University of Alberta

Mechanical Characteristics of Soils Treated with a Liquid Stabilizer

by

Dong Ming Zheng

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Abstract

Conaid is a liquid ionic soil stabilizers used as a compaction aid or a stabilizer for soil improvement. The material is a mixture of surfactant, stabilizer and high-charge ion sulfide compound. This product may have the potential benefits of removing absorbed water from soil particles with the aid of compaction which results in an increase in shear strength. Despite these potential advantages, there is a lack of geotechnical testing on the material as well as knowledge on its chemical composition and field performance. As a result, a laboratory testing program was conducted at the University of Alberta between September 2010 and June 2012.

In this study, standard ASTM laboratory tests were carried out to examine the potential benefits of Conaid. Two natural soils and two clayey soils (kaolinite, montmorillonite) were selected for this study. A wide range of Conaid contents, varying from 0.01% to 0.5% by weight, were applied in treating soils. The effects on the compaction characteristics, shear strength and volume change in triaxial compression tests were evaluated. In addition, scanning electron microscope (SEM) was used to examine the microstructure of untreated and treated soil samples.

The test results show some changes, negligible in some cases, on the mechanical properties of the soils treated with Conaid. Also, both Standard and Modified Proctor compaction tests show negligible or even negative effects on maximum dry density and optimum moisture content of test soils except for kaolinite. However, no significant, consistent increases in the shear strength are observed from consolidated drained triaxial compression tests and unconfined compression tests. There is no obvious alteration in the

microstructure of the treated soils when they are examined using a SEM except some minor changes are noted in the soil structure.

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1. Introduction

1.1. Project background

Traditional soil stabilizers, such as cement, lime, fly ash and bituminous materials, have been successfully used in pavement base and highway subgrade to improve the engineering properties of soils for decades. However, these bulk stabilizer products are very costly to use and they may have a significant impact on the environment. Furthermore, quality control on these materials cannot be easily achieved as well. In addition, stabilization with those traditional products may cause excessive soil expansion induced by the reaction between a calcium-based stabilizer and sulfate present in the soil (Harris *et al*, 2005).

In recent years, numerous nontraditional soil stabilizers are commercially available in the market. The nontraditional stabilizers are listed as acids, electrolytes, enzymes, polymers, resins and sulfonated oils in the research conducted by Harris, *et al* (2005). Scholen (1992) classified these chemical stabilizers as electrolytes, enzymes, acrylic polymers and mineral pitches. Most of these products are usually advertised as either a compaction aid or a stabilizer (Greening *et al*, 2003).

Among these nontraditional stabilizers mentioned above, liquid ionic stabilizers (sulfonated oils) have been made more aware by commercial suppliers and distributors. A number of successful uses of this product in the field have been reported. Liquid ionic stabilizers are included in the class of electrolytes (Katz *et al*, 2001). Usually, liquid ionic stabilizers are distributed as concentrate, diluted with water at the site and then sprayed on subgrade soil. According to the potential advantages claimed by suppliers and distributors, higher compacted dry density, shear strength as well as waterproofing effect of this product could be obtained for the treated soils.

Despite the potential advantage performances claimed by liquid ionic stabilizer providers, most agencies and engineers are hesitate to accept the use of these products. This lack of confidence can be caused by the following:

1) The liquid ionic stabilizer products are usually patented and their chemical composition is often not listed fully or unknown. This makes it difficult to understand the mechanism of stabilization.
2) Many manufacturers failed to demonstrate the benefits of their products with data from standard laboratory testing methods. Field performance data from treated and untreated sections are poorly documented and lack of long term results.
3) Stabilizer suppliers cannot specify application ratios according soil types, temperatures and curing conditions. Standard laboratory testing protocol concerning application ratios needs to be set up. In many cases, application ratios recommended by stabilizer suppliers are poorly defined.

As a result, an independent laboratory testing program was conducted at the University of Alberta for a commercial stabilizer Conaid. In this study, standard ASTM laboratory tests were carried out to determine the potential advantages of Conaid, a liquid ionic stabilizer provided by a supplier, whose effects on compaction characteristics, shear strength and microstructure of test soils were evaluated.

1.2. Objective of current research

The objective of this research is to measure and understand the potential effects of the selected liquid ionic stabilizer on the mechanical characteristics of soils.

Recent published studies have tested natural soils obtained in the field (Greening *et al*, 2003, Zhang *et al*, 2007), which contain a mixture of many different minerals. Data obtained from laboratory tests on natural soils can be used as a reference in field construction. However, knowledge about the natural soils lacks information on clay mineralogy. In some cases, it is hard to observe the engineering property caused by the stabilizer in a natural soil due to its complex constituent. Therefore, in this study two natural soils and two “pure” clayey soils were selected.

Conaid was selected as the soil stabilizer in this research, which is a kind of sulfonated petroleum products (SPP). According to Katz *et al* (2001), Conaid can be classified as one of the electrolytes. The major chemical ingredient of Conaid is alkyl-benzene-sulphonate derivatives. Recent studies on this product have reported either positive or negative changes in soil characteristics. Zhang *et al* (2007) reported that the strength of the stabilized soil has increased under different application ratios with this product. California Bearing Ratio (CBR) tests were conducted by Basir *et al* (2010) on Canlite BTS soil stabilizer, which is a similar product manufactured by the Conaid supplier. But improvement effects from Canlite BTS cannot be verified due to inconsistent CBR results covering a wide range of values. However, Katz *et al* (2001) indicated that similar chemical changes induced from a sulfonated oil electrolyte can be accomplished through the addition of sulfuric acid. The test conducted by Rauch *et al* (2002) failed to show consistent and significant changes in the properties of three reference clays (kaolinite, illite and montmorillonite) and two CH natural clays. One may argue that standard laboratory testing cannot exactly simulate field conditions. However, it is believed that if test conducted on a well-prepared laboratory specimen does not show significant effects, it is difficult to expect much better results in the field. In this study, two reference clays (kaolinite, montmorillonite) and two natural soils (Devon silt, Anthony Henday clay) were selected to test with Conaid. Conaid contents, the stabilizer/ soil ratios (by weight) of 0.01%, 0.025%, 0.25% and 0.5% were applied in samples in compaction tests followed by shear strength tests. The first step in laboratory testing is to determine the Atterberg limits of the soils, which is followed by mixing high-quality laboratory samples and compacting them according to Standard or Modified Proctor methods. After compaction, a series of unconfined compression tests and drained isotropically consolidated triaxial compression tests were conducted.

Also, scanning electron microscope (SEM) was used to capture high resolution images of soil specimens in understanding the microstructure of treated soils and the effects of the stabilizer on compaction and shear strength characteristics. The influencing factors, such as curing time and conditions, moisture content and Conaid content on the microstructure of the compacted soil were taken into account.

All the laboratory tests were carried out at the University of Alberta between September 2010 and June 2012. A lot of efforts were spent in establishing a standard protocol to examine the effects of this chemical additive on the engineering properties of the soils.

This research provides the framework for further study of this product or other similar products. Field tests are recommended to verify the conclusions obtained from laboratory tests. Besides, durability tests, such as thaw/freeze circle and wet & dry test are also recommended.

1.3. Outline of thesis

This thesis includes seven chapters. A brief description of the contents in each chapter is given as follows.

Chapter 1: the research background, objectives and outlines of the thesis.

Chapter 2: a literature review of nontraditional soil stabilizers, the mechanism of Conaid, previous studies carried out by other researchers, and recommended field construction procedures with Conaid application.

Chapter 3: selection and description of test soils, introduction of index properties tests, Proctor compaction tests, sample preparation, triaxial tests and unconfined compression tests on untreated and treated soils.

Chapter 4: effects of Conaid on Atterberg limits, Standard and Modified Proctor tests for Conaid-treated soils.

Chapter 5: results of consolidated drained tests and unconfined compression tests on Conaid-treated soils and control samples.

Chapter 6: SEM tests on selected soils, microstructures of both treated and untreated soils were observed.

Chapter 7: summary and conclusion.

Appendixes: tables, plots and photos showing additional information on laboratory apparatus, samples before and after the tests, SEM results.

1.4. Extent and limitations

This thesis is limited to the mechanical behavior of the selected soils treated by Conaid. And conclusion is based upon the results from the tests conducted in the laboratories at the University of Alberta.

The study is focused on short-term standard laboratory conditions; no attention has been given to long-term field conditions.

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2. Mechanisms of soil stabilization with Conaid

2.1. Literature Review

Many researches have been carried out on traditional soil stabilizers, but much fewer published documents and reports are available on nontraditional soil stabilizers, especially liquid ionic stabilizers (SPP). Most suppliers of SSP try to demonstrate the benefits of their products through successful projects experiences. However, information disclosed is usually poorly documented and subjective. There is still lack of confidence towards these products for engineers. As a result, independent researches sponsored by government agencies, like the Texas Department of Transportation, the Department of the Army Headquarters and Alberta Research Council, had been carried out as early as 1990s or even earlier. However conclusions are variable and testing methods were not standardized and therefore further studies are warranted.

Scholen (1995) reported a number of successful projects in which several nontraditional products were used although there were some failures. Zhou (1999) verified that NRS (a cationic soil stabilizer) is able to greatly reduce the water imbibition into the soils, implying that frost damage can be reduced. But a fully assessment upon field performance wasn't discussed in this paper. Through testing several combinations of seven (7) kinds of SPP and five (5) test soils, Greening *et al* (2003) concluded that a significant beneficial effect on the strength of some soils can be achieved if the combination of soil and stabilizer is correct. In other words, liquid ionic stabilizers may only affect certain soils with an optimal application ratio. Santoni *et al* (2002) found that the unconfined compressive strength of the SM soil stabilized with polymers was increased by an average of 57% and 221% in dry and wet test conditions respectively when compared with values of the control samples. However, experiments conducted by Harris *et al* (2005) indicated that the majority of the nontraditional stabilizers (including liquid ionic stabilizers) tested at the manufacturers' recommended concentrations did not reduce swelling any more than just adding water to soils. Some of the nontraditional soil stabilizers reduced the swelling in short-term but long-term effects on improvement is questionable. Katz *et al* (2001) examined the effects of sulfonated limonene on sodium montmorillonite with X-ray diffraction (XRD) and SEM. The results showed that there were similar changes in clay mineralogy between soils treated by sulfonated limonene and sulfuric acid. Katz presumed that higher application ratios (much more than manufacturer's suggestion) may improve the properties of some soils. With standard laboratory soil tests, Rauch *et al* (2002) did not observe consistent and significant changes in the properties of the soils treated with ionic, polymer and enzyme stabilizers. Petry (2001) concluded that some chemical agents improve the overall clay behavior and some provide specific improvements.

2.2. Conaid as a liquid soil stabilizer

Conaid, like other liquid ionic stabilizers, may be used to improve compaction workability and shear strength during and after road construction. The selection of liquid ionic stabilizers not only depends on the type of soils available for road construction on site, but also on the expected improvement of engineering properties to be achieved. Various similar products are available in the market, such as ROADBOND EN1, ClayStar 7, CBR PLUS and ISS 2500. These products are nearly all marketed with the benefits of reducing the thickness of absorbed water layer in the soil particles with the aid of compaction and it will protect roadbed effectively from damage of water and therefore increase the shear strength, reduce swelling and avoid mudding effect, etc.

According to the manufacturer, Conaid is a synthetic brown viscous fluid, which is water soluble, non-toxic, non-flammable and non-polluting artificial liquid. Based on the manufacturer specifications, the major chemical ingredients of Conaid are shown in the Table 2.1. The specific gravity of Conaid is 1.02 and the pH value ranges from 3 to 4.

Table 2.1 indicates that the major reactive component in Conaid is known as Alkyl-benzene-sulphonate, which is assumed to be able to (Greening *et al*, 2003):

1). reduce the surface tension of compaction water;

2). neutralize the exchangeable cation in the soil and allow the adsorption of molecules with hydrophobic properties to the clay particles.

Table 2.1 Major Chemical Ingredients of Conaid

Weight Ratio (%)	Chemical Ingredient
32~42	Alkyl-benzene-sulphonate Derivatives
6~12	Alcohol-polyoxyethylene-ether-sodium-sulphate
0.5~3	Dilute Sulphuric Acid
2~4	Sulpho-sulfosuccinate-monoester-disodium-salt
1~3	Neutral Silicate Sodium
38.5~54	Water

2.3. Mechanism of Conaid

According to Conaid product specifications, the chemical molecules contain a hydrophilic side and a hydrophobic side. After dissolving it in water, these molecules form chemical bonds with the soil particles so that the soil particles can be encapsulated with a permanent, oily membrane. Hence, the treated soil can be changed from hydrophilic to hydrophobic. Furthermore, through ion exchange process with the aid of external mechanical compaction, water can be expelled from the double layer and higher compacted dry density can be achieved (Conaid product pamphlet, 2007), whose mechanism is demonstrated in Figure 2.1.

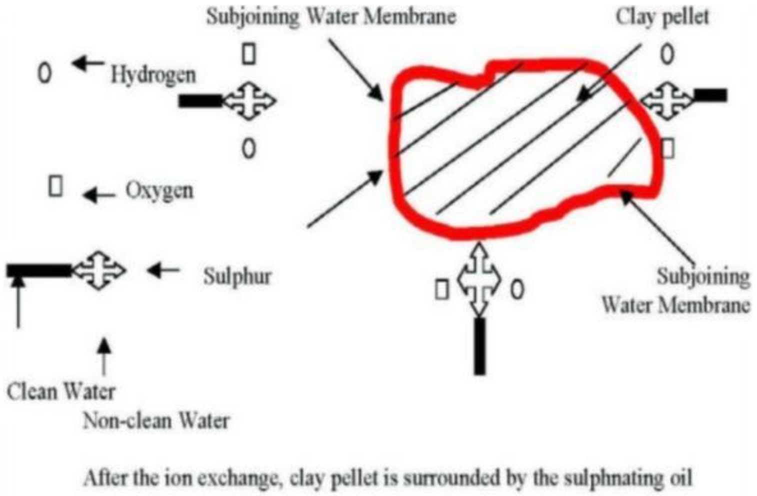

Figure 2.1 Mechanism of Conaid effects on soil particles (Conaid product pamphlet, 2007)

It seems like that the explanation given by Conaid supplier is reasonable. However, it doesn't show the mechanism of improvement in compacted density due to the addition of Conaid as a compaction aid. Regardless of the type of liquid soil stabilizers, compaction both in laboratory and on site plays a vital role in capitalizing the potential benefits from liquid soil stabilizers.

2.4. Recommended field construction procedures

Field construction steps with Conaid application are recommended by the supplier. The major steps are (Conaid product pamphlet, 2007):

1. Sprinkle the diluted Conaid solution on excavated soil on roadbed evenly;
2. Pulverize and well blend the soil to the optimum moisture content (OMC) and compact the soil by rollers;
3. In the following 5~10 days after the completion of construction, Conaid solution (1:1000 of Conaid : water) should be re-sprayed for curing and the roadbed should be continuously compacted by heavy loaded vehicles.

In addition, it was mentioned that clay content in roadbed should be 25% and Conaid will achieve the best results after 50 days. Besides, several successful applications are listed in the specifications and significant effects on liquid limit, dry density and OMC are also demonstrated by the manufacturer.

2.5. Discussion

In essence, Conaid can be grouped as one of several liquid ionic stabilizers (SPP) with regard to its major chemical ingredients and mechanisms described in its specifications. According to Conaid inventor, clay minerals are broken down by expulsion of double layer water through exchanging of interlayer cations. Thus the double layer thickness becomes thinner and the electrical repulsion is reduced, which in turn causes more flocculation in soil particles. As a result, reduced interparticle electrical repulsion and more random orientations can result in denser structure and higher strength for soils with the same moisture content (Lambe, 1958). In addition, it was claimed that soil particles could be encapsulated by the chemical additive and becomes waterproof against deterioration from excessive water. Therefore, less volume change can be expected during the specimen saturation and consolidation in drained triaxial compression test.

If the mechanism described above is effective using Conaid, soil improvements could be expected in standard laboratory tests as long as the combination of this additive (including Conaid contents) and selected soils is correct. The expected improvements are:

1) The maximum dry density (MDD) in compaction test should increase and the OMC could decrease;
2) Significant improvement on shear strength for treated samples than control ones should be achieved.

Besides, some changes in clay mineralogy and soil microstructure due to the addition of Conaid would be observable using the SEM. Also, long term effects on the engineering properties of the treated soils should be evaluated upon the application of Conaid on highway subgrade construction.

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3. General description of laboratory testing program

3.1. Selection of test soils

Three types of soils, listed in Table 3.1, were tested in the laboratory. The first two soils contain two most common clay minerals: kaolinite and montmorillonite. The reason to choose these two homogenous pure clay soils is to better observe the potential physical-chemical changes induced by Conaid. Two natural soil samples, Devon silt and Anthony Henday clay, are soils commonly encountered in highway construction around capital region of Alberta.

Table 3.1 Test soils information

Soil	Source	Modified Unified Soil Classification System (PFRA, 1985)
Kaolinite	“Pioneer Kaolin” from Kentucky –Tennessee Clay Company, Mayfield, Kentucky	Clay (CI)
Sodium Montmorillonite	“Aquagel Gold Seal Bentonite” from Halliburton Company	Fat clay (CH)
Devon Silt	Devon County, Alberta	Silty clay (CI)
Anthony Henday Clay	Anthony Henday Drive, Edmonton	Silty clay (CI)

3.1.1. Kaolinite clay description

The kaolinite used in the laboratory tests is called Pioneer Kaolin which came from Kentucky-Tennessee Clay Company, Mayfield, Kentucky. The major ingredients and physical & chemical properties of the kaolinite sample are summarized in Table 3.2 and Table 3.3 (IMERYS Pigments & Additives Group, 2000).

Table 3.2 Ingredient analysis of Pioneer Kaolin

Ingredients	Weight (%)
Kaolin - $Al_2Si_2O_5(OH)_4$	>98%
Crystalline Silica, Quartz	0.1% ~1%
Water	<2%

Table 3.3 Physical & chemical properties of Pioneer Kaolin

Physical State	Solid
Appearance & Odor	Odorless, white powder
pH	4.0~6.0
Specific Gravity	~2.6
Liquid Limit (%)	47.7
Plastic Limit (%)	27.1
Plastic Index (%)	20.6
C.E.C (meq./100g)*	3~15

* from Mitchell (1976)

3.1.2. Description of sodium montmorillonite

The brand of the sodium montmorillonite sample used in the laboratory tests is Aquagel Gold Seal®, which is a registered trademark used for bentonite clay and owned by Halliburton Energy Services, Inc. The major ingredients and physical & chemical properties of the bentonite sample are summarized in Table 3.4 and Table 3.5 (MSDS of Aquagel Gold Seal®, 2011).

Table 3.4 Ingredient analysis of Aquagel Gold Seal®

Ingredients	Weight (%)
Bentonite	60%~100%
Crystalline silica, cristobalite	0%~ 1%
Crystalline silica, tridymite	0%~1%
Crystalline silica, quartz	1%~5%

Table 3.5 Physical & chemical properties of bentonite

Physical State	Solid
Appearance & Odor	Mild earthy, tan powder
pH	8.0 - 10.0
Specific Gravity	2.6
Bulk Density (lbs/ft^3)	50~73
Liquid Limit (%)	495
Plastic Limit (%)	48
Plastic Index (%)	447
C.E.C (meq./100g)*	80~150

* from Mitchell (1976)

With extremely high cation exchange capacity (CEC) (80~150 mpg/mg) and large specific surface, bentonite is widely used for drilling mud, backfill chips and hydraulic containment liner (Mitchell 1976, Chalermyanont 2005). Conaid is expected to have significant effect on engineering properties of bentonite clay due to its comparably high CEC and large specific surface.

3.1.3. Devon silt and Anthony Henday clay

Devon silt, found in Devon County, Alberta, is brownish silty clay with low to intermediate plasticity, which can be classified as a CI soil in the Modified Unified Classification System for Soils (PFRA, 1985). Anthony Henday clay, collected from Anthony Henday Drive construction site, Edmonton, Alberta, has similar appearance and consistency with Devon silt. It is not as homogenous as Devon silt and it contains some sand, fine gravel and other substances, which is closer to an engineering soil. Atterberg limits and specific gravity of these two soils are listed in Table 3.6.

Table 3.6 Physical properties of Devon silt and Anthony Henday clay

Soil Type	Devon Silt	Anthony Henday Clay
Specific gravity	2.69	-
Liquid limit (%)	36	39
Plastic limit (%)	21	23
Plastic index (%)	15	16
C.E.C (meq./100g)	-	-

3.2 Conaid content

The recommended Conaid content, that is Conaid/soil mass ratio for laboratory specimens, is 0.1% (percentage by weight of Conaid concentrate to dry soil). To find the optimal Conaid content in laboratory, a variety of Conaid contents from 0.01% to 1% were used in the testing program.

3.3 Conaid effects on index properties

Atterberg limits (ASTM D 4318) test was conducted on each untreated and treated soil. Tests were carried out within a few hours so that the effects of curing time were not considered. The index properties of each soil treated with different Conaid contents are summarized in Table 3.7.

Table 3.7 Liquid limit for soils treated with Conaid

Conaid Content, A_c (%)	Devon Silt	Pioneer Kaolin	Bentonite
0	36.1	47.7	495
0.05	38.8	44.7	450
0.25	38.0	45.3	428
0.5	38.9	46.4	-
1	-	-	435

Table 3.7 shows that Conaid caused some changes on the liquid limits of the soil indicating that the thickness of the double layer surrounding soil particles has changed with the addition of Conaid. Reducing the thickness of the double layer means more water can be released from it and therefore less water need to be added to lubricate the soil particles.

3.4 Proctor compaction test

Not only the OMC and MDD can be determined from the Proctor compaction test, but also all the specimens in the following shear strength tests were extruded from the compacted soil samples in a mold. As a result, Proctor compaction test played a vital role in studying the potential effects of liquid ionic stabilizers on test soils. Soil compaction has been widely used to determine the OMC and MDD in the construction of roads, highways, dams and landfills. Since Proctor's work in 1933, compaction tests in laboratory have been carried out by many researchers to study the mechanism in densification. Lee and Suedkamp (1972) conducted research on the shapes of the compaction curves for different soils. Lambe (1958) explained the curves of compaction in terms of physico-chemical interactions. Nowadays a compacted field dry density of 90% to 95% of the MDD determined by Standard Proctor test or Modified Proctor test is required in most specifications in engineering projects.

3.4.1 Standard Proctor test

The ASTM D-698 recommends the diameter of a standard mold is 101.6 mm and has an effective height of 116.63 mm. The mold has a detachable base plate and 2-inch height of collar at its top. Soil sample is compacted in the mold in three equal layers by a 24.4 N rammer with a drop of 304.8 mm, and 25 times for each layer. A compaction curve can be obtained between the dry density and moisture content. Then, the OMC and MDD can be obtained through the compaction curve which has a single peak.

3.4.2 Modified Proctor test

While requiring a higher subgrade density greater than that through the Standard Proctor compaction, the Modified Proctor compaction test was introduced during World War II. The ASTM D-1557 and AASHTO T-180 describe all the details of this method, which is believed to be able to better mimic field conditions if heavy rollers are equipped. The essentials of the Modified Proctor test are the same as the Standard Proctor test. In the Modified Proctor test, there are five (5) layers compacted with a 44.5 N hammer other than 3 layers by 24.5 N hammer. Consequently, the compaction energy of the Modified Proctor test is about 5 times more than that of the Standard Proctor test. For the increased energy applied on the soil, there is a 5% to 10% increase in density together with a lower OMC.

3.4.3 Review of literatures on compaction test for the test soil treated with liquid stabilizers

In terms of the product specifications of Conaid, compaction with rollers on site at OMC is suggested to improve the MDD and therefore achieve higher shear strength or bearing capacity for the treated subgrade.

Besides the Standard Proctor compaction test mentioned above, other compaction methods were also used to compact soil samples in laboratory, such as the Harvard miniature compaction apparatus (Wilson *et al*, 1950), the vibrating table (Greening *et al*, 2003), the gyratory compaction machine (Santoni *et al*, 2002) and the static compaction method (Wibawa *et al*, 2009). Greening *et al* (2003) used the Transportek vibrating table and Modified Proctor compaction procedure to examine the potential effects of SPP on soil compaction characteristics. A significant increase in the soil density was observed using the modified AASHTO hammer rather than the vibrating table. Similarly, Rauch *et al* (2002) indicated that the soil samples in his study were compacted with a modified effort (ASTM D 1557). However, In Zhou's research (1999), all compaction tests were carried out following the Standard Proctor test (ASTM D 698) except for using a pneumatic kneading hammer. Also, Harris *et al* (2005) reported that the Standard Proctor compaction tests were used to determine the OMC and the corresponding density. But the 3-D swelling samples were prepared with the Superpave gyratory compactor by molding the soil in one lift. In addition, a gyratory compaction machine was used to compact silty-sand (SM) samples treated with various liquid stabilizers (Santoni *et al*, 2002). The gyratory machine was believed to be able to produce the same density as the ASTM D 1557 compaction with a suitable ram pressure and number of revolutions.

To better compare test results with those obtained by other researches on liquid soil stabilizers, the Standard Proctor test (ASTM D 698) as well as the Modified Proctor test (ASTM D 1557) were used in this research to prepare soil samples treated and untreated with Conaid.

3.4.4 Sample preparation and curing method

The strength of treated soils achieved in laboratory is greatly affected by sample preparation and curing method after compaction. To mimic field conditions as much as possible, construction procedures should be well understood while preparing the laboratory samples. Three major procedures in the construction of highway subgrade are described as mixing, compaction and curing respectively.

3.4.5 Laboratory sample mixing and standing (curing)

There are no details upon mixing procedure specified in the Conaid product specifications more than statements such as "well-mixing soils and getting rid of oversize clods larger than 100 mm" or "Conaid is diluted with a large amount of water and sprayed into the subgrade soils loosened by disc harrow prior to pressing with roller". In accordance with the Method A in the ASTM D 698, air-dry test soil passing through a No. 4 sieve (4.75 mm) should be thoroughly mixed with the appropriate mass of distilled water and Conaid dilution to obtain the desired moisture content. To prepare the Conaid dilution, the mass of Conaid concentrate should be calculated with the specified Conaid content, i.e. 0.025%. Then the amount of Conaid concentrate calculated was carefully dripped into a flask and mixed with distilled water. The amount of distilled water should be calculated according to the molding water content and moisture content of air-dry soil. The prepared Conaid dilution should stand at least 30 minutes prior to mixing. The distilled water and Conaid solution should be sprayed into the soil evenly during mixing. Mixing was done thoroughly using an electrical mixer fitted with a paddle type mixing blade (Figure 3.1) until a satisfied homogenous consistency was achieved throughout. The wet soil sample was sealed in a plastic bag and kept in a moisture control room at 4 ℃ and 100% humidity for at least 12 hours prior to compaction. This allows the moisture or liquid soil stabilizer to penetrate any small soil pellets and distribute more evenly. After the standing period, the mixed wet soil is ready for compaction.

Figure 3.1 Electrical mixer used in soil mixing

Most of compactions conducted here applied the method of Standard Proctor compaction because Bowles (1992) thought that no better method developed until now was used to establish the project quality control on field compaction unit weight. However, the only exception is for bentonite because compaction curves of bentonite produced by the Standard Proctor compaction were too flat to find the OMC. Therefore, Modified Proctor compaction was conducted in bentonite samples.

3.4.6 Sample trimming and curing

After compaction with a standard hammer in the laboratory, soil specimens can be extruded from the compaction mold using a hydraulic sample extruder and followed by careful cutting and trimming immediately. Specimens compacted dry of optimum are usually very brittle and easily broken while trimming. Extreme care is needed. The length/diameter ratios of all specimens used for compression tests are between 2 and 2.5 in accordance with ASTM standards. Some specimens were wrapped up with plastic film cured in moisture room, others cured in moisture room without plastic film or air-dried in room temperature directly until required time prior to compression tests.

3.5 Consolidated drained triaxial compression tests

To measure the effective shear strength of soil, consolidated drained (CD) triaxial compression tests were carried out. However, there is no standardized ASTM or AASHTO procedure for the CD test yet. Hence, the procedure described by Bishop and Henkel (1962), Head (1986) and Bowles (1992) were followed here in laboratory testing. The triaxial machine used in this research was shown in Figure 3.2.

The change of axial displacement was measured by a Linear Variable Differential Transformer (LVDT). Test data from transducers of LVDT, axial load, pressure and volume change can be automatically recorded in the *Dolphin* data recording system and downloaded at the completion of each test. The CD tests were performed at an effective confining stress of 50 kPa and 150 kPa.

Figure 3.2 Photo of triaxial test apparatus

The CD triaxial tests were also conducted on untreated samples with the same moisture content and curing condition as treated ones.

Table 3.8 Summary of CD triaxial tests performed

Moisture Content, w (%)	Conaid Content, A_c (%)
23	0
24	0.25
25	0.5

The strain rate for all tests was set at a constant 0.003mm/min. Prior to specimen saturation, the piston should not bear against the loading cap. However, while increasing cell pressure and back pressure to 400 kPa or higher, which often resulted in a significant axial loading reading in specimens. It is also noted that the friction between the piston and the top of the cell, the weight of piston and the strength of membrane were never measured directly. Therefore, determination of the initial loading due to deviator stress is very important.

3.5.1 Specimen set-up procedure

Before setting up the triaxial apparatus, all lines connecting cell, pore and back pressure should be flushed with de-aired water to remove any air bubble trapped in the lines and the apparatus. Porous stones were de-aired by boiling for at least 30 minutes and filter paper was saturated by placing in a bath of distilled water.

With the cells prepared, specimens wrapped up in plastic film were picked up from moisture room and the quality of the sample was carefully examined. If it is necessary, trimming should be conducted again by using thin wire saw and knife to ensure that the two ends' cross-sections were even.

After examination was done, the specimen was weighted and the moisture content was calculated from the trimmings. The procedures about specimen setting up are described as follows (Head, 1986):

1) Cover the base pedestal with a film of de-aired water.
2) Slide a saturated and de-aired porous disc on the pedestal without trapping any air.
3) Place a saturated piece of filter paper, trimmed to the exact diameter of the specimen, on the porous disc.
4) Place the prepared specimen on the filter paper without delay.
5) Place a second piece of saturated filter paper, followed by a second saturated and de-aired porous disc on top of the specimen.
6) Apply a thin smear of silicone grease at the vertical surfaces of the pedestal and loading cap to enable a good seal between the rubber membrane and the steel.
7) Using a membrane stretcher, fit a thin rubber membrane and 2 to 3 rubber O-rings around the specimen.
8) Place the triaxial cell over the specimen and screw tightly into place.
9) Carefully lower the cell body into position over the specimen with the piston raised to its maximum extent.
10) Allow the piston to fall slowly into contact with the ball on the top cap.
11) Fill the cell with water.
12) Keep the bleed open to maintain the cell at atmospheric pressure. The specimen is ready for saturation.

3.5.2 Specimen saturation

Following the specimen was set up on the triaxial cell pedestal and the cell was filled with water, all the valves on the triaxial cell were kept closed except for that of pore pressure transducer and cell pressure. Method 1 (Head, 1986), namely application of pressure increments to achieve full saturation, was used in this study. Under conditions of no drainage, the cell pressure was increased in increment of 60 kPa until it reached 410 kPa with a back pressure of 400 kPa. The cell pressure was increased simultaneously with back pressure. Then the specimen was left to saturate for overnight. B-test was carried out the next day. If a B-value ($B = {}^{\Delta u}/_{\Delta \sigma_3}$) was higher than 0.95, the sample can be considered to be fully saturated.

In this study, saturation of specimen took over a period of time, from approximately 24 to 72 hours. A minimum final back pressure of 400 kPa was applied to the kaolinite. Pressure increments proceeded until the desired confining pressure was achieved.

3.5.3 Specimen consolidation

Consolidation started once the back pressure valve and volume change valve were opened and it took about 24 hours. Plots of change in volume vs. time were generated. The coefficient of consolidation and coefficient of volume compressibility can be calculated based on the isotropic consolidation triaxial data by using the following equations (Head, 1986):

$$c_{vi} = \frac{\pi D^2}{\lambda t_{100}} \tag{3.1}$$

$$m_{vi} = \frac{\Delta V_c}{V_s} \times \frac{1000}{\Delta \sigma'} \tag{3.2}$$

3.5.4 Specimen shearing

Following specimen consolidation, the cell and back pressure valves as well as the volume change valve were kept open. Once the pressures were stabilized, the initial axial displacement and load were recorded, and the shearing phase of the triaxial test commenced.

The strain rate for all tests was set to 0.003mm/min, which is able to keep the fully drained condition to be applicable by the time failure is reached (Head, 1986). Shear tests normally run until at least 15% strain or until the deviator stress remained approximately constant with increasing strain. If practicable, drained tests were allowed to continue to 25% strain or more. Dismantling after test should follow the procedures specified by Head (1986) to prevent water from being sucked into specimen from the porous stones.

In calculating the deviator stress, some corrections made to take into account for the changes in area and volume due to barreling and drainage during consolidation and shear. The following equations were applied (Head, 1986):

$$(\sigma_1 - \sigma_3) = \frac{P}{A_c} \times \frac{100 - \varepsilon}{100 - \varepsilon_{vs}} \times 1000 \ \ kPa \qquad (3.3)$$

For each test, stress-strain (q vs. ε_a) curves were plotted, as well as the volumetric strain vs. axial strain. All triaxial samples were taken photos after shearing. These photos are provided in Appendix A.

3.5.5 Corrections to data

Head (1986) suggested the corrected area A due to drainage in a drained test by considering the change in volume:

$$A = \frac{1 - \frac{\Delta V}{V_c}}{1 - \frac{\varepsilon\%}{100}} A_c \qquad (3.4)$$

Here V_c is the volume at the start of compression, i.e. immediately after consolidation.

All other corrections, including membrane corrections, volume change corrections, piston friction and pipeline losses, are also discussed by Head (1986).

3.6 Unconfined compression tests

The unconfined compression (UC) test (ASTM D 2166) is often used in determining the shear strength of highway subgrade materials. In spite of wide range of error, it is still a popular test due to its simplicity (Bowels, 1992). Also, the UC test is a special type of unconsolidated-undrained (UU) test where the confining pressure is zero. Zhou (1999) measured the unconfined compressive strength (UCS) of treated and untreated Devon silt specimens soaked for 48 hours, and he found significant effects of NRS on the soil. Rauch *et al* (2002) stated that testing in triaxial cell in UU tests yield a more reliable measurement because specimen is kept intact under confining pressure. Bowels (1992) provided similar suggestion on testing samples with cracks or fissures which may results in too low UCS.

The UC tests were conducted on all treated and untreated soil specimens with different curing times and methods. Samples were extruded from compaction mold, and then trimmed into 38 mm in diameter by 76 mm in high. The strain rate was set as 0.3 mm/minute.

Samples were prepared the same way as the triaxial tests, using the same mixing equipment and compaction method. The compacted soil samples were trimmed into specimens and cured as required. While being air dried, fissures were noticeable on the surfaces of some UC specimens. Therefore, the major errors of UC test are from sample quality related to compaction and curing method.

All measured data were automatically recorded by the *Dolphin* data recording system. The peak stress, residual stress and strain at peak conditions can be obtained from the stress-strain curves.

Figure 3.3 Photo of unconfined compression test apparatus

Moisture contents were measured before curing and after sample failure by compression, indicating the relationship between strength and moisture content. All UC samples were photographed at the end of the test; these photos are included in Appendix B~F.

3.7 Experimental errors and repeatability

For all the tests mentioned above, experimental errors always exist even if the experiment were carried out by the same person. Judgment is needed to distinguish the effects of Conaid on soils from experimental errors. According to ASTM D4318-10, ASTM D698-07 and ASTM D1577-09, acceptable ranges of errors are listed in ASTM standards. In this testing program, at least two samples or multiple samples were tested to ensure repeatability of test results.

3.8 References

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4 Results of laboratory testing program

4.1 Introduction

In order to quantify the potential benefits on soils treated with Conaid liquid stabilizer, standard laboratory tests were conducted to determine the Atterberg limits, compaction characteristics and shear strength of both untreated and treated soils. It is believed that improvements achieved through standard laboratory tests could be expected on site if construction procedures are followed and properly controlled.

4.2 Effects on Atterberg limits of test soils treated with Conaid

The adsorbed water surrounding clay particles influences the plasticity of a soil. It is suggested that the thickness of the adsorbed water surrounding the clay particles can be reduced with the addition of Conaid, and therefore, the maximum dry density (MDD) can be achieved with less compaction effort or higher MDD with the same compaction effort.

It is known that Atterberg limits are related to the thickness of the diffuse double layer (Sridharan, 2002). Between the plastic and liquid limits, the liquid limit is susceptible to more variations than plastic limits and it is more easily interpreted (Mitchell, 1976). Casagrande (1932) concluded that a soil with moisture content around the liquid limit has shear strength of about 2.5 kN/m^2. Petry (2002) proposed that the Atterberg limits can be used as an indicator to measure the improvement of an additive in a clay sample although it does not necessary work for all types of clays. Therefore, liquid and plastic limits, especially liquid limit, may be used in the preliminarily assessment of potential benefits from a soil additive such as a liquid ionic stabilizer, i.e. Conaid or other similar products.

Treatment of Devon silt with Conaid contents between 0.05% and 0.5% caused a minor increase or decrease in the liquid and plastic limits (Table 4.1). As a result, there is a slight increase in the plasticity index (PI) of the soils. Treated Devon silt shows a slightly higher plasticity than the control samples, implying the shear strength of Devon silt could be affected with the addition of Conaid.

Table 4.1 Atterberg limits of Devon silt with and without treatment of Conaid

Conaid Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
0	37.8	21.4	16.4
0.05	38.8	-	-
0.25	38.0	18.9	19.1
0.5	38.9	19.5	19.4

Out of the three major types of clay minerals, the Atterberg limit of kaolinite is least likely to be affected by Conaid due to its low cation exchange capacity (CEC). Compared with the plastic limit, liquid limit is more sensitive to the change of Conaid contents. The effects of Conaid on Atterberg limits are examined on various Conaid to soil ratios.

Table 4.2 Atterberg limits of kaolinite clay with and without treatment of Conaid

Conaid Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
0	47.7	27.1	20.6
0.05	44.7	-	-
0.25	45.3	-	-
0.5	46.4	27.9	18.5

Similar to the Devon silt, a minor decrease in the liquid limit occurs when Conaid content varies from 0.05% to 0.5% (Table 4.2). Conaid content of 0.05% is the most effective ratio followed by 0.25%. Whereas 0.5% has the least change in the liquid limit, indicating that beyond the optimal Conaid content, Conaid may even make the material less compactable, thus leading to a possible decrease in shear strength as well. The treated soil may be too “oily” and the addictive may lubricate the soil particles instead of strengthening them (Greening, 2003).

Hence, compaction characteristics and shear strength of kaolinite clay may be improved with the proper Conaid contents, somewhere between 0.05% and 0.25% according to Table 4.2.

The expansive properties of montmorillonite make it an ideal candidate for examining the effects of commercial soil stabilizers (Katz, 2001). In theory, the negatively charged sulfite anions attract the cations and ionized water from the clay lattice resulting in a net negative potential toward the outside of the layer and destabilizing the clay lattice. Thus, the breakdown of clay lattice allows water to drain and leaves the clay lattice, which gradually forms a permanently hardened mineral material (Scholen, 1995). It may be postulated that montmorillonite particles could be positively charged after reacting with Conaid. Therefore, the montmorillonite lattice would breakdown and water in the double layer would be freed. The double layer water surrounding the montmorillonite particles becomes thinner resulting in a decrease in liquid limit. The most significant decrease in the liquid limit is 67% when Conaid is increased to 0.25%, which indicates potential effectiveness on reducing the swell potential for bentonite and other CH clay.

