



Foundation Investigation and Design Report

Retaining Walls, Simcoe Street to Albert Street Highway 401 Replacement of Three Underpasses and Rehabilitation of Oshawa Creek Bridge, Ministry of Transportation, Ontario, G.W.P. 2298-13-00

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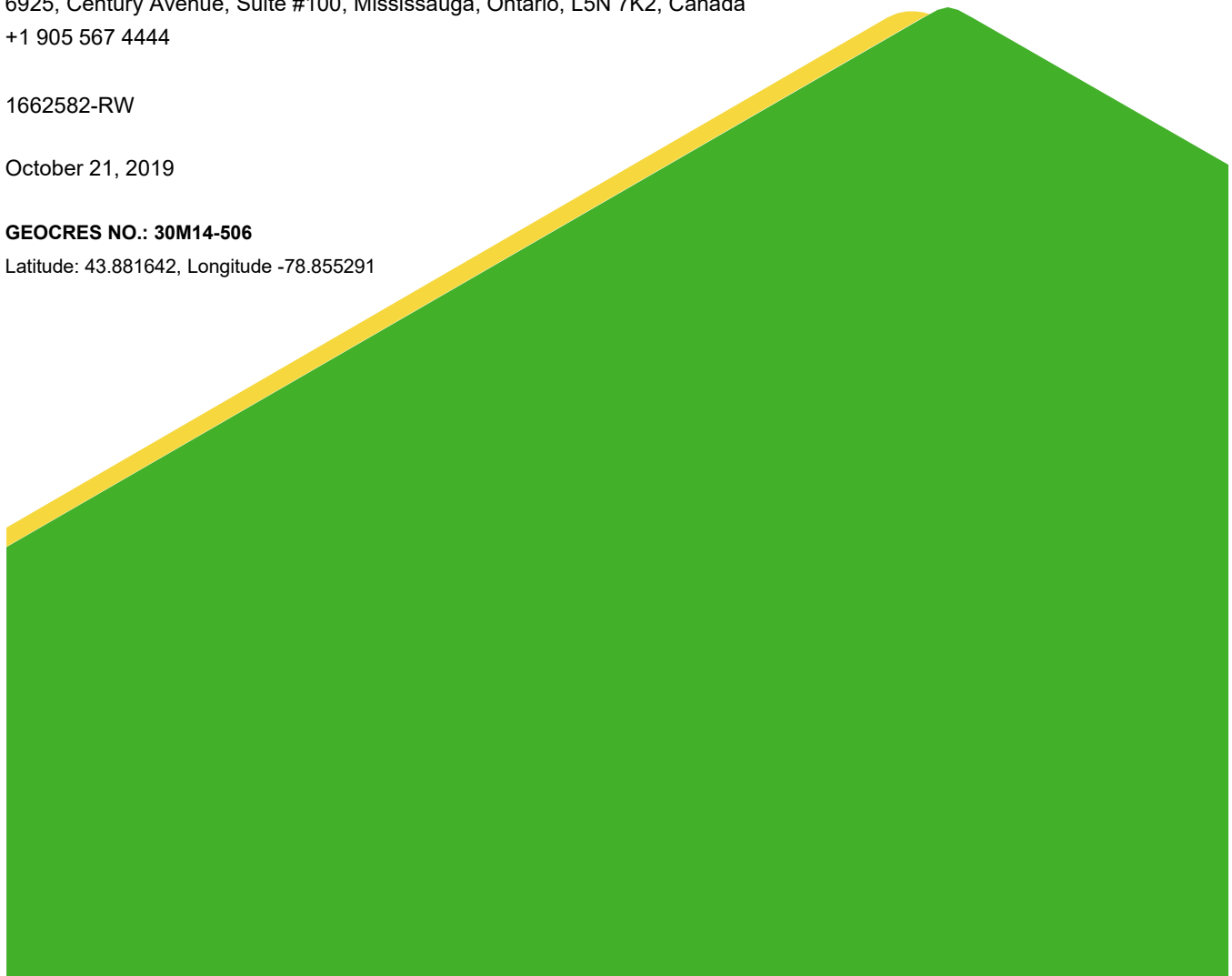
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October 21, 2019

GEOCRES NO.: 30M14-506

Latitude: 43.881642, Longitude -78.855291



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Certificate of Analysis Report # R5031865

Certificate of Analysis Report # R5031866

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PART A

FOUNDATION INVESTIGATION REPORT
RETAINING WALLS, ALBERT STREET TO SIMCOE STREET
REPLACEMENT OF THREE UNDERPASSES AND REHABILITATION OF
OSHAWA CREEK BRIDGE
MINISTRY OF TRANSPORTATION, ONTARIO
G.W.P. 2298-13-00

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by WSP on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the Highway 401 replacement and rehabilitation works in the City of Oshawa, Regional Municipality of Durham, Ontario. This report presents the results of the foundation investigation along the proposed retaining walls located on the north and south sides of Highway 401, extending between about Simcoe Street and 100 m east of Albert Street, associated with the widening of Highway 401. The location of the site is shown on the Key Plan on Drawing 1.

The purpose of this investigation is to establish the subsurface soil and bedrock conditions at the proposed retaining wall locations, by borehole drilling and laboratory testing on selected soil samples. The results of the foundation investigations at other structures associated with the widening and structural replacements are presented in separate reports.

The Terms of Reference (TOR) for the foundation engineering services are outlined in MTO's Request for Proposals (RFP) for Assignment No. 2016-E-0022, dated May 2016, and the associated Addendum 1 and Clarifications Nos. 1 and 2. The scope of work for the foundation engineering services is presented in Section 17.8 of WSP's Technical Proposal for this assignment.

2.0 SITE DESCRIPTION

Along the proposed retaining walls, Highway 401 conveys eastbound and westbound vehicular traffic through the City of Oshawa, in the Regional Municipality of Durham, Ontario from the Simcoe Street Underpass to east of the Albert Street Underpass of Highway 401. There are presently no existing retaining walls along this section.

The natural ground surface at the site is generally between about Elevations 100 m and 102 m. Highway 401 is constructed in a cut with the highway grade between about Elevations 95 m and 96 m, sloping down eastward. The road grade of Simcoe Street ranges from about Elevations 101 m to 103 m, rising northward. Similarly, the grade of Albert Street at the site ranges from about Elevations 101 m to Elevation 102 m, also rising northward. Commercial businesses, a church and several houses are located along Lviv Boulevard, immediately north and paralleling Highway 401, while the Oshawa Visitor Information Centre and a service station are located along Bloor Street East to the south of Highway 401 and residential dwellings are present along both sides of Albert Street approaching Bloor Street East.

3.0 INVESTIGATION PROCEDURES

3.1 Previous Investigation

In March 2015, Golder carried out a preliminary field investigation at the site, during which time three boreholes (designated as Boreholes S1, A1, and A2) were advanced at the northwest, northeast, and southeast quadrants of the Highway 401 – Simcoe Street and Highway 401 – Albert Street underpasses, as shown on Drawings 1 and 2. The boreholes were advanced to depths of approximately 6.8 m, 9.5 m, and 8 m below existing ground surface, respectively.

The results of the investigation are presented in Golder Associates reports titled *"Preliminary Foundation Investigation Report, Simcoe Street Underpass, Structure Site No. 22-176, Highway 401 Improvements from Brock Road to Courtice Road, Regional Municipality of Durham, W.O. 10-20011"*, Report No. 11-1184-0143-9, dated May 19, 2017 (GEOCRE 30M14-451) and *"Preliminary Foundation Investigation Report, Albert Street Underpass, Structure Site No. 22-177, Highway 401 Improvements from Brock Road to Courtice Road, Regional Municipality of Durham, W.O. 10-20011"*, Report No. 11-1184-0143-10, dated May 19, 2017 (GEOCRE 30M14-452).

The borehole locations provided on Drawings 1 and 2 and on the borehole records in Appendix A are positioned relative to MTM NAD 83 northing and easting (Zone 10) coordinates and the ground surface elevations are referenced to Geodetic datum. The borehole locations, including geographic coordinates, ground surface elevations, and borehole depths are summarized below.

Borehole Number	MTM NAD83 Northing (m) (Latitude, °)	MTM NAD83 Easting (m) (Longitude, °)	Ground Surface Elevation (m)	Borehole Depth (m)
S1	4,860,398.8 (43.881760)	356,528.6 (-78.856300)	101.3	6.8
A1	4,860,447.6 (43.882190)	356,680.2 (-78.854380)	102.1	9.5
A2	4,860,377.6 (43.881556)	356,679.4 (-78.854396)	102.0	8.0

3.2 Current Investigation

The field work for the current investigation was carried out between October 30, 2017 and March 6, 2018 during which time a total of seven boreholes (designated as SS-9, SS-8, NRW-1, AS-7, NRW-2, SRW-1, and AS-2) were advanced in the vicinity and along the north and south retaining walls as shown on Drawing 1. The boreholes were advanced to depths ranging from 10.8 m to 21.9 m, including 3 m of rock coring in Borehole AS-7.

The investigation was carried out using truck-mounted CME 75 drill rigs, supplied and operated by Pontil Drilling of Mount Albert, Ontario. The boreholes were advanced through the overburden using 216 mm outside diameter (O.D.) hollow-stem augers. Soil samples were obtained at 0.75 m and 1.5 m intervals of depth, using a 50 mm O.D. split-spoon sampler driven by an automatic hammer in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586-08)¹. The split-spoon samplers used in the investigation limit the maximum particle size that can be sampled and tested to about 35 mm. Therefore, particles or objects that may exist within the soils that are larger than this dimension would not be sampled or represented in the grain size distributions. HQ-diameter coring equipment was used to advance Borehole AS-7 into bedrock.

The groundwater conditions were noted in the boreholes upon removal of the hollow stem augers at completion of drilling. Standpipe piezometers were installed in Boreholes SS-9, NRW-1, SRW-1, and AS-2 to allow for monitoring of groundwater levels. The remaining boreholes were backfilled with bentonite and the ground surface was restored to near original condition as practical using cold-patch asphalt, where applicable.

The field work was observed by members of Golder's engineering and technical staff, who located the boreholes, arranged for the clearance of underground services, observed the drilling, sampling and in situ testing operations, logged the boreholes, and examined and cared for the soil and bedrock core samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Mississauga laboratory where the samples underwent further visual examination. Geotechnical laboratory testing (water content, grain size distribution, and Atterberg limits) was carried out on select soil samples. All laboratory testing was carried out to MTO and / or ASTM Standards, as appropriate. In addition, two soil samples were submitted to Maxxam Analytics (Maxxam) of Mississauga, Ontario for analysis of select parameters to assess for the potential corrosion to buried steel and deterioration of concrete.

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

The borehole locations were measured relative to existing site features and were plotted on the digital terrain model from which the borehole coordinates and ground surface elevations were then obtained. The borehole locations provided on Drawings 1 and 2 and on the borehole records in Appendix A are positioned relative to MTM NAD 83 northing and easting (Zone 10 CSRS CBNv6-2010.0) coordinates and the ground surface elevations are referenced to Geodetic datum. The borehole locations, including geographic coordinates, ground surface elevations, and borehole depths are summarized below.

Location	Borehole Number (West to East along Retaining Wall Alignment)	MTM NAD83 Northing (Latitude, °)	MTM NAD83 Easting (Longitude, °)	Ground Surface Elevation (m)	Borehole Depth (m)
North Retaining Wall	SS-9	4,860,394.2 (43.881720)	356,513.4 (-78.856460)	101.1	11.1
	SS-8	4,860,396.0 (43.881733)	356,523.4 (-78.856336)	101.2	10.8
	NRW-1	4,860,427.8 (43.882015)	356,592.0 (-78.855479)	102.2	10.8
	AS-7	4,860,444.2 (43.882157)	356,672.9 (-78.854472)	102.1	21.9 ¹
	NRW-2	4,860,477.8 (43.882455)	356,731.9 (-78.853734)	101.3	10.8
South Retaining Wall	SRW-1	4,860,350.5 (43.881317)	356,620.6 (-78.855131)	103.0	15.7
	AS-2	4,860,377.8 (43.881557)	356,697.2 (-78.854175)	101.6	14.2

Note: 1. Includes 3.0 m of bedrock coring

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 and *Urban Geology of Canadian Cities* (Brennand, 1998)³. The Iroquois Plain extends around the western shores of Lake Ontario and is comprised of the flat to undulating lakebed and beaches of the former glacial Lake Iroquois, which occupied this area during the last glacial recession.

The surficial soils in this area of the Iroquois Plain are typically comprised of glaciolacustrine clays, silts, and sands to gravelly sands, which are underlain by an extensive till deposit that is mapped in this area as the Bowmanville Till. More recent alluvial deposits of gravel, sand, silt, and/or clay are present in the creek valleys.

Bedrock at the site location is described as Shale of the Whitby Formation.

4.2 Subsurface Conditions

Subsurface soil, bedrock and groundwater conditions as encountered in the boreholes are presented on the borehole records in Appendix A. *Method of Soil Classification, Abbreviations and Terms Used on Records of*

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.)

³ Brennand, T.A., 1998. *Urban Geology Note: Oshawa Ontario*. In P.F. Karrow, and O.L. White (Eds.), *Geological Association of Canada, Special Papers 42: Urban Geology of Canadian Cities*, p. 353-364.

Boreholes and Test Pits and *List of Symbols* sheets are provided in Appendix A to assist in the interpretation of the borehole records. The geotechnical laboratory test results are presented on the Record of Borehole sheets in Appendix A and on the test plot sheets in Appendix B. The analytical laboratory test results are presented in Appendix C.

The results of the in-situ field tests (i.e., SPT “N”-values) as presented on the borehole records and in Section 4.2 are uncorrected. The boundaries between the strata on the borehole records have been inferred from drilling observations and non-continuous sampling. Therefore, these boundaries represent transitions between soil types rather than exact planes of geological change. The interpreted stratigraphic profiles along the retaining wall structures as shown on Drawing 2 are simplifications of the subsurface conditions. Variation in the stratigraphic boundaries between and beyond boreholes will exist and is to be expected.

4.2.1 North Retaining Wall

In general, the subsurface conditions encountered in the boreholes advanced along the alignment of the proposed north retaining wall consist of non-cohesive and cohesive fill, underlain by an upper deposit of silt and sand to silty sand till. The upper non-cohesive till deposit is underlain by a silty sand to gravelly sand deposit, which in turn is underlain by a lower deposit of sandy silt to silty sand till and/or a clayey silt till, underlain by shale bedrock.

4.2.1.1 Asphalt

An approximately 120 mm to 180 mm thick layer of asphalt pavement was encountered at ground surface in Boreholes SS-9, SS-8, AS-7, NRW-1 and NRW-2.

4.2.1.2 Topsoil

An approximately 400 mm and 500 mm thick layer of topsoil was encountered at ground surface in Boreholes S1 and A1, respectively.

4.2.1.3 Silty Sand to Gravelly Sand Fill

A 0.5 m to 0.6 m thick layer of non-cohesive fill was encountered underlying the asphalt in Boreholes SS-9, SS-8, AS-7, NRW-1 and NRW-2. The non-cohesive fill extended to depths ranging from 0.6 m to 0.7 m below ground surface (Elevations 101.5 m to 100.4 m) and consists of sand, some silt, trace to some gravel, to gravelly sand, some silt.

The SPT “N”-values measured within the non-cohesive fill range from 3 blows to 21 blows per 0.3 m of penetration, indicating a very loose to compact state of compactness.

The water content measured on samples of the non-cohesive fill ranges from about 2 per cent to 25 per cent.

4.2.1.4 Clayey Silt to Silty Clay Fill

A 0.4 m to 3.0 m thick layer of cohesive fill was encountered underlying the topsoil / non-cohesive fill in Boreholes SS-9, SS-8, NRW-1, AS 7, A1 and NRW-2. The cohesive fill was encountered at depths ranging from 0.5 m to 0.7 m below ground surface (Elevations 101.6 m to 100.4 m) and extended to depths ranging from 0.9 m to 3.7 m below ground surface (Elevation 101.2 m to 98.4 m). The cohesive fill consists of clayey silt to silty clay, trace sand to sandy clayey silt to clayey silt with sand, trace gravel. Asphalt fragments were observed in the cohesive fill recovered from Borehole NRW-1.

The SPT “N”-values measured within the cohesive fill range from 4 blows to 49 blows per 0.3 m of penetration, suggesting a soft to hard consistency.

The water content measured on samples of the deposit ranges from about 9 per cent to 41 per cent.

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

Atterberg limit testing was carried out on three samples of the cohesive fill and the results are presented on Figure B-2 in Appendix B. The Atterberg limits tests measured liquid limits ranging from 16 per cent to 44 per cent, plastic limits ranging from 10 per cent to 19 per cent, and plasticity indices ranging from 6 per cent to 25 per cent, indicating the cohesive fill is clayey silt of low plasticity to silty clay of intermediate plasticity.

4.2.1.5 Clayey Silt to Silty Clay

A 0.5 m and 0.9 m thick layer of clayey silt and silty clay was encountered underlying the cohesive fill and topsoil in Boreholes A1 and S1, respectively. The surface of the cohesive layer was encountered at depths ranging from 0.9 m to 0.4 m below ground surface (Elevations 101.2 m to 100.9 m) and extends to depths ranging from 1.4 m to 1.5 m below ground surface (Elevations 100.7 m to 99.8 m) in the respective boreholes. The cohesive layers contain trace sand, trace gravel, and trace organics as noted in the layer in Borehole S1.

The SPT “N”-values measured within the cohesive fill layer are 18 blows and 21 blows per 0.3 m of penetration, suggesting a very stiff consistency.

The water content measured on a sample of the clayey silt layer is about 24 per cent.

4.2.1.6 Silt and Sand to Silty Sand Till – Upper Deposit

A 3.0 m to 4.0 m thick non-cohesive glacial till deposit was encountered underlying the fill / clayey silt / silty clay layers at all borehole locations along the north retaining wall, except at Borehole NRW-1. The non-cohesive till deposit was encountered at depths ranging from 1.4 m to 2.9 m below ground surface (Elevations 100.7 m to 98.4 m) and extends to depths ranging from 4.4 m to 6.8 m below ground surface (Elevations 97.5 m to 94.5 m). The deposit consisted of silt and sand to silty sand, trace to some gravel, trace to some clay. Cobbles and boulders are commonly encountered in glacially derived materials and should be expected within this deposit as inferred from auger grinding in Borehole S1.

The SPT “N”-values measured within the non-cohesive till deposit range from 12 blows to 104 blows per 0.3 m of penetration, and one SPT “N”-value of 100 blows for 0.13 m of penetration, indicating a compact to very dense relative density.

The water content measured on samples of the deposit range from about 5 per cent to 10 per cent.

Grain size distribution testing was carried out on seven samples of the upper non-cohesive till deposit and the results are presented on Figure B-3 in Appendix B.

Atterberg limit testing was carried out on two samples of the deposit and the results are presented on Figure B-4 in Appendix B. The Atterberg limits tests measured liquid limits of about 12 per cent and 13 per cent, plastic limits of about 10 per cent, and plasticity indices of about 2 per cent and 3 per cent, indicating the till material is classified as a silt of slight plasticity.

4.2.1.7 Sand to Sand and Gravel

A 1.1 m to 4.5 m thick deposit of sand to gravelly sand to sand and gravel, containing trace to some silt was encountered underlying the upper silt and sand to silty sand till deposit at all borehole locations along the north retaining wall alignment, except at Borehole NRW-2. The sand to sand and gravel deposit was encountered at

depths ranging from 3.7 m to 5.6 m below ground surface (Elevations 98.5 m to 95.5 m) and extends to depths ranging from 6.6 m to 10.1 m below ground surface (Elevations 95.4 m to 91.0 m).

The SPT “N”-values measured within the sand to sand and gravel deposit range from 38 blows to 116 blows per 0.3 m of penetration and up to 100 blows per 0.1 m of penetration, indicating a dense to very dense state of compactness.

The water content measured on samples of the deposit range from about 2 per cent to 19 per cent.

Grain size distribution testing was carried out on four samples of the gravelly sand to sand and gravel portion of the deposit and the results are presented on Figure B-5 in Appendix B. In addition, grain size distribution testing was carried out on two samples of the sand interlayers of the deposit and the results are presented on Figure B-6 in Appendix B.

4.2.1.8 Sandy Silt

A 1.8 m thick deposit of sandy silt, trace to some clay, trace gravel was encountered underlying the upper silt and sand till deposit in Borehole NRW-2. The sandy silt deposit was encountered at a depth of 6.8 m below ground surface (Elevation 94.5 m) and extends to a depth of 8.6 m below ground surface (Elevation 92.7 m).

The SPT “N”-value measured within the sandy silt deposit is 70 blows per 0.3 m of penetration, indicating a very dense state of compactness.

The water content measured on a sample of this deposit is about 18 per cent.

Grain size distribution testing was carried out on one sample of the sandy silt deposit and the result is presented on Figure B-7 in Appendix B.

4.2.1.9 Sandy Silt to Silty Sand Till – Lower Deposit

A 0.2 m to 4.0 m thick deposit of sandy silt to silty sand till was encountered underlying the sand to sand and gravel/sandy silt deposits in Boreholes S1, NRW-1, AS-7, A1 and NRW-2. This lower non-cohesive till deposit was encountered at depths ranging from 6.6 m to 9.1 m below ground surface (Elevations 95.4 m to 92.7 m) and extends to depths ranging from 6.8 m to 10.8 m below ground surface (Elevations 94.5 m to 90.5 m), and Boreholes A1, NRW-1 and NRW-2 were terminated in this deposit. The deposit consists of sandy silt, some gravel, trace clay to silty sand, trace clay, trace gravel. Cobbles and boulders are commonly present in glacially derived materials and should be expected within this deposit, although these materials were not encountered in this deposit at boreholes advanced at this site.

The SPT “N”-values measured within the lower sandy silt to silty sand till range from 100 blows for 0.15 m of penetration to 100 blows for 0.08 m of penetration, indicating a very dense state of compactness.

The water content measured on samples of the till deposit range from about 4 per cent to 16 per cent.

Grain size distribution testing was carried out on one sample of the non-cohesive till lower deposit and the result is presented on Figure B-8A in Appendix B.

4.2.1.10 Clayey Silt to Clayey Silt with Sand Till

A 0.5 m to 7.2 m thick cohesive till deposit comprised of clayey silt to clayey silt with sand till was encountered underlying the sand to sand and gravel / silt in Boreholes SS-9, SS-8, AS-7 and A1. This cohesive till was encountered at depths ranging from 8.5 m to 10.5 m (Elevations 93.5 m to 91.0 m) and extends to depths ranging

from 9.1 m to 17.7 m (Elevations 93.5 m to 84.4 m). Borehole SS-9 was terminated in this deposit. The deposit consists of clayey silt, some sand, trace gravel to clayey silt with sand, trace to some gravel. Cobbles and boulders are commonly present in glacially derived materials and should be expected within this deposit as inferred from auger grinding in Borehole AS-7.

The SPT “N”-values measured within the cohesive till deposit range from 64 blows to 124 blows per 0.3 m of penetration with “N”-values up to 100 blows per 0.07 m of penetration, suggesting a hard consistency.

The water content measured on samples of this cohesive till deposit range from about 6 per cent to 16 per cent.

Grain size distribution testing was carried out on three samples of the cohesive till lower deposit and the results are presented on Figure B-8B in Appendix B.

Atterberg limit testing was carried out on five samples of the cohesive till lower deposit for the North Retaining Wall and the results are presented on Figure B-9 in Appendix B. The Atterberg limits tests measured liquid limits ranging from 17 per cent to 31 per cent, plastic limits ranging from 9 per cent to 14 per cent, and plasticity indices ranging from 7 per cent to 17 per cent, indicating the cohesive till is clayey silt of low plasticity.

4.2.1.11 Silt

An approximately 0.7 m thick silt interlayer containing trace sand, trace clay was encountered within the lower cohesive till deposit in Boreholes AS-7 and SS-8 at depths of 9.8 m and 10.1 m below ground surface (Elevations 92.3 m and 91.1 m), respectively. Borehole SS-8 was terminated in the silt interlayer.

The SPT “N”-values measured within the silt interlayers are 100 blows for 0.15 m of penetration and 100 blows for 0.13 m of penetration, indicating a very dense state of compactness.

A water content measured on the silt interlayer is about 9 per cent.

4.2.1.12 Bedrock

Bedrock was encountered underlying the cohesive till in Borehole AS-7 at a depth of 17.7 m below ground surface (Elevation 84.4 m) and the bedrock was cored from depths of 18.9 m to 21.9 m below ground surface (Elevations 83.3 m to 80.2 m).

Based on the cored bedrock samples, the bedrock is fresh, thinly to medium bedded, black, fine grained, faintly porous shale, interbedded with fossiliferous limestone. Details of the rock cores are presented on the Record of Drillhole sheet in Appendix A. The degree of weathering of the bedrock samples (i.e., fresh – W1), and the strength classification of the intact rock mass based on field identification (i.e., fresh – R3) are described in accordance with the International Society for Rock Mechanics (ISRM⁴) standard classification system. The Total Core Recovery (TCR) and Solid Core Recovery (SCR) of the bedrock core samples are 100 per cent and 100 per cent, respectively. The Rock Quality Designation (RQD) of the two core runs obtained is 91 per cent and 100 per cent, indicating that the rock is of excellent quality in accordance with Table 3.10 of the Canadian Foundation Engineering Manual (2006).⁵

⁴ International Society for Rock Mechanics Commission on Test Methods, 1985. Int. J. Rock Mech. Min. Sci. & Geomech. Abstr. Vol 22, No. 2, pp. 51-60.

⁵ Canadian Geotechnical Society, 2006. Canadian Foundation Engineering Manual, 4th Edition, BiTech Publications.

4.2.2 South Retaining Wall

In general, the subsurface conditions along the south retaining wall as encountered in Boreholes SRW-1, A2, and AS-2 generally consist of cohesive and non-cohesive fill, underlain by an upper deposit of silt and sand to gravelly silty sand till, which is in turn underlain by interlayered deposits of clayey silt, silt and sand, sand to gravelly sand underlain by a lower deposit of sandy silt till and/or clayey silt with sand till.

4.2.2.1 Asphalt

An approximately 100 mm thick layer of asphalt pavement was encountered at ground surface in Borehole AS-2.

4.2.2.2 Topsoil

An approximately 500 mm thick layer of topsoil was encountered at ground surface in Borehole A2.

4.2.2.3 Clayey Silt to Silty Clay Fill

A 0.9 m and 1.4 m thick layer of cohesive fill was encountered underlying the topsoil and at ground surface in Boreholes A2 and SRW-1, respectively. The surface of the cohesive fill was encountered at Elevations 101.5 m to 103 m and the deposit extends to depths of 1.4 m below ground surface (Elevations 100.6 m and 101.6 m) in the respective boreholes. The cohesive fill consists of silty clay to sandy clayey silt trace gravel, trace to some organics were noted in the silty clay layer encountered in Borehole A2.

The SPT “N”-values measured within the cohesive fill range from 7 to 15 blows per 0.3 m of penetration, suggesting a firm to stiff consistency.

The water content measured on samples of the cohesive fill ranges from about 11 per cent to 23 per cent.

4.2.2.4 Silty Sand to Gravelly Sand Fill

A 2 m thick layer of non-cohesive fill was encountered underlying the asphalt in Borehole AS-2. The non-cohesive fill extends to a depth of 2.1 m below ground surface (Elevation 99.5 m) and consists of an upper 0.7 m thick layer of gravelly sand, some silt, and a lower 1.6 m thick layer of silty sand, some gravel.

