March 13, 2018

FOUNDATION INVESTIGATION AND DESIGN REPORT

HIGHWAY 400 / 6th LINE UNDERPASS (SITE NO. 30-211/1&2) AND HIGH FILL EMBANKMENTS TOWN OF INNISFIL, SIMCOE COUNTY MINISTRY OF TRANSPORTATION, ONTARIO G.W.P. 2289-13-00

Submitted to:
Morrison Hershfield Limited
Suite 300, 125 Commerce Valley Drive West
Markham, Ontario
L3T 7W4

GEOCRES No.: 31D-695

Report No.: 1670268-1
Distribution:

1 Copy + PDF - MTO Central Region
1 Copy + PDF - MTO Foundations Section
1 PDF - Morrison Hershfield Limited
1 PDF - Golder Associates Ltd.

REPORT

Table of Contents

PART A

FOUNDATION INVESTIGATION REPORT
HIGHWAY 400 / 6th LINE UNDERPASS (SITE NO. 30-211/1&2)
AND ASSOCIATED HIGH FILL EMBANKMENTS
TOWN OF INNISFIL, SIMCOE COUNTY, ONTARIO
MINISTRY OF TRANSPORTATION, ONTARIO
G.W.P. 2289-13-00

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Morrison Hershfield Limited (MH) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the proposed reconstruction of Highway 400/6th Line underpass. The new Highway 400/6th Line underpass will be located north of the existing 6th Line overpass structure, in the Town of Innisfil, Simcoe County, Ontario.

The purpose of this investigation is to establish the subsurface soil and groundwater conditions at the proposed structure including the associated approach embankments, by borehole drilling and geotechnical/analytical laboratory testing on selected soil samples.

The Terms of Reference and the scope of work for the foundation investigation are outlined in MTO's Request for Proposal, dated November 30, 2016, which forms part of the Consultant Agreement for Assignment No. 2016-E-0057. The work has been carried out in accordance with Golder's Supplementary Specialty Plan for foundation engineering services for this project, dated May 19, 2017.

2.0 SITE DESCRIPTION

The proposed Highway 400/6th Line underpass is located about 7 km north of the Highway 400/Highway 89 underpass in the Town of Innisfil, Ontario. Highway 400 is oriented in a north-south direction, and 6th Line is oriented in an east-west direction. Highway 400 consists of three northbound and three southbound lanes, while 6th Line consists of one lane in each direction.

It is understood that the Town of Innisfil plans to realign 6th Line to the north of the existing local road alignment, and that a new underpass will be constructed approximately 40 m to 45 m north of the existing 6th Line overpass to accommodate the realignment.

Agricultural fields are located east and west of the proposed underpass site. At the location of the proposed structure site, the Highway 400 grade is at about Elevation 296.6 m and the highway embankment is about 1.5 m high relative to the immediately surrounding natural ground surface. The ground surface elevation within the agricultural fields varies from about Elevation 290 m to 295 m.

3.0 INVESTIGATION PROCEDURES

The field work for the foundation investigation was carried out between October 11 and October 27, 2017 and between January 3 and 19, 2018, during which time a total of 21 boreholes, designated as Boreholes 6UP-01 to 6UP-08, HF-01 to HF-10, and CE-01 to CE-03, were advanced near the location of the structure foundation footprints, approach and high fill embankments, and culvert, as summarized below.

Foundation Element	Relevant Boreholes
High Fill Embankment West of Underpass	HF-01 to HF-04, and CE-01 to CE-03
West Approach Embankment	6UP-01

Foundation Element	Relevant Boreholes
West Abutment	6UP-02 and 6UP-03
Center Pier	6UP-04 and 6UP-05
East Abutment	6UP-06 and 6UP-07
East Approach Embankment	6UP-08
High Fill Embankment East of Underpass	HF-05 to HF-10

The locations of the boreholes are shown on Drawings 1, 3 and 4, and the borehole records are provided in Appendix A. Lists of abbreviations and symbols are also provided in Appendix A to assist in the interpretation of the borehole records.

The field work was carried out using D-90 truck-mounted and D-50 track-mounted drill rigs supplied and operated by Walker Drilling Ltd. of Utopia, Ontario. The boreholes were advanced through the overburden using 203 mm outer diameter hollow stem augers. Soil samples were obtained at 0.75 m, 1.5 m and 3 m intervals of depth, using a 50 mm outer diameter split-spoon sampler driven by an automatic hammer in accordance with the Standard Penetration Test (SPT) procedures outlined in ASTM D1586-08[1].

The groundwater conditions and water levels in the open boreholes were observed during and immediately following drilling operations. A standpipe piezometer was installed in Boreholes 6UP-03, 6UP-06, HF-06, HF-09 and CE-03 to permit monitoring of the groundwater level at the borehole locations. The standpipe piezometers consist of a 50 mm diameter PVC pipe with a slotted screen sealed at a selected depth within the borehole. Details of the piezometer installation and water level readings are presented on the borehole records in Appendix A. All boreholes were backfilled with bentonite upon completion in accordance with Ontario Regulation 903: Wells (as amended).

The field work was observed by a member of Golder's engineering and technical staff, who located the boreholes, arranged for the clearance of underground services including both public and where applicable private locates, observed the drilling, sampling and in situ testing operations, logged the boreholes, and examined the soil samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to Golder's Mississauga geotechnical laboratory where the samples underwent further visual examination and geotechnical laboratory testing. All of the geotechnical laboratory tests were carried out in accordance with MTO and/or ASTM Standards, as appropriate. Classification testing (water content, Atterberg limits and grain size distribution) was carried out on selected soil samples. The results of the geotechnical laboratory testing are included in Appendix B.

Four selected soil samples were submitted, under chain-of-custody procedures, to Maxxam Analytics of Mississauga, Ontario (a Standards Council of Canada (SCC) accredited laboratory) for corrosivity testing. The soil samples were analyzed for a suite of parameters, including conductivity, resistivity, soluble chloride concentration, soluble sulphate concentration and pH. The results of the analytical tests are presented in Appendix B.

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

The borehole locations and ground surface elevations were measured using a GPS unit (Trimble XH 3.5G), having an accuracy of 0.1 m in the vertical and horizontal directions. The locations provided on the borehole records and shown on Drawings 1 to 4 are positioned relative to MTM NAD 83 (Zone 10) coordinates system, and the ground surface elevations are referenced to Geodetic datum. The borehole locations and ground surface elevations and drilled depths are summarized below.

Borehole No.	Location (MTM NAD 83, Zone 10)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (m) (Latitude, °)	Easting (m) (Longitude, °)		
6UP-01	4,902,370.2 (44.261202)	290,897.2 (-79.674120)	293.6	11.3
6UP-02	4,902,380.2 (44.261292)	290,904.6 (-79.674030)	293.6	29.4
6UP-03	4,902,366.1 (44.261165)	290,907.9 (-79.673990)	293.7	29.6
6UP-04	4,902,392.3 (44.261402)	290,945.4 (-79.673518)	296.6	27.7
6UP-05	4,902,380.4 (44.261294)	290,948.1 (-79.673484)	296.6	29.1
6UP-06	4,902,402.4 (44.261493)	290,985.5 (-79.673017)	295.1	23.1
6UP-07	4,902,391.6 (44.261396)	290,988.0 (-79.672985)	295.2	23.3
6UP-08	4,902,398.0 (44.261454)	290,997.8 (-79.672863)	295.2	11.3
HF-01	4,902,295.4 (44.260529)	290,693.0 (-79.676674)	292.7	5.2
HF-02	4,902,323.1 (44.260772)	290,758.3 (-79.675865)	291.8	11.3
HF-03	4,902,332.8 (44.260929)	290,786.1 (-79.675278)	292.0	11.3
HF-04	4,902,353.0 (44.261072)	290,842.7 (-79.674684)	292.8	11.3
HF-05	4,902,417.5 (44.261630)	291,056.3 (-79.672130)	294.6	9.8
HF-06	4,902,429.4 (44.261739)	291,104.7 (-79.671524)	294.0	9.8
HF-07	4,902,438.9 (44.261825)	291,153.6 (-79.670912)	293.0	8.2
HF-08	4,902,446.0 (44.261890)	291,203.0 (-79.670293)	291.5	6.7
HF-09	4,902,451.6 (44.261941)	291,253.0 (-79.669667)	290.3	6.7
HF-10	4,902,471.4 (44.262120)	291,300.2 (-79.669077)	290.1	5.2

Borehole No.	Location (MTM NAD 83, Zone 10)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (m) (Latitude, °)	Easting (m) (Longitude, °)		
CE-01	4,902,340.6 (44.260932)	290,710.9 (-79.676453)	291.3	11.3
CE-02	4,902,291.7 (44.260490)	290,740.0 (-79.676090)	290.0	11.3
CE-03*	4,902,292.1 (44.260504)	290,740.9 (-79.676079)	290.0	5.2

* Purpose of borehole was to install a monitoring well and to confirm the low SPT 'N' value in Borehole CE-02 at a depth of 4.5 m below ground surface.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

This project area is located within the Peterborough Drumlin Field physiographic region, as delineated in *The Physiography of Southern Ontario* (Chapman and Putman, 1894)[2]. The surficial soils in the Peterborough Drumlin Field frequently consist of gravelly sand till or sand and gravel deposits, although clayey silt to silt/sand till deposits are also common in the vicinity of the Highway 400 corridor. Drumlins (glacially-shaped hills comprised of till) are more frequent in the southern portion of the section of the Peterborough Drumlin Field that is traversed by Highway 400. Deposits of silt, clay or peat may be found in the low-lying areas between drumlins. The Peterborough Drumlin Field is underlain at depth by bedrock of the Lindsay and Verulam Formations, which consists mainly of fossiliferous limestone.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions as encountered in the boreholes advanced during the investigation, together with the results of the laboratory tests carried out on selected soil samples, are presented on the borehole records provided in Appendix A. The results of the in-situ field tests (i.e. SPT "N" values) as presented on the borehole records and in sub-sections of Section 4.2 are uncorrected. The geotechnical laboratory testing plots are contained in Appendix B.

The stratigraphic boundaries shown on the borehole records and on the stratigraphic profile and cross sections on Drawings 1 to 4 are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Furthermore, subsurface conditions will vary between and beyond the borehole locations. It should be noted that the interpreted stratigraphy shown on Drawings 1 to 4 is a simplification of the subsurface conditions.

In general, the subsurface conditions consist of a layer of topsoil or pavement structure underlain by fill that varies in composition from sand and gravel to clayey silt with sand to silty clay. At some borehole locations the topsoil is

[2] Chapman, L.J. and Putman, D.F., 1984, The Physiography of Southern Ontario, Ontario Geological Society, Special Volume 2, Third Edition. Accompanied by Map p. 2715, Scale 1:600,000.)

underlain by a surficial deposit of clayey silt with sand or silty sand. At all boreholes, the fill and surficial deposits are underlain by a glacial till deposit, which varies in composition from clayey silt with sand, to silt and sand, to silty gravelly sand. Layers or lenses of clayey silt, to silt, to silty sand to sand were observed throughout the till deposit. The till deposit is underlain by a lower deposit that varies in composition from clayey silt to silt. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Asphalt Pavement

Boreholes 6UP-04 and 6UP-05 were advanced through the west shoulder of the Highway 400 southbound lanes, near the location of the proposed pier. The encountered asphalt is approximately 215 mm to 240 mm thick.

4.2.2 Topsoil

Topsoil was encountered at ground surface in Boreholes 6UP-01, 6UP-02, 6UP-03, 6UP-06, 6UP-07, 6UP-08, HF-01 to HF-10 and CE-01 to CE-03. The topsoil was between approximately 150 mm and 690 mm in thickness. The topsoil was classified based on visual and textural observations; organic content testing was not carried out.

4.2.3 Fill

Non-cohesive fill was encountered underlying the asphalt at Boreholes 6UP-04 and 6UP-05, and underlying the topsoil in Boreholes 6UP-08 and CE-01. This fill is variable in composition and generally consists of silt and sand, to sand, to sand and gravel. The surface of the non-cohesive fill was encountered at Elevation 296.4 m in Boreholes 6UP-04 and 6UP-05, at Elevation 295.1 m in Borehole 6UP-08 and at Elevation 290.6 in Borehole CE-01. In the boreholes advanced on Highway 400, the non-cohesive fill extends to depths of 0.5 m and 4.1 m (Elevation 296.1 m and 292.5 m). In Borehole 6UP-08, advanced near the toe of the existing embankment east of Highway 400, the sand and gravel fill extends to a depth of 1.4 m (Elevation 293.7 m) below the present ground surface, while in Borehole CE-01, the silty sand fill extends to a depth of 2.2 m (Elevation 289.1 m).

Cohesive fill was encountered underlying the non-cohesive fill in Borehole 6UP-04, and underlying the topsoil in Boreholes 6UP-06, 6UP-07, HF-05 and HF-07 to HF-10. The fill consists of sandy clayey silt, and clayey silt to silty clay. The surface of the cohesive fill was encountered between Elevations 296.1 m and 289.9 m, and the fill extends to depths between 0.7 m and 3.7 m (Elevation 294.4 m and 287.9 m) below the ground surface.

The SPT “N” values within the non-cohesive fill range from 9 blows to 46 blows per 0.3 m of penetration, indicating a loose to dense compactness condition. The SPT “N” values measured within the cohesive fill range from 4 blows to 20 blows per 0.3 m of penetration, indicating a firm to very stiff consistency.

The results of grain size distribution tests completed on four samples of the non-cohesive fill are presented on Figure B1 in Appendix B. The silt and sand fill contains trace to some clay and gravel, and the sand and gravel contains some fines. Atterberg limits tests were carried out on the fines portion of two samples of the granular fill, and measured liquid limits of about 13 and 15 percent, plastic limits of about 9 and 11 per cent, and plasticity indices of about 2 and 4 per cent. The results of the Atterberg limits tests are shown on a plasticity chart on Figure B2 in Appendix B, and indicate that the fines portion of the fill can be classified as a silt of slight plasticity.

The results of grain size distribution tests completed on two samples of the cohesive fill are presented on Figure B3 in Appendix B. Atterberg limits testing was carried out on the fines portion of four samples of this cohesive fill and measured liquid limits ranging from about 19 to 45 per cent, plastic limits ranging from about 11 to 19 per cent, and plasticity indices ranging from about 8 to 27 per cent. The results of the Atterberg limits tests are shown on a plasticity chart on Figure B4 in Appendix B, and indicate that the fines portion of this fill can be classified as a clayey silt of low to medium plasticity. The water content measured in the fill material ranges from about 4 to 27 per cent.

4.2.4 Surficial Sandy Silt to Silty Sand and Clayey Silt with Sand

Thin surficial layers of sandy silt to silty sand, and clayey silt with sand were encountered below the topsoil in Boreholes 6UP-01, 6UP-02, 6UP-03, HF-04 and CE-02. The upper surface of these layers was encountered between Elevation 293.3 m and 289.3 m, and where encountered they total approximately 1.0 m to 1.8 m in thickness, with their base between Elevation 292.2 m and 287.8 m at the borehole locations.

SPT “N” values ranging from 3 to 5 blows per 0.3 m of penetration were measured in all of the sandy silt to silty sand layers, indicating a loose relatively density. SPT “N” values of 4 to 14 blows per 0.3 m of penetration were measured in the sandy clayey silt layers, indicating a firm to stiff consistency.

The results of grain size distribution tests completed on four samples of the surficial cohesive deposit are shown on Figure B5 in Appendix B. The clayey silt with sand to clayey silt contains trace to some gravel.

Atterberg limits tests were carried out on the fines portion of four samples of the surficial cohesive deposit, and measured liquid limits ranging from about 18 to 33 per cent, plastic limits ranging from about 10 to 15 per cent, and plasticity indices ranging from about 7 to 19 per cent. The results of the Atterberg limits tests are shown on a plasticity chart on Figure B6 in Appendix B, and indicate that the fines portion of the deposit can be classified as a clayey silt of low plasticity.

4.2.5 Clayey Silt with Sand Till to Silt and Sand to Silty Gravelly Sand Till

An extensive glacial till deposit was encountered underlying the asphalt, topsoil, fill and/or surficial soil layers in all boreholes advanced at the site. The till is variable in composition, grading between low plasticity clayey silt, clayey silt with sand or sandy clayey silt, and silt and sand or silty gravelly sand. The surface of the till deposit was encountered at depths ranging between about 0.3 m and 4.1 m (Elevation 294.4 m and 287.9 m). Where fully penetrated in the boreholes, the till deposit extends to depths of about 20.9 m to 27.7 m (between Elevations 274.2 m and 268.9 m).

The SPT “N” values recorded within the till deposit are variable, ranging from 5 blows per 0.3 m of penetration to 160 blows per 0.29 m of penetration. This suggests a firm to hard consistency within the cohesive till deposit, and loose to very dense compactness condition within the granular till deposit. In general, the lower SPT “N” values (and the firm to stiff or loose portions of the deposit) are found within the upper 1 m to 2 m of the till deposit.

The results of grain size distribution tests completed on 43 samples of the till deposit are shown on Figures B7A to B7G in Appendix B. Auger refusal, likely on cobbles or a boulder, was encountered within the till deposit at a depth of 3 m below ground surface in Borehole 6UP-03 and the borehole was abandoned and re-advanced at a location 1 m to the east. Although obstructions were not encountered and grinding of the augers during drilling was not evident at other boreholes, the till deposits in southern Ontario typically contain such materials and they should be expected within the glacial till deposits.

Atterberg limits tests were carried out on the fines portion of 51 samples of the till deposit, including three samples that were found to be non-plastic. The Atterberg limits tests completed on cohesive samples of this deposit measured liquid limits ranging from about 11 to 23 per cent, plastic limits ranging from about 8 to 12 per cent, and plasticity indices ranging from about 4 to 12 per cent. For the samples of the cohesive till, the results of the

Atterberg limits tests are shown on the plasticity charts on Figures B8A to B8E in Appendix B, and indicate that the fines portion of the deposit can be classified as a clayey silt of low plasticity. For the samples of the "non-cohesive" portions of the till, the results of the Atterberg limits tests are shown on the plasticity charts on Figures B9A and B9B in Appendix B, illustrating that the plasticity index of the tested samples is less than 4 per cent, and indicating that the fines in this portion of the deposit can be classified as a silt of slight plasticity. The natural water content measured on samples of this till deposit range from about 6 to 16 per cent.

4.2.6 Clayey Silt and Sandy Silt to Sand Interlayers Within the Till

Interlayers of clayey silt, sandy silt, silty sand and sand were encountered within the till deposit in many of the boreholes. The interlayers vary in thickness from about 0.2 m to 2.3 m, and these layers occur at variable depths throughout the till deposit. A much thicker interlayer or localized deposit/lens of clayey silt to sand was encountered in Boreholes 6UP-03, CE-01 and CE-02; at these locations, the layer is at least 4.1 m to 7.6 m in thickness. It is noted that additional interlayers of granular soil are likely present throughout the till, but may have not been encountered considering the 1.5 m sampling interval at depth in the boreholes.

The SPT "N" values recorded within the non-cohesive interlayers ranges from 11 blows per 0.3 m of penetration to 173 blows per 0.22 m of penetration, indicating a compact to very dense compactness condition. In Borehole CE-02 the SPT 'N' value recorded within the sand interlayer at a depth of 3.8 m was "weight of hammer"; Borehole CE-03 was advanced adjacent to CE-02 using techniques to counterbalance the water pressures, and the SPT 'N' value recorded at the same depth was 43 blows per 0.3 m, confirming that the low value measured in Borehole CE-02 is the result of sample disturbance due to groundwater inflow to the borehole. An SPT 'N' value of 57 blows per 0.3 m of penetration was recorded in the clayey silt interlayer in Borehole 6UP-03, suggesting a hard consistency.

The results of grain size distribution testing completed on three samples of the clayey silt to silt interlayers are shown on Figure B10. The results of grain size distribution testing completed on nine samples of the silty sand to sand interlayers are shown on Figures B11A and B11B. Atterberg limits testing was carried out on two samples of the fines from a granular interlayer, and confirmed these materials were non-plastic. The natural water content measured on selected samples of the non-cohesive interlayers range from about 8 to 21 per cent. The natural water content measured on the recovered sample of the clayey silt interlayer is about 11 per cent.

4.2.7 Lower Clayey Silt to Silt

A lower deposit of clayey silt to silt was encountered underlying the till deposit in Boreholes 6UP-03 to 6UP-07. This lower deposit varies in composition from clayey silt, to sandy silt, to a silt of slight plasticity. The surface of the deposit was encountered at depths of 22.3 m to 27.7 m (between about Elevations 272.8 m and 266.7 m). All of these boreholes were terminated within this deposit at depths of about 23.1 m to 29.6 m (between Elevation 272.0 m and 264.1 m).

SPT "N" values of 108 blow per 0.3 m of penetration and 100 blows per 0.08 m of penetration were measured within the clayey silt portion of this lower deposit, suggesting a hard consistency. The SPT "N" values measured within the silt to sandy silt portions of this lower deposit range from 176 blows per 0.3 m of penetration to 100 blows per 0.13 m of penetration, suggesting a very dense compactness condition.

Grain size distribution testing carried out on three samples of this lower deposit are shown on Figure B12 in Appendix B. Atterberg limits tests were carried out on the fine portions of five samples of this deposit and measured liquid limits ranging from about 19 to 22 per cent, plastic limits ranging from about 11 to 19 per cent, and plasticity indices ranging from about 2 to 11 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure B13, and indicate that the fines portions of this lower deposit can be classified as a clayey silt of low plasticity to a silt of slight plasticity. The natural water content measured on samples of the lower clayey silt to silt ranges from about 16 to 23 per cent.

4.2.8 Groundwater Conditions

The groundwater levels in the open boreholes were measured upon completion of drilling operations. The details of these measurements are shown on the borehole records contained in Appendix A; however, it is noted that these measurements are not considered to represent the stabilized groundwater level at the site.

While advancing and sampling Borehole 6UP-02 near the west abutment, about 3 m of sand “blew back” inside the hollow stem augers after advancing them to depths of about 14 m and 15.2 m (Elevations 279.6 m and 278.4 m). Subsequently, Borehole 6UP-03 was drilled with the addition of quick-gel to counter-balance the sub-artesian groundwater pressures in the sand layer; although no “blow-back” of sand was observed, difficulties occurred in retrieving the rods and split-spoon sampler, and this is considered to be due to the water pressures in the sand layer. Similarly in Borehole CE-02, sample disturbance occurred due to groundwater inflow to the borehole, when the augers were at a depth of 3.8 m below ground surface.

Standpipe piezometers were installed in Boreholes CE-03, 6UP-03, 6UP-06, and HF-09 (from west to east across the site) to permit monitoring of groundwater levels. Details of the piezometer installations and measured groundwater levels are shown on the borehole records in Appendix A. The measured groundwater levels are summarized below:

Borehole No.	Ground Surface Elevation (m)	Depth to Water Level (m)	Groundwater Elevation (m)	Date (dd/mm/yyyy)	Comments
CE-03	290.0	1.2	288.8	16/01/2018	Upon completion of drilling
		1.3	288.7	09/02/2018	Measured in standpipe piezometer
		1.1	288.9	05/03/2018	
6UP-03	293.7	7.3	286.4	10/01/2018	Upon completion of drilling
		2.2	291.5	09/02/2018	Measured in standpipe piezometer
		1.8	291.9	05/03/2018	
6UP-06	295.1	7.4	287.7	20/10/2017	Upon completion of drilling
		3.5	291.6	03/11/2017	Measured in standpipe piezometer
		3.0	292.1	14/11/2017	
		2.6	292.5	04/12/2017	
		3.0	292.1	10/01/2018	
		2.7	292.4	09/02/2018	
		2.3	292.8	05/03/2018	
HF-09	290.3	0.9	289.4	27/10/2017	Upon completion of drilling

Borehole No.	Ground Surface Elevation (m)	Depth to Water Level (m)	Groundwater Elevation (m)	Date (dd/mm/yyyy)	Comments
		0.5 above ground surface	290.8	03/11/2017	Measured in standpipe piezometer
		0.5 above ground surface	290.8	14/11/2017	
		0.5 above ground surface	290.8	04/12/2017	

It should be noted that the groundwater level in the area is subject to seasonal fluctuations and precipitation events, and should be expected to be higher during wet periods of the year.

4.2.9 Analytical Testing Results

Analytical testing was carried out on selected soil samples recovered from Boreholes 6UP-03, 6UP-05, 6UP-06 and CE-02 (advanced at the proposed west abutment, pier, east abutment, and culvert respectively). The soil samples were submitted to Maxxam Analytics of Mississauga, Ontario for corrosivity testing. Detailed analytical laboratory test results are provided on the Certificate of Analysis presented in Appendix B, and summarized below.

Borehole No.	Sample ID	Depth (m)	Parameters				
			Resistivity (ohm-cm)	Electrical Conductivity (μmho-cm)	Soluble Sulphate (SO_4) Content (μg-g)	Chlorides (CL) Content (μg-g)	pH (pH)
6UP-03	SS5	3.0 – 3.7	1,900	531	25	250	7.97
6UP-05	SS7[1]	4.6 – 5.2	910	1,100	<20[2]	610	7.99
6UP-06	SS4A	2.3 – 2.7	4,700	215	<20	57	7.88
CE-02	SS4	2.3 – 2.7	6,500	153	<20	22	7.79

Note:

1. "SS" refers to a split-spoon sampler used to carry out the soil sampling in the boreholes.
2. The sulphate concentration are below the reportable detection limit of 20 μg/g.

5.0 CLOSURE

This Foundation Investigation Report was prepared by Ms. Sandra McGaghran, M.Eng., P.Eng., a geotechnical engineer and Associate with Golder. Ms. Lisa Coyne, P.Eng., a Principal and Designated MTO Contact for Golder, conducted an independent quality control review of the report.

GOLDER ASSOCIATES LTD.

LICENSED PROFESSIONAL ENGINEER
S.M. McGAGHRAN
Mar 13/18
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
L. C. COYNE
13 MAR 2018
PROVINCE OF ONTARIO

Sandra McGaghran, M.Eng., P.Eng.
Geotechnical Engineer, Associate

Lisa Coyne, P.Eng.
Principal, Designated MTO Foundations Contact

SMM/LCC/sm

https://golderassociates.sharepoint.com/sites/13019g/6 - deliverables/fnds/underpass/final/1670268 fidr 2018mar13 hwy 400 6th line.docx

PART B

FOUNDATION DESIGN REPORT
HIGHWAY 400 / 6th LINE UNDERPASS (SITE NO. 30-211/1&2)
AND ASSOCIATED HIGH FILL EMBANKMENTS
TOWN OF INNISFIL, SIMCOE COUNTY, ONTARIO
MINISTRY OF TRANSPORTATION, ONTARIO
G.W.P. 2289-13-00

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides detail foundation engineering design recommendations for the proposed Highway 400 underpass at the realigned 6th Line (Site No. 30-211/1&2). These recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation. The discussion and recommendations presented are intended to provide the designer with sufficient information to assess the feasible foundation alternatives and carry out the design of the bridge foundations.

The foundation investigation report, discussion and recommendations are intended for the use of the Ministry of Transportation, Ontario (MTO) and their designers for G.W.P 2289-13-00, and shall not be used or relied upon for any other purpose or by any other parties, including the construction or design build contractor. The contractor must make their own interpretation based on the factual data in Part A (Foundation Investigation) of the report. Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project and for which special provisions may be required in the Contract Documents. Those requiring information on the aspects of construction must make their own interpretation of the factual information provided, as such interpretation may affect equipment selection, proposed construction methods, scheduling, and the like.

6.1 General

Based on the General Arrangement (GA) drawing provided by MH on January 9, 2018, the proposed two-span structure will be 84 m long, with two equal 42 m spans. The boreholes for the proposed pier were advanced through the existing Highway 400 embankment with the ground surface at about Elevation 296.6 m. The boreholes for the west and east abutments were advanced within the existing agricultural fields and the ground surface at the borehole locations is at about Elevation 293.7 m and 295.1 m, respectively. It is understood that the proposed 6th Line grade at the west and east abutments will be approximately Elevation 304 m, resulting in west and east approach embankment heights reaching a maximum of about 10.3 m and 8.9 m, respectively, above the existing ground surface.

6.2 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the 2014 *Canadian Highway Bridge Design Code* and its *Commentary* (CHBDC 2014), the proposed bridge and its foundation system are classified as having a “typical consequence level” associated with exceeding limits states design. In addition, given the level of foundation investigation completed to date at this location in comparison to the degree of site understanding in Section 6.5 of the CHBDC (2014), the level of confidence for design is considered to be a “typical degree of site and prediction model understanding.” Accordingly, the appropriate corresponding ULS and SLS consequence factor, ψ, from Table 6.1 and geotechnical resistance factors, ϕ_{gu} and ϕ_{gs}, from Table 6.2 of the CHBDC (2014) have been used for design.

6.3 Foundation Options

Both shallow and deep foundation options have been considered for support of the new underpass. Temporary protection systems will be required along the western edge of the Highway 400 southbound lanes to facilitate the construction of the centre pier (depending on the foundation alternative selected). It is anticipated that some groundwater seepage may occur into the excavations from within the non-cohesive fills and surficial native soils.

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

- **Strip or spread footings founded on the very stiff to hard clayey silt till:** Shallow footings are feasible at this site due to the generally very stiff to hard nature of the overburden soils. This option would require excavation to a depth of about 4 m to 4.5 m below the existing highway grade at the proposed centre pier to extend below the existing fill materials, with associated protection systems parallel to the Highway 400 lanes. At the proposed abutments, excavations would extend to a depth of about 2.3 m to 3 m below the existing ground surface to found on the very stiff to hard clayey silt till to compact to dense silt and sand till deposit. This option does not allow for the construction of integral abutments, but could permit semi-integral abutments.
- **Footings "perched" on a compacted granular pad in the approach embankments:** Shallow abutment footings "perched" within the proposed approach embankments are feasible and could minimize the depth of excavation below the existing grade. This option would not allow for the construction of integral abutments.
- **Steel H-piles or pipe piles driven to found within the hard clayey silt/silt and sand till at Elevation 273 m to 275 m, or alternatively within the "100 blow" clayey silt to silt at about Elevation 270 m to 266 m:** Steel HP310x110 friction piles driven to within the hard clayey silt/silt and sand till at Elevation 273 m to 275 m, or alternatively end-bearing piles driven into "100-blow" material at about Elevation 270 m to 266 m, are suitable and feasible for the support of the proposed abutments and central pier, and would allow for integral abutment construction. The varying depth to "100-blow" soil within the footprint of each foundation element will result in the potential for variable pile lengths, which will need to be accommodated in the contract documents.
- **Drilled shafts (caissons) founded at Elevation 285 m in the hard clayey silt till, or alternatively within the "100-blow" clayey silt to silt deposit at Elevation 270 m to 266 m:** Drilled shafts are feasible for support of the abutments (although they would not permit integral abutment construction) and pier for the proposed new structure. The drilled shafts could be socketed into the 100-blow soil at approximately Elevation 270 m to 266 m or, alternatively, they may be founded as high as Elevation 285 m in the hard clayey silt till to silt and sand till. However, if deep foundations are adopted, the use of driven piles would be preferred over drilled shafts, from a foundations perspective, due to the presence of water-bearing cohesionless soils (i.e., the sand and silt to silt till and the interlayers or lenses of sand to silt within the clayey silt till). Temporary or permanent liners would be required during caisson installation to control the ground and groundwater within these water-bearing cohesionless zones, which would result in the caisson foundations being less cost-effective than the installation of driven steel H-piles. At the pier, caissons may in fact be advantageous over spread footings from a geotechnical/foundations perspective if they can be constructed as structural columns/caissons without a below-grade pile cap, which could reduce the depth of excavation and temporary protection systems that would be required adjacent to the highway.

Based on the above considerations, the preferred option from a geotechnical/foundations perspective is to support the abutments for the proposed new structure on shallow spread footings founded on the very stiff to hard clayey silt till deposit if integral abutments are not adopted, or on steel H-piles founded within the hard clayey silt to very dense silt in an integral abutment configuration. At the pier, the preferred option from a geotechnical/foundations perspective is driven steel pile foundations to minimize the depth of excavation adjacent to the heavily-travelled Highway 400 lanes, as compared with a strip footing option; as noted above, drilled shafts are also feasible and may be advantageous if the below-grade pile cap can be eliminated and the structural columns supported directly on the drilled shafts.

6.4 Strip Footings

6.4.1 Founding Elevations

Detailed below, for each foundation element, are the recommended founding elevations for strip footings on very stiff to hard clayey silt till / dense silt and sand till deposits.

Structural Element	Reference Boreholes	Founding Stratum (m)	Maximum Founding Elevation (m)
West Abutment	6UP-02 and 6UP-03	Very stiff to hard clayey silt till / dense silt and sand till	291.5
Centre Pier	6UP-04 and 6UP-05		292.0
East Abutment	6UP-06 and 6UP-07		292.0

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

Consideration could also be given to subexcavation of the loose/soft soils to the founding elevation given above and replacement with compacted granular fill to permit footings to be founded at a higher elevation. Notwithstanding these requirements, strip footings should be founded at a minimum depth of 1.5 m below the lowest surrounding grade to provide adequate protection against frost penetration (per OPSD 3090.101 – *Foundation Frost Depths for Southern Ontario)*. If adequate soil cover cannot be provided for footings (for example, for retaining walls adjacent to the abutments), rigid styrofoam insulation could be installed to compensate for the lack of soil cover and provide protection from frost penetration.

Alternatively, the abutment foundations could be "perched" on compacted granular pads in the approach embankments above the Highway 400 grade. In this case, the compacted granular pad should have a minimum thickness of 2 m; any existing fill, organic soils and/or loose soils within the zone of influence below the compacted granular pad should be subexcavated and replaced with engineered fill, or the pad thickened to found on the very stiff to hard clayey silt with sand till to dense silt and sand till deposits at the elevations given above for footings founded on these deposits. The pad should consist of OPSS.PROV 1010 (Aggregates) Granular 'A' material extending at least 1 m beyond the edges of the footing(s), then outward and downward at 1H:1V. The granular fill should be placed in accordance with OPSS.PROV 501 (Compacting).

6.4.2 Geotechnical Resistances

Strip footings placed on the native clayey silt / silt and sand till, or perched on compacted Granular 'A' pads within the approach embankments founded at or below the design elevations given in the preceding section, should be designed based on the factored ultimate geotechnical resistances and factored serviceability geotechnical resistances (for 25 mm of settlement) given below.

Foundation Element	Founding Stratum	Maximum (Highest) Founding Elevation	Factored Ultimate Geotechnical Resistance	Factored Serviceability Geotechnical Resistance[1]
West Abutment	Very stiff to hard clayey silt till	291.5 m[4]	650 kPa[4]	500 kPa[4]
	Compacted Granular 'A' pad following subexcavation of soft/loose soils [2]	293.5 m[4]	700 kPa[4]	400 kPa[4]

Foundation Element	Founding Stratum	Maximum (Highest) Founding Elevation	Factored Ultimate Geotechnical Resistance	Factored Serviceability Geotechnical Resistance[1]
	Perched in approach embankments on compacted Granular 'A' pad[3]	N/A[4]	600 kPa[4]	300 kPa[4]
Centre Pier	Hard native clayey silt till	292 m	650 kPa	500 kPa
East Abutment	Firm to very stiff clayey silt till	293.5 m	500 kPa	300 kPa
	Very stiff to hard clayey silt till	292 m	650 kPa	500 kPa
	Compacted Granular 'A' pad following subexcavation of soft/loose soils [2]	293.5 m	700 kPa	400 kPa
	Perched in approach embankments on compacted Granular 'A' pad[3]	N/A	600 kPa	300 kPa

Notes:

1. For 25 mm of settlement.
2. For minimum 2 m thick granular pad with base of pad at Elevation indicated.
3. For minimum 2 m thick granular pad founded within approach embankment fill.
4. Founding elevations and resistances to be confirmed pending laboratory analysis.

The geotechnical resistances should be reviewed if the selected footing width or founding elevations differ from those given above. The factored geotechnical resistances provided above are given for loads that will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the footing, inclination of the load should be taken into account in accordance with Section 6.10.4 of the CHBDC (2014).

The footing subgrade should be inspected, in accordance with OPSS.PROV 902 *(Excavating and Backfilling Structures)* to check that all existing fill and softened/disturbed native soils have been removed.

The native soil subgrade will be susceptible to disturbance from ponded water, precipitation from inclement weather and/or construction traffic. If the concrete for the footings cannot be poured immediately after excavation and inspection, it is recommended that a concrete working slab (100 mm thick of 20 MPa compressive strength concrete) be placed in the excavation within four hours to protect the integrity of the subgrade. If shallow foundations are adopted, an NSSP to address this item should be included in the Contract Documents.

6.4.3 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the new concrete footings and the subgrade should be calculated in accordance with Section 6.10.5 of the CHBDC (2014). For cast-in-place concrete footings constructed directly on the native soils or granular pad, or on a concrete working slab, the sliding resistance may be calculated based on the unfactored coefficient of friction, tan δ, which can be taken as follows:

- Cast-in-place footing or working slab to native deposits: tan $\delta = 0.65$
- Cast-in-place footing or working slab to Granular A pad: tan $\delta = 0.7$
- Cast-in-place footing to concrete working tan $\delta = 0.7$

6.4.4 Frost Protection

The footings should be provided with a minimum 1.5 m of soil cover to provide adequate protection against frost penetration (per OPSD 3090.101 – *Foundation Frost Depths for Southern Ontario)*. As a guide, the MTO has adopted the use of 25 mm (1 inch) of rigid polystyrene foam insulation as equivalent to a 0.3 m reduction in soil cover.

6.5 Driven Steel H-Piles or Tube Piles

Steel H-piles or steel tube (pipe) piles driven to found within the hard clayey silt till to dense silt and sand till or alternatively to the “100-blow” clayey silt to silt, may be used for support of the abutments and the centre pier.

Consideration must be given to the potential presence of cobbles and boulders within the till deposits at the site. In this regard, steel H-piles are preferred over steel tube piles as tube piles are considered to pose a higher risk of “hanging-up” or being deflected away from their vertical or battered orientation during installation. Where end-bearing piles are adopted, it is recommended that pile tip reinforcement be incorporated to reduce the potential for damage to the pile during driving. In this regard, pile driving shoes (such as Titus standard “H” points or equivalent) are recommended over flange reinforcement. Pile tip reinforcement is not recommended where friction piles are adopted at this site,

6.5.1 Pile Founding Elevation

Steel H-piles or steel pipe piles used as shorter “friction” piles should be driven to the following elevations at the foundation units, to found within the hard clayey silt till to dense silt and sand till.

Foundation Element	Reference Borehole Nos.	Founding Stratum	Estimated Pile Tip Elevation
West Abutment	6UP-02 and 6UP-03	Hard clayey silt till to dense silt and sand till	273 m
Centre Pier	6UP-04 and 6UP-05		273 m
East Abutment	6UP-06 and 6UP-07		275 m

Alternatively, steel H-piles or pipe piles could be driven into the “100 blow” soil, the surface of which varies across the site. For design, the following ranges in pile tip elevations may be used based on the borehole results, assuming approximately 1.5 m to 2 m of penetration into materials having SPT “N” values of greater than 100 blows per 0.3 m of penetration.

