



January 10, 2018

FOUNDATION INVESTIGATION AND DESIGN REPORT

**QEW-DIXIE ROAD UNDERPASS REPLACEMENT
STRUCTURE SITE No. 24-193,
QEW WIDENING FROM EAST OF CAWTHRA ROAD
TO THE EAST MALL,
CITIES OF MISSISSAUGA AND ETOBICOKE
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 2102-13-00 & 2432-13-00**

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REPORT

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PART A

FOUNDATION INVESTIGATION REPORT
QEW-DIXIE ROAD UNDERPASS REPLACEMENT
STRUCTURE SITE NO 24-193
QEW WIDENING FROM EAST OF CAWTHRA ROAD TO THE EAST MALL
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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 realignment and replacement of the Queen Elizabeth Way (QEW) underpass at Dixie Road in the City of Mississauga, Regional Municipality of Peel.

The purpose of this investigation is to establish the subsurface soil and bedrock conditions at the proposed structure location, including the associated high fill and approach embankments, by borehole drilling, rock coring and laboratory testing on selected soil and rock 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 (Number 2015-E-0001) for this project. The work has been carried out in accordance with Golder's Supplementary Specialty Plan for foundation engineering services for this project, dated June 6, 2016.

2.0 SITE DESCRIPTION

The existing QEW-Dixie Road underpass is located approximately 1.9 km east of the QEW-Cawthra Road interchange and 2.5 km west of the Highway 427-QEW interchange in the City of Mississauga. The QEW alignment in the project area is oriented generally in a southwest-northeast direction; for the purposes of this report, the QEW is described as being in an east – west orientation.

The existing underpass was constructed in 1953 and is approximately 62 m long by 18 m wide; is a three-span bridge with abutments and piers supported on spread footings founded at about Elevation 104.7 m. The existing approach embankments are about 5 m high relative to the surrounding grade. The natural ground surface in the vicinity of the underpass is at about Elevation 107 m; the QEW and Dixie Road in the vicinity of the underpass are at about Elevation 106 m and 112 m, respectively.

Land use in the northeast, northwest and southeast quadrants is primarily residential, and a large commercial development is located in the southwest quadrant.

3.0 INVESTIGATION PROCEDURES

3.1 Previous Investigation

In August 2014, a preliminary foundation investigation for the QEW / Dixie Road bridge structure replacement, was carried out at the site by Thurber Engineering Ltd. (Thurber) during which time a total of two boreholes, designated as Boreholes DR 14-01 and DR 14-02, were advanced. The results of the Thurber investigation are contained in their report titled "Preliminary Foundation Investigation and Design Report, QEW/Dixie Road Interchange Structure, Mississauga, Ontario", Report No. 19-1351-219, dated July 13, 2015 (GEOCRE 30M11-251).

The locations of the boreholes advanced by Thurber are shown on Drawing 1, and the borehole records including a summary of the laboratory testing results from this investigation are presented in Appendix A.



3.2 Current Investigation

The field work for the current foundation investigation was carried out in September and October 2016 and June 2017, during which time a total of eleven sampled boreholes (designated as Boreholes NW6-1, NW6-2, NW4-3, HF-2, HF-3 and DO-1 to DO-6) were advanced near the location of the structure foundation footprints and high fill approach embankments as follows:

Foundation Element	Nearest Boreholes
North High Fill / Approach Embankment	NW4-3, HF-3, DO-1
North Abutment	DO-2
Center Pier	DO-3, DO-4
South Abutment	DO-6
South High Fill / Approach Embankment	NW6-2, DO-5, NW6-1, HF-2

The location of the boreholes are shown on Drawing 1 and the borehole and drillhole records are included in Appendix B.

The field borehole investigation was carried out using a truck-mounted CME 75 drill rig, supplied and operated by Davis Drilling of Milton, Ontario. The boreholes were advanced through the overburden using 150 mm or 108 mm outside diameter solid stem augers and NW casing. Soil samples were obtained at 0.75 m and 1.5 m intervals of depth, using a 50 mm outer diameter split-spoon sampler driven by an automatic hammer in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586-08)¹.

The boreholes were typically advanced to auger and/or sampler refusal (i.e. inferred bedrock), at depths ranging from about 2.9 m to 9.8 m below existing ground surface. Samples of the bedrock were obtained using an 'NQ'-size rock core barrel and coring techniques in Boreholes DO-2 and DO-3. Photographs of the recovered bedrock core samples are provided in Appendix C.

The groundwater conditions and water levels in the open boreholes were observed during the drilling operations. 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, observed the drilling, sampling and in situ testing operations, and logged the boreholes. The samples were identified in the field, placed in appropriate containers, labelled and transported to Golder's Mississauga geotechnical laboratory where the samples underwent further visual examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate. Classification testing (water content, Atterberg limits and grain size distribution) was carried out on selected samples. Uniaxial unconfined compression strength (UCS) tests, Young's modulus and Poisson's ratio tests and core density determinations were carried out on selected specimens of the bedrock core samples by Geomechanics Inc., on behalf of Golder. The results of the geotechnical laboratory testing for the current investigation are included in Appendix C.

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



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A selected bedrock core sample and two selected soil samples were submitted to Maxxam Analytics (Maxxam) a Standards Council of Canada (SCC) accredited laboratory of Mississauga, Ontario for chemical analysis. The sample of bedrock core, specifically collected from Borehole DO-2 Run 1 advanced at the north abutment, was crushed and homogenized by Maxxam prior to testing. The soil and crushed bedrock samples were analyzed for a suite of corrosivity parameters, including conductivity, resistivity, soluble chloride, soluble sulphate and pH. The results of the chemical analyses are presented in Appendix C.

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 borehole/drillhole records and shown on Drawing 1 are positioned relative to MTM NAD 83 northing and easting coordinates and the ground surface elevations are referenced to Geodetic datum. The borehole locations and ground surface elevations are summarized below.

Borehole No.	Location (MTM NAD 83)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (Latitude, °)	Easting (Longitude, °)		
NW6-1	4,828,491.3 (43.596336)	299,318.3 (-79.567893)	106.0	3.5
NW6-2	4,828,522.7 (43.596619)	299,296.8 (-79.568160)	106.3	2.9
NW4-3	4,828,683.3 (43.598063)	299,100.6 (-79.570592)	108.1	4.7
HF-2	4,828,441.3 (43.595886)	299,370.6 (-79.567245)	105.2	4.0
HF-3	4,828,621.9 (43.597511)	299,190.3 (-79.569480)	107.5	3.2
DO-1	4,828,582.4 (43.597155)	299,214.9 (-79.569175)	107.3	3.3
DO-2	4,828,567.2 (43.597019)	299,225.4 (-79.569044)	107.0	9.8*
DO-3	4,828,536.7 (43.596744)	299,241.7 (-79.568842)	106.5	7.6*
DO-4	4,828,556.5 (43.596923)	299,257.5 (-79.568648)	106.8	3.8
DO-5	4,828,517.1 (43.596569)	299,294.2 (-79.568192)	106.4	4.7
DO-6	4,828,525.6 (43.596645)	299,285.0 (-79.568306)	106.5	3.5

* includes bedrock core of between 3.5 m and 6.2 m lengths

The current investigation was supplemented with previous boreholes advanced by Thurber. In its report, Thurber does not indicate the coordinate system to which the borehole locations were referenced; however, it is deduced that they were referenced to the UTM coordinate system, and the borehole locations have been converted to MTM NAD 83 for this report. The northing and easting coordinates for these boreholes in addition to the boreholes advanced by Golder are shown in plan and tabulated on Drawing 1.



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 Georgian Bay Formation that 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 detailed subsurface soil, bedrock and groundwater conditions as encountered in the boreholes advanced during the current 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 provided in Appendix B. The results of the in situ field tests (i.e. SPT “N” values) as presented on the borehole records and in sub-sections of Section 4.2 are uncorrected. The geotechnical laboratory testing plots are contained in Appendix C.

The stratigraphic boundaries shown on the borehole records and on the stratigraphic profile and cross-sections on Drawings 1 and 2 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 borehole and drillhole records govern any interpretation of the site conditions. It should be noted that the interpreted stratigraphy shown on Drawings 1 and 2 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 and granular fill (road base) or topsoil, underlain by a deposit of sand, underlain by deposits of silty clay residual soil at some borehole locations. The native soil deposits are underlain by shale bedrock. A more detailed description of the subsurface conditions encountered in the boreholes of the current and previous investigations is provided in the following sections.

4.2.1 Asphalt

Boreholes HF-2, DO-3 to DO-6, DR14-01 and DR14-02 were advanced through the roadway surface on the existing QEW or South Service Road and encountered a layer of asphalt varying in thickness from about 65 mm to 200 mm.

It is noted that in Borehole HF-2, a 255 mm thick layer of concrete was penetrated under the asphalt, and in Boreholes DO-6 and DO-5, a 100 mm to 230 mm thick layer of concrete was penetrated underlying the asphalt and granular fill.

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.)



4.2.2 Topsoil

In Boreholes NW6-1, NW6-2, NW4-3, HF-3, DO-1 and DO-2 a 0.2 m to 0.3 m thick layer of topsoil was encountered at the ground surface.

4.2.3 Fill

Fill was encountered underlying the asphalt or concrete, where present, (or between the asphalt and concrete in Boreholes DO-5 and DO-6) in all of the boreholes advanced through the roadways, and underlying the topsoil in Boreholes NW6-2 and DO-1. The fill varies in composition from sandy silt to sand to sand and gravel. The surface of the fill was encountered between about Elevation 104.8 m and 107.0 m and extends to depths of about 0.2 m to 1.7 m below ground surface (to between Elevation 104.7 m and 106.2 m).

The Standard Penetration Test (SPT) “N” values measured within the fill range from 8 blows to 22 blows per 0.3 m of penetration, indicating that the non-cohesive fill has a loose to compact relative density.

A grain size distribution test was carried out on a sample of the granular fill from Borehole DR14-02 and the results are included in Appendix A.

The water content measured on five samples of the granular fill ranges between 3 per cent and 18 per cent.

4.2.4 Silt to Silty Sand to Sand to Sand and Gravel

In all of the boreholes advanced at the site, a granular deposit consisting of silt to silty sand to sand to sand and gravel to gravelly sand was encountered underlying the topsoil and fill (where present). The surface of the granular deposit was encountered at depths of about 0.2 m to 1.7 m below ground surface (about Elevation 104.7 m to 107.9 m) and the thickness of the deposit was measured to be between about 1.0 m and 3.1 m (extending to between Elevation 102.5 m and 104.9 m). A 0.5 m thick layer of gravel was encountered within the sand to silty sand deposit in Borehole DO-6 at a depth of about 2.6 m below ground surface (Elevation 103.9 m).

The SPT “N” values measured within the granular deposit are between 0 blows (weight of hammer) and 53 blows per 0.3 m of penetration, indicating a very loose to very dense relative density.

The results of grain size distribution tests completed on two samples of the granular deposit from Boreholes DR14-01 and DR14-02 are included in Appendix A. Grain size distribution tests were carried out on ten selected samples of the granular deposit by Golder, and the results are shown on Figures C1A and C1B in Appendix C.

The natural water content measured on samples of the granular deposit are between 5 per cent and 25 per cent.

4.2.5 Gravelly Silty Sand Till

A 0.7 m thick till layer was encountered underlying the sand deposit in Borehole NW6-1 at a depth of about 2.5 m below ground surface (Elevation 103.5 m). The deposit is about 0.7 m thick at this location, extending to about Elevation 102.8 m. This non-cohesive till layer is comprised of gravelly silty sand and contains clayey silt pockets.

An SPT “N” value measured across the interface between the till deposit and the underlying bedrock is 54 blows for 0.25 m of penetration.

A grain size distribution test was carried out on a sample of the till deposit and the result is shown on Figure C2 in Appendix C. An Atterberg limits test was carried out on the fines portion of a sample of this till deposit and measured a liquid limit about of 16 per cent, a plastic limit of about 12 per cent, and a plasticity index of about 4 per cent. The test result, which is plotted on a plasticity chart on Figure C3 in Appendix C, indicates that the fines portion of the till deposit can be classified as a clayey silt of low plasticity.



The natural water content measured on a sample of the till is 13 per cent.

4.2.6 Clayey Silt (Residual Soil)

Underlying the native granular deposits in Boreholes HF-3, DO-1 to DO-3 and DO-5, a 0.1 m to 0.3 m thick deposit of residual soil was encountered at depths between about 2.7 m and 3.3 m below ground surface (between Elevation 103.3 m and 104.8 m). This deposit is interpreted to be derived from weathering of the underlying shale bedrock, and consists of clayey silt trace gravel to gravelly, some sand, containing varying amounts of shale fragments.

An Atterberg limits test was carried out on one sample of the residual soil deposit and measured a liquid limit about of 29 per cent, a plastic limit of about 18 per cent, and a plasticity index of about 11 per cent. The test result, which is plotted on a plasticity chart on Figure C4 in Appendix C, indicates that the residual soil can be classified as a clayey silt of low plasticity.

The natural water content measured on a sample of the till is 14 per cent.

4.2.7 Shale Bedrock

Bedrock was encountered and confirmed by augering and sampling in Boreholes HF-2, HF-3, DO-1, DO-4, DO-5, DO-6, NW4-3, NW6-1 and NW6-2, while bedrock core samples were recovered in Boreholes DO-2, DO-3, DR14-01 and DR14-02. The depths to bedrock below ground surface, and the corresponding bedrock surface elevation are summarized below.

Foundation Element / Approach Embankment	Borehole	Depth to Bedrock Surface / Refusal (m)	Bedrock Surface / Refusal Elevation (m)	Comments
North High Fill / Approach Embankment	NW4-3	3.2	104.9	Split-Spoon Sample
	HF-3	2.8	104.7	Split-Spoon Sample
	DO-1	3.2	104.1	Split-Spoon Sample
North Abutment	DO-2	3.3	103.7	Bedrock Cored
	DR 14-02 ¹	3.0	103.7	Bedrock Cored
Center Pier	DO-3	3.4	103.1	Bedrock Cored
	DO-4	3.0	103.8	Split-Spoon Sample
South Abutment	DO-6	3.3	103.2	Split-Spoon Sample
South High Fill / Approach Embankment	NW6-2	2.5	103.8	Split-Spoon Sample
	DO-5	3.4	103.1	Split-Spoon Sample
	NW6-1	3.2	102.8	Split-Spoon Sample
	HF-2	2.7	102.5	Split-Spoon Sample
	DR 14-01 ¹	3.6	102.9	Bedrock Cored

Note:

1. Thurber Engineering Ltd. Report No. 19-1351-219, dated July 13, 2015, GEOCRE 30M11-251.

In general, the bedrock surface as encountered in the area of the proposed underpass replacement is relatively level to gently sloping towards the south.



Based on a review of the bedrock core samples from the current investigation, the bedrock consists of shale of the Georgian Bay Formation. In general, the bedrock samples are described as highly to slightly weathered, thinly to very thinly laminated, very fine to fine grained, non-porous, weak, grey, with medium strong to strong limestone interbeds at varying intervals, as presented in the drillhole records in Appendix B, and shown on the photographs of the recovered core samples from Drillhole DO-3 on Figure C5 in Appendix C. The degree of weathering of the bedrock samples (i.e. fresh to slightly weathered – W1 to W2), 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 (ISRM³) standard classification system.

The Rock Quality Designation (RQD) measured on the core samples generally ranges from about 60 per cent to 100 per cent, except for an upper completely weathered zone in Borehole DO-3, indicating a rock mass of generally fair 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 generally between 90 per cent and 100 per cent and between 40 per cent and 100 per cent, respectively.

Two Unconfined Compression (UC) tests (ASTM D7012)⁵ were carried out on selected core samples of the shale bedrock obtained in Boreholes DO-2 and DO-3 and measured compressive strengths of about 13.1 MPa and 5.5 MPa, as shown on the Rock Laboratory Test Result Reports from Geomechanica Inc. in Appendix C. The measured Young's moduli are 302 MPa and 1010 MPa.

Based on the laboratory UCS tests, in accordance with Table 3.5 in CFEM (2006)⁴, the shale bedrock is classified as weak (R2, 5 MPa < UCS < 25 MPa). As noted above, however, the shale contains medium strong to strong limestone interlayers.

4.2.8 Groundwater Conditions

The overburden samples obtained from the current investigation boreholes 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 1.5 m and 4.1 m (between Elevation 106.0 m and 103.6 m), to dry upon completion of drilling at depths between 3.4 m and 4.7 m below ground surface (between Elevations 103.1 and 101.2 m); however, these observations are not necessarily representative of the stabilized groundwater level at the site. Standpipe piezometers were installed in the boreholes advanced in the preliminary investigation by Thurber and the following summarizes the water levels recorded in the piezometers about one to two months after the 2014 borehole drilling:

Borehole	Nearest Foundation Unit	Stratum Sealed Into	Water Level Depth (m)	Water Elevation (m)	Date
DR 14-01	South Approach Embankment	Bedrock	3.8	102.7	September 29, 2014
			3.5	103.0	October 27, 2014
DR 14-02	North Abutment	Bedrock	3.4	103.3	September 29, 2014
			2.8	103.9	October 27, 2014

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



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

As discussed in Section 3.2, a bedrock core sample and two soil samples were submitted for analysis of parameters used to assess the potential corrosivity of the site soil and bedrock to steel and concrete. Detailed analytical test results are included in Appendix C and the following table summarizes the results of the testing:

Parameter	Borehole DO-2 Bedrock Run #1 (Elev. 102.9 m)	Borehole DO-4 Soil Sample 4 (Elev. 104.2 m)	Borehole DO-5 Soil Sample 2 (Elev. 104.5 m)
pH	8.02	8.14	8.10
Resistivity (ohm-cm)	3,500	1,200	440
Electrical Conductivity (umho/cm)	284	828	2,250
Chlorides (ug/g)	28	450	1,300
Soluble Sulphates (ug/g)	110	61	27



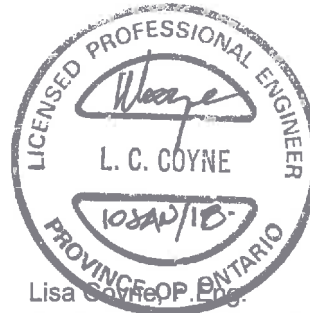
5.0 CLOSURE

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

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GWP 2102-13-00 & 2432-13-00



6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides detail foundation engineering design recommendations for the proposed Dixie Road underpass replacement (Site No. 24-193) as part of the widening of the Queen Elizabeth Way (QEW) from Cawthra Road to the Highway 427 area. These recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigations at this site. The discussion and recommendations presented are intended to provide the designer with sufficient information to assess the feasible foundation alternatives and carry out the design of the bridge foundations.

The foundations discussions and recommendations are intended for the use of the Ministry of Transportation, Ontario (MTO) and their 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 the Foundation Investigation Report (Part A of this 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

The existing 62 m long, three-span structure will be replaced on a new alignment, with the centreline of Dixie Road re-aligned approximately 35 m to the east. Based on the General Arrangement (GA) drawing provided by AECOM on May 26, 2017, the proposed 74.5 m long, two-span replacement structure will consist of a 39 m long southern span and a 35.5 m long northern span, with a pier located at the QEW median. The existing ground surface at the borehole locations varies between Elevations 105.2 and 108.1 m, generally sloping gently downward to the south. It is understood that the proposed grade at the abutments will be approximately Elevation 113.9 m, resulting in north and south approach embankments that are about 6.9 m high relative to the existing ground surface.

6.2 Consequence and Site Understanding Classification

The proposed bridge crosses over the QEW which carries large volumes of traffic with the potential to impact alternative transportation corridors. In accordance with Section 6.5 of the 2014 *Canadian Highway Bridge Design Code* and its *Commentary* (CHBDC 2014), the proposed bridge and its foundation system are 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 (2014), 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 (2014) have been used for design.

6.3 Foundations Options

Both shallow and deep foundation options have been considered for support of the new underpass. For all foundation options, temporary protection systems will be required along the edges of the QEW to facilitate the removal of the existing structure foundations, and possibly along the highway median and along the North and South Service Roads to facilitate construction of the new foundations (depending on the foundation alternative selected). It is anticipated that some groundwater seepage may occur into the excavations from within the cohesionless fills and native soils.



A summary of the advantages and disadvantages associated with each option is provided below, and a comparison of the alternative foundation options based on advantages, disadvantages, risks and relative costs is provided in Table 1 following the text of this report.

- **Strip or spread footings founded on the loose to compact silt/sand/gravel deposit, or on the shale bedrock:** Shallow footings are feasible at this site due to the generally compact nature of the overburden soils and shallow depth to bedrock (at or less than about 1 m below the existing structure footing founding level). The existing structure is founded on spread footings and has performed well. This option would require excavation to a depth of about 2 m below the existing grade for footings founded on the compact native silt/sand/gravel deposit, or about 3.5 m below the existing grade for footings founded on the shale bedrock. Deeper excavations to bedrock would require greater groundwater control. This option does not allow for the construction of integral abutments, but could permit semi-integral abutments.
- **Footings “perched” on a compacted granular pad in the approach embankments:** Shallow footings “perched” within the proposed approach embankments are feasible and could minimize the depth of excavation below the existing grade (notwithstanding the requirement to remove the existing structure foundations on the previous alignment). This option would not allow for the construction of integral abutments and would provide lower bearing resistances than for footings founded on bedrock.
- **Steel H-piles or pipe piles founded within the shale bedrock:** Due to the shallow depth to bedrock, deep foundations for the proposed structure would be most practical if used in an integral abutment configuration with the pile caps “perched” within the approach embankments; this option would minimize the depth of excavation (particularly compared to a strip footing founded on the bedrock) and associated protection systems and groundwater control. Depending on the abutment geometry, it may be necessary to form sockets in the bedrock by coring and/or churn drilling to allow the piles to be placed within and concreted into the bedrock, essentially similar to drilled shaft (caisson) construction, to achieve the required pile lengths for integral abutment construction. Although the shale bedrock is weak, it contains interlayers of medium strong to strong bedrock, and appropriate equipment and techniques will be required to form the sockets within the bedrock.
- **Drilled shafts (caissons) founded in the shale bedrock:** Drilled shafts are feasible for support of the abutments (although they would not permit integral abutment construction) and pier for the proposed new structure. Due to the relatively shallow depth to bedrock, it would be necessary to form sockets in the bedrock by pre-augering/coring; in addition, temporary liners are recommended to support the sides of the caisson holes through the non-cohesive overburden soils and minimize ground loss during construction. At the pier, caissons may in fact be advantageous over spread footings from a geotechnical/foundations perspective if they can be constructed as structural columns/caissons without a below-grade pile cap, which could reduce the depth of excavation and temporary protection systems that would be required adjacent to the highway.

Based on the above considerations, the preferred option from a geotechnical/foundations perspective is to support the abutments for the proposed new structure on shallow spread footings founded on the near-surface bedrock if integral abutments are not adopted, or on steel H-piles founded within the shale bedrock in an integral abutment configuration; this option is likely to require some pre-augering/coring to form sockets in the rock to achieve the minimum pile length. At the pier, the preferred option from a geotechnical/foundations perspective is drilled shafts socketted within the bedrock, particularly if the pile cap can be eliminated with the structural columns placed



directly on the drilled shafts. Alternatively, strip footings founded on the native soils or bedrock present a viable alternative from a geotechnical/foundations perspective.

