Foundation Investigation and Design Report

Retaining Wall at Station 14+072 to 14+087, East of Etobicoke Creek
QEW Improvements from East of Cawthra Road to The East Mall, Mississauga and Etobicoke
Ministry of Transportation, Ontario
G.W.P. 2102-13-00; 2432-13-00

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GEOCRES No.: 30M11-296
(LAT. 43.605677°, LONG. -79.559576°)

November 18, 2019

Distribution List

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PART A

FOUNDATION INVESTIGATION REPORT
RETAINING WALL FROM STATION 14+072 TO 14+087, EAST OF ETOBICOKE CREEK
QEW IMPROVEMENTS FROM EAST OF CAWTHRA ROAD TO THE EAST MALL, CITIES OF MISSISSAUGA AND ETOBICOKE
MINISTRY OF TRANSPORTATION, ONTARIO
G.W.P. 2102-13-00; 2432-13-00

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by AECOM, on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for a retaining wall to be constructed on the north side of the Queen Elizabeth Way (QEW) and east of Etobicoke Creek. This retaining wall is required for the QEW widening near The West Mall on-ramp to the Niagara bound QEW, as part of the QEW improvements from east of Cawthra Road to The East Mall, in the City of Mississauga and the City of Etobicoke, Ontario.

The purpose of this foundation investigation is to establish the subsurface soil and groundwater conditions in the vicinity of the proposed retaining wall by borehole drilling and geotechnical laboratory testing and analytical chemistry laboratory testing on selected soil samples.

The Terms of Reference and scope for the foundation investigation are outlined in MTO's Request for Proposal, dated June 2011, which forms part of the Consultant Agreement for Assignment No. 2015-E-0001 for this project. The scope of work is further outlined in Golder's Change Request, dated August 26, 2019. The work has been carried out in accordance with Golder's Supplementary Specialty Plan for this project, dated June 2016.

2.0 SITE DESCRIPTION

The proposed retaining wall between Stations 14+072 and 14+087 will be located on the north side of the QEW Niagara bound lanes and approximately 65 m east of Etobicoke Creek, in the City of Etobicoke, Ontario. The proposed retaining wall is required adjacent to an existing sanitary sewer maintenance hole, to permit widening of the QEW Niagara bound lanes to the north. The central portion of the retaining wall will be parallel to the QEW and will have a length of about 6 m with wing walls about 6 m to 7 m in length extending outward and downwards from the ends of the central portion to retain the existing slope.

The top of the existing sanitary maintenance hole at the toe of the existing embankment side slope is at about Elevation 96.4 m. At the time of the investigation the site was under construction for the replacement of the Etobicoke Creek bridge and there was a construction access road that extended from the N-W on-ramp sloping downwards toward the site of the Etobicoke Creek replacement bridge. In the vicinity of the maintenance hole, the ground surface along the construction access road varied from about Elevation 100 m to Elevation 97.5 m declining to the west towards Etobicoke Creek. The side slope of the construction access road was inclined at about 1 horizontal to 1 vertical (1H:1V). The toe of the access road embankment is between approximately Elevations 95 m to 94 m and existing ground surface between the toe of the embankment and the maintenance hole is inclined at about 3H:1V. The ground surface between the construction access road and the top of the slope contained construction debris consisting of concrete, the rubber base of a traffic barrel, pieces of wood, cobbles and boulders. At the time of the investigation, there was standing water at the toe of the access road embankment in the vicinity of the proposed wall. The grade of the QEW Niagara bound lanes in this area varies from about Elevations 102 m to 100 m; therefore, the existing QEW roadway embankment is approximately 5 m to 6 m in height relative to the ground surface at the toe.

The existing ground surface along the alignment of the proposed retaining wall is at about Elevation 97.5 m.

A photograph of the site conditions in the vicinity of the proposed wall at the time of our investigation is shown in Photograph 1 below.

Photograph 1: Looking northwest and down-slope from top of existing QEW embankment in the vicinity of the proposed retaining wall.

3.0 INVESTIGATION PROCEDURES

The field work for the foundation investigation was carried out on September 15 and 16, 2019, during which time three sampled boreholes, designated as Boreholes 19-1 to 19-3 were advanced. Boreholes 19-1 and 19-2 were drilled from the construction access road in the vicinity of the existing maintenance hole and Borehole 19-3 was advanced at the toe of the slope and downslope from the existing maintenance hole. The boreholes were located approximately 10 m from the maintenance hole (and associated sewer pipe) due to a requirement of the City of Toronto utility locate department. The locations of the boreholes are shown on Drawing 1 and the Records of Boreholes are provided in Appendix A.

Field drilling was carried out using a track-mounted drilling rig, supplied and operated by Walker Drilling (Walker) of Utopia, Ontario. The boreholes were advanced through the overburden using 57 mm inside diameter (I.D.) hollow-stem augers. Soil samples were obtained in Boreholes 19-1 and 19-2 at 0.75 m intervals of depth, and continuously sampled in Borehole 19-3, using a 50 mm outside diameter split-spoon sampler driven by a manual hammer, in accordance with the Standard Penetration Test (SPT) procedures outlined in ASTM D1586-11[1]. The boreholes were typically advanced to sampler refusal and/or auger refusal and bedrock was inferred by split-spoon sampling. Boreholes 19-1 and 19-2 were advanced to depths of about 4.6 m and 6.2 m below the existing construction access road ground surface at the time of the investigation and Borehole 19-3 was advanced to a depth of 2 m below the ground surface at the toe of the embankment slope.

Groundwater conditions and water levels in the open boreholes were observed during drilling and immediately following the drilling operations, and are noted on the borehole records in Appendix A and summarized in Section 4.2.4. All boreholes were backfilled with bentonite upon completion in accordance with Ontario Regulation 903 Wells (as amended).

[1] ASTM D1586 Standard Test Method for Standard Penetration Test and Split-Barrel Sampling of Soils

Field work was observed on a full-time basis by a member of Golder's engineering staff who located the boreholes in the field, arranged for the clearance of underground services, observed the drilling and sampling operations, logged the boreholes, and examined the soil samples. The samples were identified in the field, placed in labelled containers and transported to Golder's Mississauga laboratory where the samples underwent further visual examination and laboratory testing. Index testing (including water content, Atterberg limits, grain size distribution, and organic content) was carried out on selected soil samples, in accordance with MTO and/or ASTM standards, as applicable.

Two selected soil samples were submitted, under chain-of-custody procedures, to Bureau Veritas (formerly Maxxam) of Mississauga, Ontario (a Standards Council of Canada (SCC) accredited laboratory) for a suite of parameters that indicate corrosivity potential including pH, resistivity, conductivity, chloride content and sulphate content.

The as drilled borehole locations and ground surface elevations were measured using a Trimble GPS unit (Trimble XH 3.5G), having an accuracy of 0.1 m in the vertical and horizontal directions. The locations provided on the borehole records and shown on Drawing 1 are positioned relative to MTM NAD 83 (Zone 10) CSRS (V6) (2010 epoch) northing and easting coordinates, and the ground surface elevations are referenced to the Canadian Geodetic Vertical Datum (CGVD) 1928. The borehole locations, geographic coordinates, ground surface elevations and drilled depths are summarized below.

Borehole No.	Location (MTM NAD 83, Zone 10)		Ground Surface Elevation(m)	Borehole Depth (m)
	Northing (m) (Latitude, °)	Easting (m) (Longitude, °)		
19-1	4,829,516.8 (43.605580)	299,986.0 (-79.559636)	97.4	4.6
19-2	4,829,529.5 (43.605695)	300,001.9 (-79.559440)	99.7	6.2
19-3	4,829,540.3 (43.605791)	299,993.9 (-79.559539)	95.5	2.0

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 Lake 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 glacial Lake Iroquois, primarily sands, silts and gravels, with a shallow cover of till remaining over the bedrock.

[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.)

The Georgian Bay Formation which 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

Subsurface soil, bedrock and groundwater conditions as encountered in the boreholes, details of the water level readings, and the results of geotechnical laboratory testing carried out on selected soil samples are presented on the Record of Borehole sheets provided in Appendix A. 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. Lists on abbreviations and symbols are also included in Appendix A to assist in the interpretation of the borehole records. The results of the geotechnical laboratory testing on the soil samples are presented in Appendix B. The analytical laboratory test report is included in Appendix B and the test results are summarized in Section 4.2.5.

The stratigraphic boundaries shown on the borehole records and on the stratigraphic profile on Drawing 2 are inferred from both non-continuous and continuous sampling, observations of drilling progress and the results of Standard Penetration Tests. It should be noted that the interpreted stratigraphy shown on Drawing 2 is a simplification of the subsurface conditions. The factual data presented on the Record of Borehole sheets governs any interpretation of the site conditions. The stratigraphic boundaries shown on Drawing 2, therefore, represent transitions between soil types rather than exact planes of geological change. Furthermore, subsurface conditions will vary between and beyond the borehole locations.

In general, the subsurface conditions encountered in the boreholes advanced through the construction access road and in the vicinity of the proposed retaining wall consist of fill comprised of granular material overlying cohesive material, underlain by clayey silt with sand, which is in turn underlain by shale bedrock. The subsurface conditions encountered in the borehole advanced at the toe of the embankment slope consists of a cohesive deposit consisting of sandy silty clay to clayey silt with sand overlying shale bedrock. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Fill

Fill was encountered at ground surface in Boreholes 19-1 and 19-2, advanced through the existing construction access road which is located on the north side of the QEW Niagara bound lanes. The fill extends to depths of 3.0 m and 5.0 m (Elevations 94.4 m and 94.7 m) in Boreholes 19-1 and 19-2, respectively. The depth and elevation of the surface and base of the fill and the corresponding overall thicknesses are summarized below.

Borehole No.	Surface of Layer		Base of Layer		Thickness (m)	Fill Type
	Depth (m)	Elevation (m)	Depth (m)	Elevation (m)		
19-1	0	97.4	0.6	96.8	0.6	Sand and Gravel
	0.6	96.8	3.0	94.4	2.4	Clayey Silt with sand
19-2	0	99.7	1.4	98.3	1.4	Sandy Gravel to Sand and Gravel
	1.4	98.3	5.0	94.7	3.6	Clayey Silt with sand

The Standard Penetration Test (SPT) “N”-values within the non-cohesive fill are 24 blows and 31 blows per 0.3 m of penetration, suggesting a compact to dense compactness condition. The SPT “N”-values within the cohesive fill range from 13 blows to 45 blows per 0.3 m of penetration, suggesting a stiff to hard (but generally very stiff) consistency.

The fill is variable in composition and is comprised of non-cohesive sandy gravel to sand and gravel, trace fines, and cohesive clayey silt with sand, trace to some gravel and trace organics. In Borehole 19-2, the fill contains wood fragments at a depth of about 0.8 m below ground surface.

Grain size distribution testing was carried out on two selected samples of the cohesive fill and the results are shown on Figure B-1 in Appendix B.

Atterberg limits testing was carried out on three samples of cohesive fill and measured liquid limits ranging between about 22 per cent and 26 per cent, plastic limits ranging between about 13 per cent and 15 per cent, and plasticity indices ranging between about 9 per cent and 11 per cent. These results, which are plotted on a plasticity chart on Figure B-2 in Appendix B, indicate that the cohesive fill deposit consists of clayey silt of low plasticity.

The water contents measured on two samples of the non-cohesive fill are approximately 8 per cent and 10 per cent, and the water contents measured on three samples of the cohesive fill range from about 10 per cent to 12 per cent. Two organic content tests conducted on samples of the cohesive fill measured organic contents of approximately 2 per cent and 3 per cent.

4.2.2 Clayey Silt with Sand to Sandy Silty Clay

A 1.1 m to 1.8 m thick cohesive deposit consisting of clayey silt with sand, trace to some gravel, trace organics was encountered underlying the cohesive fill in Boreholes 19-1 and 19-2 and at ground surface in Borehole 19-3. The depth and elevation of the surface and base of the cohesive deposit, and the corresponding thicknesses are summarized below.

Borehole No.	Surface of Deposit		Base of Deposit		Thickness (m)
	Depth (m)	Elevation (m)	Depth (m)	Elevation (m)	
19-1	3.0	94.4	4.1	93.3	1.1
19-2	5.0	94.7	6.1	93.6	1.1
19-3	0	95.5	1.8	93.7	1.8

The SPT "N"-values measured in the clayey silt to sandy silty clay deposit generally range from 13 blows to 16 blows per 0.3 m of penetration, suggesting a stiff consistency. However, one SPT "N"-value of 1 blow per 0.3 m of penetration was recorded in the sandy silty clay deposit at ground surface in Borehole 19-3, suggesting a very soft consistency. A SPT "N"-value of 130 blows per 0.3 m of penetration was recorded at the bottom of the clayey silt with sand deposit in Borehole 19-1; however, it is noted on the borehole record that shale fragments were observed in the spit-spoon and the high SPT "N"-value is considered to reflect the presence of such shale fragments and contact with the inferred shale bedrock.

Grain size distribution testing was carried out on two selected samples of the cohesive deposit and the results are shown on Figure B-3 in Appendix B. Atterberg limits testing was carried out on three samples of the cohesive deposit and measured liquid limits ranging from about 25 per cent to 27 per cent, plastic limits ranging from about 18 per cent to 21 per cent, and plasticity indices ranging from about 5 per cent to 9 per cent. These results which are plotted on a plasticity chart on Figure B-4 in Appendix B, indicate that the cohesive deposit consists of a clayey silt of low plasticity. The natural water content measured on four samples of the cohesive deposit are between about 15 per cent to 22 per cent.

4.2.3 Inferred Shale Bedrock / Refusal

Bedrock was inferred from split-spoon refusal or auger refusal and samples were recovered in the split-spoon in all boreholes advanced for the proposed retaining wall. The depth to inferred bedrock or refusal below ground surface and the corresponding inferred bedrock surface or refusal elevation are summarized below.

Borehole No.	Inferred Depth to Bedrock Surface / Refusal (m)	Inferred Bedrock Surface / Refusal Elevation (m)	Comments
19-1	4.1	93.3	0.5 m split-spoon sample penetration; split-spoon and auger refusal on inferred bedrock
19-2	6.1	93.6	0.1 m split-spoon sample penetration; split-spoon refusal on inferred bedrock
19-3	1.8	93.7	0.2 m split-spoon sample penetration; split-spoon refusal on inferred bedrock

Based on a review of bedrock core samples obtained from previous geotechnical investigations carried out by Golder on the west side of Etobicoke Creek, the bedrock is anticipated to consist of shale of the Georgian Bay Formation.

4.2.4 Groundwater Conditions

Groundwater levels in the open boreholes were measured upon completion of drilling operations. Water level observations are described in the table below.

Borehole No.	Depth to Water Level (m)	Water Level Elevation (m)	Comments
19-1	3.3	94.1	Upon completion of drilling, in open borehole
19-2	Dry	-	Upon completion of drilling and removal of augers
19-3	Ground Surface	95.5	Upon completion of split-spoon sampling, in open borehole

During and following precipitation events, perched groundwater conditions may be present in the fill layers. Additionally, 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.

