



Foundation Investigation and Design Report

Replacement of Simcoe Street Underpass (Site No. 22X-176/B0)

Highway 401, Replacement of Three Underpasses and Rehabilitation of Oshawa Creek Bridge, Region of Durham, Ontario

Ministry of Transportation, Ontario G.W.P. 2298-13-00

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Certificate of Analysis Report # R5031866

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PART A

FOUNDATION INVESTIGATION REPORT

REPLACEMENT OF SIMCOE STREET UNDERPASS (SITE NO. 22X-176/B0)

REPLACEMENT OF THREE UNDERPASSES AND REHABILITATION OF
OSHAWA CREEK BRIDGE, HIGHWAY 401, OSHAWA, ONTARIO

MTO 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 replacement of underpass structures and culverts, and rehabilitation of a creek crossing structure, associated with the improvements and future widening of the Highway 401 in the Oshawa area, in the Regional Municipality of Durham, Ontario.

This report addresses the proposed replacement of the existing Simcoe Street Underpass (MTO Structure Site No. 22X-176/B0) and approach embankments. The purpose of this investigation is to establish the subsurface soil and bedrock conditions at the proposed Simcoe Street Underpass replacement location, including the associated approach embankments, by borehole drilling and in situ and laboratory testing on selected soil samples. The results of the foundation investigations at other structures associated with the widening and structure replacement are presented in separate reports.

2.0 SITE DESCRIPTION

The Simcoe Street underpass carries northbound and southbound traffic over the eastbound and westbound lanes of Highway 401 and is located in the City of Oshawa, in the Regional Municipality of Durham, Ontario. The existing Simcoe Street underpass is a two-span structure with a total span length of about 29 m. The underpass was constructed in 1941 and is founded on spread footings at about Elevation 93.9 m at the abutments and pier. Based on visual observations, the existing abutments are considered to have performed satisfactorily.

The natural ground surface at the site is at about Elevation 100 m to 102 m. Highway 401 was constructed in a cut with the highway grade at about Elevation 96 m. The road grade of Simcoe Street at the site ranges from about Elevation 101 m to 103 m, rising northward at the structure site. A commercial business is located in the northeast quadrant of the site, the Oshawa Visitor Information Centre is located in the southeast quadrant of the site, and greenspace areas are present in the northwest and southwest quadrants of the site.

3.0 INVESTIGATION PROCEDURES

3.1 Previous Investigation

In March 2015, Golder carried out a preliminary foundation investigation at the site, during which time two boreholes (designated as Boreholes S1 and S2) were advanced at the northeast and southwest quadrants of the existing underpass, as shown on Drawing 1. The boreholes were advanced to depths of approximately 6.8 m and 15.7 m below existing ground surface, respectively. A standpipe piezometer was installed in Borehole S2 to allow for monitoring of the groundwater level at the site.

The results of the investigation are presented in Golder's report 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. 10-2001"*, dated May 19, 2017 (GEOCRE 30M14-451).

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 are provided on the borehole records in Appendix A and shown on Drawing 1. The locations are positioned relative to MTM NAD 83 northing and easting coordinates and the ground surface elevations are referenced to Geodetic datum. The borehole locations, ground surface elevations, and borehole depths are summarized in the following table.

Borehole Number	MTM NAD83 Northing (m) (Latitude, °)	MTM NAD83 Easting (m) (Longitude, °)	Ground Surface Elevation (m)	Borehole Depth (m)
S1	4,860,398.8 (43.881758)	356,528.6 (-78.856271)	101.3	6.8
S2	4,860,328.0 (43.881121)	356,527.1 (-78.856297)	100.4	15.7

3.2 Current Investigation

The field work for the current investigation was carried out between October 20, 2017 and March 19, 2018 and between January 27 to 31, 2022, during which time a total of eleven boreholes (designated as Boreholes SS-1 to SS-11) were advanced in the vicinity of the foundation elements and approach embankments as shown on Drawing 1. The boreholes were advanced to depths ranging from 9.4 m to 21.4 m.

The investigation was carried out using truck-mounted CME 75 drill rigs, supplied and operated by Pontil Drilling of Mount Albert, Ontario and Walker Drilling of Utopia, Ontario. The boreholes were advanced through the overburden using 200 mm and 216 mm outside diameter hollow-stem augers. Soil samples were generally obtained at 0.75 m and 1.5 m intervals of depth, using a 50 mm outside diameter split-spoon sampler driven by an automatic hammer in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586)¹. Borehole SS-11 was advanced without sampling to about 7.6 m depth (Elevation 88.2 m) before SPT and soil sampling was carried out at sampling intervals between 0.75 m and 1.5 m to the top of bedrock. Upon encountering the shale bedrock at a depth of about 12.5 m below ground surface, 3.4 m of bedrock was cored using HQ coring equipment. 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. In situ shear vane tests were carried out in soft to firm cohesive soils.

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-3 and SS-9 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, as 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 selected soil samples, to MTO and / or ASTM Standards, as appropriate. Unconfined Compressive Strength (UCS) testing was carried out on a sample of the bedrock core by Geomechanics of Toronto, Ontario. In addition, selected soil samples were submitted to Maxxam Analytics (Maxxam) of Mississauga, Ontario for analysis of select parameters to assess corrosion potential.

The borehole locations are provided on the borehole records in Appendix A and shown on Drawing 1. The locations are positioned relative to MTM NAD 83 northing and easting coordinates (Zone 10 CSRS CBNv6-2010.0) and the

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

ground surface elevations are referenced to Geodetic datum (CGVD28 / HT2_0). The borehole locations, ground surface elevations, and borehole depths are summarized in the following table.

Borehole Number	Location	MTM NAD83 Northing (Latitude, °)	MTM NAD83 Easting (Longitude, °)	Ground Surface Elevation (m)	Borehole Depth (m)
SS-1	South Embankment	4,860,303.5 (43.880898)	356,553.7 (-78.855968)	100.4	11.1
SS-2	South Abutment	4,860,319.6 (43.881044)	356,549.0 (-78.856025)	100.8	18.8
SS-3		4,860,315.8 (43.881011)	356,535.9 (-78.856188)	100.4	21.4
SS-4	South Pier	4,860,339.7 (43.881226)	356,540.4 (-78.856130)	101.0	12.7
SS-5		4,860,336.9 (43.881200)	356,532.3 (-78.856231)	100.8	11.1
SS-6	North Pier	4,860,359.2 (43.881402)	356,515.7 (-78.856435)	96.0	9.4
SS-7		4,860,365.7 (43.881459)	356,541.9 (-78.856109)	96.0	9.5
SS-8	North Abutment	4,860,396.0 (43.881734)	356,523.4 (-78.856336)	101.2	10.8
SS-9		4,860,394.2 (43.881718)	356,513.4 (-78.856461)	101.1	11.1
SS-10	North Embankment	4,860,412.3 (43.881882)	356,506.7 (-78.856542)	100.3	11.1
SS-11	North Pier	4,860,365.0 (43.881455)	356,512.3 (-78.856477)	95.8	15.9

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. The Plain 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.

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.

4.2 Subsurface Conditions

Subsurface soil, bedrock and groundwater conditions as encountered in the boreholes are presented on the borehole records in Appendix A. In addition, bedrock core photos are included in Appendix A. The geotechnical laboratory results and analytical laboratory results are presented in Appendix B and C, respectively.

The results of in situ field tests (i.e., SPT “N” -values and shear strength 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 and across the structure, as shown on Drawings 1 and 2, are simplifications of the subsurface conditions. Variation in the stratigraphic boundaries between and beyond boreholes will exist and is to be expected.

In general, the subsurface conditions generally consist of asphalt and non-cohesive fill, underlain by a deposit of clay to clayey silt and/or an upper deposit of silt and sand till. The clay to clayey silt/upper silt and sand till is underlain by silt and sand to sand and/or gravelly sand to sand and gravel, which in turn is underlain by a lower deposit of silt and sand to gravelly silt and sand till and/or a clayey silt with sand till overlying shale bedrock.

A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Topsoil

An approximately 400 mm and 500 mm thick layer of topsoil was encountered at ground surface in Boreholes S1 and S2, respectively.

4.2.2 Asphalt

An approximately 120 mm to 250 mm thick layer of asphalt pavement was encountered at ground surface in all boreholes, excluding Boreholes S1 and S2.

4.2.3 Sand to Gravelly Sand Fill

A 0.5 m to 3.4 m thick layer of non-cohesive fill was encountered underlying the topsoil and/or asphalt at all borehole locations, except at Borehole S2. The non-cohesive fill, which consists of sand to gravelly sand, some silt, extends to depths of 0.7 m to 3.7 m below ground surface (Elevations 100.5 m to 92.3 m). In Boreholes SS-6, the non-cohesive fill was silty and contained clayey silt pockets and hydrocarbon odours.

The SPT “N”-values measured within the non-cohesive fill range from 2 to 53 blows per 0.3 m of penetration, indicating a very loose to very dense state of compactness.

The water contents measured on samples of the non-cohesive fill range from about 3% to 10%.

4.2.4 Silty Clay to Clayey Silt with Sand Fill

A 0.6 m to 1.8 m thick layer of cohesive fill was encountered underlying the topsoil / non-cohesive fill in Boreholes S2, SS-2, SS-4, SS-5, and SS-8 to SS-10. The cohesive fill was encountered at depths of 0.5 m to 1.5 m (Elevations 100.5 m to 99.5 m) and extended to depths of 1.4 m to 3.0 m (Elevations 99.8 m to 97.8 m). The cohesive fill consists of silty clay to clayey silt with sand, and trace gravel. In Borehole S2, trace amounts of organics were encountered within the cohesive fill.

The SPT “N”-values measured within the cohesive fill range from 4 to 18 blows per 0.3 m of penetration, suggesting a firm to very stiff consistency.

Grain size distribution testing was carried out on one sample of the clay to clayey silt with sand deposit and the results are presented on Figure B1 in Appendix B. Atterberg limit testing was carried out on four samples of the cohesive fill and the results are presented on Figure B2 in Appendix B. The Atterberg limit tests measured liquid limits ranging from 16% to 46%, plastic limits ranging from 10% to 19%, and plasticity indices ranging from 6% to 27%, indicating the cohesive fill is a silty clay to clayey silt of low to medium plasticity. The water contents measured on samples of the cohesive fill range from about 10% to 44%.

4.2.5 Clay to Clayey Silt with Sand

A 1.1 m to 5.9 m thick deposit of clay to clayey silt with sand was encountered underlying the topsoil / fill along Simcoe Street in Boreholes S1, S2, SS-1, SS-3, and SS-5. The deposit generally thickens toward the south, and toward the west. The cohesive deposit was encountered at depths of 0.4 m to 2.1 m (Elevations 100.9 m to 98.3 m) and extended to depths of 1.5 m to 7.1 m (Elevations 99.8 m to 93.3 m). The deposit consists of clay to clayey silt with sand, trace to some gravel.

The SPT “N”-values measured within the clay to clayey silt with sand deposit range from 2 to 21 blows per 0.3 m of penetration, suggesting a soft to very stiff consistency. In situ shear vane tests measured within the deposit range from 77 kPa to over 100 kPa, indicating a stiff to very stiff consistency at the test locations.

Grain size distribution testing was carried out on four samples of the clay to clayey silt with sand deposit and the results are presented on Figure B3 in Appendix B. Atterberg limit testing was carried out on seven samples of the deposit and measured liquid limits ranging from 16% to 52%, plastic limits ranging from 10% to 19%, and plasticity indices ranging from 6% to 33%. The Atterberg limit test results are presented on Figure B4 in Appendix B and indicate the clay to silty clay with sand ranges from low to high plasticity. The water content measured on samples of the deposit range from about 9% to 36%.

4.2.6 Silt and Sand to Silty Sand Till – Upper Deposit

A 0.2 m to 4.2 m thick upper non-cohesive glacial deposit was encountered underlying the fill / silty clay to clayey silt with sand deposit in Boreholes SS-4, SS-5, and SS-8 to SS-10. The upper non-cohesive till deposit was encountered at depths of 1.4 m to 7.5 m (Elevations 99.8 m to 93.5 m) and extended to depths of 4.4 m to 11.7 m (Elevations 96.8 m to 89.3 m). The deposit consists of silt and sand to silty sand, trace gravel to gravelly. Although not encountered, cobbles and boulders are commonly encountered in glacially derived materials and should be expected within this deposit.

The SPT “N”-values measured within the upper non-cohesive till range from 21 blows per 0.3 m of penetration to 100 blows per 0.1 m of penetration, indicating a compact to very state dense of compactness.

Grain size distribution testing was carried out on five samples of the non-cohesive till and the results are presented on Figure B5 in Appendix B. Atterberg limit testing was carried out on two samples of the deposit and one sample measured a liquid limit of 14%, a plastic limit of 10%, and a plasticity index of 4%; while the other sample was non-plastic. The Atterberg limit test results are presented on Figure B6 in Appendix B and indicate the upper non-cohesive till deposit ranges from non-plastic to slightly plastic. The water content measured on samples of the deposit range from about 4% to 11%.

4.2.7 Silt and Sand to Sand – Upper Deposit

An upper 1.5 m to 3.7 m thick deposit of silt and sand to sand was encountered underlying the fill / clay to clayey silt deposit south of Highway 401 in Boreholes S2, and SS-1 to SS-4. The upper silt and sand to sand deposit was encountered at depths of 2.1 m to 7.1 m (Elevations 98.9 m to 93.3 m) and extended to depths of 6.7 m to 8.6 m (Elevations 94.1 m to 91.8 m). The upper deposit consists of silt and sand to sand, trace to some gravel, trace to some clay. Hydrocarbon odours were noted within the silt and sand deposit in Borehole SS-3.

The SPT “N”-values measured within the upper silt and sand to sand deposit range from 16 to 108 blows per 0.3 m of penetration, with one measurement of 61 blows per 0.8 m of penetration and one measurement of 126 blows per 0.25 m of penetration, indicating a compact to very dense state of compactness.

Grain size distribution testing was carried out on two samples of the silt and sand to sand deposit; the results are presented on Figure B7 in Appendix B. The water content measured on samples of the deposit range from about 3% to 17%.

4.2.8 Gravelly Sand to Sand and Gravel

A 0.8 m to 5.5 m thick deposit of gravelly sand to sand and gravel was encountered underlying the upper silt and sand to silty sand till and upper silt and sand to sand deposit at all boreholes, except Boreholes SS-4 and SS-7. The gravelly sand to sand and gravel deposit was encountered at depths of 3.7 m to 8.6 m (Elevations 96.8 m to 91.8 m) and extended to depths of 4.5 m to 12.2 m (Elevations 94.7 m to 88.2 m). The deposit consists of gravelly sand to sand and gravel, trace to some silt with an interlayer of sand encountered in Borehole SS-8.

The SPT “N”-values measured within the gravelly sand to sand and gravel deposit range from 35 blows per 0.3 m of penetration to 100 blows per 0.1 m of penetration, indicating a compact to very dense state of compactness.

Grain size distribution testing was carried out on nine samples of the gravelly sand to sand and gravel deposit and the results are presented on Figures B8A and B8B in Appendix B. In addition, grain size distribution testing was carried out on the sample of sand interlayer and the results are presented on Figure B9 in Appendix B. The water content measured on samples of the deposit range from about 1% to 11%.

4.2.9 Silt and Sand Till – Lower Deposit

A 4.9 m and 5.7 m thick lower non-cohesive glacial deposit was encountered underlying the gravelly sand to sand and gravel in Boreholes SS-6, SS-7, and SS-11. The lower non-cohesive till deposit was encountered at depths of 2.9 m and 4.5 m (Elevations 93.1 m and 91.5 m) and extended to depths of 8.6 m to 8.7 m (Elevations 97.4 m to 87.1 m), or to the borehole termination depth of 9.4 m (Elevations 86.6 m). The deposit consists of silt and sand, trace to some gravel. Although not encountered, cobbles and boulders are commonly encountered in glacially derived materials and should be expected within this deposit.

The SPT “N”-values measured within the lower non-cohesive till range from 53 blows per 0.3 m of penetration to 100 blows per 0.1 m of penetration, indicating a very state dense of compactness.

Grain size distribution testing was carried out on four samples of the lower non-cohesive till and the results are presented on Figure B10 in Appendix B. Atterberg limit testing was carried out on one sample of the deposit and indicate the material is non-plastic. The water content measured on samples of the deposit range from about 7% to 10%.

4.2.10 Clayey Silt to Clayey Silt with Sand Till

A 0.9 m to 7.7 m thick cohesive glacial deposit was encountered generally underlying the gravelly sand to sand and gravel and non-cohesive till at all boreholes, except Boreholes SS-6 and SS-7. The cohesive till deposit was encountered at depths of 6.1 m to 11.7 m (Elevations 94.3 m to 86.0 m) and extended to depths of 7.0 m to 17.4 m (Elevations 93.4 m to 83.0 m). The deposit consisted of clayey silt to clayey silt with sand, trace gravel. Although not encountered, cobbles and boulders are commonly encountered in glacially derived materials and should be expected within this deposit.

The SPT “N”-values measured within the cohesive till range from 38 blows per 0.3 m of penetration to 100 blows per 0.03 m of penetration, suggesting a hard consistency.

Grain size distribution testing was carried out on eight samples of the cohesive till and the results are presented on Figures B11A and B11B in Appendix B. Atterberg limit testing was carried out on ten samples of the deposit and measured liquid limits ranging from 14% to 31%, plastic limits ranging from 8% to 12%, and plasticity indices ranging from 6% to 17%. The Atterberg limit test results are presented on Figure B12 and indicate the cohesive till is a clayey silt of low plasticity. The water content measured on samples of the deposit range from about 7% to 15%.

4.2.11 Silt to Sand – Lower Deposit

A lower 0.7 m to 0.9 m thick deposit of silt to sand was encountered underlying the lower silt and sand till deposit and clayey silt to clayey silt with sand till deposit in Boreholes SS-2, SS-7, SS-8, and SS-11. The lower silt to sand deposit was encountered at depths of 8.6 m to 16.2 m (Elevations 91.1 m to 84.6 m) and extended to depths of 9.5 m to 17.1 m (Elevations 91.4 m to 83.7 m). The lower deposit consists of sand, trace silt, trace gravel, trace clay, at Boreholes SS-2 and S-7, consists of silty sand at Borehole SS-11, and consists of silt at Borehole SS-8.

The SPT “N”-values measured within the lower silt to sand deposit range from 140 blow per 0.22 m of penetration to 120 blows per 0.18 m of penetration, indicating a very dense state of compactness.

Grain size distribution testing was carried out on three samples of the lower silt to sand deposit; the results are presented on Figure B13 in Appendix B. The water content measured on samples of the deposit range from about 8% to 20%.

4.2.12 Shale Bedrock

Bedrock was encountered in Boreholes SS-2, SS-3, and SS-11, at depths of 17.1 m, 17.4 m, and 12.5 m (Elevations 83.0 m, 83.7 m, and 83.3 m), respectively. Based on the cored bedrock samples from Borehole SS-11, the bedrock consists of fresh, thinly bedded, black, fine grained, faintly porous, strong shale / limestone of the Whitby Formation. Residual soil layers were encountered within the bedrock at Borehole SS-3 and hydrocarbon odours were encountered in the bedrock in both Boreholes SS-2 and SS-3.

The degree of weathering of the bedrock samples and the strength classification of the intact rock mass based on field identification are described in accordance with the International Society for Rock Mechanics (ISRM⁴) standard classification system. The Rock Quality Designation (RQD) measured on the core samples ranges from about 63% to 94%, indicating a rock mass of fair to good quality, as per Table 3.10 of CFEM (2006)⁵. The measured Total Core Recovery (TCR) and the Solid Core Recovery (SCR) of the rock samples are about 100%. The UCS strength

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

measured on one core sample is 82 MPa; the results are presented in the report prepared by Geomechanica in Appendix B.

4.3 Groundwater Conditions

Details of the water levels observed in the boreholes upon completion of drilling are summarized on the borehole records. Standpipe piezometers were installed in three boreholes to monitor the groundwater level at the site, as shown on the borehole records and in the following table. The water level in the standpipe piezometers was measured at depths ranging from 6.2 m to 7.9 m (Elevation 93.2 m to 94.2 m). 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	June 7, 2016		August 13, 2018	
		Depth(m)	Elevation (m)	Depth (m)	Elevation (m)
S2	Silt and Sand / Sand and Gravel	6.8	93.6	6.7	93.7
SS-3	Sand and Gravel	NA	NA	6.2	94.2
SS-9	Gravelly Sand	NA	NA	7.9	93.2

4.4 Analytical Testing

Three soil samples were collected and submitted to Maxxam for analysis of parameters used to assess corrosion potential and sulphate attack. A summary of the results of the analyses is presented in the following table. The Certificate of Analysis is provided in Appendix C.

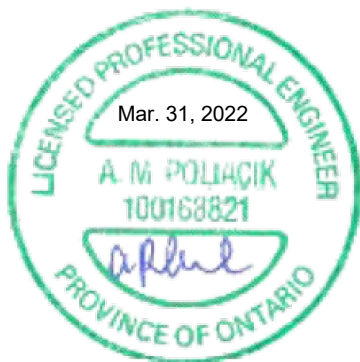
Borehole Number	Sample	Sample Depth / Elevation	Soil Type	Parameters				
				Chloride (µg/g)	Sulphate (µg/g)	pH	Conductivity (mS/cm)	Resistivity (ohm-cm)
SS-2	3	1.8 / 99.0	Silty Clay Fill	1,100	160	7.72	2,600	380
SS-2	10	9.2 / 91.6	Sandy Clayey Silt Till	28	150	7.94	297	3,400
SS-8	10	7.8 / 93.4	Sand / Sand and gravel	140	140	8.12	432	2,300

5.0 CLOSURE

This Foundation Investigation Report was prepared by Ms. Anastasia Poliacik, P.Eng., and was reviewed Mr. Christopher Ng, P.Eng., a senior geotechnical engineer with Golder. Ms. Lisa Coyne, P.Eng., Golder's MTO Foundation Designated Contact for this project, conducted an independent technical and quality control review of this report.

Signature Page

Golder Associates Ltd.



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Senior Geotechnical Engineer

A handwritten signature in blue ink, appearing to read "Chris Ng".

Christopher Ng, P.Eng.
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AP/CN/LCC/rb;ljb/rb

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PART B

FOUNDATION DESIGN REPORT

REPLACEMENT OF SIMCOE STREET UNDERPASS (SITE NO. 22X-176/B0)

REPLACEMENT OF THREE UNDERPASSES AND REHABILITATION OF
OSHAWA CREEK BRIDGE, HIGHWAY 401, OSHAWA, ONTARIO

MTO G.W.P. 2298-13-00

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides foundation design recommendations for construction of a new Simcoe Street Underpass (Site No. 22X-176/B0) to replace the existing structure. These recommendations are based on interpretation of the 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 foundation alternatives and carry out the design of the foundations.

The Foundation Design Report, discussion and recommendations are intended for the use of MTO and their designers 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.

Contractors undertaking the work must make their own interpretation based on the data presented in the Foundation Investigation Report (Part A of this report). Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project and for which special provisions may be required in the Contract Documents. Contractors must make their own interpretation of the information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling, and the like.

6.1 General

Based on the General Arrangement (GA) drawing prepared by WSP, dated February 2022, the proposed Simcoe Street underpass will consist of a three-span structure with a total span length of about 85.6 m and a width of about 29.8 m, carrying five lanes of traffic and a partial turning lane. The new structure will remain on the existing alignment but will be lengthened to allow for widening of Highway 401 from six lanes to ten lanes, including a grade raise of about 0.1 m on Highway 401. Grade raises of up to about 1.3 m and 1.7 m are proposed at the north and south approach embankments on Simcoe Street, respectively. The bridge replacement will be constructed in two stages so that northbound and southbound traffic on Simcoe Street can be maintained throughout construction.

As detailed on the GA drawing, and based on discussions between WSP and Golder as the borehole investigation results became available and the structural design progressed, the proposed bridge foundations include spread footings at the north abutment, 1.5 m diameter drilled shafts at the north and south piers, and driven piles at the south abutment. The existing and final road elevations, foundation types, and proposed foundation elevations, as shown from the GA drawing, are summarized in the following table; further discussion on the various foundation types is provided in Section 6.2, to support the selection of these preferred alternatives for the foundation types at this structure replacement.

Foundation Element	Existing Grade (m)	Final Grade (m)	Foundation Type on GA Drawing
North Abutment	100.5	103.1 ² / 95.2 ³	Spread Footing
North Pier	95.5	95.8	1.5 m dia. Drilled Shafts
South Pier	99.5	95.0	1.5 m dia. Drilled Shafts
South Abutment	100.2	102.2 ²	Spread Footing

Notes:

1. Proposed elevations correspond to the footing depth or underside of pile caps.
2. Elevation at Simcoe Street
3. Elevation at Highway 401

6.2 Foundation Options

Based on the proposed structure configuration and the subsurface conditions encountered at this site, the following shallow and deep foundation options have been considered for support of the new abutments and piers. A summary of the advantages and disadvantages associated with each option is provided below and a comparison of the alternative foundation options based on advantages, disadvantages, risks, and relative costs is provided in Table 1 following the text of this report.