The SPT “N”-values measured within the non-cohesive fill layer range from 9 blows to 38 blows per 0.3 m of penetration, indicating a loose to dense state of compactness.

The water content measured on a sample of the non-cohesive fill is about 11 per cent.

4.2.2.5 Clayey Silt

A 0.4 m thick layer of clayey silt, trace sand, trace gravel was encountered underlying the cohesive fill in Borehole A2. This clayey silt was encountered at a depth of 1.4 m below ground surface (Elevation 100.6 m) and extended to a depth of 1.8 m below ground surface (Elevation 100.2 m).

The SPT “N”-value measured within the clayey silt layer is 35 blows per 0.3 m of penetration, suggesting a hard consistency.

The water content measured on a sample of the deposit is about 14 per cent.

Atterberg limit testing was carried out on one sample of the cohesive layer and the result is presented on Figure B-10 in Appendix B. The Atterberg limits test measured a liquid limit of 25 per cent, a plastic limit of 13 per cent, and a corresponding plasticity index of 12 per cent, indicating the cohesive deposit is clayey silt of low plasticity.

4.2.2.6 Sandy Silt to Silt and Sand Till

A 1.9 m to 4.7 m thick deposit of non-cohesive till was encountered underlying the fill / clayey silt in Boreholes SRW-1, A2 and AS-2. The non-cohesive till deposit was encountered at depths ranging from 1.4 m to 2.1 m below ground surface (Elevations 101.6 m to 99.5 m) and extends to depths ranging from 4.0 m to 13.4 m (Elevations 97.6 m to 89.6 m). The deposit consisted of sandy silt to silt and sand, trace gravel to gravelly, trace to some clay. Cobbles and boulders are commonly present in glacially derived materials and should be expected within this deposit, as inferred from auger grinding in Borehole SRW-1. In Borehole SRW-1, a 1.0 m thick clayey silt interlayer was encountered within the non-cohesive till at a depth of 6.1 m below ground surface (Elevation 96.9 m). In addition, an approximately 0.6 m and 1.6 m thick interlayer of sand and gravelly sand was encountered within the non-cohesive till deposit in Boreholes A2 and SRW-1, respectively, at depths of 4.9 m and 7.1 m below ground surface (Elevations 97.1 m and 95.9 m).

The SPT “N”-values measured within the non-cohesive till deposit range from 18 to 135 blows per 0.3 m of penetration with “N”-values up to 100 blows per 0.08 m of penetration, suggesting a compact to very dense relative density.

The water content measured on samples of the deposit range from about 6 per cent to 7 per cent.

Grain size distribution testing was carried out on four samples of the non-cohesive till deposit and the results are presented on Figure B-11 in Appendix B.

4.2.2.7 Silt and Sand

A 2.5 m and 9.2 m thick deposit of silt and sand, trace gravel, trace clay was encountered underlying the non-cohesive till in Boreholes A2 and AS-2, respectively. The silt and sand deposit was encountered at depths of 5.5 m and 4 m below ground surface (Elevations 97.6 m and 96.5 m) in the respective boreholes and extends to depths of 8.0 m and 13.2 m below ground surface (Elevations 94.0 m and 88.4 m), respectively. Borehole A2 was terminated in this deposit.

The SPT “N”-values measured within the silt and sand deposit range from 59 blows to 132 blows per 0.3 m of penetration with “N”-values up to 100 blows per 0.01 m of penetration, indicating a very dense state of compactness.

The water content measured on samples of the deposit range from about 12 per cent to 16 per cent.

Grain size distribution testing was carried out on two samples of the silt and sand, and the results are presented on Figures B-12 in Appendix B.

4.2.2.8 Clayey Silt with Sand Till

A till deposit comprised of clayey silt with sand, trace to some gravel, was encountered underlying the silt and sand and non-cohesive till deposits in Boreholes AS-2 and SRW-1, respectively. This cohesive till deposit was encountered at depths of 13.2 m and 13.4 m below ground surface (Elevations 88.4 m and 89.6 m) and extends to the borehole termination depths of 14.2 m and 15.7 m below ground surface (Elevations 87.4 m and 87.3 m) in the respective boreholes. Cobbles and boulders are in glacially derived soil deposits and should be expected within this deposit, as inferred from auger grinding in Borehole SRW-1.

The SPT “N”-values measured within the cohesive till deposit are 102 blows per 0.3 m of penetration and up to 159 blows for 0.2 m of penetration, suggesting a hard consistency.

The water content measured on samples of the deposit range between about 6 per cent and 9 per cent.

Grain size distribution testing was carried out on two samples of the cohesive till deposit and the results are presented on Figure B-13 in Appendix B.

Atterberg limit testing was carried out on two samples of the cohesive till deposit and the results are presented on Figure B-14 in Appendix B. The Atterberg limits tests measured liquid limits of 15 per cent and 20 per cent, plastic limits of 9 per cent and 10 per cent, and plasticity indices of 6 per cent and 10 per cent, indicating the till deposit is clayey silt of low plasticity.

4.3 Groundwater Conditions

Details of the water levels observed in the boreholes upon completion of drilling are summarized on the Record of Boreholes, in Appendix A. Standpipe piezometers were installed in four boreholes to allow for monitoring the groundwater level at the site along each of the proposed retaining wall alignments, as shown on the borehole records and the water levels are presented below. It should be noted that the groundwater level is subject to seasonal fluctuations and precipitation events and should be expected to be higher during wet periods of the year.

Borehole Number	Screened Stratigraphy	August 13, 2018		February 8, 2019	
		Depth (m)	Elevation (m)	Depth (m)	Elevation (m)
AS-2	Silt and Sand	7.9	93.7	6.7	94.9
NRW-1	Silt and Sand (Till)	6.8	95.4	6.3	95.9
SRW-1	Clayey Silt with Sand (Till)	8.3	94.7	8.2	94.8
SS-9	Gravelly Sand	7.9	93.2	7.0	94.1

4.4 Analytical Testing

Two soil samples were collected and submitted to Maxxam, an accredited analytical laboratory, for analyses of parameters used to assess corrosion potential and sulphate attack. A summary of the results of the analyses is presented below and the detailed test results and Certificate of Analysis Laboratory Test Results are presented in Appendix C.

Borehole Number	Sample	Sample Depth / Elevation	Soil Type	Parameters				
				Chloride (µg/g)	Sulphate (µg/g)	pH	Conductivity (mS/cm)	Resistivity (ohm-cm)
SS-8	10	7.8 / 93.4	Sand / Sand and gravel	140	140	8.12	432	2,300
AS-7	8	7.8 / 94.3	Sand and gravel	28	770	8.08	727	1,400

5.0 CLOSURE

This Foundation Investigation Report was prepared by Ms. Alysha Kobylinski, EIT, and was reviewed Mr. Christopher Ng, P.Eng., a senior geotechnical engineer and Associate with Golder. Mr. Jorge Costa, P.Eng., a MTO Foundation Designated Contact and senior consultant with Golder, conducted an independent technical and quality control review of this report.

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PART B

FOUNDATION DESIGN REPORT
RETAINING WALLS, ALBERT STREET TO SIMCOE STREET
REPLACEMENT OF THREE UNDERPASSES AND REHABILITATION OF
OSHAWA CREEK BRIDGE,
MINISTRY OF TRANSPORTATION, ONTARIO
G.W.P. 2298-13-00

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides foundation design recommendations for the construction of new retaining walls along the north and south sides of Highway 401, from Simcoe Street to about 100 m east of Albert Street, associated with the widening of Highway 401 in the City of Oshawa, Ontario. These recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the previous and current field investigations. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible retaining walls and foundation alternatives and to carry out the design of the retaining walls foundations. The foundation investigation, discussion and recommendations are intended for the use of the Ministry of Transportation, Ontario (MTO) and shall not be used or relied upon for any other purpose or by any other parties, including the construction contractor or design-build proponents. The contractor must make their own interpretation based on the factual data presented in Part A (Foundation Investigation 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 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

It is understood that as part of the improvements and widening of Highway 401 in the City of Oshawa, the existing Simcoe Street and Albert Street Underpasses will be replaced by longer span structures, and retaining walls will be constructed along the north and south sides of the existing Highway 401 in the area between the new underpasses. The north retaining wall will extend from about Simcoe Street to about 100 m east of Albert Street and the south retaining wall will extend from about 50 m east of Simcoe Street to about 55 mm east of Albert Street to allow for the construction of additional travelled lanes. The Highway 401 widening will be accommodated by cutting back into the existing cut slopes and constructing retaining walls in line with the abutments of the new underpasses.

The following summarizes the details of the proposed retaining walls:

Retaining Wall Designation	Location	Approximate Wall Length (m)	Final Grade Elevation at Top of Wall (m)	Highway 401 Grade Elevation at Toe of Wall (m)	Maximum Wall Height (m)
North Retaining Wall (NRW)	North of Highway 401, from 23 m west of Simcoe Street to 110 m east of Albert Street (STA 12+876 to STA 13+192)	276	102.5 to 101	95	8.5
South Retaining Wall (SRW)	South of Highway 401, from 95 m west of Albert Street to 55 m east of Albert Street (STA 12+970 to STA 13+134)	151	102.5 to 101	96 to 95	8.5

Based on the geometries shown on the 30% design drawings by WSP, and the subsurface conditions at the site, secant pile (caisson) walls, concrete cantilever walls and retained soil system (RSS) walls are considered as potentially feasible options. It is understood that the secant pile wall option is generally preferred as it allows for “top-down” construction methods as part of the cutting into the existing side slopes for the highway widening,

although RSS walls are being considered for sections of the south retaining wall. The concrete cantilever wall and RSS wall options may require excavations beyond the limits of the highway right-of-way, which would impact the properties along the south side of Highway 401 and affect traffic on and access from Lviv Boulevard on the north side of Highway 401. These options and their associated foundation types are discussed further in Section 6.2.

6.2 Retaining Walls and Foundations Options

This section of the report presents a detailed comparison of alternative retaining wall/foundation types based on advantages and disadvantages of wall types and provides geotechnical/foundation recommendations for the feasible types of walls and foundation alternatives for these sites. A summary comparison of feasible, retaining wall options based on geotechnical related advantages/disadvantages, relative costs, and risks/consequences is presented in Table 1, following the test of this report.

It should be noted that the selection of the type of walls and foundation alternative will also depend on factors beyond foundation recommendations. From a foundation perspective, the following retaining wall types are considered feasible alternatives.

- **Secant Pile (Caisson) Wall:** Secant pile (caisson) walls are feasible and considered the preferred wall alternative for the proposed retaining walls as competent subsurface soil deposits are present below the finished highway grade (i.e., about Elevation 95 m), providing adequate axial and lateral resistances. This retaining wall system allows for “top-down” construction as part of cut widenings. Further, the secant pile wall option does not require excavating behind the proposed wall alignment and eliminates the need for a separate temporary excavation support system during the construction. This wall type consists of king (main) caissons and soldier (secondary) caissons founded within the native soils at depths that provide sufficient axial and passive (lateral) resistances for the retained soil behind the wall and other surcharge loads applicable to the wall design. Soil anchors can be used to provide additional lateral stiffness to maintain horizontal movement within the tolerable limits. If soil anchors are adopted, the construction schedule should consider the time required for installing, pre-stressing and testing of the soil anchors. Further, the design of the soil anchors, if required, should consider the available right-of-way and the potential need for additional easement for installing the soil anchors. The “top-down” construction of the secant walls precludes placement of free-draining materials behind the wall and as such other measures must be undertaken for resisting or controlling the groundwater and frost pressures behind the wall.
- **Soldier Pile and Lagging Wall with Reinforced Concrete Facing Panels:** A soldier pile and lagging system (or similar, including proprietary cast-in-place and pre-cast panel wall systems) may be considered for the proposed retaining walls, as this type of wall is also advantageous in “top-down” construction applications, similar to the secant pile wall system. Further, the soldier pile and concrete panel wall option does not require excavating behind the proposed wall alignment and eliminates the need for a separate temporary excavation support system during the construction. However, the soldier pile and lagging wall is not as rigid as a secant wall and as such, soil anchors will likely be required to provide additional lateral stiffness. As such, the construction schedule should consider the time required for installing, pre-stressing and testing of the soil anchors, and the space requirement for installation of soil anchors. Similar to the secant pile wall option, the “top-down” construction of a soldier pile and lagging wall precludes placement of free-draining materials behind the wall and as such, other measures must be undertaken for resisting or controlling the groundwater and frost pressures behind the wall.
- **Concrete Retaining Wall on Shallow Foundations:** A concrete cantilever retaining wall supported on shallow foundations (concrete strip footing) or deep foundations (driven piles or caissons) is feasible for the

retaining walls at this site, although not the preferred option considering that temporary protection systems will be required to support adjacent property/structures/highway to allow for the construction of the retaining wall footings. Consideration should be given to the space requirements and the potential for encroaching onto Lviv Boulevard on the north side of Highway 401, as well as onto the private properties on the south of the Highway 401.

- **Reinforced Soil System (RSS) Wall:** An RSS wall with the front facing supported on shallow foundations is feasible for the proposed retaining walls, although the excavations to accommodate the retained soil mass (granular backfill and reinforcing mesh/strips) will extend behind the wall face for a distance (width) equal to approximately 80 per cent of the wall height. Such excavations would extend beyond the limits of the highway right-of-way and would require disposal of excavated soil and import of granular backfill. This option is not considered appropriate for the north retaining wall based on the space constraints in that area, but could be considered for the south retaining wall.

Based on a comparison of the advantages/disadvantages between the various wall types and supporting foundation alternatives presented in Table 1 and described above, and given the subsurface conditions encountered in the boreholes, the preferred retaining wall alternative from a geotechnical/foundations perspective for the proposed retaining walls is the secant pile wall system.

The following sections of this report present the results of the assessment/analyses of global stability for the retaining walls along the north and south sides of Highway 401 and provide a comparison of the wall/foundation alternatives and geotechnical recommendations for the preferred options.

6.3 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the 2014 Canadian Highway Bridge Design Code CAN/CSA S6-14 (*CHBDC, 2014*) and its Commentary, the retaining walls and their foundations may be classified as geotechnical systems designed for application along a transportation corridor with large traffic volumes and with potential impacts on other transportation corridors, resulting in a “typical consequence level” associated with exceeding limit states design. In addition, given the typical project specific foundation investigation carried out at this site as part of the previous and current foundation investigations (as presented in Foundation Investigation section (Part A) of the report), in comparison to the degree of site understanding in Section 6.5 of *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 ultimate limit state (ULS) and serviceability limit state (SLS) consequence factor, Ψ , and geotechnical resistance factors, ϕ_{gu} and ϕ_{gs} , from Tables 6.1 and 6.2 of the *CHBDC* (2014) have been used for design.

6.3.1 Seismic Design

6.3.1.1 Seismic Site Classification and Importance Category

The *CHBDC* (2014) states that the seismic hazard values associated with the design earthquakes should be those specified National Building Code of Canada (NBCC, 2015) as developed by the Geological Survey of Canada (GSC). The current seismic hazard maps (referred to as the 5th generation seismic hazard maps) were developed by the GSC.

Subsurface ground conditions for seismic site characterization were established based on the results of the field investigation (soil types and in situ SPT 'N' values). The average SPT 'N'-values measured in the soil layers below the founding level were used to define the seismic site classification in accordance with Table 4.1 of the *CHBDC* (2014). Based on the measured SPT 'N'-values and refusal depths in boreholes, the foundations at this site may be designed using a Site Class C designation.

6.3.1.2 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, peak ground velocity (PGV) and design spectral acceleration (Sa) values for Site Class C are presented below.

Seismic Hazard Values	10% Exceedance in 50 years (475-year return period)	5% Exceedance in 50 years (975-year return period)	2% Exceedance in 50 years (2,475-year return period)
PGA (g)	0.040	0.069	0.126
PGV (m/s)	0.032	0.052	0.089
Sa (0.2) (g)	0.068	0.112	0.197
Sa (0.5) (g)	0.044	0.067	0.110
Sa (1.0) (g)	0.025	0.037	0.059
Sa (2.0) (g)	0.012	0.018	0.029
Sa (5.0) (g)	0.0026	0.0042	0.0071
Sa (10.0) (g)	0.0011	0.0018	0.0030

6.4 Secant Pile (Caisson) Wall and Soldier Pile and Lagging Wall

Drilled shaft (caisson) walls (foundations used in a secant pile wall configuration) or a soldier pile and lagging wall with reinforced concrete facing panels (including proprietary cast-in-place and pre-cast panel wall systems) are feasible for the north and south retaining walls along the proposed cut slope for the Highway 401 widening. These walls are advantageous at this site, since they do not require excavations into the cut slope behind the proposed retaining wall alignment compared to the other wall types (i.e., for construction of spread footings for concrete cantilever or reinforced soil masses).

These wall systems consist of king (primary) caissons or soldier (secondary) caissons socketted to sufficient depth within the native soils to provide the necessary axial and passive (lateral) resistance for the retained soil height. Axial geotechnical resistance recommendations for the caissons are provided in Section 6.4.1.2 of this report. It is assumed that the king caissons will extend to a depth at least equal to the height of the wall above the finished grade in front of the wall (i.e., retained soil height/thickness). If required, additional lateral support to the wall system could be provided in the form of permanent soil anchors located at strategic locations along the retaining walls.

If the soldier pile and lagging wall option is adopted, the lagging should be installed as the excavation for the cut progresses such that the unsupported height does not exceed 1.2 m at any time, and the space behind the lagging should be immediately packed with granular material to ensure intimate contact of the soil with the back of the wall and to aid in achieving proper drainage. If sufficient thickness of free-draining granular soil for adequate drainage and frost protection cannot be provided behind the laggings, consideration should be given to using a drainage net/sheet placed against the back of the wall and connected to a positive discharge element. An insulation layer could also be provided immediately behind the wall to provide frost protection from frost penetration.

The construction of deep foundations such as caissons for secant pile wall or soldier pile wall should be carried out in accordance with OPSS.PROV 903 (*Deep Foundations*).

6.4.1 Drilled Shaft Foundations

6.4.1.1 Founding Elevations

It is anticipated that primary caissons for the secant pile walls, or soldier piles for soldier pile and lagging wall if adopted, will extend into the “100-blow” strata comprised of hard clayey silt till or very dense silt and sand, at or below Elevation 88 m to develop lateral resistance for support of the retained soil height.

6.4.1.2 Factored Geotechnical Axial Resistances

The following factored ultimate and serviceability (for 25 mm of settlement) geotechnical resistances, based on base resistances only, may be used for design of primary caissons of the secant pile wall or soldier pile wall system:

Foundation Element	Factored Ultimate Geotechnical Resistance (kN)	Factored Serviceability Geotechnical Resistance ¹ (kN)
1.2 m diameter Primary Caissons for the secant pile wall system or for the soldier pile and lagging wall	2,000	N/A
1.5 m diameter Primary Caissons for the secant pile wall system or for the soldier pile and lagging wall	3,000	N/A

Note:

1. The factored serviceability geotechnical resistance (at SLS) for 25 mm of settlement will be greater than the factored ultimate geotechnical resistance (at ULS) and as such, the SLS condition does not apply.

A liner (temporary or permanent) should be utilized to support the overburden soils during construction to minimize disturbance to the side walls. Further and considering the groundwater level measured in Boreholes SRW-1 and AS-2, a head of water/drilling slurry will be required to mitigate the base disturbance.

6.4.2 Passive Resistance for Primary Caissons/Soldier Piles

The resistance to lateral loading in front of a single primary caisson of the secant pile wall or soldier pile may be assessed using subgrade reaction theory. The coefficient of horizontal subgrade reaction, k_h (kPa/m), is determined based on the equations given below (*Canadian Foundation Engineering Manual (CFEM)*, 1992).

For non-cohesive soils:

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

Where

k_h is the coefficient of horizontal subgrade reaction (kPa/m);

n_h is the constant of subgrade reaction (kPa/m);

z is the depth (m) below the final grade in front of the wall; and,

B is the caisson diameter (m).

The values of n_h to be used to calculate the coefficient of horizontal subgrade reaction (k_h) in the structural analysis are given below, as interpreted from Reese (1974).

Soil Unit	n_h (kPa/m)
Compact non-cohesive soils	10,000
Dense to very dense non-cohesive soils	20,000

For cohesive soils:

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

where: s_u is the undrained shear strength of the soil (kPa); and,
 B is the caisson diameter (m).

The value of s_u to be used to calculate the coefficient of horizontal subgrade reaction (k_h) in the structural analysis is given below.

Soil Unit	s_u (kPa)
Hard Clayey Silt	200

The lateral response of a single primary caisson/soldier pile can be analyzed using the P-y curves. The P-y curves, representing the non-linear response of the soils under lateral loading, have been generated for 1.2 m diameter caisson/soldier pile using the commercially available program LPILE (version 2016) produced by ENSOFT Inc. The P-y curves are presented at 0.5 m depth increments for single 1,050 mm and 1,200 m diameter primary caissons in Tables 2 and 3, respectively. The P-y curves should be adjusted in accordance with Section C6.11.3.4 to account for group effects.

The ultimate passive lateral pressure in front of the primary caissons/soldier piles should not exceed the passive soil resistance of the soil between the head and toe of the secant wall. The following design parameters / values can be used to estimate the passive earth resistance in front of a single secant pile using the method outlined in Section 24.3.2 of the CFEM (2006) assuming level backfill/grade behind the wall:

- K_p the coefficient of passive earth pressure, which may be taken as 3.8, based on an angle of internal friction of 36 degrees, for the native soils below Elevation 93 m at this site. The 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); and,
- γ' the effective unit weight of the soil in front of the soldier pile, which may be taken as 10 kN/m³ below the groundwater level (assumed to be at the highway grade).

The upper zone of soil down to a depth equal to 1.5 times the caisson diameter below the finished grade in front of the wall should be ignored in the calculation of the passive resistance, to account for disturbance during installation and due to frost penetration effects.

6.4.3 Secondary (Soldier) Caissons

Based on the design drawings provided by WSP, secondary caissons will be constructed between and overlapping with the adjacent structural caissons for the secant caisson wall section. It is assumed that the secondary caissons will neither carry any vertical load nor be subjected to lateral loads, with the bearing pressure exerted due to self-weight only. The secondary caissons should be founded below any existing fill materials and loosened/softened soils within the native soils, and should be founded below the depth of frost penetration which at this site is estimated to be 1.3 m below ground surface, as interpreted from OPSD 3090.101 (*Foundations, Frost Penetration Depth for Southern Ontario*).

6.4.4 Permanent Soil Anchors

If required, additional lateral support to the wall system could be provided in the form of permanent soil anchors. The soil anchors can be designed to accommodate the loads from the soil mass behind the wall and any surcharge design loads on the final grade behind the wall. It should be noted that depending on the required anchorage length, additional easement beyond the MTO right-of-way may be required.

The bond zone for soil anchors should be located beyond the “active” earth pressure zone behind the wall, and beyond potential circular slip surfaces associated with a global stability failure. As the wall will be constructed “top down”, the vertical spacing of soil anchors must correspond to expected excavation levels to ensure that the wall (or portion of the wall during construction) remains safe at all times taking into accounts construction loads, etc.

The geotechnical capacity of soil anchors has been assessed using the method provided in the CFEM (2006). The design of soil anchors may be based on the factored ultimate value (based on a typical degree of understanding) of 100 kPa per anchor or 50 kN/m length of fixed/bond zone assuming 150 mm diameter for the anchor drill hole. The capacity of soil anchors should be verified by proof tests. The geotechnical capacity of soil anchors can be improved by multi-stage secondary grouting of the bond zone. Secondary grout pressures will need to be sufficiently high to fracture primary grout and the surrounding soil mass. The structural capacity of soil anchors should be verified by the Structural Engineer. The sustained working load should not be greater than 60 per cent of the ultimate tensile strength of the anchor tendons or bars.

The SLS value for 25 mm of displacement will not govern and may be greater than the ULS value. For design purposes an SLS value equal to the ULS value should be used.

Soil anchors should have their fixed length (bond zone) formed within the native dense to very dense non-cohesive or very stiff to hard cohesive soil deposits and should be installed at a downward angle of 5 degrees or steeper. The first row of soil anchors should be installed below the frost penetration depth (i.e., about Elevation 100.5 m). A minimum of 5.0 m of overburden is required above the tip of the fixed length (bond zone) to provide the necessary overburden pressure to develop anchor capacity in gravity-grouted soil anchors, to prevent grout leakage during installation of pressure grouted anchors and to prevent heaving of the ground surface for higher grout pressure operations (CFEM, 2006 and FHWA, 1999). The fixed length (bond zone) of the anchors should be at least 3 m (and may be up to 12 m) and be maintained at least 0.15 times wall height behind a line drawn upward at 30 degrees from the toe of the proposed wall (relative to the exposed wall height), as shown on Figure 26.16 of the CFEM (2006). The horizontal spacing between anchors will be dependent of the spacing of the primary caissons or soldier piles but should be greater than four times the anchor hole diameter (grouted section) or 1.2 m whichever is greater. The permanent soil anchors should be provided with suitable corrosion protection in accordance with requirements of Chapter 6 of the Ground Anchors and Anchored Systems (FHWA 1999). The length of the free stressing length (free zone) should be designed by the Structural Engineer.

Anchor installation, grouting and testing should be carried out in accordance with OPSS 942 (*Pre-Stressed Soil and Rock Anchors*) and SP 109S58 (Amendment to OPSS 942).

Lateral earth pressures for design of the retaining walls are discussed in Section 6.7.

6.4.5 Global Stability

Slope stability analyses have been performed for the proposed retaining walls using the commercially available program SLIDE (V.2018) produced by Rocscience Inc., employing the Morgenstern-Price method of analysis. For all analyses, the Factor of Safety (FoS) of numerous potential failure surfaces was computed in order to establish the minimum FoS. The FoS is defined as the ratio of the forces tending to resist failure to the driving forces tending

to cause failure. A target minimum factored FoS of 1.5 is adopted for the design of retaining wall height and geometry under static conditions at the end of construction as per the *CHBDC* (2014). This FoS is considered adequate for the retaining walls at this site considering the design requirements and the field data available. In general, circular slip surfaces were analysed in the design.

The following parameters have been used in the stability analyses, based on field and laboratory test data as well as accepted correlations (Bowles, 1984 and Kulhawy and Mayne, 1990):

Stratigraphic Unit	γ (kN/m ³)	ϕ^* ($^{\circ}$)
New Granular Fill	22	34
Existing Loose / Stiff to Hard Fill	19	31
Very dense silt and sand / gravelly silty sand (Till)	21	34
Dense to very dense sand to sand and gravel	22	35
Very dense silt	20	32
Hard clayey silt with sand (Till)	21	34

A maximum retained wall height of 8.5 m was assumed for both the north and south retaining walls. The groundwater level was considered at Elevation 95.9 m behind the retaining walls, as measured in the piezometers installed at the site. A live load surcharge of 18 kPa, equivalent to 0.8 m high fill, was applied in accordance with Section 6.12.5 of the *CHBDC* (2014).

The stability analysis indicates that the proposed caisson retaining walls will have a FoS equal to or greater than 1.5 against global instability. An example of the static global stability analysis is provided on Figure 1.

It is understood that a temporary excavation will be required in front (south) of and parallel to the north retaining wall, for installation of a future storm sewer. Details on the excavation for this installation are not known at this time but assuming a 2 m deep, 3 m wide temporary excavation located immediately in front of the wall, with primary caissons founded at or below the design Elevation 88 m and secondary caissons founded 2 m below the highway grade, the global stability analysis indicates that the FoS against global instability for the temporary condition is equal to 1.4 (as shown on Figure 2). Considering the temporary nature of the excavations, it is recommended that the current design includes provisions for extending the secondary caissons to sufficient depths below the base of the future temporary excavations such that the temporary excavations do not undermine the secant pile wall.

