

FOUNDATION INVESTIGATION AND DESIGN REPORT

*High Fill Embankment from Station 15+260 to Station 15+360
Highway 400 Widening from North of King Road to
South of Lloydtown-Aurora Road, King City, Ontario
Assignment No. 2017-E-0016-015, G.W.P. 2835-02-00*

Submitted to:

Morrison Hershfield Limited

125 Commerce Valley Drive West
Markham, ON
L3T 7W4

Submitted by:

Golder Associates Ltd.

6925 Century Avenue, Suite #100 Mississauga, Ontario, L5N 7K2 Canada
+1 905 567 4444

1786658-WO15-HF

July 11, 2022

GEOCRES No: 30M13-299

Latitude: 43.949836°

Longitude: -79.572619°



Distribution List

1 PDF Copy - MTO Central Region

1 PDF Copy - MTO Foundations Section

1 PDF Copy - Morrison Hershfield Limited

1 PDF Copy - Golder Associates Ltd.

Table of Contents

PART A – FOUNDATION INVESTIGATION REPORT

1.0 INTRODUCTION	1
2.0 SITE DESCRIPTION	1
3.0 INVESTIGATION PROCEDURES	2
3.1 2010-2015 Investigation (GEOCRETS No. 30M13-217)	2
3.2 2016 Investigation	3
3.2.1 Laboratory Testing	5
4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS	6
4.1 Regional Geology	6
4.2 Subsurface Conditions	6
4.2.1 Asphalt	7
4.2.2 Topsoil	7
4.2.3 Fill	7
4.2.4 Upper Non-Cohesive Deposit (Sandy Silt to Silty Sand to Sand)	8
4.2.5 Upper Cohesive Deposit (Clayey Silt-Silt to Clayey Silt)	9
4.2.6 Organic Deposit	10
4.2.7 Lower Cohesive Deposit (Clayey Silt to Silty Clay to Clay)	15
4.2.8 Lower Non-Cohesive Deposit	17
4.2.9 Clayey Silt Till	19
4.2.10 Groundwater Conditions	19
4.3 Analytical Chemical Testing (Soil and Groundwater)	20
4.4 Ground Improvement Laboratory Testing	21
5.0 CLOSURE	21

PART B - FOUNDATION DESIGN REPORT

6.0 FOUNDATION ENGINEERING RECOMMENDATIONS	18
6.1 General	18

6.2	Embankment Widening Requirements and Options	18
6.3	Design Criteria.....	19
6.3.1	Global Stability Criteria	19
6.3.2	Embankment Settlement Criteria	20
6.4	Global Stability and Settlement	20
6.4.1	Parameter Selection	20
6.4.2	Global Stability	21
6.4.3	Settlement	22
6.4.3.1	Immediate Elastic Settlement	23
6.4.3.2	Primary Consolidation Settlement	23
6.4.3.3	Secondary Creep Settlement.....	25
6.5	Construction Considerations	25
6.5.1	Subgrade Preparation and Embankment Construction	25
6.5.2	Groundwater and Surface Water Control	26
6.6	Monitoring.....	26
7.0	CLOSURE	27

TABLES

Table 1 – Comparison of Stability and Settlement Mitigation Measures

Table 2 – Soil Parameters Used in Global Stability and Settlement Analyses

REFERENCES

FIGURES

Figure 1 Embankment Widening, Station 15+260 To 15+360

DRAWINGS

Drawing 1 Borehole Location Plan
Drawing 2 Soil Strata
Drawing 3 Soil Strata
Drawing 4 Monitoring Instrumentation Plan
Drawing 5 Typical Monitoring Instrumentation and Installation Details

APPENDICES

APPENDIX A 2010-2015 INVESTIGATION – BOREHOLE RECORDS AND LABORATORY RESULTS

Record of Boreholes	C29-3, C29-4, F1-5A, 5B, 5C, 5D, 5E, 5F and 5G
Figure A-1A	Grain Size Distribution – Silty Sand to Clayey Silt Fill
Figure A-1B	Grain Size Distribution - Silty Sand to Clayey Silt Fill
Figure A-2	Plasticity Chart – Clayey Silt Fill
Figure A-3	Grain Size Distribution – Upper Clayey Silt and Sand
Figure A-4	Plasticity Chart – Upper Clayey Silt to Sandy Clayey Silt
Figure A-5A	Grain Size Distribution – Silt to Sand
Figure A-5B	Grain Size Distribution – Silt to Sand
Figure A-6A	Grain Size Distribution – Clayey Silt Till
Figure A-6B	Grain Size Distribution – Clayey Silt Till
Figure A-6C	Grain Size Distribution – Clayey Silt Till
Figure A-6D	Grain Size Distribution – Clayey Silt Till
Figure A-7A	Plasticity Chart – Clayey Silt Till
Figure A-7B	Plasticity Chart – Clayey Silt Till
Figure A-7C	Plasticity Chart – Clayey Silt to Silt Till
Figure A-7D	Plasticity Chart – Clayey Silt Till
Figure A-8	Plasticity Chart – Lower Clayey Silt to Silty Clay
Figure A-9	Plasticity Chart – Lower Clayey Silt to Silty Clay
Figure A-10A	Grain Size Distribution – Organic Silt
Figure A-10B	Grain Size Distribution – Organic Sandy Silt to Silty Sandy Peat
Figure A-10C	Grain Size Distribution – Organic Clay
Figure A-10D	Grain Size Distribution – Silty Clay, Trace Organics
Figure A-11A	Plasticity Chart – Organic Silt to Organic Clayey Silt to Organic Clay
Figure A-11B	Plasticity Chart – Organic Silt to Organic Clayey Silt
Figure A-11C	Plasticity Chart – Organic Sandy Silt
Figure A-11D	Plasticity Chart – Silty Clay, Trace Organics
Figure A-12	Consolidation Test Summary – Void Ratio Vs Log Pressure
Figure A-13	Consolidation Test Summary – Total Work Vs Stress
Figure A-14	Grain Size Distribution – Silty Sand to Sand and Gravel

APPENDIX B 2016 INVESTIGATION – BOREHOLE AND TEST PIT RECORDS

List of Symbols	
List of Abbreviations	
Record of Boreholes	16-B1, 16-B2, 16-M1, 16-M2, and 16-T1
Record of Test Pits	TP1 and TP2
Cone Penetration	CPT16-1
Cone Penetration	CPT16-2

APPENDIX C 2016 INVESTIGATION – GEOTECHNICAL LABORATORY TEST RESULTS

Figure C-1	Grain Size Distribution – Sandy Silt to Silt and Sand to Silty Sand to Sand and Gravel (Fill)
Figure C-2	Plasticity Chart – Silt (Fill)
Figure C-3	Plasticity Chart – Sandy Clayey Silt
Figure C-4	Grain Size Distribution – Sandy Silt to Silty Sand to Sand – Upper
Figure C-5	Plasticity Chart – Sandy Silt
Figure C-6	Grain Size Distribution – Organic Silt and Sand
Figure C-7	Plasticity Chart – Clayey Organic Silt to Organic Silt
Figure C-8	Plasticity Chart – Clayey Silty to Silty Clay to Clay
Figure C-9	Consolidation Test Summary – Borehole B16-B2 SA 5
Figure C-10	Consolidation Test Summary - Borehole B16-B2 SA 10
Figure C-11	Consolidation Undrained Triaxial Test Summary - Borehole B16-B2 SA 5
Figure C-12	Grain Size Distribution – Sandy Silt to Silty Sand - Lower
Figure C-13	Plasticity Chart – Sandy Silt

APPENDIX D 2016 INVESTIGATION – ANALYTICAL LABORATORY TESTING RESULTS

Maxxam Certificate of Analysis R4133563 (Soil)
AGAT Groundwater Certificate of Analysis 16T149988 (Groundwater)

APPENDIX E LABORATORY TESTING REPORT BY RYERSON UNIVERSITY

Laboratory Investigation and Deep Soil Mixing in Treatment of Organic Clays in Ontario (November 16, 2016)

APPENDIX F GROUND IMPROVEMENT METHODS

APPENDIX G PARAMETER SELECTION DATA

Figure G-1	Shear Strength
Figure G-2	Preconsolidation Pressure
Figure G-3	Void Ratio
Figure G-4	Compression Index
Figure G-5	Recompression Index
Figure G-6	Global Stability Analysis – Station 15+310 – Stage B Temporary Condition (B-Bar Analysis)
Figure G-7	Global Stability Analysis – Station 15+310 – Stage C Temporary Condition (B-Bar Analysis)

APPENDIX H GLOBAL STABILITY ANALYSIS RESULTS

Figure H-1	Global Stability Analysis – Station 15+290 Stage A - Temporary Condition (Undrained Analysis)
Figure H-2	Global Stability Analysis – Station 15+290 Stage B - Temporary Condition (B-Bar Analysis)
Figure H-3	Global Stability Analysis – Station 15+290 Stage C - Temporary Condition (B-Bar Analysis)
Figure H-4	Global Stability Analysis – Station 15+290 Stage C - Permanent Condition (Drained Analysis)
Figure H-5	Global Stability Analysis – Station 15+310 Stage A - Temporary Condition (Undrained Analysis)
Figure H-6	Global Stability Analysis – Station 15+310 Stage B - Temporary Condition (B-Bar Analysis)
Figure H-7	Global Stability Analysis – Station 15+310 Stage C - Temporary Condition (B-Bar Analysis)
Figure H-8	Global Stability Analysis – Station 15+310 Stage C - Permanent Condition (Drained Analysis)

APPENDIX I SETTLEMENT ANALYSIS RESULTS

Figure I-1	Settlement Analysis Results – Station 15+290 – Total Settlement
Figure I-2	Settlement Analysis Results – Station 15+310 – Total Settlement
Figure I-3	Settlement Analysis Results – Station 15+290 – Immediate Settlement
Figure I-4	Settlement Analysis Results – Station 15+310 – Immediate Settlement
Figure I-5	Settlement Analysis Results – Station 15+290 – Consolidation Settlement
Figure I-6	Settlement Analysis Results – Station 15+310 – Consolidation Settlement

APPENDIX J SPECIAL PROVISIONS AND OPERATIONAL CONSTRAINTS

NSSP	3/8-inch Chip Stone
OC	Staged Embankment Construction and Preloading Between Station 15+260 and 15+360
NSSP	Supply and Installation of Embankment Monitoring Equipment

PART A

FOUNDATION INVESTIGATION REPORT
HIGH FILL EMBANKMENT FROM STATION 15+260 TO 15+360
HIGHWAY 400 WIDENING FROM KING ROAD TO LLOYDTOWN-AURORA
ROAD, REGIONAL MUNICIPALITY OF YORK, ONTARIO
MTO GWP 2835-02-00

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Morrison Hershfield (MH) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services in support of the detail design of the widening of Highway 400 from north of King Road to south of 16th Sideroad, and from north of 16th Sideroad to south of Lloydtown-Aurora Road (i.e., from King Road to Lloydtown-Aurora Road, excluding those portions of the highway widening completed around the 16th Sideroad interchange under a previous contract), as part of MTO Agreement No. 2017-E-0016, Assignment #15.

This report addresses the supplementary foundation investigation carried out for the easterly widening of Highway 400 (northbound lanes) embankment between approximately Stations 15+260 and 15+360, south of 16th Sideroad in King Township, Ontario (see the Key Plan on Drawing 1), as part of a 2016 investigation completed under a separate foundation retainer assignment. The purpose of this investigation was to establish the subsurface soil and groundwater conditions in the vicinity of the proposed embankment widening by test pit excavations, borehole and CPT drilling, and geotechnical and analytical laboratory testing on selected soil and groundwater samples.

This report was developed based on information from the supplementary foundation investigation and the relevant boreholes advanced at the site from Golder's previous foundation investigation, presented in the following report:

- **GEOCRES No. 30M13-217:** "Foundation Investigation and Design Report, High Fill Embankments and Deep Cut, Highway 400 Widening from North of King Road to South Canal Bank Road, King City, Ontario, G.W.P. 2835-02-00", Golder Report No 09-1111-0018-12, dated March 4, 2016.

2.0 SITE DESCRIPTION

The 100 m long section of high fill embankment investigated for this assignment is located on the east side of Highway 400 between approximately Station 15+260 and 15+360, approximately 800 m south of 16th Sideroad in the Regional Municipality of York, Ontario. In general, the topography in the area of the site consists of rolling terrain including agricultural fields, a swampy area and densely treed areas, as shown on Photograph 1.

The existing Highway 400 embankment at this site was constructed in 1951 and is up to about 10 m high relative to the natural ground surface adjacent to the toe of the existing embankment. The embankment is generally sloped at about 2 horizontal to 1 vertical (2H:1V), and the slope face is generally vegetated with grass. No evidence of embankment or pavement settlement or slope instability was observed within this area of the embankment at the time of the 2016 investigation. It is noted that the area to the east of the toe of the slope is characterized as wetland due to the presence of bull rushes and dead trees.

An existing 2.4 m by 1.5 m box culvert (Culvert 29) crosses under Highway 400 at the south end of the investigation area, at about Station 15+260. It is understood that this culvert is to be abandoned and replaced under the widened embankment at about Station 15+250. The subsurface conditions and details for the culvert replacement are provided under a separate cover.

The general configuration of the embankment and topography adjacent to the toe are shown in the following photograph:



Photograph 1: Existing embankment side slope and ground beyond embankment toe

3.0 INVESTIGATION PROCEDURES

3.1 2010-2015 Investigation (GEOCRES No. 30M13-217)

Golder carried out a foundation investigation for the high fill and deep cut embankment widening for Highway 400 from north of King Road to South Canal Bank Road in the Regional Municipality of York during the period between 2010 and 2015. Of the boreholes advanced as part of the previous investigation, a total of nine boreholes (Boreholes C29-3 and C29-4, and F1-5A to F1-5G) were advanced in the area of embankment widening for the current investigation, at the locations shown on Drawing 1. The borehole locations were generally measured by Golder relative to the survey staking along the alignment and site features, and the ground surface elevations at the borehole locations were determined from the digital terrain model for this project. The borehole locations (in MTM NAD 83 Zone 10 northing and easting coordinates and latitude and longitude), ground surface elevations, and borehole depths are summarized below.

Borehole No.	Location (MTM NAD83)		Ground Surface Elevation (m)	Borehole / CPT / Test Pit Depth (m)
	Northing (m)	Easting (m)		
C29-3	4867673.0	298966.3	308.2	18.9
C29-4	4867683.6	298994.4	297.5	11.3
F1-5A	4867746.9	298982.5	299.5	12.8
F1-5B	4867722.2	298986.7	299.1	16.5
F1-5C	4867707.5	298989.6	298.6	14.3
F1-5D	4867706.6	298999.5	298.5	12.8
F1-5E	4867761.6	298979.6	300.5	6.7
F1-5F	4867736.7	298956.0	310.0	14.0
F1-5G	4867733.6	298994.2	298.9	14.3

Classification testing (organic content, water content, grain size distribution and Atterberg limits) was carried out on selected soil samples. In addition, two one-dimensional consolidation (oedometer) tests were carried out on selected samples. The geotechnical laboratory tests were carried out in Golder's geotechnical laboratory to MTO and/or ASTM Standards, as appropriate.

3.2 2016 Investigation

The field work for the current investigation was carried out between August 8 and 15, 2016 during which time a total of two test pits (TP1 and TP2), five boreholes (Boreholes 16-B1, 16-B2, 16-M1, 16-M2, and 16-T1) and two cone penetration tests (CPT16-1 and CPT16-2) were advanced on the east side of Highway 400. The test pits were excavated at the toe of the embankment; one borehole was advanced at the crest of the embankment on the right shoulder of the northbound lanes, two boreholes were advanced at about mid-height of the embankment, and two boreholes were advanced at the toe of the embankment; and two CPT soundings were carried out at the toe of the embankment. The locations of these test pits, boreholes and CPTs are shown in plan on Drawing 1, and the records are included in Appendix B.

The test pits were excavated to depths of 3.3 m and 3.7 m below ground surface using a John Deere 75G excavator and soil samples were obtained from the excavator bucket and placed in 50 litre plastic pails for soil classification testing and laboratory soil mixing testing carried out by Ryerson University (Ryerson) in Toronto, Ontario. The test pits were backfilled with the excavated material in loose lifts and compacted with the flat side of the excavator bucket.

Borehole 16-T1 was advanced using a truck-mounted drilling rig and Boreholes 16-M1, 16-M2, 16-B1 and 16-B2 were advanced using a track-mounted Diedrich D-50 drill rig supplied and operated by Walker Drilling Ltd. of Utopia, Ontario. To access the location of the mid-height embankment boreholes, a bench was formed at about the mid-height of the embankment east slope by modifying the existing side slope geometry using an excavator. Prior to carrying out this field work, a slope stability analysis was carried out to confirm that the factor of safety for

slope stability was adequate. Upon completion of the mid-height embankment boreholes, the slope was restored to its prior condition and the area was seeded.

The boreholes were advanced through the overburden using nominal 210 mm diameter hollow stem augers or wash boring techniques to depths ranging from 11.6 m to 21.0 m below ground surface. Soil samples were obtained at intervals of depth of about 0.75 m and 1.5 m in the boreholes using a nominal 50 mm outside diameter split-spoon sampler driven by an automatic hammer in accordance with the Standard Penetration Test (SPT) procedure.¹ Thin-walled Shelby tube² samples were also taken within the cohesive and organic materials at selected intervals in Boreholes 16-M1 and 16-B2. In-situ field vane testing³ using MTO standard “N” sized vanes, was carried out in the soft to stiff cohesive soils, between split-spoon samples and Shelby tube samples, to measure the undrained shear strength of the organic silt and clayey silt to silty clay deposits.

The cone penetration tests (CPTs) were advanced using the hydraulic system on a track-mounted drill rig, supplied and operated by Walker Drilling, Ontario. The CPTs were advanced through ‘NW’ type casing pre-drilled prior to advancement of the CPTs. Pre-drilling was required to remove compact/stiff layers at shallow depths at the locations of CPT16-1 and CPT16-2.

The CPT is an in-situ technique used to assess a nearly continuous characterization of subsurface soils and was carried out to refusal, which was encountered at depths of about 10.7 m and 12.5 m below ground surface at CPT16-1 and CPT16-2, respectively. The CPT consists of a special probe (Geotech AB 10 cm²) equipped with electronic sensing elements to continuously measure tip resistance (q_c), local side friction on a sleeve (f_s) and pore water pressure (u_2). The CPT probe was pushed into the ground at a constant rate (per ASTM D5778-07 Standard Test Method for Piezocone Penetration) and the data collected nearly continuously (approximately every 2 cm). The CPT data are presented in Appendix B and indicate the depth (m) against measured tip resistance (Q_t), local side Friction, pore water pressure (PWP), blows per 305 mm ($N(60)$), and Normalized Soil Behavior Type index (I_c). The characteristics of the equipment used for this investigation are shown in the following table.

Cone Type/ Description	Cross Sectional Area (cm ²)	Sleeve Area (cm ²)	Tip Capacity (MPa)	Sleeve Capacity (MPa)	Pore Pressure Capacity (MPa)
Geotech AB 10 cm ²	10	150	50	0.5	2.5

The groundwater conditions and groundwater level in the open test pits and boreholes were observed during and upon completion of the excavation and drilling operations. A standpipe piezometer was installed in Borehole 16-B1 to permit monitoring of the groundwater level at the site. The piezometer consists of a 50 mm diameter PVC pipe with a 3 m long slotted screen sealed within a sand filter pack across the silty clay and sand deposits. The borehole and annulus surrounding the pipe above the sand filter was backfilled with bentonite to the ground surface. All other boreholes and CPT holes were backfilled with bentonite upon completion in accordance with Ontario Regulation 903 (Wells, as amended).

The field work was observed by members of Golder’s engineering staff, who located the test pits, boreholes and CPTs, arranged for the clearance of underground services, observed the drilling, sampling and in-situ testing

¹ ASTM D1586-08a – Standard Test Method for Standard Penetration Test (SPT) and Split Barrel Sampling of Soils.

² ASTM D1587M-15 – Standard Practice for Thin-Walled Tube Sampling of Fine-Grained Soils for Geotechnical Purposes.

³ ASTM D2573M-18 – Standard Test Method for Field Vane Shear Test in Saturated Fine-Grained Soils.

operations, logged the test pits, boreholes, and CPTs, and examined the soil samples. The samples were identified in the field, placed in labelled containers, and transported to Golder's Mississauga geotechnical laboratory where the samples underwent further visual examination and laboratory testing.

The as-drilled test pit, borehole, and CPT locations and ground surface elevations were measured using a handheld Global Positioning System (GPS) unit with an accuracy of 0.5 m in the horizontal and 0.1 m in the vertical directions. The locations provided on the records in Appendix B and shown in plan on Drawing 1 are positioned using MTM NAD83 Zone 10 northing and easting coordinates, and the ground surface elevations are referenced to Geodetic datum. The test pit, borehole and CPT locations, ground surface elevations and drilled depths are summarized below.

Borehole / CPT / Test Pit No.	Location (MTM NAD83)		Ground Surface Elevation (m)	Borehole / CPT / Test Pit Depth (m)
	Northing (m)	Easting (m)		
Test Pit 1	4,867,733.2	298,983.5	299.2	3.7
Test Pit 2	4,867,719.7	298,984.9	298.8	3.3
16-B1	4,867,708.5	298,984.0	300.4	21.0
16-B2	4,867,709.5	298,983.8	300.4	14.9
16-M1	4,867,712.2	298,974.0	302.9	15.5
16-M2	4,867,733.4	298,971.1	303.3	11.6
16-T1	4,867,726.5	298,955.8	309.7	20.4
CPT-1	4,867,697.9	298,983.8	298.7	10.7
CPT-2	4,867,719.7	298,984.9	299.9	12.5

3.2.1 Laboratory Testing

Classification testing (organic content, water content, grain size distribution and Atterberg limits) was carried out on selected soil samples. In addition, two one-dimensional consolidation (Oedometer) tests and a consolidated undrained triaxial (CIU) test with pore pressure measurements were carried out on selected samples. The geotechnical laboratory tests were carried out in Golder's geotechnical laboratory to MTO and/or ASTM Standards, as appropriate.

Selected soil samples were submitted to Maxxam Analytics (Maxxam) of Mississauga, Ontario, which is a Standards Council of Canada (SCC) accredited laboratory, for chemical analysis of a suite of parameters that indicate corrosivity potential including pH, resistivity, conductivity, chloride content and sulphate content. A sample of the groundwater from the standpipe piezometer installed in Borehole 16-B1 was submitted to Agat of Mississauga, Ontario, which is a Standards Council of Canada (SCC) accredited laboratory, for chemical analysis of a suite of parameters that indicate corrosivity potential including pH, resistivity, conductivity, chloride content and sulphate content.

The plastic pail samples obtained from the test pits and the Shelby tube samples from Borehole 16-B2 were submitted to Ryerson University (Ryerson) for laboratory soil mixing and associated strength testing of treated samples. The soil samples were mixed by different dosages of cement or lime and unconfined compressive strength testing was completed on treated samples cured for 7, 14, 28 and 56 days. The testing program at Ryerson was completed on treated samples as summarized below:

Soil Source	Binder	Binder Dosage (kg/m ³)	Curing Duration
Combined Test Pits 1 and 2	Cement	150	7, 28, 56
Combined Test Pits 1 and 2	Cement	200	7, 14, 28, 56
Combined Test Pits 1 and 2	Cement	250	7, 14, 28, 56
Combined Test Pits 1 and 2	Lime	100	7, 14, 28, 56
Combined Test Pits 1 and 2	Lime	200	7, 14, 28, 56
Shelby Tubes	Cement	250	7, 14, 28, 56

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

This section of Highway 400 traverses the physiographic region known as Oak Ridges Moraine, according to The Physiography of Southern Ontario (Chapman and Putman, 1984).⁴ The Oak Ridges Moraine predominately consists of sand and gravel, although the southern half of the moraine through King Township consists of clay till. It is understood that during grading for the initial construction of Highway 400 in this area, deep cuts exposed up to about 10 m of till overlying the sand and gravel deposit(s). The northern boarder of the Oak Ridges Moraine is deeply indented by swamp-floored valleys, along which many outwash terraces are found. The Oak Ridges Moraine also contains sandy soils subject to blowing. Some nearly level topography is provided by sandy outwash, or occasionally fine sandy loam (Chapman and Putman, 1984).

Further east of King Township, the Oak Ridges Moraine consists of extensive lacustrine clays and silts under the surface sand.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions encountered in the test pits and boreholes advanced as part of the 2010-2015 and 2016 investigations are presented on the test pit records, borehole records, and CPT records in Appendices A and B, respectively. Lists of abbreviations and symbols are also included in Appendices A and B to assist in the interpretation of the borehole records. The Standard Penetration Test (SPT) "N"-values and vane undrained shear strengths, as presented on the borehole records and in the following sub-sections are uncorrected.

The geotechnical laboratory test results from the 2010-2015 investigation are provided in Appendix A; these figures include results for boreholes within the specific limits of Station 15+260 to 15+360, as well as other

⁴ 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.

boreholes advanced as part of the 2010-2015 investigation within the overall limits of this high fill section for context. The geotechnical laboratory results from the 2016 investigation are provided in Appendix C. The analytical laboratory test results are provided in Appendix D. The results of the geotechnical laboratory soil mixing and associated strength testing of treated samples carried out by Ryerson are included in Appendix E.

The stratigraphic boundaries shown on the borehole records are inferred from observations of drilling progress and non-continuous sampling, observations during drilling and the results of the Standard Penetration Tests (SPTs) and in-situ testing (field vane and CPTs). These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. The interpreted stratigraphic cross-sections shown on Drawings 2 and 3 are a simplification of the subsurface conditions. Furthermore, subsurface conditions will vary between and beyond the test pit, borehole, and CPT locations.

In general, the subsurface conditions in the area between approximately Station 15+260 and 15+360 consist of embankment fill underlain by an upper non-cohesive deposit of silty sand to sand containing pockets of organic soils. The upper non-cohesive deposit is underlain by an upper cohesive deposit, an organic deposit, and a lower cohesive deposit, which are subsequently underlain by a lower non-cohesive deposit of sandy silt to sand.

The organic soil classification at this site is determined based on the organic content of the soil. Soils with organic contents between 5% and 30% are classified as “organic silt” and soils with organic contents greater than 30% are classified as “peat”. Further, organic soils which measured liquid limits are considered “cohesive” and “compressible”. The 2010-2015 borehole records in Appendix A have been annotated to re-classify select organic soil layers, for consistency with the 2016 investigation.

The organic and cohesive deposits are more predominant and thicker beyond the toe of the existing embankment and are encountered as interlayers beneath the existing embankment (in the boreholes advance mid-slope and at the shoulder of the highway). In a test pit excavated at the toe of the embankment, tree limbs and branches were encountered in the sand deposit overlying the organic soils. “Blowing sand conditions” were experienced within the lower non-cohesive deposit due to groundwater pressure, which are considered to have impacted (lowered) the SPT “N”-values in the lower non-cohesive deposit at some locations.

A more detailed description of the subsurface conditions encountered in the test pits, boreholes, and CPTs advanced as part of the 2010-2015 and 2016 investigations are provided in the following sections.

4.2.1 Asphalt

Boreholes 16-T1, C29-3 and F1-5F were advanced from the existing Highway 400 embankment and encountered a 150 mm to 200 mm thick layer of asphalt between about Elevation 310.0 m and 308.2 m.

4.2.2 Topsoil

Boreholes F1-5A, F1-5B, F1-FC and F1-5E and Test Pits TP-1 and TP-2 were advanced/excavated at or beyond the toe of the existing Highway 400 embankment and encountered a 0.1 to 0.3 m thick layer of topsoil between about Elevation 300.5 m and 298.6 m.

4.2.3 Fill

Boreholes 16-T1, C29-3 and F1-5F were drilled from Highway 400 grade and encountered between 8.5 m and 12.2 m of fill generally comprised of silt and sand to silty sand to sand and gravel below the asphalt surface (between about Elevation 309.8 m and 308.0 m), and the fill extends to depths ranging from 8.7 m to 12.4 m below the pavement grade (to Elevation 301.1 m to 297.3 m). In Borehole C29-3, the upper 1.2 m zone of the fill

is comprised of clayey silt. Boreholes 16-M1 and 16-M2 were advanced on the existing Highway 400 side slopes and encountered sandy silt to silt and sand fill from ground surface to a depth of 6.1 m and 6.4 m below ground surface (to Elevation 296.8 m and 296.9 m), respectively. Borehole 16-B1 was drilled at the toe of the existing embankment and penetrated a 1.4 m thick layer of sand fill extending to a depth of 1.4 m (Elevation 299.0 m) underlain by a 0.6 m thick layer of sand and gravel fill extending to a depth of 2 m below the ground surface (Elevation 298.4 m). Test Pits 1 and 2, excavated beyond the existing embankment toe, encountered granular fill extending to depths of 0.9 m (Elevation 298.3 m) and 2.1 m (Elevation 296.7 m) below ground surface, respectively. The fill layer in places contains pockets and lenses of sandy silt.

The measured SPT “N”-values within the non-cohesive fill range from 6 blows to 62 blows per 0.3 m of penetration, but generally less than 39 blows per 0.3 m of penetration, indicating a generally loose to dense relative density with very dense zones. Within the sand and gravel layer in Borehole 16-B1, an SPT “N”-value of 50 blows per 0.1 m of penetration was measured, suggesting the presence of coarse gravel in the fill material.

As part of the 2010-2015 investigation, a grain size distribution analysis was carried out on eleven samples of the fill and the results are presented on Figure A-1A and A-1B in Appendix A. As part of the 2016 investigation, a grain size distribution analysis was carried out on six samples of the fill and the results are presented on Figure C-1 in Appendix C.

Atterberg limits tests were carried out on four samples of the fill from the 2010-2015 investigation and one sample from the 2016 investigation, and the results are presented on Figure A-2 in Appendix A and Figure C-2 in Appendix C, respectively. The Atterberg limits tests measured liquid limits ranging from 15% to 31% and plasticity indices ranging from 4% to 14%, indicating the cohesive portion of the fill is classified as silt of slight plasticity to clayey silt of low plasticity. The water content measured on samples of the fill obtained during the current investigation ranges from about 6% to 17%.

4.2.4 Upper Non-Cohesive Deposit (Sandy Silt to Silty Sand to Sand)

An upper non-cohesive deposit of sandy silt to silty sand to sand was encountered at ground surface in Borehole F1-5G, underlying the topsoil in Boreholes F1-5C and F1-5E, underlying the fill in Boreholes C29-3, F1-5F, 16-B1, 16-B2, 16M-1, 16M-2, and underlying the upper cohesive deposit in Boreholes C29-4, F1-5B, and 16-T1. The elevation of the surface and base of the upper non-cohesive deposit and the thickness as encountered in the boreholes are summarized below.

Borehole / Test Pit No.	Depth to Surface of Deposit (m)	Deposit Surface Elevation (m)	Deposit Thickness (m)	Deposit Base Elevation (m)
Boreholes advanced from Highway 400 grade				
C29-3	8.7	299.5	3.0	296.5
F1-5F	8.9	301.1	4.4	296.7
16-T1	12.7	297.0	4.5	292.5
Borehole advanced on the side slope of the existing embankment				
16-M1	6.1	296.8	6.5	290.3
16-M2	6.4	296.9	2.7	294.2
Boreholes advanced at the toe of the existing embankment (listed from south to north)				
C29-4	2.2	295.3	3.4	291.9
F1-5B	0.7	298.4	1.1	297.3
F1-5C	0.1	298.5	2.2	296.3
F1-5G	At ground surface	298.9	0.8	298.1
F1-5E	0.1	300.4	2.1	298.3
16-B1	2.0	298.4	2.5	295.9
16-B2	Higher than 4.6 m	Higher than 295.8	0.1 or greater	295.7 or lower

The measured SPT “N”-values within the upper non-cohesive deposit generally range from 1 blow to 32 blows per 0.3 m of penetration, indicating a very loose to dense relative density. It is noted that some of the SPT “N”-values were impacted by the hydrostatic pressure in the sand layer.

Grain size distribution tests were carried out on eight samples from the 2010-2015 investigation and four samples from the 2016 investigation, and the results are presented on Figures A-5A and A-5B in Appendix A and Figure C-4 in Appendix C, respectively.

As part of the 2016 investigation, an Atterberg limits test was carried out on one sample of the upper non-cohesive deposit and measured a liquid limit of 14%, and a plastic limit of 12%, corresponding to a plasticity index of 2%. The test result is shown on the plasticity chart on Figure C-5 in Appendix C and indicates that the material is classified as a silt of slight plasticity. The water contents measured on samples of the upper non-cohesive deposit range from about 12% to 30%.

4.2.5 Upper Cohesive Deposit (Clayey Silt-Silt to Clayey Silt)

An upper cohesive deposit consisting of clayey silt-silt to clayey silt to sandy clayey silt was encountered at ground surface in Borehole C29-4, underlying the fill in Borehole 16-T1 and Test Pit 1, underlying the topsoil in Boreholes F1-5A and F1-5B, and underlying the upper non-cohesive deposit in Boreholes 16-B1, 16-M2, and F1-

5E to F1-5G,. The elevation of the surface and base of the upper cohesive deposit and the thickness as encountered in the boreholes are summarized below.

Borehole / Test Pit No.	Depth to Surface of Deposit (m)	Deposit Surface Elevation (m)	Deposit Thickness (m)	Deposit Base Elevation (m)
<i>Boreholes advanced from Highway 400 grade</i>				
16-T1	12.4	297.3	0.3	297.0
F1-5F	13.3	296.7	0.7	296.0
<i>Boreholes advanced on the side slope of the existing embankment</i>				
16-M2	9.1	294.2	0.4	293.8
<i>Boreholes advanced at the toe of the existing embankment (listed from south to north)</i>				
C29-4	At ground surface	297.5	2.2	295.3
16-B1	4.5	295.9	1.0	294.9
F1-5B	0.1	299.0	0.6	298.4
F1-5G	0.8	298.1	2.4	295.7
Test Pit 1	0.9	298.3	2.8 or greater	295.5 or lower
F1-5A	0.2	299.3	2.5	296.5
F1-5E	2.2	298.3	1.3	297.0

The measured SPT “N”-values within the upper cohesive deposit generally range from 2 blows to 14 blows per 0.3 m of penetration, suggesting a soft to stiff consistency. In Borehole C29-4, advanced near the outlet of an existing culvert, SPT “N”-values of 1 blow per 0.3 m of penetration were measured. In Borehole 16-T1, an SPT “N”-value of 26 blows per 0.3 m of penetration was recorded, indicating a very stiff consistency.

As part of the 2010-2015 investigation, a grain size distribution analysis was carried out on three samples of the upper cohesive deposit and the results are presented on Figure A-3 in Appendix A. As part of the previous investigation, Atterberg limits tests were carried out on four samples of the upper cohesive deposit from the 2010-2015 investigation and one sample from the 2016 investigation, and the results are presented on Figure A-4 in Appendix A and Figure C-4 in Appendix C, respectively. The Atterberg limits tests measured liquid limits ranging from about 24% to 32% and plasticity indices ranging from 7% to 10%, indicating that the material is classified as a clayey silt-silt to clayey silt of low plasticity. The water contents measured on samples of the upper cohesive deposit range from about 10% to 45%.

4.2.6 Organic Deposit

A deposit of organic soils comprised of clayey organic silt to organic silt to silty sandy peat was encountered below the upper non-cohesive deposit or below the upper cohesive deposit in the test pits, boreholes, and CPTs

noted below. The elevation of the surface and base of the organic deposit and the thickness as encountered in the test pits, boreholes, and CPTs is summarized below.

Borehole / Test Pit / CPT No.	Depth to Surface of Deposit (m)	Deposit Surface Elevation (m)	Deposit Thickness (m)	Deposit Base Elevation (m)
Borehole advanced from Highway 400 grade				
16-T1	17.2	292.5	0.6	291.9
Borehole advanced on the side slope of the existing embankment				
16-M2	9.5	293.8	0.6	293.2
Boreholes advanced at the toe of the existing embankment (listed from south to north)				
16-CPT1 ¹	3.1	295.6	3.7	291.9
16-CPT2 ¹	3.2	296.8	3.7	293.1
Test Pit 2	2.1	296.7	1.2 or greater	295.5 or lower
F1-5A	3.0	296.5	2.6	293.9
F1-5B	1.8	297.3	5.7	291.6
	12.5 ²	286.6	0.9	285.7
F1-5C	2.3	296.3	8.1	288.2
F1-5D	1.5	297.0	6.1	290.9
F1-5E	3.5	297.0	0.8	296.2
F1-5G	3.2	295.7	5.2	290.5
16-B1	5.5	294.9	3.0	291.9
16-B2	4.7	295.7	3.5	292.2

Notes:

- 1- The top and bottom depths/elevations and thickness are inferred from a correlation after Jefferies and Davies⁵.
- 2- Beneath the silty clay deposit.

The longitudinal extent of the organic deposit is interpreted to extend from Station 15+270 (between Boreholes C29-4 and CPT16-1) to Station 15+350 (about 15 m north of Borehole F1-5E), and the deposit is interpreted to extend laterally from approximately the midpoint of the existing embankment side slope easterly beyond the toe of the embankment (although a thin (0.6 m thick) layer of sandy clayey organic silt was also encountered below the embankment in Borehole 16-T1).

Test Pit 2 encountered tree limbs and tree branches within the organic deposit near the base of the test pit at a depth of about 3.3 m below ground surface as shown in the following photographs. During the short duration of the test pit excavation, the side walls remained near-vertical and the test pit was dry upon completion of excavation; however, this test pit was excavated in August and these conditions may not represent the groundwater table during wetter seasons.



Photograph 2: View of Test Pit 2



Photograph 3: Tree Limb at Base of Test Pit 2

The measured SPT “N”-values within the organic deposit range from 0 blows (weight of hammer) to 9 blows per 0.3 m of penetration. Four in-situ field vane tests carried out within the organic deposit measured undrained shear strengths ranging from 33 kPa to 75 kPa. The sensitivity of the organic deposit ranges between 2 and 4. The field vane test results together with the SPT “N”-values suggest that the organic deposit has a very soft to stiff consistency/very loose to loose relative density.

As part of the 2010-2015 investigation, grain size distribution tests were carried out on five samples of the organic deposit and the results are presented on Figures A-10A to A-10C in Appendix A. As part of the 2016 investigation, a grain size distribution test was carried out on one sample of organic deposit and the results are presented on Figure C-6 in Appendix C.

As part of the 2010-2015 investigation, Atterberg limits tests were carried out on twelve samples of the organic deposit and the results are presented on Figures A-11A to A-11C in Appendix A. As part of the 2016 investigation, Atterberg limits tests were carried out on three samples of the organic deposit and the results are

presented on Figure C-7 in Appendix C. The results of the Atterberg limits tests are summarized in the table below and indicate that the samples are classified as clayey organic silt of medium to high plasticity.

Borehole / Sample No.	Liquid Limit	Plastic Limit	Plasticity Index	Plasticity Classification
F1-5A / 5	49	31	18	Medium plasticity
F1-5B / 8	83	62	21	High plasticity
F1-5B / 10	82	59	23	High plasticity
F1-5B / 15B	53	38	15	High plasticity
F1-5C / 4	65	37	28	High plasticity
F1-5C / 7	89	77	12	High plasticity
F1-5C / 8B	67	41	26	High plasticity
F1-5C / 10	56	38	18	High plasticity
F1-5D / 3A	55	33	22	High plasticity
F1-5D / 7B	61	43	18	High plasticity
F1-5D / 8	63	28	35	High plasticity
F1-5E / 6A	54	41	13	High plasticity
16-B1 / 10	106	95	11	High plasticity
16-T1 / 18B	45	29	16	Medium plasticity
TP2 / 1	45	37	8	Medium plasticity
Average	65	46	19	Medium to High plasticity

The water contents of samples from the organic deposits obtained during the 2016 investigation range from about 8% to 76%. The measured organic content and the corresponding organic soil classification of selected samples from the test pits and boreholes obtained during the 2010-2015 and 2016 investigations are summarized below.

Borehole / Sample No.	Depth (m)	Elevation (m)	Organic Content (%)	Organic Soil Classification
F1-5B / 5	3.4	295.7	68	Peat
F1-5B / 7	4.9	294.2	54	Peat
F1-5D / 6	4.2	294.3	28	Organic Silt
F1-5D / 7B	5.2	293.3	8	Organic Silt
F1-5G / 5	4.1	294.8	76	Peat
F1-5G / 7B	5.6	293.3	18	Organic Silt
16-B1 / 8B	5.7	294.7	40	Peat
16-B1 / 9	6.4	294.0	25	Organic Silt
16-B1 / 10	7.2	293.2	25	Organic Silt
16-B2 / 1	4.8	295.6	66	Peat
Test Pit 2 / 1	0.7	298.1	14	Organic Silt

As part of the 2010-2015 investigation, a laboratory one-dimensional consolidation (oedometer) test was carried out on one specimen of the organic deposit obtained from the Shelby tube sample in Borehole F1-5D and the results are presented on Figure A-13 in Appendix A. A pre-consolidation pressure of about 55 kPa was estimated from the void ratio versus logarithmic pressure plot and from the total work versus pressure plot. A bulk unit weight of about 17.8 kN/m³ and a specific gravity of about 2.7 were measured on the consolidation test specimen. Details of the test results are summarized below.

Borehole Sample No.	Sample Depth / Elevation	σ_{vo}' (kPa)	σ_p' (kPa)	$\sigma_p' - \sigma_{vo}'$ (kPa)	OCR	C_c	C_r	e_o	c_v^* (cm ² /s)
Borehole F1-5D Sample 8	6.5 m / 292.0 m	55	55	0	1.00	0.29	0.045	1.18	1.2x10 ⁻³

Note: * For stress range of 45 kPa $\leq \sigma_v' \leq$ 180 kPa

where: σ_{vo}' is the effective overburden pressure in kPa

σ_p' is the pre consolidation pressure in kPa

OCR is over consolidation ratio

e_o is initial void ratio

C_c is the compression index

C_r is the recompression index

c_v is the coefficient of consolidation in cm²/s

4.2.7 Lower Cohesive Deposit (Clayey Silt to Silty Clay to Clay)

A lower cohesive deposit of clayey silt to silty clay to clay was encountered underlying the upper non-cohesive deposit in Borehole 16-M1, underlying the organic deposit in Boreholes 16-M2, 16-B1, 16-B2, F1-5B, and F1-5G. The deposit in some places contains organic matter and shell pieces. The elevation of the surface and base of the lower cohesive deposit and the thickness as encountered in the boreholes is summarized below.

Borehole / CPT No.	Depth to Surface of Deposit (m)	Deposit Surface Elevation (m)	Deposit Thickness (m)	Deposit Base Elevation (m)
<i>Boreholes advanced on the side slope of the existing embankment</i>				
16-M1	12.6	290.3	0.5	289.8
	14.9	288.0	0.6 or greater	287.4 or lower
16-M2	10.1	293.2	1.0	292.2
<i>Boreholes advanced at the toe of the existing embankment (listed from south to north)</i>				
F1-5B	7.5	291.6	5.0	286.6
F1-5G	8.4	290.5	3.2	287.3
16-B1	8.5	291.9	3.6	288.3
16-B2	8.2	292.2	5.8	286.4

The measured SPT “N”-values within the lower cohesive deposit range from 0 blows (weight of hammer) to 4 blows per 0.3 m of penetration. Measurements of the undrained shear strength in this deposit during the 2010-2015 investigation were 40 kPa and 60 kPa. An in-situ field vane test was carried out within the lower cohesive deposit during the 2016 investigation and measured an undrained shear strength of 30 kPa and a sensitivity of 3. The field vane test results together with the SPT “N”-values indicate that the lower cohesive deposit generally has a very soft to firm consistency, except at Boreholes C29-3 and C29-4 where the deposit is considered stiff to very stiff.

As part of the 2010-2015 investigation, a grain size distribution test was carried out on two samples of the lower cohesive deposit and the results are presented on Figures A-8 in Appendix A. Atterberg limits tests were carried out on five samples of the lower cohesive deposit from the 2010-2015 investigation and the results are presented on Figure A-9 and A-11D in Appendix A. As part of the 2016 investigation, Atterberg limits testing was carried out on six samples of this lower cohesive deposit and the results are presented on Figure C-8 in Appendix C. The results of the Atterberg limits tests are summarized in the table below and indicate that the material is classified as clayey silt of low plasticity to clay of high plasticity.

Borehole / Sample No.	Liquid Limit (%)	Plastic Limit (%)	Plasticity index (%)	Plasticity Classification
F1-5B / 12	43	22	21	Medium plasticity
F1-5B / 14	46	26	20	Medium plasticity
F1-5G / 9	45	23	22	Medium plasticity
F1-5G / 11	37	24	13	Medium plasticity
F1-5E / 4	37	24	13	Medium plasticity
16-B1 / 13	49	26	23	Medium plasticity
16-B1 / 14	41	24	17	Medium plasticity
16-B1 / 15	39	22	17	Medium plasticity
16-B2 / 5	61	26	35	High plasticity
16-B2 / 10	37	22	15	Medium plasticity
16-M2 / 8C	37	21	15	Medium plasticity
Average	42	23	19	Medium to high plasticity

The water content measured on samples of the lower cohesive deposit obtained during the 2016 investigation ranges from 11% to 90%, but is generally greater than 41%. The organic content measured from samples from this deposit east of the toe of the embankment, obtained during the 2010-2015 and 2016 investigations, ranges from about 4% to 8%.

As part of the 2010-2015 investigation, a laboratory one-dimensional consolidation (oedometer) test was carried out on a specimen of the cohesive deposit obtained from Shelby tube sample in Borehole F1-5B and the results are presented on Figure A-12 in Appendix A. As part of the 2016 investigation, laboratory one-dimensional consolidation (oedometer) tests were carried out on two specimens of the cohesive deposit obtained from Shelby tube samples in Borehole 16-B2 and the results are presented on Figures C-9 and C-10 in Appendix C. Pre-consolidation pressures of about 65 kPa, 65 kPa and 115 kPa were estimated from the void ratio versus logarithmic pressure plot and from the total work versus pressure plot for the respective samples. Bulk unit weights of about 17.3 kN/m³, 16.0 kN/m³ and 17.4 kN/m³ and specific gravities of about 2.7 were measured on the consolidation test specimen. Details of the test results are summarized below.

Borehole / Sample No.	Sample Depth / Elevation	σ_{vo}' (kPa)	σ_p' (kPa)	$\sigma_p' - \sigma_{vo}'$ (kPa)	OCR	C_c	C_r	e_o	c_v^* (cm ² /s)
F1-5B / 12	8.4 m / 290.7 m	65	65	0	1.00	0.35	0.036	1.20	$2.3 \times 10^{-3**}$
16-B2 / 5	8.7 m / 291.7 m	100	65	-35	1.00	0.42	0.041	1.49	2.7×10^{-3}
16-B2 / 10	14.0 m / 286.4 m	125	125	0	1.00	0.28	0.02	1.17	1.5×10^{-3}

Notes:

* For stress range of $100 \text{ kPa} \leq \sigma_v' \leq 150 \text{ kPa}$

** For stress range of $75 \text{ kPa} \leq \sigma_v' \leq 200 \text{ kPa}$

where: σ_{vo}' is the effective overburden pressure in kPa

σ_p' is the pre consolidation pressure in kPa

OCR is over consolidation ratio

e_o is initial void ratio

C_c is the compression index

C_r is the recompression index

c_v is the coefficient of consolidation in cm²/s

Laboratory consolidated isotropic undrained triaxial compression tests (CIU) with pore pressure measurement were carried out on a Shelby tube sample of the cohesive deposit obtained in Borehole 16-B2. In total, three specimens were tested under consolidation pressures ranging from 25 kPa to 125 kPa. The details of the test results are shown on Figure C-11 in Appendix C and the results are summarized below.

Borehole / Sample No.	Sample Depth / Elevation	Effective Cohesion, c' (kPa)	Effective Angle of Internal Friction, ϕ' (°)
16-B2 / 5	8.4 m / 292.0 m	11	27

The triaxial test specimens were consolidated to pressures representative of the estimated in-situ effective stress as well as stresses above and below this stress level, at the same depth. The interpreted effective strength parameters provided above are applicable only to design situations for which the stress conditions during testing are representative. Reference should be made to the test results for details of the testing conditions.

4.2.8 Lower Non-Cohesive Deposit

A lower non-cohesive deposit comprised of sandy silt to silt and sand to silty sand to sand to sand and gravel was encountered below the organic and/or lower cohesive deposits in the boreholes listed below. The elevation of the surface and base of the lower non-cohesive deposit and its thickness as encountered in the boreholes are summarized below.

Borehole No.	Depth to Surface of Deposit (m)	Deposit Surface Elevation (m)	Deposit Thickness (m)	Deposit Base Elevation (m)
<i>Boreholes advanced from Highway 400 grade (listed from south to north)</i>				
16-T1 ¹	12.7	297.0	7.7 or greater	289.3 or lower
F1-5F	14.0	296.0	6.4 or greater	289.6 or lower
<i>Boreholes advanced on the side slope of the existing embankment</i>				
16-M1 ²	13.1	289.8	1.8	288.0
16-M2	11.1	292.2	0.5 or greater	291.7 or lower
<i>Boreholes advanced at the toe of the existing embankment (listed from south to north)</i>				
F1-5A	5.6	293.9	7.2 or greater	286.7 or lower
F1-5B	13.4	285.7	0.9 or greater	284.8 or lower
F1-5C	10.4	288.2	3.9 or greater	284.3 or lower
F1-5D	7.6	290.9	5.2 or greater	285.7 or lower
F1-5E	4.3	296.2	2.4 or greater	293.8 or lower
F1-5G	11.6	287.3	2.7 or greater	284.6 or lower
16-B1	12.1	288.3	6.8 or greater	281.5 or lower
16-B2	14.0	286.4	0.9 or greater	285.5 or lower

Notes:

- 1- Contains a 0.6 m thick clayey silt pocket at a depth of 17.2 m (Elevation 292.5 m).
- 2- Borehole 16-M1 penetrated 0.6 m into a silty clay deposit below the sand layer. Borehole 16-M1 was terminated within the clay deposit.

The measured SPT “N”-values within the lower non-cohesive deposit range from 1 blow to 47 blows per 0.3 m of penetration but generally greater than 9 blows per 0.3 m of penetration, indicating a generally loose to dense relative density. Generally, the majority of the boreholes were advanced using wash boring techniques in order to counterbalance the groundwater pressure in the sand through the addition of water inside the hollow stem augers and or casing; however, low SPT “N”-values were recorded and are inferred in part to be due to the “blow back” in the sand deposit that occurred as the boreholes were advanced below the underlying the cohesive layer in Boreholes 16-T1, 16-B1, 16-B2, F1-5A and F1-5C.

As part of the 2010-2015 investigation, a grain size distribution test was carried out on six samples of the lower non-cohesive deposit and the results are presented on Figures A-14 in Appendix A. Four samples of the lower non-cohesive deposit were tested from the 2016 investigation and the results are presented on Figures C-12 in Appendix C. As part of the 2016 investigation, an Atterberg limit test were carried out on a sample of the lower non-cohesive deposit and the results are presented on Figure C-13 in Appendix C. The Atterberg limit test measured a liquid limit of 19%, a plastic limit of 15 %, and a plasticity index of about 4%, indicating the fines portion of the sample is classified as a silt of slight plasticity. The water content measured on samples of the non-

cohesive deposit obtained during the 2016 investigation ranges from about 12% to 31%. The organic content measured on one sample of the sand is 2%.

4.2.9 Clayey Silt Till

Underlying the lower non-cohesive deposit at Boreholes C29-3 and C29-4, a 6.1 m and 5.1 m thick cohesive clayey silt till deposit was encountered at about Elevations 296.5 m and 291.9 m, respectively. The till deposit extends to about Elevations 290.4 m and 286.8 m in Boreholes C29-3 and C29-4, respectively, and is underlain by a clayey silt layer, which is inferred to be an interlayer within the till deposit. Both boreholes were terminated within this deposit at Elevations 289.3 m and 286.2 m, respectively.

The SPT “N”-values measured within the clayey silt till deposit range from 13 blows to 29 blows per 0.30 m of penetration, suggesting a stiff to very hard consistency. SPT “N”-values of 14 blows and 26 blows per 0.30 m of penetration were measured within the clayey silt interlayer, suggesting a stiff to very stiff consistency.

As part of the 2010-2015 investigation, grain size distribution tests were carried out on 23 samples of the till deposit taken throughout this overall high fill embankment area, and the results are presented on Figures A-6A to A-6D and A-8 in Appendix A. As part of the 2010-2015 investigation, Atterberg limit tests were carried out on 3320 samples of the till deposit and clayey silt/silty clay from the overall high fill embankment area and the results are presented on Figures A-7A to A-7D and A-9 in Appendix A.

4.2.10 Groundwater Conditions

In general, the soil samples taken during the current investigations were moist to wet. Boreholes 16-T1, 16-M1 and 16-B1 were advanced with wash boring techniques/tricone and therefore the groundwater level inside the casing/open borehole is not representative of the groundwater conditions.

A standpipe piezometer was installed in Borehole F1-5B of the previous investigation and in Borehole 16-B1 of the current investigation. The groundwater level measurements in the piezometers are summarized below.

Borehole No.	Screened Deposit	Screened Elevations (m)	Date	Depth (m)	Elevation (m)
F1-5B	Clayey organic silt / silty sandy peat	294.7 – 293.0	November 10, 2011	5.2	293.9
			November 11, 2011	2.3	296.8
			August 15, 2016	0.1	299.0
			October 17, 2016	0.2	298.9
16-B1	Clayey organic silt / silty clay / sand	292.1 – 284.4	August 15, 2016	1.8	298.6
			October 17, 2016	2.3	298.1

The recorded groundwater level in Borehole F1-5B fluctuated depending on the time of year that the measurement was taken. It is noted that the area to the east of the toe of the slope is characterized as wetland, which is corroborated by the presence of bullrushes and dead trees. The groundwater level measured in the piezometer installed in Borehole 16-B1 is under hydrostatic pressure as the water level is above the cohesive deposit, therefore the cohesive deposit is acting as a confining layer.

The groundwater level will fluctuate seasonally in response to changes in precipitation and snow melt and is expected to be higher in the spring following snow melt, as well as during and following periods of sustained precipitation. It is anticipated that the groundwater level will be at or near ground surface in the area beyond the current embankment toe during spring and other such wet periods. In addition, groundwater may be “perched” locally in non-cohesive fill materials, atop the cohesive deposit, particularly following periods of precipitation.

4.3 Analytical Chemical Testing (Soil and Groundwater)

A total of four soil samples and one groundwater sample were submitted for chemical testing to evaluate the aggressiveness of the soils and groundwater with respect to buried ground improvement elements constructed with concrete. The soil samples were submitted under chain-of-custody protocol to Maxxam Analytics (Maxxam) of Mississauga, Ontario, which is a Standards Council of Canada (SCC) accredited laboratory. Summarized below are the results of the analysis and the analytical laboratory test report is provided in Appendix D.

Parameter	Units	Borehole No. / Soil Sample No. (Depth interval below ground surface)			
		16-B1 / Sample 11 (7.6-8.2 m)	16-B1 / Sample 14 (9.9-10.5 m)	16-B1 / Sample 17 (12.2-12.8 m)	16-B1 / Sample 18 (13.7-14.3 m)
Resistivity	ohm-cm	520	2,000	1,700	1,400
Soluble (20:1) Chloride (Cl)	ug/g	700	170	220	350
Electrical Conductivity	umho/cm	1,940	494	573	695
pH	pH	7.22	7.24	7.54	7.77
Soluble (20:1) Sulphate (SO ₄)	ug/g	1,100	<20	100	80

A groundwater sample was obtained from the standpipe piezometer installed in Borehole 16-B1, which was screened in the clayey organic silt, silty clay and sand deposits between Elevation 292.1 m and 284.4 m, for chemical analysis. Three piezometer volumes of water were purged from the piezometer prior to taking the sample in order to obtain a representative sample of the groundwater. The sample of groundwater was submitted under chain-of-custody protocol to Agat Laboratories. Agat is a SCC accredited analytical laboratory. Summarized below are the results of the analysis and the analytical laboratory report is provided in Appendix D.

Parameter	Unit	Groundwater Sample 1
Resistivity	ohms-cm	280
Chloride	ug/L	856,000
Electrical Conductivity	umho/cm	3,570
pH	pH	7.98
Sulphate	ug/L	45,900

4.4 Ground Improvement Laboratory Testing

Soil samples from the organic and cohesive soil deposits retrieved in the test pits and boreholes during the 2016 investigation were submitted to Ryerson for laboratory soil mixing and associated strength testing of treated samples. Most of the mixes were conducted on samples from the organic deposit while some batches of mixes were prepared with samples of the cohesive deposit. Cement and lime were used as the binder. Cement dosages ranging from 150 kg/m³ to 250 kg/m³ and lime dosages of 100 kg/m³ and 200 kg/m³ were used in the soil mixtures. Samples of the organic and cohesive deposits were mixed with cement and lime binders and cured for durations of 7, 14, 28, and 56 days. A water to binder ratio of 0.8 to 1 by mass was implemented to prepare the binder slurries. Details of the mixing procedures are summarized in the Ryerson Report included in Appendix E.

Unconfined Compressive Strength (UCS) tests were carried out to evaluate the changes in shear strength of the treated samples mixed with various amounts and types of binders. Samples of the mixed soils with different binders were sheared at a strain rate of 1% per minute until 25% strain was reached. The results of the UCS tests completed on untreated and treated samples are summarized below.



Native Soil	Binder	Dosage (kg/m ³)	UCS range (kPa)	Average UCS (kPa)
Silty Clay to Clay	Untreated	0	27-56	43
Silty Clay to Clay	Cement	150	211-407	321
Silty Clay to Clay	Cement	200	152-497	329
Silty Clay to Clay	Cement	250	378-609	487
Silty Clay to Clay	Lime	100	17 – 21	19
Silty Clay to Clay	Lime	200	5 – 37	24

5.0 CLOSURE


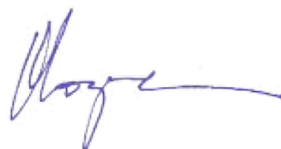
This Foundation Investigation Report was prepared by Anastasia Poliacik, P.Eng. a senior geotechnical engineer with Golder. Lisa Coyne, P.Eng., an MTO Foundations Designated Contact and Principal with Golder, conducted an independent quality review of the report.

Signature Page

GOLDER ASSOCIATES LTD.



Anastasia Poliacik, P.Eng.
Senior Geotechnical Engineer



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

AMP/LCC/ljv

Golder, Golder Associates and the GA globe design are trademarks of Golder Associates Corporation.

<https://golderassociates.sharepoint.com/sites/21998g/deliverables/wo15-hwy400wideninglloydton/8.highfill/5.final/1786658-wo15rev020220711fidr-hwy400widening-lloydton-highfill.docx>

PART B

FOUNDATION DESIGN REPORT

HIGH FILL EMBANKMENT FROM STATION 15+260 TO 15+360

HIGHWAY 400 WIDENING FROM KING ROAD TO LLOYDTOWN-AURORA
ROAD, REGIONAL MUNICIPALITY OF YORK, ONTARIO

MTO GWP 2835-02-00

6.0 FOUNDATION ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides discussion and recommendations on foundation engineering aspects for the proposed widening of an approximately 100 m long section of high fill embankment, extending from approximately Station 15+260 to Station 15+360 along the east side of Highway 400, approximately 800 m south of 16th Sideroad, in the Regional Municipality of York, Ontario. The recommendations are based on interpretation of the factual data obtained from the boreholes, test pits and CPTs advanced during a series of investigation from 2010-2015 and 2016.

The foundation investigation report, discussion and recommendations are intended for the use of MTO and its designers and shall not be used or relied upon for any other purpose or by any other parties, including the construction contractor. The contractor must make their own interpretation based on the factual data in Part A of the report (Foundation Investigation 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 or operational constraints may be required in the Contract Documents. Contractors must make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, schedule and the like.

6.2 Embankment Widening Requirements and Options

The proposed eastward construction of the Highway 400 embankment between approximately Station 15+260 and 15+360 will require a widening of approximately 12 m as measured from the existing crest to the proposed crest, based on the design cross-sections. The design cross-sections further indicate a design highway grade varying from approximately Elevation 308 m at Station 15+260 to approximately Elevation 312 m at Station 15+360. The existing ground surface at the east toe of the embankment ranges from approximately Elevation 297 m at Station 15+260 to Elevation 304 m at Station 15+360.

The existing and widened embankments have a maximum height of approximately 10 m. In accordance with MTO's standard practice, to minimize surficial erosion on the embankment side slopes, a minimum 2 m wide bench must be provided where embankment slopes are greater than 8 m in height.

The 2010-2015 investigation identified the presence of relatively thick, localized organic and soft soils in this embankment widening area, which were documented in a previous Foundation Investigation and Design Report (MTO GEOCRE No. 30M13-217). Subsequently, the 2016 investigation was completed to further assess the extent of weak/organic material and included sampling and testing of various mix proportions to support a potential intrusive ground improvement mitigation option. The 2016 investigation confirmed there are relatively limited thicknesses of weak/organic materials below the existing embankment side slopes, although these weak materials are up to approximately 9 m in thickness within the footprint of the widening.

Due to the presence of these weak soils, conventional embankment widening using earth or granular fill at maximum side slopes of 2 horizontal to 1 vertical (2H:1V) will not achieve the required minimum Factor of Safety for global stability, as described in Sections 6.3 and 6.4. Accordingly, Golder worked with MH and MTO throughout the design to assess numerous stability and settlement mitigation options including the following:

- **Subexcavation of weak/organic soils:** This option was deemed impracticable adjacent to Highway 400, as the depth of required subexcavation would necessitate temporary protection systems along the highway, generate significant quantities of excess soil to be managed, and require significant import of granular fill to be placed in wet conditions. Hence, this option is not considered further in this report.
- **Application of various intrusive ground improvement techniques (controlled modulus columns, stone columns and deep soil mixing):** These intrusive ground improvement options had been contemplated early in the design process for this site, but were not developed in detail once it became apparent that sufficient time was available in the construction schedule for staged construction and preloading to address global slope stability and settlement.
- **Assessment of various side slope geometries with various conventional and lightweight fill materials:** Numerous embankment geometries were assessed with varying side slope angles and toe berm widths, using conventional earth/granular fill as well as (in order from heaviest to lightest) 3/8-inch chip stone, lightweight and ultra-lightweight slag fill, cellular concrete, and expanded polystyrene foam.

Table 1 following the text of this report presents a comparison of advantages, disadvantages, constructability, risks, and relative costs for staged construction/preloading with the use of lighter weight fill and various intrusive ground improvement methods noted above. Through consideration of numerous options in this design process, the use of 3/8-inch chip stone was selected by the design team and MTO, and the embankment geometry and staging were determined to achieve the required minimum Factors of Safety. The construction staging considered acquisition of property in Spring 2023 to permit an initial stage of filling, with a second stage of filling in Spring 2024 followed by final paving prior to Fall 2024.

Therefore, the following sections of this report are specific to staged construction with the use of lightweight fill materials, specifically 3/8-inch Chip Stone having a maximum bulk unit weight of 18 kN/m³. Discussion regarding intrusive ground improvement methods is included in Appendix F for reference purposes only.

For any stability and settlement mitigation option (i.e., adjustments to embankment geometry - flatter side slopes and/or berms, the use of lighter weight fill materials, preloading or intrusive ground improvement methods), due to the presence of organic and compressible soils beyond the current toe of the embankment, a settlement and porewater pressure monitoring plan must be carried out to measure the development/dissipation of porewater pressure conditions in the organic and cohesive soils at depth adjacent/beyond the toe of the section of widened embankment and monitor the settlement and deformation response of the widened embankment. Details of the monitoring plan are discussed further in Section 6.6.

6.3 Design Criteria

The embankment widening must be designed to satisfy the requirements of CHBDC (2019) for global stability and MTO's requirement for long-term settlement of widened embankments.

6.3.1 Global Stability Criteria

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 global stability analyses, the Factor of Safety is equal to the inverse of the product of the consequence factor, Ψ , and the geotechnical resistance factor, ϕ_{gu} . (i.e., $FoS = 1/(\Psi \cdot \phi_{gu})$) as defined in the Canadian Highway Bridge Design Code (CHBDC, 2019).

The minimum Factors of Safety listed below have been established as the target for the design of the embankment widening at this site, considering a “high degree of understanding” per Table 6.2 of CHBDC (2019).

- 1.25 for temporary (undrained) conditions; and
- 1.43 for long-term (drained) conditions.

A “high degree of understanding” is considered appropriate at this site given the number of boreholes, CPTs and test pits carried out within the limits of the embankment widening and the associated geotechnical laboratory testing program. The above Factors of Safety have been established for a consequence factor of 1.0.

6.3.2 Embankment Settlement Criteria

The performance of the widened embankment is also required to meet MTO’s “Embankment Settlement Criteria for Design” (MTO, 2010). These criteria consider three types of settlement of the native soils: elastic compression, primary compression and secondary consolidation (or creep). The total settlement is the sum of these three components for the embankment fill itself and the settlement of the subsurface soils. The elastic compression of the native soils occurs immediately upon construction of the widened embankment and primary and secondary consolidation are time-dependent and occur in cohesive deposits.

The total and differential settlement at the shoulder of the widened Highway 400 embankment must meet MTO’s criteria for widened embankments as specified below, for a period of 20 years:

	Maximum Limits During Pavement Design Life	
	Total Settlement (mm)	Differential Settlement Rate
Freeways	50	200:1

Total and differential settlements are defined relative to both the longitudinal profile of the traffic lanes of the roadway and transversely across the top of the roadway surface. Differential settlement between any two points is the primary concern when assessing embankment settlement performance. When differential settlements exceed limits as governed by the flexibility of the pavement structure, asphalt cracking and distortions will occur. Limits on differential settlement are required to ensure adequate transitions and to avoid unacceptable surface distortions.

6.4 Global Stability and Settlement

6.4.1 Parameter Selection

Global stability and settlement analysis were carried out at two critical sections: at Station 15+289 and Station 15+310. The interpreted stratigraphic cross-sections at these locations are shown on Drawings 3 and 4. For the purpose of the analyses, the groundwater level was assumed to be at ground surface at the toe of the existing embankment (i.e., at about Elevation 298.5 m).

The soil parameters used in the stability and settlement analyses were estimated from empirical correlations with the SPT “N”-values, field vanes, CPT data, laboratory test data and engineering judgement from experience with similar soils in this region of Ontario (Bowles, 1984; Kulhawy and Mayne, 1990; Peck et al., 1974). Table 2 following the text of this report presents the simplified soil stratigraphy at the critical sections and the parameters employed in the analyses. Plots supporting the selection of the geotechnical parameters are also shown graphically on Figures G-1 to G-5 in Appendix G.

Due to the staged construction further discussed in Section 6.4.2, two sets of B-Bar values were used to assess the temporary condition stability assessment in Stages B and C, as presented in Table 2. The first set of values was determined by adjusting the B-Bar values such that the Factor of Safety for the undrained condition is a similar value to the Factor of Safety for the B-Bar analysis for the same geometry. This first set of B-Bar values was applied to the compressible soils under new loading in each stage. The second set of B-Bar values used in Stage B and C was reduced to 25% of the first set of B-Bar values to account for 75% dissipation of porewater pressures in Stage A. The second set of B-Bar values was then applied to the compressible soils under the original loading condition only (i.e., Stage A only). The stability analyses comparing the two assessments are shown on Figures G-6 and G-7 in Appendix G to support the B-Bar value selection.

The coefficient of consolidation for the organic and cohesive soil deposits (2.9×10^{-3} m/s) was determined by averaging the coefficients of consolidation calculated from the laboratory oedometer tests and applying a factor of approximately 1.5 to account for a faster coefficient of consolidation in the field due to the presence of zones and seams of more permeable soils within the deposits that are not present within the laboratory specimens.

6.4.2 Global Stability

Two-dimensional limit equilibrium slope stability analyses for the widened embankment were carried out using Slide2 (Version 9.017), developed by Rocscience Inc., engaging the Morgenstern Price method of slope stability analysis. Factors of Safety of numerous potential failure surfaces were computed for the critical embankment cross-section to establish the minimum FoS for the embankment and subsoils.

The global stability analyses indicate that to achieve a minimum Factor of Safety equal to or greater than 1.25 in short-term (undrained) conditions, the approximately 10 m high embankment widening must be designed using the following materials, geometry and staging:

- The embankment widening (excluding the pavement structure) must be constructed of 3/8-inch Chip Stone, which shall have a maximum unit weight of 18 kN/m^3 . See Section 6.5.1 for additional information.
- The embankment must be constructed in three phases (Stage A, Stage B, and Stage C) to provide sufficient time between phases such that the porewater pressures may adequately dissipate prior to the full loading of the widened embankment. It is stressed that the duration between Stages A and B is a global stability mitigation measure required to achieve the minimum required Factor of Safety for global stability, while that between Stages B and C represents both a stability mitigation measure and a settlement mitigation measure to meet MTO's post-paving settlement performance criteria at the shoulder of the widened Highway 400 embankment.
- In Stage A, the lower portion of the embankment is to be constructed to a maximum height of 5 m, as measured from the toe of the embankment. This Stage A embankment must be constructed with side slopes oriented at 3.5H:1V.
- In Stage B, the upper portion of the embankment must be constructed to the top of the granular subbase course. This upper portion of the embankment must be constructed with side slopes oriented at 2H:1V. Construction of the Stage B embankment may not proceed until approximately 75% dissipation of excess porewater pressures has been achieved following placement of the Stage A embankment. It is estimated that the required dissipation of excess porewater pressures will be achieved within approximately 12 months from completion of construction of the Stage A embankment. This timeframe was based on the time for 75%

of consolidation, as determined from the rate of consolidation of the compressible soil deposits provided in Table 2, assuming two-way drainage and a deposit thickness of 9 m.

- In Stage C, the granular base materials and hot mix asphalt may be placed to final grade, with the side slopes through the pavement structure oriented at 3.5H:1V. This stage shall not proceed until 4 months following completion of Stage B embankment; as noted above, the duration between Stages B and C is also required for settlement mitigation, as discussed further in Section 6.4.3.

The results of the slope stability analyses at the two critical locations (i.e., at Stations 15+290 and 15+310) are summarized in the table below and presented on Figures H-1 to H-8 in Appendix H.

Critical Section	Minimum Factor of Safety for Global Stability			
	Stage A Temporary Condition (Undrained Analysis)	Stage B Temporary Condition (B-Bar Analysis)	Stage C Temporary Condition (B-Bar Analysis)	Stage C Long-term Condition (Drained Analysis)
Station 15+290	1.37	1.63	1.46	1.70
Station 15+310	1.30	1.29	1.25	1.50

6.4.3 Settlement

Settlement of the widened embankment will occur due to compression of the existing embankment fill and underlying native soils as well as within the new embankment fill in response to the increased load from the new widened embankment. Settlement of the compacted new embankment fill (3/8-inch Chip Stone) itself is expected to occur during embankment construction, assuming compaction to 98% of the material's Standard Proctor Maximum Dry Density (SPMDD) is achieved.

Settlement analyses for the soils below the widened embankments were carried out at the two critical sections using Settle3 from Rocscience, using consolidation settlement parameters for the organic and cohesive soils and elastic deformation moduli for non-cohesive soils as described in Section 6.4.1 and Table 2 following the text of this report.

The maximum total settlements estimated in the settlement analyses are presented in the table below for Stage A, Stage B, and Stage C at the two critical sections. It is noted that due to varying loading conditions and varying subsurface conditions along the widened highway profile, the maximum immediate settlement and the maximum consolidation settlement do not occur at the same point, and therefore the maximum total settlement provided below may not equal the sum of the immediate and consolidation settlements in the individual stages. In general, for this embankment widening, the maximum increase in loading will occur near the toe of the existing embankment where there will be approximately 6 m of new fill placed over the greatest thickness of compressible soils.

Maximum Total Settlement (mm) ¹		
Stage	Station 15+290	Station 15+310
Stage A	310	585
Stage B	95	110
Stage C	15	10
Total ¹	380	616

Note 1: The maximum settlements in each stage will occur at different locations along the widened embankment and therefore the total maximum settlements for each of the critical sections do not equal the sum of the settlements in each stage.

The graphical results of settlement analyses are presented in Appendix I, illustrating the estimated magnitudes of total, immediate (elastic) and primary consolidation settlements measured at various points across the widened highway section at Stations 15+290 and 15+310 during Stage A, Stage B, and Stage C. Further discussion and summaries of the estimated immediate elastic and primary consolidation settlement are provided in the sub-sections below.

6.4.3.1 Immediate Elastic Settlement

The total elastic (i.e., immediate) settlement of the non-cohesive subsurface soils over all three stages is estimated to be about 70 mm to 95 mm under the widened embankment loading. A detailed breakdown of the maximum elastic settlement estimated from each stage of embankment construction is provided in the table below and shown on Figures I-3 and I-4 in Appendix I.

Maximum Elastic Settlement (mm) ¹		
Stage	Station 15+290	Station 15+310
Stage A	45	35
Stage B	65	45
Stage C	10	5
Total	95	70

Note 1: The maximum settlements in each stage will occur at different locations along the widened embankment and therefore the total maximum settlements for each of the critical sections do not equal the sum of the settlements in each stage.

The elastic settlements are expected to occur during or shortly after construction in response to the placement of the new embankment fill.

6.4.3.2 Primary Consolidation Settlement

The total primary consolidation settlement of the cohesive subsurface soils (i.e., the organic deposit and the silty clay deposits) over all three stages is estimated to be approximately 320 mm to 590 mm under the footprint of the

embankment widening. A detailed breakdown of the maximum primary consolidation settlement estimated from each stage of embankment construction is provided in the table below and shown on Figures I-5 and I-6 in Appendix I.

Maximum Primary Consolidation Settlement (mm) ¹		
Stage	Station 15+290	Station 15+310
Stage A	265	550
Stage B	70	85
Stage C	5	5
Total ¹	320	590

Note 1: The maximum settlements in each stage will occur at different locations along the widened embankment and therefore the total maximum settlements for each of the critical sections do not equal the sum of the settlements in each stage.

Based on an estimated coefficient of consolidation (C_v) equal to $2.9 \times 10^{-3} \text{ cm}^2/\text{s}$ for the combined organic silt and silty clay deposits, and assuming two-way drainage of the approximately 9 m thick combined organic silt and silty clay deposits, it is estimated that about 90% of the primary consolidation settlement will be completed in approximately two years, after which approximately 30 mm to 60 mm of primary consolidation settlement will remain. However, the majority of this consolidation settlement will occur beyond the crest of the widened embankment, under the new embankment side slope.

The embankment widening will be complete approximately 1.5 years from commencement of the staged construction for the widening. Accordingly, the following table provides a summary of the remaining primary consolidation settlement after this 1.5-year staging period at various locations along the widened embankment, based on the coefficient of consolidation and the varying compressible deposit thicknesses along the widened embankment profile.

Remaining Primary Consolidation Settlement (mm) After Approximately 1.5-Year Staging Period		
Location	Station 15+290	Station 15+310
Crest of Existing Embankment	<5	<5
Crest of Widened Embankment	<5	<5
Toe of Existing Embankment	70	95
Crest of Embankment Bench	60	110
Toe of Widened Embankment	10	65

The table above indicates that after the approximately 1.5-year staging period, the remaining primary consolidation settlement below the road surface meets the design criteria outlined in Section 6.3.2 (i.e., total settlement is less than 50 mm and differential settlement is less than 200:1).

6.4.3.3 Secondary Creep Settlement

In addition to primary consolidation within the cohesive subsurface soils (i.e., the organic deposit and the silty clay deposits) at this site, secondary compression of the organic and silty clay deposits will also occur. Secondary compression is referred to as creep settlement and occurs over a long period of time, after substantial dissipation of excess pore pressure under a constant stress (load).

If preloading measures are implemented to achieve the majority of the primary consolidation settlement in advance of completion of the paving, it is estimated that for this embankment widening constructed using 3/8-inch Chip Stone, up to approximately 80 mm of creep settlement could be realized along the widened embankment profile, within 20 years. The following table provides a summary of the secondary consolidation settlement over a 20-year period at various locations along the widened embankment, based 90% of the primary consolidation settlement being completed in approximately two years and based on the varying compressible deposit thicknesses along the widened embankment profile.

Secondary Consolidation Settlement (mm) within 20 years		
Location	Station 15+290	Station 15+310
Crest of Existing Embankment	5	5
Crest of Widened Embankment	5	15
Toe of Existing Embankment	70	70
Crest of Embankment Bench	70	70
Toe of Widened Embankment	70	80

The table above indicates that the secondary consolidation (over a 20-year period) below the road surface meets the design criteria outlined in Section 6.3.2 (i.e., total settlement is less than 50 mm and differential settlement is less than 200:1).

6.5 Construction Considerations

6.5.1 Subgrade Preparation and Embankment Construction

All topsoil and surficial organic soil should be removed from below the footprint of the widening embankment. Prior to placement of new fill, a geotextile should be placed at the subgrade level within the widening footprint and extending 1 m beyond the footprint in all directions, to mitigate mixing of fine particle soils from the subgrade into the new embankment fill.

As noted in Section 6.4.2, the embankment widening must be constructed using 3/8-inch Chip Stone (which may also be referred to by suppliers as High Performance Bedding – HPB, or High Performance Chip Stone), which shall have a maximum unit weight of 18 kN/m³ in order to achieve the required Factors of Safety for global

stability. This embankment widening must be constructed in three stages (Stage A, Stage B, and Stage C) as summarized below, to provide sufficient time between phases such that the porewater pressures adequately dissipate to mitigate global instability and to meet MTO's post-paving settlement performance criteria at the shoulder of the widened Highway 400 embankment.

- **Stage A:** The lower portion of the embankment is to be constructed to a maximum height of 5 m, as measured from the toe of the embankment. This Stage A embankment must be constructed with side slopes oriented at 3.5H:1V.
- **Stage B:** The upper portion of the embankment must be constructed to the top of the granular subbase course. This upper portion of the embankment must be constructed with side slopes oriented at 2H:1V. Construction of the Stage B embankment may not proceed until approximately 75% dissipation of excess porewater pressures has been achieved following placement of the Stage A embankment. It is estimated that the required dissipation of excess porewater pressures will be achieved within approximately 12 months from completion of construction of the Stage A embankment, based on the time for 75% of consolidation to be completed.
- **Stage C:** The granular base materials and hot mix asphalt may be placed to final grade, with the side slopes through the pavement structure oriented at 3.5H:1V. This stage shall not proceed until four months following completion of Stage B embankment.

The material and construction specifications of the 3/8-inch Chip Stone are summarized in the Non-Standard Special Provision (NSSP) included in Appendix J. The requirements for the staged embankment construction are described in the Operational Constraint contained in Appendix J, for inclusion in the Contract Documents.

The new fill should be placed in accordance with OPSS.PROV 206 (*Grading*) and compacted in accordance with OPSS.PROV 501 (*Compacting*). The new fill should be benched into the existing embankment in accordance with OPSD 208.010 (*Benching of Earth Slopes*), and a minimum 2 m wide mid-height bench must be incorporated for embankment heights greater than 8 m on accordance with OPSD 202.010 (*Slope Flattening*). To reduce the potential for erosion of the embankment side slopes due to surface water run-off, placement of topsoil and seeding or pegged sod is recommended as soon as practicable after construction of the embankments, in accordance with OPSS.PROV 804 (*Seed and Cover*).

6.5.2 Groundwater and Surface Water Control

The groundwater level at the site is interpreted to be near the existing ground surface elevation at and beyond the existing embankment toe, particularly during spring conditions following snow melt and periods of heavy precipitation. Lower groundwater levels have been measured in the piezometers, suggesting that the groundwater/surface water level in this localized area may be lower in drier periods of the year. Considering only surficial soils will be stripped from the embankment widening footprint, it is anticipated that dewatering will not be required; however, wet / soft ground conditions are to be expected.

Surface water should be directed away from the embankment widening area. Any existing drainage paths / ditches will need to be diverted around the proposed widening footprint to reduce surface infiltration / seepage.

6.6 Monitoring

As discussed in Section 6.4.2 and 6.5.1, in order to achieve the minimum required Factor of Safety for global embankment stability in undrained (short-term) conditions, the embankment widening must be constructed in

stages and Stage B may not proceed until approximately 75% dissipation of excess porewater pressures has been achieved following placement of the lower portion of the embankment in Stage A. It is estimated that the required dissipation of excess porewater pressures will be achieved approximately twelve months from completion of construction of the Stage A embankment. Further, Stage C shall not proceed until four months following completion of the Stage B embankment, subject to settlement monitoring.

A monitoring program has been developed to measure porewater pressures and settlement during and following construction of the embankment widening. The monitoring program consists of vibrating wire piezometers (VWPs) to monitoring groundwater levels and porewater pressures within the widening area, settlement plates (SPs) to measure the settlement profile along the widening embankment, and temporary benchmarks for control reference points.

An NSSP for the Supply and Installation of Settlement Monitoring Instrumentation is presented in Appendix J, for inclusion in the Contract Documents. The Settlement Monitoring Plan showing the location of the monitoring points and details of the instrument installation is presented on Drawing 4.

The monitoring points should be installed and baselined prior to placement of fill. The monitoring frequency of the VWPs and SPs shall be as follows:

- Upon completion of Stage A embankment: Weekly for two months, biweekly for four months, and monthly for six months or until approximately 75% dissipation of porewater pressures.
- Upon completion of Stage B embankment: Weekly for two months and biweekly for two months, or until approximately 75% dissipation of porewater pressures and less than 15 mm of remaining settlement at the widened crest of the highway embankment.
- Upon completion of Stage C embankment: Biweekly for two months.

7.0 CLOSURE

This Foundation Design Report was prepared by Anastasia Poliacik, P.Eng. a Senior Geotechnical / Foundation Engineer with Golder. Lisa Coyne, P.Eng., an MTO Foundation Designated Contact and Principal at Golder, conducted an independent technical and quality review of the report.

Signature Page

GOLDER ASSOCIATES LTD.



An Poliacik

Anastasia Poliacik, P.Eng.
Senior Geotechnical Engineer



L. C. Coyne

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

AMP/LCC/ljv

Golder, Golder Associates and the GA globe design are trademarks of Golder Associates Corporation.

[https://golderassociates.sharepoint.com/sites/21998g/deliverables/wo15-hwy 400 widening lloydton/8. high fill/5. final/1786658-wo15 rev0 2022'07'11 fidr-hwy 400 widening-lloydton-high fill.docx](https://golderassociates.sharepoint.com/sites/21998g/deliverables/wo15-hwy400wideninglloydton/8.highfill/5.final/1786658-wo15rev020220711fidr-hwy400widening-lloydton-highfill.docx)

REFERENCES

- Bowles, J.E., 1984. *Physical and Geotechnical Properties of Soils*, Second Edition. McGraw Hill Book Company, New York.
- Canadian Geotechnical Society, 2006. *Canadian Foundation Engineering Manual*, 4th Edition. The Canadian Geotechnical Society, BiTech Publisher Ltd., British Columbia.
- Canadian Standards Association. 2019. Canadian Highway Bridge Design Code (CHBDC) and Commentary on CAN/CSA-S6-14. CSA Special Publication, S6.1 14.
- Chapman, L.J., and Putnam, D.F., 1984. *The Physiography of Southern Ontario*, 3rd Edition. Ontario Geological Survey, Special Volume 2. Ontario Ministry of Natural Resources.
- Golder Associates, "Foundation Investigation and Design Report, High Fill and Deep Cut Embankment Widening Highway 400 Widening from North of King Road to South Canal Bank Road, King City, Ontario, GWP 2835-02-00", Geocres No. 30M13-217, Report No 09-1111-0018-12, dated March 4, 2016.
- 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.
- Ministry of the Environment, March 2004. "Soil, Ground Water and Sediment Standards for Use Under Part XV.1 of the Environmental Protection Act (MOE, March 2004)".
- Ministry of Transportation Ontario, July 2, 2010 "Embankment Settlement Criteria For Design"
- O'Rourke T.D. and McGinn A.J., 2004, GeoTrans, "Case History of Deep Mixing Soil Stabilization for Boston Central Artery"

ASTM International:

- | | |
|------------|---|
| ASTM D1586 | Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils |
| ASTM D2573 | Standard Test Method for Field Vane Test in Cohesive Soil. |

Contract Design Estimating and Documentation (CDED)

Ontario Occupational Health and Safety Act:

- Ontario Regulation 213/91 Construction Projects
 Ontario Regulation 443/09 Wells, Amendment to Ontario Regulation 213

Ontario Provincial Standard Drawings (OPSD)

- OPSD 202.010 Slope Flattening Using Surplus Excavated Material on Earth of Rock Embankment
 OPSD 208.010 Benching of Earth Slopes

Ontario Provincial Standard Specifications (OPSS)

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

Ontario Water Resources Act:

- Ontario Regulation 372/97 Amendment to Ontario Regulation 903/90, Wells

Table 1 – Comparison of Stability and Settlement Mitigation Measures

Ground Improvement Method (Feasibility)	Advantages	Disadvantages	Constructability/ Risks	Estimated Costs
Construct widened embankment using lighter weight fill materials in stages with monitoring (Feasible and preferred based on anticipated timing for property acquisition and construction staging)	<ul style="list-style-type: none">• Lower cost compared to subexcavation and likely compared to intrusive ground improvement options• Minimizes subexcavation and generation of excess soils for off-site disposal, compared with a full subexcavation option• Dewatering not expected to be required• Can be designed to fit within the construction schedule, based on acquisition of adjacent property by Spring 2023 to allow first stage of filling, completion of second stage of filling in Spring 2024, and paving by Summer/Fall 2024	<ul style="list-style-type: none">• Requires an initial loading stage of approximately 12 months to maintain minimum Factors of Safety for global stability, followed by a second stage of filling with a four-month preloading period, for the selected fill material and embankment geometry	<ul style="list-style-type: none">• Faster initial construction of highway embankment widening using conventional filling and compaction techniques• Magnitude and duration of settlement to be monitored during the staging/preloading to confirm porewater pressure dissipation/global stability and settlement performance• Compressibility / decomposition of organic deposits / interlayers may result in greater settlement than estimated in some localized areas, although this will generally occur under the widened embankment side slope and not be reflected at the widened highway shoulder.• Some potential for long-term differential settlement between existing embankment and widened embankment section associated with creep settlement under the new widening, although this will generally occur under the widened embankment side slope.	<ul style="list-style-type: none">• Lower cost
Subexcavation of weak cohesive and organic soils (Feasible but not practicable adjacent to 400-series highway)	<ul style="list-style-type: none">• None	<ul style="list-style-type: none">• Significant excavation required adjacent to Highway 400 to remove sufficient weak soils to improve factor of safety for global stability; extensive protection systems would be required	<ul style="list-style-type: none">• Significant protection system required to permit depth of subexcavation required to achieve minimum Factors of Safety for global stability (shallow subexcavation is not sufficient), although this can be designed to address required performance level per OPSS.PROV 539 to minimize impacts on existing highway	<ul style="list-style-type: none">• Likely highest cost based on requirements for protection systems, excess soils management and importation of backfill
Controlled Modulus Column (Feasible)	<ul style="list-style-type: none">• Can be used in various soil conditions• Dewatering is not required• Required equipment is readily available in Ontario• Installation procedure (i.e., reverse flight augers) may be best for removing tree branches / limbs should they be encountered• No spoils generated• Surrounding soil is densified during installation, therefore soils beyond the column area are improved• Uses grout to backfill the columns; therefore, higher strength and stiffness compared to the stone columns and deep soil mixing	<ul style="list-style-type: none">• Require space to set up the grout mixing plant• Additional environmental considerations/control measures considering the proximity of grout mixing to a neighbouring wetland area at the site• Due to the depth to bearing stratum the design of the ground improvement (i.e. spacing and diameter of columns and strength of grout) will need to take into account the loading from the embankment and the passive resistance provided by the site soils beyond the ground improvement to ensure that the columns do not bulge at depth	<ul style="list-style-type: none">• Some site grading to access the site will be required;• Requires a pump, water supply and concrete supply in addition to the rig; site logistics and access could be more challenging compared to vibro-replacement	<ul style="list-style-type: none">• Estimated to be higher than staged construction with lighter weight fill materials, given availability of time in construction schedule
Stone Columns (Vibro-Replacement) (Feasible)	<ul style="list-style-type: none">• Can be used in various soil conditions• Dewatering is not required• Does not generate spoils• Larger replacement diameter compared to controlled modulus column	<ul style="list-style-type: none">• The vibrations may have environmental effects on the neighbouring wetland area• Lower strength and stiffness gains compared to the controlled modulus columns• Similar to Controlled Column Modulus the stiffness of the columns and the passive resistance pressure of the	<ul style="list-style-type: none">• Some site grading to access the site will be required	<ul style="list-style-type: none">• Estimated to be higher than staged construction with lighter weight fill

Ground Improvement Method (Feasibility)	Advantages	Disadvantages	Constructability/ Risks	Estimated Costs
	<ul style="list-style-type: none">• Vibro-probe may be able to displace an obstruction such as a tree branch / limb should it be encountered• Concrete is not used; therefore, in contrast with controlled concrete modulus or deep soil mixing, curing time is not required	surrounding soil needs to be considered when designing the ground improvement		materials, given availability of time in construction schedule
Deep Soil Mixing (Dry) (Feasible)	<ul style="list-style-type: none">• Can be used in various soil conditions• Dewatering of the site is not required• No excess spoils generated with dry soil mixing• Can treat the entire footprint of the widening with deep soil mixing as opposed to columns; therefore, the entire site area would be improved, reducing the concern with bulging columns due to lack of restraining pressure in the soils outside the ground improvement	<ul style="list-style-type: none">• Requires space to set up the cement mixing plant• There may be difficulties in achieving a uniform soil mixture along the entire column• With wet mixing an area for mixing of the slurry is required; the area to the east of the site is a wetland and may require some protection prior to placement of construction equipment;• Depending on the time of year of construction the water level may be near ground surface and may require additional preparation prior to placement of construction equipment	<ul style="list-style-type: none">• Some site grading to access the site will be required• Requires a batch plant, water supply and concrete supply in addition to the rig; therefore, site logistics and access could be more challenging compared to vibro-replacement	<ul style="list-style-type: none">• \$1.2 Million

Table 2 – Soil Parameters Used in Global Stability and Settlement Analyses

Soil Description	Bulk Unit Weight (kN/m³)	Stability Stage A Temporary Condition (Undrained Analysis)		Stability Stage B and C Temporary Condition (B-Bar Analysis)			Stability Stage 2 Long-Term Condition (Drained Analysis)		Settlement Analysis					
		Undrained Shear Strength (kPa)	Effective Friction Angle (degrees)	Undrained Shear Strength (kPa)	Effective Friction Angle (degrees)	B-Bar	Undrained Shear Strength (kPa)	Effective Friction Angle (degrees)	Elastic Modulus (MPa)	OCR	e _o	C _c	C _r	C _v
New Fill (Stage C)	21	0	35	0	35	0	0	35	NA	NA	NA	NA	NA	NA
New Fill (Stage A and B)	18	0	35	0	35	0	0	35	NA	NA	NA	NA	NA	NA
Existing embankment fill	20	0	33	0	33	0	0	33	20	NA	NA	NA	NA	NA
Upper non-cohesive deposit - very loose to compact sandy silt to sand	20	0	32	0	32	0	0	32	10	NA	NA	NA	NA	NA
Upper cohesive deposit – soft to stiff silty clay	17.5	25	0	1	27	0.7 (0.18) ²	0	27	NA	1	1.26	0.33	0.04	2.9x10 ⁻³
Upper portion of organic deposit - very soft to soft organic silt to clayey organic silt	17	20	0	0	24	0.85 (0.21) ²	0	24	NA	1	1.26	0.33	0.04	2.9x10 ⁻³
Lower portion of organic deposit - very soft to soft organic silt to clayey organic silt	17	25	0	0	24	0.85 (0.21) ²	0	24	NA	1	1.26	0.33	0.04	2.9x10 ⁻³
Lower cohesive deposit - very soft to firm silty clay	17.5	30	0	0	27	1 (0.25) ²	0	27	NA	1	1.26	0.33	0.04	2.9x10 ⁻³
Lower non-cohesive deposit – compact to dense silty sand to sand	20	0	33	0	33	0	0	33	30	NA	NA	NA	NA	NA

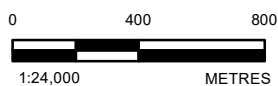
Note:

1. The B-bar values presented in parenthesis represent 25% of the original B-bar value, representative of 75% dissipation of porewater pressures assumed at 75% consolidation. These lower B-bar values are used under Stage 1 of the new embankment for application of Stage 2 loading which is estimated to occur at 75% consolidation.