Foundation Element	Reference Borehole Nos.	Founding Stratum	Estimated Pile Tip Elevation
West Abutment	6UP-02 and 6UP-03	Hard clayey silt and sand/very dense silty sand till	266 m
Centre Pier	6UP-04 and 6UP-05	Hard clayey silt to very dense silt	268 m
East Abutment	6UP-06 and 6UP-07	Very dense silt	273 m

There should be provisions made in the contract for dealing with varying pile lengths.

6.5.2 Geotechnical Axial Resistances

For HP310x110 or HP310x132 steel H-piles driven to Elevation 275 m to 273 m (per Section 6.5.1) to found within the hard clayey silt till to silt and sand till, and similarly for pipe piles as noted above, a factored ultimate geotechnical resistance of 1,200 kN per pile may be used for design. The factored serviceability geotechnical resistance for 25 mm of settlement will be greater than the factored ultimate geotechnical resistance; as such, the factored ultimate geotechnical resistance will govern for this foundation type. Larger pile sections (e.g., HP360x132) may be used to achieve higher resistances; recommendations for such larger pile sizes will be provided if required.

For HP310x110 or HP310x132 steel H-piles founded within the "100 blow" clayey silt to silt at the tip elevations given in Section 6.5.1, and similarly for pipe piles as noted above, a factored ultimate geotechnical resistance of 1,600 kN per pile may be used for design. The factored serviceability geotechnical resistance for 25 mm of settlement (for the length of piles required at this site) will be greater than the factored ultimate geotechnical resistance; as such, the factored ultimate geotechnical resistance will govern for this foundation type. Larger pile sections (e.g., HP360x132) may be used to achieve higher resistances; recommendations for such larger pile sizes will be provided if required.

Given the very stiff to hard/dense to very dense nature of the overburden soils, downdrag loads do not need to be taken into account in the pile design.

Pile installation should be in accordance with OPSS.PROV 903 (Deep Foundations). The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile size and length of pile. The set criteria must therefore be established at the time of construction after the piling equipment is known. The pile capacity should be verified in the field by the use of both the Hiley formula (MTO Standard Drawing SS103-11) AND PILE DYNAMIC ANALYZER (PDA) TESTING during the final stages of driving to achieve an ultimate capacity. Special Provision SSP 903S06 *(High Strain Dynamic Testing, Deep Foundations – Amendment to OPSS 903)* should be included in the Contract Documents to address the requirement for PDA testing. Based on MTO experience with the Hiley formula in Southern Ontario, a resistance factor equal to 0.5 may be used on the ultimate resistance to verify the factored ULS design values.

The following note from MTO's Structural Manual should be shown on the Contract Drawing, based on the application of a resistance factor of 0.5 to the use of the Hiley formula (per MTO experience in Southern Ontario) and to the ultimate capacity as assessed by PDA testing:

- *Piles to be driven in accordance with Standard SS103-11 plus PDA testing using an ultimate geotechnical resistance of 3,200 kN per pile at the abutments and piers, but should be driven to no higher than 1.5 m above the design pile tip elevations shown below at each foundation element:*
  - *West Abutment: Elevation 266.0 m*
  - *Centre Pier: Elevation 268.0 m*
  - *East Abutment: Elevation 273.0 m*

Alternatively, for shorter friction piles driven to Elevation 273 m to 275 m, the following note from MTO's Structural Manual should be shown on the Contract Drawing, based on a resistance factor of 0.5:

- *Piles to be driven in accordance with Standard SS103-11 plus PDA testing using an ultimate geotechnical resistance of 2,400 kN per pile at the abutments, but should be driven to no higher than 1.5 m above the design pile tip of Elevation 273 m at the west abutment and pier and Elevation 275 m at the east abutment.*

Assessment of ultimate geotechnical resistance by the Hiley formula and PDA testing should commence once the pile reaches a depth of not more than 1.5 m above the design pile tip elevation shown above and at 0.5 m intervals of depth until the ultimate axial resistance is achieved. If the ultimate capacity as determined by the Hiley formula and/or PDA testing is not achieved within the 1.5 m interval down to the design pile tip elevation, the Contractor should stop pile driving and notify the Contract Administrator. At this depth, the pile should be allowed to rest for 48 hours and the Hiley formula and PDA testing should then be applied immediately upon re-striking the pile. If the ultimate capacity is still not achieved after the 48-hour wait period, the Contract Administrator should be notified and authorization given prior to driving the pile below the design pile tip elevation. An NSSP has been developed to amend OPSS.PROV 903 *(Deep Foundations)* to address the 48-hour wait period between initial driving and retapping, for inclusion in the Contract Documents (see Appendix C).

Given the variable depth to the “100-blow” soils and the resulting variability in the pile founding elevations, it is recommended that the greater pile lengths be stipulated in the Contract Drawings for piles located between the north and south sides of the pier to ensure that adequate pile lengths are available on site and to reduce splicing needs. It is also recommended that the axial capacity be calculated by the Hiley formula on every pile installed.

6.5.3 Resistance to Lateral Loads

Resistance to lateral loading may be derived using vertical piles, with enhanced support offered by inclined (battered) piles, if required. For vertical piles, the resistance to lateral loading will be derived solely from the soil in front of the piles, whereas inclined piles derive lateral resistance from the soil in front of the piles as well as the horizontal component of the axial load present in the inclined pile. For integral abutment design the steel H-piles would be installed within a 3 m long corrugated steel pile filled with sand fill of a gradation in accordance with Table 1 in the NSSP for integral abutments (see Appendix C).

Where ground conditions are generally competent and the lateral loads on piles are relatively small such that the maximum lateral pile deflections will be relatively small, the resistance to lateral loading in front of a single pile can be estimated using subgrade reaction theory (as outlined below). However, the response of a pile to lateral loads is highly nonlinear and methods that assume linear behavior (such as subgrade reaction theory) are most appropriate where the maximum pile deflections are less than 1 percent of the pile diameter, where the loading is static (no cycling) and where the pile material is linear (CFEM, 2006). Where these conditions are not met, and where required for the structural engineering model, the non-linear lateral behavior of the soil should be considered by the use of P-y curves.

The factored serviceability geotechnical response of the soil in front of the piles under lateral loading at this site may be calculated using subgrade reaction theory suggested in CHBDC (2014) Commentary (Section C6.11.2.2), where the coefficient of horizontal subgrade reaction, k_h, (kPa/m) is based on the equation given below, as described by Terzaghi (1955) and the Canadian Foundation Engineering Manual (CFEM 1992).

For non-cohesive soils:

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

Where: n_h is the constant of horizontal subgrade reaction (kPa/m), as given below;

Z is the depth (m) below ground surface, except for the loose sand within the CSP where z is the depth (m) below the top of the CSP; and,

B is the pile diameter/width (m)

For cohesive soils:

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

Where: s_u is the undrained shear strength of the soil (kPa); and,

B is the pile diameter/width (m)

The following values of n_h and s_u (Terzaghi, 1955) may be incorporated into the calculations of horizontal subgrade reaction (k_h) for structural analyses for a single vertical pile. The ranges in values reflect the variability in the subsurface conditions, the soil properties and the approximate nature of the analysis and the non-linear nature of the soil behaviour (such that k_h is a function of deflection).

Foundation Element	Soil Unit	Elevation Interval (m)	n_h (kPa/m)	s_u (kPa)
West Abutment	Loose sand within CSP	297 to 294	2,000	--
	Firm to stiff clayey silt with sand	294 to 291.5	--	50
	Very stiff to hard clayey silt with sand till to dense silt and sand till (below the water table)	291.5 to 266.7	7,000	300
	"100-blow" clayey silt (below the water table)	Below 266.7	11,000	500
Center Pier	Loose to dense silt and sand fill (above the water table)	294.5 to 292.5	1,500	--
	Very stiff to hard clayey silt with sand till to compact to very dense silt and sand till (below the water table)	292.5 to 272	7,000	300
	"100-blow" clayey silt to silt (below the water table)	Below 272	11,000	500
East Abutment	Loose sand within CSP	297 to 294	2,000	--
	Stiff clayey silt fill (above the water table)	294 to 293.3	--	75
	Very stiff to hard clayey silt with sand till to dense silt and sand till (below the water table)	293.4 to 275.5	7,000	300
	"100-blow" silt (below the water table)	Below 275.5	11,000	--

The till layer varies both laterally and vertically from plastic to non-plastic, and both the horizontal subgrade reaction and undrained shear strength are provided to address the range of behaviour; both conditions should be checked. Both the structural and geotechnical resistances of the piles should be evaluated to establish the governing case at Ultimate Limit States (ULS). At Serviceability Limit States (SLS), the horizontal reaction of the piles will be controlled by deflections and the horizontal resistance of the pile should be calculated based on the coefficient of horizontal subgrade reaction (k_h) of the soil as discussed above. The SLS reaction should be taken as that corresponding to a horizontal deflection of 10 mm at the underside of the pile cap for units supporting the abutments (CHBDC (2014) Commentary Section 6.11.2.2). The minimum strength of concrete should be checked by the structural engineer based on the anticipated lateral loads.

The upper zone of the soil (down to a depth below the pile cap equal to about 1.5xB (where B is the pile diameter) should be neglected in the calculation of lateral resistance of the pile to account for disturbance effects during installation.

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters between rows of driven steel H-piles. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading by a reduction factor, R (NAVFAC DM7.2, 1986) as follows:

Pile Spacing in Direction of Loading (D = Pile Diameter)	Subgrade Reaction Reduction Factor, R
8D	1.00
6D	0.70
4D	0.40
3D	0.25

The subgrade reaction reduction factor should be interpolated for pile spacings in between those provided in the above summary. Reduction for group effects is negligible when the centre to centre pile spacing exceeds three pile diameters measured in the direction perpendicular to loading.

6.5.4 Frost Protection

All pile caps should be provided with a minimum of 1.5 m of soil cover for frost protection as interpreted from OPSD 3090.101 (Foundation, Frost Penetration Depths for Southern Ontario). As a guide, the MTO has adopted the use of 25 mm (1 inch) of rigid polystyrene foam insulation as equivalent to a 0.3 m reduction in soil cover.

6.6 Drilled Shafts (Caissons)

Drilled shafts socketed into the “100 blow” lower clayey silt to silt could be considered for support of the abutments and centre pier, particularly if it is desired to minimize the depth of excavation compared to footings at the pier, or to eliminate a below-grade pile cap at the pier. If drilled shaft foundations are adopted for support of any of the foundation elements in this founding stratum, a temporary or permanent liner would be required to support the soils during construction, to minimize disturbance and loss of ground in the water-bearing cohesionless soil zones (the silt and sand to silty gravelly sand till and interlayers or lenses of silty sand to sand) within the till deposit. These cohesionless layers were generally encountered below Elevation 284 m, and they are under some piezometric pressure as there was about 3 m of “blow-back” of the sand layer up inside the hollow stem augers during advancement of Borehole 6UP-02 (west abutment) through this layer. It is noted that granular interlayers under pressure may be present elsewhere within the deposit; however with the sampling interval of 1.5 m there is the possibility that the borehole advanced through a layer if it was between sampling depths.

Alternatively, consideration could also be given to founding the drilled shafts higher, at Elevation 285 m, in the hard clayey silt till to dense silt and sand till deposit. Drilled shafts founded at this elevation would have lesser risks related to ground/groundwater control, although a temporary liner is still recommended for advancement to permit cleaning and inspection of the base.

For either of the two founding elevations, if there is water infiltration such that there is standing water within the drilled shaft excavation prior to concrete placement, the concrete must be placed using tremie techniques, in accordance with OPSS 903. After initial placement of concrete at the bottom of the drilled shaft, the tremie discharge point should be maintained a minimum of 1 m below the surface of the wet concrete during placement, in accordance with OPSS 903. The need for control of the ground and groundwater during drilled shaft construction is discussed further under Construction Considerations in Section 6.13.8.

It is expected that the liner would be installed (and removed, if a temporary liner is used) using a vibratory hammer. In this case, vibration monitoring is recommended during liner installation and removal.

The performance of drilled shafts will depend upon the final cleaning and verification of the subgrade quality (hard clayey silt to very dense silt) at the base of the drilled shaft. Each drilled shaft excavation should be carefully cleaned to remove all loosened debris to ensure that the concrete is in intimate contact with the competent bearing stratum. The inspection of the base of the drilled shafts can be accomplished by means of a Shaft Inspection Device (SID) such as a video camera. Should the camera inspection indicate that loosened/unacceptable soil is present at the base of the drilled shaft, the base would need to be re-cleaned and re-inspected. A Foundation Engineer must confirm that the conditions encountered are consistent with the information obtained from the boreholes and that the required cleanliness has been obtained. Concrete must be placed using tremie methods immediately following cleaning and inspection of the base the.

6.6.1 Founding Elevation

The following drilled shaft base elevations and strata may be used in the design, based on the lowest elevation within each foundation element to achieve at least 1.5 m to 2 m of penetration into the “100-blow” clayey silt to silt soils:

Foundation Element	Boreholes No.	Founding Stratum	Estimated Drilled Shaft Founding Elevation
West Abutment	6UP-02 and 6UP-03	Very dense silt[1]	266 m
Central Pier	6UP-04 and 6UP-05	Hard clayey silt to very dense silt	268 m
East Abutment	6UP-06 and 6UP-07	Very dense silt	273 m

1. Soil description to be confirmed based on laboratory data.

Alternatively, as discussed above, the drilled shafts could be founded at Elevation 285 m in the hard clayey silt with sand till to silt and sand till deposit.

6.6.2 Geotechnical Axial Resistances

The following provides the recommended factored ultimate geotechnical resistances and factored serviceability geotechnical resistances (for 25 mm of settlement) for drilled shafts socketed approximately 1.5 m into the “100-blow” material at the founding elevations given in Section 6.6.1.

Foundation Element	Drilled Shaft Diameter	Founding Stratum	Factored Ultimate Geotechnical Resistance (kN)	Factored Serviceability Geotechnical Resistance for 25 mm of Settlement (kN)
West Abutment	0.9 m	Hard clayey silt to very dense silt	3,300	1,400
	1.2 m		4,800	1,650
	1.5 m		6,500	1,850
Pier	0.9 m		2,700	1,300
	1.2 m		4,100	1,550
	1.5 m		5,600	1,800
East Abutment	0.9 m		2,300	1,250
	1.2 m		3,500	1,500
	1.5 m		4,900	1,750

The following provides the recommended factored ultimate geotechnical resistances and factored serviceability geotechnical resistances (for 25 mm of settlement) for drilled shafts founded at Elevation 285 m.

Drilled Shaft Diameter	Founding Stratum	Factored Ultimate Geotechnical Resistance (kN)	Factored Serviceability Geotechnical Resistance for 25 mm of Settlement (kN)
0.9 m	Hard clayey silt with sand till to dense silt and sand till	1,300	900
1.2 m		2,100	1,500
1.5 m		3,000	2,000

Given the very stiff to hard/dense to very dense nature of the overburden soils, downdrag loads do not need to be taken into account in the drilled shaft design.

6.6.3 Resistance to Lateral Loads

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

6.6.4 Frost Protection

All drilled shaft caps should be provided with a minimum of 1.5 m of soil cover or equivalent thickness of insulation below the cap for frost protection, in accordance with OPSD 3090.101 *(Foundation Frost Penetration Depths for Southern Ontario).* As a guide, the MTO has adopted 25mm (1 inch) of rigid polystyrene foam insulation as equivalent to a 0.3 m reduction in soil cover.

6.7 Seismic Design

6.7.1 Seismic Site Classification

The site may be classified as Site Class C in accordance with Table 4.1 of the CHBDC (2014), in the absence of any geophysical testing. Geophysics testing, if carried out, can often provide a more favourable Site Class designation, but this may not be feasible at this site. For example, Table 4.1 of the CHBDC (2014) indicates that Site Class A and B are not to be used if there is more than 3 m of soil between the underside of the bridge foundations and the bedrock.

6.7.2 Spectral Response Values and Seismic Performance Category

In accordance with Section 4.4.3.4 of the CHBDC (2014), the peak ground acceleration (PGA) values and design spectral acceleration (Sa) values for Site Class C are presented below.

Seismic Hazard Values	10% Exceedance in 50 years (475-year return period)	5% Exceedance in 50 years (975-year return period)	2% Exceedance in 50 years (2,475 return period)
PGA (g)	0.041	0.075	0.144
PGV (m/s)	0.031	0.052	0.092
Sa (0.2) (g)	0.069	0.120	0.223
Sa (0.5) (g)	0.042	0.067	0.116
Sa (1.0) (g)	0.023	0.036	0.059
Sa (2.0) (g)	0.011	0.017	0.028
Sa (5.0) (g)	0.0023	0.0039	0.0067
Sa (10.0) (g)	0.001	0.0016	0.0028

6.8 Lateral Earth Pressures for Design of Abutments and Wingwalls

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

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

- Select, free draining granular fill meeting the specifications of OPSS.PROV 1010 (Aggregates) Granular A or Granular B Type II, should be used as backfill behind the walls. Longitudinal drains or weep holes should be installed to provide positive drainage of the granular backfill. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS.PROV 501 (Compacting). Other aspects of the granular backfill requirements with respect to subdrains and frost taper should be in accordance with OPSD 3101.150 (Walls, Abutment, Backfill, Minimum Granular Requirement), OPSD 3121.150 (Walls, Retaining, Backfill, Minimum Granular Requirement), and 3190.100 (Walls, Retaining and Abutment, Wall Drain).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with CHBDC (2014) Section 6.12.3 and Figure 6.6. Hand

operated compaction equipment should be used to compact the backfill soils immediately behind the walls as per OPSS.PROV 501. 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 1.5 m behind the back of the wall, per Figure C6.20(a) of the Commentary to the CHBDC (2014). For unrestrained walls, fill should be placed within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing or pile cap, per Figure C6.20(b) of the Commentary to the CHBDC (2014).

6.8.1 Static Lateral Earth Pressures for Design

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

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

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

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

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

- If the wall support and superstructure allow lateral yielding, active earth pressures may be used in the geotechnical design of the structure. The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure, may be taken as:
  - Rotation of approximately 0.002 about the base of a vertical wall (where the rotation is calculated as the horizontal displacement divided by the height of the wall);
  - Horizontal translation of 0.001 times the height of the wall; or,
  - A combination of both.

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

6.8.2 Seismic Lateral Earth Pressures for Design

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

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

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

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

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

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

Where: $\sigma_h(d)$ is the (static plus seismic) lateral earth pressure at depth, d, (kPa);

K_a is the static active earth pressure coefficient;

K_o is the static at-rest earth pressure coefficient;

K_{AE} is the seismic active earth pressure coefficient;

γ is the unit weight of the backfill soil (kN/m^3), as given in Section 6.8.1;

d is the depth below the top of the wall (m); and,

H is the total height of the wall (m).

6.9 Analytical Testing for Construction Materials

The results of analytical testing on three selected samples of the till deposit near the proposed underpass foundation elements are presented in Section 4.2.9 and in Appendix B. The analytical test results were compared to CSA A23.1 Table 3 *("Additional requirements for concrete subjected to sulphate attack")* for potential sulphate attack on concrete. The sulphate concentrations measured in the tested samples are below the exposure class of S-3 (Moderate). Therefore, when the designer is selecting the exposure class for the structure, the effects of sulphates may not need to be considered.

The analytical test results of the soil samples were also compared to Table 2 of the U.S. Criteria for Assessing Ground Corrosion Potential (as derived from Federal Highways Administration (FHWA) 2003) for the potential attack on buried steel. The resistivity and chloride concentrations measured in the soil sample obtained from a borehole advanced at the east abutment indicate “mild to no corrosion potential”, while “strong corrosion potential” occurs for the soil sample obtained from the proposed pier area and the proposed west abutment. Based on the results of the samples tested, and given that the structure will be exposed to de-icing salt, consideration should be given by the designer to designing for a “C” type exposure class as defined by CSA A23.1 Table 1.

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

6.10 Retained Soil System (RSS) Walls

It is understood that mechanically-reinforced soil retaining systems (retained soil system or RSS walls) are proposed as wingwalls/retaining walls on the north and south sides of the west and east abutments (refer to Drawing 1). The RSS retaining walls are to be designed for high performance and appearance in accordance with MTO Special Provision (SP) 599S22 (Retained Soil System).

6.10.1 Founding Elevations

A typical RSS wall has a front facing supported on a strip footing placed at shallow depth below the ground surface in front of the wall. At its lowest point, the underside of the proposed RSS wall is at Elevation 299.0 m which is above the existing ground surface at the abutments; engineered earth, select subgrade or granular fill will be required to raise the grade to the underside of the granular pad below the facing footing/alignment element. Prior to placement of the engineered fill, the existing topsoil must be removed and the existing fill/reworked soil is required to be proof-rolled. As the RSS wall is proposed to "step up" into the embankment away from the back of the abutment, the facing footing may also be stepped up provided that it is founded below any topsoil or softened/disturbed soil; for design, a minimum founding depth of 0.8 m is recommended as the facing footing steps up into the cut slope.

The facing footing should be placed on a 300 mm thick layer of compacted OPSS.PROV 1010 Granular 'A', as detailed in Figure 5.2, MTO RSS Wall Design Guidelines (September 2008). The compacted granular pad should extend at least 1.0 m beyond the outside edge of the facing footing, then downward at 1H:1V.

The compacted Granular 'A' pad and the reinforced soil mass should be keyed into the existing embankment by benching into the embankment fill, as per OPSD 208.010 (Benching of Earth Slopes).

6.10.2 Global Stability

The static global slope stability of RSS walls for the underpass structure has been analyzed using the commercially-available program SLIDE, produced by Rocscience Inc., employing the Morgenstern-Price method of analysis. For all analyses, the factor of safety of numerous potential failure surfaces was computed in order to establish the minimum Factor of Safety (FoS). A target minimum factored FoS of 1.5 is adopted for the design of embankment slopes under static conditions at the end of construction as per the CHBDC (2014). In general, circular slip surfaces were analysed in the design. These factors of safety are considered appropriate for the RSS walls at this site, considering the design requirements and the field data available.

The soil parameters used in the analysis, as given below, were estimated from empirical correlations using the results of in-situ Standard Penetration Tests (SPTs) (Bowles, 1984) and geotechnical classification testing. The groundwater table was taken at Elevation 292.5 m in the analyses.

Soil Type	Unit Weight (kN/m^3)	Undrained Shear Strength (kPa)	Angle of Internal Friction, φ' (degrees)
New embankment fill behind reinforced soil mass	21	--	35
Firm to stiff clayey silt till/compact silt and sand till	21	100	-
Very stiff to hard clayey silt till/dense to very dense silt and sand till	22	-	35
Hard clayey silt to very dense silt	22	-	34

Three RSS wall sections were analyzed for the varying wall heights as shown on the drawings provided by MH, dated January 9, 2018. In these analyses, the height of the RSS wall was considered to extend from the top of the pavement elevation to the underside of the lowest panel (top of the front facing footing). The analysis was carried out using a minimum of 0.8 m of soil cover over the front facing footing and a 2H:1V slope in front of the toe of the RSS wall. If the wall configuration changes during the course of the detail design and is different from that assumed above, further stability analyses should be completed as the results are sensitive to the buried depth of wall and the presence of the 2H:1V slope at the base of the wall.

The minimum reinforcement length has been assessed in order to obtain a factor of safety equal to 1.5 or greater against deep-seated global instability. The ratio of minimum reinforcement length to wall height for three representative RSS wall heights is provided below. The result of the analysis for the RSS wall adjacent to the abutment wall (i.e., a 5.0 m high wall) is shown on Figure 1 for the static condition.

RSS Wall Height	Ratio of Minimum Reinforcement Length to Wall Height
5.0 m	1.0
4.0 m	0.8
3.0 m	0.8

The above ratios for walls with a height of approximately 5.0 m or less are greater than the "typical" ratios that are used by wall designers (i.e. approximately 0.7 to 0.8 times the wall height), because of the presence of the 2H:1V slope in front of the wall. The contract drawings will need to specify the width of the reinforced soil mass.

6.10.3 Geotechnical Resistances

Assuming that the RSS wall acts as a unit and uses the full width of the reinforced soil mass, as recommended in Section 6.10.2, the factored ultimate and serviceability geotechnical resistances given below may be used for assessment of the reinforced mass founded on the properly prepared compacted granular fill, or on the native soil subgrade at the sub-excavation elevations given above.

Wall Height	Factored Ultimate Geotechnical Resistance	Factored Serviceability Geotechnical Resistance (for 25 mm of Settlement)
5.0 m	375 kPa	150 kPa
4.0 m	300 kPa	175 kPa
3.0 m	250 kPa	200 kPa

6.10.4 Resistance to Lateral Loads

The resistance to lateral forces / sliding resistance between the compacted granular fill of the RSS wall and the subgrade should be calculated in accordance with Section 6.10.5 of the CHBDC (2014). The coefficient of friction, tan φ', between the compacted granular fill of the RSS wall and the properly prepared subgrade may be taken as 0.55.

6.11 Concrete Toe Wall

Based on the design drawings provided by MH on January 31, 2018, a Type III concrete toe wall is proposed at the northeast embankment toe adjacent to the east abutment, to accommodate construction of a culvert.

The concrete toe wall should be designed and constructed in accordance with OPSD 3120.100 (*Concrete Toe Wall*). Per Note 1 on OPSD 3120.100, a factored ultimate geotechnical resistance of 300 kPa is required for a Type III wall. Based on the near-surface soil conditions, this minimum bearing capacity may not be achieved, although this will be further assessed once the founding level for the wall is confirmed. At this stage, it is recommended that any softened near-surface soils (cohesive fill or native cohesive soils) within the footprint of the toe wall be subexcavated, and replaced with Granular A or Granular B Type II that is placed and compacted in accordance with OPSS.PROV 501 (*Compacting*). The area to be subexcavated should be defined by a line extending from the top of the engineering fill pad outward and downward at 1H:1V.

6.12 High Fill and Approach Embankment Design and Construction

The new underpass will be located approximately 40 m north of the existing overpass, with new approach embankments up to about 10 m high relative to the existing ground surface. The proposed new high fill embankments (i.e., embankments greater than 4.5 m in height) will extend approximately 250 m west and 300 m east of the new underpass.

Boreholes HF-01 to HF-04, CE-01 to CE-03 and 6UP-01 were advanced in the vicinity of the west approach / high fill embankment and encountered topsoil underlain by very loose silty sand / firm to stiff clayey silt with sand, which in turn is underlain by very stiff to hard clayey silt with sand till.

Boreholes 6UP-08 and HF-05 to HF-12 were advanced in the vicinity of the east approach / high fill embankment and generally encountered topsoil underlain by firm to very stiff clayey silt fill/reworked which is in turn underlain by clayey silt with sand till.

6.12.1 Subgrade Preparation and Embankment Construction

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

Fill for construction of the new embankments may consist of OPSS.PROV 1010 granular materials, Select Subgrade Material (SSM), or clean earth fill. The embankment fill should be placed and compacted in accordance with OPSS.PROV 501 (Compacting) and OPSS.PROV 206 (Grading). Embankment side slopes should be constructed no steeper than 2 horizontal to 1 vertical (2H:1V) in granular or earth fill.

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

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

6.12.2 Global Stability

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

For the new earth/granular fill and native soil deposits, both short-term, undrained analyses (to address the presence of surficial layers of firm cohesive soils) and long-term, effective stress analyses were completed using the applicable parameters outlined in the table below. The parameters were estimated from the SPT "N" values, using empirical correlations proposed by Terzaghi and Peck (1967), and the results were tempered by engineering judgment based on precedent experience in similar soils. Summarized below are the simplified stratigraphy and the associated strengths and unit weights employed for the different soil types in the proposed high fill embankment areas.

Soil Deposit	Bulk Unit Weight (kN/m^3)	Effective Friction Angle (°)	Undrained Shear Strength (kPa)
New embankment fill (earth fill assumed; granular fill parameters will be higher)	21	32° (Cohesion: 1 kPa)	-
Existing compact silty sand / firm clayey silt fill	20	30°	35 kPa
Firm to stiff surficial layer of clayey silt with sand till	21	30°	50 kPa
Very stiff to hard clayey silt with sand till to dense silt and sand till	21	34°	200 kPa

The analysis indicates that the east and west approach embankments constructed of compacted earth or granular fill, with side slopes oriented at 2H:1V or flatter, will have a factored FoS greater than 1.5 against global instability in long-term conditions, as shown on Figure 2. The FoS will be greater than 1.3 for short-term, undrained conditions.

6.12.3 Settlement

Settlement of the founding soils under the east and west approach embankment areas can be expected as a result of the loading from the new fills on the loose silty sand and firm to stiff clayey silt. Settlement of new granular fill that is properly placed and compacted for construction of the widened embankments would occur during construction.

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

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

Area	Soil Type	Approximate Thickness (m)	Bulk Unit Weight (kN/m^3)	Elastic Modulus (MPa)
West Approach (Boreholes HF-01 to HF-04 and 6UP-01)	Firm to stiff clayey silt with sand	1.5	20	25
	Very stiff to hard clayey silt with sand till, containing non-cohesive interlayers	24	21	100
	"100"-blow lower soils	>3 m	21	200
	Firm sandy clayey silt fill	1.0	20	25
	Firm clayey silt with sand till	1.5	20	25

Area	Soil Type	Approximate Thickness (m)	Bulk Unit Weight (kN/m^3)	Elastic Modulus (MPa)
East Approach (Boreholes 6UP-08, HF-05 to HF-10)	Very stiff to hard clayey silt with sand till, containing non-cohesive interlayers	17	21	100
	"100"-blow lower soils	>3 m	21	200

6.12.4 Settlement Performance Requirements

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

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

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

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

6.12.5 Results of Analysis

Based on the analysis using the above parameters, the estimated maximum settlement under a 10 m high embankment is approximately 50 mm. The majority of this settlement will occur during and immediately following the fill placement. Therefore, it is anticipated that MTO's post-construction settlement performance requirements will be achieved without any further settlement mitigation measures.

The above estimates do not include compression of the fill itself, which would occur during and immediately after the construction of the embankment depending on the type of materials used. The magnitude of fill compression may range from 0.5 to 1 per cent of the height of the embankment, assuming approximately 98 per cent compaction of the embankment fill is achieved, relative to the material's standard Proctor maximum dry density.

6.13 Construction Considerations

6.13.1 Open Cut Excavation

The foundation excavations for spread footings or pile cap construction will extend through existing fill and into the till deposit, which contain zones, interlayers and lenses of water-bearing non-cohesive soil. Where space permits, open-cut excavations into these materials should be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) for Construction Activities. The existing fill materials are classified as Type 3 soil and the till is classified as Type 2 soil, according to the OHSA. Temporary excavations (i.e. those which are open for a relatively short time period) should be made with side slopes no steeper than 1H:1V.

6.13.2 Temporary Protection Systems

It is expected that temporary excavation support will be required to maintain traffic lanes in operation along Highway 400 southbound during construction of the new pier. The temporary excavation support systems should be designed and constructed in accordance with OPSS.PROV 539 *(Temporary Protection Systems).* The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS.PROV 539, provided that any adjacent utilities can tolerate this magnitude of deformation.

The protection system is expected to be required for a maximum excavation depth of approximately 1.5 m for pile caps, or 4 m to 4.5 m if a strip footing is adopted for support of the centre pier. It is considered that driven sheetpiles would be suitable for the temporary excavation support at this site, based on the subsurface soil and groundwater conditions; alternatively a soldier pile and timber lagging system could be used. Some groundwater seepage is anticipated at the base of the non-cohesive fill, and it would be necessary to control seepage or include measures to mitigate loss of soil particles through the lagging boards. In this regard use of a sheet pile wall would be advantageous.

The sheet pile wall would have to be socketed to sufficient depth to provide the necessary passive resistance for the retained soil height. Additional lateral support to the sheet pile wall or soldier pile wall, if required, could be provided in the form of rakers or temporary anchors. The selection and design of the protection system will be the responsibility of the Contractor.

6.13.3 Groundwater Control

The groundwater level measured in the standpipe piezometers installed in the till deposit are about 3 m below the ground surface in the adjacent agricultural fields, corresponding to about Elevation 293 m, which is about 1 m above the recommended founding level for strip footings. If the structure is supported on deep foundations, excavations for the pile cap will be above the groundwater level; however, depending on the time of year there may be water perched at the base of the non-cohesive fill layer or the surficial sand/silt layer.

It is anticipated that water inflow from these layers can be handled by pumping from filtered sump pumps placed at the base of the excavation. Surface water seepage into the excavations should be expected and will be heavier during periods of sustained precipitation and all surface water should be directed away from the excavations.

6.13.4 Obstructions During Installation of Deep Foundation Elements and Protection Systems

It is anticipated that cobbles and/or boulders may be encountered within the till deposits, which may affect the installation of deep foundations and/or protection system elements. It is recommended that driving shoes (such as Titus standard “H” point or equivalent) be used on all end-bearing steel H-piles to facilitate driving into the hard/very dense, 100-blow till. In addition, it is recommended that an NSSP or an Operational Constraint be included in the Contract Documents to warn the Contractor of the possible presence of cobbles and/or boulders within the overburden soils; such an Operational Constraint is provided in Appendix C.

6.13.5 Pile Driving

As discussed in Section 6.5.2, pile dynamic analyzer (PDA) testing is recommended in conjunction with Hiley testing during initial driving and retapping. Further, a 48-hour wait period is recommended following initial driving and prior to retapping. An NSSP is provided in Appendix C for incorporation into the Contract Documents, to amend OPSS.PROV 903 *(Deep Foundations)* to address both PDA testing and the 48-hour wait period prior to retapping.

6.13.6 Vibration Monitoring During Pile Installation or Caisson Construction

If driven steel H-piles are adopted and if the temporary protection systems are installed using vibratory methods, significant vibrations are not anticipated, given the very stiff to hard nature of the native soil deposits. 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 overpass structure 40 m to the south and, therefore, vibration monitoring for the existing overpass structure is not expected to be required during construction at this site.

Residential/commercial buildings are present in the vicinity of the site, at distances of approximately 250 m to 550 m from the proposed foundation elements for the new underpass. 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 the structures, it is understood that MTO has elected to incorporate pre- and post-construction condition surveys and vibration monitoring at or near the buildings. An NSSP has been provided in Appendix C to address condition surveys and vibration monitoring within 650 m of the underpass structure.

6.13.7 Subgrade Protection

If shallow foundations are adopted, the soils exposed at the footing subgrade level would be susceptible to disturbance from construction traffic and/or ponded water. To limit degradation of a strip footing subgrade, it is recommended that a working slab of concrete be placed on the subgrade within four hours after preparation, inspection and approval. This would be addressed via an NSSP as well as notes on the drawing; an NSSP can be provided if the design changes and shallow foundations are adopted.

6.13.8 Ground and Groundwater Control for Caisson Installation

As discussed in Section 6.6, running or flowing of water-bearing cohesionless soils (the silt and sand to silty gravelly sand till and interlayers or lenses of silty sand to sand) could occur during or after drilling of the caissons. If caisson foundations are adopted for support of any of the foundation elements, temporary or permanent caisson liners would be required to support the soils during construction and permit inspection and cleaning of the caisson base. In addition, in order to counter-balance the groundwater pressure, the liner must be advanced with water inside the liner and the auger may not at any time advanced beyond the tip of the liner. If this foundation option is adopted it is recommended that an NSSP be included in the Contract Documents to warn the contractor of these conditions and the need to control the ground and groundwater during caisson construction; Golder will provide this NSSP if the design changes to adopt caisson foundations.

7.0 CLOSURE

This Foundation Design Report was prepared by Ms. Sandra McGaghran, M.Eng., P.Eng., a geotechnical engineer and Associate with Golder. Ms. Lisa Coyne, P.Eng., a Principal and Designated MTO Contact for Golder, conducted an independent technical and quality control review of the report.

GOLDER ASSOCIATES LTD.

LICENSED PROFESSIONAL ENGINEER
S.M. McGAGHRAN
Mar 13/18
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
L. C. COYNE
13 MAR 2018
PROVINCE OF ONTARIO

Sandra McGaghran, M.Eng., P.Eng.
Geotechnical Engineer, Associate

Lisa Coyne, P.Eng.
Principal, Designated MTO Foundations Contact

SMM/LCC/sm

https://golderassociates.sharepoint.com/sites/13019g/6 - deliverables/fnds/underpass/final/1670268 fidr 2018mar13 hwy 400 6th line.docx

REFERENCES

Bowles, J.E. 1984. Physical and Geotechnical Properties of Soils, Second Edition. McGraw Hill Book Company, New York.

Canadian Geotechnical Society. 1992. Canadian Foundation Engineering Manual (CFEM), 3rd Edition. The Canadian Geotechnical Society, BiTech Published Ltd., British Columbia.

Canadian Geotechnical Society. 2006. Canadian Foundation Engineering Manual (CFEM), 4th Edition. The Canadian Geotechnical Society, BiTech Publisher Ltd., British Columbia.

Canadian Highway Bridge Design Code (CHBDC (2014)) and Commentary on CAN/CSA-S6-14. Canadian Standard Association. (CSA) Group.

Canadian Standards Association (CSA). 2006. Canadian Highway Bridge Design Code and *Commentary* on CAN/CSA-S6-06. CSA Special Publication, S6.1 06.

Chapman, L.J. and Putnam, D.F. 1984. The Physiography of Southern Ontario, Ontario Geological Survey, Special Volume 2, Third Edition. Accompanied by Map P.2715, Scale 1:600,000.

Kulhawy, F.H. and Mayne, P.W. 1990. Manual on Estimating Soil Properties for Foundation Design. EL6800, Research Project 14936. Prepared for Electric Power Research Institute, Palo Alto, California, U.S.

Terzaghi, K., 1955. Evaluation of Coefficients of Subgrade Reaction/ Geotechnique, Vol. 5, No. 4, pp. 297-326. Discussion in Vol. 6, No. 2, pp. 94-98.

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

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

U.S. Criteria for Assessing Ground Corrosion Potential (as derived from Federal Highways Administration (FHWA) 2003)

ASTM International:

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

Commercial Software:

Slide (Version 6) by Rocscience Inc.

Ontario Provisional Standard Drawing:

OPSD 208.010	Benching of Earth Slopes
OPSD 3000.100	Foundation, Piles, Steel H-Pile Driving Shoe
OPSD 3000.100	Foundation, Piles, Tube Pile Driving Shoe
OPSD 3090.101	Foundation Frost Penetration Depths for Southern Ontario
OPSD 3101.150	Walls, Abutments, Backfill, Minimum Granular Requirements
OPSD 3121.150	Walls, Retaining, Backfill, Minimum Granular Requirements

OPSD 3190.100	Walls, Retaining and Abutment, Wall Drain

Ontario Provincial Standard Specification:

OPSS.PROV 206	Construction Specification for Grading
OPSS.PROV 501	Construction Specifications for Compacting
OPSS.PROV 517	Construction Specification for Dewatering
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS 802	Construction Specification for Topsoil
OPSS 803	Construction Specification for Sodding
OPSS.PROV 804	Construction Specification for Seed and Cover
OPSS 902	Construction Specification for Excavating and Backfilling Structures
OPSS.PROV 903	Construction Specification for Deep Foundations
OPSS.PROV 1004	Material Specification for Aggregates – Miscellaneous
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

Ontario Water Resources Act:

Ontario Regulation 903	Wells (as amended)

Ontario Occupational Health and Safety Act:

Ontario Regulation 213/91	Construction Projects (as amended)

Ministry of Transportation, Ontario

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

Ministry of Transportation Ontario. Structural Office Report SO9601. Integral Abutment Bridges.

