

## Foundation Investigation and Design Report

*Replacement of Albert Street Underpass (Site No. 22-177)*

*Highway 401 Rehabilitation and Widening, Oshawa, Ontario*

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

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NSSP	Piezometer Decommissioning



# PART A

FOUNDATION INVESTIGATION REPORT  
REPLACEMENT OF ALBERT STREET UNDERPASS (SITE NO. 22-177)  
HIGHWAY 401 REHABILITATION AND WIDENING, 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 underpasses at Simcoe Street, Albert Street and Bennett Street and the rehabilitation of Oshawa Creek Bridge and three mainline culverts, in the Regional Municipality of Durham, Ontario. This report addresses the proposed replacement of the existing Albert Street Underpass (MTO Structure Site No. 22-177) and approach embankments at the location shown on the Key Plan on Drawing 1.

The purpose of this investigation is to establish the subsurface soil and bedrock conditions at the proposed underpass replacement location, including the associated approach embankments, by borehole drilling and laboratory testing on selected soil samples.

Golder's professional services for this assignment address only the geotechnical (physical) aspects of the subsurface conditions at this site. The geo-environmental (chemical) aspects, including the consequences of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources, or removal options of existing site materials, are outside the terms of reference for this report.

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

## 2.0 SITE DESCRIPTION

The Albert Street underpass carries northbound and southbound traffic over the eastbound and westbound lanes of Highway 401 and is in the City of Oshawa, in the Regional Municipality of Durham, Ontario. The existing Albert Street underpass is a two-span structure with a total span length of about 30 m. Based on visual observations at the time of the foundation exploration field work, it is considered that the existing abutments (hence foundations) and approach embankments are/have performed satisfactorily with no sign of instability or settlement issues. The underpass is founded on spread footings at about Elevation 93.5 m.

The natural ground surface at the site is between about Elevations 100 m and 101 m. Highway 401 was constructed in a cut with the highway grade at about Elevation 96 m. The road grade of Albert Street at the site ranges from about Elevation 102 m to 103 m, rising northward at the structure site. A church and heritage centre are located in the northeast and northwest quadrants of the structure site, respectively. Residential areas are present in the southeast and southwest quadrants of the structure 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 A1 and A2) were advanced at the northeast and southwest quadrants of the existing underpass, as shown on Drawing 1. The results of the investigation are presented in Golder's report titled *"Preliminary Foundation Investigation Report, Albert Street Underpass, Structure Site No. 22-177, Highway 401 Improvements from Brock Road to Courtice Road, Regional Municipality of Durham, W.O. 10-20011"*, dated May 19, 2017, Project No. 11-1184-0143 (GEOCRE 30M14-452).



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 below.

Borehole Number	MTM NAD83 Northing (m) (Latitude, °)	MTM NAD83 Easting (m) (Longitude, °)	Ground Surface Elevation (m)	Borehole Depth (m)
A1	4,860,447.6 (43.882190)	356,680.2 (-78.854380)	102.1	9.5
A2	4,860,377.6 (43.881560)	356,679.4 (-78.854400)	102.0	8.0

### 3.2 Current Investigation

The field work for the current investigation was carried out between November 6, 2017 and December 19, 2018 during which time a total of ten boreholes (designated as Boreholes AS-1, AS-2, AS-4, AS-5, AS-6, AS-6B, AS-7, AS-8, AS-9, AS-9B) were advanced in the vicinity of the proposed foundation elements and approach embankments as shown on Drawing 1. One or two boreholes were drilled in the vicinity of each proposed foundation element and one borehole was advanced at each of the north and south approach embankments. Except for Boreholes AS-6 and AS-9, all boreholes were advanced to depths ranging from 10.8 m to 21.9 m and included 3 m of bedrock coring at Boreholes AS-7 and AS-9B. Borehole AS-6 was terminated at a depth of 2.1 m due to an obstruction and Borehole AS-9 was terminated at a depth of 2.1 m, due to the presence of a storm sewer.

The investigation was carried out using truck-mounted CME-55 and CME-75 drilling rigs, supplied and operated by Geo-Environmental Drilling Inc. of Milton, Ontario and Pontil Drilling of Mount Albert, Ontario. The boreholes were advanced through the overburden using 150 mm and 152 mm diameter solid stem augers and 210 mm and 216 mm outside diameter (O.D.) hollow-stem augers. Soil samples were generally obtained at 0.75 m and 1.5 m intervals of depth, using a 50 mm O.D. split-spoon sampler driven by an automatic hammer in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586)<sup>1</sup>. 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. Core samples of the bedrock were obtained using an 'HQ' size rock core barrel and coring techniques in Borehole AS-7 and AS-9B.

The groundwater conditions were noted in the boreholes during drilling and generally upon removal of the hollow stem augers at completion of drilling. A standpipe piezometer was installed in Borehole AS-2 to allow for monitoring of the groundwater level. The remaining boreholes were backfilled with bentonite in accordance with O.Reg. 903 (Wells) as amended, 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

<sup>1</sup> ASTM D1586 – Standard Test Method for Standard Penetration Tests and Split Barrel Sampling of the soil.



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. 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 (Zone 10) CSRS CBNv6-2010.0 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 below.

Borehole Number	Location	MTM NAD83 Northing (Latitude, °)	MTM NAD83 Easting (Longitude, °)	Ground Surface Elevation (m)	Borehole Depth (m)
AS-1	South Embankment	4,860,362.2 (43.881420)	356,699.0 (-78.854150)	101.8	10.9
AS-2	South Abutment	4,860,377.8 (43.881557)	356,697.2 (-78.854175)	101.6	14.2
AS-4	South Pier	4,860,397.1 (43.881730)	356,692.1 (-78.854240)	101.3	12.7
AS-5		4,860,392.2 (43.881690)	356,689.6 (-78.854270)	101.6	18.9
AS-6	North Pier	4,860,410.0 (43.881840)	356,669.3 (-78.854510)	96.0	2.1
AS-6B		4,860,408.0 (43.881830)	356,668.9 (-78.854520)	96.0	12.2
AS-7	North Abutment	4,860,444.2 (43.882160)	356,672.9 (-78.854470)	102.1	21.9
AS-8	North Embankment	4,860,464.3 (43.882370)	356,666.4 (-78.854560)	102.6	10.8
AS-9	North Pier	4,860,416.0 (43.881900)	356,693.3 (-78.854219)	96.0	2.1
AS-9B		4,860,414.6 (43.881880)	356,693.8 (-78.854210)	96.0	15.3



## 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)<sup>2</sup> and *Urban Geology of Canadian Cities* (Brennand, 1998)<sup>3</sup>. 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. Shale bedrock of the Whitby Formation underlies the overburden deposits in this area.

### 4.2 Subsurface Conditions

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

The results of in situ field tests (i.e., SPT “N”-values) as presented on the borehole records and in Section 4.2 are uncorrected. The boundaries between the strata on the borehole records have been inferred from drilling observations and non-continuous sampling. Therefore, these boundaries represent transitions between soil types rather than exact planes of geological change. The interpreted stratigraphic profile and cross-sections 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 consist of asphalt and fill or surficial topsoil in places, underlain by an upper deposit of silt and sand to gravelly silty sand till which is underlain by an interlayered deposit of silt to silt and sand to sand and gravel. The silt to sand and gravel deposit is in turn underlain by a lower deposit of silt and sand to sand till and/or a clayey silt with sand till, overlying 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 500 mm thick layer of topsoil was encountered at ground surface in Boreholes A1 and A2.

#### 4.2.2 Asphalt Pavement

An approximately 90 mm to 170 mm thick layer of asphalt pavement was encountered at ground surface in Boreholes AS-1, AS-2, and AS-4 to AS-9, including AS-6B and AS-9B.

#### 4.2.3 Concrete

An approximately 255 mm thick layer of concrete was encountered underlying the asphalt pavement in Borehole AS-6B and underlying a 75 mm thick layer of sand and gravel fill in Boreholes AS-9 and AS-9B.

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<sup>2</sup> 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.)

<sup>3</sup> 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.



#### 4.2.4 Sand to Gravelly Sand to Sand and Gravel (Fill)

A 0.5 m to 3.9 m thick layer of granular fill was encountered underlying the asphalt at all borehole locations, except at Boreholes A1 and A2. The granular fill, which consists of sand to gravelly sand to sand and gravel, some silt, extends to depths of 0.6 m to 4.0 m below ground surface (Elevations 102.0 m to 93.7 m). In Borehole AS-2, a 1.3 m thick layer of silty sand fill was encountered underlying the gravelly sand fill, extending to a depth of 2.1 m below ground surface (Elevation 99.5 m). Borehole AS-9 penetrated an MTO storm sewer at a depth of 1.5 m below ground surface (Elevation 94.5 m) and was terminated at 2.1 m depth within the void space of the sewer pipe; the storm sewer was repaired with a rubber seal at the obvert and the borehole was backfilled with a lower layer of concrete and sand to near ground surface.

The SPT “N”-values measured within the non-cohesive fill range from 3 blows to 38 blows per 0.3 m of penetration, indicating a very loose to dense state of compactness. An SPT “N”-value of 50 blows for 0.05 m of penetration was measured on concrete fragments within the fill in Borehole AS-4 and therefore is not representative of the overall compactness of the fill.

The water content measured on samples of the non-cohesive fill ranges from about 2 per cent to 11 per cent.

Grain size distribution testing was carried out on one sample of the non-cohesive fill and the result is shown on Figure B1 in Appendix B.

#### 4.2.5 Silty Clay to Clayey Silt with Sand (Fill)

A 0.4 m to 1.5 m thick layer of cohesive fill was encountered underlying the topsoil/non-cohesive fill in Boreholes A1, A2, AS-1, AS-5, AS-7, and AS-8. The cohesive fill was encountered at depths of 0.5 m to 0.9 m below ground surface (Elevations 102.0 m to 100.7 m) and extends to depths of 0.9 m to 2.1 m below ground surface (Elevations 101.2 m to 99.7 m). The cohesive fill consists of silty clay to clayey silt with sand, and trace gravel. In Borehole A1 and A2, trace amounts of organics were encountered within the cohesive fill.

The SPT “N”-values measured within the cohesive fill range from 6 blows to 34 blows per 0.3 m of penetration, suggesting a firm to hard consistency.

The water content measured on samples of the cohesive fill ranges from about 10 per cent to 25 per cent.

Grain size distribution testing was carried out on two samples of the cohesive fill and the results are shown on Figure B2 in Appendix B.

Atterberg limit testing was carried out on two samples of the cohesive fill and the results are presented on Figure B3 in Appendix B. The Atterberg limit tests measured liquid limits of 15 per cent and 22 per cent, plastic limits of 10 per cent and 11 per cent, and plasticity indices of 5 per cent and 11 per cent, indicating the cohesive fill is a clayey silt of low plasticity.

#### 4.2.6 Clayey Silt

A 0.5 m and 0.4 m thick pocket of clayey silt was encountered underlying the cohesive fill in Boreholes A1 and A2, respectively. The cohesive pockets were encountered at depths of 0.9 m and 1.4 m below ground surface (Elevations 101.2 m and 100.6 m) and extend to depths of 1.4 m and 1.8 m below ground surface (Elevations 100.7 m and 100.2 m) in the respective boreholes. The cohesive material consists of clayey silt, trace sand and trace gravel.



The SPT “N”-values measured within the clayey silt pockets are 18 blows and 35 blows per 0.3 m of penetration, suggesting a very stiff and hard consistency, respectively.

The water content measured on two samples of the cohesive pockets are about 14 per cent and 24 per cent.

Atterberg limit testing was carried out on one sample of the cohesive pocket and measured a liquid limit of 25 per cent, a plastic limit of 13 per cent, and a plasticity index of 12 per cent. The Atterberg limit test result is presented on Figure B4 in Appendix B and indicates the soil is a clayey silt of low plasticity.

#### **4.2.7 Silt and Sand to Sand (Till) – Upper Deposit**

A 1.9 m to 4.1 m thick upper non-cohesive glacial deposit was encountered underlying the fill/clayey silt deposit in all boreholes, except at Boreholes AS-4, AS-6/AS-6B, and AS-9/AS-9B. The upper non-cohesive till deposit was encountered at depths of 1.4 m to 2.1 m below ground surface (Elevations 100.7 m to 99.5 m) and extended to depths of 4.0 m to 5.6 m below ground surface (Elevations 98.6 m to 96.1 m). The deposit consisted of silt and sand to silty sand, trace gravel to gravelly, trace to some clay and containing some cobbles. Although not encountered in these boreholes, boulders are commonly present within glacially derived soils and should be expected within this deposit.

The SPT “N”-values measured within the upper non-cohesive till range from 26 blows to 135 blows per 0.3 m of penetration, with one measurement of 100 blows for 0.15 m of penetration where cobbles were encountered, indicating a compact to very state dense of compactness.

The water content measured on samples of the deposit ranges from about 5 per cent to 8 per cent.

Grain size distribution testing was carried out on six samples of the upper non-cohesive till deposit and the results are presented on Figure B5 in Appendix B.

#### **4.2.8 Silt to Sand and Gravel**

A 3 m to 9.2 m thick interlayered deposit of silt to sand and gravel was encountered underlying the fill and upper non-cohesive till at all borehole locations, except at Borehole AS-6/AS-6B. The silt to sand and gravel deposit was encountered at depths ranging from 2.3 m to 5.6 m below ground surface (Elevations 98.6 m to 93.7 m) and extends to depths ranging from 5.5 m to 13.2 m below ground surface (Elevations 94.5 m to 87.5 m). The deposit interlayers consist of silt, sandy silt, silt and sand, sand, and sand and gravel.

It should be noted that flowing sand condition was encountered while advancing through the non-cohesive deposit in Boreholes AS-4, AS-5 and AS-9B.

The SPT “N”-values measured within the silt to sand and gravel deposit range from 31 blows to 141 blows per 0.3 m of penetration, with blow counts up to 100 blows per 0.05 m of penetration, indicating a dense to very dense state of compactness.

The water content measured on samples of the deposit ranges from about 2 per cent to 21 per cent.

Grain size distribution testing was carried out on eight samples of the silt to sand, and the results are presented on Figures B6A and B6B in Appendix B.

Atterberg limit testing was carried out on two samples of the deposit, and one sample measured a liquid limit of 14 per cent, a plastic limit of 12 per cent, and a plasticity index of 2 per cent; while the other sample was non-plastic.



The Atterberg limit test results, as presented on Figure B7 in Appendix B, indicate the silt portion of the deposit ranges from non-plastic to slightly plastic.

#### 4.2.9 Silt and Sand to Silty Sand (Till) – Lower Deposit

A 0.4 m to 2.7 m thick lower non-cohesive glacial till deposit was encountered underlying the silt to sand and gravel deposit in Boreholes A1, AS-7, and AS-8. The lower non-cohesive till deposit was encountered at depths ranging from 8.1 m to 9.1 m below ground surface (Elevations 94.5 m to 93.0 m) and extends to termination depths between 9.5 m and 10.8 m below ground surface (Elevations 92.6 m to 91.8 m). The deposit consists of sandy silt to silt and sand, trace to some gravel, and trace to some clay. Although not encountered in these boreholes, cobbles and boulders are commonly present within glacially derived soils and should be expected within this deposit.

The SPT “N”-values measured within the lower non-cohesive till range from 118 blows per 0.3 m of penetration to 100 blows per 0.08 m of penetration, indicating a very dense state of compactness.

The water content measured on two samples of the deposit are about 7 per cent and 8 per cent.

Grain size distribution testing was carried out on one sample of the lower non-cohesive till and the result is presented on Figure B8 in Appendix B.

#### 4.2.10 Silt

A 0.7 m thick deposit of silt was encountered interlayered between the silt and sand till and the lower clayey silt with sand till deposits in Borehole AS-7. The silt deposit was encountered at a depth of 9.8 m below ground surface (Elevation 92.3 m) and extends to a depth of 10.5 m below ground surface (Elevation 91.6 m).

An SPT “N”-value measured within the silt interlayer is 100 blows for 0.13 m of penetration, indicating a very dense state of compactness.

#### 4.2.11 Clayey Silt with Sand (Till)

A cohesive till deposit was encountered underlying the non-cohesive fill, silt to sand and gravel deposit, and lower non-cohesive till deposit in Boreholes AS-1, AS-2, AS-4, AS-5, AS-6B, AS-7, and AS-9B. The cohesive till deposit was encountered at depths ranging from 2.3 m to 13.2 m below ground surface (Elevations 93.7 m to 87.5 m) and extends to depths between 9.1 m and 17.7 m below ground surface where fully penetrated, and in some boreholes to the termination depths of 12.0 m to 14.2 m below ground surface (Elevation 93.0 m to 83.8 m). The thickness of the deposit ranges between 0.5 m and 9.9 m consists of clayey silt with sand, trace to some gravel. Although not encountered in these boreholes, cobbles and boulders are commonly present within glacially derived soils and should be expected within this deposit, as inferred from auger grinding in Borehole AS-7. In addition, strong hydrocarbon odours were noted from a depth of 10.4 m below ground surface to the borehole termination depth of 12.2 m below ground surface (Elevations 85.6 m and 83.8 m) when advancing Boreholes AS-6B.

The SPT “N”-values measured within the cohesive till deposit range from 22 blows to 142 blows per 0.3 m of penetration, with one measurement of 100 blows for 0.07 m of penetration, suggesting a very stiff to hard consistency.

The water content measured on samples of the deposit range from about 5 per cent to 17 per cent.

Grain size distribution testing was carried out on ten samples of the clayey silt with sand till and the results are presented on Figures B9A and B9B in Appendix B.



Atterberg limit testing was carried out on nine samples of the deposit and measured liquid limits ranging from 14 to 21 per cent, plastic limits ranging from 9 per cent to 10 per cent, and plasticity indices ranging from 5 per cent to 9 per cent. The Atterberg limit test results are presented on Figure B10 and indicate the cohesive till is a clayey silt of low plasticity.

#### 4.2.12 Bedrock

Bedrock was encountered in Boreholes AS-5, AS-7, and AS-9B at depths of 17.7 m, 17.7 m and 11.9 m below ground surface (Elevations 83.9 m, 84.4 m, and 84.1 m), respectively. Borehole AS-5 was advanced 1.2 m into the bedrock to a depth of 18.9 m (Elevation 82.7 m) by augering and split spoon sampling, while the bedrock was cored from depths of 18.9 m to 21.9 m below ground surface (Elevations 83.2 m to 80.2 m) in Borehole AS-7 and from depths of 12.1 m to 15.3 m below ground surface (Elevations 83.9 m to 80.7 m) in Borehole AS-9B after penetration into the bedrock by augering and split spoon sampling for depths of 1.2 m and 0.2 m, respectively.

Based on the cored bedrock samples, the bedrock is shale of the Whitby Formation. The bedrock core samples are described as fresh, thinly to medium bedded, black, fine grained, faintly porous shale, interbedded with fossiliferous limestone. Details of the rock cores are presented on the Record of Drillhole sheets in Appendix A. The degree of weathering of the bedrock samples (i.e., fresh – W1), and the strength classification of the intact rock mass based on field identification (i.e., medium strong – R3) are described in accordance with the International Society for Rock Mechanics (ISRM<sup>4</sup>) standard classification system.

The Rock Quality Designation (RQD) measured on the core samples generally ranges from about 61 per cent to 100 per cent, indicating a rock mass of fair to excellent quality, as per Table 3.10 of CFEM (2006)<sup>5</sup>. The measured Total Core Recovery (TCR) and the Solid Core Recovery (SCR) of the rock samples from the two cored boreholes are about 100 per cent.

### 4.3 Groundwater Conditions

Details of the water levels observed in the boreholes upon completion of drilling are summarized on the borehole records. The water level in the open boreholes advanced outside the Highway 401 limits was measured at depths ranging from 5.8 m to 10.1 m below ground surface, corresponding to between about Elevations 96.3 m and 92.5 m. The water level in the open boreholes advanced from the Highway 401 grade was measured at depths of 0.9 m and 2.7 m below ground surface, corresponding to about Elevations 95.1 m and 93.3 m, respectively.

A standpipe piezometer was installed in one borehole to monitor the groundwater level at the site, as shown on the borehole record and summarized below. It should be noted that the groundwater level is subject to seasonal fluctuations and precipitation events and should be expected to be higher during wet periods of the year.

<sup>4</sup> 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.

<sup>5</sup> Canadian Geotechnical Society, 2006. Canadian Foundation Engineering Manual (CFEM), 4<sup>th</sup> Edition. The Canadian Geotechnical Society, BiTech Published Ltd., British Columbia.