6.4.6 Frost Protection and Drainage Requirements for Secant Wall

The secant pile wall will be subjected to freezing ambient temperatures at the wall face as the interface will be in direct contact with the retained ground behind it. Concrete can potentially serve as a thermal conductor and unless insulation is provided at the wall face, the freezing temperatures will result in ice lenses forming and frost pressures developing behind the wall. The design of the secant wall should consider the frost, if applicable, and hydrostatic pressures behind the wall.

For secant walls, drainage may be achieved using prefabricated vertical drainage elements placed behind the caissons at a spacing of not greater than 1.5 m. The vertical drains should be connected to weep holes installed through the secondary caissons. To prevent loss of the soils behind the secant wall through the weep holes, perforated pipes encased in Class II non-woven geotextiles with Filtration Opening Size (FOS) of no greater than 150 μ m should be installed in each weep hole. Further and considering that free draining materials cannot be

placed behind the secant wall, it is recommended that the secant wall be designed for a design static groundwater level at Elevation 97.0 m.

6.4.7 Frost Protection and Drainage Requirements for Soldier Pile and Lagging Wall

Soldier pile and lagging systems will be subjected to freezing ambient temperatures at the wall face as the interface will be in direct contact with the retained ground behind it. Concrete can potentially serve as a thermal conductor and unless insulation is provided behind the concrete panels, the freezing temperatures will result in ice lenses forming and frost pressures developing behind the wall. The design and construction of the soldier pile and lagging should consider including insulation onto the back of the wall facing panels to protect the retained soil from freezing.

For soldier pile and lagging walls, drainage can be achieved using prefabricated drainage elements placed behind the permanent facing. Where precast panels are used, the space between the temporary wall face and the back of the permanent facing may be backfilled with a drainage element consisting of an approved granular drainage layer or pre-fabricated drainage material. Water intercepted by the drainage layer will flow downward to the base of the wall where it can be removed by sub-drains or conveyed through the permanent facing by means of weep holes.

6.5 Concrete Cantilever Retaining Walls

6.5.1 Founding Elevations

Strip footing (shallow) foundations are feasible for supporting the proposed north and south retaining walls and should be founded below any fill or softened/loosened surficial soils. The strip footings for concrete cantilever retaining walls, if adopted, should be founded on the very dense non-cohesive soils. All footings should be founded at a minimum depth of 1.3 m below the adjacent final grade to provide adequate protection against frost penetration, as interpreted from accordance with OPSD 3090.101 (*Foundation, Frost Penetration Depths for Southern Ontario*).

The non-cohesive till subgrade will be susceptible to disturbance and degradation on exposure to water and construction traffic. It is recommended that a concrete working slab be placed on the prepared subgrade if footing construction is not carried out within four hours following inspection and approval of the subgrade, to protect the subgrade from softening. A sample NSSP is included for this item in Appendix D for inclusion into the Contract Documents.

The following founding elevations for the retaining walls are recommended for strip footings founded on a competent native stratum:

Retaining Wall Site	Approximate Station	Founding Stratum	Highest Founding Elevation (m)*
North Retaining Wall	12+876 to 13+192	Very dense sand to sand and gravel; sandy silt to silty sand (till); very dense sandy silt	93.7
South Retaining Wall	12+970 to 13+134	Very dense silt and sand; sandy silt (till)	93.7

* The highest founding elevations provided are based on the finished highway grade the minimum soil cover depth of 1.3 m against frost penetration.

The footing subgrade should be inspected by qualified geotechnical personnel following excavation, in accordance with OPSS 902 (*Excavating and Backfilling Structures*) to check that all existing fill and/or other unsuitable material

have been removed. Where sub-excavation of fill or unsuitable materials is required, the sub-excavated area could be backfilled with granular material meeting OPSS.PROV 1010 (*Aggregates*) Granular 'A' or Granular 'B' Type II that is placed and compacted in accordance with OPSS.PROV 501 (*Compacting*), or the thickness of the footing increased to the full excavation depth.

6.5.2 Factored Geotechnical Resistance

Strip footings constructed on the properly prepared subgrade, at or below the design elevations given in the Section 6.5.1, should be designed based on the factored ultimate geotechnical resistance and the factored serviceability geotechnical resistance (for 25 mm of settlement) given below.

Retaining Wall	Assumed Footing Width (m)	Factored Ultimate Geotechnical Resistance (kPa)	Factored Serviceability Geotechnical Resistance (kPa) (for 25 mm of Settlement)
North Retaining Wall	3.0	600	450
South Retaining Wall	3.0	600	450

If the strip footings are founded at higher elevations on a compacted Granular 'A' or Granular 'B' Type II pad that is not less than 2 m thick, a factored geotechnical ultimate resistance of 700 kPa and a factored serviceability geotechnical resistance (for 25 mm of settlement) of 350 kPa could be employed for the design of the retaining wall foundations.

The geotechnical resistances provided above are given for loads applied perpendicular to the surface of the footing. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with Sections 6.10.4 of the *CHBDC* (2014).

6.5.3 Resistance to Lateral Loads

Resistance to lateral forces / sliding between the concrete footings and the subgrade should be calculated in accordance with Section 6.10.5 of the *CHBDC* (2014). The following presents the coefficient of friction, $\tan \delta$ or $\tan \phi$, for cast-in-place concrete footings constructed on a concrete working slab and cast-in-place concrete footing or working slab on very dense non-cohesive soils or granular pad as interpreted from Naval Facility Engineering Command (NAVFAC, 1986).

Subgrade Material	$\tan \phi'$
Cast-in-place footing to concrete working slab	0.70
Cast-in-place concrete footing or working slab to native very dense non-cohesive soils or granular pad	0.50

6.5.4 Global Stability

Static global stability analyses of the proposed concrete retaining walls supported on shallow foundations were completed using the parameters outlined in Section 6.4.5. A maximum retained soil height of 8.5 m was assumed in the analyses. The groundwater level was considered at Elevation 95.9 m behind the retaining walls, as measured in the piezometers installed at the site. A live load surcharge of 18 kPa, equivalent to 0.8 m high fill, was applied in accordance with Section 6.12.5 of the *CHBDC* (2014).

The stability analysis results indicate that the proposed concrete retaining wall founded on shallow foundations will have a FoS equal to 1.5 against global instability.

6.6 Retained Soil System (RSS) Walls

It is understood that mechanically-reinforced soil retaining systems (retained soil system or RSS walls) are considered to be an option for the south retaining wall. The RSS wall must be designed for high performance and appearance in accordance with MTO Special Provision (SP) 599S22 (Retained Soil System).

As this wall will be constructed by cutting into the existing side slope, the back of the reinforced soil mass should be keyed into the existing embankment by benching, as per OPSD 208.010 (Benching of Earth Slopes). Consideration should be given to the required space behind the RSS wall to place granular backfill and reinforcing straps.

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 Granular 'A', placed in accordance with OPSS.PROV 501 (Compacting), as detailed in Figure 5.2 of MTO's *RSS Wall Design Guidelines* (September 2008). There should be a minimum of 300 mm of Granular A below the facing footing. The compacted granular levelling pad should extend at least 1.0 m beyond the outside edge of the facing footing, then downward and outward at 1H:1V.

As shown 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 base of the RSS wall. The minimum soil cover to the base of the wall and top of the footing/leveling pad should be 0.5 m below the finished grade in front of the base of the RSS wall. Prior to placement of the levelling pad and the reinforced soil mass, any existing topsoil, organic and deleterious materials must be removed and the existing fill is required to be proof-rolled to identify any softened/disturbed areas for sub-excavation and replacement, where applicable, with compacted OPSS.PROV 1010 Granular 'A', placed in accordance with OPSS.PROV 501 (Compacting).

The top of the 500 mm thick Granular 'A' levelling pad/facing footing and reinforced soil mass are recommended to be founded no higher than the maximum founding elevations in the table below. Depending on the final grade at the base of the RSS wall, the levelling pad may need to be installed below these recommended elevations to achieve the minimum embedment depth of 1.0 m between the underside of the levelling pad and the finished grade.

Retaining Wall Site	Maximum (Highest) Founding Elevation (m)	Anticipated Bearing Soil
South Retaining Wall	96.0	Very dense gravelly sand; very dense silt and sand

6.6.2 Factored Geotechnical Resistance

Assuming that the RSS wall acts as a unit and uses the full width of the reinforced soil mass (which can be taken as approximately 0.8 times the wall height), the factored ultimate and serviceability geotechnical resistances given below may be used for assessment of the reinforced mass founded on the properly prepared compacted granular fill, or on the proof-rolled subgrade at the highest founding elevations provided in Section 6.6.1. The reinforced soil mass itself, founded at or below the design elevations given above, should be designed based on the factored

ultimate geotechnical resistance and the factored serviceability geotechnical resistance (for 25 mm of settlement) given below.

Retaining Wall	Recommended Minimum Strip Length (m)	Factored Ultimate Geotechnical Resistance (kPa)	Factored Serviceability Geotechnical Resistance (kPa) (for 25 mm of Settlement)
South Retaining Wall	Approximately 0.8 times the wall height	850	400

The geotechnical resistances provided above are given for loads applied perpendicular to the surface of the subgrade/facing footing. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be considered in accordance with Sections 6.10.4 of the *CHBDC* (2014).

6.6.3 Resistance to Lateral Loads

Resistance to lateral forces / sliding between the 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, $\tan \phi$, between the compacted granular fill of the RSS wall and the properly prepared subgrade is provided below. The coefficient of friction value should be reviewed and revised, if necessary, by the proprietary RSS wall designer.

Subgrade Material	$\tan \phi$
Compacted granular fill on native very dense non-cohesive soils	0.55

6.6.4 Global Stability

Static global stability analyses of the proposed RSS retaining walls were completed using the parameters outlined in Section 6.4.5. A maximum retained soil height of 8.5 m was assumed in the analyses. The groundwater level was considered at Elevation 95.9 m behind the retaining walls, as measured in the piezometers installed at the site.

The stability analysis results indicate that the proposed concrete retaining wall founded on shallow foundations will have a FoS equal to or greater than 1.5 against global instability. The design of the proposed retaining walls should consider the future temporary excavations for the installation of a future storm sewer.

6.7 Lateral Earth Pressures for Design

The lateral earth pressures acting on the retaining walls will depend on the type and method of placement of backfill materials, the nature of the soils behind the backfill, the magnitude of the surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls.

The following recommendations are made concerning the design of the walls. These design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- 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 concrete cantilever walls. A prefabricated drainage element should be attached to the back of secant pile walls or soldier pile and concrete panel walls. Longitudinal drains or weep holes should be installed to provide positive drainage of the granular backfill or the prefabricated drainage element. Aspects of the granular backfill requirements with respect to subdrains

and frost taper for a concrete cantilever wall should be in accordance with OPSP 3121.150 (*Walls, Retaining, Backfill, Minimum Granular Requirement*), and OPSP 3190.100 (*Walls, Retaining and Abutment, Wall Drain*).

- Where granular backfill is used behind the retaining walls, 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. Compaction should be carried out in accordance with OPSS.PROV 501 (*Compacting*). Care must be taken during the compaction operation not to overstress the wall, with limitations required on heavy construction equipment and requirements for the use of hand-operated compaction equipment per OPSS.PROV 501 (*Compacting*). Other surcharge loadings should be accounted for in the design, as required.
- For concrete cantilever retaining walls, if the wall is designed as a restrained wall, granular fill should be placed in a zone with the width equal to at least 1.3 m behind the back of the wall in accordance with Figure C6.20(a) of the *Commentary to the CHBDC* (2014). If the concrete cantilever wall is design as an unrestrained walls, fill should be placed within the wedge-shaped zone defined by a line drawn or flatter than at 1.1 horizontal to 1 vertical (1.1H:1V) extending up and back from the rear face of the footing or pile cap in accordance with Figure C6.20(b) of the *Commentary to the CHBDC* (2014).

6.7.1 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 design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

For restrained walls, the pressures acting on the back of the wall are based on the existing native soil or proposed backfill behind the free draining granular backfill zone or the prefabricated drainage element and the following parameters (unfactored) may be used:

Fill Type	Unit Weight of Material (kN/m ³)	Coefficients of Static Lateral Earth Pressure	
		At-Rest, K_o	Active, K_a
Select Subgrade Material	20	0.47	0.31
Existing Fill	19	0.48	0.32
Compact to very dense Silty Sand to silt and sand to silty sand (Till)	21	0.46	0.29
Very dense silt and sand/ very dense sand and gravel	21	0.44	0.28

For an unrestrained wall, in the case of the cantilever wall option, 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/m ³	0.43	0.27
Granular 'B' Type II	21 kN/m ³	0.43	0.27

If the wall support allows for lateral yielding, active earth pressures may be used in the geotechnical design of the retaining wall. 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 of the CHBDC* (2014).

If the wall does not allow lateral yielding (i.e., restrained structure where the rotational or horizontal movement is not sufficient to mobilize an active earth pressure condition), at-rest earth pressures (plus any compaction surcharge) should be assumed for geotechnical design.

6.7.2 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 retaining walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced earth pressure.
- In accordance with Sections 4.6.5 and C.4.6.5 of the *CHBDC* (2014) and its *Commentary*, for structures which allow lateral yielding, the horizontal seismic coefficient, k_h , used in the calculation of the seismic active pressure coefficient, is taken as 0.5 times the site-specific PGA. For structures that do not allow lateral yielding, k_h is taken as equal to the site-specific PGA. For both cases the value of the vertical seismic coefficient k_v is taken as zero.
- The following seismic active pressure coefficients (K_{AE}) may be used in design; these coefficients reflect the maximum K_{AE} obtained for each of the earthquake design periods and backfill conditions. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is level. Where sloping backfill is present above the top of the wall, the lateral earth pressures under seismic loading conditions should be calculated by treating the weight of the backfill located above the top of the wall as a surcharge.
- The K_{AE} value for a yielding wall is applicable provided that the wall that can move up to $250 \cdot k_h$ (measured in mm), where k_h is the site-specific PGA as given above. This corresponds to displacements of about 10 mm, 18 mm, and 32 mm for the 475-year, 975-year, and 2,475-year design earthquakes at this site.
- The earthquake-induced pressure distribution, which should 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).

Wall Type	Design Earthquake	Site PGA	Seismic Active Pressure Coefficients, K_{AE}			
			Granular 'A' / Granular 'B' Type II	SSM / Existing Fill	Compact to very dense silty sand to silt and sand to silty sand (Till)	Very dense silt and sand/very dense sand and gravel
Restrained Wall (Non-Yielding)	475-Year	0.040g	0.27	0.33	0.31	0.28
	975-Year	0.069g	0.29	0.35	0.33	0.30
	2,475-Year	0.126g	0.32	0.39	0.37	0.34
Unrestrained Wall (Yielding)	475-Year	0.040g	0.26	0.31	0.30	0.27
	975-Year	0.069g	0.27	0.32	0.31	0.28
	2,475 Year	0.126g	0.28	0.34	0.33	0.29

6.8 Construction Considerations

6.8.1 Open-Cut Excavations and Temporary Protection Systems

It is understood that open-cut excavations are not anticipated considering that the preferred retaining wall option would allow for the “top-down” construction techniques which preclude the need for excavations. Where required, such as in the case that a cantilever retaining wall option is adopted, excavations must be carried out in accordance with Ontario Regulation 213 of the Ontario Occupational Health and Safety Act for Construction Projects (OHSA), as amended. The soils to be excavated can be classified according to OHSA as Type 3 soils. Temporary open-cut excavations (i.e., those open for a relatively short time period) should be made with side slopes no steeper than 1H:1V based on the soil profile.

If required, temporary protection systems should be designed and constructed in accordance with OPSS.PROV 539 (Temporary Protection System), and the lateral movement should meet Performance Level 1b given the presence of nearby residential/commercial dwellings. Existing adjacent utilities should be checked to ensure that they can tolerate this magnitude of deformation. The selection, design and performance of the protection system will be the responsibility of the Contractor.

6.8.2 Control of Groundwater and Surface Water

The groundwater level in the standpipe piezometers installed at the time of investigation ranged from Elevations 95.9 m to 93.2 m; however, it is expected that the groundwater level will be higher during periods of heavy/sustained precipitation or during the wet seasons.

Excavations that extend below the groundwater level will require dewatering measures to ensure the retaining walls can be constructed in dry conditions. It is anticipated that dewatering measures will only be required for construction of spread / strip footings for the concrete cantilever wall. The control of water from dewatering operations should be managed in accordance with OPSS.PROV 517 (Dewatering), and SP 517F01 (Dewatering System; Temporary Flow Passage System). A copy of SP 517F01 is provided in Appendix D, with the applicable foundation-related fill-in completed; the drainage engineers should supply the applicable design storm information, for inclusion into the Contract Documents.

Construction water takings in excess of 50,000 L/day are regulated by the Ministry of the Environment, Conservation and Parks (MECP). Certain takings of groundwater for construction dewatering purposes with a combined total less than 400,000 L/day qualify for self-registration on the MECP's Environmental Activity and Sector Registry (EASR). Registry on the EASR replaces the need to obtain a PTTW for water taking and a Section 53 approval for discharge of water to the environment. A "Water Taking Plan" and a "Discharge Plan" are required by the MECP if water is taken in accordance with an EASR. In all cases, discharge under the EASR must be in accordance with a Discharge Plan (to be developed by a qualified professional). The contractor will be responsible for obtaining any required discharge approvals. A Category 3 PTTW would be required for water takings in excess of 400,000 L/day.

Depending on the final depth to which excavations are required to be made, an EASR or PTTW may be required and a hydrogeological assessment should be conducted to estimate the expected construction water extraction requirements, assist in registration, and to provide the required documentation.

Surface water should be directed away from the excavations at all times.

6.8.3 Obstructions

The native site soils are glacially derived and as such should be expected to contain cobbles and boulders as inferred present from auger grinding in Boreholes S1 and AS-7 which could affect the installation main caissons for the secant wall or soldier piles. Similarly, the existing fill may contain cobble and boulder size materials. The construction equipment should be capable of advancing the liner through such obstructions.

It is recommended that a Non-Standard Special Provision be included in the contract documents to address obstructions, and an example NSSP is provided in Appendix D.

6.8.4 Analytical Testing of Construction Materials

The summary results of analytical tests carried out on two soil samples (sand to sand and gravel) and are presented in Section 4.4 and on the Certificate of Analysis in Appendix C. The analytical test results for sulphate were compared to CSA A23.1 Table 3 (*Additional requirements for concrete subjected to sulphate attack*) to assess the potential severity of sulphate attack on concrete during its service life. The sulphate concentrations measured on the soil samples are less than 0.1 per cent, which is below the Moderate degree of exposure (i.e., below the Class S3 exposure limits), and the degree of sulphate attack is considered "Negligible" according to Table 7.2 in MTO's Gravity Pipe Design Guidelines (2014). Therefore, based on the two soil samples tested, when the designer is selecting the exposure class for the concrete structure, the effects of sulphates from within the site soils in contact with any portion of the proposed structure constructed below the ground surface may not need to be considered.

The analytical test results of the soil samples for resistivity were also compared to Table 3.2 of MTO's Gravity Pipe Design Guidelines (2016), to assess the relative level of corrosion potential on buried steel in contact with soil. The resistivity values measured on the soil samples from Borehole SS-8 and Borehole AS-7, are 2,300 ohm-cm and 1400 ohm-cm indicating a "Moderate to severe corrosiveness" and "corrosive" potential, respectively. Given that the proposed structure will be exposed to de-icing salt/chemicals, consideration should be given by the designer to designing the concrete structure for a "C" type exposure class as defined by CSA A23.1 Table 1.

It is also noted that the measured pH levels were about 8.1, suggesting the presence of alkaline soils.

Ultimately, it is the structural designer's decision to determine the appropriate exposure class and to ensure that all aspects of CSA A23.1 Section 4.1.1 (Durability Requirements) are satisfied.

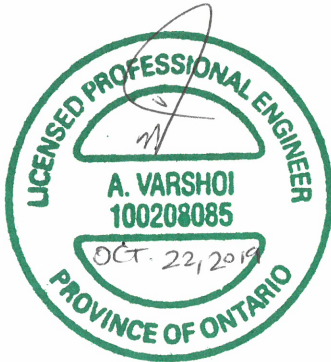
6.8.5 Monitoring Well Decommissioning

Four standpipe piezometers were installed on the site to permit monitoring of the groundwater levels at the site, specifically in Boreholes AS-2, NRW-1, SRW-1 and SS-9. Ontario Regulation (O.Reg.) 903 amended by O.Reg. 128/03 of the Ontario Water Resources Act requires that monitoring wells are properly abandoned/decommissioned by qualified personnel. We recommend that the decommissioning of the standpipe piezometers be carried out as part of the construction activities at the site so that water level measurements can be taken during the detailed design phase as well as immediately prior to construction.

7.0 CLOSURE

This Foundation Design Report was prepared by Mr. Al Varshoi, P.Eng., a geotechnical engineer with Golder, and reviewed by Mr. Christopher Ng, P.Eng., a geotechnical engineer and Associate with Golder. Mr. Jorge Costa, P.Eng., Golder's MTO Foundations Designated Contact for Golder, conducted an independent technical and quality control review of this report.

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Ontario Occupational Health and Safety Act:

Ontario Reg. 213 Construction Projects (as amended)

Ontario Provincial Standard Specifications (OPSS)

OPSS.PROV 501 Construction Specification for Compacting

OPSS.PROV 518	Construction Specification for Control of Water from Dewatering Measures
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS 902	Construction Specification for Excavating and Backfilling-Structures
OPSS.PROV 903	Construction Specification for Deep Foundations
OPSS 942	Construction Specification for Pre-stressed Soil and Rock Anchors
OPSS.PROV 1010	Construction Specification for Material Specification for Aggregates – Base, Subbase, Select Subgrade, and Backfill Material

Ontario Provincial Standard Drawings (OPSD)

OPSD 3090.101	Foundation, Frost Penetration Depths for Southern Ontario
OPSD 3121.150	Walls, Retaining, Backfill, Minimum Granular Requirements
OPSD 3190.100	Walls, Retaining and Abutments, Walls

Special Provision

SP 109S58	Amendment to OPSS 942
SP 517F01	Amendment to OPSS.PROV 517

Ontario Water Resources Act

Ontario Regulation 903	Wells (as amended)
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Table 1: Comparison of Retaining Wall Options – Highway 401 Widening

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks / Consequences
Secant pile wall	Feasible	<ul style="list-style-type: none"> Minimizes excavation into the existing cut slopes and eliminates requirement for protection systems adjacent to the travelled lanes during construction staging. Greater geotechnical resistances available than shallow foundations. 	<ul style="list-style-type: none"> Easement for soil anchors may be required depending on distance from wall to property limits. cobbles and boulders maybe encountered in the till deposits, which could impact drillhole advancement and installation of the secant piles/caissons. Likely more time-consuming to install than other wall types due to steps involved (pre-augering for socket holes, placing soldier piles, placing backfill in lifts, installing laggings, installing, pre-stressing and testing soil anchors). Free draining materials cannot be placed behind the wall. Other measures should be adopted to address hydrostatic pressure behind the wall. 	<ul style="list-style-type: none"> Lower than concrete retaining walls and temporary protection systems. 	<ul style="list-style-type: none"> If soil anchors are required, a wider right-of-way may be required to accommodate them. Least demanding on right-of-way space if soil anchors not required.
Soldier Pile and Lagging Wall	Feasible	<ul style="list-style-type: none"> Conventional construction methods for caissons/pile installation. Greater geotechnical resistances available than shallow foundations. 	<ul style="list-style-type: none"> Concrete panels may not meet desired aesthetic requirements. Cobbles and boulders maybe encountered in the till deposits, which could impact drillhole advancement and installation of the soldier piles. 	<ul style="list-style-type: none"> Comparable to secant pile wall. 	<ul style="list-style-type: none"> If soil anchors are required right-of-way may be required for them.
Concrete retaining wall on shallow foundations	Feasible, but less advantageous compared to secant pile or soldier pile and lagging walls	<ul style="list-style-type: none"> Conventional excavation and construction techniques. Lower geotechnical resistances available augered to deep foundation units (secant/soldier pile walls). 	<ul style="list-style-type: none"> Requires sub-excavation below existing fill and into the existing cut. Deeper excavation required than for secant walls or soldier pile and concrete lagging wall necessitating more robust protection system adjacent to the highway, and potentially requiring a protection system along the excavation. Wider/larger excavation area required than for secant pile or soldier pile and lagging wall. 	<ul style="list-style-type: none"> Comparable cost to secant pile wall. 	<ul style="list-style-type: none"> Temporary protection systems will be required.
Retained Soil System	Feasible for south retaining wall. Not feasible for the north retaining wall due to limited right-of-way.	<ul style="list-style-type: none"> Relative ease of construction. Aesthetically appealing panels are available. 	<ul style="list-style-type: none"> Large excavation will be required than (similar to excavation area required for concrete retaining wall). Proprietary product is required, with specialized design of internal stability of the proprietary system. 	<ul style="list-style-type: none"> Lower than concrete walls or secant walls 	<ul style="list-style-type: none"> Temporary protection systems will be required.