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

TABLE 1 – COMPARISON OF FOUNDATION ALTERNATIVES – HIGHWAY 400 / 6TH LINE UNDERPASS RECONSTRUCTION G.W.P. 2289-13-00

Option	Feasibility	Advantages	Disadvantages	Risks/Consequences	Constructability	Relative Costs
Strip or spread footing founded on very stiff to hard clayey silt till or dense silt and sand till	• Feasible for support of abutments and pier; however, requires deep excavations and temporary protection for staged construction at the pier	• Allows for semi-integral abutments • Lower vibration impacts on existing structures than for driven steel H-pile installation • Low risk of post-construction settlement • Minor groundwater seepage from perched water in the fill	• At the pier, excavations will be 4 m to 4.5 m deep through the existing embankment fill and native soil would be required • Temporary protection system required during construction • Precludes use of integral abutments; potentially greater maintenance required at abutments	• Risk with control of groundwater at the east abutment due to presence of native silt till at the footing subgrade • Potential traffic disruption during construction	• Conventional excavation and construction techniques	• Estimated cost is approximately \$600/m^3 of shallow foundations volume, plus protection system costs for 4 m to 4.5 m deep excavation at centre pier
Strip footing perched in approach embankments on granular pad	• Feasible for support of the abutments	• Low risk of post-construction settlement, although some subexcavation of firm cohesive layers may be required in footprint of approach embankments • Minimizes excavation and groundwater control requirements	• Longer bridge spans likely to be required • Does not allow for integral abutment construction • Potential for differential settlement between abutments and pier	• Potential traffic disruption during construction	• Conventional excavation and construction techniques	• Approximately the same cost as spread/strip footings founded on native till. The cost of temporary protection system and concrete for abutment walls would be reduced, but cost for bridge likely increased due to longer span

Option	Feasibility	Advantages	Disadvantages	Risks/Consequences	Constructability	Relative Costs
Steel H-piles driven within “100-blow” lower clayey silt to silt Or Steel H-piles driven within hard clayey silt till / dense silt and sand till at Elevation 278 m	• Feasible and preferred for support of abutments and pier	• Higher geotechnical axial resistance, compared to spread footings • Negligible post construction settlement • Can be used for support of conventional or integral abutments • For piles driven to Elevation 278 m no splicing would be required	• Temporary protection system required during construction to drive piles at the pier • Long piles and a splice will be required to reach “100-blow” materials	• Potential traffic disruption during construction • Negligible risk of post-construction settlement of underpass structure, or of differential settlement of foundation elements • Potentially less costly maintenance over life of the structure than semi-integral abutment structures • Limited risk of vibrations exceeding thresholds at nearby residential/commercial properties, but pre- and post-construction condition surveys and vibration monitoring may be desirable	• Conventional construction methods for driven piles	• Higher costs than spread or strip footings • Estimated cost is approximately \$250/m length for pile installation and \$600/m^3 for pile cap construction • Potentially less costly maintenance over life of the structure than semi-integral abutment structures
Caissons founded within “100-blow” lower clayey silt to silt	• Feasible for support of abutments and pier	• Higher geotechnical axial resistance compared to spread footings and piles; so reduced number of deep foundation elements compared to steel H- or tube piles • At piers, may result in less excavation and a smaller footprint/working area than for spread footing option; may also reduce protection system and groundwater control requirements, particularly if the pile cap can be eliminated and the structural columns extended directly on top of the drilled shafts • Negligible post construction settlement	• Potential for blowout of the caisson base due to the presence of the silty sand to sand deposits under high hydrostatic head • Need for temporary or permanent liners • Temporary protection system required during construction • Concrete would have to be placed by tremie methods below the water level • Cleaning of the base below the water table could be difficult • Not suitable for integral abutment design • Greater risk of encountering obstructions due to larger size of drill hole required	• Risk of disturbance of water-bearing silty sand to sand within the till soils, requiring special construction procedures including use of temporary or permanent liners • Significant traffic disruption during construction due to space required for caisson drilling equipment • Negligible risk of post-construction settlement of overpass structure, or of differential settlement of foundation elements	• Conventional construction methods for drilled shaft foundations; temporary liners required for ground and groundwater control	• Higher cost than steel H-piles • Installation cost could be impacted by need for liner to minimize disturbance and loss of ground and for tremie concrete placement. • Estimated cost is approximately \$1000/m length for caisson installation and \$600/m^3 for pile cap construction (if pile caps are adopted at the pier).

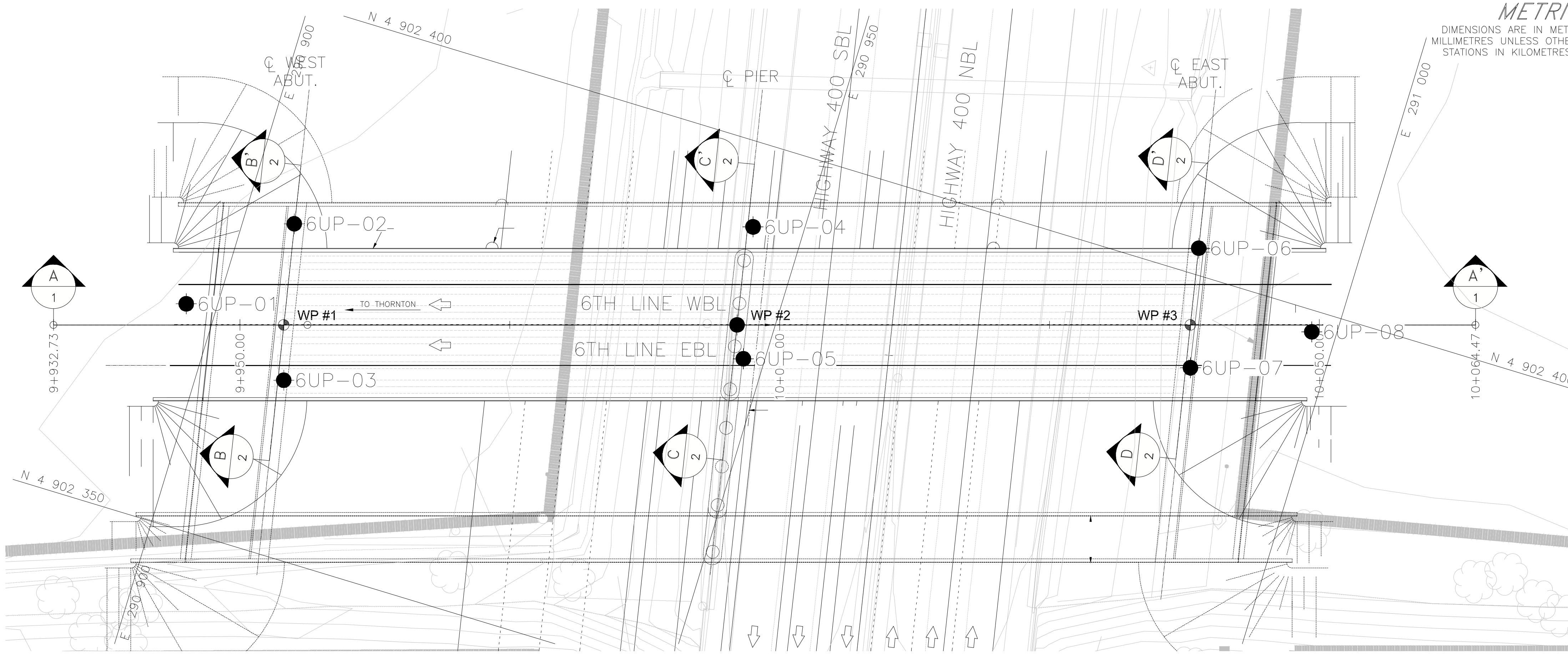
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

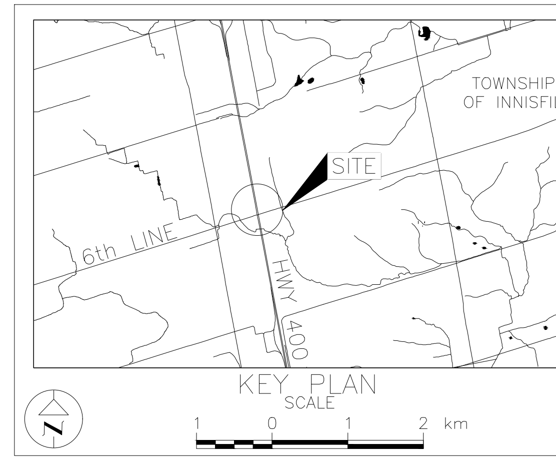
CONT No. 2018-2003
GWP No. 2289-13-00

HIGHWAY 400 / 6TH LINE UNDERPASS
BOREHOLE LOCATIONS AND
SOIL STRATA

SHEET

Golder Associates

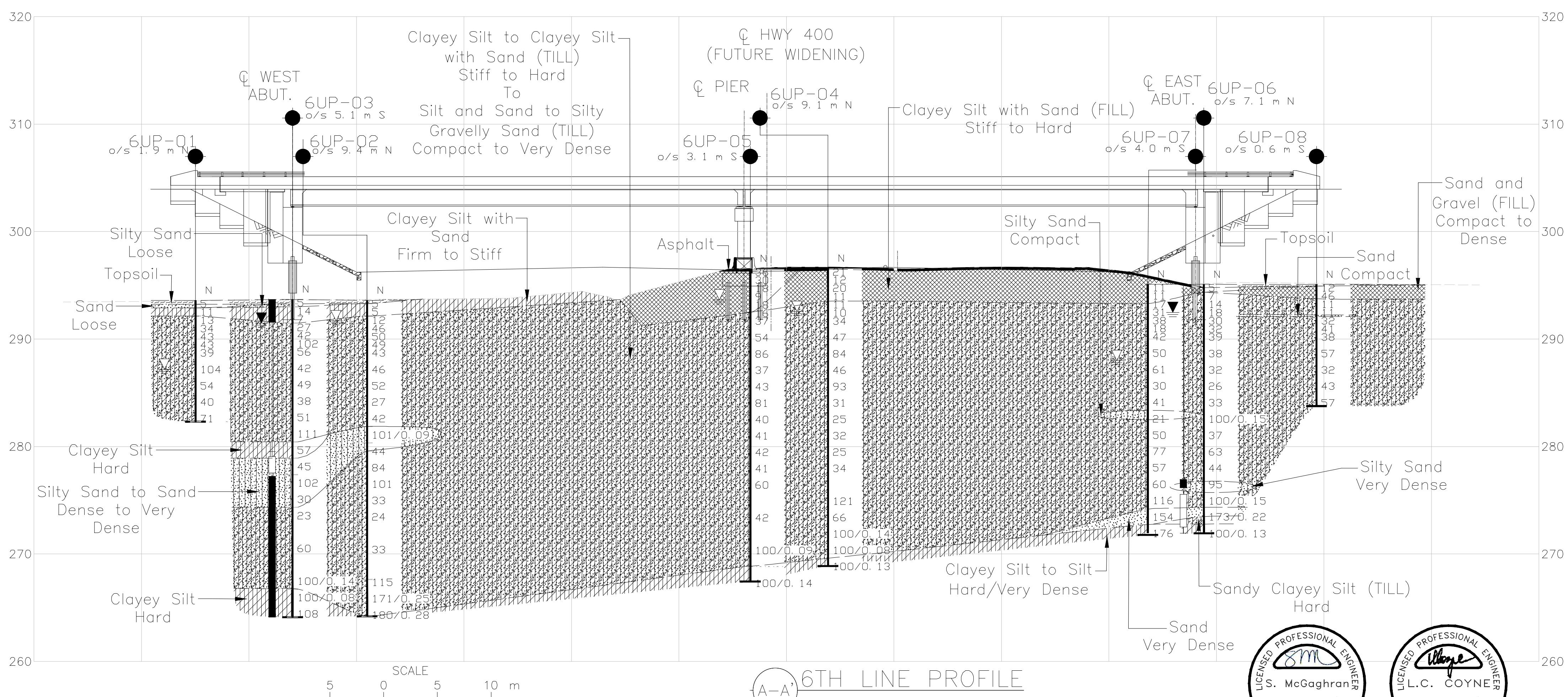




LEGEND

●	Borehole – Current Investigation
▮	Seal
▯	Piezometer
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
▼	WL in piezometer, measured on March 5, 2018
▽	WL upon completion of drilling

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
6UP-01	293.6	4902370.2	290897.2
6UP-02	293.6	4902380.2	290904.6
6UP-03	293.7	4902366.1	290907.9
6UP-04	296.6	4902392.3	290945.4
6UP-05	296.6	4902380.4	290948.1
6UP-06	295.1	4902402.4	290985.5
6UP-07	295.2	4902391.6	290988.0
6UP-08	295.2	4902398.0	290997.8



SCALE 5 0 5 10 m

NOTES

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

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

REFERENCE

Base plans and interim design plan provided in digital format by MorrisonHershfield, drawing file nos. 1170234-MNC - Interim.dwg and x1170234base, dated 2017, received August 28, 2017.
Bridge General Arrangement provided in digital format by Morrison Hershfield, drawing file no. 1170234-01.dwg, Received on January 09, 2018.

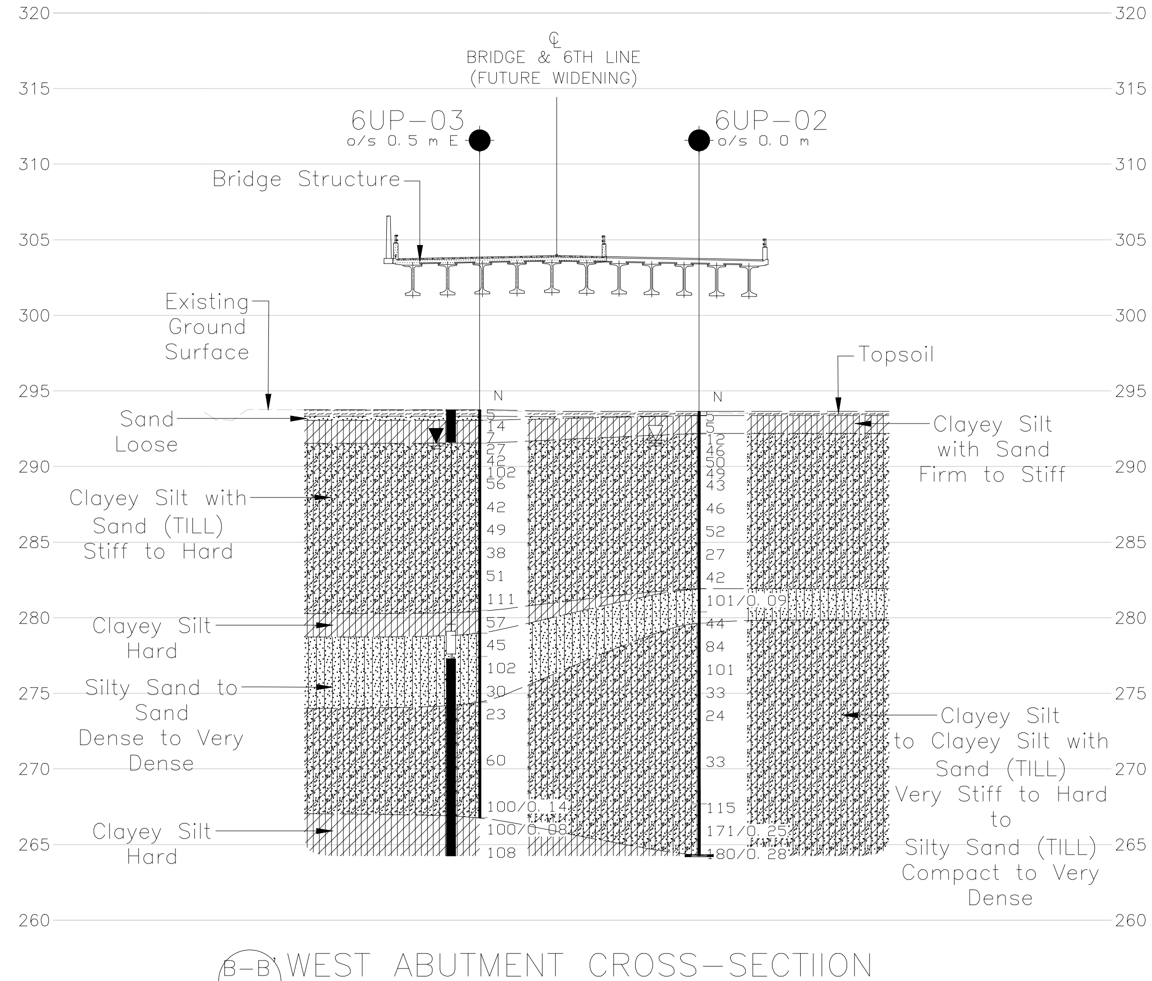
LICENSED PROFESSIONAL ENGINEER S. McGaghran Mar. 09, 2018 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER L.C. COYNE Mar. 09, 2018 PROVINCE OF ONTARIO

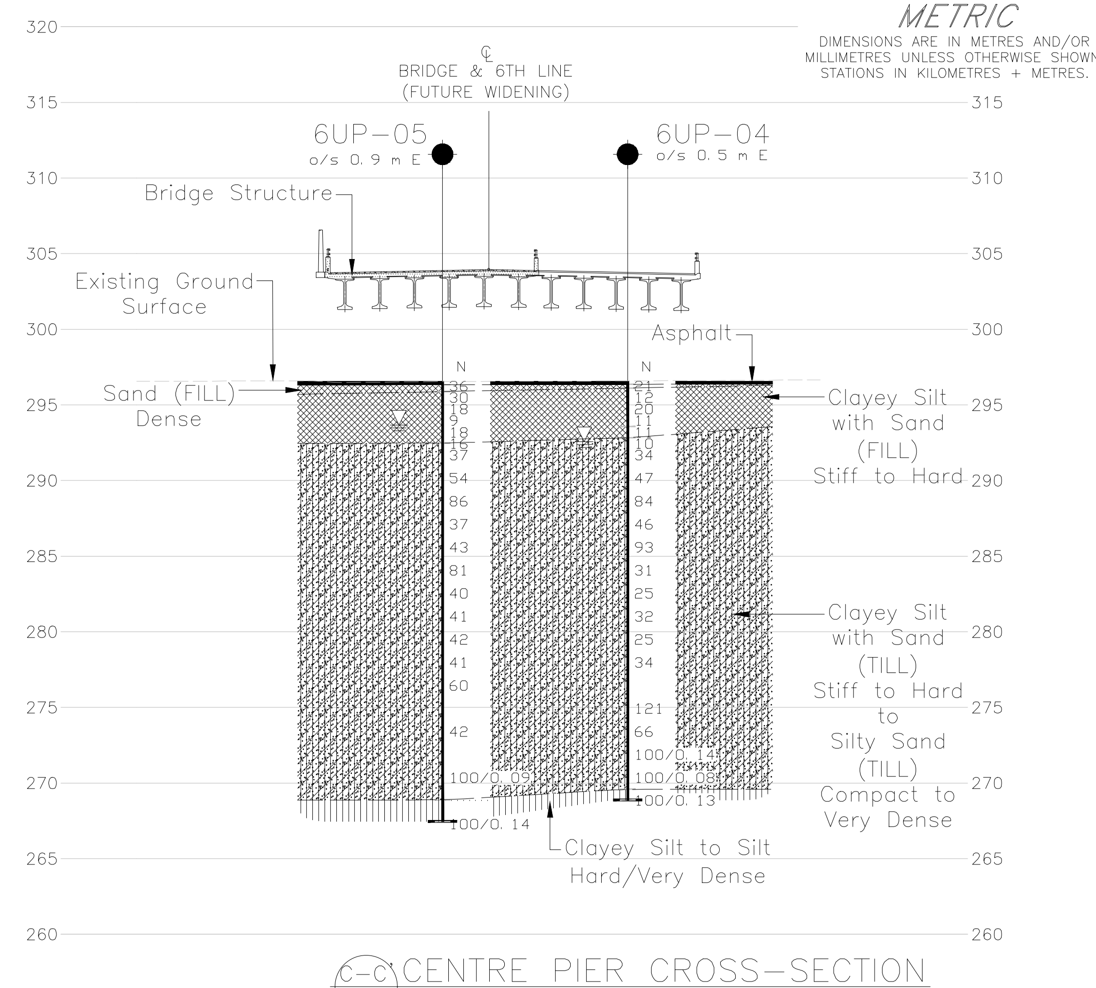
NO.	DATE	BY	REVISION

Geocres No. 31D-695		
HWY. 400	PROJECT NO. 1670268	DIST. CENTRAL
SUBM'D. DF	CHKD. KN	DATE: 3/9/2018	SITE:30-211/1&2
DRAWN: SMD	CHKD. SMM	APPD. LCC	DWG. 1

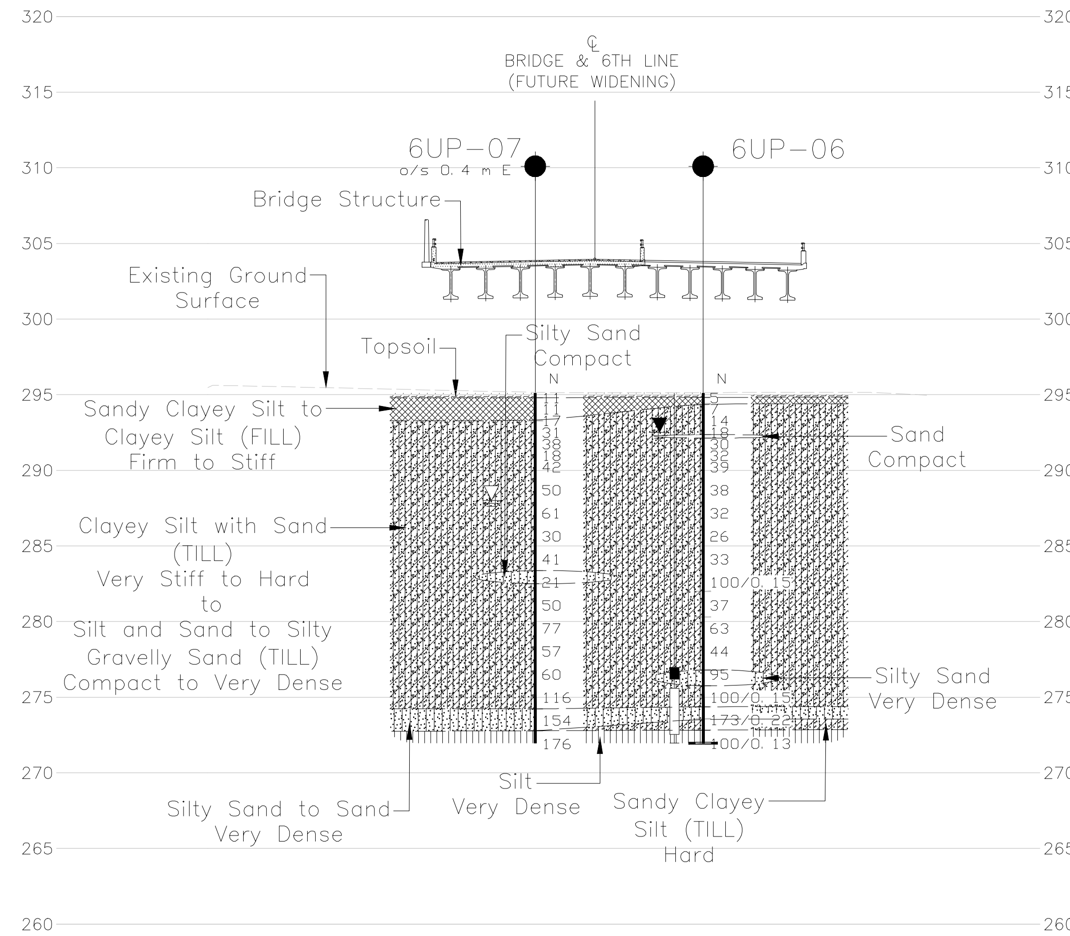
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.



B-B 01 WEST ABUTMENT CROSS-SECTIION



C-C 01 CENTRE PIER CROSS-SECTION



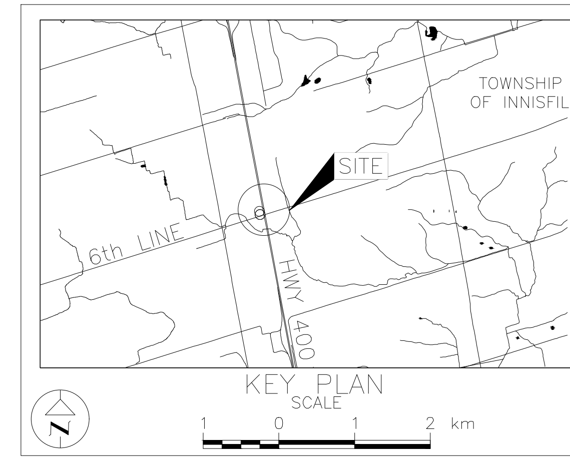
D-D 01 EAST ABUTMENT CROSS-SECTION

CONT No. 2018-2003
GWP No. 2289-13-00

HIGHWAY 400 / 6TH LINE UNDERPASS
SOIL STRATA

SHEET

Golder Associates



KEY PLAN
SCALE 1 0 1 2 km

LEGEND

- Borehole – Current Investigation
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL in piezometer, measured on March 5, 2018
- WL upon completion of drilling

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
6UP-02	293.6	4902380.2	290904.6
6UP-03	293.7	4902366.1	290907.9
6UP-04	296.6	4902392.3	290945.4
6UP-05	296.6	4902380.4	290948.1
6UP-06	295.1	4902402.4	290985.5
6UP-07	295.2	4902391.6	290988.0

NOTES

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

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

REFERENCE

Base plans and interim design plan provided in digital format by MorrisonHershfield, drawing file nos. 1170234-MNC – Interim.dwg and x1170234base, dated 2017, received August 28, 2017.
Bridge General Arrangement provided in digital format by Morrison Hershfield, drawing file no. 1170234-01.dwg, Received on January 09, 2018.

LICENSED PROFESSIONAL ENGINEER S. McGAGHRAN Mar. 09, 2018 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER L.C. COYNE Mar. 09, 2018 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No. 31D-695

HWY. 400	PROJECT NO. 1670268		DIST. CENTRAL
SUBM'D. DF	CHKD. KN	DATE: 3/9/2018	SITE: 30-211/1&2
DRAWN: SMD	CHKD. SMM	APPD. LCC	DWG. 2

MINISTRY OF TRANSPORTATION, ONTARIO

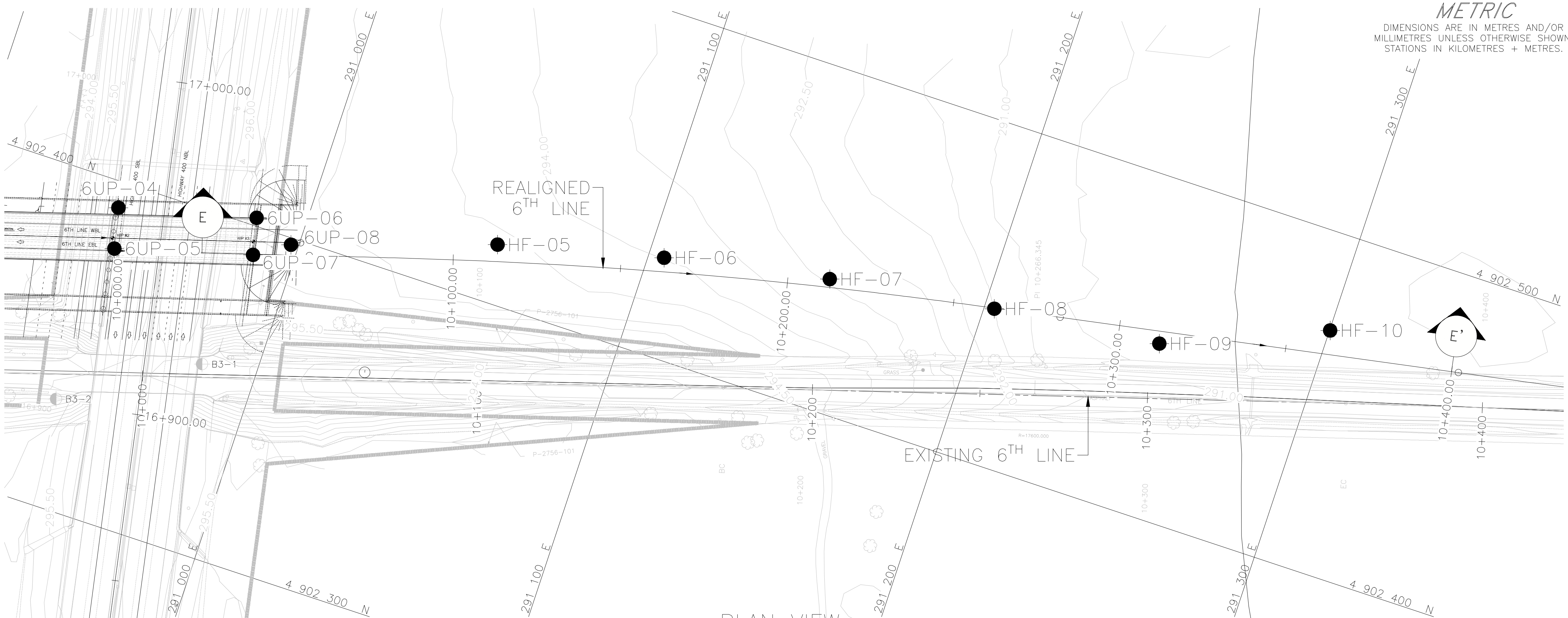
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No. 2018-2003
GWP No. 2289-13-00

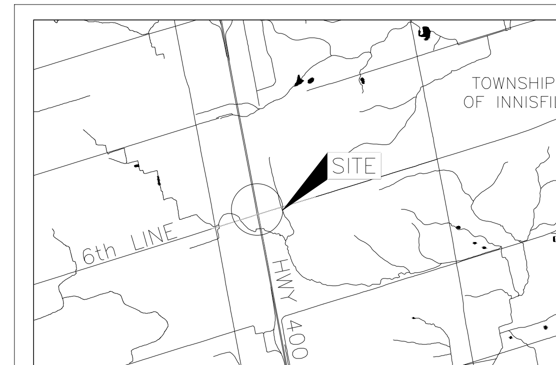
HIGHWAY 400 / 6TH LINE UNDERPASS
HIGH FILL EMBANKMENT
BOREHOLE LOCATIONS AND
SOIL STRATA

SHEET

Golder Associates



PLAN VIEW
SCALE
15 0 15 30 m



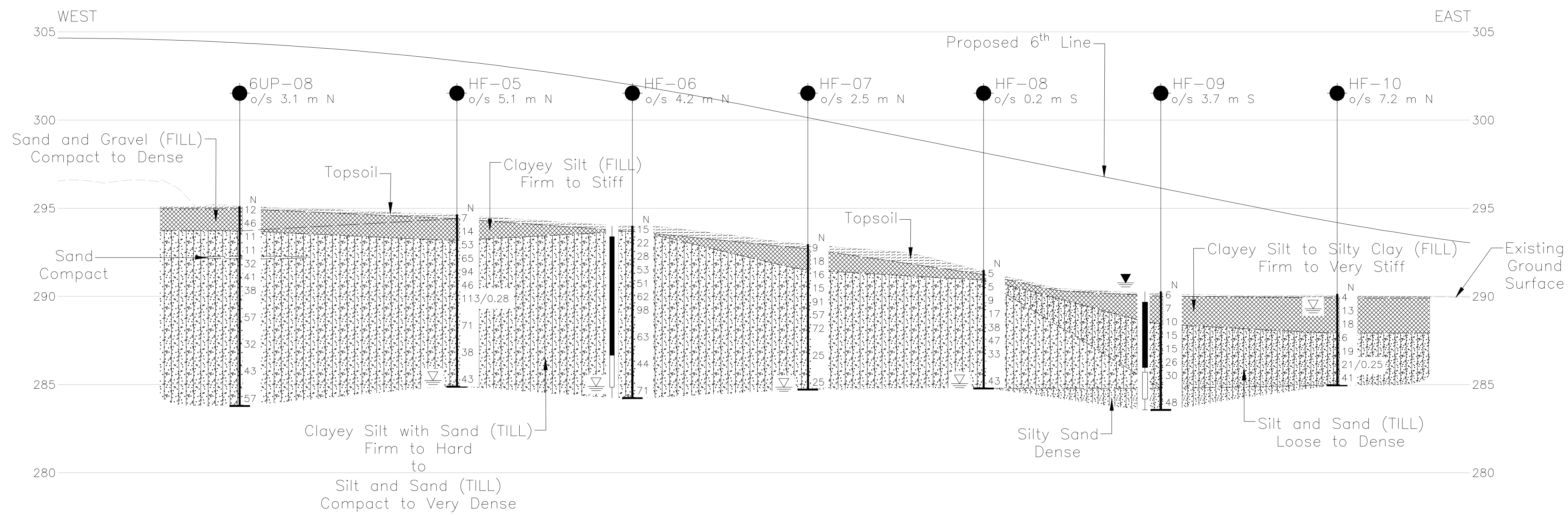
KEY PLAN
SCALE
1 0 1 2 km

LEGEND

●	Borehole – Current Investigation
▮	Seal
▯	Piezometer
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
▼	WL in piezometer, measured on December 4, 2017
▽	WL upon completion of drilling

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
6UP-04	296.6	4902392.3	290945.4
6UP-05	296.6	4902380.4	290948.1
6UP-06	295.1	4902402.4	290985.5
6UP-07	295.2	4902391.6	290988.0
6UP-08	295.2	4902398.0	290997.8
HF-05	294.6	4902417.5	291056.3
HF-06	294.0	4902429.4	291104.7
HF-07	293.0	4902438.9	291153.6
HF-08	291.5	4902446.0	291203.0
HF-09	290.3	4902451.6	291253.0
HF-10	290.1	4902471.4	291300.2



HORIZONTAL SCALE
15 0 15 30 m

VERTICAL SCALE
3 0 3 6 m

E-E' 6TH LINE EMBANKMENT CENTRELINE PROFILE

NOTES

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

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

REFERENCE

Base plans and ultimate design plan provided in digital format by MorrisonHershfield, drawing file ACAD-1170234-Alignment - Ultimate-Model.dwg, dated 2017, received September 18, 2017 and x1170234base, dated 2017, received August 28, 2017.
Bridge General Arrangement provided in digital format by Morrison Hershfield, drawing file no. 1170234-01.dwg, Received on January 09, 2018.