LEGEND

SITE



REFERENCE(S)
BASE DATA - MNR LIO, OBTAINED 2016; ESRI, HERE, DELORME, TOMTOM, INTERMAP, INCREMENT P CORP., GEBCO, USGS, FAO, NPS, NRCAN, GEOBASE, IGN, KADASTER NL, ORDNANCE SURVEY, ESRI JAPAN, METI, ESRI CHINA (HONG KONG), SWISSSTOPO, MAPMYINDIA, © OPENSTREETMAP CONTRIBUTORS, AND THE GIS USER COMMUNITY
IMAGERY - © THE REGIONAL MUNICIPALITY OF YORK (2017)
PRODUCED BY GOLDER ASSOCIATES LTD UNDER LICENCE FROM
ONTARIO MINISTRY OF NATURAL RESOURCES, © QUEENS PRINTER 2017
PROJECTION: TRANSVERSE MERCATOR DATUM: NAD 83 COORDINATE SYSTEM: UTM ZONE 17

CLIENT
MINISTRY OF TRANSPORTATION, ONTARIO

PROJECT
HIGHWAY 400 WIDENING FROM KING ROAD TO LLOYDTOWN-
AURORA ROAD, GWP 2835-02-00

TITLE
EMBANKMENT WIDENING, STATION 15+260 TO 15+360

CONSULTANT	YYYY-MM-DD	2022-03-18
	DESIGNED	SO
	PREPARED	DB
	REVIEWED	AMP
	APPROVED	LCC



PROJECT NO.	CONTROL	REV.	FIGURE
1786658-WO15-GI		A	1

S:\Golder\1786658-WO15-GI\1786658-WO15-GI\1786658-0002-BG-0001.mxd

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: 25mm

CONT No. 2022-2017
GWP No. 2835-02-00



HIGHWAY 400 EMBANKMENT WIDENING
STATIONS 15+260 TO 15+360 (NBL)

BOREHOLE LOCATIONS







KEY PLAN
SCALE



2 0 2 4 km

LEGEND

-  Borehole – Current Investigation
-  Borehole – Previous Investigation
-  CPTU Locations
-  Test Pit Locations

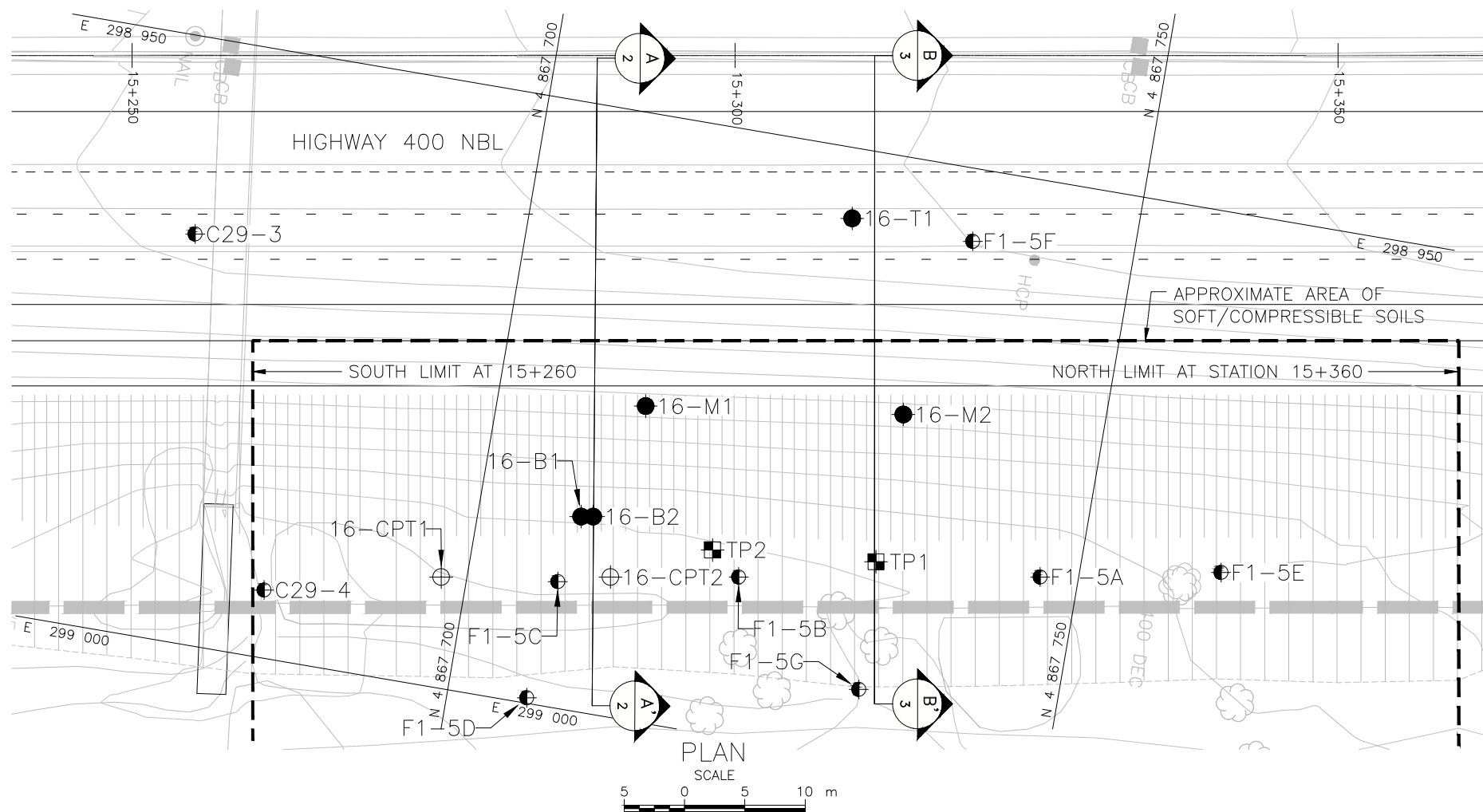
BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
16-B1	300.4	4867708.5	298984.0
16-B2	300.4	4867709.5	298983.8
16-CPT1	298.7	4867697.9	298990.9
16-CPT2	299.9	4867711.7	298988.5
16-M1	302.9	4867712.2	298974.0
16-M2	303.3	4867733.4	298971.1
16-T1	309.7	4867726.5	298955.8
C29-3	308.2	4867673.0	298966.3
C29-4	297.5	4867683.6	298994.4
F1-5A	299.5	4867746.9	298982.5
F1-5B	299.1	4867722.2	298986.7
F1-5C	298.6	4867707.5	298989.6
F1-5D	298.5	4867706.6	298999.5
F1-5E	300.5	4867761.6	298979.6
F1-5F	310.0	4867736.7	298956.0
F1-5G	298.9	4867733.6	298994.2
TP1	299.2	4867733.2	298983.5
TP2	298.8	4867719.7	298984.9

REFERENCE

Base plans provided in digital format by URS, drawing files no. Hwy400_bgd.dwg and Hwy400_plan.dwg received October 17, 2011.

NO.	DATE	BY	REVISION		
Geocres No. 30M13-299					
HWY. 400			PROJECT NO. 1786658-WO15		DIST. CENTRAL
SUBM'D. ARV	CHKD. ARV	DATE: 03/17/2022		SITE:	
DRAWN: MR	CHKD. AMP	APPD. LCC		DWG. 1	



NOTES

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

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

The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.



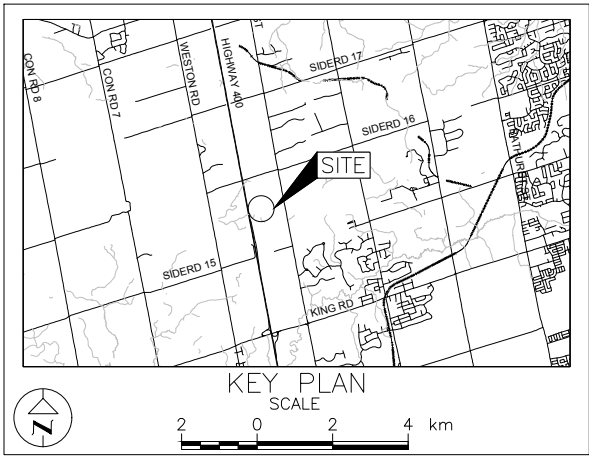
METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No. 2022-2017
GWP No. 2835-02-00

HIGHWAY 400 EMBANKMENT WIDENING
STATIONS 15+260 TO 15+360 (NBL)

SOIL STRATA

SHEET



- LEGEND
- Borehole - Current Investigation
 - Borehole - Previous Investigation
 - CPTU Locations
 - Test Pit Locations
 - Seal
 - Piezometer
 - N Standard Penetration Test Value
 - 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
 - WL in piezometer, measured on Aug. 15, 2016

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
16-B1	300.4	4867708.5	298984.0
16-M1	302.9	4867712.2	298974.0
16-T1	309.7	4867726.5	298955.8
F1-5C	298.6	4867707.5	298989.6
F1-5D	298.5	4867706.6	298999.5
TP2	298.8	4867719.7	298984.9

NOTES

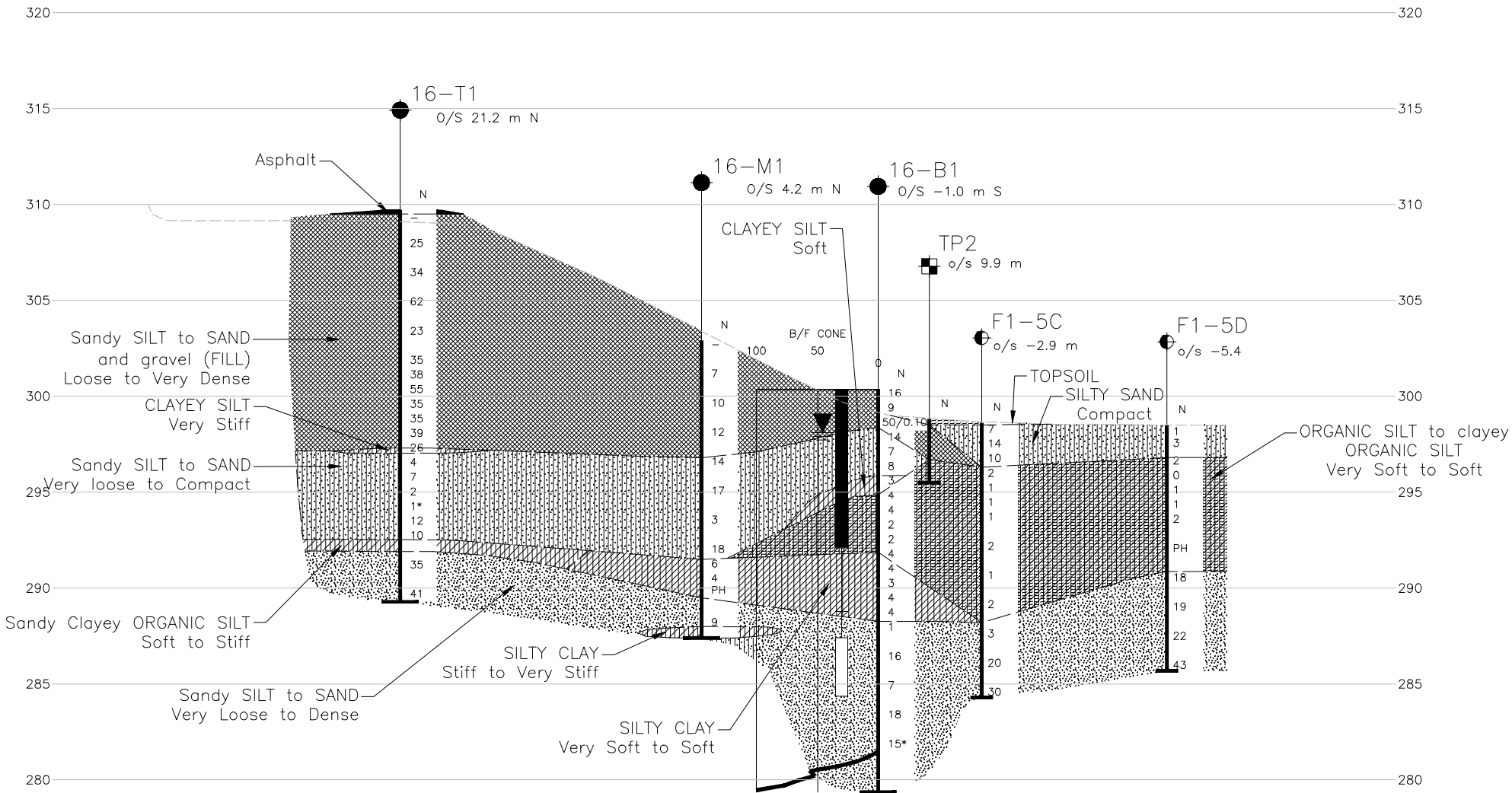
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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

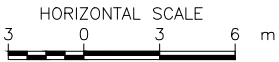
The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

REFERENCE

Base plans provided in digital format by URS, drawing files no. Hwy400_bgd.dwg and Hwy400_plan.dwg received October 17, 2011.



A-A HIGH FILL EMBANKMENT - NBL CROSS-SECTION
(STATION 15+289)

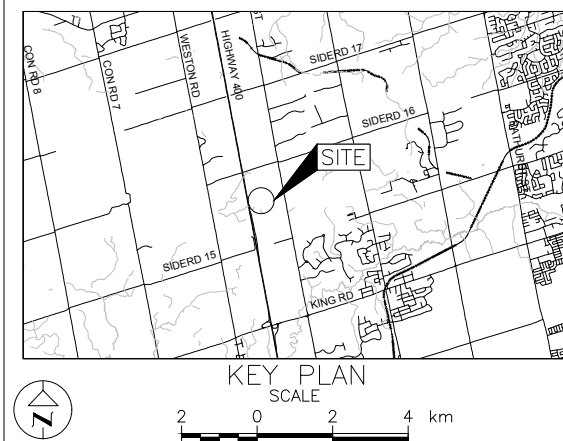
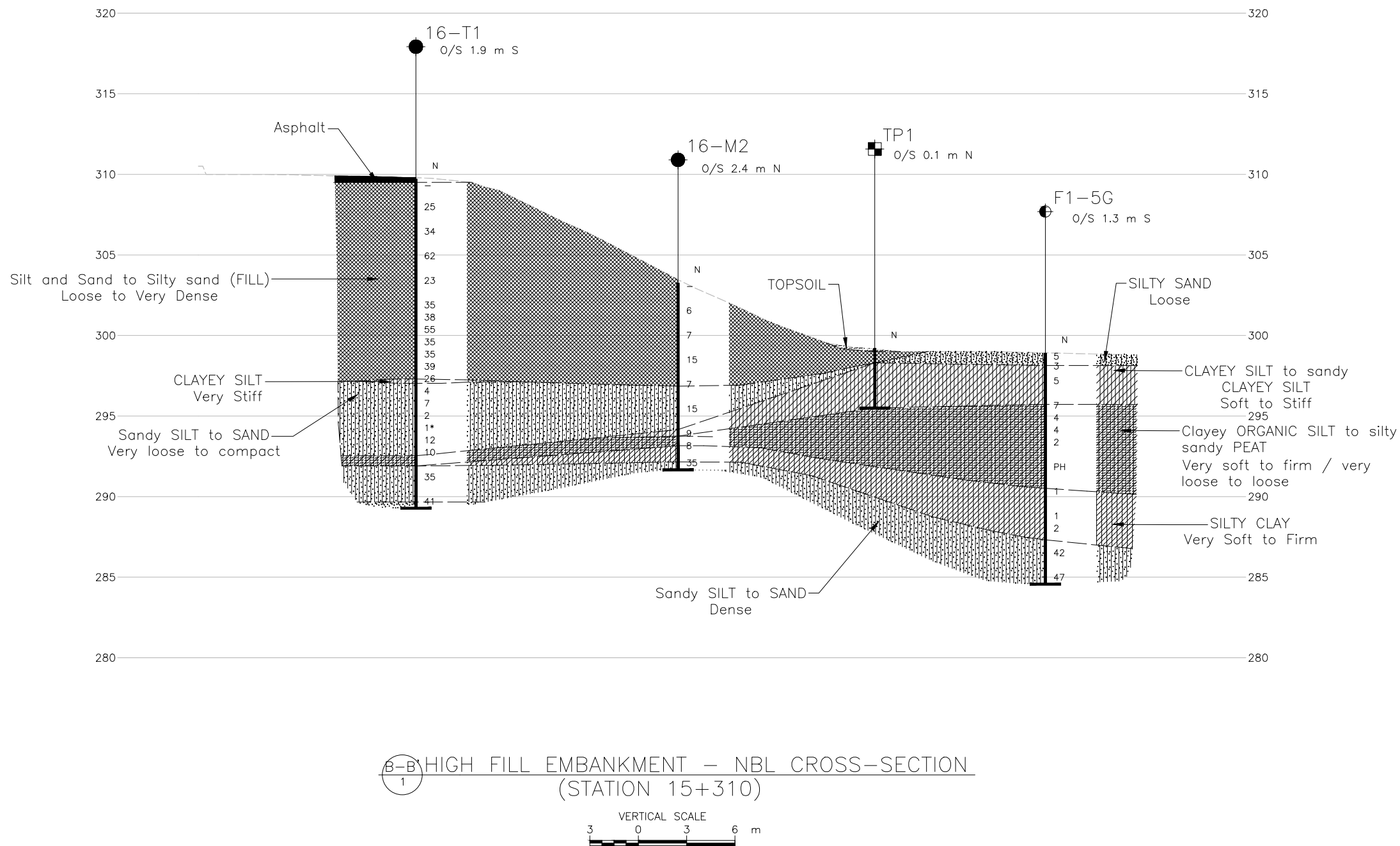


NO.	DATE	BY	REVISION
Geocres No. 30M13-299			
HWY. 400	PROJECT NO. 1786658-W015		DIST. CENTRAL
SUBM'D. ARV	CHKD. ARV	DATE: 03/17/2022	SITE:
DRAWN: MR	CHKD. AMP	APPD. LCC	DWG. 2




CONT No. 2022-2017
GWP No. 2835-02-00

HIGHWAY 400 EMBANKMENT WIDENING
STATIONS 15+260 TO 15+360 (NBL)
SOIL STRATA

SHEET



LEGEND

-  Borehole – Current Investigation
 Borehole – Previous Investigation
 Test Pit Locations
 N Standard Penetration Test Value
 16 Blows/0.3m unless otherwise stated
 (Std. Pen. Test, 475 j/blow)

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
16-M2	303.3	4867733.4	298971.1
16-T1	309.7	4867726.5	298955.8
F1-5G	298.9	4867733.6	298994.2
TP1	299.2	4867733.2	298993.5

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.

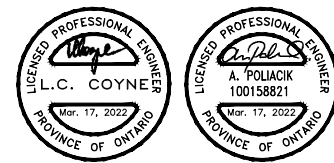
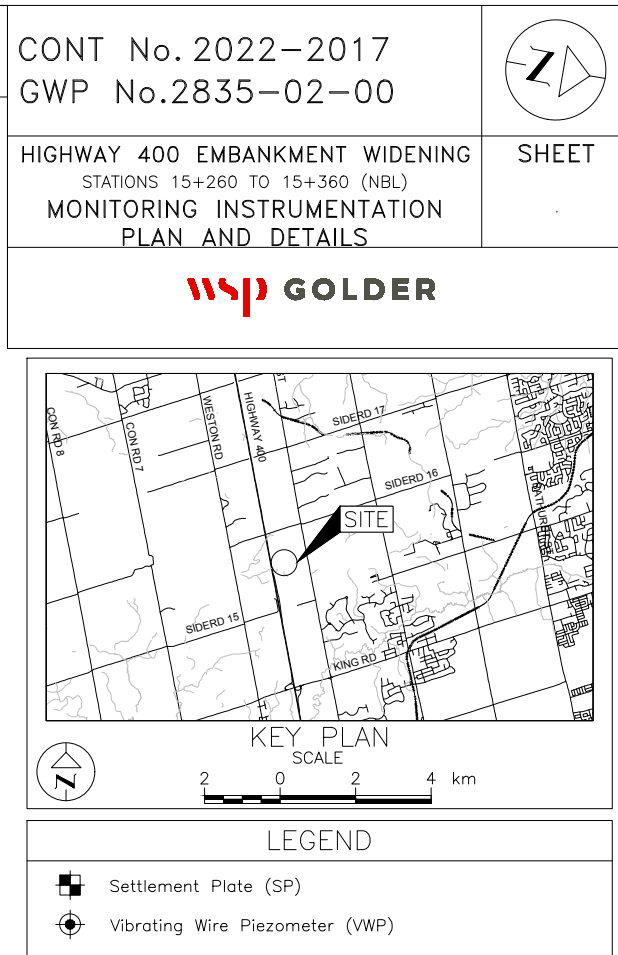
The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

REFERENCE

Base plans provided in digital format by URS, drawing files no.
Hwy400_bqd.dwg and Hwy400_plan.dwg received October 17, 2011.



NO.		DATE		BY	
				REVISION	
Geocres No. 30M13-299					
HWY. 400		PROJECT NO. 1786658-WO15		DIST. CENTRAL	
SUBM'D. ARV		CHKD. ARW		DATE: 03/17/2022	
DRAWN: MR		CHKD. AMP		APPD. LLC	
				DWG. 3	



NOTES

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.

REFERENCE

Base plans provided in digital format by URS, drawing files no.
Hwy400_bgd.dwg and Hwy400_plan.dwg received October 17, 2011.

.
NO.	DATE	BY	REVISION						
Geocres No. 30M13-299									
HWY. 400			PROJECT NO. 1786658--W01S					DIST. CENTRAL	
SUBM'D. SMM		CHKD. AMP		DATE: 03/17/2022				SITE: .	
DRAWN: MR		CHKD. AMP		APPD. LCC				DWG. 4	

APPENDIX A

**2010-2015 Investigations –
Borehole Records and Laboratory
Results**

PROJECT 09-1111-0018		RECORD OF BOREHOLE No C29-3		SHEET 1 OF 2		METRIC	
G.W.P. 2835-02-00		LOCATION N 4867673.0 ; E 298966.3		ORIGINATED BY TT			
DIST Central HWY 400		BOREHOLE TYPE D-90 Truck Mount, 108 mm Inside Diameter Hollow Stem Augers		COMPILED BY SKB/HS			
DATUM Geodetic		DATE November 8 and 9, 2010		CHECKED BY SMM			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)							
								20 40 60 80 100		20 40 60 80 100		10 20 30								
308.2	GROUND SURFACE																			
0.0	ASPHALT																			
0.2	Clayey silt, some sand, trace gravel, sand zones (FILL) Stiff Brown Moist		1	SS	9															
307.0																				
1.2	Sand and silt to silty sand, trace to some gravel, trace clay, zones of clayey silt to a depth of 5.6 m (FILL) Compact to very dense Brown Moist		2	SS	23															
			3	SS	33															
			4	SS	32															
			5	SS	56															
			6	SS	34															
			7	SS	32															
			8	SS	47															
299.5																				
8.7	SAND, some silt, trace clay, trace gravel Compact Brown and grey to brown below 9.8 m Moist to wet below 10.7 m Interlayer of clayey silt between depths of 9.5 m and 9.8 m		9	SS	27															
			10	SS	21															
296.5																				
11.7	CLAYEY SILT, trace to some sand, trace gravel (TILL) Stiff to very stiff Brown and grey to grey below 13.7 m Moist to wet below 15.2 m		11	SS	13															
			12	SS	17															

Continued Next Page

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

GTA-MTO 001 T:\PROJECTS\2009\09-1111-0018 (URS, YORK REGION)\LOG\0911110018.GPJ GAL-GTA.GDT 11/30/15 SIB

PROJECT <u>09-1111-0018</u>		RECORD OF BOREHOLE No C29-3		SHEET 2 OF 2		METRIC	
G.W.P. <u>2835-02-00</u>		LOCATION <u>N 4867673.0 ; E 298966.3</u>		ORIGINATED BY <u>TT</u>			
DIST <u>Central</u> HWY <u>400</u>		BOREHOLE TYPE <u>D-90 Truck Mount, 108 mm Inside Diameter Hollow Stem Augers</u>		COMPILED BY <u>SKB/HS</u>			
DATUM <u>Geodetic</u>		DATE <u>November 8 and 9, 2010</u>		CHECKED BY <u>SMM</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W _p	W	W _L		GR	SA	SI	CL
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× REMOULDED	WATER CONTENT (%)								
	--- CONTINUED FROM PREVIOUS PAGE ---																			
	CLAYEY SILT, trace to some sand, trace gravel (TILL) Stiff to very stiff Brown and grey to grey below 13.7 m Moist to wet below 15.2 m		13	SS	19															
			14	SS	20															
290.4 17.8	Sandy CLAYEY SILT Stiff Grey Wet																			
289.3 18.9	END OF BOREHOLE																			
	NOTE: 1. Water level in open borehole at a depth of 5.8 m (Elev. 302.4 m) upon completion of drilling.																			

PROJECT 09-1111-0018		RECORD OF BOREHOLE No C29-4		SHEET 1 OF 1		METRIC												
W.P. 2835-02-00		LOCATION N 4867683.6 ; E 298994.4		ORIGINATED BY CS														
DIST Central HWY 400		BOREHOLE TYPE D-50 Track Mount, 108 mm Diameter Solid Stem Augers		COMPILED BY SKB/HS														
DATUM Geodetic		DATE November 5, 2010		CHECKED BY SMM														
SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER			TYPE	"N" VALUES						SHEAR STRENGTH kPa		WATER CONTENT (%)			
297.5	GROUND SURFACE					20	40	60	80	100								
0.0	Sandy CLAYEY SILT, trace gravel, trace rootlets Very soft Grey Wet Interlayer of sand between depths of 0.7 m and 1.0 m		1	SS	1/0.61													
			2	SS	1/0.4													
			3	SS	1													
295.3																		
2.2	SAND, some silt, trace gravel, trace clay Loose to compact Grey Wet		4	SS	10													
			5	SS	10													
			6	SS	7													
			7	SS	13													
291.9																		
5.6	CLAYEY SILT, some sand, trace gravel (TILL) Stiff to very stiff Grey Wet		8	SS	13													
			9	SS	27													
			10	SS	29													
286.8																		
10.7	CLAYEY SILT, trace sand Very stiff Grey Wet		11	SS	26													
286.2																		
11.3	END OF BOREHOLE																	
	NOTE: 1. Water level in open borehole at a depth of 0.2 m (Elev. 297.3 m) upon completion of drilling.																	

PROJECT 09-1111-0018		RECORD OF BOREHOLE No F1-5A		SHEET 1 OF 1		METRIC	
W.P. 2835-02-00		LOCATION N 4867746.9 ; E 298982.5		ORIGINATED BY CS			
DIST Central HWY 400		BOREHOLE TYPE D-50 Track Mount, 108 mm Outside Diameter Solid Stem Auger		COMPILED BY SKB			
DATUM Geodetic		DATE November 8 and 9, 2010		CHECKED BY SMM			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT CONTENT CONTENT W _P W W _L			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	20 40 60 80 100	10 20 30				
299.5	GROUND SURFACE													
0.0	TOPSOIL													
0.2	Sandy CLAYEY SILT, trace gravel, trace rootlets Soft to stiff Brown and grey Wet Clay seams between depths of 0.7 m and 1.5 m		1	SS	4									
			2	SS	2									
			3	SS	14									
	Wood fragments at a depth of 2.2 m		4	SS	3									
296.5	ORGANIC SILT, trace sand, trace wood fragments Loose Grey Wet		5	SS	7									
295.8	SILTY PEAT, some sand, trace clay Very loose Brown Moist to wet		6	SS	3									
			7	SS	3									
293.9	SILT and SAND, trace to some clay Loose Grey Wet		8	SS	4									
292.3	SAND, some gravel to sand and gravel, trace to some silt, trace clay Compact to dense Grey Wet Blowing sands at 7.6 m depth from filling the augers		9	SS	17									
			10	SS	15									
			11	SS	38									
287.8	Silty SAND, trace gravel, trace clay Compact Grey Wet		12	SS	27									
286.7	END OF BOREHOLE													
12.8	NOTES: 1. Water level in open borehole at a depth of 0.4 m below ground surface (Elev. 299.1 m) upon completion of drilling. 2. At 7.6 m the drilling method was switched from Solid Stem Auger to Hollow Stem Auger. Water was used to prevent blowing sands from filling the augers.													

GTA-MTO 001 T:\PROJECTS\2009\09-1111-0018 (URS, YORK REGION)\LOG\0911110018.GPJ GAL-GTA.GDT 01/13/16 SIB

PROJECT 09-1111-0018		RECORD OF BOREHOLE No F1-5B				SHEET 1 OF 2		METRIC			
W.P. 2835-02-00		LOCATION N 4867722.2 ; E 298986.7				ORIGINATED BY CS					
DIST Central HWY 400		BOREHOLE TYPE D-50 Track Mount, 108 mm Inside Diameter Hollow Stem Auger				COMPILED BY SKB					
DATUM Geodetic		DATE November 9, 2010				CHECKED BY SMM					
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		SHEAR STRENGTH kPa				
							20 40 60 80 100			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W _p W W _L	
							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED			WATER CONTENT (%) 10 20 30	
							20 40 60 80 100				
299.1	GROUND SURFACE										
0.9	TOPSOIL										
298.4	CLAYEY SILT, some sand, trace rootlets		1	SS	3						
0.7	Soft Grey Wet										
	Silty SAND, some clay, trace gravel, shell fragments, trace organics		2	SS	7						1 66 26 7
297.3	Very loose to loose Grey Wet		3	SS	3						
1.8	Organic SILTY CLAY, trace sand		4	SS	3						
296.7	Soft Grey Wet										
2.4	SILTY SANDY PEAT, trace clay, trace rootlets and wood fragments										
	Very loose Black Wet		5	SS	2						WC=320.4% OC=67.9
			6	SS	2						
			7	SS	2						OC=54.1
293.8	ORGANIC Sandy SILT, trace rootlets and wood fragments		8	SS	WH						WC=134.2% PL=62.3% LL=82.5%
5.3	Very loose Black Wet		9	SS	1						
			10	SS	WH						WC=106.7% PL=59.2% LL=82.1%
291.6	SILTY CLAY, silt seams, trace organics		11	SS	WH						
7.5	Very soft Grey Wet		12	TO	-						OC=3.5 WC=42.4 PL=22.3 LL=42.8
			13	SS	1						
			14	SS	WH						WC=45.9% PL=25.5% LL=45.9%
286.6	Organic SILT, some clay, trace sand, shell fragments		15A	SS	2						WC=54.5% PL=37.8% LL=53.3%
12.5	Very loose Grey Wet		15B								
285.7	SAND, some gravel, some silt, trace clay										
13.4	Loose Grey Wet		16	SS	8						14 71 11 4
284.8	Dynamic Core Penetration Test										
14.3											

Continued Next Page

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

GTA-MTO 001 T:\PROJECTS\2009\09-1111-0018 (URS, YORK REGION)\LOG\0911110018.GPJ GAL-GTA.GDT 01/13/16 SIB

PROJECT		2835-02-00		LOCATION		N 4867722.2 ; E 298986.7		ORIGINATED BY		CS																						
DIST		Central HWY 400		BOREHOLE TYPE		D-50 Track Mount, 108 mm Inside Diameter Hollow Stem Auger		COMPILED BY		SKB																						
DATUM		Geodetic		DATE		November 9, 2010		CHECKED BY		SMM																						
SOIL PROFILE				SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)														
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	20			40	60	80	100	20						40	60	80	100	10	20	30	GR	SA	SI	CL			
282.6	Dynamic Core Penetration Test						284																									
16.5	END OF BOREHOLE						283																									
NOTES: 1. Water level in open borehole at a depth of 1.5 m below ground surface (Elev. 297.6 m) upon completion of drilling. 2. Water level measurements in piezometer: <table border="1" style="margin-left: 40px;"> <thead> <tr> <th>Date</th> <th>Depth (m)</th> <th>Elev. (m)</th> </tr> </thead> <tbody> <tr> <td>11/10/11</td> <td>5.2</td> <td>293.9</td> </tr> <tr> <td>11/11/11</td> <td>2.3</td> <td>296.8</td> </tr> </tbody> </table>		Date	Depth (m)	Elev. (m)	11/10/11	5.2	293.9	11/11/11	2.3	296.8																						
Date	Depth (m)	Elev. (m)																														
11/10/11	5.2	293.9																														
11/11/11	2.3	296.8																														

T:\PROJECTS\2009\09-1111-0018 (URS, YORK REGION)\LOG\0911110018.GPJ GAL-GTA.GDT 01/13/16 SIB

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

PROJECT 09-1111-0018		RECORD OF BOREHOLE No F1-5C				SHEET 2 OF 2		METRIC									
W.P. 2835-02-00		LOCATION N 4867707.5 ; E 298989.6				ORIGINATED BY TWB											
DIST Central HWY 400		BOREHOLE TYPE D-50 Track Mount, 108 mm Inside Diameter Hollow Stem Auger				COMPILED BY SKB											
DATUM Geodetic		DATE May 16, 2012				CHECKED BY SMM											
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa									
--- CONTINUED FROM PREVIOUS PAGE ---							<div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 20 40 60 80 100 </div> <div style="display: flex; justify-content: space-between;"> ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED </div>										
NOTES: 1. Water level in open borehole at a depth of 0.9 m below ground surface (Elev. 297.7 m) upon completion of drilling. 2. Blowing sands encountered at a depth of 7.9 m below ground surface. 3. Borehole backfilled with a 1:1 cement to water mixture.																	

GTA-MTO 001 T:\PROJECTS\2009\09-1111-0018 (URS, YORK REGION)\LOG\0911110018.GPJ GAL-GTA.GDT 01/13/16 SIB

PROJECT 09-1111-0018				RECORD OF BOREHOLE No F1-5D				SHEET 1 OF 1				METRIC				
W.P. 2835-02-00				LOCATION N 4867706.6 ; E 298999.5				ORIGINATED BY JIL								
DIST Central HWY 400				BOREHOLE TYPE D-25 Rubber Track Mount, 89 mm O.D. Tricone Wash Bore, N Casing				COMPILED BY JC								
DATUM Geodetic				DATE October 30, 2015				CHECKED BY TWB								
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					WATER CONTENT (%)			
298.5	GROUND SURFACE						<div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 20 40 60 80 100 </div> <div style="display: flex; justify-content: space-between;"> 10 20 30 10 20 30 </div>								kN/m ³	GR SA SI CL
0.0	NO RECOVERY		1	SS	1											
			2	SS	3											
297.0																
1.7	Organic CLAYEY SILT Grey Wet		3A	SS	2									WC=67.2% PL=32.6% LL=55.0%		
	ORGANIC SILT to SILTY PEAT, some sand, trace clay Very loose Dark brown to grey Wet Shells between depths of 3.0 m and 4.8 m		3B													
			4	SS	0											
			5	SS	1											
			6	SS	1											
293.6			7A	SS	2									WC=269.1% OC=28%		
4.9	ORGANIC CLAYEY SILT to ORGANIC SILT to ORGANIC CLAY, trace sand Very soft Brown to grey Moist		7B													
			8	TO	PH									OC=8% PL=42.8% LL=60.7%		
290.9														γ =17.8 PL=28.0% LL=62.5%	0 1 72 27	
7.6	SILTY SAND, trace clay, trace gravel Compact to dense Grey Wet		9	SS	18											
			10	SS	19											
			11	SS	22											
			12	SS	43											
285.7	END OF BOREHOLE													17 53 28 2		
12.8	NOTE: 1. Borehole was advanced using wash boring techniques and therefore the water level inside casing, which was maintained at ground surface is not representative of the groundwater conditions.															

GTA-MTO 001 T:\PROJECTS\2009\09-1111-0018 (URS, YORK REGION)\LOG\0911110018.GPJ GAL-GTA.GDT 01/13/16 SIB

PROJECT		RECORD OF BOREHOLE		No F1-5E		SHEET 1 OF 1		METRIC						
W.P. 2835-02-00		LOCATION		N 4867761.6 ; E 298979.6		ORIGINATED BY		TWB						
DIST Central HWY 400		BOREHOLE TYPE		D-50 Turbo, 210 mm Inside Diameter Hollow Stem Augers		COMPILED BY		SKB						
DATUM Geodetic		DATE		May 17, 2012		CHECKED BY		SMM						
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
300.5	GROUND SURFACE							20 40 60 80 100	20 40 60 80 100	10 20 30				
0.0	TOPSOIL													
	SILTY SAND, trace clay, trace gravel, trace organics and rootlets to 0.6 m Loose Brown Moist to wet below 0.8 m		1	SS	5									
			2	SS	5									
			3	SS	9									
298.3														
2.2	SILTY CLAY, trace to some sand, trace organics and rootlets Soft Dark grey Moist		4	SS	2									
			5A	SS	4									
297.0			5B	SS										
	Silty PEAT, some sand, trace clay, trace rootlets and wood fragments Very loose Dark brown Moist		6A	SS	1									
3.7			6B	SS										
296.2														
4.5	ORGANIC SILT, trace sand, trace shells and wood fragments Very loose Dark brown, grey and light grey Moist		7	SS	9									
294.9														
5.6	SILTY SAND, trace clay, some gravel, trace organics Very loose Brown and dark grey Wet													
	SAND and GRAVEL, trace to some silt Loose Grey Wet		8	SS	11									
293.8														
6.7														
	SILTY SAND, trace clay, trace gravel Compact Brown Wet													
	END OF BOREHOLE													

3.7 m to 4.3 m:
 ORGANIC SILT, trace sand,
 trace shells and wood
 fragments
 Very soft
 Dark brown, grey and light
 grey
 Moist

WC=88.6%
 PL=41.1%
 LL=54.0%

NOTES:
 1. Water level in open borehole at a depth of 2.5 m below ground surface (Elev. 298.0 m) upon completion of drilling.

GTA-MTO 001 T:\PROJECTS\2009\09-1111-0018 (URS, YORK REGION)\LOG\0911110018.GPJ GAL-GTA.GDT 01/13/16 SIB

PROJECT 09-1111-0018		RECORD OF BOREHOLE No F1-5F		SHEET 1 OF 2		METRIC	
W.P. 2835-02-00		LOCATION N 4867736.7 ; E 298956.0		ORIGINATED BY TWB			
DIST Central HWY 400		BOREHOLE TYPE D-90 Track Mount, 89 mm O.D. Tricone Wash Bore, N Casing		COMPILED BY SKB			
DATUM Geodetic		DATE June 4, 2012		CHECKED BY SMM			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	W _p	W	W _L	WATER CONTENT (%)					
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED									
310.0	GROUND SURFACE																
0.0	Asphalt																
0.2	Sand and gravel, some silt, trace clay (FILL) Brown moist																
308.8							309										
1.2	Silty sand, trace clay (FILL) Compact Brown Moist		1A	SS	21												
307.9			1B				308										
2.1	Clayey silt, trace to some sand (FILL) Brown Moist																
307.1																	
2.9	Silty sand, trace clay, pockets of clayey silt (FILL) Compact Brown Moist		2	SS	19		307										
							306										
			3	SS	14		305										
304.4																	
5.6	Sandy silt, trace clay, trace gravel (FILL) Compact Brown Moist		4	SS	25		304										
			5	SS	28		303										
			6	SS	21		302										
			7	SS	17		301										
301.1																	
8.9	SILTY SAND, trace clay, trace gravel Compact to dense Brown Moist		8	SS	19		300										
	dense below 10.2 m		9	SS	27												
	Clayey silt pockets below 10.7 m		10	SS	32		299										
298.3																	
11.7	SAND and SILT, trace clay, trace gravel Compact Brown Moist		11A	SS	29		298										
297.4			11B														
12.6	SAND, some silt, trace gravel Compact Brown Wet						297										
296.7																	
13.3	CLAYEY SILT, trace sand, trace gravel Stiff Brown Moist		12A	SS	10		296										
296.0			12B														
14.0																	

Continued Next Page

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

GTA-MTO 001 T:\PROJECTS\2009\09-1111-0018 (URS, YORK REGION)\LOG\0911110018.GPJ GAL-GTA.GDT 01/13/16 SIB

PROJECT 09-1111-0018		RECORD OF BOREHOLE No F1-5F				SHEET 2 OF 2		METRIC								
W.P. 2835-02-00		LOCATION N 4867736.7 ; E 298956.0				ORIGINATED BY TWB										
DIST Central HWY 400		BOREHOLE TYPE D-90 Track Mount, 89 mm O.D. Tricone Wash Bore, N Casing				COMPILED BY SKB										
DATUM Geodetic		DATE June 4, 2012				CHECKED BY SMM										
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa								
	--- CONTINUED FROM PREVIOUS PAGE ---															
	SILTY SAND, trace clay, trace gravel Loose to dense Brown Wet Clayey silt pockets to 16.8 m		13	SS	9											
						294										
	Silt seam, shell fragments from 16.8 to 16.9		14	SS	44											
						293										
						292										
			15	SS	35											
						291										
						290										
289.6	END OF BOREHOLE		16	SS	46											
20.4	NOTE: 1. Water level in open borehole at a depth of 11.3 m below ground surface (Elev. 298.7 m) upon completion of drilling.															

GTA-MTO 001 T:\PROJECTS\2009\09-1111-0018 (URS, YORK REGION)\LOG\0911110018.GPJ GAL-GTA.GDT 01/13/16 SIB

PROJECT 09-1111-0018		RECORD OF BOREHOLE No F1-5G		SHEET 1 OF 2		METRIC	
W.P. 2835-02-00		LOCATION N 4867733.6 ; E 298994.2		ORIGINATED BY JIL			
DIST Central HWY 400		BOREHOLE TYPE D-25 Rubber Track Mount, 89 mm O.D. Tricone Wash Bore, N Casing		COMPILED BY SKB			
DATUM Geodetic		DATE October 30, 2015 - November 2, 2015		CHECKED BY SMM			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W _p W W _L				WATER CONTENT (%)
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED						
298.9	GROUND SURFACE							20 40 60 80 100					GR SA SI CL	
0.0	SILTY SAND, rootlets in upper 0.15 m, trace organics		1	SS	5	298							3 24 61 12	
298.1	Loose Grey Wet		2A	SS	3									
0.8	Sandy CLAYEY SILT, trace gravel, trace organics		2B											
	Soft to firm Grey Moist		3	SS	5	297								
						296								
295.7	SILTY SANDY PEAT, trace clay		4A	SS	7	295							WC=53.0%	
3.2	Very loose to loose Dark brown Moist to wet		4B											
			5	SS	4									
			6	SS	4	294								
						293								
293.2	- Trace shells below a depth of 5.5 m		7A	SS	2	292							OC=18%	
5.7	ORGANIC CLAYEY SILT		7B											
	Very soft Dark brown to black Moist to wet		7C											
			8	TO	PH	291								
						290								
290.5	SILTY CLAY, trace organics		9	SS	1	289							OC=4% WC=45.2% PL=21.9% LL=45.0%	
8.4	Very soft Grey with black laminations Moist		10	SS	1									
			11	SS	2									
	- Some sand to sandy at a depth of 11.0 m					288								
287.3	SILTY SAND, trace gravel, trace clay		12	SS	42	287							4 74 19 3	
11.6	Dense Grey Wet													
			13	SS	47									
284.6	END OF BOREHOLE					285								
14.3														

5.7 m to 8.4 m
Clayey ORGANIC SILT
Very soft
Dark brown to black
Moist to wet

Continued Next Page

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

GTA-MTO 001 T:\PROJECTS\2009\09-1111-0018 (URS, YORK REGION)\LOG\0911110018.GPJ GAL-GTA.GDT 01/13/16 SIB

PROJECT <u>09-1111-0018</u>		RECORD OF BOREHOLE No F1-5G		SHEET 2 OF 2		METRIC	
W.P. <u>2835-02-00</u>		LOCATION <u>N 4867733.6 ;E 298994.2</u>		ORIGINATED BY <u>JIL</u>			
DIST <u>Central</u> HWY <u>400</u>		BOREHOLE TYPE <u>D-25 Rubber Track Mount, 89 mm O.D. Tricone Wash Bore, N Casing</u>		COMPILED BY <u>SKB</u>			
DATUM <u>Geodetic</u>		DATE <u>October 30, 2015 - November 2, 2015</u>		CHECKED BY <u>SMM</u>			

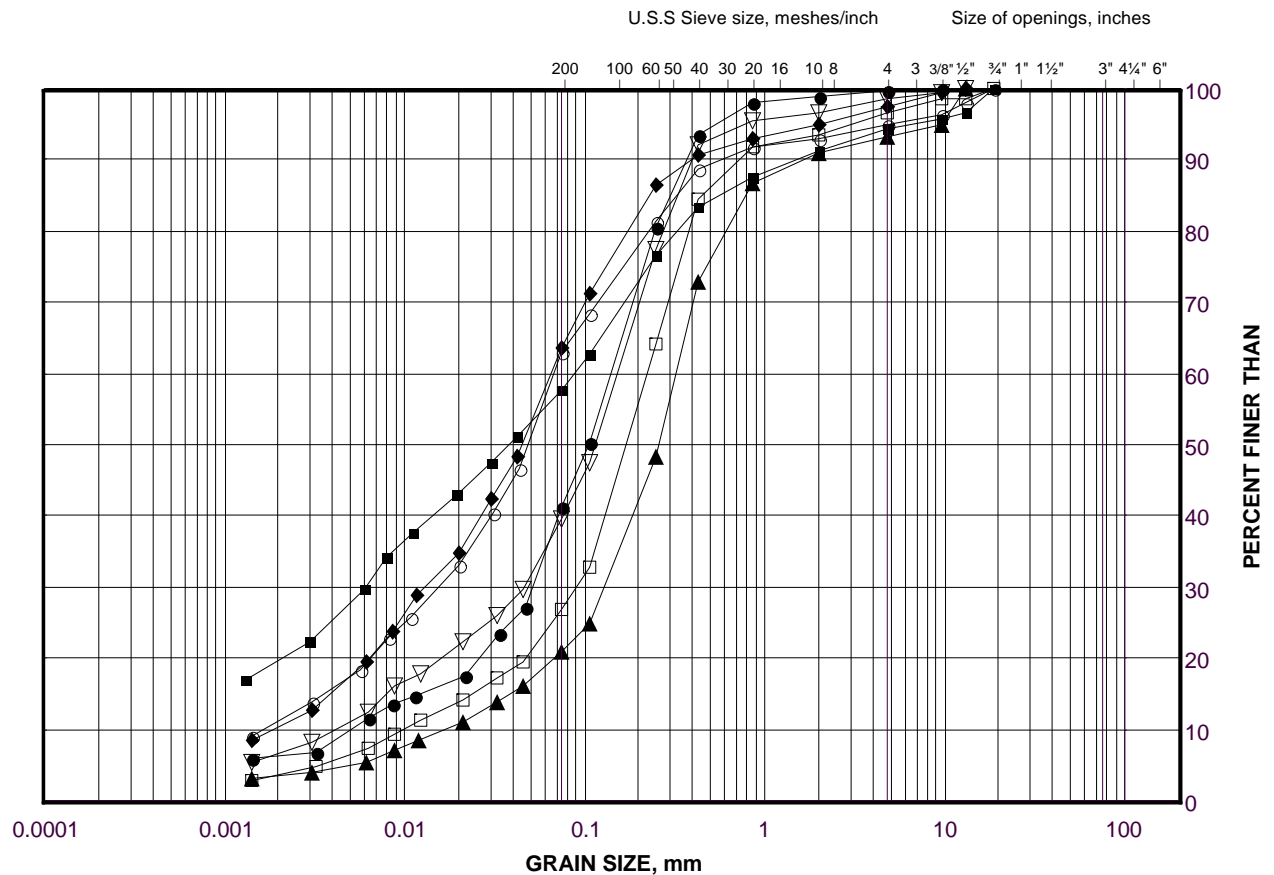
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W _p	W	W _L		GR	SA	SI	CL	
--- CONTINUED FROM PREVIOUS PAGE ---					○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x REMOULDED					WATER CONTENT (%)											
	NOTE: 1. Borehole was advanced using wash boring techniques and therefore the water level inside casing, which was maintained at ground surface is not representative of the groundwater conditions.																				

GTA-MTO 001 T:\PROJECTS\2009\09-1111-0018 (URS, YORK REGION)\LOG\0911110018.GPJ GAL-GTA.GDT 01/13/16 SIB

GRAIN SIZE DISTRIBUTION

Silty Sand to Clayey Silt Fill

FIGURE A-1A



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	C29-3	2	306.4
■	F1-3	2	298.5
◆	C29-2	3	305.6
▲	C29-3	4	304.9
▽	C29-2	6	303.3
○	C29-3	7	301.8
□	C29-2	9	298.8

Project Number: 09-1111-0018

Checked By: TWB

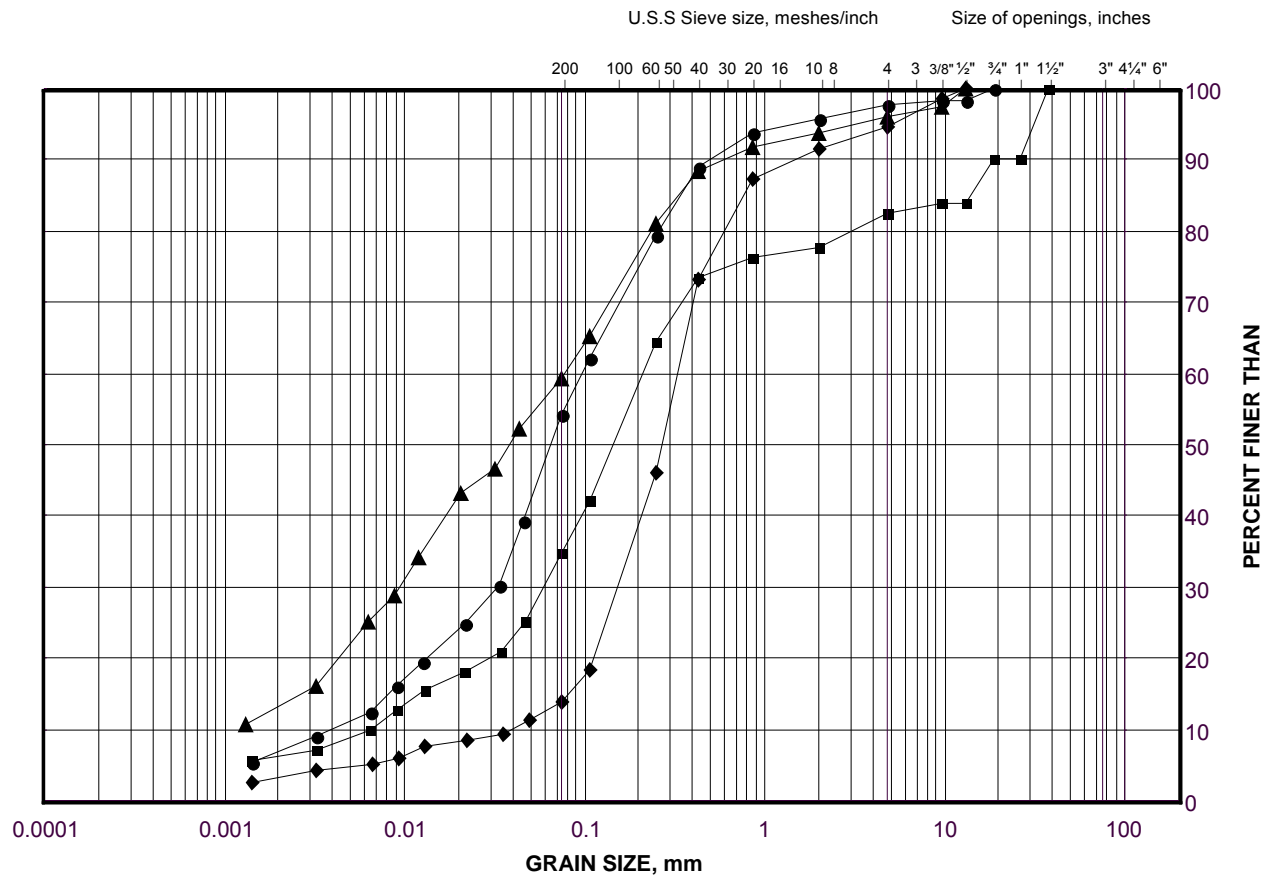
Golder Associates

Date: 06-Jan-16

GRAIN SIZE DISTRIBUTION

Silty Sand to Clayey Silt Fill

FIGURE A-1B



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

LEGEND

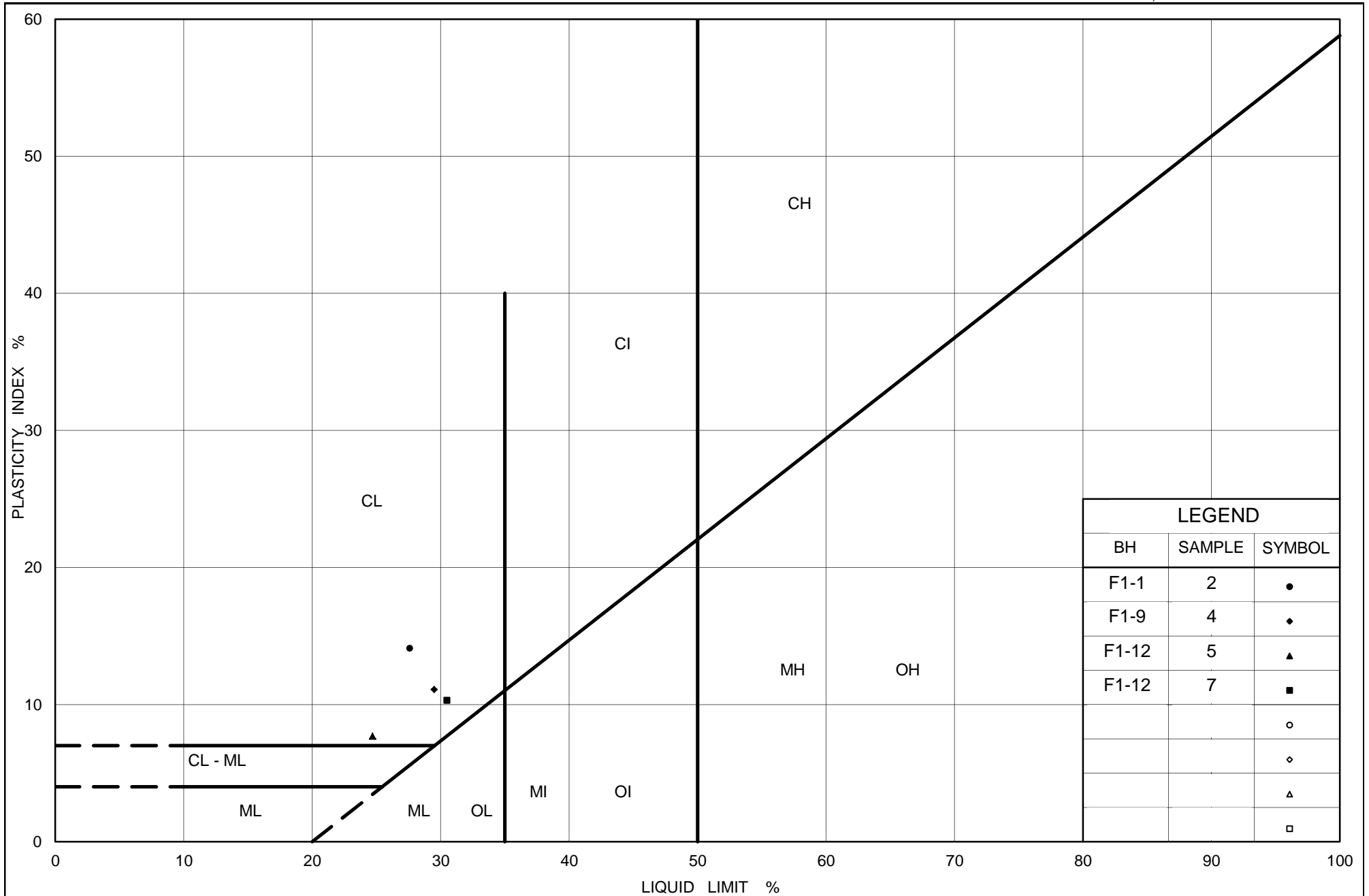
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	F1-12	3	311.8
■	F1-6	3	309.0
◆	F1-6	5	306.7
▲	F1-6	7	305.2

Project Number: 09-1111-0018

Checked By: TWB

Golder Associates

Date: 06-Jan-16



Ministry of Transportation

Ontario

PLASTICITY CHART

Clayey Silt Fill

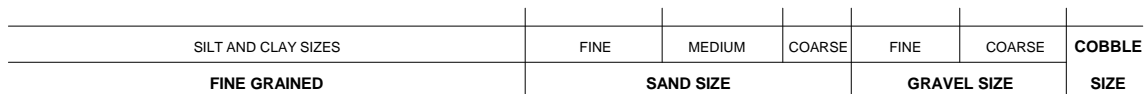
Figure No. A-2

Project No. 09-1111-0018

Checked By: TWB

Upper Clayey Silt with Sand

Date: 06-Jan-16

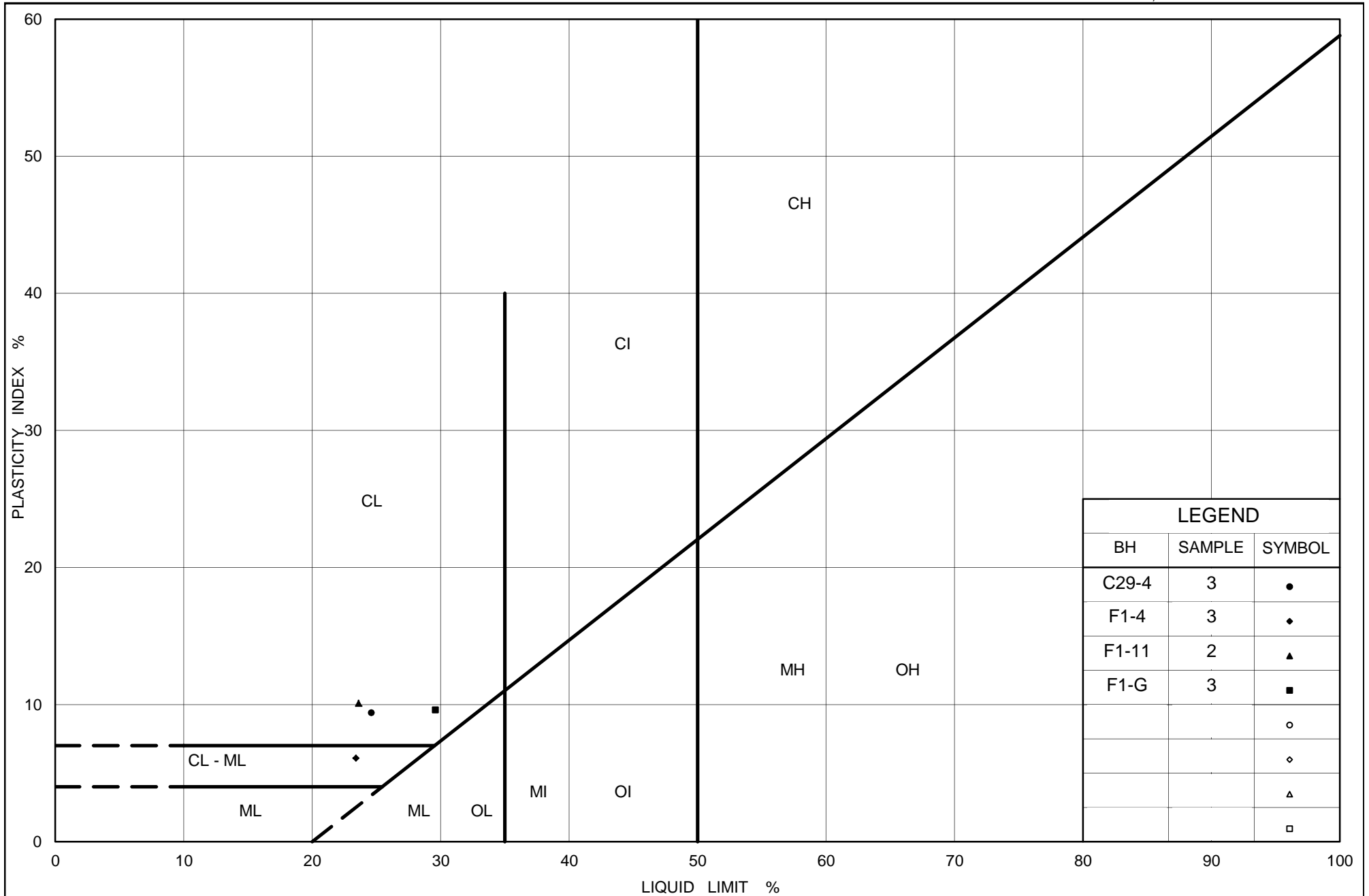


Project Number: 09-1111-0018

Checked By: TWB

Golder Associates

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	F1-5A	2	298.4
■	F1-5G	3	297.2
◆	C29-4	3	297.7



Ministry of Transportation

Ontario

PLASTICITY CHART Upper Clayey Silt to Sandy Clayey Silt

Figure No. A-4

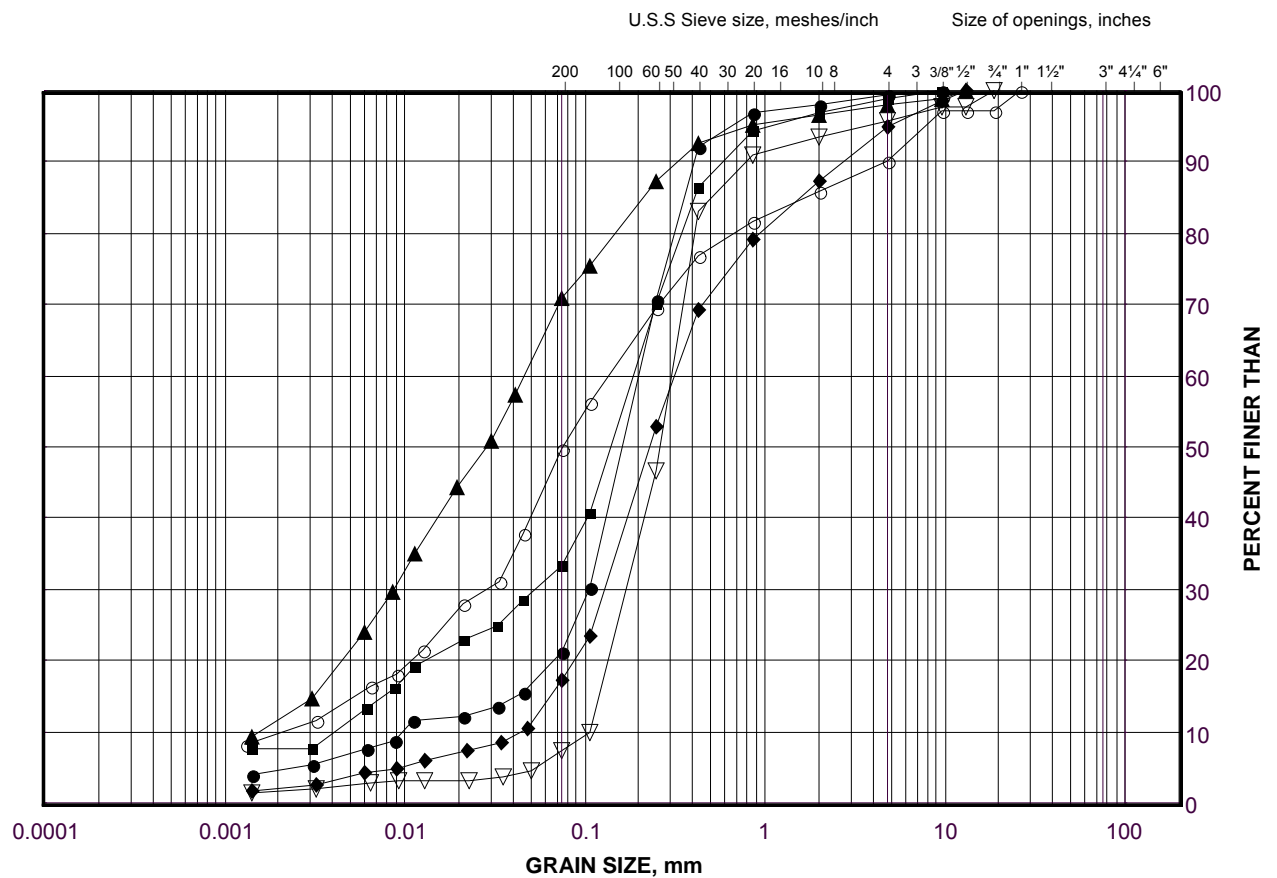
Project No. 09-1111-0018

Checked By: TWB

GRAIN SIZE DISTRIBUTION

Silt to Sand

FIGURE A-5A



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	C29-3	10	297.2
■	F1-5B	2	298.0
◆	C29-4	4	296.9
▲	F1-4	4A	297.0
▽	C29-4	6	295.4
○	F1-6	8B	303.6

Project Number: 09-1111-0018

Checked By: TWB

Golder Associates

Date: 06-Jan-16

Silt to Sand

FIGURE A-5B



SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	F1-5C	2	297.6
■	F1-11	8	296.3

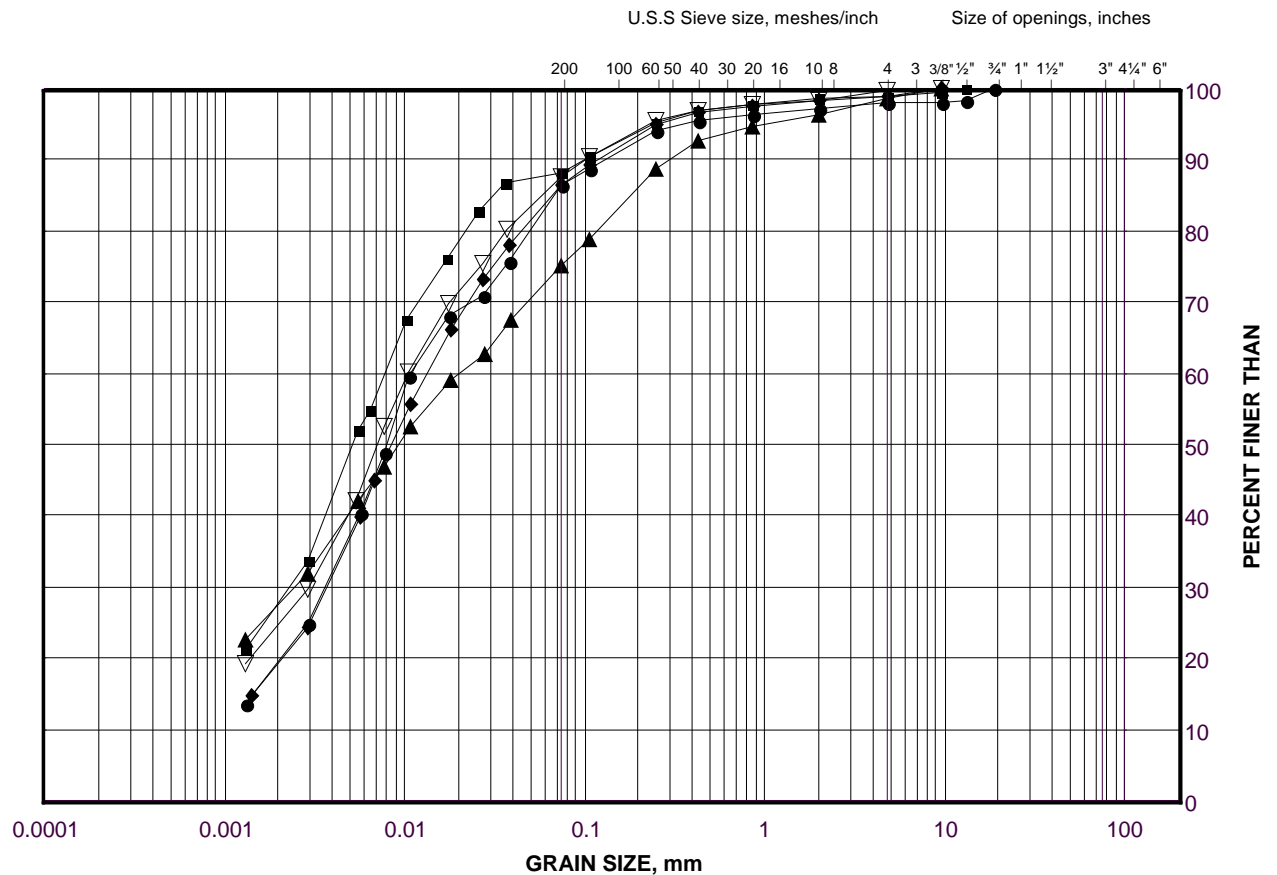
Checked By: TWB

Date: 06-Jan-16

GRAIN SIZE DISTRIBUTION

Clayey Silt Till

FIGURE A-6A



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	C29-2	11	295.7
■	C29-3	12	294.2
◆	F1-10	3	299.6
▲	F1-8	3	301.0
▽	F1-2	3	299.8

Project Number: 09-1111-0018

Checked By: TWB

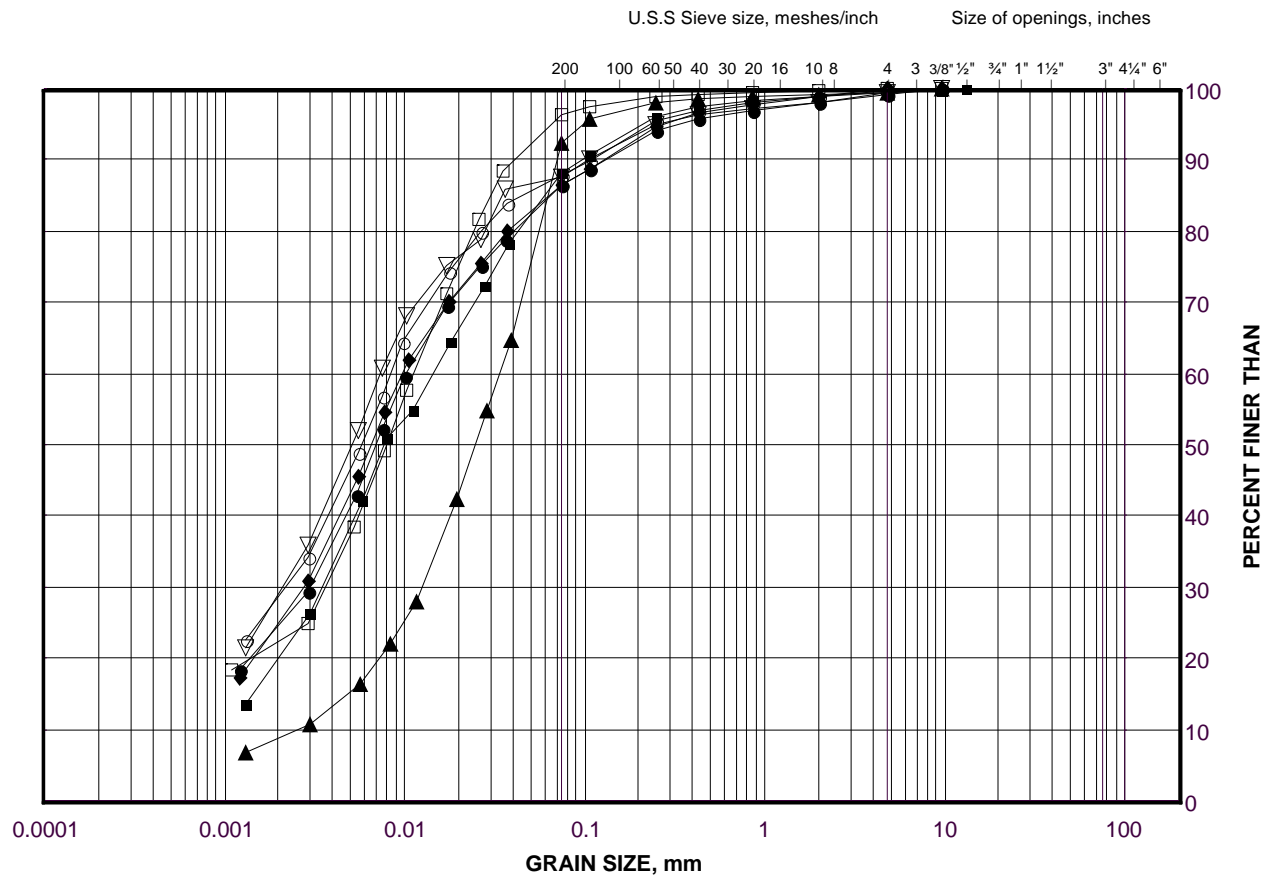
Golder Associates

Date: 06-Jan-16

GRAIN SIZE DISTRIBUTION

Clayey Silt Till

FIGURE A-6B



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	C29-1	4	295.7
■	F1-1	4	302.1
◆	F1-7	4A	305.4
▲	F1-8	5	299.4
▽	F1-9	6	301.4
○	F1-4	6	295.4
□	F1-11	6	298.6

Project Number: 09-1111-0018

Checked By: TWB

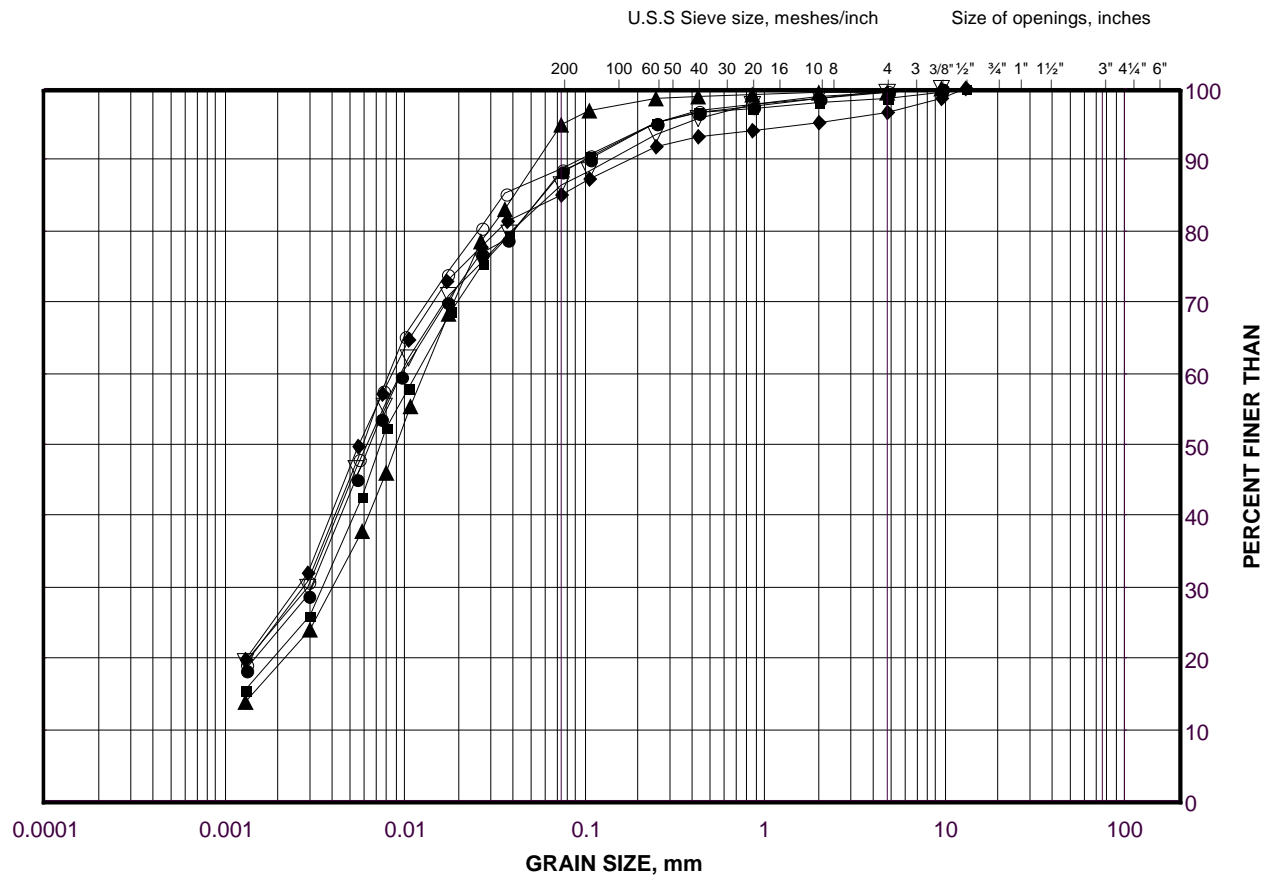
Golder Associates

Date: 06-Jan-16

GRAIN SIZE DISTRIBUTION

Clayey Silt Till

FIGURE A-6C



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	F1-10	6	297.3
■	F1-3	6	295.5
◆	F1-2	7	296.7
▲	F1-7	8	301.4
▽	F1-1	8	297.6
○	C29-4	8	293.1

Project Number: 09-1111-0018

Checked By: TWB

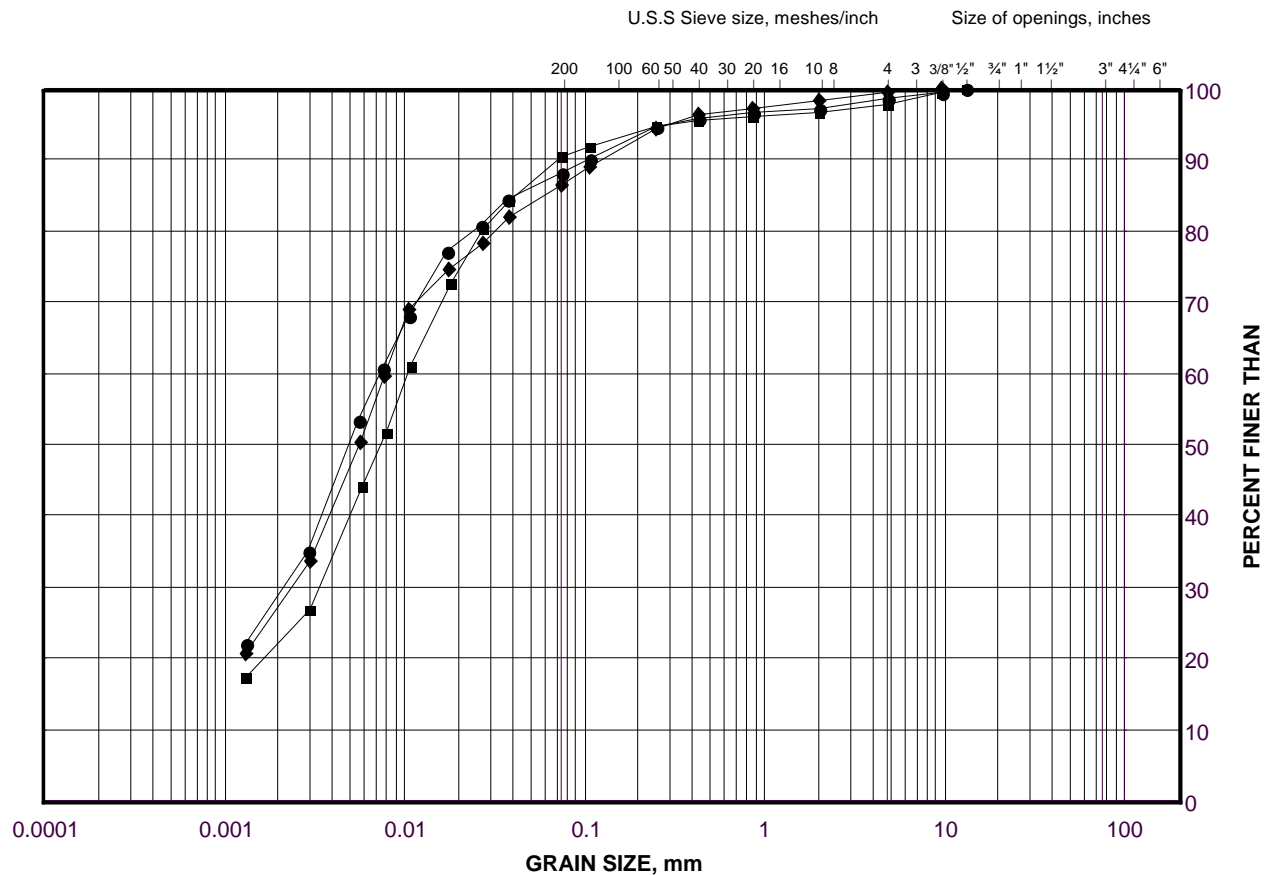
Golder Associates

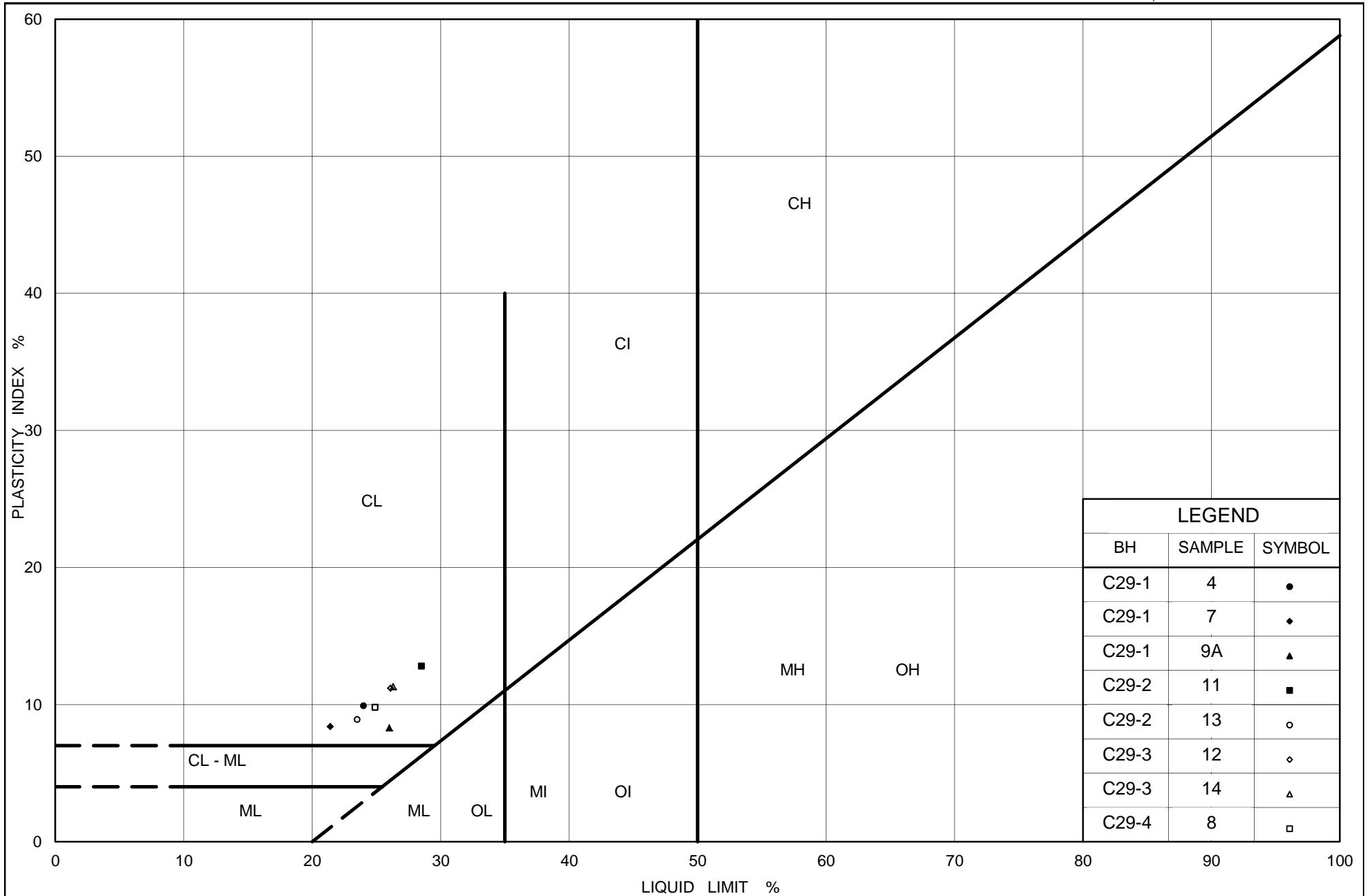
Date: 06-Jan-16

GRAIN SIZE DISTRIBUTION

Clayey Silt Till

FIGURE A-6D





Ministry of Transportation

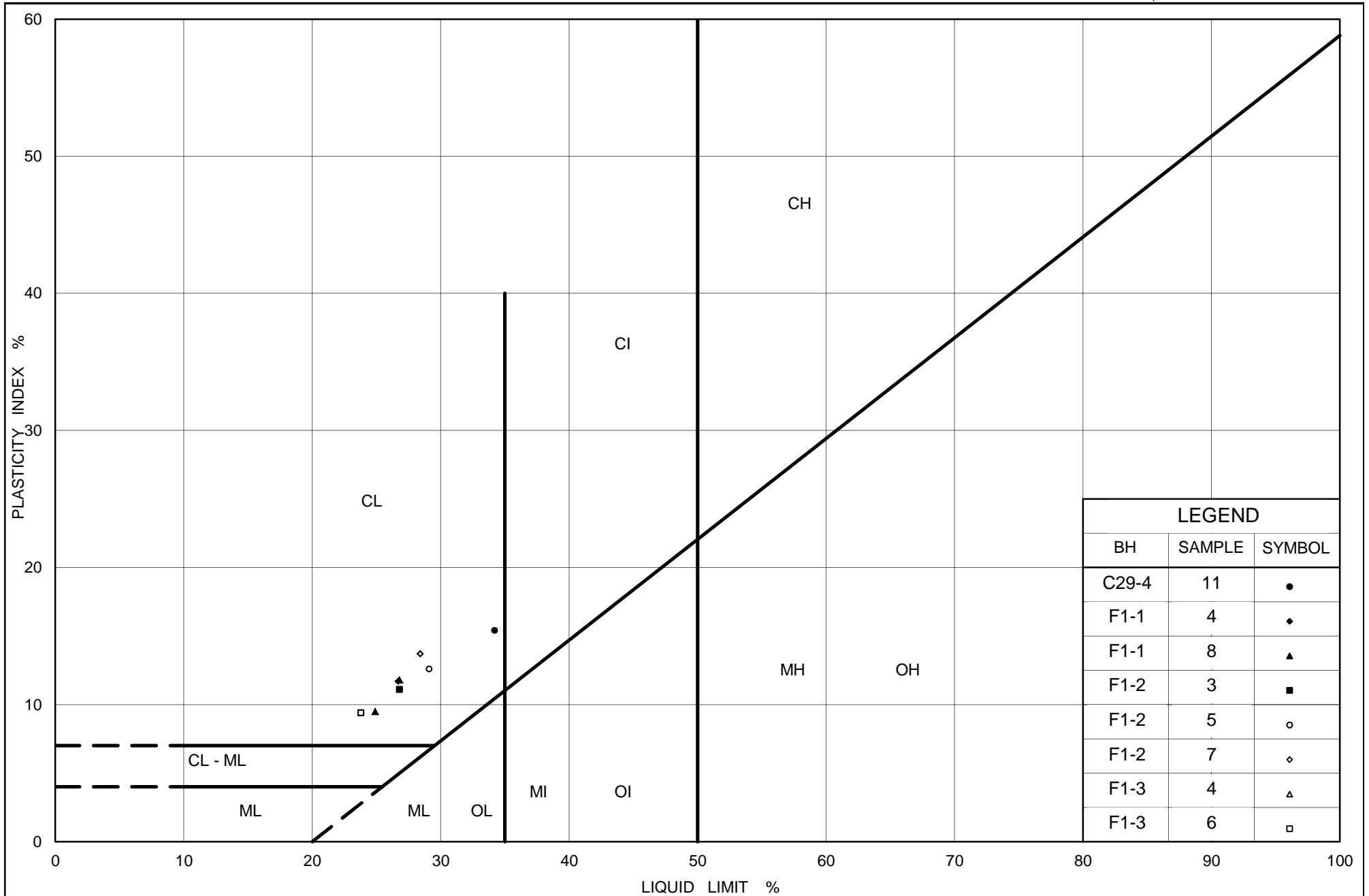
Ontario

PLASTICITY CHART Clayey Silt Till

Figure No. A-7A

Project No. 09-1111-0018

Checked By: TWB



Ministry of Transportation

Ontario

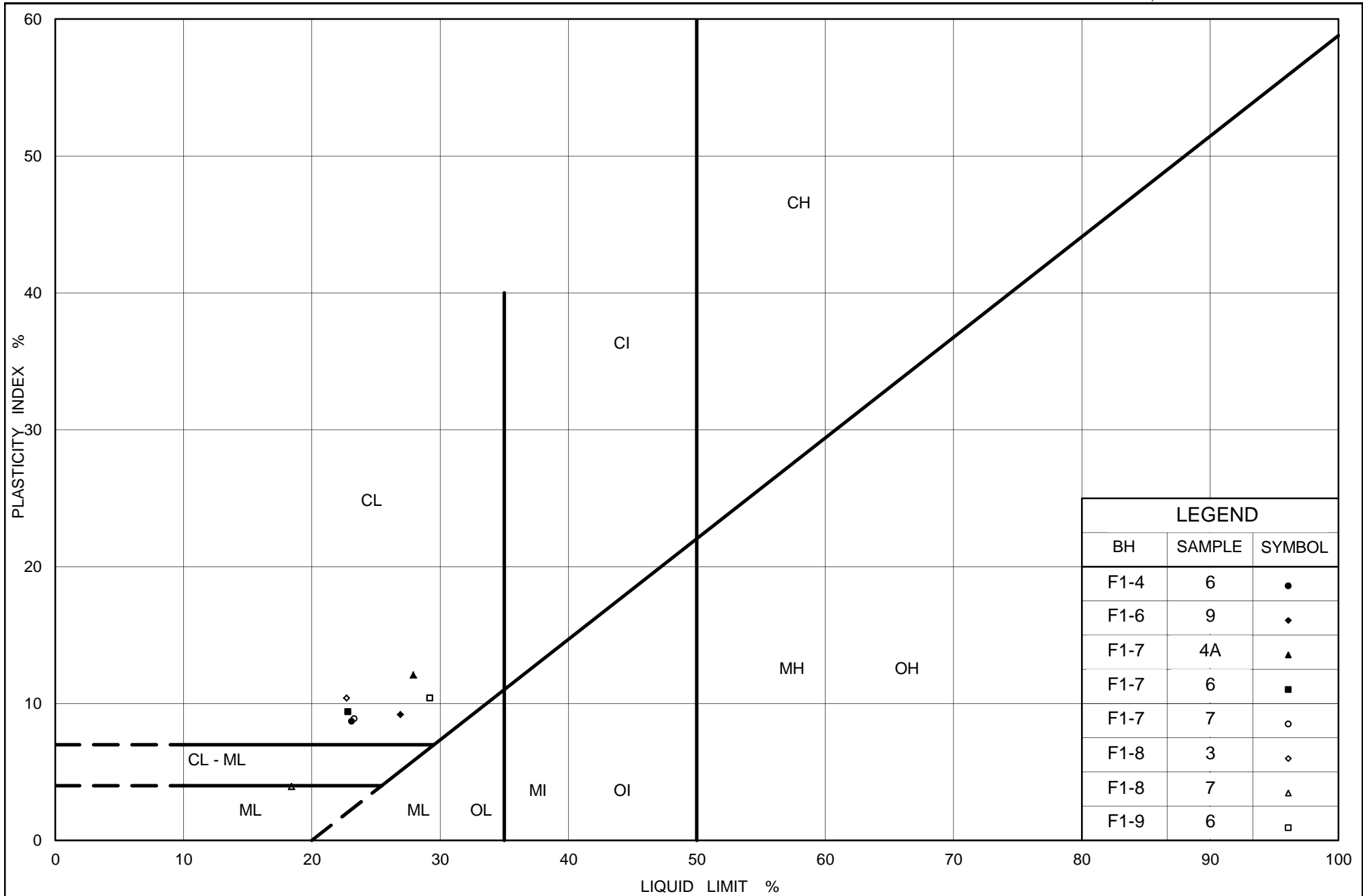
PLASTICITY CHART

Clayey Silt Till

Figure No. A-7B

Project No. 09-1111-0018

Checked By: TWB



Ministry of Transportation

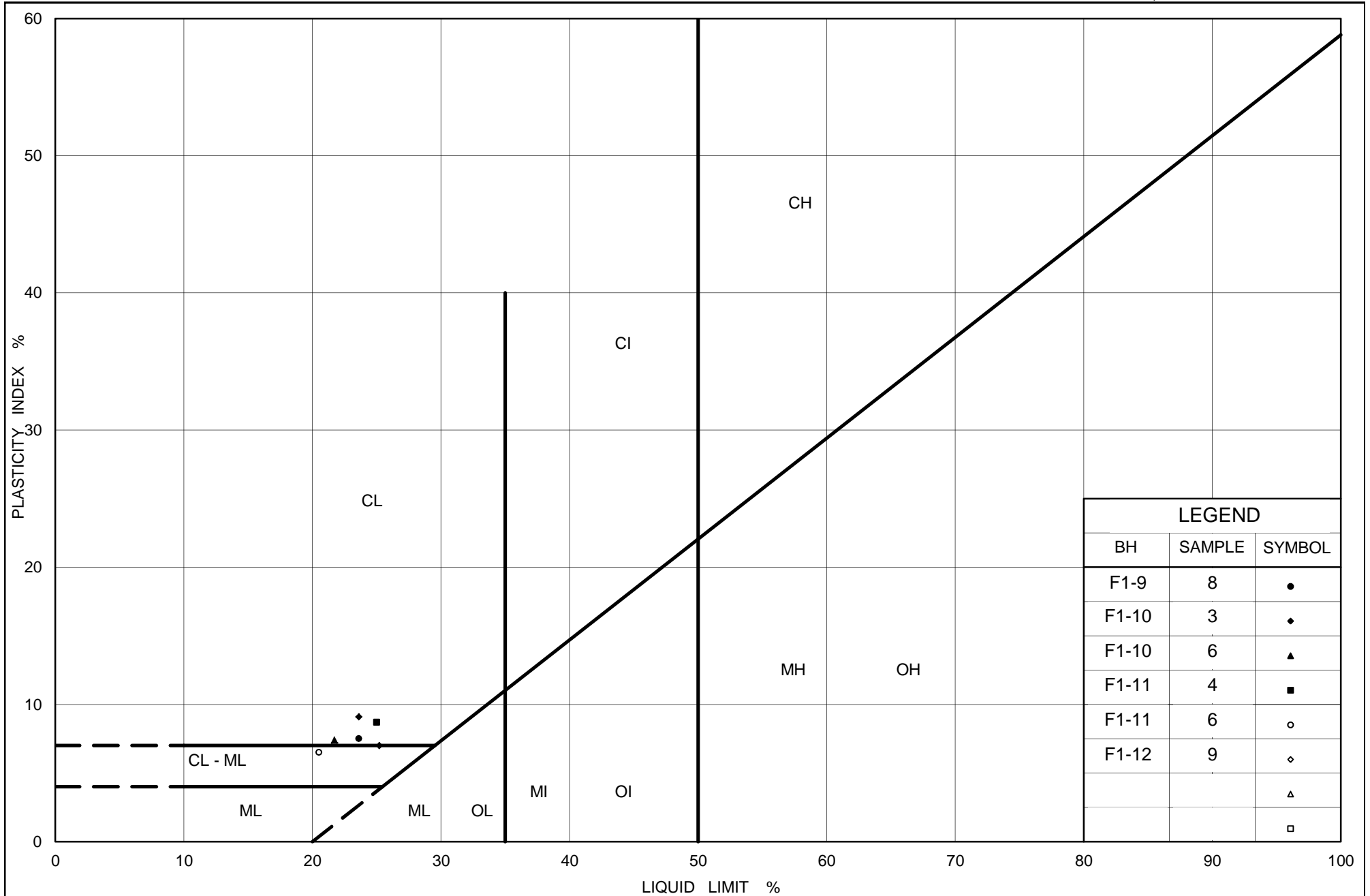
Ontario

PLASTICITY CHART Clayey Silt to Silt Till

Figure No. A-7C

Project No. 09-1111-0018

Checked By: TWB



Ministry of Transportation

Ontario

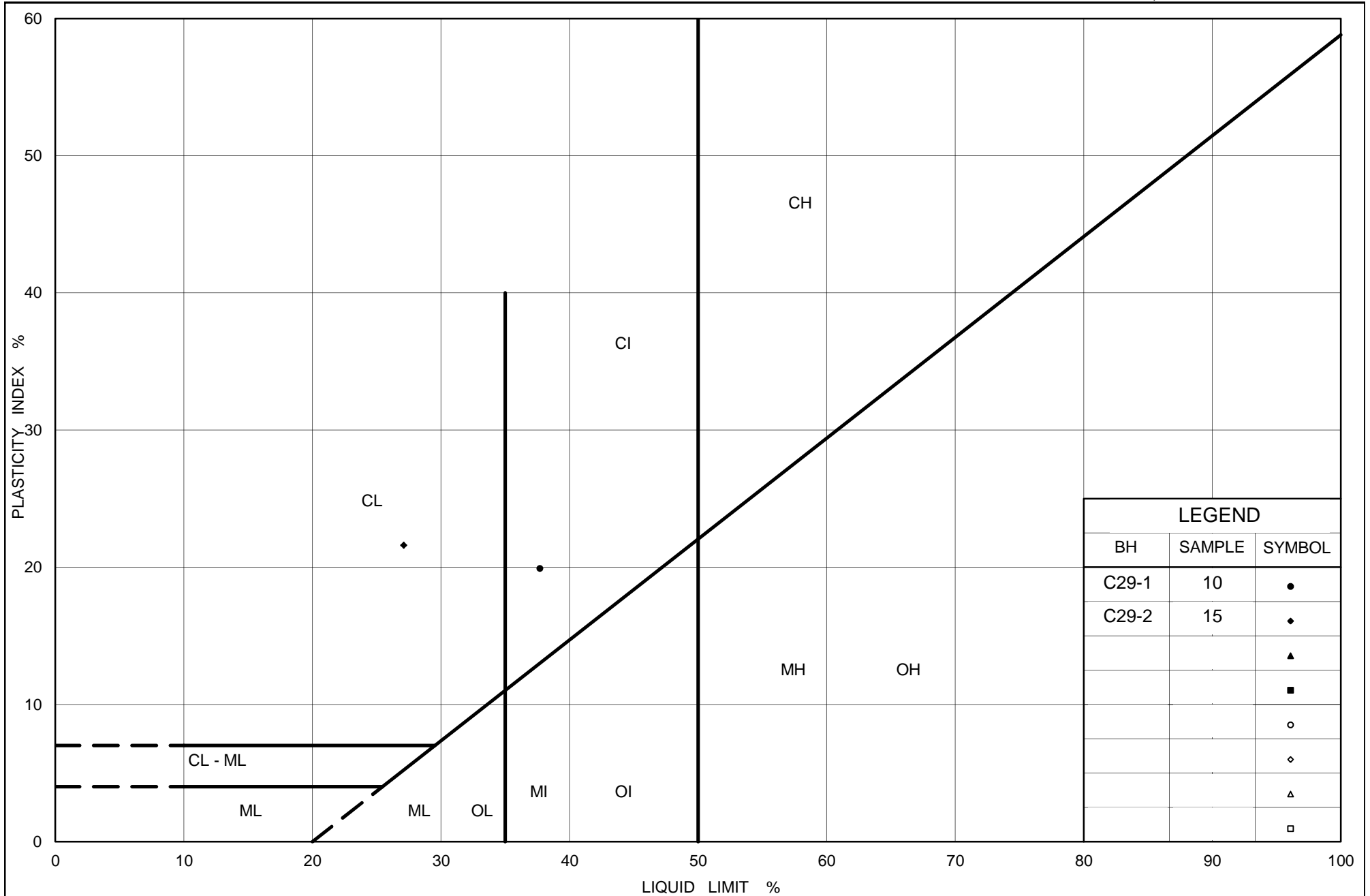
PLASTICITY CHART

Clayey Silt Till

Figure No. A-7D

Project No. 09-1111-0018

Checked By: TWB



Ministry of Transportation

Ontario

PLASTICITY CHART Lower Clayey Silt to Silty Clay

Figure No. A-9

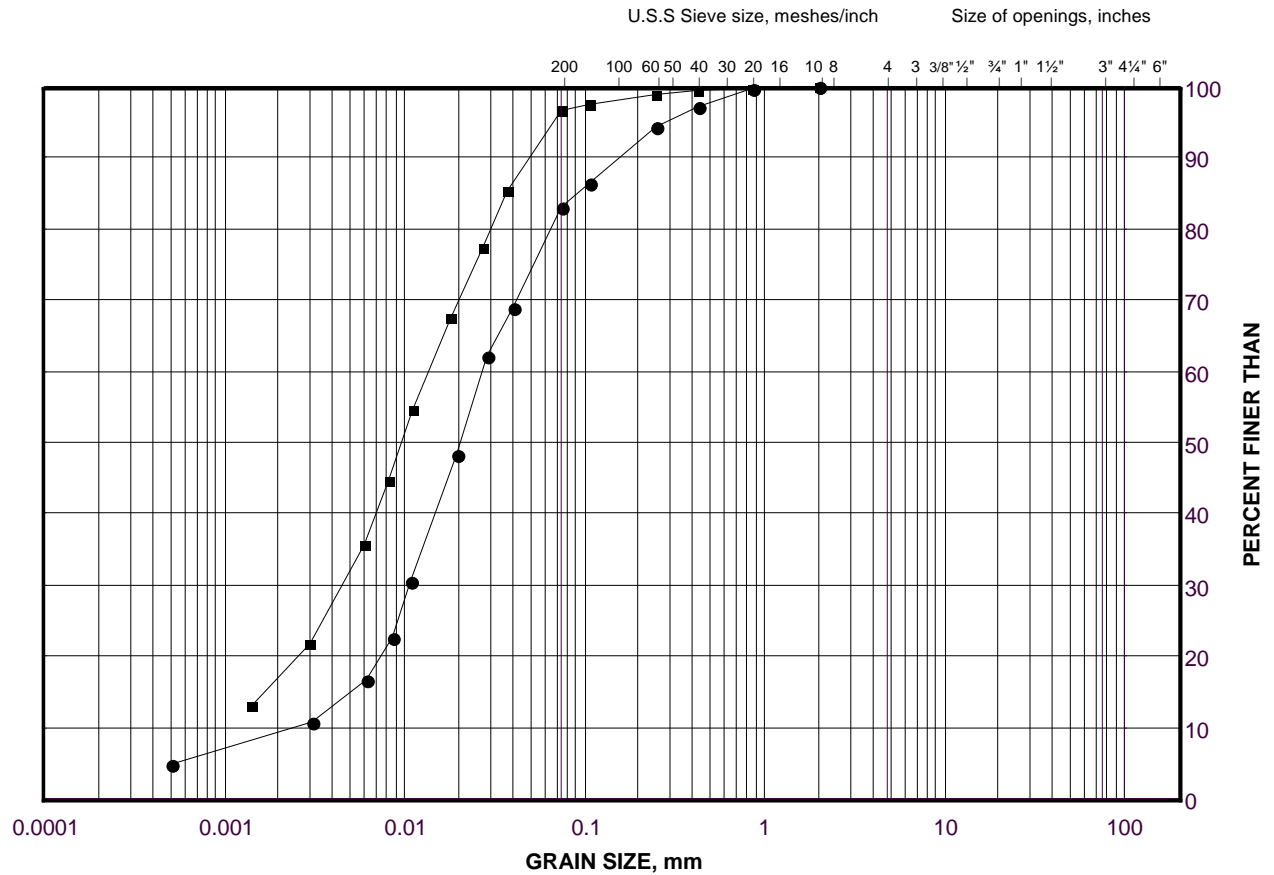
Project No. 09-1111-0018

Checked By: TWB

GRAIN SIZE DISTRIBUTION

Organic Silt

FIGURE A-10A



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	F1-5B	15B	286.4
■	F1-5A	5	296.2

