



SUPPLEMENTARY REPORT ON

Clay Consolidation Characteristics
County Road 2/34 Approach Embankments
Highway 401, Site No. 31-232
Lancaster, Ontario
W.P. 4013-11-01

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PART A

Foundation Investigation Report
Clay Consolidation Characteristics
County Road 2/34 Approach Embankments
Highway 401, Site 31-232
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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) was previously retained by Dillon Consulting Limited (Dillon) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services associated with replacement of the existing County Road 2/34 Bridge (Site No. 31-232) over Highway 401 which is located in Lancaster, Ontario (WP 4013-11-01) as part of the 23 Structures MEGA 3 project. The results of the foundation investigation for this project were provided in Golder report dated September 2017 titled "*Foundation Investigation and Design, Country Road 2/34 Underpass Replacement, Highway 401, Site 31-232, Lancaster, Ontario, W.P. 4013-11-01*" (Geocres 31G-259). This report should be read in conjunction with the previous Golder report dated September 2017 (Geocres 31G-259).

Based on the previous investigation, the subsurface conditions at the site consist of up to 10 m of sensitive and compressible clay. Analysis results show that, if conventional earth fill or granular fill is used for the new 8 m high embankments (using conventional construction techniques), settlement of the approach embankments will be significant. Ground improvement alternatives are therefore being evaluated to mitigate the anticipated settlement.

The purpose of the current investigation is to provide additional information about the clay subsoil to support the detail design of ground improvement alternatives for the new approach embankments as part of the bridge replacement. This investigation included drilling two boreholes, as well as carrying out in-situ testing, piezocone penetration test (CPT) and advanced laboratory testing on selected soil samples. The advanced laboratory testing consists of standard and long-term consolidation testing, constant rate of strain testing, and bench-scale soil mixing experiments.

The terms of reference for the original scope of work are outlined in the MTO's Request for Proposal (RFP) dated August 2012. In addition, Golder's proposal letter dated October 31, 2017 described the work plan for additional lab testing and engineering services required for detail design.

The work was carried out in accordance with Golder's Quality Control Plan dated December 2012.

2.0 SITE DESCRIPTION AND GEOLOGY

The County Road 2/34 Underpass is located along Highway 401 in Lancaster, Ontario. The existing bridge (Site No. 31-232) is located at about Station 27+600 on Highway 401 (see Key Plan in Drawing 1).

It is understood that the preferred arrangement for detail design is to replace the existing bridge on a new alignment immediately west of the existing, and tying into the existing ramps.

The results of the foundation investigation for this project were provided in the report by Golder, dated September 2017 titled "*Foundation Investigation and Design, County Road 2/34 Underpass Replacement, Highway 401, Site No. 31-232, Lancaster, Ontario, W.P. 4013-11-01*" (Geocres 31G-259). Previous investigations were also carried out for the design of the original/existing bridge. The results of those investigations are contained in the following reports:

- Report on "*Foundation Investigation, Proposed Crossing, Highway No. 401 and Highway No. 2, 1-1/2 Miles South of Lancaster, Township of Charlottenburg, District No. 9, Bridge No. 11, WP 108-59*" (Geocres 31G00-144), by H.G. Acres & Company Limited, dated 1960.
- Report on "*Foundation for Proposed Underpass Bridge, HWY #401 and HWY #2 at Lancaster*" (Geocres No. 31G00-145), by the Department of Highways, Ontario, dated 1955.

The regional geological conditions and description of the existing and future structure are described in recent Golder reports and will not be repeated herein. This report should be read in conjunction with the previous Golder report submitted in September 2017 (Geocres 31G-259). In summary, the existing structure, built in 1963, is to be replaced with a four lane, two-span structure with a shift in the alignment to the west immediately adjacent to the existing alignment. A widening/new embankment will be required about 8 m in height and up to about 20 m in width at the crest to accommodate this shift. The west side slopes of the existing embankments will therefore form a small part of the new embankments.

3.0 INVESTIGATION PROCEDURES

The subsurface investigation for the current scope was carried out between November 10 and 14, 2017. During this time, two piezocone penetration tests (CPT) (numbered 17-1 and 17-2) and two boreholes (numbered 17-11 and 17-12) were advanced at locations shown on Drawing 1. Boreholes 17-11 and 17-12 were located within the proposed south approach embankment. The boreholes were advanced to a depth of about 9.2 m (Elevation 39.0 m) below the existing ground surface in the overburden. Both boreholes were terminated in the clay layer. The boreholes were advanced using 200 mm inside diameter (I.D.) continuous-flight hollow-stem augers on a truck-mounted drill rig (CME 850), supplied and operated by Marathon Drilling Ltd. of Ottawa, Ontario.

Within Borehole 17-11, a sample of the clay was obtained at 1.5 m depth using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedure. In addition, three 73 mm diameter thin-walled Shelby tube samples of the clay were retrieved from Borehole 7-11 using a fixed piston sampler. Shelby tube samples were retrieved at three depth intervals, at about 4 m, 6 m, and 8 m (shallow, mid-level, and deep samples). In situ vane shear strength testing was carried out between sampling intervals within the clay, using an MTO 'N'-sized vane, with the reaction (torque) measured by a pair of calibrated scales. After determining the undrained shear strength, remoulded shear strengths were also measured.

Within Borehole 17-12, samples of the clay were obtained at 2 m intervals of depth, using an 80 mm outside diameter split-spoon sampler in accordance with the Standard Penetration Test (SPT) procedure. These samples were placed into 3 containers based on their depth interval, at about 3.7 m, 5.5 m, and 7.8 m (shallow, mid-level, and deep samples), and were shipped to Ryerson University for soil mixing experiments.

The field investigation program also included 2 piezocone penetration tests, numbered CPT 17-1 and CPT 17-2. CPT 17-1 and 17-2 were advanced within the proposed south and north approach embankments, respectively. The CPT's were carried out using portable CPT equipment supplied and operated by ConeTec Investigations Ltd. of Richmond Hill, Ontario. The CPT equipment was advanced using a track-mounted drill rig, supplied and operated by Marathon Drilling Ltd. of Ottawa, Ontario.

In each CPT hole, the piezocone was pushed starting from the top of the clay layer (either from ground surface or about 1.5 m depth), through the inside of the hollow-stem augers, and using the loading head of the drill rig to advance the piezocone at a rate of about 2 cm per second. The tip resistance, shaft friction, and pore water pressure were measured at approximately 0.025 m depth intervals. The CPT holes were advanced to depths of about 10.0 and 11.3 m below the existing ground surface at the locations of CPT 17-1 and 17-2, respectively.

The boreholes and CPT holes were backfilled with bentonite pellets, mixed with native soils in the overburden and the site conditions were restored following completion of work.

The field work was supervised by members of Golder's technical and engineering staff, who located the boreholes, supervised the drilling, sampling and in situ testing operations, logged the boreholes, and examined and cared for the soil and bedrock samples.

The soil samples were identified in the field, placed in labelled containers and transported to Golder's laboratory in Ottawa for further examination before shipping to Ryerson University, University of Western Ontario and Golder's laboratory in Mississauga for testing. The samples obtained from Borehole 17-12 were shipped to Ryerson University for soil mixing testing. Shelby tube samples from Borehole 17-11 were shipped to the University of Western Ontario (UWO) for constant rate of strain (CRS) testing and to Golder's Mississauga laboratory for consolidation testing. Three oedometer consolidation tests (i.e., two incremental loading (IL) consolidation tests and one long-term consolidation test) were performed on selected Shelby tube samples at Golder's Mississauga laboratory. Three CRS tests were carried out on samples collected at shallow, mid-level, and deep Shelby tube samples at the UWO laboratory. Index and classification tests consisting of water content determinations, Atterberg limits testing, and specific gravity measurements were carried out on selected soil samples at both the UWO and Golder laboratories. All of the laboratory tests at the Golder laboratory were carried out to MTO and/or ASTM standards, as appropriate.

Prior to drilling, the borehole and CPT locations were staked and surveyed by Golder personnel using a Trimble R8 GPS unit. The borehole and CPT hole locations, including MTM NAD83 northing and easting coordinates and ground surface elevations referenced to Geodetic datum, are summarized in the following table and are shown on Drawing 1.

Borehole Number	Borehole Location	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)	Borehole/CPT depth (m)
17-11	South Approach Embankment	5000034.8	226411.6	48.1	9.2
17-12	South Approach Embankment	5000033.3	226411.6	48.1	9.1
CPT 17-1	South Approach Embankment	5000026.2	226416.6	48.0	10.0
CPT 17-2	North Approach Embankment	5000135.2	226356.0	47.9	11.3

4.0 SITE STRATIGRAPHY

4.1 General

As part of the subsurface investigation at this site, two boreholes and two piezocone holes were advanced within the footprint of the approach embankments for the proposed bridge widening. The borehole locations from the current and previous investigations at the site are shown on Drawing 1. The stratigraphic boundaries shown on the borehole records and on the interpreted stratigraphic profile on Drawing 1 are inferred from observations of drilling progress and from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

The subsurface conditions encountered in the testholes advanced during the current investigation are shown on the Record of Borehole and Record of CPT sheets in Appendix A. The results of the water content and Atterberg limit testing obtained at Golder Laboratory are indicated on the Record of Borehole sheets. The CPT results including profiles of the tip resistance (q_t), sleeve friction (f_s), porewater pressure (u_2) during pushing and the corrected tip resistance (q_t) and sleeve friction (f_t) are presented on the Record of Cone Penetration Test sheets in Appendix B. The laboratory test results including, grain size distribution graphs, plasticity charts, results of

oedometer consolidation testing obtained at Golder laboratory and the results of the CRS testing obtained at University of Western Ontario are provided in Appendix C. The Record of Borehole sheets and selected laboratory testing results from the previous investigations at the site are provided for reference in Appendix D. The results of the cement mixing carried out by Ryerson University are provided in Appendix E.

In summary, the subsurface conditions along County Rd 2/34 at Hwy 401 crossing consist of about 0.8 to 2.0 m of heterogeneous fill material, locally underlain or overlain by about 0.1 m to 0.3 m of topsoil or peat, and then by about 0.2 m of silty sand. These surficial deposits are underlain by a thick deposit of sensitive clay, which extends to depths ranging from 9.9 to 11.4 m below the existing ground surface and varies from about 8.8 to 9.9 m in thickness. The clay has been weathered to a stiff to very stiff grey brown crust to depths ranging from about 2.1 to 3.6 m. The clay below the depth of weathering is grey in color and has a soft to firm consistency, becoming stiff with depth. The clay is underlain by sandy silt, gravelly sand, sand and gravel or glacial till, typically forming a layer over the bedrock surface with thicknesses ranging from about 0.8 m to 2.7 m. The surface of the underlying limestone bedrock appears to be relatively uniform, at depths ranging from about 12.6 to 13.9 m (Elevations ranging from 34.9 to 35.9 m). The groundwater level is near ground surface, at depths ranging from about 0.3 to 1.1 m below the existing ground surface.

A more detailed description of the subsurface conditions encountered in the previous boreholes is provided in the previous Golder report dated September 2017 (Geocres 31G-259). The following sections provide more detailed information for the boreholes drilled during the current investigation, with the focus on characterizing the clay layer.

4.2 Clay

The surficial materials at the site are currently underlain by a thick deposit of sensitive clay. The clay deposit, where fully penetrated in Boreholes 16-1 to 16-5, inclusive, and Borehole 17-10, extends to depths of about 9.9 to 11.4 m below existing ground surface and varies from about 8.8 to 9.9 m in thickness.

The upper portion of the clay has been weathered to form a grey brown crust. The thickness of the crust ranges from about 1.0 to 2.9 m and extends to depths ranging from about 2.1 to 3.6 m. Standard penetration tests carried out within the weathered crust gave 'N' values ranging from "Weight of Hammer" to about 17 blows per 0.3 m of penetration. In situ shear vane testing carried out where possible within this deposit measured undrained shear strengths of 94 to 118 kPa, indicating a stiff to very stiff consistency.

Grain size distribution testing was previously carried out on one sample of the weathered clay, the results of which are provided on Figure B2 in Appendix D. The results of Atterberg limit testing carried out on five samples of the weathered clay are summarized on Figure B3 in Appendix D and indicate plasticity index values generally ranging from 44 to 78 percent and liquid limit values ranging from 67 to 100 percent, reflecting a clay of high plasticity. The measured water content of the weathered clay ranges from approximately 29 to 52 percent.

The clay below the depth of weathering is grey and unweathered. In situ shear vane testing carried out within the unweathered portion of the deposit measured undrained shear strengths ranging from about 19 to more than 96 kPa, indicating a soft to very stiff consistency, however the deposit was found to be more generally firm. The average sensitivity ratio based on remoulded shear strengths in this deposit is about 6, indicating a sensitive material. A summary of engineering properties for the clay deposit is included in Figure C1 in Appendix C, which includes the parameters estimated/measured within the clay during both the current and past investigations.

Grain size distribution testing was previously carried out on one sample of the unweathered clay, the results of which are provided on Figure B4 in Appendix D. The results of Atterberg limit testing of unweathered clay carried out at Golder, UWO and Ryerson University are presented in Table C1 and summarized on Figure C2, in

Appendix C. The previous results of Atterberg limit testing carried out on ten samples of the unweathered clay are also summarized on Figure B5 in Appendix D. Based on Golder's previous and current test results, the plasticity index values range from about 31 to 78 percent and the liquid limit values range from about 57 to 100 percent, indicating a high plasticity clay. The measured water contents of the unweathered portion of the deposit were between about 58 to 93 percent.

It should be noted that the water content and Atterberg limit values reported by UWO and Golder are significantly different. This may be the result of different testing protocols at each facility. Golder follows the ASTM and carries out two tests for each sample and if the results of the two tests are more than 1% different, the test will be repeated. It is not clear if UWO follows the same procedure.

4.2.1 Oedometer and CRS Consolidation Testing

Incremental loading (IL) oedometer consolidation testing was previously carried out on three samples of the unweathered clay, the results of which are provided on Figures B6 to B8, inclusive, in Appendix D. IL oedometer consolidation testing was also carried out on three Shelby tube samples that were retrieved during the current investigation, the results of which are provided on Figures C3 to C5, inclusive, in Appendix C. A long term (LT) consolidation test was also carried out on one of the samples to assess the creep and secondary consolidation characteristics of the clay and the results are provided on Figures C4 and C6. Constant rate of strain consolidation testing was carried out on the same Shelby tube samples at UWO and the results are provided on Figures C7 to C9, inclusive, in Appendix C.

The results from the three oedometer consolidation tests carried out during the previous Golder investigation and the new results for the current investigation, as well as the CRS test results from the UWO laboratory, are summarized in the table below.

Borehole / Sample Number / Lab	Type of Test	Sample Depth/Elevation (m)	Unit Weight (kN/m ³)	σ_p' (kP)	σ_{vo}' (kP)	$\sigma_p' - \sigma_{vo}'$ (kPa)	C _c	C _r	e _o	OCR
17-11 / 2 / GAL	IL	4.8/43.3	14.7	125	35	90	1.55	0.010	2.29	3.5
17-11 / 3 / GAL	LT	6.4/41.7	14.9	-	44	-	-	0.019	2.39	-
17-11 / 4 / GAL	IL	7.9/40.2	15.6	130	53	77	1.30	0.025	1.86	2.4
17-11 / 2 / UWO	CRS	4.8/43.3	-	180	35	145	1.41	0.081	1.73	5.1
17-11 / 3 / UWO	CRS	6.4/41.7	-	120	44	76	1.64	0.146	2.37	2.7
17-11 / 4 / UWO	CRS	7.9/40.2	-	190	53	137	1.43	0.094	1.75	3.6
16-5B / 1 / GAL	IL	5.7 / 42.4	15.2	100	60	40	1.23	0.039	2.21	1.7
16-5B / 2 / GAL	IL	7.6 / 40.5	16.3	100	70	30	0.81	0.020	1.71	1.4
16-3 / 8 / GAL	IL	8.9 / 39.5	15.9	140	80	60	2.09	0.017	1.86	1.8

Notes:

GAL Golder Associates laboratory

UWO University of Western Ontario laboratory

σ_p' Apparent preconsolidation pressure

σ_{vo}' Computed existing vertical effective stress

C_c Compression index

C_r Recompression index

e_o Initial void ratio

OCR Overconsolidation ratio

IL Incremental Loading Oedometer test

LT Long-term Oedometer test

CRS Constant Rate of Strain test

The results of the IL testing indicate that the clay is overconsolidated with preconsolidation pressures ranging from about 100 to 140 kPa and overconsolidation ratios of 1.4 to 3.5.

The CRS tests were carried out using constant loading strain rate of 5, 10 and 0.75 %/hr for sample 2, 3 and 4, respectively. The CRS test results indicate higher preconsolidation pressures (i.e., ranging from about 120 to 190 kPa) than obtained with the IL consolidation tests. The CRS overconsolidation ratios (i.e., ranging from 1.7 to 5.1) are also higher. However, CRS test results are highly dependent on the chosen strain rate and are not directly comparable with the IL Oedometer results. Additional analysis and normalization of the test results would be required to relate CRS results to the Oedometer results.

4.2.2 CPT Results

The undrained shear strength profile of the clay has been evaluated based on the results of the piezocone testing program, using the following equation:

$$S_u = (q_t - \sigma_{vo}) / N_K$$

Where:

- S_u = Calculated undrained shear strength (kPa);
- q_t = Measured net tip resistance (kPa);
- σ_{vo} = Calculated total vertical stress (kPa); and,
- N_K = Correlation factor, selected by ConeTec.

The undrained shear strength profiles for the clay determined from the results of the piezocone testing, as described above, are summarized in Appendix B. The piezocone test results indicate an undrained shear strength ranging from about 60 kPa to 20 kPa over the depth of unweathered clay.

The undrained shear strength of the clay, based on the estimation from the CPT results, decreases steadily with depth from 90 kPa at the top of the weathered crust, generally reaching about 30 kPa below Elevation 44 m. The undrained shear strength below that elevation remains in the range of about 20 to 30 kPa, for CPT 17-01 and CPT 17-02.

The piezocone test results have also been interpreted and calibrated against the laboratory consolidation test results to provide a profile of the preconsolidation pressure with elevation, as shown on Figure C10 in Appendix C. The method used to process the data is suggested by Demers and Leroueil (2002) for Champlain Sea clay, with:

$$\sigma_{p'} = (q_t - \sigma_{vo}) / N_{ot}$$

Where:

- $\sigma_{p'}$ = Calculated preconsolidation pressure (kPa);
- q_t = Measured net tip resistance (kPa);
- σ_{vo} = Calculated total vertical stress (kPa); and,
- N_{ot} = Correlation factor, selected as 3.0 based on Bjerrum (1975) correlation.

As can be seen on Figure C10, lower preconsolidation pressures were recorded for the north approach embankment, in CPT 17-02; possibly indicating more compressible material in that area. The results for CPT 17-02 indicate that the preconsolidation pressure of the silty clay decreases steadily with depth from 150 kPa at the top of unweathered portion of the deposit, generally reaching about 90 to 100 kPa below Elevation 44 m, consistent with the undrained field vane shear strengths measured in the conventionally drilled boreholes. The results for CPT 17-01 indicate that the preconsolidation pressure is more variable, ranging from about 100 to 140 kPa and increasing before decreasing with depth below about Elevation 40 m.

4.2.3 Soil-Cement Mix Test Results

Three buckets of clay samples retrieved from Borehole 17-12 at depths of about 3.7 m, 5.5 m, and 7.8 m (shallow, mid-level, and deep samples) were shipped to Ryerson University for soil-cement mix testing. The samples were treated with the four different cement dosages of 100, 150, 200 and 250 kg/m³. Geotechnical tests, such as water content measurement, specific gravity, and Atterberg limit tests were also carried out on both the remoulded clay samples and cement treated clay samples.

The samples were then subjected to CRS and unconfined compression strength (UCS) testing. The CRS testing was carried out on remoulded samples, as well as 7 and 14-day cured soil-cement mixed samples, and the UCS testing was carried out on soil-cement mixed samples that had cured for 7, 14, 28 and 56 days after mixing.

Further details on the testing procedures and the results are presented in the report prepared by Ryerson University titled "Cement Mixing to Treat Sensitive Champlain Sea Clay", dated March 2018. That report is provided as an attachment in Appendix E.

The UCS test results on treated (i.e., soil-cement mixed) samples gave peak UCS strength values of about 860 to 900 kPa at the lowest cement dosage of 100 kg/m³, after 7 days of curing.

The CRS test results from the treated samples indicate that the compressibility of the soil-cement mixed samples is less than the untreated clay, with C_c values of about 0.1 for cement treated samples compared with C_c values of 0.8 to 2.2 for untreated samples. The preconsolidation pressure of cement-treated samples also increased to more than 1000 kPa compared with preconsolidation pressures ranging from 100 to 140 kPa for untreated clay samples.

5.0 CLOSURE

This Foundation Investigation Report was prepared by Ms. Sahar Soleimani, P.Eng., and reviewed by Mr. Bill Cavers, P.Eng., a geotechnical engineer and Associate with Golder. Mr. Fin Heffernan, P.Eng., the MTO Foundations Designated Contact for this assignment, conducted an independent quality review of this report.

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PART B

Foundation Design Report
Clay Consolidation Characteristics
County Road 2/34 Approach Embankments
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6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides recommendation for the approach embankment design for the proposed replacement of the existing County Road 2/34 Bridge (Site No. 31-232) over Highway 401 in Lancaster, Ontario. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current subsurface investigation as well as previous Golder investigation (Geocres 31G-259). The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible ground improvement alternatives.

The existing bridge is shown in plan on Drawing 1 and consists of a two-lane, two-span cast-in-place concrete box girder structure with abutments and central pier founded on piles. The existing structure is aligned approximately northwest to southeast and is about 67 m long and 12 m wide. It is understood that the existing structure, built in 1963, is to be replaced with a four lane, two-span structure with a shift in the alignment to west of the existing structure (immediately adjacent to the existing alignment). It is understood that a widening/new embankment will be required about 8 m in height and about 15 to 20 m in width at the crest to accommodate this shift.

In general, the surficial soils at the location of the proposed County Road 2/34 Underpass alignment consist of a surficial layer of fill and/or topsoil (with some localized peat) underlain by a thick compressible clay deposit. The clay is underlain by relatively thin deposits of glacial till and/or sand and gravel over limestone bedrock.

The existing embankment loading over the deep sensitive and compressible clay deposit has led to very large settlements of the embankments since the original construction. These settlements resulted in the abutments of the existing structure tilting back towards the approach embankments.

Analysis indicates that, if conventional earth fill or granular fill is used for the new 8 m high embankments (using conventional construction techniques), settlement of the approach embankments will be significant. It is important to limit these settlements not only to avoid roadway distortion, but also to reduce the potential for differential settlements between the approach embankments and the bridge structure (which will be supported on deep foundations on bedrock, and therefore not subjected to these consolidation settlements) and to eliminate the corresponding downdrag loads on the piles at the abutments, which could in turn reduce the available capacity of the piles.

6.2 Summary of Design Consideration for Granular Fill Embankment

As part of the available correspondence in Geocres 31G00-144, the results of the settlement monitoring between 1962 and 1984 showed that the construction of the original approach embankments led to total settlements (i.e., primary and secondary) of over 1.5 m. Furthermore, the Geocres information indicates horizontal movements between the bridge deck and the abutments in 1967 to be about 125 mm at the north abutment and about 75 mm at the south abutment. High settlement of the existing embankments since the original construction indicates the clay deposit below the existing roadway embankment are most likely now normally consolidated and cannot take on any additional load without overstressing and causing unacceptable settlements. Based on the indicated embankment heights as well as the assessed existing stress level and preconsolidation pressure profile within the clay deposit, if granular embankment fill is used to construct the new embankment to full height, the calculated primary consolidation settlements are estimated to be in the order of 0.8 m (at the location of greatest settlement in the transverse direction). In the longer term, these settlements would increase beyond the estimates given above due to secondary compression (i.e., creep) of the deposit. It is expected that, over a period of 20 years

following construction (the likely approximate time until the first repaving, when the profile could be corrected) secondary compression could increase these settlements by between 50 and 75 mm. The calculated settlement values exceed the usual values accepted by MTO for the approaches to bridges for non-freeways, therefore various options were considered for mitigating the anticipated settlements (see Section 6.3).

The stability of the embankments was also evaluated under undrained (i.e., short-term) conditions, drained conditions, and seismic loading conditions. The results of the stability analyses indicate that the 8 m high embankments with side slopes at 2H:1V would *not* have an acceptable factor of safety against deep-seated rotational stability for the undrained static or for seismic conditions. Stage construction, using flatter side slope or using temporary stabilizing berms were recommended to achieve acceptable factor of safety against undrained static condition. However, neither staged construction nor stabilizing berms, would provide an acceptable factor of safety against deep-seated rotational instability during an earthquake event.

6.3 Embankment Design Alternatives

Given the significant magnitude of the anticipated settlements, and their continuous/long-term nature, it is considered that periodic re-paving to correct for the settlement is not a feasible option for addressing/mitigating the settlement effects. Subexcavation of the clay would also not be feasible due to its thickness. The following feasible options may therefore be considered for mitigating the anticipated settlements:

- 1) Lightweight Fill: Lightweight fill materials such as expanded polystyrene (EPS) could be used for the embankment construction, reducing the stress increase on the compressible clay deposit and the long-term settlement magnitudes to acceptable levels.
- 2) Preloading/Surcharging with Wick Drains together with Lightweight Fill: The new embankment areas could be preloaded and surcharged, in part, and allowed to settle in advance of the roadway being paved or put into service over the new approach embankments. Wick-drains would be used to decrease the preload time. Due to the sensitive nature of the clay and consolidation characteristics, the preload/surcharge height would have to be limited and the use of some EPS would still be required for this option to be feasible. This option would also require multiple stages of construction to satisfy embankment stability during the surcharge/preload period. It is expected that the preload time (including surcharge, staged construction and assuming wick drains) could take two to three years to complete.
- 3) Rigid Inclusions: The installation of Rigid Inclusions (RI) is another alternative for mitigating settlements beneath embankments. RI's constructed of ready-mix concrete installed within the clay soil using specialty equipment would be suitable for this site. Rigid Inclusions could be installed in the clay deposit, up to original ground surface, to transfer the stress from the embankment loads down to the glacial till or bedrock. A Load Transfer Platform (LTP) created using granular material and geogrid, and/or concrete would be constructed above the Rigid Inclusions (i.e., beneath the embankment) to transfer the embankment loads to the columns.
- 4) Deep Soil Mixing: Deep soil mixing is another alternative for mitigating settlements beneath embankments. Deep soil mixing consists of in situ mechanical mixing of the native soil using a process that breaks down the soil without extraction while injecting a stabilizing agent into the mix at low pressure.

Based on discussions with the design team and the MTO, it is understood that the preference is to use ground improvement (i.e., rigid inclusions or deep soil mixing) at the site and then to construct the embankments for the new alignment using granular fill. The selection of the ground improvement method will be the responsibility of the contractor, who will obtain a proprietary design for this project by a ground improvement specialty contractor.

If Options 3 or 4 are selected, the design of the rigid inclusions or soil mixing columns shall consider the potential for interference with underground utilities located within the specified ground improvement area or with the proposed pile configuration at the abutment locations. The construction of a suitable/stable working platform by the general contractor may be required for stability of the drill rig or other equipment used by the ground improvement contractor.

The advantages, disadvantages, relative costs, and risks associated with the options short-listed by MTO (i.e., rigid inclusions and deep soil mixing) are provided in Table 1 following the text of this report. For details on the other embankment construction and settlement mitigation alternatives (i.e., lightweight fill and preloading with wick drains), reference can be made to the previous Golder report dated September 2017 (Geocres 31G-259).

In addition to the items outlined in Table 1 (and discussed further in the sections below), the existing adjacent embankment makes for a complicated design and construction since the settlements will also have to be mitigated under the existing side slopes (otherwise there is the potential for significant post construction maintenance). This settlement will have to be mitigated by the contractor.

Also, due to the geometry of the new embankments with respect to the existing embankments, the installation area and effectiveness of concrete columns or deep soil mixing would be limited to areas where the specialized equipment can install the various ground improvement components. It is therefore anticipated that the ground improvement methods would be installed from at least two working platform elevations and that some amount of shoring (i.e., roadway protection) may be required for these options.

Some limited further details for each ground improvement alternative are presented in the following sub-sections.

6.3.1 Rigid Inclusions

Rigid inclusions (RI) are used to transfer unacceptable embankment loads through compressible soils to stiffer soil or rock. This ground improvement method increases the load carrying capacity of the soil, reduces the compressibility (and therefore the settlement magnitudes) and helps prevent slope instability.

Rigid inclusions can consist of cement-treated aggregate, grouted aggregate, grout mixed with soil, or concreted columns. Aggregate columns are not considered feasible for this site considering the thickness and low strength of the underlying clay deposit. The clay would likely offer little resistance/confinement during the installation of the aggregates and therefore there would be a high risk of column bulging in addition to the potential for shearing failures and remoulding of the clay structure.

Concrete column rigid inclusions would be the most feasible RI system for the site, with a specifically designed Load Transfer Platform (LTP). The LTP, which transfers the load from the embankments to the rigid inclusions, is a key element of the design that distributes the loads to the columns. The system should be designed to satisfy MTO settlement and global stability criteria.

Due to the configuration of the proposed embankment with respect to the existing embankment, it may be recommended to install the RI's from two working elevations (i.e., in two stages). The first stage would include installation of rigid inclusions from the existing ground level (i.e., about Elevation 48 m) beneath the footprint of the widened/new embankment. The second stage would include installation of RI's from a higher elevation, following granular embankment construction to a certain height (i.e., about Elevation 51 m), so that RI's can be placed beneath the future roadway and within the existing embankment side slopes. The RI will go down to the bottom of the clay layer at about Elevation 37 to 38 m reach to a stiffer material. Roadway protection may be required for this second stage, adjacent to the existing lanes of traffic that will need to remain in service.

Additional slope stability analyses would be required to assess the stability of the existing and future embankments.

The LTP would also need to be designed by the specialty ground improvement contractor to limit the load that is directly transmitted to the compressible clay soils. The load transfer system could consist of several alternating layers of geogrid and engineered fill or could be a concrete layer at the top of the RI's.

Wick drains may be required, in addition to RI's, depending on the specialty contractor that is retained and their proprietary design. Wick drains or other structural reinforcement can be used to mitigate against seismic instability.

At the abutment locations, the RI pattern is typically modified to allow for pile installation after some settlement has occurred.

Some amount of EPS may still be required directly behind the abutments if the design cannot acceptably limit the settlements and resulting downdrag forces (i.e., if the settlements at the abutments will exceed 10 mm, then the downdrag forces indicated in the FIDR will need to be applied for design of the piles).

Field trials would be recommended prior to or at the same time as design of the rigid inclusions to establish the range of strength that can be achieved from the ground improvement. The CPT testing carried out during the current investigation will be useful to the specialty contractor for this option.

As a preliminary guideline, based on discussions with ground improvement contractors, it is anticipated that the construction of the RI's and LTP could take about 2 to 3 months to complete (1.5 to 2 weeks per 50 m length of embankment).

6.3.2 Deep Soil Mixing

Improvement of weak and compressible soils by deep soil mixing (DSM) can be achieved by mixing the existing soils using either a slurry with binder (wet DSM) or a dry binder (dry DSM). Jetting of slurry can be also used to enhance mechanical mixing. Similar to rigid inclusions, deep soil mixing creates large columns of improved ground for embankment support.

Approximately 1.8 m diameter columns would be created with the DSM procedures, placed in a specific pattern. A track-mounted drill rig would be used to directly inject the binder into the column areas, add the stabilizing agent, and mixing.

A soil-cement mixing investigation was carried out to understand the strength gain and compressibility of cement-treated samples at four different cement dosages of 100, 150, 200 and 250 kg/m³. The results of this investigation (provided in Appendix E) indicate that the lowest cement dosage that was selected for the laboratory testing (i.e., 100 kg/m³) could provide sufficient strength for the construction of the proposed approach embankments at this site. However, the interaction of deep soil mixing columns with surrounding soils needs to be further investigated to understand the possibility of settlement of the existing embankment (which needs to remain in service during construction) during remolding of the underlying clay and subsequent cement mixing.

The high plasticity clays with high shear strengths (i.e., the weathered clay crust at the site) may require pre-treatment for successful performance. Furthermore, areas with stiff soils and/or obstructions, such as the existing embankments and side slopes, may require pre-drilling ahead of the soil mixing process.

Some amount of EPS may still be required directly behind the abutments if the design cannot acceptably limit the settlements and resulting downdrag forces (i.e., if the settlements at the abutments will exceed 10 mm, then the downdrag forces indicated in the FIDR will need to be applied for design of the piles).

It is recommended that at least one pre-production test column be advanced prior to construction. The ground surface adjacent to the test column should be monitored for settlement during installation to assess the potential impacts during remoulding of the clay soil. In addition, the completed test column should be cored at 7 and 14 days to obtain samples for strength (UCS) testing of the in-situ soil-cement mix.

As a preliminary guideline, based on discussions with ground improvement contractors, it is anticipated that the DSM could take about 2 to 3 months to complete.

7.0 CLOSURE

This Foundation Design Report was prepared by Ms. Sahar Soleimani, P.Eng., and reviewed by Mr. Bill Cavers, P.Eng., a geotechnical engineer and Associate with Golder. Mr. Fin Heffernan, P.Eng., the Designated MTO Foundations Contact for this assignment, conducted an independent quality review of this report.

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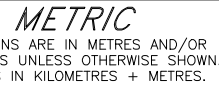
Fin Heffernan, P.Eng.
MTO Foundations Designated Contact

SS/KSL/WC/FJH/mvrd/ca

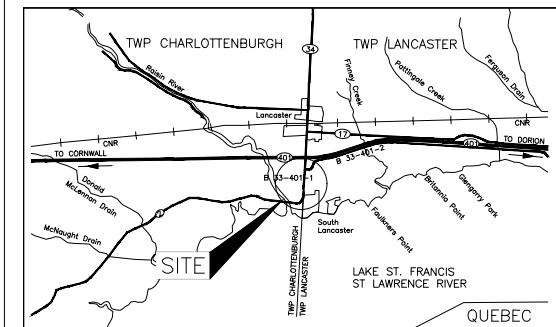
n:\active\2017\3 proj\1772182 mto ref 4016-e-0012 er-e mega 6\assignment #12 - clay characterization and ground improvement\reports\lancaster\1772182 rpt-001 rev1 final hwy 401 lancaster september 2018.docx

Table 1: Comparison of Embankment Settlement Mitigation Alternatives

Embankment Option	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Rigid Inclusions (RI) (e.g., Concrete Columns)	<ul style="list-style-type: none"> ■ Relatively rapid installation ■ Allows for greater bearing pressures and limited settlements 	<ul style="list-style-type: none"> ■ Mobilizing specialty subcontractor may have impact on schedule 	<ul style="list-style-type: none"> ■ The same cost, or slightly higher (up to 15% more), as DSM 	<ul style="list-style-type: none"> ■ Some field testing ahead of production would be recommended ■ Design should consider risk of interference with piles advanced for abutment construction and impact on foundations or other utilities ■ Settlement monitoring recommended prior to final paving
Deep Soil Mixing (DSM)	<ul style="list-style-type: none"> ■ Relatively rapid installation ■ Allows for greater bearing pressures and limited settlements 	<ul style="list-style-type: none"> ■ Mobilizing specialty subcontractor may have impact on schedule ■ Sensitive to installation sequence and radial distance between the mixing columns. 	<ul style="list-style-type: none"> ■ The same cost, or slightly lower (up to 15% lower, than rigid inclusions) 	<ul style="list-style-type: none"> ■ Some field testing ahead of production would be recommended ■ Slightly higher risk option for high plasticity clay ■ May require predrilling where columns are required beneath the existing side slopes ■ Design should consider risk of interference with piles advanced for abutment construction and impact on foundations or other utilities ■ Settlement monitoring recommended prior to final paving ■ Unknown interaction of deep soil mixing columns with surrounding soils and possibility of existing embankment settlement during clay remolding and cement mixing procedure.












SHEET



KEY PLAN

SCALE
1.5 0 1.5 3 km

LEGEND

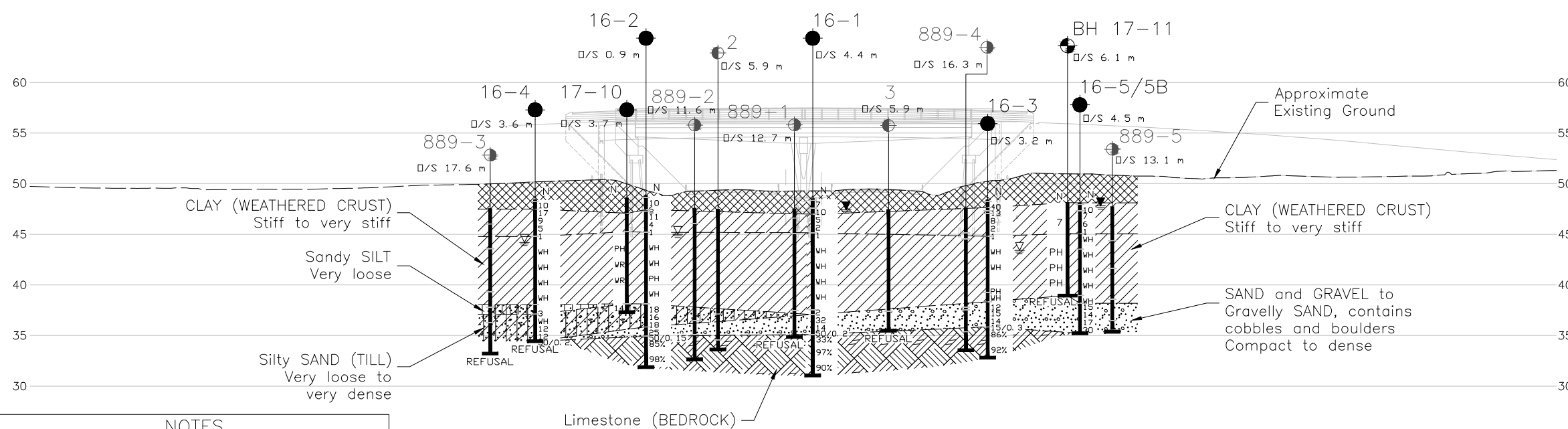
- | | |
|---|--|
|  | Borehole – Current Investigation |
|  | Cone Penetration Test – Current Investigation |
|  | Borehole – Previous Investigation |
|  | Borehole – Previous Investigation
(Geocres No. 31G00–144) |
|  | Borehole – Previous Investigation
(Geocres No. 31G00–145) |
| N | Standard Penetration Test Value |
| 16 | Blows/0.3m unless otherwise stated
(Std. Pen. Test, 475 j/blow) |
| 100% | Rock Quality Designation (RQD) |
|  | WL upon completion of drilling |
|  | WL in piezometer, measured on July 29, 2017 |
|  | Seal |
|  | Piezometer |

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
17-11	48.1	5000034.8	226411.6
17-12	48.1	5000033.3	226411.6
CPT-1	48.0	5000026.2	226415.6
CPT-2	47.9	5000135.2	226356.0
16-1	48.5	5000079.4	226388.0
16-2	48.8	5000109.7	226374.7
16-3	48.4	5000049.9	226406.2
16-4	48.4	5000127.4	226361.5
16-5	48.1	5000033.5	226414.2
16-5B	48.1	5000033.5	226414.2
16-6	48.6	5000055.0	226344.1
16-7	48.4	5000068.1	226365.7
16-8	48.5	5000109.9	226436.1
16-9	48.4	5000123.4	226457.1
17-10	48.6	5000111.7	226370.4
889-1	47.5	5000088.9	226397.4
889-2	47.6	5000105.5	226386.6
889-3	47.5	5000143.5	226371.8
889-4	47.6	5000057.6	226419.4
889-5	47.7	5000034.5	226428.9
BH-1	47.5	5000112.7	226406.3
BH-2	47.5	5000100.8	226387.7
BH-3	47.5	5000071.5	226404.5
BH-4	47.5	5000083.4	226423.0

A	.	.	.
NO.	DATE	BY	REVISION

Geocres No.			
HWY. 401		PROJECT NO. 1772182	DIST. EASTERN
SUBM'D. SS	CHKD. SS	DATE: 02/13/2018	SITE: 31-232
DRAWN: JIM	CHKD. KSJ	APPD. F.J.H	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.

PROFILE A-A'

REFERENCE

Base plans provided in digital format by Dillon, drawing file nos.
4013-09-GA.dwg, received August 15, 2017 and Mega 6 - 4 - Profile
Alternatives-New Ramp.dwg, received January 11, 2018.

DRAFT

APPENDIX A

Borehole Records, Current Investigation

List of Abbreviations and Symbols

Lithological and Geotechnical Rock Description Terminology

Record of Boreholes 17-11 And 17-12 And CPT Holes CPT 17-1 and CPT 17-2



LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta \sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
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 = $(w_l - w_p)$
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 = σ'_p / σ'_{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

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

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



LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

(b) Cohesive Soils Consistency

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

Dynamic Cone Penetration Resistance; N_d:

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

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

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

IV. SOIL TESTS

w	water content
w _p	plastic limit
w _l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D _R	relative density (specific gravity, G _s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO ₄	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

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

V. MINOR SOIL CONSTITUENTS

Per cent by Weight

Modifier

0 to 5	Trace
5 to 12	Trace to Some (or Little)
12 to 20	Some
20 to 30	(ey) or (y)
over 30	And (non-cohesive (cohesionless)) or With (cohesive)

Example

Trace sand
Trace to some sand
Some sand
Sandy
Sand and Gravel
Silty Clay with sand / Clayey Silt with sand



LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERINGS STATE

Fresh: no visible sign of weathering

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

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

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

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

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

BEDDING THICKNESS

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

JOINT OR FOLIATION SPACING

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

GRAIN SIZE

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

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

CORE CONDITION

Total Core Recovery (TCR)

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

Solid Core Recovery (SCR)

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

Rock Quality Designation (RQD)

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

DISCONTINUITY DATA

Fracture Index

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

Dip with Respect to Core Axis

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

Description and Notes

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

Abbreviations

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

PROJECT <u>1772182-1125</u>		RECORD OF BOREHOLE No 17-11		SHEET 1 OF 1		METRIC	
W.P. <u>4260-15-00</u>		LOCATION <u>N 5000034.8; E 226411.6 MTM ZONE 8 (LAT. 45.135940; LONG. -74.496650)</u>		ORIGINATED BY <u>CR & PV</u>			
DIST <u>Eastern</u> HWY <u>401</u>		BOREHOLE TYPE <u>Power Auger, 200 mm Diam. (Hollow Stem)</u>		COMPILED BY <u>ZS</u>			
DATUM <u>Geodetic</u>		DATE <u>November 13, 2017</u>		CHECKED BY _____			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL LIMIT MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa			W _p	W	W _L			
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	WATER CONTENT (%)							
							20 40 60 80 100				25 50 75					
48.1 0.0	GROUND SURFACE For soil stratigraphy refer to Record of Borehole 16-5B															
46.6 1.5	(CH) CLAY (WEATHERED CRUST) Very stiff to stiff Grey-brown		1	SS	7											
45.1 3.1	(CH) CLAY Soft to firm Grey with black mottling															
				2	TP	PH										
				3	TP	PH										
			4	TP	PH											
39.0 9.2	END OF BOREHOLE SAMPLER REFUSAL															

+³, X³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

GTA-MTO 001 N:\ACTIVE\SPATIAL_IMMTO\MEGAEASTERN\RETAINER602_DATA\GINT\1772182-1125.GPJ GAL-GTA.GDT 3/14/18 ZS

PROJECT 1772182-1125		RECORD OF BOREHOLE No 17-12		SHEET 1 OF 1		METRIC							
W.P. 4260-15-00		LOCATION N 5000033.3; E 226411.6 MTM ZONE 8 (LAT. 45.135920; LONG. -74.496640)		ORIGINATED BY PAH									
DIST Eastern HWY 401		BOREHOLE TYPE CPT - Push Hydraulic		COMPILED BY ZS									
DATUM Geodetic		DATE November 14, 2017		CHECKED BY									
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					
48.1 0.0	GROUND SURFACE						20 40 60 80 100	20 40 60 80 100	25 50 75				GR SA SI CL
44.5 3.7	(CH) CLAY, trace shells Soft to firm Grey with black mottling		1	SS	PH								
			2	SS	PH								
			3	SS	PH								
39.0 9.1	END OF BOREHOLE												

GTA-MTO 001 N:ACTIVE\SPATIAL IMMTO\MEGA\EASTERN\RETAINER6\02 DATA\GINT\1772182-1125.GPJ GAL-GTA.GDT 3/14/18 ZS

PROJECT <u>1772182-1125</u>		RECORD OF BOREHOLE No CPT 17-1		SHEET 2 OF 2		METRIC	
W.P. <u>4260-15-00</u>		LOCATION <u>N 5000026.2; E 226415.6 MTM ZONE 8 (LAT. 45.135860; LONG. -74.496590)</u>		ORIGINATED BY <u>PAH</u>			
DIST <u>Eastern</u> HWY <u>401</u>		BOREHOLE TYPE <u>CPT - Push Hydraulic</u>		COMPILED BY <u>ZS</u>			
DATUM <u>Geodetic</u>		DATE <u>November 10, 2017</u>		CHECKED BY _____			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL LIMIT MOISTURE LIQUID CONTENT LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL	
								○ UNCONFINED	+	FIELD VANE	● QUICK TRIAXIAL	×	REMOULDED	w _p	w		w _L				
	--- CONTINUED FROM PREVIOUS PAGE ---						20	40	60	80	100		25	50	75						
30.8	END OF CPT EFFECTIVE REFUSAL																				

GTA-MTO 001 N:\ACTIVE\SPATIAL_IMMTOWMEGAEASTERNRETAINER602_DATA\GINT\1772182-1125.GPJ GAL-GTA.GDT 3/14/18 ZS

PROJECT <u>1772182-1125</u>		RECORD OF BOREHOLE No CPT 17-2		SHEET 1 OF 2		METRIC	
W.P. <u>4260-15-00</u>		LOCATION <u>N 5000135.2; E 226356.0 MTM ZONE 8 (LAT. 45.136840; LONG. -74.497370)</u>		ORIGINATED BY <u>PAH</u>			
DIST <u>Eastern</u> HWY <u>401</u>		BOREHOLE TYPE <u>CPT - Push Hydraulic</u>		COMPILED BY <u>ZS</u>			
DATUM <u>Geodetic</u>		DATE <u>November 10, 2017</u>		CHECKED BY _____			

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		W _P	W	W _L		
47.9 0.0	GROUND SURFACE Fill/Topsoil						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	20 40 60 80 100	25 50 75				kN/m ³	GR SA SI CL
46.4 1.5	Silty Clay (Weathered Crust)													
44.9 3.0	Silty Clay Grey													
									</					

Continued Next Page

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

GTA-MTO 001 N:\ACTIVE\SPATIAL_IMMTOMEGA\EASTERN\RETAINER602_DATA\GINT\1772182-1125.GPJ GAL-GTA.GDT 3/14/18 ZS

PROJECT		RECORD OF BOREHOLE				No CPT 17-2		SHEET 2 OF 2		METRIC							
W.P. 4260-15-00		LOCATION				N 5000135.2; E 226356.0 MTM ZONE 8 (LAT. 45.136840; LONG. -74.497370)				ORIGINATED BY PAH							
DIST Eastern HWY 401		BOREHOLE TYPE				CPT - Push Hydraulic				COMPILED BY ZS							
DATUM Geodetic		DATE				November 10, 2017				CHECKED BY							
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					
36.7	Silty Clay Grey						37										
11.3	END OF CPT EFFECTIVE REFUSAL																

GTA-MTO 001 N:\ACTIVE\SPATIAL_IMMTO\MEGAEASTERN\RETAINER602_DATA\GINT\1772182-1125.GPJ GAL-GTA.GDT 3/14/18 ZS

APPENDIX B

ConeTec Investigation Report

PRESENTATION OF SITE INVESTIGATION RESULTS

Hwy 401 at 2-34

Prepared for:

Golder Associates

ConeTec Job No: 17-05066

Project Start Date: 10-Nov-2017

Project End Date: 10-Nov-2017

Report Date: 16-Nov-2017



Prepared by:

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Introduction

The enclosed report presents the results of the site investigation program conducted by ConeTec Investigations Ltd. for Golder Associates in Lancaster, ON. The program consisted of two cone penetration tests (CPT).

Project Information

Project	
Client	Golder Associates
Project	Hwy 401 at 2-34
ConeTec project number	17-05066

The CPT test locations are presented in the following map image.



Rig Description	Deployment System	Test Type
Portable	Drill rig	CPT

Coordinates		
Test Type	Collection Method	EPSG Number
CPT	Consumer grade GPS	32618

Cone Penetration Test (CPT)	
Depth reference	Depths are referenced to the existing ground surface at the time of each test.
Tip and sleeve data offset	0.1 meter This has been accounted for in the CPT data files.
Additional plots	Soil behavior type (SBT) scatter plots and advanced plots with I_c , $S_u(Nkt)$, OCR and $N1(60)I_c$ are provided in the data release package.

Cone Penetrometers Used for this Project						
Cone Description	Cone Number	Cross Sectional Area (cm ²)	Sleeve Area (cm ²)	Tip Capacity (bar)	Sleeve Capacity (bar)	Pore Pressure Capacity (psi)
465:T1500F15U500	465	15	225	1500	15	500
Cone 465 was used for all CPT soundings.						

Calculated Geotechnical Parameter Tables	
Additional information	<p>The Normalized Soil Behaviour Type Chart based on Q_{tn} (SBT Q_{tn}) (Robertson, 2009) was used to classify the soil for this project. A detailed set of calculated CPT parameters have been generated and are provided in Excel format files in the release folder. The CPT parameter calculations are based on values of corrected tip resistance (q_t) sleeve friction (f_s) and pore pressure (u_2).</p> <p>Soils were classified as either drained or undrained based on the Normalized Soil Behaviour Type Chart (SBT Q_{tn}) (Robertson, 2009). Calculations for both drained and undrained parameters were included for materials that classified as silt mixtures – clayey silt to silty clay (zone 4).</p>

Limitations

This report has been prepared for the exclusive use of Golder Associates (Client) for the project titled "Hwy 401 at 2-34". The report's contents may not be relied upon by any other party without the express written permission of ConeTec Investigations Ltd. (ConeTec). ConeTec has provided site investigation services, prepared the factual data reporting and provided geotechnical parameter calculations consistent with current best practices. No other warranty, expressed or implied, is made.

The information presented in the report document and the accompanying data set pertain to the specific project, site conditions and objectives described to ConeTec by the Client. In order to properly understand the factual data, assumptions and calculations, reference must be made to the documents provided and their accompanying data sets, in their entirety.



The cone penetration tests (CPTu) are conducted using an integrated electronic piezocone penetrometer and data acquisition system manufactured by Adara Systems Ltd. of Richmond, British Columbia, Canada.

ConeTec's piezocone penetrometers are compression type designs in which the tip and friction sleeve load cells are independent and have separate load capacities. The piezocones use strain gauged load cells for tip and sleeve friction and a strain gauged diaphragm type transducer for recording pore pressure. The piezocones also have a platinum resistive temperature device (RTD) for monitoring the temperature of the sensors, an accelerometer type dual axis inclinometer and a geophone sensor for recording seismic signals. All signals are amplified down hole within the cone body and the analog signals are sent to the surface through a shielded cable.

ConeTec penetrometers are manufactured with various tip, friction and pore pressure capacities in both 10 cm² and 15 cm² tip base area configurations in order to maximize signal resolution for various soil conditions. The specific piezocone used for each test is described in the CPT summary table presented in the first Appendix. The 15 cm² penetrometers do not require friction reducers as they have a diameter larger than the deployment rods. The 10 cm² piezocones use a friction reducer consisting of a rod adapter extension behind the main cone body with an enlarged cross sectional area (typically 44 mm diameter over a length of 32 mm with tapered leading and trailing edges) located at a distance of 585 mm above the cone tip.

The penetrometers are designed with equal end area friction sleeves, a net end area ratio of 0.8 and cone tips with a 60 degree apex angle.

All ConeTec piezocones can record pore pressure at various locations. Unless otherwise noted, the pore pressure filter is located directly behind the cone tip in the "u₂" position (ASTM Type 2). The filter is 6 mm thick, made of porous plastic (polyethylene) having an average pore size of 125 microns (90-160 microns). The function of the filter is to allow rapid movements of extremely small volumes of water needed to activate the pressure transducer while preventing soil ingress or blockage.

The piezocone penetrometers are manufactured with dimensions, tolerances and sensor characteristics that are in general accordance with the current ASTM D5778 standard. ConeTec's calibration criteria also meets or exceeds those of the current ASTM D5778 standard. An illustration of the piezocone penetrometer is presented in Figure CPTu.



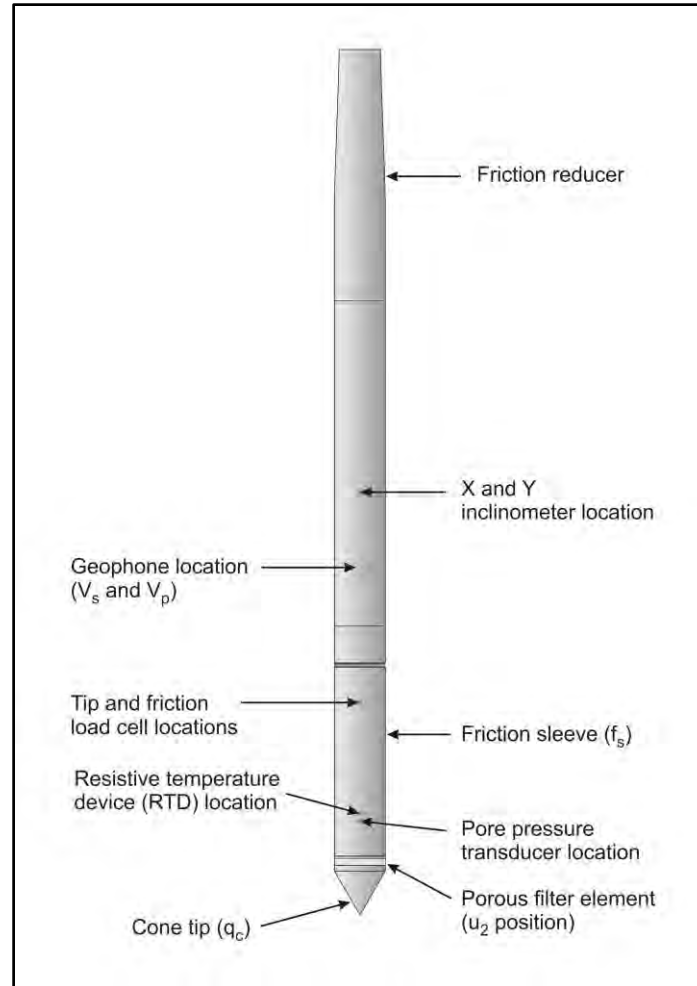


Figure CPTu. Piezocone Penetrometer (15 cm²)

The ConeTec data acquisition systems consist of a Windows based computer and a signal conditioner and power supply interface box with a 16 bit (or greater) analog to digital (A/D) converter. The data is recorded at fixed depth increments using a depth wheel attached to the push cylinders or by using a spring loaded rubber depth wheel that is held against the cone rods. The typical recording intervals are either 2.5 cm or 5.0 cm depending on project requirements; custom recording intervals are possible. The system displays the CPTu data in real time and records the following parameters to a storage media during penetration:

- Depth
- Uncorrected tip resistance (q_c)
- Sleeve friction (f_s)
- Dynamic pore pressure (u)
- Additional sensors such as resistivity, passive gamma, ultra violet induced fluorescence, if applicable

All testing is performed in accordance to ConeTec's CPT operating procedures which are in general accordance with the current ASTM D5778 standard.

Prior to the start of a CPTu sounding a suitable cone is selected, the cone and data acquisition system are powered on, the pore pressure system is saturated with either glycerine or silicone oil and the baseline readings are recorded with the cone hanging freely in a vertical position.

The CPTu is conducted at a steady rate of 2 cm/s, within acceptable tolerances. Typically one meter length rods with an outer diameter of 1.5 inches are added to advance the cone to the sounding termination depth. After cone retraction final baselines are recorded.

Additional information pertaining to ConeTec's cone penetration testing procedures:

- Each filter is saturated in silicone oil or glycerine under vacuum pressure prior to use
- Recorded baselines are checked with an independent multi-meter
- Baseline readings are compared to previous readings
- Soundings are terminated at the client's target depth or at a depth where an obstruction is encountered, excessive rod flex occurs, excessive inclination occurs, equipment damage is likely to take place, or a dangerous working environment arises
- Differences between initial and final baselines are calculated to ensure zero load offsets have not occurred and to ensure compliance with ASTM standards

The interpretation of piezocone data for this report is based on the corrected tip resistance (q_t), sleeve friction (f_s) and pore water pressure (u). The interpretation of soil type is based on the correlations developed by Robertson (1990) and Robertson (2009). It should be noted that it is not always possible to accurately identify a soil type based on these parameters. In these situations, experience, judgment and an assessment of other parameters may be used to infer soil behaviour type.

The recorded tip resistance (q_c) is the total force acting on the piezocone tip divided by its base area. The tip resistance is corrected for pore pressure effects and termed corrected tip resistance (q_t) according to the following expression presented in Robertson et al, 1986:

$$q_t = q_c + (1-a) \cdot u_2$$

where: q_t is the corrected tip resistance

q_c is the recorded tip resistance

u_2 is the recorded dynamic pore pressure behind the tip (u_2 position)

a is the Net Area Ratio for the piezocone (0.8 for ConeTec probes)

The sleeve friction (f_s) is the frictional force on the sleeve divided by its surface area. As all ConeTec piezocones have equal end area friction sleeves, pore pressure corrections to the sleeve data are not required.