Borehole Location	Screened Stratigraphy	Water Level		Date of Measurement
		Depth (m)	Elevation (m)	
AS-2	Silt and Sand	7.9	93.7	August 13, 2018
		6.7	94.9	February 8, 2019

## 4.4 Analytical Testing

### 4.4.1 Corrosivity Testing

Two 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 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 (µmho/cm)	Resistivity (ohm-cm)
AS-5	6	4.7 (96.9)	Silt and Sand Till	300	33	8.06	578	1700
AS-7	8	7.9 (94.2)	Sand and Gravel	28	770	8.08	727	1400

### 4.4.2 Environmental Quality

Although geo-environmental (chemical) aspects of the subsurface conditions are outside the Terms of Reference for this report, it is noted that during advancement of Borehole AS-6B, strong hydrocarbon odours were noted within the clayey silt with sand till deposit from a depth of 10.4 m below ground surface to the borehole termination depth of 12.2 m below ground surface (Elevations 85.6 m and 83.8 m). In coordination with WSP, samples of the clayey silt with sand till deposit with strong hydrocarbon odours were submitted for analytical testing on behalf of WSP for their review and reporting purposes.

## 5.0 CLOSURE

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



## Signature Page

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# PART B

## FOUNDATION DESIGN REPORT

REPLACEMENT OF ALBERT STREET UNDERPASS (SITE NO. 22-177)  
HIGHWAY 401 REHABILITATION AND WIDENING, 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 the construction of a new Albert Street Underpass (Site No. 22-177). These recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the previous and current field investigations. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and carry out the design of the underpass foundations. The Foundation Design Report, discussion and recommendations are intended for the use of MTO and its designers and shall not be used or relied upon for any other purpose or by any other parties, including the construction contractor. Contractors must make their own interpretation based on the factual 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. Those requiring information on aspects of construction must make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

### 6.1 General

Based on the General Arrangement (GA) drawing provided by WSP, dated February 2018, the proposed Albert Street underpass will consist of a three-span structure with a total span length of about 72.8 m and a width of about 14.1 m, carrying two lanes of traffic. The new structure will allow for widening of Highway 401 from the existing six lanes to ten lanes and two ramp lanes and will include a nominal grade raise of about 0.1 m on Highway 401. A grade raise of up to about 0.4 m and 0.6 m is proposed at the north and south approach embankments on Albert Street, respectively. Further, given the proposed extent of the construction works to widen Highway 401, and to construct new foundations and raise the approach embankment on Albert Street, full closure of Albert Street will be implemented.

As detailed on the GA drawing, and based on discussions with WSP as the borehole investigation results became available and the structural design progressed, the proposed bridge foundations options include spread footings and drilled shafts (caissons). The existing and final road elevations, foundation types, and proposed foundation elevations, as shown from the GA drawing, are summarized below. Further discussion on the spread footings and other foundation types is provided in Section 6.2 to support the selection of the preferred foundation type for this replacement structure.

Foundation Element	Approximate Existing Grade Elevation (m)	Approximate Final Grade Elevation (m)	Foundation Type on GA Drawing
North Abutment	102.3	102.7 <sup>1</sup> / 95.3 <sup>2</sup>	Spread Footing
North Pier	95.8	95.8	Spread Footing
South Pier	102.0	95.7	Spread Footing
South Abutment	102.0	102.6 <sup>1</sup> / 95.8 <sup>2</sup>	Spread Footing

Notes:

1. Elevation at Albert Street.
2. Elevation at Highway 401 grade.

It is understood that full lane closure along the existing Albert Street underpass will be required for reconstruction of the structure. The existing abutments will be removed during Highway 401 overnight full closure.



## 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, considering the excavations required at both ends of the existing Albert Street Underpass to widen Highway 401. 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.

- **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 dense to very dense native soils. This option would require excavations up to depths of about 8.1 m and 8.7 m below the existing Albert Street grade at the abutments, that is to depths of about 2.1 m below the final Highway 401 grade. 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 near or below the groundwater level and therefore groundwater control (i.e., active dewatering) is anticipated to be required for construction in dry conditions. The use of shallow foundations does not allow for integral abutment design.
- **Driven steel H-piles or pipe piles:** Due to the requirement to found the pile caps at about 1.3 m depth below final ground surface for frost protection and presence of “100-blow” soil at shallow depths, the pile lengths at the abutment would be too short to develop sufficient axial and lateral geotechnical resistances prior to reaching effective refusal, and there would be potential for these relatively short piles to become misaligned and/or for pile damage due to driving in very dense/hard soils, although this could be mitigated by pre-drilling. While such a foundation option is technically feasible and adequately longer piles could allow for the abutments to be designed as integral abutments, this option is not preferred as it is not seen to offer any significant advantage over the spread/strip footings or drilled shaft (caisson) foundation options. As such, discussions on driven steel piles are not included herein.
- **Drilled shafts (caissons):** Drilled shafts (caissons) founded within the dense to very dense / hard native soils or bedrock are also considered feasible for the support of the abutments and piers. Drilled shafts require 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 to minimize the potential for fines from the non-cohesive silt to silt and sand to sand and gravel deposit flowing into the casing due to groundwater pressure, as was experienced during the drilling of Boreholes AS-4, AS-5, and AS-9B. 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. This option does not allow for integral abutment design.

Based on the above considerations, both shallow and deep foundation options are considered feasible from a foundation perspective for the support of the abutments and piers at this site. Typically, at the abutments, pile foundations would be preferred with a perched pile cap in a false abutment configuration to minimize excavation and groundwater control requirements, and to allow for integral abutment design. However, due to limited right-of-way and the need for retaining walls at the north and south limits of the right-of-way, perched abutments are not



feasible at this site. Drilled shafts (caissons) are deemed suitable at this site; however, it is understood that shallow foundations are feasible and preferred from a structural and traffic staging perspective. From a foundations perspective, shallow foundations are preferred at this site given the presence of “100-blow” soil at shallow depths below the Highway 401 grade and, excavation for the widened Highway 401 cut easily allows for additional excavation to about 2.1 m depth below the Highway 401 grade for construction of shallow foundations.

## 6.3 Design Considerations

### 6.3.1 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the *Canadian Highway Bridge Design Code* (CHBDC, 2014) 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 Part A of the report), in comparison to the degree of site understanding in Section 6.5 of *CHBDC, 2014*, the level of confidence for design is considered to be a “typical degree of site and prediction model understanding.” Accordingly, the appropriate corresponding ultimate limit state (ULS) and serviceability limit state (SLS) consequence factor,  $\Psi$ , and geotechnical resistance factors,  $\phi_{gu}$  and  $\phi_{gs}$ , from Tables 6.1 and 6.2 of the *CHBDC, 2014* 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 laboratory testing. The SPT “N”-values measured in the soil layers and the interpreted shear wave velocity of soils up to 30 m below founding level were used to define the seismic site classification in accordance with Table 4.1 of the 2014 CHBDC. Based on this methodology, it is considered that a Site Class C would be applicable for the design of the replacement structure.

The CHBDC 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. It is also understood from WSP that the proposed replacement structure has an Importance Category of “Other” bridge.

#### 6.3.2.2 Spectral Response Values and Seismic Performance Category

In accordance with Section 4.4.3.4 of the *CHBDC* (2014), the peak ground acceleration (PGA) values, peak ground velocity (PGV) and design spectral acceleration (S(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 (0.2) (g)	0.068	0.112	0.197
S (0.5) (g)	0.044	0.067	0.110
S (1.0) (g)	0.025	0.037	0.059
S (2.0) (g)	0.012	0.018	0.029
S (5.0) (g)	0.0026	0.0042	0.0071
S (10.0) (g)	0.0011	0.0018	0.0030



In accordance with Table 4.10 of the CHBDC (2014), the bridge structure (Importance Category of “Other”) with a fundamental period greater or less than 0.5 s, falls within Seismic Performance Category 1. Based on this Seismic Performance Category, it is understood that no seismic analysis is required.

## 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 dense to very dense / hard native soils. The highest founding elevations recommended for design of footings founded on the native soils are summarized below. All footings should be constructed in accordance with OPSS 902 (Excavating and Backfilling – Structures), as amended by SP 109S12 and FOUN0003 (Dewatering Structure Excavations), copies of which are provided in Appendix D for inclusion in the Contract Documents.

Foundation Element	Founding Elevation (m)	Founding Depth (m)	Founding Stratum
North Abutment	93.6	9.1 <sup>1</sup> /1.7 <sup>2</sup>	Very dense sand and gravel / very dense silt and sand till
North Pier	93.7	2.1 <sup>2</sup>	Hard clayey silt and sand till / dense to very dense sandy silt to sand
South Pier	94.4	1.4 <sup>2</sup>	Dense to very dense silt and sand to sand
South Abutment	94.3	8.3 <sup>1</sup> /1.5 <sup>2</sup>	Very dense silt and sand

Notes:

1. Depth of excavation relative to final grade at Simcoe Street.
2. Depth of excavation relative to final grade at Highway 401 grade.

### 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 m wide strip footings founded on the properly prepared soil at or below the elevations given above, or on compacted OPSS 1010 (Aggregates) Granular ‘A’ at the abutments following subexcavation to the elevations given above.

Foundation Element	Factored Ultimate Geotechnical Resistance (kPa)	Factored Serviceability Geotechnical Resistance (kPa)
North Abutment	600	300
North Pier	600	300
South Pier	600	300
South Abutment	600	300

### 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 6.10.5 of the CHBDC (2014). The following presents the coefficient of friction,  $\tan \phi$ , for the interface between the concrete footing and native soils at the proposed founding elevations, as interpreted from NAVFAC (1984):



Subgrade Material	$\tan \phi'$
Very dense sand and gravel / very dense sand and silt till	0.60
Dense to very dense silt and sand to sandy silt to sand and silt to sand	0.55
Hard clayey silt and sand till	0.45

#### 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. If adequate soil cover cannot be provided for the footing, rigid Styrofoam insulation could be installed to compensate for the lack of soil cover and provide protection from frost penetration.

#### 6.4.5 Key Challenges and Considerations

- Based on the groundwater level measurements in February 2019, excavations for the spread/strip footings at the abutments and piers will extend near or below the groundwater level. Therefore, groundwater control measures (active dewatering) will be required to achieve and maintain a dry and stable foundation subgrade. Additional dewatering details are provided in Section 6.9.3.
- 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” should be assessed in conjunction with the structural designed based on the final founding level. The concrete should have a minimum 28-day strength of 20 MPa.
- If a concrete tremie “plug” is not adopted, even with active dewatering, subgrade soils will be susceptible to loosening and disturbance due to water seepage, ponded water and construction traffic. It is recommended that a concrete working slab be placed on the subgrade within four hours to protect the integrity of the bearing stratum. A Special Provision should be included in the Contract Documents for a working slab, a copy of which is provided in Appendix D (FOUN0001).

### 6.5 Drilled Shafts (Caissons)

#### 6.5.1 Founding Elevations

The new abutments and piers for the proposed underpass replacement may also be supported on drilled shafts (caissons) founded on very dense / hard native soils and/or about 0.5 m into shale bedrock. The following drilled shaft founding elevations may be used for design purposes.



Foundation Element	Assumed Underside of Pile Cap Elevation (m)	Bottom of Caisson Elevation (m)	Founding Stratum
North Abutment	94.1	88.1	Hard clayey silt with sand till
North Pier	94.2	83.6	Shale Bedrock
South Pier	94.2	83.4	Shale Bedrock
South Abutment	94.4	88.4	Hard clayey silt with sand till

If drilled shaft foundations are adopted, a temporary liner 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 bentonite slurry to minimize the potential for non-cohesive materials ("flowing sands") to migrate into the drillhole 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 D. Drilled shaft foundations should be constructed in accordance with OPSS.PROV 903 (Deep Foundations), as amended by SP109F57, a copy of which is included in Appendix D for inclusion into the contract document.

### 6.5.2 Geotechnical Resistances

The factored ultimate and serviceability geotechnical resistances (at ULS and SLS for 25 mm of settlement, respectively) to be used for design of 1.2 m diameter drilled shaft (caisson) foundations are presented below. The factored ultimate geotechnical resistances of drilled shafts socketed 0.5 m into the good quality (i.e. RQD greater than 75 per cent) shale bedrock at the north and south piers was calculated based on a uniaxial compressive strength of 5 MPa; this uniaxial compressive strength was determined based on Golder's experience with similar rock types in the area.

Foundation Element	Factored Ultimate Geotechnical Resistance (kN)	Factored Serviceability Geotechnical Resistance (kN)
North Abutment	2,200	N/A <sup>1</sup>
North Pier	3,500	N/A <sup>1</sup>
South Pier	3,500	N/A <sup>1</sup>
South Abutment	2,250	N/A <sup>1</sup>

Notes:

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

The following factored ultimate and serviceability geotechnical resistances (at ULS and SLS for 25 mm of settlement, respectively) may be used for design of 1.5 m diameter drilled shaft (caisson) foundations:



Foundation Element	Factored Ultimate Geotechnical Resistance (kN)	Factored Serviceability Geotechnical Resistance (kN)
North Abutment	3,300	N/A <sup>1</sup>
North Pier	5,500	N/A <sup>1</sup>
South Pier	5,500	N/A <sup>1</sup>
South Abutment	3,400	N/A <sup>1</sup>

Notes:

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

### 6.5.3 Frost Protection

All drilled shaft pile caps, if located within the ground, 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 ground surface 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.

### 6.5.4 Key Challenges and Considerations

- If drilled shafts are adopted, a liner (temporary or permanent) and a head of water/drilling slurry will be required to support the overburden soils during construction and balance groundwater pressures to minimize disturbance to the side walls, loss of material from the non-cohesive strata and to groundwater pressures, and to control base disturbance/basal heave. In addition, placement of concrete using tremie methods will be required to construct the drilled shafts.
- 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 subgrade soils or bedrock. 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 visual remote inspection of the base of the drilled shafts (via a shaft inspection device such as a video camera), the drilled shaft excavations must be lined. The liner must be maintained tight to the sides of the soil.
- Based on the groundwater level measurements in February 2019, excavations for the drilled shafts pile caps, if located within the ground, will extend to within about 0.4 m of the groundwater level and therefore groundwater control measures (active dewatering) are likely required. Additional dewatering details are provided in Section 6.9.3. The requirement for dewatering could be eliminated if the pile caps are placed at the underside of the bridge deck, rather than below grade.



## 6.6 Resistance to Lateral Loads for Drilled Shaft Foundations

The design of drilled shaft (caissons) subjected to lateral loads should take into account such factors as the batter of the drilled shaft (if any), the relative rigidity of the caisson to the surrounding soil, the fixity condition at the head of the caisson (pile cap level), the structural capacity of the caisson to withstand bending moments, the soil resistance that can be mobilized, the tolerable lateral deflections at the head of the drilled shaft and group effects. For a longer, more flexible drilled shaft, the maximum yield moment of the caisson 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 can be resisted fully or partially using battered piles.

The resistance to lateral loading in front of a single drilled shaft may be estimated using subgrade reaction theory and the coefficient of horizontal subgrade reaction,  $k_h$  (kPa/m). However, the response of a drilled shaft to lateral loads is highly nonlinear and methods that assume linear behavior (such as subgrade reaction theory) are only appropriate where the maximum drilled shaft deflections are less than 1 per cent of the drilled shaft diameter, where the loading is static (no cycling) and where the pile material is linear (CFEM, 2006). If one or more of these conditions are not satisfied, lateral pile analysis should be carried out using p-y curves.

The resistance to lateral loading in front of a single drilled shaft may be calculated using subgrade reaction theory, provided that above conditions are met, where the coefficient of horizontal subgrade reaction,  $k_h$  (kPa/m), is based on the following equations (CFEM, 1992 as referenced in the *Commentary of the CHBDC, 2014*):

For non-cohesive soils:

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

where:

$n_h$	=	coefficient related to soil density (kPa/m)
$z$	=	depth (m)
$B$	=	drilled shaft diameter (m)

For cohesive soils:

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

where:

$s_u$	=	undrained shear strength of the soil (kPa)
$B$	=	drilled shaft diameter (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.



Soil Unit	Elevation (m)	$n_h$ (kN/m <sup>2</sup> /m)	$s_u$ (kPa)
Very Dense Silt to Sand and Gravel	Above Groundwater	17,500	Not Applicable
	Below Groundwater	10,500	
Hard Clayey Silt with Sand Till	Above Groundwater	17,500	Not Applicable
	Below Groundwater	10,500	

For a single drilled shaft, the factored lateral ultimate limit state (ULS) and factored lateral serviceability limit state (SLS) for 10 mm of horizontal deflection at the pile/caisson 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 Lateral Ultimate Geotechnical Resistance (kN)	Factored Lateral Serviceability Geotechnical Resistance for 10 mm of Deflection (kN)
North and South Abutments	1.2 m dia. Drilled shaft (6 m long)	275	225
	1.5 m dia. Drilled shaft (6 m long)	450	250
North and South Piers	1.2 m dia. Drilled shaft (10.6 m and 10.8 m long)	650	375
	1.5 m dia. Drilled shaft (10.6 m and 10.8 m long)	825	450

Note:

1. 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 the CHBDC, 2014*.

## 6.7 Lateral Earth Pressures for Design of Abutments and Wingwalls

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

The following recommendations are made concerning the design of the abutment's walls (and wingwalls):

- 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 OPSD 3190.100 (Walls, Retaining and Abutment, Wall Drain).

- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with *CHBDC* (2014) Section 6.12.3 and Figure 6.6. 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 C6.20(a) of the *Commentary to the CHBDC* (2014). For unrestrained walls, fill should be placed within the wedge-shaped zone defined by a line drawn at 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 C6.20(b) of the *Commentary to the CHBDC* (2014).

### 6.7.1 Static Lateral Earth Pressures for Design

The following guidelines and recommendations are provided regarding the lateral earth pressures for static 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 <sup>3</sup> )	Coefficients of Static Lateral Earth Pressure	
		At-Rest, $K_o$	Active, $K_a$
Earth Fill / SSM	20	0.50	0.33

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 <sup>3</sup> )	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 C6.6 of the *Commentary to the CHBDC*, 2014.



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

### 6.7.2 Seismic Lateral Earth Pressures for Design

Seismic (earthquake) loading must also be considered in the design of retaining and wingwalls in accordance with Section 4.6.5 of the *CHBDC* (2014). In this regard, the following should be included in the assessment of lateral earth pressures:

Seismic loading will result in increased lateral earth pressures acting on the abutment stem and / or retaining walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake induced dynamic earth pressure.

In accordance with Sections 4.6.5 and C.4.6.5 of the *CHBDC* (2014) and its *Commentary*, for structures which allow lateral yielding, the horizontal seismic coefficient,  $k_h$ , used in the calculation of the seismic active pressure coefficient, is taken as 0.5 times the site-specific PGA. For structures that do not allow lateral yielding,  $k_h$  is taken as equal to the site-specific PGA. For both cases the value of the vertical seismic coefficient  $k_v$  is taken as zero.

The following seismic active pressure coefficients ( $K_{AE}$ ) may be used in design; these coefficients reflect the maximum  $K_{AE}$  obtained for each of the earthquake design periods and backfill conditions. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is level. Where sloping backfill is present above the top of the wall, the lateral earth pressures under seismic loading conditions should be calculated by treating the weight of the backfill located above the top of the wall as a surcharge.

Wall Type	Design Earthquake	Site PGA	Seismic Active Pressure Coefficients, $K_{AE}$		
			Granular 'A'	Granular 'B' Type II	SSM
Yielding Wall (Non-Restrained)	475-Year	0.04	0.26	0.26	0.31
	975-Year	0.069	0.27	0.27	0.32
	2,475 Year	0.126	0.28	0.28	0.34
Non-Yielding Wall (Restrained)	475-Year	0.04	0.27	0.27	0.33
	975-Year	0.069	0.29	0.29	0.35
	2,475-Year	0.126	0.32	0.32	0.39

The  $K_{AE}$  value for a yielding wall is applicable provided that the wall can move up to  $250 \cdot k_h$  (measured in mm), where  $k_h$  is the site-specific PGA as given in the table above. This corresponds to displacements of 10 mm, 18 mm, and 32 mm for the 475-year, 975-year, and 2,475-year design earthquakes at this site.

The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e. an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined per Section C4.6.5 of the *Commentary* to *CHBDC* (2014).