Project Number: 09-1111-0018

Checked By: TWB

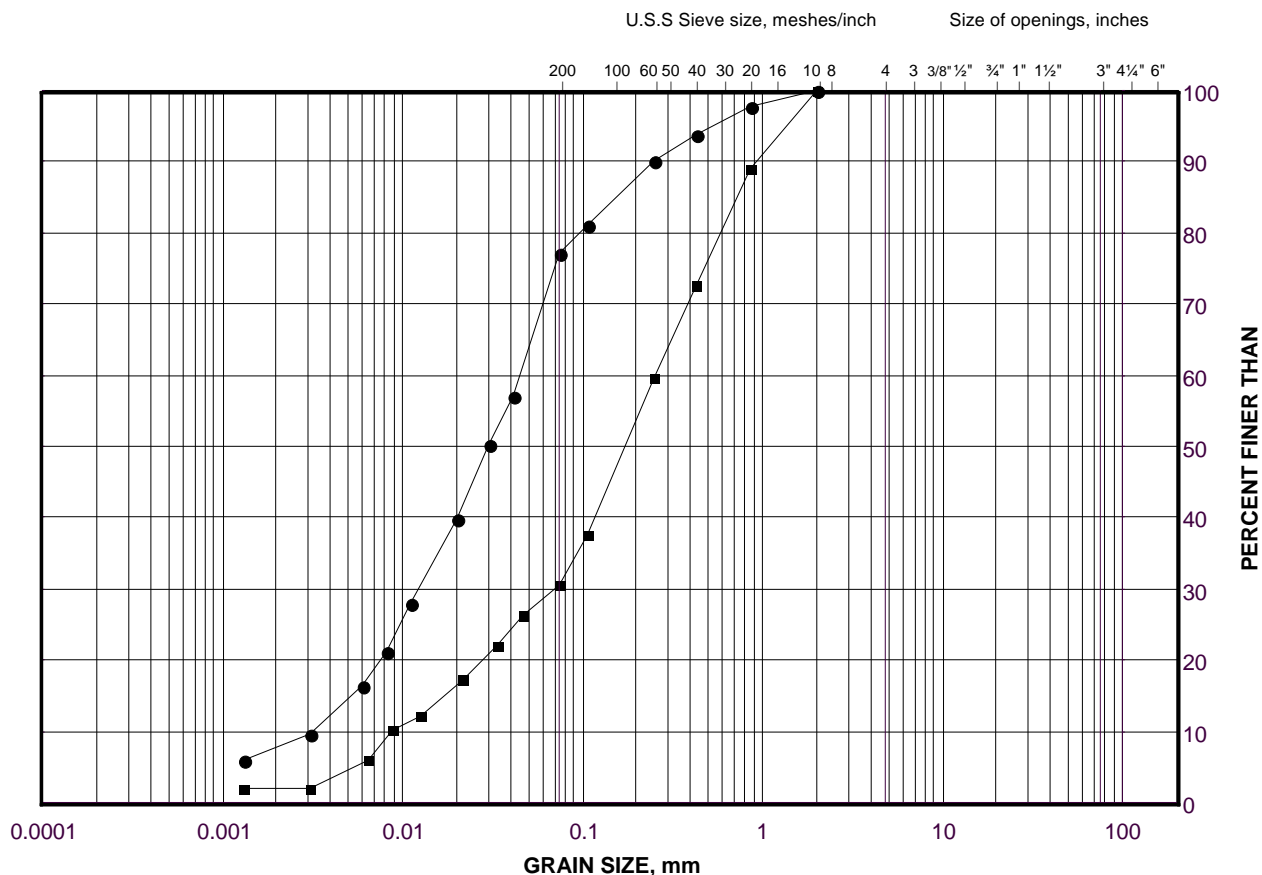
Golder Associates

Date: 06-Jan-16

GRAIN SIZE DISTRIBUTION

Organic Sandy Silt to Silty Sandy Peat

FIGURE A-10B



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
●	F1-5B	10	291.9
■	F1-5B	5	295.7

Project Number: 09-1111-0018

Checked By: TWB

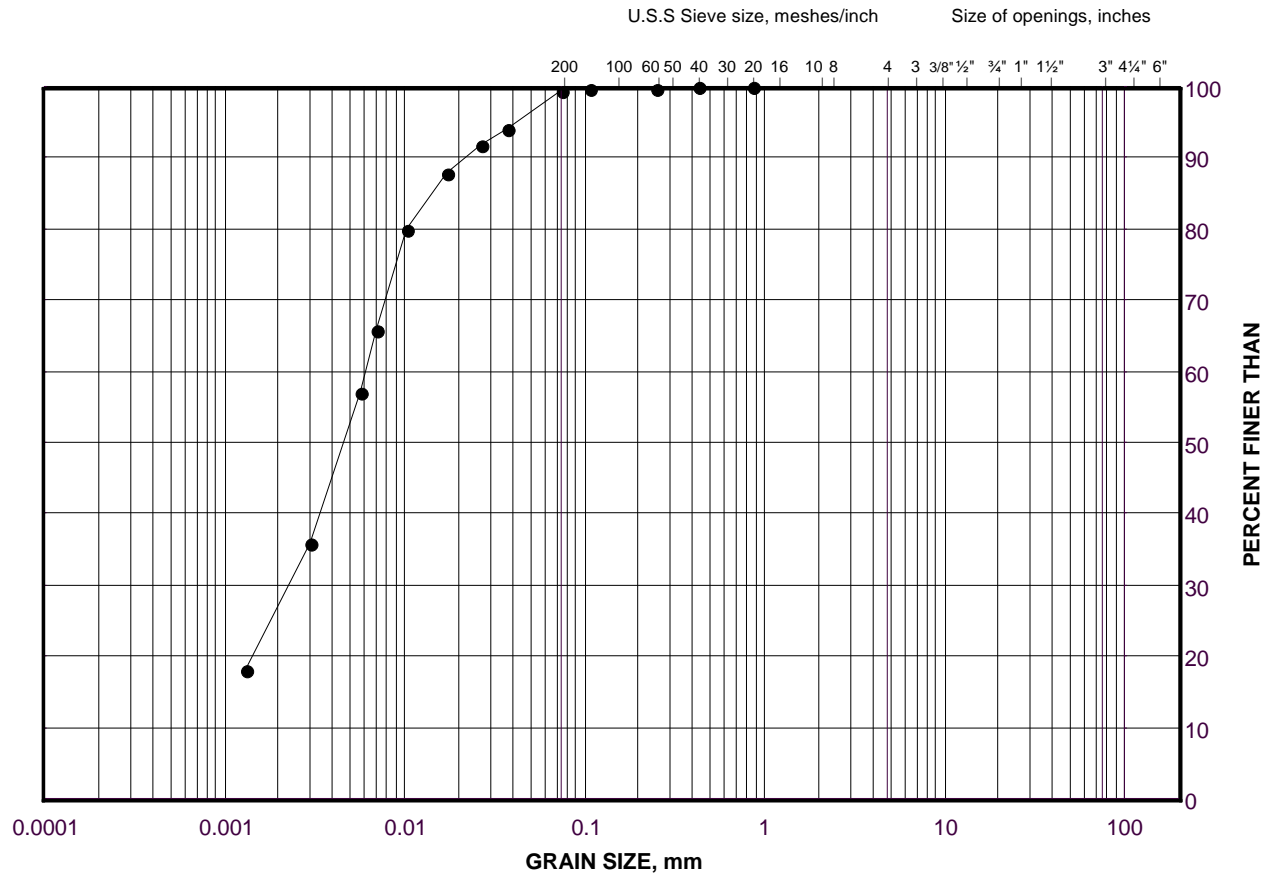
Golder Associates

Date: 06-Jan-16

GRAIN SIZE DISTRIBUTION

Organic Clay

FIGURE A-10C



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	F1-5D	8	292.0

Project Number: 09-1111-0018

Checked By: TWB

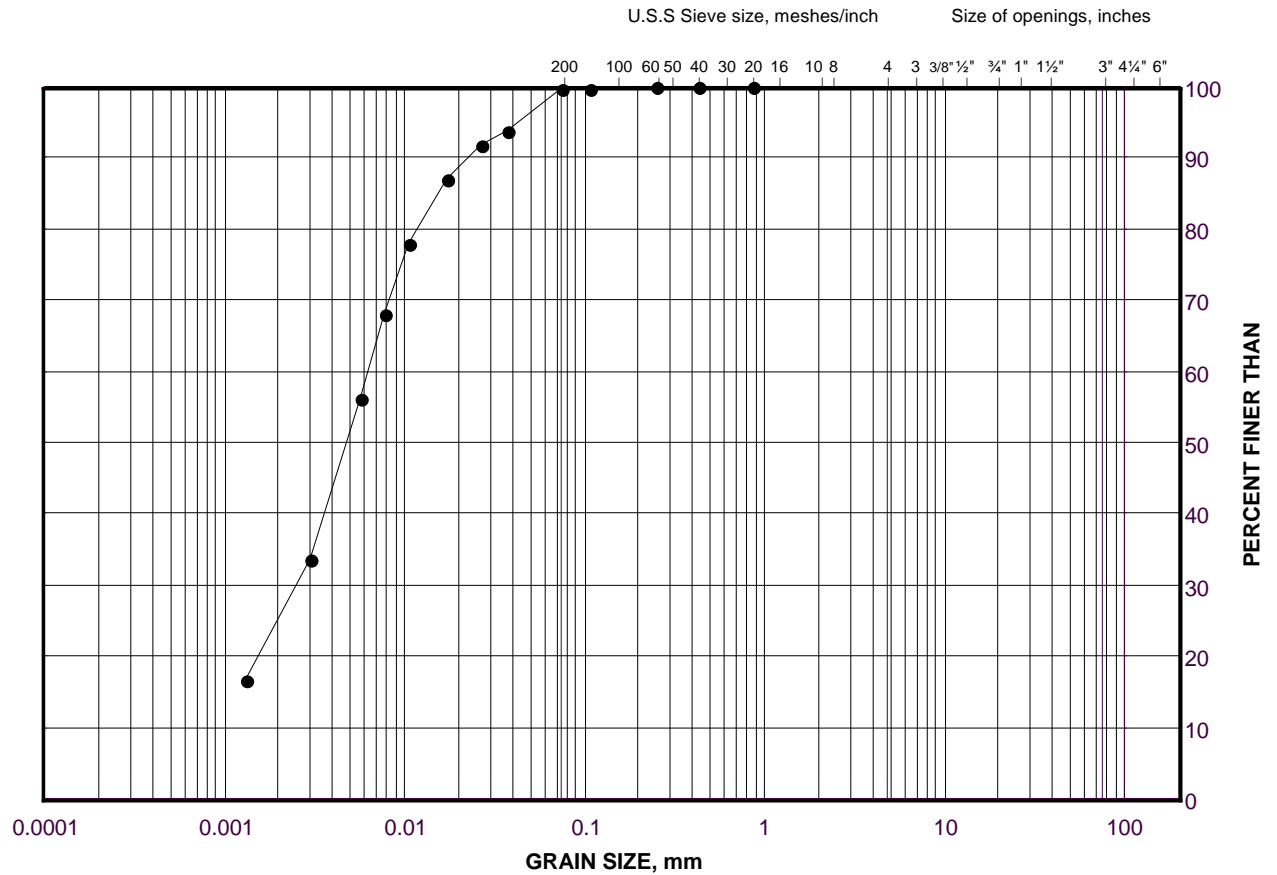
Golder Associates

Date: 06-Jan-16

GRAIN SIZE DISTRIBUTION

Silty Clay, Trace Organics

FIGURE A-10D



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

LEGEND

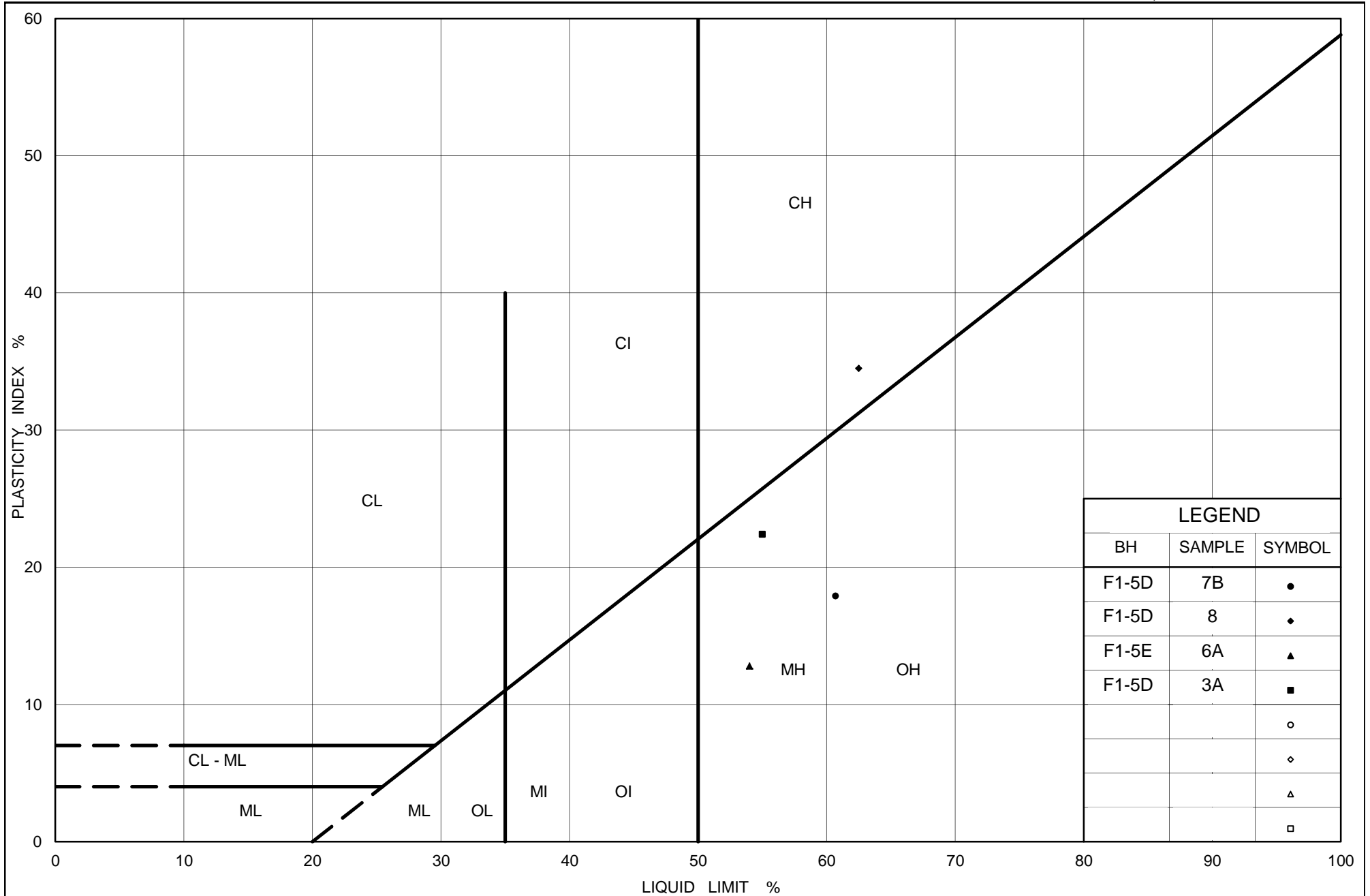
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	F1-5G	9	290.2

Project Number: 09-1111-0018

Checked By: TWB

Golder Associates

Date: 06-Jan-16



Ministry of Transportation

Ontario

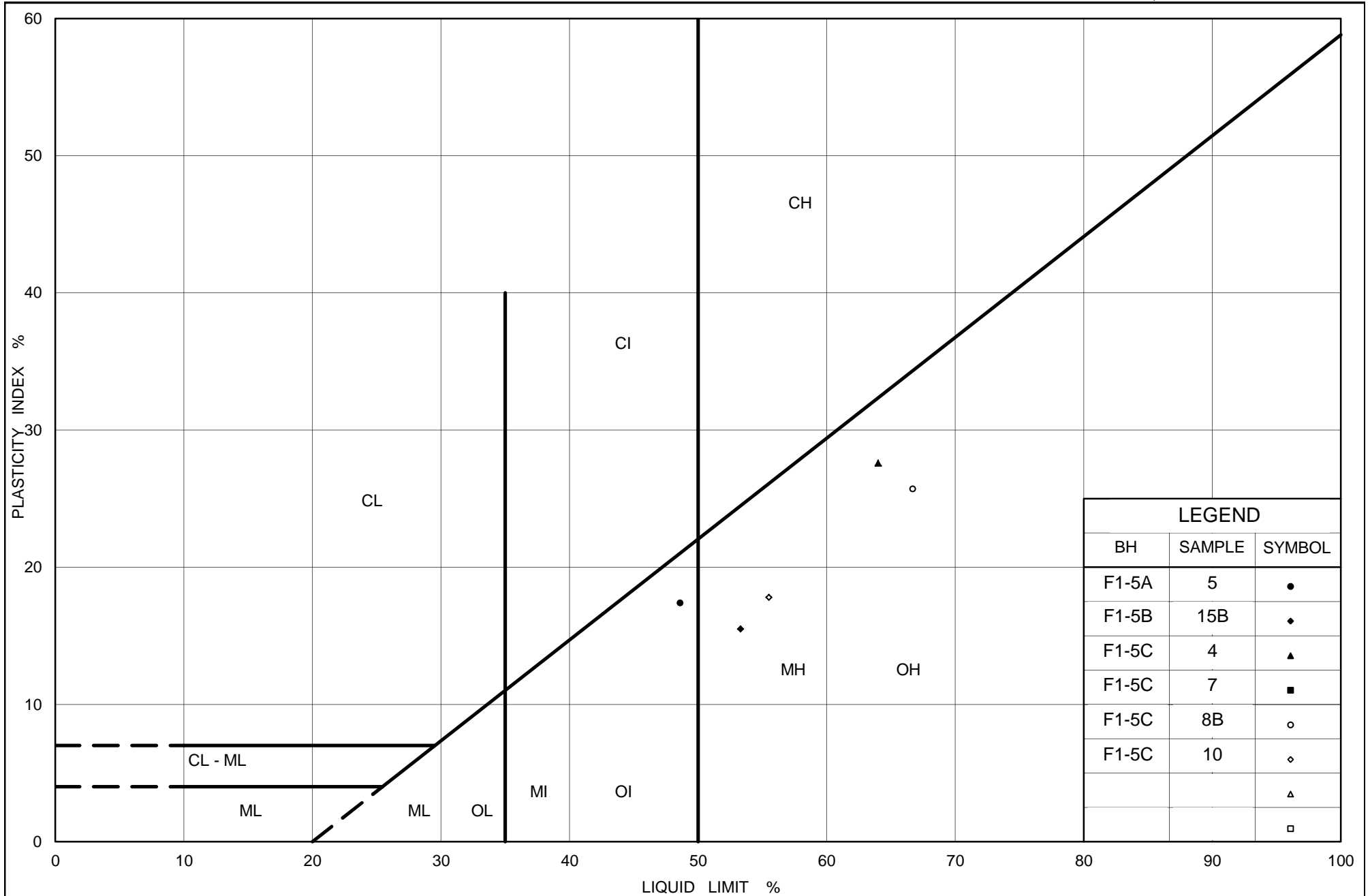
PLASTICITY CHART

Organic Silt to Organic Clayey Silt to Organic Clay

Figure No. A-11A

Project No. 09-1111-0018

Checked By: TWB



Ministry of Transportation

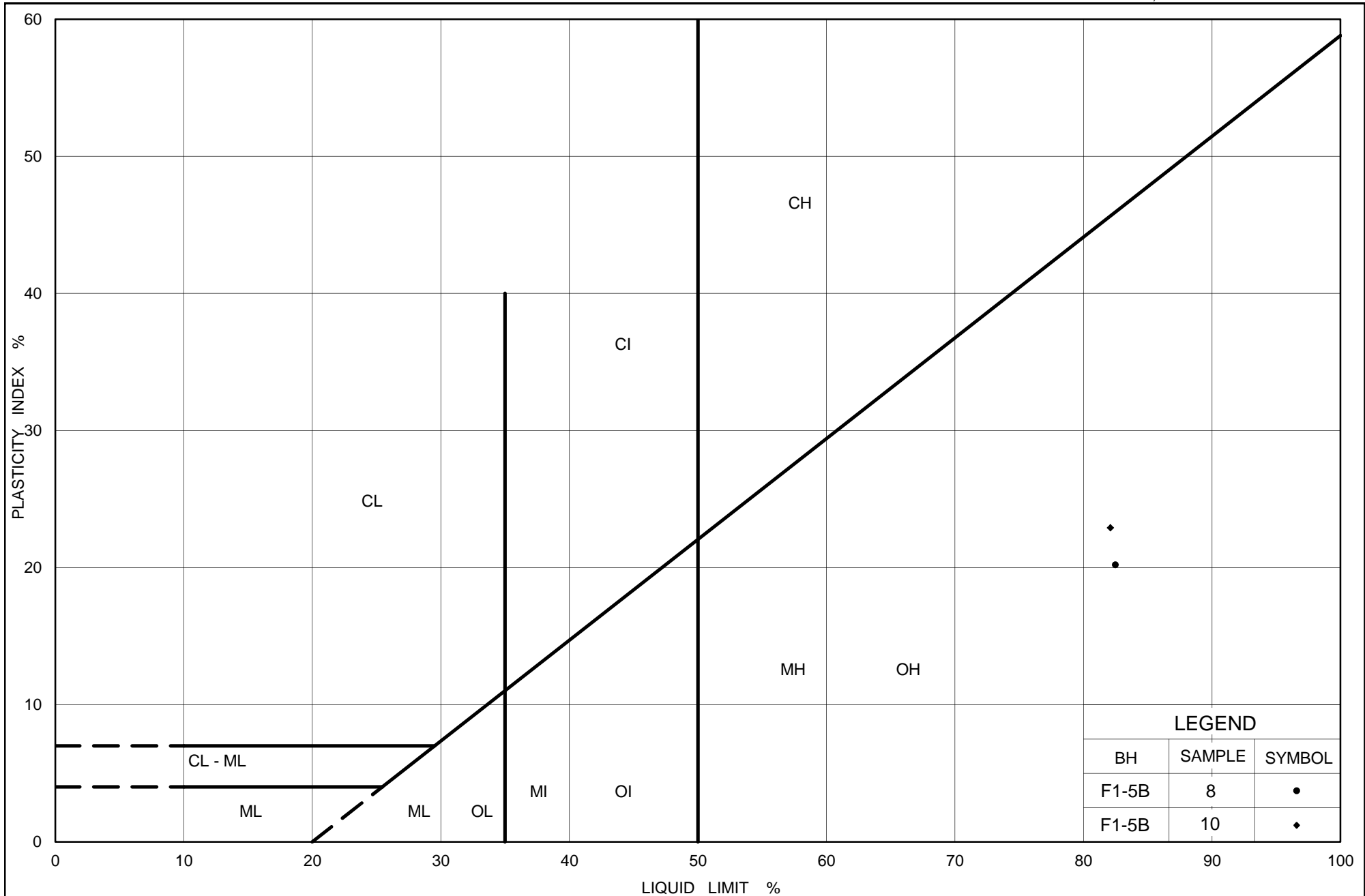
Ontario

PLASTICITY CHART Organic Silt to Organic Clayey Silt

Figure No. A-11B

Project No. 09-1111-0018

Checked By: TWB



Ministry of Transportation

Ontario

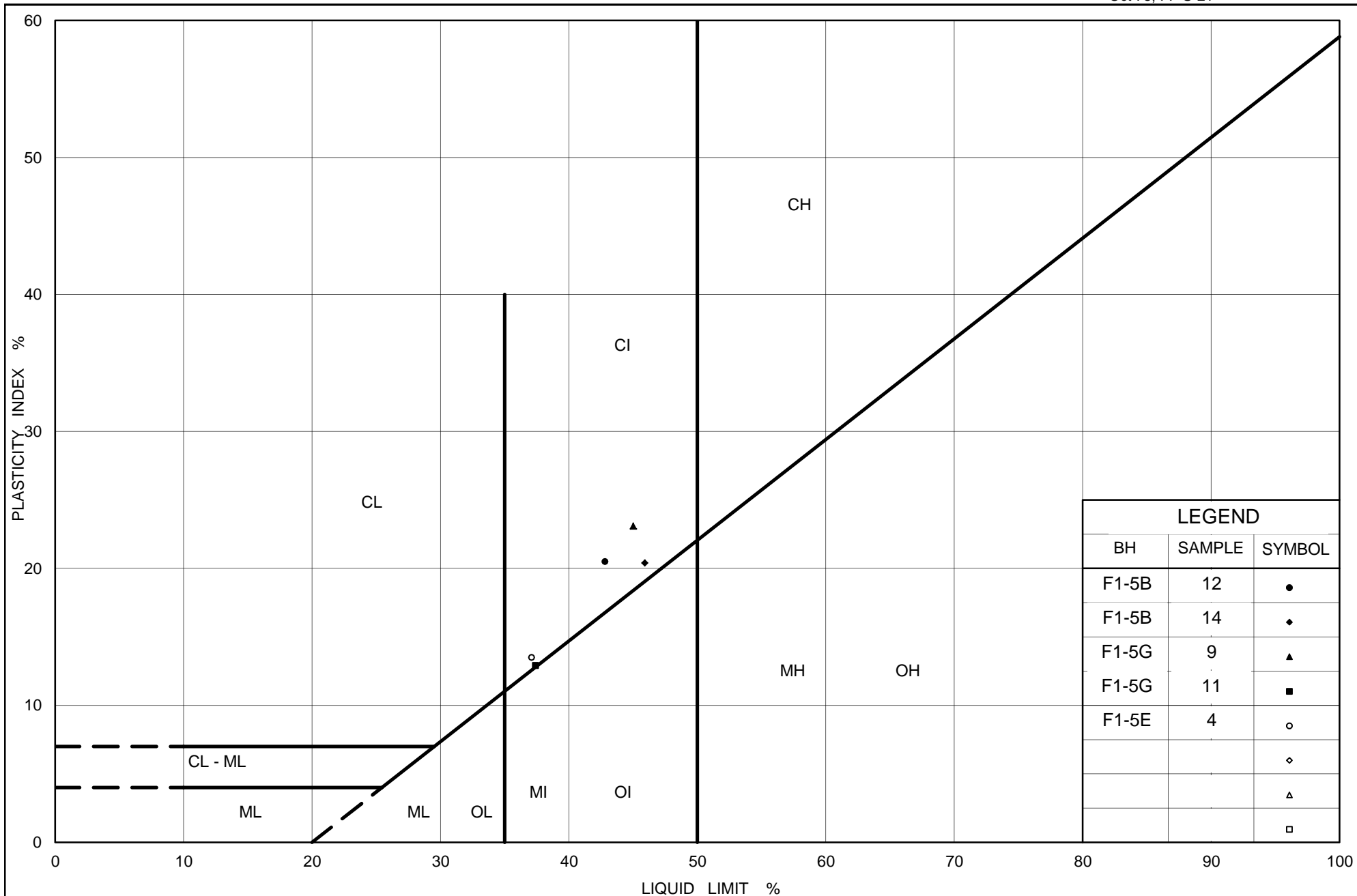
PLASTICITY CHART

Organic Sandy Silt

Figure No. A-11C

Project No. 09-1111-0018

Checked By: TWB



Ministry of Transportation

Ontario

PLASTICITY CHART Silty Clay, Trace Organics

Figure No. A-11D

Project No. 09-1111-0018

Checked By: TWB

CONSOLIDATION TEST SUMMARY**FIGURE A-12****SAMPLE IDENTIFICATION**

Project Number	09-1111-0018	Sample Number	12
Borehole Number	F1-5B	Sample Depth, m	8.38-8.91

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	8		
Date Started	11/30/2010		
Date Completed	12/14/2010		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.91	Unit Weight, kN/m ³	17.33
Sample Diameter, cm	6.32	Dry Unit Weight, kN/m ³	12.17
Area, cm ²	31.40	Specific Gravity, measured	2.75
Volume, cm ³	59.97	Solids Height, cm	0.862
Water Content, %	42.39	Volume of Solids, cm ³	27.07
Wet Mass, g	105.98	Volume of Voids, cm ³	32.91
Dry Mass, g	74.43	Degree of Saturation, %	95.9

TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t ₉₀ sec	c _v cm ² /s	m _v m ² /kN	k cm/s
0.00	1.910	1.216	1.910				
4.97	1.905	1.210	1.908	18	4.29E-02	5.27E-04	2.21E-06
10.00	1.893	1.196	1.899	305	2.51E-03	1.30E-03	3.20E-07
20.00	1.871	1.171	1.882	327	2.30E-03	1.12E-03	2.52E-07
40.00	1.837	1.131	1.854	240	3.04E-03	9.01E-04	2.68E-07
80.00	1.776	1.061	1.807	360	1.92E-03	7.89E-04	1.49E-07
160.00	1.700	0.972	1.738	290	2.21E-03	4.99E-04	1.08E-07
320.00	1.619	0.878	1.659	202	2.89E-03	2.67E-04	7.56E-08
640.00	1.537	0.783	1.578	178	2.96E-03	1.34E-04	3.88E-08
1280.00	1.461	0.695	1.499	109	4.37E-03	6.22E-05	2.66E-08
2560.00	1.384	0.605	1.422	94	4.56E-03	3.15E-05	1.41E-08
1280.00	1.393	0.616	1.388				
320.00	1.416	0.643	1.405				
80.00	1.446	0.678	1.431				
20.00	1.482	0.720	1.464				
4.98	1.525	0.769	1.503				

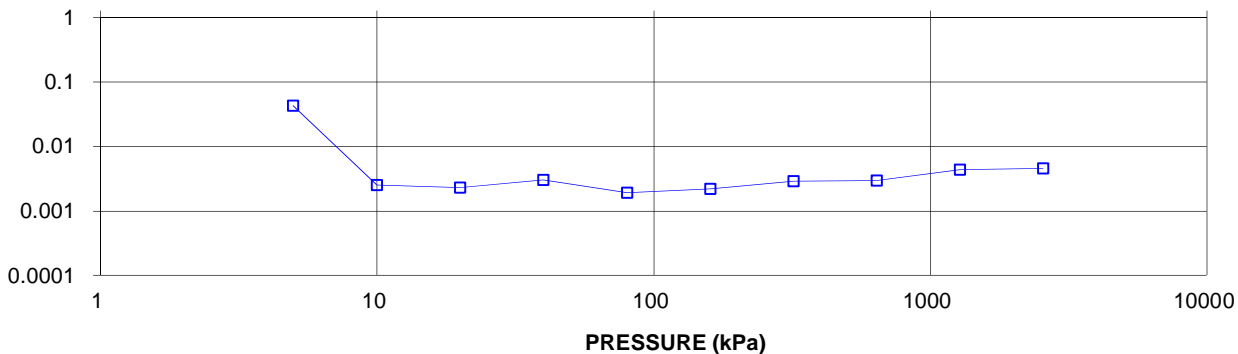
Note:
k calculated using cv based on t₉₀ values.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.52	Unit Weight, kN/m ³	19.78
Sample Diameter, cm	6.32	Dry Unit Weight, kN/m ³	15.25
Area, cm ²	31.40	Specific Gravity, measured	2.75
Volume, cm ³	47.87	Solids Height, cm	0.862
Water Content, %	29.72	Volume of Solids, cm ³	27.07
Wet Mass, g	96.55	Volume of Voids, cm ³	20.80
Dry Mass, g	74.43		

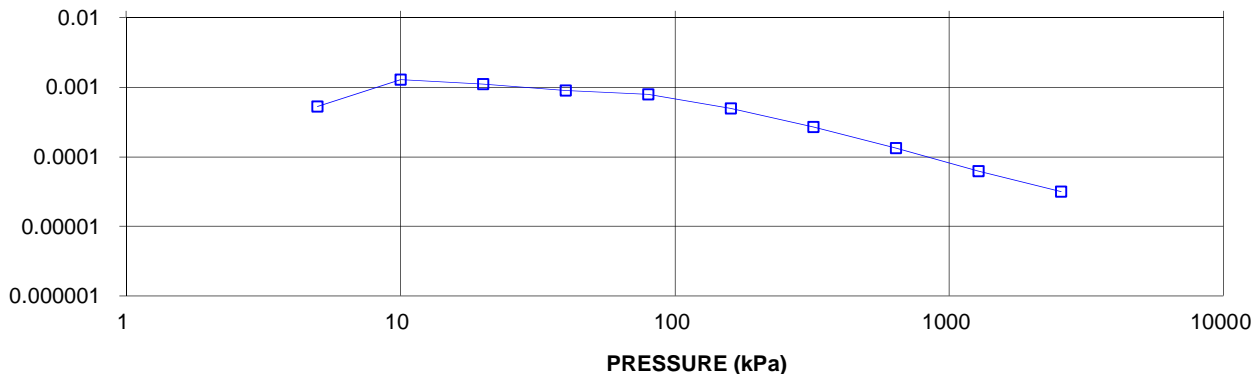
COEFFICIENT OF CONSOLIDATION,
cm²/s

CONSOLIDATION TEST
C_v cm²/s VS PRESSURE (kPa)
BH F1-5B SA 12



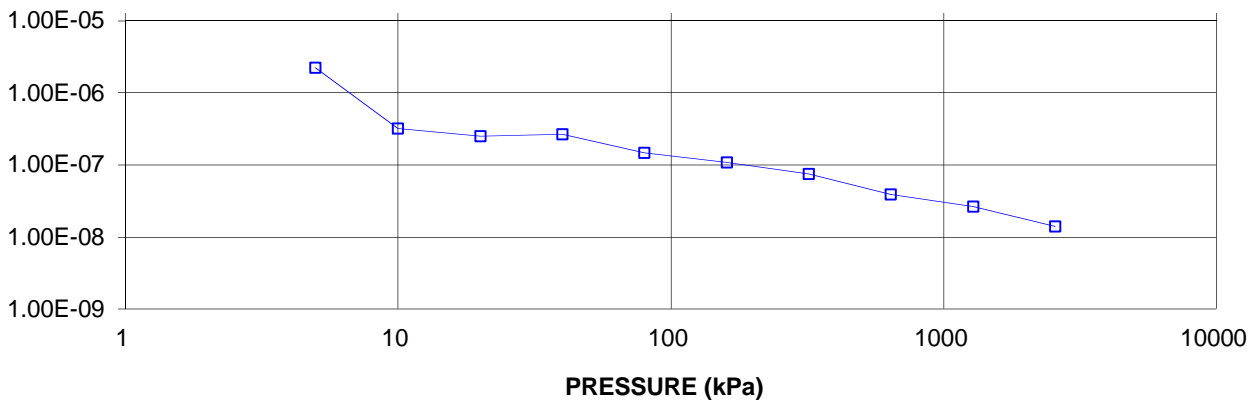
VOLUME COMPRESSIBILITY, m²/kN

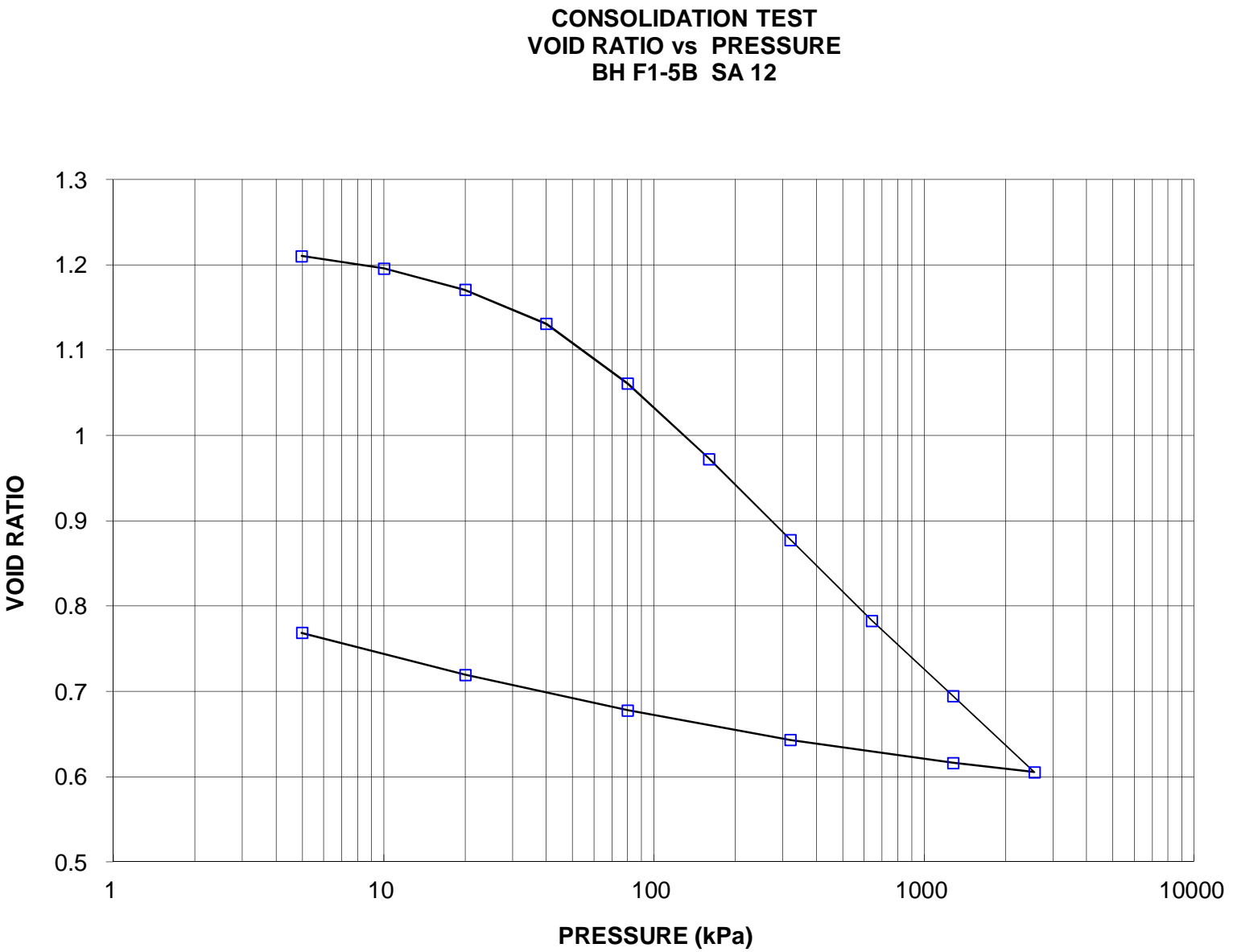
CONSOLIDATION TEST
M_v m²/kN vs PRESSURE (kPa)
BH F1-5B SA 12

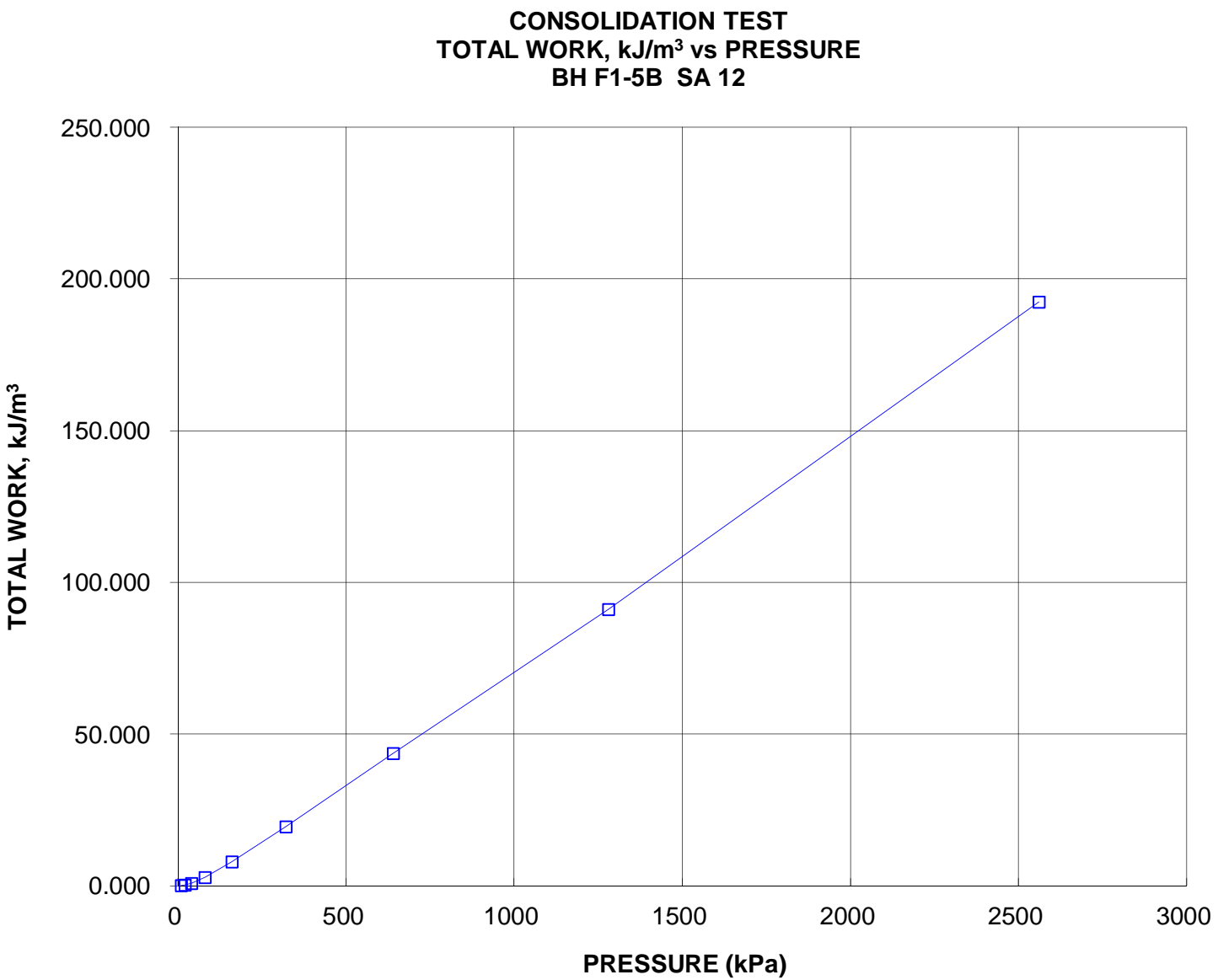


HYDRAULIC CONDUCTIVITY,
cm/s

CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BH F1-5B SA 12







CONSOLIDATION TEST SUMMARY**ASTM D2435/D2435M****FIGURE A-13****SAMPLE IDENTIFICATION**

Project Number	09-1111-0018	Sample Number	8
Borehole Number	F1-5D	Sample Depth, m	6.10-6.71

TEST CONDITIONS

Test Type	Laboratory Standard	Load Duration, hr	24
Oedometer Number	7		
Date Started	11/10/2015		
Date Completed	11/30/2015		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.89	Unit Weight, kN/m ³	17.81
Sample Diameter, cm	6.31	Dry Unit Weight, kN/m ³	12.22
Area, cm ²	31.23	Specific Gravity, measured	2.72
Volume, cm ³	58.90	Solids Height, cm	0.864
Water Content, %	45.77	Volume of Solids, cm ³	26.97
Wet Mass, g	106.95	Volume of Voids, cm ³	31.93
Dry Mass, g	73.37	Degree of Saturation, %	105.2

TEST COMPUTATIONS

Stress	Corr. Height	Void	Average Height	t ₉₀	cv.	mv	k
kPa	cm	Ratio	cm	sec	cm ² /s	m ² /kN	cm/s
0.00	1.886	1.184	1.886				
5.80	1.865	1.159	1.875	2774	2.69E-04	1.97E-03	5.18E-08
10.70	1.858	1.151	1.861	3840	1.91E-04	7.57E-04	1.42E-08
20.59	1.828	1.116	1.843	3937	1.83E-04	1.60E-03	2.86E-08
40.26	1.778	1.059	1.803	3330	2.07E-04	1.34E-03	2.72E-08
79.50	1.707	0.976	1.742	3745	1.72E-04	9.63E-04	1.62E-08
20.56	1.723	0.994	1.715				
5.80	1.741	1.016	1.732				
20.56	1.731	1.004	1.736	470	1.36E-03	3.70E-04	4.93E-08
79.50	1.694	0.961	1.712	346	1.80E-03	3.33E-04	5.86E-08
157.79	1.628	0.885	1.661	1109	5.27E-04	4.45E-04	2.30E-08
314.53	1.554	0.799	1.591	1033	5.19E-04	2.52E-04	1.28E-08
627.83	1.484	0.718	1.519	645	7.58E-04	1.18E-04	8.80E-09
1255.23	1.414	0.637	1.449	487	9.14E-04	5.87E-05	5.25E-09
2509.74	1.335	0.546	1.375	205	1.95E-03	3.33E-05	6.37E-09
1255.23	1.349	0.562	1.342				
314.53	1.376	0.593	1.363				
79.50	1.409	0.632	1.392				
20.56	1.450	0.678	1.429				
5.80	1.489	0.724	1.469				

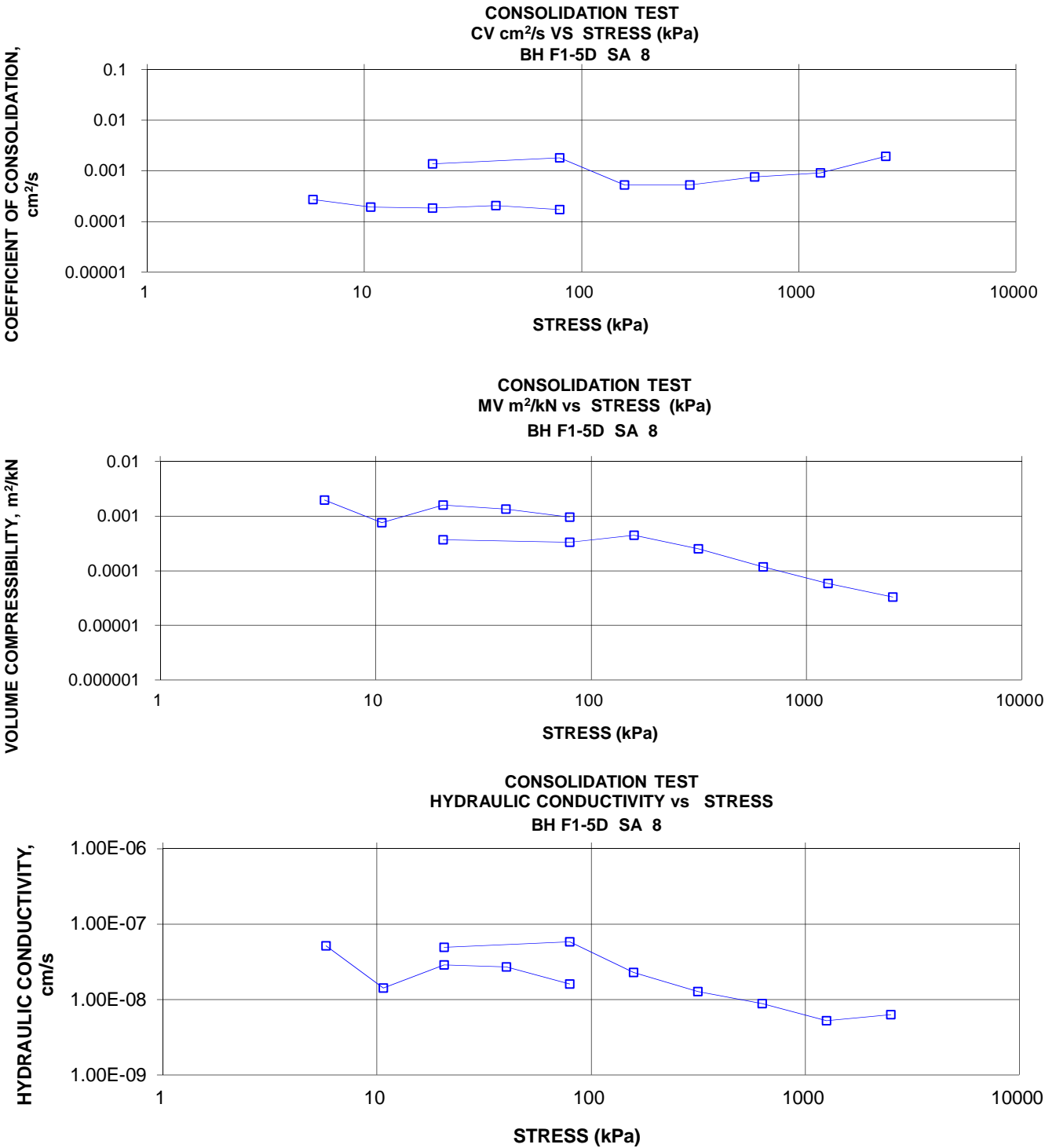
Note:

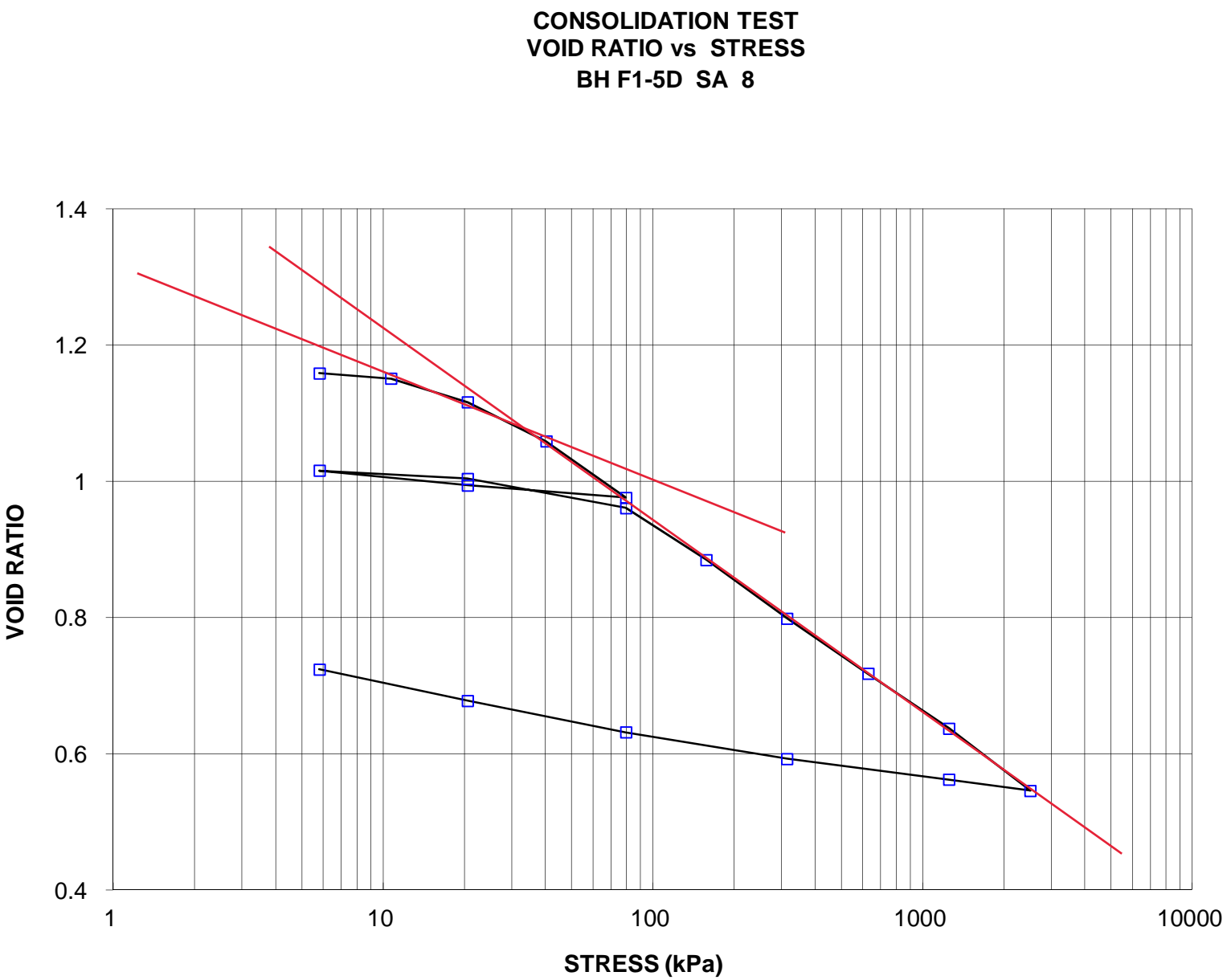
Consolidation loading and unloading schedule assigned by the client.

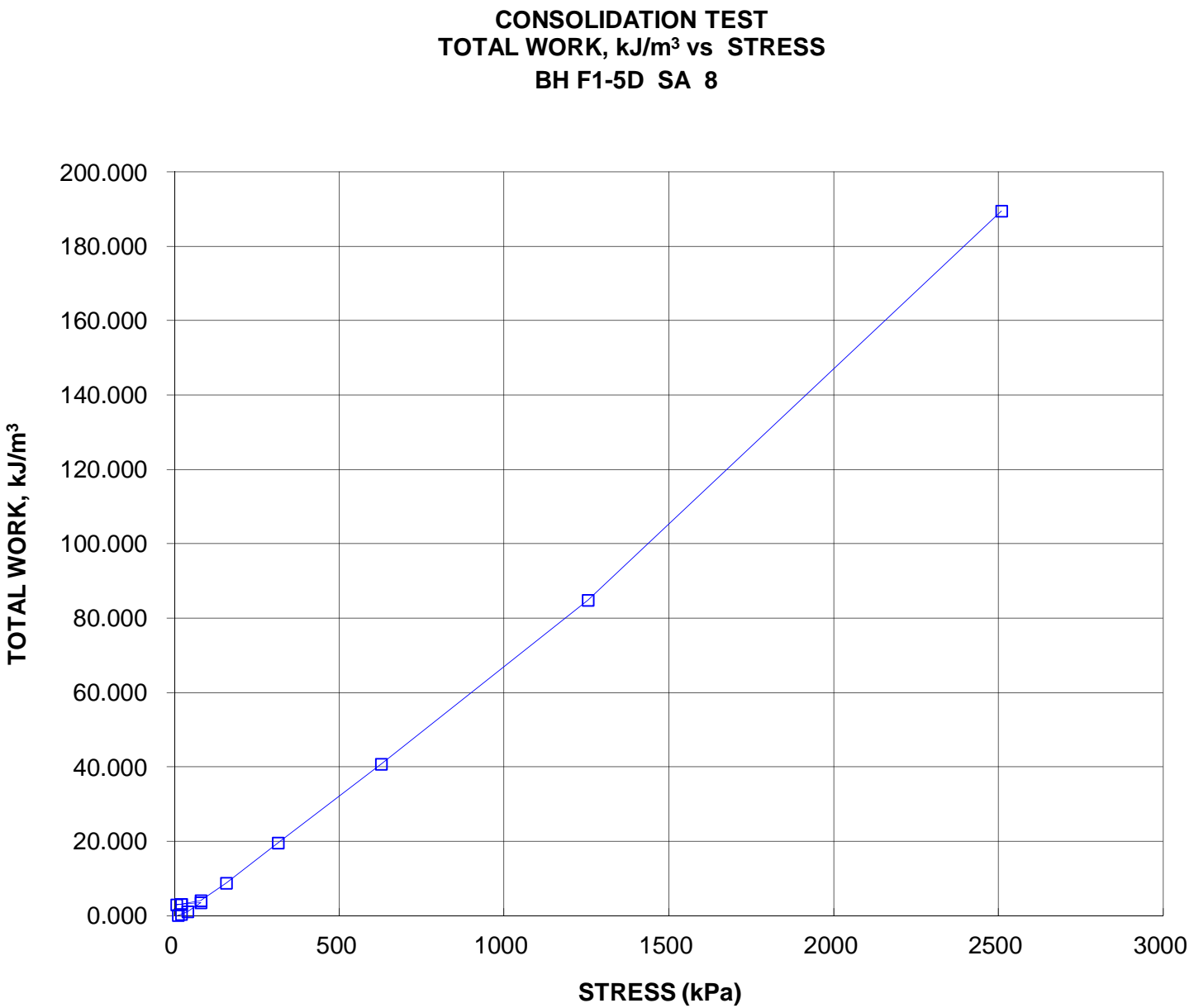
Specimen taken 23-33 cm from bottom of the tube

k calculated using cv based on t₉₀ values.**SAMPLE DIMENSIONS AND PROPERTIES - FINAL**

Sample Height, cm	1.49	Unit Weight, kN/m ³	20.08
Sample Diameter, cm	6.31	Dry Unit Weight, kN/m ³	15.47
Area, cm ²	31.23	Specific Gravity, measured	2.72
Volume, cm ³	46.50	Solids Height, cm	0.864
Water Content, %	29.81	Volume of Solids, cm ³	26.97
Wet Mass, g	95.24	Volume of Voids, cm ³	19.53
Dry Mass, g	73.37		



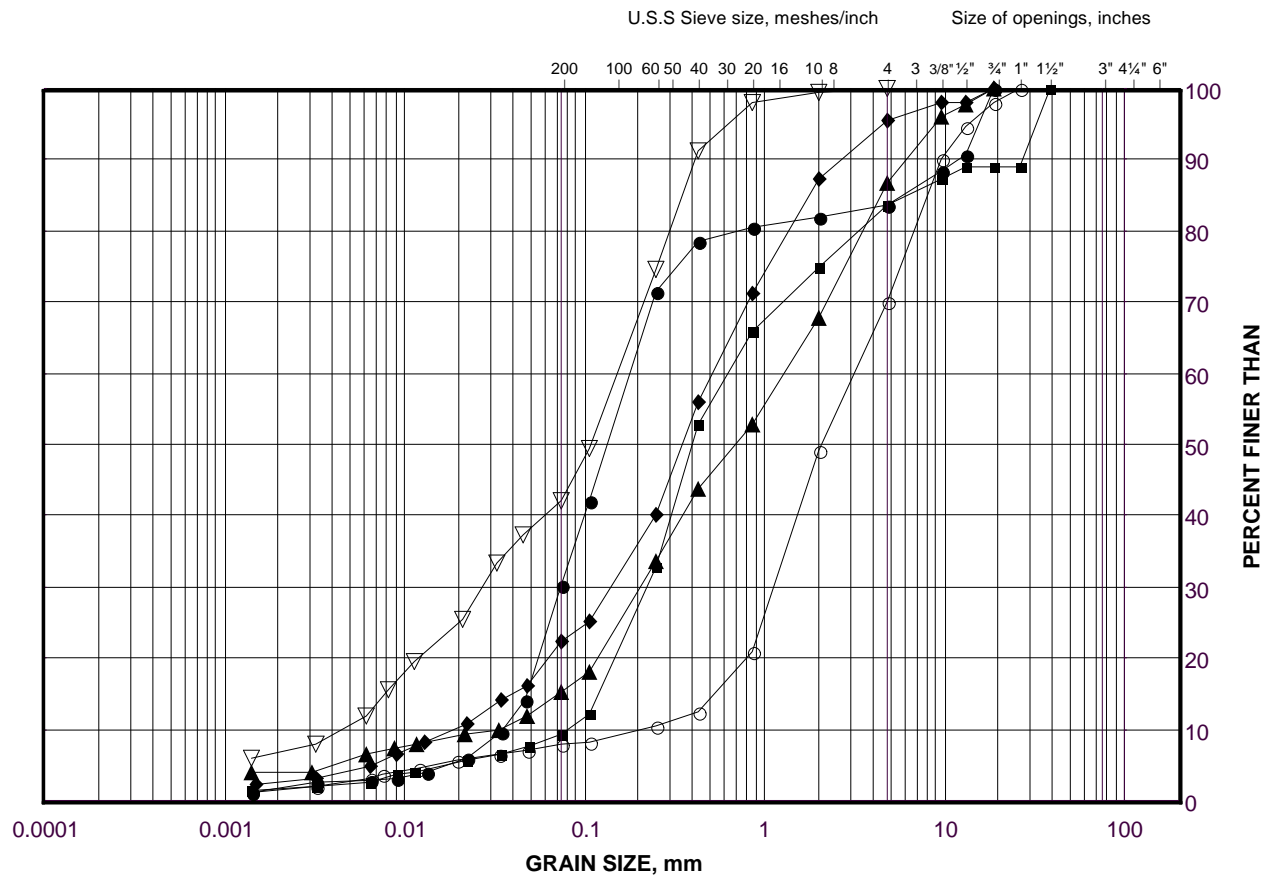




GRAIN SIZE DISTRIBUTION

Silty Sand to Sand and Gravel

FIGURE A-14



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	F1-5D	10	289.0
■	F1-5A	11	288.5
◆	F1-5G	13	284.8
▲	F1-5B	16	285.1
▽	F1-5A	8	293.1
○	F1-5A	9	291.6

Project Number: 09-1111-0018

Checked By: TWB

Golder Associates

Date: 06-Jan-16

APPENDIX B

**2016 Investigation – Borehole and
Test Pit Records**

ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS

MINISTRY OF TRANSPORTATION, ONTARIO

PARTICLE SIZES OF CONSTITUENTS

Soil Constituent	Particle Size Description	Millimetres	Inches (US Std. Sieve Size)
BOULDERS	Not Applicable	>300	>12
COBBLES	Not Applicable	75 to 300	3 to 12
GRAVEL	Coarse	19 to 75	0.75 to 3
	Fine	4.75 to 19	(4) to 0.75
	Coarse Medium Fine	2.00 to 4.75 0.425 to 2.00 0.075 to 0.425	(10) to (4) (40) to (10) (200) to (40)
FINES	Classified by plasticity	<0.075	< (200)

MODIFIERS FOR SECONDARY COMPONENTS^{1,2}

Percentage by Mass	Modifier
> 35	Use 'and' to combine primary and secondary component (<i>i.e.</i> , SAND and gravel)
> 20 to 35	Primary soil name prefixed with "gravelly, sandy" as applicable
> 10 to 20	some (<i>i.e.</i> , some sand)
≤ 10	trace (<i>i.e.</i> , trace fines)

1. Only applicable to components not described by Primary Group Name.

2. Classification of Primary Group Name based on Unified Soil Classification System (ASTM D2487) for coarse-grained soils; fine-grained soils described per current MTO Soil Classification System.

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.) split-spoon sampler for a distance of 300 mm (12 in.). Values reported are as recorded in the field and are uncorrected.

Cone Penetration Test (CPT)

An electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (*q_t*), porewater pressure (*u*) and sleeve friction (*f_s*) are recorded electronically at 25 mm penetration intervals.

Dynamic Cone Penetration Resistance (DCPT); 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

SAMPLES

AS	Auger sample
BS	Block sample
CS	Chunk sample
DD	Diamond Drilling
DO or DP	Seamless open ended, driven or pushed tube sampler – note size
DS	Denison type sample
GS	Grab Sample
MC	Modified California Samples
MS	Modified Shelby (for frozen soil)
RC / SC	Rock core / Soil core
SS	Split spoon sampler – note size
ST	Slotted tube
TO	Thin-walled, open – note size (Shelby tube)
TP	Thin-walled, piston – note size (Shelby tube)
WS	Wash sample
OD / ID	Outer Diameter / Inner Diameter
HSA / SSA	Hollow-Stem Augers / Solid-Stem Augers

SOIL TESTS

w	water content
PL, w _p	plastic limit
LL, w _L	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D _R	relative density (specific gravity, G _s)
DS	direct shear test
GS	specific gravity
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 (FV)	field vane (LV-laboratory vane test)
Y	unit weight

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

COARSE-GRAINED SOILS

Compactness¹

Term	SPT 'N' (blows/0.3m) ²
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	≥ 50

3. Definition of compactness terms are based on SPT 'N' ranges as provided in Terzaghi, Peck and Mesri (1996). Many factors affect the recorded SPT 'N' value, including hammer efficiency (which may be greater than 60% in automatic trip hammers), overburden pressure, groundwater conditions, and grain size. As such, the recorded SPT 'N' value(s) should be considered only an approximate guide to the soil compactness. These factors need to be considered when evaluating the results, and the stated compactness terms should not be relied upon for design or construction.

4. SPT 'N' in accordance with ASTM D1586, uncorrected for the effects of overburden pressure.

FINE-GRAINED SOILS

Consistency

Term	Undrained Shear Strength (kPa)	SPT 'N' ^{1,2} (blows/0.3m)
Very Soft	< 12	0 to 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	> 200	> 30

1. SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects; approximate only.

2. SPT 'N' values should be considered ONLY an approximate guide to consistency; for sensitive clays (e.g., Champlain Sea clays), the N-value approximation for consistency terms does NOT apply. Rely on direct measurement of undrained shear strength or other manual observations.

Field Moisture Condition

Term	Description
Dry	Soil flows freely through fingers.
Moist	Soils are darker than in the dry condition and may feel cool.
Wet	As moist, but with free water forming on hands when handled.

LIST OF SYMBOLS

MINISTRY OF TRANSPORTATION, ONTARIO

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

I. GENERAL

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)

σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
U	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

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

Notes: 1
2

$$\tau = c' + \sigma' \tan \phi'$$

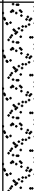

$$\text{shear strength} = (\text{compressive strength})/2$$

PROJECT		1413191 (1080)		RECORD OF BOREHOLE No 16-B1		SHEET 1 OF 2		METRIC									
G.W.P.				LOCATION		N 4867708.5; E 298984.0 MTM NAD 83 ZONE 10 (LAT. 43.949342; LONG. -79.572463)		ORIGINATED BY JIL									
DIST		Central HWY 400		BOREHOLE TYPE		Power Auger to 4.6 m; then Wash Boring		COMPILED BY ACK									
DATUM		Geodetic		DATE		August 9, 2016		CHECKED BY ARV									
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	γ	GR	SA	SI	CL	
300.4	GROUND SURFACE																
0.0	Sand, trace to some silt, trace gravel, trace organics, trace silt lenses (FILL) Loose to compact Brown to grey Moist		1	SS	16		300										
			2	SS	9												
299.0			3	SS	50/0.10		299		○							39	42 17 2
1.4	Sand and gravel, some silt, trace clay, trace rootlets (FILL) Dense Brown to grey Moist to wet																
298.4			4	SS	14		298		○								
2.0	SAND, some silt, trace clay, trace clayey silt lenses Loose to compact Grey Wet		5	SS	7		297		○								
			6	SS	8				○								
295.9							296										
4.5	CLAYEY SILT, trace organics and silt lenses Soft Grey to dark grey Wet		7	SS	3					○							
294.9			8A				295										
5.5	Clayey ORGANIC SILT and SAND to PEAT, trace rootlets, roots and shell pieces Very loose/soft Black Wet		8B	SS	4							160	○				
			9	SS	4		294					136	○				
			10	SS	2		293					106	○				
			11	SS	2							157	○				
291.9							292									10	42 43 5
8.5	SILTY CLAY, trace organics and shells Soft Grey to dark grey Moist to wet		12A									160	○				
			12B	SS	4								○				
			13	SS	4		291										
			14	SS	3		290										
			15	SS	4												
			16	SS	4		289										
288.3																	
12.1	SAND, trace to some silt, trace to some gravel, trace clay Very loose to compact Grey Wet		17	SS	1		288		○							2	83 13 2
							287										
			18	SS	16		286										
285.4																	

Continued Next Page

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

GTA-MTO 001 S:\CLIENTS\MTOWHY_400\02_DATA\GINT\GEOTECH_INVEST_A#8_HWY_400_EMBANKMENT\HWY_400_1413191_1080.GPJ GAL-GTA.GDT 3/16/22

PROJECT 1413191 (1080)				RECORD OF BOREHOLE No 16-B1				SHEET 2 OF 2				METRIC													
G.W.P.				LOCATION N 4867708.5; E 298984.0 MTM NAD 83 ZONE 10 (LAT. 43.949342; LONG. -79.572463)				ORIGINATED BY JIL																	
DIST Central HWY 400				BOREHOLE TYPE Power Auger to 4.6 m; then Wash Boring				COMPILED BY ACK																	
DATUM Geodetic				DATE August 9, 2016				CHECKED BY ARV																	
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)								
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W _p	W	W _L										
--- CONTINUED FROM PREVIOUS PAGE ---																									
15.0	SAND, some gravel, trace to some silt, trace clay Very loose to compact Grey Wet		19	SS	7		285																		
								284																	
								283																	
								282																	
281.5	- Clayey silt lenses at a depth of 18.7 m		20	SS	18																				
18.9	END OF BOREHOLE Dynamic Cone Penetration Test (DCPT)		21	SS	15*																				
279.4																									
21.0	END OF DCPT																								
NOTE: 1. Water level measurement in piezometer: <table border="1" style="margin: 10px auto; border-collapse: collapse;"> <thead> <tr> <th>Date</th> <th>Depth (m)</th> <th>Elev. (m)</th> </tr> </thead> <tbody> <tr> <td>08/15/2016</td> <td>1.8</td> <td>298.6</td> </tr> <tr> <td>11/17/2016</td> <td>2.3</td> <td>298.1</td> </tr> </tbody> </table> * Sand blow-back was noted in the borehole; N-value may have been impacted by groundwater.																	Date	Depth (m)	Elev. (m)	08/15/2016	1.8	298.6	11/17/2016	2.3	298.1
Date	Depth (m)	Elev. (m)																							
08/15/2016	1.8	298.6																							
11/17/2016	2.3	298.1																							

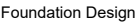
GTA-MTO 001 S:\CLIENTS\MTOWHY_400\02_DATA\GINT\GEOTECH_INVEST_A#8_HWY_400_EMBANKMENT\HWY_400_1413191_1080.GPJ GAL-GTA.GDT 3/16/22

PROJECT		RECORD OF BOREHOLE		No 16-B2		SHEET 1 OF 2		METRIC											
1413191 (1080)		LOCATION		N 4867709.5; E 298983.8 MTM NAD 83 ZONE 10 (LAT. 43.949351; LONG. -79.572465)		ORIGINATED BY		JIL											
DIST Central HWY 400		BOREHOLE TYPE		210 mm O.D., 133 mm I.D. Continuous Flight Hollow Stem Augers		COMPILED BY		ACK											
DATUM Geodetic		DATE		August 12, 2016		CHECKED BY		ARV											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40						60	80	100	20	40
300.4	GROUND SURFACE						300												
0.0	No soil sample taken; refer to Borehole 16-B1																		
295.8							296												
4.7	Silty SAND Grey Wet		1	SS	3														
	Clayey ORGANIC SILT to SILTY PEAT Soft Dark grey Wet		2	TO	PH														
			3	TO	PH														
			4	TO	PH														
292.2							292												
8.2	SILTY CLAY to CLAY, trace organics Grey Wet		5	TO	PH														
			6	TO	PH														
			7	TO	PH														
			8	TO	PH														
			9	TO	PH														
			10	TO	PH														
286.4			11A				287												
14.0	SAND, trace silt to silty, trace silty clay lenses Very loose Grey Wet		11B	TO	PH														
			12	SS	3		286												
285.5																			

Continued Next Page

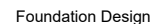
+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

GTA-MTO 001 S:\CLIENTS\TOHWY_400\02_DATA\GINT\GEOTECH_INVEST_A#8_HWY_400_EMBANKMENT\HWY_400_1413191_1080.GPJ GAL-GTA.GDT 10/01/19



GTA-MTO	001	S:CLIENTS	MTO\HWY	400\02	DATA\GINT\GEOTECH	INVEST	A#8	HWY	400	EMBANKMENT\HWY	400	1413191	1080	GPJ	GAL-GTA	GDT	10/01/19
---------	-----	-----------	---------	--------	-------------------	--------	-----	-----	-----	----------------	-----	---------	------	-----	---------	-----	----------

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



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

PROJECT <u>1413191 (1080)</u>		RECORD OF BOREHOLE No 16-M1				SHEET 2 OF 2		METRIC	
LOCATION <u>N 4867712.2; E 298974.0 MTM NAD 83 ZONE 10 (LAT. 43.949376; LONG. -79.572586)</u>		ORIGINATED BY <u>JIL</u>							
DIST <u>Central</u> HWY <u>400</u>		BOREHOLE TYPE <u>Power Auger, 210 mm O.D. Hollow Stem Augers to 12.8 m then Open Hole Tricone</u>				COMPILED BY <u>ACK</u>			
DATUM <u>Geodetic</u>		DATE <u>August 15, 2016</u>				CHECKED BY <u>ARV</u>			

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa								
	--- CONTINUED FROM PREVIOUS PAGE ---															
14.9	SILTY CLAY	X														
287.4	Stiff to very stiff															
15.5	Grey Wet															
	END OF BOREHOLE															
	NOTE: 1. Water level could not be measured due to the addition of drilling fluid inside the borehole.															

GTA-MTO 001 S:\CLIENTS\TOHWY_400\02_DATA\GINT\GEOTECH_INVEST_A#8_HWY_400_EMBANKMENT\HWY_400_1413191_1080.GPJ GAL-GTA.GDT 10/01/19

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

STAM-TO 001 S:\CLIENTS\TOHWY 400\02 DATA\GIN\GEO\TECH INVEST A#8 HWY 400 EMBANKMENT\HWY 400 1413191 1080.GPJ GAL-GTA.GDT 3/16/22

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

PROJECT 1413191 (1080)			RECORD OF BOREHOLE No 16-T1			SHEET 2 OF 2			METRIC								
G.W.P.			LOCATION N 4867726.5; E 298955.8 MTM NAD 83 ZONE 10 (LAT. 43.949504; LONG. -79.572814)			ORIGINATED BY JIL											
DIST Central HWY 400			BOREHOLE TYPE 210 mm O.D. Continuous Flight Hollow Stem Augers to 15.2m; then Wash Boring			COMPILED BY ACK											
DATUM Geodetic			DATE August 8, 2016			CHECKED BY ARV											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
	--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100					
	SAND, some silt, trace clay, trace gravel Very loose to compact Grey Wet		16	SS	1*		294										
			17	SS	12												
			18A	SS	10		293										
292.5			18B	SS													
17.2	Sandy Clayey ORGANIC SILT, trace gravel Stiff Grey Wet						292										
291.9																	
17.8	Sandy SILT, trace to some clay Dense Grey Wet		19	SS	35		291										0 22 67 11
289.7			20A	SS			290										0 30 63 7
289.3			20B	SS	41												
20.4	SAND, trace silt Dense Grey Wet END OF BOREHOLE																
NOTE: 1. Water level could not be measured upon completion of drilling due to the addition of water for drilling below a depth of 15.2 m. * Sand blow-back was noted in the borehole; N-value may have been impacted.																	

GTA-MTO 001 S:\CLIENTS\MTOWHY_400\02_DATA\GINT\GEOTECH_INVEST_A#8_HWY_400_EMBANKMENT\HWY_400_1413191_1080.GPJ GAL-GTA.GDT 3/16/22

RECORD OF TEST PIT 1

Job Number: 1413191 – Ph1080

Project Name: Highway 400 NBL Ground Improvement

Test Pit Number: TP1

Test Pit Location: N 4867732.1 m
E 298986.9 m

Date: August 9, 2016

Logged by: MK

Approximate Test Pit Ground Elevation: 299.2 m **Test Pit Size:** 3.0 m X 2.0 m X 3.7 m

Depth (m)		Soil Description	Sample No.	Sample Depth (m)	Remarks
From	To				
0.0	0.2	Topsoil	-	-	
0.2	0.9	Silty sand (FILL) Grey Moist	-	-	
0.9	3.7	CLAYEY SILT, some sand, pockets of organics below a depth of 2.5 m Dark brown to brown, Moist to wet	1	1.0 m to 3.7 m	Water Content = 32% Organic Content = 8% Liquid Limit = 32%, Plastic Limit = 22% Plasticity Index = 10% (Refer to Figure C-4) Specific Gravity = 2.65
	3.7	End of Test Pit (Elev. 295.5 m)			

Notes:

1. Test pit was moist upon completion of excavation.
2. Side walls caving was noted.
3. Test pit backfilled upon completion.

Checked: SMM

RECORD OF TEST PIT 2

Job Number: 1413191 – Ph1080

Project Name: Highway 400 Ground Improvement

Test Pit Location: N 4867723.2 m
E 298986.5 m

Test Pit Number: TP2

Date: August 15, 2016

Logged by: ARV

Approximate Test Pit Ground Elevation: 298.8 m Test Pit Size: 3.0 m X 3.0 m X 3.5 m

Depth (m)		Soil Description	Sample No.	Sample Depth (m)	Remarks
From	To				
0.0	0.3	Topsoil	-	-	
0.3	2.1	Silty sand, some clayey silt lenses (FILL) Brown and grey Moist to wet	-	-	
2.1	3.3	Organic SILT, some sand, trace to some clay, containing roots, branches and wood pieces, Grey with oxidation stains Moist	1	0.7	Water Content = 63% Organic Content = 14% Liquid Limit = 45% Plastic Limit = 37% Plasticity Index = 8% (Refer to Figure C-8)
	3.3	End of Test Pit (Elev. 295.5 m)			

Notes:

1. Limited groundwater seepage was noted within the top 0.3 m the test pit.
2. Side walls caving was noted.
3. Grinding of the excavator bucket was noted at a depth of 1.5 m.
4. Test pit backfilled upon completion.