SUMMARY OF P-y CURVES FOR A 1,050 mm Single Caisson - Static Loading Condition - Retaining Walls

Description Depth (z) * Elevation P-y Curves	Very Dense Sandy Silt to Silty Sand (Till) to Sand to Sand and Gravel																	
	z= 5 m		z= 1.0 m		z= 1.5 m		z= 2.0 m		z= 2.5 m		z= 3.0 m		z= 3.5 m		z= 4.0 m		z= 4.5 m	
	Elev.= 94.5 m		Elev.= 94.0 m		Elev.= 93.5 m		Elev.= 93.0 m		Elev.= 92.5 m		Elev.= 92.0 m		Elev.= 91.5 m		Elev.= 91.0 m		Elev.= 90.5 m	
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.001	15.336	0.002	34.093	0.002	52.244	0.002	65.767	0.001	70.636	0.001	79.162	0.001	103.448	0.002	130.904	0.002	161.529	0.002
0.003	29.561	0.003	65.715	0.003	100.704	0.003	126.770	0.003	136.155	0.002	152.588	0.003	199.402	0.003	252.325	0.003	311.357	0.004
0.004	41.865	0.005	93.066	0.005	142.617	0.005	179.531	0.004	192.822	0.004	216.095	0.004	282.392	0.005	357.342	0.005	440.943	0.005
0.006	51.878	0.006	115.326	0.006	176.730	0.006	222.474	0.005	238.944	0.005	267.783	0.005	349.938	0.006	442.815	0.007	546.413	0.007
0.007	59.633	0.008	132.565	0.008	203.147	0.008	255.729	0.007	274.661	0.006	307.811	0.007	402.247	0.008	509.006	0.008	628.090	0.009
0.008	65.410	0.009	145.408	0.010	222.827	0.009	280.503	0.008	301.270	0.007	337.632	0.008	441.216	0.009	558.318	0.010	688.938	0.011
0.010	69.591	0.011	154.702	0.011	237.070	0.011	298.433	0.009	320.526	0.009	359.213	0.010	469.418	0.011	594.005	0.012	732.974	0.013
0.011	72.554	0.013	161.288	0.013	247.163	0.012	311.137	0.010	334.172	0.010	374.505	0.011	489.402	0.012	619.293	0.013	764.178	0.015
0.013	74.622	0.014	165.886	0.014	254.208	0.014	320.006	0.012	343.697	0.011	385.180	0.012	503.352	0.014	636.946	0.015	785.961	0.016
0.014	76.051	0.016	169.062	0.016	259.075	0.015	326.133	0.013	350.278	0.012	392.555	0.014	512.989	0.015	649.141	0.017	801.009	0.018
0.016	77.031	0.017	171.240	0.018	262.413	0.017	330.335	0.014	354.791	0.013	397.612	0.015	519.598	0.017	657.504	0.018	811.328	0.020
0.017	77.699	0.019	172.726	0.019	264.691	0.018	333.202	0.016	357.870	0.015	401.063	0.016	524.108	0.018	663.211	0.020	818.371	0.022
0.018	78.154	0.020	173.737	0.021	266.240	0.020	335.152	0.017	359.965	0.016	403.411	0.018	527.176	0.020	667.092	0.021	823.160	0.024
0.020	78.462	0.022	174.423	0.022	267.291	0.021	336.475	0.018	361.386	0.017	405.003	0.019	529.257	0.021	669.726	0.023	826.410	0.026
0.021	78.671	0.024	174.888	0.024	268.003	0.023	337.372	0.020	362.348	0.018	406.082	0.020	530.667	0.023	671.510	0.025	828.611	0.027
0.023	78.813	0.025	175.202	0.026	268.485	0.024	337.978	0.021	363.000	0.019	406.812	0.022	531.621	0.024	672.717	0.026	830.101	0.029

Description Depth (z) * Elevation P-y Curves	Hard Clayey Silt with Sand																	
	z= 5.5 m		z= 6.0 m		z= 6.5 m		z= 7.0 m		z= 7.5 m		z= 8.0 m		z= 8.5 m		z= 9.0 m		z= 9.5 m	
	Elev.= 89.5 m		Elev.= 89.0 m		Elev.= 88.5 m		Elev.= 88.0 m		Elev.= 87.5 m		Elev.= 87.0 m		Elev.= 86.5 m		Elev.= 86.0 m		Elev.= 85.5 m	
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.002	212.440	0.002	247.328	0.002	284.858	0.002	325.032	0.002	367.849	0.003	413.309	0.003	461.413	0.003	512.159	0.003	565.549	0.003
0.004	409.491	0.004	476.738	0.004	549.080	0.005	626.518	0.005	709.050	0.005	796.677	0.005	889.399	0.006	987.216	0.006	1090.128	0.006
0.006	579.919	0.006	675.155	0.007	777.606	0.007	887.272	0.007	1004.154	0.008	1128.251	0.008	1259.564	0.009	1398.092	0.009	1543.835	0.009
0.008	718.632	0.008	836.647	0.009	963.603	0.009	1099.501	0.010	1244.340	0.010	1398.120	0.011	1560.842	0.012	1732.504	0.012	1913.109	0.013
0.010	826.052	0.010	961.708	0.011	1107.641	0.012	1263.853	0.012	1430.342	0.013	1607.109	0.014	1794.154	0.014	1991.477	0.015	2199.078	0.016
0.012	906.079	0.013	1054.876	0.013	1214.948	0.014	1386.293	0.015	1568.911	0.016	1762.804	0.016	1967.969	0.017	2184.409	0.018	2412.121	0.019
0.014	963.994	0.015	1122.302	0.016	1292.605	0.016	1474.903	0.017	1669.194	0.018	1875.479	0.019	2093.759	0.020	2324.033	0.021	2566.300	0.022
0.016	1005.033	0.017	1170.081	0.018	1347.634	0.019	1537.692	0.020	1740.255	0.021	1955.322	0.022	2182.894	0.023	2422.971	0.024	2675.553	0.025
0.018	1033.681	0.019	1203.434	0.020	1386.049	0.021	1581.524	0.022	1789.861	0.024	2011.059	0.025	2245.118	0.026	2492.038	0.027	2751.820	0.028
0.020	1053.472	0.021	1226.475	0.022	1412.585	0.024	1611.803	0.025	1824.128	0.026	2049.561	0.027	2288.102	0.029	2539.749	0.030	2804.505	0.031
0.022	1067.044	0.023	1242.276	0.024	1430.784	0.026	1632.569	0.027	1847.630	0.029	2075.967	0.030	2317.581	0.032	2572.470	0.033	2840.637	0.035
0.023	1076.306	0.025	1253.059	0.027	1443.203	0.028	1646.740	0.030	1863.667	0.031	2093.986	0.033	2337.697	0.035	2594.799	0.036	2865.293	0.038
0.025	1082.605	0.027	1260.392	0.029	1451.650	0.031	1656.377	0.032	1874.574	0.034	2106.241	0.036	2351.378	0.037	2609.985	0.039	2882.062	0.041
0.027	1086.879	0.029	1265.368	0.031	1457.380	0.033	1662.916	0.035	1881.974	0.037	2114.556	0.038	2360.661	0.040	2620.289	0.042	2893.440	0.044
0.029	1089.774	0.031	1268.739	0.033	1461.263	0.035	1667.346	0.037	1886.988	0.039	2120.189	0.041	2366.950	0.043	2627.269	0.045	2901.148	0.047
0.031	1091.734	0.033	1271.020	0.036	1463.890	0.038	1670.344	0.040	1890.381	0.042	2124.001	0.044	2371.205	0.046	2631.993	0.048	2906.364	0.050

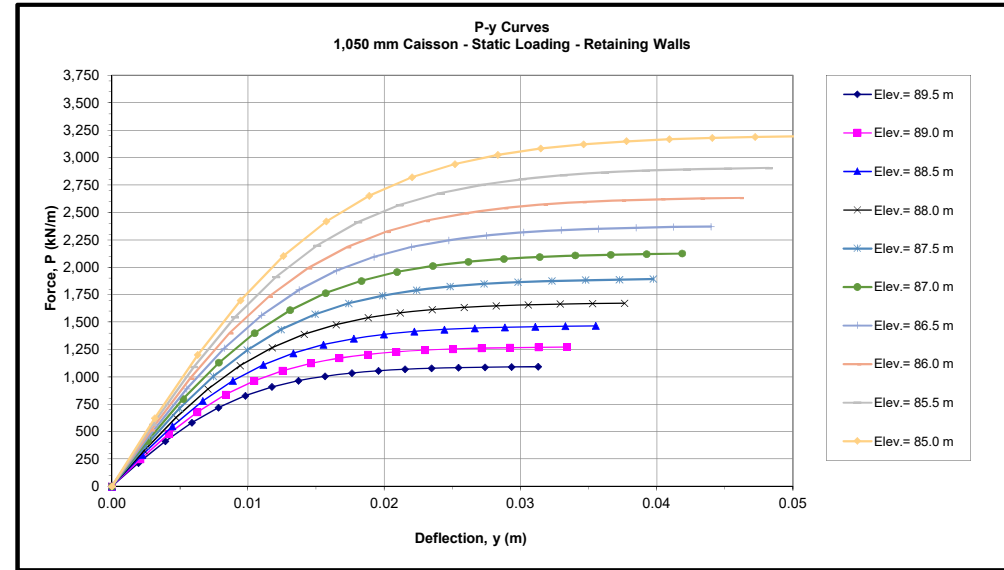
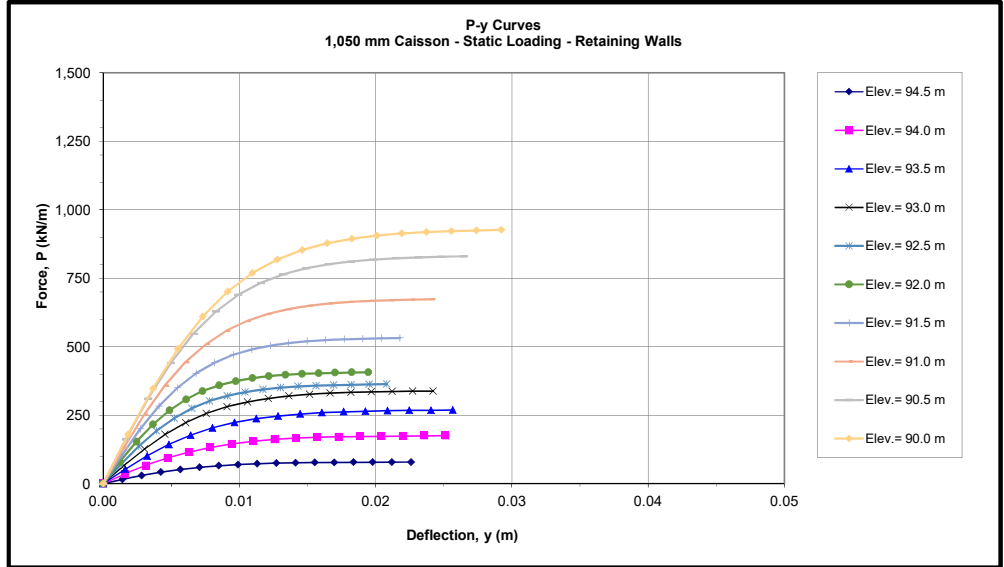
NOTES: * Depth (z) is measured to be positive below the proposed front face elevation (i.e. Elevation 95 m).

The P-y curves have been generated based on the following assumptions:

1. P-y curves are generated for vertical caissons (i.e. no inclination)
2. Static loading condition is considered.
3. There are no pile group effects (i.e. analysis is based on a single pile).
4. P-y curves represent the resistance from the mass of soil behind the caissons in the passive earth pressure mode.

P-y CURVES
Highway 401 Oshawa - Secant Retaining walls - P-y Curves
1050 mm Diameter Caisson

Table 2



Date: October 2019
Project No: 1662582

Prepared By: ARV
Checked By: CN



SUMMARY OF P-y CURVES FOR A 1,200 mm Single Caisson - Static Loading Condition - Retaining Walls

Description Depth (z) * Elevation P-y Curves	Very Dense Sandy Silt to Silty Sand (Till) to Sand to Sand and Gravel																	
	z= .5 m		z= 1.0 m		z= 1.5 m		z= 2.0 m		z= 2.5 m		z= 3.0 m		z= 3.5 m		z= 4.0 m		z= 4.5 m	
	Elev.= 94.5 m		Elev.= 94.0 m		Elev.= 93.5 m		Elev.= 93.0 m		Elev.= 92.5 m		Elev.= 92.0 m		Elev.= 91.5 m		Elev.= 91.0 m		Elev.= 90.5 m	
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
P-y Curves	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	0.002	17.175	0.002	38.273	0.002	59.773	0.002	78.153	0.002	89.892	0.001	91.467	0.001	107.133	0.002	135.116	0.002	185.270
	0.003	33.106	0.004	73.774	0.004	115.216	0.004	150.645	0.003	173.271	0.003	176.308	0.003	206.506	0.003	260.443	0.003	357.119
	0.005	46.884	0.005	104.478	0.006	163.169	0.005	213.342	0.005	245.386	0.004	249.687	0.004	292.453	0.005	368.839	0.005	505.750
	0.006	58.099	0.007	129.469	0.007	202.197	0.007	264.372	0.007	304.081	0.006	309.410	0.006	362.405	0.006	457.063	0.007	562.441
	0.008	66.783	0.009	148.821	0.009	232.421	0.009	303.890	0.008	349.534	0.007	355.660	0.007	416.577	0.008	525.384	0.009	646.514
	0.009	73.253	0.011	163.239	0.011	254.938	0.011	333.330	0.010	383.396	0.008	390.116	0.008	456.934	0.009	576.282	0.010	709.148
	0.011	77.935	0.012	173.673	0.013	271.233	0.013	354.636	0.012	407.903	0.010	415.052	0.010	486.141	0.011	613.117	0.012	754.475
	0.013	81.253	0.014	181.066	0.015	282.780	0.014	369.734	0.013	425.268	0.011	432.721	0.011	506.837	0.012	639.219	0.014	786.595
	0.014	83.569	0.016	186.228	0.017	290.841	0.016	380.273	0.015	437.390	0.013	445.056	0.013	521.284	0.014	657.440	0.015	809.017
	0.016	85.169	0.018	189.793	0.018	296.409	0.018	387.554	0.017	445.764	0.014	453.577	0.014	531.265	0.016	670.027	0.017	824.506
	0.017	86.266	0.019	192.238	0.020	300.228	0.020	392.547	0.018	451.507	0.015	459.421	0.016	538.109	0.017	678.659	0.019	835.128
	0.019	87.015	0.021	193.907	0.022	302.834	0.022	395.954	0.020	455.426	0.017	463.408	0.017	542.780	0.019	684.550	0.020	842.377
	0.021	87.524	0.023	195.042	0.024	304.606	0.023	398.271	0.022	458.092	0.018	466.120	0.018	545.957	0.020	688.556	0.022	847.307
	0.022	87.870	0.025	195.812	0.026	305.809	0.025	399.844	0.023	459.900	0.020	467.961	0.020	548.112	0.022	691.274	0.024	850.652
	0.024	88.104	0.026	196.334	0.028	306.623	0.027	400.909	0.025	461.125	0.021	469.207	0.021	549.572	0.023	693.116	0.026	852.918
	0.025	88.262	0.028	196.687	0.029	307.175	0.029	401.630	0.026	461.954	0.022	470.051	0.023	550.560	0.025	694.362	0.027	854.452
																	0.030	952.106

Description Depth (z) * Elevation P-y Curves	Hard Clayey Silt with Sand																	
	z= 5.5 m		z= 6.0 m		z= 6.5 m		z= 7.0 m		z= 7.5 m		z= 8.0 m		z= 8.5 m		z= 9.0 m		z= 9.5 m	
	Elev.= 89.5 m		Elev.= 89.0 m		Elev.= 88.5 m		Elev.= 88.0 m		Elev.= 87.5 m		Elev.= 87.0 m		Elev.= 86.5 m		Elev.= 86.0 m		Elev.= 85.5 m	
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
P-y Curves	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	0.002	217.908	0.002	253.252	0.002	291.239	0.002	331.869	0.003	375.142	0.003	421.059	0.003	469.619	0.003	520.822	0.003	574.668
	0.004	420.030	0.004	488.157	0.005	561.379	0.005	639.696	0.005	723.108	0.005	811.614	0.006	905.216	0.006	1003.913	0.006	1107.705
	0.006	594.845	0.006	691.326	0.007	795.023	0.007	905.935	0.008	1024.062	0.008	1149.405	0.008	1281.964	0.009	1421.738	0.009	1568.727
	0.008	737.127	0.009	856.686	0.009	985.186	0.010	1122.627	0.010	1269.010	0.011	1424.334	0.011	1588.600	0.012	1761.807	0.012	1943.955
	0.010	847.312	0.011	984.742	0.011	1132.450	0.012	1290.436	0.013	1458.700	0.013	1637.242	0.014	1826.062	0.015	2025.159	0.015	2234.535
	0.012	929.398	0.013	1080.143	0.014	1242.161	0.014	1415.452	0.015	1600.017	0.016	1795.856	0.017	2002.968	0.018	2221.354	0.018	2451.013
	0.014	988.804	0.015	1149.184	0.016	1321.558	0.017	1505.926	0.018	1702.288	0.019	1910.644	0.020	2130.995	0.021	2363.339	0.021	2607.678
	0.016	1030.899	0.017	1198.107	0.018	1377.819	0.019	1570.036	0.020	1774.758	0.021	1991.984	0.022	2221.715	0.023	2463.951	0.025	2718.692
	0.018	1060.285	0.019	1232.259	0.020	1417.094	0.022	1614.790	0.023	1825.347	0.024	2048.766	0.025	2285.045	0.026	2534.186	0.028	2796.188
	0.020	1080.585	0.021	1255.851	0.023	1444.225	0.024	1645.706	0.025	1860.294	0.027	2087.990	0.028	2328.794	0.029	2582.705	0.031	2849.723
	0.022	1094.507	0.024	1272.031	0.025	1462.831	0.026	1666.908	0.028	1884.261	0.029	2114.891	0.031	2358.797	0.032	2615.979	0.034	2886.438
	0.024	1104.007	0.026	1283.072	0.027	1475.529	0.029	1681.377	0.030	1900.617	0.032	2133.248	0.034	2379.271	0.035	2638.686	0.037	2911.492
	0.026	1110.468	0.028	1290.581	0.029	1484.164	0.031	1691.217	0.033	1911.740	0.035	2145.733	0.036	2393.195	0.038	2654.128	0.040	2928.531
	0.028	1114.852	0.030	1295.676	0.032	1490.023	0.034	1697.894	0.035	1919.287	0.037	2154.204	0.039	2402.643	0.041	2664.606	0.043	2940.092
	0.030	1117.822	0.032	1299.128	0.034	1493.993	0.036	1702.417	0.038	1924.400	0.040	2159.942	0.042	2409.044	0.044	2671.705	0.046	2947.924
	0.032	1119.832	0.034	1301.464	0.036	1496.679	0.038	1705.478	0.041	1927.860	0.043	2163.826	0.045	2413.375	0.047	2676.508	0.049	2953.225
																	0.051	3243.525

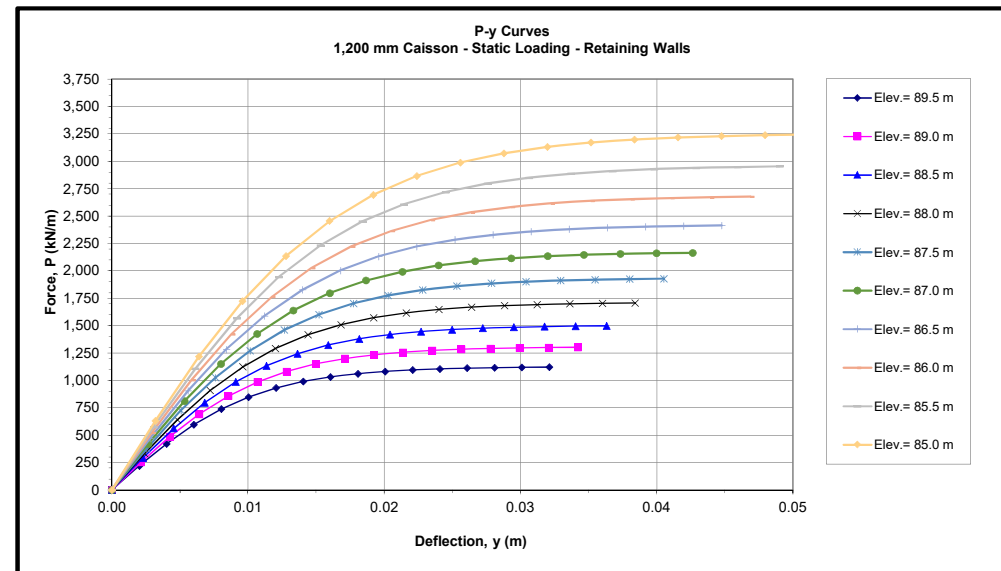
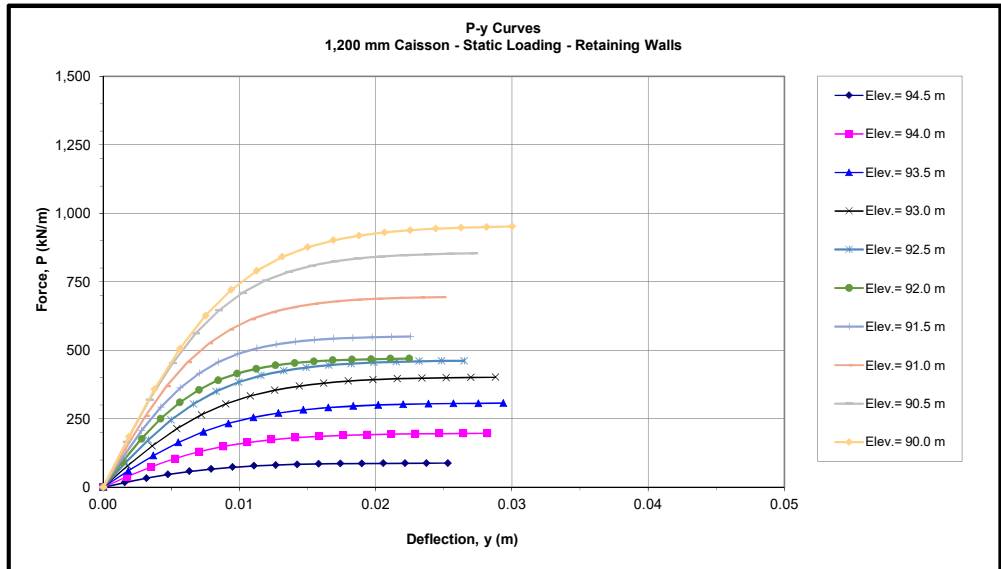
NOTES: * Depth (z) is measured to be positive below the proposed front face elevation (i.e. Elevation 95 m).

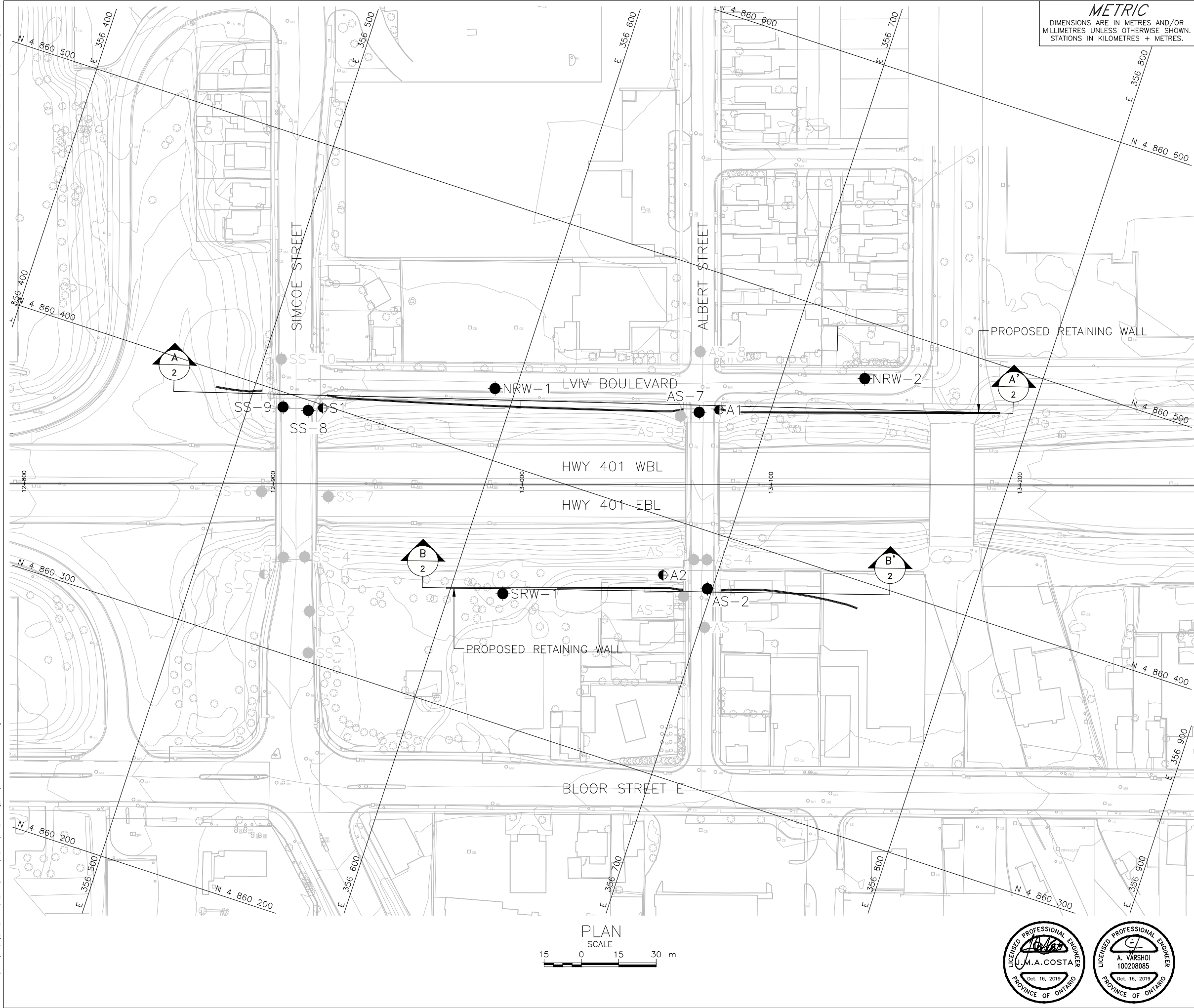
The P-y curves have been generated based on the following assumptions:

1. P-y curves are generated for vertical caissons (i.e. no inclination)
2. Static loading condition is considered.
3. There are no pile group effects (i.e. analysis is based on a single pile).
4. P-y curves represent the resistance from the mass of soil behind the caissons in the passive earth pressure mode.

P-y CURVES
Highway 401 Oshawa - Secant Retaining walls - P-y Curves
1200 mm Diameter Caisson

Table 3

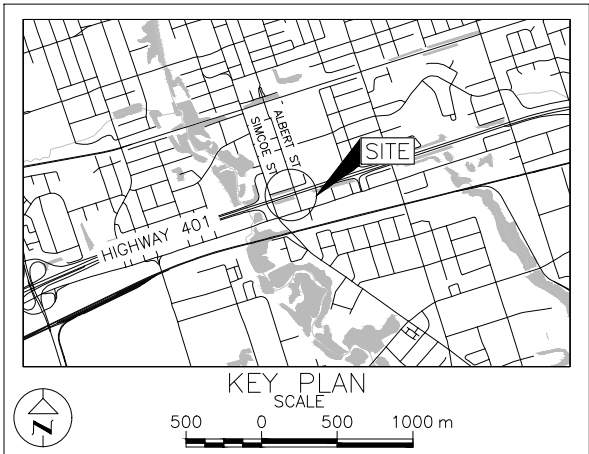
Date: October 2019
Project No: 1662582Prepared By: ARV
Checked By: CN



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

CONT No.
GWP No. 2298-13-00

RETAINING WALLS
HIGHWAY 401 WIDENING, SIMCOE STREET
TO ALBERT STREET, OSHAWA
BOREHOLE LOCATIONS



LEGEND

- Borehole - Current Investigation
- Borehole - Previous Investigation

BOREHOLE CO-ORDINATES (MTM NAD 83 ZONE 10)			
No.	ELEVATION	NORTHING	EASTING
A1	102.1	4860447.6	356680.2
A2	102.0	4860377.6	356679.4
AS-2	101.6	4860377.9	356698.1
AS-7	102.1	4860444.2	356672.9
NRW-1	102.2	4860427.8	356592.0
NRW-2	101.3	4860477.8	356731.9
S1	101.3	4860398.8	356528.6
SRW-1	103.0	4860350.5	356620.6
SS-8	101.2	4860396.0	356523.4
SS-9	101.1	4860394.2	356513.4

NOTES

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

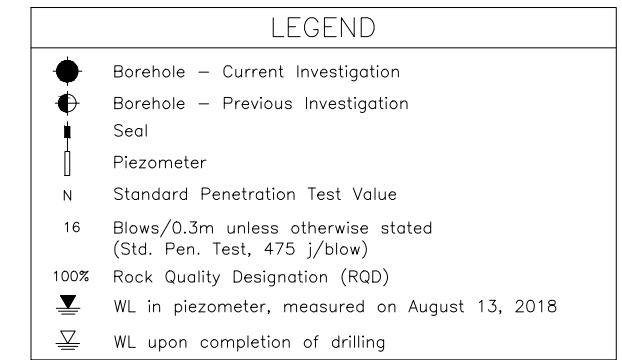
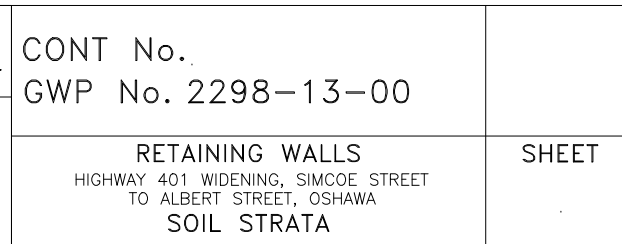
The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

REFERENCE

Base plans provided in digital format by URS, drawing file nos. X-Base.dwg and X-Property.dwg, received April 11, 2014.
Retaining walls provided in digital format by WSP, drawing no. 171-04557-00_XR_Retaining Walls.dwg, received July 09, 2018.