- i **Spread/strip footings:** Shallow foundations comprised of spread or strip footings are considered feasible for support of the new abutments and piers, provided they extend to the very dense/hard native soils. This option would require excavations up to depths of 11 m and 4.5 m below existing ground surface at the abutments and piers, respectively; however, the footing level can be located at a higher elevation if a compact granular pad is adopted. Staged construction and constrained workspace will be a challenge if spread/strip footings are considered for the north pier. In addition, spread/strip footings will be founded below the groundwater level and therefore groundwater control (i.e., active dewatering) will be required for construction in dry conditions.
- i **Driven steel H-piles or pipe piles:** Steel H-piles or pipe piles driven to bedrock is considered feasible for support of the south abutment only. Due to the required 1.3 m pile cap depth (for frost protection) and the shallow depths to “100-blow” soil, the pile lengths at the north abutment and piers would be too short to develop sufficient axial and lateral geotechnical resistances prior to reaching effective refusal; in addition, there would be potential for these relatively short piles to become misaligned and/or for pile damage due to driving in dense/hard soils, although this could be mitigated by pre-drilling. While such a foundation option is technically feasible, it is not preferred as it is not seen to offer any significant advantage over the spread/strip footings or drilled shaft (caisson) foundation options for the north abutment and piers, respectively. As such, driven steel piles are only considered for the south abutment and further discussion of driven piles at the north abutment and piers is not included herein. At the south abutment, pile driving shoes are recommended to protect the pile tips from damage during driving into the very dense / hard native site soils and anticipated cobbles / boulders in the glacial till deposits.
- i **Drilled shafts:** Drilled shafts founded within the very dense / hard native soils or socketed into bedrock is also considered feasible for the support of the abutments and piers. Drilled shafts can offer a narrower footprint for construction in constrained working areas, as compared with shallow foundations and driven piles, and can be affixed directly to the underside of the superstructure at the piers, eliminating the need for foundation excavations to construct below-grade pile caps. If drilled shafts are adopted for support of the abutments and/or piers, temporary or permanent liners will be required and will need to be advanced with a water/bentonite drilling slurry inside the liners. Drilled shafts would be more expensive than spread/strip footings and driven pile foundations; however, the higher costs per drilled shaft element would be offset by schedule and cost savings associated with minimizing the working footprint for traffic staging, and potentially minimizing excavation and groundwater control if the below-grade pier pile caps can be eliminated.

Based on the above considerations, both shallow and deep foundation options are considered feasible for the support of abutments and piers. Typically, at the abutments, pile foundations would be practicable with a perched pile cap in a false abutment configuration to minimize excavation and groundwater control requirements, and to facilitate integral abutments. However, due to the closed structure type at the north abutment as well as shallow depths to “100-blow” soil following excavation for the widened Highway 401 cut, shallow foundations for the north abutment are preferred. Due to the presence of a stiff to very stiff clay to clayey silt deposit at the south abutment, the preferred foundation option is to sub-excavate and replace the cohesive deposit with a compacted granular pad

to allow for abutment design similar to that at the north abutment. At the piers, shallow foundations would normally be considered the preferred option from a foundations perspective due to the presence of a suitable bearing stratum at shallow depth; however, due to the extremely constrained highway corridor and staging requirements at this site, drilled shaft (caisson) foundations are considered more advantageous from a foundations perspective for the piers.

6.3 Design Considerations

6.3.1 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the 2019 Canadian Highway Bridge Design Code (CHBDC, 2019) and its Commentary, the proposed bridge and its foundation system are expected to carry medium to high traffic volumes and its performance will have potential impacts on other transportation corridors; hence, the structure is classified as having a “typical consequence level” associated with exceeding limits states design.

In addition, given the typical project specific foundation investigation carried out at this site (as presented in the Foundation Investigation Report (Part A of the report)), in comparison to the degree of site understanding in Section 6.5 of CHBDC (2019), the level of confidence for design is considered to be a “typical degree of site and prediction model understanding.” Accordingly, the appropriate corresponding 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 CHBDC have been used for design.

6.3.2 Seismic Design

6.3.2.1 Seismic Site Classification and Importance Category

The subsurface conditions for seismic site characterization were assessed based on the results of the field investigation and in situ testing. Based on the energy-corrected average penetration resistance, \bar{N}_{60} below the founding level, the site may be classified as Site Class C in accordance with Table 4.1 of *CHBDC (2019)*, in the absence of any geophysical testing.

CHBDC (2019) states that the seismic hazard values associated with the design earthquakes should be those established for the National Building Code of Canada (NBCC) 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 and were made available for public use in December 2015.

6.3.2.2 Spectral Response Values and Seismic Performance Category

In accordance with Section 4.4.3.1 of *CHBDC (2019)*, the peak ground acceleration (*PGA*), peak ground velocity (*PGV*) and 5% damped spectral response acceleration ($S_a(T)$) 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
$S_a(0.2)$ (g)	0.068	0.112	0.197
$S_a(0.5)$ (g)	0.044	0.067	0.110
$S_a(1.0)$ (g)	0.025	0.037	0.059
$S_a(2.0)$ (g)	0.012	0.018	0.029
$S_a(5.0)$ (g)	0.0026	0.0042	0.0071
$S_a(10.0)$ (g)	0.0011	0.0018	0.0030

Given that the seismic hazard values above reference ground conditions Site Class C, further modification to obtain site-specific seismic hazard values are not required and therefore, the design spectral response acceleration ($S(T)$) values are equal to the 5% damped spectral response acceleration ($S_a(T)$) values.

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

6.4 Spread / Strip Footings

6.4.1 Founding Elevations

For the support of the new abutments and piers spread/strip footings should be founded on very dense / hard native soils, or on compacted granular pads following sub-excavation of looser soil deposits. The founding elevations recommended for design of footings founded on the native soils are summarized below; these elevations also represent the target sub-excavation level prior to construction of engineered granular pads:

Foundation Element	Recommended Founding Elevation (m)	Founding Stratum
North Abutment	93.5	Very dense gravelly sand to sand and gravel / very dense sand
North Pier	91.5	Dense gravelly sand / very dense silt and sand till
South Pier	93.0	Very dense sand and gravel / very dense gravelly silt and sand till
South Abutment	92.5	Very dense sand and gravel / hard clayey silt with sand till

Alternatively, consideration could be given to founding the spread/strip footings at shallower elevations (i.e. at a depth equal to frost penetration) as summarized below.

Foundation Element	Alternative Founding Elevation (m)	Founding Stratum
North Abutment	93.9	Very dense gravelly sand to sand and gravel
North Pier	94.5	Very loose to compact silty sand fill
South Pier	93.7	Compact silt and sand / dense to very dense silty sand till
South Abutment	95.5	3 m Granular 'A' Pad over the dense to very dense sand and gravel / hard clayey silt to clayey silt

6.4.2 Geotechnical Resistances

The following factored ultimate geotechnical resistance and factored serviceability geotechnical resistance (for 25 mm of settlement) may be used for the design of 5.5 m wide (abutments) or 3 m wide (piers), 30.1 m long strip footings founded on the properly prepared soil or on compacted Granular 'A' at the elevations given below.

Foundation Element	Founding Elevation (m)	Factored Ultimate Geotechnical Resistance (kPa)	Factored Serviceability Geotechnical Resistance (kPa)
North Abutment (5.5 m wide)	93.5	525	N/A ¹
	93.9	525	N/A ¹
North Pier (3 m wide)	91.5	550	N/A ¹
	94.5	250	100
South Pier (3 m wide)	93.0	550	N/A ¹
	93.7	500	475
South Abutment (5.5 m wide)	92.5	550	N/A ¹
	95.5	475	300

Note:

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

6.4.3 Resistance to Lateral Loads

Resistance to lateral forces/sliding resistance between cast-in-place concrete footings and the founding soils should be calculated in accordance with Section 6.10.4 of CHBDC (2019). The following presents the unfactored coefficient of friction, $\tan \phi'$, for the interface between the concrete footing and native soils or Granular 'A' pad as interpreted from NAVFAC (1984):

Subgrade Material	$\tan \phi'$
Very dense gravelly sand to sand and gravel	0.70
Very dense sand	0.65
Very dense silt and sand till to gravelly silt and sand till	0.70
Hard clayey silt and sand till	0.70
Granular 'A' Pad	0.70

6.4.4 Frost Protection

All footings should be provided with a minimum 1.3 m of soil cover for frost protection as per OPSD 3090.101 (Frost Penetration Depths for Southern Ontario), as measured vertically and perpendicular from the face of the abutment slope to the edge of the underside of the footing.

6.4.5 Key Challenges and Considerations

- i Due to the limited working space at the north pier, temporary protection systems would be required along the perimeter of a footing excavation at this location. Temporary protection systems could consist of driven steel sheet piles (which would serve to cut-off groundwater inflow from the sides of the excavation, if not the base), or soldier piles and timber lagging (a system that is not watertight, and which would require greater groundwater control measures to minimize the potential for loss of fine-grained soil particles through the lagging boards). Variations on protection systems, such as a slide rail shoring system, may also be feasible at this site.
- i Based on the groundwater level measurements in August 2018, excavations for the spread/strip footings at the north abutment as well as at the north and south piers will extend to or below the groundwater level. Therefore, groundwater control measures (active dewatering) will be required at these locations to achieve and maintain a dry and stable foundation subgrade. Additional dewatering details are provided in Section 6.11.3.
- i Alternatively, footing excavations could be completed within a shored, water-filled excavation to balance the groundwater pressure at the excavation, with a concrete “plug” placed via tremie methods, to reduce or eliminate the requirement for dewatering. If this option is adopted, the minimum thickness of the concrete “plug” will be assessed in conjunction with the structural designers based on the final founding level. The concrete should have a minimum 28-day strength of 20 MPa.
- i If a concrete tremie “plug” is not adopted, even with active dewatering, the subgrade soils will be susceptible to loosening and disturbance due to water seepage, ponded water, and construction traffic. It is recommended that a 100 mm thick concrete working slab be placed on the subgrade within four hours to protect the integrity of the bearing stratum.
- i An appropriate strategy will be required to address the on-site storage and disposal of pumped water at the north pier, if dewatering is required at this location, considering the constrained workspace within the highway.

6.5 Driven Steel H-Piles or Pipe Piles

6.5.1 Founding Elevations

The south abutment for the replacement structure may be supported on steel H-piles or pipe piles driven to found within the shale bedrock. As discussed in Section 6.2, piles are not considered practicable for the north abutment and piers.

The south abutment pile cap should be perched as high as structurally possible within the south approach embankment, relative to the Simcoe Street grade of approximately 102 m, as long as adequate soil cover is provided to the pile cap for frost protection. The following pile tip elevation for the south abutment is provided for design purposes.

Foundation Element	Assumed Underside of Pile Cap Elevation (m)	Estimated Pile Tip Elevation/Depth to Bedrock (m)	Approx. Pile Length (m)	Founding Stratum
South Abutment	96 – 98	83.0	13 – 15	Shale Bedrock

6.5.2 Factored Geotechnical Axial Resistances

The factored ultimate and serviceability (for 25 mm of settlement) geotechnical axial resistance for driven steel HP310x 110 piles are presented below. The same factored resistances may be used for closed-end, concrete filled 324 mm (12 ¾ in) diameter steel pipe piles having a minimum wall thickness of 9.5 mm (3/8 in.).

Foundation Element	Factored Ultimate Geotechnical Resistance (kN)	Factored Serviceability Geotechnical Resistance ¹ (kN)
South Abutment	1800	N/A

Note:

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

The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile and length of pile; the criteria must therefore be established at the time of construction after the piling equipment is known. The pile capacity should then be verified in the field by using the Hiley formula (MTO's Standard Drawing SS103-11, Pile Driving Control) together with high strain dynamic testing (i.e., Pile Driving Analyzer (PDA) testing) during the final stages of driving to verify that the required ultimate capacity has been achieved. All pile installation/driving should be carried out in accordance with OPSS.PROV 903 (Deep Foundations). If this option were adopted, it is recommended that an NSSP be included in the Contract Documents to specify the proportion of piles for PDA testing.

6.5.3 Frost Protection

All pile caps should be provided with a minimum 1.3 m of soil cover for frost protection as per OPSD 3090.101 (Frost Penetration Depths for Southern Ontario), as measured vertically from and perpendicular to the face of the abutment slope to the edge of the underside of the pile cap. If adequate soil cover cannot be provided for the pile cap, rigid styrofoam insulation could be installed to compensate for the lack of soil cover and provide protection from frost penetration.

6.5.4 Key Challenges and Considerations

- i The till deposits are anticipated to contain cobbles and boulders. These potential obstructions may affect the installation of piles and appropriate measures will need to be implemented. Where steel H-piles are adopted, the pile tips should be reinforced with pile points (refer to Section 6.11.5 for more details).
- i If the south abutment pile cap is maintained above approximately Elevation 96 m, dewatering is not expected to be required. If a deeper pile cap is adopted, dewatering may be required to achieve and maintain a dry and stable subgrade.

6.6 Drilled Shafts

6.6.1 Founding Elevations

The new abutments and piers for the proposed underpass replacement may also be supported on drilled shafts founded on very dense / hard native soils or shale bedrock. The following drilled shaft founding elevations may be used for design purposes.

Foundation Element	Elevation of Assumed Underside of Pile Cap (m)	Elevation of Bottom of Drilled Shaft (m)	Founding Stratum
North Abutment	94.0	88.0	Very dense silt / very dense silt and sand till / very dense clayey silt till
North Pier	94.6	82.8	0.5 m socket into shale bedrock
South Pier	93.8	82.8	0.5 m socket into shale bedrock
South Abutment	92.8	91.0	Very dense sand and gravel / very dense sandy clayey silt till

It should be noted that boreholes at the north abutment and south pier did not extend to the proposed founding elevation and therefore, the founding deposit is inferred. However, based on our understanding of the geology within the project limits through nearby boreholes (at the north pier and south abutment), and the foundation investigation in the general vicinity (including that from the adjacent north retaining wall and nearby Albert Street Underpass), the risk associated with the bottom of drilled shaft encountering loose/soft soils is considered to be low.

If drilled shaft foundations are adopted, temporary casings and water or drilling slurry should be used to support the overburden soils during construction to minimize disturbance to the side walls and to control base disturbance/basal heave due to groundwater pressure/seepage. In addition, placement of concrete by tremie methods would be required.

6.6.2 Geotechnical Axial Resistances

The following factored ultimate and serviceability geotechnical axial resistances may be used for design of 1.5 m diameter drilled shaft foundations founded within the till deposit at the abutments:

Foundation Element	Factored Ultimate Geotechnical Resistance (kN)	Factored Serviceability Geotechnical Resistance ¹ (kN)
North Abutment	3,100	N/A
South Abutment	2,000	N/A

Note:

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

The following factored ultimate and serviceability geotechnical axial resistances may be used for sizing and design of drilled shaft foundations founded within the shale bedrock at the north and south piers:

Drilled Shaft Diameter at Piers	Factored Ultimate Geotechnical Resistance (kN)	Factored Serviceability Geotechnical Resistance ¹ (kN)
0.9 m	14,000	N/A
1.2 m	25,000	N/A
1.5 m	38,000	N/A

Note:

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

It will be necessary to verify the cleanliness of the base of the drilled shafts at the piers to achieve the above capacities. It may be necessary to use a Shaft Quantitative Inspection Device (SQUID) to verify the base cleanliness given the wet overburden conditions, use of drilling fluids, and potential difficulties in seating the temporary casing into the strong bedrock.

6.6.3 Frost Protection

All drilled shaft pile caps should be provided with a minimum 1.3 m of soil cover for frost protection as per OPSD 3090.101 (Frost Penetration Depths for Southern Ontario), as measured vertically and perpendicular from the face of the abutment slope to the edge of the underside of the pile cap. If adequate soil cover cannot be provided for the pile cap, rigid Styrofoam insulation could be installed to compensate for the lack of soil cover and provide protection from frost penetration.

6.6.4 Key Challenges and Considerations

- i If drilled shafts are adopted, temporary casing should be utilized to support the overburden soils during construction to minimize disturbance to the side walls. The liner should be advanced while filled with a head of water or drilling slurry to minimize the potential for non-cohesive materials ("flowing sands") to migrate into the advancing drilled shaft and to control base disturbance/basal heave due to groundwater pressures/seepage. In addition, placement of concrete by tremie methods would be required. The Contractor should be alerted to the presence of "flowing sand" conditions; an example NSSP is included in Appendix E.
- i Given that the above drilled shaft capacities have a significant end-bearing component, the performance of the drilled shafts in compression will depend to a large degree upon the final cleaning and verification of the condition of the base of the drilled shaft. As such, the base of each drilled shaft excavation must be cleaned to remove all loose cuttings to ensure that the concrete is in intimate contact with the very dense / hard till deposit or hard clayey silt deposit. A qualified geotechnical engineer should be retained during construction to inspect the drilled shafts to verify that the conditions encountered are consistent with the information obtained from the boreholes and to confirm the base elevation of the drilled shaft and cleanliness. To allow for remote inspection of the base of the drilled shafts (via a shaft inspection device), the drilled shaft excavations must be cased. The casing must be maintained tight to the sides of the soil and extended nominally into the bedrock.
- i Based on the groundwater level measurements in August 2018, excavations for the pile caps at the north abutment as well as at the north and south piers will extend to near the groundwater level and therefore groundwater control measures (active dewatering) are expected to be required at these locations. Additional dewatering details are provided in Section 6.11.3. The requirement for dewatering could be eliminated with

the use of drilled shaft foundations at the north and south piers if the pile caps are placed at the underside of the bridge deck, rather than below grade.

An appropriate strategy will be required to address the on-site storage and disposal of pumped water at the north pier, if below-grade pile caps are adopted at this location, considering the constrained working space within the highway.

6.7 Resistance to Lateral Loads for Driven Pile and Drilled Shaft Foundations

The design of piles and drilled shafts subjected to lateral loads should take into account such factors as the batter of the pile (if any), the relative rigidity of the pile / drilled shaft to the surrounding soil, the fixity condition at the head of the pile / drilled shaft (i.e., at the pile cap level), the structural capacity of the pile / drilled shaft to withstand bending moments, the soil resistance that can be mobilized, the tolerable lateral deflections at the head of the pile / drilled shaft and group effects. For a longer, more flexible pile, the maximum yield moment of the pile may be reached prior to mobilization of the lateral geotechnical resistance. For design purposes, both the structural and geotechnical resistances should be evaluated to establish the governing case.

Lateral loading could be resisted fully or partially by the use of battered piles, where possible.

The resistance to lateral loading in front of a single pile / drilled shaft may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h (kN/m²/m), is based on the following equations:

For non-cohesive soils:

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

where:

n_h	=	coefficient related to soil density (kN/m ² /m)
z	=	depth (m)
B	=	pile / drilled shaft diameter or width (m)

For cohesive soils:

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

where:

s_u	=	undrained shear strength of the soil (kN/m ²)
B	=	pile / drilled shaft diameter or width (m)

The values of n_h (Terzaghi, 1955 and Reese, 1975) and s_u to be incorporated into the calculations of the coefficient of horizontal subgrade reaction (k_h) within the native overburden, to be used for the structural analysis of the piles or drilled shafts at this site are summarized below.

Foundation Element	Soil Unit	Elevations (m)	n_h (kN/m ² /m)	S_u (kPa)
North Abutment	Dense to Very Dense Sand to Sand and Gravel	96.0 – 92.0	16,000	Not Applicable
	Hard Silty Clay Till	92.0 – 83.0	Not Applicable	200
South Abutment	Dense to Very Dense Gravelly Sand to Sand and Gravel	92.5 – 88.0	16,000	Not Applicable
North Pier	Loose to compact fill	96.0 – 92.1	3,000	Not Applicable
	Dense gravelly sand / Very dense silt and sand till / Very dense silty sand	92.1 – 86.0	11,000	Not Applicable
	Hard sandy clayey silt-silt to clayey silt till	86.0 – 83.3	Not Applicable	200
South Pier	Stiff clayey silt	96.0 – 95.0	Not Applicable	55
	Dense to very dense silty sand till / dense to very dense sand and gravel	95.0 – 92.5	11,000	Not Applicable
	Hard clayey silt with sand till	89.0 – 83.3	Not Applicable	200

For a single H-pile and a single drilled shaft, the factored ultimate limit state (ULS) and factored serviceability limit state (SLS) for 10 mm of horizontal deflection at the pile caps are presented below. These values are based on analyses carried out using the commercially available program LPILE Plus (Version 2016), developed by Ensoft Inc.

Foundation Element	Deep Foundation Unit	Factored Ultimate Geotechnical Resistance (kN)	Factored Serviceability Geotechnical Resistance for 10 mm of Deflection (kN)
South Abutment ^{1,2}	HP 310x110	200	70
Abutments and Piers ²	1.5 m dia. drilled shaft	300	200

Notes:

1. The analysis assumes that the steel H-piles at the abutments are oriented for strong axis bending.
2. The analysis assumes a free head condition.

Group action for lateral loading should be considered in accordance with Section C6.11.3.4 of the *Commentary to CHBDC (2019)*.

6.8 Lateral Earth Pressures for Design of Abutments and Wingwalls

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

The following recommendations are made concerning the design of the abutments.

- Free-draining granular fill meeting the specifications of OPSS.PROV 1010 (Aggregates) Granular 'A' or Granular B Type II should be used as backfill behind the walls. Longitudinal drains or weep holes should be

installed to provide positive drainage of the granular backfill. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS.PROV 501 (Compacting). Other aspects of the granular backfill requirements with respect to subdrains and frost taper should be in accordance with OPSD 3101.150 (Walls, Abutment, Backfill, Minimum Granular Requirement), OPSD 3121.150 (Walls, Retaining, Backfill, Minimum Granular Requirement), and 3190.100 (Walls, Retaining and Abutment, Wall Drain).

A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with CHBDC (2019) Section 6.12.3 and Figure 6.8. 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 restrained walls, 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 C6.31(a) of the *Commentary to CHBDC* (2019). For unrestrained walls, fill should be placed within the wedge-shaped zone defined by a line drawn at flatter than 1 horizontal to 1 vertical (1H:<1V) extending up and back from the rear face of the footing or pile cap in accordance with Figure C6.31(b) of the *Commentary to CHBDC* (2019).

The following guidelines and recommendations are provided regarding the lateral earth pressures for static 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 are based on the proposed new embankment fill behind the granular backfill zone, and the following parameters (unfactored) may be used assuming the use of earth fill or Select Subgrade Material (SSM) for the general embankment fill:

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

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

Fill Type	Unit Weight of Material (kN/m ³)	Coefficients of Static Lateral Earth Pressure	
		At-Rest, K _o	Active, K _a
Granular 'A'	22	0.43	0.27
Granular 'B' Type II	21	0.43	0.27

If the wall support and superstructure allow lateral yielding, active earth pressures may be used in the geotechnical design of the structure. 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 C6.12 of the *Commentary to CHBDC* (2019).

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.

6.9 Approach Embankment Design

6.9.1 Parameter Selection

The foundation engineering parameters for the soil types encountered in the boreholes at the approach embankments are summarized in the following table. For the stability and settlement analyses, the groundwater level was assumed to be at Elevation 94.2 m.

Stratigraphic Unit	γ (kN/m ³)	ϕ' (°)	s_u (kPa)	σ_p' (kPa)	e_o	C_c	C_c	E' (MPa)
New fill (Granular 'B' Type II)	21	35	--	--	--	--	--	20
Loose to dense sand to gravelly sand fill	20	32	--	--	--	--	--	9
Firm to very stiff clay to silty clay fill	19	30	50	225	0.6	0.4	0.04	--
Compact to very dense silt to sand	21	32	--	--	--	--	--	30 – 150
Soft to stiff clayey silt to silty clay with sand	19	29	75	325	0.6	0.4	0.04	--
Dense to very dense gravelly sand to sand and gravel	21	35	--	--	--	--	--	100
Compact to very dense silt and sand till	21	35	--	--	--	--	--	30 – 150
Hard clayey silt to clayey silt with sand till	21	35	200	--	--	--	--	150

6.9.2 Global Stability

Global stability analyses were carried out for the proposed approach embankments and the abutment walls/fore slopes. Limit equilibrium slope stability analyses were performed using the commercially available program Slide (Version 2018), developed by Rocscience Inc., employing the Morgenstern-Price method of analysis. For all analyses, the Factors of Safety of numerous potential failure surfaces were computed to establish the minimum Factor of Safety. The Factor of Safety is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. The Factor of Safety is equal to the inverse of the product of the consequence factor, Ψ , and the geotechnical resistance factor, ϕ_{gu} . (i.e., $FoS = 1/(\Psi \cdot \phi_{gu})$). Accordingly, minimum Factors of Safety of 1.5 have been used for the design of the embankment slopes for the long-term / permanent conditions, respectively, as per Table 6.2 of CHBDC (2019).

The stability analyses for the approach embankments and abutment walls/fore slopes indicate that in long-term conditions, the embankments and abutments will have a Factor of Safety greater than the required 1.5. It is noted that the stability analyses assume the embankments are constructed to the side slopes indicated on the GA Drawing (summarized in the table below) and that the embankment material consists of Granular 'B' Type II, placed and compacted in accordance with Section 6.11.2. The results of the stability analyses are summarized in the following table and are shown on Figures D1 to D6 in Appendix D.

Foundation Element	Relevant Boreholes	Location	Slope	Factor of Safety
North Approach Embankment	SS-8 and SS-9	West Slope	2H :1V	1.6
		East Slope	2H :1V	1.6
South Approach Embankment	SS-2 and SS-3	West Slope	4H:1V	2.7
		East Slope	4H:1V	2.7
North Abutment	SS-8 and SS-9	Abutment/ Retaining Wall	N/A	2.4
South Abutment	SS-2 and SS-3	Front Slope	2H :1V	2.3

6.9.3 Settlement

To estimate the magnitude of expected settlement at the embankments due to the grade increase along Simcoe Street, settlement analyses were carried out at the north and south approach embankments. The analyses were carried out using the commercially available program Settle3D (Version 4.0), developed by Rocscience Inc. The analyses assumed a 1.3 m grade raise at the north approach and a 1.7 m grade raise at the south approach.

