



# Foundation Investigation and Design Report

*Airport Road Connection Structure*

*QEW/Glendale Avenue Interchange Improvements*

*Niagara-on-the-Lake, Ontario*

*MTO GWP 2423-15-00*

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# Table of Contents

## PART A – FOUNDATION INVESTIGATION REPORT

<b>1.0 INTRODUCTION .....</b>	<b>1</b>
<b>2.0 SITE DESCRIPTION .....</b>	<b>1</b>
<b>3.0 INVESTIGATION PROCEDURES .....</b>	<b>1</b>
<b>4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS.....</b>	<b>3</b>
4.1 Regional Geology.....	3
4.2 General Overview of Subsurface Conditions .....	3
4.2.1 Topsoil.....	4
4.2.2 Asphalt .....	4
4.2.3 Fill (Including Former Structure Foundations) .....	4
4.2.4 Silty Clay to Clay .....	5
4.2.5 Clayey Silt .....	6
4.2.6 Boulder.....	7
4.2.7 Silt and Sand to Silt.....	7
4.2.8 Lower Clayey Silt Till/Residual Soil.....	7
4.2.9 Shale Bedrock.....	7
4.3 Groundwater Conditions .....	8
4.4 Analytical Testing Results.....	9
<b>5.0 CLOSURE .....</b>	<b>10</b>

## PART B – FOUNDATION DESIGN REPORT

<b>6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS .....</b>	<b>11</b>
6.1 General.....	11
6.2 General Foundation Design Context.....	11
6.2.1 Consequences and Level of Understanding .....	11
6.2.2 Seismic Design .....	12
6.2.2.1 Seismic Site Classification .....	12
6.2.2.2 Spectral Response Values and Seismic Performance Category .....	12

6.3	Foundations Options .....	12
6.4	Shallow Foundations.....	13
6.4.1	Founding Elevations and Frost Protection Requirements .....	13
6.4.2	Geotechnical Resistances .....	14
6.4.3	Resistance to Lateral Loads .....	14
6.5	Driven Steel H-Piles or Pipe Piles.....	14
6.5.1	Founding Elevations and Geotechnical Resistances.....	14
6.5.2	Downdrag and Drag Loads .....	15
6.5.3	Resistance to Lateral Loads .....	16
6.5.4	Group Action .....	18
6.6	Lateral Earth Pressures for Design of Abutments.....	18
6.6.1	Static Lateral Earth Pressures for Design.....	19
6.6.2	Seismic Lateral Earth Pressures for Design .....	19
6.7	Embankment Design.....	20
6.7.1	Global Stability .....	20
6.7.1.1	Method of Analysis.....	21
6.7.1.2	Parameter Selection .....	21
6.7.2	Settlement.....	22
6.7.2.1	Method of Analysis.....	22
6.7.2.2	Parameter Selection .....	22
6.7.2.3	Settlement Performance .....	23
6.7.3	Results of Analyses .....	23
6.7.3.1	Global Stability .....	23
6.7.3.2	Settlement.....	23
6.7.3.3	Settlement Mitigation – Preloading .....	24
6.8	Retained Soil System (RSS) Walls .....	26
6.8.1	Founding Level.....	26
6.8.2	Geotechnical Resistances .....	26
6.8.3	Frost Protection.....	27



6.8.4	Resistance to Lateral Loads/Sliding Resistance.....	27
6.8.5	Global Stability .....	27
6.8.6	Settlement .....	27
6.9	Analytical Testing of Construction Materials .....	27
6.10	Construction Considerations .....	28
6.10.1	Excavation and Control of Groundwater and Surface Water.....	28
6.10.3	Former Bridge Substructure.....	29
6.10.4	Obstructions .....	30
6.10.5	Footing Subgrade Protection .....	30
6.10.6	Embankment Construction and Erosion Protection .....	30
6.10.7	Preloading and Settlement Monitoring.....	30
<b>7.0</b>	<b>CLOSURE .....</b>	<b>31</b>

## REFERENCES

### TABLES

Table 1	Comparison of Foundation Alternatives – Airport Road Connection Structure
Table 2	Summary of Foundation Engineering Parameters – North and South Approach Embankments

### DRAWINGS

Drawing 1	Borehole Locations and Soil Strata
Drawing 2	Soil Strata

### FIGURES

Figure 1	Summary Plot of Engineering Parameters for Cohesive Deposits
Figure 2A	Global Stability of 2H:1V Embankment Side Slope – Short-Term (Undrained) Condition
Figure 2B	Global Stability of 2H:1V Embankment Side Slope – Long-Term (Effective Stress) Condition
Figure 3A	Global Stability of Concrete Cantilever Retaining Wall – Short-Term (Undrained) Condition
Figure 3B	Global Stability of Concrete Cantilever Retaining Wall – Long-Term (Effective Stress) Condition
Figure 4A	Global Stability of RSS Wall – Short-Term (Undrained) Condition
Figure 4B	Global Stability of RSS Wall – Long-Term (Effective Stress) Condition

## APPENDICES

### APPENDIX A – Former Structure Information

#### APPENDIX B – Borehole Records

Lists of Symbols and Abbreviations  
Lithological and Geotechnical Rock Description Terminology  
Records of Boreholes ARB-1 to ARB- 4 and HF-1

**APPENDIX C – Geotechnical Laboratory Test Results**

Figure C-1	Grain Size Distribution – Silty Clay to Clay Fill and Sand and Gravel Fill
Figure C-2	Plasticity Chart – Silty Clay to Clay Fill
Figure C-3A-B	Grain Size Distribution – Clayey Silt to Silty Sand
Figure C-4A-B	Plasticity Chart – Clayey Silt to Silty Sand
Figure C-5A-D	Consolidation Test Summary – Sample GAU-2, Sample 16
Figure C-6A-D	Consolidation Test Summary – Sample GAU-5, Sample 13
Figure C-7A-D	Consolidation Test Summary – Sample GAU-6, Sample 10
Figure C-8	Grain Size Distribution – Clayey Silt
Figure C-9	Plasticity Chart – Clayey Silt
Figure C-10	Grain Size Distribution – Silt and Sand to Silt
Figure C-11	Plasticity Chart – Silt
Figure C-12	Rock Core Photographs

Rock Laboratory Test Result Report

**APPENDIX D – Analytical Laboratory Test Results**

# PART A

FOUNDATION INVESTIGATION REPORT  
AIRPORT ROAD CONNECTION STRUCTURE  
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 Airport Road connection structure for the QEW/Glendale Avenue 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 Drawing 1.

This report addresses the foundation investigation carried out to support the design of the new Airport Road connection structure under Glendale Avenue. This report was developed based on the results from a 2018 investigation and laboratory testing.

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 present in the northeast quadrant of the interchange.

Prior to 2007, a three-span bridge supported on spread footings was present along the existing Glendale Avenue, to carry Glendale Avenue over a then-existing North Service Road, on a similar alignment to the proposed Airport Road connection structure. Based on the available information, the abutments for this former structure were founded at approximately Elevation 116.8 m, and the pier footings were founded at approximately Elevation 112.5 m. In 2007, the bridge superstructure was demolished, with the former bridge span infilled with earth fill material. Based on the demolition contract drawings provided by Niagara Region, it is understood that the former abutment and pier footings, abutment foreslopes and the former road structure were left in place. Information regarding this former bridge structure and its demolition is included in Appendix A.

The proposed new Airport Road connection from Glendale Avenue will implement a new S-W loop ramp in the northeast quadrant of the interchange. The ramp will be connected to Airport Road via a single-span structure that allows the ramp to pass under Glendale Avenue. The new connector road will be 9.25 m wide to carry two lanes of traffic. The existing ground surface at the proposed structure site varies from approximately Elevation 117 m to 118 m, and the approximately 4 m high Glendale Avenue embankment (with its grade at about Elevation 121.0 m to 121.5 m) is present at the western end of the proposed structure.

## 3.0 INVESTIGATION PROCEDURES

The field work for the Airport Road connection structure investigation was carried out between July 23 and August 24, 2018, and between September 17 and October 30, 2018. During this time a total of five boreholes (designated as Boreholes ARB-1 to ARB-4, and HF-1) were advanced within the footprint of the proposed Airport Road connection structure. The borehole locations are shown on Drawing 1.

Boreholes ARB-1 to ARB-4 and HF-1 were drilled using 152 mm outer diameter hollow-stem augers by a CME-55 track-mounted drill 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 with an automatic hammer in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586)<sup>1</sup>.

Borehole ARB-1 was advanced through the Glendale Avenue embankment to a depth of 4.0 m and encountered auger refusal. Boreholes ARB-2 to ARB-4 were advanced to depths of 29 m to 36.3 m below existing ground surface, including bedrock coring in Boreholes ARB-2 and ARB-3. Boreholes HF-1 was advanced to a depth of 16.5 m. Boreholes details are provided in the borehole and drillhole records in Appendix B.

The groundwater conditions in the open boreholes were observed during and immediately following the drilling operations. A standpipe piezometer was installed in Borehole ARB-4 to permit monitoring of the water level. The installed piezometer consists of a 50 mm diameter PVC pipe, with a 1.5 m slotted screen within a filter sand pack sealed within the clayey silt deposit about 2 m above the bottom of the borehole. 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 ARB-1 to ARB-3 and HF-1 were backfilled to ground surface with bentonite, and the upper 200 mm of Boreholes ARB-1 and ARB-2 were sealed to the roadway surface with cold patch asphalt 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 in 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 in geographic (latitude / longitude) coordinates, the ground surface elevations and borehole drilled depths are summarized below.

Borehole No.	Foundation Element	MTM NAD83		Ground Surface Elevation (m)	Borehole Depth (m)
		Northing (m) (Latitude)	Easting (m) (Longitude)		
ARB-1	North Abutment	4,779,855.5 (43.158062)	332,139.8 (-79.163817)	121.0	4.6
ARB-2	South Abutment	4,779,841.8 (43.157938)	332,133.3 (-79.163898)	121.5	36.3
ARB-3	North Abutment	4,779,841.4 (43.157933)	332,172.2 (-79.163420)	117.7	33.2

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

Borehole No.	Foundation Element	MTM NAD83		Ground Surface Elevation (m)	Borehole Depth (m)
		Northing (m) (Latitude)	Easting (m) (Longitude)		
ARB-4	South Abutment	4,779,827.2 (43.157806)	332,176.8 (-79.163365)	117.1	29.0
HF-1	Centre	4,779,836.5 (43.157890)	332,159.9 (-79.163572)	116.5	16.5

## 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) 2 and *Urban Geology of Canadian Cities* (Menzies and Taylor, 1998) 3.

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.

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 boreholes of the current investigation 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 B. 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 from current investigation are based on use of an automatic hammer and the values are reported with

<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.

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 also presented on the laboratory test figures in Appendix C. The results of the analytical testing are provided in Appendix D.

The stratigraphic boundaries shown on the borehole records and on the stratigraphic profile 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 soils encountered consist of surficial layers of topsoil and asphalt, underlain by non-cohesive and cohesive fill material; one of the boreholes encountered the substructure that was left in place from the former North Service Road bridge on Glendale Avenue, which was demolished in 2007. The fill materials are underlain by a silty clay to clay deposit (consisting of a very stiff to hard crust and becoming firm to stiff with depth) which is in turn underlain by deposits of generally hard clayey silt, hard clayey silt to silty clay till, and/or dense to very dense sand and silt, overlying shale bedrock.

#### 4.2.1 Topsoil

A 50 mm to 102 mm thick layer of topsoil was encountered from ground surface in Boreholes ARB-3, ARB-4 and HF-1. The topsoil was classified based on visual and textural observations; organic content testing was not carried out during the current investigations.

#### 4.2.2 Asphalt

Boreholes ARB-1 and ARB-2 were advanced through the Glendale Avenue embankment and encountered between 120 mm and 150 mm of asphalt at ground surface.

#### 4.2.3 Fill (Including Former Structure Foundations)

Fill was encountered underlying the topsoil or asphalt at all borehole locations advanced for the proposed structure. Borehole ARB-1 terminated within the fill at a depth of 4.6 m due to auger refusal on a concrete obstruction; the last sample in the borehole also contained concrete fragments. After review of historic documentation, it was determined that Borehole ARB-1 had been advanced along the former North Service Road alignment, and encountered the substructure of a former bridge that was demolished under a 2007 Niagara Region contract. Based on the demolition drawings, the abutment and pier footings and abutment slope were left in place when this former bridge was decommissioned, and the bridge span infilled with earth fill. Information regarding this former bridge structure and its demolition is contained in Appendix A.

In general, a thick layer of non-cohesive fill is present below the asphalt in Borehole ARB-2 and below the cohesive fill in Borehole ARB-1 on the east abutment of Airport Road. This fill ranges from 1.2 m to 3.6 m, and consists of sand and gravel, some silt and trace clay. An 800 mm thick layer of non-cohesive fill was also encountered at a depth of 5.6 m in Borehole ARB-2. The SPT “N”-values measured within these non-cohesive fill layers range from 5 blows to 65 blows per 0.3 m of penetration, indicating a variable, loose to very dense relative density.

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

The majority of the fill at the site is cohesive, generally consisting of silty clay, but also varying to clay to clayey silt, containing trace to some sand and gravel, along with trace rootlets. The cohesive fill layer is approximately 0.9 m to 4.2 m in thickness, and extends to approximately Elevation 115.9 m to 114.1 m in the boreholes. The SPT “N”-values measured within the cohesive fill range from 6 blows to 29 blows per 0.3 m of penetration, indicating a firm to very stiff consistency.

The results of grain size distribution tests completed on one sample of the cohesive fill and one sample of the non-cohesive fill are presented on Figure C-1 in Appendix C. Atterberg limits testing was carried out on three samples of the cohesive fill and measured liquid limits about 39 to 50 per cent, plastic limits ranging from about 19 to 22 per cent, and plasticity indices of about 20 to 28 per cent, indicating that the fines portion of the fill has intermediate plasticity as presented on the plasticity chart on Figure C-2 in Appendix C. The water content measured in the fill materials ranges from 19 per cent to 26 per cent, and field observations indicate moist to wet conditions.

#### 4.2.4 Silty Clay to Clay

An extensive cohesive deposit consisting of silty clay to clay was encountered below the fill in Boreholes ARB-2, ARB-3, ARB-4 and HF-1. The surface of this deposit was encountered at depths ranging from about 1.4 m to 6.4 m (between about Elevation 115.1 m and 114.1 m). The thickness of the deposit ranges from about 13.7 m to 17.0 m where it was fully penetrated, and the deposit extends to depths between about 16.5 m and 20.1 m (between Elevation 101.4 m and 97.6 m) below existing ground surface.

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

- The SPT “N”-values recorded within the cohesive upper “crust” of this deposit, above approximately Elevation 108 m, range from 5 blows to 30 blows per 0.3 m of penetration but are generally between about 10 blows and 15 blows per 0.3 m of penetration, indicating a firm to hard consistency. In situ vane tests carried out within this upper portion of the deposit measured undrained shear strengths greater than 96 kPa. The in situ field vane test results together with the SPT “N”-values indicate that the upper crust has a predominantly very stiff consistency.
- The SPT “N”-values measured within the lower portion of the deposit generally range from 0 blows (weight of hammer) to 12 blows per 0.3 m of penetration, suggesting a soft to stiff consistency. In situ vane tests carried out within the lower portion of this deposit measured undrained shear strengths generally ranging from about 24 kPa to greater than 96 kPa, but typically greater than 80 kPa. The sensitivity generally ranges from about 1.3 to 3.7. The in situ field vane tests results together with the SPT “N”-values indicate that the lower portion of the silty clay to clay deposit has a predominantly stiff consistency.

The results of grain size distribution testing completed on thirteen samples are shown on Figures C-3A and C-3B in Appendix C. The deposit generally contains trace to some sand and trace to some gravel with trace sand seams; a varved structure was observed within the lower portion of the deposit in some of the boreholes.

Atterberg limits testing carried out on thirteen samples of the cohesive deposit measured liquid limits ranging from about 28 per cent to about 55 per cent, plastic limits ranging from about 16 per cent to about 23 per cent, and plasticity indices ranging from about 11 per cent to about 31 per cent. These results indicate that the deposit predominantly consists of silty clay to clay of intermediate to high plasticity as presented on the plasticity charts on Figures C-4A and C-4B in Appendix C; however, as shown on these figures, two tested samples are classified as



clayey silt of low plasticity. The natural water contents measured on samples of this 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 thin-walled Shelby tubes in Borehole GAU-2, GAU-5 and GAU-6, which were drilled as part of the 2018 investigations for the Glendale Avenue underpass replacement located immediately south of this structure. The results from these three tests have been considered in selecting the design parameters for this cohesive deposit at the Airport Road connection structure.

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 1.2 to 1.7. Details of the test results are shown on Figures C-5A-D, C-6A-D and C-7A-D in Appendix C, 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	$3.7 \times 10^{-3}$ $3.1 \times 10^{-3}$

**Notes:**

- 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

## 4.2.5 Clayey Silt

A cohesive deposit comprised of clayey silt to sandy clayey silt was encountered underlying the silty clay to clay deposit in Boreholes ARB-2 to ARB-4. The cohesive deposit is generally varved (typically comprised of silty clay with thin clayey silt and silt laminae), but also includes homogenous zones of silty clay, and in Borehole ARB-3 this deposit has a till-like appearance. The surface of this cohesive deposit was encountered at depths between about 17.1 m and 20.1 m below ground surface (Elevations 101.4 m to 97.6 m). This deposit is approximately 2.9 m to 6.0 m thick as encountered in the boreholes, extending to about Elevation 95.8 m to 94.0 m.

The SPT "N"-values measured within the upper portion of this clayey silt deposit in Boreholes ARB-2 and ARB-4 are 2 blows and 12 blows per 0.3 m of penetration, indicating a soft to stiff consistency; in situ vane tests carried out immediately below the 2-blow sample measured undrained shear strengths of about 75 kPa to 85 kPa and sensitivity values of about 1.6 and 2.0, indicating that this upper portion of the deposit is generally stiff. In the

remainder of the deposit, the SPT “N” values range from 70 blows to greater than 100 blows (e.g., 50 blows per 0.13 m of penetration, and 50 blows per 0.08 m of penetration), indicating a hard consistency.

The results of grain size distribution tests completed on three samples of the cohesive deposit are shown on Figure C-8 in Appendix C.

Atterberg limits tests were carried out on three samples of this cohesive deposit and measured liquid limits ranging from about 21 per cent to 31 per cent, plastic limits ranging from about 13 per cent to 18 per cent and plasticity indices ranging from about 8 per cent to 14 per cent. The results of the Atterberg limits tests are shown on the plasticity charts on Figure C-9 in Appendix C, and indicate that the cohesive deposit can be classified as clayey silt of low plasticity.

#### **4.2.6 Boulder**

A granite boulder was encountered in Borehole ARB-2 near the base of the clayey silt deposit, above the silt and sand to silt deposit. The boulder was penetrated by coring for 0.2 m. Although not encountered in other boreholes, cobbles and boulders should be anticipated in the glacially-derived soils at this site.

#### **4.2.7 Silt and Sand to Silt**

A silt and sand to silt deposit was encountered underlying the clayey silt deposit in Boreholes ARB-2 to ARB-4. This deposit varies in composition from silt and sand, to sandy silt, to silt containing trace to some sand, and trace to some clay. The surface of this deposit was encountered at depths ranging from about 23.0 m to 25.7 m (between Elevations 95.6 m and 94.0 m) and the deposit extends to depths of about 28.9 m to 32.6 m (approximately Elevation 88.8 m to 88.9 m). The deposit thickness varies from about 5.7 m to 5.9 m.

The SPT “N”-values measured within this deposit range from 54 blows to greater than 100 blows per 0.3 m of penetration, indicating a very dense compactness condition.

The results of grain size distribution testing carried out on three samples of this deposit are shown on Figure C-10 in Appendix C. Atterberg limits testing was carried out on one sample of the silt deposit and measured a liquid limit of about 21 per cent, plastic limit about 18 per cent, and a plasticity index about 3 per cent. The results of the Atterberg limits test are shown on a plasticity chart on Figure C-11 in Appendix C, and indicate that the silt deposit can be classified as having slight plasticity. The natural water content measured in samples of this deposit ranges from about 9 to 20 per cent.

#### **4.2.8 Lower Clayey Silt Till/Residual Soil**

A layer of clayey silt till/residual soil was encountered below the silt deposit and immediately overlying shale bedrock in Borehole ARB-3. The surface of this layer was encountered at a depth of 28.9 m (Elevation 88.8 m), and the layer is about 1.1 m thick. The deposit is described as containing some sand and some shale fragments.

One SPT “N”-value of 50 blows for 0.05 m of penetration was measured within this layer, suggesting a hard consistency.

#### **4.2.9 Shale Bedrock**

Bedrock was encountered in Boreholes ARB-2 and ARB-3, below the silt and sand to silt or clayey silt residual soil deposits, respectively. The depth to bedrock below ground surface and the corresponding bedrock surface elevations at each borehole location (from north to south) are summarized below.

Borehole No.	Depth to Bedrock Surface (m)	Bedrock Surface Elevation (m)	Comments
ARB-2	32.6	88.9	Bedrock cored for 3.7 m
ARB-3	30.0	87.7	Bedrock cored for 3.2 m

In general, the bedrock surface as encountered in the boreholes advanced in the area of the proposed Airport Road connection structure varies from about Elevation 87.7 m to 88.9 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 to fresh, 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 on the drillhole records in Appendix B. Bedrock core photographs are included in Appendix C.

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 ranges from 52 per cent to 100 per cent, with the lowest RQD of 52 per cent measured in the first core run below the bedrock surface in Borehole ARB-2. The RQD values indicate a rock mass of fair to excellent quality, as classified 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 52 per cent and 100 per cent, respectively.

Uniaxial compression (UC) tests (ASTM D7012) were carried out on selected core samples from boreholes advanced at the Glendale Avenue underpass immediately to the south of the Airport Road connector structure location. 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 C.

Borehole Number	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-5 (Run #2)	29.6 – 29.8	87.2 to 87.0	25.7	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). This bedrock formation contains moderately strong interlayers of limestone/dolostone, and such conditions should be expected even where such interlayers have not been specifically encountered in the boreholes.

### 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 B, but do not represent the stabilized groundwater level at the site. A piezometer was installed in Borehole ARB-4, and the measured groundwater level is summarized below.

Borehole No.	Ground Surface Elevation (m)	Depth to Groundwater (m)	Groundwater Elevation (m)	Date
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

This measured groundwater level at approximately Elevation 109 m is associated with the lower silt and sand to silt deposit. Based on piezometers installed at the Glendale Avenue underpass site immediately to the south, the groundwater table within the silty clay to clay deposit was measured to be approximately Elevation 111 m to 112 m. 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.

#### 4.4 Analytical Testing Results

Two selected samples from Boreholes ARB-2 and ARB-3 were assessed for potential corrosivity of the soil from ; this data has been supplemented with testing results on three selected samples from boreholes at the Glendale Avenue underpass site immediately to the south. Detailed analytical test results are included in Appendix D and the test results are summarized below:

Borehole Number / Sample Number	pH	Resistivity (ohm-cm)	Electrical Conductivity ( $\mu\text{mho/cm}$ )	Chloride ( $\mu\text{g/g}$ )	Soluble Sulphates ( $\mu\text{g/g}$ )
GAU-2 / 11	7.82	390	2570	51	3400
GAU-5 / 6	7.97	460	2160	120	2600
GAU-6 / 11	7.99	390	2150	49	2900
ARB-2 / 5	7.78	1600	643	77	400
ARB-3 / 6	8.03	560	1790	22	2800

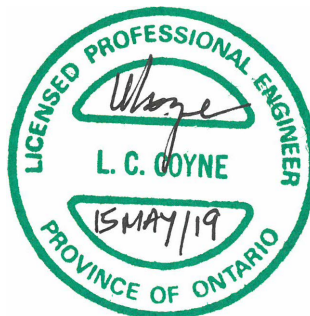
## 5.0 CLOSURE

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  
AIRPORT ROAD CONNECTION STRUCTURE  
QEW / GLENDALE AVENUE INTERCHANGE IMPROVEMENTS  
NIAGARA-ON-THE-LAKE, ONTARIO  
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 proposed Airport Road connection structure associated with the Glendale Avenue underpass replacement and interchange development in the Town of Niagara-on-the-Lake, Regional Municipality of Niagara. 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 designer with sufficient information to assess the feasible foundation alternatives and carry out the detail design of the replacement structure foundations.

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. The contractor 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 the aspects of construction must make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling, and the like.

It is understood that the new Airport Road connection from Glendale Avenue entails a new ramp in the northeast quadrant of the interchange. This ramp will provide access to Airport Road through a bridge structure under Glendale Avenue, located north of the proposed Glendale Avenue underpass. The proposed bridge will be a single-span concrete rigid frame structure with a 9.25 m span length between the abutments, and with an overall width of approximately 37 m to carry the lanes of the diverging diamond interchange above. The Glendale Avenue embankment will be approximately 4 m in height relative to the existing ground surface in this quadrant, and the new connection ramp will require a cut below the existing ground surface. Retaining walls are required in all four quadrants of the proposed structure; these walls are typically 8 m in length, except in the southwest quadrant where an approximately 33 m long wall is required.

The proposed Airport Road connection structure is located along a similar alignment to a former three-span bridge that carried Glendale Avenue over the former North Service Road. In 2007, this structure was demolished; based on the demolition contract drawings provided by Niagara Region, it is understood that the former abutment and pier footings, abutment foreslopes and the former road structure were left in place (see Appendix A). Based on available information, these abutments were founded at approximately Elevation 116.8 m, and the pier footings were founded at approximately Elevation 112.5 m. These elements are anticipated to be encountered within excavations for the proposed Airport Road connection structure and associated retaining walls.

### 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 underpass structure and its foundation system may be classified as having medium to high traffic volumes and their performance as having potential impacts on other transportation corridors, resulting in a “typical consequence level” associated with exceeding limit states design.

Based on the level of foundation investigation completed at this site in comparison to the degree of site understanding in Section 6.5 of *CHBDC (2014)*, the level of confidence for design for the Airport Road overhead structure has been assessed as “typical degree of site and prediction model understanding” based on having two boreholes near each foundation element.

The corresponding 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.

## 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 Airport Road overhead structure.

### 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)
PGA (g)	0.074	0.139	0.240
PGV (m/s)	0.053	0.094	0.164
Sa (0.2) (g)	0.113	0.208	0.364
Sa (0.5) (g)	0.072	0.123	0.212
Sa (1.0) (g)	0.040	0.064	0.104
Sa (2.0) (g)	0.019	0.030	0.048
Sa (5.0) (g)	0.0040	0.0066	0.012
Sa (10.0) (g)	0.0016	0.0025	0.004

## 6.3 Foundations Options

Based on the proposed structure/embankment geometry and the subsurface conditions at this site, both shallow and deep foundation options have been considered for support of the new abutments and associated retaining walls for the Airport Road connection structure. For all options, temporary protection systems are expected to be required along the east side of the existing Glendale Avenue embankment to facilitate the staged construction of the new bridge and retaining wall foundations, as well as the removal of the former structure foundations where these conflict with the new works.



- **Shallow foundations:** The eastward shift of Glendale Avenue will require placement of approximately 4 m to 5 m of fill at the north approach, and approximately 7 m of fill at the south approach. This loading will result in settlement of approximately 65 mm to 85 mm below the north and south approach embankments, which will affect the settlement performance of shallow foundations if adopted. In addition, the factored serviceability geotechnical resistance for shallow foundations is relatively low for 25 mm of settlement under the foundation loading. However, as the connection structure is located “offline” within the northeast quadrant of the interchange, if the structure area can be preloaded for a period of 30 to 60 days (for the north approach) and 90 to 120 days (for the south approach) prior to removal of the fills and construction of the connection structure and its associated retaining walls, the settlement performance and geotechnical resistances will be improved. This preload period will reduce the post-preload settlement under the embankment and structure foundation loading to less than 10 mm at the north approach, and to less than 25 mm at the south approach, as discussed further in Section 6.7. As the earthworks associated with this preloading alternative do not interfere with the existing operation and traffic staging on the QEW or Glendale Avenue, and provided that the construction schedule permits, this is the preferred option from a geotechnical/foundations perspective. If the construction schedule does not permit this period of time for preloading, additional settlement mitigation options including surcharging and/or the use of lightweight fill may be employed.
- **Driven pile foundations:** Steel H-piles or pipe piles driven to/into the shale bedrock are considered feasible for support of the new abutments. Driven piles would permit design of conventional and semi-integral abutments (for H-piles or pipe piles), or integral abutments (generally for H-piles). This option would be preferred from a geotechnical perspective if settlement mitigation measures such as preloading, surcharging and/or the use of lightweight fill cannot be employed within the available construction timeline to facilitate the shallow foundation option.

A summary of the advantages, disadvantages and risks for each foundation option, from a geotechnical/foundations perspective, is presented in Table 1 following the text of this report.

## 6.4 Shallow Foundations

### 6.4.1 Founding Elevations and Frost Protection Requirements

Strip footing (shallow) foundations may be feasible for the support of the new Airport Road connection structure and associated retaining walls, likely with settlement mitigation as discussed above and in Section. The footing should be founded on the generally very stiff crust of the silty clay to clay deposit, below any fill or softened/loosened soils. In order to extend below the existing fill, the footings should be founded at or below Elevation 114.0 m for both abutments; however, it is anticipated that deeper founding elevations will be required to extend below the ramp cut grade, unless the footings are perched in the approach embankments in conjunction with a longer structure span length. It is recommended that the footings be maintained as high as possible (i.e., with depth only to satisfy frost protection requirements below the Airport Road connection grade) to minimize the influence of the footing loading below the cohesive “crust”.

Spread/strip footings should be founded at a minimum depth of 1.2 m for frost protection as per OPSD 3090.101 (*Frost Penetration Depths for Southern Ontario*). If adequate soil cover cannot be provided for the footing or pile cap, rigid styrofoam insulation could be installed to compensate for the lack of soil cover and provide protection from frost penetration. However, if the depth of embedment for the footings is less than 1.2 m, the factored ultimate and serviceability geotechnical resistances would have to be re-evaluated.

At this level, the foundations are expected to be maintained above the groundwater level at the site; further, the silty clay to clay soils have relatively low permeability and groundwater seepage is anticipated to be relatively minor.

The founding on silty clay deposit will be susceptible to loosening and disturbance due to precipitation, water seepage and construction traffic. It is recommended that a 100 mm thick concrete working slab/mud mat be placed on the subgrade within four hours to protect the integrity of the bearing stratum.

#### 6.4.2 Geotechnical Resistances

Strip footings placed on the native soils founded at or below the design elevations given in the preceding section should be designed based on the factored ultimate geotechnical resistances and factored serviceability geotechnical resistances (for 25 mm of settlement) given below. It is noted that the factored serviceability geotechnical resistance values are predicated on the application of preloading or other settlement mitigation measures, as the magnitude of settlement under the footings due to the structure an approach embankment loading would otherwise be excessive and pose greater risk to the structure performance.

Footing Width (m)	Factored Ultimate Geotechnical Resistance (kPa)	After Preloading or Other Settlement Mitigation
		Factored Serviceability Geotechnical Resistance (kPa) (for 25 mm of Settlement)
4	400	250
5	500	220
6	650	200

The geotechnical resistances should be reviewed if the selected footing width or founding elevations differ from those given above. As illustrated above, the factored serviceability geotechnical resistances can be increased if preloading of the structure area is completed prior to bridge construction.

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

#### 6.4.3 Resistance to Lateral Loads

Resistance to lateral forces/sliding resistance between the cast-in-place concrete strip footings and the subgrade should be calculated in accordance with Section 6.10.5 of the *CHBDC* (2014). The unfactored coefficient of friction ( $\tan \phi'$ ) between the cast-in-place concrete strip footing and the native soils as interpreted from Naval Facility Engineering Command (NAVFAC, 1986) is as follows:

- Cast-in-place footing or cast-in-place concrete working slab to native deposits:  $\tan \phi' = 0.58$

### 6.5 Driven Steel H-Piles or Pipe Piles

#### 6.5.1 Founding Elevations and Geotechnical Resistances

Driven steel H-piles or pipe piles may be founded on/within the weathered shale bedrock, although there is a possibility that the piles will terminate above the bedrock surface within the very dense silt and sand deposit (as encountered in Borehole ARB-2 near the west end of the site).

The design pile tip elevation and the factored ultimate and serviceability geotechnical resistances that may be used for the design of steel HP 310x110 or HP 310x132 piles are presented below, based on the piles penetrating a nominal distance into the bedrock; the piles may terminate at a higher elevation than the estimated design pile tip elevation given in this table. The same values apply to closed end, concrete filled 324 mm (12 3/4 inch) diameter steel pipe piles having a minimum wall thickness of 9.5 mm (3/8 inch).

Bedrock Surface Elevation (m)	Estimated Design Pile Tip Elevation (m)	Founding Stratum	Factored Ultimate Geotechnical Resistance	Factored Serviceability Geotechnical Resistance (for 25 mm of settlement)
87.7-88.9	87.0-88.0	Shale bedrock	1,400 kN	-- <sup>1</sup>

**Note:**

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

If and where the piles terminate above the bedrock within the very dense (100-blow") silt and sand deposit as encountered in Borehole ARB-2, a factored ultimate geotechnical resistance of 1,300 kN may be used for design; the factored serviceability resistance for 25 mm of settlement will be greater than the factored ultimate geotechnical resistance in this case also, and as such, the SLS condition does not apply.

Drag loads will be imposed on the steel H-piles if they are driven prior to construction of the approach embankments, as discussed in Section 6.5.2.

The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile and length of pile; the criteria must therefore be established at the time of construction after the piling equipment is known to ensure that the piles are not overdriven and to avoid possible damage to the piles. In addition to Hiley testing, pile dynamic analyzer (PDA) testing should be completed on at least 10% of piles or two piles (whichever is greater) at each foundation element in each stage of construction to verify the ultimate resistance.