LICENSED PROFESSIONAL ENGINEER S. McGaghran Mar. 09, 2018 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER L.C. COYNE Mar. 09, 2018 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No. 31D-695

HWY. 400	PROJECT NO. 1670268		DIST. CENTRAL
SUBM'D. DF	CHKD. KN	DATE: 3/9/2018	SITE: 30-211/1&2
DRAWN: SMD	CHKD. SMM	APPD. LCC	DWG. 3

PLOT DATE: March 9, 2018
FILENAME: S:\Clients\MTO\Hwy_400_and_6th_Line_Innisfil\99_PROJ\1670268_MorrisonHershfield_GeotechInvestigation\40_PROD\0005_HighFillProfiles\1670268-0005-HC-0001.dwg

MINISTRY OF TRANSPORTATION, ONTARIO

METRIC

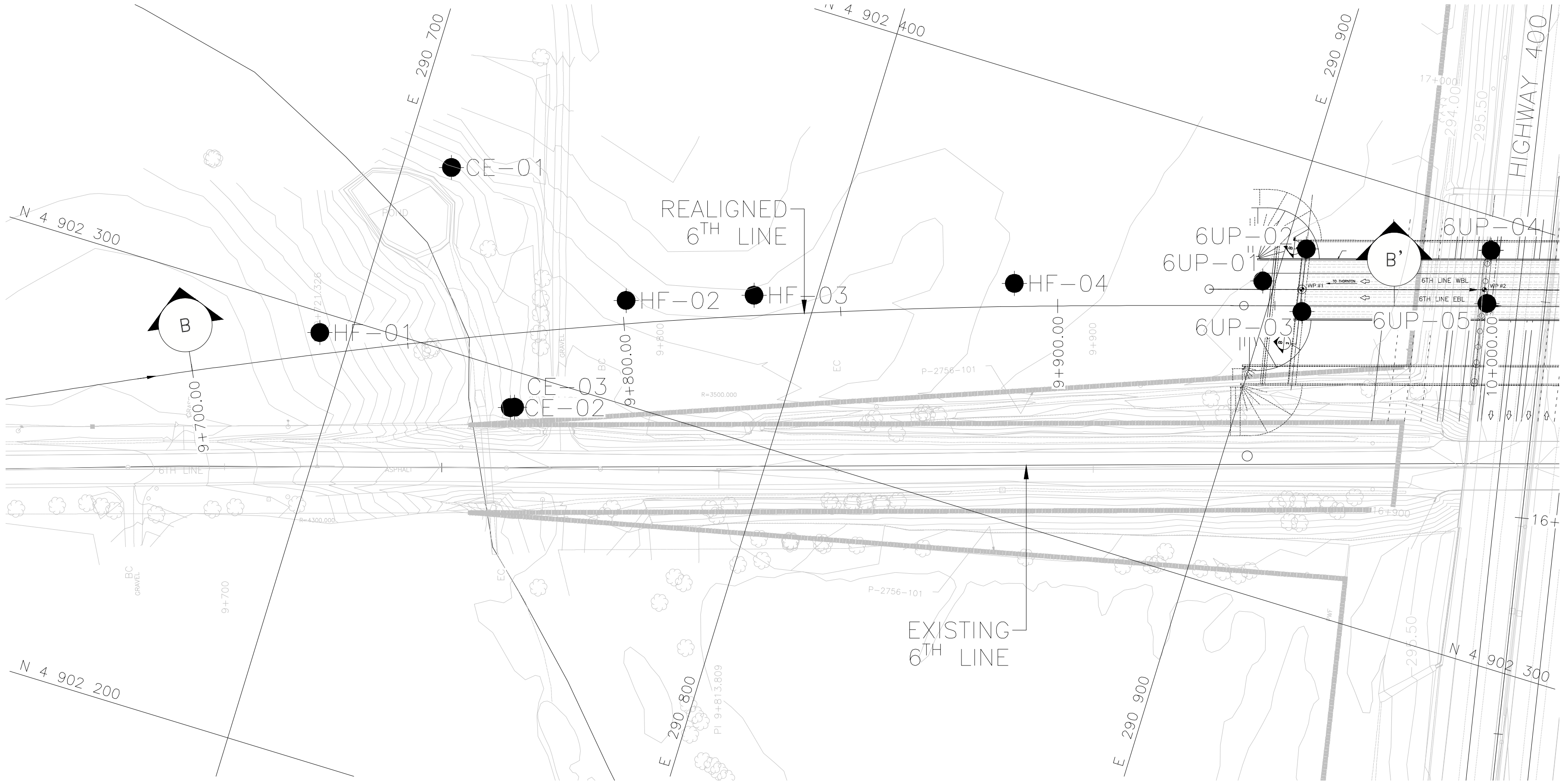
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No. 2018-2003
GWP No. 2289-13-00

SHEET

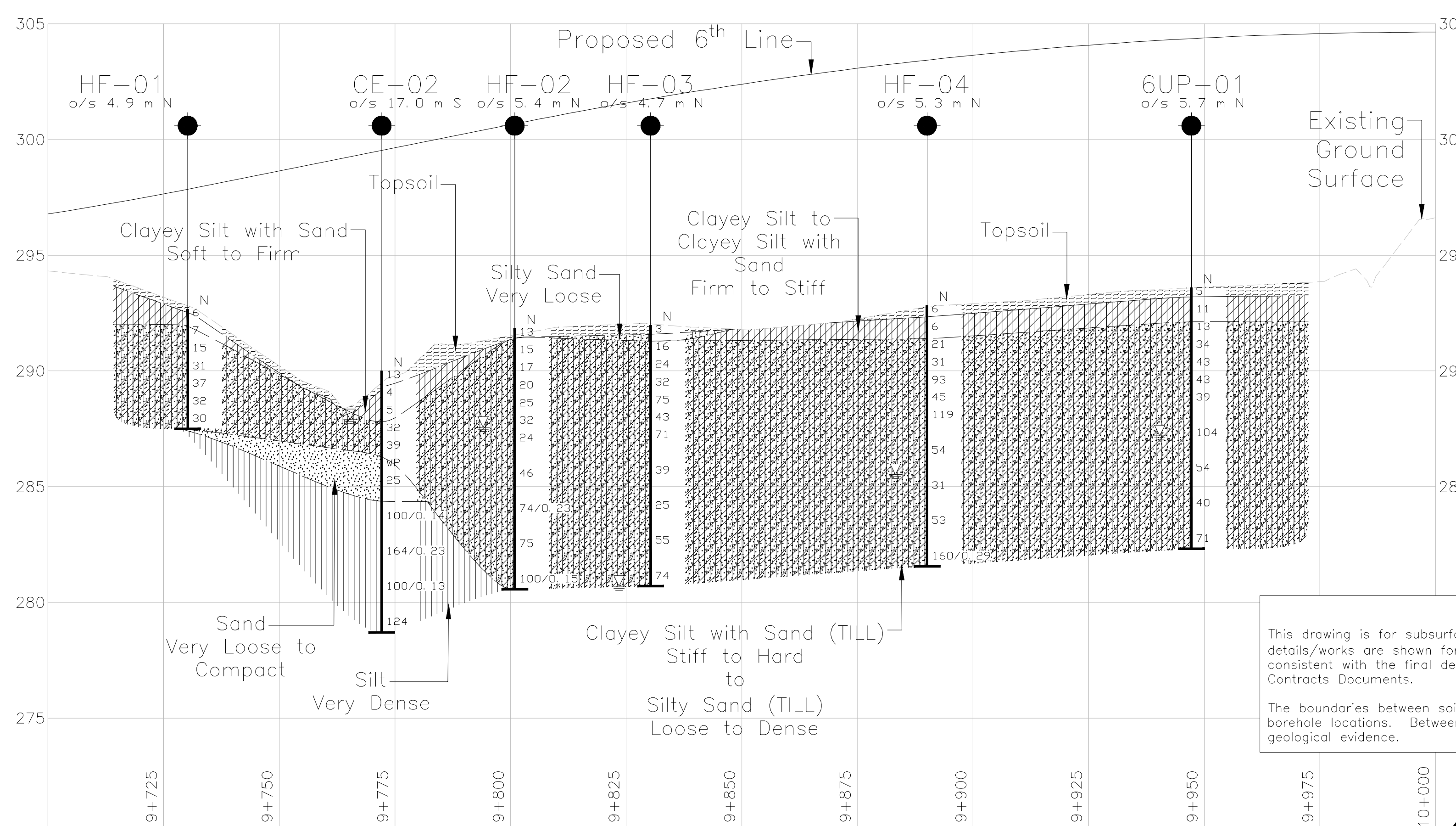
HIGHWAY 400 / 6TH LINE UNDERPASS
HIGH FILL EMBANKMENT
BOREHOLE LOCATIONS AND SOIL STRATA

Golder Associates



SCALE
15 0 15 30 m

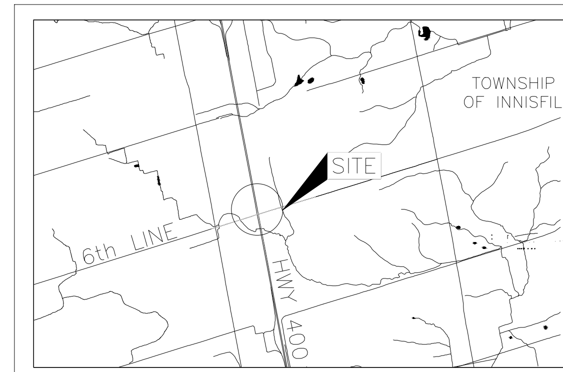
PLAN VIEW



HORIZONTAL SCALE
15 0 15 30 m

VERTICAL SCALE
3 0 3 6 m

B-B' 6TH LINE EMBANKMENT CENTRELINE PROFILE



KEY PLAN

SCALE
1 0 1 2 km

LEGEND

- Borehole – Current Investigation
- Seal
- Piezometer
- N — Standard Penetration Test Value
- WH* — Sample Disturbed
- 16 — Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% — Rock Quality Designation (RQD)
- WL in piezometer, measured on March 5, 2018
- WL upon completion of drilling

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
6UP-01	293.5	4902370.2	290897.2
6UP-02	293.5	4902380.2	290904.6
6UP-03	293.7	4902366.1	290907.9
6UP-04	296.6	4902392.3	290945.4
6UP-05	296.6	4902380.4	290948.1
CE-01	291.3	4902340.6	290710.9
CE-02	290.0	4902291.7	290740.0
CE-03	290.0	4902292.1	290740.9
HF-01	292.7	4902295.4	290693.0
HF-02	291.8	4902323.1	290758.3
HF-03	292.0	4902332.8	290786.1

NOTES

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

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

REFERENCE

Base plans and ultimate design plan provided in digital format by MorrisonHershfield, drawing file ACAD-1170234-Alignment - Ultimate-Model.dwg, dated 2017, received September 18, 2017 and x1170234base, dated 2017, received August 28, 2017.
Bridge General Arrangement provided in digital format by Morrison Hershfield, drawing file no. 1170234-01.dwg, Received on January 09, 2018.

LICENSED PROFESSIONAL ENGINEER
S. McGaghran
Mar. 09, 2018
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
L.C. COYNE
Mar. 09, 2018
PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

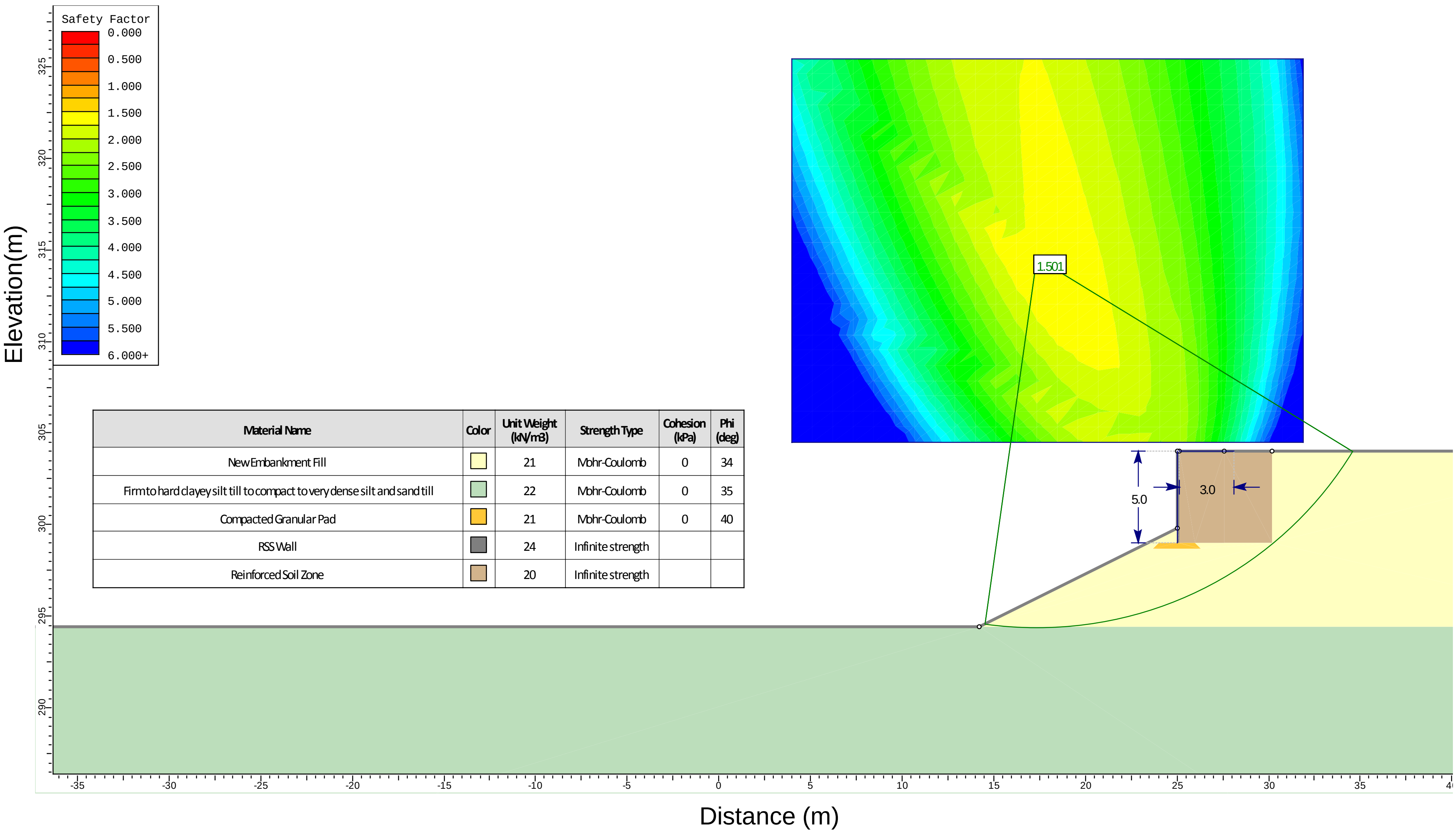
Geocres No. 31D-695

HWY. 400	PROJECT NO. 1670268		DIST. CENTRAL
SUBM'D. DF	CHKD. KN	DATE: 3/9/2018	SITE: 30-211/1&2
DRAWN: SMD	CHKD. SMM	APPD. LCC	DWG. 4

PLOT DATE: March 9, 2018

Highway 400 and 6th Line Underpass
RSS Wall Static Global Slope Stability Results

Figure 1



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
New Embankment Fill		21	Mohr-Coulomb	0	34
Firm to hard clayey silt till to compact to very dense silt and sand till		22	Mohr-Coulomb	0	35
Compacted Granular Pad		21	Mohr-Coulomb	0	40
RSS Wall		24	Infinite strength		
Reinforced Soil Zone		20	Infinite strength		

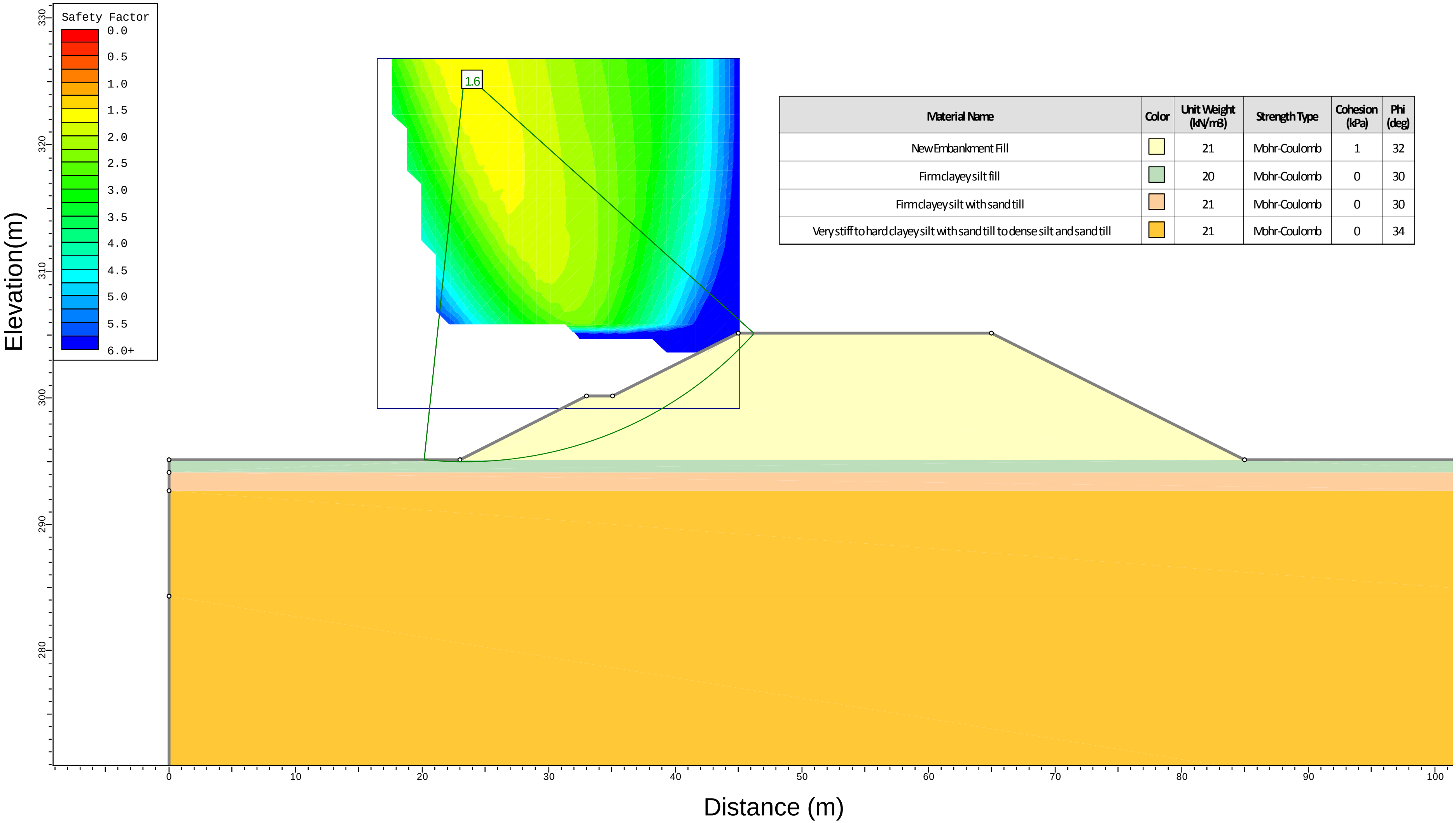
Date: February 2018
Project No: 1670268

Analysis By: MWK **Reviewed By:** SMM

Golder Associates

Highway 400 and 6th Line Underpass, Static Global Slope Stability Results

Figure 2



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
New Embankment Fill		21	Mohr-Coulomb	1	32
Firm clayey silt fill		20	Mohr-Coulomb	0	30
Firm clayey silt with sand till		21	Mohr-Coulomb	0	30
Very stiff to hard clayey silt with sand till to dense silt and sand till		21	Mohr-Coulomb	0	34

Date: February 2018

Project No: 1670268

Analysis By: MWK **Reviewed By:** SMM

APPENDIX A

Borehole Records

LIST OF SYMBOLS

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

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

II.	**STRESS AND STRAIN**
γ	shear strain
Δ	change in, e.g. in stress: $\Delta\,\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
υ	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
σ_1, σ_2, σ_3	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

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

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

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

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

(c)	**Consolidation (one-dimensional)**
C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_α	secondary compression index
m_v	coefficient of volume change
c_v	coefficient of consolidation (vertical direction)
c_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

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

Notes:
1 $\tau = c' + \sigma' \tan\phi'$
2 shear strength = (compressive strength)/2

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance; N_d:

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

PH: Sampler advanced by hydraulic pressure
PM: Sampler advanced by manual pressure
WH: Sampler advanced by static weight of hammer
WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

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

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

Compactness Condition	**N** **Blows/300 mm or Blows/ft**
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils
Consistency

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

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

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

V. MINOR SOIL CONSTITUENTS

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

RECORD OF BOREHOLE No 6UP-01

SHEET 1 OF 1 **METRIC**

PROJECT 1670268
G.W.P. 2289-13-00
LOCATION N 4902370.2; E 290897.2 MTM NAD 83 ZONE 10 (LAT. 44.261202; LONG. -79.674120)
ORIGINATED BY JLS
DIST Central HWY 400
BOREHOLE TYPE Power Auger - 203 mm O.D. Hollow Stem Augers
COMPILED BY KN
DATUM Geodetic
DATE January 10, 2018
CHECKED BY SMM

ELEV DEPTH (m)	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
293.6	GROUND SURFACE					
0.0	TOPSOIL (360 mm)	1	SS	5		
293.2 / 0.4	CLAYEY SILT, some sand, trace gravel. Stiff. Brown-grey. Moist	2	SS	11		1 18 43 38
292.2 / 1.5	CLAYEY SILT with SAND, trace to some gravel, contains cobble fragments (TILL). Stiff to hard. Grey to brown-grey. Moist	3	SS	13		11 40 37 12
		4	SS	34		
		5	SS	43		
		6	SS	43		6 57 23 14
		7	SS	39		
		8	SS	104	∇	
		9	SS	54		
		10	SS	40		
		11	SS	71		
282.3 / 11.3	END OF BOREHOLE					

Elevation scale: 293, 292, 291, 290, 289, 288, 287, 286, 285, 284, 283

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

NOTE:

1. Water level recorded in open borehole at a depth of about 6.4 m (Elev. 287.2 m) below ground surface upon completion of drilling.

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

RECORD OF BOREHOLE No 6UP-02 SHEET 1 OF 3 METRIC

PROJECT 1670268
G.W.P. 2289-13-00
LOCATION N 4902380.2; E 290904.6 MTM NAD 83 ZONE 10 (LAT. 44.261292; LONG. -79.674030)
ORIGINATED BY DMF
DIST Central HWY 400
BOREHOLE TYPE Power Auger - 203 mm O.D. Hollow Stem Augers
COMPILED BY KN
DATUM Geodetic
DATE January 3, 4, and 8, 2018
CHECKED BY SMM

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
293.6	GROUND SURFACE						
0.0 / 293.3	TOPSOIL (280 mm)						
0.3	CLAYEY SILT with SAND, trace gravel; Firm; Brown to grey; Moist	1	SS	5		293	
		2	SS	5			4 42 41 13
292.2 / 1.5	CLAYEY SILT with SAND, trace to some gravel, contains cobble fragments (TILL); Sitff to hard; Brown; Moist	3	SS	12	∇	292	
		4	SS	46		291	6 56 25 13
		5	SS	50		290	
		6	SS	49			
		7	SS	43		289	
	- Grey below a depth of about 6.1 m	8	SS	46		288, 287	
		9	SS	52		286	
		10	SS	27		285, 284	2 58 25 15
		11	SS	42		283	
281.9 / 11.7	Silty SAND; Very dense; Grey; Wet	12	SS	101/0.09		282, 281	
280.3 / 13.3	SAND, trace to some silt, trace clay; Dense; Grey; Moist	13A	SS	44		280	0 86 12 2
279.6 / 14.0		13B				279	

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30
UNIT WEIGHT γ kN/m^3

Continued Next Page

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

GOLDER

RECORD OF BOREHOLE No 6UP-02

SHEET 2 OF 3

METRIC

PROJECT 1670268

G.W.P. 2289-13-00

LOCATION N 4902380.2; E 290904.6 MTM NAD 83 ZONE 10 (LAT. 44.261292; LONG. -79.674030)

ORIGINATED BY DMF

DIST Central HWY 400

BOREHOLE TYPE Power Auger - 203 mm O.D. Hollow Stem Augers

COMPILED BY KN

DATUM Geodetic

DATE January 3, 4, and 8, 2018

CHECKED BY SMM

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION	WATER CONTENT (%)	GR SA SI CL
	CLAYEY SILT with SAND, trace gravel, contains cobble fragments (TILL) Very stiff to hard Grey Moist To Silty SAND, trace to some clay, trace gravel, contains cobble fragments (TILL) Compact to very dense Grey Moist	14	SS	84	278		
		15	SS	101	277–276		
		16	SS	33	275		
		17	SS	24	274–273		
		18	SS	33	271–270		
267.6 26.0	CLAYEY SILT, some sand (TILL) Hard Grey Moist	19	SS	115	268–267		
		20	SS	71/0.25	266		0 19 58 23
		21	SS	80/0.28	265		
264.2 29.4							

Continued Next Page

+3, ×3: Numbers refer to Sensitivity

○3% STRAIN AT FAILURE

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

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

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

UNIT WEIGHT γ kN/m³

RECORD OF BOREHOLE No 6UP-02

SHEET 3 OF 3

METRIC

PROJECT 1670268

G.W.P. 2289-13-00

LOCATION N 4902380.2; E 290904.6 MTM NAD 83 ZONE 10 (LAT. 44.261292; LONG. -79.674030)

ORIGINATED BY DMF

DIST Central HWY 400

BOREHOLE TYPE Power Auger - 203 mm O.D. Hollow Stem Augers

COMPILED BY KN

DATUM Geodetic

DATE January 3, 4, and 8, 2018

CHECKED BY SMM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20 40 60 80 100	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	END OF BOREHOLE										

NOTES:

1. After advancing the borehole to depths of about 14 m and 15.2 m (Elevations 279.6 m and 278.4 m) about 3 m of sand "blew back" inside the hollow stem augers. Water was added to counterbalance the water pressure.

2. Water level measured in open borehole at a depth of about 1.9 m (Elev. 291.7 m) below ground surface upon completion of drilling.

$+^3, \times^3$: Numbers refer to Sensitivity

$O^{3\%}$ STRAIN AT FAILURE

GOLDER

RECORD OF BOREHOLE No 6UP-03

SHEET 1 OF 3 **METRIC**

PROJECT 1670268
G.W.P. 2289-13-00
DIST Central HWY 400
DATUM Geodetic
LOCATION N 4902366.1; E 290907.9 MTM NAD 83 ZONE 10 (LAT. 44.261165; LONG. -79.673990)
BOREHOLE TYPE Power Auger - 203 mm O.D. Hollow Stem Augers
DATE January 8, 9, and 10, 2018
ORIGINATED BY DMF
COMPILED BY KN
CHECKED BY SMM

ELEV DEPTH (m)	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	GR	SA	SI	CL
293.7	GROUND SURFACE								
0.0	TOPSOIL (405 mm)	1	SS	5					
293.3 / 293.0 / 0.7	Silty SAND, trace gravel, trace clay; Loose; Brown; Moist — CLAYEY SILT with SAND, trace gravel, trace rootlets; Firm to stiff; Brownish-grey; Moist	2	SS	14					
		3	SS	7					
291.5 / 2.2	CLAYEY SILT with SAND, some gravel, cobble fragments (TILL); Very stiff to hard; Brown; Moist; - Auger refusal encountered at 3.0 m depth on possible cobble or boulder; borehole advanced 1 m east	4	SS	27					
		5	SS	42					
		6	SS	102					
		7	SS	56		13	52	18	17
	- Grey below a depth of 6.1 m	8	SS	42					
		9	SS	49					
		10	SS	38					
		11	SS	51					
		12	SS	111					
280.4 / 13.3	CLAYEY SILT, trace to some sand; Hard; Grey; Moist	13	SS	57		0	11	63	26
278.9 / 14.8									

Elevation scale: 293, 292, 291, 290, 289, 288, 287, 286, 285, 284, 283, 282, 281, 280, 279

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED — 20 40 60 80 100
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30
UNIT WEIGHT γ kN/m³
REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

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

Foundation Design

GOLDER

RECORD OF BOREHOLE No 6UP-03

SHEET 2 OF 3 **METRIC**

PROJECT 1670268
G.W.P. 2289-13-00
LOCATION N 4902366.1; E 290907.9 MTM NAD 83 ZONE 10 (LAT. 44.261165; LONG. -79.673990)
ORIGINATED BY DMF
DIST Central HWY 400
BOREHOLE TYPE Power Auger - 203 mm O.D. Hollow Stem Augers
COMPILED BY KN
DATUM Geodetic
DATE January 8, 9, and 10, 2018
CHECKED BY SMM

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
	SAND, trace gravel, trace to some silt, trace clay Dense Grey Wet	14	SS	45	278	w	1	88	6	5
277.4 16.3	Silty SAND, trace gravel, plastic fine inclusions Dense to very dense Grey Moist	15	SS	102	277, 276					
		16	SS	30	275					
274.4 19.4	CLAYEY SILT with SAND, trace to some gravel (TILL) Very stiff to hard Grey Moist	17	SS	23	274, 273	w_P–w–w_L	9	42	21	28
		18	SS	60	272, 271, 270, 269					
		19	SS	100/0.14	268	w_P–w–w_L	1	26	48	25
266.7 27.0	CLAYEY SILT, trace to some sand Hard Grey Moist	20	SS	100/0.08	267, 266					
		21	SS	108	265	w_P–w–w_L	0	8	54	38
264.1 29.6										

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED, 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L, WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity

$O^{3\%}$ STRAIN AT FAILURE

GOLDER

Foundation Design

RECORD OF BOREHOLE No 6UP-03

SHEET 3 OF 3 **METRIC**

PROJECT 1670268
G.W.P. 2289-13-00
LOCATION N 4902366.1; E 290907.9 MTM NAD 83 ZONE 10 (LAT. 44.261165; LONG. -79.673990)
ORIGINATED BY DMF
DIST Central HWY 400
BOREHOLE TYPE Power Auger - 203 mm O.D. Hollow Stem Augers
COMPILED BY KN
DATUM Geodetic
DATE January 8, 9, and 10, 2018
CHECKED BY SMM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	END OF BOREHOLE										

NOTES:

1. Water level recorded at a depth of 7.3 m (Elev. 286.4 m) below ground surface upon completion of drilling.

2. Water level measurements in standpipe piezometer:

Date	Depth (m)	Elev. (m)
10/01/18	2.2	291.5
09/02/18	2.2	291.5
05/03/18	1.8	291.9

GTA-MTO 001 S:\CLIENTS\MTO\HWY_400_AND_6TH_LINE_INNISFIL\02_DATA\GINT\1670268.GPJ GAL-GTA.GDT 3/12/18

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

GOLDER

RECORD OF BOREHOLE No 6UP-04

SHEET 1 OF 3 **METRIC**

PROJECT 1670268
G.W.P. 2289-13-00
LOCATION N 4902392.3; E 290945.4 MTM NAD 83 ZONE 10 (LAT. 44.261402; LONG. -79.673520)
ORIGINATED BY DMF
DIST Central HWY 400
BOREHOLE TYPE Power Auger - 203 mm O.D. Hollow Stem Augers
COMPILED BY JL
DATUM Geodetic
DATE October 11, 16 and 17, 2017
CHECKED BY SMM

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
296.6	GROUND SURFACE					
0.0	ASPHALT (215 mm)					
	Sand, some gravel, trace to some fines (FILL) Compact Brown Moist	1A 1B	SS	21		
0.5	Sandy clayey silt, trace gravel, contains clayey organic silt zones (FILL/REWORKED) Stiff to very stiff Mottled brown-grey/brown Moist	2	SS	12		
		3	SS	20		
		4	SS	11		
		5	SS	11		1 21 37 41
292.9 / 3.7	CLAYEY SILT with SAND, trace to some gravel, contains sand pockets, oxidation staining to 4.6 m (TILL) Stiff to hard Brown becoming grey at 8.7 m Moist becoming wet at 8.7 m to Silty SAND, trace to some clay, trace gravel (TILL) Compact to very dense Brown becoming grey at 8.7 m Moist becoming wet at 8.7 m	6	SS	10	∇	6 41 40 13
		7	SS	34		
		8	SS	47		
		9	SS	84		
		10	SS	46		
		11	SS	93		
		12	SS	31		
		13	SS	25		5 55 25 15

ELEVATION SCALE: 296, 295, 294, 293, 292, 291, 290, 289, 288, 287, 286, 285, 284, 283, 282

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L

WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

Continued Next Page

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

GOLDER

PROJECT 1670268

RECORD OF BOREHOLE No 6UP-04 SHEET 2 OF 3 **METRIC**

G.W.P. 2289-13-00 LOCATION N 4902392.3; E 290945.4 MTM NAD 83 ZONE 10 (LAT. 44.261402; LONG. -79.673520) ORIGINATED BY DMF

DIST Central HWY 400 BOREHOLE TYPE Power Auger - 203 mm O.D. Hollow Stem Augers COMPILED BY JL

DATUM Geodetic DATE October 11, 16 and 17, 2017 CHECKED BY SMM

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT	GR	SA	SI	CL
	CLAYEY SILT with SAND, trace to some gravel, contains sand pockets, oxidation staining to 4.6 m (TILL) Stiff to hard Brown becoming grey at 8.7 m Moist becoming wet at 8.7 m to Silty SAND, trace to some clay, trace gravel (TILL) Compact to very dense Brown becoming grey at 8.7 m Moist becoming wet at 8.7 m	14	SS	32	281					
					280					
		15	SS	25		o				
					279					
		16	SS	34	278					
					277					
					276					
		17	SS	121	275	⊢o⊣	5	56	27	12
					274					
		18	SS	66	273					
		19	SS	100/0.14	272	o				
					271					
		20	SS	100/0.08	270					
269.6 27.0	SILT, trace to some clay, trace sand Very dense Grey Moist	21	SS	100/0.13	269	⊢o⊣	0	2	88	10
268.9 27.7										

Continued Next Page

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

GOLDER

Foundation Design

RECORD OF BOREHOLE No 6UP-04

SHEET 3 OF 3 **METRIC**

PROJECT 1670268			
G.W.P. 2289-13-00	LOCATION N 4902392.3; E 290945.4 MTM NAD 83 ZONE 10 (LAT. 44.261402; LONG. -79.673520)	ORIGINATED BY DMF
DIST Central HWY 400	BOREHOLE TYPE Power Auger - 203 mm O.D. Hollow Stem Augers	COMPILED BY JL
DATUM Geodetic	DATE October 11, 16 and 17, 2017	CHECKED BY SMM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	END OF BOREHOLE										

Notes:

1. Water level measured in open borehole at a depth of 4.0 m (Elev. 292.6 m) on October 17, 2017 before start of drilling when borehole was at a depth of 24.7 m.

* The water level measurement is not considered to be representative of the groundwater level due to the introduction of drilling mud/water during boring operations.

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

GOLDER

Foundation Design

PROJECT 1670268

RECORD OF BOREHOLE No 6UP-05

SHEET 1 OF 3

METRIC

G.W.P. 2289-13-00

LOCATION N 4902380.4; E 290948.1 MTM NAD 83 ZONE 10 (LAT. 44.261294; LONG. -79.673480)

ORIGINATED BY DMF

DIST Central HWY 400

BOREHOLE TYPE Power Auger - 203 mm O.D. Hollow Stem Augers

COMPILED BY JL

DATUM Geodetic

DATE October 12 and 16, 2017

CHECKED BY SMM

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
296.6	GROUND SURFACE				
0.0	ASPHALT (240 mm)	1A 1B	SS	36	
0.2	Sand, trace to some gravel, contains clayey organic silt (FILL) Dense Brown Moist				
295.9 0.7	Silt and sand, trace to some clay, trace to some gravel, oxidation staining to 3.7 m (FILL/REWORKED) Loose to Dense Mottled brown/grey Moist	2	SS	30	
		3	SS	18	8 51 34 12
		4	SS	9	
		5	SS	18	
292.5 4.1	CLAYEY SILT with SAND, trace to some gravel, contains sand pockets (TILL) Very stiff to hard Brown, becoming grey at 7.2 m Moist to SILT and SAND, trace to some clay, trace gravel (TILL) Compact to very dense Brown becoming grey at 7.2 m Moist	6A 6B	SS	16	5 50 37 8
		7	SS	37	
		8	SS	54	2 59 27 12
		9	SS	86	10 55 23 12
		10	SS	37	
		11	SS	43	
		12	SS	81	
		13	SS	40	

Continued Next Page

Elevation scale: 296, 295, 294, 293, 292, 291, 290, 289, 288, 287, 286, 285, 284, 283, 282

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m³

$+^3, \times^3$: Numbers refer to Sensitivity

$\circ^{3\%}$ STRAIN AT FAILURE

GTA-MTO 001 S:\CLIENTS\MTO\HWY_400_AND_6TH_LINE_INNISFIL\02_DATA\GINT\1670268.GPJ GAL-GTA.GDT 3/12/18

GOLDER

Foundation Design

RECORD OF BOREHOLE No 6UP-05

SHEET 2 OF 3 **METRIC**

PROJECT 1670268
G.W.P. 2289-13-00
LOCATION N 4902380.4; E 290948.1 MTM NAD 83 ZONE 10 (LAT. 44.261294; LONG. -79.673480)
ORIGINATED BY DMF
DIST Central HWY 400
BOREHOLE TYPE Power Auger - 203 mm O.D. Hollow Stem Augers
COMPILED BY JL
DATUM Geodetic
DATE October 12 and 16, 2017
CHECKED BY SMM

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAYEY SILT with SAND, trace to some gravel, contains sand pockets (TILL) Very stiff to hard Brown, becoming grey at 7.2 m Moist to SILT and SAND, trace to some clay, trace gravel (TILL) Compact to very dense Brown becoming grey at 7.2 m Moist	14	SS	41	281	
		15	SS	42	280	
		16	SS	41	279, 278	
		17	SS	60	277	
		18	SS	42	276, 275, 274	
		19	SS	100/0.09	273, 272, 271	
268.9 27.7	CLAYEY SILT Hard Grey Wet				270, 269, 268	
267.5 29.1		20	SS	100/0.14		0 2 82 16

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity

$\circ^{3\%}$ STRAIN AT FAILURE

GTA-MTO 001 S:\CLIENTS\MTO\HWY_400_AND_6TH_LINE_INNISFIL\02_DATA\GINT\1670268.GPJ GAL-GTA.GDT 3/12/18

GOLDER

PROJECT 1670268	RECORD OF BOREHOLE No 6UP-05	SHEET 3 OF 3	METRIC
G.W.P. 2289-13-00	LOCATION N 4902380.4; E 290948.1 MTM NAD 83 ZONE 10 (LAT. 44.261294; LONG. -79.673480)	ORIGINATED BY DMF	
DIST Central HWY 400	BOREHOLE TYPE Power Auger - 203 mm O.D. Hollow Stem Augers	COMPILED BY JL	
DATUM Geodetic	DATE October 12 and 16, 2017	CHECKED BY SMM	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L / WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	END OF BOREHOLE NOTE: 1. Water level measured in open borehole at depth of 2.9 m (Elev. 293.7 m) on October 16, 2017 before start of decommissioning. * The water level measurement is not considered to be representative of the groundwater level due to introduction of water/drilling mud during boring operations.										