The settlement performance criterion for design of approach embankments is in accordance with MTO's "Embankment Settlement Criteria for Design", dated July 2, 2010. In general, embankments approaching structural elements such as bridge abutments are to be designed such that total settlements and differential settlement rates do not exceed 25 mm, over a 15-year period following completion of construction for a secondary highway.

The sources of settlement at this site are considered to include immediate settlement of the granular soils (short-term) and primary time-dependent consolidation of the cohesive deposits (using Terzaghi's one-dimensional consolidation theory long-term). Given that the cohesive deposits are over-consolidated, secondary time dependent (creep) consolidation is not a concern at this site.

The thickness of the compressible foundation soils and the height of the approach embankments vary along the proposed alignment, and as such the settlements along the length of alignment will similarly vary; however, the settlements estimated from the settlement analyses represent the maximum anticipated value at the embankments.

Based on the results of the settlement analyses, the total settlement of the existing site soils under the loading imposed by a 1.3 m and 1.7 m high embankment is estimated to be 25 mm and 40 mm at the north and south abutment, respectively. The total settlement (immediate and consolidation) is expected to occur during construction and therefore settlement mitigation measures are not required. It is noted that the analyses assume the existing fill materials remain in place and new fill consists of Granular 'B' Type II, placed and compacted in accordance with Section 6.11.4.

6.10 Liquefaction Potential Below Embankments

Liquefaction is a phenomenon whereby seismically-induced shaking generates shear stresses within the soil under undrained conditions. These stresses tend to densify the soil (i.e. leading to potentially large surface deformations) and under undrained conditions generate excess pore water pressures. The excess pore water pressures also lead to sudden temporary losses in strength. Where existing static shear stresses are present, the loss of strength can lead to significant lateral movements (i.e. analogous to a slope failure) often referred to as "lateral spreading" or under certain conditions even catastrophic failure of the slope often referred to as "flow slides". Lateral spreading and flow slides often accompany liquefaction along rivers and other shorelines.

The liquefaction susceptibility of granular soils was evaluated by comparing the penetration resistance required to trigger liquefaction with the available penetration resistance. Liquefaction is predicted to occur when the available penetration resistance is less than the resistance required.

The methodology used to assess liquefaction potential at the site is consistent with that presented in the Commentary to CHBDC (2019). It involves comparing the cyclic shear stresses applied to the soil by the design earthquake, represented as the cyclic stress ratio (*CSR*), to the cyclic shear strength, represented as the cyclic resistance ratio (*CRR*) provided by the soil.

The liquefaction analysis was carried out using in-situ testing data collected at the borehole locations. The design groundwater level was determined based on the highest measured groundwater level in the standpipe piezometer installed in Borehole SS-3 at about Elevation 94.2 m. The *CRR* with depth was calculated at each borehole location using the parameter, $(N1)_{60cs}$, that is based on the SPT 'N'-value obtained in the field and corrected for overburden stress, rod length during sampling, hammer energy efficiencies, and fines content.

The results of the liquefaction assessment indicate that the silts and sands at the site are not considered liquefiable during the 2,475-year design earthquake.

6.11 Construction Considerations

6.11.1 Open-Cut Excavations

The topsoil and any organic / deleterious materials encountered within the footprint of the proposed foundation elements and approach embankments should be sub-excavated and replaced with granular fill. All 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 excavations should be made with side slopes no steeper than 1H:1V based on the soil profile. However, if water inflow is observed, flatter slopes and dewatering measures may need to be implemented.

Temporary excavations should be observed and reviewed during construction to confirm that the soil and groundwater conditions are as anticipated. If unexpected conditions are encountered, a geotechnical engineer should review the excavation plan considering the conditions at that time.

6.11.2 Temporary Protection Systems

Temporary protection systems will likely be required to facilitate the staged construction at the abutments and, depending on the selected foundation option and location of the pile cap, temporary protection systems may also be required to facilitate construction of the piers. Where 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 2 provided that any existing adjacent utilities can tolerate this magnitude of deformation. The selection and design of the protection system will be the responsibility of the contractor.

6.11.3 Control of Groundwater and Surface Water

The groundwater level measurements ranged from Elevation 93.2 m to 94.2 m in August 2018. However, it is noted that the groundwater level could 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 foundation elements can be constructed in dry conditions. It is anticipated that dewatering measures will be required for

construction of spread/strip footings at all foundation elements. Dewatering operations should be carried out / managed in accordance with OPSS.PROV 902 (*Excavation and Backfilling – Structures*), as amended by FOUN0003 (*Dewatering Structure Excavation*), a copy of which is included in Appendix E.

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 and stormwater for construction dewatering purposes with a combined total less than 400,000 L/day qualify for self-registration on the MECP 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 foundation option, an EASR or PTTW may be required, and a hydrogeological assessment should be conducted to estimate the expected water extraction requirements, assist in registration, and to provide the required documentation.

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

6.11.4 Embankment Construction and Erosion Protection

Placement of Select Subgrade Material (SSM) or granular fill (satisfying OPSS.PROV 1010 SSM or Granular ‘B’ Type I or Type II requirements) above the water table for construction of the new embankments (including backfilling operations) should be carried out in accordance with the requirements outlined in OPSS.PROV 206 (Grading). The SSM or granular fill should be placed and compacted in accordance with OPSS.PROV 501 (Compacting).

Side slopes for the SSM or granular fill embankment should be no steeper than 2H:1V. In addition, benching of the existing Simcoe Street side slopes should be carried out in accordance with OPSD 208.010 (Benching of Earth Slopes) prior to the placement of new embankment fill. Inspection and field testing should be carried out by qualified personnel during construction to confirm that appropriate materials are used and that adequate levels of compaction are being achieved.

To reduce erosion of the embankment side slopes due to surface water runoff, placement of topsoil and seeding or pegged sod should be carried out as soon as practicable after construction of the embankments. In the short-term, if placement of cover material cannot be carried out soon after the construction of the embankments, erosion control blankets should be installed to minimize erosion of the embankment slopes. The erosion protection should be in accordance with OPSS.PROV 804 (Temporary Erosion Control).

6.11.5 Obstructions

The site soils are generally glacially derived and as such should be expected to contain cobbles and boulders, which could affect the installation of piles, drilled shafts and/or protection systems.

If drilled shaft foundations are selected, the construction equipment should be capable of advancing the liner through such obstructions.

It is recommended that a Non-Standard Special Provision (NSSP) or Notice to Contractor be included in the contract documents to address obstructions (refer to Appendix E for an NSSP, which can be revised to a Notice to Contractor if desired).

6.11.6 Vibration Monitoring

Vibration monitoring and pre- and post- construction condition surveys are recommended during pile driving and installation of temporary protection systems to confirm that construction techniques and associated vibration levels experienced at nearby structures and utilities are maintained below acceptable levels, and to mitigate potential claims from property owners. NSSPs for vibration monitoring, and pre- and post-construction condition surveys are provided in Appendix E.

6.11.7 Analytical Testing of Construction Materials

The results of analytical tests carried out on three soil samples and are presented in Section 4.4 and on the Certificate of Analysis in Appendix C. The analytical test results 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 0.2%, which indicates a moderate degree of exposure (i.e., class S3 exposure limits). Therefore, based on the three 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 the spread footing or pile cap and any portion of the proposed structure constructed below the ground surface may need to be considered. However, 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.

The analytical test results of the soil samples were also compared to Table 7.1 (Relative Effect of Resistivity on Corrosion Potential/Aggressiveness (from NCHRP 1978)), as presented in the Federal Highway Administration / National Highway Institute Publication No. FHWANHI14007 (Federal Highway Administration, 2015), to assess the relative level of corrosion potential on buried steel in contact with soil. The resistivity values measured on the fill soil sample from Borehole SS-2 indicate a “very corrosive” potential and the deeper native soil samples from Boreholes SS-2 and SS-8 indicate “moderately corrosive” potential.

The analytical test results of the soil samples were also compared to Table 7.2 (Criteria of the U.S. Criteria for Assessing Ground Corrosion Potential), as outlined in FHWANHI14007 for the potential corrosion of buried steel, based on the measured values of pH, resistivity, chlorides, and sulfates in the soil samples tested. Based on the measured values, the soil samples are classified as “aggressive”.

It is also noted that the measured pH levels range between about 7.7 and 8.1, suggesting the presence of alkaline soils.

Ultimately, it is the 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.

6.11.8 Monitoring Well Decommissioning

Three standpipe piezometers were installed on the site to permit monitoring of the groundwater levels at the site. Ontario Regulation (O.Reg.) 903 amended by O.Reg. 128 of the Ontario Water Resources Act requires that monitoring wells are properly abandoned/decommissioned by qualified personnel. It is recommended 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 immediately prior to and during construction.

7.0 CLOSURE

This Foundation Design Report was prepared by Ms. Anastasia Poliacik, P.Eng., and was reviewed by Mr. Christopher Ng, P.Eng., a senior geotechnical engineer with Golder. Ms. Lisa Coyne, P.Eng., Golder's MTO Foundations Designated Contact for this project, conducted an independent technical and quality control review of this report.

Signature Page

Golder Associates Ltd.



Anastasia Poliacik, P.Eng.
Senior Geotechnical Engineer

A handwritten signature in blue ink, appearing to read "Christopher Ng".

Christopher Ng, P.Eng.
Senior Geotechnical Engineer



Lisa Coyne, P.Eng.
MTO Foundations Designated Contact

AP/CN/LCC/rb;ljb/rb

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Commercial Software:

- Slide (Version 2018) by Rocscience Inc.
- Settle3D (Version 4.0) by Rocscience Inc
- LPILE Plus (Version 2016) by Ensoft Inc

Ministry of Transportation Ontario:

- Drawing SS103-11 Pile Driving Control

Ontario Occupational Health and Safety Act:

- O. Reg. 213 Construction Projects (as amended)

Ontario Provincial Standard Specifications (OPSS)

- | | |
|---------------|---|
| OPSS.PROV 206 | Construction Specification for Grading |
| OPSS.PROV 501 | Construction Specification for Compacting |
| OPSS.PROV 539 | Construction Specification for Temporary Protection Systems |
| OPSS.PROV 804 | Construction Specification for Temporary Erosion Control |
| OPSS.PROV 902 | Construction Specification for Excavating and Backfill – Structures |

OPSS.PROV 903 Construction Specification for Deep Foundations

OPSS.PROV 1010 Construction Specification for Material Specification for Aggregates – Base, Subbase, Select Subgrade, and Backfill Material

Ontario Provincial Standard Drawings (OPSD)

OPSD 208.010 Benching of Earth Slopes

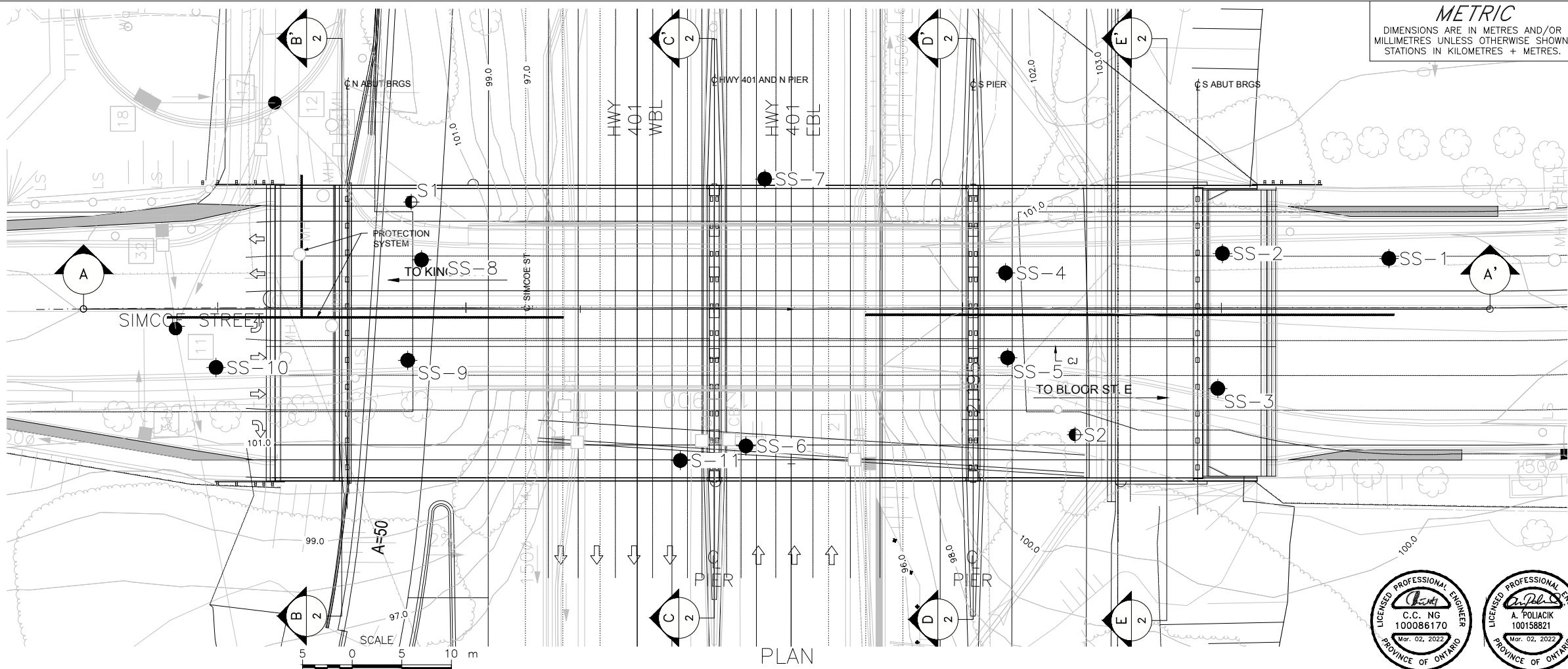
OPSD 3090.101 Foundation, Frost Penetration Depths for Southern Ontario

OPSD 3000.100 Foundation, Piles, Steel H-Pile Driving Shoe

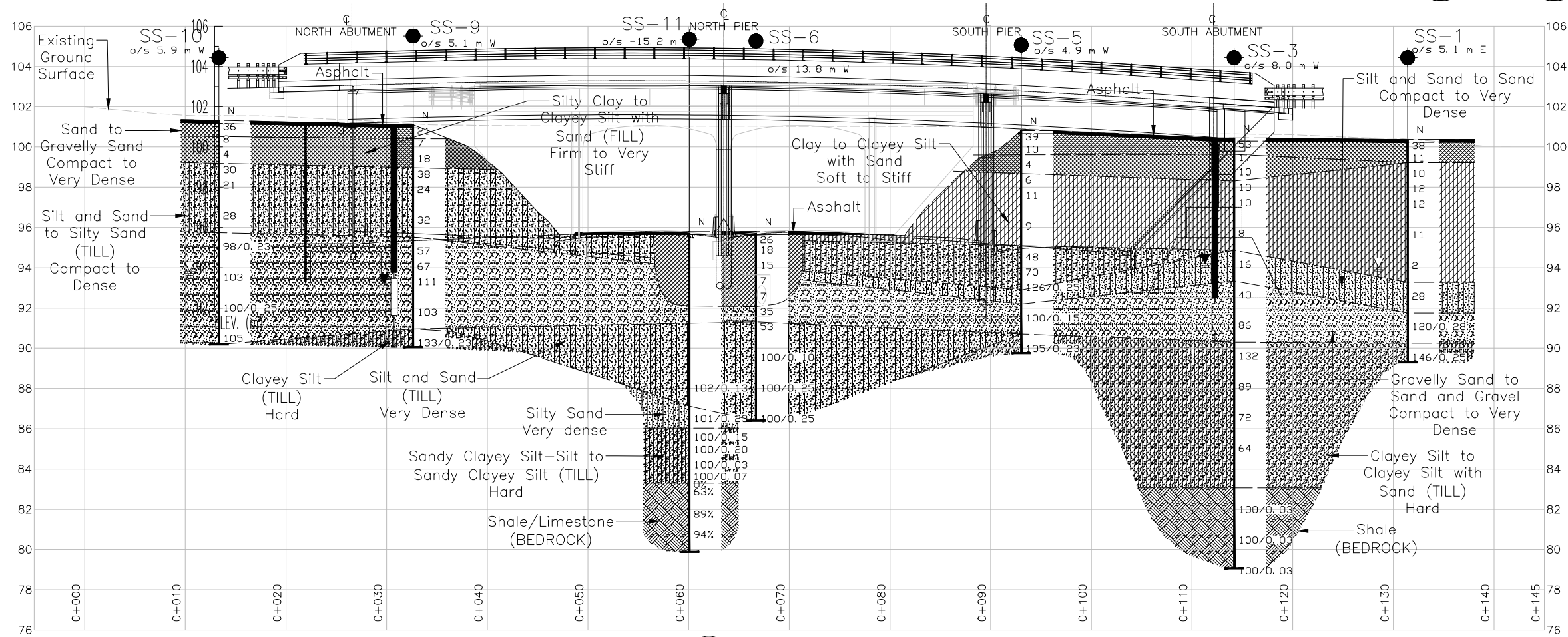
https://golderassociates.sharepoint.com/sites/16625822/12-foundations/6-reporting/3-simcoe-street/4-reports/5-final-fidr_rev2/1662582-fidr-2022'03'25-simcoe-street-underpass_rev2.docx

Table 1: Comparison of Foundation Alternatives – Simcoe Street Underpass

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risk / Consequences
Spread / strip footings founded on existing very dense / hard native soils, or on a compacted Granular 'A' pad	<ul style="list-style-type: none"> Feasible for all foundation elements, but would require significant excavation at south abutment 	<ul style="list-style-type: none"> Conventional construction techniques. 	<ul style="list-style-type: none"> Temporary protection system is required at the abutments and pier locations. Significant excavations would be required at south abutment to found below the stiff to very stiff clay to clayey silt Dewatering measures are required at abutments and pier locations. 	<ul style="list-style-type: none"> Lower relative cost than deep foundations for foundation elements only, but this would be offset by additional costs associated with deeper excavation and protection systems. 	<ul style="list-style-type: none"> Challenging to construct at north pier due to limited workspace Dewatering challenges at north pier due to location. Longer staging and more significant protection systems adjacent to south abutment
Driven Steel H-piles (HP 310x110) or Pipe Piles	<ul style="list-style-type: none"> Feasible at south abutment Not practical at north abutment and piers due to shallow depth to "100-blow" soil 	<ul style="list-style-type: none"> Conventional construction methods for H-pile foundations. Pile cap may be "perched" within the south approach embankment to reduce the amount of excavation. Dewatering not required at south abutment 	<ul style="list-style-type: none"> Short piles due to shallow "100-blow" soils at north abutment and piers. Temporary protection system is required at the abutments and pier locations. Dewatering measures are required at abutments and pier locations for the construction of pile caps. Requires driving shoes due to potential presence of cobbles / boulders within the till deposits. 	<ul style="list-style-type: none"> Higher relative cost than spread / strip footings. Lower relative cost than drilled shafts. 	<ul style="list-style-type: none"> Risk of damage to the piles due to cobbles / boulders within the till deposits. Challenging to construct at north pier due to limited workspace Dewatering challenges at north pier due to location
Drilled Shafts founded within the very dense / hard native soils or socketed into bedrock.	<ul style="list-style-type: none"> Feasible for all foundation elements 	<ul style="list-style-type: none"> Conventional construction methods for drilled shaft foundations. Offers higher geotechnical resistance compared to driven steel piles, requiring fewer foundation elements. Pile cap may be "perched" within the south approach embankment to reduce the amount of excavation and potentially eliminate requirement for protection systems. If underside of pile caps is extended to higher elevations, dewatering potentially not required. Requires a smaller footprint for construction in constrained working areas, as compared with spread/strip footings or driven piles. 	<ul style="list-style-type: none"> Temporary casing will be required, plus special measures such as use of a head of water or slurry to counterbalance groundwater pressures and minimize ground disturbance. Requires inspection of shaft base to verify design capacity, potentially including SQUID (Shaft Quantitative Inspection Device) given wet overburden conditions and potential difficulty in seating temporary casings into the surface of the strong shale bedrock. 	<ul style="list-style-type: none"> Higher relative cost than spread / strip footings and driven piles. 	<ul style="list-style-type: none"> May be difficult to inspect the base of the drilled shaft due to potential need for slurry during drill shaft advancement. Risk associated with the base of the drilled shaft.



PLAN



SIMCOE STREET UNDERPASS PROFILE

CONT No.
GWP No. 2298-13-00

SIMCOE STREET UNDERPASS
HIGHWAY 401 WIDENING
BOREHOLE LOCATION PLAN
AND SOIL STRATA

GOLDER



KEY PLAN

LEGEND

- Borehole - Current Investigation
- ⊕ Borehole - Previous Investigation

BOREHOLE CO-ORDINATES (MTM NAD83 ZONE 10)

No.	ELEVATION	NORTHING	EASTING
SS-1	100.4	4860303.5	356553.7
SS-2	100.8	4860319.6	356549.0
SS-3	100.4	4860315.8	356535.9
SS-4	101.0	4860339.7	356540.4
SS-5	100.8	4860336.9	356532.3
SS-6	95.8	4860359.2	356515.7
SS-7	95.8	4860365.7	356541.9
SS-8	101.2	4860396.0	356523.4
SS-9	101.1	4860394.2	356513.4
SS-10	101.3	4860412.3	356506.7
SS-11	95.8	4860365.0	356512.3
S1	101.3	4860398.8	356528.6
S2	100.4	4860328.0	356527.1

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.

