



# Foundation Investigation and Design Report

*High Fill Embankments, QEW / Glendale Avenue Interchange Improvements  
Niagara-on-the-Lake, Ontario  
MTO GWP 2423-15-00*

Submitted to:

**AECOM**

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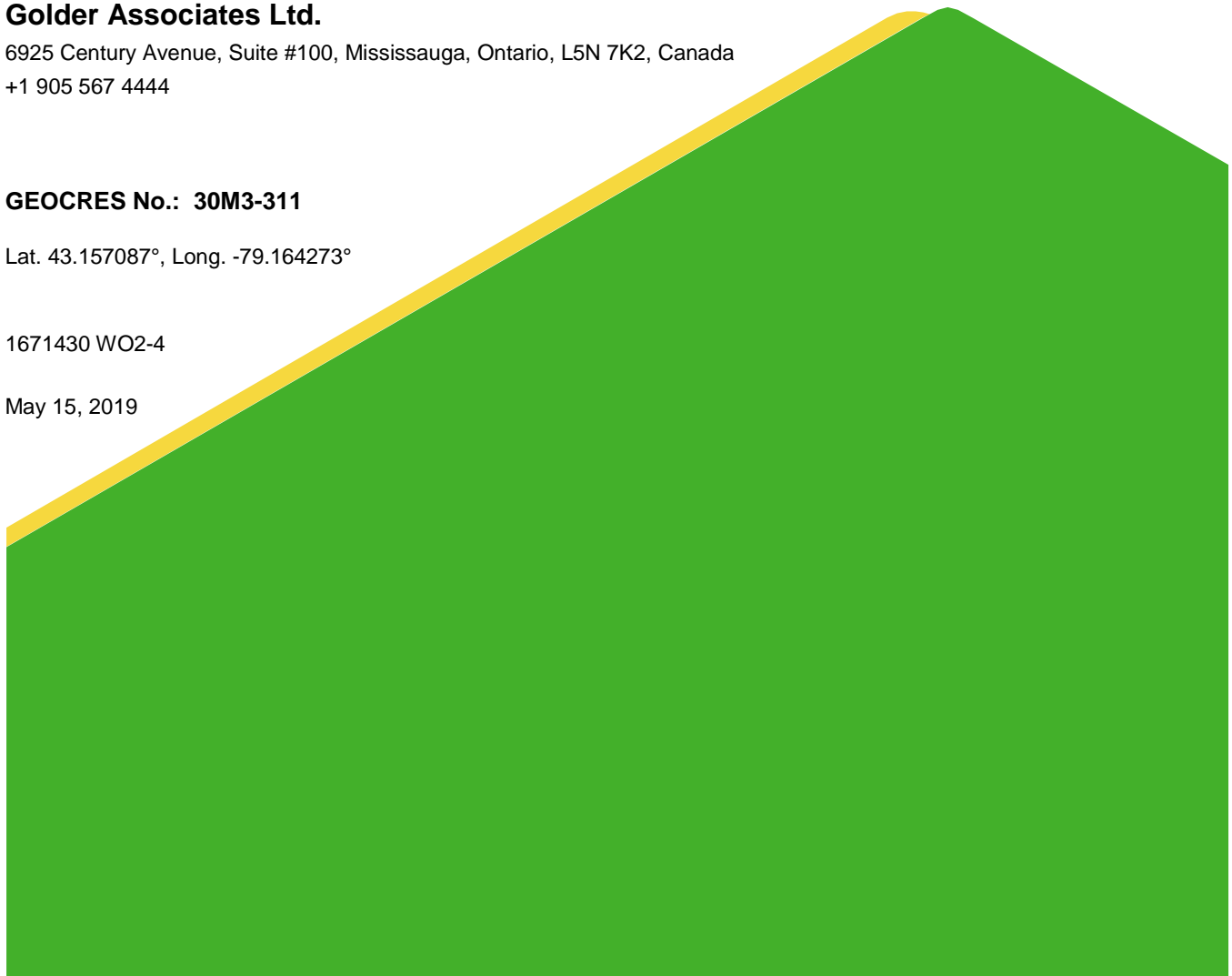
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Rock Laboratory Test Result Report

# PART A

FOUNDATION INVESTIGATION REPORT  
HIGH FILL EMBANKMENTS  
QEW / GLENDALE AVENUE INTERCHANGE IMPROVEMENTS  
NIAGARA-ON-THE-LAKE, ONTARIO  
MTO GWP 2423-15-00

## 1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by AECOM on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the detail design of the replacement of the Glendale Avenue underpass and associated interchange improvements in the Town of Niagara-on-the-Lake, Regional Municipality of Niagara (Assignment No. 2016-E-0029-002), located as shown on the attached Key Plan on Figure 1.

This report addresses the proposed new or widened high fill embankment areas requiring placement of more than 4.5 m of fill along Glendale Avenue, as well as along portions of the following ramp alignments on the east side of Glendale Avenue:

- Glendale Avenue Ramp N/S-E in the southeast quadrant of the interchange; and
- QEW Ramps E-N/S in the northeast quadrant of the interchange.

Some limited high fill embankments are also proposed on the west flank of the existing Glendale Avenue embankment, where the proposed N/S-W and W-N/S Ramps rise to meet the existing and new Glendale Avenue grade.

The purpose of this investigation is to establish the subsurface soil and groundwater conditions in the general vicinity of the high fill embankment areas by borehole drilling, in situ testing and laboratory testing on selected soil samples. The investigation areas are shown in plan on Drawing 1. This report complements the high fill embankment investigation included for the approach embankments at the Glendale Avenue underpass replacement structure and the Airport Road connection structure.

The Terms of Reference for the foundation engineering services are outlined in MTO's Work Item Order No. 2016-E-0029-002, dated July 2017, which forms part of the Consultant's Assignment for the Central Region Large Value Retainer under Agreement No. 2016-E-0029-002.

## 2.0 SITE DESCRIPTION

The existing Glendale Avenue underpass and interchange is located east of the Garden City Skyway and west of the General Brock Parkway (Highway 405) - Queen Elizabeth Way (QEW) interchange in the Town of Niagara-on-the-Lake, Ontario. For the purposes of this report, QEW is assumed to be oriented in an east-west direction, and Glendale Avenue in a north-south direction. Commercial developments are located in the southwest and northwest quadrants of the interchange, Niagara College is located in the southeast quadrant of the interchange, and an undeveloped vegetated area is present in the northeast quadrant of the interchange.

The ground surface along the QEW at the existing and proposed Glendale Avenue underpass locations varies from approximately Elevation 117 m to 118 m, rising from west to east. The existing ground surface outside of the QEW and Glendale Avenue embankments varies from approximately Elevation 115 m to 118 m in the interchange area. The existing Glendale Avenue was constructed on embankments up to approximately 6 m high relative to the natural ground surface, with the existing Glendale Avenue grade at approximately Elevation 123 m to 124 m over the QEW; based on visual observation by Golder at the time of the field investigation, the existing embankments do not exhibit evidence of global or surficial instability nor of ongoing settlement.

### 3.0 INVESTIGATION PROCEDURES

The field work for this investigation was carried out on between July 23 and August 24, 2018, and between September 17 and October 30, 2018. During this time total of twenty-three boreholes were advanced at the site; nine of these boreholes (Boreholes HF-1 to HF-5, GUA-1, GUA-3, GUA-7 and GUA-8) are included in this high fill embankment report, including use of selected deeper boreholes at the foundation elements for the Glendale Avenue underpass structure. The boreholes were advanced at the locations shown on Drawing 1 to provide broad coverage of the high fill embankment areas. Drawing 1 also illustrates the locations of other boreholes advanced at this site for the Glendale Avenue underpass and Airport Road connection structures, the details of which are presented in the Foundation Investigation Reports for those structures.

Boreholes GAU-1, GAU-3 and GAU-7 were drilled using 210 mm outer diameter hollow stem augers, and Boreholes HF-1 to HF-5 and GAU-8 were drilled using 152 mm outer diameter hollow stem augers, using a CME-55 track-mounted drilling rig supplied and operated by Geo-Environmental Drilling of Halton Hills, Ontario. Soil samples were obtained at 0.75 m and 1.5 m intervals of depth using a 50 mm outer diameter split-spoon sampler driven by an automatic hammer in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586)<sup>1</sup>.

The groundwater conditions in the open boreholes were observed during and immediately following the drilling operations. Standpipe piezometers were installed in Boreholes GAU-1, GAU-3 and GAU-8 to permit monitoring of the water levels. The installed piezometers consist of a 50 mm diameter PVC pipe, with a 1.5 m slotted screen within a filter sand pack sealed at a selected interval as shown on the borehole records in Appendix A. The borehole and annulus surrounding the piezometer pipe above the filter sand pack were backfilled to near ground surface with bentonite pellets, and the upper 200 mm of the borehole was capped with cold patch asphalt to the roadway surface. Boreholes GAU-7 and HF-1 to HF-5, in which no piezometers were installed, were backfilled to ground surface with bentonite upon completion, in accordance with Ontario Regulation 903 (Wells, as amended).

The field work was monitored on a full-time basis by a member of Golder's technical staff who located the boreholes in the field, directed the sampling and in situ testing operations, logged the boreholes and examined the soil samples. The soil samples were identified in the field, placed in labelled containers and transported to Golder's laboratory in Mississauga for further visual review and geotechnical laboratory testing on selected samples, consisting of natural moisture content, Atterberg limits and grain size distribution conducted in accordance with MTO and / or ASTM Standards as applicable.

The borehole locations and elevations were surveyed by Callon Dietz Surveying using survey equipment with a horizontal and vertical accuracy of 0.05 m. The locations given on the borehole records and shown on Drawing 1 are positioned relative to MTM NAD 83 (Zone 10) northing and easting coordinates and the ground surface elevations are referenced to Geodetic datum. The borehole locations, including geographic (latitude / longitude) coordinates, ground surface elevations and borehole depths are summarized below.

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

Borehole No.	MTM NAD83		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (m) (Latitude)	Easting (m) (Longitude)		
GAU-1	4,779,785.4 (43.157430)	332,141.3 (-79.163803)	122.1	17.4
GAU-3	4,779,764.4 (43.157241)	332,138.9 (-79.163833)	119.1	33.8
GAU-7	4,779,711.3 (43.156764)	332,115.4 (-79.164125)	116.1	34.1
GAU-8	4,779,697.2 (43.156638)	332,094.7 (-79.164388)	115.6	15.9
HF-1	4,779,836.5 (43.157890)	332,159.9 (-79.163572)	116.5	16.5
HF-2	4,779,777.7 (43.157359)	332,179.2 (-79.163337)	118.1	16.5
HF-3	4,779,749.9 (43.157108)	332,220.6 (-79.162829)	117.6	16.6
HF-4	4,779,686.0 (43.156536)	332,139.0 (-79.163836)	114.8	15.9
HF-5	4,779,652.6 (43.156237)	332,071.9 (-79.164663)	117.5	16.2

## 4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

### 4.1 Regional Geology

This section of the QEW lies within the physiographic region known as the Iroquois Plain, as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, 1984)<sup>2</sup> and *Urban Geology of Canadian Cities* (Menzies and Taylor, 1998)<sup>3</sup>.

The Iroquois Plain extends around the western shores of Lake Ontario; on the south side of the lake, in the St. Catharines area, the Plain is located between the present Lake Ontario shorebluffs and the foot of the Niagara Escarpment. The Plain is comprised of the flat to undulating lakebed and beaches of the former glacial Lake Iroquois, which occupied this area during the last glacial recession.

<sup>2</sup>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.

<sup>3</sup>Menzies, J., and Taylor, E.M., 1998. *Urban Geology of St. Catharines-Niagara Falls, Region Niagara*. In *Urban Geology of Canadian Cities*, Geological Association of Canada Special Paper 42, Ed. P.F. Karrow and O.L. White.



The surficial soils in the Iroquois Plain are typically comprised of glaciolacustrine clays and silts. The surficial sands, silts and clays are underlain by an extensive till deposit; portions of the till are considered to be “water-lain” (that is, formed by sediment rain-out either from a floating ice margin or from iceberg dumping), resulting in a predominantly massive, matrix-supported structure, as well as relatively thin sand to silt stringers or interlayers. This extensive till deposit may be underlain by or interlayered with a lower glaciolacustrine clay deposit, although this glaciolacustrine layer is absent in some portions of the Iroquois Plain in the St. Catharines area. Finally, the till and/or glaciolacustrine layer may be underlain by a lower till unit, that typically has increasing gravel content with proximity to the underlying bedrock (Menzies and Taylor, 1998).

The overburden soils are underlain by red shale bedrock of the Queenston Formation. This shale formation contains siltstone interlayers as well as “occasional patches of gypsum” (Menzies and Taylor, 1998).

## 4.2 General Overview of Subsurface Conditions

The detailed subsurface soil and groundwater conditions encountered in the selected boreholes included in this report, including piezometer installation details and water level readings, and the results of the in situ and laboratory tests are provided on the borehole and drillhole records in Appendix A. The results of the in-situ field tests (i.e., SPT “N”-values) as presented on the borehole records and in Section 4 are uncorrected. The Standard Penetration Test “N”-values are based on use of an automatic hammer and the values are reported with no adjustment in this report, although it is recognized that SPT “N” values obtained using an automatic hammer are frequently lower than those obtained using a manual hammer (CFEM, 2006)<sup>4</sup>. The results of the geotechnical laboratory testing on soil samples are presented on the laboratory test figures in Appendix B.

The stratigraphic boundaries shown on the borehole records and on the stratigraphic profiles on Drawings 1 and 2 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. Variation in the stratigraphic boundaries between and beyond boreholes will exist and is to be expected; however, the factual data presented on the borehole records governs any interpretation of the site conditions.

In general, the subsurface conditions encountered at the site consist of surficial layers of topsoil and asphalt, underlain by non-cohesive and cohesive fill material. The predominant native soil deposit at the site is silty clay to clay, consisting of a very stiff to hard crust and becoming typically stiff or firm to stiff with depth. This deposit is overlain in some areas by a stiff to very stiff clayey silt layer, and underlain in places by a typically stiff to hard clayey silt to clayey silt till deposit. These cohesive deposits are underlain by a compact to very dense silt and sand to sand deposit that generally overlies the shale bedrock, although thin layers of clayey silt till or residual soil were observed above the bedrock in one of the boreholes included in this report, as well as in other boreholes elsewhere at the site.

### 4.2.1 Topsoil

An approximately 75 mm to 300 mm thick layer of topsoil was encountered from ground surface in Boreholes GAU-1, GAU-3, GAU-7 and HF-1. The topsoil was classified based on visual and textural observations; organic content testing was not carried out during the current investigations.

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<sup>4</sup> Canadian Geotechnical Society, 2006. *Canadian Foundation Engineering Manual*, 4th Edition.

### 4.2.2 Fill

Fill material was encountered in all of the boreholes included in this report, either underlying the topsoil or immediately below the ground surface. The base of the fill layer was encountered between approximately Elevation 116.9 m to 113.4 m in the boreholes, at depths between 1.4 m and 5.6 m below the ground surface at the borehole locations.

The fill is generally cohesive, varying in composition from clayey silt to silty clay to clay, with trace to some sand and gravel along with trace rootlets. A thin layer (about 400 mm thick) of non-cohesive sand and gravel fill was encountered below the topsoil and above the cohesive fill in Borehole GAU-1 included in this report; this is consistent with thin layers of non-cohesive fill encountered in other boreholes advanced through or near the existing Glendale Avenue embankment or other roadways.

The SPT “N”-values measured within the cohesive fill range from 9 blows to 17 blows per 0.3 m of penetration, indicating a soft to stiff consistency. One SPT “N”-value of 32 blows per 0.3 m of penetration was measured within the granular fill, indicating a dense relative density.

The results of grain size distribution tests completed on three samples of the cohesive fill are presented on Figure B-1 in Appendix B. Atterberg limits testing was carried out on four samples of the cohesive fill and measured plastic limits ranging from about 20 per cent to 22 per cent, liquid limits ranging from about 42 per cent to 54 per cent, and plasticity indices ranging from about 22 per cent to 34 per cent, indicating that the cohesive fill varies in composition from silty clay of intermediate plasticity to clay of high plasticity as presented on the plasticity chart on Figure B-2 in Appendix B. The water content measured in the fill material ranges from 6 per cent to 29 per cent, and field observations indicate moist to wet conditions.

### 4.2.3 Silty Clay to Clay

The predominant native soil deposit at the site is a cohesive deposit generally consisting of silty clay to clay; portions of the deposit are varved, and the deposit typically contains trace sand and trace gravel. This deposit was encountered below the fill with its surface between Elevation 116.9 m and 114.1 m in Boreholes GAU-1, GAU-3, GAU-7, GAU-8, HF-1 and HF-5, and below the surficial clayey silt layer (described below in Section 4.2.4) with its surface between Elevation 115.4 m and 112.5 m in Boreholes HF-2 to HF-4. Where fully penetrated, the deposit ranges in thickness from about 6.1 m to 17.2 m, extending to approximately Elevation 104.3 m to 99.7 m.

The deposit consists of a very stiff upper “crust” zone, and becomes less stiff with depth, as follows:

- The SPT “N”-values recorded within the upper “crust”, generally above approximately Elevation 106 m to 108 m, range from 5 blows to 32 blows per 0.3 m of penetration, indicating a firm to hard consistency. In situ vane tests carried out within this portion of the deposit measured undrained shear strengths greater than 96 kPa. In consideration of the field vane test results, the upper crust generally has a very stiff consistency.
- The SPT “N”-values measured within the lower portion of the deposit range from 0 blows (weight of hammer, WH) to 18 blows per 0.3 m of penetration, but are typically between approximately 2 blows and 8 blows per 0.3 m of penetration. In situ vane tests carried out within this deposit measured undrained shear strength ranging from about 24 kPa to greater than 96 kPa, but typically greater than about 80 kPa. The sensitivity ranges from about 1.3 to 3.7 in this lower portion of the deposit. Based on the field vane tests results together with the SPT “N”-values, this portion of the deposit has a firm to very stiff consistency, but is typically stiff.

The results of grain size distribution testing completed on 23 samples of this deposit are shown on Figures B-3A to B-3D in Appendix B. Atterberg limits testing was carried out on 25 samples of this deposit and generally measured plastic limits ranging from about 17 per cent to 23 per cent, liquid limits ranging from about 36 per cent to 54 per cent, and plasticity indices ranging from about 17 per cent to 33 per cent. These test results, which are plotted on plasticity charts on Figures B-4A to B-4D in Appendix B, indicate that the deposit consists of silty clay of intermediate plasticity to clay of high plasticity; one tested sample is classified as a clayey silt of low plasticity, based on a plastic limit of about 17 per cent, a liquid limit of about 28 per cent, and a plasticity index of about 11 per cent (see Figure B-4B in Appendix B). The natural water content measured on samples of the deposit range from about 18 per cent to 44 per cent.

Laboratory consolidation tests were carried out on three samples of the cohesive deposit obtained from Shelby tubes in Boreholes GAU-2, GAU-5 and GAU-6 drilled as part of the 2018 investigation at this site; these boreholes are included in the Foundation Investigation Report for the Glendale Avenue underpass structure, located within the high fill embankment area. A pre-consolidation stress ranging between about 309 kPa and 344 kPa was estimated from the void ratio versus logarithmic pressure plots and from the total work versus pressure plots. Unit weights ranging between about 18.6 kN/m<sup>3</sup> and 19.6 kN/m<sup>3</sup> and specific gravities between about 2.72 and 2.78 were measured on the consolidation test samples. The over consolidation ratio (OCR) ranges from approximately 1.1 to 1.7. Details of the test results are shown on Figures B-5A to B-5D, B-6A to B-6D and B-7A to B-7D in Appendix B, and the test results are summarized below.

Borehole and Sample No.	Sample Depth / Elevation	$\sigma_{vo}'$ (kPa)	$\sigma_p'$ (kPa)	$\sigma_p' - \sigma_{vo}'$ (kPa)	OCR	$C_c$	$C_r$	$e_o$	$c_v^1$ (cm <sup>2</sup> /s)
GAU-2 Sample 16	18.3 m / 104.1 m	261	309	48	1.19	0.42	0.01	0.83	$1.4 \times 10^{-3}$ $1.5 \times 10^{-3}$
GAU-5 Sample 13	15.2 m / 101.6 m	235	325	90	1.38	0.28	0.02	0.68	$1.8 \times 10^{-3}$ $2.9 \times 10^{-3}$
GAU-6 Sample 10	10.7 m / 105.6 m	199.8	343.9	144	1.72	0.34	0.09	0.84	$3.7 \times 10^{-3}$ $3.1 \times 10^{-3}$

**Note:**

- Two coefficients of consolidation ( $c_v$ ) have been presented for each sample. The first value (top line) is based on a stress range below the effective overburden stress (i.e., within the over consolidated stress range). The second value (bottom line) is based on a stress range between the effective overburden stress and the final stress due to 8 m high embankment and a 4.5 m high embankment.

$\sigma_{vo}'$  is the in situ vertical effective overburden stress in kPa

$\sigma_p'$  is the pre-consolidation stress in kPa

OCR is the over-consolidation ratio

$e_o$  is the initial void ratio

$C_c$  is the compression index

$C_r$  is the recompression index

$c_v$  is the coefficient of consolidation in cm<sup>2</sup>/s

#### 4.2.4 Clayey Silt

Zones of clayey silt were encountered in the boreholes advanced at the site, as follows:

- Immediately below the fill (with surface between Elevation 115.4 m and 113.4 m) in Boreholes HF-2 to HF-4; in these boreholes, this layer is about 2.6 m to 5.0 m thick, with its base at approximately Elevation 112.5 m to 109.2 m. This upper clayey silt zone is typically described as containing trace sand, trace gravel and trace rootlets; sand seams were noted in some of the recovered samples. The SPT "N"-values in this upper clayey silt zone range from 11 blows to 21 blows per 0.3 m of penetration, suggesting a stiff to very stiff consistency.

- At depth below the silty clay to clay deposit in the deeper boreholes, GAU-3, GAU-7, and GAU-8, as well as HF-3 and HF-4. The surface of the clayey silt layer was encountered in these boreholes between approximately Elevation 104.3 m and 99.7 m, and this layer is 3.8 m to 3.9 m thick where fully penetrated. This lower deposit is described as clayey silt, trace to some sand, trace gravel, to sandy clayey silt, trace to some gravel; in some locations, it has been interpreted as a till deposit based on its broad gradation. The SPT “N”-values within this lower clayey silt zone generally range from 80 to greater than 100 blows per 0.3 m of penetration, suggesting a hard consistency. However, two lower SPT “N”-values of 7 and 8 blows per 0.3 m of penetration were measured in this zone in Borehole HF-3; field vane testing measured undrained shear strengths of greater than 96 kPa following these samples, suggesting a stiff to very stiff consistency.

The results of grain size distribution tests completed on six samples of the clayey silt deposits are shown on Figure B-8 in Appendix B. Atterberg limits tests were carried out on six samples of this cohesive deposit and measured plastic limits ranging from about 13 per cent to 18 per cent, liquid limits ranging from about 18 per cent to 32 per cent and plasticity indices ranging from about 5 per cent to 14 per cent. The results of the Atterberg limits tests are shown on a plasticity chart on Figure B-9 in Appendix B, and indicate that these zones are classified as clayey silt of low plasticity.

#### 4.2.5 Silt and Sand to Sand

A non-cohesive deposit was encountered underlying the cohesive deposits in Boreholes GAU-3 and GAU-7. This deposit varies in composition from silt and sand to sand, trace to some silt and trace to some clay. The surface of the deposit was encountered between Elevations 96.2 m and 95.9 m, and the deposit is about 6.3 m to 10.5 m in thickness, with its base at approximately Elevation 89.6 m to 85.7 m.

The SPT “N”-values measured within the silt and sand to sand deposit range from 14 blows to greater than 100 blows per 0.3 m of penetration, indicating a compact to very dense, but typically very dense compactness condition.

Grain size distribution testing carried out on three samples of this deposit are shown on Figure B-10 in Appendix B. The natural water content measured on selected samples of this deposit range from about 12 per cent to 24 per cent.

#### 4.2.6 Clayey Silt Till/Residual Soil

A thin (0.2 m thick) layer of clayey silt till/residual soil was encountered below the silt to sand deposit and above the bedrock in Borehole GAU-3; the surface of this layer was encountered at Elevation 89.6 m. This layer is described as sandy clayey silt, some gravel and some shale fragments.

#### 4.2.7 Shale Bedrock

Bedrock was encountered and confirmed by bedrock coring in Boreholes GAU-3 and GAU-7. The depth to bedrock below ground surface and the corresponding bedrock surface elevations at each borehole location are summarized below.

Borehole No.	Depth to Bedrock Surface (m)	Bedrock Surface Elevation (m)	Comments
GAU-3	29.7	89.4	Bedrock cored 4.1 m
GAU-7	30.4	85.7	Bedrock cored 3.7 m

In general, the inferred / confirmed bedrock surface as encountered in the boreholes advanced in the area of the proposed Glendale Avenue underpass and Airport Road connection structures (as reported under the Foundation Investigation Reports for those structure sites) varies from about Elevation 85.7 m to 89.4 m.

Based on a review of the bedrock core samples, the bedrock consists of shale of the Queenston Formation. In general, the bedrock samples are described as moderately weathered to slightly weathered, very thin to medium bedded, fine grained, faintly to non-porous, weak, grey, with very thin to thin medium strong limestone interbeds at varying intervals, as presented in the drillhole records in Appendix A.

Typically, the upper portion of the bedrock surface is weathered and transitions to slightly weathered to fresh at depth. The degree of weathering of the bedrock samples (e.g. slightly weathered – W2), and the strength classification of the intact rock mass based on field identification (e.g. weak – R2) are described in accordance with the International Society for Rock Mechanics (ISRM3) standard classification system.

The Rock Quality Designation (RQD) measured on the core samples throughout the site (including boreholes from the Glendale Avenue underpass and Airport Road connection structure) ranges between 0 per cent and 91 per cent, with core runs near the bedrock surface ranging from about 0 per cent to 25 per cent (indicating a rock mass of very poor to poor quality) and core runs at depth (typically below about 1 m below bedrock surface) ranging from about 50 per cent to 100 per cent (indicating a rock mass of fair to excellent quality), per Table 3.10 of CFEM (2006)<sup>4</sup>. The Total Core Recovery (TCR) and Solid Core Recovery (SCR) of samples recovered range between 59 per cent and 100 per cent and between 22 per cent and 100 per cent, respectively.

Uniaxial compression tests (ASTM D7012) were carried out on selected core samples of the shale bedrock and the uniaxial compressive strength (UCS), and bulk density of the intact samples are summarized below, and the details are presented in the Rock Laboratory Test Result report from Geomechanica in Appendix B.

Borehole No.	Sample Depth Interval (m)	Sample Elevation Interval (m)	Uniaxial Compressive Strength (UCS) (MPa)	Bulk Density (g/cm <sup>3</sup> )
GAU-3 (Run #2)	32.0 – 32.2	87.1 to 86.8	13.7	2.66
GAU-7 (Run #3)	32.6 – 32.8	83.5 to 83.3	24.5	2.56

Based on the laboratory UCS tests, in accordance with Table 3.5 in CFEM (2006)<sup>4</sup>, the shale bedrock is generally classified as weak (R2, 5 MPa < UCS < 25 MPa).

### 4.3 Groundwater Conditions

The groundwater levels in the open boreholes were measured upon completion of drilling operations; these measurements are shown on the borehole records in Appendix A, but do not represent the stabilized groundwater level at the site. Piezometers were installed in Boreholes GAU-1, GAU-3, GAU-8 and ARB-4 (as reported in the Foundation Investigation Reports for the adjacent Glendale Avenue underpass and Airport Road connection structures) and the measured groundwater levels are summarized below.