## 6.8 Approach Embankment Design

### 6.8.1 Parameter Selection

The foundation engineering parameters for the soil types encountered in the boreholes at the approach embankments are summarized below. For the stability and settlement analyses, the groundwater level behind the abutment stem walls was assumed to be at Elevation 94.9 m.

Stratigraphic Unit	$\gamma$ (kN/m <sup>3</sup> )	$\phi'$ (°)	$s_u$ (kPa)	$\sigma_p'$ (kPa)	$e_o$	$C_c$	$C_r$	$E'$ (MPa)
New fill (Granular 'A' or 'B' Type II)	21	35	--	--	--	--	--	75
Compact to dense gravelly sand to sand and gravel fill	20	32	--	--	--	--	--	25
Firm to very stiff clayey silt with sand fill	19	30	50	225	0.6	0.4	0.04	--
Dense to very dense silt and sand till	21	35	--	--	--	--	--	150
Dense to very dense silt to sand and gravel	20	32	--	--	--	--	--	100
Hard clayey silt with sand till	21	35	--	--	--	--	--	100

### 6.8.2 Global Stability

Limit equilibrium global slope stability analyses were carried out for the proposed abutment walls 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,  $\Psi$ , and the geotechnical resistance factor,  $\phi_{gu}$ . (i.e.,  $FoS = 1/(\Psi \cdot \phi_{gu})$ ). Accordingly, minimum Factors of Safety of 1.5 have been used for the design of the approach embankment slopes for the long-term / permanent conditions, as per Table 6.2 of CHBDC (2014).

Based on the road profile provided on the GA drawings, the approach embankment side slopes will be inclined at 2 horizontal to 1 vertical (2H:1V) with an overall height of 0.4 m and 0.6 m of new fill over the native subgrade materials at the north and south approach embankments, respectively, and as such, global stability of the approach embankment side slopes is not expected to be an issue.

The stability analyses for the abutment walls indicate that in long-term (permanent) conditions, the approach embankments / abutments will have a global Factor of Safety greater than 1.5. The results of the stability analyses are summarized below and are shown on Figures 1 and 2 following the text of this report.

Foundation Element	Relevant Boreholes	Location	Slope	Factor of Safety
North Abutment	A1, AS-7	Abutment Wall	Front	2.3
South Abutment	A2, AS-2	Abutment Wall	Front	2.4



### 6.8.3 Settlement

To estimate the magnitude of expected settlement of the approach embankments due to the anticipated design grade raise of 0.4 m to 0.6 m at the north and south approach embankments, respectively, settlement analyses were carried out at the north and south approach embankments. The analyses were carried out using the commercially available program Settle<sup>3D</sup> (Version 4.0), developed by Rocscience Inc.

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.

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 Albert Street 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 of the approach embankments.

Based on the results of the settlement analyses, employing the commercially available program Settle 3D (Version 4.0) by Rocscience Inc., the total settlement of the existing site soils under the loading imposed by a 0.4 m and 0.6 m high embankment (grade raise) is estimated to be less than 25 mm at the north and south approach embankments. 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 OPSS.PROV 1010 (Aggregates) Granular 'A' or Granular 'B' Type II, placed and compacted in accordance with Section 6.9.4.

## 6.9 Construction Considerations

### 6.9.1 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 OPSS.PROV 1010 (Aggregates) Granular 'A' or Granular 'B' Type II material and Select Subgrade Material, respectively. 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 (i.e., those open for a relatively short time period) should be made with side slopes no steeper than 1H:1V based on the soil profile. 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.



## 6.9.2 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), as amended by SP 105S09. 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.

## 6.9.3 Groundwater and Surface Water Control

The groundwater level measured in the piezometer at the south approach embankment was about Elevation 95 m in February 2019 and the water level in the open boreholes drilled from the Highway 401 grade was between about Elevations 95 m and 93 m. 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 (including drilled shaft pile caps, if applicable) can be constructed in dry conditions. It is anticipated that dewatering measures will be required for construction of spread/strip footings and for drilled shaft pile cap construction, if applicable, at all foundation elements. Dewatering operations should be carried out / managed in accordance with OPSS.PROV 517 (Dewatering) as amended by Special Provision SP 517F01 (Dewatering System; Temporary Flow Passage System), a copy of which is included in Appendix D.

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.

## 6.9.4 Embankment Construction and Erosion Protection

Placement of Select Subgrade Material (SSM) or granular fill (satisfying OPSS.PROV 1010 SSM or Granular 'A' and Granular 'B' Type I or Type II requirements) above the water table for construction of the approach embankments grade raise (including backfill behind the abutments) 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), as amended by SP 105S22.

Side slopes for the SSM or granular fill embankment should be constructed no steeper than 2H:1V. In addition, benching of the existing Albert Street cut slopes and side slope reconstruction 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 as noted in OPSS.PROV 501 (Compacting), as amended by SP 105S22.

To reduce erosion of the embankment side slopes due to surface water runoff, placement of topsoil and seeding 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 802 (Topsoil) and OPSS.PROV 804 (Seed and Cover).

### 6.9.5 Obstructions

Glacially derived soil deposits are present at this site and as such should be expected to contain cobbles and boulders, as inferred by grinding of the augers during advancement of Borehole AS-7, which could affect the installation of piles and drilled shafts.

If drilled shaft pile 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) be included in the contract documents to address obstructions, a copy of which is included in Appendix D.

### 6.9.6 Vibration Monitoring

If drilled shafts (caissons) are adopted and if temporary protection systems are installed using vibratory methods, significant vibrations are not anticipated. A maximum peak particle velocity (PPV) of 100 mm/s is generally considered applicable for bridge structures in good condition. Based on vibration monitoring experience, it is considered unlikely that vibrations will reach this threshold level at the existing underpass and, therefore, vibration monitoring for the existing underpass structure is not expected to be required during construction at this site.

Commercial developments are located within 50 m and 60 m of the proposed underpass structure and additional commercial/residential developments are located within 200 m of the proposed underpass structure. A lower PPV threshold of 50 mm/s is generally considered applicable for buildings. While it is expected that vibration levels will not reach these thresholds at these offsite structures, it is anticipated that MTO may wish to incorporate pre- and post-construction condition surveys and vibration monitoring at or near the buildings. An NSSP has been included in Appendix D to address condition surveys and vibration monitoring within 100 m of the underpass structure.

### 6.9.7 Analytical Testing of Construction Materials

The results of analytical tests carried out on two soil samples and are presented in Section 4.4.1 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 0.003 per cent and 0.08 per cent, which indicates a less than Moderate degree of exposure (i.e., below the class S3 exposure limits) and may be considered negligible according to Table 7.2 of MTO's Gravity Pipe Design Guidelines (2004). Therefore, based on the 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 not need to be considered. However, given that the proposed structure will be exposed to de-icing salt/chemicals, consideration should also 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 silt and sand till and the sand and gravel soil samples tested have a pH of about 8.1 and resistivity of 1,700 ohm-cm and 1,400 ohm-cm, respectively. According to the MTO Gravity Pipe Design Guidelines (2014), the pH is not considered detrimental to culvert durability as it is less than a pH of 8.5. The resistivity is less than 2,000 ohm-cm which indicates that the soil corrosiveness is severe ( $2,000 > R$ ) as per Table 3.2 of the MTO Gravity Pipe Design Guideline (2014).

These recommendations are provided as guidance only; the structural designer should take the results of the laboratory testing, the potential for corrosion and the corrosion susceptibility of materials to be used in construction of the structure foundations in Table 7.1 of the MTO Gravity Pipe Design Guideline (2014) into consideration of the ultimate selection of materials. 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.

## 6.10 Piezometer Decommissioning

A standpipe piezometer was installed in Borehole AS-2 to permit monitoring of the groundwater level at the site. Ontario Regulation (O.Reg.) 903 amended by O.Reg. 128/03 of the Ontario Water Resources Act requires that monitoring wells are properly abandoned/decommissioned by qualified personnel. It is recommended that the decommissioning of the standpipe piezometer be carried out as part of the construction activities at the site to allow for water level measurements to be taken immediately prior to and during construction as may be appropriate. The standpipe piezometer in Borehole AS-2 should then be abandoned under the Construction Contract work; an NSSP for this item is included in Appendix D.

## 7.0 CLOSURE

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

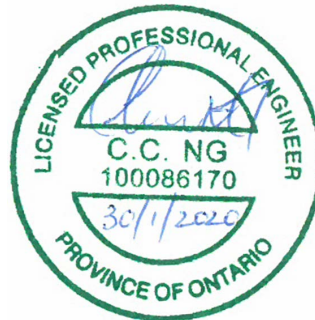


## Signature Page

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- Unified Facilities Criteria, U.S. Navy. 1986. NAVFAC Design Manual 7.02. Soil Mechanics, Foundation and Earth Structures. Alexandria, Virginia.

### Commercial Software:

- Slide (Version 2018) by Rocscience Inc.
- Settle<sup>3D</sup> (Version 4.0) by Rocscience Inc.
- LPILE Plus (Version 2016) by Ensoft Inc.

### Ontario Occupational Health and Safety Act:

- O. Reg. 213 Construction Projects (as amended)
- Ontario Regulation 903 Wells (as amended)

### Ontario Provincial Standard Specifications (OPSS)

- |               |                                                                       |
|---------------|-----------------------------------------------------------------------|
| OPSS.PROV 206 | Construction Specification for Grading                                |
| OPSS.PROV 501 | Construction Specification for Compacting                             |
| OPSS.PROV 517 | Construction Specification for Dewatering                             |
| OPSS.PROV 539 | Construction Specification for Temporary Protection Systems           |
| OPSS.PROV 802 | Construction Specification for Topsoil                                |
| OPSS.PROV 804 | Construction Specification for Seed and Cover                         |
| OPSS 902      | Construction Specification for Excavating and Backfilling – Structure |



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OPSS.PROV 903	Construction Specification for Deep Foundations
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade, and Backfill Material

**Ontario Provincial Standard Drawings (OPSD)**

OPSD 208.010	Benching of Earth Slopes
OPSD 3090.101	Foundation, Frost Penetration Depths for Southern Ontario
OPSD 3101.150	Walls, Abutments, Backfill, Minimum Granular Requirements
OPSD 3121.150	Walls, Retaining, Backfill, Minimum Granular Requirements
OPSD 3190.100	Walls, Retaining and Abutments, Wall Drain

**Special Provisions**

SP 105S09	Amendment to OPSS 539
SP 109S12	Amendment to OPSS 902
SP 105S22	Amendment to OPSS 501
SP 109F57	Amendment to OPSS 903
SP 517F01	Dewatering System – Temporary Flow Passage System



**Table 1: Comparison of Foundation Alternatives – Albert Street Underpass**

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risk / Consequences
Spread / strip footings founded on existing very dense / hard stratum	<ul style="list-style-type: none"> <li>Feasible for all foundation elements</li> <li>Most feasible at north abutment</li> </ul>	<ul style="list-style-type: none"> <li>Conventional construction techniques.</li> <li>Presence of suitable stratum at shallow depths to provide required axial resistance.</li> </ul>	<ul style="list-style-type: none"> <li>Temporary protection system for construction of foundations is required at the abutments and pier locations.</li> <li>Dewatering measures are required at abutments and pier locations.</li> <li>Does not allow for integral abutment design.</li> </ul>	<ul style="list-style-type: none"> <li>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.</li> </ul>	<ul style="list-style-type: none"> <li>Challenging to construct at north pier due to limited workspace</li> <li>Dewatering challenges at north pier due to location.</li> <li>Longer staging and more significant protection systems required adjacent to south abutment</li> </ul>
Driven Steel H-piles (HP 310x110) or Pile Piles	<ul style="list-style-type: none"> <li>Not practical due to relatively shallow depth to “100-blow” soil (would require pre-auguring to achieve required pile length)</li> </ul>	<ul style="list-style-type: none"> <li>Conventional construction methods for H-pile foundations.</li> <li>High axial resistances available at shallow depths</li> <li>Allow for integral abutment design, if piles have adequate length.</li> </ul>	<ul style="list-style-type: none"> <li>Short piles due to very dense/hard (“100-blow”) soils at shallow depth at abutments and piers.</li> <li>Pre-drilling through the “100-blow” soils is required prior to pile driving.</li> <li>Temporary protection system is required at the abutments and pier locations.</li> <li>Dewatering measures are required at abutments and pier locations for the construction of pile caps.</li> <li>Requires driving shoes due to potential presence of cobbles / boulders within the till deposits.</li> </ul>	<ul style="list-style-type: none"> <li>Higher relative cost than spread / strip footings.</li> <li>Lower relative cost than drilled shafts (caissons).</li> </ul>	<ul style="list-style-type: none"> <li>Risk of misalignment of piles due to the shallow depth of “100-blow” soil below pile cap level.</li> <li>Risk of damage to piles due to cobbles / boulders within the till deposits.</li> </ul>
1.2 m or 1.5 m Diameter Drilled Shafts (Caissons)	<ul style="list-style-type: none"> <li>Feasible for all foundation elements</li> </ul>	<ul style="list-style-type: none"> <li>Conventional construction methods for drilled shaft foundations.</li> <li>Offers higher geotechnical resistance compared to driven steel piles, requiring fewer foundation elements. If underside of pile caps is extended to higher elevations, dewatering potentially not required.</li> <li>Requires a smaller footprint for construction in constrained working areas, as compared with spread/strip footings or driven piles.</li> </ul>	<ul style="list-style-type: none"> <li>Liners will be required, plus special measures such as use of bentonite slurry to counterbalance groundwater pressures and minimize ground disturbance.</li> <li>Generation of soil cuttings during drilled shaft advancement.</li> <li>Limited potential for inspection of shaft base due to presence of bentonite slurry (or potential loss of base resistance if slurry is recovered)</li> <li>Does not allow for integral abutment design.</li> </ul>	<ul style="list-style-type: none"> <li>Higher relative cost than spread / strip footings and driven piles.</li> </ul>	<ul style="list-style-type: none"> <li>May be difficult to inspect the base of the drilled shaft due to potential need for bentonite slurry inside the liners.</li> </ul>