Table 4.3 Atterberg limits of bentonite with and without treatment of Conaid

Conaid Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
0	495	60	435
0.05	450	-	-
0.25	428	-	-
1.0	435	57	378

After testing three types of soils treated with Conaid, Conaid seems to have some effects on the Atterberg limits even though some changes seem like within experimental errors(ASTM 4318-10), which are between 1% and 2%. Since the optimal Conaid content has not been determined, the above conclusions are only applicable to the tests conducted in this laboratory testing program.

4.3 Effects on compaction characteristics of test soils treated with Conaid

4.3.1 Selection of compaction test methods

Since Conaid is known to be a compaction aid which can increase the MDD and decrease the OMC at the same time, the objective of this laboratory program is to examine whether or not Conaid has a positive effect on the compaction characteristics of soils. If it does, how much improvements can be expected?

In most specifications for earth work, the dry unit weight of the material compacted in the field is compared to the optimum dry density determined in the laboratory using the Standard Proctor test (ASTM D 698). Therefore, most of the compaction tests conducted in this investigation were Standard Proctor test. However, some Modified Proctor tests (ASTM D 1557) were also carried out because higher compaction effort is needed to produce a typical single peak compaction curve. For example, the compaction curve of bentonite produced by the Standard Proctor test is too flat to determine the MDD. But a single peak curve can be obtained by Modified Proctor method.

4.3.2 Standard Proctor test

A total of four (4) soils were tested, i.e. kaolinite, BK20 (mixture of 20% bentonite and 80% kaolinite by weight), Devon silt and Anthony Henday clay, and they were compacted following the Standard Proctor method to evaluate the effects of Conaid.

4.3.2.1 Preparation of kaolinite samples

The natural kaolinite clay used in the laboratory is called Pioneer Kaolin, an air-dry, very fine particle powder with white or light yellowish white color. After the soil was thoroughly mixed by adding distilled water to the molding water content (the adjusted water content of a soil that will be compacted), the wet soil sample was found to contain some lumps with diameters of 2.5~5.0 mm. The centers of these lumps were very dry. Hence, water and Conaid could not be evenly distributed in the lump and therefore they could not react with clay minerals. In addition, because the weight of the standard hammer (24.47 N) cannot break down all the lumps in the mold, there are many voids existed between these lumps (Figure 4.1). The randomly distributed voids will inevitably result in significant variations in compacted dry densities.

Figure 4.1 Compacted samples with and without passing No.8 sieve (2.36 mm)

However, ASTM D 698 does not discuss how to deal with these oversized lumps in wet mixed samples after the addition of water. Obviously, the quality of compacted soil specimen in the mold can be improved with finer mixed sample (Figure 4.1). To maximize the effects of Conaid on the kaolinite, the wet mixed samples in this laboratory were sieved to pass through No. 8 (2.36 mm) before compaction. To achieve this requirement, a pestle and mortar were used to grind those oversize pellets into finer particles that can pass through the No.8 (2.36 mm) sieve. Control samples were compacted directly without passing through the No. 8 sieve. Compaction curves can be compared between wet kaolinite samples with and without passing the No. 8 sieve.

4.3.2.2 Compaction curves of kaolinite wet sample without passing No.8 sieve

Figure 4.2 shows the compaction results of kaolinite being mixed in the desired water contents without passing through the No.8 sieve. The addition of 0.1% Conaid caused at least a 5% increase in the MDD. The difference of OMC between the treated and untreated soil is only about 0.5% which is considered to be negligible. Also, the compaction curve of treated kaolinite is much closer to the zero-air-void line than the untreated one. It seems that Conaid may provide some benefits for kaolinite in laboratory compaction.

4.3.2.3 Compaction curves of kaolinite wet sample passing No.8 sieve

If all the oversize fraction of wet particles larger than 2.36mm were grinded into finer particles and passed through the No.8 sieve prior to compaction, the compaction curves are shown in Figure 4.3. As coarse fraction was pulverized, voids were much reduced in the mold after compaction and therefore kaolinite sample can be more densely compacted (Figure 4.1). Besides, it is assumed that the effect of Conaid can be more significant in soil particles which are closer to each other after compaction.

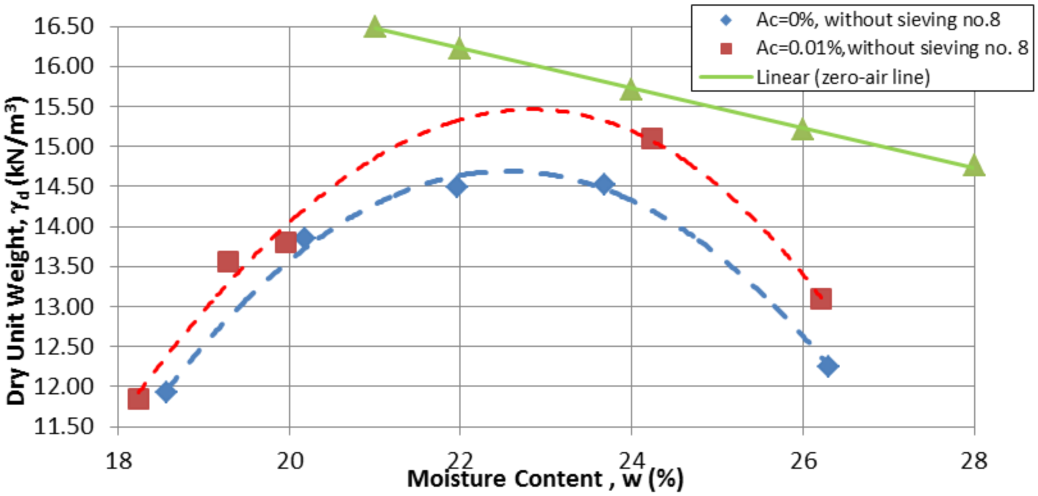

Figure 4.2 Compaction curve of kaolinite without passing No. 8 sieve

The addition of 0.25% Conaid causes an increase of 3.4% in dry unit weight while the molding water content is 23%. This variation in dry unit weight is probably outside experimental errors or variability (ASTM D 698-07). Also there is an approximately 1% difference in the OMC between treated (23.5%) and untreated soils (24.5%), indicating that a little less molding water is needed to achieve the OMC for the treated soil under the same compaction effort. In addition, the compaction curve of treated kaolinite is very close to the zero-air-void line, indicating that an improved compacted density can be achieved with the addition of 0.25% Conaid.

Figure 4.3 shows that the most significant improvement on dry unit weight is obtained while the molding water content lies between 20% and 23.5%. Conaid is effective if the soil is compacted around the optimum moisture content rather than too dry or too wet (Figure 4.3). Otherwise, the benefits of adding Conaid on the compaction characteristics may not be realized.

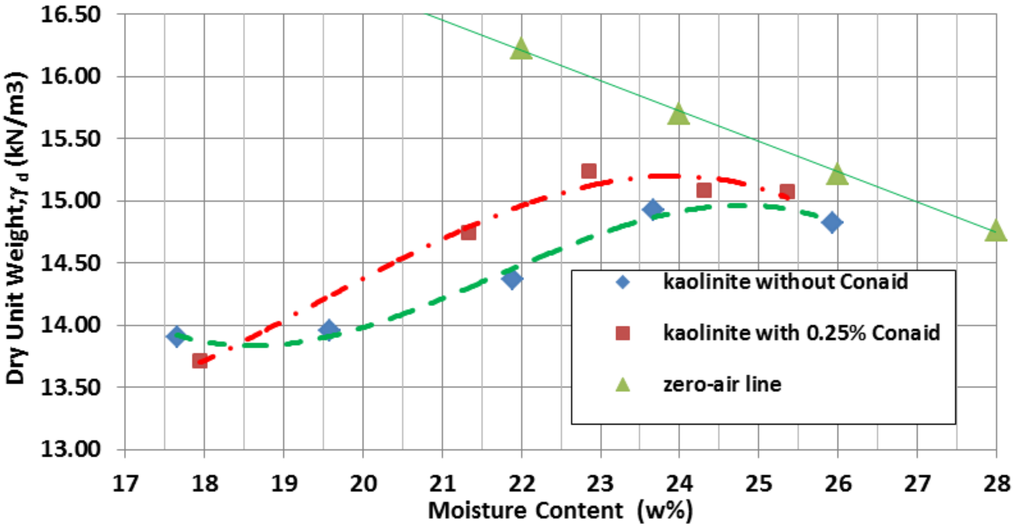

Figure 4.3 Compaction curve of kaolinite passing No. 8 sieve

4.3.2.4 Effects of Conaid content on the MDD of kaolinite

Manufacturer of Conaid suggests 1 kg Conaid concentrate can be used for 25~35 m^2 of roadbed of 20 cm thickness. In other words, the Conaid content is around 0.01% in construction sites. However the optimal Conaid content for laboratory testing is still unknown because compaction effort and sample preparation method in laboratory are different from those in the construction field. The recommended Conaid content in the laboratory by the manufacturer is 0.1%. The Conaid contents adopted here are based upon the recommended value.

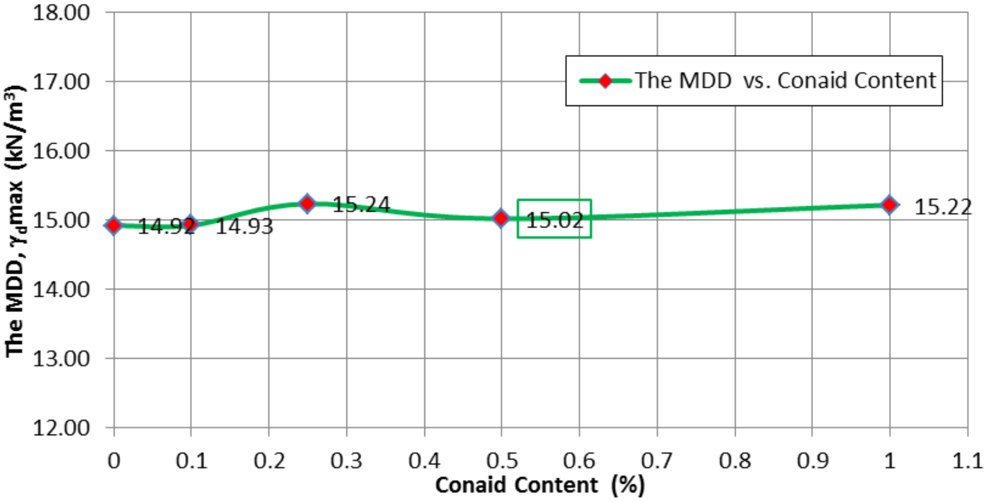

Figure 4.4 Maximum dry unit weight vs. Conaid content

The OMC of the kaolinite being tested is about 23.5%, according to the compaction test conducted. For Conaid contents between 0.1% and 1%, it was found that only 0.25% and 1% of Conaid results in an increase in the maximum dry unit weight by 2%. Other Conaid contents, 0.1% and 0.5%, have insignificant influence on soils being tested (Figure 4.4). However, the OMC of 23.5% was obtained from the compaction test for 0.25% Conaid content. Strictly speaking, compaction test for each Conaid content should be conducted individually and then the corresponding OMC can be determined accordingly. The Figure 4.4 shows that 0.25% Conaid may be the optimal Conaid content for kaolinite.

4.3.2.5 BK20 compaction curve

The compacted BK20 (a 80% bentonite and 20% kaolinite mixture) has been widely used in landfill as clay liner. The potential effects of Conaid on this mixture should also be examined. Pure bentonite has a CEC value of about 10 times that of the kaolinite. Bentonite is considered to be the ideal material to measure the potential benefits of liquid ionic stabilizers. However, due to its expansive property and insensitivity to the change in the moisture content, the compaction curve produced by the Standard Proctor method is too flat to determine the MDD and OMC. Therefore the BK20, which is composed of 20% bentonite and 80% kaolinite by weight, is expected to show the most significant effect of Conaid compared to all the other soils being tested. Standard Proctor tests were carried out on the BK20 soil treated with and without Conaid.

The addition of 0.25% Conaid results in a decrease on the dry unit weight compacted at a moisture contact both dry and wet of optimum moisture content (Figure 4.5). The compaction curve of treated BK20 is a little sharper than that of the untreated one indicating that the compacted density of treated BK20 is more sensitive to the changes of the molding moist content. However, the MDD and OMC of both treated and untreated soils remain approximately unchanged. Figure 4.5 shows the effect of adding 0.25% Conaid which results in lower dry unit weight when the soil is compacted at dry and wet of the optimum moisture content. Besides, the compaction curve of untreated BK20 is closer to the zero-air-void line than the treated one, indicating that there is not much improvement in the compaction characteristics.

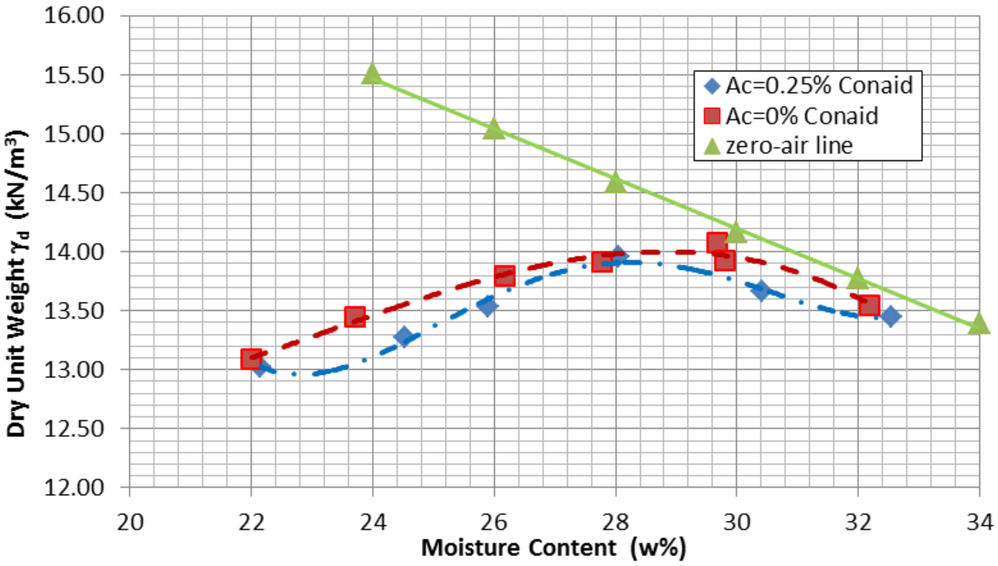

Figure 4.5 Standard Proctor compaction curves for bentonite-kaolinite Mixture

4.3.2.6 Devon silt compaction curve

Devon silt was air-dried in room temperature and then pulverized to pass through the No. 10 sieve (2.0mm) prior to conducting the compaction test. Standard Proctor tests were carried out on the treated and untreated Devon silt. Devon silt was completely mixed with 0.1% Conaid. The mixed wet soil was sealed and left in the moisture room for at least 12 hours to stand (cure) prior to conducting the Standard Proctor test.

The compaction curve for the untreated Devon silt is a typical compaction curve which lies right below the zero-air line. There is a decrease of the dry unit weight for the Devon silt treated with 0.1% Conaid compacted either at dry or wet of optimum moisture content. However, the addition of 0.1 % Conaid results in only minor decrease in dry unit weight.

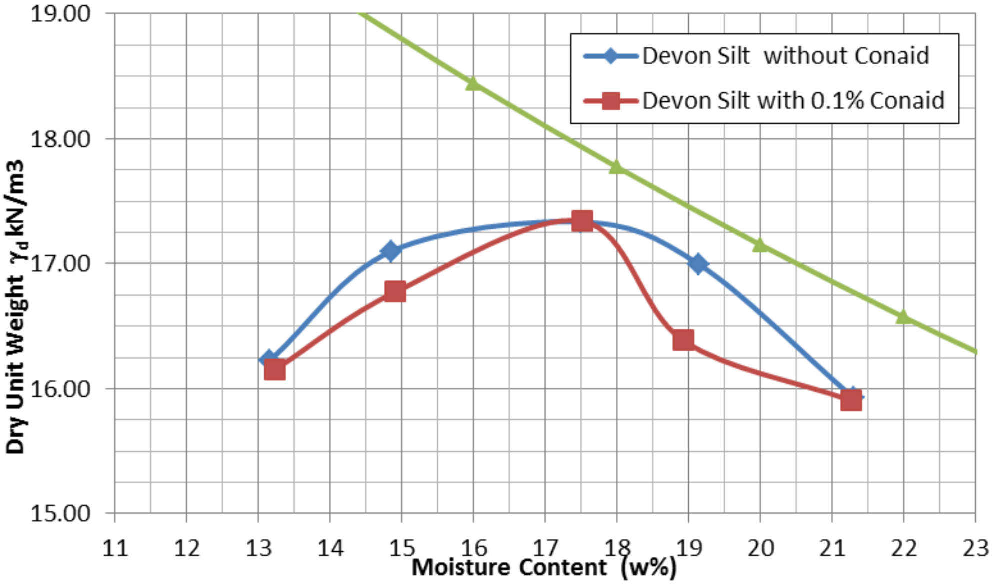

Figure 4.6 Standard Proctor compaction curves for Devon silt

In theory, non-cohesive fine particle soils, such as silt, are less likely to be benefit from liquid ionic stabilizer. According to the grain size distribution curve (Soil Mechanics Laboratory, University of Alberta), there is about 17% of clay fraction in the Devon silt. As a result, the marginal changes of compaction characteristics are due to the clay fraction in the Devon silt treated with Conaid. Conaid has no effects on silt and other non-cohesive materials in the Devon silt.

4.3.3 Modified Proctor test

4.3.3.1 Bentonite compaction curve

Bentonite has been widely used in compacted clay liners required in landfills due to its very low hydraulic conductivity which is in the range of 10^{-11} to 10^{-12} m/s (Karunaratne et al, 2001). With the highest CEC, in the range of 80 to 150 meq/100 gm, among major clay minerals, theoretically speaking, bentonite, which is composed of mainly sodium montmorillonite, is the most susceptible to react with the sulfonated oil molecules in Conaid. As it has been discussed before, the compaction curve produced by the Standard Proctor test is too flat to determine the OMC and MDD. Hence, Modified Proctor tests were conducted in bentonite. The MDD of treated bentonite will be compared with the control samples. With nearly 5 times the compaction effort provided in the Modified Proctor test compared with the Standard Proctor test, the Modified Proctor test may produce significant changes on the compaction characteristics of Conaid-treated bentonite. Considering the large specific surface of montmorillonite (50~120 m^2/gm), water is strongly being held to the clay particle surfaces. The thickness of the diffuse double layer could be very thick and about 20 times that of montmorillonite crystal itself (Lambe, 1958). Therefore, there are more tendencies for Conaid to reduce the thickness of double layer.

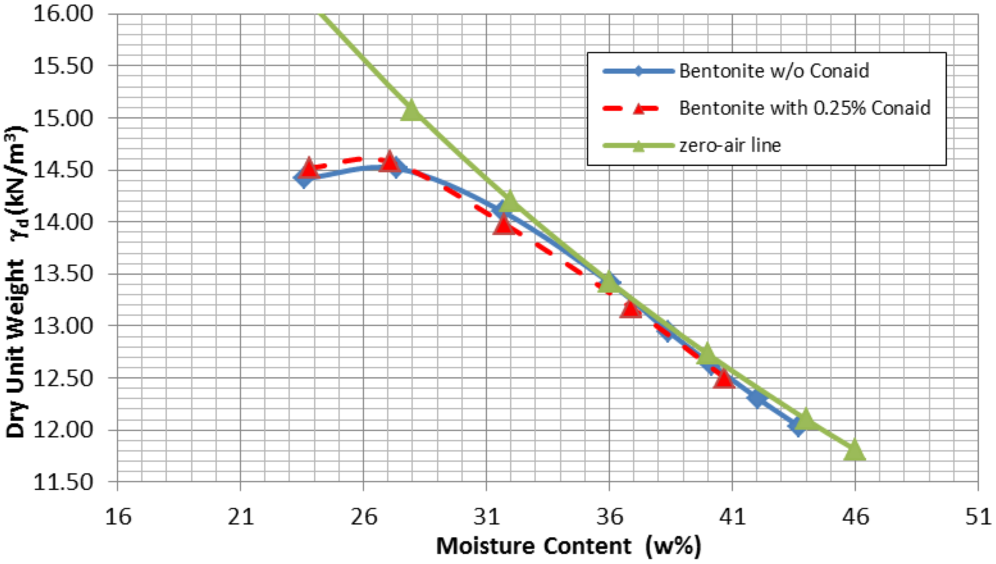

Figure 4.7 Modified Proctor compaction curves for bentonite

However, Figure 4.7 shows no improvement in the dry unit weight of the Conaid-treated bentonite. On the dry side of optimum, bentonite with 0.25% Conaid has a very minor increase in dry unit weight whereas on the wet of optimum, only a negligible decrease in dry unit weight is observed. Both treated and untreated bentonite are very close to zero-air line, which means no more efforts can be made to reach higher dry unit weight.

4.3.4 Discussion

According to Lambe (1958), additives can be classified as dispersants and aggregants. For a compacted soil, dispersants reduce the net interparticle attractive forces (increase repulsion) and therefore reduce the OMC and increase the MDD by improving particles orientation. However, aggregants do the reverse of the dispersants by reducing repulsion and increasing flocculation. Therefore, for a given compaction effort and molding water content, it is the dispersant rather than that aggregant that should improve the density of a compacted soil. Also, for a given compaction condition, the dispersive soil structure can be more easily compacted than the flocculated structure because there is more gain in the shear strength of the material due to the reduction in particle spacing than the reduction in strength due to the dispersion. However, it is not known whether Conaid belongs to be dispersant or aggregant. The results from the tests conducted here show that all the treated soils, except kaolinite, fail to achieve higher unit weight than the untreated soils.

4.4 References

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5. Shear strength of untreated and Conaid-treated soils

5.1. General

To quantify soil strength of compacted soil specimens, consolidated drained triaxial compression test (CD test) and unconfined compression strength test (UC test, ASTM D2166) were performed. The UC tests were conducted on air-dried, moisture room-cured and as-molded specimens. According to the supplier of Conaid, air-dried specimens cured in room temperature are expected to show more significant effects of Conaid than those as-molded or moisture room-cured ones. CD tests were only performed on treated and untreated kaolinite because specimens compacted with soils other than kaolinite usually takes too many days to be saturated under regular back pressure.

In the CD tests, three Conaid Contents, i.e. 0.25%, 0.5% and 1%, were applied in stabilizing kaolinite compacted at the OMC. For the UC tests, more Conaid contents, such as 0.01%, 0.05% and 0.025%, were applied in the treated soils. Both CD and UC tests were also conducted on control samples without the addition of Conaid. Results will be compared between untreated and treated samples.

For all the shear strength tests, graphs of deviator stress were plotted against axial strain. Also, plots were compared to understand the independent effects of Conaid content, moisture content and confining pressure. Peak conditions as well as the following fully-softened conditions were also considered.

5.2. Consolidated drained triaxial tests results

The results of CD tests are discussed as follows. The results of these tests can be found in Appendix A.

5.2.1 Deviator stress and stress ratio

The peak deviator stress, q and stress ratio, $\eta = q/p'$, for each CD test is plotted against various Conaid contents (Figure 5.1, Figure 5.2). Fully-softened values are also plotted (Figure 5.3, Figure 5.4). For all the CD tests performed on kaolinite, the maximum peak deviator stress is 249.5 kPa; this is achieved when untreated kaolinite is compacted at 24% moisture content with a confining pressure of 150 kPa.

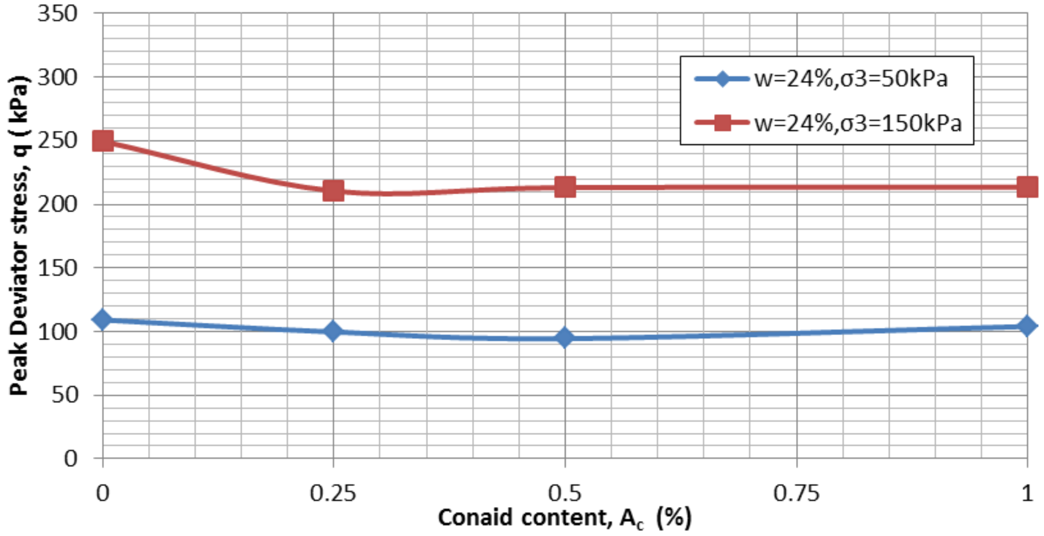


Figure 5.1 Peak deviator stress vs. Conaid content for CD tests at confining pressures of 50kPa and 150kPa

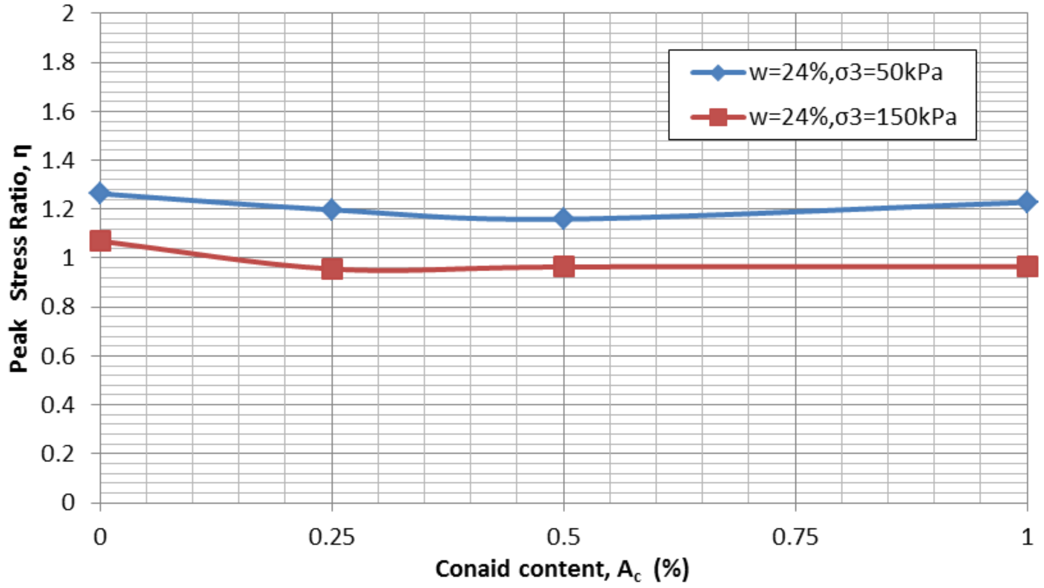


Figure 5.2 Peak stress ratio vs. Conaid content for CD tests at confining pressures of 50kPa and 150kPa

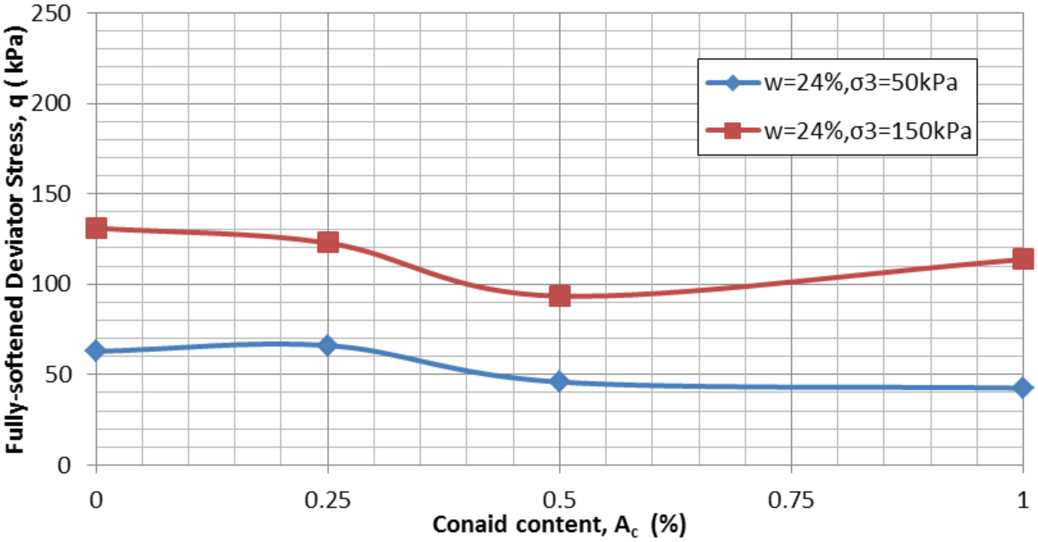


Figure 5.3 Fully-softened deviator stress vs. Conaid content for CD tests at confining pressures of 50kPa and 150kPa

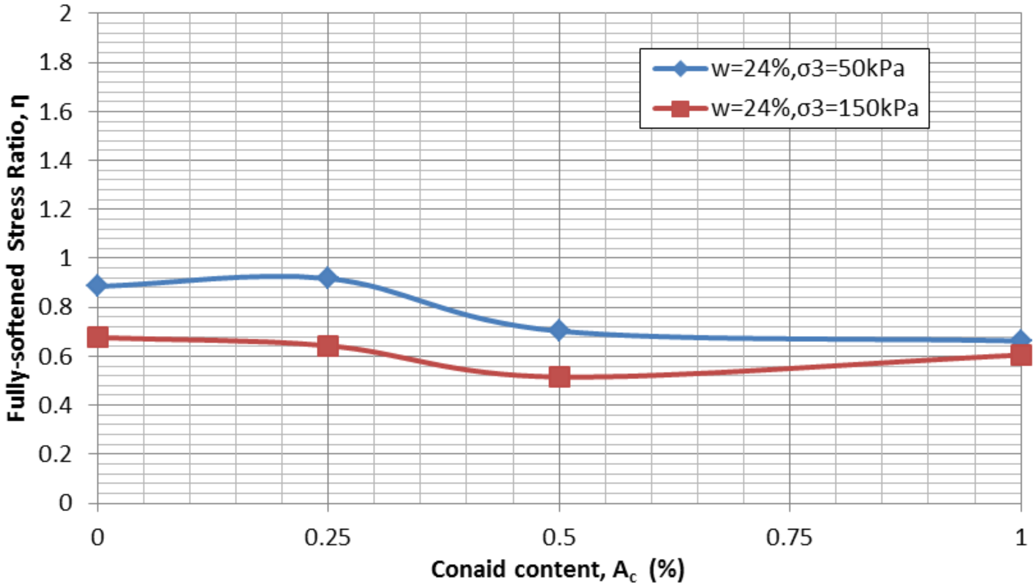


Figure 5.4 Fully-softened deviator stress ratio vs. Conaid content for CD tests at confining pressures of 50kPa and 150kPa

5.2.1.1 Effect of Conaid content

Unexpectedly, an increase in Conaid content from 0% to 0.25% causes a decrease in peak deviator stress and peak stress ratio (Figure 5.1, Figure 5.2). However, Figure 4.4 indicates a minor increase in dry density in the same range of Conaid

Content. Even though both the increase in dry density and decrease in peak deviator stress (ratio) are marginal, those changes should more or less indicate some mechanism behind this phenomenon.

At a confining pressure of 150 kPa, with Conaid contents between 0% and 0.5%, the fully-softened stress of kaolinite decreases significantly from 130 kPa to 93 kPa. At a confining pressure of 50 kPa, the fully-softened stress reaches the maximum value of 66 kPa while Conaid content is 0.25% and then continues to decrease to 42.5 kPa until 1% of Conaid content (Figure 5.3, Figure 5.4). Based on the limited test results, it is postulated that Conaid may cause more dispersion in the kaolinite particle structure, which could increase the compacted dry density whereas decrease the shear strength (Lambe, 1958).

5.2.1.2 Effect of confining pressure

If confining pressure is increased, it is found that the peak deviator stress increased and peak stress ratio decreases for all cases considered herein. The same trend is observed in the fully-softened values (Figure 5.1~Figure 5.4). As for the fissured, compacted soils, the confining pressure keeps the specimen intact under load (Rauch, 2002). For more reliable test results, higher confining pressures are suggested for compacted specimens in triaxial compression test.

5.2.2 Axial strain at peak condition

The axial strain corresponding to peak conditions for the CD tests ranges between 6% and 9% approximately. Note that there is an initial local softened stage appearing for kaolinite specimens at confining pressure of 150 kPa when the axial strain is between 0.5% and 1.5% (Figure 5.5). It may be caused by the closure of some fissures existing in specimens under high confining pressure. The local softened stages at the beginning of shearing cannot be found at confining pressure of 50 kPa (Figure 5.6).

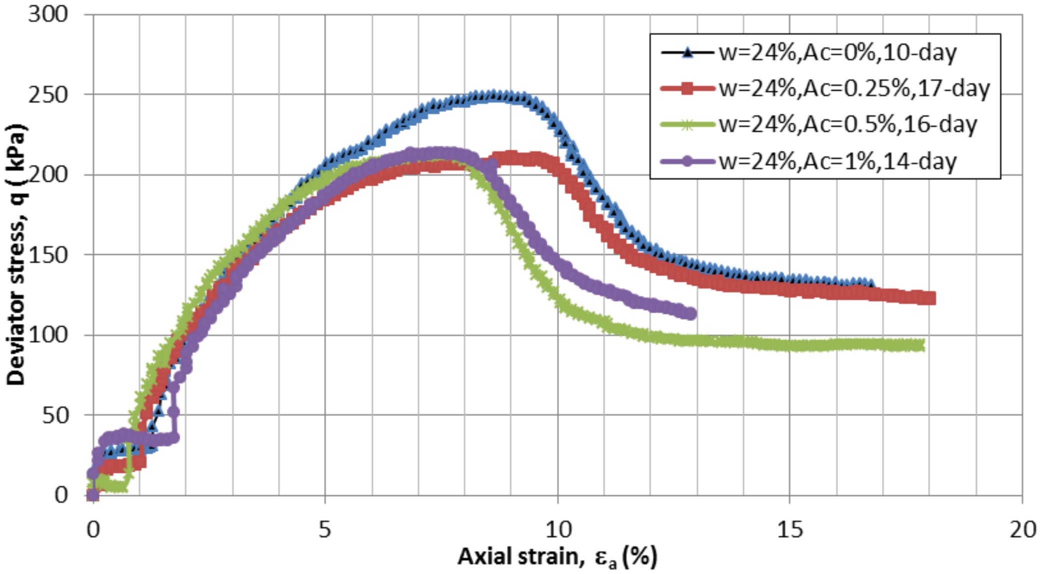


Figure 5.5 q vs. ε_a for w=24%, p_0'=150kPa

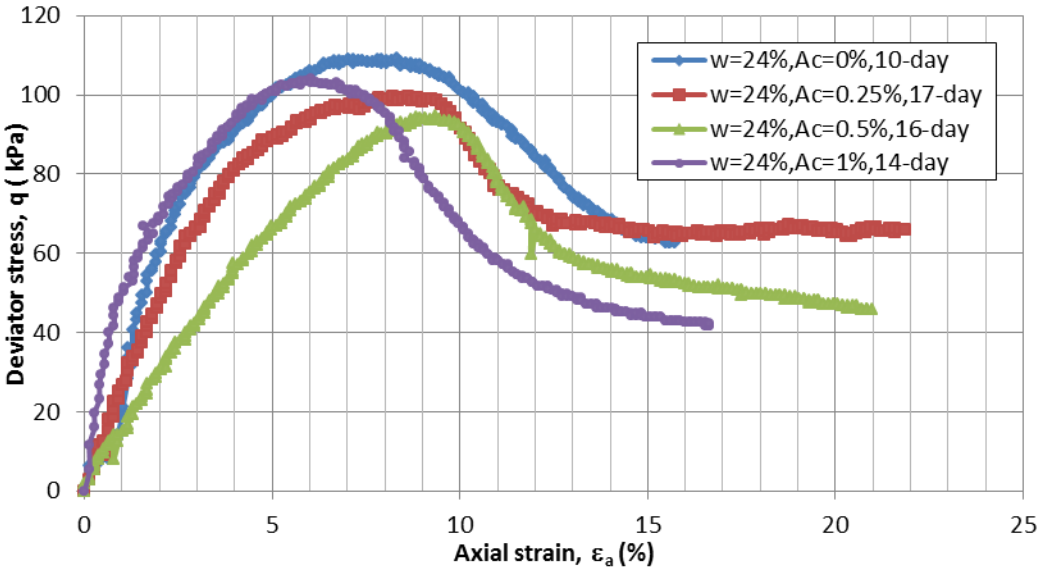


Figure 5.6 q vs. ε_a for w=24%, p_0'=50kPa

5.2.2.1 Effect of Conaid content

At 150 kPa confining pressure, the axial strain corresponding to the peak conditions decreases with an increase in Conaid content. However, the opposite is true at 50 kPa confining pressure for Conaid between 0% and 0.5% (Figure 5.5, Figure 5.6). The first observation in Figure 5.5 is an indication of almost no change in the material's stiffness with an increase of Conaid contents from 0% to

1%. However, at lower confining pressure, the material's stiffness decreased with the Conaid contents varying between 0.5% and 1%. Beyond Conaid content of 0.5%, the stiffness of kaolinite specimen increased. Maybe too much Conaid caused a more dispersed structure in kaolinite, which caused treated material more rigid but weakened.

5.2.2.2 Effect of confining pressure

For Conaid contents between 0% and 1%, an increase in confining pressure resulted in a decrease in stiffness except for the case of 0.5% Conaid. The axial strain corresponding to peak conditions at 50 kPa confining pressure is about 1% more than that at 150 kPa confining pressure. Specimen became more plastic under high confining pressure.

5.2.3 Volume change during drained shear

At the beginning of the drained triaxial tests on treated and untreated kaolinite specimens, the volume change generally indicates contractive behavior and then becomes dilative prior to reaching the peak stress conditions. While Conaid content is 0.25%, dilation continued until the volume change is negative. Dilation at the early stage of shearing indicates that both the untreated and treated kaolinite behaved as over-consolidated soil. The volume change curves show contractive behavior, followed by dilative behavior before peak stress conditions are achieved, at about half of axial strains at peak stress conditions.

5.2.3.1 Effect of Conaid content

Based on the test results, it is observed that volume change is associated with various Conaid contents under the same moisture content. The addition of Conaid had some significant effects on the volume changes of kaolinite specimen.

At a confining pressure of 150 kPa, the volume change decreases with the amount of Conaid (Figure 5.7). Furthermore, the addition of 0.25% Conaid in compacted kaolinite causes a lower volume change than 0.5% Conaid but there is a negative volume change found at the final stage of shearing. Both 0.25% and 0.5% Conaid contents have some impacts on the volume change at the stages of shearing. To understand this phenomenon, the volume change during consolidation should be understood.