Consideration must be given to the potential presence of cobbles and boulders within the glacially-derived soils at this site. In this regard, steel H-piles are preferred over steel tube piles given that steel tubes are considered to pose a slightly higher risk of "hanging up" or being deflected from their vertical or battered orientation during installation, due to their larger end area. The piles should be reinforced at the tip for protection during driving to reduce the potential for damage to the piles. Driving shoes (such as Titus Standard "H" Bearing Pile Points) are preferred over flange plates and should be installed in accordance with OPSP 3001.100 Type II. The requirement for driving shoes should be included in the Contract Drawings. All pile installation/driving should be carried out in accordance with OPSS.PROV 903 (*Deep Foundations*).

## 6.5.2 Downdrag and Drag Loads

As a result of the loading from the new approach embankments along the new Glendale Avenue alignment, elastic compression of the very stiff "crust", and long-term consolidation settlement of the underlying cohesive deposit will occur. The difference in the vertical movement between the overburden and the piles (i.e., from the elastic deformation of the piles under the load from the bridge and from the movement of the piles into the moderately weathered bedrock below the pile tip) will likely result in the development of negative skin friction on the piles and downdrag. Consequently, if the piles are installed prior to the construction of the approach embankments, drag loads will need to be considered in the assessment of the pile's structural capacity.

Analysis to estimate drag loads and geotechnical resistances for the recommended pile foundation option at the abutments was carried out in accordance with Section 6.11.4.10 of the *Commentary to the CHBDC (2014)* using the method proposed by Briaud and Tucker (1994). It is noted that the method used to assess the deformation of the piles and the associated drag loads is dependent on a number of factors including the pile length, foundation conditions at the pile tip, the unfactored dead load on the pile and the anticipated settlement profile of the foundation soils. The proposed dead load on the pile should be confirmed by the structural designer and the estimated drag loads reassessed if necessary.

The calculated drag loads at the neutral plane for the piles at this structure site are as follows:

Pile Type	Approximate Pile Length (m)	Estimated Depth from Ground Surface to Neutral Plane (m)	Calculated Drag Load at Neutral Plane
Steel HP 310x110 pile	30.0	17 m	900 kN

**Note:** Drag load does not include dead load on the pile. Estimated drag loads should be re-assessed based on the actual structural load on the piles.

If the piles for the abutments are installed prior to the construction of the approach embankments, then in accordance with the requirements of the *Canadian Foundation Engineering Manual (2006)*, an assessment is required to be carried out to estimate if the structural capacity of the steel H-pile would be exceeded when taking into account the factored dead load combined with the factored drag load. For this site, the factored dead load plus the factored drag load may exceed the structural capacity of an HP 310x110 pile.

Several measures to mitigate downdrag may be considered such as the use of a heavier pile section (e.g., HP 310x132) with a higher structural capacity to accommodate the drag load, application of bitumen coating to the piles to reduce the drag load (although it is anticipated that this would not be cost effective for the relatively small number of piles required on this contract), and preloading and/or surcharging or the use of light-weight fill to mitigate settlements and eliminate the drag load. Further details on preloading are provided in Section 6.7.3, but in summary, preloading for a period of 30 to 60 days with conventional weight fill at the north abutment and approach will limit the post-preload, post-construction settlement to less than 10 mm, while preloading for a period of 90 to 120 days at the south abutment and approach would still yield approximately 20 mm of post-preload, post-construction settlement. Based on these analyses, it is anticipated that the downdrag may be able to be mitigated at the north abutment via preloading alone, but that at the south abutment, heavier pile sections are likely to be required in addition to preloading. The DB-ready designer and design-builder will need to confirm the remaining settlement for their available preloading period, and then confirm whether a requirement for a heavier pile section is indeed applicable.

### 6.5.3 Resistance to Lateral Loads

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

Where ground conditions are generally competent and the lateral loads on piles are relatively small such that the maximum lateral pile deflections will be relatively small, the resistance to lateral loading in front of a single pile can

be estimated using subgrade reaction theory (as outlined below). However, it should be noted that the response of a pile to lateral loads is highly nonlinear and methods that assume linear behavior (such as subgrade reaction theory) are only appropriate where the maximum pile deflections are less than 1 percent of the pile diameter, where the loading is static (no cycling) and where the pile material is linear (CFEM, 2006). Where these conditions are not met, the non-linear lateral behavior of the soil should be considered by the use of P-y curves.

The resistance to lateral loading in front of a single pile may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction,  $k_h$ , (kPa/m) is based on the following equations based on the Canadian Foundation Engineering Manual (CFEM, 1992 as referenced in the *Commentary of the CHBDC, 2014*).

For non-cohesive soils:

$$k_h = \frac{n_h z}{B} \quad \text{where} \quad \begin{array}{l} n_h \text{ is the constant of subgrade reaction (kPa/m);} \\ z \text{ is the depth (m); and} \\ B \text{ is the pile diameter or width (m).} \end{array}$$

For cohesive soils:

$$k_h = \frac{67 s_u}{B} \quad \text{Where} \quad \begin{array}{l} s_u \text{ is the undrained shear strength of the soil (kPa); and} \\ B \text{ is the pile diameter or width (m).} \end{array}$$

The following values of  $n_h$  and  $s_u$  (Terzaghi, 1995) may be incorporated into the calculations of horizontal subgrade reaction ( $k_h$ ) for structural analyses for a single vertical pile, based on the interpreted stratigraphic profiles for each of the foundation elements shown on Drawing 2. The ranges in values reflect the variability in the subsurface conditions, the soil properties and the approximate nature of the analysis and the non-linear nature of the soil behaviour (such that  $k_h$  is a function of deflection).

Soil Unit	$n_h$ (kPa/m)	$s_u$ (kPa)
Generally very stiff silty clay to clay "crust" above Elevation 108 m	-	150
Stiff silty clay to clay below "crust", between Elevation 108 m and 99 m	-	100
Hard clayey silt between Elevation 99 m and 95 m	-	400
Very dense silt and sand to silt (below the water table) below Elevation 95 m to shale bedrock at approximately Elevation 88 m	13,000	-

Both the structural and geotechnical resistances of the piles should be evaluated to establish the governing case at Ultimate Limit States (ULS). At Serviceability Limit States (SLS), the horizontal reaction of the piles will be controlled by deflections and the horizontal resistance of the pile should be calculated based on the coefficient of horizontal subgrade reaction ( $k_h$ ) of the soil as discussed above. The SLS reaction should be taken corresponding to a horizontal deflection of 10 mm at the underside of the pile cap for units supporting the abutments (*CHBDC (2014) Commentary Section 6.11.2.2*).

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

#### 6.5.4 Group Action

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters between rows of driven steel H-piles. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading by a reduction factor,  $R$  (NAVFAC DM-7.2, 1986) as follows:

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

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

### 6.6 Lateral Earth Pressures for Design of Abutments

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

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

- Select, free draining granular fill meeting the specifications of OPSS.PROV 1010 (Aggregates) Granular 'A' or Granular B Type II, should be used as backfill behind the walls. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS.PROV 501 (Compacting).
- A minimum compaction surcharge of 12.0 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with CHBDC (2014) Section 6.12.3 and Figure 6.6. Hand-operated compaction equipment should be used to compact the backfill soils immediately behind the walls as per OPSS.PROV 501. Other surcharge loadings should be accounted for in the design, as required.
- For restrained walls, granular fill should be placed in a zone with the width equal to at least 1.2 m behind the back of the wall on Figure C6.20(a) of the Commentary to the CHBDC (2014). For unrestrained walls, fill should be placed within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing on Figure C6.20(b) of the Commentary to CHBDC (2014).

### 6.6.1 Static Lateral Earth Pressures for Design

The following guidelines and recommendations are provided regarding the lateral earth pressures for static (i.e., not earthquake) loading conditions. These lateral earth pressures assume that the ground above the wall will be flat, not sloping. For sloping conditions above the wall, then lateral earth pressures will need to be re-calculated based on Mononobe-Okabe (M-O) theory.

For a restrained wall, the pressures are based on the fill behind the granular backfill zone, and the following parameters (unfactored) may be used assuming the use of earth fill or Select Subgrade Material (SSM):

Material	Earth Fill or SSM
Soil Unit Weight:	20 kN/m <sup>3</sup>
Coefficients of static lateral earth pressure:	
Active, $K_a$	0.33
At rest, $K_o$	0.50

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

Material	Granular 'A'	Granular 'B' Type II
Soil Unit Weight:	22 kN/m <sup>3</sup>	21 kN/m <sup>3</sup>
Coefficients of static lateral earth pressure:		
Active, $K_a$	0.27	0.27
At rest, $K_o$	0.43	0.43

- If the wall does not allow lateral yielding (i.e., restrained structure where the rotational or horizontal movement is not sufficient to mobilize an active earth pressure condition), at-rest earth pressures (plus any compaction surcharge) should be assumed for geotechnical design.
- If the wall support and superstructure allow lateral yielding, active earth pressures may be used in the geotechnical design of the structure. The movement required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.12.1 and Table C6.6 of the Commentary to CHBDC (2014).

### 6.6.2 Seismic Lateral Earth Pressures for Design

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

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

In accordance with Sections 4.6.5 and C.4.6.5 of the CHBDC (2014) and its Commentary, for structures which allow lateral yielding, the horizontal seismic coefficient,  $k_h$ , used in the calculation of the seismic active pressure coefficient, is taken as 0.4 to 0.5 times the site-specific PGA. For structures that do not allow lateral yielding,  $k_h$  is



taken as equal to the site-specific PGA. For both cases the value of the vertical seismic coefficient  $k_v$  is taken as zero.

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

	Design Earthquake	Site PGA	Seismic Active Pressure Coefficients, $K_{AE}$		
			Granular A	Granular B Type II	Earth Fill
Yielding Wall	475-Yr	0.074g	0.27	0.27	0.30
	975-Yr	0.139g	0.29	0.29	0.32
	2,475 Yr	0.266g	0.33	0.33	0.37
Non-Yielding Wall	475-Yr	0.074g	0.29	0.29	0.32
	975-Yr	0.139g	0.33	0.33	0.37
	2,475 Yr	0.266g	0.44	0.44	0.49

The  $K_{AE}$  value for a yielding wall is applicable provided that the wall can move up to  $250k_h$  mm, where  $k_h$  is the site adjusted horizontal PGA as given in the table above. This corresponds to displacements of 10 mm, 18 mm, and 34 mm for the 475-year, 975-year, and 2,475-year design earthquakes at this site.

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

## 6.7 Embankment Design

The proposed Glendale Avenue grade above the structure will be at approximately Elevation 122.4 m, requiring placement of up to about 4 m to 5 m of new fill at the south approach, and 6 m to 7 m of new fill at the north approach.

### 6.7.1 Global Stability

The following subsections outline the method used to evaluate static global stability of the proposed approach embankments. The geotechnical soil parameters used in the analyses are also presented. The results of the stability analyses are presented in Section 6.7.3 where they are discussed together with the results of the settlement analyses and recommendations regarding possible design and construction alternatives to mitigate post-construction settlement.



### 6.7.1.1 Method of Analysis

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. 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).

### 6.7.1.2 Parameter Selection

The simplified stratigraphy together with the foundation engineering parameters for the different soil types encountered at the site are summarized in Table 2. The following is a summary of the embankment slope inclination, unit weight and effective friction angle for new earth fill and new granular fill modelled in the slope stability analyses.

Fill Type	Maximum (Steepest) Slope Inclination	Unit Weight, $\gamma$	Effective Friction Angle, $\phi'$	Effective Cohesion
SSM Fill <sup>1</sup>	2H:1V	20 kN/m <sup>3</sup>	32°	0
Granular Fill <sup>1</sup>	2H:1V	21 kN/m <sup>3</sup>	45°	0

**Note:**

1. The overall strength of the SSM fill is lower compared to the granular fill. As such, approach embankments constructed predominantly with SSM fill represent the worst-case scenario in terms of global slope stability of the embankments. All slope stability figures presented in this report illustrate embankments using SSM fill, where feasible.

For the non-cohesive soils present at this site, the effective stress parameters employed in the analysis were estimated from empirical correlations based on the results of the in situ Standard Penetration Tests (SPT). 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.

For cohesive deposits, total stress parameters were employed in the analyses assuming 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 assessed based on the results of in situ field vane shear tests, inferred from the laboratory consolidation test results, and estimated from correlations with the SPT results and other laboratory test data (i.e., natural water content, liquid limit, etc.), where appropriate. A plot of the undrained shear strength versus elevation is shown on Figure 1.

Effective stress parameters were also employed to evaluate the stability of the embankments based on long-term, drained conditions (i.e., permanent conditions). The effective stress parameters (i.e., effective friction angle ( $\phi'$ ) and effective cohesion ( $c'$ )) for the cohesive deposits were estimated from empirical correlations based on the plasticity index. The correlations proposed by Mitchell (1993), Kulhawy and Mayne (1990), and Ladd et al. (1977) were employed and the results were adjusted by engineering judgment based on precedent experience in similar soil conditions.

For the purpose of the stability analysis, the groundwater level was assumed to be at Elevation 112 m, based on the groundwater levels measured in piezometers installed within the silty clay to clay deposit at the Glendale Avenue underpass site immediately to the south.

### 6.7.2 Settlement

The following subsections outline the methods used to carry out the settlement analyses at the proposed approach embankments for the realigned Glendale Avenue. The results of the analyses are presented in Section 6.7.3, where they are discussed together with the results of the stability analyses and recommendations regarding potential design and construction alternatives to mitigate stability issues and/or post-construction settlement, where applicable.

#### 6.7.2.1 Method of Analysis

To estimate the magnitude of expected settlement, analyses were carried out at the north and south approach embankments. 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.

The sources of settlement at this site to include the following:

- Immediate settlement of the existing fill and the very stiff to hard “crust” portion of the silty clay to clay deposit (short-term);
- Primary time dependent consolidation of the firm to stiff portion of the cohesive deposits (using Terzaghi's one-dimensional consolidation theory long-term); and,
- Secondary time dependent (creep) consolidation of the firm to stiff portions of the cohesive deposits (long-term).

The thickness of the compressible foundation soils and the height of the approach embankments vary both along Glendale Avenue, and along the proposed Airport Road connection structure (as the west limit of the structure and western retaining walls is located within the footprint of the existing Glendale Avenue embankment). As such, the settlements along the length of the alignment will similarly vary. The settlement magnitudes estimated from the settlement analyses represent the maximum anticipated values at the maximum embankment height at/immediately behind the abutments. Where the embankment height decreases the amount of settlement will be less than that presented herein, relative to the critical embankment height which is immediately behind the abutments.

#### 6.7.2.2 Parameter Selection

The simplified stratigraphy together with the elastic deformation and time-rate consolidation parameters, where applicable, employed for the different soil types encountered at the site are summarized in Table 2. The parameters associated with the extensive cohesive deposit encountered at the site are presented on Figure 2 and are based on the graphical presentation of this data. The upper cohesive deposit was sub-divided into two layers based on the preconsolidation stress (function of OCR), void ratio, and compression index/recompression index.

The immediate compression of the non-cohesive deposits (i.e., sand and gravel fill) were modelled by estimating an elastic modulus of deformation based on the corrected 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 C6.9.3.6 of the *Commentary to the CHBDC* (2014) and adjusted, if necessary.

The consolidation settlement of the cohesive deposits was assessed using the results of the laboratory consolidation test, 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,  $c_v$  ( $\text{cm}^2/\text{s}$ ), 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.

In addition to primary consolidation within the cohesive deposits, secondary compression may also occur. Secondary compression is referred to as creep settlement and occurs over a long period of time, after full dissipation of excess pore pressure under a constant stress.

For the purpose of the settlement analyses, the groundwater level was assumed to be at Elevation 112 m, based on the groundwater levels measured in piezometers installed within the silty clay to clay deposit at the Glendale Avenue underpass site immediately to the south.

### 6.7.2.3 Settlement Performance

The settlement performance criterion for design of approach embankments is in accordance with MTO's "Embankment Settlement Criteria for Design", dated July 2, 2010. In general, embankments approaching structural elements such as a bridge abutment are to be designed as follows:

- Total settlements and differential settlement rates are not to exceed 25 mm, over a 15-year period following completion of construction for a highway.

## 6.7.3 Results of Analyses

### 6.7.3.1 Global Stability

The stability analyses for the approach embankments indicate that after completion of construction, the approximately 5 m to 7 m high Glendale Avenue embankments will have a Factor of Safety of greater than 1.5 during the long-term/permanent condition 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 1.3 is achieved in the short-term (undrained) condition. Figures 2A and 2B 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/existing fill and any surficial organic/deleterious material and replacement with engineered fill.

The stability analyses for concrete retaining walls parallel to the Airport Road connection roadway indicate that a factor of safety of greater than 1.5 is achieved for these walls, provided that the footing width also satisfies overturning and sliding per the structural engineer's assessment. Based on input and feedback from AECOM, these analyses have been based on the critical case (maximum wall height) involving an approximately 6 m wide footing for a 6 m high retained height; Figures 3A and 3B illustrate the results of these static global stability analyses. The global stability for RSS walls, if and where adopted, is addressed in Section 6.8.3.

### 6.7.3.2 Settlement

Based on the results of the settlement analysis (with the topsoil, existing fill 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 5 m to 7 m high embankments at the north and south approaches, respectively, is summarized in the following table.

Approach	Total Settlement (mm)	Immediate Settlement (mm)	Consolidation Settlement (mm)
North Approach (Approximately 4 m to 5 m high)	65	30	35
South Approach (Approximately 6 m to 7 m high)	85	30	55

The magnitude of settlement will decrease toward the western portion of the proposed structure footprint, where the existing Glendale Avenue embankment will be excavated to construct the new structure. 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 grade raise for this portion of the structure site 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 at the north approach embankment and 90 to 120 days at the south approach embankment to reach a point beyond which 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 10 mm over a 15-year period following completion of construction.

To reduce the post-construction settlement of the approach embankment together with the connection structure and associated retaining wall foundations, the alternative mitigation option of preloading (as discussed in the following section) could be considered, assuming that the construction schedule can accommodate this timeline.

### 6.7.3.3 Settlement Mitigation – Preloading

In order to meet the settlement performance criterion, a minimum preload period of 30 to 60 days is recommended at the north approach embankment, and a minimum preload period of 90 to 120 days is recommended at the south approach embankment. The total factored post-construction consolidation settlement of the foundation soils under the loading imposed by the structure foundations and approach embankment is estimated to be less than 25 mm, as summarized below, provided that the connection structure, associated retaining walls and final approach embankments are constructed after this preloading period.

Approach	Preload Period (Days)	Remaining Factored Post-Construction Consolidation Settlement (mm)
North Approach (Approximately 4 m to 5 m high)	30-60	10
South Approach (Approximately 6 m to 7 m high)	90-120	20

If the construction schedule can accommodate this preload period, preloading the foundation soils for a duration of 30 to 60 days for the north abutment/approach area, and 90 to 120 days for the south abutment/approach areas, respectively is ranked as the preferred settlement mitigation option for this area.

The Factor of Safety for a 5 m to 7 m high preload embankment constructed with SSM will be greater than 1.3 for the temporary/short-term condition.

It is recommended that the magnitude and time-rate of settlement during and after construction of the preload embankment should be assessed by a monitoring program consisting of settlement plates, to confirm the end of the preload period.

#### 6.7.3.4 Alternate Settlement Mitigation Options – Surcharging or Lightweight Fill

If it is desirable to reduce the preloading period at the north and south approaches below the estimated 30-60 days and 90-120 days, respectively, consideration may be given to surcharging or incorporating lightweight fill. The use of wick drains could be considered at the south approach given the estimated duration of preloading in this area; however, the wick drain installation methods would have to address the presence of the stiffer silty clay to clay “crust”, and preloading and/or surcharging will still be required. If a wick drain solution is adopted, the design-builder will have to complete further assessment and detailed design of the wick drain spacing and resulting settlement mitigation impacts. Ground improvement is not recommended for this site, as the stiff zone to be “improved” is generally at a depth of 8 m to 15 m below the existing ground surface; the time and cost to complete such improvement will have limited return given the relatively small magnitudes of settlement estimated for this site.

If a 2 m surcharge is added to the abutment/approach areas, the following loading durations and remaining settlement will apply (assuming the use of SSM for the embankment fill):

Approach	Preload + Surcharge Period (Days)	Remaining Factored Post-Construction Consolidation Settlement (mm)
North Approach (Approximately 4 m to 5 m high)	15	5-10
South Approach (Approximately 6 m to 7 m high)	60	25

Alternatively, where lightweight fill is used for construction of the approach embankments (considering a 1 m thick layer pavement structure including the asphalt, subbase and base), the estimated maximum settlements are summarized below. Based on the estimated magnitude of consolidation settlement in the firm to stiff portion of the silty clay to clay deposit, some degree of preloading will still be required to achieve the post-construction settlement

objectives. The design-builder should assess the specific combination of lightweight fill materials and pavement structure, together with the time-rate of settlement, to achieve the desired timeline for construction.

Approach Embankment Area	Lightweight Fill Option	Total Settlement (mm)	Immediate Settlement (mm)	Consolidation Settlement (mm)
North approach (approx. 4 m to 5 m high)	Ultra-lightweight slag fill (bulk unit weight of approx. 11 kN/m <sup>3</sup> )	45	20	25
	Cellular concrete (assuming bulk unit weight of approx. 5 kN/m <sup>3</sup> )	35	15	20
South approach (approx. 7 m high)	Ultra-lightweight slag fill (bulk unit weight of approx. 11 kN/m <sup>3</sup> )	65	25	40
	Cellular concrete (assuming bulk unit weight of approx. 5 kN/m <sup>3</sup> )	50	20	30

## 6.8 Retained Soil System (RSS) Walls

### 6.8.1 Founding Level

The Airport Road connection structure will require retaining walls extending from the abutments parallel to the connector road, in all four quadrants of the structure. While concrete retaining walls are proposed in most quadrants, RSS walls are also feasible. For RSS walls adjacent to structures, design should follow the guidance provided in MTO Bridge Office Memorandum #2019-02, dated March 2019.

The front facing panels should be supported on a footing constructed on a granular pad. The granular pad should consist of a minimum thickness of 0.3 m of compacted Granular 'A' material, which should extend at least 1 m beyond the outside edge of both sides of the facing footing, then outward/downward at an inclination of 1H:1V.

### 6.8.2 Geotechnical Resistances

For the RSS facing panels supported on a 0.3 m wide footing constructed on a compacted granular pad as described in Section 6.8.1, the following geotechnical resistances may be used for design:

Foundation Unit	Factored Ultimate Geotechnical Resistance	Factored Serviceability Geotechnical Resistance for 25 mm of Settlement
RSS facing panel footing on a 0.3 m thick compacted Granular 'A' pad	200 kPa	N/A <sup>1</sup>

**Note:**

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

Assuming that the RSS walls act as a unit and utilize the full width of the reinforced soil mass, taken as 1.0 times the height of the wall to account for sloping ground above the wall where applicable, a factor of safety of 1.5 will be achieved for the global stability of the RSS wall system. Where the ground surface above and in front of the wall is level, the width of the reinforcing zone may be taken as 0.8 times the height of the wall to achieve the required factor of safety for global stability.

### 6.8.3 Frost Protection

Based on MTO's *RSS Design Guidelines* (2008), it is understood that the minimum soil cover to the underside of the levelling pad supporting the footings for the RSS facing panels should be at least 0.8 m or 40 per cent of the frost depth (i.e., 1.2 m at this site), whichever is greater; and the minimum soil cover to the top of the levelling pad should be at least 0.5 m. It is further understood that the levelling pad at the last step of the RSS wall should be provided with 1 m of soil cover or 50 per cent of the frost depth, whichever is greater.

However, if annual wall movements resulting from freeze thaw cycles must be minimized, consideration should be given to providing the footings with a minimum 1.2 m of soil cover for frost protection as per OPSD 3090.101 (*Frost Penetration Depths for Southern Ontario*), as measured vertically and perpendicular from the face of the abutment slope to the edge of the underside of the footing.

### 6.8.4 Resistance to Lateral Loads/Sliding Resistance

Resistance to lateral forces/sliding resistance between the compacted fill of the RSS wall mass and the subgrade should be calculated in accordance with Section 6.10.5 of the *CHBDC* (2014). The coefficient of friction ( $\tan \phi'$ ), between the compacted granular fill of the RSS wall and the native silty clay to clay may be taken as 0.58.

### 6.8.5 Global Stability

As discussed in Section 6.7.3, a target minimum Factor of Safety of 1.5 is considered appropriate for design of the retaining walls (including RSS walls) for global stability. The results of the stability analyses for RSS walls indicate that a Factor of Safety equal to or greater than 1.5 is achieved provided that the following minimum reinforcing strip lengths are incorporated in the wall design:

- 1.0 times the height of the wall to account for sloping ground above the wall where applicable; and
- 0.8 times the height of the wall where the ground surface both in front of and above the wall is level.

The results of static global stability analyses for the condition where level ground is present above and below the wall are shown on Figures 4A and 4B for the short-term and long-term (permanent) conditions, respectively.

These strip lengths must be specified on the Contract Drawings. The internal stability of a reinforced earth structure is to be designed and assessed by the proprietary product designer/manufacture to ensure that the internal stability of the walls is acceptable.

### 6.8.6 Settlement

The estimated factored settlement along the RSS walls is estimated to be less than 25 mm assuming the walls are constructed following completion of preloading of the approach embankments for a period of 60 days at the south approach and 120 days at the north approach embankment, as discussed in Section 6.7.3.3. The estimated magnitude of settlement is also based on the assumption that the RSS walls will be founded within the bridge approach embankments comprised of new compacted earth fill, SSM or granular fill as outlined in Section 6.10.5. However, if the RSS walls have to be constructed prior to employing the preloading period, consideration should be given to constructing a more flexible, two-stage construction RSS wall which is designed to tolerate higher post-construction settlements.

## 6.9 Analytical Testing of Construction Materials

Soil corrosivity may affect the concrete foundations and reinforced steel and other concrete elements buried in the soil. The long-term performance and durability of the foundations are directly related to their respective corrosion



resistance. Generally, the corrosivity potential to a structure depends on the soil resistivity / electrical conductivity, hydrogen ion concentration, and salts (chloride and sulphate) concentrations. The analytical results for the samples submitted for testing are summarized in Section 4.4 and the analytical laboratory test reports are included in Appendix D.

The analytical test results were compared to CSA A23.1 Table 3 (Additional requirements for concrete subjected to sulphate attack) to assess the potential severity of sulphate attack on concrete during its service life. The sulphate concentrations measured on the soil samples range between less than 0.002 per cent and 0.022 per cent, which is below the moderate degree of exposure (i.e., below the class S3 exposure limits). Therefore, based on the soil sample tested recovered from depth of about 10.6 m below ground surface, when the designer is selecting the exposure class for the concrete structure, the effects of sulphates from within the cohesive deposit in contact with the spread footing and any portion of the proposed structure constructed below the ground surface may not need to be considered.

The analytical test results of the soil samples were also compared to Table 7.1 (Relative Effect of Resistivity on Corrosion Potential/Aggressiveness (from NCHRP 1978)), as presented in the Federal Highway Administration/National Highway Institute Publication No. FHWA/NHI14007 (Federal Highway Administration, 2015), to assess the relative level of corrosion potential on buried steel in contact with soil. The resistivity values measured on the soil samples from Boreholes ARB-3, GAU-2, GAU-5 and GAU-6 drilled during the current investigations measured resistivity values of on the order of 2,600 ohm-cm to 3,400 ohm-cm. These results indicate a “moderately corrosive” to “very corrosive” potential. The sample from Borehole ARB-2 had a resistivity value of 400 ohm-cm which indicates a “low corrosive” potential. It is also noted that the measured pH level of 7.7 and 8.0, suggesting the presence of alkaline soils.

According to the Gravity Pipe Design Guidelines (MTO, 2014), the pH is not considered detrimental to concrete durability. However, the resistivity indicates that the soil corrosiveness is “moderate” ( $2,000 \text{ ohm-cm} < R < 4,500 \text{ ohm-cm}$ ), as per Table 3.2 of the Gravity Pipe Design Guidelines (MTO, 2014), and some level of corrosion protection should be applied to the foundation element / materials. Further, given that the foundations are located adjacent to the roadway shoulder and will be exposed to de-icing salt, consideration should be given to selection of a “C” type exposure class as defined by CSA A23.1 Table 1.

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

## 6.10 Construction Considerations

### 6.10.1 Excavation and Control of Groundwater and Surface Water

The foundation excavations at the abutments for the retaining wall will extend to depths of about 7 m below the Glendale Avenue through the existing fill and into the stiff to firm silty clay to clayey silt deposit.

Open-cut excavations must be carried out in accordance with the guidelines outlined in the most recent version of the Occupational Health and Safety Act and Regulation for Construction Activities. The existing fill materials are classified as Type 3 soils, while the native deposits are classified as Type 2 soils, according to the OHSA. Temporary excavations (i.e. those that are open for a relatively short time period) should be made with side slopes no steeper than 1H:1V.



It is expected that for construction staging, temporary protection systems will be required to facilitate the staged construction of the new ramps and wingwalls/retaining walls. Recommendations for temporary protection systems are provided in Section 6.10.5 below.

Excavations for the abutment foundations will be maintained above the groundwater level, which has been interpreted to be at approximately Elevation 111 m to 112 m, with the potential for seasonally higher water levels; groundwater inflow is expected to be relatively minor through the silty clay to clay deposit, especially during drier periods of the year. Some water inflow should be expected into the foundation excavations, particularly from water perched in granular fills on top of the silty clay to clay deposit or from seams/interlayers within the cohesive soils, and particularly during wet months; however, it is anticipated that water inflow can be handled by pumping from filtered sump pumps placed at the base of the excavations.

Surface water seepage into the excavations should be expected and will be heavier during periods of sustained precipitation and all surface water should be directed away from the excavations.

### 6.10.2 Temporary Protection Systems

To facilitate construction of the new structure foundations and removal of the former structure foundations (if required), temporary protection systems are expected to be required along the east side of the existing Glendale Avenue embankment. The temporary protection systems should be designed and constructed in accordance with OPSS.PROV 539 (*Temporary Protection Systems*). The lateral movement of the temporary protections systems should meet Performance Level 2 as specified in OPSS.PROV 539, provided that any existing adjacent structures or utilities can tolerate this magnitude of deformation.

Although the selection and design of the protection systems will be the responsibility of the contractor, it is considered that both a driven, interlocking sheet pile system and a soldier pile and lagging system are feasible at this site. However, the presence of the former bridge substructure (see Section 6.10.3) will impact the feasibility and construction methods for temporary protection systems.

The sheet piles or soldier piles will have to be driven or socketted to sufficient depth to provide the necessary passive resistance for the retained soil height, including any surcharge loads behind the protection system within at least a 1H:1V zone relative to the base of the excavation. Lateral support to the sheet piles or soldier piles could be provided in the form of rakers or temporary anchors.

### 6.10.3 Former Bridge Substructure

Prior to 2007, a three-span bridge supported on spread footings was present along a similar alignment to the proposed Airport Road connection structure. Based on the limited information on the previous bridge, the abutments were founded at approximately Elevation 116.8 m, and the pier footings were founded at approximately 112.5 m. In 2007 the bridge was demolished. Based on the demolition contract drawings provided by Niagara Region, the former abutment and pier footings, along with the abutment slopes and the former North Service Road structure were left in place and the road was decommissioned with the bridge structure filled in with earth fill material (see Appendix A). During the field investigation, Borehole ARB-1 encountered a concrete obstruction that was interpreted to be either a former footing or abutment foreslope based on auger refusal conditions and the concrete fragments encountered in the last sample. Based on an overlay of the former North Service Road and the proposed Airport Road connection structure alignment, it is likely that the old substructure will be encountered during construction (refer to the last figure in Appendix A).

The presence of this former substructure must be considered in the installation of temporary protection systems and excavation on this contract.

#### **6.10.4 Obstructions**

Cobbles and/or boulders were encountered and inferred due to difficulty to augering at varying depths in the boreholes drilled during the current subsurface investigation, which may affect the installation of steel H-piles if adopted, as well as temporary protection systems. If driven piles are adopted, it is recommended that driving shoes such as Titus Standard “H” Bearing Pile Point design as per OPSD 3000.100 be used on all steel H-piles to facilitate driving into the overburden soils.

#### **6.10.5 Footing Subgrade Protection**

The silty clay to clay that will be exposed at the subgrade level for strip footings will be susceptible to disturbance from construction traffic and ponded water. To limit this degradation, it is recommended that a concrete working slab be placed on the subgrade within four hours after preparation, inspection and approval of the footing subgrade.

#### **6.10.6 Embankment Construction and Erosion Protection**

Side slopes for the Glendale Avenue embankment adjacent to the connection structure should be no steeper than 2H:1V. The use of Select Subgrade Material (SSM), or granular fill (satisfying OPSS.PROV 1010 (*Aggregates*) Granular ‘B’ Type I or Type II requirements) is required in order to achieve adequate surficial stability (i.e., a factor of safety of 1.5 in permanent/long-term condition). The fill placement should be carried out in accordance with the requirements as outlined in OPSS.PROV 206 (*Grading*) and compacted in accordance with OPSS.PROV 501 (*Compacting*). Inspection and field testing should be carried out by qualified personnel during construction to confirm that appropriate materials are used and that adequate levels of compaction are being achieved.

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

#### **6.10.7 Preloading and Settlement Monitoring**

As discussed in Section 6.7.3, based on the estimated magnitude and time-rate of settlement, settlement mitigation will be required particularly if shallow foundations are to be adopted for the proposed connection structure. The design-builder will have to develop an instrumentation and settlement monitoring program to confirm the magnitude and time-rate of settlement at the abutments/approach embankments.

## 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 Foundation Designated Contact of Golder, conducted an independent technical and quality control review of this report.

### Golder Associates Ltd.



Manisha Ahuja, P.Eng., P.E.  
*Geotechnical Engineer*



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

MA/LCC/rb

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<https://golderassociates.sharepoint.com/sites/15994g/6.deliverables/wo002-glendaleinterchange/foundations/4.airportrd/3.final/1671430wo2fidr2019-05-15airportrdconnectionstructure.docx>

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Canadian Standards Association (CSA). 2014. *Canadian Highway Bridge Design Code and Commentary on CAN/CSA-S6-14*. CSA Special Publication.

