC (2019)*.

6.7.3 Potential for Liquefaction

Based on the compactness and fines content of the sandy silt to sand deposit, the presence of stiff to hard cohesive soils and the site-specific PGA, the soils at this site are considered to have a low potential for liquefaction during a design seismic event.

6.8 Analytical Testing for Construction Materials

The results of analytical testing on two clayey silt with sand till samples from Boreholes OPB-1 and OPB-2 are presented in Section 4.2.9 and in Appendix D. The potential for sulphate attack and corrosion are discussed in the following sub sections. However, it is ultimately up to the designer to determine the appropriate construction materials, including the exposure class and ensuring that all aspects of CSA A23.1 14 Section 4.1.1 “Durability Requirements” are followed when designing concrete elements.

6.8.1 Potential for Sulphate Attack

The analytical test results were compared to CSA Standard, CAN/CSA-A23.1-14 Table 3 (“Additional requirements for concrete subjected to sulphate attack”) to assess the potential severity of sulphate attack on concrete during its service life. The sulphate concentrations measured in the tested samples (about 0.02%) are below the exposure class of S-3 (Moderate) and may be considered negligible according to Table 2 of the MTO Gravity Pipe Design Guidelines (2014). Therefore, based on the two samples of soil tested, when the designer is selecting the exposure class for the structure, the effects of sulphates may not need to be considered. However, given that the structure is located above/adjacent to roadways/parking lots and will be exposed to de-icing salt, consideration should be given by the designed to designing for a ‘C’ type exposure class as defined by CSA A23.1 Table 1.

6.8.2 Potential for Corrosion

Based on the test results from the two soil samples the pH is about 7.7 and 7.8 and the resistivity values are 1,400 ohm-cm and 2,000 ohm-cm. According to the MTO Gravity Pipe Design Guidelines (2014), the pH is not considered detrimental to the buried steel. The resistivity is less than and equal to 2,000 ohm-cm, which indicates that the soil corrosiveness is severe ($R < 2,000$ ohm cm), as per Table 3.2 of the MTO *Gravity Pipe Design Guidelines* (2014).

6.9 Construction Considerations

6.9.1 Excavation and Control of Groundwater and Surface Water

Open-cut excavations must be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act and Regulation for Construction Activities (OHSA, O. Reg. 213). The existing site soils are generally classified as Type 3, for soil above the groundwater table, according to the OHSA. Below the groundwater table the site soils would be classified as Type 4 soil. Temporary excavations (i.e. those that are open for a relatively short time period) in Type 3 soil should be made with side slopes no steeper than 1H:1V and in Type 4 soil should be no steeper than 3H:1V.

It is expected that for construction staging, temporary protection systems will be required along the north and south sides of the excavations for the piers adjacent to the highway. Recommendations for temporary protection systems are provided in Section 6.9.3.

The groundwater level at the site is anticipated to be at about Elevation 101 m, and may be higher during wetter periods of the year. Depending on the elevation of the spread footings/pile caps, excavations at the piers may extend below the water level. It is expected that water inflow through the native cohesive soils and shale bedrock can be handled by pumping from well filtered sumps located outside the foundation footprint, although significant pumping capacity may be required for deeper excavations that extend completely through the sandy deposits or to the surface of the bedrock. If dewatering is required, it should be carried out in accordance with OPSS.PROV 517.

Surface water seepage into the excavations should be expected and will be heavier during periods of sustained precipitation and all surface water should be directed away from the excavations.

6.9.2 Bedrock Excavation

If the piers are supported on strip/spread footings founded on bedrock at the elevations provided in Section 6.4.1, bedrock excavation will be required to achieve the geotechnical resistances provided in Section 6.4.2. Bedrock excavation may likely be carried out using hoe-ramming techniques. Excavations in the bedrock may be made with near vertical side slopes/faces.

As noted in Section 6.6.1 the shale bedrock at the site is weak (i.e., calculated UCS in the range of 11 MPa to 21 MPa), but contains medium strong to very strong limestone interbeds. It is recommended that an NSSP be included in the Contract Documents to warn the Contractor of the bedrock characteristics, that excavation or drilling into the bedrock will require appropriate equipment and construction procedures, and that the bedrock excavation shall not disturb the existing bridge footings during construction staging (if required). An NSSP is provided in Appendix E for inclusion in the Contract Documents.

If drilled shafts are selected for foundations at this bridge site, during advancement the rock socket must be filled with water at all times, until concrete is placed in the rock socket using tremie methods. Following drilling of the rock sockets, the walls of the socket and the base will be covered by a “cake” of rock powder/mud from the drilling operations. Therefore, flushing and cleaning of the rock sockets walls and base are required and this requirement

must be included in the Contract Documents. For drilled shafts designed based on a combination of end-bearing resistance and a portion of the shaft resistance within the rock socket, the base of each rock socket must be thoroughly cleaned to remove all loose cuttings to ensure that the tremied concrete is in intimate contact with the competent shale bedrock. Upon completion of flushing and cleaning of the rock socket, the base will need to be inspected by means of a Shaft Inspection Device (SID) such as a video camera to confirm that the base is free of debris. For the caissons that are designed based on shaft resistance only, the rock socket walls of the caissons are required to be free from mud slurry; therefore, flushing and cleaning of the rock sockets walls are required, and this requirement must be included in the Contract Documents. Following acceptance of the rock socket by the Foundation Engineer, concrete must be placed using tremie methods within 6 hours of the final cleaning and within seven days from when the rock socket was completed (i.e., prior to cleaning). An NSSP for cleaning requirements and concreting of the caisson is provided in Appendix E. All drilled installation should be carried out in accordance with OPSS.PROV 903 (*Deep Foundations*), as amended by SP 109F57.

6.9.3 Temporary Protection Systems

Temporary protection systems will be required along the north and south side of the QEW lanes, along municipal roads and along adjacent properties to facilitate the staged removal of the existing bridges foundations and the construction of the new foundations.

The temporary protection systems should be designed and constructed in accordance with OPSS.PROV 539 (Temporary Protection Systems). The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS.PROV 539, provided that any existing adjacent structures or utilities can tolerate this magnitude of deformation. If excavations must be completed for removals in close proximity to the existing or new foundations, it is recommended that such protection systems meet Performance Level 1b as specified in OPSS.PROV 539.

To handle removal of existing pier footings, and for subexcavation and construction of new footings, the protection systems are required for an estimated excavation depth of up to approximately 6 m relative to the QEW grade at the site (depending on the footing elevations). It is considered that either a driven, interlocking sheet pile system or a soldier pile and lagging system would be suitable for the temporary excavation support, based on the subsurface soil and groundwater conditions. An interlocking sheet pile system would contribute to both ground control and, where applicable, groundwater/seepage control. For a soldier pile and lagging system, it would be necessary to control groundwater/seepage or include measures to mitigate loss of soil particles through the lagging boards.

The sheet piles or soldier piles would have to be driven to or socketed into bedrock to a sufficient depth to provide the necessary passive resistance for the retained soil height, including any surcharge loads behind the protection system within at least a 1H:1V zone relative to the base of the excavation. Lateral support to the sheet piles or soldier piles could be provided in the form of struts, rakers or temporary anchors depending on the size of the excavation made to accommodate one or more pier foundations or pile caps.

The selection and design of the protection system will be the responsibility of the Contractor.

6.9.4 Obstructions

The native soils at this site are loose/soft to very dense/hard and, where encountered, the residual soil and till above the bedrock surface contains rock fragments, particularly immediately above the bedrock, as noted on the Record of Borehole sheets, and the till deposit should be expected to contain cobbles and/or boulders, which could affect the installation of deep foundations, if this option is pursued. An NSSP should be included in the Contract

Documents to identify to the contractor the possible presence of rock fragments, or slabs in cobble/boulder sizes within the overburden soils or immediately above the bedrock; an example NSSP is provided in Appendix E.

6.9.5 Bedrock Subgrade Inspection and Protection

As discussed in Section 6.4.1, if consideration is given to founding the new spread/strip footings on the bedrock, some subexcavation or scaling may be required to expose competent bedrock. It is estimated that up to 1 m of subexcavation below the bedrock surface may be required to expose competent bedrock. The footing subgrade should be inspected in accordance with OPSS 902 (*Excavating and Backfilling Structures*) to check that all existing fill materials and fractured, softened or loosened portions of the shale bedrock are removed prior to construction of the footings for the piers.

The shale bedrock that will be exposed at the foundation subgrade level will be susceptible to weathering and/or disturbance from water and construction traffic. To limit this degradation, it is recommended that a concrete working slab be placed on the subgrade within four hours after preparation, inspection and approval of the footing subgrade. This requirement can be addressed with a note on the General Arrangement drawing and/or with an NSSP. An NSSP is provided in Appendix E for inclusion in the Contract Documents.

6.9.6 Vibration Monitoring During Construction

A maximum peak particle velocity (PPV) of 100 mm/s is generally considered applicable for bridge structures in good condition. Based on vibration monitoring experience, it is considered unlikely that vibrations induced by conventional construction activities such as hoe-ramming will reach this threshold level and, therefore, vibration monitoring for the existing bridge is not expected to be required during construction at this site.

Residential homes are located immediately adjacent to the southern ramp of the existing pedestrian bridge. A lower PPV threshold of 50 mm/s is generally considered applicable for vibration impacts on buildings. Therefore, it is recommended that vibration monitoring be carried out at the existing structures within 100 m the bridge site during bedrock excavating, temporary protection system installation and/or pile driving operations. An NSSP describing the requirements for vibration monitoring is presented in Appendix E.

7.0 CLOSURE

This report was prepared by Mr. Matthew Kelly, P.Eng. a Geotechnical Engineer with Golder. Ms. Lisa Coyne, P.Eng., Principal and MTO Foundations Designated Contact for Golder, conducted an independent technical and quality control review of this report.

Golder Associates Ltd.



Matthew Kelly, P.Eng.
Geotechnical Engineer



Lisa Coyne, P.Eng.
MTO Foundations Designated Contact, Principal

MWK/WC/LCC/ml

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[https://golderassociates.sharepoint.com/sites/19542g/1 foundations/09 - reports/02 - ogden bridge/4 - final/1530382-r-rev-0-fidr ogden ped bridge - 2021-11-22.docx](https://golderassociates.sharepoint.com/sites/19542g/1%20foundations/09%20-%20reports/02%20-%20ogden%20bridge/4%20-%20final/1530382-r-rev-0-fidr%20ogden%20ped%20bridge%20-%202021-11-22.docx)

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ASTM International:

- | | |
|------------|---|
| ASTM D1586 | Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils |
|------------|---|

Ontario Provisional Standard Drawing:

- | | |
|---------------|--|
| OPSD 3000.100 | Foundation, Piles, Steel H-Pile Driving Shoe |
| OPSD 3000.100 | Foundation, Piles, Tube Pile Driving Shoe |
| OPSD 3090.101 | Foundation Frost Penetration Depths for Southern Ontario |

Ontario Provincial Standard Specification:

- | | |
|----------|--|
| OPSS 539 | Construction Specification for Temporary Protection Systems |
| OPSS 902 | Construction Specification for Excavating and Backfilling Structures |
| OPSS 903 | Construction Specification for Deep Foundations |

Ontario Water Resources Act:

- | | |
|------------------------|--------------------|
| Ontario Regulation 903 | Wells (as amended) |
|------------------------|--------------------|

Ontario Occupational Health and Safety Act:

- | | |
|---------------------------|------------------------------------|
| Ontario Regulation 213/91 | Construction Projects (as amended) |
|---------------------------|------------------------------------|

TABLES

Table 1: Comparison of Foundation Alternatives

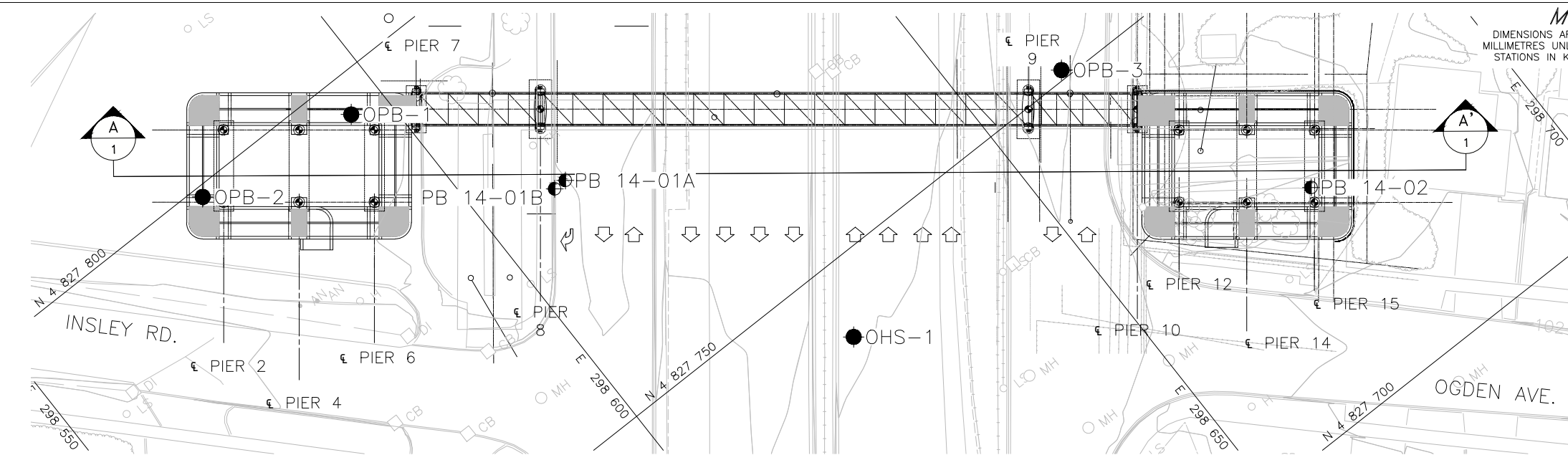
Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Caissons socketed into shale bedrock	<ul style="list-style-type: none"> Feasible for support of piers. 	<ul style="list-style-type: none"> Higher bearing resistances than for steel H-piles, possibly requiring fewer elements. Pile caps could be eliminated reducing excavation depth and associated protection system requirements. Minor groundwater seepage anticipated if pile cap constructed below ground surface, so pumping from filtered sumps will provide adequate groundwater control. 	<ul style="list-style-type: none"> Temporary or permanent liners would be required during construction to control potential ground losses and to mitigate for groundwater inflow. Shale bedrock is very weak to weak with medium strong to very strong limestone layers, so more expensive coring or churn drilling required to form bedrock socket through these layers. The rock socket is required to be cleaned using airlift methods and if the design relies on resistance from the base then inspection with a video camera would be required. Concrete would have to be placed by tremie methods below the water level. 	<ul style="list-style-type: none"> Conventional construction methods for caisson foundations; temporary liners required for ground and groundwater control. Could eliminate or significantly reduce the temporary protection system requirements. Limited dewatering required (assuming tremie concrete placement). Reduced spoil volumes compared to shallow foundations supported on native soil or bedrock. Less working room required and less potential impact on highway operations or adjacent properties. Reduced vibrations compared to pile driving. 	<ul style="list-style-type: none"> Potentially higher cost than driven piles.

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Steel H-piles or pipe piles founded on shale bedrock	<ul style="list-style-type: none"> Feasible for support of the piers. 	<ul style="list-style-type: none"> Pile caps could be maintained higher than shallow footings founded native soil, reducing excavation depth and associated protection system requirements. Minor groundwater seepage anticipated in pile cap excavations, so pumping from filtered sumps will likely provide adequate groundwater control. Higher geotechnical resistance available than for shallow foundations. 	<ul style="list-style-type: none"> Temporary protection systems likely required along edges of QEW and adjacent properties to facilitate excavation to pile cap level. Larger/specialized equipment required for installation of piles than for construction of shallow foundations. Driving piles in close proximity to residential homes will have an impact on those neighborhood residents. 	<ul style="list-style-type: none"> Conventional construction methods for H-pile. Vibration mitigation at the south end (adjacent to residential properties) may be difficult. 	<ul style="list-style-type: none"> May be equal to or modestly higher cost than shallow foundations on bedrock but potentially lower cost than caissons.