6.4 Strip Footings

6.4.1 Founding Elevations

Detailed below, for each foundation element and associated concrete wingwalls / retaining walls, are the recommended founding elevation for strip footings on bedrock or on the native compact silty sand to sand and gravel deposits.

Structural Element	Reference Borehole(s)	Highest Founding Elevation on Native Deposits (m)	Bedrock Surface Elevation (m)	Highest Founding Elevation on Bedrock (m)
North Abutment	DO-2 DR 14-02	105.0	103.7 103.7	102.0
Centre Pier	DO-3 DO-4	105.0	103.1 103.8	101.7
South Abutment	DO-5 DO-6	104.0	103.1 103.2	101.5

Factored geotechnical resistances for footing founded at the elevations recommended above are provided in Section 6.4.2.

Consideration could also be given to subexcavation of the loose/soft soils to the founding elevation given above and replacement with compacted granular fill to permit footings to be founded at a higher elevation. Notwithstanding these requirements, strip footings should be founded at a minimum depth of 1.2 m below the lowest surrounding grade to provide adequate protection against frost penetration (per OPSD 3090.101 – *Foundation Frost Depths for Southern Ontario*). If adequate soil cover cannot be provided for footings (for example, for retaining walls adjacent to the abutments), rigid styrofoam insulation could be installed to compensate for the lack of soil cover and provide protection from frost penetration.

Alternatively, the abutment foundations could be “perched” on a compacted granular pad in the approach embankments above the QEW grade. In this case, the compacted granular pad should have a minimum thickness of 2 m; any existing fill, organic soils and/or loose soils within the zone of influence below the compacted granular pad should be subexcavated and replaced with engineered fill, or the pad thickened to found on the compact native silty sand to sand and gravel deposits at the elevations given above for footings founded on these deposits. The pad should consist of OPSS.PROV 1010 (Aggregates) Granular ‘A’ material extending at least 1 m beyond the edges of the footing(s), then outward and downward at 1H:1V. The granular fill should be placed in accordance with OPSS.PROV 501 (Compacting).

6.4.2 Geotechnical Resistances

Strip footings placed on the native sandy soils, on the sound shale bedrock, or perched on a compacted Granular ‘A’ pads within the approach embankments founded at or below the design elevations given in the preceding section, should be designed based on the factored ultimate geotechnical resistances and factored serviceability geotechnical resistances (for 25 mm of settlement) given below.



Founding Stratum	Assumed Footing Width	Factored Ultimate Geotechnical Resistance	Factored Serviceability Geotechnical Resistance¹
Abutments, pier, and/or retaining wall footings on compact native sandy soils	3 m	500 kPa	300 kPa
	4 m	500 kPa	225 kPa
Abutments, pier, and/or retaining wall footings on “sound” shale bedrock	3 m	850 kPa	700 kPa
	4 m	850 kPa	550 kPa
Abutments, pier, and/or retaining wall footings on compacted Granular ‘A’ pad following subexcavation of soft/loose soils ²	3 m	550 kPa	350 kPa
	4 m	550 kPa	300 kPa
Abutments or retaining wall perched in approach embankments on compacted Granular ‘A’ pad ³	3 m	600 kPa	350 kPa
	4 m	600 kPa	300 kPa

1. For 25 mm of settlement.
2. For minimum 2 m thick granular pad founded at Elevation 105.0 m for north abutment and center pier, and at Elevation 104.0 m at south abutment.
3. For minimum 2 m thick granular pad founded within approach embankment fill.

The geotechnical resistances should be reviewed if the selected footing width or founding elevations differ from those given above. 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 6.10.4 of the CHBDC (2014).

The footing subgrade should be inspected, in accordance with OPSS.PROV 902 (*Excavating and Backfilling Structures*) to check that all existing fill, native soils have been removed. Furthermore, following excavations into the 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 shale or native soil 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 (100 mm thick of 20 MPa compressive strength concrete) be placed in the excavation within four hours to protect the integrity of the subgrade. A NSSP to address this item is included in Appendix D, which should be included in the Contract Documents.

Descriptions of the shale bedrock core samples, and laboratory testing, indicates the shale is weak with medium strong to strong limestone layers, and where excavation for the abutment and pier 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 6.11.2. As also discussed in Section 6.11.6,



vibration monitoring is not anticipated to be required during hoe-ramming activities, neither on the existing bridge nor on the nearest residences which are approximately 60 m away from the proposed bridge.

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 6.10.5 of the CHBDC (2014). For cast-in-place concrete footings constructed directly on bedrock, on native soils, or on a concrete working slab, the sliding resistance may be calculated based on the unfactored coefficient of friction, $\tan \delta$, which can be taken as follows:

- Cast-in-place footing or working slab to native sandy deposits: $\tan \delta = 0.6$
- Cast-in-place footing or working slab to shale bedrock: $\tan \delta = 0.5$
- Cast-in-place footing or working slab to Granular A pad: $\tan \delta = 0.6$
- Cast-in-place footing to concrete working slab: $\tan \delta = 0.7$

If necessary, the sliding resistance between the concrete footing and/or working slab and the bedrock at the abutments and pier can be supplemented by dowelling into the bedrock. The horizontal resistance of the dowels is dependent on the strength of the bedrock, grout and steel. The dowels should have a minimum length within the bedrock 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 value of 250 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 D. These values assume that construction is carried out in dry conditions.

6.4.4 Frost Protection

The footings should be provided with a minimum 1.2 m of soil cover to provide adequate protection against frost penetration (per OPSD 3090.101 – *Foundation Frost Depths for Southern Ontario*).

6.5 Steel H-Pile or Pipe Pile Foundations

6.5.1 Founding Elevations

Consideration can be given to supporting the abutments and pier on steel HP 310x110 piles, or closed-end, concrete-filled, 324 mm (12 ¾ in.) diameter steel pipe piles having a minimum wall thickness of 9.5 mm (3/8 in.). Due to the shallow depth to bedrock, pile foundations for the proposed structure would be most practical if used in an integral abutment configuration with the pile tops “perched” within the approach embankments. At the centre pier, and possibly at the abutments, depending on geometry, it may be necessary to form sockets in the bedrock by coring and/or churn drilling to allow the piles to be placed within and concreted into the bedrock, to achieve the required pile lengths for integral abutment construction. A minimum 600 mm diameter socket is typically required for the installation of approximately 300 mm diameter piles (HP310x110) due to out-of-plumb / out-of-alignment drilling and to allow for accurate placement and alignment of the piles. To provide pile toe fixity, the bottom 1 m of the pre-augered hole should be backfilled with minimum 30 MPa concrete, the suitability of which should also be checked by the structural engineer. Above this level, the pre-augered hole should be backfilled with OPSS.PROV 1010 Granular B Type II.

The table below provides founding elevations for pre-augered socketted piles, based on extending into “sound” bedrock below observed clay seams and zones of weathering, along with an approximate pile length based on



design pile top elevations of 107.0 m at the south abutment and 107.2 m at the north abutment (from the GA Drawing supplied by AECOM). It has been assumed that if pre-augered socketted pile foundations are adopted at the centre pier, the underside of the pile cap will be at approximately at Elevation 105.3 m to provide the minimum founding depth of 1.2 m for frost protection purposes. The structural designers should assess whether the pile lengths given below are sufficient from a structural perspective. If longer pile lengths are required from a structural perspective, deeper pre-augering/coring would be needed to penetrate further into the shale bedrock. 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 with respect to socket formation, as well as the backfilling requirements; an NSSP is included in Appendix D for this purpose.

Foundation Element	Borehole	Estimated Underside of Pile Cap Elevation (m)	Bedrock Surface Elevation (m)	Highest Design Pile Tip Elevation (m)	Estimated Approx. Pile Length below Pile Cap (m)
North Abutment	DO-2	107.2	103.7	102.0	5.2
	DR 14-02		103.7		
Centre Pier	DO-3	105.3	103.1	101.7	3.6
	DO-4		103.8		
South Abutment	DO-5	107	103.1	101.5	5.5
	DO-6		103.2		

As discussed further in Section 6.11.6, vibration monitoring is not anticipated to be required during deep foundation construction activities, either for the existing bridges or for the nearest residential buildings.

The shale bedrock is weak (with unconfined compressive strengths typically in the range of 5 MPa to 15 MPa), with medium strong to strong limestone interbeds (with unconfined compressive strengths of greater than 50 MPa), and so the sockets would have to be advanced into the rock by churn drilling or rock coring. If pre-augered socketted pile foundations 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 D for this purpose

6.5.2 Geotechnical Axial Resistances

For HP310x110 steel H-piles founded within the shale 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 for 25 mm of settlement (for the length of piles required at this site) will be greater than the factored ultimate geotechnical resistance; as such, the factored ultimate geotechnical resistance will govern for this foundation type.

6.5.3 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. For integral abutment design the steel H-piles would be installed within a 3 m long corrugated steel pipe (CSP) filled with sand fill in accordance with Table 1 in the NSSP for integral abutments.



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, it should be 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 percent 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 (2014) 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).

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).

Foundation Element	Soil Deposit	Elevation (m)	n_h (kPa/m)
North Abutment	Loose sand within CSP	107.2 to 104.2	2,000
	Loose to compact silty sand to sand	94.2 to bedrock	7,000
Centre Pier	Sand and gravel to sandy silt (fill)	106.5 to 106.2	5,000
	Compact to dense sand	106.2 to bedrock	7,500
South Abutment	Loose sand within CSP	107.0 to 104.0	2,000
	Compact silty sand to sand to gravel	104.0 to bedrock	7,500
General	Compacted Granular B Type II	As established by structural designers	7,000
	Granular B Type II in annulus of pre-augered hole		6,000

Both the structural and geotechnical resistances of the piles should be evaluated to establish the governing case at Ultimate Limit States (ULS). At Serviceability Limit States (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 reaction 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 (2014) Commentary Section 6.11.2.2). For piles concreted into pre-augered holes in the shale bedrock, the lateral resistance of the portion of the pile within the concrete can be calculated using the diameter of the socket/pre-augered. The minimum strength of concrete should be checked by the structural engineer based on the anticipated lateral loads.



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.

6.5.4 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 Drilled Shafts (Caissons)

6.6.1 Founding Elevations

Drilled shaft foundations could also be considered for support of the abutments and/or pier. Drilled shafts should be socketed a minimum of 3 m into the good quality shale bedrock. A temporary or permanent liner and placement of concrete by the tremie method will likely be required to install drilled shafts. The following details the estimated rock socket elevations at the abutments and piers:

Foundation Element	Borehole Nos.	Bedrock Surface Elevation (m)	Top of Rock Socket Elevation (m)	Estimated Caisson Tip Elevation (m)
North Abutment	DO-2	103.7	102.0	99.0
	DR 14-02	103.7		
Centre Pier	DO-3	103.1	101.7	98.7
	DO-4	103.8		
South Abutment	DO-5	103.1	101.5	98.5
	DO-6	103.2		



The shale bedrock is weak with unconfined compressive strengths in the range of 5 MPa to 15 MPa and with medium strong to strong limestone layers, and so the sockets may likely be advanced into the rock by churn drilling or rock coring. If drilled shafts 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 D for this purpose.

6.6.2 Geotechnical Axial Resistance

For drilled shafts socketed 3 m or more into the good quality bedrock (with the top of the rock socket commencing at the elevations in Section 6.6.1 or below these elevations), the factored ultimate geotechnical resistance may be taken as follows:

Foundation Element	Caisson Diameter (m)	Factored Ultimate Geotechnical Resistance (kN)
North Abutment	1.0	2,500
	1.2	3,800
	1.5	6,000
Centre Pier	1.0	2,500
	1.2	3,800
	1.5	6,000
South Abutment	1.0	2,500
	1.2	3,800
	1.5	6,000

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 conditions do not apply for this structure site.

For drilled shafts designed for end-bearing and shaft friction combined, 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. Immediately following cleaning and inspection of the rock socket the concrete must be tremied into the caisson.

The centre-to-centre spacing between drilled shafts within a group founded in bedrock should be greater than 2.5 times the drilled shaft diameter to limit interaction between drilled shafts. Provided this minimum drilled shaft 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).



6.6.3 Resistance to Lateral Loads

The geotechnical resistance to lateral loading for the caissons should be calculated in accordance with Section 6.5.3, using the horizontal subgrade formulas and parameter values presented therein.

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/within the bedrock, the site may be classified as Site Class C in accordance with Table 4.1 of the CHBDC (2014), 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 abutment foundations. For example, Table 4.1 of the CHBDC (2014) 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 (2014), the peak ground acceleration (PGA) values and design spectral acceleration (Sa) values for Site Class C are presented below.

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)	0.041	0.075	0.144
PGV (m/s)	0.031	0.052	0.092
Sa (0.2) (g)	0.069	0.120	0.223
Sa (0.5) (g)	0.042	0.067	0.116
Sa (1.0) (g)	0.023	0.036	0.059
Sa (2.0) (g)	0.011	0.017	0.028
Sa (5.0) (g)	0.0023	0.0039	0.0067
Sa (10.0) (g)	0.001	0.0016	0.0028

6.8 Lateral Earth Pressures for Design of Abutments and Wingwalls

The lateral earth pressures acting on the abutment walls and any associated wingwalls will depend on the type and method of placement of the backfill materials, the nature of the soils behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls. Seismic (earthquake) loading must also be taken into account in the design.

The following recommendations are made concerning the design of the abutment/wing walls:



- Select, free draining granular fill meeting the specifications of OPSS.PROV 1010 (Aggregates) Granular 'A' or Granular B Type II, should be used as backfill behind the walls. Longitudinal drains or weep holes should be installed to provide positive drainage of the granular backfill. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS.PROV 501 (Compacting). Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSS 3101.150 (Walls, Abutment, Backfill, Minimum Granular Requirement), OPSS 3121.150 (Walls, Retaining, Backfill, Minimum Granular Requirement), and 3190.100 (Walls, Retaining and Abutment, Wall Drain).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with CHBDC (2014) Section 6.12.3 and Figure 6.6. Care must be taken during the compaction operation not to overstress the wall. Heavy construction equipment should be maintained at a distance of at least 1 m away from the walls while the backfill soils are being placed. Hand-operated compaction equipment should be used to compact the backfill soils immediately behind the walls as per OPSS.PROV 501. Other surcharge loadings should be accounted for in the design, as required.
- For restrained walls, granular fill should be placed in a zone with the width equal to at least 1.2 m behind the back of the wall on Figure C6.20(a) of the Commentary to the CHBDC (2014). For unrestrained walls, fill should be placed within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing or pile cap on Figure C6.20(b) of the Commentary to the CHBDC (2014).

6.8.1 Static Lateral Earth Pressures for Design

The following guidelines and recommendations are provided regarding the lateral earth pressures for static (i.e., not earthquake) loading conditions. These lateral earth pressures assume that the ground above the wall will be flat, not sloping. If the inclination of the slope above the wall changes then new lateral earth pressures will need to be calculated.

- For a restrained wall, the pressures are based on the proposed embankment fill and the following parameters (unfactored) may be used assuming the use of earth fill:

Material	Earth Fill
Soil Unit Weight:	20 kN/m ³
Coefficients of static lateral earth pressure:	
Active, K_a	0.33
At rest, K_o	0.50
Passive, K_p	3.0

- For an unrestrained wall, the pressures are based on using engineered granular fill and the following parameters (unfactored) may be used:



Material	Granular 'A'	Granular 'B' Type II
Soil Unit Weight:	22 kN/m ³	21 kN/m ³
Coefficients of static lateral earth pressure:		
Active, K_a	0.27	0.27
At rest, K_o	0.43	0.43
Passive, K_p	3.7	3.7

- If the wall support and superstructure allow lateral yielding, active earth pressures may be used in the geotechnical design of the structure. The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure, may be taken as:
 - Rotation of approximately 0.002 about the base of a vertical wall (where the rotation is calculated as the horizontal displacement divided by the height of the wall);
 - Horizontal translation of 0.001 times the height of the wall; or,
 - A combination of both.
- If the wall does not allow lateral yielding (i.e., restrained structure where the rotational or horizontal movement is not sufficient to mobilize an active earth pressure condition), at-rest earth pressures (plus any compaction surcharge) should be assumed for geotechnical design.

6.8.2 Seismic Lateral Earth Pressures for Design

Seismic (earthquake) loading must also be taken into account in the design of retaining / wing walls in accordance with Section 4.6.5 of the CHBDC (2014). In this regard, the following should be included in the assessment of lateral earth pressures:

- Seismic loading will result in increased lateral earth pressures acting on the abutment stem and/or retaining walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure.
- In accordance with Sections 4.6.5 and C.4.6.5 of the CHBDC (2014) and its Commentary, for structures which allow lateral yielding, the horizontal seismic coefficient, k_h , used in the calculation of the seismic active pressure coefficient, is taken as 0.5 times the site-specific PGA. For structures that do not allow lateral yielding, k_h is taken as equal to the site-specific PGA. For both cases the value of the vertical seismic coefficient k_v is taken as zero.
- The following seismic active pressure coefficients (K_{AE}) may be used in design; these coefficients reflect the maximum K_{AE} obtained for each of the earthquake design periods and backfill conditions. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is level. Where sloping backfill is present above the top of the wall, the lateral earth pressures under seismic loading conditions should be calculated by treating the weight of the backfill located above the top of the wall as a surcharge.



	Design Earthquake	Site PGA	Seismic Active Pressure Coefficients, K_{AE}		
			Granular A	Granular B Type II	Earth Fill
Yielding Wall	475-Yr	0.041g	0.26	0.26	0.31
	975-Yr	0.075g	0.27	0.27	0.32
	2,475 Yr	0.144g	0.29	0.29	0.35
Non-Yielding Wall	475-Yr	0.041g	0.27	0.27	0.33
	975-Yr	0.075g	0.29	0.29	0.35
	2,475 Yr	0.144g	0.34	0.34	0.40

- The K_{AE} value for a yielding wall is applicable provided that the wall can move up to $250k_h$ mm, where k_h is the site specific PGA as given in the table above. This corresponds to displacements of 10, 19, and 36 mm for the 475-year, 975-year, and 2,475-year design earthquakes at this site.
- The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e. an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

$$\sigma_h(d) = K_a \gamma d + (K_{AE} - K_a) \gamma (H-d), \text{ yielding walls}$$

$$\sigma_h(d) = K_o \gamma d + (K_{AE} - K_a) \gamma (H-d), \text{ non-yielding walls}$$

Where: $\sigma_h(d)$ is the (static plus seismic) lateral earth pressure at depth, d , (kPa);
 K_a is the static active earth pressure coefficient;
 K_o is the static at-rest earth pressure coefficient;
 K_{AE} is the seismic active earth pressure coefficient;
 γ is the unit weight of the backfill soil (kN/m^3), as given in Section 6.8.1;
 d is the depth below the top of the wall (m); and,
 H is the total height of the wall (m).

6.9 Analytical Testing for Construction Materials

The results of an analytical test on one sample of bedrock from Borehole DO-2 and two samples of soil from Boreholes DO-4 and DO-5 are presented in Section 4.2.9 and in Appendix C. The analytical test results were compared to CSA A23.1 Table 3 ("*Additional requirements for concrete subjected to sulphate attack*") for potential sulphate attack on concrete. The sulphate concentrations measured in the tested samples are below the exposure class of S-3 (Moderate). Therefore, based on the single sample of bedrock and 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.

The analytical test results of the bedrock and soil samples were also compared to Table 2 of the U.S. Criteria for Assessing Ground Corrosion Potential (as derived from Federal Highways Administration (FHWA) 2003) for the potential attack on buried steel. The sulphate and chloride concentrations measured in the bedrock sample



indicates “Mild to no corrosion potential” and the resistivity measured indicates the corrosion potential to be less than strong, but greater than mild. The chloride concentration and resistivity values measured in the soil samples indicate “strong corrosion potential”. Based on the results of the samples tested, and given that the structure is located under/adjacent to the roadway and will be exposed to de-icing salt, consideration should be given by the designer to designing for a “C” type exposure class as defined by CSA A23.1 Table 1.

It is ultimately up to the structural designer to determine the appropriate exposure class and to ensure that all aspects of CSA A23.1 Section 4.1.1 “Durability Requirements” are followed.

6.10 High Fill and Approach Embankment Design and Construction

The replacement structure will be located to the east of the existing Dixie Road embankment and structure, and new approach embankments up to about 6.9 m high will be constructed. The proposed new high fill embankments (i.e. embankments greater than 4.5 m in height) will extend approximately 80 m south and 150 m north of the new underpass structure.

Boreholes NW4-3, HF-3 and DO-1 were advanced in the vicinity of the north approach / high fill embankment and encountered topsoil underlain by compact sand and gravel fill and/or very loose to dense native sand to a depth of about 3.2 m, underlain by shale bedrock.

Boreholes NW6-1, NW6-2, DO-5, HF-2 and DR 14-01 were advanced in the vicinity of the south approach / high fill embankment and generally encountered topsoil or asphalt pavement underlain by compact sand to sand and gravel fill and/or very loose to dense native silty sand to sand to depths between about 2.5 m and 3.6 m, underlain by shale bedrock.

6.10.1 Subgrade Preparation and Embankment Construction

Prior to construction of the new approach embankments it is recommended that any topsoil/organic soils and loosened/softened fill be stripped from within the embankment footprint.

Fill for construction of the new embankments should consist of Granular ‘B’ Type I, Type II or Select Subgrade Material meeting the specifications of OPSS.PROV 1010 (Aggregates). The embankment fill should be placed and compacted in accordance with OPSS.PROV 501 (Compacting) and OPSS.PROV 206 (Grading). Embankment side slopes should be constructed no steeper than 2 horizontal to 1 vertical (2H:1V) in granular or earth fill.

In accordance with MTO’s standard practice, a minimum 2 m wide bench should be provided where embankment slopes are greater than 8 m in height, such that the uninterrupted slope height does not exceed 8 m, consistent with OPSD 202.010 (Slope Flattening).

To reduce surface water erosion on the granular embankment side slopes, topsoil and seeding as per OPSS 802 (Topsoil) and OPSS.PROV 804 (Seed and Cover) should be carried out as soon as possible after construction of the embankments. If this slope protection is not in place before winter, then alternate protection measures, such as covering the slope with straw, or gravel sheeting as per OPSS 511 (Rip Rap, Rock Protection and Granular Sheeting), and OPSS.PROV 1004 (Aggregates – Miscellaneous) will be required to reduce the potential for erosion and to reduce the potential for the requirement of remedial works on the side slopes in the spring prior to topsoil dressing and seeding.



6.10.2 Global Stability

Limit equilibrium slope stability analyses were performed on the north and south approach embankment side slopes using the commercially available program “Slide V.6” published by Rocscience Inc., employing the Morgenstern-Price method of analysis. For all analyses, the Factor of Safety (FoS) of numerous potential failure surfaces was computed in order to establish the minimum FoS. The FoS is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. A target minimum factored FoS of 1.5 is adopted for the design of embankment slopes under static conditions at the end of construction as per the CHBDC (2014). This FoS is considered adequate for the embankments at this site considering the design requirements and the field data available. The stability analyses were performed to assess if the target minimum FoS was achieved for the design embankment height and geometries. In general, circular slip surfaces were analysed in the design. ,

For the new granular/earth fill and native soil deposits, effective stress parameters were employed in the analysis assuming drained conditions and the parameters were estimated from empirical correlations using the in-situ SPT ‘N’-values. The correlations proposed by Terzaghi and Peck (1967) were employed and the results were tempered by engineering judgment based on precedent experience in similar soils.