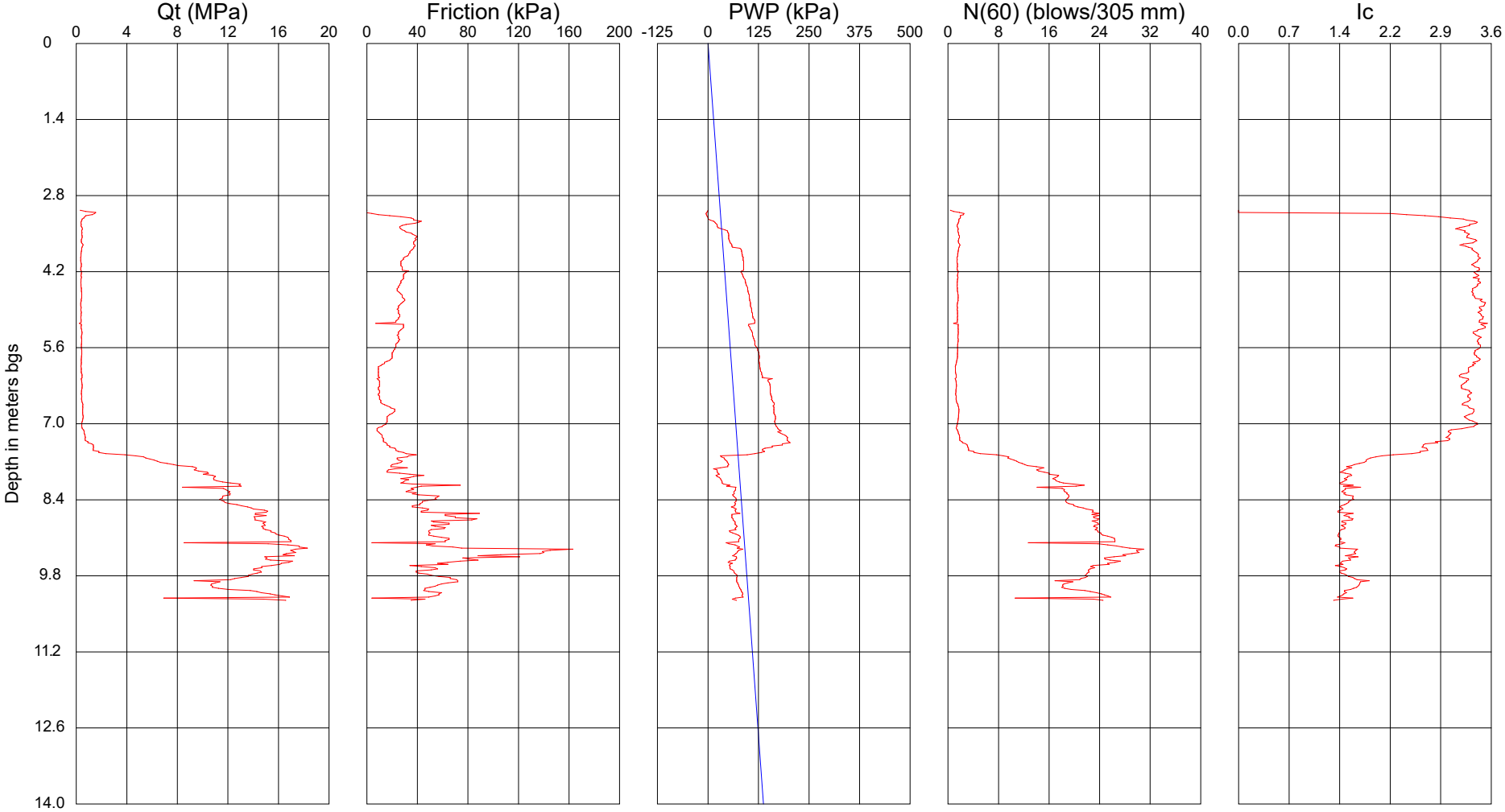
Checked: SMM

Cone Penetration Test - CPT16-1

Test Date : 2016-08-10
Location :
N 4,867,697.9, E 298,983.8
MTM NAD 83 ZONE 10

Operator : Golder Associates

Ground Surf. Elev. : 298.7
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

After Jefferies and Davies (1993)

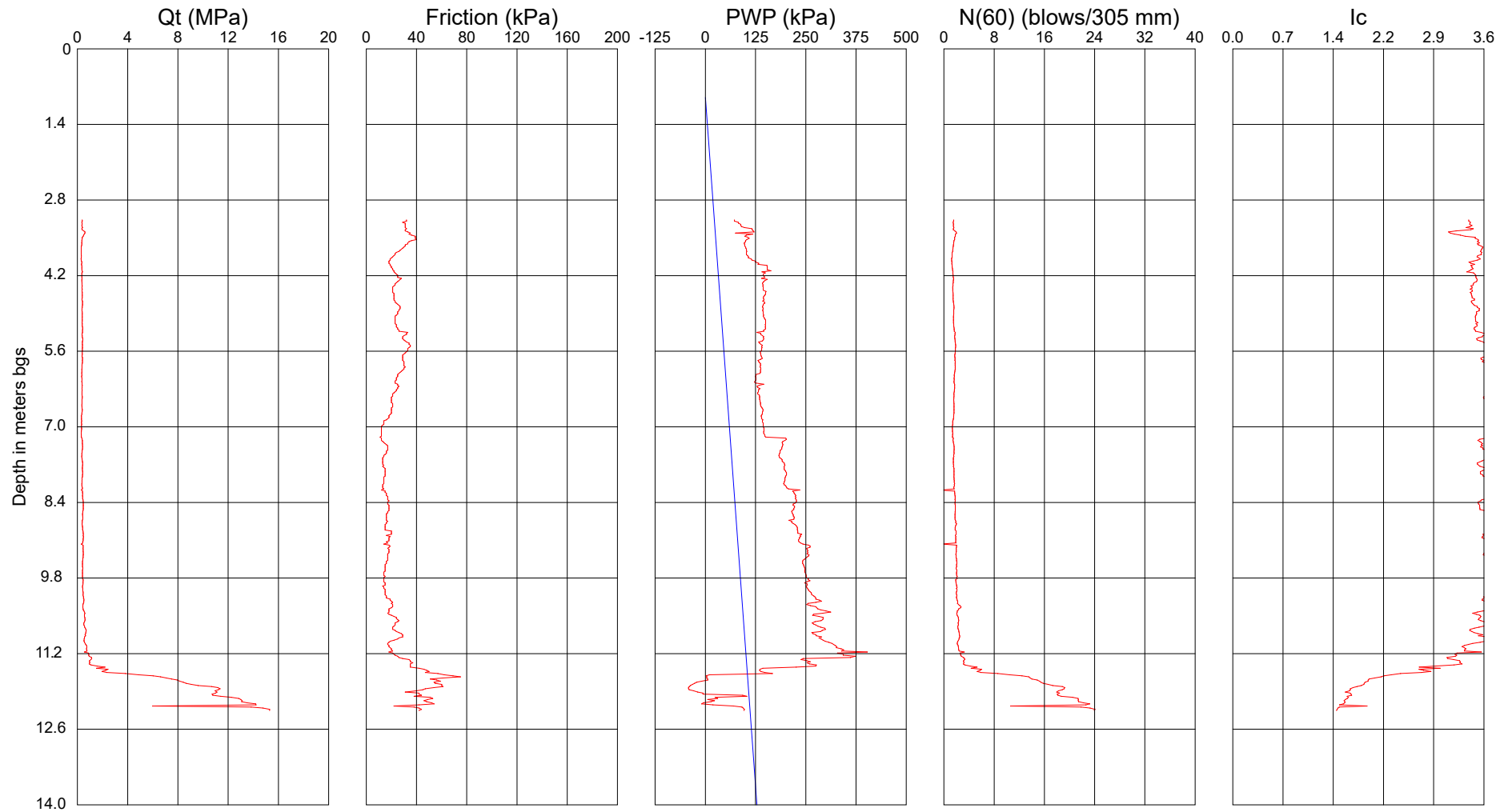
After Robertson and (Fear) Wride (1998)
Ic < 1.31 - Gravelly sands
1.31 < Ic < 2.05 - Clean to silty sand
2.05 < Ic < 2.60 - Silty sand to sandy silt
2.60 < Ic < 2.95 - Clayey silt to silty clay
2.95 < Ic < 3.60 - Clays

Cone Penetration Test - CPT16-2

Test Date : 2016-08-11
Location :
N 4,867,719.7, E 298,984.9
MTM NAD 83 ZONE 10

Operator : Golder Associates

Ground Surf. Elev. : 299.9
Water Table Depth : 0.90



Qt normalized for
unequal end area effects

After Jefferies and Davies (1993)

After Robertson and (Fear) Wride (1998)

Ic < 1.31 - Gravelly sands
1.31 < Ic < 2.05 - Clean to silty sand
2.05 < Ic < 2.60 - Silty sand to sandy silt
2.60 < Ic < 2.95 - Clayey silt to silty clay
2.95 < Ic < 3.60 - Clays

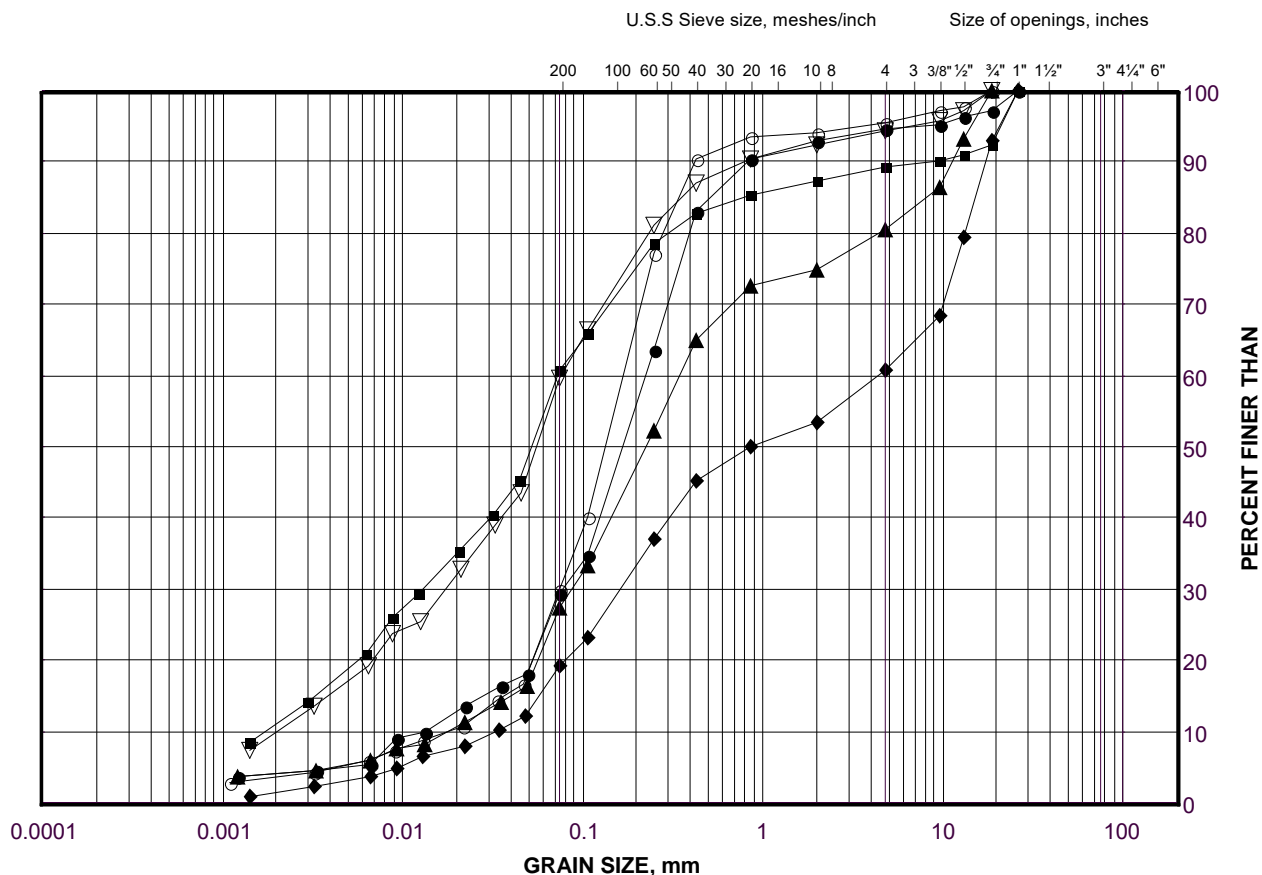
APPENDIX C

**2016 Investigation – Geotechnical
Laboratory Testing Results**

GRAIN SIZE DISTRIBUTION

Sandy Silt to Silt and Sand to Silty Sand to 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

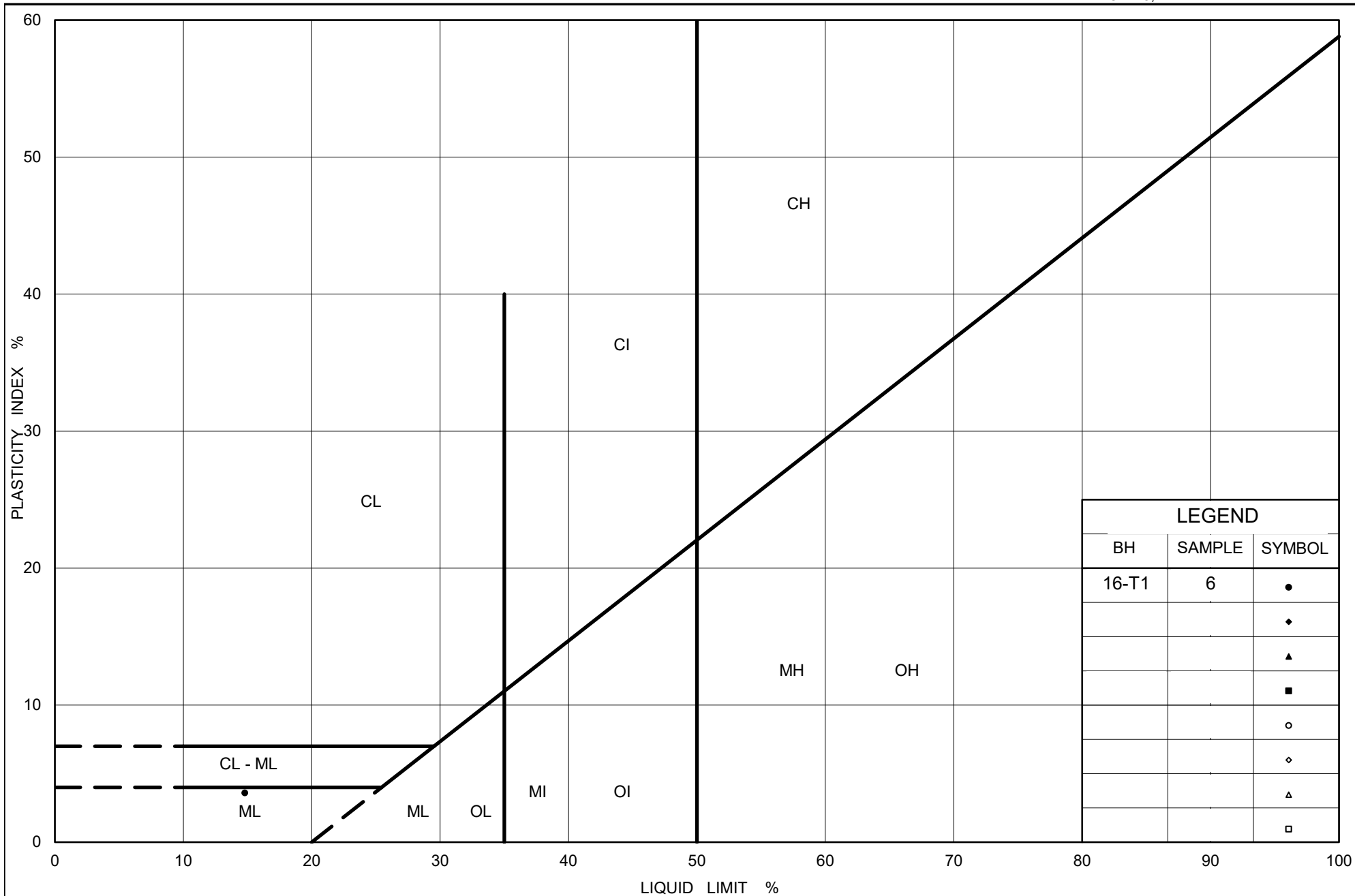
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	16-T1	11	298.0
■	16-M1	3	299.5
◆	16-B1	3	298.8
▲	16-T1	3	306.4
▽	16-M2	4	298.4
○	16-T1	8	300.2

Project Number: 1413191

Checked By: ARV

Golder Associates

Date: 24-Sep-19



Ministry of Transportation

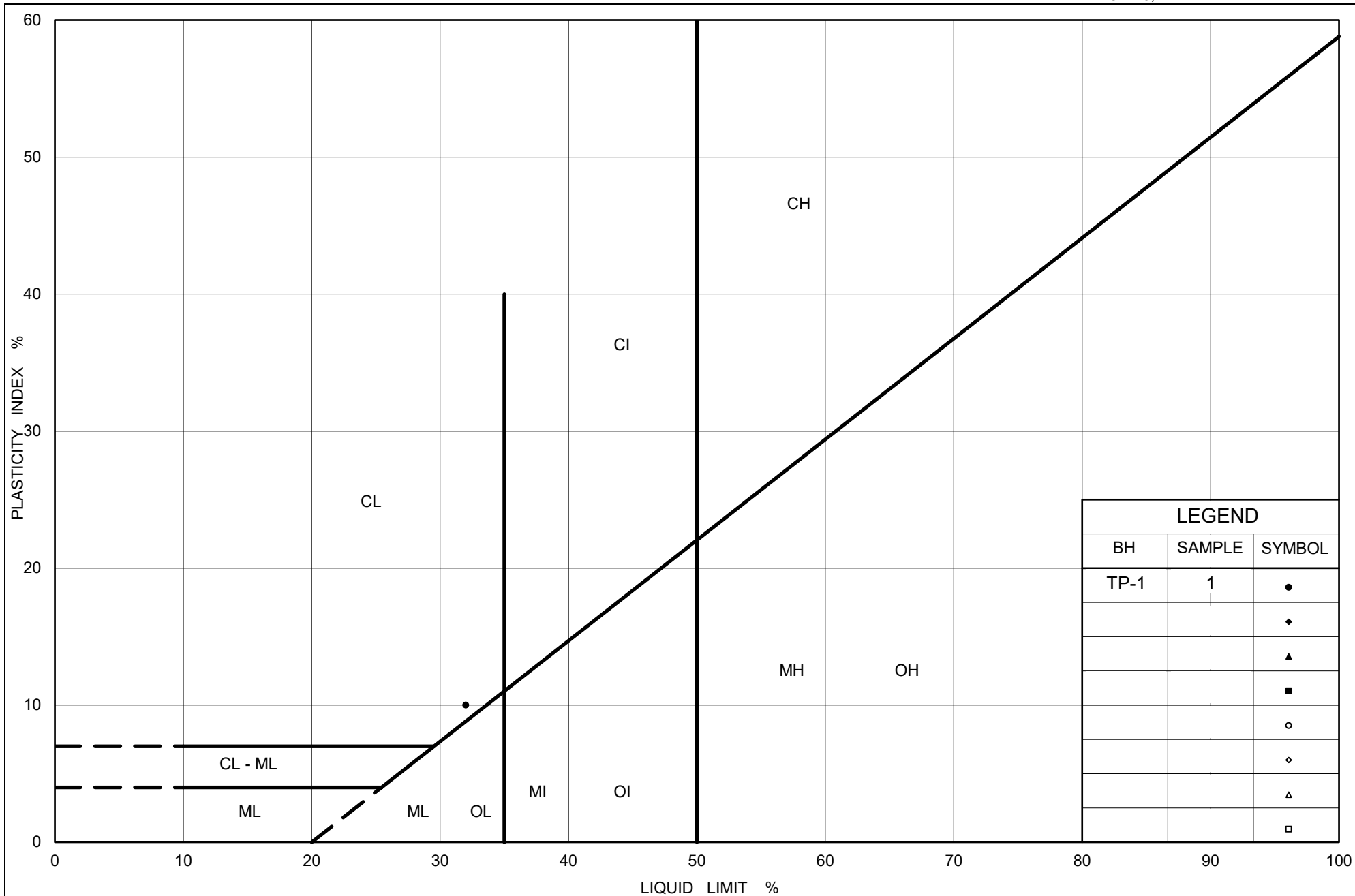
Ontario

PLASTICITY CHART Silt (Fill)

Figure No. C-2

Project No. 1413191 (1080)

Checked By: ARV



Ministry of Transportation

Ontario

PLASTICITY CHART Sandy Clayey Silt

Figure No. C-3

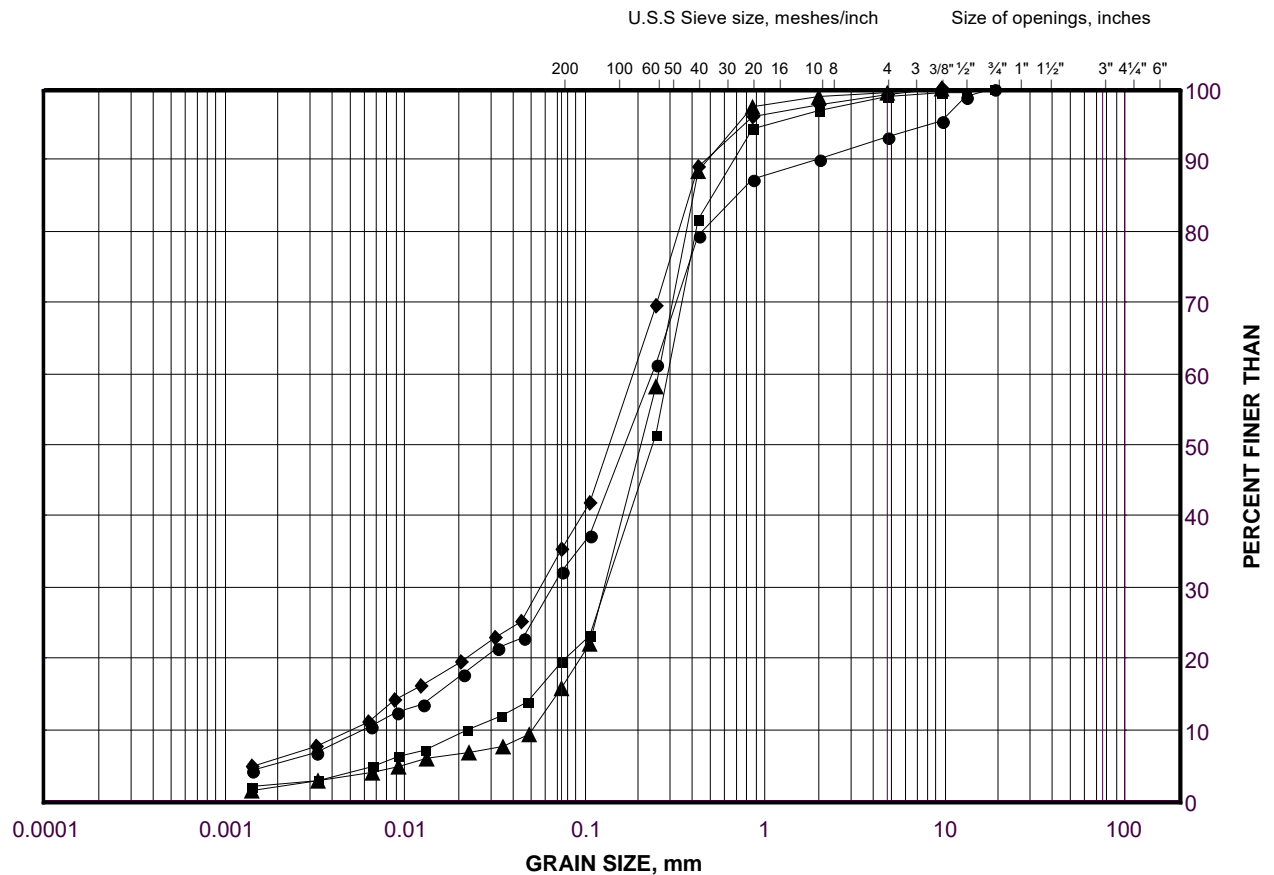
Project No. 1413191 (1080)

Checked By: ARV

GRAIN SIZE DISTRIBUTION

Sandy Silt to Silty Sand to Sand - Upper

FIGURE C-4



LEGEND

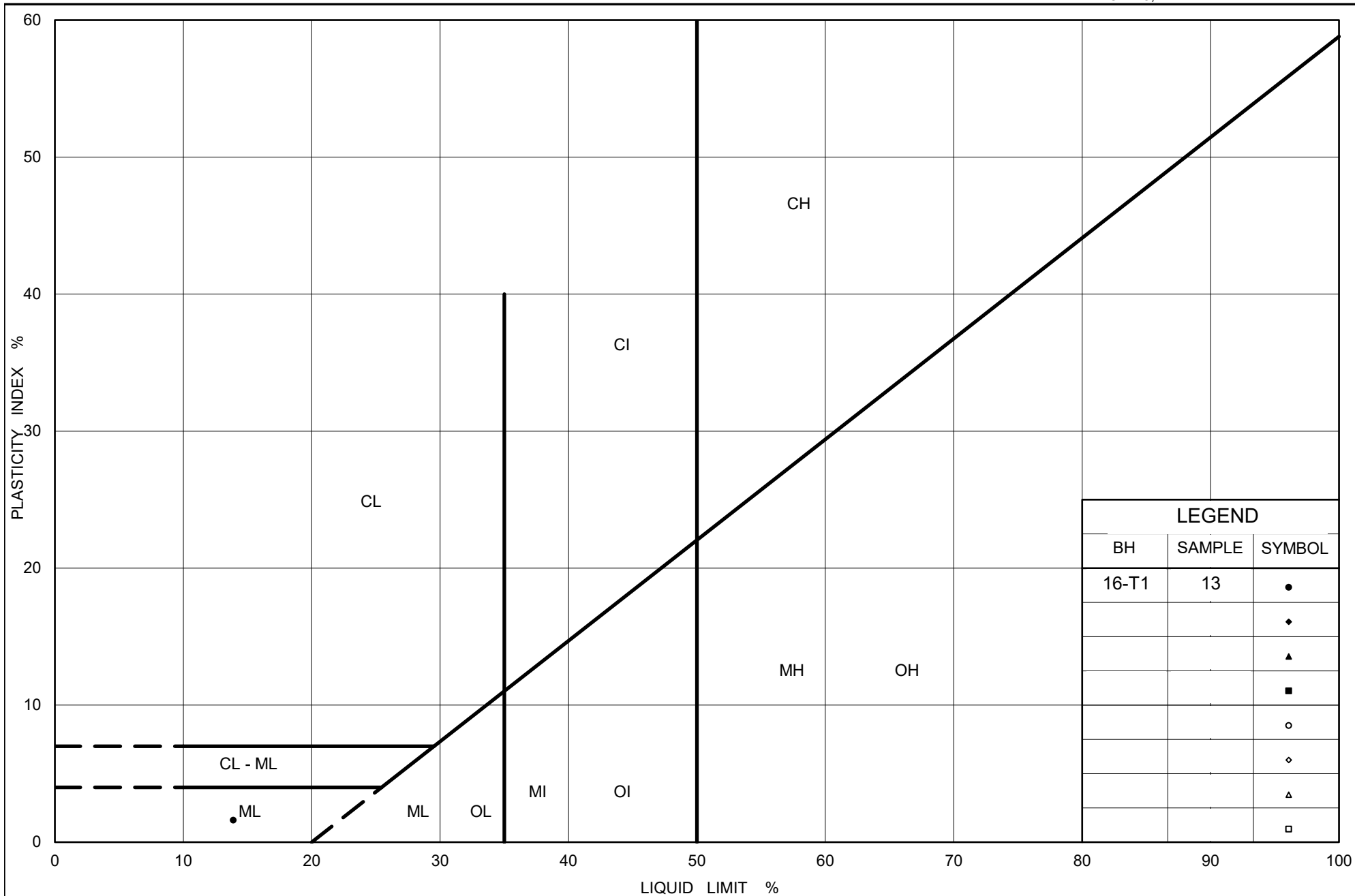
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	16-M1	10A	290.5
■	16-T1	15	294.9
◆	16-M2	6	295.4
▲	16-M1	7	293.5

Project Number: 1413191

Checked By: ARV

Golder Associates

Date: 24-Sep-19



Ministry of Transportation

Ontario

PLASTICITY CHART

Sandy Silt

Figure No. C-5

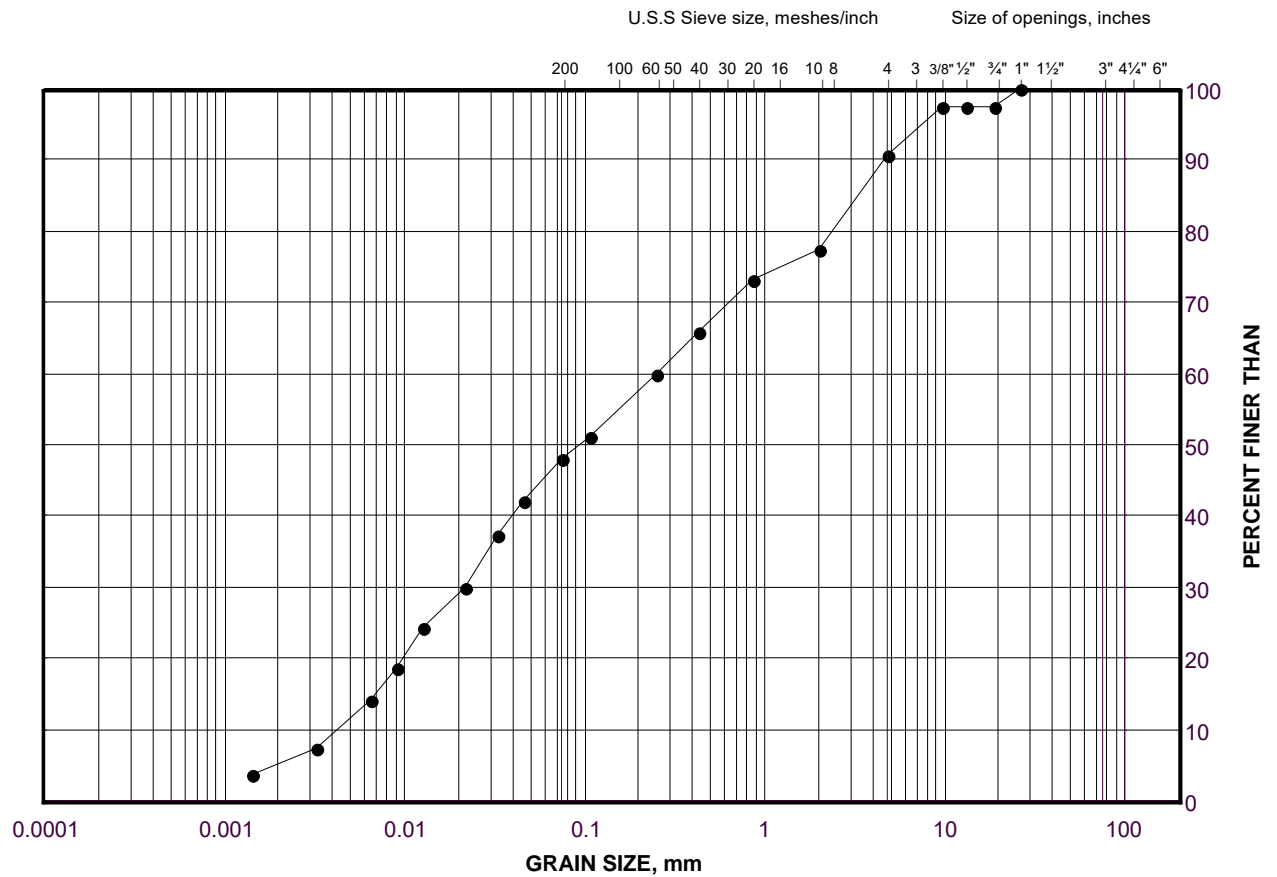
Project No. 1413191 (1080)

Checked By: ARV

GRAIN SIZE DISTRIBUTION

Organic Silt and Sand

FIGURE C-6



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

LEGEND

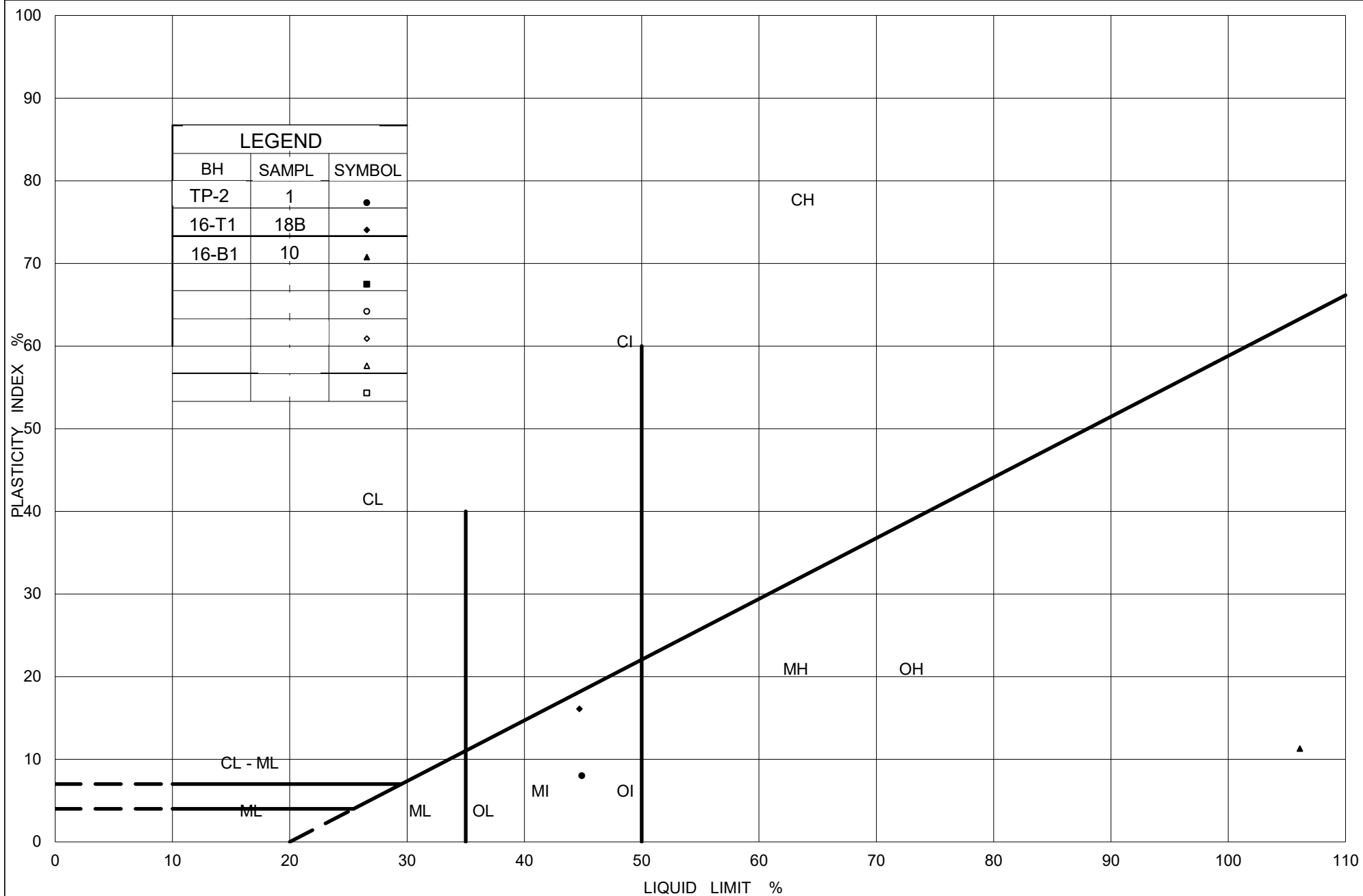
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	16-B1	11	292.5

Project Number: 1413191

Checked By: ARV

Golder Associates

Date: 24-Sep-19



Ministry of Transportation

PLASTICITY CHART

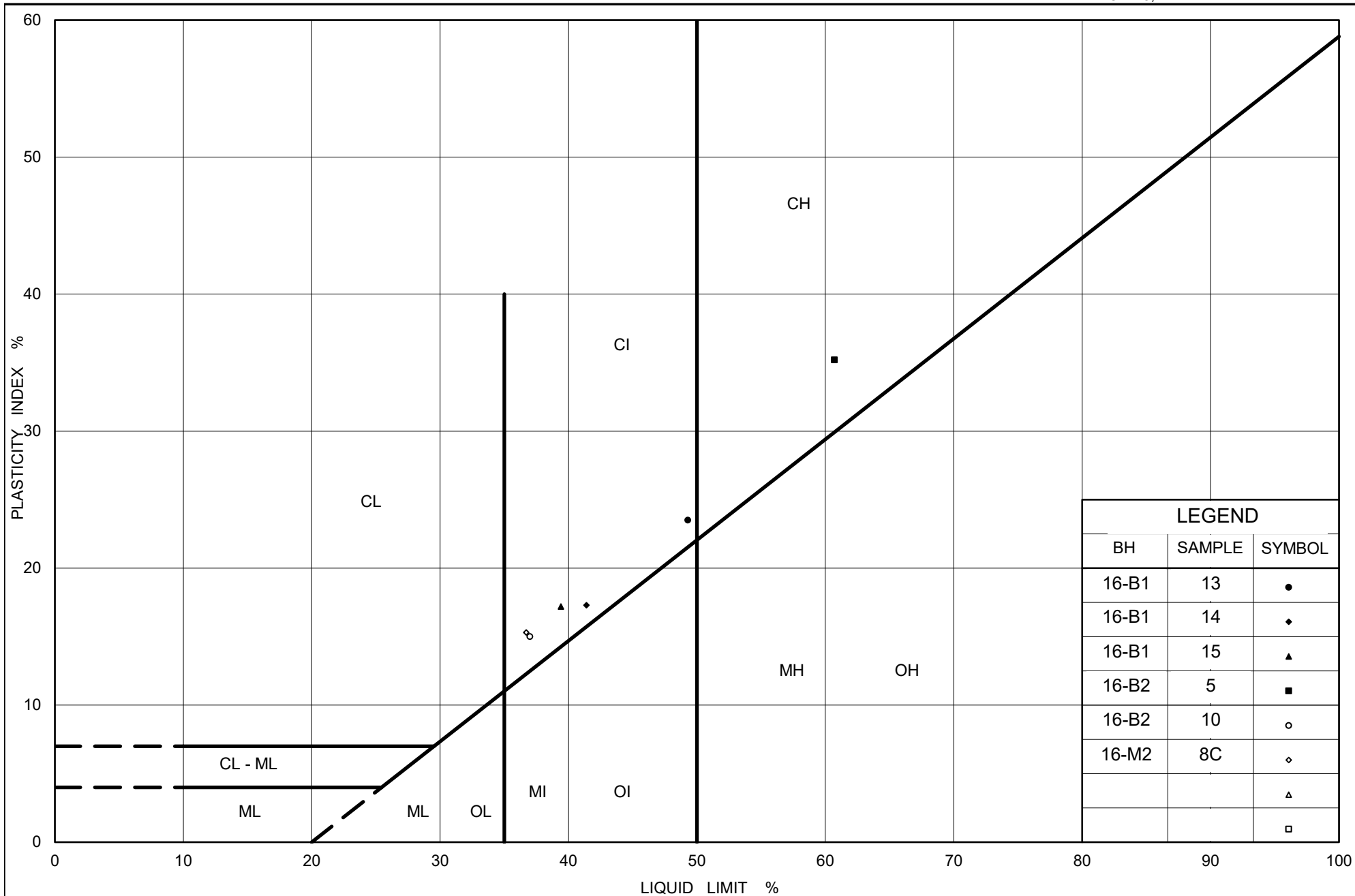
Clayey Organic Silt to Organic Silt

Ontario

Figure No. C-7

Project No. 1413191 (1080)

Checked By: ARV



Ministry of Transportation

Ontario

PLASTICITY CHART

Clayey Silty to Silty Clay to Clay

Figure No. C-8

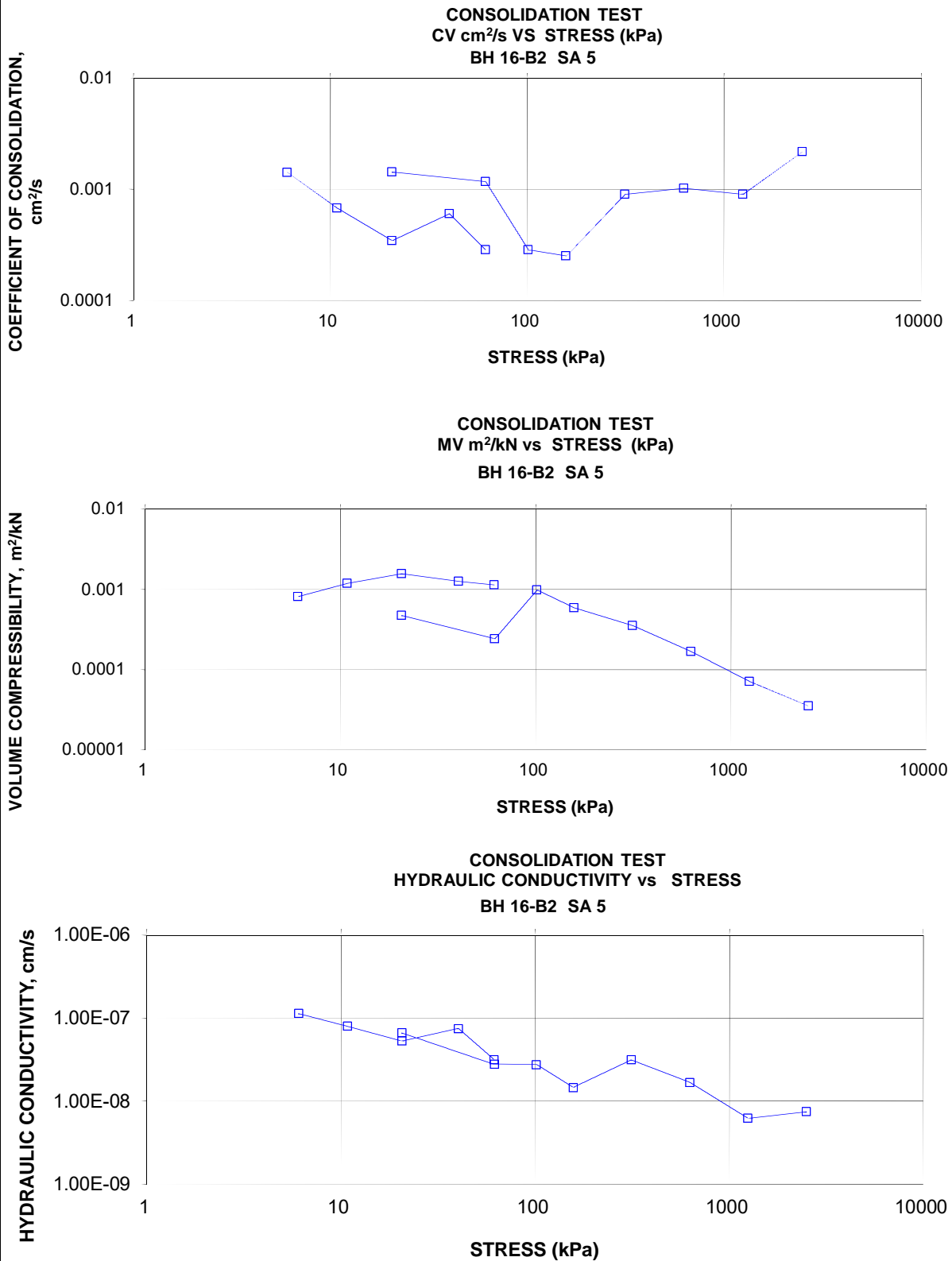
Project No. 1413191 (1080)

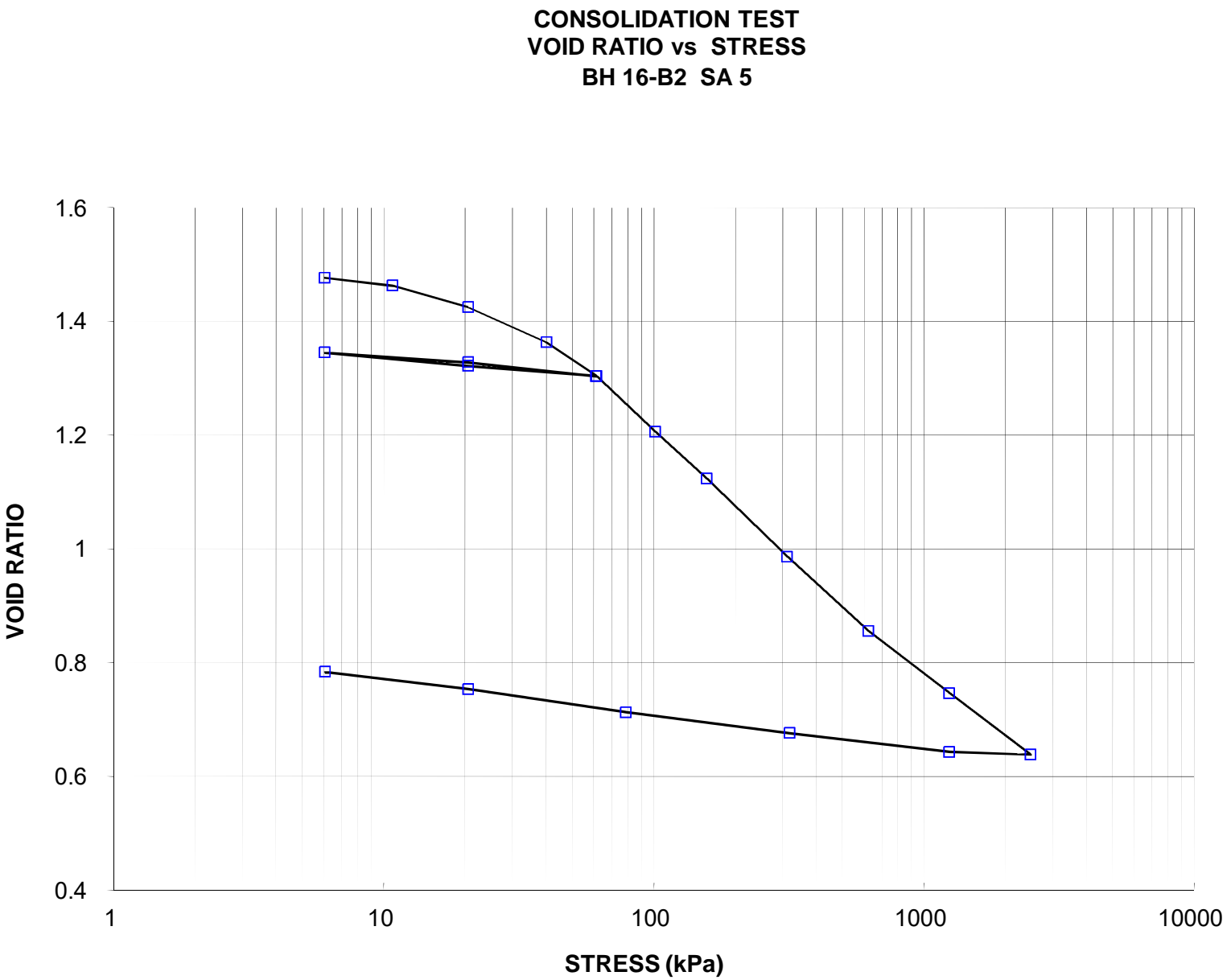
Checked By: ARV

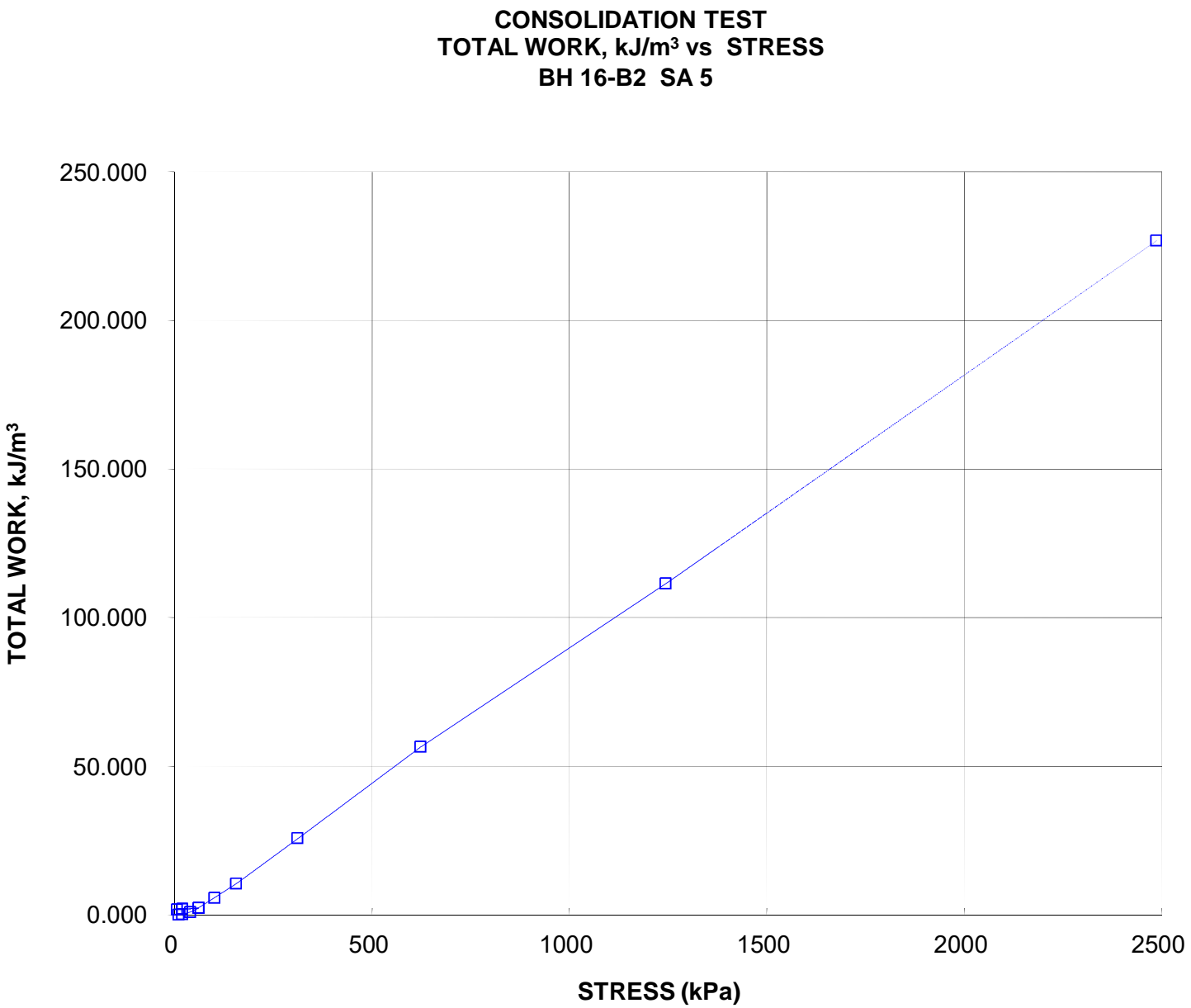
CONSOLIDATION TEST SUMMARY					FIGURE C9		
ASTM D2435/D2435M							
SAMPLE IDENTIFICATION							
Project Number	1413191(1080)				Sample Number	5	
Borehole Number	16-B2				Sample Depth, m	7.62-8.23	
TEST CONDITIONS							
Test Type	Laboratory Standard				Load Duration, hr	24	
Oedometer Number	2						
Date Started	08/17/2016						
Date Completed	09/07/2016						
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL							
Sample Height, cm	2.55				Unit Weight, kN/m ³	16.03	
Sample Diameter, cm	6.33				Dry Unit Weight, kN/m ³	10.68	
Area, cm ²	31.50				Specific Gravity, measured	2.71	
Volume, cm ³	80.17				Solids Height, cm	1.022	
Water Content, %	50.15				Volume of Solids, cm ³	32.20	
Wet Mass, g	131.04				Volume of Voids, cm ³	47.96	
Dry Mass, g	87.27				Degree of Saturation, %	91.3	
TEST COMPUTATIONS							
	Corr.		Average				
Stress	Height	Void	Height	t ₉₀	cv.	mv	k
kPa	cm	Ratio	cm	sec	cm ² /s	m ² /kN	cm/s
0.00	2.545	1.489	2.545				
6.04	2.533	1.477	2.539	960	1.42E-03	8.13E-04	1.13E-07
10.78	2.518	1.463	2.525	1984	6.81E-04	1.19E-03	7.97E-08
20.55	2.479	1.425	2.499	3826	3.46E-04	1.57E-03	5.32E-08
40.08	2.416	1.364	2.448	2089	6.08E-04	1.26E-03	7.52E-08
61.14	2.356	1.304	2.386	4234	2.85E-04	1.13E-03	3.17E-08
20.58	2.374	1.322	2.365				
6.04	2.398	1.345	2.386				
20.55	2.380	1.328	2.389	844	1.43E-03	4.74E-04	6.66E-08
61.43	2.355	1.304	2.368	1009	1.18E-03	2.41E-04	2.78E-08
101.20	2.256	1.206	2.305	3937	2.86E-04	9.83E-04	2.76E-08
156.71	2.172	1.124	2.214	4133	2.51E-04	5.94E-04	1.46E-08
312.03	2.031	0.987	2.101	1033	9.06E-04	3.56E-04	3.16E-08
622.73	1.897	0.856	1.964	802	1.02E-03	1.69E-04	1.69E-08
1242.80	1.786	0.747	1.841	799	9.00E-04	7.08E-05	6.24E-09
2485.65	1.675	0.638	1.730	290	2.19E-03	3.50E-05	7.50E-09
1242.80	1.680	0.643	1.677				
318.88	1.714	0.676	1.697				
78.99	1.751	0.713	1.732				
20.55	1.793	0.753	1.772				
6.07	1.824	0.784	1.808				
Note: Consolidation loading and unloading schedule assigned by the client. Specimen taken 50-56 cm from top of the tube k calculated using cv based on t ₉₀ values.							
SAMPLE DIMENSIONS AND PROPERTIES - FINAL							
Sample Height, cm	1.82				Unit Weight, kN/m ³	19.31	
Sample Diameter, cm	6.33				Dry Unit Weight, kN/m ³	14.90	
Area, cm ²	31.50				Specific Gravity, measured	2.71	
Volume, cm ³	57.44				Solids Height, cm	1.022	
Water Content, %	29.60				Volume of Solids, cm ³	32.20	
Wet Mass, g	113.10				Volume of Voids, cm ³	25.24	
Dry Mass, g	87.27						
Prepared By: LH		Golder Associates				Checked By:	

CONSOLIDATION TEST SUMMARY

FIGURE C9



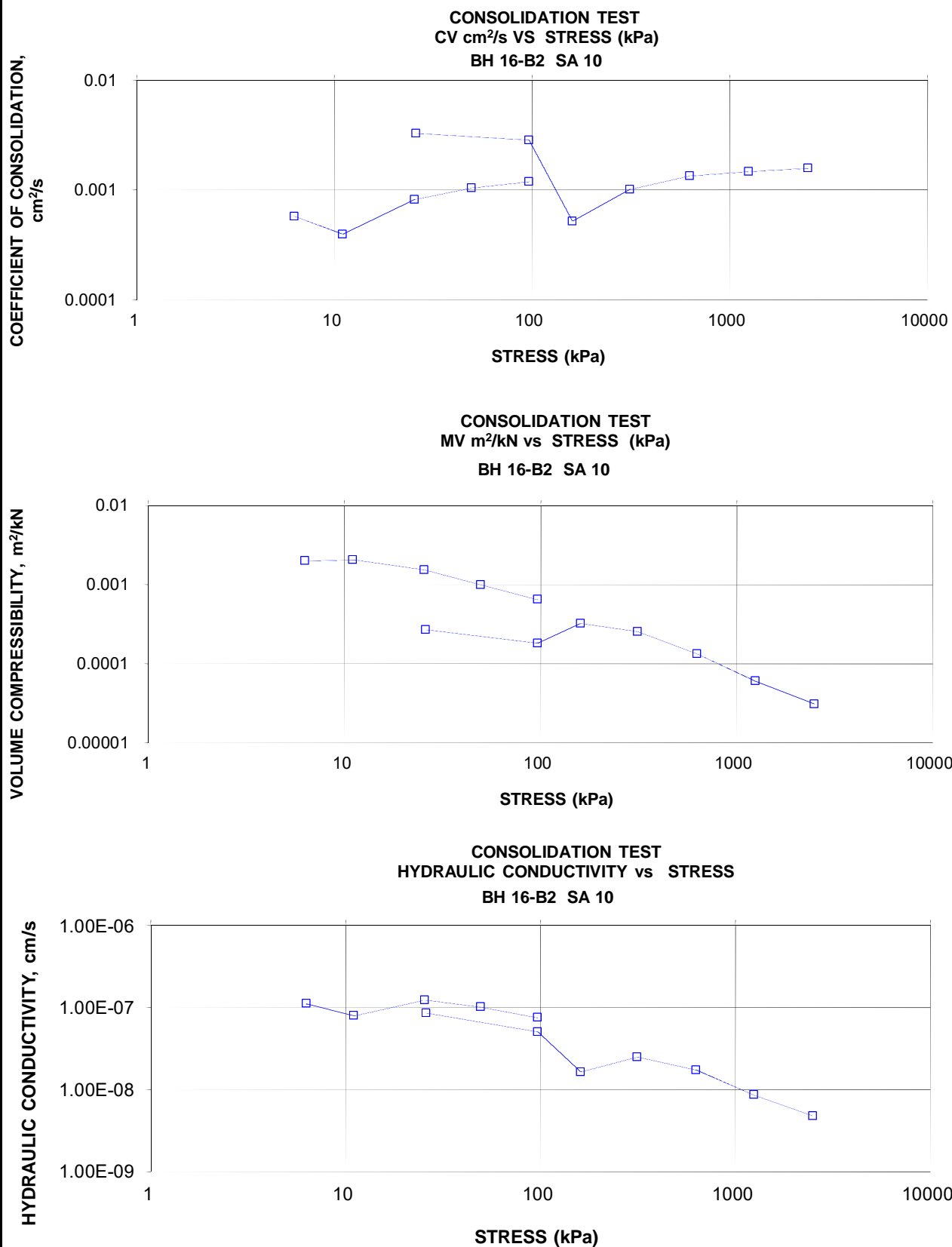


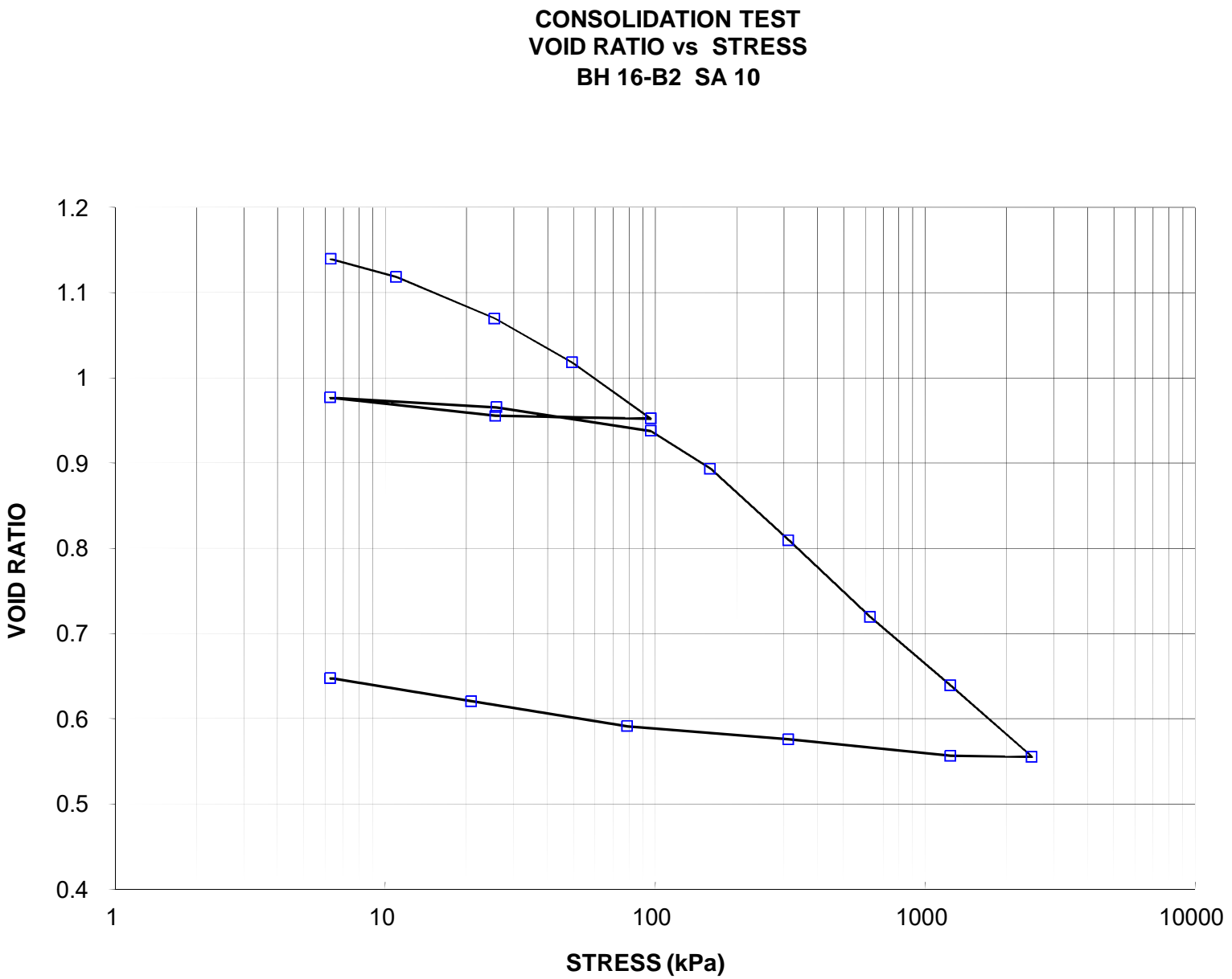


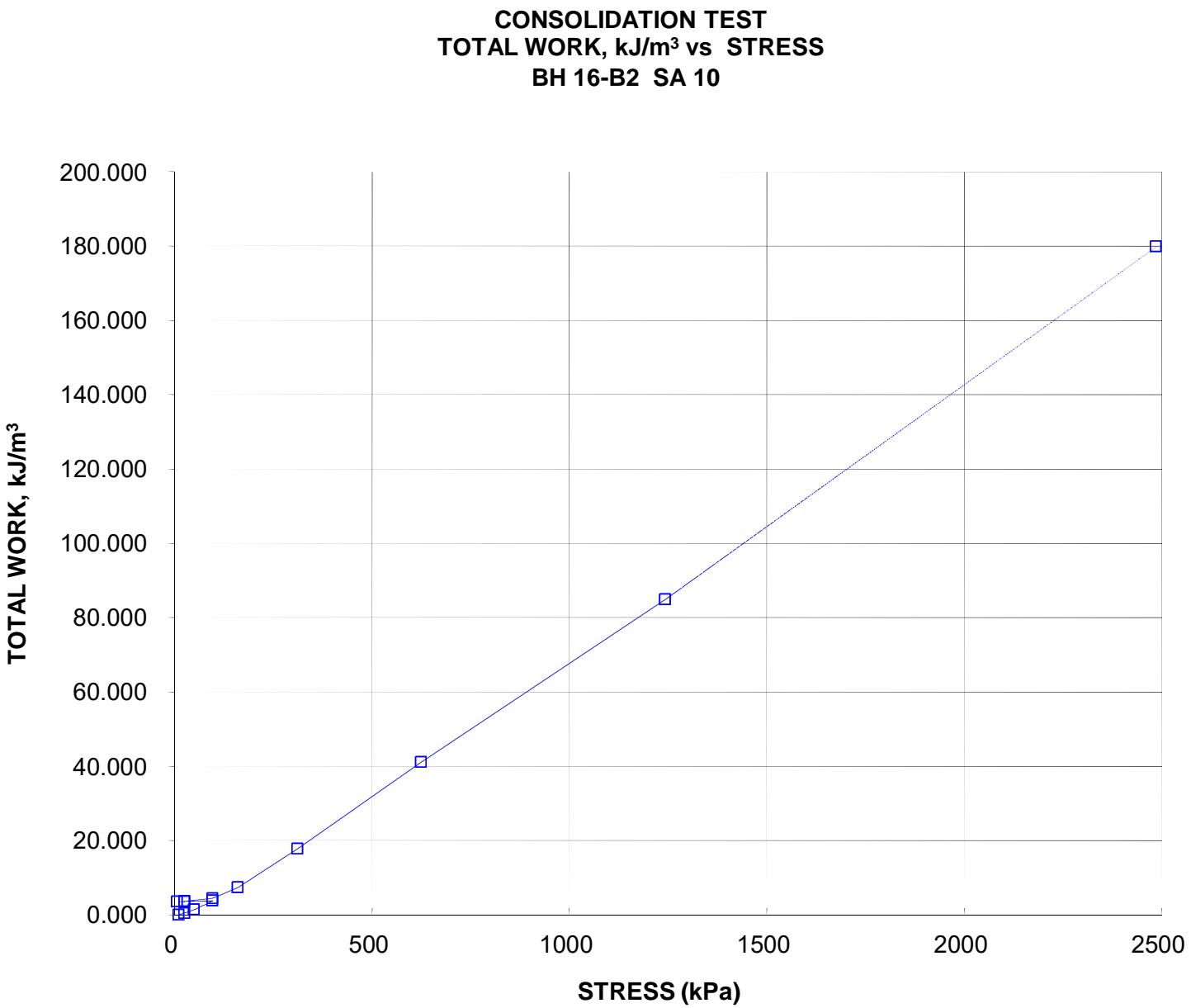
CONSOLIDATION TEST SUMMARY					FIGURE C10		
ASTM D2435/D2435M							
SAMPLE IDENTIFICATION							
Project Number	1413191(1080)			Sample Number	10		
Borehole Number	16-B2			Sample Depth, m	13.72-14.20		
TEST CONDITIONS							
Test Type	Laboratory Standard			Load Duration, hr	24		
Oedometer Number	4						
Date Started	08/18/2016						
Date Completed	09/12/2016						
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL							
Sample Height, cm	2.54			Unit Weight, kN/m ³	17.41		
Sample Diameter, cm	6.34			Dry Unit Weight, kN/m ³	12.13		
Area, cm ²	31.59			Specific Gravity, measured	2.68		
Volume, cm ³	80.21			Solids Height, cm	1.172		
Water Content, %	43.53			Volume of Solids, cm ³	37.02		
Wet Mass, g	142.40			Volume of Voids, cm ³	43.19		
Dry Mass, g	99.21			Degree of Saturation, %	100.0		
TEST COMPUTATIONS							
	Corr.		Average				
Stress	Height	Void	Height	t ₉₀	cv.	mv	k
kPa	cm	Ratio	cm	sec	cm ² /s	m ² /kN	cm/s
0.00	2.539	1.167	2.539				
6.28	2.507	1.139	2.523	2344	5.76E-04	1.99E-03	1.13E-07
10.99	2.482	1.118	2.495	3342	3.95E-04	2.07E-03	8.02E-08
25.45	2.426	1.070	2.454	1561	8.18E-04	1.55E-03	1.24E-07
49.35	2.365	1.018	2.395	1162	1.05E-03	9.97E-04	1.02E-07
96.26	2.288	0.952	2.327	960	1.20E-03	6.48E-04	7.59E-08
25.60	2.292	0.956	2.290				
6.25	2.317	0.977	2.304				
25.84	2.304	0.966	2.310	346	3.27E-03	2.69E-04	8.63E-08
96.43	2.271	0.938	2.287	390	2.84E-03	1.82E-04	5.08E-08
159.85	2.219	0.893	2.245	2053	5.20E-04	3.23E-04	1.65E-08
312.11	2.121	0.810	2.170	984	1.01E-03	2.54E-04	2.53E-08
624.85	2.015	0.720	2.068	677	1.34E-03	1.33E-04	1.74E-08
1242.01	1.921	0.639	1.968	558	1.47E-03	6.02E-05	8.69E-09
2484.37	1.823	0.555	1.872	470	1.58E-03	3.10E-05	4.81E-09
1242.01	1.824	0.557	1.823				
312.11	1.847	0.576	1.836				
78.99	1.865	0.591	1.856				
20.86	1.899	0.621	1.882				
6.25	1.931	0.648	1.915				
<p>Note:</p> <p>Consolidation loading and unloading schedule assigned by the client.</p> <p>Specimen taken 35-40 cm from top of the tube</p> <p>k calculated using cv based on t₉₀ values.</p>							
SAMPLE DIMENSIONS AND PROPERTIES - FINAL							
Sample Height, cm	1.93			Unit Weight, kN/m ³	20.07		
Sample Diameter, cm	6.34			Dry Unit Weight, kN/m ³	15.95		
Area, cm ²	31.59			Specific Gravity, measured	2.68		
Volume, cm ³	61.00			Solids Height, cm	1.172		
Water Content, %	25.85			Volume of Solids, cm ³	37.02		
Wet Mass, g	124.86			Volume of Voids, cm ³	23.98		
Dry Mass, g	99.21						
Prepared By: LH				Golder Associates		Checked By:	

CONSOLIDATION TEST SUMMARY

FIGURE C10





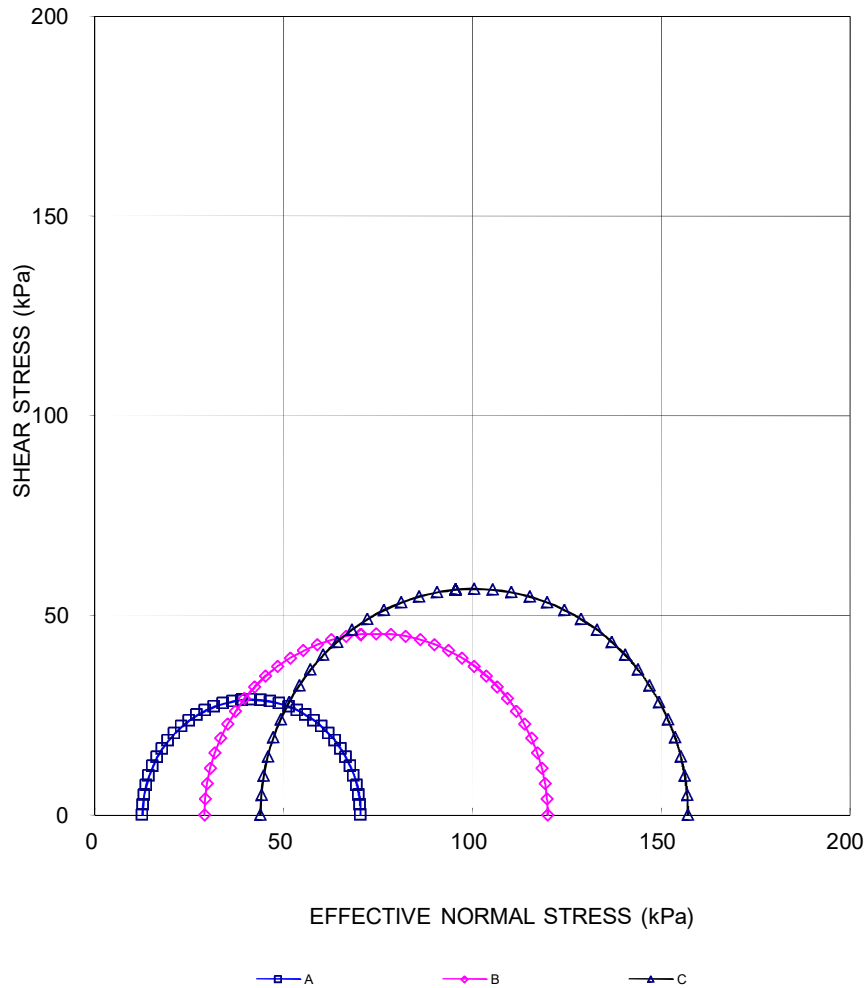


CONSOLIDATED UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENTS SHEET 1 OF 4			FIGURE C11
TEST STAGE	A	B	C
BOREHOLE NUMBER	16-B2		
SAMPLE	5		
DEPTH, m	7.62-8.23		
SPECIMEN DIAMETER, cm	5.07	5.05	5.10
SPECIMEN HEIGHT, cm	9.99	9.95	10.04
NATURAL WATER CONTENT, %	47.2	48.4	37.7
DRY DENSITY, Mg/m ³	1.17	1.21	1.32
WATER CONTENT AFTER SATURATION, %	47.7	47.6	37.4
CELL PRESSURE, σ_3 , kPa	160.0	210.0	260.0
BACK PRESSURE, kPa	135.0	135.0	135.0
PORE PRESSURE PARAMETER "B"	0.96	0.99	0.96
EFFECTIVE CONSOLIDATION STRESS, σ_c , kPa	25.0	75.0	125.0
VOLUMETRIC STRAIN DURING CONSOLIDATION, %	3.1	9.5	1.3
WATER CONTENT AFTER CONSOLIDATION, %	45.1	39.7	32.2
AVERAGE RATE OF STRAIN, %/hr	0.5	0.5	0.5
TIME TO FAILURE, HOURS	28.3	29.9	29.9
WATER CONTENT AFTER TEST, %	44.5	40.8	31.8
MAX. DEVIATOR STRESS, $(\sigma_1 - \sigma_3)$, kPa	57.7	90.9	113.2
AXIAL STRAIN AT $(\sigma_1 - \sigma_3)$ maximum, %	14.2	15.0	15.0
MAX EFFECTIVE PRINCIPAL STRESS RATIO, (σ'_1 / σ'_3) maximum	5.8	4.2	3.6
DEVIATOR STRESS AT (σ'_1 / σ'_3) maximum, kPa	54.2	90.0	105.1
AXIAL STRAIN AT (σ'_1 / σ'_3) maximum, %	8.4	14.4	10.4
PORE PRESSURE PARAMETER, Af, AT $(\sigma_1 - \sigma_3)$ maximum	0.21	0.51	0.72
PORE PRESSURE PARAMETER, Af, AT (σ'_1 / σ'_3) maximum	0.25	0.52	0.80
FILTER DRAINS USED, y/n	y	y	y
TEST NOTES: Effective consolidation stresses are assigned by the client. Specimen A taken 39-50 cm from top of tube. Specimen B taken 28-39 cm from top of tube. Specimen C taken 16-27 cm from top of tube.			
FAILURE PLANE NUMBER	-	-	-
ANGLE OF FAILURE PLANE, DEGREES	Bulged	Bulged	Bulged
<div> <div>Date: 9/6/2016</div> <div>Project No. 1413191(1080)</div> </div> <div> <div>Golder Associates</div> </div> <div> <div>Prepared By: LH</div> <div>Checked By:</div> </div>			

CONSOLIDATED UNDRAINED TRIAXIAL
WITH PORE PRESSURE MEASUREMENTS
SHEET 2 OF 4

FIGURE C11

BH 16-B2 SA 5



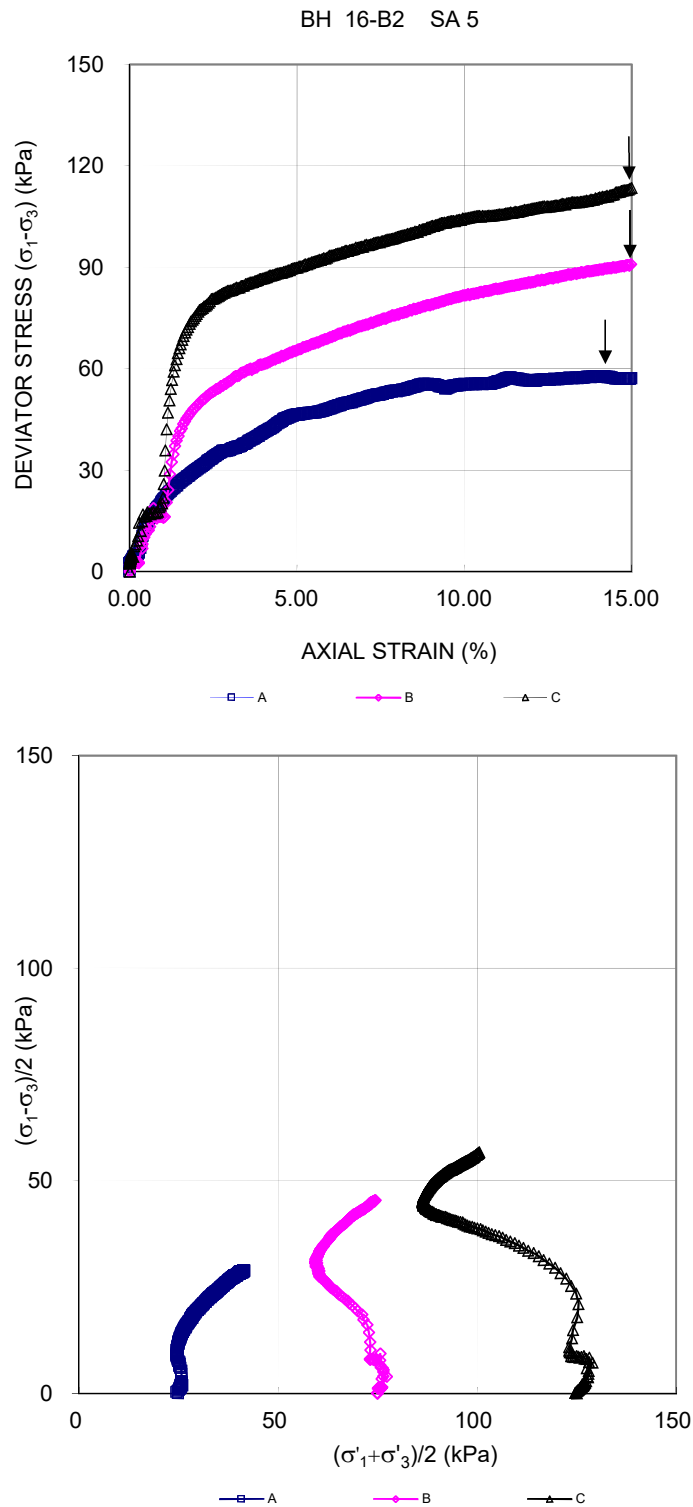
Date: 9/6/2016
Project No. 1413191(1080)

Golder Associates

Prepared By: LH
Checked By:

**CONSOLIDATED UNDRAINED TRIAXIAL
WITH PORE PRESSURE MEASUREMENTS
SHEET 3 OF 4**

FIGURE C11



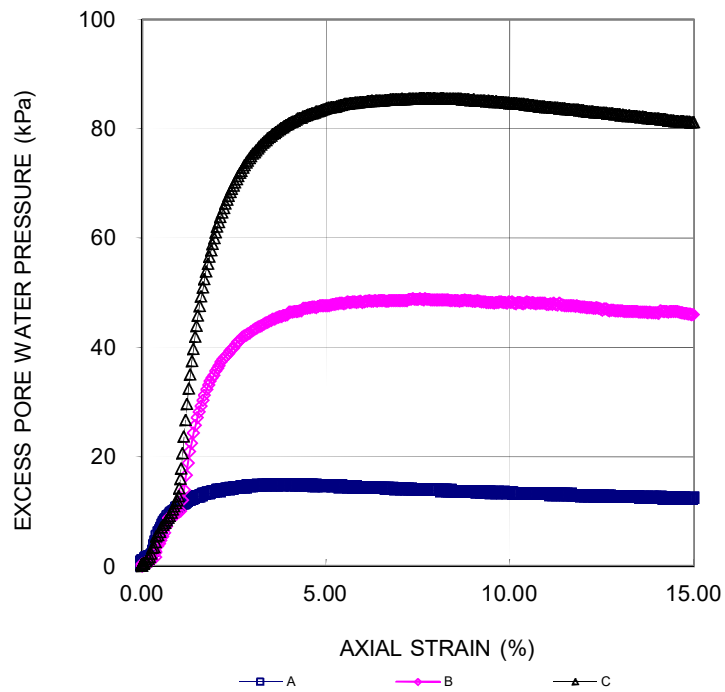
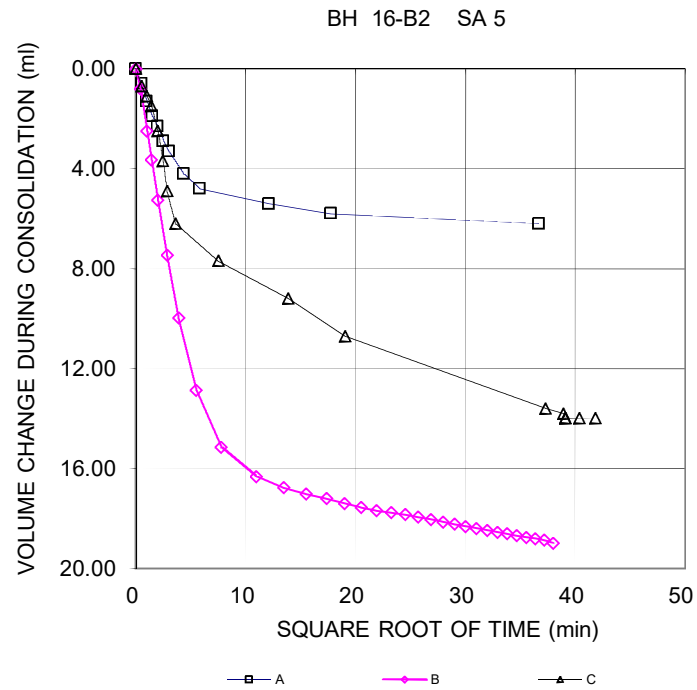
Date: 9/6/2016
Project No. 1413191(1080)

Golder Associates

Prepared By: LH
Checked By:

**CONSOLIDATED UNDRAINED TRIAXIAL
WITH PORE PRESSURE MEASUREMENTS
SHEET 4 OF 4**

FIGURE C11



Date: 9/6/2016
Project No. 1413191(1080)

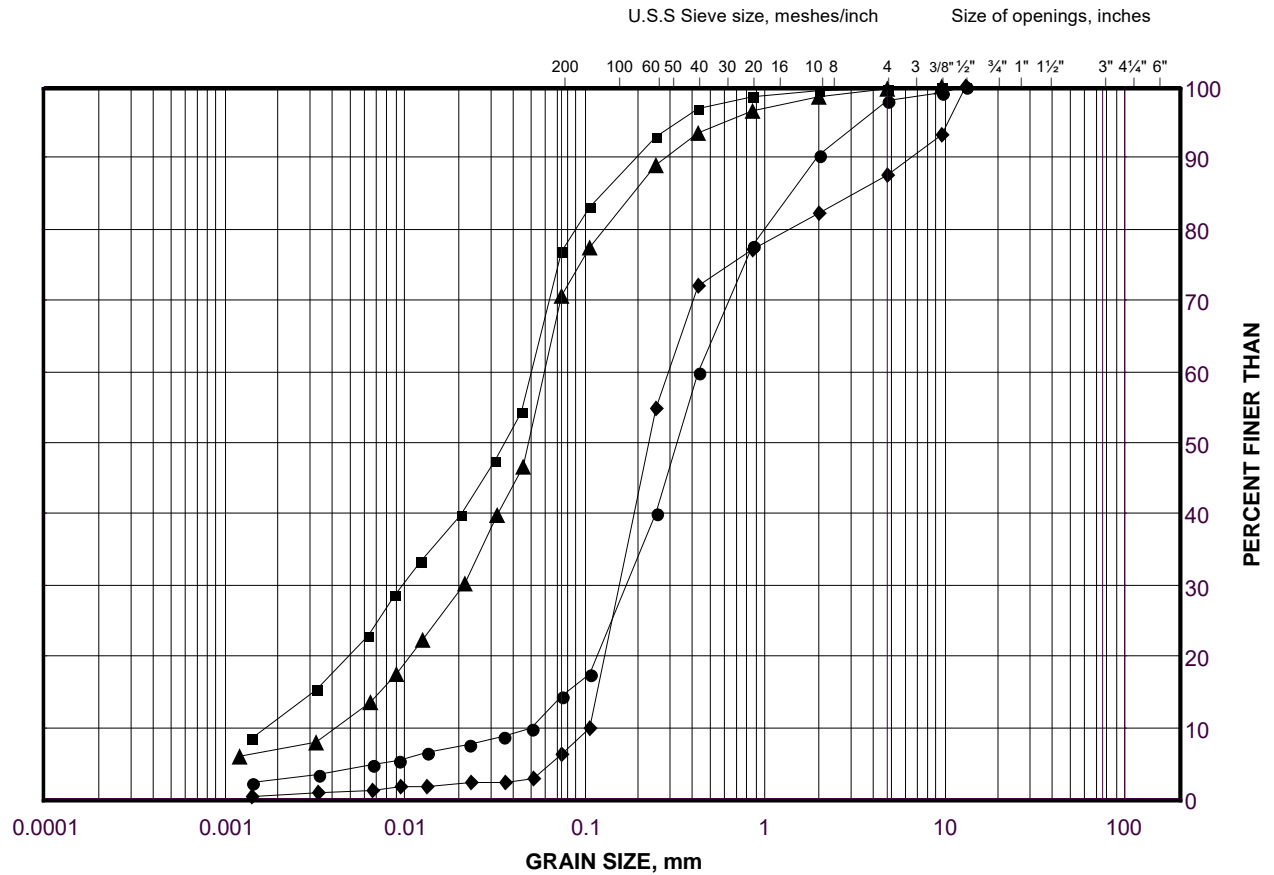
Golder Associates

Prepared By: LH
Checked By:

GRAIN SIZE DISTRIBUTION

Sandy Silt to Silty Sand - Lower

FIGURE C-12



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

LEGEND

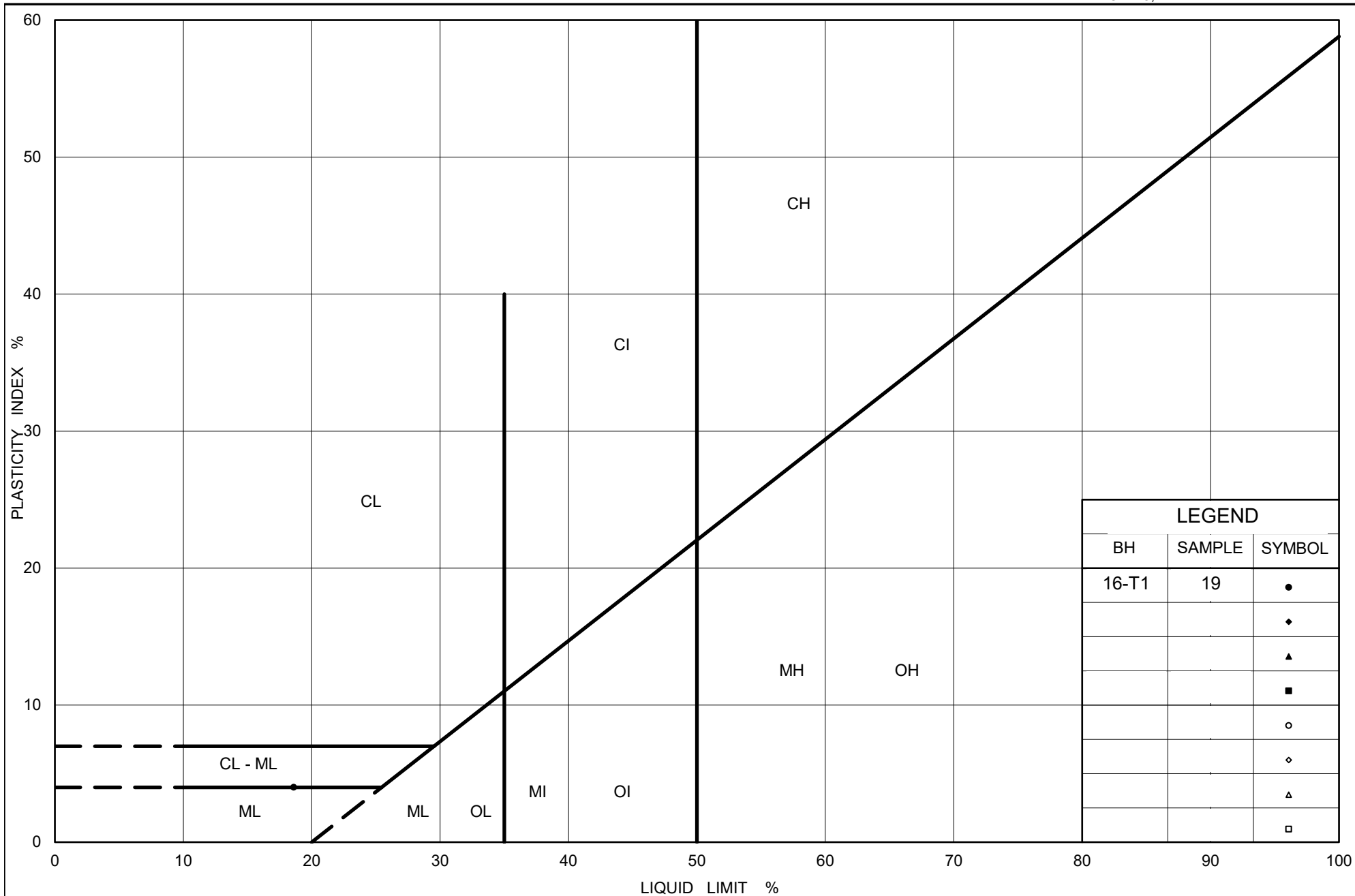
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	16-B1	17	287.8
■	16-T1	19	291.1
◆	16-B1	20	283.3
▲	16-T1	20A	289.8

Project Number: 1413191

Checked By: ARV

Golder Associates

Date: 24-Sep-19



Ministry of Transportation

Ontario

PLASTICITY CHART

Sandy Silt

Figure No. C-13

Project No. 1413191 (1080)

Checked By: ARV

APPENDIX D

**2016 Investigation – Analytical
Laboratory Test Results**

Your Project #: 1413191/1080

Site Location: HWY 400

Your C.O.C. #: 70341

Attention: Alysha Kobylinski

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

Report Date: 2016/08/25

Report #: R4133563

Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B6H7173

Received: 2016/08/19, 17:06

Sample Matrix: Soil
Samples Received: 4

Analyses	Date		Date Analyzed	Laboratory Method	Reference
	Quantity	Extracted			
Chloride (20:1 extract)	4	N/A	2016/08/25	CAM SOP-00463	EPA 325.2 m
Conductivity	4	N/A	2016/08/25	CAM SOP-00414	OMOE E3530 v1 m
pH CaCl2 EXTRACT	4	2016/08/24	2016/08/24	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	4	2016/08/20	2016/08/25	CAM SOP-00414	SM 22 2510 m
Sulphate (20:1 Extract)	4	N/A	2016/08/25	CAM SOP-00464	EPA 375.4 m

Remarks:

Maxxam Analytics has performed all analytical testing herein in accordance with ISO 17025 and the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act. All methodologies comply with this document and are validated for use in the laboratory. The methods and techniques employed in this analysis conform to the performance criteria (detection limits, accuracy and precision) as outlined in the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act.

Maxxam Analytics is accredited for all specific parameters as required by Ontario Regulation 153/04. Maxxam Analytics is limited in liability to the actual cost of analysis unless otherwise agreed in writing. There is no other warranty expressed or implied. Samples will be retained at Maxxam Analytics for three weeks from receipt of data or as per contract.

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

Email: EGitej@maxxam.ca

Phone# (905)817-5829

=====

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

RESULTS OF ANALYSES OF SOIL

Maxxam ID		CXQ259		CXQ260	CXQ261			
Sampling Date		2016/08/18		2016/08/18	2016/08/18			
COC Number		70341		70341	70341			
	UNITS	16B1-11-7.62-8.23	RDL	16B1-14-9.91-10.52	16B1-17-12.19-12.80	RDL	MDL	QC Batch
Calculated Parameters								
Resistivity	ohm-cm	520		2000	1700			4628553
Inorganics								
Soluble (20:1) Chloride (Cl)	ug/g	700	20	170	220	20	20	4632752
Conductivity	umho/cm	1940	2	494	573	2	1	4634277
Available (CaCl2) pH	pH	7.22		7.24	7.54			4632680
Soluble (20:1) Sulphate (SO4)	ug/g	1100	60	<20	100	20	N/A	4632770
RDL = Reportable Detection Limit QC Batch = Quality Control Batch N/A = Not Applicable								

Maxxam ID		CXQ262			
Sampling Date		2016/08/18			
COC Number		70341			
	UNITS	16B1-18-13.72-14.33	RDL	MDL	QC Batch
Calculated Parameters					
Resistivity	ohm-cm	1400			4628553
Inorganics					
Soluble (20:1) Chloride (Cl)	ug/g	350	20	20	4632752
Conductivity	umho/cm	695	2	1	4634277
Available (CaCl2) pH	pH	7.77			4632680
Soluble (20:1) Sulphate (SO4)	ug/g	80	20	N/A	4632770
RDL = Reportable Detection Limit QC Batch = Quality Control Batch N/A = Not Applicable					

Maxxam Job #: B6H7173
Report Date: 2016/08/25

Golder Associates Ltd
Client Project #: 1413191/1080
Site Location: HWY 400
Sampler Initials: AK

TEST SUMMARY

Maxxam ID: CXQ259
Sample ID: 16B1-11-7.62-8.23
Matrix: Soil

Collected: 2016/08/18
Shipped:
Received: 2016/08/19

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	4632752	N/A	2016/08/25	Alina Dobreanu
Conductivity	AT	4634277	N/A	2016/08/25	Neil Dassanayake
pH CaCl2 EXTRACT	AT	4632680	2016/08/24	2016/08/24	Neil Dassanayake
Resistivity of Soil		4628553	2016/08/25	2016/08/25	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	4632770	N/A	2016/08/25	Deonarine Ramnarine

Maxxam ID: CXQ260
Sample ID: 16B1-14-9.91-10.52
Matrix: Soil

Collected: 2016/08/18
Shipped:
Received: 2016/08/19

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	4632752	N/A	2016/08/25	Alina Dobreanu
Conductivity	AT	4634277	N/A	2016/08/25	Neil Dassanayake
pH CaCl2 EXTRACT	AT	4632680	2016/08/24	2016/08/24	Neil Dassanayake
Resistivity of Soil		4628553	2016/08/25	2016/08/25	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	4632770	N/A	2016/08/25	Deonarine Ramnarine

Maxxam ID: CXQ261
Sample ID: 16B1-17-12.19-12.80
Matrix: Soil

Collected: 2016/08/18
Shipped:
Received: 2016/08/19

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	4632752	N/A	2016/08/25	Alina Dobreanu
Conductivity	AT	4634277	N/A	2016/08/25	Neil Dassanayake
pH CaCl2 EXTRACT	AT	4632680	2016/08/24	2016/08/24	Neil Dassanayake
Resistivity of Soil		4628553	2016/08/25	2016/08/25	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	4632770	N/A	2016/08/25	Deonarine Ramnarine

Maxxam ID: CXQ262
Sample ID: 16B1-18-13.72-14.33
Matrix: Soil

Collected: 2016/08/18
Shipped:
Received: 2016/08/19

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	4632752	N/A	2016/08/25	Alina Dobreanu
Conductivity	AT	4634277	N/A	2016/08/25	Neil Dassanayake
pH CaCl2 EXTRACT	AT	4632680	2016/08/24	2016/08/24	Neil Dassanayake
Resistivity of Soil		4628553	2016/08/25	2016/08/25	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	4632770	N/A	2016/08/25	Deonarine Ramnarine

GENERAL COMMENTS

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

Package 1	23.0°C
-----------	--------

Results relate only to the items tested.