The dynamic pore pressure (u) is a measure of the pore pressures generated during cone penetration. To record equilibrium pore pressure, the penetration must be stopped to allow the dynamic pore pressures to stabilize. The rate at which this occurs is predominantly a function of the permeability of the soil and the diameter of the cone.

The friction ratio (R_f) is a calculated parameter. It is defined as the ratio of sleeve friction to the tip resistance expressed as a percentage. Generally, saturated cohesive soils have low tip resistance, high



friction ratios and generate large excess pore water pressures. Cohesionless soils have higher tip resistances, lower friction ratios and do not generate significant excess pore water pressure.

A summary of the CPTu soundings along with test details and individual plots are provided in the appendices. A set of interpretation files were generated for each sounding based on published correlations and are provided in Excel format in the data release folder. Information regarding the interpretation methods used is also included in the data release folder.

For additional information on CPTu interpretations, refer to Robertson et al. (1986), Lunne et al. (1997), Robertson (2009), Mayne (2013, 2014) and Mayne and Peuchen (2012).

The cone penetration test is halted at specific depths to carry out pore pressure dissipation (PPD) tests, shown in Figure PPD-1. For each dissipation test the cone and rods are decoupled from the rig and the data acquisition system measures and records the variation of the pore pressure (u) with time (t).

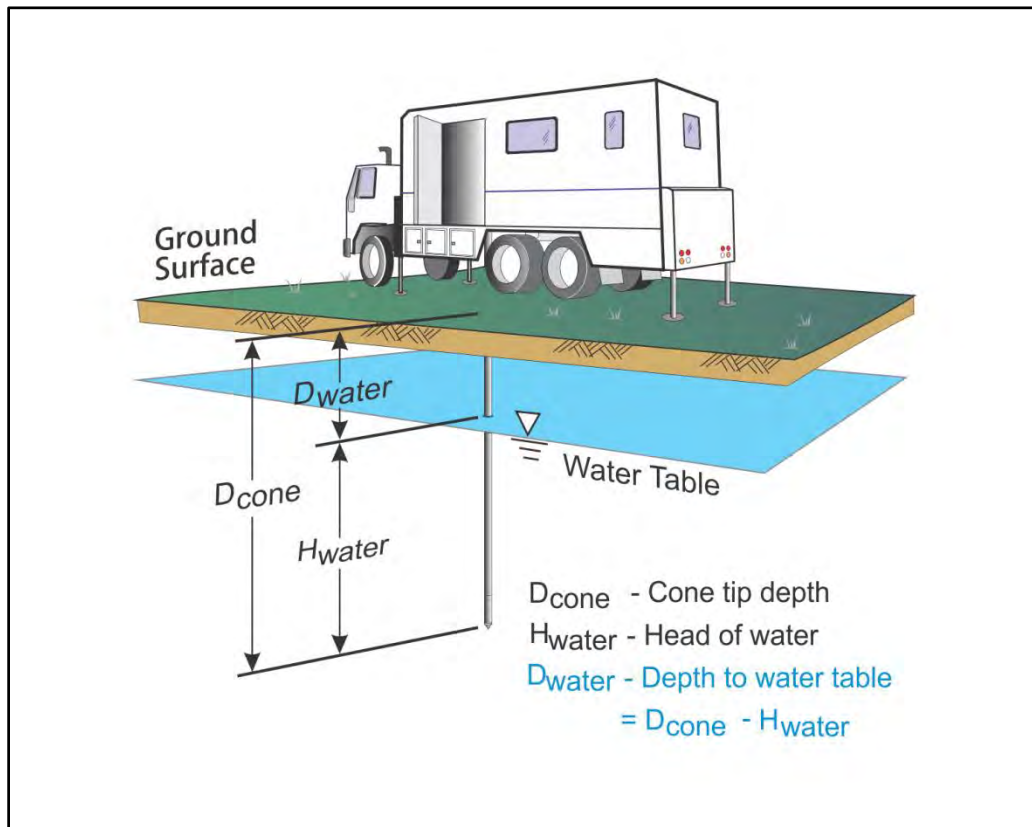


Figure PPD-1. Pore pressure dissipation test setup

Pore pressure dissipation data can be interpreted to provide estimates of ground water conditions, permeability, consolidation characteristics and soil behaviour.

The typical shapes of dissipation curves shown in Figure PPD-2 are very useful in assessing soil type, drainage, in situ pore pressure and soil properties. A flat curve that stabilizes quickly is typical of a freely draining sand. Undrained soils such as clays will typically show positive excess pore pressure and have long dissipation times. Dilative soils will often exhibit dynamic pore pressures below equilibrium that then rise over time. Overconsolidated fine-grained soils will often exhibit an initial dilatory response where there is an initial rise in pore pressure before reaching a peak and dissipating.

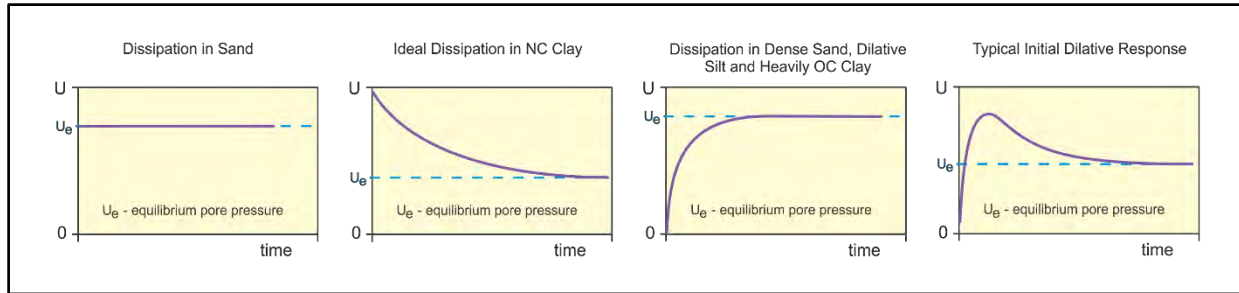


Figure PPD-2. Pore pressure dissipation curve examples

In order to interpret the equilibrium pore pressure (u_{eq}) and the apparent phreatic surface, the pore pressure should be monitored until such time as there is no variation in pore pressure with time as shown for each curve of Figure PPD-2.

In fine grained deposits the point at which 100% of the excess pore pressure has dissipated is known as t_{100} . In some cases this can take an excessive amount of time and it may be impractical to take the dissipation to t_{100} . A theoretical analysis of pore pressure dissipations by Teh and Houlsby (1991) showed that a single curve relating degree of dissipation versus theoretical time factor (T^*) may be used to calculate the coefficient of consolidation (c_h) at various degrees of dissipation resulting in the expression for c_h shown below.

$$c_h = \frac{T^* \cdot a^2 \cdot \sqrt{I_r}}{t}$$

Where:

- T^* is the dimensionless time factor (Table Time Factor)
- a is the radius of the cone
- I_r is the rigidity index
- t is the time at the degree of consolidation

Table Time Factor. T^* versus degree of dissipation (Teh and Houlsby, 1991)

Degree of Dissipation (%)	20	30	40	50	60	70	80
$T^* (u_2)$	0.038	0.078	0.142	0.245	0.439	0.804	1.60

The coefficient of consolidation is typically analyzed using the time (t_{50}) corresponding to a degree of dissipation of 50% (u_{50}). In order to determine t_{50} , dissipation tests must be taken to a pressure less than u_{50} . The u_{50} value is half way between the initial maximum pore pressure and the equilibrium pore pressure value, known as u_{100} . To estimate u_{50} , both the initial maximum pore pressure and u_{100} must be known or estimated. Other degrees of dissipations may be considered, particularly for extremely long dissipations.

At any specific degree of dissipation the equilibrium pore pressure (u at t_{100}) must be estimated at the depth of interest. The equilibrium value may be determined from one or more sources such as measuring the value directly (u_{100}), estimating it from other dissipations in the same profile, estimating the phreatic surface and assuming hydrostatic conditions, from nearby soundings, from client provided information, from site observations and/or past experience, or from other site instrumentation.

For calculations of c_h (Teh and Houlsby, 1991), t_{50} values are estimated from the corresponding pore pressure dissipation curve and a rigidity index (I_r) is assumed. For curves having an initial dilatory response in which an initial rise in pore pressure occurs before reaching a peak, the relative time from the peak value is used in determining t_{50} . In cases where the time to peak is excessive, t_{50} values are not calculated.

Due to possible inherent uncertainties in estimating I_r , the equilibrium pore pressure and the effect of an initial dilatory response on calculating t_{50} , other methods should be applied to confirm the results for c_h .

Additional published methods for estimating the coefficient of consolidation from a piezocone test are described in Burns and Mayne (1998, 2002), Jones and Van Zyl (1981), Robertson et al. (1992) and Sully et al. (1999).

A summary of the pore pressure dissipation tests and dissipation plots are presented in the relevant appendix.

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The appendices listed below are included in the report:

- Cone Penetration Test Summary and Standard Cone Penetration Test Plots
- Advanced Cone Penetration Test Plots
- Soil Behavior Type (SBT) Scatter Plots
- Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

Cone Penetration Test Summary and Standard Cone Penetration Test Plots





Job No: 17-05066
Client: Golder Associates
Project: Hwy 401 at 2-34
Start Date: 10-Nov-2017
End Date: 10-Nov-2017

CONE PENETRATION TEST SUMMARY

Sounding ID	File Name	Date	Cone	Assumed Phreatic Surface ¹ (m)	Final Depth (m)	Northing ² (m)	Easting (m)	Refer to Notation Number
CPT17-01	17-05066_CP01	10-Nov-2017	465:T1500F15U500	0.5	10.300	4998167	539582	
CPT17-02	17-05066_CP02	10-Nov-2017	465:T1500F15U500	0.5	11.725	4998275	539520	

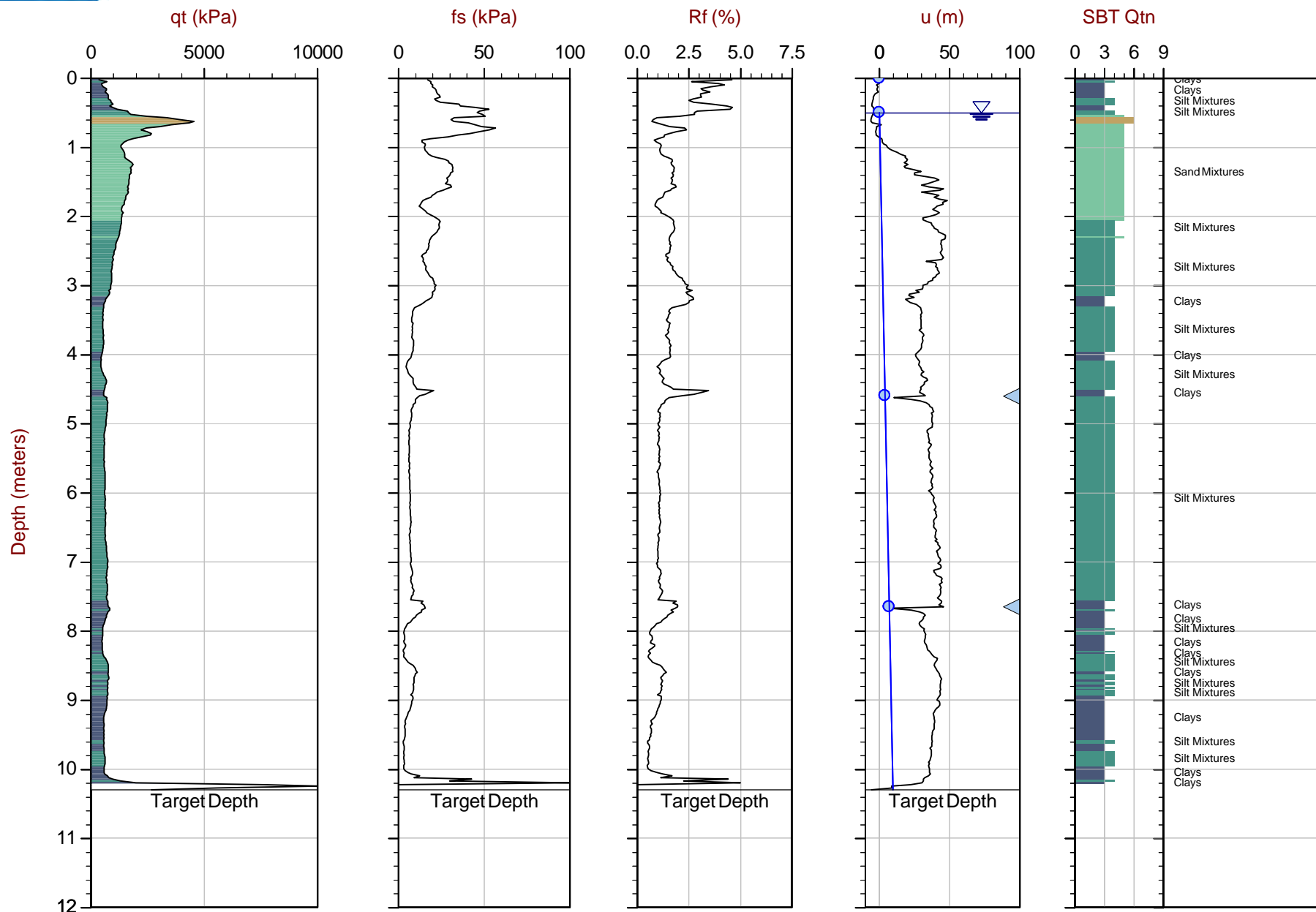
1. The assumed phreatic surface was based on an adjacent well. Hydrostatic conditions were assumed for the calculated parameters.
2. Coordinates were collected with a consumer grade GPS device in datum WGS84/UTM Zone 18 North.



Golder

Job No: 17-05066
Date: 2017-11-10 10:16
Site: Lancaster, Ontario

Sounding: CPT17-01
Cone: 465:T1500F15U500



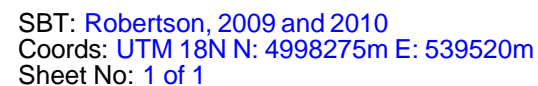
Max Depth: 10.300 m / 33.79 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 17-05066_CP01.COR
Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 4998167m E: 539582m
Sheet No: 1 of 1

Assumed Ueq Dissipation, equilibrium assumed Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Advanced Cone Penetration Test Plots



Golder

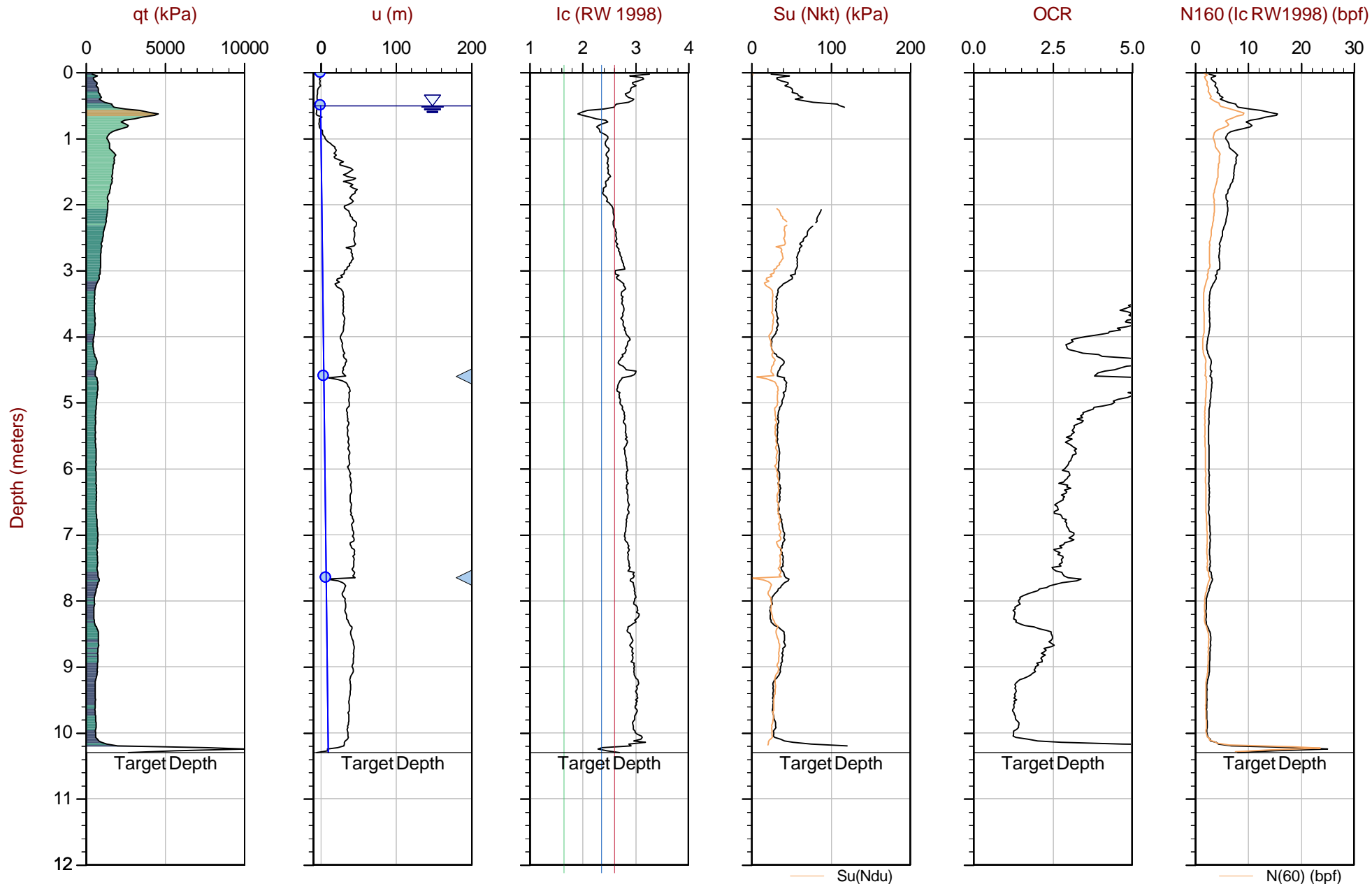
Job No: 17-05066

Date: 2017-11-10 10:16

Site: Lancaster, Ontario

Sounding: CPT17-01

Cone: 465:T1500F15U500



Max Depth: 10.300 m / 33.79 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 17-05066_CP01.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 10.0

SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 4998167m E: 539582m
Sheet No: 1 of 1

● Assumed Ueq ◀ Dissipation, equilibrium assumed — Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



Golder

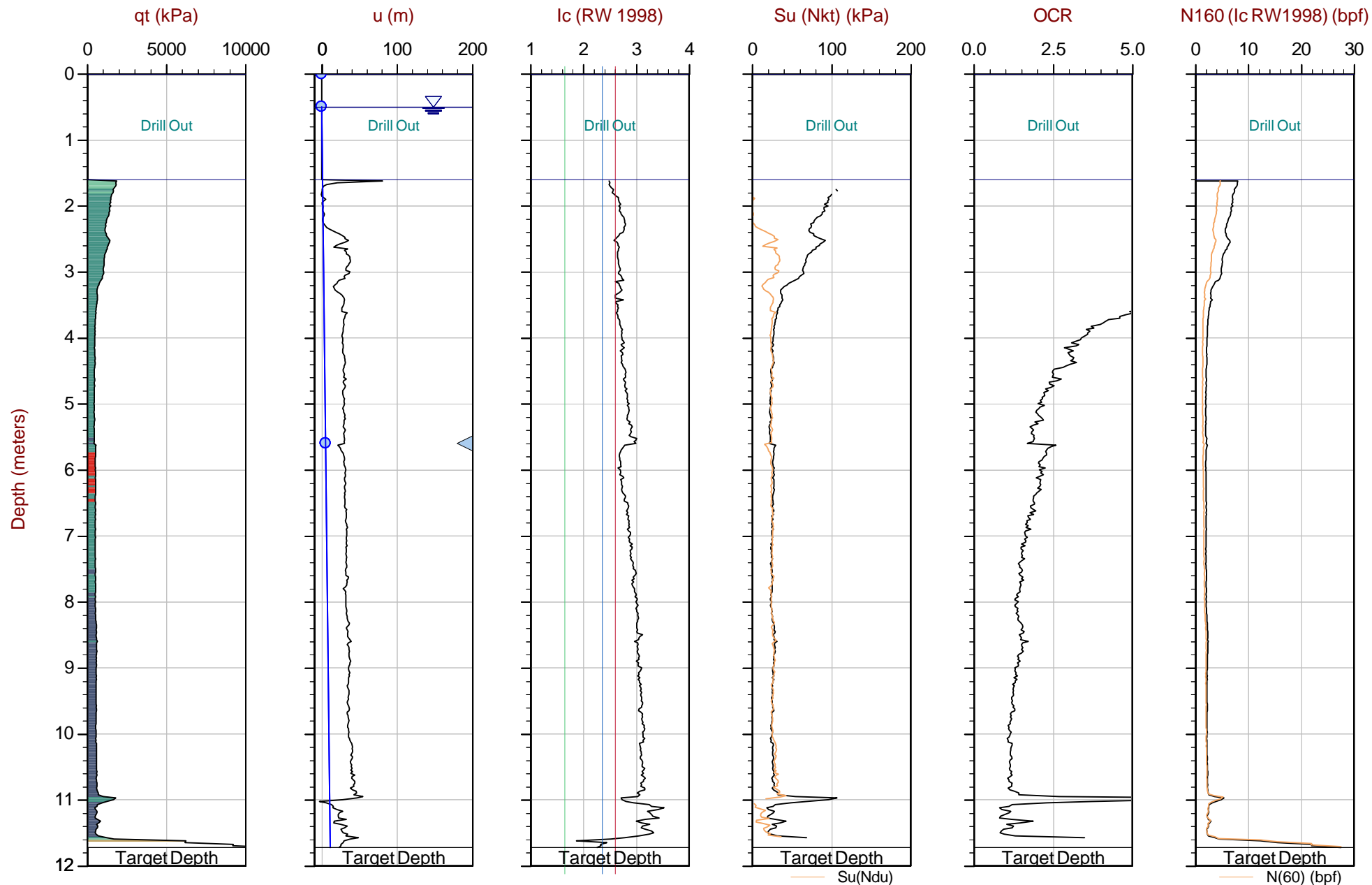
Job No: 17-05066

Date: 2017-11-10 06:52

Site: Lancaster, Ontario

Sounding: CPT17-02

Cone: 465:T1500F15U500



Max Depth: 11.725 m / 38.47 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 17-05066_CP02.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 10.0

SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 4998275m E: 539520m
Sheet No: 1 of 1

● Assumed Ueq ▲ Dissipation, equilibrium assumed — Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Soil Behaviour Type (SBT) Scatter Plots



Golder

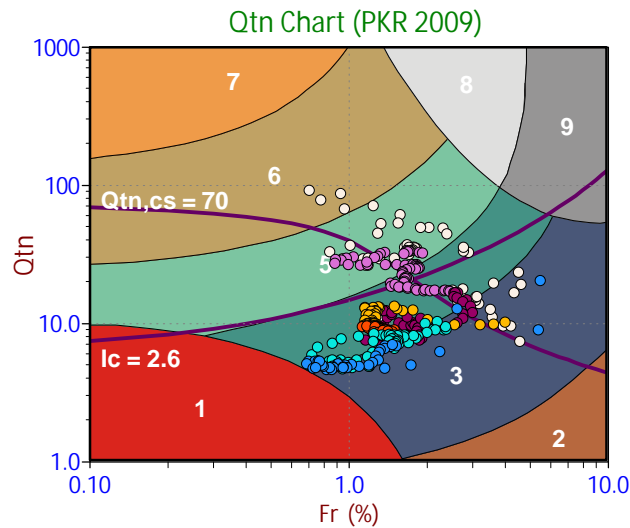
Job No: 17-05066

Date: 2017-11-10 10:16

Site: Lancaster, Ontario

Sounding: CPT17-01

Cone: 465:T1500F15U500

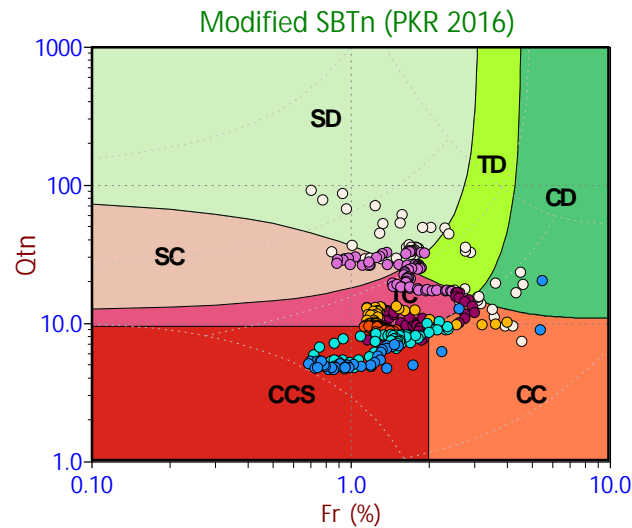


Depth Ranges

- >0.0 to 1.5 m
- >1.5 to 3.0 m
- >3.0 to 4.5 m
- >4.5 to 6.0 m
- >6.0 to 7.5 m
- >7.5 to 9.0 m
- >9.0 to 10.5 m
- >10.5 to 12.0 m
- >12.0 to 13.5 m
- >13.5 to 15.0 m
- >15.0 m

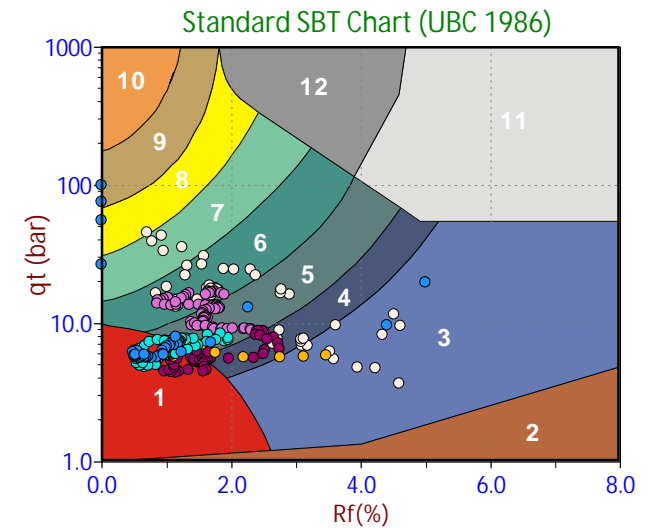
Legend

- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained



Legend

- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)



Legend

- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand



Golder

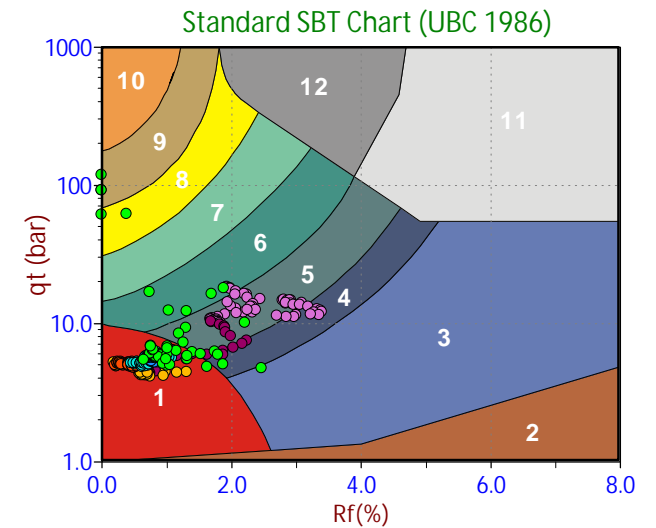
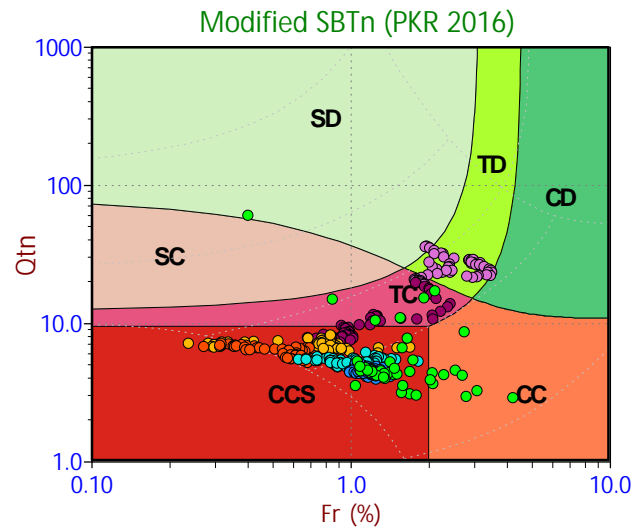
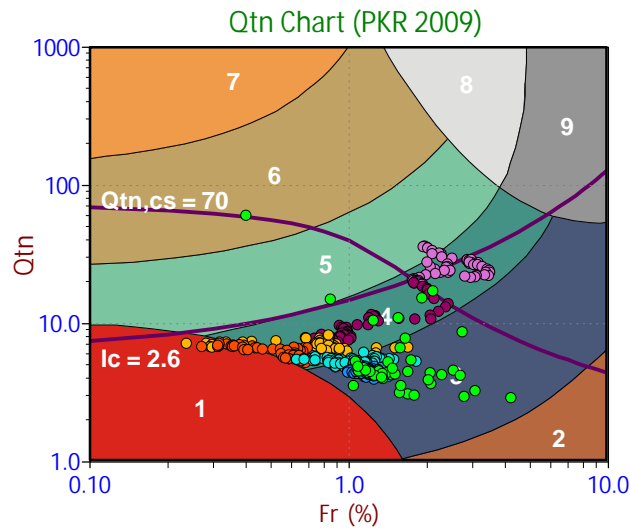
Job No: 17-05066

Date: 2017-11-10 06:52

Site: Lancaster, Ontario

Sounding: CPT17-02

Cone: 465:T1500F15U500



Depth Ranges

- >0.0 to 1.5 m
- >1.5 to 3.0 m
- >3.0 to 4.5 m
- >4.5 to 6.0 m
- >6.0 to 7.5 m
- >7.5 to 9.0 m
- >9.0 to 10.5 m
- >10.5 to 12.0 m
- >12.0 to 13.5 m
- >13.5 to 15.0 m
- >15.0 m

Legend

- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

Legend

- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

Legend

- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots



Job No: 17-05066
Client: Golder Associates
Project: Hwy 401 at 2-34
Start Date: 10-Nov-2017
End Date: 10-Nov-2017

CPT_u PORE PRESSURE DISSIPATION SUMMARY

Sounding ID	File Name	Cone Area (cm ²)	Duration (s)	Test Depth (m)	Estimated Equilibrium Pore Pressure U _{eq} (m)	Calculated Phreatic Surface (m)	Estimated Phreatic Surface (m)	t ₅₀ ^a (s)	Assumed Rigidity Index (I _r)	c _h ^b (cm ² /min)
CPT17-01	17-05066_CP01	15	4300	4.600	4.1		0.5	3018	100	0.2
CPT17-01	17-05066_CP01	15	4100	7.650	7.2		0.5	2182	100	0.3
CPT17-02	17-05066_CP02	15	3245	5.600	5.1		0.5	2493	100	0.3

a. Time is relative to where umax occurred

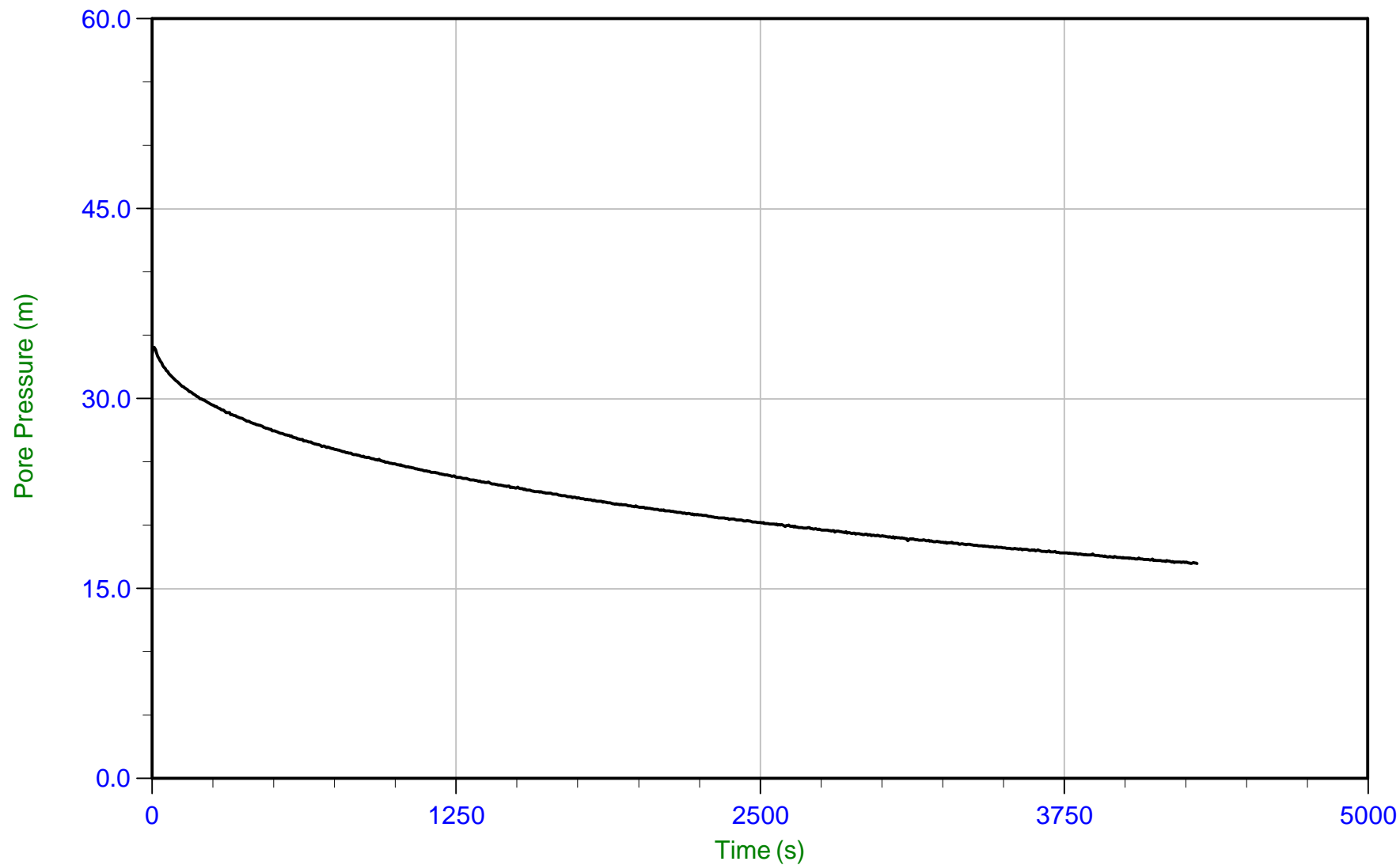
b. Houlsby and Teh, 1991



Golder

Job No: 17-05066
Date: 11/10/2017 10:16
Site: Lancaster, Ontario

Sounding: CPT17-01
Cone: 465:T1500F15U500 Area=15 cm²



Trace Summary: Filename: 17-05066_CP01.PPF
Depth: 4.600 m / 15.092 ft
Duration: 4300.0 s

U Min: 17.0 m
U Max: 34.1 m

WT: 0.500 m / 1.640 ft
Ueq: 4.1 m
U(50): 19.08 m

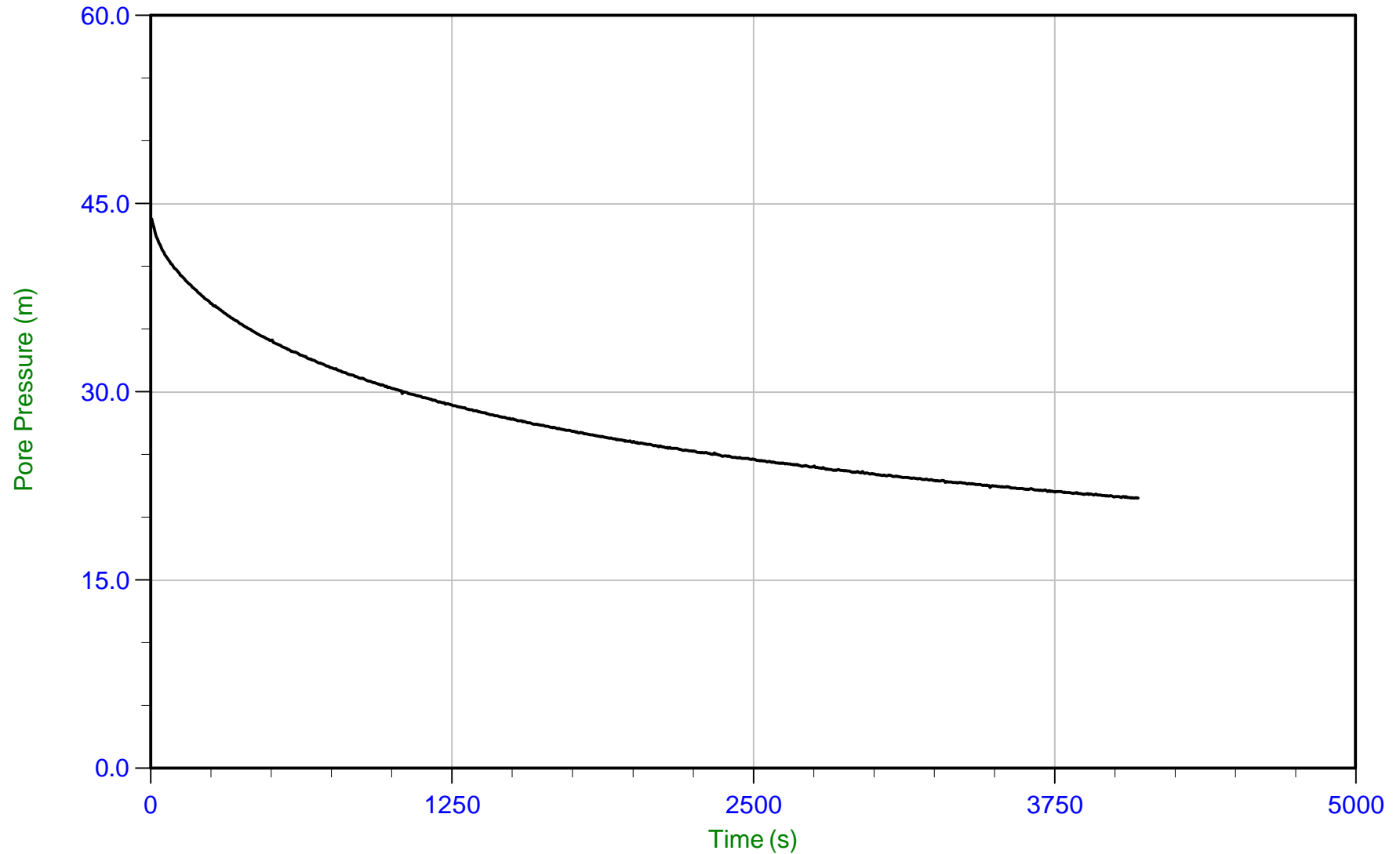
T(50): 3017.9 s
Ir: 100
Ch: 0.2 sq cm/min



Golder

Job No: 17-05066
Date: 11/10/2017 10:16
Site: Lancaster, Ontario

Sounding: CPT17-01
Cone: 465:T1500F15U500 Area=15 cm²



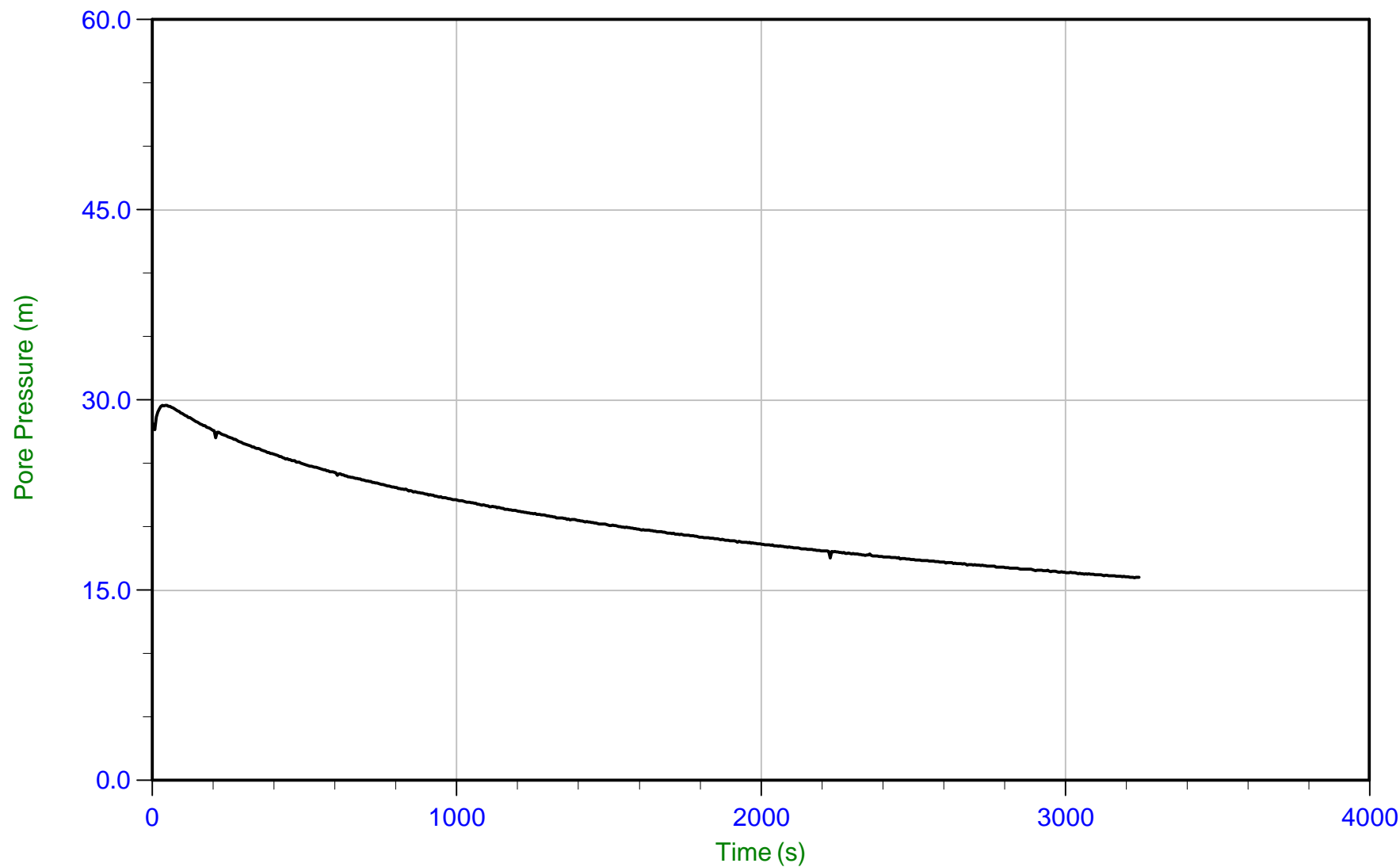
Trace Summary: Filename: 17-05066_CP01.PPF U Min: 21.5 m WT: 0.500 m / 1.640 ft T(50): 2182.1 s
Depth: 7.650 m / 25.098 ft U Max: 43.8 m Ueq: 7.2 m Ir: 100
Duration: 4100.0 s U(50): 25.47 m Ch: 0.3 sq cm/min



Golder

Job No: 17-05066
Date: 11/10/2017 06:52
Site: Lancaster, Ontario

Sounding: CPT17-02
Cone: 465:T1500F15U500 Area=15 cm²



Trace Summary:	Filename: 17-05066_CP02.PPF	U Min: 16.0 m	WT: 0.500 m / 1.640 ft	T(50): 2492.5 s
	Depth: 5.600 m / 18.372 ft	U Max: 29.6 m	Ueq: 5.1 m	Ir: 100
	Duration: 3245.0 s		U(50): 17.34 m	Ch: 0.3 sq cm/min

APPENDIX C

Laboratory Test Results

Figure C1 - Summary of Engineering Properties for the Clay Deposit

Figure C2 - Plasticity Chart - Clay

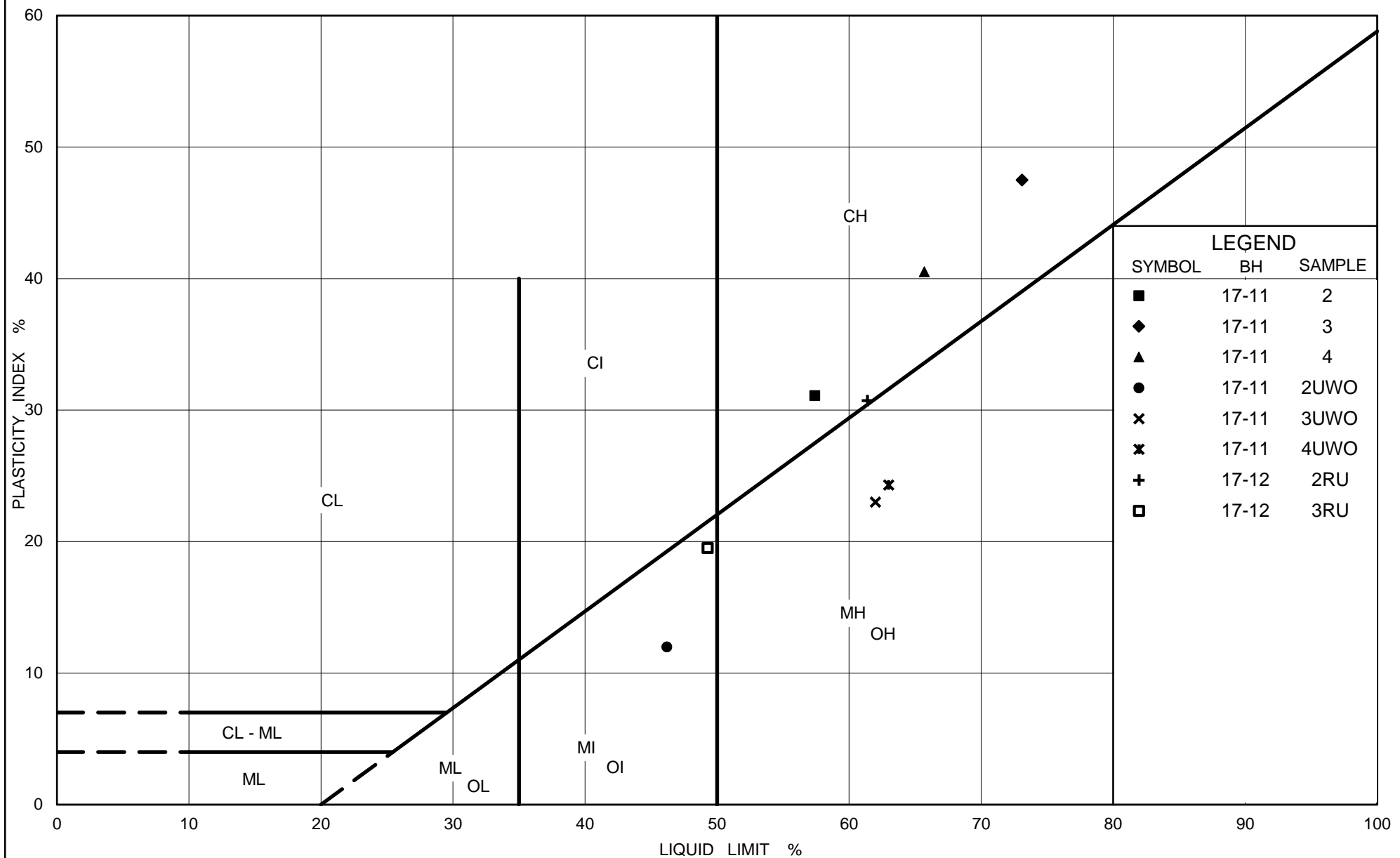
Figures C3 - C5 - Consolidation Test Results - Current Investigation

Figure C6 - Long-Term Consolidation Test Results - Current Investigation

Figures C7 - C9 Constant Rate of Strain (CRS) Test Results— Current Investigation

Figure C10 - Estimated Preconsolidation Pressure Using CPT Data

Table C1 - Atterberg Limits and water content data measure at Golder, UWO and RU laboratories



Ministry of Transportation

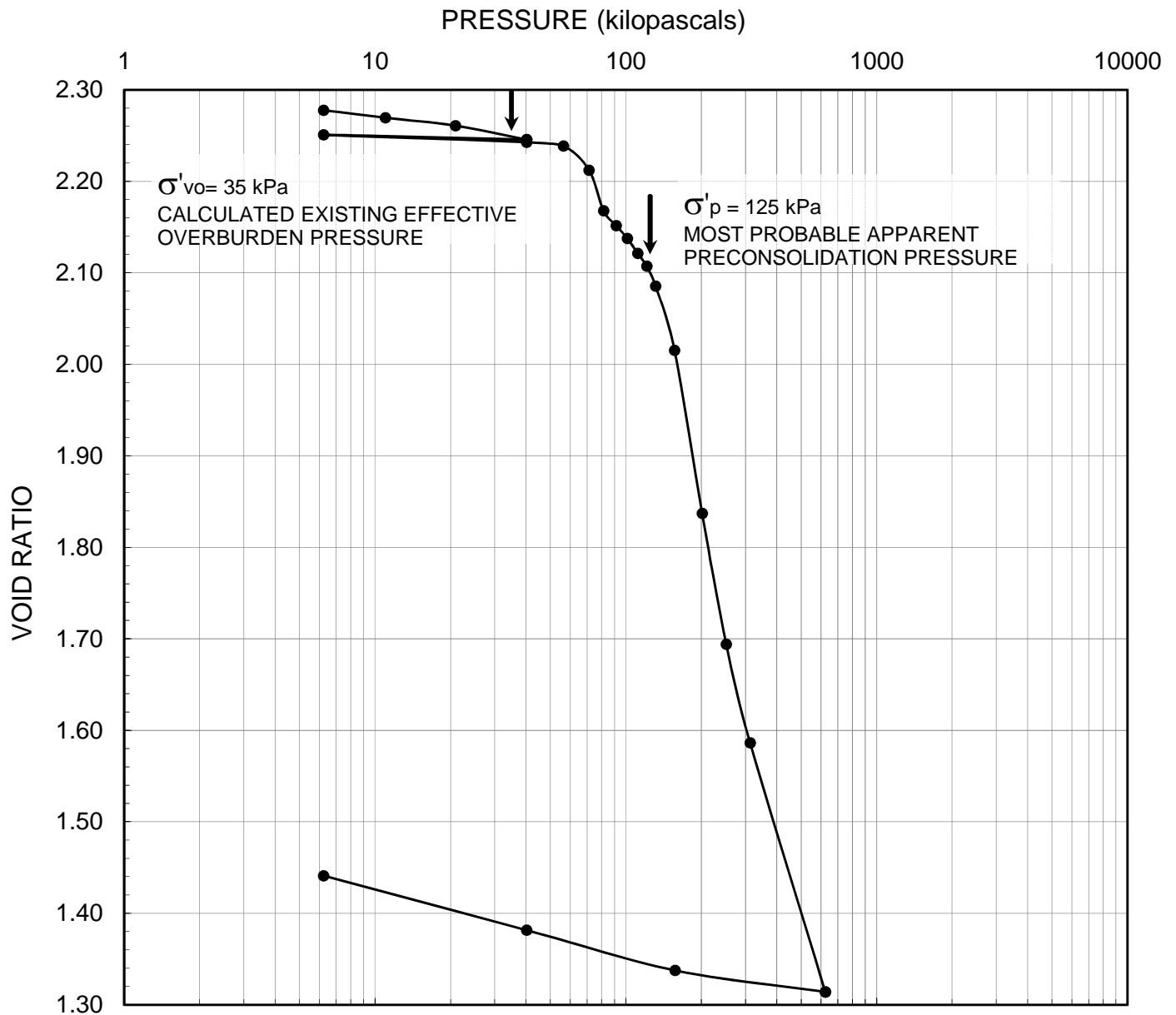
Ontario

PLASTICITY CHART

FIG No. C2

Project No. 1772182 / 1125

Compiled By : CNM Checked By : MI



LEGEND

Borehole: 17-11	$w_i = 82\%$	$S_o = 97\%$	$\gamma = 14.7 \text{ kN/m}^3$
Sample: 2	$w_f = 54\%$	$e_o = 2.29$	$G_s = 2.70$
Depth (m): 4.8	$w_l = 57\%$	$C_c = 1.55$	
Elevation (m): 43.3	$w_p = 26\%$	$C_r = 0.010$	



SCALE	AS SHOWN
DATE	04/03/18
CADD	N/A
ENTERED	CNM

TITLE

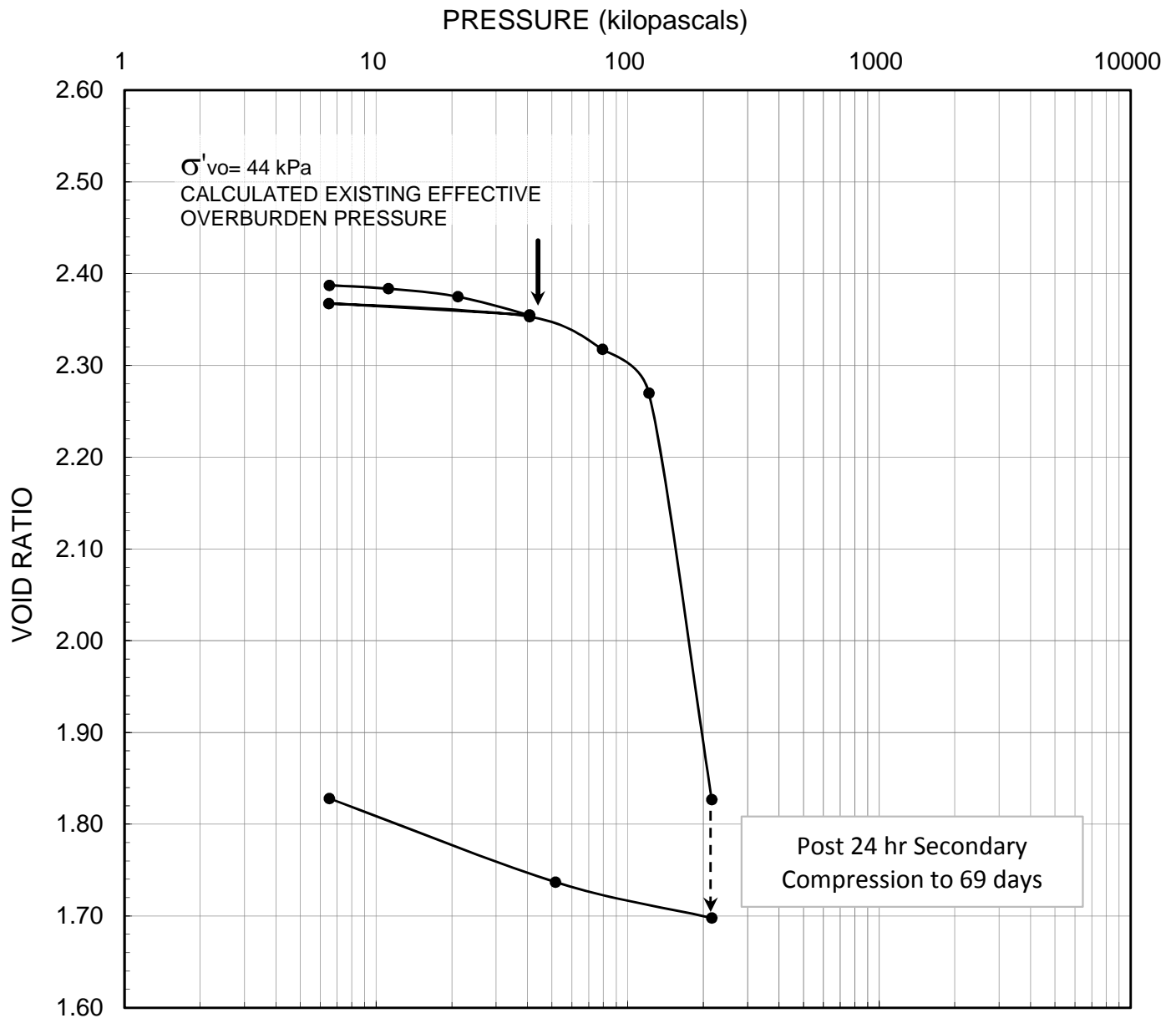
CONSOLIDATION TEST RESULTS

FILE No.	Consolidation summary
PROJECT No.	1772182 / 1125
REV.	2

CHECK	MI
REVIEW	SS

FIGURE

C3



LEGEND

Borehole: 17-11	$w_i = 87\%$	$S_o = 100\%$	$\gamma = 14.9 \text{ kN/m}^3$
Sample: 3	$w_f = 72\%$	$e_o = 2.39$	$G_s = 2.75$
Depth (m): 6.4	$w_l = 73\%$	$C_c = \text{N/A}$	
Elevation (m): 41.7	$w_p = 26\%$	$C_r = 0.010$	