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

Kulhawy, F.H. and Mayne, P.W. 1990. *Manual on Estimating Soil Properties for Foundation Design*. EL 6800, Research Project 1493 6. Prepared for Electric Power Research Institute, Palo Alto, California.

National Resources Canada, 2017. *Earthquake Hazard*. [http://www.earthquakescanada.nrcan.gc.ca/hazard-alea/interpolat/index\\_2015-en.php](http://www.earthquakescanada.nrcan.gc.ca/hazard-alea/interpolat/index_2015-en.php). Accessed on July 18, 2018.

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

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

### ASTM International:

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

### Commercial Software:

Slide (Version 2018) by Rocscience Inc.

Settle (Version 2018) by Rocscience Inc.

### Ontario Provisional Standard Drawing:

OPSD 208.010	Benching of Earth Slopes
OPSD 3090.101	Foundation Frost Penetration Depths for Southern Ontario

### Ontario Provincial Standard Specification:

OPSS.PROV 501	Construction Specifications for Compacting
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS.PROV 902	Construction Specification for Excavating and Backfilling Structures
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

**Special Provisions**

SP105S10      Compaction

**Ontario Water Resources Act:**

Ontario Regulation 903      Wells (as amended)

**Ontario Occupational Health and Safety Act:**

Ontario Regulation 213/91      Construction Projects (as amended)

**Ministry of Transportation, Ontario**

*Structural Manual*, Provincial Highways Management Division, Highway Standards Branch, Bridge Office, August 2016.

*RSS Design Guidelines*, Ministry of Transportation Engineering Standards Branch, September 2008

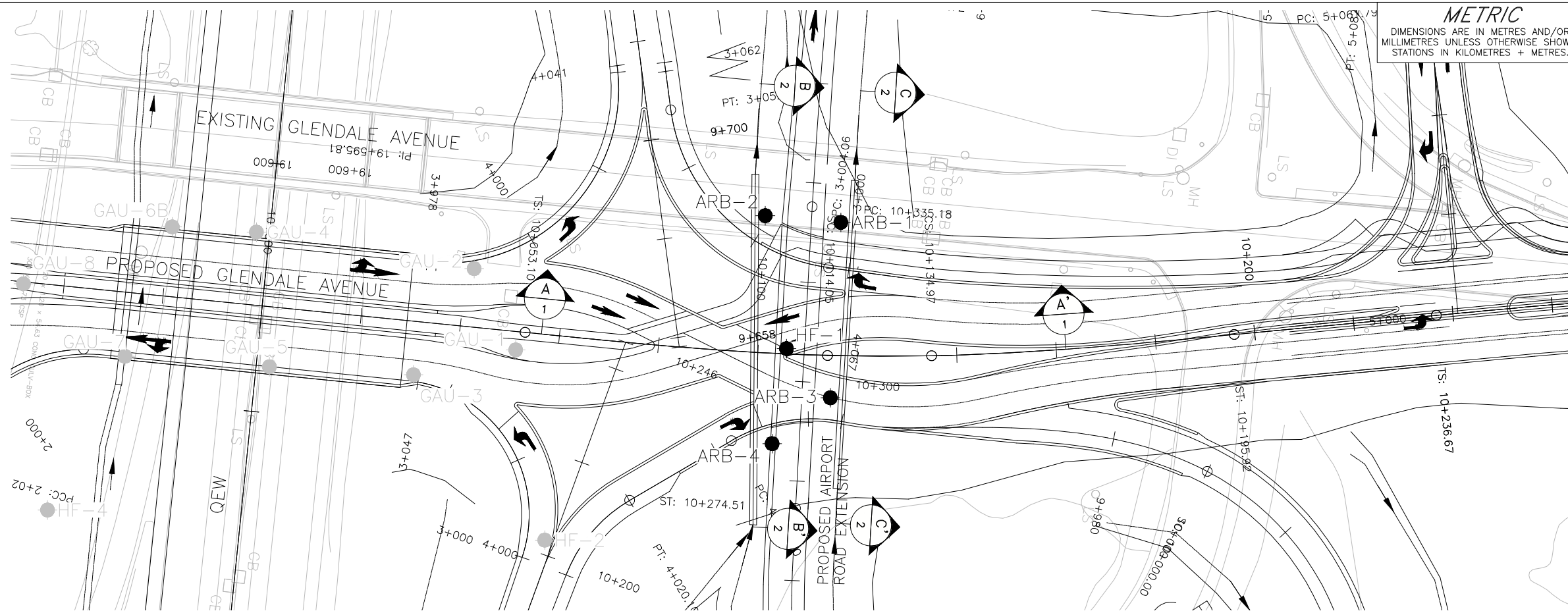
TABLE 1 – COMPARISON OF FOUNDATION ALTERNATIVES – AIRPORT ROAD CONNECTION STRUCTURE

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Strip footings founded on native soils	Feasible if preloading or other settlement mitigation is adopted to minimize total and differential settlement along and between foundation elements; as this structure is located “offline” from the existing QEW and Glendale Avenue alignments, there is sufficient space for preloading operations (in conjunction with other settlement mitigation as may be adopted). This option is preferred if there is sufficient time available in the construction schedule for settlement mitigation to be completed.	<ul style="list-style-type: none"><li>Where feasible, generally results in lower construction costs; however, in this case, would require additional preloading time and material handling costs</li><li>Excavations will be maintained above the groundwater level and only minor groundwater seepage anticipated</li></ul>	<ul style="list-style-type: none"><li>Relatively low factored serviceability geotechnical resistance if footings constructed in advance of preloading or other settlement mitigation; these values are improved where footings are constructed after settlement mitigation</li><li>Temporary protection systems required along east side of existing Glendale Avenue to reach design founding level for new structure on the east side of the existing embankment, and to facilitate removals and new construction under existing Glendale Avenue embankment</li><li>Precludes use of integral abutments; potentially greater maintenance required at abutments</li></ul>	<ul style="list-style-type: none"><li>Conventional excavation and construction techniques</li><li>Protection system installation, excavation and foundation construction must take account of presence of former North Service Road structure that was demolished and infilled under existing Glendale Avenue embankment</li></ul>	<ul style="list-style-type: none"><li>Lower relative cost than deep foundations, excluding costs and time associated with settlement mitigation, and with protection systems/removals</li></ul>
Driven steel H-piles or pipe piles	Feasible for support of the abutments and centre pier. If the construction schedule is constrained such that settlement mitigation cannot be completed within the available timeframe, this option would be preferred to mitigate differential settlement along and between foundation elements.	<ul style="list-style-type: none"><li>Meets settlement performance criteria without settlement mitigation treatment at abutments</li><li>Allows for integral abutment construction</li></ul>	<ul style="list-style-type: none"><li>Temporary protection systems required, similar to strip footings</li><li>Risk of encountering obstructions that could impact pile installation; piles may terminate above design pile tip elevation</li><li>Larger/specialized equipment required for installation of piles than for construction of shallow foundations</li></ul>	<ul style="list-style-type: none"><li>Conventional construction methods for driven piles</li><li>Protection system installation, excavation and foundation installation must take account of presence of former North Service Road structure that was demolished and infilled under existing Glendale Avenue embankment</li></ul>	<ul style="list-style-type: none"><li>Estimated cost is approximately \$250/m length for pile installation and \$600/m<sup>3</sup> for pile cap construction, plus costs and time associated with protection system installation and removal of former North Service Road structure</li><li>Potentially less costly maintenance over life of the structure than semi-integral abutment structures</li></ul>

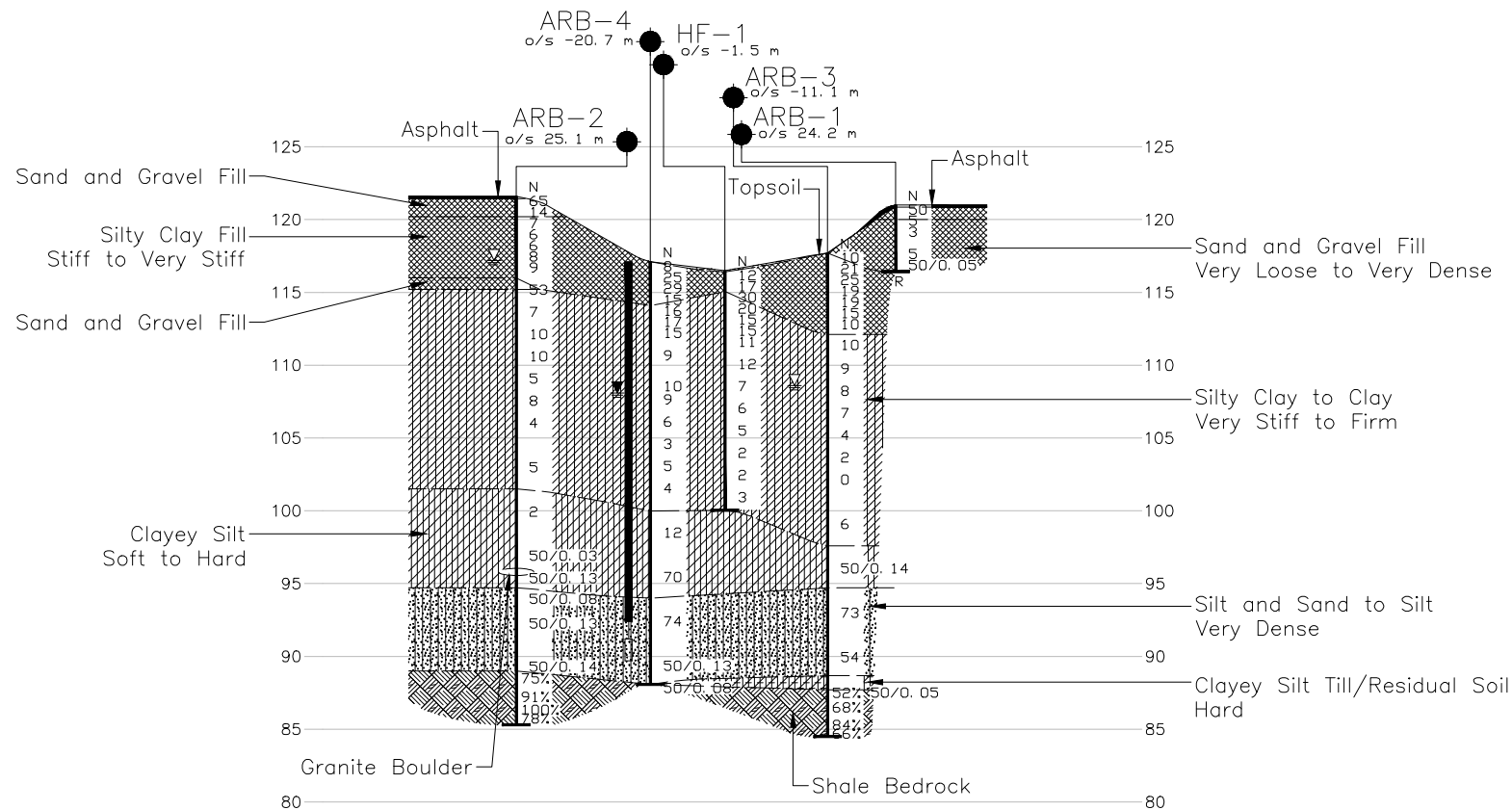
TABLE 2 – SUMMARY OF FOUNDATION ENGINEERING PARAMETERS – NORTH AND SOUTH APPROACH EMBANKMENTS

Foundation Investigation Area	Stratigraphic Unit	Elevation (m)	Thickness (m)	$\gamma$ (kN/m <sup>3</sup> )	$\phi'$ (°)	$c'$ (kPa)	$\sigma_u$ (kPa)	$\sigma_p'$ (kPa)	$e_0$	$C_c$	$C_r$	$m_v$ (kPa)	$E'$ (MPa)	$c_v$ (cm <sup>2</sup> /s)
North Approach Embankment (ARB-1, ARB-4 & HF-1)	Silty Clay to Clay (Fill)	121.0 - 115.1	1.4 - 3.0	19	30	0	-	-	-	-	-	-	5 - 25	-
	Sand and Gravel (Fill)	120 - 115.1	0.8 - 3.6	19	32	0	-	-	-	-	-	-	10 - 25	-
	Silty Clay to Clay	114.1 - 108.0	6.1	18.5	27	0	85 - 95	-	0.85	-	-	-	30 - 50	-
	Silty Clay to Clay (Firm to Stiff)	108.0 - 100.0	8.0 - 9.0	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>
	Silt	94.0 - 88.1	6.0	19	30	0	-	-	-	-	-	-	25 - 75	-
	Clayey Silt	100.0 - 94.0	6.0	21	32	0	-	-	-	-	-	-	175	-
South Approach Embankment (ARB-2 & ARB-3)	Silty Clay to Clay (Fill)	120.1 - 114.7	3 - 4.2	19	30	0	-	-	-	-	-	-	5 - 25	-
	Silty Clay to Clay	114.7 - 110.5	4.2	18.5	27	0	85 - 95	-	0.85	-	-	-	30 - 50	-
	Silty Clay to Clay (Firm to Stiff)	115.1 - 97.6	12.9 - 13.7	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>	30 - 50	2.5 x 10 <sup>-3</sup> - 3.5 x 10 <sup>-3</sup>
	Clayey Silt to Clayey Silt Till	101.4 - 94.7	2.9 - 5.6	19	30	0	-	-	-	-	-	-	25 - 75	-
	Silt and Sand to Silt	94.6 - 88.8	5.7 - 5.9	21	32	0	-	-	-	-	-	-	175	-
	Shale Bedrock	88.9 - 84.5	3.2 - 3.7	22	35	0	-	-	-	-	-	-		-





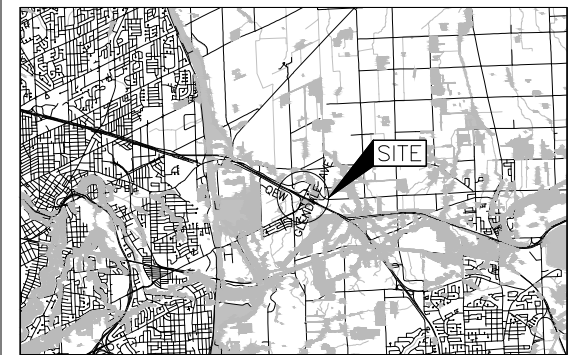
PLAN

SCALE  
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PROFILE A-A'

HORIZONTAL SCALE  
10 0 10 20 m  
VERTICAL SCALE  
10 0 10 20 m**METRIC**  
DIMENSIONS ARE IN METRES AND/OR  
MILLIMETRES UNLESS OTHERWISE SHOWN.  
STATIONS IN KILOMETRES + METRES.CONT No.  
GWP No.2423-15-00QEW/GLENDALE AVENUE INTERCHANGE IMPROVEMENTS  
AIRPORT ROAD CONNECTION STRUCTURE  
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)
- R Refusal
- 100% Rock Quality Designation (RQD)
- ≡ WL in piezometer, measured on May 7, 2019
- ≡ WL upon completion of drilling

## BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
ARB-1	121.0	4779855.5	332139.8
ARB-2	121.5	4779841.8	332133.3
ARB-3	117.7	4779841.4	332172.2
ARB-4	117.1	4779827.2	332176.8
HF-1	116.5	4779836.5	332159.9

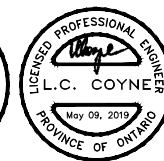
## NOTES

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

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

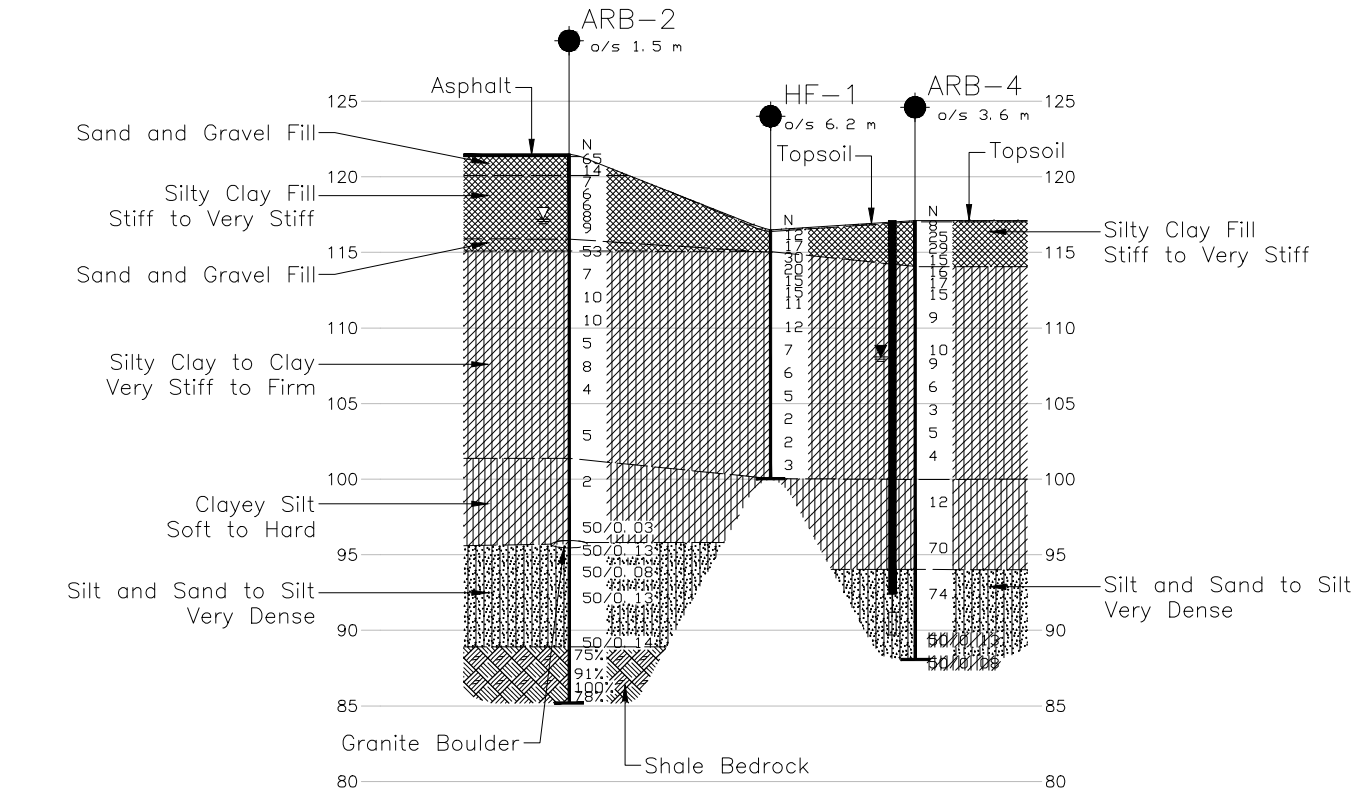
## REFERENCE

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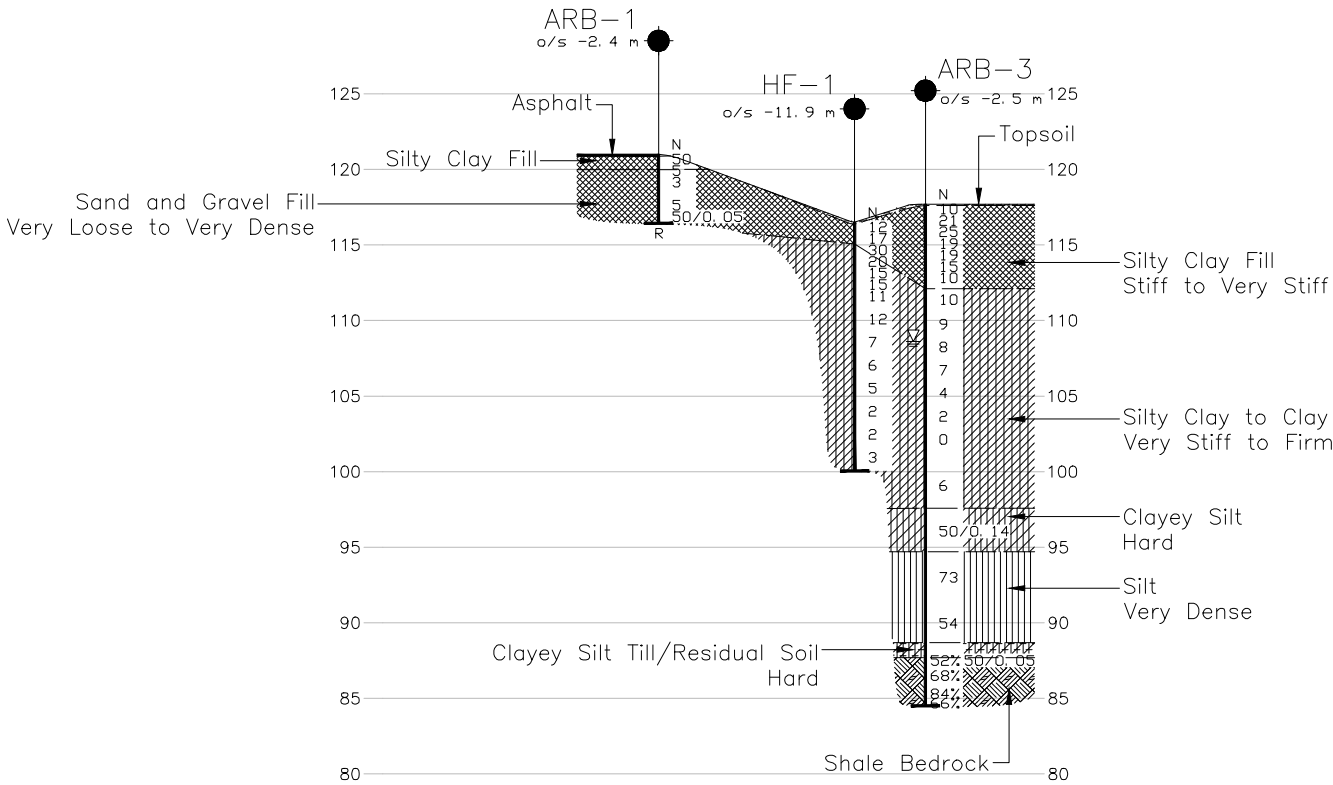


NO.	DATE	BY	REVISION
Geocres No. 30M3-310			
HWY. QEW	PROJECT NO. 1671430	DIST. .	
SUBM'D. NK	CHKD. MA	DATE: 05/10/2019	SITE: .
DRAWN: DD	CHKD. MA	APPD. LCC	DWG. 1

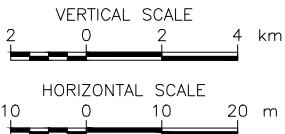




CROSS SECTION B-B'  
SOUTH ABUTMENT



CROSS SECTION C-C'  
NORTH ABUTMENT

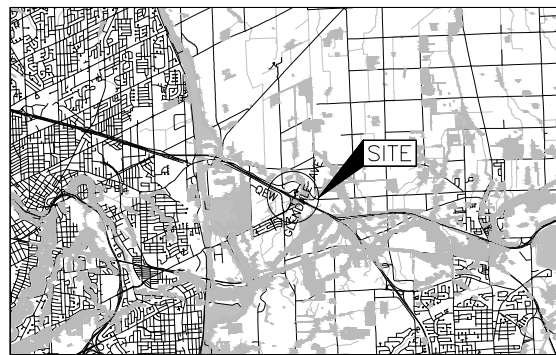


**METRIC**  
DIMENSIONS ARE IN METRES AND/OR  
MILLIMETRES UNLESS OTHERWISE SHOWN.  
STATIONS IN KILOMETRES + METRES.

CONT No.  
GWP No.2423-15-00

QEW/GLENDALE AVENUE INTERCHANGE IMPROVEMENTS  
AIRPORT ROAD CONNECTION STRUCTURE

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)
- R Refusal
- 100% Rock Quality Designation (RQD)
- ⬮ WL in piezometer, measured on May 7, 2019
- ⬮ WL upon completion of drilling

BOREHOLE CO-ORDINATES			
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NO.	DATE	BY	REVISION
Geocres No. 30M3-310			
HWY.	QEW	PROJECT NO.	1671430
SUBM'D.	NK	CHKD.	MA
DRAWN:	SW	CHKD.	MA
DATE:	05/10/2019	APPD.	LCC
SITE:		DWG.	2

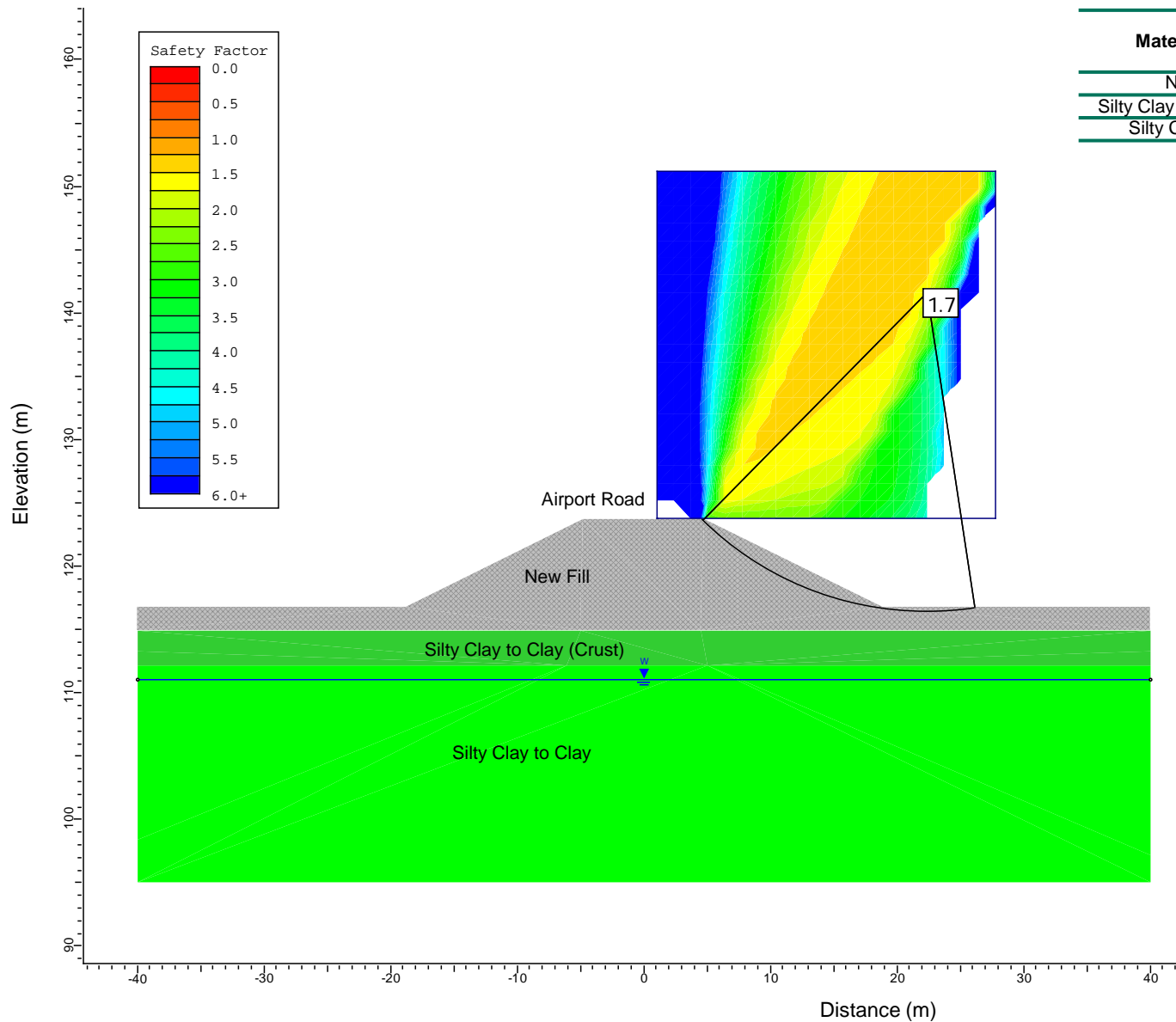


**GOLDER**



# QEW – Airport Road Connection Structure Global Stability of 2H:1V Embankment Side Slope – Short-Term (Undrained) Condition

Figure 2A

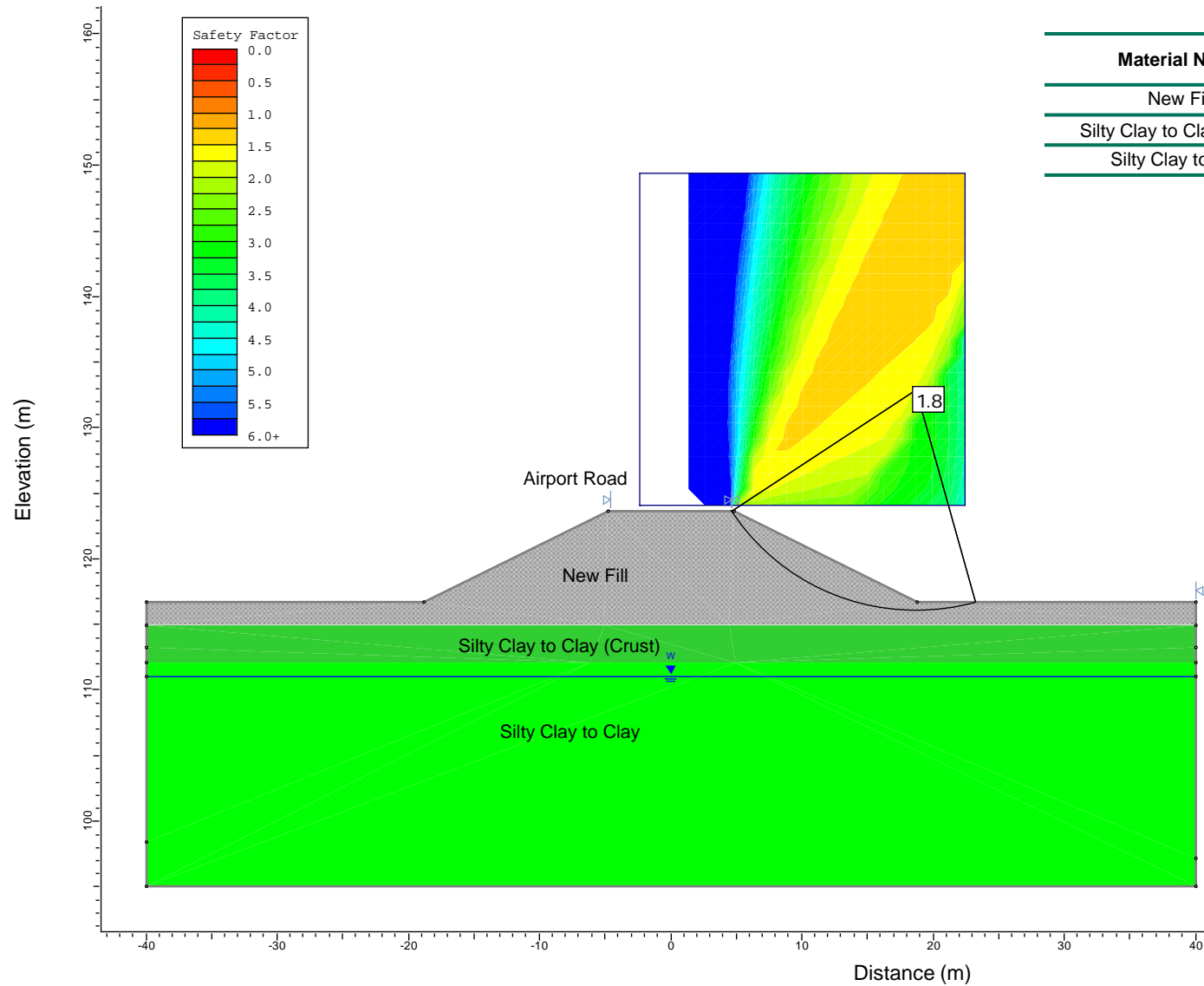


Material Name	$\gamma$ (kN/m <sup>3</sup> )	$s_u$ (kPa)	$\phi'$ (degrees)
New Fill	21	-	35
Silty Clay to Clay (Crust)	18.5	100	-
Silty Clay to Clay	18	75	-



# QEW – Airport Road Connection Structure Global Stability of 2H:1V Embankment Side Slope – Long-Term (Effective Stress) Condition