4.2.5 Analytical Testing Results

Two selected soil samples were submitted for analysis to Bureau Veritas (formerly Maxxam), a Standards Council of Canada (SCC) accredited laboratory, of Mississauga, Ontario, for chemical analysis of the of parameters used to assess the potential corrosivity of the site soil to steel and concrete. The Bureau Veritas test report is provided in Appendix B, and summarized below:

Parameter	Borehole 19-2 Sample 5 Elev. 96.3 m	Borehole 19-3 Sample 2 Elev. 94.6 m
pH	7.71	7.82
Resistivity (ohm-cm)	2,800	2,000
Electrical Conductivity (umho/cm)	355	494
Chlorides (ug/g)	90	130
Soluble Sulphates (ug/g)	<20	160

5.0 CLOSURE

This Foundation Investigation Report was prepared by Ms. Alysha Kobylinski, E.I.T. and reviewed by Ms. Sandra McGaghran, M.Eng., P.Eng., an Associate and Senior Geotechnical Engineer with Golder. Mr. Paul Dittrich, P.Eng., a MTO Foundations Designated Contact and Principal with Golder, conducted a technical and quality control review of the report.

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PART B

FOUNDATION DESIGN REPORT
RETAINING WALL FROM STATION 14+072 TO 14+087,
QEW IMPROVEMENTS FROM EAST OF CAWTHRA ROAD TO THE EAST MALL, CITIES OF MISSISSAUGA AND ETOBICOKE
MINISTRY OF TRANSPORTATION, ONTARIO
G.W.P. 2432-13-00

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides foundation engineering recommendations for the proposed retaining wall to be constructed adjacent to an existing sanitary sewer maintenance hole on the north side of the Queen Elizabeth Way (QEW) and east of Etobicoke Creek, between Stations 14+072 and 14+087. This retaining wall is required for the QEW widening near The West Mall on-ramp, as part of the QEW improvements from east of Cawthra Road to The East Mall, in the City of Mississauga and the City of Etobicoke, Ontario. These recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation. The discussion and recommendations presented are intended to provide the designer with sufficient information to assess the feasible retaining wall foundation alternatives and carry out the design of the retaining wall foundations, and to provide the designers with sufficient information to assess the feasible roadway protection system alternatives where required, develop construction cost estimates, and identify items or issues to be addressed in the Contract Documents.

The foundation investigation report, discussion and recommendations are intended for the use of the MTO and their designers for G.W.P 2432-13-00, and shall not be used or relied upon for any other purpose or by any other parties, including the construction contractor or design-build contractor. The contractor must make their own interpretation based on the factual data in Part A (Foundation Investigation) of the report. Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project and for which special provisions may be required in the Contract Documents. Those requiring information on the aspects of construction must make their own interpretation of the factual information provided, as such interpretation may affect equipment selection, proposed construction methods, scheduling, and the like.

6.1 General

The General Arrangement (GA) drawing provided by AECOM on October 23, 2019 indicates that the portion of the proposed retaining wall that will be parallel to the QEW will have a length of about 6 m with wing walls about 6 m to 7 m in length, extending outward and downwards to retain the proposed widened QEW. The GA drawing also indicates that in the vicinity of the proposed retaining wall between about Stations 14+072 and 14+087, the final ground surface along the widened QEW will be at about Elevation 100 m. The top of the existing sanitary maintenance hole at the toe of the existing embankment slope is at about Elevation 96.4 m and the proposed grade at the base of the retaining wall will be at about Elevation 97 m. Therefore, an approximately 3 m high retaining wall will be required to retain the widened highway embankment above the existing sanitary maintenance hole. The existing ground surface downslope of the maintenance hole is inclined at about 3 horizontal to 1 vertical (3H:1V) to about Elevation 95.5 m and it is understood that the ground surface is to remain at this inclination following construction of the retaining wall. During construction of the retaining wall it is understood that the staged traffic on the QEW will be at a distance of approximately 20 m south of the proposed retaining wall, therefore, temporary protection systems may not be required for the construction of the retaining wall.

AECOM provided Golder on October 23, 2019 a drawing titled, “Etobicoke Creek Sanitary Trunk Sewer, from Valley Park to Evans Avenue, Construction Details” Drawing No 1239-R-7 dated October 1962, which presents the design details for the existing maintenance hole and associated sanitary sewer at the site. The drawing indicates that the sanitary sewer was installed using trenchless methods from the maintenance hole southwards toward and under the QEW and in an open cut northwards away from the maintenance hole along the sewer alignment. Based on the limited information shown on this drawing, we have inferred for the purposes of our analysis that the existing maintenance hole would have been installed in an open cut excavation with near vertical side slopes within the bedrock and with cut slopes at an inclination of 1 horizontal to 1 vertical (1H:1V) through the overburden. Based on this assumed geometry, the proposed retaining wall alignment would be located beyond the edges of the open cut excavation into bedrock and partially overlying the backfill associated with the maintenance hole construction. We have also assumed that the excavation for the maintenance hole construction would have been backfilled with compacted native clayey silt fill.

6.2 Foundations Options for Retaining Wall

Based on the generally stiff to very stiff fill and native soils encountered in the boreholes advanced in the vicinity of the proposed retaining wall alignment, the following wall types and reinforced earth options are considered practicable, constructible and appropriate at this site from a geotechnical/foundations perspective:

- **Concrete Cantilever Retaining Wall on Shallow Foundations:** A concrete cantilever retaining wall supported on a shallow foundation is geotechnically feasible for the proposed retaining structure. Due to the expected traffic staging at the time of construction of the retaining wall, a temporary protection system may not be required, and construction could be carried out within an open cut excavation. Excavations to the recommended founding stratum (below the existing fill) would be on the order of about 2.5 m deep relative to the ground surface in front of the wall and about 3 m to 5 m deep below the grade of the construction access road at the time of this investigation. The use of deep foundations for support of a concrete retaining wall option is not required at this site as adequate bearing resistance and settlement performance can be achieved on the native soils, which are underlain by relatively shallow bedrock, with the use of a shallow foundation.

- **Reinforced Soil System (RSS) Wall:** An RSS wall is considered feasible from a foundations perspective at this location and we understand that it is the preferred option for the proposed retaining structure. Typically, the zone of the reinforced soil mass extends a distance behind the wall face equal to about 80 per cent of the wall height and therefore results in a smaller excavation footprint; however, as discussed in Section 6.9.2 the reinforced soil mass at this location is required to extend a distance of about 1.5 times the wall height to satisfy global slope stability requirements. This is a disadvantage in comparison to the excavation footprint for the concrete cantilever retaining wall option; however, the advantage of the RSS wall is that the depth of excavation will be less in comparison to the concrete cantilever retaining wall option. Due to the expected staging at the time of construction of the retaining wall, a temporary protection system may not be required, and construction could be carried out within an open cut excavation. It should be noted that, as discussed in Section 6.9.2, the Regional storm event is predicted to have a high water level at the site which would be above the ground surface in front of the proposed RSS wall; given this, project specific approval from MTO’s RSS Committee will be required for the use of an RSS wall at this location.

- **Soldier Pile and Concrete Panel Wall:** A soldier pile and concrete panel wall could be considered for the proposed retaining structure. This type of wall is generally more advantageous in “top-down” construction applications (i.e., as part of a cut, rather than new fill construction). However, this option would minimize excavation into the existing north slope of the QEW embankment, as would be required for the above two options, and would allow the widening embankment fill placement to proceed above the existing embankment behind the proposed wall alignment. The location of the existing maintenance hole is visible; however, it is recommended that if this option is adopted that the contractor obtain the as-constructed location of the existing sanitary sewer by detailed survey, since advancing piles into the bedrock in this area for permanent support of a soldier pile wall poses the risk of damaging the existing underlying utility.

- **Reinforced Earth Slope:** A 1H:1V or steeper reinforced earth slope could be considered at this location instead of a retaining wall. The excavation footprint for the reinforced earth slope mass would require up to an additional 3 m in width compared to the first two options discussed above, in order to accommodate a slope inclined at 1H:1V. A temporary protection system may not be required as we understand that the expected staging at the time of construction could be carried out within an open cut excavation. However, it may be challenging to establish vegetation on a steepened, north-facing slope, rendering this option less desirable from an aesthetic and long-term maintenance perspective.

The feasibility, advantages and disadvantages for the various retaining wall and reinforced earth options are summarized in Table 1 following the text of this report. From a geotechnical/foundations perspective, the use of an RSS wall is the preferred option based on tolerance to settlement, minimization of excavation into the existing embankment side slope relative to the concrete retaining wall option, and cost. The following sections of this report provide geotechnical recommendations for the feasible options.

6.3 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the 2014 *Canadian Highway Bridge Design Code* and its *Commentary* (CHBDC 2014), the proposed retaining wall 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.

6.4 Seismic Design

6.4.1 Seismic Site Classification

Subsurface ground conditions for the proposed retaining wall characterization were established based on the results of the field investigation and laboratory testing. The SPT "N"-values measured in the soil layers were used to evaluate the seismic site classification in accordance with Table 4.1 of the CHBDC (2014). Based on this method it is considered that Site Class C is applicable for the design of the replacement retaining wall structure.

6.4.2 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 based on the National Resource Canada (NRC) website 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	0.224
Sa (0.5) (g)	0.042	0.067	0.117

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)
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

6.5 Concrete Retaining Wall on Shallow Foundations

6.5.1 Founding Elevations

Shallow footings must be founded at a minimum depth of 1.2 m below the lowest surrounding final grade to provide adequate protection against frost penetration (per OPSD 3090.101 – *Foundation Frost Depths for Southern Ontario)*. For support of a concrete retaining wall, footings must be founded below any existing fill and/or softened, disturbed or soft/firm soils, and on the native stiff clayey silt with sand. Based on the borehole information, a founding Elevation of 94.3 m or lower is recommended; alternatively, the foundation footprint may be subexcavated to the elevation identified above and backfilled with engineered granular fill to a higher elevation that still satisfies the frost protection requirements.

The footing subgrade should be inspected by an experienced geotechnical engineer following excavation, in accordance with OPSS 902 *(Excavating and Backfilling Structures)* as amended by SP 109S12, to check that all existing fill and soft/firm soils and soils containing organics have been removed. Any softened/disturbed or otherwise deleterious materials should be further subexcavated and backfilled with compacted Granular A or Granular B Type II soils meeting the requirements set out in OPSS 1010 *(Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material)*, that is placed and compacted in accordance with OPSS.PROV 501 (*Compacting*), as amended by Special Provision SP105S10 *(Amendment to OPSS 501 – Construction).*

The footing subgrade will be susceptible to disturbance on exposure to construction activities and/or ponded water. If the concrete for the retaining wall footing cannot be poured immediately after excavation and inspection, it is recommended that a 100 mm thick concrete working slab be placed on the subgrade within four hours after its inspection and approval, to protect the integrity of the subgrade. A Non-Standard Special Provision (NSSP) to address this item is included in Appendix C, and this should be included in the Contract Documents if this wall/foundation type is adopted.

6.5.2 Geotechnical Resistances

Strip footings placed on the properly prepared subgrade, at or below the design elevation given in the preceding section, or on engineered granular fill following subexcavation to the elevation given in the preceding section, should be designed based on a factored ultimate geotechnical resistance of 450 kPa and factored serviceability geotechnical resistance (for 25 mm of settlement) of 200 kPa for a footing width up to about 2 m.

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).

6.5.3 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the new concrete footing and the subgrade should be calculated in accordance with Section 6.10.5 of the CHBDC (2014). For cast-in-place concrete footings constructed on the stiff clayey silt with sand, the coefficient of friction, tan φ', can be taken as 0.35.

6.6 Retained Soil System (RSS) Walls

The RSS wall should be designed in accordance with MTO Special Provision (SP) 599S22 (*Retained Soil System*) for the performance and appearance conditions as specified by the MTO.

As this wall will be constructed partially by cutting into the existing north embankment side slope, it is recommended that the back of the reinforced soil mass be keyed into the existing embankment by benching into the existing fill, as per OPSD 208.010 (*Benching of Earth Slopes*).

6.6.1 Founding Elevations

A typical RSS wall has a front facing panel system that is supported on a strip footing placed at a shallow depth below the ground surface in front of the wall. The 200 mm thick facing footing should be placed within a 500 mm thick levelling pad comprised of OPSS.PROV 1010 (*Aggregates*) Granular A, placed in accordance with OPSS.PROV 501 (*Compacting*), as amended by Special Provision SP105S10 *(Amendment to OPSS 501 – Construction)* and as detailed on Figure 5.2 of MTO's *RSS Wall Design Guidelines* (September 2008). In accordance with this guideline, there should be a minimum 300 mm thickness of Granular A below the facing footing; the compacted granular levelling pad is not required below the reinforced soil mass. The compacted granular levelling pad should extend horizontally at least 1.0 m beyond the outside and inside edges of the facing footing, then downward and outward at 1H:1V. The exception to this is for the portion of the RSS wall located directly south of the maintenance hole where the levelling pad should extend beyond the outer edge of the facing footing and abut the maintenance hole; the 1H:1V slope on the front of the granular pad is not required at this location. However, in order to reduce the impact of horizontal loads from the RSS wall on the existing manhole chamber, we recommend that a void form or a minimum 50 mm (2") thickness of expanded polystyrene be installed between the levelling pad backfill and the maintenance hole chamber.

As detailed on Figure 5.22 of MTO's *RSS Wall Design Guidelines*, it is recommended that the underside of the levelling pad be founded at a minimum depth of 1.0 m below the finished grade at the front base of the RSS wall. The minimum soil cover below the finished grade in front of the base of the RSS wall and over the top of the facing footing should be 0.5 m. Prior to placement of the levelling pad and the reinforced soil mass, any existing topsoil and organics or other deleterious materials must be removed and the subgrade is required to be proof-rolled to identify any softened/disturbed areas that will then require sub-excavation and replacement with compacted OPSS.PROV 1010 (*Aggregates*) Granular A, placed in accordance with OPSS.PROV 501 (*Compacting*) as amended by Special Provision SP105S10 *(Amendment to OPSS 501 – Construction)*.

Based on the borehole information and the proposed ground surface at the base of the proposed RSS wall, the top of the Granular A levelling pad/facing footing (and bottom of the reinforced soil mass) are recommended to be founded no higher than Elevation 96.0 m and at least 0.5 m below the final finished grade along all sections (i.e. central and wings) of the RSS wall.

6.6.2 Geotechnical Resistance

As discussed in Section 6.1, AECOM provided Golder on October 23, 2019, a drawing titled, "Etobicoke Creek Sanitary Trunk Sewer, from Valley Park to Evans Avenue, Construction Details" Drawing No 1239-R-7 dated

October 1962, which presents the design details for the existing maintenance hole at the site. Based on this limited information we have inferred for the purposes of our analysis that the existing maintenance hole would have been installed in an open cut excavation with near vertical side slopes within the bedrock and with cut slopes at an inclination of 1H:1V through the overburden. We have also assumed that the excavation would have been backfilled with compacted native clayey silt fill.

Assuming that the RSS wall acts as a unit and distributes load across the full width of the reinforced soil mass (described below), the factored ultimate geotechnical resistance and factored serviceability geotechnical resistances given in the table below can be used for design of the retained soil system founded on the properly prepared compacted granular fill (i.e. levelling pad below the facing footing) and/or on the proof-rolled and inspected clayey silt fill subgrade at or below the sub-excavation elevation given above. Given that this RSS wall will not be constructed in close proximity to a bridge or other structure, higher settlement tolerances may be considered acceptable by the designers in the Serviceability Limit States (SLS) design condition. As such, the factored serviceability geotechnical resistance for 25 mm and 50 mm of settlement are provided. The proprietary designer of the RSS wall should confirm that the structure can tolerate the settlements indicated below and the wall designed accordingly.

Alternatively, if the resistances provided below for the RSS wall and levelling pad founded directly on the clayey silt fill subgrade are not sufficient to support the RSS wall, the RSS wall and the reinforced soil mass could be constructed on a 1 m thick OPSS.PROV 1010 (*Aggregates*) compacted Granular A pad and the factored ultimate geotechnical resistance and factored serviceability geotechnical resistances given below can be used for design.