REVISION			
NO.	DATE	BY	REVISION
Geocres No. 30M14-506			
HWY. 401	PROJECT NO. 1662582		DIST. .
SUBM'D. ACK	CHKD. ACK	DATE: 10/16/2019	SITE: .
DRAWN: DD	CHKD. ARV	APPD. JMAC	DWG. 1



BOREHOLE CO-ORDINATES (MTM NAD 83 ZONE 10)			
No.	ELEVATION	NORTHING	EASTING
A1	102.1	4860447.6	356680.2
A2	102.0	4860377.6	356679.4
AS-2	101.6	4860377.9	356698.1
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S1	101.3	4860398.8	356528.6
SRW-1	103.0	4860350.5	356620.6
SS-8	101.2	4860396.0	356523.4
SS-9	101.1	4860394.2	356513.4

NOTES

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

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

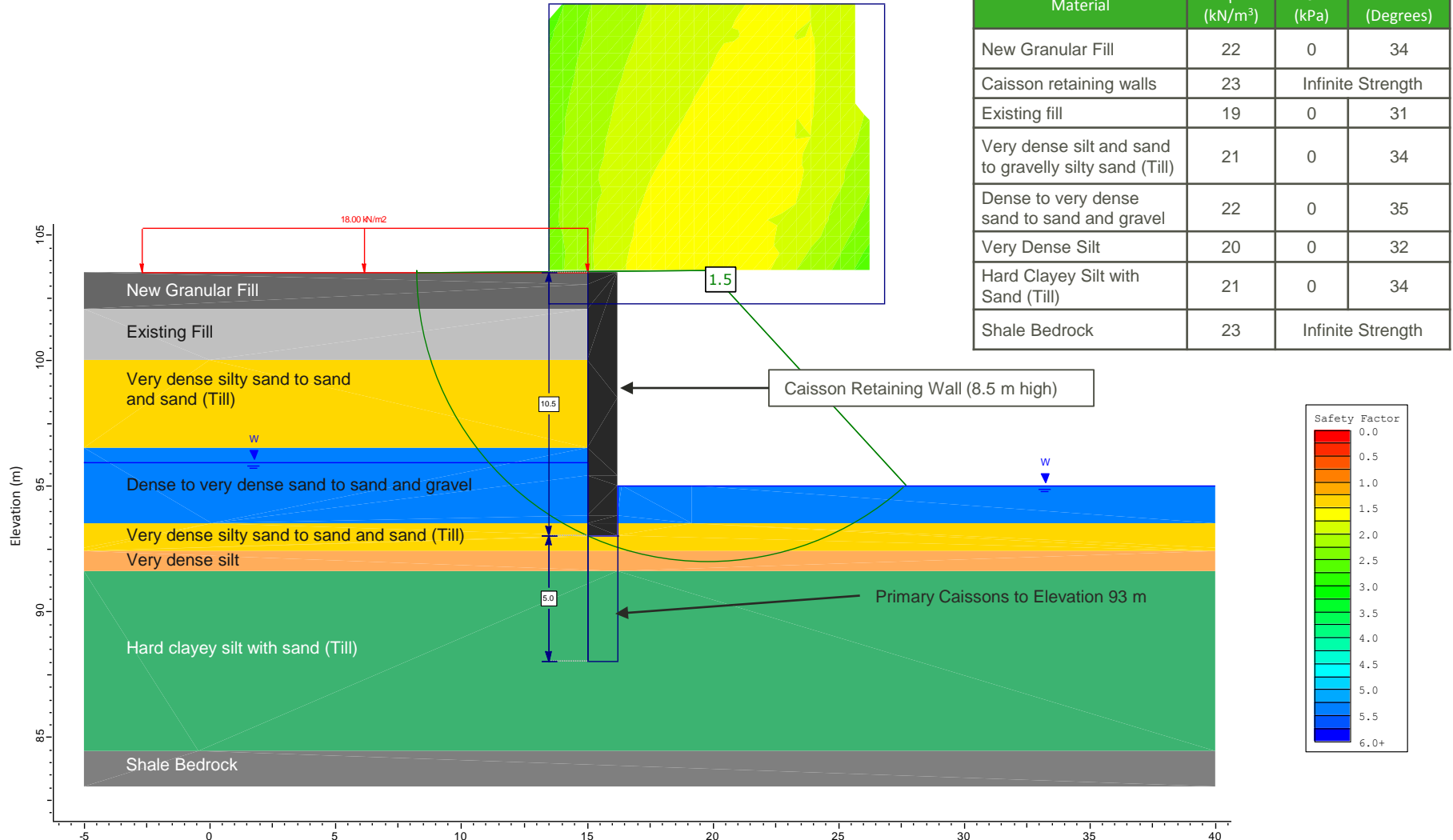
REFERENCE

Base plans provided in digital format by URS, drawing file nos. X-Base.dwg and X-Property.dwg, received April 11, 2014.
Retaining walls provided in digital format by WSP, drawing no. 171-04557-00_XR_Retaining Walls.dwg, received July 09, 2018.



NO.	DATE	BY	REVISION
Geocres No. 30M14-506			
HWY. 401		PROJECT NO. 1662582	DIST.
SUBM'D. ACK	CHKD. ACK	DATE: 10/15/2019	SITE:
DRAWN: DD	CHKD. ARV	APPD. JMAC	DWG. 2

Figure 1





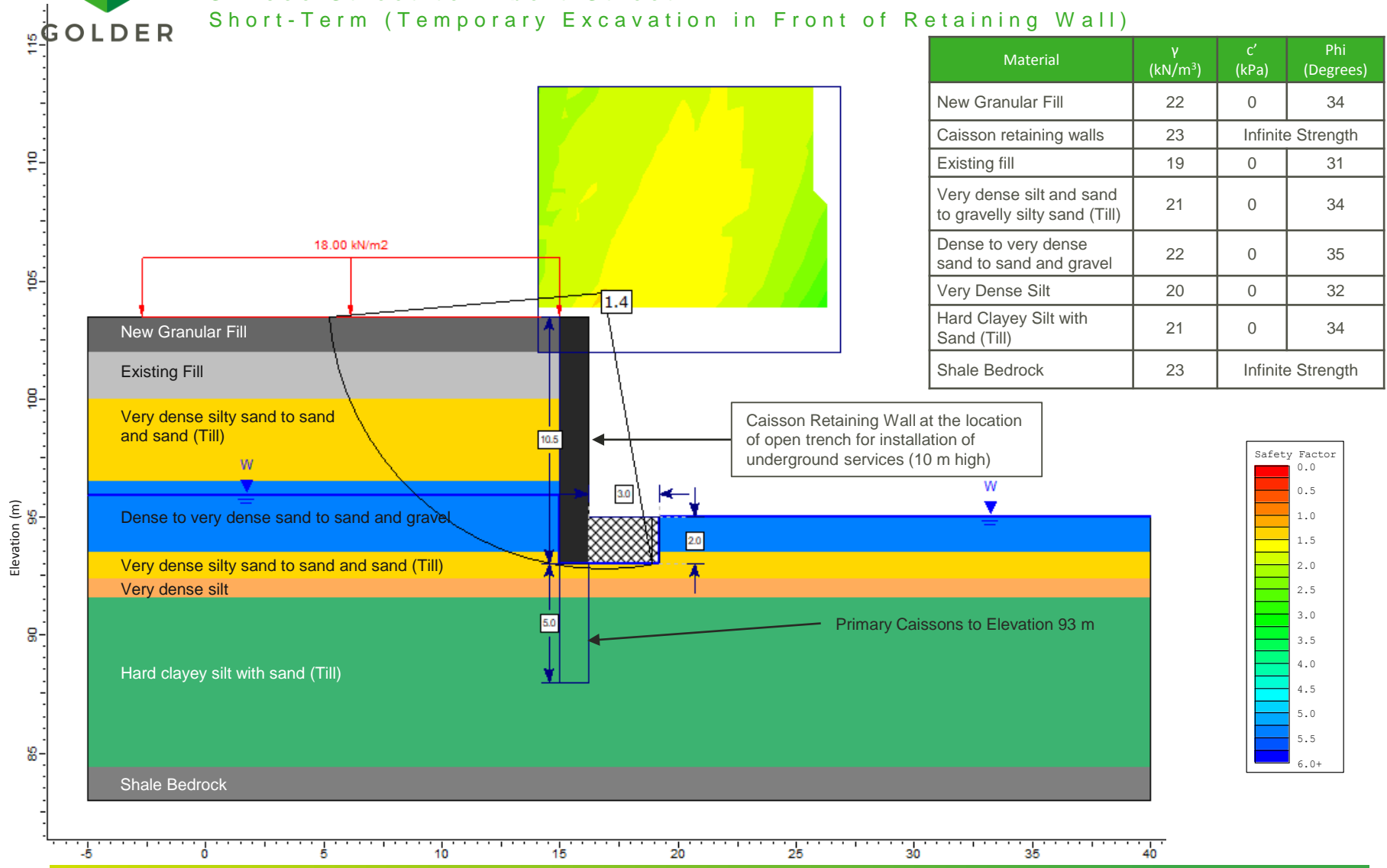
Global Stability Analysis

Secant Caissons Retaining Walls

Simcoe Street to Albert Street

Short-Term (Temporary Excavation in Front of Retaining Wall)

Figure 2



APPENDIX A

Borehole Records

LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta \sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

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

Notes: 1
2

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

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)

Dynamic Cone Penetration Resistance; N_d :

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

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

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

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

Compactness	N
Condition	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	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

WEATHERINGS STATE

Fresh: no visible sign of weathering

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

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

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

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

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

BEDDING THICKNESS

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

JOINT OR FOLIATION SPACING

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

GRAIN SIZE

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

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

CORE CONDITION

Total Core Recovery (TCR)

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

Solid Core Recovery (SCR)

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

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varied from 0% for completely broken core to 100% for core in solid sticks.

DISCONTINUITY DATA

Fracture Index

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

Dip with Respect to Core Axis

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

Description and Notes

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

Abbreviations

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

PROJECT		1662582 (2000)		RECORD OF BOREHOLE No SS-9		SHEET 1 OF 1		METRIC																
G.W.P.		2298-13-00		LOCATION		N 4860394.2; E 356513.4 MTM NAD ZONE 10 (LAT. 43.881720; LONG. -78.856460)		ORIGINATED BY LP																
DIST		Central HWY 401		BOREHOLE TYPE		216 mm O.D Hollow Stem Augers		COMPILED BY AK																
DATUM		Geodetic		DATE		October 31, 2017		CHECKED BY																
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS			ELEVATION SCALE			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES																		
101.1		GROUND SURFACE																						
0.0		ASPHALT (150 mm)																						
0.2		Gravelly sand, some silt (FILL)		1	SS	21																		
100.4		Compact Brown Moist																						
0.7		Silty clay, trace to some sand (FILL)		2	SS	7																		
		Firm to very stiff Brown Moist																						
				3	SS	18																		
99.0																								
2.1		SILT and SAND, some gravel, trace to some clay (TILL)		4	SS	38																		
		Compact to dense Brown, becoming grey below a depth of 3.3 m (Elev. 97.8 m)																						
		Moist		5	SS	24																		
				6	SS	32																		
95.5																								
5.6		Gravelly SAND, trace to some silt, trace clay		7	SS	57																		
		Very dense Grey																						
		Moist to wet below a depth of 7.6 m (Elev. 93.5 m)																						
		- Crushed rock fragments recovered at a depth of 6.1 m (Elev. 95.0 m)		8	SS	67																		
				9	SS	111																		
				10	SS	103																		
91.0																								
10.1		CLAYEY SILT, some sand, trace gravel (TILL)																						
		Hard Grey Moist																						
90.1				11	SS	133/0.2																		
11.1		END OF BOREHOLE																						
		NOTES:																						
		1. Groundwater level in borehole measured at a depth of 8.5 m below ground surface (Elev. 92.6 m) upon completion of drilling.																						
		2. Water level in piezometer measured as follows:																						
		Date	Depth (m)	Elev. (m)																				
		Aug 13/18	7.9	93.2																				



PROJECT 1662582 (2000)			RECORD OF BOREHOLE No SS-8			SHEET 1 OF 1			METRIC							
G.W.P. 2298-13-00			LOCATION N 4860396.0; E 356523.4 MTM NAD ZONE 10 (LAT. 43.881730; LONG. -78.856340)			ORIGINATED BY LP										
DIST Central HWY 401			BOREHOLE TYPE 216 mm O.D Hollow Stem Augers			COMPILED BY AK										
DATUM Geodetic			DATE October 30, 2017			CHECKED BY										
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
101.2	GROUND SURFACE															
0.0	ASPHALT (180 mm)															
0.2	Gravelly sand, some silt (FILL)		1	SS	21											
100.5	Compact Brown Moist															
0.7	Silty clay, trace sand, trace gravel (FILL)		2	SS	14											
99.8	Stiff Brown Moist															
1.4	SILT and SAND, trace to some gravel, trace to some clay (TILL)		3	SS	19											
	Compact to dense Brown Moist to wet															
			4	SS	41											11 43 38 8
			5	SS	25											
			6	SS	32											
96.8	SAND and GRAVEL, trace to some silt, trace clay															
4.4	Dense to very dense Grey Moist to wet		7	SS	48											
			8	SS	76/0.23											52 40 6 2
			9	SS	92											
93.7	SAND, trace silt, trace gravel, trace clay															
7.5	Very dense Brown Moist		10A	SS	108											2 96 1 1
93.2			10B													
8.0	SAND and GRAVEL, trace to some silt, trace clay															
92.7	Very dense Grey Wet															
8.5	CLAYEY SILT, some sand, trace gravel (TILL)		11	SS	106/0.25											
	Hard Grey Moist															
91.1	SILT, trace sand															
10.1	Very dense Grey Moist		12	SS	100/0.15											
90.4																
10.8	END OF BOREHOLE															
NOTES:																
1. Groundwater level measured at a depth of 9.1 m below ground surface (Elev. 92.1 m) upon completion of drilling.																
2. Borehole caved to a depth of 7.6 m below ground surface (Elev. 93.6 m) upon completion of drilling and removal of augers.																

PROJECT 11-1184-0143		RECORD OF BOREHOLE No S1				SHEET 1 OF 1		METRIC									
G.W.P. 10-20011		LOCATION N 4860398.8 ; E 356528.6				ORIGINATED BY TD											
DIST Central HWY 401		BOREHOLE TYPE 150 mm O.D. Continuous Flight Solid Stem Augers				COMPILED BY PKS											
DATUM Geodetic		DATE March 9, 2015				CHECKED BY LCC											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
101.3	GROUND SURFACE																
0.0	TOPSOIL																
100.9																	
0.4	SILTY CLAY, trace gravel, trace sand, trace organics, containing rootlets Very stiff Brown Moist		1	AS	-												
			2	SS	21												
99.8																	
1.5	SILT and SAND, trace to some clay, trace gravel (TILL) Compact to very dense Brown, oxidation staining Moist		3	SS	24												
			4	SS	55												
	- auger grinding on inferred cobble or boulder between a depth of 3.1 m and 6.1 m - becoming grey below a depth of 3.4 m		5	SS	57												
			6	SS	100/0.13												
95.8																	
5.5	SAND and GRAVEL, trace to some silt, trace clay Very dense Grey, oxidation staining Moist		7	SS	100/0.1												
94.7																	
6.8	Silty SAND, trace clay, trace gravel (TILL) Very dense Brown Moist		8	SS	100/0.1												
	END OF BOREHOLE																
	NOTES: 1. Borehole dry upon completion of drilling, March 9, 2015 2. Borehole caved to a depth of 5.5 m below ground surface (Elev. 95.8 m) upon completion of drilling, March 9, 2015																

PROJECT 1662582		RECORD OF BOREHOLE No NRW-1		SHEET 1 OF 1		METRIC						
G.W.P. 2298-13-00		LOCATION N 4860427.8; E 356592.0 MTM NAD 83 ZONE 10 (LAT. 43.882015; LONG. -78.855480)		ORIGINATED BY LP								
DIST Central HWY 401		BOREHOLE TYPE 216 mm O.D Hollow Stem Augers		COMPILED BY AK								
DATUM Geodetic		DATE March 5, 2018		CHECKED BY CN								
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC NATURAL LIQUID UNIT REMARKS			
ELEV	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	W _p W W _L	WATER CONTENT (%)	γ	GRAIN SIZE DISTRIBUTION (%)
102.2	GROUND SURFACE											
0.0	ASPHALT (150 mm)											
0.2	Sand, some silt, some gravel (FILL)		1	SS	3		102					
101.5	Very loose Brown Moist		2	SS	16		101					
0.7	Sandy clayey silt to sand with clayey silt with sand, trace to some gravel, trace asphalt pieces (FILL)		3	SS	9		100					
	Firm to very stiff Brown Moist		4	SS	4		99					
	- Trace organics between depths of 0.7 m and 2.1 m (Elev. 101.5 m and 100.1 m)		5	SS	8		98					18 49 22 11
98.5	SAND and GRAVEL, trace to some silt, trace clay		6	SS	68		97					47 43 7 3
3.7	Very dense Brown to grey Moist		7	SS	100/0.15		96					
			8	SS	100/0.15		95					
95.4	SILT and SAND, trace to some clay, trace gravel (TILL)		9	SS	100/0.15		94					
6.8	Very dense Grey Moist		10	SS	100/0.15		93					2 43 44 11
			11	SS	100/0.15		92					
91.4	END OF BOREHOLE											
10.8	NOTES: 1. Borehole dry upon completion of drilling. 2. Water level in piezometer measured as follows: Date Depth (m) Elev. (m) Aug. 13/18 6.8 95.4 Feb. 8/19 6.3 95.9											

GTA-MTO 001 S:\CLIENTS\MTO\HWY 401 OSHAWA\02 DATA\GINTHWY 401 OSHAWA.GPJ GAL-GTA.GDT 09/10/19

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

PROJECT <u>1662582 (2000)</u>				RECORD OF BOREHOLE No AS-7				SHEET 2 OF 2				METRIC						
G.W.P. <u>2298-13-00</u>				LOCATION <u>N 4860444.2; E 356672.9 MTM NAD 83 ZONE 10 (LAT. 43.882157; LONG. -78.854472)</u>				ORIGINATED BY <u>LP</u>										
DIST <u>Central</u> HWY <u>401</u>				BOREHOLE TYPE <u>216 mm O.D. Hollow Stem Augers, HQ Coring</u>				COMPILED BY <u>AK/KN</u>										
DATUM <u>Geodetic</u>				DATE <u>November 16 and 17, 2017</u>				CHECKED BY <u>AP</u>										
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT			LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W _p — W — W _L					
--- CONTINUED FROM PREVIOUS PAGE ---																		
84.4 17.7	CLAYEY SILT with SAND, trace to some gravel (TILL) Hard Grey Moist		14	SS	64		87											RQD = 91%
								86										
			15	SS	122		85											
			16	SS	100/0.02		84											
			17	SS	100/0.02		83											
80.2 21.9	SHALE (BEDROCK) Bedrock cored between depths of 18.9 m and 21.9 m (Elev. 83.2 m and 80.2 m) For bedrock coring details refer to Record of Drillhole AS-7.		1	RC	REC 100%		82											RQD = 100%
			2	RC	REC 100%		81											
END OF BOREHOLE																		
NOTE: 1. Water level not recorded upon completion of drilling.																		

PROJECT: 1662582 (2000)

RECORD OF DRILLHOLE: AS-7

SHEET 1 OF 1

LOCATION: N 4860444.2 ;E 356672.9

DRILLING DATE: November 20, 2017

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 75 Truck

DRILLING CONTRACTOR: Pontil Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	JN - Joint FLT - Fault SHR- Shear VN - Vein CJ - Conjugate BD- Bedding FO- Foliation CO- Contact OR- Orthogonal CL - Cleavage PL - Planar CU- Curved UN- Undulating ST - Stepped IR - Irregular PO- Polished K - Slickensided SM- Smooth Ro - Rough MB- Mechanical Break BR - Broken Rock NOTE: For additional abbreviations refer to list of abbreviations & symbols.																FEATURES	RQ/R1 ZONES	NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																															
						RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.25 m	DISCONTINUITY DATA					ROCK STRENGTH INDEX		WEATH- ERING INDEX																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																						
						TOTAL CORE %	SOLID CORE %			TYPE AND SURFACE DESCRIPTION	Jr	Ja	R4	R3	R2	R1	W1	W2	W3	W4	W5				W6																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																														
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19	Rotary Drill HQ Coring	Continued from Record of Borehole AS-7		83.26 18.85																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																			

FEATURES LEGEND



BROKEN CORE



CLAY SEAM



LIMESTONE



LOST CORE

DEPTH SCALE

1 : 50

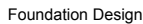
**GOLDER**

LOGGED: LP

CHECKED: ACK

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



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

PROJECT 1662582		RECORD OF BOREHOLE No SRW-1				SHEET 1 OF 2		METRIC								
G.W.P. 10-20011		LOCATION N 4860350.5; E 356620.6 MTM NAD 83 ZONE 10 (LAT. 43.881317; LONG. -78.855131)				ORIGINATED BY LP										
DIST Central HWY 401		BOREHOLE TYPE 216 mm O.D. Hollow Stem Augers				COMPILED BY AK										
DATUM Geodetic		DATE February 1, 2018				CHECKED BY CN										
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40						60	80
103.0	GROUND SURFACE															
0.0	Sandy clayey silt, trace gravel, trace metal and brick fragments (FILL) Firm to stiff Brown Moist		1	SS	13											
			2	SS	7											
101.6																
1.4	Gravelly SILT and SAND to SILT and SAND, trace gravel (TILL) Compact to very dense Brown, organic staining Moist		3	SS	24											
			4	SS	18											
			5A	SS	60											
	- Crushed Rock fragments between depths 3.4 m and 3.5 m (Elev. 99.6 m and 99.5 m)		5B	SS	60											
			6	SS	71											
	- Auger grinding at a depth of 5.2 m (Elev. 97.8 m)															
	- Auger grinding between depths 5.5 m and 6.1 m (Elev. 97.5 m and 96.9 m)															
96.9																
6.1	CLAYEY SILT, some sand Hard Grey Moist		7	SS	42											
95.9																
7.1	Gravelly SAND, some silt Very dense Brown Moist		8	SS	101											
	- Auger grinding between depths 7.3 m and 7.6 m (Elev. 95.7 m and 95.4 m)															
	- Auger grinding between depths 8.2 m and 8.5 m (Elev. 94.8 m and 94.5 m)															
94.3																
8.7	Sandy SILT, trace to some clay, trace gravel (TILL) Very dense Grey Moist		9	SS	100/0.1											
	- Auger grinding between depths 9.8 m and 10.4 m (Elev. 93.2 m and 92.6 m)															
			10	SS	100/0.1											
			11	SS	100/0.0											
89.6																
13.4	CLAYEY SILT with SAND, trace to some gravel (TILL) Hard Grey Moist		12	SS	159/0.2											
	- Auger grinding between depths 13.7 m and 15.2 m (Elev. 89.3 m and 87.8 m)															

Continued Next Page

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

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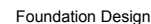
PROJECT <u>1662582</u>		RECORD OF BOREHOLE No SRW-1				SHEET 2 OF 2		METRIC								
G.W.P. <u>10-20011</u>		LOCATION <u>N 4860350.5; E 356620.6 MTM NAD 83 ZONE 10 (LAT. 43.881317; LONG. -78.855131)</u>				ORIGINATED BY <u>LP</u>										
DIST <u>Central</u> HWY <u>401</u>		BOREHOLE TYPE <u>216 mm O.D. Hollow Stem Augers</u>				COMPILED BY <u>AK</u>										
DATUM <u>Geodetic</u>		DATE <u>February 1, 2018</u>				CHECKED BY <u>CN</u>										
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					W _p W W _L			
	--- CONTINUED FROM PREVIOUS PAGE ---						20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					WATER CONTENT (%)				
87.3		13	SS	102												
15.7	END OF BOREHOLE NOTES: 1. Borehole dry upon completion of drilling. 2. Water level in piezometer measured as follows: Date Depth (m) Elev. (m) Aug. 13/18 8.3 94.7 Feb. 8/19 8.2 94.8															

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PROJECT		11-1184-0143		RECORD OF BOREHOLE No A2		SHEET 1 OF 1		METRIC					
G.W.P.		10-20011		LOCATION		N 4860377.6 ; E 356679.4		ORIGINATED BY		TD			
DIST		Central HWY 401		BOREHOLE TYPE		150 mm O.D. Continuous Flight Solid Stem Augers		COMPILED BY		PKS			
DATUM		Geodetic		DATE		March 10, 2015		CHECKED BY		LCC			
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)			
102.0	GROUND SURFACE												
0.0	TOPSOIL												
101.5													
0.5	Silty clay, trace to some organics, trace gravel, trace sand (FILL) Stiff Dark to light brown Moist		1	AS	-								
100.6			2A	SS	15								
1.4	CLAYEY SILT, trace gravel, trace sand Hard Brown Moist		3A	SS	35								
100.2			3B										
1.8	SILT and SAND, trace to some clay, trace gravel (TILL) Dense to very dense Brown to grey, oxidation staining Moist		4	SS	47								
			5	SS	135								
97.1	- becoming grey and wet below a depth of 4.6 m		6A	SS	84								
4.9	SAND, trace gravel, trace silt Very dense Brown Moist		6B										
96.5													
5.5	SILT and SAND, trace gravel, trace clay Very dense Grey Wet		7	SS	132								
94.0			8	SS	100/0.0								
8.0	END OF BOREHOLE												
NOTES:													
1. Borehole caved to a depth of 7.3 m below ground surface (Elev. 94.7 m) upon completion of drilling, March 10, 2015. Borehole dry above this depth on completion of drilling.													