GTA-MTO 001 S:\CLIENTS\MTO\HWY_400_AND_6TH_LINE_INNISFIL\02_DATA\GINT\1670268.GPJ GAL-GTA.GDT 3/12/18

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

RECORD OF BOREHOLE No 6UP-06

SHEET 1 OF 2

METRIC

PROJECT 1670268

G.W.P. 2289-13-00

LOCATION N 4902402.4; E 290985.5 MTM NAD 83 ZONE 10 (LAT. 44.261493; LONG. -79.673020)

ORIGINATED BY DMF

DIST Central HWY 400

BOREHOLE TYPE Power Auger - 203 mm O.D. Hollow Stem Augers

COMPILED BY JL

DATUM Geodetic

DATE October 19 to 20, 2017

CHECKED BY SMM

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
295.1	GROUND SURFACE
294.8 0.0	TOPSOIL (280 mm)
0.3	Clayey silt, some sand, contains rootlets (FILL/REWORKED) Firm Brown Moist
294.4 0.7	Sandy CLAYEY SILT, trace gravel, oxidation staining (TILL) Firm to very stiff Brown Moist
292.4	SAND, trace to some gravel, trace silt Compact Brown Moist
3.0	CLAYEY SILT with SAND, trace to some gravel (TILL) Very stiff to hard Brown, becoming grey at 7.2 m Moist to SILT and SAND, trace to some clay, trace gravel (TILL) Compact to very dense Brown becoming grey at 7.2 m Moist
282.0 13.1	Sandy CLAYEY SILT, trace to some gravel (TILL) Hard Grey Moist
280.3 14.8	

Continued Next Page

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1A 1B	SS	5	
2	SS	7	2 29 34 35
3	SS	14	
4A 4B	SS	18	
5	SS	30	
6	SS	32	
7	SS	39	
8	SS	38	4 59 25 12
9	SS	32	
10	SS	26	
11	SS	33	
12	SS	100/0.15	
13	SS	37	9 25 40 26

STRAT PLOT | GROUND WATER CONDITIONS | ELEVATION SCALE: 295, 294, 293, 292, 291, 290, 289, 288, 287, 286, 285, 284, 283, 282, 281

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT W_P | NATURAL MOISTURE CONTENT W | LIQUID LIMIT W_L

WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m³

$+^3, \times^3$: Numbers refer to Sensitivity

$\circ^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No 6UP-06

SHEET 2 OF 2 **METRIC**

PROJECT 1670268
G.W.P. 2289-13-00
LOCATION N 4902402.4; E 290985.5 MTM NAD 83 ZONE 10 (LAT. 44.261493; LONG. -79.673020)
ORIGINATED BY DMF
DIST Central HWY 400
BOREHOLE TYPE Power Auger - 203 mm O.D. Hollow Stem Augers
COMPILED BY JL
DATUM Geodetic
DATE October 19 to 20, 2017
CHECKED BY SMM

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAYEY SILT, trace to some gravel (TILL) Hard Brown, becoming grey at 7.2 m Moist	14	SS	63		
		15	SS	44		
276.8 18.3	Silty SAND, trace to some clay, trace gravel Very dense Grey Wet	16	SS	95	o	1 65 28 6
275.7 19.4	Sandy CLAYEY SILT, trace to some gravel (TILL) Hard Grey Moist	17	SS	100/0.15		
274.4 20.7	SAND, trace fines Very dense Brown Wet	18A 18B	SS	73/0.22	o	0 97 (3)
273.6 21.5	Sandy CLAYEY SILT, trace to some gravel (TILL) Hard Grey Moist					
272.8 22.3	SILT, trace sand, clay Very dense Grey Wet	19	SS	100/0.13	oH	
272.0 23.1	END OF BOREHOLE					

Elevation scale: 280, 279, 278, 277, 276, 275, 274, 273, 272

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

NOTES:

1. Water level in open borehole at a depth of 7.4 m (Elev. 287.7 m) upon completion of drilling on Ocotober 20, 2017.

2. Water level measurements in standpipe piezometer:

Date	Depth (m)	Elev. (m)
03/11/17	3.5	291.6
14/11/17	3.0	292.1
04/12/17	2.6	292.5
10/01/18	3.0	292.1
09/02/18	2.7	292.4
05/03/18	2.3	292.8

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

RECORD OF BOREHOLE No 6UP-07

SHEET 1 OF 2

METRIC

PROJECT 1670268

G.W.P. 2289-13-00

LOCATION N 4902391.6; E 290988.0 MTM NAD 83 ZONE 10 (LAT. 44.261396; LONG. -79.672990)

ORIGINATED BY DH

DIST Central HWY 400

BOREHOLE TYPE Power Auger - 203 mm O.D. Hollow Stem Augers

COMPILED BY JL

DATUM Geodetic

DATE October 23 to 24, 2017

CHECKED BY SMM

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
295.2	GROUND SURFACE					295						
294.9 / 0.0	TOPSOIL (300 mm)	1A / 1B	SS	11								
0.3	Sandy clayey silt, trace gravel (FILL/REWORKED) Stiff Brown Moist to wet	2	SS	11		294	o					
293.4 / 1.8	CLAYEY SILT with SAND, some gravel, contains sand pockets (TILL) Very stiff to hard Brown, becoming grey at 7.2 m Moist, becoming wet at 7.2 m to Silty Gravelly SAND, trace to some clay (TILL) Compact to very dense Brown becoming grey at 7.2 m Moist becoming wet at 7.2 m	3	SS	17		293						
		4	SS	31			o	NP	26	47	21	6
		5	SS	38		292						
		6	SS	18		291						
		7	SS	42		290						
		8	SS	50		289	o					
					∇	288						
		9	SS	61		287	o├┤		15	51	23	11
		10	SS	30		286						
						285						
		11	SS	41		284						
283.5 / 11.7	Silty SAND, trace to some clay, trace gravel Compact Grey Moist to wet	12A	SS	21		283	o		3	70	21	6
282.6 / 12.6	CLAYEY SILT with SAND, trace to some gravel, contains sand pockets, sand layers encountered between depths of about 15.5 m to 15.6 m (TILL) Very stiff to hard Grey Moist to wet	12B				282						
		13	SS	50		281	o					

Continued Next Page

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

RECORD OF BOREHOLE No 6UP-07

SHEET 2 OF 2 **METRIC**

PROJECT 1670268

G.W.P. 2289-13-00

LOCATION N 4902391.6; E 290988.0 MTM NAD 83 ZONE 10 (LAT. 44.261396; LONG. -79.672990)

ORIGINATED BY DH

DIST Central HWY 400

BOREHOLE TYPE Power Auger - 203 mm O.D. Hollow Stem Augers

COMPILED BY JL

DATUM Geodetic

DATE October 23 to 24, 2017

CHECKED BY SMM

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	UNIT WEIGHT kN/m³	GR	SA	SI	CL
	CLAYEY SILT with SAND, trace to some gravel, contains sand pockets, sand layers encountered between depths of about 15.5 m to 15.6 m (TILL) Very stiff to hard Grey Moist to wet	14	SS	77					
		15	SS	57		9	55	24	12
		16	SS	60					
		17	SS	116					
274.3 / 20.9	Silty SAND, trace to some gravel, trace clay Very dense Grey Moist to wet	18	SS	154	NP	7	69	20	4
272.9 / 22.3	SILT, trace sand, trace clay Very desne Grey Wet	19	SS	176					
271.9 / 23.3	END OF BOREHOLE								

NOTE:

1. Water level measured in open borehole at a depth of 7.1 m (Elev. 288.1 m) on October 24, 2017 before start of drilling when the borehole was at a depth of 7.1 m

Elevation scale: 280, 279, 278, 277, 276, 275, 274, 273, 272

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

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

RECORD OF BOREHOLE No 6UP-08

SHEET 1 OF 1 **METRIC**

PROJECT 1670268
G.W.P. 2289-13-00
LOCATION N 4902398.0; E 290997.8 MTM NAD 83 ZONE 10 (LAT. 44.261454; LONG. -79.672860)
ORIGINATED BY DH
DIST Central HWY 400
BOREHOLE TYPE Power Auger - 203 mm O.D. Hollow Stem Augers
COMPILED BY JL
DATUM Geodetic
DATE October 24, 2017
CHECKED BY SMM

ELEV / DEPTH (m)	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
295.2	GROUND SURFACE				
0.0	TOPSOIL (150 mm)				
0.1	Sand and gravel, some fines (FILL) Compact to dense Brown Moist	1A 1B	SS	12	
		2	SS	46	32 53 (15)
293.8 / 1.4	CLAYEY SILT with SAND, trace to some gravel, contains sand pockets (TILL) Stiff to hard Brown Moist to wet	3	SS	11	8 40 39 13
		4A 4B	SS	11	
292.4 / 3.0	SAND, trace to some gravel, some fines Compact Brown Moist				
	CLAYEY SILT with SAND, trace to some gravel, contains sand pockets (TILL) Hard Brown Moist to wet to Silty SAND, trace to some clay, trace gravel (TILL) Dense to very dense Brown Moist to wet	5	SS	32	5 59 26 10
		6	SS	41	
		7	SS	38	
		8	SS	57	
		9	SS	32	
		10	SS	43	7 55 27 11
283.9		11	SS	57	
11.3	END OF BOREHOLE				

NOTE:

1. Water level not meaured upon completion of drilling.

Column headings: GROUND WATER CONDITIONS; ELEVATION SCALE (295 to 284); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED, 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L, WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³.

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

GOLDER

Foundation Design

RECORD OF BOREHOLE No CE-01

SHEET 1 OF 1 **METRIC**

PROJECT 1670268
G.W.P. 2289-13-00
LOCATION N 4902340.6; E 290710.9 MTM NAD 83 ZONE 10 (LAT. 44.260932; LONG. -79.676453)
ORIGINATED BY LJS
DIST Central HWY 400
BOREHOLE TYPE Power Auger - 203 mm O.D. Hollow Stem Augers
COMPILED BY KN
DATUM Geodetic
DATE January 16, 2018
CHECKED BY SMM

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
291.3	GROUND SURFACE											
0.0	TOPSOIL (690 mm)	1	DO	6		291						
290.6 / 0.7	Silty sand, trace gravel, some clay (FILL / REWORKED TILL) Loose to compact Brown, mottled Moist	2	DO	9		290	o		4	59	24	13
		3	DO	12								
289.1 / 2.2	CLAYEY SILT with SAND, trace gravel, containing cobble fragments (TILL) Hard Brown to grey Moist	4	DO	30		289	o ├─┤		2	55	23	20
		5	DO	80		288						
		6	DO	47		287						
286.3 / 5.0	SAND, trace gravel, trace non-plastic fines Compact Grey Wet	7	DO	58	∇	286						
285.7 / 5.6	Silty SAND, trace gravel (TILL) Very dense Grey Moist	8	DO	126		285						
284.1 / 7.2	SAND, trace gravel, trace non-plastic fines (TILL) Very dense Grey Wet	9	DO	74		284, 283						
		10	DO	65		282						
281.1 / 10.2	SILT, some clay Very dense Grey Moist	11	DO	65		281	o	NP	0	0	87	13
280.0 / 11.3	END OF BOREHOLE					280						

Chart headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; REMARKS & GRAIN SIZE DISTRIBUTION (%)

NOTES:

1. Borehole advanced with water inside the hollow stem augers in order to counterbalance the water pressures.

2. Water level recorded in open borehole at a depth of about 3.0 m (Elev. 288.3 m) below ground surface upon completion of drilling.

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

GTA-MTO 001 S:\CLIENTS\MTO\HWY_400_AND_6TH_LINE_INNISFIL\02_DATA\GINT\1670268.GPJ GAL-GTA.GDT 3/12/18

GOLDER

Foundation Design

RECORD OF BOREHOLE No CE-02

SHEET 1 OF 2 **METRIC**

PROJECT 1670268

G.W.P. 2289-13-00 LOCATION N 4902291.7; E 290740.0 MTM NAD 83 ZONE 10 (LAT. 44.260490; LONG. -79.676090) ORIGINATED BY DMF

DIST Central HWY 400 BOREHOLE TYPE Power Auger - 203 mm O.D. Hollow Stem Augers COMPILED BY KN

DATUM Geodetic DATE January 15, 2018 CHECKED BY SMM

Column heads: SOIL PROFILE (ELEV / DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20, 40, 60, 80, 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (10, 20, 30); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) (GR SA SI CL)

ELEV / DEPTH	DESCRIPTION
290.0	GROUND SURFACE
0.0	TOPSOIL (690 mm)
289.3 / 0.7	CLAYEY SILT with SAND, trace gravel Soft to firm Brown Moist to wet
287.8 / 2.2	CLAYEY SILT with SAND, some gravel, trace cobble fragments (TILL) Hard Brown to grey Moist to wet
286.3 / 3.7	SAND, some silt, trace to some clay, trace gravel Very loose to compact Brown to grey Wet
284.4 / 5.6	SILT, trace sand, trace to some clay Very dense Grey Moist to wet
278.7 / 11.3	END OF BOREHOLE

NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
1	DO	13				
2	DO	4	2	32	43	23
3	DO	5				
4	DO	32	18	50	15	17
5	DO	39				
6	DO	WH*				
7	DO	25	2	72	19	7
8	DO	100/0.14				
9	DO	164/0.23	0	3	92	5
10	DO	100/0.13				
11	DO	124				

Elevation scale: 289, 288, 287, 286, 285, 284, 283, 282, 281, 280, 279

NOTES:

1. Sample 6 is likely disturbed, see borehole record for CE-03 which was advanced with water inside the hollow stem augers in order to counterbalance the water pressures.

2. Water level recorded in open borehole at a depth of 2.1 m (Elev. 287.9 m) below ground surface upon completion of drilling.

Continued Next Page

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

RECORD OF BOREHOLE No CE-02

SHEET 2 OF 2 **METRIC**

PROJECT 1670268

G.W.P. 2289-13-00

LOCATION N 4902291.7; E 290740.0 MTM NAD 83 ZONE 10 (LAT. 44.260490; LONG. -79.676090)

ORIGINATED BY DMF

DIST Central HWY 400

BOREHOLE TYPE Power Auger - 203 mm O.D. Hollow Stem Augers

COMPILED BY KN

DATUM Geodetic

DATE January 15, 2018

CHECKED BY SMM

SOIL PROFILE		SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L / WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE			
	--- CONTINUED FROM PREVIOUS PAGE ---									
	3. Water level measurements in standpipe piezometer:									
	Date / Depth (m) / Elev. (m)									
	05/03/18 / 2.3 / 287.7									

$+^3, \times^3$: Numbers refer to Sensitivity

$\bigcirc^{3\%}$ STRAIN AT FAILURE

GOLDER

Foundation Design

RECORD OF BOREHOLE No CE-03

SHEET 1 OF 1

METRIC

PROJECT 1670268

G.W.P. 2289-13-00

LOCATION N 4902292.1; E 290740.9 MTM NAD ZONE 10 (LAT. 44.260504; LONG. -79.676079)

ORIGINATED BY DMF

DIST Central HWY 400

BOREHOLE TYPE Power Auger - 203 mm O.D. Hollow Stem Augers

COMPILED BY KN

DATUM Geodetic

DATE January 16, 2018

CHECKED BY SMM

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
290.0	GROUND SURFACE					
0.0	Refer to stratigraphy in Borehole CE-02					
287.0 / 3.0	CLAYEY SILT with SAND, trace gravel, containing cobble fragments (TILL) Dense Grey Moist	1	DO	34		
286.3 / 3.7	SAND, trace to some silt, trace gravel, trace clay Dense to compact Grey Wet	2	DO	43	20	1 84 12 3
		3	DO	29		
284.8 / 5.2	END OF BOREHOLE					

Chart column headings: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (289, 288, 287, 286, 285); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED, 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³

NOTES:

1. Borehole advanced with water inside the hollow stem augers in order to counterbalance the water pressures.

2. Water level recorded in open borehole at a depth of about 1.2 m (Elev. 288.8 m) below ground surface upon completion of drilling.

3. Water level measurements in standpipe piezometer:

Date	Depth (m)	Elev. (m)
09/02/18	1.3	288.7
05/03/18	1.1	288.9

$+^3, \times^3$: Numbers refer to Sensitivity

$\circ^{3\%}$ STRAIN AT FAILURE

GTA-MTO 001 S:\CLIENTS\MTO\HWY_400_AND_6TH_LINE_INNISFIL\02_DATA\GINT\1670268.GPJ GAL-GTA.GDT 3/12/18

GOLDER

Foundation Design

RECORD OF BOREHOLE No HF-01

SHEET 1 OF 1

METRIC

PROJECT 1670268

G.W.P. 2289-13-00

LOCATION N 4902295.4; E 290693.0 MTM NAD 83 ZONE 10 (LAT. 44.260529; LONG. -79.676674)

ORIGINATED BY DMF

DIST Central HWY 400

BOREHOLE TYPE Power Auger - 203 mm O.D. Hollow Stem Augers

COMPILED BY JL

DATUM Geodetic

DATE January 19, 2018

CHECKED BY SMM

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
292.7	GROUND SURFACE									
0.0	TOPSOIL (610 mm)	1	SS	6						
292.1 0.6	CLAYEY SILT with SAND, trace to some gravel, contains cobble fragments (TILL); Firm to hard Grey-brown Moist to Silty SAND, trace to some clay, trace to some gravel, contains cobble fragments (TILL) Loose to dense Grey-brown Moist	2	SS	7		292	6	56	25	13
		3	SS	15		291				
		4	SS	31		290				
		5	SS	37		289	6	57	23	14
		6	SS	32		288				
		7	SS	30						
287.5 5.2	END OF BOREHOLE NOTE: 1. Borehole dry upon completion of drilling.									

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L

WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m³

$+^3, \times^3$: Numbers refer to Sensitivity

$O^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No HF-02

SHEET 1 OF 1

METRIC

PROJECT 1670268

G.W.P. 2289-13-00

LOCATION N 4902323.1; E 290758.3 MTM NAD 83 ZONE 10 (LAT. 44.260772; LONG. -79.675865)

ORIGINATED BY DMF

DIST Central HWY 400

BOREHOLE TYPE Power Auger - 203 mm O.D. Hollow Stem Augers

COMPILED BY KN

DATUM Geodetic

DATE January 11, 2018

CHECKED BY SMM

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
291.8	GROUND SURFACE							
0.0	TOPSOIL (380 mm)	1	SS	13				
291.4 0.4	CLAYEY SILT with SAND, trace to some gravel, cobble fragments (TILL) Stiff to hard Grey-brown with oxidation stains Moist To Silty SAND, trace to some clay, cobble fragments (TILL) Compact to very dense Grey-brown with oxidation stains Moist	2	SS	15		291		
		3	SS	17		290		8 54 24 14
		4	SS	20		289		
		5	SS	25		288		
		6	SS	32	▽			
		7	SS	24		287		5 58 21 16
						286		
		8	SS	46		285		
		9	SS	74/0.23		284		
						283		
		10	SS	75		282		2 63 24 11
		11	SS	100/0.15		281		
280.5 11.3	END OF BOREHOLE NOTE: 1. Water level recorded in open borehole at a depth of about 4.3 m (Elev. 287.5 m) below ground surface upon completion of drilling.							

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

UNIT WEIGHT γ kN/m³

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

GOLDER

RECORD OF BOREHOLE No HF-03

SHEET 1 OF 1 **METRIC**

PROJECT 1670268
G.W.P. 2289-13-00
LOCATION N 4902332.8; E 290786.1 MTM NAD 83 ZONE 10 (LAT. 44.260929; LONG. -79.675278)
ORIGINATED BY DMF
DIST Central HWY 400
BOREHOLE TYPE Power Auger - 203 mm O.D. Hollow Stem Augers
COMPILED BY KN
DATUM Geodetic
DATE January 11, 2018
CHECKED BY SMM

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
292.0	GROUND SURFACE						
0.0	TOPSOIL (405 mm)	1	SS	3			
291.6 / 291.3 / 0.7	Silty SAND, trace gravel; Very loose; Brown, mottled; Moist						
	CLAYEY SILT with SAND, trace to some gravel, trace cobble fragments (TILL); Very stsiff to hard; Brown, becoming grey at 3.0 m; Moist	2	SS	16		291	19 46 22 13
		3	SS	24		290	
		4	SS	32		289	
		5	SS	75			5 54 26 15
		6	SS	43		288	
		7	SS	71	∇	287	
						286	
		8	SS	39		285	
		9	SS	25		284	1 57 25 17
						283	
		10	SS	55		282	
280.7		11	SS	74		281	
11.3	END OF BOREHOLE						

NOTE:

1. Water level recorded at a depth of 4.9 m (287.1 m) below ground surface upon completion of drilling.

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

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

Foundation Design

GOLDER

RECORD OF BOREHOLE No HF-04

SHEET 1 OF 1

METRIC

PROJECT 1670268

G.W.P. 2289-13-00

LOCATION N 4902353.0; E 290842.7 MTM NAD 83 ZONE 10 (LAT. 44.261072; LONG. -79.674684)

ORIGINATED BY DMF

DIST Central HWY 400

BOREHOLE TYPE Power Auger - 203 mm O.D. Hollow Stem Augers

COMPILED BY KN

DATUM Geodetic

DATE January 11, 2018

CHECKED BY SMM

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
292.8	GROUND SURFACE						
0.0	TOPSOIL (510 mm)	1	SS	6			
292.3 / 0.5	CLAYEY SILT with SAND, trace gravel Firm Brown Moist	2	SS	6		292	3 43 37 17
291.4 / 1.5	CLAYEY SILT with SAND, trace to some gravel, cobble fragments (TILL) Very stiff to hard Brown, becoming grey at 6.1 m Moist	3	SS	21		291	
		4	SS	31		290	
		5	SS	93		289	
		6	SS	45			
		7	SS	119		288	9 51 24 16
		8	SS	54		287, 286	
					▽		
		9	SS	31		285	4 58 23 15
		10	SS	53		284, 283	
		11	SS	60/0.29		282	
281.5 / 11.3	END OF BOREHOLE						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

NOTE:

1. Water level recorded in open borehole at a depth of about 7.3 m (Elev. 285.5 m) below ground surface upon completion of drilling.

GTA-MTO 001 S:\CLIENTS\MTO\HWY_400_AND_6TH_LINE_INNISFIL\02_DATA\GINT\1670268.GPJ GAL-GTA.GDT 3/12/18

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

GOLDER

Foundation Design

RECORD OF BOREHOLE No HF-05

SHEET 1 OF 1

METRIC

PROJECT 1670268

G.W.P. 2289-13-00

LOCATION N 4902417.5; E 291056.3 MTM NAD 83 ZONE 10 (LAT. 44.261630; LONG. -79.672130)

ORIGINATED BY DH

DIST Central HWY 400

BOREHOLE TYPE Power Auger - 203 mm O.D. Hollow Stem Augers

COMPILED BY JL

DATUM Geodetic

DATE October 24, 2017

CHECKED BY SMM

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
294.6	GROUND SURFACE					
0.0	TOPSOIL (230 mm)	1A 1B	SS	7		
0.2	Clayey silt, some sand to sandy (FILL) Firm to stiff Brown Moist - Rootlets from a depth of about 0.2 m to 0.6 m	2	SS	14		
293.2 / 1.5	CLAYEY SILT with SAND, trace to some gravel, contains sand pockets (TILL) Hard Brown, becoming grey at 7.6 m Moist	3	SS	53		
		4	SS	65		
		5	SS	94		
		6	SS	46		
		7	SS	113/0.28		11 51 27 11
		8	SS	71		
		9	SS	38		
284.9		10	SS	43	∇	
9.8	END OF BOREHOLE					

Elevation scale: 294, 293, 292, 291, 290, 289, 288, 287, 286, 285

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

NOTES:

1. Water level measured in open borehole at a depth of 9.4 m (Elev. 285.2 m) below ground surface upon completion of drilling.

$+^3$, $\times^3$: Numbers refer to Sensitivity

$\bigcirc^{3\%}$ STRAIN AT FAILURE

Foundation Design

GOLDER

RECORD OF BOREHOLE No HF-06

SHEET 1 OF 1 **METRIC**

PROJECT 1670268
G.W.P. 2289-13-00
LOCATION N 4902429.4; E 291104.7 MTM NAD 83 ZONE 10 (LAT. 44.261739; LONG. -79.671524)
ORIGINATED BY DH
DIST Central HWY 400
BOREHOLE TYPE Power Auger - 203 mm O.D. Hollow Stem Augers
COMPILED BY JL
DATUM Geodetic
DATE October 25, 2017
CHECKED BY SMM

ELEV DEPTH (m)	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
294.0	GROUND SURFACE					
0.0	TOPSOIL (300 mm)	1	SS	15		
293.7 / 0.3	CLAYEY SILT with SAND, trace to some gravel, contains sand pockets (TILL) Very stiff to hard Brown Moist, becoming wet at 9.1 m	2	SS	22	293	8 43 38 11
		3A / 3B	SS	28	292	
		4	SS	53	291	
		5	SS	51		
		6	SS	62	290	
		7	SS	98	289	13 51 26 10
		8	SS	63	288	
		9	SS	44	287, 286	
		10A / 10B	SS	71	285	
284.3 / 9.8	END OF BOREHOLE					

NOTES:

1. Water level measured in open borehole at a depth of about 9.1 m (Elev. 284.9 m) below ground surface upon completion of drilling on October 25, 2017.

2. Standpipe piezometer damaged on site after installation, and unable to be monitored.

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

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

GTA-MTO 001 S:\CLIENTS\MTO\HWY_400_AND_6TH_LINE_INNISFIL\02_DATA\GINT\1670268.GPJ GAL-GTA.GDT 3/12/18

GOLDER

RECORD OF BOREHOLE No HF-07

SHEET 1 OF 1 **METRIC**

PROJECT 1670268			
G.W.P. 2289-13-00	LOCATION N 4902438.9; E 291153.6 MTM NAD 83 ZONE 10 (LAT. 44.261825; LONG. -79.670912)		ORIGINATED BY DH
DIST Central	HWY 400	BOREHOLE TYPE Power Auger - 203 mm O.D. Hollow Stem Augers	COMPILED BY JL
DATUM Geodetic		DATE October 25, 2017	CHECKED BY SMM

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
293.0	GROUND SURFACE						
0.0	TOPSOIL (250 mm)	1A 1B	SS	9			
0.3	Sandy clayey silt, trace gravel, containing rootlets (FILL) Stiff to very stiff Light brown Moist	2	SS	18		292	
291.6 1.5	SILT and SAND, trace to some gravel, trace to some clay (TILL) Compact to very dense Brown, becoming grey at 6.1 m Moist to CLAYEY SILT with SAND, trace to some gravel, contains sand pockets (TILL) Very stiff to hard Brown, becoming grey at 6.1 m Moist	3	SS	16		291	6 46 37 11
		4	SS	15		290	
		5	SS	91			
		6	SS	57		289	8 56 23 13
		7	SS	72		288	
						287	
		8	SS	25		286	
284.8		9	SS	25	∇	285	
8.2	END OF BOREHOLE						

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

NOTES:

1. Water level measured in open borehole at a depth of 8.1 m (Elev. 284.9 m) below ground surface upon completion of drilling.

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

GOLDER

Foundation Design

RECORD OF BOREHOLE No HF-08

SHEET 1 OF 1 **METRIC**

PROJECT 1670268

G.W.P. 2289-13-00

LOCATION N 4902446.0; E 291203.0 MTM NAD 83 ZONE 10 (LAT. 44.261890; LONG. -79.670293)

ORIGINATED BY DH

DIST Central HWY 400

BOREHOLE TYPE Power Auger - 203 mm O.D. Hollow Stem Augers

COMPILED BY JL

DATUM Geodetic

DATE October 25, 2017

CHECKED BY SMM

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
291.5	GROUND SURFACE							
0.0	TOPSOIL (250 mm)	1A / 1B	SS	5				
290.9 / 0.6	Sandy clayey silt to clayey silt and sand (FILL) Firm Light brown Moist					291		
	SILT and SAND, trace to some clay, trace to some gravel (TILL) Loose to dense Brown, becoming grey at 3.0 m Moist	2	SS	5				5 46 39 10
	to	3	SS	9		290		
	CLAYEY SILT with SAND, trace to some gravel, contains sand pockets (TILL) Firm to hard Brown, becoming grey at 3.0 m Moist - Oxidation staining at depths between 2.3 m and 2.9 m	4	SS	17		289		
		5	SS	38		288		16 50 23 11
		6	SS	47		287		
		7	SS	33		286		
284.8		8	SS	43	∇	285		
6.7	END OF BOREHOLE							

Dynamic cone penetration resistance plot: 20 40 60 80 100

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

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

UNIT WEIGHT γ kN/m³

NOTES:

1. Water leval measured in open borehole at a depth of 6.4 m (Elev. 285.1 m) below ground surface upon completion of drilling.

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

Foundation Design

RECORD OF BOREHOLE No HF-09

SHEET 1 OF 1 **METRIC**

PROJECT 1670268

G.W.P. 2289-13-00 LOCATION N 4902451.6; E 291253.0 MTM NAD 83 ZONE 10 (LAT. 44.261941; LONG. -79.669667) ORIGINATED BY DH

DIST Central HWY 400 BOREHOLE TYPE Power Auger - 203 mm O.D. Hollow Stem Augers COMPILED BY JL

DATUM Geodetic DATE October 27, 2017 CHECKED BY SMM

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
290.3	GROUND SURFACE						
0.0	TOPSOIL (250 mm)	1A	SS	6	290		
0.3	Clayey silt, some sand, oxidation staining (FILL) Firm Brown Moist	1B					
289.4 / 0.9	Silty clay, trace sand (FILL) Firm to stiff Brown to grey Moist	2	SS	7	289		
		3A	SS	10		43	0 4 36 60
288.5 / 1.8	SILT and SAND, some clay, trace to some gravel (TILL) Compact to dense Grey Moist to wet	3B			288		
		4	SS	15			5 45 38 12
		5	SS	15	287		
		6	SS	26	286		
		7	SS	30	285		
284.8 / 5.5	Silty SAND, some gravel, trace clay Dense Grey Wet						
		8	SS	48	284		16 54 27 3
283.6 / 6.7	END OF BOREHOLE						

NOTES:

1. Water level measured in open borehole at a depth of 0.9 m (Elev. 289.4 m) below ground surface upon completion of drilling.

2. Water level measurements in standpipe piezometer:

Date	Depth*(m)	Elev. (m)
03/11/17	-0.5	290.8
14/11/17	-0.5	290.8
04/12/17	-0.5	290.8

* Water level measured above ground surface within stick up.

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

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

GOLDER

RECORD OF BOREHOLE No HF-10

SHEET 1 OF 1 **METRIC**

PROJECT 1670268

G.W.P. 2289-13-00

LOCATION N 4902471.4; E 291300.2 MTM NAD 83 ZONE 10 (LAT. 44.262120; LONG. -79.669077)

ORIGINATED BY DH

DIST Central HWY 400

BOREHOLE TYPE Power Auger - 203 mm O.D. Hollow Stem Augers

COMPILED BY JL

DATUM Geodetic

DATE October 27, 2017

CHECKED BY SMM

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
290.1	GROUND SURFACE							
0.0	TOPSOIL (230 mm)	1A 1B	SS	4		290		
0.2	Silty clay, oxidation staining (FILL) Firm to very stiff Brown Moist to wet	2A 2B	SS	13	∇	289		
		3	SS	18			45	
287.9 / 2.2	SILT and SAND, trace to some gravel, trace to some clay (TILL) Loose to dense Grey Moist to wet - Contains CLAYEY SAND layers at 2.3 m, 2.9 m, 3.8 m and 4.2 m	4	SS	6		288		
		5A 5B	SS	19		287		
		6	SS	21/0.25		286		6 52 34 8
		7	SS	41				
284.9 / 5.2	END OF BOREHOLE					285		

Column headers: SOIL PROFILE; STRAT PLOT; SAMPLES; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED, 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L, WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³.

NOTES:

1. Water level measured in open borehole at a depth of 0.8 m (Elev. 289.3 m) below ground surface upon completion of drilling.

2. Borehole caved to a depth of 0.8 m (Elev. 289.3 m) below ground surface upon completion of drilling.

+3, ×3: Numbers refer to Sensitivity

O 3% STRAIN AT FAILURE

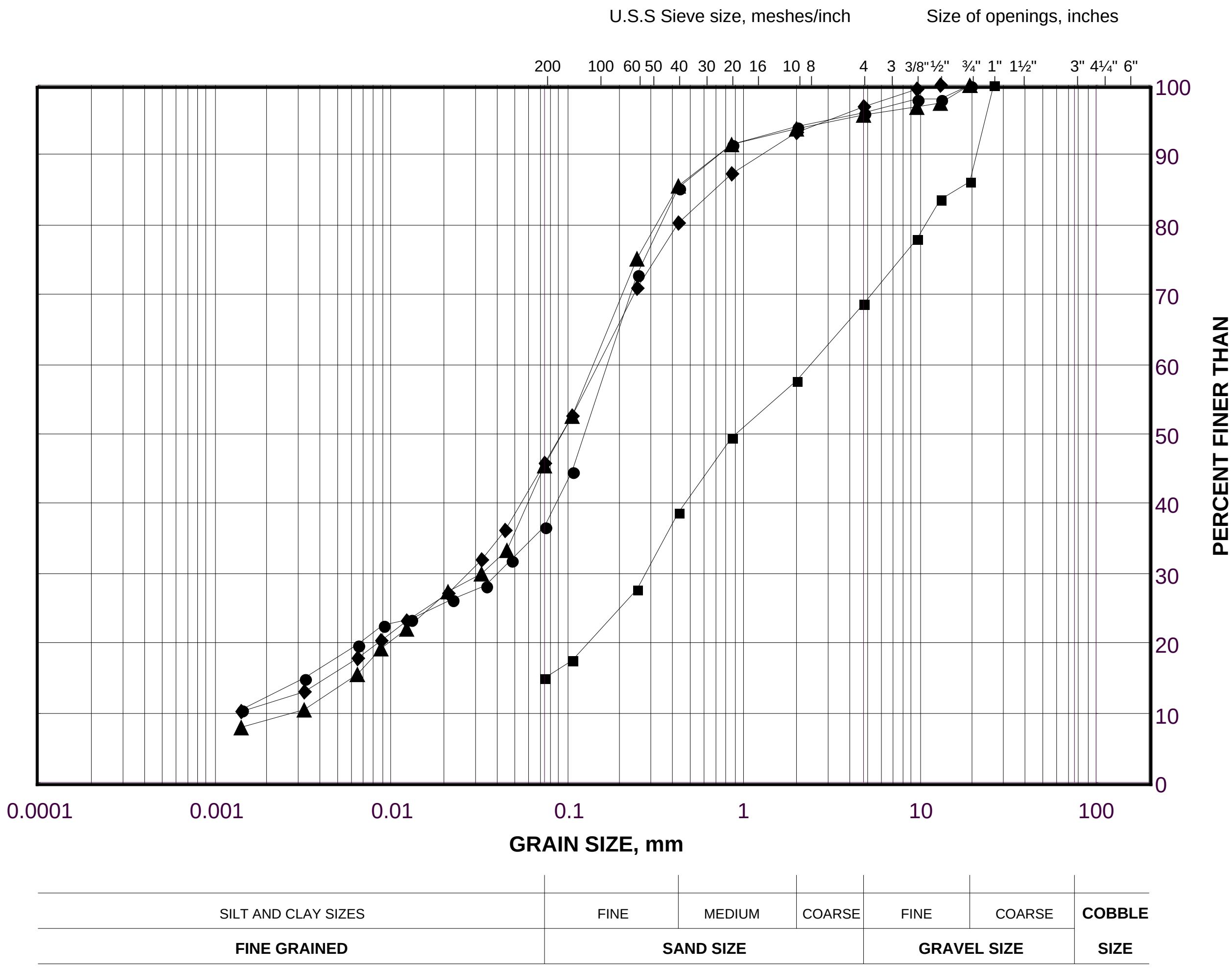
APPENDIX B

Geotechnical Laboratory Test Results and Chemical Test Results

GRAIN SIZE DISTRIBUTION

Silt and Sand to Sand and Gravel (Fill/Reworked)

FIGURE B1



LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	CE-01	2	290.2
■	6UP-08	2	294.2
◆	6UP-05	3	294.8
▲	6UP-05	6A	292.6

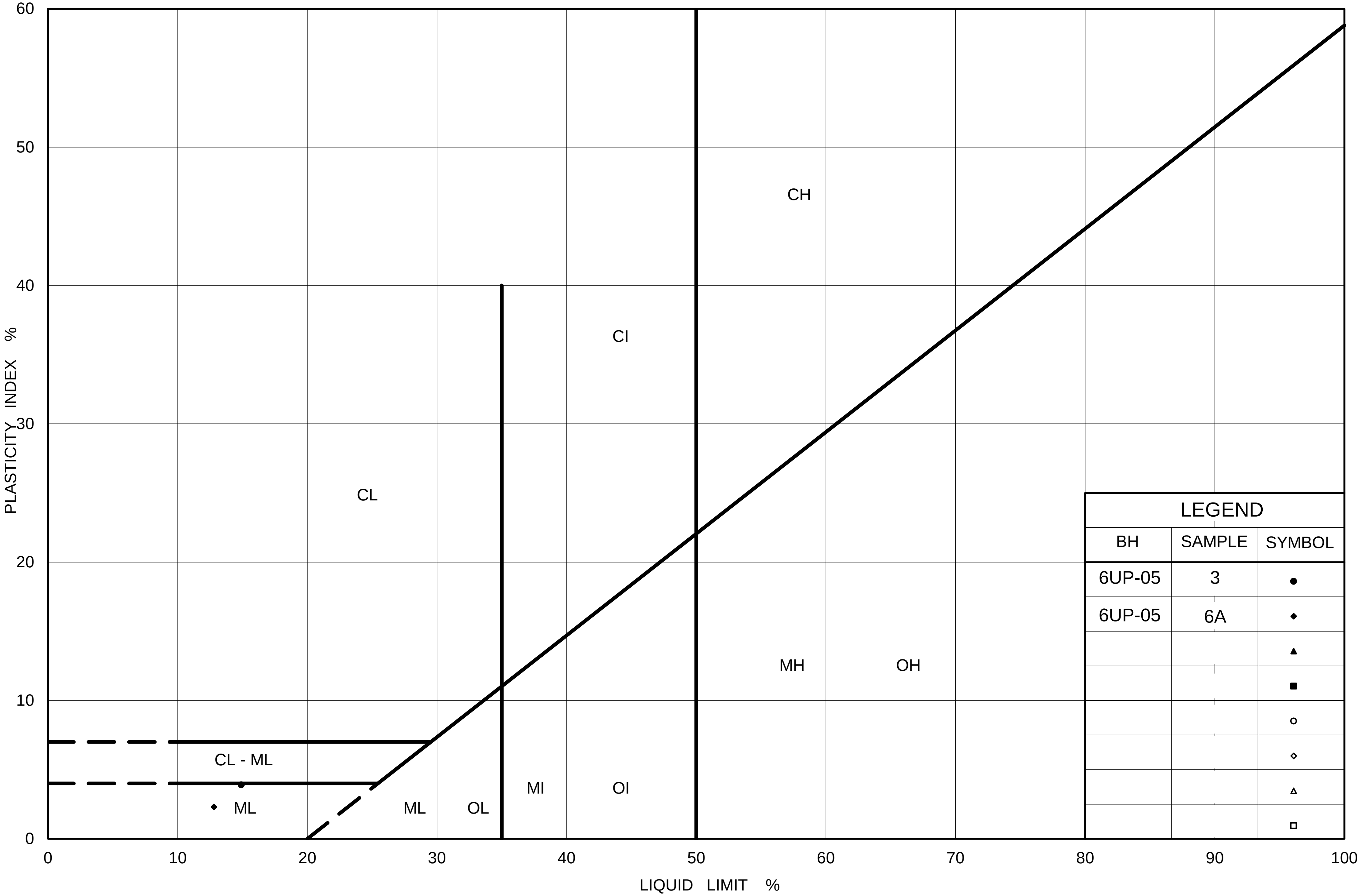
Project Number: 1670268

Checked By: SMM

Golder Associates

Date: 08-Mar-18

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART
Silt and Sand (Fill/Reworked)

Figure No. B2

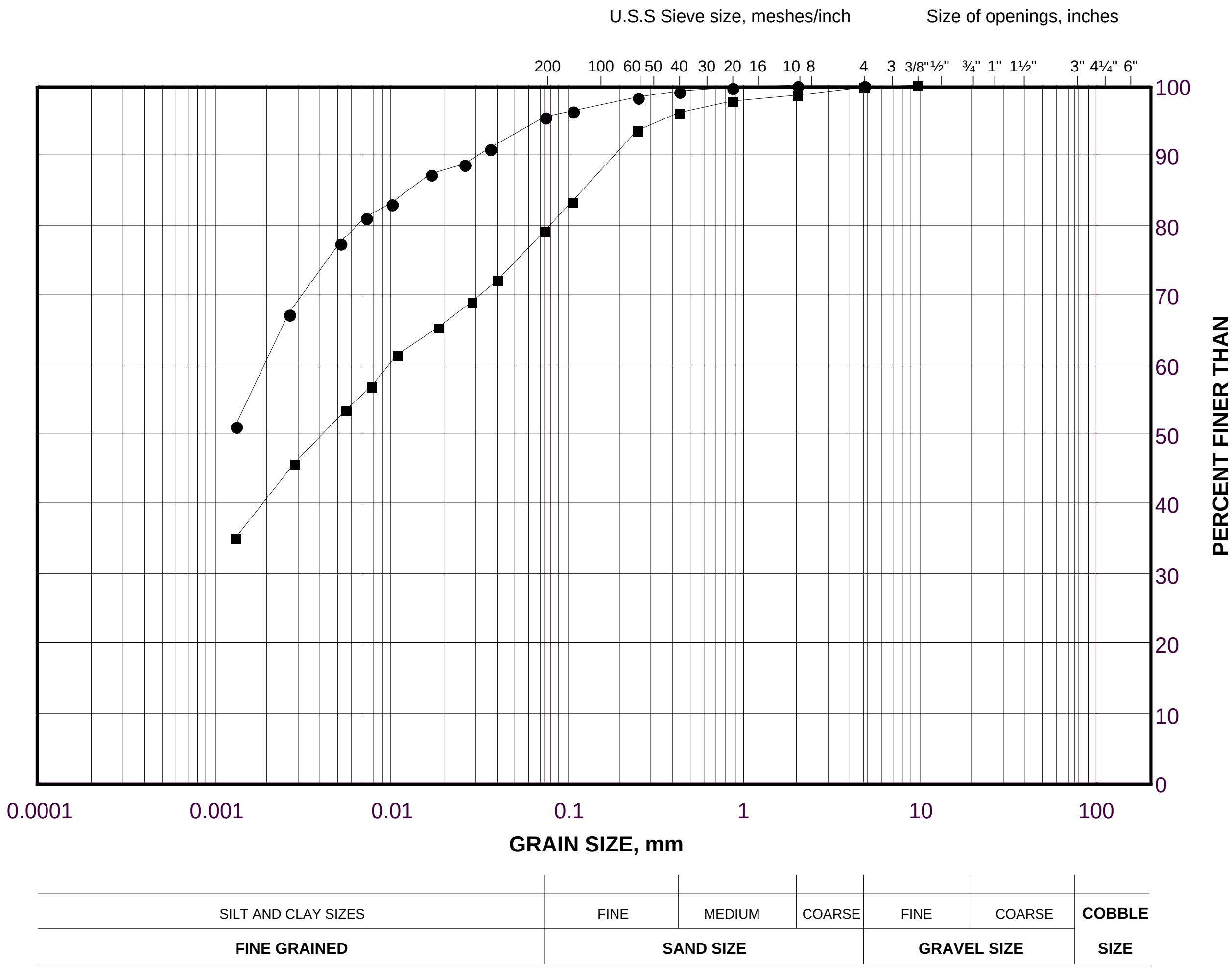
Project No. 1670268

Checked By: SMM

GRAIN SIZE DISTRIBUTION

Sandy Clayey Silt to Silty Clay (Fill)