SCALE	AS SHOWN
DATE	04/03/18
CADD	N/A
ENTERED	CNM
CHECK	MI
REVIEW	SS

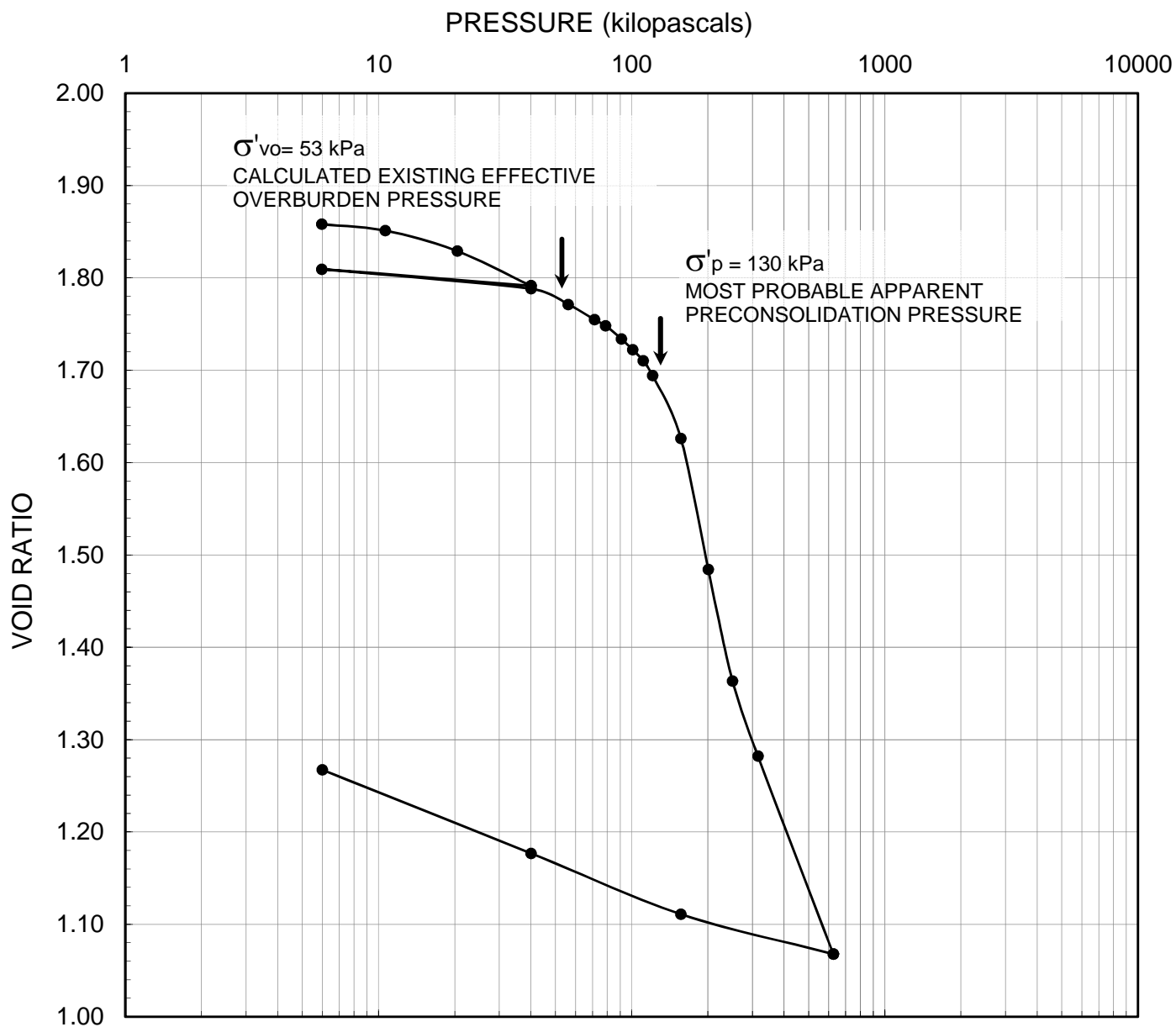
TITLE

CONSOLIDATION TEST RESULTS

FILE No.	Consolidation summary
PROJECT No.	1772182 / 1125
REV.	1

FIGURE

C4



LEGEND

Borehole: 17-11	$w_i = 68\%$	$S_o = 99\%$	$\gamma = 15.6 \text{ kN/m}^3$
Sample: 4	$w_f = 48\%$	$e_o = 1.86$	$G_s = 2.71$
Depth (m): 7.9	$w_l = 66\%$	$C_c = 1.30$	
Elevation (m): 40.2	$w_p = 25\%$	$C_r = 0.025$	



SCALE	AS SHOWN
DATE	04/03/18
CADD	N/A
ENTERED	CNM

TITLE

CONSOLIDATION TEST RESULTS

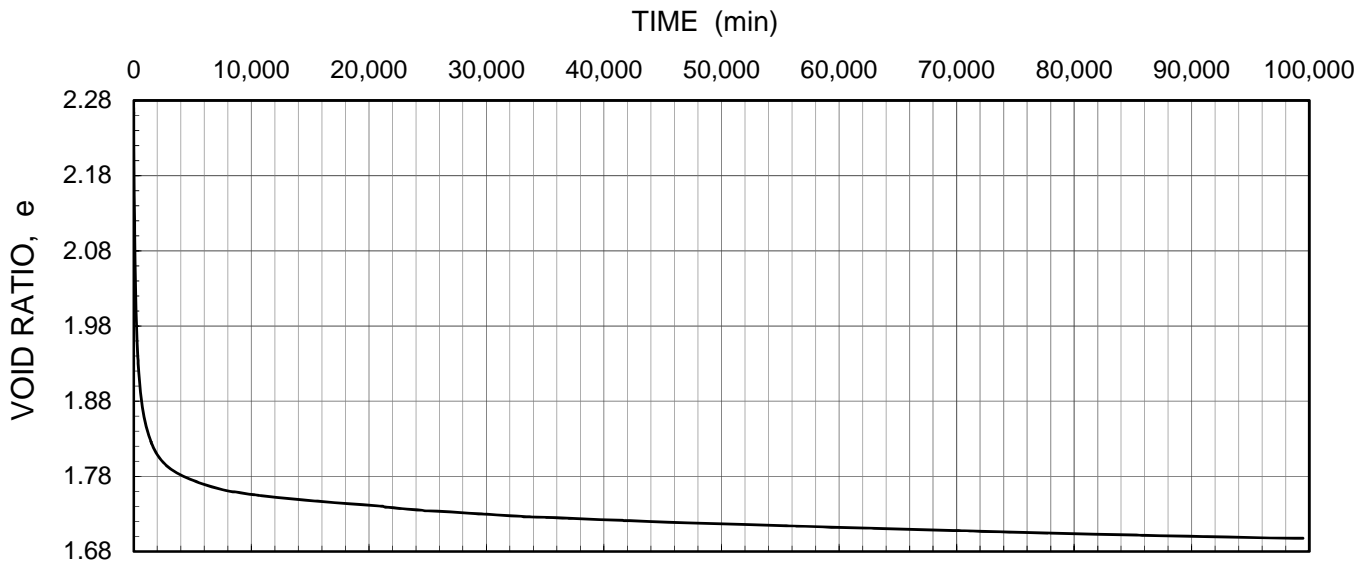
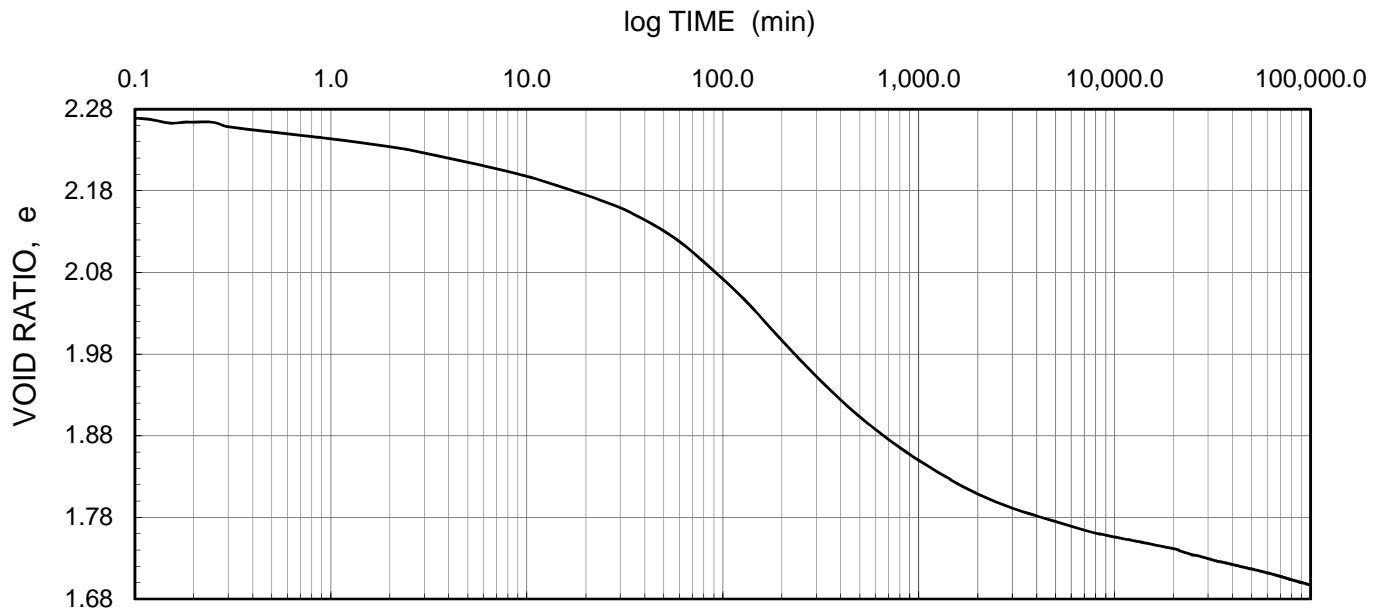
FILE No.	Consolidation summary
PROJECT No.	1772182 / 1125
REV.	2

CHECK	MI
REVIEW	SS

FIGURE

C5

PRESSURE = 216 kPa



LEGEND

Borehole: 17-11	$w_i = 87\%$	$S_o = 100\%$
Sample: 3	$w_f = 72\%$	$C_\alpha = 0.049$
Depth (m): 6.4	$w_l = 73\%$	
	$w_p = 26\%$	



SCALE	AS SHOWN
DATE	04/03/18
DESIGN	N/A
CADD	CNM
CHECK	MI
REVIEW	SS

TITLE

**SUMMARY OF
SECONDARY COMPRESSION TEST**

FILE No. Consolidation summary

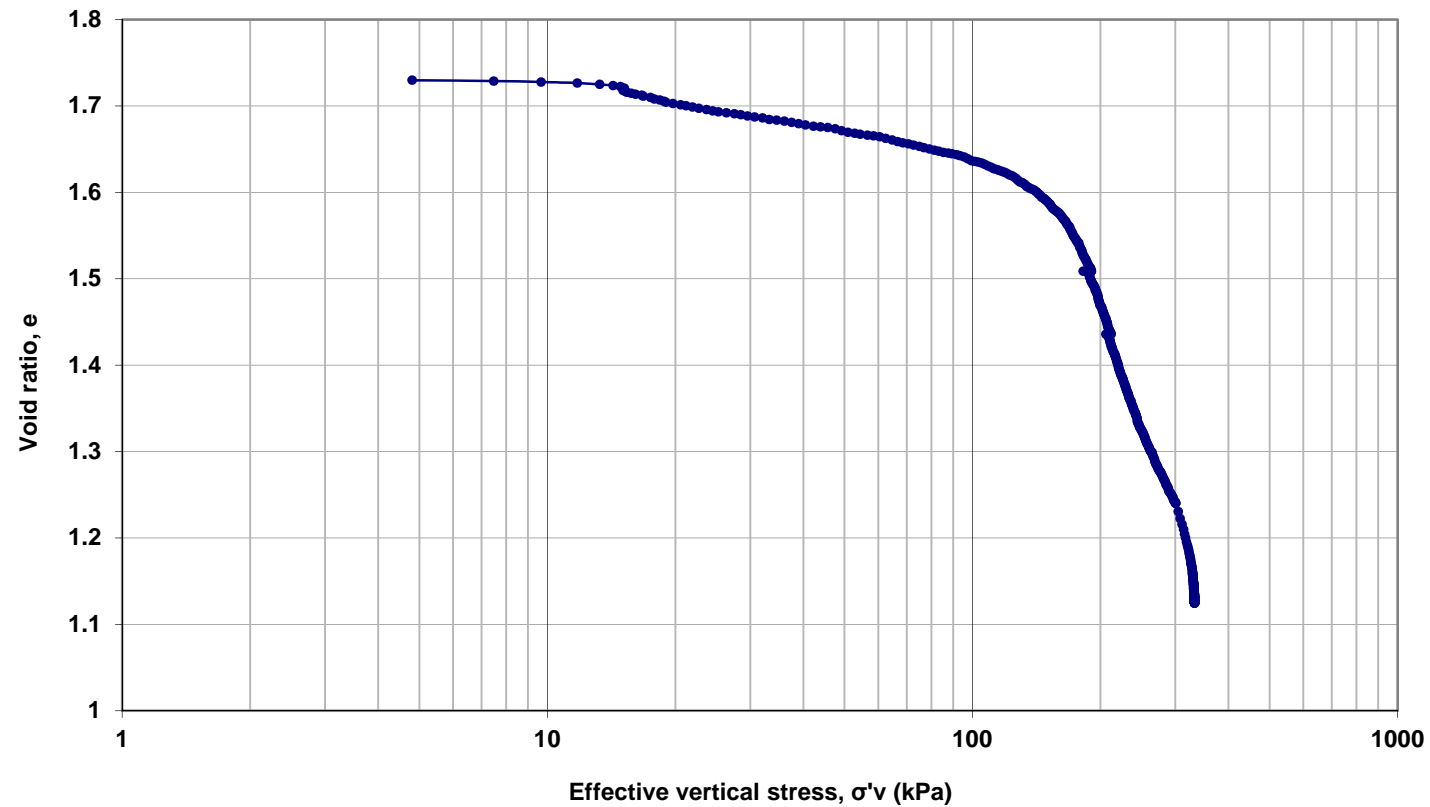
PROJECT No. 1772182 /1125 REV. 1

FIGURE

C6

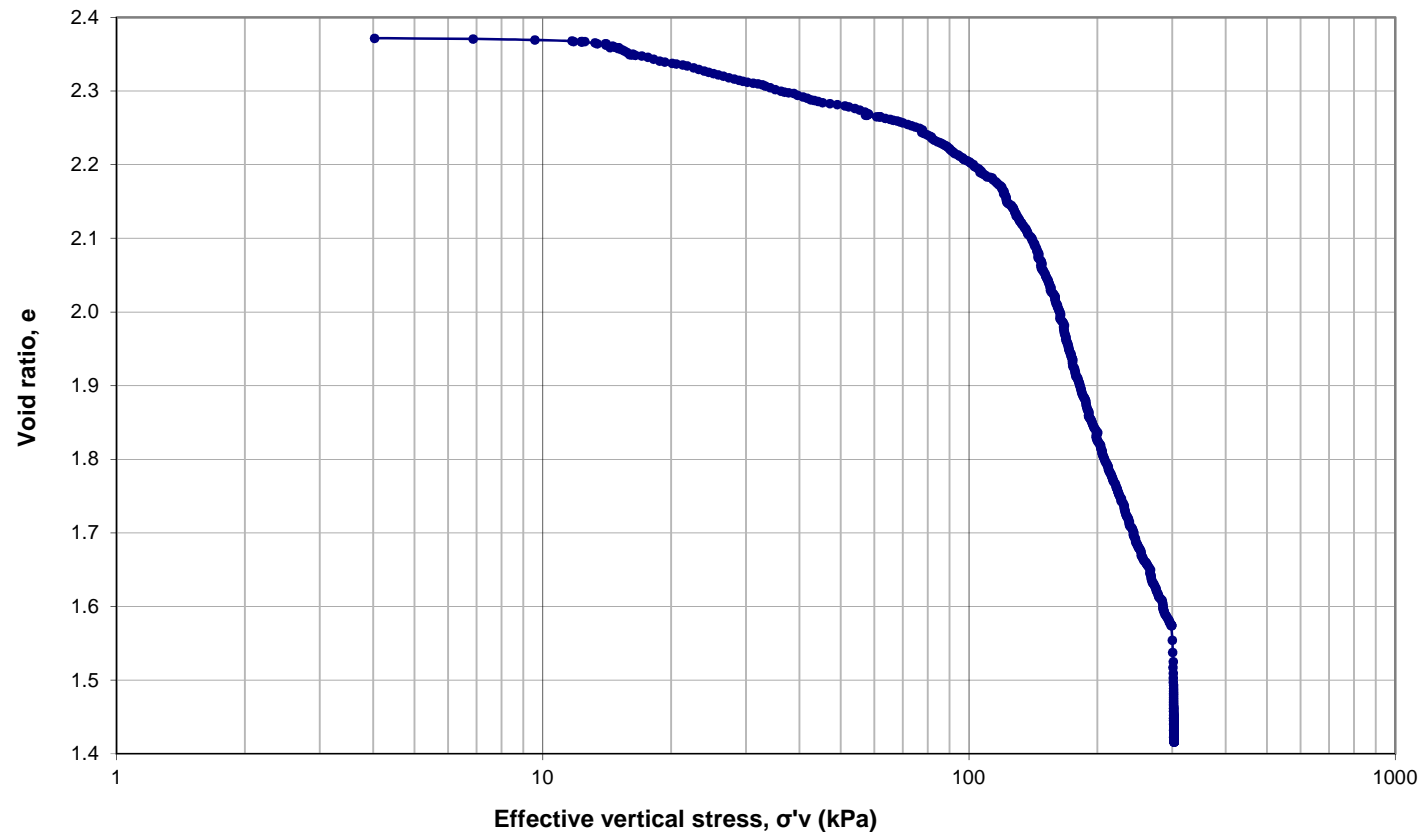
CRS TEST RESULTS
BOREHOLE 17-11, SAMPLE #2

FIGURE C7



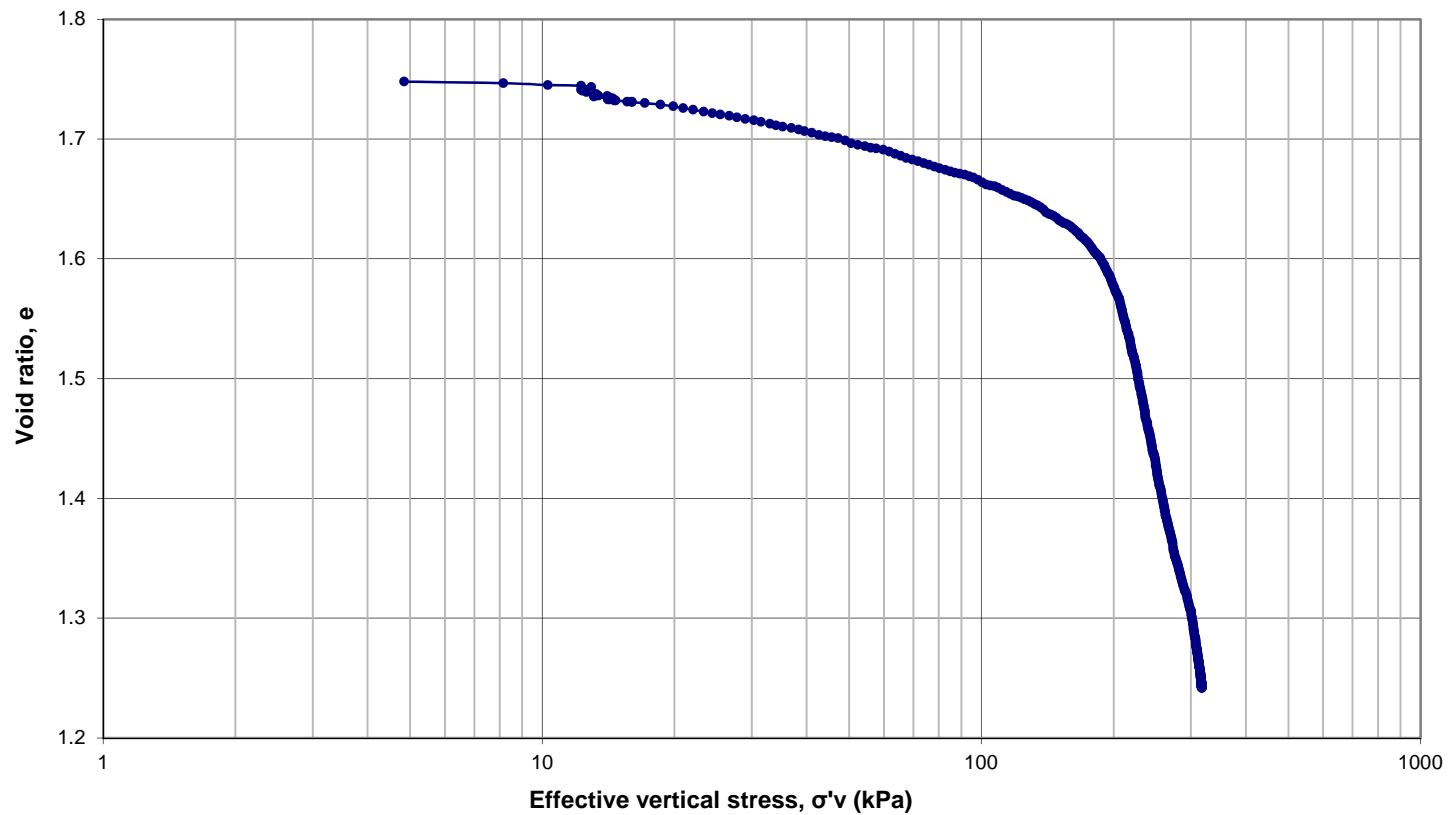
CRS TEST RESULTS
BOREHOLE 17-11, SAMPLE #3

FIGURE C8



CRS TEST RESULTS
BOREHOLE 17-11, SAMPLE #4

FIGURE C9

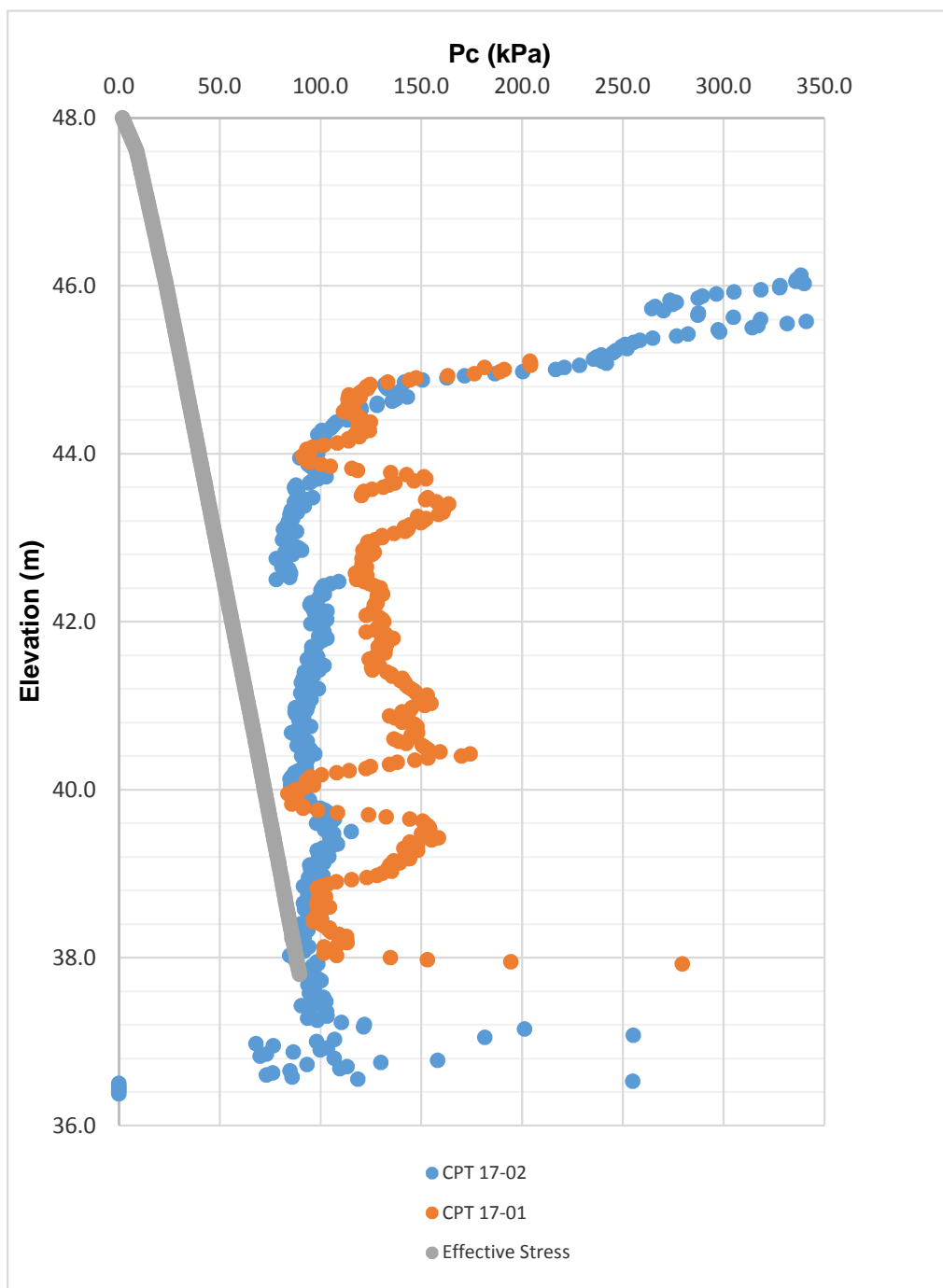


Western
Engineering

Civil & Environmental Engineering

ESTIMATED PRECONSOLIDATION PRESSURE BASED ON CPT RESULTS

FIGURE C10



Date March 05, 2018
Project 1772182/1125

Golder Associates

Drawn SS
Chkd WC

TABLE C1**SUMMARY OF WATER CONTENT AND ATTERBERG LIMIT DETERMINATIONS**

PROJECT NUMBER		1772182 /1125					
PROJECT NAME		Dillon/ Mega 6 RET-ER-E/ CR2+34 / Lancaster, Ontario					
DATE TESTED		09-Jan-18					
Borehole No.	Sample No.	Depth (m)	Water Content (%)	Atterberg Limits			
				W _L	W _P	LI	PI
<u>Golder Mississauga Lab :</u>							
17-11	2	4.57-5.13	82.1%	57.4	26.3	1.8	31.1
17-11	3	6.10-6.65	87.4%	73.1	25.6	1.3	47.5
17-11	4	7.62-8.23	67.8%	65.7	25.2	1.1	40.5
<u>University of Western Ontario Lab :</u>							
17-11	2	4.57-5.13	63.3%	46.2	34.2	2.4	12.0
17-11	3	6.10-6.65	87.0%	62.0	39.1	2.1	23.0
17-11	4	7.62-8.23	60.5%	63.0	38.8	0.9	24.3
<u>Ryerson University Lab:</u>							
17-12	1	3.66-5.49	61.6%	61.4	30.8	1.0	30.7
17-12	2	5.49-7.32	64.8%	49.3	29.8	1.8	19.5
17-12	3	7.32-9.14	65.5%				

APPENDIX D

**Record of Previous Boreholes 16-1 To 16-9 and 17-10
(Geocres No. 31G-259)**

PROJECT 12-1121-0193-1140		RECORD OF BOREHOLE No 16-1				SHEET 2 OF 3		METRIC								
G.W.P. 4013-11-01		LOCATION N 5000079.4; E 226388.0 MTM ZONE (LAT. ; LONG.)				ORIGINATED BY DG										
DIST Eastern HWY 401		BOREHOLE TYPE Power Auger, 200 mm Diam. (Hollow Stem)/Rotary Drill, NQ Core				COMPILED BY JM										
DATUM Geodetic		DATE September 19 and 20, 2016				CHECKED BY KSL										
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 ---																
35.1	Gravelly SAND, some silt, trace clay, contains cobbles and boulders Compact to dense Grey Wet	A	12	SS	14											
13.4	Limestone (BEDROCK) Bedrock cored from 13.4 m depth to 17.5 m depth. For coring details see Record of Drillhole 16-1.	B	1	RC	REC 32.5%											
			2	RC	REC 100%											
			3	RC	REC 100%											
31.1	END OF BOREHOLE															
17.5	Note: 1. Water level in piezometer at a depth of 1.1 m below ground surface (Elev. 47.4 m) on July 29, 2017.															

PROJECT: 12-1121-0193-1140

RECORD OF DRILLHOLE: 16-1

SHEET 3 OF 3

LOCATION: N 5000079.4 ;E 226388.0

DRILLING DATE: September 19 and 20, 2016

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 850

DRILLING CONTRACTOR: CCC

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY														FEATURES																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																									
						FLUSH RETURN	RECOVERY		R.Q.D. %	FRACT. INDEX PER	DIP w.r.t. CORE AXIS	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY K, cm/sec		WEATH- ERING INDEX																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													
							TOTAL CORE %	SOLID CORE %				Jr	Js	10 ⁻⁶	10 ⁻⁵	10 ⁻⁴	10 ⁻³	W1	W2		W3	W4	W5	W6																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																					
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		Continued from Record of Borehole 16-1		35.11																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																									</

DEPTH SCALE

1 : 60



LOGGED: DG

CHECKED:




GTA-RCK 031 N:\ACTIVE\2012\1121 - GEOTECHNICAL\12-1121-0193 DILLON MEGA 3 EASTERN REGION\SPATIAL_IMG\INT\PHASE 1140\1211210193-1140.GPJ GAL-MISS GDT 9/12/17 J.L

PROJECT	12-1121-0193-1140	RECORD OF BOREHOLE No 16-2		SHEET 1 OF 3	METRIC
G.W.P.	4013-11-01	LOCATION	N 5000109.7; E 226374.7 MTM ZONE (LAT. ; LONG.)	ORIGINATED BY	DG
DIST	Eastern	HWY	401	BOREHOLE TYPE	Power Auger, 200 mm Diam. (Hollow Stem)/Rotary Drill, NQ Core
DATUM	Geodetic	DATE	September 23 and 26, 2016	COMPILED BY	JM
				CHECKED BY	KSL

[illegible]

Continued Next Page

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

PROJECT		RECORD OF BOREHOLE No 16-2				SHEET 2 OF 3		METRIC									
12-1121-0193-1140																	
G.W.P. 4013-11-01		LOCATION N 5000109.7; E 226374.7 MTM ZONE (LAT. ; LONG.)				ORIGINATED BY DG											
DIST Eastern HWY 401		BOREHOLE TYPE Power Auger, 200 mm Diam. (Hollow Stem)/Rotary Drill, NQ Core				COMPILED BY JM											
DATUM Geodetic		DATE September 23 and 26, 2016				CHECKED BY KSL											
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 ---																
35.9	Silty SAND, some gravel, trace clay (TILL) Compact Grey Wet		13	SS	18												
13.0	SAND and GRAVEL, some silt to silty Compact to dense Grey Wet		14	SS	25												
34.9			15	SS	50/0.15												
13.9	Limestone (BEDROCK) Bedrock cored from 13.9 m depth to 16.9 m depth. For coring details see Record of Drillhole 16-2.		1	RC	REC 100%												RQD = 85%
			2	RC	REC 100%												RQD = 98%
31.9	END OF BOREHOLE																
16.9	Note: 1. Water level at a depth of 3.8 m below ground surface (Elev. 45.0 m) upon completion of drilling.																

GTA-MTO 001 N:\ACTIVE\2012\1121 - GEOTECHNICAL\12-1121-0193 DILLON MEGA 3 EASTERN REGION\SPATIAL_IM\GINT\PHASE 1140\11210193-1140.GPJ GAL-GTA.GDT 9/12/17 JUL

SHEET 3 OF 3

DATUM: Geodetic

DRILLING CONTRACTOR: CCC

[illegible]

DEPTH SCALE

1 : 60



LOGGED: DG

CHECKED:

PROJECT 12-1121-0193-1140				RECORD OF BOREHOLE No 16-3				SHEET 2 OF 3				METRIC					
G.W.P. 4013-11-01				LOCATION N 5000049.9; E 226406.2 MTM ZONE (LAT. ; LONG.)				ORIGINATED BY DG									
DIST Eastern HWY 401				BOREHOLE TYPE Power Auger, 200 mm Diam. (Hollow Stem)/Rotary Drill, NQ Core				COMPILED BY JM									
DATUM Geodetic				DATE September 28, 2016				CHECKED BY KSL									
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 (%)				
	--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100	W _p	W	W _L		
35.9			13	SS	50/0.3		36										
12.6	Limestone (BEDROCK) Bedrock cored from 12.6 m depth to 15.6 m depth. For coring details see Record of Drillhole 16-3.		1	RC	REC 100%		35										RQD = 86%
			2	RC	REC 100%		34										RQD = 92%
32.8	END OF BOREHOLE Note: 1. Water level at a depth of 5.0 m below ground surface (Elev. 43.4 m) upon completion of drilling.						33										
15.6																	

INCLINATION: -90° AZIMUTH: --

DRILL RIG: CME 850
DRILLING CONTRACTOR: CCC

DATUM: Geodetic

GTA-RCK 031 N:\ACTIVE\2012\121 - GEOTECHNICAL\12-121-0193 DILLON MEGA 3 EASTERN REGION\SPATIAL_IMG\INT\PHASE 1140\121210193-1140.GPJ GAL-MISS.GDT 9/12/17 JUL

1 : 60



PROJECT 12-1121-0193-1140		RECORD OF BOREHOLE No 16-4		SHEET 1 OF 2		METRIC	
G.W.P. 4013-11-01		LOCATION N 5000127.4; E 226361.5 MTM ZONE (LAT. ; LONG.)				ORIGINATED BY DG	
DIST Eastern HWY 401		BOREHOLE TYPE Power Auger, 200 mm Diam. (Hollow Stem)				COMPILED BY JM	
DATUM Geodetic		DATE September 22, 2016				CHECKED BY KSL	

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							
								○ UNCONFINED	+	FIELD VANE					
								● QUICK TRIAXIAL	×	REMOULDED					
								WATER CONTENT (%)							
48.4	GROUND SURFACE														
0.0	Sandy silt (TOPSOIL/FILL)														
0.1	Dark brown Moist		1	SS	10										
	Silty clay, some sand and gravel, contains rootlets (FILL)														
47.5	Grey-brown Moist														
0.9	CLAY (Weathered Crust)		2	SS	17										
	Stiff to very stiff Grey-brown Moist														
			3	SS	9										
			4	SS	5										
			5	SS	1										
44.8	CLAY														
3.6	Firm Grey with black organic mottling Wet														
			6	SS	WH										
			7	SS	WH										
			8	SS	WH										
			9	SS	WH										
38.0	Sandy SILT														
10.4	Very loose Grey Wet		10	SS	3										
37.1															
11.3			11	SS	WH										

Continued Next Page

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

PROJECT <u>12-1121-0193-1140</u>		RECORD OF BOREHOLE No 16-4		SHEET 2 OF 2		METRIC	
G.W.P. <u>4013-11-01</u>		LOCATION <u>N 5000127.4; E 226361.5 MTM ZONE (LAT. ; LONG.)</u>		ORIGINATED BY <u>DG</u>			
DIST <u>Eastern</u> HWY <u>401</u>		BOREHOLE TYPE <u>Power Auger, 200 mm Diam. (Hollow Stem)</u>		COMPILED BY <u>JM</u>			
DATUM <u>Geodetic</u>		DATE <u>September 22, 2016</u>		CHECKED BY <u>KSL</u>			

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					WATER CONTENT (%)				GR	SA	SI	CL	
								○ UNCONFINED + FIELD VANE	● QUICK TRIAXIAL × REMOULDED												
	--- CONTINUED FROM PREVIOUS PAGE ---																				
	Silty SAND, some gravel, trace clay, contains cobbles and boulders (TILL) Compact Grey Wet		12	SS	12																
			13	SS	12																
34.7																					
34.4	SAND and GRAVEL, some silt, trace clay (TILL) Very dense Wet		14	SS	50/0.2												46	36			
14.0	END OF BOREHOLE AUGER REFUSAL																15	3			
	Note: 1. Water level at a depth of 4.2 m below ground surface (Elev. 44.2 m) upon completion of drilling.																				

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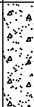
PROJECT 12-1121-0193-1140		RECORD OF BOREHOLE No 16-5		SHEET 1 OF 2		METRIC	
G.W.P. 4013-11-01		LOCATION N 5000033.5; E 226414.2 MTM ZONE (LAT. ; LONG.)		ORIGINATED BY DG			
DIST Eastern HWY 401		BOREHOLE TYPE Power Auger, 200 mm Diam. (Hollow Stem)		COMPILED BY JM			
DATUM Geodetic		DATE September 27, 2016		CHECKED BY KSL			

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 (%) 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	20 40 60	W _p W W _L						
48.1	GROUND SURFACE																
0.0	Silty sand (TOPSOIL)																
0.2	Dark brown Moist		1	SS	10												
47.3	CLAY, contains rootlets (Weathered Crust)		2	SS	7												
0.8	Stiff to very stiff Grey-brown Moist																
	CLAY (Weathered Crust)		3	SS	6												
	Stiff to very stiff Grey-brown Moist		4	SS	1												
45.1	CLAY		5	SS	WH												
3.1	Firm Grey Wet																
43.5	CLAY		6	SS	WH												
4.6	Firm Grey with black organic mottling Wet																
			7	SS	WH												
			8	SS	WH												
39.0	CLAY		9	SS	WH												
9.1	Firm Grey Wet																
38.2	Gravelly SAND, some silt, trace clay Compact Grey Wet		10	SS	15												
9.9			11	SS	14												
			12	SS	17												

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+³, X³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

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PROJECT		RECORD OF BOREHOLE No 16-5				SHEET 2 OF 2		METRIC									
12-1121-0193-1140																	
G.W.P. 4013-11-01		LOCATION N 5000033.5; E 226414.2 MTM ZONE (LAT. ; LONG.)				ORIGINATED BY DG											
DIST Eastern HWY 401		BOREHOLE TYPE Power Auger, 200 mm Diam. (Hollow Stem)				COMPILED BY JM											
DATUM Geodetic		DATE September 27, 2016				CHECKED BY KSL											
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 (%)
	--- CONTINUED FROM PREVIOUS PAGE ---						20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					20 40 60				GR SA SI CL	
35.2	Gravelly SAND, some silt, trace clay Compact Grey Wet		13	SS	30		36								o		
12.9	END OF BOREHOLE Note: 1. Water level at a depth of 2.3 m below ground surface (Elev. 45.8 m) upon completion of drilling.																

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

PROJECT <u>12-1121-0193-1140</u>		RECORD OF BOREHOLE No 16-6		SHEET 1 OF 1		METRIC	
G.W.P. <u>4013-11-01</u>		LOCATION <u>N 5000055.0; E 226344.1 MTM ZONE (LAT. ; LONG.)</u>		ORIGINATED BY <u>DG</u>			
DIST <u>Eastern</u> HWY <u>401</u>		BOREHOLE TYPE <u>Power Auger, 200 mm Diam. (Hollow Stem)</u>		COMPILED BY <u>JM</u>			
DATUM <u>Geodetic</u>		DATE <u>September 21, 2016</u>		CHECKED BY <u>KSL</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	20	40	60	80	100	w _p	w		w _L			
48.6	GROUND SURFACE																			
0.0	Silty sand (TOPSOIL/FILL) Dark brown Moist		1	SS	7															
0.2	Silty clay, trace gravel, contains rootlets and organic matter (FILL) Brown to dark grey-brown Moist																			
			2	SS	44															
			3	SS	11															
46.5	CLAY (Weathered Crust) Stiff to very stiff Grey-brown Moist																			
2.1			4	SS	4															
			5	SS	WH															
45.0	CLAY Firm Grey with black organic mottling Wet																			
3.6																				
			6	SS	WH															
			7	SS	WH															
41.3	END OF BOREHOLE																			
7.3	Note: 1. Water level at a depth of 5.5 m below ground surface (Elev. 43.1 m) upon completion of drilling.																			

PROJECT 12-1121-0193-1140		RECORD OF BOREHOLE No 16-8		SHEET 1 OF 1		METRIC	
G.W.P. 4013-11-01		LOCATION N 5000109.9; E 226436.1 MTM ZONE (LAT. ; LONG.)		ORIGINATED BY DG			
DIST Eastern HWY 401		BOREHOLE TYPE Power Auger, 200 mm Diam. (Hollow Stem)		COMPILED BY JM			
DATUM Geodetic		DATE September 21, 2016		CHECKED BY KSL			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE LIQUID CONTENT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL
												20	40	60	80					
48.5	GROUND SURFACE																			
0.0	Silty clay, contains rootlets (FILL) Grey-brown Moist			1	SS	6														
47.9	Silty clay, some gravel (FILL) Grey-brown Moist																			
0.6																				
47.4	Silty sand (FILL) Loose Grey-brown Moist			2	SS	16								o						
1.1																				
46.5				3	SS	8														
2.0	CLAY (Weathered Crust) Stiff to very stiff Grey-brown Moist																			
				4	SS	4														
				5	SS	2														
44.9	CLAY Firm Grey with black organic mottling Wet																			
3.6																				
				6	SS	WH														
				7	SS	WH														
41.2	END OF BOREHOLE																			
7.3	Note: 1. Water level at a depth of 6.3 m below ground surface (Elev. 42.2 m) upon completion of drilling.																			

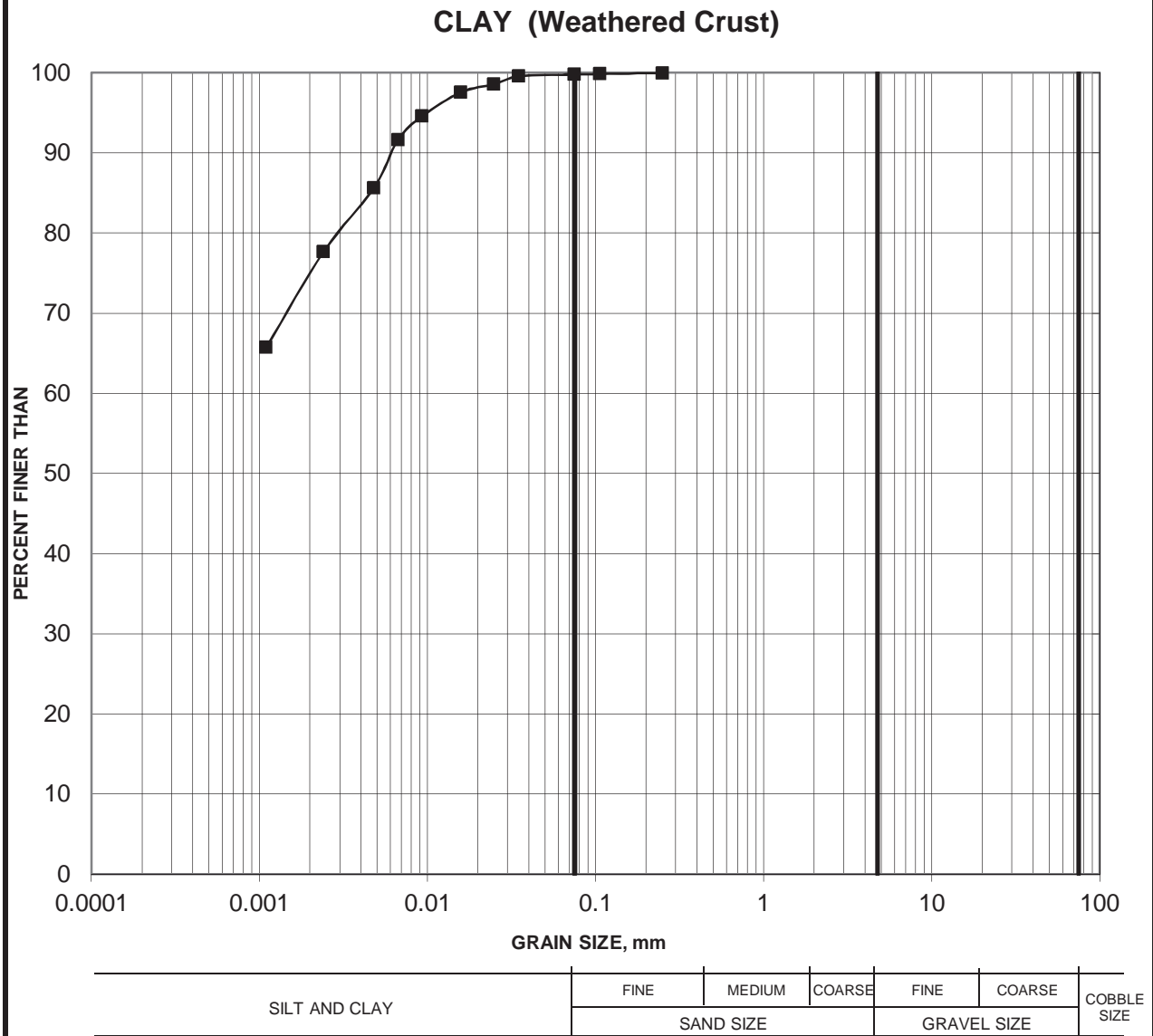
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DIST Eastern HWY 401		BOREHOLE TYPE Power Auger, 200 mm Diam. (Hollow Stem)		COMPILED BY JM			
DATUM Geodetic		DATE September 21, 2016		CHECKED BY KSL			

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 (%) 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	20 40 60	W _p W W _L						
48.4	GROUND SURFACE																
0.9	Silty sand (TOPSOIL/FILL) Dark brown Moist		1	SS	13												
47.8	Silty clay, trace gravel, contains rootlets (FILL) Dark brown Moist																
0.6	Sandy silt (FILL) Loose Dark grey to grey Moist		2	SS	8					○							
46.7	CLAY (WEATHERED CRUST) Stiff to very stiff Grey-brown Moist																
1.7			3	SS	6												
			4	SS	4							○					
45.1	CLAY Firm Grey with black organic mottling Wet		5	SS	WH												
3.3																	
			6	SS	WH												
			7	SS	WH												
41.1	END OF BOREHOLE																
7.3	Note: 1. Water level at a depth of 5.5 m below ground surface (Elev. 42.9 m) upon completion of drilling.																

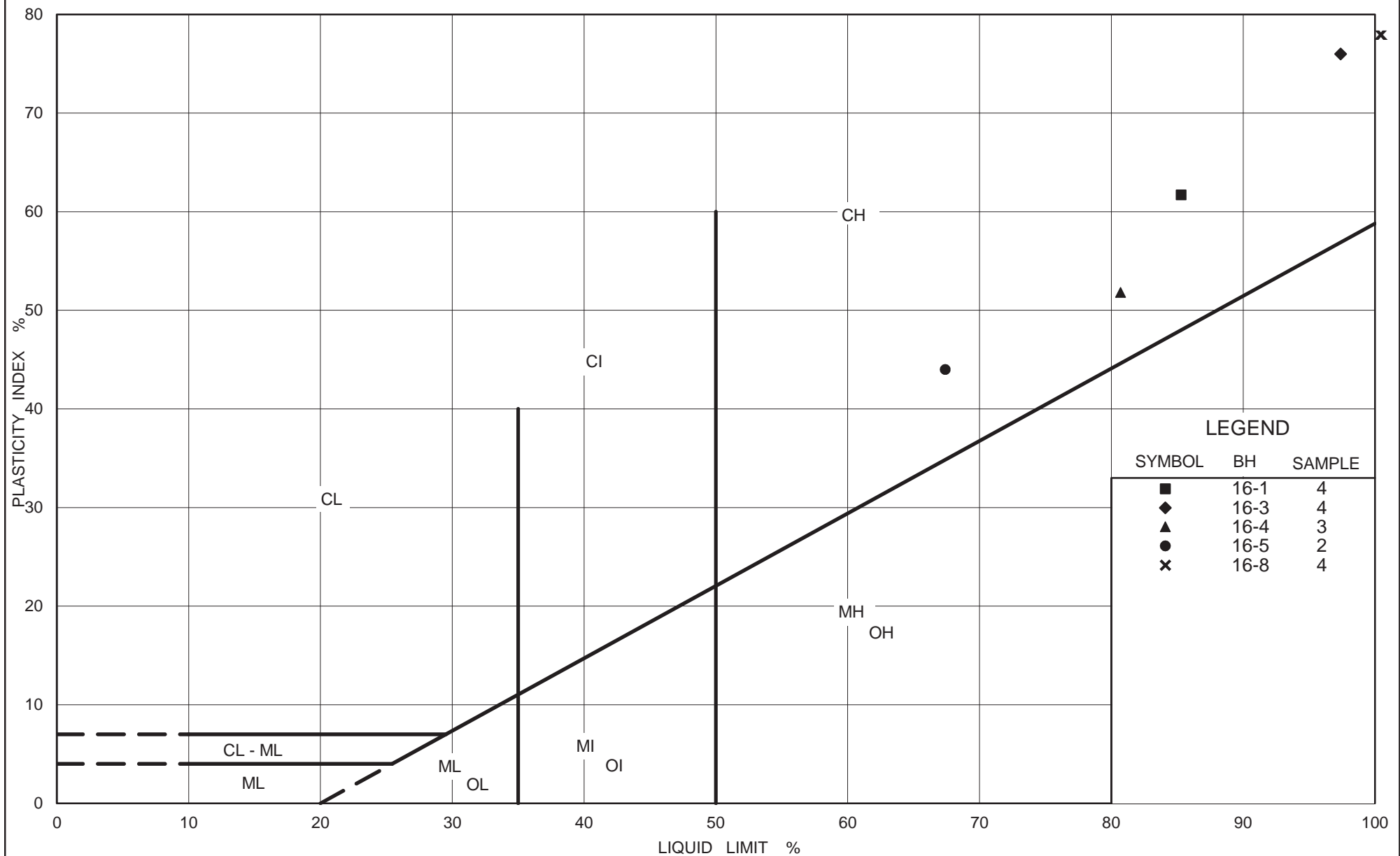
+³, ×³: Numbers refer to Sensitivity ○^{3%} STRAIN AT FAILURE

GRAIN SIZE DISTRIBUTION

FIGURE B2



Borehole	Sample	Depth (m)
16-2	5	2.29-2.90



Ontario

Ministry of Transportation

PLASTICITY CHART CLAY (Weathered Crust)

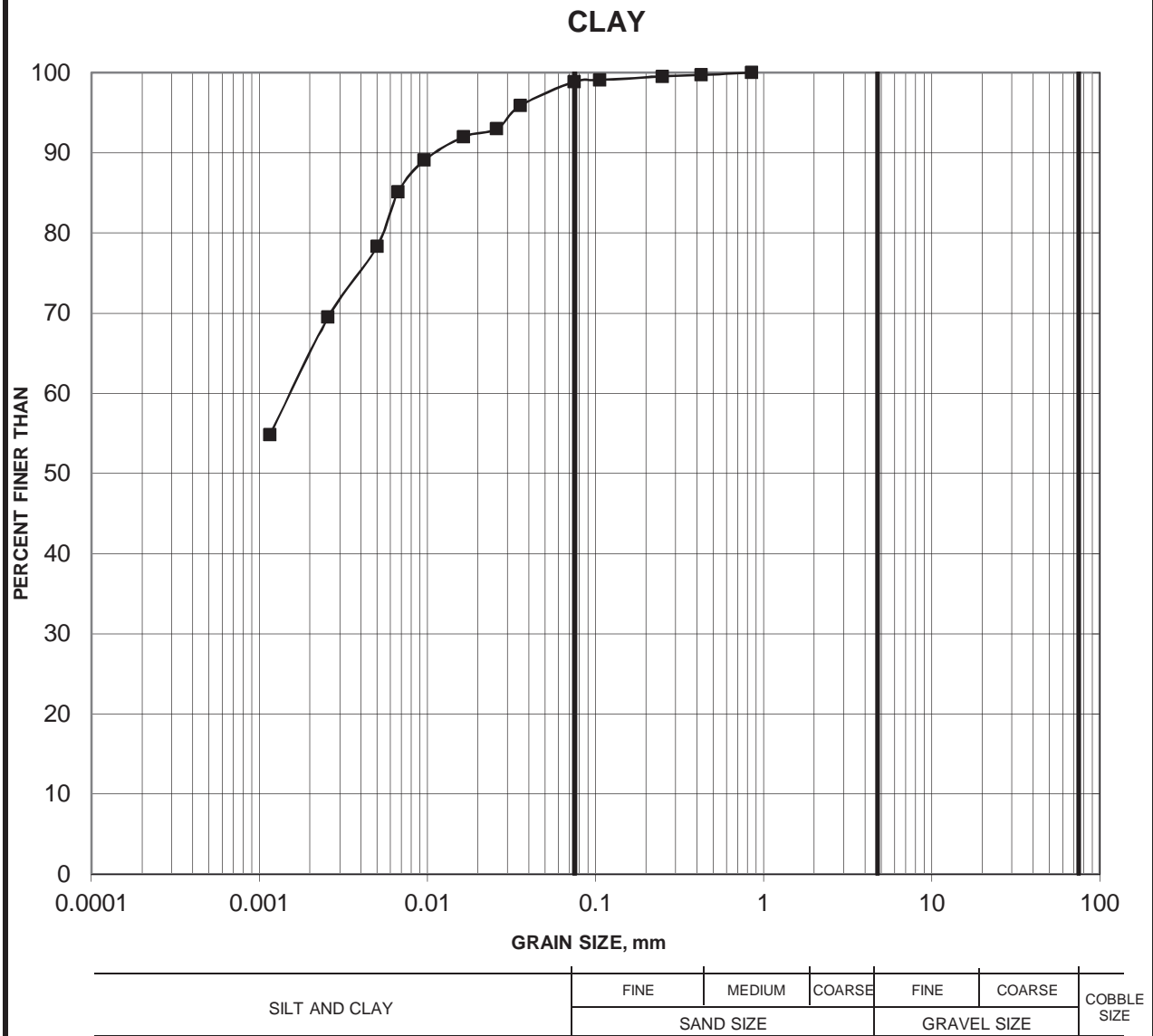
FIG No. B3

Project No. 12-1121-0193 /1140

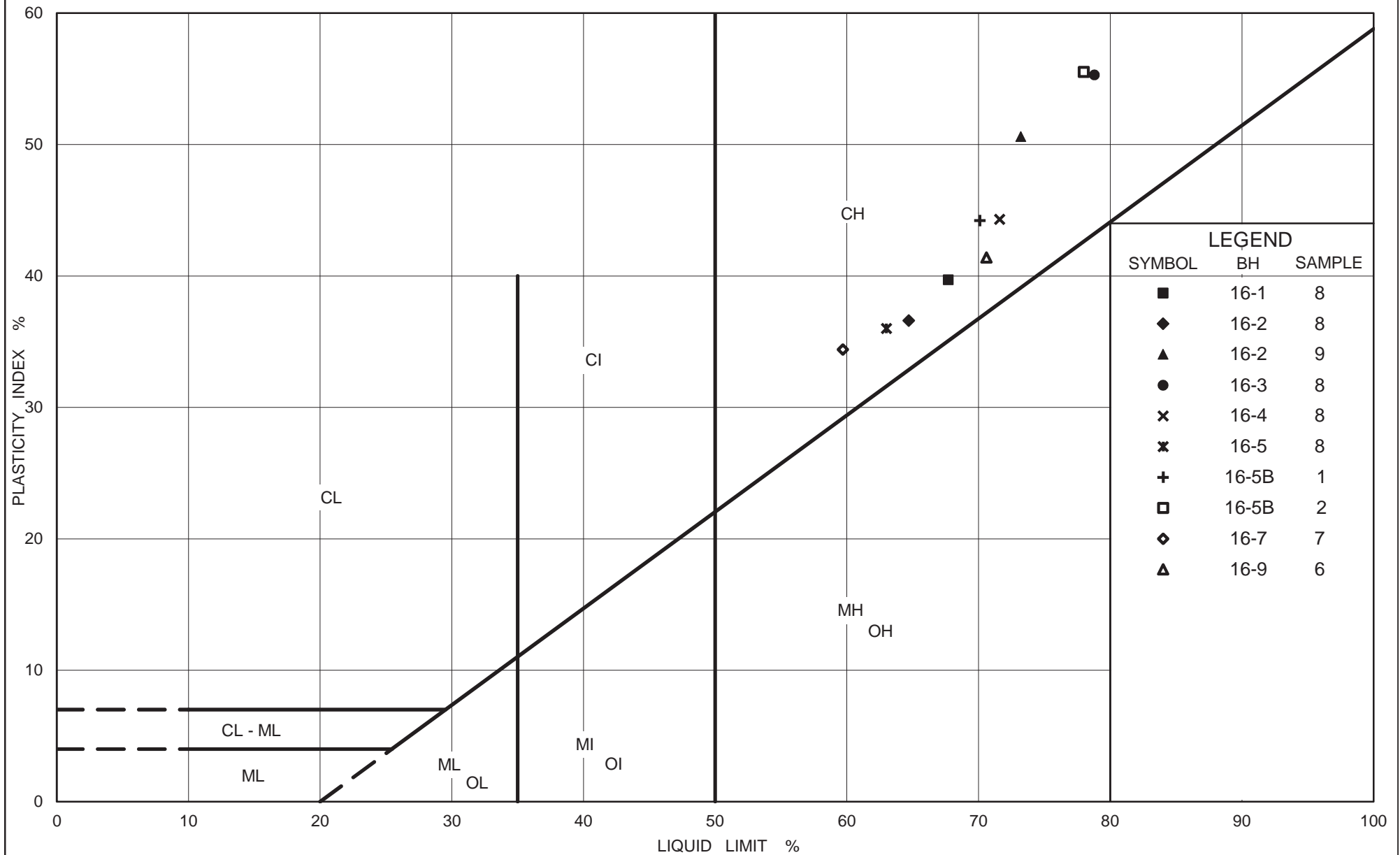
Compiled By : MI Checked By : CNM

GRAIN SIZE DISTRIBUTION

FIGURE B4



Borehole	Sample	Depth (m)
16-3	7	6.10-6.71



Ontario

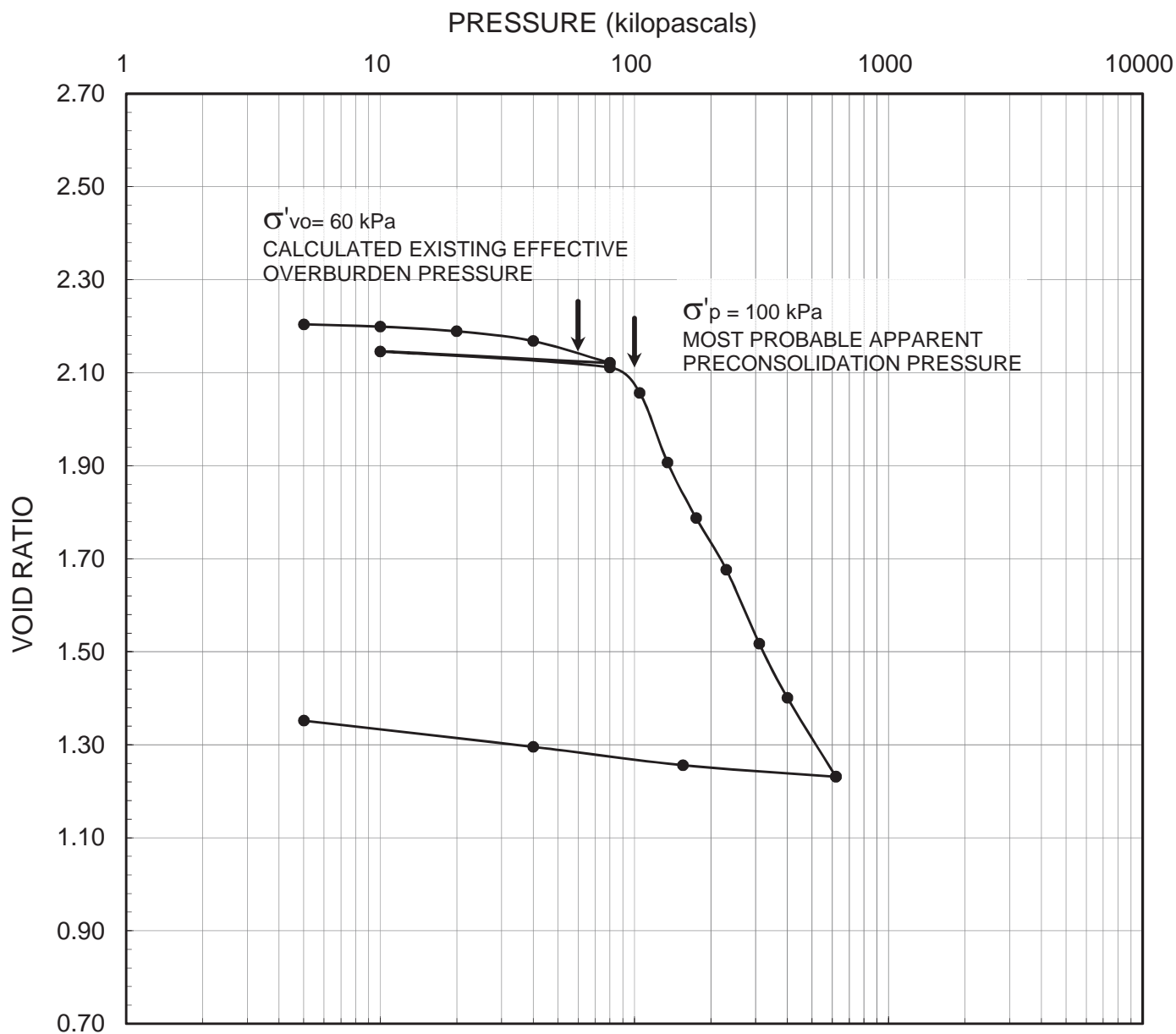
Ministry of Transportation

PLASTICITY CHART CLAY

FIG No. B5

Project No. 12-1121-0193 /1140

Compiled By : MI Checked By : CNM



LEGEND

Borehole: 16-5B	$w_i = 77\%$	$S_o = 98\%$	$\gamma = 15.2 \text{ kN/m}^3$
Sample: 1	$w_f = 49\%$	$e_o = 2.21$	$G_s = 2.80$
Depth (m): 5.7	$w_l = 70\%$	$C_c = 1.23$	
Elevation (m): 42.4	$w_p = 26\%$	$C_r = 0.039$	