GTA-MTO 001 S:\CLIENTS\MTO\HWY 401 OSHAWA\02 DATA\GINTHWY 401 OSHAWA.GPJ GAL-GTA.GDT 10/18/19

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



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

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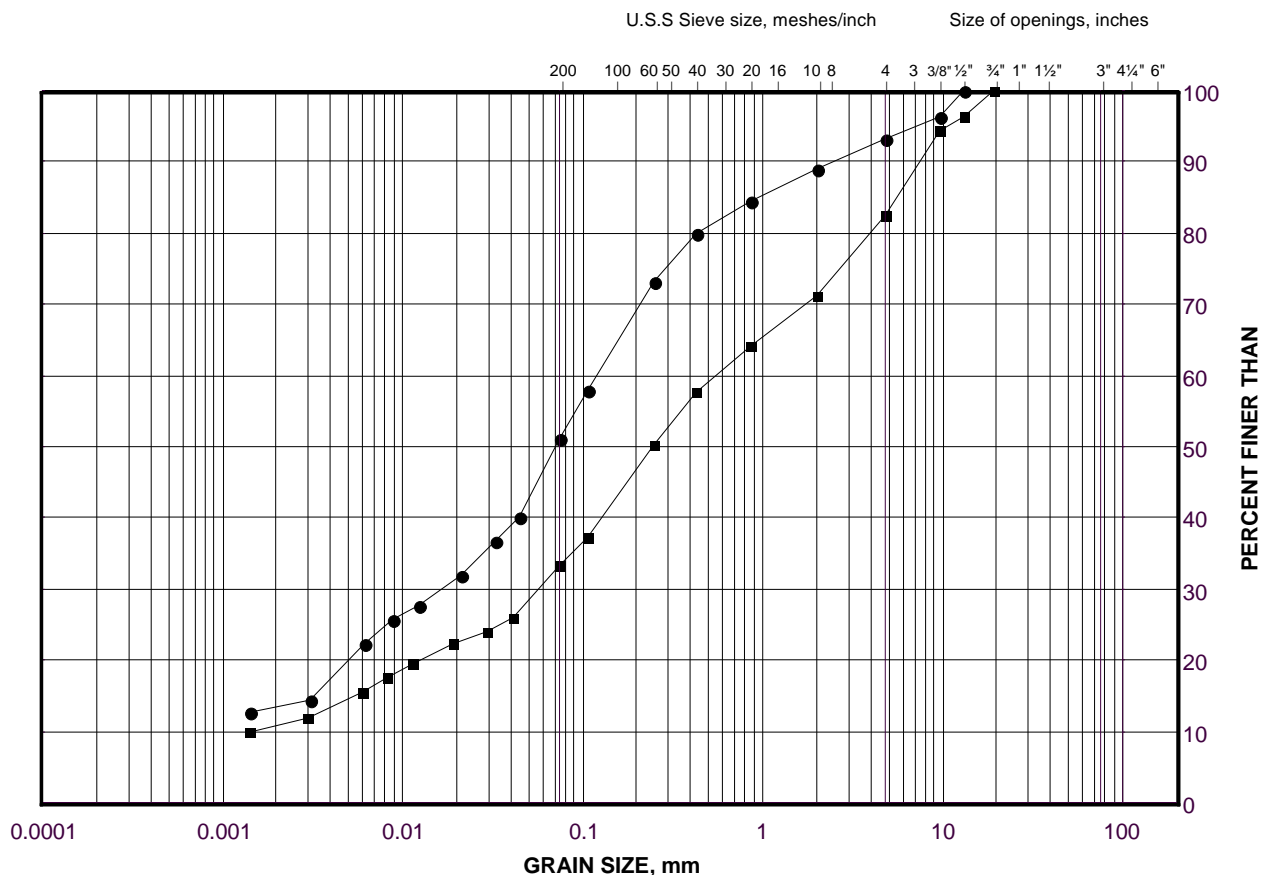
APPENDIX B

Geotechnical Laboratory Test Results

GRAIN SIZE DISTRIBUTION

Clayey Silt to Silty Clay (Fill) - North Retaining Wall

FIGURE B-1



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

LEGEND

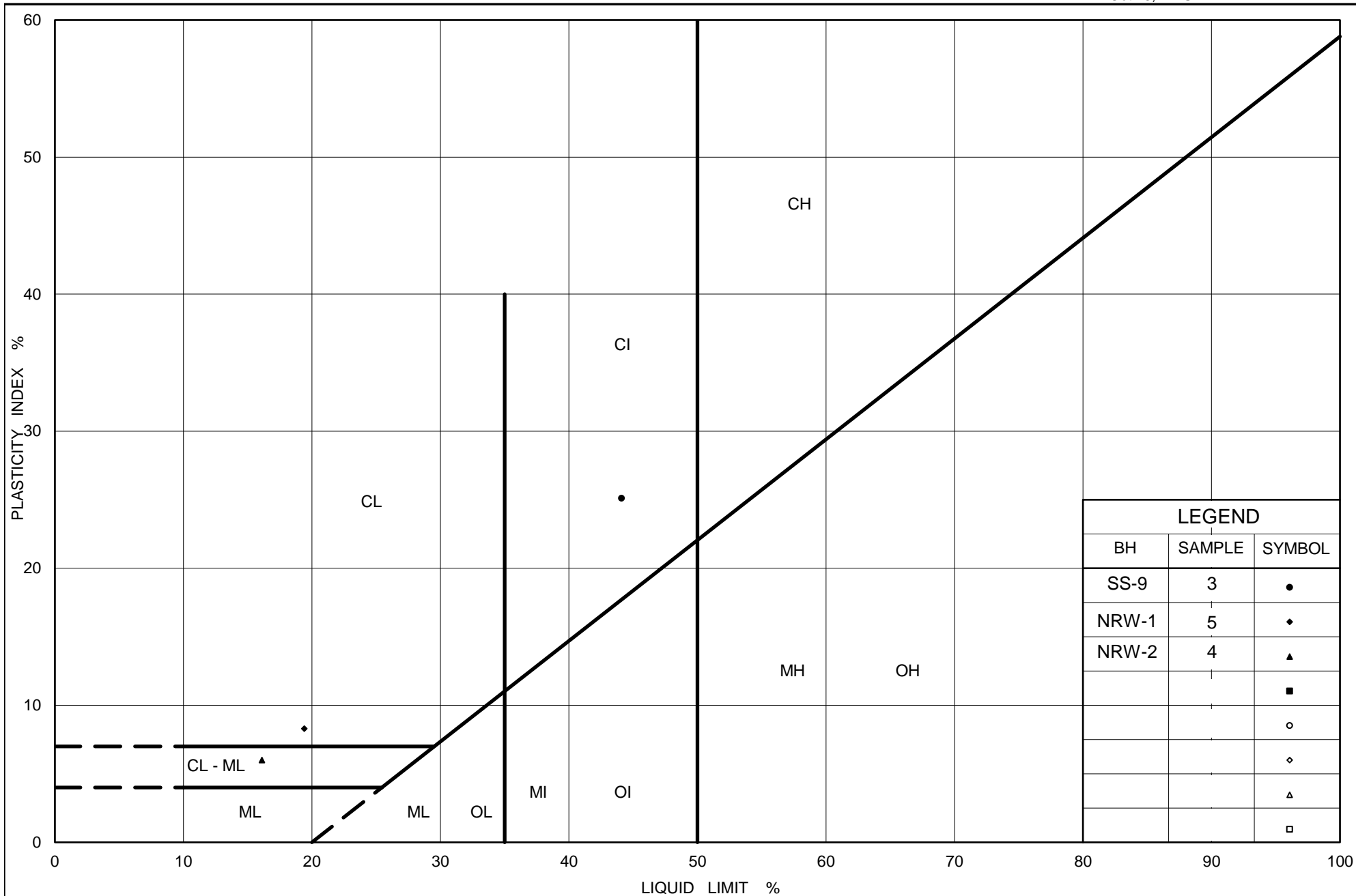
SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	NRW-2	4	98.8
■	NRW-1	5	99.0

Project Number: 1662582

Checked By: ACK

Golder Associates

Date: 28-Aug-18



Ministry of Transportation

Ontario

PLASTICITY CHART

Clayey Silt to Silty Clay (Fill) -
North Retaining Wall

Figure No. B-2

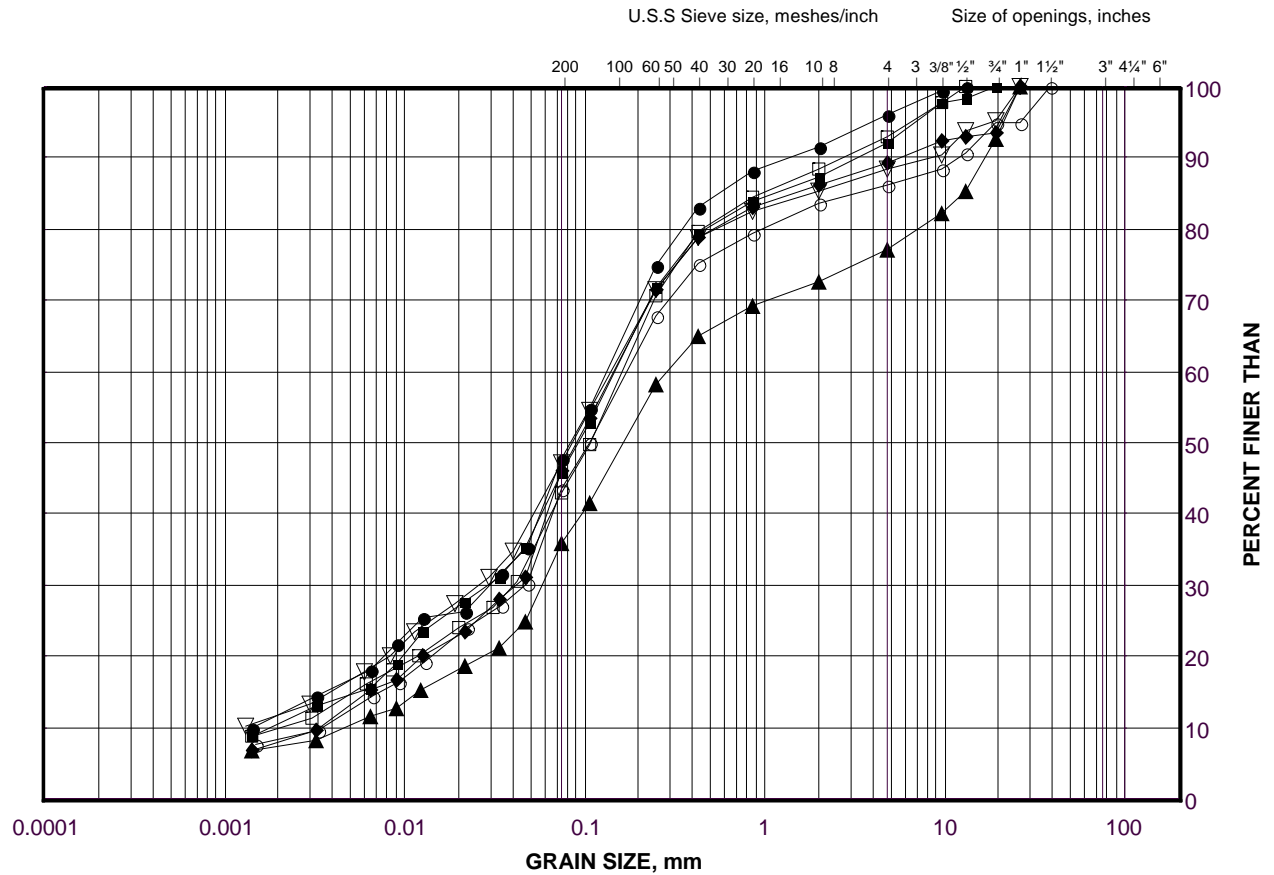
Project No. 1662582

Checked By: ACK

GRAIN SIZE DISTRIBUTION

Silt and Sand to Silty Sand Till (Upper) - North Retaining Wall

FIGURE B-3



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

LEGEND

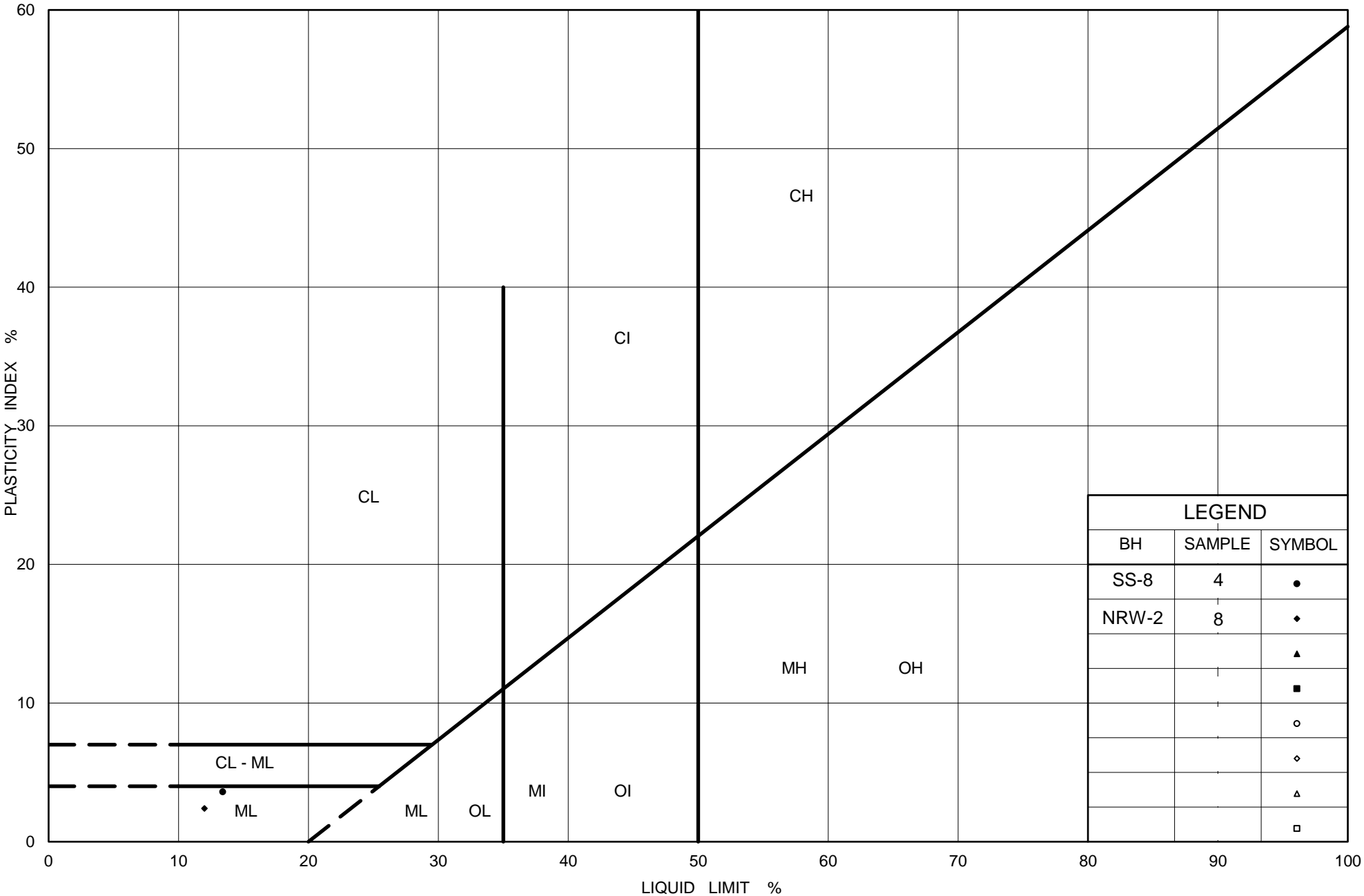
SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	S1	3	99.6
■	A1	4	99.6
◆	SS-8	4	98.7
▲	AS-7	5	98.8
▽	NRW-2	5	98.0
○	SS-9	5	97.8
□	NRW-2	8	95.0

Project Number: 1662582

Checked By: ACK

Golder Associates

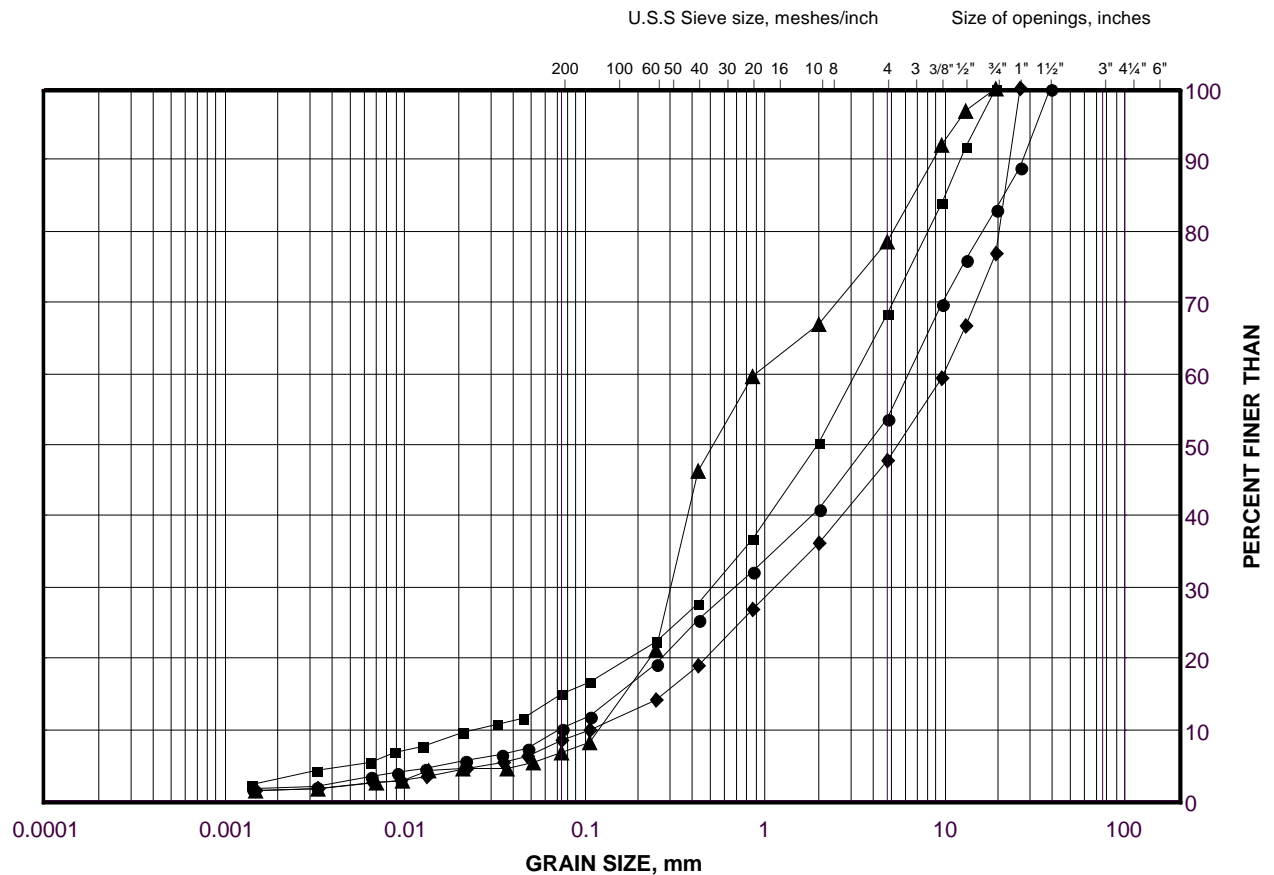
Date: 28-Aug-18



GRAIN SIZE DISTRIBUTION

Gravelly Sand to Sand and Gravel - North Retaining Wall

FIGURE B-5



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	NRW-1	6	98.2
■	S1	7	95.0
◆	SS-8	8	94.9
▲	SS-9	9	93.3