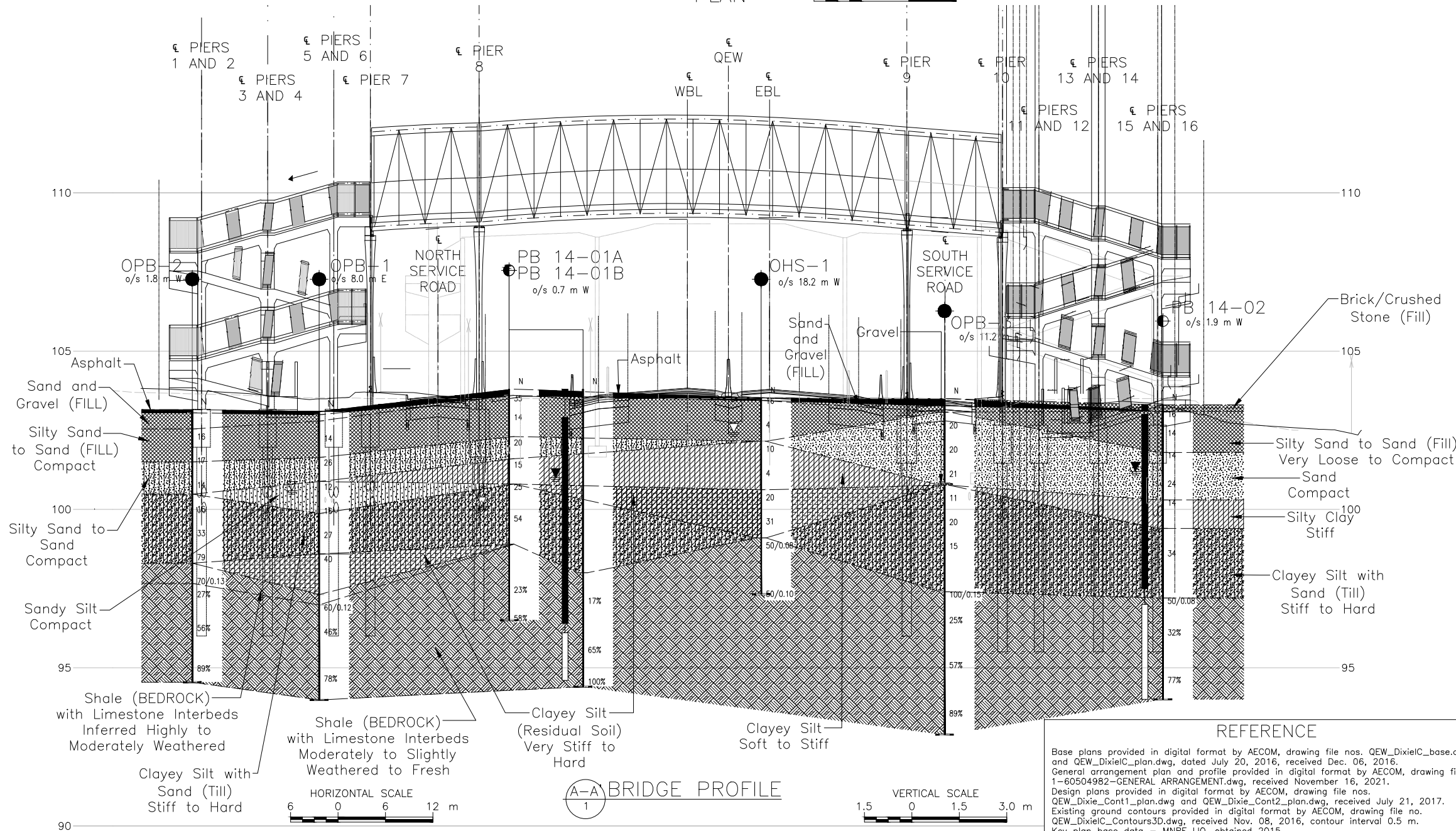
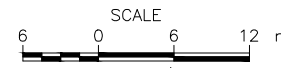
Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Spread/strip footings founded on native soils	<ul style="list-style-type: none"> Feasible for pier foundations; however, requires temporary protection for staged construction. 	<ul style="list-style-type: none"> Suitable founding strata at modest depths (~ 3 m). Design is consistent with existing piers which are supported on shallow foundations and have performed well. 	<ul style="list-style-type: none"> Temporary protection systems required along edges of QEW and adjacent properties. Protection system may be required to have a high degree of stiffness in order to not compromise the soils beneath the existing footings, depending on construction staging and footing elevations. Excavations will extend below groundwater level in sandy soils and dewatering effort may be challenging. Lower bearing geotechnical resistances compared to deep foundation options or spread footings founded on bedrock and may not be sufficient for higher loads from new bridge. 	<ul style="list-style-type: none"> Conventional excavation and construction techniques. Extensive temporary protection systems required due to site constraints. Limited working room for installation of temporary protection systems and subsequent excavation. Construction dewatering may be a challenge. 	<ul style="list-style-type: none"> Typically lower cost than other options, but the difference may be much less than typical due to site constraints.

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Spread/strip footings founded on competent shale bedrock	<ul style="list-style-type: none"> Feasible for support of the piers; however, requires temporary protection for staged construction. 	<ul style="list-style-type: none"> Existing piers are supported on shallow foundations, and have performed well. Higher geotechnical resistance than for shallow foundations bearing on native soil deposits 	<ul style="list-style-type: none"> Temporary protection systems required along edges of QEW and adjacent properties. Excavations will extend below groundwater level in sandy soils and significant dewatering effort may be required. Protection system may be required to have a high degree of stiffness in order to not comprise the soils beneath the existing footings depending on construction staging and footing elevations. Greater volume of excavation spoil and concrete for pier construction. 	<ul style="list-style-type: none"> Conventional excavation and construction techniques. Limited working room for installation of temporary protection systems and subsequent excavation. Construction dewatering is likely to be a challenge. 	<ul style="list-style-type: none"> Higher cost than shallow foundations on native soils and may approach or equal costs for deep foundations.

DRAWINGS



PLAN



A-A BRIDGE PROFILE

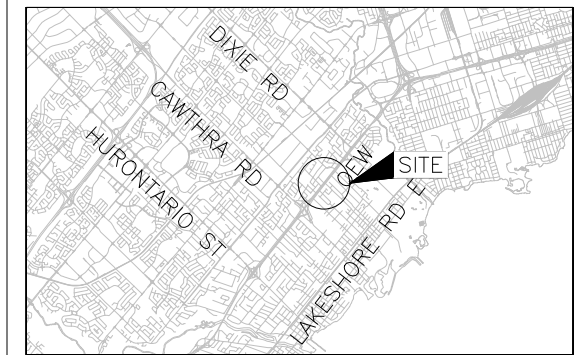
REFERENCE

Base plans provided in digital format by AECOM, drawing file nos. QEW_DixieC_base.dwg and QEW_DixieC_plan.dwg, dated July 20, 2016, received Dec. 06, 2016.
General arrangement plan and profile provided in digital format by AECOM, drawing file no. 1-60504982-GENERAL ARRANGEMENT.dwg, received November 16, 2021.
Design plans provided in digital format by AECOM, drawing file nos. QEW_Dixie_Cont1_plan.dwg and QEW_Dixie_Cont2_plan.dwg, received July 21, 2017.
Existing ground contours provided in digital format by AECOM, drawing file no. QEW_DixieC_Contours3D.dwg, received Nov. 08, 2016, contour interval 0.5 m.
Key plan base data - MNR/LIO, obtained 2015.

METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No. 2021-2127
GWP No. 2012-13-00

OGDEN AVENUE PEDESTRIAN BRIDGE
BOREHOLE LOCATIONS AND SOIL
STRATA



KEY PLAN



LEGEND

- Borehole - Current Investigation
- Borehole - Previous Investigation (2014) (Geocres No. 30M11-253)
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- WL in piezometer
- WL upon completion of drilling

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
OHS-1	103.5	4827742.0	298624.3
OPB-1	103.1	4827795.8	298595.9
OPB-2	103.2	4827798.8	298577.3
OPB-3	103.5	4827751.0	298660.7
PB 14-01A	103.8	4827775.4	298610.0
PB 14-01B	103.8	4827775.4	298608.5
PB 14-02	103.3	4827723.6	298674.4



NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

NO.	DATE	BY	REVISION
Geocres No. 30M11-310			
HWY. QEW		PROJECT NO. 1530382	DIST. CENTRAL
SUBM'D. MWK	CHKD. MWK/KN	DATE: 11/22/2021	SITE: 24X-0192
DRAWN: MR/JM	CHKD. LCC	APPD. LCC	DWG. 1

APPENDIX A

**Previous Investigation –
GEOCRES No. 30M11-253**

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


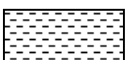

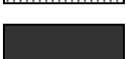

ROCK WEATHERING CLASSIFICATION

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		103							0 77 16 7
			2	SS	14									
102.3														
1.5	SAND, some silt, trace clay Compact Brown Wet		3	SS	20		102							
101.6														
2.2	Silty CLAY, trace sand Stiff Grey Wet		4	SS	15		101							0 6 45 49
100.8														
3.0	Silty CLAY with SAND, trace gravel Very Stiff to Hard Grey Wet (TILL)		5	SS	25		100							
			6	SS	54									0 39 36 25
98.9							99							
4.9	SHALE BEDROCK, slightly weathered, thinly bedded, very weak, strong to very strong limestone interbeds, grey (Georgian Bay Formation)						98							RUN #1 TCR=100% SCR=48% RQD=23%
	Very strong limestone interbeds (50mm) at 6.5m and 6.7m		1	RUN										
	Horizontal joint (25mm) at 7.2m						97							
	Clay seam (25mm) at 7.2m		2	RUN										RUN #2 TCR=100% SCR=83% RQD=58%
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.													

+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

METRIC

SOIL PROFILE						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES	"N" VALUES	GROUND WATER CONDITIONS	
<div>DYNAMIC CONE PENETRATION RESISTANCE PLOT<div><div></div><div>20406080100</div></div></div> <div><div>SHEAR STRENGTH kPa</div><div>○ UNCONFINED + FIELD VANE● QUICK TRIAXIAL × LAB VANE</div></div> <div><div>PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT</div><div>w_P w w_L</div><div>WATER CONTENT (%)</div><div>204060</div></div>						
<div>ELEVATION SCALE</div> <div>UNIT WEIGHT γ kN/m³</div> <div>REMARKS & GRAIN SIZE DISTRIBUTION (%)GR SA SI CL</div>						
103.8	GROUND SURFACE					
0.0	Augered to 5.8m, then start coring For soil stratigraphy refer to Borehole PB14-01A					
98.0						
5.8	SHALE BEDROCK , moderately to slightly weathered, thinly bedded, weak, occasional strong to very strong limestone interbeds, grey (Georgian Bay Formation) Clay seams at 6.1m, 7.1m, 7.2m and 7.3m Strong to very strong limestone interbeds: 50mm at 6.7m 50mm at 8.1m 200mm at 8.4m		1 RUN			
			2 RUN			
			3 RUN			
94.4						
9.4	END OF BOREHOLE AT 9.4m. BOREHOLE OPEN TO 9.4m AND WATER LEVEL AT 2.2m. Piezometer installation consists of					

(%) 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	W P	W	W L	WATER CONTENT (%)		
	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	Crushed Stone: (125mm)												
0.2	SAND, trace gravel, some silt, trace clay Compact Dark Brown Dry (FILL)		1	SS	16		103						
			2	SS	14		102						0 72 20 8
101.8							102						
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		97						RUN #1 TCR=100% SCR=57% RQD=32% UCS=9.7MPa Shale UCS=152MPa Limestone
			2	RUN			96						RUN #2 TCR=100% SCR=97% RQD=77% UCS=12.4MPa Shale
94.0							95						
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

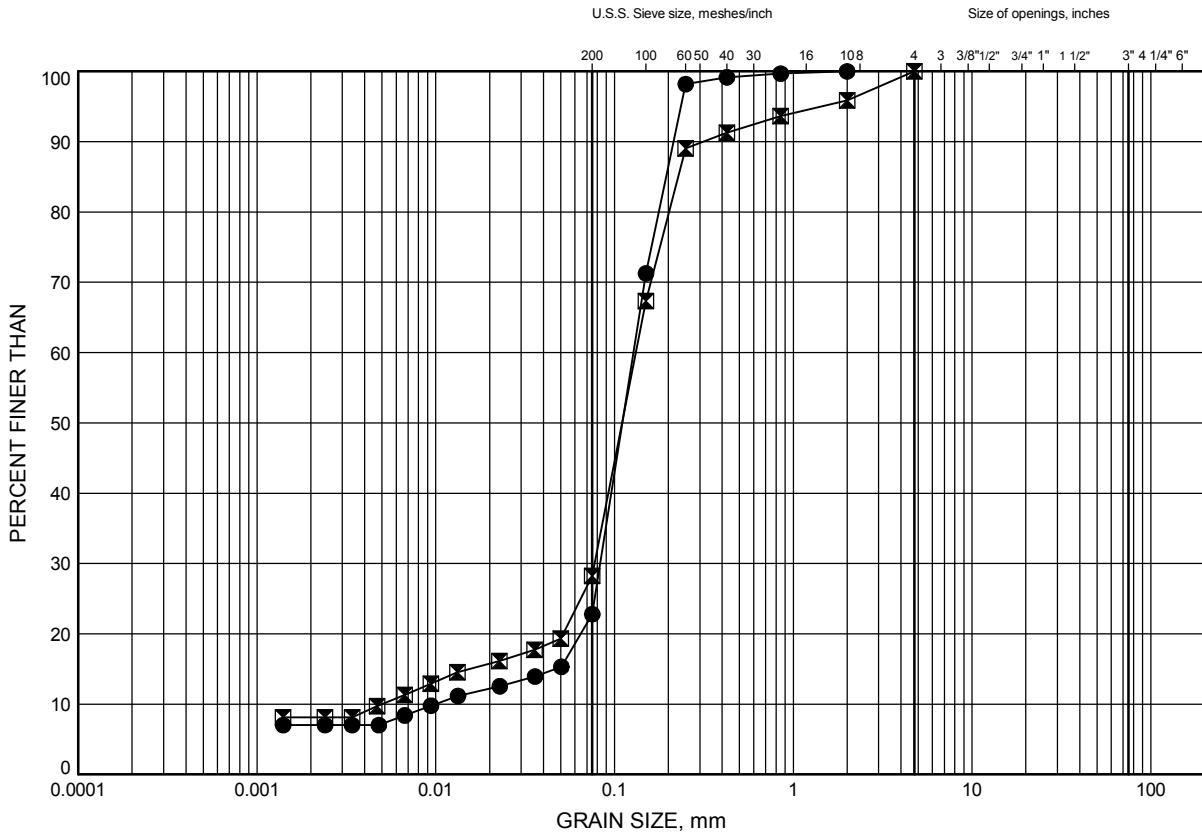
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Appendix B
Laboratory Test Results
Soil and Rock Samples