Summarized below are the simplified stratigraphy and the associated strengths and unit weights employed for the different soil types in the proposed north approach area.

Soil Deposit	Bulk Unit Weight (kN/m ³)	Effective Friction Angle (°)	Undrained Shear Strength (kPa)
Granular embankment fill	21	32°	--
Existing compact sand and gravel fill	20	32°	--
Very loose to dense native silty sand to sand	19	30°	--

The analysis indicates that the north and south approach embankments constructed of granular fill will have a factored FoS greater than 1.5 against global instability, as shown on Figure 1.

6.10.3 Settlement

Settlement of the founding soils under the north and south approach embankment areas can be expected as a result of the loading from the new fills on the shallow deposits of cohesionless sandy soils. Settlement of new granular fill that is properly placed and compacted for construction of the widened embankments would occur during construction.

To estimate the magnitude of the expected immediate settlements of the subgrade material, analyses were carried out using hand and spreadsheet calculations. The immediate compression of the existing fill and native cohesionless deposits was modelled by estimating an elastic modulus of deformation based on the SPT ‘N’-values and using correlations proposed by Bowles (1984) and Kulhawy and Mayne (1990).

The simplified stratigraphy, together with the associated strengths and unit weights employed for the different foundation soil types at the north and south approach embankments, are summarized below.



Borehole/Approach	Soil Type	Approximate Thickness (m)	Bulk Unit Weight (kN/m ³)	Elastic Modulus (MPa)
Boreholes NW4-3, HF-3 and DO-1 North Approach	Loose to compact sand and gravel fill	1.4	20	20
	Loose to compact sand	2.0	19	20
Boreholes NW6-1, NW6-2, HF-2, DO-5 and DR 14-01 South Approach	Compact to dense gravelly sand to sand	3.0	20	25
	Stiff clayey silt with sand (Residual Soil)	0.3	20	75

6.10.4 Settlement Performance Requirements

The settlement performance criterion for design of high fill embankments is in accordance with MTO's Guideline "Embankment Settlement Criteria for Design" (2010).

Where new embankments approach structural elements, the following post-construction settlement and differential settlement criteria are considered acceptable for settlements to occur within 20 years post-paving for the bridge approach embankments at this site.

Location	Maximum Limits During Pavement Design Life	
	Distance from Transition Point (i.e., Abutment)	Total Post-Construction Settlement
Transition/Taper to Bridge Abutments	0 m to 20 m	25
	20 m to 50 m	50
	50 m to 75 m	75
	>75 m	>100

The total settlement and differential settlement rate are considered to be applicable over a 20-year period following completion of construction (i.e., final paving). These performance criteria form part of the overall design performance for the embankment in the vicinity of the approach embankments.

6.10.5 Results of Analysis

Based on the analysis using the above parameters the settlement of the up to 6.9 m high approach/high fill embankment is expected to be less than 25 mm, and this settlement is expected to occur during construction.

6.11 Construction Considerations

6.11.1 Excavation and Control of Groundwater and Surface Water

For the option of shallow foundations, the excavations for foundations at the pier and abutments will extend to a depth of up to 5.5 m, assuming that spread footings are founded directly on competent bedrock surface at the maximum founding elevations provided in Section 6.4.1. If consideration is given to founding the spread/strip footings on the compact native sandy soil deposits, the excavations will only need to extend to a depth of up to about 2.5 m below the surrounding grade.

If and where feasible, open-cut excavations must be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA 2017) and Regulation for Construction Activities. The existing fill and native sandy soils are classified as Type 3 soils, while the till and residual soils are classified as Type 2 soils,



according to the OHSA. Temporary excavations (i.e. those that are open for a relatively short time period) should be made with side slopes no steeper than 1H: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 pier and abutments. Recommendations for temporary protection systems are provided in Section 6.11.3.

The groundwater level at the site varies from about Elevation 102.7 m to 103.9 m, which corresponds to about the elevation of the bedrock surface, and will be higher during wetter periods of the year. Excavations for construction of the abutments and pier may extend below the water level (depending on the foundation alternative selected); however, it is expected that water inflow through the shale bedrock can be handled by pumping from well filtered sumps located outside the foundation footprint.

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.

6.11.2 Bedrock Excavation

Depending on the foundation alternative selected, bedrock excavation may be required to allow founding the strip footings at the recommended founding elevations provided in Section 6.4.1, and to achieve the factored geotechnical resistances provided in Section 6.4.2. Bedrock excavation may be carried out using hoe-ramming techniques. Excavations in the bedrock may be made with near-vertical side slopes/faces.

The shale bedrock at the site is weak (corresponding to unconfined compressive strengths in the range of 5 MPa to 15 MPa, but contains medium strong to 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 into the bedrock will require appropriate equipment and construction procedures. An NSSP is provided in Appendix D for inclusion in the Contract Documents.

6.11.3 Temporary Protection Systems

To facilitate construction of the new underpass foundations, and removal of the existing underpass foundations (if required), temporary protection systems are expected to be required along the north and south side of the QEW lanes, along the QEW median, and along the north and south sides of North Service and South Service Roads.

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.

It is considered that either a driven, interlocking sheet pile system or a soldier pile and timber lagging system would be suitable for the temporary excavation support, based on the subsurface soil and groundwater conditions. However, where excavations will extend to the bedrock, driven sheetpiles would have to be keyed into or pinned to the bedrock, or soldier piles would have to be socketted into the bedrock. An interlocking sheet pile system would contribute to both ground and, where applicable, groundwater control – it would provide for control of seepage of groundwater from just above the bedrock surface and any perched water within the native sandy soils above the bedrock. For a soldier pile and lagging system, it would be necessary to control 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 or socketted to 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.

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

6.11.4 Obstructions

The residual soil above the bedrock surface contains rock fragments, particularly immediately above the bedrock, as noted on the borehole records, 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 cobbles boulders / sizes within the overburden soils or immediately above the bedrock; an example NSSP is provided in Appendix D.

6.11.5 Bedrock Subgrade Inspection and Protection

As discussed in Section 6.4, if consideration is given to founding the new spread/strip footings on the bedrock some subexcavation will 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 D for inclusion in the Contract Documents.

6.11.6 Vibration Monitoring During Construction

It is anticipated that hoe-ramming or churn drilling will be required to excavate into the bedrock at the proposed abutment and pier locations.

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 about 90 m from the proposed abutment locations. Although a lower PPV threshold of 50 mm/s is generally considered applicable for vibration impacts on buildings, the zone of influence would be less than 200 m and potentially even less than 100 m. Vibration monitoring is not expected to be required at the existing structures near the bridge site; however, it would be prudent to carry out such monitoring during critical stages of the construction, such as during bedrock excavating and pile driving operations. An NSSP describing the requirements for vibration monitoring is presented in Appendix D.



7.0 CLOSURE

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

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ASTM International:

- | | |
|------------|--|
| ASTM D1586 | Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils |
| ASTM D7012 | Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures |

Commercial Software:

- Slide (Version 6) by Rocscience Inc.

Ontario Provisional Standard Drawing:

- | | |
|---------------|---|
| OPSD 202.010 | Slope Flattening |
| 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 |
| OPSD 3101.150 | Walls, Abutments, Backfill, Minimum Granular Requirements |
| OPSD 3121.150 | Walls, Retaining, Backfill, Minimum Granular Requirements |
| OPSD 3190.100 | Walls, Retaining and Abutment, Wall Drain |



Ontario Provincial Standard Specification:

OPSS.PROV 501	Construction Specifications for Compacting
OPSS 511	Construction Specification for Rip Rap, Rock Protection and Granular Sheetting
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS 802	Construction Specification for Topsoil
OPSS.PROV 804	Construction Specification for Seed and Cover
OPSS.PROV 902	Construction Specification for Excavating and Backfilling Structures
OPSS.PROV 903	Construction Specification for Deep Foundations
OPSS.PROV 1004	Material Specification for Aggregates – Miscellaneous
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

Ontario Water Resources Act:

Ontario Regulation 903 Wells (as amended)

Ontario Occupational Health and Safety Act:

Ontario Regulation 213/91 Construction Projects (as amended)

Ministry of Transportation, Ontario

Structural Manual, Provincial Highways Management Division, Highway Standards Branch, Bridge Office, August 2014.

Ministry of Transportation Ontario. Structural Office Report SO-96-01. Integral Abutment Bridges.

MTO Foundations Guideline, Embankment Settlement Criteria for Design, July 2010.



TABLES



**TABLE 1 – COMPARISON OF FOUNDATION ALTERNATIVES – DIXIE ROAD UNDERPASS REPLACEMENT
G.W.P. 2102-13-00 & 2432-13-00**

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 and abutments; however, requires deeper temporary protection for staged construction. 	<ul style="list-style-type: none"> Existing abutments and piers are supported on shallow foundations, and have performed well. Lower vibration impacts on existing structures than for driven steel H-pile installation. Only minor groundwater seepage anticipated through the shale bedrock, so pumping from filtered sumps expected to provide adequate groundwater control. Higher geotechnical resistance than for shallow foundations bearing on native soil deposits or compacted granular pad. 	<ul style="list-style-type: none"> Deeper excavation and temporary protection systems required along edges of QEW, North Service and South Service Roads. More groundwater control required associated with deeper excavation. Precludes use of integral abutments, although may permit semi-integral abutments; potentially greater maintenance required at abutments. Greater volume of excavation spoil and concrete than for spread/strip footings founded on native soils. 	<ul style="list-style-type: none"> Conventional excavation and construction techniques. 	<ul style="list-style-type: none"> Estimated cost is approximately \$600/m³ of shallow foundations volume, plus protection system costs; greater than shallow foundations on native soil due to greater volume of excavation and concrete required.
Spread/strip footings founded on native soils	<ul style="list-style-type: none"> Feasible for support of the piers and abutments; however, requires temporary protection for staged construction. 	<ul style="list-style-type: none"> Suitable founding strata at shallow depths reducing depth of excavation and temporary excavation support requirements. Existing abutments and piers are supported on shallow foundations, and have performed well. Only minor groundwater seepage anticipated, so pumping from filtered sumps expected to provide adequate groundwater control. Lesser volume of excavation spoil; and concrete than for spread/strip footings founded on bedrock. 	<ul style="list-style-type: none"> Temporary protection systems required along edges of QEW, North Service and South Service Roads. Lower bearing geotechnical resistances compared to deep foundation options or spread/strip footings founded on bedrock. Precludes use of integral abutments; potentially greater maintenance required at abutments. 	<ul style="list-style-type: none"> Conventional excavation and construction techniques. 	<ul style="list-style-type: none"> Overall lesser cost than spread/strip footings founded on shale bedrock as less excavation volume required; the cost of temporary protection system and concrete would be reduced.

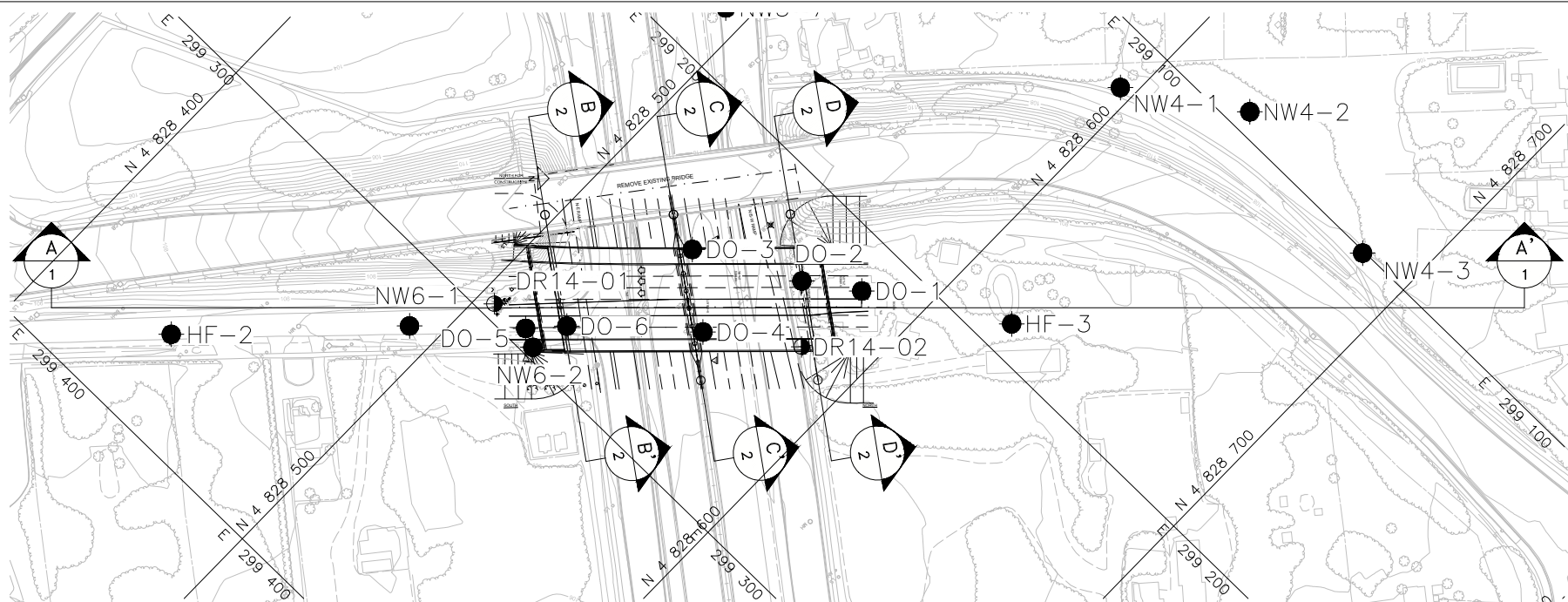


FOUNDATION REPORT – QEW-DIXIE ROAD UNDERPASS REPLACEMENT, GWP 2102-13-00 & 2432-13-00

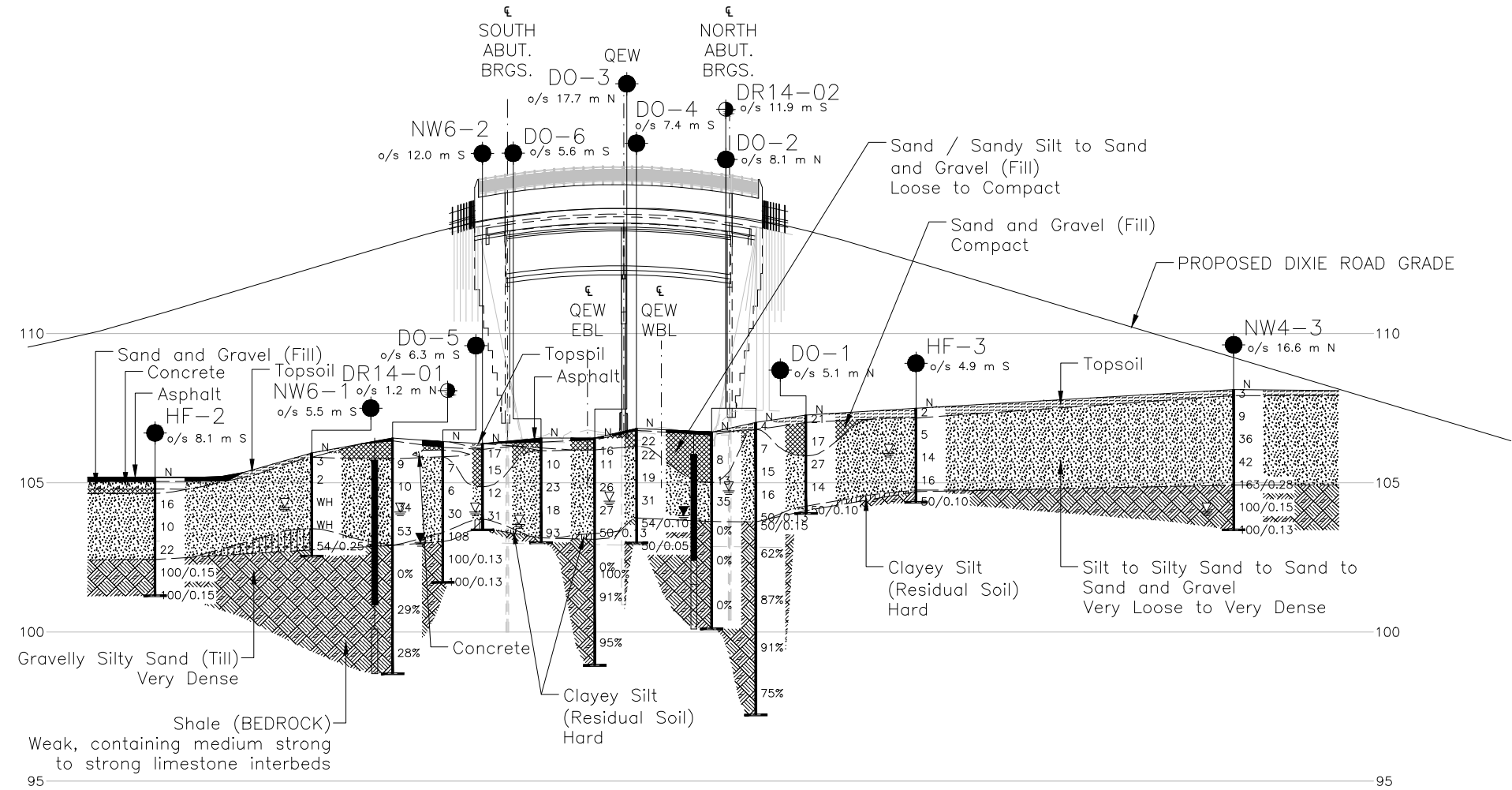
Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Spread/strip footings “perched” on compacted granular pad in approach embankments	<ul style="list-style-type: none"> Feasible for support of the abutments and associated wing walls/retaining walls. 	<ul style="list-style-type: none"> Abutment footings can be maintained higher than footings founded on bedrock deposit; reducing the requirements for subexcavation and temporary protection systems 	<ul style="list-style-type: none"> Precludes use of integral abutments; potentially greater maintenance required at abutments Potential for differential settlement between abutments and pier 	<ul style="list-style-type: none"> Conventional excavation and construction techniques. 	<ul style="list-style-type: none"> Approximately the same cost as spread/strip footings founded on shale bedrock. The cost of temporary protection system and concrete for abutment walls would be reduced, but cost for bridge increased due to longer span.
Steel H-piles founded within shale bedrock	<ul style="list-style-type: none"> Feasible for support of the abutments. Not as practical as strip footings or drilled shafts at centre pier, due to shallow depth to bedrock. 	<ul style="list-style-type: none"> Abutment pile caps could be maintained higher than footings founded on shale bedrock, reducing excavation depth and associated protection system requirements. Allows for integral abutment construction. 	<ul style="list-style-type: none"> Temporary protection systems likely required along median edge of QEW EBL and WBL to facilitate excavation to pile cap level. Pre-augering and coring into the bedrock may be required to achieve the required pile lengths. Larger/specialized equipment required for installation of piles than for construction of shallow foundations. 	<ul style="list-style-type: none"> Conventional construction methods for driven piles; augering, churn drilling and/or coring into shale bedrock may be required to achieve minimum pile lengths. 	<ul style="list-style-type: none"> Estimated cost is approximately \$250/m length for pile installation and \$600/m³ for pile cap construction; the cost may be higher to account for pre-augering/coring within bedrock and for temporary liners. Potentially less costly maintenance over life of the structure than semi-integral abutment structures.
Drilled shafts socketed into shale bedrock	<ul style="list-style-type: none"> Feasible for support of abutments and piers; may be more advantageous at centre pier than other options as . 	<ul style="list-style-type: none"> Higher bearing resistances than for steel H-piles, requiring fewer elements. At piers, may result in less excavation and a smaller footprint/working area than for spread footing option; may also reduce protection system and groundwater control requirements, particularly if the pile cap can be eliminated and the structural columns extended directly on top of the drilled shafts. 	<ul style="list-style-type: none"> Temporary liners would be required during construction to control potential ground losses in the non-cohesive soils and to mitigate for groundwater seepage. Shale bedrock is weak with medium strong to strong limestone layers, so more expensive coring/churn drilling required to form bedrock socket through these layers. Precludes use of integral abutments. The rock socket is required to be cleaned, incorporating inspection with a video camera. Concrete would have to be placed by tremie methods below the water level. 	<ul style="list-style-type: none"> Conventional construction methods for drilled shaft foundations; temporary liners required for ground and groundwater control. 	<ul style="list-style-type: none"> Estimated cost is approximately \$1000/m length for caisson installation and \$600/m³ for pile cap construction (if pile caps are adopted at the pier); this cost expected to be higher to account for pre-augering/coring through harder limestone layers and for temporary liners.



DRAWINGS



PLAN
SCALE
20 0 20 40 m



HORIZONTAL SCALE
20 0 20 40 m

DIXIE ROAD CENTRELINE PROFILE
A-A'
1

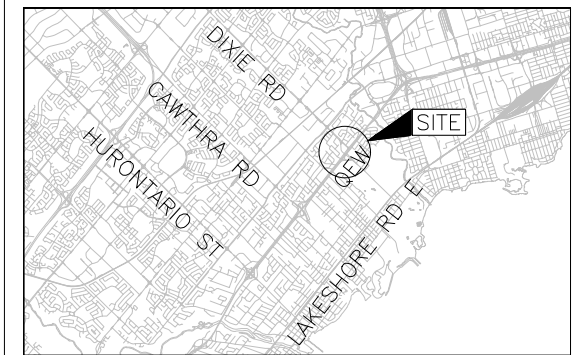
VERTICAL SCALE
2 0 2 4 m

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

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
DO-1	107.3	4828582.4	299214.9
DO-2	107.0	4828567.2	299225.4
DO-3	106.5	4828536.7	299241.7
DO-4	106.8	4828556.5	299257.5
DO-5	106.4	4828517.1	299294.2
DO-6	106.5	4828525.6	299285.0
DR14-01	106.5	4828505.1	299295.5
DR14-02	106.7	4828581.1	299239.8
HF-2	105.2	4828441.3	299370.6
HF-3	107.5	4828621.9	299190.3
NW4-3	108.1	4828683.3	299100.6
NW6-1	106.0	4828491.3	299318.3
NW6-2	106.3	4828522.7	299296.8

CONT No. _____
GWP No. 2012-13-00 & 2432-13-00

QEW DIXIE ROAD UNDERPASS
BOREHOLE LOCATIONS AND
SOIL STRATA



KEY PLAN
SCALE
2 0 2 4 km

LEGEND

- Borehole - Current Investigation
- Borehole - 2014 Investigation (Geocres No. 30M11-251)
- 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, measured on March 31, 2015
- WL upon completion of drilling
- R Refusal

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 the borehole locations. Between boreholes the boundaries are assumed from geological evidence.

Boreholes 155 to 167 from Geocres 30M11-20, dated September 23, 1966 were overlaid onto this drawing and the borehole coordinates were interpreted from the coordinate system superimposed on the plan.

The coordinates for Boreholes EC 14-01 to EC 14-04 were recorded by Thurber in UTM coordinates. Golder converted the UTM coordinates to MTM coordinates.