Project Number: 1662582

Checked By: ACK

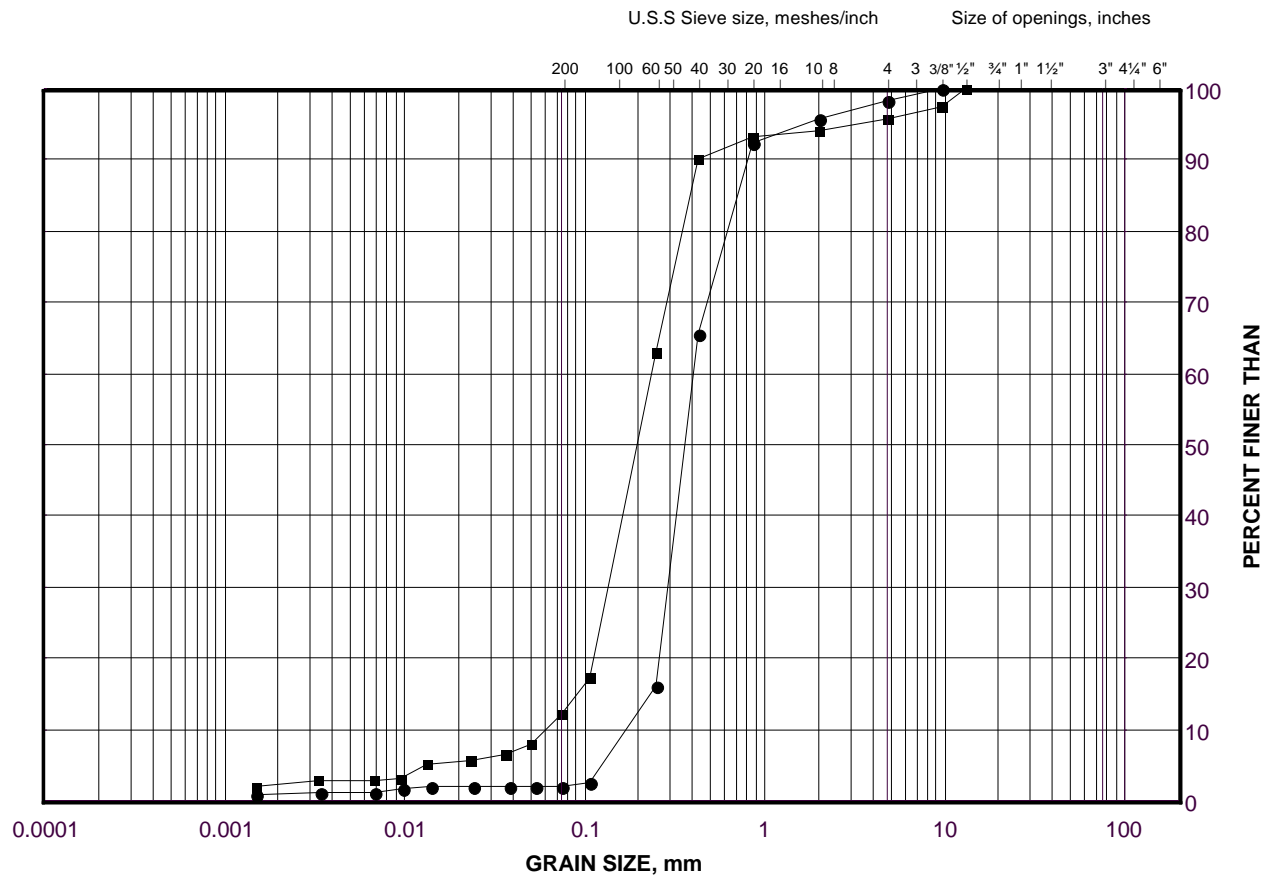
Golder Associates

Date: 28-Aug-18

GRAIN SIZE DISTRIBUTION

Sand - North Retaining Wall

FIGURE B-6



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	SS-8	10A	93.4
■	A1	7	95.8

Project Number: 1662582

Checked By: ACK

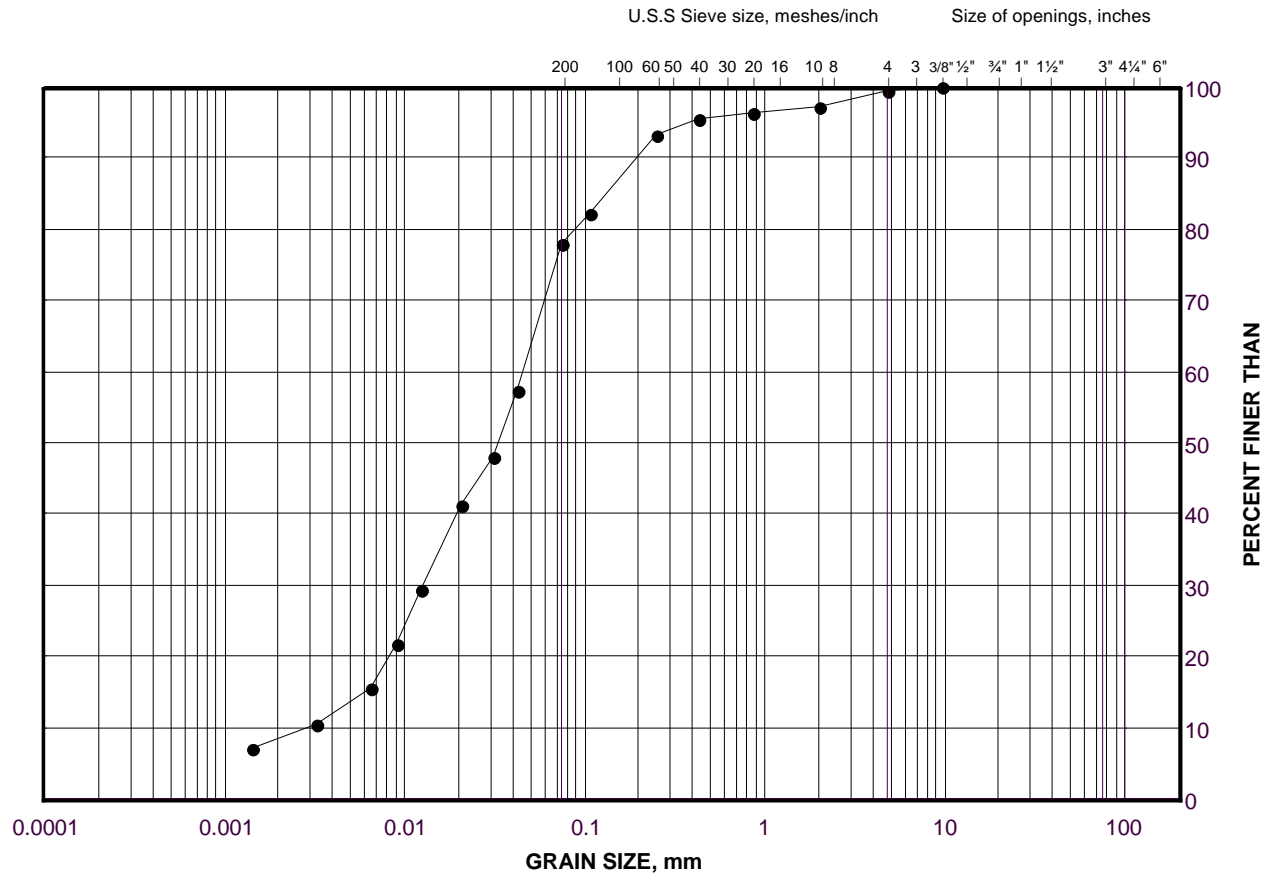
Golder Associates

Date: 11-Sep-18

GRAIN SIZE DISTRIBUTION

Sandy Silt - North Retaining Wall

FIGURE B-7



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
•	NRW-2	9	93.4

Project Number: 1662582

Checked By: ACK

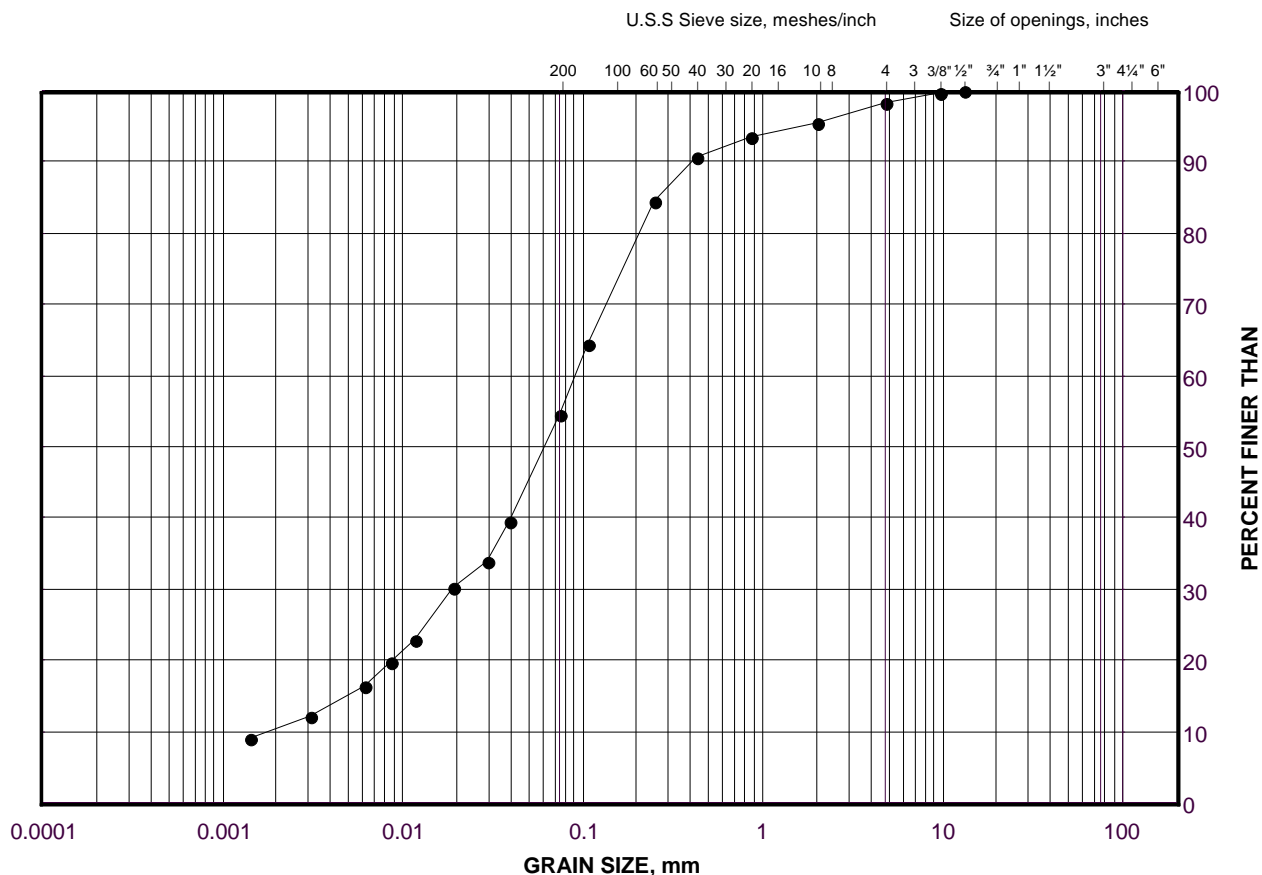
Golder Associates

Date: 11-Sep-18

GRAIN SIZE DISTRIBUTION

Silt and Sand Till (Lower) - North Retaining Wall

FIGURE B-8A



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
•	NRW-1	10	93.0

Project Number: 1662582

Checked By: ACK

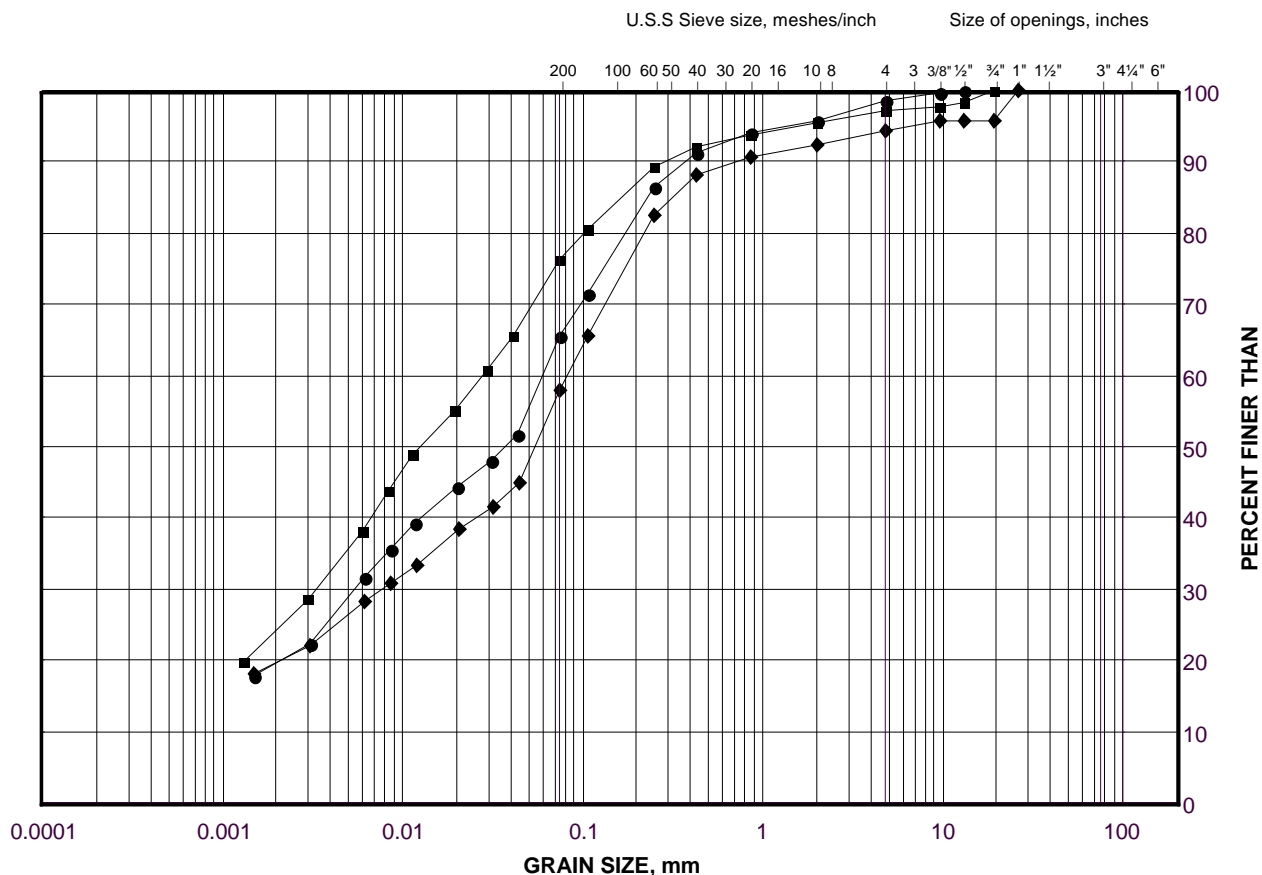
Golder Associates

Date: 28-Aug-18

GRAIN SIZE DISTRIBUTION

Clayey Silt Till (Lower) - North Retaining Wall

FIGURE B-8B



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

LEGEND

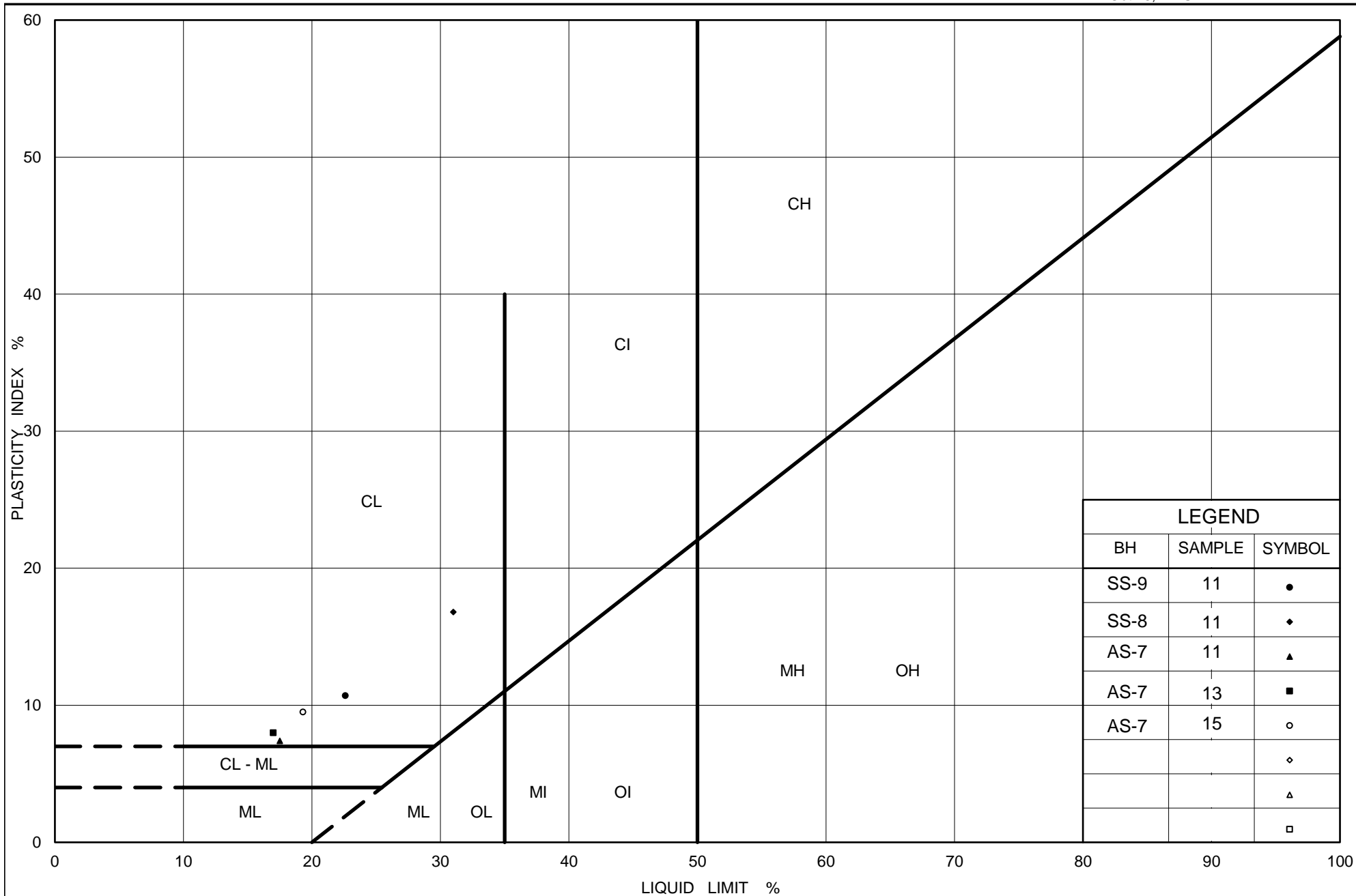
SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	AS-7	11	91.2
■	SS-9	11	90.3
◆	AS-7	13	88.2

Project Number: 1662582

Checked By: ACK

Golder Associates

Date: 28-Aug-18



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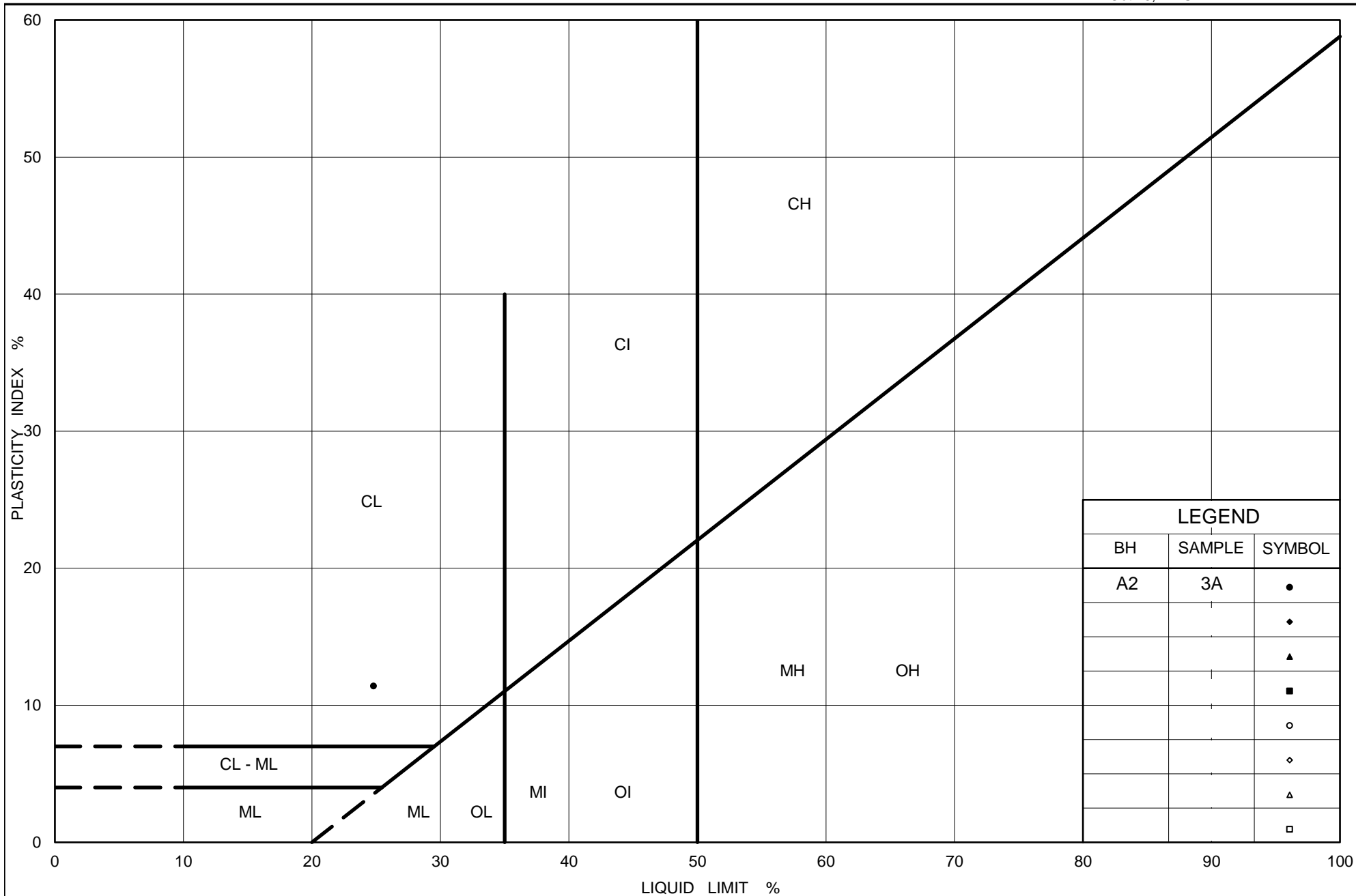
PLASTICITY CHART

Clayey Silt Till (Lower) -
North Retaining Wall

Figure No. B-9

Project No. 1662582

Checked By: ACK



Ministry of Transportation

Ontario

PLASTICITY CHART Clayey Silt - South Retaining Wall

Figure No. B-10

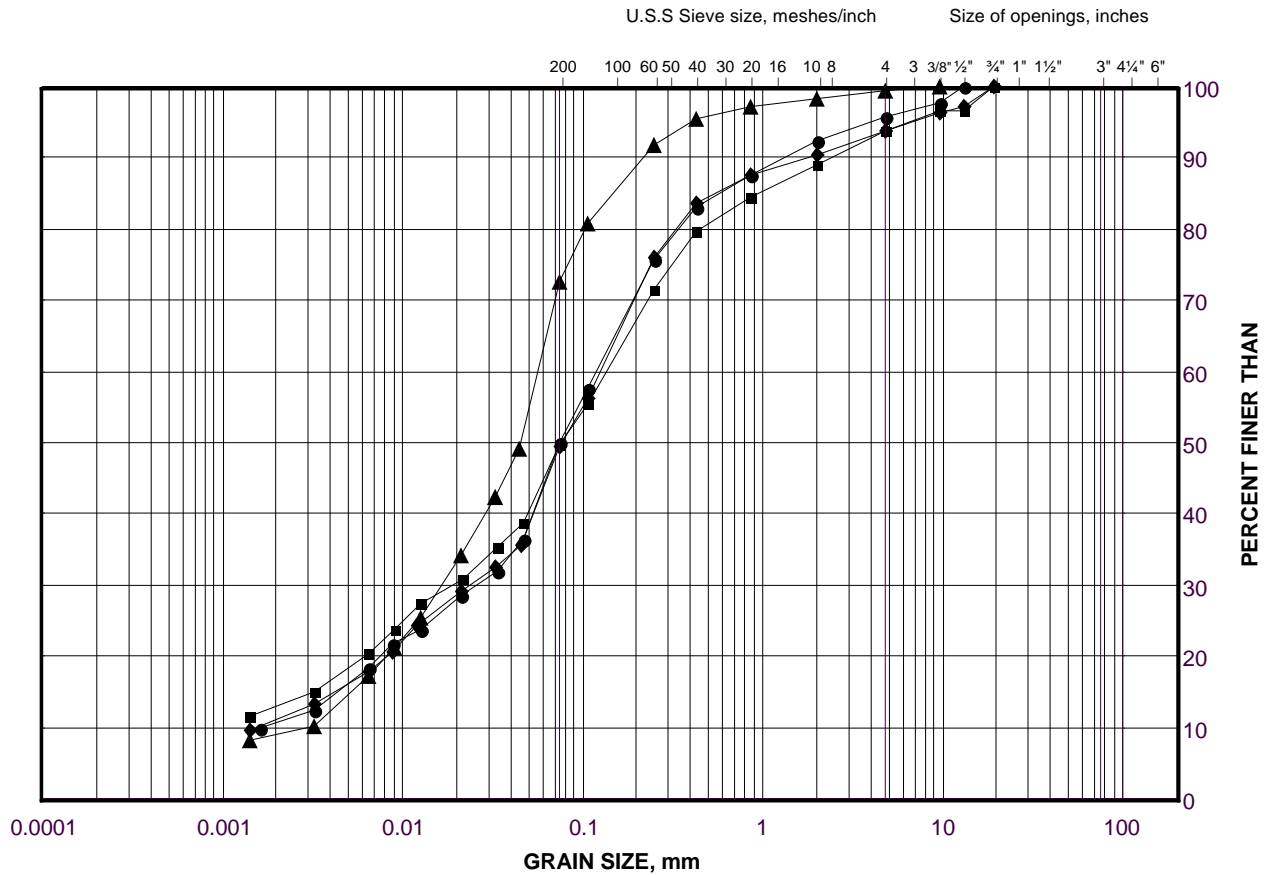
Project No. 1662582

Checked By: ACK

GRAIN SIZE DISTRIBUTION

Sandy Silt to Silt and Sand Till (Upper)- South Retaining Wall

FIGURE B-11



LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	SRW-1	4	100.5
■	A2	5	98.7
◆	AS-2	5	98.3
▲	SRW-1	9	93.8

Project Number: 1662582

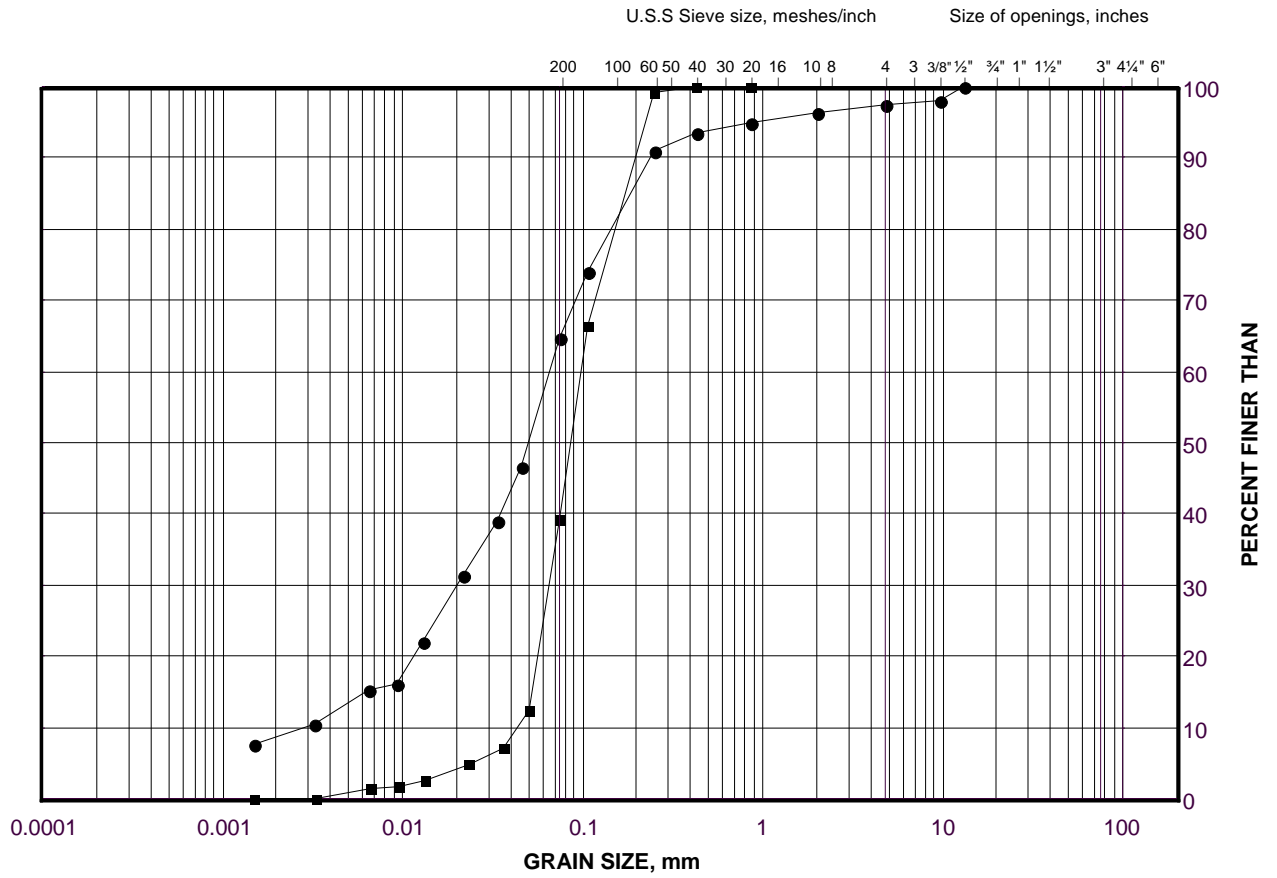
Checked By: ACK

Golder Associates

Date: 11-Sep-18

Silt and Sand - South Retaining Wall

FIGURE B-12



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

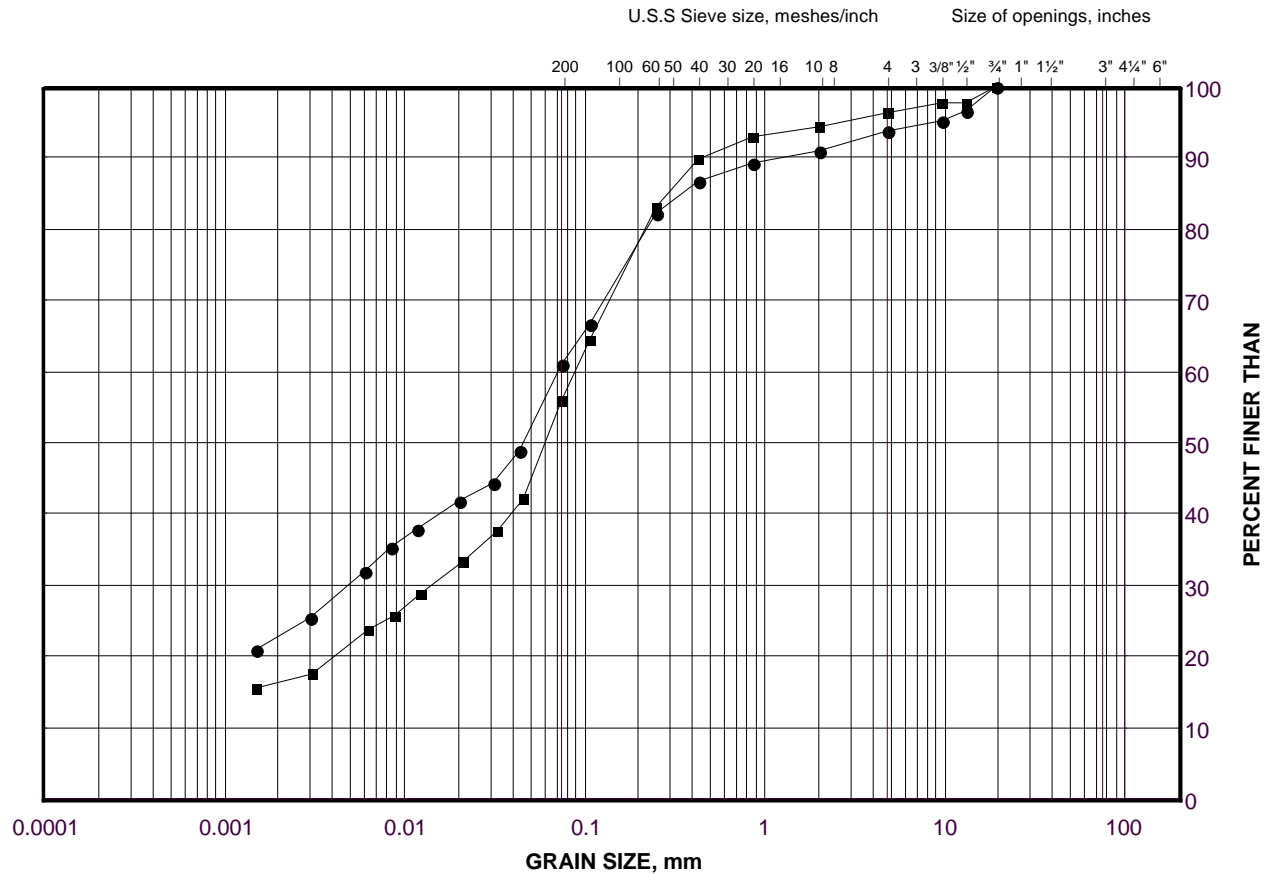
LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	A2	8	94.2
■	AS-2	9	92.2

GRAIN SIZE DISTRIBUTION

Clayey Silt Till - South Retaining Wall

FIGURE B-13



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

LEGEND

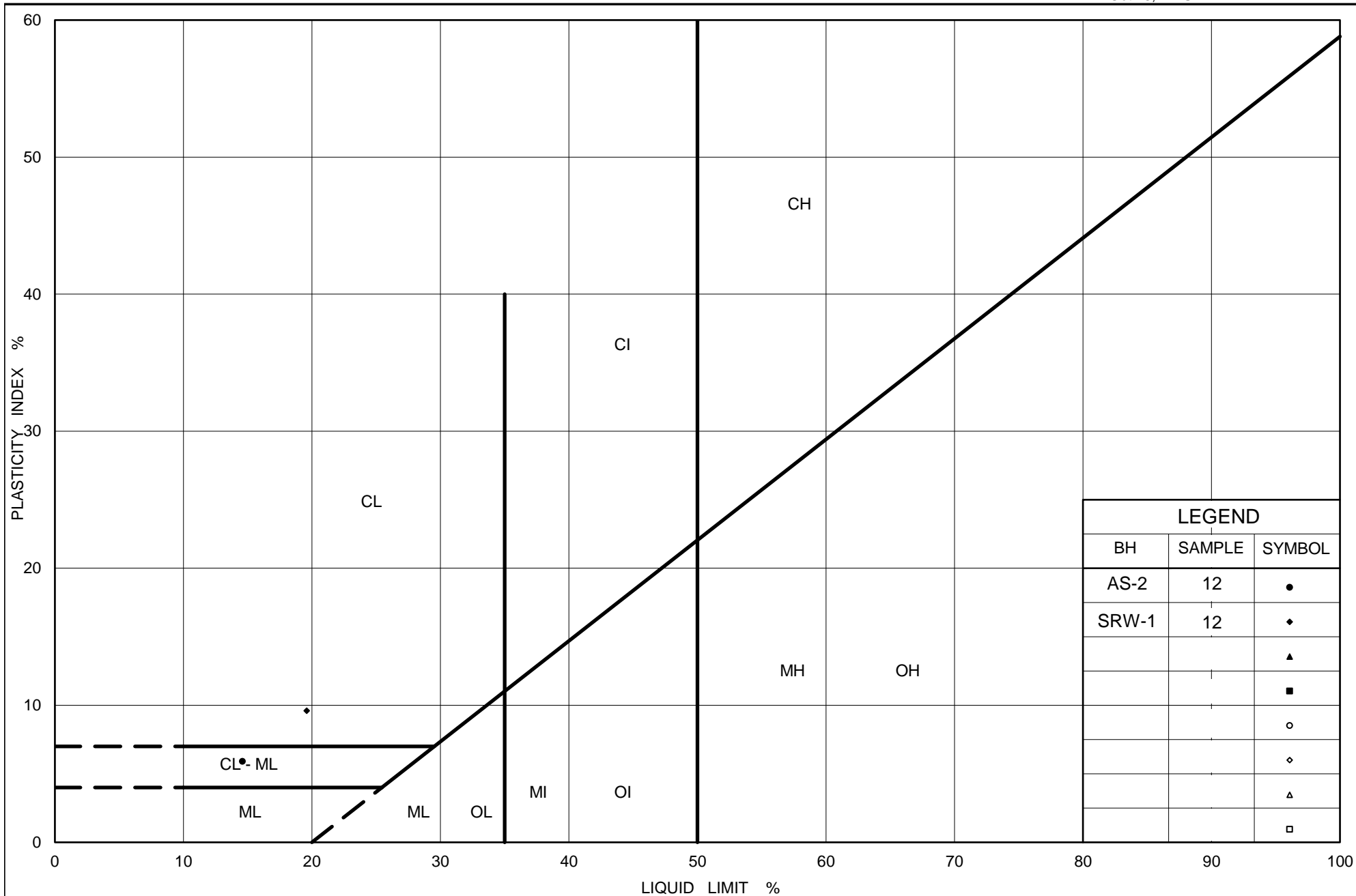
SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	SRW-1	12	89.1
■	AS-2	12	87.6

Project Number: 1662582

Checked By: ACK

Golder Associates

Date: 28-Aug-18



Ministry of Transportation

Ontario

PLASTICITY CHART Clayey Silt Till - South Retaining Wall

Figure No. B-14

Project No. 1662582

Checked By: ACK

APPENDIX C

Analytical Test Results

Your Project #: 1662582
Site Location: OSHAWA/ HWY401
Your C.O.C. #: 107484

Attention: Al Varshoi

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

Report Date: 2018/03/06
Report #: R5031865
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B845291

Received: 2018/02/28, 09:39

Sample Matrix: Soil
Samples Received: 2

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Reference
Chloride (20:1 extract)	2	N/A	2018/03/05	CAM SOP-00463	EPA 325.2 m
Conductivity	2	N/A	2018/03/06	CAM SOP-00414	OMOE E3530 v1 m
pH CaCl2 EXTRACT	2	2018/03/05	2018/03/05	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	2	2018/02/28	2018/03/06	CAM SOP-00414	SM 23 2510 m
Sulphate (20:1 Extract)	2	N/A	2018/03/05	CAM SOP-00464	EPA 375.4 m

Remarks:

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

All work recorded herein has been done in accordance with procedures and practices ordinarily exercised by professionals in Maxxam's profession using accepted testing methodologies, quality assurance and quality control procedures (except where otherwise agreed by the client and Maxxam in writing). All data is in statistical control and has met quality control and method performance criteria unless otherwise noted. All method blanks are reported; unless indicated otherwise, associated sample data are not blank corrected.