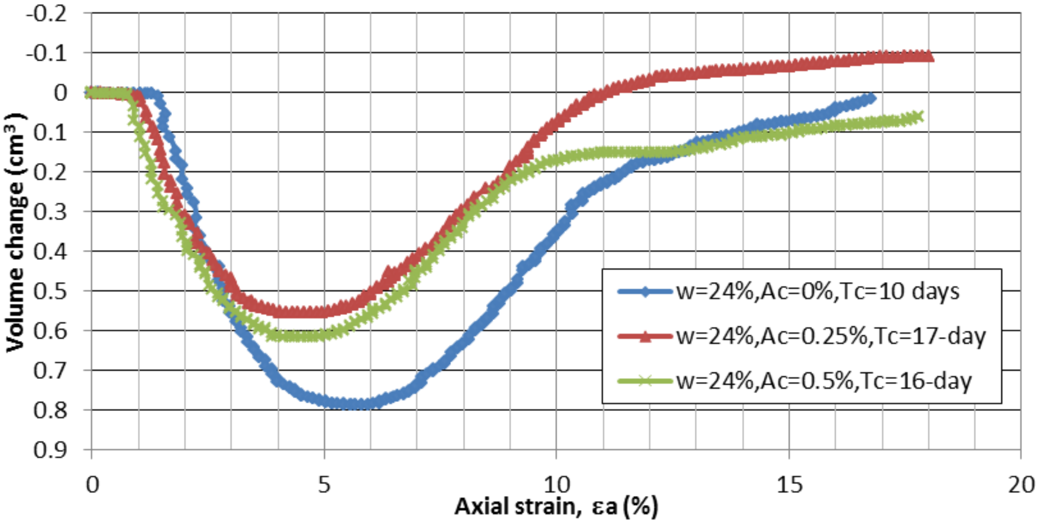


Figure 5.7 Volume change vs. ε_a for w=24% & p_0'=150kPa

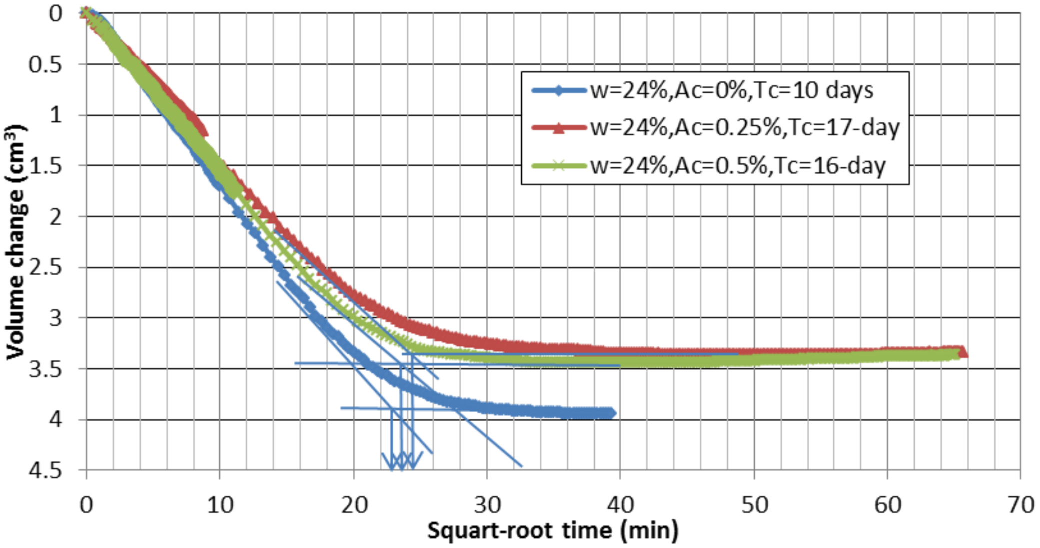


Figure 5.8 Volume change vs. time for w=24% & p_0'=150kPa

Figure 5.8 shows that there was a minimum volume change during consolidation when the amount of Conaid is equal to 0.25% which is closely followed by 0.5% Conaid. The untreated kaolinite has the largest volume change during consolidation among all the specimens. The time needed to reach stabilization during consolidation for each specimen are very close to each other indicating that Conaid does not change the permeability of compacted kaolinite significantly.

The effects of Conaid on volume change during the consolidation and shearing stages can be assumed to be in a dispersed structure in treated kaolinite, whereas untreated kaolinite has a more flocculated structure. A dispersed soil structure is not only hydrophobic but also more densely compacted than a flocculated one (Lambe, 1958). As a result, less water is imbibed during saturation stage in the CD test. If the benefits on volume change can be verified in field test, the application of Conaid in highway construction could be expected to reduce the damage caused by soil swelling in a wetting and drying cycle.

5.2.3.2 Effect of confining pressure

Confining pressure seems to have more significant influence on volume change than Conaid content (Figure 5.9). As the confining pressure is increased, volume changes due to shearing decrease accordingly. At 50 kPa confining pressure, specimen treated with 0.5% Conaid tends to dilate at the final stage of shearing. There is a greater contraction at failure for the same Conaid-kaolinite sample with 150kPa confining pressure. Low confining pressure increases the tendency for compacted samples to dilate following peak stress conditions (Figure 5.9).

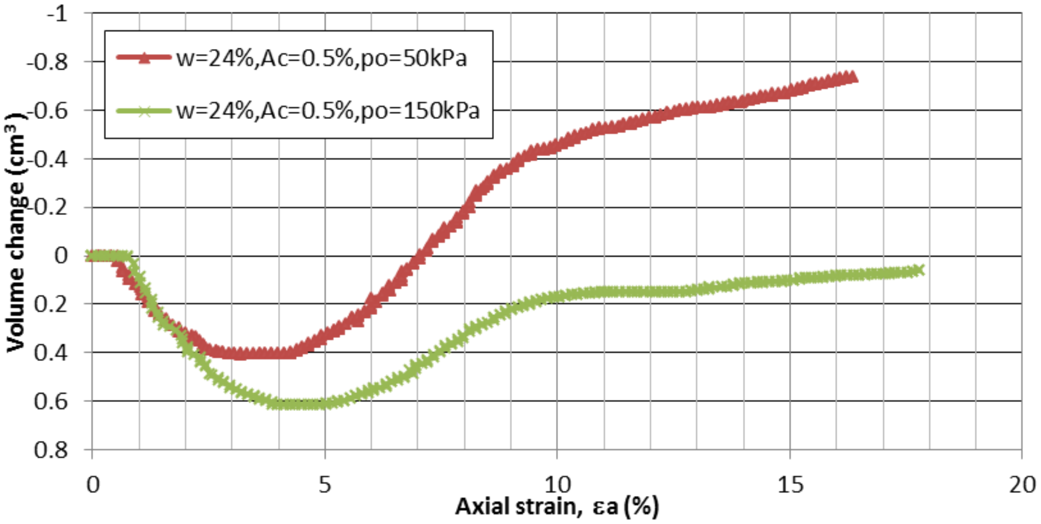


Figure 5.9 Volume change vs. ε_a for w=24% & p_0'=50kPa, 150kPa

5.2.4 Cohesion and friction angle

Because the compacted kaolinite samples were all saturated prior to consolidation and shearing, the Mohr-Coulomb failure criterion should be applicable to the

Conaid-treated compacted clay in the CD test. Based on the range of confining pressures applied in the current study (i.e. 50 kPa, 150 kPa), a linear failure envelope along with a constant cohesion and friction angle is thought to be appropriate for drained test results. In such case, cohesion and friction angle affect each other, and the friction angle is lower while cohesion is higher.

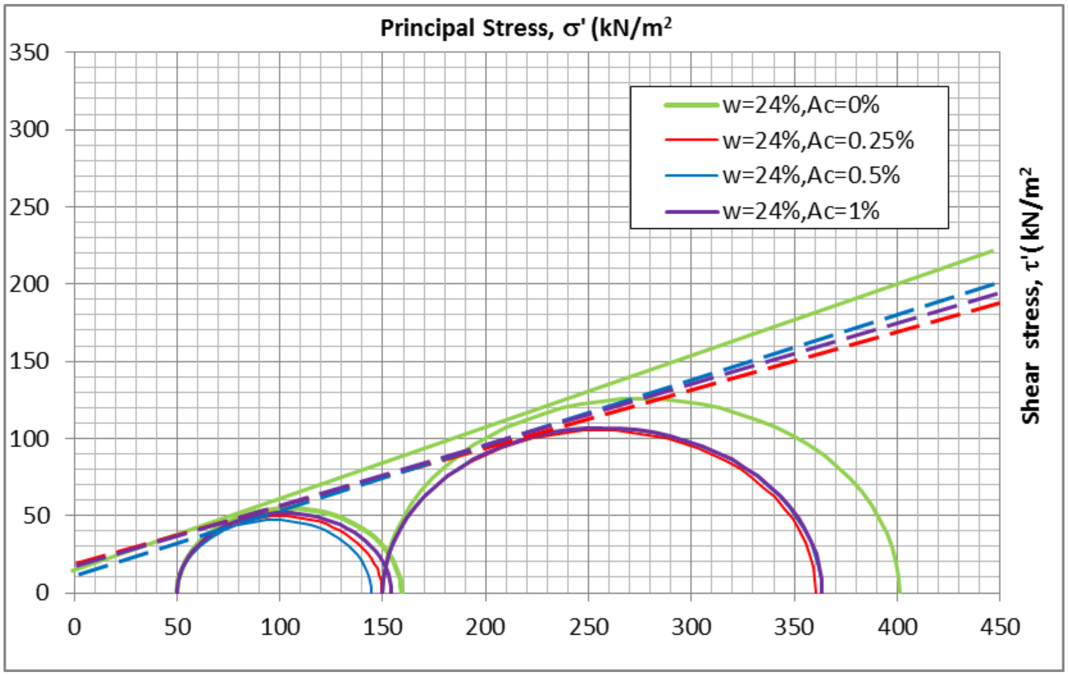


Figure 5.10 Peak envelope of Mohr circles for w=24% & p_0'=50 kPa, 150 kPa

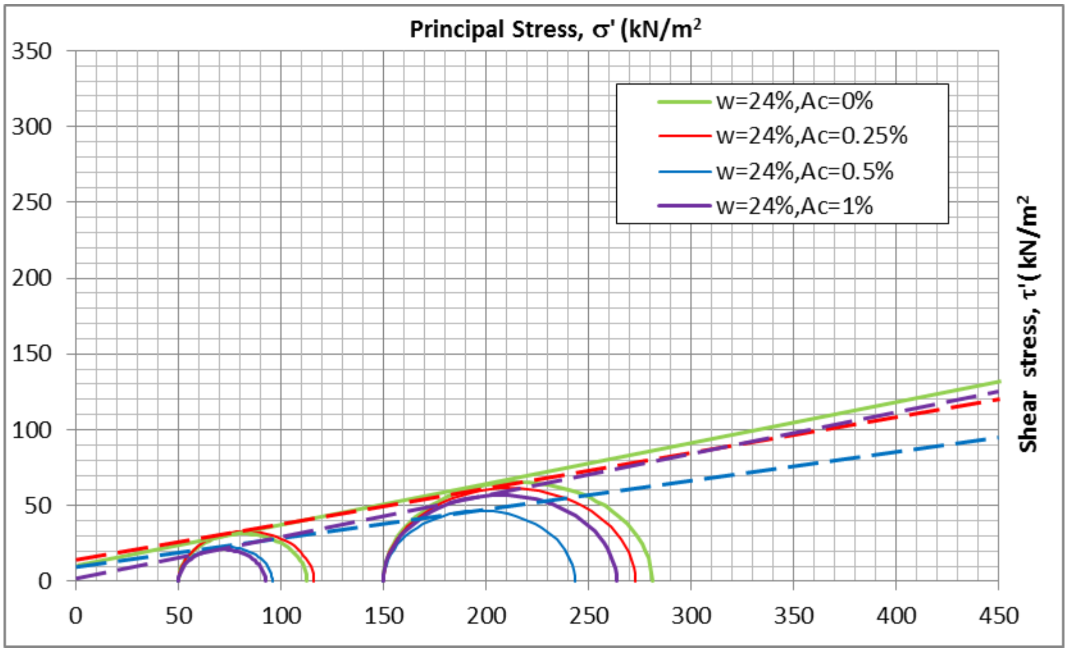


Figure 5.11 Fully-softened envelope of Mohr circles for w=24% & p_0'=50kPa, 150kPa

The cohesion and friction angle (peak and fully-softened values) for different Conaid contents were measured from the drained effective stress failure envelope in the Mohr's circle plots (Figure 5.10, Figure 5.11).

5.2.4.1 Effect of Conaid on Strength Parameters

It is found that Conaid has some effects on friction angle and cohesion of compacted kaolinite. In the current study, an increase in Conaid to 0.25% results in a maximum increase in peak cohesion followed by a sharp decrease between Conaid of 0.25% and 0.5%. Beyond 0.5% Conaid, there is a slight increase in peak cohesion up to 1% Conaid (Figure 5.12).

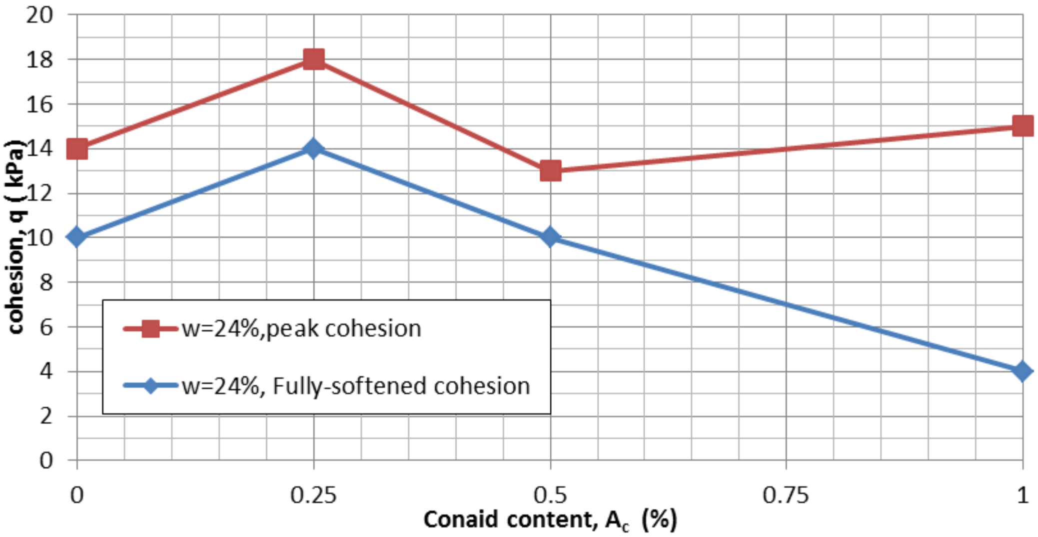


Figure 5.12 Effective cohesions vs. Conaid content

An increase in the peak cohesion is associated with a corresponding decrease in the peak friction angle (Figure 5.13). The fully-softened cohesion increases sharply initially and then decreases beyond 0.25% Conaid. The test results suggested that cohesion comes from compacted clay particles and it not entirely destroyed at the residual condition. There is no clear trend in the fully-softened friction angles with Conaid. Between Conaid of 0% and 0.25%, the fully-softened friction angles is almost unchanged. After 0.25% of Conaid, the fully-softened friction angles decreases slightly by 2 degree and then increases back 10 degree.

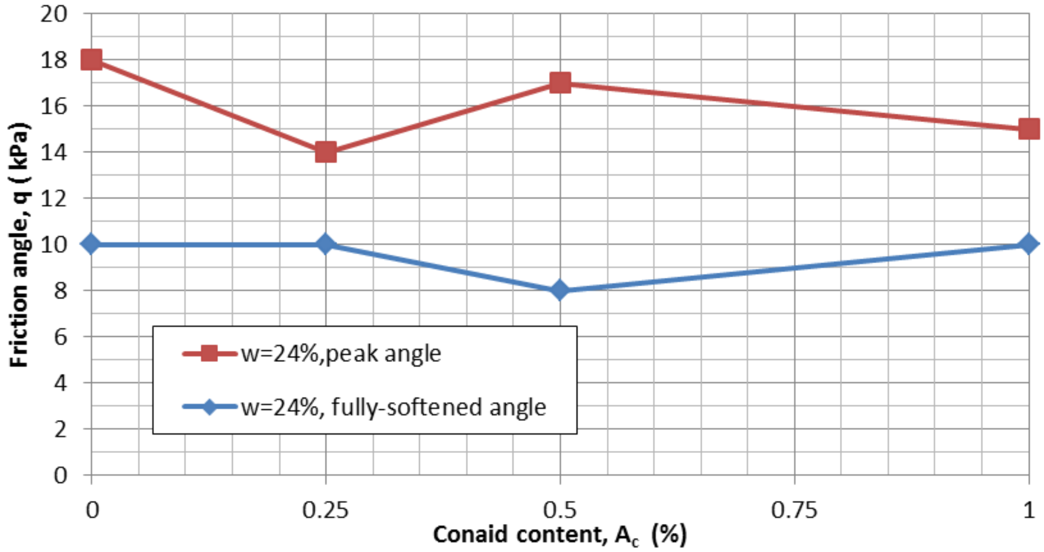


Figure 5.13 Effective friction angles vs. Conaid content

These observations lead to the assumption that peak cohesion and peak friction may be related. Cohesion is not completely destroyed at residual condition, the fully-softened friction angle is nearly constant for all Conaid content values.

The slight decrease in friction angle with the addition of 0.25% Conaid could be caused by a dispersed structure in treated sample. Also, a minor increase in cohesion from 10 kPa to 14 kPa is probably derived from more densely compacted soil structure due to the dispersed soil structure. Soil particles can be better compacted in a dispersed structure than flocculated one with the same compaction effort (Lambe, 1958). The approximate trend of decrease in both friction angle and cohesion following 0.25% Conaid implied that extra Conaid may deteriorate soil strength because the density improvement by compaction could be offset by strength reduction due to the dispersed structure. It has been proven that dispersed soil can be compacted easier than flocculated soil with lower shear strength.

5.2.5 Failure behavior

Photographs of three specimens after the CD test were taken as shown in Figure 5.14. Figure 5.14a shows that an untreated sample failed without significant barreling and a failure plane about 25 ° was formed during slow shearing.

However, samples treated with 0.25% and 0.5% Conaid experienced a little more barreling than the untreated one. Also, Figure 5.5 shows that an untreated specimen has a higher shear strength and axial strain at peak strength than a treated one. In other words, kaolinite was softened by Conaid and became a little more rigid.

(a) (b) (d)

Figure 5.14: Photos of specimens after CD test: (a) A_c=0%, w=24%, p_0'=150kPa; (b) A_c=0.25%, w=24%, p_0'=150kPa; (c) A_c=0.5%, w=24%, p_0'=150kPa.

5.3. Unconfined compression strength test results

The unconfined compression (UC) strength test (ASTM D2166) is widely accepted by engineers to determine the shear strength of highway subgrade or pavement materials. Errors resulted from sampling disturbance, curing condition and fissures cannot be completely eliminated, which range between 40% and 85% of the true in-situ shear strength. In spite of such a wide range of experimental errors, the UC test is a practical method to measure the strength of soils due to its simplicity.

For the current laboratory program, the treated and untreated samples prepared with different moisture contents were either air-dried or cured in moisture room. Voids and cracks were observed on the surface of air-dried specimens. Test results and photos of each sample are presented in Appendix B~F. In general, the UC tests on two (2) specimens were conducted for each test condition. While interpreting the UC test results, only the maximum value from the same mixture was selected. All the specimens were extruded from the compaction mold and carefully trimmed before being cured. The stress-strain curves of the UC tests are plotted in Appendix B~F.

5.3.1 Unconfined compressive strength of untreated and treated kaolinite

5.3.1.1 Samples compacted with Standard Proctor method

The UC test results indicate that the strength of kaolinite treated with 0.25% Conaid is about 30% lower than that of the untreated kaolinite (Figure 5.15). Also, the axial strain of treated kaolinite at peak conditions is about 2.8%, indicating a slight increase in stiffness compared to the treated kaolinite at 24% moisture content.

A decrease in the UC strength for the treated kaolinite can also associated with the improvement on dry density of compacted kaolinite treated by Conaid (see section 4.3.2.3). In other words, the dispersed structure caused by the addition of 0.25% Conaid could increase the dry density of the compacted soil. It is known that the interparticle attraction and difficulty of displacing particles in a disorderly array in flocculated soil result in a higher strength and lower compressibility than dispersed soil at the same void ratio (Lambe, 1979). If the deteriorating effect on strength by dispersion offsets its benefit on reducing void ratio of compacted soil, the strength of compacted kaolinite treated by Conaid would be lower than the untreated kaolinite.

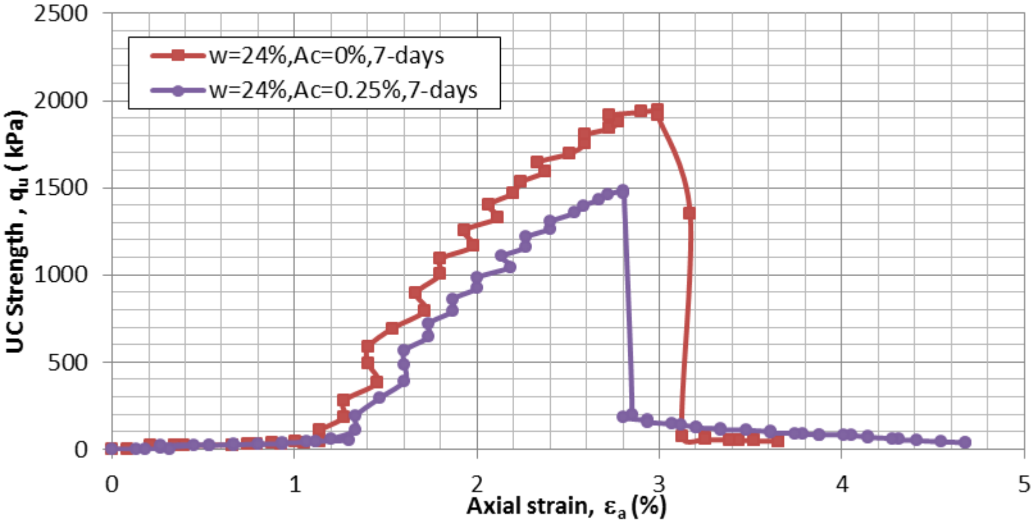


Figure 5.15 UC stress-strain curves for air-dried specimens, T_c=7 days

The opposite is true for untreated kaolinite. Following 7-day air dry, the moisture content in the specimens decreased from 24% to around 2%, showing that moisture in air-dried specimens had almost reached a minimal value and was not a factor in influencing shear strength anymore.

5.3.1.2 Samples compacted with Modified Proctor method

While only considering the maximum strength in each mixture prepared using the Modified Proctor method, the strength of the untreated and treated kaolinite are almost equal. However, the axial strain at peak decreased by more than 1% with Conaid ranging from 0% to 0.05% (Figure 5.16).

The shear strength of untreated and treated specimens is almost the same even though the moisture contents after shearing are a little different.

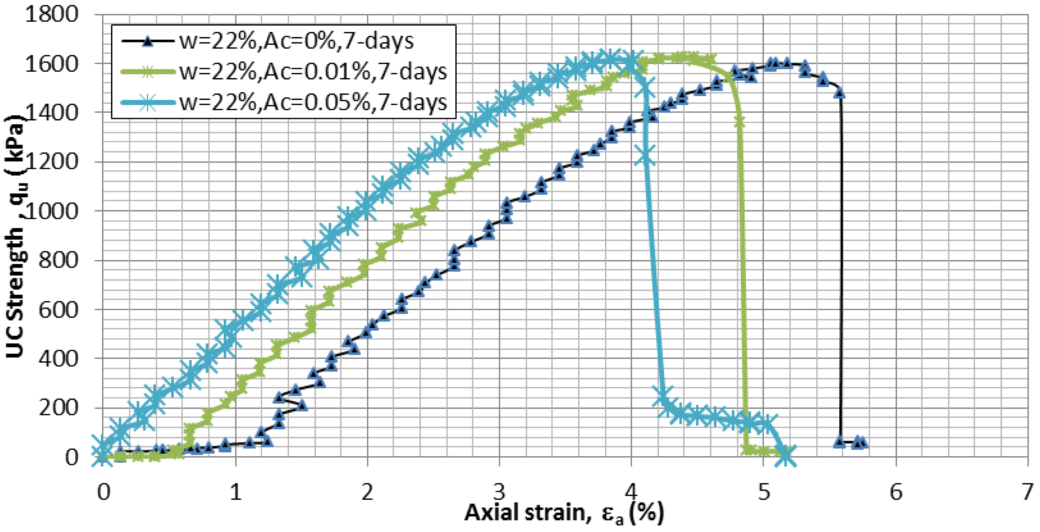


Figure 5.16 UC stress-strain curves for moisture room dry, T_c=7 days

Nevertheless, the strains at peak strength are obviously a function of the amount of Conaid. The compacted kaolinite samples tended to be more rigid after the addition of Conaid (Figure 5.16). Table 5.1 shows only about 5% moisture loss in specimens following 7-day curing in the moisture room. Strain at peak strength is related to the amount of Conaid rather than the moisture content of the specimen. Compared to 0.25% Conaid content, the addition of 0.01% and 0.05% Conaid in water are much more diluted solution. Compared to the kaolinite treated with

0.25% Conaid (5.3.1.1), compacted kaolinite with 0.05% Conaid becomes more rigid than untreated one subjected to the same (Modified) compaction effort. Higher energy from the Modified Proctor compaction could crash almost all the lumps and eliminate most of the voids between soil particles. Theoretically, the effects of Conaid should be more observable for samples compacted using the Modified Proctor method than using the Standard Proctor method.

Table 5.1 Moisture content change after 7-day moisture room-dried

Conaid Content (%)	Moisture Content(before) (%)	Moisture Content(after) (%)
0	22	17.19
0.01	22	17.32
0.05	22	17.56

The results of the UC test are generally affected by various factors, such as specimen quality (fissures or voids), curing condition and compaction effort (Standard or Modified Proctor test), etc. Besides, the optimal Conaid content for kaolinite in compaction test is still unknown, which might range from 0.01% to 0.5%. Besides, the pH value of Conaid solution could influence the kaolinite structure significantly. Also there are experimental errors and variations during the test. In most cases Conaid changed the stiffness of kaolinite at peak strength significantly with no improvement on the shear strength of treated kaolinite. The stiffness of the material is affected by the amount of Conaid, curing condition and compaction effort.

5.3.1.3 Failure behavior

Under unconfined compression condition, kaolinite became a brittle material after being air-dried for 7-days. Crushing happened in the air-dried sample. Also, spalling was noticed while shearing (Figure 5.17a, b). Specimens cured in moisture room were divided into two pieces by a failure plane of about 30 º~40 º.

(a) (b) (c) (d)

Figure 5.17: Photos of specimens after UC test: (a) A_c=0%, w=24%, 7-day air dry; (b) A_c=0.25%, w=24%, 7-day air dry; (c) A_c=0%, w=22%, 7-day moisture room dry; (d) A_c=0.01%, w=22%, 7-day moisture room dry

While samples were dried in moisture room, no spalling or crushing was observed during shearing for both treated and untreated samples. In addition, specimen treated with 0.01% Conaid shows more brittle failure behavior than untreated one (Figure 5.17c, d). The failure behaviors between treated and untreated specimens show little difference.

5.3.2 Unconfined compressive strength of untreated and treated bentonite

According to the Modified Proctor compaction test (4.3.3.1), the OMC for bentonite is around 28%. Therefore, two groups of UC tests were performed; for the first group, bentonite treated with 0%, 0.01%, 0.025% and 0.25% Conaid were compacted at 32% water content. Altogether there were five (5) specimens cured in moisture room for 21 days prior to conducting UC test. As for the second group, bentonite treated with 0% and 0.25% Conaid was compacted at various moisture contents. In total, there were 12 specimens cured in moisture room for 31 days prior to shearing.

5.3.2.1 Effect of Conaid content (w=32%)

At 32% molding water content, the UC strength of bentonite treated with 0.01% Conaid increases by 9.6%. Bentonite treated with 0.025% Conaid only achieves an increased in strength by 2.4%, whereas the addition of 0.25% Conaid causes an abrupt decrease of 384 kPa in strength (Figure 5.18, Figure 5.19). The addition of

0.25% Conaid seems to have softened the bentonite rather than strengthened it.

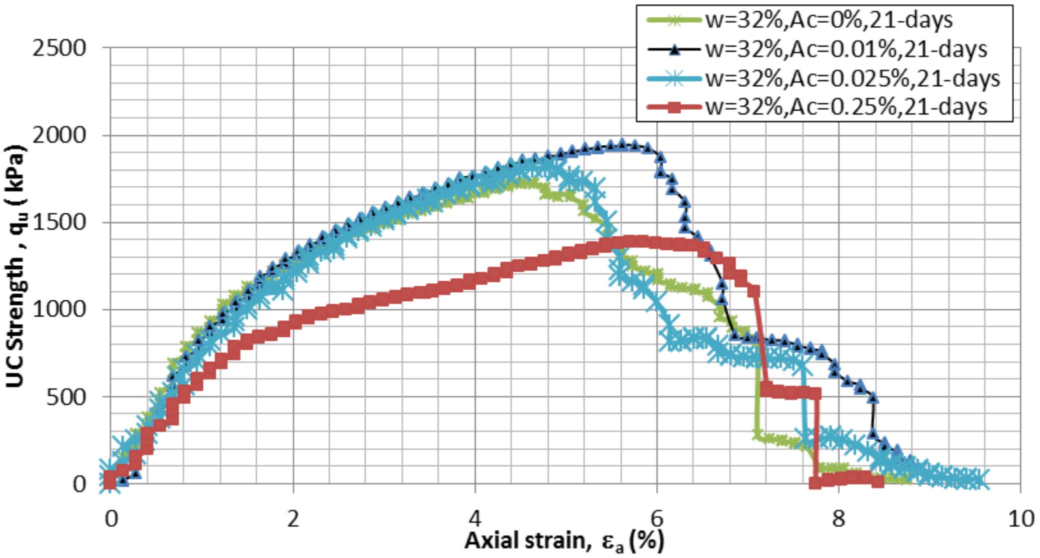


Figure 5.18 UC stress- strain curve for w=32%, T_c=31 days

Specimens treated with 0.25% Conaid reached their peak conditions with an axial strain about 5~7%. For untreated bentonite, the axial strain at peak strength is less than 4.5%. An increase of Conaid content resulted in a reduction in the stiffness of the bentonite, indicating that treated bentonite became less stiff than the untreated one.

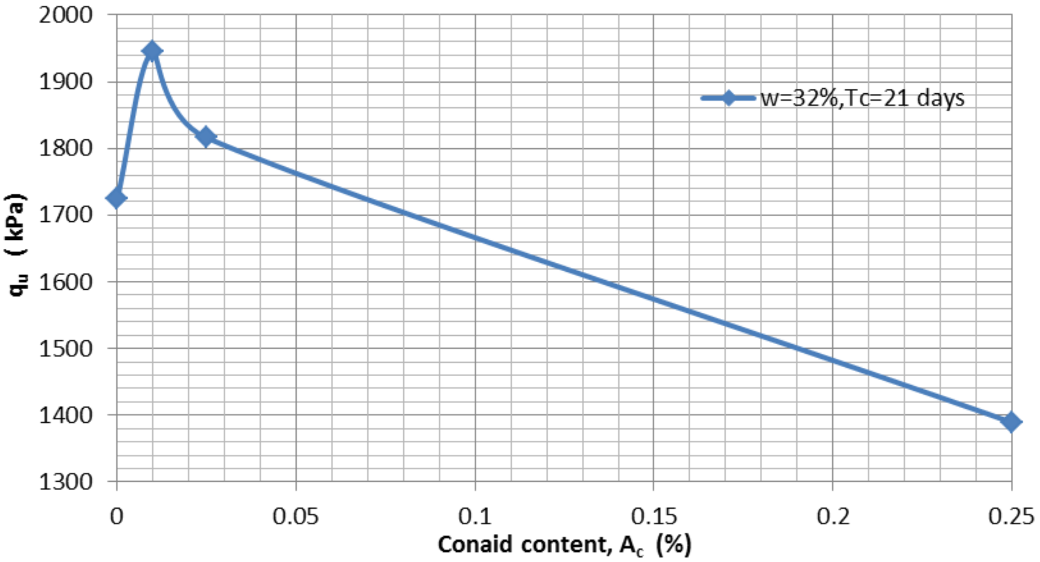


Figure 5.19 UC peak strength vs. Conaid content for w=32%, T_c=21 days

Following 21 days of curing in moisture room, the moisture contents of cured samples were almost the same and no fissures were found on the surface of each specimen, giving evidence that curing in moisture room provided better quality samples.

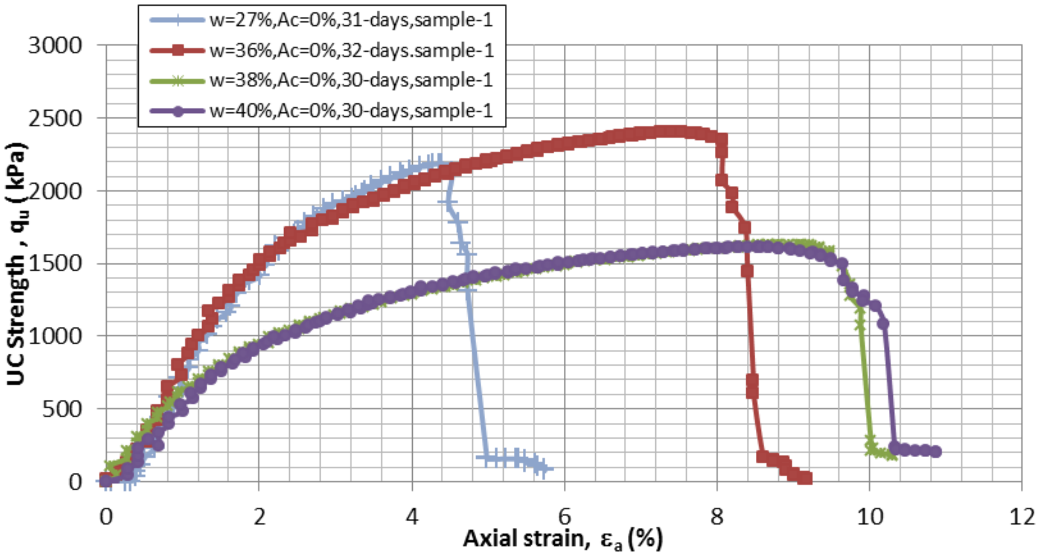


Figure 5.20 UC stress- strain curve for Ac=0%, T_c=30~32 days

5.3.2.2 Effect of moisture content

With increasing moisture contents from 27% to 40%, the UC strength of untreated bentonite decreases significantly by about 33%. As expected, the stiffness is also obviously reduced with the moisture content is changed from 27% to 36%. The axial strain at peak strength with a moisture content of 36% is twice as much as that with a moisture content of 27%, indicating that water can reduce the rigidity of bentonite in a relative short range of moisture content. There is only a 1.5% increase in axial strain between 36% and 40% moisture content, but the corresponding strength decrease is more than 50%. Beyond a moisture content of 38%, the stress-strain curves show little or no differences (Figure 5.20). Strength and stiffness are not sensitive to the change of moisture content somewhere beyond the wet of optimum condition.

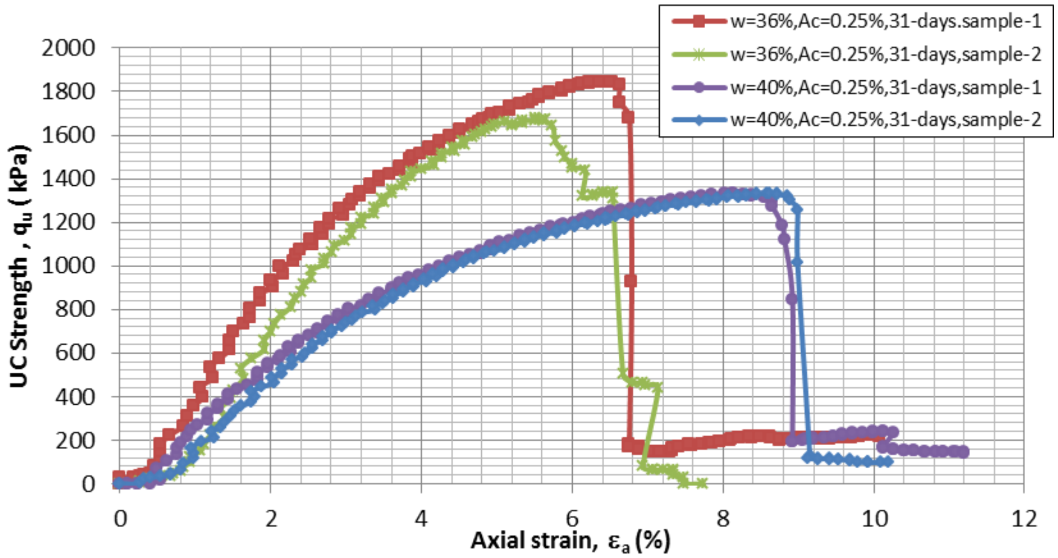


Figure 5.21 UC stress- strain curve for Ac=0.25%, T_c=31 days

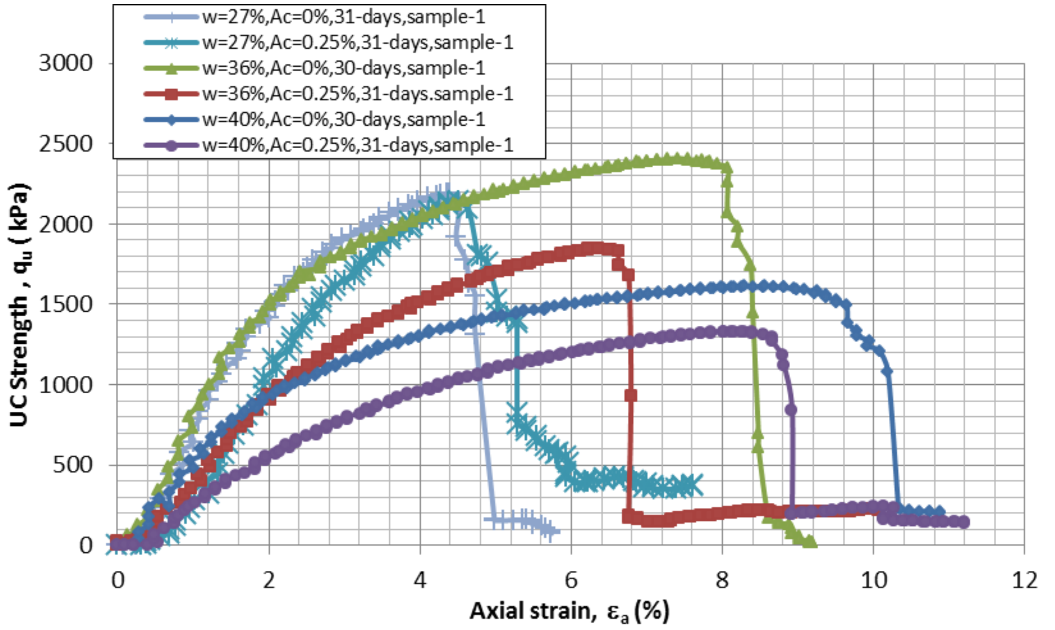


Figure 5.22 UC stress- strain curve for Ac=0 & 0.25%, T_c=31 days

However, bentonite treated with 0.25% Conaid is somewhat less sensitive to the change of moisture content (Figure 5.21). The UC strength of treated bentonite specimens only decreased 38% compared to 50% for the untreated ones in the same range of moisture content changes (from 36% to 40%).

In Figure 5.22, specimens were softened with the addition of 0.25% Conaid and they became stiffer than untreated bentonite. At a moisture content of 27%, the difference in shear strength is very small due to a moisture content at dry of optimum condition. It is hypothesized that at dry of optimum moisture content, diffusion double layer is not fully developed therefore the effect of Conaid on reducing the double layer thickness is negligible. At moisture content wet of optimum, there is enough water available for the bentonite to develop double layers, consequently, Conaid is able to impose more influences on bentonite particles.

The softening effect on the strength can be explained if 0.25% Conaid content is not the optimal Conaid content. It is possible that too much Conaid results in a flocculated structure in bentonite resulting in less densely compacted soil in the mold. The gain in strength due to flocculation is offset by the reduced dry density caused by the flocculated structure in Proctor compaction. Based on limited test results, the optimal Conaid content for bentonite could be as low as 0.01%.