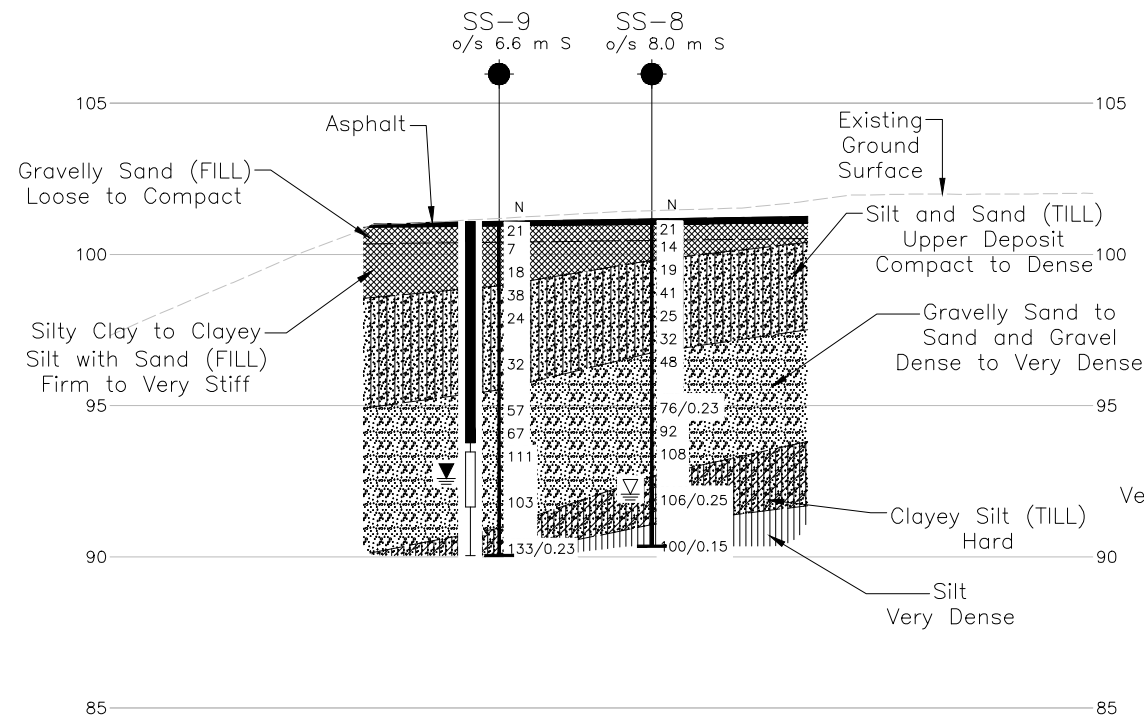
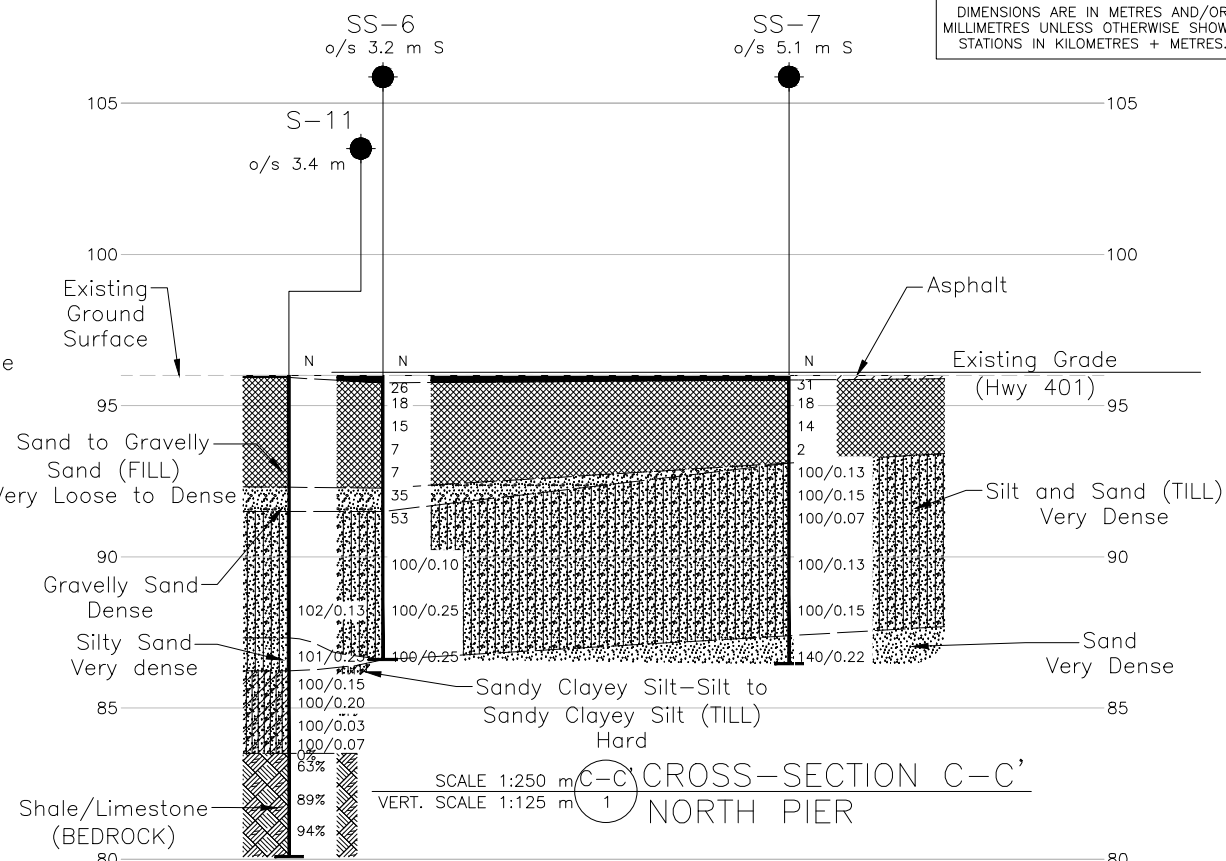
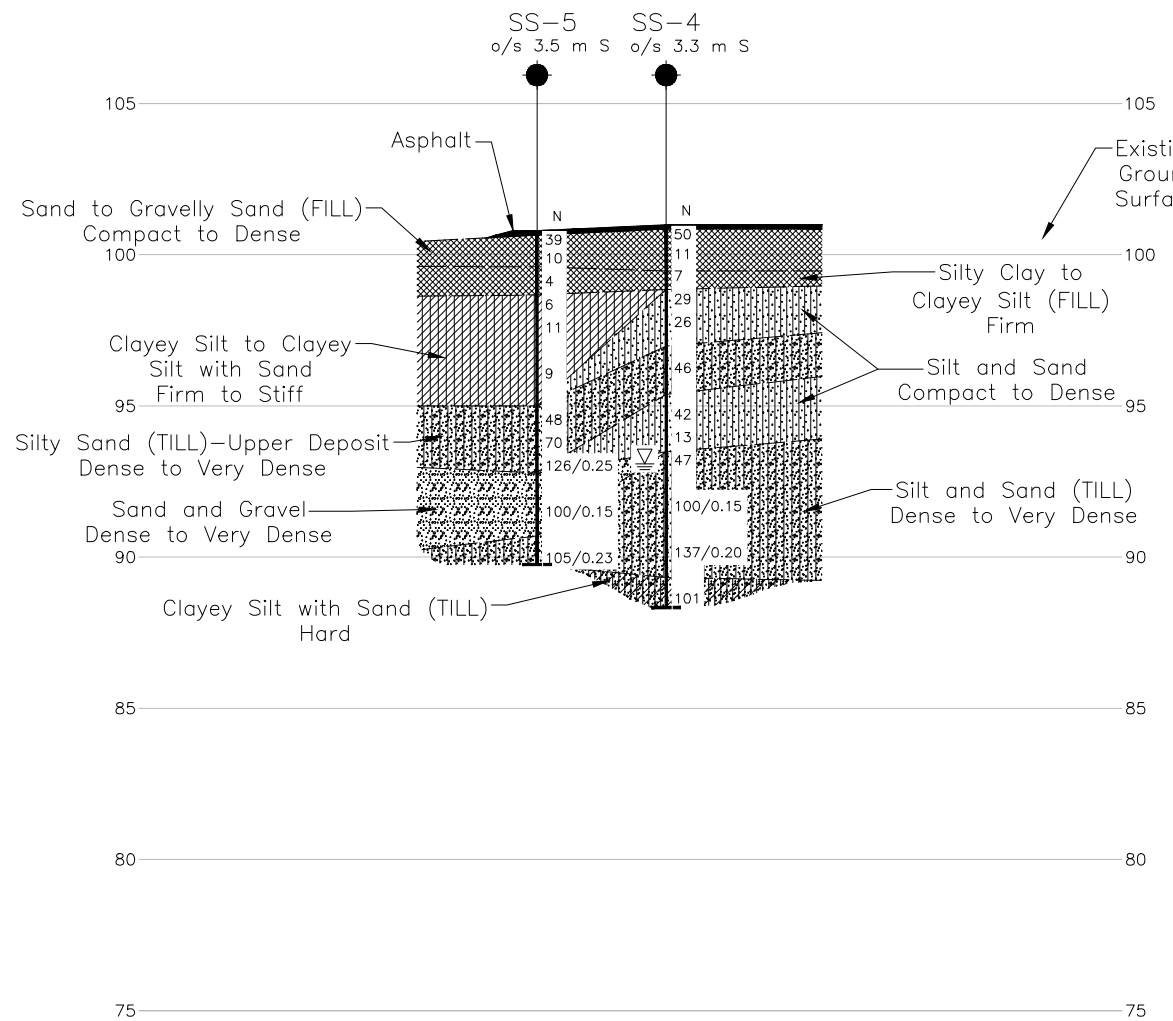
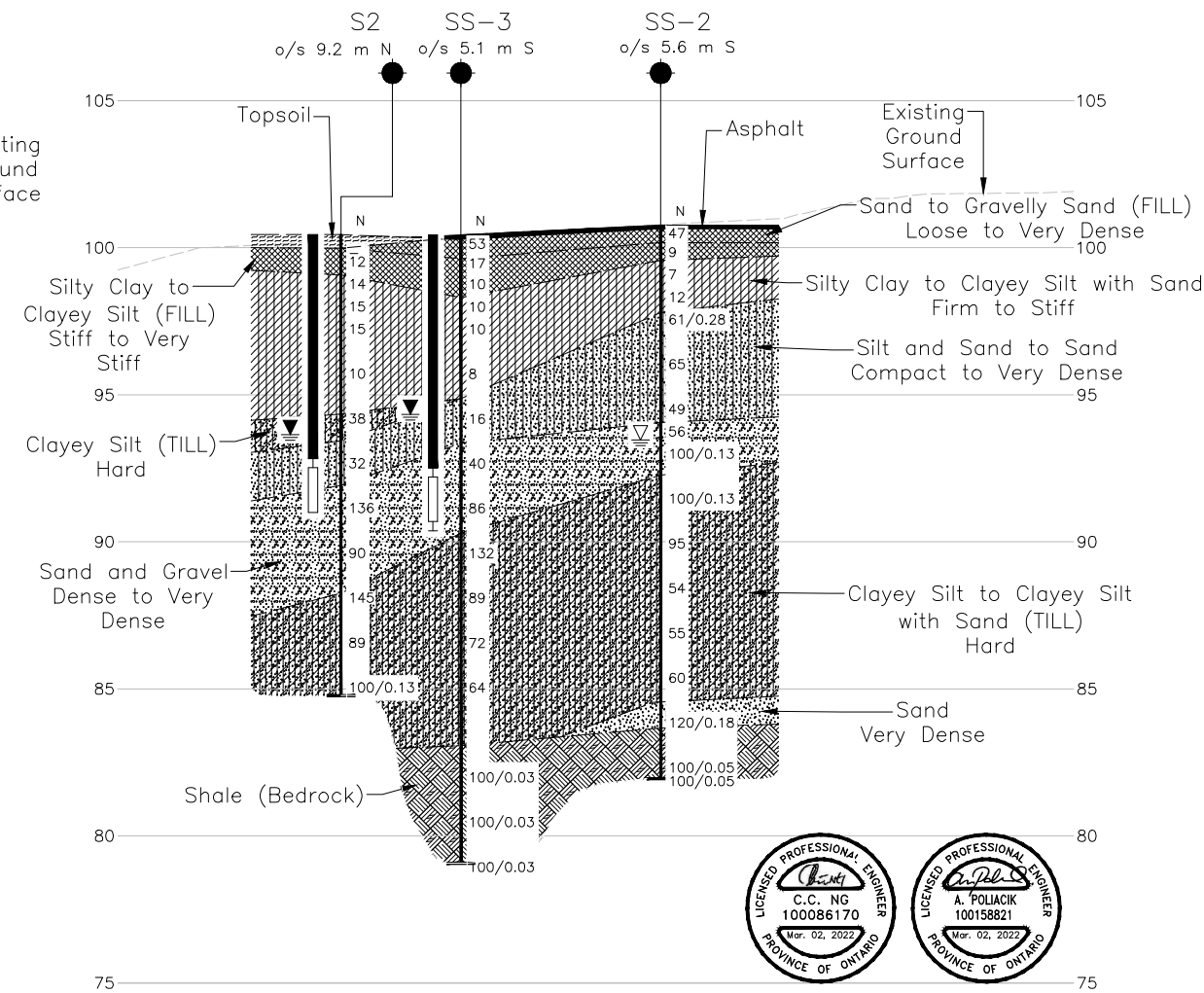
The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

REFERENCE

Base plans provided in digital format by client, dated June 6, 2015.

BridgeGA provided by client in digital format, titled S17104557-SIMCOE-01-GA.dwg, received on February 25, 2022.

NO.	DATE	BY	REVISION
1	03/02/2022	AK	ISSUED FOR CONSTRUCTION
2	03/02/2022	AK	REVISED
3	03/02/2022	AK	REVISED
4	03/02/2022	AK	REVISED
5	03/02/2022	AK	REVISED
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261	03/02/2022	AK	REV

CROSS-SECTION B-B'
NORTH ABUTMENTCROSS-SECTION C-C'
NORTH PIERCROSS-SECTION D-D'
SOUTH PIERCROSS-SECTION E-E'
SOUTH ABUTMENT**METRIC**
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.CONT No.
GWP No. 2298-13-00SIMCOE STREET UNDERPASS
HIGHWAY 401 WIDENING
SOIL STRATA 2

SHEET

WSP **GOLDER**

KEY PLAN

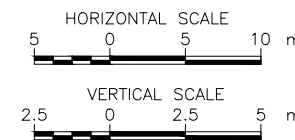


LEGEND

- Borehole - Current Investigation
- ⊕ Borehole - Previous Investigation

BOREHOLE CO-ORDINATES (MTM NAD83 ZONE 10)

No.	ELEVATION	NORTHING	EASTING
SS-1	100.4	4860303.5	356553.7
SS-2	100.8	4860319.6	356549.0
SS-3	100.4	4860315.8	356535.9
SS-4	101.0	4860339.7	356540.4
SS-5	100.8	4860336.9	356532.3
SS-6	95.8	4860359.2	356515.7
SS-7	95.8	4860365.7	356541.9
SS-8	101.2	4860396.0	356523.4
SS-9	101.1	4860394.2	356513.4
SS-10	101.3	4860412.3	356506.7
SS-11	95.8	4860365.0	356512.3
S1	101.3	4860398.8	356528.6
S2	100.4	4860328.0	356527.1



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 client, dated June 6, 2015.

NO.	DATE	BY	REVISION
Geocres No. 30M15-324			
HWY. 401	PROJECT NO. 21466052		DIST. CENTRAL
SUBM'D. AK	CHKD. AK/AJS	DATE: 03/02/2022	SITE: .
DRAWN: SMD/DD	CHKD. CN	APPD. LCC	DWG. 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

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 MTM NAD ZONE 10 (LAT. 43.881760; LONG. -78.856300)			ORIGINATED BY TD																		
DIST Central HWY 401			BOREHOLE TYPE 152 mm O.D. 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 NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES																			
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 3.1 m and 6.1 m depth - Becoming grey below 3.4 m depth		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																								
	Silty SAND, trace clay, trace gravel (TILL) Very dense Brown Moist		8	SS	100/0.1																			
6.8																								
	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																							

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PROJECT 11-1184-0143			RECORD OF BOREHOLE No S2			SHEET 1 OF 2			METRIC					
G.W.P. 10-20011			LOCATION N 4860328.0; E 356527.1 MTM NAD ZONE 10 (LAT. 43.881120; LONG. -78.856300)			ORIGINATED BY TD								
DIST Central HWY 401			BOREHOLE TYPE 152 mm O.D. Solid Stem Augers			COMPILED BY PKS								
DATUM Geodetic			DATE March 11, 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 (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
								20 40 60 80 100	20 40 60 80 100	W _p	W	W _L		
100.4	GROUND SURFACE													
0.0	TOPSOIL													
99.9														
0.5	Silty clay, trace to some sand, trace organics (FILL)		1	AS	-									
	Stiff		2	SS	12									
99.0	Brown													
	Moist													
1.4	Hydrocarbon odour													
	SILTY CLAY to CLAY, trace sand		3	SS	14									
	Stiff to very stiff													
	Brown		4	SS	15									
	Moist		5	SS	15									
			6	SS	10									
94.3	CLAYEY SILT, some sand, trace gravel (TILL)		7	SS	38									
	Hard													
93.4	Grey													
	Moist													
7.0	SILT and SAND, trace to some gravel													
	Dense		8	SS	32									
	Grey													
	Wet													
91.9	SAND and GRAVEL, trace silt													
	Very dense		9	SS	136									
	Grey													
	Moist													
			10	SS	90									
88.2	CLAYEY SILT, some sand, trace to some gravel (TILL)		11	SS	145									
	Hard													
	Grey													
	Moist													
			12	SS	89									

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

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

PROJECT		11-1184-0143		RECORD OF BOREHOLE No S2		SHEET 2 OF 2		METRIC								
G.W.P.		10-20011		LOCATION		N 4860328.0; E 356527.1 MTM NAD ZONE 10 (LAT. 43.881120; LONG. -78.856300)		ORIGINATED BY								
DIST		Central HWY 401		BOREHOLE TYPE		152 mm O.D. Solid Stem Augers		COMPILED BY								
DATUM		Geodetic		DATE		March 11, 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								
	--- CONTINUED FROM PREVIOUS PAGE ---															
84.7			13	SS	100/0.13	85										
15.7	END OF BOREHOLE NOTES: 1. Water level measured in borehole at a depth of 7.6 m below ground surface (Elev. 92.8 m) upon completion of drilling 2. Borehole caved to a depth of 8.2 m below ground surface (Elev. 92.2 m) upon completion of drilling 3. Water level in piezometer measured as follows: Date Depth (m) Elev. (m) Jun 7/16 6.8 93.6 Aug 13/18 6.7 93.7															

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PROJECT		RECORD OF BOREHOLE				No SS-1				SHEET 1 OF 1				METRIC			
G.W.P.		2298-13-00		LOCATION		N 4860303.5; E 356553.7 MTM NAD ZONE 10 (LAT. 43.880900; LONG. -78.855970)				ORIGINATED BY				LP			
DIST		Central HWY 401		BOREHOLE TYPE		216 mm O.D Hollow Stem Augers				COMPILED BY				AK			
DATUM		Geodetic		DATE		November 21, 2017				CHECKED BY				AP			
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa			Wp W WL					γ	
100.4	GROUND SURFACE							20 40 60 80 100							GR SA SI CL		
0.0	ASPHALT (140 mm)																
99.9	Gravelly sand, some silt (FILL)		1	SS	38		100										
0.5	Dense Brown Moist																
99.3	Sand, some silt, trace gravel (FILL)		2	SS	11		99										
1.2	Compact Brown Moist																
	CLAYEY SILT, trace to some sand, trace to some gravel		3	SS	10												
	Very soft to stiff Grey and brown Moist						98										
			4	SS	12												
	- Hydrocarbon odour present from depths of 3.0 m to 3.5 m (Elev. 97.4 m to 96.9 m)		5	SS	12		97										
							96										
			6	SS	11												
							95										
							94										
			7	SS	2												
							93										
93.3	SILTY SAND, trace to some gravel, trace to some clay																
7.1	Compact Grey Wet		8	SS	28												
							92										
91.8	SAND and GRAVEL, some silt																
8.6	Very dense Grey Wet		9	SS	120/0.28		91										
							90										
90.3	CLAYEY SILT with SAND, trace to some gravel (TILL)																
10.1	Very dense Grey Moist																
89.3			10	SS	146/0.25												
11.1	END OF BOREHOLE																
	1. Groundwater level measured at a depth of 6.4 m below ground surface (Elev. 94.0 m) upon completion of drilling.																
	2. Borehole caved to a depth of about 7.9 m below ground surface (Elev. 92.5 m) upon completion of drilling.																

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

PROJECT		1662582 (2000)		RECORD OF BOREHOLE No SS-2		SHEET 2 OF 2		METRIC						
G.W.P.		2298-13-00		LOCATION		N 4860319.6; E 356549.0 MTM NAD ZONE 10 (LAT. 43.881040; LONG. -78.856020)		ORIGINATED BY LP						
DIST		Central HWY 401		BOREHOLE TYPE		216 mm O.D Hollow Stem Augers		COMPILED BY AK						
DATUM		Geodetic		DATE		November 03, 2017		CHECKED BY AP						
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
	--- CONTINUED FROM PREVIOUS PAGE ---							20 40 60 80 100						
84.6	Sandy CLAYEY SILT, trace to some gravel (TILL) Hard Grey Moist		14	SS	60									7 27 31 35
16.2	SAND, trace to some silt, trace to some gravel, trace clay Very dense Grey Wet		15A 15B	SS	120/0.18									7 78 11 4
83.7	SHALE (BEDROCK) Black Hydrocarbon odour													
17.1			16	SS	100/0.05									
82.0			17	SS	100/0.05									
18.8	END OF BOREHOLE DUE TO SPLIT SPOON / AUGER REFUSAL NOTES: 1. Groundwater level measured at a depth of 7.3 m below ground surface (Elev. 93.5 m) upon completion of drilling. 2. Borehole caved to a depth of 7.6 m below ground surface (Elev. 93.2 m) upon completion of drilling and removal of augers.													


PROJECT 1662582 (2000)		RECORD OF BOREHOLE No SS-3		SHEET 1 OF 2		METRIC	
G.W.P. 2298-13-00		LOCATION N 4860315.8; E 356536.0 MTM NAD ZONE 10 (LAT. 43.881010; LONG. -78.856190)		ORIGINATED BY LP			
DIST Central HWY 401		BOREHOLE TYPE 216 mm O.D Hollow Stem Augers		COMPILED BY AK			
DATUM Geodetic		DATE November 08, 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 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		
								20 40 60 80 100	20 40 60 80 100					
100.4	GROUND SURFACE													
0.0	ASPHALT (145 mm)													
0.1	Gravelly sand, some silt (FILL)		1	SS	53									
99.6	Very dense Brown Moist													
0.8	Sand, some silt, trace gravel (FILL)		2	SS	17									
99.0	Compact Brown Moist													
1.4	Clayey silt, trace sand (FILL)		3	SS	10									
98.3	Stiff Brown and grey Moist													
2.1	SILTY CLAY, trace sand, trace gravel		4	SS	10									
	Stiff Brown and grey Moist													
	- Trace rootlets at a depth of 2.3 m (Elev. 98.1 m)		5	SS	10									
	- Hydrocarbon odour at a depth of 4.6 m (Elev. 95.8 m)		6	SS	8									
94.8	SILT and SAND, trace to some clay, trace to some gravel													
5.6	Compact Grey Moist Hydrocarbon odour		7	SS	16									
93.3	SAND and GRAVEL, trace to some silt, trace clay													
7.1	Dense to very dense Grey Wet		8	SS	40									
			9	SS	86									
90.3	CLAYEY SILT with SAND, trace to some gravel (TILL)													
10.1	Hard Grey Moist		10	SS	132									
			11	SS	89									
			12	SS	72									

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

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PROJECT		1662582 (2000)		RECORD OF BOREHOLE No SS-3		SHEET 2 OF 2		METRIC				
G.W.P.		2298-13-00		LOCATION		N 4860315.8; E 356536.0 MTM NAD ZONE 10 (LAT. 43.881010; LONG. -78.856190)		ORIGINATED BY LP				
DIST		Central HWY 401		BOREHOLE TYPE		216 mm O.D Hollow Stem Augers		COMPILED BY AK				
DATUM		Geodetic		DATE		November 08, 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 γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER			TYPE	"N" VALUES	20 40 60 80 100	20 40 60 80 100			W _p W W _L
--- CONTINUED FROM PREVIOUS PAGE ---												
83.0	CLAYEY SILT with SAND, trace to some gravel (TILL) Hard Grey Moist		13	SS	64							2 35 39 24
17.4	SHALE (BEDROCK) Containing residual soil layers Black Hydrocarbon odour											
79.0	END OF BOREHOLE		14	SS	100/0.03							
21.4	NOTES:											
	1. Groundwater level in hollow stem augers measured at a depth of 6.1 m below ground surface (Elev. 94.3 m) upon completion of drilling.											
	2. Monitoring well installed 1.0 m south of Borehole SS-3 location.											
	3. Depth to bedrock inferred from drilling resistance.											
	4. Shear strength data obtained from monitoring well borehole, located 1.0 m south of Borehole SS-3 location.											
	5. The geodetic elevation is approximate.											
	6. Water level in piezometer measured as follows:											
	Date Depth (m) Elev. (m)											
	Aug 13/18 6.2 94.2											

PROJECT 1662582 (2000)			RECORD OF BOREHOLE No SS-4			SHEET 1 OF 1			METRIC							
G.W.P. 2298-13-00			LOCATION N 4860339.7; E 356540.4 MTM NAD ZONE 10 (LAT. 43.881230; LONG. -78.856130)			ORIGINATED BY LP										
DIST Central HWY 401			BOREHOLE TYPE 216 mm O.D Hollow Stem Augers			COMPILED BY AK										
DATUM Geodetic			DATE November 06, 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								
101.0	GROUND SURFACE															
0.0	ASPHALT (130 mm)															
0.1	Gravelly sand, some silt (FILL)		1	SS	50											
100.2	Dense to very dense															
0.8	Brown Moist		2	SS	11											
99.6	Sand, some silt, some gravel (FILL)															
1.4	Compact Brown Moist		3	SS	7											
98.9	Silty clay, trace sand (FILL)															
2.1	Firm Brown and grey Moist		4	SS	29											
	SILT and SAND, trace gravel, containing silty clay layers															
	Compact Grey Moist		5	SS	26											
97.0																
4.0	Gravelly SILTY SAND, trace to some clay (TILL)		6	SS	46											
	Dense Grey Moist															
95.4																
5.6	SILT AND SAND, some gravel, trace clay															
	Compact to dense Grey Moist		7	SS	42											
			8	SS	13											
93.5																
7.5	Gravelly SILT and SAND, trace to some clay (TILL)		9	SS	47											
	Dense to very dense Grey Moist															
			10	SS	100/0.15											
			11	SS	137/0.20											
89.3																
11.7	CLAYEY SILT with SAND, trace gravel (TILL)															
	Hard Grey Moist		12	SS	101											
88.4																
12.7	END OF BOREHOLE NOTES:															
1. Groundwater level measured at a depth of 7.9 m below ground surface (Elev. 93.1 m) upon completion of drilling. 2. Borehole caved to a depth of 8.5 m below ground surface (Elev. 92.5 m) upon removal of augers and completion of drilling.																

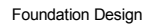
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PROJECT		RECORD OF BOREHOLE				No SS-5		SHEET 1 OF 1		METRIC					
G.W.P.		2298-13-00		LOCATION		N 4860336.9; E 356532.3 MTM NAD ZONE 10 (LAT. 43.881200; LONG. -78.856230)				ORIGINATED BY LP					
DIST		Central HWY 401		BOREHOLE TYPE		216 mm O.D Hollow Stem Augers				COMPILED BY AK					
DATUM		Geodetic		DATE		November 01, 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							
100.8	GROUND SURFACE														
0.0	ASPHALT (155 mm)														
100.3	Gravelly sand, some silt (FILL)		1	SS	39										
0.5	Dense Brown Moist														
99.6	Sand, some silt, trace gravel (FILL)		2	SS	10										
1.2	Compact Brown Moist														
98.7	Clayey silt, trace sand, trace gravel (FILL)		3	SS	4										
2.1	Soft Grey Moist		4	SS	6										
	CLAYEY SILT, trace sand Firm to stiff Brown and grey Moist Hydrocarbon odour		5	SS	11										
96.8	CLAYEY SILT with SAND, some gravel Stiff Grey Moist		6	SS	9										
95.1	SILTY SAND, trace to some gravel, trace clay (TILL) Dense to very dense Grey Moist to wet		7	SS	48										
			8	SS	70										
92.9	SAND, trace silt, trace gravel, trace clay Very dense Grey Moist		9A 9B	SS	126/0.25										
92.2	SAND and GRAVEL, trace to some silt, trace clay Very dense Grey Moist to wet		10	SS	100/0.15										
90.7	CLAYEY SILT with SAND, trace gravel (TILL) Hard Grey Moist		11	SS	105/0.23										
89.8	END OF BOREHOLE														
11.1	NOTES: 1. Borehole dry upon completion of drilling. 2. Borehole caved to a depth of 10.1 m below ground surface (Elev. 90.7 m) upon completion of drilling and removal of augers.														