Figure 2B



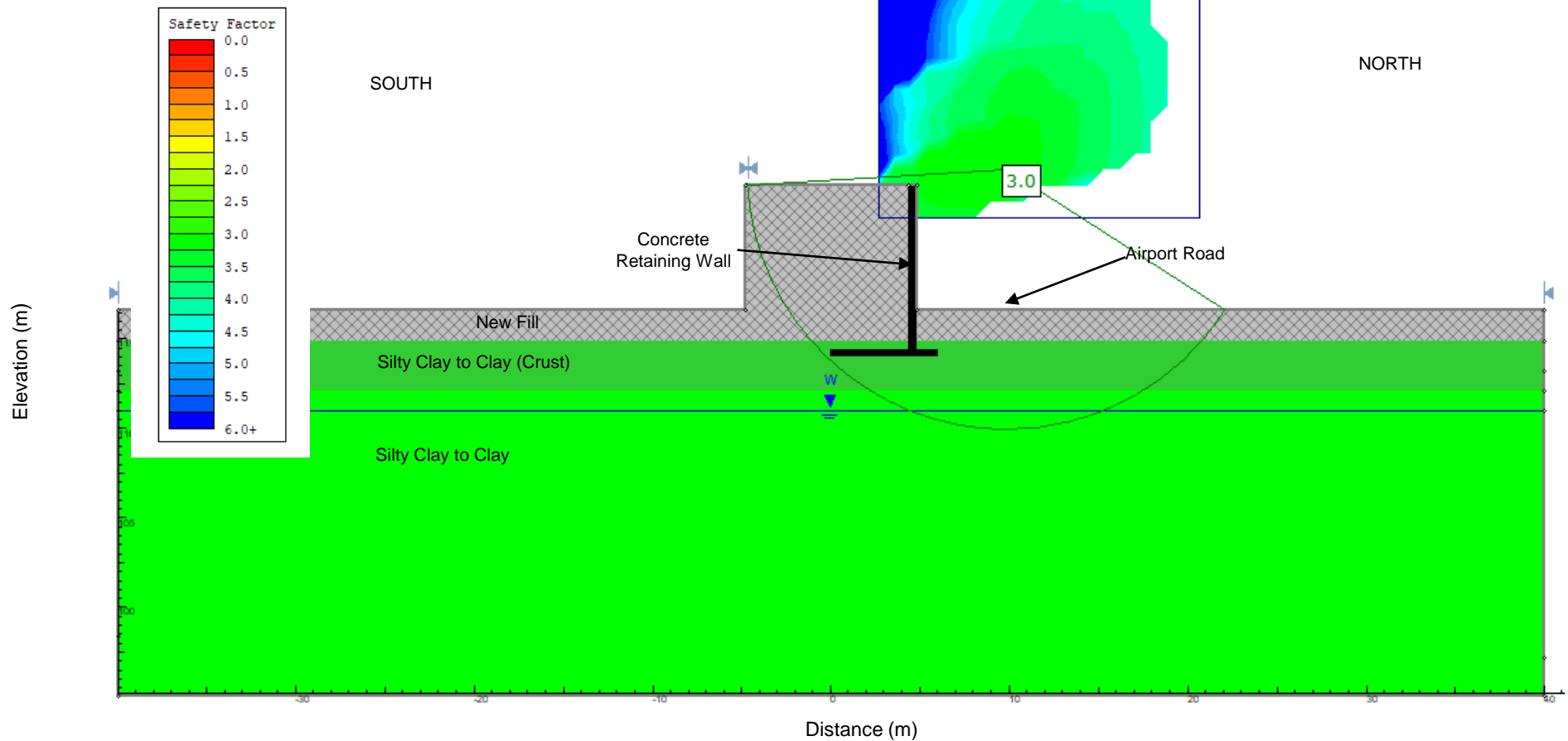
Material Name	$\gamma$ (kN/m <sup>3</sup> )	$s_u$ (kPa)	$\phi'$ (degrees)
New Fill	21	-	35
Silty Clay to Clay (Crust)	18.5	-	27
Silty Clay to Clay	18	-	25



# QEW – Airport Road Connection Structure Global Stability of Concrete Cantilever Retaining Wall – Short-Term (Undrained) Condition

Figure 3A

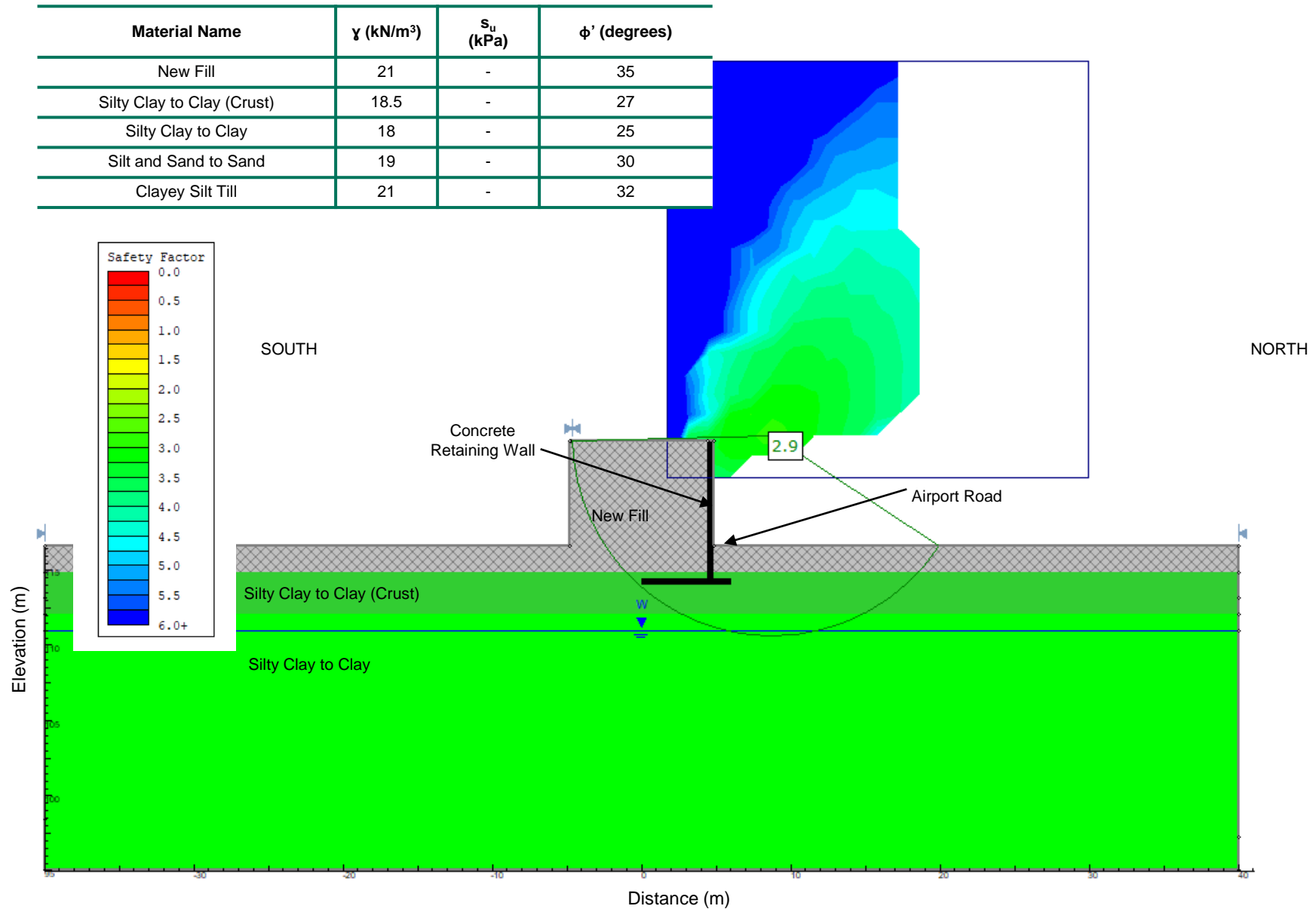
Material Name	$\gamma$ (kN/m <sup>3</sup> )	$s_u$ (kPa)	$\phi'$ (degrees)
New Fill	21	-	35
Silty Clay to Clay (Crust)	18.5	100	-
Silty Clay to Clay	18	75	-
Silt and Sand to Sand	19	-	30
Clayey Silt Till	21	-	32





# QEW – Airport Road Connection Structure Global Stability of Concrete Cantilever Retaining Wall – Long-Term (Effective Stress) Condition

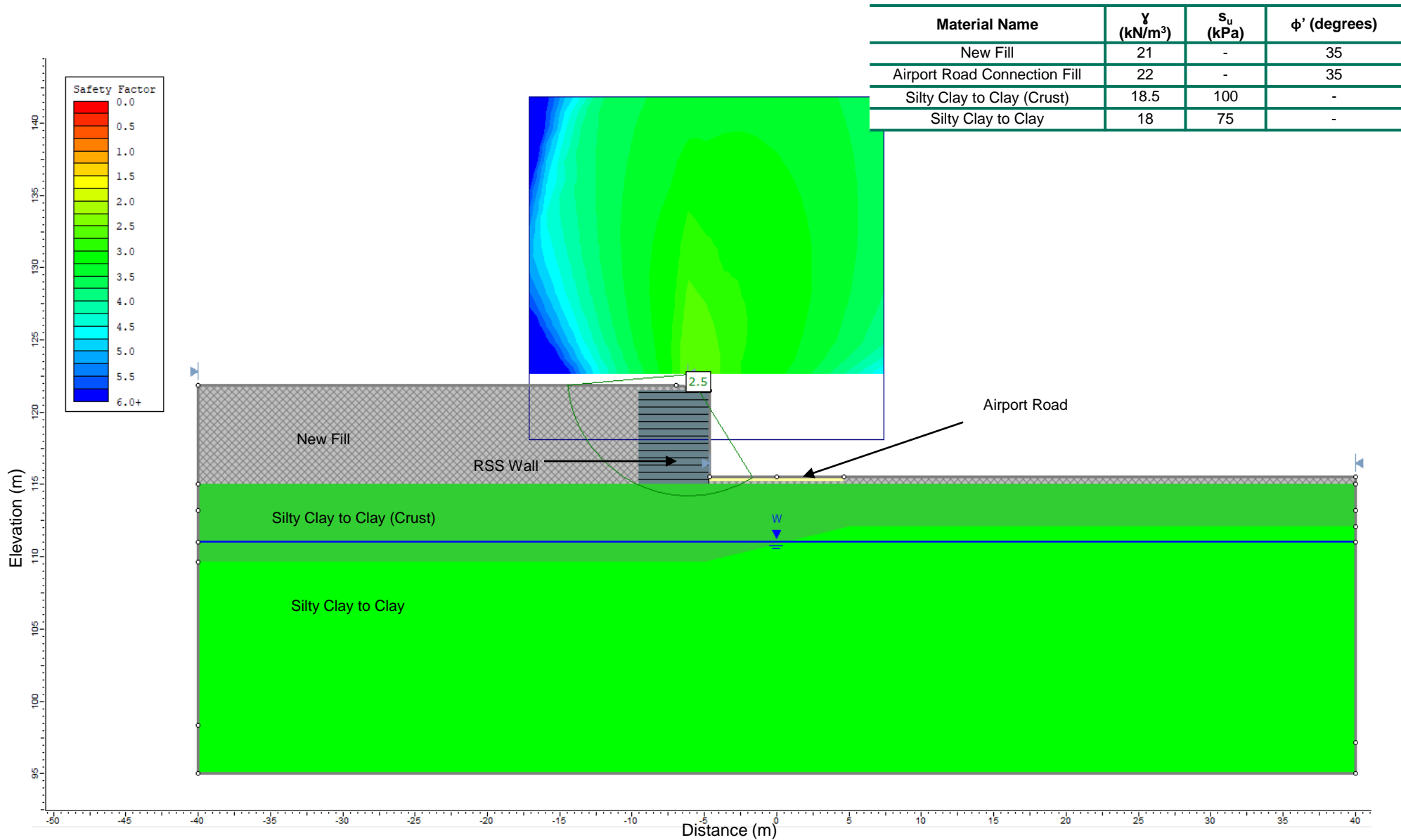
Figure 3B





# QEW – Airport Road Connection Structure Global Stability of RSS Wall – Short-Term (Undrained) Condition

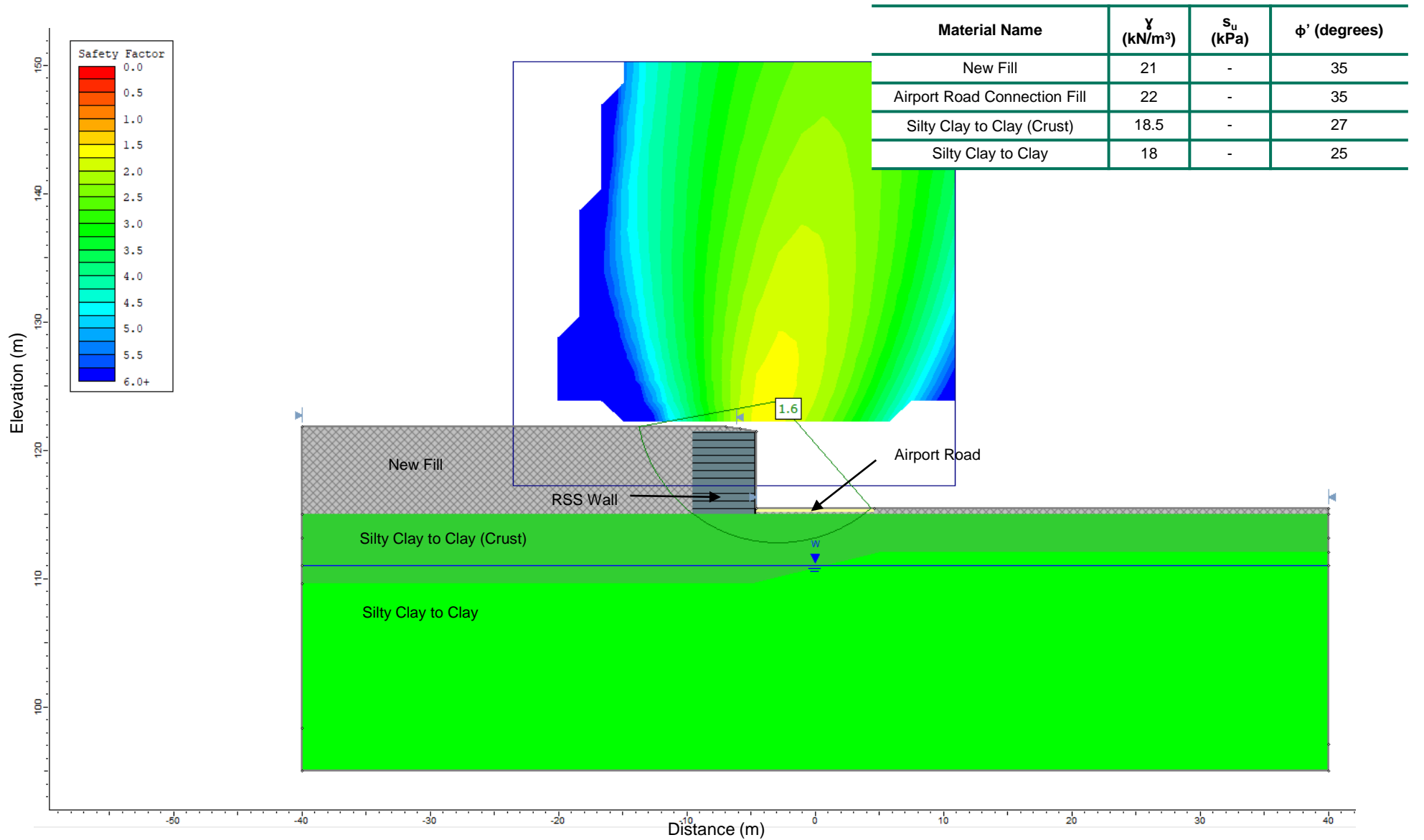
Figure 4A





# QEW – Airport Road Connection Structure Global Stability of RSS Wall – Long-Term (Effective Stress) Condition

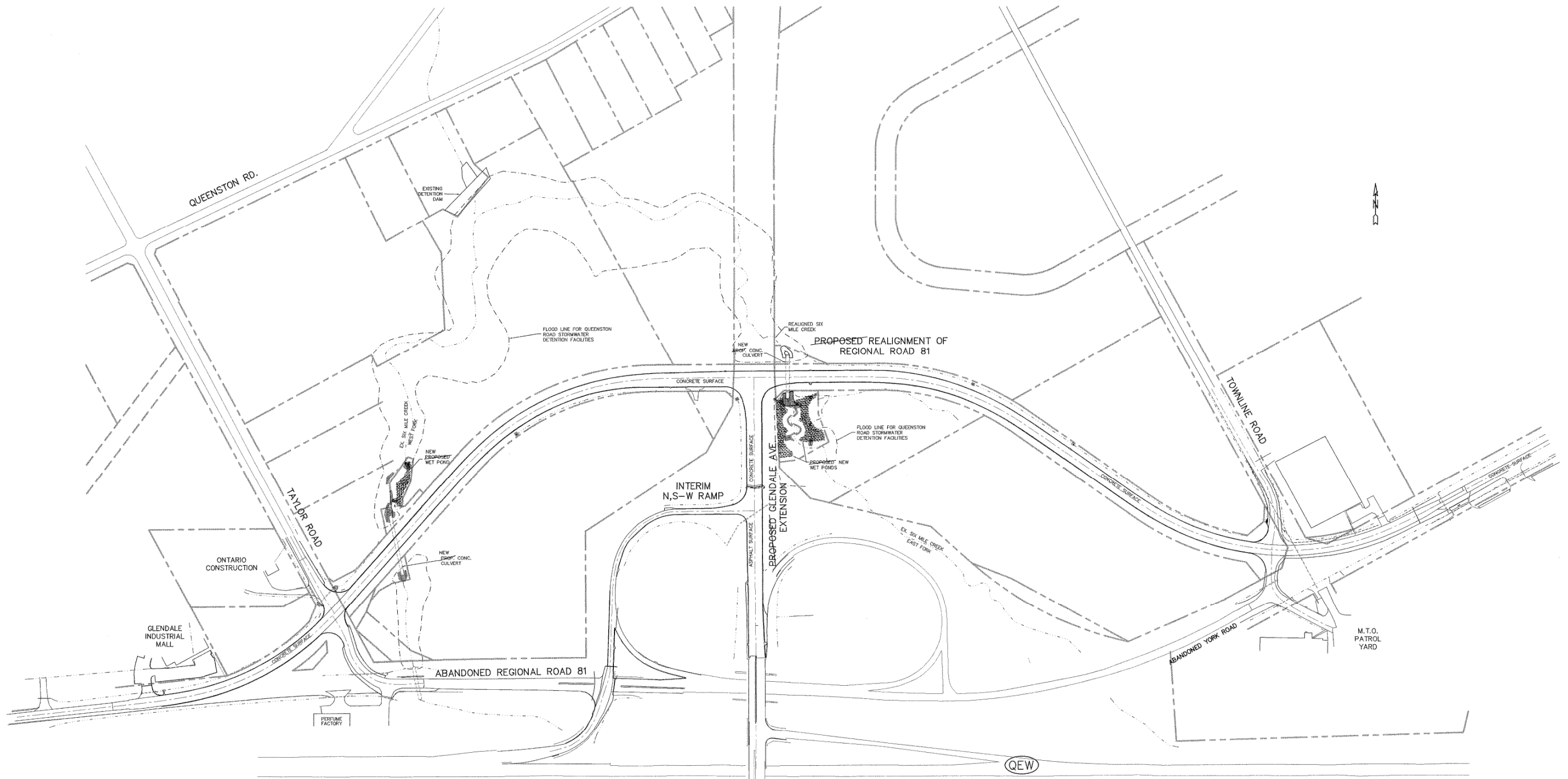
Figure 4B





**APPENDIX A**

# Former Structure Information



**NOTE:**  
 THE INFORMATION ON THIS CONSTRUCTION DRAWING IS DERIVED FROM LIMITED SPOT CHECKS MADE DURING THE CONSTRUCTION REVIEW PROCESS AND IN SOME INSTANCES FROM INFORMATION PROVIDED BY THE CONTRACTOR OR OTHERS. THEREFORE, MPS REINDERS NIAGARA DOES NOT ACCEPT ANY RESPONSIBILITY FOR THE COMPLETENESS OF THE INFORMATION SHOWN ON THIS DRAWING NOR FOR THE WAY IN WHICH THIS INFORMATION IS USED BY OTHERS. THOSE USING THIS DRAWING SHALL SATISFY THEMSELVES REGARDING THE ACCURACY OF THE SAME AND SHALL REPORT ANY CONFLICTING INFORMATION TO MPS REINDERS NIAGARA INC. IMMEDIATELY.

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NO.	REVISION	DATE	INIT.
4	CONSTRUCTION RECORD REVISION	15 JAN 96	DAA
3	FLOOD LINE EXTENDED	29 MAY 95	DAA
2	ISSUED FOR CONSTRUCTION	28 MAR 95	DAA
1	ISSUED FOR TENDER	23 SEP 94	DAA
0	ISSUED FOR APPROVAL	30 JUN 94	JDT

drawn by:  
J.D.T.  
 design by:  
H.E.K.  
 approved by:  
P.A.M.  
 date:  
SEP 94

MPS Reinders Niagara Inc.  
 Architects, Engineers, & Planners  
 phone: (905) 984-8676  
 fax: (905) 682-5896

drawing title  
**GENERAL  
 ARRANGEMENT  
 PLAN**

PUBLIC WORKS DEPARTMENT

REALIGNMENT  
 OF  
**YORK ROAD**  
 (Regional Road 81)  
 NIAGARA-ON-THE-LAKE

scale 1:2000	revision # 4
drawing no. 6875-G2	contract no. RN. 94-21

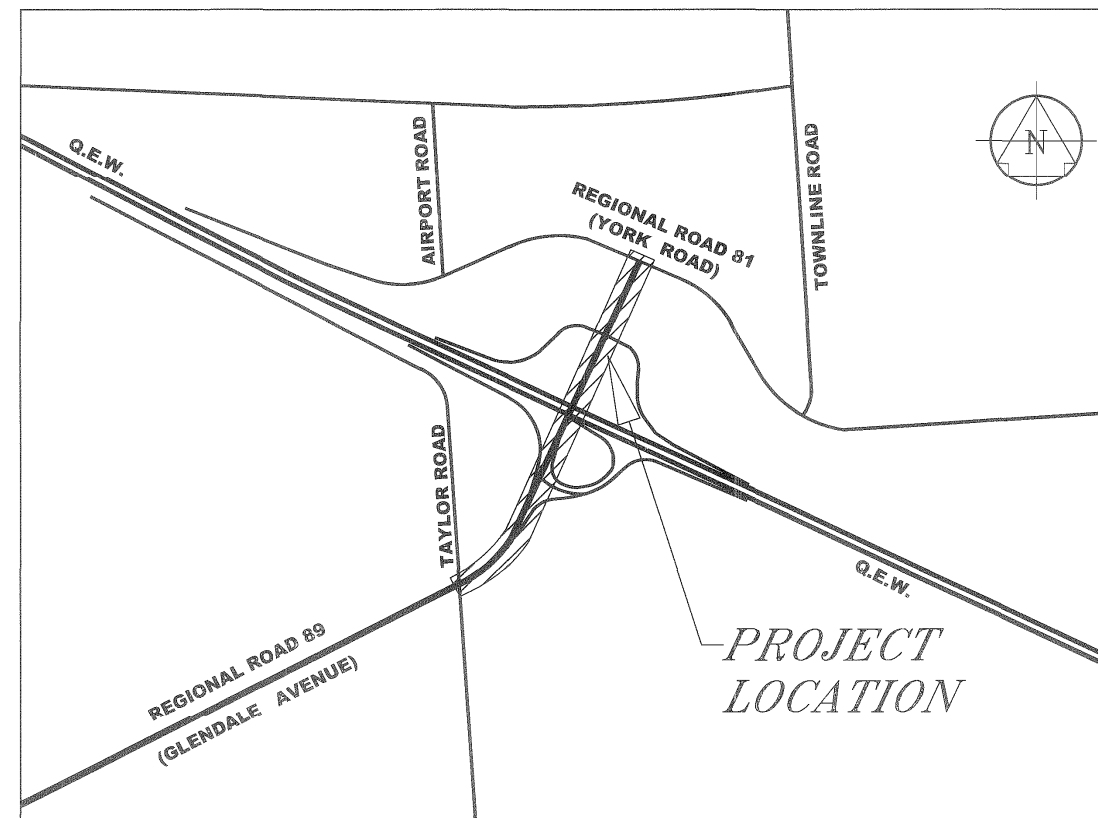
1522



**The Regional Municipality of Niagara  
Public Works Department**

**GLENDALE AVENUE (REGIONAL ROAD 89) BRIDGE DEMOLITION,  
ROAD AND SIDEWALK CONSTRUCTION  
from YORK ROAD (REGIONAL ROAD 81)  
to TAYLOR ROAD  
in the Town of NIAGARA-ON-THE-LAKE**

**IAN NEVILLE, M.P.A., P.ENG.  
DIRECTOR OF PUBLIC WORKS**



KEY PLAN - (Not to Scale)

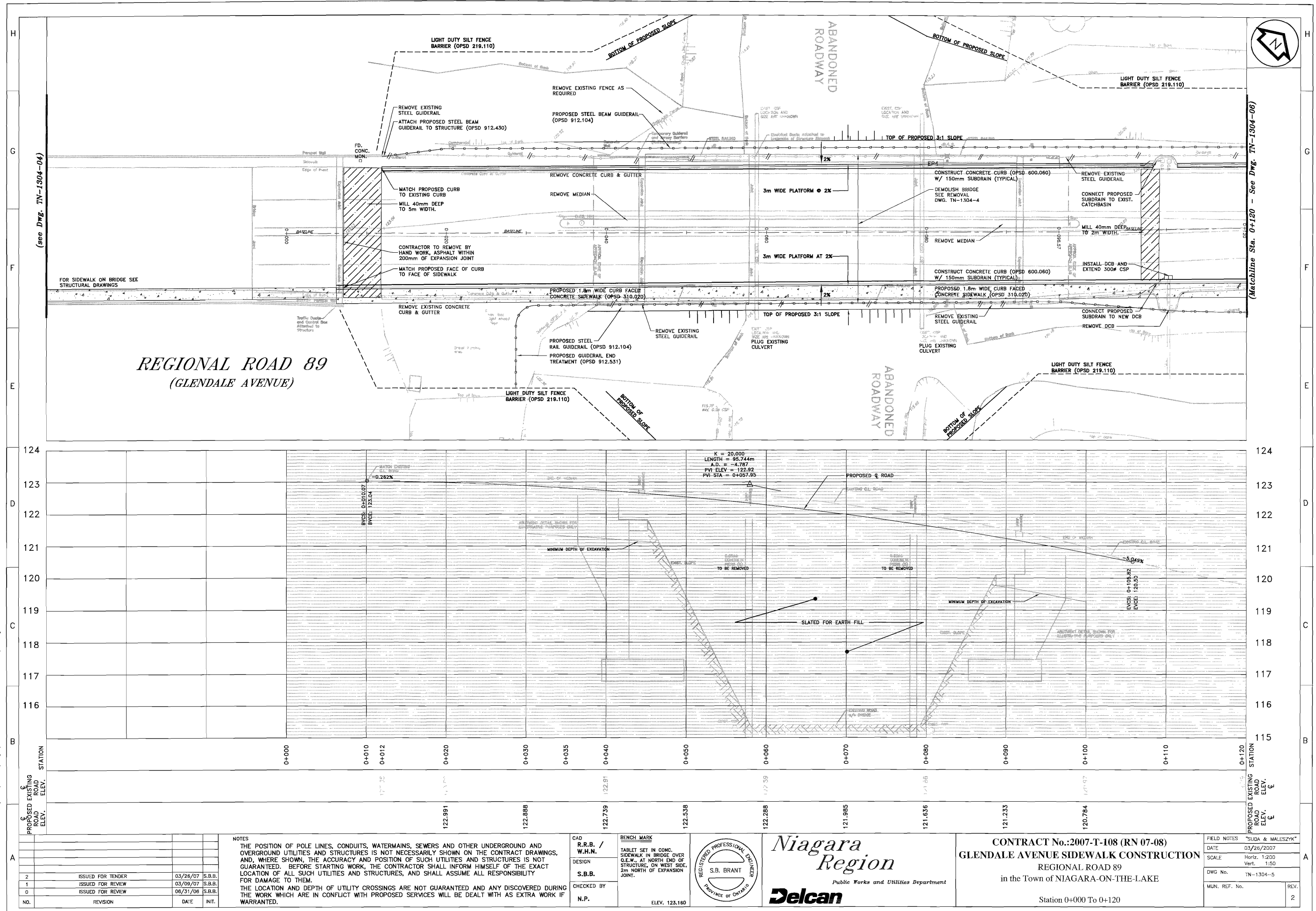
**PETER PARTINGTON  
REGIONAL CHAIR**

**Delcan**

**CONTRACT NO.: 2007-T-108  
(RN 07-08)**

ISSUED FOR TENDER  
MARCH 26, 2007

CONTRACT No.: RN. 2007-T-108 (RN 07-08)  
GLENDALE AVENUE SIDEWALK CONSTRUCTION  
REGIONAL ROAD 89  
in the Town of NIAGARA-ON-THE-LAKE

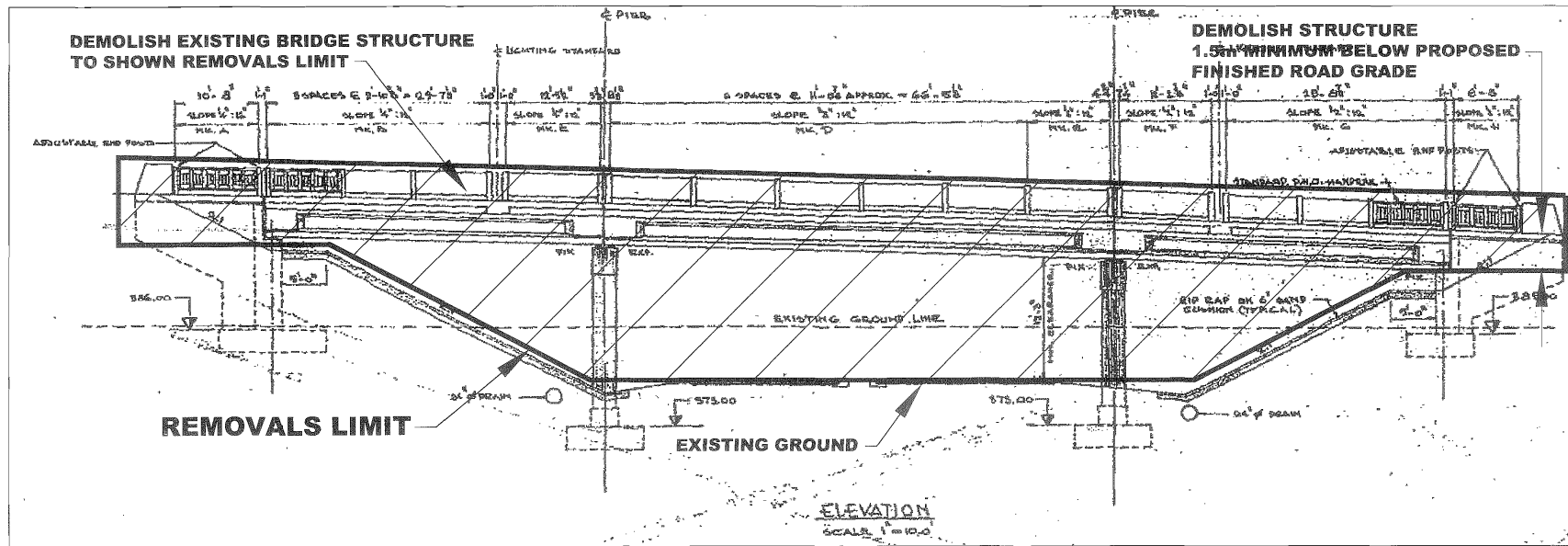
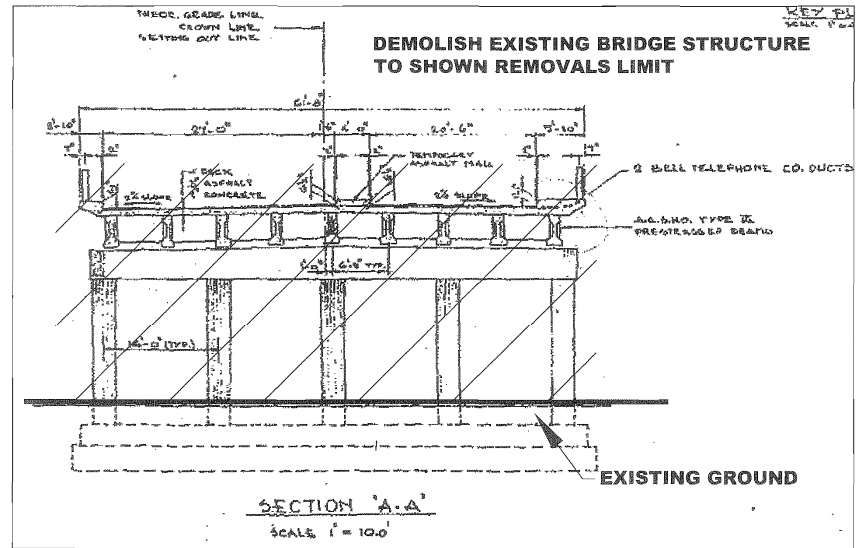
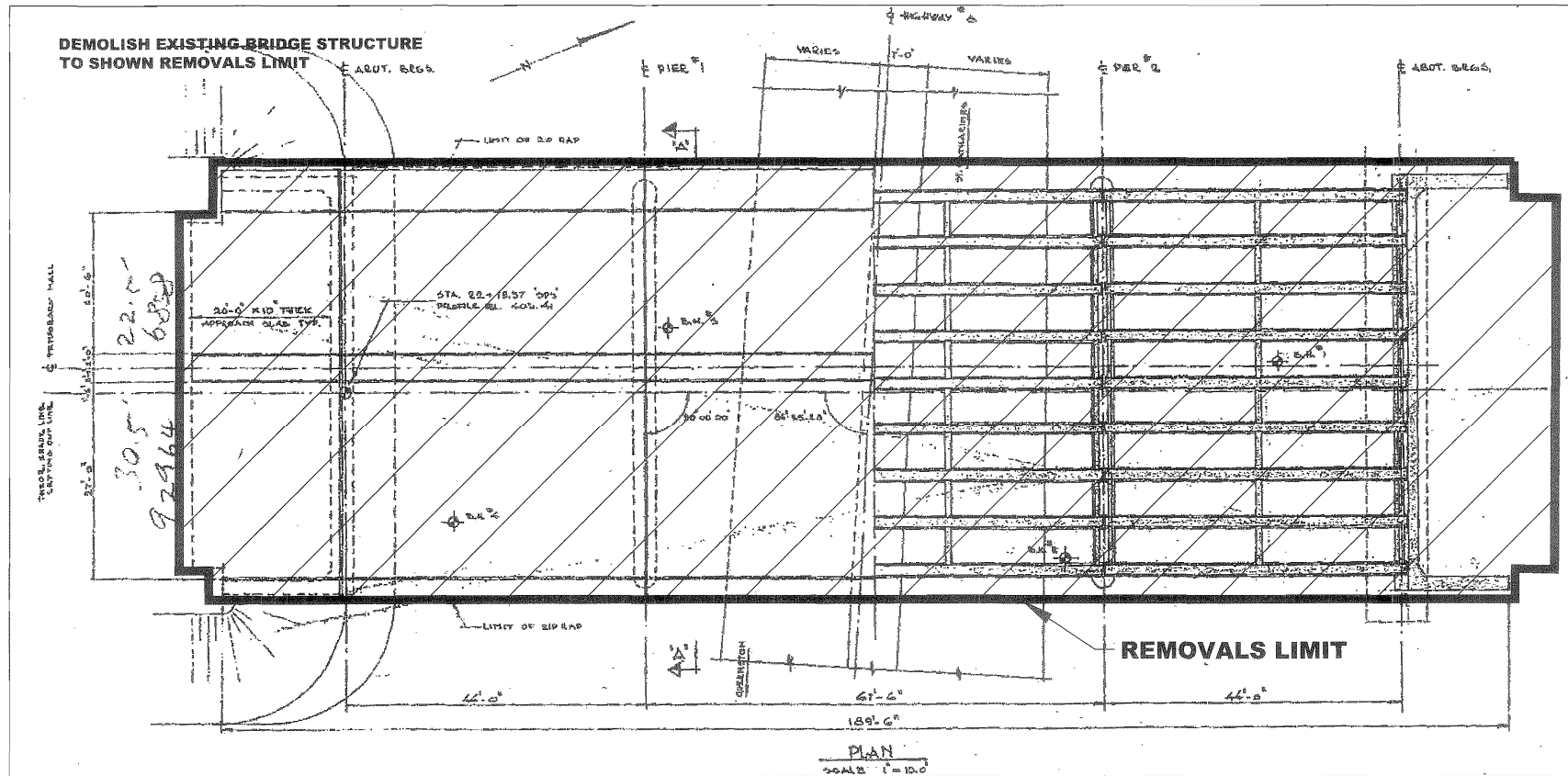






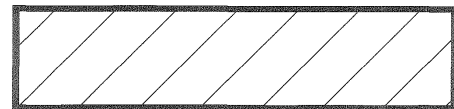


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XREF = 1: DATA\1304\1304.MF #6\GAD FILES.....Glendale - QEW Baseplan.dwg



**NOTES:**  
**BACKGROUND DRAWING FROM THE ORIGINAL "GENERAL ARRANGEMENT" PLAN**

**DENOTES AREAS TO BE REMOVED/DEMOLISHED**



NO.	REVISION	DATE	INIT.
2	ISSUED FOR TENDER	03/26/07	S.B.B.
1	ISSUED FOR REVIEW	03/09/07	S.B.B.
0	ISSUED FOR	01/01/02	S.B.B.