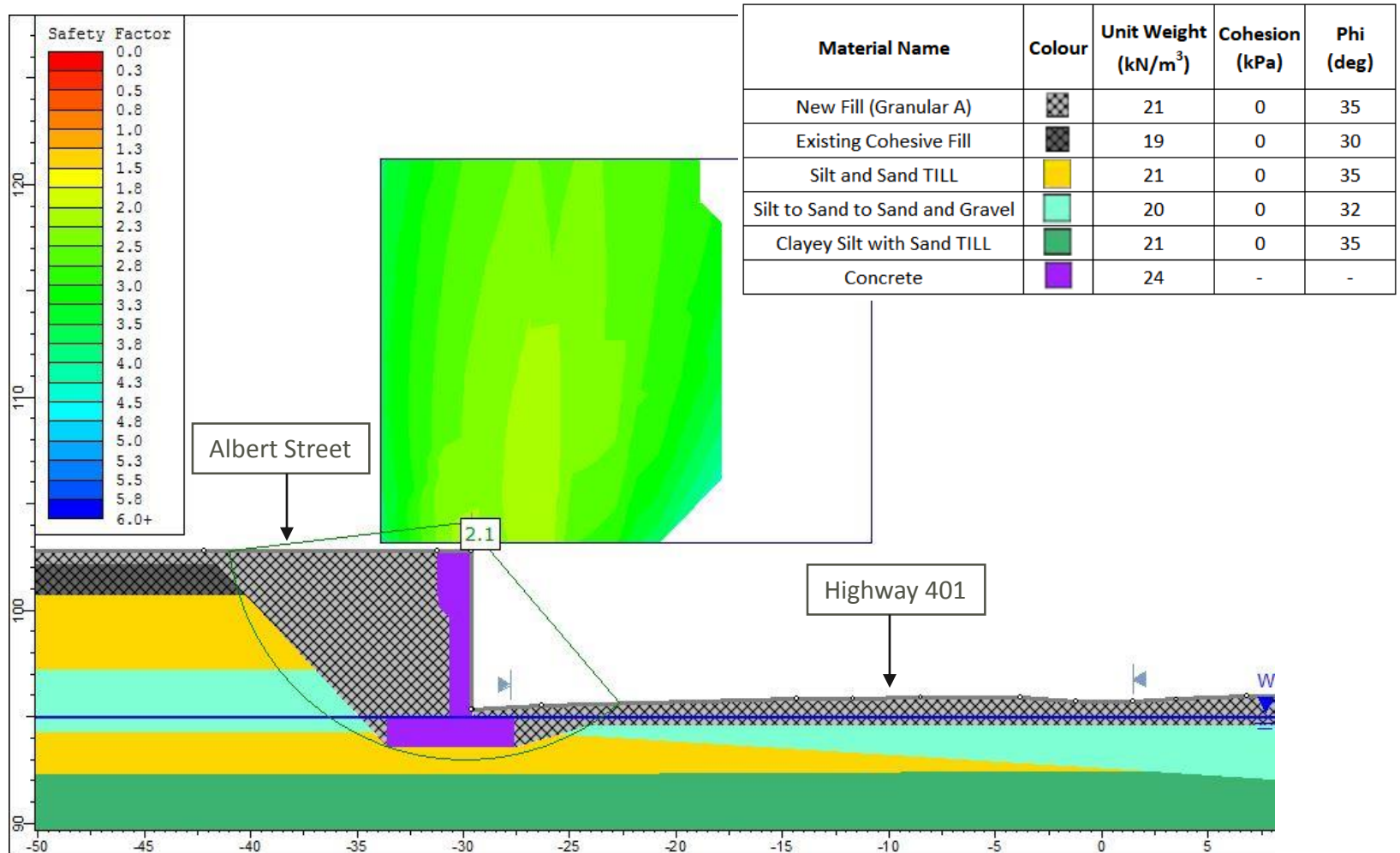


# Global Stability Analysis

## Albert Street - North Abutment

### Long-Term Analysis

FIGURE 1



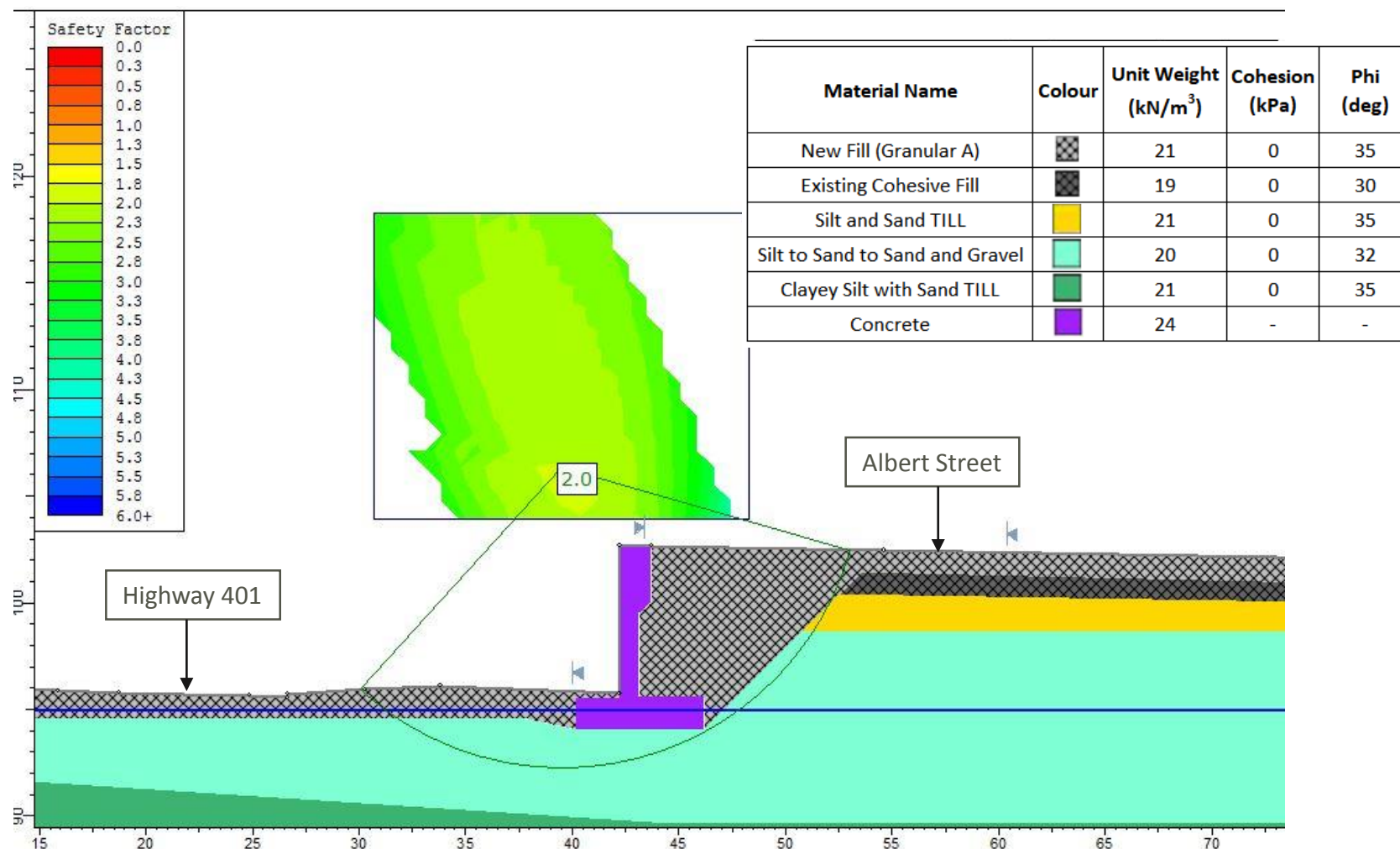


# Global Stability Analysis

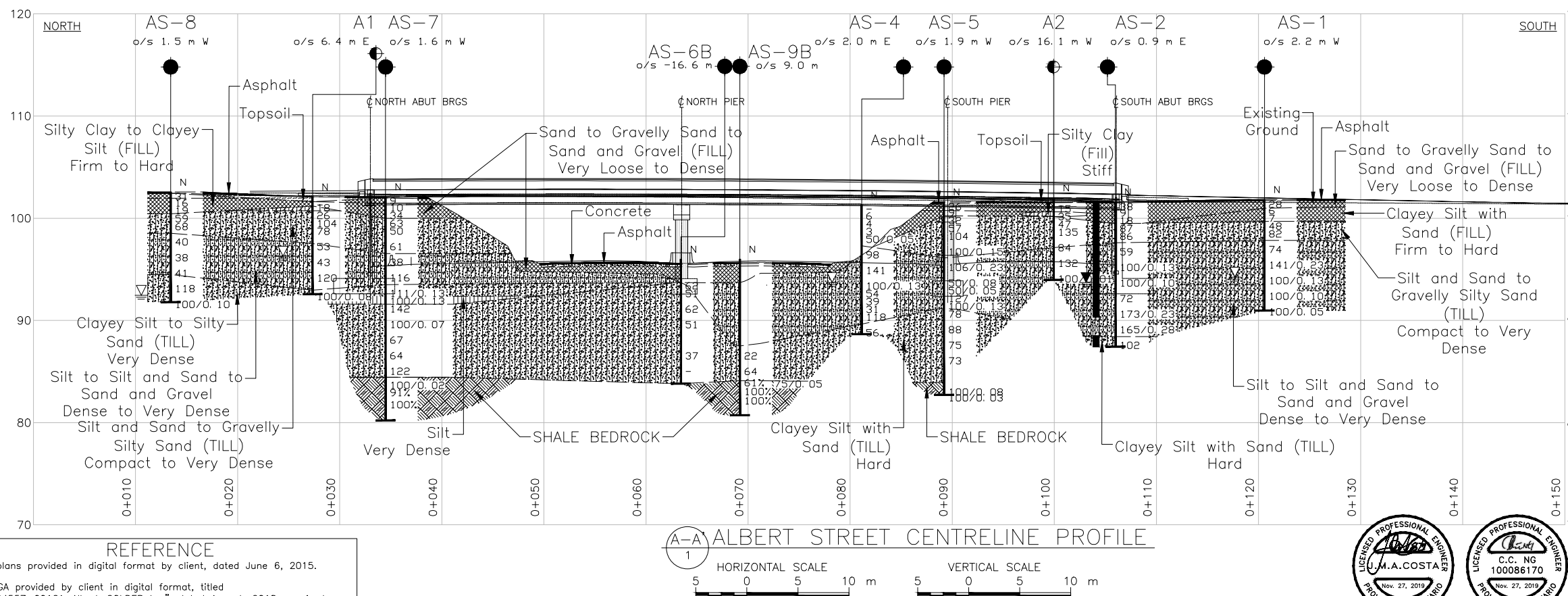
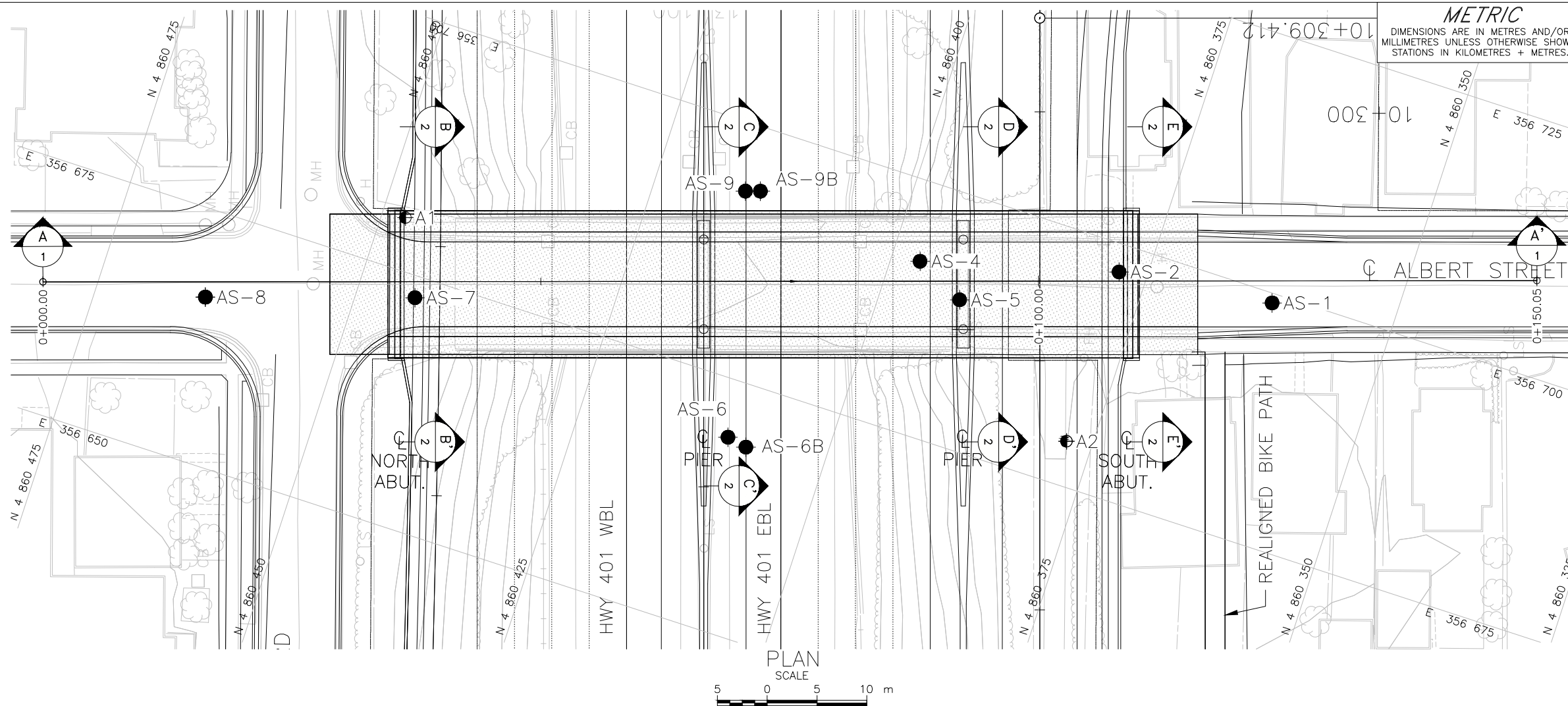
## Albert Street – South Abutment

### Long-Term Analysis

**FIGURE 2**







## REFERENCE

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

BridgeGA provided by client in digital format, titled "S17104557-001GA-Albert-GOLDER.dwg", dated August, 2018, received on August 23, 2018.

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

CONT No.  
GWP No. 2298-13-00

HIGHWAY 401 REHABILITATION/WIDENING  
ALBERT STREET UNDERPASS  
BOREHOLE LOCATION AND SOIL  
STRATA



KEY PLAN  
SCALE  
1 0 1 2 km

## LEGEND

- Borehole - Current Investigation
- Borehole - Previous Investigation
- ⊥ Seal
- ⊥ Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated  
(Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- ≡ WL in piezometer, measured February 8, 2019
- ≡ WL upon completion of drilling

## BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
A1	102.1	4860447.6	356680.2
A2	102.0	4860377.6	356679.4
AS-1	101.8	4860362.2	356699.0
AS-2	101.6	4860377.8	356697.2
AS-4	101.3	4860397.1	356692.1
AS-5	101.6	4860392.2	356689.6
AS-6	96.0	4860410.0	356669.3
AS-6B	96.0	4860408.0	356668.9
AS-7	102.1	4860444.2	356672.9
AS-8	102.6	4860464.3	356666.4
AS-9	96.0	4860416.0	356693.3
AS-9B	96.0	4860414.6	356693.8

## NOTES

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

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

NO.	DATE	BY	REVISION
-----	------	----	----------

Geocres No. 30M15-330

HWY. 401	PROJECT NO. 1662582	DIST.
SUBM'D. AP	CHKD. AP	DATE: 11/27/2019
DRAWN: SMD/DD	CHKD. CN	APPD. CN/JMAC
SITE: 22-177	DWG. 1	





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

CONT No.  
GWP No. 2298-13-00

HIGHWAY 401 REHABILITATION/WIDENING  
ALBERT STREET UNDERPASS  
SOIL STRATA

SHEET



KEY PLAN  
SCALE  
1 0 1 2 km

## LEGEND

- Borehole - Current Investigation
- Borehole - Previous Investigation
- ⊥ Seal
- ⊥ Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- ≡ WL in piezometer, measured February 8, 2019
- ≡ WL upon completion of drilling

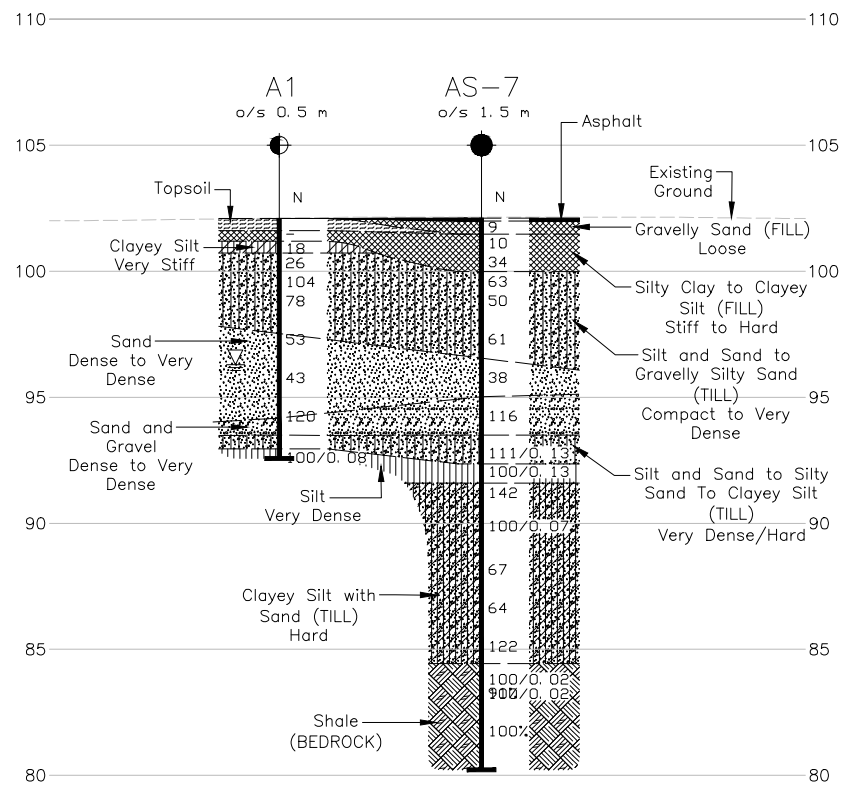
## BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
A1	102.1	4860447.6	356680.2
A2	102.0	4860377.6	356679.4
AS-2	101.6	4860377.8	356697.2
AS-4	101.3	4860397.1	356692.1
AS-5	101.6	4860392.2	356689.6
AS-6B	96.0	4860408.0	356668.9
AS-7	102.1	4860444.2	356672.9
AS-9B	96.0	4860414.6	356693.8

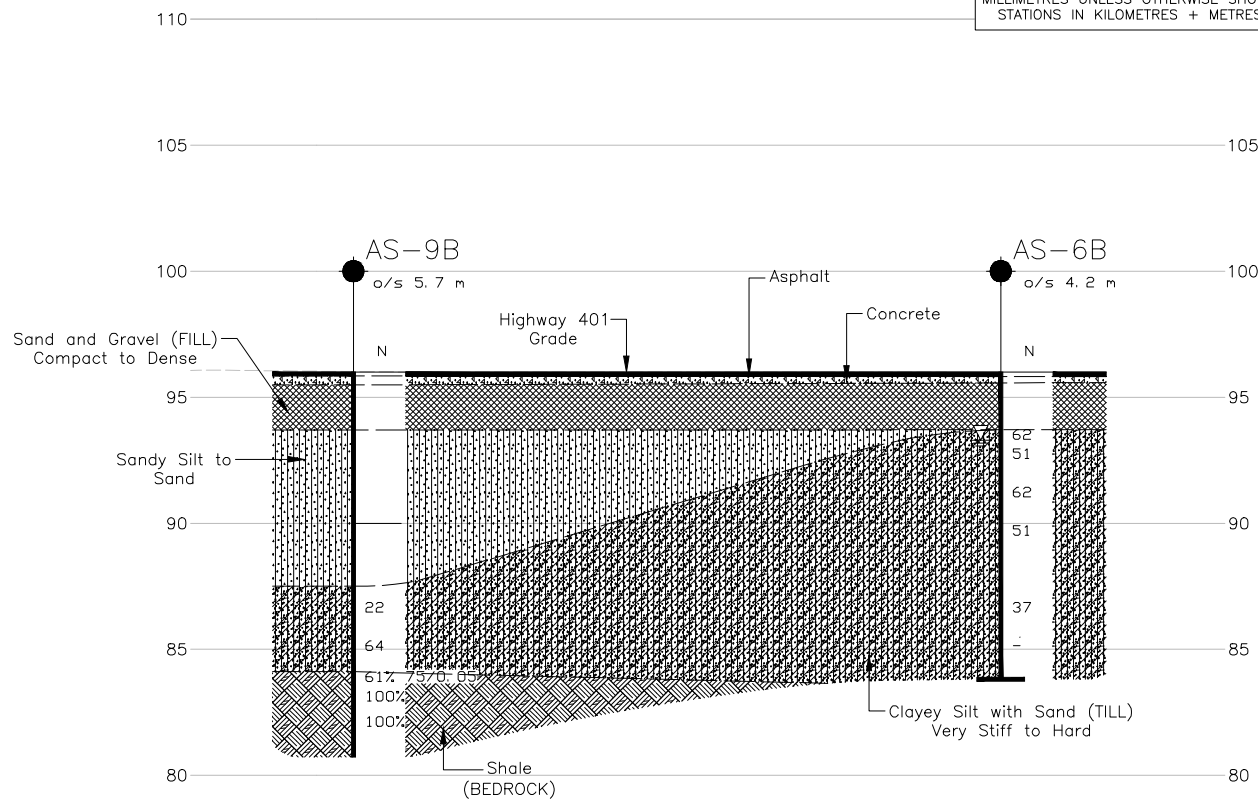
## NOTES

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.

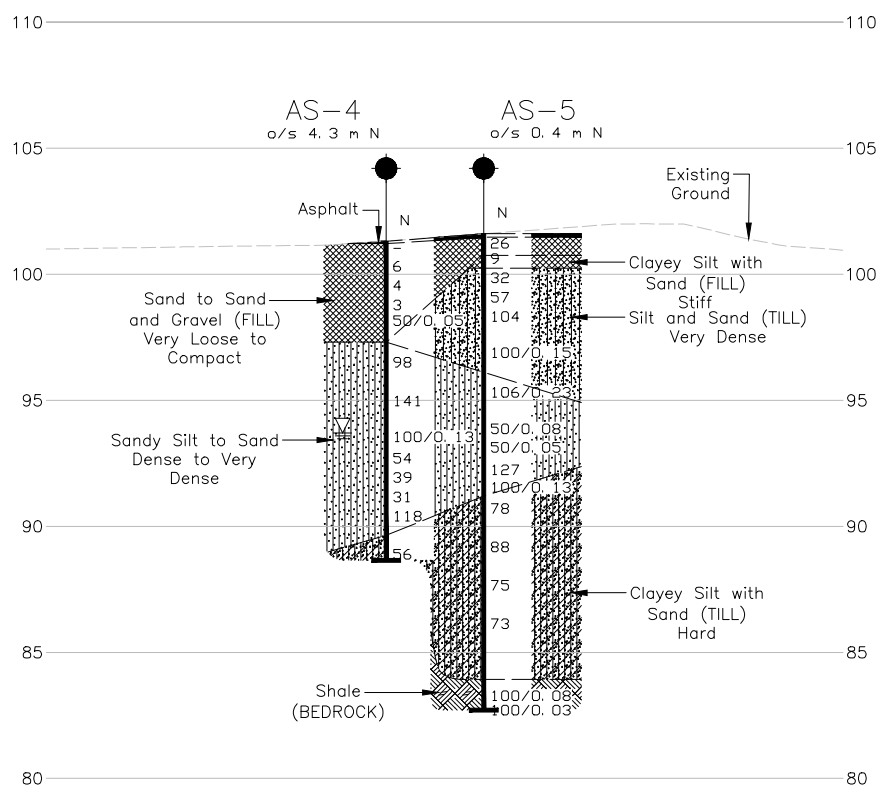
The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.



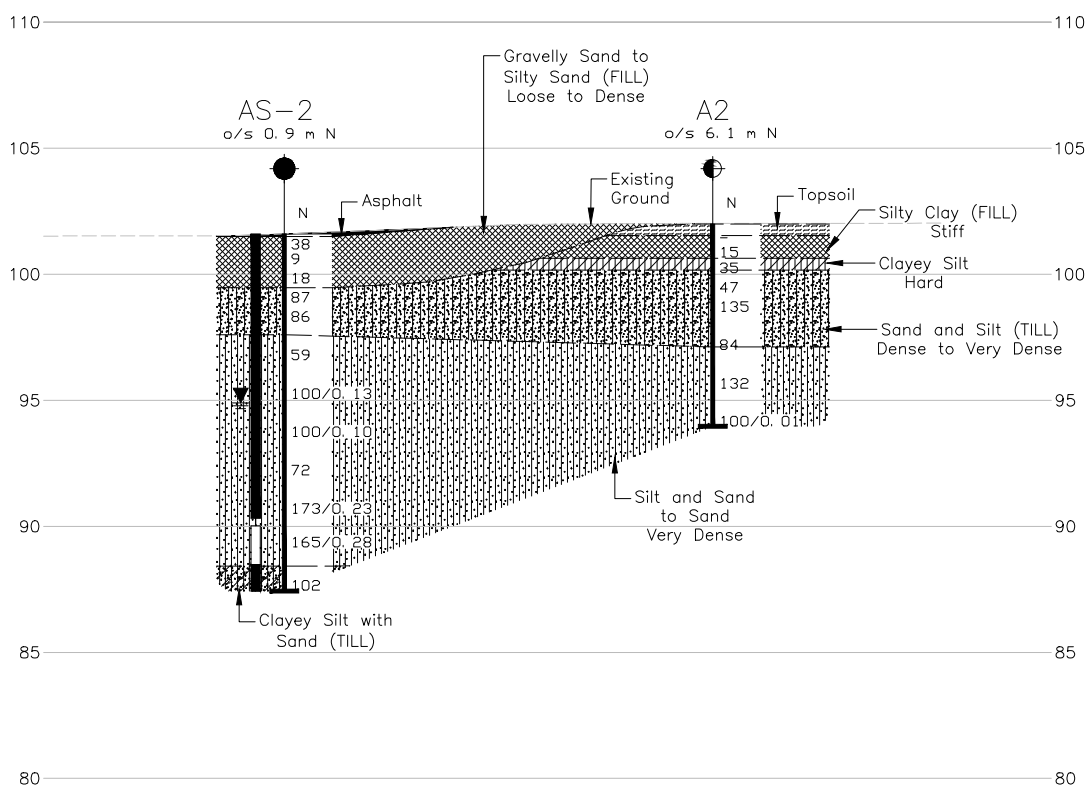
B-B NORTH ABUTMENT CROSS-SECTION  
1



C-C NORTH PIER CROSS-SECTION  
1



D-D SOUTH PIER CROSS-SECTION  
1



E-E SOUTH ABUTMENT CROSS-SECTION  
1

## REFERENCE

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

BridgeGA provided by client in digital format, titled "S17104557-001GA-Albert-GOLDER.dwg", dated August, 2018, received on August 23, 2018.