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

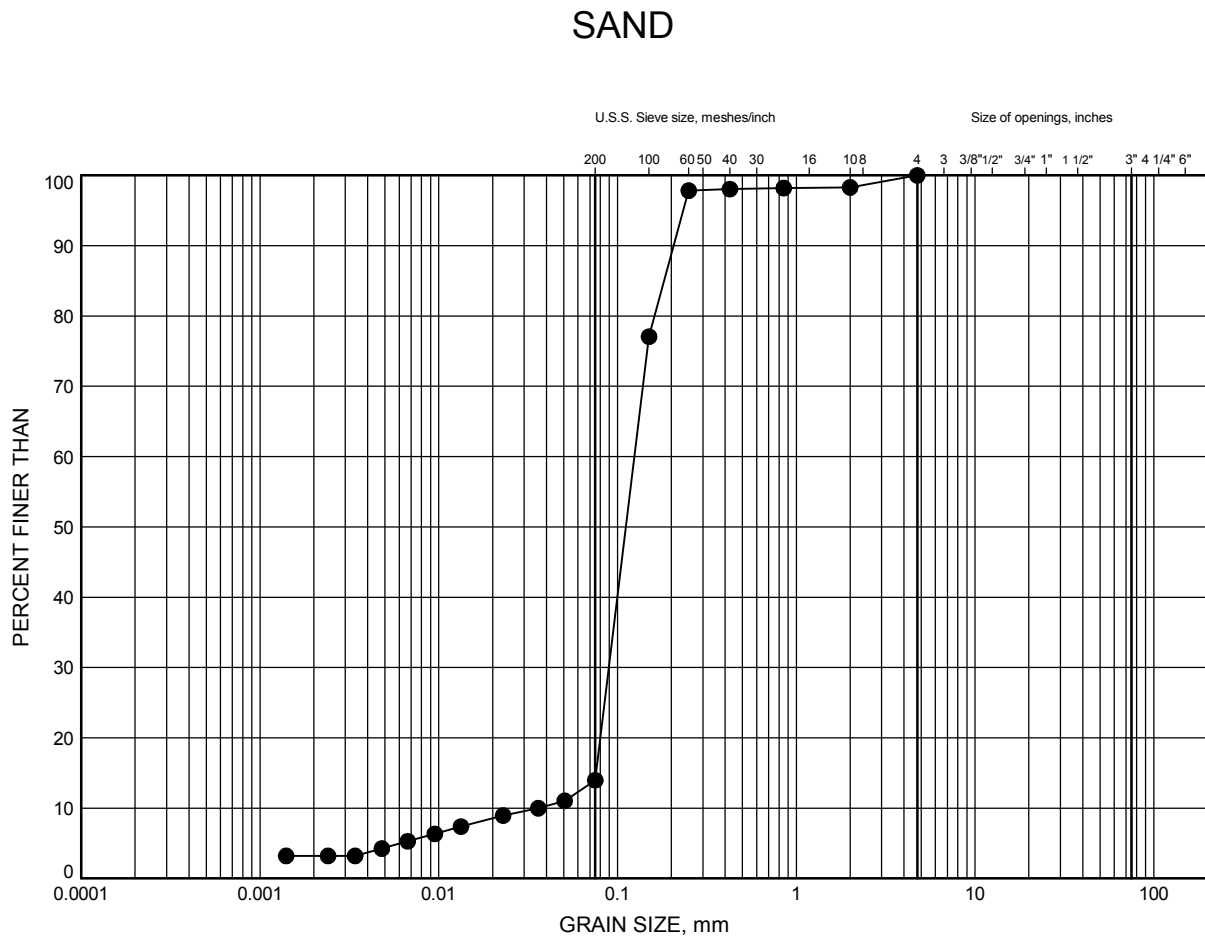
Date January 2015
W.P. 09-20003



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

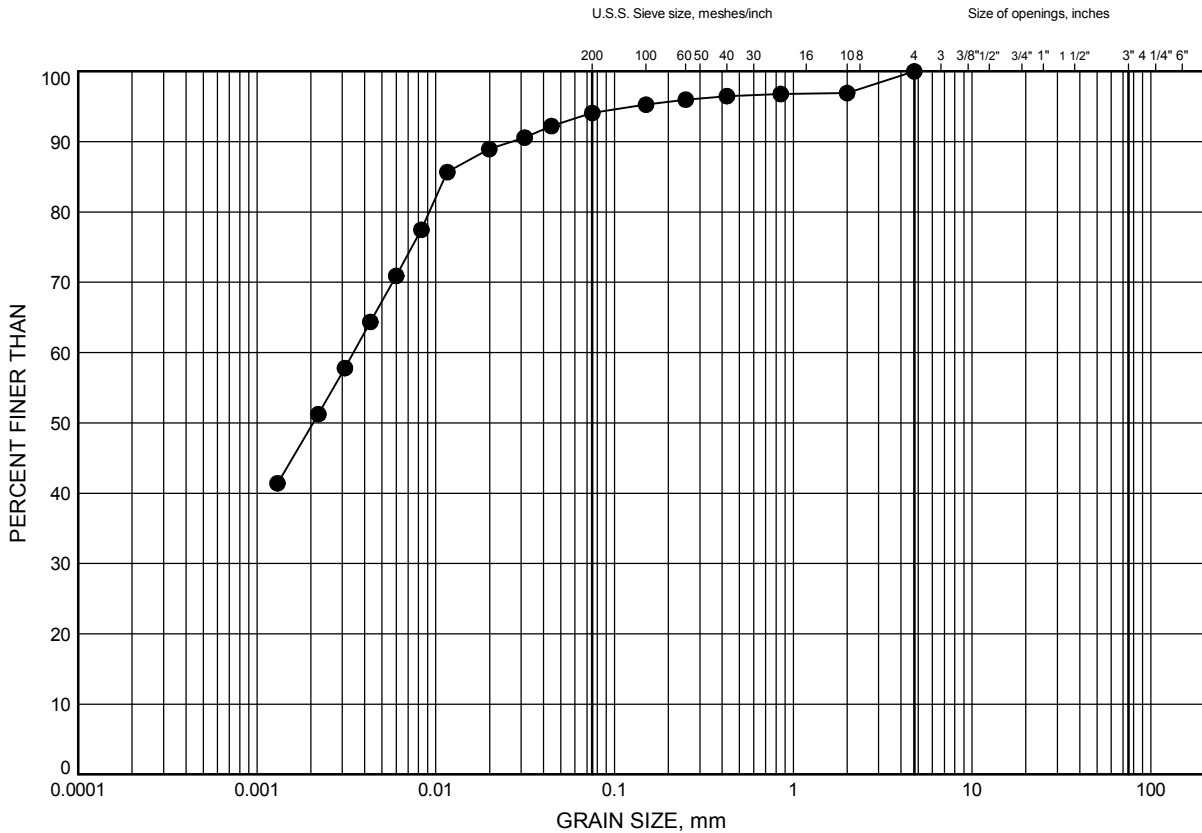


Prep'd MFA
Chkd. AMP

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

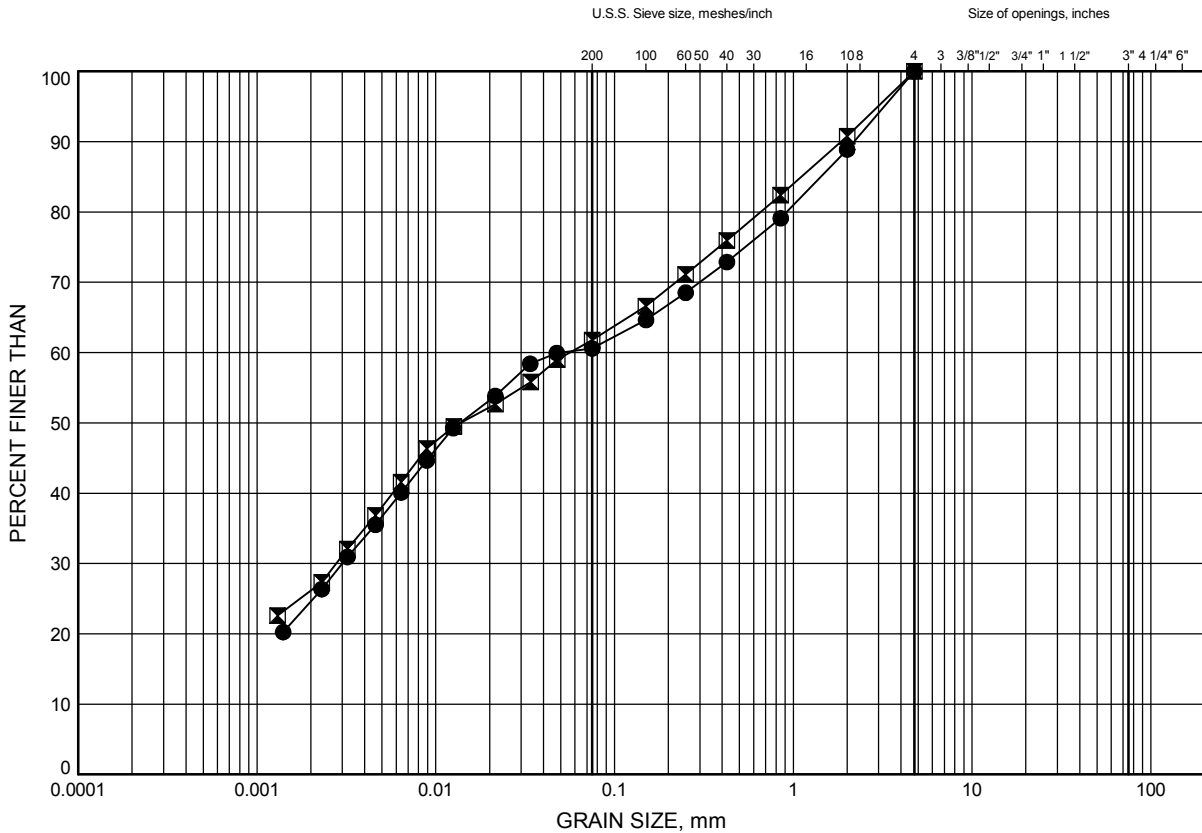


Prep'd MFA
Chkd. AMP

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



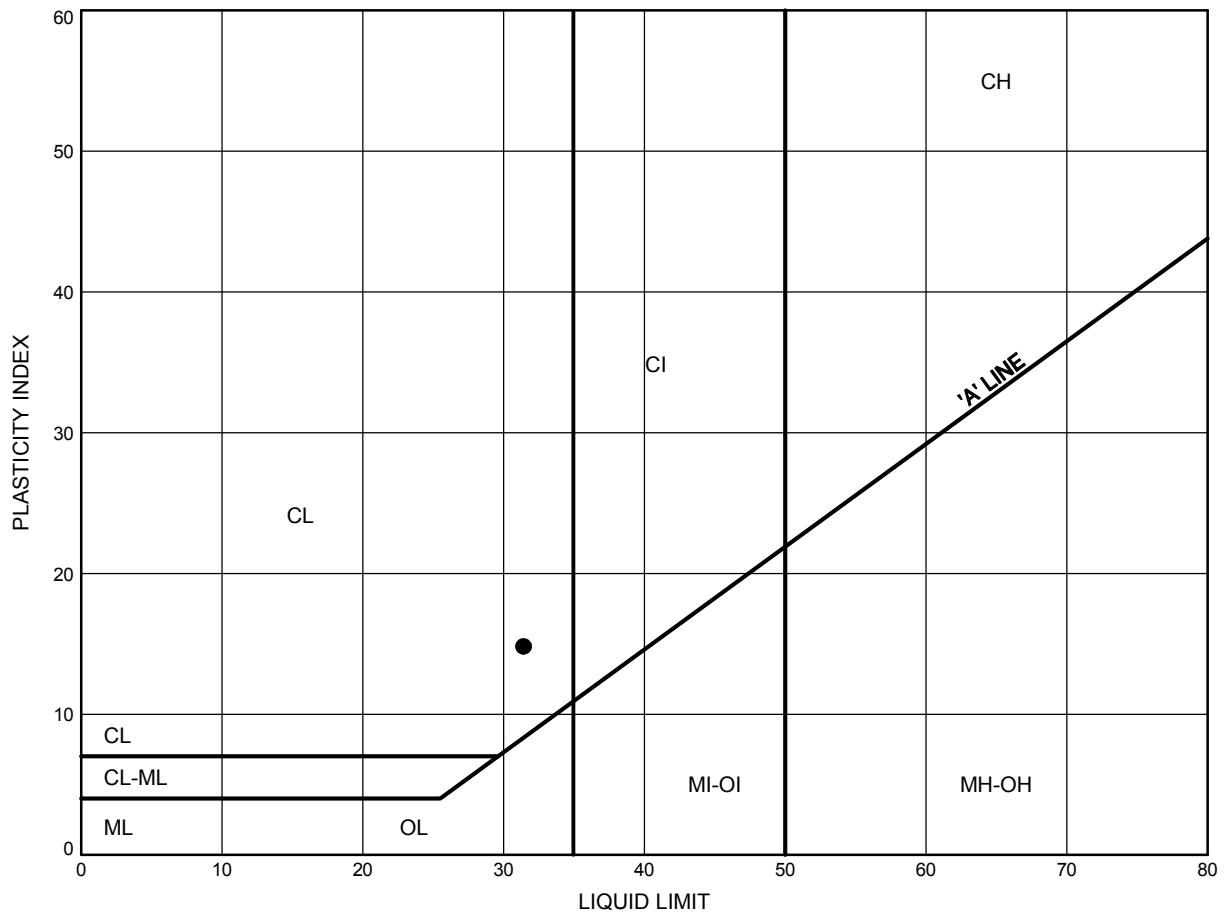
Prep'd MFA
Chkd. AMP

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



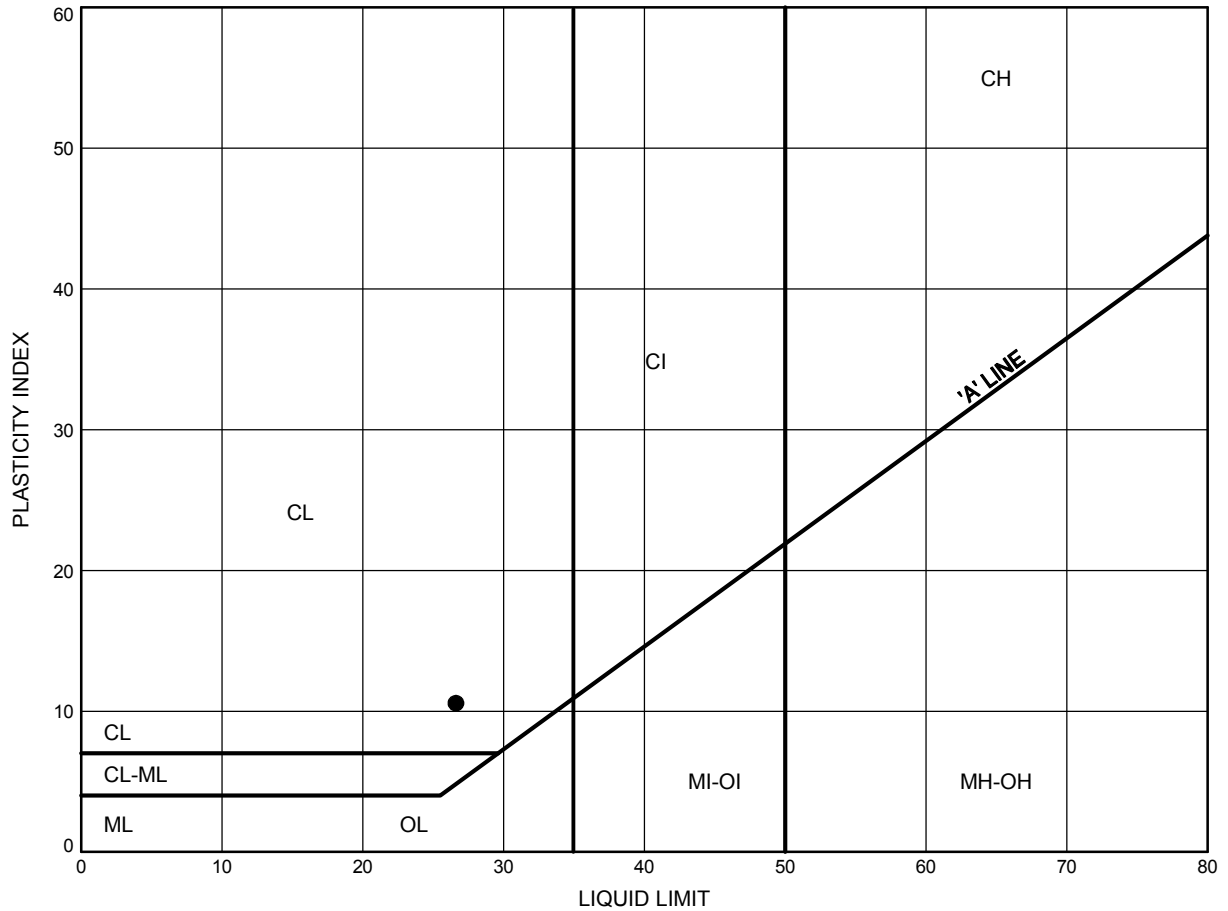
Prep'd MFA
Chkd. AMP

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

Project Name : QEW CAWTHRA ROAD
PEDESTRIAN BRIDGE

Date Drilled : 9 Dec, 2014

Core Size : NQ BH No : PB14-01B

Date Tested : 15 Dec, 2014

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									
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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

APPENDIX B

**Borehole Records and
Bedrock Core Photographs –
2016 to 2020 Investigation**

ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS

MINISTRY OF TRANSPORTATION, ONTARIO

PARTICLE SIZES OF CONSTITUENTS

Soil Constituent	Particle Size Description	Millimetres	Inches (US Std. Sieve Size)
BOULDERS	Not Applicable	>200	>8
COBBLES	Not Applicable	75 to 200	3 to 8
GRAVEL	Coarse Fine	19 to 75 4.75 to 19	0.75 to 3 (4) to 0.75
SAND	Coarse Medium Fine	2.00 to 4.75 0.425 to 2.00 0.075 to 0.425	(10) to (4) (40) to (10) (200) to (40)
FINES	Classified by plasticity	<0.075	< (200)

MODIFIERS FOR SECONDARY COMPONENTS^{1,2}

Percentage by Mass	Modifier
> 35	Use 'and' to combine primary and secondary component (<i>i.e.</i> , SAND and gravel)
> 20 to 35	Primary soil name prefixed with "gravelly, sandy" as applicable
> 10 to 20	some (<i>i.e.</i> , some sand)
≤ 10	trace (<i>i.e.</i> , trace fines)

1. Only applicable to components not described by Primary Group Name.

2. Classification of Primary Group Name based on Unified Soil Classification System (ASTM D2487) for coarse-grained soils; fine-grained soils described per current MTO Soil Classification System.

PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) split-spoon sampler for a distance of 300 mm (12 in.). Values reported are as recorded in the field and are uncorrected.

Cone Penetration Test (CPT)

An electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (*q_t*), porewater pressure (*u*) and sleeve friction (*f_s*) are recorded electronically at 25 mm penetration intervals.