PROJECT		1662582 (2000)		RECORD OF BOREHOLE No SS-6		SHEET 1 OF 1		METRIC								
G.W.P.		2298-13-00		LOCATION		N 4860359.2; E 356515.7 MTM NAD ZONE 10 (LAT. 43.881400; LONG. -78.856440)		ORIGINATED BY PS								
DIST		Central HWY 401		BOREHOLE TYPE		216 mm O.D Hollow Stem Augers		COMPILED BY AK								
DATUM		Geodetic		DATE		March 19, 2018		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								
95.8	GROUND SURFACE															
0.0	ASPHALT (250 mm)															
0.3	Sand, some gravel, trace silt (FILL)		1	SS	26											
95.0	Compact Brown Moist		2	SS	18											
0.8	Silty sand, trace to some gravel, trace clayey silt pockets (FILL)		3	SS	15											
	Loose to compact Brown Moist to wet		4	SS	7											
	- Trace organics at a depth of 2.4 m (Elev. 93.4 m)		5	SS	7											
	- Hydrocarbon odour noted between depths of 2.7 m and 3.7 m (Elev. 93.1 m to 92.1 m)															
92.1	Gravelly SAND, trace to some silt, trace clay		6	SS	35											
3.7	Dense Brown Wet		7	SS	53											
91.3	SILT and SAND, trace to some gravel, trace to some clay (TILL)		8	SS	100/0.10											
4.5	Very dense Brown Wet		9	SS	100/0.25											
			10	SS	100/0.25											
86.4	END OF BOREHOLE															
9.4	NOTE: 1. Borehole dry upon completion of drilling.															

PROJECT		1662582 (2000)		RECORD OF BOREHOLE No SS-7		SHEET 1 OF 1		METRIC								
G.W.P.		2298-13-00		LOCATION		N 4860365.7; E 356541.9 MTM NAD ZONE 10 (LAT. 43.881460; LONG. -78.856110)		ORIGINATED BY LP								
DIST		Central HWY 401		BOREHOLE TYPE		216 mm O.D Hollow Stem Augers		COMPILED BY AK								
DATUM		Geodetic		DATE		March 19, 2018		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								
95.8	GROUND SURFACE															
0.0	ASPHALT (150 mm)															
0.2	Gravelly sand, some silt (FILL) Very loose to dense Brown Moist to wet		1	SS	31											
			2	SS	18											
			3	SS	14											
			4	SS	2											
92.9																
2.9	SILT and SAND, trace to some gravel, trace to some clay (TILL) Very dense Grey Moist		5	SS	100/0.13											
			6	SS	100/0.15											
			7	SS	100/0.07											
			8	SS	100/0.13											
			9	SS	100/0.15											
87.2																
8.6	SAND, trace to some silt, trace to some gravel Very dense Grey Wet		10	SS	140/0.22											
86.3																
9.5	END OF BOREHOLE															
NOTES:																
1. Groundwater level measured at a depth of 6.4 m below ground surface (Elev. 89.6 m) upon completion of drilling.																

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							20	40	60	80	100					
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												
			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												
			9	SS	92												
93.7	SAND, trace silt, trace gravel, trace clay																
7.5	Very dense Brown Moist		10A	SS	108												
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.																	



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


PROJECT		1662582 (2000)		RECORD OF BOREHOLE No SS-10				SHEET 1 OF 1		METRIC							
G.W.P.		2298-13-00		LOCATION		N 4860412.3; E 356506.7 MTM NAD ZONE 10 (LAT. 43.881880; LONG. -78.856540)				ORIGINATED BY LP							
DIST		Central HWY 401		BOREHOLE TYPE		216 mm O.D Hollow Stem Augers				COMPILED BY AK							
DATUM		Geodetic		DATE		November 01, 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									
101.3	GROUND SURFACE																
0.0	ASPHALT (135 mm)																
0.1	Sand, some gravel, some silt (FILL)		1	SS	36												
100.5	Dense Brown Moist		2	SS	8												
0.8	Clayey silt, some sand (FILL)																
	Firm Dark brown Moist		3	SS	4												
99.2																	
2.1	SILT and SAND, trace to some clay, trace to some gravel (TILL)		4	SS	30												
	Compact to dense Brown, becoming grey below a depth of 4.6 m (Elev. 96.7 m)		5	SS	21												
	- Oxidation staining from a depth of 2.1 m to 4.6 m (Elev. 96.7 m to 99.2 m)		6	SS	28												
95.7																	
5.6	SAND and GRAVEL, trace to some silt, trace clay		7	SS	98/0.23												
	Very dense Grey Moist to wet																
	- Crushed rock fragments recovered at a depth of 6.1 m (Elev. 95.2 m)																
			8	SS	103												
			9	SS	100/0.25												
			10	SS	105												
90.2																	
11.1	END OF BOREHOLE																
	NOTES:																
	1. Groundwater level in borehole measured at a depth of 7.5 m below ground surface (Elev. 93.8 m) upon completion of drilling.																
	2. Borehole caved to a depth of 7.6 m (Elev. 93.7 m) below ground surface upon completion of drilling and auger removal.																

PROJECT		21466052		RECORD OF BOREHOLE No SS-11		SHEET 1 OF 2		METRIC											
G.W.P.		2298-13-00		LOCATION		N 4860365.0; E 356512.3 MTM NAD 83 ZONE 10 (LAT. 43.881455; LONG. -78.856477)		ORIGINATED BY MTI											
DIST		Central HWY 401		BOREHOLE TYPE		200 mm O.D. Hollow Stem Augers; HQ3 Coring		COMPILED BY AM											
DATUM		HT2 0 (Geodetic)		DATE		January 27-31, 2022		CHECKED BY AP/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	100	20	40
95.8 0.0	GROUND SURFACE Refer to Borehole SS-6 from GEOCRETS Report No. 30M15-324 for stratigraphy between ground surface (Elev. 95.7 m) to a depth of 7.6 m (Elev. 88.2 m).																		
88.2 7.6	SILT (ML) and sand of slight plasticity, trace gravel (TILL) Very dense Grey Moist		1	SS	102/0.13														
87.1 8.7	SILTY SAND (SM), some gravel Very dense Grey Moist		2	SS	101/0.23														
86.0 9.8	Sandy CLAYEY SILT-SILT(CL-ML) to sandy CLAYEY SILT (CL), some gravel (TILL) Hard Grey Moist		3	SS	100/0.15														
			4	SS	100/0.20														
			5	SS	100/0.03														
83.3 12.5	SHALE/LIMESTONE (BEDROCK)		1A	SS	08/0.01														
			1B	RC	REC 100%														
			2	RC	REC 100%														
			3	RC	REC 100%														

Continued Next Page

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

GTA-MTO 001 S:\CLIENTS\TOHWY_401_OSHAWA\02_DATA\GINT\HWY_401_OSHAWA.GPJ GAL-GTA.GDT 3/3/22

PROJECT <u>21466052</u>		RECORD OF BOREHOLE No SS-11				SHEET 2 OF 2		METRIC									
G.W.P. <u>2298-13-00</u>		LOCATION <u>N 4860365.0; E 356512.3 MTM NAD 83 ZONE 10 (LAT. 43.881455; LONG. -78.856477)</u>				ORIGINATED BY <u>MTI</u>											
DIST <u>Central</u> HWY <u>401</u>		BOREHOLE TYPE <u>200 mm O.D. Hollow Stem Augers; HQ3 Coring</u>				COMPILED BY <u>AM</u>											
DATUM <u>HT2_0 (Geodetic)</u>		DATE <u>January 27-31, 2022</u>				CHECKED BY <u>AP/CN</u>											
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa									
	--- CONTINUED FROM PREVIOUS PAGE ---						20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					10 20 30 WATER CONTENT (%)					
79.9	SHALE/LIMESTONE (BEDROCK)		3	RC	REC 100%											RQD = 94%	
15.9	END OF BOREHOLE																
	NOTE: 1. Groundwater level measured at a depth of 2.8 m below ground surface (Elev. 93.0 m) prior to rock coring																

GTA-MTO 001 S:\CLIENTS\MTI\HWY_401_OSHAWA\02_DATA\GINT\HWY_401_OSHAWA.GPJ GAL-GTA.GDT 3/3/22

[illegible]

FEATURES LEGEND



BROKEN CORE



CLAY SEAM



LIMESTONE



LOST CORE

DEPTH SCALE

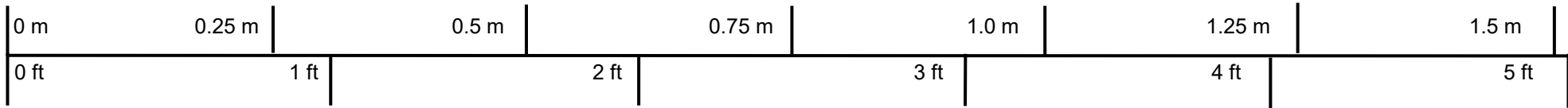
1 : 50




LOGGED: MTI

CHECKED: AP/CN

Borehole SS-11



Scale

PROJECT					
Detail Design for Replacement of Simcoe Street Underpass					
Site No. 22-176					
MTO Agreement 2016-E-0022, GWP 2298-13-00					
TITLE					
Core Photographs					
Borehole SS-11 (12.4 m – 15.85 m)					
			PROJECT No. 21466052		FILE No. 21466052 (2000)
			DRAFT	LJV	FEB 2022
			CADD	--	
			CHECK	AMP	FEB 2022
			REVIEW		FEB 2022
			SCALE	AS SHOWN	VER. 1.
			FIGURE A1		

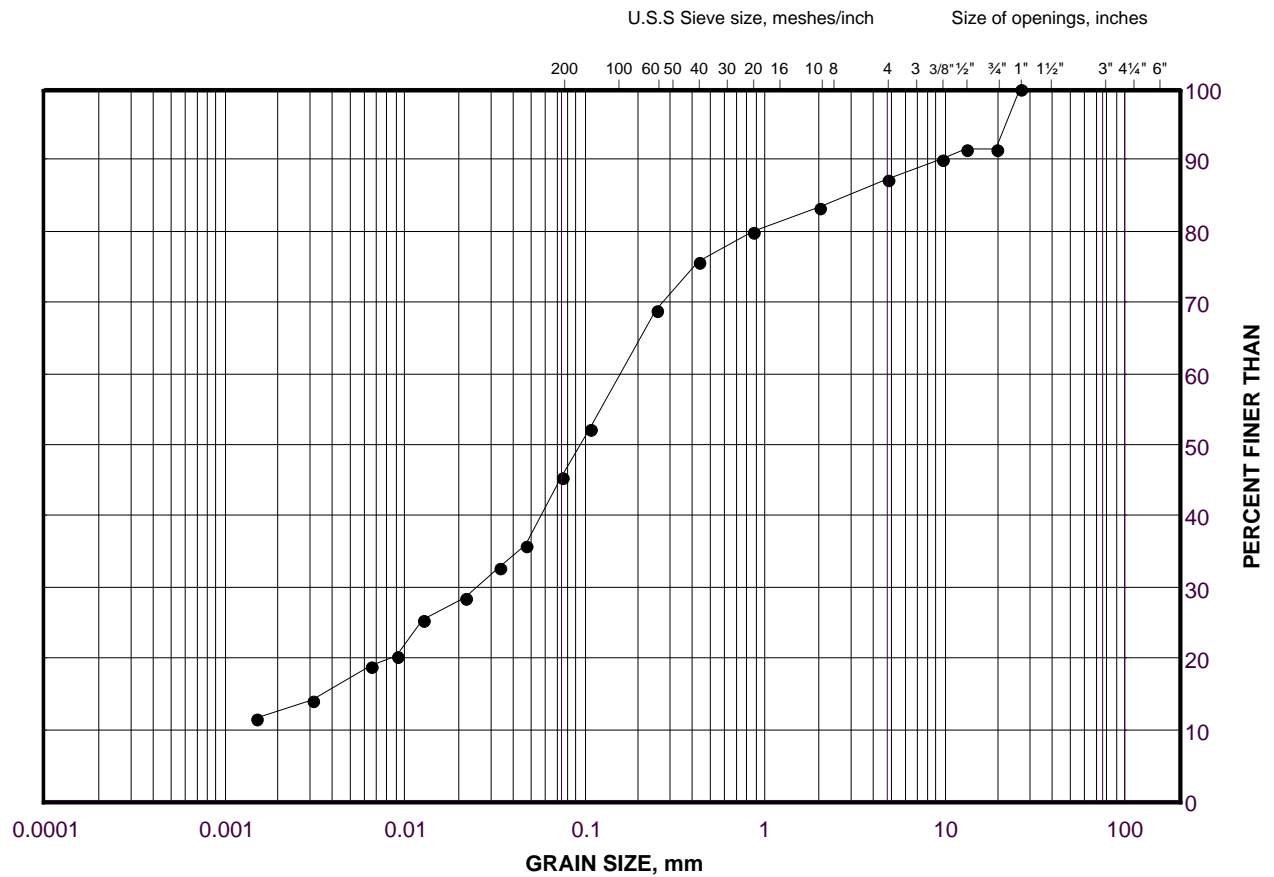
APPENDIX B

Geotechnical Laboratory Results

GRAIN SIZE DISTRIBUTION

Clayey Silt with Sand (FILL)

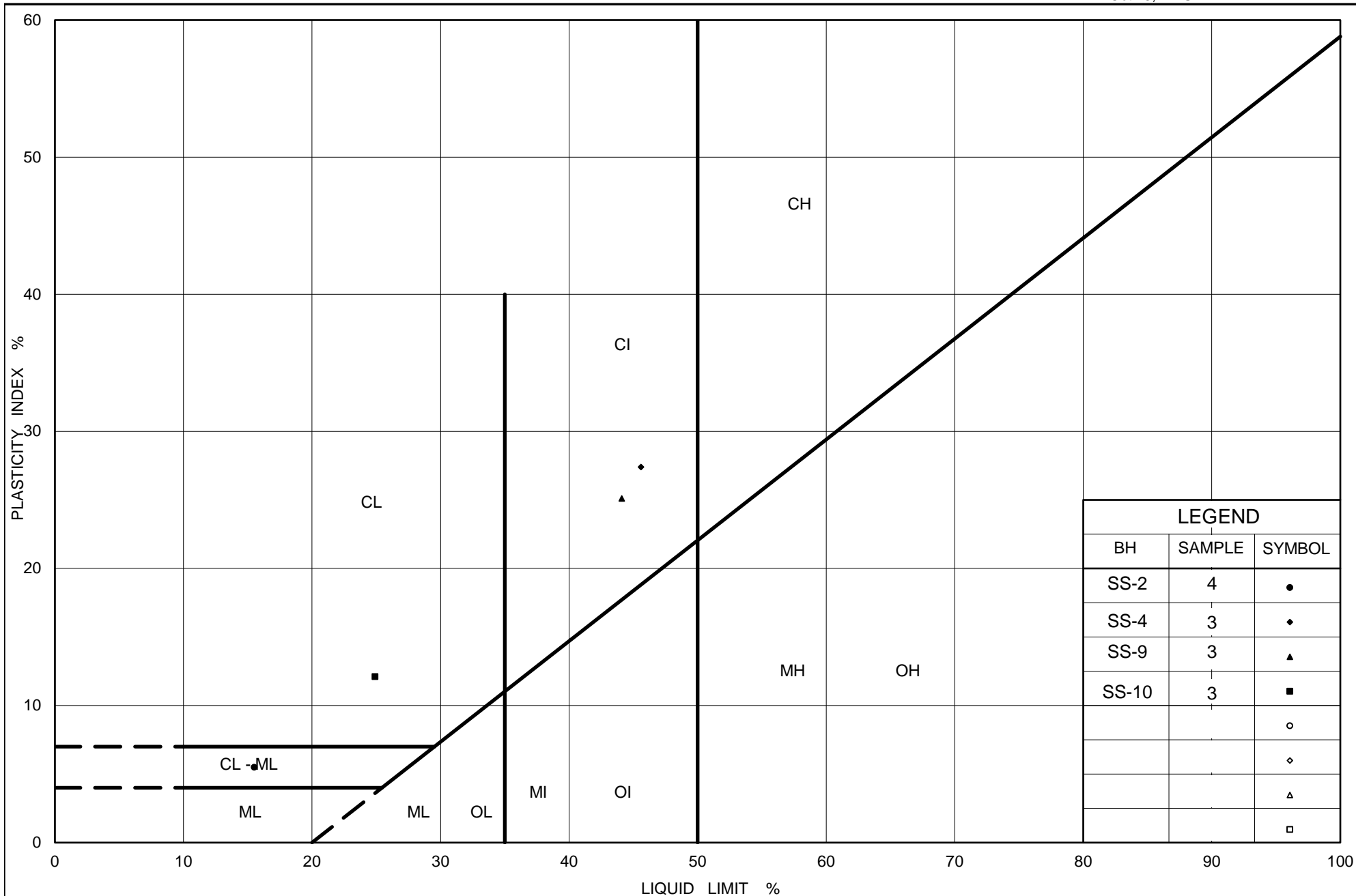
FIGURE B1



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
•	SS-2	4	98.2



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PLASTICITY CHART Silty Clay to Clayey Silt with Sand (FILL)

Figure No. B2

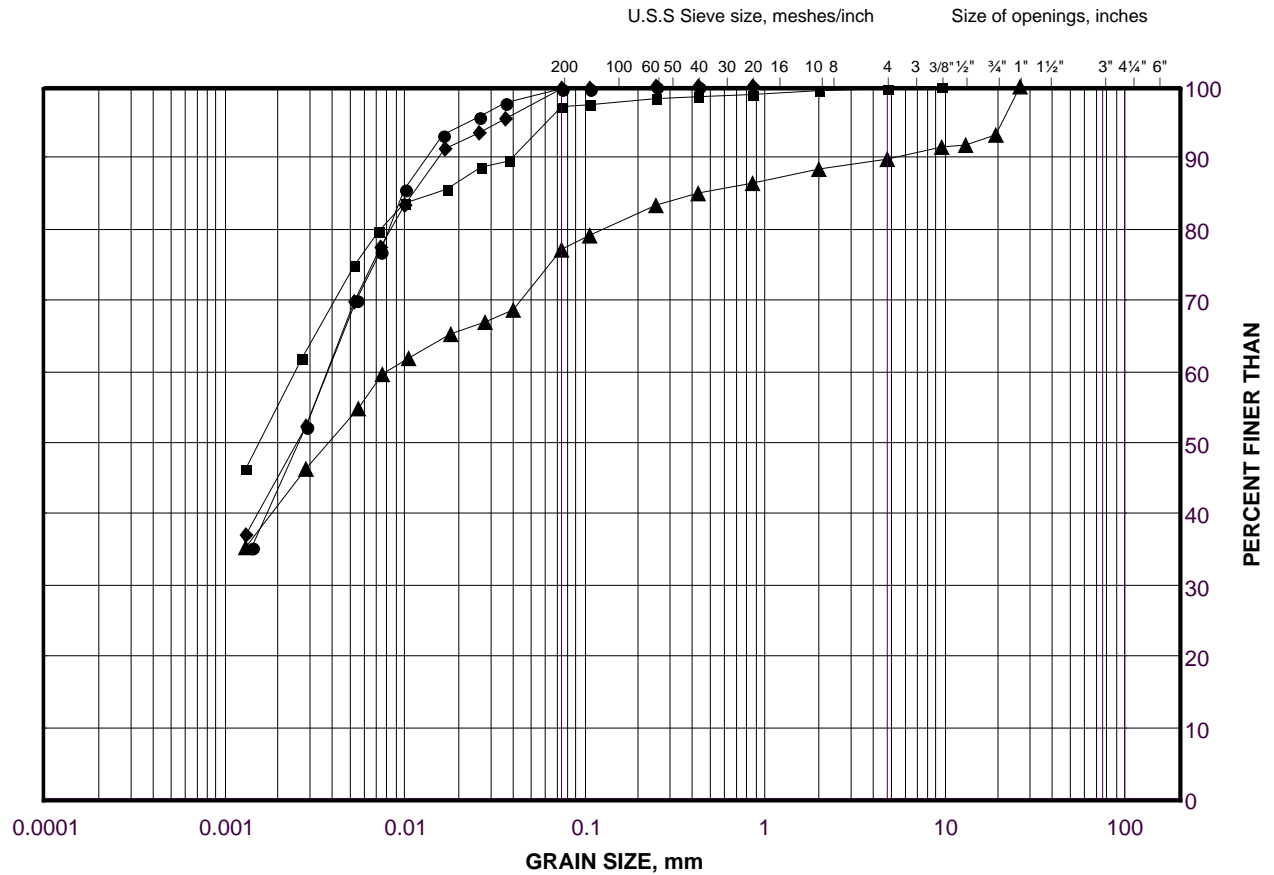
Project No. 1662582

Checked By: AP

GRAIN SIZE DISTRIBUTION

CLAY to CLAYEY SILT

FIGURE B3



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

LEGEND

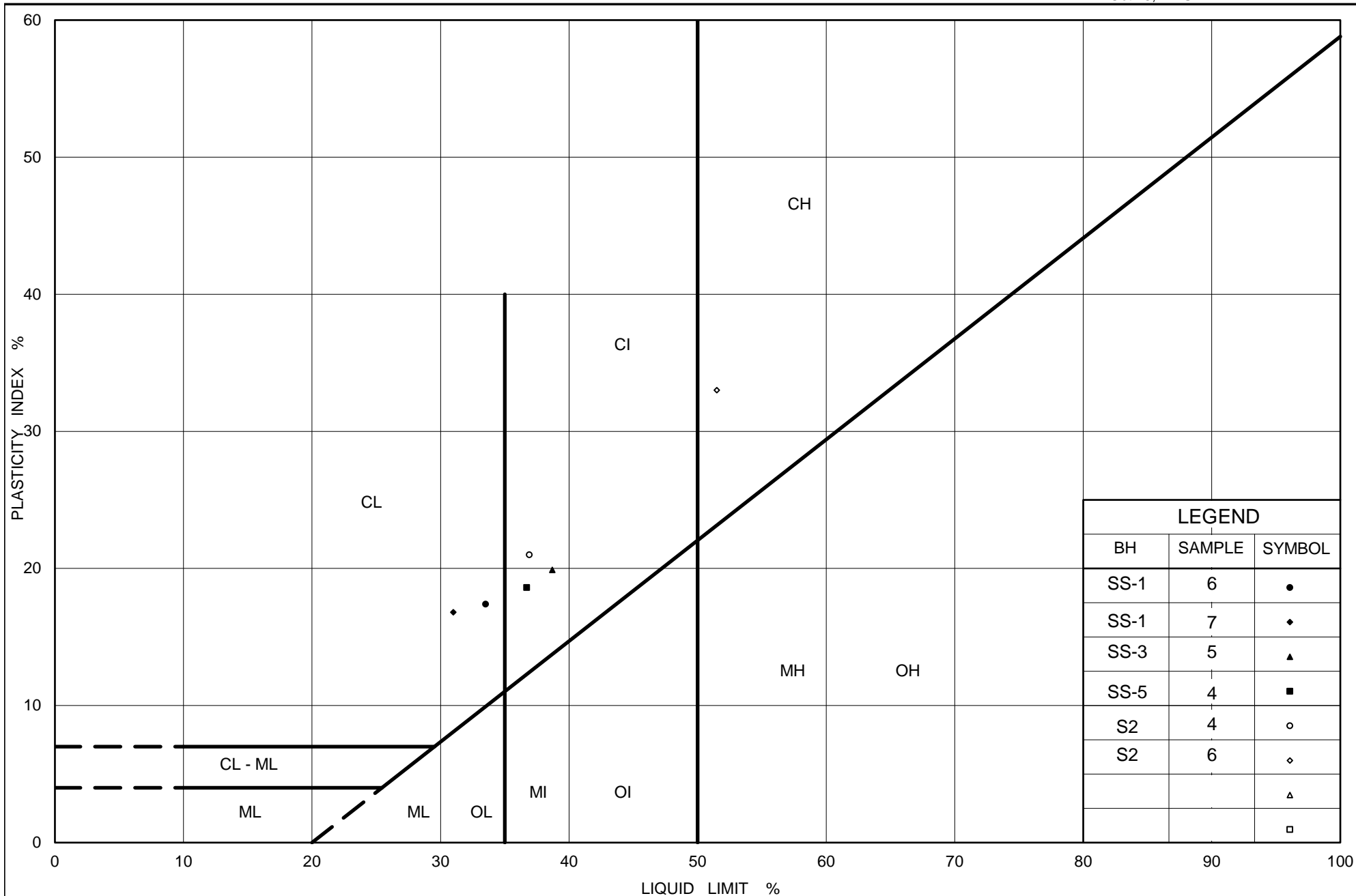
SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	SS-5	4	98.2
■	S2	6	95.6
◆	SS-1	6	95.5
▲	SS-1	7	94.0