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 profile and cross section plan provided in digital format by AECOM, drawing file no. 01_DixieRdUnderpass_GA.dwg, received June 16, 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.
Dixie Road Profile (shown as approximate) based on PDF file of 30% Executive Review Submission, provided by AECOM, Dated January 10, 2017. Key plan base data - MNRF LIO, obtained 2015.

NO.	DATE	BY	REVISION
Geocres No. 30M11-272			
HWY. QEW	PROJECT NO. 1530382	DIST. CENTRAL	
SUBM'D. SMM	CHKD. MWK	DATE: 10/01/2018	SITE:
DRAWN: MR/DD	CHKD. SMM	APPD. LCC	DWG. 1



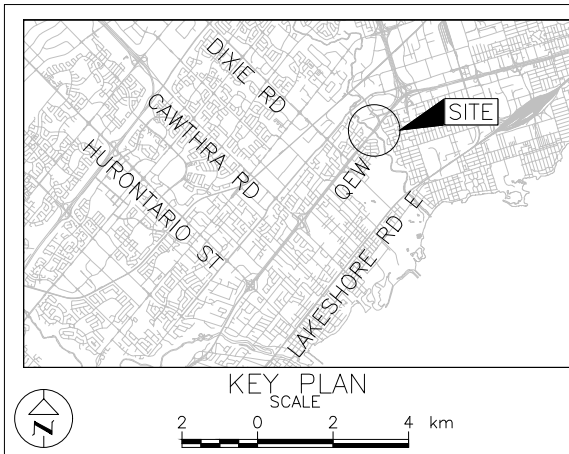
METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No. _____
GWP No. 2012-13-00 & 2432-13-00

QEW DIXIE ROAD UNDERPASS

SOIL STRATA

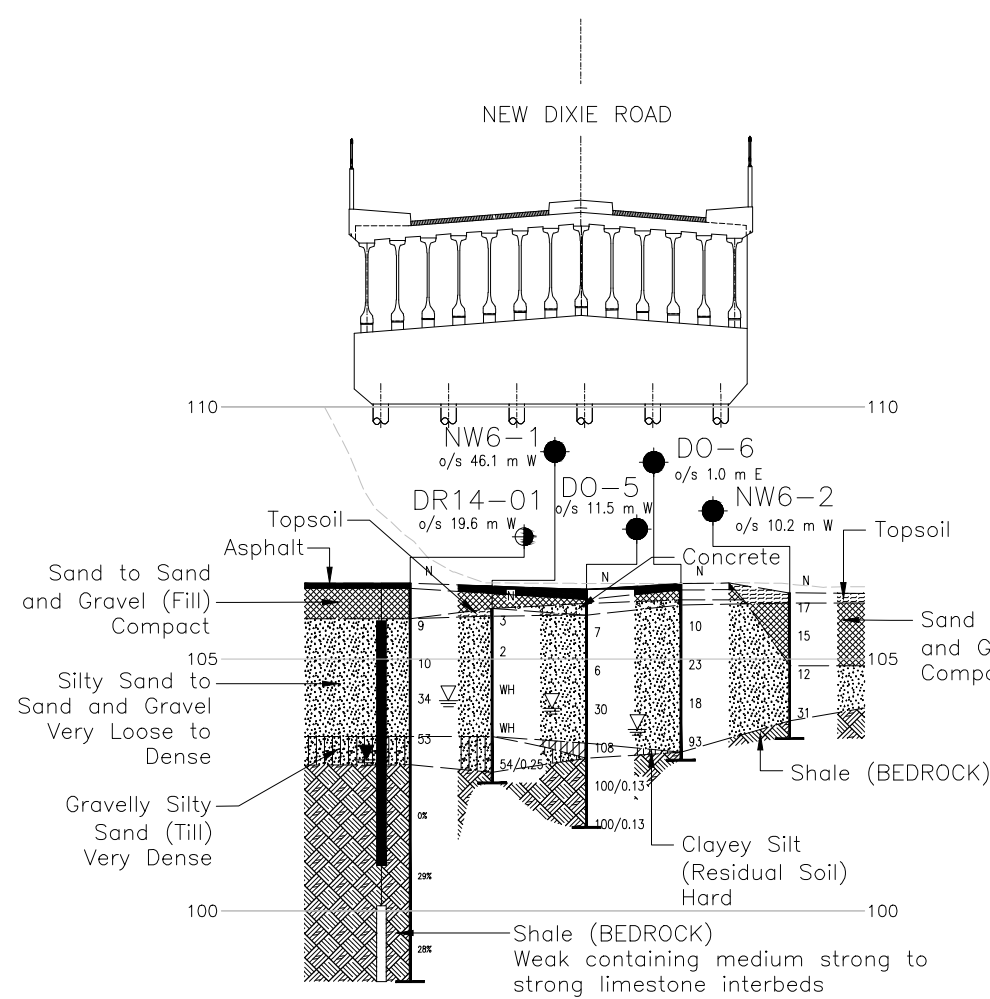
SHEET



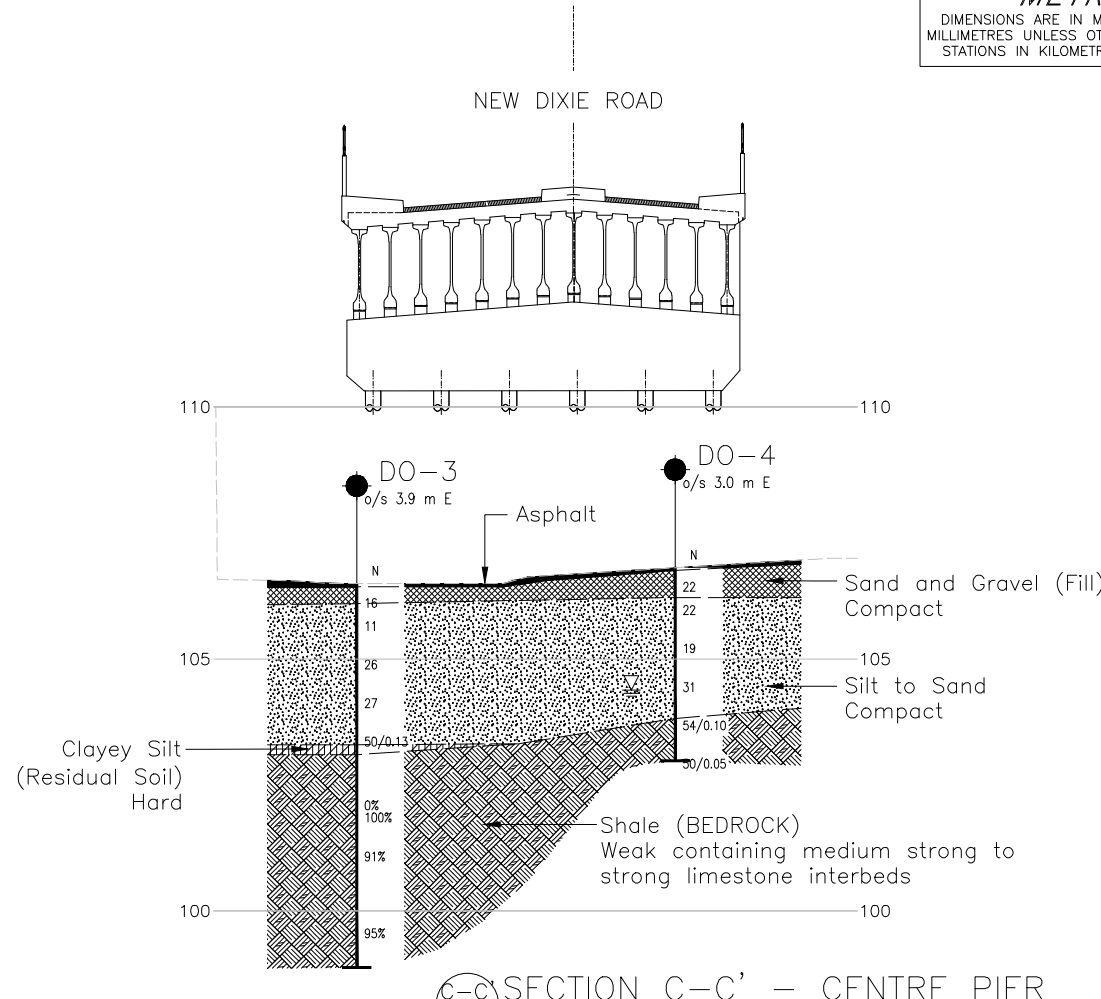
LEGEND

●	Borehole - Current Investigation
●	Borehole - 1966 Investigation (Geocres No. 30M11-20)
●	Borehole - 2014 Investigation (Geocres No. 30M11-252)
⊥	Seal
⊥	Piezometer
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, measured on March 31, 2015
≡	WL upon completion of drilling

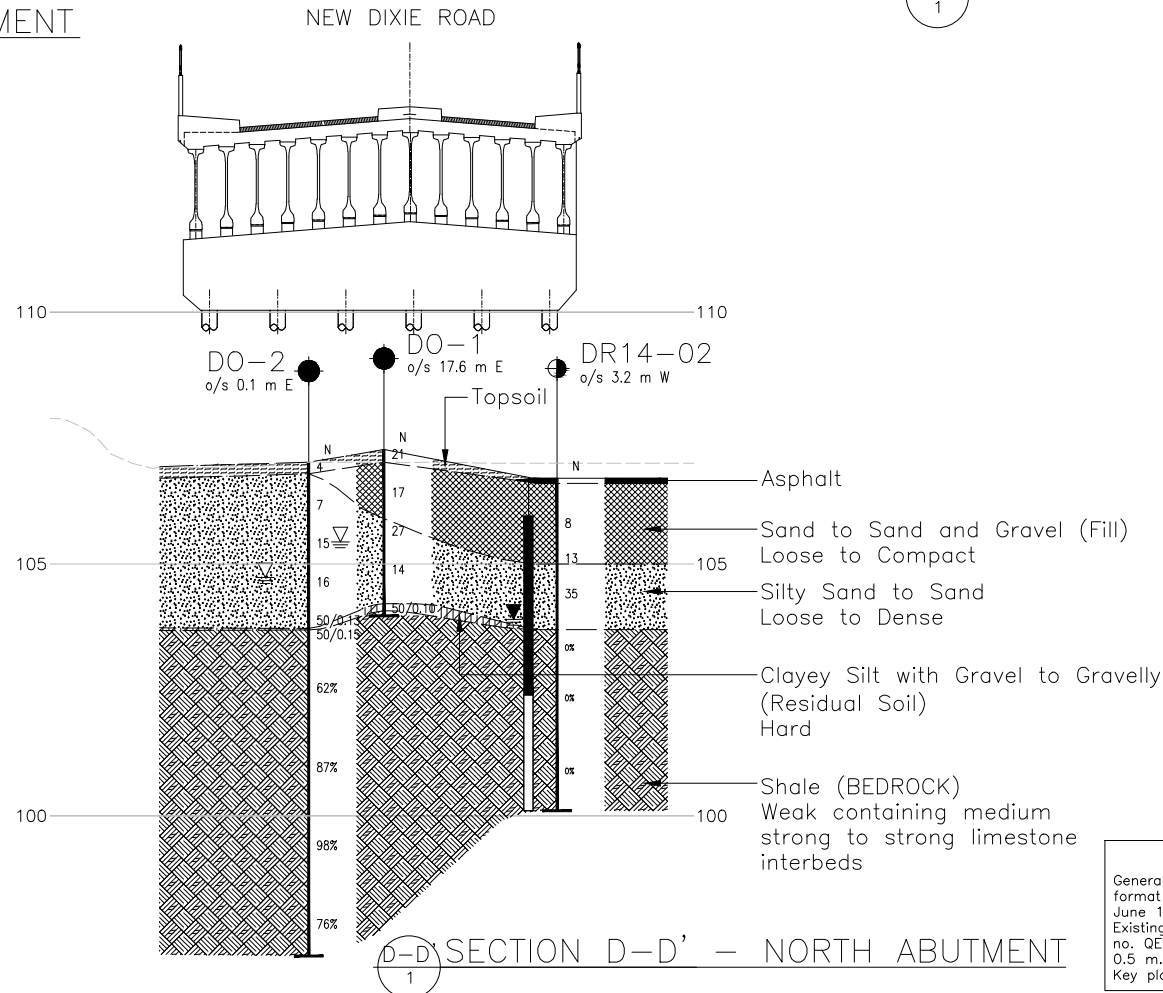
BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
DO-1	107.3	4828582.4	299214.9
DO-2	107.0	4828567.2	299225.4
DO-3	106.5	4828536.7	299241.7
DO-4	106.8	4828556.5	299257.5
DO-5	106.4	4828517.1	299294.2
DO-6	106.5	4828525.6	299285.0
DR14-01	106.5	4828505.1	299295.5
DR14-02	106.7	4828581.1	299239.8
HF-3	107.5	4828621.9	299190.3
NW6-1	106.0	4828491.3	299318.3
NW6-2	106.3	4828522.7	299296.8



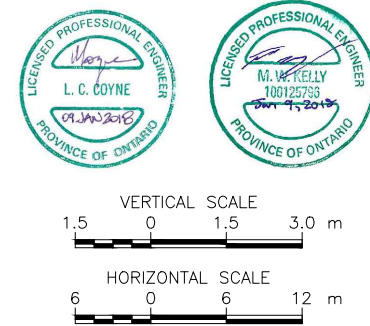
B-B' SECTION B-B' - SOUTH ABUTMENT



C-C' SECTION C-C' - CENTRE PIER



D-D' SECTION D-D' - NORTH ABUTMENT



REFERENCE

General arrangement plan profile and cross section provided in digital format by AECOM, drawing file no. 01_DixieRdUnderpass_GA.dwg, received June 16, 2017.

Existing ground contours provided in digital format by AECOM, drawing file no. QEW_DixieRd_Contours3D.dwg, received Nov. 08, 2016, contour interval 0.5 m.

Key plan base data - MNRF LIO, obtained 2015.

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 the borehole locations. Between boreholes the boundaries are assumed from geological evidence.

Boreholes 155 to 167 from Geocres 30M11-20, dated September 23, 1966 were overlaid onto this drawing and the borehole coordinates were interpreted from the coordinate system superimposed on the plan.

The coordinates for Boreholes EC 14-01 to EC 14-04 were recorded by Thurber in UTM coordinates. Golder converted the UTM coordinates to MTM coordinates.

NO.	DATE	BY	REVISION
Geocres No. 30M11-272			
HWY. QEW	PROJECT NO. 1530382	DIST. CENTRAL	
SUBM'D. SMM	CHKD. MWK	DATE: 1/10/2018	SITE:
DRAWN: MR	CHKD. SMM	APPD. LCC	DWG. 2

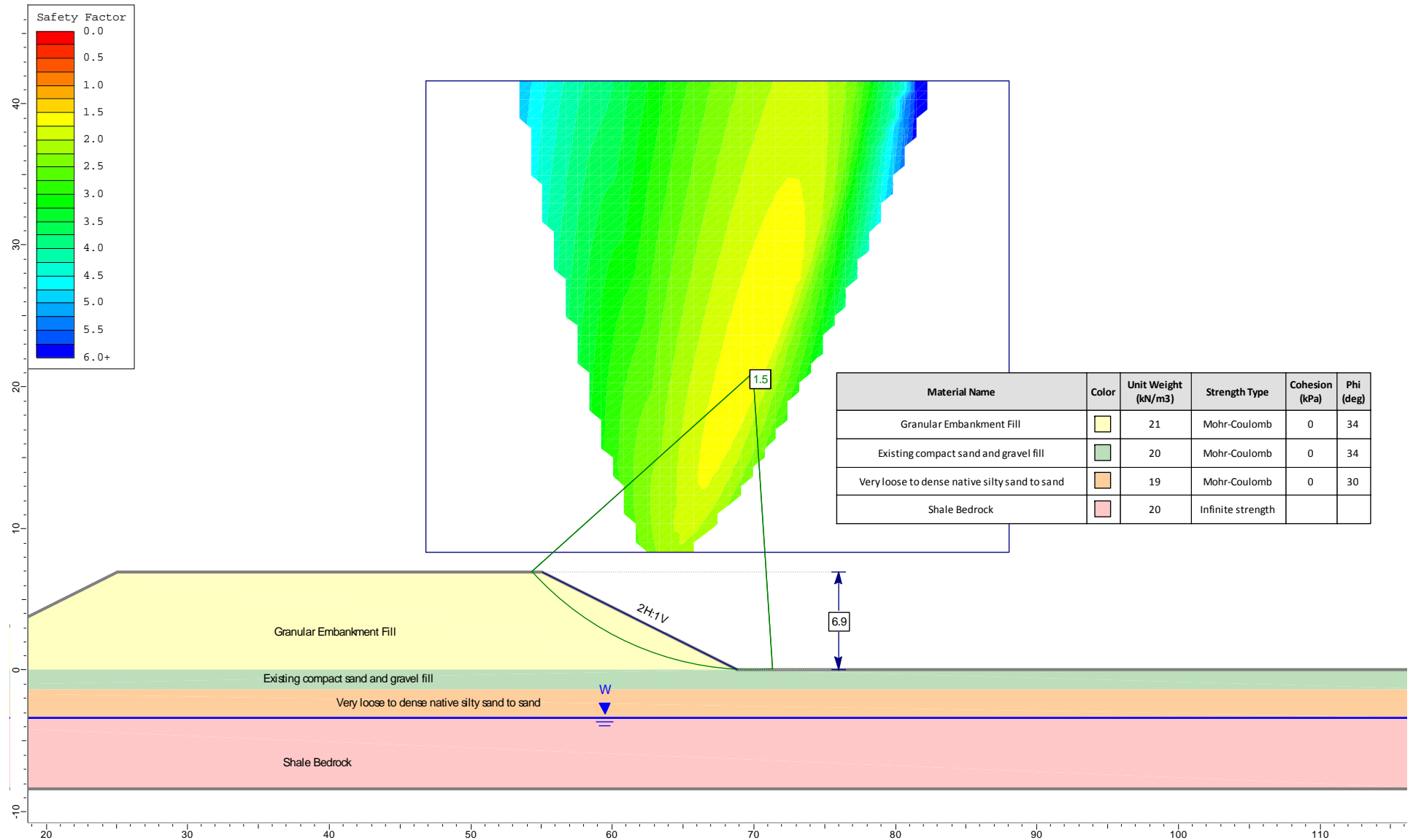


FIGURES



QEW – Dixie Road Underpass Replacement Approach Embankment Static Global Stability Analysis

Figure 1





APPENDIX A

Borehole Records and Laboratory Test Results – Previous Investigation (GEOCRES 30M11-251)

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


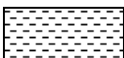



ROCK WEATHERING CLASSIFICATION

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 DR 14-01

1 OF 1

METRIC

W.P. 09-20003 LOCATION Dixie Rd. Underpass N 4 828 505.1 E 299 295.5 ORIGINATED BY SLL
 HWY QEW BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2014.08.08 - 2014.08.08 CHECKED BY MW

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					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						
								20 40 60 80 100						
106.5	GROUND SURFACE													
0.0	ASPHALT: (113mm)													
0.1	SAND, trace gravel, trace silt, occasional cobbles		1	GS										
105.8	Brown Moist (FILL)													
0.7	Silty SAND, trace clay, trace gravel, trace rootlets Loose to Dense Brown Moist		2	SS	9									
			3	SS	10									5 67 25 3
			4	SS	34									
			5	SS	53									
102.9														
3.6	SHALE, highly to moderately weathered, fine grained, thinly bedded, grey, weak to medium strong, with strong limestone interbeds: (Georgian Bay Formation)													
	Highly broken zone (125mm) at 4.2m		1	RUN										RUN #1 TCR=94% SCR=83% RQD=0%
	Highly broken zone (75mm) at 5.5m		2	RUN										RUN #2 TCR=100% SCR=95% RQD=29% UCS=22.4MPa UCS=17.4 MPa
	Thin clay seam (less than 25mm) from 6.8m to 7.0m		3	RUN										RUN #3 TCR=100% SCR=96% RQD=28% UCS=30.8MPa UCS=11.9 MPa
98.6														
7.9	END OF BOREHOLE AT 7.9m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Sep 29, 14 3.8 102.7 Oct 27, 14 3.5 103.0													

ONTMT4S 1219.GPJ 2012TEMPLATE(MTO).GDT 12/9/14

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

RECORD OF BOREHOLE No DR 14-02

1 OF 1

METRIC

W.P. 09-20003 LOCATION Dixie Rd. Underpass N 4 828 581.1 E 299 239.8 ORIGINATED BY SLL
 HWY QEW BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2014.08.08 - 2014.08.08 CHECKED BY MW

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 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)
106.7	GROUND SURFACE							20	40	60	80	100					
0.0 0.1	ASPHALT: (125mm) SAND, trace gravel, trace silt and clay Loose Brown Moist (FILL)		1	GS													
			2	SS	8												3 84 10 3
105.0																	
1.7	SAND, trace gravel, trace silt, trace rootlets Compact to Dense Brown Moist		3	SS	13												
			4	SS	35												7 80 13 (SI+CL)
103.7																	
3.0	SHALE, highly to moderately weathered, fine grained, thinly bedded, grey, weak to medium strong, with very strong limestone interbeds: (Georgian Bay Formation) Clay seam (25mm) at 3.6m Highly broken zone at: 50mm at 4.1m 100mm at 4.3m 100mm at 4.5m Vertical joint (125mm) at 4.9m Highly broken zone at: 50mm at 5.1m 50mm at 5.5m 75mm at 6.1m Limestone interbeds		1	RUN													RUN #1 TCR=100% SCR=63% RQD=0%
			2	RUN													RUN #2 TCR=100% SCR=69% RQD=0%
			3	RUN													RUN #3 TCR=100% SCR=84% RQD=0% UCS=25.3MPa UCS=133.6 MPa
100.1																	
6.6	END OF BOREHOLE AT 6.6m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Sep 29, 14 3.4 103.3 Oct 27, 14 2.8 103.9																