QUALITY ASSURANCE REPORT

Golder Associates Ltd
Client Project #: 1413191/1080
Site Location: HWY 400
Sampler Initials: AK

QC Batch	Parameter	Date	Matrix Spike		SPIKED BLANK		Method Blank		RPD	
			% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
4632680	Available (CaCl ₂) pH	2016/08/24			98	97 - 103			0.47	N/A
4632752	Soluble (20:1) Chloride (Cl)	2016/08/25	109	70 - 130	104	70 - 130	<20	ug/g	NC	35
4632770	Soluble (20:1) Sulphate (SO ₄)	2016/08/25	NC	70 - 130	108	70 - 130	<20	ug/g	NC	35
4634277	Conductivity	2016/08/25			100	90 - 110	<2	umho/cm	1.3	10

N/A = Not Applicable

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 spiked amount was too small to permit a reliable recovery calculation (matrix spike concentration was less than 2x that of the native sample concentration).

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 (one or both samples < 5x RDL).

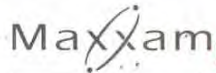
VALIDATION SIGNATURE PAGE

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



Brad Newman, Scientific Specialist

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



6740 Carmichael Road, Mississauga, ON L5N 2L8
Phone: 905-817-5700 Fax: 905-817-5779 Toll Free: (800) 563-6266

CHAIN OF CUSTODY RECORD

70341

Page 1 of 1

INVOICE INFORMATION		REPORT INFORMATION (if differs from invoice)		PROJECT INFORMATION		MAXXAM JOB NUMBER	
Company Name:	GOLDER ASSOCIATES	Company Name:		Quotation #:		CHAIN OF CUSTODY # 00	
Contact Name:	ALYSHA KOBYLINSKI	Contact Name:		P.O. #:			
Address:	6925 CENTURY AVE, SUITE 100 MISSISSAUGA, ON L5N 7K2	Address:		Project #:	1413191/1080		
Phone:	647 618 1364	Phone:		Site Location:	HWY 400		
Fax:		Fax:		Site #:			
Email:	Alysha_Kobylinski@golder.com	Email:		Sampled By:			

Note: For MOE Regulated Drinking Water samples, please use the Drinking Water CoC.					ANALYSIS REQUESTED (Please be specific)					TURNAROUND TIME (TAT) REQUIRED				
Regulation 153 (2011)					Other Regulations					PLEASE PROVIDE ADVANCE NOTICE FOR RUSH PROJECTS.				
Table 1	Res/Park	Med/Fine	CCME	Sanitary Sewer Bylaw	MOE Regulated Drinking Water? (Y/N)	Metals Field Filtered? (Y/N)	PH	chloride	sulphate	resistivity	conductivity	Regular (Standard) TAT: (5-7 working days for most tests)		
Table 2	Ind/Comm	Coarse	Reg. 558	Storm Sewer Bylaw								Rush TAT: ***Samples must be received by 3pm to guarantee your TAT***		
Table 3	Agri/Other	For RSC	MISA	Municipality:								Rush Confirmation #: PN		
Table		Yes	PWQO									<input type="checkbox"/> 1 day <input type="checkbox"/> 2 days <input type="checkbox"/> 3 days		
Table		No	Other (specify):									Date Req'd:		
Include Criteria on Certificate of Analysis (Y/N)?										TATs for certain tests are > 5 days. Please contact your Project Manager for details.				
SAMPLES MUST BE KEPT COOL (<10°C) FROM TIME OF SAMPLING UNTIL DELIVERY TO MAXXAM.										# of Cont. COMMENTS / TAT COMMENTS				
1	16B1-11-7.62-8.23	2016/08/18	AM	SOIL			X	X	X	X		2		
2	16B1-14-9.91-10.52	2016/08/18	AM	SOIL			X	X	X	X		2		
3	16B1-17-12.19-12.80	2016/08/18	AM	SOIL			X	X	X	X		2		
4	16B1-18-13.72-14.33	2016/08/18	AM	SOIL			X	X	X	X		2		
5														
6														
7														
8														
9														
10														
*RELINQUISHED BY (Signature/Print)					RECEIVED BY: (Signature/Print)					#JARS USED AND NOT SUBMITTED				
Alysha Kobylinski					Raj Rameer					Laboratory Use Only				
Date (YYYY/MM/DD)					Date (YYYY/MM/DD)					Custody Seal				
2016/08/19					2016/08/19					Yes No				
Time					Time					Present				
17:04					17:06					Intact				
										Temperature (°C) on Receipt				
										24.3/22				

*MANDATORY SECTIONS IN GREY MUST BE FILLED OUT. AN INCOMPLETE CHAIN OF CUSTODY MAY RESULT IN ANALYTICAL TAT DELAYS. notice

COC-1004 (10/11) - ENY ENG

Maxxam Analytics International Corporation or a Maxxam Analytics

White: Maxxam

Yellow: Mail

Pink: Client

CLIENT NAME: GOLDER ASSOCIATES LTD.
6925 CENTURY AVE, SUITE#100
MISSISSAUGA, ON L5N7K2
(905) 567-4444

ATTENTION TO: Al Varshoi

PROJECT: 1413191(1080)

AGAT WORK ORDER: 16T149988

WATER ANALYSIS REVIEWED BY: Amanjot Bhela, Inorganic Coordinator

DATE REPORTED: Oct 24, 2016

PAGES (INCLUDING COVER): 5

VERSION*: 1

Should you require any information regarding this analysis please contact your client services representative at (905) 712-5100

*NOTES

All samples will be disposed of within 30 days following analysis. Please contact the lab if you require additional sample storage time.



AGAT Laboratories

Certificate of Analysis

AGAT WORK ORDER: 16T149988

PROJECT: 1413191(1080)

5835 COOPERS AVENUE
MISSISSAUGA, ONTARIO
CANADA L4Z 1Y2
TEL (905)712-5100
FAX (905)712-5122
<http://www.agatlabs.com>

CLIENT NAME: GOLDER ASSOCIATES LTD.

SAMPLING SITE:

ATTENTION TO: AI Varshoi

SAMPLED BY: Jason Cornell

Inorganic Chemistry (Water)

DATE RECEIVED: 2016-10-18

DATE REPORTED: 2016-10-24

SAMPLE DESCRIPTION: SA1
SAMPLE TYPE: Water
DATE SAMPLED: 10/17/2016
G / S RDL 7937671

Parameter	Unit	G / S	RDL	
Electrical Conductivity	uS/cm		2	3570
pH	pH Units		NA	7.98
Resistivity	ohms.cm			280
Chloride	mg/L		2.0	856
Sulphate	mg/L		2.0	45.9

Comments: RDL - Reported Detection Limit; G / S - Guideline / Standard

7937671 Elevated RDL indicates the degree of sample dilution prior to the analysis for Anions in order to keep analytes within the calibration range of the instrument and to reduce matrix interference.

Certified By:

Amanjot Bhela

Quality Assurance

CLIENT NAME: GOLDER ASSOCIATES LTD.

PROJECT: 1413191(1080)

SAMPLING SITE:

AGAT WORK ORDER: 16T149988

ATTENTION TO: AI Varshoi

SAMPLED BY: Jason Cornell

Water Analysis

RPT Date: Oct 24, 2016

DUPLICATE

REFERENCE MATERIAL

METHOD BLANK SPIKE

MATRIX SPIKE

PARAMETER	Batch	Sample Id	Dup #1	Dup #2	RPD	Method Blank	Measured Value	Acceptable Limits		Recovery	Acceptable Limits		Recovery	Acceptable Limits	
								Lower	Upper		Lower	Upper		Lower	Upper

Inorganic Chemistry (Water)

Electrical Conductivity	7936236		946	943	0.3%	< 2	106%	80%	120%	NA			NA		
pH	7936236		7.79	7.69	1.3%	NA	101%	90%	110%	NA			NA		
Chloride	7941313		40.1	39.1	2.5%	< 0.10	97%	90%	110%	97%	90%	110%	101%	80%	120%
Sulphate	7941313		43.9	43.7	0.5%	< 0.10	95%	90%	110%	99%	90%	110%	97%	80%	120%

Comments: NA signifies Not Applicable.

Certified By:



Method Summary

CLIENT NAME: GOLDER ASSOCIATES LTD.

AGAT WORK ORDER: 16T149988

PROJECT: 1413191(1080)

ATTENTION TO: AI Varshoi

SAMPLING SITE:

SAMPLED BY: Jason Cornell

PARAMETER	AGAT S.O.P	LITERATURE REFERENCE	ANALYTICAL TECHNIQUE
Water Analysis			
Electrical Conductivity	INOR-93-6000	SM 2510 B	PC TITRATE
pH	INOR-93-6000	SM 4500-H+ B	PC TITRATE
Resistivity		SM 2510 B	EC METER
Chloride	INOR-93-6004	SM 4110 B	ION CHROMATOGRAPH
Sulphate	INOR-93-6004	SM 4110 B	ION CHROMATOGRAPH

APPENDIX E

**Laboratory Testing Report –
Ryerson University**

Laboratory Investigation of Deep Soil Mixing in Treatment of Organic Clays in Ontario

Final REPORT

A Project Funded by Ontario Centre for Excellence and Golder Associates Ltd.



Department of Civil Engineering, Ryerson University



Jinyuan Liu
Bill (Shuihan) Li
Ali Ahmad
Chandra Pouydal

November 16, 2016

EXECUTIVE SUMMARY

This research presents a preliminary experimental investigation of applying deep soil mixing method (DMM) in treating soft clays at a project site near the intersection between Highway 400 and Side Road 16 in the York region in Ontario. Both disturbed and undisturbed soil samples were collected from the site and treated with differing binder types, binder dosages, and curing durations. Two types of clay samples were received: clayey silt with organics to organic silt from test pit and clayey organic silt to silty clay from Shelby tube. A series of geotechnical tests were conducted on both untreated clay samples and treated clay samples, see the table below. Silty clay was used to perform the majorities of tests. Both cement and lime were used in treating the clay samples. Cement was shown to improve the strength and compressibility of both clayey silt and organic clays based on preliminary test results, while lime showed a minimal effect in treating the silty clay.

Table 0-1 Summaries of Tests Conducted to Date

Test Type	ASTM Standard	Number of Tests
Water Content Test	ASTM D2216-10	33
Specific Gravity Test	ASTM D854	5
Mini-Vane Shear Test	ASTM D4648	3
Constant Rate of Strain (CRS) Test	ASTM D4168	6
Unconfined Compression Strength (UCS) Test	ASTM D2166	37
Consolidated Undrained (CIU) Triaxial Test	ASTM D4767	3

DISCLAIMER

The contents of this report reflect the views of the authors who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the views or policies of the Ontario Centre of Excellence (OCE) and Golder Associates Ltd. The authors and the OCE do not endorse any products or manufacturers. Trade or manufacturers' names appear herein solely because they are considered essential to the objectives of this report.

TABLE OF CONTENT

Executive Summary	II
Disclaimer	II
Table of Content	III
List of Figures	IV
List of Tables	VI
1 Introductions.....	1
1.1 Background	1
1.2 Objectives.....	1
1.3 Research Methodology.....	1
2 Procedure	2
2.1 Sample Preparations	2
2.2 Unconfined compression strength Test.....	4
2.3 Specific Gravity Test.....	5
2.4 CRS Test	5
3 Results	6
3.1 UCS Test Result	6
3.1.1 Untreated Organic Clay	6
3.1.2 Cement Treated Soils.....	7
3.1.3 Lime Treated Soils.....	12
3.2 CRS Test Result	16
3.3 Triaxial Test Results.....	21
4 Summary and Future Research.....	24
5 Acknowledgement	25
6 Reference	26
7 Appendix	28
7.1 UCS Test Results of cement treated silty clay (150kg/m ³).....	29
7.2 UCS Test Results of cement treated silty clay (200kg/m ³).....	35
7.3 UCS Test Results of cement treated silty clay (250kg/m ³).....	42
7.4 UCS Test Results of cement treated Organic clay (250kg/m ³).....	48

7.5	UCS Test Results of Lime treated Silty clay (100 kg/m ³)	50
7.6	UCS Test Results of Lime treated Silty clay (200 kg/m ³)	56
7.7	UCS Test Results of Untreated Organic Clays	62
7.8	Specific Gravity Test.....	68
7.9	Miniature Vane Shear Test.....	71
7.10	Triaxial Test Results of Organic Clays	74

LIST OF FIGURES

Figure	Page
Figure 2-1 Slurry mixing and sample mixing procedure	3
Figure 2-2 Sample compaction and curing	4
Figure 2-3 Sample trimming mechanical saw and before/after of an UCS test.....	4
Figure 2-4 CRS sample preparation and set up	5
Figure 3-1 Stress-strain curves of untreated clayey organic silt to silty clay from Shelby tube.....	7
Figure 3-2 Stress-strain curves of UCS tests for 150 kg/m ³ cement treated soil samples	9
Figure 3-3 Stress-strain curves of UCS tests for 200 kg/m ³ cement treated soil samples	9
Figure 3-4 Stress-strain curves of UCS tests for 250 kg/m ³ cement treated soil samples	10
Figure 3-5 Peak UCS vs. curing duration plot for cement treated clayey silt with organics to organic silt.....	10
Figure 3-6 Residual UCS vs. curing duration plot for cement treated clayey silt with organics to organic silt.....	11
Figure 3-7 Peak UCS vs. curing duration plot for cement treated clayey organic silt to silty clay	11
Figure 3-8 Residual UCS vs. curing duration plot for cement treated clayey organic silt to silty clay	12
Figure 3-9 Stress-strain curves of UCS tests for 100 kg/m ³ lime treated soil samples	14
Figure 3-10 Stress-strain curves of UCS tests for 200 kg/m ³ lime treated soil samples	14
Figure 3-11 Peak UCS vs. curing duration plot for lime treated clayey silt with organics to organic silt.....	15
Figure 3-12 Residual UCS vs. curing duration plot for lime treated clayey silt with organics to organic silt.....	15
Figure 3-13 e vs. $\log p$ for cement treated clayey silt with organics to organic silt and untreated clayey silt with organics to organic silt.....	17
Figure 3-14 Compression index and recompression index for 250 kg/m ³ cement treated clayey silt with organics to organic silt.....	17

Figure 3-15 Compression index and recompression index for 200 kg/m ³ cement treated clayey silt with organics to organic silt.....	18
Figure 3-16 Compression index and recompression index for 150 kg/m ³ cement treated clayey silt with organics to organic silt.....	18
Figure 3-17 Compression index and recompression index for untreated clayey silt with organics to organic silt sample	19
Figure 3-18 e vs. $\log p$ for lime treated clayey silt with organics to organic silt.....	19
Figure 3-19 Compression index and recompression index for lime treated clayey silt with organics to organic silt.....	20
Figure 3-20 e vs. $\log p$ for an untreated clayey organic silt to silty clay sample.....	20
Figure 0-21 Volume change of untreated clayey organic silt to silty clay samples with time during consolidation	22
Figure 0-22 Stress-strain curves of untreated clayey organic silt to silty clay samples during shearing	23
Figure 0-23 Stress path curves of untreated clayey organic silt to silty clay samples during shearing	23

LIST OF TABLES

Table	Page
Table 0-1 Summaries of Tests Conducted to Date	II
Table 2-1 Experimental program for DMM mixed tests	2
Table 3-1 UCS test results for untreated clayey organic silt to silty clay from Shelby tubes	6
Table 3-2 UCS Test Results for Cement Treated Soils	8
Table 3-3 UCS Test Results for Lime Treated Soils	13
Table 3-4 CRS calculated parameters.....	21

1 INTRODUCTIONS

1.1 BACKGROUND

Ryerson University was retained by Golder Associates Ltd. to investigate using the deep mixing method (DMM) to improve soft organic clays encountered in one of its projects near the intersection between Highway 400 and Side Road 16th in the York Region.

Two kinds of samples were collected from the project: Shelby tube samples for deeper depths around 6.4m – 12.5m below ground and test pit soil samples at shallower depths. Both samples were tested using the DMM in the GeoOptical Research Lab at Ryerson University.

1.2 OBJECTIVES

The main objective of this research is to confirm the feasibility of using DMM to improve strength and compressibility of the organic clays of the project site. DMM has been applied successfully around the world for more than half a century. However, soil type and its characteristics are one of a few control factors for its efficiency in treating problematic soils. Currently, there are no available DMM applications available for treating organic clays in Ontario. In order to address the issue, soil samples were collected from the project site and a series of geotechnical tests were conducted following pertinent ASTM standards, including the unconfined compression strength (UCS) test, constant rate of strain (CRS) consolidation test, and isotropically consolidated-undrained (CIU) triaxial test.

1.3 RESEARCH METHODOLOGY

In this study the feasibility of applying DMM to treat silty and organic clay for the project site was investigated through an experimental program in the laboratory. The test variables include binder type, binder dosage, and curing duration. Both cement and lime were used due to their popularities and successful applications for treating problematic soils around the world (Baker, 2015; Bruce et al., 2013; Bergado et al., 1999; Locat, 1990). The binder dosage was designed based on literature review. Though its optimization can be addressed if time and budget permit. The curing durations used in this studies were 7, 14, 28, and 56 days. The comparison studies between treated and untreated samples in term of strength and compressibility were used to evaluate the efficiency of DMM in treating organic clay.

2 PROCEDURE

2.1 SAMPLE PREPARATIONS

Silty clay samples and organic clay samples were mixed and treated with a binder based on experimental program. Experimental variables included two (2) binder types, three (3) binder dosage, and three (3) curing conditions. The summary of the mixing tests is shown in Table 2-1. The binder dosage and curing conditions were chosen based on past researches (Li et al, 2016; Pathivada, 2005; Kitazume & Terashi, 2012; Hwang, 2006; Ramirez, 2009) and industry design standards.

Table 2-1 Experimental program for DMM mixed tests

Soil Type	Binder	Binder Dosage	Curing Duration
clayey silt with organics to organic silt	Cement	150 kg/m ³	7, 28, 56
clayey silt with organics to organic silt	Cement	200 kg/m ³	7,14,28,56
clayey silt with organics to organic silt	Cement	250 kg/m ³	7,14,28,56
clayey silt with organics to organic silt	Lime	100 kg/m ³	7,14,28,56
clayey silt with organics to organic silt	Lime	200 kg/m ³	7,14,28,56
clayey organic silt to silty clay	Cement	250 kg/m ³	7,14,28,56

First, wet cement slurry was prepared by mixing cement with water according to a 0.8:1 water to cement ratio based on the finding by Pathivada (2005). Water was added in three separate batches into the cement to allow slurry homogenization. The slurry was mixed with two spatulas and hand for one minute in between each interval of pouring. Clumps and aggregates were crushed to assure a homogeneous mixture. The final state of the slurry should be liquid and runny with no visible clumps or sludge (Figure 2-1).

Second, untreated soils were homogenized with the Hobart A200 Mixer (Hobart, 2005). Samples were transferred to the mixer's metal bowl and placed under the mixer's mechanical mixing hook. A mixing speed setting of Level 1, or 60 RPM, was used for 10 minutes to homogenize the clay soil before cement addition.

Next, a binder slurry was added to the untreated soil in four batches. The mixer was turned off first to add the first batch of binder slurry. Then the mixture was allowed to mix for one minute at the speed Level 1. After one minute, the second batch was added with the same mixing procedure until



Figure 2-1 Slurry mixing and sample mixing procedure

After the soil was thoroughly mixed, it was compacted into pre-fabricated paper tubes. This procedure was to simulate the field condition after mixing was conducted in field. Figure 2-2 illustrates these steps in detail. The steps are described as follows:

1. Cut 6.35cm diameter paper tubes into 33 cm segments
2. Mould the treated soils into a ball by hand with a diameter of approximately 70 mm, which prevents soil sticking to the side of the tube
3. Drop the soil-ball into the tube and use the nail hammer to compact the ball 30 times. This action will puncture air holes within the soil to facilitate pressure compaction, while also applying force to the soil. This step is a modification from the procedures used by Pathivada (2005)
4. Repeat Step 2 and Step 3 until the tube is filled to the top
5. Seal the soil tube with caps and get it ready for curing

Finally, the prepared samples were placed for curing in a water tank (Figure 2-2). This will allow cement to fully react with soil to trigger the pozzolanic reaction (Kitazume & Terashi, 2012), which increases the strength properties of the soil.



Figure 2-2 Sample compaction and curing

2.2 UNCONFINED COMPRESSION STRENGTH TEST

The UCS test was used in this research as the main benchmark test to determine the efficiency of DMM in improving the strength of clays. The tests were conducted in accordance to ASTM D2166. The sample was trimmed with a power saw to ensure two flat end surfaces. Figure 2-3 shows the mechanical saw used in the cutting and sample pictures before and after an UCS test.

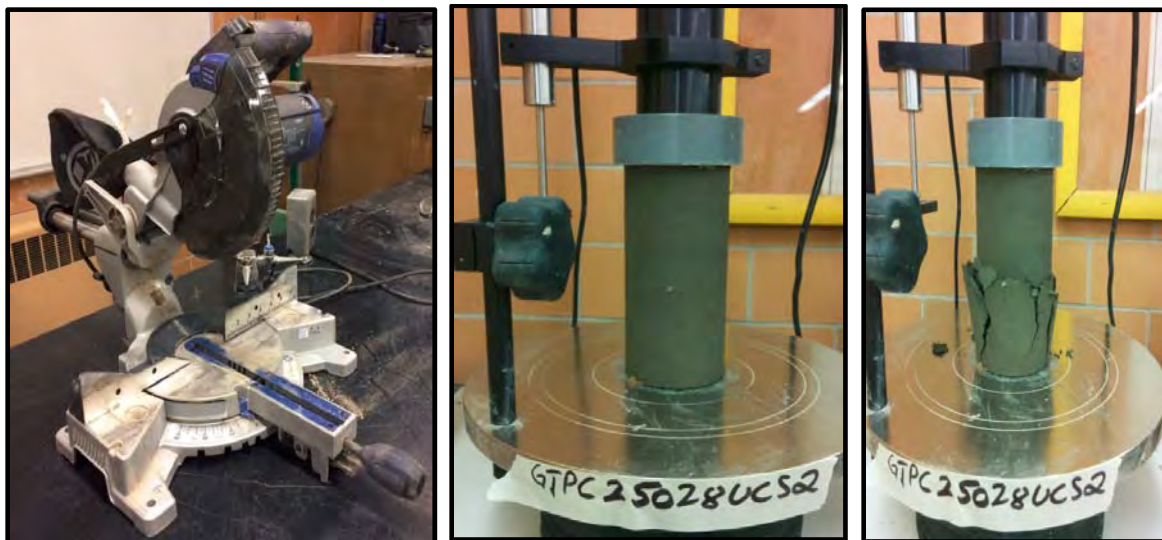


Figure 2-3 Sample trimming mechanical saw and before/after of an UCS test

2.3 SPECIFIC GRAVITY TEST

Specific gravity test was conducted according to ASTM D854. Since organic contents were found in soil, the organic contents were separated from the sample itself using the water floating method. The remaining soil minerals were used to conduct specific gravity test. This procedure was developed specifically to our specific gravity tests after initial test result showed mismatching values due to organic contents. It was not part of the ASTM D854 Standard.

2.4 CRS TEST

The constant rate of strain (CRS) consolidation test was performed according to ASTM D4186. Both cement or lime treated samples and untreated samples were tested to study the influence of DMM on compressibility of soil. Figure 2-4 illustrates the preparation of a sample and the CRS cell.

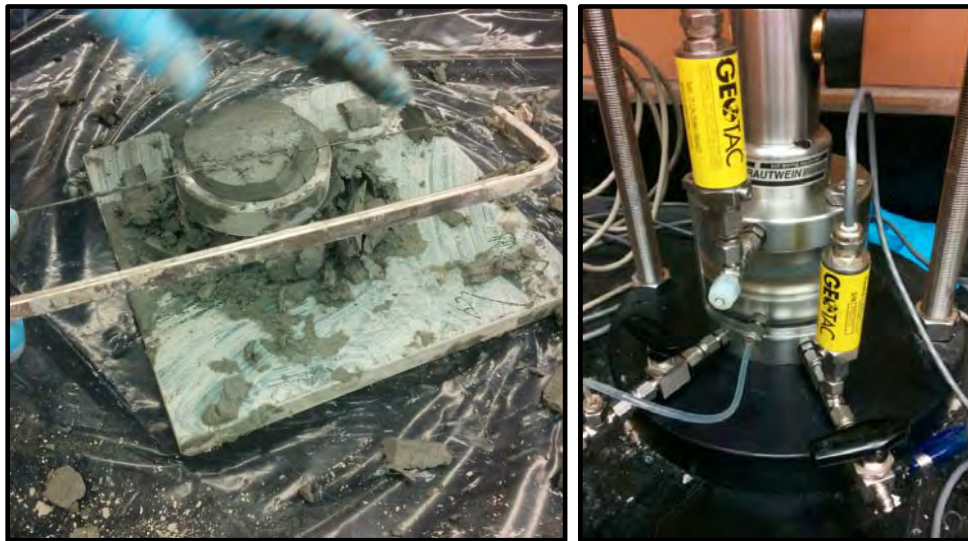


Figure 2-4 CRS sample preparation and set up

3 RESULTS

3.1 UCS TEST RESULT

UCS tests were performed on both untreated samples and cement or lime treated samples. The 76 mm curing paper tubes were changed in the middle of test program to 64 mm tubes due to the limited quantity of soil samples received from the site. Samples treated with 150 kg/m³ and 200 kg/m³ of cement were prepared using 76 mm paper tubes. Samples treated with 250 kg/m³ of cement, 100 kg/m³ and 200 kg/m³ of lime were prepared with 64 mm paper tubes. This section is divided into three sub-sections to present results from untreated clayey organic silt to silty clay soil samples, cement treated soil samples, and lime treated soil samples.

3.1.1 UNTREATED ORGANIC CLAY

Soils from Borehole No. 16-B2 were used to perform UCS test due to the relative undisturbed nature of the sample. GS25 corresponds to Shelby tube No. 4 with a depth of 7.6 m below ground, GS31 corresponds to Shelby tube No. 6 with a depth of 9.4 m below ground, and GS34 corresponds to Shelby tube No. 7 with a depth of 10.4 m below ground. Measured peak UCS ranged from 26.9 kPa to 55.7 kPa with an average value of 42.7 kPa. The UCS tests are shown in Figure 3-1 and summarized in Table 3-1.

Table 3-1 UCS test results for untreated clayey organic silt to silty clay from Shelby tubes

No.	Experiment No.	Peak UCS (kPa)	Approximate Secant Modulus (kPa)
1	GS25UCS1	43.7	542
2	GS31UCS1	47.9	591
3	GS31UCS2	26.9	128
4	GS34UCS1	27.1	219
5	GS34UCS2	55.1	498
6	GS34UCS3	55.7	619

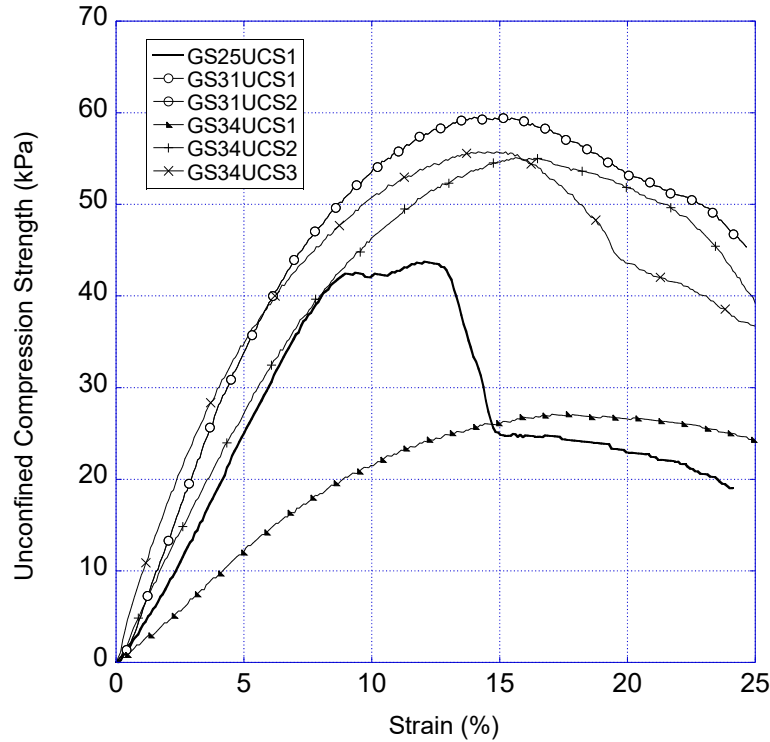


Figure 3-1 Stress-strain curves of untreated clayey organic silt to silty clay from Shelby tube

3.1.2 CEMENT TREATED SOILS

Clayey silt with organics to organic silt were used to perform cement treatment mixing. Cement dosages ranged from 150 kg/m^3 to 250 kg/m^3 with 7, 14, 28, and 56 day curing durations. The UCS tests of cement treated soil are summarized in Table 3-2. The notation of sample number is designed as Golder Test Pit (GTP) soil treated with Cement at a dosage of 150 kg/m^3 and cured at 7 days (C1507) for Unconfined Compression Test number 1 (UCS-1). In similar way, GS33C2507UCS1 represents Golder Shelby 33 feet (GS33) below ground sample treated with Cement at a dosage of 250 kg/m^3 dosage and cured at 7 days for Unconfined Compression Test number 1.

The general use Type 10 cement produced by Holcim was used. After cement treatment, a significant increase was observed in both the peak UCS and Young's modulus of samples. Cement treatment of 250 kg/m^3 was also performed on clayey organic silt to silty clay GS33, which has more organic contents. A similar strength increase was also found in cement treated clayey organic silt to silty clay soil sample. The stress-strain curves of UCS tests for cement treated soils at different dosages and curing time are shown in Figures 3-2 to 3-4. Peak UCS vs. curing duration plots were displayed below to illustrate the effect of cementitious strengthening. Residual UCS vs. curing duration plots were also displayed below. Note that residual UCS of a sample was selected at 15% strain of the corresponding stress vs. strain curve.

Table 3-2 UCS Test Results for Cement Treated Soils

Experiment No.	Binder	Dosage (kg/m ³)	Curing Duration (days)	Peak UCS (kPa)	Residual UCS (kPa)	Approximate Secant Modulus (kPa)
GTPC1507 UCS - 1	C	150	7	407	55	12942
GTPC1507 UCS - 2	C	150	7	287	63	10037
GTPC15028 UCS - 1	C	150	28	280	45	12887
GTPC15028 UCS - 2	C	150	28	397	21	25951
GTPC15056 UCS - 1	C	150	56	342	11	14045.6
GTPC15056 UCS - 2	C	150	56	211	19	9658.1
GTPC2007UCS -1	C	200	7	375	12	23112
GTPC2007UCS -2	C	200	7	315	71	18954
GTPC20014UCS -1	C	200	14	275	28	39680
GTPC20028UCS -1	C	200	28	152	62	6030
GTPC20028UCS -2	C	200	28	390	15	29410
GTPC20056UCS -1	C	200	56	295	79	17850
GTPC20056UCS -2	C	200	56	497	60	22657
GTPC2507UCS -1	C	250	7	385	25	37649
GTPC2507UCS -2	C	250	7	395	68	29614
GTPC25014UCS -1	C	250	14	600	117	29146
GTPC25014UCS -2	C	250	14	609	63	57916
GTPC25028UCS -1	C	250	28	518	40	59971
GTPC25028UCS -2	C	250	28	422	92	56076
GTPC25056UCS -1	C	250	56	489	54	23169
GTPC25056UCS -2	C	250	56	475	7	23133
GS33C2507UCS – 1	C	250	7	404	97	22118
GS33C2507UCS – 2	C	250	7	571	143	34840
GS33C25014UCS – 1	C	250	14	378	26	17860
GS33C25014UCS – 2	C	250	14	601	17	29682

Note: C-cement

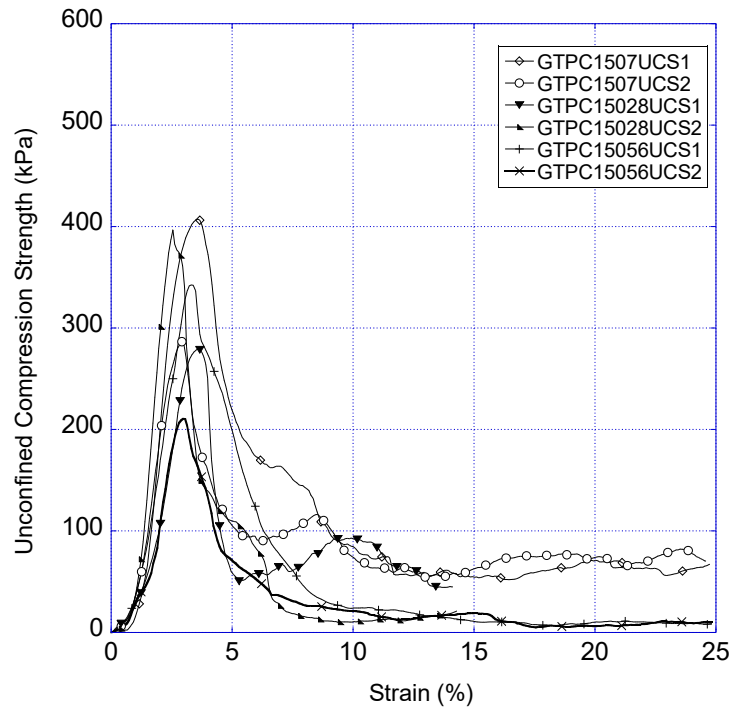


Figure 3-2 Stress-strain curves of UCS tests for 150 kg/m³ cement treated soil samples

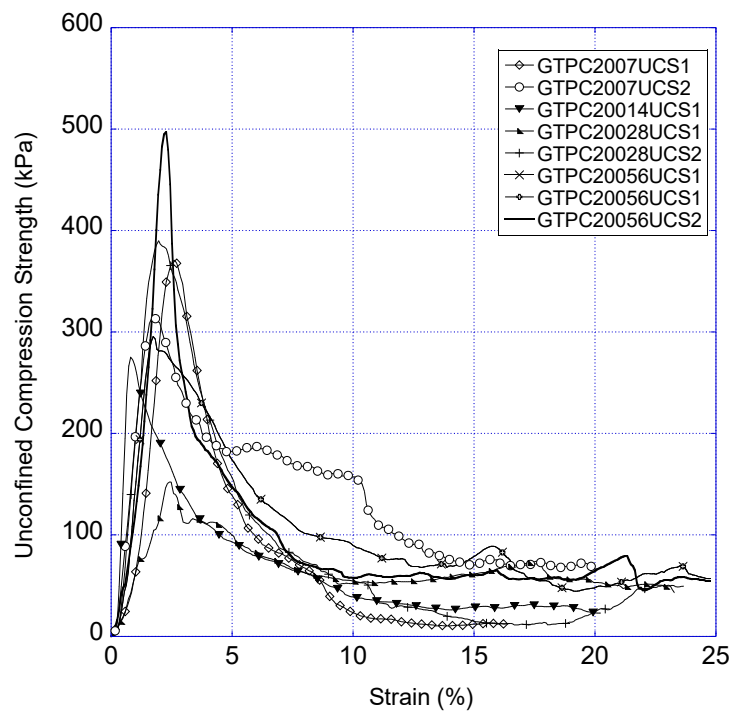


Figure 3-3 Stress-strain curves of UCS tests for 200 kg/m³ cement treated soil samples

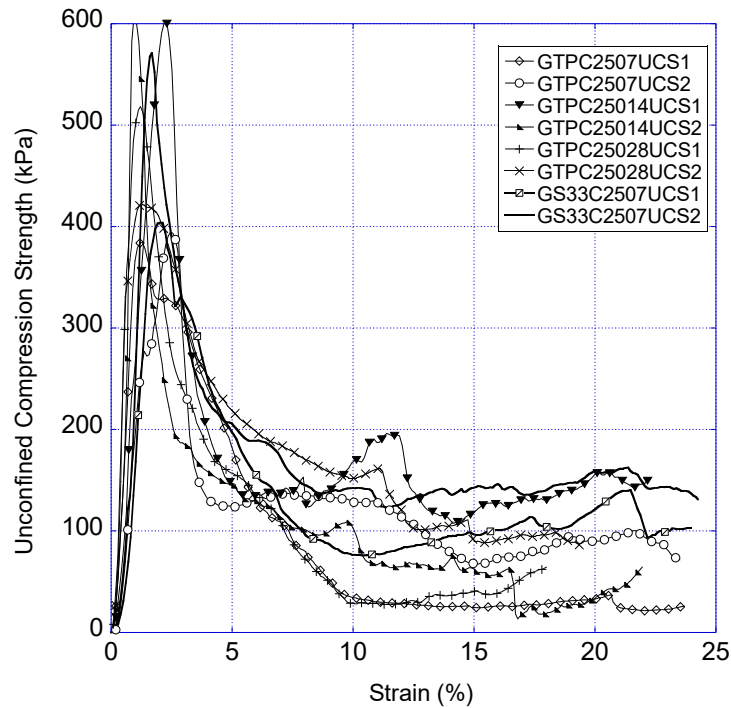


Figure 3-4 Stress-strain curves of UCS tests for 250 kg/m³ cement treated soil samples

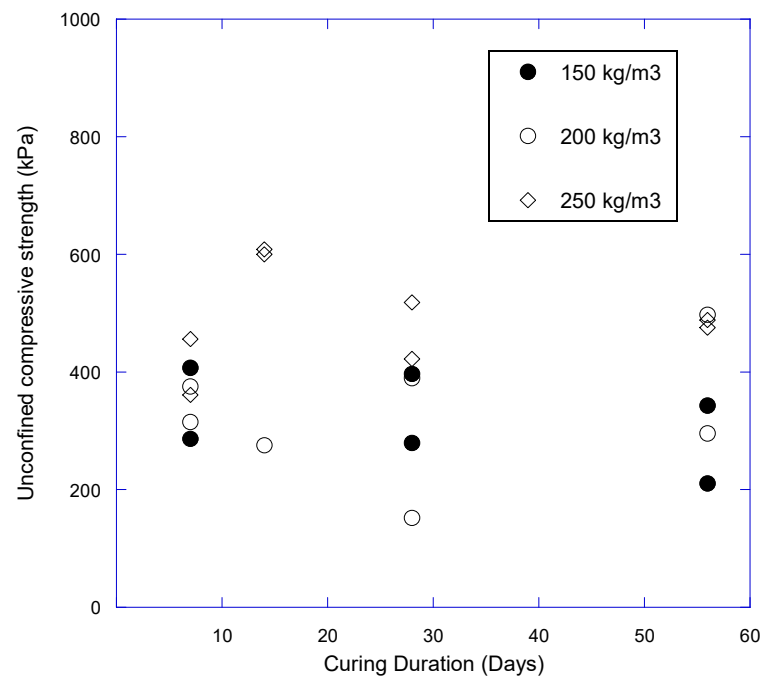


Figure 3-5 Peak UCS vs. curing duration plot for cement treated clayey silt with organics to organic silt

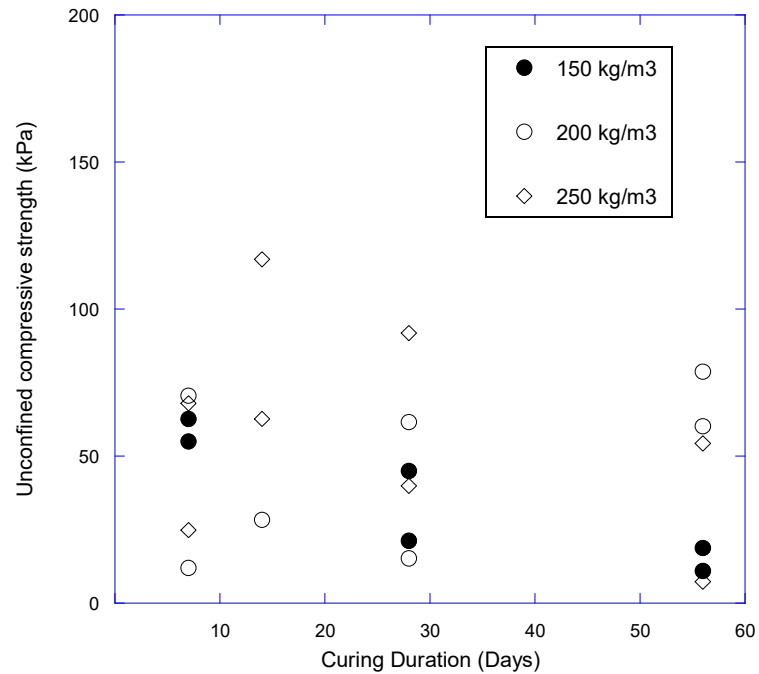


Figure 3-6 Residual UCS vs. curing duration plot for cement treated clayey silt with organics to organic silt

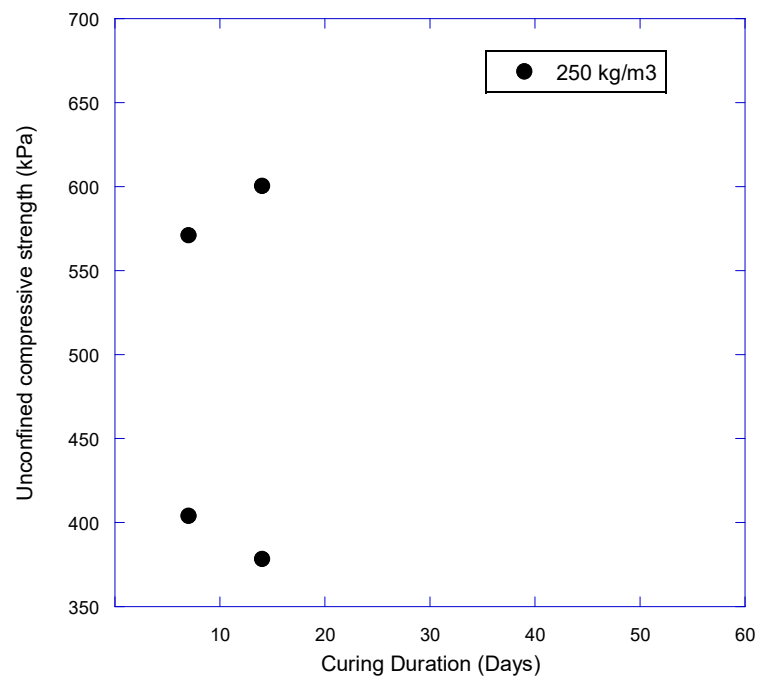


Figure 3-7 Peak UCS vs. curing duration plot for cement treated clayey organic silt to silty clay

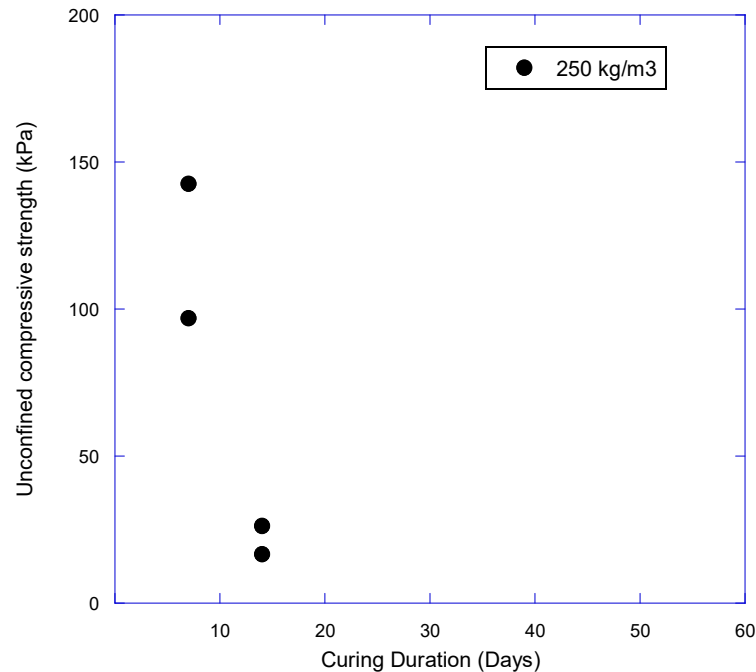


Figure 3-8 Residual UCS vs. curing duration plot for cement treated clayey organic silt to silty clay

3.1.3 LIME TREATED SOILS

The efficiency of lime treatment of clay was tested with clayey silt with organics to organic silt samples taken from the test pits. Lime stabilisation was reported to be effective in strengthening soft clay and marine clay (Balasubramaniam & Bergado, 1989; Locat, 1990). High calcium quicklime donated from Graymont in Boucherville, QC was used in this study. The lime dosages were 100 kg/m³ and 200 kg/m³ and the curing durations are 7, 14, 28, and 56 days. Based on the UCS test results, the efficiency of lime in treating the site clay is very low. Normally lime is not efficient in treating organic clays though it may be excellent for other problematic clays (Eades & Grim, 1960). The summary of UCS tests for lime treated soils is shown in Table 3-3. The stress-strain curves of UCS tests on the lime treated samples for two dosages are shown in Figure 3-5 and Figure 3-6, respectively.

Table 3-3 UCS Test Results for Lime Treated Soils

Experiment No.	Binder	Dosage (kg/m ³)	Curing Duration (days)	Peak UCS (kPa)	Residual UCS (kPa)	Approximate Secant Modulus (kPa)
GTPL1007UCS -1	L	100	7	20	19	187
GTPL1007UCS -2	L	100	7	20	20	178
GTPL10014UCS -1	L	100	14	19	17	175
GTPL10014UCS -2	L	100	14	17	16	180
GTPL10028UCS -1	L	100	28	18	17	146
GTPL10028UCS -2	L	100	28	21	21	149
GTPL2007UCS-1	L	200	7	37	19	665
GTPL2007UCS-2	L	200	7	35	23	520
GTPL20014UCS-1	L	200	14	29	15	450
GTPL20014UCS-2	L	200	14	28	21	334
GTPL20028UCS-1	L	200	28	6	5	36
GTPL20028UCS-2	L	200	28	5	4	42

Note: L-Lime

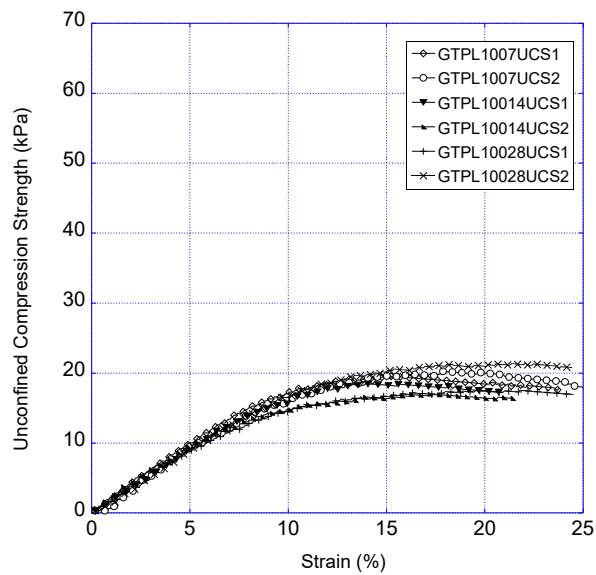


Figure 3-9 Stress-strain curves of UCS tests for 100 kg/m³ lime treated soil samples

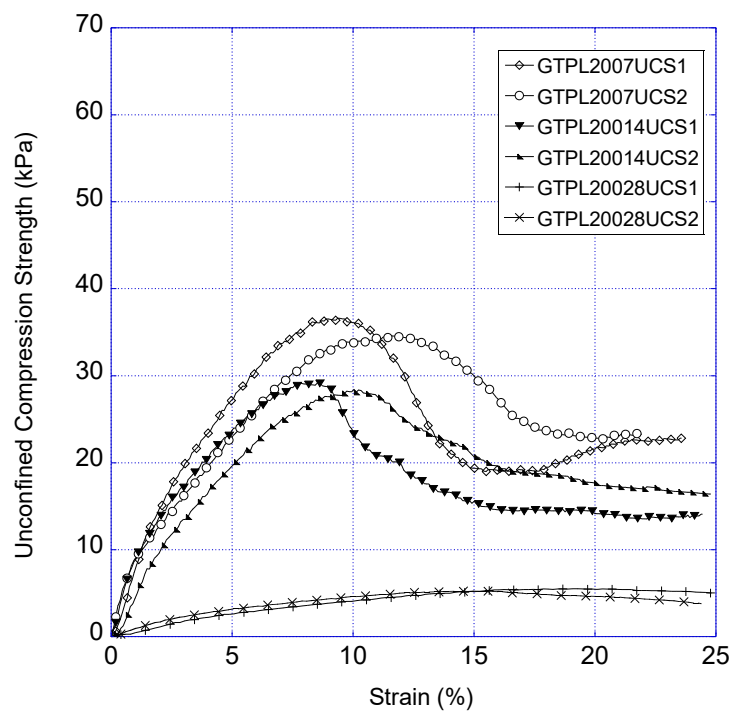


Figure 3-10 Stress-strain curves of UCS tests for 200 kg/m³ lime treated soil samples

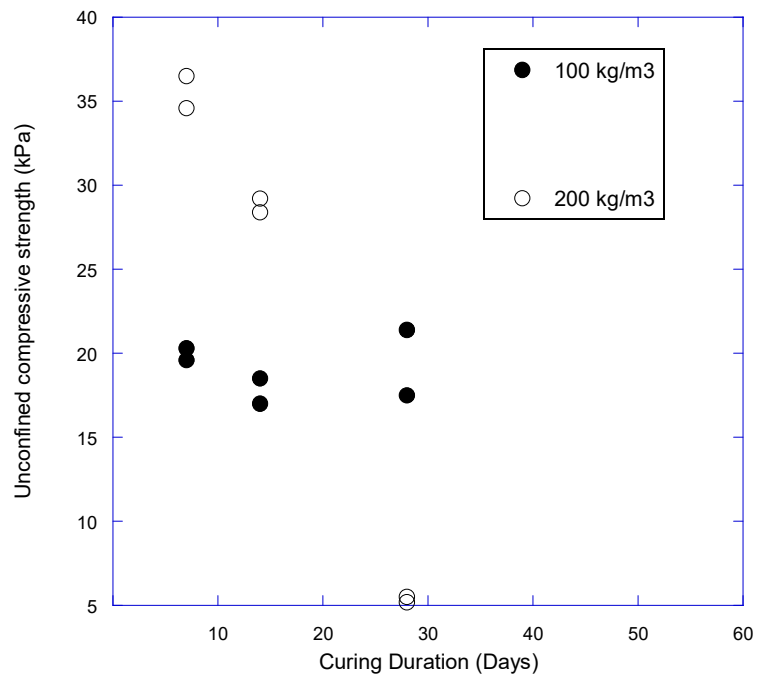


Figure 3-11 Peak UCS vs. curing duration plot for lime treated clayey silt with organics to organic silt

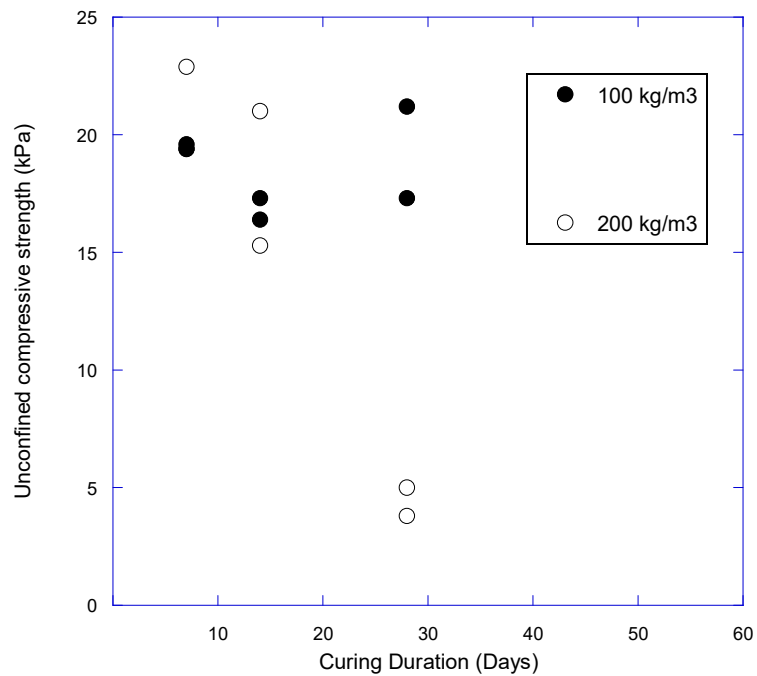


Figure 3-12 Residual UCS vs. curing duration plot for lime treated clayey silt with organics to organic silt

3.2 CRS TEST RESULT

A series of constant rate of strain (CRS) tests were conducted for cement treated samples at a dosage of 150 kg/m³, 200 kg/m³, and 250 kg/m³ and cured at 7 days. The CRS tests was also performed for a lime treated samples at a dosage of 100 kg/m³ and cured at 7 days and an untreated samples. The untreated clayey organic silt to silty clay sample was taken from Shelby tube while other samples were clayey silt with organics to organic silt from test pit. For GTPC150, GTPC200, and GTPC250, the CRS device's linear variable differential transformer (LVDT) sensor experienced a position error, which required a thorough data investigation. Data curves were modified to represent the most accurate portions of the data. Figures 3-13, Figure 3-18, and Figure 3-20 shows the e-log p curves for cement treated, lime treated, and untreated soil samples respectively. Compression index (C_c), recompression index (C_R), and effective compression index (C'_c) were interpreted from the e log P curves. Equations 3-1 to 3-3 were used to calculate C_c , C_R , and C'_c respectively.

$$C_c = \frac{e_1 - e_2}{\log P_2 - \log P_1} \quad \text{Equation 3-1}$$

$$C_R = \frac{e_1 - e_2}{\log P_2 - \log P_1} \quad \text{Equation 3-2}$$

$$C'_c = \frac{C_c}{1 + e_0} \quad \text{Equation 3-3}$$

Where e_1 & e_2 are the respective void ratios on the second loading stage of the e log P curves, and P_1 & P_2 are the respective applied loads. Table 3-4 displays the interpolated parameters.

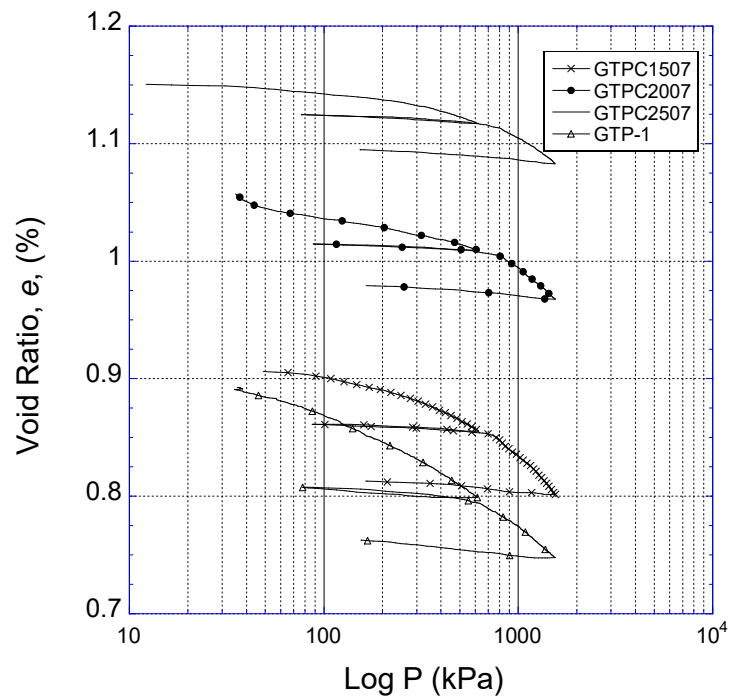


Figure 3-13 e vs. $\log p$ for cement treated clayey silt with organics to organic silt and untreated clayey silt with organics to organic silt

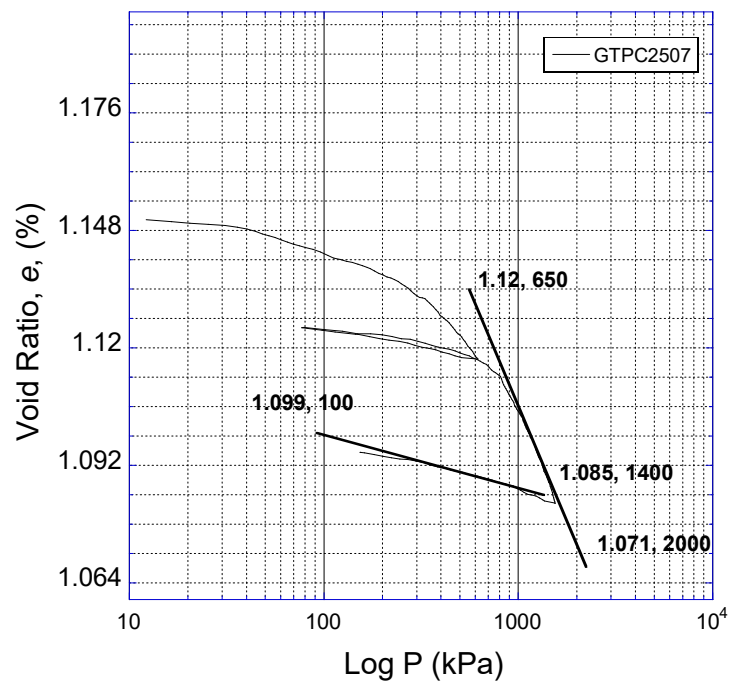


Figure 3-14 Compression index and recompression index for 250 kg/m³ cement treated clayey silt with organics to organic silt

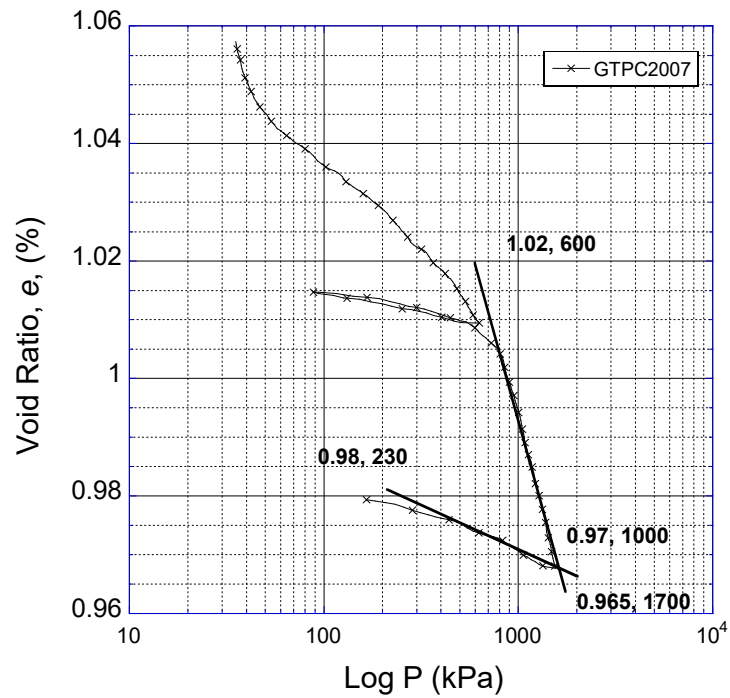


Figure 3-15 Compression index and recompression index for 200 kg/m³ cement treated clayey silt with organics to organic silt

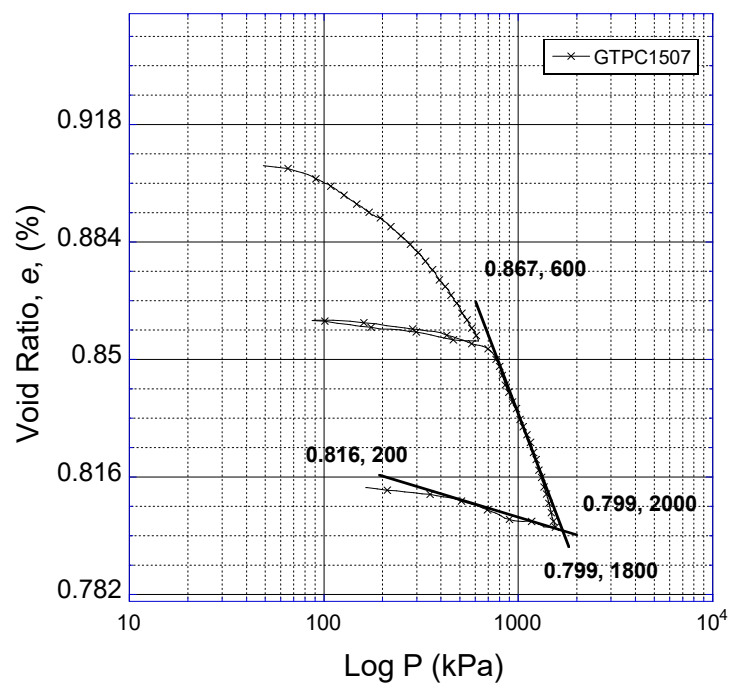


Figure 3-16 Compression index and recompression index for 150 kg/m³ cement treated clayey silt with organics to organic silt

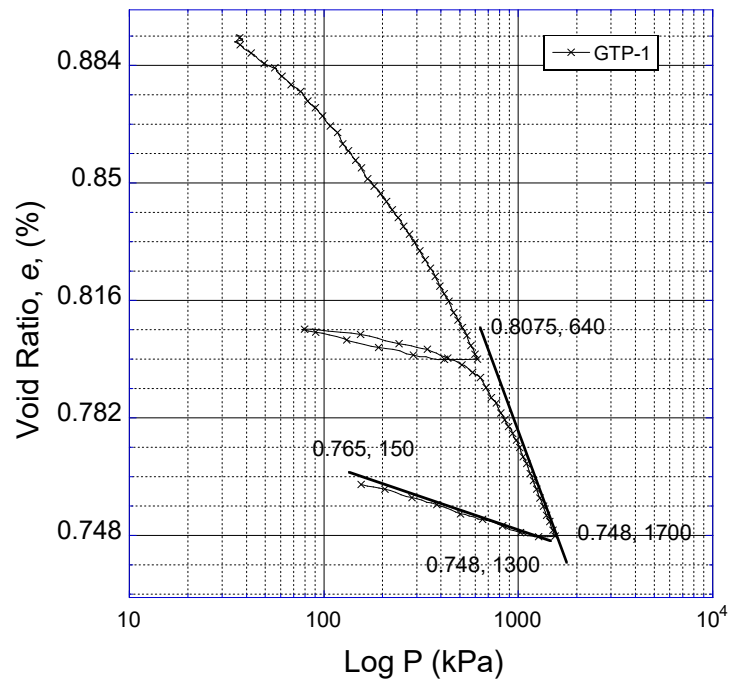


Figure 3-17 Compression index and recompression index for untreated clayey silt with organics to organic silt sample

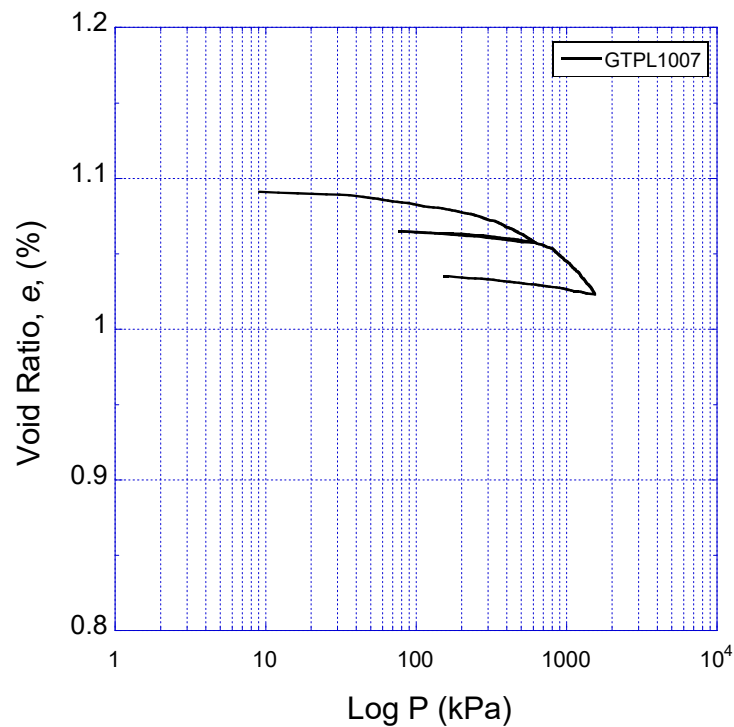


Figure 3-18 e vs. $\log p$ for lime treated clayey silt with organics to organic silt

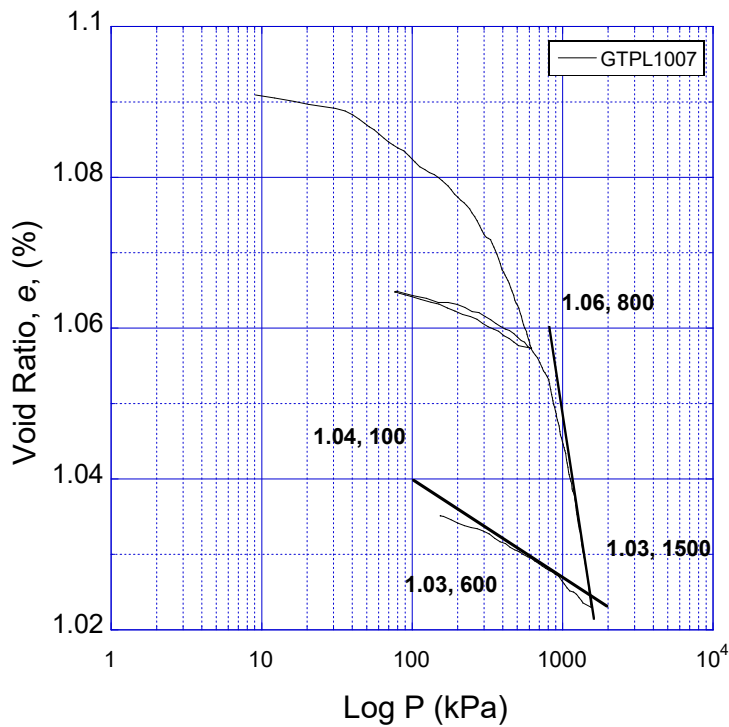


Figure 3-19 Compression index and recompression index for lime treated clayey silt with organics to organic silt

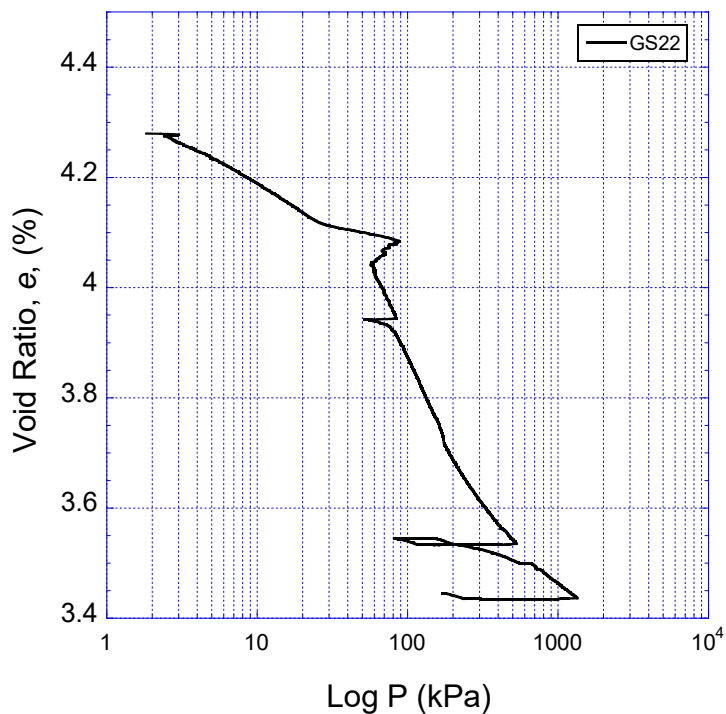


Figure 3-20 e vs. $\log p$ for an untreated clayey organic silt to silty clay sample

Table 3-4 CRS calculated parameters

Experiment No.	C_c	C_R	e_0	C'_c
GTP-1	0.14	0.018	1.05	0.068
GTPC1507	0.14	0.017	1.18	0.064
GTPC2007	0.122	0.016	1.36	0.052
GTPC2507	0.1	0.012	1.21	0.045
GTPL1007	0.11	0.013	1.15	0.051
GS22	0.22	0.075	4.28	0.042

3.3 TRIAXIAL TEST RESULTS

A total of three isotropically consolidated undrained (CIU) triaxial compression tests were conducted on clayey organic silt to silty clay samples taken from Shelby tube No. GS22. The confining pressures were instructed by Golder Associates Ltd. as 125, 75, and 25 kPa for the three tests. The summaries of test results are shown in Table 3-4. The volume change of sample with time during consolidation is shown in Figure 3-10. The stress-strain curves along with the stress path curves are shown in Figures 3-11 and 3-12, respectively.

Table 3-4 Summary of CIU Triaxial Compression Tests on Clayey Organic Silt to Silty Clay Samples

Test Name	OCEGS22- CIU1	OCEGS22 -CIU 3	OCEGS22- CIU 4
Shelby tube number	GS 22	GS 22	GS 22
Sample depth from ground surface (m)	6.7	6.5	6.4
Specimen diameter (mm)	38.32	40.13	41.04
Specimen height (mm)	75.33	80.50	82.00
Natural water content (%)	212.9	175.9	142.7
Water content after test (%)	168.7	141.2	152.3
Dry density (kN/m^3)	3.77	4.45	5.23
Cell pressure, σ_3 , (kPa)	260	210	160
Back pressure (kPa)	135	135	135
Pore pressure parameter, B	0.90	0.94	0.90

Volumetric strain during consolidation (%)	6.0	2.7	4.6
Average rate of strain (%/hour)	1.0	1.0	1.0
Maximum deviator stress, $(\sigma_1 - \sigma_3)_{\max}$, (kPa)	150.7	110.6	56.5
Axial strain at $(\sigma_1 - \sigma_3)_{\max}$, (%)	16.8	13.7	19.7
Excess pore pressure at $(\sigma_1 - \sigma_3)_{\max}$ (kPa)	97.1	54.6	17.8

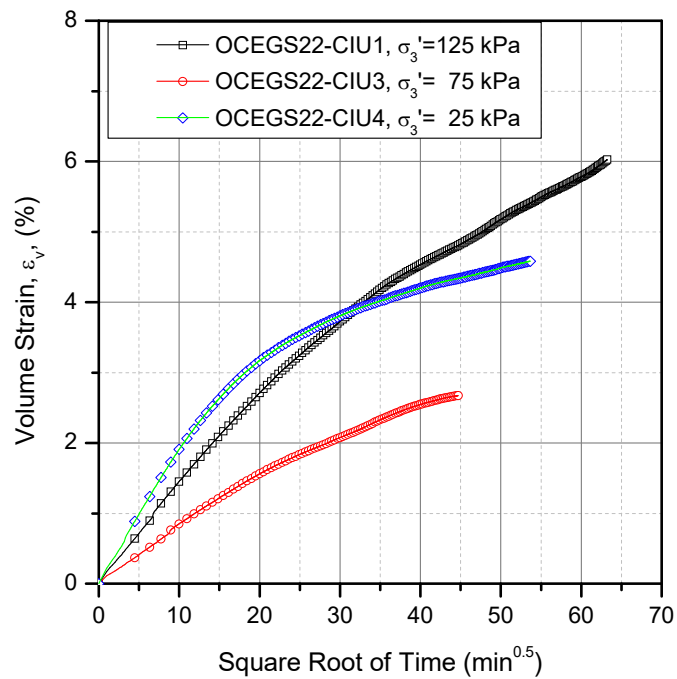


Figure 3-21 Volume change of untreated clayey organic silt to silty clay samples with time during consolidation

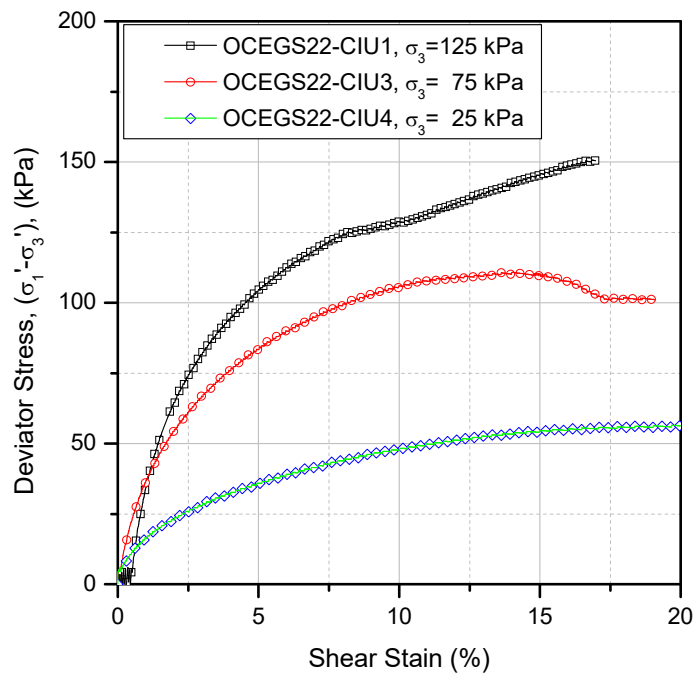


Figure 3-22 Stress-strain curves of untreated clayey organic silt to silty clay samples during shearing

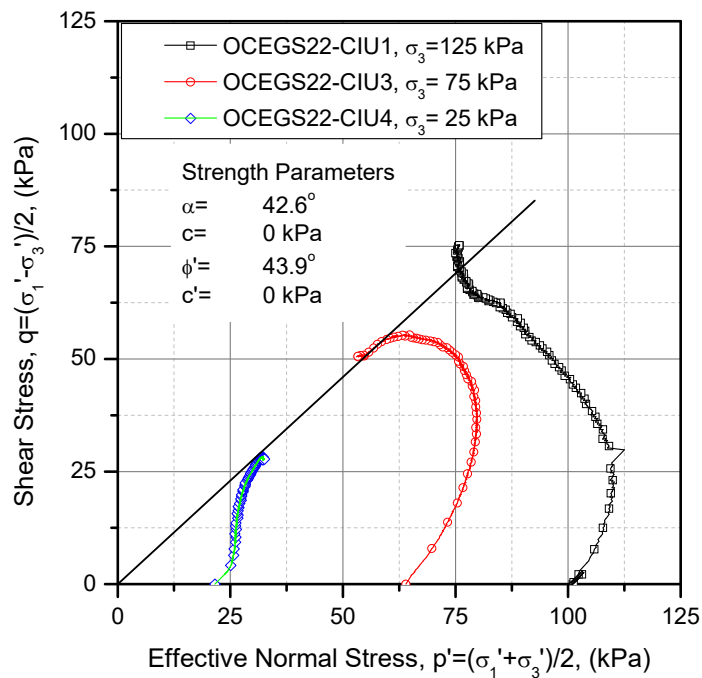


Figure 3-23 Stress path curves of untreated clayey organic silt to silty clay samples during shearing

4 SUMMARY AND FUTURE RESEARCH

The feasibility of applying deep soil mixing method in treating organic clays in Ontario was investigated in this study. Two types of clay samples were collected from a project site near the intersection between Highway 400 and Side Road 16: disturbed clayey silt with organics to organic silt collected from a test pit and shipped in buckets and undisturbed clayey organic silt to silty clay in Shelby tubes taken from deeper depths. Due to the quantity of soil samples available for tests, more tests were conducted using the clayey silt with organics to organic silt. The test variables included binder type, binder dosage, curing duration, test methods. Both cement and lime were tested as a binder for the clayey silt with organics to organic silt samples. clayey organic silt to silty clay soils were mixed with cement at a dosage of 250 kg/m³ and cured for varying durations. The raw test curves are attached in the appendices.

Based on the test results, a few preliminary conclusions can be drawn as follows:

1. Cement is efficient in treating both silty and organic clays at the project site. Based on the UCS and CRS tests, a significant improvement was observed in strength and compressibility.
2. Lime is not efficient for treating the soil samples from the UCS test results. The reason behind of this less encouraging results will be investigated.

The current findings are preliminary. Future studies will be required to gain a complete knowledge and interaction between organic clay and strengthening binders. The subjects include:

1. Conduct further detailed analysis of test results presented in this report
2. Conduct further testing for long-term performance of DMM in treating organic clays in Ontario
3. Perform a thorough characterization of clays in the site with more tests
4. Applying cement/slag, or other binders to treat organic clay
5. Investigate the influence from soil pH level on the efficiency of DMM
6. Utilize scanning electron microscopy (SEM) or transmission electron microscopy (TEM) to understand soil-binder interactions
7. Perform X-ray diffraction analysis (XRD) to understand the interaction between soil and binder
8. Conduct a pilot project to evaluate the field performance of DMM in treating organic clays in Ontario
9. Study the interaction of DMM block or column with surrounding soil
10. Study the failure modes of DMM blocks in soft organic clay
11. Perform exhaustive cost benefit analysis for the deep soil mixing technique in Ontario

5 ACKNOWLEDGEMENT

The authors wish to acknowledge the financial support from the Ontario Centres of Excellence and the sponsorship of Golders Associates Ltd. The supports from Mr. Al Varshoi and Ms. Sandra McGaghran of Golders Associates Ltd. throughout of this project are greatly acknowledged. The helps from Ms. Naresh Gurpersaud of Geo-Foundations Contractor Inc., Mr. Andries Kirstein, Mr. Cong Shi, and Mr. Markus Jesswein of GeoOptical Research Lab of Ryerson University are also greatly appreciated.

6 REFERENCE

- ASTM. (2000). Standard Test Method for Laboratory Miniature Vane Shear Test for Saturated Fine-Grained Clayey Soil. West Conshohocken, PA: American Society for Testing and Materials.
- ASTM. (2000). Standard Test Method for Unconfined Compressive Strength of Cohesive Soil. West Conshohocken, PA: American Society for Testing and Materials.
- ASTM. (2010). *Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass*. West Conshohocken: American Society of Testing and Materials.
- ASTM. (2014). *Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils*. West Shohocken: American Society of Testing and Materials.
- ASTM. (2014). *Standard Test Method for One-Dimensional Consolidation Properties of Saturated Cohesive Soils Using Controlled-Strain Loading D4186*. West Conshohocken: American Society of Testing and Materials.
- ASTM. (2014). *Standard Test Methods for Specific Gravity of Soil Solids by Water Pycnometer D854-14*. West Conshohocken: American Society of Testing and Materials.
- Baker, S. (2015). Laboratory Evaluation of Organic Soil Mixing. Tampa: University of South Florida.
- Balasubramaniam, A., & Bergado, D. (1989). Strength and deformation characteristics of lime-treated soft clays. *Geotechnical Engineering*, 49-65.
- Bergado, D., Ruenkairergsa, T., Taesiri, Y., & Balasubramaniam, A. (1999). Deep Soil Mixing Used to Reduce Embankment Settlement. *Ground Improvement*, 145-162.
- Bruce, M., Berg, R., Collin, J., Filz, G., Terashi, M., & Yang, D. (2013). *Federal Highway Administration Design Manual: Deep Mixing for Embankment and Foundation Support*. McLean: US Department of Transportation.
- Eades, J., & Grim, R. (1960). *Reaction of hydrated lime with pure clay mineral in soil stabilization*. Highway Research Board Bulletin.
- Hobart. (2005, April). *A200 Mixer Technical Manual*. Retrieved March 3rd, 2016, from Hobart Corp:
<https://my.hobartcorp.com/kroger/TechnicalServiceManuals/A200%20Technical%20Manual.pdf>
- Hwang, J. (2006). *Effects of Cement Treatment on the 1-D Consolidation Behavior of a Highly Organic Soil*. West Lafayette: 2006.
- Kitazume, M., & Terashi, M. (2012). *The Deep Mixing Method*. Tokyo: CRC Press.

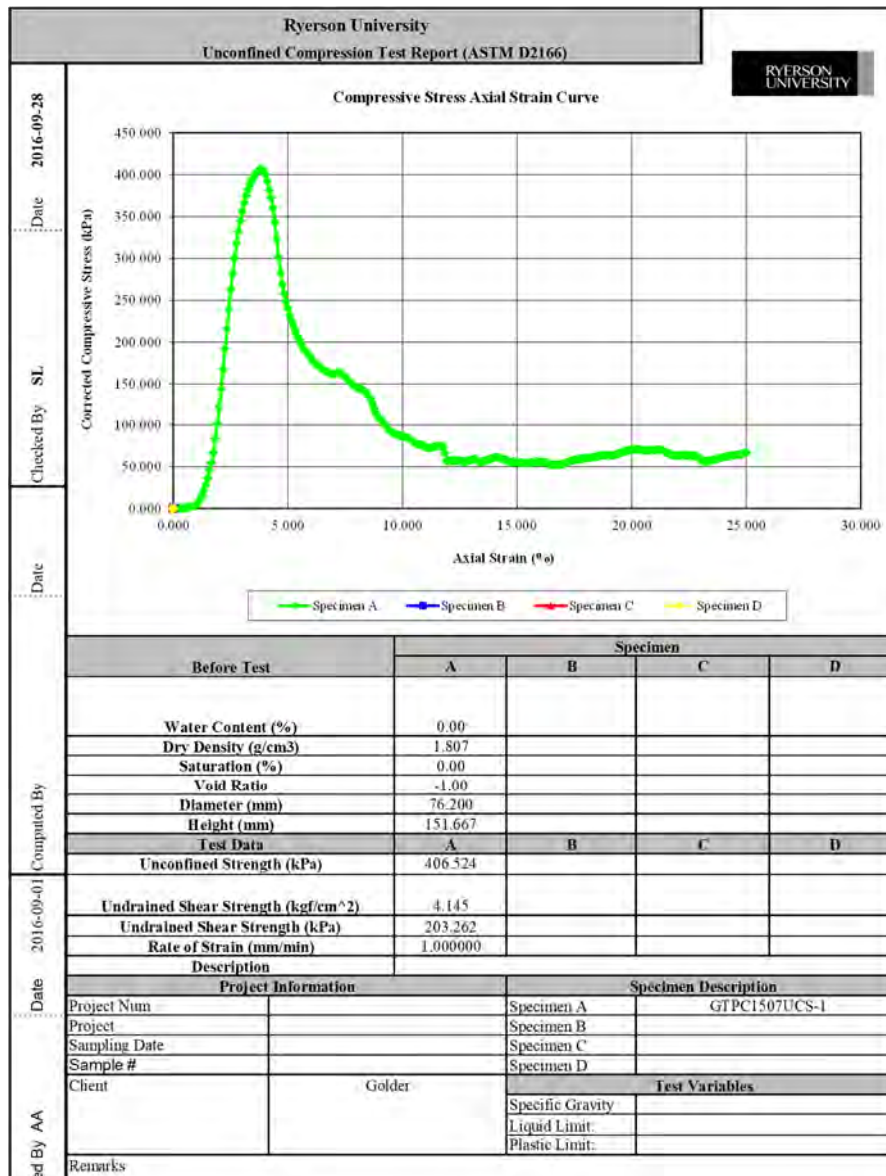
- Li, S., Kirstein, A., Gurbarsud, N., & Liu, J. (2016). Experimental investigation of cement mixing to improve Champlain Sea clay. *GeoVancouver 2016*. Vancouver: Canadian Geotechnical Society.
- Locat, J., Berube, M., & Choquette, M. (1990). Laboratory Investigation on the Lime Stabilization of Sensitive Clays: Shear Strength Development. *Canadian Geotechnical Journal*, 294 - 305.
- Pathivada, S. (2005). *Effects of Water-Cement Ratio on Deep Mixing Treated Expansive Clay Characteristics*. The University of Texas at Arlington.
- Ramirez, J. (2009). *Cement Stabilization of Organic Soils for Controlling Secondary Compression Behavior*. Boca Raton: Florida Atlantic University.