Project Number: 1662582,11-1184-0143

Checked By: AP

Golder Associates

Date: 14-Aug-18



Ministry of Transportation

Ontario

PLASTICITY CHART CLAY to CLAYEY SILT

Figure No. B4

Project No. 1662582

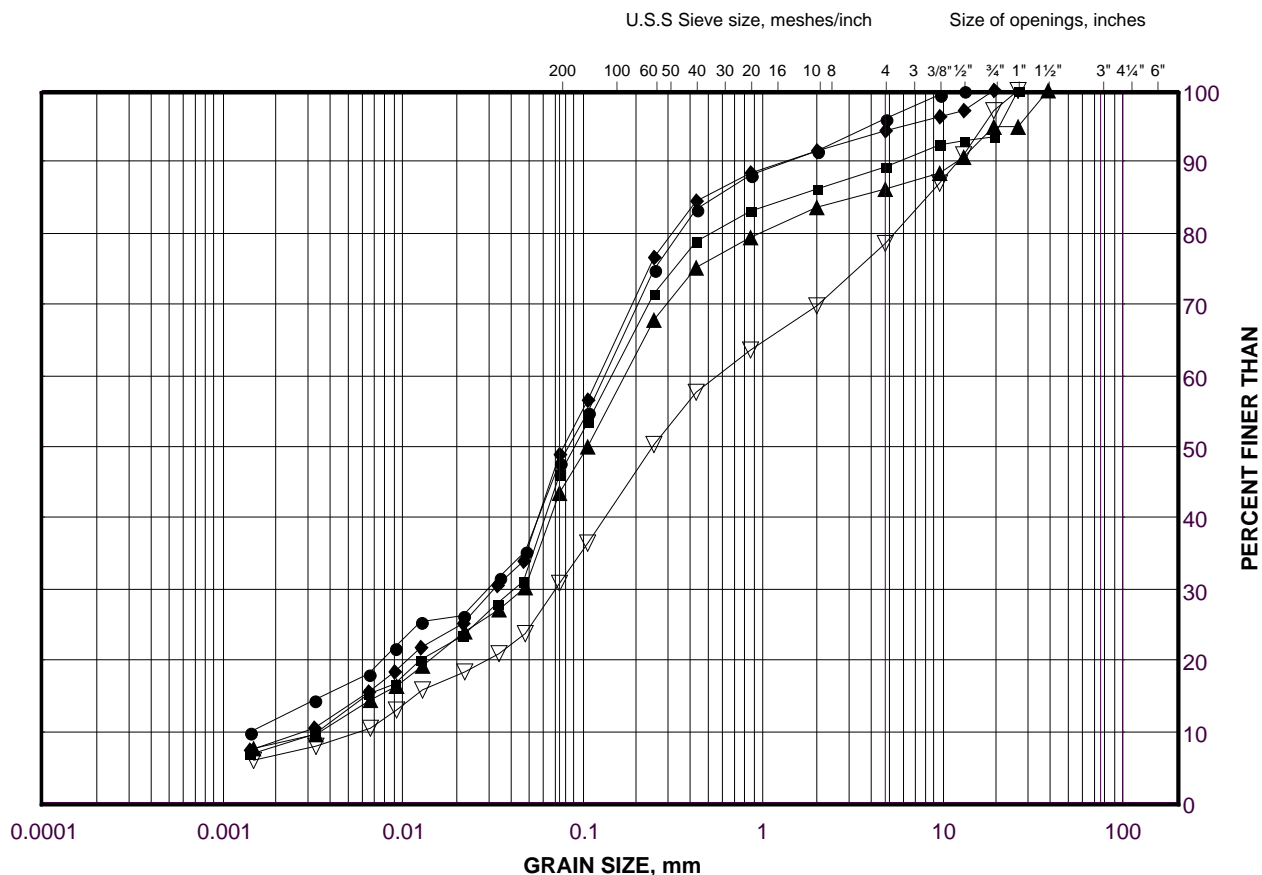
Checked By: AP

GRAIN SIZE DISTRIBUTION

SILT and SAND to SILTY SAND (TILL)

(Upper Deposit)

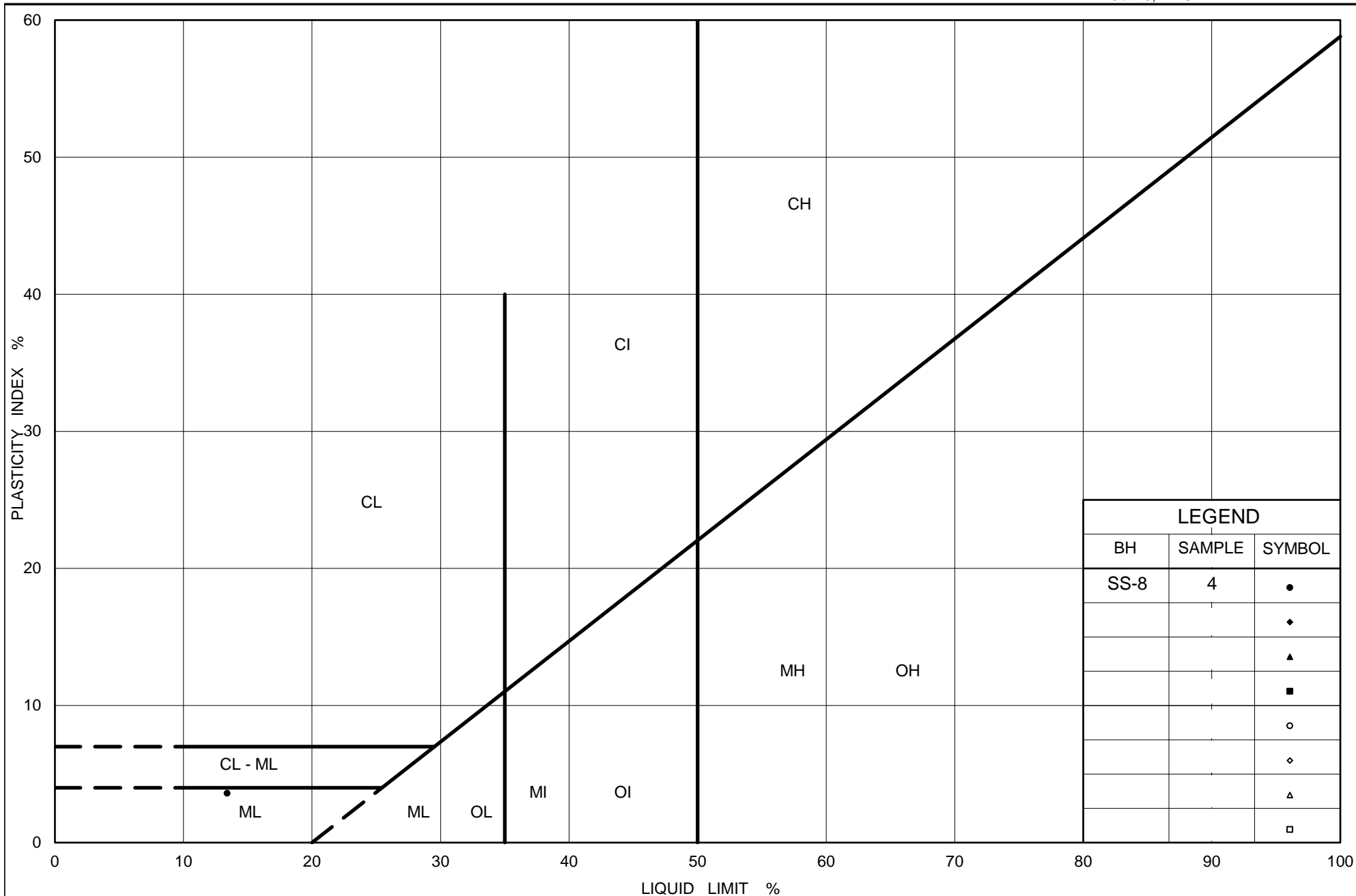
FIGURE B5



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	99.5
■	SS-8	4	98.7
◆	SS-10	5	98.0
▲	SS-9	5	97.8
▽	SS-4	6	96.3



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PLASTICITY CHART SILT and SAND (TILL) Fines Portion

Figure No. B6

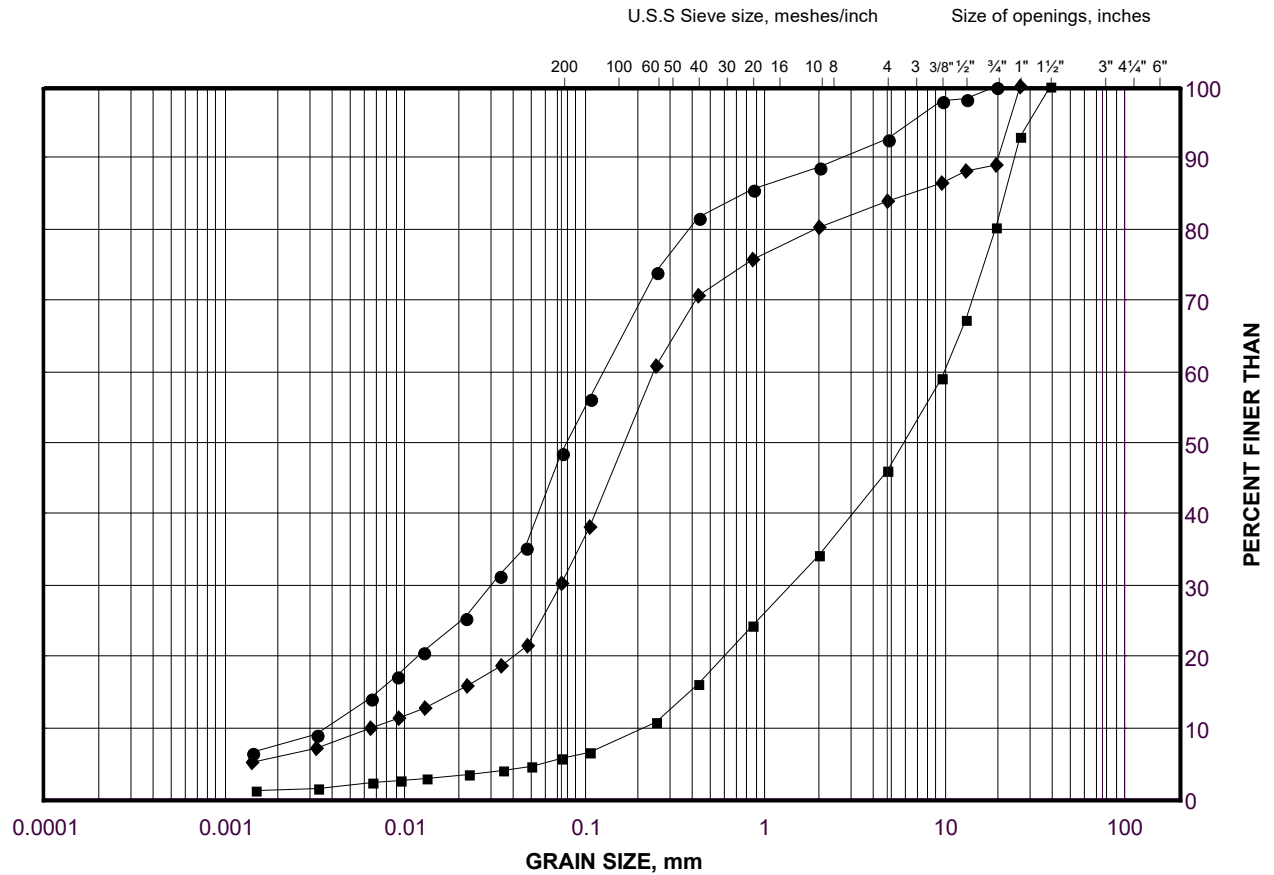
Project No. 1662582

Checked By: AP

GRAIN SIZE DISTRIBUTION

SILT and SAND to SAND
(Upper Deposit)

FIGURE B7



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	SS-3	SA7	94.0
■	SS-2	SA8	92.5
◆	SS-11	SA2	86.5

Project Number: 21466052

Checked By: AP

Golder Associates

Date: 30-Mar-22

FIGURE B8A



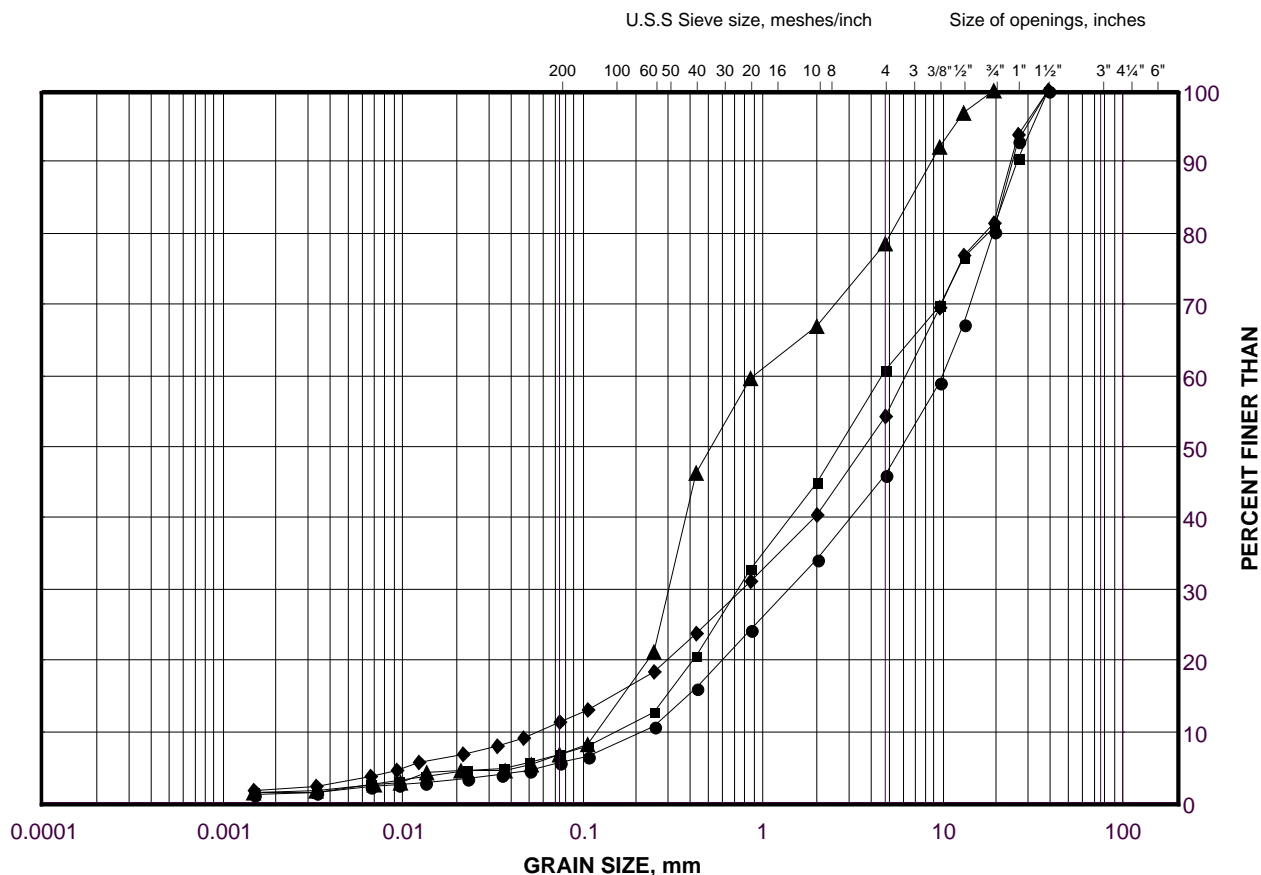
SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	S2	10	89.5
■	SS-5	10	91.5
◆	SS-6	6	91.9
▲	S1	7	95.0
▽	SS-8	8	94.9

Date: 14-Aug-18

GRAIN SIZE DISTRIBUTION

GRAVELLY SAND to SAND and GRAVEL

FIGURE B8B



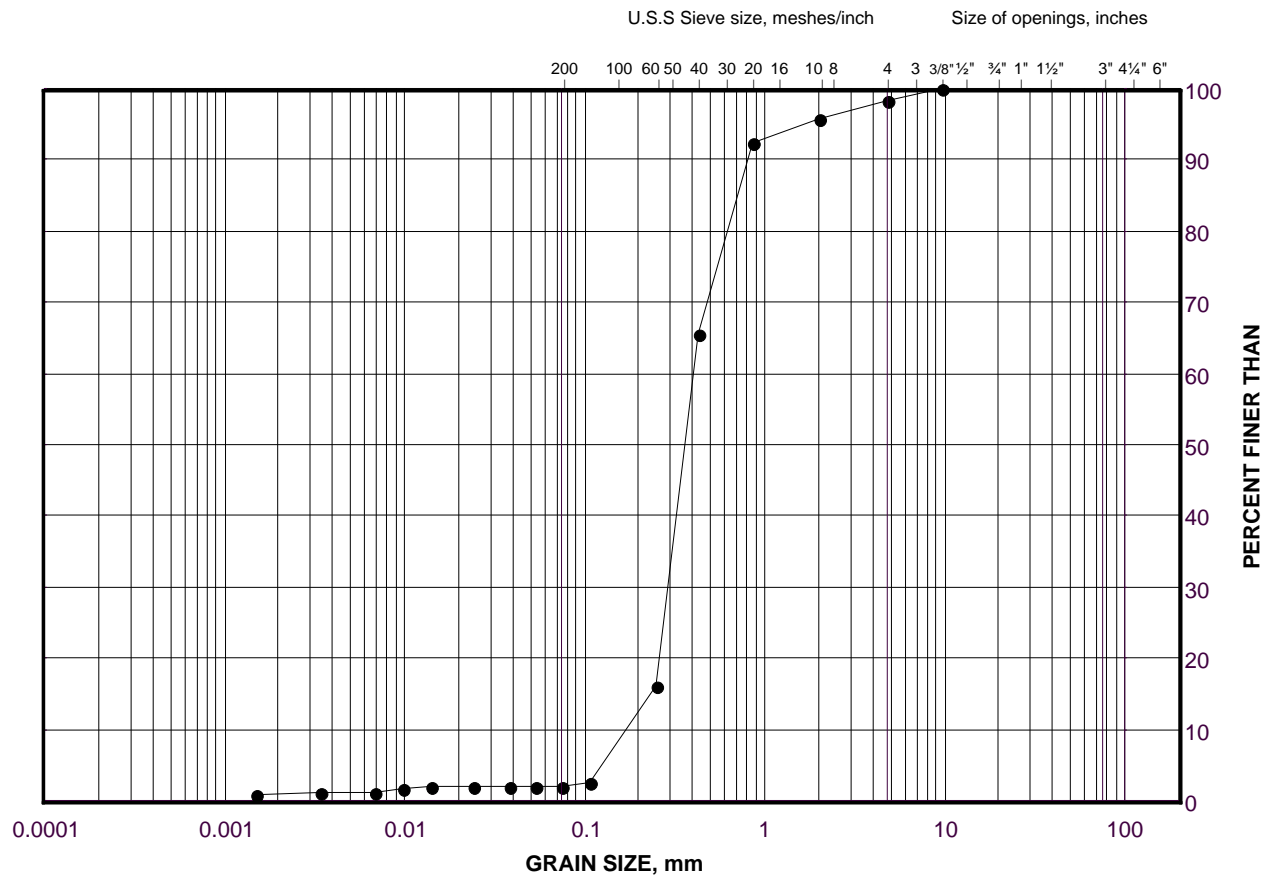
LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	SS-2	8	93.8
■	SS-10	8	93.5
◆	SS-3	9	90.9
▲	SS-9	9	93.3

GRAIN SIZE DISTRIBUTION

SAND
(Interlayer)

FIGURE B9



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

Project Number: 1662582,11-1184-0143

Checked By: AP

Golder Associates

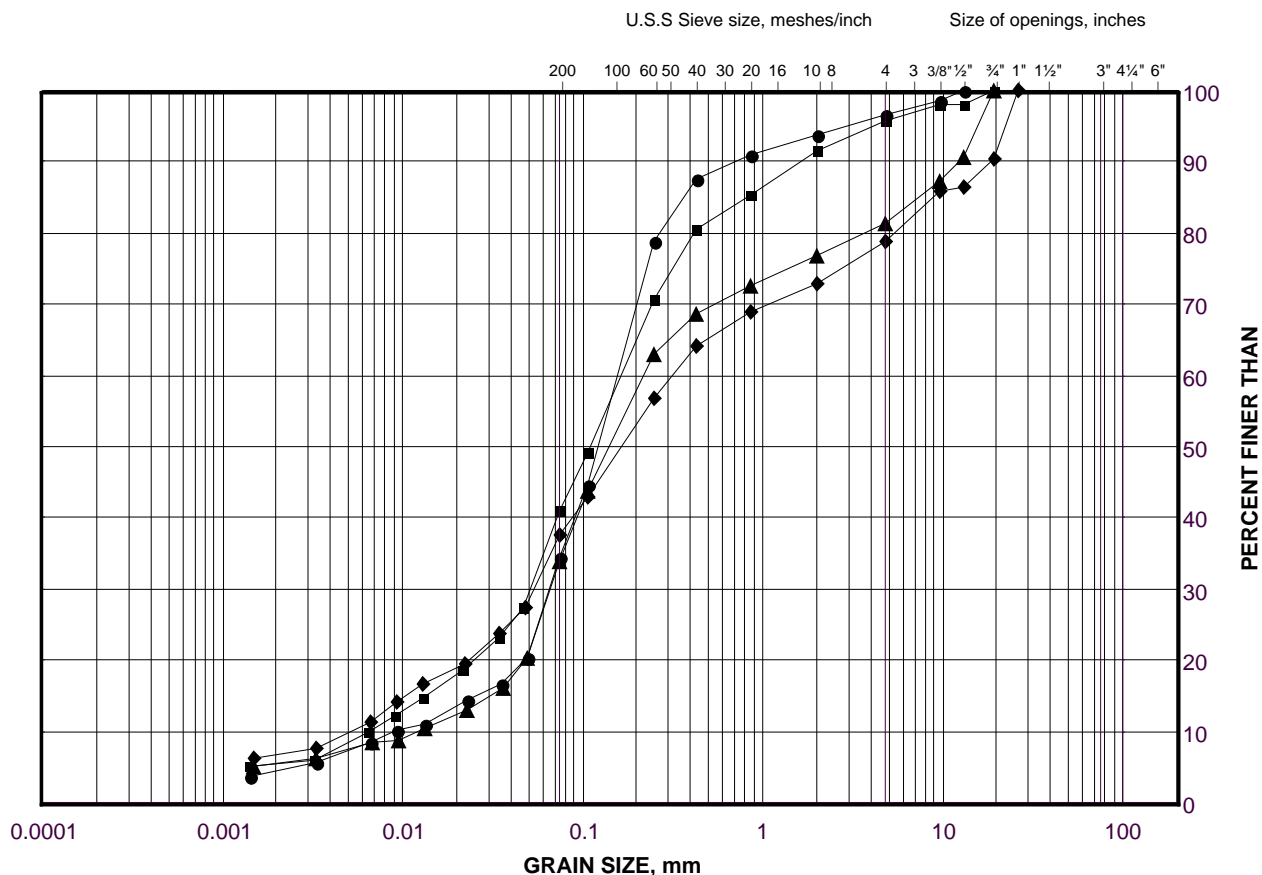
Date: 16-Aug-18

GRAIN SIZE DISTRIBUTION

SILT and SAND to SILTY SAND (TILL)

(Lower Deposit)

FIGURE B10



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

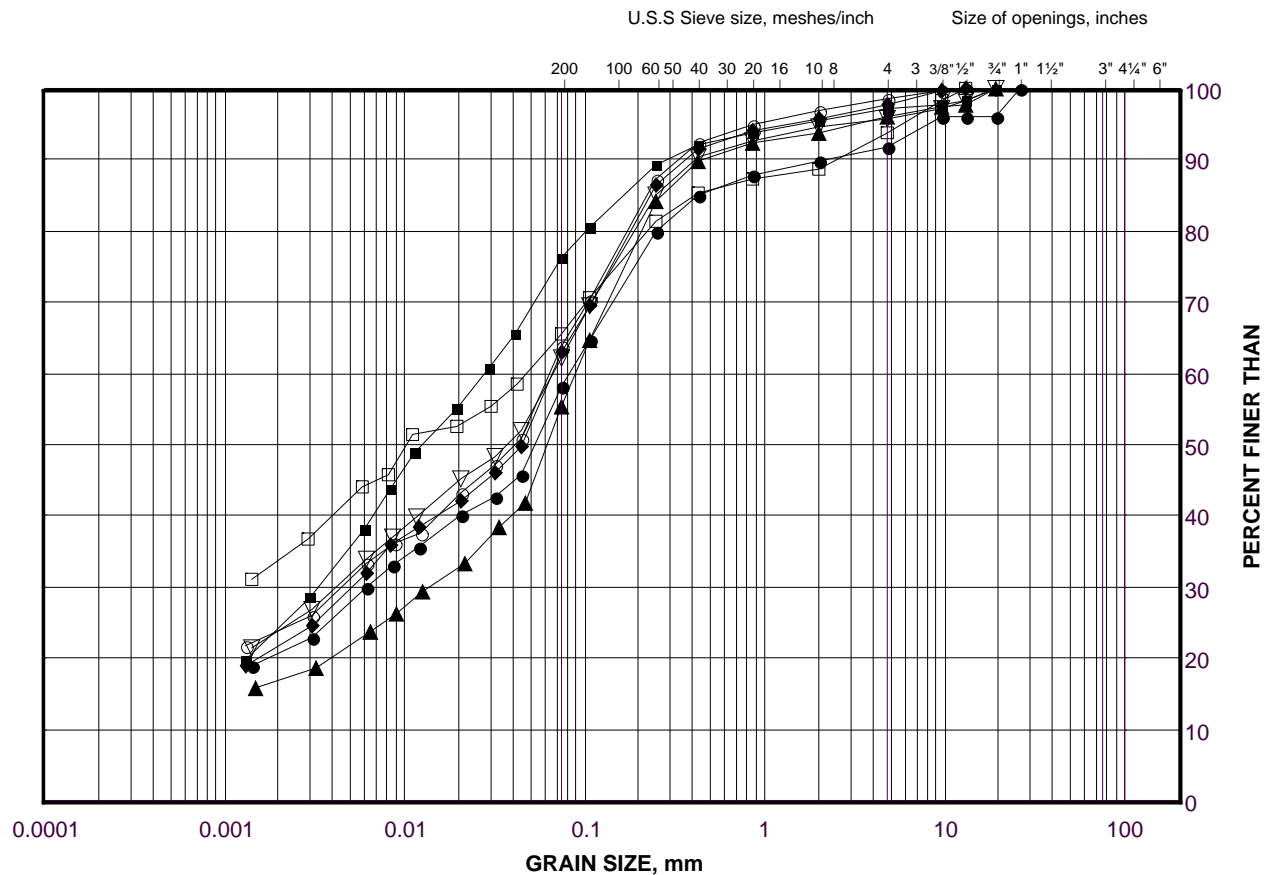
LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	SS-7	6	91.9
■	SS-6	8	89.8
◆	SS-4	9	93.2
▲	SS-5	9A	93.0