+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

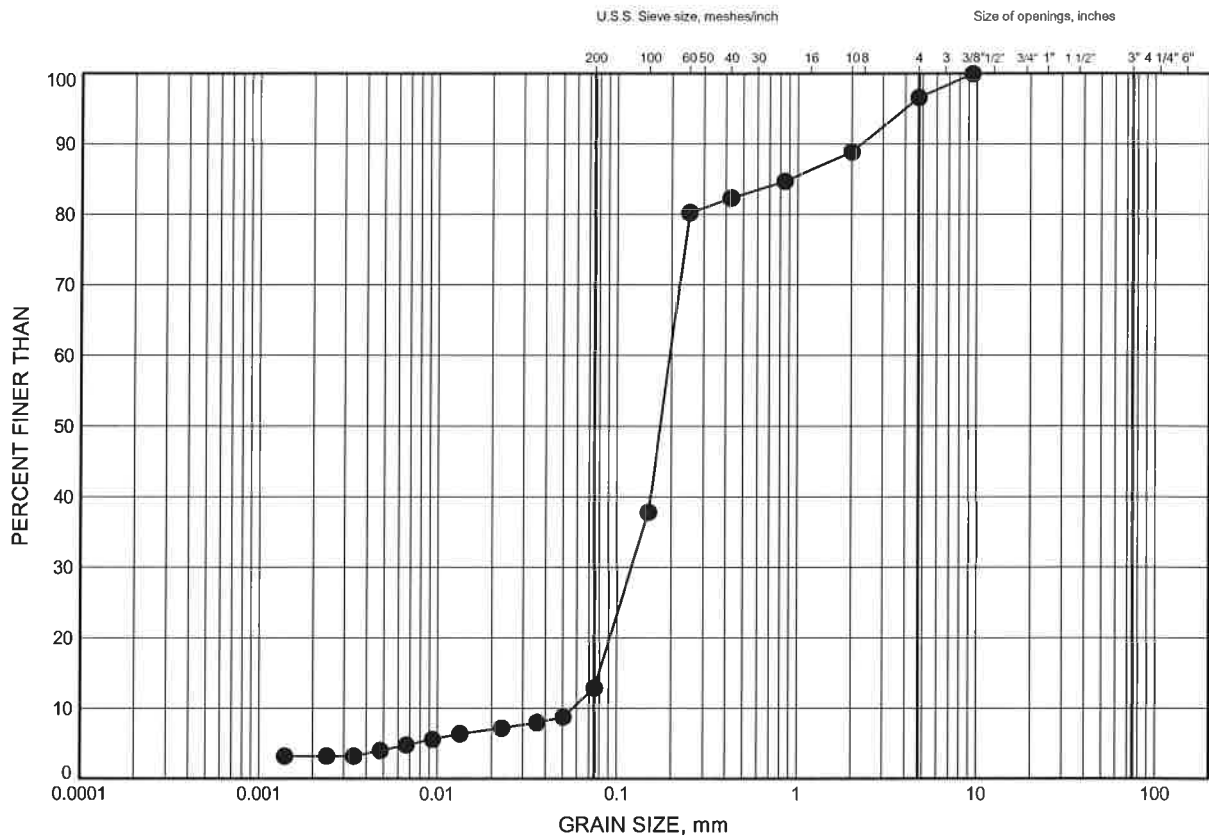
Appendix B
Laboratory Test Results

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)
●	DR 14-02	1.07	105.63

Date December 2014
W.P. 09-20003

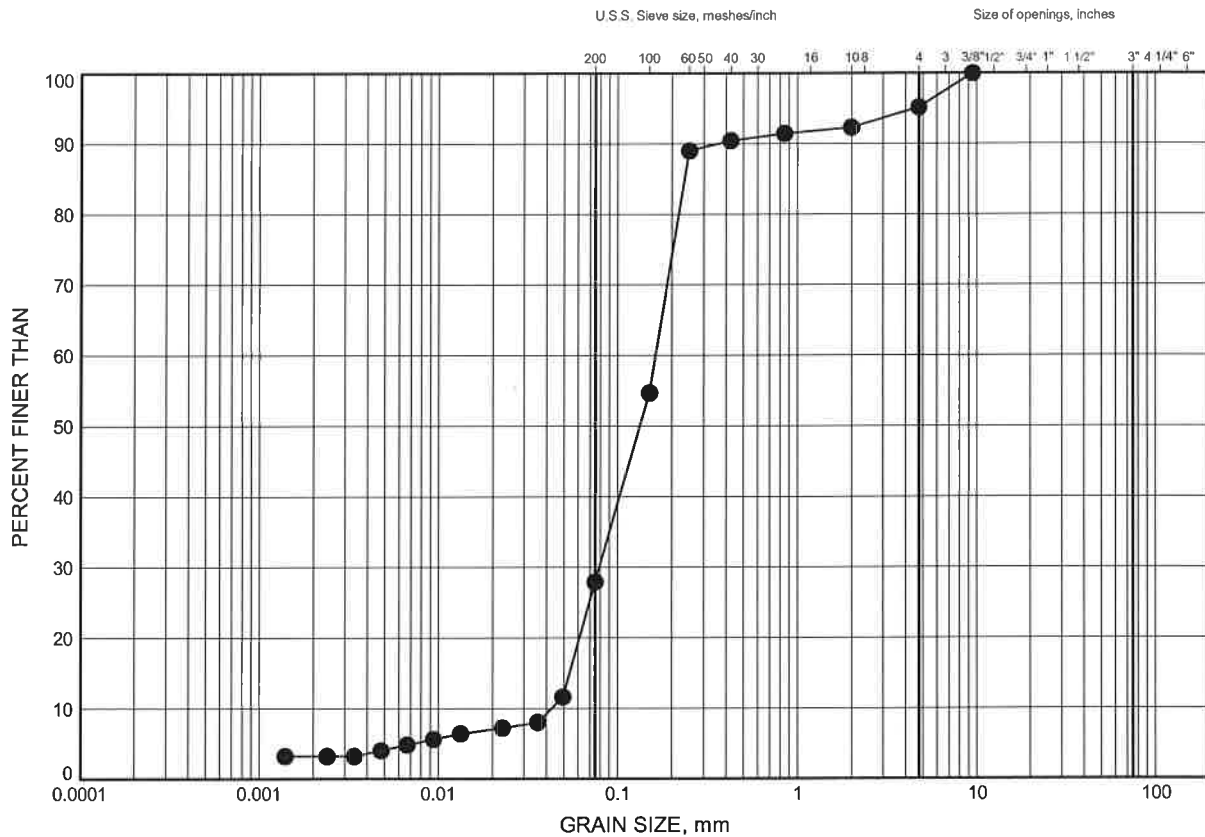


Prep'd AN
Chkd. AP

QEW Cawthra Road
GRAIN SIZE DISTRIBUTION

FIGURE B2

SILTY SAND



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	DR 14-01	1.83	104.67

Date December 2014
W.P. 09-20003

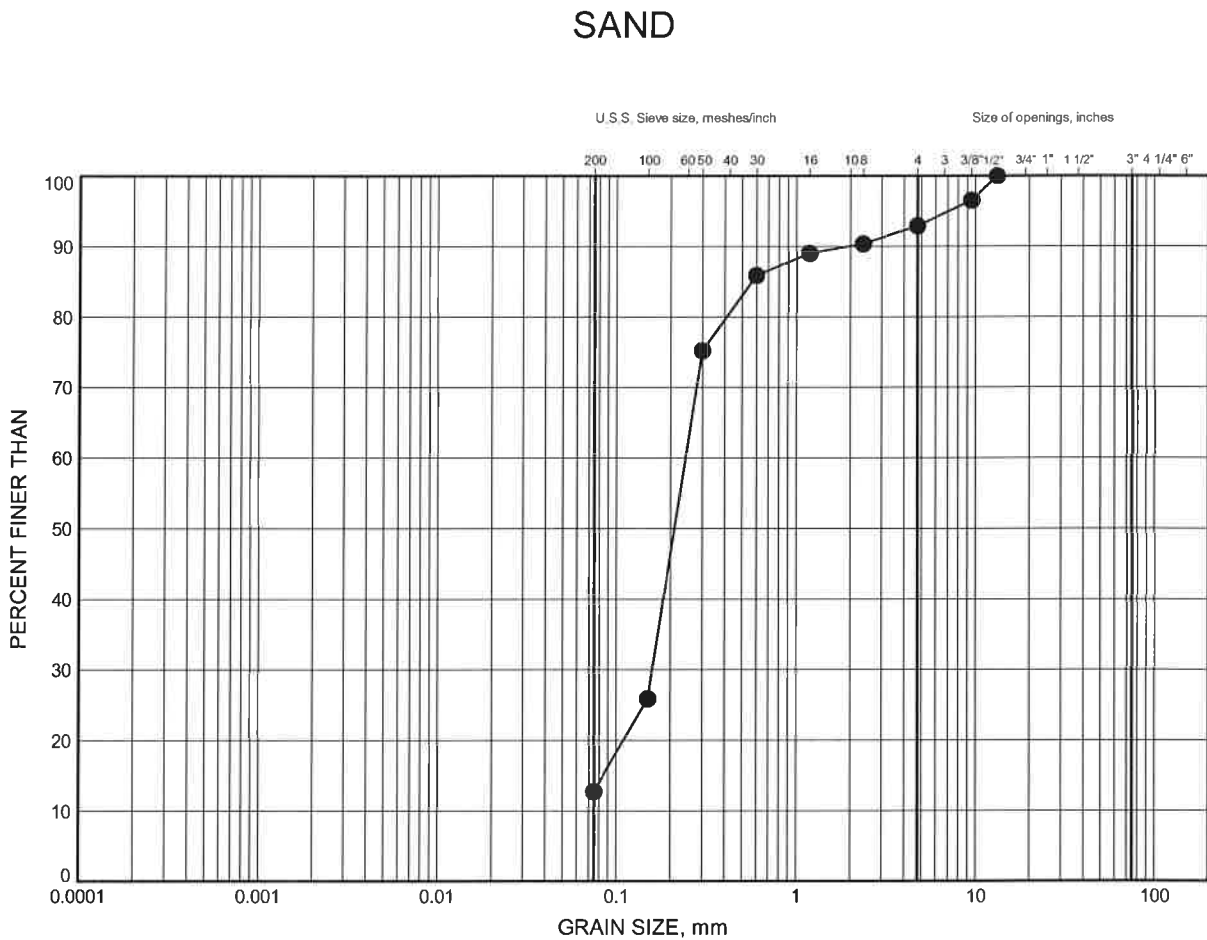


Prep'd AN
Chkd. AP

QEW Cawthra Road

GRAIN SIZE DISTRIBUTION

FIGURE B3



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	DR 14-02	2.51	104.19



Appendix C

Point load Test Results and Rock Core Photographs



THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

Job No :	19-1351-219	Client :	MMM Group
Project Name :	QEW CAWTHRA ROAD	Date Drilled :	08-Aug-14
Core Size :	NQ BH No : DR14-01	Date Tested :	08-Aug-14
		Tester :	GAZ

Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (MPa)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	2	5.8	A	3.0	47.1	54.3	22.4	Shale	Weak
2	2	6.0	A	2.2	47.0	50.2	17.4	Shale	Weak
3	3	7.3	A	4.0	47.0	51.2	30.8	Shale	Medium Strong
4	3	7.9	D	1.2	46.9	long	11.9	Shale	Weak
5									
6									
7									
8									
9									
10									
11									
12									
13									
14									
15									
16									
17									
18									
19									
20									
21									
22									
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24									
25									
26									
27									
28									
29									
30									
31									
32									
33									
34									
35									

- * It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1
- Long pieces of core can be tested diametrically to produce suitable lengths for axial testing
- * Diametral Test should have $0.7 \times D$ on either side of test point.



THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

Job No : 19-1351-219 Client : MMM Group
Date Drilled : 08-Aug-14
Project Name : QEW CAWTHRA ROAD Date Tested : 08-Aug-14
Core Size : NQ BH No : DR14-02 Tester : GAZ

Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (MPa)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	3	6.1	D	2.5	47.0	84.9	25.3	Shale	Medium Strong
2	3	6.4	A	21.2	47.0	66.9	133.6	Limestone	Very Strong
3									
4									
5									
6									
7									
8									
9									
10									
11									
12									
13									
14									
15									
16									
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30									
31									
32									
33									
34									
35									

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

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

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

Last Modified: August 15, 2013



Photograph 1. Rock cores recovered from Borehole DR14-01



Photograph 2. Rock cores recovered from Borehole DR14-02



APPENDIX B

Borehole and Drillhole Records – Current Investigation



LIST OF SYMBOLS

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

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

(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 = $(w_l - w_p)$
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_{α}	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 = σ'_p / σ'_{vo}

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = $\tan \delta$
c'	effective cohesion
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	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

Notes: 1
2

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



LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
SS	Split-spoon
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

II. 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.) drive open sampler for a distance of 300 mm (12 in.).

Dynamic Cone Penetration Resistance; 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

Piezo-Cone Penetration Test (CPT)

A 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 (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Non-Cohesive Soils

Density Index	N
Relative Density	Blows/300 mm or Blows/ft
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils Consistency

	C_u, S_u	psf
	kPa	
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

IV. SOIL TESTS

w	water content
w_p	plastic limit
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
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	field vane (LV-laboratory vane test)
γ	unit weight

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

V. MINOR SOIL CONSTITUENTS

Per cent by Weight	Modifier	Example
0 to 5	Trace	Trace sand
5 to 12	Trace to Some (or Little)	Trace to some sand
12 to 20	Some	Some sand
20 to 30	(ey) or (y)	Sandy
over 30	And (non-cohesive) or With (cohesive)	Sand and Gravel Silty Clay with sand / Clayey Silt with sand



LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERINGS STATE

Fresh: no visible sign of weathering

Faintly weathered: weathering limited to the surface of major discontinuities.

Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

Moderately weathered: weathering extends throughout the rock mass but the rock material is not friable.

Highly weathered: weathering extends throughout rock mass and the rock material is partly friable.

Completely weathered: rock is wholly decomposed and in a friable condition but the rock and structure are preserved.

BEDDING THICKNESS

<u>Description</u>	<u>Bedding Plane Spacing</u>
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

<u>Description</u>	<u>Spacing</u>
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

<u>Term</u>	<u>Size*</u>
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, measured relative to the length of the total core run. RQD varied from 0% for completely broken core to 100% for core in solid sticks.

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

JN Joint	PL Planar
FLT Fault	CU Curved
SH Shear	UN Undulating
VN Vein	IR Irregular
FR Fracture	K Slickensided
SY Stylolite	PO Polished
BD Bedding	SM Smooth
FO Foliation	SR Slightly Rough
CO Contact	RO Rough
AXJ Axial Joint	VR Very Rough
KV Karstic Void	
MB Mechanical Break	

PROJECT <u>1530382</u>		RECORD OF BOREHOLE No NW6-1		SHEET 1 OF 1		METRIC	
G.W.P. <u>2102-13-00; 2432-13-00</u>		LOCATION <u>N 4828491.3; E 299318.3 MTM NAD 83 ZONE 10 (LAT. 43.596336; LONG. -79.567893)</u>		ORIGINATED BY <u>ML</u>			
DIST <u>Central</u> HWY <u>QEW</u>		BOREHOLE TYPE <u>150 mm O.D. Solid Stem Augers</u>		COMPILED BY _____			
DATUM <u>Geodetic</u>		DATE <u>June 28, 2017</u>		CHECKED BY _____			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)							
													20	40	60		80	100	W _p	W
106.0	GROUND SURFACE																			
0.0	TOPSOIL																			
0.2	SAND, trace to some silt, some gravel Very loose Brown Moist to wet		1	SS	3		105													
			2	SS	2															
			3	SS	WH															
			4A	SS	WH															
			4B	SS	WH															
103.5	Gravelly Silty SAND, containing clayey silt pockets (TILL) Very dense Grey Wet		5A	SS	54/0.25	103														
2.5			5B																	
102.8	SHALE (BEDROCK)																			
102.5	END OF BOREHOLE																			
3.5	NOTE: 1. Water level in open borehole at a depth of 1.8 m below ground surface (Elev. 104.2 m) upon completion of overburden drilling.																			

PROJECT <u>1530382</u>		RECORD OF BOREHOLE No NW6-2		SHEET 1 OF 1		METRIC	
G.W.P. <u>2102-13-00; 2432-13-00</u>		LOCATION <u>N 4828522.7; E 299296.8 MTM NAD 83 ZONE 10 (LAT. 43.596619; LONG. -79.568160)</u>		ORIGINATED BY <u>EN</u>			
DIST <u>Central</u> HWY <u>QEW</u>		BOREHOLE TYPE <u>150 mm O.D. Solid Stem Augers</u>		COMPILED BY _____			
DATUM <u>Geodetic</u>		DATE <u>June 28, 2017</u>		CHECKED BY _____			

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					WATER CONTENT (%)				GR	SA	SI	CL	
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× REMOULDED										
106.3	GROUND SURFACE																				
0.0	TOPSOIL																				
0.2	Sand, some silt, trace gravel to gravelly, trace organics (FILL) Compact Black to brown Moist		1A																		
			1B	SS	17																
			2	SS	15																
104.9																					
1.5	SAND, some silt, trace clay Compact Brown Wet		3	SS	12																
103.8			4A																		
2.5	SHALE (BEDROCK)		4B	SS	31																
103.4																					
2.9	END OF BOREHOLE																				
	NOTE: 1. Water level in open borehole at a depth of 2.3 m below ground surface (Elev. 104.0 m) upon completion of overburden drilling.																				

PROJECT		1530382		RECORD OF BOREHOLE No NW4-3		SHEET 1 OF 1		METRIC										
G.W.P.		2102-13-00; 2432-13-00		LOCATION		N 4828683.3; E 299100.6 MTM NAD 83 ZONE 10 (LAT. 43.598063; LONG. -79.570592)		ORIGINATED BY										
DIST		Central HWY QEW		BOREHOLE TYPE		108 mm O.D. Continuous Flight Solid Stem Augers		COMPILED BY										
DATUM		Geodetic		DATE		October 6, 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										WATER CONTENT (%)
108.1	GROUND SURFACE							20	40	60	80	100						
0.0	TOPSOIL							20	40	60	80	100						
0.2	SAND, some silt Very loose to dense Brown to grey Moist		1	SS	3													
			2	SS	9													
			3	SS	36													
	- Becoming wet below 2.1 m depth		4	SS	42													
104.9	SHALE (BEDROCK)		5	SS	163/0.28													
3.2			6	SS	100/0.15													
103.4	END OF BOREHOLE		7	SS	100/0.13													
4.7	NOTE: 1. Water level in open borehole at a depth of 4.1 m below ground surface (Elev. 104.0 m) upon completion of drilling.																	

PROJECT		1530382		RECORD OF BOREHOLE		No HF-2		SHEET 1 OF 1		METRIC							
G.W.P.		2102-13-00; 2432-13-00		LOCATION		N 4828441.3; E 299370.6 MTM NAD 83 ZONE 10 (LAT. 43.595886; LONG. -79.567245)		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 13, 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									
105.2	GROUND SURFACE																
0.0	ASPHALT (130 mm)																
104.8	CONCRETE (255 mm)																
0.5	Sand and gravel (FILL) Brown Moist SAND, trace silt, trace gravel Compact Brown Moist		1	SS	16												
			2	SS	10												
103.1	Gravelly SAND, trace to some silt, trace clay Compact Brown Wet		3A	SS	22												
102.5	SHALE (BEDROCK)		3B														
			4	SS	100/0.15												
101.2	END OF BOREHOLE		5	SS	100/0.15												
4.0	NOTE: 1. Open borehole dry upon completion of drilling.																

PROJECT <u>1530382</u>		RECORD OF BOREHOLE No HF-3		SHEET 1 OF 1		METRIC	
G.W.P. <u>2102-13-00; 2432-13-00</u>		LOCATION <u>N 4828621.9; E 299190.3 MTM NAD 83 ZONE 10 (LAT. 43.597511; LONG. -79.569480)</u>		ORIGINATED BY <u>KG</u>			
DIST <u>Central</u> HWY <u>QEW</u>		BOREHOLE TYPE <u>CME 75, 150 mm O.D. Solid Stem Augers</u>		COMPILED BY _____			
DATUM <u>Geodetic</u>		DATE <u>June 15, 2017</u>		CHECKED BY <u>MWK</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
107.5	GROUND SURFACE																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																				

NOTE:

1. Water level in open borehole at
a depth of 1.5 m below ground
surface (Elev. 106.0 m) upon
completion of overburden drilling.

PROJECT		1530382		RECORD OF BOREHOLE No DO-1		SHEET 1 OF 1		METRIC						
G.W.P.		2102-13-00; 2432-13-00		LOCATION		N 4828582.4; E 299214.9 MTM NAD 83 ZONE 10 (LAT. 43.597155; LONG. -79.569175)		ORIGINATED BY						
DIST		Central HWY QEW		BOREHOLE TYPE		CME 55, 150 mm O.D. Solid Stem Augers		COMPILED BY						
DATUM		Geodetic		DATE		June 15, 2017		CHECKED BY						
								MWK						
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						
107.3 0.0	GROUND SURFACE TOPSOIL							20 40 60 80 100	20 40 60 80 100	10 20 30				
107.0 0.3	Sand and gravel (FILL) Compact Grey Moist		1	SS	21		107							
			2	SS	17									
105.9 1.4	SAND, trace to some silt Compact Brown Moist to wet - Becoming wet below 1.8 m		3	SS	27		106							
			4	SS	14		105							
104.2 3.3	CLAYEY SILT with GRAVEL, some shale fragments (RESIDUAL SOIL) Hard Grey Moist SHALE (BEDROCK) END OF BOREHOLE		5	SS	50/0.10		104							
NOTE: 1. Water level in open borehole at a depth of 1.8 m below ground surface (Elev. 105.5 m) upon completion of overburden drilling.														

PROJECT 1530382		RECORD OF BOREHOLE No DO-2		SHEET 1 OF 2		METRIC	
G.W.P. 2102-13-00; 2432-13-00		LOCATION N 4828567.2; E 299225.4 MTM NAD 83 ZONE 10 (LAT. 43.597019; LONG. -79.569044)		ORIGINATED BY KG			
DIST Central HWY QEW		BOREHOLE TYPE CME 75, 150 mm O.D. Solid Stem Augers		COMPILED BY			
DATUM Geodetic		DATE June 15, 2017		CHECKED BY MWK			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
								20	40	60	80	100	W _p	W	W _L		
107.0	GROUND SURFACE																
0.0	TOPSOIL																
0.2	Silty SAND to SAND, some silt, trace rootlets Loose to compact Brown Moist		1	SS	4												
			2	SS	7												
			3	SS	15												
	- Becoming wet below 2.3 m		4	SS	16												
103.7			5	SS	50/0.13												
3.3	CLAYEY SILT with GRAVEL (RESIDUAL SOIL) Hard Grey Moist SHALE (BEDROCK) Bedrock cored from depths of 3.6 m to 9.8 m. For bedrock coring details refer to Record of Drillhole DO-2.		6	SS	50/0.15												
			1	RC	REC 92%											RQD = 62%	
			2	RC	REC 100%											RQD = 87%	
			3	RC	REC 100%											RQD = 91%	
			4	RC	REC 100%											RQD = 75%	
97.2																	
9.8	END OF BOREHOLE																

Continued Next Page

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

GTA-MTO 001 S:\CLIENTS\MTQ\QEW-DIXIE\GPJ GAL-GTA.GDT 01/10/18 GPJ



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

SHEET 1 OF 1

DATUM: Geodetic

DRILLING CONTRACTOR: Davis Drilling Ltd.