NO.	DATE	BY	REVISION
Geocres No. 30M15-330			
HWY. 401	PROJECT NO. 1662582		DIST.
SUBM'D. AP	CHKD. AP	DATE: 11/27/2019	SITE: 22-177
DRAWN: SMD/DD	CHKD. CN	APPD. CN/JMAC	DWG. 2



**APPENDIX A**

# Borehole Records



## LIST OF SYMBOLS

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

### I. GENERAL

$\pi$	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

$\gamma$	shear strain
$\Delta$	change in, e.g. in stress: $\Delta \sigma$
$\varepsilon$	linear strain
$\varepsilon_v$	volumetric strain
$\eta$	coefficient of viscosity
$\nu$	Poisson's ratio
$\sigma$	total stress
$\sigma'$	effective stress ( $\sigma' = \sigma - u$ )
$\sigma'_{vo}$	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
$\sigma_{oct}$	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
$\tau$	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
$\gamma'$	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
$C_{\alpha}$	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
$\sigma'_p$	pre-consolidation stress
OCR	over-consolidation ratio = $\sigma'_p / \sigma'_{vo}$

#### (d) Shear Strength

$\tau_p, \tau_r$	peak and residual shear strength
$\phi'$	effective angle of internal friction
$\delta$	angle of interface friction
$\mu$	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  $\rho$ . Unit weight symbol is  $\gamma$  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 cm<sup>2</sup> 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 <sup>1</sup>
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement <sup>1</sup>
$D_R$	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)
$\gamma$	unit weight

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

### V. MINOR SOIL CONSTITUENTS

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



## WEATHERINGS STATE

**Fresh:** no visible sign of weathering

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

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

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

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

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

## BEDDING THICKNESS

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

## JOINT OR FOLIATION SPACING

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

## GRAIN SIZE

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

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

## CORE CONDITION

### Total Core Recovery (TCR)

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

### Solid Core Recovery (SCR)

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

### Rock Quality Designation (RQD)

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

## DISCONTINUITY DATA

### Fracture Index

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

### Dip with Respect to Core Axis

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

### Description and Notes

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

### Abbreviations

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



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



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

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+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3</sup>% STRAIN AT FAILURE



PROJECT 1662582 (2000)			RECORD OF BOREHOLE No AS-2		SHEET 1 OF 2		METRIC	
G.W.P. 2298-13-00			LOCATION N 4860377.8; E 356697.2 MTM NAD 83 ZONE 10 (LAT. 43.881557; LONG. -78.854175)			ORIGINATED BY LP		
DIST Central HWY 401			BOREHOLE TYPE 216 mm O.D. Hollow Stem Augers			COMPILED BY AK/KN		
DATUM Geodetic			DATE November 21 and 22, 2018			CHECKED BY AP		
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100
101.6	GROUND SURFACE							PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W <sub>p</sub> — W — W <sub>L</sub> WATER CONTENT (%)
100.8	ASPHALT (100 mm)		1	SS	38		101	
	Gravelly sand, some silt (FILL)							
	Dense Brown Moist		2	SS	9		100	
	Silty sand, some gravel (FILL)							
	Loose to compact Brown Moist		3	SS	18			
99.5								
	SILT and SAND, trace to some clay, trace to some gravel (TILL)		4	SS	87		99	
	Very dense Brown, oxidation staining Moist							
			5	SS	86		98	
97.6								
	SILT and SAND		6	SS	59		97	
	Very dense Grey Moist to wet below a depth of 6.1 m (Elev. 95.5 m)							
			7	SS	100/0.13		96	
			8	SS	100/0.14		95	
							94	
			9	SS	72		93	
			10	SS	73/0.23		92	
			11	SS	65/0.28		91	
							90	
							89	
88.4								
	CLAYEY SILT with SAND, trace gravel (TILL)		12	SS	102		88	
	Hard Grey Moist							
87.4								
	END OF BOREHOLE							
14.2								

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+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

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+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3</sup>% STRAIN AT FAILURE

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PROJECT		1662582 (2000)		RECORD OF BOREHOLE		No AS-4		SHEET 1 OF 1		METRIC			
G.W.P.		2298-13-00		LOCATION		N 4860397.1; E 356692.1 MTM NAD 83 ZONE 10 (LAT. 43.881732; LONG. -78.854237)		ORIGINATED BY		MB			
DIST		Central HWY 401		BOREHOLE TYPE		210 mm O.D. Hollow Stem Augers		COMPILED BY		AK/KN			
DATUM		Geodetic		DATE		March 9, 2018		CHECKED BY		AP			
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT $\gamma$	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	W <sub>p</sub> W W <sub>L</sub>	WATER CONTENT (%)		
101.3	GROUND SURFACE												
0.0	ASPHALT (90 mm)												
0.1	Gravelly sand to sand, some silt, some gravel (FILL) Very loose to loose Brown Moist		1	AS	-								
			2	SS	6								
			3	SS	4								
			4	SS	3								
	- Concrete fragments at a depth of 3.0 m (Elev. 98.3 m)		5	SS	50/0.05								
97.3													
4.0	Sandy SILT, trace to some clay, trace gravel Very dense Brown to grey Moist		6	SS	98								
			7	SS	141								
	- Grey below a depth of 7.6 m (Elev. 93.7 m)		8	SS	100/0.13								
93.1													
8.2	SAND, some silt to silty Dense to very dense Grey Wet		9	SS	54								
			10	SS	39								
			11	SS	31								
			12	SS	118								
89.6													
11.7	CLAYEY SILT with SAND, trace gravel (TILL) Hard Grey Moist		13	SS	56								
88.6													
12.7	END OF BOREHOLE												
	NOTES:  1. Water encountered at a depth of 7.6 m below ground surface (Elev. 93.7 m) during drilling.												

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

PROJECT		1662582 (2000)		RECORD OF BOREHOLE		No AS-5		SHEET 1 OF 2		METRIC		
G.W.P.		2298-13-00		LOCATION		N 4860392.2; E 356689.6 MTM NAD 83 ZONE 10 (LAT. 43.881687; LONG. -78.854268)		ORIGINATED BY		LP		
DIST		Central HWY 401		BOREHOLE TYPE		152 mm Solid Stem Augers / 210 mm O.D Hollow Stem Augers		COMPILED BY		AK/KN		
DATUM		Geodetic		DATE		November 6 and 7, 2017		CHECKED BY		AP		
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT  γ  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
101.6	GROUND SURFACE											
0.0	ASPHALT (140 mm)											
0.1	Sand and gravel, some silt (FILL) Compact Brown Moist		1	SS	26							
100.7												
0.9	Clayey silt with sand (FILL) Stiff Brown Moist		2	SS	9							
100.2												
1.4	SILT and SAND, trace to some clay, trace to some gravel (TILL) Dense to very dense Brown, oxidation staining Moist		3	SS	32							
			4	SS	57							
			5	SS	104							
	- Cobble fragments at a depth of 4.6 m (Elev. 97.0 m)		6	SS	100/0.15							
96.1												
5.5	SILT and SAND, trace to some clay, trace gravel Very dense Grey Moist		7	SS	106/0.23							
			8	SS	50/0.08							
			9	SS	50/0.05							
92.8												
8.8	SAND, some silt Very dense Grey Moist											
			10	SS	127							
			11	SS	100/0.13							
91.2												
10.4	CLAYEY SILT with SAND, trace gravel (TILL) Hard Grey Moist		12	SS	78							
			13	SS	88							
			14	SS	75							

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

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PROJECT <u>1662582 (2000)</u>				RECORD OF BOREHOLE <b>No AS-5</b>				SHEET 2 OF 2				<b>METRIC</b>					
G.W.P. <u>2298-13-00</u>				LOCATION <u>N 4860392.2; E 356689.6 MTM NAD 83 ZONE 10 (LAT. 43.881687; LONG. -78.854268)</u>				ORIGINATED BY <u>LP</u>									
DIST <u>Central</u> HWY <u>401</u>				BOREHOLE TYPE <u>152 mm Solid Stem Augers / 210 mm O.D Hollow Stem Augers</u>				COMPILED BY <u>AK/KN</u>									
DATUM <u>Geodetic</u>				DATE <u>November 6 and 7, 2017</u>				CHECKED BY <u>AP</u>									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					WATER CONTENT (%)				
--- CONTINUED FROM PREVIOUS PAGE ---																	
	CLAYEY SILT with SAND, trace gravel (TILL) Hard Grey Moist		15	SS	73		86										3 39 40 18
							85										
							84										
83.9 17.7	SHALE (BEDROCK) Black-grey		16	SS	100/0.00		83										
82.7 18.9	END OF BOREHOLE DUE TO SPLIT-SPOON AND AUGER REFUSAL  NOTES:  1. Borehole caved to a depth of 10.1 m below ground surface (Elev. 91.5 m) upon removal of augers.		17	SS	100/0.00												

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PROJECT		1662582 (2000)		RECORD OF BOREHOLE No AS-6		SHEET 1 OF 1		METRIC									
G.W.P.		2298-13-00		LOCATION		N 4860410.0; E 356669.3 MTM NAD 83 ZONE 10 (LAT. 43.881849; LONG. -78.854519)		ORIGINATED BY KN									
DIST		Central HWY 401		BOREHOLE TYPE		216 mm O.D Hollow Stem Augers		COMPILED BY AK									
DATUM		Geodetic		DATE		November 9, 2018		CHECKED BY AMP									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
96.0	GROUND SURFACE							20	40	60	80	100					
95.1	ASPHALT (75 mm)																
	Sand and gravel, trace silt, trace clay (FILL)		1	SS	29												
	Compact to dense		2	SS	14												
	Grey to brown																
	Dry to wet																
	- Concrete fragments, auger grinding below 1.5 m depth (Elev. 94.5 m)		3	SS	32												
93.9																	
2.1	END OF BOREHOLE																
NOTES: 1. Water encountered at a depth of 0.9 m below ground surface (Elev. 95.1 m) upon completion of drilling. 2. Due to auger grinding on inferred obstruction Borehole AS-6 was abandoned and Borehole AS-6B was drilled approximately 2 m southwest of Borehole AS-6 (Refer to Record of Borehole AS-6B).																	



PROJECT		1662582 (2000)		RECORD OF BOREHOLE No AS-6B		SHEET 1 OF 1		METRIC						
G.W.P.		2298-13-00		LOCATION		N 4860408.0; E 356668.9 MTM NAD 83 ZONE 10 (LAT. 43.881831; LONG. -78.854524)		ORIGINATED BY KN						
DIST		Central HWY 401		BOREHOLE TYPE		216 mm O.D Hollow Stem Augers		COMPILED BY AK						
DATUM		Geodetic		DATE		November 9, 2018		CHECKED BY AMP						
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100					
96.0	GROUND SURFACE													
0.0	ASPHALT (180 mm)													
	CONCRETE (255 mm)													
0.4	Sand and gravel, trace silt, trace clay (FILL) Grey to brown Dry to wet													
93.7														
2.3	CLAYEY SILT with SAND, trace to some gravel (TILL) Hard Grey Dry to moist		4	SS	62	▽								3 36 37 24
			5	SS	51									
			6	SS	62									
			7	SS	51									6 41 38 15
			8	SS	37									
			9	AS	-									
83.8	- Hydrocarbon odours from 10.4 m to 12.2 m depth - Increased drilling resistance below 10.7 m depth (Elev. 85.3 m)		10	AS	-									13 45 24 18
12.2	- Wet at 12.2 m depth (Elev. 83.9 m) END OF BOREHOLE													
NOTES: 1. Borehole caved to a depth of 11.9 m below ground surface upon removal of augers. 2. Water level measured in open borehole at a depth of 2.7 m below ground surface (Elev. 93.3 m) upon removal of augers.														

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PROJECT 1662582 (2000)			RECORD OF BOREHOLE No AS-7			SHEET 1 OF 2			METRIC											
G.W.P. 2298-13-00			LOCATION N 4860444.2; E 356672.9 MTM NAD 83 ZONE 10 (LAT. 43.882157; LONG. -78.854472)			ORIGINATED BY LP														
DIST Central HWY 401			BOREHOLE TYPE 216 mm O.D. Hollow Stem Augers, HQ Coring			COMPILED BY AK/KN														
DATUM Geodetic			DATE November 16 and 17, 2017			CHECKED BY AP														
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40						60	80	100	20	40	60
102.1	GROUND SURFACE																			
0.0	ASPHALT (120 mm)						102													
0.1	Gravelly sand, some silt (FILL)		1	SS	9															
101.5	Loose Brown Moist		2	SS	10		101													
0.6	sandy clayey silt to clayey silt, some sand, trace gravel (FILL)		3	SS	34															
	Stiff to hard Brown Moist																			
100.0	Gravelly Silty SAND to Silty SAND, trace to some clay, trace gravel (TILL)		4	SS	63		100													
2.1	Very dense Brown Moist		5	SS	50		99													
			6	SS	61		98													
96.5	SAND, some silt		7	SS	38		96													
5.6	Dense Brown Wet																			
95.0	SAND and GRAVEL, some silt		8	SS	116		95													
7.1	Very dense Brown Moist to wet																			
93.5	SILT and SAND, trace clay, some gravel (TILL)		9	SS	111/0.13		93													
8.6	Very dense Grey Moist																			
92.3	SILT, trace sand, trace clay		10	SS	100/0.13		92													
9.8	Very dense Grey Wet																			
91.6	CLAYEY SILT with SAND, trace to some gravel (TILL)		11	SS	142		91													
10.5	Hard Grey Moist																			
	- Auger grinding at a depth of 11.4 m (Elev. 90.7 m)		12	SS	100/0.07		90													
	- Auger grinding between depths of 12.2 m and 13.7 m (Elev. 89.9 m and Elev. 88.4 m)																			
			13	SS	67		88													

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

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PROJECT <u>1662582 (2000)</u>			RECORD OF BOREHOLE <b>No AS-7</b>			SHEET 2 OF 2			<b>METRIC</b>									
G.W.P. <u>2298-13-00</u>			LOCATION <u>N 4860444.2; E 356672.9 MTM NAD 83 ZONE 10 (LAT. 43.882157; LONG. -78.854472)</u>			ORIGINATED BY <u>LP</u>												
DIST <u>Central</u> HWY <u>401</u>			BOREHOLE TYPE <u>216 mm O.D. Hollow Stem Augers, HQ Coring</u>			COMPILED BY <u>AK/KN</u>												
DATUM <u>Geodetic</u>			DATE <u>November 16 and 17, 2017</u>			CHECKED BY <u>AP</u>												
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)
	--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100						
	CLAYEY SILT with SAND, trace to some gravel (TILL) Hard Grey Moist		14	SS	64		87											
							86											
			15	SS	122		85											
84.4	SHALE (BEDROCK)						84											
17.7	Bedrock cored between depths of 18.9 m and 21.9 m (Elev. 83.2 m and 80.2 m)  For bedrock coring details refer to Record of Drillhole AS-7.		16	SS	100/0.02		83											RQD = 91%
			17	SS	100/0.02		82											
			1	RC	REC 100%		81											RQD = 100%
			2	RC	REC 100%													
80.2	END OF BOREHOLE																	
21.9	NOTE:  1. Water level not recorded upon completion of drilling.																	

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PROJECT: 1662582 (2000)

## RECORD OF DRILLHOLE: AS-7

SHEET 1 OF 1

LOCATION: N 4860444.2 ;E 356672.9

DRILLING DATE: November 20, 2017

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 75 Truck

DRILLING CONTRACTOR: Pontil Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV.	RUN No.	JN - Joint FLT - Fault SHR- Shear VN - Vein CJ - Conjugate BD- Bedding FO- Foliation CO- Contact OR- Orthogonal CL - Cleavage PL - Planar CU- Curved UN- Undulating ST - Stepped IR - Irregular PO- Polished K - Slickensided SM- Smooth Ro - Rough MB- Mechanical Break BR - Broken Rock NOTE: For additional abbreviations refer to list of abbreviations & symbols.														FEATURES	R0/R1 ZONES	NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											
				DEPTH (m)		RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.25 m	DISCONTINUITY DATA						ROCK STRENGTH INDEX	WEATH- ERING INDEX																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																
						TOTAL CORE %	SOLID CORE %			TYPE AND SURFACE DESCRIPTION	Jr	Ja	R4	R3	R2		R1	W1	W2				W3	W4	W5	W6																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																							

## FEATURES LEGEND



BROKEN CORE



CLAY SEAM



LIMESTONE



LOST CORE

DEPTH SCALE

1 : 50



GOLDER

LOGGED: LP

CHECKED: ACK

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PROJECT 1662582 (2000)			RECORD OF BOREHOLE No AS-8			SHEET 1 OF 1			METRIC								
G.W.P. 2298-13-00			LOCATION N 4860464.3; E 356666.4 MTM NAD 83 ZONE 10 (LAT. 43.882338; LONG. -78.854550)			ORIGINATED BY LP											
DIST Central HWY 401			BOREHOLE TYPE 152 mm Solid Stem Augers			COMPILED BY AK/KN											
DATUM Geodetic			DATE November 20, 2017			CHECKED BY AP											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
102.6	GROUND SURFACE							20	40	60	80	100					
0.0	ASPHALT (110 mm)																
0.1	Gravelly sand, some silt (FILL)		1	SS	31												
102.0	Dense Brown Moist																
0.6	Clayey silt with sand, trace to some gravel (FILL)		2	SS	16												
	Very stiff Brown Moist																
			3	SS	19												
100.5																	
2.1	SILT and SAND, trace to some clay, trace gravel (TILL)		4	SS	59												
	Very dense Brown, oxidation staining Moist																
			5	SS	68												
98.6																	
4.0	SAND and GRAVEL, some silt Dense Brown and grey Moist to wet		6	SS	40												
	- Wet below a depth of 6.1 m (Elev. 96.5 m)		7	SS	38												
94.5	- 50 mm SILTY CLAY layer at 8.0 m depth (Elev. 94.6 m)		8	SS	41												
8.1	SILT and SAND, trace to some clay, trace to some gravel (TILL)																
	Very dense Grey Moist																
			9	SS	118												
91.8																	
10.8	END OF BOREHOLE		10	SS	100/0.10												
NOTES:																	
1. Water level measured at a depth of 10.1 m below ground surface (Elev. 92.5 m) upon completion of drilling.																	
2. Borehole caved to a depth of 6.4 m below ground surface (Elev. 96.2 m) upon removal of augers.																	

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PROJECT		1662582 (2000)		RECORD OF BOREHOLE No AS-9		SHEET 1 OF 1		METRIC										
G.W.P.		2298-13-00		LOCATION		N 4860416.0; E 356693.3 MTM NAD 83 ZONE 10 (LAT. 43.881901; LONG. -78.854220)		ORIGINATED BY LP										
DIST		Central HWY 401		BOREHOLE TYPE		216 mm O.D Hollow Stem Augers		COMPILED BY AK/KN										
DATUM		Geodetic		DATE		November 12, 2018 and December 18, 2018		CHECKED BY AP										
SOIL PROFILE			SAMPLES			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	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ	GR SA SI CL
								20 40 60 80 100	20 40 60 80 100	W <sub>p</sub>	W	W <sub>L</sub>	10 20 30					
96.0		GROUND SURFACE																
0.0		ASPHALT (150 mm)																
95.5		Sand and gravel (FILL)																
0.5		CONCRETE (280 mm)																
		Sand and gravel, trace silt, trace clay (FILL)																
		Grey to brown																
		Dry to wet																
94.5		CONCRETE (STORM SEWER)																
1.6		VOID																
93.9																		
2.1		END OF BOREHOLE																
		NOTES: 1. MTO Storm Sewer encountered and penetrated at a depth of 1.5 m below ground surface (Elev. 94.5 m). Upon penetration of sewer Borehole AS-9 was terminated and augers were buried until repair operators could commence. On Dec. 18, 2018, augers were removed, sewer roof was blocked with a rubber seal and borehole was backfilled with concrete and sand. 2. Borehole AS-9A was drilled approximately 2 m south of Borehole AS-9 (Refer to Record of Borehole AS-9B).																