SCALE	AS SHOWN
DATE	05/27/17
CADD	N/A
ENTERED	CW
CHECK	CNM
REVIEW	KSL

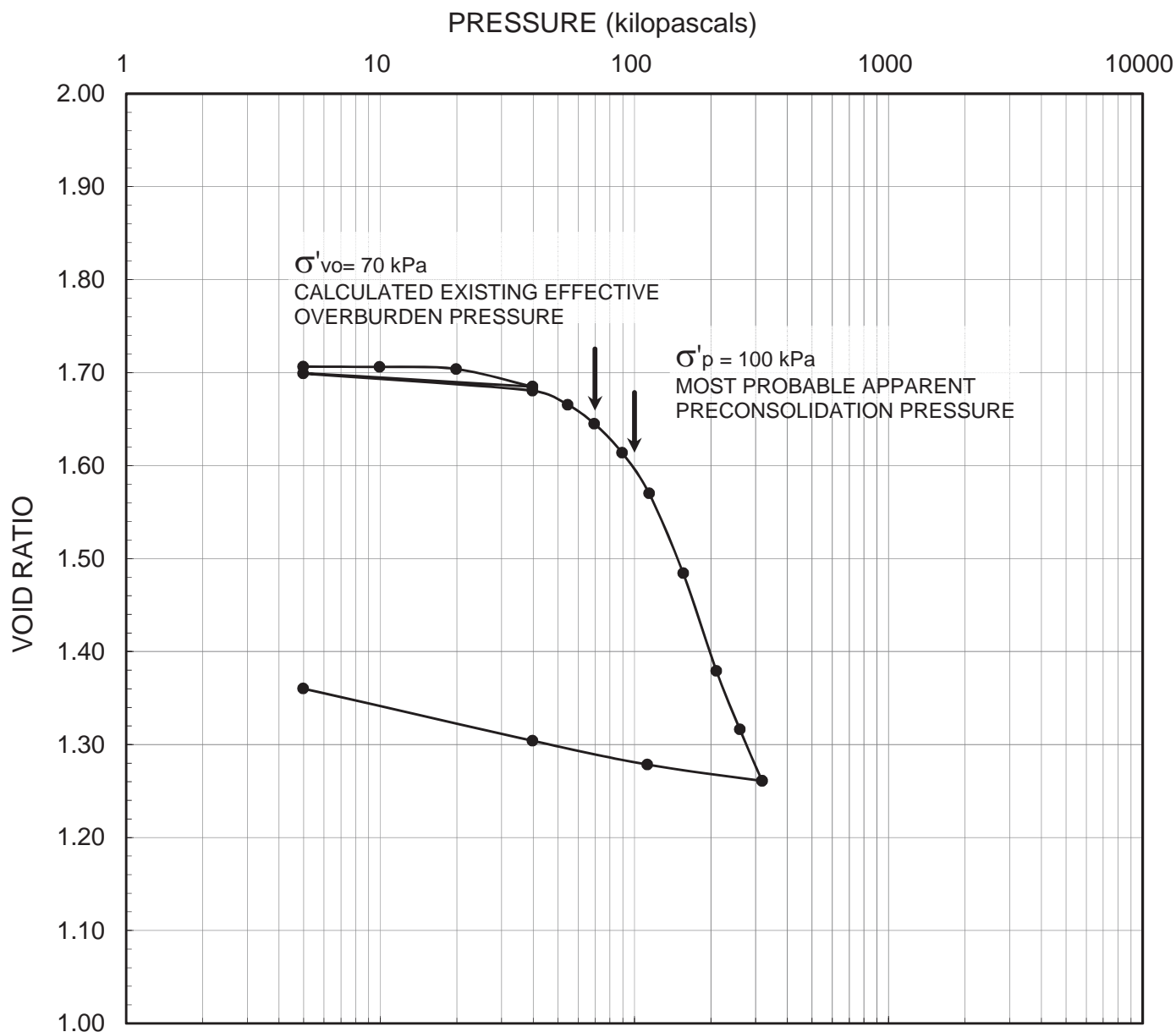
TITLE

CONSOLIDATION TEST RESULTS

FILE No.	Consolidation summary
PROJECT No.	12-1121-0193-1140
REV.	1

FIGURE

B6



LEGEND

Borehole: 16-5B	$w_i = 60\%$	$S_o = 98\%$	$\gamma = 16.3 \text{ kN/m}^3$
Sample: 2	$w_f = 48\%$	$e_o = 1.71$	$G_s = 2.81$
Depth (m): 7.6	$w_l = 78\%$	$C_c = 0.81$	
Elevation (m): 40.5	$w_p = 23\%$	$C_r = 0.020$	



SCALE	AS SHOWN
DATE	05/27/17
CADD	N/A
ENTERED	CW
CHECK	CNM
REVIEW	KSL

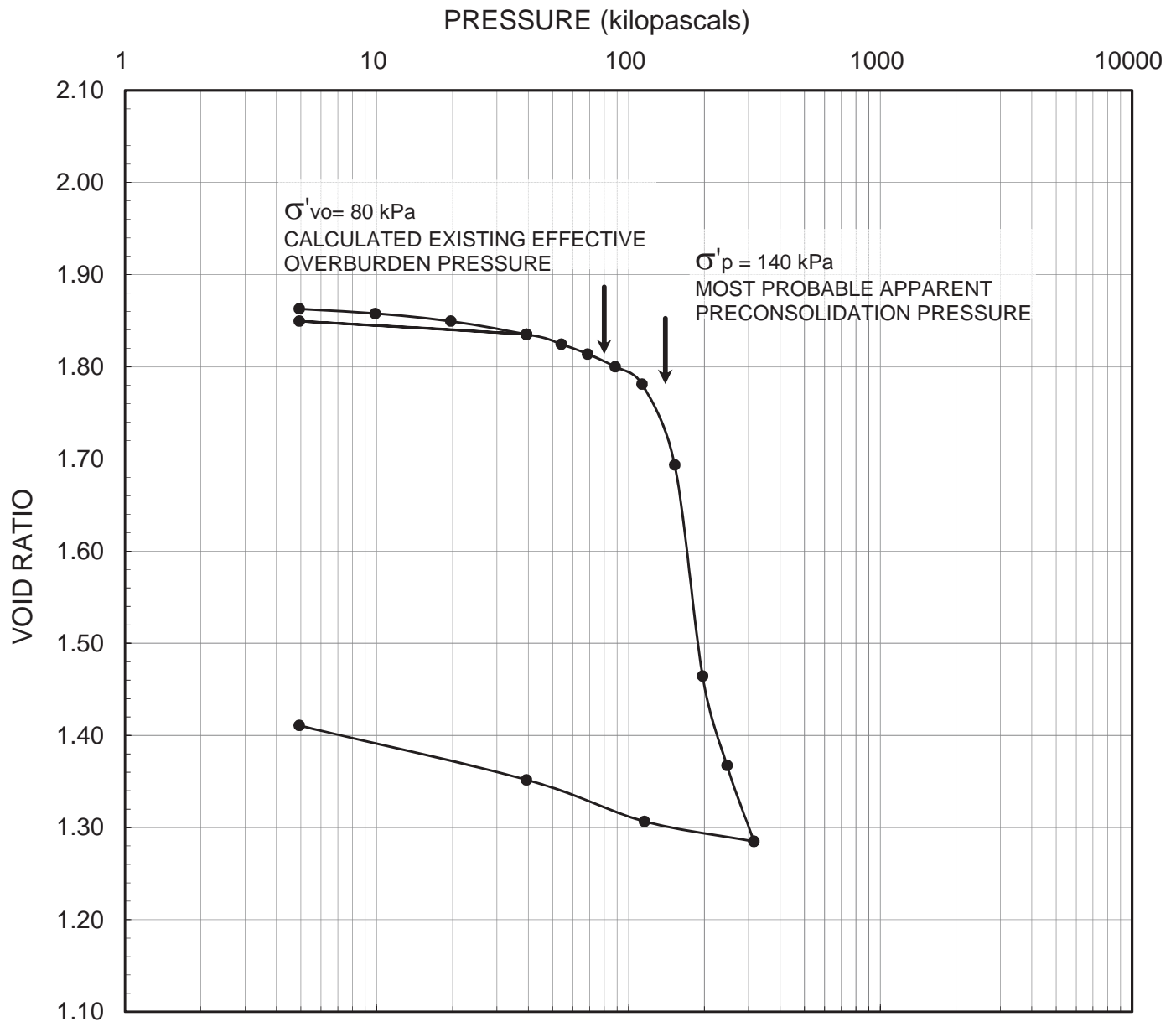
TITLE

CONSOLIDATION TEST RESULTS

FILE No.	Consolidation summary
PROJECT No.	12-1121-0193-1140
REV.	1

FIGURE

B7



LEGEND

Borehole: 16-3	$w_i = 67\%$	$S_o = 100\%$	$\gamma = 15.9 \text{ kN/m}^3$
Sample: 8	$w_f = 52\%$	$e_o = 1.86$	$G_s = 2.78$
Depth (m): 8.9	$w_l = 79\%$	$C_c = 2.09$	
Elevation (m): 39.5	$w_p = 24\%$	$C_r = 0.017$	



SCALE	AS SHOWN
DATE	05/27/17
CADD	N/A
ENTERED	CW
CHECK	CNM
REVIEW	KSL

TITLE

CONSOLIDATION TEST RESULTS

FILE No.	Consolidation summary
PROJECT No.	12-1121-0193-1140
REV.	1

FIGURE

B8

APPENDIX E

Soil-Cement Mix Testing Report - Ryerson University

Cement Mixing to Treat Sensitive Champlain Sea Clay

[Final Report]

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



March 6, 2018

Submitted to
Ms. Kim Lesage

Prepared by
Jinyuan Liu, Mohammad Afroz, Ali Ahmad

Department of Civil Engineering, Ryerson University

EXECUTIVE SUMMARY

This report presents an experimental investigation of applying deep soil mixing method (DSM) to treat Champlain Sea clay at a bridge project site near the intersection of Highway 401 and Country Road 2/34 in Lancaster, Ontario. Disturbed soil samples collected from the site were treated with ordinary Portland cement at four different dosages (100, 150, 200, 250 kg/m³ of untreated soil) and then cured at curing durations (7, 14, 28 and 56 days). A series of geotechnical tests were conducted on both native clay samples and treated clay samples, see the table below. Based on the test results, cement is able to significantly improve the strength and reduce the compressibility of Champlain Sea clay at the project site.

Summaries of tests conducted in this study		
Test Type	ASTM Standard	Number of Tests
Water Content Test	ASTM D2216-10	36
Specific Gravity Test	ASTM D854	4
Grain Size Distribution Test	ASTM D422	1
Atterberg Limits	ASTM D423, D424	2
Salinity Test	-	2
Constant Rate of Strain Consolidation Test	ASTM D4168	6
Unconfined Compressive Strength Test	ASTM D2166	32

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.

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1 INTRODUCTION

1.1 BACKGROUND

Ryerson University was retained by Golder Associates Ltd. to investigate the feasibility of applying cement to improve the ground at one of its projects. The project is for a bridge approach structure at the intersection of Highway 401 and County Road 2/34 in Lancaster, Ontario. At the site, there is a 7 m thick soft to stiff Champlain Sea clay layer approximately 3 m below the ground. The soil at the project site has an undrained shear strength ranging from 20 to 40 kPa and can lose more than 70% of its strength due to disturbance. Excess settlement would occur without any treatment as occurred on the existing embankments for the bridge to the east, which settled approximately 1.5 m. An in-situ ground improvement technique has been selected for the project where deep soil mixing (DSM) provides the best fit.

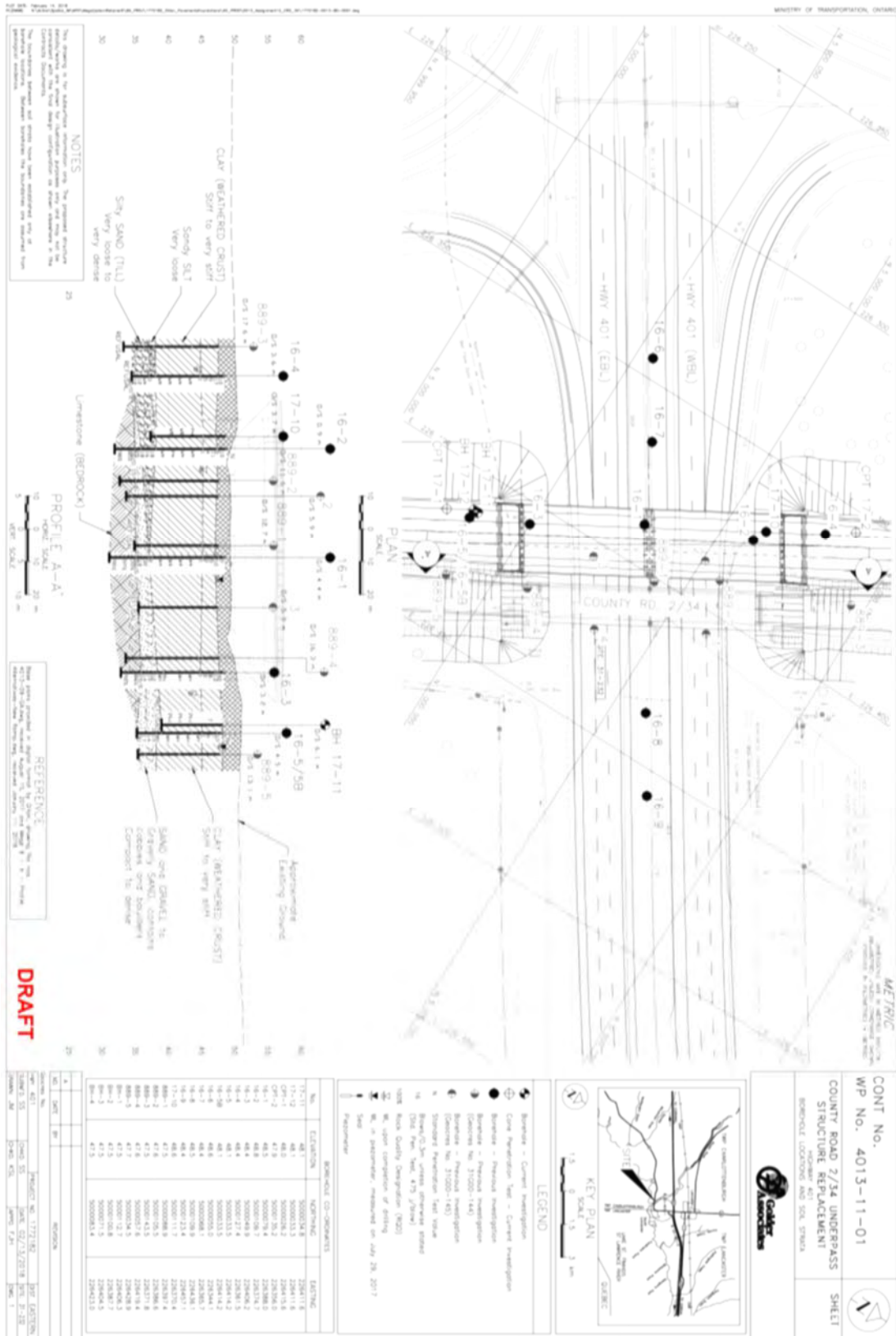
Soil samples were retrieved by Golder Associates Ltd and delivered in disturbed conditions to Ryerson University for experimental study. Three buckets of samples were collected from borehole 17-12: Sample 1 was from 3.66 to 5.49 m depth; Sample 2 from 5.49 to 7.32 m depth; and Sample 3 from 7.32 to 9.14 m depth. The shear strength profile of the ground can be seen from the borehole log for Borehole BH 17-11, which is right beside the BH17-12 for soil samples. More detailed geotechnical information can be found in Figure 1.1.

1.2 OBJECTIVES

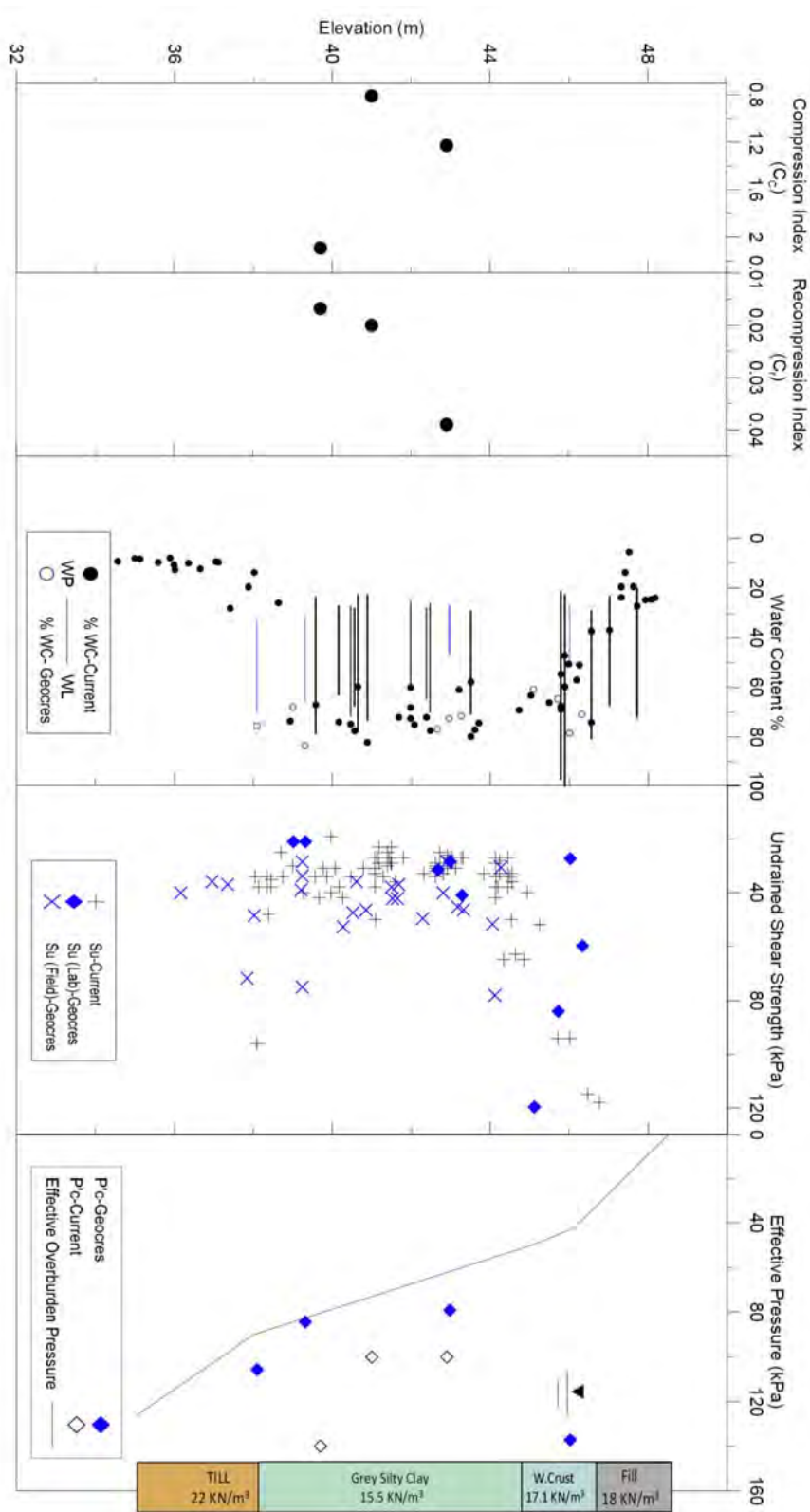
Due to the lack of precedent DSM cases in Ontario and unknown reactions of cement with local clay, a comprehensive laboratory program is needed to address these issues.

The main objective of this research is to assess the feasibility of using ordinary Portland cement to treat sensitive Champlain Sea clay at the project site. DSM has been successfully applied around the world, including the U.S. and western Canada; however, due to lack of testing data, DSM has never been applied in practice to treat sensitive Champlain Sea clay in Ontario. In order to address the issue, a series of geotechnical tests are conducted following pertinent ASTM standards, including the unconfined compressive strength (UCS) test and constant rate of strain (CRS) consolidation test.

Based on this study, a DSM design can be developed for the project. This research helps to promote the introduction of cost-effective DSM technique to treat sensitive Champlain Sea clay and eventually helps safe and efficient infrastructure development in eastern Ontario.



a) Plan and section view of the project



b) Soil properties at the project site

Figure 1.1 Geotechnical information for the project site

1.3 RESEARCH METHODOLOGY

In this study, the efficiency of applying DSM to sensitive Champlain Sea clay for the project site was investigated through an experimental program in the laboratory. The test variables include cement dosage, mixing method, and curing duration. The cement dosage designed based on literature review and past experiences. The curing durations used in this study were 7, 14, 28 and 56 days. Both UCS and CRS tests are carried out on cement-treated samples with different curing and dosage conditions to assess strength increase and compressibility change. The changes in strength and compressibility due to mixing treatment can be used to evaluate the efficiency of DSM in treating clay at the site and avoid excess settlement of the new approach structure.

2 EXPERIMENTAL DESIGN AND SAMPLE PREPARATION

2.1 EXPERIMENTAL VARIABLES

Samples were mixed and treated with Ordinary Portland cement. Experimental variables include two (2) mixing methods, four (4) binder dosages, and four (4) curing conditions. The summary of the mixing design is shown in Table 2-1. The binder dosage and curing conditions were chosen based on previous research (Li et al, 2016; Pathivada, 2005; Kitazume & Terashi, 2012; Hwang, 2006; Ramirez, 2009) and related industry design standards. Samples were prepared in two different mixing methods: dry mixing and wet mixing. Please be noted that different notations can be used for the dosage. In some literature, the volume of soil is the volume of treated soil, while others refer to the volume of soil to be treated. The latter is used in this study.

Table 2-1 Experimental program for DSM tests

Mixing Type	Cement Dosage (kg/m ³)	Curing Duration (days)
Dry, Wet	100, 150, 200, 250	7, 14, 28, 56

2.2 SAMPLE MIXING AND PREPARATION

For wet mixing method, first, cement slurry was prepared according to a 0.7:1 water to cement ratio based on a similar water content of received soil samples. The slurry was mixed with two spatulas by hand for about two minutes. 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. For dry mixing, cement powder was prepared according to required dosage and was applied to soil during mixing.

Second, a Hobart A200 Mixer (Hobart, 2005), was used to mix the samples as shown in Figure 2.1. Sample was transferred to the mixer's metal bowl and placed under the mixer's mechanical mixing hook. Clay sample was mixed at 205 rotations per minute (RPM) for about one minute.

Next, the mixer was turned off first to add the half batch of binder (cement slurry in wet mixing or cement powder in dry mixing) to the clay in the mixer. Then the mixture was allowed to mix for one minute at the speed of 205 RPM. After that, the second half of the binder was added to the batch. After the addition of the binder, clay and cement mixture was then mixed at the same speed for a total of 10 minutes. This duration of mixing is similar to the one used by other researchers (Bhadriraju et al., 2008; Pagan-Ortiz, 2013). A few intermittent interruptions were applied for about every two-minute mixing to manually remove with a spatula the mixture sticking to the wall of the mixing bowl for a good mixing.

After the soil and cement mixture was thoroughly mixed, it was compacted into plastic cylindrical molds, shown in Figure 2.2. These recyclable molds are 3-inch diameter and 6-inch high with an engineered liner for extruding the specimen easily after curing. A bottom insert in the tube also protects bottom edges of the specimen and helps to eliminate vacuum (Deslauries, 2017). The steps, shown in Figure 2.3, are described as follows:

1. Each mold was then filled with approximately three lifts according to recommendations of federal highway design manual (Pagan-Ortiz, 2013)
2. Each layer was compacted by hitting the mould 10 times against the floor and poking 30 times with a 35-mm diameter rod to remove the entrapped air voids within the specimen
3. Repeat Step 1 and Step 2 until the tube is filled to the top
4. Seal the mold with the plastic cap

Finally, the prepared samples were placed into a humid chamber for curing. The chamber has a relative humidity of 95-100 percent and a temperature of 22-25 °C (Pagan-Ortiz, 2013). This will allow cement to fully react with soil to trigger the Pozzolanic reaction (Kitazume & Terashi, 2012). After a specimen has reached its designated curing time, it was carefully removed from the mold by applying air shock to the end of the mold.



Figure 2.1. Hobart A200 mixer used for soil mixing



Figure 2.2. Reusable plastic mold used for sample curing (Deslauries, 2017)



Figure 2.3. Picture of soil and cement after dry mixing



Figure 2.4. Picture of compacting the mixture into the plastic mold



Figure 2.5. Picture of samples numbered and put in the humid chamber for curing

Based on the measured density of samples, the sample preparation method can generate reasonably consistent results. However, there are still voids in the samples observed during UCS tests. To achieve better samples in future research, compaction with static pressure or some needles attached to a poking rod can be used for a better compaction. In addition, more layers than 3 layers may be needed for a more homogeneous sample.

3 PHYSICAL PROPERTIES OF CHAMPLAIN SEA CLAY

3.1 INTRODUCTION

Soil samples were retrieved from the field by Golder Associates Ltd and delivered in disturbed conditions to Ryerson University for experimental study, as shown in Figure 3.1. Three buckets of samples were collected from borehole 17-12: Sample 1 was from 3.66 to 5.49 m depth; Sample 2 from 5.49 to 7.32 m depth; and Sample 3 from 7.32 to 9.14 m depth. The color of clay was mainly grey, varying from dark gray, greenish grey, and grey.



Figure 3.1 Disturbed soil samples received at Ryerson Laboratory

3.2 MOISTURE CONTENT

Moisture content data were collected regularly from each bucket in accordance with ASTM D2216. The results are shown in Table 3.1. It was noted that samples taken from the bottom part of the bucket had water contents of about 10% higher than the ones taken from the top part of the bucket. The values shown in Table 3.1 were for samples taken at the top part of the bucket.

3.3 ATTERBERG LIMITS

The Atterberg limits were determined for the native soil samples. The Liquid Limits (LL) were determined according to ASTM D 423 while Plastic Limit (PL) according to ASTM D 424. The test results are also shown in Table 3.1.

3.4 NATURAL DENSITY

A 63.5 mm (2.5-inch) diameter and 25 mm (1-in) high consolidation ring was used to measure the sample density. Density data were obtained by dividing the direct weight of the sample by the ring volume. The results can be found in Table 3.1.

3.5 SPECIFIC GRAVITY

Specific gravity was determined according to ASTM D854. The results are shown in Table 3.1.

3.6 PORE-FLUID SALINITY

There are two methods which can be used in measuring the salinity in a soil sample: the diluted fluid method or the squeezed fluid method. In the first method, salinity is measured by the following procedures: First, obtain the mass of dry sample through oven drying the sample; Second, dissolve dry sample thoroughly into a known volume of distilled water; Third, measure the salt concentration in the liquid by a salinity meter. A Horiba ES-51 portable salinity meter was used in this study. In the squeezed fluid method, the pore fluid is squeezed out of the clay sample and the salinity is measured directly from the pore fluid. Based on a previous study on Champlain Sea clay (Liu et al. 2017), it was found that the salinity from the diluted fluid method was about 4-5 % higher than the one from the squeezed fluid method. This is similar to the finding that the dissolving sample in distilled water method results in an overestimation of the pore water salinity compared to the squeezed fluid method (Torrance 1979).

The diluted fluid method was used for the salinity measurement in the clay sample. The salinity level can be presented in two ways: one is the salinity in the pore fluid (g/L), the other is the salinity in dry soil (g/kg). The salinity values in the clay samples are shown Table below.

Table 3.1 Physical properties of soil received for research

Physical Property	Sample 1 (from 3.66 to 5.49 m depth)	Sample 2 (from 5.49 to 7.32 m depth)	Sample 3 (from 7.32 to 9.14 m depth)
Wet Density (kN/m ³)	16.3	16.47	16.49
Liquid Limit	61.41	49.26	-
Plastic Limit	30.75	29.79	-
Water Content* (%)	61.64	64.82	65.54
Specific Gravity	2.72	2.60	-
Salinity (g/L)	1.956	1.615	-
Salinity (g/kg)	1.252	1.034	-

*_water content about 10% higher was noticed for soil samples taken from the bottom part of sample in the bucket. Results in the table are for samples taken from the top part of the buckets

3.7 GRAIN SIZE DISTRIBUTION

The standard according to ASTM D 6913 was found difficult to obtain the right grain size distribution of Champlain Sea clay due to the difficulties in breaking the fine particles through mechanical measures (Liu et al. 2017). A modified method was adopted to address these issues above. First, the sample was broken into pieces and oven dried to determine its dry weight. Second, the sample was grinded using a mortar and pestle and then placed in a dispersion cup with water.

The resulting slurry was then mechanically mixed in order to fully separate clay particles from one another. After that, all soil particles were passed through the No. 10 sieve to conform to ASTM D422. Next, the soil slurry was transferred to a glass beaker for a hydrometer test. After completing the hydrometer test, the slurry was then passed through a series of sieves smaller than No. 10 with the smallest size being No. 200. The grain size distribution of clay of Sample 1 retrieved from 3.66-5.49 m depth can be found in the figure below. Due to lack of samples, the grain size distribution of clays at deeper depths could not be conducted in this study.

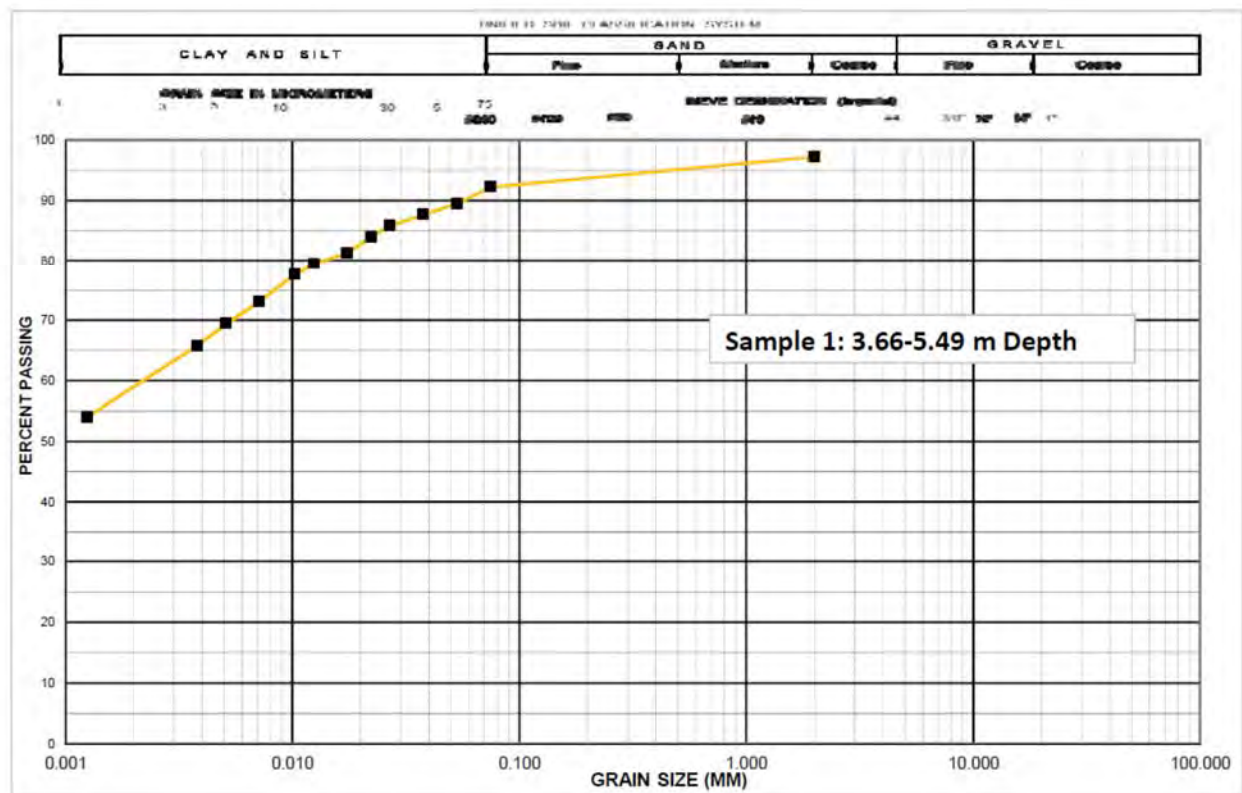


Figure 3.2 Grain size distribution of clay from 3.66-5.49 m depth

3.8 SUMMARY

A series of index properties were conducted on received clay samples, including density, water content, specific gravity and grain size distribution.

4 UNCONFINED COMPRESSIVE STRENGTH TEST

4.1 INTRODUCTION

The compressive strength from the UCS test was used as the main benchmark to determine the efficiency of cement in improving the strength of clay collected at the project site. These tests were

conducted in accordance with ASTM D2166. A typical stress-strain curves from a UCS test can be found in Figure 4.1. It can be seen that the sample mostly reaches its peak strength at a strain value of less than 1%. After that, it gradually loses its strength with increasing strain due to mainly the brittle failure mode of the samples and also lack of lateral confinement. The strain softening behaviour shall be considered in the design. The impact of lateral confinement on the stress-strain behaviour of cement treated clay will be investigated in the future.

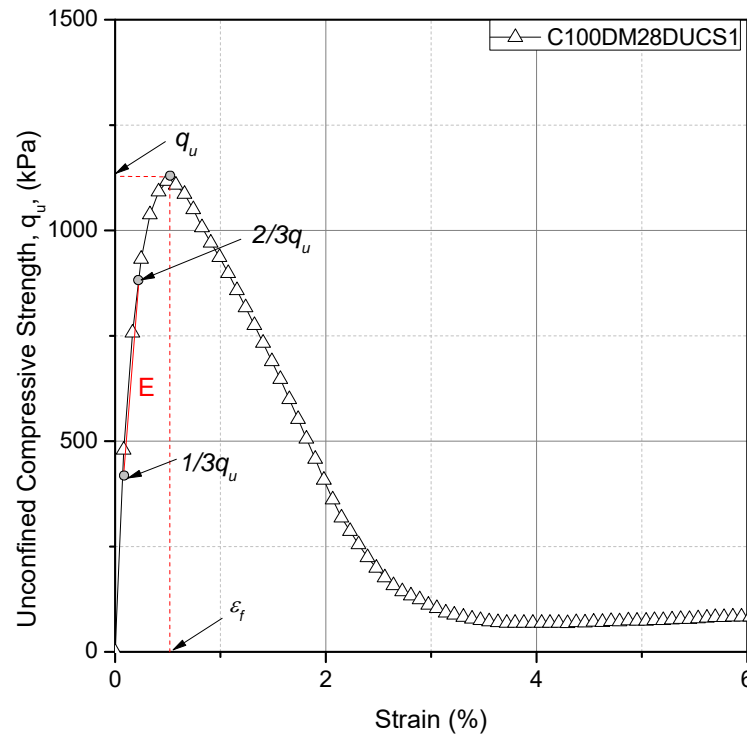


Figure 4.1 A typical stress-strain curve of UCS test on 28-day curing sample

Figure 4.2 shows a typical UCS test conducted on dry mixing sample before and after failure, while a wet mixing sample is shown in Figure 4.3. From these figures, it can be found more voids were found in the dry mixing samples, which were not prominent in the wet mixing sample. The reason is due to the fact more difficulties were experienced in compacting a dry mixing sample due to its higher stiffness. More improvement is needed in future sample compaction.

There were two failure planes observed in the samples: a conical failure plane and a planar one. The conical failure plane was found mainly in the dry mixing samples, while a planar failure plane was noticed mainly in the wet mixing sample even under the same dosage and curing conditions. More details of the samples before and after failure can be found in Appendix B.



Figure 4.2 A typical UCS test on a dry mixing sample before and after failure

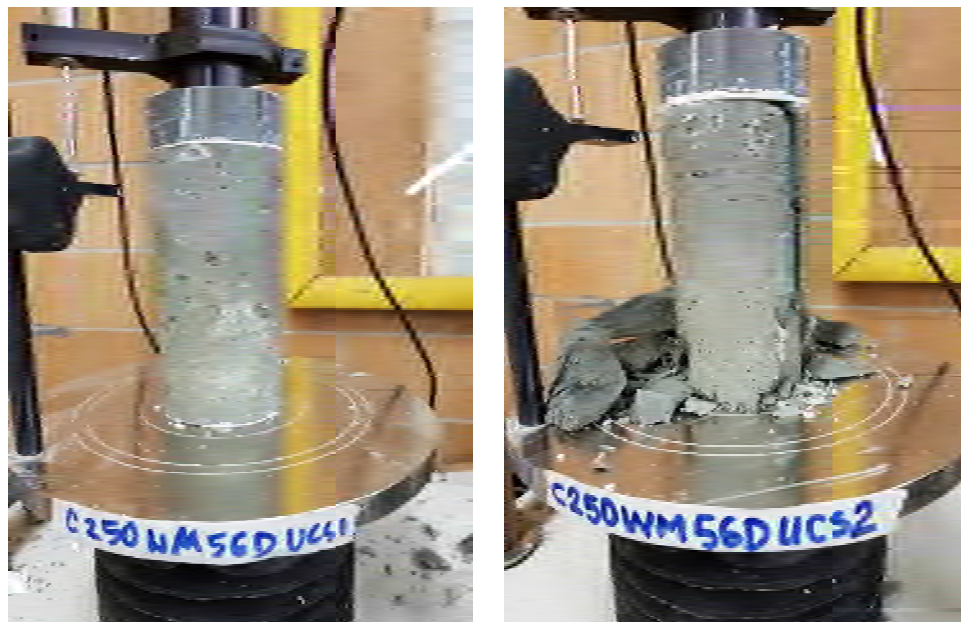


Figure 4.3 A typical UCS test on a wet mixing sample before and after failure

4.2 UCS TEST RESULTS AND ANALYSES

The results from UCS tests are summarized in Table 4.1. Each sample is noted in the same manner as the cement dosage, mixing method, curing time, last followed by UCS test number. For example, C150DM7DUCS1 is the sample with a cement dosage of 150 kg/m³ (C150), mixed by dry mixing

(DM), cured at 7 days (7D), and the first UCS test (UCS-1). The value of q_u is related to the peak compressive strength of the sample, as shown in Figure 4.1. In addition, the corresponding shear strain related to q_u is noted as failure strain, ε_f . Young's modulus, E is calculated based on the two stress levels approximately at $1/3q_u$ and $2/3q_u$ from the stress-strain curve of each sample, as shown in Figure 4.1.

Table 4.1 UCS test results for cement treated Champlain Sea clay samples

Experiment No.	Type of Mixing	Cement Dosage (kg/m ³)	γ_w Before test (kg/m ³)	W_c Before Test (%)	W_c After Test (%)	Curing (days)	Peak UCS, q_u (kPa)	Failure Strain, ε_f (%)	Young's Modulus, E (MPa)
C100DM7D UCS1	Dry Mix	100	1571	65.02	65.83	7	860.0	0.75	253
C100WM7D UCS1	Wet Mix	100	1560	66.68	63.46	7	904.9	0.83	228
C150DM7D UCS1	Dry Mix	150	1612	55.81	48.45	7	1602.9	1.32	172
C150WM7D UCS1	Wet Mix	150	1569	64.57	58.73	7	763.4	1.24	110
C200DM7D UCS1	Dry Mix	200	1621	48.31	44.54	7	1600.2	0.66	499
C200WM7D UCS1	Wet Mix	200	1625	57.95	55.15	7	769.6	0.90	139
C250DM7D UCS1*	Dry Mix	250	1660	54.69	46.66	7	2815.9	1.42	494
C250WM7D UCS1*	Wet Mix	250	1546	70.71	63.40	7	1606.9	0.98	399
C100DM14D UCS1	Dry Mix	100	1531	65.02	64.69	14	1388.9	0.57	1277
C100WM14 DUCS1	Wet Mix	100	1511	66.68	62.06	14	1247.1	0.66	413
C150DM14D UCS1	Dry Mix	150	1623	55.81	52.86	14	1609.5	0.74	258
C150WM14 DUCS1	Wet Mix	150	1571	64.57	59.36	14	801.4	0.50	299
C200DM14D UCS1	Dry Mix	200	1584	48.31	44.92	14	886.5	0.33	407
C200WM14 DUCS1	Wet Mix	200	1598	57.95	55.29	14	917.9	0.66	273
C250DM14D UCS1*	Dry Mix	250	1638	54.69	49.62	14	3175.8	0.83	977
C250WM14 DUCS1*	Wet Mix	250	1548	70.71	61.30	14	1142.6	0.73	234
C100DM28D UCS1	Dry Mix	100	1564	65.02	60.99	28	1116.9	0.58	385
C100WM28 DUCS1	Wet Mix	100	1555	66.68	63.28	28	929.4	0.67	436
C150DM28D UCS1	Dry Mix	150	1652	55.81	52.66	28	1990.1	0.42	651
C150WM28 DUCS1	Wet Mix	150	1591	64.57	59.61	28	1052.6	0.92	251

Experiment No.	Type of Mix	Cement Dosage (kg/m ³)	ρ_d Before test (kg/m ³)	W_c Before Test (%)	W_c After Test (%)	Curing (days)	Peak UCS, q_u (kPa)	Failure Strain, ϵ_f (%)	Young's Modulus, E (MPa)
C200DM28D UCS1	Dry Mix	200	1557	48.31	44.07	28	1567.9	0.50	504
C200WM28 DUCS1	Wet Mix	200	1626	57.95	55.87	28	1358.3	0.50	684
C250DM28D UCS1*	Dry Mix	250	1645	54.69	49.35	28	2786.0	0.66	617
C250WM28 DUCS1*	Wet Mix	250	1542	70.71	65.36	28	1750.6	0.50	530
C100DM56D UCS1	Dry Mix	100	1569	65.02	55.55	56	1357.93	0.92	404
C100WM56 DUCS1	Wet Mix	100	1551	66.68	63.28	56	1877.69	0.75	468
C150DM56 DUCS1	Dry Mix	150	1627	55.81	50.49	56	1569.72	0.92	188
C150WM56 DUCS1	Wet Mix	150	1570	64.57	61.49	56	1230.55	1.00	276
C200DM56 DUCS1	Dry Mix	200	1519	48.31	43.24	56	2021.89	0.50	808
C200WM56 DUCS1	Wet Mix	200	1638	57.95	53.22	56	1255.21	1.24	202
C250DM56 DUCS1*	Dry Mix	250	1623	54.69	47.24	56	4157.57	0.58	2681
C250WM56 DUCS1*	Wet Mix	250	1555	70.71	64.17	56	1578.65	0.83	393

*_clay was taken from Sample 1 bucket

The stress-strain curves of UCS tests for cement-treated samples for each dosage and mixing condition are shown in Figures 4.4 to 4.7.

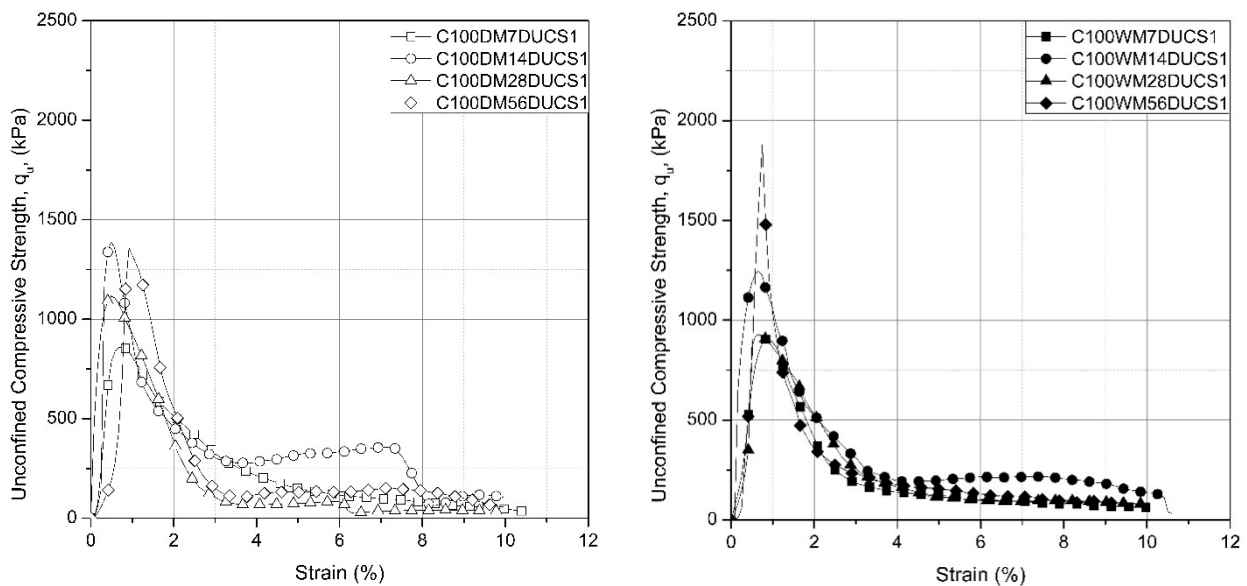


Figure 4.4 Stress-strain curves of 100 kg/m³ cement dosage samples

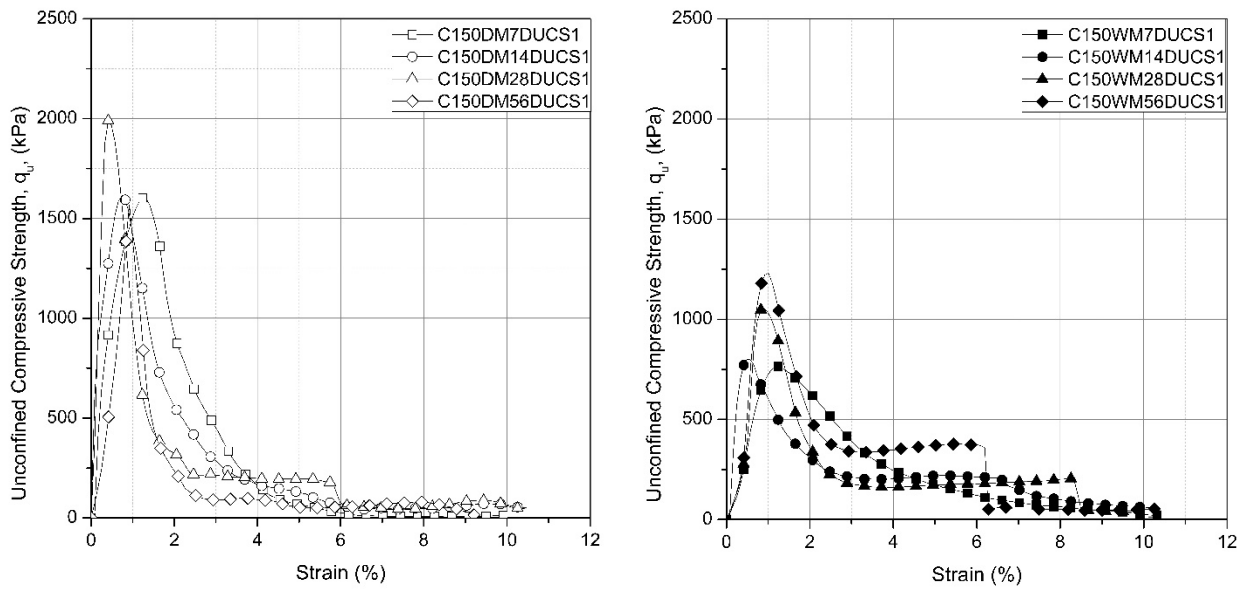


Figure 4.5 Stress-strain curves of 150 kg/m³ cement dosage samples

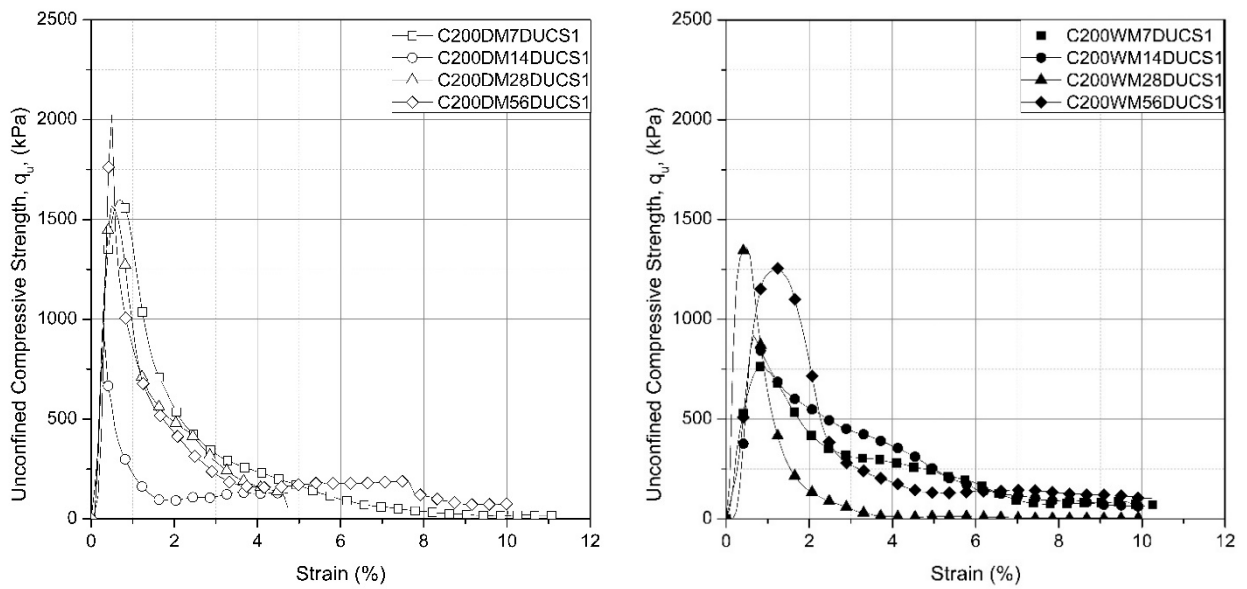


Figure 4.6 Stress-strain curves of 200 kg/m³ cement dosage samples

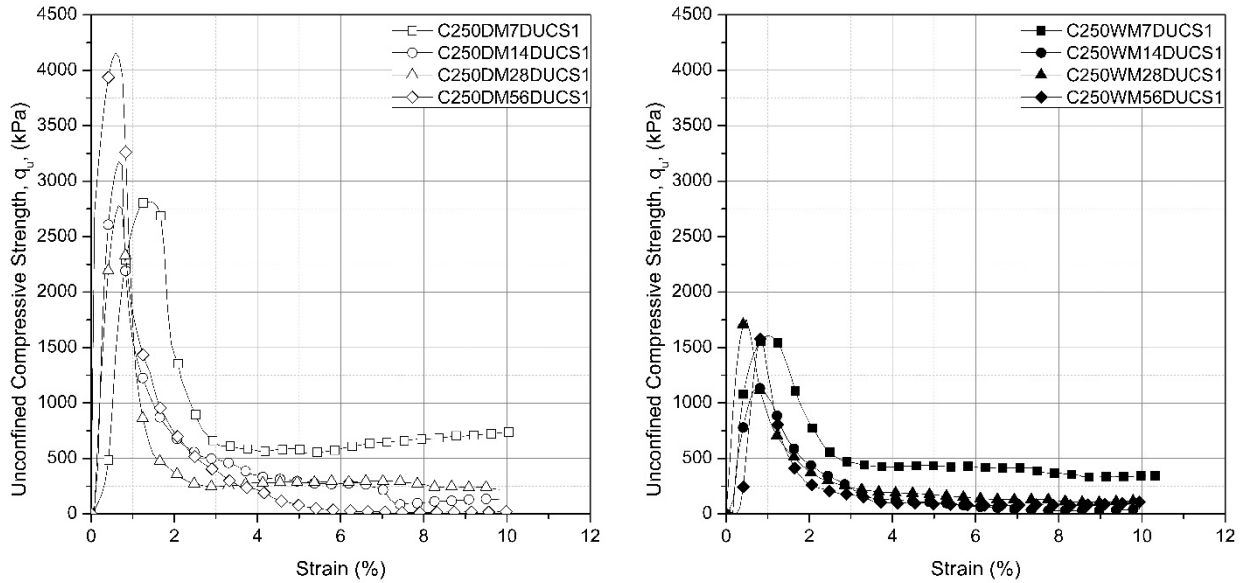
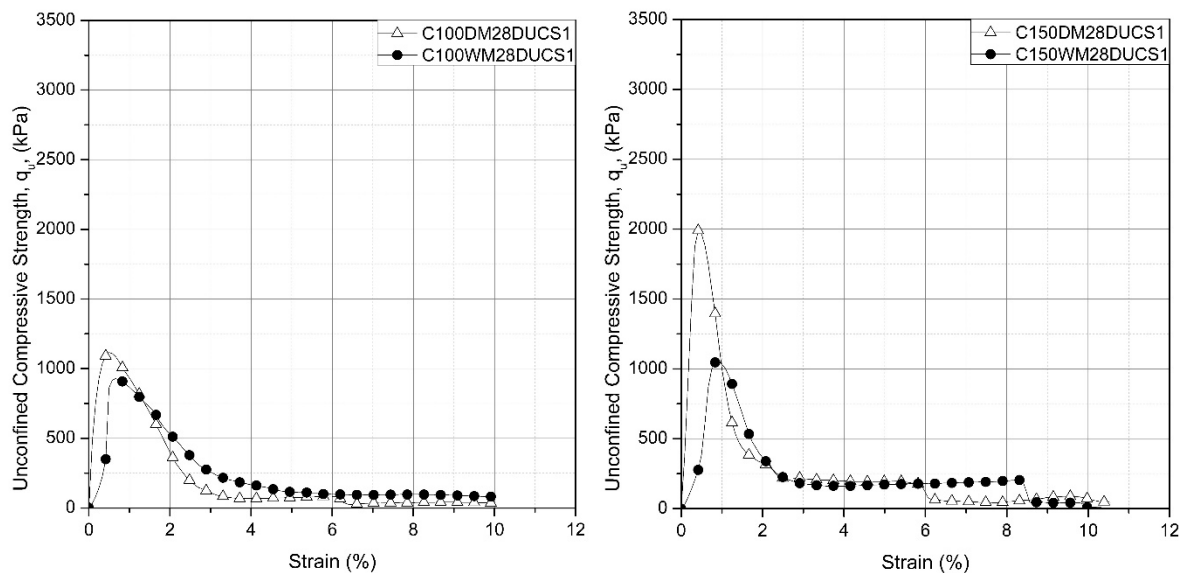


Figure 4.7 Stress-strain curves of 250 kg/m³ cement dosage samples

The impact of different mixing methods on the stress-strain curve is shown in Figure 4.8, where 28-day curing samples are selected for comparison of each dosage condition. In addition, the impact on q_u is also shown in Figure 4.9 for both 28-day and 56-day curing samples.