Dynamic Cone Penetration Resistance (DCPT); N_d:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

SAMPLES

AS	Auger sample
BS	Block sample
CS	Chunk sample
DD	Diamond Drilling
DO or DP	Seamless open ended, driven or pushed tube sampler – note size
DS	Denison type sample
GS	Grab Sample
MC	Modified California Samples
MS	Modified Shelby (for frozen soil)
RC / SC	Rock core / Soil core
SS	Split spoon sampler – note size
ST	Slotted tube
TO	Thin-walled, open – note size (Shelby tube)
TP	Thin-walled, piston – note size (Shelby tube)
WS	Wash sample
OD / ID	Outer Diameter / Inner Diameter
HSA / SSA	Hollow-Stem Augers / Solid-Stem Augers

SOIL TESTS

w	water content
PL, w _p	plastic limit
LL, w _L	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D _R	relative density (specific gravity, G _s)
DS	direct shear test
GS	specific gravity
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO ₄	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V (FV)	field vane (LV-laboratory vane test)
Y	unit weight

1. Tests anisotropically consolidated prior to shear are shown as CAD, CAU.

COARSE-GRAINED SOILS

Compactness¹

Term	SPT 'N' (blows/0.3m) ²
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	> 50

- Definition of compactness terms are based on SPT 'N' ranges as provided in Terzaghi, Peck and Mesri (1996). Many factors affect the recorded SPT 'N' value, including hammer efficiency (which may be greater than 60% in automatic trip hammers), overburden pressure, groundwater conditions, and grain size. As such, the recorded SPT 'N' value(s) should be considered only an approximate guide to the soil compactness. These factors need to be considered when evaluating the results, and the stated compactness terms should not be relied upon for design or construction.
- SPT 'N' in accordance with ASTM D1586, uncorrected for the effects of overburden pressure.

FINE-GRAINED SOILS

Consistency

Term	Undrained Shear Strength (kPa)	SPT 'N' ^{1,2} (blows/0.3m)
Very Soft	< 12	0 to 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	> 200	> 30

- SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects; approximate only.
- SPT 'N' values should be considered ONLY an approximate guide to consistency; for sensitive clays (e.g., Champlain Sea clays), the N-value approximation for consistency terms does NOT apply. Rely on direct measurement of undrained shear strength or other manual observations.

Field Moisture Condition

Term	Description
Dry	Soil flows freely through fingers.
Moist	Soils are darker than in the dry condition and may feel cool.
Wet	As moist, but with free water forming on hands when handled.

LIST OF SYMBOLS

MINISTRY OF TRANSPORTATION, ONTARIO

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π	3.1416
$\ln x$	natural logarithm of x
\log_{10}	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
FoS	factor of safety

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)

σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight)*
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

(a) Index Properties (continued)

w	water content
w_L or LL	liquid limit
w_P or PL	plastic limit
I_P or PI	plasticity index $= (w_L - w_P)$
NP	non-plastic
w_s	shrinkage limit
I_L	liquidity index $= (w - w_P) / I_P$
I_c	consistency index $= (w_L - w) / I_P$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index $= (e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
$C_{a(e)}$	secondary compression index
C_a	rate of secondary compression
$C_{a(e)}$	modified secondary compression index
m_v	coefficient of volume change
C_v	coefficient of consolidation (vertical direction)
C_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
c'	effective cohesion
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction $= \tan \delta$
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q or q'	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

* Density symbol is ρ . Unit weight symbol is γ .
where $\gamma = \rho \cdot g$ (i.e., mass density multiplied by
acceleration due to gravity)

Notes: 1
2

$\tau = c' + \sigma' \tan \phi'$
shear strength = (compressive strength)/2

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING CLASSIFICATION

Fresh (W1): no visible sign of rock material weathering.

Slightly Weathered (W2): discoloration indicates weathering of rock mass material on discontinuity surfaces. **Less than 5%** of rock mass is altered or weathered.

Moderately Weathered (W3): less than 50% of the rock mass is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.

Highly Weathered (W4): more than 50% of the rock mass is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.

Completely Weathered (W5): 100% of the rock mass is decomposed and/or disintegrated to a soil. The original mass structure is still largely intact.

Residual Soil (W6): all rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.

BEDDING THICKNESS

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

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	Greater than 3 m
Wide	1 m to 3 m
Moderately close	0.3 m to 1 m
Close	50 mm to 300 mm
Very close	Less than 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	Greater than 60 mm
Coarse Grained	2 mm to 60 mm
Medium Grained	60 microns to 2 mm
Fine Grained	2 microns to 60 microns
Very Fine Grained	Less than 2 microns

Note: * Grains greater than 60 microns diameter are visible to the naked eye

CORE CONDITION

Total Core Recovery (TCR)

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, as measured along the centerline axis of the core, relative to the length of the total core run. RQD varies from 0% for completely broken core to 100% for core in solid segments.

DISCONTINUITY DATA

Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

Dip with Respect to Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole, a discontinuity with a 90° angle is horizontal.

Description and Notes

An abbreviation description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

AXJ Axial Joint	KV Karstic Void
BD Bedding	K Slickensided
BC Broken Core	LC Lost Core
CC Continuous Core	MB Mechanical Break
CL Closed	PL Planar
CO Contact	PO Polished
CU Curved	RO Rough
CT Coated	SA Slightly Altered
FLT Fault	SH Shear
FOL Foliation	SM Smooth
FR Fracture	SR Slightly Rough
GO Gouge	SY Stylolite
IN Infilled	UN Undulating
IR Irregular	VN Vein
JN Joint	VR Very Rough

ISRM Intact Rock Material Strength Classification

Grade	Description	Approx. Range of Uniaxial Compressive Strength (MPa)
R0	Extremely weak rock	0.25 – 1.0
R1	Very weak rock	1.0 – 5.0
R2	Weak rock	5.0 – 25
R3	Medium strong rock	25 – 50
R4	Strong rock	50 -100
R5	Very strong rock	100 -250
R6	Extremely strong rock	>250

FIELD ESTIMATION OF ROCK HARDNESS

Grade	Description	Field Identification	Approx. Range of UCS (MPa)
R0	Extremely Weak Rock	Indented by thumbnail	0.25 - 1
R1	Very Weak Rock	Material can be peeled or shaped with a knife. Crumbles under firm blows from geological hammer.	1 - 5
R2	Weak Rock	Knife cuts material but too hard to shape into triaxial specimens or material can be peeled with a knife with difficulty. Shallow (<5mm) indentations made by firm blows from pick of a geological hammer.	5 - 25
R3	Moderately Strong Rock	Cannot be peeled or scraped with a knife. Hand held specimens can be fractured with single firm blow of geological hammer.	25 - 50
R4	Strong Rock	Hand held specimen requires more than one blow of geological hammer to fracture.	50 - 100
R5	Very Strong Rock	Hand held specimen requires many blows of geological hammer to fracture.	100 - 250
R6	Extremely Strong Rock	Specimen can only be chipped under repeated hammer blows, rings when hit.	> 250

Notes:

1. Hand held specimens should have height approximately 2 times the diameter.
2. Materials having a uniaxial compressive strength of less than approximately 0.5 MPa and cohesionless materials should be classified using soil classification systems.
3. Rocks with a uniaxial compressive strength below 25 MPa (i.e. below R2) are likely to yield highly ambiguous results under point load testing.

Reference:

Brown, 1981. "Suggested Methods for Rock Characterization Testing and Monitoring", International Society for Rock Mechanics.

Hoek, E., Kaiser, P.K., Bawden, W.F., 1995. "Support of Underground Excavations in Hard Rock", Balkema, Rotterdam.

ROCK WEATHERING CLASSIFICATION

Term	Symbol	Description	Discoloration Extent	Fracture Condition	Surface Characteristics
Residual soil	W6	All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.	Throughout	N/A	Resembles soil
Completely weathered	W5	100% of rock material is decomposed and/or disintegrated to soil. The original mass structure is still largely intact.	Throughout	Filled with alteration minerals	Resembles soil
Highly weathered	W4	More than 50% of the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.	Throughout	Filled with alteration minerals	Friable and possibly pitted
Moderately weathered	W3	Less than 50% of the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones. Visible texture of the host rock still preserved. Surface planes are weathered (oxidized or carbonate filling) even when breaking the "intact rock".	>20% of fracture spacing on both sides of fracture	Discoloured, may contain thick filling	Partial to complete discoloration, not friable except poorly cemented rocks
Slightly weathered	W2	Discoloration indicates weathering of rock material on discontinuity surfaces (usually oxidized). Less than 5% of rock mass altered.	<20% of fracture spacing on both sides of fracture	Discoloured, may contain thin filling	Partial discoloration
Fresh	W1	No visible sign of rock material weathering.	None	Closed or discoloured	Unchanged

Reference:

Brown, 1981. "Suggested Methods for Rock Characterization Testing and Monitoring", International Society for Rock Mechanics.

PROJECT		1530382		RECORD OF BOREHOLE No OHS-1		SHEET 1 OF 1		METRIC						
G.W.P.		2102-13-00; 2432-13-00		LOCATION		N 4827742.0; E 298624.3 MTM NAD 83 ZONE 10 (LAT. 43.589586; LONG. -79.576480)		ORIGINATED BY						
DIST		Central HWY QEW		BOREHOLE TYPE		108 mm O.D. Continuous Flight Solid Stem Augers		COMPILED BY						
DATUM		Geodetic		DATE		September 13 and 14, 2016		CHECKED BY						
								SMM						
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
103.5	GROUND SURFACE							20 40 60 80 100	20 40 60 80 100	10 20 30				
0.0	ASPHALT (80 mm)													
0.1	Silty sand, trace gravel (FILL) Very loose to compact Dark brown/brown Dry to wet		1	SS	16									
			2	SS	4									
102.1														
1.4	CLAYEY SILT, trace to some sand Soft to stiff Grey Moist		3	SS	10									
			4	SS	4									
100.6														
2.9	CLAYEY SILT, some sand, trace gravel, trace shale fragments (RESIDUAL SOIL) Very stiff Brown/Grey Moist to wet		5	SS	20									
			6	SS	31									
99.1														
4.4	SHALE (BEDROCK) Grey		7	SS	50/0.08									
97.3														
6.2	END OF BOREHOLE		8	SS	50/0.10									
	NOTE: 1. Water in open borehole at a depth of 1.1 m below ground surface (Elev. 102.4 m) upon completion of drilling.													

GTA-MTO 001 S:\CLIENTS\MTO\QEW-DIXIE\02 DATA\GINT\QEW-DIXIE.GPJ GAL-GTA.GDT 3/31/21

+³, ×³: Numbers refer to Sensitivity ○^{3%} STRAIN AT FAILURE

PROJECT <u>1530382</u>	RECORD OF BOREHOLE No OPB-1	SHEET 2 OF 1	METRIC
G.W.P. <u>2102-13-00; 2432-13-00</u>	LOCATION <u>N 4827795.8; E 298595.9 MTM NAD 83 ZONE 10 (LAT. 43.596105; LONG. -79.569451)</u>	ORIGINATED BY <u>KN</u>	
DIST <u>Central</u> HWY <u>QEW</u>	BOREHOLE TYPE <u>200 mm O.D. Hollow Stem Augers</u>	COMPILED BY <u>CC</u>	
DATUM <u>Geodetic</u>	DATE <u>February 11 to 12, 2020</u>	CHECKED BY <u>SMM</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W _p	W	W _L		GR	SA	SI	CL	
-- CONTINUED FROM PREVIOUS PAGE --					○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					WATER CONTENT (%)											
								20	40	60	80	100	10	20	30						
	NOTES: 1. Borehole open and dry to a depth of 6.1 m below ground surface (Elev. 97.0 m) on February 11, 2020. 2. Water level measured at a depth of 2.5 m below ground surface (Elev. 100.6 m) on February 12, 2020, prior to rock coring.																				

GTA-MTO 001 S:\CLIENTS\MTQEQW-DIXIE\02_DATA\INTQEW-DIXIE.GPJ GAL-GTA.GDT 3/31/21

[illegible]

FEATURES LEGEND



BROKEN CORE



CLAY SEAM



LIMESTONE



LOST CORE

DEPTH SCALE

1 : 50



GOLDER

LOGGED: KN

CHECKED: AK

PROJECT		RECORD OF BOREHOLE No OPB-2				SHEET 1 OF 1		METRIC							
G.W.P.		2102-13-00; 2432-13-00		LOCATION		N 4827798.8; E 298577.3 NAD 83 MTM ZONE 10 (LAT. 43.599621; LONG. -79.565717)		ORIGINATED BY		KN					
DIST		Central HWY QEW		BOREHOLE TYPE		200 mm O.D. Hollow Stem Augers		COMPILED BY		CC					
DATUM		Geodetic		DATE		February 12, 2020		CHECKED BY		SMM					
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					WATER CONTENT (%)		
								20 40 60 80 100							
103.2	GROUND SURFACE														
0.0	ASPHALT (100 mm)														
0.1	Sand and gravel (FILL) Brown Moist						103								
102.6															
0.6	Sand, trace to some silt (FILL) Compact Orange-brown Moist		1	SS	16		102								
101.5															
1.7	Silty SAND Compact Grey Moist		2A	SS	17		101								
			2B												
			3A	SS	14										
100.5			3B												
2.7	CLAYEY SILT with SAND, some gravel, silt pockets (TILL) Stiff to hard Grey Moist						100								
			4	SS	16										
			5	SS	33		99								
			6A	SS	79										
98.3			6B				98								
4.9	Inferred highly to moderately weathered, grey, SHALE (BEDROCK)														
97.7			7	SS	70/0.13										
5.5	SHALE (BEDROCK) Grey Slightly weathered		1	RC	REC 89%		97						RQD = 27%		
	Bedrock cored between depths of 5.5 m and 8.6 m. For rock coring details refer to Record of Drillhole OPB-2.		2	RC	REC 100%		96						RQD = 56%		
			3	RC	REC 100%		95						RQD = 89%		
94.6															
8.6	END OF BOREHOLE														
	NOTE: 1. Borehole open and dry prior to rock coring.														

PROJECT: 1530382

RECORD OF DRILLHOLE: OPB-2

SHEET 1 OF 1

LOCATION: N 4827798.8 ; E 298577.3

DRILLING DATE: February 12, 2020

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: D120 (Truck Mounted)

DRILLING CONTRACTOR: Walker Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	JN - Joint FLT - Fault SHR- Shear VN - Vein CJ - Conjugate BD- Bedding FO- Foliation CO- Contact OR- Orthogonal CL - Cleavage PL - Planar CU- Curved UN- Undulating ST - Stepped IR - Irregular PO- Polished K - Slickensided SM- Smooth Ro - Rough MB- Mechanical Break BR - Broken Rock NOTE: For additional abbreviations refer to list of abbreviations & symbols.																FEATURES	RQ/R1 ZONES	NOTES PIEZOMETER OR STANDPIPE INSTALLATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																								
						RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.25 m	DISCONTINUITY DATA				ROCK STRENGTH INDEX			WEATH- ERING INDEX																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																															
						TOTAL CORE %	SOLID CORE %			B Angle DIP w.r.t CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	R4 R3 R2 R1	W1 W2 W3 W4 W5 W6																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
		Continued from Record of Borehole OPB-2		97.66																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																												

FEATURES LEGEND



BROKEN CORE



CLAY SEAM



LIMESTONE



LOST CORE

DEPTH SCALE

1 : 50



GOLDER

LOGGED: KN/TG

CHECKED: AK


GTA-RCK 054 S:\CLIENTS\MTQ\QEW-DIXIE\02 DATA\GINTQEW-DIXIE.GPJ GAL-MISS.GDT 20-3-12

PROJECT		1530382		RECORD OF BOREHOLE				No OPB-3		SHEET 1 OF 2		METRIC			
G.W.P.		2102-13-00; 2432-13-00		LOCATION		N 4827751.0; E 298660.7 MTM NAD 83 ZONE 10 (LAT. 43.589666; LONG. -79.576029)				ORIGINATED BY		PKS			
DIST		Central	HWY	QEW	BOREHOLE TYPE		108 mm O.D. Continuous Flight Solid Stem Augers				COMPILED BY		ACK		
DATUM		Geodetic		DATE		October 3, 2016				CHECKED BY		SMM			
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	W _P W W _L	WATER CONTENT (%)	10 20 30	kN/m ³		
103.5	GROUND SURFACE														
0.0	ASPHALT (230 mm)														
103.3															
103.1	Sand and gravel (FILL)														
0.4	Brown Moist														
	SAND, some silt, trace clay														
	Compact														
	Brown														
	Moist to wet														
	- Wet below a depth of 2.1 m														
100.8															
2.7	GRAVEL, trace sand, trace silt														
	Grey														
	Wet														
	CLAYEY SILT with SAND, trace to some gravel (TILL)														
	Stiff to very stiff														
	Grey														
	Wet														
									</						