5.3.2.3 Failure behavior

Figure 5.23 shows the failure behaviors for bentonite after 21-day curing in the moisture room. Oblique failure plane about 40 °splits the untreated specimen into two unequal parts while the one treated with 0.25% Conaid is divided into two almost equal parts. Bending was observed in the specimen treated with 0.025% Conaid (Figure 5.23b), which might be caused by the movement of the top pedestal. However, crushing happened in the sample treated with 0.25% Conaid. Various failure behaviors can be seen for both treated and untreated specimens.

(a) (b) (c) (d) (e)

Figure 5.23: Photos of specimens after UC test: (a) A_c=0%, w=32%, 21-day moisture room dry; (b) A_c=0.025%, w=32%, 21-day moisture room dry; (c) A_c=0.25%, w=32%, 21-day moisture room dry; (d) A_c=0%, w=27%, 31-day moisture room dry; (e) A_c=0.25%, w=27%, 31-day moisture room dry

5.3.3 Unconfined compressive strength of untreated and treated BK20

To better understand the effects of Conaid on major clay minerals, a mixture of 20:80 bentonite and kaolinite (BK20) ratio was tested in laboratory test. Two groups of UC tests were performed; for the first group, BK20 with 0% and 0.25% Conaid content were compacted at 20% moisture content. Altogether four (4) specimens were cured in moisture room for 7 days prior to shearing. For the second group, BK20 with 0%, 0.025% and 0.25% Conaid content were compacted at 28% moisture content. Six (6) specimens were cured in air for 7 days prior to the UC test.

5.3.3.1 Effect of Conaid content

Similar to kaolinite, the strength of BK20 is decreased by 26% with the addition of 0.25% Conaid. Also, there is a decrease of 1% in axial strain at peak stress for the compacted BK20 specimens treated with 0.25% Conaid. Following 7-day curing in the moisture room, the change of moisture content in the specimens is insignificant compared with those of the air dried samples. The results in Figure 5.24 show that the compacted BK20 is softened by 0.25% Conaid and becomes a little stiffer at the same time.

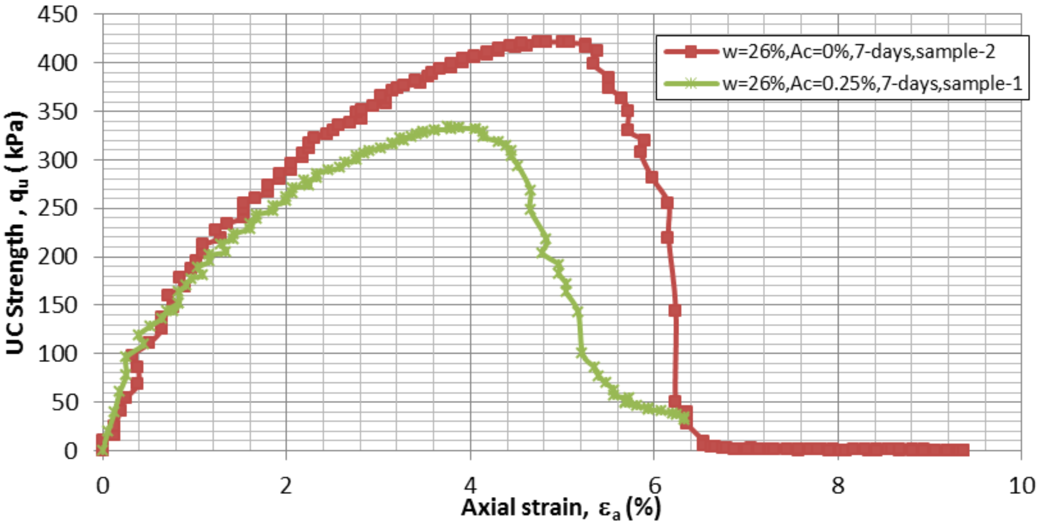


Figure 5.24 UC stress-strain curves for moisture room curing, T_c=7 days

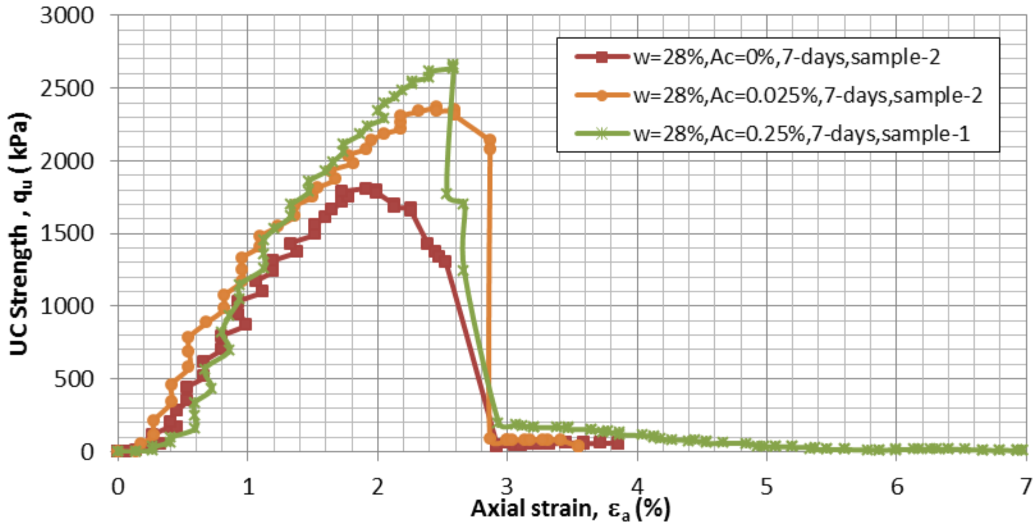
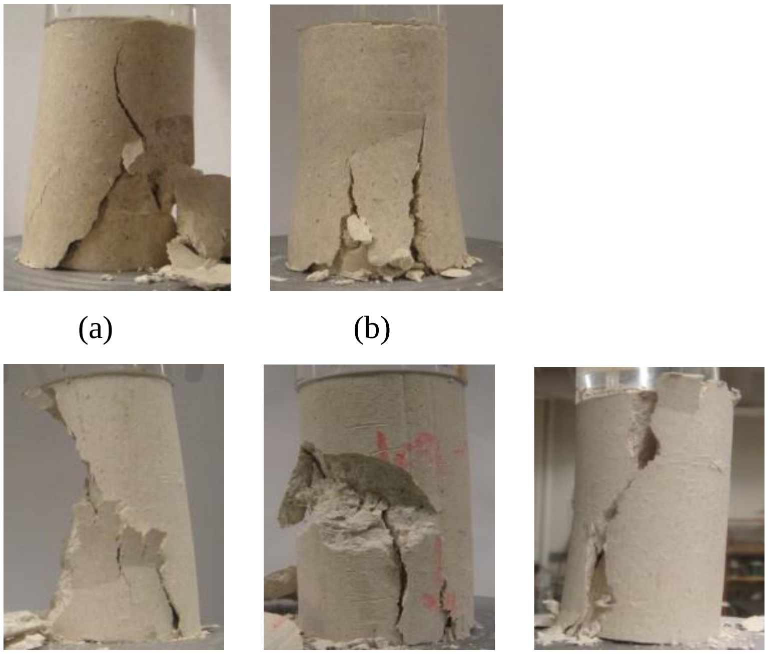


Figure 5.25 UC stress-strain curves for air-dried, T_c=7 days

When BK20 is compacted at the optimal moisture content and air dried, Conaid has some positive effects on the shear strength of the compacted BK20 samples. There is an increase of 550 kPa in strength for BK20 treated with 0.025% Conaid and an increase of 830 kPa for 0.25% Conaid. In this case, the sample was compacted at the optimum moisture content and air-dried for 7 days. Consequently, the moisture content in the specimens decreased from 28% to 3% after 7-day air dry. Besides, the axial strain corresponding to the peak strength increases for 0.025% and 0.25% Conaid indicating that both shear strength and ductility are considerably improved at the same time with the addition of Conaid.

5.3.3.2 Failure behavior

Since there is only 20% of bentonite mixed with kaolinite, the failure behaviors of BK20 are similar to kaolinite. Both crushing and oblique fractures occurs in BK20 samples. Two failure planes develop in the samples as shown in Figure 5.26 (a), whereas the 0.25% Conaid specimen fails by crushing rather than developing a clear shear plane. Even though the positions of shear plane varied in different air dry specimens, failure behaviors are similar to each other and not significantly related to Conaid contents.



(a) (b)

(c) (d) (e)

Figure 5.26: Photos of specimens after UC test: (a) A_c=0%, w=26%, 7-day moisture room dry; (b) A_c=0.25%, w=26%, 7-day moisture room dry; (c) A_c=0%, w=28%, 7-day air dry; (d) A_c=0.025%, w=28%, 7-day air dry; (e) A_c=0.25%, w=28%, 7-day air dry moisture

5.3.4 Unconfined compressive strength of untreated and treated Devon silt

Devon silt untreated and treated with 0.05% Conaid were compacted at optimum moisture content. Altogether three (3) specimens were cured in moisture room for 21 days prior to shearing. Also, Devon silt samples were compacted wet of optimum. Four (4) specimens were cured in moisture room for 12 days before conducting UC test.

5.3.4.1 Effect of Conaid content and moisture content

If Devon silt samples are compacted at optimum moisture content, there is a minor improvement (10.5%) in strength for treated samples compared to the untreated one. The axial strain corresponding to the peak stress decreases by 1.2% with the addition of Conaid, indicating the treated Devon silt becomes a little stiffer (Figure 5.27). When samples are compacted wet of optimum, the strength of the Devon silt sample with 0.05% Conaid is slightly lower (6.5%) than that of untreated one following 21 days curing in the moisture room (Figure 5.28). The axial strains for both untreated and treated Devon silt are almost the same. So 0.05% Conaid in treated samples has a negligible negative effect on the shear strength but not on the stiffness, which corresponds to the compaction curves shown in Figure 4.6.

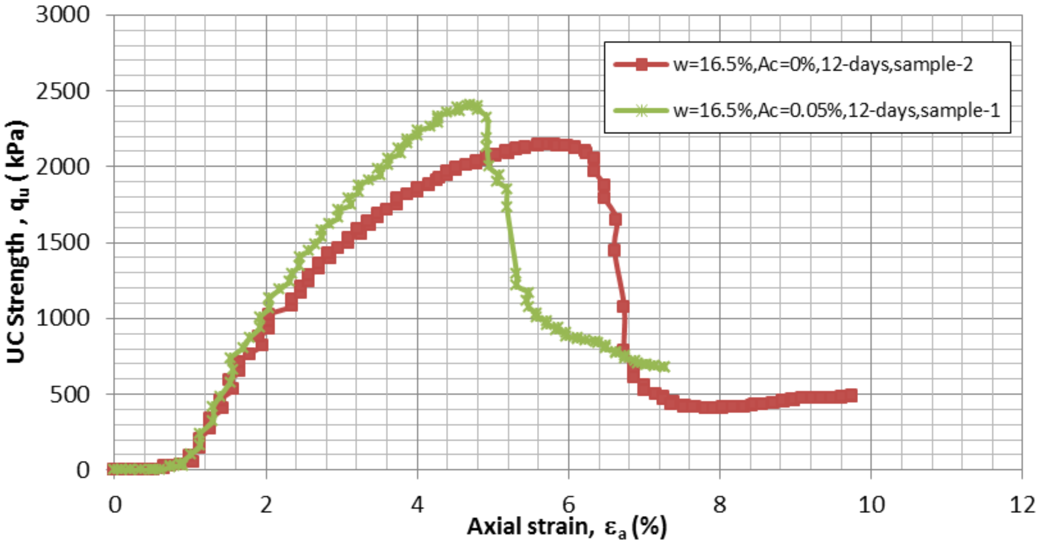


Figure 5.27 UC stress- strain curve for w=16.5%, T_c=12 days

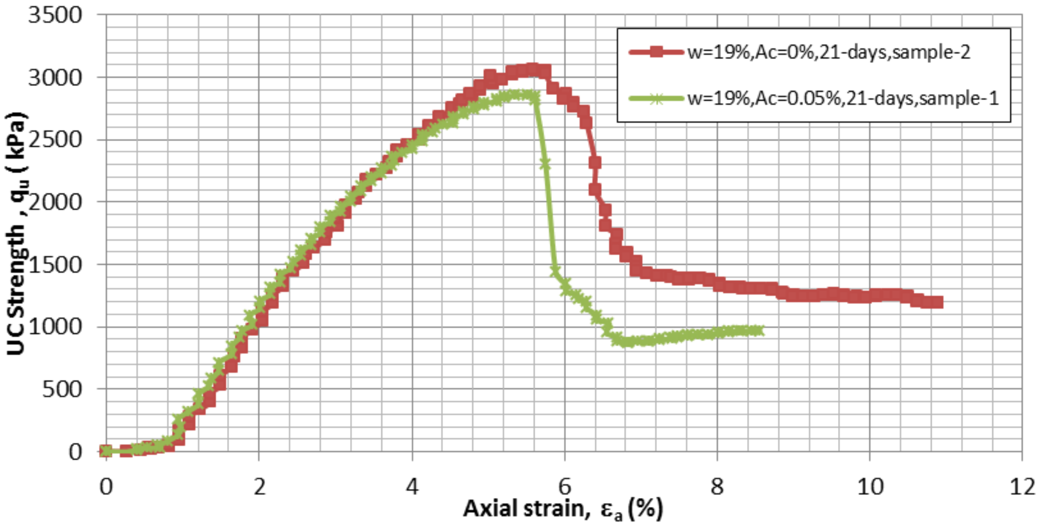


Figure 5.28 UC stress- strain curve for w=19%, T_c=21 days

5.3.4.2 Failure behavior

Oblique fracture mode is common as shown in Figure 5.29. Also, barreling can be seen in Figures 5.29 (a) and (b). A clear failure plane about 30 ° is developed in each specimen along with some spalling. The failure behaviors between untreated and treated specimens are also similar to each other.


(a) (b) (c) (d)

Figure 5.29: Photos of specimens after UC test: (a) A_c=0%, w=16.5%, 12-day moisture room dry; (b) A_c=0.05%, w=16.5%, 12-day moisture room dry; (c) A_c=0%, w=19%, 21-day moisture room dry; (d) A_c=0.05%, w=19%, 21-day moisture room dry.

5.3.5 Unconfined compressive strength of untreated and treated Anthony Henday clay

The OMC for Anthony Henday clay is estimated to be 17% based on the liquid limit and plastic limit of the material (Johnson, 1962). All the samples of the Anthony Henday clay were compacted at 17% moisture content. Altogether two (2) specimens were cured in the moisture room for 5 days before conducting the UC test. Also another two (2) specimens were sheared immediately without being cured.

5.3.5.1 Effect of Conaid content and moisture content

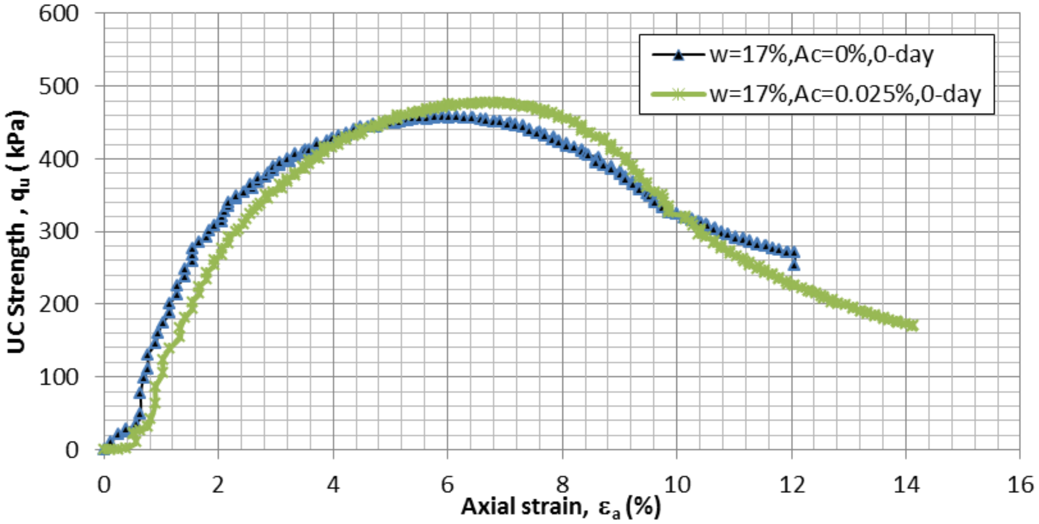


Figure 5.30 UC stress-strain curve for w=17%, T_c=0 day

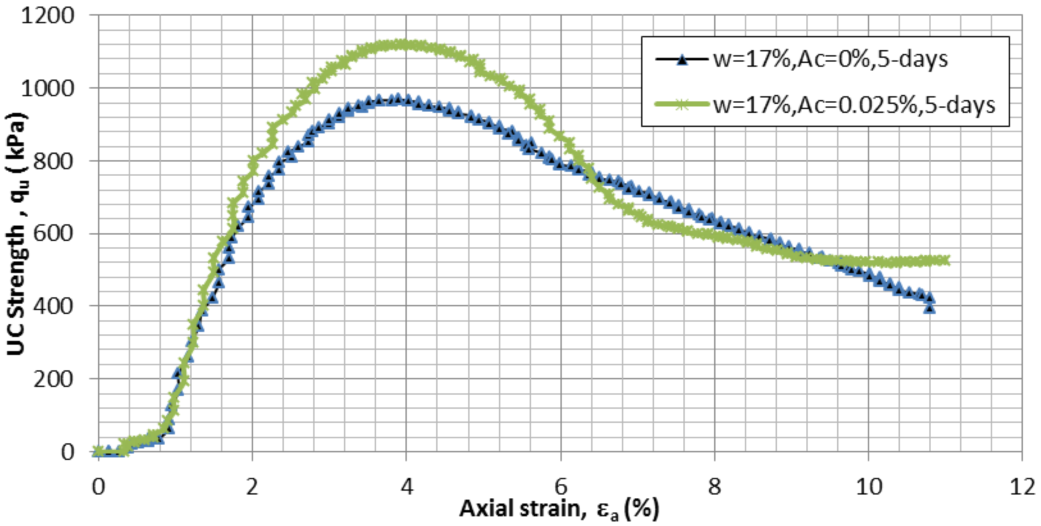


Figure 5.31 UC stress- strain curve for w=17%, T_c=5 days

At a moisture content of samples of 17%, the UC strength of the uncured specimens treated with 0.025% Conaid is slightly higher than the untreated one (Figure 5.31). After curing in the moisture room for 5 days, there is a more than 15% improvement in shear strength for specimens with 0.025% Conaid (Figure 5.32).

5.3.5.2 Failure behavior

The common failure mode of the uncured samples is crushing. An unclear failure plane can be seen in the 5-day moisture room cured specimens. Because it is a non-homogenous clay with some sand and gravel, the effects of Conaid is not clear.


(a) (b) (c) (d)

Figure 5.32: Photos of specimens after UC test: (a) A_c=0%, w=17%, 0-day; (b) A_c=0.025%, w=17%, 0-day; (c) A_c=0%, w=17%, 5-day moisture room dry; (d) A_c=0.025%, w=17%, 5-day

moisture room dry.

5.4. Summary and Conclusion

Based on limited test results from the CD tests and UC tests, the effects of Conaid on clayey material are summarized as follows:

1) The shear strength of kaolinite treated with 0.25% and 1% Conaid decreases compared to the untreated kaolinite samples. Also, the treated samples became slightly stiffer than the untreated ones. However, it is observed that volume change is associated with different Conaid contents with the same moisture content. The addition of Conaid can somewhat reduce the volume change of kaolinite specimen during consolidation and shearing.
2) Bentonite is also weakened with the addition of 0.25% Conaid, especially on the wet of optimum samples. However, adding 0.01% and 0.025% Conaid does not noticeably decrease the shear strength of bentonite. In addition, 0.25% Conaid may change the stiffness of treated bentonite.
3) Positive effects on the shear strength of treated BK20 are observed with 0.025% and 0.25% Conaid.
4) For Devon silt, there is no significant effect on the shear strength. Maybe silt particles in Devon silt are too coarse to react with Conaid. Also, there may be potential benefits of Conaid on Anthony Henday clay which cannot be distinguished from other factors due to its nonhomogeneous characteristics.
5) It is postulated that soil structure of kaolinite treated with Conaid tends to be more dispersed than that of the untreated one. The dispersive structure may marginally increase the dry density of the compacted soil, which cannot compensate the loss in shear strength weakened by this dispersive structure. On the other hand, the soil structure of bentonite may become more flocculated with the addition of 0.25% Conaid than untreated one. However, the gain in strength due to flocculation is usually offset by the reduction in dry density. Nevertheless, the above hypothesis should be verified with further study, such as using the SEM.
6) For the CD tests, barreling with a failure plane of about 25 °is observed under slow shearing. For the UC tests, various failure behaviors are observed, which

are related to the curing method or the specimen moisture content. The failure behaviors between untreated and treated specimens are also similar to each other.

5.5. References

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6. Microstructure of treated and untreated soils

6.1 General

Since the first scanning electron microscope (SEM) image was obtained by M. Knoll in 1935 (Chen, 1994), SEM has been widely used in studying materials such as soils. The maximum resolution of an optical microscope is about 200 nm, whereas SEM resolution can be as high as 0.5 nm. The latest commercial transmission electron microscopy (TEM) with an ultra-thin specimen achieved a resolution of 0.002 nm. As a result, the influence of soil stabilizer on soil structure can be examined with the aid of SEM.

There are two extreme forms of soil structure, the flocculated structure and dispersed structure. The soil particles in a flocculated structure are edge to face and they tend to move toward each other. The net force between two soil particles in a dispersed structure is repulsive and the particles are parallel to each other. Most soil structures are between these two extremes.

In general, under the same compaction effort, higher dry density can be acquired in a dispersed structure than a flocculated one. If an ionic soil stabilizer acts as compaction aid, deflocculated or dispersed structure in a treated soil should be observed after the addition of the stabilizer. Reversely, flocculated structure caused by an ionic soil stabilizer may reduce the efficiency of compaction effort. Thus, dry density in treated soil will be decreased.

On the other hand, the fundamental hypothesis of the mechanism of ionic soil stabilizer is that the double-layer thickness of the treated soil particles can be altered. If the double-layer thickness can be decreased, soil particles tend to attract to each other and they get closer. A tendency toward flocculation can happen. On the contrary, a dispersed structure due to repulsion between soil particles may be created by the soil stabilizer if the double-layer thickness is increased. Comparison of SEM images between treated and control specimens can provide visual information on the changes of soil structure.

6.2 Sample preparation for SEM

SEM photomicrographs were taken at the Scanning Electron Microscope Laboratory, Department of Earth and Atmospheric Sciences, University of Alberta. A Zeiss EVO SEM was used in capturing these images, which provides magnification from 20x to 100 000x with a resolution of about 5 nm.

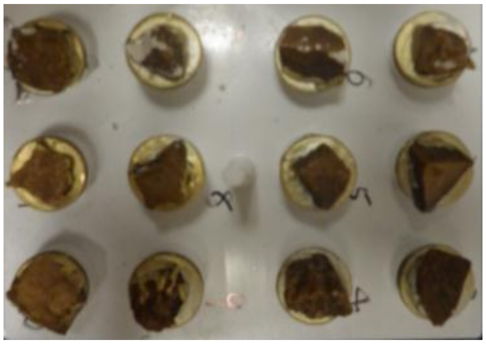
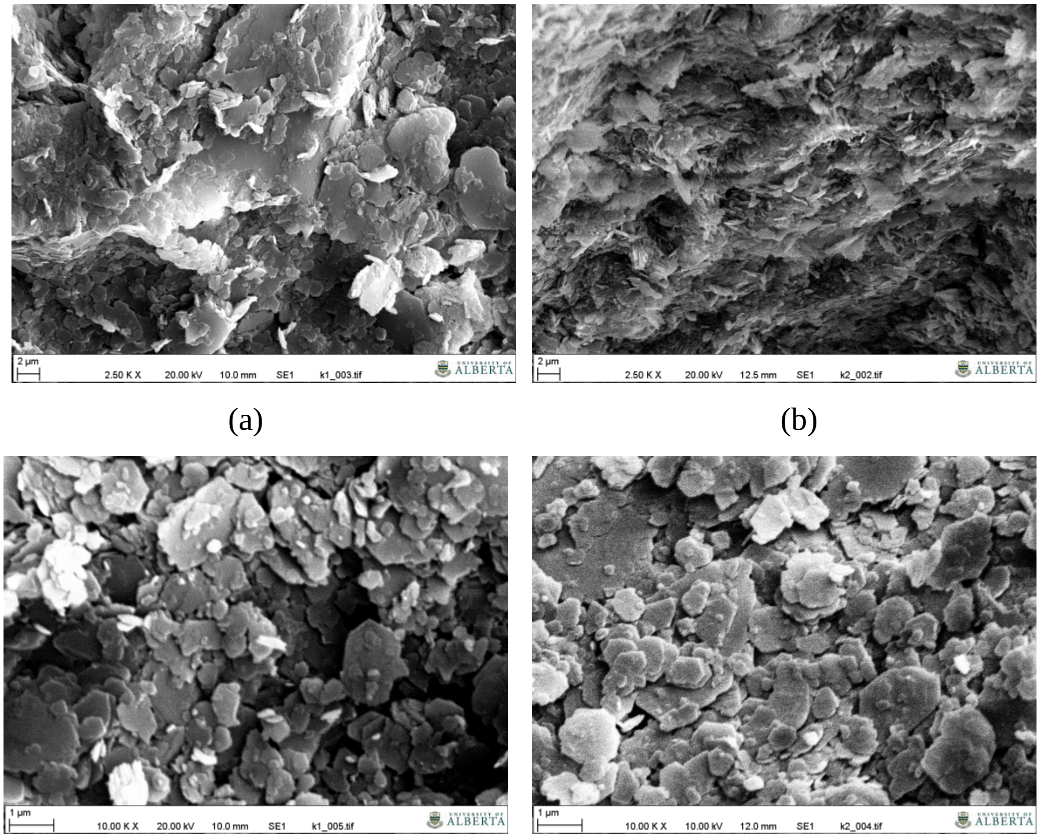


Figure 6.1 SEM samples glued on stubs and coated with gold coating

With the results from drained triaxial and unconfined compression tests, fourteen (14) samples were examined under the SEM. All samples were air-dried in room temperature. Specimens were less than 1.5 cm in diameter and 1 cm high. PELCO® Colloidal Silver Liquid was used as conductive adhesive to glue the specimens on the metal stubs. The silver liquid should also be dry enough before vacuum to remove any loose particles. The last step before placing in the SEM is coating. Conductive coating using gold was applied in all the samples with Xenosput XE200 sputter coater.

6.3 Soil structure of untreated and treated kaolinite

Well-crystallized kaolinite particle are formed by hexagonal plates, which consist of repeating stacks of silica and gibbsite sheets. Figure 6.2 illustrates the images of air-dried kaolinite samples. These specimens were taken from untreated and treated kaolinite samples after drained triaxial test with a confining pressure of 150kPa.



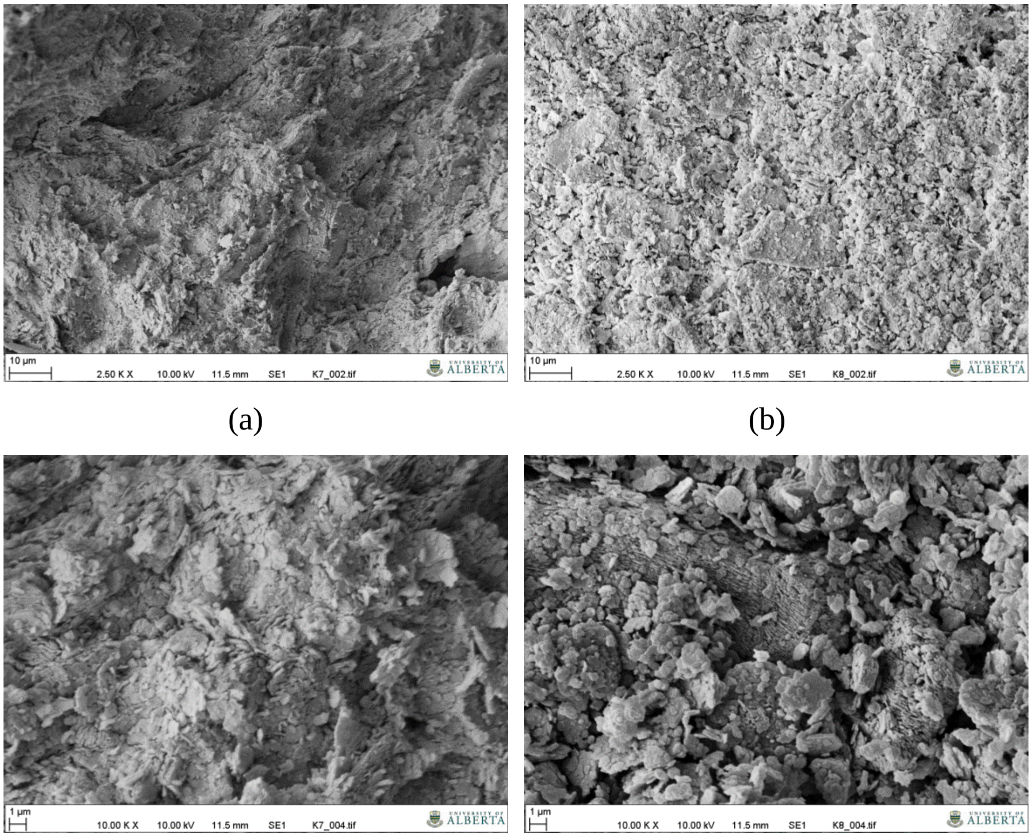
(a) (b)

(c) (d)

Figure 6.2 Photomicrographs of kaolinite at w=24%, (a) M~2,500X, A_c=0%, T_c=10 days; (b) M~2,500X, A_c=0.25%, T_c=17 days; (c) M~10, 000X, A_c=0%, T_c=10 days; (d) M~10, 000X, A_c=0.25%, T_c=17 days.

From Figure 6.2, no cementitious material is visible in the treated kaolinite sample, suggesting that the mechanism of stabilization from Conaid (if it has) is different from traditional stabilizers. The SEM images in treated kaolinite (Figure 6.2b, 6.2d) show a slightly denser and less flocculated structure than that of the untreated kaolinite, which is supported by the compaction curves in Figure 4.3. The benefit from Conaid as a compaction aid may be somewhat verified here but no significant changes are observed. Furthermore, Figure 5.5 indicates that shear strength of the treated kaolinite sample is weakened by Conaid. It is presumed that less flocculated structure may slightly increase the dry density of compacted soil but shear strength could be deteriorated with this structure at the same time.

Also, specimens from unconfined compression test were also examined under the SEM. Because soil microstructure could be altered due to high confining pressure, degree of saturation and consolidation in drained triaxial test, specimens from unconfined compression test are more suitable in studying soil microstructure.



(a) (b)

(c) (d)

Figure 6.3 Photomicrographs of kaolinite at w=24%, T_c=7 days (a) M~2,500X, A_c=0%; (b) M~2,500X, A_c=0.25%; (c) M~10,000X, A_c=0%; (d) M~10,000X, A_c=0.25%

The surface of untreated kaolinite is uneven. Voids and fissures can be obviously seen (Figure 6.3a). However, the treated kaolinite illustrates a comparatively smooth surface composed of many smaller size voids (Figure 6.3b). Both untreated and treated kaolinite specimens display a "card-house" type of particle arrangement (Figure 6.3c, 6.3d).

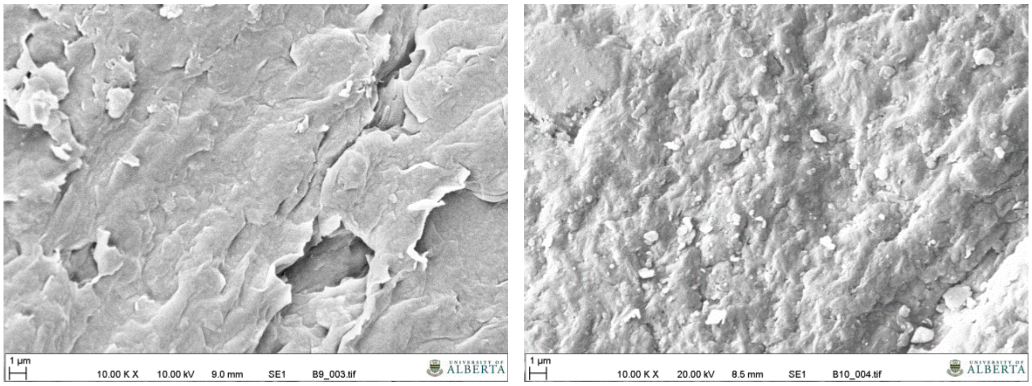
The small poorly crystallized kaolinite particles with a face-to-face arrangement cling to the surface of well-crystallized stacks of kaolinite sheets (Figure 6.3d). The arrangement of small poorly crystallized kaolinite particles in treated kaolinite appears to be more aggregated than that of untreated one. But voids between aggregates and stacks of sheet-like layers are obvious in treated kaolinite, which explains why porous structure is observed in Figure 6.3b. It is possible that a small fraction of poorly crystallized kaolinite particles may react with Conaid, resulting in a face-to-face and aggregated soil structure. However, Conaid cannot break down the well-crystallized stacks of sheet-like layers which are held together tightly by hydrogen bonds. If the majority of kaolinite consists of well crystallized platelets, this explains why only minor dry density increase can be achieved in Proctor Compaction test (Figure 4.3).

It is concluded that Conaid cannot break down the "card-house" structure of well crystallized kaolinite stacks and only some poorly crystallized particles may be affected. A face-to-face and aggregated soil structure may appear in those poorly crystallized particles, which could contribute to marginal change in the dry density of the compacted soil. It is expected that Conaid at optimal application ratio would have significant effects on poorly crystallized kaolinite.

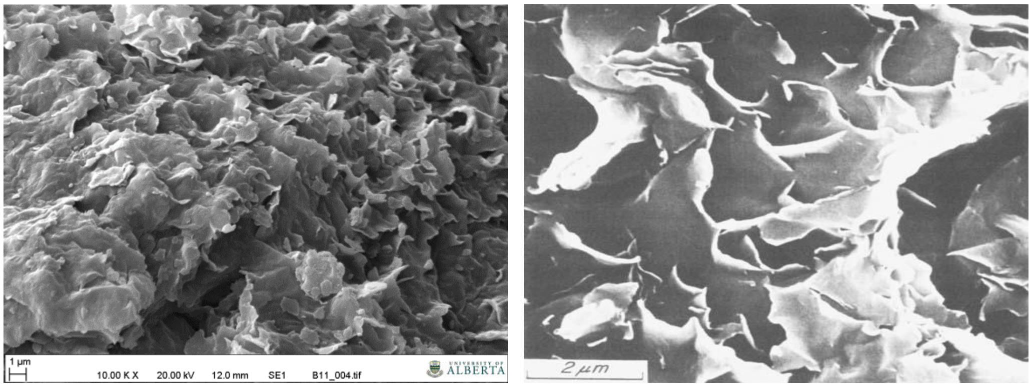
6.4 Soil structure of untreated and treated bentonite

Bentonite is the name for clay whose major mineral is montmorillonite. Montmorillonite is a three-layer mineral consisting of an octahedral sheet sandwiched between two silica sheets. Bonding between layers is by van der Waals forces and by cations. The bonding is weak which can be easily separated by cleavage or adsorption of water or other polar liquids (Mitchell, 1976).

It is postulated that Conaid can alter the three-layer structure and reduce the thickness of double-layer water. Therefore, both treated and untreated compacted bentonite samples were examined with the Zeiss EVO SEM.



(a) (b)



(c) (d)

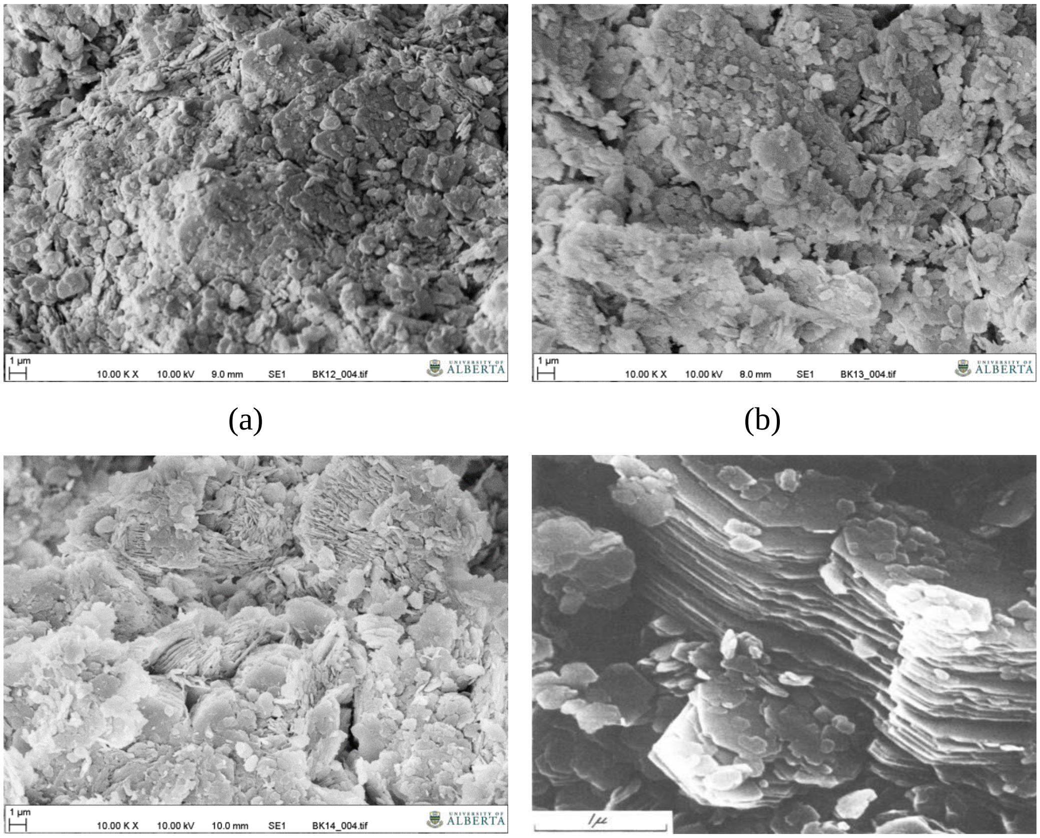
Figure 6.4 Photomicrographs of bentonite at w=32%, T_c=21 days and M~10,000X, (a) A_c=0%; (b) A_c=0.025%; (c) A_c=0.25%; (d) montmorillonite after Terzaghi (1996)

Compared with Figure 6.4c, soil surfaces in Figure 6.4a and 6.4b are relatively flat and smooth, indicating specimens may be taken from the shear zone. The film-like flakes can be observed in Figure 6.4c, which are very similar to those in Figure 6.4d (Terzaghi, 1996).

If it can be verified that Conaid causes more flocculation of the montmorillonite particles (Figure 6.4c) than those in Figure 6.4d, this could explain the minor decrease in dry density for compacted bentonite treated with 0.25% (Figure 4.7). Flocculation caused by 0.25% Conaid reduces the effort of compaction energy. The gain in strength due to flocculation in bentonite is usually offset by the reduced dry density, which eventually deteriorates the shear strength of compacted bentonite (Figure 5.18).

6.5 Soil structure of untreated and treated BK20

BK20 is the mixture of 20% bentonite and 80% kaolinite. The majority of clay minerals are kaolinite. There are no significant visual changes between untreated and treated BK20 (Figure 6.5a, 6.5b). Stacks of card-house sheets can easily be observed in Figure 6.5c. Montmorillonite films are hardly seen in all the SEMs in Figure 6.5a, 6.5b and 6.5c. However, small poorly crystallized kaolinite particles can be coated with bentonite cling to the surface of the card-house in this mixture and connect the well crystallized stacks. Also, there are more edge-to-face flocculated and aggregated stacks found in Figure 6.5c, which may lead to the significant increase of shear strength for treated BK20.