FIGURE B3



LEGEND

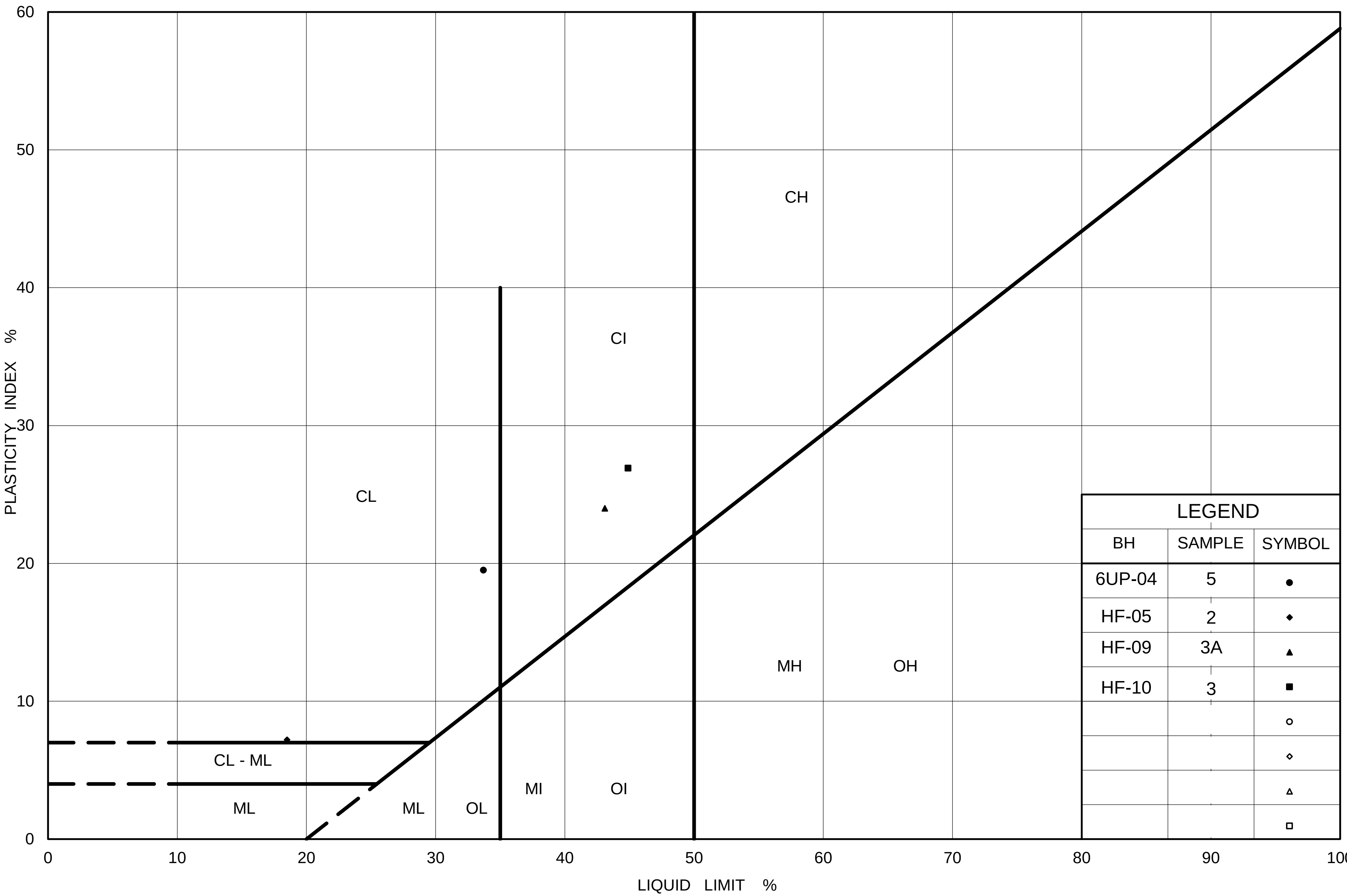
SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	HF-09	3A	288.6
■	6UP-04	5	293.2

Project Number: 1670268

Checked By: SMM

Golder Associates

Date: 08-Mar-18



Ministry of Transportation

Ontario

PLASTICITY CHART
Sandy Clayey Silt to Silty Clay (Fill)

Figure No. B4

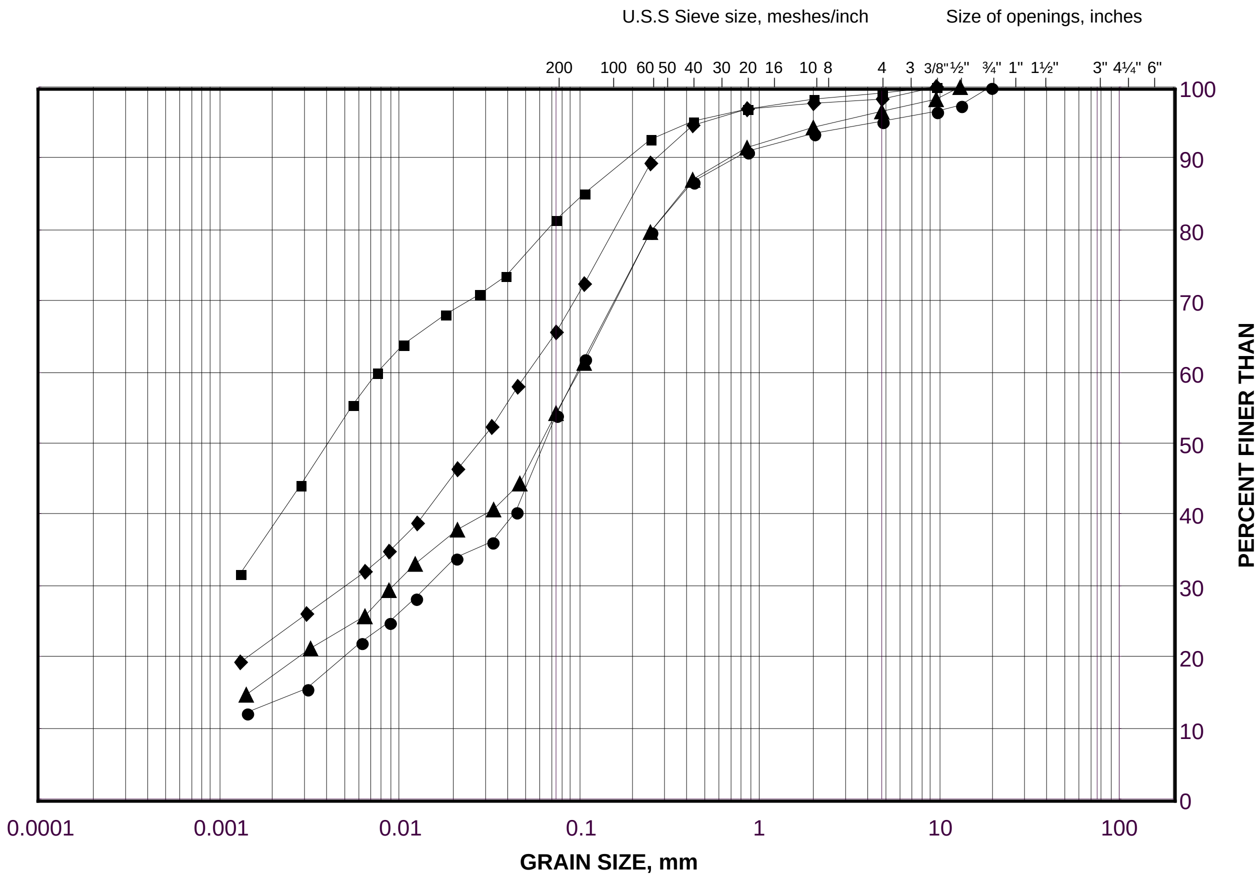
Project No. 1670268

Checked By: SMM

GRAIN SIZE DISTRIBUTION

Clayey Silt with Sand to Clayey Silt

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)
●	6UP-02	2	292.6
■	6UP-01	2	292.5
◆	CE-02	2	288.9
▲	HF-04	2	291.8

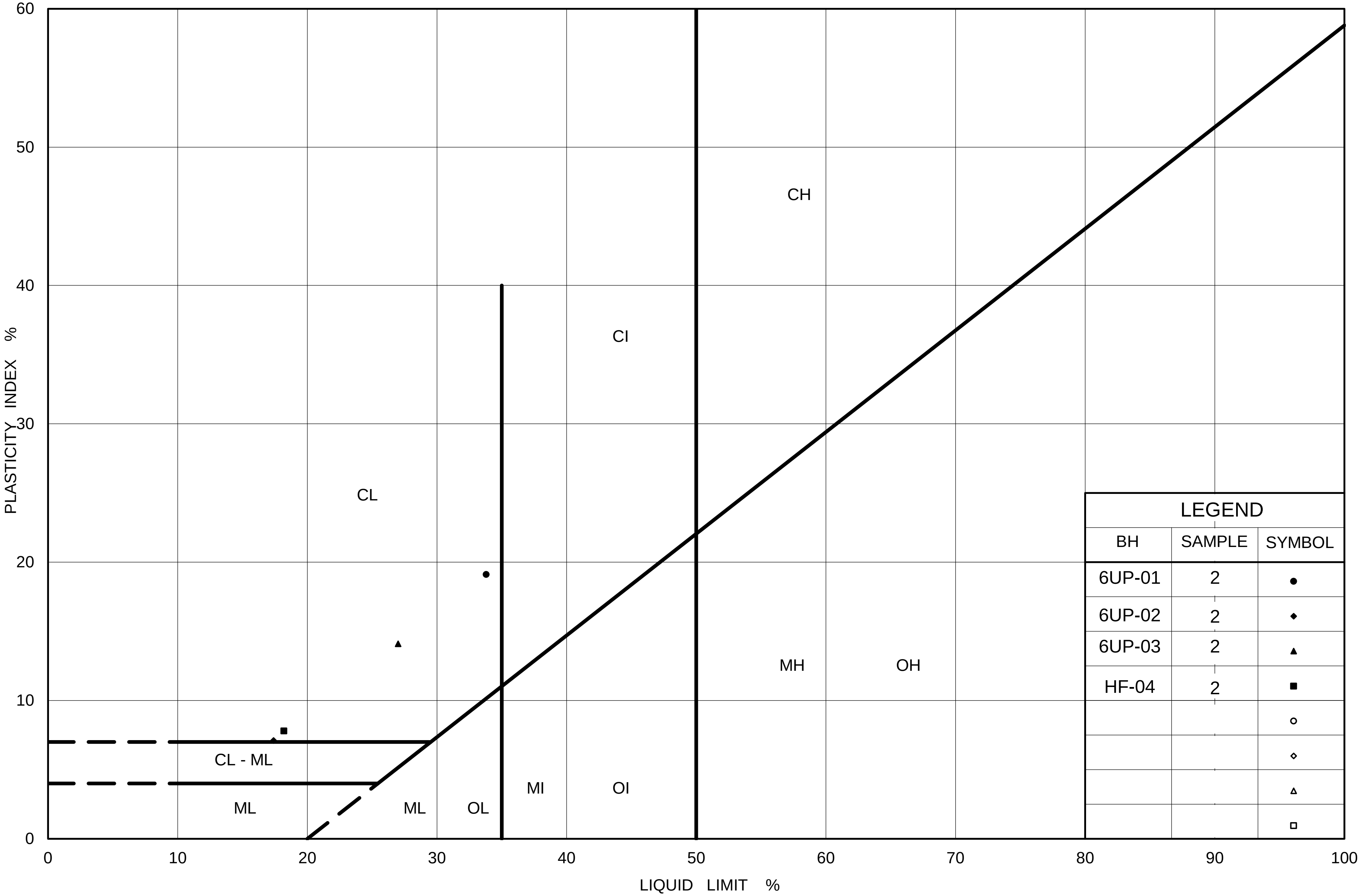
Project Number: 1670268

Checked By: SMM

Golder Associates

Date: 08-Mar-18

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART
Sandy Clayey Silt to Clayey Silt

Figure No. B6

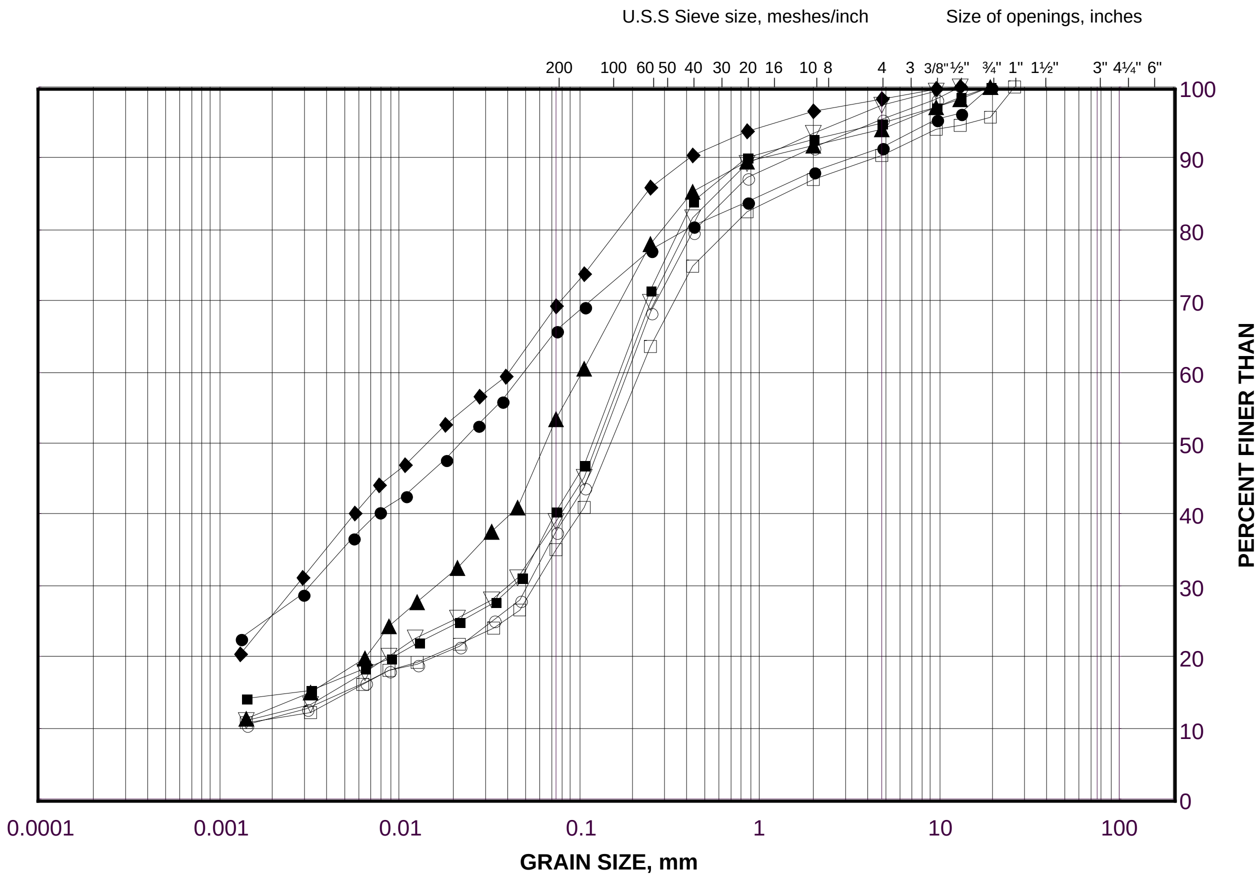
Project No. 1670268

Checked By: SMM

GRAIN SIZE DISTRIBUTION

Sandy Clayey Silt to Clayey Silt with Sand (Till)

FIGURE B7A



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	6UP-06	13	281.1
■	6UP-04	13	282.6
◆	6UP-06	2	294.0
▲	6UP-04	6	292.5
▽	6UP-05	8	290.2
○	6UP-06	8	288.7
□	6UP-05	9	288.7

Project Number: 1670268

Checked By: SMM

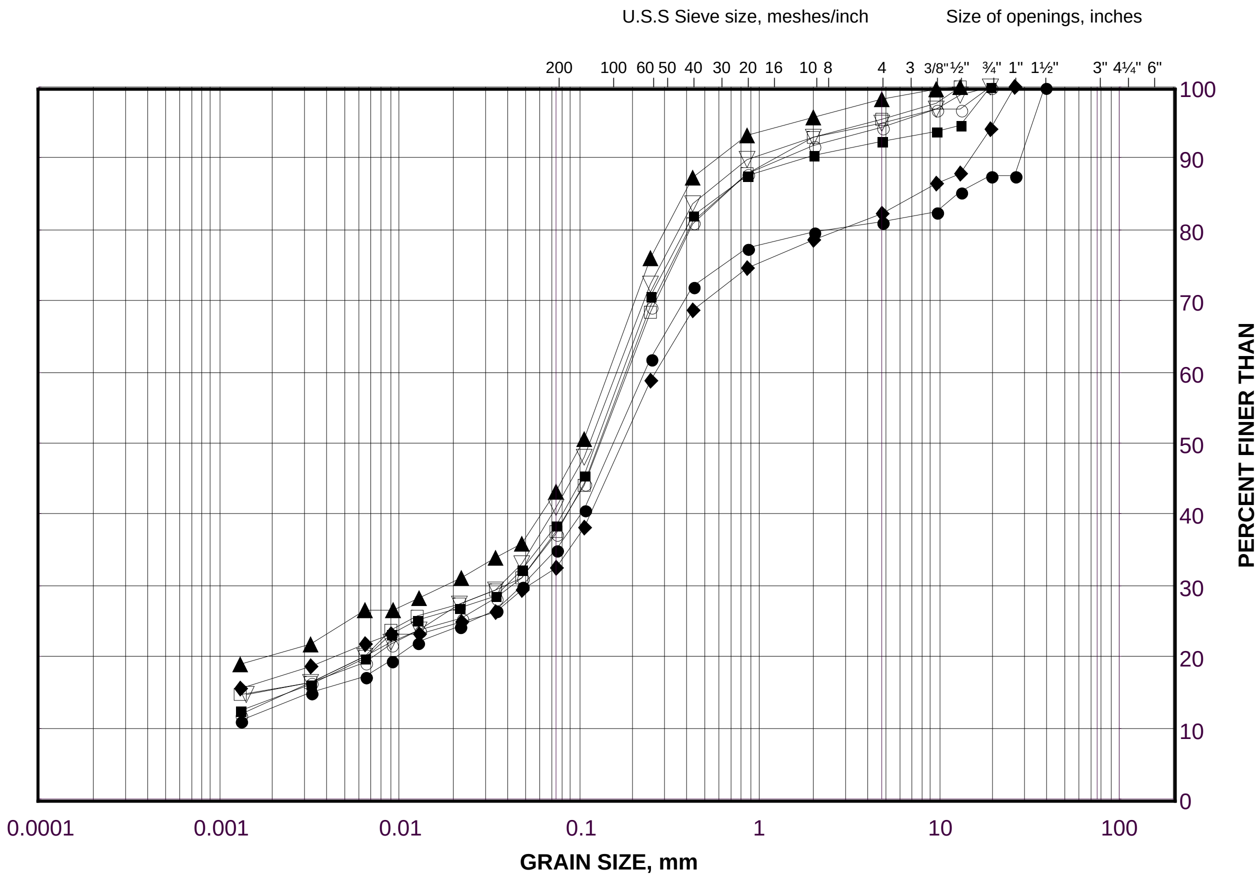
Golder Associates

Date: 23-Jan-18

GRAIN SIZE DISTRIBUTION

Clayey Silt with Sand (Till)

FIGURE B7B



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	HF-03	2	290.9
■	HF-02	3	290.0
◆	CE-02	4	287.4
▲	CE-01	4	288.7
▽	HF-03	5	288.6
❍	HF-01	5	289.3
☐	HF-02	7	287.0

Project Number: 1670268

Checked By: SMM

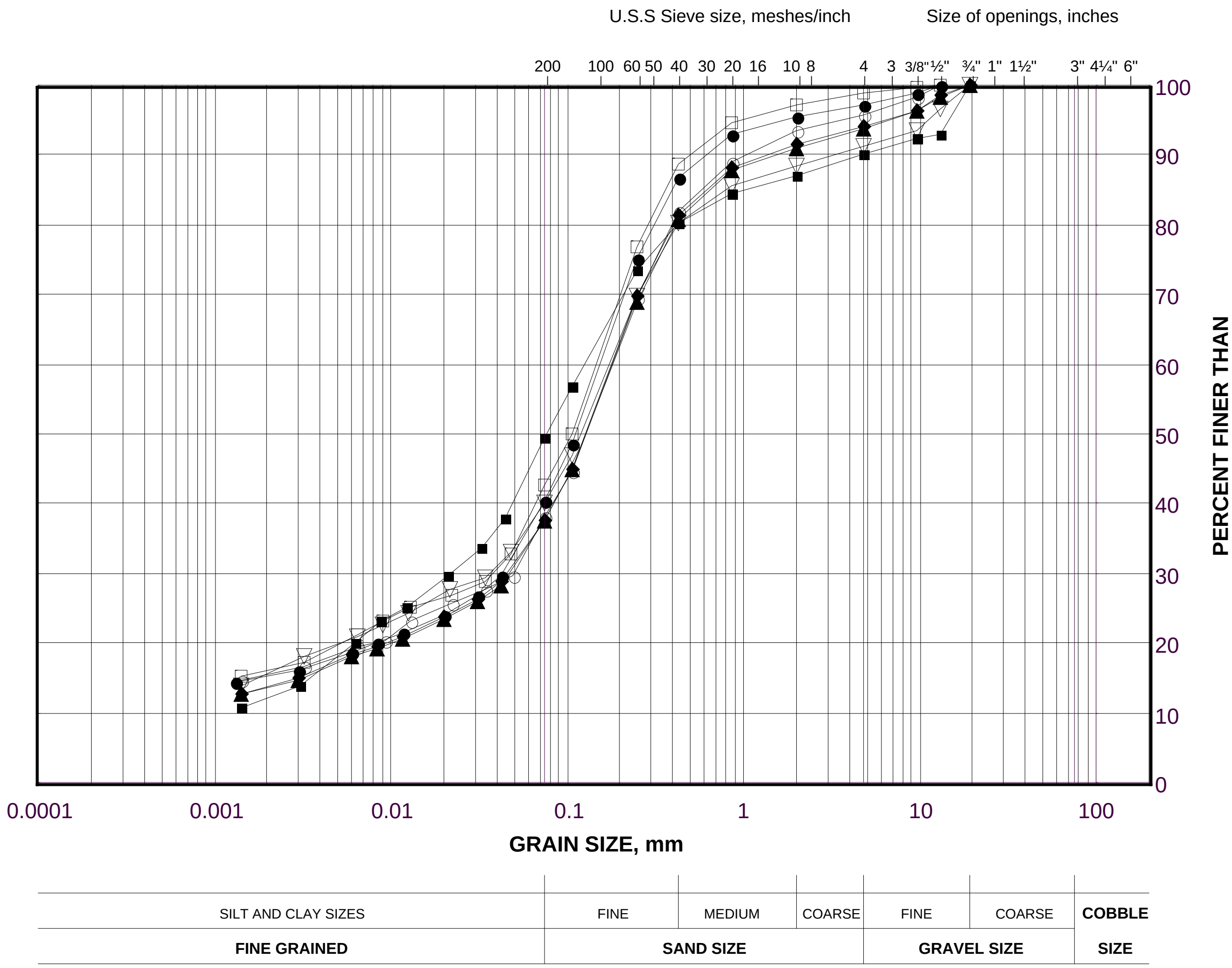
Golder Associates

Date: 08-Mar-18

GRAIN SIZE DISTRIBUTION

Clayey Silt with Sand (Till)

FIGURE B7C



LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	6UP-02	10	284.1
■	6UP-01	3	291.8
◆	6UP-02	4	291.0
▲	6UP-01	6	289.5
▽	HF-04	7	288.0
❍	HF-04	9	284.9
☐	HF-03	9	284.1

Project Number: 1670268

Checked By: SMM

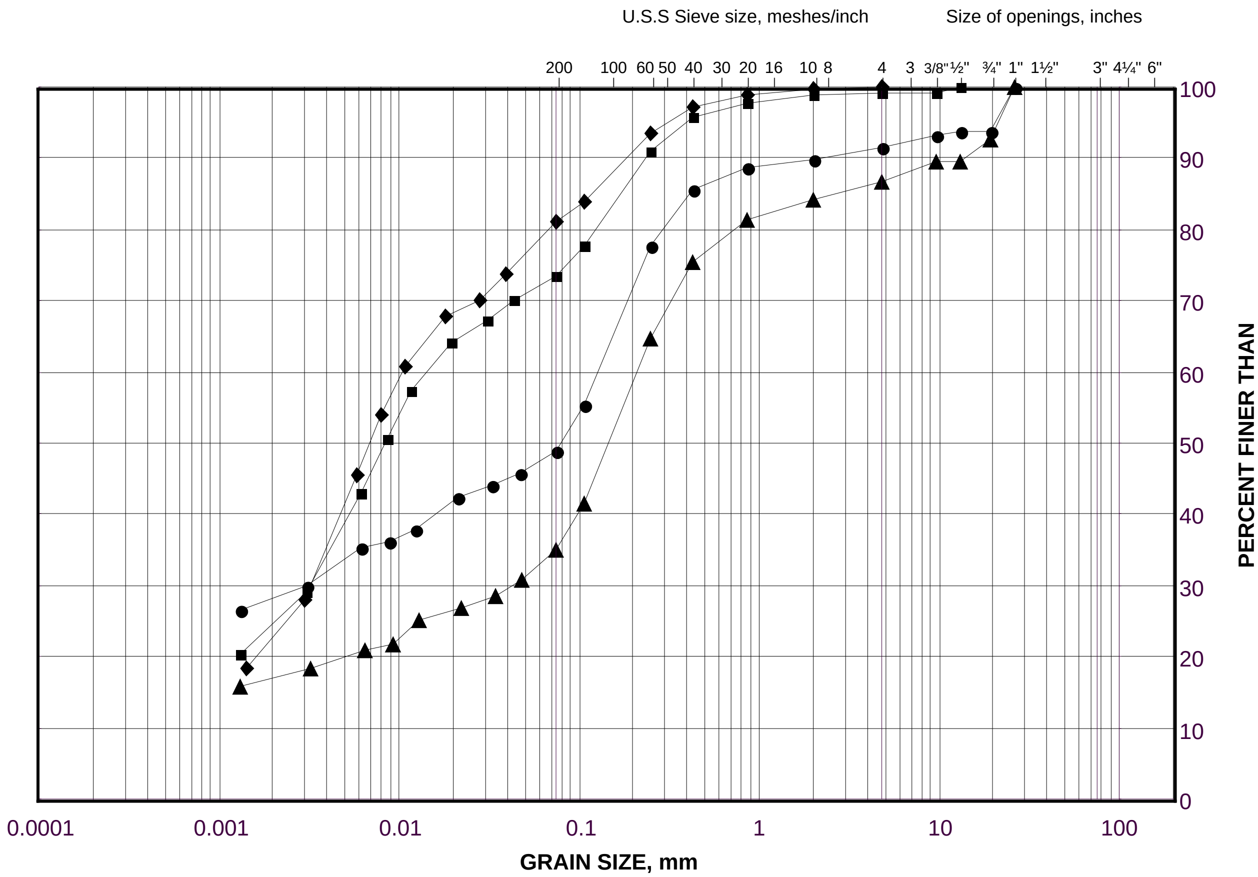
Golder Associates

Date: 08-Mar-18

GRAIN SIZE DISTRIBUTION

Clayey Silt with Sand (Till)

FIGURE B7D



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	6UP-03	17	273.6
■	6UP-03	19	267.7
◆	6UP-02	20	265.9
▲	6UP-03	7	288.8

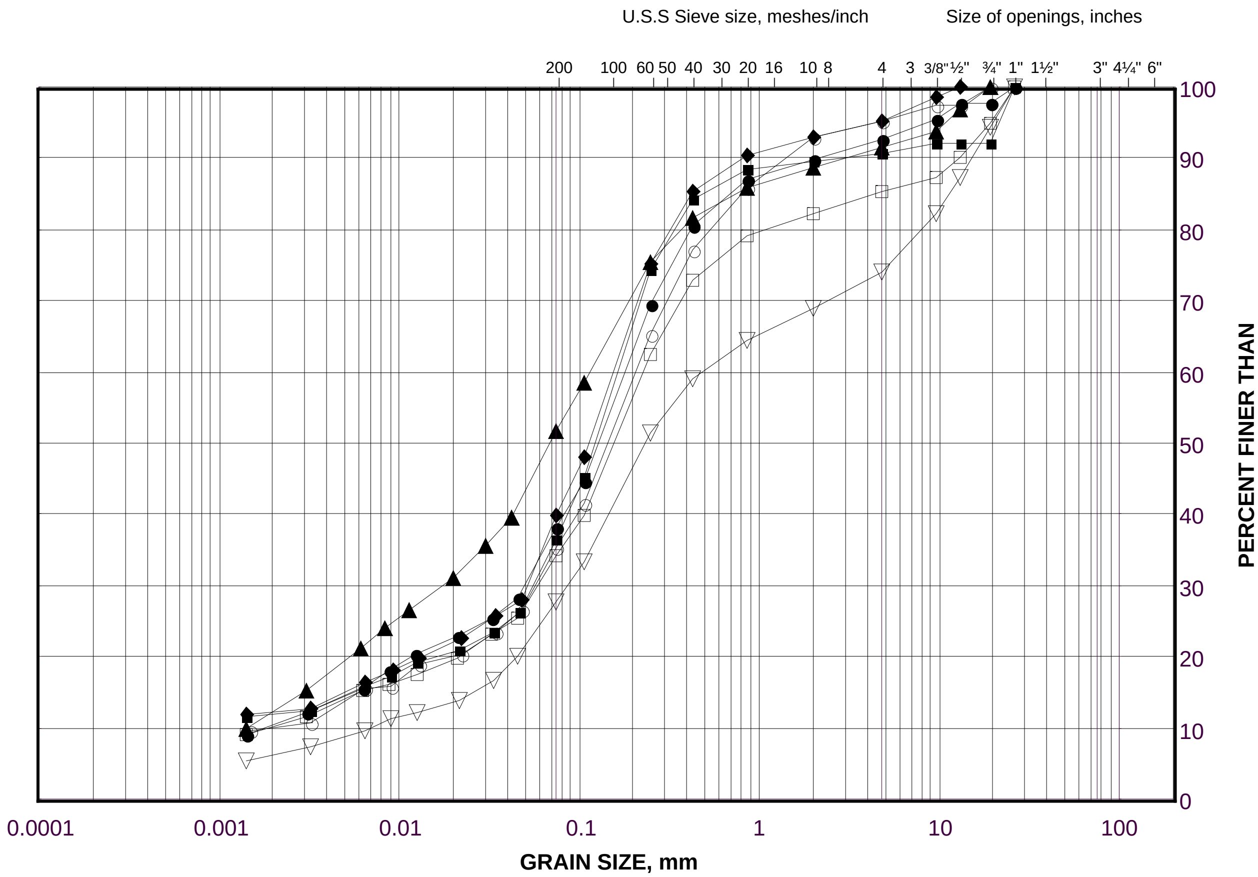
Project Number: 1670268

Checked By: SMM

Golder Associates

Date: 08-Mar-18

GRAIN SIZE DISTRIBUTION Sandy Clayey Silt to Silty Gravelly Sand (Till)	FIGURE B7E



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	6UP-08	10	285.7
■	6UP-07	15	278.0
◆	6UP-04	17	275.0
▲	6UP-08	3	293.3
▽	6UP-07	4	292.5
❍	6UP-08	5	291.7
☐	6UP-07	9	287.2

Project Number: 1670268

Checked By: SMM

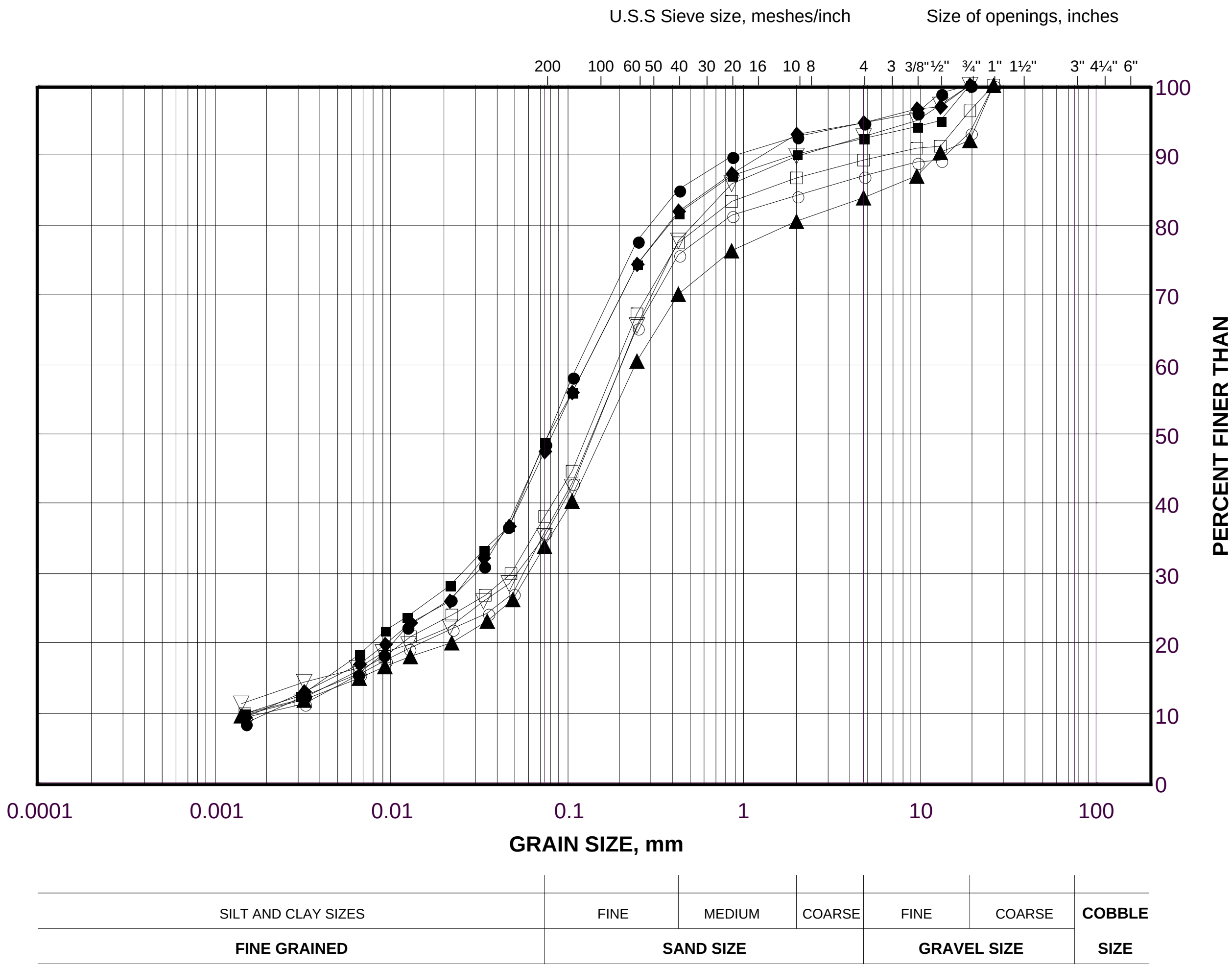
Golder Associates

Date: 23-Jan-18

GRAIN SIZE DISTRIBUTION

Clayey Silt with Sand to Silt and Sand (Till)

FIGURE B7F



LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	HF-08	2	290.4
■	HF-06	2	292.9
◆	HF-07	3	291.2
▲	HF-08	5	288.1
▽	HF-07	6	288.8
❍	HF-06	7	289.1
□	HF-05	7	289.8

Project Number: 1670268

Checked By: SMM

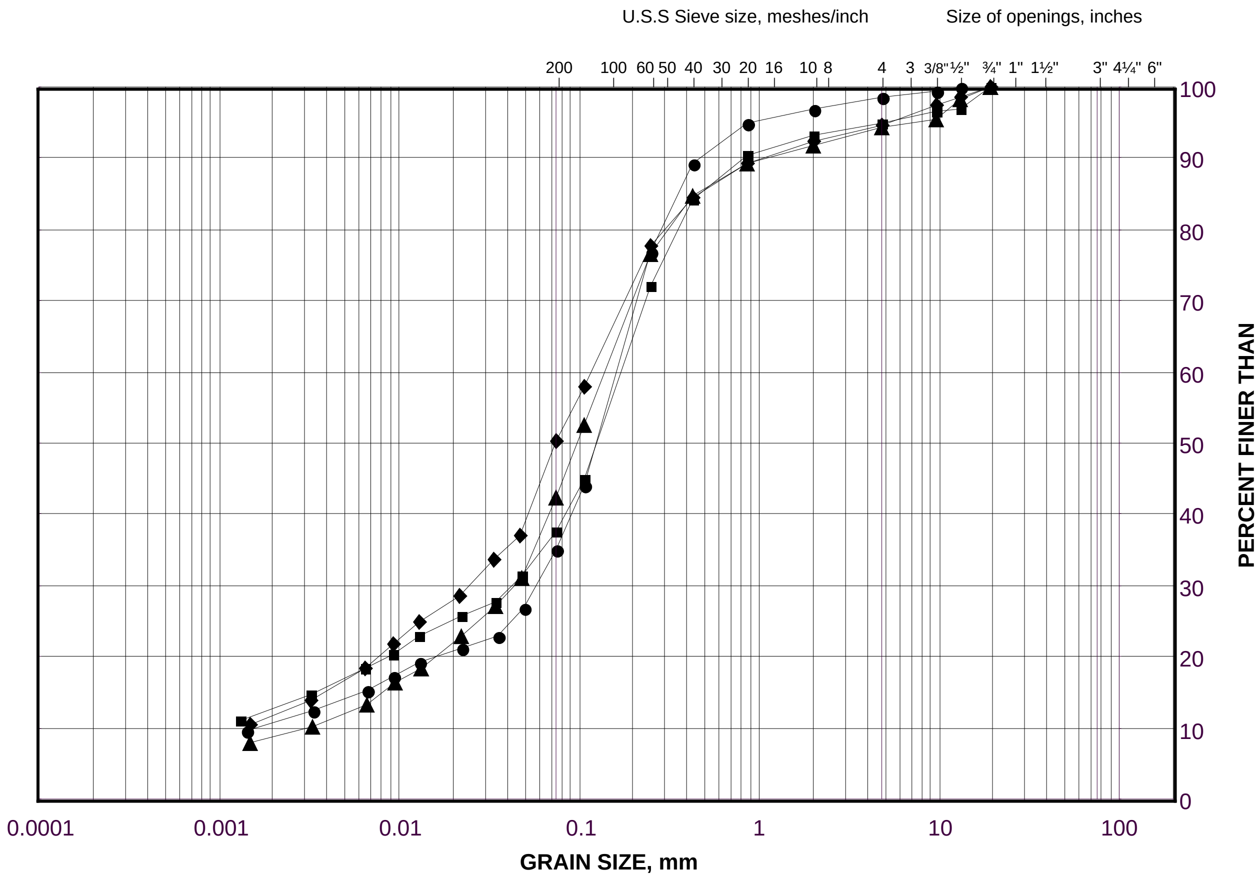
Golder Associates

Date: 23-Jan-18

GRAIN SIZE DISTRIBUTION

Silt and Sand to Silty Sand (Till)