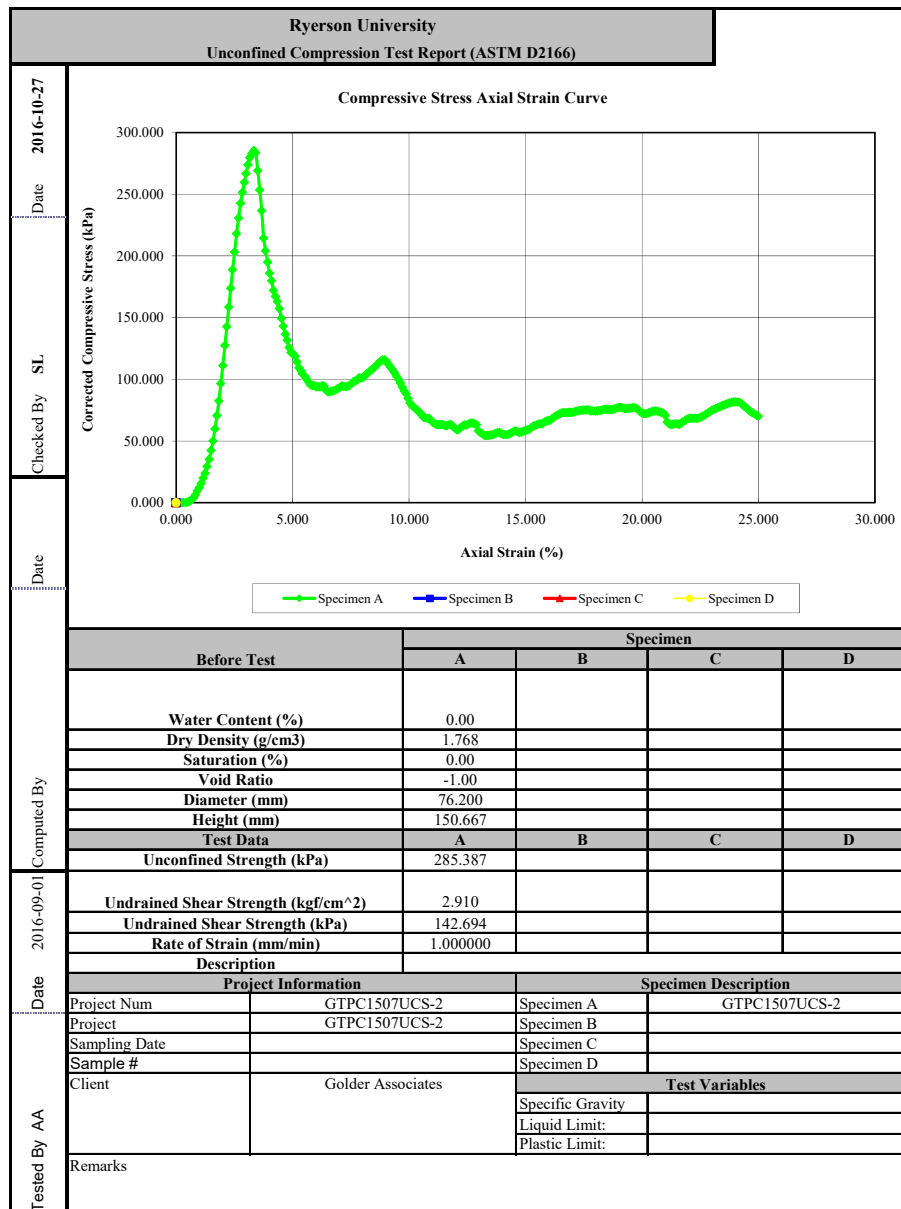
7 APPENDIX

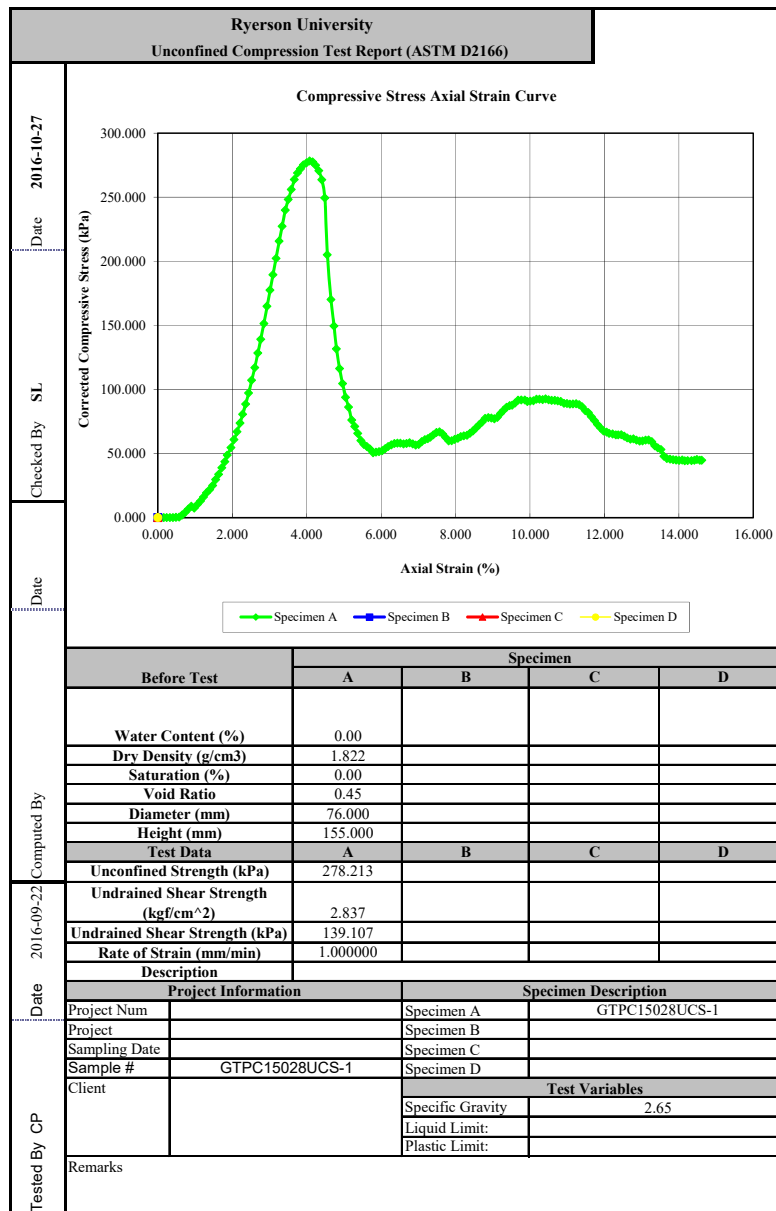
List of appendices:

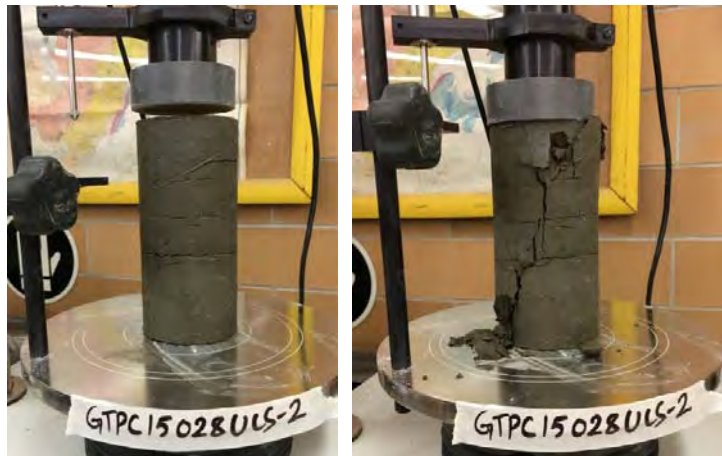
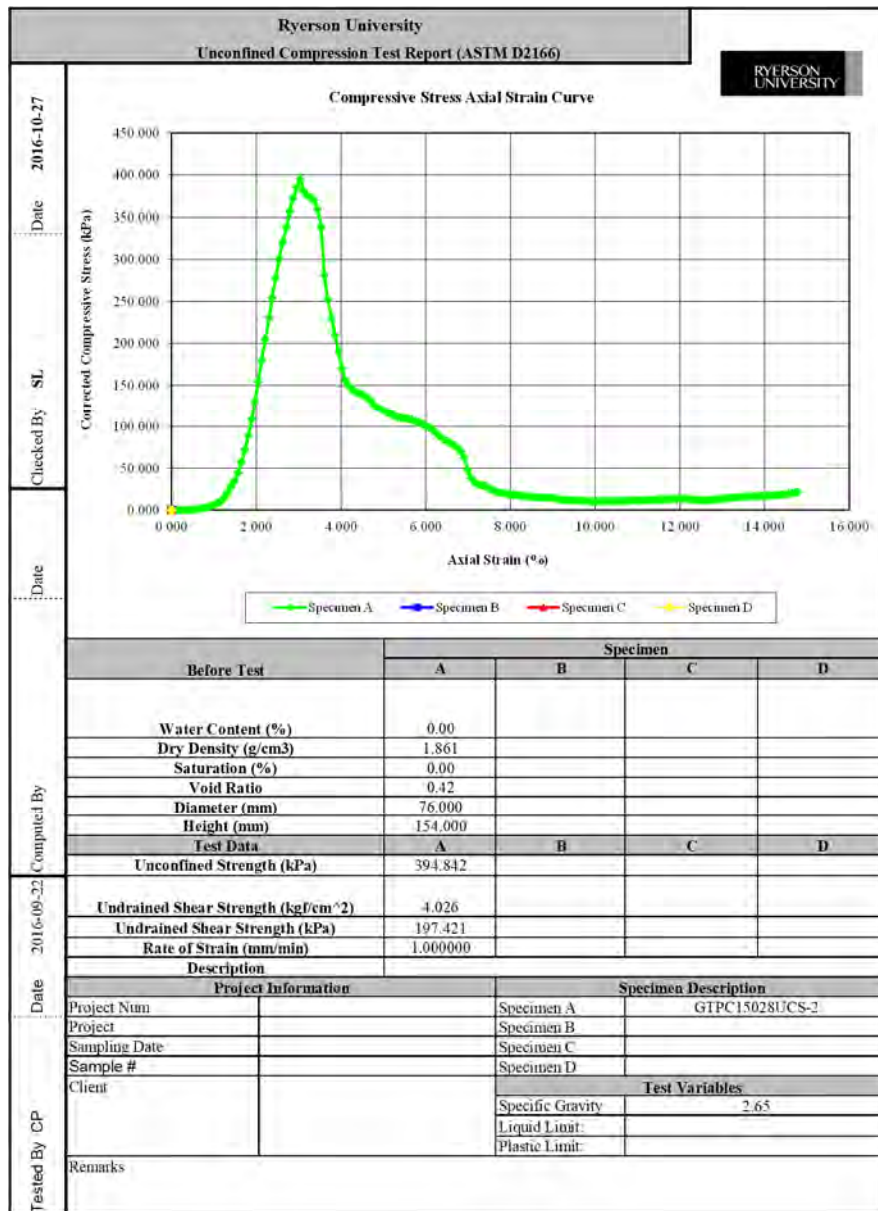
7.1	UCS Test Results of cement treated silty clay (150kg/m ³).....
7.2	UCS Test Results of cement treated silty clay (200kg/m ³).....
7.3	UCS Test Results of cement treated silty clay (250kg/m ³).....
7.4	UCS Test Results of cement treated Organic clay (250kg/m ³).....
7.5	UCS Test Results of Lime treated Silty clay (100 kg/m ³)
7.6	UCS Test Results of Lime treated Silty clay (200 kg/m ³)
7.7	UCS Test Results of Untreated Organic Clays
7.8	Specific Gravity Test.....
7.9	Miniature Vane Shear Test.....
7.10	Triaxial Test Results of Organic Clays

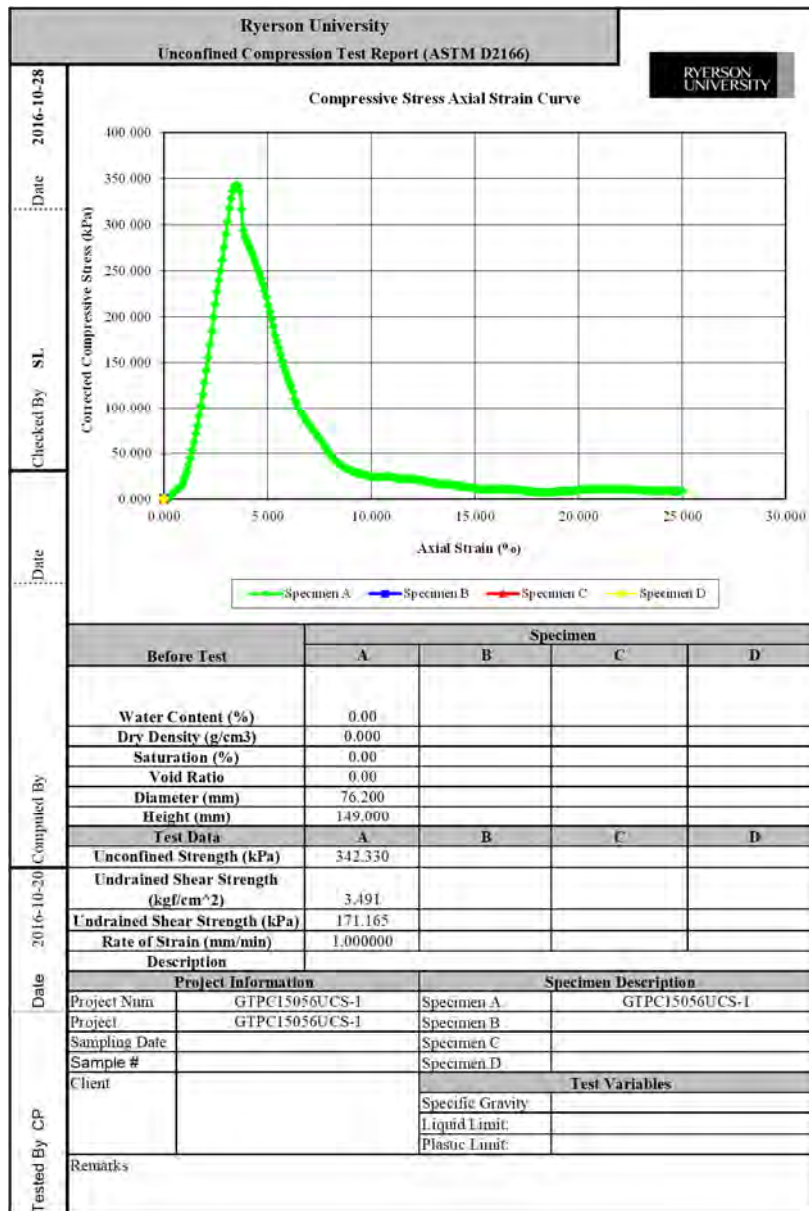
7.1 UCS TEST RESULTS OF CEMENT TREATED SILTY CLAY (150KG/M³)

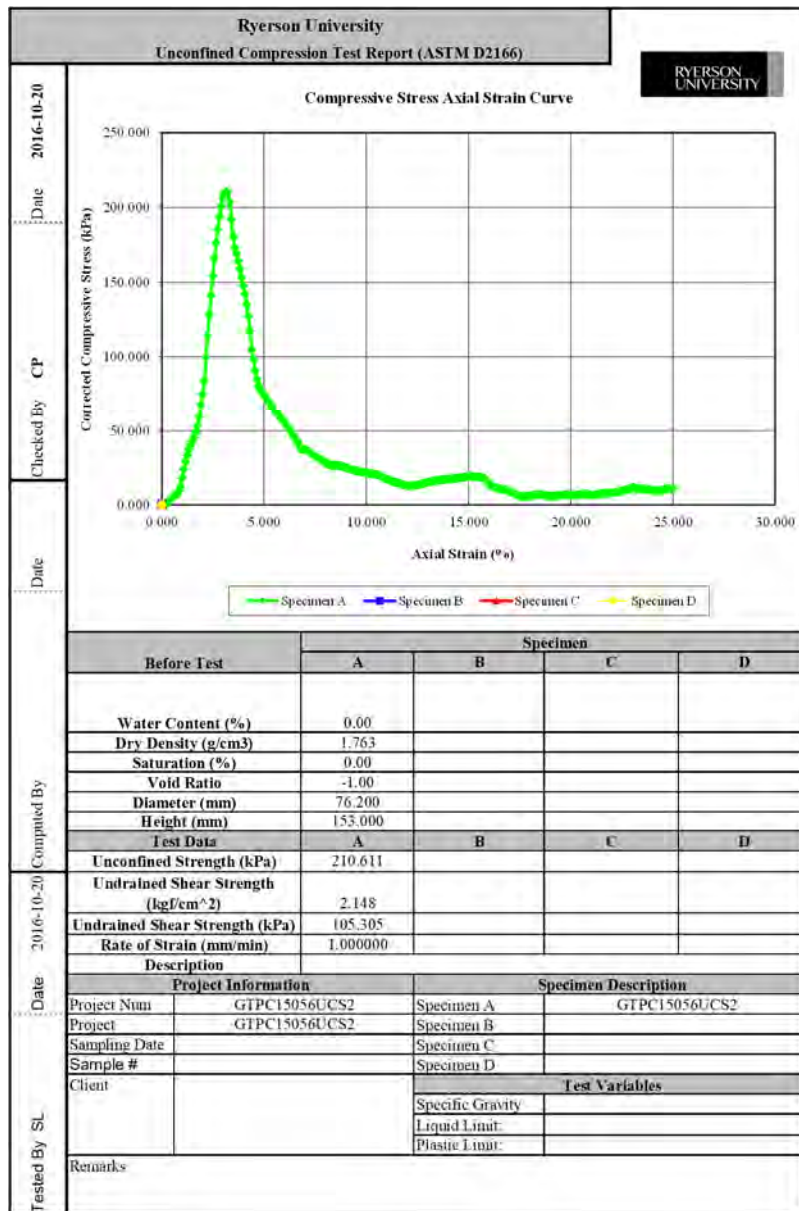




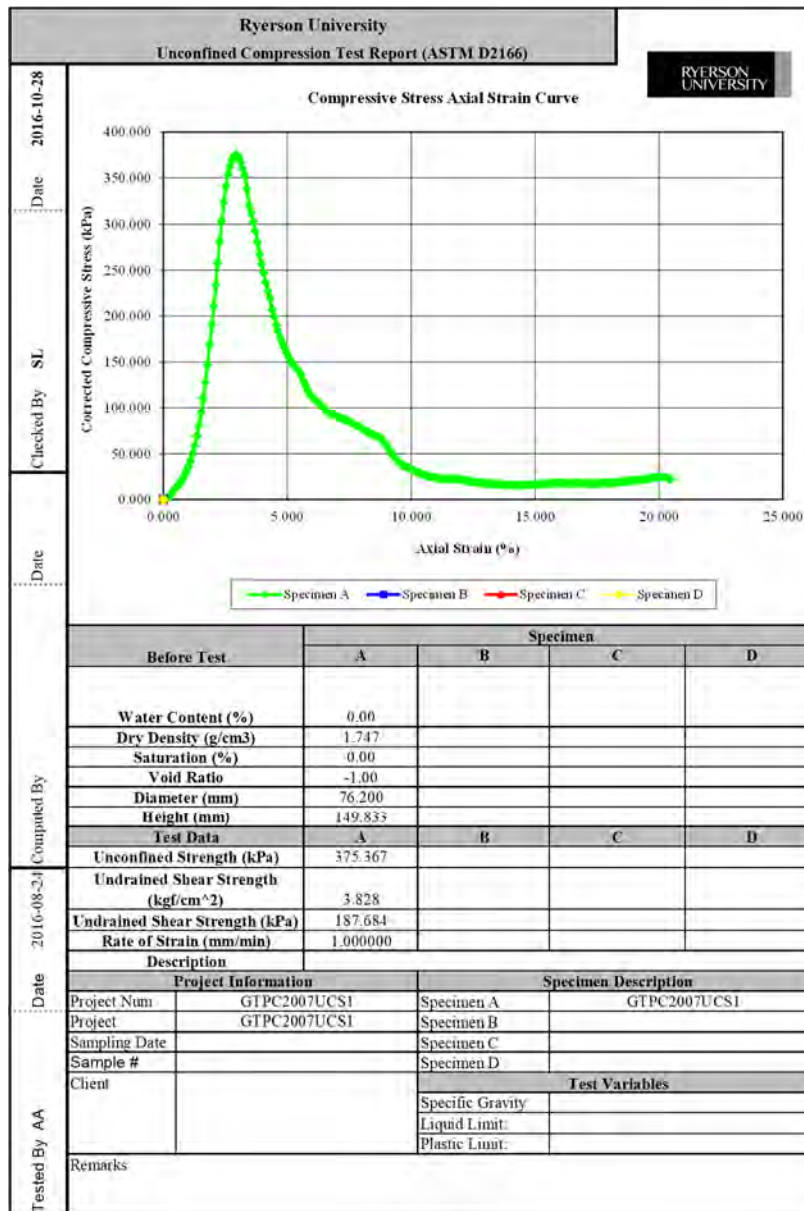


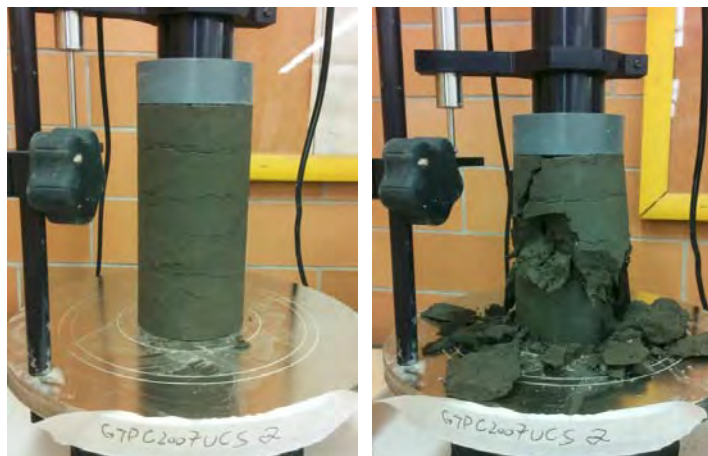
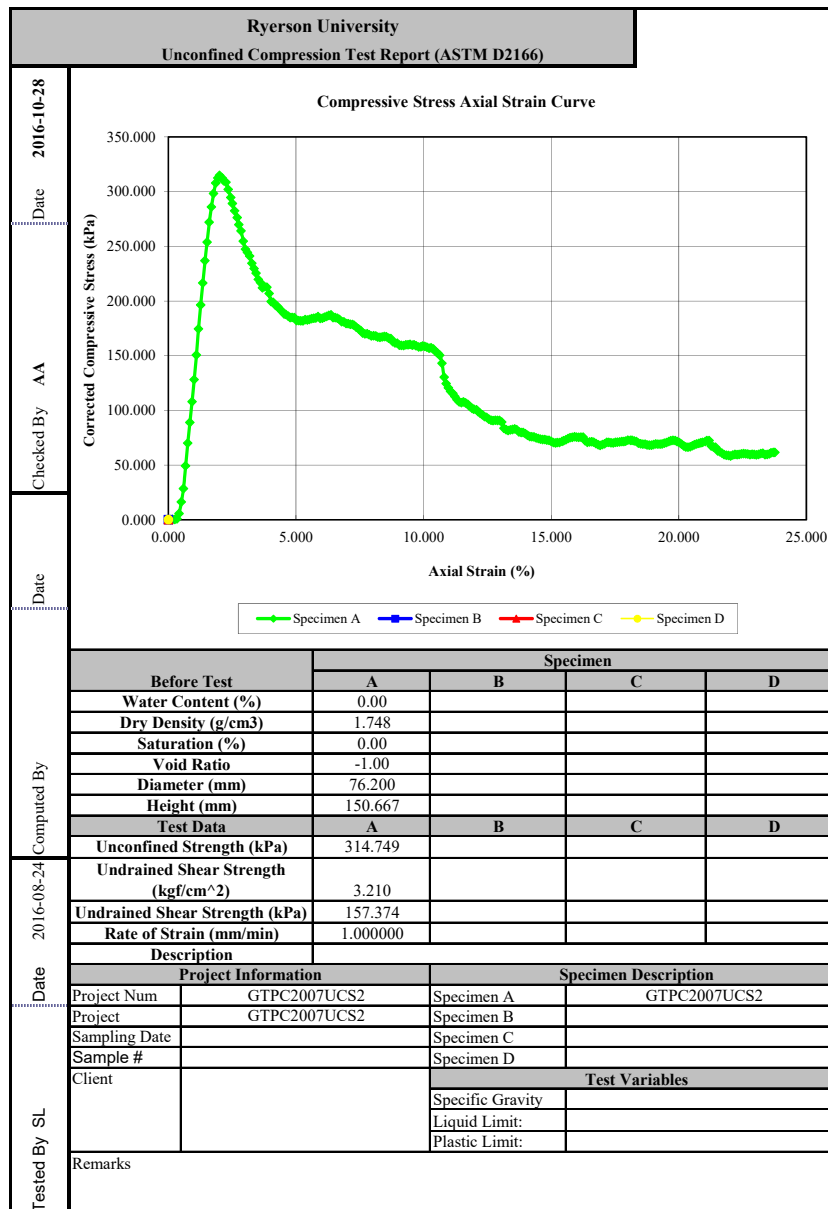


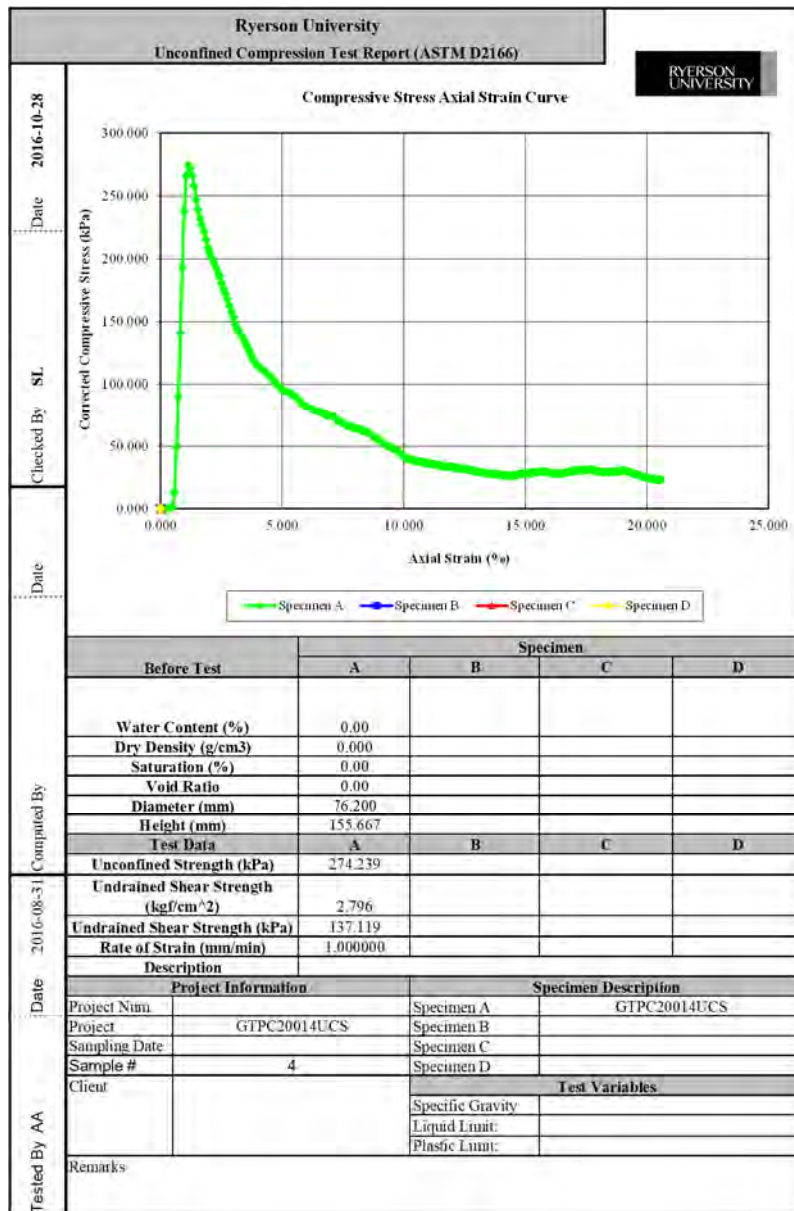


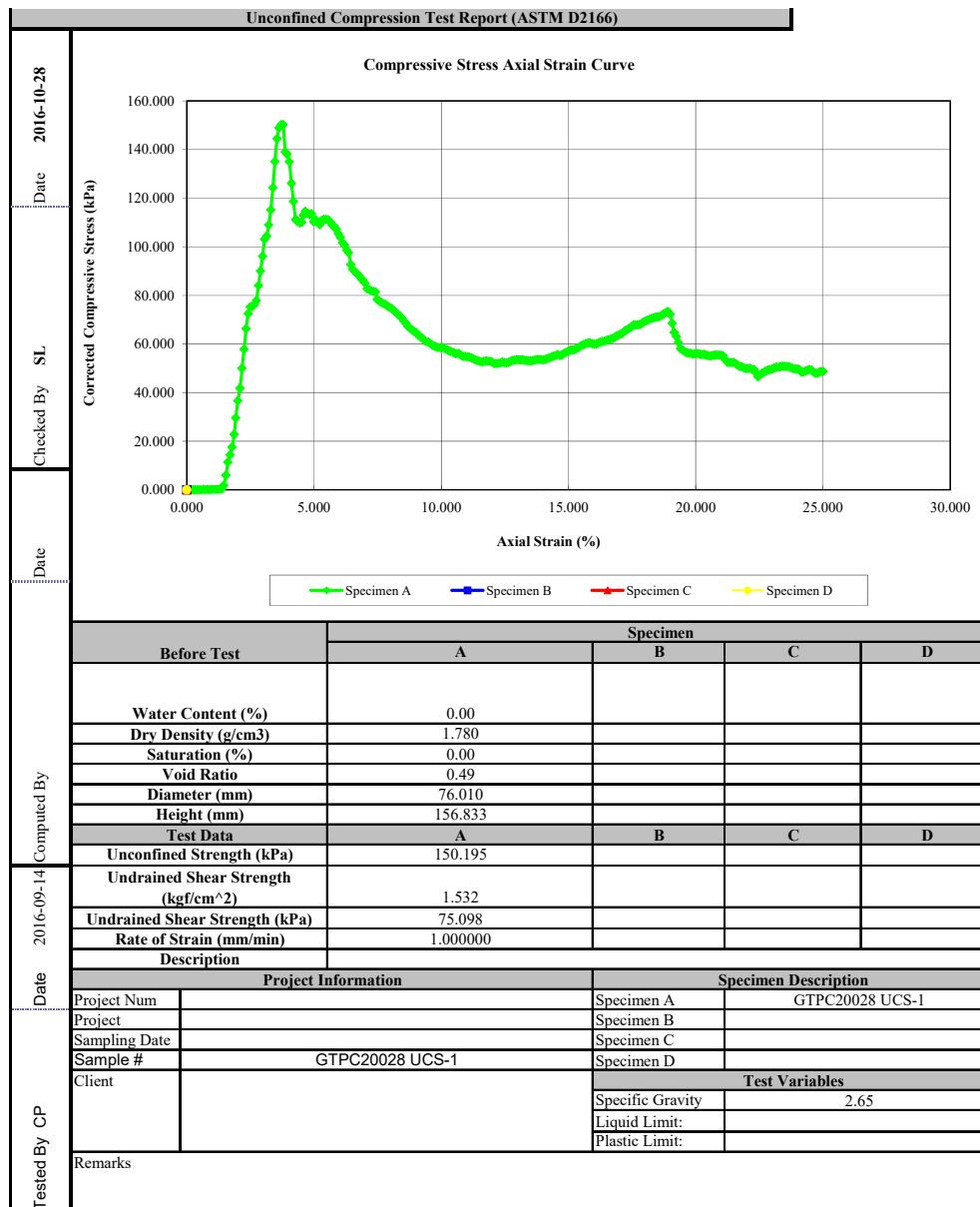


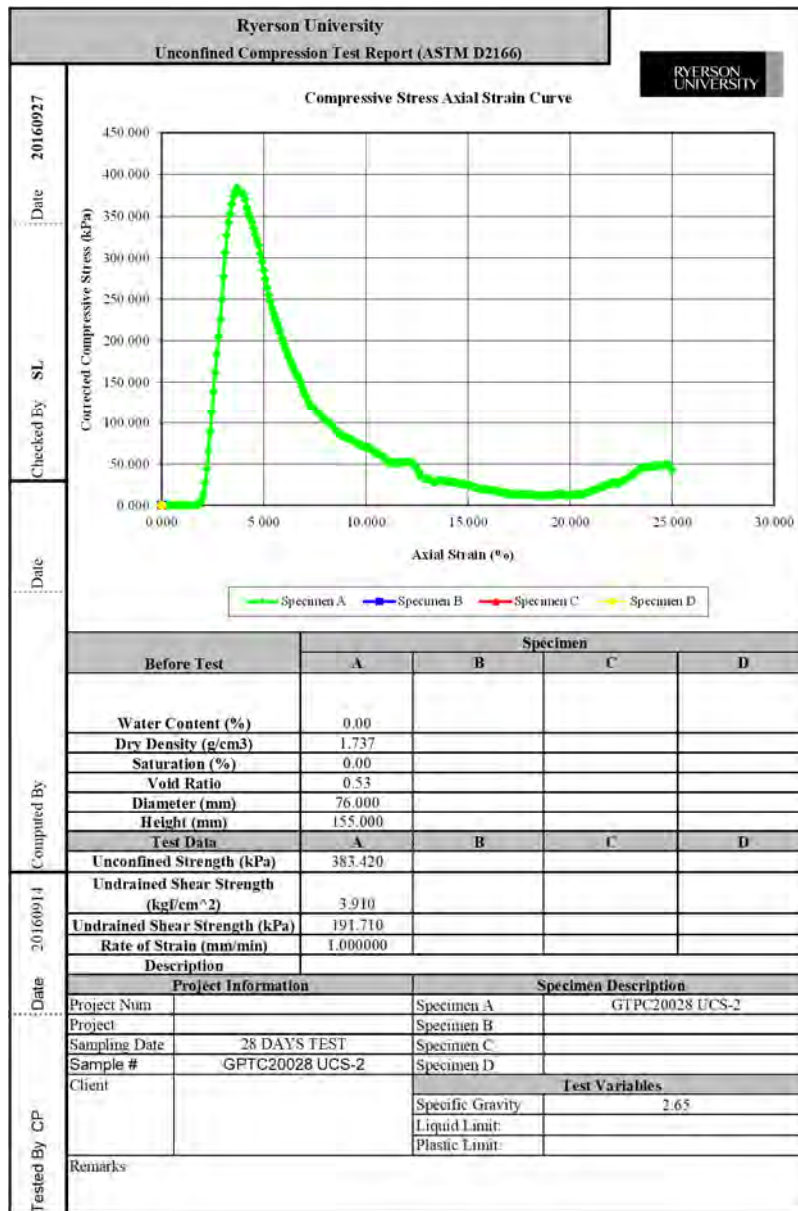
7.2 UCS TEST RESULTS OF CEMENT TREATED SILTY CLAY (200KG/M³)

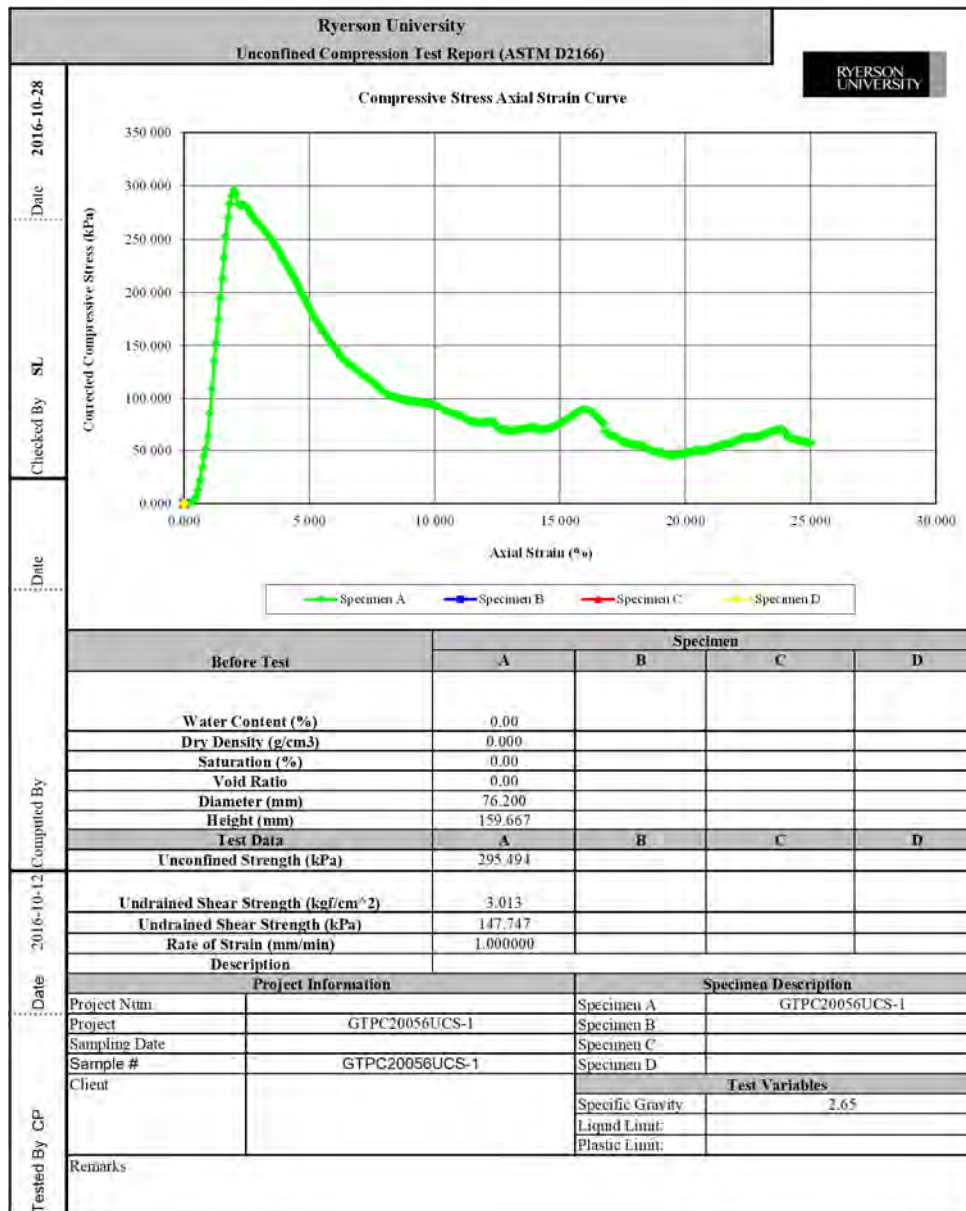


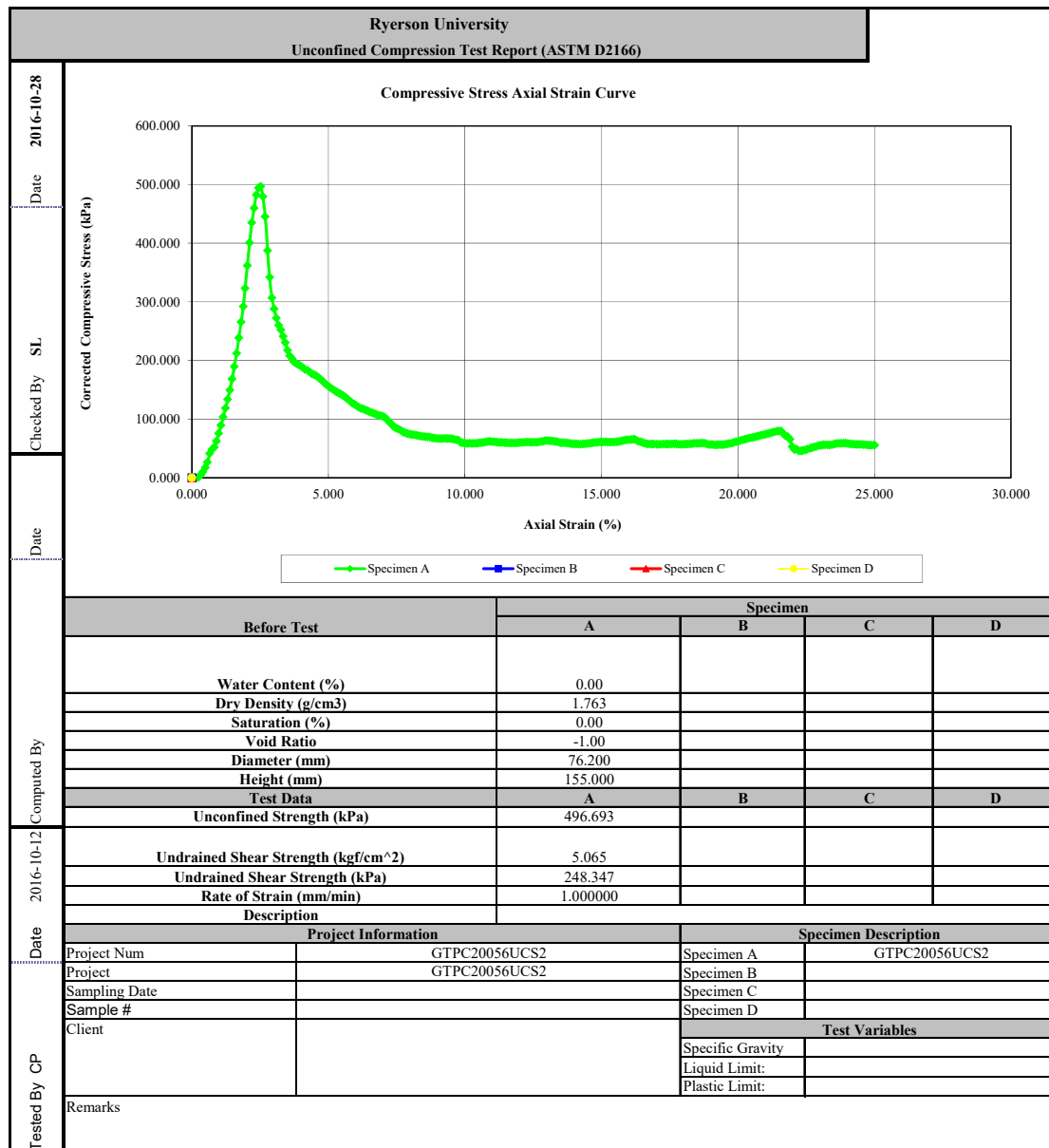




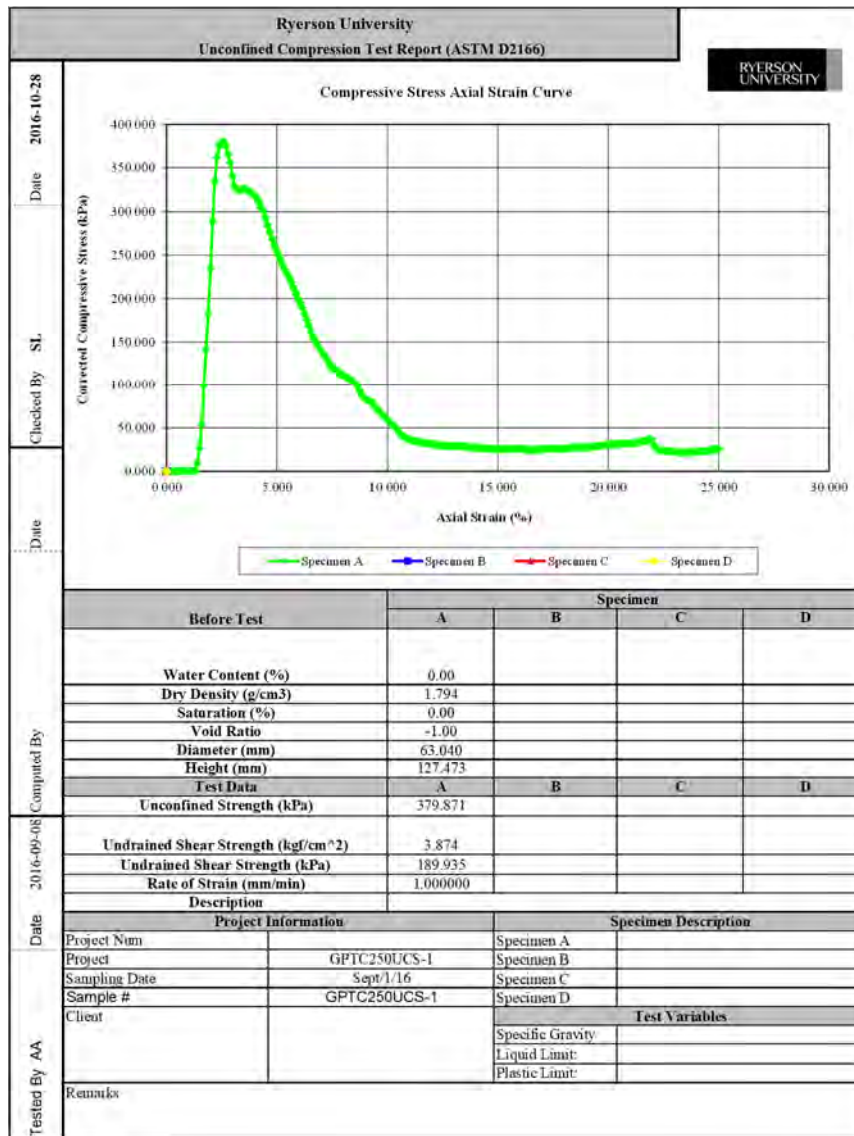


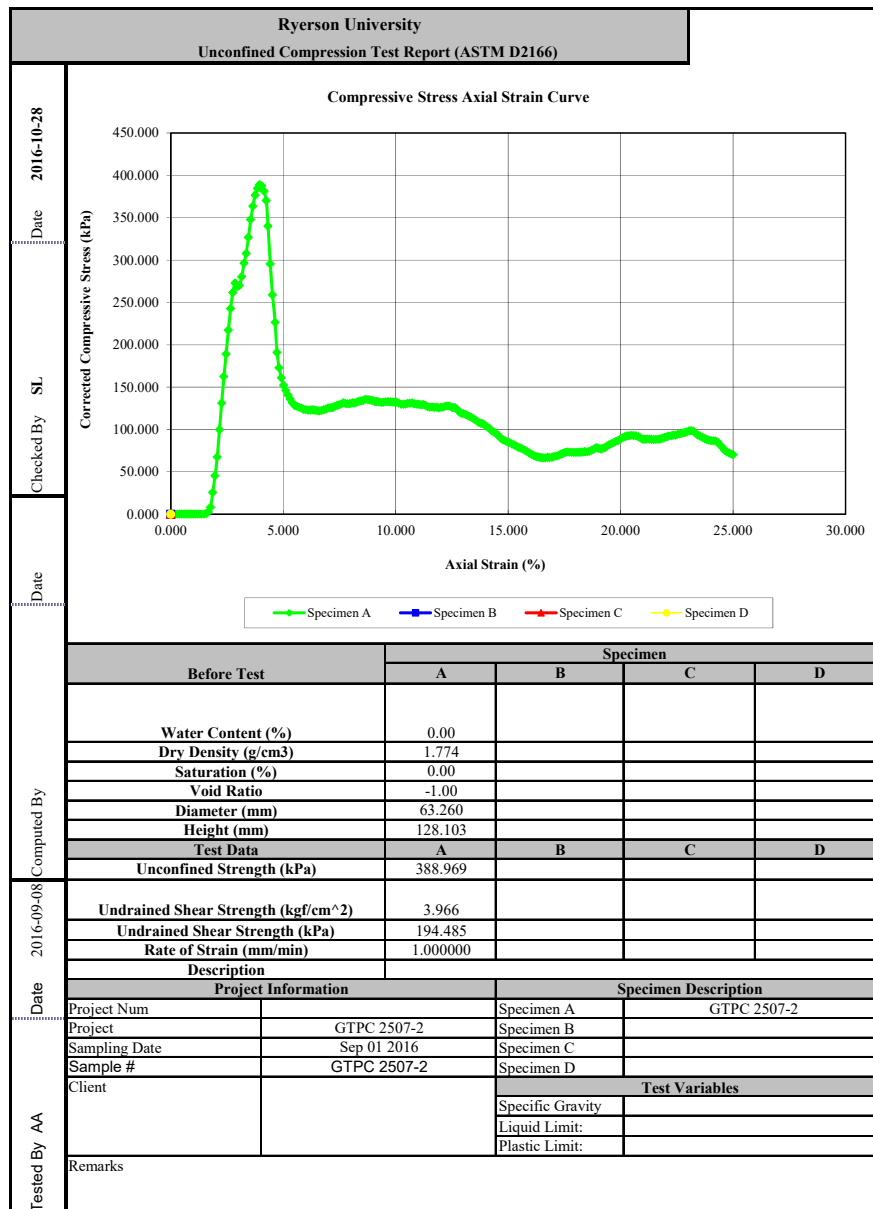


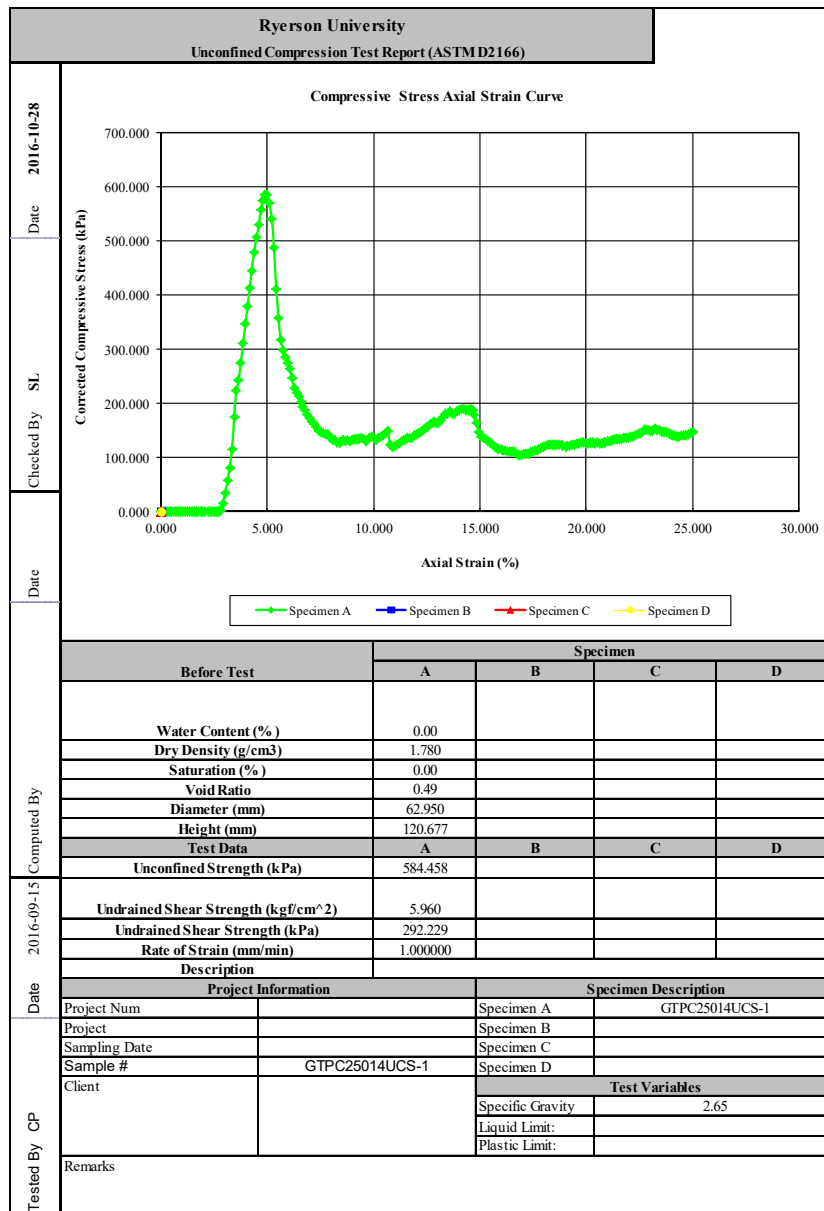


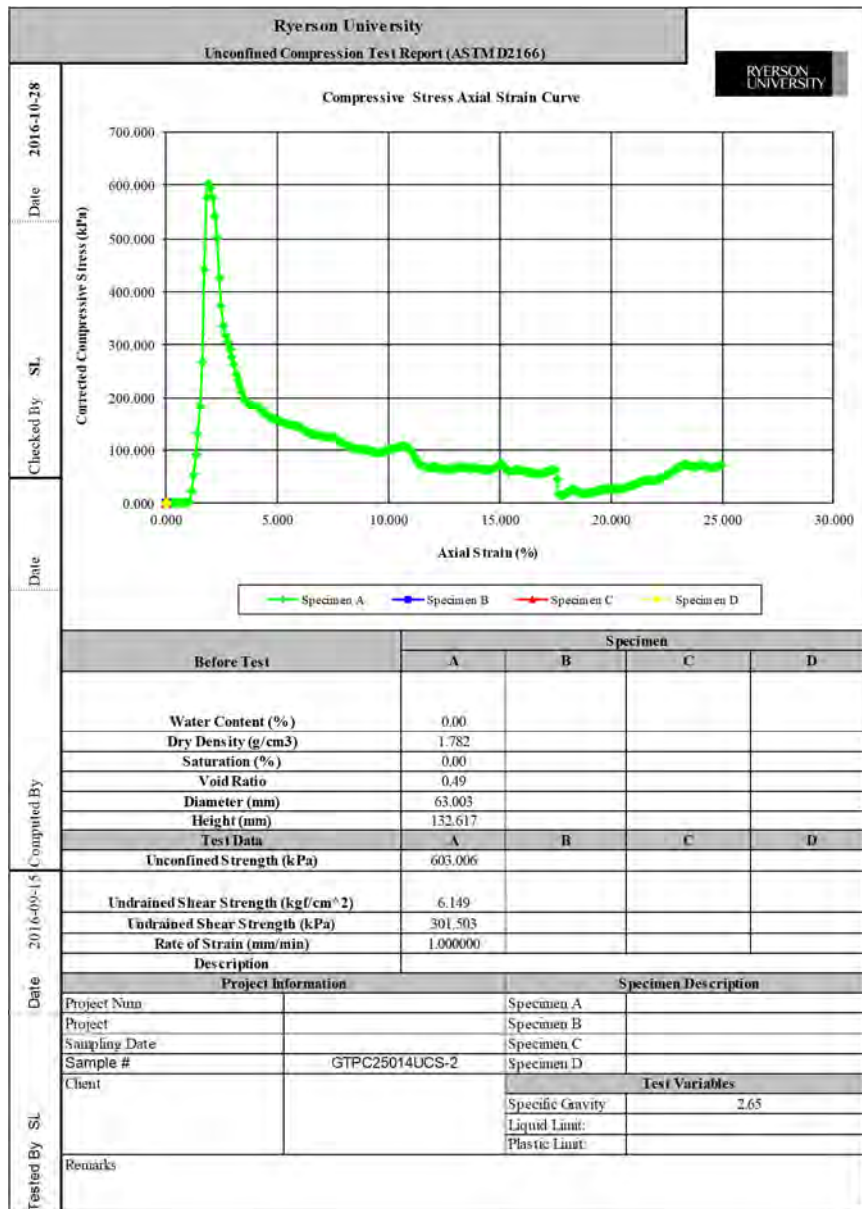


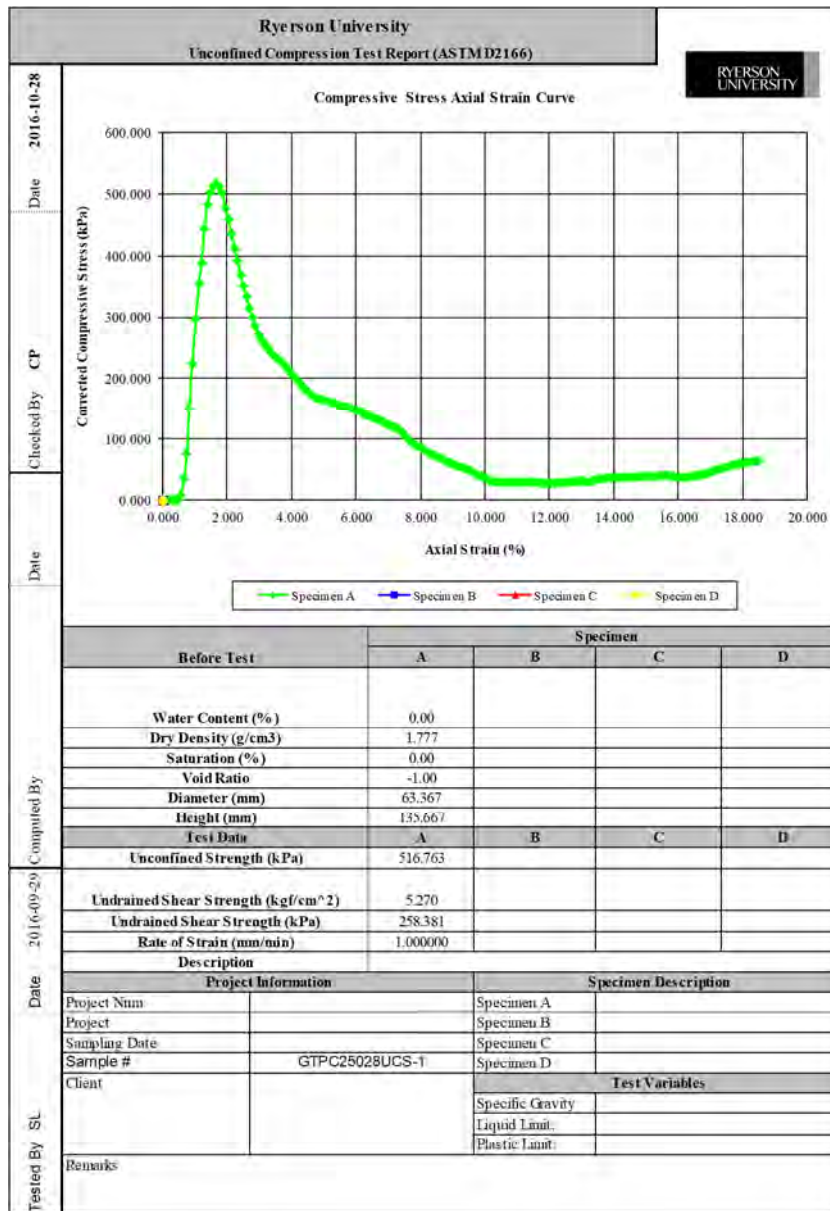
7.3 UCS TEST RESULTS OF CEMENT TREATED SILTY CLAY (250KG/M³)

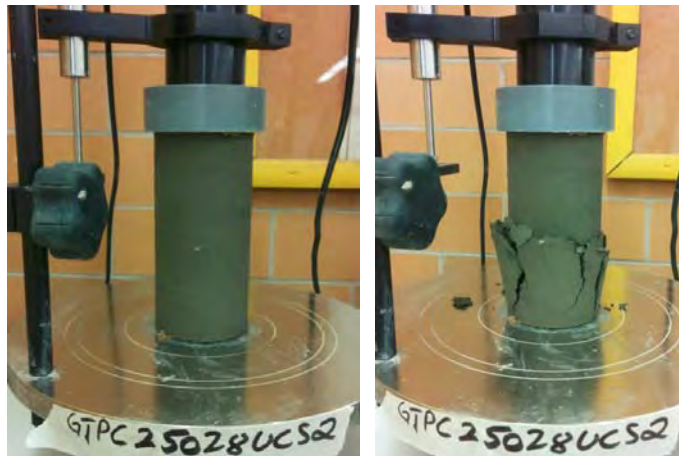
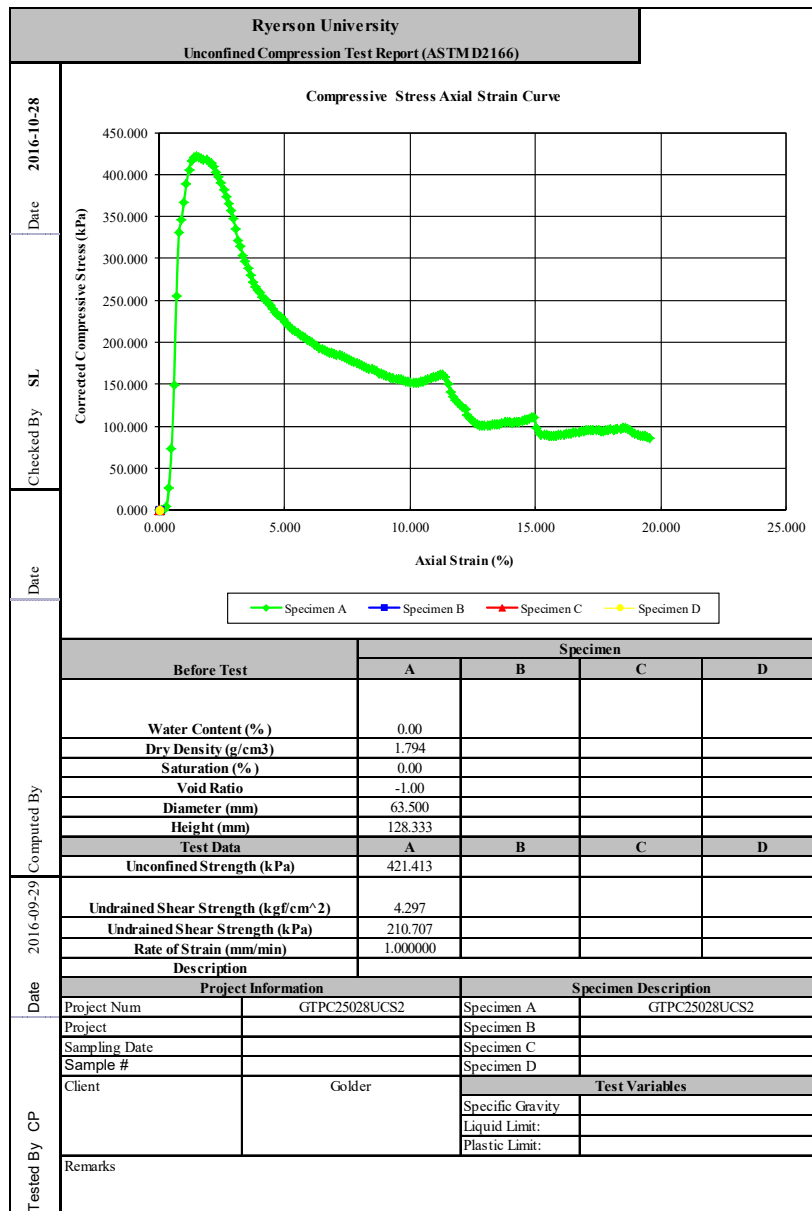




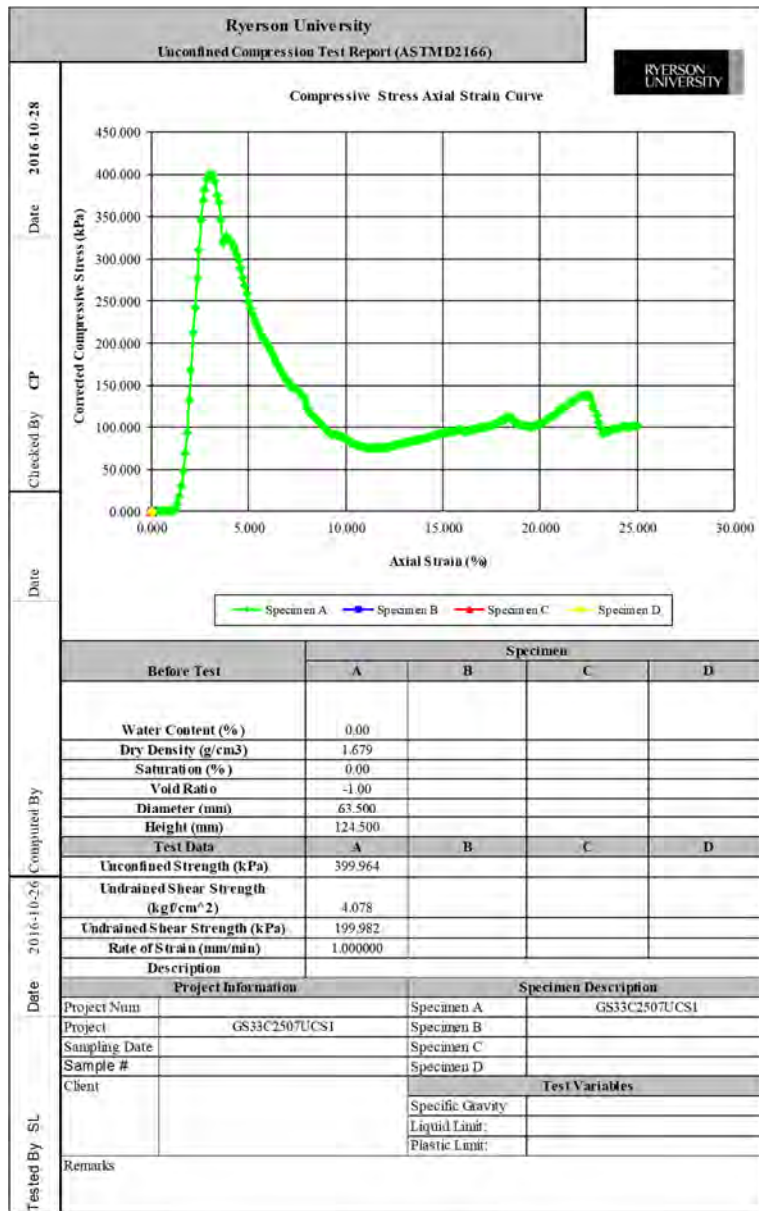


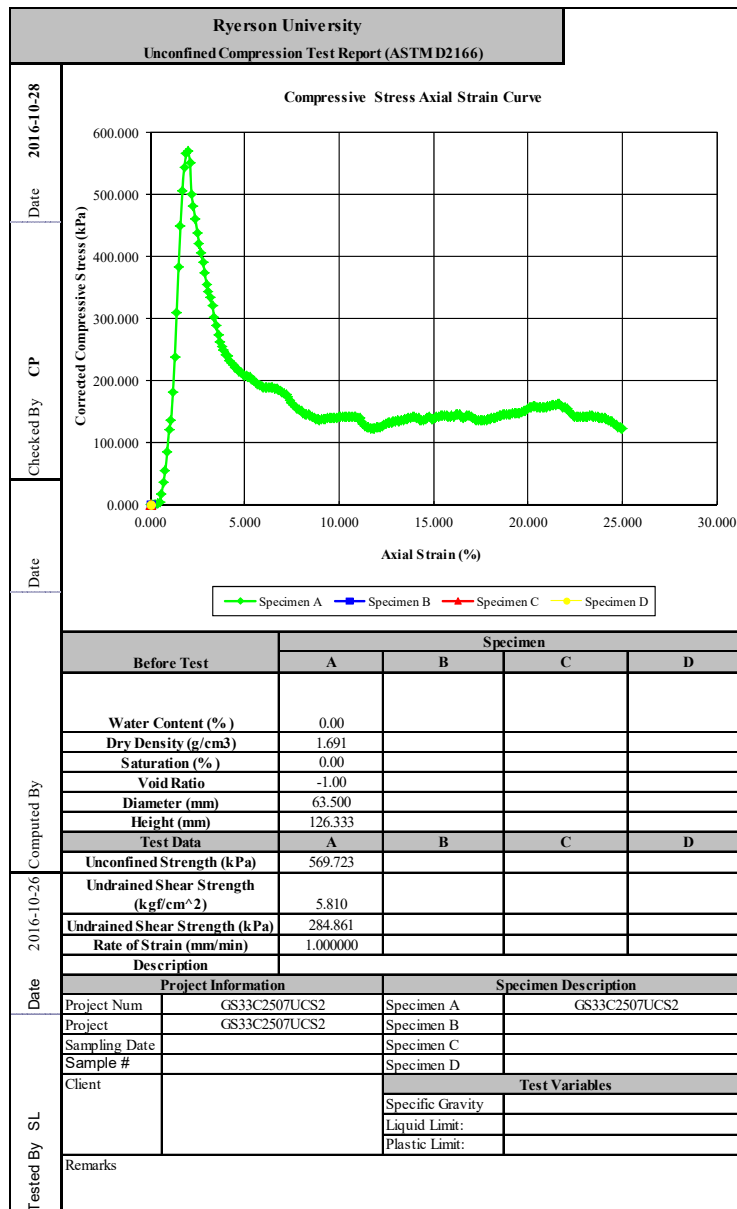




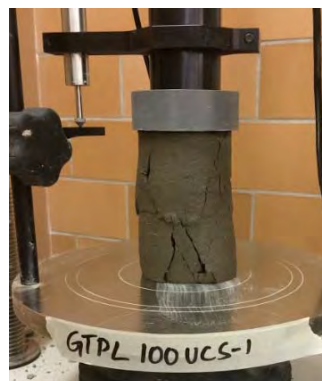
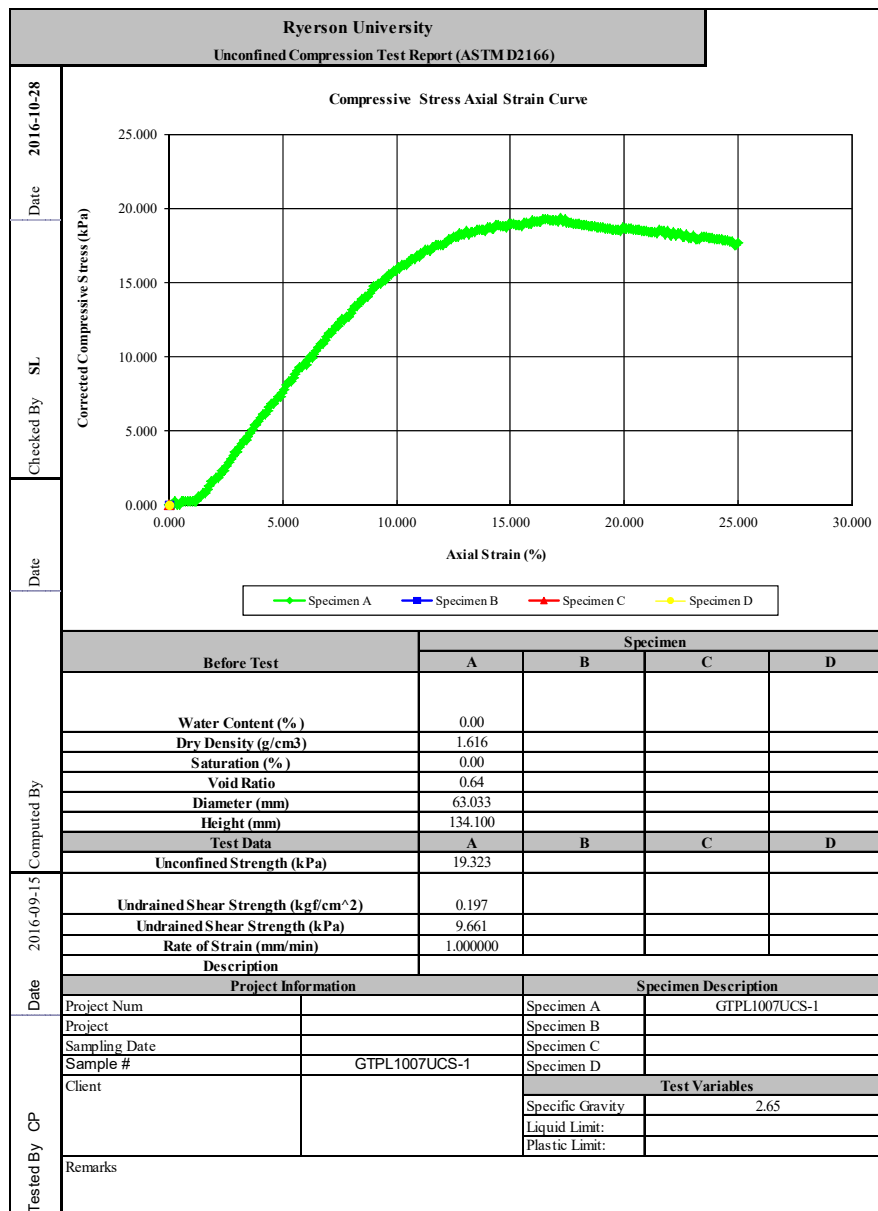


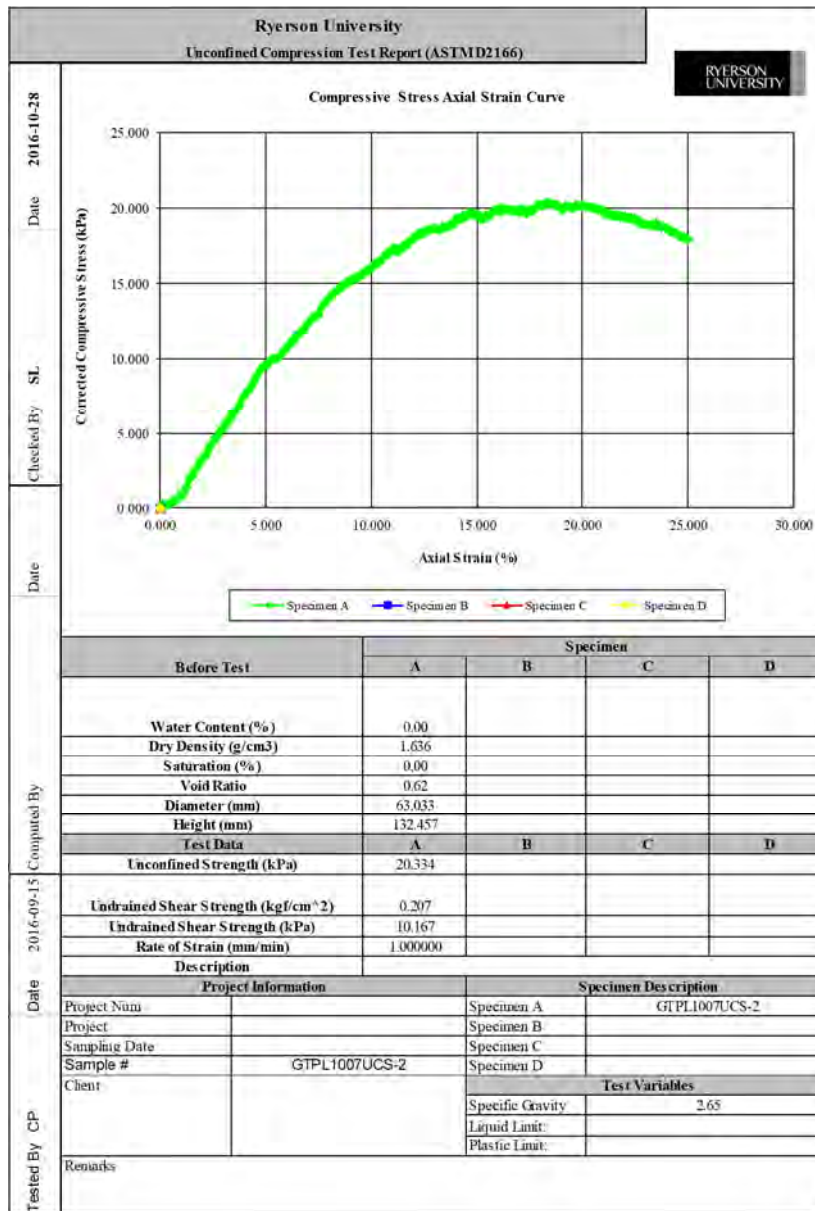
7.4 UCS TEST RESULTS OF CEMENT TREATED ORGANIC CLAY (250KG/M³)

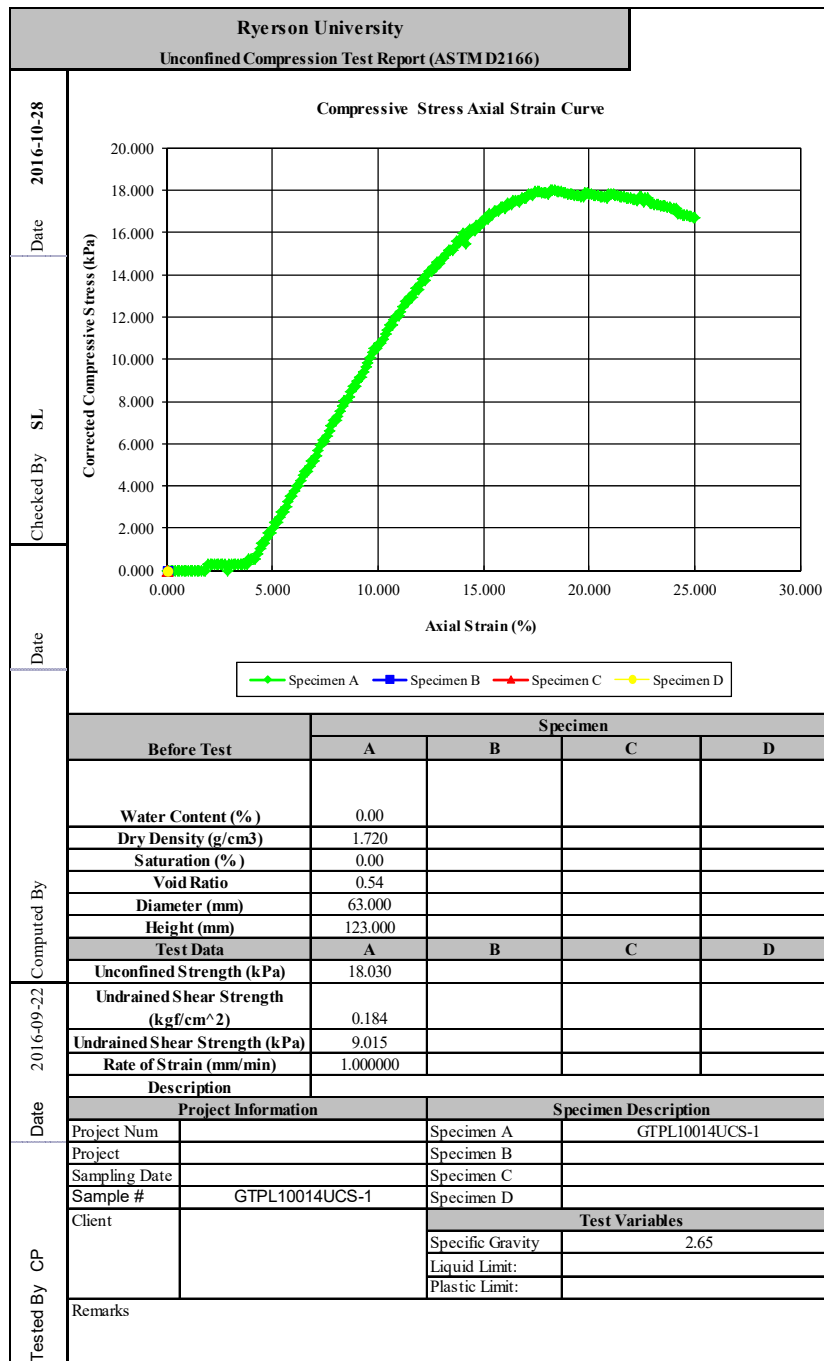




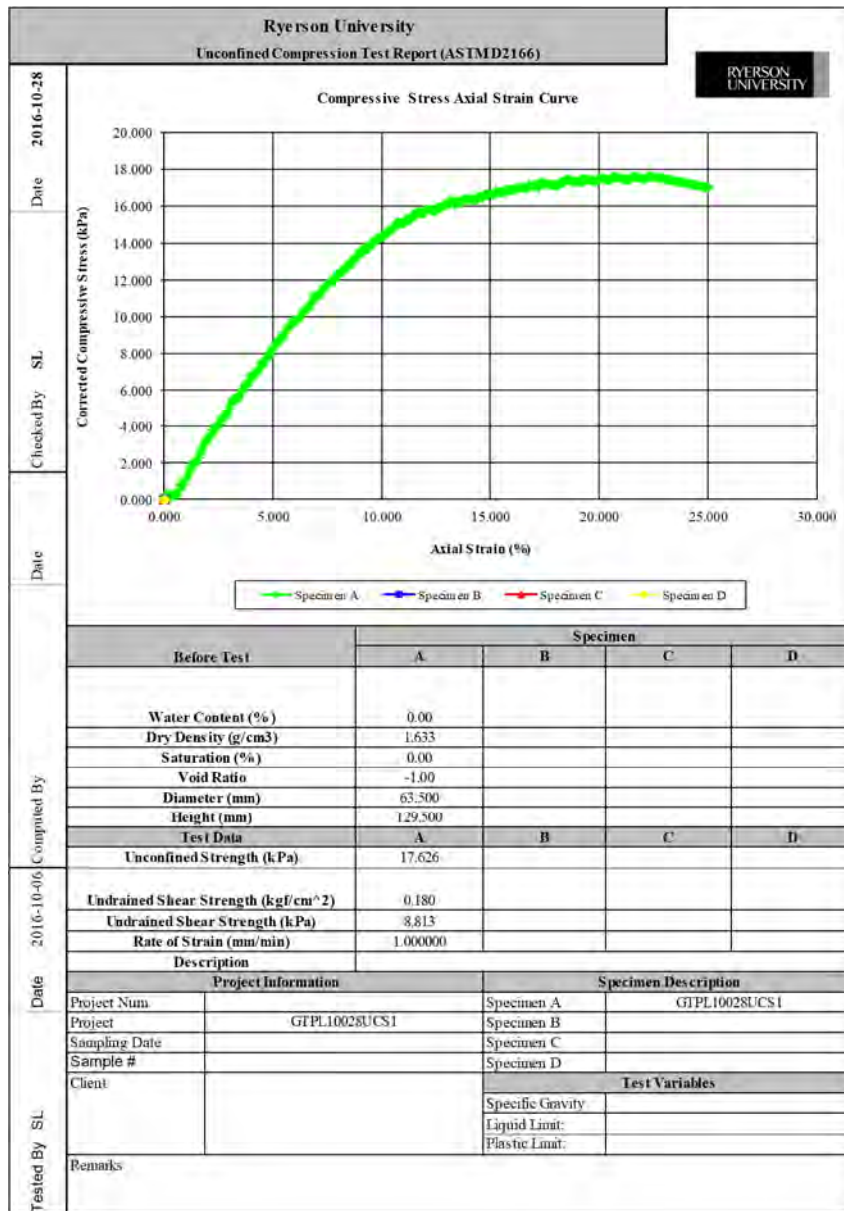
7.5 UCS TEST RESULTS OF LIME TREATED SILTY CLAY (100 KG/M³)

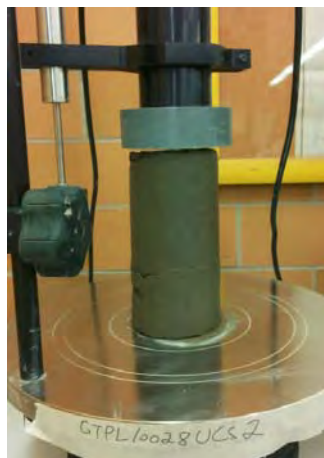
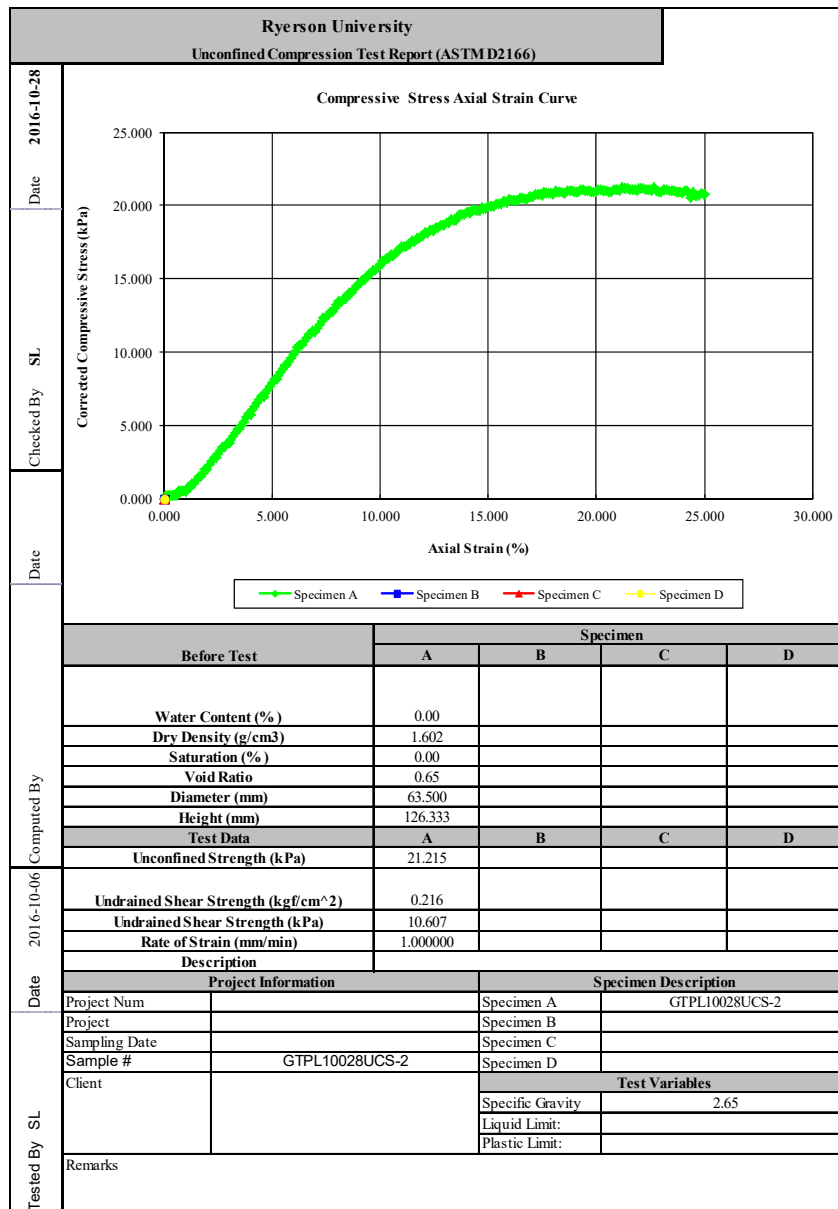




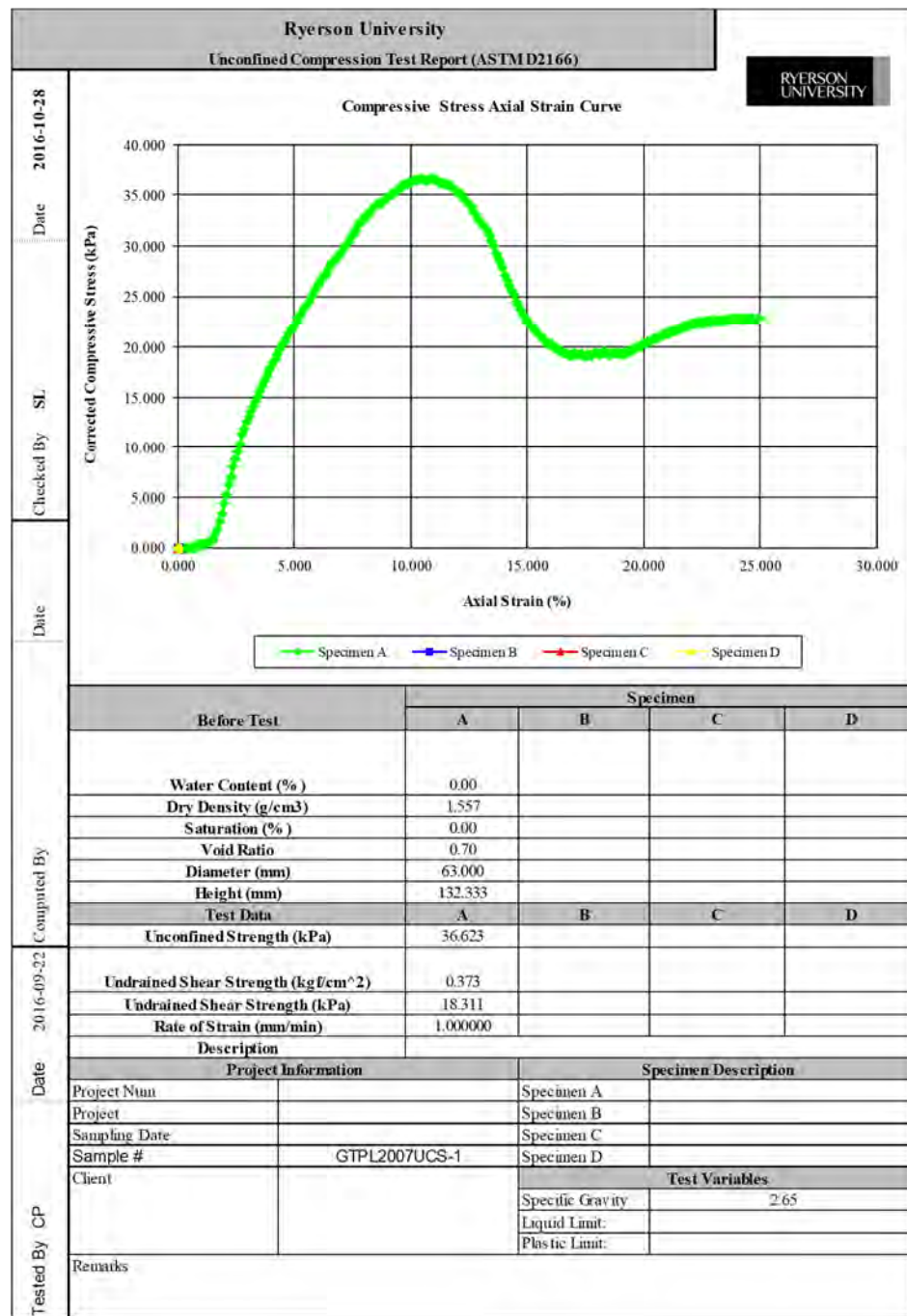


Ryerson University		Unconfined Compression Test Report (ASTMD2166)		RYERSON UNIVERSITY																																																																																																													
Checked By	SL	Date	2016-10-28																																																																																																														
Computed By	Date	<div style="text-align: center;">Compressive Stress Axial Strain Curve</div> <div style="text-align: center; margin-top: 10px;"> —◆— Specimen A —◆— Specimen B —◆— Specimen C —◆— Specimen D </div>																																																																																																															
Tested By	CP	Date	2016-09-22		<table border="1" style="width: 100%; border-collapse: collapse;"> <thead> <tr> <th style="width: 50%;">Before Test</th> <th colspan="4" style="text-align: center;">Specimen</th> </tr> <tr> <th></th> <th style="text-align: center;">A</th> <th style="text-align: center;">B</th> <th style="text-align: center;">C</th> <th style="text-align: center;">D</th> </tr> </thead> <tbody> <tr> <td>Water Content (%)</td> <td style="text-align: center;">0.00</td> <td></td> <td></td> <td></td> </tr> <tr> <td>Dry Density (g/cm³)</td> <td style="text-align: center;">1.662</td> <td></td> <td></td> <td></td> </tr> <tr> <td>Saturation (%)</td> <td style="text-align: center;">0.00</td> <td></td> <td></td> <td></td> </tr> <tr> <td>Void Ratio</td> <td style="text-align: center;">0.59</td> <td></td> <td></td> <td></td> </tr> <tr> <td>Diameter (mm)</td> <td style="text-align: center;">63.000</td> <td></td> <td></td> <td></td> </tr> <tr> <td>Height (mm)</td> <td style="text-align: center;">134.000</td> <td></td> <td></td> <td></td> </tr> <tr> <th style="text-align: center;">Test Data</th> <th style="text-align: center;">A</th> <th style="text-align: center;">B</th> <th style="text-align: center;">C</th> <th style="text-align: center;">D</th> </tr> <tr> <td>Unconfined Strength (kPa)</td> <td style="text-align: center;">16.576</td> <td></td> <td></td> <td></td> </tr> <tr> <td>Undrained Shear Strength (kgf/cm²)</td> <td style="text-align: center;">0.169</td> <td></td> <td></td> <td></td> </tr> <tr> <td>Undrained Shear Strength (kPa)</td> <td style="text-align: center;">8.288</td> <td></td> <td></td> <td></td> </tr> <tr> <td>Rate of Strain (mm/min)</td> <td style="text-align: center;">1.000000</td> <td></td> <td></td> <td></td> </tr> <tr> <th colspan="2" style="text-align: center;">Description</th> <th colspan="2" style="text-align: center;">Project Information</th> <th colspan="2" style="text-align: center;">Specimen Description</th> </tr> <tr> <td colspan="2" style="text-align: center;">Project Num</td> <td colspan="2" style="text-align: center;">Project</td> <td colspan="2" style="text-align: center;">Specimen A</td> </tr> <tr> <td colspan="2" style="text-align: center;">Sampling Date</td> <td colspan="2" style="text-align: center;">Sample #</td> <td colspan="2" style="text-align: center;">Specimen B</td> </tr> <tr> <td colspan="2" style="text-align: center;">Client</td> <td colspan="2" style="text-align: center;">GTPL10014UCS-2</td> <td colspan="2" style="text-align: center;">Specimen C</td> </tr> <tr> <td colspan="2" rowspan="4" style="text-align: center; vertical-align: middle;">Remarks</td> <td colspan="2" rowspan="4" style="text-align: center; vertical-align: middle;"></td> <td colspan="2" style="text-align: center;">Specimen D</td> </tr> <tr> <th colspan="2" style="text-align: center;">Test Variables</th> </tr> <tr> <td colspan="2" style="text-align: center;">Specific Gravity</td> </tr> <tr> <td colspan="2" style="text-align: center;">Liquid Limit:</td> </tr> <tr> <td colspan="2" style="text-align: center;">Plastic Limit:</td> <td colspan="2" style="text-align: center;">2.65</td> <td colspan="2"></td> </tr> </tbody> </table>		Before Test	Specimen					A	B	C	D	Water Content (%)	0.00				Dry Density (g/cm ³)	1.662				Saturation (%)	0.00				Void Ratio	0.59				Diameter (mm)	63.000				Height (mm)	134.000				Test Data	A	B	C	D	Unconfined Strength (kPa)	16.576				Undrained Shear Strength (kgf/cm ²)	0.169				Undrained Shear Strength (kPa)	8.288				Rate of Strain (mm/min)	1.000000				Description		Project Information		Specimen Description		Project Num		Project		Specimen A		Sampling Date		Sample #		Specimen B		Client		GTPL10014UCS-2		Specimen C		Remarks				Specimen D		Test Variables		Specific Gravity		Liquid Limit:		Plastic Limit:		2.65			
		Before Test	Specimen																																																																																																														
			A	B			C	D																																																																																																									
		Water Content (%)	0.00																																																																																																														
		Dry Density (g/cm ³)	1.662																																																																																																														
		Saturation (%)	0.00																																																																																																														
		Void Ratio	0.59																																																																																																														
		Diameter (mm)	63.000																																																																																																														
		Height (mm)	134.000																																																																																																														
		Test Data	A	B			C	D																																																																																																									
Unconfined Strength (kPa)	16.576																																																																																																																
Undrained Shear Strength (kgf/cm ²)	0.169																																																																																																																
Undrained Shear Strength (kPa)	8.288																																																																																																																
Rate of Strain (mm/min)	1.000000																																																																																																																
Description		Project Information		Specimen Description																																																																																																													
Project Num		Project		Specimen A																																																																																																													
Sampling Date		Sample #		Specimen B																																																																																																													
Client		GTPL10014UCS-2		Specimen C																																																																																																													
Remarks				Specimen D																																																																																																													
				Test Variables																																																																																																													
				Specific Gravity																																																																																																													
				Liquid Limit:																																																																																																													
Plastic Limit:		2.65																																																																																																															

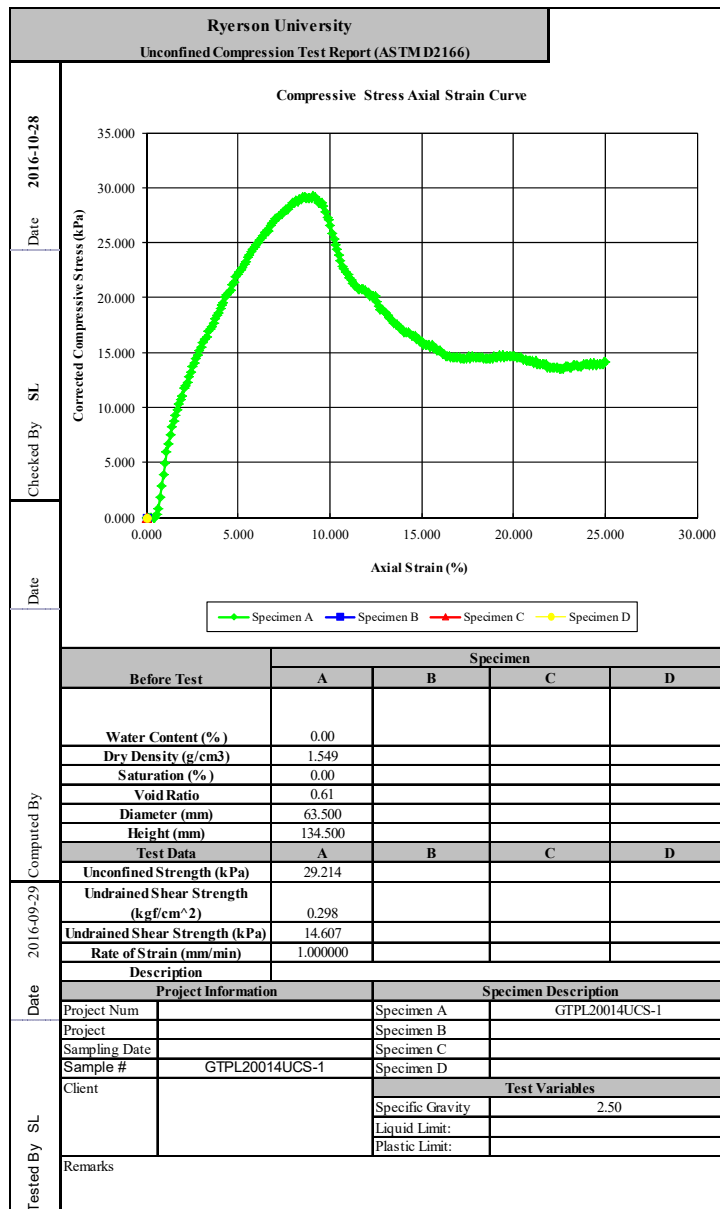


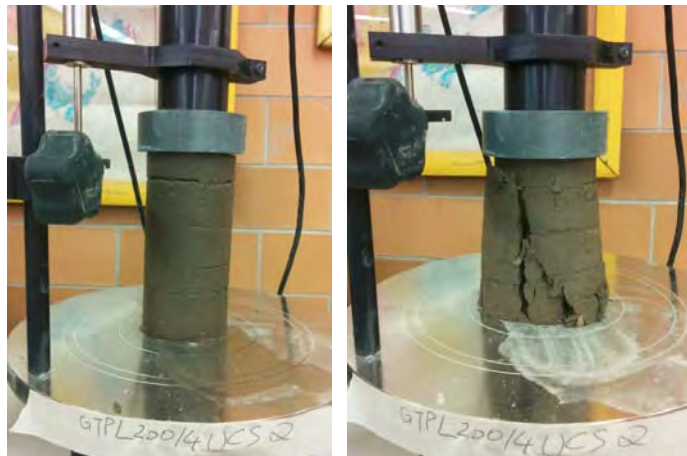
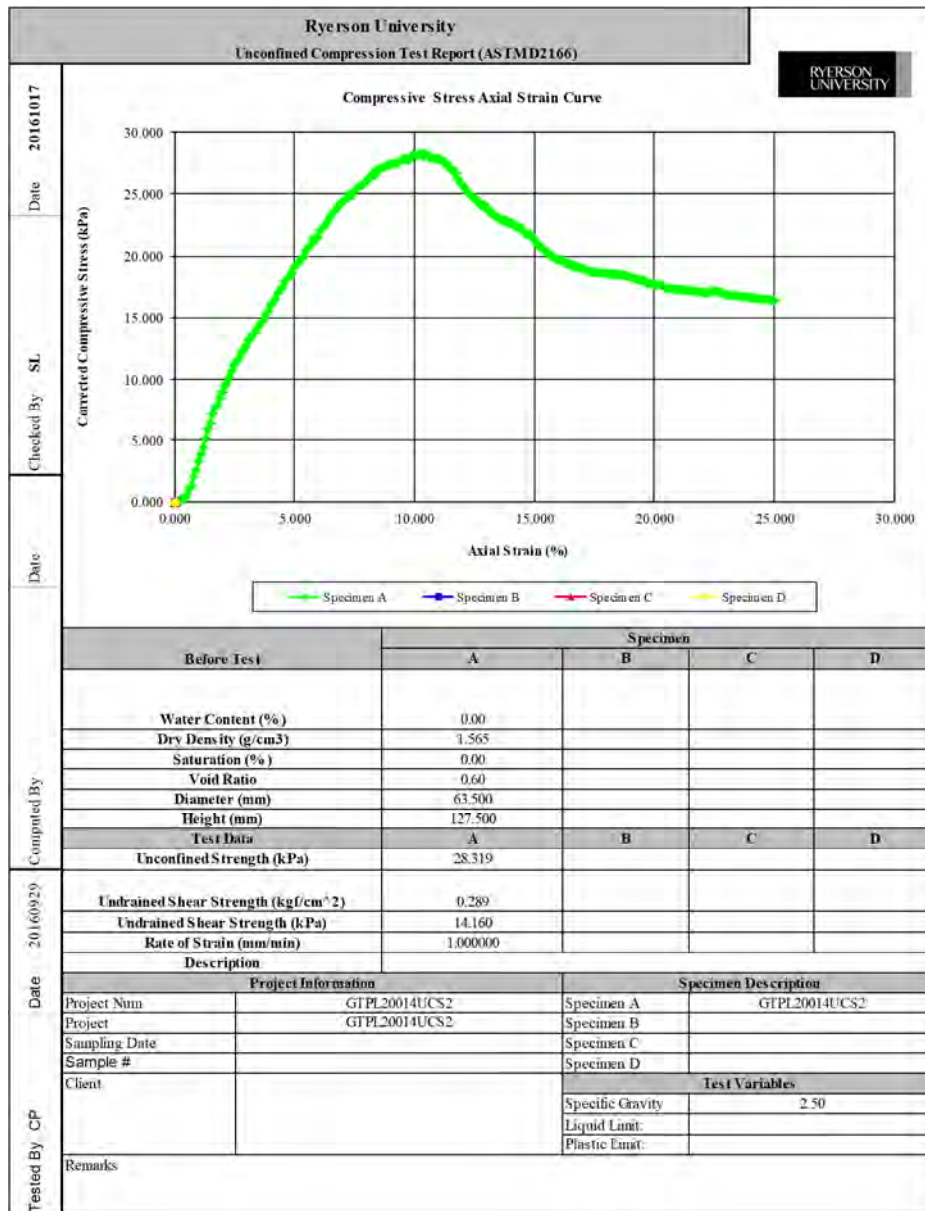