[illegible]

FEATURES LEGEND



BROKEN CORE



CLAY SEAM



LIMESTONE



LOST CORE

DEPTH SCALE

1 : 50


**Golder
Associates**

LOGGED:

CHECKED:

STA-RCK 054 S:\CLIENTS\MT0\QEW-DIXIE\02 DATA\GINT\QEW-DIXIE.GPJ GAL-MISS.GDT 01/10/18 GPK

PROJECT		1530382		RECORD OF BOREHOLE No DO-3		SHEET 1 OF 1		METRIC																
G.W.P.		2102-13-00; 2432-13-00		LOCATION		N 4828536.7; E 299241.7 MTM NAD 83 ZONE 10 (LAT. 43.596744; LONG. -79.568842)		ORIGINATED BY MK																
DIST		Central HWY QEW		BOREHOLE TYPE		CME 75, 108 mm O.D. Solid Stem Augers		COMPILED BY PKS																
DATUM		Geodetic		DATE		September 6 and 7, 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																		
106.5		GROUND SURFACE																						
0.0		ASPHALT (65 mm)																						
0.1		Sand and gravel (FILL)																						
106.1		Compact Brown Moist		1	SS	16																		
0.4																								
105.7		SILT, some sand, trace gravel																						
0.8		Compact Brown Moist		2	SS	11																		
		SAND, trace to some silt, trace clay																						
		Compact Brown/grey Moist to wet		3	SS	26																		
				4	SS	27																		
103.3		CLAYEY SILT, trace sand, trace gravel, some shale fragments (RESIDUAL SOIL)		5	SS	50/0.13																		
103.1		Hard Grey Moist																						
3.4		SHALE (BEDROCK)																						
		Bedrock cored from depths of 4.1 m to 7.6 m.		1	RC	REC 100%																		
		Bedrock coring not stated to 4.1 m depth due to difficulty seating casing.		2	RC	REC 100%																		
		For bedrock coring details refer to Record of Drillhole DO-3.		3	RC	REC 92%																		
				4	RC	REC 100%																		
98.9		END OF BOREHOLE																						
7.6		NOTE: 1. Borehole dry upon completion of drilling and prior to rock coring.																						

PROJECT: 1530382

RECORD OF DRILLHOLE: DO-3

SHEET 1 OF 1

LOCATION: N 4828536.7 ; E 299241.7

DRILLING DATE: September 6 and 7, 2016

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 75 (Truck Mounted)

DRILLING CONTRACTOR: Davis Drilling Ltd.

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	R0/R1 ZONES	NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																														
						RECOVERY		R.Q.D. %	FRACT. INDEX PER Meter	DISCONTINUITY DATA					ROCK STRENGTH INDEX			WEATH- ERING INDEX																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																				
						TOTAL CORE %	SOLID CORE %			TYPE AND SURFACE DESCRIPTION	Jr	Ja	R4	R3	R2	R1	W1	W2	W3	W4	W5				W6																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													
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		Continued from Record of Borehole DO-3		102.41																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																		

FEATURES LEGEND



BROKEN CORE



CLAY SEAM



LIMESTONE



LOST CORE

DEPTH SCALE

1 : 50



LOGGED: MK

CHECKED: KG/AB

GTA-RCK 054 S:\CLIENTS\MTQ\QEW-DIXIE\02_DATA\GINTQEW-DIXIE.GPJ GAL-MISS.GDT 01/10/18 GPK

PROJECT		2150382		RECORD OF BOREHOLE		No DO-4		SHEET 1 OF 1		METRIC							
G.W.P.		2102-13-00; 2432-13-00		LOCATION		N 4828556.5; E 299257.5 MTM NAD 83 ZONE 10 (LAT. 43.596923; LONG. -79.568648)		ORIGINATED BY		MK							
DIST		Central HWY QEW		BOREHOLE TYPE		CME 75, 108 mm O.D. Solid Stem Augers		COMPILED BY		FRC							
DATUM		Geodetic		DATE		September 7, 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									
106.8	GROUND SURFACE																
0.0	ASPHALT (65 mm)																
106.4	Sand and gravel (FILL) Brown Moist		1	SS	22												
106.2	Sandy silt, trace gravel (FILL) Compact Grey Moist		2	SS	22												
0.6	SAND, some silt, trace gravel, trace clay Compact to dense Brown Moist to wet		3	SS	19												
			4	SS	31												
103.8	SHALE (BEDROCK)		5	SS	54/0.10												
3.0																	
103.0	END OF BOREHOLE		6	SS	50/0.05												
3.8	NOTE: 1. Water level in open borehole at a depth of 2.4 m below ground surface (Elev. 104.4 m) upon completion of drilling.																

PROJECT 1530382		RECORD OF BOREHOLE No DO-5		SHEET 1 OF 1		METRIC	
G.W.P. 2102-13-00; 2432-13-00		LOCATION N 4828517.1; E 299294.2 MTM NAD 83 ZONE 10 (LAT. 43.596569; LONG. -79.568192)		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 13, 2016		CHECKED BY SMM			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			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
								20	40	60	80	100								
106.4	GROUND SURFACE																			
0.0	ASPHALT (200 mm)																			
106.2																				
105.9	Sand and gravel (FILL) Brown Moist																			
0.5	CONCRETE (230 mm)																			
	SAND, some silt, trace clay Loose to compact Brown Moist to wet below 2.3 m depth		1	SS	7															
			2	SS	6															
			3	SS	30															
103.4																				
3.1	CLAYEY SILT, some sand, trace gravel, trace shale fragments (RESIDUAL SOIL)		4	SS	108															
103.1	Hard Grey Wet																			
3.4	SHALE (BEDROCK)		5	SS	100/0.13															
101.7			6	SS	100/0.13															
4.7	END OF BOREHOLE																			
	NOTE: 1. Open borehole dry upon completion of drilling.																			

PROJECT		1530382		RECORD OF BOREHOLE		No DO-6		SHEET 1 OF 1		METRIC							
G.W.P.		2102-13-00; 2432-13-00		LOCATION		N 4828525.6; E 299285.0 MTM NAD 83 ZONE 10 (LAT. 43.596645; LONG. -79.568306)		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 13, 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									
106.5	GROUND SURFACE																
0.0	ASPHALT (150 mm)																
106.2	Sand and gravel (FILL) Brown Moist																
0.4	CONCRETE (100 mm)																
	SAND, some silt Compact Brown to grey Moist		1	SS	10												
	- Grey below a depth of 1.8 m		2	SS	23												
	- Becoming wet below a depth of 2.1 m																
103.9																	
2.6	GRAVEL, some sand, trace silt Compact Brown Wet		3	SS	18												
103.5																	
3.1	Silty SAND Very dense Brown Wet																
103.2			4	SS	93												
3.5	SHALE (BEDROCK) END OF BOREHOLE																
NOTE:																	
1. Water level in open borehole at a depth of 2.9 m below ground surface (Elev. 103.6 m) upon completion of drilling.																	

GTA-MTO 001 S:\CLIENTS\MTQ\QEW-DIXIE02_DATAGINT\QEW-DIXIE.GPJ GAL-GTA.GDT 01/10/18 GPK



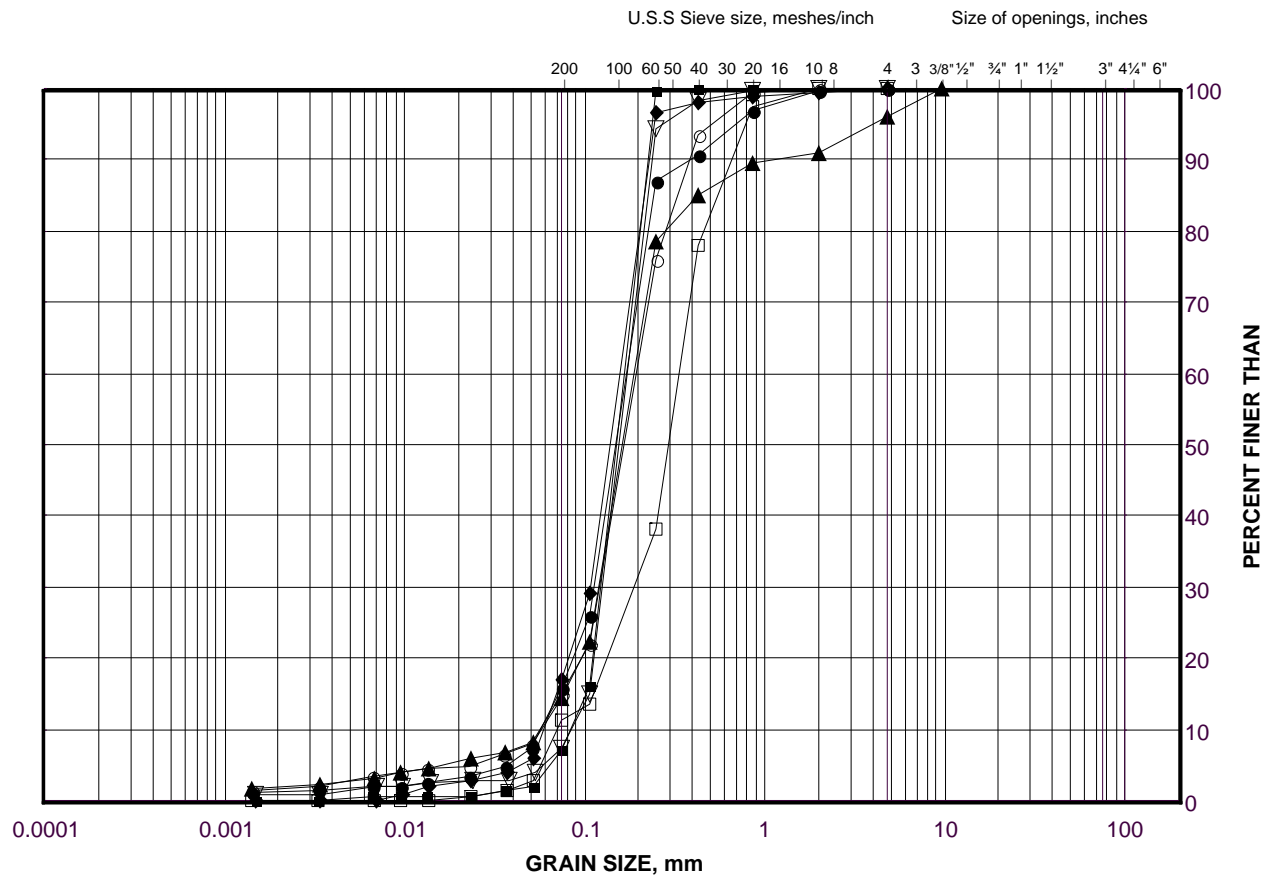
APPENDIX C

Laboratory Test Results, Bedrock Core Photographs and Chemical Test Results – Current Investigation

GRAIN SIZE DISTRIBUTION

Silty Sand to Sand to Gravelly Sand

FIGURE C1-A



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	NW6-2	3	104.5
■	HF-3	3	105.7
◆	NW 4-3	3	106.2
▲	DO -4	3	105.0
▽	DO -3	3	104.7
○	DO-2	4	104.4
□	DO-1	4	104.7

Project Number: 1530382

Checked By: MWK

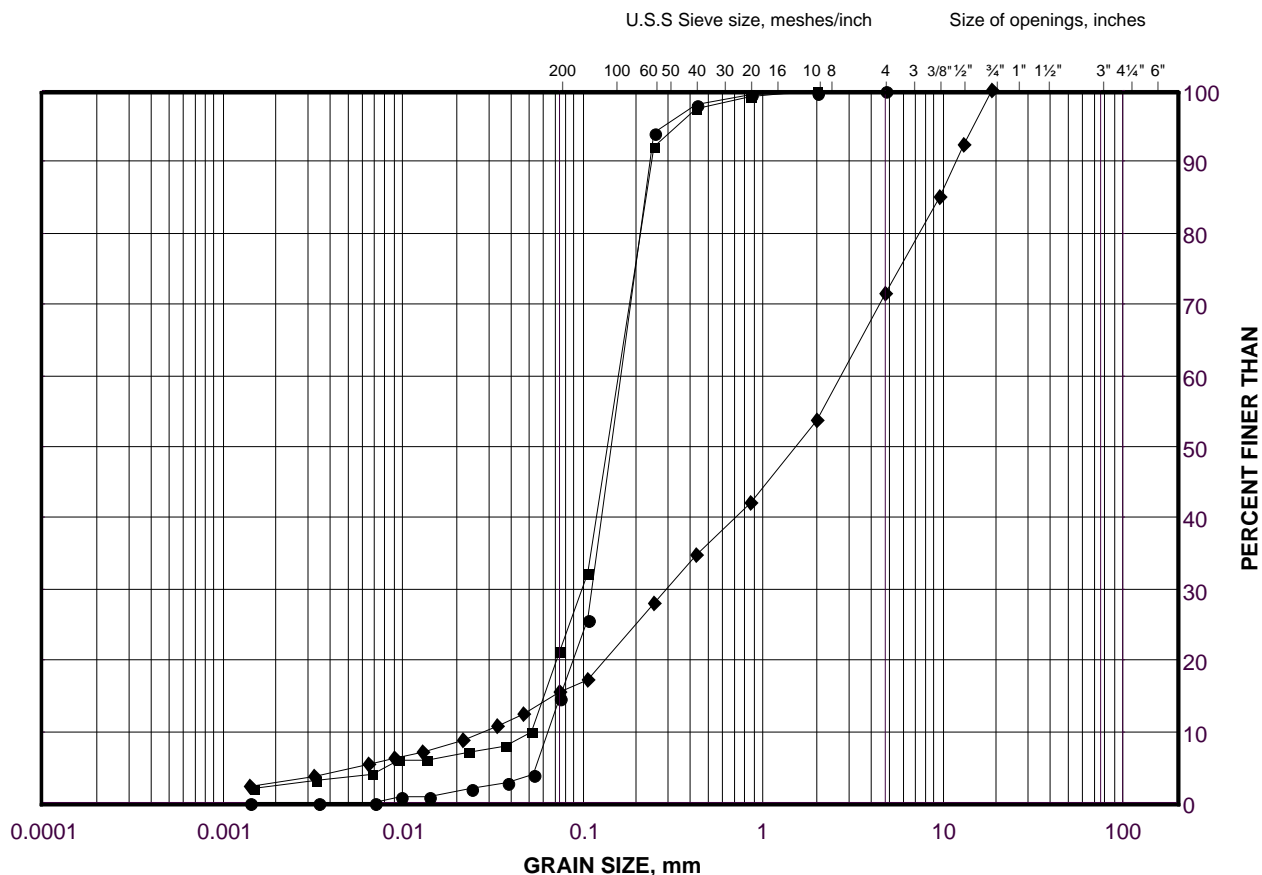
Golder Associates

Date: 16-Aug-17

GRAIN SIZE DISTRIBUTION

Silty Sand to Sand to Gravelly Sand

FIGURE C1-B



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	DO -6	2	104.7
■	DO -5	2	104.6
◆	HF -2	3A	102.7

Project Number: 1530382

Checked By: MWK

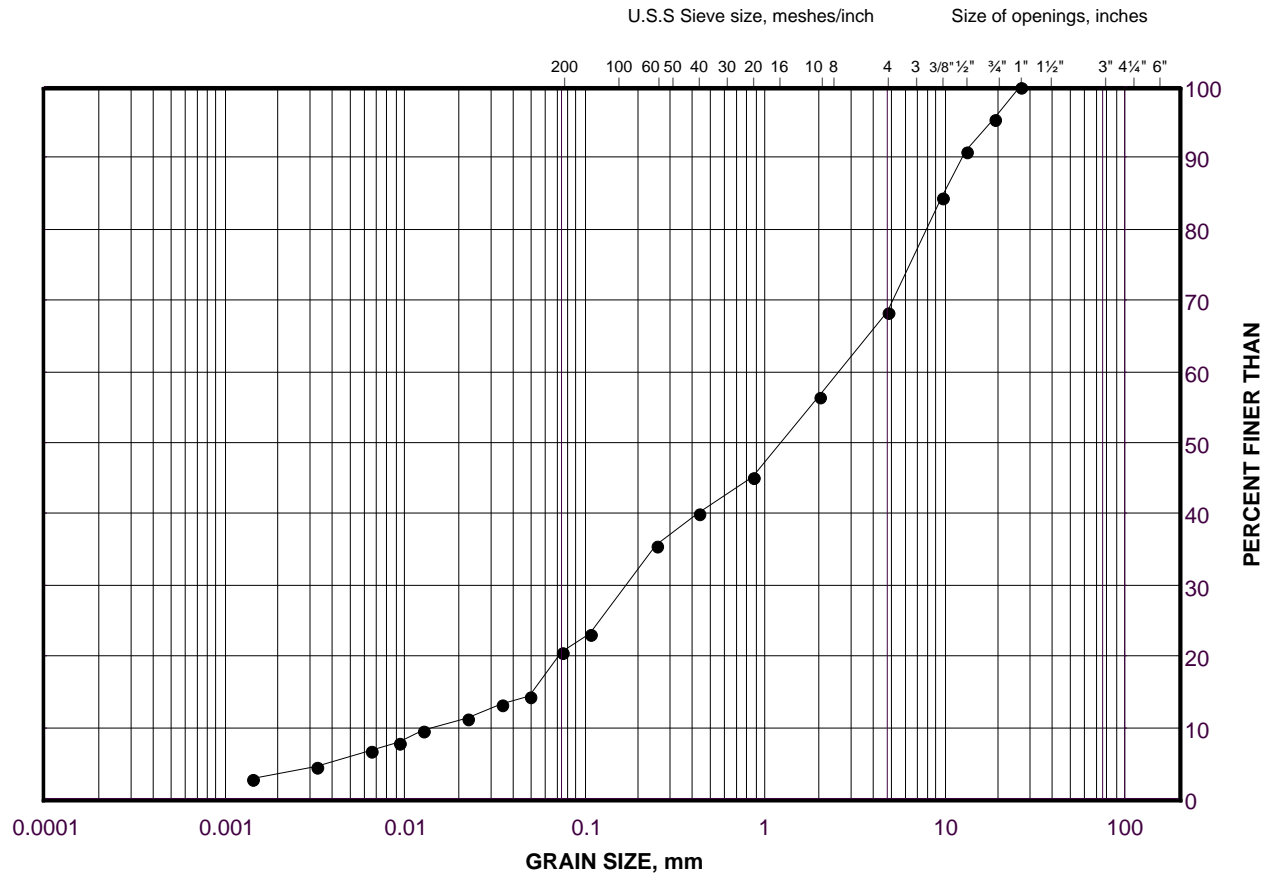
Golder Associates

Date: 16-Aug-17

GRAIN SIZE DISTRIBUTION

Gravelly Silty Sand (Till)

FIGURE C2



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

LEGEND

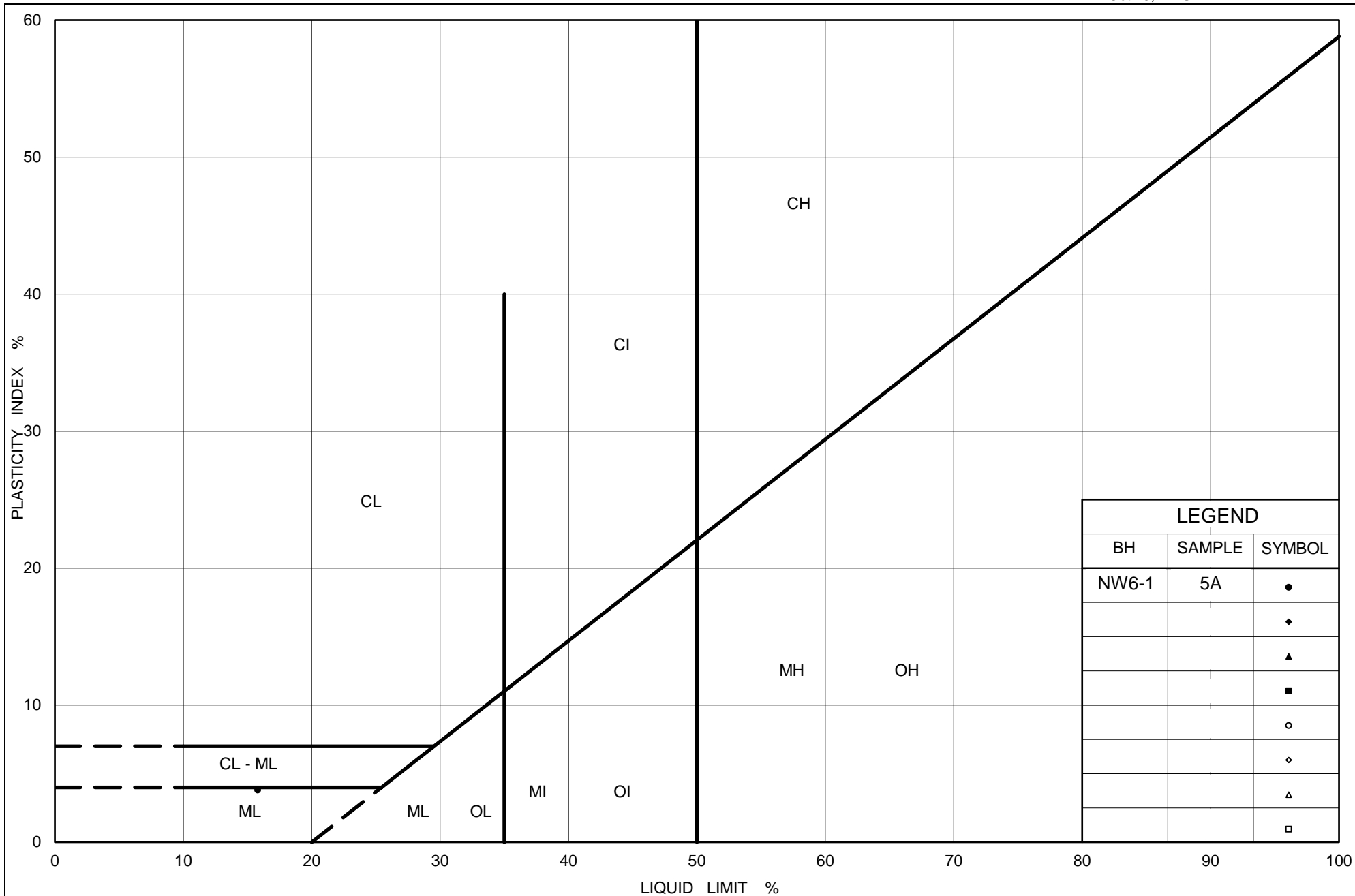
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	NW6-1	5A	102.9