(a) (b)

(c) (d)

Figure 6.5 Photomicrographs of BK20 at w=28%, T_c=7 days and M~10,000X, (a) A_c=0%; (b) A_c=0.025%; (c) A_c=0.25%; (d) kaolinite after Terzaghi (1996)

6.6 Soil structure of untreated and treated Devon silt

More porous surface texture in Figure 6.6a for Devon silt without Conaid is observed whereas sample treated with 0.05% Conaid (Figure 6.6b) has flatter surface with denser texture. The untreated Devon silt has flocculated flaky film-like particles, and the treated one displays a dispersed microstructure. Silt grains can be found on the surface of the flakes. There is much less voids and fissures in treated Devon silt than untreated one (Figure 6.6a, 6.6b).

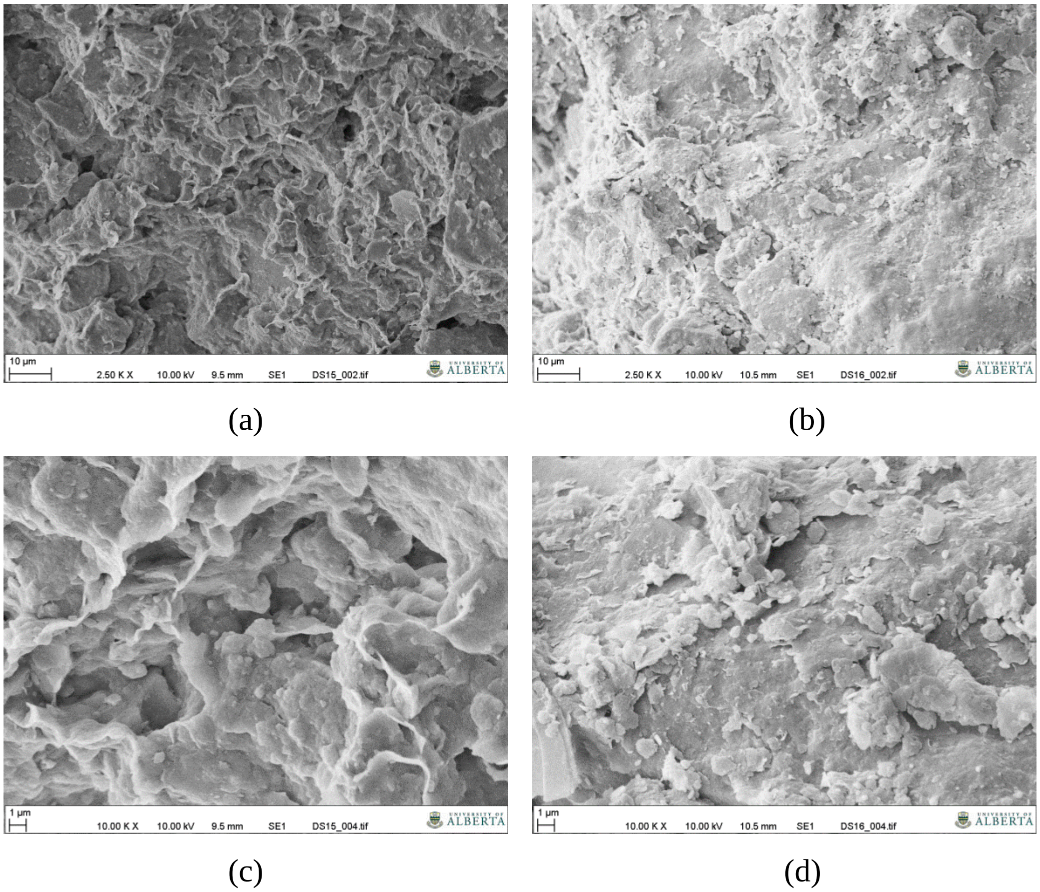


Figure 6.6 Photomicrographs of Devon silt at w=16.5%, T_c=12 days, (a) M~2,500X, A_c=0%; (b) M~2,500X, A_c=0.05%; (c) M~10,000X, A_c=0%; (d) M~10,000X, A_c=0.05%.

It is possible that Conaid has some effects on Devon silt. However there are only two specimens being tested and examined under the SEM. To confirm this point, more samples and SEM examinations are required.

6.7 Summary and conclusion

Through the analysis of photomicrographs from four types of soils, some preliminary conclusions can be drawn as follows:

1) There is a slight microstructure changes for the poorly-crystallized kaolinite treated by 0.25% Conaid. The small pieces of kaolinite particles have a face-to-face arrangement cling to the surface of well-crystallized stacks of kaolinite sheets. However, Conaid cannot break down the card-house structure consisting of stacks of silica and gibbsite sheets. Therefore, Conaid has little effect on the well-crystallized kaolinite minerals.

2) Compared with the reference image, montmorillonite particles in treated sample seem to be somewhat more flocculated than those in untreated one. However, such minor changes observed in the arrangement of the montmorillonite particles could reduce the compaction effort in the Proctor compaction test.

3) It can be observed that small poorly crystallized kaolinite particles coated with montmorillonite particle sheets cling to the surface of the card-house in the treated BK20 and connect the well crystallized stacks. The edge-to-face flocculated and aggregated stacks are found in sample treated with 0.25% Conaid, which may explain the significant increase of shear strength.

4) Conaid probably has some benefits on Devon silt as a stabilizer, which is able to reduce void ratio in compacted Devon silt at the optimal moisture content. As silt grains are probably too coarse to react with Conaid, a fraction of the clay particles in the Devon silt may be susceptible to Conaid. However, it is questionable that too much Conaid may lubricate the coarse silt particles and decrease the shear strength of the soil.

5) In most cases, there are little or only minor changes with the addition of Conaid for the soil samples being tested. Some positive changes should be verified with more well-prepared SEM specimens along with mineral analysis, such as X Ray Diffraction (XRD).

6.8 References

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[2] Mitchell, J. K. 1976. Fundamentals of Soil Behavior, 2nd ed. John Wiley and Sons, Inc., New York.

[3] Shi, B., Liu, Z., Cai, Y. and Zhang, X. 2007. Micropore structure of aggregates in treated soils. Journal of Materials in Civil Engineering, ASCE.

[4] Terzaghi, K., Peck, R. B., Mesri, G. 1996. Soil Mechanics Practice in

Engineering, John Wiley & Sons, Inc., New York.

[5] Wang, Y. H., Siu, W.K. 2006. Structure characteristics and mechanical properties of kaolinite soils. I. Surface charges and structural characterizations. Canadian Geotechnical Journal 43: 587-600.

7. Conclusion and recommendations

7.1 Summary and conclusion

To understand the effects of Conaid on selected test soils, various standard laboratory tests were conducted. Atterberg limits were carried out on Devon silt, kaolinite and bentonite to exam the possible changes due to the addition of Conaid. Besides, Conaid effects on compaction characteristics, shear strength of soil samples were evaluated by using of Standard and Modified Proctor test, triaxial compression test and unconfined compression test. Finally, SEM was conducted to observe the microstructure of test soils.

With limited test results, some preliminary general conclusions are made as follows:

1) Conaid has some effects on the Atterberg limits of the soils being tested, especially on liquid limit. Some noticeable changes on bentonite can be observed.

2) As a potential additive to improve the compaction of a soil, Conaid has been observed to increase the dry density of compacted soils marginally except for kaolinite treated with 0.25% Conaid. However, even though Conaid can improve the compaction characteristics of kaolinite, no consistent and significant results from other soils being tested have been obtained. Under Standard or Modified Proctor compaction test conditions, there is no significant improvement for montmorillonite with the addition of Conaid. Unexpectedly, preliminary results show that Conaid may even reduce the compaction effort and decrease the dry density of bentonite at a moisture content wet of optimum.

3) Even though the dry density of kaolinite could be somewhat increased by adding Conaid in the Standard Proctor test, shear strength was found to be lowered. When bentonite and kaolinite are mixed together, the BK20 material shows improvement in strength by adding Conaid. Both shear strength and ductility are increased with 0.025% and 0.25% Conaid. With

limited Devon silt specimens treated with 0.05% Conaid, minor improvement was observed.

4) Conaid decreases the volume change during shearing and consolidation of the kaolinite material. Less water is imbibed during saturation stage in the CD test, which is expected to reduce the damage caused by soil swelling in a dry-wet cycle. Further tests are needed to verify this benefit.
5) Poorly-crystallized kaolinite particles may become more dispersive or less flocculated after being treated with 0.25% Conaid. However, Conaid appears to have no change on the card-house structure of well-crystallized kaolinite sheets due to strong hydrogen bonding.
6) Some positive results still need to be verified with more SEM and mineral analysis, such as X Ray Diffraction (XRD).

7.2 Recommendations

Further study is necessary to verify the preliminary conclusions listed above. To fully understand the effects on test soils with Conaid, here are some suggestions:

1) Since specimen are retrieved from soil sample compacted in three lifts in a compaction mold which leads to inhomogeneity in the compacted soil. It is preferred to use a gyratory compactor by molding the soil in one lift. Also, proper mixing equipment, such as an electrical mixer, and procedure are critical to ensure high-quality laboratory soil samples for compaction test. Also, moisture room curing for retrieved specimens can decrease the rate of drying, which can minimize the development of fissures on specimen surface.
2) Instead of UC test, UU test is recommended because confining pressure can eliminate the deficiency in specimens, such as voids and fissures. Besides, durability tests, such as freeze/thaw circle and wet/dry test, swell potential tests are also recommended.
3) The Proctor compaction curve should be developed individually for each Conaid content. The OMC from untreated soil shouldn’t be directly applied to treated soils. Also, optimal amount of Conaid for different soils should be

determined.

4) Specimens for SEM analysis should be well prepared to represent the original fabric of the test sample. Comparison should be made at various angles. Besides, X Ray Diffraction (XRD) analysis is needed to examine mineral changes accomplished through the addition of Conaid.

5) Field test should be conducted to monitor the long term effects of Conaid on permeability, swell potential and maintenance cost of treated soils subjected to seasonal variation in life-cycle.

6) With reference to some index properties, soils frequently encountered in construction field should be categorized with the optimal Conaid contents and OMCs.

8. Appendix

8.1 Appendix A: Consolidated drained triaxial compression test

Consolidated drained triaxial compression test (CD test) were conducted on kaolinite samples at the University of Alberta between September 2010 and September 2011. Samples with Conaid contents 0.25%, 0.5% and 1% mixed at the optimum moisture content were tested. Specimens trimmed from the compacted sample were cured in moisture room for 10, 14, 16 and 17 days. Confining pressures of 50 kPa and 150 kPa were applied, with a back pressure of 400 kPa. Graphs of all the shear data are attached in this following appendix. Details of the triaxial test can be referred to Chapter 5.

Table A.1 Specimen Information for A_c=0%, w=24%, 10-day & p_0'=150kPa

Specimen Information	Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)
Before Test	37.83	76.68	1124	168.5	23.28
After Test	-	-	-	175.2	27.93

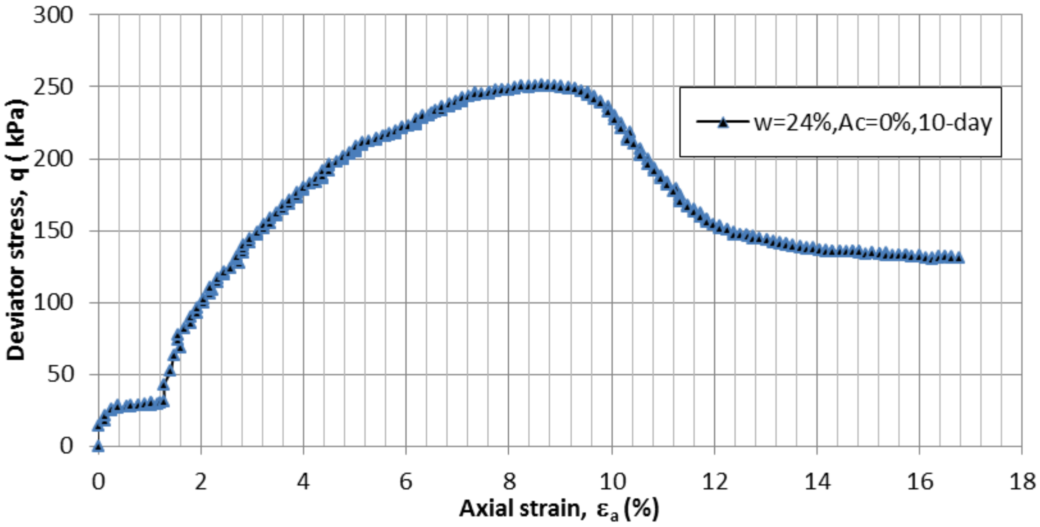


Figure A.1 q vs. ε_a for A_c=0%, w=24%, 10-day & p_0'=150kPa

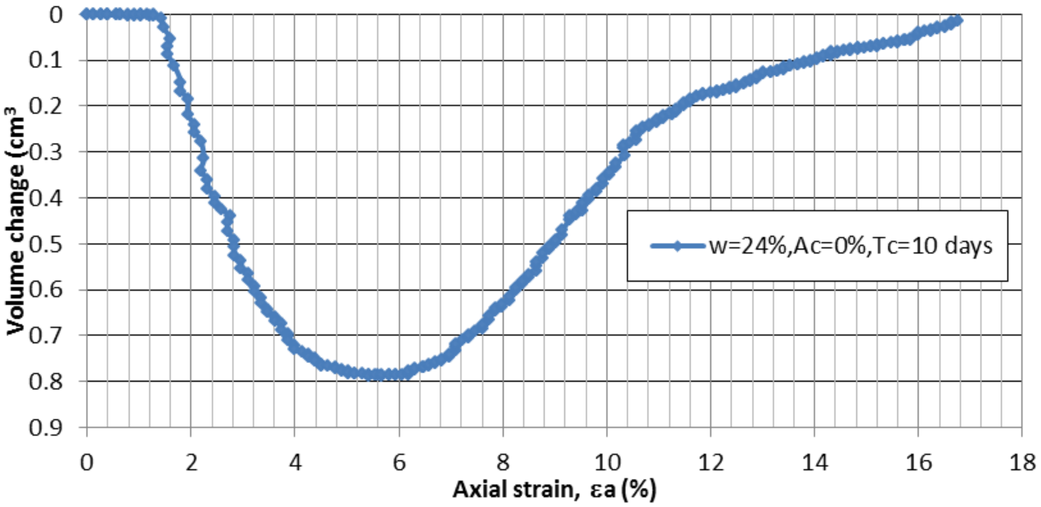


Figure A.2 Volume change vs. εa for Ac=0%, w=24%, 10-day & p_0'=150kPa

Figure A.3 Photo after CD test for Sample: Ac=0%, w=24%, 10-day & p0'=150kPa

Table A.2 Specimen Information for A_c=0%, w=24%, 10-day & p_0'=50kPa

Specimen Information	Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)
Before Test	37.87	76.42	1126	167.1	23.34
After Test	-	-	-	174.2	27.59

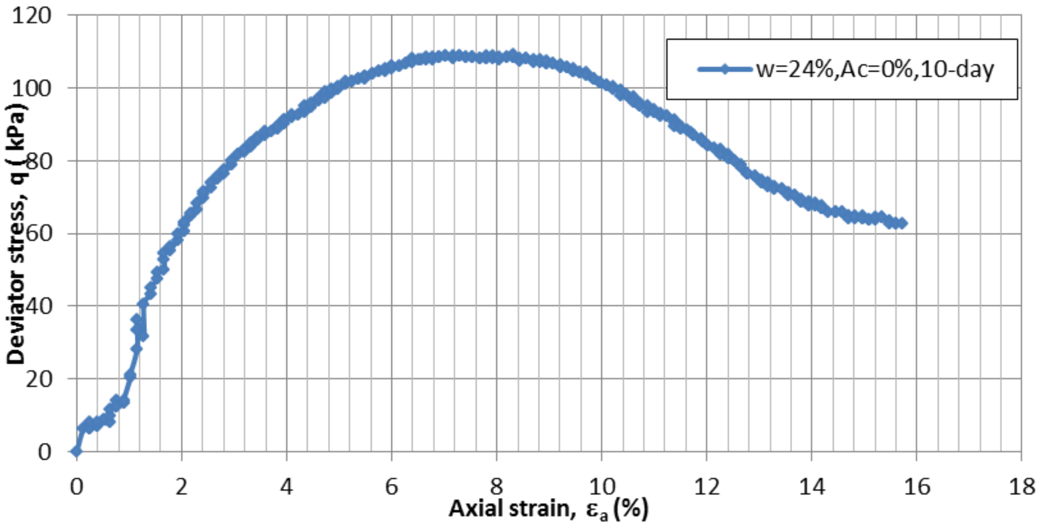


Figure A.4 q vs. ε_a for A_c=0%, w=24%, 10-day & p_0'=50kPa

Figure A.5 Photo after CD test for Sample: A_c=0%, w=24%, 10-day & p_0'=50kPa

Table A.3 Specimen Information for A_c=0.25%, w=24%, 17-day & p_0'=150kPa

Specimen Information	Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)
Before Test	37.86	75.58	1126	165.3	23.55
After Test	-	-	-	169.2	26.76

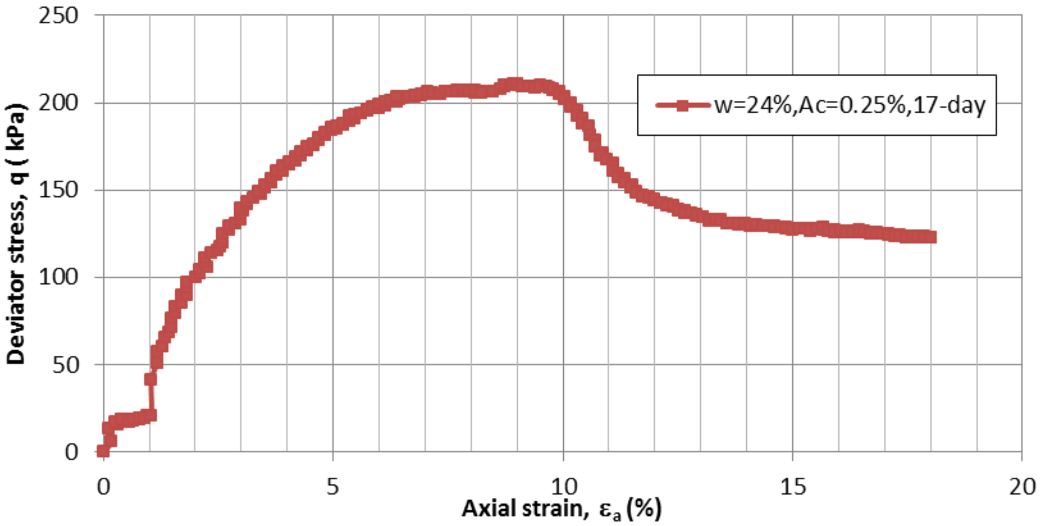


Figure A.6 q vs. ε_a for A_c=0.25%, w=24%, 17-day & p_0'=150kPa

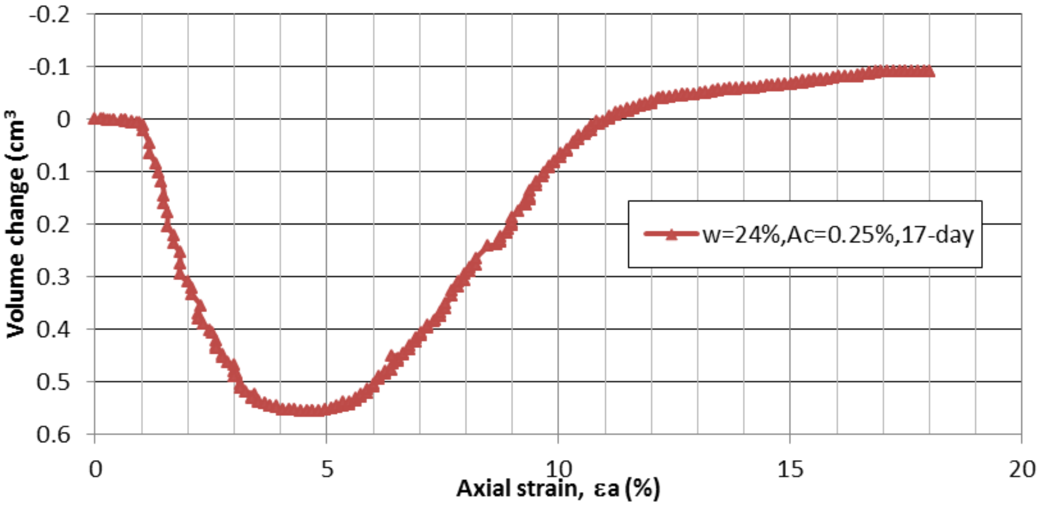


Figure A.7 Volume change vs. ε_a for A_c=0.25%, w=24%, 17-day & p_0'=150kPa

Figure A.8 Photo after CD test for Sample: A_c=0.25%, w=24%, 17-day & p_0'=150kPa

Table A.4 Specimen Information for A_c=0.25%, w=24%, 17-day & p_0'=50kPa

Specimen Information	Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)
Before Test	37.78	76.07	1121	166.9	23.55
After Test	-	-	-	173.6	28.98

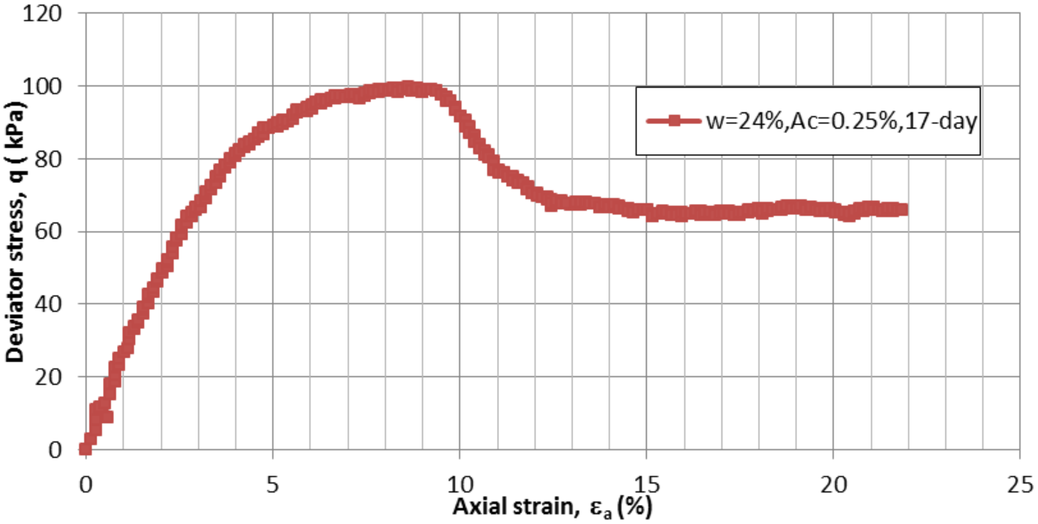


Figure A.9 q vs. ε_a for A_c=0.25%, w=24%, 17-day & p_0'=50kPa

Figure A.10 Photo after CD test for Sample: A_c=0.25%, w=24%, 17-day & p_0'=50kPa

Table A.5 Specimen Information for A_c=0.5%, w=24%, 16-day & p_0'=150kPa

Specimen Information	Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)
Before Test	37.8	76.28	1122	166.6	23.76
After Test	-	-	-	173.7	29.36

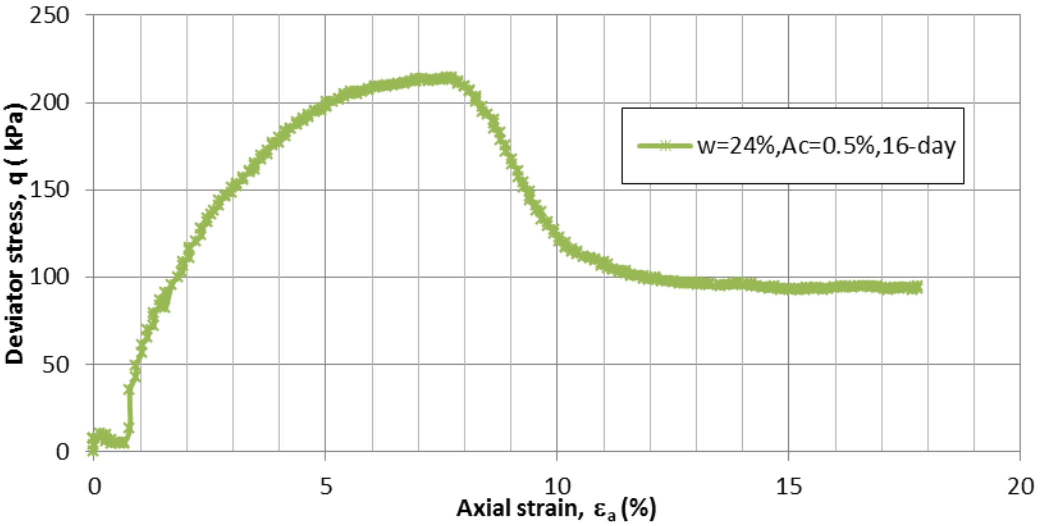


Figure A.11 q vs. ε_a for A_c=0.5%, w=24%, 16-day & p_0'=150kPa

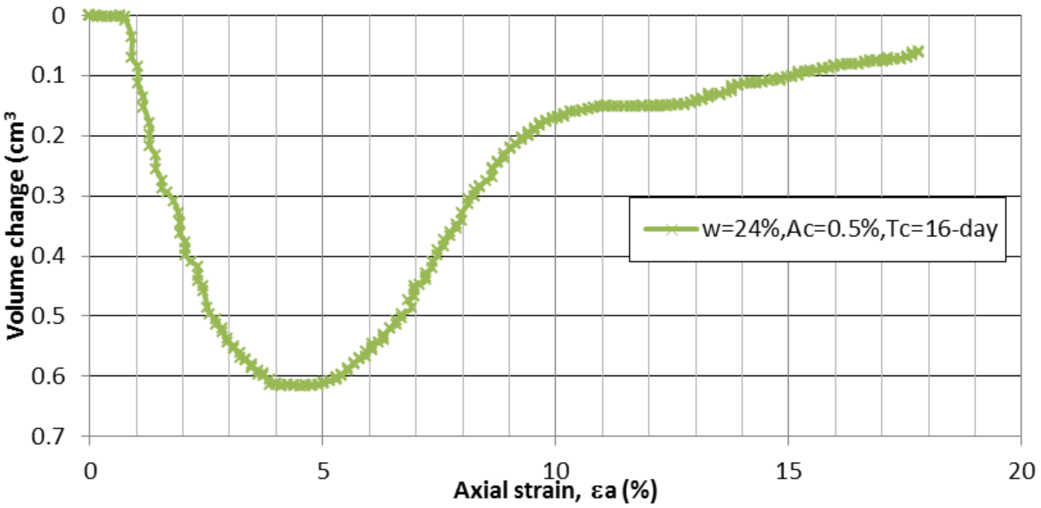


Figure A.12 Volume change vs. ε_a for A_c=0.5%, w=24%, 16-day & p_0'=150kPa

Figure A.13 Photo after CD test for Sample: A_c=0.5%, w=24%, 16-day & p_0'=150kPa

Table A.6 Specimen Information for A_c=0.5%, w=24%, 16-day & p_0'=50kPa

Specimen Information	Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)
Before Test	37.8	76.15	1124	166.6	23.76
After Test	-	-	-	173.3	27.54

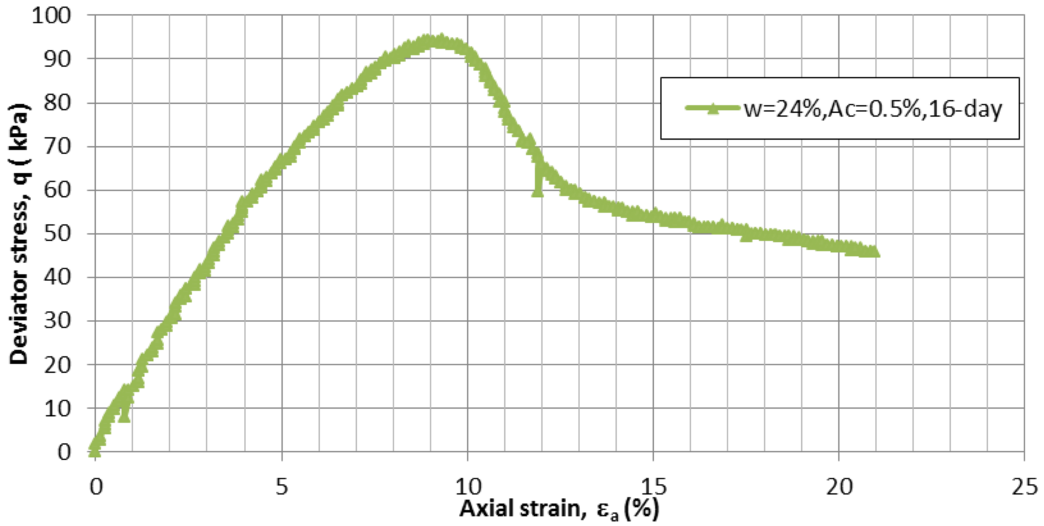


Figure A.14 q vs. ε_a for A_c=0.5%, w=24%, 16-day & p_0'=50kPa

Figure A.15 Photo after CD test for Sample: A_c=0.5%, w=24%, 16-day & p_0'=50kPa

Table A.7 Specimen Information for A_c=1%, w=24%, 14-day & p_0'=150kPa

Specimen Information	Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)
Before Test	38.05	74.27	1137	164.2	23.40
After Test	-	-	-	169.4	27.75

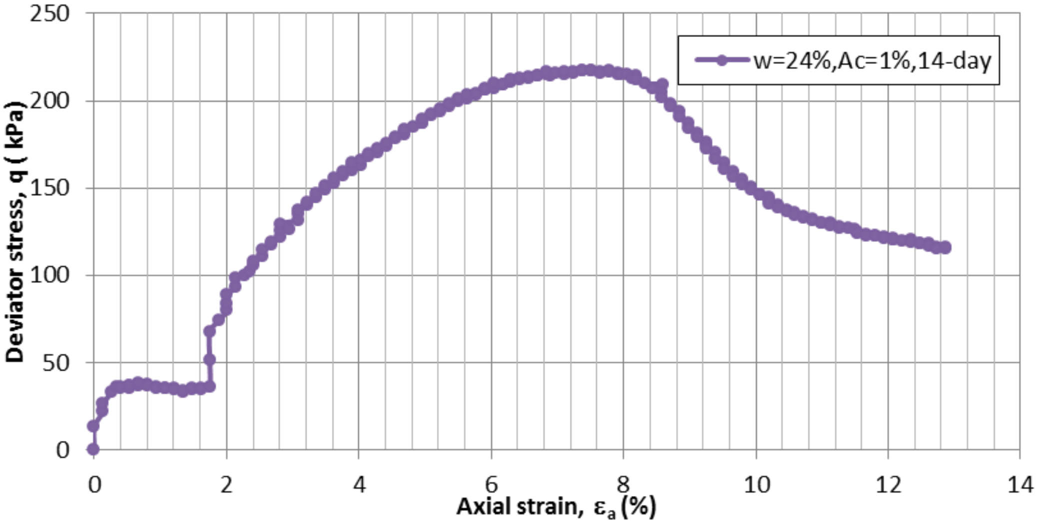


Figure A.16 q vs. ε_a for A_c=1%, w=24%, 14-day & p_0'=150kPa

Figure A.17 Photo after CD test for Sample: A_c=1%, w=24%, 14-day & p_0'=150kPa

Table A.8 Specimen Information for A_c=1%, w=24%, 14-day & p_0'=50kPa

Specimen Information	Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)
Before Test	37.85	74.32	1125	163.7	23.37
After Test	-	-	-	167.9	27.54

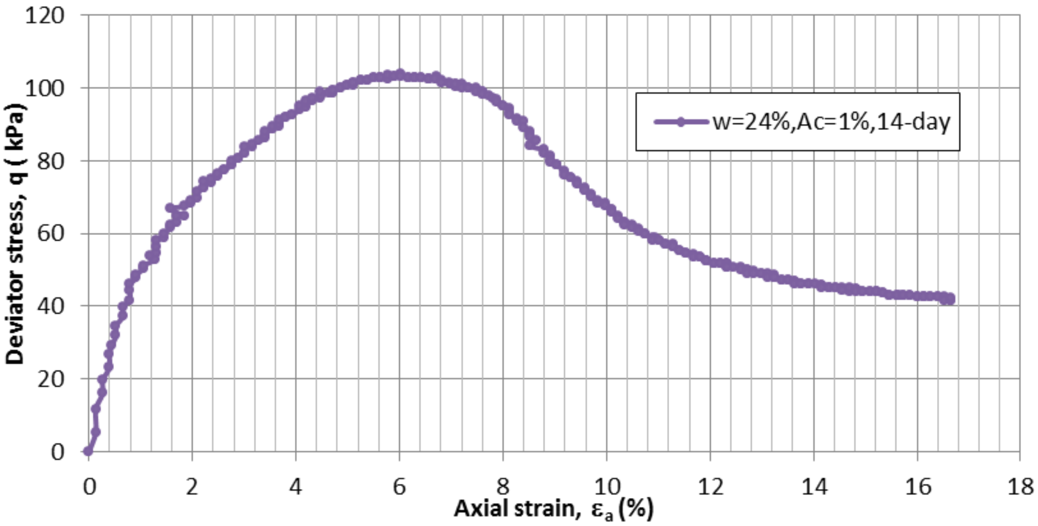


Figure A.18 q vs. ε_a for A_c=1%, w=24%, 14-day & p_0'=50kPa

Figure A.19 Photo after CD test for Sample: A_c=1%, w=24%, 14-day & p_0'=50kPa

8.2 Appendix B: Unconfined compression test on kaolinite

Unconfined compression test (UC test) were conducted on kaolinite at the University of Alberta between May 2011 and September 2011. Samples with Conaid contents 0.01%, 0.05% and 0.25% mixed at the optimum moisture content were tested. Control samples without Conaid were prepared in exactly the same way as treated ones. Specimens trimmed from the compacted sample were cured in moisture room or air-dried for 7 days. Typically, two tests were conducted for each mix. Graphs of all shear data are attached in Appendix B. Details of the UC test can be referred to Chapter 5.