Subgrade Soil	Factored Ultimate Geotechnical Resistance	Factored Serviceability Geotechnical Resistance (for 25 mm of settlement)	Factored Serviceability Geotechnical Resistance (for 50 mm of settlement)
Existing clayey silt fill	175 kPa	75 kPa	150 kPa
1 m thick compacted Granular A pad over existing clayey silt fill	225 kPa	150 kPa	300 kPa

Note(s): The geotechnical resistances provided above are based on a RSS wall (including the reinforced soil zone) having a total width of 4.5 m. If a width other than 4.5 m is adopted, the geotechnical resistances should be re-evaluated.

The 1 m thick compacted granular pad should be constructed similar to the granular levelling pad as described in Section 6.6.1.

6.6.3 Resistance to Lateral Loads / Sliding Resistance

The resistance to lateral forces / sliding resistance between the compacted granular fill of the RSS wall and the subgrade should be calculated in accordance with Section 6.10.5 of the CHBDC (2014). The coefficient of friction, tan φ’, between the compacted granular fill of the RSS wall and the properly prepared subgrade may be taken as 0.53. The actual values used should be reviewed and revised, if necessary, by the proprietary RSS wall designer during detail design.

6.7 Soldier Pile and Concrete Panel Wall

A soldier pile and concrete panel wall could be adopted at this site and would minimize excavation into the existing north slope of the QEW embankment as compared with the concrete cantilever retaining wall or RSS wall options. This wall system would consist of soldier piles socketed into bedrock to sufficient depth to provide the necessary passive resistance for the maximum retained soil height. Due to the height of the fill slope above the top of the wall, if additional lateral support to the soldier pile and concrete panel wall system is required, this could be provided by the installation of anchors. A conventional drilled and grouted anchor system would likely necessitate the anchor bond zone extending into the shale bedrock. A deadman anchor system may also be feasible, although the deadmen would likely need to be installed below the travelled lanes of the QEW, and the top portion of the deadmen would need to be designed to avoid interfering with the roadway pavement structure.

The concrete panels would have to be installed such that the unsupported height does not exceed 1.2 m at any time, and the space behind the concrete panels would have to be immediately packed with granular material to aid in achieving proper drainage. If sufficient thickness of free-draining granular soil is not provided behind the concrete panels to provide adequate drainage and frost protection, consideration should be given to using a drainage sheet. An insulation layer could also be provided immediately behind the wall to provide frost protection.

6.7.1 Passive Resistance for Soldier Pile Sockets

The ultimate passive lateral pressure in front of the soldier piles may be assessed using Brom's equation (1964) using the design parameters / values for the overburden soils as follows:

K_p the coefficient of passive earth pressure, which may be taken as 2.8 for the existing cohesive fill and 3.1 for the clayey silt deposits. This K_p value must be reduced by an appropriate factor that considers the allowable wall movement in accordance with Figure C6.16 of CHBDC (2014).

γ' the effective unit weight of the soil in front of the soldier pile socket, which may be taken as 10 kN/m^3 below the groundwater level (assumed to be at Elevation 95.5 m).

The upper 1.2 m of soil in front of the soldier piles should be ignored in the calculation of the passive resistance, to account for disturbance during installation, and for frost effects as interpreted from OPSD 3090.101 (*Frost Penetration Depths for Southern Ontario*).

Core samples of the shale bedrock were not recovered during the subsurface investigation for this retaining wall; however, based on nearby boreholes from this assignment drilled for the QEW / Etobicoke Creek replacement structure and proposed retaining wall west of Etobicoke Creek, a factored passive lateral resistance (f_{horiz}) for the portion of the soldier pile socketed into the near surface shale bedrock mass may be taken as 1.5 MPa.

6.7.2 Permanent Rock Anchors

If required, a rock anchor support system can be designed to accommodate the loads applied from lateral earth pressures and surcharge pressures from area, line or point loads and also take into account any sloping ground behind the retaining wall system. For design, the rock anchors may be sized based an unfactored bond stress acting between the grout and shale bedrock of 200 kPa.

In accordance with the CHBDC (2014), a factor of 0.4 should be applied to the unfactored bond stress value for ULS conditions. The SLS value for 25 mm of displacement will not govern; for design purposes an SLS value equal to the ULS value should be used.

The sustained working load should not be greater than 80 per cent of the ultimate tensile strength of the steel anchor tendons or bars. Rock tie-back anchors should have their fixed length (bond zone) formed within the shale bedrock and should be installed at a downwards angle of 20 degrees or steeper. A minimum of 4.5 m of overburden is typically required above the center of the fixed length (bond zone) to provide the necessary overburden pressure to develop anchor capacity in gravity-grouted anchors; to prevent grout leakage during installation of pressure grouted anchors and to prevent heaving of the ground surface for higher grout pressure operations (FHWA, 1999). The fixed length (bond zone) of the anchors should be at least 3 m (and may be up to 8 m) and should be maintained behind a line drawn upward at 45 degrees from the toe of the proposed wall. The horizontal spacing between anchors will be dependent of the spacing of the soldier piles but should be greater than four times the diameter of the anchor (grouted section) and at least 1.2 m. The permanent rock anchors should be provided with suitable corrosion protection.

Lateral earth pressures for design are discussed in Section 6.12. Anchor installation, grouting and testing should be carried out in accordance with OPSS 942 (*Pre-Stressed Soil and Rock Anchors*).

6.8 Reinforced Earth Slope

A reinforced earth slope could be considered with slopes oriented at 1H:1V or potentially steeper. This option could be less expensive than a vertical retaining structure solution; however, it may be challenging to establish vegetation on a steepened slope as the north-facing embankment slope may not receive sufficient sunlight during the day to promote growth of vegetation cover on the face of the slope, rendering this option less desirable from an aesthetic and long-term maintenance perspective. This could be addressed in the long-term by periodic maintenance, although it is a disadvantage that may preclude the use of reinforced earth slopes as a viable option for this site. If this option is adopted, it is recommended that an interceptor drain or swale be constructed along the crest of the slope to minimize surface water flow over the crest and slope face, and to reduce erosion potential on the reinforced slope face. The design of the reinforced earth slope would be up to the proprietary supplier of the system, subject to the global stability considerations as outlined in Section 6.9.4.

Prior to placement of the engineered fill, the existing topsoil must be removed and the existing fill/reworked soil is required to be proof-rolled. The reinforced soil mass should be keyed into the existing embankment by benching into the embankment fill, as per OPSD 208.010 (*Benching of Earth Slopes*). Vegetation cover should be established on the slope face to protect against surficial erosion, as per OPSS 572 *(Seeding and Cover)*, if and where such vegetation is compatible with the selected, proprietary reinforced earth slope system. Appropriate treatment of the steepened slope face will be required to allow vegetation to become established and to maintain the vegetation cover.

6.9 Global Stability for Retaining Wall Options

Limit equilibrium slope stability analyses were performed for the various retaining wall options using the commercially available program SLIDE (version 8.018), produced by Rocscience Inc., employing the Morgenstern-Price method of analysis. For all analyses, the factor of safety of numerous potential failure surfaces was computed to establish the minimum Factor of Safety (FoS). The FoS is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. For the purpose of the stability analysis, the Factor of Safety is equal to the inverse of the product of the consequence factor, Ψ, and the geotechnical resistance factor, ϕ_{gu}. (i.e., $FoS = 1/(\Psi \cdot \phi_{gu})$). Typically a target minimum FoS of 1.33 is adopted for design of walls and embankment slopes during and immediately following construction (temporary condition), while a target minimum FoS of 1.54 is adopted for walls and embankment slopes under static conditions in the long-term (permanent condition), as per the CHBDC (2014); and a target minimum FoS of 1.1 is typically adopted for the design of walls and embankment slopes under seismic conditions, as per the CHBDC (2014). In general, circular slip surfaces were considered in the global stability analyses. These factors of safety are considered appropriate for the design of the proposed retaining wall at this site, considering the design requirements and the available borehole and laboratory test data.

The soil parameters used in the short-term (undrained, temporary condition) and long-term (drained, permanent condition) analyses, as given below, were estimated from empirical correlations using the results of in-situ Standard Penetration Tests (SPTs) (Bowles, 1984) and geotechnical index testing. The groundwater table was assumed to be at Elevation 95.5 m in the analyses.

Soil Deposit	Bulk Unit Weight (kN/m^3)	Effective Friction Angle	Undrained Shear Strength (kPa)
New earth fill or SSM	20	32°	-
Existing compact non-cohesive fill	20	33°	-
Existing stiff to hard cohesive fill	19	28°	90 and 65
Stiff to very stiff clayey silt with sand	19	31°	90

6.9.1 Concrete Retaining Wall

The results of the static global stability analyses indicate a minimum factor of safety of 1.5 is achieved for a 3 m high (exposed face) concrete retaining wall, founded at the elevation provided in Section 6.5.1, at this site in both short-term and long-term conditions. A seismic global stability analysis was also completed and indicates a minimum factor of safety of 1.1 is achieved for this wall height.

6.9.2 RSS Wall

The results of the static global stability analyses indicate that a minimum factor of safety of 1.5 is achieved in both the short-term (temporary) and long-term (permanent) conditions for an RSS wall of up to approximately 3 m in total height (H) above the finished ground surface at the front of the wall; however, the required ratio of reinforcing strip length to height of the RSS wall (%H) to achieve the minimum FoS for global stability is 1.5 (i.e. the length of the reinforcing strips must be at least 1.5 x 3 m = 4.5 m). If the RSS wall height is greater than 3 m, the required length of the reinforcing strips will need to be re-evaluated. The results of the static global stability analysis for the RSS wall in the short-term and long-term conditions are shown on Figures 1 and 2, respectively.

AECOM have indicated that the 100-year storm event and Regional storm event would have water levels at Elevations 95.76 m and 98.6 m, respectively at the site. The ground surface at the proposed toe of the RSS wall will be at approximately Elevation 97.0 m; therefore, the Regional storm event water level would be above the ground surface in front of the RSS wall. As such, project specific approval from MTO's RSS Committee will be required for the use of an RSS wall at this location.

A seismic global stability analysis was also completed for non-flooded conditions and indicates a minimum factor of safety of 1.1 is achieved for this wall/slope geometry and reinforcement ratio under the design earthquake loading.

The RSS wall is to be designed, and the internal stability assessed, by the proprietary product designer. This design and assessment should include a check of all potential failure mechanisms associated with the reinforced soil mass, the facing elements and the associated connections. Extending the reinforcement zone beyond the minimum lengths identified above may be required to satisfy internal stability requirements.

6.9.3 Soldier Pile and Concrete Panel Wall

If a soldier pile and concrete panel wall option is adopted for this site, the global stability analysis will be assessed once the structural design has established the soldier pile embedment depths and the details of the dimensions and lateral restraint provided by rock anchors are made available.

6.9.4 Reinforced Earth Slope

The results of the static global stability analysis for the reinforced earth slope indicate that a factor of safety greater than 1.5 is achieved against deep-seated instability in both short- and long-term conditions for a 1H:1V reinforced earth slope up to approximately 3 m high. This analysis assumes the reinforcing strips have a minimum width of up to 5.5 m (i.e., 1.8 times the slope height), and that the reinforced soil mass has been designed by the proprietary designer to avoid failure through/within the reinforced soil mass. A seismic global stability analysis was also completed and indicates a minimum factor of safety of 1.1.

6.10 Settlement of Widened Embankment/Retaining Wall

The northward widening of the QEW will result in the placement of up to about 3 m (height of wedge of soil along existing slope and behind the proposed retaining wall alignment) of embankment fill material on top of the existing embankment side slope, and this new fill will induce some settlement of the existing embankment fill soils as well as the underlying foundation soils above the bedrock.

The settlement analysis for the embankment widening and retaining wall was carried out using the commercially available program Settle-3D from Rocscience (Version 4.023), using estimated elastic deformation moduli as given in the table below, based on correlations with SPT “N” values and engineering judgement from experience with similar soils in this region of Ontario.

Soil Deposit	Bulk Unit Weight	Elastic Modulus
New Embankment Fill	21 kN/m^3	--
Existing non-cohesive fill	20 kN/m^3	15 MPa
Existing cohesive fill	19 kN/m^3	10 MPa
Clayey silt with sand	19 kN/m^3	15 MPa

The settlement performance criterion for design of embankments within the high fill areas is outlined in MTO’s Guideline titled, “Embankment Settlement Criteria for Design”, dated July 2010. In general, widened embankments not approaching a structural element are to be designed as follows:

- Total settlements and differential settlement rates are to be less than 50 mm and 200:1, respectively, over a 20-year period following completion of construction for a King’s highways.

The settlement of the foundation soils under the approximately 3 m thick wedge of additional fill on the existing slope face, that will be placed for the widened highway embankment, is estimated to be less than 25 mm for an RSS wall, and less than 10 mm for retaining wall options founded below the existing fill soils; these estimated settlement magnitudes are less than the criteria referenced above.

6.11 Scour Protection and Erosion Control

Requirements for scour protection in front of the toe of the wall should be assessed and confirmed by the hydraulic designer, taking into consideration the 100-year Design Storm and Regional Storm levels. However, from a geotechnical perspective, it is recommended that consideration be given to placing rip-rap on the slope toe/face in front of the wall facing panels if the toe of the wall is at or below the 100 year storm levels. As discussed in Section 6.9.2 the predicted elevation of the Regional storm event is above the base of the RSS wall and as such project specific approval from MTO's RSS Committee will be required for the use of an RSS wall at this location.

6.12 Lateral Earth Pressures for Design of Retaining Walls

The lateral earth pressures acting on the retaining wall 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 and traffic loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the wall. Seismic (earthquake) loading must also be taken into account in the design. The following recommendations are made concerning the design of the retaining wall:

- 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 wall. 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*) as amended by Special Provision SP105S10 *(Amendment to OPSS 501 – Construction)*. Other aspects of the granular backfill requirements with respect to subdrains and frost taper should be in accordance with OPSD 3121.150 (*Walls, Retaining, Backfill, Minimum Granular Requirement*), and OPSD 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. Hand operated compaction equipment should be used to compact the backfill soils immediately behind the walls as per OPSS.PROV 501(*Compacting*) as amended by Special Provision SP105S10 *(Amendment to OPSS 501 – Construction)*. 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, per 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, per Figure C6.20(b) of the Commentary to the CHBDC (2014).
- Where space is restricted and the walls are constructed in a top-down fashion with only a thin zone of granular backfill behind the wall or in cases where it is not possible to place granular behind the wall, it is recommended that drainage measures (e.g., pre-fabricated sheets) be incorporated on the back of the walls, before or concurrent with the panel installation, to promote drainage and minimize the risk of frost action during freezing temperatures. The wall system and facing should also incorporate subdrains and weep holes at intervals through the wall face.

6.12.1 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. The coefficients of static lateral earth pressure must be adjusted to take account of the slope above the wall, if applicable.