Borehole No.	Ground Surface Elevation (m)	Depth to Groundwater (m)	Groundwater Elevation (m)	Date
GAU-1	122.1	Dry	Below 105.4	14-Aug-18
		9.8	112.3	07-May-19
GAU-3	119.1	10.2	108.9	19-Sep-18
		10.5	108.6	28-Sep-18
		10.4	108.7	26-Nov-18
		10.4	108.7	07-May-19
GAU-8	115.6	14.5	101.1	28-Sep-18
		4.5	111.1	26-Nov-18
		4.6	111.1	07-May-19
ARB-4	117.1	14.3	102.8	25-Oct-18
		9.3	107.8	26-Nov-18
		9.1	108.9	07-May-19

Based on the water level measurements in the piezometers, it is estimated that the groundwater level associated with the silty clay to clay deposit is at approximately Elevation 111 m to 112 m. The groundwater measurements at approximately Elevation 109 m are associated with the underlying sand/silt deposit. The groundwater level will be subject to seasonal fluctuations and should be expected to be higher during the spring season or during and following periods of heavy precipitation.

## 5.0 CLOSURE

This Foundation Investigation Report was prepared by Ms. Manisha Ahuja, P.Eng., P.E., a geotechnical engineer with Golder. Ms. Lisa Coyne, P.Eng., a Principal and MTO Foundations Designated Contact of Golder, conducted an independent technical and quality control review of this report.

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# PART B

FOUNDATION DESIGN REPORT  
HIGH FILL EMBANKMENTS  
QEW / GLENDALE AVENUE INTERCHANGE IMPROVEMENTS  
NIAGARA-ON-THE-LAKE, ONTARIO  
MTO GWP 2423-15-00



## 6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

### 6.1 General

This section of the report provides detail foundation design recommendations for the high fill embankments associated with the Glendale Avenue improvements in the Town of Niagara-on-the-Lake, Regional Municipality of Niagara (Assignment No. 2016-E-0029-002). These recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the 2018 subsurface investigation at this site. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the requirements for stability and settlement of the high fill embankments and underlying soils.

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

The replacement of the Glendale Avenue underpass and reconstruction of the interchange will involve an eastward shift of the Glendale Avenue embankment and replacement structure. High fill embankments, on the order of 4.5 m to 7 m in height, will be required along Glendale Avenue to carry the local road over the QEW and the Airport Road connection; the limits of the high fill embankments extend from approximately Station 9+920 to 10+100, based on the profiles and cross-sections received from AECOM in January 2019. Portions of the E-N/S and N/S-E Ramps near the tie-ins with Glendale Avenue also require fill placement of greater than 4.5 m, and some limited high fill areas will occur on the west flank of the existing Glendale Avenue embankment, where the proposed N/S-W and W-N/S Ramps rise to meet the Glendale Avenue grade.

### 6.2 General Foundation Design Context

#### 6.2.1 Consequences and Level of Understanding

In accordance with Section 6.5 of the *Canadian Highway Bridge Design Code* CAN/CSA S6-14 (*CHBDC (2014)*) and its *Commentary*, the proposed embankments are expected to carry medium to high traffic volumes and their performance may have potential impacts on other transportation corridors; hence, the proposed high fill embankment works have been assessed as having a “typical consequence level” associated with exceeding limits states design.

In addition, given the project-specific foundation investigation carried out across the site, in comparison to the degree of site understanding in Section 6.5 of *CHBDC (2014)*, the level of confidence for design is considered to be a “typical degree of site and prediction model understanding”. Accordingly, the appropriate corresponding ultimate limit state (ULS) and serviceability limit state (SLS) consequence factor,  $\Psi$ , and geotechnical resistance factors,  $\phi_{gu}$  and  $\phi_{gs}$ , from Tables 6.1 and 6.2 of the *CHBDC (2014)* have been used for the design.

For a “typical degree of understanding”, *CHBDC (2014)* requires a minimum Factor of Safety of 1.3 for short-term/temporary condition and 1.5 for the long-term/permanent condition for global stability of embankments.



## 6.2.2 Seismic Design

### 6.2.2.1 Seismic Site Classification

The subsurface conditions for seismic site characterization were assessed based on the results of the field investigation and laboratory testing. The SPT “N”-values measured in the soil layers and the interpreted shear wave velocity of soils up to 30 m below founding level were used to define the seismic site classification in accordance with Table 4.1 of the *CHBDC (2014)*. Based on this methodology, it is considered that a Site Class D would be applicable for the design of the new embankment.

### 6.2.2.2 Spectral Response Values and Seismic Performance Category

In accordance with Section 4.4.3.4 of the *CHBDC (2014)* and as obtained from NRC (2015) website, the peak ground acceleration (PGA) and peak ground velocity (PGV) values and design spectral acceleration ( $S_a$ ) values for Site Class D 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)
<b>PGA (g)</b>	0.074	0.139	0.266
<b>PGV (m/s)</b>	0.053	0.094	0.178
<b><math>S_a</math> (0.2) (g)</b>	0.113	0.208	0.396
<b><math>S_a</math> (0.5) (g)</b>	0.072	0.123	0.229
<b><math>S_a</math> (1.0) (g)</b>	0.040	0.064	0.112
<b><math>S_a</math> (2.0) (g)</b>	0.019	0.030	0.050
<b><math>S_a</math> (5.0) (g)</b>	0.0040	0.0066	0.0120
<b><math>S_a</math> (10.0) (g)</b>	0.0016	0.0025	0.0045

## 6.3 Global Stability

The following subsections outline the method and parameters used to evaluate the static global stability of the proposed high fill embankments, and the results of the stability analyses.

### 6.3.1 Method of Analysis

The stability analyses have been completed for a representative critical section based on the highest embankment height encountered within the embankment areas. The embankment geometry is based on the typical cross sections provided by AECOM with a maximum embankment height of approximately 7 m; this represents the maximum height within the south approach to the Glendale Avenue underpass and the north approach to the Airport Road connection structure, as presented in the Foundation Reports for those structures; further from these structures, the high fill embankment heights vary from approximately 4.5 m to 6 m. The embankment side slopes have been analyzed for a maximum (i.e., steepest) side slope orientation of 2 horizontal to 1 vertical (2H:1V). It is assumed for analysis that topsoil and other organic/unsuitable fill material will be removed from the footprint of the embankments during foundation preparation.

Two-dimensional limit equilibrium slope stability analyses were performed using the commercially available program Slide (Version 6.0), developed by Rocscience Inc., employing the Morgenstern-Price method of analysis. Morgenstern-Price is a general method of slices which is based on equilibrium of forces and moments acting on each slice of soil mass above the potential failure surface. For all analyses, the Factors of Safety of numerous potential failure surfaces were computed in order to establish the minimum Factor of Safety. The Factor of Safety is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. For the purpose of the stability analysis, the Factor of Safety is equal to the inverse of the product of the consequence factor,  $\Psi$ , and the geotechnical resistance factor,  $\phi_{gu}$ . (i.e.,  $FoS = 1/(\Psi \cdot \phi_{gu})$ ). Accordingly, minimum Factors of Safety of 1.3 and 1.5 have been used for the design of the embankment slopes for the short-term/temporary and long-term/permanent conditions, respectively, as per Table 6.2 of CHBDC (2014).

### 6.3.2 Soil Parameters

For the cohesive deposits, total stress parameters were employed in the analyses of the short-term, undrained conditions (i.e., temporary conditions). The total stress parameters (i.e., average mobilized undrained shear strength –  $s_u$ ) for the cohesive soils were estimated from the field vanes, as well as from correlations with the SPT results and other laboratory test data (i.e., natural water content, liquid limit, oedometer testing, etc.), where appropriate. A plot of the undrained shear strength versus elevation is shown on Figure 1.

Effective stress parameters were also assigned to the cohesive and non-cohesive deposits to evaluate the stability based on long-term, drained conditions (i.e., permanent conditions). The effective stress parameters (i.e., effective friction angle ( $\phi'$ )) for the cohesive deposits were estimated from empirical correlations based on the plasticity indices. The correlations proposed by Mitchell (1993), Kulhawy and Mayne (1990), and Ladd et al. (1977) were employed and the results were adjusted using engineering judgement based on precedent experience in similar soil conditions. For the non-cohesive soils present at this site, the effective stress parameters employed in the analyses were estimated from empirical correlations based on the results of the corrected Standard Penetration Test (SPT) “N”-values. The correlations proposed by Peck et al (1974) and U.S. Navy (1986) were employed and the results were adjusted by engineering judgment based on precedent experience in similar soil conditions.

The simplified stratigraphy together with the foundation engineering parameters for the different soil types encountered in the proposed high fill embankment areas are summarized below. For the purpose of the stability analyses, the groundwater level was assumed to be at Elevation 109 m, based on the observations in the field investigation program.

Soil Deposit	Bulk Unit Weight, $\gamma$ (kN/m <sup>3</sup> )	Effective Friction Angle, $\phi'$ (°)	Cohesion, $c'$ (kPa)	Undrained Shear Strength, $s_u$ (kPa)
Select subgrade material (SSM) <sup>1</sup> for new embankments	20	32	-	-
Granular fill <sup>1</sup> for new embankments	21	36		
Compact to very dense / stiff to hard existing embankment fill or existing surficial fill	19	30	-	-
Silty clay to clay crust	18.5	27	-	100
Silty clay to clay (firm to stiff zone)	18	25		75
Compact to very dense silt and sand to sand	20	32	-	-
Very stiff to hard clayey silt / clayey silt till	20	32	-	150

Soil Deposit	Bulk Unit Weight, $\gamma$ (kN/m <sup>3</sup> )	Effective Friction Angle, $\phi'$ (°)	Cohesion, $c'$ (kPa)	Undrained Shear Strength, $s_u$ (kPa)
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**Note:**

1. The slope stability figures presented in this report illustrate embankments using select subgrade material (SSM) fill, to minimize the potential for slumping and surficial failures which could occur if embankments are constructed of the intermediate to high plasticity native silty clay to clay deposit.

### 6.3.3 Global Stability Analysis Results

The global stability analyses indicate that after completion of construction, high fill embankments up to 7 m high will have a Factor of Safety of greater than 1.3 in short-term/temporary (undrained) conditions for deep-seated, global failure surfaces of the embankment side slope (2H:1V) that would impact the operation of the roadway. A minimum factor of safety of 1.5 is achieved in the long-term/permanent (effective stress) condition. Figures 2 and 3 illustrate the results of these static global stability analyses.

These analyses and factors of safety assume the embankment is constructed of SSM following subexcavation of topsoil and any surficial organic/deleterious material. Earth fill may be used for construction of the embankments; however, side slopes oriented no steeper than 2.5H:1V would be required to obtain the required factor of safety for global stability, as well as to promote surficial stability.

## 6.4 Settlement

The construction of the realigned Glendale Avenue high fill embankment to the east of the existing embankment, together with the construction of the ramps in proximity to the Glendale Avenue embankment, will result in placement of up to approximately 4.5 m to 7 m of new fill on top of the existing ground surface. In general, lesser quantities of fill are required in the western portion of the realignment of Glendale Avenue, where the new fill will be placed on top of the existing east embankment side slope. The sources of settlement will include the following:

- Compression of the new embankment fill (short-term);
- Immediate settlement of the non-cohesive soils and the very stiff to hard portions of cohesive deposits (short-term);
- Primary time-dependent consolidation of the firm to stiff portions of the cohesive deposits (using Terzaghi's one-dimensional consolidation theory long-term); and
- Secondary time dependent (creep) consolidation of the cohesive deposits (long-term).

The following subsections outline the methods and parameters used to carry out the settlement analyses within the proposed high fill embankment areas. The results of the analyses are presented and recommendations regarding potential design and construction alternatives to mitigate post-construction settlement are provided.

### 6.4.1 Method of Analysis

To estimate the magnitude of expected settlement, analyses were carried out at the critical sections of the proposed high fill areas, corresponding to the greatest embankment heights of approximately 7 m at the south approach to the Glendale Road underpass structure, and at the north approach to the Airport Road connection structure. Settlement analyses have also been completed for the shorter embankment heights (on the order of 4.5 m to 6 m high) to illustrate the range of settlement that will occur within the high fill embankment areas. The thickness of the compressible foundation soils is relatively consistent at the site, while the height of the embankments varies along the existing and proposed Glendale Avenue alignments, and as such the settlements along the length and width of

the new Glendale Avenue embankment will similarly vary. The settlement analyses assume that any topsoil, organics or other deleterious materials have been removed and replaced with SSM or granular fill prior to construction of the high fill embankments.

The settlement analyses were carried out using the commercially available program Settle<sup>3D</sup> (Version 4.0), developed by Rocscience Inc. The stress distribution calculations used in the settlement analyses were based on Westergaard's (1938) solution.

#### 6.4.2 Soil Parameters

The simplified stratigraphy along and across the site is shown on Drawings 1 and 2, and the deformation and time-rate consolidation parameters, where applicable, employed for the soil types encountered at the site are summarized in Table 1 following the text of this report. The parameters associated with the extensive cohesive deposit encountered at the site are presented on Figure 1 and are based on the graphical presentation of this data the upper cohesive deposit was sub-divided into multiple layers based on the pre-consolidation stress (function of OCR), void ratio, compression index/recompression index. For the purpose of the settlement analyses, the groundwater level was assumed to be at Elevation 109, based on the observations during the field investigation.

The immediate compression of the non-cohesive deposits (i.e., silt, sandy silt, silt and sand to silty sand) and the stiffer portions of the cohesive 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). These estimated values were also compared with the typical range of expected values for similar soil types, as outlined in Section C 6.9.3.6 of the *Commentary to the CHBDC (2014)* and adjusted, if necessary.

The consolidation settlement of the firm to stiff portions of the cohesive deposits was assessed using the results of the laboratory consolidation tests, where appropriate, and in situ field vane tests to estimate the stress history and deformation parameters for the cohesive deposits. In addition, the results of the laboratory index tests were employed to further assess deformation parameters (i.e., compression and recompression indices) using empirical correlations proposed in literature by Azzouz et al. (1976), Koppula (1986), Kulhawy and Mayne (1990), Nishida (1956) and Terzaghi and Peck (1967).

The coefficient of consolidation,  $c_v$  (cm<sup>2</sup>/s), required in the time-rate settlement analysis was established using the results of the laboratory consolidation tests and/or estimated from the U.S. Navy (1986) correlation with liquid limit assuming normally consolidated or over-consolidated soils, as applicable.

The following correlation relating in situ undrained shear strength to preconsolidation stress (Mesri, 1975) was employed:

$$\sigma'_p = \frac{s_{u(mob)}}{0.22}$$

where:

$$\begin{aligned} \sigma'_p &= \text{preconsolidation stress (kPa); and,} \\ s_{u(mob)} &= \mu s_{u(FV)} \text{ (after Bjerrum, 1973), where } s_{u(mob)} = \text{average mobilized undrained shear strength (kPa)} \\ s_{u(FV)} &= \text{undrained shear strength from field vane test (kPa)} \\ \mu &= \text{Bjerrum's correction factor based on Plasticity Index} \end{aligned}$$

In addition to primary consolidation within the cohesive deposits, secondary compression may also occur. Secondary compression or creep settlement occurs over a long period of time, after full dissipation of excess pore pressure under a constant stress. The following relationship has been employed for estimating the magnitude of creep settlement over the life of the embankments following the completion of primary settlement:

$$S_c = HC_{\alpha\epsilon} \log\left(\frac{t}{t_{EOP}}\right)$$

where:

$S_c$	=	secondary consolidation (creep) settlement (mm)
$C_{\alpha\epsilon}$	=	modified secondary compression index as estimated from laboratory consolidation tests and correlation by Mesri (1973)
$H$	=	initial thickness of compressible clay deposit (mm)
$t$	=	post-construction period of interest (20 years)
$t_{EOP}$	=	time to reach end of primary consolidation (years)

### 6.4.3 Settlement Performance Criteria

The settlement performance criterion for design of approach embankments within the high fill areas is in accordance with MTO's Guideline "Embankment Settlement Criteria for Design", dated July 2, 2010. In accordance with Table 1.1 in MTO's Guideline, new embankments associated with this QEW interchange are to be designed as follows:

- For King's highways on non-compressible soils, total settlements of 50 mm are permissible during the 20-year pavement design life, and differential settlements are to be less than 200:1.
- For King's highways on compressible soils, total settlements of 100 mm are permissible during the 20-year pavement design life, and differential settlements are to be less than 200:1.

The QEW is classified as a King's highway and the Glendale Avenue ramps are classified as being components of a King's highway. For the QEW/Glendale Avenue interchange, due to the presence of firm to stiff zones of cohesive soils below the site, the "compressible soil" category has been selected in establishing the post-construction settlement criterion.

However, where new embankments approach structural elements (such as bridge abutments), a more stringent settlement criterion associated with such a transition point will apply in accordance with Section 1.2 of MTO's Guideline. In this case, the approach embankments adjacent to the proposed Glendale Avenue underpass and Airport Road connection structure must satisfy a post-construction settlement criterion of 25 mm over a 15-year period following completion of construction. The settlement of the bridge approach embankments is addressed in Golder's Foundation Investigation and Design Reports for these components.

### 6.4.4 Settlement Analysis Results

Based on the results of the settlement analyses (with the topsoil and any organic/deleterious materials subexcavated and replaced with SSM or granular fill), the factored settlement of the foundation soils under the loading imposed by the approximately 4.5 m to 7 m high embankments is summarized in the following table.

Location	Approximate Maximum Embankment Height/New Fill Thickness	Total Settlement (mm)	Immediate Settlement (mm)	Consolidation Settlement (mm)
Glendale Avenue south of Station 9+950, including proximal portions of N/S-E and W-N/S Ramp embankments	4.5 m to 5.5 m	65	30	35
South approach to Glendale underpass structure, including proximal portion of N/S-E Ramp left-turn lanes	6 m to 7 m	85	40	45
North approach to Glendale underpass structure, including proximal portion of E-N/S Ramp left-turn lanes	4 m to 5 m	65	35	30
South approach to Airport Road connection structure, including proximal portion of E-N/S Ramp right-turn lanes	6 m to 7 m	85	30	55
North Approach to Airport Road connection structure, and northward	4.5 m to 5.5 m high	65	30	35

The magnitude of settlement will decrease toward the western portion of the realigned Glendale Avenue embankment, where the new fill will be placed on top of the existing embankment's east side slope. The existing Glendale Avenue embankment is effectively preloading the western portion of the proposed structure/retaining wall footprint, although some grade raise will be required over a portion of this existing fill to match the new Glendale Avenue grade. The post-construction consolidation settlement under the filling and where applicable the grade raise at the western crest of the realigned Glendale Avenue embankment is estimated to be less than 15 mm.

Considering the variability in the coefficient of consolidation ( $c_v$ ) and thickness/drainage path of the consolidating layers, attributed primarily to the varved nature of the clayey silt to silty clay deposit, it is estimated that it will take approximately 30 to 60 days over most portions of the high fill embankments to reach a point beyond which less than 25 mm of primary consolidation settlement remains. However, at the south approach to the Airport Road connection structure, it is estimated that it will take 90 to 120 days to achieve sufficient settlement that less than 25 mm of primary consolidation settlement remains.

The magnitude of secondary consolidation (creep) settlement for the cohesive deposit is estimated to be about 5 mm to 10 mm over a 15-year period following completion of construction of the high fill embankments at this site.

The settlements beyond the immediate approach embankments for the structures (which are addressed in greater detail in the Foundation Reports for the Glendale Avenue underpass and Airport Road connection structures) meet MTO's settlement criteria for freeways and associated components on both compressible and non-compressible soils. Therefore, settlement mitigation measures are not strictly required for the high fill embankments beyond the immediate structure approaches.

However, to reduce the post-construction settlement of the pavements on the high fill embankments, it is recommended that final paving be delayed for a minimum period of 30 to 60 days following construction of the embankments, with the exception of the section of Glendale Avenue between the new underpass and the Airport Road connection structure, where it is recommended that final paving be completed at least 90 days after embankment construction. The embankment construction in this area may consist of preloading in advance of or following the bridge construction, or any combination thereof.

## 6.5 Construction Considerations

### 6.5.1 Removal of Topsoil and Organic Materials

Based on the information from the boreholes advanced during the field investigation, the thickness of organic deposits (mainly topsoil) generally ranges between about 100 mm and 300 mm. Greater thicknesses of organic material may be present particularly along the proposed E-N/S and N/S-E ramps, which will traverse vegetated/wooded areas in the northeast and southeast quadrants of the Glendale Avenue interchange. To improve settlement performance, it is recommended that all surficial layers of topsoil, organic soils, and any other deleterious materials should be stripped from the plan limits of the proposed high fill embankment footprints, regardless of the embankment height, in accordance with OPSS.PROV 206 (*Grading*).

### 6.5.2 Embankment Construction

Placement of Select Subgrade Material (SSM) or granular fill (satisfying OPSS.PROV 1010 (*Aggregates*) Granular 'B' Type I or Type II requirements) is recommended for construction of the new embankments. Earth fill is not recommended in this area as the primary source consists of intermediate to high plasticity silty clay to clay, which may promote surficial instability on embankment side slopes unless flatter slopes are adopted.

Embankment construction should be carried out in accordance with the requirements as outlined in OPSS.PROV 206 (*Grading*), and the fill should be compacted in accordance with OPSS.PROV 501 (*Compacting*). The embankment side slopes should be no steeper than 2H:1V. The embankment side slopes should also include a minimum 2 m wide bench at mid-height for all fill heights greater than 8 m per OPSD 202.010 (*Slope Flattening*), although this is not anticipated to be applicable for the fill heights at this site. In addition, benching of the existing Glendale Avenue side slopes should be carried out to "key in" the new fill materials for the realigned roadway and ramp fills, in accordance with OPSD 208.010 (*Benching of Earth Slopes*).

### 6.5.3 Erosion Protection

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



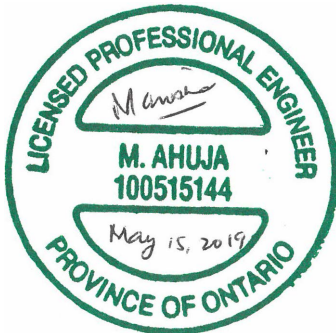
## 6.5.4 Groundwater and Surface Water Control

Excavations within the footprint of the high fill embankments will be required to remove topsoil, organic soils and/or deleterious materials prior to embankment fill placement. It is expected that such excavations will be maintained above the groundwater table. However, if construction operations are carried out during the wet season or periods of heavy or sustained precipitation, some limited groundwater seepage may occur from near-surface seams/layers of more permeable soil, and surface water flow may occur into sub-excavation areas. Dewatering is not expected for the excavation and backfilling along the high fill areas; however, surface water should be directed away from the excavations.

## 7.0 CLOSURE

This Foundation Design Report was prepared by Ms. Manisha Ahuja, P.Eng., P.E., a geotechnical engineer with Golder. Ms. Lisa Coyne, P.Eng., a Principal and MTO Foundations Designated Contact of Golder, conducted an independent technical and quality control review of this report.

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### ASTM International:

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

### Commercial Software:

- Settle3D (Version 4.0) by Rocscience Inc.
- Slide (Version 2018) by Rocscience Inc.

### Ontario Provisional Standard Drawing:

- |              |                          |
|--------------|--------------------------|
| OPSD 202.010 | Slope Flattening         |
| OPSD 208.010 | Benching of Earth Slopes |

### Ontario Provincial Standard Specification:

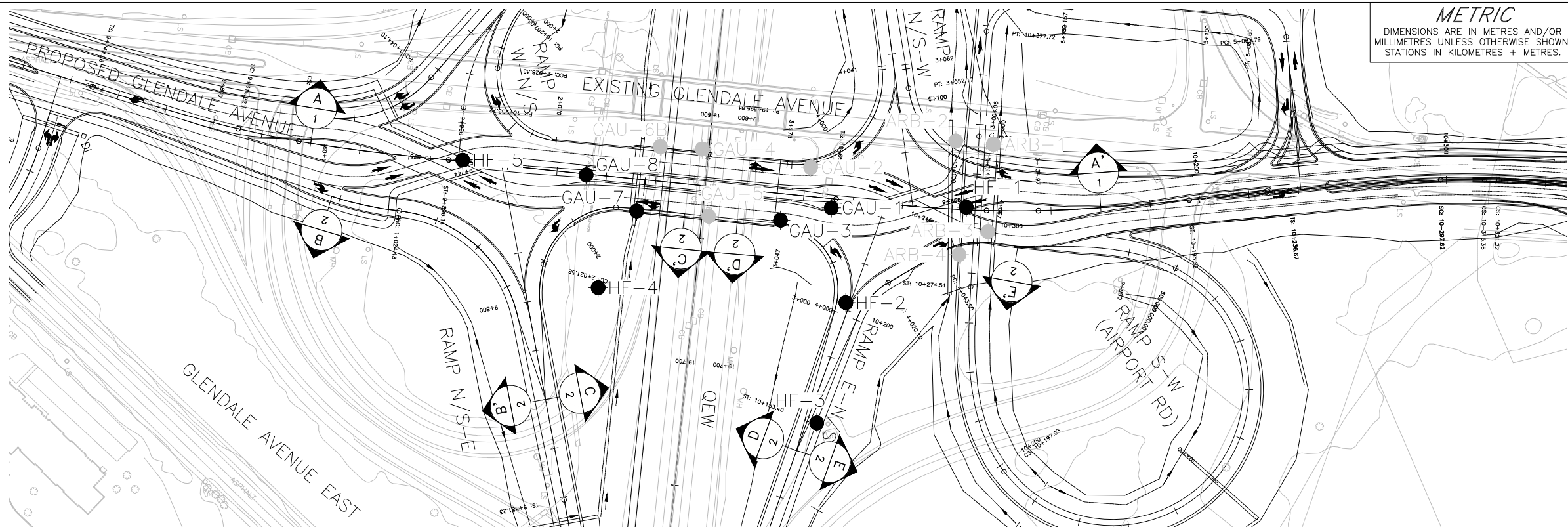
- |                |  |
|----------------|--|
| OPSS.PROV 206  | Construction Specification for Grading   |
| OPSS.PROV 501  | Construction Specification for Compacting  |
| OPSS.PROV 804  | Construction Specification for Seed and Cover  |
| OPSS.PROV 1010 | Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material |

### Ontario Water Resources Act:

- |                        |                    |
|------------------------|--------------------|
| Ontario Regulation 903 | Wells (as amended) |
|------------------------|--------------------|

TABLE 1 – SUMMARY OF FOUNDATION ENGINEERING PARAMETERS – HIGH FILL EMBANKMENTS

High Fill Embankment Area	Stratigraphic Unit	Elevation (m)	Thickness Thickness (m)(m)	$\gamma$	$\phi'$	$c'$	$\sigma_u$	$\sigma_p'$	$e_0$	$C_c$	$C_r$	$m_v$	$E'$ (MPa)	$C_v$
				(kN/m <sup>3</sup> )	(°)	(kPa)	(kPa)	(kPa)				(kPa)		(cm <sup>2</sup> /s)
High Fill Embankments North of QEW including E-N/S Ramp  (Boreholes GAU-1, GAU-3, HF-1, HF-2 and HF-3)	Silty Clay to Clay Fill	121.4 - 115.1	1.3 - 4.9	19	30	0	-	-	-	-	-	-	5 - 25	-
	Sand and Gravel Fill	121.8 - 115.1	0.4 - 3.0	19	32	0	-	-	-	-	-	-	10 - 25	-
	Silty Clay to Clay	116.9 - 109.5	5.6 - 6.4	18.5	27	0	85 - 100	-	0.85	-	-	-	30 - 50	-
	Silty Clay to Clay (Firm to Stiff)	112.5 - 99.7	5.8 - 10.9	18	25	0	75 - 90	300 - 330	0.80 - 0.84	0.3 - 0.45	0.01 - 0.02	1.8 x 10 <sup>-4</sup>	-	1.4 x 10 <sup>-3</sup> - 2.0 x 10 <sup>-3</sup>
	Sandy Silt to Silt and Sand	95.9 - 89.6	6.3	19	30	0	-	-	-	-	-	-	25 - 75	-
	Clayey Silt	115.4 - 95.9	2.6 - 5.0	21	32	0	-	-	-	-	-	-	175	-
	Shale Bedrock	89.4 - 85.3	4.1	22	35	0	-	-	-	-	-	-	2000	-
High Fill Embankments South of QEW including N/S-E Ramp  (Boreholes GAU-7, GAU-8, HF-4 and HF-5)	Silty Clay to Clay Fill	117.5 - 113.4	0.8 - 1.5	19	30	0	-	-	-	-	-	-	5 - 25	-
	Silty Clay to Clay	116.7 - 109.5	2.6 - 7.7	18.5	27	0	85 - 95	-	0.85	-	-	-	30 - 50	-
	Silty Clay to Clay (Firm to Stiff)	111.5 - 100.3	9.2 - 13.3	18	25	0	75 - 90	325 - 400	0.68 - 0.84	0.25 - 0.35	0.02 - 0.04	1.4 x 10 <sup>-4</sup>	-	2.5 x 10 <sup>-3</sup> - 3.5 x 10 <sup>-3</sup>
	Clayey Silt to Clayey Silt Till	113.4 - 96.2	2.4 - 4.2	19	30	0	-	-	-	-	-	-	25 - 75	-
	Silt and Sand to Silt	96.2 - 85.7	10.5	21	32	0	-	-	-	-	-	-	175	-
	Shale Bedrock	85.7 - 82	3.7	22	35	0	-	-	-	-	-	-	2000	-