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
PROJECT 1662582 (2000)		RECORD OF BOREHOLE No AS-9B		SHEET 1 OF 3		METRIC							
G.W.P. 2298-13-00		LOCATION N 4860414.6; E 356693.8 MTM NAD 83 ZONE 10 (LAT. 43.881888; LONG. -78.854214)		ORIGINATED BY KN/AK									
DIST Central HWY 401		BOREHOLE TYPE 216 mm O.D Hollow Stem Augers, P-Casing Advancement		COMPILED BY ACK									
DATUM Geodetic		DATE December 17 to 19, 2018		CHECKED BY AMP									
SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20 40 60 80 100	20 40 60 80 100	W <sub>p</sub>	W		
96.0	GROUND SURFACE												
0.0	ASPHALT (150 mm)												
95.5	SAND and GRAVEL (FILL)												
0.5	CONCRETE (280 mm)												
	Sand and gravel, trace silt, trace clay (FILL)												
	Grey to brown												
	Dry to wet												
93.7													
2.3	Sandy SILT to SAND, trace clay												
	Grey												
	Moist to wet												
	- Wet at about 4.6 m depth (Elev. 91.4 m)												
	- Flowing sands encountered at a depth of 6.1 m (Elev. 89.9 m)												
87.5													
8.5	CLAYEY SILT with SAND, trace to some gravel (TILL)												
	Very stiff to hard												
	Grey												
	Moist												
			1	SS	22								5 37 34 24
			2	SS	64								
84.1													
11.9	SHALE (BEDROCK)												
	Bedrock cored from a depth of 12.1 m to 15.3 m												RQD = 61%
	For bedrock coring details, refer to Record of Drillhole No. AS-9B												RQD = 100%
			3	SS	75/0.05								
			1	RC	REC 100%								
			2	RC	REC 100%								
			3	RC	REC 100%								RQD = 100%

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

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PROJECT		1662582 (2000)		RECORD OF BOREHOLE No AS-9B		SHEET 2 OF 3		METRIC											
G.W.P.		2298-13-00		LOCATION		N 4860414.6; E 356693.8 MTM NAD 83 ZONE 10 (LAT. 43.881888; LONG. -78.854214)		ORIGINATED BY											
DIST		Central HWY 401		BOREHOLE TYPE		216 mm O.D Hollow Stem Augers, P-Casing Advancement		COMPILED BY											
DATUM		Geodetic		DATE		December 17 to 19, 2018		CHECKED BY											
AMP																			
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20	40	60	80	100	W <sub>p</sub>	W			W <sub>L</sub>	GR	SA
80.7 15.3	SHALE (BEDROCK) END OF BOREHOLE  NOTES: 1. Borehole advanced to a depth of 9.1 m below ground surface (Elev. 86.9 m), on December 17, 2018 using hollow stem augers. At 9.1 m depth, heaving sand/silt was noted inside augers to a depth of 6.1 m below ground surface (Elev. 89.9 m).  2. Borehole advancement recommenced using P-Casing advancement on December 18, 2018 from a depth of 9.1 m to 11.9 m below ground surface (Elev. 86.4 m to 84.1 m).		3	RC															

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PROJECT: 1662582 (2000)

**RECORD OF DRILLHOLE: AS-9B**

SHEET 3 OF 3

LOCATION: N 4860414.6 ;E 356693.8

DRILLING DATE: January 7, 2019

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55 Truck

DRILLING CONTRACTOR: Geo-Environmental Drilling

DEPTH SCALE METRES	DRILLING RECORD		DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	JN - Joint FLT - Fault SHR- Shear VN - Vein CJ - Conjugate BD- Bedding FO- Foliation CO- Contact OR- Orthogonal CL - Cleavage PL - Planar CU- Curved UN- Undulating ST - Stepped IR - Irregular PO- Polished K - Slickensided SM- Smooth Ro - Rough MB- Mechanical Break BR - Broken Rock NOTE: For additional abbreviations refer to list of abbreviations & symbols.																FEATURES	R/O/R1 ZONES	NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																										
							RECOVERY			FRACT. INDEX PER 0.25 m by core	DISCONTINUITY DATA						ROCK STRENGTH INDEX		WEATH- ERING INDEX																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																
							TOTAL CORE % R Q D %	SOLID CORE % R Q D %	R.Q.D. %		B Angle 0 to 90 N	DIP w.r.t. CORE AXIS 0 to 90 N	TYPE AND SURFACE DESCRIPTION	Jr	Ja	R4	R3	R2	R1	W1	W2	W3				W4	W5	W6																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																							
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## FEATURES LEGEND



BROKEN CORE



CLAY SEAM



LIMESTONE



LOST CORE

DEPTH SCALE

1 : 50

**GOLDER**LOGGED: KN  
CHECKED: ACK

GTA-RCK 054 S:\CLIENTS\MT01HWY\_401\_0SHAWA02\_DATA\GINT\HWY\_401\_0SHAWA.GPJ GAL-MISS.GDT 09/10/19



**APPENDIX B**

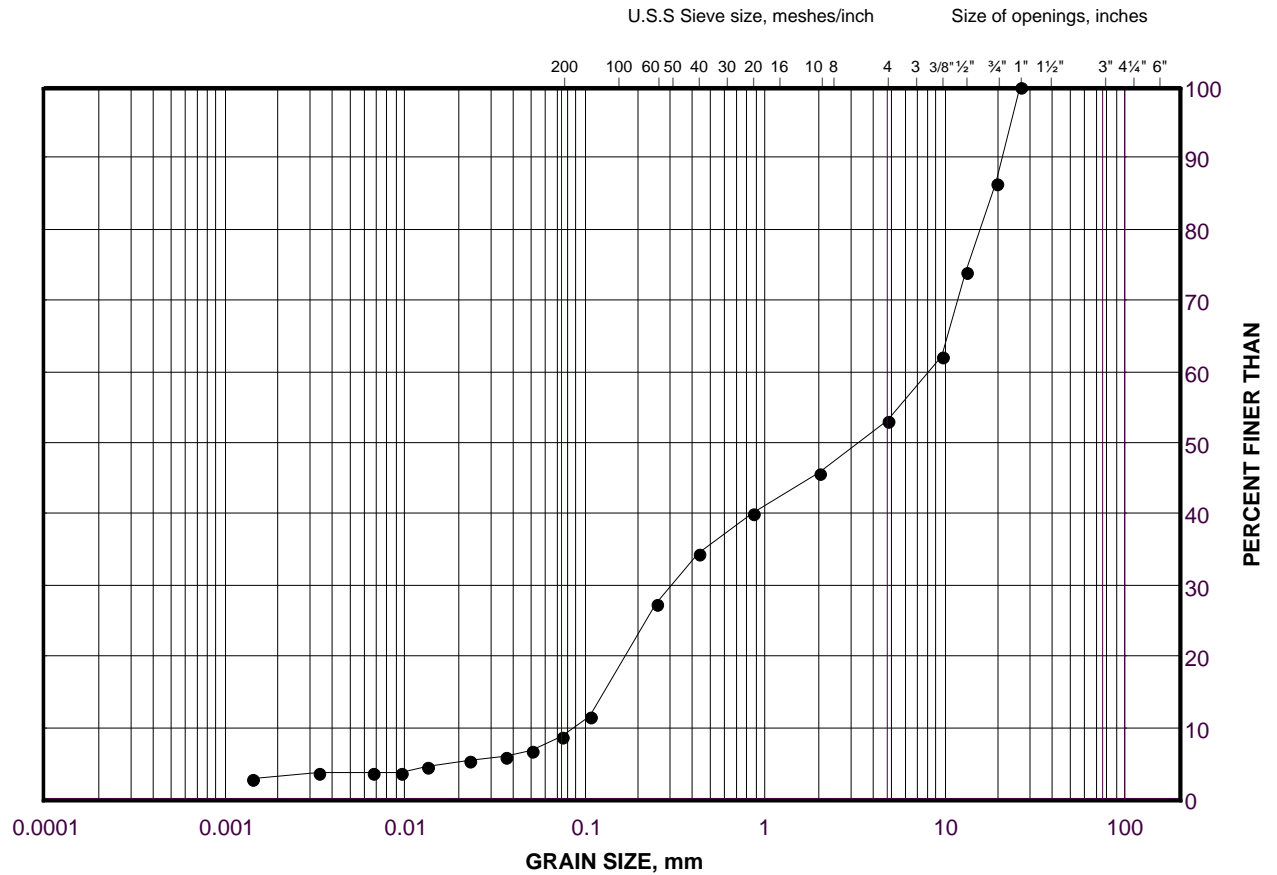
# Geotechnical Laboratory Test Results



# GRAIN SIZE DISTRIBUTION

Sand and Gravel (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)
•	AS-6	3	94.2

Project Number: 1662582

Checked By: AMP

**Golder Associates**

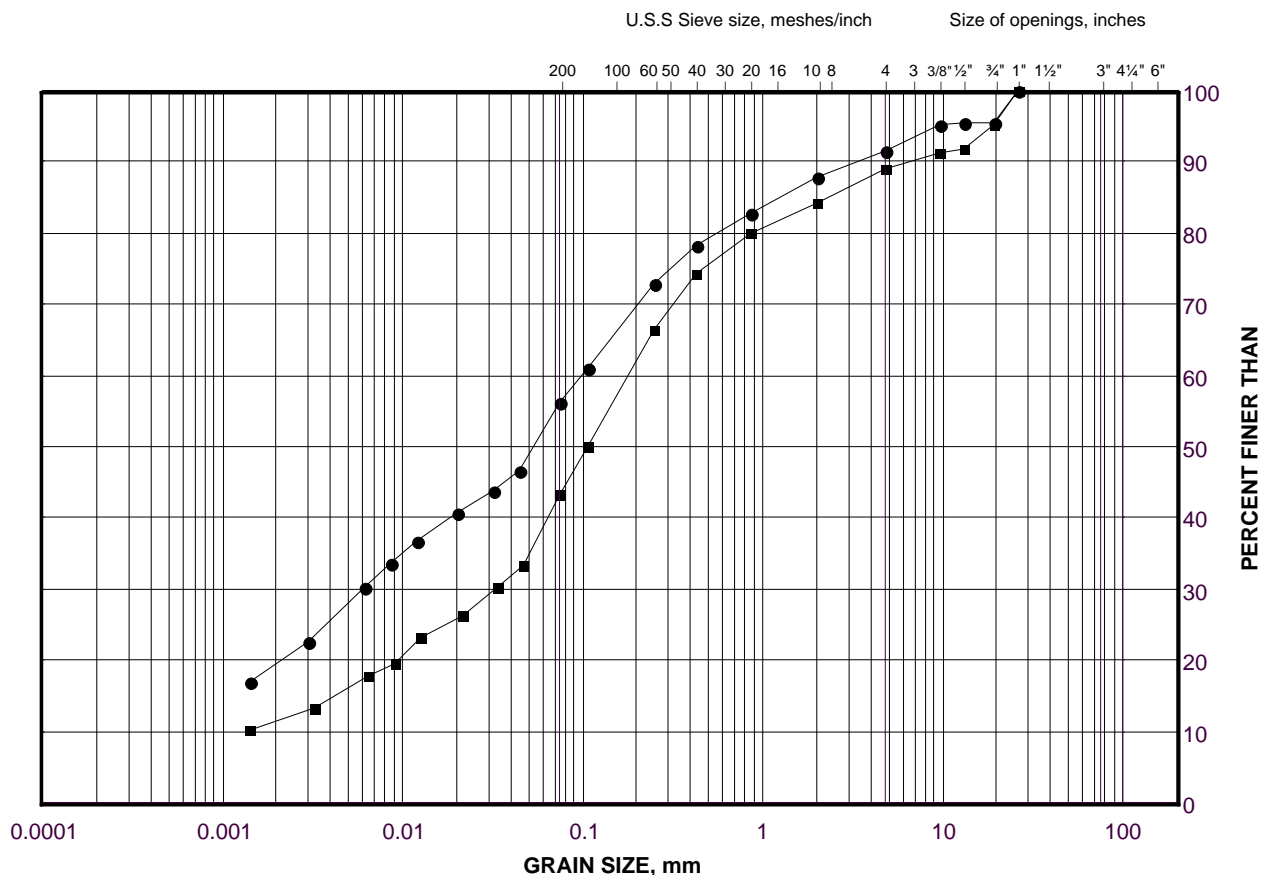
Date: 20-Feb-19



# GRAIN SIZE DISTRIBUTION

Clayey Silt with Sand (FILL)

FIGURE B2



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

## LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	AS-8	3	100.8
■	AS-1	3	100.1

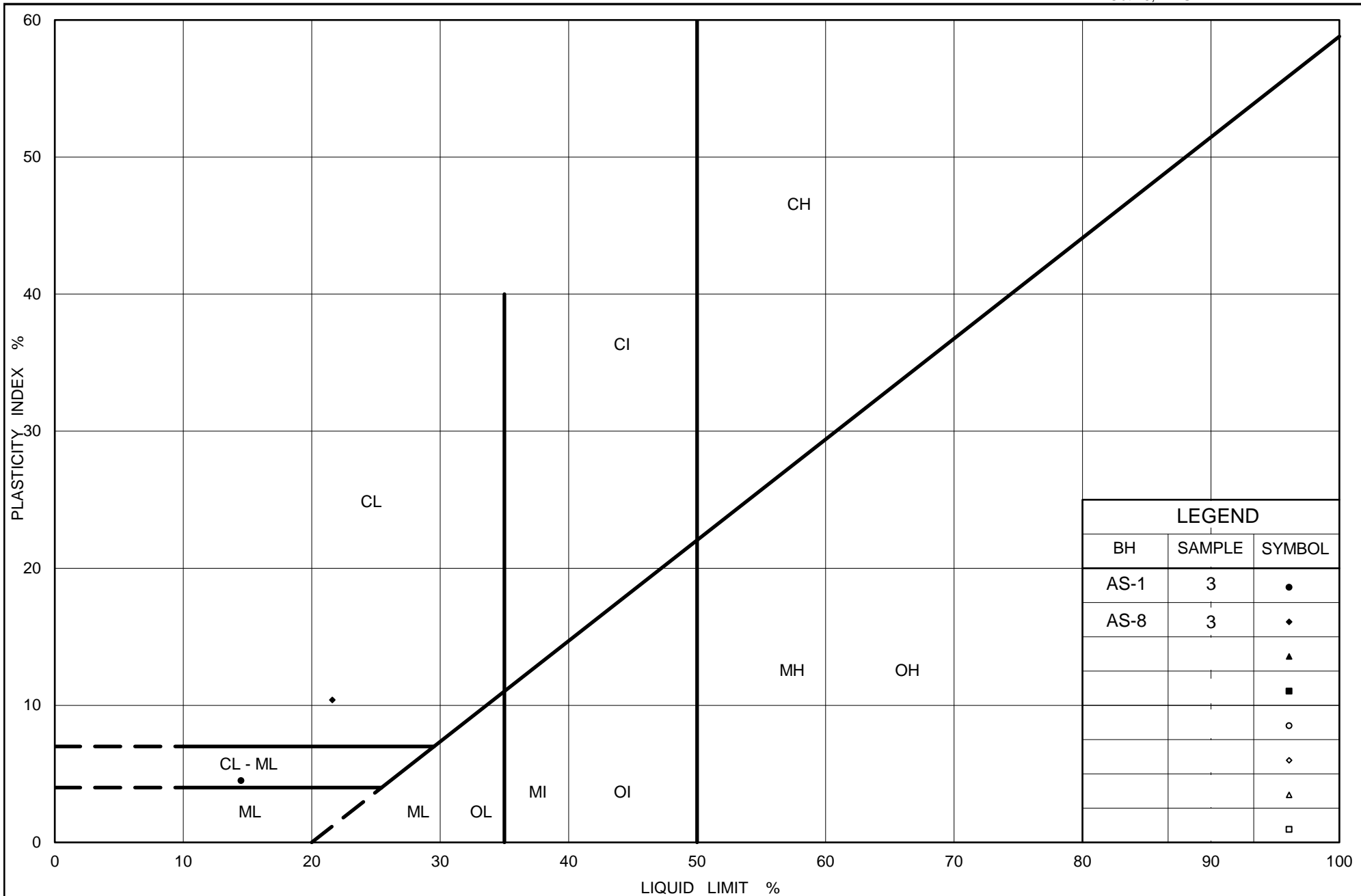
Project Number: 1662582

Checked By: AMP

**Golder Associates**

Date: 20-Feb-19





Ministry of Transportation

Ontario

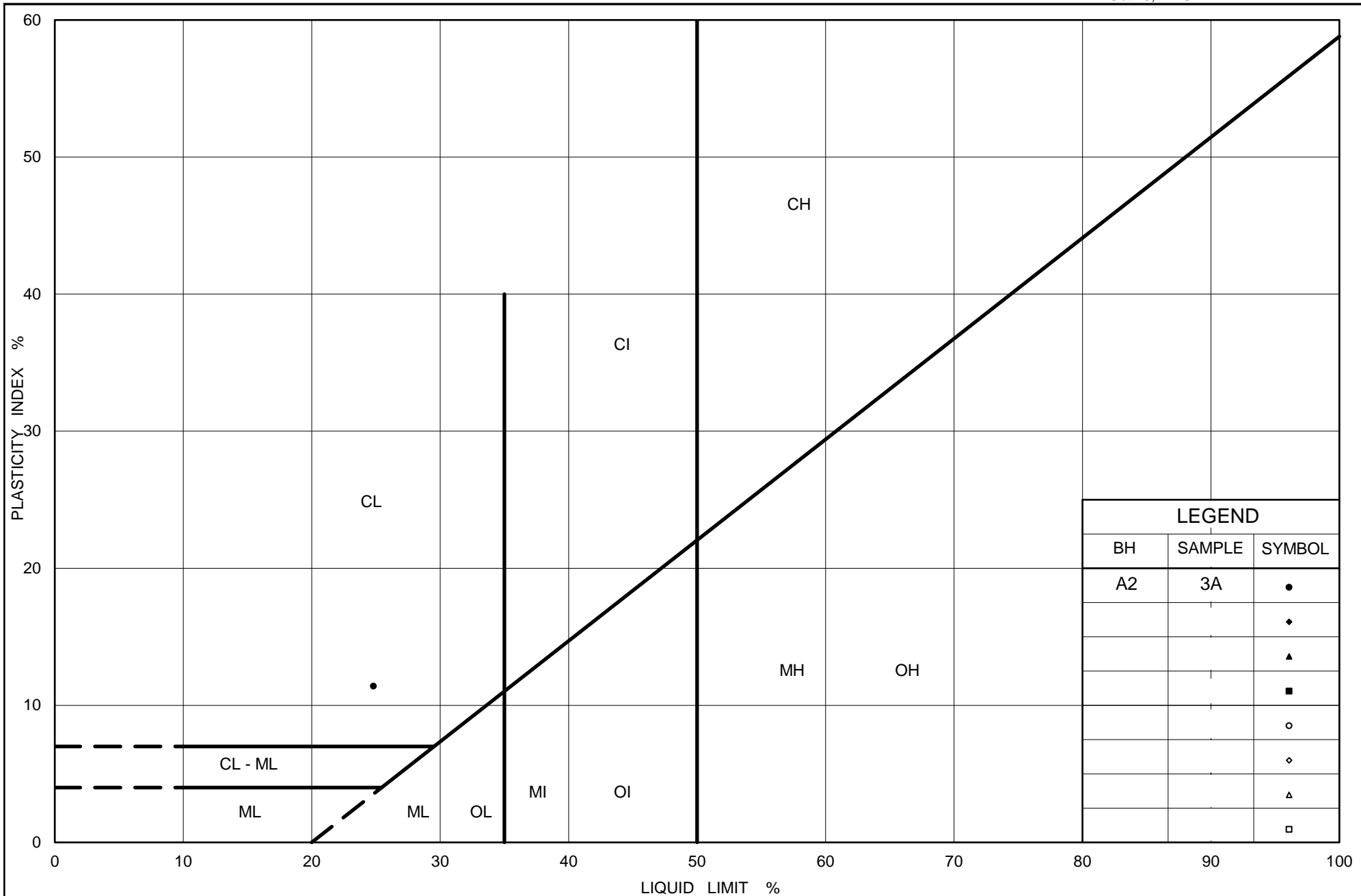
# PLASTICITY CHART Clayey Silt with Sand (FILL)