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

Fill Type	Unit Weight of Material	Coefficients of Static Lateral Earth Pressure	
		At-Rest, K_o	Active, K_a
Granular 'A'	22 kN/m³	0.43	0.27
Granular 'B' Type II	21 kN/m³	0.43	0.27

- For a soldier pile and concrete panel wall, or for a restrained wall, the pressures are based on the existing and proposed embankment fill, and the following parameters (unfactored) may be used assuming the use of earth fill (behind the granular zone where applicable):

Fill Type	Unit Weight of Material	Coefficients of Static Lateral Earth Pressure	
		At-Rest, K_o	Active, K_a
Earth Fill	20 kN/m³	0.53	0.36

- If the retaining wall structure allows for lateral yielding, active earth pressures should be used in the geotechnical design. The movement required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.12.1 and Table C6.6 of the *Commentary to the CHBDC* (2014).
- If the retaining wall structure does not allow lateral yielding (i.e., restrained structure where the rotational or horizontal movement is not sufficient to mobilize and active earth pressure condition), at-rest earth pressures (plus any surcharge) should be assumed for geotechnical design.

6.12.2 Seismic Lateral Earth Pressures for Design

Seismic (earthquake) loading must also be taken into account in the design of retaining 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 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 that 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 mm, 19 mm, 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 per Section C4.6.5 of the *Commentary* to *CHBDC* (2014).

6.13 Assessment of the Effect of the Retaining Wall on the Existing Sanitary Sewer

As discussed in Section 6.1, AECOM provided Golder on October 23, 2019 a drawing titled, “Etobicoke Creek Sanitary Trunk Sewer, from Valley Park to Evans Avenue, Construction Details” Drawing No 1239-R-7 dated October 1962. According to the drawing, the section of the sanitary sewer running south of the maintenance hole (and under the QEW) which will be underlying the newly constructed retaining wall has a diameter of about 1.37 m, with an obvert at about Elevation 91 m. Based on the borehole information from the current investigation, the inferred bedrock surface was encountered at elevations ranging from about 93.7 m to 93.3 m. Therefore, based on this information and the as-constructed drawings, the obvert of the sanitary sewer is between about 2.3 m and 2.7 m below the inferred bedrock surface at the proposed wall location. Further, it is understood that this portion of the sanitary sewer was installed using trenchless (i.e. tunnelling) methods.

Based on the above, it is considered that construction of the retaining wall at this location and the resulting new loading associated with the placement of up to about a 3 m high wedge of new fill on the existing slope face will have a negligible effect on the majority of the existing sanitary sewer which will be directly underlying the new wall. However, for the portion of the sanitary sewer immediately adjacent to the maintenance hole (i.e. at the connection) that would have most likely been constructed using open-cut methods, the construction of the new retaining wall and the additional loading associated with the new fill placement will apply additional loading onto this portion of the sewer and on the maintenance hole structure. Preliminary analysis indicates that an additional horizontal load of up to 70 kPa will be applied to the maintenance hole riser structure and an additional vertical load of up to 20 kPa will be applied to the portion of the sewer pipe constructed within the open-cut section. If these loads cannot be accommodated by the existing sanitary sewer and maintenance hole structure, additional information (i.e. a more detailed geometry of the extent of the open-cut section between the maintenance hole and the proposed retaining wall) will be required and a more detailed analysis will have to be carried out.

6.14 Analytical Testing for Construction Material

Soil corrosivity may affect the concrete or steel elements buried in the soil. Generally, the corrosivity of a structure depends on the soil resistivity, hydrogen ion concentration, salts (chloride and sulphate) concentrations and redox potential. The results of analytical test on two soil samples from Boreholes 19-2 and 19-3 are summarized in Section 4.2.5 and the analytical testing reports are presented in Appendix B. The potential for sulphate attack and corrosion are discussed in the following sub-sections. However, it is ultimately up to the designer to determine the appropriate construction materials for all elements of the retaining wall, including the exposure class and ensuring that all aspects of CSA A23.1-14 Section 4.1.1 “Durability Requirements” are followed when designing concrete elements.

6.14.1 Potential for Sulphate Attack

The analytical test results were compared to CSA A23.1 14 Table 3 ("Additional requirements for concrete subjected to sulphate attack”) for the potential sulphate attack on concrete. The sulphate concentrations measured in all samples of the native soils range from less than 0.002 per cent to about 0.016 per cent, which are below the exposure class of “S-3” (Moderate - 0.1 – 0.2 per cent) and the sulphate concentrations are considered negligible according to the Gravity Pipe Design Guidelines Table 7.2 (MTO, 2014). Therefore, based on the samples tested, when the designer is selecting the exposure class for the structure, the effects of sulphates from within the native soil deposits on concrete elements may not need to be considered.

6.14.2 Potential for Corrosion

Based on the test results from the soil samples the pH are about 7.71 and 7.82 and the resistivity are 2,000 and 2,800 ohm-cm. According to the MTO Gravity Pipe Design Guidelines (2014), the pH is not considered detrimental to concrete durability. The soil corrosiveness is generally moderate (2,000 ohm-cm < R < 4,500 ohm-cm), as per Table 3.2 of the MTO Gravity Pipe Design Guidelines (2014). As the concrete retaining wall will be located under and adjacent to the roadway / highway shoulders and will be exposed to de-icing salt, concrete should be designed for a “C” type exposure class as defined by CSA A23.1-14 Table 1. The concrete elements should be designed with consideration given to Table 7.1 of the MTO Gravity Pipe Design Guidelines (2014).

6.15 Construction Considerations

6.15.1 Open-Cut Excavation

Open-cut excavations must be carried out in accordance with the guidelines outlined in the Ontario Occupational Health and Safety Act (OHSA) for Construction Activities. The excavations for a concrete retaining wall on a strip footing, RSS wall, or reinforced earth slope construction will extend through existing fill that may contain zones of water-bearing soils. The existing fill materials are classified as Type 3 soil according to the soil types defined by

OHSA. If and where space permits, temporary excavations (i.e. those which are open for a relatively short time period) should be made with side slopes no steeper than 1H:1V. Where insufficient space is available to accommodate this slope, and to protect both the workers and the travelled lanes of the QEW during construction staging, a temporary protection system may be required.

6.15.2 Temporary Protection Systems

Where a temporary protection system is required, it should be designed and constructed in accordance with OPSS.PROV 539 *(Temporary Protection Systems),* as amended by SP 105S09. The lateral movement of the protection system should meet Performance Level 2 as specified in OPSS.PROV 539 *(Temporary Protection Systems)*.

The Standard Penetration Test (SPT) “N”-values in the existing fill and clayey silt in the vicinity of the proposed retaining wall are generally less than about 20 blows per 0.3 m of penetration. While the installation of driven sheet piles is feasible in the less stiff materials, some challenges may be encountered due to the presence of relatively shallow bedrock, which may impact the feasibility of a driven sheet-pile wall at this site. As a result, the contractor may elect to use a soldier pile and lagging system.

The sheet piles or soldier piles will need to extend/be socketed to a sufficient depth to provide the necessary passive resistance for the retained soil height, plus any surcharge and traffic loads behind the protection system. Lateral support to the sheet pile wall or soldier pile wall could be provided in the form of rakers or temporary anchors, if and as required.

While the selection and design of the temporary protection system will be the responsibility of the Contractor, the following information is provided to MTO and its designers to aid in assessment of the approximate construction costs during detail design.

Soil Type	Unit Weight	Internal Angle of Friction	Undrained Shear Strength	Coefficient of Lateral Earth Pressure[1]		
	(γ, kN/m^3)	(ϕ, degrees)	(S_u, kPa)	Active K_a	At Rest K_o	Passive K_p [2]
New earth fill or SSM	20	32°	-	0.31	0.47	3.25
Existing non-cohesive fill	20	33°	-	0.29	0.46	3.39
Existing cohesive fill	19	28°	65	0.36	0.53	2.77
Clayey silt with sand	19	31°	90	0.32	0.48	3.12

Notes:

1. The earth pressure coefficients noted above are based on a horizontal surface adjacent to the excavation. If sloped surfaces are present, the coefficient of earth pressure should be adjusted accordingly.
2. The total passive resistance below the base of the excavation (i.e. adjacent to the temporary protection system) may be calculated based on the values of K_p indicated above, but reduced by an appropriate factor that considers the allowable wall movement in accordance with Figure C6.16 of the CHBDC (2014) to account for the fact that a large strain would be required for mobilization of the full passive resistance.

It should be noted that the pressure distributions calculated based on the parameters given above will be the minimum for the ultimate stress condition; a stiffer design may be required than predicted by these distributions in order to maintain displacements within an acceptable range.

For the temporary protection system behind the retaining wall, a design ground water level of Elevation 95.5 m should be assumed. If groundwater is encountered, it will be necessary to control seepage or include measures to mitigate loss of soil particles through lagging boards.

Consideration should be given to either partial or full removal of the protection system upon completion of construction or each stage of construction (as required). Where possible, full removal of the protection system should be considered to mitigate potential impediments to future rehabilitation or reconstruction work. An NSSP is included in Appendix C which addresses the removal or cut-off of the protection system.

6.15.3 Obstructions During Installation of Temporary Protection Systems

It is anticipated that pieces of wood, construction debris, cobbles and/or boulders may be encountered within the existing fill and clayey silt with sand deposits, which may affect the installation of protection system elements. It is recommended that a Notice to Contractor be included in the Contract Documents to warn the Contractor of the possible presence of pieces of wood, construction debris, cobbles and/or boulders within the overburden soils; a Notice to Contractor is provided in Appendix C.

6.15.4 Vibration Monitoring During Temporary Protection System Installation

If the temporary protection systems are installed using vibratory methods, significant vibrations are not anticipated, given the generally stiff to very stiff nature of the native soil deposits; however, if the sheetpile is vibrated into the underlying bedrock, then this may result in increased vibrations. A maximum peak particle velocity (PPV) of 100 mm/s is generally considered applicable for bridge structures in good condition, and this criterion is expected to be applicable to the existing Etobicoke Creek bridge. Based on vibration monitoring experience, it is considered unlikely that vibrations associated with the wall construction will reach this threshold level at the existing Etobicoke Creek bridge.

Residential buildings are present in the vicinity of the site, at distances of approximately 150 m east of the proposed retaining wall. A lower PPV threshold of 25 mm/s to 50 mm/s is generally considered applicable for buildings. While it is expected that vibration levels will not reach these thresholds at the structures, it is recommended that pre- and post-construction condition surveys and vibration monitoring be undertaken at or near the buildings located within 200 m of the protection systems, to defend against potential damage claims. A sample NSSP has been provided in Appendix C, and this NSSP will be modified as required to address condition surveys and/or vibration monitoring if elected for this contract.

6.15.5 Groundwater Control

The groundwater level is anticipated to be between about Elevations 94 m and 95.5 m, within the native clayey silt with sand deposit.

Excavations for construction of the various retaining structure alternatives are expected to be maintained above the groundwater level, with minimal groundwater inflow from perched groundwater above or within the fill or native deposits; where ground water or perched water is encountered, it is anticipated that this can be handled by pumping from filtered sump pumps placed at the base of the excavation and outside of the foundation footprint.

In addition, 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.

6.15.6 Subgrade Protection

The subgrade soils will be susceptible to disturbance from construction traffic and/or ponded water. To limit this degradation, it is recommended that a concrete working slab be placed on the subgrade if construction of the retaining wall (i.e. placement of the concrete or granular fill) is not started within four hours after preparation, inspection and approval of the footing subgrade for the retaining wall. This requirement can be addressed with a note on the General Arrangement drawing and/or with an NSSP. A sample NSSP for the working slab is included in Appendix C.

7.0 CLOSURE

This Foundation Design Report was prepared by Mr. Matthew Kelly, P.Eng., and the technical aspects were reviewed by Ms. Sandra McGaghran, M.Eng., P.Eng. an Associate and Senior Geotechnical Engineer with Golder. Mr. Paul Dittrich, P.Eng., a MTO Foundations Designated Contact and Principal of Golder, conducted a technical and quality control review of this report.

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ACK/MWK/JPD/rb;ljv

https://golderassociates.sharepoint.com/sites/19542g/1 foundations/9 - reports/18 - rw e of et crk/2 - final/1530382 fidr retaining wall e of et crk 2019nov18.docx

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Terzaghi, K. and Peck, R.B., 1967. *Soil Mechanics in Engineering Practice*, 2nd Edition, John Wiley and Sons, New York.

U.S. Department of Transportation, Federal Highway Administration. *Geotechnical Engineering Circulation No. 4, Ground Anchors and Anchored Systems.* June 1999.

ASTM International

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

Commercial Software:

Slide (Version 8.018) by Rocscience Inc.
Settle^{3D} (Version 4.023) by Rocscience Inc.

Ontario Occupational Health and Safety Act:

Ontario Regulation 213Construction Projects (as amended)

Ontario Provincial Standard Specifications (OPSS)

OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS 572	Construction Specification for Seeding and Cover
OPSS 902	Construction Specification for Excavating and Backfilling Structures
OPSS 942	Construction Specification for Prestressed Soil and Rock Anchors
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

Ontario Provincial Standard Drawings (OPSD)

OPSD 208.010	Benching of Earth Slopes
OPSD 3090.101	Foundation, Frost Penetration Depths for Southern Ontario
OPSD 2131.150	Walls, Retaining, Backfill, Minimum Granular Requirement
OPSD 3190.100	Walls, Retaining and Abutment, Wall Drain

Ontario Regulations:

R.R.O 1990, Regulation 903Wells, under Ontario Water Resources Act, R.S.O. 1990, c. O.40

Special Provisions (SP)

SP 599S22	Retained Soil System
SP 105S10	Amendment to OPSS 501 – Construction

TABLE 1 – COMPARISON OF FOUNDATION ALTERNATIVES – RETAINING WALL BETWEEN STATION 14+072 AND 14+087

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs
Concrete retaining wall on a shallow foundation	Feasible, but less advantageous compared to other options	• Conventional excavation and construction techniques.	• Less tolerant to settlement than RSS wall, and hence requires subexcavation below existing fill to found on native soils at this site. • Deeper excavation required than for RSS wall, potentially necessitating a temporary protection system adjacent to the highway or a larger open cut excavation footprint. • Larger excavation area required than for soldier pile and concrete panel wall option. • Excavation depth for subexcavation/footing founding level will extend close to the groundwater table, potentially requiring greater groundwater control as compared to other alternatives.	• Higher cost relative to RSS wall.
Reinforced soil system (RSS) wall	Feasible, and preferred from a foundations perspective; however project specific approval required by MTO's RSS Committee	• Relative ease of construction but proprietary product required, with specialized design of internal stability of proprietary system. • Shallower and potentially smaller excavation as compared with concrete retaining wall option, minimizing or eliminating groundwater control requirements. • More tolerant of total and differential settlements. • RSS may be more cost-effective due to shorter retained heights as compared with concrete retaining wall option.	• Larger excavation area required than for soldier pile and concrete panel wall option. • Ground surface elevation at toe of proposed RSS wall is below the predicted high water level for the Regional Storm event; as such, project specific approval my MTO's RSS Committee will be required. •	• Lower cost than concrete retaining wall.
Soldier pile and concrete panel wall	Feasible, but would likely require permanent rock anchors or deadman anchors	• Minimizes excavation into the existing north embankment roadway slope, and potentially eliminates requirement for protection systems adjacent to the travelled lanes during construction staging.	• May not meet desired aesthetic requirements. • Soldier piles may need to extend/be socketed into the bedrock to achieve the required embedment depth/pile toe fixity, which would like necessitate coring/churn drilling to form the socket.	• Comparable costs to concrete retaining wall, but higher costs than RSS wall.

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs
Reinforced earth slope	Feasible	• Relative ease of construction but proprietary product required, with specialized design of internal stability of proprietary system. • Vegetated surface could be used to improve aesthetics.	• Special treatment of reinforced earth slope surfaces likely required to allow vegetation to grow and minimize erosion. • Potentially larger open cut excavation footprint compared to RSS wall to accommodate the inclination of the reinforced slope, which may then require the use of a temporary protection system adjacent to the travelled lanes during staging. • Reinforced earth slope will be north facing and therefore may require additional long-term maintenance to promote and maintain growth which is required to enhance surficial stability.	• Lower cost than RSS wall.