Table B.1 Specimen Information for A_c=0%, w=24%, 7-day air dry, sample-1

Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)	Loading Rate (mm/min)
37.39	74.39	1098	134.6	1.86	0.3

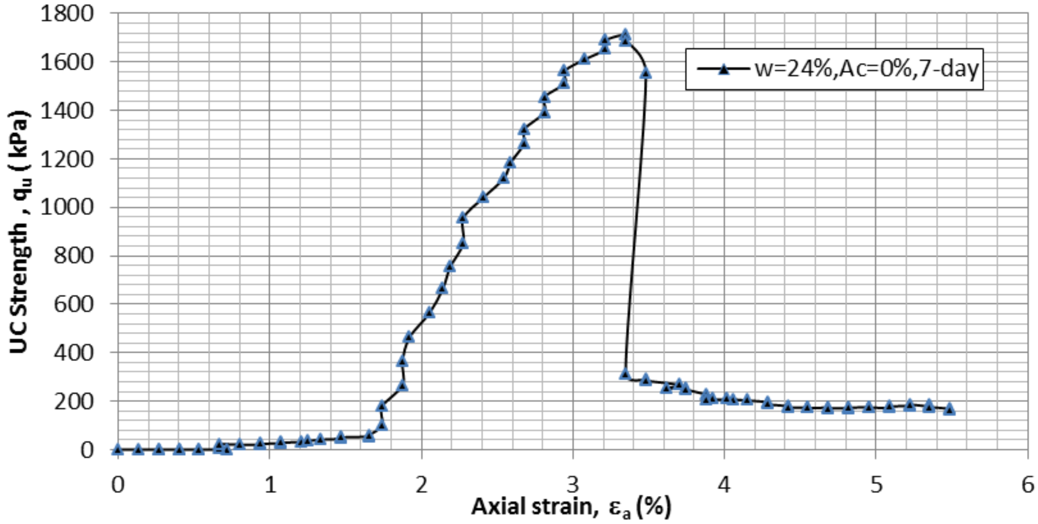


Figure B.1 q vs. ε_a for A_c=0%, w=24%, 7-day air dry, sample-1

Figure B.2 Photo after UC test for Sample-1: A_c=0%, w=24%, 7-day air dry

Table B.2 Specimen Information for A_c=0%, w=24%, 7-day air dry, sample-2

Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)	Loading Rate (mm/min)
37.34	73.43	1095	133.35	1.89	0.3

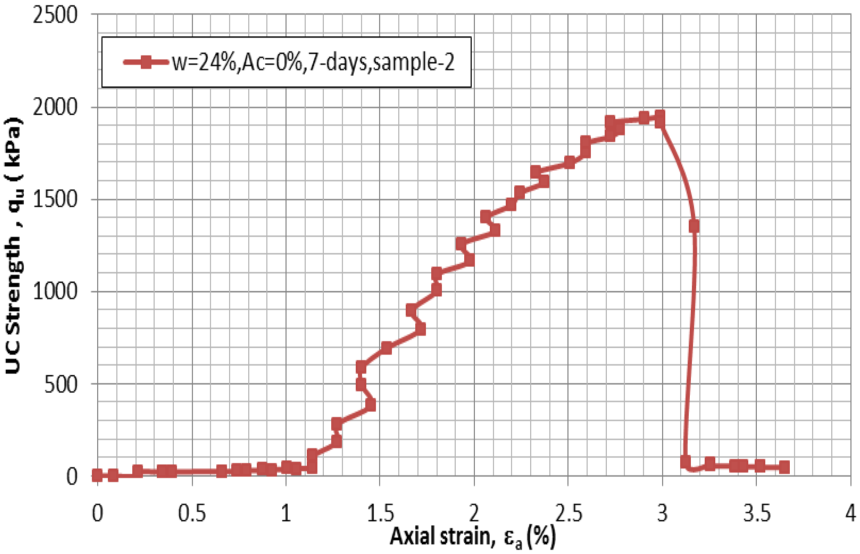


Figure B.3 q vs. ε_a for A_c=0%, w=24%, 7-day air dry, sample-2

Figure B.4 Photo after UC test for Sample-2: A_c=0%, w=24%, 7-day air dry

Table B.3 Specimen Information for A_c=0.25%, w=24%, 7-day air dry, sample-1

Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)	Loading Rate (mm/min)
37.25	72.65	1090	135.15	1.79	0.3

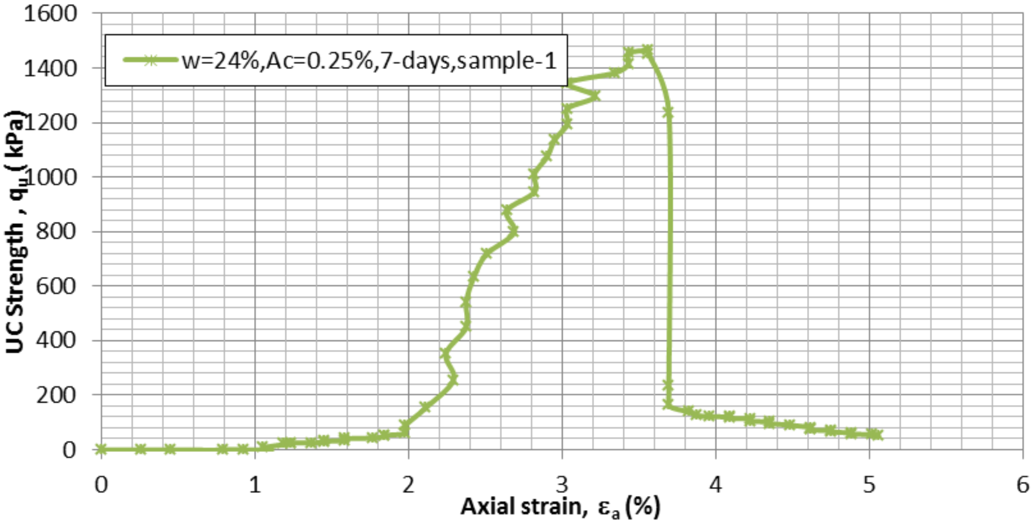


Figure B.5 q vs. ε_a for A_c=0.25%, w=24%, 7-day air dry, sample-1

Figure B.6 Photo after UC test for Sample-1: A_c=0.25%, w=24%, 7-day air dry

Table B.4 Specimen Information for A_c=0.25%, w=24%, 7-day air dry, sample-2

Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)	Loading Rate (mm/min)
37.11	72.9	1082	135.15	1.75	0.3

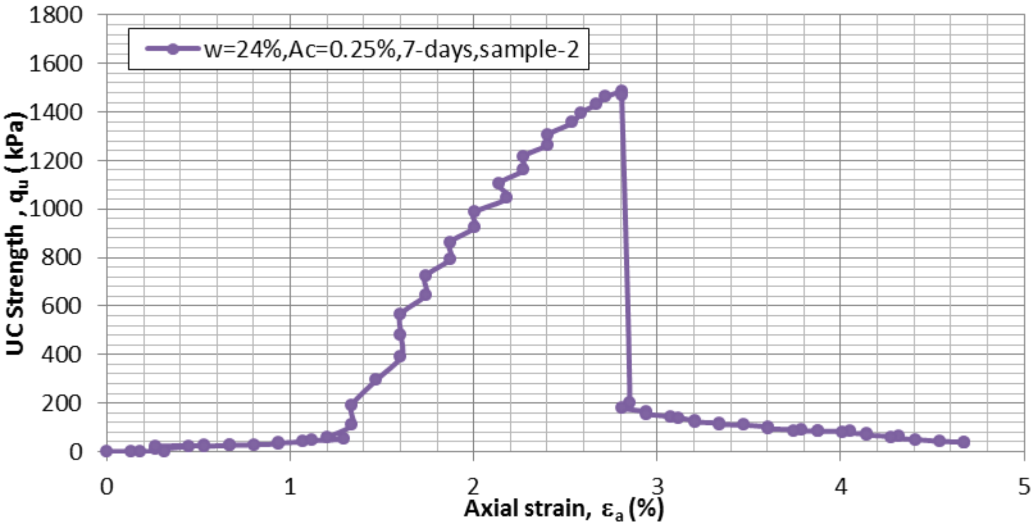


Figure B.7 q vs. ε_a for A_c=0.25%, w=24%, 7-day air dry, sample-2

Figure B.8 Photo after UC test for sample-2: A_c=0.25%, w=24%, 7-day air dry

Table B.5 Specimen Information for A_c=0%, w=22%, 7-day moisture room dry

Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)	Loading Rate (mm/min)
37.37	73.2	1097	162.9	17.2	0.3

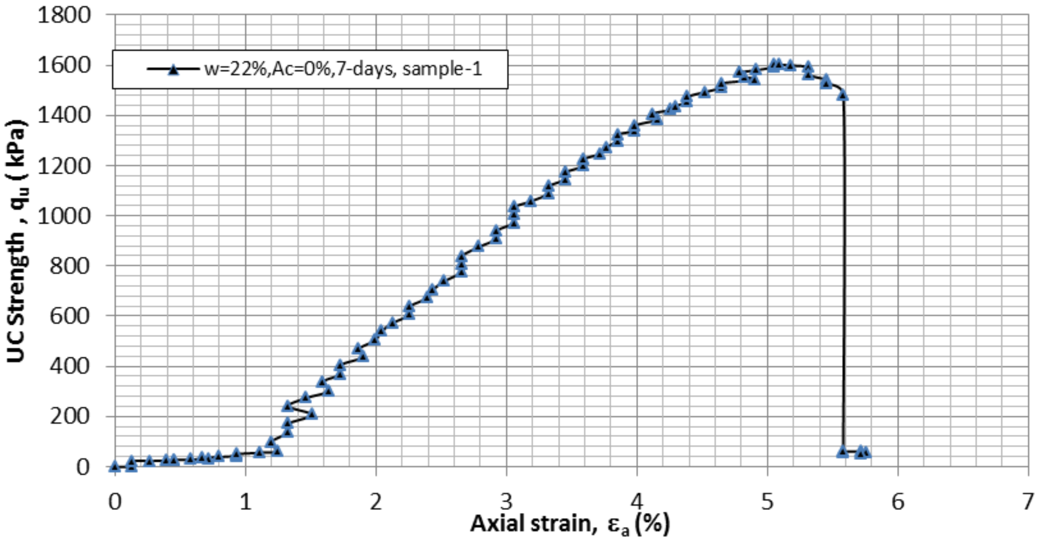


Figure B.9 q vs. ε_a for A_c=0%, w=22%, 7-day moisture room dry, sample-1

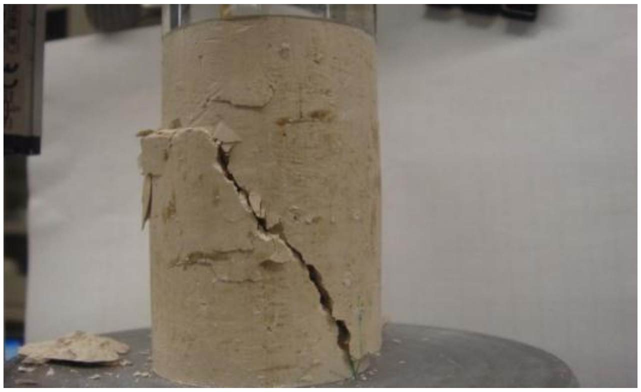


Figure B.10 Photo after UC test for sample-1: A_c=0%, w=22%, 7-day moisture room dry

Table B.6 Specimen Information for A_c=0%, w=22%, 7-day moisture room dry

Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)	Loading Rate (mm/min)
37.41	68.67	1099	153.7	17.5	0.3

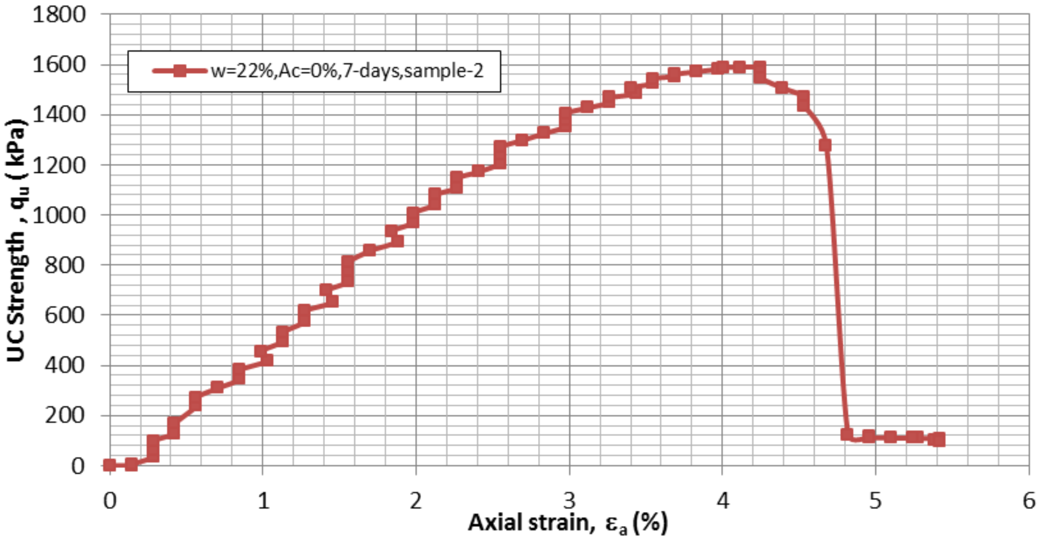


Figure B.11 q vs. ε_a for A_c=0%, w=22%, 7-day moisture room dry, sample-2

Figure B.12 Photo after UC test for sample-2: A_c=0%, w=22%, 7-day moisture room dry

Table B.7 Specimen Information for A_c=0.05%, w=22%, 7-day moisture room dry

Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)	Loading Rate (mm/min)
37.5	73.4	1104	162.85	17.56	0.3

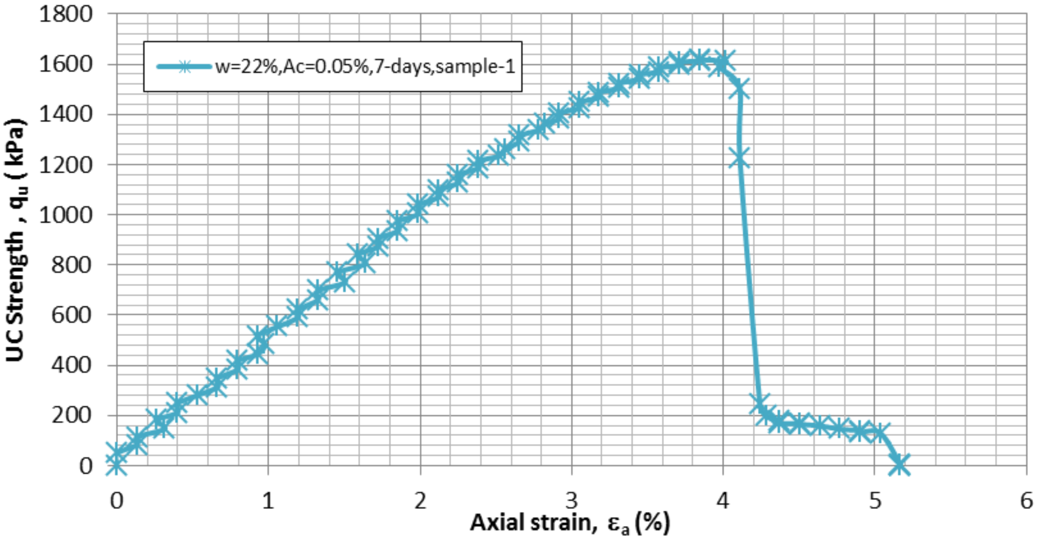


Figure B.13 q vs. ε_a for A_c=0.05%, w=22%, 7-day moisture room dry, sample-1

Figure B.14 Photo after UC test for sample-1: A_c=0.05%, w=22%, 7-day moisture room dry

Table B.8 Specimen Information for A_c=0.05%, w=22%, 7-day moisture room dry

Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)	Loading Rate (mm/min)
37.46	72.3	1102	161.25	17.23	0.3

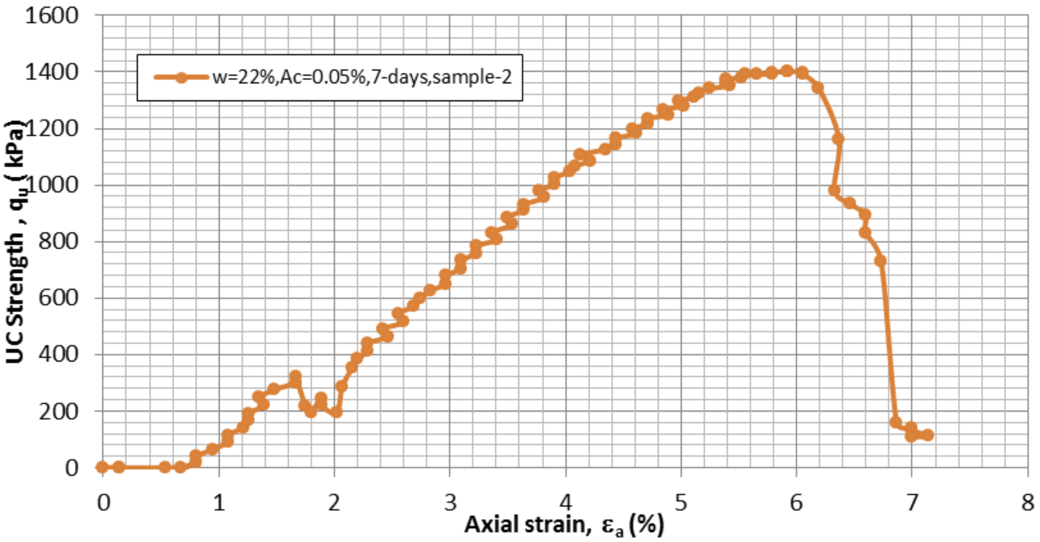


Figure B.15 q vs. ε_a for A_c=0.05%, w=22%, 7-day moisture room dry, sample-2

Figure B.16 Photo after UC test for sample-2: A_c=0.05%, w=22%, 7-day moisture room dry

Table B.9 Specimen Information for A_c=0.01%, w=22%, 7-day moisture room dry

Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)	Loading Rate (mm/min)
37.52	73.93	1106	165.65	17.32	0.3

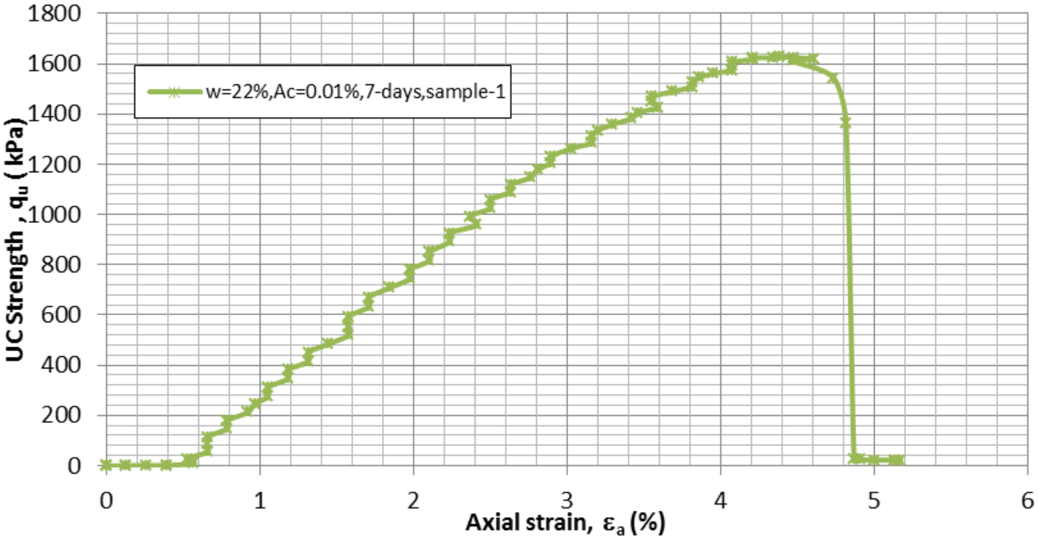


Figure B.17 q vs. ε_a for A_c=0.01%, w=22%, 7-day moisture room dry, sample-1

Figure B.18 Photo after UC test for sample-1: A_c=0.01%, w=22%, 7-day moisture room dry

Table B.10 Specimen Information for A_c=0.01%, w=22%, 7-day moisture room dry

Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)	Loading Rate (mm/min)
37.3	74.27	1093	164.45	17.28	0.3

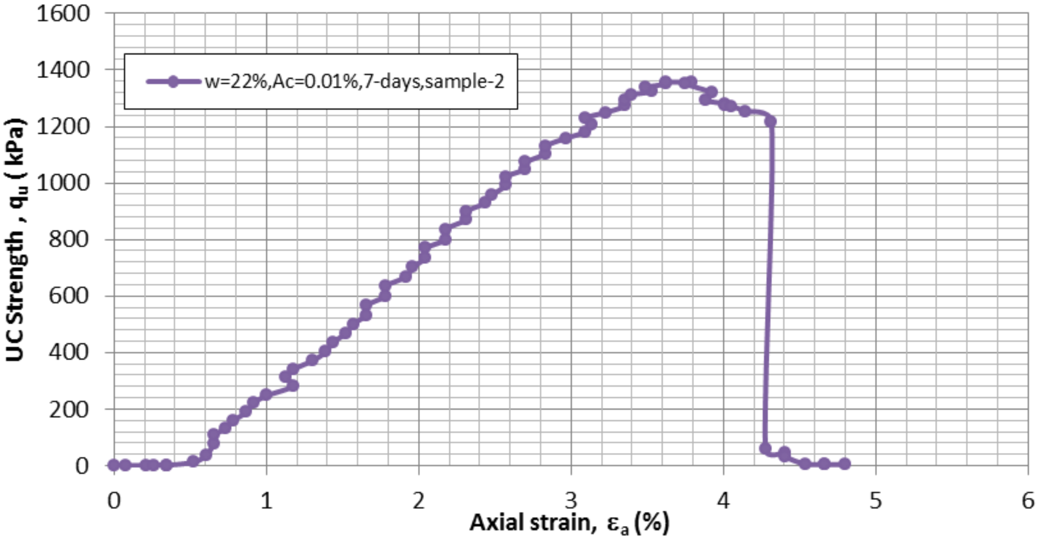


Figure B.19 q vs. ε_a for A_c=0.01%, w=22%, 7-day moisture room dry, sample-2

Figure B.20 Photo after UC test for sample-2: A_c=0.01%, w=22%, 7-day moisture room dry

8.3 Appendix C: Unconfined compression test on bentonite

Unconfined compression test (UC test) were conducted on bentonite at the University of Alberta between August 2011 and September 2011. Samples with Conaid contents 0.01%, 0.025% and 0.25% mixed at various moisture contents were tested. Untreated samples without Conaid were prepared in exactly the same way as treated ones. Specimens trimmed from the compacted sample were cured in moisture room between 7 and 31 days. Typically, one or two tests were conducted for each mix. Graphs of all shear data are attached in Appendix C. Details of the UC test can be referred to Chapter 5.

Table C.1 Specimen Information for A_c=0%, w=32%, 21-day moisture room dry

Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)	Loading Rate (mm/min)
37.29	75.21	1092	159.1	28.75	0.3

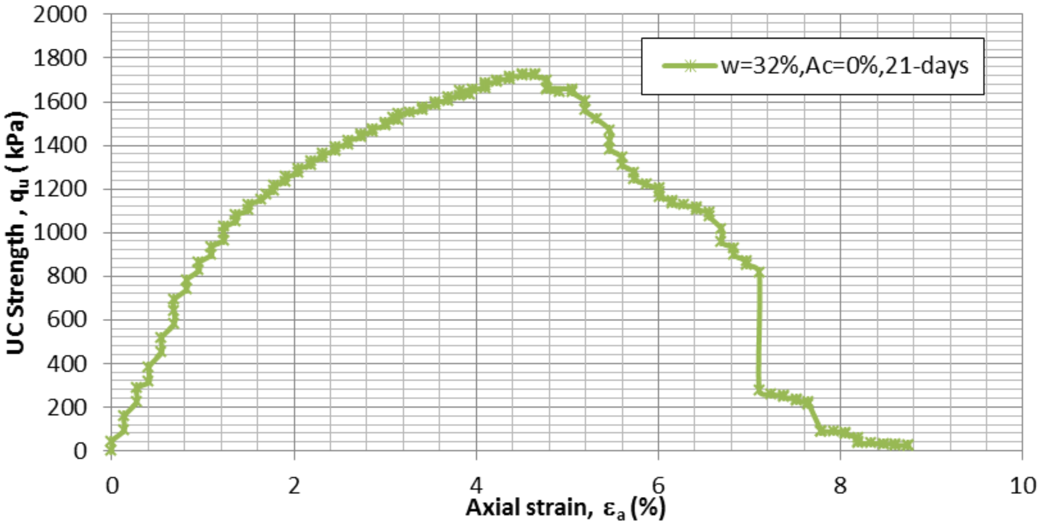


Figure C.1 q vs. ε_a for A_c=0%, w=32%, 21-day moisture room dry, sample-1

Figure C.2 Photo after UC test for sample-1: A_c=0%, w=32%, 21-day moisture room dry

Table C.2 Specimen Information for A_c=0.01%, w=32%, 21-day moisture room dry

Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)	Loading Rate (mm/min)
37.12	74.89	1082	158.1	28.49	0.3

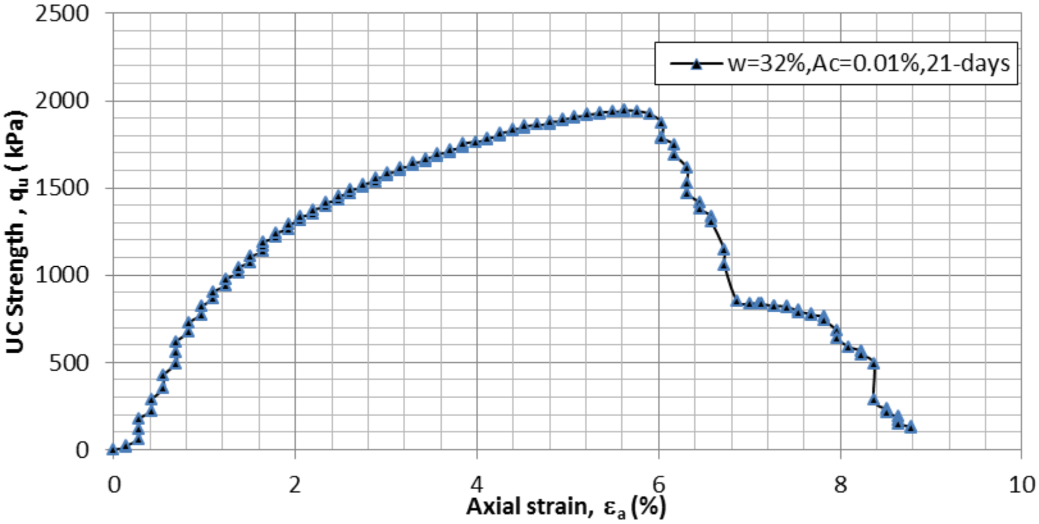


Figure C.3 q vs. ε_a for A_c=0.01%, w=32%, 21-day moisture room dry, sample-1

Figure C.4 Photo after UC test for Sample-1: A_c=0.01%, w=32%, 21-day moisture room dry

Table C.3 Specimen Information for A_c=0.025%, w=32%, 21-day moisture room dry

Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)	Loading Rate (mm/min)
38.24	74.15	1148	164.05	28.26	0.3

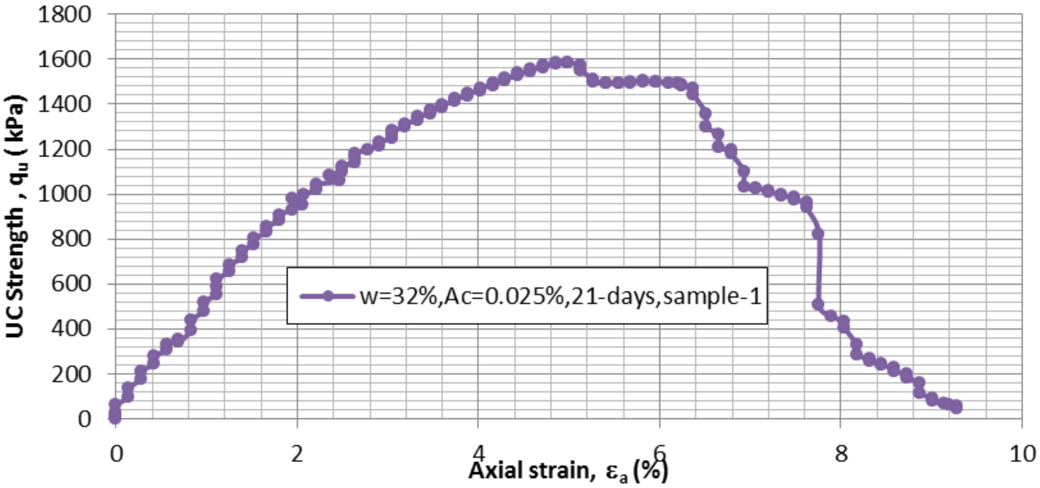


Figure C.5 q vs. ε_a for A_c=0.025%, w=32%, 21-day moisture room dry, sample-1

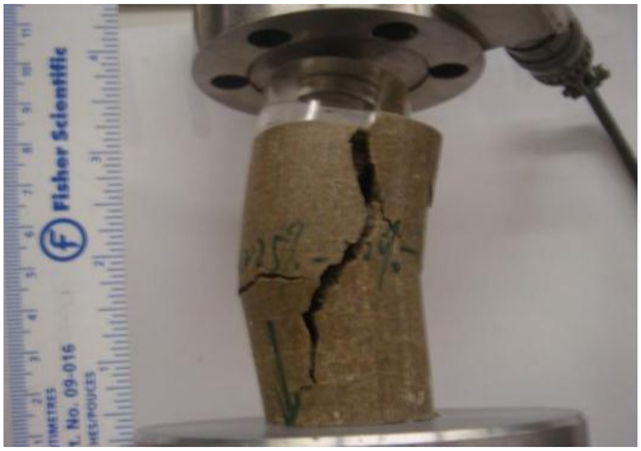


Figure C.6 Photo after UC test for Sample-1: A_c=0.025%, w=32%, 21-day moisture room dry

Table C.4 Specimen Information for A_c=0.025%, w=32%, 21-day moisture room dry

Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)	Loading Rate (mm/min)
38.18	75.34	1086	157.95	27.89	0.3

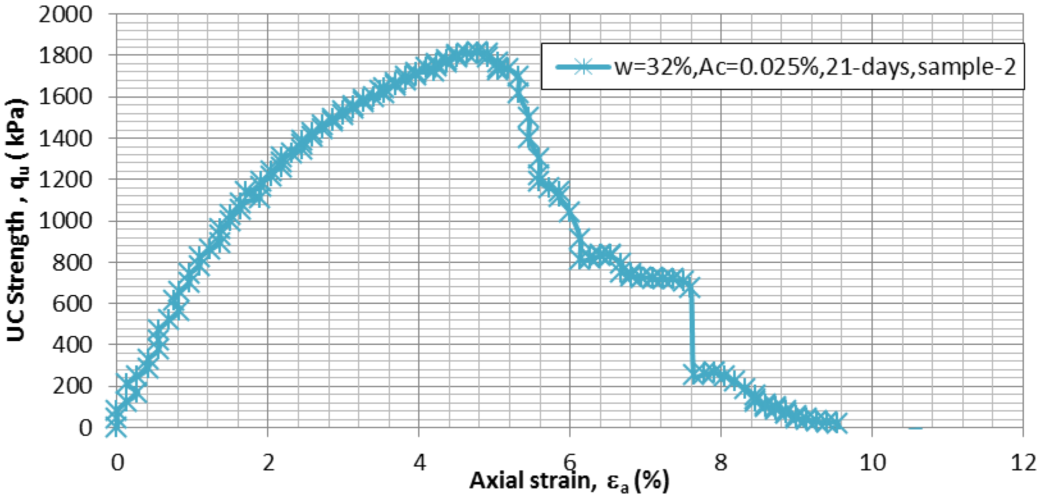


Figure C.7 q vs. ε_a for A_c=0.025%, w=32%, 21-day moisture room dry, sample-2

Figure C.8 Photo after UC test for sample-2: A_c=0.025%, w=32%, 21-day moisture room dry

Table C.5 Specimen Information for A_c=0.25%, w=32%, 21-day moisture room dry

Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)	Loading Rate (mm/min)
38.45	75.57	1161	168.5	28.23	0.3

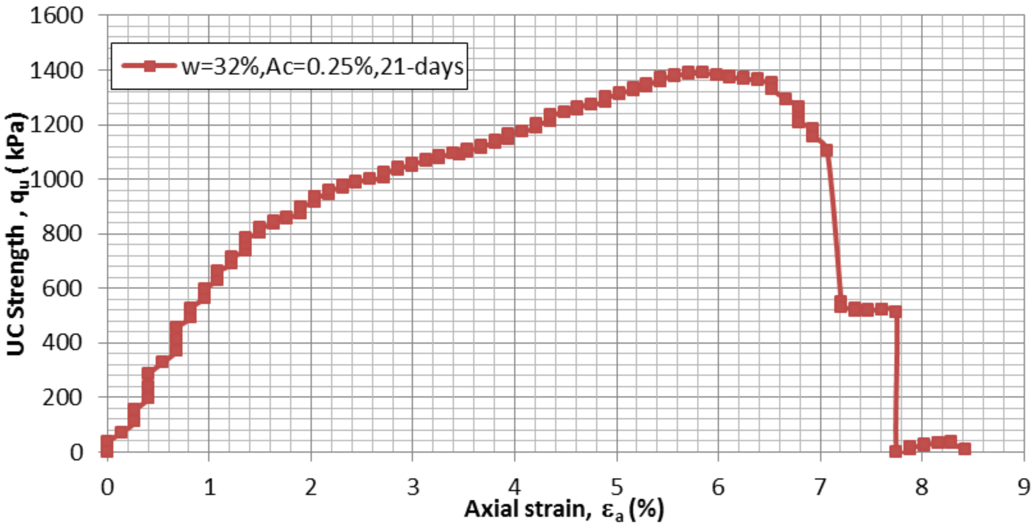


Figure C.9 q vs. ε_a for A_c=0.25%, w=32%, 21-day moisture room dry, sample-1

Figure C.10 Photo after UC test for sample-1: A_c=0.25%, w=32%, 21-day moisture room dry

Table C.6 Specimen Information for A_c=0.25%, w=40%, 31-day moisture room dry

Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)	Loading Rate (mm/min)
36.25	72.72	1032	141.5	31.52	0.3

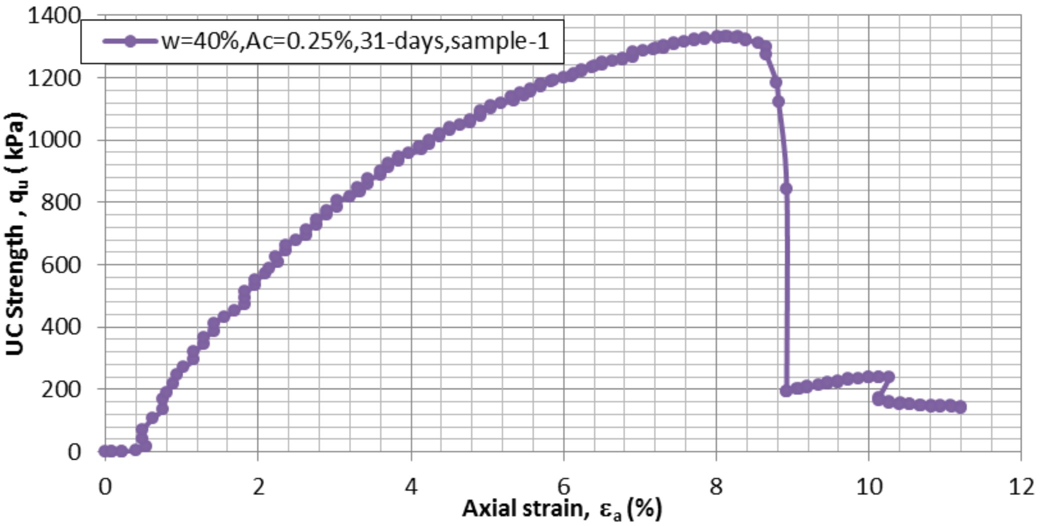


Figure C.11 q vs. ε_a for A_c=0.25%, w=40%, 31-day moisture room dry, sample-1

Figure C.12 Photo after UC test for sample-1: A_c=0.25%, w=40%, 31-day moisture room dry

Table C.7 Specimen Information for A_c=0.25%, w=36%, 31-day moisture room dry

Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)	Loading Rate (mm/min)
36.61	73.47	1053	149.6	28.31	0.3

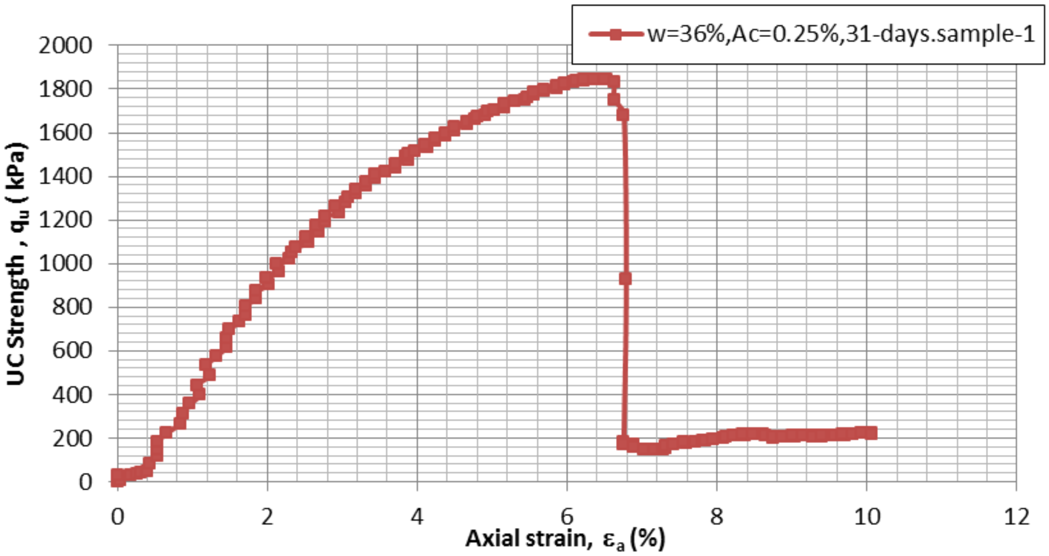


Figure C.13 q vs. ε_a for A_c=0.25%, w=36%, 31-day moisture room dry, sample-1

Figure C.14 Photo after UC test for Sample-1: A_c=0.25%, w=36%, 31-day moisture room dry

Table C.8 Specimen Information for A_c=0.25%, w=27%, 31-day moisture room dry

Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)	Loading Rate (mm/min)
37.66	75.52	1114	159.95	25.15	0.3

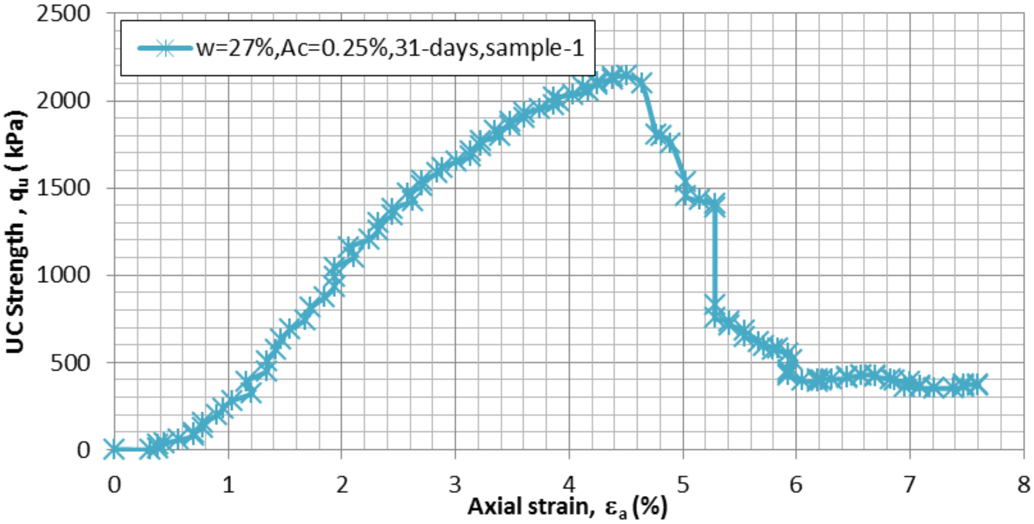


Figure C.15 q vs. ε_a for A_c=0.25%, w=27%, 31-day moisture room dry, sample-1

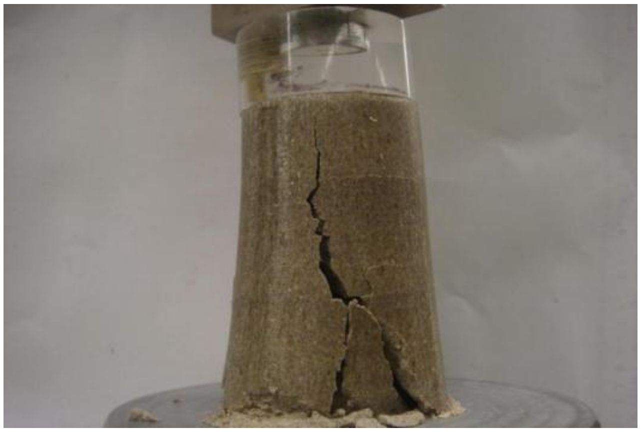


Figure C.16 Photo after UC test for sample-1: A_c=0.25%, w=27%, 31-day moisture room dry

Table C.9 Specimen Information for A_c=0%, w=27%, 31-day moisture room dry

Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)	Loading Rate (mm/min)
37.74	77.29	1118.6	163.1	25.47	0.3

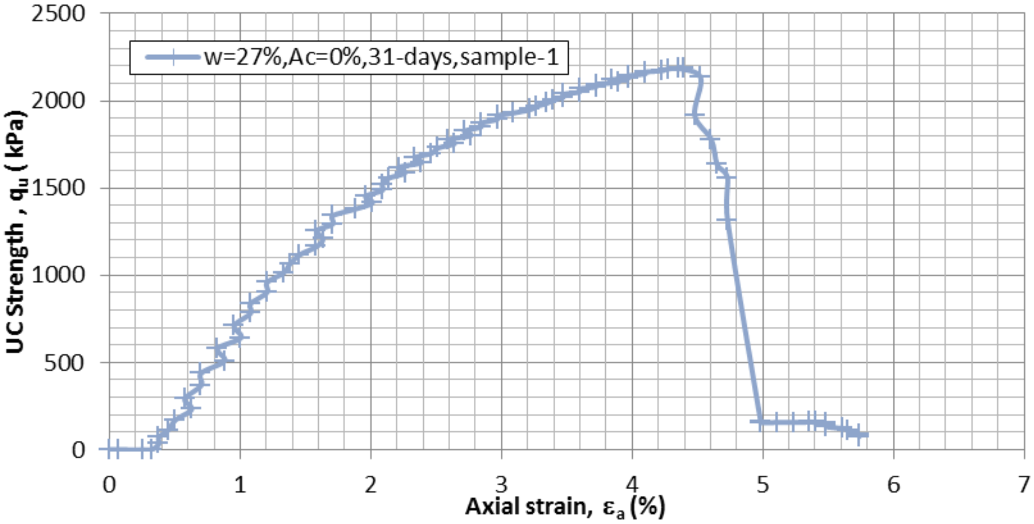


Figure C.17 q vs. ε_a for A_c=0%, w=27%, 31-day moisture room dry, sample-1

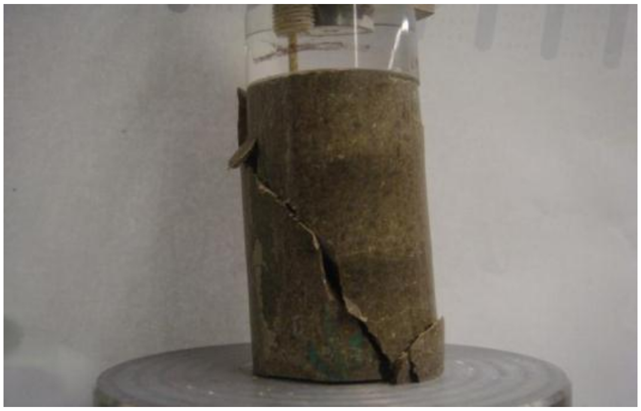


Figure C.18 Photo after UC test for sample-1: A_c=0%, w=27%, 31-day moisture room dry

Table C.10 Specimen Information for A_c=0%, w=36%, 31-day moisture room dry

Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)	Loading Rate (mm/min)
36.07	72.34	1022	147.15	27.37	0.3

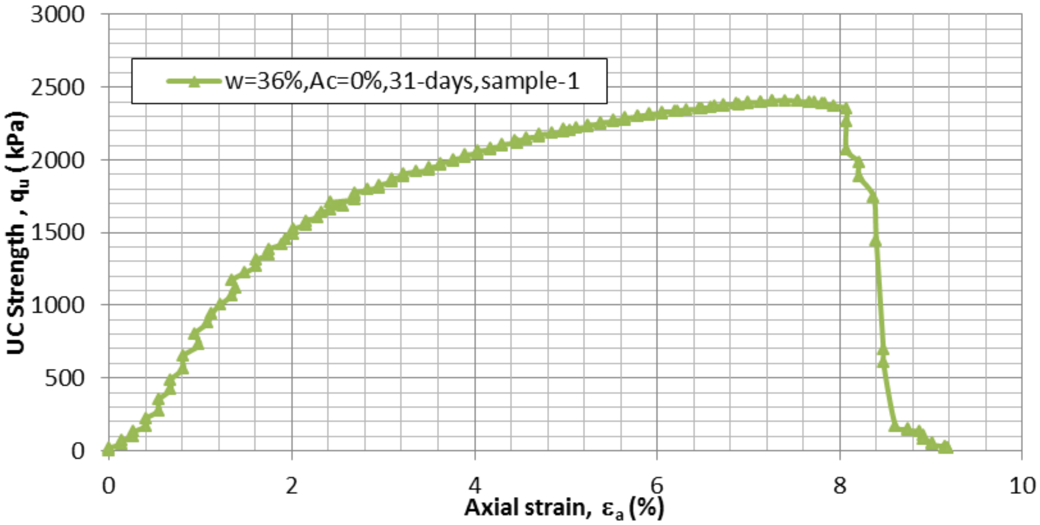


Figure C.19 q vs. ε_a for A_c=0%, w=36%, 31-day moisture room dry, sample-1

Figure C.20 Photo after UC test for sample-1: A_c=0%, w=36%, 31-day moisture room dry

Table C.11 Specimen Information for A_c=0%, w=40%, 31-day moisture room dry

Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)	Loading Rate (mm/min)
36.12	70.69	1024.5	145.4	30.39	0.3

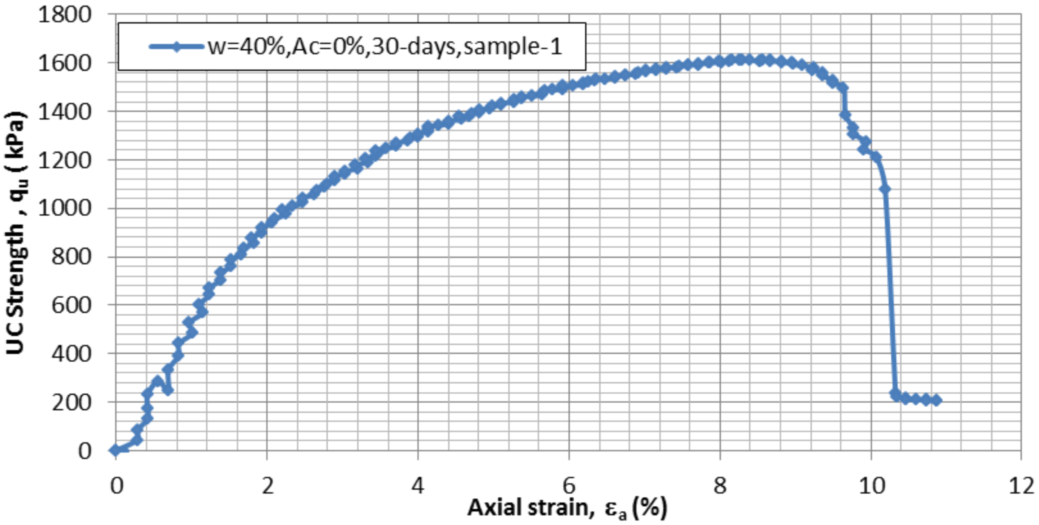


Figure C.21 q vs. ε_a for A_c=0%, w=40%, 31-day moisture room dry, sample-1

Figure C.22 Photo after UC test for sample-1: A_c=0%, w=40%, 31-day moisture room dry

8.4 Appendix D: Unconfined compression test on BK20

Unconfined compression test (UC test) were conducted on the mixture of 20% bentonite and 80% kaolinite (BK20). Samples with Conaid contents 0.25% mixed at 26% moisture content were compacted and then cured in moisture room for 7 days. Besides, Samples with Conaid contents 0.025% and 0.25% mixed at 28% moisture content were compacted and then air-dried for 7 days. In addition, time effect on samples cured for 90 days in moisture room was also examined. Control samples without Conaid were prepared and tested in the same way accordingly. Typically, one or two tests were conducted for each mix. Graphs of all shear data are attached in Appendix D. Details of the UC test can be referred to Chapter 5.