FIGURE B7G



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

LEGEND

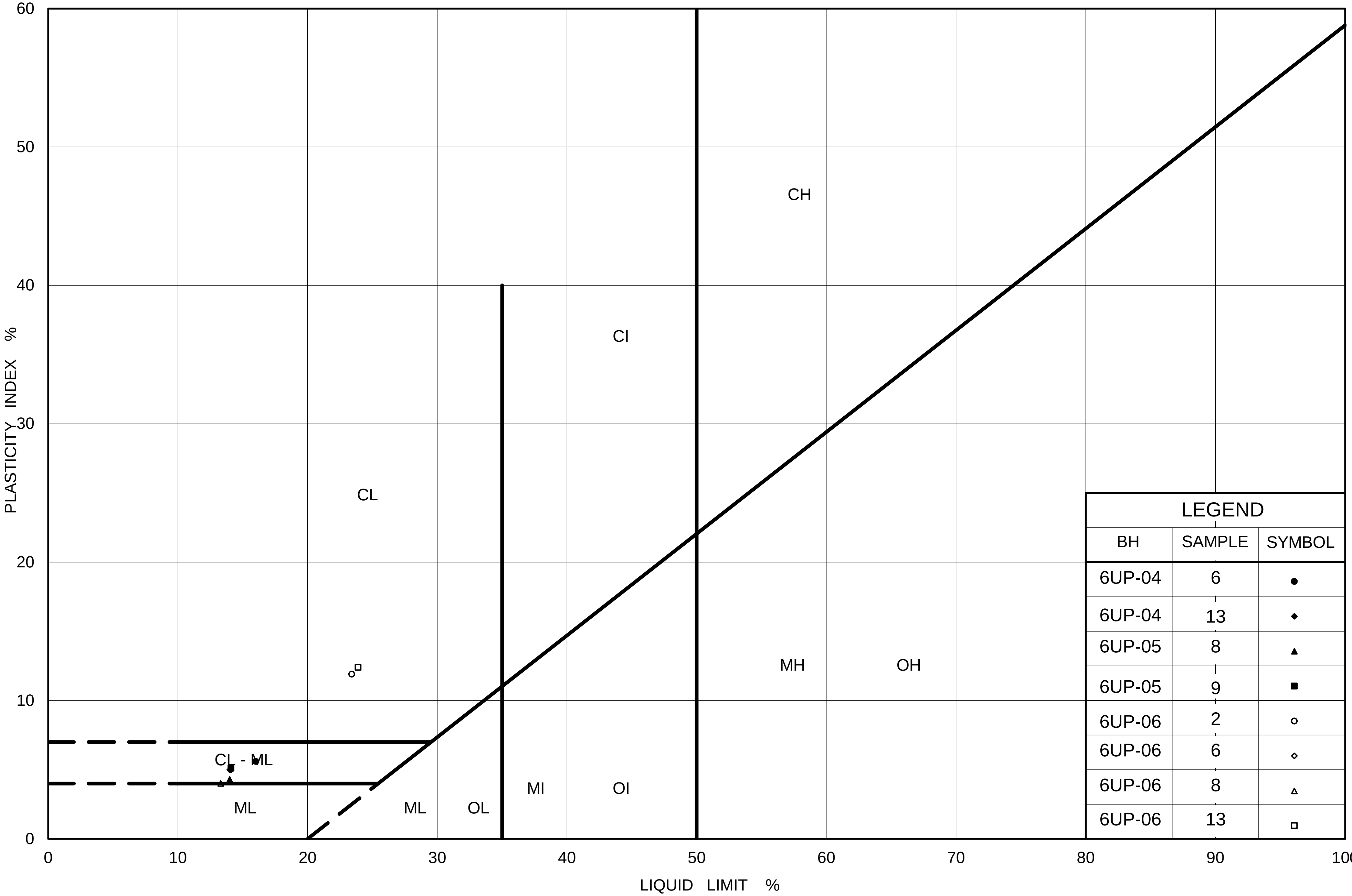
SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	HF-02	10	282.4
■	HF-01	2	291.6
◆	HF-09	4	287.7
▲	HF-10	6	286.0

Project Number: 1670268

Checked By: SMM

Golder Associates

Date: 08-Mar-18



LEGEND

BH	SAMPLE	SYMBOL
6UP-04	6	●
6UP-04	13	◆
6UP-05	8	▲
6UP-05	9	■
6UP-06	2	○
6UP-06	6	◇
6UP-06	8	△
6UP-06	13	□

Ministry of Transportation

Ontario

PLASTICITY CHART

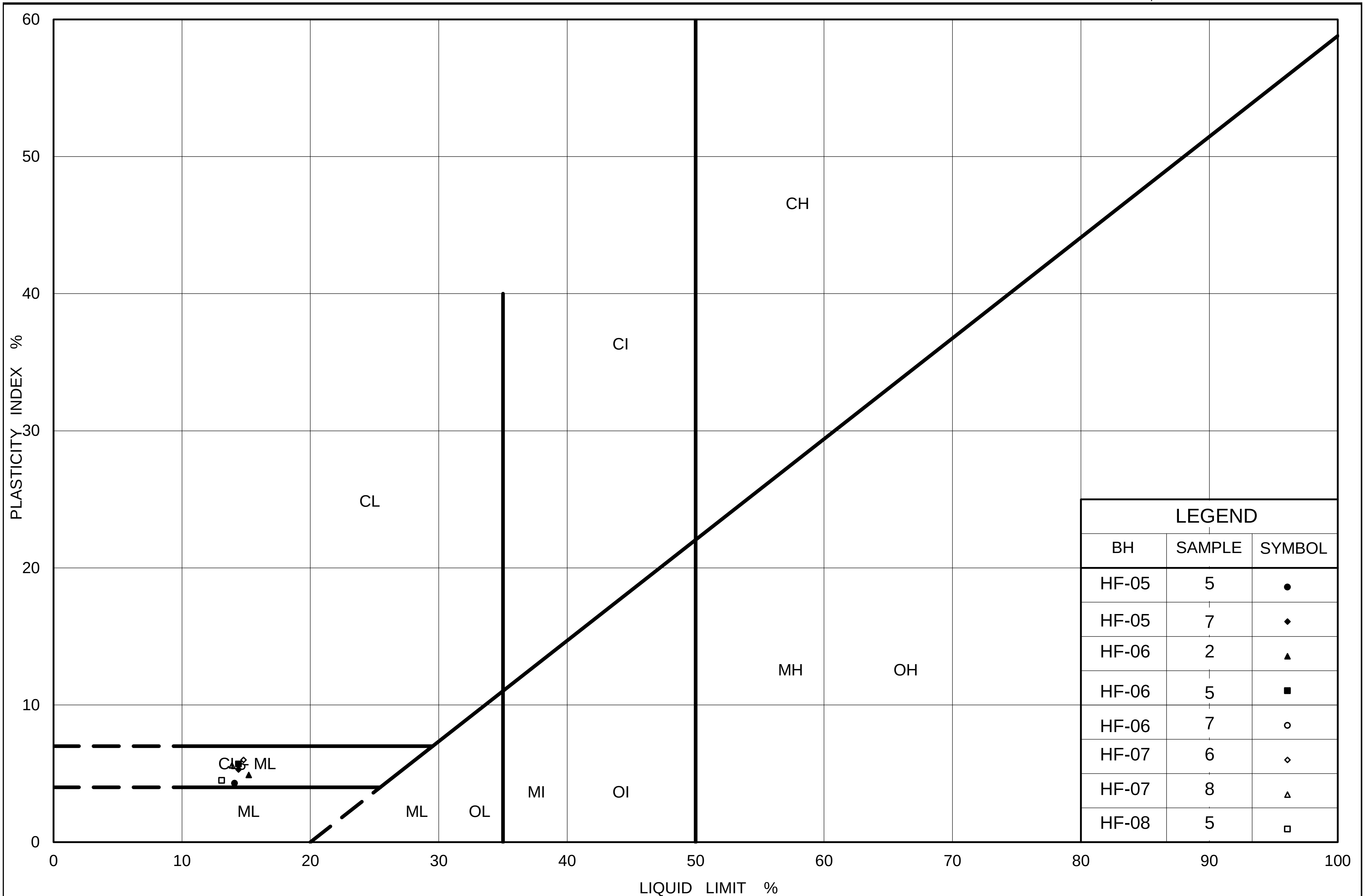
Sandy Clayey Silt to Clayey Silt with Sand (Till)

Figure No. B8A

Project No. 1670268

Checked By: SMM

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART

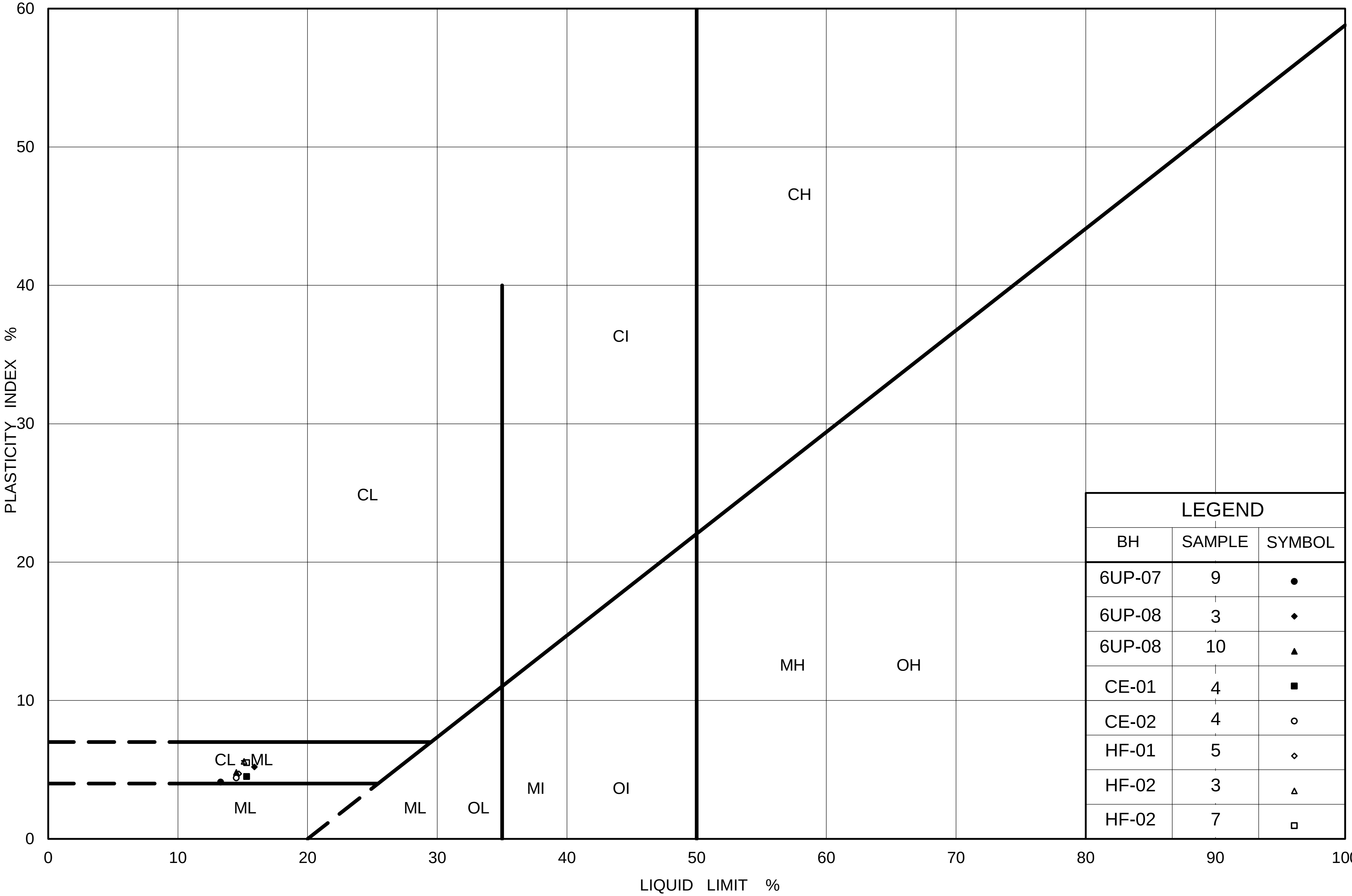
Sandy Clayey Silt to Clayey Silt with Sand (Till)

Figure No. B8B

Project No. 1670268

Checked By: SMM

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART

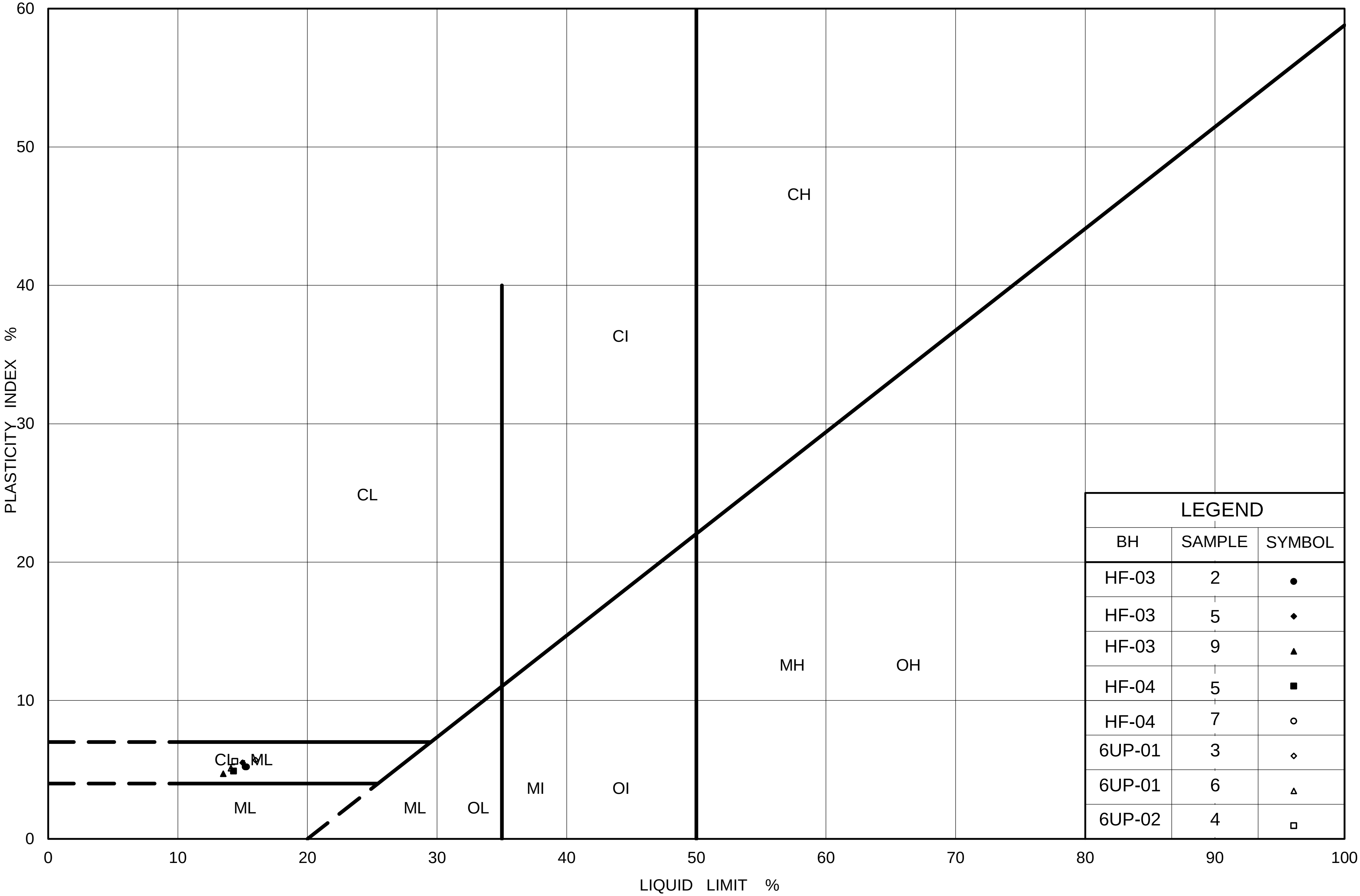
Sandy Clayey Silt to Clayey Silt with Sand (Till)

Figure No. B8C

Project No. 1670268

Checked By: SMM

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART

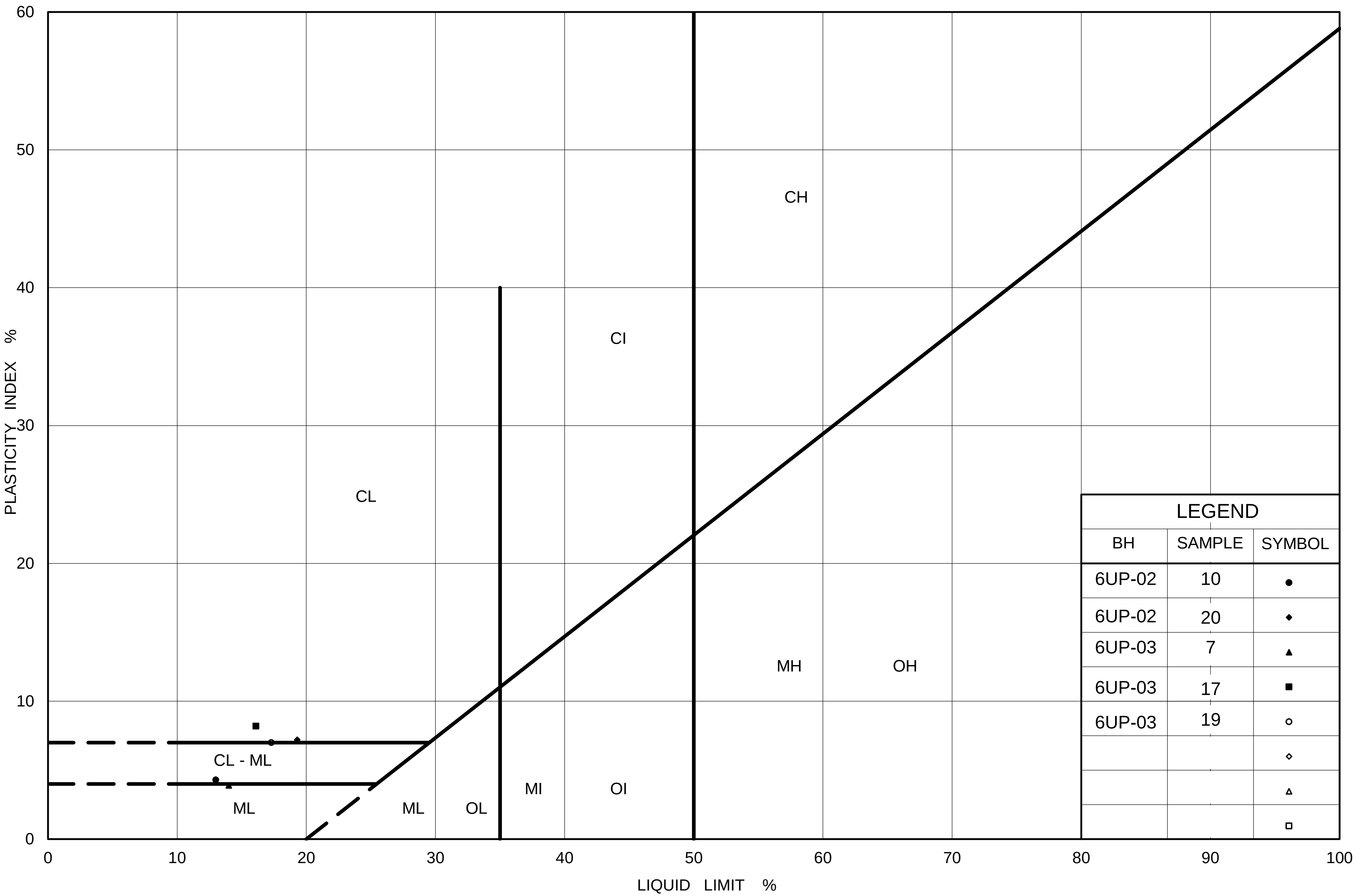
Clayey Silt with Sand (Till)

Figure No. B8D

Project No. 1670268

Checked By: SMM

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART

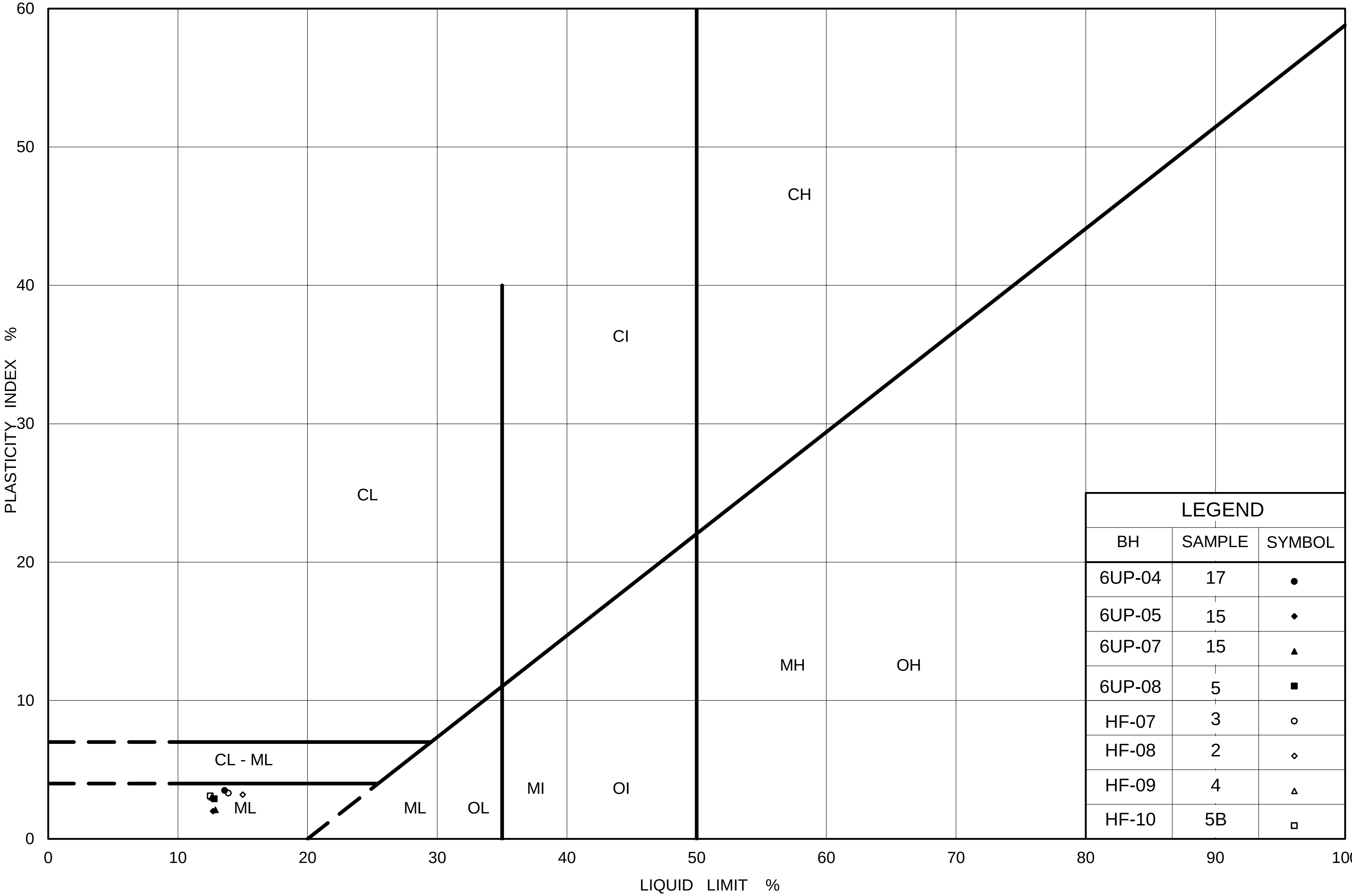
Clayey Silt with Sand (Till)

Figure No. B8E

Project No. 1670268

Checked By: SMM

Oct 75, FF-S-21



LEGEND

BH	SAMPLE	SYMBOL
6UP-04	17	●
6UP-05	15	◆
6UP-07	15	▲
6UP-08	5	■
HF-07	3	○
HF-08	2	◇
HF-09	4	△
HF-10	5B	□

Ministry of Transportation

Ontario

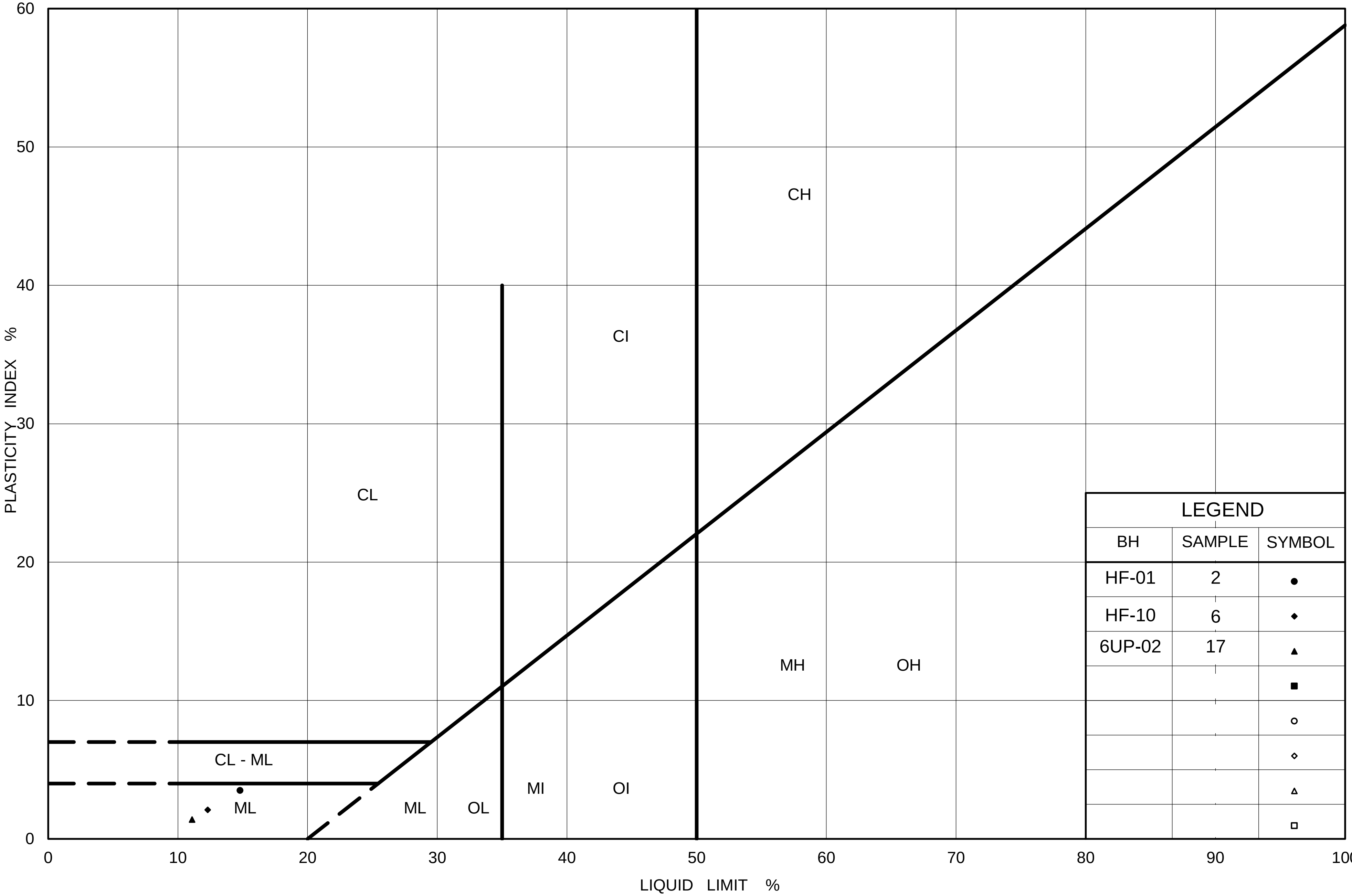
PLASTICITY CHART
Silt and Sand to Silty Sand (Till)

Figure No. B9A

Project No. 1670268

Checked By: SMM

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART

Silt and Sand to Silty Sand (Till)

Figure No. B9B

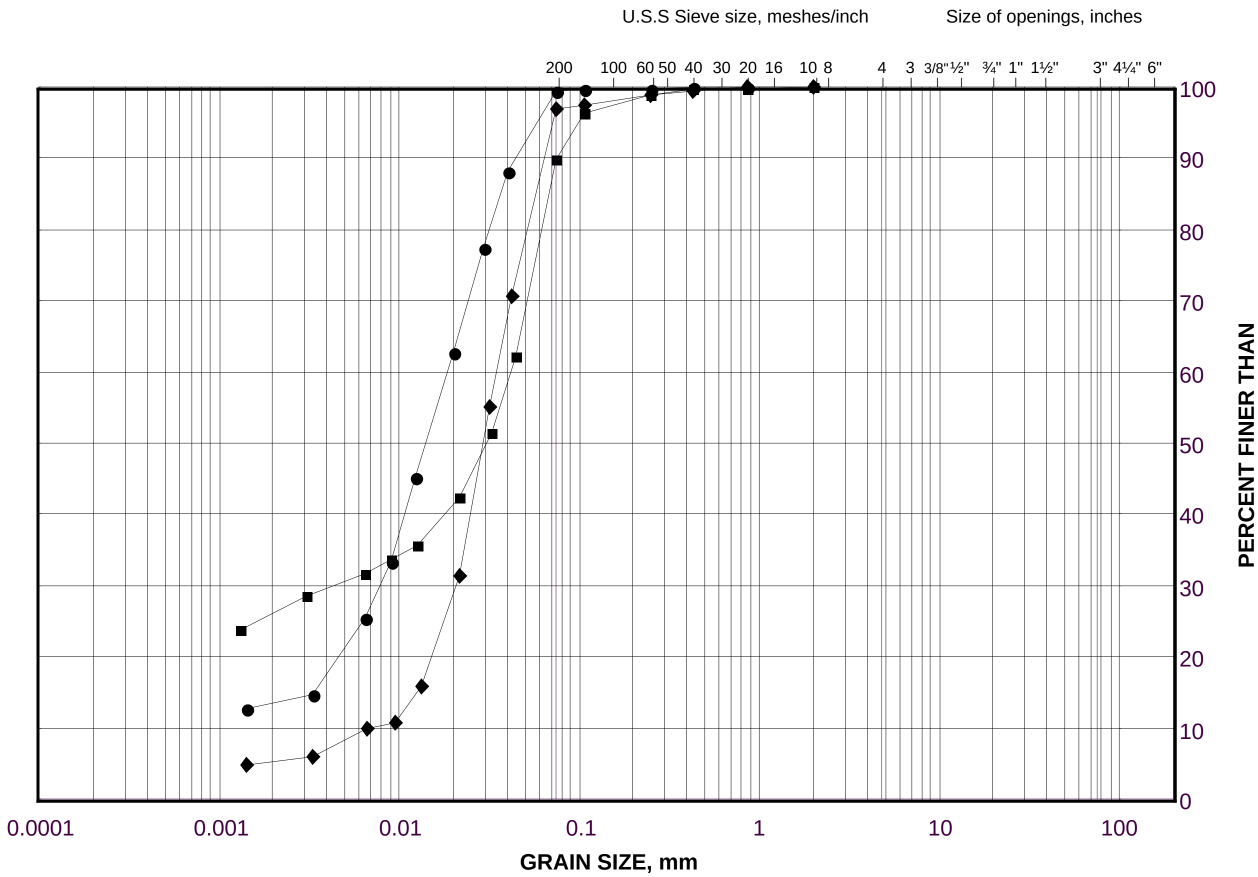
Project No. 1670268

Checked By: SMM

GRAIN SIZE DISTRIBUTION

Clayey Silt to Silt (Upper Interlayer)

FIGURE B10



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	CE-01	11	280.3
■	6UP-03	13	279.7
◆	CE-02	9	282.2

Project Number: 1670268

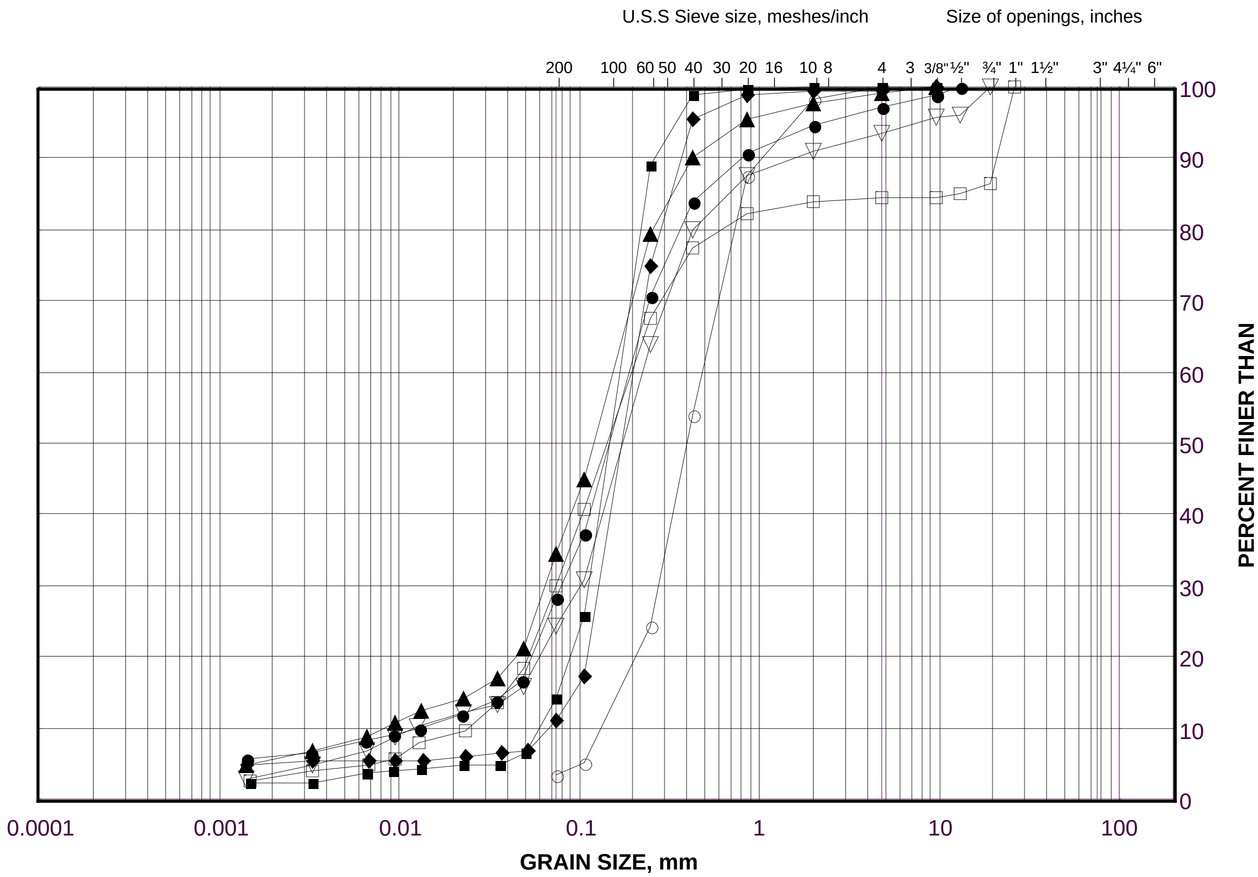
Checked By: SMM

Golder Associates

Date: 08-Mar-18

GRAIN SIZE DISTRIBUTION
Silty Sand to Sand

FIGURE B11A



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	6UP-07	12A	282.8
■	6UP-02	13	279.6
◆	6UP-03	14	278.2
▲	6UP-06	16	276.5
▽	6UP-07	18	273.5
❍	6UP-06	18A	273.7
□	HF-09	8	283.9

Project Number: 1670268

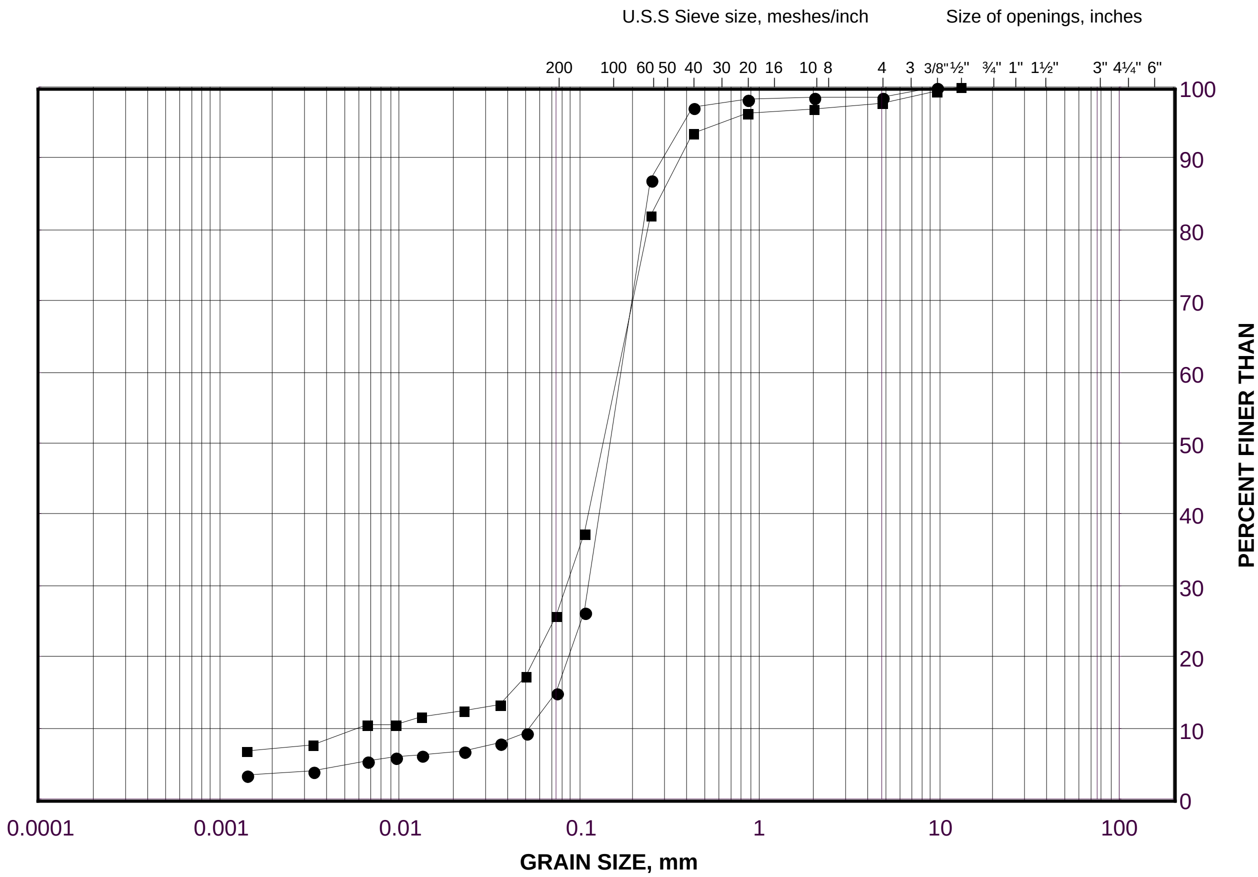
Checked By: SMM

Golder Associates

Date: 08-Mar-18

GRAIN SIZE DISTRIBUTION
Silty Sand to Sand

FIGURE B11B



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	CE-03	2	285.9
■	CE-02	7	285.1