It is premature to provide any conclusive trends due to a limited number of samples. Generally speaking, the dry mixing samples exhibit higher q_u values than those of wet mixing samples. The failure strains are similar for both dry mixing and wet mixing samples. The difficulties for compacting much stiffer mixture into consistent samples may contribute to a larger scatter in the strength values in dry mixing samples.



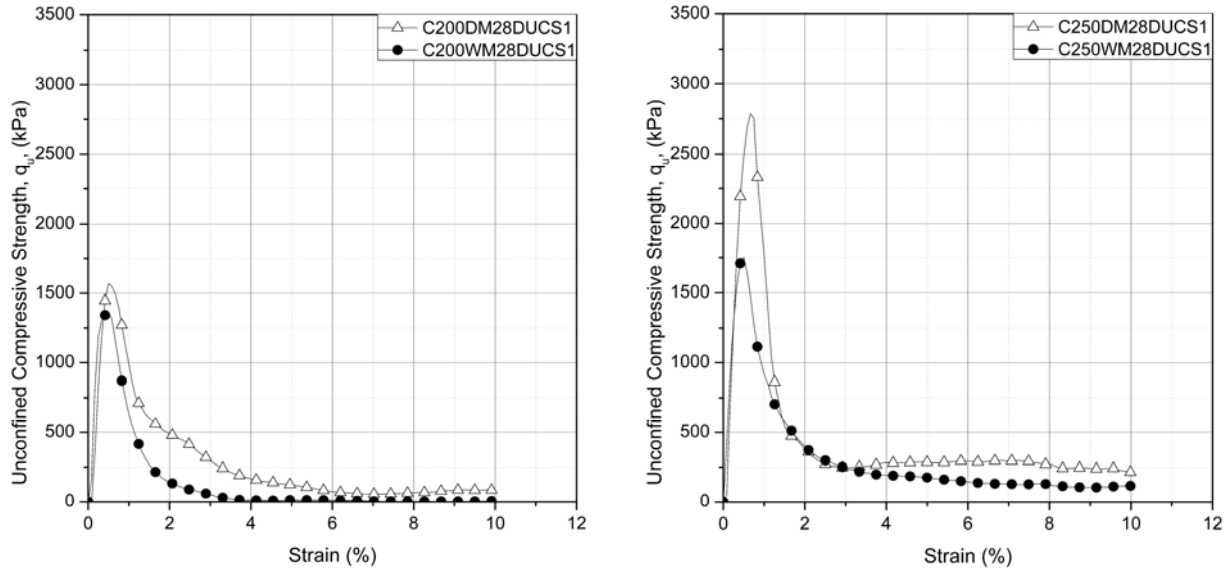


Figure 4.8 Comparison of impact of mixing method on stress-strain curves of samples

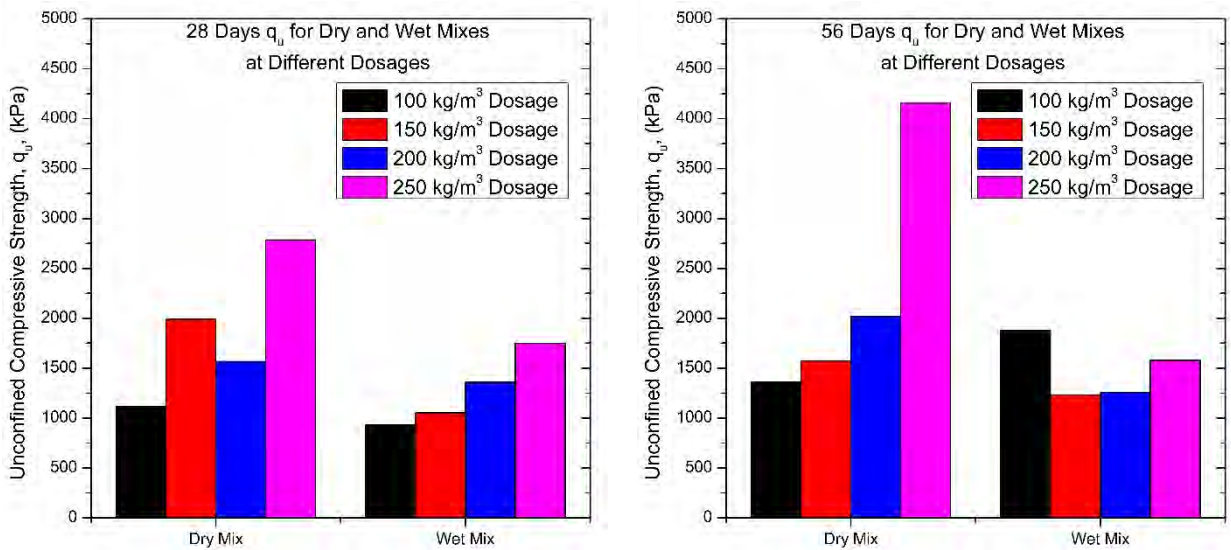


Figure 4.9 Impact of mixing method on q_u of cement-treated samples

The impact of water content on q_u is shown in Figure 4.10 for the 28-day and 56-day curing samples. The trend of reduction of q_u with increasing water content is shown clearer in dry mixing samples and not prominent in wet mixing samples.

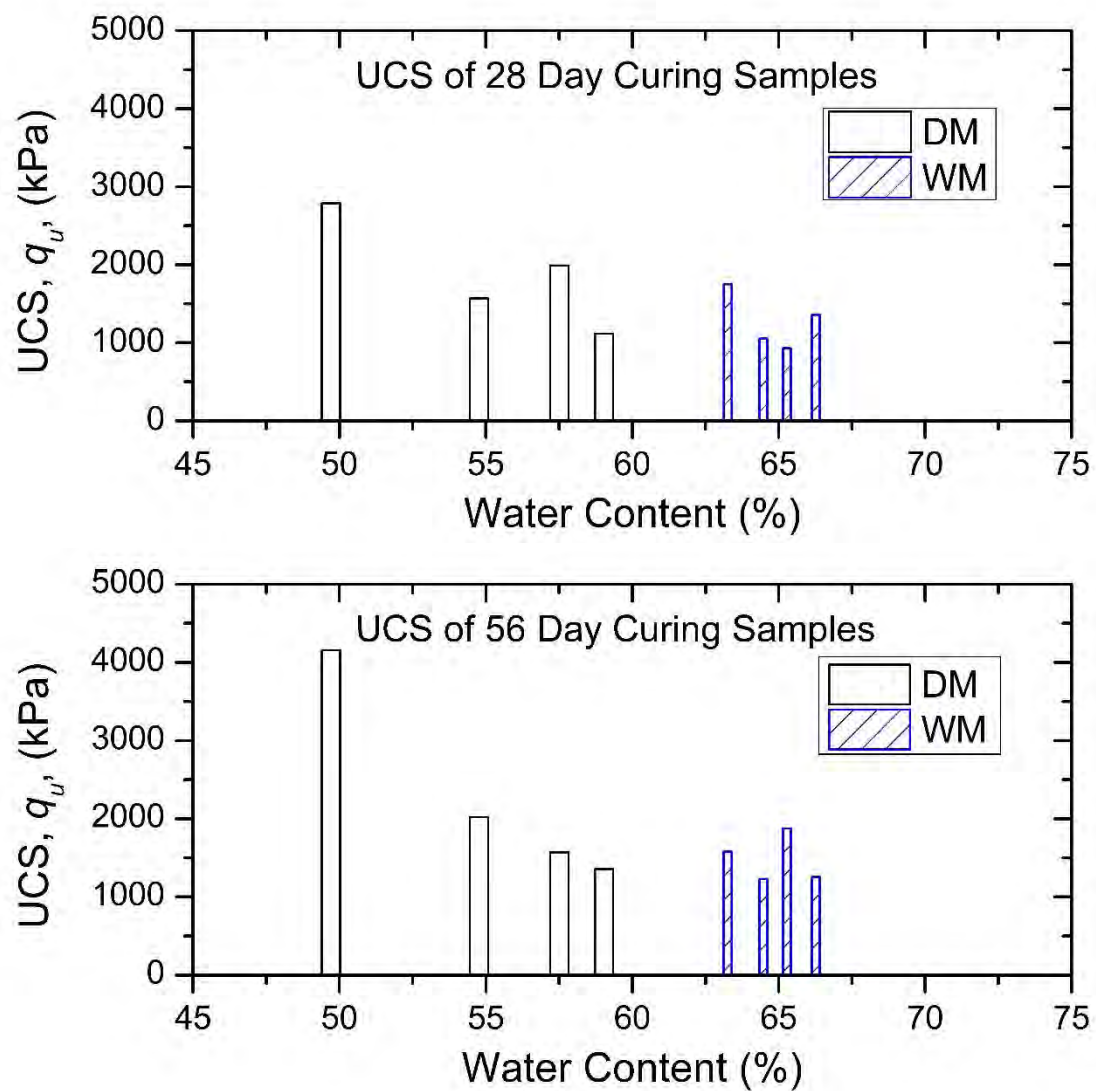


Figure 4.10 Impact of water content on q_u of cement-treated samples

The impact of sample density on q_u is shown in Figure 4.11 for all the samples. It can be seen q_u increases with increasing dry density of the treated samples except one of the samples. This also implies the possibility of quality control of DSM in the field by specifying the required dry density after treatment.

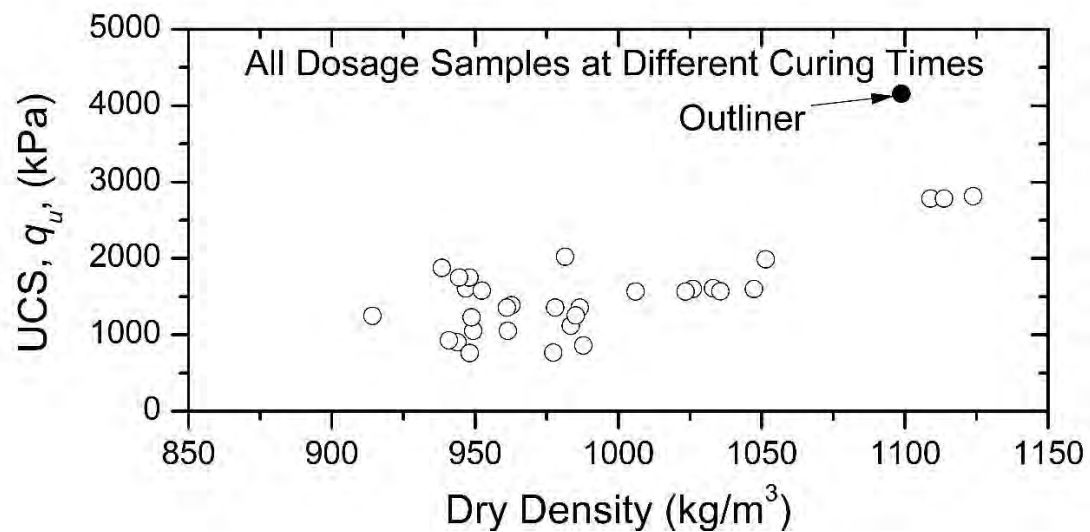
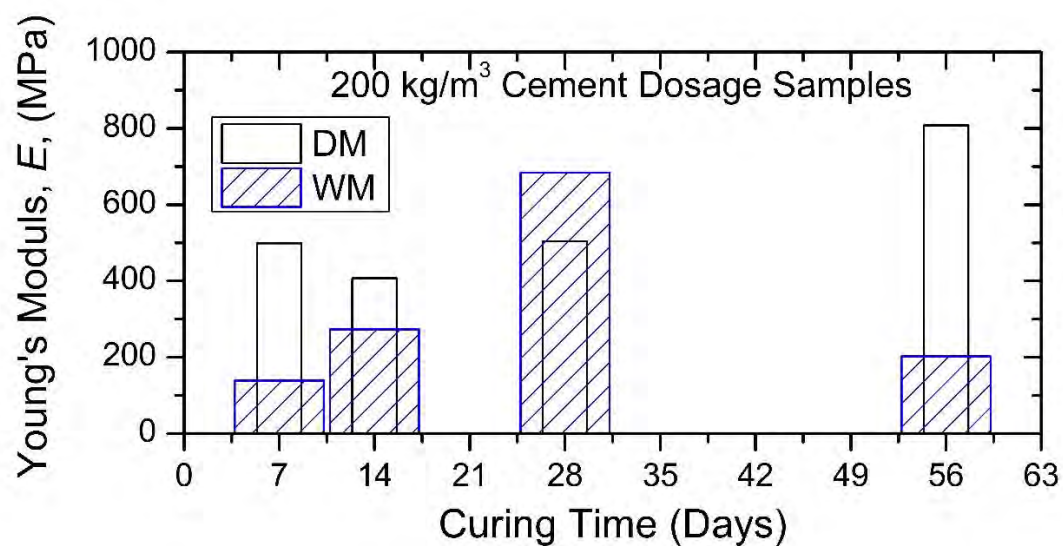


Figure 4.11 Impact of dry density on q_u of all samples

The Young Modulus, E is calculated from two points at stress levels about 1/3 and 2/3 of q_u for each sample. The change of E with time for the 150 and 200 kg/m^3 dosage samples are shown in Figure 4.12. There is not a clear trend of E change with time observed in these samples.



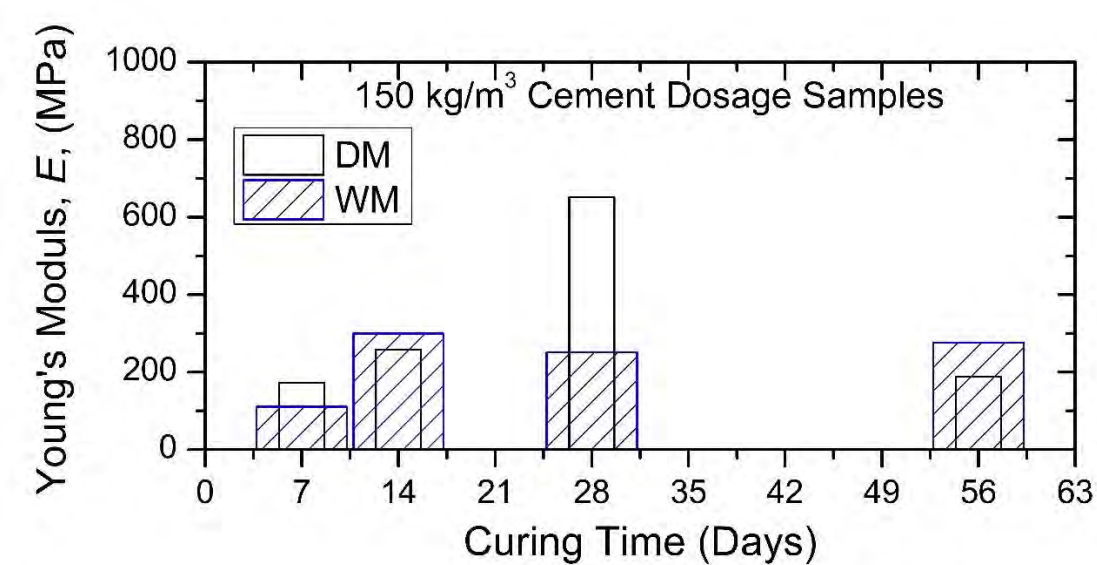
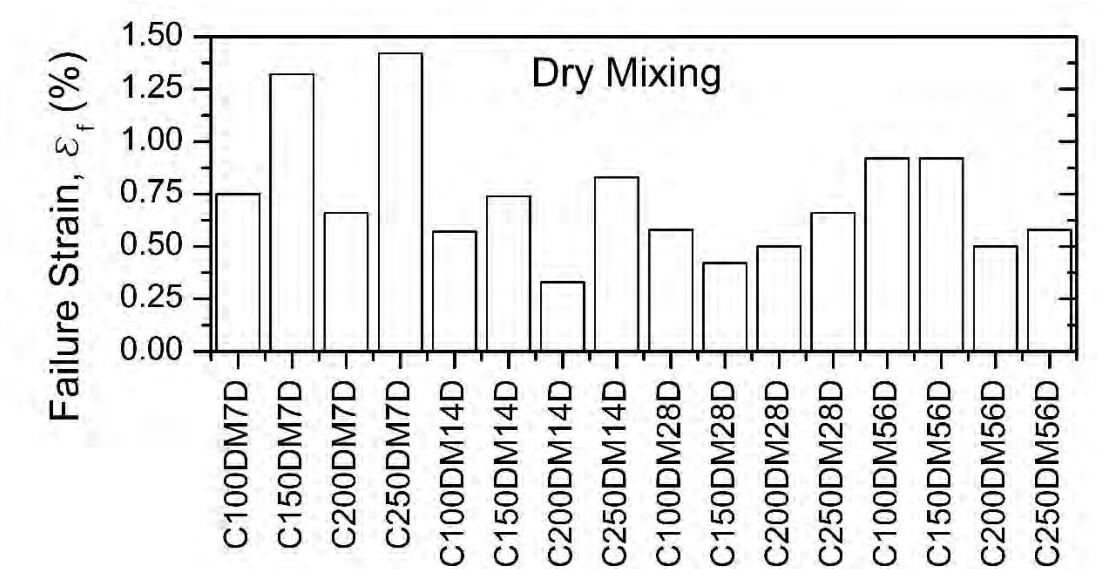


Figure 4.12 Young's Modulus of Elasticity, E for dry and wet mixes at different curing days

Most of the failure strain, ϵ_f for both dry and wet mixing samples are less than 1%, as shown in Figures 4.13 for all samples. This implies that the cement-treated samples will likely experience a brittle shear failure mode.



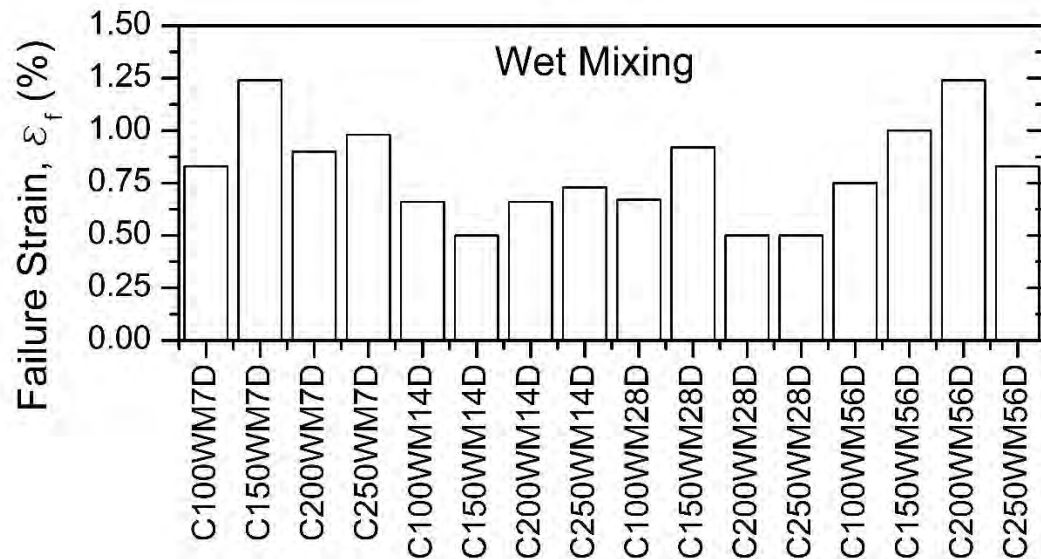
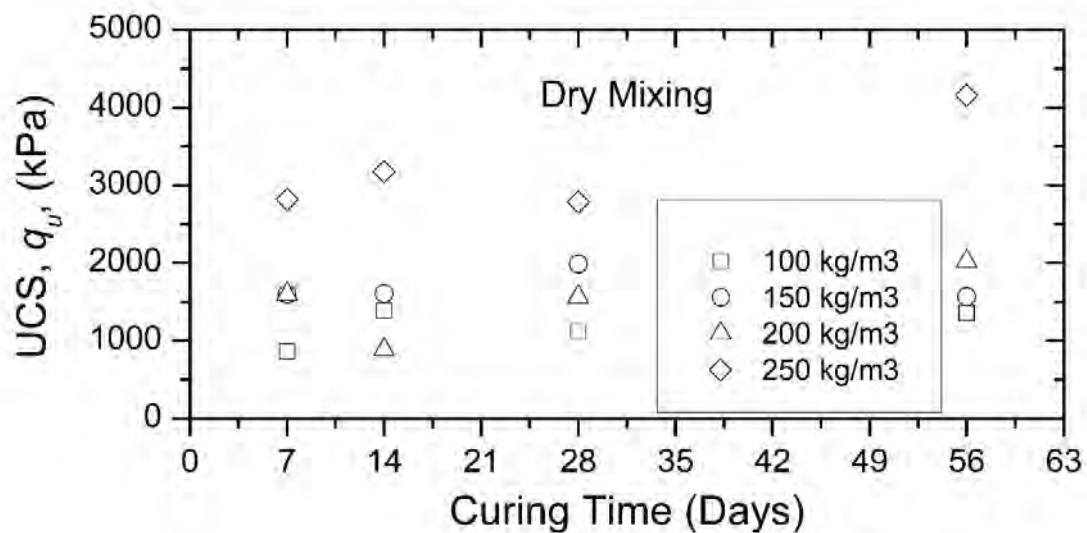


Figure 4.13 Failure strain, ε_f of UCS tests for the dry and wet mixing samples

The impact of curing time on q_u of cement-treated samples is shown in Figure 4.14. There is no prominent trend noticed from the samples on the strength change with time after 7-day initial curing, particularly for dry mixing samples. The test results also imply that cement can quickly increase the shear strength of Champlain Sea clay, shorter than 7 days in this study. The high salinity in the clay sample may contribute to less prominent change with time. More research is being conducted to address the impact of pore fluid chemistry on DSM in Champlain Sea clay.



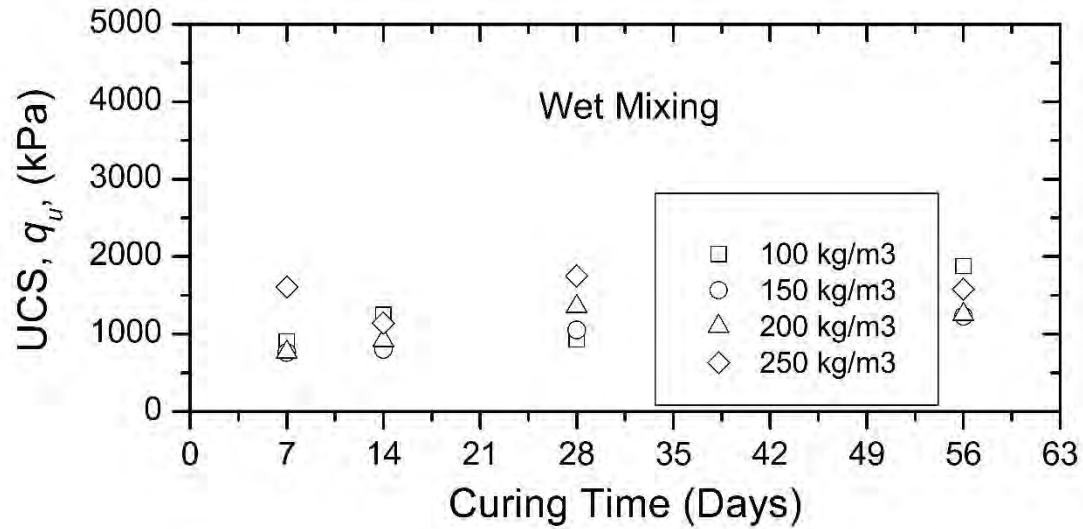


Figure 4.14 Impact of curing time on q_u of samples

From Figure 1.1, it can be said that the soil properties of Sample 2 and 3 (from 5.49 m to 9.14 m) are quite different from Sample 1 (3.66 m to 5.49 m depth), soil from lower depth is weaker compared to Sample 1. A total of 8 out of 32 samples (4 dry mixing and 4 wet mixing) were prepared from Sample 1 for the 250 kg/m³ cement dosage samples. The difference in the geotechnical properties of native soil will impact the treatment results. It can not make any comparison on this impact on q_u in this study due to different dosage usage. This will be investigated in the future.

q_u is decreasing exponentially with increasing total water to cement ratio, as shown in Figure 4.15. It implies the possibility of specifying the total water to cement ratio for strength control of DSM.

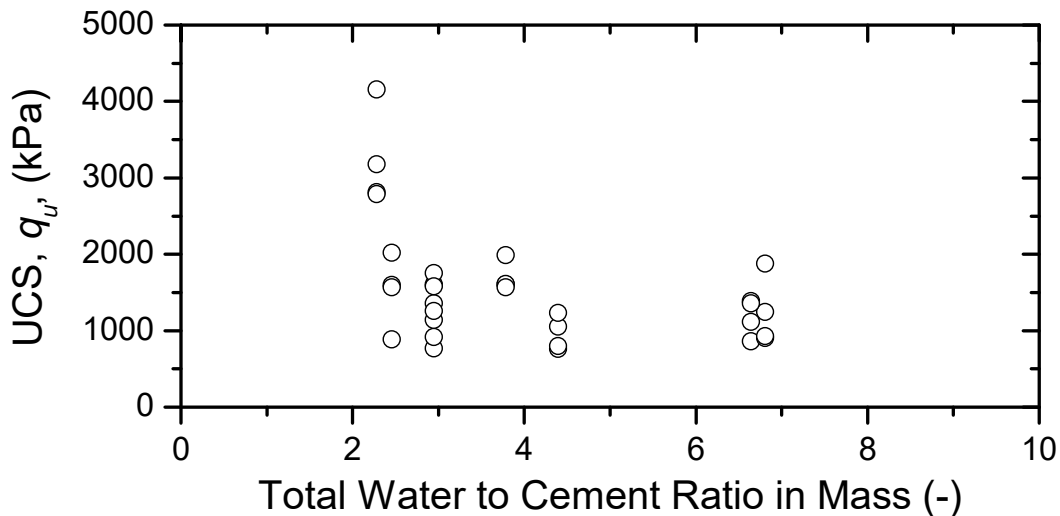


Figure 4.15 Impact of total water to cement ratio on q_u of all samples

A preliminary statistical analysis is conducted based on the UCS test results. The average value of q_u is 1553 kPa with a standard deviation of 763 kPa for all 32 samples. More detailed analyses are shown in Table 4.2.

Table 4.2 Statistical analysis of UCS test results

Cases	Average value of q_u (kPa)	Standard deviation
32 samples	1553	763
24 samples excluding Sample 1 samples	1278	389
8 samples from Sample 1 samples*	2377	1022
All 16 dry mixing samples	1907	899
All 16 wet mixing samples	1199	356

*_Sample 1 clay samples have the highest dosage of 250 kg/m³

4.3 SUMMARY

A total of 32 UCS tests were conducted on samples treated with cement dosages ranging from 100 kg/m³ to 250 kg/m³ and cured from 7 days to 56 days. Based on these UCS values, it can be found cement can quickly increase the shear strength of Champlain Sea clay at the project site. A clear trend of UCS increase with the dry density of samples is noticed from the results. The dry mixing method can generate higher UCS values with a larger scatter than the wet mixing method. However, some inconsistencies can be seen in the UCS test results due to stickiness of the mixture while compaction in the molds and difficulties to produce consistent samples, particularly in dry mixing samples.

5 CONSTANT RATE OF STRAIN COMPRESSIBILITY TEST

5.1 CRS EQUIPMENT

The CRS testing machine used for this study is an automated consolidation system designed by GEOTAC (Trautwein, 2001). The main components of the CRS test system is shown in Figure 5.1.

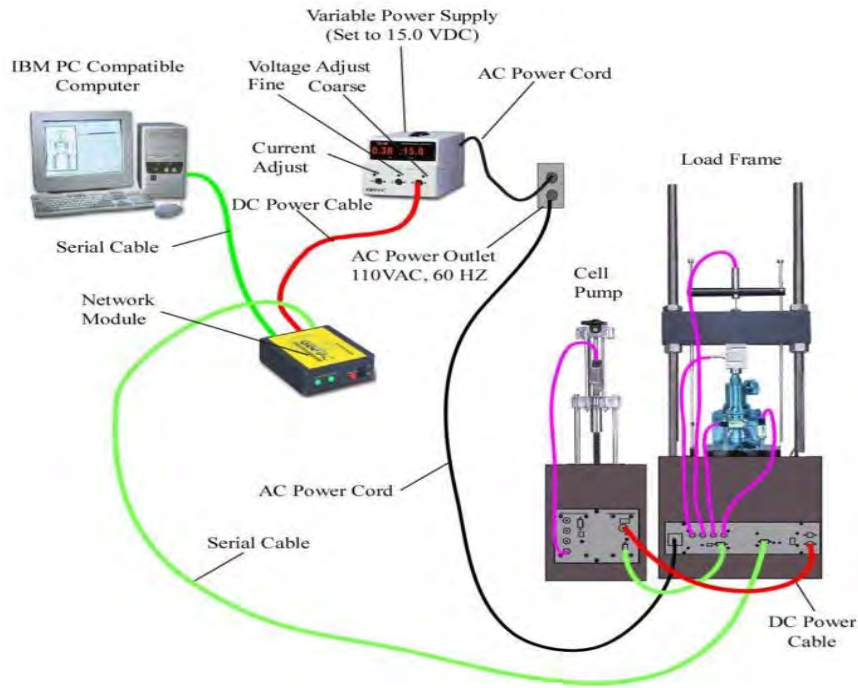


Figure 5.1 Schematic of CRS testing machine components

5.2 SAMPLE PREPARATION

CRS tests were conducted only on treated samples prepared by the wet mixing method. Cement and water were mixed to form a slurry at a water: cement ratio of 0.7:1. The slurry and clay were then manually mixed for 5 minutes using two spatulas. Next, a CRS ring was filled with the mixture. At the final stage, the sample was wrapped in soft nylons and kept sealed prior to placing in the curing chamber for a desired curing time. The water content of the mixture was measured for each sample and a degree of saturation ranging from 90 to 100% confirmed the reasonability of the method used above (see Appendix C for void ratios of samples). However, the inconsistency was noticed with a wide range of initial void ratios recorded among samples. A more consistent method should be developed for the future tests.

After curing, each sample was trimmed and placed onto the machine for testing. The first step was to saturate the sample. A back pressure of about 350 kPa was applied for sample saturation for about 24 hours. The second step was to conduct tests at a loading strain rate of 1.0% /hr to a limit pressure of 2.4 MPa (50 ksf) and then unload the sample at an unloading strain rate of 0.25% /hr to a limit pressure of 0.1 MPa (2 ksf). The same loading and unloading strain rates were applied for all CRS tests. This loading strain rate was selected to keep the pore water pressure to mean effective stress ratio within the recommended range of 3-15 % as per ASTM standard.

5.3 CRS TEST RESULTS AND ANALYSES

Based on the mass and density of each component and a known cement to solid ratio, the initial void ratio of each specimen can be calculated. Detailed calculation of void ratio and CRS test results of each specimen can be found in Appendix C. All calculations of parameters from CRS tests are according to ASTM D4186 (ASTM, 2012). The CRS test curves of each sample can be found in Appendix C.

Figure 5.2 shows schematically the compressibility change of five samples after their compression tests. The remolded sample was prepared from clay without any cement treatment. The sample heights at the beginning were the same. It shows clearly the reduction in compressibility due to cement mixing.

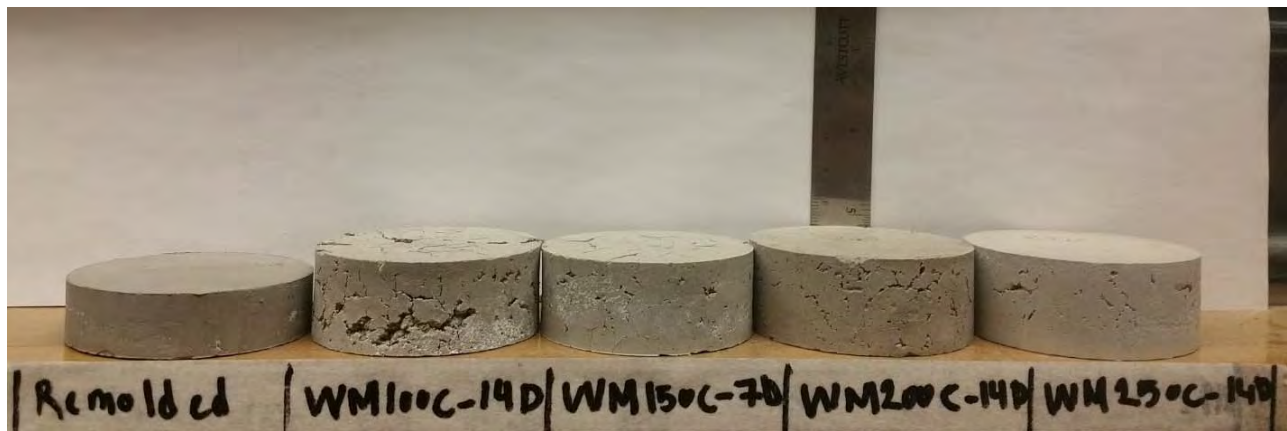


Figure 5.2 Comparison of compressibility change due to cement mixing

The change in the compressibility due to cement mixing is shown in Figure 5.3. Compared to a C_c value of approximately 0.5 for the remolded sample, C_c reduces to less than 0.1 for a pressure range from 100 to 1000 kPa. There is no apparent yielding for all cement treated samples until after 1 MPa.

For the 150 kg/m^3 dosage sample, two CRS tests were conducted on samples cured at 7 days and 14 days. It can be found that there is almost no compressibility change after 7 days. It confirms the same finding in the UCS tests that cement can improve the strength and compressibility of clay at the project site within 7 days in this study. After that there is no prominent change with time observed in the tests.

The large scatter in initial void ratios shows the need for a standardized sample preparation, particularly for small mixed samples. This will be investigated in the future study.

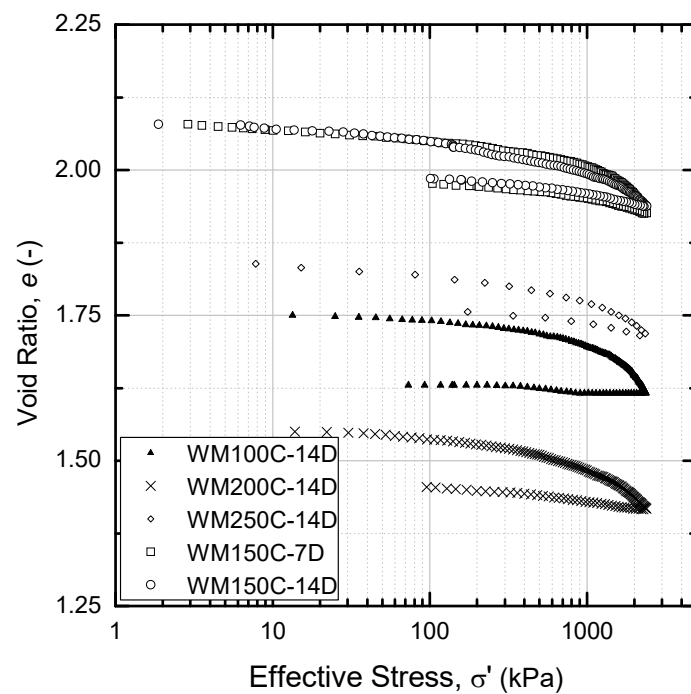
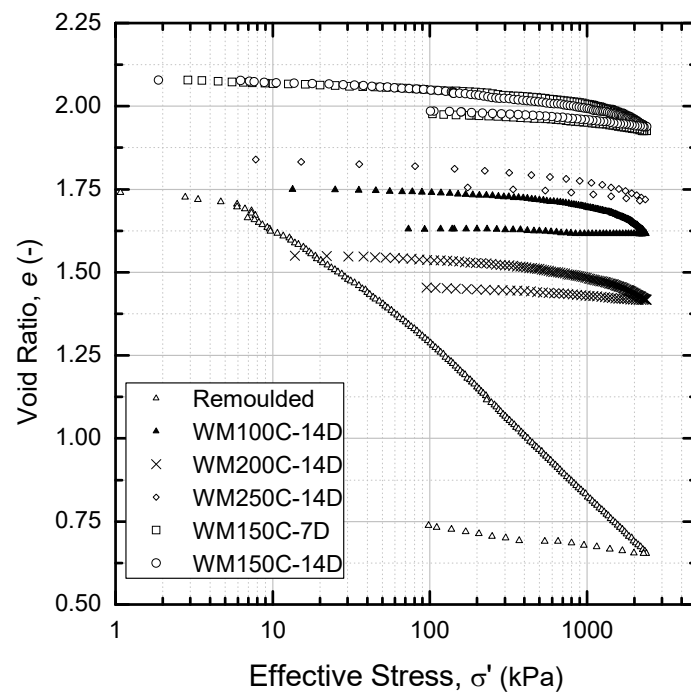


Figure 5.3 Compressibility change due to cement mixing

The change in the coefficient of volume change, M_v , defined as $\Delta\varepsilon_v/\Delta\sigma_v$, due to cement mixing is shown in Figure 5.4. M_v reduces at least one order of magnitude due to cement mixing. No appreciable impact on M_v from different cement dosages was found in the tested samples.

The impact on hydraulic conductivity due to cement mixing is shown in Figure 5.5. It can be found that the permeability change of cement-treated sample is in a much wider range than the remolded sample. The permeability coefficient of cement-treated soil is less than that of the remolded clay sample. However, all the coefficients of consolidation for cement-treated samples are larger than that of the untreated remolded clay sample, as shown in Figure 5.6. It is mainly due to a higher reduction of compressibility.

5.4 SUMMARY

A series of CRS tests were conducted on a disturbed untreated clay sample and cement-treated clay samples. Based on the test results, it can be found that cement-treated samples did not yield until axial pressure higher than 1MPa. In addition, a significant reduction was recorded in the compressibility of all cement-treated samples compared to untreated disturbed sample. In summary, cement is very efficient to reduce the compressibility of Champlain Sea clay at the project site.

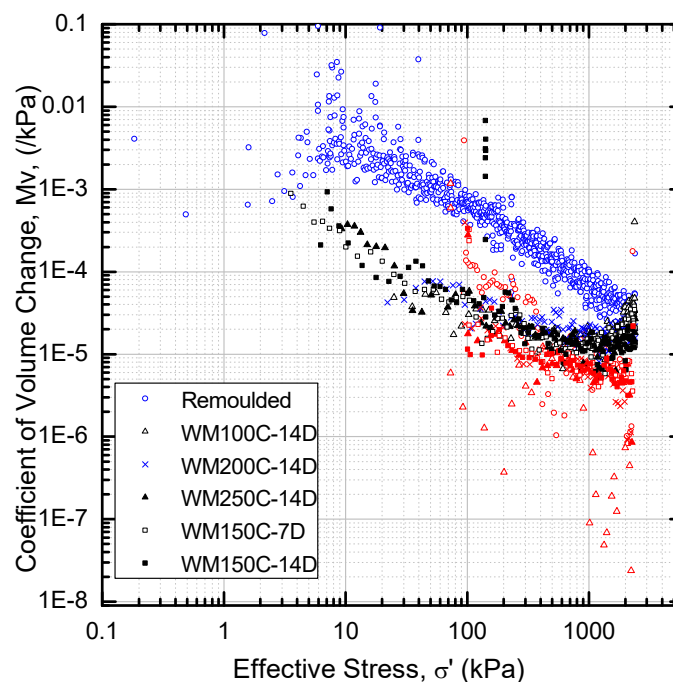


Figure 5.4 Change in coefficient of volume change due to cement mixing

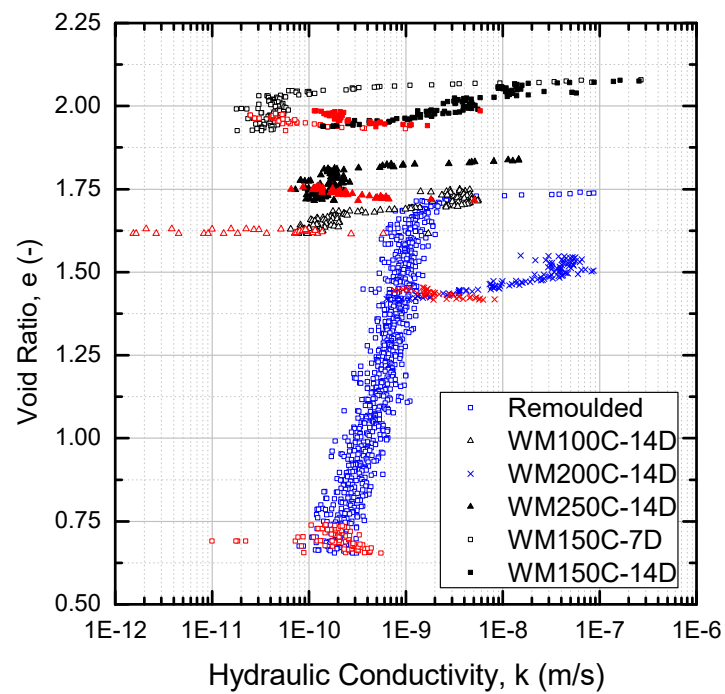


Figure 5.5 Change in hydraulic conductivity due to cement mixing

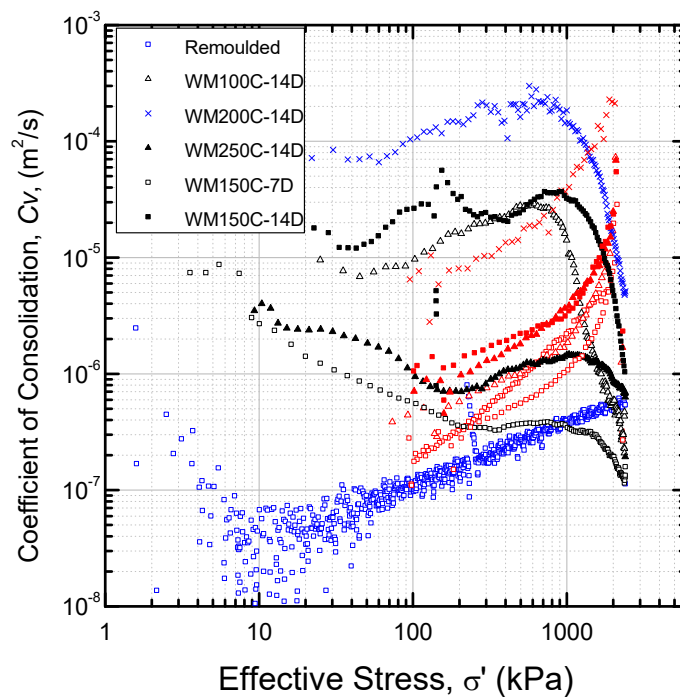


Figure 5.6 Change in coefficient of consolidation due to cement mixing

6 SUMMARY AND FUTURE RESEARCH

The efficiency of treating sensitive Champlain Sea clay with ordinary Portland cement was investigated in this study. Disturbed clay samples were collected from a project site near the intersection of Highway 401 and Country Road 2/34 in Lancaster, Ontario. Clay from the site was mixed with cement with both dry and wet mixing methods at different dosages and curing durations. A series of geotechnical tests were conducted on both native clay samples and treated clay samples, including UCS tests and CRS consolidation tests.

Based on the test results, it is found that cement can significantly improve the strength of Champlain Sea clay at the project site and reduce its compressibility. Regardless of cement dosage, curing time, and mixing method, the average value of UCS is 1553 kPa with a standard deviation of 763 kPa for all 32 samples tested in this study.

Compared with different mixing methods, the samples prepared by the dry mixing method tend to exhibit higher UCS values than corresponding samples from the wet mixing method. Due to the difficulty in compacting a stiffer mixture into a consistent dry mixing sample, a large scatter was observed in the UCS values of dry mixing samples compared to the ones from the wet mixing samples. An improved sample preparation procedure is needed for more consistent samples in the future.

Cement can improve the compressibility of site clay within a short time. At 7-day or 14-day curing conditions, the cement-treated soil has already become significantly less compressible than the remolded clay. The treated soil samples did not yield before the axial stress reaches 1MPa. Correspondingly, cement can improve the shear strength of Champlain Sea clay within 7 days curing. After that, there was not much appreciable strength improvement with time observed in the samples. It is believed that the high salinity may contribute to this phenomenon. The impact of pore fluid chemistry is being investigated on the long-term performance of DSM for Champlain Sea clay.

The geotechnical properties of original soil will impact the treatment results. Based on geotechnical investigation at the site, Sample 1 (from 3.66 to 5.49 m depth) soil has different properties than soil from Sample 2 (from 5.49 to 7.32 m depth) and Sample 3 (from 7.32 to 9.14 m depth). In this study, the comparison cannot be easily made due to the fact that only 250 kg/m³ dosage samples were prepared with Sample 1 soil. It may be due to the higher cement dosage, the cement-treated soil samples from Sample 1 exhibits much higher UCS values than samples from deeper depths.

The current findings are preliminary based on the laboratory tests conducted in this study. Future studies will be required to gain a complete knowledge and interaction between Champlain Sea clay and cement, including standardizing sample preparation and laboratory testing procedure, field

performance compared to laboratory tests, and quality control of DSM in Champlain Sea clay in the field.

7 ACKNOWLEDGEMENT

The authors wish to acknowledge the financial support from the Ontario Centres of Excellence and Golders Associates Ltd. The supports from Ms. Kim Lesage, Mr. William Cavers, Mr. Murty Devata of Golders Associates Ltd. and Mr. Tony Sangiuliano of MTO for this project are greatly acknowledged. The helps from two undergraduate Research Assistants, Mr. Moulay Youssef Monsif and Ms. Niloofarsadat Heirani of GeoOptical Research Lab of Ryerson University are also greatly appreciated.

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APPENDICES

Appendix A – INDEX PROPERTIES

Specific Gravity, ASTM D-854		Ryerson University	
PROJECT INFORMATION			
Client	Golder Associates	Boring Number	
Project Name	OCE-DMM	Sample Number	1
Sample Location	Ottawa, ON	Sample Depth	3.66-5.50
Specimen Description		Specimen Remarks	
Name	Trial 1	Trial 2	Trial 3
Flask Number	32	4	
Mass of Flask(g)	167.800	161.650	
Mass of Flask + Water(g)	665.220	659.630	
Mass of water (g)	497.420	497.980	
Temperature of water (deg)	24.000	24.000	
Water density based on table (g/mL)	0.997	0.997	
Volume of flask (mL)	498.752	499.314	
Mass of wet flask (g)	167.810	162.180	
Mass of soil+ Flask (g)	217.910	212.100	
Mass of soil (g)	50.100	49.920	
Mass of flask+ soil+water+soil (g)	696.830	691.700	
Temperature of water (deg)	25.000	24.000	
Mass of water (g)	478.920	479.600	
density of water (g/mL)	0.997	0.997	
Volume of water (mL)	480.324	480.884	
Volume of soil (mL)	18.428	18.429	
Density of soil (g/mL)	2.719	2.709	
Specific Gravity	2.7266	2.7160	
Specific Gravity (Average)	2.721		
Test Date	Dec. 18, 2017	Tested by	NH
Check Date	Dec. 24, 2017	Checked by	JL

Specific Gravity, ASTM D-854		Ryerson University	
PROJECT INFORMATION			
Client	Golder Associates	Boring Number	
Project Name	OCE-DMM	Sample Number	2
Sample Location	Ottawa, ON	Sample Depth	5.50-7.3
Specimen Description		Specimen Remarks	
Name	Trial 1	Trial 2	Trial 3
Flask Number	34	15	
Mass of Flask(g)	188.510	185.060	
Mass of Flask + Water(g)	686.070	682.360	
Mass of water (g)	497.560	497.300	
Temperature of water (deg)	24.000	24.000	
Water density based on table (g/mL)	0.997	0.997	
Volume of flask (mL)	498.893	498.632	
Mass of wet flask (g)	189.210	185.900	
Mass of soil+ Flask (g)	239.250	235.070	
Mass of soil (g)	50.040	49.170	
Mass of flask+ soil+water+soil (g)	717.460	713.470	
Temperature of water (deg)	25.000	24.000	
Mass of water (g)	478.210	478.400	
density of water (g/mL)	0.997	0.997	
Volume of water (mL)	479.612	479.681	
Volume of soil (mL)	19.281	18.951	
Density of soil (g/mL)	2.595	2.595	
Specific Gravity	2.6030	2.6016	
Specific Gravity (Average)	2.602		
Test Date	Dec. 18, 2017	Tested by	NH
Check Date	Dec. 24, 2017	Checked by	JL

LL, PL, ASTM 4318, , MOISTURE CONTENTS, ASTM D2216 (Oven Dry)					Ryerson University					
PROJECT INFORMATION										
Client	Golder Associates			Boring Number						
Project Name	OCE-DMM			Sample Number	1					
Sample Location	Ottawa, ON			Sample Depth	3.66-5.50					
Specimen Description				Specimen Remarks						
SOIL MOISTURE CONTENT DETERMINATION										
TEST ITEMS		LIQUID LIMIT			PLASTIC LIMIT		MOISTURE CONTENT			
STEPS		Trial 1	Trial 2	Trial 3	Trial 4	Trial 1	Trial 2	Trial 1 / Sample 1	Trial 2 / Sample 2	Trial 3 / Sample 3
No of blows		7	27	19	35					
Dish No.		#2-S6	#3-S2	G3-S72	#3-S2	1S-8	4-S9			
Mass of dish (g)		14.13	14.14	14.33	14.39	14.13	15.09	215.85	215.71	215.47
Mass of wet soil + dish (g)		39.37	45.75	45.17	48.11	18.85	20.72	347.81	349.30	348.22
Mass of dry soil + dish (g)		28.38	34.26	32.1	36.27	17.72	19.42	297.49	296.76	295.66
Mass of water, Mw (g)		10.99	11.49	13.07	11.84	1.13	1.3	50.32	52.54	52.56
Mass of dry soil, Ms (g)		14.25	20.12	17.77	21.88	3.59	4.33	81.64	81.05	80.19
Water content, w (%)		77.12	57.11	73.55	54.11	31.48	30.02	61.64%	64.82%	65.54%
LL:	61.41	STD:	11.55	PL	30.75	STD	1.03	WC	64.00%	
<p style="text-align: center;">Liquid Limit</p> <p style="text-align: center;">Water contents, w (%)</p> <p style="text-align: center;">Number of blows, N</p> <p style="text-align: center;">$y = -14.4\ln(x) + 107.76$ $R^2 = 0.7747$</p>										
Test Date:	17/12/2017	Tested By : NH	Check Date	26-Dec-17	Checked By	JL				

LL, PL, ASTM 4318, , MOISTURE CONTENTS, ASTM D2216 (Oven Dry)						Ryerson University				
PROJECT INFORMATION										
Client	Golder Associates			Boring Number						
Project Name				Sample Number		2				
Sample Location	Ottawa, ON			Sample Depth		5.50-7.3				
Specimen Description				Specimen Remarks						
SOIL MOISTURE CONTENT DETERMINATION										
TEST ITEMS		LIQUID LIMIT				PLASTIC LIMIT		MOISTURE CONTENT		
STEPS		Trial 1	Trial 2	Trial 3	Trial 4	Trial 1	Trial 2	Trial 1 / Sample 1	Trial 2 / Sample 2	Trial 3 / Sample 3
No of blows		42	17	28	12					
Dish No.		#2-S7	G1S4	G4S1	G3S3	G4S2	G4-S4			
Mass of dish (g)		14.11	13.79	14.18	14.09	13.95	14.11	215.85	215.71	215.47
Mass of wet soil + dish (g)		36.84	51.58	36.47	45.26	20.2	19.25	347.81	349.30	348.22
Mass of dry soil + dish (g)		30.18	38.15	29.04	34.06	18.79	18.05	297.49	296.76	295.66
Mass of water, Mw (g)		6.66	13.43	7.43	11.2	1.41	1.2	50.32	52.54	52.56
Mass of dry soil, Ms (g)		16.07	24.36	14.86	19.97	4.84	3.94	81.64	81.05	80.19
Water content, w (%)		41.44	55.13	50.00	56.08	29.13	30.46	61.64%	64.82%	65.54%
LL:	49.26	STD:	6.70	PL	29.79	STD	0.94	WC	64.00%	
Liquid Limit 										
Test Date:	17/12/2017		Tested By : NH		Check Date	26-Dec-17		Checked By	JL	

Salinity, Diluted Fluid Method		Ryerson University	
PROJECT INFORMATION			
Client	Golder Associates	Boring Number	
Project Name	OCE-DMM	Sample Number	1
Sample Location	Ottawa, ON	Sample Depth	3.66-5.50
Specimen Description		Specimen Remarks	
Soil index and salinity reading-1			
Sample depth(m)	3.66	Specific gravity (measured)	2.72
Mass of dry soil(g)	17.31	Water content of original soil	0.64
Volume of mixture(mL)	199.71	Void ratio	1.74
Salinity Reading	0.07	Specimen Remarks/Comments	
Calibration Factor	1.60	salinity reading of distilled water	0.00
Salt Mass Calculation			
Salt Concentration (g/L)	0.11	Mass of salt(g)	0.02
Volume of water of mixture(mL)	193.35	Mass of pore water(g)	11.08
Salinity Calculation			
Ratio of salt mass to volume of pore water (g/L)	1.956	Ratio of salt mass to dry soil mass (g/kg)	1.252
Salinity readings			
Reading of salinity meter	Temperature	time (minutes)	Mass of the mixture (g)
0.05	23.80	4	200.94
0.06	23.80	5	200.94
0.07	23.70	1440	199.71
0.07	23.30	1680	199.71
Test Date	Dec. 18, 2017	Tested by	NH
Check Date	Dec. 24, 2017	Checked by	JL

Salinity, Diluted Fluid Method		Ryerson University	
PROJECT INFORMATION			
Client	Golder Associates	Boring Number	
Project Name	OCE-DMM	Sample Number	2
Sample Location	Ottawa, ON	Sample Depth	5.50-7.3
Specimen Description		Specimen Remarks	
Soil index and salinity reading-1			
Sample depth(m)	5.50-7.35	Specific gravity (measured)	2.6
Mass of dry soil(g)	18.70	Water content of original soil	0.64
Volume of mixture(mL)	208.43	Void ratio	1.66
Salinity Reading	0.06	salinity reading of distilled water	0.00
Calibration Factor	1.60	Specimen Remarks/Comments	
Salt Mass Calculation			
Salt Concentration (g/L)	0.10	Mass of salt(g)	0.02
Volume of water of mixture(mL)	201.24	Mass of pore water(g)	11.97
Salinity Calculation			
Ratio of salt mass to volume of pore water (g/L)	1.615	Ratio of salt mass to dry soil mass (g/kg)	1.034
Salinity readings			
Reading of salinity meter	Tempreture	time (minutes)	Mass of the mixture (g)
0.04	27.40	4	209.87
0.05	27.40	5	209.87
0.06	21.70	1440	208.43
0.06	22.00	1680	208.43
Test Date	Dec. 18, 2017	Tested by	NH
Check Date	Dec. 24, 2017	Checked by	JL

File Name:	Grain Size Distribution
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PROJECT INFORMATION			
Client	Golder Associates	Boring Number	
Project Name	OCE-DMM	Sample Number	1
Sample Location	Ottawa, ON	Sample Depth	3.66-5.49
Specimen Description		Specific Gravity	2.72