Table D.1 Specimen Information for A_c=0.25%, w=26%, 7-day moisture room dry

Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)	Loading Rate (mm/min)
37.71	75.28	1116.7	151.6	25.13	0.3

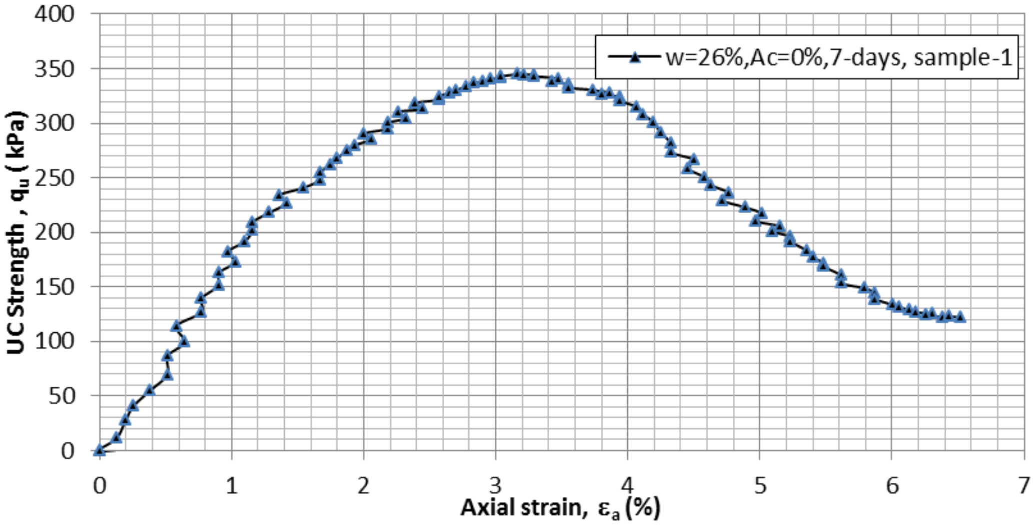


Figure D.1 q vs. ε_a for A_c=0.25%, w=26%, 7-day moisture room dry, sample-1

Figure D.2 Photo after UC test for sample-1: A_c=0.25%, w=26%, 7-day moisture room dry

Table D.2 Specimen Information for A_c=0.25%, w=26%, 7-day moisture room dry

Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)	Loading Rate (mm/min)
37.62	75.18	1111.3	148.7	25.56	0.3

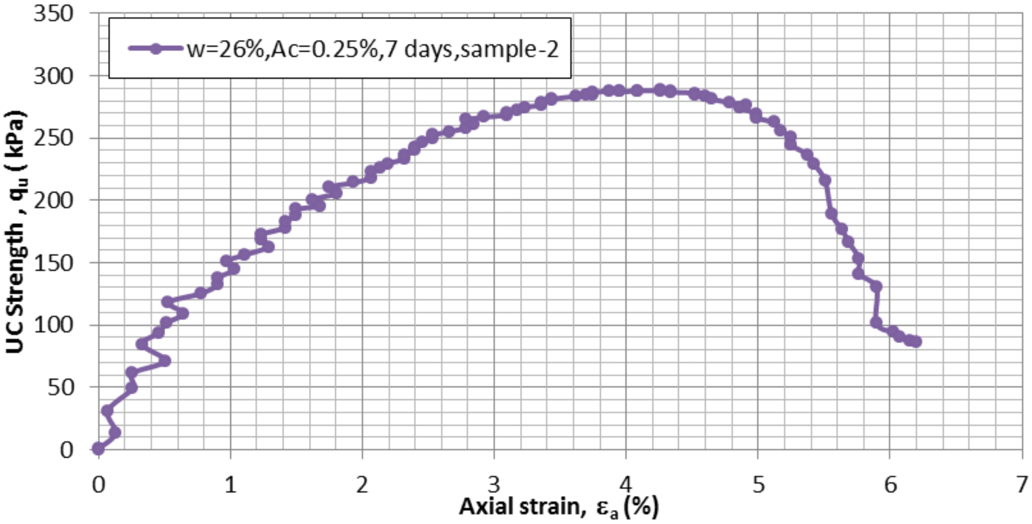


Figure D.3 q vs. ε_a for A_c=0.25%, w=26%, 7-day moisture room dry, sample-2

Figure D.4 Photo after UC test for Sample-2: A_c=0.25%, w=26%, 7-day moisture room dry

Table D.3 Specimen Information for A_c=0 %, w=26%, 7-day moisture room dry

Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)	Loading Rate (mm/min)
37.66	75.47	1114.1	148.9	25.36	0.3

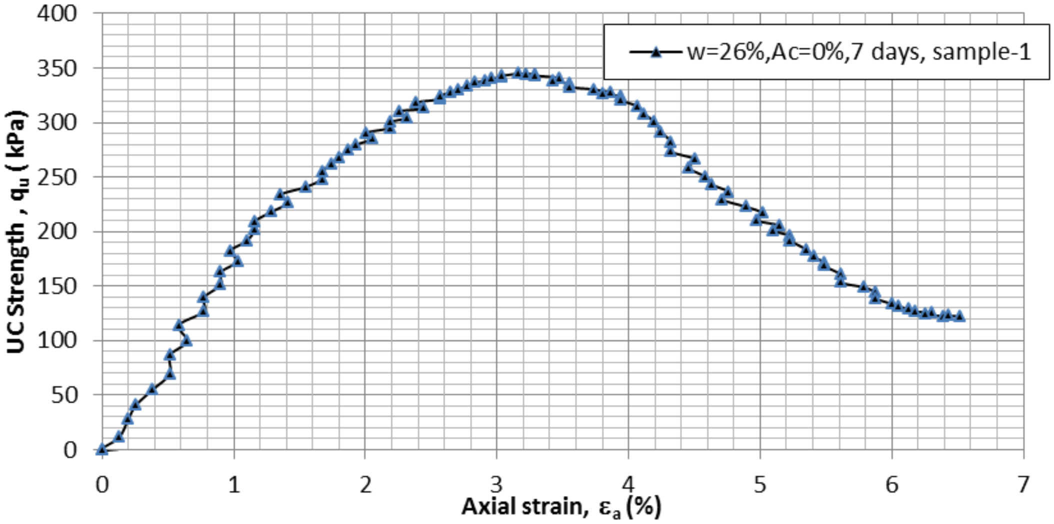


Figure D.5 q vs. ε_a for A_c=0%, w=26%, 7-day moisture room dry, sample-1

Figure D.6 Photo after UC test for sample-1: A_c=0%, w=26%, 7-day moisture room dry

Table D.4 Specimen Information for A_c=0 %, w=26%, 7-day moisture room dry

Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)	Loading Rate (mm/min)
37.76	75.75	1119.8	153.55	25.18	0.3

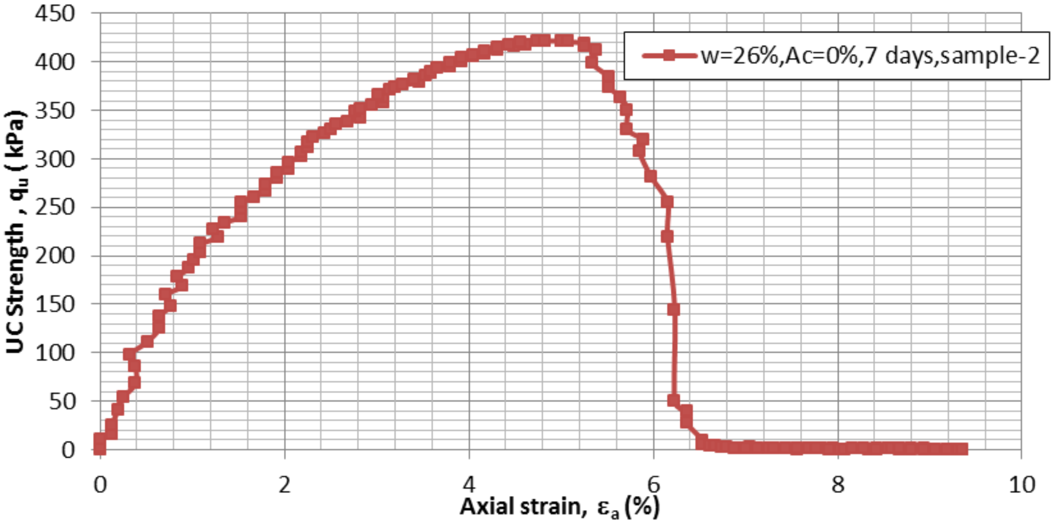


Figure D.7 q vs. ε_a for A_c=0%, w=26%, 7-day moisture room dry, sample-2

Figure D.8 Photo after UC test for sample-2: A_c=0%, w=26%, 7-day moisture room dry

Table D.5 Specimen Information for A_c=0 %, w=28%, 7-day air dry

Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)	Loading Rate (mm/min)
36.72	73.31	1058.8	124.6	3.24	0.3

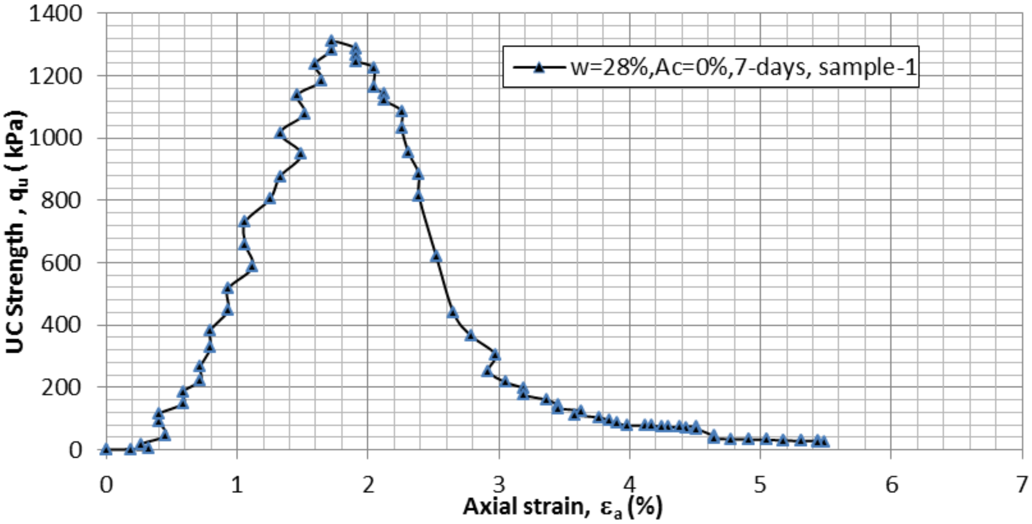


Figure D.9 q vs. ε_a for A_c=0%, w=28%, 7-day air dry, sample-1

Figure D.10 Photo after UC test for sample-1: A_c=0%, w=28%, 7-day air dry

Table D.6 Specimen Information for A_c=0 %, w=28%, 7-day air dry, sample-2

Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)	Loading Rate (mm/min)
36.73	73.26	1059.4	127.15	3.24	0.3

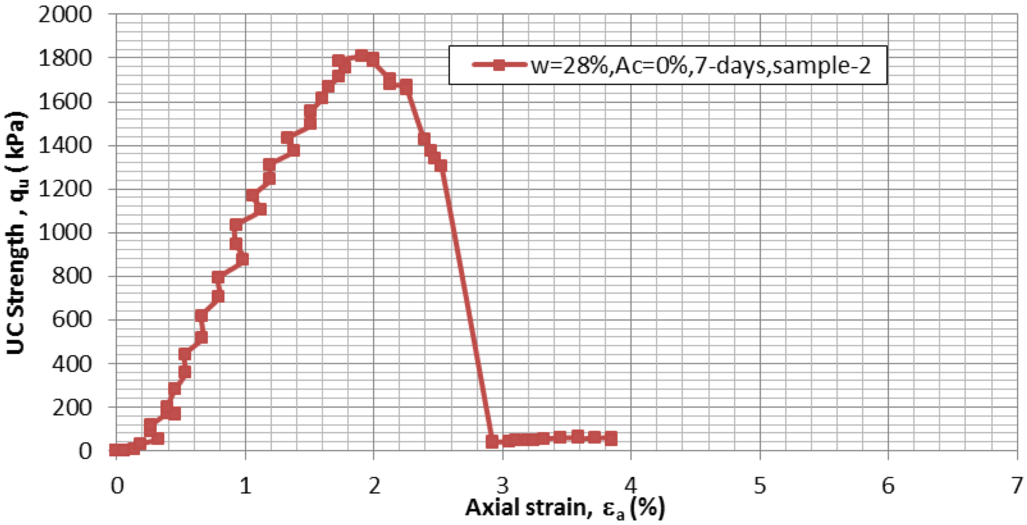


Figure D.11 q vs. ε_a for A_c=0%, w=28%, 7-day air dry, sample-2

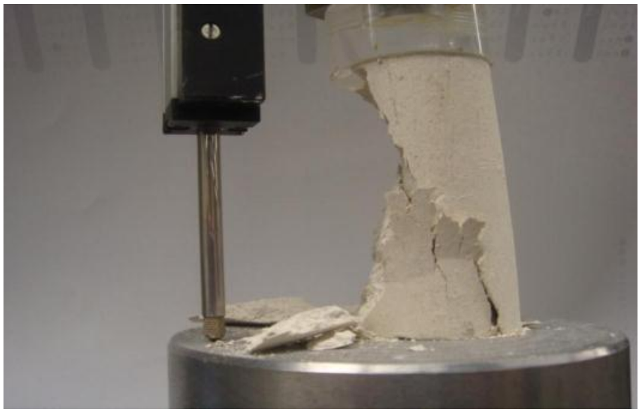


Figure D.12 Photo after UC test for sample-2: A_c=0%, w=28%, 7-day air dry

Table D.7 Specimen Information for A_c=0.025 %, w=28%, 7-day air dry, sample-1

Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)	Loading Rate (mm/min)
36.56	72.31	1049.6	131	4.77	0.3

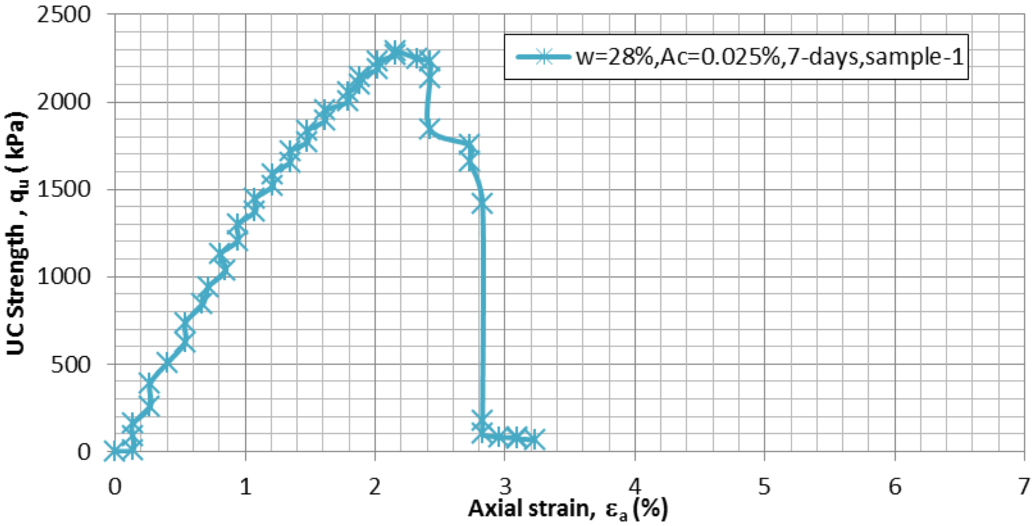


Figure D.13 q vs. ε_a for A_c=0.025%, w=28%, 7-day air dry, sample-1

Figure D.14 Photo after UC test for sample-1: A_c=0.025%, w=28%, 7-day air dry

Table D.8 Specimen Information for A_c=0.025 %, w=28%, 7-day air dry, sample-2

Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)	Loading Rate (mm/min)
36.35	71.33	1038	128.8	4.54	0.3

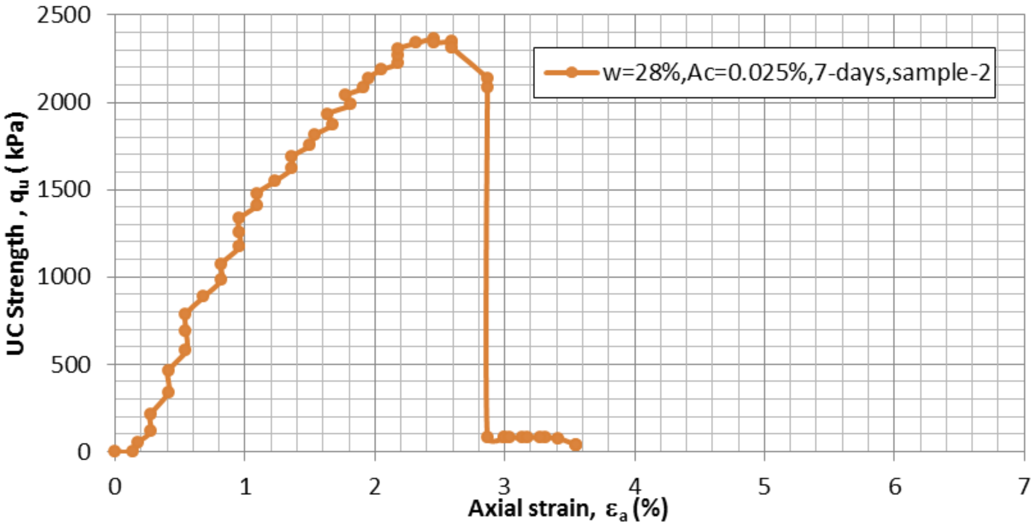


Figure D.15 q vs. ε_a for A_c=0.025%, w=28%, 7-day air dry, sample-2

Figure D.16 Photo after UC test for sample-2: A_c=0.025%, w=28%, 7-day air dry

Table D.9 Specimen Information for A_c=0.25 %, w=28%, 7-day air dry, sample-1

Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)	Loading Rate (mm/min)
36.43	72.97	1042	128	3.05	0.3

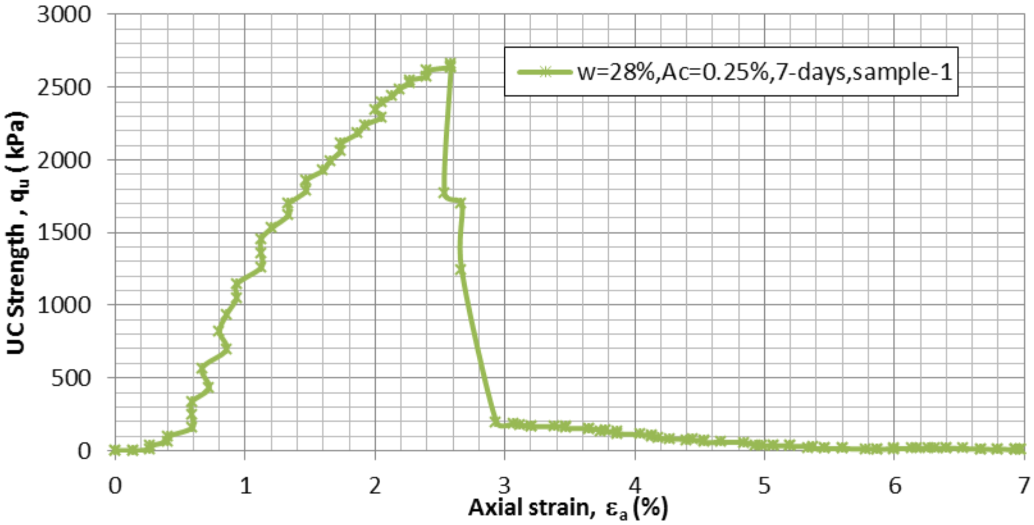


Figure D.17 q vs. ε_a for A_c=0.25%, w=28%, 7-day air dry, sample-1

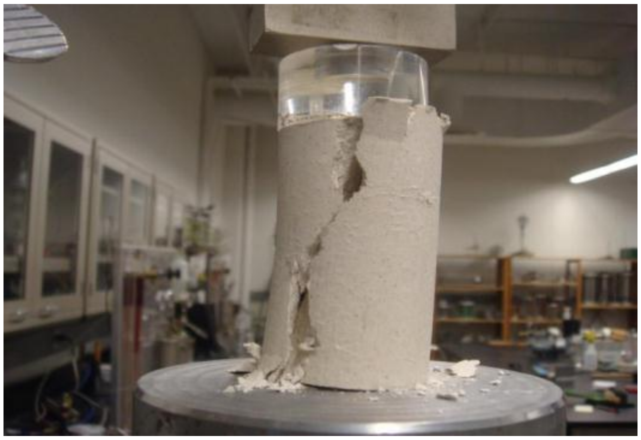


Figure D.18 Photo after UC test for sample-1: A_c=0.25%, w=28%, 7-day air dry

Table D.10 Specimen Information for A_c=0.25 %, w=28%, 7-day air dry, sample-2

Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)	Loading Rate (mm/min)
36.6	73.31	1052	128.1	3.15	0.3

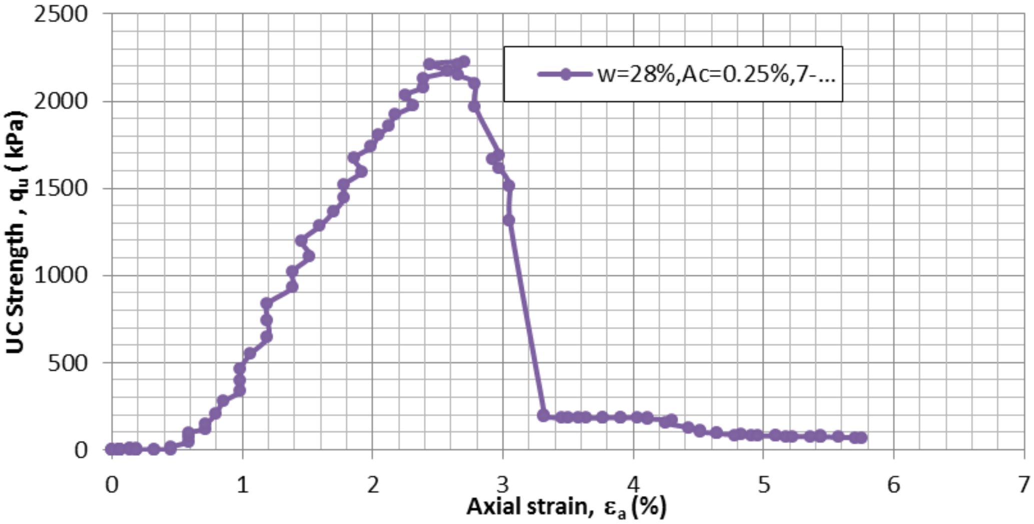


Figure D.19 q vs. ε_a for A_c=0.25%, w=28%, 7-day air dry, sample-2

Figure D.20 Photo after UC test for sample-2: A_c=0.25%, w=28%, 7-day air dry

Table D.11 Specimen Information for A_c=0 %, w=24%, 90-day air dry, sample-1

Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)	Loading Rate (mm/min)
37.38	74.59	1097.2	123.35	2.84	0.3

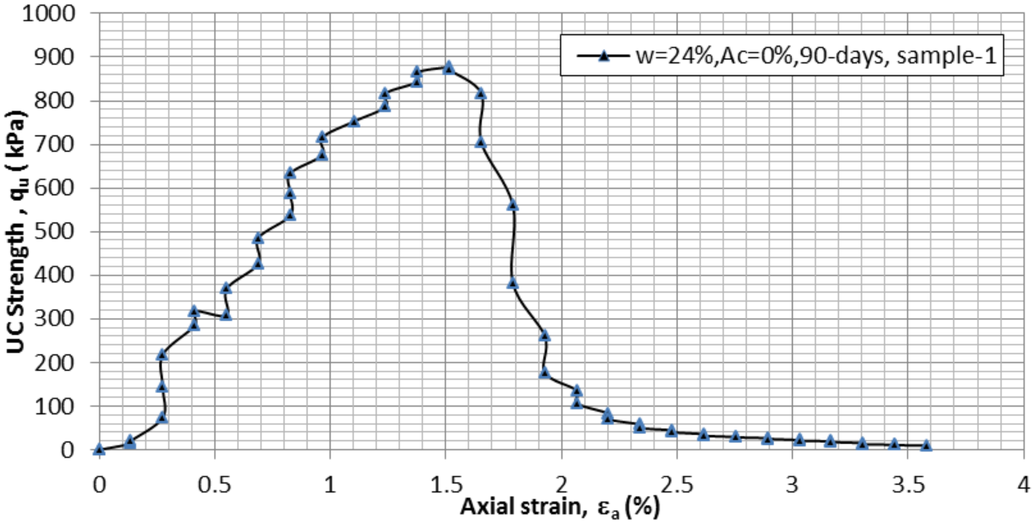


Figure D.21 q vs. ε_a for A_c=0 %, w=24%, 90-day air dry, sample-1

Figure D.22 Photo after UC test for sample-1: A_c=0%, w=28%, 90-day air dry

Table D.12 Specimen Information for A_c=0 %, w=24%, 90-day air dry, sample-2

Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)	Loading Rate (mm/min)
37.12	75.24	1082	121.1	2.86	0.3

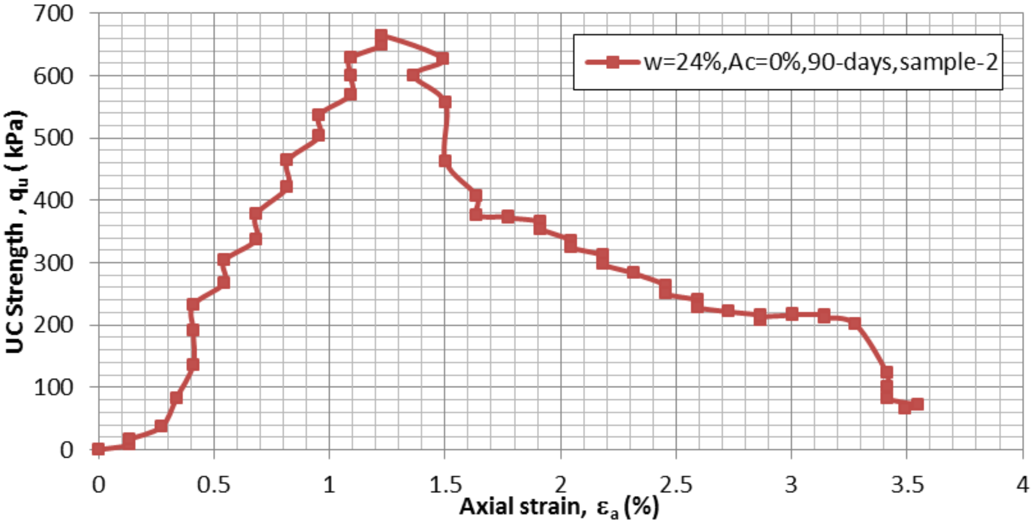


Figure D.23 q vs. ε_a for A_c=0 %, w=24%, 90-day air dry, sample-2

Figure D.24 Photo after UC test for sample-2: A_c=0%, w=28%, 90-day air dry

Table D.13 Specimen Information for A_c=0.25%, w=24%, 90-day air dry, sample-1

Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)	Loading Rate (mm/min)
37.15	71.54	1083.8	115.1	2.84	0.3

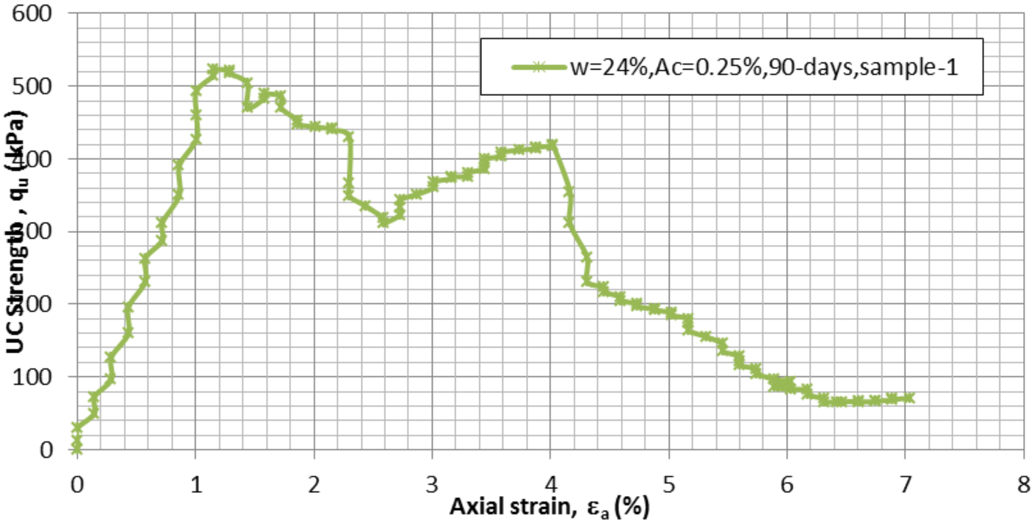


Figure D.25 q vs. ε_a for A_c=0.25%, w=24%, 90-day air dry, sample-1

Figure D.26 Photo after UC test for sample-1: A_c=0.25%, w=24%, 90-day air dry

Table D.14 Specimen Information for A_c=0.25%, w=24%, 90-day air dry, sample-2

Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)	Loading Rate (mm/min)
37.20	74.53	1086.7	122.3	2.88	0.3

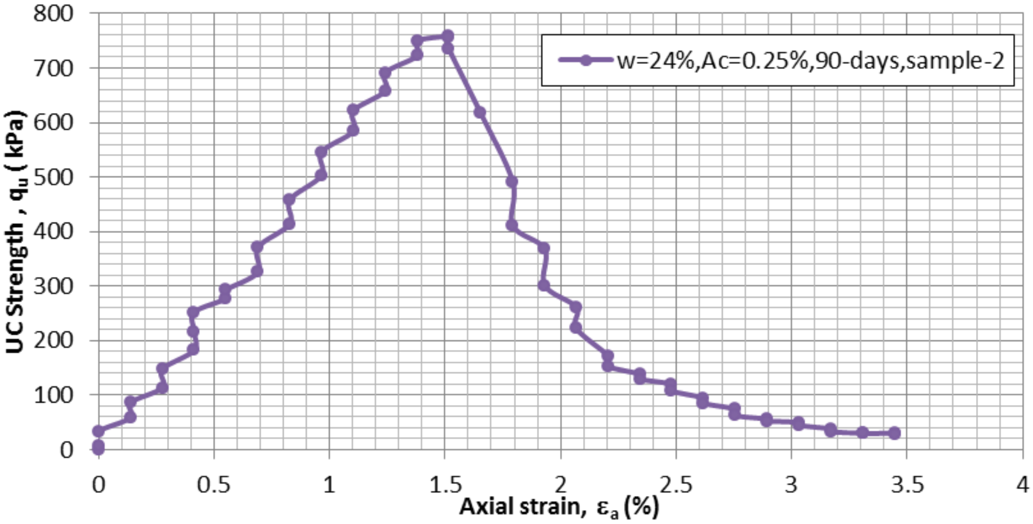


Figure D.27 q vs. ε_a for A_c=0.25%, w=24%, 90-day air dry, sample-2

Figure D.28 Photo after UC test for sample-2: A_c=0.25%, w=24%, 90-day air dry

8.5 Appendix E: Unconfined compression test on Devon silt

Unconfined compression test (UC test) were conducted on Devon silt. Samples with Conaid contents 0.05% were compacted at OMC and wet of optimum and then cured in moisture room for 12 and 21 days, respectively. Control samples were also prepared accordingly. Graphs of all shear data are attached in Appendix E. Details of the UC test can be referred to Chapter 5.