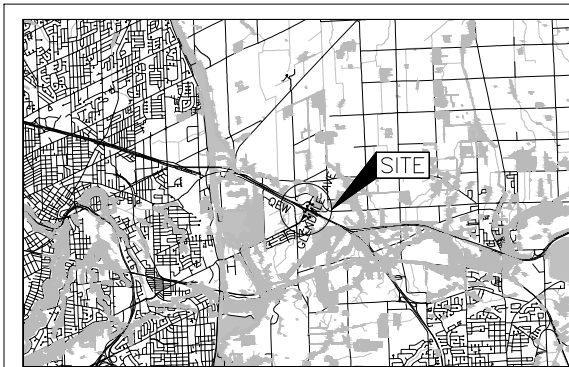


**METRIC**  
DIMENSIONS ARE IN METRES AND/OR  
MILLIMETRES UNLESS OTHERWISE SHOWN.  
STATIONS IN KILOMETRES + METRES.

CONT No.  
GWP No. 2423-15-00

QEW/GLENDALE AVENUE INTERCHANGE IMPROVEMENTS  
HIGH FILL EMBANKMENTS  
BOREHOLE LOCATION AND SOIL  
STRATA

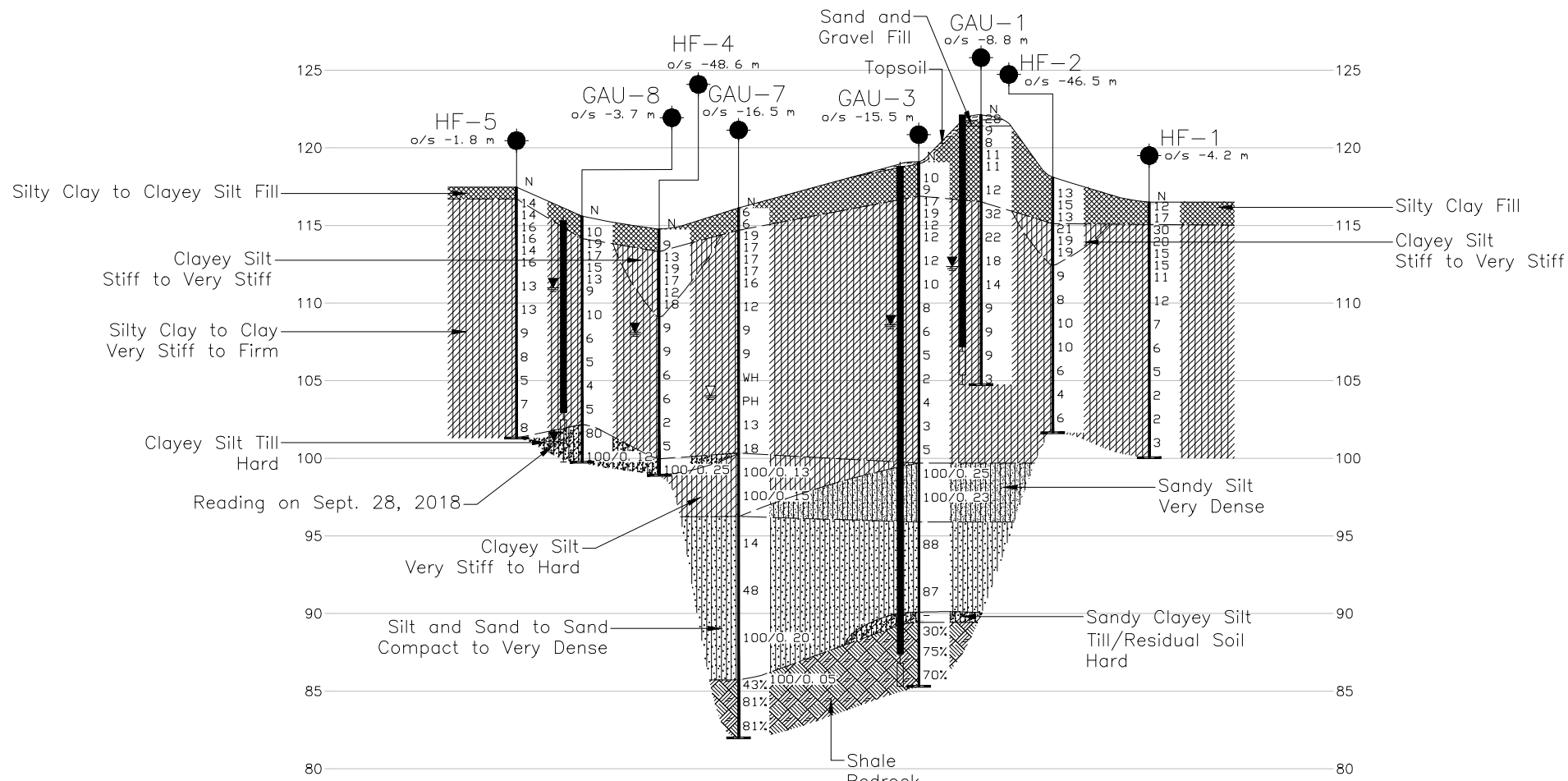
SHEET



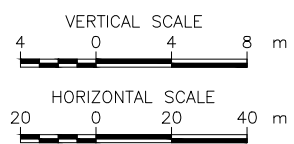
KEY PLAN  
SCALE  
2 0 2 4 km

LEGEND

- Borehole – Current Investigation
- ⬮ Seal
- ⬮ Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- ≡ WL in piezometer, measured on May 7, 2019
- ≡ WL upon completion of drilling



PROFILE A-A'  
GLENDALE AVENUE



BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
GAU-1	122.1	4779785.4	332141.3
GAU-3	119.1	4779764.4	332138.9
GAU-7	116.1	4779711.3	332115.4
GAU-8	115.6	4779697.2	332094.7
HF-1	116.5	4779836.5	332159.9
HF-2	118.1	4779777.7	332179.2
HF-3	117.6	4779749.9	332220.6
HF-4	114.8	4779686.0	332139.0
HF-5	117.5	4779652.6	332071.9

NOTES

This drawing is for subsurface information only. The proposed are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Preliminary Design Report.

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

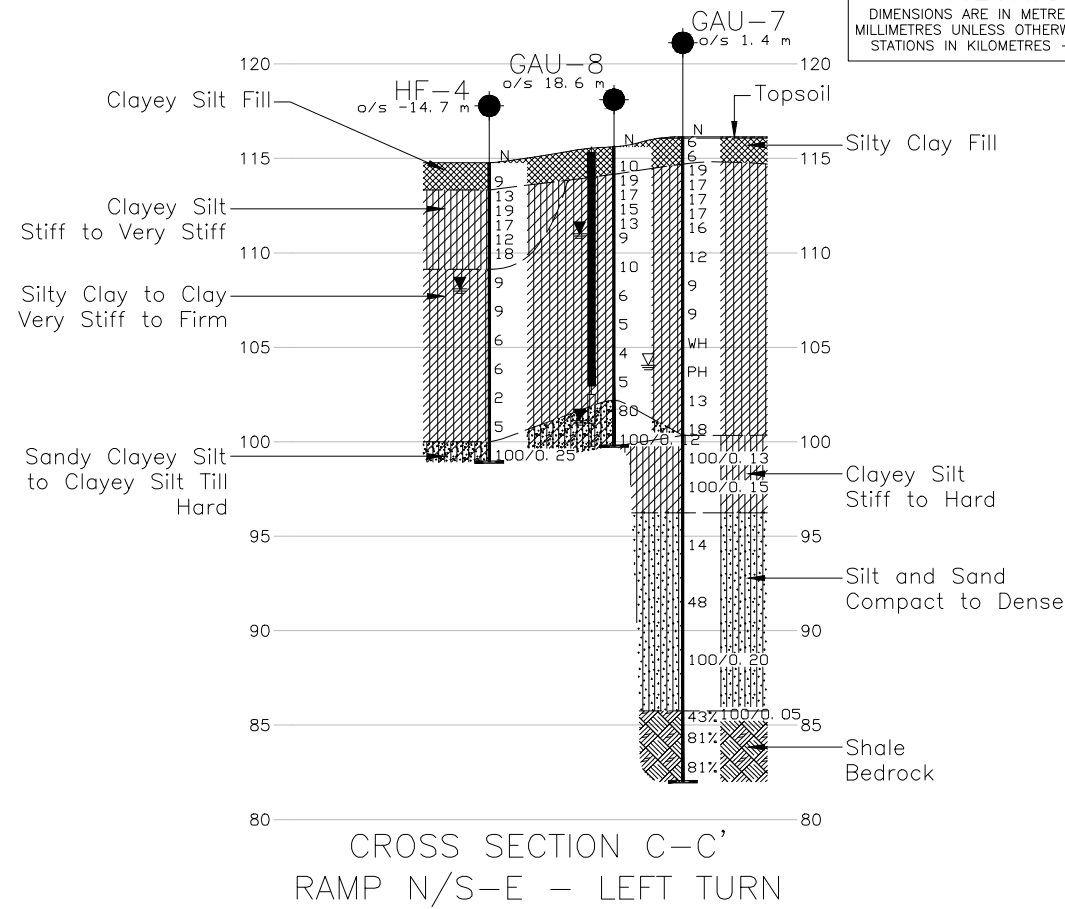
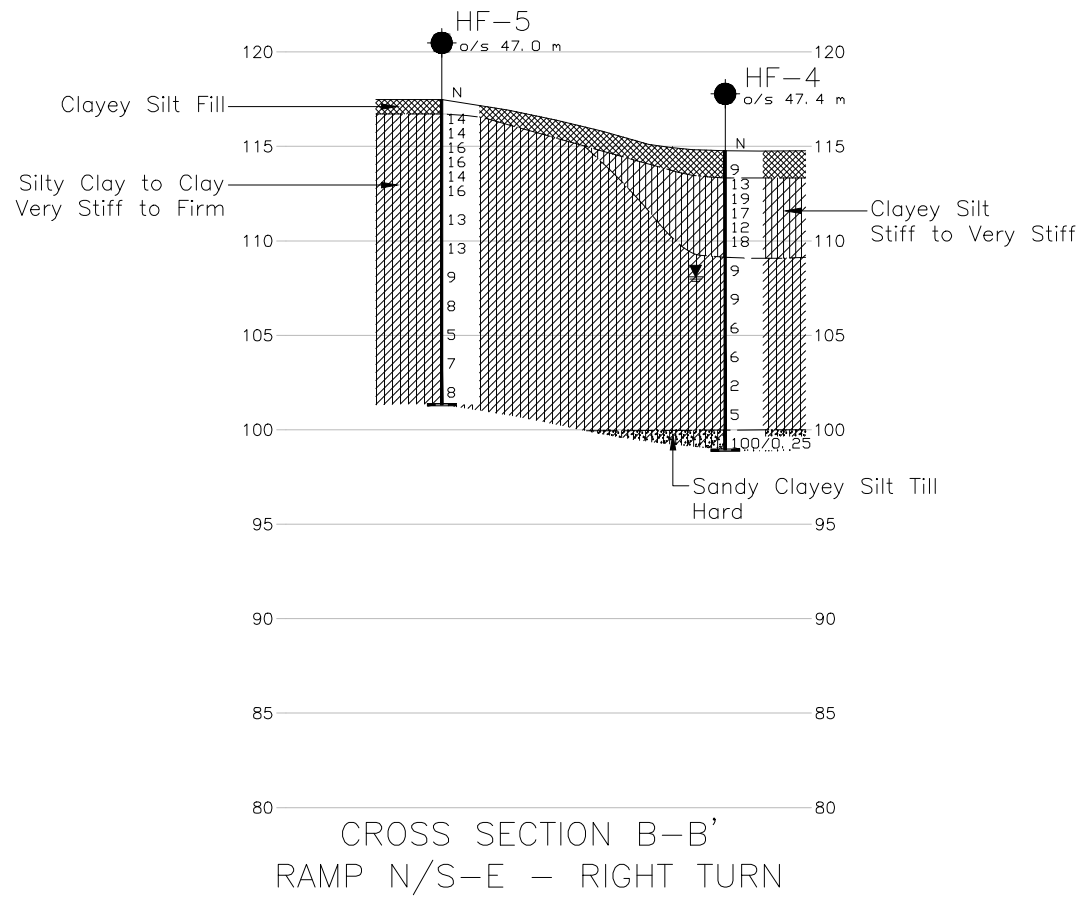
REFERENCE

Base plans provided in digital format by Aecom, drawing file nos. X\_Base.dwg, X\_Property.dwg, York Roundabout\_1 Lane.dwg, Diverging Diamond.dwg and Diverging Diamond with Airport Rd connection.dwg, received October 23, 2018.



NO.	DATE	BY	REVISION
Geocres No. 30M3-311			
HWY. QEW	PROJECT NO. 1671430		DIST. .
SUBM'D. NK	CHKD. MA	DATE: 4/30/2019	SITE: .
DRAWN: DD	CHKD. MA	APPD. LCC	DWG. 1



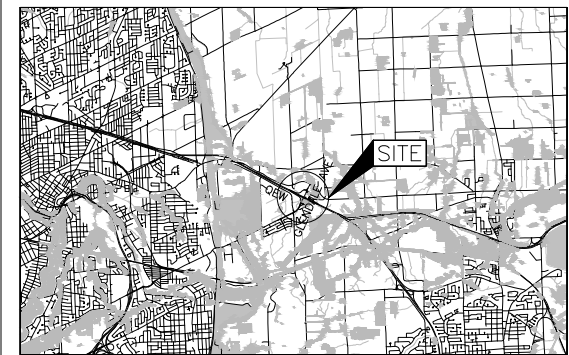


**METRIC**  
DIMENSIONS ARE IN METRES AND/OR  
MILLIMETRES UNLESS OTHERWISE SHOWN.  
STATIONS IN KILOMETRES + METRES.

CONT No.  
GWP No. 2423-15-00

QEW/GLENDALE AVENUE INTERCHANGE IMPROVEMENTS  
HIGH FILL EMBANKMENTS

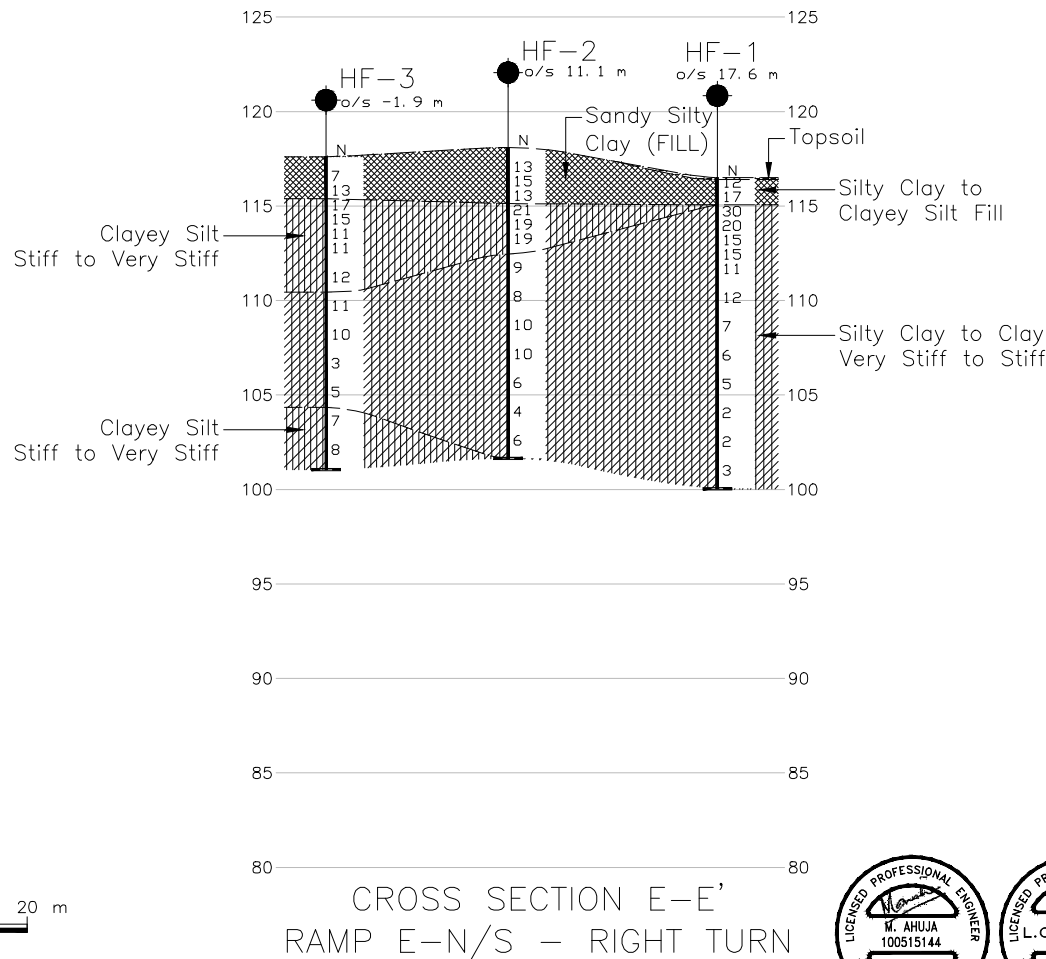
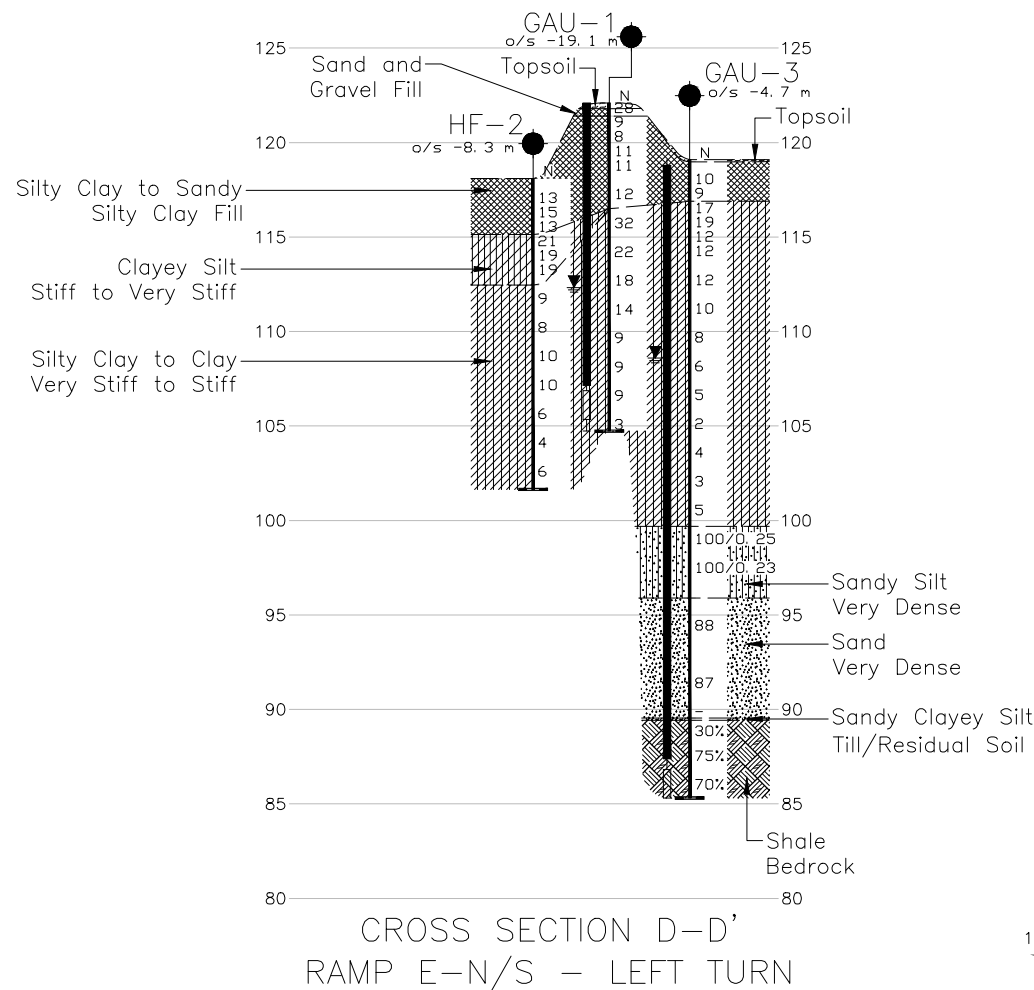
SHEET



KEY PLAN  
SCALE  
0 2 4 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)
- 100% Rock Quality Designation (RQD)
- ≡ WL in piezometer, measured on May 7, 2019
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BOREHOLE CO-ORDINATES			
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HF-3	117.6	4779749.9	332220.6
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HF-5	117.5	4779652.6	332071.9

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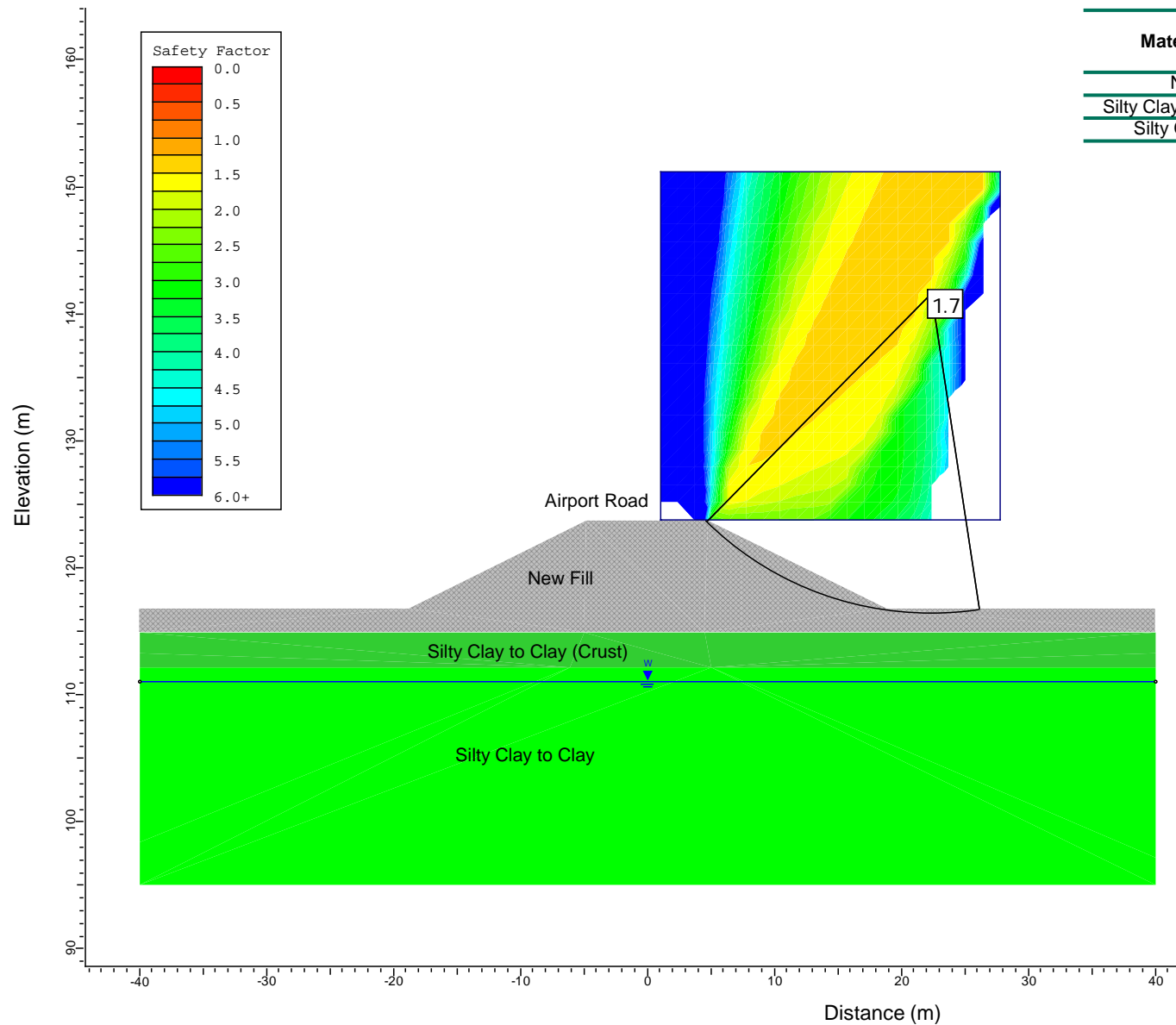
NO.	DATE	BY	REVISION
1			
Geocres No. 30M3-311			
HWY. QEW	PROJECT NO. 1671430	DIST.	
SUBM'D. NK	CHKD. MA	DATE: 2019-05-10	SITE:
DRAWN: SW	CHKD. MA	APPD. LCC	DWG. 2





# QEW – Glendale Avenue Interchange Improvements - High Fill Embankments Static Global Stability Analysis – Short-Term (Undrained) Condition