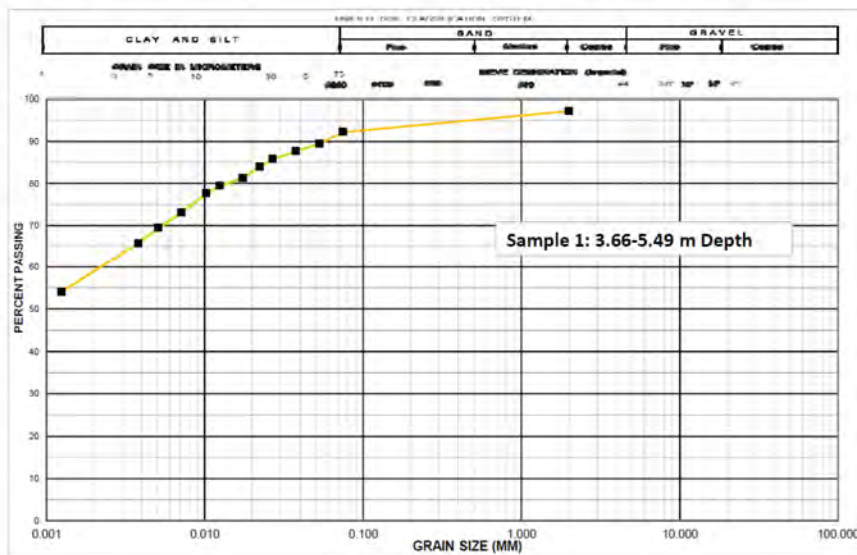
Sieve Analysis					
Sieve No.	Mass of sieve(g)	Mass of Sieve+Soil (g)	Mass of Soil (g)	Percent Retained (%)	Percent Passing (%)
10	482.12	483.6	1.48	2.82	97.18
200	337.2	339.8	2.6	7.77	92.23

Total dry Mass (g)	52.5	Hydrometer	152
Cum- M	48.42	Adjustment Factor (a)	0.98

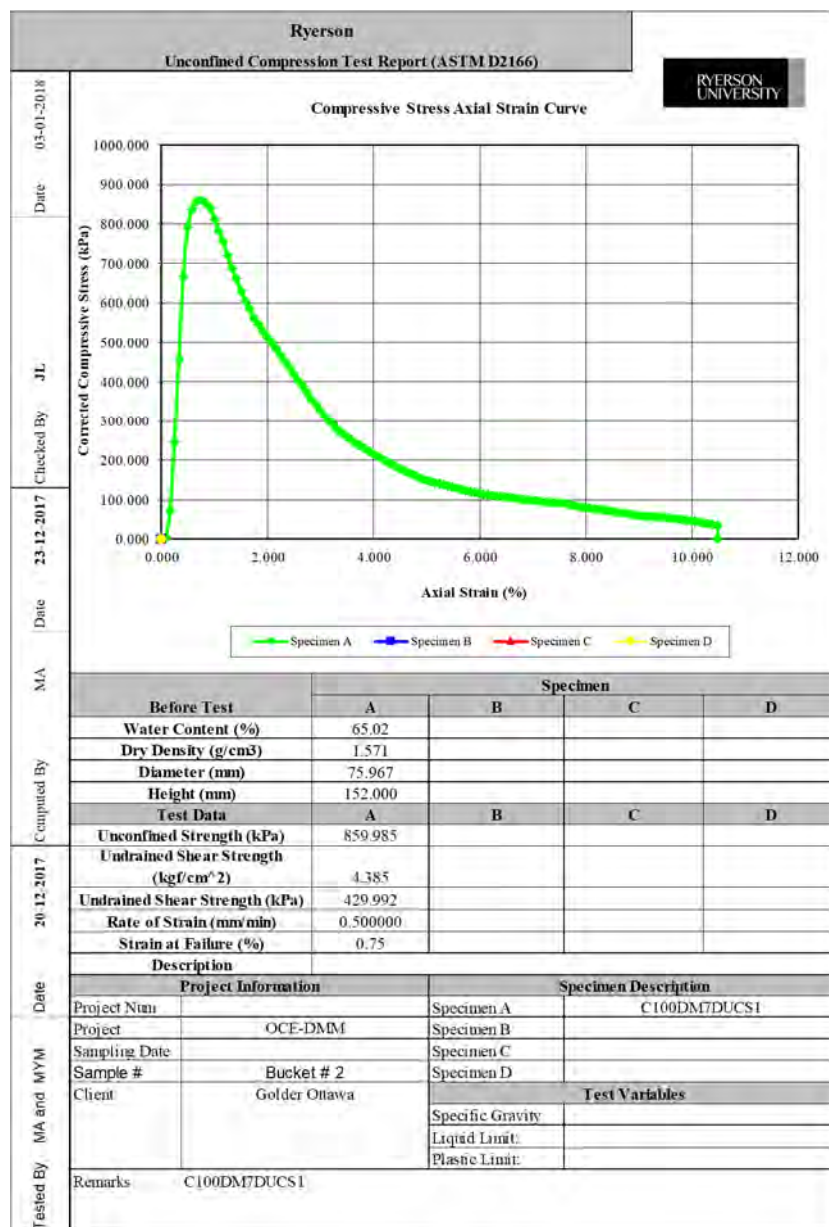
Start time		4:00 PM		Mass of fine (g)	
Time	Time Interval (min)	Temperature of Slurry	Reading of Slurry	Temperature of Distilled water	Reading of Distilled Water
4:00:30 PM	0.5	24.0	47.0	24.0	0.5
4:01:00 PM	1.0	24.0	46.0	24.0	0.5
4:03:00 PM	2.0	24.0	45.0	24.0	0.5
4:06:00 PM	3.0	24.0	44.0	24.0	0.5
4:11:00 PM	5.0	24.0	42.5	24.0	0.5
4:21:00 PM	10.0	24.0	41.5	24.0	0.5
4:36:00 PM	15.0	24.0	40.5	24.0	0.5
5:08:00 PM	32.0	24.0	38.0	25.0	0.5
6:14:00 PM	65.0	24.0	36.0	24.0	0.5
8:14:00 PM	120.0	24.0	34.0	24.0	0.5
5:42:00 PM	1288.0	22.0	27.0	23.0	1.0

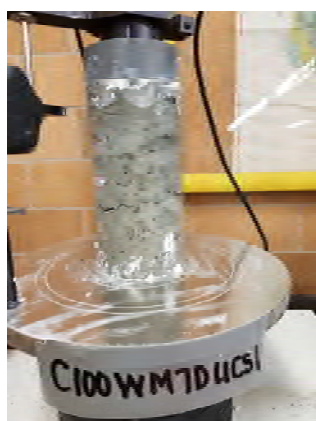
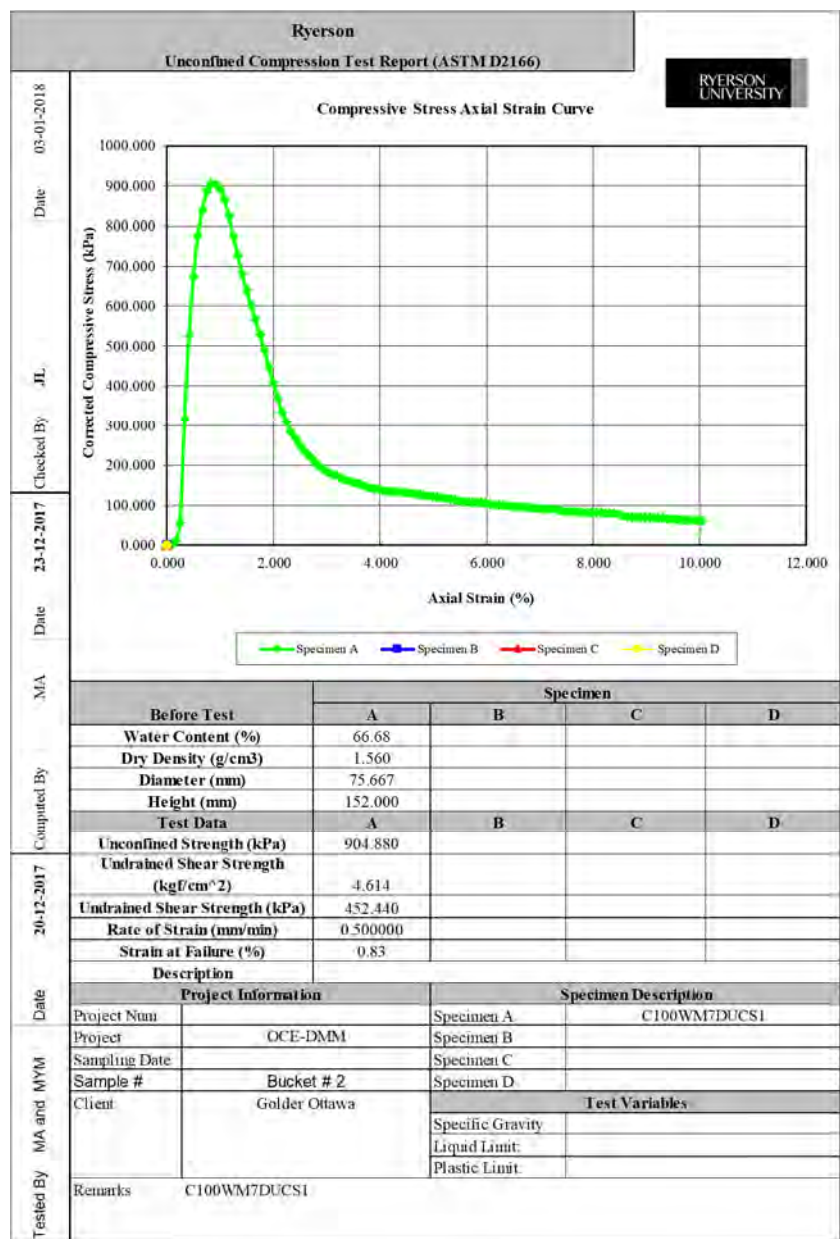
Calculations								
Corr. Hdr. Rdg	Meniscus Corr.	Corr. Rdg	Percent Finer	L	L/t	K	D	PF*
							2.000	97.18
							0.075	92.23
0.58	1	49.080	92.050	87.000	174.000	0.0040	0.053	89.46
0.58	1	48.080	90.174	88.000	88.000	0.0040	0.038	87.63
0.58	1	47.080	88.299	90.000	45.000	0.0040	0.027	85.81
0.58	1	46.080	86.423	92.000	30.667	0.0040	0.022	83.99
0.58	1	44.580	83.610	94.000	18.800	0.0040	0.017	81.25
0.58	1	43.580	81.735	96.000	9.600	0.0040	0.012	79.43
0.58	1	42.580	79.859	97.500	6.500	0.0040	0.010	77.61
0.58	1	40.080	75.170	101.000	3.156	0.0040	0.007	73.05
0.58	1	38.080	71.419	105.000	1.615	0.0040	0.005	69.41
0.58	1	36.080	67.668	108.000	0.900	0.0040	0.004	65.76
0.63	1	29.630	55.571	118.000	0.092	0.0041	0.001	54.00

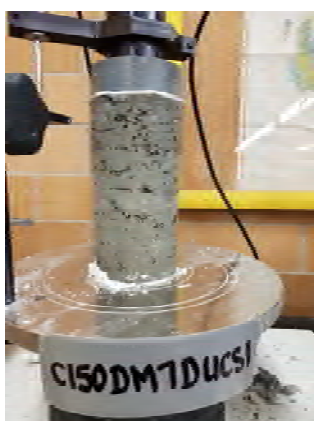
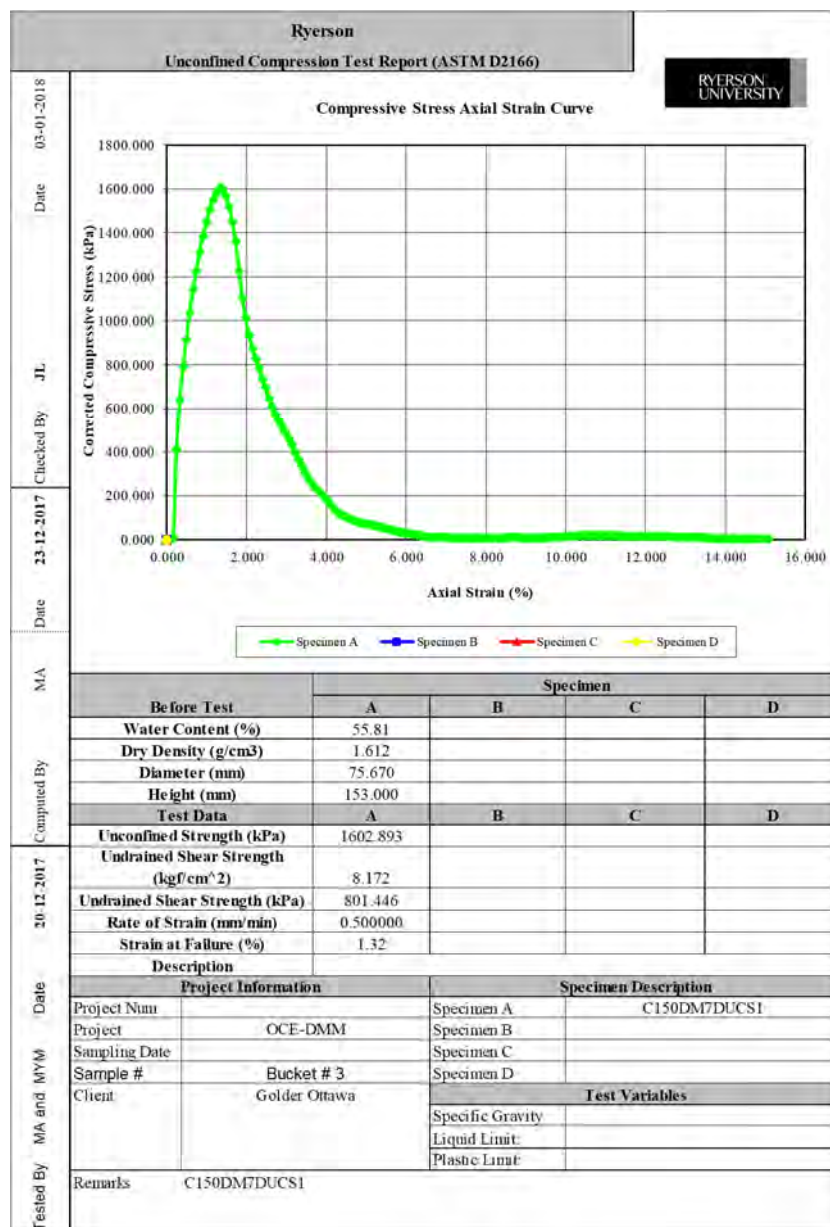
Test Date	1/14/2018	Tested By	NH
Check Date	1/16/2018	Checked By	JL

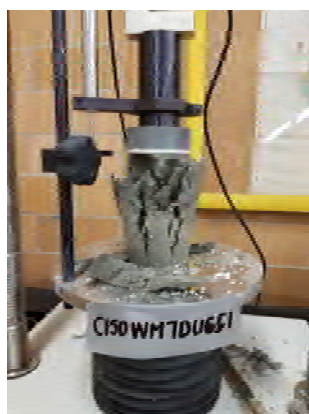
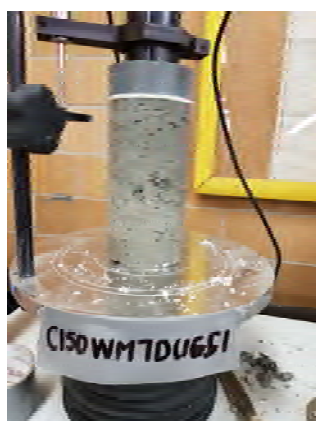
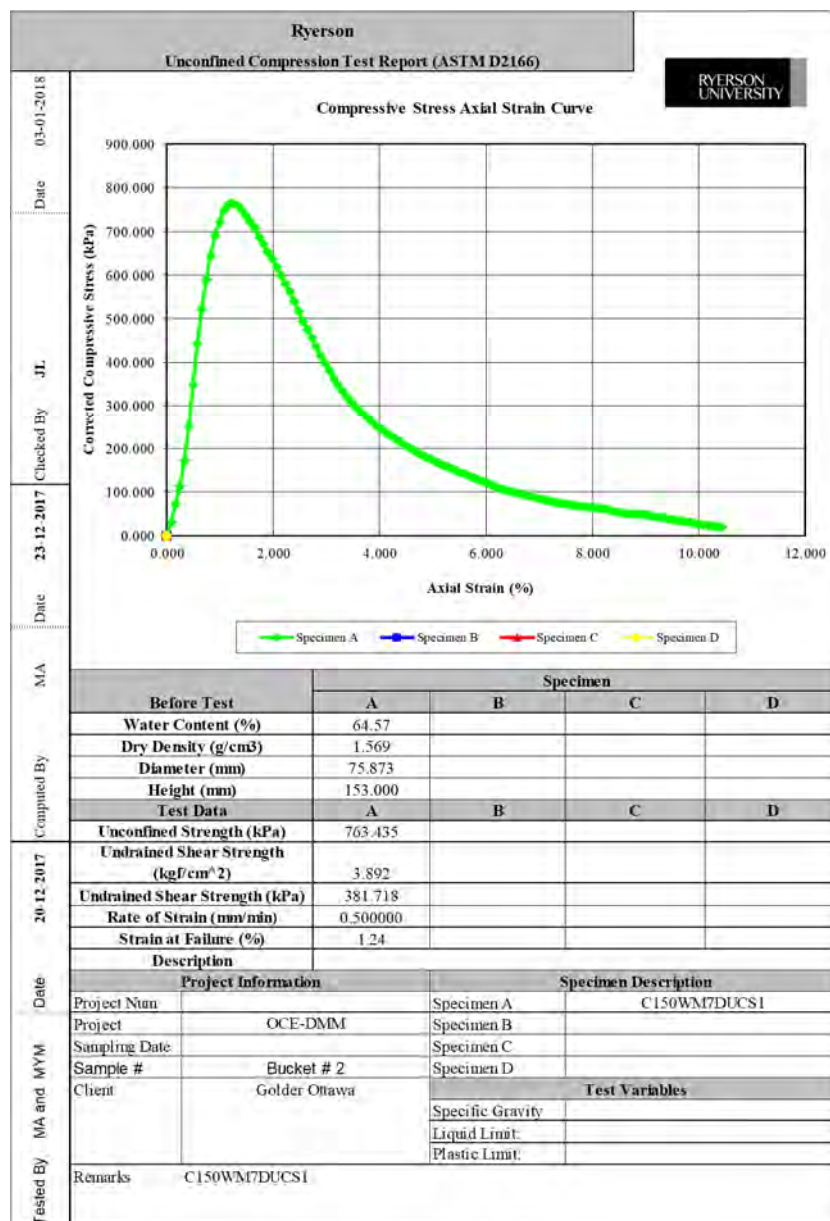


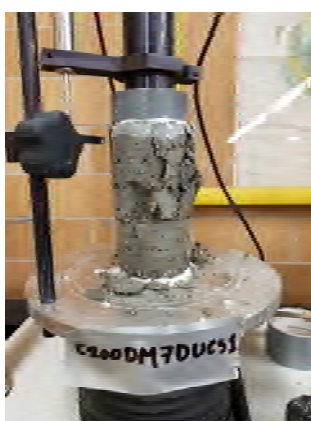
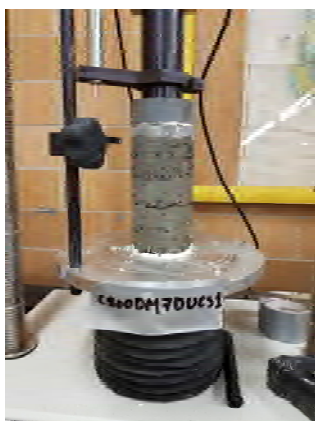
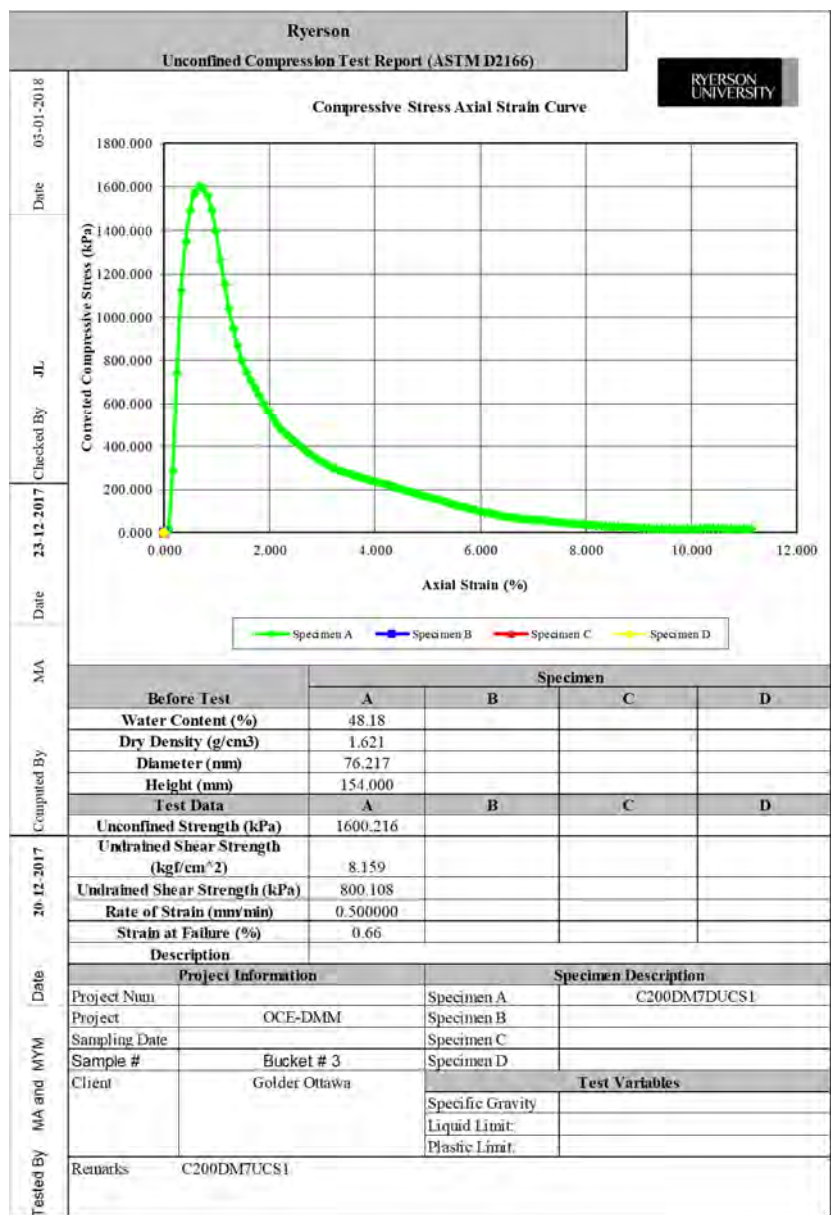
Appendix B – UCS TEST RESULTS

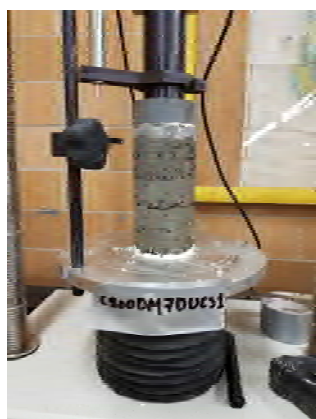
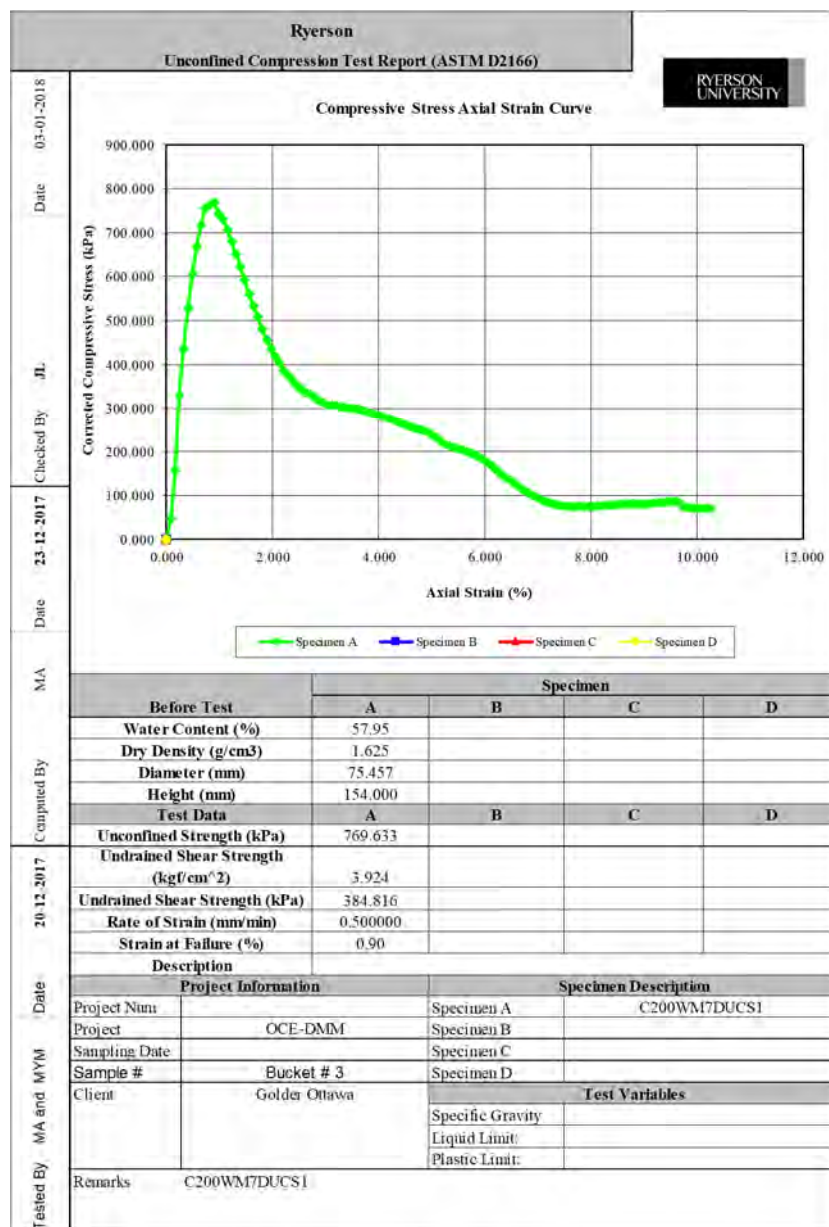


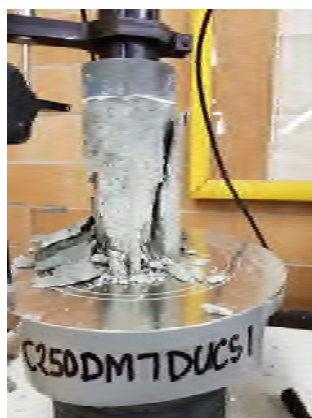
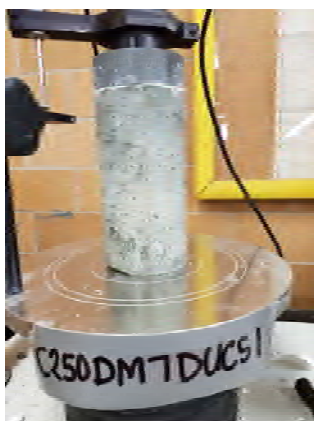
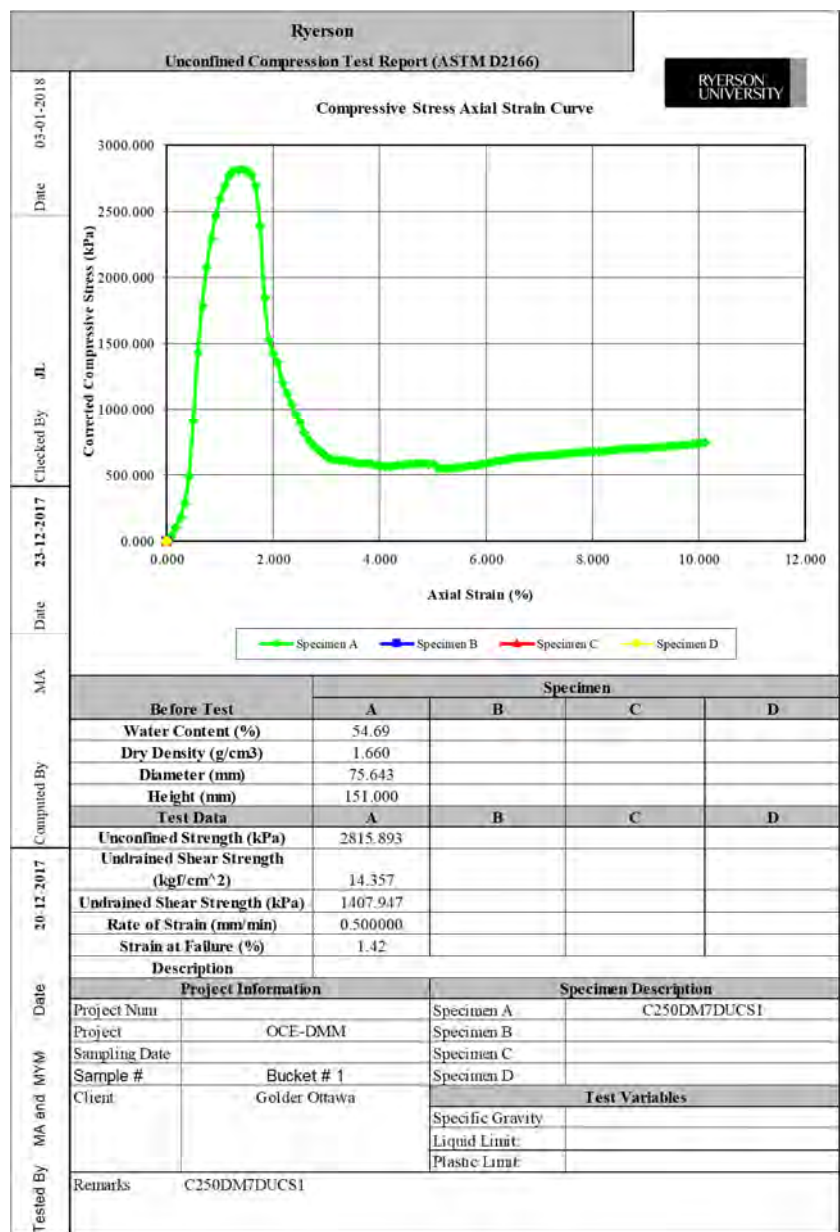


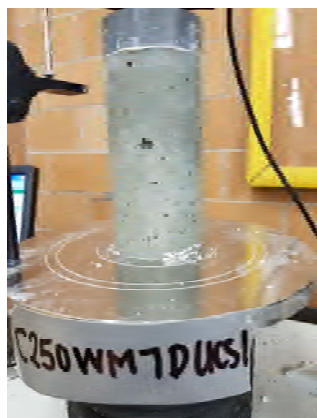


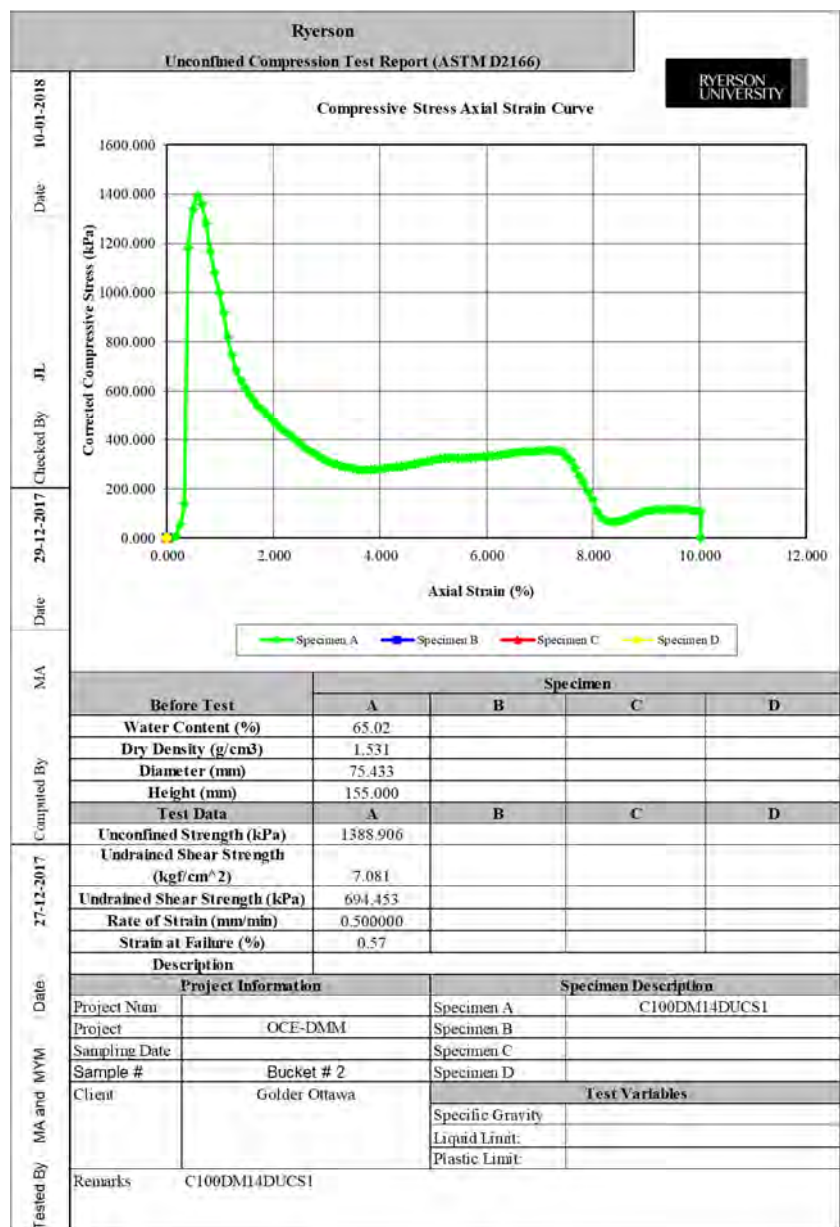


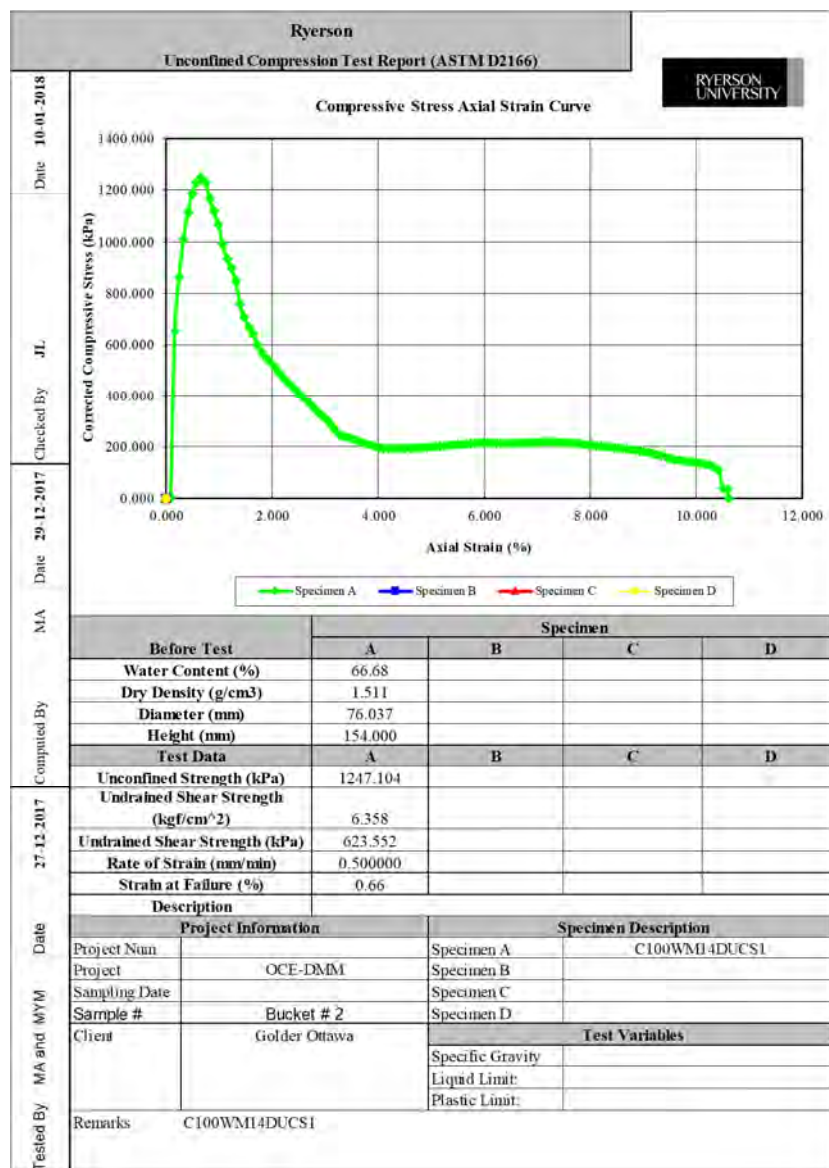


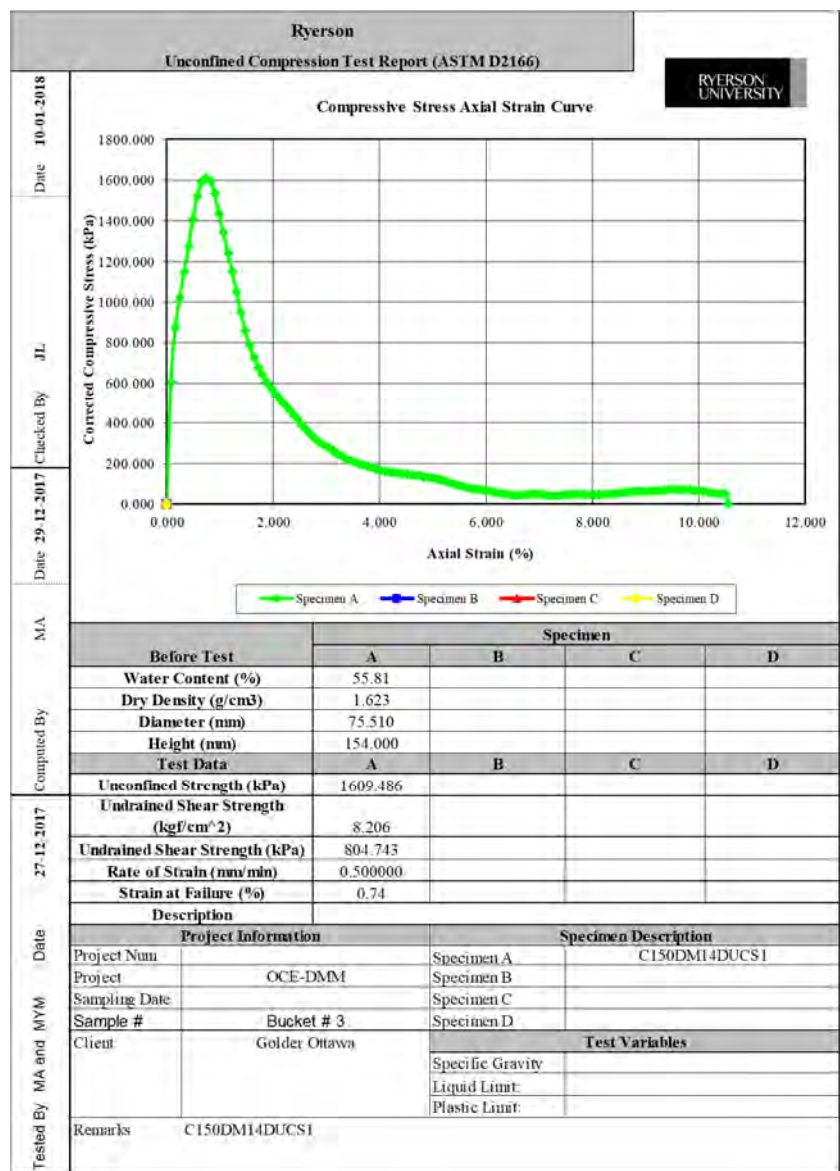


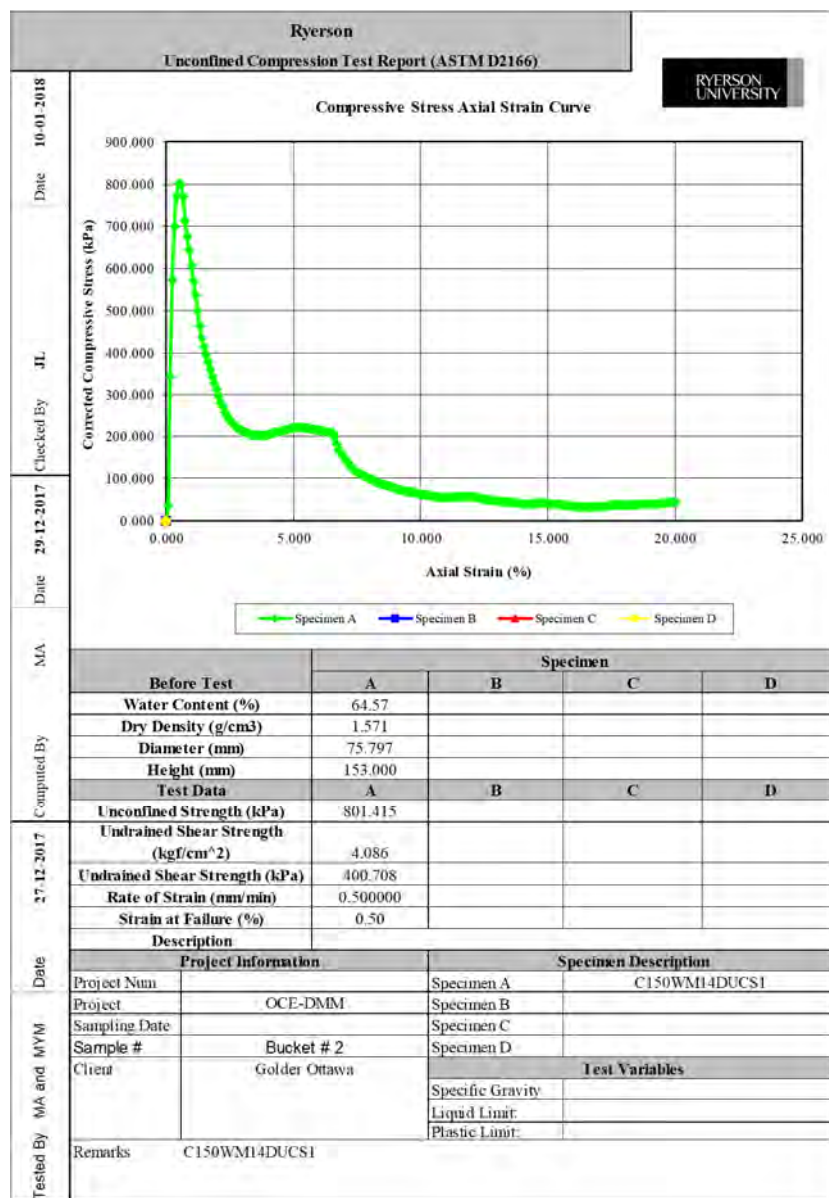


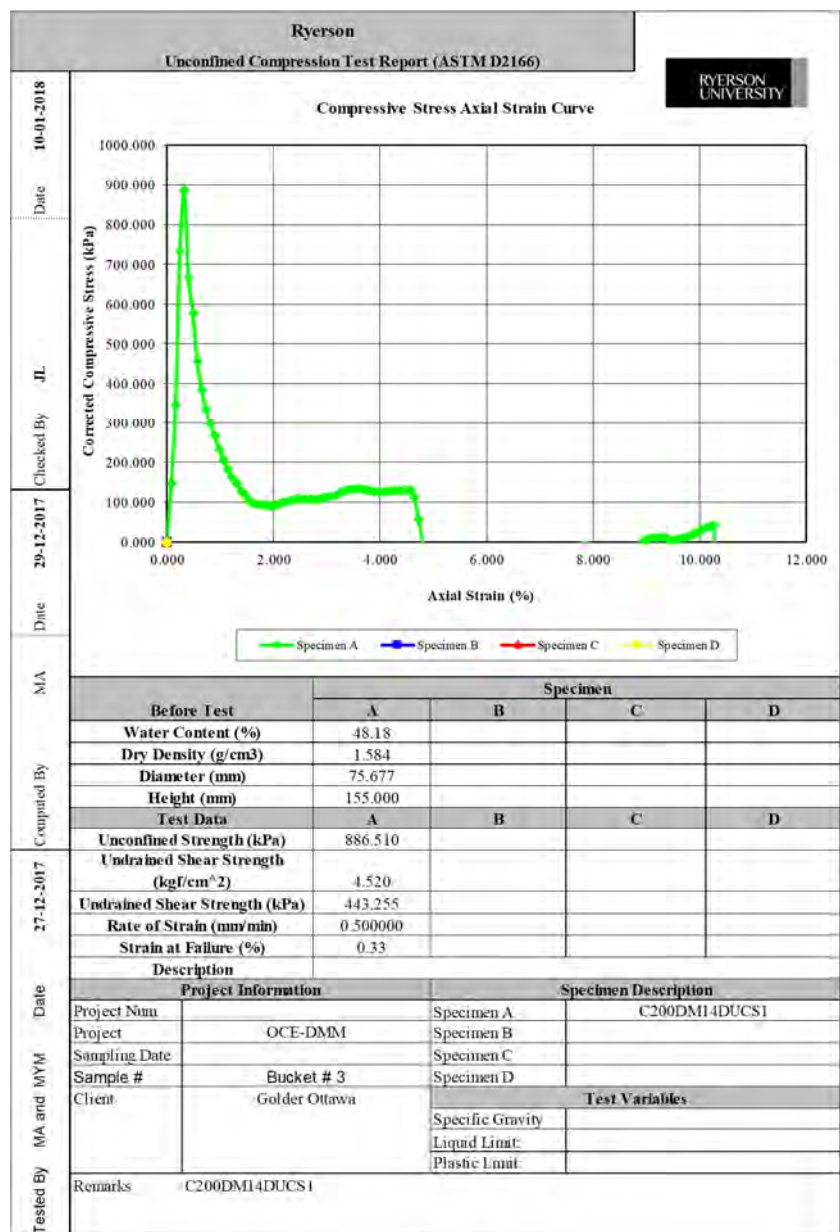


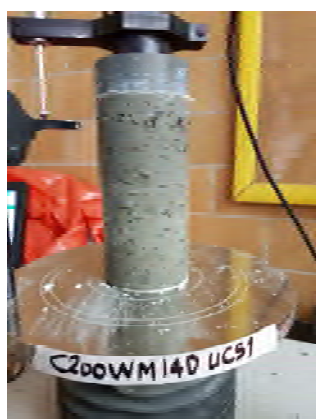
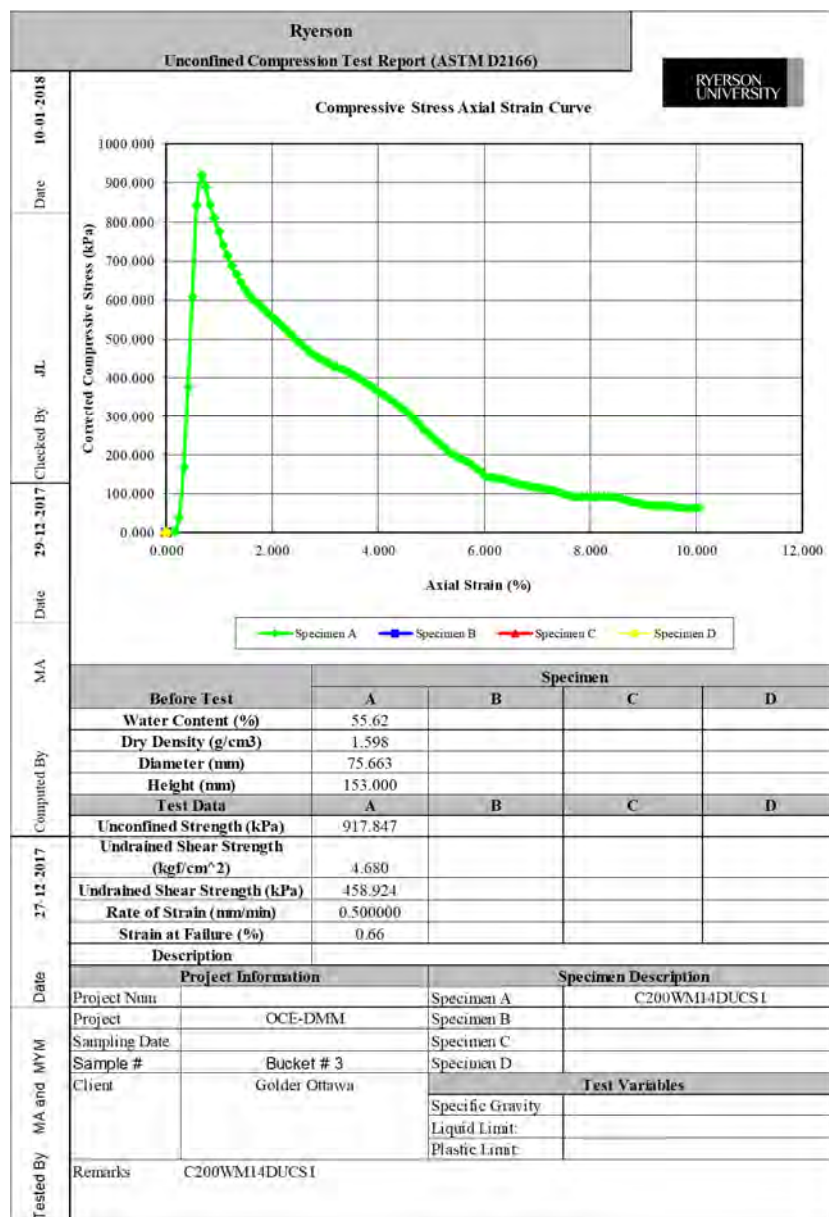


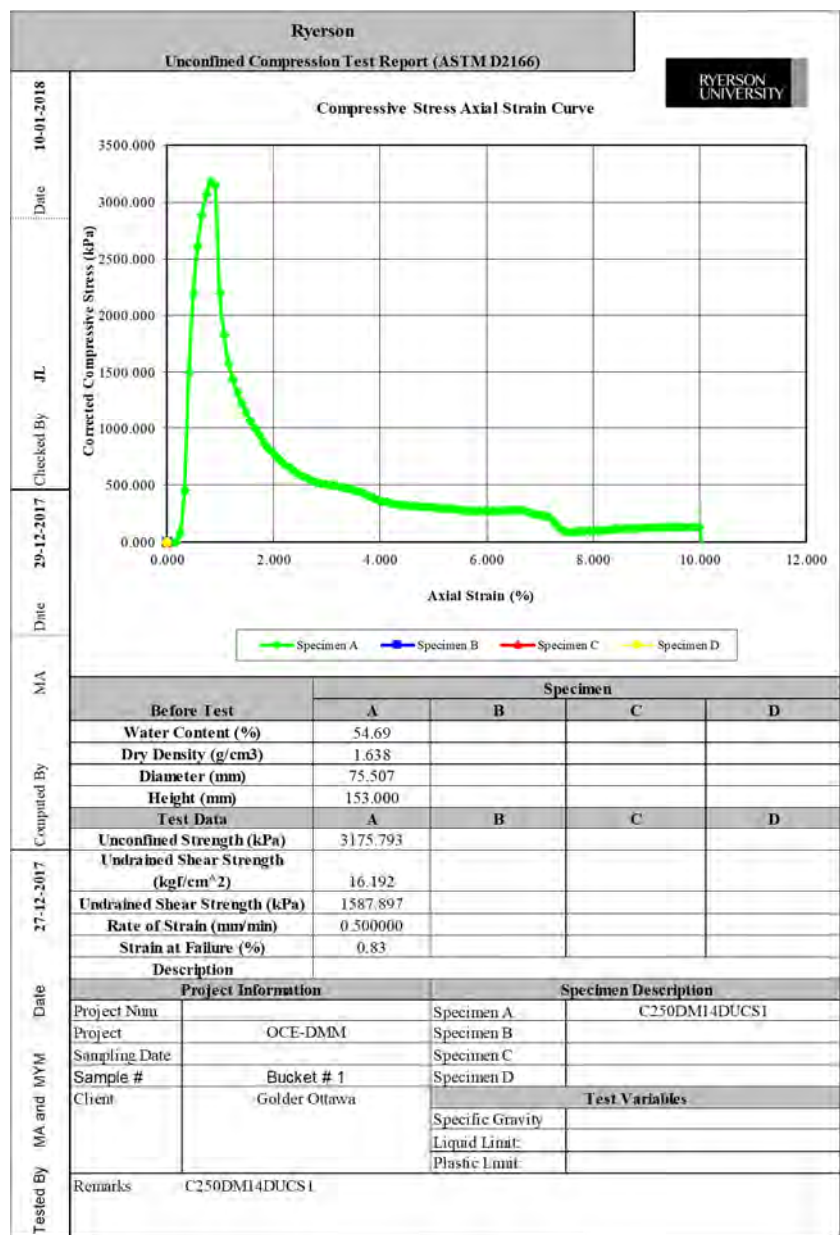


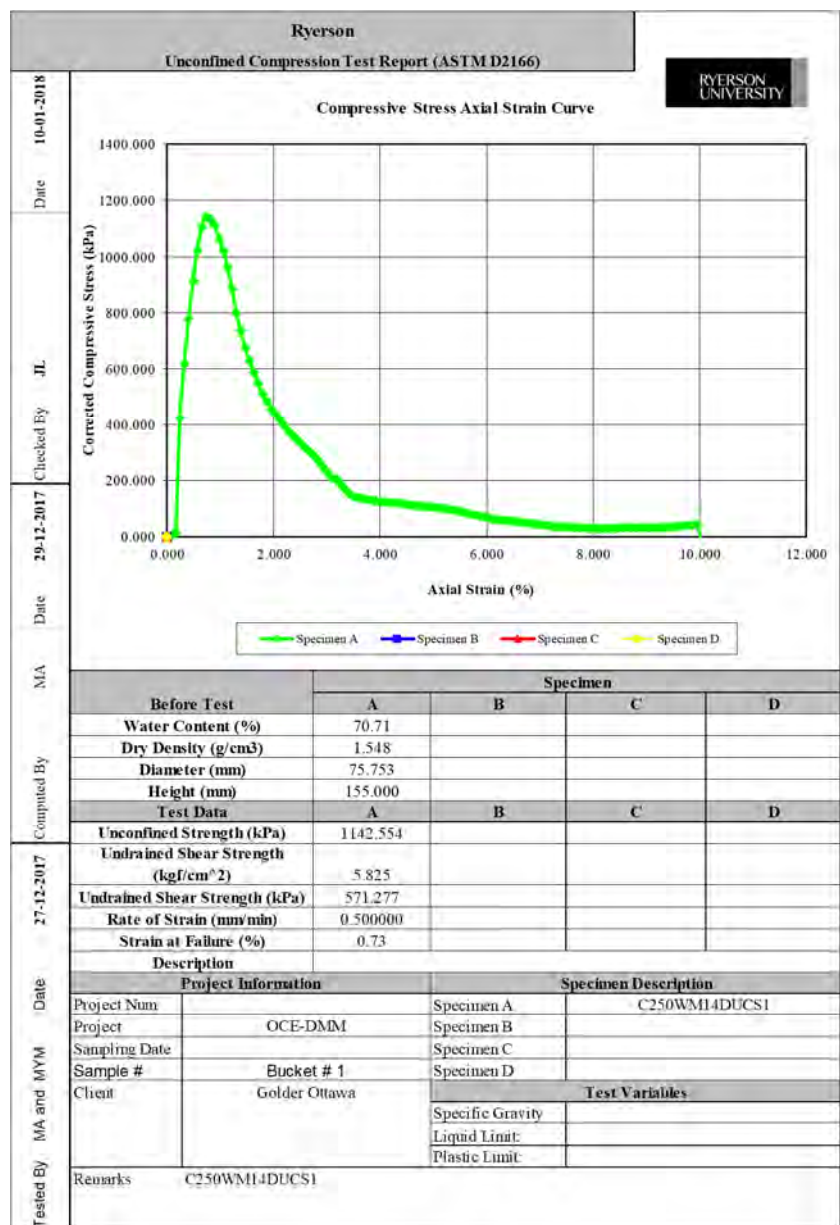


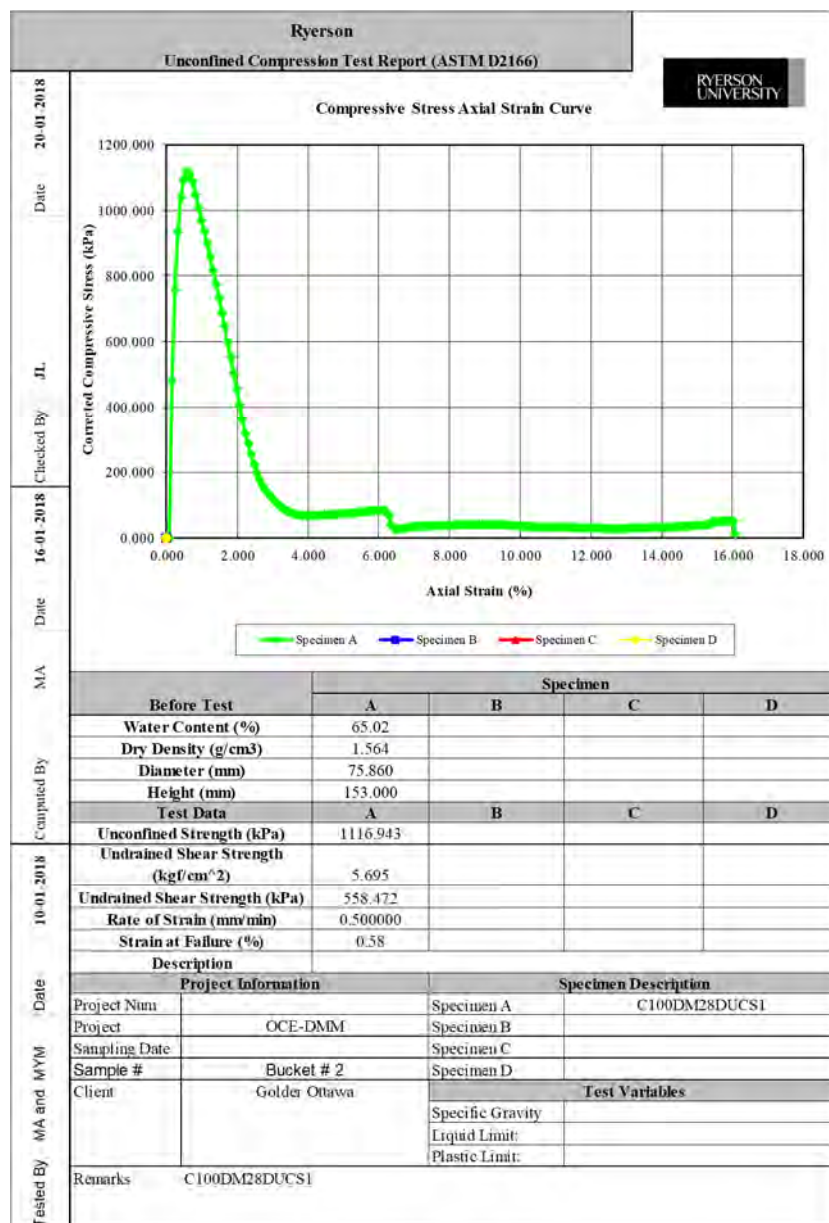


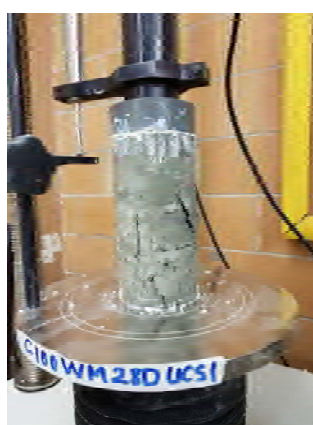
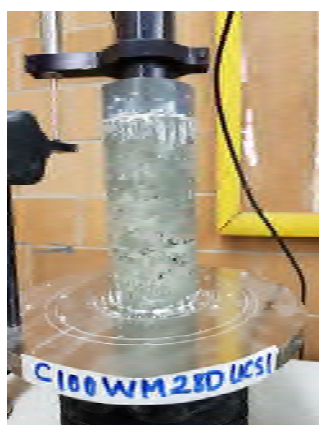
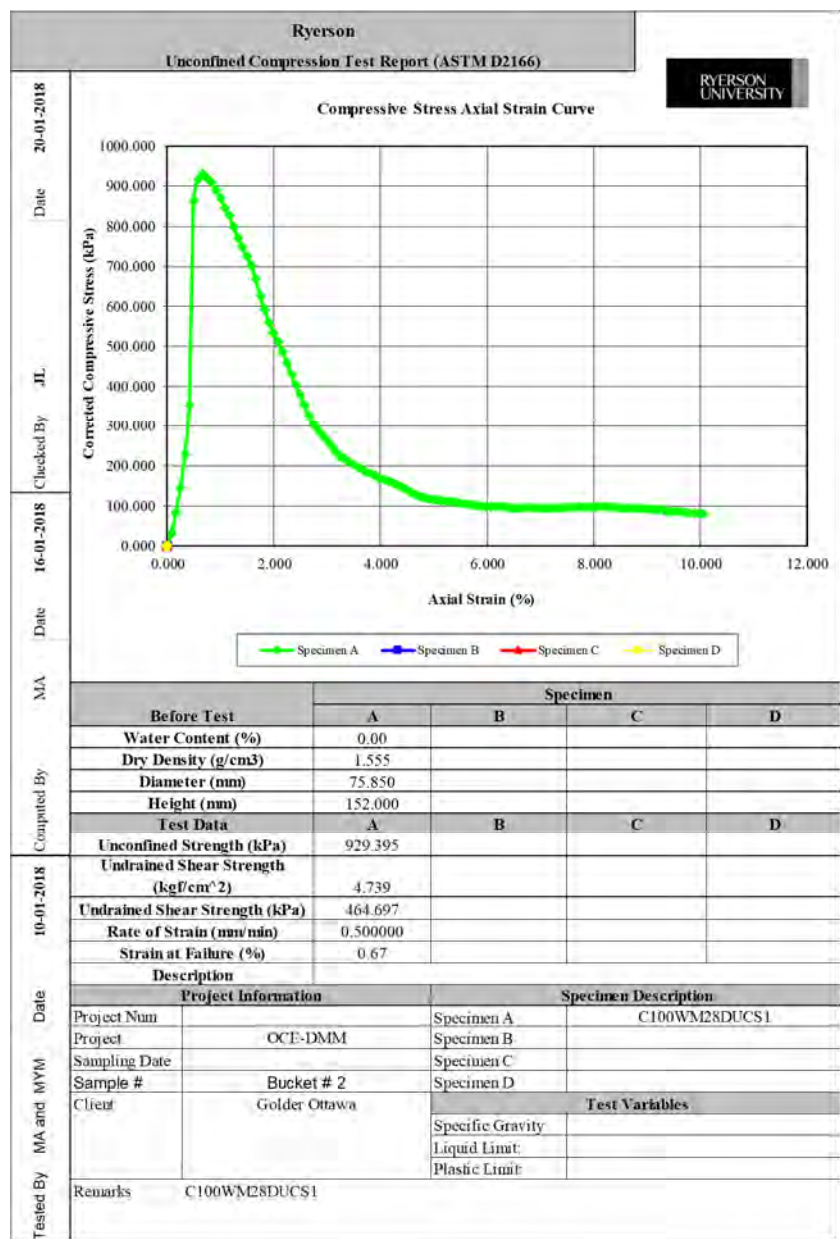