MINISTRY OF TRANSPORTATION, ONTARIO

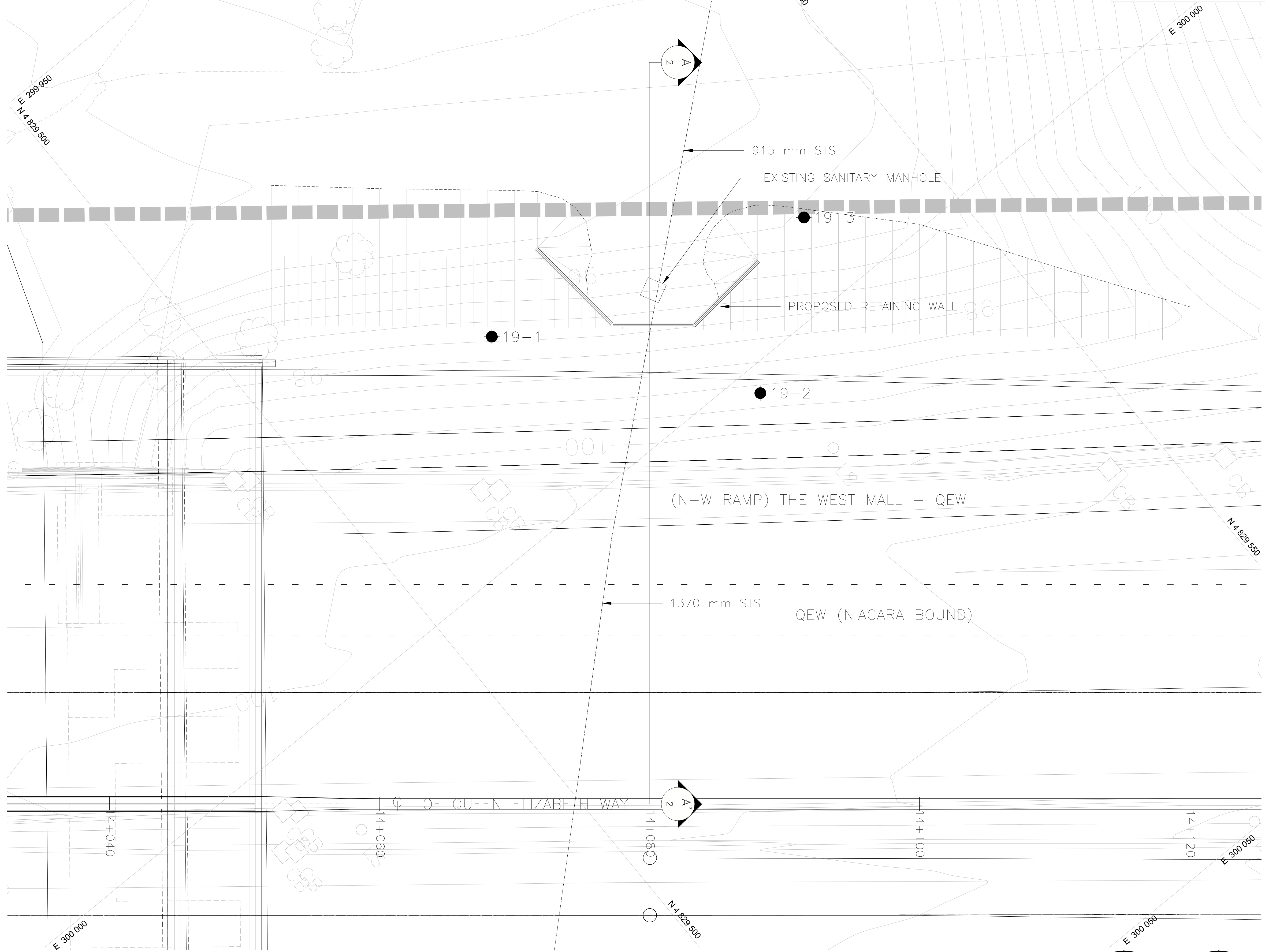
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No. 2018–2001
GWP No. 2102–13–00 & 2432–13–00

RETAINING WALL EAST OF ETOBICOKE CREEK
QEW STATION 14+072 TO 14+087
BOREHOLE LOCATIONS PLAN

SHEET

GOLDER



PLAN
SCALE
3 0 3 6 m

KEY PLAN
SCALE
2 0 2 4 km

LEGEND

● Borehole – Current Investigation

BOREHOLE CO–ORDINATES (NAD 83 ZONE 10)

No.	ELEVATION	NORTHING	EASTING
19–1	97.4	4829516.8	299986.0
19–2	99.7	4829529.5	300001.9
19–3	95.5	4829540.3	299993.9

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.

Horizontal Datum: MTM NAD 83 (Zone 10) CSRS (V6) (2010 epoch).

REFERENCE

Retaining wall plan provided in digital format by AECOM, drawing file nos. xs_QEW_C1_RSS_14+080.dwg, received October 4, 2019 and sanitary–MH–protection–East–of–Et–Crk_March–20–2019–REV1.dwg, received October 23, 2019.
Sanitary sewer plan provided in digital format by AECOM, drawing file no. QEW_DixieIC_utilities.dwg, received March 26, 2018.
Base plans provided in digital format by AECOM, drawing file nos. QEW_DixieIC_base.dwg and QEW_DixieIC_plan.dwg, dated July 20, 2016, received Dec. 06, 2016.
Design plans provided in digital format by AECOM, drawing file nos. QEW_Dixie_Cont1_plan.dwg and QEW_Dixie_Cont2_plan.dwg, received July 21, 2017.
Existing ground contours provided in digital format by AECOM, drawing file no. QEW_DixieIC_Contours3D.dwg, received Nov. 08, 2016, contour interval 0.5 m.
Key plan base data – MNRF LIO, obtained 2015.

LICENSED PROFESSIONAL ENGINEER S. McGaghran Nov. 15, 2019 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER J.P. DITTRICH Nov. 15, 2019 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No. 30M11–296

HWY. QEW		PROJECT NO. 1530382	DIST. CENTRAL
SUBM'D. MWK	CHKD. MWK	DATE: 11/15/2019	SITE:
DRAWN: DD	CHKD. SMM	APPD. JPD	DWG. 1

PLOT DATE: November 15, 2019

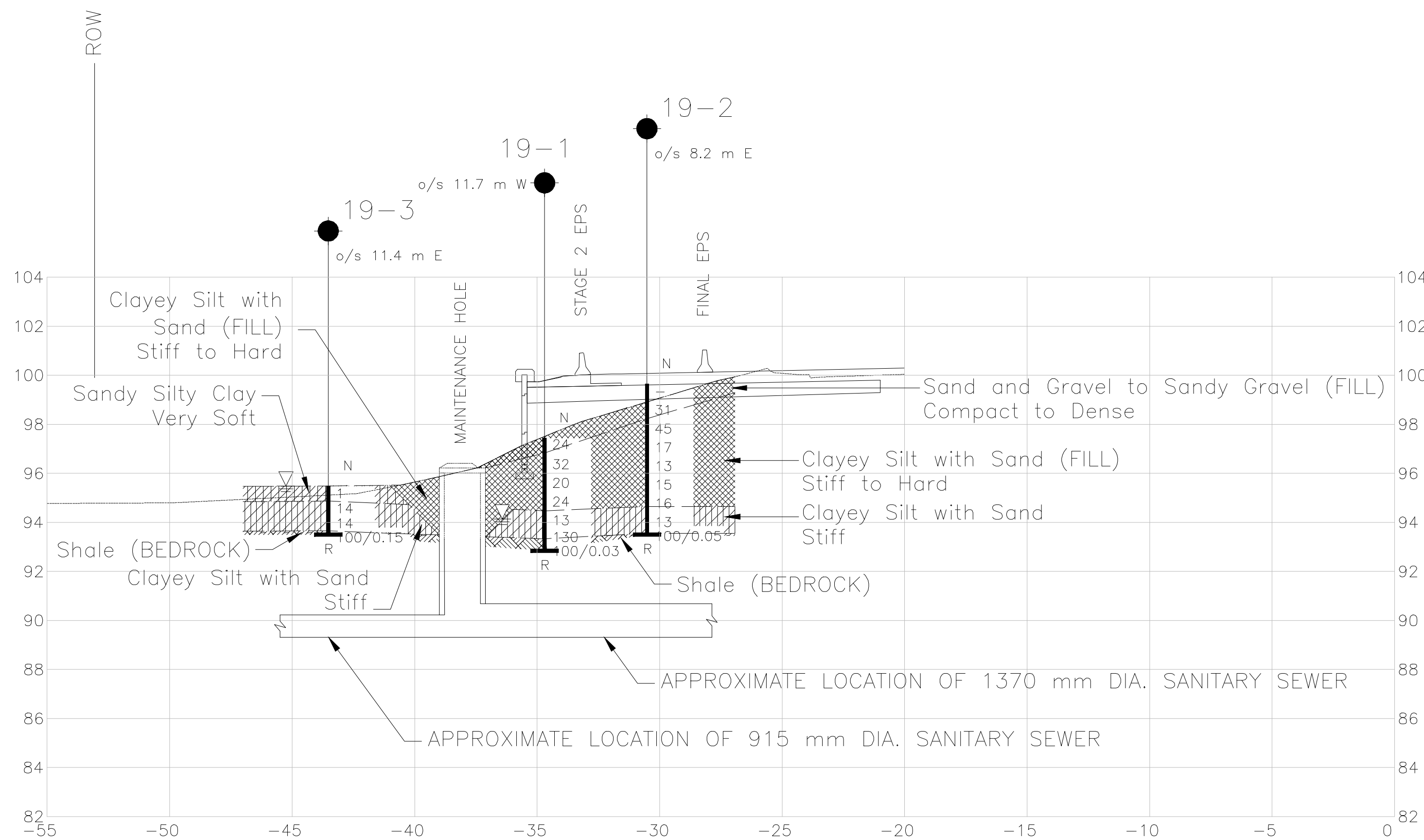
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No. 2018–2001
GWP No. 2102–13–00 & 2432–13–00

RETAINING WALL EAST OF ETOBICOKE CREEK
QEW STATION 14+072 TO 14+087
SOIL STRATA

SHEET

GOLDER



A–A / 1 CROSS–SECTION A–A" AT STATION 14+080

SCALE
3 0 3 6 m

KEY PLAN
SCALE
2 0 2 4 km

LEGEND

●	Borehole – Current Investigation
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
R	Refusal (Split–Spoon and/or Auger Refusal)
▽	WL upon completion of drilling

BOREHOLE CO–ORDINATES (NAD 83 ZONE 10)

No.	ELEVATION	NORTHING	EASTING
19–1	97.4	4829516.8	299986.0
19–2	99.7	4829529.5	300001.9
19–3	95.5	4829540.3	299993.9

NOTES

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

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

Horizontal Datum: MTM NAD 83 (Zone 10) CSRS (V6) (2010 epoch).

Vertical Datum: Canadian Geodetic Vertical Datum (CGVD) 1928.

REFERENCE

Retaining wall profile provided in digital format by AECOM, drawing file nos. xs_QEW_C1_RSS_14+080.dwg, received October 4, 2019 sanitary–MH–protection–East–of–Et–Crk_March–20–2019–REV1.dwg, received October 23, 2019.
Existing ground contours provided in digital format by AECOM, drawing file no. QEW_DixieIC_Contours3D.dwg, received Nov. 08, 2016, contour interval 0.5 m.
Key plan base data – MNRF LIO, obtained 2015.

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J.P. DITTRICH
Nov. 15, 2019
PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

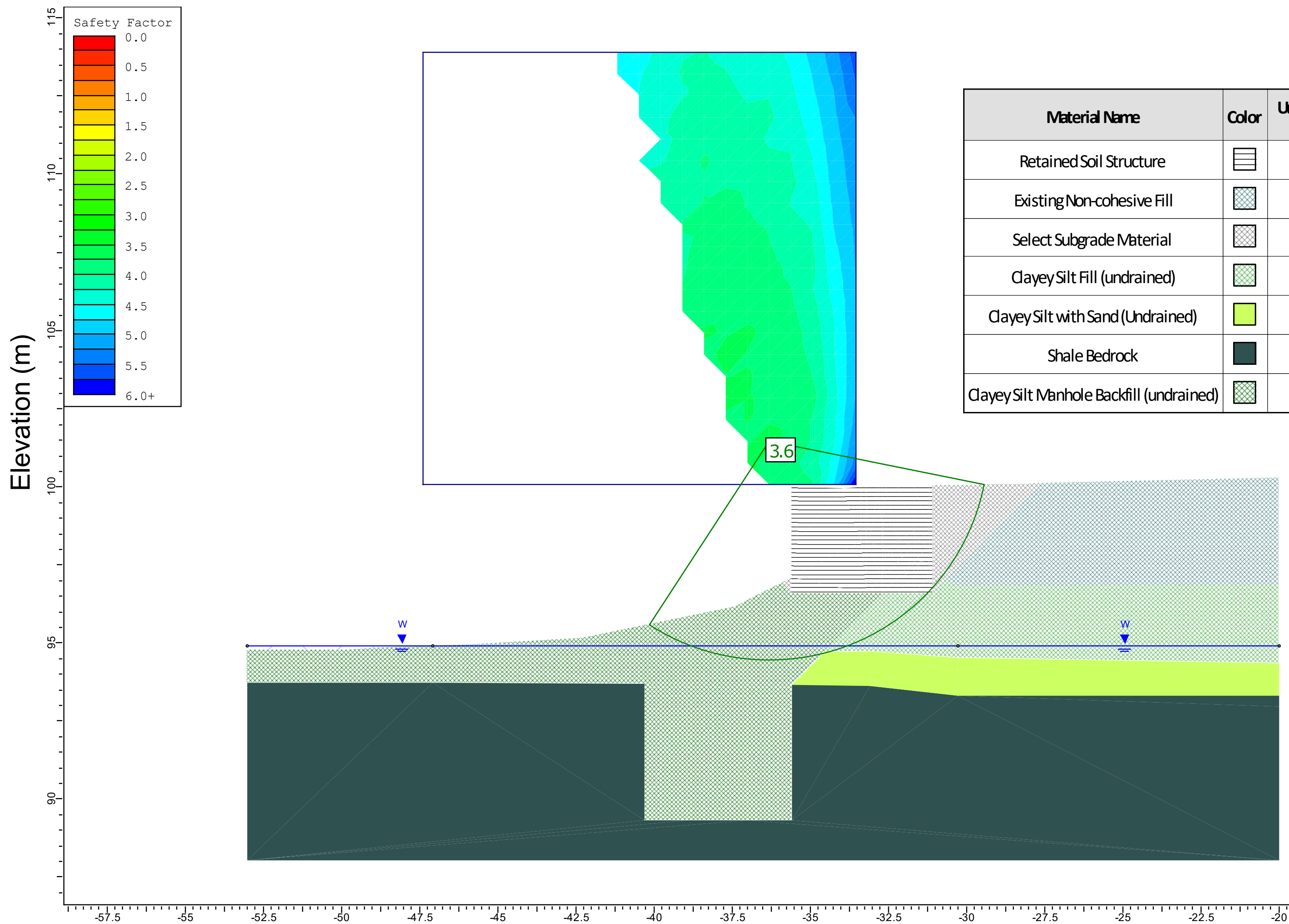
Geocres No. 30M11–296

HWY. QEW	PROJECT NO. 1530382	DIST. CENTRAL
SUBM'D. MWK	CHKD. MWK	DATE: 11/15/2019	SITE:
DRAWN: DD	CHKD. SMM	APPD. JPD	DWG. 2