Figure 2

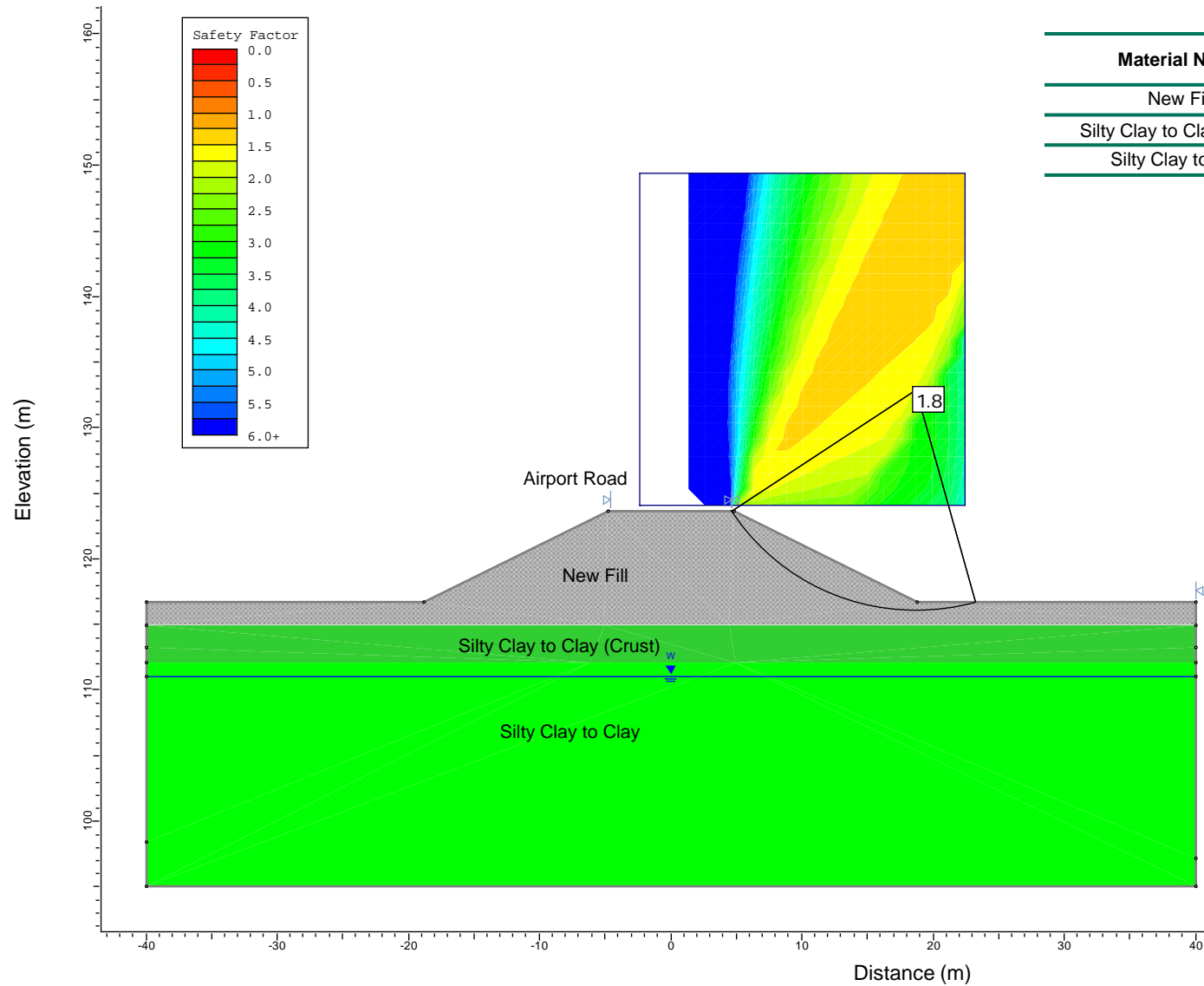


Material Name	$\gamma$ ( $\text{kN/m}^3$ )	$s_u$ (kPa)	$\phi'$ (degrees)
New Fill	21	-	35
Silty Clay to Clay (Crust)	18.5	100	-
Silty Clay to Clay	18	75	-



# QEW – Glendale Avenue Interchange Improvements - High Fill Embankments Static Global Stability Analysis– Long-Term (Effective Stress) Condition

Figure 3



Material Name	$\gamma$ (kN/m <sup>3</sup> )	$s_u$ (kPa)	$\phi'$ (degrees)
New Fill	21	-	35
Silty Clay to Clay (Crust)	18.5	-	27
Silty Clay to Clay	18	-	25

**APPENDIX A**

**Borehole Records from 2018  
Investigation**



## LIST OF SYMBOLS

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

### I. GENERAL

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

### II. STRESS AND STRAIN

$\gamma$	shear strain
$\Delta$	change in, e.g. in stress: $\Delta \sigma$
$\varepsilon$	linear strain
$\varepsilon_v$	volumetric strain
$\eta$	coefficient of viscosity
$\nu$	Poisson's ratio
$\sigma$	total stress
$\sigma'$	effective stress ( $\sigma' = \sigma - u$ )
$\sigma'_{vo}$	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
$\sigma_{oct}$	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
$\tau$	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

### III. SOIL PROPERTIES

#### (a) Index Properties

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

#### (a) Index Properties (continued)

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

#### (b) Hydraulic Properties

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

#### (c) Consolidation (one-dimensional)

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

#### (d) Shear Strength

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

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

Notes: 1  
2

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

## LIST OF ABBREVIATIONS

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

### I. SAMPLE TYPE

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

### II. PENETRATION RESISTANCE

#### Standard Penetration Resistance (SPT), N:

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

#### Dynamic Cone Penetration Resistance; $N_d$ :

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

**PH:** Sampler advanced by hydraulic pressure

**PM:** Sampler advanced by manual pressure

**WH:** Sampler advanced by static weight of hammer

**WR:** Sampler advanced by weight of sampler and rod

#### Piezo-Cone Penetration Test (CPT)

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

### III. SOIL DESCRIPTION

#### (a) Non-Cohesive (Cohesionless) Soils

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

#### (b) Cohesive Soils Consistency

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

### IV. SOIL TESTS

w	water content
$w_p$	plastic limit
$w_l$	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test <sup>1</sup>
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement <sup>1</sup>
$D_R$	relative density (specific gravity, $G_s$ )
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO <sub>4</sub>	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
$\gamma$	unit weight

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

### V. MINOR SOIL CONSTITUENTS

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

## WEATHERINGS STATE

**Fresh:** no visible sign of weathering

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

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

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

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

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

## BEDDING THICKNESS

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

## JOINT OR FOLIATION SPACING

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

## GRAIN SIZE

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

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

## CORE CONDITION

### Total Core Recovery (TCR)

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

### Solid Core Recovery (SCR)

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

### Rock Quality Designation (RQD)

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

## DISCONTINUITY DATA

### Fracture Index

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

### Dip with Respect to Core Axis

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

### Description and Notes

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

### Abbreviations

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



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

PROJECT		RECORD OF BOREHOLE				No HF-1		SHEET 2 OF 2		METRIC						
G.W.P. 1671430 WO2		LOCATION		N 4779836.5; E 332159.9 MTM NAD 83 ZONE 10 (LAT. 43.157890; LONG. -79.163572)				ORIGINATED BY KN								
DIST Central HWY QEW		BOREHOLE TYPE		152 mm Hollow Stem Augers; CME 55 Track-mounted Drill Rig				COMPILED BY KG								
DATUM Geodetic		DATE		September 20, 2014				CHECKED BY LCC								
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa								
	--- CONTINUED FROM PREVIOUS PAGE ---															
100.0	SILTY CLAY, trace sand, trace gravel Hard to stiff Brown to grey below 9.1 m Moist to wet below 11.7 m		14	SS	3		101									
16.5	END OF BOREHOLE															
	NOTES:  1. Open borehole dry upon completion of drilling.  2. Borehole open to 15.2 m below ground surface on removal of augers.															

GTA-MTO 001 S:\CLIENTS\MTQEW-LENDALE\02\_DATA\INTQEW-LENDALE.GPJ GAL-GTA.GDT 19-4-30

PROJECT <u>1671430 WO2</u>		<b>RECORD OF BOREHOLE No HF-2</b>		SHEET 1 OF 2		<b>METRIC</b>	
G.W.P. <u>2423-15-00</u>		LOCATION <u>N 4779777.7; E 332179.2 MTM NAD 83 ZONE 10 (LAT. 43.157359; LONG. -79.163337)</u>		ORIGINATED BY <u>SE</u>			
DIST <u>Central</u> HWY <u>QEW</u>		BOREHOLE TYPE <u>152 mm Hollow Stem Augers; CME 55 Track-mounted Drill Rig</u>		COMPILED BY <u>KG</u>			
DATUM <u>Geodetic</u>		DATE <u>September 17, 2018</u>		CHECKED BY <u>LCC</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT   NATURAL MOISTURE   LIQUID LIMIT			UNIT WEIGHT  γ  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)		
								20	40	60	80	100			W <sub>p</sub>	W	W <sub>L</sub>
118.1	GROUND SURFACE						118										
0.0	Sandy silty clay, trace to some gravel, trace wood fragments (FILL) Stiff Brown Moist		1	SS	13		117										
			2	SS	15		116										
			3	SS	13		115										
115.1	CLAYEY SILT, trace sand, trace gravel, trace rootlets Very stiff Brown Moist		4	SS	21		114										
3.0			5	SS	19		113										
			6	SS	19		112										
112.5	SILTY CLAY, trace gravel, trace sand Very stiff to stiff Brown to grey below 10.2 m Moist to wet below 11.7 m		7	SS	9		111										
5.6			8	SS	8		110										
			9	SS	10		109										
			10	SS	10		108										
			11	SS	6		107										
			12	SS	4		106										
							105										
							104										

Continued Next Page

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

GTA-MTO 001 S:\CLIENTS\MTQEW-LENDALE\02\_DATA\INTQEW-LENDALE.GPJ GAL-GTA.GDT 19-4-30



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

GTA-MTO 001 S:\CLIENTS\MTO\QEW-GLENDALE\02 DATA\GINT\QEW-GLENDALE.GPJ GAL-GTA.GDT 19-4-30

<b>PROJECT</b> 1671430 WO2		<b>RECORD OF BOREHOLE No HF-3</b>		SHEET 1 OF 2		<b>METRIC</b>	
<b>G.W.P.</b> 2423-15-00		<b>LOCATION</b> N 4779749.9; E 332220.6 MTM NAD 83 ZONE 10 (LAT. 43.157108; LONG. -79.162829)		<b>ORIGINATED BY</b> SE			
<b>DIST</b> Central <b>HWY</b> QEW		<b>BOREHOLE TYPE</b> 152 mm Hollow Stem Augers; CME 55 Track-mounted Drill Rig		<b>COMPILED BY</b> KG			
<b>DATUM</b> Geodetic		<b>DATE</b> September 24, 2018		<b>CHECKED BY</b> LCC			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT  γ  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		w <sub>p</sub>	w	w <sub>L</sub>		
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × REMOULDED					
117.6	GROUND SURFACE							20 40 60 80 100						GR SA SI CL
0.0	Clayey silt, some sand to sandy, trace to some gravel, trace to some rootlets (FILL) Firm to stiff Brown Moist		1	SS	7		117							
			2	SS	13		116							
115.4														
2.2	CLAYEY SILT, trace to some sand, trace gravel, contains trace sand seams, trace rootlets Very stiff to stiff Brown to mottled grey below 6.1 m Moist		3	SS	17		115							
			4	SS	15		114							
			5	SS	11		113							
			6	SS	11		112							
							111							
			7	SS	12		110							
110.4							109							
7.2	SILTY CLAY TO CLAY, trace sand Very stiff Grey Moist to wet below 11.7 m		8	SS	11		108							
							107							
			9	SS	10		106							
							105							
			10	SS	3		104							
							103							
			11	SS	5									
104.3														
13.3	CLAYEY SILT, trace sand Stiff to very stiff Grey Wet		12	SS	7									

Continued Next Page

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

GTA-MTO 001 S:\CLIENTS\MTQ\QEW-GLENDALE\02\_DATA\GINT\QEW-GLENDALE.GPJ GAL-GTA.GDT 19-4-30



PROJECT <u>1671430 WO2</u>			<b>RECORD OF BOREHOLE No HF-3</b>			SHEET 2 OF 2			<b>METRIC</b>		
G.W.P. <u>2423-15-00</u>			LOCATION <u>N 4779749.9; E 332220.6 MTM NAD 83 ZONE 10 (LAT. 43.157108; LONG. -79.162829)</u>			ORIGINATED BY <u>SE</u>					
DIST <u>Central</u> HWY <u>QEW</u>			BOREHOLE TYPE <u>152 mm Hollow Stem Augers; CME 55 Track-mounted Drill Rig</u>			COMPILED BY <u>KG</u>					
DATUM <u>Geodetic</u>			DATE <u>September 24, 2018</u>			CHECKED BY <u>LCC</u>					

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
								20	40	60	80	100	W <sub>p</sub>	W	W <sub>L</sub>		
101.1	CLAYEY SILT, trace sand Stiff to very stiff Grey Wet		13	SS	8												
16.6	END OF BOREHOLE  NOTES:  1. Open borehole dry on completion of drilling.																

GTA-MTO 001 S:\CLIENTS\MTQ\QEW-GLENDALE\02\_DATA\GINT\QEW-GLENDALE.GPJ GAL-GTA.GDT 19-4-30


<b>PROJECT</b> 1671430 WO2		<b>RECORD OF BOREHOLE No HF-4</b>		SHEET 1 OF 2		<b>METRIC</b>	
<b>G.W.P.</b> 2423-15-00		<b>LOCATION</b> N 4779686.0; E 332139.0 MTM NAD 83 ZONE 10 (LAT. 43.156536; LONG. -79.163836)		<b>ORIGINATED BY</b> SE			
<b>DIST</b> Central <b>HWY</b> QEW		<b>BOREHOLE TYPE</b> 152 mm Hollow Stem Augers; CME 55 Track-mounted Drill Rig		<b>COMPILED BY</b> KG			
<b>DATUM</b> Geodetic		<b>DATE</b> September 27, 2018		<b>CHECKED BY</b> LCC			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT  <b>γ</b>  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				GR	SA	SI	CL
								○ UNCONFINED   + FIELD VANE	● QUICK TRIAXIAL   × REMOULDED	W <sub>P</sub>	W	W <sub>L</sub>					
114.8	GROUND SURFACE																
0.0	Clayey silt, trace to some sand, trace gravel, trace rootlets (FILL) Stiff Brown Moist		1	SS	9												
113.4																	
1.4	CLAYEY SILT, trace to some sand, trace to some gravel Stiff to very stiff Brown to mottled grey below 3.0 m Moist		2	SS	13										2	8	60 30
			3	SS	19												
			4	SS	17												
			5	SS	12												
			6	SS	18												
109.2																	
5.6	SILTY CLAY to CLAY, trace sand, trace gravel Very stiff to stiff Brown-grey to grey below 10.2 m Moist to wet below 10.2 m		7	SS	9										0	2	33 65
			8	SS	9												
			9	SS	6										1	3	27 69
			10	SS	6												
			11	SS	2										0	1	42 57
			12	SS	5												
100.0																	
14.8																	

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

GTA-MTO 001 S:\CLIENTS\MTQEW-GLENDALE\02\_DATA\INTQEW-GLENDALE.GPJ GAL-GTA.GDT 19-4-30

PROJECT		1671430 WO2				RECORD OF BOREHOLE No HF-4				SHEET 2 OF 2				METRIC					
G.W.P.		2423-15-00		LOCATION		N 4779686.0; E 332139.0 MTM NAD 83 ZONE 10 (LAT. 43.156536; LONG. -79.163836)				ORIGINATED BY				SE					
DIST		Central		HWY		QEW		BOREHOLE TYPE		152 mm Hollow Stem Augers; CME 55 Track-mounted Drill Rig				COMPILED BY		KG			
DATUM		Geodetic		DATE		September 27, 2018				CHECKED BY		LCC							
SOIL PROFILE		SAMPLES				GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)						
	--- CONTINUED FROM PREVIOUS PAGE ---																		
99.0 15.9	Sandy CLAYEY SILT, trace to some gravel, contains trace shale fragments (TILL) Hard Red Moist END OF BOREHOLE  NOTES: 1. Borehole caved to 15.0 m on removal of augers 2. Water level at a depth of 14.9 m (Elev. 99.9 m) on removal of augers.		13	SS	100/0.25		99										9 21 56 14		

GTA-MTO 001 S:\CLIENTS\MTQEW-LENDALE\02\_DATA\INTQEW-LENDALE.GPJ GAL-GTA.GDT 19-4-30



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

PROJECT 1671430 WO2			RECORD OF BOREHOLE No HF-5			SHEET 2 OF 2			METRIC								
G.W.P. 2423-15-00			LOCATION N 4779652.6; E 332071.9 MTM NAD 83 ZONE 10 (LAT. 43.156237; LONG. -79.164663)			ORIGINATED BY SE											
DIST Central HWY QEW			BOREHOLE TYPE 152 mm Hollow Stem Augers; CME 55 Track-mounted Drill Rig			COMPILED BY KG											
DATUM Geodetic			DATE September 28, 2018			CHECKED BY LCC											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
	--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100					
101.4	SILTY CLAY to CLAY, trace sand, trace gravel, trace rootlets to a depth of 1.3 m Very stiff to stiff Brown to grey below 8.7 m Moist to wet below 9.4 m		13	SS	8		102										
16.2	END OF BOREHOLE																
	NOTES:  1. Water level in open borehole at depth of 14.5 m (Elev. 103.0 m) on completion of drilling and prior to removal of augers.  2. Borehole caved to 14.7 m on removal of augers.																

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PROJECT		1671430 WO2		RECORD OF BOREHOLE No GAU-1		SHEET 1 OF 2		METRIC																
G.W.P.		2423-15-00		LOCATION		N 4779785.4; E 332141.3 MTM NAD 83 ZONE 10 (LAT. 43.157430; LONG. -79.163803)		ORIGINATED BY																
DIST		Central HWY QEW		BOREHOLE TYPE		203 mm O.D. Hollow Stem Augers; CME 55 Track Mounted Drill Rig		COMPILED BY																
DATUM		Geodetic		DATE		August 14, 2018		CHECKED BY																
								LCC																
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS			ELEVATION SCALE			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES																		
122.1	0.0	GROUND SURFACE																						
121.8	0.3	TOPSOIL (305 mm)		1A	SS	28																		
121.4	0.7	Sand and gravel (FILL) Compact Grey to red Moist		1B																				
		Silty clay, trace sand (FILL) Stiff to very stiff Brown to grey-brown below 4.6 m Moist		2	SS	9																		
				3	SS	8																		
				4	SS	11																		
				5	SS	11																		
				6	SS	12																		
		- Hydrocarbon odour noted at 5.2 m																						
116.5	5.6	SILTY CLAY to CLAY, trace sand Very stiff to hard, becoming firm to stiff with depth Grey-brown to grey below 8.7 m Moist		7	SS	32																		
		- Pockets of clay below 7.6 m		8	SS	22																		
				9	SS	18																		
				10	SS	14																		
				11	SS	9																		
				12	SS	9																		

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

GTA-MTO 001 S:\CLIENTS\MTQEW-LENDALE\02\_DATA\INTQEW-LENDALE.GPJ GAL-GTA.GDT 19-5-10

PROJECT		RECORD OF BOREHOLE				No GAU-1		SHEET 2 OF 2		METRIC															
G.W.P. 1671430 WO2		LOCATION		N 4779785.4; E 332141.3 MTM NAD 83 ZONE 10 (LAT. 43.157430; LONG. -79.163803)				ORIGINATED BY		AMM															
DIST Central HWY QEW		BOREHOLE TYPE		203 mm O.D. Hollow Stem Augers; CME 55 Track Mounted Drill Rig				COMPILED BY		EN															
DATUM Geodetic		DATE		August 14, 2018				CHECKED BY		LCC															
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)									
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa									WATER CONTENT (%)								
	--- CONTINUED FROM PREVIOUS PAGE ---						20	40	60	80	100														
							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED																		
							20	40	60	80	100	10	20	30											
104.7	SILTY CLAY to CLAY, trace sand Very stiff to hard, becoming firm to stiff with depth Grey-brown to grey below 8.7 m Moist		13	SS	9										42	0 2 35 63									
106																									
105			14	SS	3																				
17.4	END OF BOREHOLE																								
	NOTE:  1. Open borehole dry upon completion of drilling and well installation.  2. Water level measured in piezometer as follows:  <table style="margin-left: 40px;"> <tr> <td>Date (mm/dd/yy)</td> <td>Depth (mbgs)</td> <td>Elevation (m)</td> </tr> <tr> <td>08/14/18</td> <td>16.7</td> <td>105.4</td> </tr> <tr> <td>05/07/19</td> <td>9.7</td> <td>112.3</td> </tr> </table>	Date (mm/dd/yy)	Depth (mbgs)	Elevation (m)	08/14/18	16.7	105.4	05/07/19	9.7	112.3															
Date (mm/dd/yy)	Depth (mbgs)	Elevation (m)																							
08/14/18	16.7	105.4																							
05/07/19	9.7	112.3																							

GTA-MTO 001 S:\CLIENTS\MTQEW-LENDALE\02\_DATA\INTQEW-LENDALE.GPJ GAL-GTA.GDT 19-5-10

<b>PROJECT</b> 1671430 WO2		<b>RECORD OF BOREHOLE No GAU-3</b>		SHEET 1 OF 4		<b>METRIC</b>	
<b>G.W.P.</b> 2423-15-00		<b>LOCATION</b> N 4779764.4; E 332138.9 MTM NAD 83 ZONE 10 (LAT. 43.157241; LONG. -79.163833)		<b>ORIGINATED BY</b> SE			
<b>DIST</b> Central <b>HWY</b> QEW		<b>BOREHOLE TYPE</b> 203 mm O.D. Hollow Stem Augers; HQ Coring; CME 55 Track Mounted Drill Rig		<b>COMPILED BY</b> EN			
<b>DATUM</b> Geodetic		<b>DATE</b> September 17, 18 and 19, 2018		<b>CHECKED BY</b> LCC			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT  <b>γ</b>  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		W <sub>P</sub>	W	W <sub>L</sub>	GR	SA	SI	CL
								SHEAR STRENGTH kPa		WATER CONTENT (%)										
								○ UNCONFINED + FIELD VANE												
								● QUICK TRIAXIAL × REMOULDED												
119.1	GROUND SURFACE																			
0.0	TOPSOIL (102 mm)																			
	Silty clay, trace sand, trace to some gravel, contains rootlets and wood fragments to 2.1 m (FILL)																			
	Stiff																			
	Brown																			
	Moist																			
116.9			1	SS	10															
2.2			2	SS	9															
	SILTY CLAY to CLAY, trace sand																			
	Very stiff, becoming stiff with depth																			
	Brown to grey below 10.2 m																			
	Wet																			
			3	SS	17															
			4	SS	19															
			5	SS	12															
			6	SS	12															
			7	SS	12															
			8	SS	10															
			9	SS	8															
			10	SS	6															
			11	SS	5															
			12	SS	2															
												</								

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

GTA-MTO 001 S:\CLIENTS\MTQEW-GLENDALE\02\_DATA\INTQEW-GLENDALE.GPJ GAL-GTA.GDT 19-5-10






GTA-MTO 001 S:\CLIENTS\MTO\QEW-GLENDALE\02 DATA\GINT\QEW-GLENDALE.GPJ GAL-GTA.GDT 19-5-10

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

PROJECT		RECORD OF BOREHOLE				No GAU-3		SHEET 3 OF 4		METRIC																		
G.W.P. 2423-15-00		LOCATION N 4779764.4; E 332138.9 MTM NAD 83 ZONE 10 (LAT. 43.157241; LONG. -79.163833)				ORIGINATED BY SE																						
DIST Central HWY QEW		BOREHOLE TYPE 203 mm O.D. Hollow Stem Augers; HQ Coring; CME 55 Track Mounted Drill Rig				COMPILED BY EN																						
DATUM Geodetic		DATE September 17, 18 and 19, 2018				CHECKED BY LCC																						
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)												
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa									WATER CONTENT (%)											
	--- CONTINUED FROM PREVIOUS PAGE ---						20	40	60	80	100																	
85.3	Sandy CLAYEY SILT, some gravel, some shale fragments (RESIDUAL SOIL) Red Wet SHALE (BEDROCK)  Bedrock cored from 29.7 m to 33.8 m.  For rock coring details refer to Record of Drillhole GAU-3.		1	RC	REC 100%												RQD = 30%											
			2	RC	REC 100%													RQD = 75%										
			3	RC	REC 100%													RQD = 70%										
33.8			END OF BOREHOLE  NOTES: 1. Open borehole dry prior to installation of casing at 4.6 m.  2. Water level in standpipe piezometer at a depth of 10.2 m (Elev. 108.9 m) on completion of installation.  3. Water level in standpipe piezometer measured as follows: <table border="1" style="margin: 10px auto; border-collapse: collapse;"> <thead> <tr> <th>Date (mm/dd/yy)</th> <th>Depth (mbgs)</th> <th>Elevation (m)</th> </tr> </thead> <tbody> <tr> <td>09/28/18</td> <td>10.5</td> <td>108.6</td> </tr> <tr> <td>11/26/18</td> <td>10.4</td> <td>108.7</td> </tr> <tr> <td>05/07/19</td> <td>10.4</td> <td>108.7</td> </tr> </tbody> </table>																Date (mm/dd/yy)	Depth (mbgs)	Elevation (m)	09/28/18	10.5	108.6	11/26/18	10.4	108.7	05/07/19
Date (mm/dd/yy)	Depth (mbgs)	Elevation (m)																										
09/28/18	10.5	108.6																										
11/26/18	10.4	108.7																										
05/07/19	10.4	108.7																										

GTA-MTO 001 S:\CLIENTS\MTQEW-LENDALE\02\_DATA\INTQEW-LENDALE.GPJ GAL-GTA.GDT 19-5-10

PROJECT: 1671430 W02

**RECORD OF DRILLHOLE: GAU-3**

SHEET 4 OF 4

LOCATION: N 4779764.40 ;E 332138.90

DRILLING DATE: September 19, 2018

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME55 Track-Mount

DRILLING CONTRACTOR: Geo-Environmental

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY												FEATURES	PIEZOMETER																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											
						FLUSH RETURN	RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.25 m CORE AXIS	DIP w.r.t. CORE AXIS	DISCONTINUITY DATA			WEATH- ERING INDEX	Diametral Point Load Index (MPa)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																														
							TOTAL CORE %	SOLID CORE %				TYPE AND SURFACE DESCRIPTION	Jr	Ja			Jzon																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													
		Continued from Record of Borehole GAU-3		89.43																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																										</

PROJECT		1671430 WO2		RECORD OF BOREHOLE No GAU-7				SHEET 1 OF 4		METRIC					
G.W.P.		2423-15-00		LOCATION				N 4779711.3; E 332115.4 MTM NAD 83 ZONE 10 (LAT. 43.156764; LONG. -79.164125)		ORIGINATED BY		KN			
DIST		Central HWY QEW		BOREHOLE TYPE				203 mm O.D. Hollow Stem Augers; HQ Coring; CME 55 Track Mounted Drill Rig		COMPILED BY		EN			
DATUM		Geodetic		DATE				July 23,24, 25, 26, 29 and 30, 2018		CHECKED BY		LCC			
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)		
								20 40 60 80 100							
							20 40 60 80 100					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			
							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W <sub>P</sub> W W <sub>L</sub>			
							20 40 60 80 100					10 20 30			
116.1	GROUND SURFACE														
0.7	TOPSOIL (76 mm)		1A		6										
	Silty clay, trace to some organics, trace gravel (FILL)		1B	SS											
	Firm		1C												
	Brown														
	Moist		2	SS	6										
	- 0.1 m wood fragment at 0.3 m														
114.7															
1.4	- Hydrocarbon odour between														
	SILTY CLAY to CLAY, trace sand		3	SS	19										
	Very stiff to firm														
	Brown becoming grey below		4	SS	17										
	13.3 m														
	Moist		5	SS	17										
			6	SS	17										
			7	SS	16										
			8	SS	12										
			9	SS	9										
			10	SS	9										
			11	SS	WH										
			12	TO	PH										
			13	SS	13										

Continued Next Page


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

GTA-MTO 001 S:\CLIENTS\MTQEW-LENDALE\02\_DATA\INTQEW-LENDALE.GPJ GAL-GTA.GDT 19-4-30

Continued Next Page

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

PROJECT <u>1671430 WO2</u>		<b>RECORD OF BOREHOLE No GAU-7</b>		SHEET 3 OF 4		<b>METRIC</b>	
G.W.P. <u>2423-15-00</u>		LOCATION <u>N 4779711.3; E 332115.4 MTM NAD 83 ZONE 10 (LAT. 43.156764; LONG. -79.164125)</u>		ORIGINATED BY <u>KN</u>			
DIST <u>Central</u> HWY <u>QEW</u>		BOREHOLE TYPE <u>203 mm O.D. Hollow Stem Augers; HQ Coring; CME 55 Track Mounted Drill Rig</u>		COMPILED BY <u>EN</u>			
DATUM <u>Geodetic</u>		DATE <u>July 23,24, 25, 26, 29 and 30, 2018</u>		CHECKED BY <u>LCC</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT   NATURAL LIMIT   MOISTURE   LIQUID CONTENT   LIMIT			UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W <sub>p</sub> W   W <sub>L</sub>				WATER CONTENT (%)			
								○ UNCONFINED   + FIELD VANE ● QUICK TRIAXIAL   × REMOULDED												
--- CONTINUED FROM PREVIOUS PAGE ---																				
85.7	SHALE (BEDROCK)  Bedrock cored from 30.4 m to 34.1 m.  For rock coring details refer to Record of Drillhole GAU-7.		20	SS	100/0.05		86													
30.4			1	RC	REC 100%		85												RQD = 43%	
			2	RC	REC 100%		84													RQD = 81%
			3	RC	REC 100%		83													RQD = 81%
82.0	END OF BOREHOLE						82													
34.1	NOTE:  1. Water level in open borehole at a depth of 12.1 m (Elev. 104.0 m) prior to coring.																			