GRAIN SIZE DISTRIBUTION

Sandy CLAYEY SILT to CLAYEY SILT with SAND (TILL)

FIGURE B11A



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	SS-1	10	89.6
■	SS-9	11	90.5
◆	SS-5	11	90.0
▲	SS-3	11	88.0
▽	SS-4	12	88.5
○	SS-3	13	84.9
□	SS-2	14	85.3

Project Number: 21466052

Checked By: AP

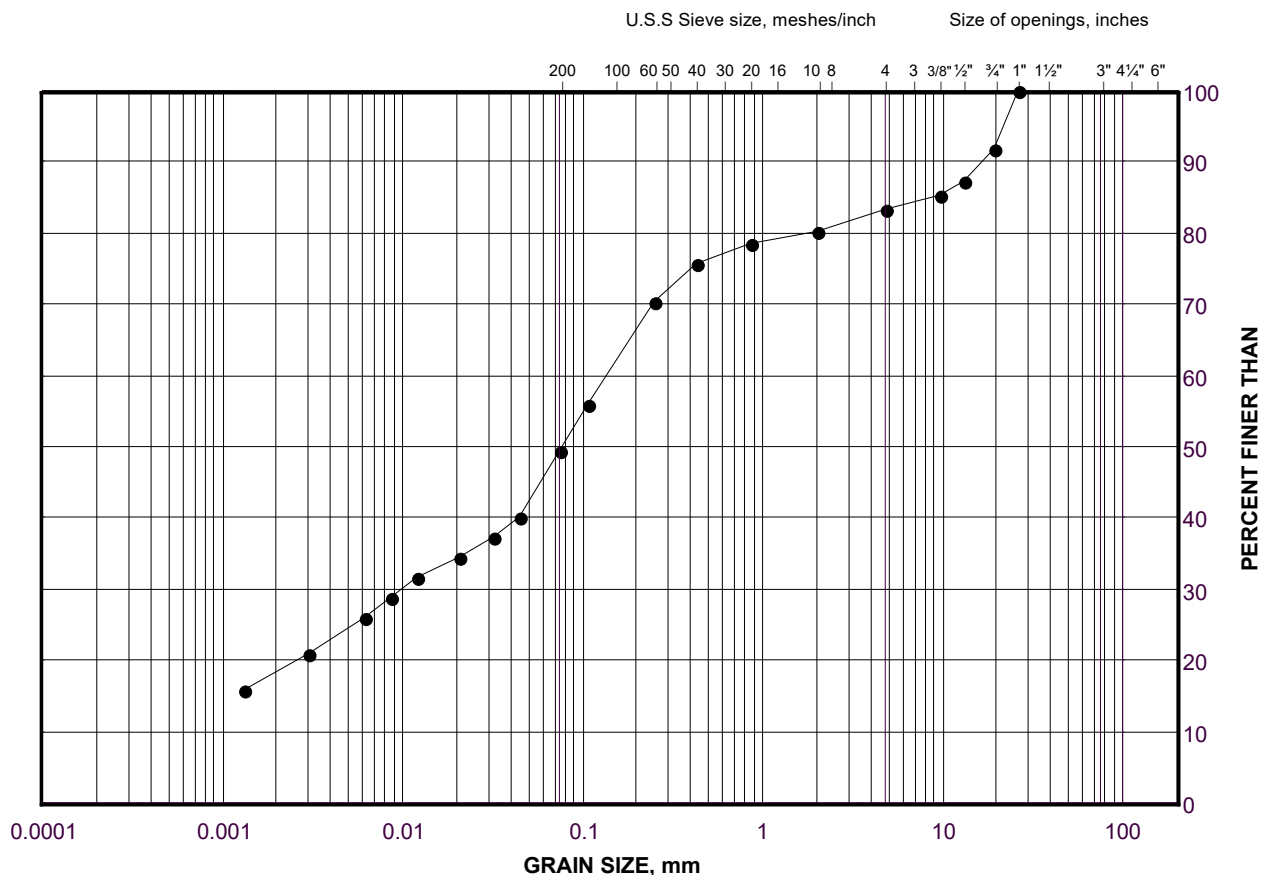
Golder Associates

Date: 30-Mar-22

GRAIN SIZE DISTRIBUTION

Sandy CLAYEY SILT to CLAYEY SILT with SAND (TILL)

Figure B11B



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

LEGEND

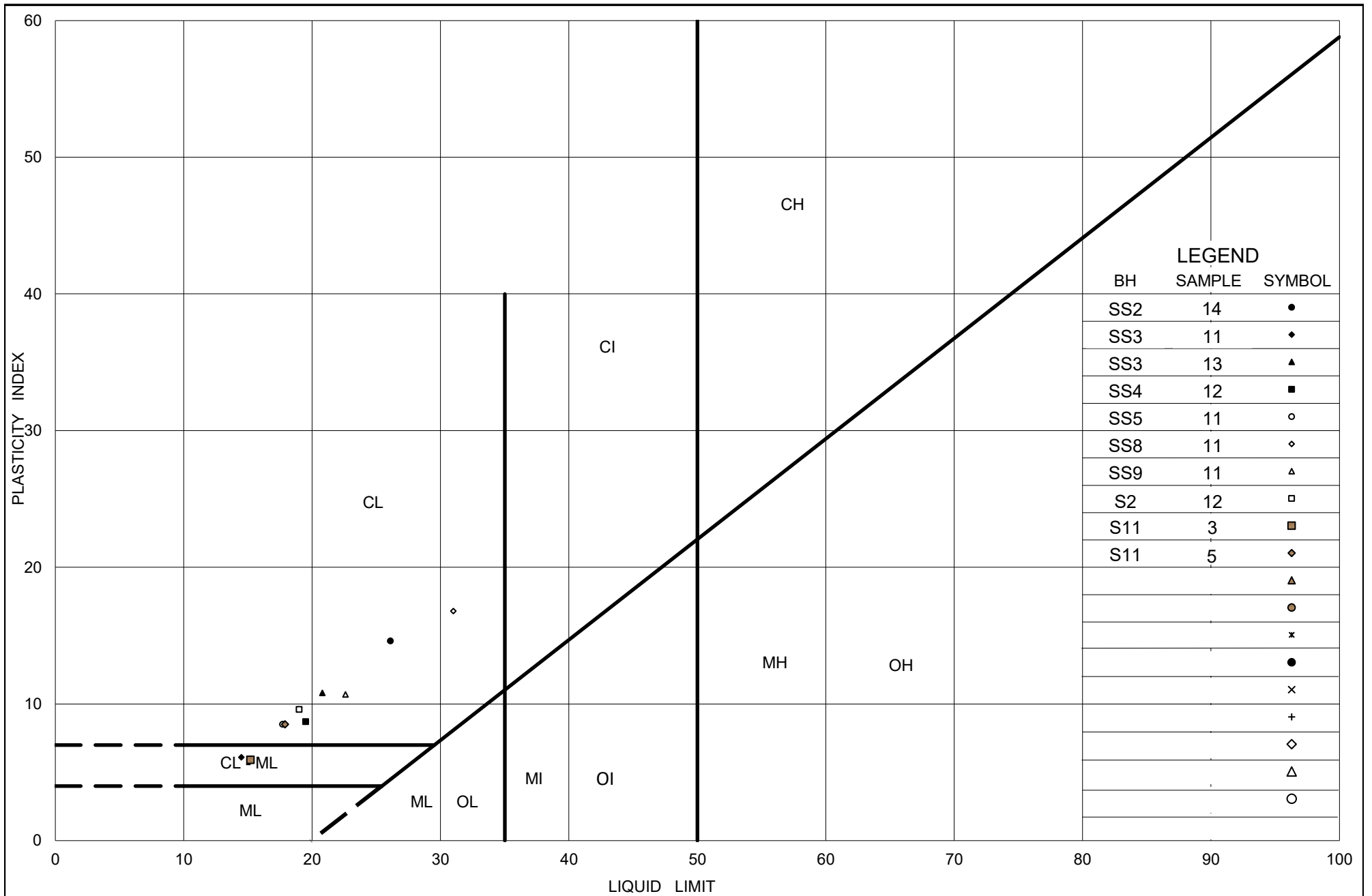
SYMBOL	Borehole	SAMPLE	ELEVATION(m)
•	SS-11	SA-05	84.3

Project Number: 21466052

Checked By: AP

Golder Associates

Date: 30-Mar-22



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PLASTICITY CHART Sandy CLAYEY SILT-SILT to CLAYEY SILT with SAND (TILL)

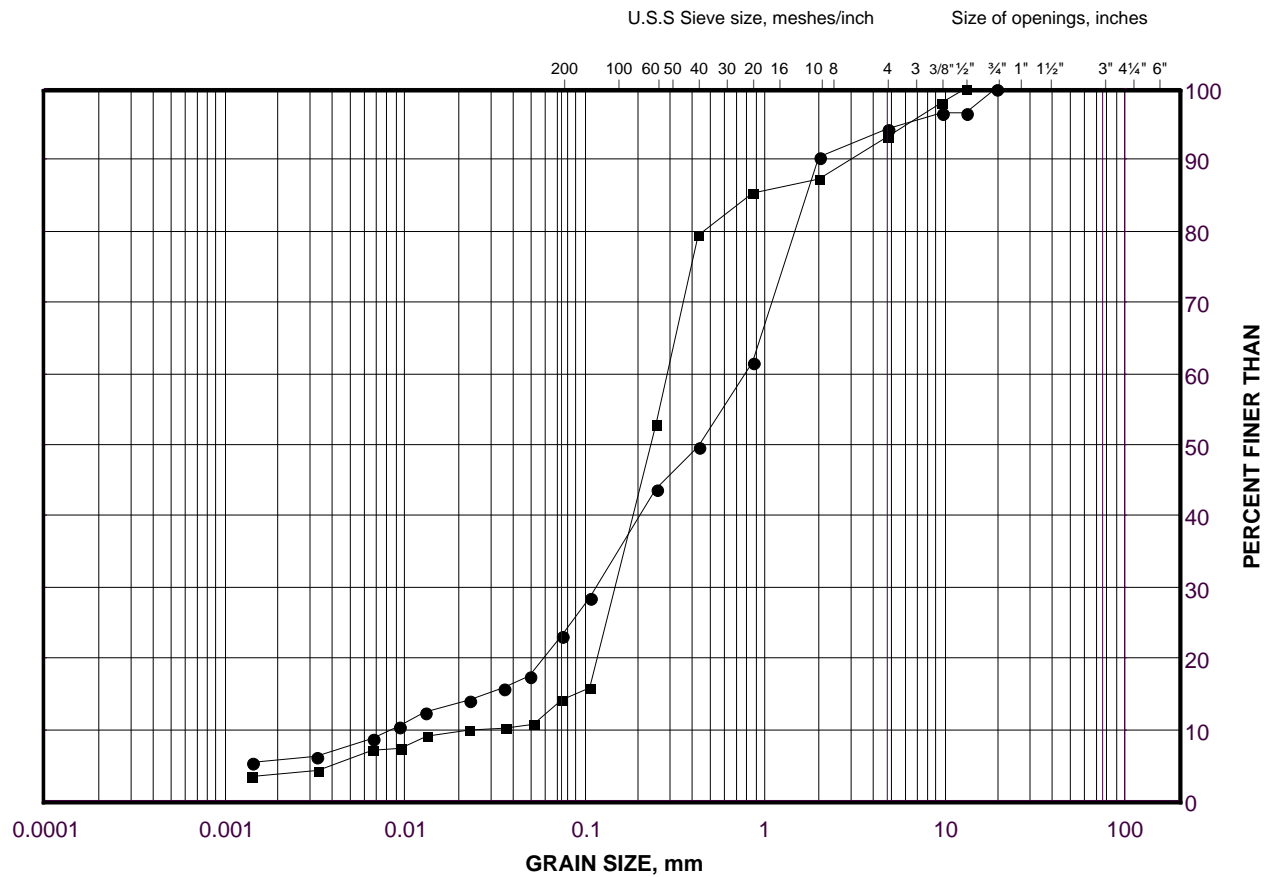
Figure No. B12

Project No. 21466052

Checked By: AP

SAND
(Lower Deposit)

FIGURE B13



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	SS-7	10	86.7
■	SS-2	15A	83.9

Rock Laboratory Testing Results

A report submitted to:

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

Prepared by:

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Toronto ON
M5H 2Y2 Canada
Tel: +1-647-478-9767
lab@geomechanica.com

February 14, 2022

Project number: 21466052

Abstract

This document summarizes the results of rock laboratory testing, including 1 Uniaxial Compression Strength (UCS) test. The UCS value along with photographs of the specimen before and after testing are presented herein.

In this document:

Appendices

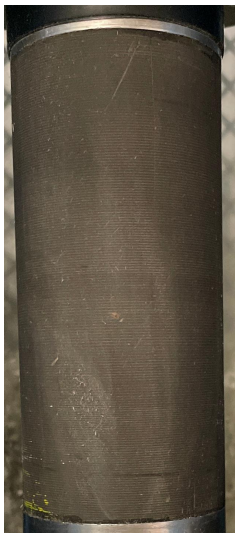

1

Appendices

Specimen sheets

- S-11, SA1

Uniaxial Compression Test

Client	Golder Associates Ltd.	Project	21466052
Sample	S-11, SA1	Depth	13.05 - 13.27
<div>Specimen parameters</div>		Prior to testing	After testing
Diameter (mm) ^a	60.73		
Length (mm) ^a	128.42		
Bulk density ρ (g/cm ³)	2.643		
UCS (MPa)	82.0		
Lithology	Brown shale		
Failure description ^b	1		
<div>^a Additional specimen measurement/details provided in accompanying summary spreadsheet.</div> <div>^b Failure description: ¹ Inclined shear fracture and axial splitting failure;</div>			
Remarks: Loading rate: 0.15 mm/min			
Performed by	BSAT/HS	Date	2022-02-14

APPENDIX C

Analytical Test Results

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 CaCl2 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		Report Information (if differs from invoice)		Project Information (where applicable)		Turnaround Time (TAT) Required																											
Company Name: <u>Golden Associates Ltd.</u>		Company Name: <u>AI Varshoi</u>		Quotation #: _____		<input checked="" type="checkbox"/> Regular TAT (5-7 days) Most analyses																											
Contact Name: <u>[REDACTED]</u>		Contact Name: <u>AI Varshoi</u>		P.O. #/ A/E #: _____		PLEASE PROVIDE ADVANCE NOTICE FOR RUSH PROJECTS																											
Address: <u>6925 Century Ave. #100</u> <u>MISSISSAUGA ON</u>		Address: _____		Project #: <u>1602582</u>		Rush TAT (Surcharges will be applied)																											
Phone: <u>905-567-4444</u> Fax: _____		Phone: _____ Fax: _____		Site Location: <u>OSHANA/HWY 401</u>		<input type="checkbox"/> 1 Day <input type="checkbox"/> 2 Days <input type="checkbox"/> 3-4 Days																											
Email: <u>[REDACTED]</u>		Email: <u>AI-Varshoi@golder.com</u>		Site #: _____		Date Required: _____																											
Sampled By: <u>LP</u>		Rush Confirmation #: _____		LABORATORY USE ONLY		CUSTODY SEAL																											
MOE REGULATED DRINKING WATER OR WATER INTENDED FOR HUMAN CONSUMPTION MUST BE SUBMITTED ON THE MAXXAM DRINKING WATER CHAIN OF CUSTODY		Analysis Requested		Present: <u>Y</u> Intact: <u>N</u>		COOLER TEMPERATURES																											
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 _____ 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: _____ <input type="checkbox"/> Other (Specify) _____ <input type="checkbox"/> REG 558 (MIN. 3 DAY TAT REQUIRED)		REG 153 METALS & INORGANICS REG 153 ICP/MS METALS REG 153 METALS (Pb, Cr, VI, ICP/MS Metals, HWS - B) Corrosivity Package		COOLING MEDIA PRESENT: Y / <u>N</u>		COMMENTS																									
Include Criteria on Certificate of Analysis: Y / N		SAMPLES MUST BE KEPT COOL (< 10 °C) FROM TIME OF SAMPLING UNTIL DELIVERY TO MAXXAM		SAMPLER IDENTIFICATION		DATE SAMPLED (YYYY/MM/DD)		TIME SAMPLED (HH:MM)		MATRIX		# OF CONTAINERS SUBMITTED		FIELD FILTERED (CIRCLE) Metals / Ig / Cvx		BTEX / HIC F1		PH/2 / 4		VOCs		REG 153 METALS & INORGANICS		REG 153 ICP/MS METALS		REG 153 METALS (Pb, Cr, VI, ICP/MS Metals, HWS - B)		Corrosivity Package		HOLD- DO NOT ANALYZE			
1		SS2-SA10		2017/11/03		AM		SOIL		1		1		1		1		1		1		1		1		1		1		1		1	
2		[REDACTED]		[REDACTED]		[REDACTED]		[REDACTED]		[REDACTED]		[REDACTED]		[REDACTED]		[REDACTED]		[REDACTED]		[REDACTED]		[REDACTED]		[REDACTED]		[REDACTED]		[REDACTED]		[REDACTED]		[REDACTED]	
3		SS8-SA10		2017/10/30		AM		SOIL		1		1		1		1		1		1		1		1		1		1		1		1	
4		SS2-SA3		2017/11/06		AM		SOIL		1		1		1		1		1		1		1		1		1		1		1		1	
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)																							
Katherine Karkhanavich		2018/02/28		9:38		[Signature]		2018/02/28		09:39																							

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28-Feb-18 09:39
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TLI ENV-410