**NOTES**

THE POSITION OF POLE LINES, CONDUITS, WATERMAINS, SEWERS AND OTHER UNDERGROUND AND OVERGROUND UTILITIES AND STRUCTURES IS NOT NECESSARILY SHOWN ON THE CONTRACT DRAWINGS, AND, WHERE SHOWN, THE ACCURACY AND POSITION OF SUCH UTILITIES AND STRUCTURES IS NOT GUARANTEED. BEFORE STARTING WORK, THE CONTRACTOR SHALL INFORM HIMSELF OF THE EXACT LOCATION OF ALL SUCH UTILITIES AND STRUCTURES, AND SHALL ASSUME ALL RESPONSIBILITY FOR DAMAGE TO THEM.

THE LOCATION AND DEPTH OF UTILITY CROSSINGS ARE NOT GUARANTEED AND ANY DISCOVERED DURING THE WORK WHICH ARE IN CONFLICT WITH PROPOSED SERVICES WILL BE DEALT WITH AS EXTRA WORK IF WARRANTED.

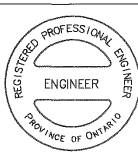
CAD  
R.R.B. /  
W.H.N.

DESIGN  
S.B.B.

CHECKED BY  
N.P.

BENCH MARK  
TABLET SET IN CONC.  
SIDEWALK IN BRIDGE OVER  
Q.E.W. AT NORTH END OF  
STRUCTURE, ON WEST SIDE,  
2m NORTH OF EXPANSION  
JOINT.

ELEV. 123.160



**Niagara Region**  
Public Works and Utilities Department  
**Delcan**

CONTRACT No.:2007-T-108 (RN 07-08)  
**GLENDALE AVENUE SIDEWALK**  
REGIONAL ROAD 81 - (GLENDALE AVENUE)  
in the TOWN OF NIAGARA-ON-THE-LAKE

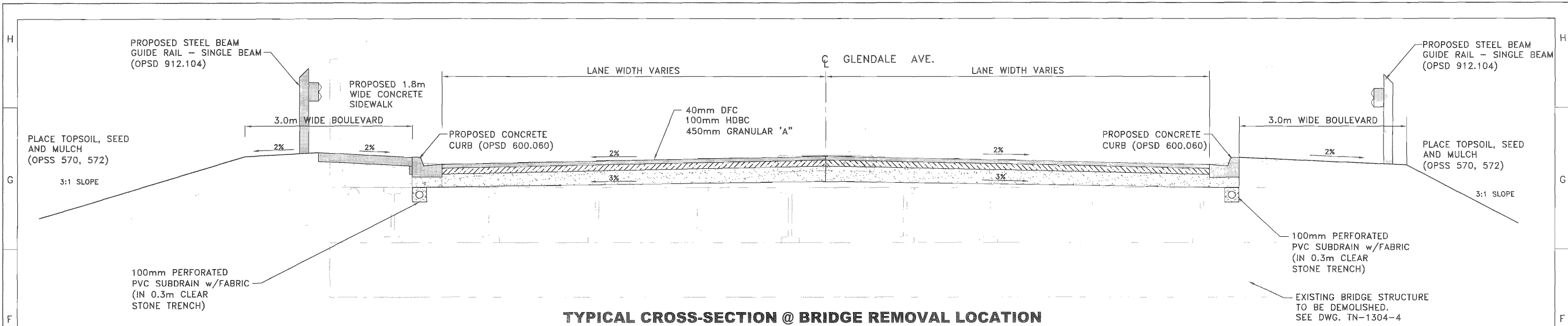
DEMOLITION PLAN

FIELD NOTES	"M.T.O."
DATE	03/26/2007
SCALE	Horiz. 1:200 Vert. 1:50
DWG No.	TN-1304-9
MUN. REF. No.	
REV.	2

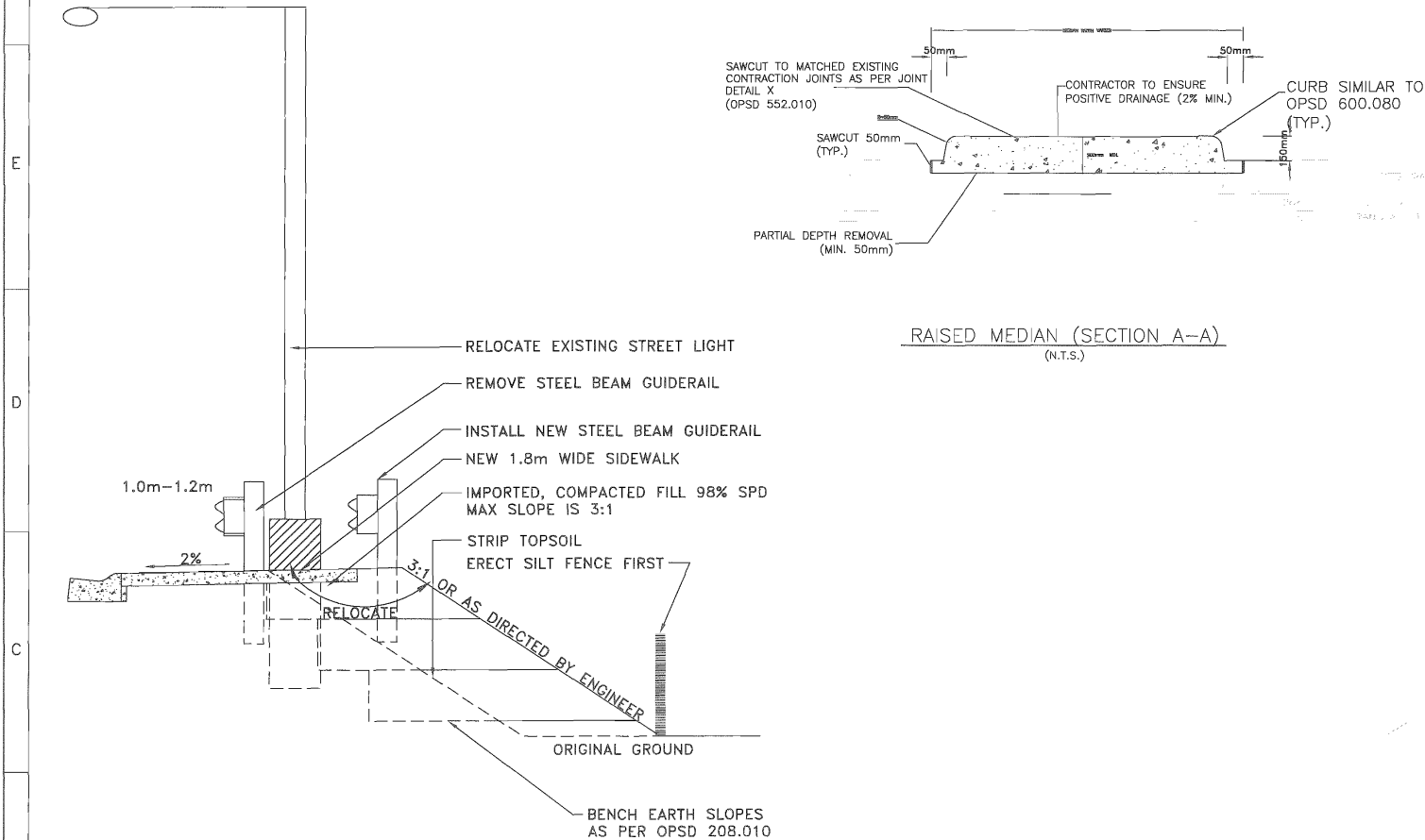
ISSUED FOR TENDER  
MARCH 26, 2007

TN-1304-9  
REMOVALS PLAN  
GLENDALE AVENUE

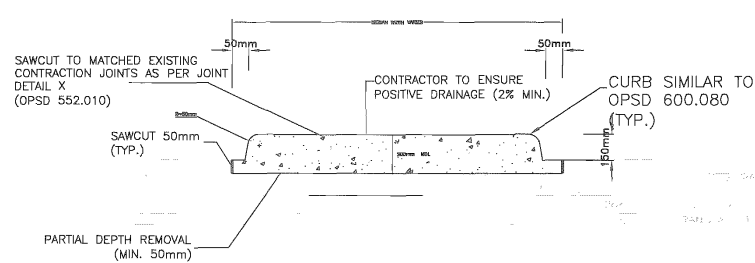
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XREF = J:\DATA\1304\1304.MXD #1304.MXD - QEW Baseplan.dwg



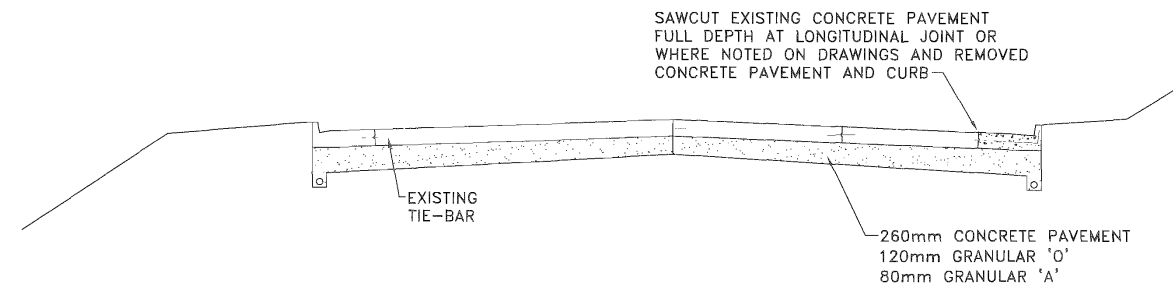
**TYPICAL CROSS-SECTION @ BRIDGE REMOVAL LOCATION**



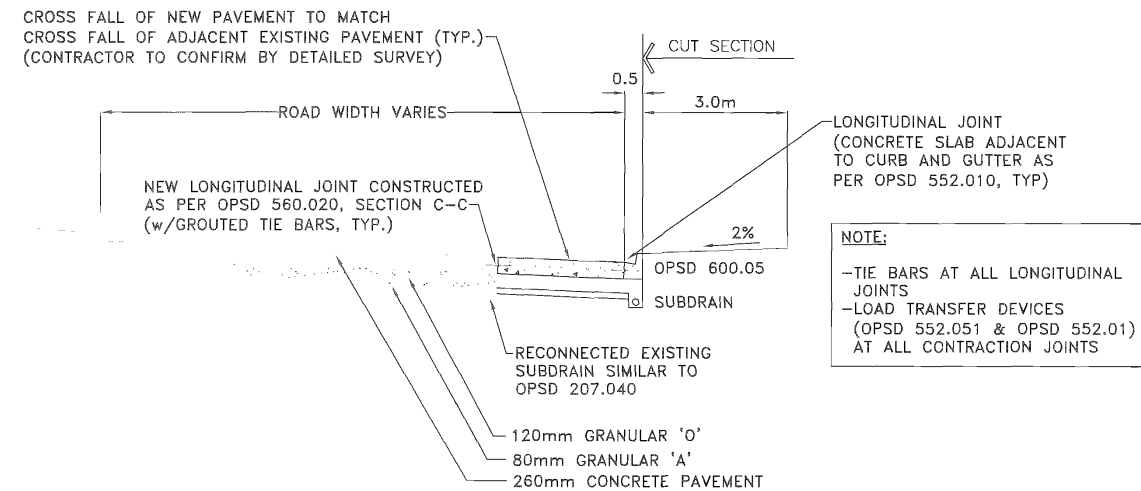
**TYPICAL CROSS-SECTION**  
(N.T.S.)



**RAISED MEDIAN (SECTION A-A)**  
(N.T.S.)



**TYPICAL REMOVALS FOR CONCRETE PAVEMENT WIDENING**  
(N.T.S.)



**TYPICAL NEW CONCRETE PAVEMENT CONSTRUCTION**  
(N.T.S.)

**NOTE:**

- TIE BARS AT ALL LONGITUDINAL JOINTS
- LOAD TRANSFER DEVICES (OPSD 552.051 & OPSD 552.01) AT ALL CONTRACTION JOINTS

NO.	REVISION	DATE	INIT.
2	ISSUED FOR TENDER	03/26/07	S.B.B.
1	ISSUED FOR REVIEW	03/09/07	S.B.B.
0	ISSUED FOR REVIEW	08/01/06	S.B.B.

**NOTES**

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THE LOCATION AND DEPTH OF UTILITY CROSSINGS ARE NOT GUARANTEED AND ANY DISCOVERED DURING THE WORK WHICH ARE IN CONFLICT WITH PROPOSED SERVICES WILL BE DEALT WITH AS EXTRA WORK IF WARRANTED.

CAD R.R.B. / W.H.N.	<b>BENCH MARK</b> TABLET SET IN CONC. SIDEWALK IN BRIDGE OVER G.E.W. AT NORTH END OF STRUCTURE, ON WEST SIDE, 2m NORTH OF EXPANSION JOINT.
DESIGN S.B.B.	
CHECKED BY N.P.	

ELEV. 123.150



**Niagara Region**  
Public Works and Utilities Department  
**Delcan**

CONTRACT No.:2007-T-108 (RN 07-08)  
**GLENDALE AVENUE SIDEWALK**  
REGIONAL ROAD 81  
in the TOWN OF NIAGARA-ON-THE-LAKE

TYPICAL SECTIONS AND DETAILS

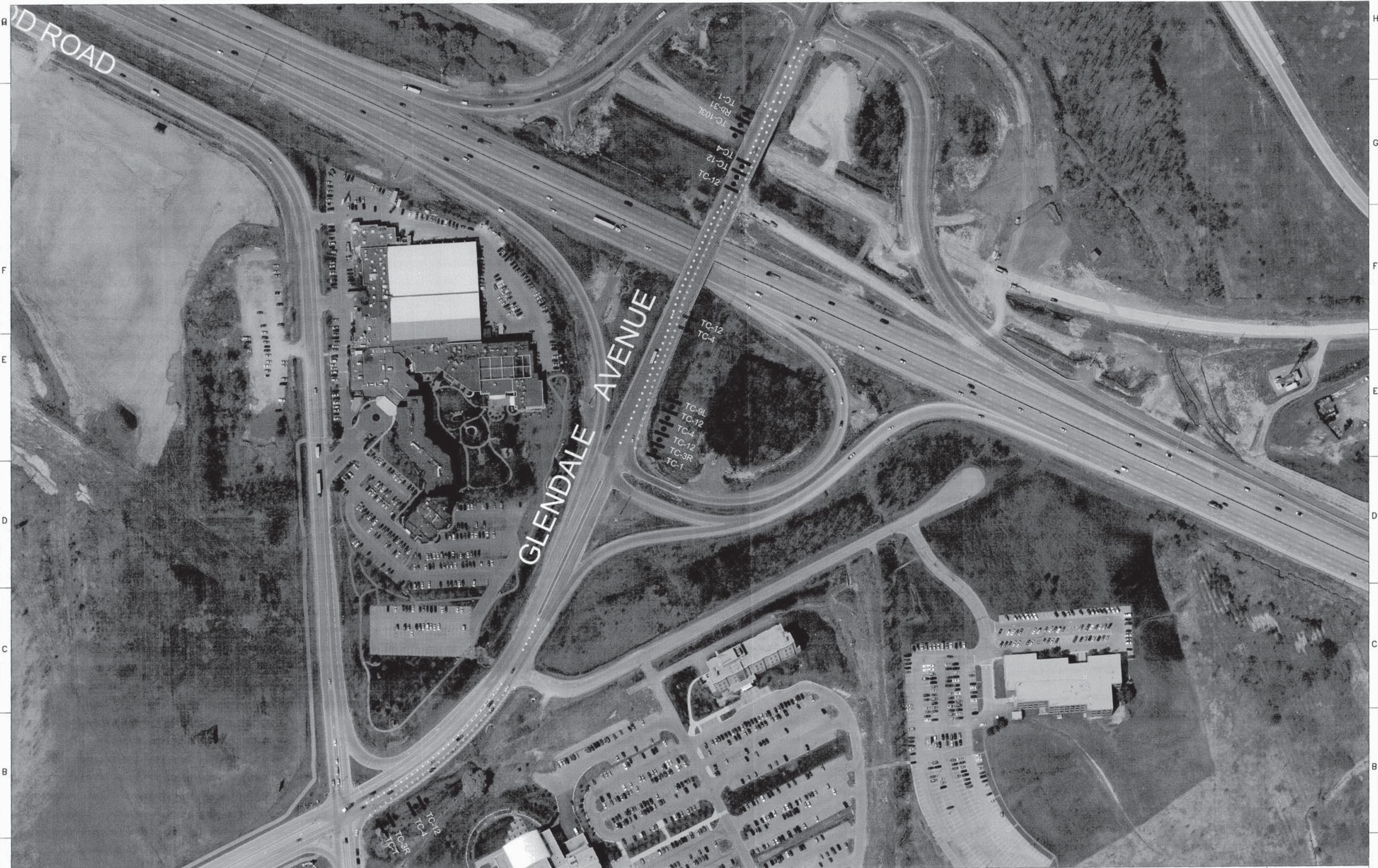
FIELD NOTES "M.T.O."	
DATE 03/26/2007	
SCALE N.T.S.	
DWG No. TN-1304-10	
MUN. REF. No.	REV. 2

ISSUED FOR TENDER  
MARCH 26, 2007

TN-1304-10 TYPICAL SECTIONS AND DETAILS GLENDALE AVENUE



\\DATA\TN1304\VF #0\CAD FILES.....Glendale Avenue Sidewalk - Detailed Design.dwg  
XREF = \\DATA\TN1304\VF #0\CAD FILES.....Glendale - QEW Baseplan.dwg



NO.	REVISION	DATE	INT.

NOTES  
THIS PLAN IS SCHEMATIC ONLY. IN ALL CASES, THE ONTARIO TRAFFIC MANUAL "BOOK 7 - TEMPORARY CONDITIONS" TAKES PRECEDENCE.

CAD  
R.R.B. /  
W.H.N.  
DESIGN  
S.B.B.  
CHECKED BY  
N.P.  
ELEV. 123.160

BENCH MARK  
TABLET SET IN CONC.  
SIDEWALK IN BRIDGE OVER  
Q.E.W., AT NORTH END OF  
STRUCTURE, ON WEST SIDE,  
2m NORTH OF EXPANSION  
JOINT.  
REGISTERED PROFESSIONAL ENGINEER  
S.B. BRANT  
PROVINCE OF ONTARIO

Niagara  
Region  
Public Works and Utilities Department  
Delcan

CONTRACT No.:2007-T-108 (RN 07-08)  
GLENDALE AVENUE SIDEWALK CONSTRUCTION  
REGIONAL ROAD 89  
in the Town of NIAGARA-ON-THE-LAKE  
CONSTRUCTION STAGE 2

FIELD NOTES "SUDA & MALESZYK"  
DATE 03/26/07  
SCALE N.T.S.  
DWG No. TN-1304-STAGE2  
MUN. REF. No. REV. 0

TN-1304-STAGE2 GLENDALE AVENUE SIDEWALK CONSTRUCTION STAGE 1 AND DETOUR PLAN

ISSUED FOR TENDER  
MARCH 26, 2007





**APPENDIX B**

# Borehole Records



## LIST OF SYMBOLS

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

### I. GENERAL

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

### II. STRESS AND STRAIN

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

### III. SOIL PROPERTIES

#### (a) Index Properties

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

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

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

#### (b) Hydraulic Properties

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

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

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

#### (d) Shear Strength

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

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

Notes: 1  
2

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

## LIST OF ABBREVIATIONS

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

### I. SAMPLE TYPE

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

### II. PENETRATION RESISTANCE

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

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

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

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

**PH:** Sampler advanced by hydraulic pressure

**PM:** Sampler advanced by manual pressure

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

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

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

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

### III. SOIL DESCRIPTION

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

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

#### (b) Cohesive Soils Consistency

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

### IV. SOIL TESTS

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

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

### V. MINOR SOIL CONSTITUENTS

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

## WEATHERINGS STATE

**Fresh:** no visible sign of weathering

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

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

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

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

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

## BEDDING THICKNESS

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

## JOINT OR FOLIATION SPACING

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

## GRAIN SIZE

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

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

## CORE CONDITION

### Total Core Recovery (TCR)

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

### Solid Core Recovery (SCR)

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

### Rock Quality Designation (RQD)

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

## DISCONTINUITY DATA

### Fracture Index

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

### Dip with Respect to Core Axis

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

### Description and Notes

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

### Abbreviations

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

PROJECT		1671430 WO2		RECORD OF BOREHOLE No ARB-1		SHEET 1 OF 1		METRIC										
G.W.P.		2423-15-00		LOCATION		N 4779855.5; E 332139.8 MTM NAD 83 ZONE 10 (LAT. 43.158062; LONG. -79.163817)		ORIGINATED BY JP										
DIST		Central HWY QEW		BOREHOLE TYPE		152 mm O.D. Hollow Stem Augers; CME 55 Track Mounted Drill Rig		COMPILED BY EN										
DATUM		Geodetic		DATE		October 26, 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 (%)
121.0	GROUND SURFACE							20	40	60	80	100						
0.0	ASPHALT (120 mm)																	
0.1	Silty clay, some sand (FILL) Firm Brown Moist		1	SS	50													
120.0			2A	SS	5													
1.0	Sand and gravel, some silt, trace to some clay (FILL) Very loose to very dense Brown Moist		2B	SS	5													
			3A	SS	3													
			3B	SS	3													
			4	SS	5													
			5	SS	50/0.05													
116.4	END OF BOREHOLE																	
4.6	NOTES:  1. Borehole terminated at 4.6 m due to auger refusal on an obstruction.  2. Open borehole dry on completion of drilling.																	

GTA-MTO 001 S:\CLIENTS\MTQEW-GLENDALE\02\_DATA\GINTQEW-GLENDALE.GPJ GAL-GTA.GDT 19-4-23

PROJECT		RECORD OF BOREHOLE No ARB-2				SHEET 1 OF 3		METRIC				
G.W.P.		2423-15-00		LOCATION		N 4779841.8; E 332133.3 MTM NAD 83 ZONE 10 (LAT. 43.157938; LONG. -79.163898)		ORIGINATED BY JP				
DIST		Central HWY QEW		BOREHOLE TYPE		152 mm O.D. Hollow Stem Augers; CME 55 Track Mounted Drill Rig		COMPILED BY EN				
DATUM		Geodetic		DATE		October 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 (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w		
121.5	GROUND SURFACE											
0.0	ASPHALT (150 mm)											
0.2	Sand and gravel, some silt to silty (FILL) Compact to very dense Brown Moist		1	SS	65							
			2	SS	14							
120.1												
1.4	Silty clay, some sand to sandy, trace to some gravel, trace organics at 4.0 m (FILL) Firm to stiff Brown, oxidation staining at 4.9 m Moist		3	SS	7							
			4	SS	6							
			5	SS	6							6 13 44 35
			6	SS	8							
			7	SS	9							
115.9												
5.6	Sand and gravel, some silt, trace to some clay (FILL) Very dense Red-brown to black Moist		8A									
115.1	- Concrete fragments between 6.1 m to 6.2 m		8B	SS	53							
6.4	SILTY CLAY to CLAY, trace to some sand, trace gravel Stiff to firm Brown grey to grey below 14.8 m Moist to wet below 14.8 m  - Rootlets encountered at 8.0 m.		9	SS	7						46	2 7 36 55
			10	SS	10							
			11	SS	10						48	0 3 35 62
			12	SS	5							
			13	SS	8						52	0 3 35 62
	- Red sand pockets at 14.0 m and 14.3 m.											

Continued Next Page

+<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 19-4-26



PROJECT <u>1671430 WO2</u>		<b>RECORD OF BOREHOLE No ARB-2</b>		SHEET 2 OF 3		<b>METRIC</b>	
G.W.P. <u>2423-15-00</u>		LOCATION <u>N 4779841.8; E 332133.3 MTM NAD 83 ZONE 10 (LAT. 43.157938; LONG. -79.163898)</u>		ORIGINATED BY <u>JP</u>			
DIST <u>Central</u> HWY <u>QEW</u>		BOREHOLE TYPE <u>152 mm O.D. Hollow Stem Augers; CME 55 Track Mounted Drill Rig</u>		COMPILED BY <u>EN</u>			
DATUM <u>Geodetic</u>		DATE <u>October 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 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		W <sub>p</sub>	W	W <sub>L</sub>		
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × REMOULDED					
--- CONTINUED FROM PREVIOUS PAGE ---														
	SILTY CLAY to CLAY, trace to some sand, trace gravel Stiff to firm Brown grey to grey below 14.8 m Moist to wet below 14.8 m		14	SS	4		106							
							105							
							104							
			15	SS	5		103							
							102							
101.4							101							
20.1	CLAYEY SILT, some sand, trace gravel Soft to stiff Grey Wet		16	SS	2		100							3 17 50 30
							99							
							98							
			17	SS	50/0.08		97							
95.8	- Auger refusal at a depth of 25.7 m, switch to rock coring						96							
25.9	GRANITE boulder Grey		18	SS	50/0.13		95							
	Sandy SILT, some clay Very dense Red-grey Moist						94							
94.6							93							
26.9	SILT and SAND, trace to some clay, trace to some gravel Very dense Red Moist		19	SS	50/0.08		92							
			20	SS	50/0.13									1 55 38 6

Continued Next Page

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

GTA-MTO 001 S:\CLIENTS\TOQEW-LENDALE\02\_DATA\INTOQEW-LENDALE.GPJ GAL-GTA.GDT 19-4-26

PROJECT		1671430 WO2		RECORD OF BOREHOLE No ARB-2				SHEET 3 OF 3				METRIC					
G.W.P.		2423-15-00		LOCATION		N 4779841.8; E 332133.3 MTM NAD 83 ZONE 10 (LAT. 43.157938; LONG. -79.163898)				ORIGINATED BY JP							
DIST		Central HWY QEW		BOREHOLE TYPE		152 mm O.D. Hollow Stem Augers; CME 55 Track Mounted Drill Rig				COMPILED BY EN							
DATUM		Geodetic		DATE		October 26, 29 and 30, 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 $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
	--- CONTINUED FROM PREVIOUS PAGE ---																
88.9	SILT and SAND, trace to some clay, trace to some gravel Very dense Red Moist																
32.6	- Rock fragments at a depth of about 32 m		21	SS	50/0.14												
	SHALE (BEDROCK)																
	Bedrock cored from 32.6 m to 36.3 m		1	RC	REC 100%											RQD = 75%	
	For rock coring details refer to Record of Drillhole ARB-2		2	RC	REC 96%											RQD = 91%	
			3	RC	REC 100%											RQD = 100%	
			4	RC	REC 100%											RQD = 78%	
85.2	END OF BOREHOLE																
36.3	NOTE: 1. Water level in open borehole at a depth of 4.3 m (Elev. 117.2 m) during drilling.																

GTA-MTO 001 S:\CLIENTS\MTQEW-LENDALE\02\_DATA\INTQEW-LENDALE.GPJ GAL-GTA.GDT 19-4-26

PROJECT: 1671430 WO2

RECORD OF DRILLHOLE: **ARB-2**

SHEET 1 OF 1

LOCATION: N 4779841.77 ;E 332133.32

DRILLING DATE:

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME55 Truck-Mount

DRILLING CONTRACTOR: Geo-Environmental

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN	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)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																				
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33		Continued from Record of Borehole ARB-2  Slightly to moderately weathered, very thin to medium bedded, red, fine grained, non-porous to slightly porous, weak, SHALE with very thin to thin, medium strong LIMESTONE interbeds (Queenston Formation)  Hard layers: (Grey LIMESTONE) 33.46 m - 33.49 m, 34.80 m - 34.86 m, 36.21 m - 36.23 m  Clay layer: 36.10 m - 36.17 m		88.90 32.60	1																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																															

DEPTH SCALE

1 : 50



LOGGED: JP

CHECKED: EN




PROJECT <u>1671430 WO2</u>		<b>RECORD OF BOREHOLE No ARB-3</b>		SHEET 2 OF 3		<b>METRIC</b>	
G.W.P. <u>2423-15-00</u>		LOCATION <u>N 4779841.4; E 332172.2 MTM NAD 83 ZONE 10 (LAT. 43.157933; LONG. -79.163420)</u>		ORIGINATED BY <u>JP</u>			
DIST <u>Central</u> HWY <u>QEW</u>		BOREHOLE TYPE <u>152 mm O.D. Hollow Stem Augers; CME 55 Track Mounted Drill Rig</u>		COMPILED BY <u>EN</u>			
DATUM <u>Geodetic</u>		DATE <u>October 22 and 23, 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	20 40 60 80 100	10 20 30				
	--- CONTINUED FROM PREVIOUS PAGE ---													
	SILTY CLAY, trace sand, trace gravel Stiff to firm Red-grey Moist		14	SS	0		102	1.5				42		0 3 41 56
							101	1.8 1.4						
							100							
			15	SS	6		99							
							98							
97.6							97							
20.1	SANDY CLAYEY SILT, some gravel Hard Red-grey Moist		16	SS	50/0.14		96						18 24 43 15	
							95							
94.7							94							
23.0	SILT, trace to some clay, trace sand Very dense Red to grey Moist		17	SS	73		93						0 3 86 11	
							92							
							91							
			18	SS	54		90							
							89							
88.8							88							
28.9	CLAYEY SILT, some sand, some shale fragments (TILL/RESIDUAL SOIL) Hard Red to grey Moist		19	SS	50/0.05									
87.7														

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 19-4-26

PROJECT		RECORD OF BOREHOLE No ARB-3				SHEET 3 OF 3		METRIC									
G.W.P. 1671430 WO2		LOCATION N 4779841.4; E 332172.2 MTM NAD 83 ZONE 10 (LAT. 43.157933; LONG. -79.163420)				ORIGINATED BY JP											
DIST Central HWY QEW		BOREHOLE TYPE 152 mm O.D. Hollow Stem Augers; CME 55 Track Mounted Drill Rig				COMPILED BY EN											
DATUM Geodetic		DATE October 22 and 23, 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 $\gamma$ kN/m <sup>3</sup>	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	W <sub>p</sub>	W	W <sub>L</sub>			
30.0	SHALE (BEDROCK)  Bedrock cored from 30.0 m to 33.2 m.  For rock coring details refer to Record of Drillhole ARB-3.		1	RC	REC 100%												RQD = 52%
			2	RC	REC 100%												RQD = 68%
			3	RC	REC 88%												RQD = 84%
84.5			4	RC	REC 100%												RQD = 66%
33.2	END OF BOREHOLE  NOTE:  1. Water level measured at a depth of 9.1 m (Elev. 108.6 m) on completion of drilling.																