GTA-MTO 001 S:\CLIENTS\MTQEW-GLENDALE\02\_DATA\GINTQEW-GLENDALE.GPJ GAL-GTA.GDT 19-4-30

PROJECT: 1671430 WO2

**RECORD OF DRILLHOLE: GAU-7**

SHEET 4 OF 4

LOCATION: N 4779711.30 ;E 332115.40

DRILLING DATE: July 31, 2018

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME55 Track-Mount

DRILLING CONTRACTOR: Geo-Environmental

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY																FEATURES	PIEZOMETER																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																												
						RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.25 m	DIP w.r.t. CORE AXIS	DISCONTINUITY DATA				WEATH- ERING INDEX						Diametral Point Load Index (MPa)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																														
						TOTAL CORE %	SOLID CORE %				TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jzon	W1	W2	W3	W4	W5	W6																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																															
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		Continued from Record of Borehole GAU-7		85.73																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																															</



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



PROJECT		RECORD OF BOREHOLE				No GAU-8		SHEET 2 OF 2		METRIC																			
G.W.P. 2423-15-00		LOCATION				N 4779697.2; E 332094.7 MTM NAD 83 ZONE 10 (LAT. 43.156638; LONG. -79.164388)		ORIGINATED BY		SE																			
DIST Central HWY QEW		BOREHOLE TYPE				152 mm Hollow Stem Auger; CME 55 Track-mounted rig		COMPILED BY		KG																			
DATUM Geodetic		DATE				September 27, 2018		CHECKED BY		LCC																			
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)													
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa									WATER CONTENT (%)												
							20	40	60	80	100																		
							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED																						
							20	40	60	80	100	10	20	30	kN/m <sup>3</sup>	GR SA SI CL													
99.8			13	SS	100/0.12																								
15.9	END OF BOREHOLE																												
	NOTES:  1. Borehole caved to 14.6 m on removal of augers.  2. Water level in open borehole at depth 14.5 m on completion of drilling.  3. Water level in piezometer on completion of installation at depth of 14.6 m.  4. Water level in standpipe piezometer measured as follows:  <table border="1" style="margin-left: 40px;"> <thead> <tr> <th>Date (mm/dd/yy)</th> <th>Depth (mbgs)</th> <th>Elevation (m)</th> </tr> </thead> <tbody> <tr> <td>09/28/18</td> <td>14.5</td> <td>101.1</td> </tr> <tr> <td>11/26/18</td> <td>4.5</td> <td>111.1</td> </tr> <tr> <td>05/07/19</td> <td>4.6</td> <td>111.1</td> </tr> </tbody> </table>	Date (mm/dd/yy)	Depth (mbgs)	Elevation (m)	09/28/18	14.5	101.1	11/26/18	4.5	111.1	05/07/19	4.6	111.1																
Date (mm/dd/yy)	Depth (mbgs)	Elevation (m)																											
09/28/18	14.5	101.1																											
11/26/18	4.5	111.1																											
05/07/19	4.6	111.1																											

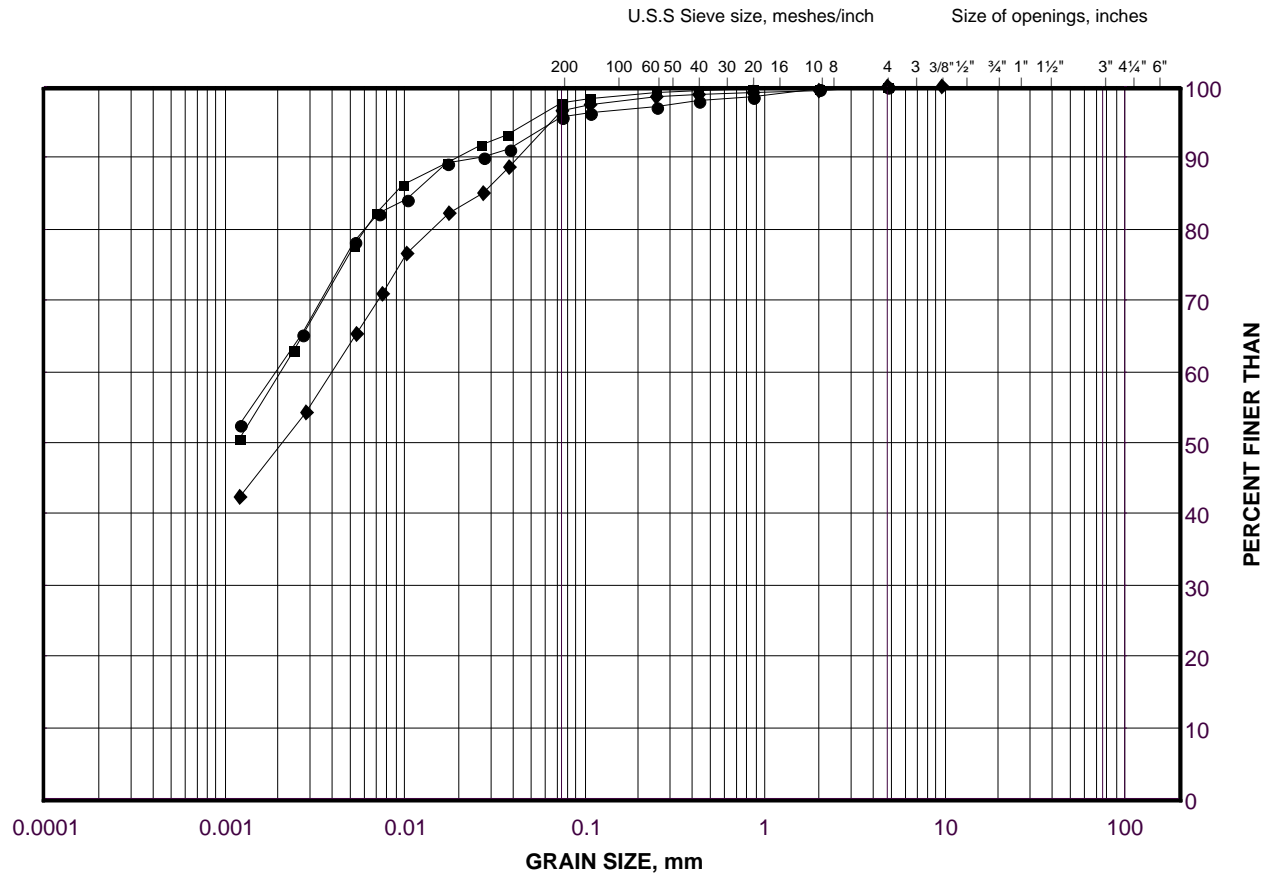
**APPENDIX B**

# Geotechnical Laboratory Test Results

# GRAIN SIZE DISTRIBUTION

Silty Clay to Clay Fill

FIGURE B-1



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

## LEGEND

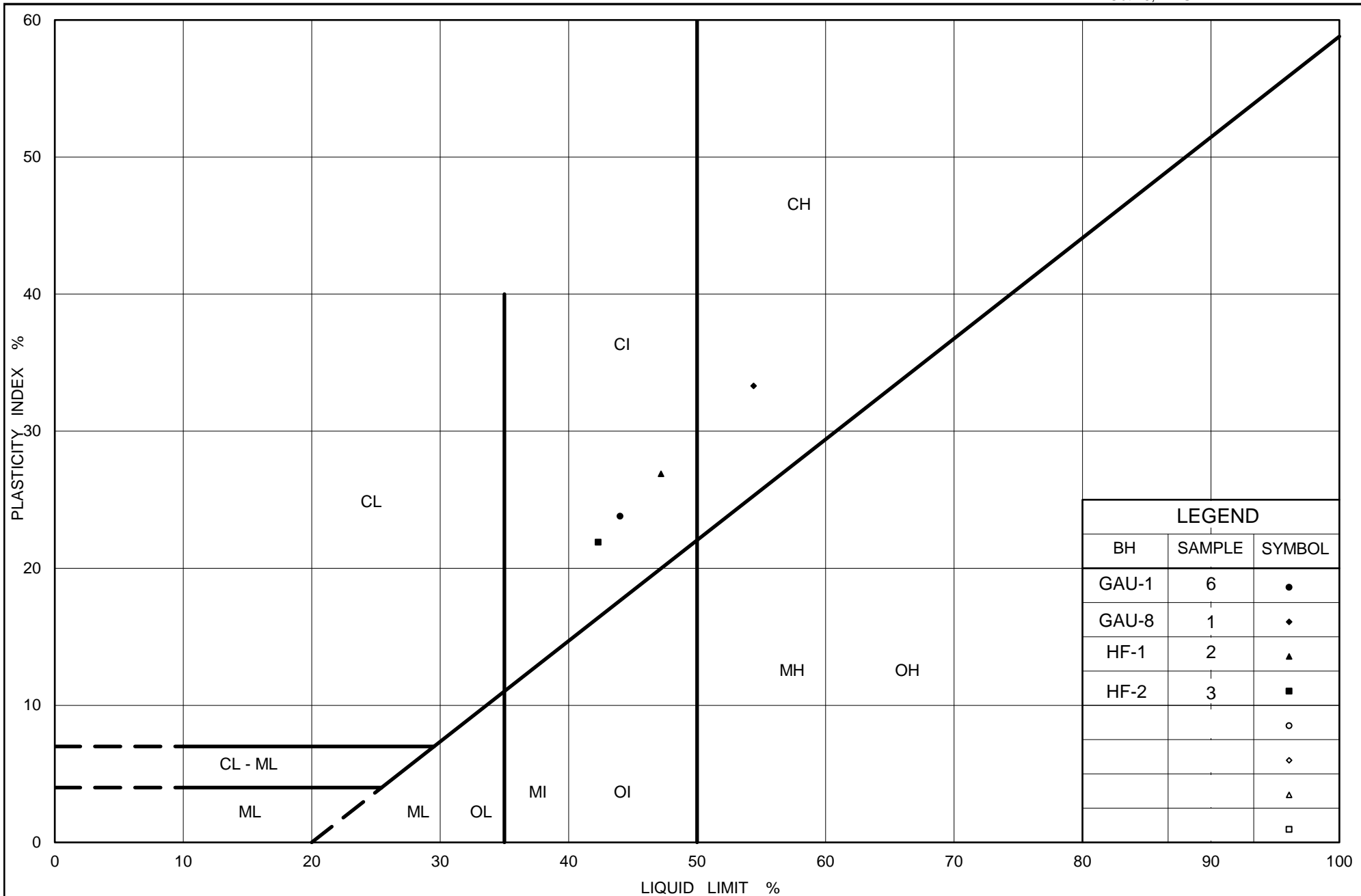
SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	GAU-8	1	114.5
■	HF-1	2	115.4
◆	GAU-1	6	117.2

Project Number: 1671430

Checked By: MA/LCC

**Golder Associates**

Date: 30-Apr-19



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# PLASTICITY CHART Silty Clay to Clay Fill

Figure No. B-2

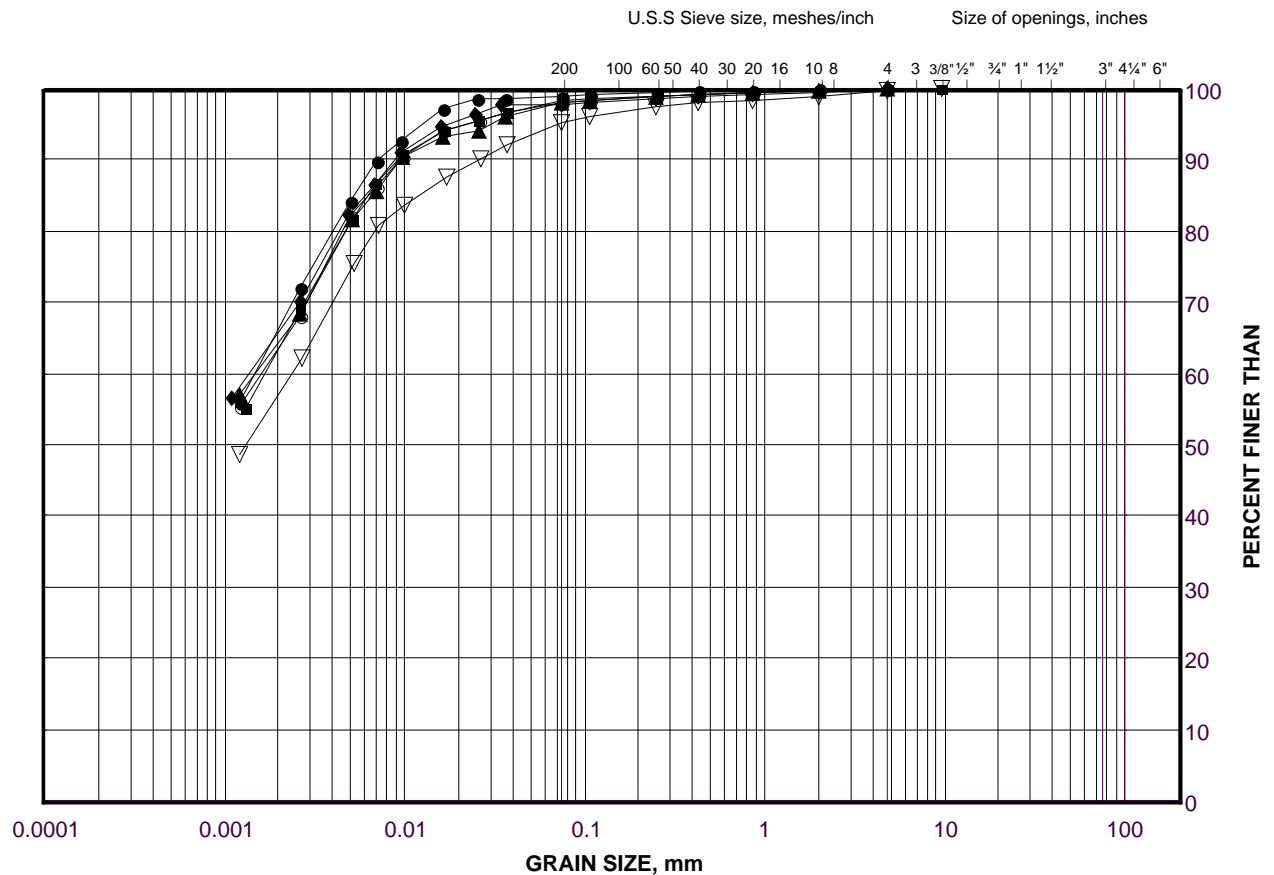
Project No. 1671430 (WO 002)

Checked By: MA/LCC

# GRAIN SIZE DISTRIBUTION

Silty Clay to Clay

FIGURE B-3A



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

## LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	GAU-3	10	108.1
■	GAU-8	6	110.7
◆	GAU-3	6	114.2
▲	GAU-7	7	111.2
▽	GAU-8	9	106.2
○	GAU-1	9	112.7

Project Number: 1671430

Checked By: MA/LCC

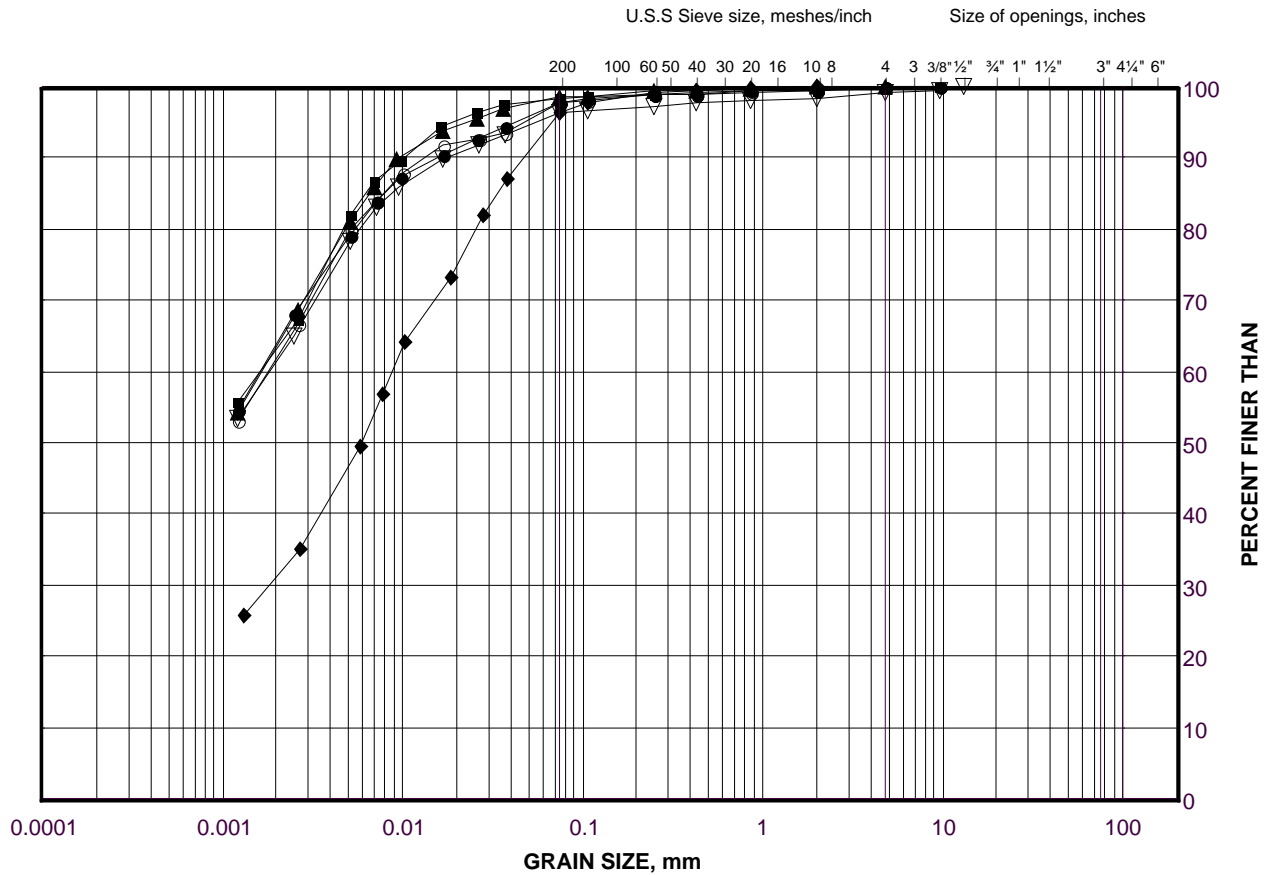
**Golder Associates**

Date: 30-Apr-19

# GRAIN SIZE DISTRIBUTION

Silty Clay to Clay

FIGURE B-3B



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

## LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	HF-1	11	105.5
■	HF-5	3	114.9
◆	HF-1	6	112.4
▲	HF-5	8	109.6
▽	HF-1	8	110.1
○	HF-2	8	110.1

Project Number: 1671430

Checked By: MA/LCC

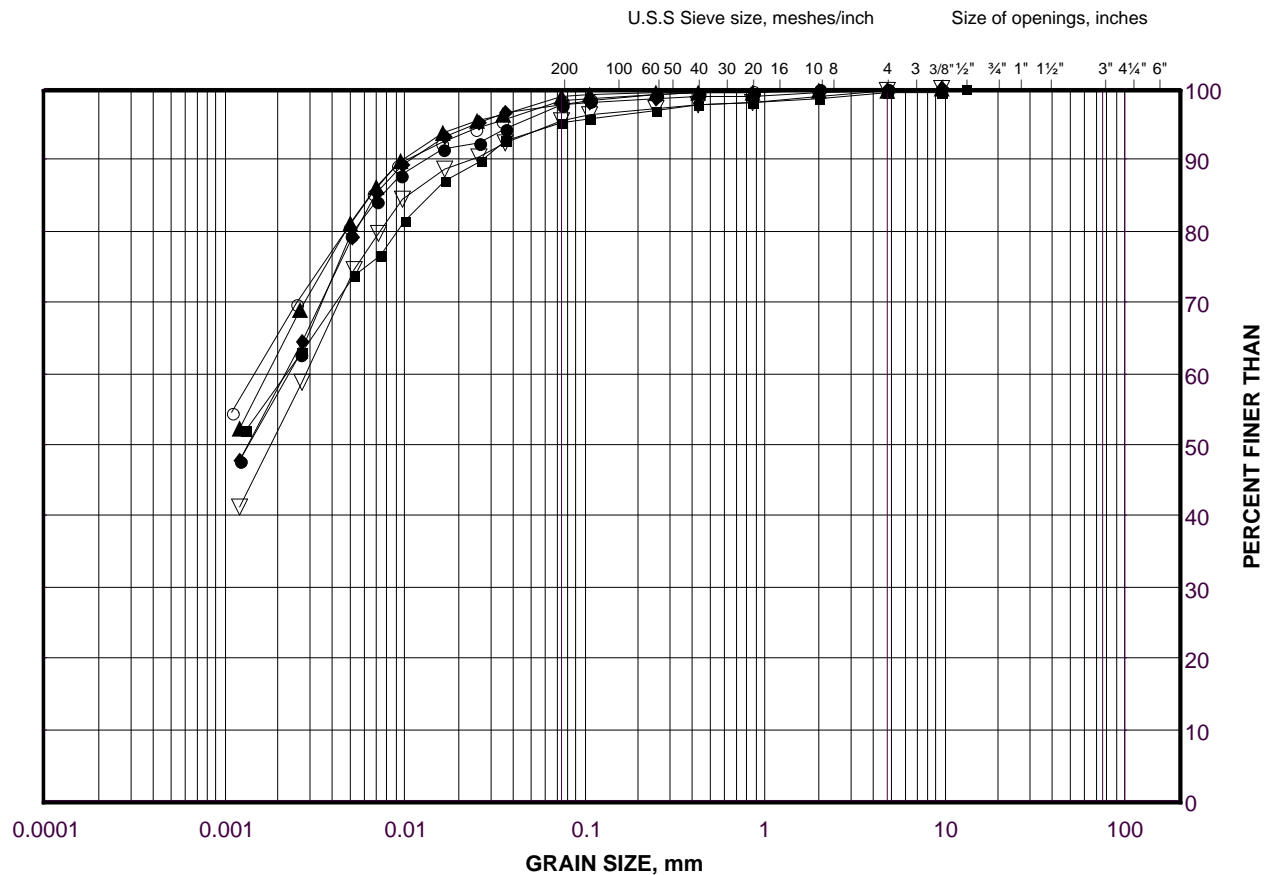
**Golder Associates**

Date: 30-Apr-19

# GRAIN SIZE DISTRIBUTION

Silty Clay to Clay

FIGURE B-3C



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

## LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	HF-2	12	104.1
■	GAU-3	12	105.1
◆	HF-1	13	102.5
▲	GAU-1	13	106.6
▽	GAU-3	14	102.0
○	HF-4	7	108.4

Project Number: 1671430

Checked By: MA/LCC

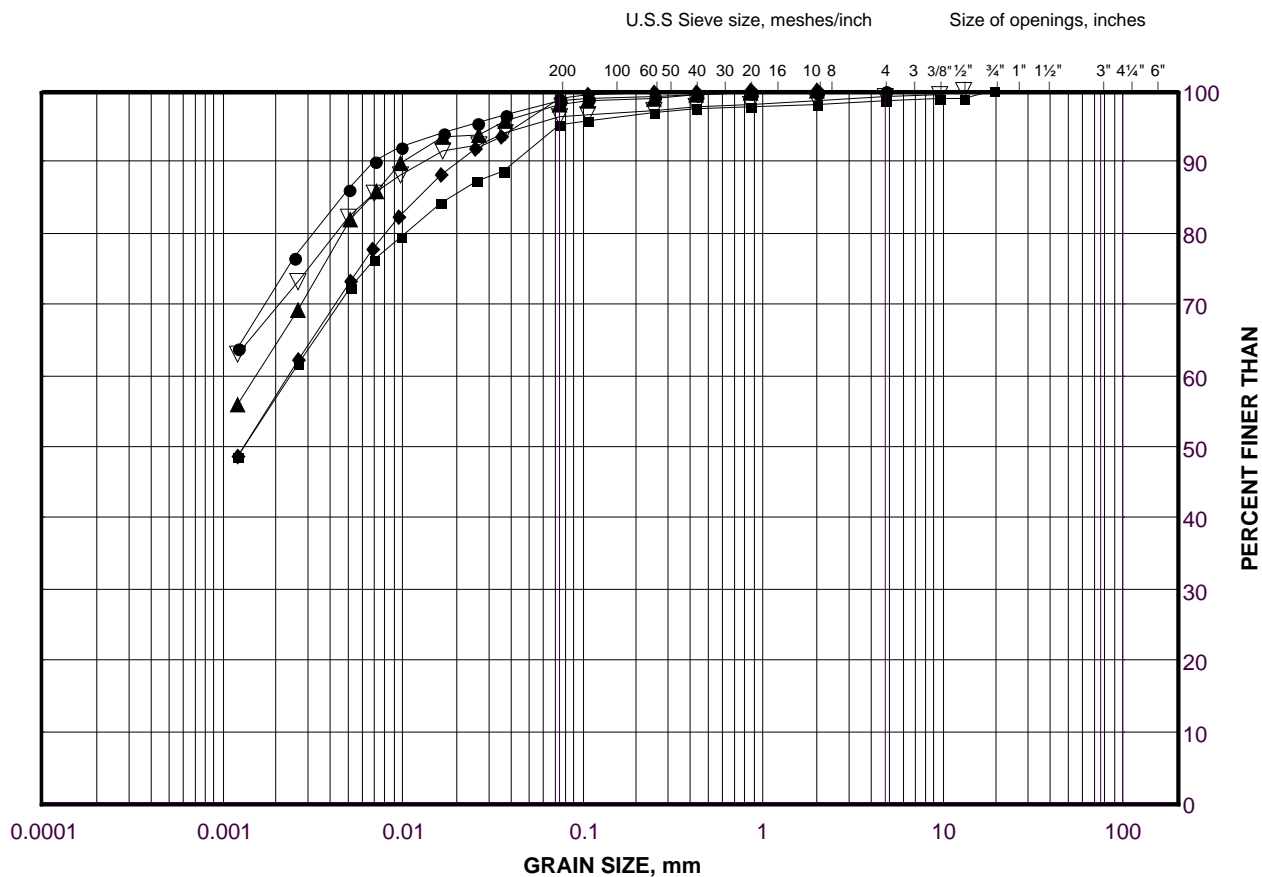
**Golder Associates**

Date: 30-Apr-19

# GRAIN SIZE DISTRIBUTION

Silty Clay to Clay

FIGURE B-3D



## LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	HF-3	10	106.5
■	HF-5	11	105.0
◆	HF-4	11	102.3
▲	HF-3	8	109.6
▽	HF-4	9	105.4