White: Maxxam - Yellow: Client

APPENDIX D

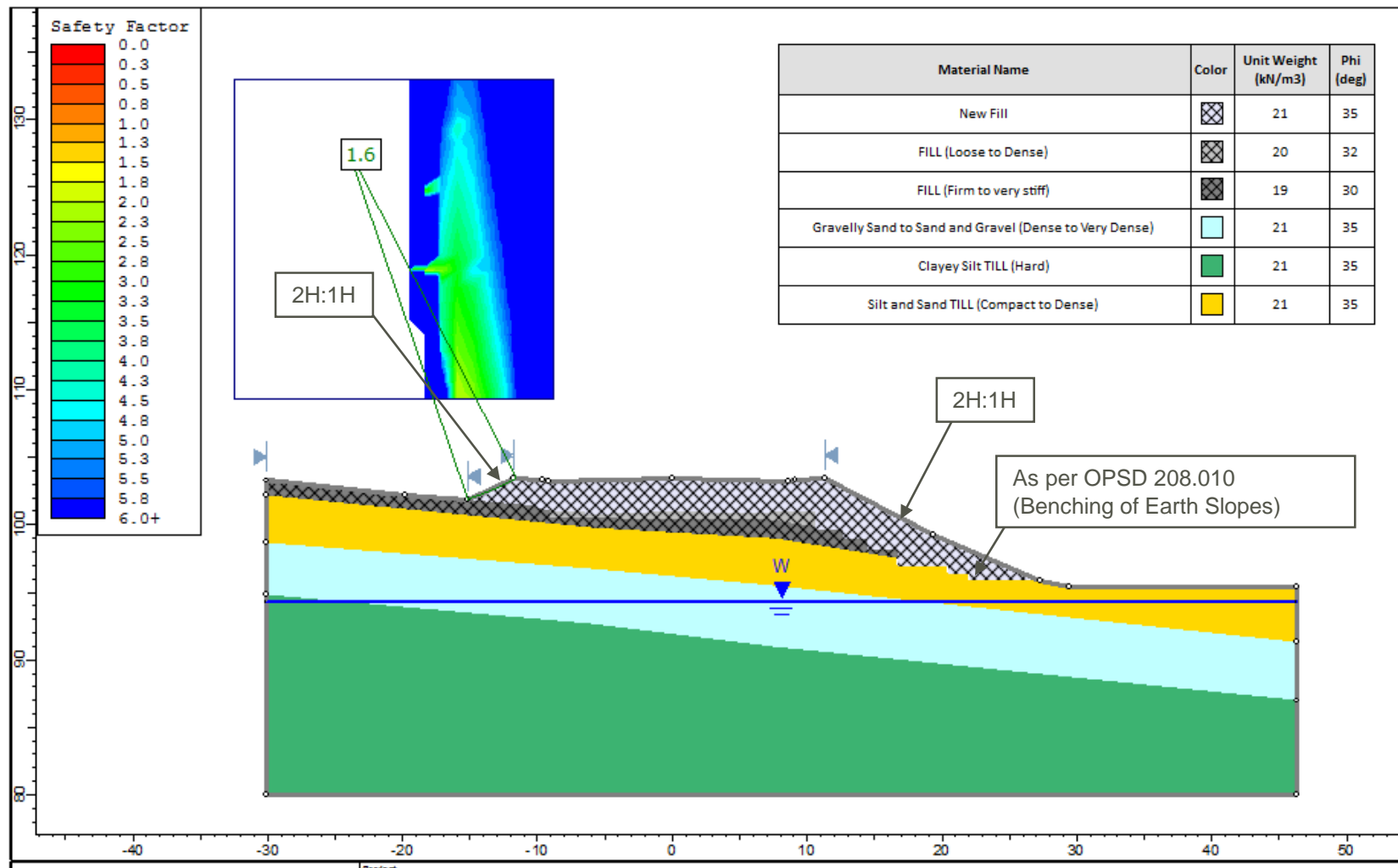
Stability Analysis Figures

Global Stability Analysis

North Approach Embankment – East Slope

Long-term (Permanent) Analysis

Figure D1

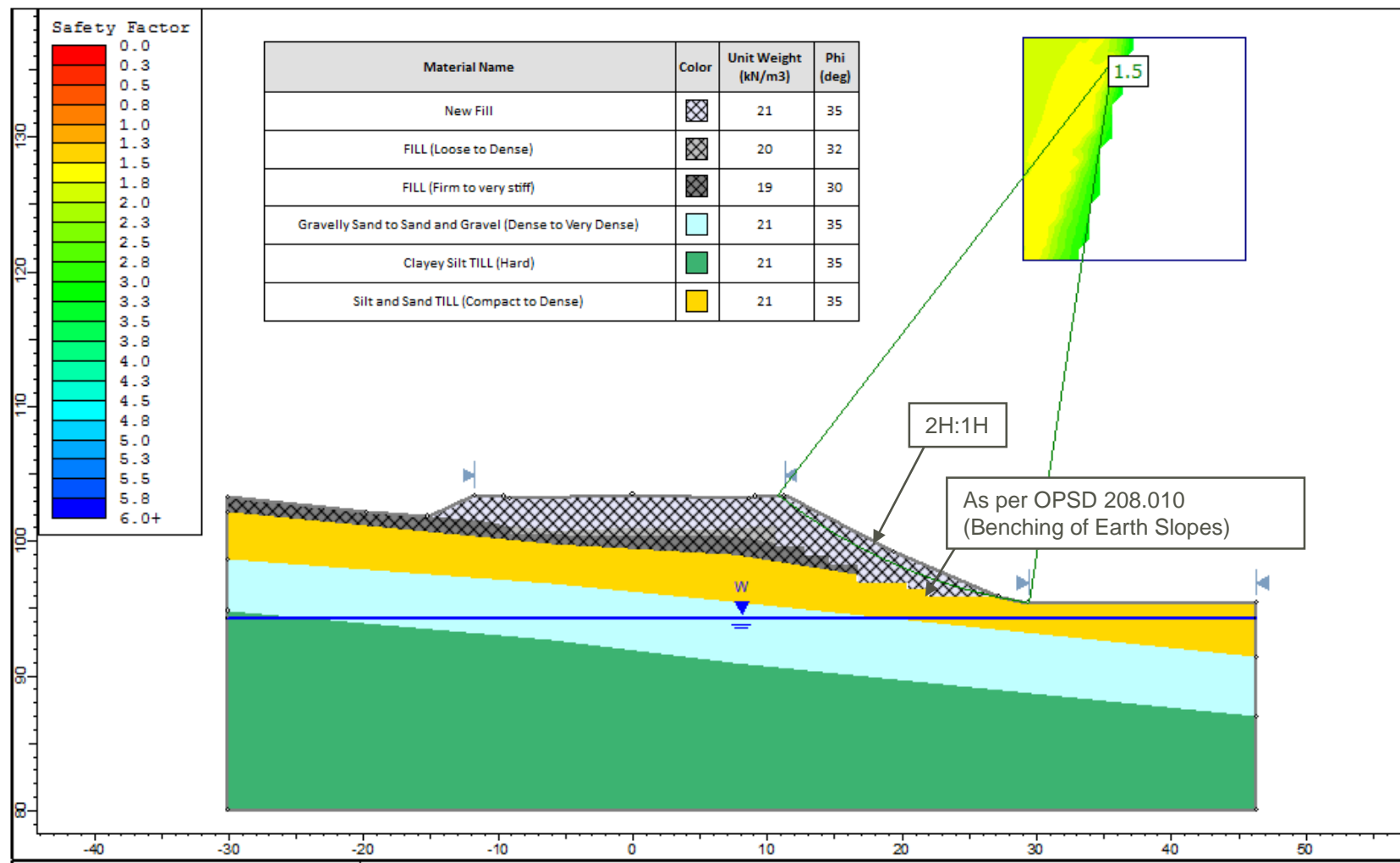


Global Stability Analysis

North Approach Embankment – West Slope

Long-term (Permanent) Analysis

Figure D2

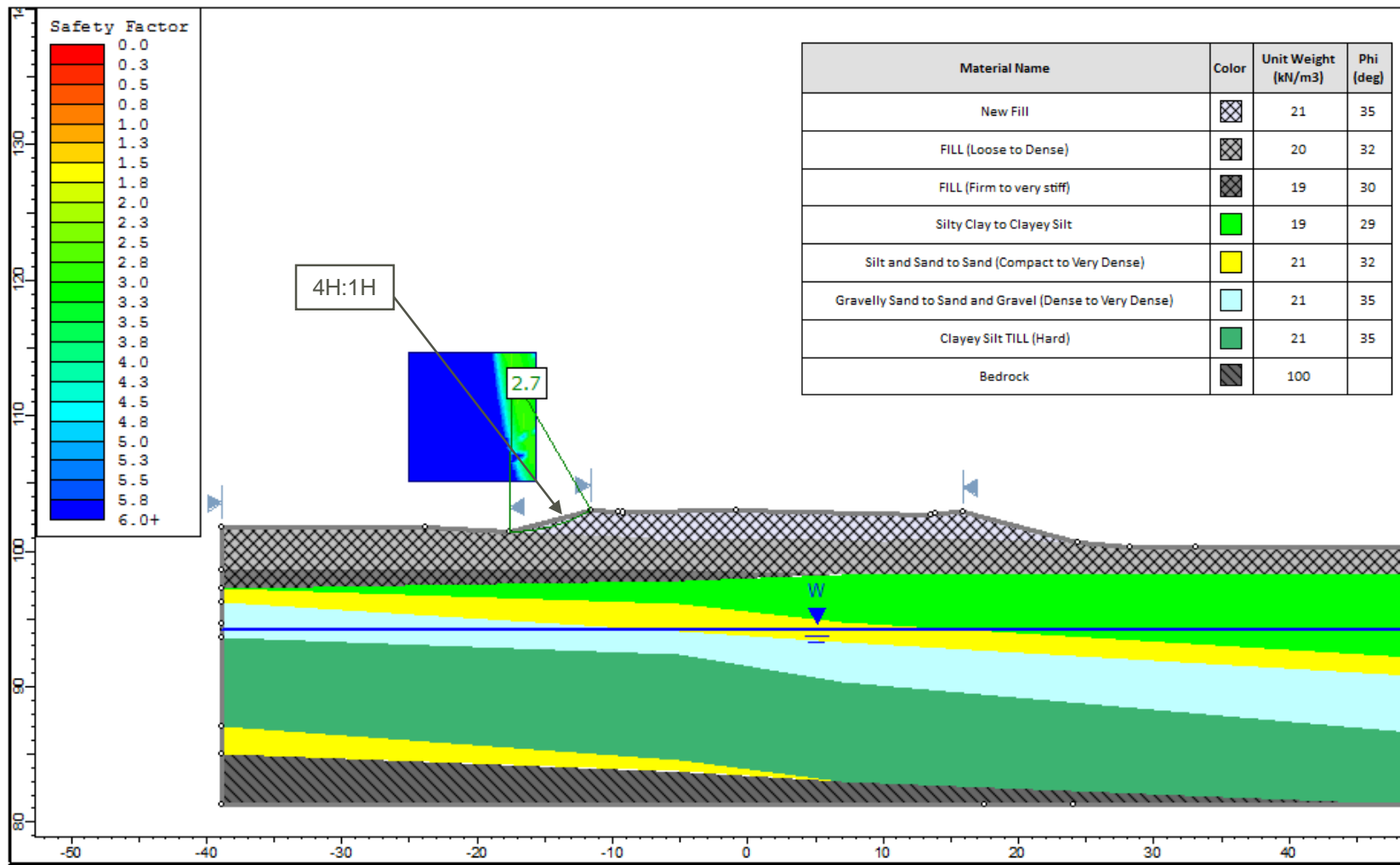


Global Stability Analysis

South Approach Embankment – East Slope

Long-term (Permanent) Analysis

Figure D3

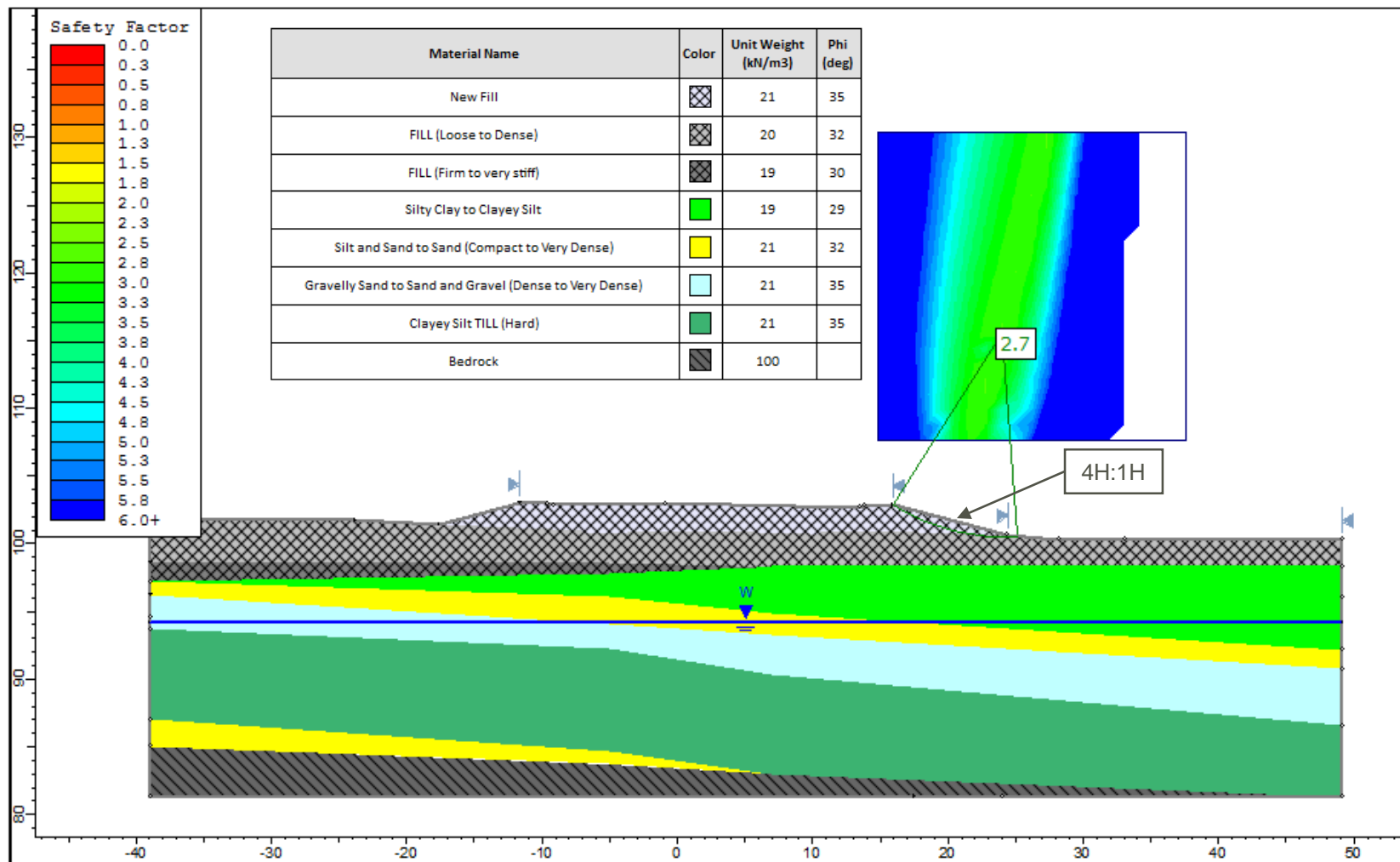


Global Stability Analysis

South Approach Embankment – West Slope

Long-term (Permanent) Analysis

Figure D4

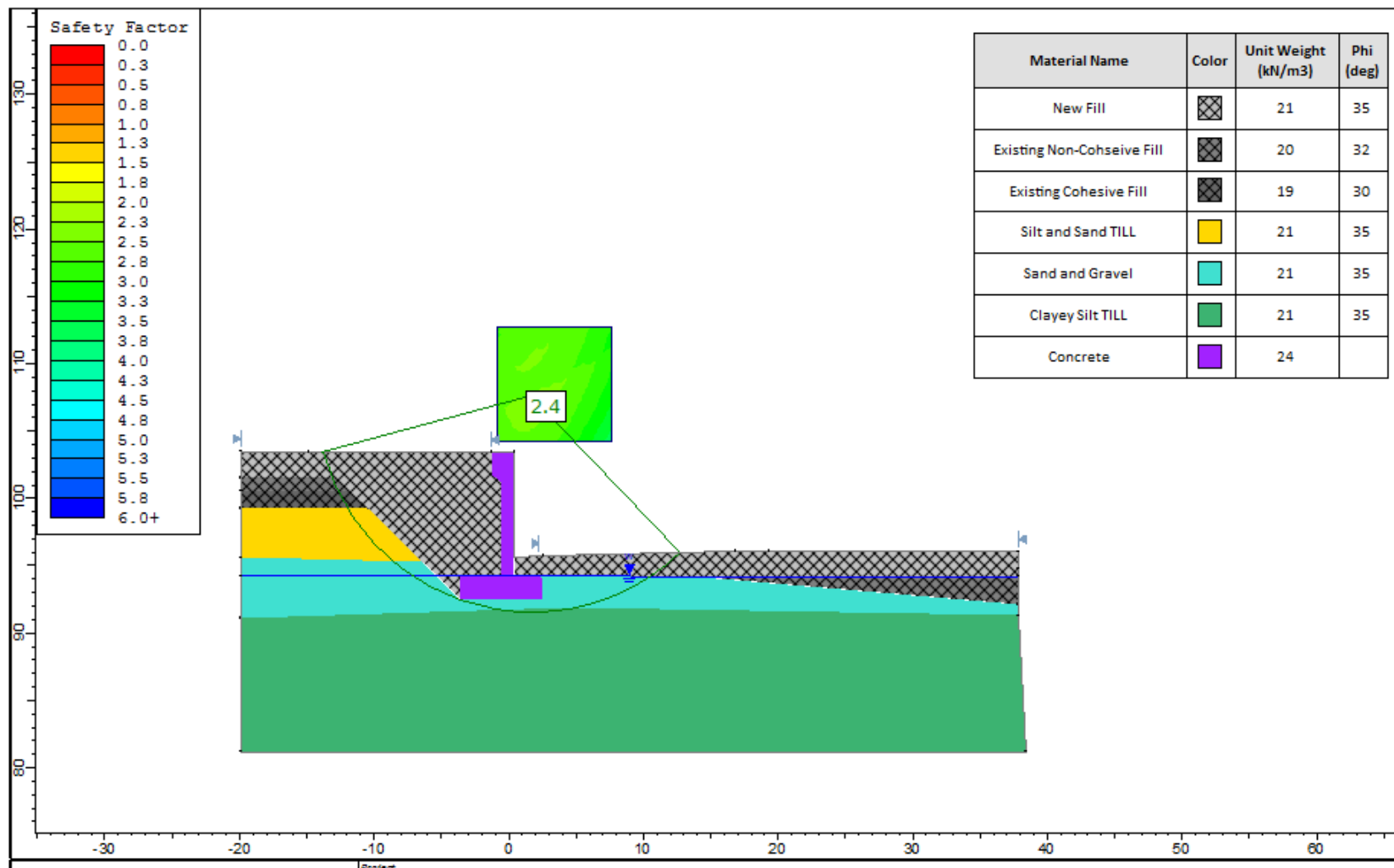


Global Stability Analysis

North Abutment - Foreslope

Long-term (Permanent) Analysis

Figure D5

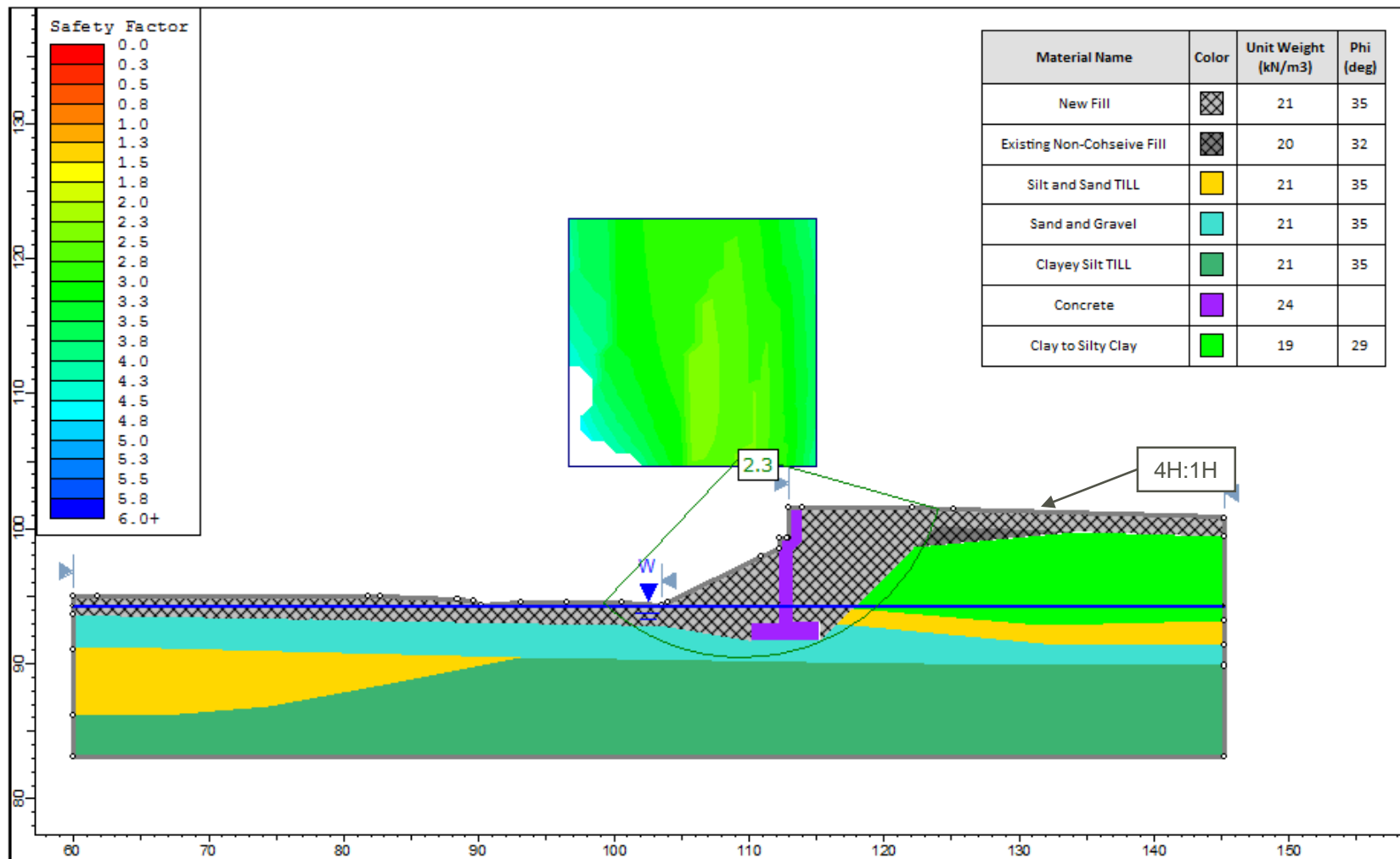


Global Stability Analysis

South Abutment - Foreslope

Long-term (Permanent) Analysis

Figure D6



APPENDIX E

Non-Standard Special Provisions

DEWATERING STRUCTURE EXCAVATIONS - Item No.

Special Provision No. FOUN0003

March 8, 2018

Amendment to OPSS 902, November 2010

OPSS 902, November 2010, Construction Specification for Excavating and Backfilling - Structures is amended as follows:

902.02 REFERENCES

Section 902.02 of OPSS 902 is amended by the addition of the following:

Ontario Provincial Standard Specifications, Construction

OPSS 517 Dewatering
OPSS 805 Temporary Erosion and Sediment Control Measures

902.03 DEFINITIONS

Section 903.03 of OPSS 902 is amended by the addition of the following:

Automatic Transfer Switch means as defined in OPSS 517.

Cofferdam means as defined in OPSS 539.

Cut-Off Wall means as defined in OPSS 517.

Design Storm Return Period means as defined in OPSS 517.

Dewatering System means as defined in OPSS 517.

Groundwater Control System means as defined in OPSS 517.

Plug means as defined in OPSS 517.

Sediment means as defined in OPSS 517.

Sediment Control Measure means as defined in OPSS 517.

Temporary Flow Passage System means as defined in OPSS 517.

Unwatering means as defined in OPSS 517.

Vegetated Discharge Area means as defined in OPSS 517.

Waterbody means as defined in OPSS 517.

Watercourse means as defined in OPSS 517.

902.04 DESIGN AND SUBMISSION REQUIREMENTS

902.04.01 Design Requirements

902.04.01.01 Dewatering

Clause 902.04.01.01 of OPSS 902 is deleted in its entirety and replaced with the following:

A dewatering system shall be designed to control water and the flow of water into the excavation, prevent disturbance of the foundation, permit the placing of concrete in the dry, and complete the excavating and backfilling for structures work.

When the system includes temporary flow passage system, the system shall be designed, as a minimum, for a 25-year design storm return period, and groundwater discharge. A longer return period shall be used when determined appropriate for the work.

The dewatering system shall be according to the design requirements specified in OPSS 517.

902.04.02 Submission Requirements

Subsection 902.04.02 of OPSS 902 is deleted in its entirety and replaced with the following:

902.04.02.01 Working Drawings

Working Drawings for the dewatering system shall be according to OPSS 517.

902.04.02.02 Preconstruction Survey

When a groundwater control system by wells or a well point system will be used, a condition survey of property and structures that may be affected by the work shall be carried out. The condition survey shall include the location and condition of adjacent properties, buildings, underground structures, water wells, Utilities, and structures, within a distance of 100 metres from the groundwater control system. In addition, all water wells used as a supply of drinking water and located within this distance shall be tested for compliance with Ontario Drinking Water Quality Standards.

Water wells within the preconstruction survey distance can be located using the website <https://www.ontario.ca/environment-and-energy/map-well-records> or its successor site.

Copies of the condition survey and water quality test results shall be submitted to the Contract Administrator prior to the operation of the groundwater control system.

902.04.02.03 Milestone Inspections

Clause 902.04.02.03 of OPSS 902 is deleted in its entirety.

902.07 CONSTRUCTION

Subsection 902.07.04 of OPSS 902 is deleted in its entirety and replaced with the following:

902.07.04 Dewatering Structure Excavation

902.07.04.01 General

The dewatering systems shall be constructed and operated according to the Working Drawings.

Activation and deactivation of a temporary flow passage system, if applicable, shall be according to OPSS 517.

The dewatering system shall be continuously operational to control buoyancy forces until such forces can be resisted by backfill and structure self-weight, to keep excavations stable, to avoid erosion impacts from the release of accumulated water, and to keep the work area in the condition required to complete the associated work as specified in the Contract Documents.

When a temporary flow passage system is to remain operational through a seasonal shutdown period, the Contractor shall be responsible for any maintenance or repair costs due to the system during the seasonal shutdown period.

Temporary erosion and sediment control measures, including controlling the discharge of water, shall be according to OPSS 805. Measures not specified in OPSS 805 shall be according to the Working Drawings. Temporary erosion and sediment control measures and cover material to protect exposed soils, as required by the Working Drawings, shall be installed as soon as is practical.

Stranded fish shall be managed as specified in the Contract Documents.

Unwatering shall be carried out as necessary.

Water suspected of being contaminated as indicated by visual or olfactory observations shall be reported to the Contract Administrator.

Dewatering and temporary flow passage systems shall be discontinued in a manner that does not disturb any structure, pipeline, or flow channel. Operation of the dewatering system shall be shut down according to the procedures specified in the Working Drawings, where applicable.

902.07.04.02 Discharge of Water

The discharge of water shall be according to OPSS 517.

902.07.04.03 Monitoring

Monitoring shall be according to OPSS 517.

902.07.04.04 System Amendments

Amendments to stop any displacement, damage, soil loss or erosion due to the operation of the dewatering system shall be according to OPSS 517.

902.07.04.05 Removal

Removal of dewatering system and temporary flow passage system components shall be according to OPSS 517.

NOTES TO DESIGNER:

Designer Fill-Ins

- * Fill in the design storm return period according to MTO Drainage Design Standard TW-1.
- ** Fill in the preconstruction survey distance as recommended by the foundation engineer.

WARRANT: Include with this standard tender item only on the recommendation of a foundation engineer.

CUSTODIAN: Tony Sangiuliano, MERO - Foundation Group.

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 deposits. Considerations of the presence of these obstructions must be made in the selection of appropriate equipment and procedures for excavations, pile installation, and installation of drilled shafts (caissons).

VIBRATION MONITORING – Item No.

Special Provision

Vibration monitoring during pile driving (as may be applicable) and/or installation of temporary protection systems at the Simcoe Street Underpass and along Lviv Boulevard, and during compaction operations in proximity of heritage building at 597 Albert Street, will be completed by the Contractor Administrator as described herein. Prior to commencement of vibration monitoring, the Contractor shall submit the proposed pile driving (if required) and temporary protection system installation methods to the Contract Administrator for review to confirm that the proposed installation methods meets the requirements outlined herein.

The approximate locations of the vibration monitoring equipment are provided in Table 1. The actual locations of the vibration monitoring equipment may be field-adjusted when agreed in writing between the Contract Administrator and Contractor, based on access and Contractor's working areas.

Table 1: Approximate Locations of Vibration Monitoring Equipment

Purpose	Monitoring Station	Latitude	Longitude
Monitoring during construction of Storm Sewer along Lviv Boulevard	#1	43.882448°	-78.854085°
	#2	43.882412°	-78.854288°
	#3	43.882269°	-78.854833°
	#4	43.881956°	-78.856053°
	#5	Mobile location adjacent to the section of storm sewer under construction	
Monitoring during construction of Simcoe Street Underpass	#4	43.881956°	-78.856053°
	#6	43.881164°	-78.855964°

The Contractor shall adequately protect the monitoring equipment such that each instrument is accessible, is not damaged during construction, and is secured against vandalism and/or theft. The Contractor shall be responsible for all costs associated with the repair and/or replacement of damaged or missing vibration monitoring equipment.

The applicable vibration limits at the monitoring stations are provided in Table 2.

Table 2: Applicable Vibration Limits

Frequency of Vibration (Hz)	Peak Particle Velocity (mm/s)		
	For Buildings	For Heritage Buildings (597 Albert St)	For Utilities
Less than 4	8	4	50
4 to 10	15	7	50
More than 10	25	12	50

The Contractor will be notified by the Contractor Administrator should the peak particle velocity thresholds noted above be exceeded at any one of the monitoring stations. Once notification has been received, the

Contractor shall suspend and investigate construction activities producing excessive vibration. The Contractor is to submit to the Contractor Administrator a mitigation strategy detailing altered procedures and/or equipment so that vibrations remain within acceptable levels. The mitigation strategy shall only be implemented upon approval by the Contract Administrator.

PRE-CONSTRUCTION AND POST-CONSTRUCTION CONDITION SURVEY – Item No.

Special Provision

TABLE OF CONTENTS

1.0	SCOPE
2.0	REFERENCES - Not Used
3.0	DEFINITIONS
4.0	DESIGN AND SUBMISSION REQUIREMENTS
5.0	MATERIALS - Not Used
6.0	EQUIPMENT - Not Used
7.0	CONSTRUCTION
8.0	QUALITY ASSURANCE - Not Used
9.0	MEASUREMENT FOR PAYMENT - Not Used
10.0	BASIS OF PAYMENT

1.0 SCOPE

This special provision describes requirements for Pre- and Post-Condition Survey associated with construction of the following components of the Contract:

- Simcoe Street Underpass
- Storm sewer replacement along Lviv Boulevard

3.0 DEFINITIONS

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

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

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

4.0 DESIGN AND SUBMISSION REQUIREMENTS

4.1 Submission Requirements

The Contractor shall submit details of the Pre- and Post-Construction Condition Survey plan to the Contract Administrator for information purposes. The submittals shall, at a minimum, contain the following specific information:

- a) Qualifications of condition survey specialist.
- b) Details of equipment and methods used by the Contractor to perform the work that may cause undue vibration.
- c) Details of equipment and methods to be used for the Pre- and Post-Construction Condition Surveys.

7.0 CONSTRUCTION

7.1 Pre- and Post-Construction Condition Surveys

A Pre-Construction Condition Survey and Post-Construction Condition Survey shall be prepared for all buildings, structures, and facilities within about 50 m of the proposed construction activities for the Simcoe Street Underpass and the storm sewer along Lviv Boulevard. The surveys shall be prepared for all buildings, structures, and facilities at the following addresses:

- a) 589 and 597 Simcoe Street
- b) 589, 597, 627, and 630, 632 to 638 Albert Street
- c) 8, 34, 38, 42, and 68 Lviv Boulevard
- d) 2, 44, 62, and 72 Bloor Street East
- e) 592, 594, and 598 Front Street

7.1.1 Pre-Construction Condition Surveys

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

The Pre-Construction Condition Survey at each structure/facility identified above shall be completed a minimum of two (2) weeks prior to commencement of vibration-inducing construction activity. Only one (1) Pre-Construction Condition Survey per structure or facility is required to be carried out in advance of construction activities that may cause undue vibration, unless more than six (6) months will elapse between these operations, in which case an interim inspection will be required.

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

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

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

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

7.1.2 Post-Construction Condition Surveys

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

A Post-Construction Condition Survey at each structure/facility identified above is required within two (2) months of completion of vibration-inducing construction activity.

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

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

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

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

7.2 Records

The Contractor shall submit details of the Pre- and Post-Construction Condition Survey to the Contract Administrator as follows:

- a) An interim report containing all relevant data including the Pre-Construction Condition Survey prior to the start of vibration inducing construction activity.
- b) A final report containing all relevant data including vibration monitoring and Pre- and Post-Construction Condition Surveys within two (2) weeks upon completing the Post-Condition Survey.

10.0 BASIS OF PAYMENT

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



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