GTA-MTO 001 S:\CLIENTS\MTQ\QEW-GLENDALE\02\_DATA\GINT\QEW-GLENDALE.GPJ GAL-GTA.GDT 19-4-26

PROJECT: 1671430 WO2

**RECORD OF DRILLHOLE: ARB-3**

SHEET 1 OF 1

LOCATION: N 4779841.38 ;E 332172.17

DRILLING DATE:

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	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																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																										
30		Continued from Record of Borehole ARB-3		87.70																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																												</

DEPTH SCALE

1 : 50

**GOLDER**

LOGGED: JP

CHECKED: EN

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<b>PROJECT</b> 1671430 WO2		<b>RECORD OF BOREHOLE No ARB-4</b>		SHEET 1 OF 3		<b>METRIC</b>	
<b>G.W.P.</b> 2423-15-00		<b>LOCATION</b> N 4779827.2; E 332176.8 MTM NAD 83 ZONE 10 (LAT. 43.157806; LONG. -79.163365)		<b>ORIGINATED BY</b> JP			
<b>DIST</b> Central <b>HWY</b> QEW		<b>BOREHOLE TYPE</b> 152 mm O.D. Hollow Stem Augers; CME 55 Track Mounted Drill Rig		<b>COMPILED BY</b> EN			
<b>DATUM</b> Geodetic		<b>DATE</b> October 24 and 25, 2018		<b>CHECKED BY</b> LCC			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W <sub>p</sub>	W	W <sub>L</sub>		
								20 40 60 80 100	20 40 60 80 100					
117.1	GROUND SURFACE													
0.9	TOPSOIL (50 mm)		1	SS	8									
	Silty clay to clay, trace to some sand, trace rootlets (FILL)		2	SS	25									
	Stiff to very stiff		3	SS	29									
	Brown		4	SS	15									
	Moist													
114.1			5	SS	16									
3.1	SILTY CLAY to CLAY, trace to some gravel, trace to some sand		6	SS	17									
	Very stiff to stiff		7	SS	15									
	Brown to red-brown to red-grey													
	Moist		8	SS	9									
			9	SS	10									
			10	SS	9									
			11	SS	6									
			12	SS	3									
			13	SS	5									

Continued Next Page

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

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





+<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 ARB-4</b>		SHEET 3 OF 3		<b>METRIC</b>	
G.W.P. <u>2423-15-00</u>		LOCATION <u>N 4779827.2; E 332176.8 MTM NAD 83 ZONE 10 (LAT. 43.157806; LONG. -79.163365)</u>		ORIGINATED BY <u>JP</u>			
DIST <u>Central</u> HWY <u>QEW</u>		BOREHOLE TYPE <u>152 mm O.D. Hollow Stem Augers; CME 55 Track Mounted Drill Rig</u>		COMPILED BY <u>EN</u>			
DATUM <u>Geodetic</u>		DATE <u>October 24 and 25, 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 CONTENT			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>		GR	SA	SI	CL														
	--- CONTINUED FROM PREVIOUS PAGE ---																																	
	END OF BOREHOLE																																	
	NOTES:																																	
	4. Water level in standpipe piezometer measured as follows:																																	
	<table><tr><td>Date</td><td>Depth</td><td>Elevation</td></tr><tr><td>(mm/dd/yy)</td><td>(mbgs)</td><td>(m)</td></tr><tr><td>10/24/18</td><td>14.3</td><td>102.8</td></tr><tr><td>11/26/18</td><td>9.3</td><td>107.8</td></tr><tr><td>05/07/19</td><td>9.0</td><td>108.1</td></tr></table>	Date	Depth	Elevation	(mm/dd/yy)	(mbgs)	(m)	10/24/18	14.3	102.8	11/26/18	9.3	107.8	05/07/19	9.0	108.1																		
Date	Depth	Elevation																																
(mm/dd/yy)	(mbgs)	(m)																																
10/24/18	14.3	102.8																																
11/26/18	9.3	107.8																																
05/07/19	9.0	108.1																																

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

PROJECT		RECORD OF BOREHOLE				No HF-1		SHEET 2 OF 2		METRIC							
G.W.P. 2423-15-00		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.																

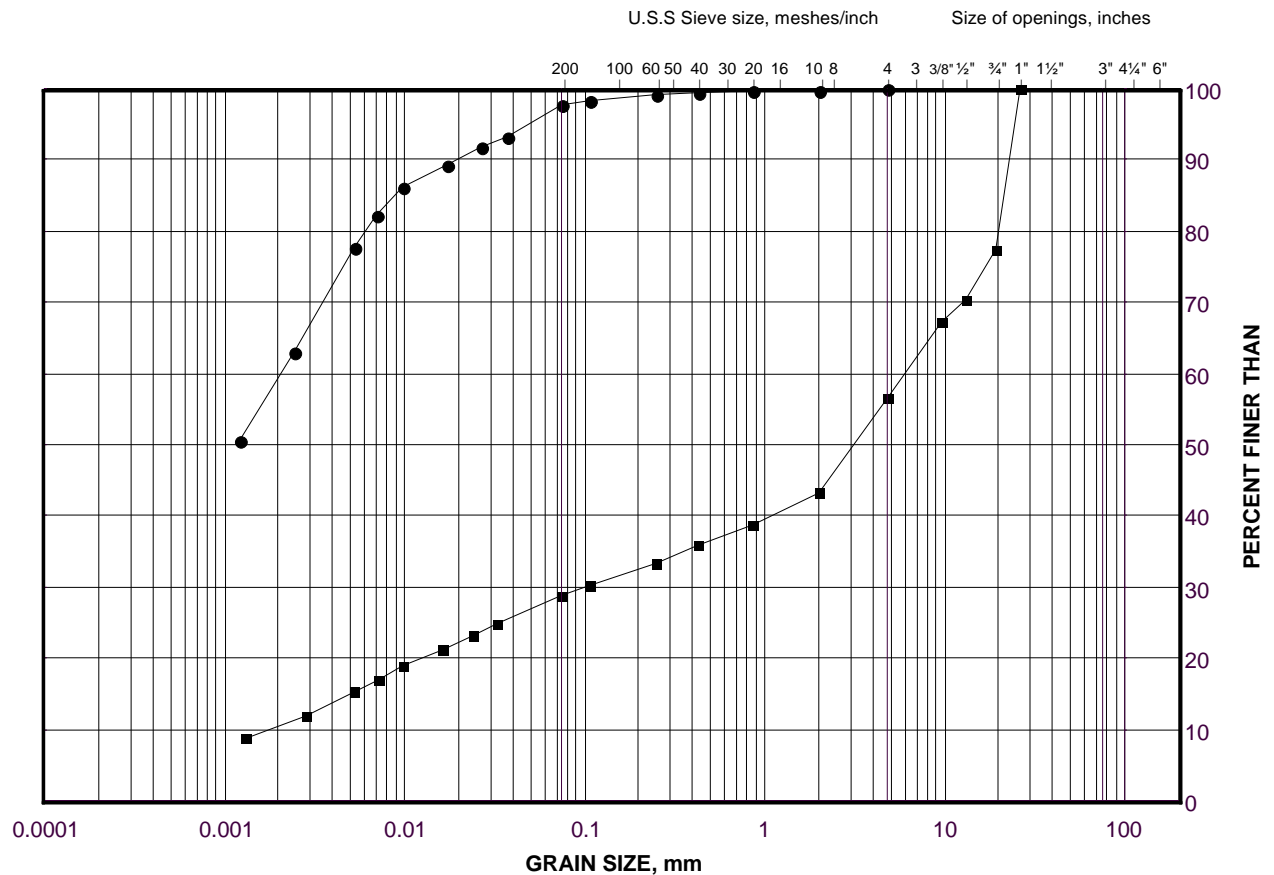
**APPENDIX C**

# Geotechnical Laboratory Test Results

# GRAIN SIZE DISTRIBUTION

Silty Clay to Clay Fill and Sand and Gravel Fill

FIGURE C-1



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	2	115.5
■	ARB-1	3B	119.3

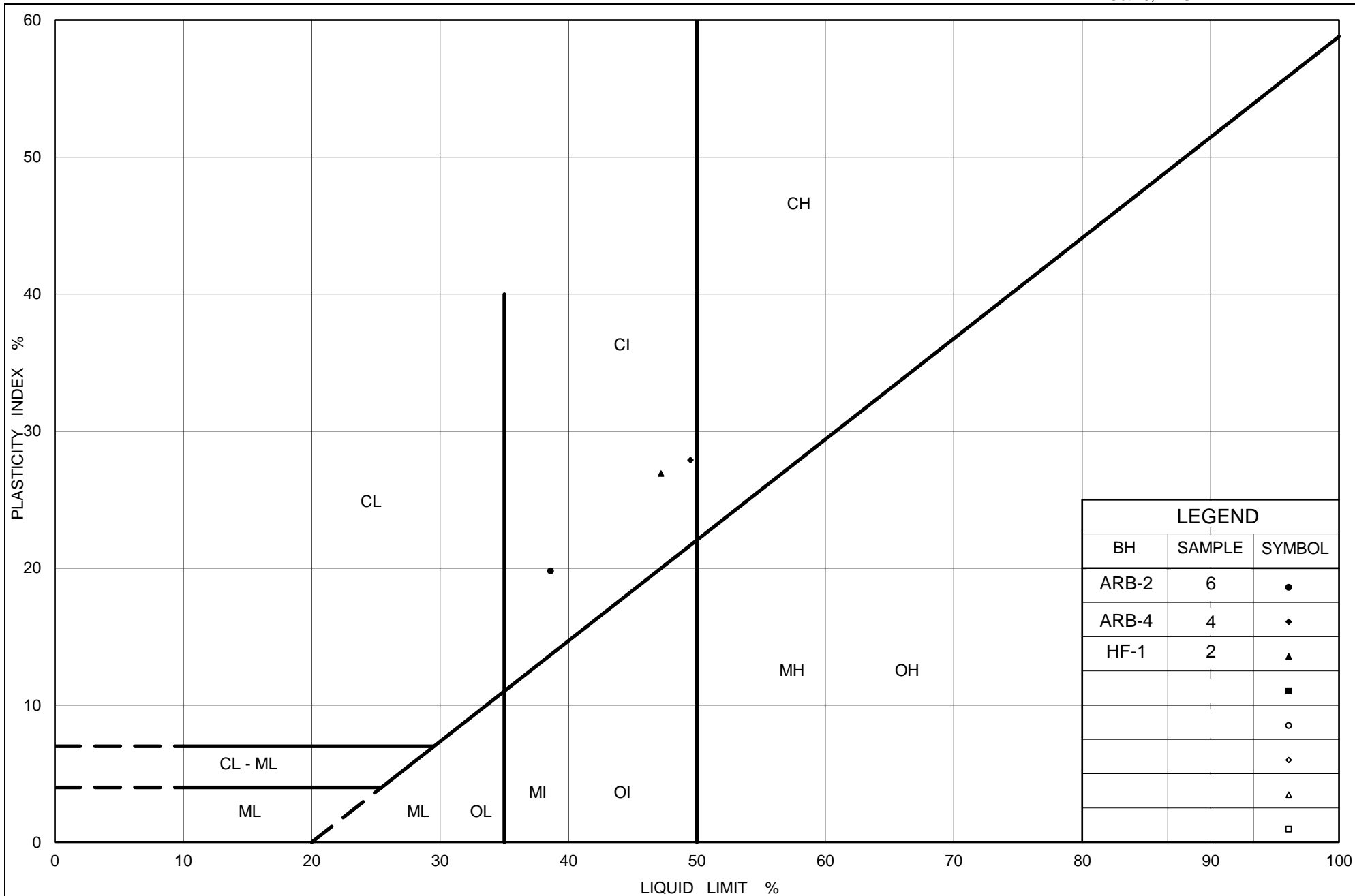
Project Number: 1671430

Checked By: MA/LCC

**Golder Associates**

Date: 22-Apr-19





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## PLASTICITY CHART

### Silty Clay to Clay Fill

Figure No. C-2

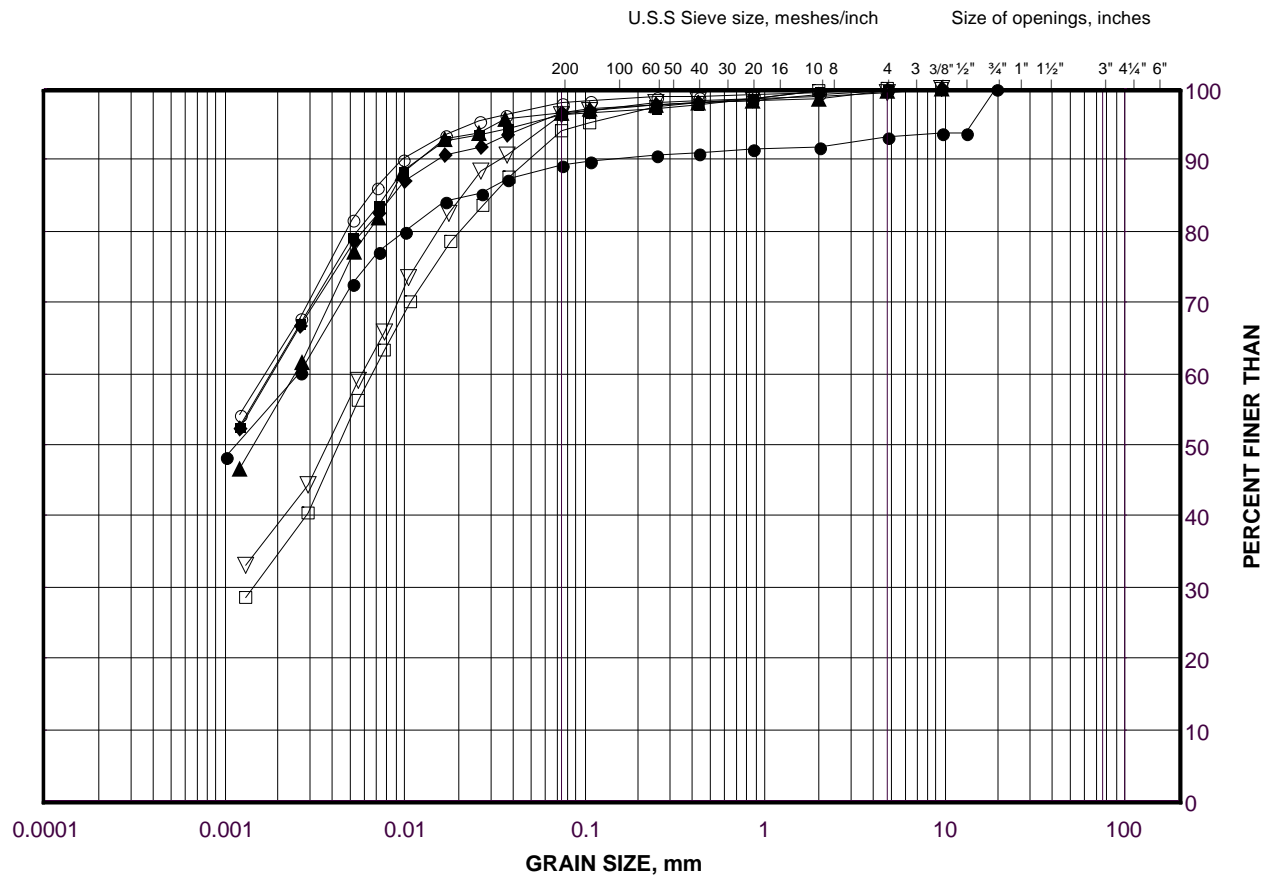
Project No. 1671430 (WO 002)

Checked By: MA/LCC

# GRAIN SIZE DISTRIBUTION

Silty Clay to Clay

FIGURE C-3A



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

## LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	ARB-4	10	107.7
■	ARB-2	11	110.5
◆	ARB-2	13	107.5
▲	ARB-3	14	102.2
▽	ARB-3	5	114.3
○	ARB-3	8	111.4
□	ARB-4	8	110.7

Project Number: 1671430

Checked By: MA/LCC

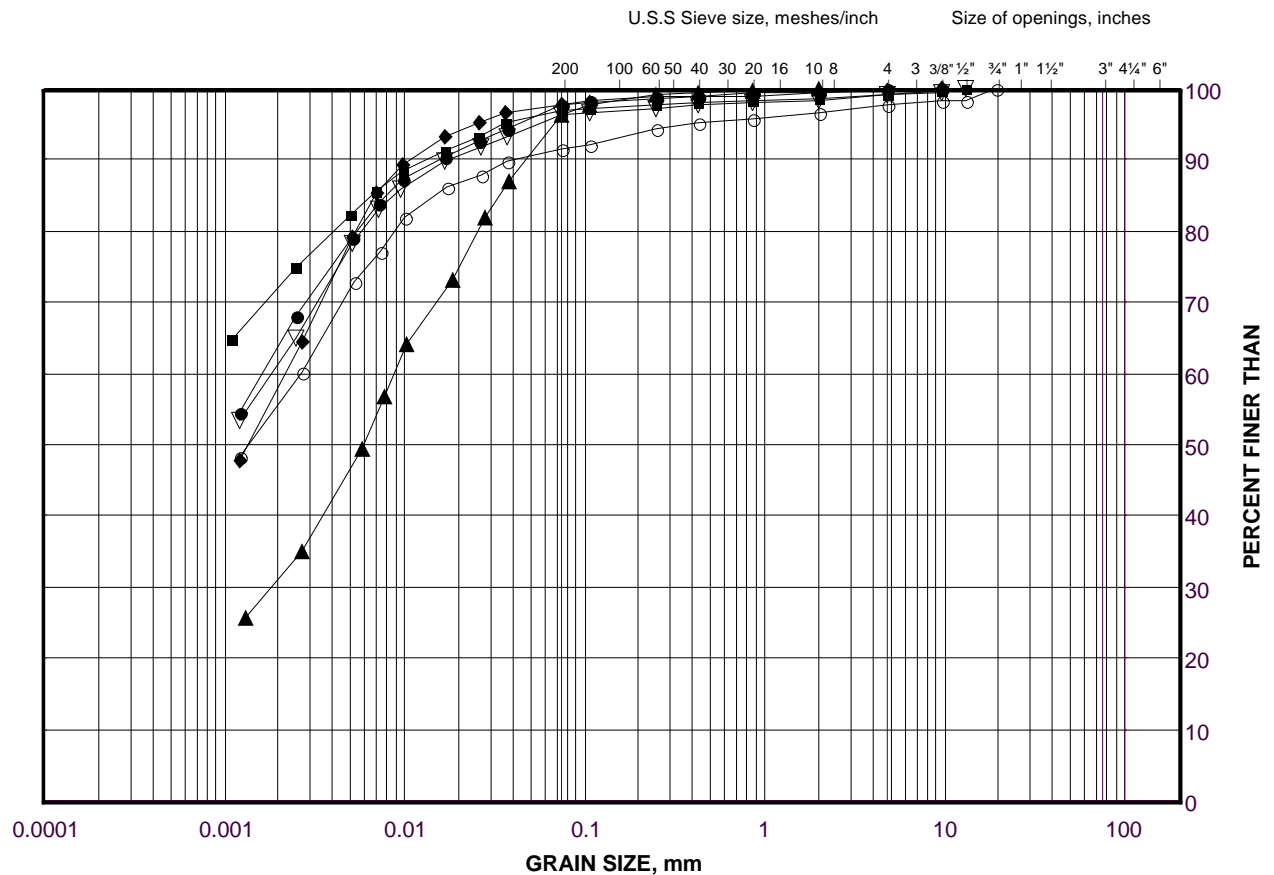
**Golder Associates**

Date: 22-Apr-19

# GRAIN SIZE DISTRIBUTION

Silty Clay to Clay

FIGURE C-3B



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

## LEGEND

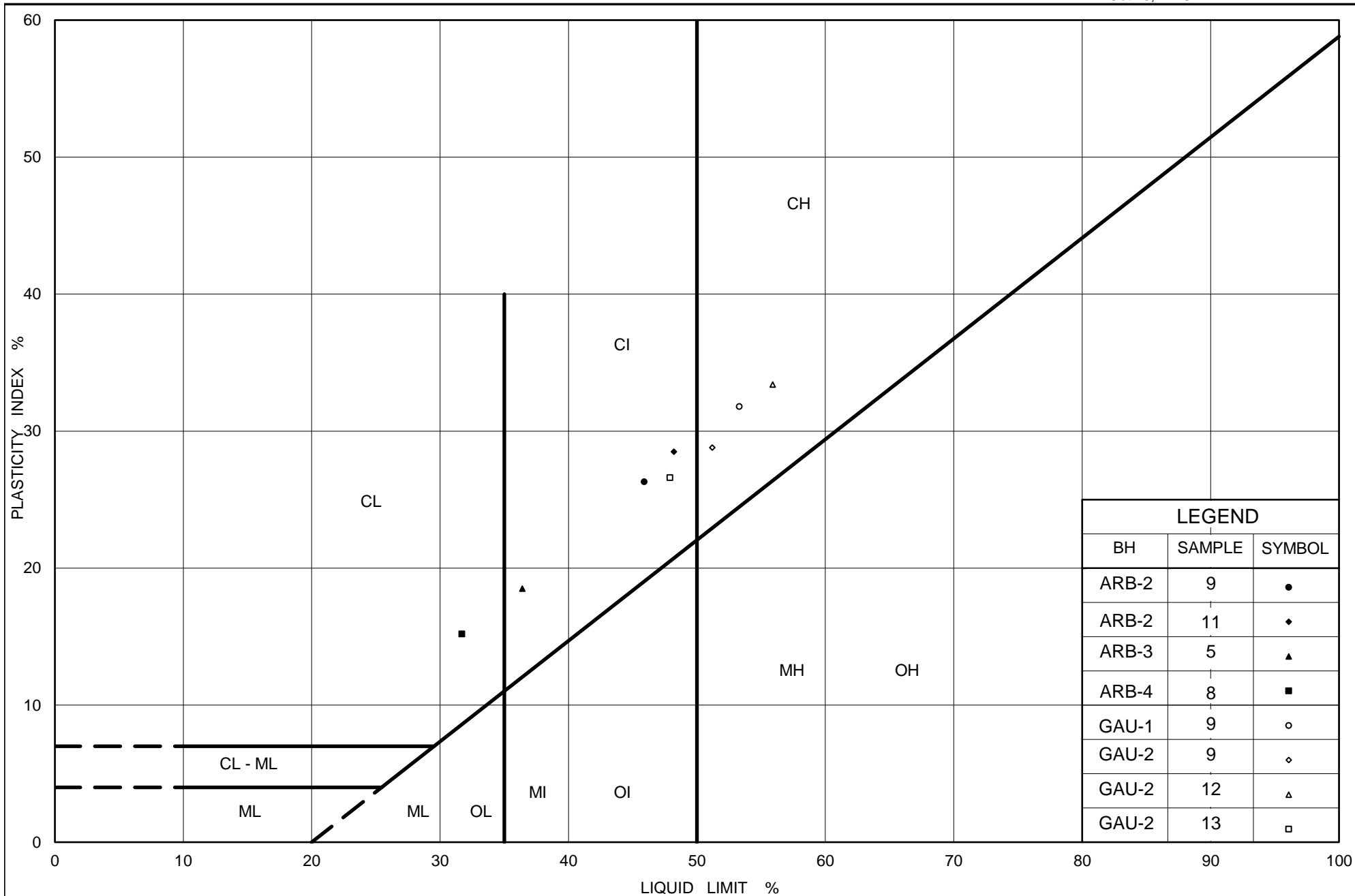
SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	HF-1	11	105.5
■	ARB-4	12	104.6
◆	HF-1	13	102.5
▲	HF-1	6	112.4
▽	HF-1	8	110.1
○	ARB-2	9	113.6

Project Number: 1671430

Checked By: MA/LCC

**Golder Associates**

Date: 22-Apr-19



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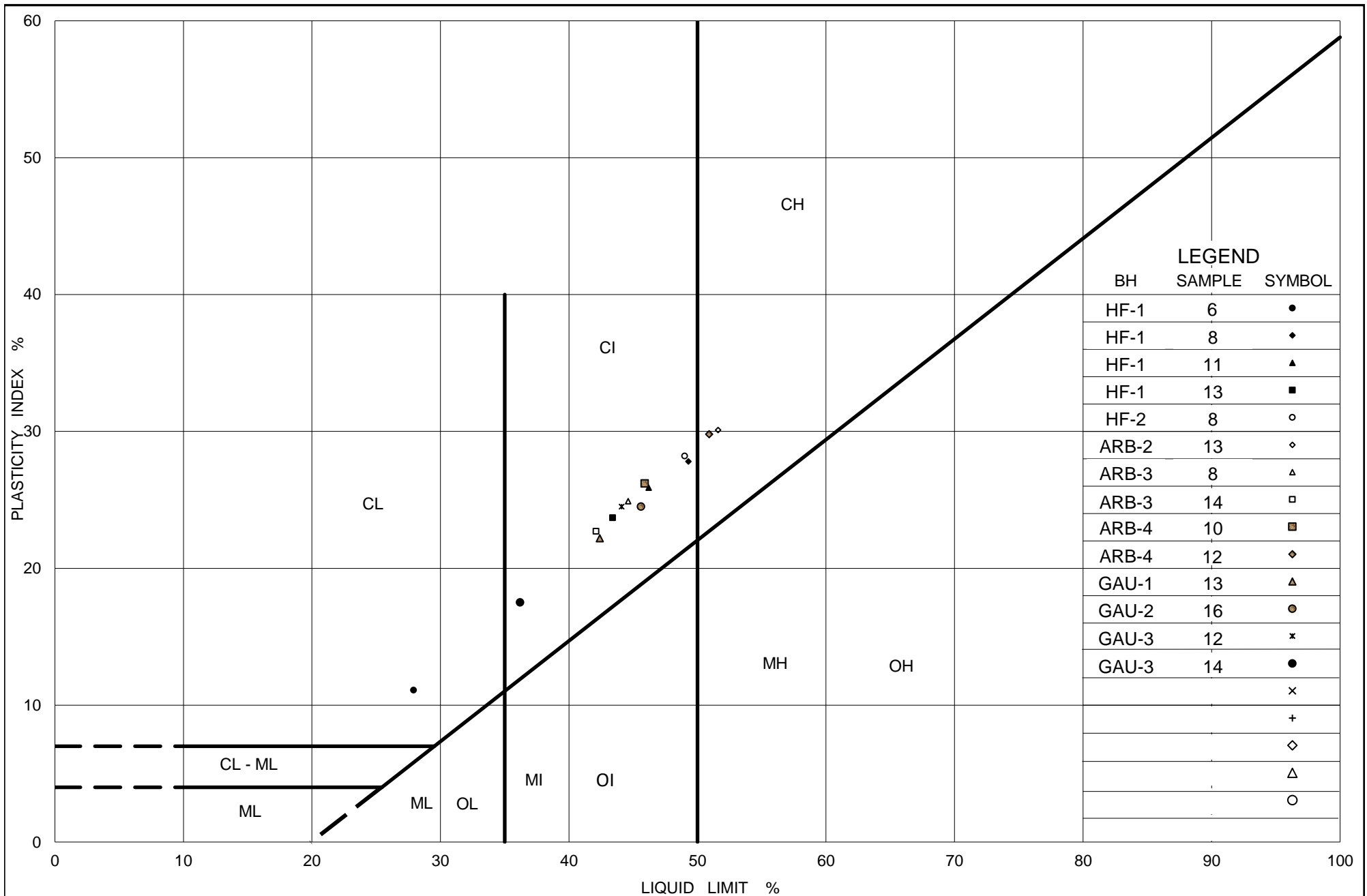
## PLASTICITY CHART

### Silty Clay to Clay

Figure No. C-4A

Project No. 1671430 (WO 002)

Checked By: MA/LCC



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## PLASTICITY CHART Silty Clay to Clay

Figure No. C-4B

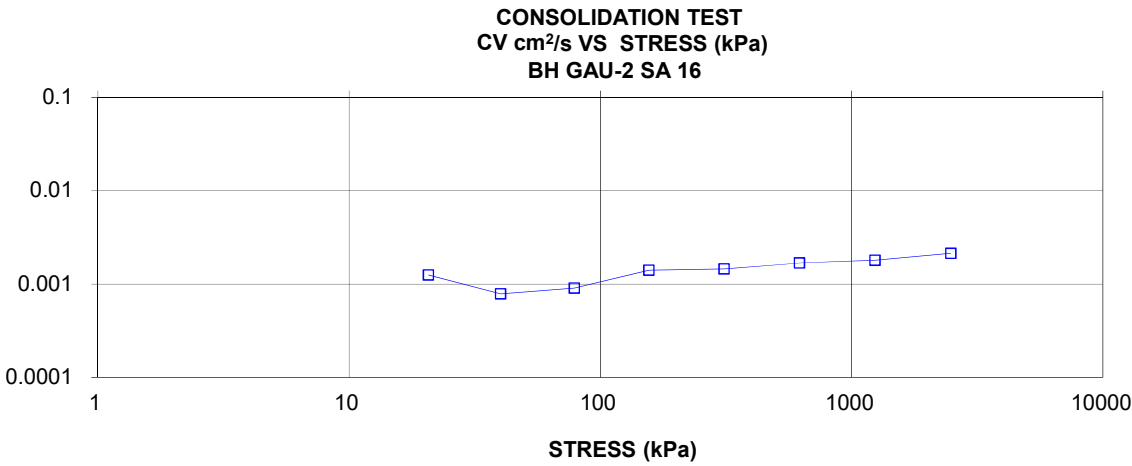
Project No. 1671430 (WO 002)

Checked By: MA/LCC

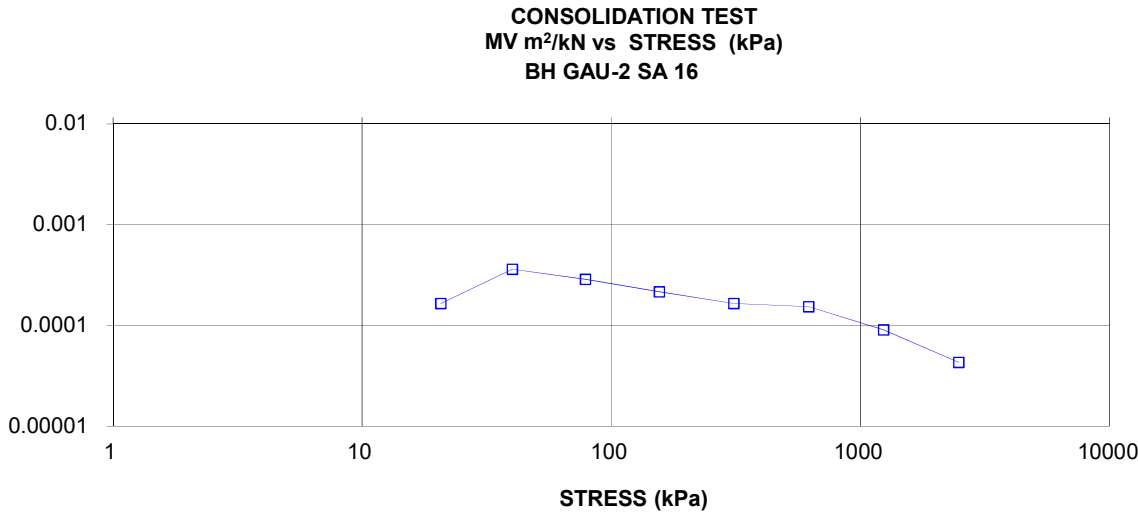
<div style="display: flex; justify-content: space-between;"> <div style="text-align: center;"> <b>CONSOLIDATION TEST SUMMARY</b>  <b>ASTM D2435/D2435M</b> </div> <div style="text-align: center;"> <b>FIGURE C-5A</b> </div> </div>				
<b>SAMPLE IDENTIFICATION</b>				
Project Number	1671430(WO002)	Sample 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>
kPa	cm	Ratio	cm	sec
				cv.
				m <sup>2</sup> /s
				mv
				m <sup>2</sup> /kN
				k
				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
40.08	2.535	0.913	2.544	1750
78.73	2.510	0.894	2.523	1500
156.03	2.468	0.862	2.489	936
310.38	2.403	0.813	2.436	866
619.74	2.282	0.722	2.343	694
1237.93	2.141	0.615	2.211	578
2480.25	2.005	0.512	2.073	427
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			
<div style="display: flex; justify-content: space-between;"> <div>Prepared By: LH</div> <div style="text-align: center;"><b>Golder Associates</b></div> <div>Checked By:</div> </div>				



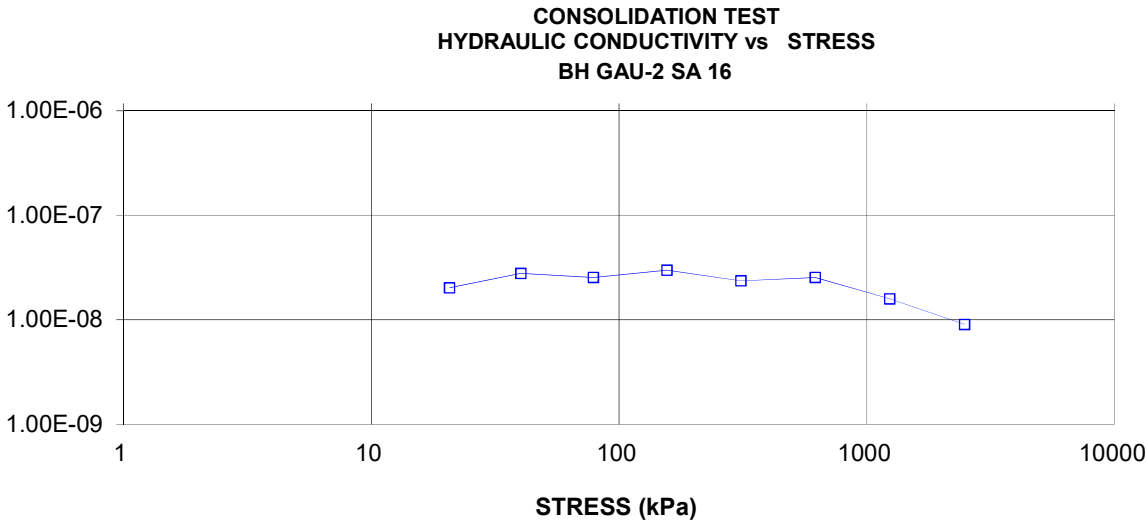
COEFFICIENT OF CONSOLIDATION,  
cm<sup>2</sup>/s