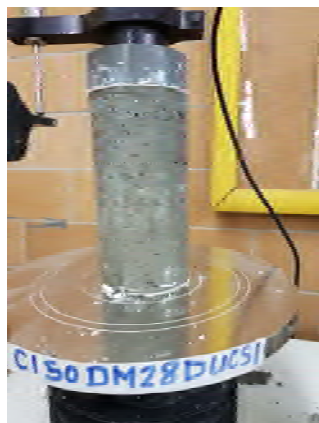
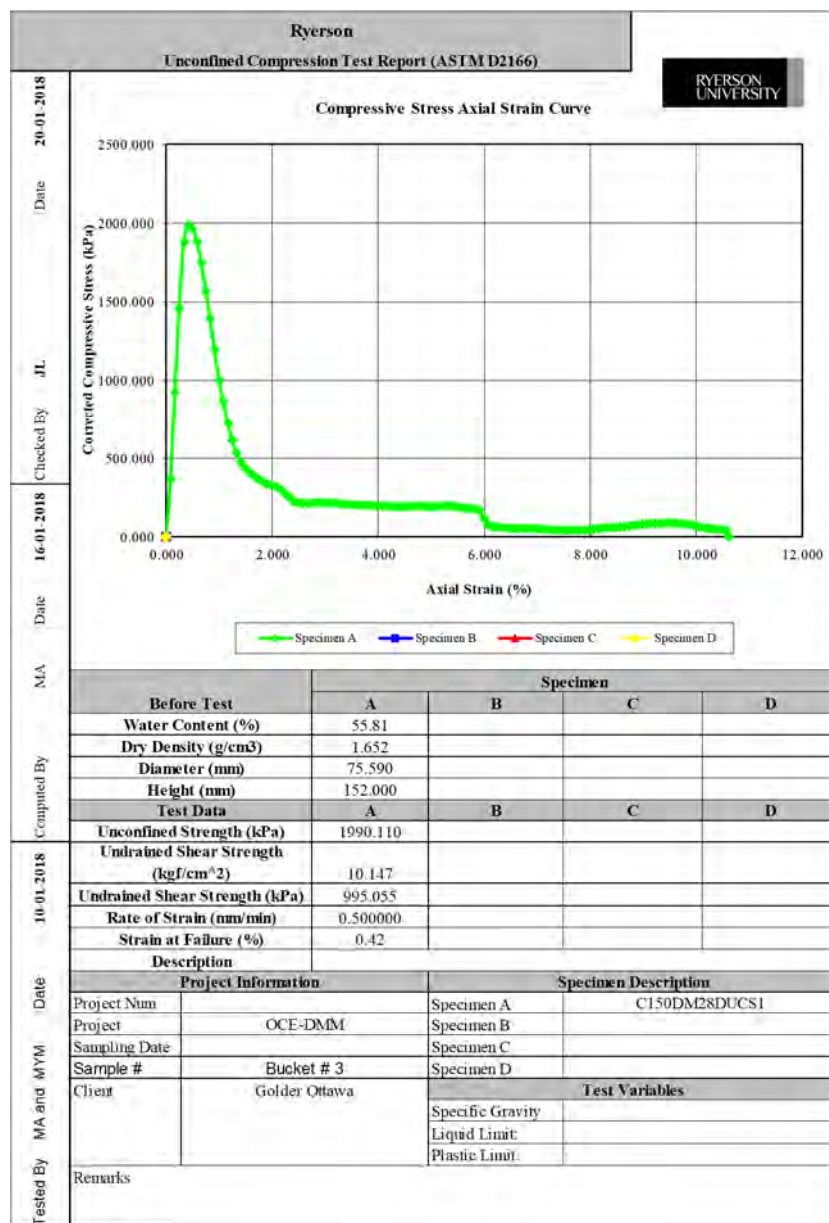


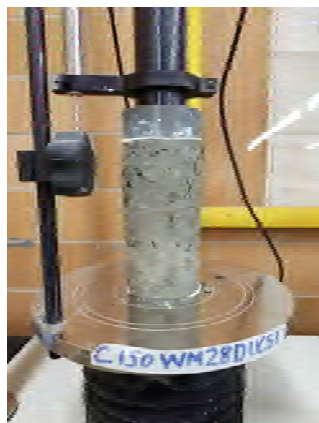
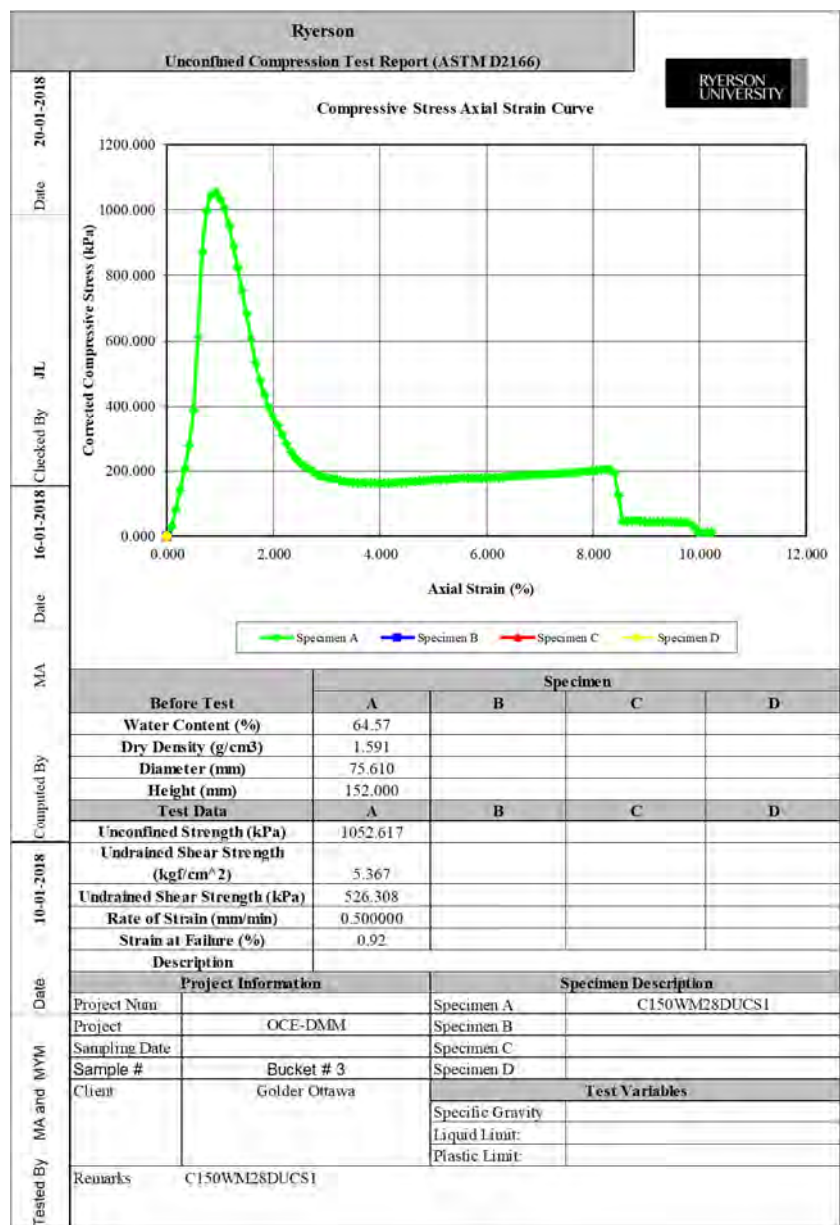


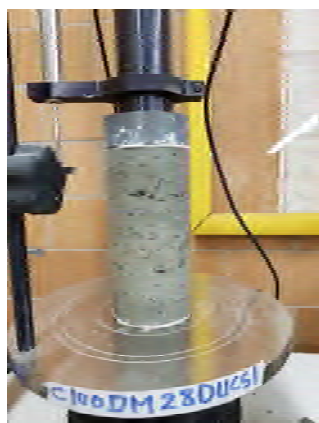
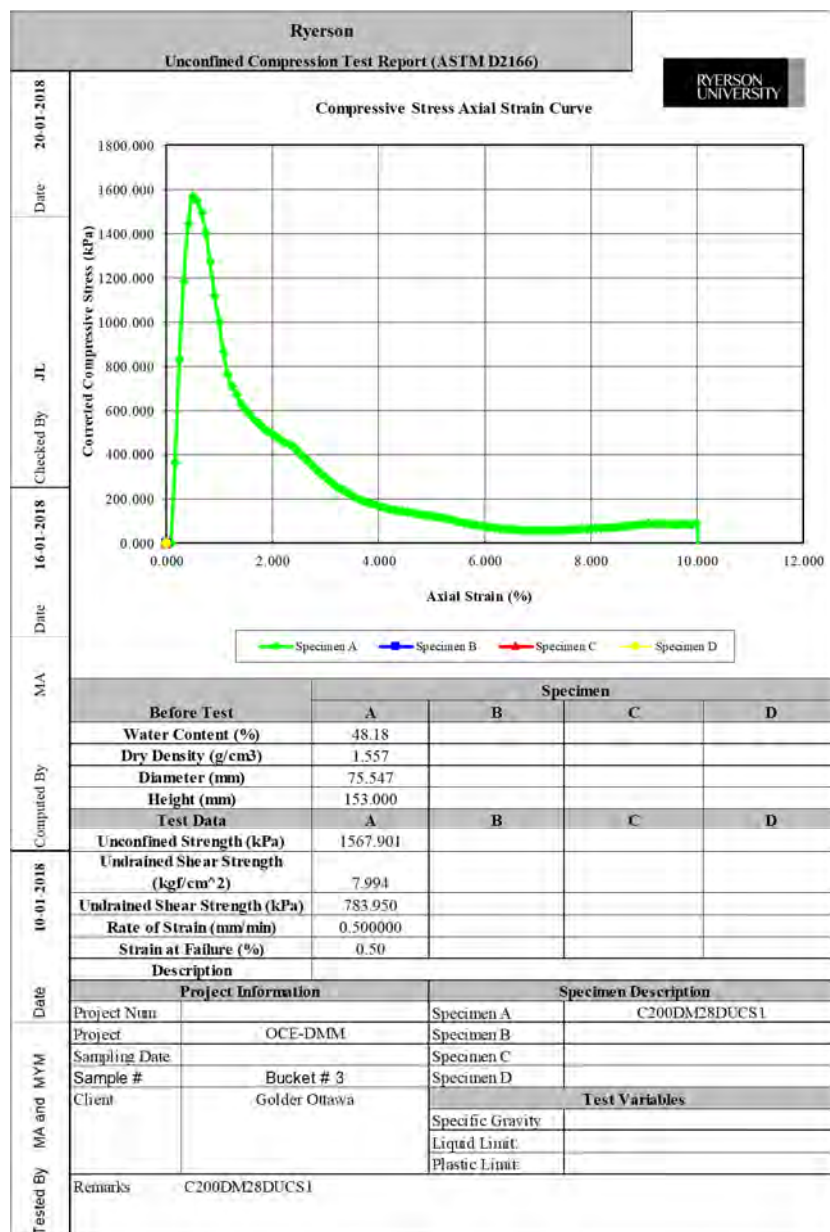


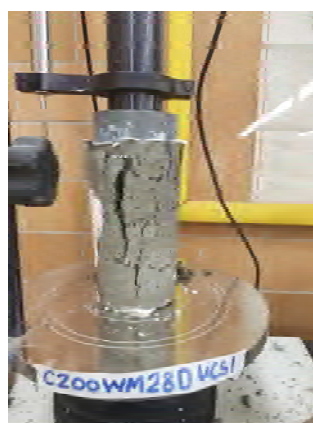
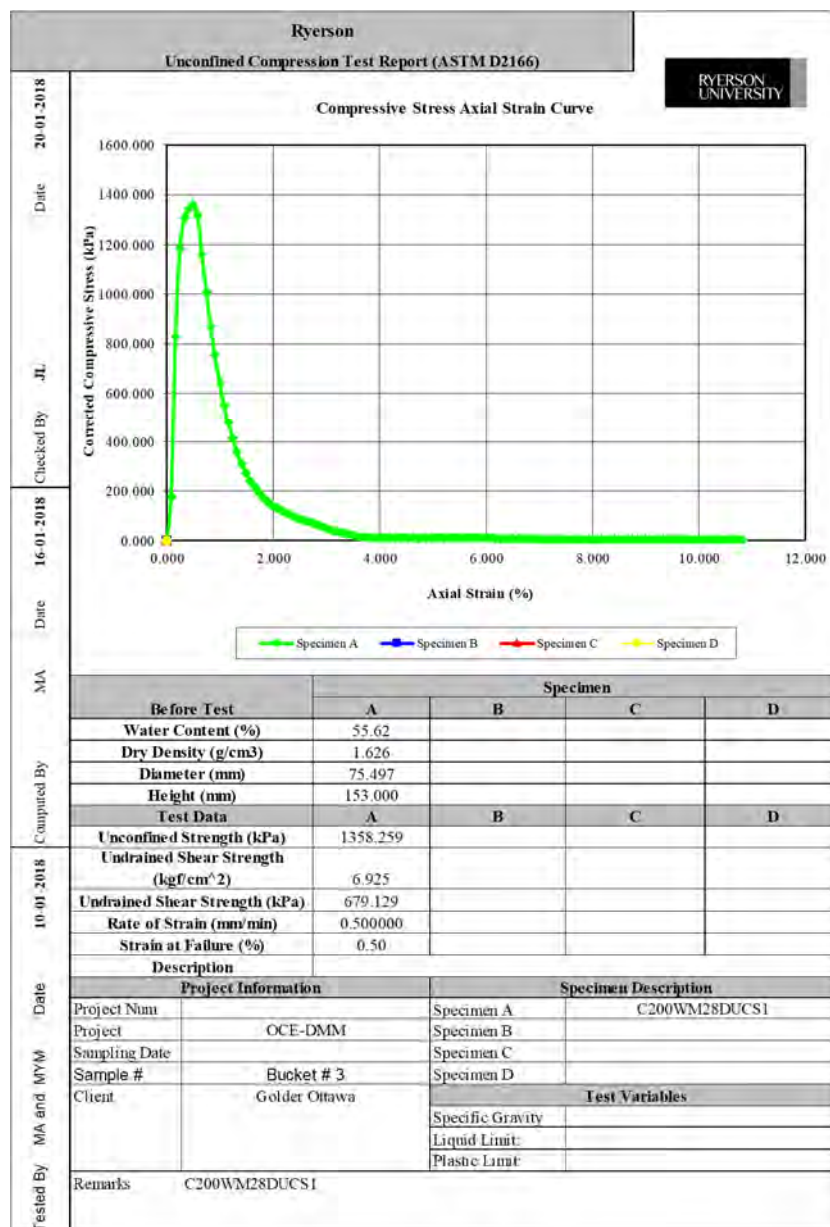


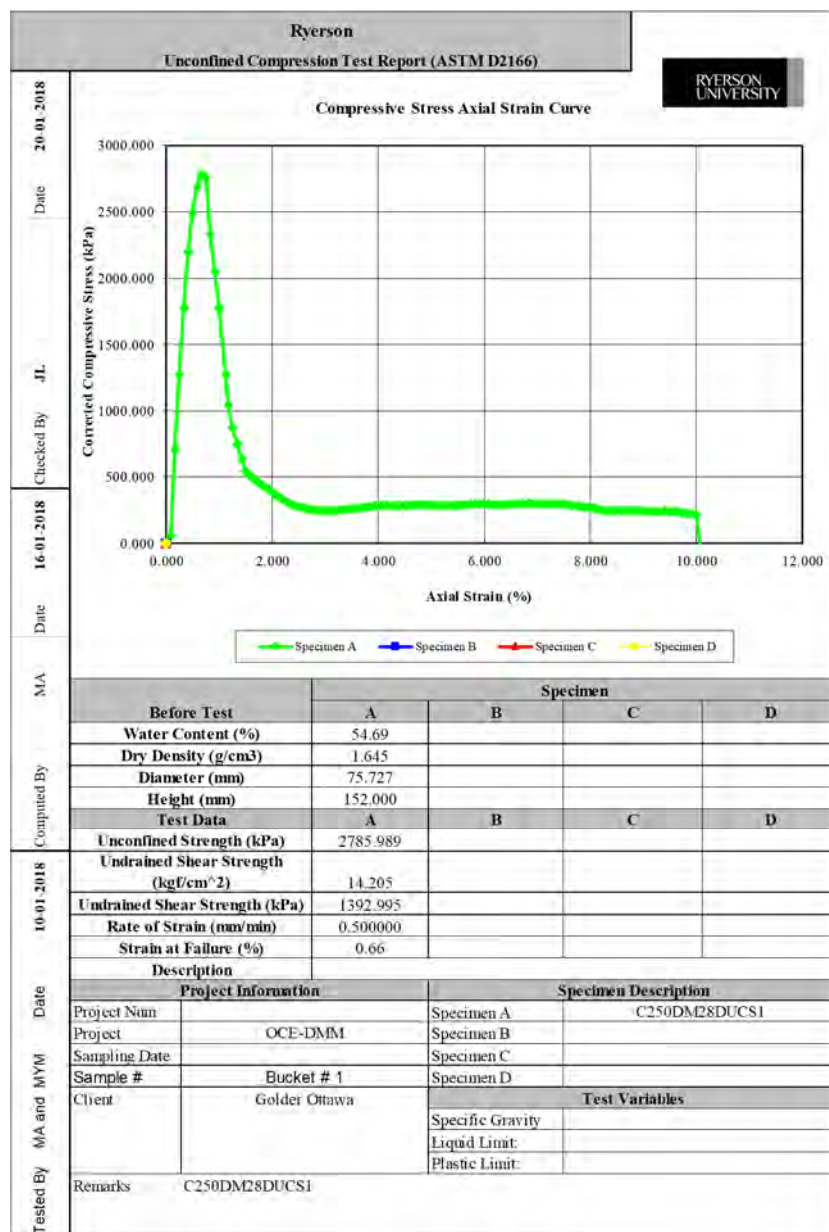


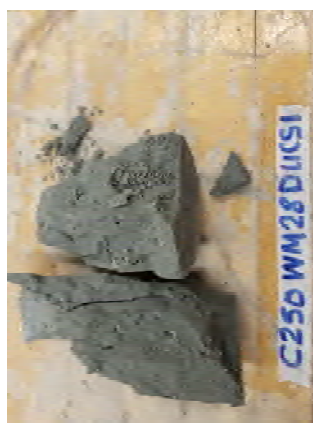
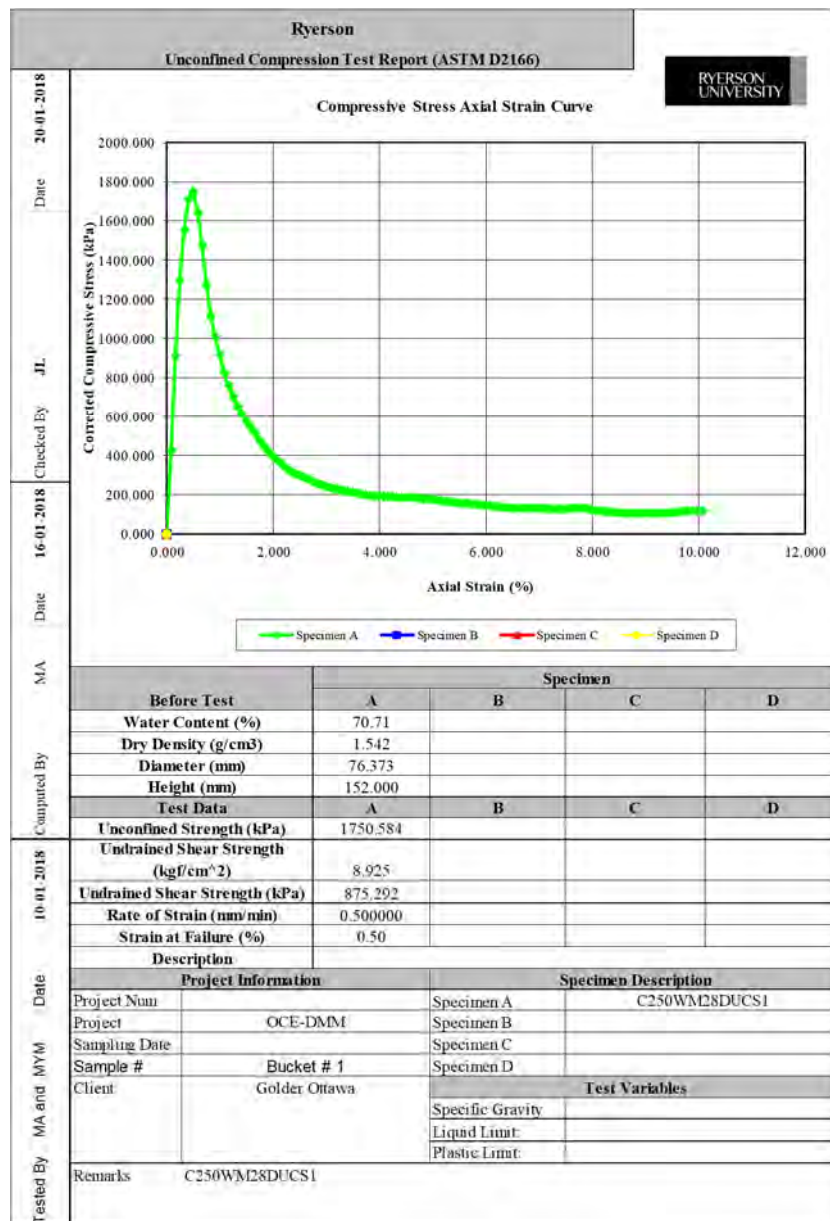


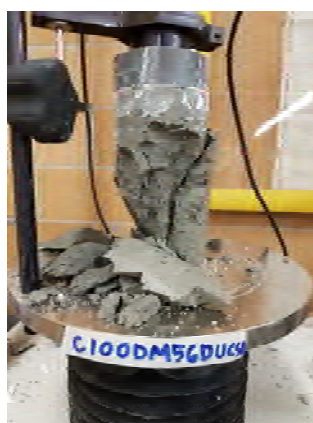
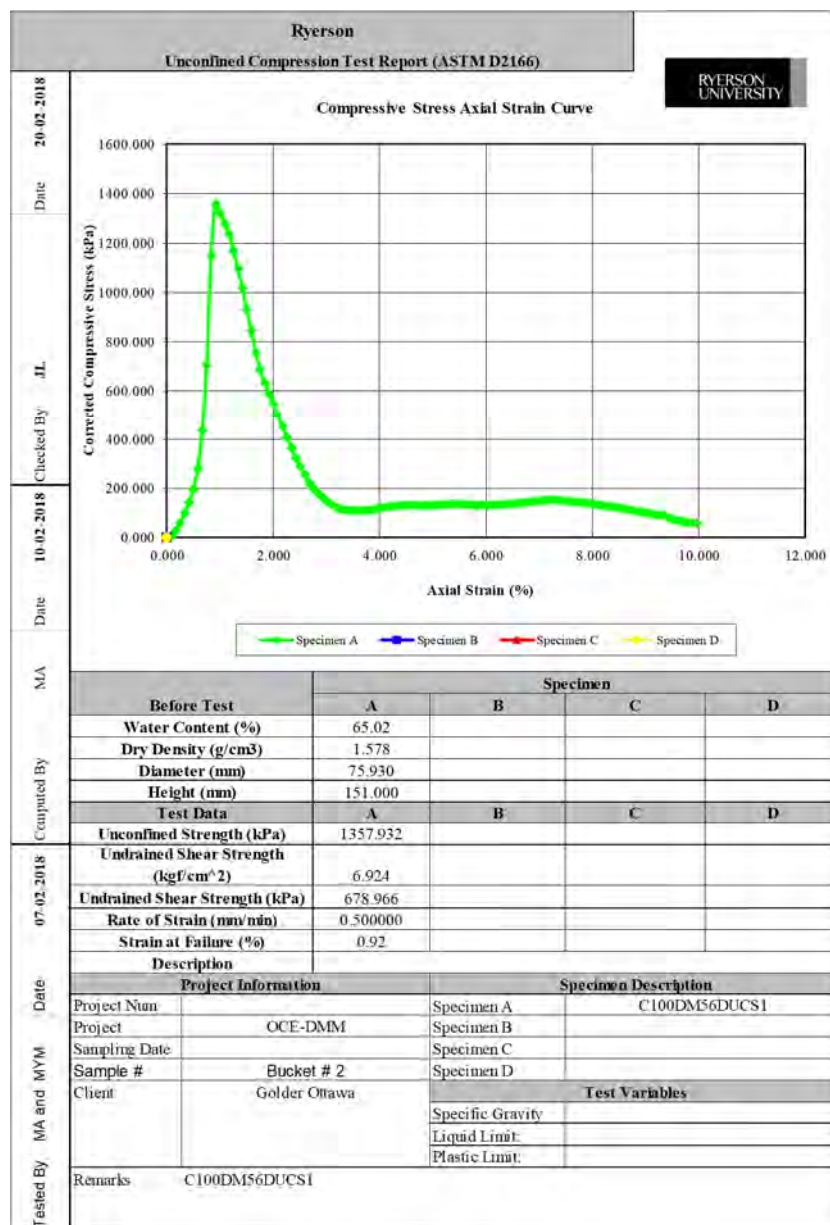


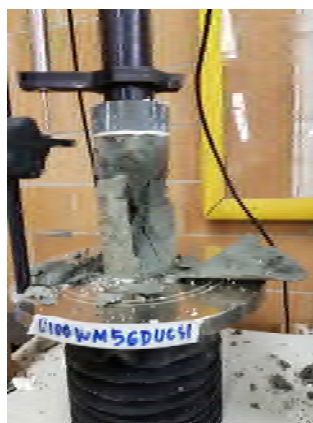
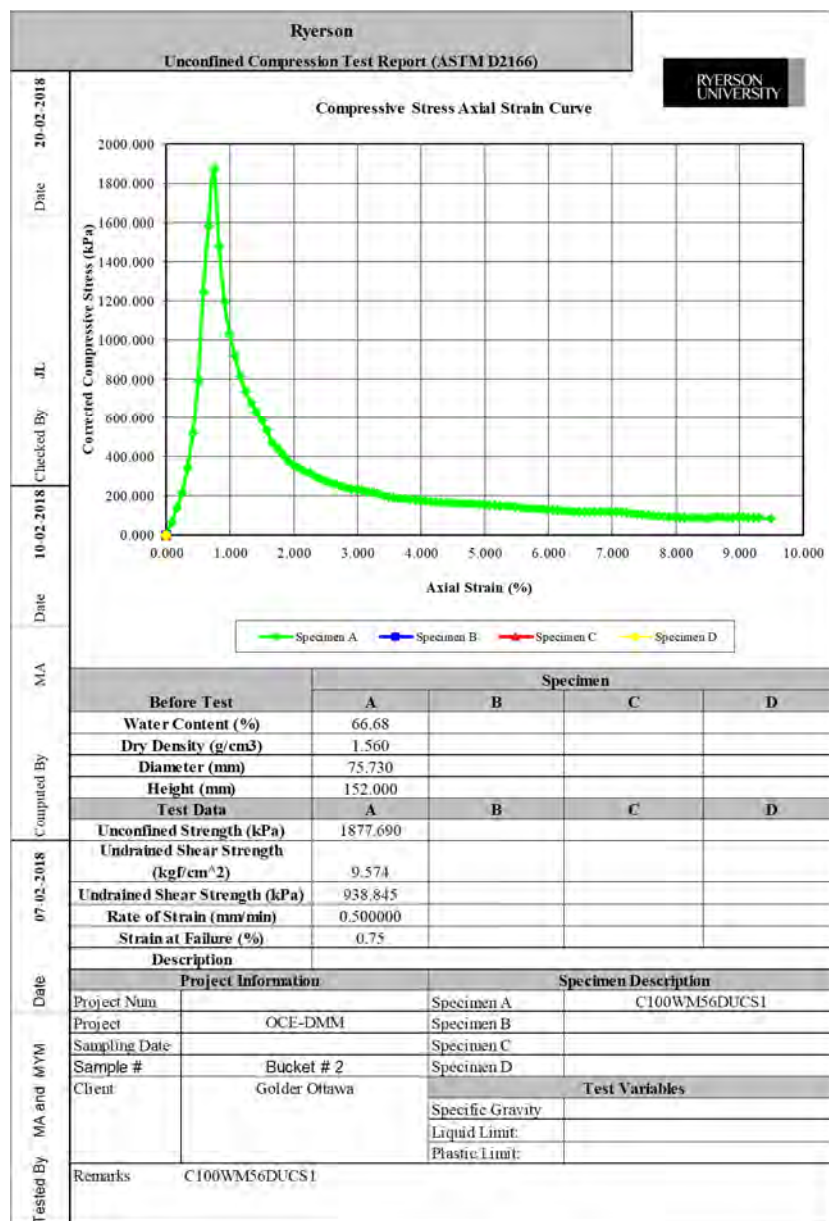


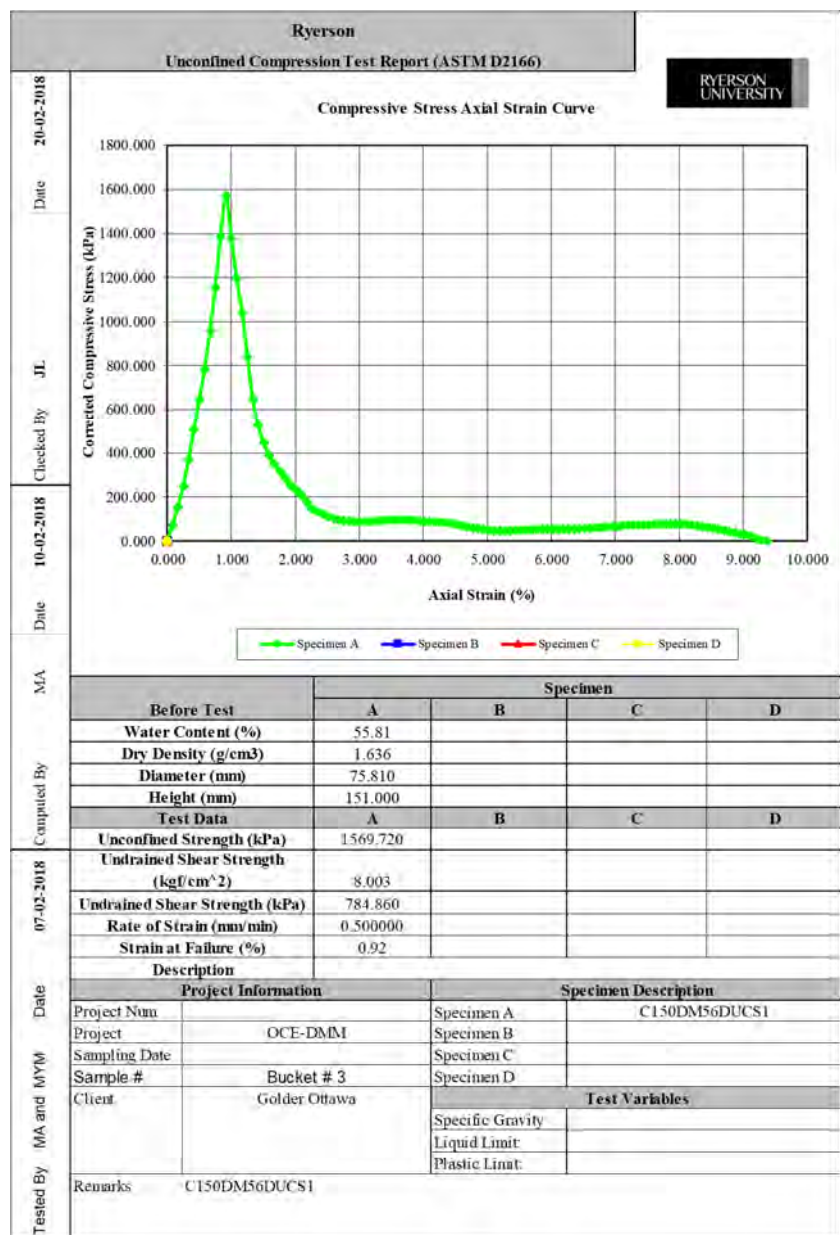


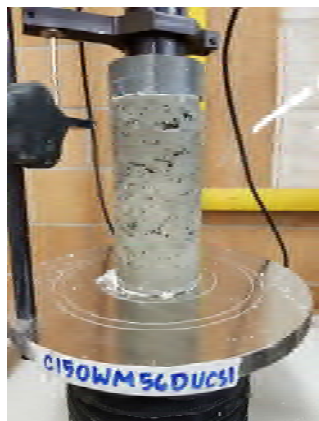
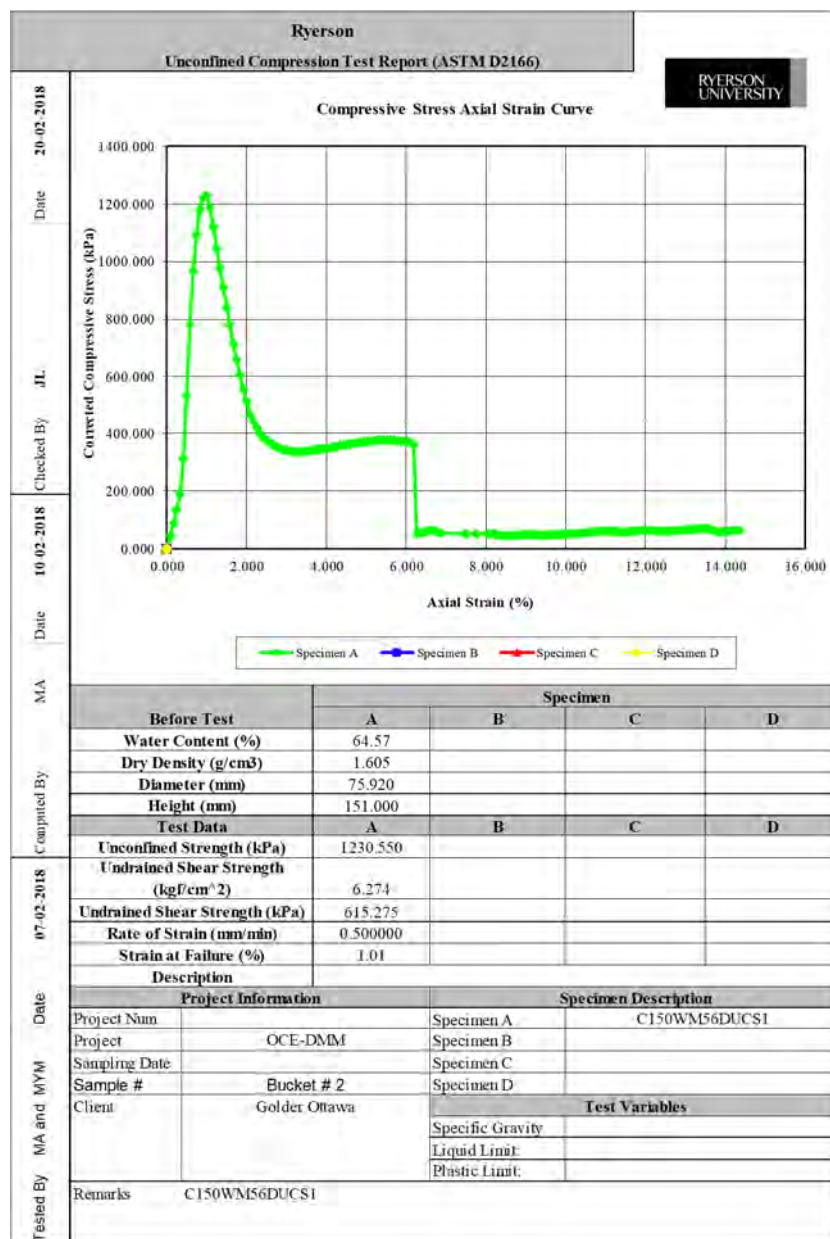


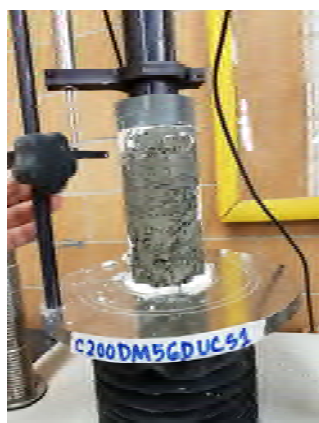
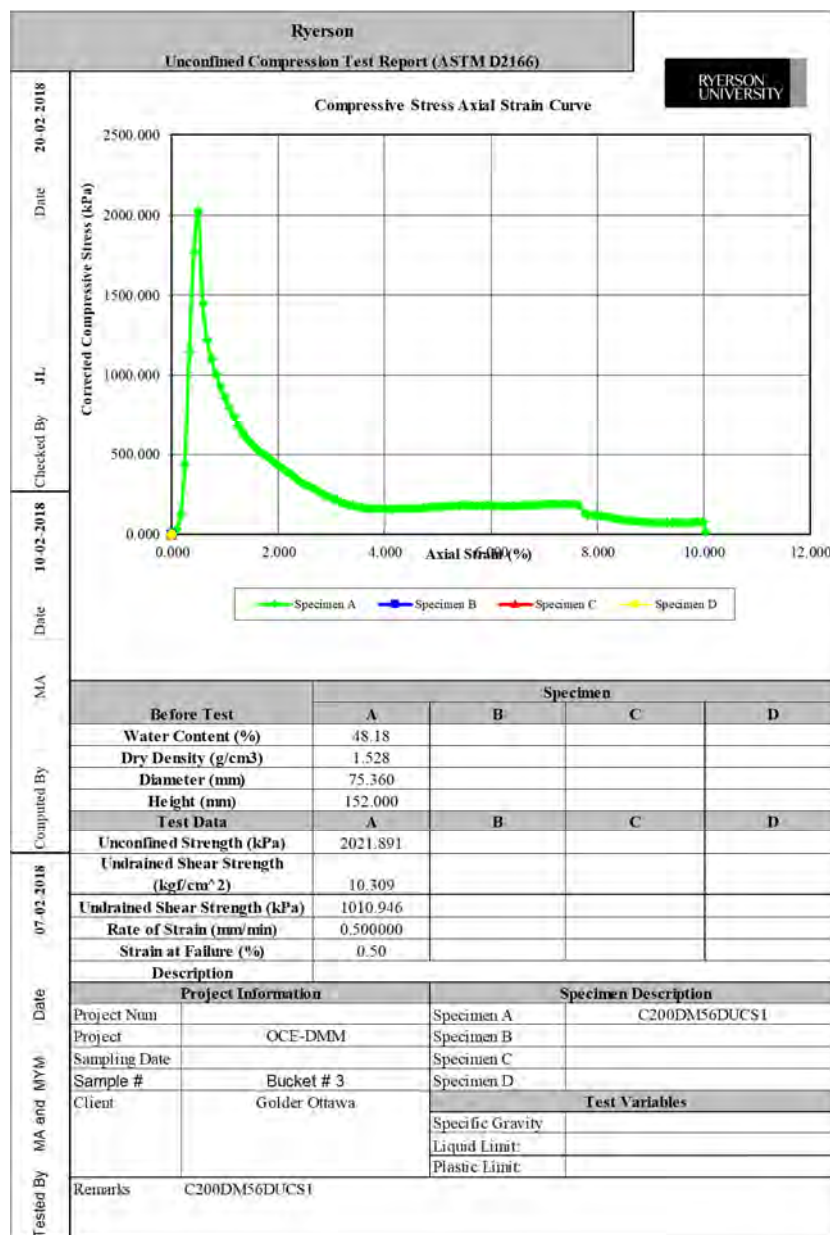


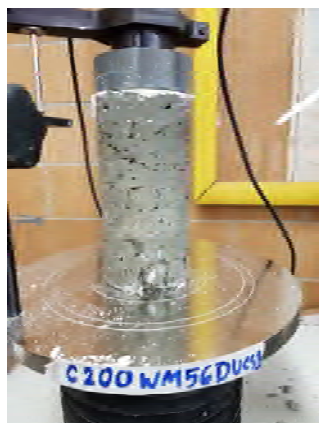
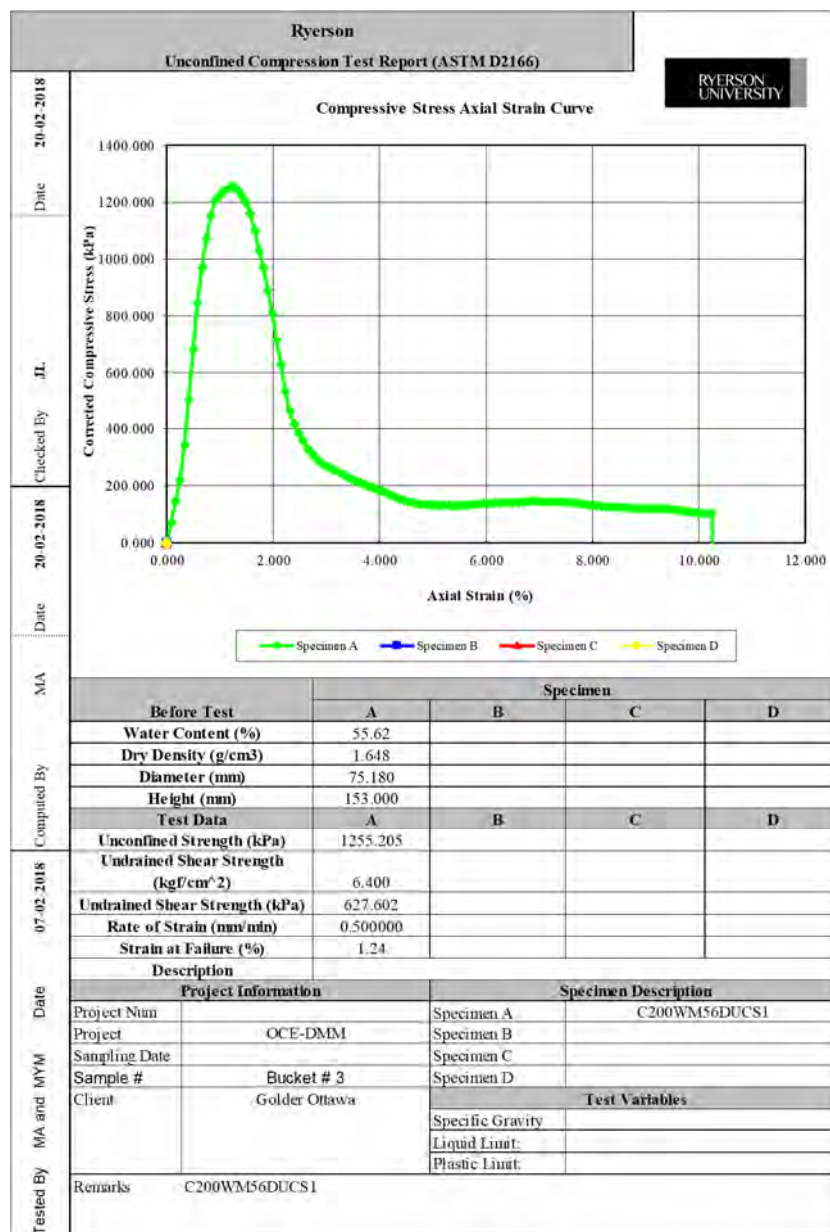


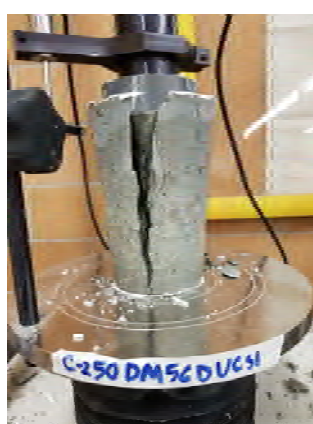
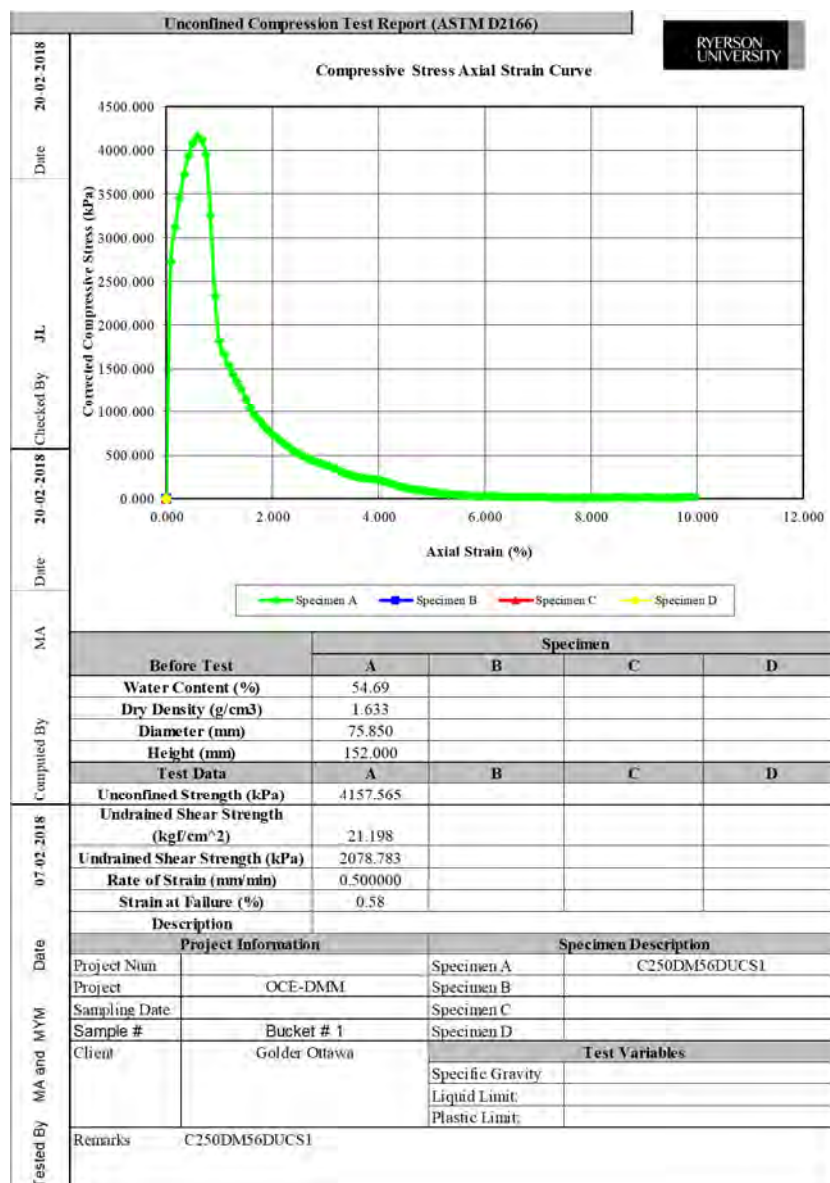


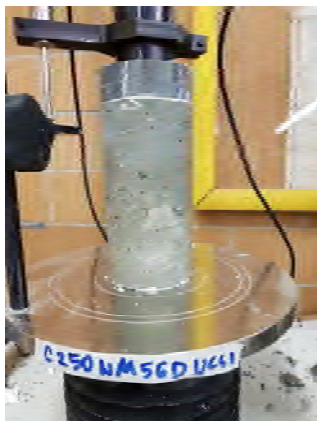
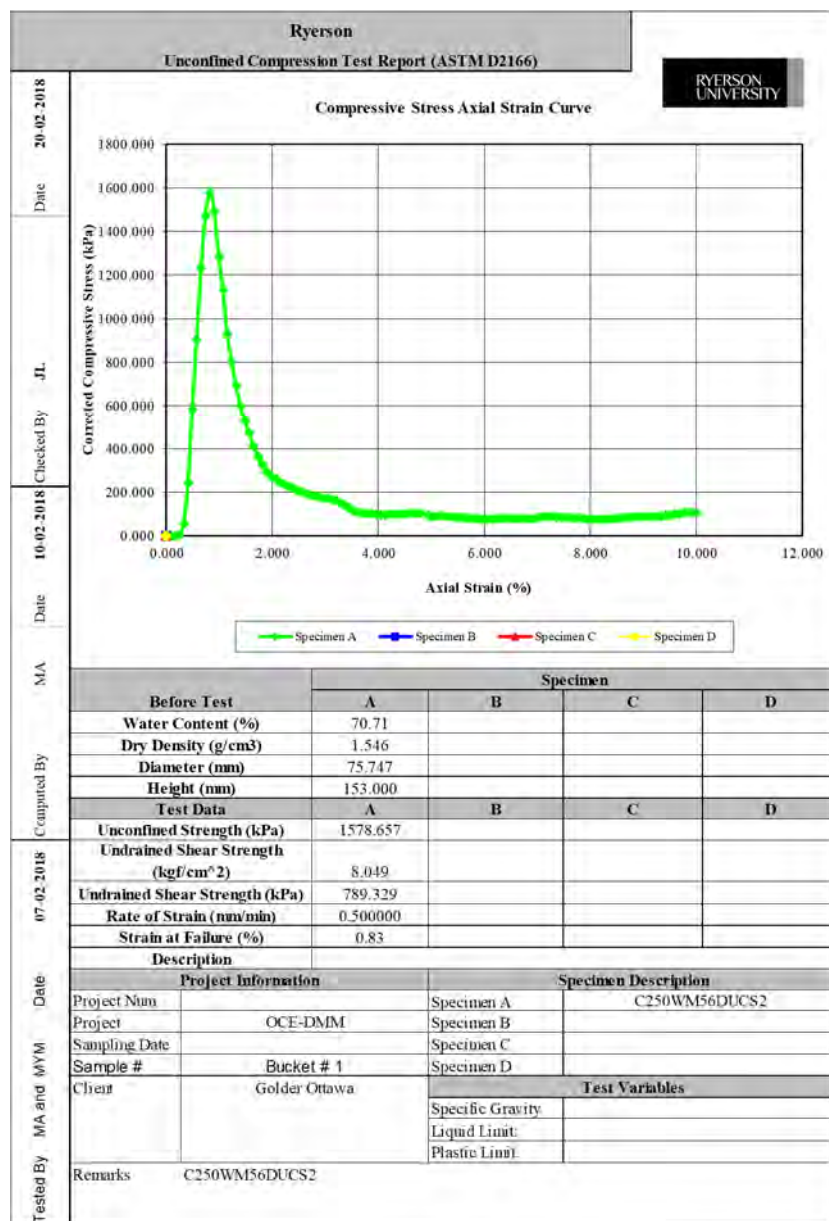