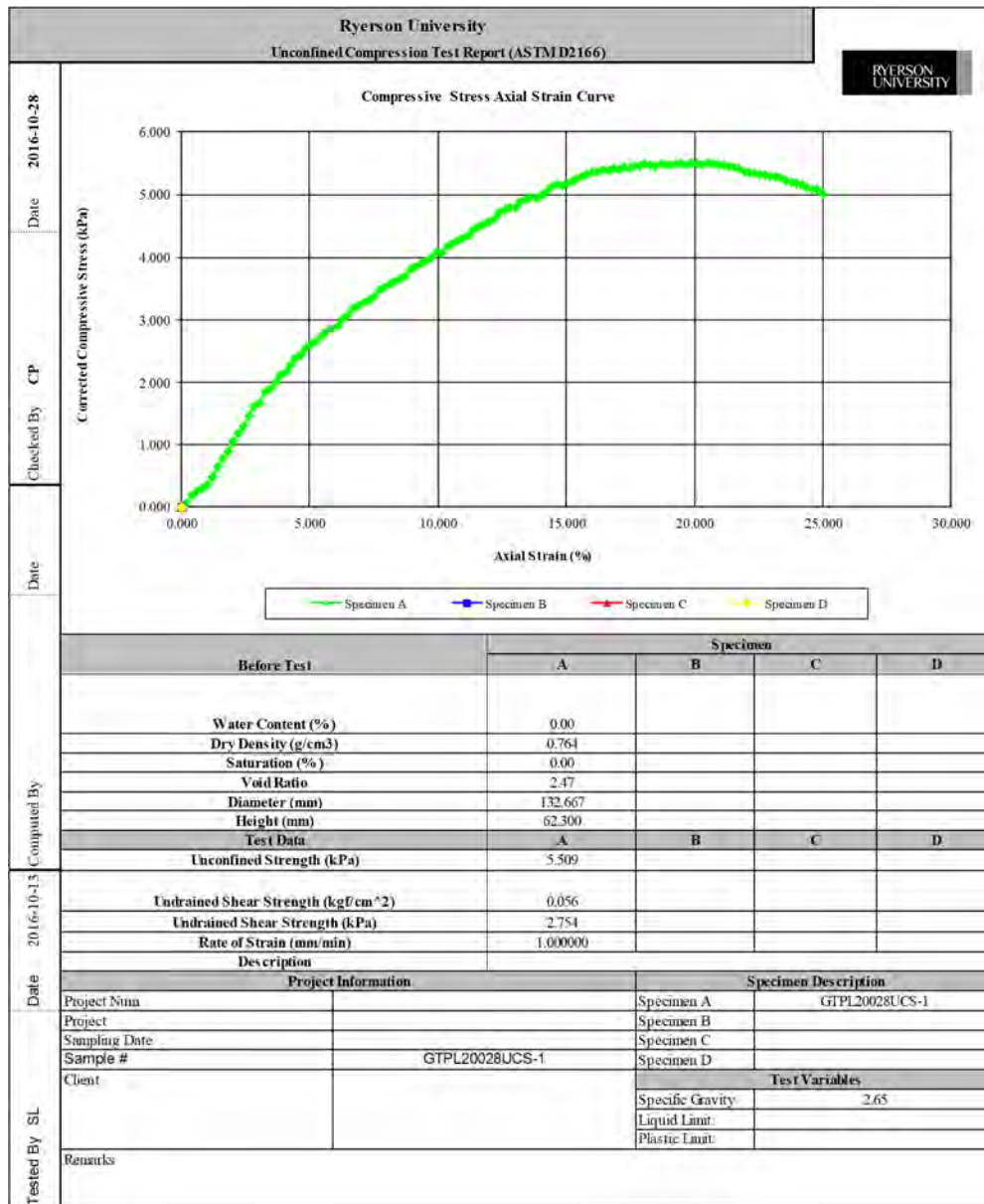
7.6 UCS TEST RESULTS OF LIME TREATED SILTY CLAY (200 KG/M³)



Ryerson University																																																																																																														
Unconfined Compression Test Report (ASTM D2166)																																																																																																														
2016-10-28	<p>Compressive Stress Axial Strain Curve</p> <p>Corrected Compressive Stress (kPa)</p> <p>Axial Strain (%)</p> <p>Specimen A Specimen B Specimen C Specimen D</p>																																																																																																													
SL	<table border="1"> <thead> <tr> <th rowspan="2">Before Test</th> <th colspan="4">Specimen</th> </tr> <tr> <th>A</th> <th>B</th> <th>C</th> <th>D</th> </tr> </thead> <tbody> <tr> <td>Water Content (%)</td> <td>0.00</td> <td></td> <td></td> <td></td> </tr> <tr> <td>Dry Density (g/cm³)</td> <td>1.580</td> <td></td> <td></td> <td></td> </tr> <tr> <td>Saturation (%)</td> <td>0.00</td> <td></td> <td></td> <td></td> </tr> <tr> <td>Void Ratio</td> <td>0.68</td> <td></td> <td></td> <td></td> </tr> <tr> <td>Diameter (mm)</td> <td>63.000</td> <td></td> <td></td> <td></td> </tr> <tr> <td>Height (mm)</td> <td>134.667</td> <td></td> <td></td> <td></td> </tr> <tr> <td>Test Data</td> <td>A</td> <td>B</td> <td>C</td> <td>D</td> </tr> <tr> <td>Unconfined Strength (kPa)</td> <td>34.540</td> <td></td> <td></td> <td></td> </tr> <tr> <td>Undrained Shear Strength (kgf/cm²)</td> <td>0.352</td> <td></td> <td></td> <td></td> </tr> <tr> <td>Undrained Shear Strength (kPa)</td> <td>17.270</td> <td></td> <td></td> <td></td> </tr> <tr> <td>Rate of Strain (mm/min)</td> <td>1.000000</td> <td></td> <td></td> <td></td> </tr> <tr> <td>Description</td> <td colspan="4"></td> </tr> <tr> <td colspan="2">Project Information</td> <td colspan="2">Specimen Description</td> </tr> <tr> <td>Project Num</td> <td></td> <td>Specimen A</td> <td></td> </tr> <tr> <td>Project</td> <td></td> <td>Specimen B</td> <td></td> </tr> <tr> <td>Sampling Date</td> <td></td> <td>Specimen C</td> <td></td> </tr> <tr> <td>Sample #</td> <td>GTPL2007UCS-2</td> <td>Specimen D</td> <td></td> </tr> <tr> <td>Client</td> <td></td> <td colspan="2">Test Variables</td> </tr> <tr> <td></td> <td></td> <td>Specific Gravity</td> <td>2.65</td> </tr> <tr> <td></td> <td></td> <td>Liquid Limit:</td> <td></td> </tr> <tr> <td></td> <td></td> <td>Plastic Limit:</td> <td></td> </tr> <tr> <td>Remarks</td> <td colspan="3"></td> </tr> </tbody> </table>	Before Test	Specimen				A	B	C	D	Water Content (%)	0.00				Dry Density (g/cm ³)	1.580				Saturation (%)	0.00				Void Ratio	0.68				Diameter (mm)	63.000				Height (mm)	134.667				Test Data	A	B	C	D	Unconfined Strength (kPa)	34.540				Undrained Shear Strength (kgf/cm ²)	0.352				Undrained Shear Strength (kPa)	17.270				Rate of Strain (mm/min)	1.000000				Description					Project Information		Specimen Description		Project Num		Specimen A		Project		Specimen B		Sampling Date		Specimen C		Sample #	GTPL2007UCS-2	Specimen D		Client		Test Variables				Specific Gravity	2.65			Liquid Limit:				Plastic Limit:		Remarks			
Before Test	Specimen																																																																																																													
	A	B	C	D																																																																																																										
Water Content (%)	0.00																																																																																																													
Dry Density (g/cm ³)	1.580																																																																																																													
Saturation (%)	0.00																																																																																																													
Void Ratio	0.68																																																																																																													
Diameter (mm)	63.000																																																																																																													
Height (mm)	134.667																																																																																																													
Test Data	A	B	C	D																																																																																																										
Unconfined Strength (kPa)	34.540																																																																																																													
Undrained Shear Strength (kgf/cm ²)	0.352																																																																																																													
Undrained Shear Strength (kPa)	17.270																																																																																																													
Rate of Strain (mm/min)	1.000000																																																																																																													
Description																																																																																																														
Project Information		Specimen Description																																																																																																												
Project Num		Specimen A																																																																																																												
Project		Specimen B																																																																																																												
Sampling Date		Specimen C																																																																																																												
Sample #	GTPL2007UCS-2	Specimen D																																																																																																												
Client		Test Variables																																																																																																												
		Specific Gravity	2.65																																																																																																											
		Liquid Limit:																																																																																																												
		Plastic Limit:																																																																																																												
Remarks																																																																																																														
2016-09-22	<table border="1"> <thead> <tr> <th colspan="2">Project Information</th> <th colspan="2">Specimen Description</th> </tr> </thead> <tbody> <tr> <td>Project Num</td> <td></td> <td>Specimen A</td> <td></td> </tr> <tr> <td>Project</td> <td></td> <td>Specimen B</td> <td></td> </tr> <tr> <td>Sampling Date</td> <td></td> <td>Specimen C</td> <td></td> </tr> <tr> <td>Sample #</td> <td>GTPL2007UCS-2</td> <td>Specimen D</td> <td></td> </tr> <tr> <td>Client</td> <td></td> <td colspan="2">Test Variables</td> </tr> <tr> <td></td> <td></td> <td>Specific Gravity</td> <td>2.65</td> </tr> <tr> <td></td> <td></td> <td>Liquid Limit:</td> <td></td> </tr> <tr> <td></td> <td></td> <td>Plastic Limit:</td> <td></td> </tr> <tr> <td>Remarks</td> <td colspan="3"></td> </tr> </tbody> </table>	Project Information		Specimen Description		Project Num		Specimen A		Project		Specimen B		Sampling Date		Specimen C		Sample #	GTPL2007UCS-2	Specimen D		Client		Test Variables				Specific Gravity	2.65			Liquid Limit:				Plastic Limit:		Remarks																																																																								
Project Information		Specimen Description																																																																																																												
Project Num		Specimen A																																																																																																												
Project		Specimen B																																																																																																												
Sampling Date		Specimen C																																																																																																												
Sample #	GTPL2007UCS-2	Specimen D																																																																																																												
Client		Test Variables																																																																																																												
		Specific Gravity	2.65																																																																																																											
		Liquid Limit:																																																																																																												
		Plastic Limit:																																																																																																												
Remarks																																																																																																														
CP	<table border="1"> <thead> <tr> <th colspan="2">Project Information</th> <th colspan="2">Specimen Description</th> </tr> </thead> <tbody> <tr> <td>Project Num</td> <td></td> <td>Specimen A</td> <td></td> </tr> <tr> <td>Project</td> <td></td> <td>Specimen B</td> <td></td> </tr> <tr> <td>Sampling Date</td> <td></td> <td>Specimen C</td> <td></td> </tr> <tr> <td>Sample #</td> <td>GTPL2007UCS-2</td> <td>Specimen D</td> <td></td> </tr> <tr> <td>Client</td> <td></td> <td colspan="2">Test Variables</td> </tr> <tr> <td></td> <td></td> <td>Specific Gravity</td> <td>2.65</td> </tr> <tr> <td></td> <td></td> <td>Liquid Limit:</td> <td></td> </tr> <tr> <td></td> <td></td> <td>Plastic Limit:</td> <td></td> </tr> <tr> <td>Remarks</td> <td colspan="3"></td> </tr> </tbody> </table>	Project Information		Specimen Description		Project Num		Specimen A		Project		Specimen B		Sampling Date		Specimen C		Sample #	GTPL2007UCS-2	Specimen D		Client		Test Variables				Specific Gravity	2.65			Liquid Limit:				Plastic Limit:		Remarks																																																																								
Project Information		Specimen Description																																																																																																												
Project Num		Specimen A																																																																																																												
Project		Specimen B																																																																																																												
Sampling Date		Specimen C																																																																																																												
Sample #	GTPL2007UCS-2	Specimen D																																																																																																												
Client		Test Variables																																																																																																												
		Specific Gravity	2.65																																																																																																											
		Liquid Limit:																																																																																																												
		Plastic Limit:																																																																																																												
Remarks																																																																																																														

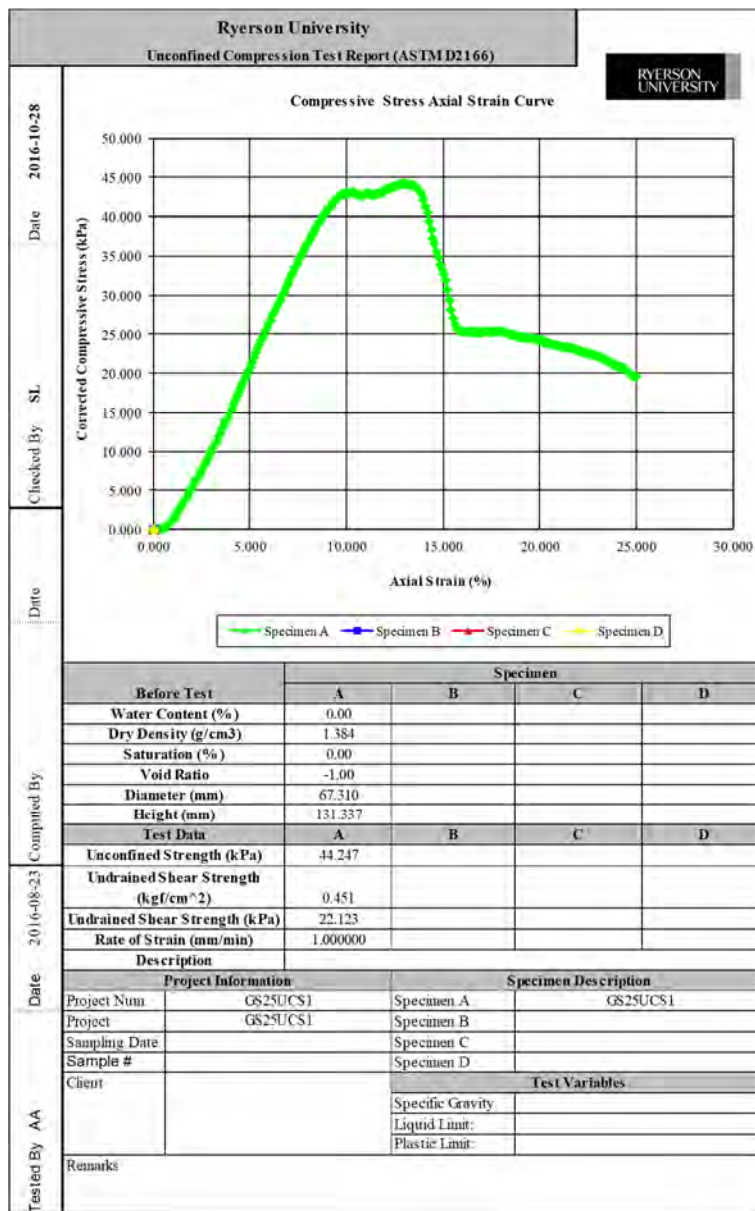


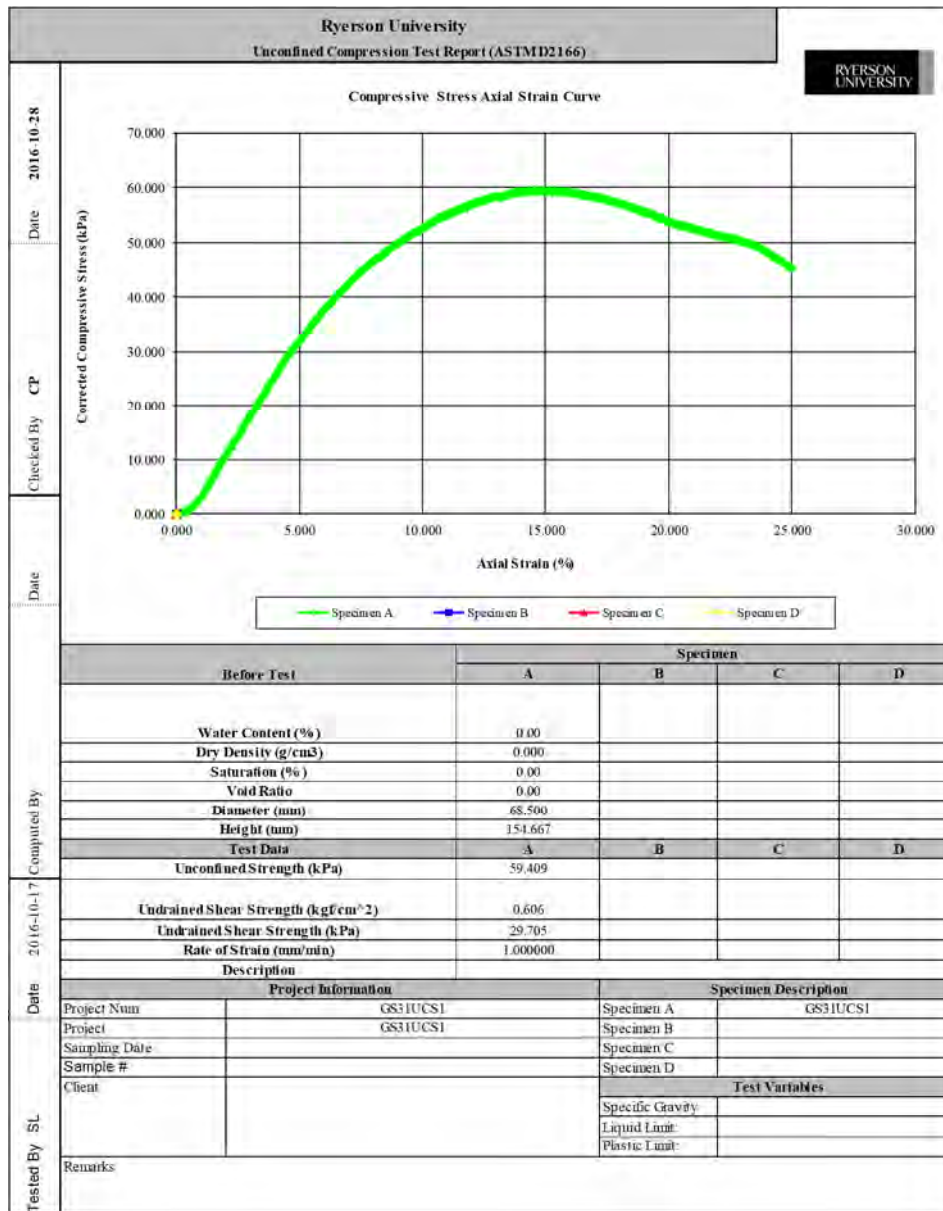


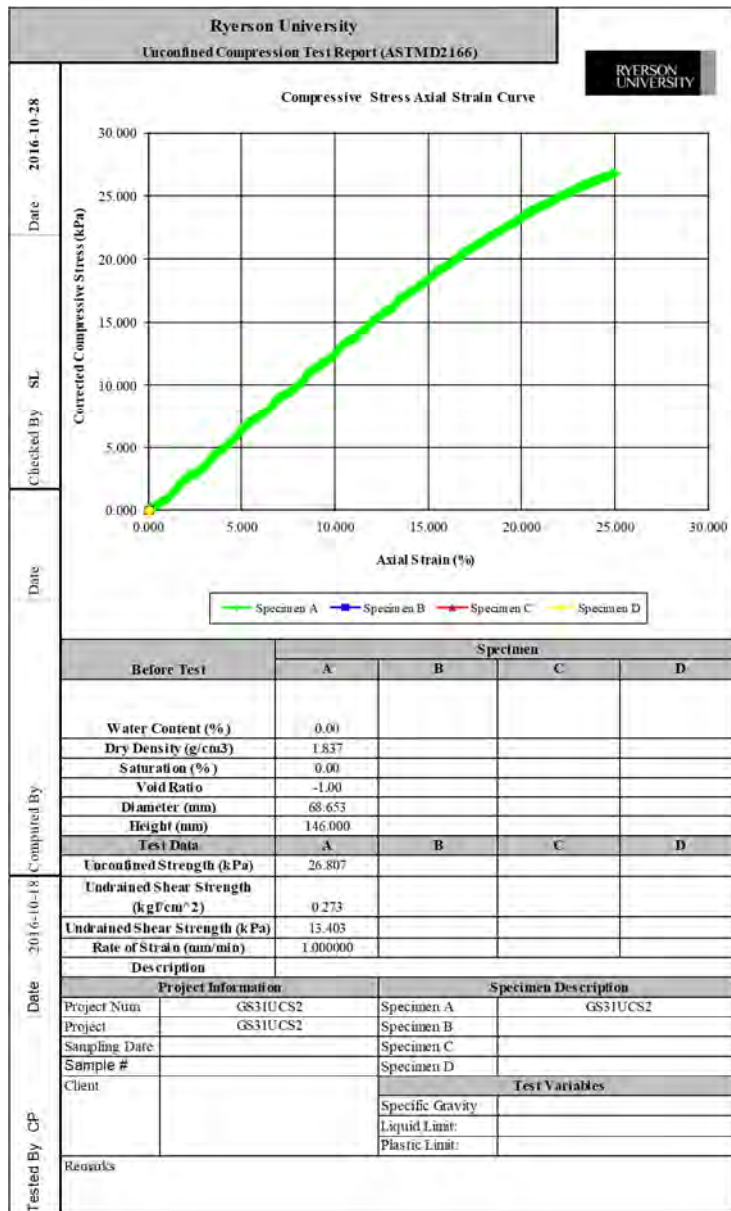


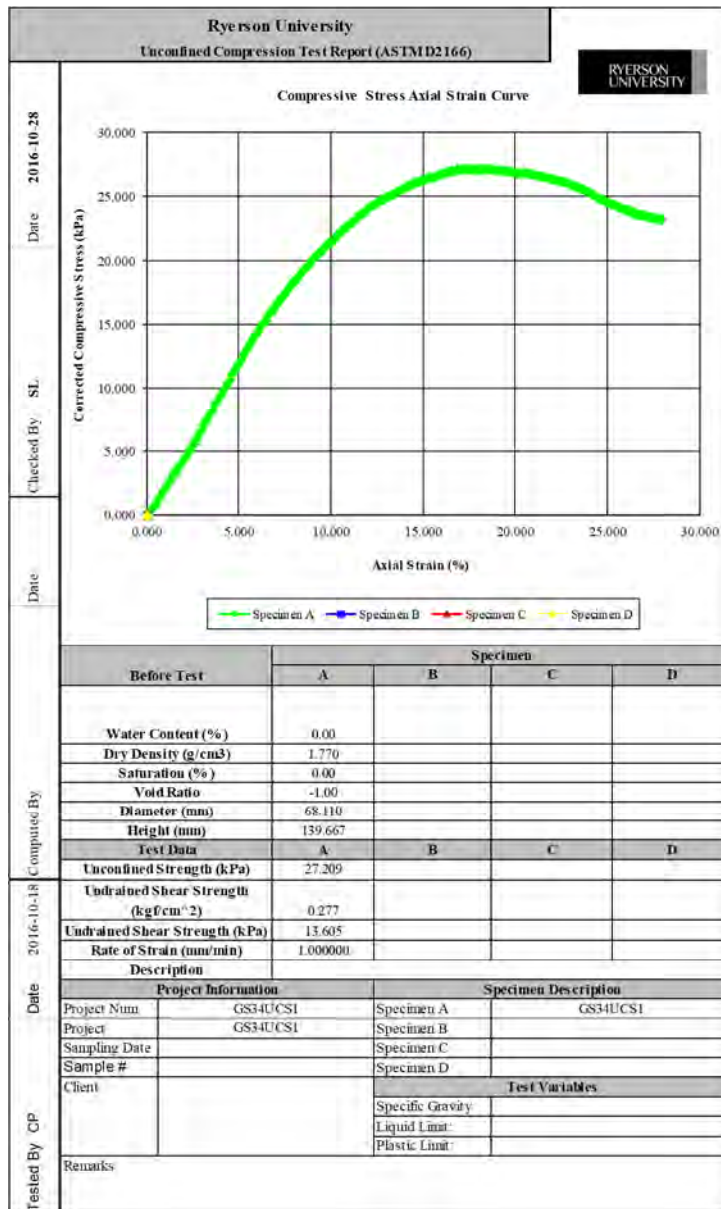
Ryerson University																																																																																																															
Unconfined Compression Test Report (ASTM D2166)																																																																																																															
<div style="text-align: center;">2016-10-28</div> <div style="text-align: center;">Date</div>	<div style="text-align: center;">Compressive Stress Axial Strain Curve</div> <div style="text-align: center;"> —●— Specimen A —●— Specimen B —●— Specimen C —●— Specimen D </div>																																																																																																														
<div style="text-align: center;">SL</div> <div style="text-align: center;">Checked By</div>																																																																																																															
<div style="text-align: center;">Date</div>																																																																																																															
<div style="text-align: center;">2016-10-13</div> <div style="text-align: center;">Date</div>	<table border="1" style="width: 100%; border-collapse: collapse;"> <thead> <tr> <th style="width: 25%;"></th> <th colspan="4" style="background-color: #cccccc;">Specimen</th> </tr> <tr> <th style="background-color: #cccccc;">Before Test</th> <th style="background-color: #cccccc;">A</th> <th style="background-color: #cccccc;">B</th> <th style="background-color: #cccccc;">C</th> <th style="background-color: #cccccc;">D</th> </tr> </thead> <tbody> <tr><td>Water Content (%)</td><td>0.00</td><td></td><td></td><td></td></tr> <tr><td>Dry Density (g/cm³)</td><td>0.759</td><td></td><td></td><td></td></tr> <tr><td>Saturation (%)</td><td>0.00</td><td></td><td></td><td></td></tr> <tr><td>Void Ratio</td><td>2.49</td><td></td><td></td><td></td></tr> <tr><td>Diameter (mm)</td><td>133.333</td><td></td><td></td><td></td></tr> <tr><td>Height (mm)</td><td>62.300</td><td></td><td></td><td></td></tr> <tr> <th style="background-color: #cccccc;">Test Data</th> <th style="background-color: #cccccc;">A</th> <th style="background-color: #cccccc;">B</th> <th style="background-color: #cccccc;">C</th> <th style="background-color: #cccccc;">D</th> </tr> <tr><td>Unconfined Strength (kPa)</td><td>5.254</td><td></td><td></td><td></td></tr> <tr><td>Undrained Shear Strength (kgf/cm²)</td><td>0.054</td><td></td><td></td><td></td></tr> <tr><td>Undrained Shear Strength (kPa)</td><td>2.627</td><td></td><td></td><td></td></tr> <tr><td>Rate of Strain (mm/min)</td><td>1.000000</td><td></td><td></td><td></td></tr> <tr><td>Description</td><td></td><td></td><td></td><td></td></tr> <tr> <th colspan="2" style="background-color: #cccccc;">Project Information</th> <th colspan="2" style="background-color: #cccccc;">Specimen Description</th> </tr> <tr><td>Project Num</td><td></td><td>Specimen A</td><td>GTPL20028UCS-2</td></tr> <tr><td>Project</td><td></td><td>Specimen B</td><td></td></tr> <tr><td>Sampling Date</td><td></td><td>Specimen C</td><td></td></tr> <tr><td>Sample #</td><td>GTPL20028UCS-2</td><td>Specimen D</td><td></td></tr> <tr><td>Client</td><td></td><td colspan="2" style="background-color: #cccccc;">Test Variables</td></tr> <tr><td></td><td></td><td>Specific Gravity</td><td>2.65</td></tr> <tr><td></td><td></td><td>Liquid Limit:</td><td></td></tr> <tr><td></td><td></td><td>Plastic Limit:</td><td></td></tr> <tr><td>Remarks</td><td colspan="3"></td></tr> </tbody> </table>		Specimen				Before Test	A	B	C	D	Water Content (%)	0.00				Dry Density (g/cm ³)	0.759				Saturation (%)	0.00				Void Ratio	2.49				Diameter (mm)	133.333				Height (mm)	62.300				Test Data	A	B	C	D	Unconfined Strength (kPa)	5.254				Undrained Shear Strength (kgf/cm ²)	0.054				Undrained Shear Strength (kPa)	2.627				Rate of Strain (mm/min)	1.000000				Description					Project Information		Specimen Description		Project Num		Specimen A	GTPL20028UCS-2	Project		Specimen B		Sampling Date		Specimen C		Sample #	GTPL20028UCS-2	Specimen D		Client		Test Variables				Specific Gravity	2.65			Liquid Limit:				Plastic Limit:		Remarks			
		Specimen																																																																																																													
Before Test		A	B	C	D																																																																																																										
Water Content (%)		0.00																																																																																																													
Dry Density (g/cm ³)		0.759																																																																																																													
Saturation (%)		0.00																																																																																																													
Void Ratio		2.49																																																																																																													
Diameter (mm)		133.333																																																																																																													
Height (mm)		62.300																																																																																																													
Test Data		A	B	C	D																																																																																																										
Unconfined Strength (kPa)	5.254																																																																																																														
Undrained Shear Strength (kgf/cm ²)	0.054																																																																																																														
Undrained Shear Strength (kPa)	2.627																																																																																																														
Rate of Strain (mm/min)	1.000000																																																																																																														
Description																																																																																																															
Project Information		Specimen Description																																																																																																													
Project Num		Specimen A	GTPL20028UCS-2																																																																																																												
Project		Specimen B																																																																																																													
Sampling Date		Specimen C																																																																																																													
Sample #	GTPL20028UCS-2	Specimen D																																																																																																													
Client		Test Variables																																																																																																													
		Specific Gravity	2.65																																																																																																												
		Liquid Limit:																																																																																																													
		Plastic Limit:																																																																																																													
Remarks																																																																																																															
<div style="text-align: center;">CP</div> <div style="text-align: center;">Tested By</div>																																																																																																															

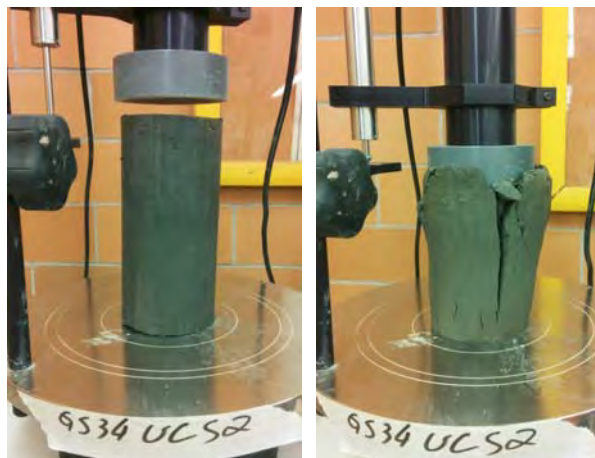
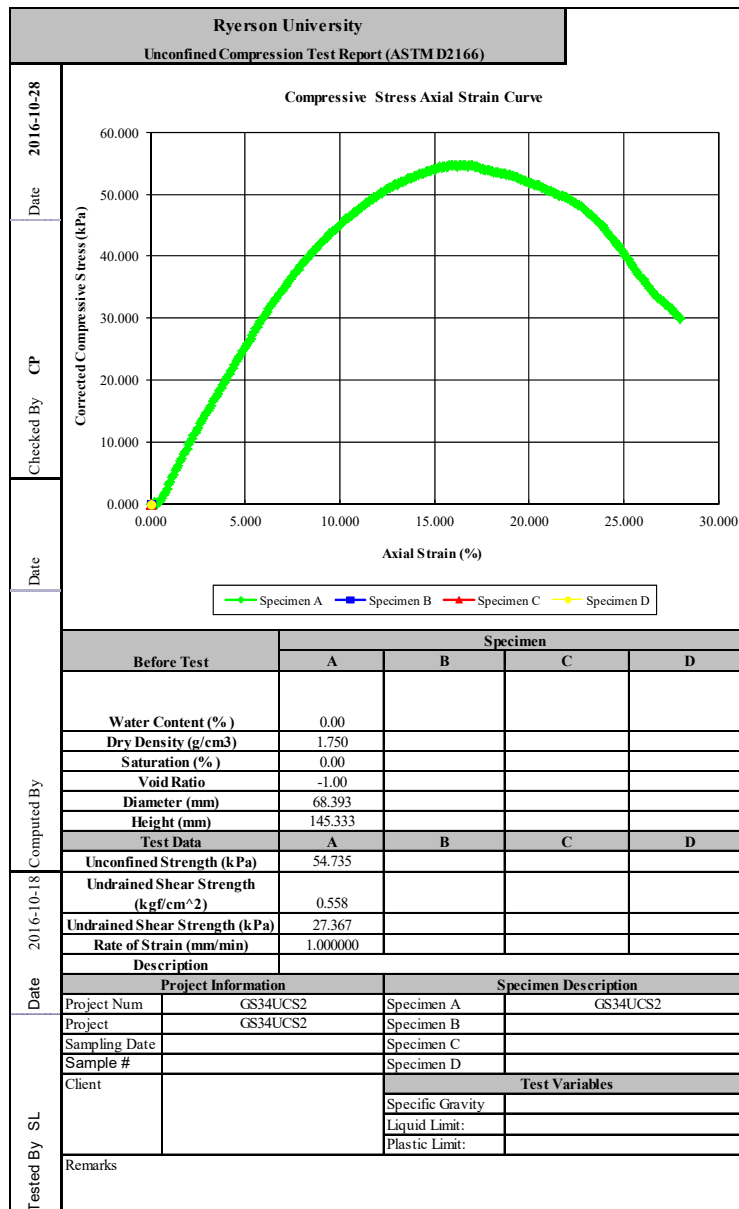
7.7 UCS TEST RESULTS OF UNTREATED ORGANIC CLAYS

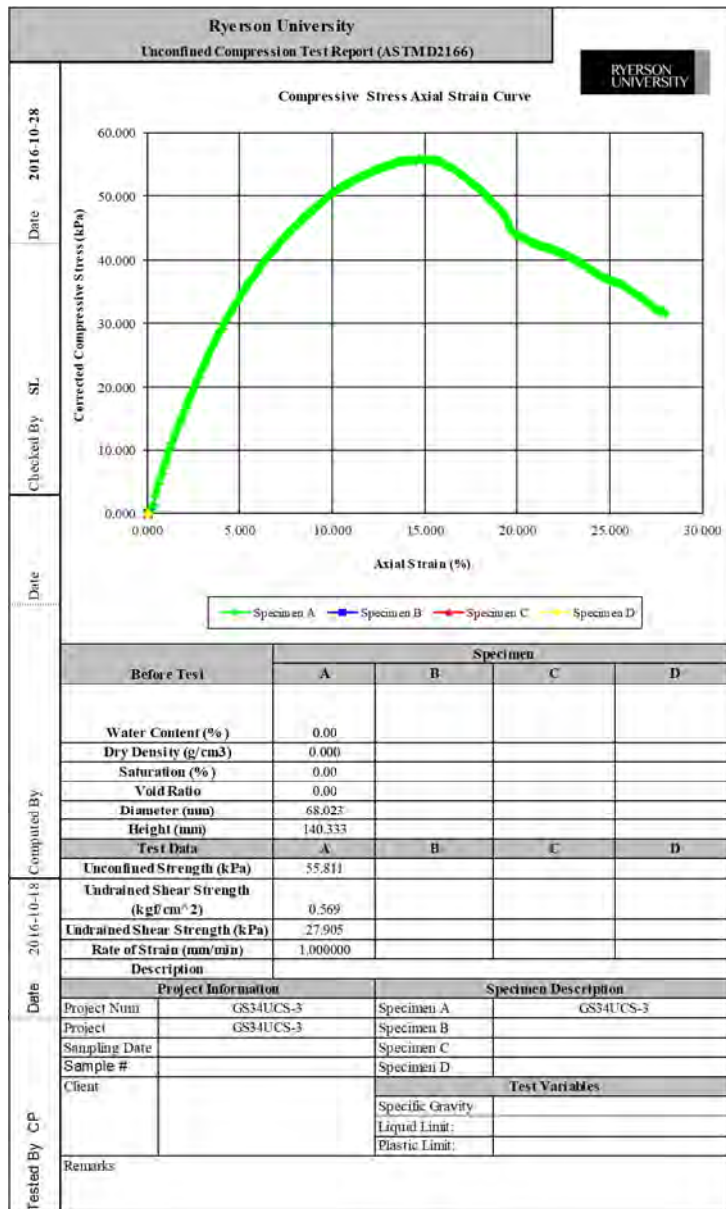












7.8 SPECIFIC GRAVITY TEST

Item No.	Calc Method	Description	Unit	Trial			
GTPSG-2				Example	1	2	3
1	Note	Flask No.		1	1	3	4
2	Measure	Mass of flask	g	171.05	190.3	167.13	171.56
3	Measure	Mass of flask+ water	g	667.88	687.92	664.29	669.16
4	#3-#2	Mass of water(500ml)	g	496.83	497.62	497.16	497.6
5	Measure	Temperature of water	oC	25	25	25	24
6	Reference	Water density based on table	g/ml	0.99708	0.99708	0.99708	0.99733
7	#4 / #6	Volume of Flask	ml	498.285	499.077	498.616	498.932
8	Measure	Mass of wet flask	g	171.05	190.77	167.23	172.25
9	Measure	Mass of soil + flask	g	209.70	257.75	237.15	246.80
10	#9 - #8	Mass of soil	g	38.65	66.98	69.92	74.55
11	Measure	Mass of flask + soil + water	g	692.05	729.24	707.02	714.2
12	Measure	Temperature of water	oC	23	25.5	27	27
13	#11 - #10 - #8	Mass of water	g	482.35	471.49	469.87	467.40
14	Reference	density of water	g/ml	0.99757	0.99695	0.99655	0.99655
15	#13 / #14	Volume of water	ml	483.525	472.932	471.497	469.018
16	#7 - #15	Volume of soil	ml	14.76003	26.14486	27.1193	29.91404
17	#10 / #16	Density of soil	g/ml	2.618559	2.56188	2.578238	2.492141
18	#17 / #14	Specific Gravity	g/ml	2.624938	2.569718	2.587164	2.500769
Specific gravity		average		2.552550095			
19	Measure	Water content	%				
Date 2016-08-18		Tested By AA		Checked By SL			

Item No.	Calc Method	Description	Unit	Trial			
GTP SG-4				E.g.	1	2	3
1	Note	Flask No.		1	3		
2	Measure	Mass of flask	g	171.05	166.95		
3	Measure	Mass of flask+ water	g	667.88	664.71		
4	#3-#2	Mass of water(500ml)	g	496.83	497.76		
5	Measure	Temperature of water	oC	25	21		
6	Reference	Water density based on table	g/ml	0.99708	0.99802		
7	#4 / #6	Volume of Flask	ml	498.285	498.748		
8	Measure	Mass of wet flask	g	171.05	167.12		
9	Measure	Mass of soil + flask	g	209.70	199.02		
10	#9 - #8	Mass of soil	g	38.65	31.90		
11	Measure	Mass of flask + soil + water	g	692.05	683.76		
12	Measure	Temperature of water	oC	23	21		
13	#11 - #10 - #8	Mass of water	g	482.35	484.74		
14	Reference	density of water	g/ml	0.99757	0.99802		
15	#13 / #14	Volume of water	ml	483.525	485.702		
16	#7 - #15	Volume of soil	ml	14.76003	13.04583		
17	#10 / #16	Density of soil	g/ml	2.618559	2.445226		
18	#17 / #14	Specific Gravity	g/ml	2.624938	2.450077		
Specific gravity		average		2.450076805			
19	Measure	Water content	%				
Date 2016-09-26		Tested By CP		Checked By SL			

Item No.	Calc Method	Description	Unit	Trial			
GTP SG-5					8.5% cem	2	3
1	Note	Flask No.		1	4		
2	Measure	Mass of flask	g	190.25	171.48		
3	Measure	Mass of flask+ water	g	688.42	669.52		
4	#3-#2	Mass of water(500ml)	g	498.17	498.04		
5	Measure	Temperature of water	oC	21	21		
6	Reference	Water density based on table	g/ml	0.99802	0.99802		
7	#4/#6	Volume of Flask	ml	499.158	499.028		
8	Measure	Mass of wet flask	g	190.81	171.94		
9	Measure	Mass of soil + flask	g	241.04	216.78		
10	#9-#8	Mass of soil	g	50.23	44.84		
11	Measure	Mass of flask + soil + water	g	718.6	696.5		
12	Measure	Temperature of water	oC	21	21		
13	#11-#10-#8	Mass of water	g	477.56	479.72		
14	Reference	density of water	g/ml	0.99802	0.99802		
15	#13/#14	Volume of water	ml	478.507	480.672		
16	#7 - #15	Volume of soil	ml	20.65089	18.35635		
17	#10 / # 16	Density of soil	g/ml	2.432341	2.442752		
18	#17/#14	Specific Gravity	g/ml	2.437166	2.447598		
Specific Gravity		Average		2.442382339			
19	Measure	Water content	%				
Date 2016-09-27		Tested By: CP		Checked By SL			

7.9 MINIATURE VANE SHEAR TEST

Filename:									
Laboratory Miniature Vane Shear Test, ASTM D4648-00									
Ryerson University									
PROJECT INFORMATION									
Client	Golder Associates			Boring Number					
Project Name				Sample Number		GS25 MV-1			
Sample Location	King, ON			Sample Depth					
Specimen Description	Organic Clay			Specimen Remarks					
Machine No.	1			Vane Diameter (mm)		12.7			
Spring Calibration Factor(N.m ⁻¹)	0.0042			Vane Height (mm)		12.7			
				Vane Constant		233644.86			
Number of Tests		Undisturbed Soil				Remolded Soil			
Trial Numbers		Trial 1	Trial 2	Trial 3	Trial 4	Trial 5	Trial 6	Trial 7	Trial 8
Initial Stress Reading(°)		346/5	80.5/-10	79/59		255/-10	255/75	346/63	
Final Stress Reading(°)		378.5/41	109/30	99/110		271/57	271/145	360/110	
Stress Scale Difference(°)		33	29	20.0		16	16.0	14.0	
Applied Torque(N.m)		0.1365	0.1197	0.0840	0.0000	0.0672	0.0672	0.0588	0.0000
Undrained Shearing Strength(kPa)		31.89	27.97	19.63	0.00	15.70	15.70	13.74	0.00
Average Undrained Shear Strength (kPa)		26.50				15.05			
Standard Deviation		6.26				1.13			
Sensitivity		1.76							
Test Date	20160824		Tested By		AA		Checked By		SL



Laboratory Investigation of Deep Soil Mixing in Treatment of Organic Clays in Ontario

Filename:									
Laboratory Miniature Vane Shear Test, ASTM D4648-00					Ryerson University				
PROJECT INFORMATION									
Client	Golder Associates			Boring Number					
Project Name				Sample Number		GS34 MV-1			
Sample Location	King, ON			Sample Depth					
Specimen Description	Organic Clay			Specimen Remarks					
Machine No.	1			Vane Diameter (mm)		12.7			
Spring Calibration Factor(N.m ⁻¹)	0.0042			Vane Height (mm)		12.7			
				Vane Constant		233644.86			
Number of Tests		Undisturbed Soil				Remolded Soil			
Trial Numbers		Trial 1	Trial 2	Trial 3	Trial 4	Trial 5	Trial 6	Trial 7	Trial 8
Initial Stress Reading(°)		52/343.5	29/73	15/164		3.5/252	49/256	19/342.5	
Final Stress Reading(°)		98/363	85/93	69/187		40/263.5	100/263	60/353.5	
Stress Scale Difference(°)		20	20	23.0		11.5	7.0	11.0	
Applied Torque(N.m)		0.0819	0.0840	0.0966		0.0483	0.0294	0.0462	
Undrained Shearing Strength(kPa)		19.14	19.63	22.57		11.29	6.87	10.79	
Average Undrained Shear Strength (kPa)		20.44				9.65			
Standard Deviation		1.86				2.42			
Sensitivity		2.12							
Test Date	20161018		Tested By	CP		Checked By	SL		

Laboratory Investigation of Deep Soil Mixing in Treatment of Organic Clays in Ontario

Filename:									
Laboratory Miniature Vane Shear Test, ASTM D4648-00					Ryerson University				
PROJECT INFORMATION									
Client	Golder Associates			Boring Number					
Project Name				Sample Number		GS34 MV-1			
Sample Location	King, ON			Sample Depth					
Specimen Description	Organic Clay			Specimen Remarks					
Machine No.	1			Vane Diameter (mm)		12.7			
Spring Calibration Factor(N.m ⁻¹)	0.0042			Vane Height (mm)		12.7			
				Vane Constant		233644.86			
Number of Tests		Undisturbed Soil				Remolded Soil			
Trial Numbers		Trial 1	Trial 2	Trial 3	Trial 4	Trial 5	Trial 6	Trial 7	Trial 8
Initial Stress Reading(°)		53/256	36/347	24/73.5		43/164	15/252.5	-15/343	
Final Stress Reading(°)		110/274	90/367.5	120/98		90/175	61/263.5	41/354	
Stress Scale Difference(°)		18	21	24.5		11	11.0	11.0	
Applied Torque(N.m)		0.0756	0.0861	0.1029		0.0462	0.0462	0.0462	
Undrained Shearing Strength(kPa)		17.66	20.12	24.04		10.79	10.79	10.79	
Average Undrained Shear Strength (kPa)		20.61				10.79			
Standard Deviation		3.22				0.00			
Sensitivity		1.91							
Test Date	20161018		Tested By	CP		Checked By	SL		



7.10 TRIAXIAL TEST RESULTS OF ORGANIC CLAYS

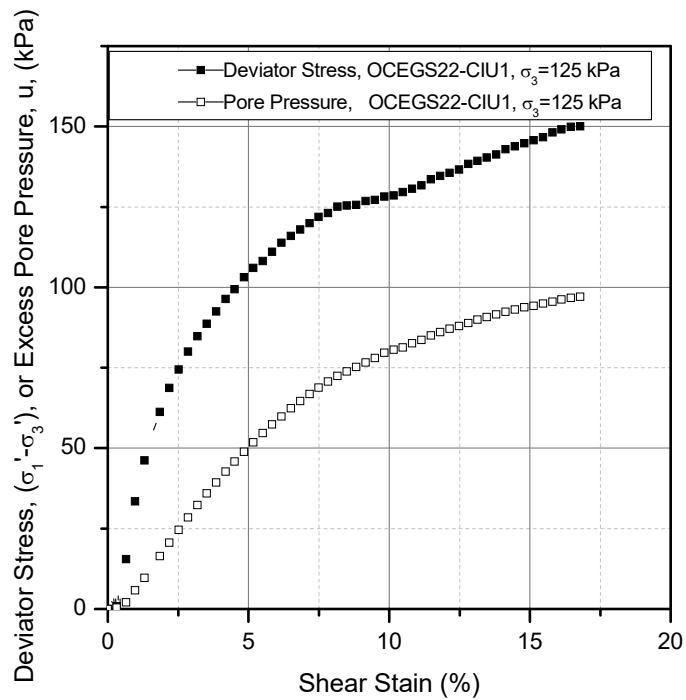


Figure 7.10a Stress and pore pressure development during shearing for OCEGS22-CIU1

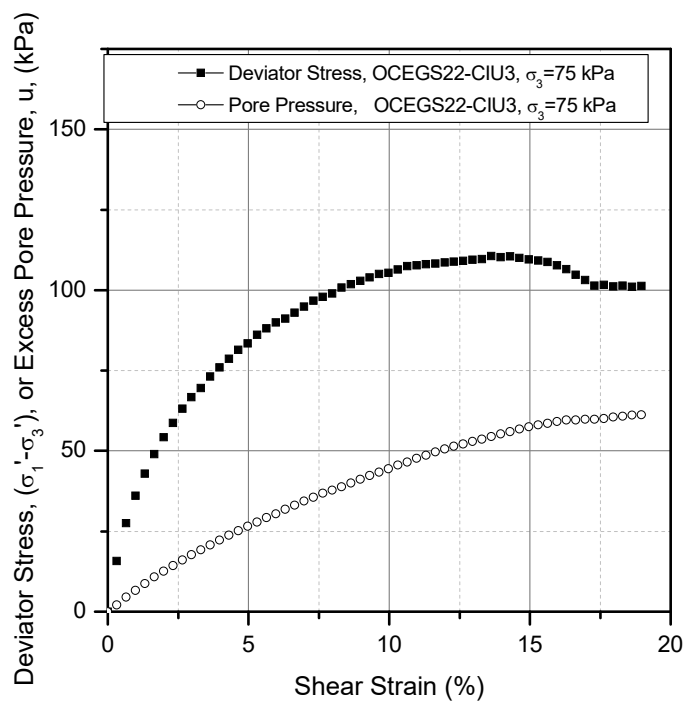


Figure 7.10b Stress and pore pressure development during shearing for OCEGS22-CIU3

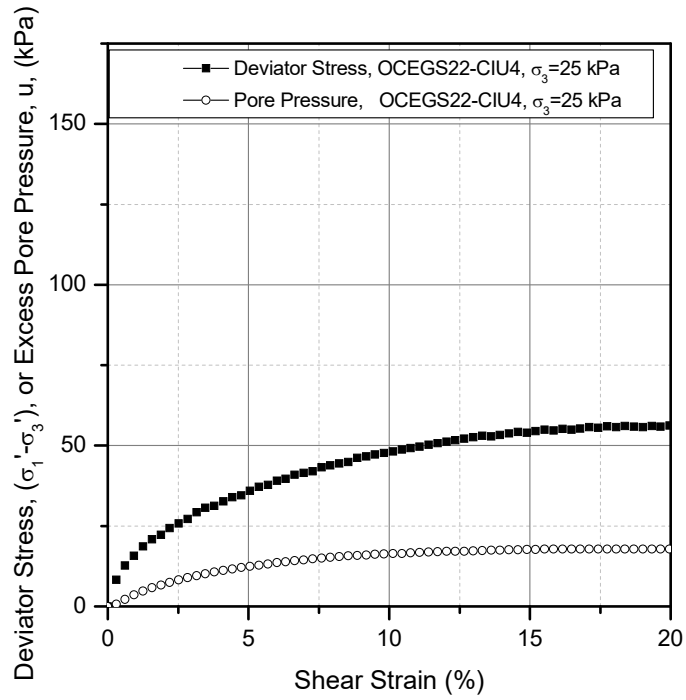


Figure 7.10c Stress and pore pressure development during shearing for OCEGS22-CIU4

APPENDIX F

Ground Improvement Methods

F.1 INTRUSIVE GROUND IMPROVEMENT METHODS

Intrusive ground improvement methods are generally used to modify the subsurface conditions at sites where the subsurface conditions are poor, in order to improve the shear strength and deformation properties of low-strength soil deposits and/or potentially enhance the hydraulic conditions of low permeability conditions.

If employed at this Highway 400 embankment widening site, intrusive ground improvement methods would enhance the shear strength and deformation properties of the cohesive and organic soils beneath the embankment widening footprint. Considering the extent of the organics and/or cohesive soils at this site, any intrusive ground improvement method at this site should extend to a depth of about 10 m to 12 m below the ground surface at the toe of the existing embankment (to approximately Elevation 285 m).

The following intrusive ground improvement methods were considered:

- Controlled modulus columns;
- Stone columns; and
- Deep soil mixing.

For all three methods a treated area is comprised of a grid (pattern) of columns of grout, aggregate or a mixture of concrete and in-situ soils. The intrusive ground improvement methods will result in an increase in the strength and the stiffness modulus of the overall soil matrix in the area of the columns; however, the area of soils between the columns remains unaltered. The spacing of the columns is a function of the soil stratigraphy, the strength and modulus of the untreated soils, the application and the intended loadings and the performance criteria. The length of the columns is designed such that they extend to a stratum where there is sufficient bearing resistance (which can be achieved by a combination of shaft friction and end bearing).

For each of the three methods noted above, a load transfer platform (LTP) consisting of 0.3 m to 1 m of granular fill is placed over the columns at ground surface. The purpose of the LTP is to promote strain compatibility and load sharing between all of the columns. Subsequently, after completion of the columns, layers of biaxial geogrid and engineered fill are placed over the full footprint of the embankment widening area. For the option of deep soil mixing where the entire footprint for the widened embankment undergoes mixing, as opposed to discrete or mixed soil-cement columns, an LTP would likely not be required.

Other intrusive ground improvement methods such as vibro-compaction (compaction of in-situ soils), and permeation (or jet) grouting are more applicable for granular soils. These methods are not considered applicable at this site as the silty clay deposit is considered to have too low a permeability to allow for the benefits of these methods; therefore, they are not considered further.

There are a number of key site constraints that would need to be taken into consideration in selecting the method and ultimately the design of ground improvement, such as:

- The presence of tree limbs/branches/roots, as observed in the test pits, may pose a challenge to the constructability implementation of the intrusive ground improvement method; however, if encountered the spacing of the columns could be adjusted during construction.
- As discussed in Section 4.2.4, 8 and 4.2.10 of the Foundation Investigation Report, “blowing sands” (i.e., a rise/incursion of soil into the auger/casing due to groundwater pressure) was encountered in

some of the boreholes and the boreholes needed to be advanced by wash boring drilling techniques to counterbalance the groundwater pressure in the sandy silt to silt and sand to sand deposit underlying the silty clay deposit. Therefore, consideration would need to be given to the method of installation of the intrusive ground improvement in order to avoid disturbance to the non-cohesive layer and to ensure that there is not loss of ground at depth due to the upward migration of the non-cohesive deposit.

- The site is located adjacent to a wetland area and consideration would need to be given to the impact of the selected method on the adjacent wetland.

For each of the intrusive ground improvement method noted above, a brief description of the method, construction details, advantages and disadvantages and suggestions for parameters to record during construction and quality assurance laboratory testing is presented below.

F.2 DESIGN CRITERIA

If controlled modulus columns or stone columns were used, it is recommended that horizontal movement or magnitude of lateral spreading of the organic and cohesive deposits at depth and beyond the toe of the embankment be limited in magnitude.

Depending on the ground improvement method adopted and the diameter of the columns selected, the silty clay deposit may not have sufficient strength and therefore may not develop sufficient passive resistance to limit the deformation of the columns. It is recommended that for a total amount of settlement of 100 mm at the location where the embankment loading is the greatest, the lateral movement of the silty clay deposit be limited to the product of magnitude of the vertical settlement times the Poisson's ratio of 0.35 (i.e. $\delta v * \mu$), resulting in a lateral movement criteria of 35 mm.

F.3 CONTROLLED MODULUS COLUMNS

Controlled modulus columns (CMC) are constructed using reverse flight augers advanced under a high torque and high static down thrust, which results in compressing the soil laterally. This displacement results in densification of the surrounding soil, which improves the load transfer into the column element. Once the target depth has been reached the hole is backfilled with pressurized cement grout, typically at about 500 kPa or less, that further densifies the surrounding soils. The result is a CMC element that is significantly stiffer than the soil around it. The displacement auger is advanced without air or water jetting and without generating spoil. The load from the embankment widening is transmitted along the shaft of the CMC column and then transmitted to the more competent deeper stratum. In addition, if additional strength is required, a central reinforcing bar can be added to the cement grout column.

The CMC system (diameter of columns and spacing) in conjunction with the LTP is designed to distribute the load from the widened embankment to the soil mass by increasing the overall soil deformation modulus of the soil mass and limiting the settlements to within the limits permitted by MTO's Embankment Settlement Criteria (2010). The columns are not designed to directly support the loads imposed by the widened embankment, but to improve the global response of the subgrade soil(s) in order to control settlement. The system is generally designed to transfer 50% to 95% of the load to the CMCs while the remainder of the load is transmitted to the soils between the CMCs. The ratio of the load sharing is dependent upon the type and stiffness of the soils between the CMCs as well as the allowable settlement for the embankment. In some applications a geogrid is also used within the LTP layer to better distribute the loads. Typical diameters for the columns are 360 mm, 450 mm and 520 mm. The process is suited for a wide variety of soils including organics soils, soft soils and mixed fills that are present overlying stiff or dense bearing strata.

Column lengths can be up to 30 m; however, for greater columns lengths, depending on the thickness and undrained shear strength of cohesive deposits outside and beyond the ground improvement, the cohesive deposit may not possess sufficient strength to develop adequate passive/restraining pressure. The result is an increase in bending/flexural stresses of the column which leads to lateral displacement/bulging of the column. This may result in cracking and loss of shear resistance and potentially axial resistance of the CMC columns. It is estimated that the length of the columns at this site would need to be in the order of 15 m to 20 m; the silty clay layer at the toe of the existing embankment comprises a significant proportion of the overall length of the CMC column. One potential design solution to increase the shear resistance of the treated area would be to extend the footprint of the columns to beyond the embankment widening so that the additional columns would provide additional lateral restraint. At depth, the increase in stress due to the load from the embankment widening is considered to extend from ground surface outwards and downwards at 0.5 horizontal to 1 vertical (0.5H:1V). Considering the silty clay layer extends to a depth of between about 11.7 m and 13.4 m below the base of the existing embankment, additional ground improvement within the 0.5H:1V zone would result in two to three additional rows of columns, depending on the spacing used in the design, which would need to be considered relative to right-of-way limits.

During column construction the following information is recorded:

- Column identification;
- Working grade level;
- Column diameter;
- Start time, time at bottom of hole and finish time;
- Bottom depth of hole;
- Speed of rotation and rate of advancement and withdrawal of the augers;
- Torque and down-thrust during advancements
- Pressure and volume of injected cement/grout from which the in-situ profile of the columns are determined; and
- Laboratory testing for compressive strength of the grout.

During the auger advancement, the changes in the down pressure, speed and torque of the auger are interpreted in the field to detect changes in subsurface conditions to allow for the interpretation the column lengths to be adjusted in the field.

Quality Control testing would include the following:

- Grout strength;
- Load testing of column (ASTM D1143);
- Pile Integrity Testing; and
- Dynamic (i.e. Statnamic) load tests.

F.4 STONE COLUMNS

Stone columns are an intrusive ground improvement method comprising dense aggregate columns (stone columns) constructed by means of compaction of aggregate placed in a pre-drilled hole through either vibration or vertical ramming with a hammer. The compacted aggregate columns create strong and stiff elements within weak soils that reduce settlement and increase global stability of embankments. Initially an auger hole is advanced and then the aggregate is placed in lifts and compacted with a downhole hammer, or the aggregate is placed in the hole and vibrated in-situ thus compacting/densifying the aggregate.

With the compaction method the hole can be advanced either with or without a sleeve which is left in place. The advantage of the sleeve is that it adds an element of stiffness to the column; however, conversely an advantage of not using a sleeve is that as the aggregate is compacted it can extend further laterally into the surrounding soil, resulting in a larger diameter stone column. The hole/sleeve is advanced to the target depth using a strong static force augmented by high frequency vertical impact energy. The aggregate is then delivered to the bottom of the hole using a hopper and compacted through static down force and dynamic vertical ramming from the hammer. This process densifies the aggregate vertically and the compaction effort causes the sleeve to bulge outwards into the surrounding soil. This process continues for the full length of the sleeve. Conversations with local contractors suggest that the compaction method has some depth limitations typically restricted to a depth of approximately 10 m below ground surface.

This vibro replacement method can be carried out in one of two ways:

- The wet top feed method involves using a water jet to remove the in-situ material and to stabilize the hole. The aggregate is then placed by gravity from the ground surface to the bottom of the hole; the vibrator then compacts the aggregate at the bottom of the hole in lifts, more aggregate is placed and the process is repeated. A disadvantage of the using the wet method for vibro replacement is that there is excess spoil generated which may not be compatible with the site conditions and has to be removed off site.
- The dry vibro method is referred to as a vibro displacement method as the in-situ soils are displaced rather than removed. The advantage of this method is that it virtually eliminates the generation of excess spoil, which is a benefit in environmentally sensitive site conditions and in terms of excess soil management. The hole is advanced using the vibrator probe and a hopper and supply tube direct/discharge the aggregate to the tip of the vibrator probe. The aggregate fills the annular space around the vibrator tip, and fills the void created as the vibrator is lifted about a metre. The vibrator is then lowered, densifying and displacing the underlying stone. The vibro displacement process is repeated until a dense stone column is constructed to the ground surface. As the aggregate is compacted it spreads laterally, into the surrounding soil thereby increasing the overall diameter of the stone column. The extent of lateral spreading of the aggregate depends on the strength and stiffness/compactness parameters of the surrounding soil. Due to the vibration action at depth the soils surrounding the stone column may consolidate and settle; therefore, some local grade raising/regrading in the area of the columns may be required.

A disadvantage of the stone columns installed by compaction is that they can typically only be installed to a depth of 10 m, whereas the columns installed with vibro compaction can extend to depths of about 30 m. Considering that the silty clay layer extends to a depth of between about 11.7 m and 13.5 m below the base of the existing embankment at this site, the use of stone columns installed by compaction method may not be suitable at this site. As with the CMC method, the design of stone columns would need to consider the

strength of the silty clay at depth and the potential for the stone columns to bulge due to the widened embankment loading.

During construction, quality control monitoring of the following key items should be carried out:

- The quantity and type of aggregate used; and
- Vibrator amperage draw, as the densification of the in-situ soils occurs the ground restricts the horizontal movement of the vibrator, resulting in an increased draw to maintain frequency; this is a qualitative measure of the ground improvement.

Prior to construction of the stone columns consideration could be given to carrying out a test installation at a convenient locale, such that three columns are installed in a triangular pattern. Radiating from the stone column the immediately surrounding soils will be relatively “highly” densified and the densification will decrease radially from the column. After construction of the three test columns, Standard Penetration Testing or in-situ vane shear tests would then be carried out within the “highly” densified soils and further away from the stone columns within the less densified soils and modifications could be made to the spacing, if warranted.

Post-construction the following quality assurance testing could be carried out to ascertain as to the efficacy of the stone columns installation in enhancing the strength/resistance of the overall soil mass:

- Standard Penetration Testing (SPT) as noted above;
- Cone Penetration Testing (CPT) / Dilatometer Testing (DMT)/ Load Testing / and Shear Wave Velocity Profiling, as may be considered applicable.

F.5 DEEP SOIL MIXING – WET AND DRY MIXING

Deep soil mixing (DSM) is an intrusive ground improvement method that involves the introduction and mechanical mixing of in-situ soft and weak soils with a cementitious compound, referred to as a binder agent, such as cement/cement lime grout, flyash, gypsum or slag. By increasing the dosage of the binder agent the resulting strength of the mixed soil mass/columns can be increased. The technique can be installed as individual columns, or rows of interlocking columns such that ground improvement can be applied to the entire area where it is required. Columns are typically 0.6 m to 1.0 m in diameter for dry mixing and in the order of up to 3.5 m in wet mixing applications.

There are two types of soil mixing: dry and wet. Dry soil mixing is used to stabilize wet soft soil and typically is best suited to silty and clayey soils with a high groundwater level and consequently a high moisture content (in the order of about 60% or greater and near the liquid limit). Wet soil mixing is used where the water content of the soil is less than 60% and typically can be applied to a greater variety of different soil types. In dry mixing applications columns can be constructed to a maximum depth of about 20 m, whereas in wet soil mixing applications the columns can extend to a maximum depth of about 30 m. Dewatering of the site is not required for either method. An advantage of dry soil mixing in comparison to wet soil mixing is that there is no excess spoil; whereas with wet mixing the excess spoil can be in the order of 50 % to 60 % of the treated volume (O'Rourke and McGinn, 2004). Factors that affect the ease with which soil can be mixed include the following:

- Soil type, plasticity and shear strength. The following soils types have proven to be challenging to mix:
 - Low plasticity clays with a shear strength greater than 75 kPa

- High plasticity clays with shear strength greater than 50 kPa; and,
- Cohesive soils with a moisture content much lower than the liquid limits.
- The organic content of the soil can limit / slow down the hydration of cement;
- Water content;
- Variability in stratigraphy;
- Texture; and
- Obstructions.

Design of DSM ground improvement consists of determining the target depth of the columns or mass treatment in order to sufficiently penetrate the soil that requires treatment and then depending on the application, determining the dosage of the binder material and ascertaining whether columns are sufficient or whether mass treatment is required. Similar to CMC and stone columns, consideration will need to be given to the effect of the presence of the silty clay deposit on the strength of the ground improvement to resist the load from the embankment widening such that tensile and flexural stresses of the DSM columns/unit is not exceeded. Interlocking the columns or creating a “lattice” structure may be required to limit flexural distortions and lateral spreading within the ground improvement area. The columns can also be installed on an incline or batter. In the current application, consideration could be given to battering the row of columns outward along the toe of the proposed slope as it would minimize lateral spreading of the silty clay deposit at depth.

The process of construction involves advancing a soil mixing tool to premix the in-situ soil to the target depth. Many different types of mixing tools have been used by different contractors. The mixing tools can be a single, double or triple blade, they can come in a set of four making a square shape, or the cutter soil mixing (CSM) unit where there are two circulating disks at the base of the Kelly bar. In mass soil mixing applications, a horizontal axis rotary mixing tool is used. The main requirement of the soil mixing tool is that it needs to be able to incorporate and spread the binder evenly in the cavity and along the full length of the cavity formed by the blades. In literature there are reported occurrences of clay materials sticking to mixing tools, or the soil/mixture becoming clumped between blades, thereby inhibiting the mixing process. It is reported that the mixed soil became clumpy particularly when lime was used as the binder agent, as it causes rapid dewatering and causes the clay to become more plastic and difficult to work with. This problem is avoided with the CSM method which uses water injection and vertical cutting heads to essentially create a paste within the clayey deposit which is then mixed with the cement agent to produce a relatively homogenous soil-cement mix throughout the column/panel/mass. A disadvantage of dry mixing is the binding agent is injected using compressed air and there is the potential to fracture the clay with the compressed air; this causes unnecessary disturbance to the surrounding soil and can result in lateral and vertical soil movement at depth, which can destabilize the soil and potentially lead to isolated settlements at the ground surface.

If the DSM were designed for application to the entire footprint of the embankment widening at this site, then the presence of obstructions such as the tree roots / tree limbs encountered in the test pits will slow down production of the installation of the DSM. For this reason, consideration would need to be given to construction of columns so that the load is not transmitted to an area where a tree limb / branch or root is left buried, as it may result in delayed settlement occurring post-construction.

Once the hole is made, a dry binder agent or slurry grout is then pneumatically injected at the base of the hole into the soil as the mixing tool is withdrawn upwards from the hole, leaving behind a dry soil mix column. The injection of the binder agent is flow controlled and synchronized with the upward movement of the mixing tool. The injection of the binder agent can be adapted by varying the amount and mix proportion.

A challenge with DSM is obtaining a uniform or consistent mixture along the full depth and within the footprint where the ground improvement is required, which is affected by variations in the subsurface conditions. For this reason, it is important that laboratory testing be carried out on samples of the site soils using different binder materials and different dosages. During construction, in-situ grab samples should be taken to confirm the homogeneity of the soil-cement mixture over the full depth and strength testing should be carried out on these samples.

Laboratory testing consists of measuring the unconfined compressive strength of cylinder samples of the soil / binder mixture. It is difficult to predict, from only laboratory test results, the field strength and deformation properties of DSM samples on a reasonable level of accuracy due to the challenges with simulating the field conditions in the laboratory, and quite often for a soil mixture with a higher binder dosage the measured unconfined compressive strength can actually be lower compared to a soil mixture with a lower binder dosage. Therefore, when assessing the results of unconfined compressive test results for different binder dosages it is important to look at the overall results and not compare individual results. There are many variables between the laboratory and the field conditions, such as the mixing process, curing conditions, environmental conditions, and field conditions that cannot be replicated in the laboratory analysis, therefore caution should be used when applying the laboratory test results in design.

For this reason, if this intrusive ground improvement method were adopted, it is recommended that a pre-test column be specified to be installed on site at the start of construction. The following routine geotechnical field tests could then be carried out through the centre of the column:

- PQ Coring – this will allow for a larger diameter and continuous sample to be obtained for unconfined compression (UC) testing of compressive strength, stiffness modulus, tensile strength and homogeneity.
- Exposing a panel of the DSM using an excavator so that the success of the mixing can be observed. While there is a limit to the depth of the test pit, it would at least show the success of the shallow portion of the mixing operation.
- Standard Penetration Test (SPT) to measure the “N”-values and allow for an assessment of the increase in stiffness or compactness. Alternatively, a dynamic cone penetration test (DCPT) could be carried out; however, the benefit with the SPT test is that samples are retrieved.
- Cone Penetration Test (CPT) to provide a continuous measurement of the penetration resistance, sleeve friction and pore water pressure at the tip of the probe. Depending on the target strength increase required the CPT probe may not be able to penetrate into the DSM mass.

In addition, the pre-test column could be load tested according to ASTM D1143 to measure the magnitude of settlement under the anticipated load conditions.

During the soil mixing process, the design-bid-build contractor should record the following information:

- Column identification;

- Working grade;
- Column diameter;
- Start time, time at bottom of hole and finish time;
- Bottom depth of hole;
- Binder type;
- Binder dosage rate and air pressure (for dry mixing);
- Slurry dosage rate and pressure (for wet mixing);
- Mixing tool revolutions per minute (rpm) during penetration and withdrawal;
- Total quantity of slurry (wet mixing) or binder (dry mixing) added during mixing; and
- Laboratory tests on binder/slurry samples for UCS, stiffness modulus and tensile strength, and the samples observed for homogeneity.

To assist with the curing process a portion of the embankment could be constructed over the ground improvement area to provide confinement. Curing of the soil and binder mixture will require about 2 weeks to 6 weeks at which point previous studies have indicated that the strength of the soil mixture will be 10 times to 50 times greater and much stiffer than at mixing time. Typically, about 90% of the ultimate strength of the mixed soil may be reached after 3 weeks following construction; however, this is dependent on the type of soil being mixed, presence of organics and whether bentonite was used in the mixing process as bentonite will tend to slow down the curing process.

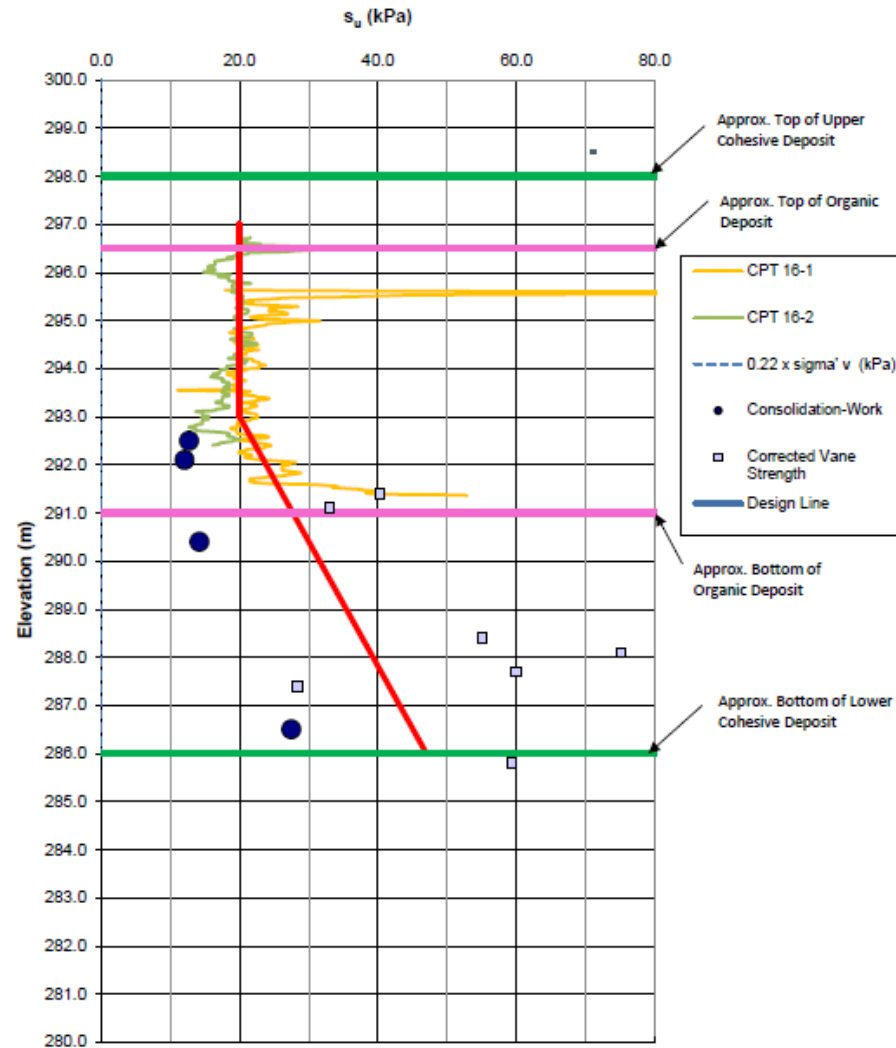
APPENDIX G

Parameter Selection Data

Undrained Shear Strength

Station 15+260 to 15+360

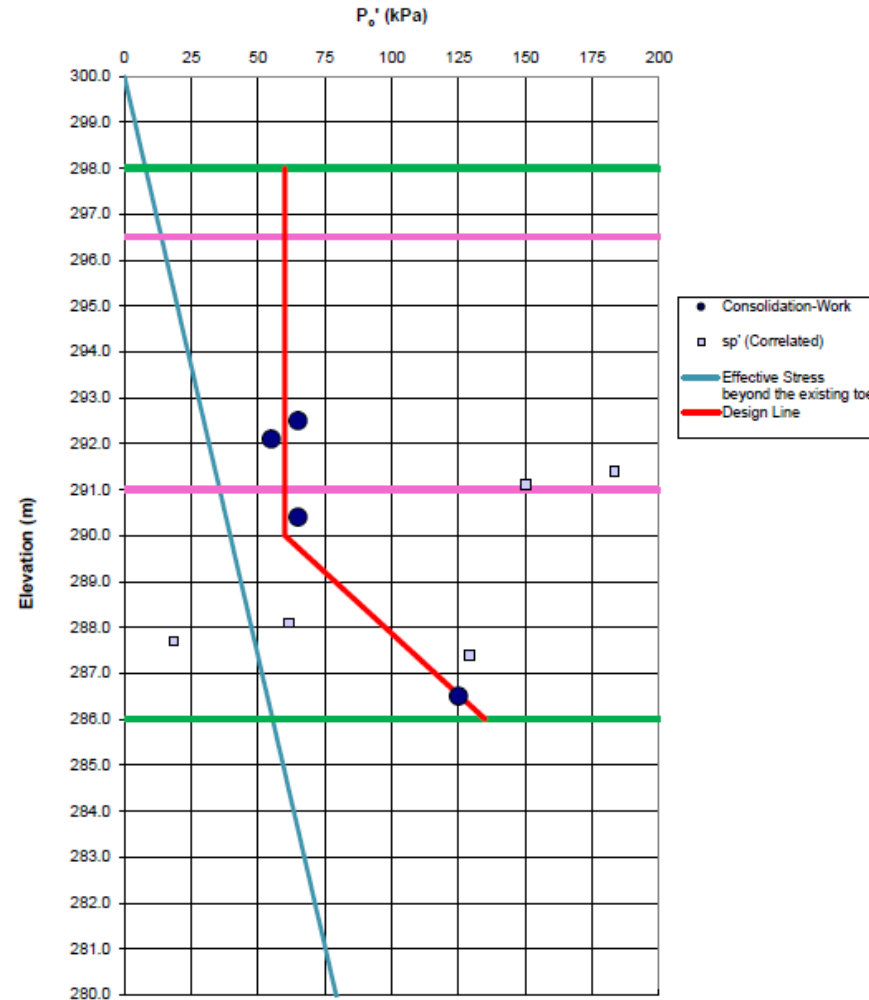
Figure G-1



Preconsolidation Pressure

Station 15+260 to 15+360

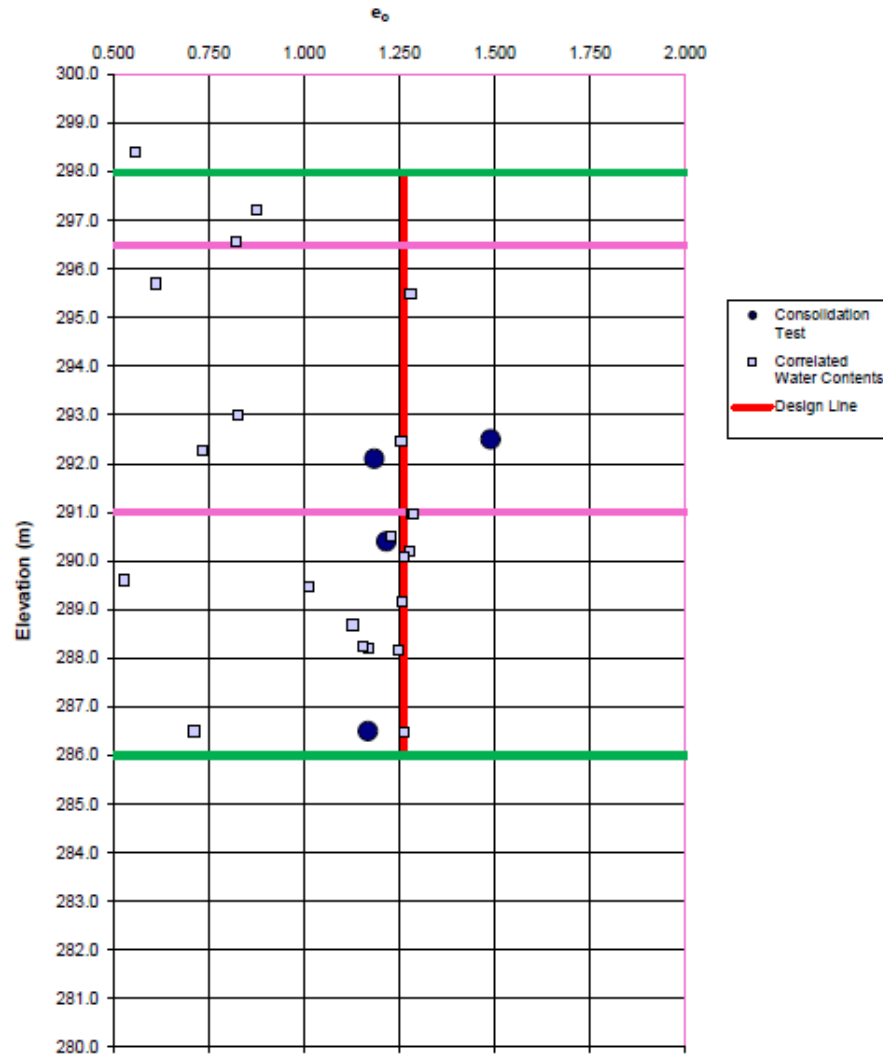
Figure G-2



Void Ratio

Station 15+260 to 15+360

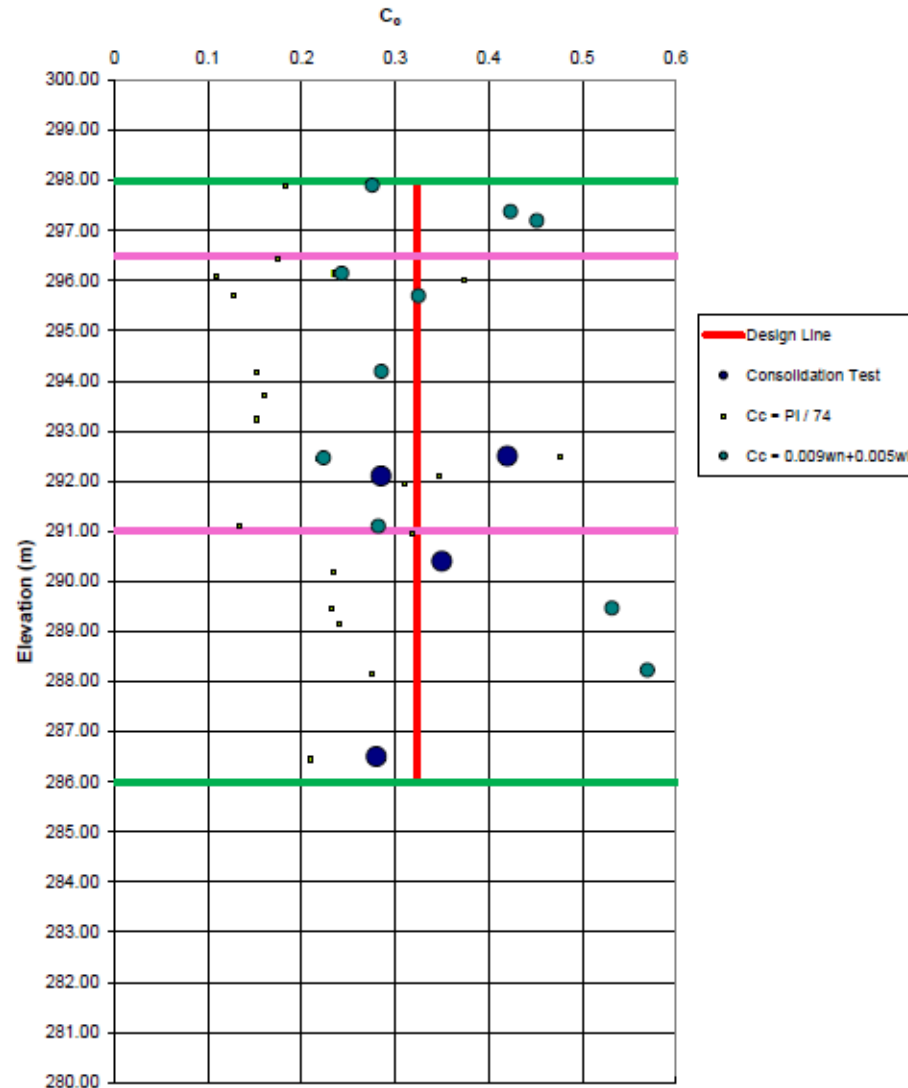
Figure G-3



Compression Index

Station 15+260 to 15+360

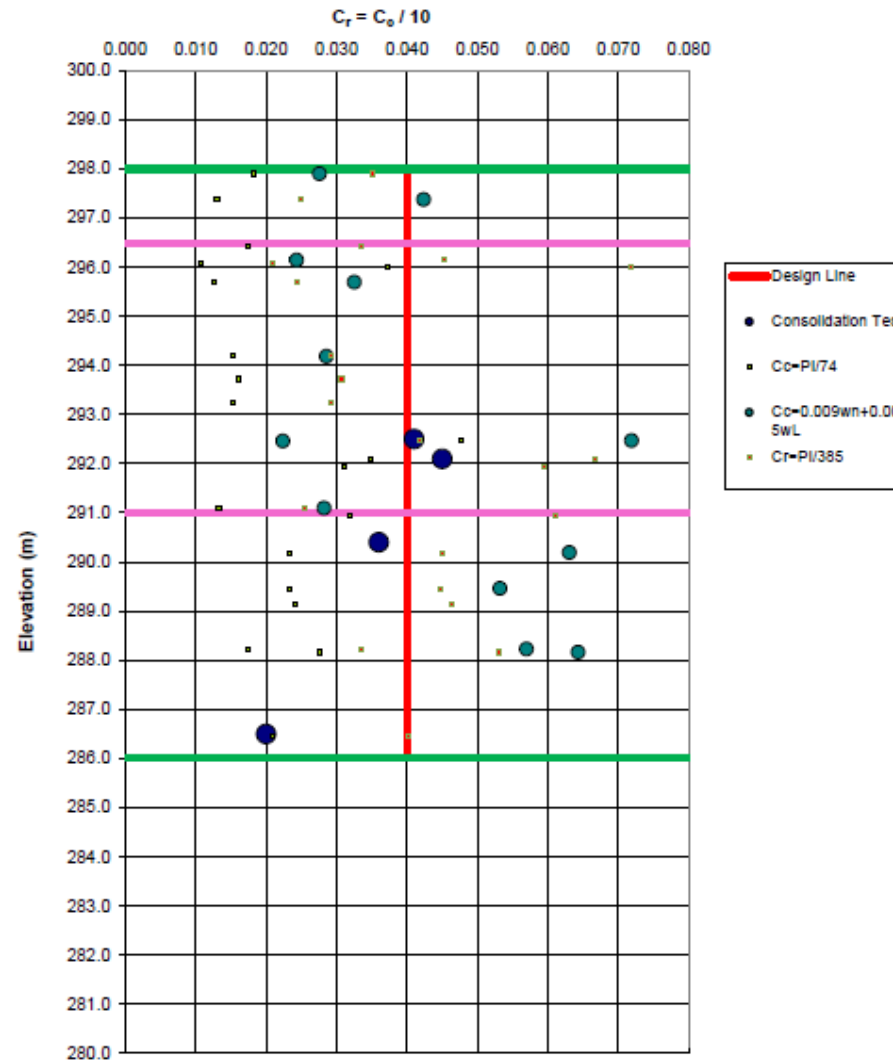
Figure G-4



Recompression Index

Station 15+260 to 15+360

Figure G-5

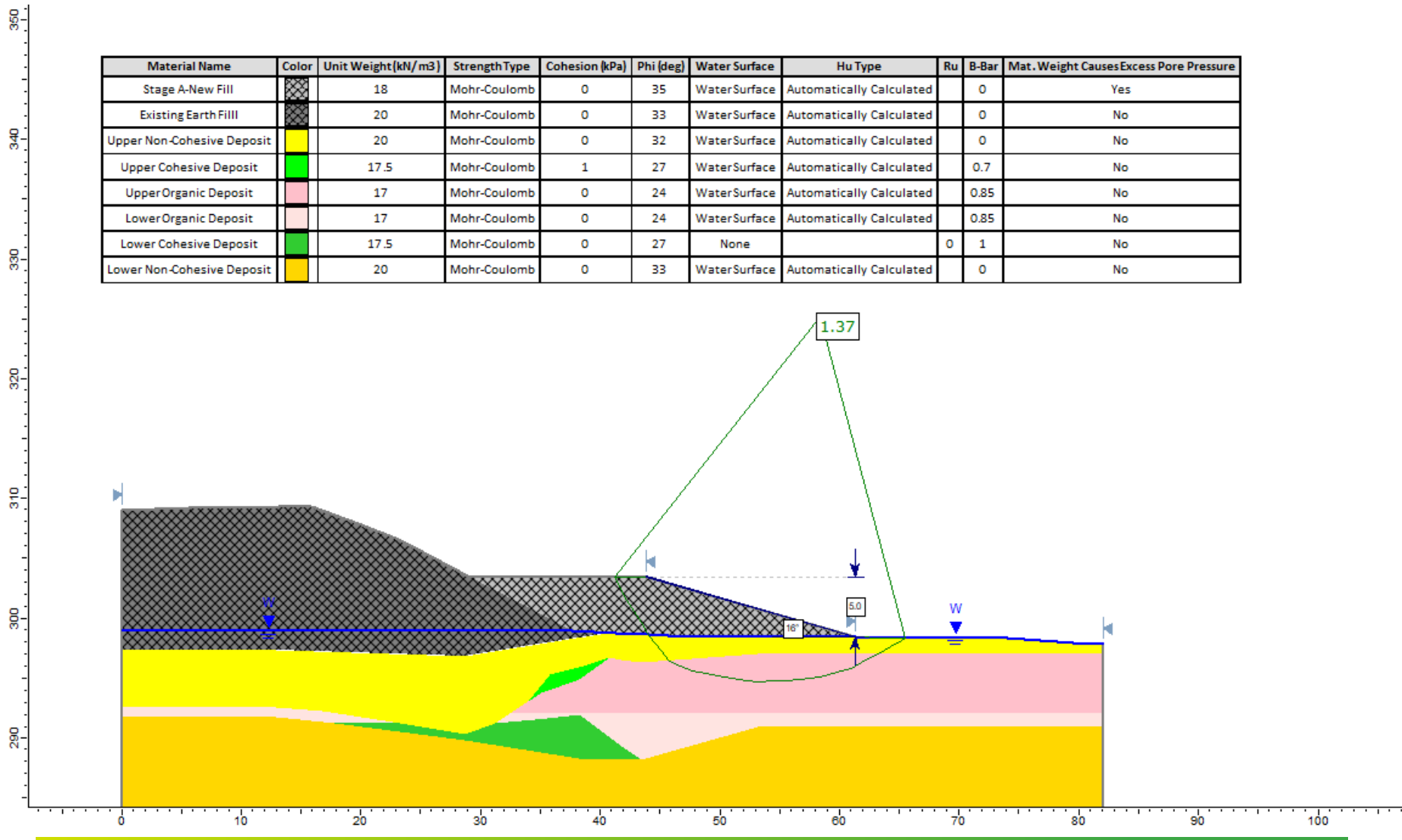




Global Stability Analysis

15+290 Stage A – Temporary Condition (B-bar Analysis)