PLOT DATE: November 15, 2019

GLOBAL STABILITY ANALYSIS
RSS WALL STA 14+079
SHORT-TERM ANALYSIS

Figure 1



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Retained Soil Structure		22	Infinite strength		
Existing Non-cohesive Fill		20	Mohr-Coulomb	0	33
Select Subgrade Material		20	Mohr-Coulomb	0	32
Clayey Silt Fill (undrained)		20	Undrained	90	
Clayey Silt with Sand (Undrained)		19	Undrained	90	
Shale Bedrock		20	Infinite strength		
Clayey Silt Manhole Backfill (undrained)		20	Undrained	65	

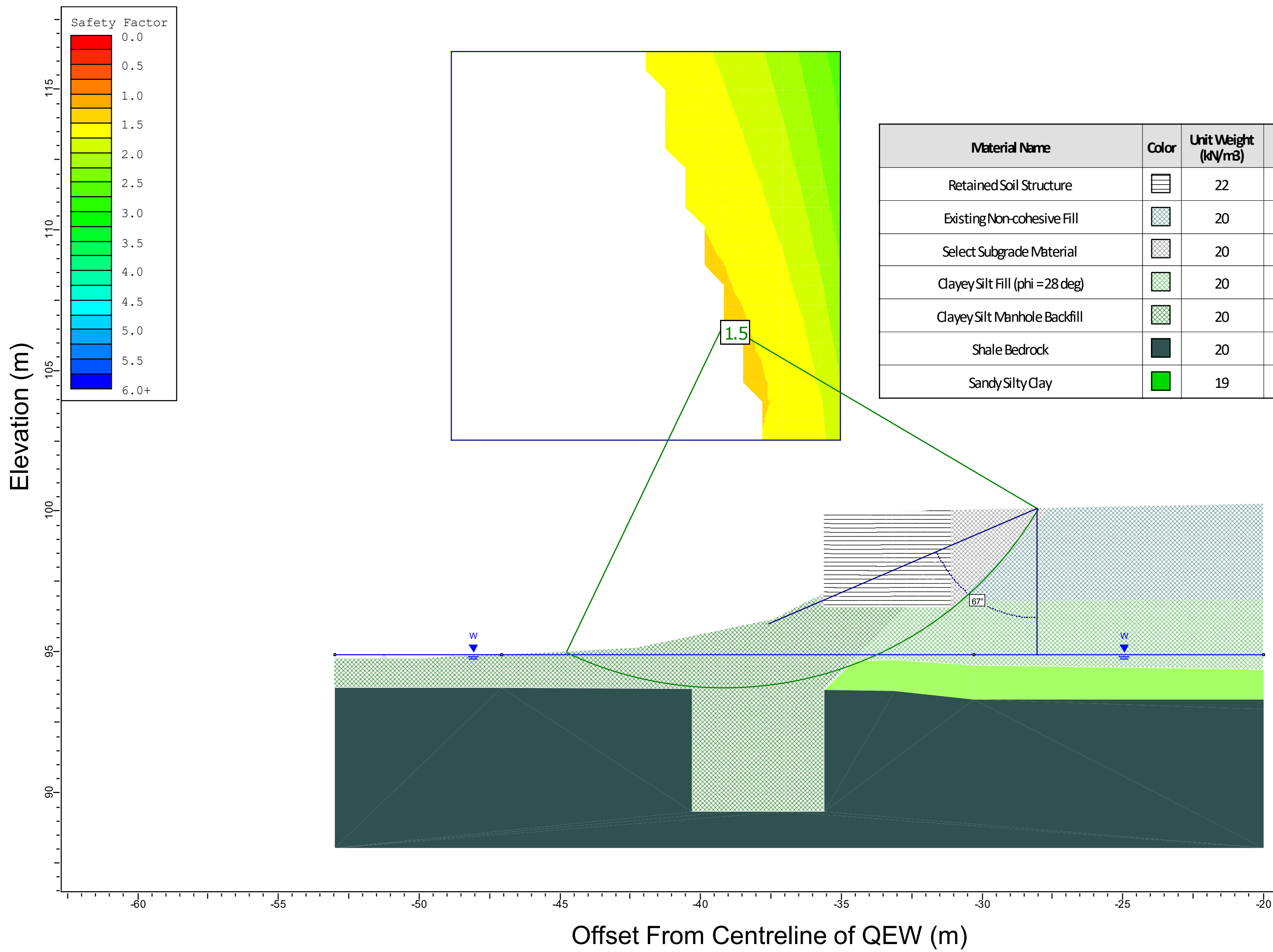
Date: October 2019
Project No: 1530382

Analysis By: ACK
Reviewed By: JPD

GLOBAL STABILITY ANALYSIS – CLAYEY SILT FILL PHI = 30 DEG
RSS WALL STA 14+079
LONG-TERM ANALYSIS

GOLDER

Figure 2



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Retained Soil Structure		22	Infinite strength		
Existing Non-cohesive Fill		20	Mohr-Coulomb	0	33
Select Subgrade Material		20	Mohr-Coulomb	0	32
Clayey Silt Fill (phi = 28 deg)		20	Mohr-Coulomb	0	28
Clayey Silt Manhole Backfill		20	Mohr-Coulomb	0	28
Shale Bedrock		20	Infinite strength		
Sandy Silty Clay		19	Mohr-Coulomb	0	25

Date: October 2019
Project No: 1530382

Analysis By: ACK
Reviewed By: JPD

APPENDIX A

Record of Borehole Sheets

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
υ	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 δ
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 $\tau = c' + \sigma' \tan \phi'$
2 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, 60º 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 cm^2 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 (Cohesionless) Soils

Compactness Condition	N 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 kPa	c_u, s_u psf
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 (cohesionless)) or With (cohesive)	Sand and Gravel Silty Clay with sand / Clayey Silt with sand

GOLDER

RECORD OF BOREHOLE No 19-1

SHEET 1 OF 1

METRIC

PROJECT 1530382

G.W.P. 2102-13-00; 2432-13-00

LOCATION N 4829516.8; E 299986.0 MTM NAD 83 ZONE 10 (LAT. 43.605580; LONG. -79.559636)

ORIGINATED BY AS

DIST Central HWY QEW

BOREHOLE TYPE 57 mm I.D. Hollow Stem Augers

COMPILED BY ACK

DATUM Geodetic

DATE September 15, 2019

CHECKED BY SMM

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
97.4	GROUND SURFACE						
0.0	Sand and gravel, trace fines (FILL) Compact Grey Moist	1	SS	24			
96.8 / 0.6	Clayey silt with sand, trace to some gravel (FILL) Very stiff to hard Grey Moist	2	SS	32			
		3	SS	20			8 43 35 14
		4	SS	24			
94.4 / 3.0	CLAYEY SILT with SAND, trace to some gravel, trace organics Stiff Grey Most to wet - Wet below a depth of 3.7 m (Elev. 93.7 m)	5	SS	13	∇		5 48 40 7
93.3 / 4.1	Inferred, grey SHALE (BEDROCK)	6A 6B	SS	130			
92.8 / 4.6	END OF BOREHOLE SPLIT-SPOON AND AUGER REFUSAL	7	SS	100/0.03			

NOTE:

1. Water level measured inside hollow stem augers at a depth of 3.3 m below ground surface (Elev. 94.1 m) upon completion of drilling.

Elevation scale: 97, 96, 95, 94, 93

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

$+^3, \times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

GOLDER

Foundation Design

RECORD OF BOREHOLE No 19-2

SHEET 1 OF 1 **METRIC**

PROJECT 1530382
G.W.P. 2102-13-00; 2432-13-00
LOCATION N 4829529.5; E 300001.9 MTM NAD 83 ZONE 10 (LAT. 43.605695; LONG. -79.559440)
ORIGINATED BY AS
DIST Central HWY QEW
BOREHOLE TYPE 57 mm I.D. Hollow Stem Augers
COMPILED BY ACK
DATUM Geodetic
DATE September 16, 2019
CHECKED BY SMM

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS (UNIT WEIGHT)	GR	SA	SI	CL
99.7	GROUND SURFACE									
0.0	Sandy gravel to sand and gravel, trace fines (FILL) Dense Grey Moist - Containing wood fragments at a depth of 0.76 m	1	AS	-						
		2	SS	31	99					
98.3 1.4	Clayey silt with sand, trace to some gravel, trace organics (FILL) Stiff to hard Grey Moist	3	SS	45	98	OC = 2.9%				
		4	SS	17	97					
		5	SS	13	96					
		6	SS	15		OC = 2.3%	11	48	30	11
		7A	SS	16	95					
94.7 5.0	CLAYEY SILT with SAND, trace organics Stiff Grey Moist	7B								
		8	SS	13	94					
93.6 6.2	Inferred, grey SHALE (BEDROCK)	9	SS	100/0.05						

END OF BOREHOLE
SPLIT-SPOON REFUSAL

NOTE:

1. Borehole dry upon completion of drilling after removal of augers.

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100
PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30
UNIT WEIGHT γ kN/m³

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

GOLDER

Foundation Design

RECORD OF BOREHOLE No 19-3

SHEET 1 OF 1 **METRIC**

PROJECT 1530382

G.W.P. 2102-13-00; 2432-13-00 LOCATION N 4829540.3; E 299993.9 MTM NAD 83 ZONE 10 (LAT. 43.605791; LONG. -79.559539) ORIGINATED BY AS

DIST Central HWY QEW BOREHOLE TYPE Continuous split spoon sampling COMPILED BY ACK

DATUM Geodetic DATE September 16, 2019 CHECKED BY SMM

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	NATURAL MOISTURE CONTENT w (%)	GR	SA	SI	CL
95.5	GROUND SURFACE									
0.0	Sandy SILTY CLAY, some gravel, trace organics Very soft Grey Moist	1	SS	1	95					
94.9 0.6	CLAYEY SILT with SAND, trace gravel, trace organics Stiff Grey Moist	2	SS	14		○				
		3	SS	14	94	├○─┤	1	37	50	12
93.7 2.0	Inferred, grey SHALE (BEDROCK)	4	SS	100/0.15						
	END OF BOREHOLE SPLIT-SPOON REFUSAL									

NOTE:

1. Water level at ground surface (Elev. 95.5 m) upon completion of borehole sampling.

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20 40 60 80 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

GTA-MTO 001 S:\CLIENTS\MTO\QEW-DIXIE\02_DATA\GINT\QEW-DIXIE.GPJ GAL-GTA.GDT 10/31/19

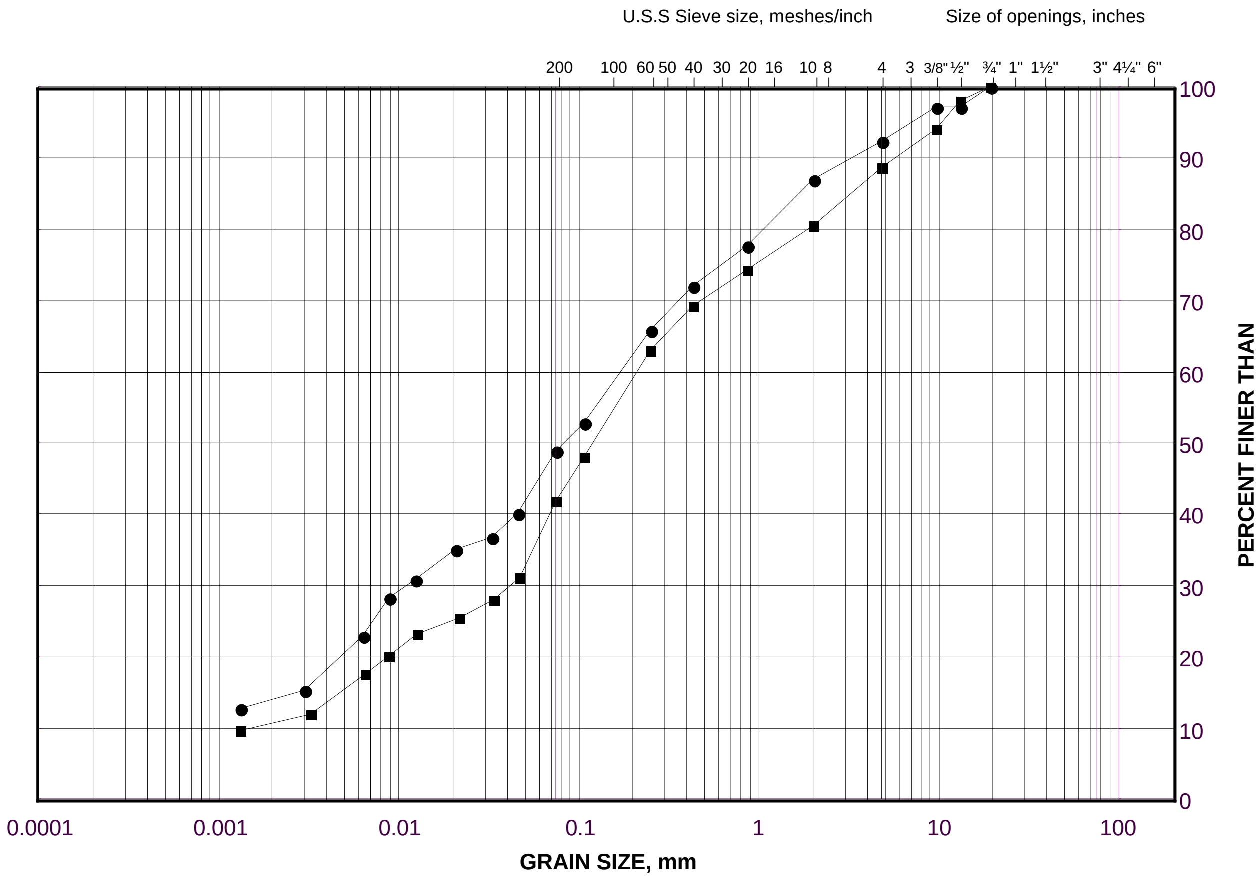
APPENDIX B

Geotechnical Laboratory Test Results and Analytical Test Results

GRAIN SIZE DISTRIBUTION

Clayey Silt with Sand (Fill)