Maxxam Analytics' liability is limited to the actual cost of the requested analyses, unless otherwise agreed in writing. There is no other warranty expressed or implied. Maxxam has been retained to provide analysis of samples provided by the Client using the testing methodology referenced in this report. Interpretation and use of test results are the sole responsibility of the Client and are not within the scope of services provided by Maxxam, unless otherwise agreed in writing.

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

Results relate to samples tested.

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

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

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

Your Project #: 1662582
Site Location: OSHAWA/ HWY401
Your C.O.C. #: 107484

Attention: Al Varshoi

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

Report Date: 2018/03/06
Report #: R5031865
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B845291

Received: 2018/02/28, 09:39

Encryption Key

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

Ema Gitej, Senior Project Manager

Email: EGitej@maxxam.ca

Phone# (905)817-5829

=====

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

RESULTS OF ANALYSES OF SOIL

Maxxam ID		GEB172	GEB173		
Sampling Date		2017/11/06	2017/11/16		
COC Number		107484	107484		
	UNITS	AS5-SA6	AS7-SA8	RDL	QC Batch
Calculated Parameters					
Resistivity	ohm-cm	1700	1400		5419656
Inorganics					
Soluble (20:1) Chloride (Cl)	ug/g	300	28	20	5425533
Conductivity	umho/cm	578	727	2	5427146
Available (CaCl2) pH	pH	8.06	8.08		5423384
Soluble (20:1) Sulphate (SO4)	ug/g	33	770	20	5425542
RDL = Reportable Detection Limit					
QC Batch = Quality Control Batch					

Maxxam Job #: B845291
Report Date: 2018/03/06

Golder Associates Ltd
Client Project #: 1662582
Site Location: OSHAWA/ HWY401
Sampler Initials: LP

TEST SUMMARY

Maxxam ID: GEB172
Sample ID: AS5-SA6
Matrix: Soil

Collected: 2017/11/06
Shipped:
Received: 2018/02/28

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5425533	N/A	2018/03/05	Alina Dobreanu
Conductivity	AT	5427146	N/A	2018/03/06	Tahir Anwar
pH CaCl2 EXTRACT	AT	5423384	2018/03/05	2018/03/05	Tahir Anwar
Resistivity of Soil		5419656	2018/03/06	2018/03/06	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5425542	N/A	2018/03/05	Alina Dobreanu

Maxxam ID: GEB173
Sample ID: AS7-SA8
Matrix: Soil

Collected: 2017/11/16
Shipped:
Received: 2018/02/28

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5425533	N/A	2018/03/05	Alina Dobreanu
Conductivity	AT	5427146	N/A	2018/03/06	Tahir Anwar
pH CaCl2 EXTRACT	AT	5423384	2018/03/05	2018/03/05	Tahir Anwar
Resistivity of Soil		5419656	2018/03/06	2018/03/06	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5425542	N/A	2018/03/05	Alina Dobreanu

GENERAL COMMENTS

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

Package 1	6.3°C
-----------	-------

Samples submitted and analyzed past the recommended hold time.

Results relate only to the items tested.

QUALITY ASSURANCE REPORT

Golder Associates Ltd
Client Project #: 1662582
Site Location: OSHAWA/ HWY401
Sampler Initials: LP

QC Batch	Parameter	Date	Matrix Spike		SPIKED BLANK		Method Blank		RPD	
			% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
5423384	Available (CaCl ₂) pH	2018/03/05			99	97 - 103			1.5	N/A
5425533	Soluble (20:1) Chloride (Cl)	2018/03/05	NC	70 - 130	102	70 - 130	<20	ug/g	0.19	35
5425542	Soluble (20:1) Sulphate (SO ₄)	2018/03/05	NC	70 - 130	98	70 - 130	<20	ug/g	4.7	35
5427146	Conductivity	2018/03/06			98	90 - 110	<2	umho/cm	0.78	10

N/A = Not Applicable

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

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

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

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

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

VALIDATION SIGNATURE PAGE

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



Brad Newman, Scientific Service Specialist

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

CHAIN OF CUSTODY RECORD 107484 Page 1 of 1

Invoice Information		Report Information (if differs from invoice)		Project Information (where applicable)		Turnaround Time (TAT) Required	
Company Name: <u>Golder Associates Ltd.</u>		Company Name:		Quotation #:		<input checked="" type="checkbox"/> Regular TAT (5-7 days) Most analyses	
Contact Name: <u>Al Varshoi</u>		Contact Name:		P.O. #/ A/E#:		PLEASE PROVIDE ADVANCE NOTICE FOR RUSH PROJECTS	
Address: <u>8925 Century Ave. #100</u>		Address:		Project #: <u>10602582</u>		Rush TAT (Surcharges will be applied)	
City: <u>Mississauga ON</u>		City:		Site Location: <u>OSHAWA/HW40</u>		<input type="checkbox"/> 1 Day <input type="checkbox"/> 2 Days <input type="checkbox"/> 3-4 Days	
Phone: <u>905-567-4444</u>		Phone:		Site #:		Date Required:	
Email: <u>Al.Varshoi@golder.com</u>		Email:		Sampled By: <u>LP</u>		Rush Confirmation #:	
MOE REGULATED DRINKING WATER OR WATER INTENDED FOR HUMAN CONSUMPTION MUST BE SUBMITTED ON THE MAXXAM DRINKING WATER CHAIN OF CUSTODY							
Regulation 153		Other Regulations		Analysis Requested			
<input type="checkbox"/> Table 1 <input type="checkbox"/> Res/Park <input type="checkbox"/> Med/ Fine <input type="checkbox"/> Table 2 <input type="checkbox"/> Ind/Comm <input type="checkbox"/> Coarse <input type="checkbox"/> Table 3 <input type="checkbox"/> Agri/ Other <input type="checkbox"/> Table <input type="checkbox"/> FOR RSC (PLEASE CIRCLE) Y / N		<input type="checkbox"/> CCME <input type="checkbox"/> Sanitary Sewer Bylaw <input type="checkbox"/> MISA <input type="checkbox"/> Storm Sewer Bylaw <input type="checkbox"/> PWQJ <input type="checkbox"/> Region <input type="checkbox"/> Other (Specify) <input type="checkbox"/> REG 558 (MIN. 3 DAY TAT REQUIRED)		# OF CONTAINERS SUBMITTED PREL FILTERED (CIRCLE) Metals / Hg / Cu BTEX / PHE F1 PHES F2 - F4 VOCs REG 153 METALS & INORGANICS REG 153 ICPMS METALS REG 153 METALS (Hg, Cd, V, ICPMS Metals, HWS - B) Corrosivity			
Include Criteria on Certificate of Analysis: Y / N							
SAMPLES MUST BE KEPT COOL (< 10 °C) FROM TIME OF SAMPLING UNTIL DELIVERY TO MAXXAM							
SAMPLE IDENTIFICATION		DATE SAMPLED (YYYY/MM/DD)	TIME SAMPLED (HH:MM)	MATRIX	HOLD - DO NOT ANALYZE		
1	A55-SAB	2017/11/06	AM	SOIL	1		
2	A57-SAB	2017/11/16	AM	SOIL	1		
3							
4							
5							
6							
7							
8							
9							
10							
RELINQUISHED BY (Signature/Print)		DATE: (YYYY/MM/DD)	TIME: (HH:MM)	RECEIVED BY: (Signature/Print)		DATE: (YYYY/MM/DD)	TIME: (HH:MM)
<u>Kate Katik</u>		2018/02/28	9:38	<u>Matthew Tauran</u>		2018/02/28	09:38

28-Feb-18 09:39
Ema Gitej
B845291

TLI ENV-410

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 acknowledged. available for viewing at www.maxxam.ca/terms. Sample container, preservation, hold time and packages information can be viewed at <http://www.maxxam.ca/wp-content/uploads/Ontario-COC.pdf>.

COC-1004 (03/17)

White: Maxxam - Yellow: Client

Your Project #: 1662582
Site Location: OSHAWA/ HWY401
Your C.O.C. #: 107477

Attention: Al Varshoi

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

Report Date: 2018/03/06
Report #: R5031866
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B845302

Received: 2018/02/28, 09:39

Sample Matrix: Soil
Samples Received: 3

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Reference
Chloride (20:1 extract)	3	N/A	2018/03/05	CAM SOP-00463	EPA 325.2 m
Conductivity	3	N/A	2018/03/06	CAM SOP-00414	OMOE E3530 v1 m
pH CaCl ₂ EXTRACT	3	2018/03/05	2018/03/05	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	3	2018/02/28	2018/03/06	CAM SOP-00414	SM 23 2510 m
Sulphate (20:1 Extract)	3	N/A	2018/03/05	CAM SOP-00464	EPA 375.4 m

Remarks:

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

All work recorded herein has been done in accordance with procedures and practices ordinarily exercised by professionals in Maxxam's profession using accepted testing methodologies, quality assurance and quality control procedures (except where otherwise agreed by the client and Maxxam in writing). All data is in statistical control and has met quality control and method performance criteria unless otherwise noted. All method blanks are reported; unless indicated otherwise, associated sample data are not blank corrected.

Maxxam Analytics' liability is limited to the actual cost of the requested analyses, unless otherwise agreed in writing. There is no other warranty expressed or implied. Maxxam has been retained to provide analysis of samples provided by the Client using the testing methodology referenced in this report. Interpretation and use of test results are the sole responsibility of the Client and are not within the scope of services provided by Maxxam, unless otherwise agreed in writing.

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

Results relate to samples tested.

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

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

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

Your Project #: 1662582
Site Location: OSHAWA/ HWY401
Your C.O.C. #: 107477

Attention: Al Varshoi

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

Report Date: 2018/03/06
Report #: R5031866
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B845302

Received: 2018/02/28, 09:39

Encryption Key

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

Ema Gitej, Senior Project Manager

Email: EGitej@maxxam.ca

Phone# (905)817-5829

=====

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

RESULTS OF ANALYSES OF SOIL

Maxxam ID		GEB219	GEB220		GEB221		
Sampling Date		2017/11/03	2017/10/30		2017/11/06		
COC Number		107477	107477		107477		
	UNITS	SS2-SA10	SS8-SA10	RDL	SS2-SA3	RDL	QC Batch
Calculated Parameters							
Resistivity	ohm-cm	3400	2300		380		5419656
Inorganics							
Soluble (20:1) Chloride (Cl)	ug/g	28	140	20	1100	40	5425533
Conductivity	umho/cm	297	432	2	2600	2	5427146
Available (CaCl2) pH	pH	7.94	8.12		7.72		5423384
Soluble (20:1) Sulphate (SO4)	ug/g	150	140	20	160	20	5425542
RDL = Reportable Detection Limit							
QC Batch = Quality Control Batch							

Maxxam Job #: B845302
Report Date: 2018/03/06

Golder Associates Ltd
Client Project #: 1662582
Site Location: OSHAWA/ HWY401
Sampler Initials: LP

TEST SUMMARY

Maxxam ID: GEB219
Sample ID: SS2-SA10
Matrix: Soil

Collected: 2017/11/03
Shipped:
Received: 2018/02/28

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5425533	N/A	2018/03/05	Alina Dobreanu
Conductivity	AT	5427146	N/A	2018/03/06	Tahir Anwar
pH CaCl2 EXTRACT	AT	5423384	2018/03/05	2018/03/05	Tahir Anwar
Resistivity of Soil		5419656	2018/03/06	2018/03/06	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5425542	N/A	2018/03/05	Alina Dobreanu

Maxxam ID: GEB220
Sample ID: SS8-SA10
Matrix: Soil

Collected: 2017/10/30
Shipped:
Received: 2018/02/28

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5425533	N/A	2018/03/05	Alina Dobreanu
Conductivity	AT	5427146	N/A	2018/03/06	Tahir Anwar
pH CaCl2 EXTRACT	AT	5423384	2018/03/05	2018/03/05	Tahir Anwar
Resistivity of Soil		5419656	2018/03/06	2018/03/06	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5425542	N/A	2018/03/05	Alina Dobreanu

Maxxam ID: GEB221
Sample ID: SS2-SA3
Matrix: Soil

Collected: 2017/11/06
Shipped:
Received: 2018/02/28

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5425533	N/A	2018/03/05	Alina Dobreanu
Conductivity	AT	5427146	N/A	2018/03/06	Tahir Anwar
pH CaCl2 EXTRACT	AT	5423384	2018/03/05	2018/03/05	Tahir Anwar
Resistivity of Soil		5419656	2018/03/06	2018/03/06	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5425542	N/A	2018/03/05	Alina Dobreanu

GENERAL COMMENTS

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

Package 1	6.3°C
-----------	-------

Samples submitted and analyzed past the recommended hold time.

Results relate only to the items tested.

QUALITY ASSURANCE REPORT

Golder Associates Ltd
Client Project #: 1662582
Site Location: OSHAWA/ HWY401
Sampler Initials: LP

QC Batch	Parameter	Date	Matrix Spike		SPIKED BLANK		Method Blank		RPD	
			% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
5423384	Available (CaCl ₂) pH	2018/03/05			99	97 - 103			1.5	N/A
5425533	Soluble (20:1) Chloride (Cl)	2018/03/05	NC	70 - 130	102	70 - 130	<20	ug/g	0.19	35
5425542	Soluble (20:1) Sulphate (SO ₄)	2018/03/05	NC	70 - 130	98	70 - 130	<20	ug/g	4.7	35
5427146	Conductivity	2018/03/06			98	90 - 110	<2	umho/cm	0.78	10

N/A = Not Applicable

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

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

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

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

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

VALIDATION SIGNATURE PAGE

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



Brad Newman, Scientific Service Specialist

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

CHAIN OF CUSTODY RECORD

107477 Page 1 of 1

Invoice Information Company Name: <u>Golden Associates Ltd.</u> Contact Name: <u>[REDACTED]</u> Address: <u>6925 Century Ave. #100</u> <u>MISSISSAUGA ON</u> Phone: <u>905-567-4444</u> Fax: <u>[REDACTED]</u> Email: <u>[REDACTED]</u>		Report Information (if differs from invoice) Company Name: <u>[REDACTED]</u> Contact Name: <u>Al Varshoi</u> Address: <u>[REDACTED]</u> Phone: <u>[REDACTED]</u> Fax: <u>[REDACTED]</u> Email: <u>Al-Varshoi@golder.com</u>		Project Information (where applicable) Quotation #: <u>[REDACTED]</u> P.O. #/ AFE #: <u>[REDACTED]</u> Project #: <u>1602582</u> Site Location: <u>OSHANA/HW/401</u> Site #: <u>[REDACTED]</u> Sampled By: <u>LP</u>		Turnaround Time (TAT) Required <input checked="" type="checkbox"/> Regular TAT (5-7 days) Most analyses PLEASE PROVIDE ADVANCE NOTICE FOR RUSH PROJECTS Rush TAT (Surcharges will be applied) <input type="checkbox"/> 1 Day <input type="checkbox"/> 2 Days <input type="checkbox"/> 3-4 Days Date Required: <u>[REDACTED]</u> Rush Confirmation #: <u>[REDACTED]</u>	
MOE REGULATED DRINKING WATER OR WATER INTENDED FOR HUMAN CONSUMPTION MUST BE SUBMITTED ON THE MAXXAM DRINKING WATER CHAIN OF CUSTODY							
Regulation 153 <input type="checkbox"/> Table 1 <input type="checkbox"/> Res/Park <input type="checkbox"/> Med/Fine <input type="checkbox"/> Table 2 <input type="checkbox"/> Ind/Comm <input type="checkbox"/> Coarse <input type="checkbox"/> Table 3 <input type="checkbox"/> Agri/ Other <input type="checkbox"/> Table <u>[REDACTED]</u> FOR RSC (PLEASE CIRCLE) Y / N		Other Regulations <input type="checkbox"/> CCME <input type="checkbox"/> Sanitary Sewer Bylaw <input type="checkbox"/> MISA <input type="checkbox"/> Storm Sewer Bylaw <input type="checkbox"/> FWQO <input type="checkbox"/> Region: <u>[REDACTED]</u> <input type="checkbox"/> Other (Specify) <u>[REDACTED]</u> <input type="checkbox"/> REG 558 (MIN. 3 DAY TAT REQUIRED)		Analysis Requested FIELD FILTERED (CIRCLE) Metals / lig / CVI BTEX / HCH F1 PHG P2 P4 VOCs REG 153 METALS & INORGANICS REG 153 ICPMS METALS REG 153 METALS (Pb, Cr VI, ICPMS Metals, HWS - B) Corrosivity Package		LABORATORY USE ONLY CUSTODY SEAL Y / N Present Intact COOLER TEMPERATURES <u>6/6/7</u> COOLING MEDIA PRESENT: Y / N COMMENTS	
Include Criteria on Certificate of Analysis: Y / N SAMPLES MUST BE KEPT COOL (< 10 °C) FROM TIME OF SAMPLING UNTIL DELIVERY TO MAXXAM							
SAMPLE IDENTIFICATION		DATE SAMPLED (YYYY/MM/DD)	TIME SAMPLED (HH:MM)	MATRIX	# OF CONTAINERS SUBMITTED	FIELD FILTERED (CIRCLE) Metals / lig / CVI	OTHER ANALYSES
1	SS2-SA10	2017/11/03	AM	SOIL	1		X
2	SS2-SA10	2017/11/03	AM	SOIL	1		X
3	SS8-SA10	2017/10/30	AM	SOIL	1		X
4	SS2-SA3	2017/11/06	AM	SOIL	1		X
5							
6							
7							
8							
9							
10							

RELINQUISHED BY: (Signature/Print)	DATE: (YYYY/MM/DD)	TIME: (HH:MM)	RECEIVED BY: (Signature/Print)	DATE: (YYYY/MM/DD)	TIME: (HH:MM)
<u>Katherine Karkhanavich</u>	<u>2018/02/28</u>	<u>9:38</u>	<u>[Signature]</u>	<u>2018/02/28</u>	<u>09:39</u>

28-Feb-18 09:39

Ema Gitej



B845302

TLI

ENV-410

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 for viewing at www.maxxam.ca/terms. Sample container, preservation, hold time and packages information can be viewed at <http://www.maxxam.ca/wp-content/uploads/Ontario-COC.pdf>.

APPENDIX D

Non-Standard Special Provisions

SUBGRADE PROTECTION - Item No.

Non-Standard Special Provision

The subgrade soils of the concrete cantilever retaining wall will be susceptible to disturbance and loosening from construction traffic and ponded water.

If the concrete for the footings on the native or engineered fill soil cannot be poured immediately after excavation and within three hours of its inspection and approval, a working mat of lean concrete or mass concrete, with minimum thickness of 100 mm, should be placed on the foundation subgrade in general accordance with OPSS.PROV 904 (*Concrete Structures*). The lean concrete shall have a compressive strength of 20 MPa. A minimum 75 mm thick uncompacted levelling pad consisting of Granular 'A' material or fine aggregates (meeting the grading requirements specified in OPSS.PROV 1010 or OPSS.PROV 1002) should be provided on top of the lean concrete mat.

Basis of Payment

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

END OF SECTION

OBSTRUCTIONS – Item No.

Non-Standard Special Provision – Notice to Contractor

The Contactor shall be alerted to the potential presence of cobbles and boulders within the glacial till and very dense sand deposits at this site. Considerations of the presence of these obstructions must be made in the selection of appropriate equipment and procedures for and installation of secant wall or soldier pile and lagging caissons.

AMENDMENT TO OPSS 942, NOVEMBER 2009

Special Provision No. 109S58

March 2018

942.03 DEFINITIONS

Section 942.03 of OPSS 942 is amended by the deletion of the definitions for **Certificate of Conformance** and **Quality Verification Engineer**.

942.04 DESIGN AND SUBMISSION REQUIREMENTS

942.04.02.05 Milestone Inspections

Clause 942.04.02.05 of OPSS 942 shall be deleted in its entirety.

942.07 CONSTRUCTION

942.07.07 Anchor Installation

Subsection 942.07.07 of OPSS 942 is amended by the addition of the following clause:

942.07.07.01 Inspection during Anchor Installation

The Contractor's Engineer shall inspect the following Work:

- a) Construction of anchor holes.
- b) Anchor installation.
- c) Primary grouting.
- d) Post grouting.
- e) Placement of slurry in free stressing length.
- f) Anchorage installation.

942.07.12 Testing

942.07.12.01 General

Clause 942.07.12.01 of OPSS 942 is amended by deleting the last sentence in its entirety and replacing it with the following:

The Contractor's Engineer shall inspect the pre-production and production anchor testing.

942.07.12.06.05 Lift-Off Tests

Clause 942.07.12.06.05 of OPSS 942 is amended by deleting the first paragraph in its entirety and replacing it with the following:

A minimum of 3 lift-off tests shall be conducted at each site. The location of the anchor to be tested shall be as determined by the Contractor. The lift-off test shall not be performed until 48 hours has elapsed after transferring the lock-off load. The method of testing shall be as detailed on the Working Drawings.

942.07.13.03 Certificate of Conformance

Clause 942.07.13.03 of OPSS 942 shall be deleted and replaced by the following:

942.07.13.03 Inspection after Anchor Installation

The Contractor's Engineer shall inspect and verify that the materials have been supplied and installed according to the Contract Documents.

A Certificate of Conformance shall be submitted to the Contract Administrator upon completion of the anchor installation.

The Certificate of Conformance shall be submitted to the Contract Administrator upon completion of the stressing operations.

DEWATERING SYSTEM - Item No.
TEMPORARY FLOW PASSAGE SYSTEM - Item No.

Special Provision No. 517F01

July 2017

Amendment to OPSS 517, November 2016

Design Storm Return Period and Preconstruction Survey Distance

517.01 SCOPE

Section 517.01 of OPSS 517 is deleted in its entirety and replaced with the following:

This specification covers the requirements for the design, operation, and removal of a dewatering or temporary flow passage system or both to control water during construction, and the control of the water prior to discharge to the natural environment and sewer systems.

517.04 DESIGN AND SUBMISSION REQUIREMENTS

517.04.01 Design Requirements

Subsection 517.04.01 of OPSS 517 is amended by deleting the first paragraph in its entirety and replacing it with the following:

A dewatering or temporary flow passage system or both shall be designed to control water at the locations specified in the Contract Documents and at any other location where a system is necessary to complete the work. The design of the system shall be sufficient to permit the work at each location to be carried out as specified in the Contract Documents.

Subsection 517.04.01 of OPSS 517 is further amended by deleting the second last paragraph in its entirety and replacing it with the following:

Temporary flow passage systems shall be designed, as a minimum, for a 2 year design storm return period and groundwater discharge, except for the work specified in Table A. For the work specified in Table A, the temporary flow passage system shall be designed, as a minimum, for the design storm return period specified in Table A and groundwater discharge. A longer return period shall be used when determined appropriate for the work.

Intensity-Duration Factor (IDF) curve location, site specific minimum return period, return period flow estimates, and other information is provided in Table A. The IDF information can be accessed through the MTO IDF Curve Look up Tool on the Drainage and Hydrology page of MTO's website. The return period flow estimates do not include flow volumes from groundwater discharge. The Owner specifically excludes these flow estimates from the warranty in the Reliance on Contract Documents subsection of OPSS 100, MTO General Conditions of Contract.

Table A

IDF Curve Location	Latitude: *		Longitude: *			
Temporary Flow Passage Systems						
Site Name / Station Reference	Minimum Return Period (Years)	Return Period Flow Estimates (m ³ /s)				Design Engineer Requirements (Note 1)
		2 Year	5 Year	10 Year	25 Year	
**	***	****	****	****	****	*****
Dewatering Systems						
Site Name / Station Reference	Preconstruction Survey Distance (Note 2) (m)				Design Engineer Requirements (Note 1)	
**	*****				*****	
<p>Note:</p> <p>1. “Yes” means the design Engineer and design-checking Engineer shall have a minimum of 5 years of experience in designing systems of similar nature and scope to the required work. “No” means a minimum experience level is not required for the design Engineer and design-checking Engineer.</p> <p>2. “N/A” indicates a preconstruction survey is not required.</p>						



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