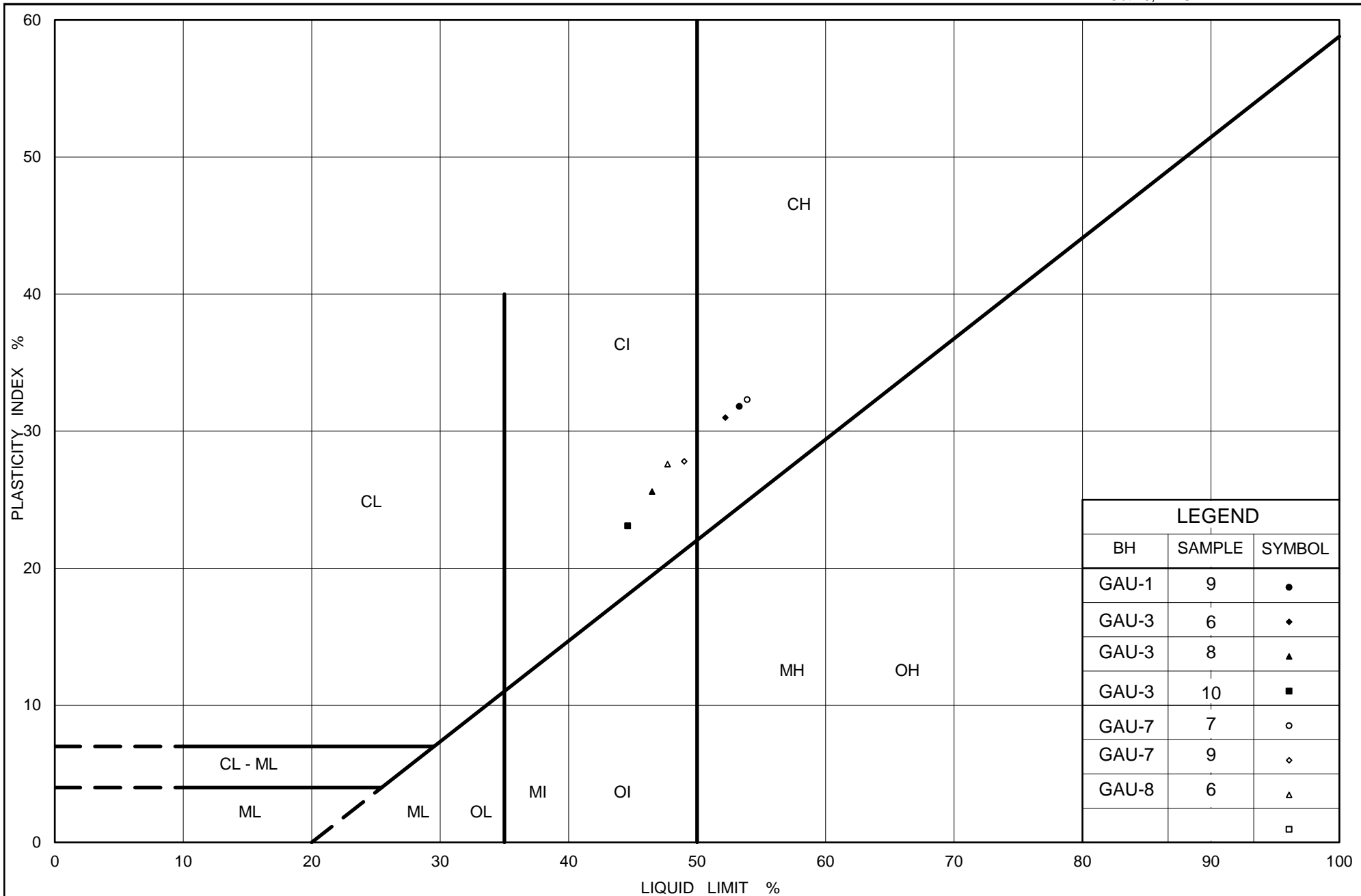
Project Number: 1671430

Checked By: MA/LCC

**Golder Associates**

Date: 30-Apr-19





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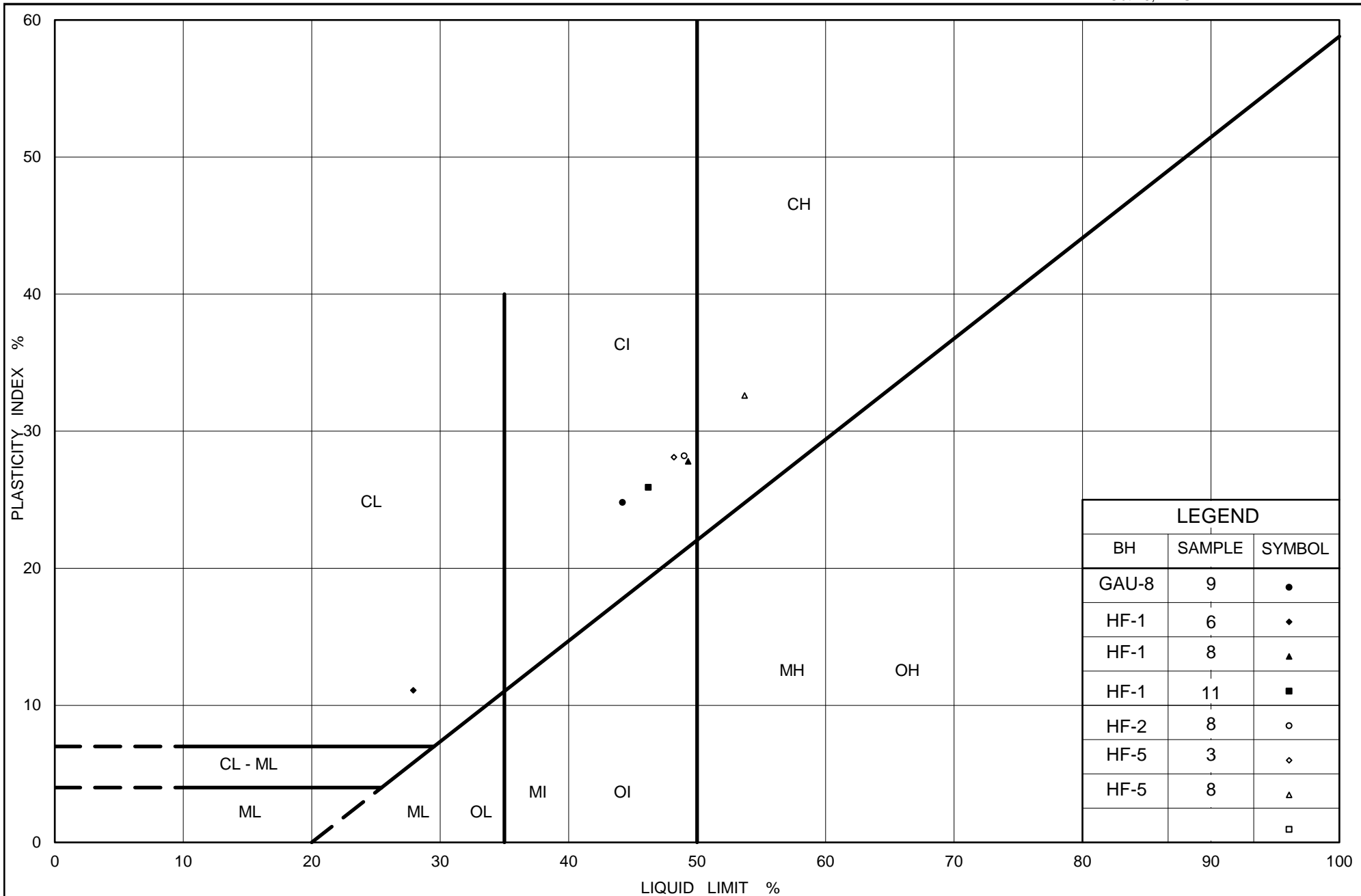
Ontario

# PLASTICITY CHART Silty Clay to Clay

Figure No. B-4A

Project No. 1671430 (WO 002)

Checked By : MA/LCC



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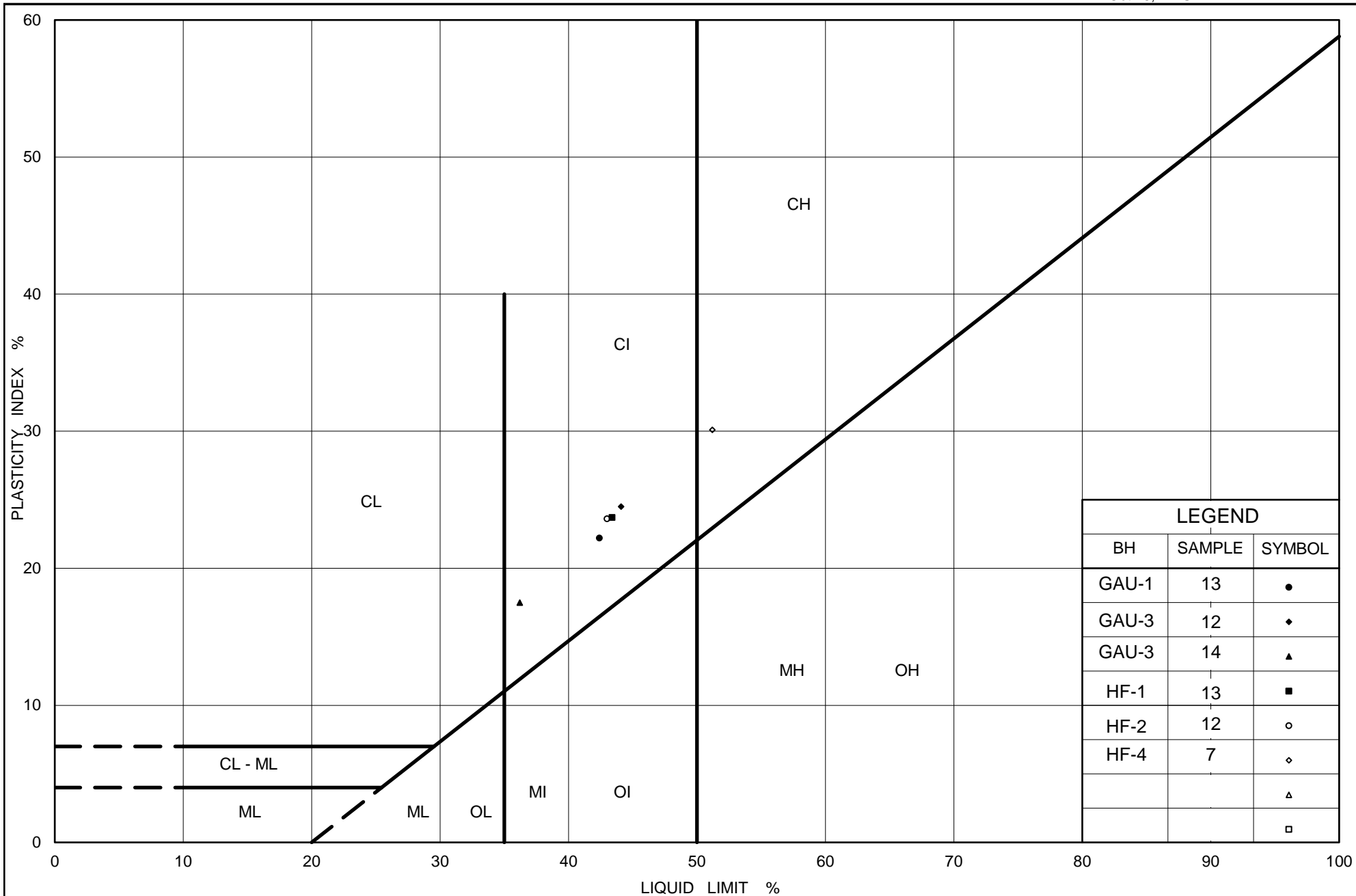
Ontario

# PLASTICITY CHART Silty Clay to Clay

Figure No. B-4B

Project No. 1671430 (WO 002)

Checked By: MA/LCC



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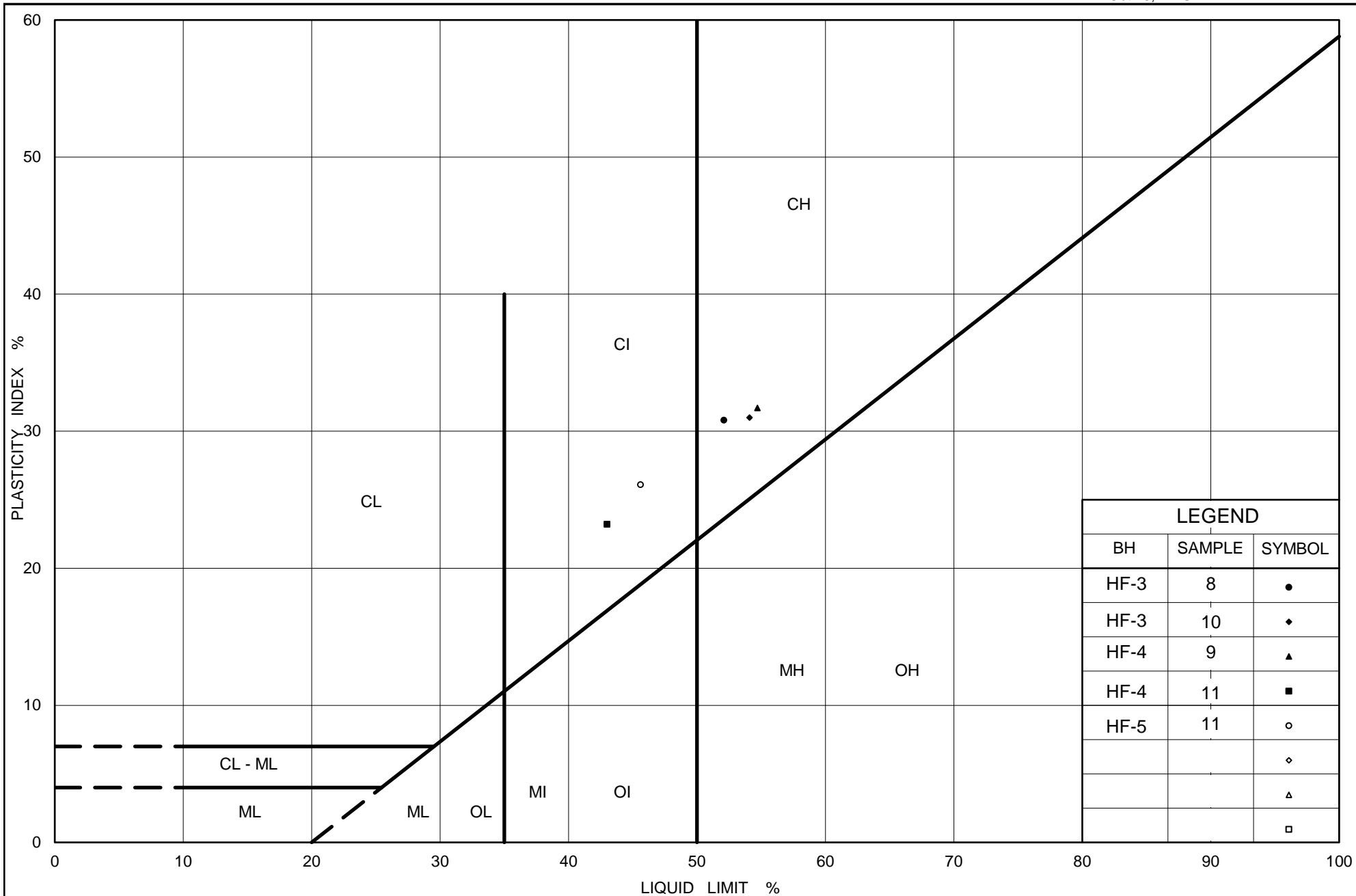
## PLASTICITY CHART

### Silty Clay to Clay

Figure No. B-4C

Project No. 1671430 (WO 002)

Checked By: MA/LCC



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# PLASTICITY CHART Silty Clay to Clay

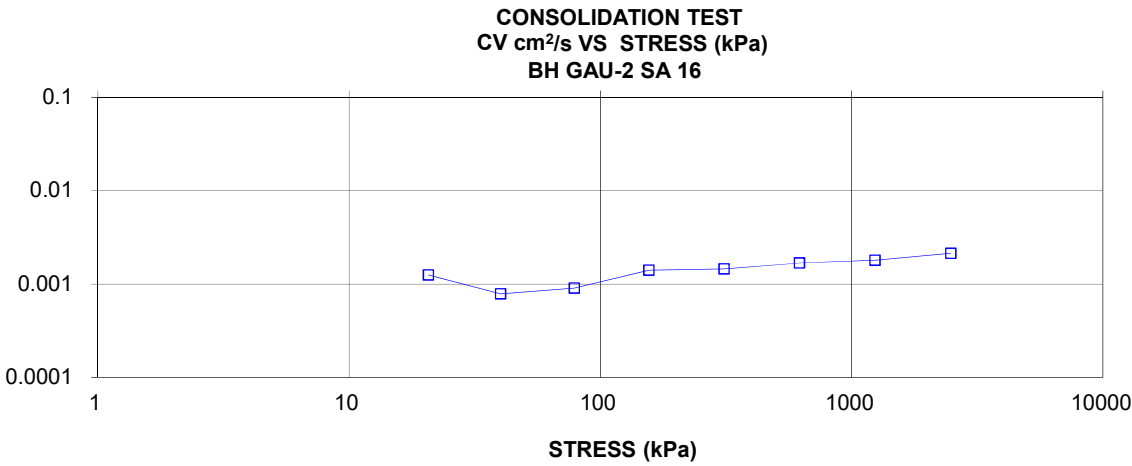
Figure No. B-4D

Project No. 1671430 (WO 002)

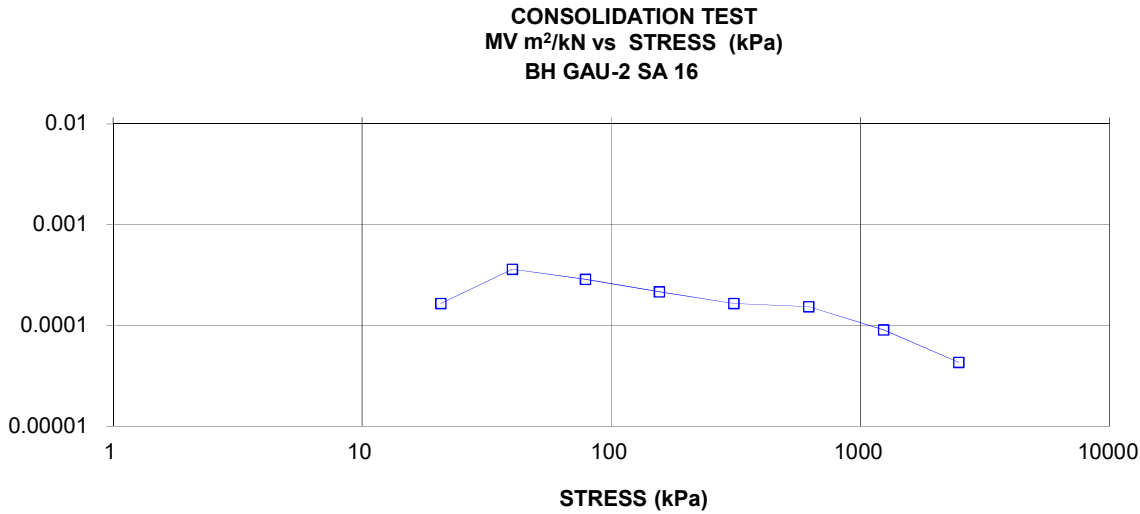
Checked By: MA/LCC

<div style="display: flex; justify-content: space-between;"> <div style="text-align: center;"> <b>CONSOLIDATION TEST SUMMARY</b>  <b>ASTM D2435/D2435M</b> </div> <div style="text-align: center;"> <b>FIGURE B-5A</b> </div> </div>							
<b>SAMPLE IDENTIFICATION</b>							
Project Number	1671430(WO002)	Sample Number	16				
Borehole Number	GAU-2	Sample Depth, ft	18.29-18.90				
<b>TEST CONDITIONS</b>							
Test Type	Laboratory Standard	Load Duration, hr	24				
Oedometer Number	2						
Date Started	10/06/2018						
Date Completed	10/19/2018						
<b>SAMPLE DIMENSIONS AND PROPERTIES - INITIAL</b>							
Sample Height, cm	2.54	Unit Weight, kN/m <sup>3</sup>	18.61				
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m <sup>3</sup>	13.94				
Area, cm <sup>2</sup>	31.65	Specific Gravity, measured	2.72				
Volume, cm <sup>3</sup>	80.29	Solids Height, cm	1.325				
Water Content, %	33.55	Volume of Solids, cm <sup>3</sup>	41.95				
Wet Mass, g	152.38	Volume of Voids, cm <sup>3</sup>	38.35				
Dry Mass, g	114.1	Degree of Saturation, %	99.8				
<b>TEST COMPUTATIONS</b>							
	Corr.		Average				
Stress	Height	Void	Height	t <sub>90</sub>	cv.	mv	k
kPa	cm	Ratio	cm	sec	cm <sup>2</sup> /s	m <sup>2</sup> /kN	cm/s
0.00	2.537	0.914	2.537				
6.01	2.547	0.922	2.542				
10.64	2.557	0.929	2.552				
20.68	2.553	0.926	2.555	1109	1.25E-03	1.65E-04	2.02E-08
40.08	2.535	0.913	2.544	1750	7.84E-04	3.62E-04	2.78E-08
78.73	2.510	0.894	2.523	1500	8.99E-04	2.51E-04	2.54E-08
156.03	2.468	0.862	2.489	936	1.40E-03	2.17E-04	2.98E-08
310.38	2.403	0.813	2.436	866	1.45E-03	1.65E-04	2.35E-08
619.74	2.282	0.722	2.343	694	1.68E-03	1.54E-04	2.53E-08
1237.93	2.141	0.615	2.211	578	1.79E-03	9.03E-05	1.59E-08
2480.25	2.005	0.512	2.073	427	2.13E-03	4.32E-05	9.02E-09
619.74	2.052	0.548	2.028				
156.03	2.130	0.607	2.091				
40.08	2.216	0.672	2.173				
10.72	2.294	0.731	2.255				
<p>Note:</p> <p>Consolidation loading and unloading schedule assigned by the client.</p> <p>cv and k are approximate only based on t<sub>90</sub> estimated from Square Root of Time Method (ASTMD2435/2435M)</p> <p>Specimen swelled under 10.64kPa.</p>							
<b>SAMPLE DIMENSIONS AND PROPERTIES - FINAL</b>							
Sample Height, cm	2.29	Unit Weight, kN/m <sup>3</sup>	20.04				
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m <sup>3</sup>	15.41				
Area, cm <sup>2</sup>	31.65	Specific Gravity, measured	2.72				
Volume, cm <sup>3</sup>	72.61	Solids Height, cm	1.325				
Water Content, %	30.04	Volume of Solids, cm <sup>3</sup>	41.95				
Wet Mass, g	148.37	Volume of Voids, cm <sup>3</sup>	30.66				
Dry Mass, g	114.1						
Prepared By: LH <div style="float: right; text-align: right;"> <b>Golder Associates</b>            Checked By:         </div>							

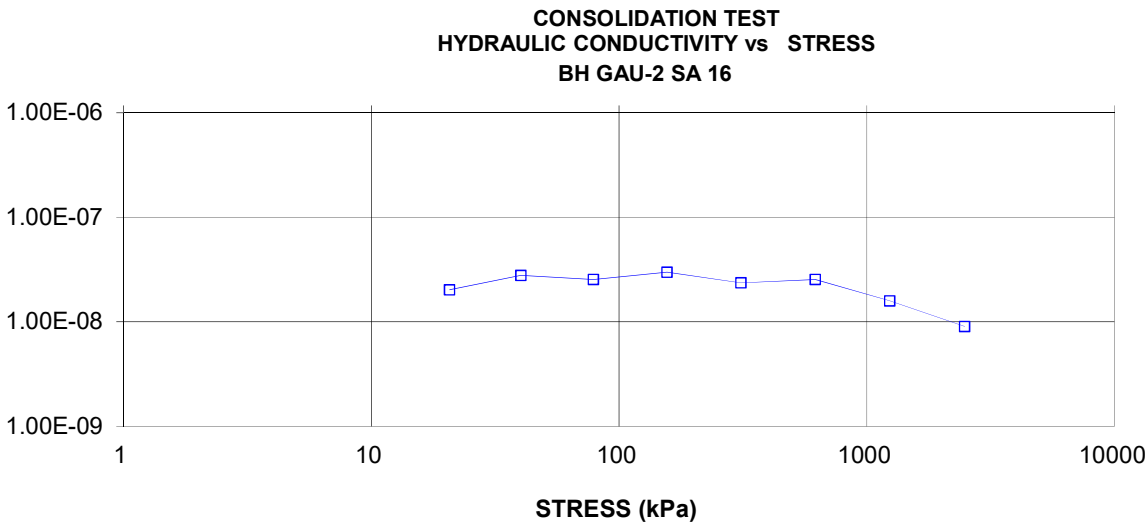
COEFFICIENT OF CONSOLIDATION,  
cm<sup>2</sup>/s