FIGURE B-1



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

LEGEND

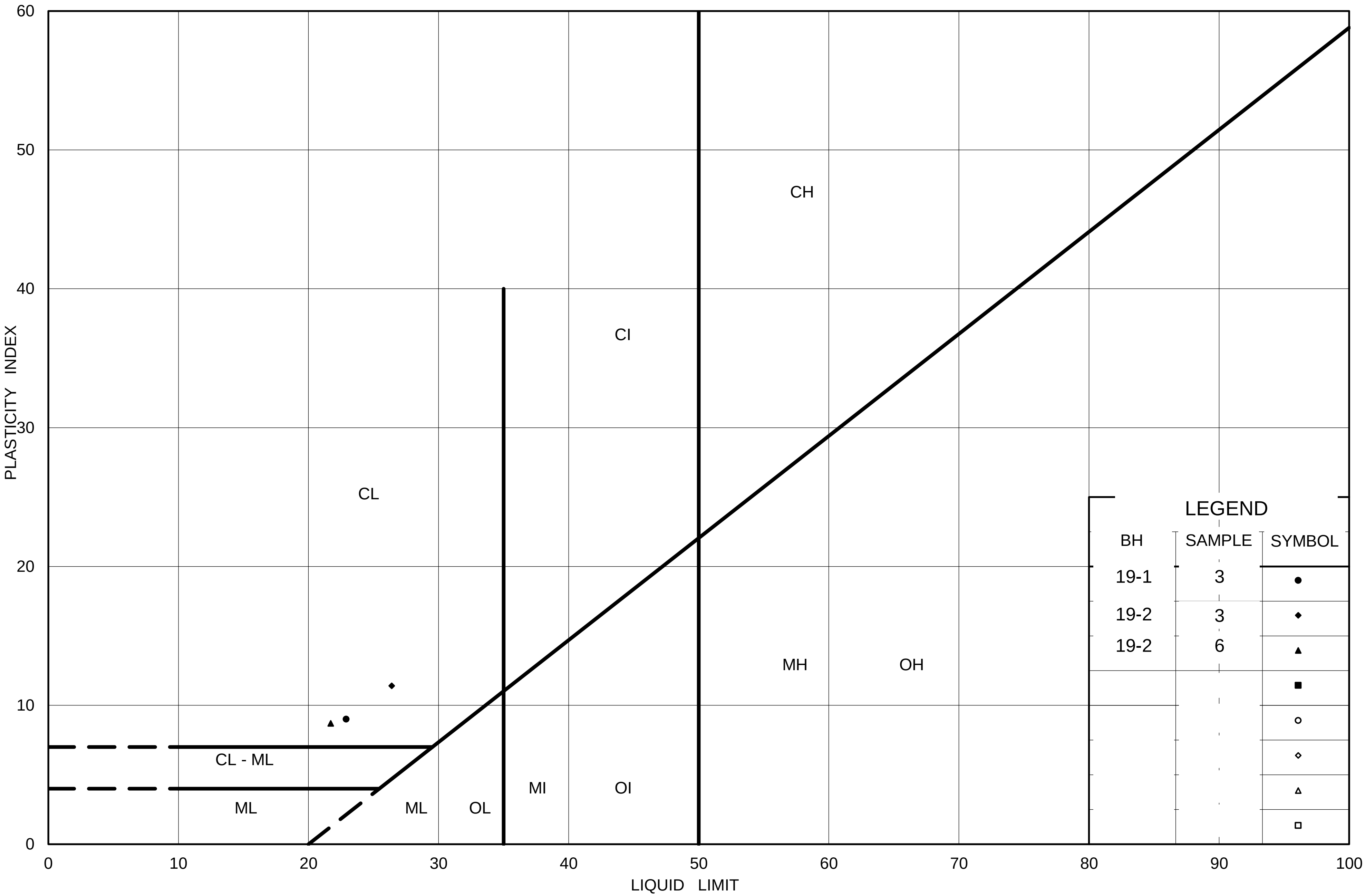
SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	19-1	3	95.6
■	19-2	6	95.5

Project Number: 1530382

Checked By: SMM

Golder Associates

Date: 09-Oct-19



Ministry of Transportation

Ontario

PLASTICITY CHART
Clayey Silt with Sand (Fill)

Figure No. B-2

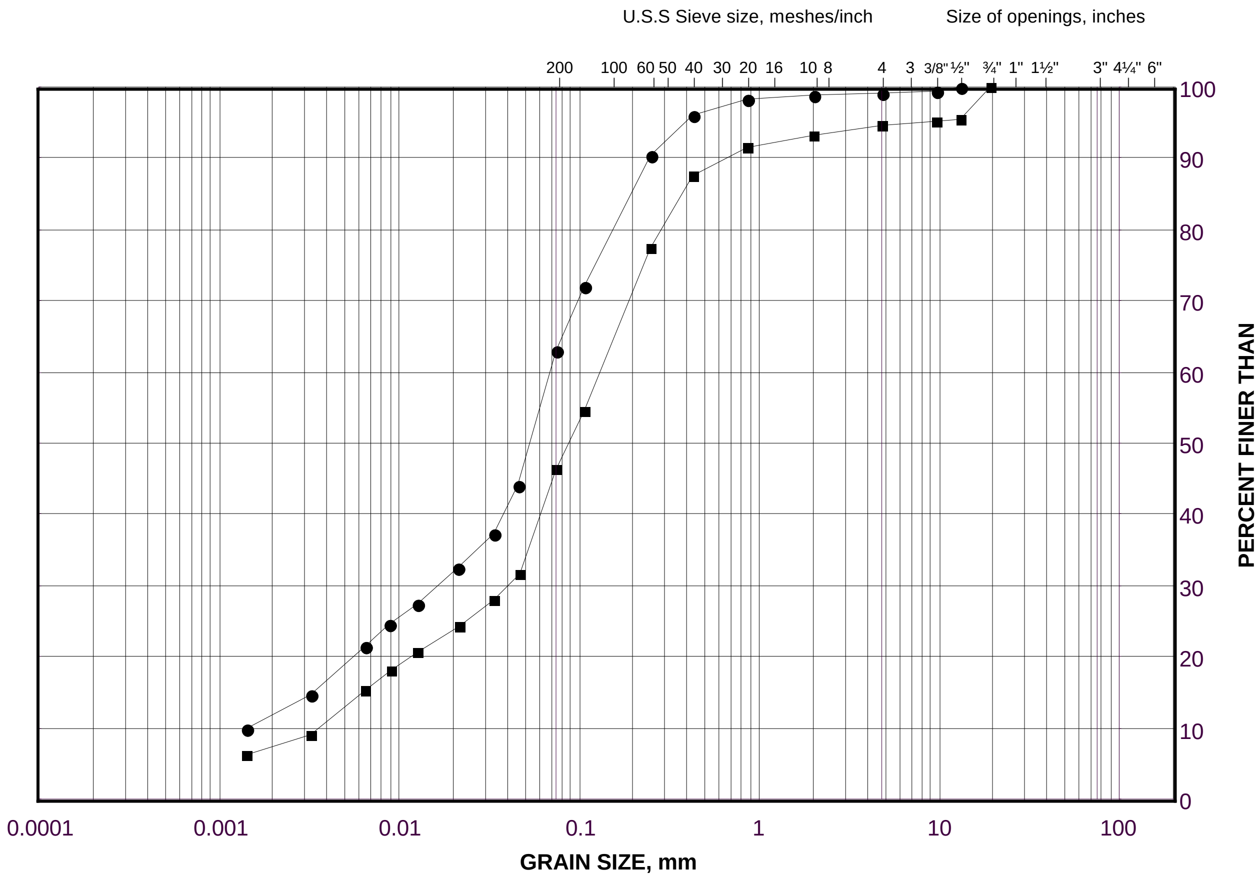
Project No. 1530382

Checked By: SMM

GRAIN SIZE DISTRIBUTION

Clayey Silt with Sand

FIGURE B-3



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

LEGEND

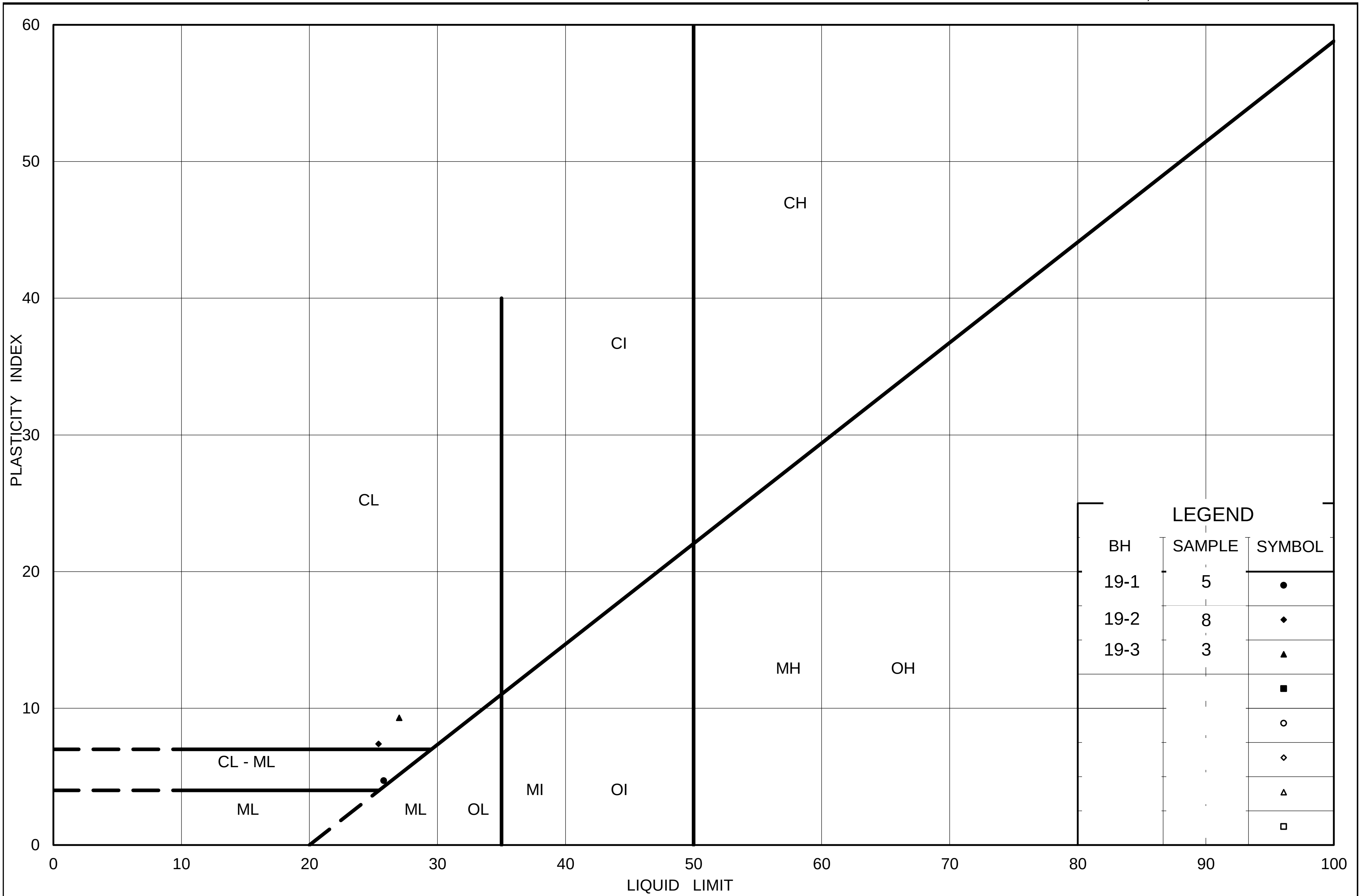
SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	19-3	3	93.9
■	19-1	5	94.1

Project Number: 1530382

Checked By: ____________________

Golder Associates

Date: 09-Oct-19



Ministry of Transportation Ontario	PLASTICITY CHART Clayey Silt with Sand	Figure No. B-4
		Project No. 1530382
		Checked By: SMM

Your Project #: 1530382
Site Location: QEW DIXIE
Your C.O.C. #: 132400

Attention: Sandra McGaghran

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

Report Date: 2019/09/26
Report #: R5897066
Version: 1 - Final

CERTIFICATE OF ANALYSIS

BV LABS JOB #: B9Q4865
Received: 2019/09/20, 18:12

Sample Matrix: Soil
Samples Received: 2

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Reference
Chloride (20:1 extract)	2	2019/09/25	2019/09/26	CAM SOP-00463	SM 23 4500-Cl E m
Conductivity	2	2019/09/25	2019/09/25	CAM SOP-00414	OMOE E3530 v1 m
pH CaCl2 EXTRACT	2	2019/09/26	2019/09/26	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	2	2019/09/21	2019/09/25	CAM SOP-00414	SM 23 2510 m
Sulphate (20:1 Extract)	2	2019/09/25	2019/09/26	CAM SOP-00464	EPA 375.4 m

Remarks:

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

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

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

Solid sample results, except biota, are based on dry weight unless otherwise indicated. Organic analyses are not recovery corrected except for isotope dilution methods.
Results relate to samples tested. When sampling is not conducted by BV Labs, results relate to the supplied samples tested.
This Certificate shall not be reproduced except in full, without the written approval of the laboratory.
Reference Method suffix “m” indicates test methods incorporate validated modifications from specific reference methods to improve performance.
* RPDs calculated using raw data. The rounding of final results may result in the apparent difference.

Bureau Veritas Laboratories 6740 Campobello Road, Mississauga, Ontario, L5N 2L8 Tel: (905) 817-5700 Toll-Free: 800-563-6266 Fax: (905) 817-5777 www.bvlabs.com

Microbiology testing is conducted at 6660 Campobello Rd. Chemistry testing is conducted at 6740 Campobello Rd.

Your Project #: 1530382
Site Location: QEW DIXIE
Your C.O.C. #: 132400

Attention: Sandra McGaghran

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

Report Date: 2019/09/26
Report #: R5897066
Version: 1 - Final

CERTIFICATE OF ANALYSIS

BV LABS JOB #: B9Q4865
Received: 2019/09/20, 18:12

Encryption Key

Please direct all questions regarding this Certificate of Analysis to your Project Manager.
Ema Gitej, Senior Project Manager
Email: Ema.Gitej@bvlabs.com
Phone# (905)817-5829
==
This report has been generated and distributed using a secure automated process.
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BV Labs Job #: B9Q4865
Report Date: 2019/09/26

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

SOIL CORROSIVITY PACKAGE (SOIL)

BV Labs ID		KVN121			KVN121			KVN122		
Sampling Date		2019/09/15			2019/09/15			2019/09/15		
COC Number		132400			132400			132400		
	UNITS	**19-2 SA 5**	**RDL**	**QC Batch**	**19-2 SA 5 Lab-Dup**	**RDL**	**QC Batch**	**19-3 SA 2**	**RDL**	**QC Batch**
Calculated Parameters										
Resistivity	ohm-cm	2800		6346192				2000		6346192
Inorganics										
Soluble (20:1) Chloride (Cl-)	ug/g	90	20	6351526	97	20	6351526	130	20	6351526
Conductivity	umho/cm	355	2	6351403				494	2	6351403
Available (CaCl2) pH	pH	7.71		6353836				7.82		6353836
Soluble (20:1) Sulphate (SO4)	ug/g	<20	20	6351527				160	20	6351527

RDL = Reportable Detection Limit
QC Batch = Quality Control Batch
Lab-Dup = Laboratory Initiated Duplicate

BV Labs ID		KVN122		
Sampling Date		2019/09/15		
COC Number		132400		
	UNITS	**19-3 SA 2 Lab-Dup**	**RDL**	**QC Batch**
Inorganics				
Soluble (20:1) Sulphate (SO4)	ug/g	150	20	6351527

RDL = Reportable Detection Limit
QC Batch = Quality Control Batch
Lab-Dup = Laboratory Initiated Duplicate

BV Labs Job #: B9Q4865
Report Date: 2019/09/26

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

TEST SUMMARY

BV Labs ID: KVN121
Sample ID: 19-2 SA 5
Matrix: Soil

Collected: 2019/09/15
Shipped:
Received: 2019/09/20

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	6351526	2019/09/25	2019/09/26	Deonarine Ramnarine
Conductivity	AT	6351403	2019/09/25	2019/09/25	Kazzandra Adeva
pH CaCl2 EXTRACT	AT	6353836	2019/09/26	2019/09/26	Neil Dassanayake
Resistivity of Soil		6346192	2019/09/25	2019/09/25	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	6351527	2019/09/25	2019/09/26	Deonarine Ramnarine

BV Labs ID: KVN121 Dup
Sample ID: 19-2 SA 5
Matrix: Soil

Collected: 2019/09/15
Shipped:
Received: 2019/09/20

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	6351526	2019/09/25	2019/09/26	Deonarine Ramnarine

BV Labs ID: KVN122
Sample ID: 19-3 SA 2
Matrix: Soil

Collected: 2019/09/15
Shipped:
Received: 2019/09/20

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	6351526	2019/09/25	2019/09/26	Deonarine Ramnarine
Conductivity	AT	6351403	2019/09/25	2019/09/25	Kazzandra Adeva
pH CaCl2 EXTRACT	AT	6353836	2019/09/26	2019/09/26	Neil Dassanayake
Resistivity of Soil		6346192	2019/09/25	2019/09/25	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	6351527	2019/09/25	2019/09/26	Deonarine Ramnarine

BV Labs ID: KVN122 Dup
Sample ID: 19-3 SA 2
Matrix: Soil

Collected: 2019/09/15
Shipped:
Received: 2019/09/20

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Sulphate (20:1 Extract)	KONE/EC	6351527	2019/09/25	2019/09/26	Deonarine Ramnarine

Bureau Veritas Laboratories 6740 Campobello Road, Mississauga, Ontario, L5N 2L8 Tel: (905) 817-5700 Toll-Free: 800-563-6266 Fax: (905) 817-5777 www.bvlabs.com

Microbiology testing is conducted at 6660 Campobello Rd. Chemistry testing is conducted at 6740 Campobello Rd.