Figure No. B3

Project No. 1662585 (2000)

Checked By: AMP





Ministry of Transportation

Ontario

## PLASTICITY CHART

### Clayey Silt

Figure No. B4

Project No. 1662582 (2000)

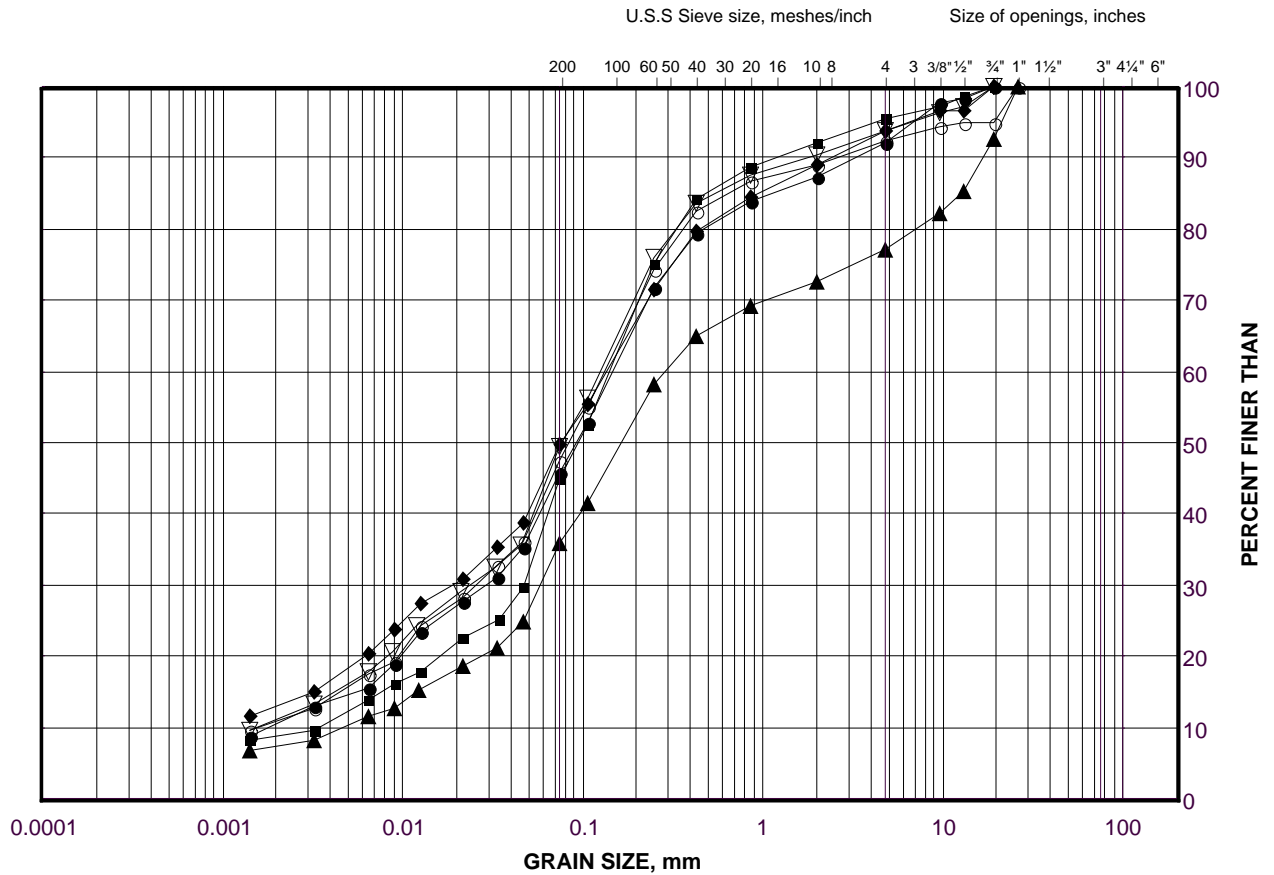
Checked By: AMP



# GRAIN SIZE DISTRIBUTION

SILT and SAND to Gravelly 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)
●	A1	4	99.6
■	AS-8	4	99.3
◆	A2	5	98.7
▲	AS-7	5	98.8
▽	AS-2	5	98.4
○	AS-5	5	98.4

Project Number: 1662582

Checked By: AMP

**Golder Associates**

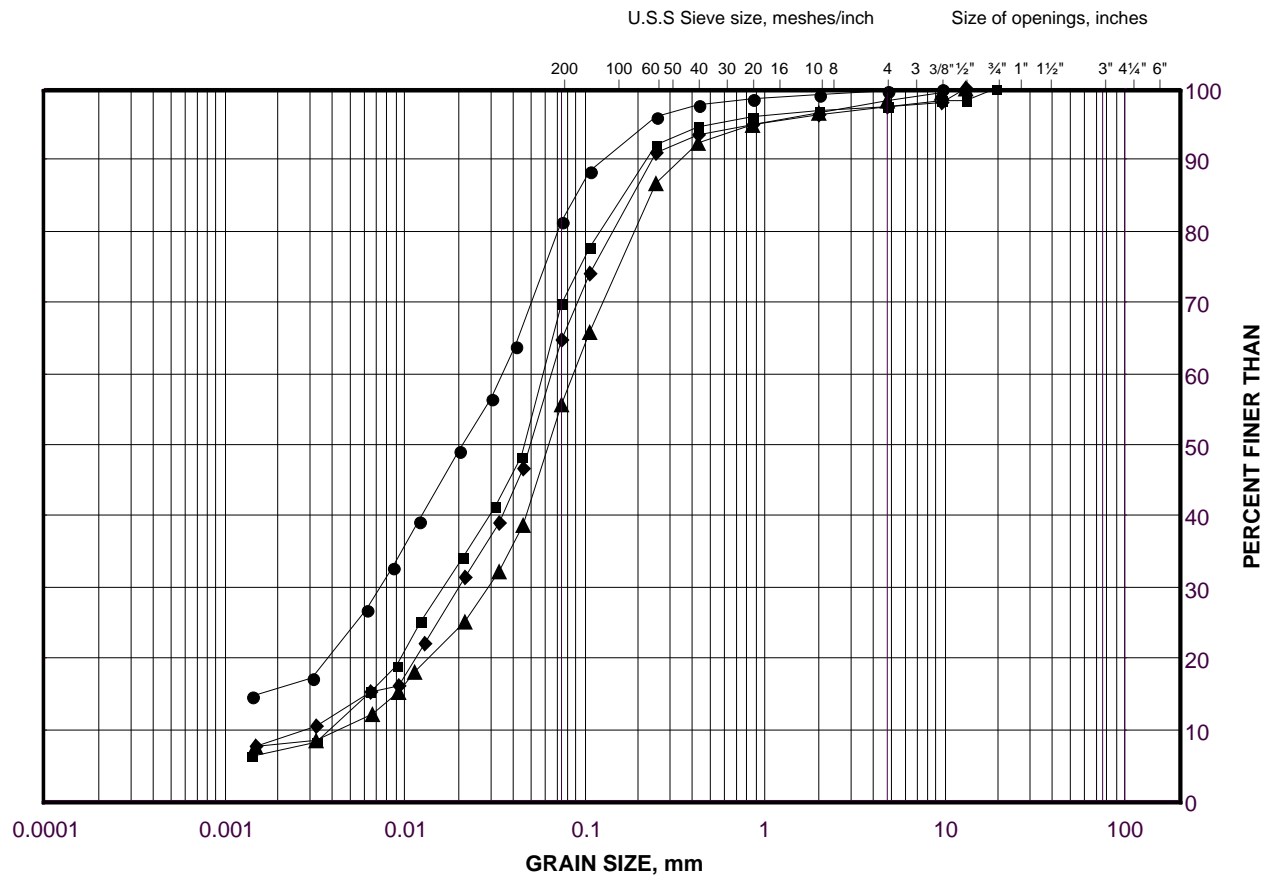
Date: 20-Feb-19



# GRAIN SIZE DISTRIBUTION

SILT to SILT and SAND

FIGURE B6A



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

## LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	AS-1	6	97.0
■	AS-4	7	95.0
◆	A2	8	94.2
▲	AS-5	9	93.2

Project Number: 1662582

Checked By: AMP

**Golder Associates**

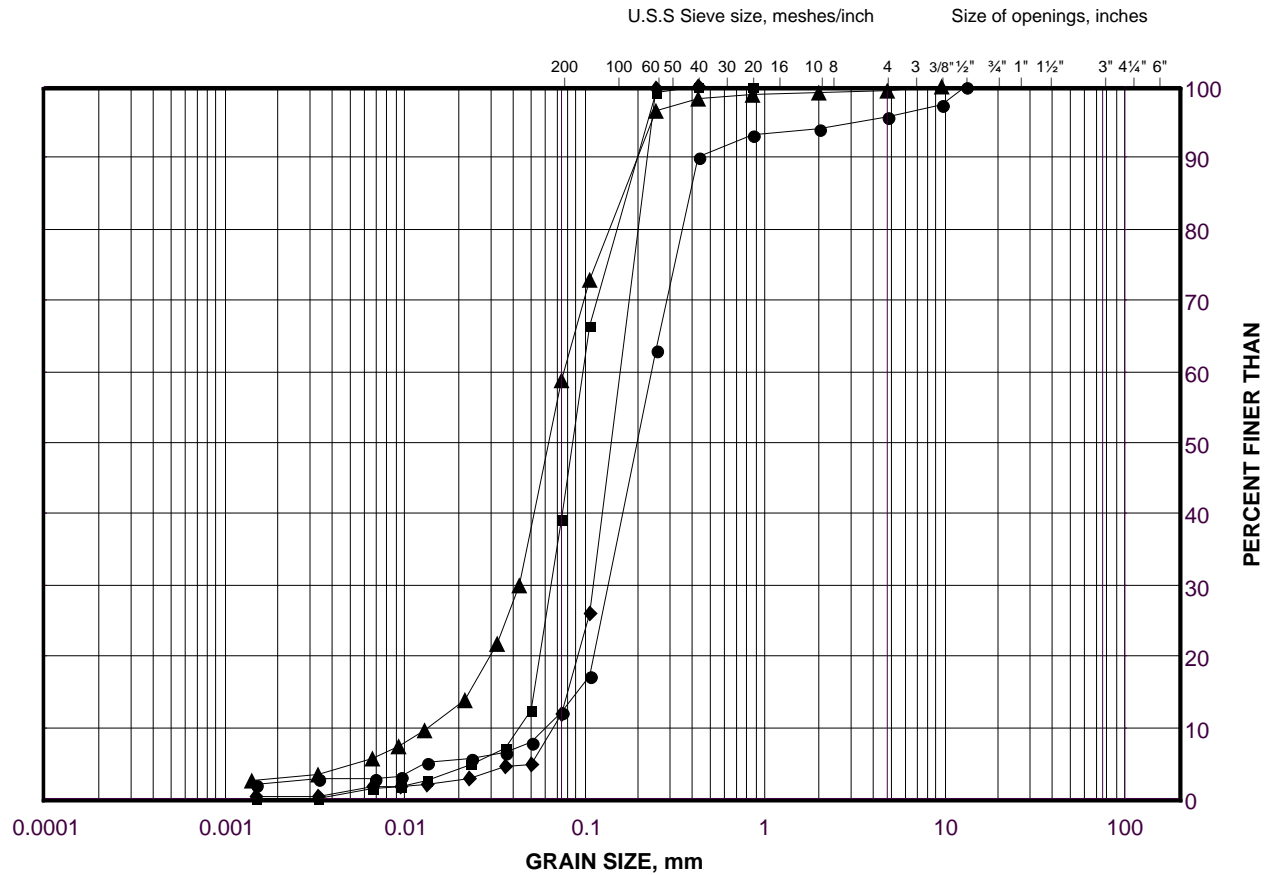
Date: 20-Feb-19



# GRAIN SIZE DISTRIBUTION

SILT and SAND to SAND

FIGURE B6B



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

## LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	A1	7	95.8
■	AS-2	9	92.3
◆	AS-4	9	92.8
▲	AS-1	9	92.6

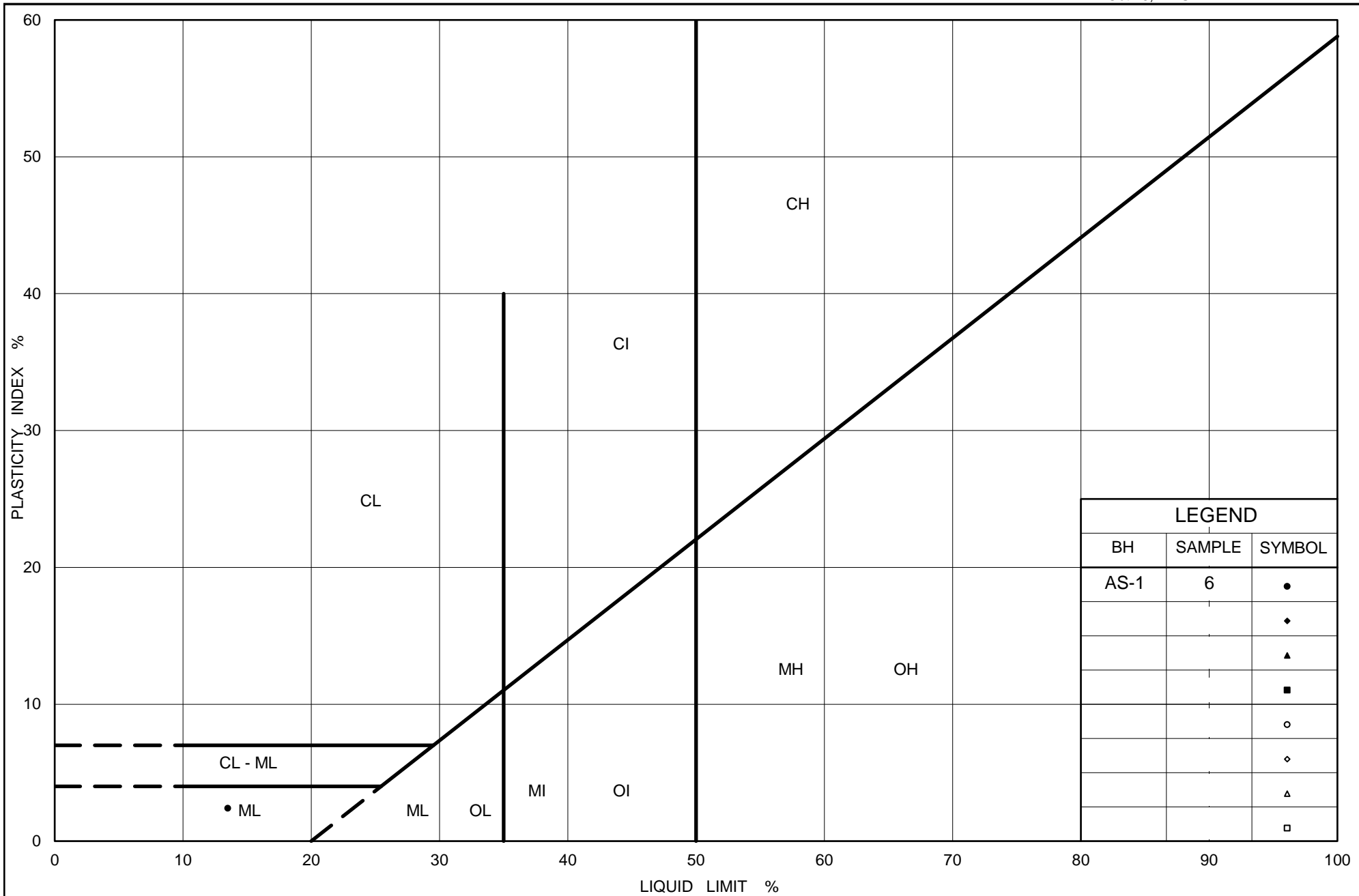
Project Number: 1662582

Checked By: AMP

**Golder Associates**

Date: 20-Feb-19





Ministry of Transportation

Ontario

# PLASTICITY CHART SILT

Figure No. Figure B7

Project No. 1662582 (2000)