Continued Next Page

+³, ×³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

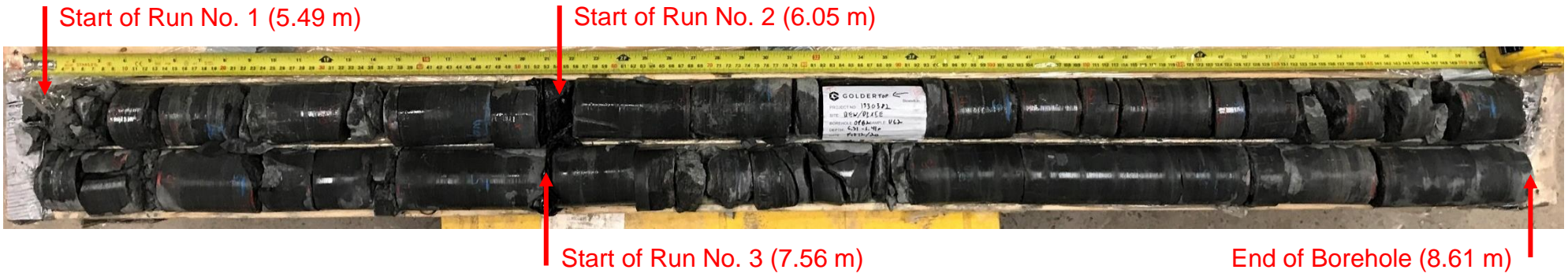
GTA-MTO 001 S:\CLIENTS\MTQEW-DIXIE\02_DATAGINT\QEW-DIXIE.GPJ GAL-GTA.GDT 20-3-12

PROJECT		RECORD OF BOREHOLE				No OPB-3		SHEET 2 OF 2		METRIC						
G.W.P. 2102-13-00; 2432-13-00		LOCATION				N 4827751.0; E 298660.7 MTM NAD 83 ZONE 10 (LAT. 43.589666; LONG. -79.576029)				ORIGINATED BY						
DIST Central HWY QEW		BOREHOLE TYPE				108 mm O.D. Continuous Flight Solid Stem Augers				COMPILED BY						
DATUM Geodetic		DATE				October 3, 2016				CHECKED BY						
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa								
	--- CONTINUED FROM PREVIOUS PAGE ---															
92.9	SHALE (BEDROCK) Grey Moderately weathered to slightly weathered at 7.25 m depth		3	RC	REC 92%											RQD = 89%
10.6	END OF BOREHOLE NOTE: 1. Water level not measured upon completion of drilling. Wet soil noted below a depth of 2.1 m (Elev. 101.4 m)															

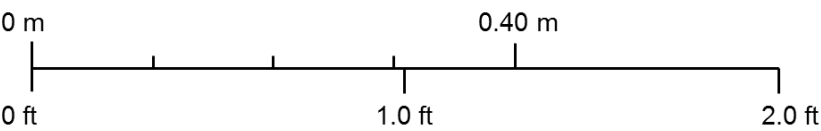
GTA-MTO 001 S:\CLIENTS\MTQ\QEW-DIXIE02_DATAGINT\QEW-DIXIE.GPJ GAL-GTA.GDT 20-3-12




OPB-1 Rock Core: 6.11 m to 9.11 m depth



OPB-2 Rock Core: 5.49 m to 8.61 m depth



Scale

PROJECT						
East of Cawthra Road to the East Mall QEW Widening						
TITLE						
Bedrock Core Photographs						
Borehole OPB-1 (6.11 m to 9.11 m) & Borehole OPB-2 (5.49 m to 8.61 m)						
 GOLDER	PROJECT No. 1530382			FILE No. ----		
	DRAFT	KNN	20200303	SCALE	NTS	VER. 1.
	CADD	--		FIGURE B-1		
	CHECK	SMM	20200310			
	REVIEW	JMAC	20200313			

Start of Run No. 1 (6.25 m)

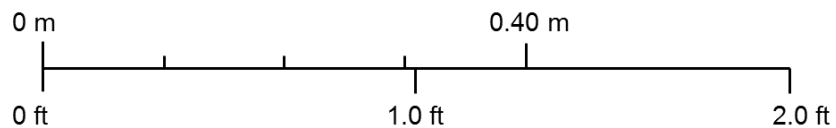
Start of Run No. 2 (7.6 m)




End of Borehole (10.62 m)

Start of Run No. 3 (9.09 m)

OPB-3 Rock Core: 6.25 m to 10.62 m depth



Scale

PROJECT						
East of Cawthra Road to the East Mall QEW Widening						
TITLE						
Bedrock Core Photographs Borehole OPB-3 (6.25 m to 10.62 m)						
 GOLDER	PROJECT No. 1530382			FILE No. ----		
	DRAFT	KNN	20200303	SCALE	NTS	VER. 1.
	CADD	--		FIGURE B-2		
	CHECK	SMM	20200310			
	REVIEW	JMAC	20200313			

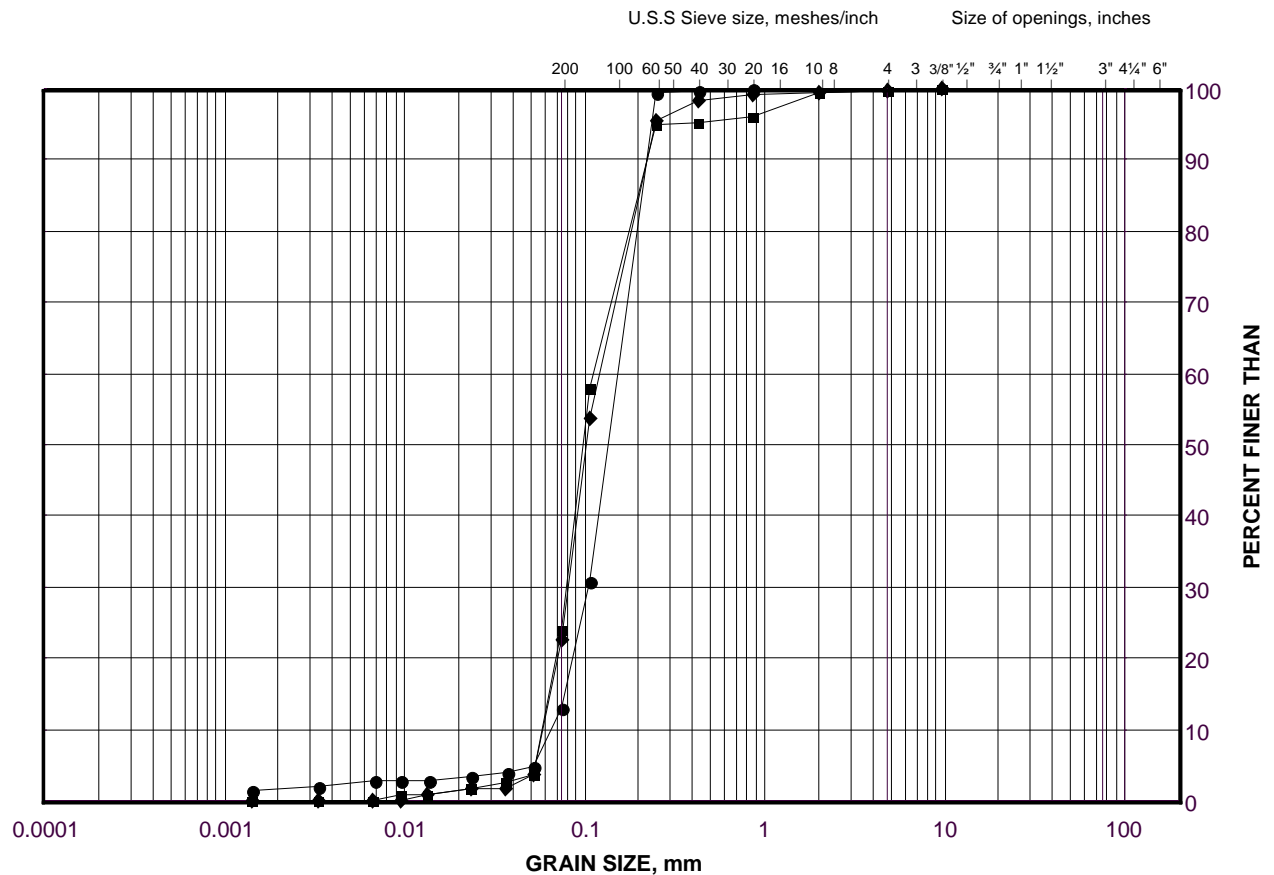
APPENDIX C

Geotechnical Laboratory Test Results

GRAIN SIZE DISTRIBUTION

Silty Sand to Sand

FIGURE C-1



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED	SAND SIZE			GRAVEL SIZE		SIZE

LEGEND

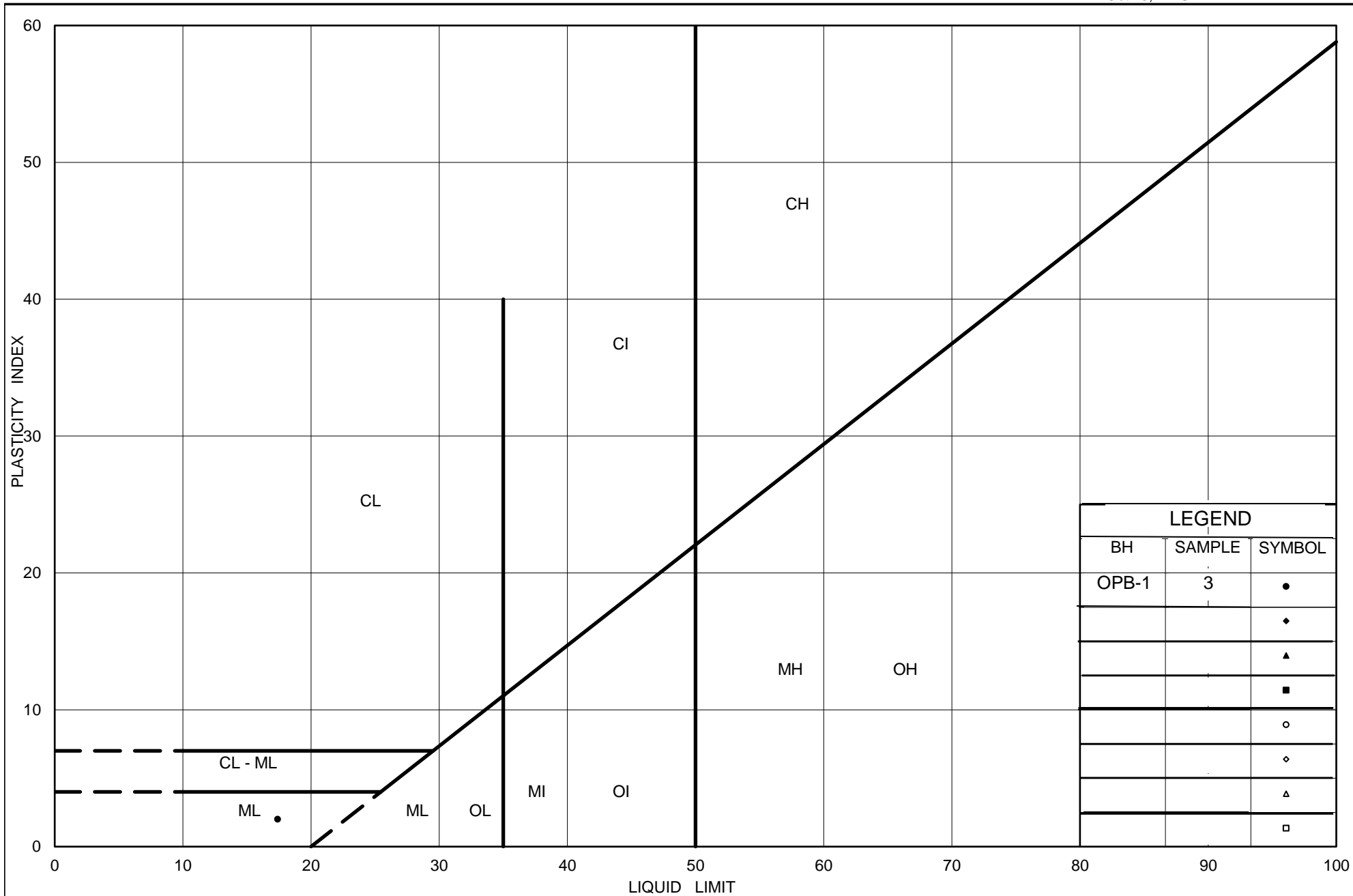
SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	OPB-3	2	101.7
■	OPB-1	2	101.3
◆	OPB-2	2B	101.2

Project Number: 1530382

Checked By: MK

Golder Associates

Date: 06-Mar-20



Ministry of Transportation

Ontario

PLASTICITY CHART

Sandy Silt

Figure No. C-2

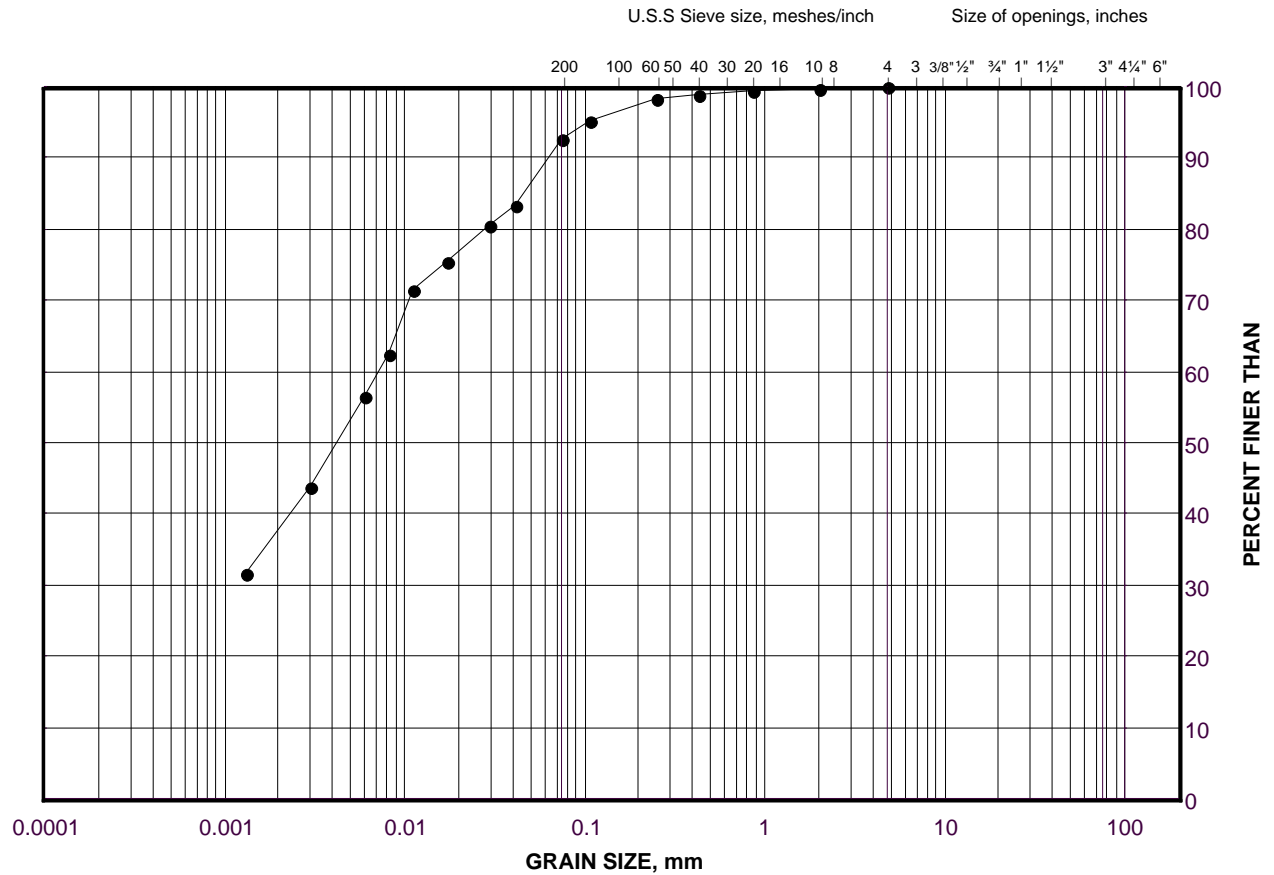
Project No. 1530382

Checked By:

GRAIN SIZE DISTRIBUTION

Clayey Silt

FIGURE C-3



SILT AND CLAY SIZES		FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED		SAND SIZE			GRAVEL SIZE		SIZE

LEGEND

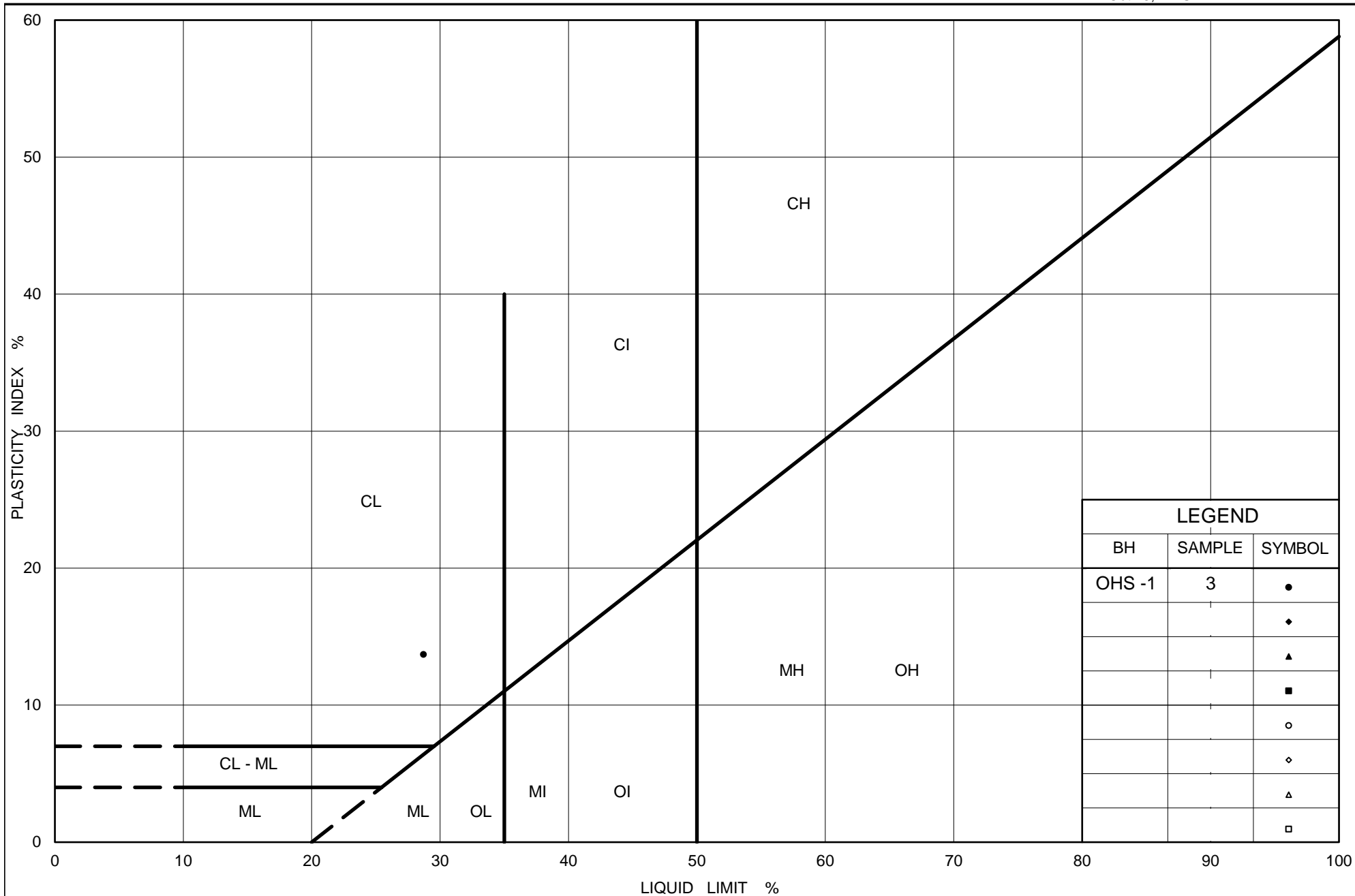
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(ft)
•	OHS -1	3	101.7

Project Number: 1530382

Checked By: MK

Golder Associates

Date: 21-Jul-17



Ministry of Transportation

Ontario

PLASTICITY CHART

Clayey Silt

Figure No. C-4

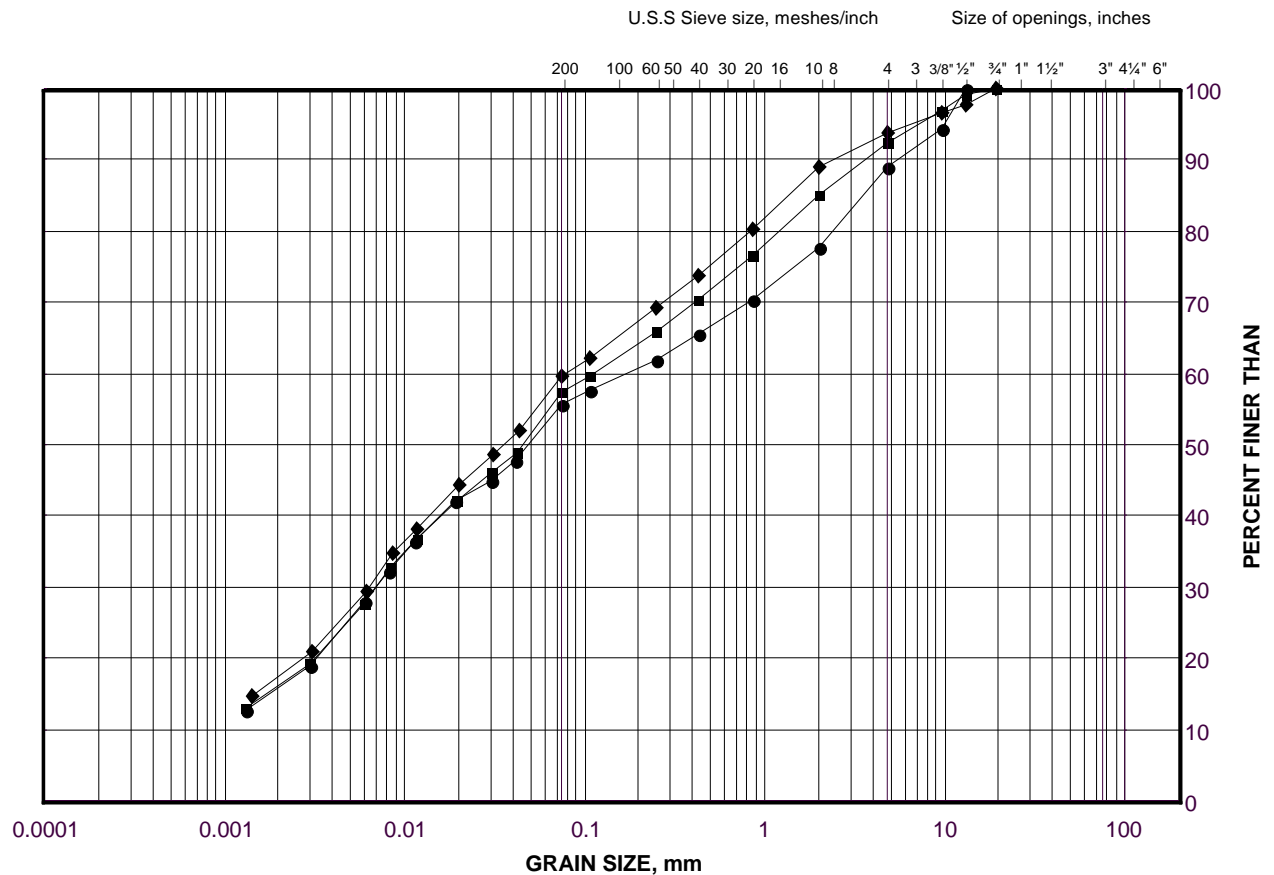
Project No. 1530382

Checked By:MK

GRAIN SIZE DISTRIBUTION

Clayey Silt with Sand (Till)

FIGURE C-5



SILT AND CLAY SIZES		FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED		SAND SIZE			GRAVEL SIZE		

LEGEND

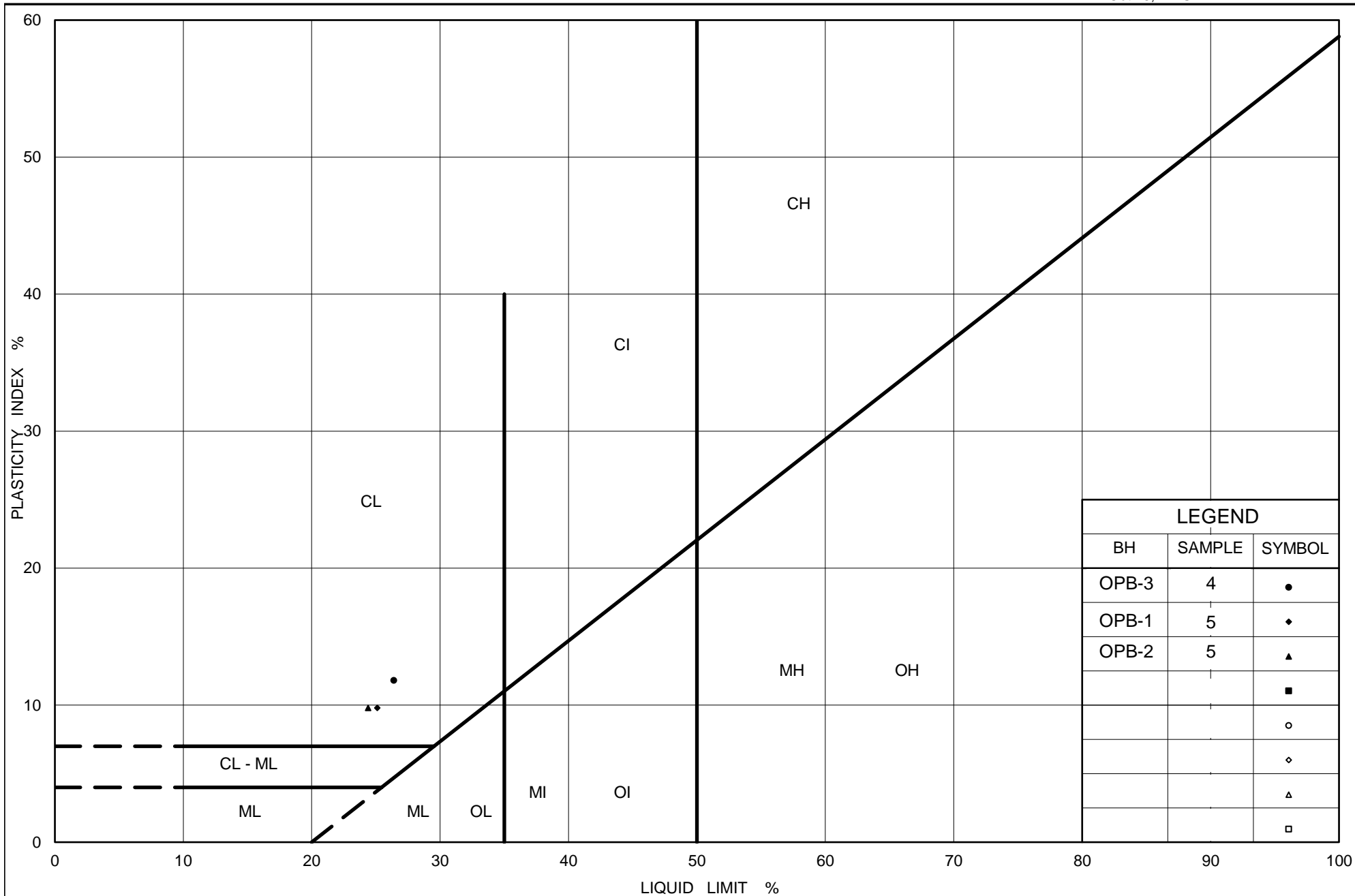
SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	OPB-2	3B	100.4
■	OPB-1	5	99.0
◆	OPB-3	6	98.6

Project Number: 1530382

Checked By: MK

Golder Associates

Date: 06-Mar-20



Ministry of Transportation

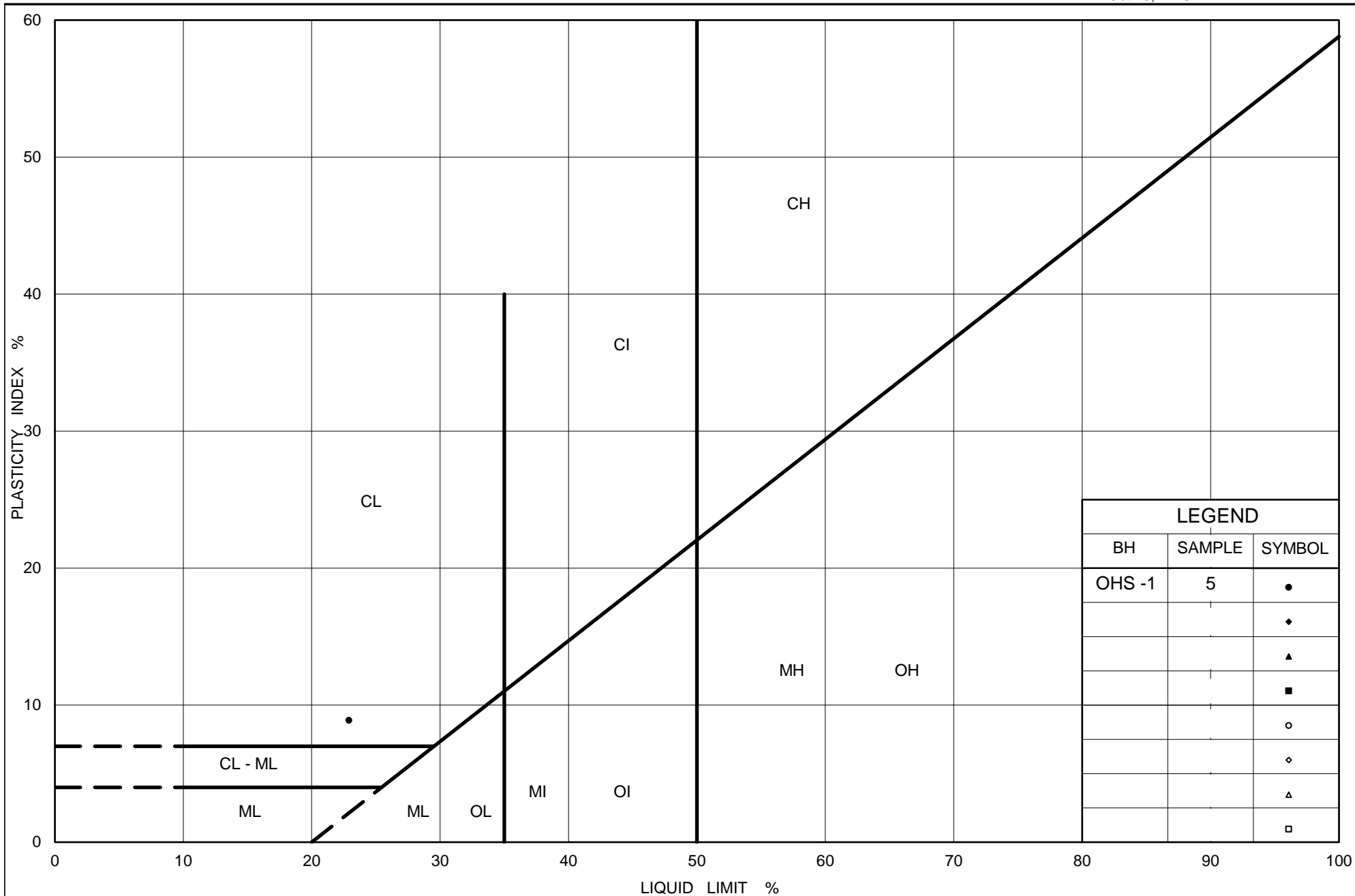
Ontario

PLASTICITY CHART Clayey Silt with Sand (Till)

Figure No. C-6

Project No. 1530382

Checked By:MK



Ministry of Transportation

Ontario

PLASTICITY CHART Clayey Silt (Residual Soil)

Figure No. C-7

Project No. 1530382

Checked By: MK

March 11, 2020

Ms. Katelyn Nero
Golder Associates Ltd.
6925 Century Avenue, Suite #100
Mississauga, Ontario
Canada L5N 7K2

Re: QEW/Dixie UCS, testing
(Golder Project 153082)

Dear Ms. Nero:

On February 24th, 2020 three(3) HQ-sized core samples were received by Geomechanica Inc. via drop-off by Golder personnel. These samples were identified as being from the QEW/Dixie Project (Golder Project No. 153082). From these samples, two (2) UCS specimens were prepared and tested.

Details regarding the steps of specimen preparation and testing along with the results and photographs of the test specimens before and after testing are presented in the accompanying laboratory report and summary spreadsheet(s).

Sincerely,



Bryan Tatone Ph.D., P. Eng.