GENERAL COMMENTS

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

Package 1	3.3°C

Results relate only to the items tested.

QUALITY ASSURANCE REPORT

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

			Matrix Spike		SPIKED BLANK		Method Blank		RPD	
QC Batch	Parameter	Date	% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
6351403	Conductivity	2019/09/25			103	90 - 110	<2	umho/cm	6.2	10
6351526	Soluble (20:1) Chloride (Cl-)	2019/09/26	NC	70 - 130	103	70 - 130	<20	ug/g	7.6	35
6351527	Soluble (20:1) Sulphate (SO4)	2019/09/26	NC	70 - 130	102	70 - 130	<20	ug/g	9.6	35
6353836	Available (CaCl2) pH	2019/09/26			100	97 - 103			0.61	N/A

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)

VALIDATION SIGNATURE PAGE

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

Anastassia Hamanov, Scientific Specialist

BV Labs has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per ISO/IEC 17025, signing the reports. For Service Group specific validation please refer to the Validation Signature Page.

Maxxam
A Bureau Veritas Group Company

6740 Campobello Road, Mississauga, Ontario L5N 2L8
Phone: 905-817-5700 Fax: 905-817-5779 Toll Free: 800-563-6266
CAM FCD-01191/4

CHAIN OF CUSTODY RECORD 132400

Page 1 of 1

Invoice Information		Report Information (if differs from invoice)		Project Information (where applicable)		Turnaround Time (TAT) Required
Company Name:	GOLDER ASSOCIATES	Company Name:		Quotation #:		[X] Regular TAT (5-7 days) Most analyses
Contact Name:	Sandra McGaghran	Contact Name:		P.O. #/ AFE#:		PLEASE PROVIDE ADVANCE NOTICE FOR RUSH PROJECTS
Address:	6925 CENTURY AVE, SUITE 100 MISSISSAUGA, ON	Address:		Project #:	1530382	Rush TAT (Surcharges will be applied)
				Site Location:	QEW DIXIE	[ ] 1 Day [ ] 2 Days [ ] 3-4 Days
Phone:	905 567 4444 Fax:	Phone: Fax:		Site #:		
Email:	Sandra_McGaghran@golder.com	Email:		Site Location Province:		Date Required:
				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

[ ] Table 1 [ ] Res/Park [ ] Med/ Fine
[ ] Table 2 [ ] Ind/Comm [ ] Coarse
[ ] Table 3 [ ] Agri/ Other
[ ] Table ____
FOR RSC (PLEASE CIRCLE) Y / N

Other Regulations

[ ] CCME [ ] Sanitary Sewer Bylaw
[ ] MISA [ ] Storm Sewer Bylaw
[ ] PWQO Region ____
[ ] Other (Specify)
[ ] REG 558 (MIN. 3 DAY TAT REQUIRED)

Include Criteria on Certificate of Analysis: Y / N

SAMPLES MUST BE KEPT COOL (< 10 °C) FROM TIME OF SAMPLING UNTIL DELIVERY TO MAXXAM

Analysis Requested

	SAMPLE IDENTIFICATION	DATE SAMPLED (YYYY/MM/DD)	TIME SAMPLED (HH:MM)	MATRIX	# OF CONTAINERS SUBMITTED	FIELD FILTERED (CIRCLE) Metals / Hg / CrVI	BTEX/ PHC F1	PHCs F2 - F4	VOCs	REG 153 METALS & INORGANICS	REG 153 ICPMS METALS	REG 153 METALS (Hg, Cr VI, ICPMS Metals, HWS - B)	CORROSIVITY PKG.							HOLD- DO NOT ANALYZE	COMMENTS
1	19-2 SA5	2019/09/15	AM	SOIL	1								X								
2	19-3 SA2	2019/09/15	AM	SOIL	1								X								
3																					
4																					
5																					
6																					
7																					
8																					
9																					
10																					

LABORATORY USE ONLY

CUSTODY SEAL Y / N Present	Intact	COOLER TEMPERATURES
N	N	3 / 4 / 3

COOLING MEDIA PRESENT: Y / N

RELINQUISHED BY: (Signature/Print)	DATE: (YYYY/MM/DD)	TIME: (HH:MM)	RECEIVED BY: (Signature/Print)	DATE: (YYYY/MM/DD)	TIME: (HH:MM)
Alysha Kobylinski / Alysha Kobylinski	2019/09/20	18:12	FAREEN JETHA	2019/09/20	18:12

20-Sep-19 18:12
Ema Gitej
B9Q4865
KFL ENV-1213

Unless otherwise agreed to in writing, work submitted on this Chain of Custody is subject to Maxxam's standard Terms and Conditions. Signing of this Chain of Custody document is acknowledgment and acceptance of our terms which are available...
Sample container, preservation, hold time and packages information can be viewed at http://maxxam.ca/wp-content/uploads/Ontario-COC.pdf.

COC-1004 (01/19) White: Maxxam ~ Yellow: Client

APPENDIX C

Non-Standard Special Provisions and Notice to Contractor

WORKING SLAB - Item No.

Non-Standard Special Provision

1.0 Scope

This Special Provision covers the requirements for the supply and placement of a concrete working slab under retaining wall foundations between Station 14+072 and 14+087, for the QEW Niagara bound widening east of Etobicoke Creek.

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.

7.04 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

VIBRATION MONITORING - Item No.

Special Provision

TABLE OF CONTENTS

1.0 SCOPE

This special provision describes requirements for vibration monitoring for the following components of the Contract:

- Deep foundation and temporary protection system installation for the construction of the Etobicoke Creek bridge
- Temporary protection system for the construction of the retaining wall between Station 13+830 to 13+975 on the north side of the QEW and 14+072 and 14+087 on the north side of the QEW
- Deep foundation installation for a retaining wall between Station 13+650 and 13+750 on the north side of the QEW
- Temporary protection system for the removal of existing retaining walls on the north side of the QEW between Station 13+501 and 13+815 and on the south side of the QEW between Station 13+748.5 and 13+847.5.

2.0 REFERENCES

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

1. Foundation Investigation and Design Report, QEW - Etobicoke Bridge Replacement (Site No. 37-237/1&2), City off Mississauga, Etobicoke, Ministry of Transportation, Ontario, GWP 2102-13-00 and 2432-13-00.

2. Retaining Wall from Station 13+830 to 13+975, QEW Improvements from East of Cawthra Road to The East Mall, Mississauga and Etobicoke, Ministry of Transportation, Ontario, G.W.P. 2102-13-00 & 2432-13-00.

3. Retaining Wall from Station 14+072 to 14+087, QEW Improvements from East of Cawthra Road to The East Mall, Mississauga and Etobicoke, Ministry of Transportation, Ontario, G.W.P. 2102-13-00 & 2432-13-00.

4. Foundation Investigation and Design Report, Retaining Walls No. 24-887/W and 24-888/W Replacement, 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.

5. Sanitary Sewer, 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

3.0 DEFINITIONS

For the purposes of this specification, the following definitions apply:

Contractor's Engineer means 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 Contractor's Engineer shall be retained by the Contractor to ensure general conformance with the Contract Documents and issue certificates of conformance.

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

Pre-Construction Condition Survey means a detailed record, accompanied by film or video, as necessary, of the condition of private or public property, prior to the commencement of vibratory or vibration-inducing construction operations.

Post-Construction Condition Survey means a detailed record, accompanied by film or video, as necessary, of the condition of private or public property, after completion of vibratory or vibration-inducing construction operations.

4.0 DESIGN AND SUBMISSION REQUIREMENTS

4.1 Submission Requirements

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

a) Equipment and methods used by the Contractor to perform the work that may cause undue vibration.
b) Qualifications of vibration monitoring specialist.
c) Details regarding proposed instrumentation.

d) Proposed location of instruments adjacent to the on the residences, utilities, wells, or other potentially vibration-sensitive structures within a 250 m radius from the Etobicoke Creek bridge, within 75 m of the protection systems for the removal of the existing retaining wall and deep foundation installation on the north side of the QEW, and within 50 m of the proposed retaining wall alignment and/or protection systems on the south side of the QEW.
e) Proposed frequency of readings.
f) Action plan to be taken to adjust deep foundation and protection system installation methods or if readings show vibrations exceeding tolerable levels.

6.0 EQUIPMENT

6.1 Vibration Monitoring Equipment

All vibration monitoring equipment shall be capable of measuring and recording ground vibration PPV up to 200 mm/s in the vertical, transverse, and radial directions. The equipment shall have been calibrated within the last 12 months either by the manufacturer or other qualified agent. Proof of calibration shall be submitted to the Contract Administrator prior to commencement of any monitoring operations.

7.0 CONSTRUCTION

7.1 Pre- and Post-Construction Condition Surveys

A Pre-Construction Condition Survey and Post-Construction Condition Survey shall be prepared for all buildings, utilities, structures, water wells, and facilities within a 250 m radius from the Etobicoke Creek bridge, within 75 m of the protection systems for the removal of the existing retaining wall and deep foundation installation on the north side of the QEW, and within 50 m of the proposed retaining wall alignment and/or protection systems on the south side of the QEW.

7.1.1 Pre-Construction Condition Surveys

The standard inspection procedure shall include the provision of an explanatory letter to the owner or occupant and owner with a formal request for permission to carry out an inspection.

The Pre-Construction Condition Survey, at each structure/well within a 250 m radius from the Etobicoke Creek bridge, within 75 m of the protection systems for the removal of the existing retaining wall and deep foundation installation on the north side of the QEW, and within 50 m of the proposed retaining wall alignment and/or protection systems on the south side of the QEW, shall be completed a minimum of two (2) weeks prior to commencement of installation of the deep foundations and/or protection system(s). Only one Pre-Construction Condition Survey per structure or facility is required to be carried out in advance of deep foundation and protection system installation, unless more than six (6) months will elapse between these operations, in which case an interim inspection will be required.

The Pre-Construction Condition Survey shall include, as a minimum, the following information:

a) Type of structure, including type of construction and if possible, the date when built.
b) Identification and description of existing differential settlements, including visible cracks in walls, floors, and ceilings, including a diagram, if applicable, room-by-room. All other apparent structural and cosmetic damage or defects shall also be noted. Defects shall be described, including dimensions, wherever possible.
c) Digital photographs or digital video or both, as necessary, to record areas of significant concern.

Photographs and videos shall be clear and shall accurately represent the condition of the property. Each photograph or video shall be clearly labelled with the location and date taken.

A copy of the Pre-Construction Construction Survey limited to a single residence or property, including copies of any photographs or videos that may form part of the report, shall be provided to the owner of that residence or property, upon request.

7.1.2 Post-Construction Condition Surveys

The standard inspection procedure shall include the provision of an explanatory letter to the owner or occupant and owner with a formal request for permission to carry out an inspection.

A Post-Construction Condition Survey at each structure within a 250 m radius from the Etobicoke Creek bridge, within 75 m of the protection systems for the removal of the existing retaining wall and deep foundation installation on the north side of the QEW, and within 50 m of the proposed retaining wall alignment and/or protection systems on the south side of the QEW, is required within two (2) months of completion of the installation of deep foundations and protection systems.

The Post-Construction Condition Survey shall include, as a minimum, the following information:

a) Identification and description of existing differential settlements, including visible cracks in walls, floors, and ceilings, including a diagram, if applicable, room-by-room. All other apparent structural and cosmetic damage or defects shall also be noted. Defects shall be described, including dimensions, wherever possible.
b) Digital photographs or digital video or both, as necessary, to record areas of significant concern.
c) Comparison between pre-condition survey documented concerns and post-condition concerns.

Photographs and videos shall be clear and shall accurately represent the condition of the property. Each photograph or video shall be clearly labelled with the location and date taken.

A copy of the Post-Construction Condition Survey limited to a single residence or property, including copies of any photographs or videos that may form part of the report, shall be provided to the owner of that residence or property, upon request. The report shall confirm that there have been no changes to the property between the Pre-Construction Condition Survey and the Post-Construction Condition Survey as a result of the installation of deep foundations and protection systems.

7.2 Monitoring

The vibration monitoring equipment shall be placed on the ground surface in the vicinity of each retaining wall section requiring deep foundation elements or protection systems, and on the ground surface at radial distances of 25 m, 50 m, and 100 m from these locations toward receptors (e.g., buildings, sensitive utilities). The Contractor shall take readings continuously during construction for the deep foundation elements of retaining walls or associated protection system installation, and shall immediately notify the Contract Administrator if the vibrations exceed the limits specified herein.

The vibrations measured on private structures, wells, etc. shall not exceed 25 mm/s. Those measured on utilities, if applicable, shall not exceed 10 mm/s.

If the readings are not within the limits stated above, the Contractor must alter the installation procedures until the vibrations at the various locations are within acceptable levels.

7.3 Records

The Contractor/Contractor's Engineer shall submit details of the vibration monitoring to the Contract Administrator as follows:

a) The time/duration of each reading.
b) Construction operations (i.e. installation of sheet piling) and timing of such relative to the readings.
c) Details of exceedances and modifications to operations.
d) Final report containing all relevant data including vibration monitoring and Pre- and Post-Construction Condition Surveys.

10.0 BASIS OF PAYMENT

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

OBSTRUCTIONS – Item No.

Notice to Contractor

The Contactor shall be alerted to the potential presence of pieces of wood, concrete, rubber base of traffic barrel, cobbles and boulders on the face of the slope in the vicinity of Station 14+072 to 14+087. These materials may be present in the fill material and at the fill / native interface. Consideration of the presence of these obstructions must be made in the selection of appropriate equipment and procedures for excavations and installation of temporary protection systems.

PROTECTION SYSTEM – Item No.

Special Provision

Amendment to OPSS 539, November 2014

593.07.02 Removal of Protection Systems

Subsection 539.07.02 of OPSS 539 is deleted in its entirety and replaced with the following:

Protection systems shall be removed from the right-of-way unless it is specified in the Contract Documents that the protection system may be left in place.

Where piles are left in place, the top shall be removed to at least 1.2 m below the finished grade or ground level.

The method and sequence of removal shall be such that there shall be no damage to the new work, existing work and facility being protected.

All disturbed areas shall be restored to an equivalent or better condition than existing prior to the commencement of construction.

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