Project Number: 1670268

Checked By: SMM

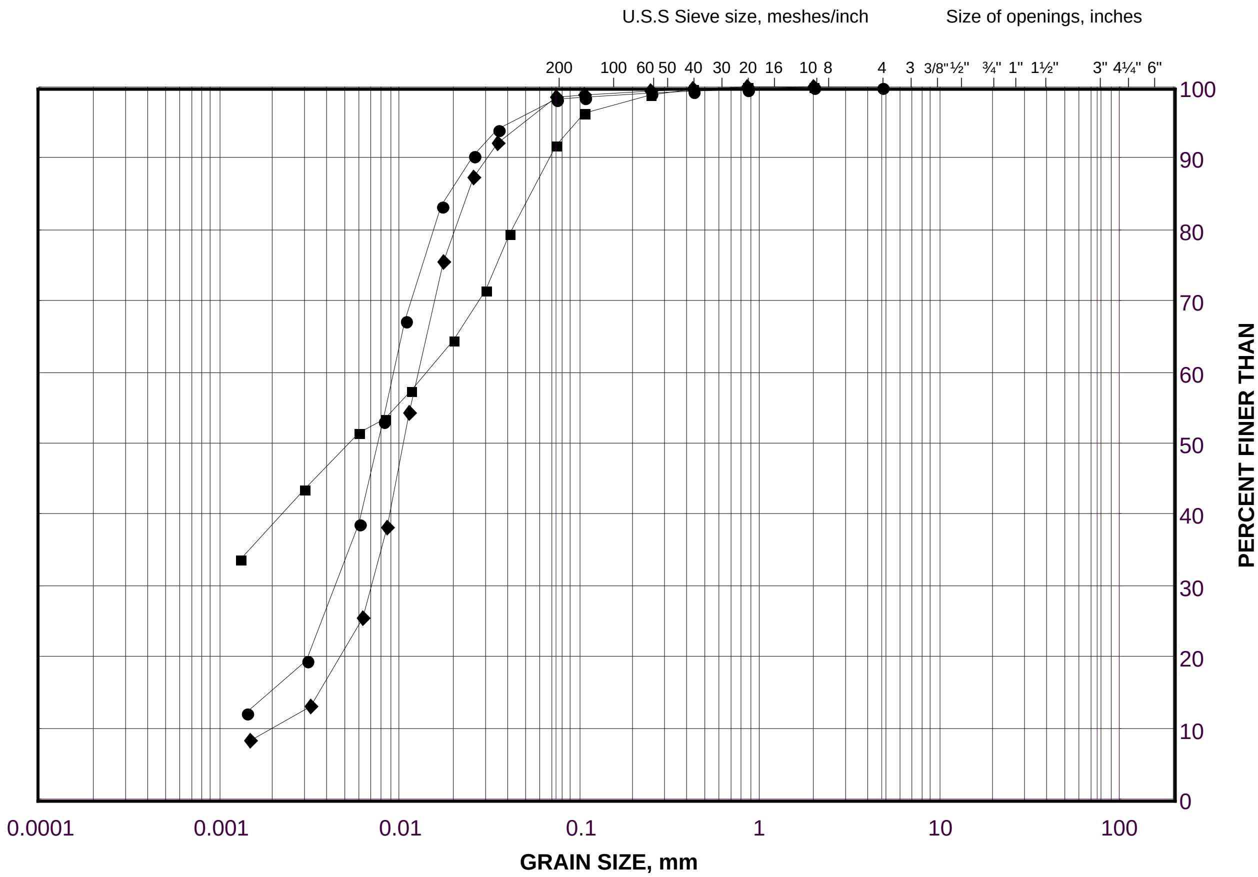
Golder Associates

Date: 08-Mar-18

GRAIN SIZE DISTRIBUTION

Clayey Silt to Silt

FIGURE B12



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	6UP-05	20	267.6
■	6UP-03	21	264.4
◆	6UP-04	21	269.0

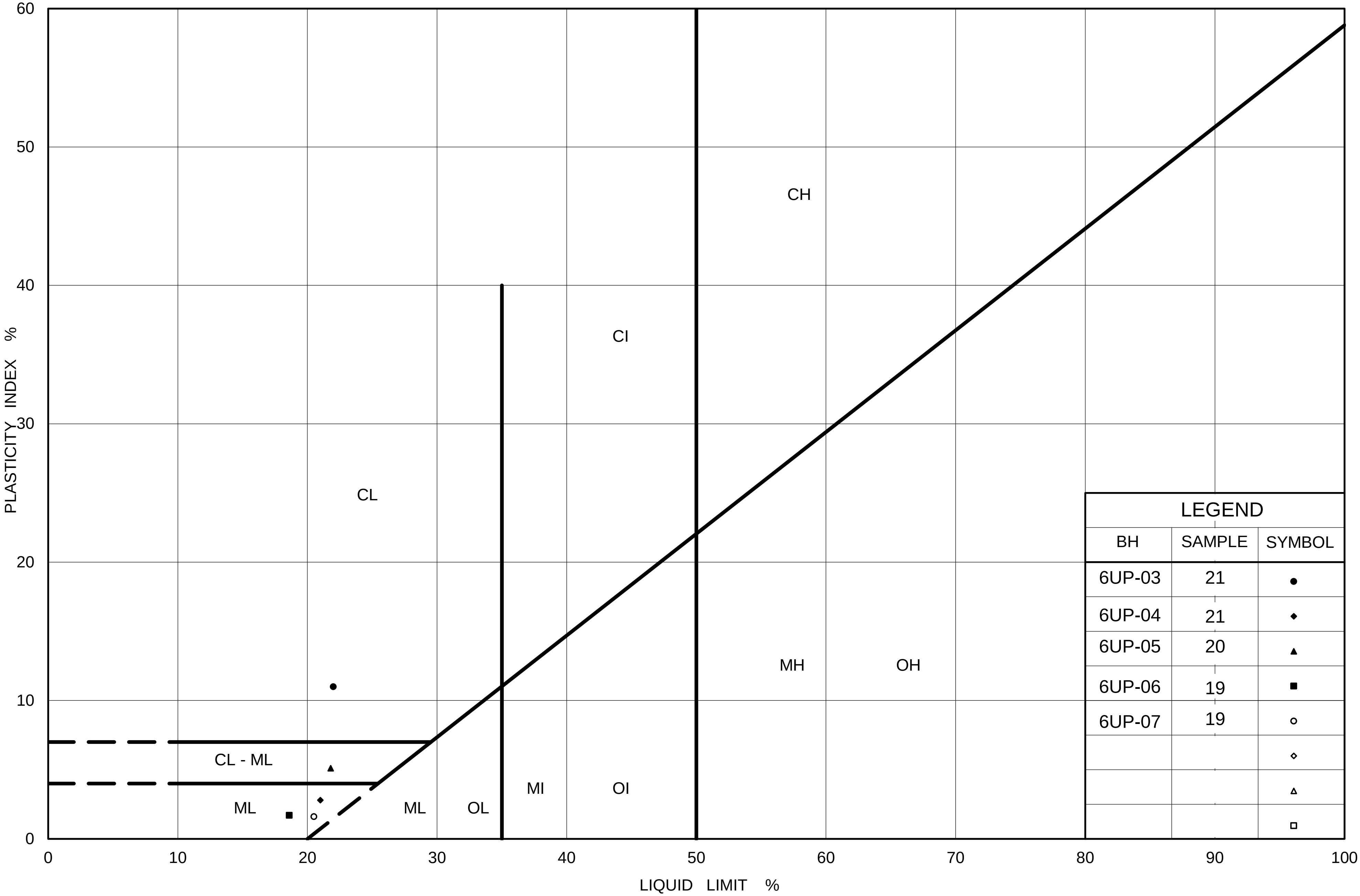
Project Number: 1670268

Checked By: SMM

Golder Associates

Date: 08-Mar-18

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART
Clayey Silt to Silt

Figure No. B13

Project No. 1670268

Checked By: SMM

Maxxam
A Bureau Veritas Group Company

Your Project #: 1670268
Site Location: HWY 400/6TH LINE
Your C.O.C. #: 60263

Attention:David Marmor

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

Report Date: 2017/11/14
Report #: R4856769
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B7P2674
Received: 2017/11/09, 15:45

Sample Matrix: Soil
Samples Received: 2

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Reference
Chloride (20:1 extract)	2	N/A	2017/11/14	CAM SOP-00463	EPA 325.2 m
Conductivity	2	N/A	2017/11/14	CAM SOP-00414	OMOE E3530 v1 m
pH CaCl2 EXTRACT	2	2017/11/14	2017/11/14	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	2	2017/11/09	2017/11/14	CAM SOP-00414	SM 22 2510 m
Sulphate (20:1 Extract)	2	N/A	2017/11/14	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.

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.

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.

Maxxam Analytics International Corporation o/a Maxxam Analytics 6740 Campobello Road, Mississauga, Ontario, L5N 2L8 Tel: (905) 817-5700 Toll-Free: 800-563-6266 Fax: (905) 817-5777 www.maxxam.ca

Your Project #: 1670268
Site Location: HWY 400/6TH LINE
Your C.O.C. #: 60263

Attention:David Marmor

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

Report Date: 2017/11/14
Report #: R4856769
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B7P2674
Received: 2017/11/09, 15:45

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.

Maxxam Job #: B7P2674
Report Date: 2017/11/14

Golder Associates Ltd
Client Project #: 1670268
Site Location: HWY 400/6TH LINE
Sampler Initials: JL

RESULTS OF ANALYSES OF SOIL

Maxxam ID		FNH590	FNH590	FNH591	FNH591		
Sampling Date		2017/10/12	2017/10/12	2017/10/19	2017/10/19		
COC Number		60263	60263	60263	60263		
	UNITS	**6UP-05-SA7**	**6UP-05-SA7 Lab-Dup**	**6UP-06-SA4A**	**6UP-06-SA4A Lab-Dup**	**RDL**	**QC Batch**
Calculated Parameters							
Resistivity	ohm-cm	910		4700			5261646
Inorganics							
Soluble (20:1) Chloride (Cl)	ug/g	610		57	57	20	5262943
Conductivity	umho/cm	1100	1130	215		2	5264045
Available (CaCl2) pH	pH	7.99		7.88			5262832
Soluble (20:1) Sulphate (SO4)	ug/g	<20		<20		20	5262956

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

Maxxam Analytics International Corporation o/a Maxxam Analytics 6740 Campobello Road, Mississauga, Ontario, L5N 2L8 Tel: (905) 817-5700 Toll-Free: 800-563-6266 Fax: (905) 817-5777 www.maxxam.ca

Maxxam
A Bureau Veritas Group Company

Maxxam Job #: B7P2674
Report Date: 2017/11/14

Golder Associates Ltd
Client Project #: 1670268
Site Location: HWY 400/6TH LINE
Sampler Initials: JL

TEST SUMMARY

Maxxam ID: FNH590
Sample ID: 6UP-05-SA7
Matrix: Soil

Collected: 2017/10/12
Shipped:
Received: 2017/11/09

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5262943	N/A	2017/11/14	Deonarine Ramnarine
Conductivity	AT	5264045	N/A	2017/11/14	Tahir Anwar
pH CaCl2 EXTRACT	AT	5262832	2017/11/14	2017/11/14	Tahir Anwar
Resistivity of Soil		5261646	2017/11/14	2017/11/14	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5262956	N/A	2017/11/14	Alina Dobreanu

Maxxam ID: FNH590 Dup
Sample ID: 6UP-05-SA7
Matrix: Soil

Collected: 2017/10/12
Shipped:
Received: 2017/11/09

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Conductivity	AT	5264045	N/A	2017/11/14	Tahir Anwar

Maxxam ID: FNH591
Sample ID: 6UP-06-SA4A
Matrix: Soil

Collected: 2017/10/19
Shipped:
Received: 2017/11/09

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5262943	N/A	2017/11/14	Deonarine Ramnarine
Conductivity	AT	5264045	N/A	2017/11/14	Tahir Anwar
pH CaCl2 EXTRACT	AT	5262832	2017/11/14	2017/11/14	Tahir Anwar
Resistivity of Soil		5261646	2017/11/14	2017/11/14	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5262956	N/A	2017/11/14	Alina Dobreanu

Maxxam ID: FNH591 Dup
Sample ID: 6UP-06-SA4A
Matrix: Soil

Collected: 2017/10/19
Shipped:
Received: 2017/11/09

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5262943	N/A	2017/11/14	Deonarine Ramnarine

Maxxam Analytics International Corporation o/a Maxxam Analytics 6740 Campobello Road, Mississauga, Ontario, L5N 2L8 Tel: (905) 817-5700 Toll-Free: 800-563-6266 Fax: (905) 817-5777 www.maxxam.ca

Maxxam Job #: B7P2674
Report Date: 2017/11/14

Golder Associates Ltd
Client Project #: 1670268
Site Location: HWY 400/6TH LINE
Sampler Initials: JL

GENERAL COMMENTS

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

Package 1	2.7°C

Results relate only to the items tested.

Maxxam Analytics International Corporation o/a Maxxam Analytics 6740 Campobello Road, Mississauga, Ontario, L5N 2L8 Tel: (905) 817-5700 Toll-Free: 800-563-6266 Fax: (905) 817-5777 www.maxxam.ca

Maxxam
A Bureau Veritas Group Company

Maxxam Job #: B7P2674
Report Date: 2017/11/14

QUALITY ASSURANCE REPORT

Golder Associates Ltd
Client Project #: 1670268
Site Location: HWY 400/6TH LINE
Sampler Initials: JL

			Matrix Spike		SPIKED BLANK		Method Blank		RPD	
QC Batch	Parameter	Date	% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
5262832	Available (CaCl2) pH	2017/11/14			99	97 - 103			0.32	N/A
5262943	Soluble (20:1) Chloride (Cl)	2017/11/14	NC	70 - 130	106	70 - 130	<20	ug/g	0.12	35
5262956	Soluble (20:1) Sulphate (SO4)	2017/11/14	NC	70 - 130	106	70 - 130	<20	ug/g	1.2	35
5264045	Conductivity	2017/11/14			100	90 - 110	<2	umho/cm	2.6	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)

Maxxam Analytics International Corporation o/a Maxxam Analytics 6740 Campobello Road, Mississauga, Ontario, L5N 2L8 Tel: (905) 817-5700 Toll-Free: 800-563-6266 Fax: (905) 817-5777 www.maxxam.ca

Maxxam Job #: B7P2674
Report Date: 2017/11/14

Golder Associates Ltd
Client Project #: 1670268
Site Location: HWY 400/6TH LINE
Sampler Initials: JL

VALIDATION SIGNATURE PAGE

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

Cristina Carriere

Cristina Carriere, Scientific 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.

Maxxam Analytics International Corporation o/a Maxxam Analytics 6740 Campobello Road, Mississauga, Ontario, L5N 2L8 Tel: (905) 817-5700 Toll-Free: 800-563-6266 Fax: (905) 817-5777 www.maxxam.ca

Maxxam
A Bureau Veritas Group Company

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

CHAIN OF CUSTODY RECORD 60263

Page 1 of 1

Invoice Information		Report Information (if differs from invoice)		Project Information (where applicable)		Turnaround Time (TAT) Required
Company Name:	Golder Associates Ltd.	Company Name:		Quotation #:		[X] Regular TAT (5-7 days) Most analyses
Contact Name:	David Marmor	Contact Name:		P.O. #/ AFE#:		PLEASE PROVIDE ADVANCE NOTICE FOR RUSH PROJECTS
Address:	100-6925 Century Ave.	Address:		Project #:	1670268	Rush TAT (Surcharges will be applied)
	Mississauga, ON L5N 7K2			Site Location:	Hwy 400 / 6th Line	[ ] 1 Day [ ] 2 Days [ ] 3-4 Days
Phone: 905-567-4444 Fax: 905-567-6561		Phone: Fax:		Site #:		
Email:	David_Marmor@golder.com	Email:		Sampled By:		Date Required:

MOE REGULATED DRINKING WATER OR WATER INTENDED FOR HUMAN CONSUMPTION MUST BE SUBMITTED ON THE MAXXAM DRINKING WATER CHAIN OF CUSTODY

Rush Confirmation #:

Regulation 153: [ ] Table 1 [ ] Res/Park [ ] Med/ Fine [ ] Table 2 [ ] Ind/Comm [ ] Coarse [ ] Table 3 [ ] Agri/ Other [ ] Table ___
FOR RSC (PLEASE CIRCLE) Y / N (circled: N)

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

Include Criteria on Certificate of Analysis: Y / N (circled: Y)

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

Analysis Requested

	SAMPLE IDENTIFICATION	DATE SAMPLED (YYYY/MM/DD)	TIME SAMPLED (HH:MM)	MATRIX	# OF CONTAINERS SUBMITTED	FIELD FILTERED (CIRCLE) Metals / Hg / CrVI	BTEX/ PHC F1	PHCs F2 - F4	VOCs	REG 153 METALS & INORGANICS	REG 153 ICPMS METALS	REG 153 METALS (Hg, Cr VI, ICPMS Metals, HWS - B)	Corrosivity Package (pH, sulfate, chloride, resistivity, EC)						HOLD- DO NOT ANALYZE
1	6UP-05-SA7	2017/10/12	am	Soil	1	N							X						
2	6UP-06-SA4A	2017/10/19	am	Soil	1	N							X						
3																			
4																			
5																			
6																			
7																			
8																			
9																			
10																			

REG 153 columns: REFER TO BACK OF COC

LABORATORY USE ONLY

CUSTODY SEAL Y / N — Present / Intact

COOLER TEMPERATURES: 3/3/1

COOLING MEDIA PRESENT: Y / N (circled: Y)

COMMENTS

09-Nov-17 15:45
Ema Gitej
B7P2674
TSP ENV-627

RELINQUISHED BY: (Signature/Print)	DATE: (YYYY/MM/DD)	TIME: (HH:MM)	RECEIVED BY: (Signature/Print)	DATE: (YYYY/MM/DD)	TIME: (HH:MM)	MAXXAM JOB #
Jeremy Lebow	2017/11/09	15:42	Tarant Tanvirsevi	2017/11/09	15:45	

COC-1004 (10/14) - ENV. ENG.

White: Maxxam ~ Yellow: Client

Maxxam
A Bureau Veritas Group Company

Your Project #: 1670268
Site Location: 400 / 6TH LINE
Your C.O.C. #: 76783

Attention: Sandra McGaghran

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

Report Date: 2018/02/06
Report #: R4971214
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B822296
Received: 2018/01/30, 18:58

Sample Matrix: Soil
Samples Received: 2

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Reference
Chloride (20:1 extract)	2	N/A	2018/02/02	CAM SOP-00463	EPA 325.2 m
Conductivity	2	N/A	2018/02/06	CAM SOP-00414	OMOE E3530 v1 m
pH CaCl2 EXTRACT	2	2018/02/02	2018/02/02	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	2	2018/01/30	2018/02/06	CAM SOP-00414	SM 22 2510 m
Sulphate (20:1 Extract)	2	N/A	2018/02/02	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.

Maxxam Analytics International Corporation o/a Maxxam Analytics 6740 Campobello Road, Mississauga, Ontario, L5N 2L8 Tel: (905) 817-5700 Toll-Free: 800-563-6266 Fax: (905) 817-5777 www.maxxam.ca

Your Project #: 1670268
Site Location: 400 / 6TH LINE
Your C.O.C. #: 76783

Attention: Sandra McGaghran

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

Report Date: 2018/02/06
Report #: R4971214
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B822296
Received: 2018/01/30, 18:58

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.

Maxxam Job #: B822296
Report Date: 2018/02/06

Golder Associates Ltd
Client Project #: 1670268
Site Location: 400 / 6TH LINE
Sampler Initials: DMF

RESULTS OF ANALYSES OF SOIL

Maxxam ID		FZU734			FZU734			FZU735		
Sampling Date		2018/01/15			2018/01/15			2018/01/08		
COC Number		76783			76783			76783		
	UNITS	**CE02_4**	**RDL**	**QC Batch**	**CE02_4 Lab-Dup**	**RDL**	**QC Batch**	**6UP-03_5**	**RDL**	**QC Batch**
Calculated Parameters										
Resistivity	ohm-cm	6500		5375407				1900		5375407
Inorganics										
Soluble (20:1) Chloride (Cl)	ug/g	22	20	5381327				250	20	5381327
Conductivity	umho/cm	153	2	5386260	153	2	5386260	531	2	5386260
Available (CaCl2) pH	pH	7.79		5381502				7.97		5381502
Soluble (20:1) Sulphate (SO4)	ug/g	<20	20	5381328				25	20	5381328

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

Maxxam Analytics International Corporation o/a Maxxam Analytics 6740 Campobello Road, Mississauga, Ontario, L5N 2L8 Tel: (905) 817-5700 Toll-Free: 800-563-6266 Fax: (905) 817-5777 www.maxxam.ca

Maxxam
A Bureau Veritas Group Company

Maxxam Job #: B822296
Report Date: 2018/02/06

Golder Associates Ltd
Client Project #: 1670268
Site Location: 400 / 6TH LINE
Sampler Initials: DMF

TEST SUMMARY

Maxxam ID: FZU734
Sample ID: CE02_4
Matrix: Soil

Collected: 2018/01/15
Shipped:
Received: 2018/01/30

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5381327	N/A	2018/02/02	Deonarine Ramnarine
Conductivity	AT	5386260	N/A	2018/02/06	Neil Dassanayake
pH CaCl2 EXTRACT	AT	5381502	2018/02/02	2018/02/02	Tahir Anwar
Resistivity of Soil		5375407	2018/02/06	2018/02/06	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5381328	N/A	2018/02/02	Deonarine Ramnarine

Maxxam ID: FZU734 Dup
Sample ID: CE02_4
Matrix: Soil

Collected: 2018/01/15
Shipped:
Received: 2018/01/30

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Conductivity	AT	5386260	N/A	2018/02/06	Neil Dassanayake

Maxxam ID: FZU735
Sample ID: 6UP-03_5
Matrix: Soil

Collected: 2018/01/08
Shipped:
Received: 2018/01/30

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5381327	N/A	2018/02/02	Deonarine Ramnarine
Conductivity	AT	5386260	N/A	2018/02/06	Neil Dassanayake
pH CaCl2 EXTRACT	AT	5381502	2018/02/02	2018/02/02	Tahir Anwar
Resistivity of Soil		5375407	2018/02/06	2018/02/06	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5381328	N/A	2018/02/02	Deonarine Ramnarine

Maxxam Analytics International Corporation o/a Maxxam Analytics 6740 Campobello Road, Mississauga, Ontario, L5N 2L8 Tel: (905) 817-5700 Toll-Free: 800-563-6266 Fax: (905) 817-5777 www.maxxam.ca

Maxxam
A Bureau Veritas Group Company

Maxxam Job #: B822296
Report Date: 2018/02/06

Golder Associates Ltd
Client Project #: 1670268
Site Location: 400 / 6TH LINE
Sampler Initials: DMF

GENERAL COMMENTS

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

Package 1	5.7°C

Results relate only to the items tested.

Maxxam Analytics International Corporation o/a Maxxam Analytics 6740 Campobello Road, Mississauga, Ontario, L5N 2L8 Tel: (905) 817-5700 Toll-Free: 800-563-6266 Fax: (905) 817-5777 www.maxxam.ca

Maxxam Job #: B822296
Report Date: 2018/02/06

QUALITY ASSURANCE REPORT

Golder Associates Ltd
Client Project #: 1670268
Site Location: 400 / 6TH LINE
Sampler Initials: DMF

			Matrix Spike		SPIKED BLANK		Method Blank		RPD	
QC Batch	Parameter	Date	% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
5381327	Soluble (20:1) Chloride (Cl)	2018/02/02	112	70 - 130	102	70 - 130	<20	ug/g	NC	35
5381328	Soluble (20:1) Sulphate (SO4)	2018/02/02	108	70 - 130	106	70 - 130	<20	ug/g	NC	35
5381502	Available (CaCl2) pH	2018/02/02			99	97 - 103			0.65	N/A
5386260	Conductivity	2018/02/06			100	90 - 110	<2	umho/cm	0.13	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 (Duplicate RPD): The duplicate RPD was not calculated. The concentration in the sample and/or duplicate was too low to permit a reliable RPD calculation (absolute difference <= 2x RDL).

Maxxam Analytics International Corporation o/a Maxxam Analytics 6740 Campobello Road, Mississauga, Ontario, L5N 2L8 Tel: (905) 817-5700 Toll-Free: 800-563-6266 Fax: (905) 817-5777 www.maxxam.ca

VALIDATION SIGNATURE PAGE

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

Cristina Carriere

Cristina Carriere, Scientific 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.

Maxxam
A Bureau Veritas Group Company

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

CHAIN OF CUSTODY RECORD

76783 Page 1 of 1

Invoice Information		Report Information (if differs from invoice)		Project Information (where applicable)		Turnaround Time (TAT) Required
Company Name:	Golder Associates	Company Name:		Quotation #:		[X] Regular TAT (5-7 days) Most analyses
Contact Name:	Sandra McGaghran	Contact Name:		P.O. #/ AFE#:		PLEASE PROVIDE ADVANCE NOTICE FOR RUSH PROJECTS
Address:	6925 Century Ave # 100 Mississauga, ON	Address:		Project #:	1670268	Rush TAT (Surcharges will be applied)
				Site Location:	400 16th Line	[ ] 1 Day [ ] 2 Days [ ] 3-4 Days
Phone: 905-567-4444	Fax:	Phone:	Fax:	Site #:		
Email:	Sandra-Mcgaghran@golder.com	Email:		Sampled By:	DMF	Date Required:

MOE REGULATED DRINKING WATER OR WATER INTENDED FOR HUMAN CONSUMPTION MUST BE SUBMITTED ON THE MAXXAM DRINKING WATER CHAIN OF CUSTODY

Rush Confirmation #:

Regulation 153: [ ] Table 1 [ ] Res/Park [ ] Med/ Fine [ ] Table 2 [ ] Ind/Comm [ ] Coarse [ ] Table 3 [ ] Agri/ Other [ ] Table ____

FOR RSC (PLEASE CIRCLE) Y / N

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

Include Criteria on Certificate of Analysis: Y / N

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

LABORATORY USE ONLY

CUSTODY SEAL Y / N Present	Intact	COOLER TEMPERATURES
N	N	7/6/4

COOLING MEDIA PRESENT: Y / N (N circled)

Analysis Requested (REFER TO BACK OF COC)

	SAMPLE IDENTIFICATION	DATE SAMPLED (YYYY/MM/DD)	TIME SAMPLED (HH:MM)	MATRIX	# OF CONTAINERS SUBMITTED	FIELD FILTERED (CIRCLE) Metals / Hg / CrVI	BTEX/ PHC F1	PHCs F2 - F4	VOCs	REG 153 METALS & INORGANICS	REG 153 ICPMS METALS	REG 153 METALS (Hg, Cr VI, ICPMS Metals, HWS - B)	Corrosivity							HOLD- DO NOT ANALYZE	COMMENTS
1	CE02-4	2018/01/15	AM	SOIL	1								X								
2	6VP-03_5	2018/01/08	AM	SOIL	1								X								
3																					
4																					
5																					
6																					
7																					
8																					
9																					
10																					

30-Jan-18 18:58
Faiz Ahmed
B822296
FCN ENV-793

RELINQUISHED BY: (Signature/Print)	DATE: (YYYY/MM/DD)	TIME: (HH:MM)	RECEIVED BY: (Signature/Print)	DATE: (YYYY/MM/DD)	TIME: (HH:MM)	MAXXAM JOB #
Katie Klen	2018/01/30	6:55	Pardeep / Pardeep K. Pureval	2018/01/30	18:58	

COC-1004 (10/14) - ENV. ENG. White: Maxxam ~ Yellow: Client

APPENDIX C

Non-Standard Special Provisions

DEEP FOUNDATIONS – Item No.

Special Provision No. 903S06 October 2017

Amendment to OPSS 903, April 2016

903.02 REFERENCES

Section 903.02 of OPSS 903 is amended by the addition of the following under **ASTM International:**

D 4945-12 Standard Test Method for High-Strain Dynamic Testing of Deep Foundations

903.03 DEFINITIONS

Section 903.03 of OPSS 903 is amended by the addition of the following:

High Strain Dynamic Testing means a method of evaluating the quality of deep foundations and/or performance of the drive system. It is a form of load testing and involves the instrumenting and application of dynamic loads to a tested pile.

903.04 DESIGN AND SUBMISSION REQUIREMENTS

903.04.02 Submission Requirements

Subsection 903.04.02 of OPSS 903 is amended by the addition of the following clause:

903.04.02.07 High-Strain Dynamic Testing

Prior to commencing high-strain dynamic testing, calibration certificates of all equipment used shall be submitted to the Contract Administrator. All equipment used shall be in good working condition, and shall have been calibrated within the last 2 years according to ASTM D 4945. Equipment set-up may be completed by trained Contractor personnel, however, testing shall be performed under the direction of an Engineer with at least 5 years of experience in high-strain dynamic testing and holding a proficiency rating at the Intermediate level or better for Dynamic Measurement and Analysis Proficiency Test as administered by the Pile Driving Contractors Association (PDCA). After December 31, 2020, the Engineer shall be required to hold a proficiency rating level of Advanced or better.

A preliminary report on the test results and its analysis shall be submitted to the Contract Administrator on the same day of the testing. The analysis shall be based on a closed-form solution (Case Method or approved equivalent) or signal-matching analyses (Case Pile Wave Analysis Program - CAPWAP or approved equivalent). As a minimum, the preliminary report shall include:

a) Pile ultimate resistance and integrity.

b) Calculated driving stresses.

c) Transferred energy and hammer efficiency at the time of the test.

A final report shall be submitted to the Contract Administrator within 10 Days of the field testing. The final report shall include the following:

a) Results of pile ultimate resistance and pile integrity based on signal-matching analyses (CAPWAP or approved equivalent), hammer performance and comparisons with any applicable static load test.

b) Discussion and recommendations for soil setup/relaxation, and/or revised pile installation criteria.

c) An appendix shall be included containing the following documents:
   i. Pile installation record
   ii. Reference subsurface information (borehole record)
   iii. Pile location drawing
   iv. Initial calibration check by the test computer unit
   v. Test set up geometry

The report shall be signed and sealed by two Engineers of the testing company, one of whom shall be identified as MTO's designated contact and one of whom shall have the required experience in high-strain dynamic testing and hold the required certificate of PDCA Proficiency Test.

903.07 CONSTRUCTION

903.07.02.07 Monitoring Driven Piles

903.07.02.07.03 Driving to a Specified Ultimate Resistance

903.07.02.07.03.01 General

Clause 903.07.02.07.03.01 of OPSS 903 is deleted in its entirety and replaced with the following:

When piles are specified to be driven to a specified ultimate resistance, the specified ultimate resistance shall be validated using high-strain dynamic testing at end of drive (EOD). If the specified ultimate resistance is not achieved, retap/restrike should be conducted after sufficient time has passed to allow soil setup. The requirements for soil setup are as specified in the Contract Documents.

The results of the high-strain dynamic tests shall be submitted to the Contract Administrator who shall, in collaboration with the independent testing company, verify that the specified ultimate resistance has been achieved.

903.07.02.07.04 Wave Equation Analysis

Clause 903.07.02.07.04 is deleted in its entirety and replaced with the following:

903.07.02.07.04 Wave Equation Analysis and High-Strain Dynamic Testing

903.07.02.07.04.01 Wave Equation Analysis

Prior to mobilizing piling equipment to the site, a Wave Equation Analysis of Piles (WEAP) analysis shall be performed by the Contractor to demonstrate the potential for the proposed piling equipment to activate the specified ultimate resistance specified in the Contract Documents.

When requested by the Contract Administrator, all equipment, material, and personnel shall be supplied to conduct the wave equation analysis procedure.

903.07.02.07.04.02 High-Strain Dynamic Testing

An independent testing company with no corporate affiliation with the Contractor shall be employed to perform the high-strain dynamic testing. The independent testing company shall be RAQs qualified (Specialty: Geotechnical (Structures and Embankments – Medium or High Complexity)).

High-strain dynamic tests shall be performed by an Engineer employed by the independent testing company. The Engineer shall have documented evidence of training and experience in foundation engineering and wave equation analyses, and a certificate of proficiency (intermediate level or better) in the PDCA Dynamic Measurement and Analysis Proficiency Test.

High-strain dynamic testing shall be performed using the Pile Driving Analyzer, or approved equivalent, for the determination of pile ultimate resistance, establishment of pile installation criteria, assessment of pile integrity, monitoring of hammer/drive system performance and driving stresses, as specified in the Contract Documents. The method and equipment for testing and its reporting shall be according to ASTM D 4945.

The location, sequencing and scheduling of the individual pile testing shall be proposed by the Contractor based on the purpose of the testing, and shall be submitted to the Contract Administrator for information purposes.

High-strain dynamic testing shall be carried out at the end of initial driving on a minimum of 10% of piles in each pile group, rounded up, but no fewer than 2 piles; or as specified in the Contract Documents.

Additional high strain dynamic testing (i.e. restrike testing) shall be carried out during the retapping of piles, as specified in the Retapping Tests on Piles clause. Restrike testing shall be performed on a minimum of 10% of piles in each pile group, rounded up, but no fewer than 2 piles; or as specified in the Contract Documents.

Restrike testing shall be carried out no sooner than 24 hours after installation of the individual pile and at a time specified in the Contract Documents. If the hammer needs to be warmed up prior to performing a restrike, it shall not be warmed up by striking the intended test pile.

903.07.02.07.06 Retapping Tests on Piles

Section 903.07.02.06 is deleted in its entirety and replaced by the following:

In each pile group, 10% of the piles rounded up to the next whole number, but no fewer than two piles, shall be retapped no sooner than 48 hours after installation of the individual pile to confirm that the ultimate axial geotechnical resistance has been achieved and/or sustained.

903.10 BASIS OF PAYMENT

Section 903.10 of OPSS 903 is amended by the addition of the following subsection:

903.10.04 High-Strain Dynamic Testing, Deep Foundations - Item

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

WARRANT: Always with this item.

<u>VIBRATION MONITORING</u> – Item No.

Special Provision

TABLE OF CONTENTS

1.0 SCOPE

This special provision describes requirements for vibration monitoring during deep foundation and temporary protection system installation for the construction of the Highway 400/6th Line underpass, and protection system installation, as required, related to backfilling of the existing Highway 400/6th Line overpass.

2.0 REFERENCES

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

Highway 400 / 6th Line Underpass (Structure Site No. 30-211/1&2) and High Fill Embankments
Town of Innisfil, Simcoe County, Ontario
MTO GWP 2289-13-00

Backfilling of Highway 400 / 6th Line Overpass (Site No. 30-211/1&2)
Town of Innisfil, Simcoe County, Ontario
MTO GWP 2289-13-00

3.0 DEFINITIONS

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

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

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

Pre-Construction Condition Survey means a detailed record, accompanied by film or video, as necessary, of the condition of private or public property, prior to the commencement of vibratory 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 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 650 m radius from the overpass and underpass structures, as applicable.
e) Proposed frequency of readings.
f) Action plan to be taken to adjust deep foundation and protection system installation methods 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 650 m of each abutment, pier and/or protection system location.

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 650 m radius of the bridge, shall be completed a minimum of two (2) weeks prior to commencement of installation of deep foundations and temporary protection systems. Only one Pre-Construction Condition Survey per structure or facility is required to be carried out in advance of deep foundation or temporary protection system installation, unless more than six (6) months will elapse between these operations, in which case an interim inspection will be required.

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

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

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

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

7.1.2 Post-Construction Condition Surveys

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

A Post-Construction Condition Survey at each structure within a 650 m radius of the bridge, is required within two (2) months of completion of the installation of deep foundations and temporary protection systems at each of the east and west sides of the bridge.

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

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

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

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

7.2 Monitoring

The vibration monitoring equipment shall be placed on the ground surface in the vicinity of each foundation element or protection system, and on the ground surface at radial distances of 25 m, 50 m, and 100 m from the foundation element or protection system locations at the bridge site(s). The Contractor shall take readings continuously during deep foundation installation and during installation of temporary protection systems, 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.

<u>CSP FOR INTEGRAL ABUTMENTS – Item No</u>

Non-Standard Special Provision

Scope

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

Submission and Design Requirements

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

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

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

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

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

Material

Corrugated steel pipe

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

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

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

Sand Fill

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

Table 1 – Sand Fill Gradation Requirements

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

Construction

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

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

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

The Contractor shall ensure the full perimeters of the tops of all CSPs at each abutment are at the elevation and orientation shown on the working drawings.
The CSP at each pile shall be constructed to the following tolerances:

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

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

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

Basis of Payment

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

END OF SECTION

OPERATIONAL CONSTRAINT (STRUCTURAL) – Soil Condition

Special Provision

The Contactor is hereby notified that the native soils at the site of the existing Highway 400 / 6th Line overpass and the proposed realigned Highway 400 / 6th Line underpass sites, as inferred from available information regarding till deposits, should be expected to contain cobbles and boulders.

These soil conditions could affect various activities such as excavations, installation of deep foundations and installation of temporary protection systems, among other activities. The presence of the above-noted subsurface obstructions shall be considered by the Contractor in the selection of appropriate equipment and procedures for various activities such as excavation, installation of the foundations and installation of the temporary protection system.

As a global, employee-owned organisation with over 50 years of experience, Golder Associates is driven by our purpose to engineer earth's development while preserving earth's integrity. We deliver solutions that help our clients achieve their sustainable development goals by providing a wide range of independent consulting, design and construction services in our specialist areas of earth, environment and energy.

For more information, visit golder.com

Africa	+ 27 11 254 4800
Asia	+ 86 21 6258 5522
Australasia	+ 61 3 8862 3500
Europe	+ 44 1628 851851
North America	+ 1 800 275 3281
South America	+ 56 2 2616 2000

solutions@golder.com
www.golder.com

Golder Associates Ltd.
6925 Century Avenue, Suite #100
Mississauga, Ontario, L5N 7K2
Canada
T: +1 (905) 567 4444