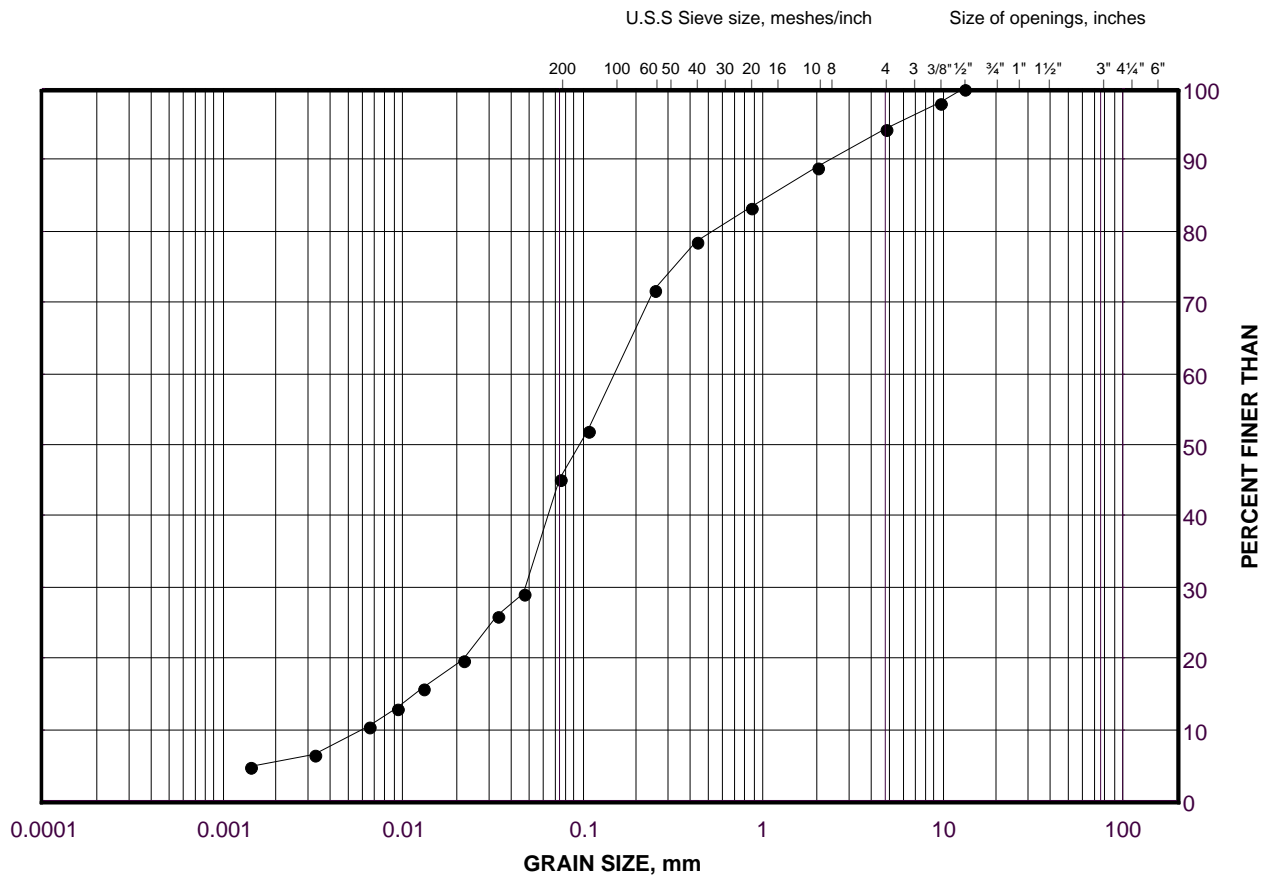
Checked By: AMP



# GRAIN SIZE DISTRIBUTION

## SILT and SAND (TILL) - Lower Deposit

FIGURE B8



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

### LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
•	AS-8	9	93.2

Project Number: 1662582

Checked By: AMP

**Golder Associates**

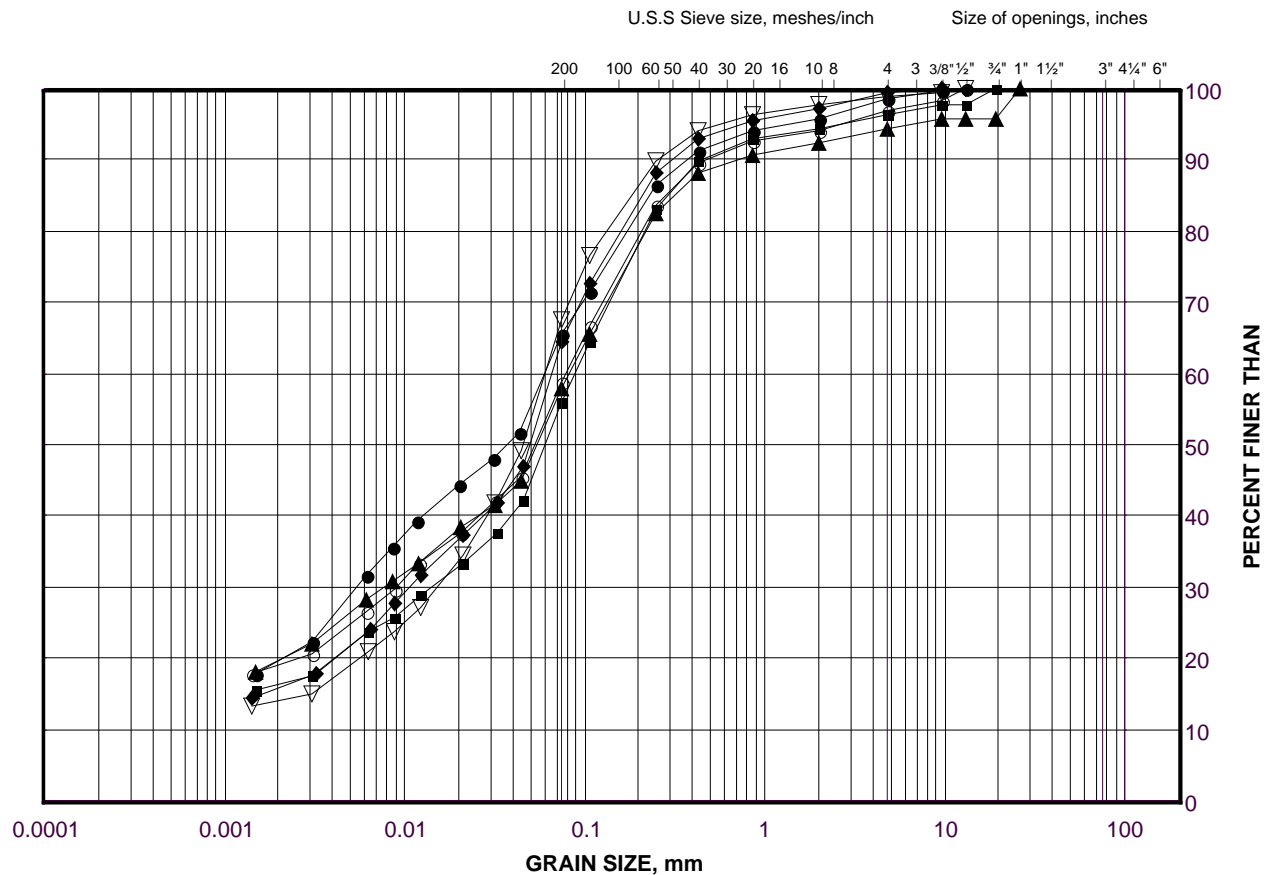
Date: 20-Feb-19



# GRAIN SIZE DISTRIBUTION

CLAYEY SILT with SAND (TILL)

FIGURE B9A



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

## LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	AS-7	11	91.2
■	AS-2	12	87.7
◆	AS-5	12	90.7
▲	AS-7	13	88.2
▽	AS-4	13	88.9
○	AS-5	15	86.2

Project Number: 1662582

Checked By: AMP

**Golder Associates**

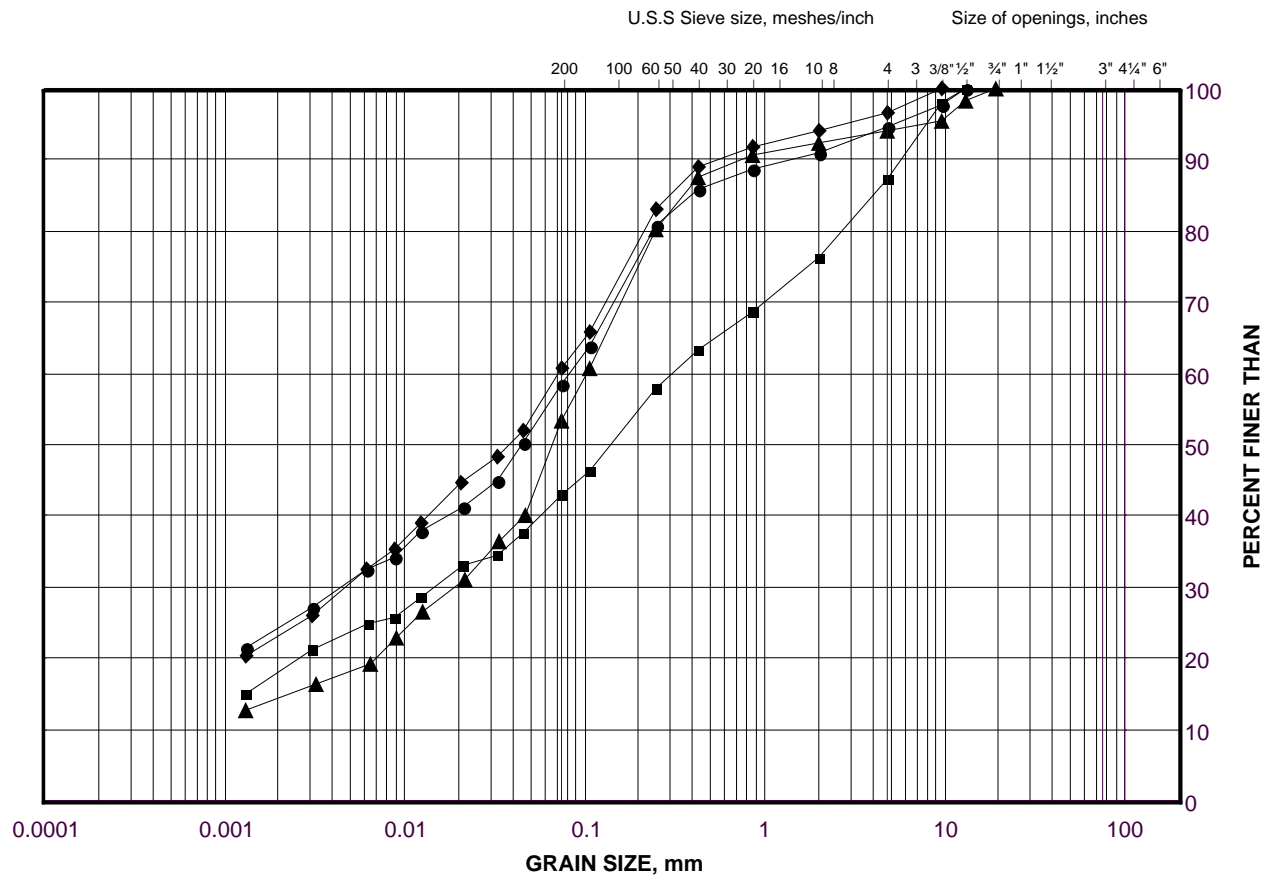
Date: 20-Feb-19



# GRAIN SIZE DISTRIBUTION

CLAYEY SILT with SAND

FIGURE B9B



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

## LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	AS-9B	1	86.7
■	AS-6B	10	83.9
◆	AS-6B	4	93.4
▲	AS-6B	7	89.6

Project Number: 1662582

Checked By: AMP

**Golder Associates**

Date: 20-Feb-19







**APPENDIX C**

# Analytical Test Results



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

**Attention: Al Varshoi**

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

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

**CERTIFICATE OF ANALYSIS**

**MAXXAM JOB #: B845291**

**Received: 2018/02/28, 09:39**

Sample Matrix: Soil  
# Samples Received: 2

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

**Remarks:**

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

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

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

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

Results relate to samples tested.

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

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

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



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

**Attention: Al Varshoi**

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

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

**CERTIFICATE OF ANALYSIS**

**MAXXAM JOB #: B845291**

**Received: 2018/02/28, 09:39**

**Encryption Key**

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

Ema Gitej, Senior Project Manager

Email: EGitej@maxxam.ca

Phone# (905)817-5829

=====

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



### RESULTS OF ANALYSES OF SOIL

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



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

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

## TEST SUMMARY

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

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

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

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

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

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



### GENERAL COMMENTS

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

Package 1	6.3°C
-----------	-------

Samples submitted and analyzed past the recommended hold time.

**Results relate only to the items tested.**



## QUALITY ASSURANCE REPORT

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

QC Batch	Parameter	Date	Matrix Spike		SPIKED BLANK		Method Blank		RPD	
			% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
5423384	Available (CaCl <sub>2</sub> ) 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 <sub>4</sub> )	2018/03/05	NC	70 - 130	98	70 - 130	<20	ug/g	4.7	35
5427146	Conductivity	2018/03/06			98	90 - 110	<2	umho/cm	0.78	10

N/A = Not Applicable

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

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

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

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

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



### VALIDATION SIGNATURE PAGE

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



Brad Newman, Scientific Service Specialist

---

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



**CHAIN OF CUSTODY RECORD** 107484 Page 1 of 1

<b>Invoice Information</b> Company Name: <u>Golder Associates Ltd.</u> Contact Name: <u>Al Varshoi</u> Address: <u>8925 Century Ave. #100</u> <u>Mississauga ON</u> Phone: <u>905-567-4444</u> Email: <u>Al.Varshoi@golder.com</u>		<b>Report Information (if differs from invoice)</b> Company Name: _____ Contact Name: _____ Address: _____ Phone: _____ Fax: _____ Email: _____		<b>Project Information (where applicable)</b> Quotation #: _____ P.O. #/ A/E #: _____ Project #: <u>10602582</u> Site Location: <u>OSHAWA/HW40</u> Site #: _____ Sampled By: <u>LP</u>		<b>Turnaround Time (TAT) Required</b> <input checked="" type="checkbox"/> Regular TAT (5-7 days) Most analyses PLEASE PROVIDE ADVANCE NOTICE FOR RUSH PROJECTS Rush TAT (Surcharges will be applied) <input type="checkbox"/> 1 Day <input type="checkbox"/> 2 Days <input type="checkbox"/> 3-4 Days Date Required: _____ Rush Confirmation #: _____	
MOE REGULATED DRINKING WATER OR WATER INTENDED FOR HUMAN CONSUMPTION MUST BE SUBMITTED ON THE MAXXAM DRINKING WATER CHAIN OF CUSTODY							
<b>Regulation 153</b> <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		<b>Other Regulations</b> <input type="checkbox"/> CCME <input type="checkbox"/> Sanitary Sewer Bylaw <input type="checkbox"/> MISA <input type="checkbox"/> Storm Sewer Bylaw <input type="checkbox"/> PWQJ <input type="checkbox"/> Region <input type="checkbox"/> Other (Specify) _____ <input type="checkbox"/> REG 558 (MIN. 3 DAY TAT REQUIRED)		<b>Analysis Requested</b> # OF CONTAINERS SUBMITTED PREL FILTERED (CIRCLE) Metals / Hg / Cu BTEX / PHE F1 PHES F2 - F4 VOCs REG 153 METALS & INORGANICS REG 153 ICPMS METALS REG 153 METALS (Hg, Cd, V, ICPMS Metals, HWS - B) <u>Corrosivity</u> HOLD - DO NOT ANALYZE		<b>LABORATORY USE ONLY</b> CUSTODY SEAL Y <input checked="" type="checkbox"/> N Present Intact COOLER TEMPERATURES <u>6/6/7</u> COOLING MEDIA PRESENT: Y <input checked="" type="checkbox"/> N COMMENTS	
Include Criteria on Certificate of Analysis: Y / N SAMPLES MUST BE KEPT COOL (< 10 °C) FROM TIME OF SAMPLING UNTIL DELIVERY TO MAXXAM							
<b>SAMPLE IDENTIFICATION</b>		<b>DATE SAMPLED</b> (YYYY/MM/DD)	<b>TIME SAMPLED</b> (HH:MM)	<b>MATRIX</b>	<b># OF CONTAINERS SUBMITTED</b>	<b>PREL FILTERED (CIRCLE) Metals / Hg / Cu</b>	<b>HOLD - DO NOT ANALYZE</b>
1	A55-SAB	2017/11/06	AM	SOIL	1		
2	A57-SAB	2017/11/16	AM	SOIL	1		
3							
4							
5							
6							
7							
8							
9							
10							
<b>RELINQUISHED BY (Signature/Print)</b> <u>Kate Katik</u>		<b>DATE:</b> (YYYY/MM/DD) <u>2018/02/28</u>	<b>TIME:</b> (HH:MM) <u>9:38</u>	<b>RECEIVED BY: (Signature/Print)</b> <u>Matthew Tauran...</u>		<b>DATE:</b> (YYYY/MM/DD) <u>2018/02/28</u>	<b>TIME:</b> (HH:MM) <u>09:39</u>

Unless otherwise agreed to in writing, work submitted on this Chain of Custody is subject to Maxxam's standard Terms and Conditions. Signing of this Chain of Custody document is acknowledged available for viewing at [www.maxxam.ca/terms](http://www.maxxam.ca/terms). Sample container, preservation, hold time and packages information can be viewed at <http://www.maxxam.ca/wp-content/uploads/Ontario-COC.pdf>.

COC-1004 (03/17)

28-Feb-18 09:39

Ema Gitej

B845291

TLI ENV-410

White: Maxxam - Yellow: Client



**APPENDIX D**

# Non-Standard Special Provisions



## **DEWATERING STRUCTURE EXCAVATIONS – Item No.**

---

Special Provision No. FOUN0003

---

### **Amendment to OPSS 902, November 2010**

#### **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.

**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 5 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 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.

#### **902.04.02.02 Working Drawings**

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

## **902.07 CONSTRUCTION**

### **902.07.04 Dewatering Structure Excavation**

Subsection 902.07.04 of OPSS 902 is amended by the addition of the following clauses:

#### **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.



## **WORKING SLAB - Item No.**

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### **Special Provision**

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#### **1.0 SCOPE**

This Special Provision covers the requirements for the supply and placement of a concrete working slab under structure foundations.

#### **2.0 REFERENCES**

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

##### **Ontario Provincial Standard Specifications, Construction**

OPSS 902      Excavating and Backfilling - Structures

#### **3.0 DEFINITIONS - Not Used**

#### **4.0 DESIGN AND SUBMISSION REQUIREMENTS - Not Used**

#### **5.0 MATERIALS**

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

#### **6.0 EQUIPMENT - Not Used**

#### **7.0 CONSTRUCTION**

##### **7.01 Excavation**

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

##### **7.02 Protection of Founding Soil**

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

##### **7.03 Protection of Founding Bedrock**

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

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

##### **7.04 Dewatering**

Dewatering shall be carried out according to OPSS 902.



**8.0** **QUALITY ASSURANCE - Not Used**

**9.0** **MEASUREMENT FOR PAYMENT - Not Used**

**10.0** **BASIS OF PAYMENT**

**10.01** **Working Slab - Item**

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



## **FLOWING SAND CONDITIONS DURING CAISSON INSTALLATION**

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Non-Standard Special Provision

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The contractor is advised that the caissons at the north pier may experience flowing sand conditions when advancing extend through non-cohesive soils under the groundwater table, and therefore soil sloughing, base instability as well as ground loss may be encountered. The construction methods and techniques shall be the responsibility of the Contractor; however, a temporary liner and bentonite slurry should be considered 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 should be considered.



**OBSTRUCTIONS – Item No.**

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Non-Standard Special Provision

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The Contactor shall be alerted to the potential presence of cobbles and boulders within the glacial till deposits at this site as inferred from auger grinding during advancement of Borehole AS-7. 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 shaft piles (caissons).



## **VIBRATION MONITORING - Item No.**

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Non-Standard Special Provision

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#### **1.0 SCOPE**

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

- Deep foundation installation for the Albert Street Underpass replacement structure.

#### **2.0 REFERENCES**

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

1. Foundation Investigation Report; Simcoe Street Underpass (Site No. 22-176), Replacement of Three Underpasses and Rehabilitation of Oshawa Creek Bridge, Highway 401, Oshawa, Ontario, MTO GWP 2298-13-00.

#### **3.0 DEFINITIONS**

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

**Contractor's Engineer** means an Engineer with a minimum of five (5) years of experience in the field of installation of piling and vibration monitoring or, alternatively, with expertise demonstrated by providing



satisfactory quality verification services for a minimum of two (2) projects of similar scope to the Contract. The Contractor's Engineer shall be retained by the Contractor to ensure general conformance with the Contract Documents and issue certificates of conformance.

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

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

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

#### **4.0 DESIGN AND SUBMISSION REQUIREMENTS**

##### **4.1 Submission Requirements**

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

- a) Equipment and methods used by the Contractor to perform the work that may cause undue vibration.
- b) Qualifications of vibration monitoring specialist.
- c) Details regarding proposed instrumentation.
- d) Proposed location of instruments adjacent to the on the residences, utilities, wells, or other potentially vibration-sensitive structures within a 100 m radius from the proposed Simcoe Street underpass.
- e) Proposed frequency of readings.
- f) Action plan to be taken to adjust deep foundation installation methods or if readings show vibrations exceeding tolerable levels.

#### **6.0 EQUIPMENT**

##### **6.1 Vibration Monitoring Equipment**

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

#### **7.0 CONSTRUCTION**

##### **7.1 Pre- and Post-Construction Condition Surveys**

A Pre-Construction Condition Survey and Post-Construction Condition Survey shall be prepared for all buildings, utilities, structures, water wells, and facilities within a 100 m radius from the Simcoe Street Underpass replacement.



### **7.1.1 Pre-Construction Condition Surveys**

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

The Pre-Construction Condition Survey, at each structure/well within a 100 m radius from the proposed Simcoe Street Underpass, shall be completed a minimum of two (2) weeks prior to commencement of installation of the deep foundations. Only one Pre-Construction Condition Survey per structure or facility is required to be carried out in advance of deep foundation installation, unless more than six (6) months will elapse between these operations, in which case an interim inspection will be required.

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

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

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

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

### **7.1.2 Post-Construction Condition Surveys**

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

A Post-Construction Condition Survey at each structure within a 100 m radius from the proposed Simcoe Street Underpass, is required within two (2) months of completion of the installation of deep foundations.

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

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

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

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



residence or property, upon request. The report shall confirm that there have been no changes to the property between the Pre-Construction Condition Survey and the Post-Construction Condition Survey as a result of the installation of deep foundations.

## **7.2 Monitoring**

The vibration monitoring equipment shall be placed on the ground surface at radial distances of 25 m, 50 m, and 100 m from the bridge structures toward the receptors (e.g., buildings, sensitive utilities). The Contractor shall take readings continuously during pile driving for the deep foundation elements, and shall immediately notify the Contract Administrator if the vibrations exceed the limits specified herein.

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

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

## **7.3 Records**

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

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

## **10.0 BASIS OF PAYMENT**

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



## **PIEZOMETER DECOMMISSIONING - Item No.**

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Non-Standard Special Provision

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### **Decommissioning**

One standpipe piezometer was installed in Borehole AS-2 as part of the foundation investigation. and the registered owner is the City of Oshawa. The standpipe piezometer has been left in place to allow for monitoring of groundwater level up to construction. General standpipe piezometer information is provided below.

<b>Piezometer Identification</b>	<b>Location (Northing / Easting)</b>	<b>PVC Pipe and Screen diameter / Borehole diameter</b>	<b>Depth (Below Ground Surface) to Tip of Screen / Borehole Depth</b>
AS-2	Northbound lane of Albert Street, 15 m south of existing south abutment.  (4,860,377.8 / 356,697.2)	50 mm / 216 mm	13.2 m / 14.2 m

As part of the construction activities the contractor shall decommission the standpipe piezometer in Borehole AS-2. The abandonment method must satisfy the minimum requirements of Ontario Regulation 903 Wells, as amended under the Ontario Water Resources Act. The contractor shall provide a written record of the decommissioning procedure to the Contract Administrator. The record shall include plugging material used, depth of plugging material and limit of the PVC standpipe/screen removal.

### **Basis of Payment**

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





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