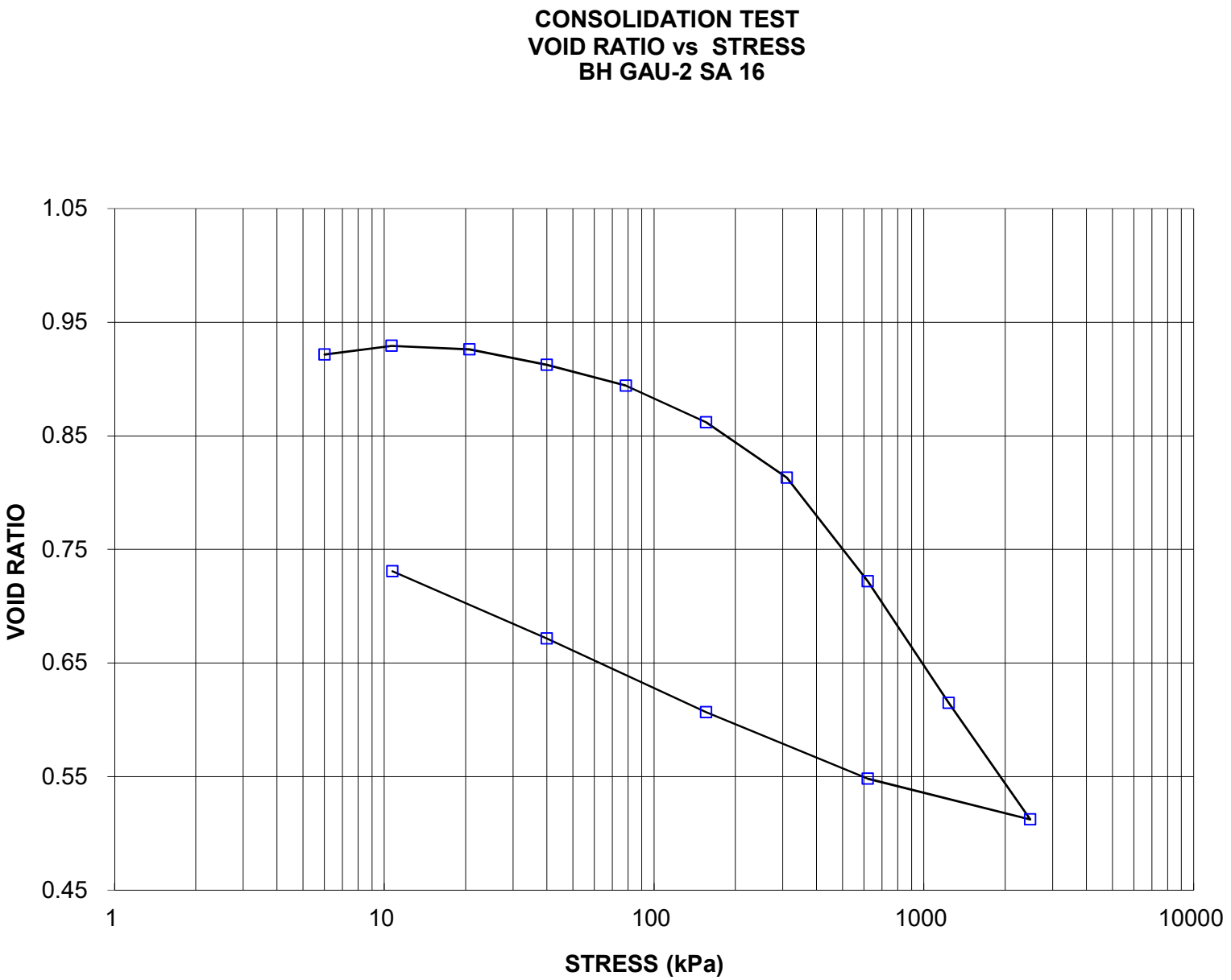


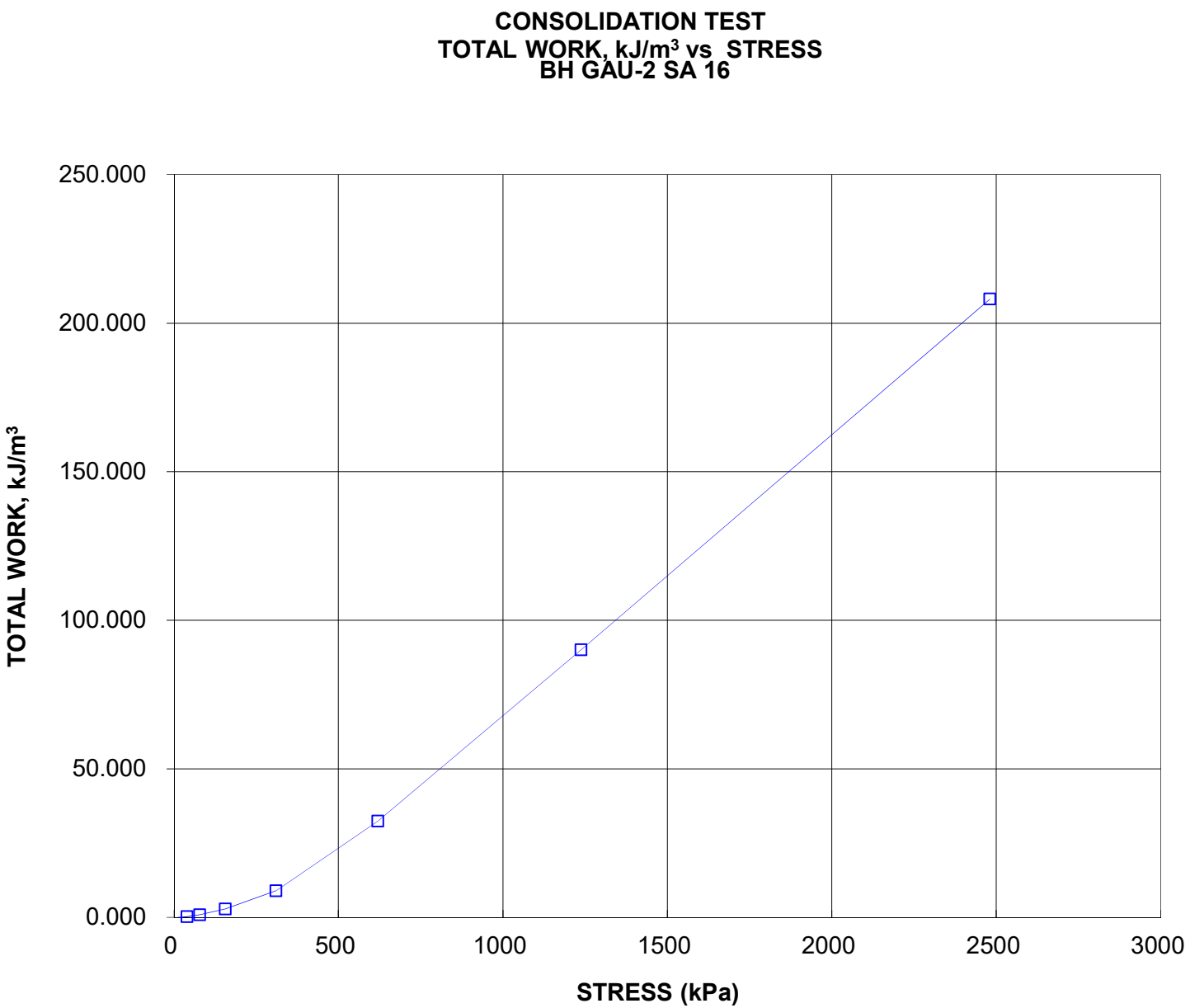
VOLUME COMPRESSIBILITY, m<sup>2</sup>/kN



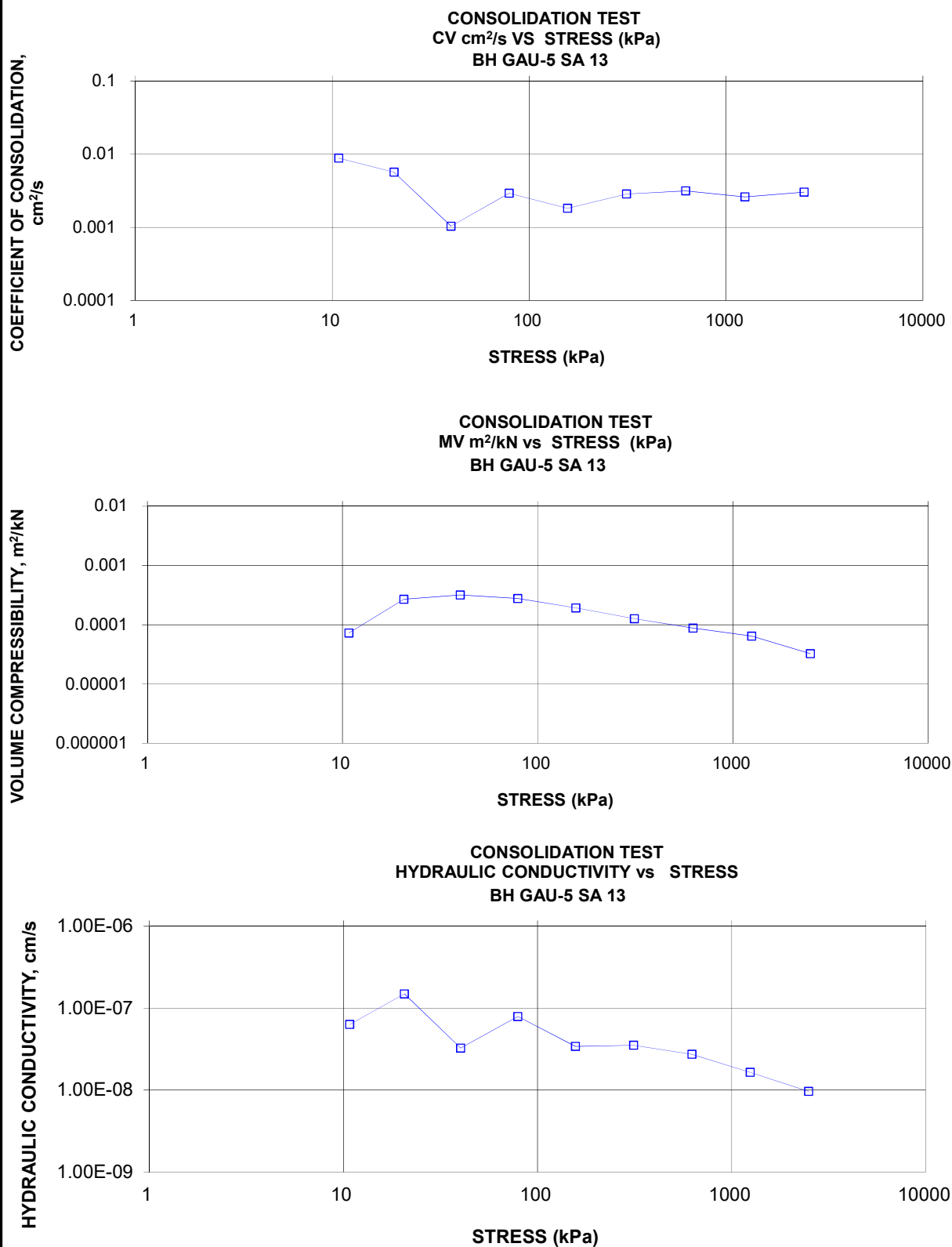
HYDRAULIC CONDUCTIVITY, cm/s

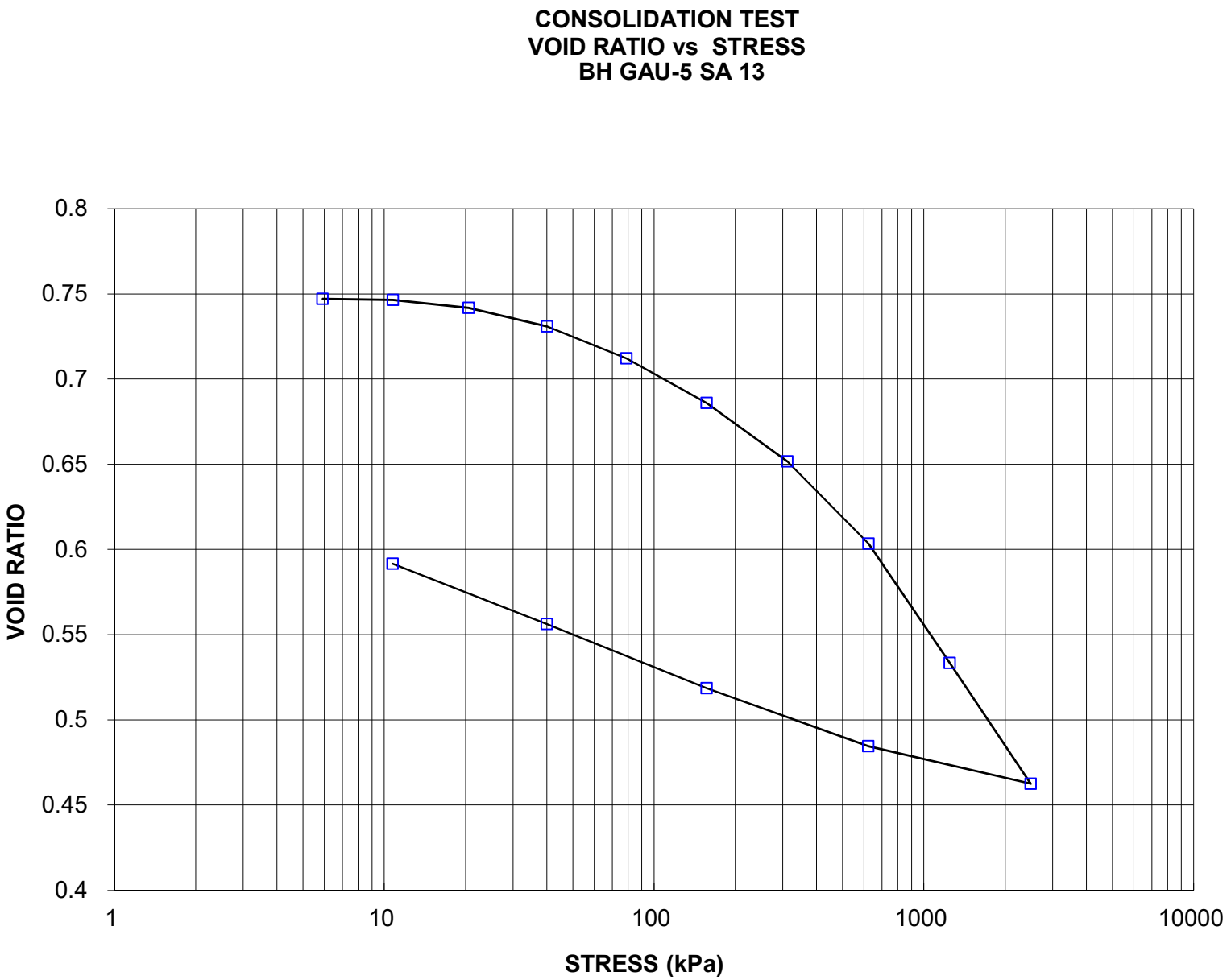


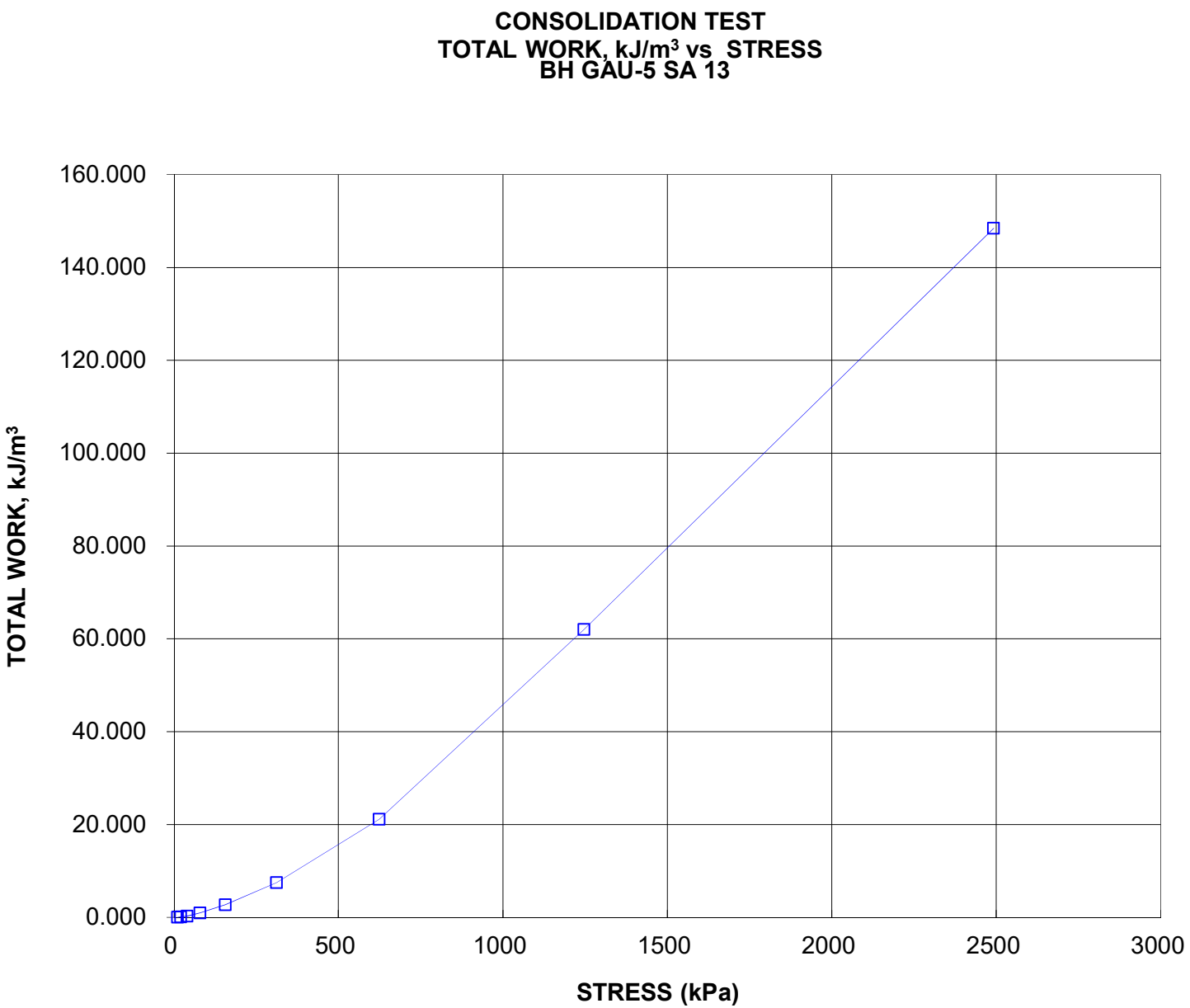




<div style="display: flex; justify-content: space-between;"> <div style="text-align: center;"> <b>CONSOLIDATION TEST SUMMARY</b>  <b>ASTM D2435/D2435M</b> </div> <div style="text-align: center;"> <b>FIGURE C-6A</b> </div> </div>				
<b>SAMPLE IDENTIFICATION</b>				
Project Number	1671430(WO002)	Sample 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 style="text-align: center;"><b>Golder Associates</b></div> <div>Checked By:</div> </div>				









**CONSOLIDATION TEST SUMMARY****FIGURE C-7A****ASTM D2435/D2435M****SAMPLE IDENTIFICATION**

Project Number	1671430(WO002)	Sample Number	10
Borehole Number	GAU-6	Sample Depth, ft	10.67-11.28

**TEST CONDITIONS**

Test Type	Laboratory Standard	Load Duration, hr	24
Oedometer Number	4		
Date Started	10/05/2018		
Date Completed	10/17/2018		

**SAMPLE DIMENSIONS AND PROPERTIES - INITIAL**

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

**TEST COMPUTATIONS**

Stress kPa	Corr. Height cm	Void Ratio	Average		t <sub>90</sub> sec	cv. cm <sup>2</sup> /s	mv m <sup>2</sup> /kN	k cm/s
	Height cm		Height cm					
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	4.42E-03	2.01E-05	8.72E-09	
40.14	2.557	0.914	2.563	778	1.79E-03	2.51E-04	4.41E-08	
79.10	2.534	0.897	2.545	756	1.82E-03	2.37E-04	4.31E-08	
156.94	2.507	0.877	2.520	360	3.74E-03	1.36E-04	4.97E-08	
312.60	2.467	0.846	2.487	413	3.17E-03	1.02E-04	3.16E-08	
624.05	2.391	0.790	2.429	452	2.77E-03	9.53E-05	2.58E-08	
1251.00	2.219	0.661	2.305	554	2.03E-03	1.08E-04	2.15E-08	
2495.95	2.074	0.553	2.147	470	2.08E-03	4.59E-05	9.35E-09	
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					

Note:

Consolidation loading and unloading schedule assigned by the client.

cv and k are approximate only based on t<sub>90</sub> estimated from Square Root of Time Method (ASTMD2435/2435M)

Specimen swelled under 10.80kPa.

**SAMPLE DIMENSIONS AND PROPERTIES - FINAL**

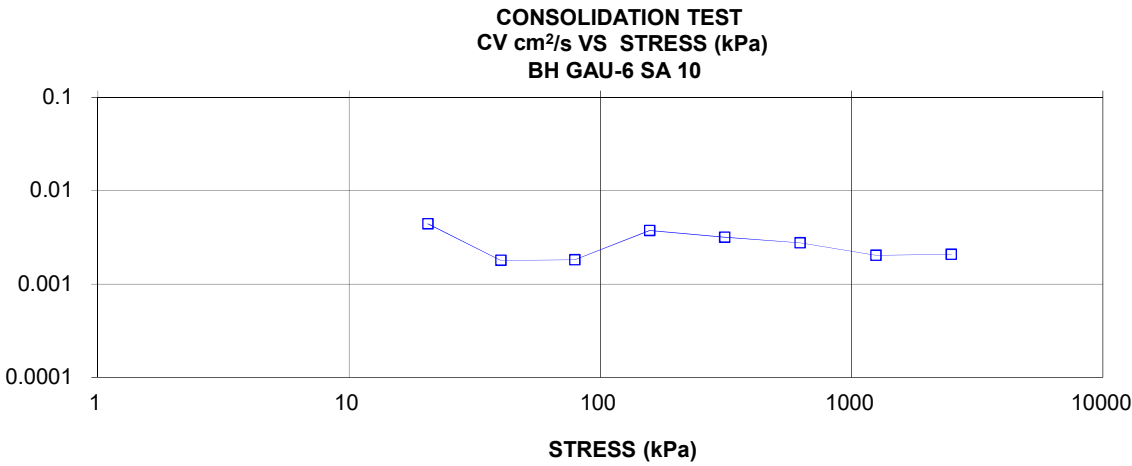
Sample Height, cm	2.36	Unit Weight, kN/m <sup>3</sup>	19.90
Sample Diameter, cm	6.33	Dry Unit Weight, kN/m <sup>3</sup>	15.40
Area, cm <sup>2</sup>	31.45	Specific Gravity, measured	2.78
Volume, cm <sup>3</sup>	74.38	Solids Height, cm	1.336
Water Content, %	29.20	Volume of Solids, cm <sup>3</sup>	42.01
Wet Mass, g	150.89	Volume of Voids, cm <sup>3</sup>	32.37
Dry Mass, g	116.79		

Prepared By: LH

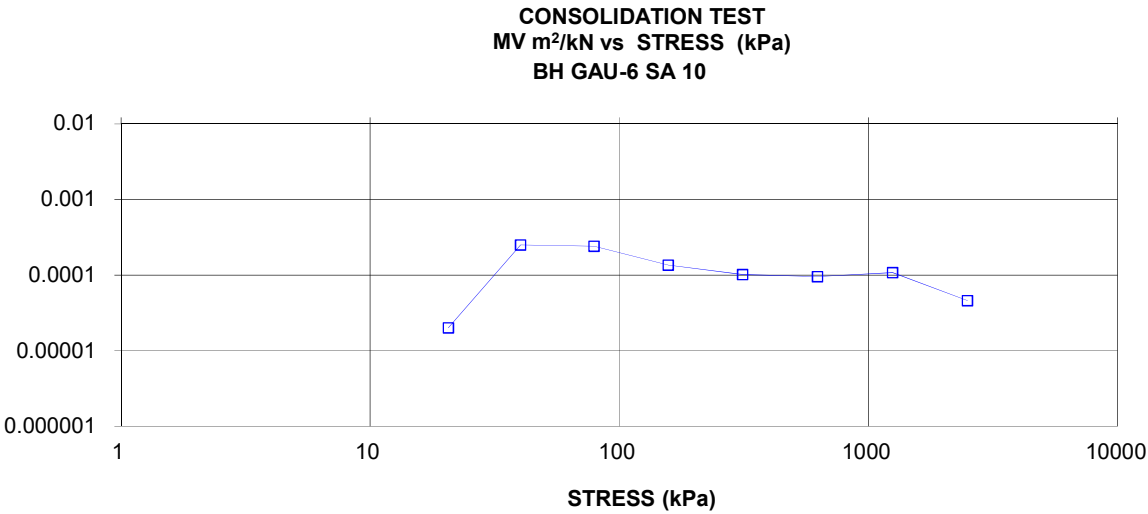
**Golder Associates**

Checked By:

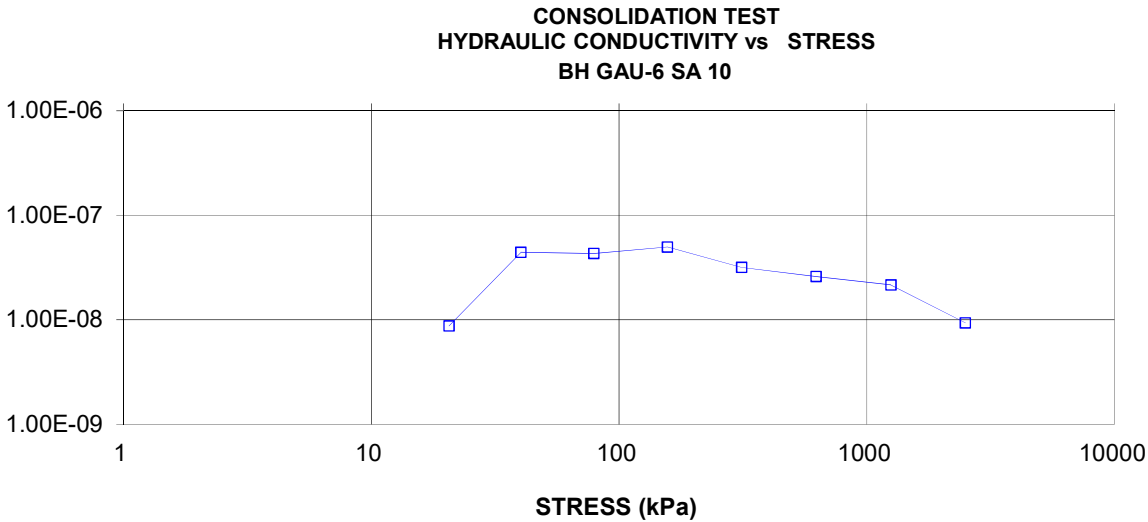
COEFFICIENT OF CONSOLIDATION,  
cm<sup>2</sup>/s

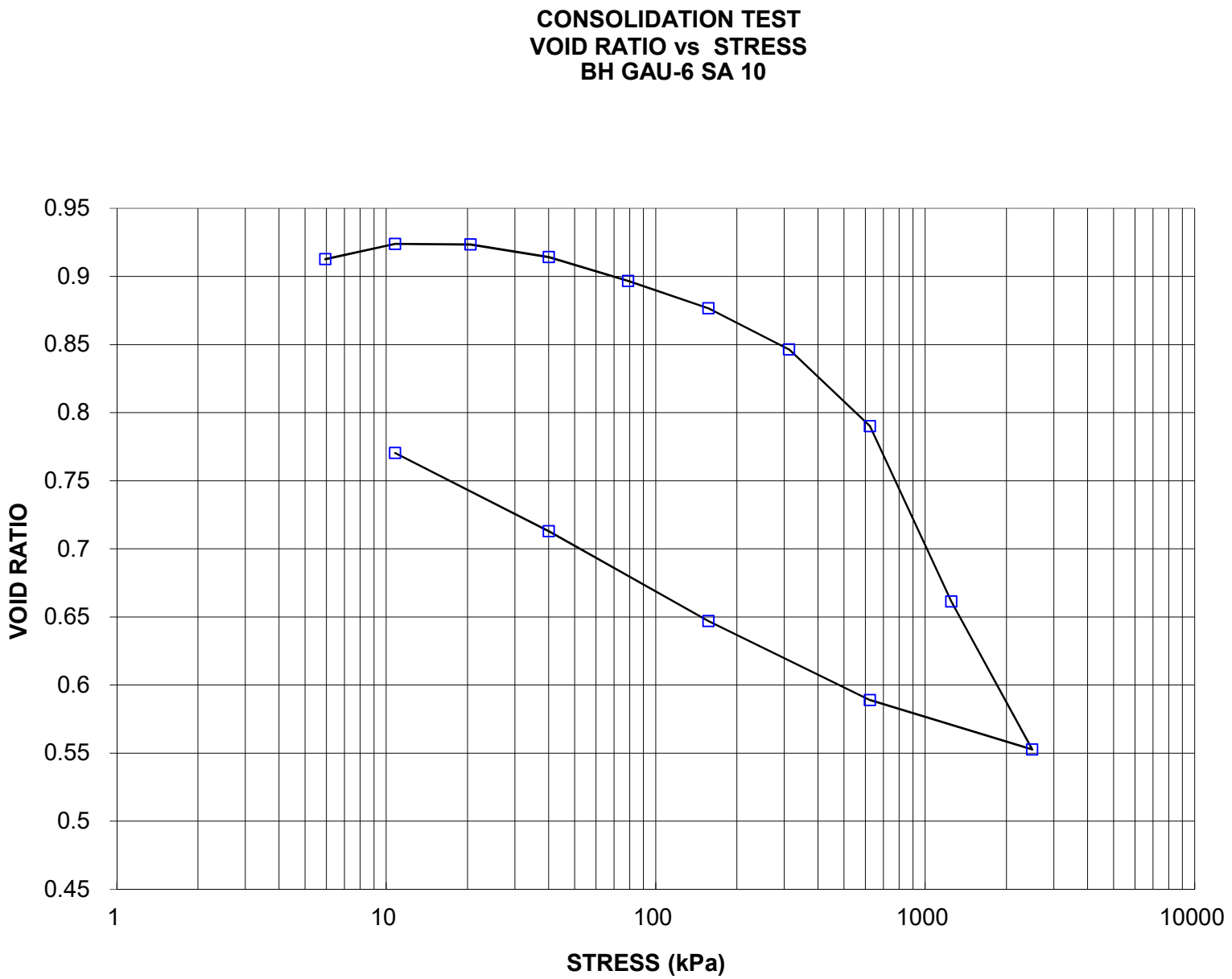


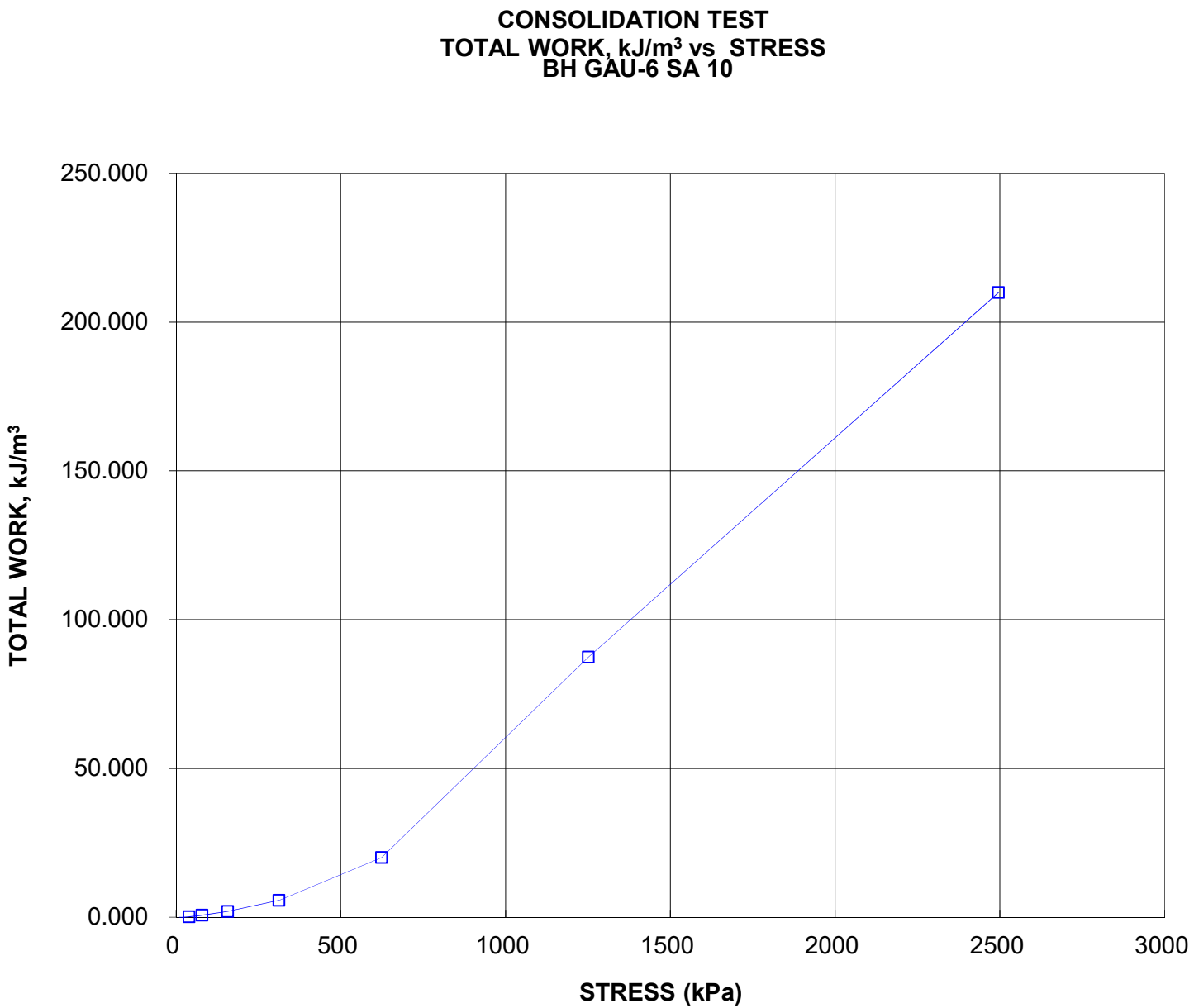
VOLUME COMPRESSIBILITY, m<sup>2</sup>/kN