Figure G-7



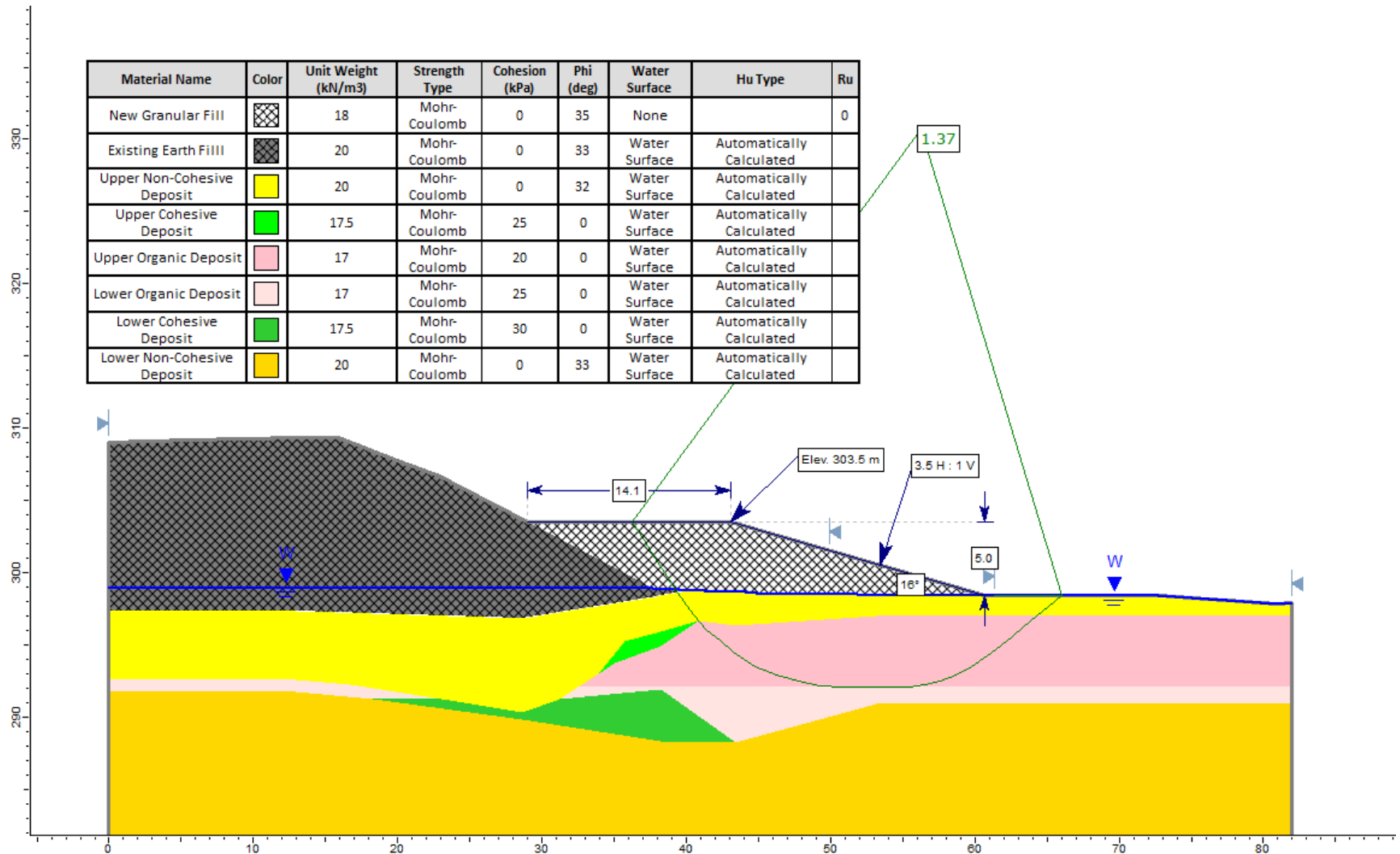
APPENDIX H

Global Stability Analysis Results

Global Stability Analysis

15+290 Stage A – Temporary Condition (Undrained Analysis)

Figure H-1

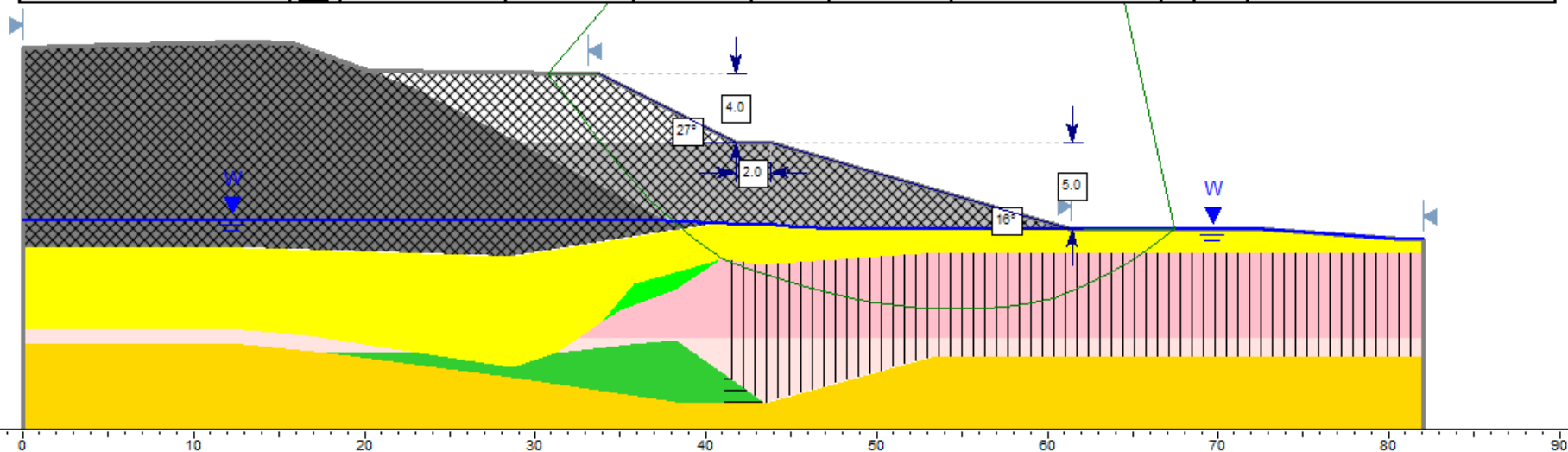


Global Stability Analysis

Figure H-2

15+290 Stage B – Temporary Condition (B-bar Analysis)

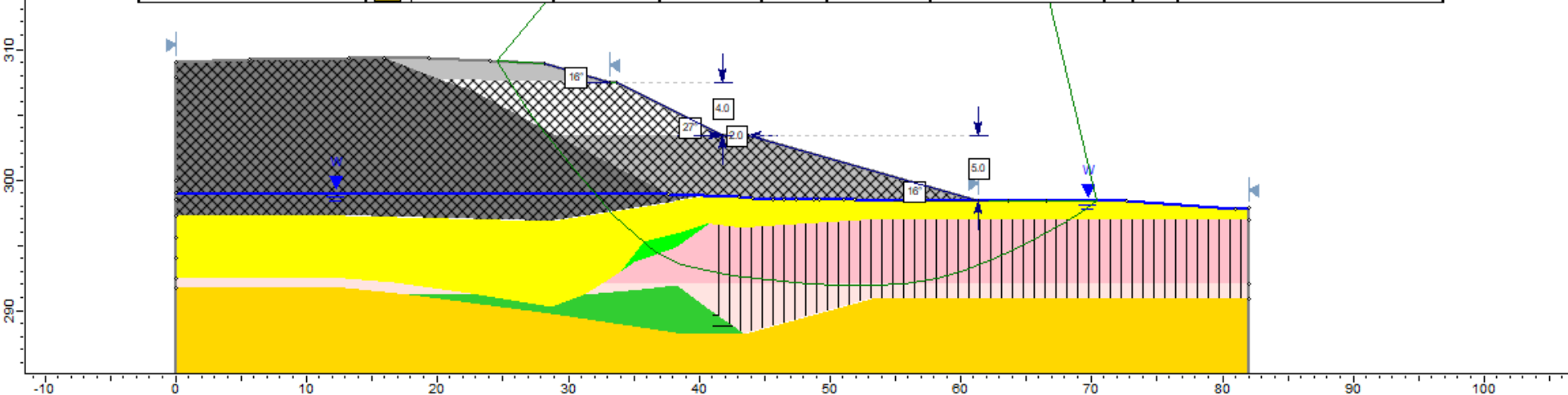
Material Name	Color	Unit Weight (kN/m ³)	Strength Type	Cohesion (kPa)	Phi (deg)	Water Surface	Hu Type	Ru	B-Bar	Mat. Weight Causes Excess Pore Pressure
Stage B-New Fill		18	Mohr-Coulomb	0	35	None		0	0	Yes
Stage A-New Fill		18	Mohr-Coulomb	0	35	Water Surface	Automatically Calculated		0	Yes
Existing Earth Fill		20	Mohr-Coulomb	0	33	Water Surface	Automatically Calculated		0	No
Upper Non-Cohesive Deposit		20	Mohr-Coulomb	0	32	Water Surface	Automatically Calculated		0	No
Upper Cohesive Deposit		17.5	Mohr-Coulomb	1	27	Water Surface	Automatically Calculated		0.7	No
Upper Organic Deposit		17	Mohr-Coulomb	0	24	None		0	0.85	No
Lower Organic Deposit		17	Mohr-Coulomb	0	24	Water Surface	Automatically Calculated		0.85	No
Lower Cohesive Deposit		17.5	Mohr-Coulomb	0	27	Water Surface	Automatically Calculated		1	No
Upper Organic Deposit-25%Bbar		17	Mohr-Coulomb	0	24	Water Surface	Automatically Calculated		0.21	No
Lower Organic Deposit-25%Bbar		17	Mohr-Coulomb	0	24	Water Surface	Automatically Calculated		0.21	No
Lower Cohesive Deposit-25%Bbar		20	Mohr-Coulomb	0	27	Water Surface	Automatically Calculated		0.25	No
Lower Non-Cohesive Deposit		20	Mohr-Coulomb	0	33	Water Surface	Automatically Calculated		0	No



15+290 Stage C – Temporary Condition (B-bar Analysis)

1.46

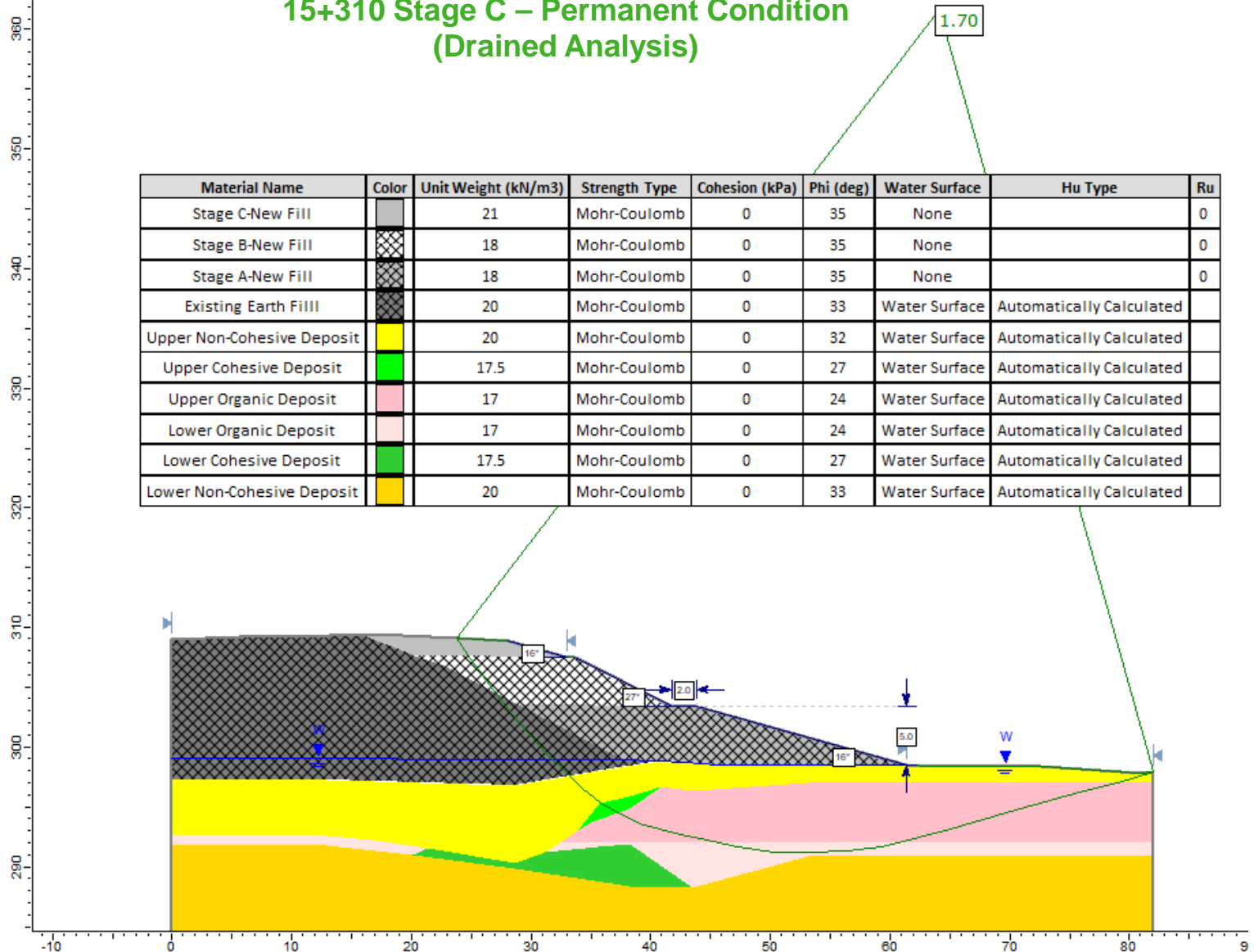
Material Name	Color	Unit Weight (kN/m ³)	Strength Type	Cohesion (kPa)	Phi (deg)	Water Surface	Hu Type	Ru	B-Bar	Mat. Weight Causes Excess Pore Pressure
Stage C-New Fill		21	Mohr-Coulomb	0	35	None		0	0	Yes
Stage B-New Fill		18	Mohr-Coulomb	0	35	None		0	0	Yes
Stage A-New Fill		18	Mohr-Coulomb	0	35	None		0	0	Yes
Existing Earth Fill		20	Mohr-Coulomb	0	33	Water Surface	Automatically Calculated		0	No
Upper Non-Cohesive Deposit		20	Mohr-Coulomb	0	32	Water Surface	Automatically Calculated		0	No
Upper Cohesive Deposit		17.5	Mohr-Coulomb	1	27	Water Surface	Automatically Calculated		0.7	No
Upper Organic Deposit		17	Mohr-Coulomb	0	24	Water Surface	Automatically Calculated		0.85	No
Lower Organic Deposit		17	Mohr-Coulomb	0	24	Water Surface	Automatically Calculated		0.85	No
Lower Cohesive Deposit		17.5	Mohr-Coulomb	0	27	Water Surface	Automatically Calculated		1	No
Upper Organic Deposit-25%Bbar		17	Mohr-Coulomb	0	24	Water Surface	Automatically Calculated		0.21	No
Lower Organic Deposit-25%Bbar		17	Mohr-Coulomb	0	24	Water Surface	Automatically Calculated		0.21	No
Lower Cohesive Deposit-25%Bbar		20	Mohr-Coulomb	0	27	None		0	0.25	No
Lower Non-Cohesive Deposit		20	Mohr-Coulomb	0	33	Water Surface	Automatically Calculated		0	No



Global Stability Analysis

Figure H-4

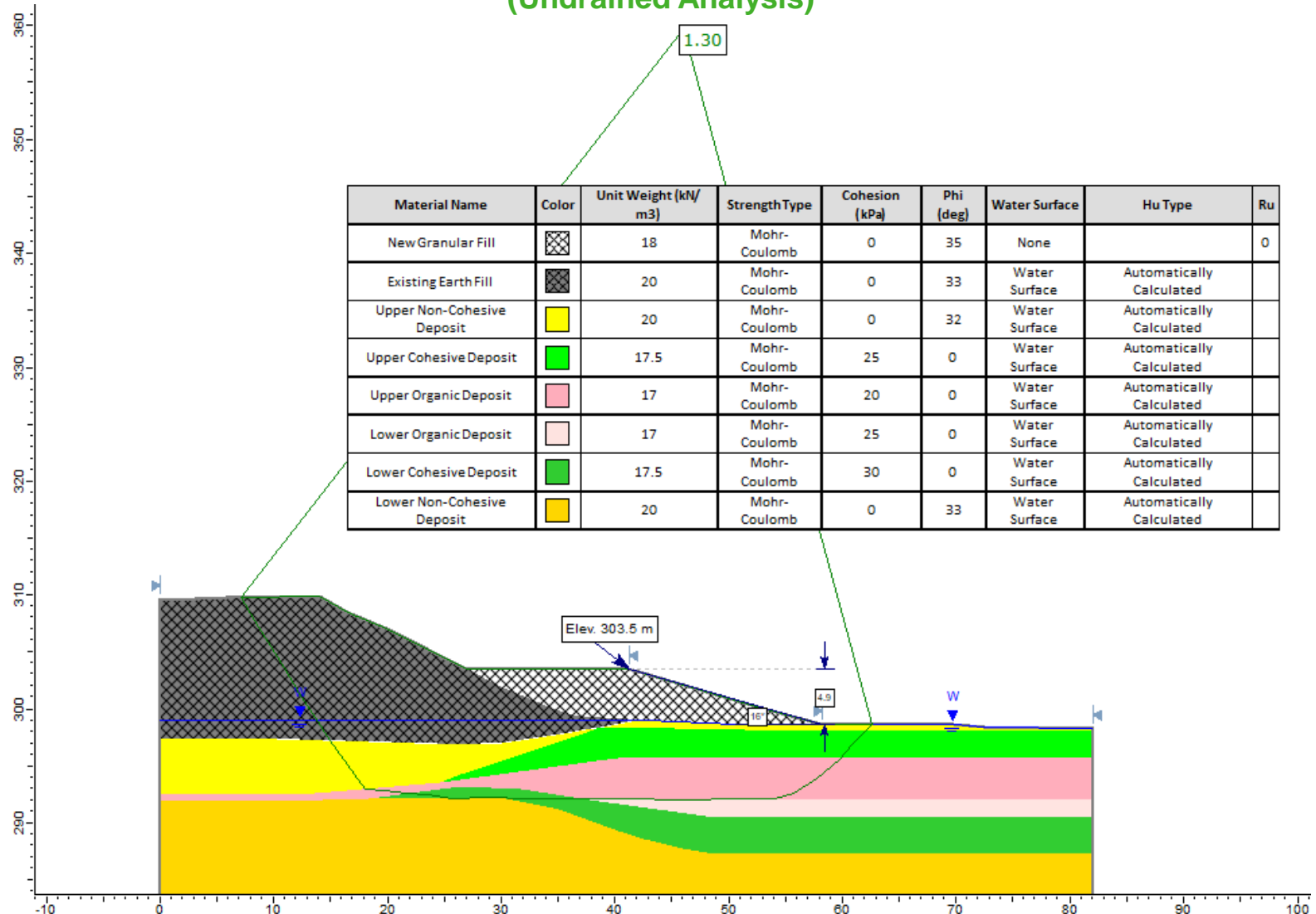
15+310 Stage C – Permanent Condition (Drained Analysis)



Global Stability Analysis

Figure H-5

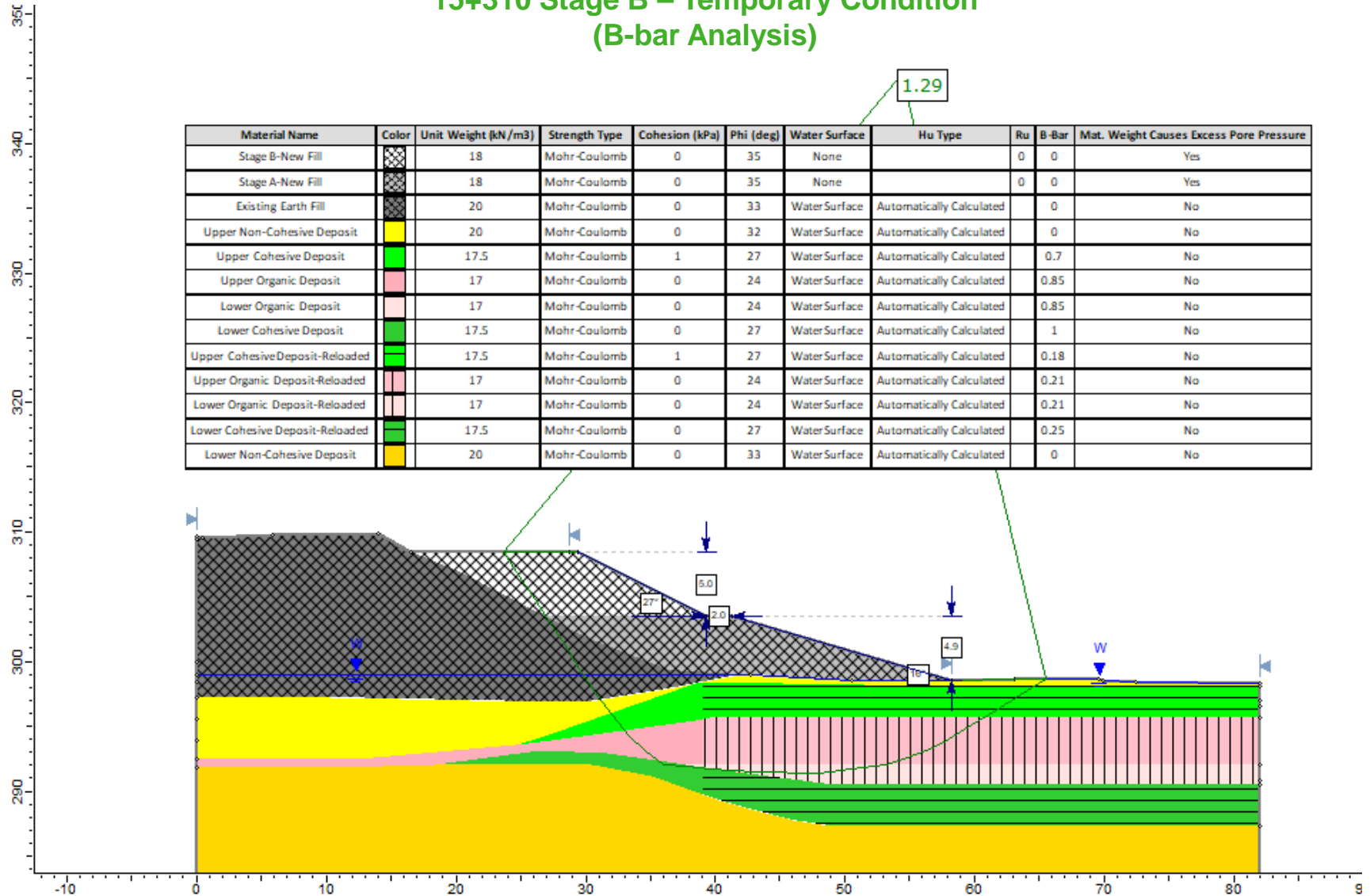
15+310 Stage A – Temporary Condition (Undrained Analysis)



Global Stability Analysis

Figure H-6

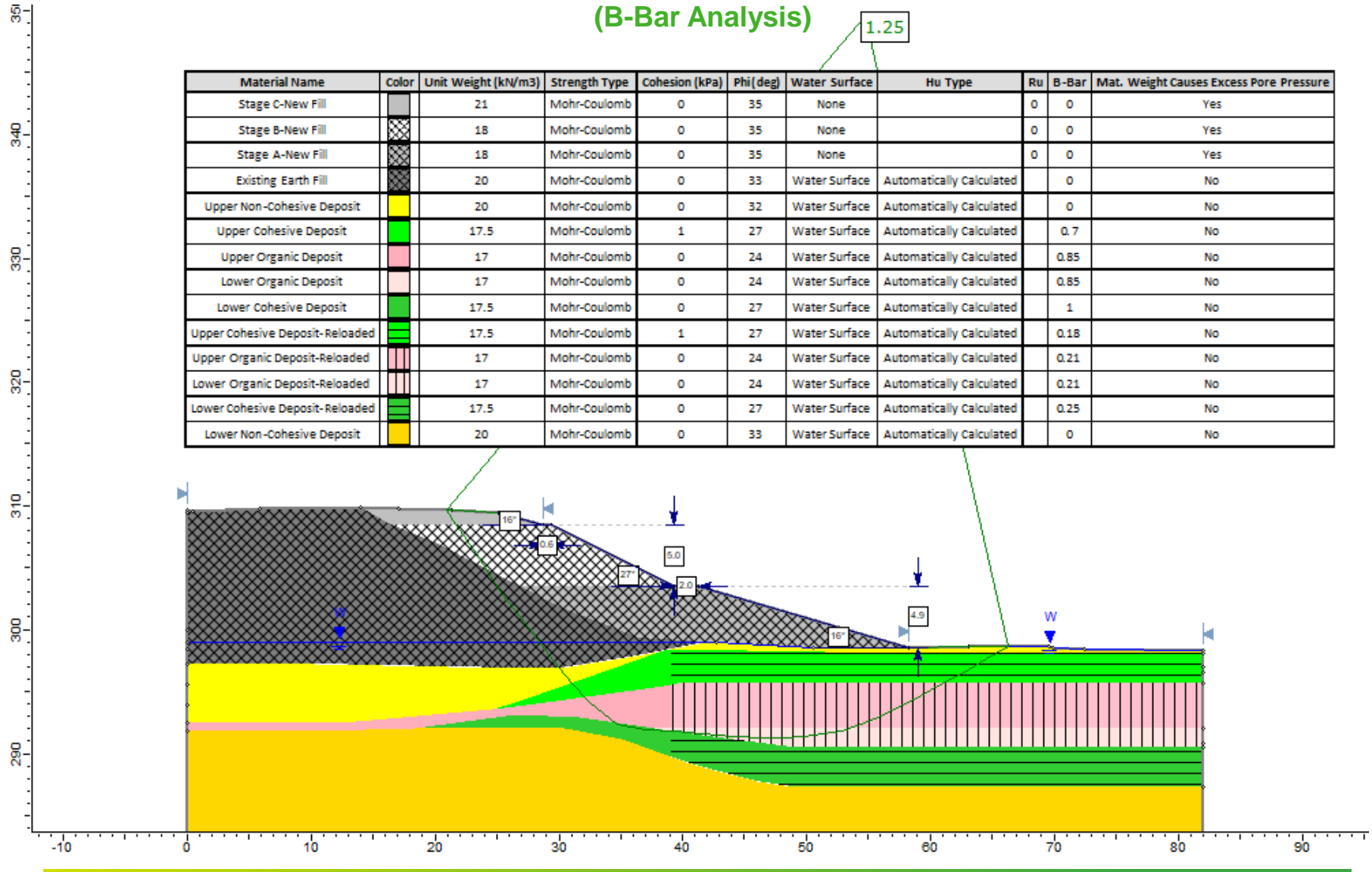
15+310 Stage B – Temporary Condition (B-bar Analysis)



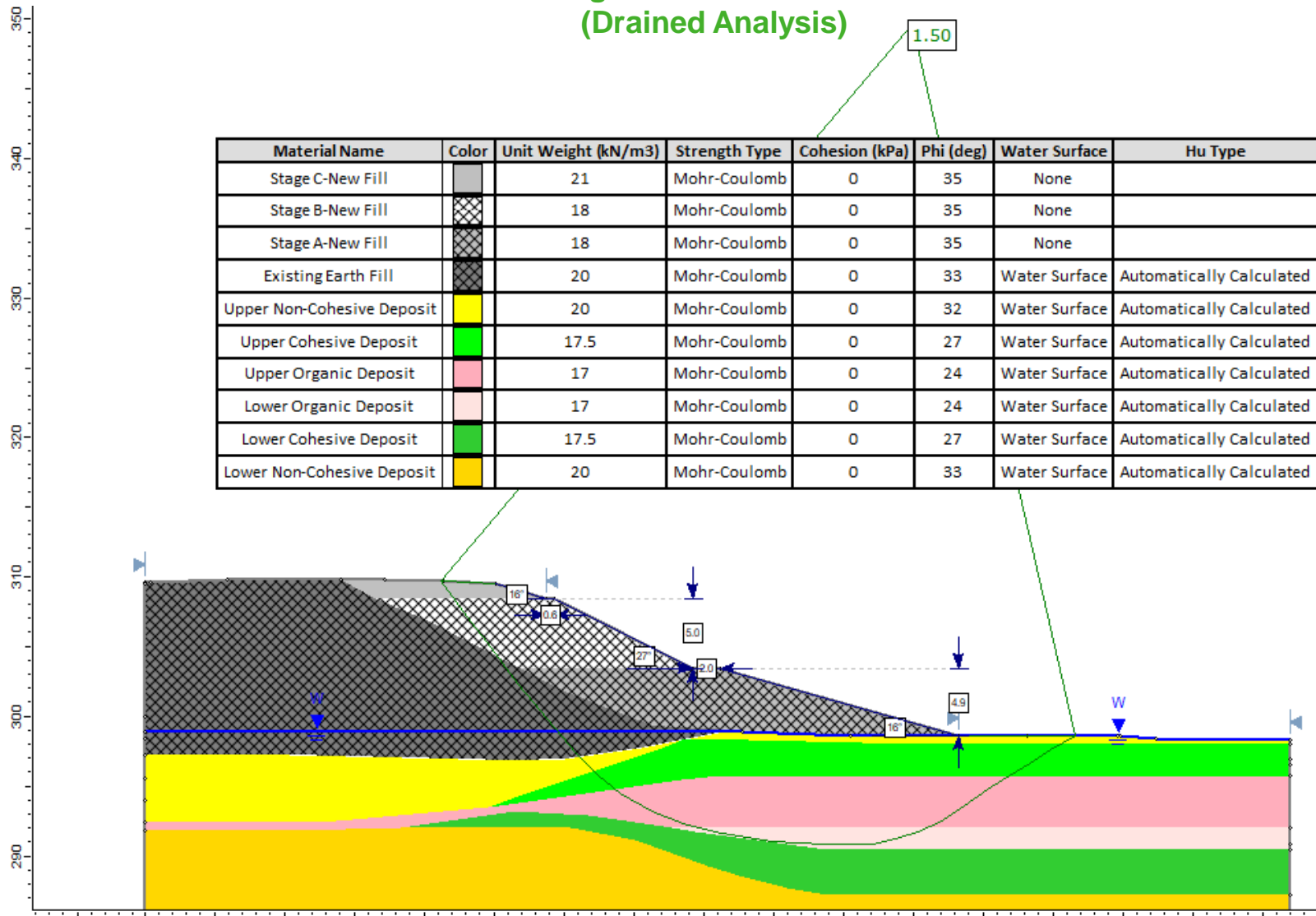
Global Stability Analysis

Figure H-7

15+310 Stage C – Temporary Condition (B-Bar Analysis)



15+310 Stage C – Permanent Condition (Drained Analysis)

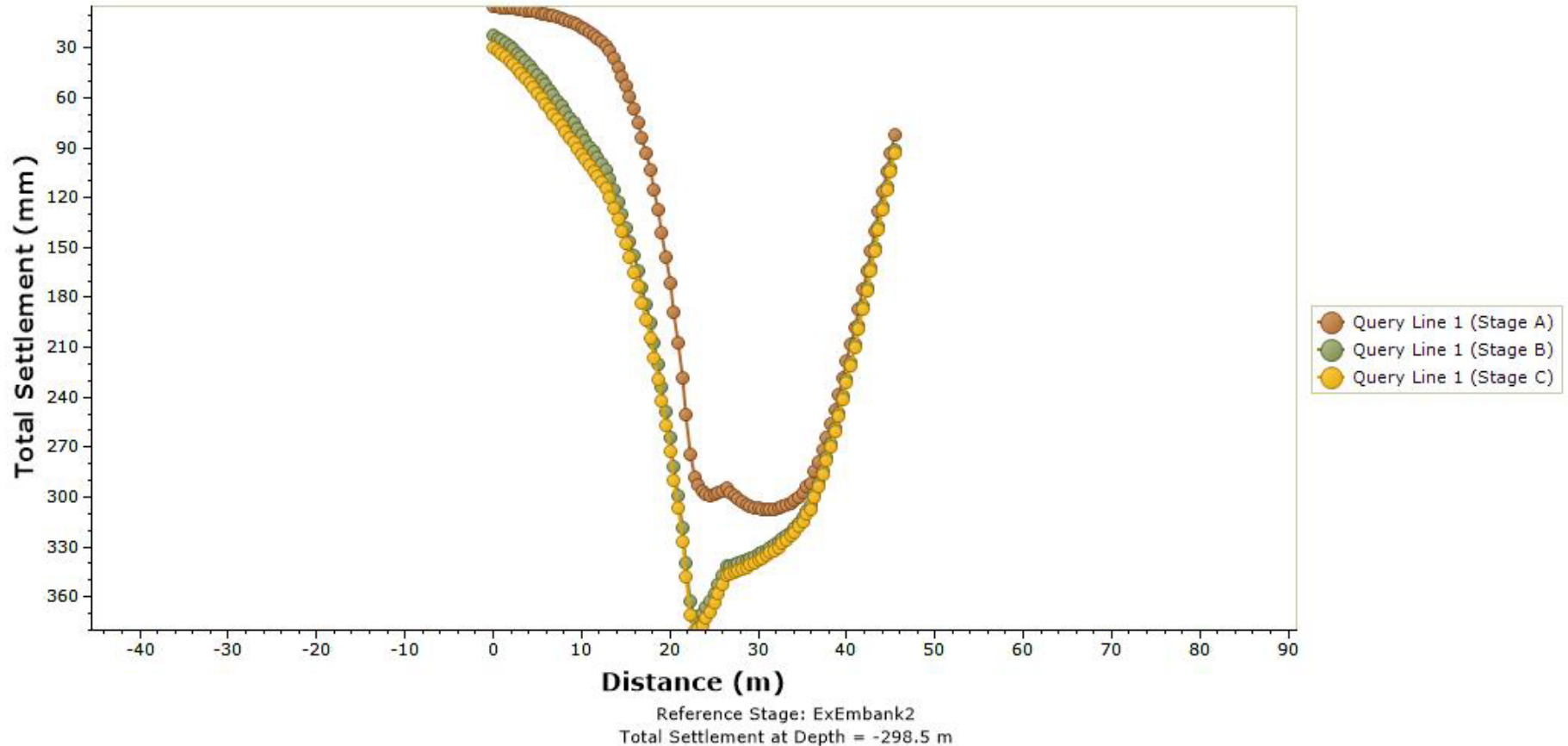


APPENDIX I

Settlement Analysis Results

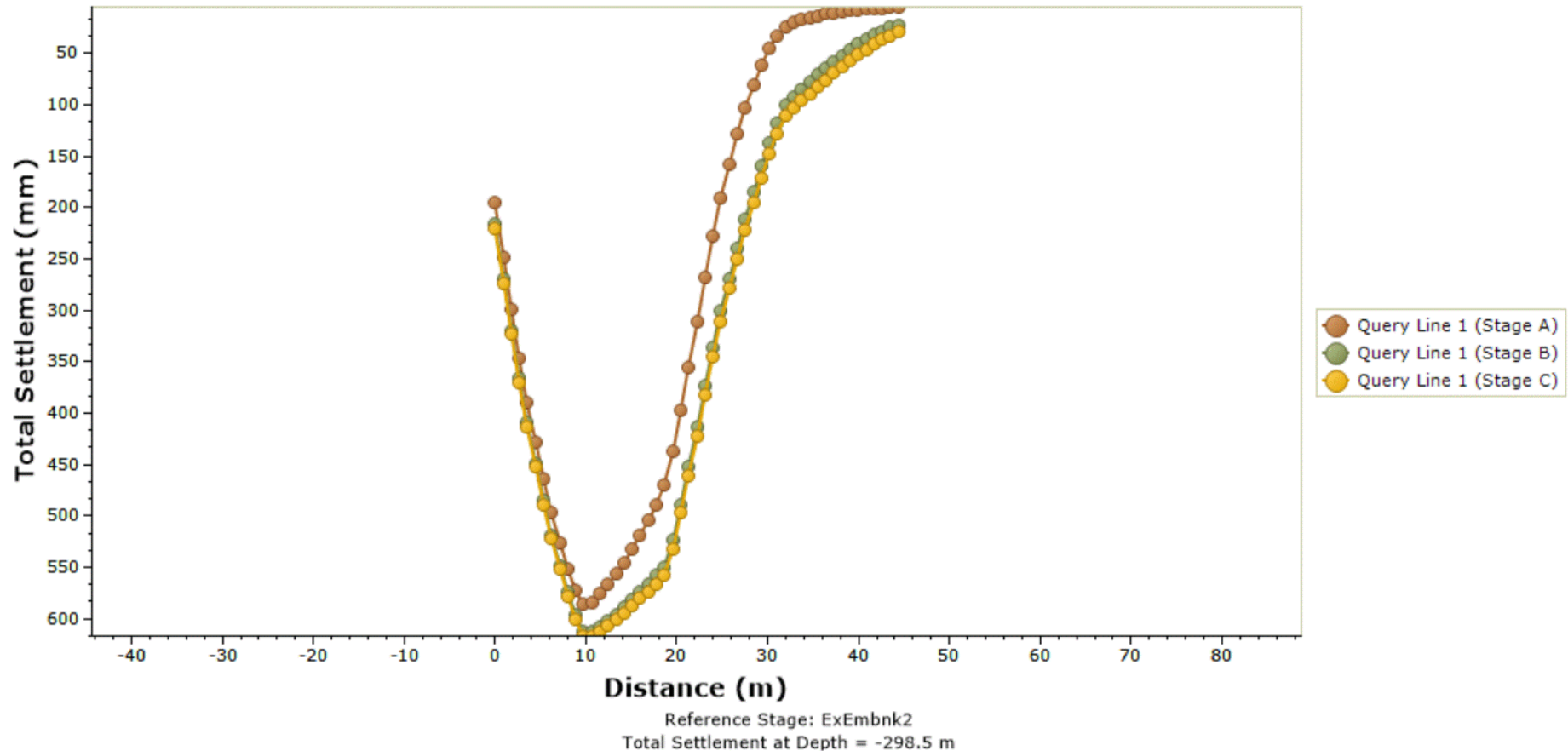
Total Settlement Station 15+290

Distance vs. Total Settlement



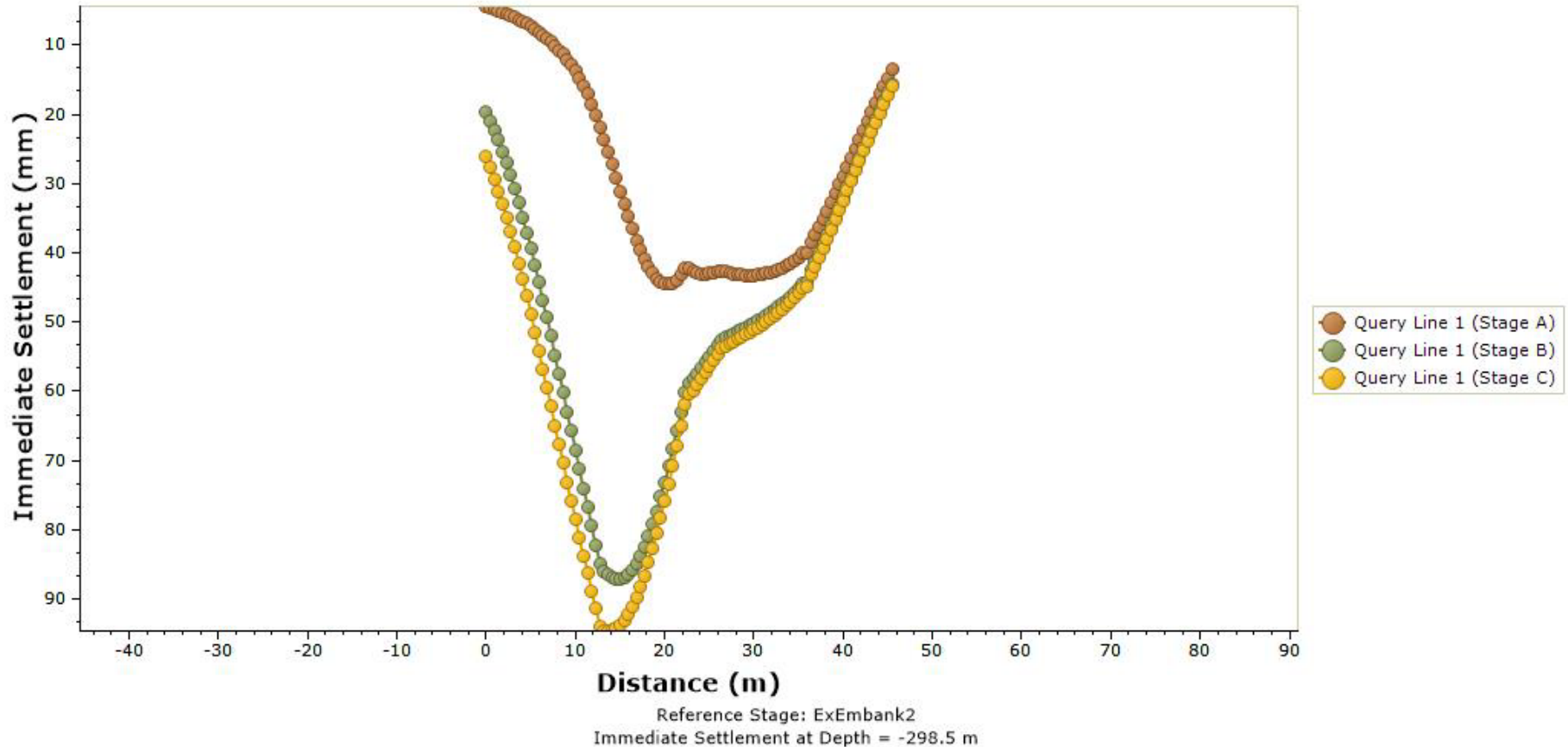
Total Settlement
Station 15+310

Distance vs. Total Settlement

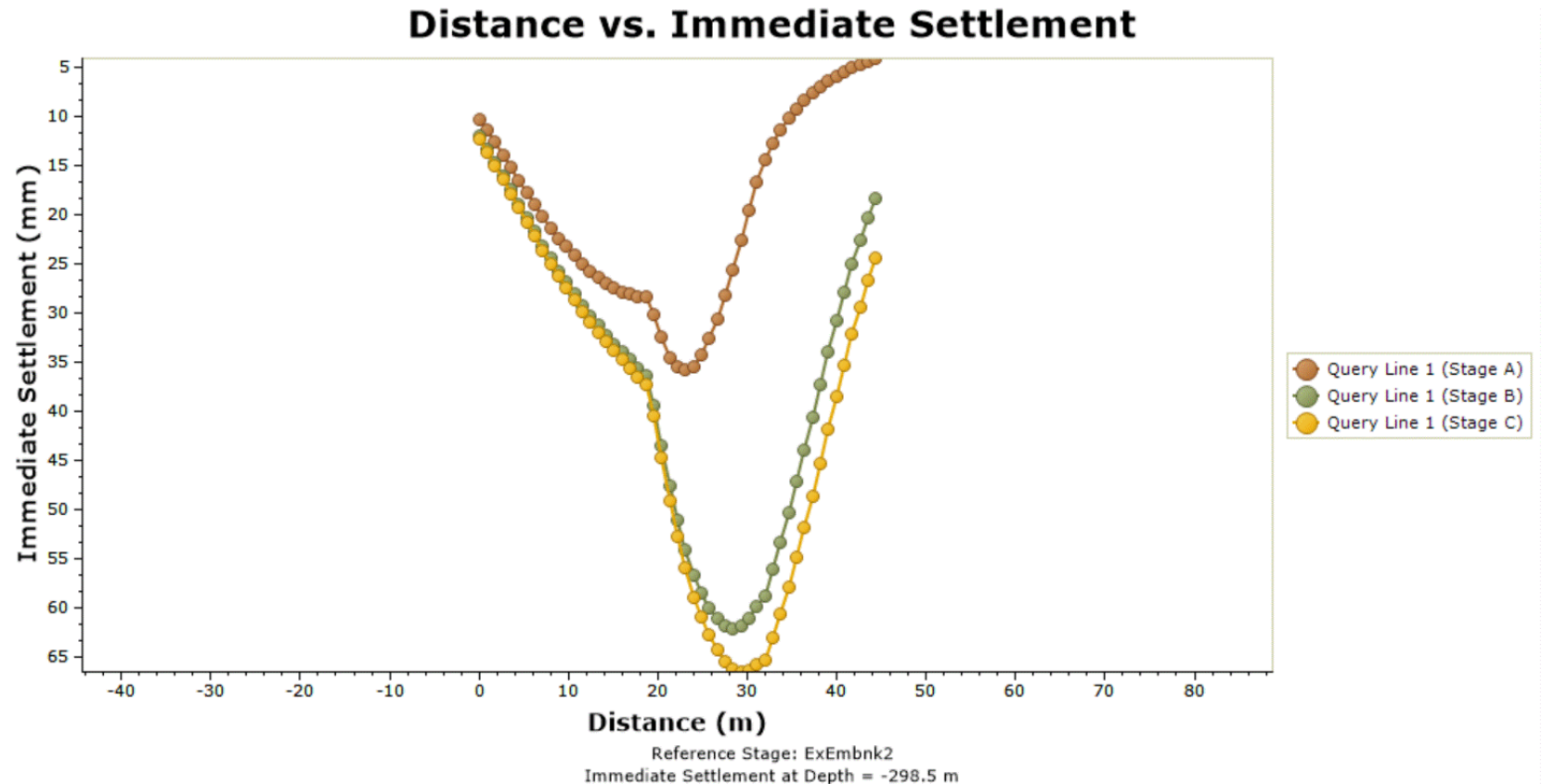


Immediate Settlement Station 15+290

Distance vs. Immediate Settlement

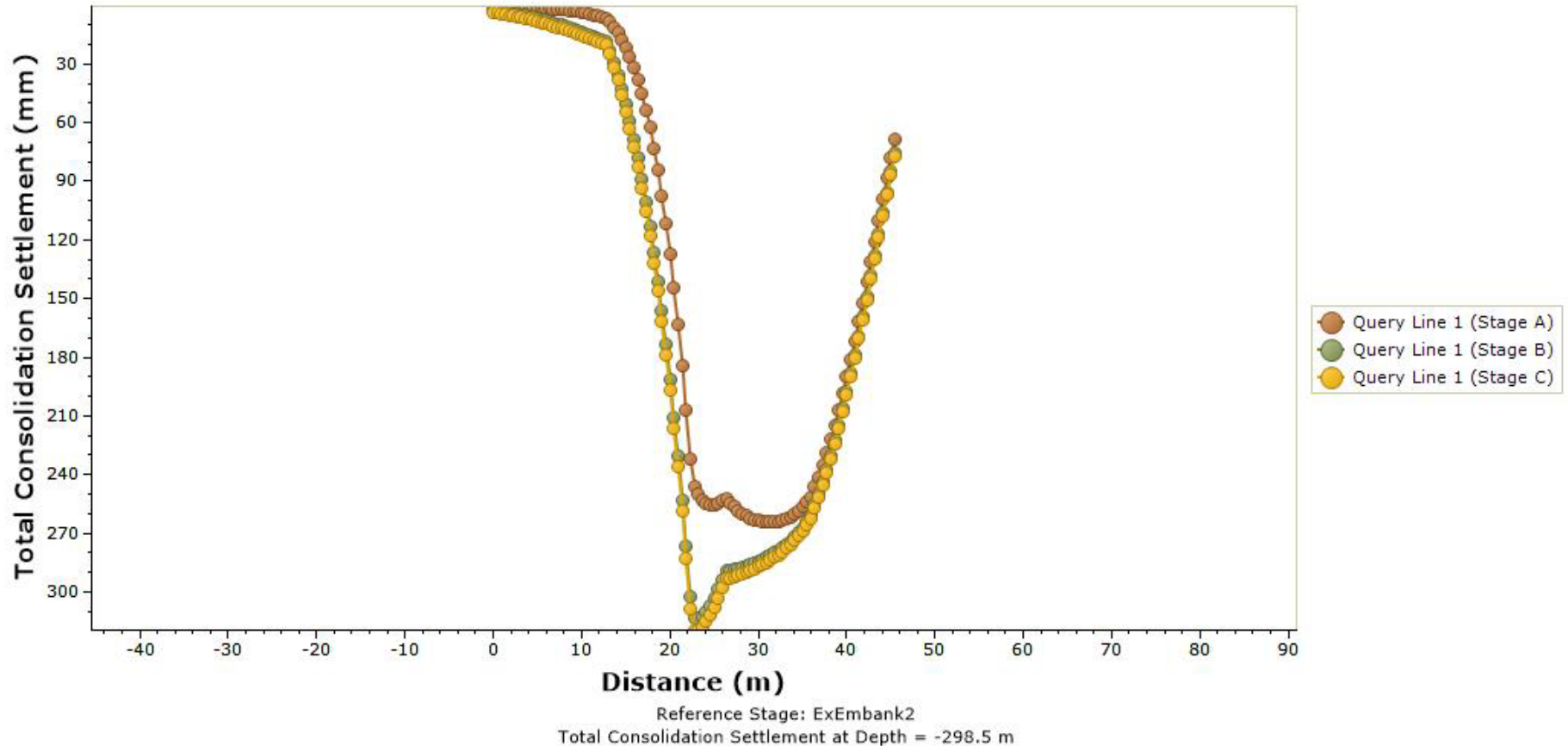


Immediate Settlement Station 15+310



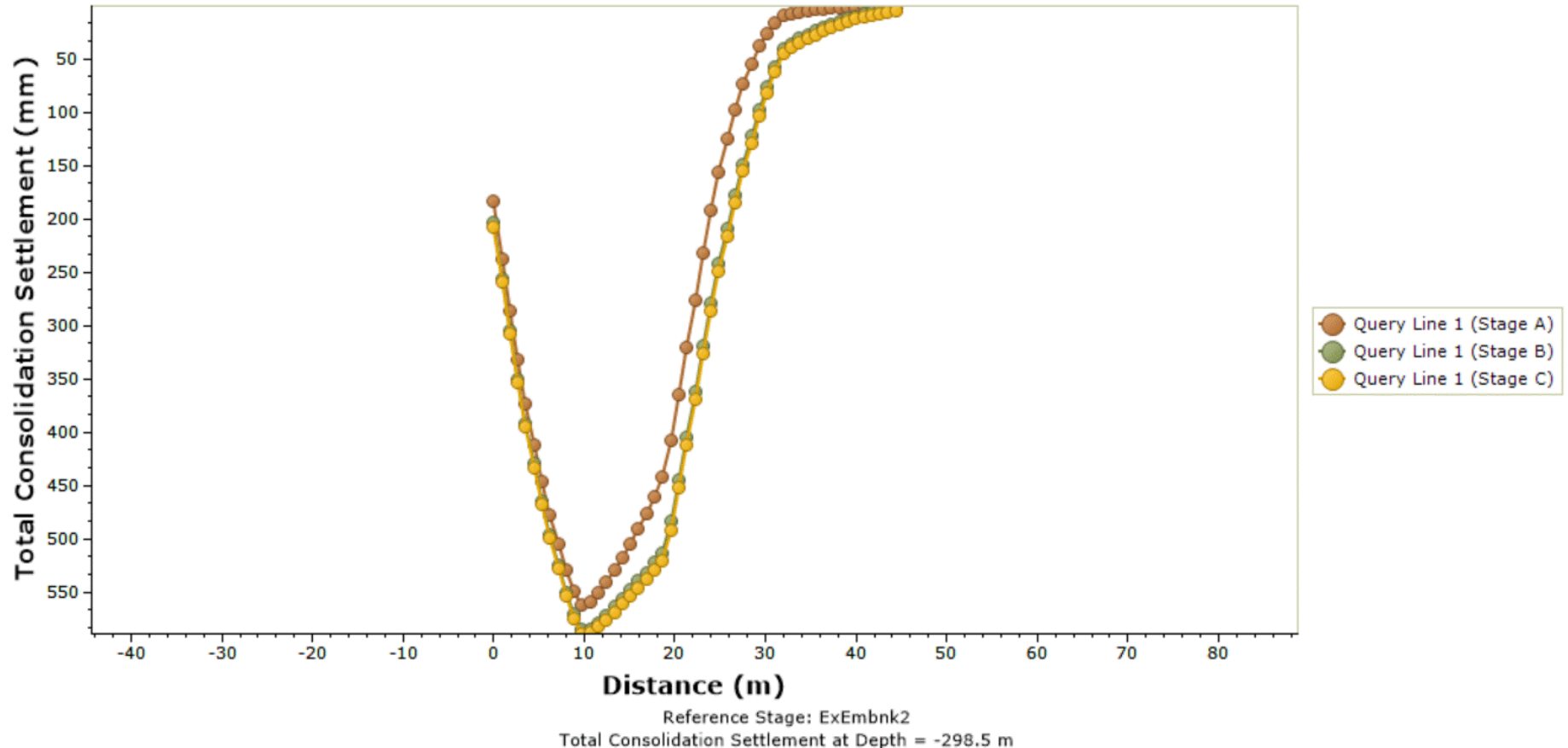
Consolidation Settlement Station 15+290

Distance vs. Total Consolidation Settlement



Consolidation Settlement Station 15+310

Distance vs. Total Consolidation Settlement



APPENDIX J

Special Provisions and Operational Constraints

3/8-INCH CHIP STONE - Item No.

Non Standard Special Provision

Amendment to OPSS 314, November 2015

314.02 REFERENCES

Section 314.02 of OPSS 314 is amended by the addition of the following:

Ontario Provincial Standard Specifications Material

OPSS 1010 Aggregates – Base, Subbase, Select Subgrade, and Backfill Material

Ontario Ministry of Transportation Publications

MTO Laboratory Testing Manual:

LS-601	Material Finer than 75 µm Sieve in Mineral Aggregates by Washing
LS-602	Sieve Analysis of Aggregates
LS-618	Resistance of Coarse Aggregate to Degradation by Abrasion in the Micro-Deval Apparatus
LS-619	Resistance of Fine Aggregate to Degradation by Abrasion in the Micro-Deval Apparatus
LS-625	Guidelines for Sampling of Granular Materials
LS-630	Determination of Amount of Contamination of Coarse Aggregates
LS-631	Qualitative Determination of Presence of Plastic Fines in Aggregates
LS-706	Moisture-Density Relationship of Soils Using 2.5 kg Rammer and 305 mm Drop

MTO Forms:

PH-D-10	Aggregate Sample Data Sheet
PH-D-1Chip	Chip Gradation Computation Acceptance & Payment Adjustment Sheet

314.03 DEFINITIONS

314.03 is amended by deleting the first sentence and replacing it with the following:

For the purpose of this specification, the definitions in OPSS 1010 and the following definitions apply:

314.04 DESIGN AND SUBMISSION REQUIREMENTS

314.04.01 Submission Requirements

The Contractor shall submit details of the proposed 3/8-Inch Chip Stone aggregates (which may also be referred to by suppliers as High Performance Bedding – HPB, or High Performance Chip Stone) to the Contract Administrator, a minimum of fifteen (15) working days prior to placement of the 3/8-Inch Chip Stone aggregates.

The submission shall include the following details:

- a) Source(s) of the 3/8-Inch Chip Stone aggregates.
- b) Test results showing complete conformance with the production and physical property requirements of this specification. Only aggregate sample test data obtained from stockpiled material to be used in the Work and tested within the past 12 months shall be submitted.

The 3/8-Inch Chip Stone aggregates shall not be placed within the Contract limits until the Contract Administrator provides written confirmation that the aggregates meet the requirements as specified in the Contract Documents. Within 4 Business Days following the delivery of all required documentation by the Contractor, the Contract Administrator shall provide the Contractor with the above confirmation or advise the Contractor of any requirements that have not been met.

314.05 MATERIALS

Section 314.05 of OPSS 314 is amended by the addition of the following subsection:

314.05.04 3/8-Inch Chip Stone

The 3/8-Inch Chip Stone aggregates shall be according to OPSS 1001, unless otherwise specified in this specification, and shall conform to the requirements of Table 1 and Table 2 when tested according to the test methods identified herein.

The 3/8-Inch Chip Stone aggregates shall be produced from a quarry or talus. The aggregates shall be clean, hard, durable particles and shall be produced from material free of earth, humus, clay coatings, and clay lumps or fragments of any size or shape. Recycled materials shall not be permitted.

314.07 CONSTRUCTION

314.07.01 Subbase, Base, Surface, and Selected Subgrade

Clause 314.07.01 of OPSS 314 is amended by the addition of the following clause:

314.07.01.01 3/8-Inch Chip Stone

The Highway 400 northbound lane embankment widening between Stations 15+260 and 15+360 shall be constructed using 3/8-Inch Chip Stone aggregates as shown elsewhere in the Contract Documents. The placement of the aggregates shall be completed in two separate stages (Stage A and Stage B); the scheduling and other staging requirements related to the staged installation is documented elsewhere in the Contract Documents.

314.07.05 Compaction

314.07.05.02 Compaction Requirements

Clause 314.07.05.02 of OPSS 314 is amended by the addition of the following:

314.07.05.02.05 3/8-Inch Chip Stone – Compaction Acceptance Based on LS-706

The 3/8-Inch Chip Stone aggregate shall be compacted according to OPSS 501 with the following changes and clarifications:.

1. The 3/8-Inch Chip Stone aggregates shall be considered to be a granular material for compaction acceptance purpose; and

2. Target densities shall be established, based on LS-706, according to the last paragraph of the Target Density clause in OPSS 501.
3. Each lift shall be considered as a single lot with four sublots for compaction acceptance purposes;
4. For a lot to be acceptable, all tests shall be at least 100% of the established target density. Otherwise, the QC lot shall be rejected for compaction.

Except as provided under the Modified Layer Compaction Method clause, the 3/8-Inch Chip Stone aggregates shall be placed in uniform lifts without segregation such that the thickness of the compacted layer is not greater than 150 mm. Each lift shall be bladed to a smooth surface according to the required cross-section and maintained until placement of a subsequent lift, when applicable.

314.08 QUALITY ASSURANCE

314.08.01 General

3.14.08.01 is amended by the addition of the following clause and subsections:

The laboratory designated by the Owner shall carry out QA testing for purposes of ensuring that the 3/8-Inch Chip Stone aggregates used in the Work conform to the physical and production requirements of this specification. Individual test results shall be forwarded to the Contractor, as they become available.

314.08.01.01 Sampling of 3/8-Inch Chip Stone

QA samples shall be taken according to the Contract Documents and LS-625 and shall be road samples or delivery samples obtained from the Work at a location determined by the Contract Administrator. When required, the Contractor shall provide a front-end loader to obtain material for QA samples.

New or clean sample bags or containers shall be provided for sampling. Sample bags or containers shall be constructed to prevent the loss of any part of the material or contamination or damage to the contents during shipment. Sample containers shall be securely fastened. Metal or cardboard containers are unacceptable. The sample shall be identified both inside and outside of the sample container. Data to be included with the sample shall be according to MTO form PH-D-10.

In the event that the Contractor is unavailable to take samples, no further material shall be placed in the Work until the required QA samples have been taken. The Contract Administrator shall seal each sample container at the time and place of sampling.

One of the duplicate QA samples shall be randomly selected for testing by the QA laboratory. The QA laboratory shall retain the remaining sample for referee testing purposes.

314.08.01.02 Sample Size

The mass of each QA sample shall be minimum 25kg. When more than 30 kg is required, additional sample containers shall be used.

314.08.02 Acceptance

314.08.02 is amended by the addition of the following subsections:

314.08.02.01 Acceptance of Physical Properties for 3/8-Inch Chip Stone

At least one set of duplicate QA samples to be used in the Work shall be randomly sampled from lots of 25,000 tonnes or part thereof for physical properties. All materials delivered to the Work shall be included within a lot.

The physical properties of a lot of aggregates shall be deemed to be acceptable if all physical property test results for the sample representing that lot meet all physical property requirement in Table 1.

If a tested sample of aggregates representing a lot does not meet all of the requirements shown in Table 1, then a reduced price payment of 20% of the tender price shall be given for that lot, as long as the lot is not rejectable and the applicable test results for that sample:

- a) do not exceed the requirement for LS-618 by more than 10% of the specified value.
- b) do not identify plastic fines within the material when determined according to LS-631, and acceptance test results for LS-602 are not subject to a payment adjustment on the 75µm sieve.
- c) meet all other physical property requirements of this specification.

Should the test results for any sample of aggregates representing a lot not meet the requirements listed above, then all of the aggregates within that lot shall be considered rejectable and any of those aggregates used in the Work shall be removed at no cost to the Owner.

Irrespective of the negotiation of a reduced price payment, the warranty provisions of the Contract Documents shall apply.

314.08.02.02 Acceptance of Production Properties for 3/8-Inch Chip Stone

All lots for production properties shall be divided into four sublots of approximately equal tonnage and one duplicate QA sample shall be randomly obtained from each subplot.

The Contract Administrator shall estimate the quantities of the aggregates obtained from each different source or process. For each of those individual sources or processes, the Contract Administrator shall identify the number and size of each lot to be sampled and tested. A lot size is generally 4,000 tonnes. When the quantity of the aggregate is less than 2,000 tonnes, that quantity shall be added to the previous lot.

If any circumstances, such as the closure of the construction season or changes in production, result in a lot not being completed, then the Contract Administrator shall be notified prior to the first sample being taken within that lot, in order for the Contract Administrator to adjust the subplot sizes to accommodate the reduced quantity. If such notification is not given in time, then acceptance shall be based on the sampled sublots for the incomplete lot. All lots shall be deemed to be complete at the end of each calendar year.

The QA laboratory shall conduct sieve analysis according to LS-602 and determine test results for each sieve designation shown in Table 2.

Test results from each subplot within a lot shall be combined to determine the mean of the lot for each test. All the means and ranges for test results carried out according to LS-602 shall be computed to one decimal place and reported on PH-D-1Chip MTO form by the Contract Administrator.

The acceptability of a lot based on LS-602 may result in payment at full price, payment at a reduced price, or rejection.

A complete or incomplete lot shall be deemed to meet the applicable requirements for LS-602, if the mean of the test results for that lot is within the limits shown in Table 2 and the range of the test results for that lot is within the limits shown in Table 4.

Lots that are subject to a total payment adjustment factor of more than 25% in respect of lot mean and range are deemed to be rejected and shall be removed from the Work at no cost to the Owner.

When a complete or incomplete lot does not meet the requirements of LS-602, and the lot is not subject to removal, then at the request of the Contractor, an adjusted payment calculated according to the following formula shall be allowed in lieu of removal:

$$\begin{aligned} \text{PAYMENT REDUCTION} &= \text{lot quantity (tonnes)} \times \text{item price (\$/tonne)} \times \text{payment adjustment factor (\%)} \\ &= \sum_{k=1}^n \text{affected quantity (m}^3\text{)}_k \times \text{item price (\$/m}^3\text{)}_k \times \text{payment adjustment factor (\%)}_k \\ &= \sum_{k=1}^n \text{affected quantity (m}^3\text{)}_k \times \text{item price (\$/m}^3\text{)}_k \times \text{payment adjustment factor (\%)}_k \\ &= \sum_{k=1}^n \text{affected quantity (m}^3\text{)}_k \times \text{item price (\$/m}^3\text{)}_k \times \text{payment adjustment factor (\%)}_k \text{ Where:} \end{aligned}$$

The lot quantity shall be expressed in tonnes as determined according to Table 3, and the item price shall be the contract price for the 3/8-Inch Chip Stone item with the tender quantity in tonnes.

The payment adjustment factor, in percent, shall be equal to the sum of the adjustment points determined as follows:

- a) Adjustment points shall be applied for each 0.1% that the mean gradation falls outside the gradation specification limits for each sieve, according to Table 4.
- b) 0.1 adjustment points shall be applied for each 0.1% that the range exceeds the maximum acceptable range for each sieve, according to Table 4.

The reduced price payment for the lot given above shall be in addition to any payment reduction determined according to the Acceptance of Physical Properties for 3/8-Inch Chip Stone clause.

Irrespective of the negotiation of a reduced price payment, the warranty provisions of the Contract Documents shall apply.

314.08.03 Referee Testing

The Contractor may invoke referee testing for one or more attributes by submitting a written request to the Contract Administrator within 5 Business Days following notification that the lot does not meet the requirements of this specification.

Referee testing shall be carried out as specified herein and elsewhere in the Contract Documents.

The retained duplicate QA samples for all sublots shall be used for referee testing of the lot.

All referee test results shall replace the respective QA tests for acceptance of the applicable lot and shall be binding on both the Owner and the Contractor.

If a lot is not accepted at full payment based on the referee test results, then the Contractor shall be responsible for the cost of the referee testing of that lot, including the cost of transporting the samples to the

referee laboratory at the rates specified elsewhere in the Contract Documents. In all other cases, the Owner shall bear the cost of the referee testing of that lot.

314.09 MEASUREMENT FOR PAYMENT

314.09.01 Actual Measurement

Clause 314.09.01.01 of OPSS 314 is amended by the addition of the following:

314.09.01.01 3/8-Inch Chip Stone

314.10 BASIS OF PAYMENT

Subsection 314.10.01 of OPSS 314 is amended by the addition of the following:

314.10.01 3/8-Inch Chip Stone - Item

TABLE 1
Physical Property Requirements for 3/8-Inch Chip Stone

MTO Test Number	Laboratory Test	Acceptance Limit
LS-618	Micro-Deval Abrasion (Coarse Aggregate), % maximum loss	30 (Note 1)
LS-619	Micro-Deval Abrasion (Fine Aggregate), % maximum loss	35
LS-630	Amount of Contamination	Note 2
LS-631	Plastic Fines	NP (Non-Plastic)
LS-706	Standard Proctor (5 points), kg/m ³ , maximum	1800
Note: 1. LS-618 shall be waived if the aggregate has more than 80% passing the 4.75 mm sieve. 2. Maximum 1.0% by mass of any combination of wood, clay brick, gypsum, gypsum wall board, or plaster.		

TABLE 2
Production Requirements for 3/8-Inch Chip Stone

MTO Sieve Designation	Sieve Analysis, % Passing
13.2 mm	100

MTO Sieve Designation	Sieve Analysis, % Passing
9.5 mm	95.0 – 100
4.75 mm	65.0 – 90.0
1.18 mm	0 – 20.0
75 µm	0 – 3.0

TABLE 3
Lot Quantity Determinations for Adjusted Payments

Item	Road or Delivery Samples
Items having the tender quantity in tonnes.	The quantity measured for payment by weighing.
All other items.	The weighed quantity when available; otherwise the theoretical quantity calculated by the Contract Administrator using a conversion factor of 1.65 tonnes per cubic metre.

TABLE 4
Adjustment Points and Range Requirements

MTO Sieve Designation	Adjustment Points Per 0.1% Deviation from Specified Limit	Maximum Acceptable Range
13.2 mm	0.1	1.0
9.5 mm	0.1	-
4.75 mm	Excess Passing 0.5 / Insufficient Passing 0.2	22.0
1.18 mm	0.1	18.0
75 µm	1.0	5.0

OPERATIONAL CONSTRAINT – Staged Embankment Construction and Preloading Between Stations 15+260 and 15+360

Special Provision

This Special Provision addresses staged embankment construction and preloading requirements for the widening of northbound Highway 400 between Stations 15+260 and 15+360. The Contractor shall construct the eastward embankment widening works in three stages (Stage A, Stage B, and Stage C) as shown elsewhere in the Contract Documents.

Stage A construction shall commence as early as possible following associated property acquisition as stipulated elsewhere in the Contract Documents and all three stages shall be completed prior to implementing Stage 2 of construction in the northbound direction in this area.

The Stage B embankment shall be constructed to the top of the granular subbase course and shall not proceed until approximately 75% dissipation of excess porewater pressures has been achieved following placement of the Stage A embankment, as measured and confirmed in writing by the Owner's Foundation Engineering Specialist (FES). It is estimated that the required dissipation of excess porewater pressures will be achieved 12 months from completion of construction of the Stage A embankment.

Placement of granular base material and hot mix asphalt in Stage C shall not proceed until 4 months following completion of the Stage B embankment, subject to settlement monitoring as measured and confirmed in writing by the FES. Stage C shall be preceded by recompacting the granular subbase.

The Contractor shall confirm that the top of embankment fill placed at the end of Stages A and B are within 150 mm of the design elevations, with surveyed elevations provided to the Contract Administrator within five (5) working days of placement of the preload. The Contractor shall keep records of the thickness of each layer of fill placed and the timing of such placement and provide these records to the Contract Administrator within five (5) working days of reaching the top of each layer.

The Contractor shall provide a minimum of one month's notice to the Contract Administrator and FES prior to commencing any of the above-noted operations in this area. The Contractor shall coordinate with and provide access to the Contract Administrator and FES to carry out settlement monitoring throughout the construction of the eastward widening in this area.

SUPPLY AND INSTALLATION OF EMBANKMENT MONITORING EQUIPMENT – Item No.

Special Provision

1.0 SCOPE

This Special Provision covers the requirements for the supply, installation, calibration and, upon completion of the monitoring program (by others), removal/decommissioning of geotechnical monitoring Equipment and instrumentation (settlement plates, vibrating wire piezometers, and benchmarks) for the eastward widening of Highway 400 between Stations 15+260 to 15+360. The geotechnical monitoring Equipment and instrumentation will be utilized by the Owner's Foundation Engineering Specialist (FES) to carry out an embankment monitoring program of the instrumentation and provide the monitoring measurements and recommendations to the Contract Administrator.

To satisfy the requirements of this Special Provision, the Contractor shall retain a Foundation Engineering consultant registered in MTO's Consultant Registry, Appraisal and Qualifications System (RAQS) for "Foundations Engineering Category: Geotechnical (Structures and Embankments) – Medium Complexity" to do the work.

1.1 General Scope

This Special Provision contains the requirements for the supply and installation of the following geotechnical monitoring instrumentation:

- Temporary Benchmarks (TBM);
- Vibrating Wire Piezometers (VWP), and
- Settlement Plates (SP).

1.2 Purpose

The purpose of the VWPs and SPs is to monitor the progress of porewater pressure dissipation and settlement during the staged construction of the eastward widening of Highway 400 between Stations 15+260 and 15+360. The duration of the preloading period for each stage of the embankment construction will be controlled by the instrumentation readings, as specified elsewhere in the Contract Documents. The preloaded staged embankments shall remain undisturbed until such time as the monitoring shall indicate that a sufficient degree of porewater pressure dissipation (for Stage A) or settlement (for Stage B) has been achieved.

The purpose of the TBMs is to provide non-settling references for the surveying of the monitoring instruments.

2.0 REFERENCES

2.1 General

When the Contract Documents indicate that provincial oriented specifications are to be used and there is a provincial oriented specification of the same number as those listed below, references within this specification to an OPSS shall be deemed to mean OPSS.PROV, unless use of a municipal oriented specification is specified in the Contract Documents. When there is not a corresponding provincial-oriented specification, the

references below shall be considered to be the OPSS listed, unless use of a municipal oriented specification is specified in the Contract Documents.

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

Ontario Provincial Standard Specifications, Construction

OPSS 905 Steel Reinforcement for Concrete

Ontario Provincial Standards Specifications, Material

OPSS 1010 Aggregates – Base, Subbase, Select Subgrade and Backfill Material
OPSS 1350 Concrete – Materials and Production
OPSS 1250 Clay Seal
OPSS 1301 Cementing Materials
OPSS 1801 Corrugated Steel Pile (CSP) Products

Ontario Water Resources Act RRO 1990:

Regulation 903 Wells

2.2 Subsurface Conditions

The subsurface conditions at the site are described in the following Foundation Investigation Report for this Contract.

Foundation Investigation Report – High Fill Embankment from Station 15+260 to 15+360, Highway 400 Widening from King Road to Lloydtown-Aurora Road, Regional Municipality of York, MTO G.W.P. 2835-02-00.

3.0 DEFINITIONS

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

Contractor means the Contractor and their Geotechnical Consultant.

Equal shall be understood to indicate that the equal product is the same or better than the specified product in function, performance, reliability, quality and general configuration.

Geotechnical Engineering Consultant means a consultant with MTO classification of “Geotechnical (Structures and Embankments) - High Complexity”, to undertake the supply and installation of geotechnical instruments.

Monitoring Program means the monitoring readings conducted by the Owner’s Foundation Engineering Specialist

Settlement Plate means a plate installed at the defined level with a series of rods attached to a plate for the purposes of settlement monitoring.

Temporary Benchmark means a non-yielding, deep-seated survey reference point.

Vibrating Wire Piezometer means a sensor attached to a cable installed in a borehole for the purposes of measuring pore pressure response.

4.0 DESIGN AND SUBMISSION REQUIREMENTS

4.1 Submission Requirements

4.1.1 Notification

The Contract Administrator and the Owner's Foundation Engineering Specialist shall be notified a minimum of fifteen (15) working days in advance of commencing the installation of instruments.

4.1.2 Installation Methods

The Contractor shall submit details of the proposed installation methods including locations and types of the data acquisition system(s), monitoring enclosure(s), temporary survey benchmarks and installation schedule, to the Contract Administrator, a minimum of fifteen (15) working days before the start of instrument installation.

5.0 MATERIALS

5.1 Materials for Temporary Benchmarks (TBM)

The Contractor shall supply all materials and equipment required for the installation of the benchmarks.

5.1.1 Rod

The Contractor shall supply a steel pipe, Schedule 40, with an outside diameter not less than 25.4 mm, supplied in lengths as required to complete the installation as described in Section 7.2.2.

The top end of each length of TBM rod shall be threaded to receive a cap or to allow for connection of successive lengths of rods. A rounded cap shall be installed at the top of the rod in such a way that a single survey point can be clearly identified and returned to.

5.1.2 Sand

The Contractor shall supply clean, washed sand. The sand shall be Sakcrete washed general-purpose sand – or equal.

5.1.3 Grout

The Contractor shall supply cement-bentonite grout. A suitable grout mix design shall consist of 23 kg of bentonite (OPSS 1205), 143 litres of water and 40 kg of cement (Type GU – OPSS 1301).

5.1.4 Rod Anchor Grout

The Contractor shall supply cement-bentonite grout. A suitable grout mix design shall consist of 14 kg of bentonite (OPSS 1205), 49 litres of water and 40 kg of cement (Type GU – OPSS 1301).

5.1.5 Friction-Reducing Sleeve

The Contractor shall supply a friction-reducing sleeve consisting of Schedule 40 – 50.8 mm (2") outer diameter PVC pipe cut perpendicular to the axis of the pipe.

5.2 Materials for Vibrating Wire Piezometer

The Contractor shall supply all materials and equipment required for the installation of the Vibrating Wire Piezometer.

5.2.1 Vibrating Wire Piezometers (VWPs)

The vibrating wire piezometer sensors shall be:

- Slope Indicator model 52611020 (-5 to 50 psi); or
- RST model VW2100-0.35; or
- Equal.

The VWPs shall be compatible with the Slope Indicator VW Minilogger, Model 52613310, or equal. All VWPs shall be of the same make/supplier.

All VWPs shall be calibrated prior to installation and the calibration data for each piezometer shall be provided to the Contract Administrator.

5.2.2 Signal Cable

The signal cable shall be:

- Slope Indicator model 50613524 cable; or
- RST model EL380004 cable; or
- Equal.

The length of cable for each piezometer shall be carefully estimated from the construction drawings to ensure that there is sufficient length of signal cable for each piezometer to provide enough slack in the borehole and along the trenches until each cable is out of the embankment footprint area where they shall be protected from earthmoving equipment and extended to the monitoring station.

5.2.3 Bentonite

Bentonite to form borehole plugs as required shall be in accordance with OPSS 1205 in pellet form in sufficient quantity.

5.2.4 Filter Sand

Sand for filters around VWP sensors shall be clean washed sand, such as “Sakcrete” washed general-purpose sand; or similar.

5.2.5 Protective Surround

Protective casing as recommended by the manufacturer shall be provided over the length of the piezometers through the rock fill. Sand for additional protection around the casing shall be clean washed sand, such as “Sakcrete” washed general-purpose sand; or similar.

5.2.6 Grout

Grout shall be cement-bentonite mix consisting of 23 kg of bentonite (OPSS 1205), 143 litres of water and 40 kg of cement (Type GU - OPSS 1301).

5.2.7 Trench Burial and Conduit

The signal cable for each VWP shall be buried in a shallow trench and taken out of the embankment footprint area and/or to an area that will not be impacted by construction operations. Conduits to protect the signal cables in the trenches and above ground surface shall consist of Schedule 40 – 75 mm - 3" - steel pipe. A minimum 300 mm protective surround consisting of OPSS.PROV 1010 Granular ‘A’ in accordance OPSS.PROV 1010 shall be placed around the conduit. If appropriate, several signal cables may be housed in a single conduit and laid in a common trench.

5.2.8 Data Acquisition System (Data Logger)

The signal cables from the vibrating wire piezometers shall be connected to the nearest data-logger. A minimum of one (1) data-logger shall be provided at a suitable location at or near this embankment widening area. The data acquisition systems shall be from the same supplier as the VWPs and shall consists of:

- Slope Indicator Model 56701000 (CR1000); or
- RST Model ELGL1200; or
- Equal.

The data-logger shall consist of the following:

- ENC 16/18 Water-proof Enclosure Model 56705020, Model ELF0638, or equal;
- SC32A Serial Interface (with RS232 transfer cable) Model 56704010, Model CS-SC32A, or equal;
- VW Interface Model 56701510 or 56701500, Model CS-AVW200, or equal;
- AM16/32 Multiplexer Model 56702110, Model ELGL2042, or equal;
- A suitable power supply which shall be able to last for a minimum of 18 months, or which can be recharged throughout or beyond this duration;

The data-loggers shall be programmed according to the following:

- Recording Software: VWP data shall be recorded six (6) times a day (i.e. one (1) reading every 4 hours); and,
- Test Software: once this program is transferred to the data-logger, the system shall be able to be tested to confirm readings can be gathered manually at the site.

5.2.9 Wooden Posts

Wooden posts for the support of the data acquisition system enclosures shall be:

- 100 mm x 100 mm (4"x4"), minimum 3 m (10') long pressured treated lumber.

5.3 Settlement Plates (SPs)

The Contractor shall supply all materials and equipment required for the installation of the settlement plates.

5.3.1 Plate

The Contractor shall supply a steel plate with a thickness of at least 6.35 mm. The plate shall be at least 0.5 m wide by 0.5 m long.

5.3.2 Rod

The SP rod shall be fixed to the centre of the plate and perpendicular to the plate. The coupling of the rods shall be such that all sections have the same axis and that no separation or contraction will occur at the couplings.

The top end of each length of rod shall be threaded to receive a cap. A rounded cap shall be installed at the top of the rod in such a way that a single survey point can be clearly identified and returned to.

5.3.3 Sand

The Contractor shall supply clean, washed sand. The sand shall be Sakcrete washed general-purpose sand – or equal.

5.3.4 Grout

The Contractor shall supply cement-bentonite grout. A suitable grout mix design shall consist of 23 kg of bentonite (OPSS 1205), 143 litres of water and 40 kg of cement (Type GU – OPSS 1301).

5.3.5 Friction-Reducing Sleeve

The Contractor shall supply a friction-reducing sleeve consisting of Schedule 40 – 50.8 mm (2") outer diameter PVC pipe cut perpendicular to the axis of the pipe.

5.3.6 Extension of Rod

The SP rods shall be extended upwards as the embankment is constructed so that the top of the rod is always at least 0.3 m but not more than 2 m above the surrounding fill.

5.3.7 Protective Surround

The Contractor shall supply a protective surround for the portion of the rod and friction-reducing sleeve within the backfill/embankment.

The surround shall consist of 300 mm diameter corrugated steel pipe (CSP – OPSS 1801) with the ends cut perpendicular to the axis of the pipe and free of burrs and sharp edges. The space between the CSP and the friction-reducing sleeve shall be filled with medium to coarse sand.

6.0 EQUIPMENT

6.1 Monitoring Equipment Operation and Weather Conditions

All monitoring equipment and associated materials shall be capable of withstanding the range of temperatures possible for their location within the ground or on the surface. The instruments shall be capable of operating within the manufacturer's stated accuracy throughout the temperature range. Monitoring will be conducted year-round throughout the staged preloading by the Owner's Foundation Engineering Specialist.

6.2 Data Logger

The Contractor shall submit a detailed proposal on the setup of the data-logging system (i.e. numbers and locations of the data-logging unit(s)) to the Contract Administrator and the Owner's Foundation Engineering Specialist for review, prior to ordering the data-logger(s).

7.0 CONSTRUCTION

7.1 Monitoring Instrument Installations

7.1.1 Drawings

Reference shall be made to the following drawing that is contained elsewhere in the Contract Documents:

- Monitoring Instrumentation Plan and Details;

7.1.2 Quantities and Locations of Instruments

The quantities and approximate location of instruments are presented in Table 1A and are shown on the Contract Drawings. The final locations shall be "field fit" by the Contractor to take account of any utilities that may be present, construction operations, and safe access conditions, upon discussion and approval with the Owner's Foundation Engineering Specialist.

Table 1A – Instrument Quantities and Locations

Location	Quantities	
	SP	VWP
Proposed Widened Embankment Crest (Stage B)	5	--
Proposed Stage A Embankment Bench	5	3
Proposed Stage A Embankment Slope	5	1
TOTAL:	15	4

7.1.3 Materials and Equipment

The Contractor shall supply all materials and equipment required for the installation of instrumentation unless otherwise noted.

7.1.4 Instrument Locations and Surveying

Prior to the installation of instruments, the Contractor shall coordinate with the Owner's Foundation Engineering Specialist to accurately survey and stake the location of each instrument and obtain a ground elevation at each instrument location.

Surveying to establish the benchmarks and other elevations shall be carried out by a registered surveyor with appropriate equipment. The surveyor shall be retained by the Owner's Foundation Engineering Specialist.

7.1.5 Underground Utilities

The Contractor shall be responsible for locating and protecting all underground utilities prior to drilling boreholes for installing instruments. Any damage to underground utilities caused by the Contractor's work shall be repaired by the Contractor at no cost to the Owner or Contract Administrator.

7.1.6 Marking and Labelling

The location of all benchmarks, VMPs and SPs shall be made clearly visible to nearby traffic before, during and after embankment construction. Marking shall be of sufficient size to be visible from a reversing vehicle and after heavy snow falls, if and where applicable.

Instruments shall be clearly labelled in the field, with each instrument having a unique identifier. The labelling shall remain legible for the entire duration of monitoring.

7.1.7 Protection of Instruments

The Contractor shall adequately protect all instruments such that they are not damaged during construction. Any instrument damaged by the Contractor's work shall be immediately replaced by the Contractor at no cost to the Owner or Contract Administrator.

7.1.10 Boreholes

The Contractor shall notify the Owner's Foundation Engineering Specialist at least three (3) working days prior to drilling boreholes for installation of VMPs; the Owner's Foundation Engineering Specialist may be present on site to observe the borehole drilling. The Contractor shall make a basic stratigraphic log of boreholes as they are being drilled for the installation of monitoring instruments. In situ or laboratory geotechnical testing is not required.

Boreholes shall be advanced using conventional drilling methods, where applicable, and shall be as straight and vertical as practicable.

7.1.11 Installation Program

The instruments shall be installed prior to the commencement of the embankment construction. Table 1B gives a summary of the installation schedule requirements.

Table 1B – Instrument Installation Program

Instrument Type	Instrument Location	Start Installation
TBM	Outside approximate area of soft/compressible soils	Prior to installation of SPs and VWPs.
VWP	Along proposed side slope of Stage A embankment and along proposed embankment bench	After stripping and prior to start of Stage A embankment construction
SP	Along proposed side slope of Stage A embankment and along proposed embankment bench	After stripping and prior to start of Stage A embankment construction.
SP	Along proposed crest of widened embankment	After preloading period of Stage A embankment is complete and prior to commencement of Stage B embankment construction.

7.2 Benchmark Installation

7.2.1 Number and Locations

The minimum number and approximate locations of the benchmarks are to be determined by the Contractor and their Foundation Engineering consultant in conjunction with the Contract Administrator and the Owner's Foundation Engineering Specialist. For bidding purposes assume that two (2) benchmarks are required anchored at 15 m depth, located at the toe of the existing embankment and outside of the approximate area of soft/compressible soils. The number and locations of benchmarks shall be determined in the field to satisfy the following conditions:

- Direct sighting is possible from all instruments to at least one benchmark.
- Each benchmark is located in an area that will not experience a change in loading (due to grade raise or excavation) that could induce settlement or heave in the ground in which the benchmark is installed (i.e. non-settling benchmark).
- Each benchmark is located in such a way to minimize interference with and damage by construction activities.
- The rod anchor elevation shall be adjusted in the field to extend approximately 1 m into soils having Standard Penetration Test 'N' values of greater than 25 blows per 0.3 m of penetration. Reference shall be made to the Foundation Investigation Report for information in order to determine the anchor elevation for each Benchmark location selected.

7.2.2 Installation

The Contractor shall install Benchmarks in accordance with the following:

7.2.2.1 Borehole

The borehole shall be advanced to rod anchor elevations controlled by the Standard Penetration Test "N" values given above, using suitable drilling techniques. The diameter of the borehole shall be sufficient to fit the rod, friction-reducing sleeve and rod anchor. The sides of the borehole shall be stable and the borehole shall be free of drilling mud and debris.

7.2.2.2 Rod

The coupling of the rods shall be such that all sections have the same axis and no separation or contraction will occur at the couplings.

7.2.2.3 Rod Anchor

The rod shall be installed vertically in the borehole with its bottom end resting at the bottom of the borehole. The bottom portion of the rod shall be fixed against the surrounding native soil by grouting the bottom 0.5 m of the borehole to form a concrete/soil anchor.

Once grouting is completed and the rod anchor grout has set, the contractor shall pour clean sand in the lower 0.5 m length of the borehole above the concrete/soil anchor to create a base for the end of the friction reducing sleeve to rest on.

The elevation of the bottom of the rod anchor shall be determined by measuring the length of the rod to the ground surface elevation.

7.2.2.4 Friction-Reducing Sleeve

The friction-reducing sleeve shall be installed over the entire length of the rod above the rod anchor and sand, extending up to ground surface.

7.2.2.5 Installation Details

The elevation, easting and northing of the top of the Benchmark rod shall be surveyed.

7.3 Vibrating Wire Piezometer (VWP) Installation

7.3.1 Number and Locations

The locations of the VWP are shown in the Contract Documents and summarized in Table 1A. The VWPs shall be installed in boreholes prior to construction of the embankment widening. The VWPs shall be installed to a tip elevation at Elevation 292.0 m. Installation of the VWPs shall be as per the manufacturer's recommendations in addition to what is stated or emphasised below.

The VWP signal cables shall be extended to the data-logger enclosure areas through a metal or plastic conduit buried in trenches with protective surround, as specified in Section 5.2.5. The final location of the monitoring enclosure should be determined on-site prior to ordering instruments to ensure there is sufficient cable length(s). Due to the restricted working area, the location of the monitoring enclosure should be determined to avoid construction traffic.

7.3.2 Borehole Installation

The borehole at each VWP location shall be advanced to 300 mm below the lowest tip elevation using suitable drilling techniques. The sides of the borehole shall be stable and the borehole shall be free of drilling mud and debris. A split spoon sample shall be taken at the proposed installation depth to confirm the soil stratum at the VWP tip elevation. The borehole shall be filled with water prior to installation of the VWP tip.

7.3.3 Protective Enclosures for Data Loggers

The data-logger shall be installed in a protective enclosure near each approach embankment to prevent vandalism and prolonged wear-out of the data-loggers against extreme weather. The protective enclosure shall be lockable and weather proofed. The Contractor shall submit a detailed proposal of the protective enclosure (i.e. materials and location(s) etc.) to the Contract Administrator for review, prior to construction/installation.

The Contractor shall ensure access to the protective enclosure at all times, including but not limited to snow clearing in the winter.

7.3.4 Completion of Installation

It is known that the process of installing VWP's can temporarily alter the pore water pressure acting on the piezometer tip. The installation of a VWP shall not be considered to be complete until the pore pressure acting on the piezometer has returned to and stabilized at the value prevailing in the surrounding, unaffected soil mass. The Contractor shall take daily reading of the pore pressures, for the period noted below until the value has stabilized as determined by the Contract Administrator. Stabilization shall be deemed to have occurred:

- When no change in the measured value has occurred over a period of five (5) consecutive days and the measured value is within 10 per cent of the anticipated hydrostatic value; and,
- When the daily rate of change is less than four (4) kPa per day for three (3) consecutive days and the measured value is within 5 per cent of the anticipated hydrostatic value.

The Contractor should be prepared to wait for a period of 10 days to 15 days after completion of installation of the instruments for the baseline readings to stabilize.

7.4 Settlement Plates

7.4.1 General

The Contractor shall install SPs at the locations shown on the Contract Drawings and given in Table 1A. The Contractor shall install SPs as shown on the Contract Drawings and the Typical Installation Detail, in addition to what is stated below.

The Contractor is responsible for preventing damage to the settlement plate during the embankment construction. If the rod is damaged during fill placement, the rods, friction-reducing sleeve and protective surround shall be replaced before resuming the fill placement.

7.4.2 Plate

The settlement plate shall be installed horizontally on the undisturbed native soil or existing embankment fill just below the existing ground surface, following stripping of topsoil. Where the plate is located on the existing embankment side slope, a flat surface shall be created at the plate base using a hand shovel.

7.4.3 Rod

The coupling of the rods shall be such that all sections have the same axis and no separation or contraction will occur at the couplings.

7.4.4 Friction-Reducing Sleeve

The friction-reducing sleeve shall be installed over the entire length of the rod that is below ground and within the embankment fill except that the cap on top of the SP rod shall extend 25 mm above the top of the friction sleeve at all times.

7.4.5 Extension of Rod

The SP rods shall be extended upwards as the embankment widening is constructed so that the top of the rod is always at least 0.3 m, but not more than 2 m above the surrounding fill.

7.4.6 Protective Surround

The CSP, friction-reducing sleeve and sand surround shall be extended concurrent with the rods, where applicable. The SP rod shall be in the centre of the CSP and friction-reducing sleeve. The annulus between the CSP and the friction-reducing sleeve shall be filled with sand to a level not higher than the top of the sleeve.

7.4.7 Installation Details

The elevation, northing and easting of the top of the rod shall be surveyed by the Contractor.

The total distance from the rod anchor to the top of the rod shall be measured and recorded by the Contractor to an accuracy of ± 2 mm or better.

7.5 Reporting

7.5.1 General

The Contractor shall notify the Contract Administrator no later than three (3) working days after the completion of installation of benchmarks, vibrating wire piezometers and settlement plates.

The Contractor shall supply the information outlined in the following sections to the Contract Administrator and the Owner's Foundation Engineering Specialist within three (3) days of completion of installation of each instrument.

7.5.1.1 Temporary Survey Benchmarks

The Contractor shall record and report relevant installation details to the Contract Administrator. These include, but are not limited to:

- TBM Northing and Easting in MTM NAD 83 coordinates;
- Elevation of the rod anchor bottom, rod anchor length, and top of rod in Geodetic datum;
- Date of installation;
- Stratigraphic log of subsurface conditions at the TBMs, including notes on drilling method obstructions it encountered;
- Installation notes/sketches; and,
- Description of TBM (rod), sleeves and rod anchors.

7.5.1.2 Vibrating Wire Piezometers

The Contractor shall record and report relevant installation details to the Contract Administrator. These include, but are not limited to:

- VWP Northings and Eastings in MTM NAD 83 coordinates;
- Elevations of VW sensors in Geodetic datum;
- Dates of installation;
- Stratigraphic log of subsurface conditions, including drilling method notes;
- Installation notes / sketches;
- Model, make and serial numbers of VW sensors, readout unit and signal cable; and,
- Calibration details of VW sensors.

7.5.1.3 Settlement Plates

The Contractor shall record and report relevant installation details to the Contract Administrator. These include, but are not limited to:

- SP Northing and Easting in MTM NAD 83 coordinates;
- Elevation of base of plate and top of rod in Geodetic datum;
- Date of installation;
- Installation notes/sketches; and,
- Description of SP rods, sleeves and plates.

Adjustments in the length of any SP rod shall be coordinated with the Contract Administrator to allow surveying by others of the elevation of the top of the rod immediately before and immediately after adjustment. This surveying is necessary to accurately track the settlement data.

7.6 Monitoring

The Contractor shall meet with the Contract Administrator and the Owner's Foundation Engineering Specialist (responsible for the ongoing monitoring) immediately after installation of the instruments and before the start of embankment construction. At this meeting, the Contractor shall hand over to the Contract Administrator all records pertaining to the installation of the instruments, and all equipment to be supplied by the Contractor, as identified in the item-specific special provisions.

Monitoring by the Owner's Foundation Engineering Specialist for the baseline readings shall commence within seven working days after the hand-over meeting. The monitoring shall continue on a schedule to be determined by the Contract Administrator and the Owner's Foundation Engineering Specialist throughout the construction of the embankment. It is estimated that the monitoring will extend approximately twelve (12) months from completion of Stage A embankment construction, and approximately six (6) months from completion of Stage B embankment construction.

7.7 Decommissioning of Instruments

At the end of the monitoring period, the Contractor shall decommission all the temporary survey benchmarks, any tie-in points, and settlement plates by removing the rod / friction-reducing sleeve to at least 1.5 m below grade by excavating and backfilling with compacted granular fill in accordance with the specifications for fill placement.

At the end of the monitoring period, the Contractor shall decommission the wiring, logging equipment and datalogger station associated with all vibrating wire piezometers.

Decommissioning of instrumentation shall be carried out per the item-specific special provisions and according to the Ontario Water Resources Act, Regulation 903 (as amended).

8.0 QUALITY ASSURANCE – Not Used

9.0 MEASUREMENT FOR PAYMENT – Not Used

10.0 BASIS OF PAYMENT

10.1 Supply and Installation of Embankment Monitoring Equipment - Item

Payment at the Contract price for the above tender item shall be full compensation for all labour, Equipment and Material to do the work, including the supply, installation, calibration and decommissioning of survey benchmarks, settlement plates, and vibrating wire piezometers, as well as performing all required reporting associated with the instrument installation works.



golder.com