Project Number: 1530382

Checked By: MWK

Golder Associates

Date: 16-Aug-17



Ministry of Transportation

Ontario

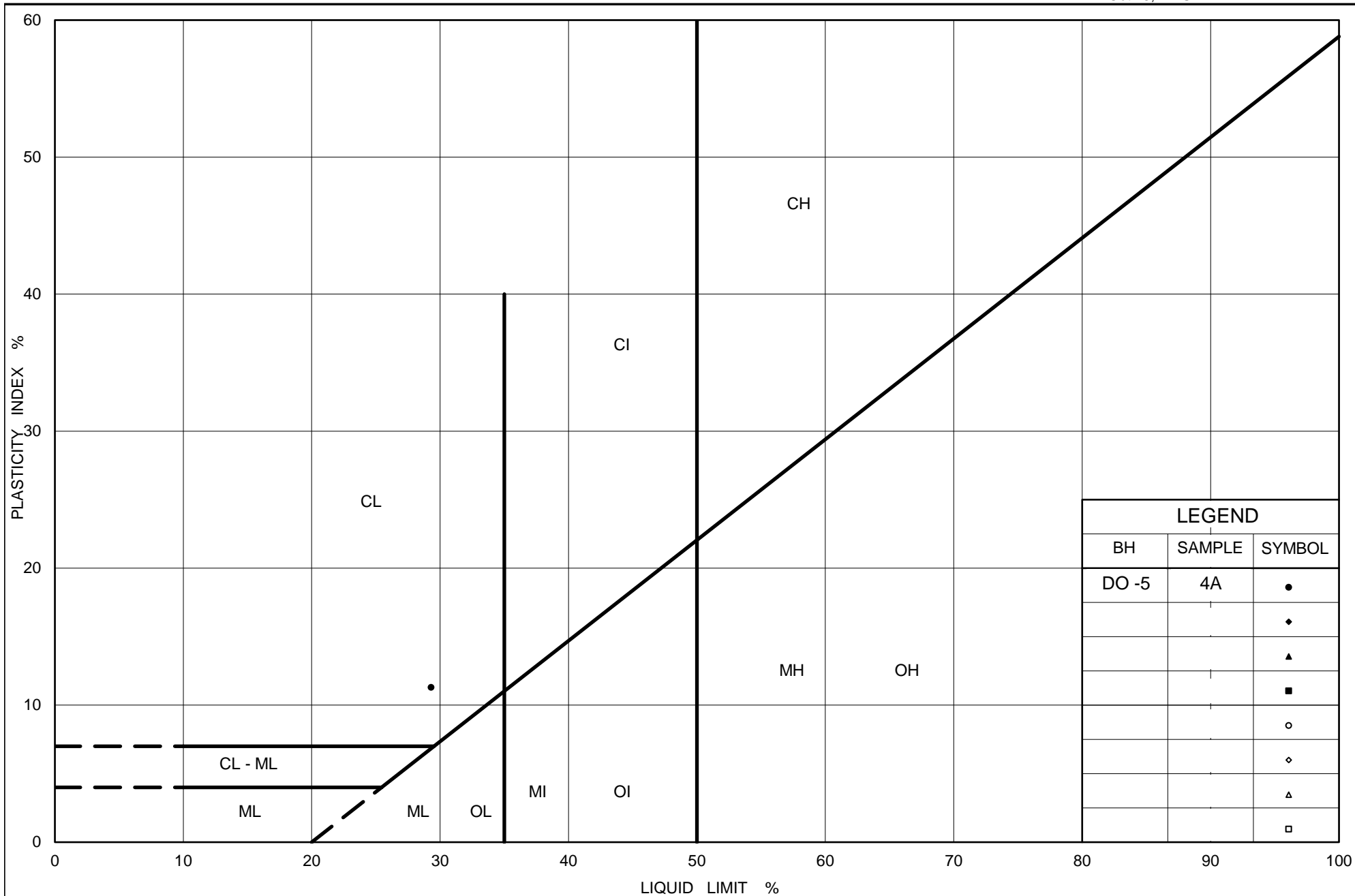
PLASTICITY CHART

Gravelly Silty Sand (Till)

Figure No. C3

Project No. 1530382

Checked By: MWK



Ministry of Transportation

Ontario

PLASTICITY CHART

Clayey Silt (Residual Soil)

Figure No. C4

Project No. 1530382

Checked By: MWK

4.09 m




6.09 m

6.09 m



7.62 m

PROJECT		FOUNDATION REPORT	
		QEW - DIXIE ROAD UNDERPASS BRIDGE REPLACEMENT, GWP 2102-13-00 & 2432-13-00 SITE 24-193	
TITLE		BEDROCK CORE PHOTOGRAPHS – DO-3	
		PROJECT No. 1530382	FILE No. ----
		DESIGN MWK	SCALE NTS
		CADD --	REV.
		CHECK	
		REVIEW JMAC	FIGURE C5

September 15, 2016

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

Re: UCS Testing of shale sample - Golder Associates Project No. 1530382

Dear Ms. McGaghran:

On September 9, 2016 two (2) NQ-sized core samples were received by Geomechanica Inc. via drop-off. These samples were identified as shale from a drilling investigation near the QEW and Dixie Rd. in Mississauga, Ontario. One (1) uniaxial compressive strength (UCS) test specimen was prepared and tested from one of the two samples. The second sample was retained as a spare.

Details regarding the steps of specimen preparation and testing along with the test results and photographs of test specimen before and after testing are presented in the accompanying laboratory report.

Sincerely,



Giovanni Grasselli Ph.D., P. Eng.

Geomechanica Inc.
Tel: (647) 478-9767
Email: giovanni.grasselli@geomechanica.com

Rock Laboratory Testing Results

A report submitted to:

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

Prepared by:

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

September 15, 2016

Project number: 1530382

Abstract

This document summarizes the results of Uniaxial Compressive Strength (UCS) testing of 1 rock core sample for Golder Associates Ltd. (Golder Project No. 1530382). The sample was identified as shale from a drilling investigation near the QEW and Dixie Rd. in Mississauga Ontario. The results, including the tabulated values of the UCS, bulk density, and elastic modulus along with photos of the test specimen before and after testing, are presented herein.

In this document:

1	Uniaxial Compressive Strength (UCS) Testing	1
---	---	---

1 Uniaxial Compressive Strength (UCS) Testing

1.1 Introduction

This section summarizes the results of UCS testing of a core sample of shale received by Geomechanica from Golder Associates Ltd. (Golder Project No. 1530382). The test was performed in Geomechanica's rock testing laboratory in Vaughan, Ontario using a 150 ton Forney loading frame equipped with pressure-compensated control valve to maintain an axial strain rate of approximately $1.3 \times 10^{-5} \text{ s}^{-1}$ (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 maintain the moisture content, avoid damage during handling, and minimize exposure to moisture during specimen preparation.
2. Diamond sawing the core sample to length such that a cylindrical specimen with a length:diameter ratio of approximately 2:1 and nearly parallel end faces was obtained.
3. Surface grinding of specimens to obtain flat and parallel end faces within $\pm 0.05 \text{ mm}$.
4. Loading the specimen into a stiff hydraulic loading frame and applying a small axial load of 0.1-0.2 kN to allow removal of the electrical tape and subsequently loading the specimen to rupture while recording axial force and axial deformation to determine the peak strength (UCS) and (tangent) Young's modulus (E).



Figure 1: Equipment setup for measuring Uniaxial Compression Strength (UCS).

1.2 Results

The results of UCS testing are summarized in Table 1. The corresponding stress-strain curve is shown in Figure 2. The Young's modulus value presented in Table 1 represents the tangent modulus, calculated as the slope of the best fit line through ± 300 data points on either side of the point representing 50% of the UCS, unless noted otherwise.

Table 1: Summary of UCS test results.

Borehole	Depth	Bulk density, ρ (g/cm ³)	UCS (MPa)	Elastic modulus, E (MPa)
BH-DO-3	6.79 - 6.94	2.64	5.5	302

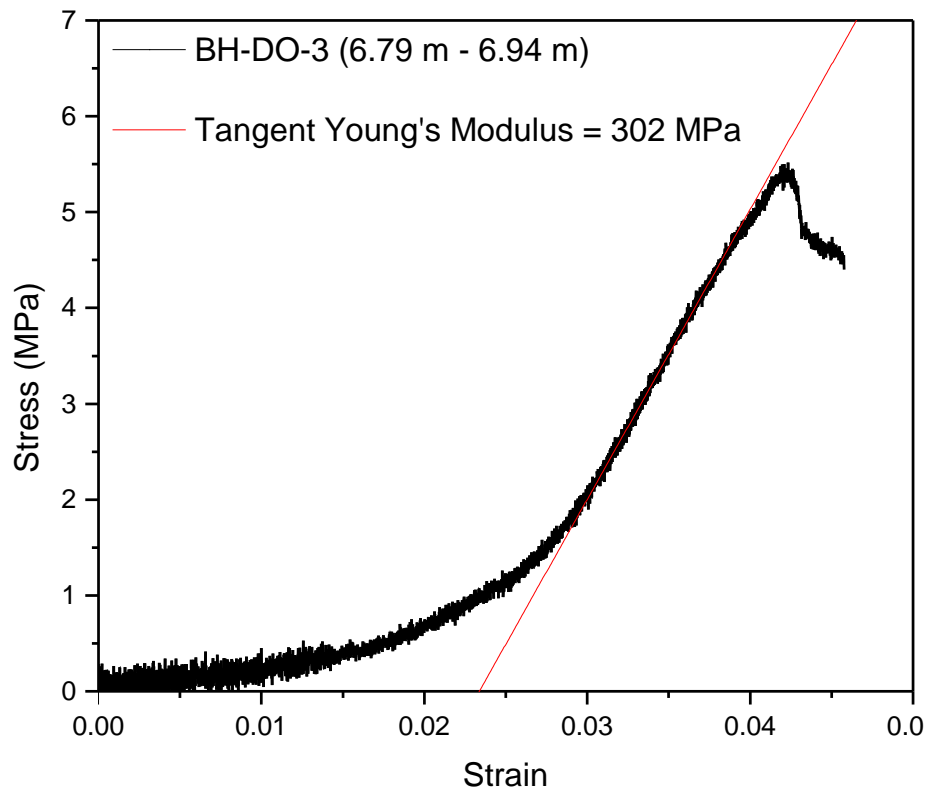
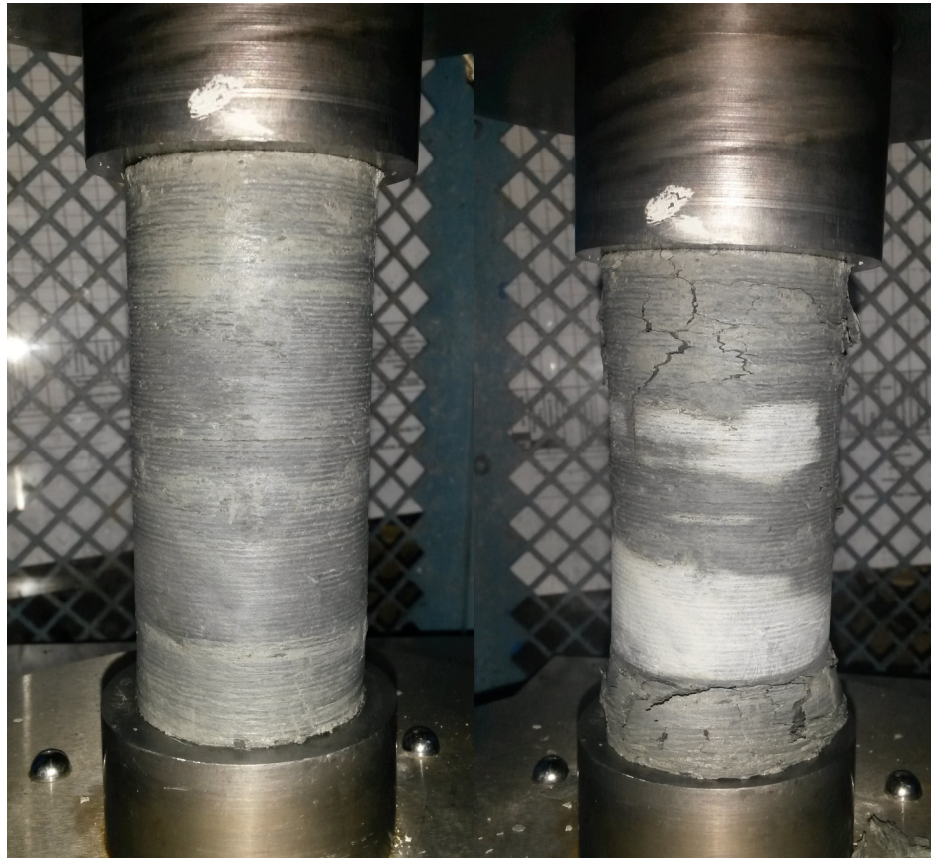


Figure 2: Measured stress-strain curve.

1.3 Specimen photographs

Photographs of the specimens before and after testing are presented in Figure 3.



BH-DO-3 6.79 m – 6.94 m
Pre-test

BH-DO-3 6.79 m – 6.94 m
Post-test

Figure 3: Photographs of UCS test specimen before and after testing. Note that the sub-horizontal lineations visible on the specimen are from the electrical tape used to protect the sample. That is, they do not reflect natural rock structure.

July 18, 2016

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

Re: UCS Testing of shale sample - Golder Associates Project No. 1530382

Dear Ms. McGaghran:

On July 12, 2017 one (1) HQ-sized core samples were received by Geomechanica Inc. via drop-off. These samples were identified as shale from a drilling investigation for Golder project 1530382 in Mississauga, Ontario. One (1) uniaxial compressive strength (UCS) test specimen was prepared and tested.

Details regarding the steps of specimen preparation and testing along with the test results and photographs of the test specimen before and after testing are presented in the accompanying laboratory report.

Sincerely,



Giovanni Grasselli Ph.D., P. Eng.

Geomechanica Inc.
Tel: (647) 478-9767
Email: giovanni.grasselli@geomechanica.com

Rock Laboratory Testing Results

A report submitted to:

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

Prepared by:

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

July 18, 2017

Project number: 1530382

Abstract

This document summarizes the results of Uniaxial Compressive Strength (UCS) testing of 1 rock core sample for Golder Associates Ltd. (Golder Project No. 1530382). The results, including the tabulated values of the UCS, bulk density, and elastic modulus along with photos of the test specimen before and after testing, are presented herein.

In this document:

1	Uniaxial Compressive Strength (UCS) Testing	1
---	---	---

1 Uniaxial Compressive Strength (UCS) Testing

1.1 Introduction

This section summarizes the results of UCS testing of a core sample of shale received by Geomechanica from Golder Associates Ltd. (Golder Project No. 1530382). The test was performed in Geomechanica's rock testing laboratory in Oakville, Ontario using a 150 ton Forney loading frame equipped with pressure-compensated control valve to maintain an axial strain rate of approximately $1.5 \times 10^{-5} \text{ s}^{-1}$ (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 maintain the moisture content, avoid damage during handling, and minimize exposure to moisture during specimen preparation.
2. Diamond cutting of the core sample to length such that a cylindrical specimen with a length:diameter ratio of approximately 2:1 and nearly parallel end faces was obtained.
3. Surface grinding the specimens to obtain flat (within $\pm 0.025 \text{ mm}$) and parallel end faces (within 0.25°).
4. Loading the specimen into a stiff hydraulic loading frame and applying a small axial load of 0.5 kN to allow removal of the electrical tape and subsequently loading the specimen to rupture while recording axial force and axial deformation to determine the peak strength (UCS) and (tangent) Young's modulus (E).

1.2 Results

The results of UCS testing are summarized in Table 1. The corresponding stress-strain curve is shown in Figure 2. The Young's modulus value presented in Table 1 represents the tangent modulus, calculated as the slope of the best fit line through ± 300 data points on either side of the point representing 50% of the UCS, unless noted otherwise.

Table 1: Summary of UCS test results.

Borehole	Depth	Bulk density, $\rho \text{ (g/cm}^3\text{)}$	UCS (MPa)	Elastic modulus, E (GPa)
BH-DO-2	9.13 - 9.27	2.59	13.1	1.01

1.3 Specimen photographs

Photographs of the test specimen before and after testing are shown in Figure 3.



Figure 1: Equipment setup for measuring Uniaxial Compression Strength (UCS).

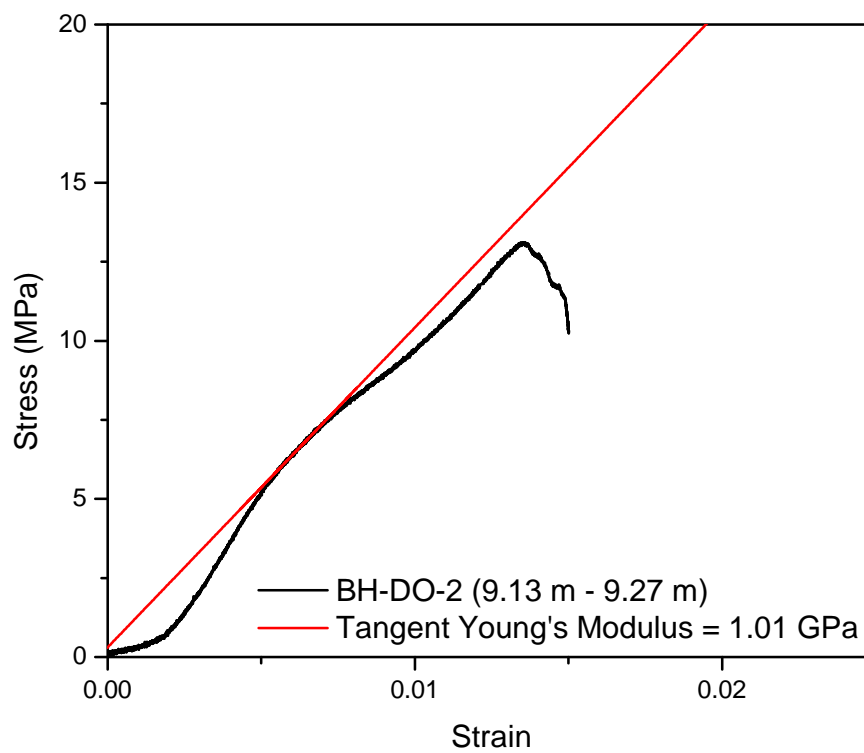


Figure 2: Measured stress-strain curve.

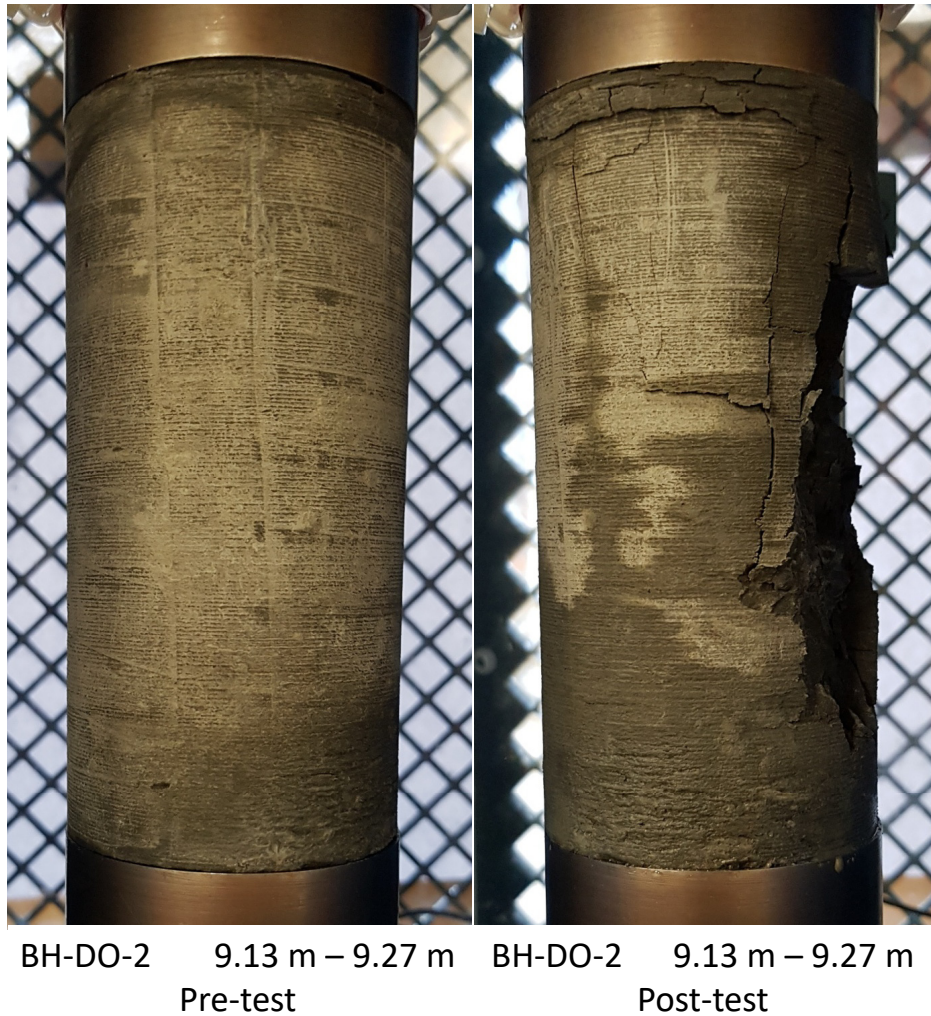


Figure 3: Photographs of UCS test specimen before and after testing. Note that the sub-horizontal lineations visible on the specimen are from the electrical tape used to protect the sample. That is, they do not reflect natural rock structure.

Your Project #: 1530382
Site Location: QEW/CAWTHRA
Your C.O.C. #: 76779

Attention: Alysha Kobylinski

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

Report Date: 2016/12/09
Report #: R4281717
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B6Q3878
Received: 2016/12/03, 15:03

Sample Matrix: Soil
Samples Received: 2

Analyses	Date		Date Analyzed	Laboratory Method	Reference
	Quantity	Extracted			
Chloride (20:1 extract)	2	N/A	2016/12/09	CAM SOP-00463	EPA 325.2 m
Conductivity	2	N/A	2016/12/09	CAM SOP-00414	OMOE E3530 v1 m
pH CaCl2 EXTRACT	2	2016/12/07	2016/12/07	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	2	2016/12/03	2016/12/09	CAM SOP-00414	SM 22 2510 m
Sulphate (20:1 Extract)	2	N/A	2016/12/09	CAM SOP-00464	EPA 375.4 m

Remarks:

Maxxam Analytics' laboratories are accredited to ISO/IEC 17025:2005 for specific parameters on scopes of accreditation. Unless otherwise noted, procedures used by Maxxam are based upon recognized Provincial, Federal or US method compendia such as CCME, MDDELCC, EPA, APHA.

All work recorded herein has been done in accordance with procedures and practices ordinarily exercised by professionals in Maxxam's profession using accepted testing methodologies, quality assurance and quality control procedures (except where otherwise agreed by the client and Maxxam 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.

Maxxam Analytics' liability is limited to the actual cost of the requested analyses, unless otherwise agreed in writing. There is no other warranty expressed or implied. Maxxam 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 Maxxam, unless otherwise agreed in writing.

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.

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 Location: QEW/CAWTHRA
Your C.O.C. #: 76779

Attention: Alysha Kobylinski

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

Report Date: 2016/12/09
Report #: R4281717
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B6Q3878
Received: 2016/12/03, 15:03

Encryption Key

Please direct all questions regarding this Certificate of Analysis to your Project Manager.

Ema Gitej, Senior Project Manager

Email: EGitej@maxxam.ca

Phone# (905)817-5829

=====

Maxxam has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per section 5.10.2 of ISO/IEC 17025:2005(E), signing the reports. For Service Group specific validation please refer to the Validation Signature Page.