HYDRAULIC CONDUCTIVITY, cm/s



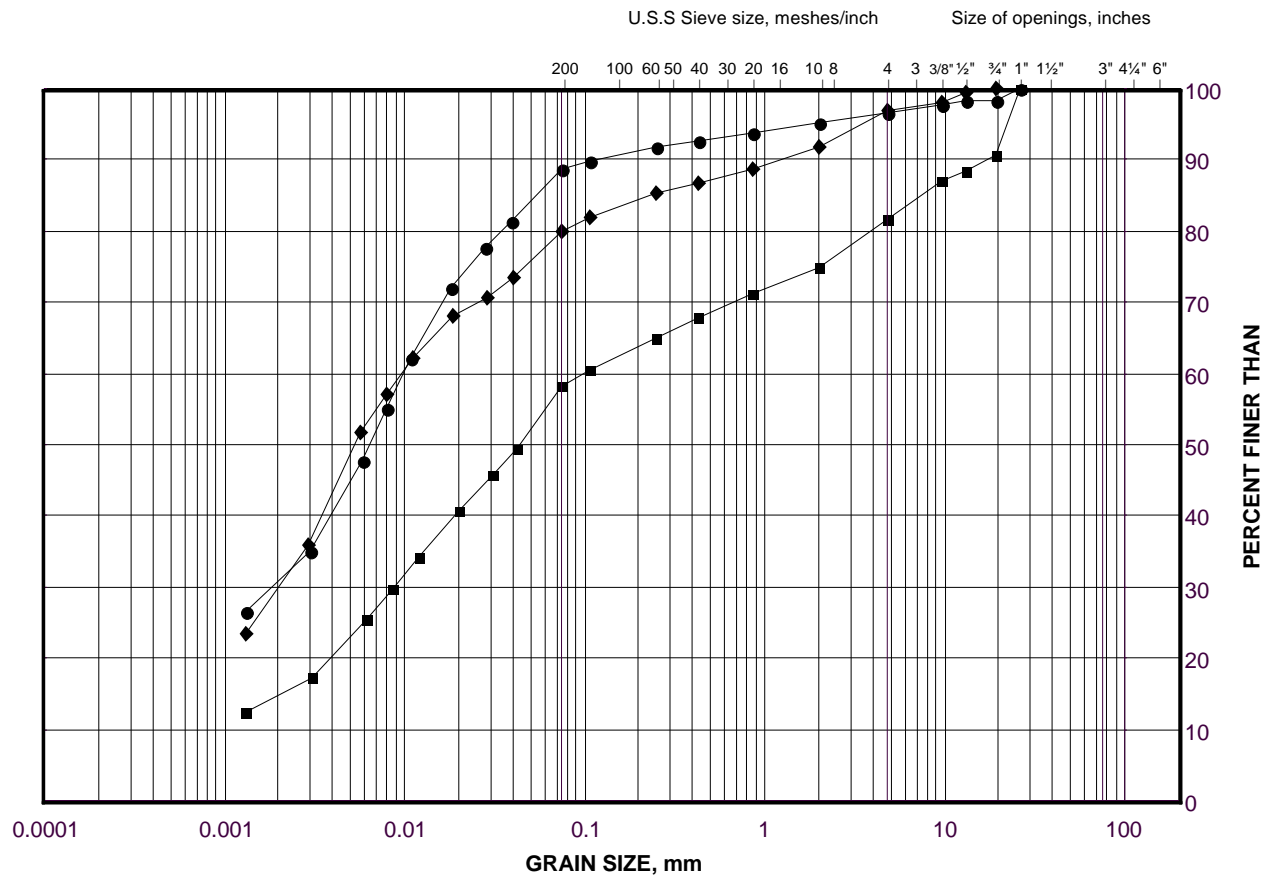




# GRAIN SIZE DISTRIBUTION

Clayey Silt

FIGURE C-8



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

## LEGEND

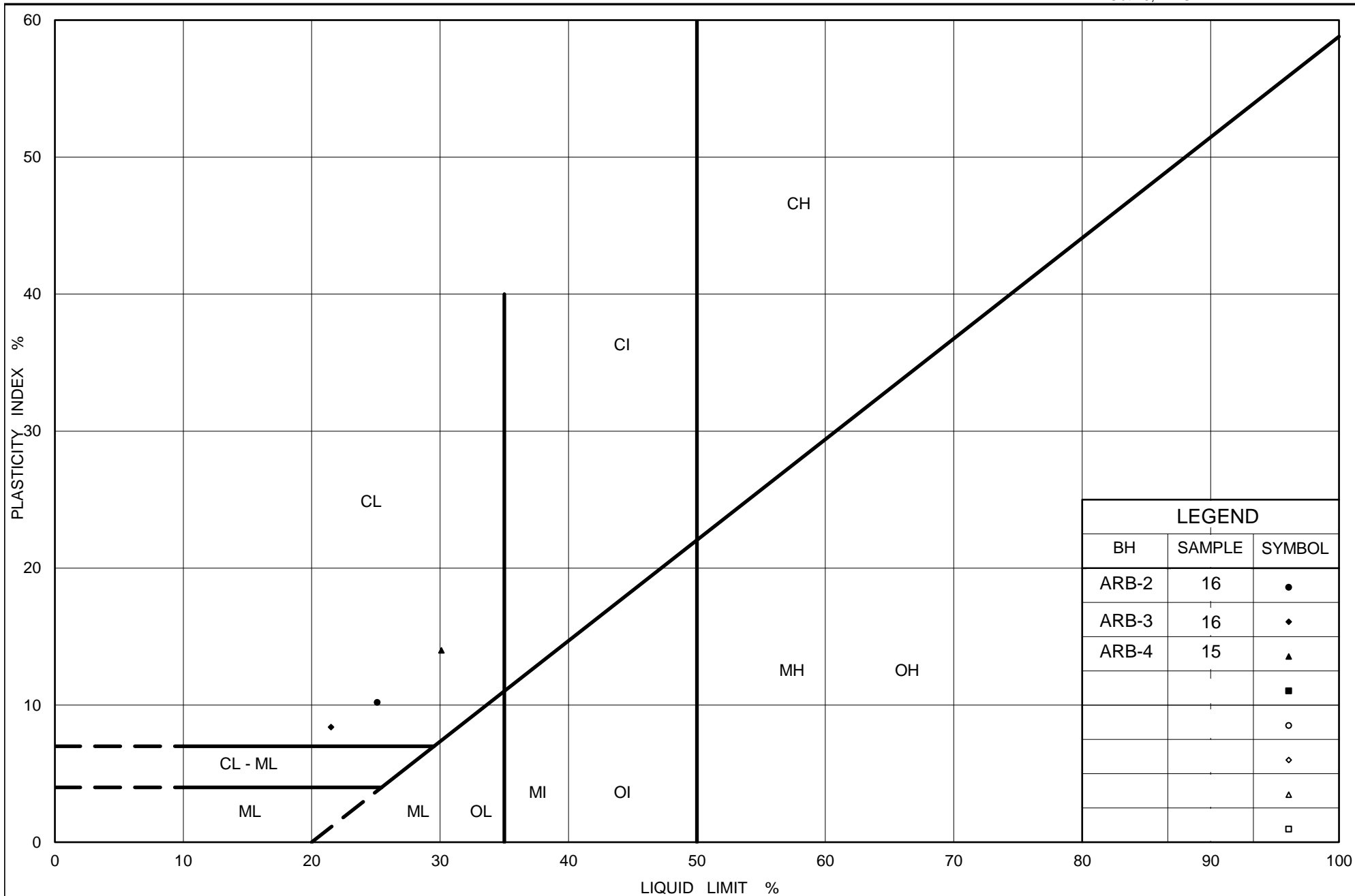
SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	ARB-4	15	98.5
■	ARB-3	16	96.2
◆	ARB-2	16	99.8

Project Number: 1671430

Checked By: MA/LCC

**Golder Associates**

Date: 22-Apr-19



Ministry of Transportation

Ontario

## PLASTICITY CHART

### Clayey Silt

Figure No. C-9

Project No. 1671430 (WO 002)

Checked By: MA/LCC

## Silt and Sand to Silt

U.S.S Sieve size, meshes/inch

Size of openings, inches

PERCENT FINER THAN

GRAIN SIZE, mm

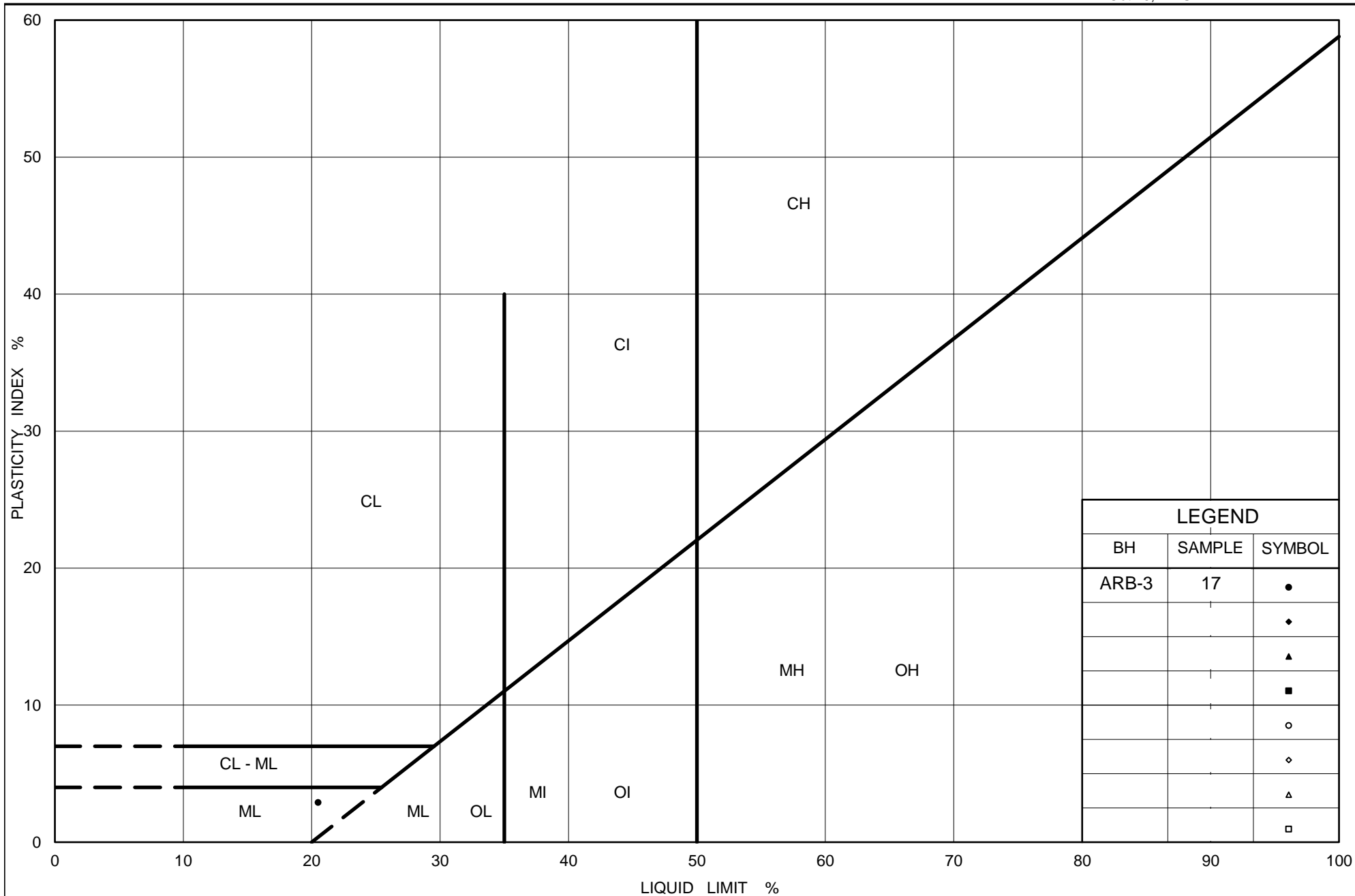
Grain Size (mm)	Percent Finer (%) - Squares	Percent Finer (%) - Circles	Percent Finer (%) - Diamonds
0.075	100	100	100
0.15	100	100	100
0.3	100	100	100
0.6	100	100	100
1.2	100	100	100
2.5	100	100	100
5.0	100	100	100
10.0	100	100	100
20.0	100	100	100
40.0	100	100	100
80.0	100	100	100
150.0	100	100	100
300.0	100	100	100
600.0	100	100	100
1200.0	100	100	100
2500.0	100	100	100
5000.0	100	100	100
10000.0	100	100	100
20000.0	100	100	100
40000.0	100	100	100
80000.0	100	100	100
160000.0	100	100	100
320000.0	100	100	100
640000.0	100	100	100
1280000.0	100	100	100
2560000.0	100	100	100
5120000.0	100	100	100
10240000.0	100	100	100
20480000.0	100	100	100
40960000.0	100	100	100
81920000.0	100	100	100
163840000.0	100	100	100
327680000.0	100	100	100
655360000.0	100	100	100
1310720000.0	100	100	100
2621440000.0	100	100	100
5242880000.0	100	100	100
10485760000.0	100	100	100
20971520000.0	100	100	100
41943040000.0	100	100	100
83886080000.0	100	100	100
167772160000.0	100	100	100
335544320000.0	100	100	100
671088640000.0	100	100	100
1342177280000.0	100	100	100
2684354560000.0	100	100	100
5368709120000.0	100	100	100
10737418240000.0	100	100	100
21474836480000.0	100	100	100
42949672960000.0	100	100	100
85899345920000.0	100	100	100
171798691840000.0	100	100	100
343597383680000.0	100	100	100
687194767360000.0	100	100	100
1374389534720000.0	100	100	100
2748779069440000.0	100	100	100
5497558138880000.0	100	100	100
10995116277760000.0	100	100	100
21990232555520000.0	100	100	100
43980465111040000.0	100	100	100
87960930222080000.0	100	100	100
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11258999068426240000.0	100	100	100
22517998136852480000.0	100	100	100
45035996273704960000.0	100	100	100
90071992547409920000.0	100	100	100
180143985094819840000.0	100	100	100
36028797			

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

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	ARB-4	17	92.5
■	ARB-3	17	93.0
◆	ARB-2	20	92.4

Date: 22-Apr-19





Ministry of Transportation

Ontario

# PLASTICITY CHART

## Silt

Figure No. C-11

Project No. 1671430 (WO 002)

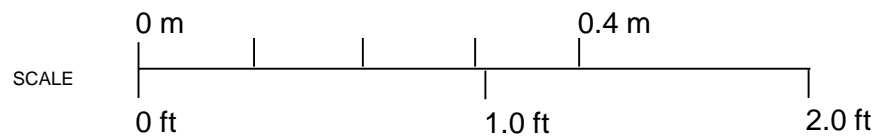
Checked By: MA/LCC



**Borehole ARB-2:** Bedrock cored between depths of about 32.6 m to 36.3 m



**Borehole ARB-3:** Bedrock cored between depths of about 30.0 m to 33.2 m



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  
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#900-390 Bay St.  
Toronto ON  
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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-WO2). 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												

**APPENDIX D**

# Analytical Laboratory Test Results

Your Project #: 1671430-W02  
Your C.O.C. #: 641804-06-01

**Attention: Nikol Kochmanova**

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

**Report Date: 2018/12/12**  
Report #: R5522779  
Version: 1 - Final

**CERTIFICATE OF ANALYSIS**

**MAXXAM JOB #: B8W6764**

**Received: 2018/12/06, 12:29**

Sample Matrix: Soil  
# Samples Received: 3

Analyses	Date		Date Analyzed	Laboratory Method	Reference
	Quantity	Extracted			
Chloride (20:1 extract)	3	N/A	2018/12/12	CAM SOP-00463	EPA 325.2 m
Conductivity	3	N/A	2018/12/12	CAM SOP-00414	OMOE E3530 v1 m
pH CaCl <sub>2</sub> EXTRACT	3	2018/12/11	2018/12/11	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	3	2018/12/06	2018/12/12	CAM SOP-00414	SM 23 2510 m
Sulphate (20:1 Extract)	3	N/A	2018/12/12	CAM SOP-00464	EPA 375.4 m

**Remarks:**

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

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

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

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

Results relate to samples tested. When sampling is not conducted by Maxxam, results relate to the supplied samples tested.

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

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

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

**Encryption Key**

Please direct all questions regarding this Certificate of Analysis to your Project Manager.  
Ema Gitej, Senior Project Manager

Your Project #: 1671430-W02  
Your C.O.C. #: 641804-06-01

**Attention: Nikol Kochmanova**

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

**Report Date: 2018/12/12**  
Report #: R5522779  
Version: 1 - Final

**CERTIFICATE OF ANALYSIS**

**MAXXAM JOB #: B8W6764**

**Received: 2018/12/06, 12:29**

Email: EGitej@maxxam.ca

Phone# (905)817-5829

=====

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

### SOIL CORROSIVITY PACKAGE (SOIL)

<b>Maxxam ID</b>		IMF982	IMF983			IMF983			IMF984		
<b>Sampling Date</b>		2018/08/20	2018/08/01			2018/08/01			2018/08/15		
<b>COC Number</b>		641804-06-01	641804-06-01			641804-06-01			641804-06-01		
	<b>UNITS</b>	<b>GAU5-SS6</b>	<b>GAU6-SS9</b>	<b>RDL</b>	<b>QC Batch</b>	<b>GAU6-SS9 Lab-Dup</b>	<b>RDL</b>	<b>QC Batch</b>	<b>GAU2-SS11</b>	<b>RDL</b>	<b>QC Batch</b>

#### Calculated Parameters

Resistivity	ohm-cm	460	470		5875238				390		5875238
-------------	--------	-----	-----	--	---------	--	--	--	-----	--	---------

#### Inorganics

Soluble (20:1) Chloride (Cl-)	ug/g	120	49	20	5883832				51	20	5883832
Conductivity	umho/cm	2160	2150	2	5883994				2570	2	5883994
Available (CaCl2) pH	pH	7.97	7.99		5881793				7.82		5881793
Soluble (20:1) Sulphate (SO4)	ug/g	2600	2900	100	5883876	3000	100	5883876	3400	100	5883876

RDL = Reportable Detection Limit

QC Batch = Quality Control Batch

Lab-Dup = Laboratory Initiated Duplicate

<b>Maxxam ID</b>		IMF984		
<b>Sampling Date</b>		2018/08/15		
<b>COC Number</b>		641804-06-01		
	<b>UNITS</b>	<b>GAU2-SS11 Lab-Dup</b>	<b>RDL</b>	<b>QC Batch</b>
<b>Inorganics</b>				
Soluble (20:1) Chloride (Cl-)	ug/g	52	20	5883832
RDL = Reportable Detection Limit				
QC Batch = Quality Control Batch				
Lab-Dup = Laboratory Initiated Duplicate				



## TEST SUMMARY

**Maxxam ID:** IMF982  
**Sample ID:** GAU5-SS6  
**Matrix:** Soil

**Collected:** 2018/08/20  
**Shipped:**  
**Received:** 2018/12/06

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5883832	N/A	2018/12/12	Deonarine Ramnarine
Conductivity	AT	5883994	N/A	2018/12/12	Kazzandra Adeva
pH CaCl2 EXTRACT	AT	5881793	2018/12/11	2018/12/11	Gnana Thomas
Resistivity of Soil		5875238	2018/12/12	2018/12/12	Anastassia Hamanov
Sulphate (20:1 Extract)	KONE/EC	5883876	N/A	2018/12/12	Deonarine Ramnarine

**Maxxam ID:** IMF983  
**Sample ID:** GAU6-SS9  
**Matrix:** Soil

**Collected:** 2018/08/01  
**Shipped:**  
**Received:** 2018/12/06

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5883832	N/A	2018/12/12	Deonarine Ramnarine
Conductivity	AT	5883994	N/A	2018/12/12	Kazzandra Adeva
pH CaCl2 EXTRACT	AT	5881793	2018/12/11	2018/12/11	Gnana Thomas
Resistivity of Soil		5875238	2018/12/12	2018/12/12	Anastassia Hamanov
Sulphate (20:1 Extract)	KONE/EC	5883876	N/A	2018/12/12	Deonarine Ramnarine

**Maxxam ID:** IMF983 Dup  
**Sample ID:** GAU6-SS9  
**Matrix:** Soil

**Collected:** 2018/08/01  
**Shipped:**  
**Received:** 2018/12/06

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Sulphate (20:1 Extract)	KONE/EC	5883876	N/A	2018/12/12	Deonarine Ramnarine

**Maxxam ID:** IMF984  
**Sample ID:** GAU2-SS11  
**Matrix:** Soil

**Collected:** 2018/08/15  
**Shipped:**  
**Received:** 2018/12/06

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5883832	N/A	2018/12/12	Deonarine Ramnarine
Conductivity	AT	5883994	N/A	2018/12/12	Kazzandra Adeva
pH CaCl2 EXTRACT	AT	5881793	2018/12/11	2018/12/11	Gnana Thomas
Resistivity of Soil		5875238	2018/12/12	2018/12/12	Anastassia Hamanov
Sulphate (20:1 Extract)	KONE/EC	5883876	N/A	2018/12/12	Deonarine Ramnarine

**Maxxam ID:** IMF984 Dup  
**Sample ID:** GAU2-SS11  
**Matrix:** Soil

**Collected:** 2018/08/15  
**Shipped:**  
**Received:** 2018/12/06

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5883832	N/A	2018/12/12	Deonarine Ramnarine

### GENERAL COMMENTS

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

Package 1	5.3°C
-----------	-------

Conductivity Analysis: Analysis was performed past sample holding time. This may increase the variability associated with these results.

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

## QUALITY ASSURANCE REPORT

Golder Associates Ltd  
Client Project #: 1671430-W02  
Sampler Initials: KNE

QC Batch	Parameter	Date	Matrix Spike		SPIKED BLANK		Method Blank		RPD	
			% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
5881793	Available (CaCl <sub>2</sub> ) pH	2018/12/11			100	97 - 103			0.10	N/A
5883832	Soluble (20:1) Chloride (Cl <sup>-</sup> )	2018/12/12	NC	70 - 130	102	70 - 130	<20	ug/g	0.57	35
5883876	Soluble (20:1) Sulphate (SO <sub>4</sub> )	2018/12/12	NC	70 - 130	104	70 - 130	<20	ug/g	4.4	35
5883994	Conductivity	2018/12/12			103	90 - 110	<2	umho/cm	0.65	10

N/A = Not Applicable

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

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

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

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

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

### VALIDATION SIGNATURE PAGE

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



\_\_\_\_\_  
Anastassia Hamanov, Scientific Specialist

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Maxxam Analytics International Corporation o/a Maxxam Analytics 5740 Campbell Road, Mississauga, Ontario Canada L5N 2L8 Tel: (905) 817-5700 Toll-free 800-563-6266 Fax: (905) 817-5777 www.maxxam.ca										<b>CHAIN OF CUSTODY RECORD</b>																																																																																					
INVOICE TO:										REPORT TO:										PROJECT INFORMATION:										Laboratory Use Only:																																																																	
Company Name: <b>#1326 Golder Associates Ltd</b> Attention: <b>Accounts Payable</b> Address: <b>6925 Century Ave Suite 100</b> <b>Mississauga ON L5N 7K2</b> Tel: <b>(905) 567-4444 x</b> Fax: <b>(905) 567-6561 x</b> Email: <b>AP_CustomerService@golder.com</b>										Company Name: <b>Golder Associates Ltd.</b> Attention: <b>Nikol Kochmanova</b> Address: <b>"</b> Tel: <b>"</b> Fax: <b>"</b> Email: <b>Nikol-Kochmanova@golder.com</b>										Quotation #: <b>870916</b> P.O. #: <b>"</b> Project: <b>1671430-W02</b> Project Name: <b>"</b> Site #: <b>"</b> Sampled By: <b>KN/EN</b>										Maxxam Job #: <b>"</b> Bottle Order #: <b>641804</b> COC #: <b>"</b> Project Manager: <b>Ema Gitej</b> C#641804-06-01																																																																	
<b>MOE REGULATED DRINKING WATER OR WATER INTENDED FOR HUMAN CONSUMPTION MUST BE SUBMITTED ON THE MAXXAM DRINKING WATER CHAIN OF CUSTODY</b>										<b>ANALYSIS REQUESTED (PLEASE BE SPECIFIC)</b>										<b>Turnaround Time (TAT) Required:</b> Please provide advance notice for rush projects																																																																											
<b>Regulation 153 (2011)</b> <input type="checkbox"/> Table 1 <input type="checkbox"/> Res/Park <input type="checkbox"/> Medium/Fine <input type="checkbox"/> Table 2 <input type="checkbox"/> Ind/Comm <input type="checkbox"/> Coarse <input type="checkbox"/> Table 3 <input type="checkbox"/> Agri/Other <input type="checkbox"/> For RSC <input type="checkbox"/> Table <b>"</b>										<b>Other Regulations</b> <input type="checkbox"/> CCME <input type="checkbox"/> Sanitary Sewer Bylaw <input type="checkbox"/> Reg 558 <input type="checkbox"/> Storm Sewer Bylaw <input type="checkbox"/> MISA Municipality <b>"</b> <input type="checkbox"/> PWQO <input type="checkbox"/> Other <b>"</b>										<b>Special Instructions</b> <b>Field Filtered (please circle):</b> <b>Metals / Hg / Cr VI</b> <b>Standard Carcinogen Package</b>										<b>Regular (Standard) TAT:</b> (will be applied if Rush TAT is not specified): Standard TAT = 5-7 Working days for most tests. Please note: Standard TAT for certain tests such as BOD and Dioxins/Furans are > 5 days - contact your Project Manager for details.																																																																	
<b>Include Criteria on Certificate of Analysis (Y/N)?</b>										<b>Job Specific Rush TAT (if applies to entire submission)</b> Date Required: <b>"</b> Time Required: <b>"</b> Rush Confirmation Number: <b>"</b> (call lab for #)										<b># of Bottles</b> <b>Comments</b>																																																																											
<table border="1" style="width:100%; border-collapse: collapse;"> <thead> <tr> <th>Sample Barcode Label</th> <th>Sample (Location) Identification</th> <th>Date Sampled</th> <th>Time Sampled</th> <th>Matrix</th> <th>Field Filtered (please circle): Metals / Hg / Cr VI</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>GAU5-SS6</td> <td>2018/8/20</td> <td>PM</td> <td>SOIL</td> <td>X</td> </tr> <tr> <td>2</td> <td>GAU6-SS9</td> <td>2018/08/01</td> <td>PM</td> <td>SOIL</td> <td>X</td> </tr> <tr> <td>3</td> <td>GAU2-SS11</td> <td>2018/08/15</td> <td>AM</td> <td>SOIL</td> <td>X</td> </tr> <tr><td>4</td><td></td><td></td><td></td><td></td><td></td></tr> <tr><td>5</td><td></td><td></td><td></td><td></td><td></td></tr> <tr><td>6</td><td></td><td></td><td></td><td></td><td></td></tr> <tr><td>7</td><td></td><td></td><td></td><td></td><td></td></tr> <tr><td>8</td><td></td><td></td><td></td><td></td><td></td></tr> <tr><td>9</td><td></td><td></td><td></td><td></td><td></td></tr> <tr><td>10</td><td></td><td></td><td></td><td></td><td></td></tr> </tbody> </table>										Sample Barcode Label	Sample (Location) Identification	Date Sampled	Time Sampled	Matrix	Field Filtered (please circle): Metals / Hg / Cr VI	1	GAU5-SS6	2018/8/20	PM	SOIL	X	2	GAU6-SS9	2018/08/01	PM	SOIL	X	3	GAU2-SS11	2018/08/15	AM	SOIL	X	4						5						6						7						8						9						10																									
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																																								Time Sensitive										Temperature (°C) on Reel <b>8/3/5</b>										Custody Seal <input checked="" type="checkbox"/> Present <input type="checkbox"/> Intact										Yes <input type="checkbox"/> No <input checked="" type="checkbox"/>																									
* UNLESS OTHERWISE AGREED TO IN WRITING, WORK SUBMITTED ON THIS CHAIN OF CUSTODY IS SUBJECT TO MAXXAM'S STANDARD TERMS AND CONDITIONS. SIGNING OF THIS CHAIN OF CUSTODY DOCUMENT IS ACKNOWLEDGMENT AND ACCEPTANCE OF OUR TERMS WHICH ARE AVAILABLE FOR VIEWING AT WWW.MAXXAM.CA/TERMS.										* IT IS THE RESPONSIBILITY OF THE RELINQUISHER TO ENSURE THE ACCURACY OF THE CHAIN OF CUSTODY RECORD. AN INCOMPLETE CHAIN OF CUSTODY MAY RESULT IN ANALYTICAL TAT DELAYS.										** SAMPLE CONTAINER, PRESERVATION, HOLD TIME AND PACKAGE INFORMATION CAN BE VIEWED AT HTTP://MAXXAM.CA/WP-CONTENT/UPLOADS/ONTARIO-COC.PDF.										White: Maxxa Yellow: Client																																																																	

Your Project #: 1671430 W02  
Site Location: QEW GLENDALE  
Your C.O.C. #: 641804-11-01

**Attention: Nikol Kochmanova**

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

**Report Date: 2019/01/23**  
Report #: R5567997  
Version: 1 - Final

**CERTIFICATE OF ANALYSIS**

**MAXXAM JOB #: B915672**

**Received: 2019/01/18, 10:35**

Sample Matrix: Soil  
# Samples Received: 2

Analyses	Date		Date Analyzed	Laboratory Method	Reference
	Quantity	Extracted			
Chloride (20:1 extract)	2	N/A	2019/01/23	CAM SOP-00463	EPA 325.2 m
Conductivity	2	N/A	2019/01/22	CAM SOP-00414	OMOE E3530 v1 m
pH CaCl2 EXTRACT	2	2019/01/22	2019/01/22	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	2	2019/01/19	2019/01/22	CAM SOP-00414	SM 23 2510 m
Sulphate (20:1 Extract)	2	N/A	2019/01/23	CAM SOP-00464	EPA 375.4 m

**Remarks:**

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

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

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

Results relate to samples tested. When sampling is not conducted by Maxxam, results relate to the supplied samples tested.

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

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

Your Project #: 1671430 W02  
Site Location: QEW GLENDALE  
Your C.O.C. #: 641804-11-01

**Attention: Nikol Kochmanova**

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

**Report Date: 2019/01/23**  
Report #: R5567997  
Version: 1 - Final

**CERTIFICATE OF ANALYSIS**

**MAXXAM JOB #: B915672**  
**Received: 2019/01/18, 10:35**

Encryption Key

Please direct all questions regarding this Certificate of Analysis to your Project Manager.  
Ema Gitej, Senior Project Manager  
Email: EGitej@maxxam.ca  
Phone# (905)817-5829

=====

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### RESULTS OF ANALYSES OF SOIL

<b>Maxxam ID</b>		IUD596		IUD597		
<b>Sampling Date</b>		2018/10/26		2018/10/22		
<b>COC Number</b>		641804-11-01		641804-11-01		
	<b>UNITS</b>	<b>ARB2 SA5</b>	<b>RDL</b>	<b>ARB3 SA6</b>	<b>RDL</b>	<b>QC Batch</b>
<b>Calculated Parameters</b>						
Resistivity	ohm-cm	1600		560		5936840
<b>Inorganics</b>						
Soluble (20:1) Chloride (Cl-)	ug/g	77	20	22	20	5940294
Conductivity	umho/cm	643	2	1790	2	5940019
Available (CaCl2) pH	pH	7.78		8.03		5939853
Soluble (20:1) Sulphate (SO4)	ug/g	400	20	2800	100	5940279
RDL = Reportable Detection Limit						
QC Batch = Quality Control Batch						



Maxxam Job #: B915672  
Report Date: 2019/01/23

Golder Associates Ltd  
Client Project #: 1671430 W02  
Site Location: QEW GLENDALE  
Sampler Initials: JMP

## TEST SUMMARY

**Maxxam ID:** IUD596  
**Sample ID:** ARB2 SA5  
**Matrix:** Soil

**Collected:** 2018/10/26  
**Shipped:**  
**Received:** 2019/01/18

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5940294	N/A	2019/01/23	Deonarine Ramnarine
Conductivity	AT	5940019	N/A	2019/01/22	Kazzandra Adeva
pH CaCl2 EXTRACT	AT	5939853	2019/01/22	2019/01/22	Gnana Thomas
Resistivity of Soil		5936840	2019/01/22	2019/01/22	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5940279	N/A	2019/01/23	Deonarine Ramnarine

**Maxxam ID:** IUD597  
**Sample ID:** ARB3 SA6  
**Matrix:** Soil

**Collected:** 2018/10/22  
**Shipped:**  
**Received:** 2019/01/18

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5940294	N/A	2019/01/23	Deonarine Ramnarine
Conductivity	AT	5940019	N/A	2019/01/22	Kazzandra Adeva
pH CaCl2 EXTRACT	AT	5939853	2019/01/22	2019/01/22	Gnana Thomas
Resistivity of Soil		5936840	2019/01/22	2019/01/22	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5940279	N/A	2019/01/23	Deonarine Ramnarine

### GENERAL COMMENTS

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

Package 1	9.0°C
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pH, Chloride, Sulphate, Conductivity/Resistivity: Sample submitted and analyzed past the recommended sample hold time. This may increase the variability associated with these results.

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

## QUALITY ASSURANCE REPORT

Golder Associates Ltd  
Client Project #: 1671430 W02  
Site Location: QEW GLENDALE  
Sampler Initials: JMP

QC Batch	Parameter	Date	Matrix Spike		SPIKED BLANK		Method Blank		RPD	
			% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
5939853	Available (CaCl <sub>2</sub> ) pH	2019/01/22			100	97 - 103			1.0	N/A
5940019	Conductivity	2019/01/22			103	90 - 110	<2	umho/cm	0.68	10
5940279	Soluble (20:1) Sulphate (SO <sub>4</sub> )	2019/01/23	117	70 - 130	108	70 - 130	<20	ug/g	NC	35
5940294	Soluble (20:1) Chloride (Cl <sup>-</sup> )	2019/01/23	112	70 - 130	103	70 - 130	<20	ug/g	NC	35

N/A = Not Applicable

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

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

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

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

NC (Duplicate RPD): The duplicate RPD was not calculated. The concentration in the sample and/or duplicate was too low to permit a reliable RPD calculation (absolute difference <= 2x RDL).

### VALIDATION SIGNATURE PAGE

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



Anastassia Hamanov, Scientific Specialist

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

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