Geomechanica Inc.
Tel: (647) 478-9767
Email: bryan.tatone@geomechanica.com

Rock Laboratory Testing Results

A report submitted to:

Katelyn Nero
Golder Associates Ltd.
6925 Century Avenue, Suite #100
Mississauga, Ontario
Canada L5N 7K2

Prepared by:

Bryan Tatone, PhD, PEng
Omid Mahabadi, PhD, PEng
Geomechanica Inc.
#900-390 Bay St.
Toronto ON
M5H 2Y2 Canada
Tel: +1-647-478-9767
lab@geomechanica.com

March 11, 2020

Project number: 1530382

Abstract

This document summarizes the results of rock laboratory testing, including the results of 2 Uniaxial Compressive Strength (UCS) tests. Along with the UCS values, the tangent Young's modulus along with photographs of specimens before and after testing are presented.

In this document:

1 Uniaxial Compressive Strength Tests	1
Appendices	4

1 Uniaxial Compressive Strength Tests

1.1 Overview

This section summarizes the results of uniaxial compressive strength testing. The testing was performed in Geomechanica's rock testing laboratory using a 150 ton (1.3 MN) Forney loading frame equipped with pressure-compensated control valve to maintain an axial displacement rate of approximately 0.2 mm/min (Figure 1). The specimen preparation and testing procedure included the following:

1. Unwrapping of the core sample, inspecting it for damage, and re-wrapping it in electrical tape to minimize exposure to moisture during subsequent specimen preparation.
2. Diamond cutting of the core sample to obtain cylindrical specimens with an appropriate length (length:diameter = 2:1) and nearly parallel end faces.
3. Diamond grinding of the specimen to obtain flat (within ± 0.025 mm) and parallel end faces (within 0.25°).
4. Placing of the specimen into the loading frame, applying a 1 kN axial load, and removing the electrical tape.
5. Axially loading the specimens to rupture while continuously recording axial force and axial deformation to determine the peak strength (UCS) and tangent Young's modulus.



Figure 1: Forney loading frame setup for UCS testing.

Using a precision V-block mounted on the magnetic chuck of the surface grinder, the test specimens met the end flatness, end parallelism, and perpendicularity criteria set out in ASTM D4543-08. The side straightness criteria, as checked with a feeler gauge, was met for all samples and the minimum length:diameter criteria was met for all specimens unless noted otherwise in Table 1. Testing of the specimens followed ASTM D7012-14 with the following exceptions:

- These tests included measurement of the UCS and tangent Young' (elastic) modulus, but not the Poisson's ratio. This represents a hybrid between Methods C and D of ASTM D7012-14.

1.2 Results

The results of the UCS tests are summarized in Table 1. Additional specimen details and measurements are provided in the summary spreadsheet that accompanies this report. The corresponding stress-strain curves are presented in Figure 2. The Young's modulus is the tangent modulus, calculated as the slope of the best fit line through ± 300 data points on either side of the point representing 50.0% of the peak strength.

Table 1: Summary of UCS results.

Sample	Depth (m)	Bulk density ρ (g/cm ³)	UCS (MPa)	Young's modulus E (GPa)	Lithology	Failure description
OPB-1, UC1	7.13 - 7.29	2.588	21.1	1.1	Georgian Bay Formation - Shale	1, 2
OPB-2, UC2	6.31 - 6.41	2.582	16.2	0.7	Georgian Bay Formation - Shale	3, 2, 4
Average		2.585	18.6	0.9		
Standard deviation		0.003	2.5	0.2		

¹ Hourglass failure

² Specimen emitted pore water upon loading

³ Axial splitting failure

⁴ Length:Diameter ratio less than 2

1.3 Specimen photographs

Photographs of the specimens before and after testing are presented in the Appendix of this report.

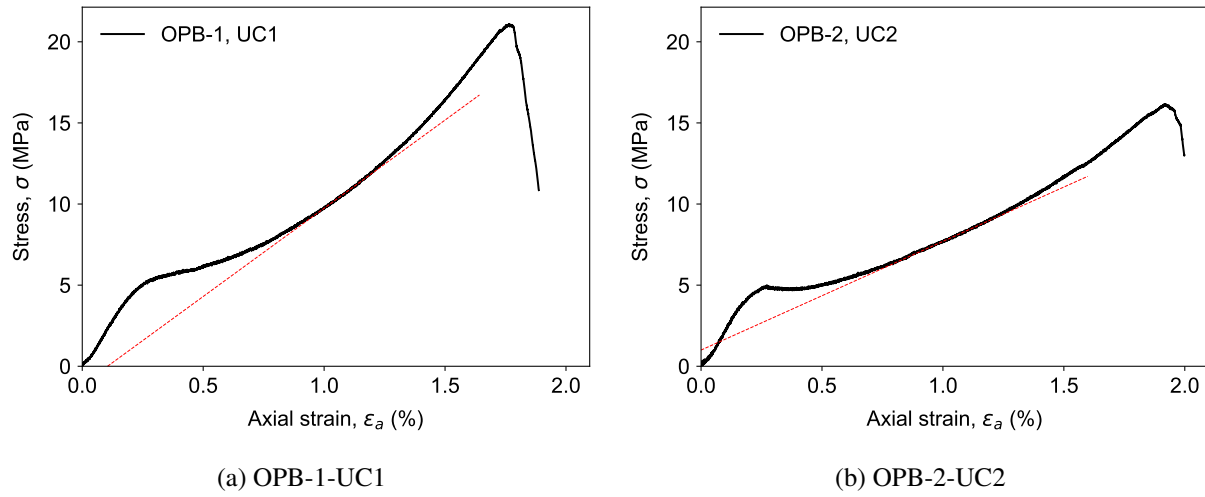




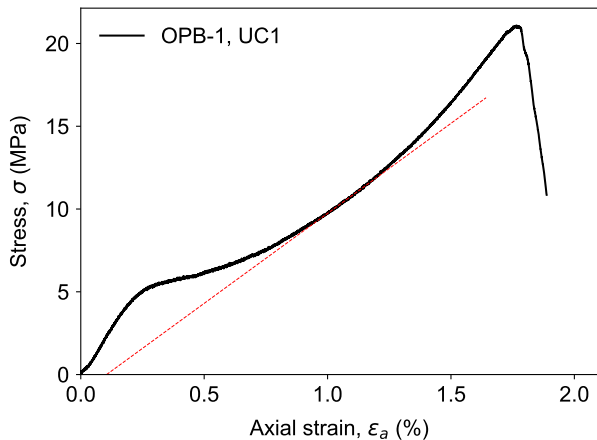
Figure 2: Measured stress-strain curves.

Appendices



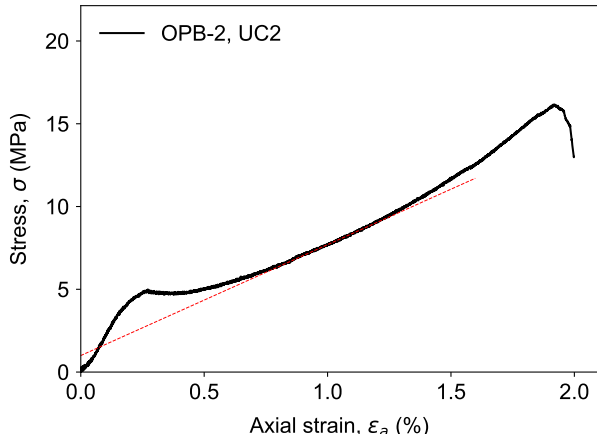
Specimen sheets

- OPB-1, UC1
- OPB-2, UC2

Uniaxial Compression Test

Client	Golder Associates Ltd.	Project	1530382
Sample	OPB-1, UC1	Depth	7.13 - 7.29
Specimen parameters		Prior to testing	After testing
Diameter (mm) ^a	63.02		
Length (mm) ^a	127.18		
Bulk density ρ (g/cm ³)	2.588		
UCS (MPa)	21.1		
Young's modulus E (GPa) ^b	1.1		
Lithology	Georgian Bay Formation - Shale		
Failure description ^c	1, 2		
^a Additional specimen measurement/details provides in accompanying summary spreadsheet.			
^b Tangent modulus, calculated as the slope of the best fit line through ±300 data points on either side of the point representing 50.0% of the peak strength.			
^c Failure description: ¹ Hourglass failure; ² Specimen emitted pore water upon loading;			
			
Remarks:			
Performed by	BSAT	Date	2020-03-10

Uniaxial Compression Test

Client	Golder Associates Ltd.	Project	1530382
Sample	OPB-2, UC2	Depth	6.31 - 6.41
Specimen parameters		Prior to testing	After testing
Diameter (mm) ^a	62.92		
Length (mm) ^a	101.03		
Bulk density ρ (g/cm ³)	2.582		
UCS (MPa)	16.2		
Young's modulus E (GPa) ^b	0.7		
Lithology	Georgian Bay Formation - Shale		
Failure description ^c	3, 2, 4		
^a Additional specimen measurement/details provides in accompanying summary spreadsheet.			
^b Tangent modulus, calculated as the slope of the best fit line through ±300 data points on either side of the point representing 50.0% of the peak strength.			
^c Failure description: ³ Axial splitting failure; ² Specimen emitted pore water upon loading; ⁴ Length:Diameter ratio less than 2;			
			
Remarks:			
Performed by	BSAT	Date	2020-03-10

APPENDIX D

Analytical Laboratory Test Results



Attention: Sandra McGaghran

Golder Associates Ltd
6925 Century Ave
Suite 100
Mississauga, ON
CANADA L5N 7K2

Your Project #: 1530382
Site#: OGDEN BRIDGE
Site Location: QEW/DIXIE
Your C.O.C. #: 657051-02-01

Report Date: 2020/02/28
Report #: R6091999
Version: 1 - Final

CERTIFICATE OF ANALYSIS

BV LABS JOB #: C049525

Received: 2020/02/24, 18:25

Sample Matrix: Soil
Samples Received: 2

Analyses	Quantity	Date	Date	Laboratory Method	Analytical Method
		Extracted	Analyzed		
Chloride (20:1 extract)	2	2020/02/27	2020/02/27	CAM SOP-00463	SM 23 4500-Cl E m
Conductivity	2	2020/02/28	2020/02/28	CAM SOP-00414	OMOE E3530 v1 m
pH CaCl2 EXTRACT	2	2020/02/26	2020/02/26	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	2	2020/02/24	2020/02/28	CAM SOP-00414	SM 23 2510 m
Sulphate (20:1 Extract)	2	2020/02/27	2020/02/27	CAM SOP-00464	EPA 375.4 m

Remarks:

Bureau Veritas Laboratories are accredited to ISO/IEC 17025 for specific parameters on scopes of accreditation. Unless otherwise noted, procedures used by BV Labs are based upon recognized Provincial, Federal or US method compendia such as CCME, MELCC, EPA, APHA.

All work recorded herein has been done in accordance with procedures and practices ordinarily exercised by professionals in BV Labs profession using accepted testing methodologies, quality assurance and quality control procedures (except where otherwise agreed by the client and BV Labs in writing). All data is in statistical control and has met quality control and method performance criteria unless otherwise noted. All method blanks are reported; unless indicated otherwise, associated sample data are not blank corrected. Where applicable, unless otherwise noted, Measurement Uncertainty has not been accounted for when stating conformity to the referenced standard.

BV Labs liability is limited to the actual cost of the requested analyses, unless otherwise agreed in writing. There is no other warranty expressed or implied. BV Labs has been retained to provide analysis of samples provided by the Client using the testing methodology referenced in this report. Interpretation and use of test results are the sole responsibility of the Client and are not within the scope of services provided by BV Labs, unless otherwise agreed in writing. BV Labs is not responsible for the accuracy or any data impacts, that result from the information provided by the customer or their agent.

Solid sample results, except biota, are based on dry weight unless otherwise indicated. Organic analyses are not recovery corrected except for isotope dilution methods.

Results relate to samples tested. When sampling is not conducted by BV Labs, results relate to the supplied samples tested.

This Certificate shall not be reproduced except in full, without the written approval of the laboratory.

Reference Method suffix "m" indicates test methods incorporate validated modifications from specific reference methods to improve performance.

* RPDs calculated using raw data. The rounding of final results may result in the apparent difference.



Your Project #: 1530382
Site#: OGDEN BRIDGE
Site Location: QEW/DIXIE
Your C.O.C. #: 657051-02-01

Attention: Sandra McGaghran

Golder Associates Ltd
6925 Century Ave
Suite 100
Mississauga, ON
CANADA L5N 7K2

Report Date: 2020/02/28
Report #: R6091999
Version: 1 - Final

CERTIFICATE OF ANALYSIS

BV LABS JOB #: C049525
Received: 2020/02/24, 18:25

Encryption Key

Please direct all questions regarding this Certificate of Analysis to your Project Manager.
Ema Gitej, Senior Project Manager
Email: Ema.Gitej@bvlabs.com
Phone# (905)817-5829

=====

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BUREAU
VERITAS

BV Labs Job #: C049525
Report Date: 2020/02/28

Golder Associates Ltd
Client Project #: 1530382
Site Location: QEW/DIXIE
Sampler Initials: KNN

SOIL CORROSIVITY PACKAGE (SOIL)

BV Labs ID		MBY061	MBY062		
Sampling Date		2020/02/11	2020/02/12		
COC Number		657051-02-01	657051-02-01		
	UNITS	OPB1-SA4B	OPB2-SA5	RDL	QC Batch
Calculated Parameters					
Resistivity	ohm-cm	1400	2000		6602604
Inorganics					
Soluble (20:1) Chloride (Cl-)	ug/g	250	130	20	6608859
Conductivity	umho/cm	699	508	2	6611014
Available (CaCl2) pH	pH	7.71	7.84		6606974
Soluble (20:1) Sulphate (SO4)	ug/g	200	190	20	6608860
RDL = Reportable Detection Limit					
QC Batch = Quality Control Batch					



BUREAU
VERITAS

BV Labs Job #: C049525
Report Date: 2020/02/28

Golder Associates Ltd
Client Project #: 1530382
Site Location: QEW/DIXIE
Sampler Initials: KNN

TEST SUMMARY

BV Labs ID: MBY061
Sample ID: OPB1-SA4B
Matrix: Soil

Collected: 2020/02/11
Shipped:
Received: 2020/02/24

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	6608859	2020/02/27	2020/02/27	Alina Dobreanu
Conductivity	AT	6611014	2020/02/28	2020/02/28	Kazzandra Adeva
pH CaCl ₂ EXTRACT	AT	6606974	2020/02/26	2020/02/26	Surinder Rai
Resistivity of Soil		6602604	2020/02/28	2020/02/28	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	6608860	2020/02/27	2020/02/27	Alina Dobreanu

BV Labs ID: MBY062
Sample ID: OPB2-SA5
Matrix: Soil

Collected: 2020/02/12
Shipped:
Received: 2020/02/24

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	6608859	2020/02/27	2020/02/27	Alina Dobreanu
Conductivity	AT	6611014	2020/02/28	2020/02/28	Kazzandra Adeva
pH CaCl ₂ EXTRACT	AT	6606974	2020/02/26	2020/02/26	Surinder Rai
Resistivity of Soil		6602604	2020/02/28	2020/02/28	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	6608860	2020/02/27	2020/02/27	Alina Dobreanu



BUREAU
VERITAS

BV Labs Job #: C049525
Report Date: 2020/02/28

Golder Associates Ltd
Client Project #: 1530382
Site Location: QEW/DIXIE
Sampler Initials: KNN

GENERAL COMMENTS

Each temperature is the average of up to three cooler temperatures taken at receipt

Package 1	2.0°C
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Results relate only to the items tested.



BUREAU
VERITAS

BV Labs Job #: C049525

Report Date: 2020/02/28

QUALITY ASSURANCE REPORT

Golder Associates Ltd

Client Project #: 1530382

Site Location: QEW/DIXIE

Sampler Initials: KNN

QC Batch	Parameter	Date	Matrix Spike		SPIKED BLANK		Method Blank		RPD	
			% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
6606974	Available (CaCl ₂) pH	2020/02/26			100	97 - 103			0.66	N/A
6608859	Soluble (20:1) Chloride (Cl ⁻)	2020/02/27	106	70 - 130	101	70 - 130	<20	ug/g	NC	35
6608860	Soluble (20:1) Sulphate (SO ₄)	2020/02/27	NC	70 - 130	101	70 - 130	<20	ug/g	3.5	35
6611014	Conductivity	2020/02/28			102	90 - 110	<2	umho/cm	1.0	10

N/A = Not Applicable

Duplicate: Paired analysis of a separate portion of the same sample. Used to evaluate the variance in the measurement.

Matrix Spike: A sample to which a known amount of the analyte of interest has been added. Used to evaluate sample matrix interference.

Spiked Blank: A blank matrix sample to which a known amount of the analyte, usually from a second source, has been added. Used to evaluate method accuracy.

Method Blank: A blank matrix containing all reagents used in the analytical procedure. Used to identify laboratory contamination.