RESULTS OF ANALYSES OF SOIL

Maxxam ID		DOL198	DOL198		DOL199		
Sampling Date		2016/11/07	2016/11/07		2016/11/07		
COC Number		76779	76779		76779		
	UNITS	DO- 4 - SA4 -2.29M-2.90M	DO- 4 - SA4 -2.29M-2.90M Lab-Dup	RDL	DO-5- SA2 -1.52M-2.13M	RDL	QC Batch

Calculated Parameters							
Resistivity	ohm-cm	1200			440		4777837
Inorganics							
Soluble (20:1) Chloride (Cl)	ug/g	450	390	20	1300	40	4784297
Conductivity	umho/cm	828		2	2250	2	4784499
Available (CaCl2) pH	pH	8.14			8.10		4782348
Soluble (20:1) Sulphate (SO4)	ug/g	61	54	20	27	20	4784302
RDL = Reportable Detection Limit							
QC Batch = Quality Control Batch							
Lab-Dup = Laboratory Initiated Duplicate							

Maxxam Job #: B6Q3878
Report Date: 2016/12/09

Golder Associates Ltd
Client Project #: 1530382
Site Location: QEW/CAWTHRA
Sampler Initials: AK

TEST SUMMARY

Maxxam ID: DOL198
Sample ID: DO- 4 - SA4 -2.29M-2.90M
Matrix: Soil

Collected: 2016/11/07
Shipped:
Received: 2016/12/03

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	4784297	N/A	2016/12/09	Deonarine Ramnarine
Conductivity	AT	4784499	N/A	2016/12/09	Tahir Anwar
pH CaCl2 EXTRACT	AT	4782348	2016/12/07	2016/12/07	Surinder Rai
Resistivity of Soil		4777837	2016/12/09	2016/12/09	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	4784302	N/A	2016/12/09	Alina Dobreanu

Maxxam ID: DOL198 Dup
Sample ID: DO- 4 - SA4 -2.29M-2.90M
Matrix: Soil

Collected: 2016/11/07
Shipped:
Received: 2016/12/03

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	4784297	N/A	2016/12/09	Deonarine Ramnarine
Sulphate (20:1 Extract)	KONE/EC	4784302	N/A	2016/12/09	Alina Dobreanu

Maxxam ID: DOL199
Sample ID: DO-5- SA2 -1.52M-2.13M
Matrix: Soil

Collected: 2016/11/07
Shipped:
Received: 2016/12/03

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	4784297	N/A	2016/12/09	Deonarine Ramnarine
Conductivity	AT	4784499	N/A	2016/12/09	Tahir Anwar
pH CaCl2 EXTRACT	AT	4782348	2016/12/07	2016/12/07	Surinder Rai
Resistivity of Soil		4777837	2016/12/09	2016/12/09	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	4784302	N/A	2016/12/09	Alina Dobreanu

GENERAL COMMENTS

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

Package 1	12.7°C
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Results relate only to the items tested.

QUALITY ASSURANCE REPORT

Golder Associates Ltd
Client Project #: 1530382
Site Location: QEW/CAWTHRA
Sampler Initials: AK

QC Batch	Parameter	Date	Matrix Spike		SPIKED BLANK		Method Blank		RPD	
			% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
4782348	Available (CaCl ₂) pH	2016/12/07			99	97 - 103			0.48	N/A
4784297	Soluble (20:1) Chloride (Cl)	2016/12/09	NC	70 - 130	103	70 - 130	<20	ug/g	14	35
4784302	Soluble (20:1) Sulphate (SO ₄)	2016/12/09	NC	70 - 130	105	70 - 130	<20	ug/g	NC	35
4784499	Conductivity	2016/12/09			100	90 - 110	<2	umho/cm	2.5	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 spiked amount was too small to permit a reliable recovery calculation (matrix spike concentration was less than 2x that of 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 (one or both samples < 5x RDL).

VALIDATION SIGNATURE PAGE

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



Ewa Pranjić, M.Sc., C.Chem, Scientific Specialist

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CHAIN OF CUSTODY RECORD

76779

Page 1 of 1

Invoice Information		Report Information (if differs from invoice)		Project Information (where applicable)		Turnaround Time (TAT) Required	
Company Name: <u>GOLDER ASSOCIATES</u>		Company Name:		Quotation #:		<input checked="" type="checkbox"/> Regular TAT (5-7 days) Most analyses	
Contact Name: <u>ALYSHA KOBYLINSKI</u>		Contact Name:		P.O. #/ AFE#:		PLEASE PROVIDE ADVANCE NOTICE FOR RUSH PROJECTS	
Address: <u>6925 CENTURY AVE</u>		Address:		Project #: <u>1530382</u>		Rush TAT (Surcharges will be applied)	
<u>MISSISSAUGA, ONTARIO</u>				Site Location: <u>QEW/CAWTHRA</u>		<input type="checkbox"/> 1 Day <input type="checkbox"/> 2 Days <input type="checkbox"/> 3-4 Days	
Phone: <u>905 567 4444</u> Fax: <u>905 567 6561</u>		Phone: Fax:		Site #:		Date Required:	
Email: <u>Alysha_Kobylinski@golder.com</u>		Email:		Sampled By:		Rush Confirmation #:	
MOE REGULATED DRINKING WATER OR WATER INTENDED FOR HUMAN CONSUMPTION MUST BE SUBMITTED ON THE MAXXAM DRINKING WATER CHAIN OF CUSTODY							
Regulation 153		Other Regulations		Analysis Requested		LABORATORY USE ONLY	
<input type="checkbox"/> Table 1 <input type="checkbox"/> Res/Park <input type="checkbox"/> Med/ Fine <input type="checkbox"/> Table 2 <input type="checkbox"/> Ind/Comm <input type="checkbox"/> Coarse <input type="checkbox"/> Table 3 <input type="checkbox"/> Agri/ Other <input type="checkbox"/> Table _____ FOR RSC (PLEASE CIRCLE) Y / N		<input type="checkbox"/> CCME <input type="checkbox"/> Sanitary Sewer Bylaw <input type="checkbox"/> MISA <input type="checkbox"/> Storm Sewer Bylaw <input type="checkbox"/> PWQO <input type="checkbox"/> Region <input type="checkbox"/> Other (Specify) <input type="checkbox"/> REG 558 (MIN. 3 DAY TAT REQUIRED)		REFERENCE TO BACK OF COC REG 153 METALS & INORGANICS REG 153 CPMS METALS REG 153 METALS (Hg, Cr VI, CPMS Metals, HWS - B) CORROSIVITY PACKAGE		CUSTODY SEAL Y / N Present Intact COOLER TEMPERATURES 11/12/16 COOLING MEDIA PRESENT: <input checked="" type="checkbox"/> Y / N	
Include Criteria on Certificate of Analysis: Y / N							
SAMPLES MUST BE KEPT COOL (<10 °C) FROM TIME OF SAMPLING UNTIL DELIVERY TO MAXXAM							
SAMPLE IDENTIFICATION		DATE SAMPLED (YYYY/MM/DD)	TIME SAMPLED (HH:MM)	MATRIX	# OF CONTAINERS SUBMITTED	HELD FILTERED (CIRCLE) Metals / Hg / CrVI	DO NOT ANALYZE
1 DO-4-S44-2-29m-2.90m		2016/11/07	-	SOIL	1	N	
2 DO-5-S42-1.52m-2.13m		2016/11/07	-	SOIL	1	N	
3							
4							
5							
6							
7							
8							
9							
10							
RELINQUISHED BY: (Signature/Print)		DATE: (YYYY/MM/DD)	TIME: (HH:MM)	RECEIVED BY: (Signature/Print)		DATE: (YYYY/MM/DD)	TIME: (HH:MM)
Alysha Kobylinski		2016/12/03	15:03	Tawir S. Tawir		2016/12/03	15:03
ALYSHA KOBYLINSKI							
MAXXAM JOB #							

Your Project #: 1530382
Site Location: QEW/DIXIE
Your C.O.C. #: na

Attention: Sandra McGaghran

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

Report Date: 2017/07/13
Report #: R4596038
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B7E2708

Received: 2017/07/06, 16:30

Sample Matrix: ROCK
Samples Received: 1

Analyses	Date		Date Analyzed	Laboratory Method	Reference
	Quantity	Extracted			
Chloride (20:1 extract)	1	N/A	2017/07/12	CAM SOP-00463	EPA 325.2 m
Conductivity	1	N/A	2017/07/12	CAM SOP-00414	OMOE E3530 v1 m
pH CaCl2 EXTRACT	1	2017/07/12	2017/07/12	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	1	2017/07/07	2017/07/12	CAM SOP-00414	SM 22 2510 m
Sulphate (20:1 Extract)	1	N/A	2017/07/12	CAM SOP-00464	EPA 375.4 m

Remarks:

Maxxam Analytics' laboratories are accredited to ISO/IEC 17025:2005 for specific parameters on scopes of accreditation. Unless otherwise noted, procedures used by Maxxam are based upon recognized Provincial, Federal or US method compendia such as CCME, MDDELCC, EPA, APHA.

All work recorded herein has been done in accordance with procedures and practices ordinarily exercised by professionals in Maxxam's profession using accepted testing methodologies, quality assurance and quality control procedures (except where otherwise agreed by the client and Maxxam 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.

Maxxam Analytics' liability is limited to the actual cost of the requested analyses, unless otherwise agreed in writing. There is no other warranty expressed or implied. Maxxam 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 Maxxam, unless otherwise agreed in writing.

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.

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 Location: QEW/DIXIE
Your C.O.C. #: na

Attention:Sandra McGaghran

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

Report Date: 2017/07/13
Report #: R4596038
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B7E2708
Received: 2017/07/06, 16:30

Encryption Key

Please direct all questions regarding this Certificate of Analysis to your Project Manager.

Ema Gitej, Senior Project Manager

Email: EGitej@maxxam.ca

Phone# (905)817-5829

=====

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Maxxam Job #: B7E2708
Report Date: 2017/07/13

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

RESULTS OF ANALYSES OF ROCK

Maxxam ID		ERW232		
Sampling Date		2017/06/15		
COC Number		na		
	UNITS	DO-2-4.19M-4.30M	RDL	QC Batch
Calculated Parameters				
Resistivity	ohm-cm	3500		5062825
Inorganics				
Soluble (20:1) Chloride (Cl)	ug/g	28	20	5066879
Conductivity	umho/cm	284	2	5068831
Available (CaCl2) pH	pH	8.02		5067289
Soluble (20:1) Sulphate (SO4)	ug/g	110	20	5066944
RDL = Reportable Detection Limit				
QC Batch = Quality Control Batch				

Maxxam Job #: B7E2708
Report Date: 2017/07/13

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

TEST SUMMARY

Maxxam ID: ERW232
Sample ID: DO-2-4.19M-4.30M
Matrix: ROCK

Collected: 2017/06/15
Shipped:
Received: 2017/07/06

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5066879	N/A	2017/07/12	Deonarine Ramnarine
Conductivity	AT	5068831	N/A	2017/07/12	Xuanhong Qiu
pH CaCl2 EXTRACT	AT	5067289	2017/07/12	2017/07/12	Tahir Anwar
Resistivity of Soil		5062825	2017/07/12	2017/07/12	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5066944	N/A	2017/07/12	Deonarine Ramnarine

GENERAL COMMENTS

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

Package 1	7.0°C
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Results relate only to the items tested.

QUALITY ASSURANCE REPORT

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

QC Batch	Parameter	Date	Matrix Spike		SPIKED BLANK		Method Blank		RPD	
			% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
5066879	Soluble (20:1) Chloride (Cl)	2017/07/12	107	70 - 130	103	70 - 130	<20	ug/g	NC	35
5066944	Soluble (20:1) Sulphate (SO4)	2017/07/12	NC	70 - 130	108	70 - 130	<20	ug/g	5.1	35
5067289	Available (CaCl2) pH	2017/07/12			100	97 - 103			0.27	N/A
5068831	Conductivity	2017/07/12			100	90 - 110	<2	umho/cm	2.2	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 <= 2x RDL).

VALIDATION SIGNATURE PAGE

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

Cristina Carriere

Cristina Carriere, Scientific Services

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CHAIN OF CUSTODY RECORD **76164** Page 1 of 1

Invoice Information		Report Information (If differs from Invoice)		Project Information (where applicable)		Turnaround Time (TAT) Required	
Company Name: <u>GOLDER ASSOCIATES</u>		Company Name: <u>GOLDER ASSOCIATES</u>		Quotation #: _____		<input checked="" type="checkbox"/> Regular TAT (5-7 days) Most analyses	
Contact Name: <u>Alysha Kobylinski</u>		Contact Name: <u>SANDRA MCGAGHRAN</u>		P.O. #/ AFE#: _____		PLEASE PROVIDE ADVANCE NOTICE FOR RUSH PROJECTS	
Address: <u>6925 CENTURY AVE, SUITE #100</u>		Address: <u>6925 CENTURY AVE, SUITE #100</u>		Project #: <u>1530382</u>		Rush TAT (Surcharges will be applied)	
<u>MISSISSAUGA, ON</u>		<u>MISSISSAUGA, ON</u>		Site Location: <u>GEW/DIXIE</u>		<input type="checkbox"/> 1 Day <input type="checkbox"/> 2 Days <input type="checkbox"/> 3-4 Days	
Phone: <u>905 567 4444</u> Fax: <u>905 567 6561</u>		Phone: <u>905 567 4444</u> Fax: <u>905 567 6561</u>		Site #: _____		Date Required: _____	
Email: <u>akobylinski@golder.com</u>		Email: <u>sandra.mcgaighran@golder.com</u>		Sampled By: _____		Rush Confirmation #: _____	
MOE REGULATED DRINKING WATER OR WATER INTENDED FOR HUMAN CONSUMPTION MUST BE SUBMITTED ON THE MAXXAM DRINKING WATER CHAIN OF CUSTODY							
Regulation 153		Other Regulations		Analysis Requested		LABORATORY USE ONLY	
<input type="checkbox"/> Table 1 <input type="checkbox"/> Res/Park <input type="checkbox"/> Med/ Fine <input type="checkbox"/> Table 2 <input type="checkbox"/> Ind/Comm <input type="checkbox"/> Coarse <input type="checkbox"/> Table 3 <input type="checkbox"/> Agri/ Other <input type="checkbox"/> Table _____ FOR RSC (PLEASE CIRCLE) Y / N		<input type="checkbox"/> CCME <input type="checkbox"/> Sanitary Sewer Bylaw <input type="checkbox"/> MISA <input type="checkbox"/> Storm Sewer Bylaw <input type="checkbox"/> PWQO <input type="checkbox"/> Region <input type="checkbox"/> Other (Specify) _____ <input type="checkbox"/> REG 558 (MIN. 3 DAY TAT REQUIRED)		REFER TO BACK OF COC REG 153 METALS & INORGANICS REG 153 ICPMs METALS REG 153 METALS (Hg, Cr VI, ICPMs Metals, HWS, B) CORROSIVITY PACKAGE		CUSTODY SEAL Y / N Present Intact 7/7/7 COOLING MEDIA PRESENT: <input checked="" type="checkbox"/> Y / N	
Include Criteria on Certificate of Analysis: Y / N							
SAMPLES MUST BE KEPT COOL (< 10 °C) FROM TIME OF SAMPLING UNTIL DELIVERY TO MAXXAM							
SAMPLE IDENTIFICATION		DATE SAMPLED (YYYY/MM/DD)	TIME SAMPLED (HH-MM)	MATRIX	# OF CONTAINERS SUBMITTED	FIELD FILTERED (CIRCLE) Metals / Hg / CrVI	BTX/ PHC F1
1 <u>D0-Z-4.19m-4.30m</u>		<u>2017/06/15</u>	<u>AM</u>	<u>ROCK</u>	<u>1</u>		
2							
3							
4							
5							
6							
7							
8							
9							
10							
RELINQUISHED BY: (Signature/Print)		DATE: (YYYY/MM/DD)	TIME: (HH-MM)	RECEIVED BY: (Signature/Print)		DATE: (YYYY/MM/DD)	TIME: (HH-MM)
<u>Alysha Kobylinski</u>		<u>2017/07/06</u>	<u>4:10 AM</u>	<u>[Signature]</u>		<u>2017/07/06</u>	<u>16:30</u>

07-Jul-17 16:30

Ema Gitej



B7E2708

MNI ENV-1196



APPENDIX D

Non-Standard Special Provisions

CSP FOR INTEGRAL ABUTMENTS – Item No

Non-Standard Special Provision

Scope

This specification covers the requirements for the installation of the corrugated steel pipes (CSPs) at the integral abutments.

Submission and Design Requirements

All submissions shall bear the seal and signature of an Engineer.

At least two weeks prior to commencement of installation of the abutment piles, the Contractor shall submit to the Contract Administrator, for information purposes only, three (3) sets of the working drawings.

The Contractor shall have a copy of the submitted working drawings on site at all times. Working drawings shall include at least the following:

1. Layout and elevations of the CSPs;
2. Location of reference points, and location of the centroid of each pile with respect to the reference points;
3. Construction sequence and details;
4. Source of the sand fill, and description of placing methods and equipment;
5. Location and details of all temporary bracing and spacers for the piles and CSPs;
6. Method for preventing water and debris from entering the CSP prior to placing sand; and
7. Method for preventing concrete from abutment pours from entering the CSPs during placement.

The Contractor shall be responsible for the complete detailed design of all temporary bracing, including spacers required to maintain the piles, CSP spacing and abutment stems in their specified positions through all stages of construction until the CSPs have been backfilled. All temporary bracing shall be removed.

Material

Corrugated steel pipe

CSP shall be in accordance with OPSS 1801, and shall be from a supplier listed under DSM#4.60.80. The CSP shall be of the diameter and wall thickness specified on the Contract drawings, and shall be galvanized in accordance with CSA G164-M.

CSPs shall be supplied in the lengths and with the end treatments, either square or skew, as specified on the Contract drawings; field cutting and splicing of CSPs will not be permitted. Cut ends shall be neat and free of burrs. The planes defined by the end treatments of each CSP shall be parallel to each other.

Handling and storage of CSPs shall be in accordance with the manufacturer's recommendations. Damaged CSPs shall be rejected. Localized areas of damaged galvanizing on otherwise acceptable CSPs shall be repaired with two coats of zinc-rich paint.

Sand Fill

The sand fill for backfilling the CSP shall meet the gradation requirements of Table 1 below:

Table 1 – Sand Fill Gradation Requirements

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

Construction

The sequence of construction shall be in accordance with the working drawings and as follows, unless otherwise approved:

1. Construct levelling pad and place CSPs and spacers.
2. Install piles by driving to design criteria.
3. Place loose sand into 600 diameter CSP.
4. Remove temporary spacers.

The CSP shall be positioned such that the piles are centrally positioned within the CSP. Temporary blocking and bracing shall be used to hold the CSP in position.

The Contractor shall ensure the full perimeters of the tops of all CSPs at each abutment are at the elevation and orientation shown on the working drawings.

The CSP at each pile shall be constructed to the following tolerances:

Criteria	Tolerance
Maximum deviation of CSP from pile centroid	+/- 50 mm
Maximum deviation of any point on the top perimeter of the CSP from the specified elevation	+/- 10 mm

The sand fill shall be placed dry of optimum and free-flowing, completely filling the volume between the CSP and pile. No additional compaction effort other than the action of placing the sand itself shall be applied to the sand fill.

The placing of the sand fill shall be carried out in a manner such as to not damage and displace the CSP.

Basis of Payment

Payment at the contract price for the above tender item shall include all labour, equipment and material required to do the work.

END OF SECTION

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 weak in the area of the Dixie Road underpass bridge and which contains medium 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 at the piers, the Contractor shall provide dowels into the bedrock at the foundations for the QEW-Dixie Road 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 at each foundation element 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 percent of the theoretical elastic elongation of the free stressing and is less than the theoretical elongation of the free stressing length plus 50 percent 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.

STEEL H-PILE FOUNDATIONS - Item No.

Non-Standard Special Provision

Where steel H-pile foundations are used for support of the bridge foundation elements, the foundations will extend into the shale bedrock, which is very weak to weak and which contains clay seams and medium strong limestone layers at varying depths/elevations. Appropriate equipment and construction procedures will be required to penetrate the overburden and concrete from the existing footings and advance sockets into the bedrock to reach the design founding level.

Where steel H-pile foundations are used for support of the bridge foundation elements, the steel H-piles must be installed in pre-drilled holes/sockets (minimum 1 m deep) within the bedrock.

The base of each pre-drilled hole/rock socket must be cleaned in accordance with OPSS 903 of all loose cuttings and debris before the H-pile is installed to the base of the socket.

If the rock sockets cannot be cleaned to the above requirements to the satisfaction of the Foundation Engineer, then the steel H-piles shall be seated in the base of the rock sockets by driving. The piles shall be driven in accordance with the requirements of OPSS 903.

If unusually excessive penetration per blow is observed, driving shall be stopped and this excessive penetration immediately reported to the Contract Administrator.

The Foundation Engineer shall determine when the hammer energy can be increased and when the driving is complete for each pile.

Following installation of the steel piles into the rock socket, the annular space between the pile and sides of the rock socket shall be filled with concrete (1 m in length). The concrete must be placed using tremie methods. The pile installation into the rock socket (including concrete placement) must be completed within 24 hours of the rock socket reaching the design tip elevation of the pile.

No sooner than 24 hours after placement of the concrete (by tremie methods) in the rock socket, the remaining annular space between the top of the concrete and the underside of the CSP forming the integral abutment shall be backfilled with OPSS.PROV 1010 Granular 'B' Type II. This material must be placed while the temporary liner remains in place. The temporary liner shall be removed in stages while the granular is added, however, at all times, a minimum 0.5 m thickness of the granular material must be present up inside and above the tip of the liner as the liner is withdrawn so that the annulus between the pile and the sides of the pre-drilled hole is entirely backfilled with granular material and so that no native material caves into the annular space.

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

OBSTRUCTIONS - Item No.

Non-Standard Special Provision

The native cohesive residual soils contain shale fragments as indicated in the Record of Borehole sheets. Although not encountered in the boreholes advanced at this site, the non-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 excavation, 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.

Non-Standard Special Provision

Scope

This special provision describes requirements for vibration monitoring during bedrock excavation, piling / caisson installation works for the construction of the QEW – Dixie Road underpass bridge structure.

References

The subsurface conditions at the site are described in the following Foundation Investigation Report for GWP 2102-13-00 and 2432-13-00:

QEW - Dixie Road Underpass Bridge Replacement Structure Site No. 24-193,
QEW Widening From East of Cawthra Road to The East Mall,
Cities of Mississauga and Etobicoke
Ministry of Transportation, Ontario
GWP 2102-13-00 & 2432-13-00

Definitions

Quality Verification Engineer (QVE): An Engineer with a minimum of five (5) years' experience in the field of installation of piling and vibration monitoring or, alternatively, with expertise demonstrated by providing satisfactory quality verification services for a minimum of two (2) projects of similar scope to the contract. The QVE shall be retained by the Contractor to ensure general conformance with the contract documents and issue certificates of conformance.

Submission Requirements

The Contractor/QVE shall submit details of the vibration monitoring plan to the Contract Administrator for review. The submittals shall satisfy the specifications and at a minimum contain the following specific information:

- Qualifications of vibration monitoring specialist.
- Details regarding proposed instrumentation.
- Proposed location of instruments.
- Proposed frequency of readings.
- Proposed methods for adjusting piling methods if readings show vibrations exceeding tolerable levels.

Monitoring

The vibration monitoring equipment shall be placed as close as possible to the works. The Contractor/QVE shall take readings on the existing residential structures located within 100 m of the works during driving of each pile, starting with the pile furthest away for each foundation element.

The vibrations measured at the site shall not exceed 50 mm/s (peak particle velocity).

The results shall be submitted to the Contract Administrator after each pile has been driven, prior to continuing with the subsequent piles. As a minimum, the pile number, location, set criteria and driving log must be submitted with vibration monitoring results.

If the vibration monitoring results are acceptable, the Contractor may continue with the next pile(s) with readings taken during driving of each pile. The results of subsequent piles should be submitted to the Contract Administrator after each pile has been driven.

If the readings are not within the limits stated above, the Contractor must alter the driving procedures until the vibrations at the existing structures are within acceptable levels. The above process must be repeated for each pile.

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 Etobicoke 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

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