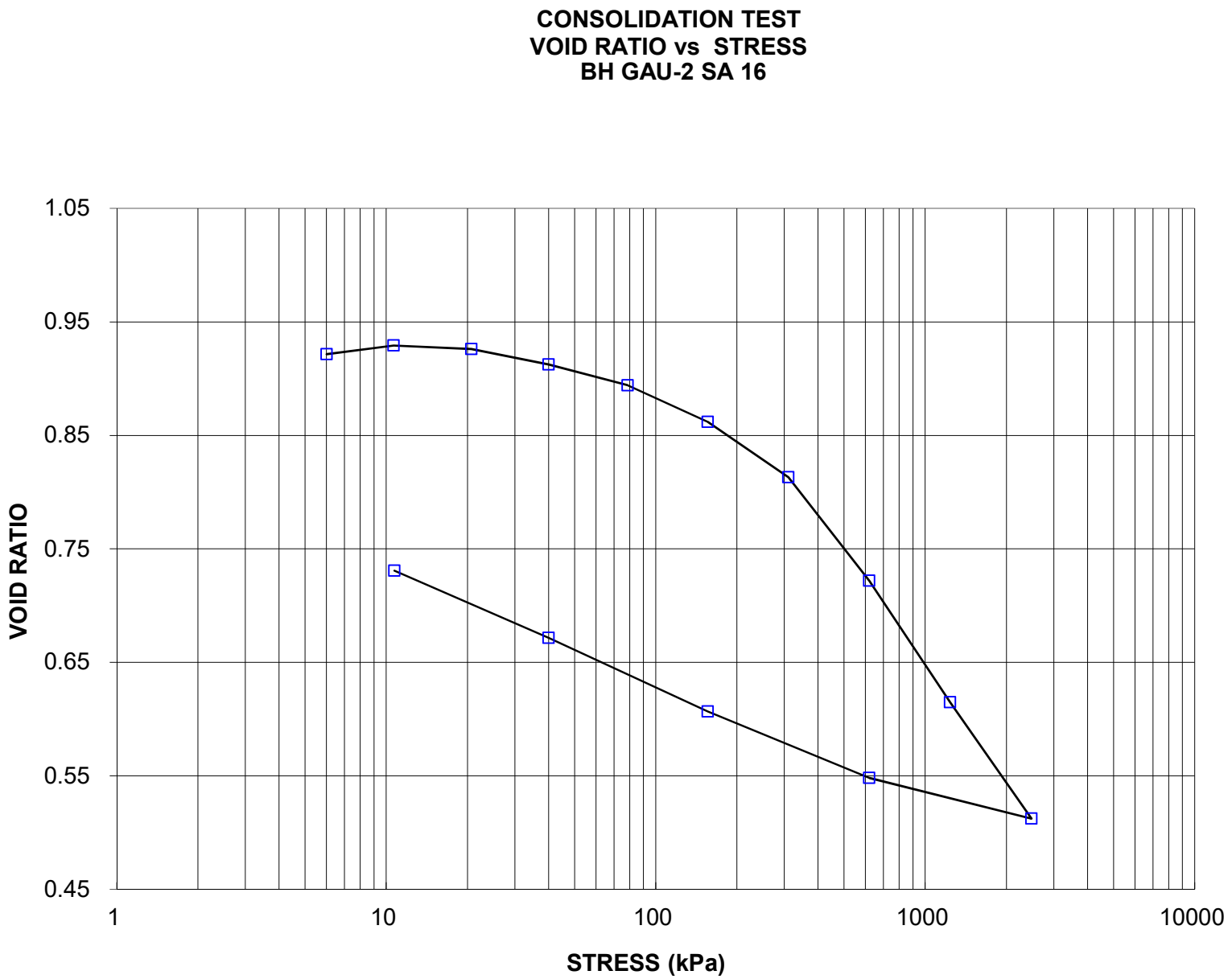


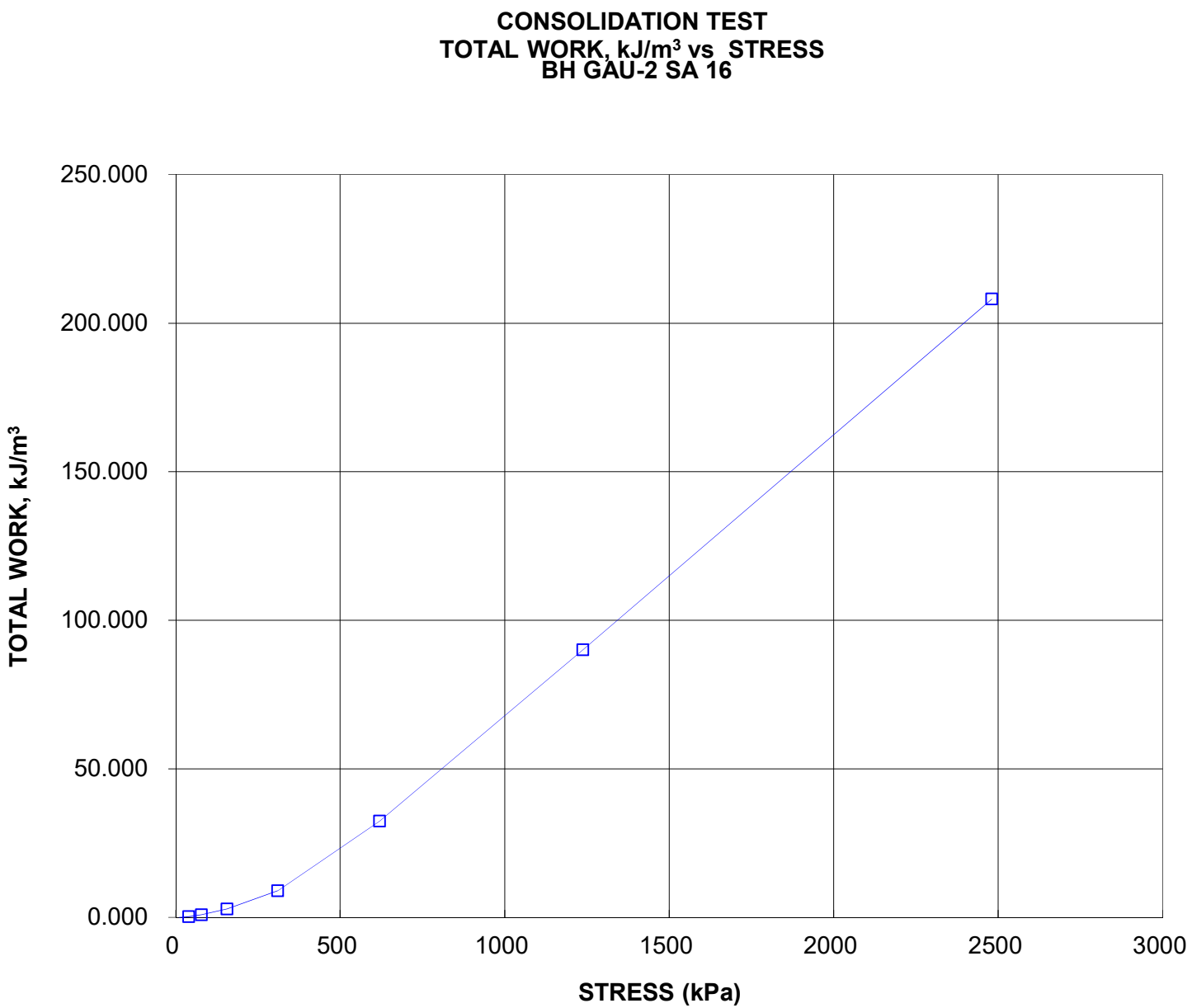
VOLUME COMPRESSIBILITY, m<sup>2</sup>/kN



HYDRAULIC CONDUCTIVITY, cm/s

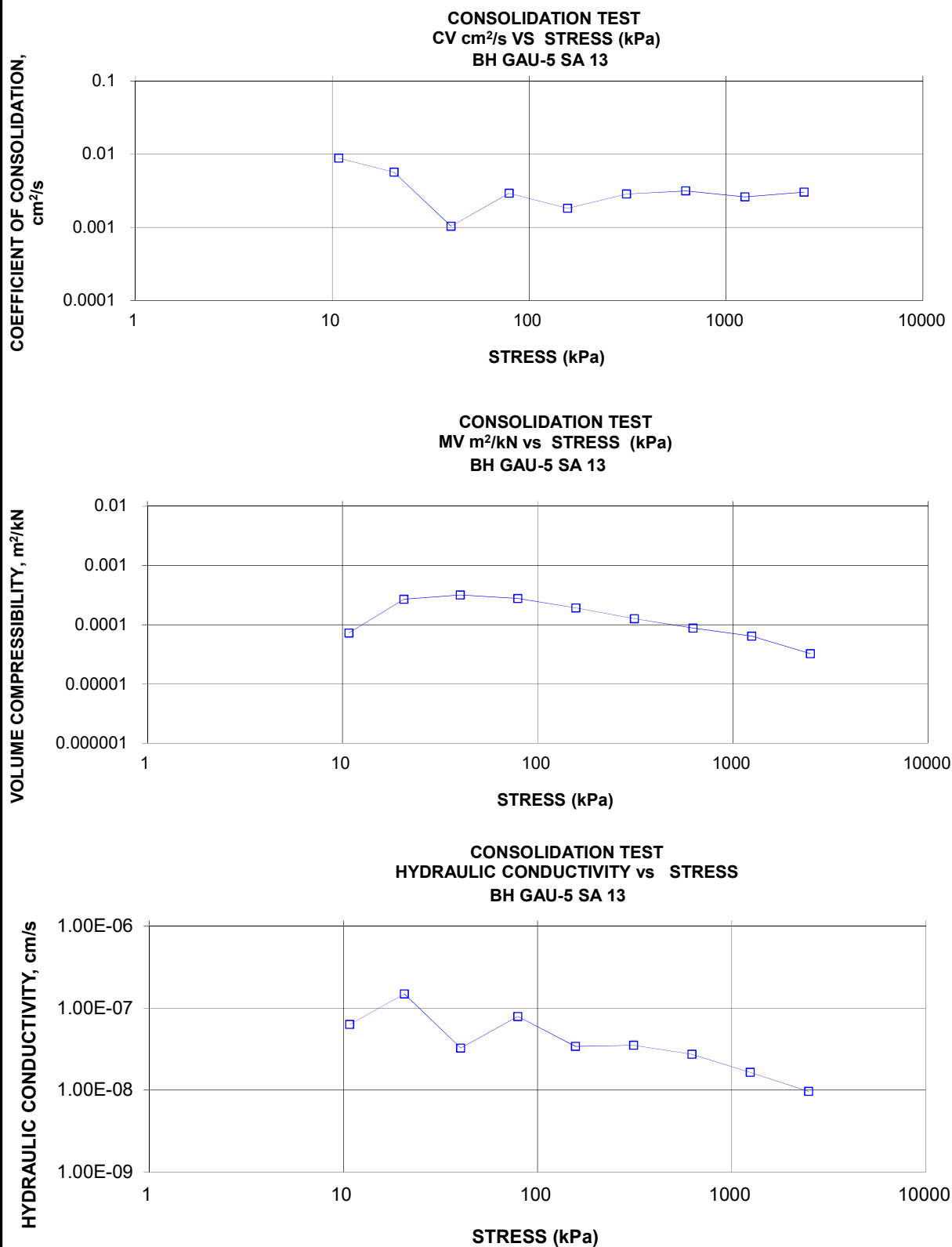


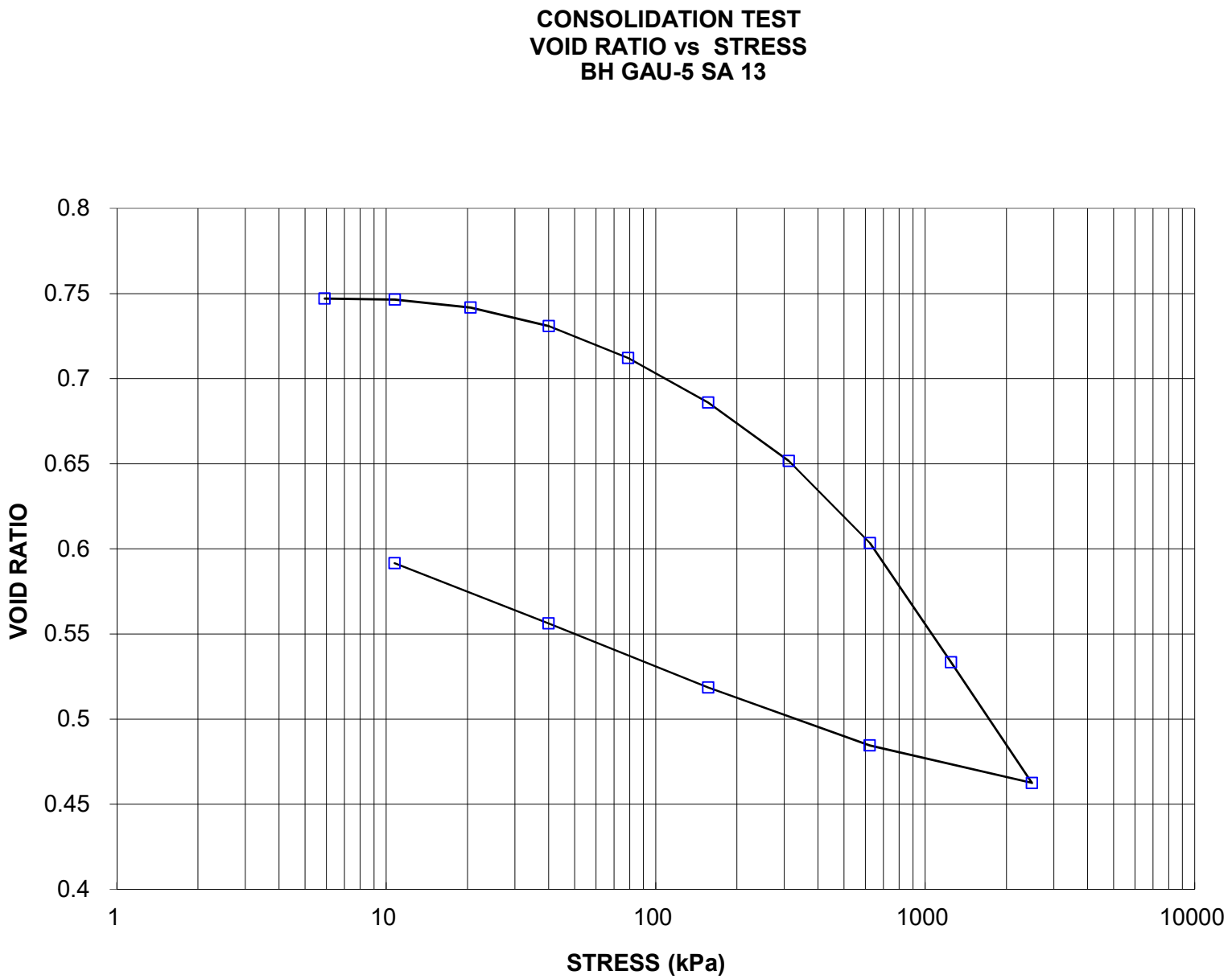


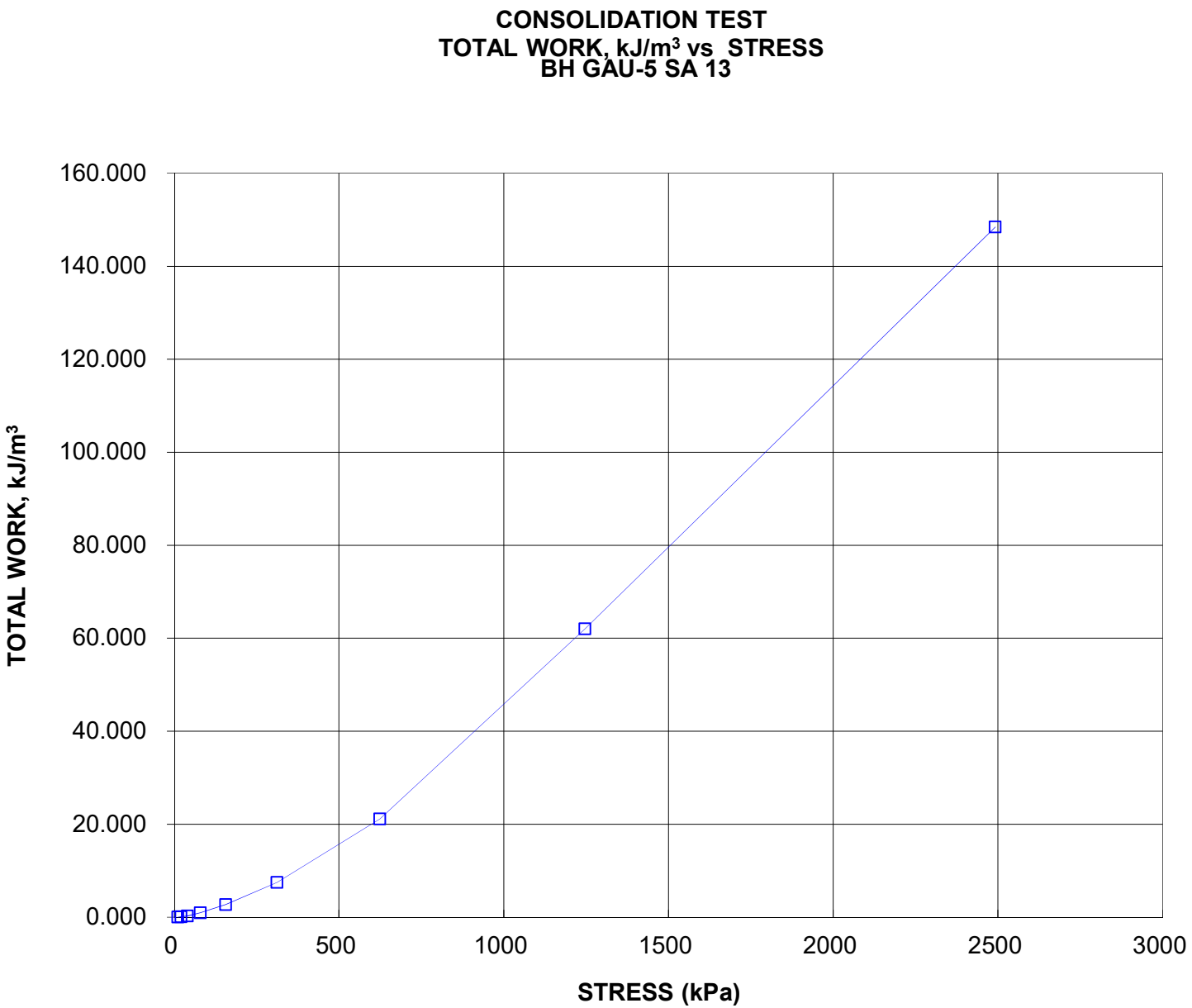




<div style="display: flex; justify-content: space-between;"> <div style="text-align: center;"> <b>CONSOLIDATION TEST SUMMARY</b>  <b>ASTM D2435/D2435M</b> </div> <div style="text-align: center;"> <b>FIGURE B-6A</b> </div> </div>				
<b>SAMPLE IDENTIFICATION</b>				
Project Number	1671430(WO002)	Sample Number	13	
Borehole Number	GAU-5	Sample Depth, ft	15.24-15.70	
<b>TEST CONDITIONS</b>				
Test Type	Laboratory Standard	Load Duration, hr	24	
Oedometer Number	3			
Date Started	10/05/2018			
Date Completed	10/18/2018			
<b>SAMPLE DIMENSIONS AND PROPERTIES - INITIAL</b>				
Sample Height, cm	2.53	Unit Weight, kN/m <sup>3</sup>	19.56	
Sample Diameter, cm	6.33	Dry Unit Weight, kN/m <sup>3</sup>	15.38	
Area, cm <sup>2</sup>	31.48	Specific Gravity, measured	2.74	
Volume, cm <sup>3</sup>	79.74	Solids Height, cm	1.450	
Water Content, %	27.16	Volume of Solids, cm <sup>3</sup>	45.64	
Wet Mass, g	159.03	Volume of Voids, cm <sup>3</sup>	34.10	
Dry Mass, g	125.06	Degree of Saturation, %	99.6	
<b>TEST COMPUTATIONS</b>				
	Corr.	Average		
Stress	Height	Void	Height	t <sub>90</sub>
kPa	cm	Ratio	cm	sec
				cv.
				mv
				k
				cm <sup>2</sup> /s
				m <sup>2</sup> /kN
				cm/s
0.00	2.533	0.747	2.533	
5.91	2.533	0.747	2.533	
10.77	2.532	0.746	2.533	154
20.58	2.525	0.742	2.529	240
40.10	2.510	0.731	2.518	1297
79.04	2.482	0.712	2.496	454
156.70	2.445	0.686	2.463	711
312.22	2.395	0.652	2.420	437
624.00	2.325	0.603	2.360	375
1246.74	2.223	0.533	2.274	421
2491.94	2.120	0.462	2.172	332
623.06	2.152	0.485	2.136	
156.70	2.202	0.519	2.177	
40.05	2.256	0.556	2.229	
10.74	2.308	0.592	2.282	
<p>Note:</p> <p>Consolidation loading and unloading schedule assigned by the client.</p> <p>cv and k are approximate only based on t<sub>90</sub> estimated from Square Root of Time Method (ASTMD2435/2435M)</p> <p>Specimen swelled under 5.91kPa.</p>				
<b>SAMPLE DIMENSIONS AND PROPERTIES - FINAL</b>				
Sample Height, cm	2.31	Unit Weight, kN/m <sup>3</sup>	20.78	
Sample Diameter, cm	6.33	Dry Unit Weight, kN/m <sup>3</sup>	16.88	
Area, cm <sup>2</sup>	31.48	Specific Gravity, measured	2.74	
Volume, cm <sup>3</sup>	72.64	Solids Height, cm	1.450	
Water Content, %	23.08	Volume of Solids, cm <sup>3</sup>	45.64	
Wet Mass, g	153.93	Volume of Voids, cm <sup>3</sup>	27.00	
Dry Mass, g	125.06			
<div style="display: flex; justify-content: space-between;"> <div>Prepared By: LH</div> <div><b>Golder Associates</b></div> <div>Checked By:</div> </div>				

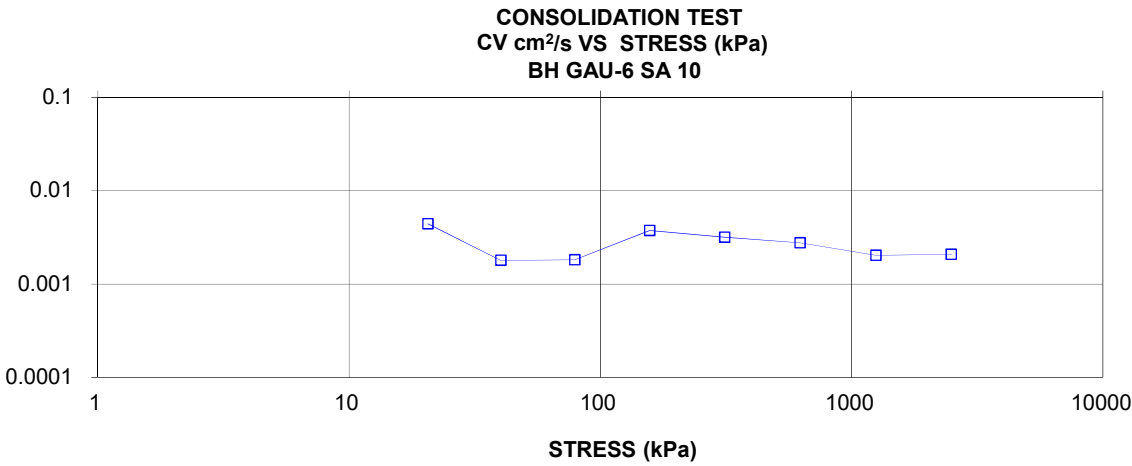




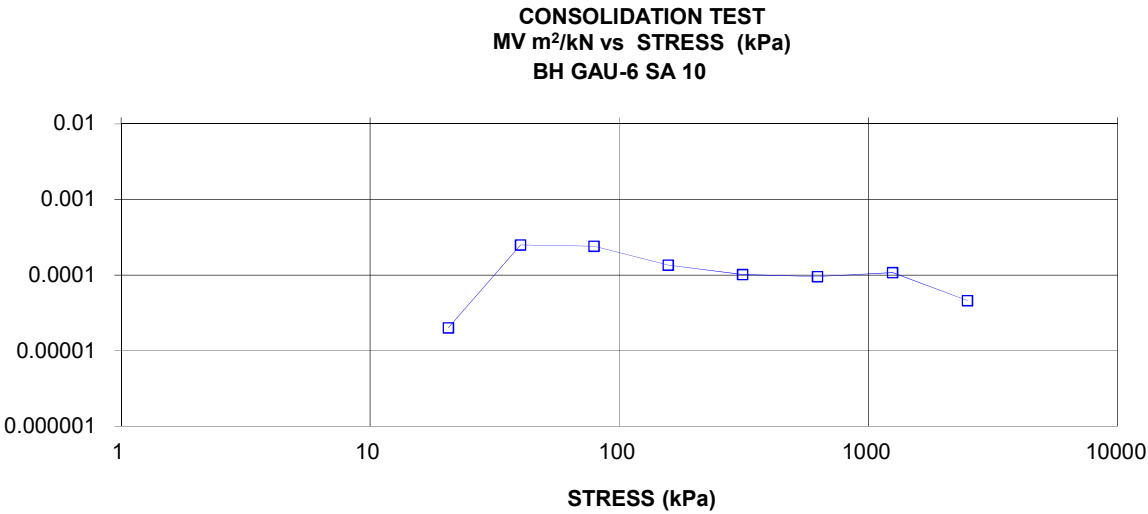


<div style="display: flex; justify-content: space-between;"> <div style="text-align: center;"> <b>CONSOLIDATION TEST SUMMARY</b>  <b>ASTM D2435/D2435M</b> </div> <div style="text-align: center;"> <b>FIGURE B-7A</b> </div> </div>				
<b>SAMPLE IDENTIFICATION</b>				
Project Number	1671430(WO002)	Sample Number	10	
Borehole Number	GAU-6	Sample Depth, ft	10.67-11.28	
<b>TEST CONDITIONS</b>				
Test Type	Laboratory Standard	Load Duration, hr	24	
Oedometer Number	4			
Date Started	10/05/2018			
Date Completed	10/17/2018			
<b>SAMPLE DIMENSIONS AND PROPERTIES - INITIAL</b>				
Sample Height, cm	2.54	Unit Weight, kN/m <sup>3</sup>	18.98	
Sample Diameter, cm	6.33	Dry Unit Weight, kN/m <sup>3</sup>	14.34	
Area, cm <sup>2</sup>	31.45	Specific Gravity, measured	2.78	
Volume, cm <sup>3</sup>	79.88	Solids Height, cm	1.336	
Water Content, %	32.37	Volume of Solids, cm <sup>3</sup>	42.01	
Wet Mass, g	154.60	Volume of Voids, cm <sup>3</sup>	37.87	
Dry Mass, g	116.79	Degree of Saturation, %	99.8	
<b>TEST COMPUTATIONS</b>				
	Corr.	Average		
Stress	Height	Void	Height	t <sub>90</sub>
kPa	cm	Ratio	cm	sec
				cv.
				cm <sup>2</sup> /s
				mv
				m <sup>2</sup> /kN
				k
				cm/s
0.00	2.540	0.901	2.540	
5.94	2.555	0.913	2.548	
10.80	2.570	0.924	2.563	
20.57	2.570	0.924	2.570	317
40.14	2.557	0.914	2.563	778
79.10	2.534	0.897	2.545	756
156.94	2.507	0.877	2.520	360
312.60	2.467	0.846	2.487	413
624.05	2.391	0.790	2.429	452
1251.00	2.219	0.661	2.305	554
2495.95	2.074	0.553	2.147	470
624.05	2.122	0.589	2.098	
157.01	2.200	0.647	2.161	
40.13	2.288	0.713	2.244	
10.78	2.365	0.770	2.326	
<p>Note:</p> <p>Consolidation loading and unloading schedule assigned by the client.</p> <p>cv and k are approximate only based on t<sub>90</sub> estimated from Square Root of Time Method (ASTMD2435/2435M)</p> <p>Specimen swelled under 10.80kPa.</p>				
<b>SAMPLE DIMENSIONS AND PROPERTIES - FINAL</b>				
Sample Height, cm	2.36	Unit Weight, kN/m <sup>3</sup>	19.90	
Sample Diameter, cm	6.33	Dry Unit Weight, kN/m <sup>3</sup>	15.40	
Area, cm <sup>2</sup>	31.45	Specific Gravity, measured	2.78	
Volume, cm <sup>3</sup>	74.38	Solids Height, cm	1.336	
Water Content, %	29.20	Volume of Solids, cm <sup>3</sup>	42.01	
Wet Mass, g	150.89	Volume of Voids, cm <sup>3</sup>	32.37	
Dry Mass, g	116.79			
<div style="display: flex; justify-content: space-between;"> <div>Prepared By: LH</div> <div><b>Golder Associates</b></div> <div>Checked By:</div> </div>				

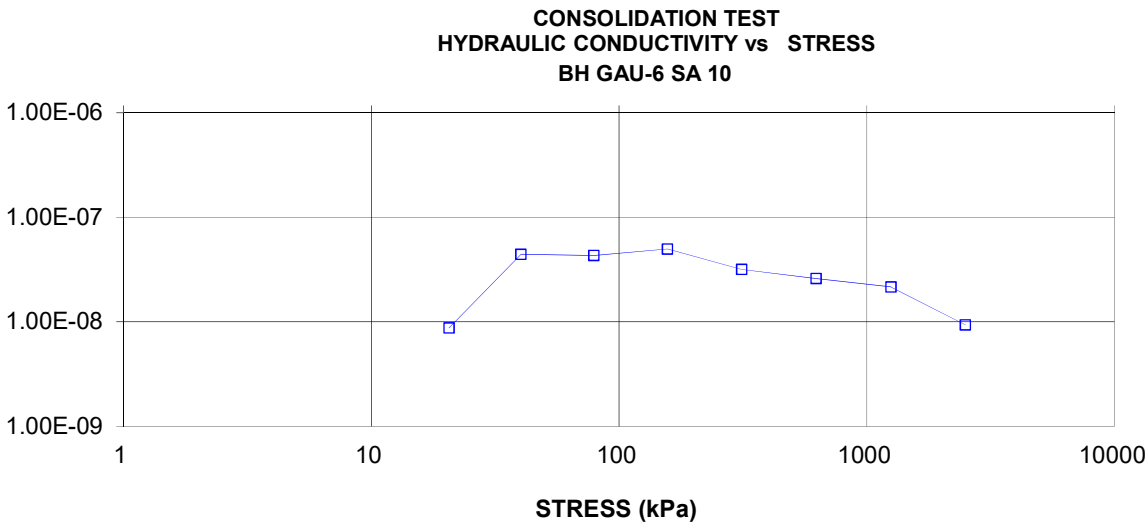
COEFFICIENT OF CONSOLIDATION,  
cm<sup>2</sup>/s