NC (Matrix Spike): The recovery in the matrix spike was not calculated. The relative difference between the concentration in the parent sample and the spike amount was too small to permit a reliable recovery calculation (matrix spike concentration was less than the native sample concentration)

NC (Duplicate RPD): The duplicate RPD was not calculated. The concentration in the sample and/or duplicate was too low to permit a reliable RPD calculation (absolute difference \leq 2x RDL).



BUREAU
VERITAS

BV Labs Job #: C049525
Report Date: 2020/02/28

Golder Associates Ltd
Client Project #: 1530382
Site Location: QEW/DIXIE
Sampler Initials: KNN

VALIDATION SIGNATURE PAGE

The analytical data and all QC contained in this report were reviewed and validated by the following individual(s).

Anastassia Hamanov, Scientific Specialist

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Maxxam Analytics International Corporation o/a Maxxam Analytics

APPENDIX E

Non-Standard Special Provisions

FOUNDATIONS ON BEDROCK – Item No.

Non-Standard Special Provision

Where strip/spread footings, steel piles or caissons extend to or into the shale bedrock, which is very weak to weak in the pier areas of the Ogden Pedestrian Bridge and which contains strong to very strong limestone layers at varying depths/elevations, appropriate equipment and construction procedures will be required to penetrate into the bedrock to reach the founding level.

Basis of Payment

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION

DOWELS INTO ROCK - Item No.

Non-Standard Special Provision

Scope of Work

Where required, the Contractor shall provide dowels into the bedrock at the pier foundations for the QEW-Ogden Pedestrian Bridge replacement structure.

Construction

Concrete shall be of the same strength as the footing concrete and placed in accordance with OPSS 904. All reinforcing steel supplied shall be in accordance with OPSS.PROV 1440 (dowel bars conforming to CAN/CSA G30.18, Grade 400).

Where dowels are to be placed in rock, hole shall be drilled to the required depth and size. Hole diameter shall be two times the nominal diameter of the dowel. Each hole shall be cleaned out, grouted and the dowel set in place. Grout shall be of the same strength as the footing concrete or at least 25 MPa at 28 days.

If hole contains water, the Contractor shall remove the water, otherwise a tremie procedure shall be used to completely fill the hole with grout. The dowel shall be forced into the hole after the grout has been placed and while it is still fresh.

Rock Dowel Testing

All proposed testing procedures shall be in general conformance with ASTM D3689 07, ASTM D1143 07 and ASTM D4435 08. Field testing must be carried out in the presence of, and the results reviewed and approved by, the Contract Administrator.

Performance Tests

Performance testing shall be carried out at two dowels to confirm that the design load of the rock dowels can be achieved. The Contract Administrator will select the rock dowels to be tested.

Performance test shall be by axial tensioning using a hydraulic jack with a capacity of at least 1.5 times the ultimate strength of the dowels.

Rock dowels shall be loaded and unloaded in 3 cycles and measurements of the displacement of the dowel shall be carried out at each load increment (step) in accordance with the following schedule:

Cycle-Step	1 1	1 2	1 3	2 1	2 2	2 3	2 4
% Design Load	50	75	25	50	75	100	25
Cycle-Step	3 1	3 2	3 3	3 4	3 5		
% Design Load	50	75	100	110	25		

The design load shall be taken as 360 kN for 35M dowels, 252 kN for 30M dowels, 180 kN, for 25M dowels, and 108 kN for 20M dowels.

Displacement measurements shall be carried out at each load increment using calibrated displacement gauges capable of measuring movements of 0.0025 cm. Measurements shall be referenced to an independent fixed referenced pint.

Rock dowels which fail to meet the acceptance criteria shall be replaced at the Contractor's expense and re tested. If a rock dowel fails, three (3) additional rock dowels shall be tested at the same abutment and pier footing as directed by the Contract Administrator.

Acceptance criteria for the rock dowels will be in accordance with the Post Tensioning Institute (1985) as follows:

- The dowels are acceptable if the total elastic movement is greater than 80% of the theoretical elastic elongation of the free stressing and is less than the theoretical elongation of the free stressing length plus 50% of the bond length.

Basis of Payment

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

OBSTRUCTIONS - Item No.

Non-Standard Special Provision

The native cohesive residual soils contain shale fragments as indicated in the borehole records. Although not encountered in the boreholes advanced at this site, the cohesive till deposit should be expected to contain cobbles and/or boulders. Consideration of the presence of these obstructions must be made in the selection of appropriate equipment and procedures for driving steel H-piles or advancing caissons, such that the design tip levels are achieved; or installation of temporary protection systems.

Basis of Payment

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION

VIBRATION MONITORING - Item No.

Special Provision

Scope

This special provision describes requirements for vibration monitoring during bedrock excavation, temporary protection system installation, and piling/caisson installation associated with the construction of the Dixie Road Underpass, Ogden Pedestrian Bridge and Applewood Creek Culvert (CV02/03).

References

The subsurface conditions at the site are described in the following Foundation Investigation Reports:

- Dixie Road Underpass Bridge Replacement (Site No. 24-193), QEW Widening From East of Cawthra Road to The East Mall, Cities Of Mississauga and Etobicoke, MTO GWP 2102-13-00 and 2432-13-00.
- Ogden Avenue Pedestrian Bridge Replacement (Structure Site No. 24-192), QEW Widening from East of Cawthra Road to The East Mall, Cities of Mississauga and Etobicoke, MTO GWP 2102-13-00 and 2432-13-00.
- Applewood Creek Culvert (CV02/03) Replacement, QEW Widening from East of Cawthra Road to The East Mall, Cities of Mississauga and Etobicoke, MTO GWP 2102-13-00 and 2432-13-00

Definitions

For the purposes of this Special Provision the following definitions apply:

Peak Particle Velocity (PPV) means the maximum component velocity in millimetres per second that ground particles move as a result of energy released from vibratory construction operations.

Pre-Construction Condition Survey means a detailed record, accompanied by film or video, as necessary, of the condition of a structure or utility, prior to the commencement of vibratory construction operations.

Intra-Construction or Post-Construction Condition Survey means a detailed record, accompanied by film or video, as necessary, of the condition of a structure or utility, during or after completion of vibratory construction operations.

Submission Requirements

The Contractor shall submit a Request to Proceed accompanied with details of their vibration control plan to the Contract Administrator at least 14 Days prior to any bedrock excavation, protection system installation in bedrock, or deep foundation installation at the Dixie Road underpass, Ogden Pedestrian Bridge and Applewood Creek Culvert (CV02/03) sites. The submittals shall satisfy the specifications and at a minimum contain the following specific information:

- a) Equipment and methods used by the Contractor to perform the work that may cause vibrations
- b) Verification that the equipment and methods are planned to satisfy the PPV limits of 25 mm/s adjacent to residential structures within 100 m of the bridge sites.
- c) Action plan to be taken to adjust excavation, protection system installation, deep foundation installation and compaction methods if monitoring readings by the Contract Administrator show vibrations measured on the ground above the thresholds shown in Table 1.

Table 1: Peak Particle Velocity (PPV) Thresholds

	Frequency (Hz)	PPV (mm/s)
Review Level	All	25
Alert Level	All	50

The Contractor shall proceed with the work only after receiving a Notice to Proceed from the Contract Administrator.

Monitoring

The Contractor shall coordinate with and provide access to the Contract Administrator and their Foundation Engineering Specialist to conduct vibration monitoring throughout vibration-inducing construction activities, including but not limited to bedrock excavation, protection system installation, and deep foundation installation. Vibration monitoring equipment will be placed on the ground surface in an array at increasing distances away from the construction activity toward the residential structures within 100 m of the bridge sites. The Contract Administrator will take readings during excavation, installation of protection systems and deep foundations, and will promptly notify the Contractor if the vibrations exceed the limits specified in Table 1 above.

If the Review Level is reached, the Contractor shall immediately review and discuss response actions with the Contract Administrator. The Contractor shall submit a plan of action to alter their construction procedures to prevent the Alert Level from being reached. All construction work shall be continued such that the Alert Level is not reached.

If the Alert Level is reached, the Contractor shall cease construction operations, and immediately review and discuss further response actions with the Contract Administrator.

Basis of Payment

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION

WORKING SLAB - Item No.

Special Provision

1.0 Scope

This Special Provision covers the requirements for the supply and placement of a concrete working slab under foundations for the QEW Ogden Pedestrian Bridge replacement structure.

2.0 References

This Special Provision refers to the following standards, specifications or publications:

Ontario Provincial Standard Specifications, Construction

OPSS 902 Excavating and Backfilling - Structures

3.0 Definitions - Not Used**4.0 Design and Submission Requirements - Not Used****5.0 Materials**

Concrete for working slabs shall have a minimum 28 day strength of 20 MPa.

6.0 EQUIPMENT - Not Used**7.0 CONSTRUCTION****7.01 Excavation**

Excavation for the working slab shall be according to OPSS 902.

7.02 Protection of Founding Soil

Following inspection and approval of the prepared subgrade, a working slab with a minimum thickness of 100 mm shall be placed on the foundation subgrade as specified in the Contract Documents.

7.03 Protection of Founding Bedrock

The surface of the footing founding rock shall be exposed, cleaned and any loose or fractured parts removed so that sound rock is exposed. The working slab shall be placed on the exposed cleaned sound founding rock surface as specified in the Contract Documents.

Thickness of the mass concrete pad shall depend on the slope and irregularities in the exposed founding rock surface. A nominal thickness and a footprint plan view area has been specified on the Contract Documents

7.04 Dewatering

Dewatering shall be carried out according to OPSS 902.

8.0 Quality Assurance - Not Used

9.0 Measurement for Payment - Not Used

10.0 Basis of Payment

10.01 Working Slab - Item

Payment at the Contract price for the above tender item shall be full compensation for all labour, Equipment and Material to do the work.

END OF SECTION

AMENDMENT TO OPSS 903 – Item No.

Non-Standard Special Provision

OPSS 903, Construction Specification for Deep Foundations, is amended by the following:

903.07 CONSTRUCTION

903.07.03 Caisson Piles

903.07.03.01 General

Drilled shaft (caisson) foundations for support of the Dixie Road Underpass centre pier and the Ogden Pedestrian Bridge will extend through overburden and into the shale bedrock, which is weak and contains clay seams and medium strong to very strong limestone interlayers of varying thicknesses at varying depths/elevations. Appropriate equipment and construction procedures will be required to penetrate the overburden and advance sockets into the bedrock to reach the design founding level, including the use of temporary casings to support the sidewalls through the overburden.

The temporary casings must be advanced sufficiently into the upper zone of weathered bedrock to create a seal that prevents the overburden and weathered shale bedrock from flowing/sloughing into the rock socket.

The drilled shaft foundations at the Dixie Road Underpass centre pier and the Ogden Pedestrian Bridge foundation elements must be constructed in either fully dry or fully wet conditions to eliminate the potential for degradation of the shale at the base and sidewalls of the rock socket due to wetting and drying prior to concrete placement. If water seepage occurs within the drilled shaft, the rock socket shall be filled with water and maintained in a wet condition throughout inspection, placement of reinforcing steel and placement of concrete using tremie methods. If no water seepage occurs within the drilled shaft below the permanent casing and there is no risk of degradation of the shale due to wetting and drying, the rock socket may be maintained in a fully dry condition provided that the base can be cleaned to meet the cleaning and inspection requirements.

Where rock sockets are “wet” (i.e., flooded), construction of the drilled shaft including placement of concrete by tremie methods shall be completed within seven days following the start of rock socket construction. Where rock sockets are “dry”, inspection and concrete placement shall be completed within 24 hours following the start of rock socket construction.

Construction of a rock socket or installation and seating of a steel casing shall not be permitted within 5 m of any caisson into which concrete has been placed within the preceding 24 hours.

903.07.03.03 Inspection of the Excavation

Section 903.07.03.03 of OPSS.PROV 903 is amended by the addition of the following:

The walls and base of each rock socket shall be thoroughly cleaned and inspected immediately thereafter. Cleaning shall be by vacuum, water flushing or other suitable means appropriate to the conditions in the Ogden Pedestrian Bridge caisson holes; where water flushing is used, the water issuing from the drilled shaft on flushing or pumping shall be clean and free of soil, rock cuttings and any other material.

Each rock socket shall be inspected and accepted by the Contract Administrator (Foundation Engineering Specialist) prior to proceeding with construction, and the Contractor shall coordinate with and support the Contract Administrator and Foundation Engineering Specialist for such inspection. The bottom of each rock socket shall be inspected using a Shaft Inspection Device (SID) or Shaft Quantitative Inspection Device (SQUID) supplied by the

Foundation Engineering Specialist to verify socket cleanliness and thickness of base sediment at the time of concreting.

The SID shall satisfy the following requirements:

- a) A remotely operated, high resolution, color video camera sealed inside a watertight bell housing.
- b) Provides a clear view of the bottom inspection on a video monitor at the surface in real time.
- c) Provides a permanent record of the entire inspection with voice annotation with a resolution of not less than 720 x 480.
- d) Provides at least two graduated measuring devices to record the thickness of debris/sediment on the bottom of the shaft excavation to a minimum accuracy of 12 mm and a length greater than 37 mm.
- e) Provides sufficient lighting to illuminate the entire field of vision at the bottom of the shaft for the operator and inspector to clearly see the depth measurement scale on the video monitor and to produce a clear recording of the inspection.
- f) Provides a compressed air or gas system to displace drilling fluids from the bell housing and a pressurized water system to assist in determination of bottom sedimentation depth.

Where SQUID is used in place of SID, the SQUID shall satisfy the following requirements and procedures:

- a) The SQUID unit (unless updated by the equipment manufacturer) shall be a hexagonal-shaped device with a height of approximately 630 mm, a diagonal of approximately 650 mm, and a weight of approximately 188 kg. The unit shall include three penetrometers each having a surface area of 10 cm² to measure force and three displacement plates each having a diameter of 152 mm and a weight of 7.75 kg to determine displacements. The unit shall also be supplied with two downhole data transmission cables and two transmitter boxes, and a SQUID tablet to permit continuous acquisition, display and storage of the data transmitted from the SQUID tests.
- b) The SQUID unit shall be pin-connected to the Kelly bar using a properly-sized Kelly bar adapter provided by the SQUID equipment supplier or contractor.
- c) After the pin connection and prior to testing of the drilled shaft base, the verticality of the SQUID unit shall be checked and confirmed. The signal transmission from the SQUID unit to the SQUID tablet shall also be confirmed prior to commencing the test. Signal transmission shall be checked by manually lifting each displacement plate and observing the increasing displacement on the SQUID tablet.
- d) The test shall proceed by slowly lowering the Kelly bar without rotation until the entire weight of the Kelly bar is transferred to, and is resting on, the SQUID unit. Penetrometer force and plate displacement measurements shall be continuously acquired, displayed and stored on the SQUID tablet during the test process. A test run shall be terminated once two of the three penetrometers have registered a force greater than 2.2 kN or the maximum penetrometer travel is reached for any one of the penetrometers.
- e) For each SQUID run, the average debris thickness determined using the force versus displacement results from a minimum of two penetrometers shall be used to determine if the drilled shaft base condition meets the specified base cleanliness criteria, or whether additional cleaning and retesting is required.
- f) A drilled shaft base often contains irregularities from a level surface due to pilot holes or grooves from cutting teeth on drilling tools. Therefore, a SQUID run shall be considered complete provided the debris thickness can be determined from a minimum of two force versus displacement plots. Interpretation of reading for determination of the thickness of debris/sediment and reporting shall be based on the manufacturer's recommended procedures.

The thickness of debris/sediment at the base of the rock socket shall be measured by SID or by SQUID testing in at least five locations: one in the center of the shaft as well as in each of the four quadrants surrounding the shaft center. The following cleaning criteria shall apply for thickness of sediments at the time of concrete placement:

- a) The average thickness of the sediments shall be less than 8 mm over at least 50 percent of the base of each shaft. The maximum thickness of sediment at any place on the base of the shaft shall not exceed 15 mm.

If any of the SID inspections indicate the cleanliness or bearing material requirements are not achieved, additional drilling and/or cleaning shall be completed and reinspection shall be carried out.

903.07.03.08 Pile Integrity Testing

The following section is added to OPSS.PROV 903 for the Dixie Road underpass centre pier and the Ogden Pedestrian Bridge Foundations:

Pile Integrity Testing (PIT) shall be carried out by the Contract Administrator (Foundation Engineering Specialist) on a minimum of two of the completed drilled shafts at the Dixie Road Underpass centre pier, and two of the completed drilled shafts at Ogden Pedestrian Bridge. The Contractor shall coordinate with and provide access to the Contract Administrator and their Foundation Engineering Specialist to conduct the Pile Integrity Tests.

These tests shall conform to the requirements of ASTM D5882-16.

A comprehensive report on the PIT test results including a profile analysis for each of the tested drilled shafts will be submitted to the Contractor within five days of testing.



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