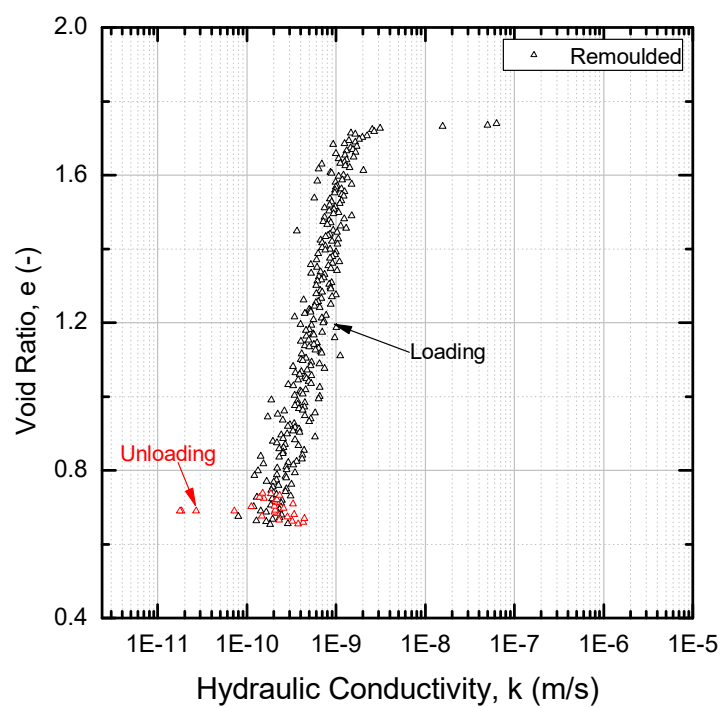
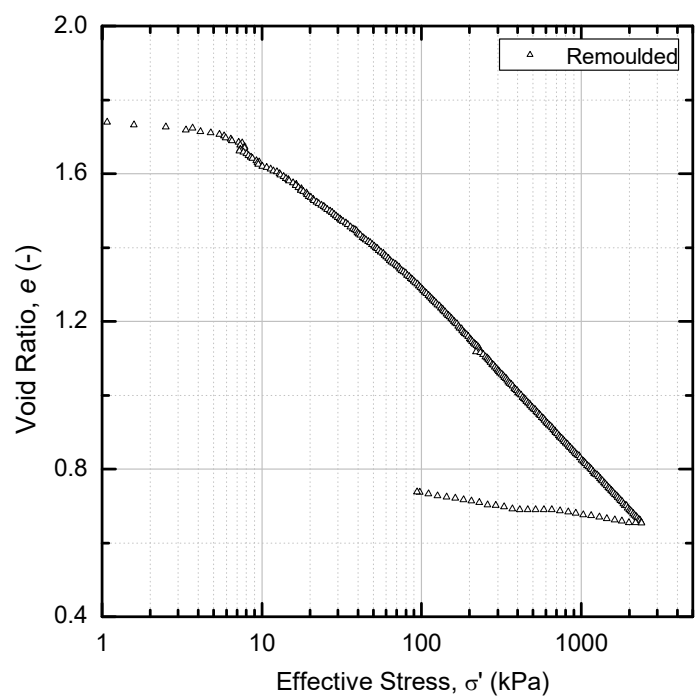








Appendix C – CRS TEST RESULTS



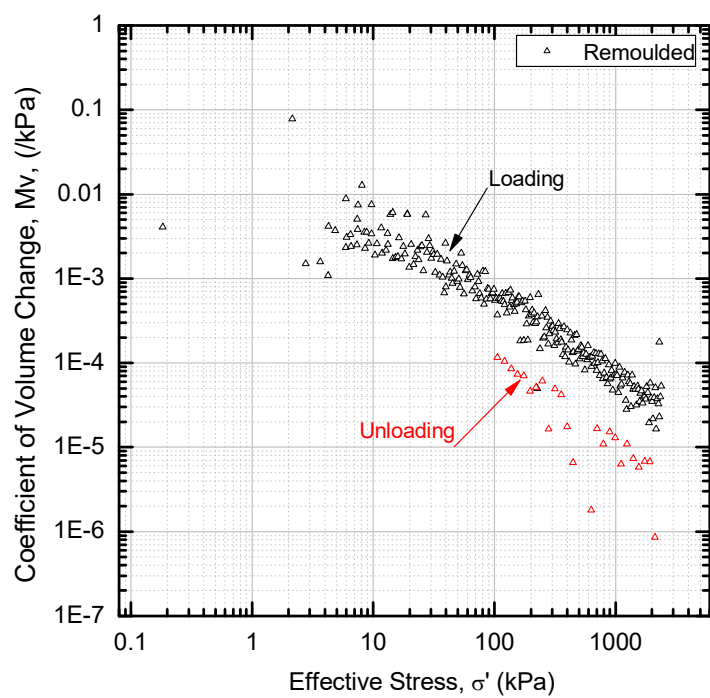
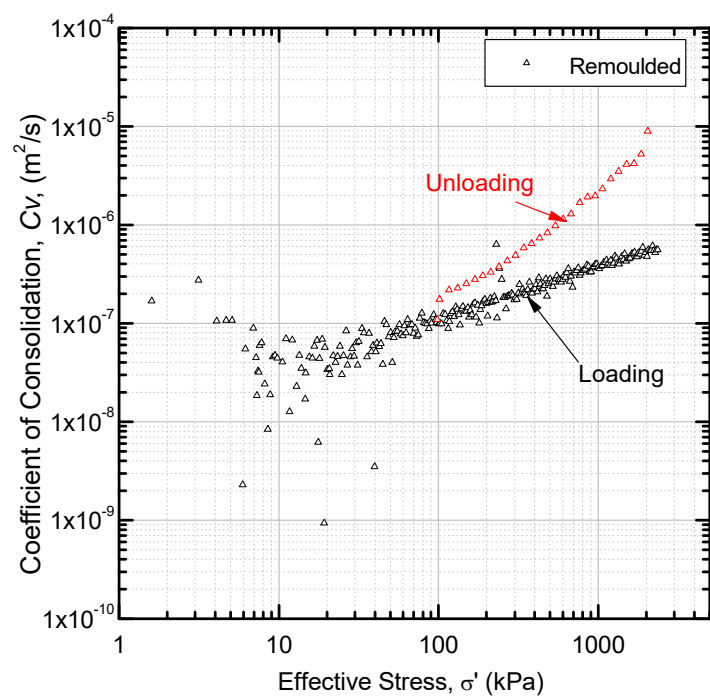
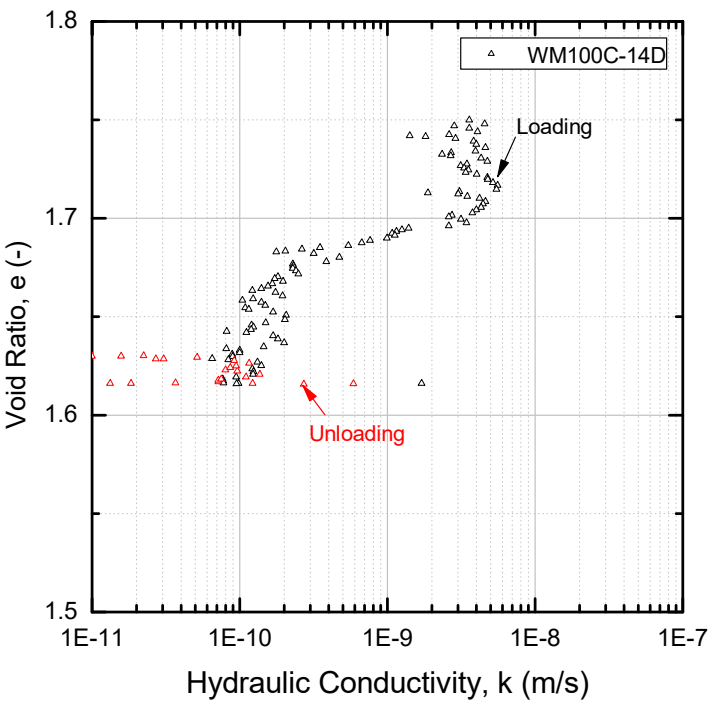
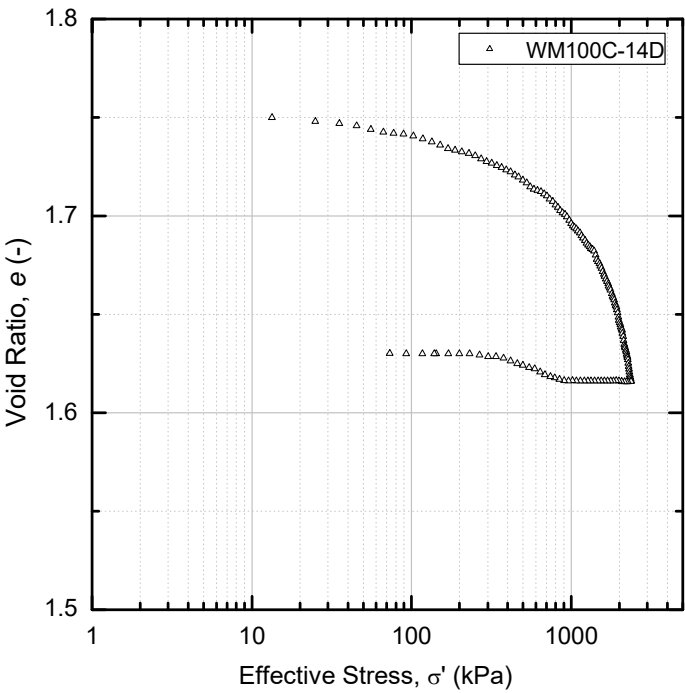


Figure C.1 Results from CRS test on remoulded clay sample



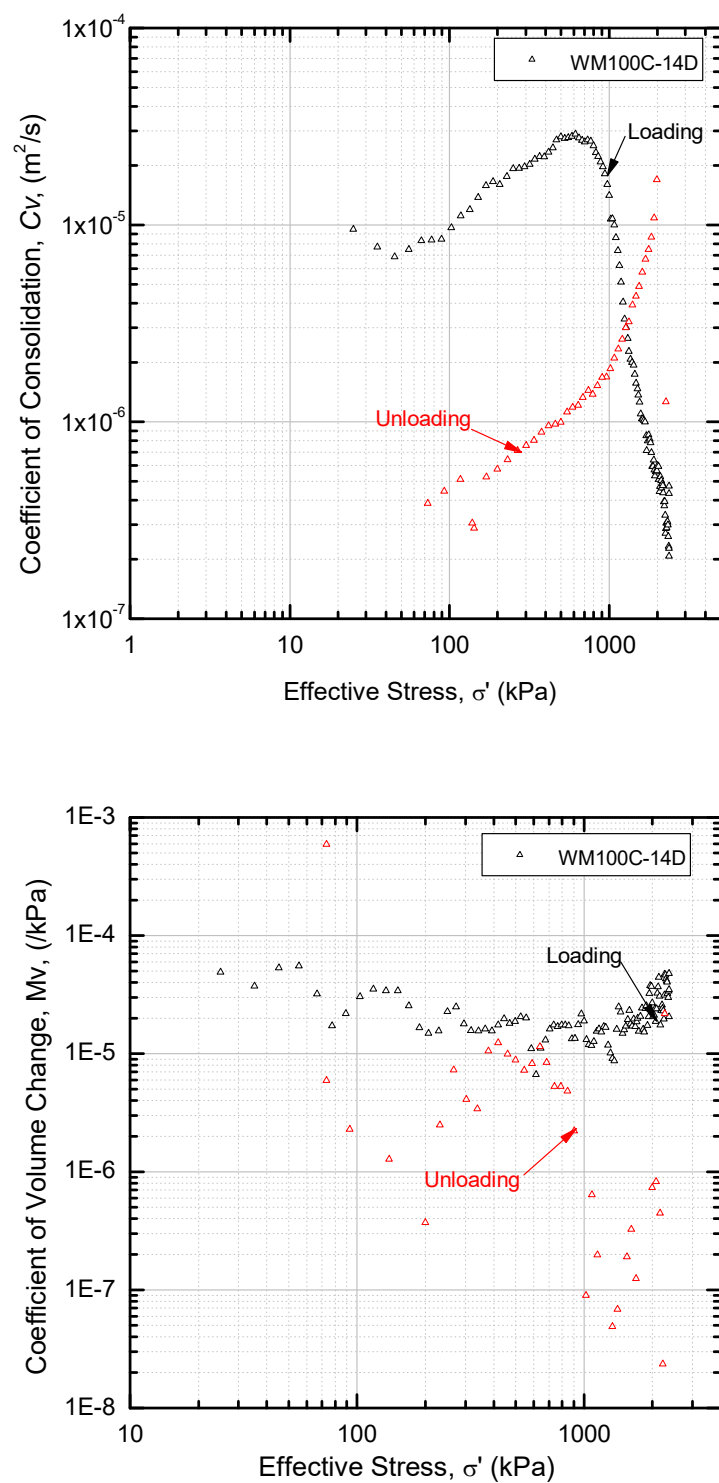
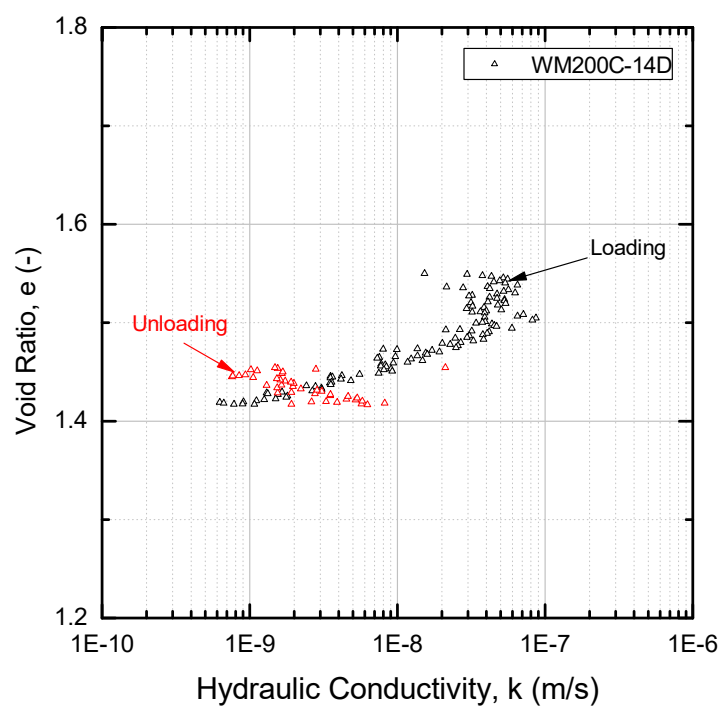
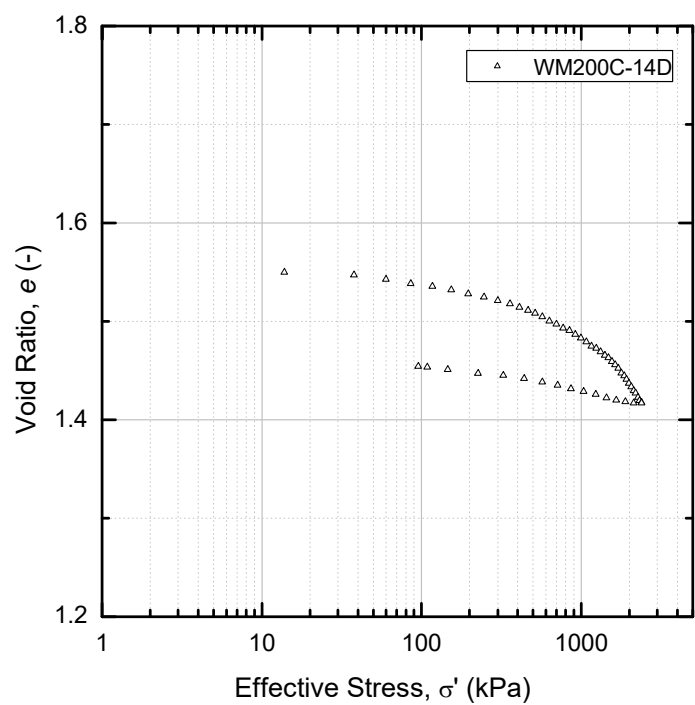


Figure C.2 Results from CRS test on wet mixed 100 kg/m³ cement dosage sample at 14-day curing



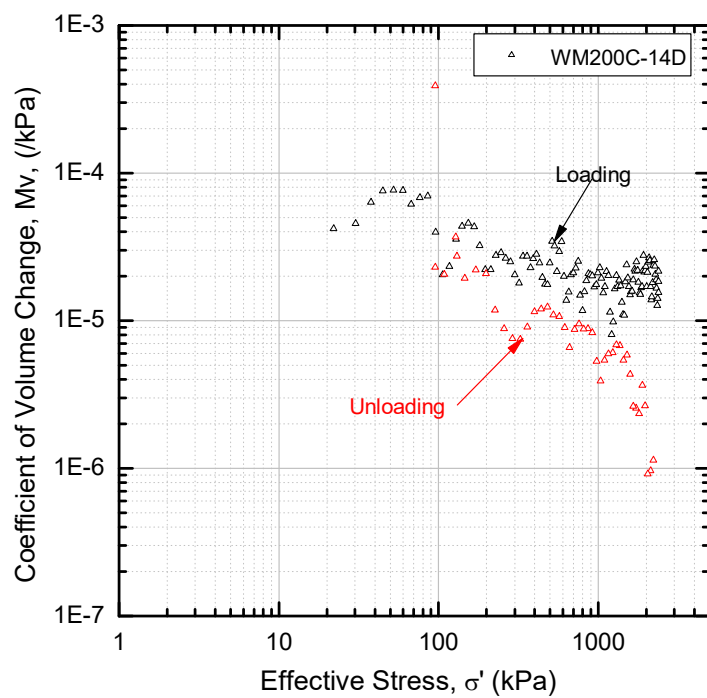
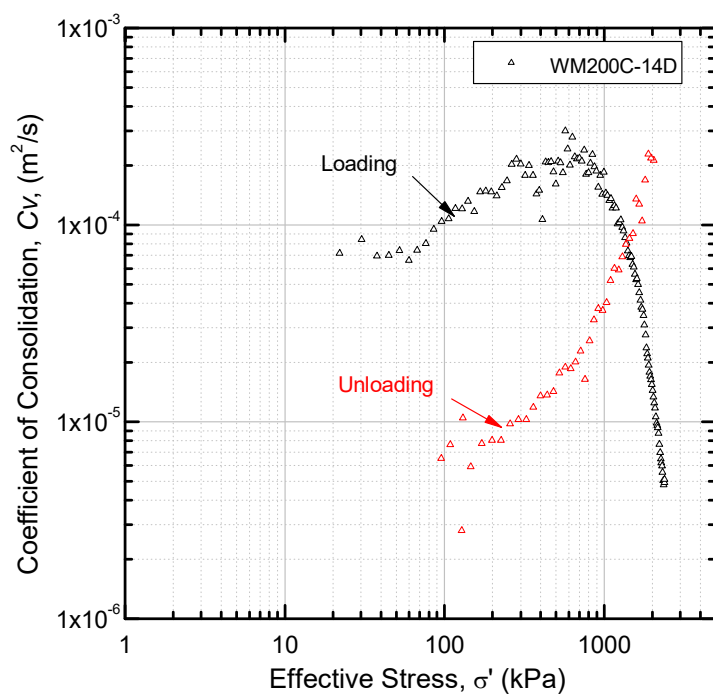
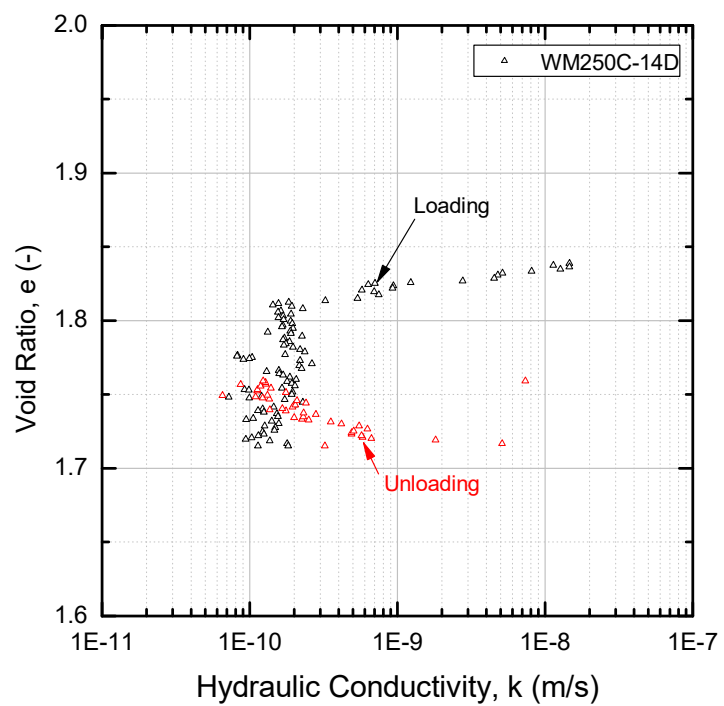
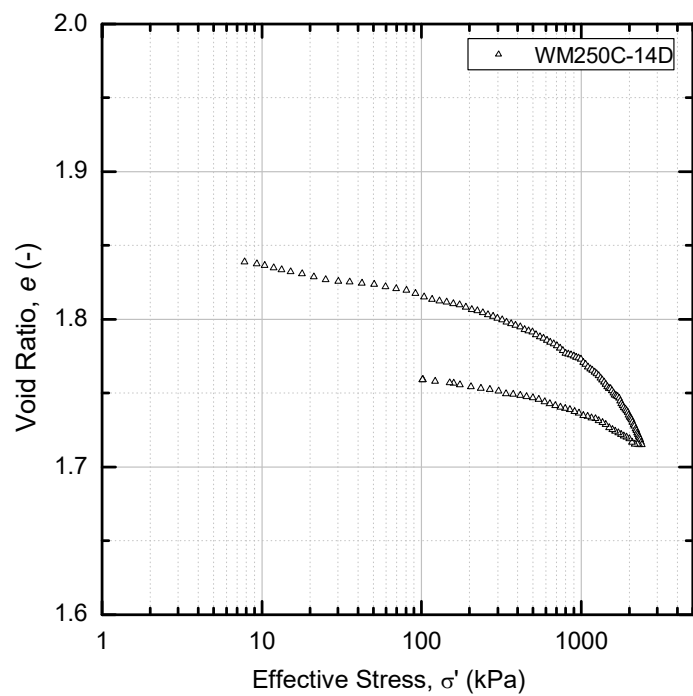


Figure C.3 Results from CRS test on wet mixed 200 kg/m³ cement dosage sample at 14-day curing



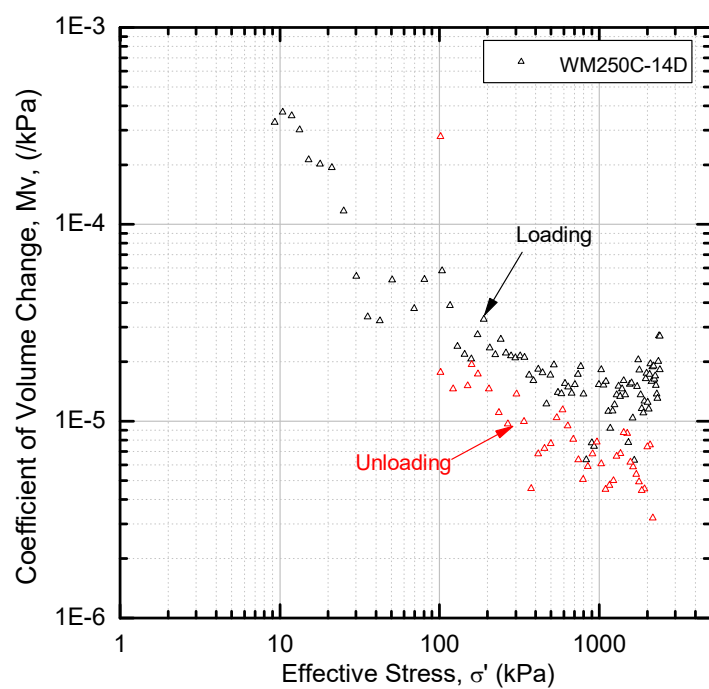
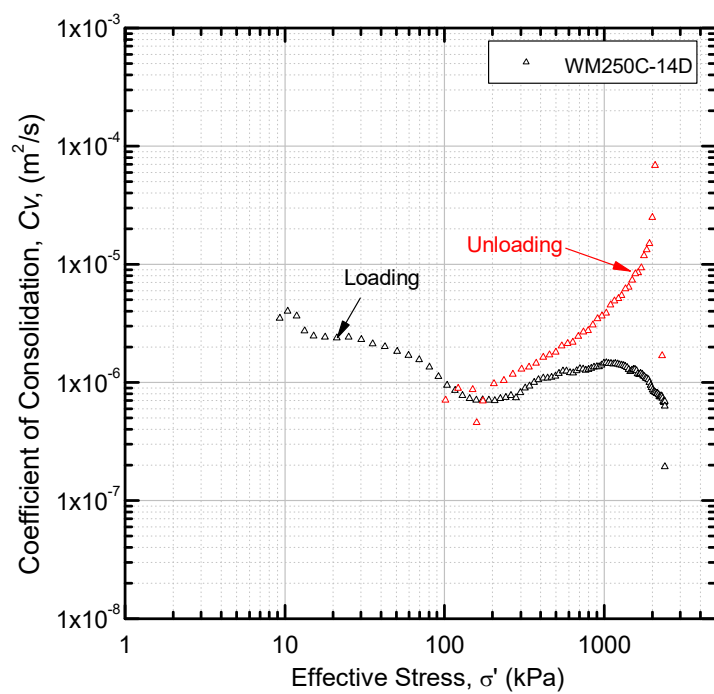
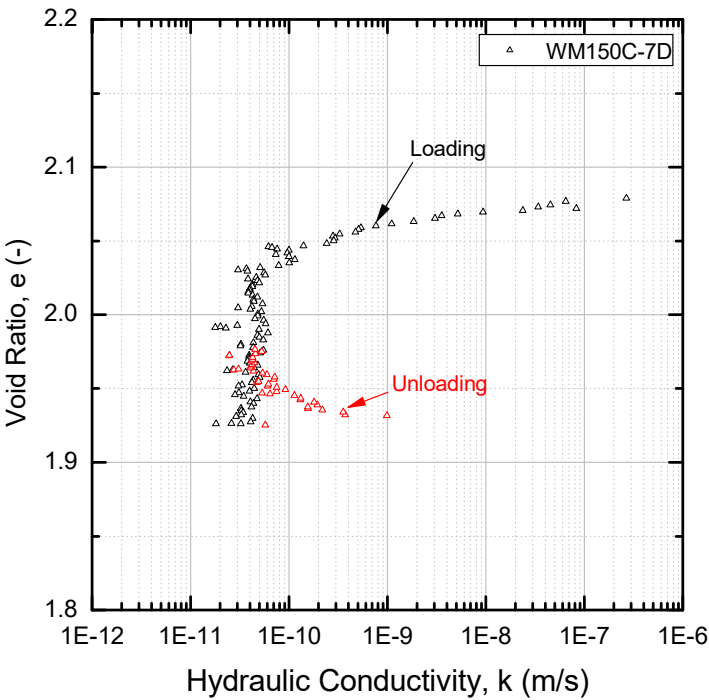
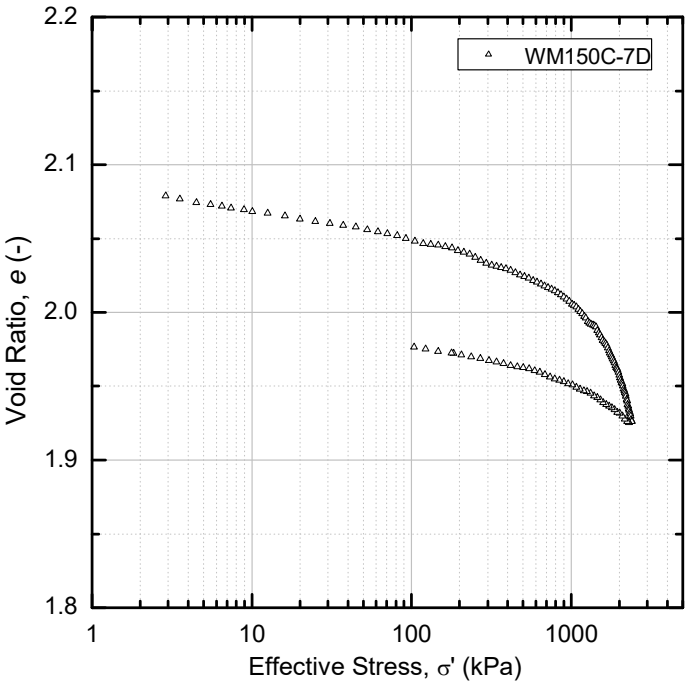


Figure C.4 Results from CRS test on wet mixed 250 kg/m³ cement dosage sample at 14-day curing



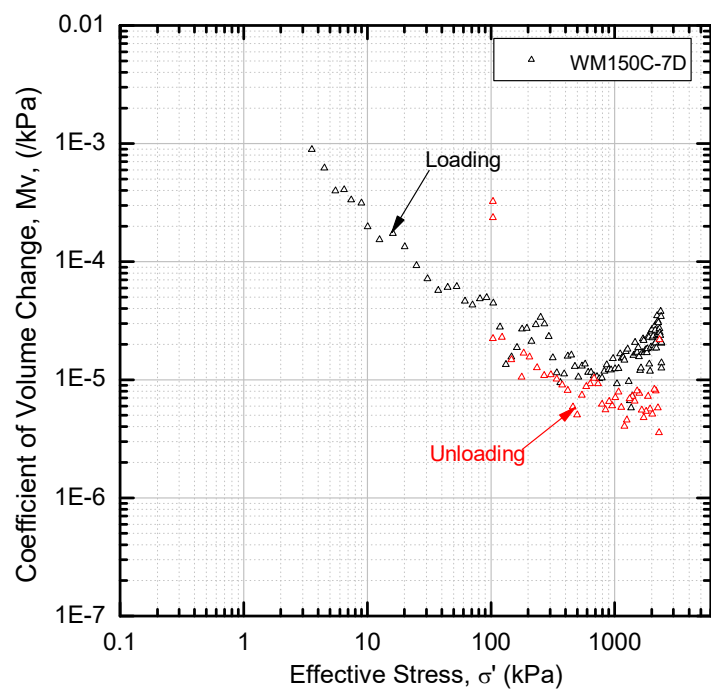
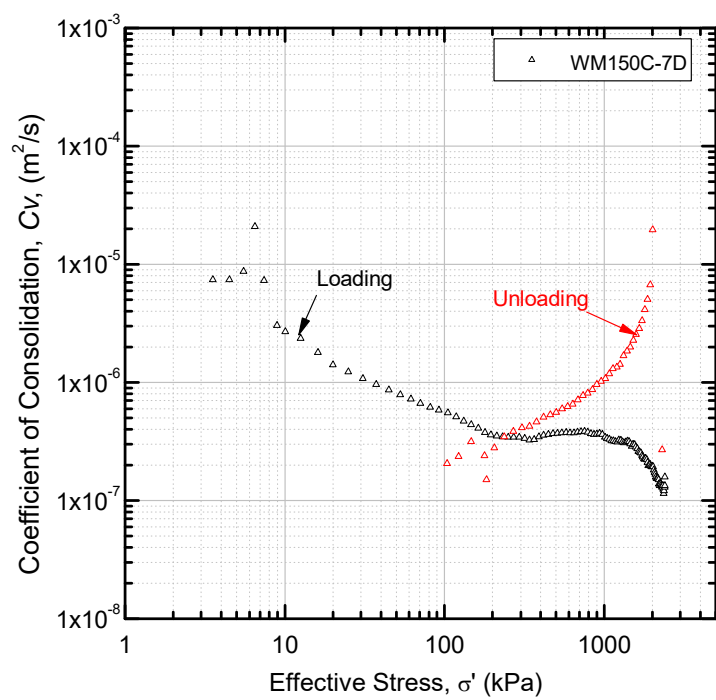
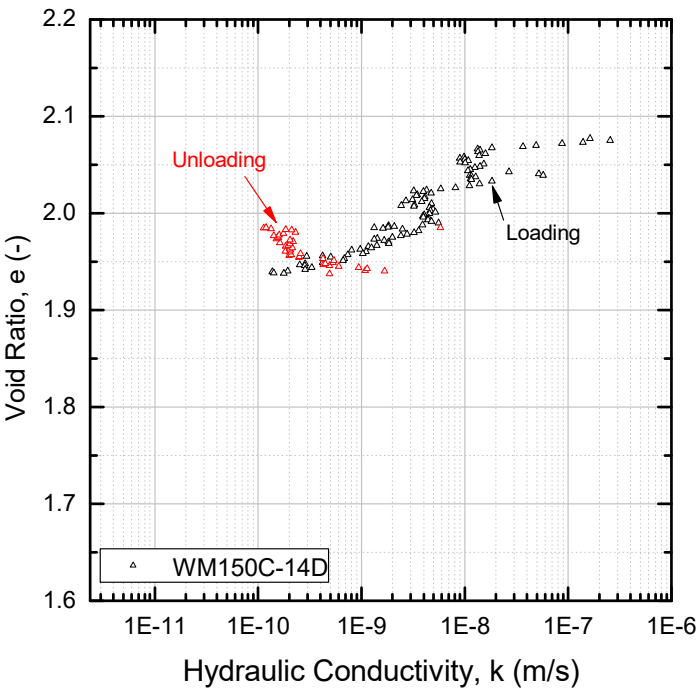
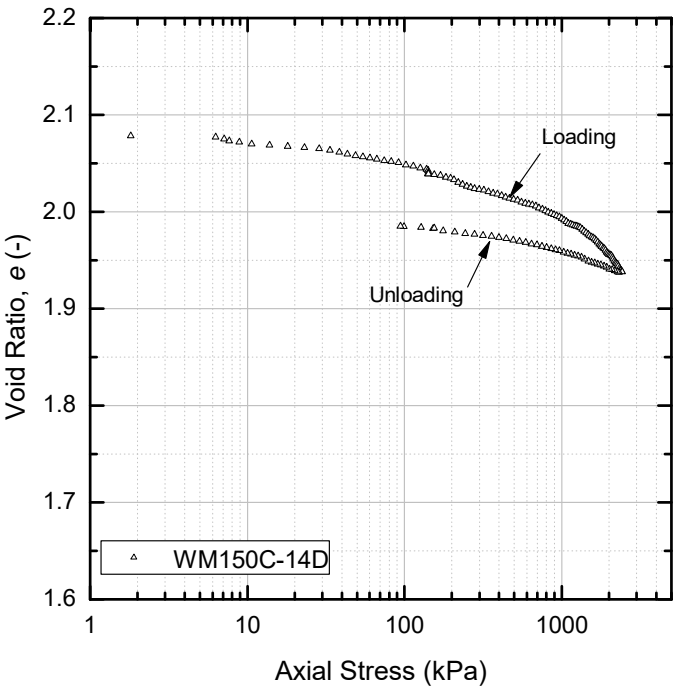


Figure C.5 Results from CRS test on wet mixed 150 kg/m^3 cement dosage sample at 7-day curing



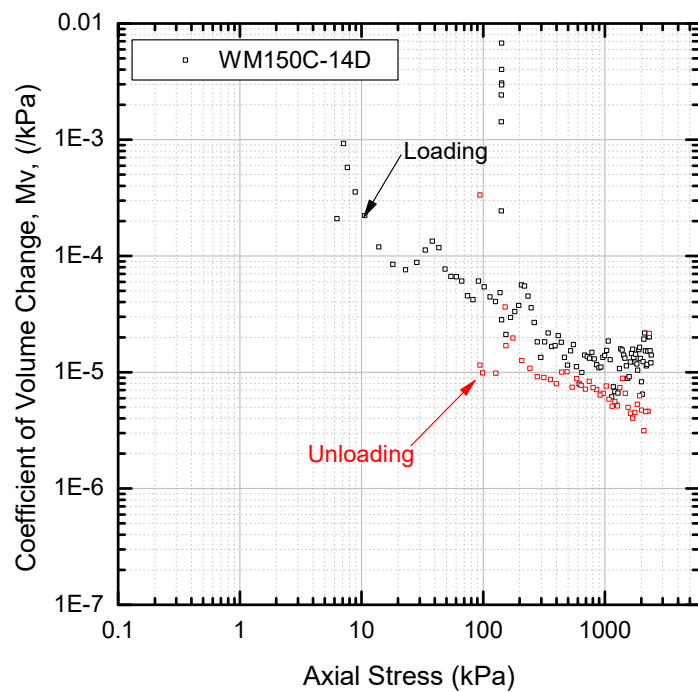
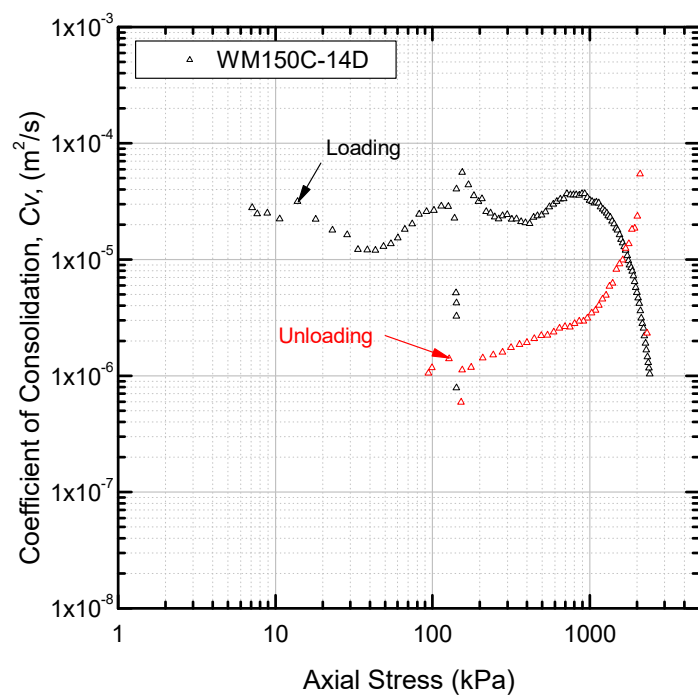





Figure C.6 Results from CRS test on wet mixed 150 kg/m^3 cement dosage sample at 14-day curing

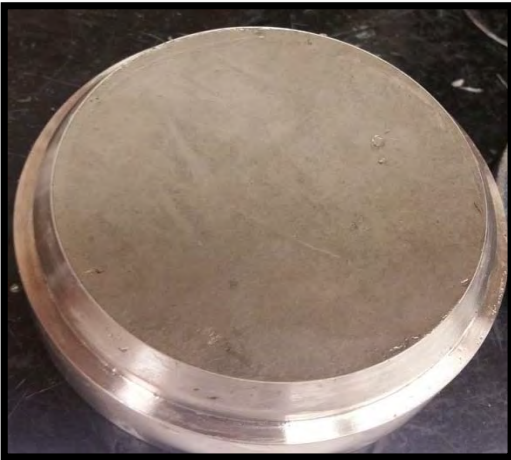

File Name:	Remoulded-1	Date mixed:	12/18/2017	
Sample Des.	Remoulded clay for Golder Ottawa	Start of test:	12/18/2017	
Tested by:	A.A	End of test:	12/22/2017	
Ring Data		Checked by:	JL	
Ring No.	1			
Dia (mm)	63.5			
Height (mm)	25.4			
Mass (g)	217.55			
Mass of sample in the ring (g)		Mass of Sample (g)		Porous stone
Before test	128.74	128.74	Height (mm)	6.33
After test	102.57	102.57	Mass (g)	93.81
Height of soil (mm)		1	2	3
Just Sample (before test)	25	24.96	24.94	24.97
Sample + filter + Porous Stone (before test)	22.7	22.9	22.76	22.79
Sample	16.37	16.57	16.43	16.46
Change in height (mm)	8.51			
Loading Information				
		Rate (%hr)	Pressure (psf)	
1	L	1	50000	
2	U	0.25	2000	
Saturation Information				
Back Pressure (psi)	50	Start of Saturation	12/18/2017 2:43	
Ramp time (min)	60	End of Saturation	12/19/2017 2:26	
Seating Strain (%)	0.5			
Water Content				
Before Test	Trial 1	Trial 2	Trial 3	
Dish No.	G3St9	#3S7	G1S4	
Mass of dish (g)	14.24	14.1	14.07	
Mass of wet soil + dish (g)	40.01	34.88	32.92	
Mass of dry soil + dish (g)	29	26.66	24.83	
Mass of water, Mw (g)	11.01	8.22	8.09	
Mass of dry soil, Ms (g)	14.76	12.56	10.76	
Water content, w (%)	74.59	65.45	75.19	
Average water content	71.74			
After Test	Trial 1	Trial 2	Trial 3	
Dish No.	20.18 Cer			
Mass of dish (g)	127.86			
Mass of wet soil + dish (g)	229.73			
Mass of dry soil + dish (g)	203.95			
Mass of water, Mw (g)	25.78			
Mass of dry soil, Ms (g)	76.09			
Water content, w (%)	33.88			
Average water content	33.88			
Comment:				

Parameters in the CRS ring	
Total sample	
Surface area (sq.mm)	3166.92
Water Content (%)	71.74
Additives (kg/m3)	
Cement	0
Mass of each portion in the <i>ring</i> (g)	
All sample (wet)	128.74
All Sample (Dry)	74.96
Total water	53.78
Cement	0.00
Water Added	0.00
Soil	74.96
Density of each portion (g/mm3)	
Water	0.001
Cement	0.00315
Soil	0.0026
Volume of each portion in the ring (mm3)	
Water	53778.61
Cement	0.00
Soil	28831.30
Height (mm)	
Sample	24.97
Water	16.98
Cement	0.00
Delta h after test	
Soil	7.99
Total Solid height	7.99
Void height	16.98
Height check	TRUE
Void Ratio	2.13
Saturation (%)	100.00
Before test	After test
	



File Name:	2a	Date mixed:	12/15/2017
Sample Des.	WMC100-14D	Start of test:	12/27/2017
Client:	Golder Ottawa	End of test:	
Ring Information		Tested by:	A.A
Ring No.	1	Checked by:	JL
Dia (mm)	63.5	Porous stone	
Height (mm)	25.4	Height (mm)	6.33
Mass (g)	217.37	Mass (g)	93.81
Mass of sample + ring (g)		Mass of Sample (g)	
Before test	342.77	125.4	
After test	344.03	126.66	
Height of soil (mm)	1	2	3
Sample (before test)	25.3	25.31	25.25
Sample + filter + Porous Stone (before test)			
Sample (After test)	24.66	24.72	24.59
Change in height (mm)	0.63		
Loading Information			
		Rate (%hr)	Pressure (psf)
1	L	1	50000
2	U	0.25	2000
Saturation Information			
Back Pressure (psi)	50	Start of saturation	End of saturation
Ramp time (min)	60	12/27/2017 12:31	12/28/2017 23:40
Seating Strain (%)	0.5		
Water Content			
Before Test	Trial 1	Trial 2	Trial 3
Dish No.	G4S6	#S208	
Mass of dish (g)	14.11	14.18	
Mass of wet soil + dish (g)	25.98	30.71	
Mass of dry soil + dish (g)	21.4	24.33	
Mass of water, Mw (g)	4.58	6.38	
Mass of dry soil, Ms (g)	7.29	10.15	
Water content, W (%)	62.83	62.86	
Average water content	62.84		
After Test	Trial 1	Trial 2	Trial 3
Dish No.	2a		
Mass of dish (g)	217.37		
Mass of wet soil + dish (g)	344.03		
Mass of dry soil + dish (g)	294.28		
Mass of water, Mw (g)	49.75		
Mass of dry soil, Ms (g)	76.91		
Water content, W (%)	64.69		
Average water content	64.69		
Comment:			

Mix design			
Dia (m)	Height (m)	Assumed vol of soil (m3)	Cement dosage (kg/m3)
0.0635	0.0253	8.0E-05	100
Soil in the mix (g)	Cement (g) in the mix	Water(g) in the mix	Total mix weight (g)
134.78	8.01	5.61	148.39
Soil in the ring (wet) (g)	Cement (g) in the ring	Water (g) in the ring	Total sample weight(g)
113.90	6.77	4.74	125.40
Each portion in the ring			
Surface area (sq.mm)	3166.92	Mass (g)	
Water Content (%)	62.84	Sample (wet)	125.40
Additives (kg/m3)		Sample (Dry)	77.01
Cement	100	Total water	48.39
		Cement	6.77
		Water Added	4.74
		Soil (dry)	70.24
Density (g/mm3)		Volume (mm3)	
Water	0.001	Water	48392.59
Cement	0.00315	Cement	2148.38
Soil	0.0026	Soil	27015.4
		Total volume	77556.36
Height (mm)		Results	
Sample	25.29	Void Ratio	1.75
Water	15.28	Saturation (%)	95.042
Cement	0.68	Height check	TRUE
Soil	8.53		
Total Solid height	9.21		
Void height	16.08		
Before test		After test	
			

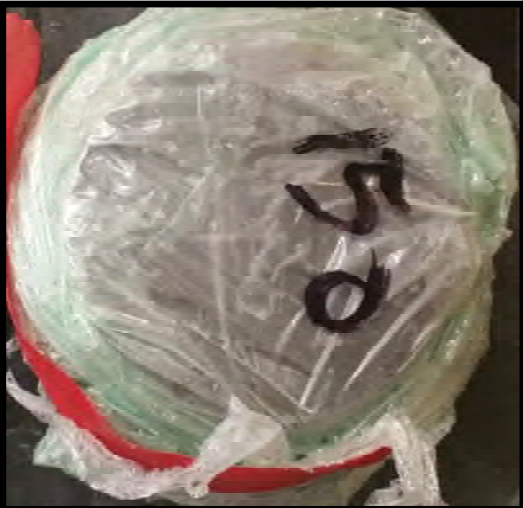

File Name:	1a	Date mixed:	12/13/2017
Sample Des.	WMC100-14D	Start of test:	12/25/2017
Client:	Golder Ottawa	End of test:	12/27/2017
Ring Information		Tested by:	A.A
Ring No.	1	Checked by:	JL
Dia (mm)	63.5	Porous stone	
Height (mm)	25.4	Height (mm)	6.33
Mass (g)	217.55	Mass (g)	93.81
Mass of sample + ring (g)		Mass of Sample (g)	
Before test	350.65	133.1	
After test	351.62	134.07	
Height of soil (mm)	1	2	3
Sample (before test)	25.68	25.8	25.81
Sample + filter + Porous Stone (before test)			
Sample (After test)	25.22	25.18	25.22
Change in height (mm)	0.56		
Loading Information			
		Rate (%hr)	Pressure (psf)
1	L	1	50000
2	U	0.25	2000
Saturation Information			
Back Pressure (psi)	50	Start of saturation	End of saturation
Ramp time (min)	60	12/25/2017 12:15	12/26/2017 12:00
Seating Strain (%)	0.5		
Water Content			
Before Test	Trial 1	Trial 2	Trial 3
Dish No.	G4s3	G1S9	G3S87
Mass of dish (g)	14.75	13.66	14.27
Mass of wet soil + dish (g)	56.8	56.28	56.02
Mass of dry soil + dish (g)	42.09	40.94	40.89
Mass of water, Mw (g)	14.71	15.34	15.13
Mass of dry soil, Ms (g)	27.34	27.28	26.62
Water content, W (%)	53.80	56.23	56.84
Average water content	55.62		
After Test	Trial 1	Trial 2	Trial 3
Dish No.	1a		
Mass of dish (g)	217.75		
Mass of wet soil + dish (g)	351.55		
Mass of dry soil + dish (g)	303.06		
Mass of water, Mw (g)	48.49		
Mass of dry soil, Ms (g)	85.31		
Water content, W (%)	56.84		
Average water content	56.84		
Comment:			

Mix design			
Dia (m)	Height (m)	Assumed vol of soil (m3)	Cement dosage (kg/m3)
0.0635	0.0258	8.2E-05	200
Soil in the mix (g)	Cement (g) in the mix	Water(g) in the mix	Total mix weight (g)
137.32	16.32	11.42	165.06
Soil in the ring (wet) (g)	Cement (g) in the ring	Water (g) in the ring	Total sample weight(g)
110.73	13.16	9.21	133.10
Each portion in the ring			
Surface area (sq.mm)	3166.92	Mass (g)	
Water Content (%)	55.62	Sample (wet)	133.10
Additives (kg/m3)		Sample (Dry)	85.53
Cement	200	Total water	47.57
		Cement	13.16
		Water Added	9.21
		Soil (dry)	72.37
Density (g/mm3)		Volume (mm3)	
Water	0.001	Water	47573.45
Cement	0.00315	Cement	4177.36
Soil	0.0026	Soil	27833.8
		Total volume	79584.61
Height (mm)		Results	
Sample	25.76	Void Ratio	1.55
Water	15.02	Saturation (%)	95.954
Cement	1.32	Height check	TRUE
Soil	8.79		
Total Solid height	10.11		
Void height	15.66		
Before test		After test	
			



File Name:	4a	Date mixed:	12/26/2017
Sample Des.	WMC250-14D	Start of test:	1/8/2018
Client:	Golder Ottawa	End of test:	
Ring Information		Tested by:	A.A
Ring No.	1	Checked by:	JL
Dia (mm)	63.5	Porous stone	
Height (mm)	25.4	Height (mm)	6.33
Mass (g)	217.14	Mass (g)	93.81
Mass of sample + ring (g)		Mass of Sample (g)	
Before test	344.71	127.57	
After test	345.15	128.01	
Height of soil (mm)	1	2	3
Sample (before test)	25.32	25.25	25.26
Sample + filter + Porous Stone (before test)			
Sample (After test)	25.08	25	25.08
Change in height (mm)	0.22		
Loading Information			
		Rate (%hr)	Pressure (psf)
1	L	1	50000
2	U	0.25	2000
Saturation Information			
Back Pressure (psi)	50	Start of saturation	End of saturation
Ramp time (min)	60	1/8/2018 11:20	1/9/2018 23:23
Seating Stress (Psf)	200		
Water Content			
Before Test	Trial 1	Trial 2	Trial 3
Dish No.	#S-208	G3S59	G1S4
Mass of dish (g)	14.15	14.24	14.08
Mass of wet soil + dish (g)	38.32	44.83	27.88
Mass of dry soil + dish (g)	28.5	32.47	22.28
Mass of water, Mw (g)	9.82	12.36	5.6
Mass of dry soil, Ms (g)	14.35	18.23	8.2
Water content, W (%)	68.43	67.80	68.29
Average water content	68.18		
After Test	Trial 1	Trial 2	Trial 3
Dish No.	4a		
Mass of dish (g)	217.14		
Mass of wet soil + dish (g)	345.15		
Mass of dry soil + dish (g)	293.8		
Mass of water, Mw (g)	51.35		
Mass of dry soil, Ms (g)	76.66		
Water content, W (%)	66.98		
Average water content	66.98		
Comment:			

Mix design			
Dia (m)	Height (m)	Assumed vol of soil (m3)	Cement dosage (kg/m3)
0.0635	0.0253	8.0E-05	250
Soil in the mix (g)	Cement (g) in the mix	Water(g) in the mix	Total mix weight (g)
134.72	20.01	14.01	168.74
Soil in the ring (wet) (g)	Cement (g) in the ring	Water (g) in the ring	Total sample weight(g)
101.85	15.13	10.59	127.57
Each portion in the ring			
Surface area (sq.mm)	3166.92	Mass (g)	
Water Content (%)	68.18	Sample (wet)	127.57
Additives (kg/m3)		Sample (Dry)	75.86
Cement	250	Total water	51.71
		Cement	15.13
		Water Added	10.59
		Soil (dry)	60.73
Density (g/mm3)		Volume (mm3)	
Water	0.001	Water	51714.50
Cement	0.00315	Cement	4802.94
Soil	0.0026	Soil	23356.2
		Total volume	79873.69
Height (mm)		Results	
Sample	25.28	Void Ratio	1.84
Water	16.33	Saturation (%)	99.662
Cement	1.52	Height check	TRUE
Soil	7.38		
Total Solid height	8.89		
Void height	16.39		
Before test		After test	
			

File Name:	5a	Date mixed:	2018-01-08
Sample Des.	WMC150-7D	Start of test:	2018-01-14
Client:	Golder Ottawa	End of test:	2018-01-16
Ring Information		Tested by:	A.A
Ring No.	1	Checked by:	J.L.
Dia (mm)	63.5	Porous stone	
Height (mm)	25.4	Height (mm)	6.33
Mass (g)	217.14	Mass (g)	93.81
Mass of sample + ring (g)		Mass of Sample (g)	
Before test	340	122.86	
After test	340.23	123.09	
Height of soil (mm)	1	2	3
Sample (before test)	25.22	25.25	25.32
Sample + filter + Porous Stone (before test)			
Sample (After test)	24.97	24.9	24.92
Change in height (mm)	0.33		
Loading Information			
		Rate (%hr)	Pressure (psf)
1	L	1	50000
2	U	0.25	2000
Saturation Information			
Back Pressure (psi)	50	Start of saturation	End of saturation
Ramp time (min)	60	2018-01-14 12:30	2018-01-15 12:35
Seating Stress (Psf)	200		
Water Content			
Before Test	Trial 1	Trial 2	Trial 3
Dish No.	#S208	S-305	G3S5a
Mass of dish (g)	14.14	14.26	14.24
Mass of wet soil + dish (g)	32.54	34.64	42.31
Mass of dry soil + dish (g)	24.49	25.74	30.12
Mass of water, Mw (g)	8.05	8.9	12.19
Mass of dry soil, Ms (g)	10.35	11.48	15.88
Water content, W (%)	77.78	77.53	76.76
Average water content	77.36		
After Test	Trial 1	Trial 2	Trial 3
Dish No.	crs1		
Mass of dish (g)	217.14		
Mass of wet soil + dish (g)	340.23		
Mass of dry soil + dish (g)	286.93		
Mass of water, Mw (g)	53.3		
Mass of dry soil, Ms (g)	69.79		
Water content, W (%)	76.37		
Average water content	76.37		
Comment:			

Mix design			
Dia (m)	Height (m)	Assumed vol of soil (m3)	Cement dosage (kg/m3)
0.0635	0.0253	8.0E-05	150
Soil in the mix (g)	Cement (g) in the mix	Water(g) in the mix	Total mix weight (g)
134.65	12.00	8.40	155.05
Soil in the ring (wet) (g)	Cement (g) in the ring	Water (g) in the ring	Total sample weight(g)
106.69	9.51	6.66	122.86
Each portion in the ring			
Surface area (sq.mm)	3166.92	Mass (g)	
Water Content (%)	77.36	Sample (wet)	122.86
Additives (kg/m3)		Sample (Dry)	69.27
Cement	150	Total water	53.59
		Cement	9.51
		Water Added	6.66
		Soil (dry)	59.76
Density (g/mm3)		Volume (mm3)	
Water	0.001	Water	53586.79
Cement	0.00315	Cement	3018.82
Soil	0.0026	Soil	22986.1
		Total volume	79591.73
Height (mm)		Results	
Sample	25.26	Void Ratio	2.08
Water	16.92	Saturation (%)	0.992
Cement	0.95	Height check	TRUE
Soil	7.26		
Total Solid height	8.21		
Void height	17.05		
Before test		After test	
			

File Name:	5b	Date mixed:	2018-01-08
Sample Des.	WMC150-14D	Start of test:	
Client:	Golder Ottawa	End of test:	
Ring Information		Tested by:	A.A
Ring No.	1	Checked by:	J.L.
Dia (mm)	63.5	Porous stone	
Height (mm)	25.4	Height (mm)	6.33
Mass (g)	217.14	Mass (g)	93.81
Mass of sample + ring (g)		Mass of Sample (g)	
Before test	339.32	122.18	
After test	341.05	123.91	
Height of soil (mm)	1	2	3
Sample (before test)	25.19	25.24	25.31
Sample + filter + Porous Stone (before test)			
Sample (After test)	25.01	25.07	25.04
Change in height (mm)	0.21		
Loading Information			
		Rate (%hr)	Pressure (psf)
1	L	1	50000
2	U	0.25	2000
Saturation Information			
Back Pressure (psi)	50	Start of saturation	End of saturation
Ramp time (min)	60	2018-01-22 9:35	2018-01-23 9:36
Seating Stress (Psf)	200		
Water Content			
Before Test	Trial 1	Trial 2	Trial 3
Dish No.	#S208	S-305	G3S5a
Mass of dish (g)	14.14	14.26	14.24
Mass of wet soil + dish (g)	32.54	34.64	42.31
Mass of dry soil + dish (g)	24.49	25.74	30.12
Mass of water, Mw (g)	8.05	8.9	12.19
Mass of dry soil, Ms (g)	10.35	11.48	15.88
Water content, w (%)	77.78	77.53	76.76
Average water content	77.36		
After Test	Trial 1	Trial 2	Trial 3
Dish No.	5b		
Mass of dish (g)	217.41		
Mass of wet soil + dish (g)	341.05		
Mass of dry soil + dish (g)	287.93		
Mass of water, Mw (g)	53.12		
Mass of dry soil, Ms (g)	70.52		
Water content, w (%)	75.33		
Average water content	75.33		
Comment:			

Mix design			
Dia (m)	Height (m)	Assumed vol of soil (m3)	Cement dosage (kg/m3)
0.0635	0.0252	8.0E-05	150
Soil in the mix (g)	Cement (g) in the mix	Water(g) in the mix	Total mix weight (g)
134.56	11.99	8.40	154.95
Soil in the ring (wet) (g)	Cement (g) in the ring	Water (g) in the ring	Total sample weight(g)
106.10	9.46	6.62	122.18
Each portion in the ring			
Surface area (sq.mm)	3166.92	Mass (g)	
Water Content (%)	77.36	Sample (wet)	122.18
Additives (kg/m3)		Sample (Dry)	68.89
Cement	150	Total water	53.29
		Cement	9.46
		Water Added	6.62
		Soil (dry)	59.43
Density (g/mm3)		Volume (mm3)	
Water	0.001	Water	53290.20
Cement	0.00315	Cement	3002.11
Soil	0.0026	Soil	22858.9
		Total volume	79151.21
Height (mm)		Results	
Sample	25.25	Void Ratio	2.09
Water	16.83	Saturation (%)	0.985
Cement	0.95	Height check	TRUE
Soil	7.22		
Total Solid height	8.17		
Void height	17.08		
Before test		After test	
			



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