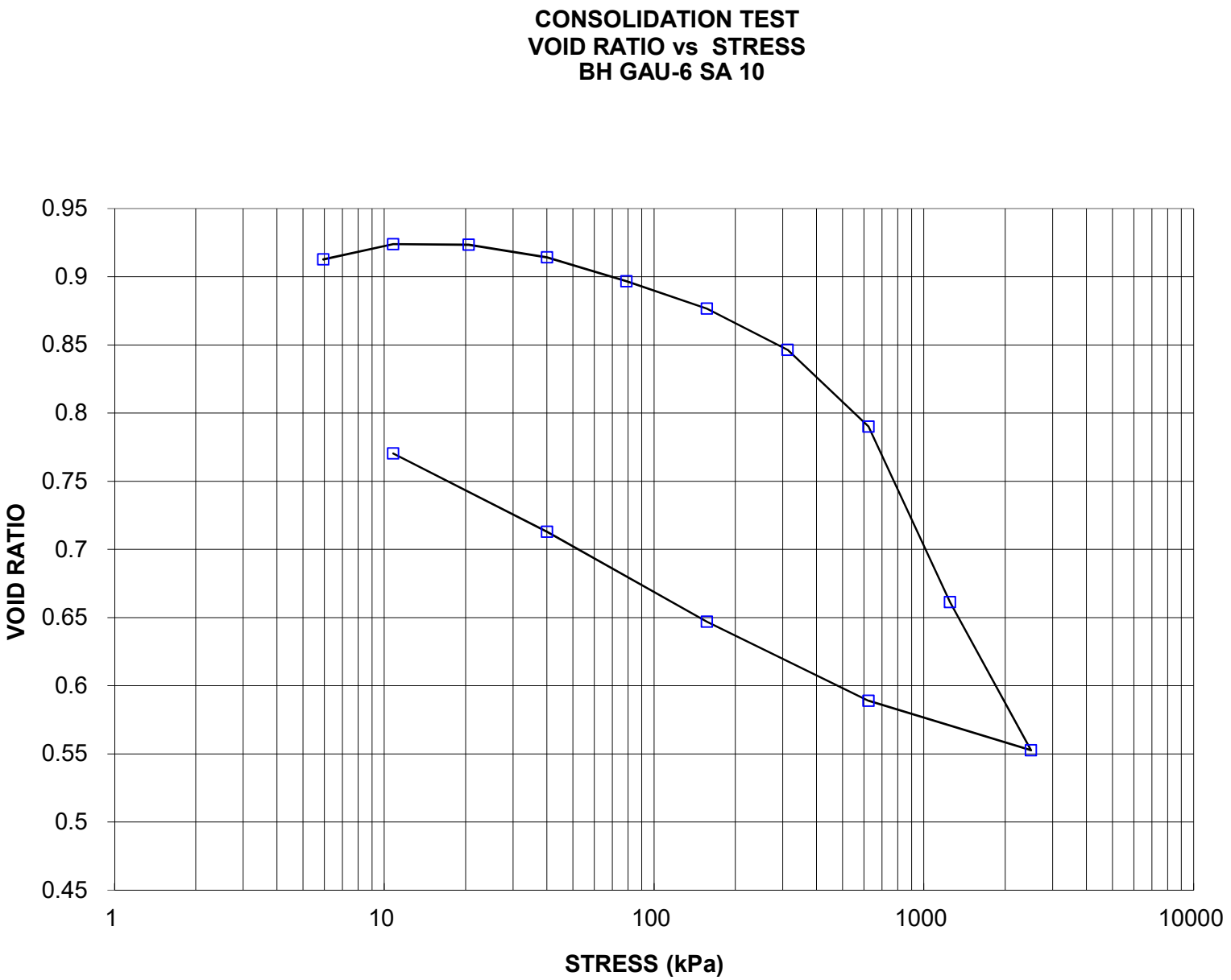


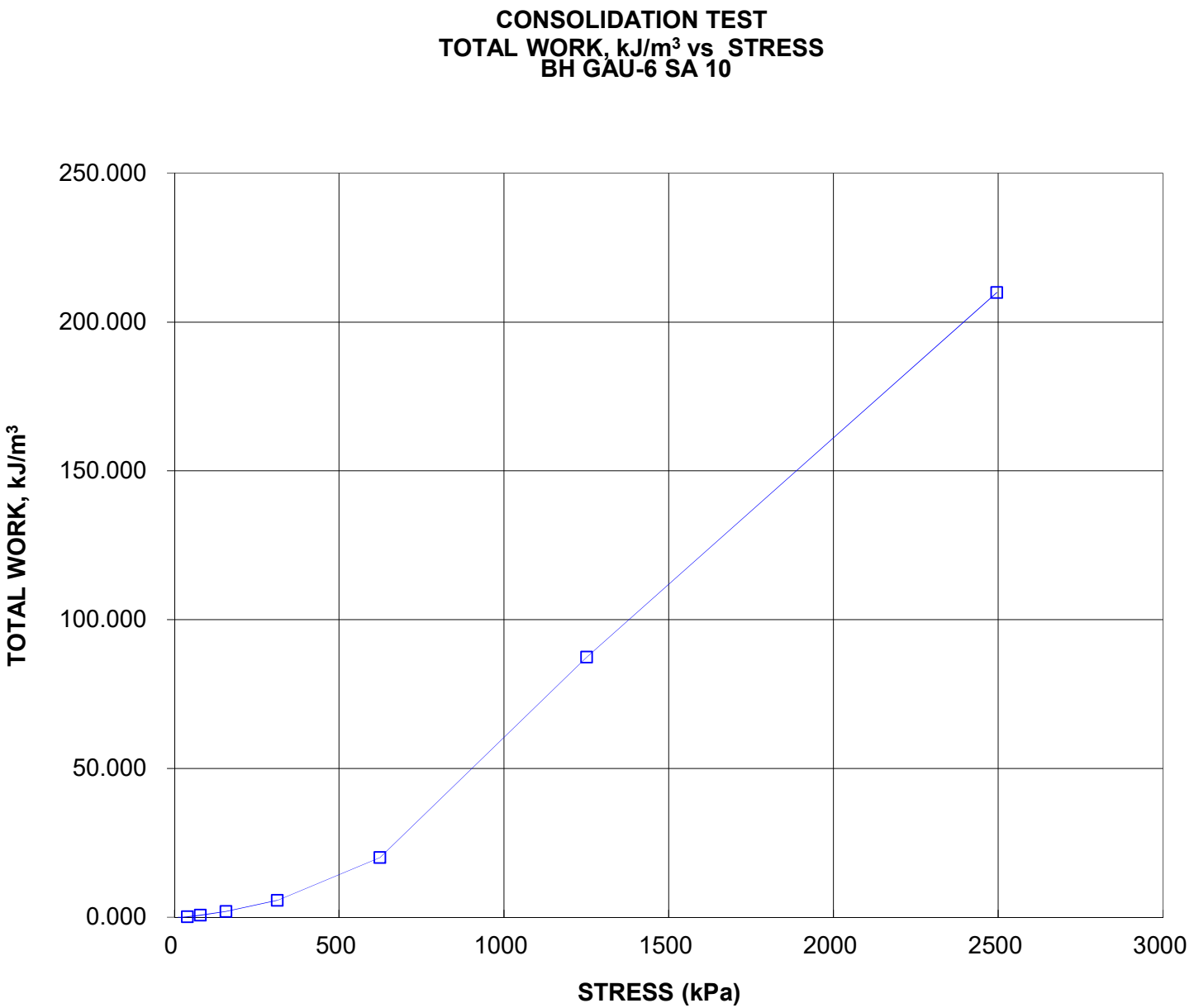
VOLUME COMPRESSIBILITY, m<sup>2</sup>/kN



HYDRAULIC CONDUCTIVITY, cm/s





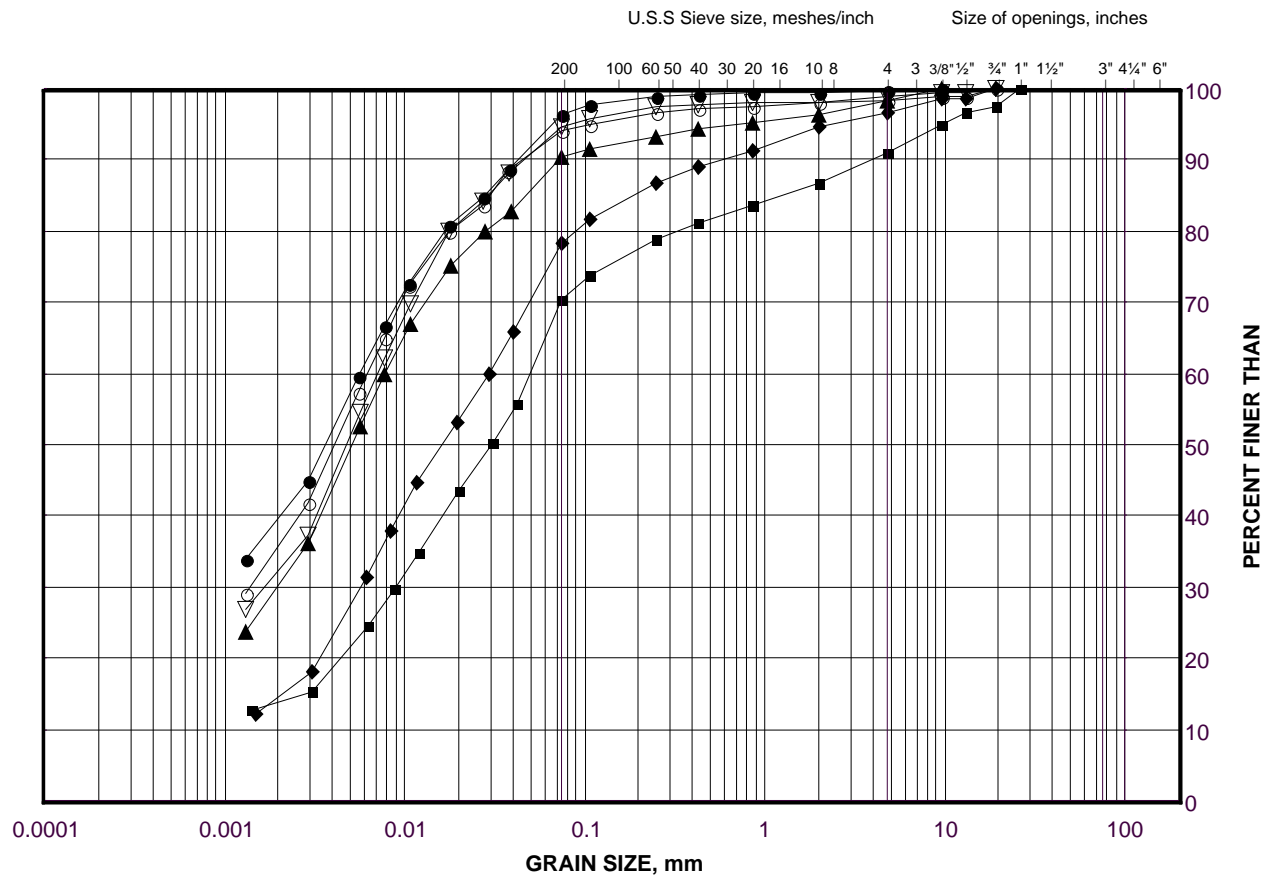




# GRAIN SIZE DISTRIBUTION

Clayey Silt to Clayey Silt Till

FIGURE B-8



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

## LEGEND

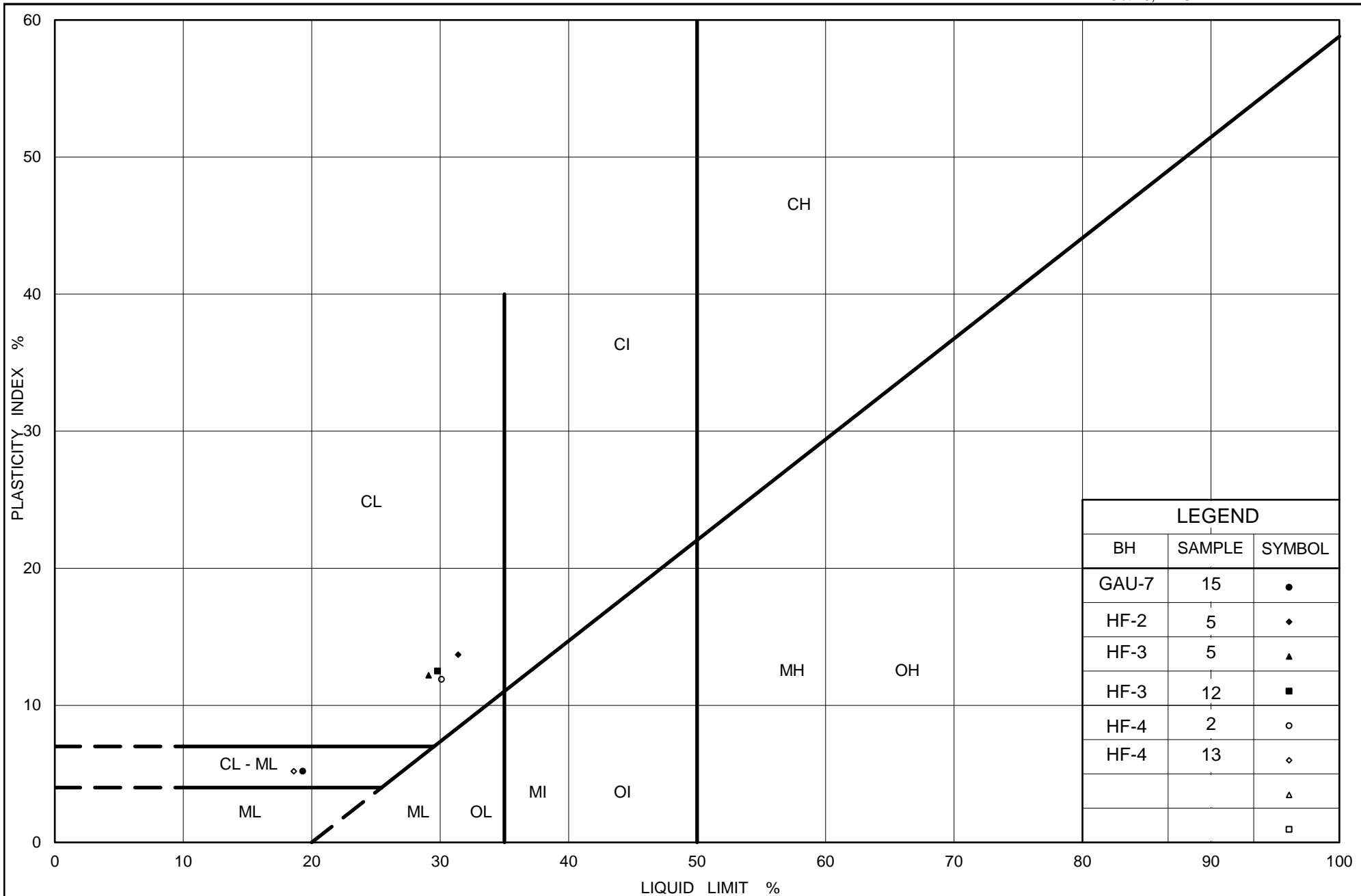
SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	HF-3	12	103.5
■	HF-4	13	99.3
◆	GAU-7	15	99.2
▲	HF-4	2	113.0
▽	HF-3	5	113.5
○	HF-2	5	114.0

Project Number: 1671430

Checked By: MA/LCC

**Golder Associates**

Date: 30-Apr-19



Ministry of Transportation

Ontario

# PLASTICITY CHART Clayey Silt to Clayey Silt Till

Figure No. B-9

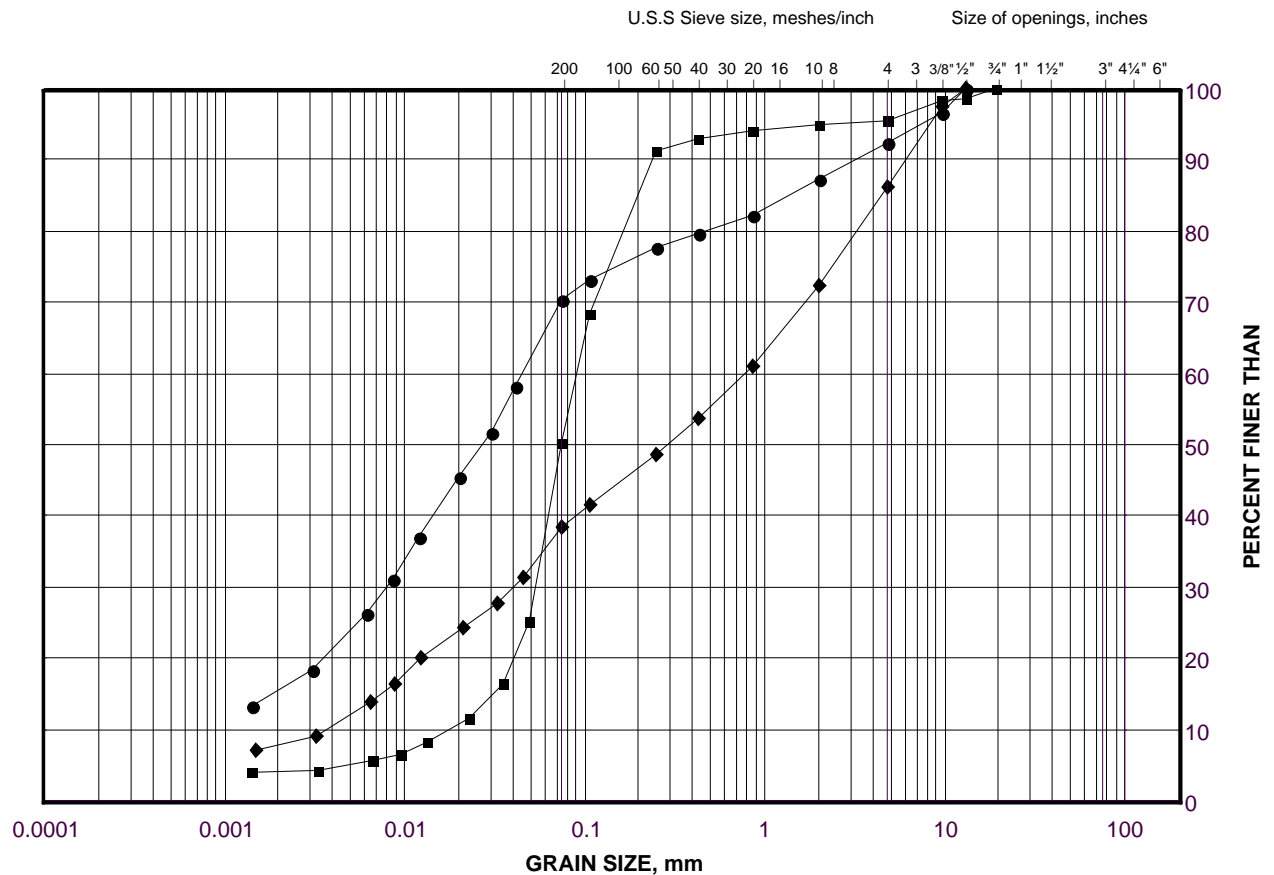
Project No. 1671430 (WO 002)

Checked By : MA/LCC

# GRAIN SIZE DISTRIBUTION

Sand and Silt

FIGURE B-10



## LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	GAU-3	16	98.8
■	GAU-7	17	94.5
◆	GAU-7	19	88.6

Project Number: 1671430

Checked By: MA/LCC

**Golder Associates**

Date: 30-Apr-19

January 22, 2019

Mr. Eric Naylor  
Golder Associates Ltd.  
6925 Century Avenue, Suite #100  
Mississauga, Ontario  
Canada L5N 7K2

Re: UCS testing  
(Golder Project No. 1671430 WO-2)

Dear Mr. Naylor:

On November 15, 2018 six (6) HQ-sized samples were received by Geomechanica Inc. via drop-off by Golder Personnel. These samples were identified as being from Golder project 1671430 WO-2 (QEW Niagara). From these samples, three (3) UCS tests were completed.

Details regarding the steps of specimen preparation and testing along with the test results and photographs of the test specimens before and after testing are presented in the accompanying laboratory report and spreadsheet.

Sincerely,



Bryan Tatone Ph.D., P. Eng.

Geomechanica Inc.  
Tel: (647) 478-9767  
Email: [bryan.tatone@geomechanica.com](mailto:bryan.tatone@geomechanica.com)

# Rock Laboratory Testing Results

**A report submitted to:**

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

**Prepared by:**

Bryan Tatone, PhD, PEng  
Omid Mahabadi, PhD, PEng  
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#900-390 Bay St.  
Toronto ON  
M5H 2Y2 Canada  
Tel: +1-647-478-9767  
lab@geomechanica.com

**January 22, 2019**

Project number: 1671430-W02

**Abstract**

This document summarizes the results of rock laboratory testing, including the results of 3 Uniaxial Compressive Strength (UCS) tests. These samples are from a drilling investigation for the QEW Niagara Project (Golder Project No. 1671430-W02). Results including uniaxial compressive strength (UCS) along with photographs of samples before and after testing are presented herein.

**In this document:**

1 Uniaxial Compressive Strength Tests	1
Appendices	3

# 1 Uniaxial Compressive Strength Tests

## 1.1 Overview

This section summarizes the results of uniaxial compressive strength (UCS) testing of HQ-sized specimens. The testing was performed in Geomechanica's rock testing laboratory using a 150 ton (1.3 MN) Forney loading frame equipped with pressure-compensated control valve to maintain an axial displacement rate of approximately 0.100 mm/min (Figure 1). The preparation and testing of each specimen included the following:

1. Unwrapping of the core sample, inspecting it for damage, and re-wrapping it in electrical tape to minimize exposure to moisture during subsequent specimen preparation.
2. Diamond cutting of core sample to obtain cylindrical specimens with an appropriate length (length:diameter = 2:1) and nearly parallel end faces.
3. Diamond grinding of specimen to obtain flat (within  $\pm 0.025$  mm) and parallel end faces (within  $0.25^\circ$ ).
4. Placing specimen into the loading frame, applying a 1 kN axial load, and removing the electrical tape.
5. Axially loading the specimens to rupture while continuously recording axial force and axial deformation to determine the peak strength (UCS).



Figure 1: Forney loading frame setup for uniaxial compression testing.

Using a precision V-block mounted on the magnetic chuck of the surface grinder, test specimens met the end flatness, end parallelism, and perpendicularity criteria set out in ASTM D4543-08. The side straightness criteria, as checked with a feeler gauge, was met for all samples and the minimum length:diameter criteria was met for all specimens unless noted otherwise in Table 1. Testing of the specimens followed ASTM D7012-14 Method C with the following exceptions:

- Rather than a spherical seat diameter equal to 1 to 2 times the specimen diameter, the setup used here employed a 25.4 mm diameter high precision ball bearing and seat. Despite the smaller diameter, this seat could move freely to accommodate small angular rotations in any direction, as needed, and therefore did not appreciably influence the results.

## 1.2 Results

The testing results are summarized in Table 1. Please note that additional specimen details and measurements are provided in the summary spreadsheet that accompanies this report.

Table 1: Summary of Uniaxial Compression test results.

Sample	Depth (m)	Bulk density $\rho$ (g/cm <sup>3</sup> )	UCS (MPa)	Lithology	Failure description
GAU-3	32.03 - 32.23	2.659	13.7	Red mudstone	1
GAU-5	29.64 - 29.80	2.659	25.7	Red mudstone with green reduction zone	1
GAU-7	32.62 - 32.85	2.669	24.5	Red mudstone	1
Average		2.663	21.3		
Standard deviation		0.005	5.4		

<sup>1</sup> Axial splitting failure

## 1.3 Specimen photographs

Photographs of the specimens prior to and after testing are presented in the Appendix.



# Appendices

## Specimen sheets



- GAU-3
- GAU-5
- GAU-7





### Uniaxial Compression Test

Client	Golder Associates Ltd.	Project	1671430-W02												
Sample	GAU-3	Depth	32.03 - 32.23												
<div>Specimen parameters</div> <table><tr><td>Diameter (mm) <sup>a</sup></td><td>63.14</td></tr><tr><td>Length (mm) <sup>a</sup></td><td>125.56</td></tr><tr><td>Bulk density <math>\rho</math> (g/cm<sup>3</sup>)</td><td>2.659</td></tr><tr><td>UCS (MPa)</td><td>13.7</td></tr><tr><td>Lithology</td><td>Red mudstone</td></tr><tr><td>Failure description <sup>b</sup></td><td>1</td></tr></table>		Diameter (mm) <sup>a</sup>	63.14	Length (mm) <sup>a</sup>	125.56	Bulk density $\rho$ (g/cm <sup>3</sup> )	2.659	UCS (MPa)	13.7	Lithology	Red mudstone	Failure description <sup>b</sup>	1	<div>Prior to testing</div> 	<div>After testing</div> 
Diameter (mm) <sup>a</sup>	63.14														
Length (mm) <sup>a</sup>	125.56														
Bulk density $\rho$ (g/cm <sup>3</sup> )	2.659														
UCS (MPa)	13.7														
Lithology	Red mudstone														
Failure description <sup>b</sup>	1														
<div><sup>a</sup> Additional specimen measurement/details provides in accompanying summary spreadsheet.</div> <div><sup>b</sup> Failure description: <sup>1</sup> Axial splitting failure;</div>															
Remarks:															
Performed by	BSAT	Date	2018-12-18												

### Uniaxial Compression Test

Client	Golder Associates Ltd.	Project	1671430-W02
Sample	GAU-5	Depth	29.64 - 29.80
Specimen parameters		Prior to testing	After testing
Diameter (mm) <sup>a</sup>	62.84		
Length (mm) <sup>a</sup>	125.14		
Bulk density $\rho$ (g/cm <sup>3</sup> )	2.659		
UCS (MPa)	25.7		
Lithology	Red mudstone with green reduction		
Failure description <sup>b</sup>	1		
<sup>a</sup> Additional specimen measurement/details provides in accompanying summary spreadsheet.			
<sup>b</sup> Failure description: <sup>1</sup> Axial splitting failure;			
Remarks:			
Performed by	BSAT	Date	2018-12-18

## Uniaxial Compression Test

Client	Golder Associates Ltd.	Project	1671430-W02												
Sample	GAU-7	Depth	32.62 - 32.85												
<div><div>Specimen parameters</div><table><tr><td>Diameter (mm) <sup>a</sup></td><td>63.22</td></tr><tr><td>Length (mm) <sup>a</sup></td><td>127.52</td></tr><tr><td>Bulk density <math>\rho</math> (g/cm<sup>3</sup>)</td><td>2.669</td></tr><tr><td>UCS (MPa)</td><td>24.5</td></tr><tr><td>Lithology</td><td>Red mudstone</td></tr><tr><td>Failure description <sup>b</sup></td><td>1</td></tr></table></div>		Diameter (mm) <sup>a</sup>	63.22	Length (mm) <sup>a</sup>	127.52	Bulk density $\rho$ (g/cm <sup>3</sup> )	2.669	UCS (MPa)	24.5	Lithology	Red mudstone	Failure description <sup>b</sup>	1	<div>Prior to testing</div> 	<div>After testing</div> 
Diameter (mm) <sup>a</sup>	63.22														
Length (mm) <sup>a</sup>	127.52														
Bulk density $\rho$ (g/cm <sup>3</sup> )	2.669														
UCS (MPa)	24.5														
Lithology	Red mudstone														
Failure description <sup>b</sup>	1														
<div><div><sup>a</sup> Additional specimen measurement/details provides in accompanying summary spreadsheet.</div><div><sup>b</sup> Failure description: <sup>1</sup> Axial splitting failure;</div></div>															
Remarks:															
Performed by	BSAT	Date	2018-12-18												



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