Table E.1 Specimen Information for A_c=0 %, w=16.5%, 12-day moisture room dry

Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)	Loading Rate (mm/min)
36.95	74.39	1072.5	172.65	10.92	0.3

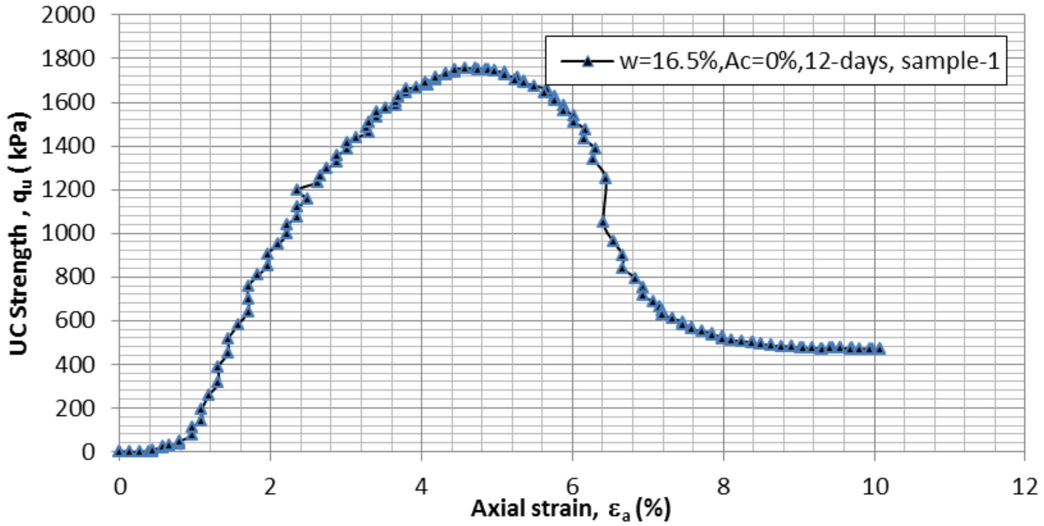


Figure E.1 q vs. ε_a for A_c=0%, w=16.5%, 12-day moisture room dry, sample-1

Figure E.2 Photo after UC test for sample-1: A_c=0%, w=16.5%, 12-day moisture room dry

Table E.2 Specimen Information for A_c=0 %, w=16.5%, 12-day moisture room dry

Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)	Loading Rate (mm/min)
36.75	74.66	1060.7	174.45	10.39	0.3

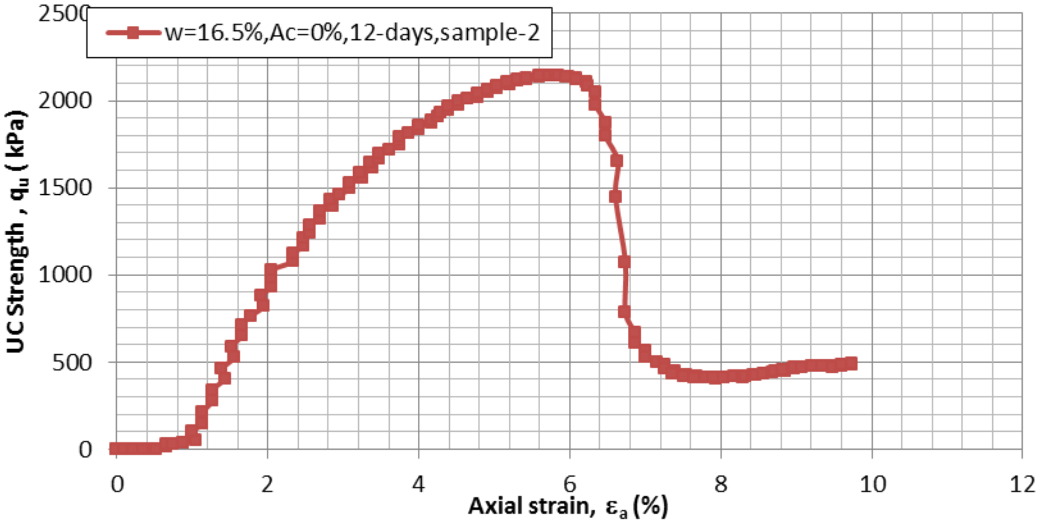


Figure E.3 q vs. ε_a for A_c=0%, w=16.5%, 12-day moisture room dry, sample-2

Figure E.4 Photo after UC test for sample-2: A_c=0%, w=16.5%, 12-day moisture room dry

Table E.3 Specimen for A_c=0.05%, w=16.5%, 12-day moisture room dry

Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)	Loading Rate (mm/min)
36.86	74.50	1067.1	170.3	9.18	0.3

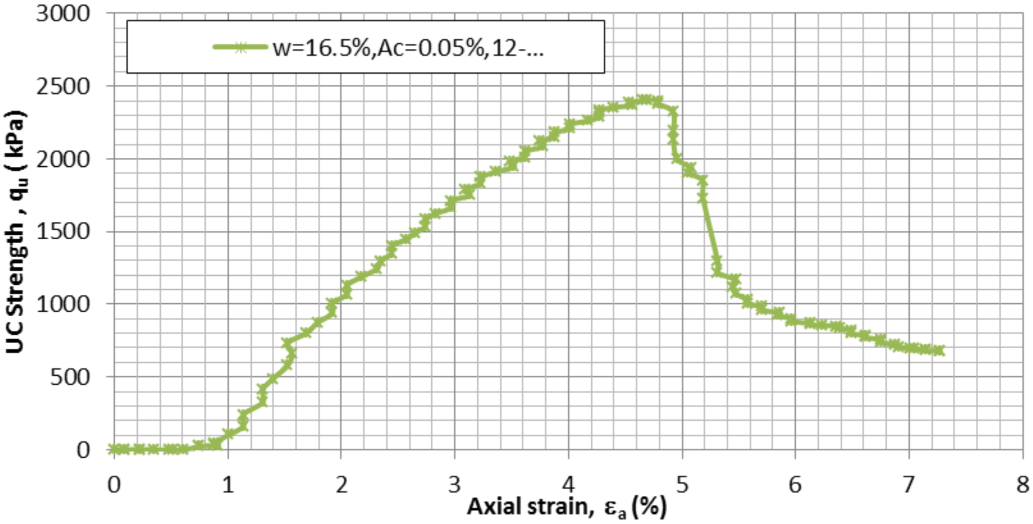


Figure E.5 q vs. ε_a for A_c=0.05%, w=16.5%, 12-day moisture room dry, sample-1

Figure E.6 Photo after UC test for sample-1: A_c=0.05%, w=16.5%, 12-day moisture room dry

Table E.4 Specimen for A_c=0.05%, w=16.5%, 12-day moisture room dry

Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)	Loading Rate (mm/min)
37.24	74.63	1089.4	168.55	9.61	0.3

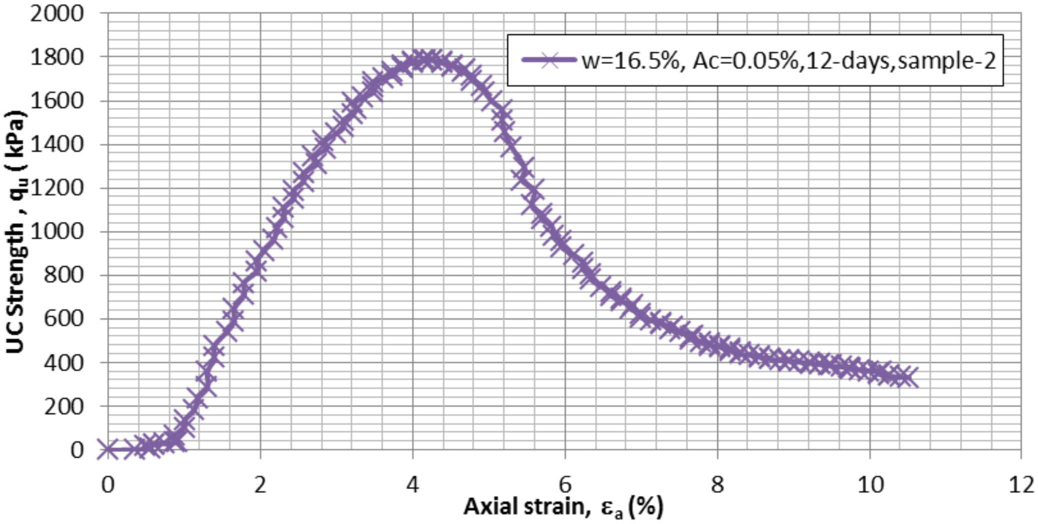


Figure E.7 q vs. ε_a for A_c=0.05%, w=16.5%, 12-day moisture room dry, sample-2

Figure E.8 Photo after UC test for Sample-2: A_c=0.05%, w=16.5%, 12-day moisture room dry

Table E.5 Specimen for A_c=0%, w=19%, 21-day moisture room dry

Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)	Loading Rate (mm/min)
36.35	72.64	1038	164.85	9.84	0.3

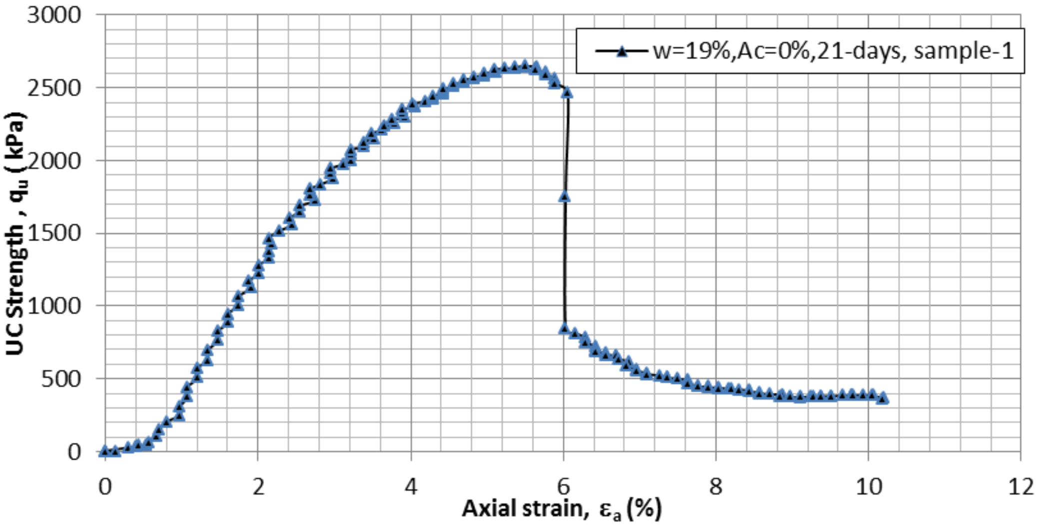


Figure E.9 q vs. ε_a for A_c=0%, w=19%, 21-day moisture room dry, sample-1

Figure E.10 Photo after UC test for sample-1: A_c=0%, w=19%, 21-day moisture room dry

Table E.6 Specimen Information for A_c=0 %, w=19%, 21-day moisture room dry

Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)	Loading Rate (mm/min)
34.43	71.48	1042.5	162.9	8.81	0.3

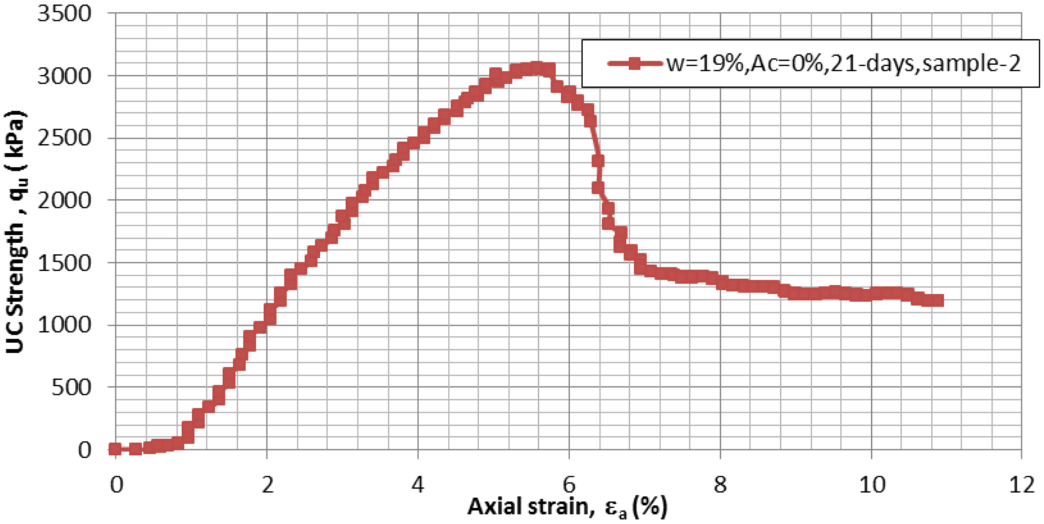


Figure E.11 q vs. ε_a for A_c=0%, w=19%, 21-day moisture room dry, sample-2

Figure E.12 Photo after UC test for sample-2: A_c=0%, w=19%, 21-day moisture room dry

Table E.7 Specimen for A_c=0.05%, w=19%, 21-day moisture room dry

Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)	Loading Rate (mm/min)
36.18	72.79	1028.3	162.8	9.44	0.3

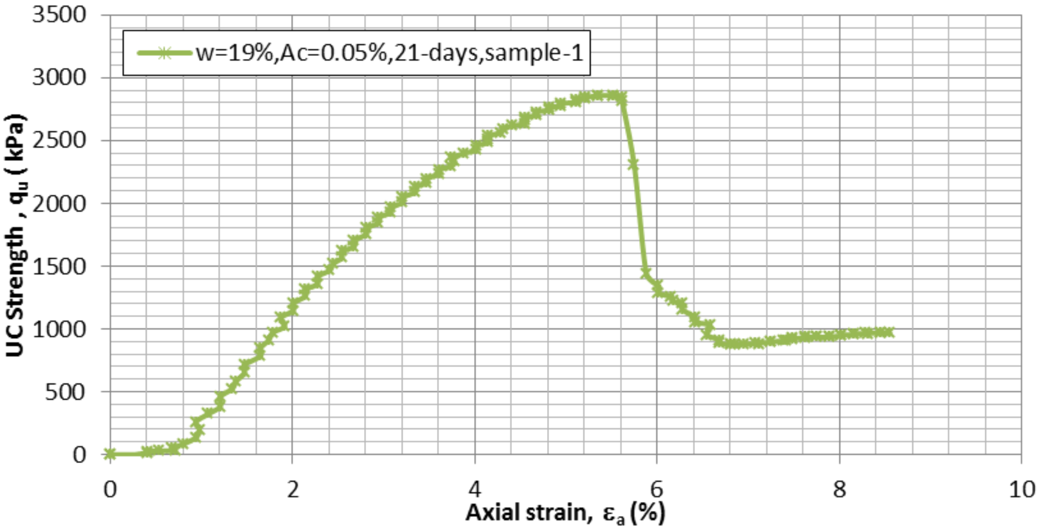


Figure E.13 q vs. ε_a for A_c=0.05%, w=19%, 21-day moisture room dry, sample-1

Figure E.14 Photo after UC test for sample-1: A_c=0.05%, w=19%, 21-day moisture room dry

8.6 Appendix F: Unconfined compression test on Anthony Henday clay

Unconfined compression test (UC test) were conducted on the silt clay collected from Anthony Henday Drive, Edmonton. Samples with Conaid contents 0.025% was compacted at OMC and then as-molded specimens were sheared or cured in moisture room for 5 days before shearing. Control samples without Conaid were also prepared completely the same way with treated ones. Graphs of all shear data are attached in Appendix E. Details of the UC test can be referred to Chapter 5.

Table F.1 Specimen Information for A_c=0 %, w=17%, 5-day moisture room dry

Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)	Loading Rate (mm/min)
37.32	74.85	1093.7	167.8	12.89	0.3

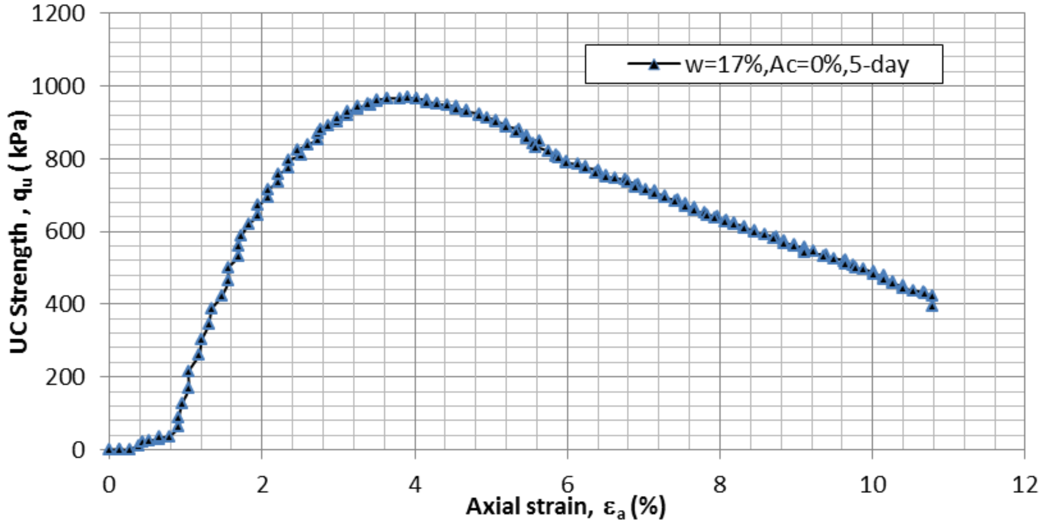


Figure F.1 q vs. ε_a for A_c=0%, w=17%, 5-day moisture room dry, sample-2

Figure F.2 Photo after UC test for sample-2: A_c=0%, w=17%, 5-day moisture room dry

Table F.2 Specimen Information for A_c=0.025%, w=17%, 5-day moisture room dry

Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)	Loading Rate (mm/min)
37.15	75.78	1084.1	172.1	12.94	0.3

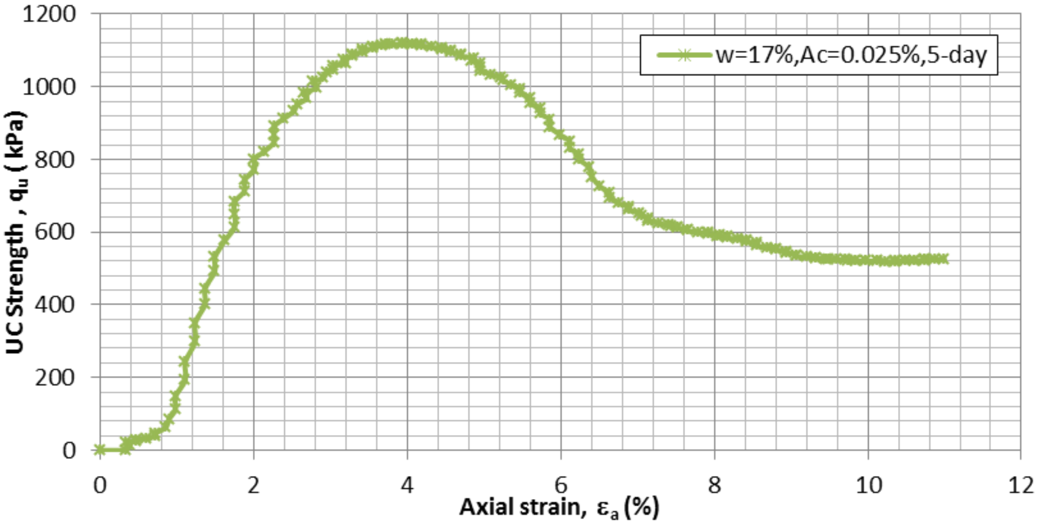


Figure F.3 q vs. ε_a for A_c=0.025%, w=17%, 5-day moisture room dry, sample-2

Figure F.4 Photo after UC test for sample-2: A_c=0.025%, w=17%, 5-day moisture room dry

Table F.3 Specimen Information for A_c=0%, w=17%, 0-day (as-molded)

Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)	Loading Rate (mm/min)
38.04	75.9	1136.5	177.78	15.71	0.3

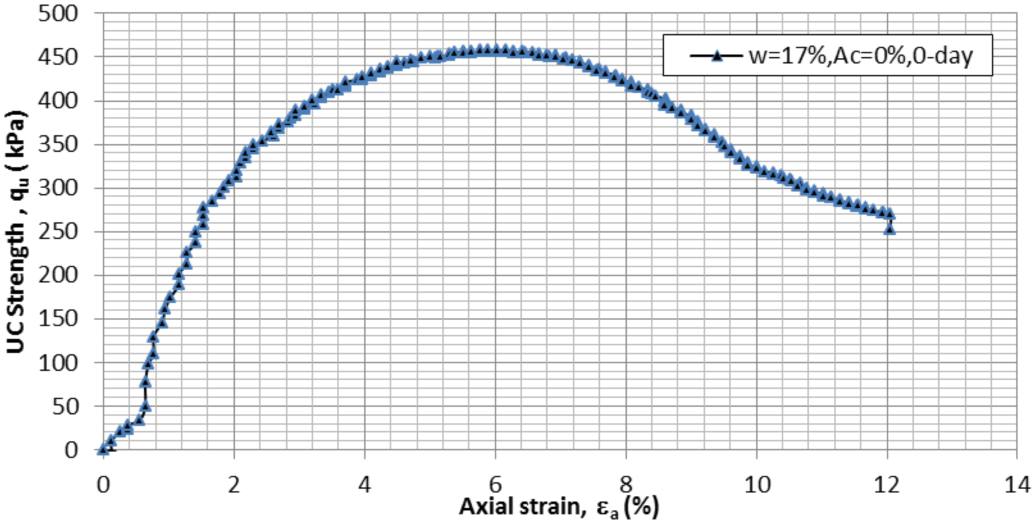


Figure F.5 q vs. ε_a for A_c=0%, w=17%, 0-day (as-molded), sample-1

Figure F.6 Photo after UC test for sample-1: A_c=0%, w=17%, 0-day (as-molded)

Table F.4 Specimen Information for A_c=0.025%, w=17%, 0-day (as-molded)

Diameter (mm)	Length (mm)	Area (mm^2)	Mass (g)	Moisture Content (%)	Loading Rate (mm/min)
38.04	75.9	1136.5	180.79	15.75	0.3

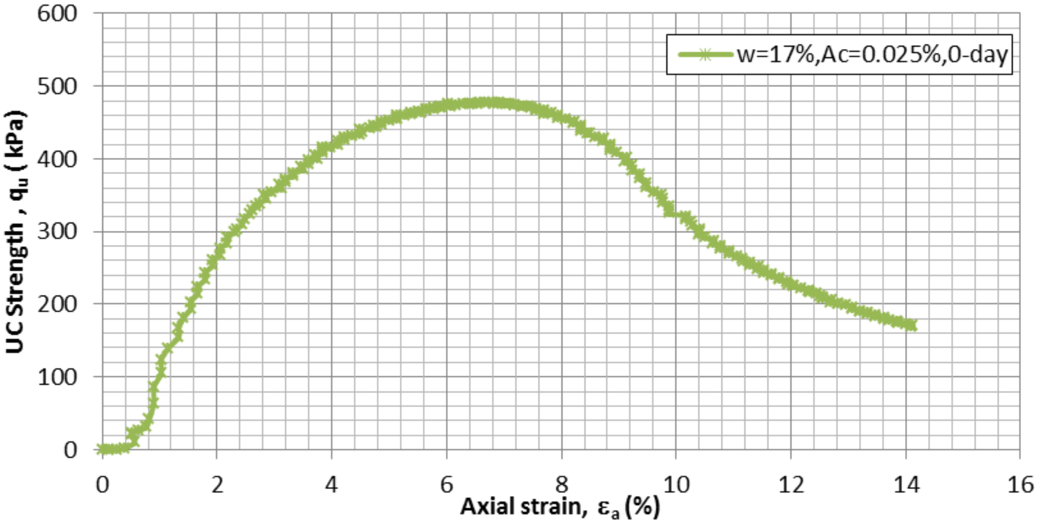


Figure F.7 q vs. ε_a for A_c=0.025%, w=17%, 0-day (as-molded), sample-1

Figure F.8 Photo after UC test for sample-1: A_c=0.025%, w=17%, 0-day (as-molded)

8.7 Appendix G: Photomicrographs of untreated and treated test soils

SEM photomicrographs were taken at the Scanning Electron Microscope Laboratory, Department of Earth and Atmospheric Sciences, University of Alberta on June 29 and June 30, 2012. A Zeiss EVO SEM was used in capturing these images, which provided magnification from 500x to 30,000x for all the samples.

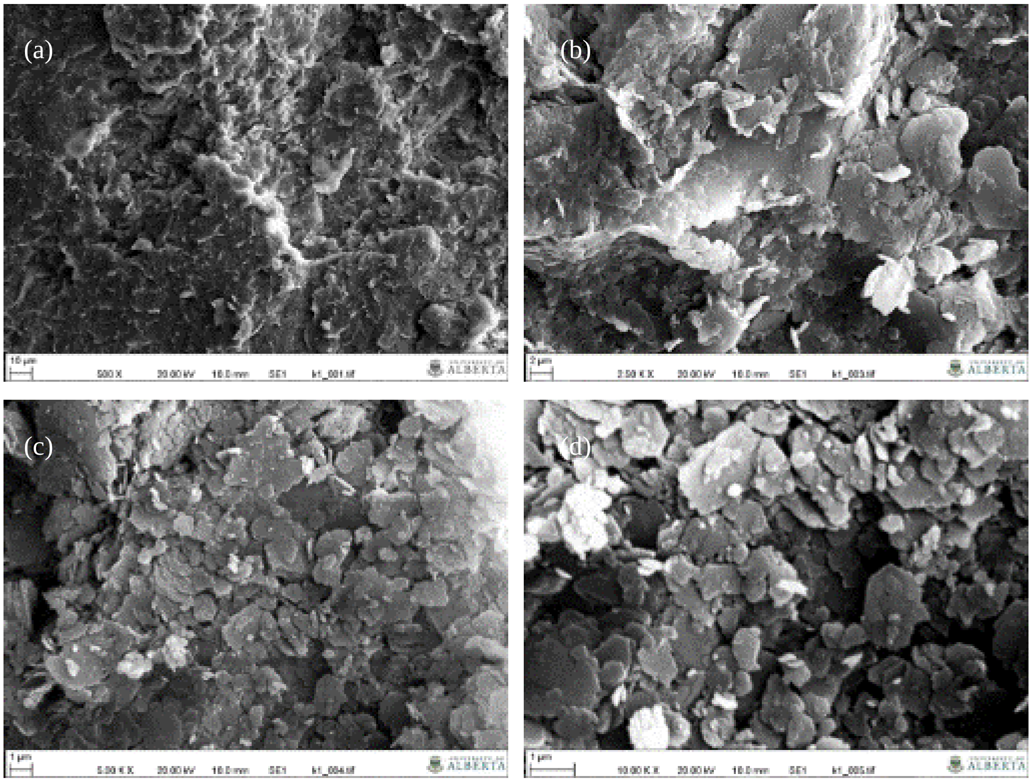
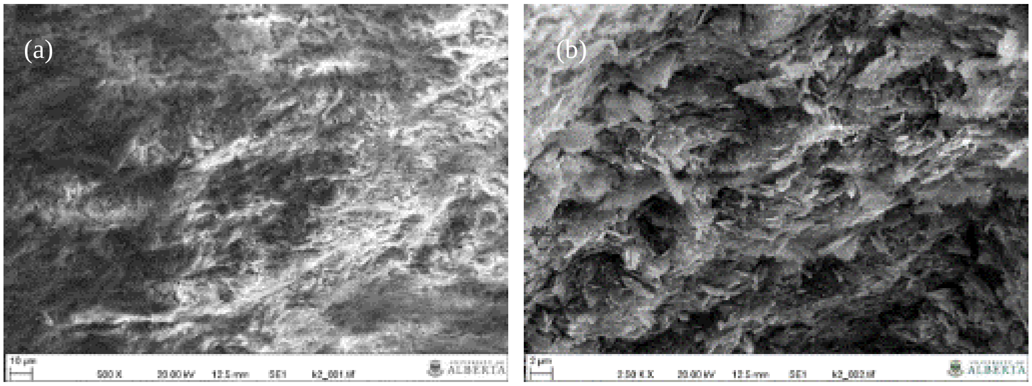


Figure G.1 Photomicrographs of kaolinite in drained triaxial test at w=24%, A_c=0%, T_c=10 days, (a) M~500X; (b) M~2,500X; (c) M~5, 000X; (d) M~10, 000X.



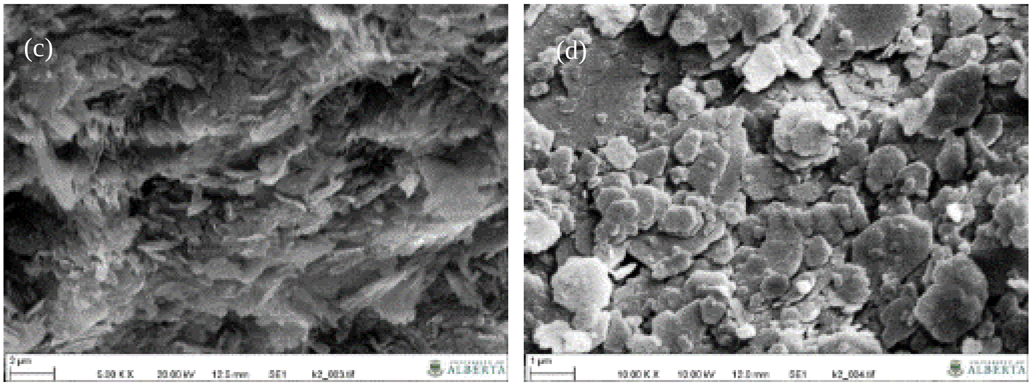


Figure G.2 Photomicrographs of kaolinite in drained triaxial test at w=24%, A_c=0.25%, T_c=17 days, (a) M~500X; (b) M~2,500X; (c) M~5, 000X; (d) M~10, 000X.

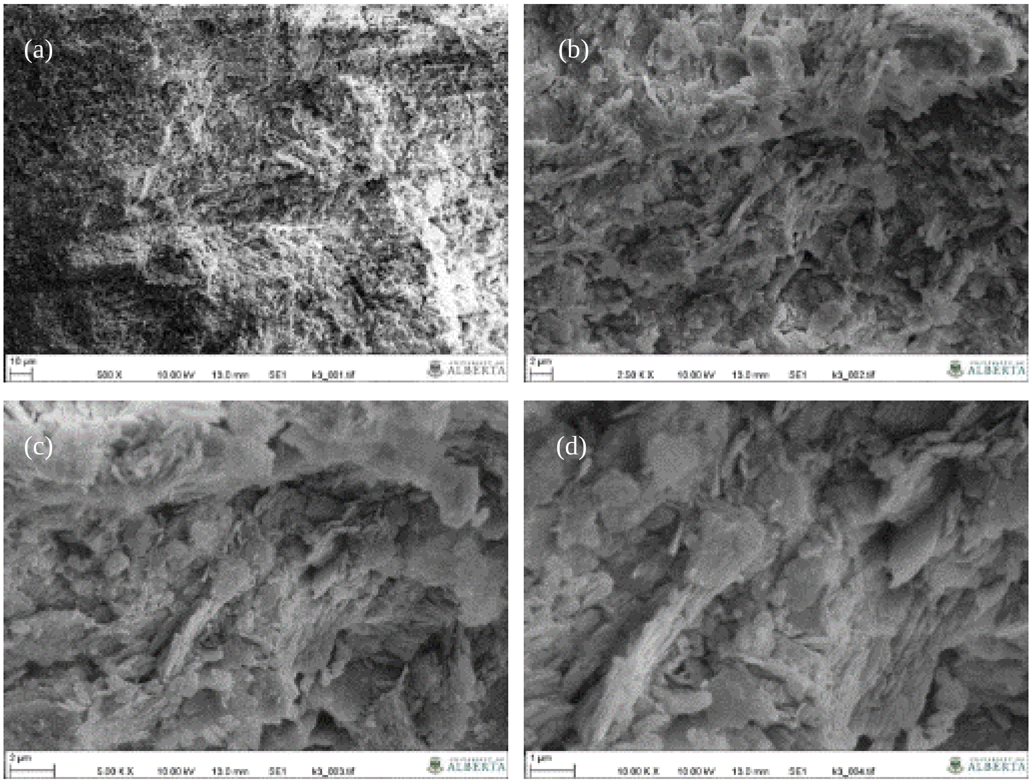
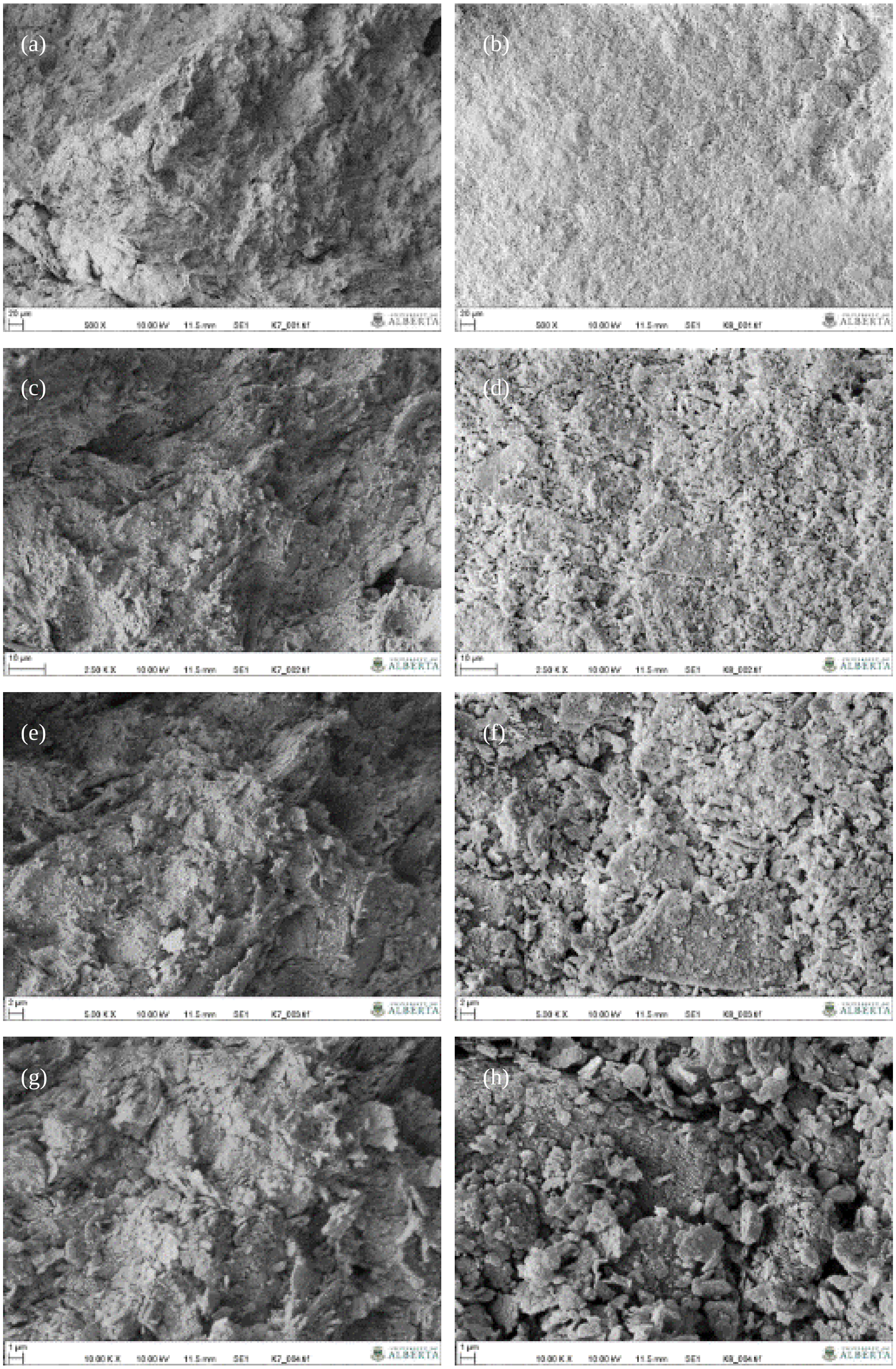


Figure G.3 Photomicrographs of kaolinite in drained triaxial test at w=24%, A_c=0.5%, T_c=16 days, (a) M~500X; (b) M~2,500X; (c) M~5, 000X; (d) M~10, 000X.


(a)
20 µm 500 X 10.00 kV 11.5 mm SE1 K7_001.tif
(b)
20 µm 500 X 10.00 kV 11.5 mm SE1 K8_001.tif
(c)
10 µm 2.50 K X 10.00 kV 11.5 mm SE1 K7_002.tif
(d)
10 µm 2.50 K X 10.00 kV 11.5 mm SE1 K8_002.tif
(e)
2 µm 5.00 K X 10.00 kV 11.5 mm SE1 K7_003.tif
(f)
2 µm 5.00 K X 10.00 kV 11.5 mm SE1 K8_003.tif
(g)
1 µm 10.00 K X 10.00 kV 11.5 mm SE1 K7_004.tif
(h)
1 µm 10.00 K X 10.00 kV 11.5 mm SE1 K8_004.tif

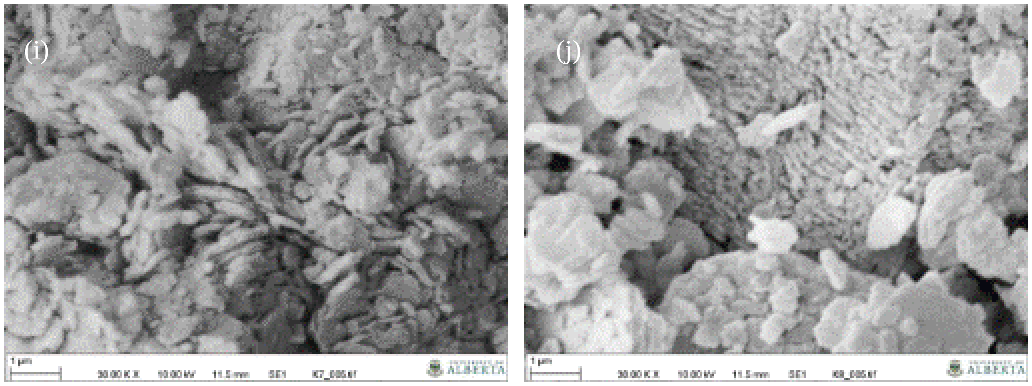
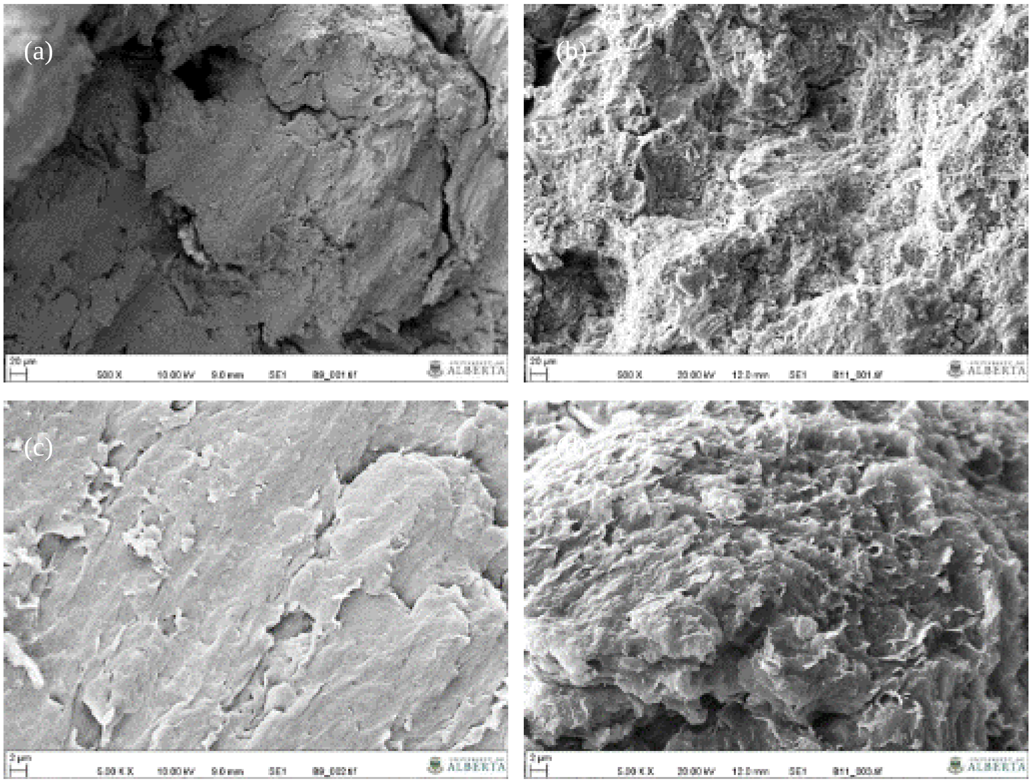


Figure G.4 Photomicrographs of kaolinite in unconfined compression test at w=24%, T_c=7 days, (a) A_c=0%, M~500X; (b) A_c=0.25%, M~500X; (c) A_c=0%, M~2,500X; (d) A_c=0.25%, M~2,500X; (e) A_c=0%, M~5, 000X; (f) A_c=0.25%, M~5, 000X; (g) A_c=0%, M~10, 000X; (h) A_c=0.25%, M~10, 000X; (i) A_c=0%, M~30, 000X; (j) A_c=0.25%, M~30, 000X.



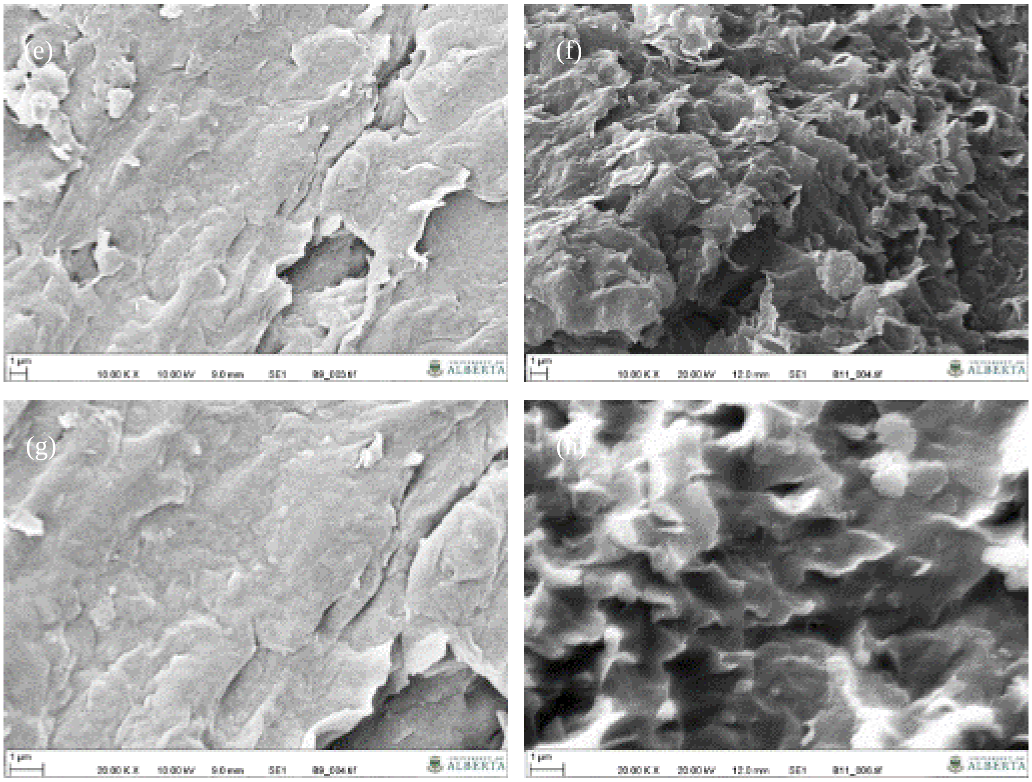
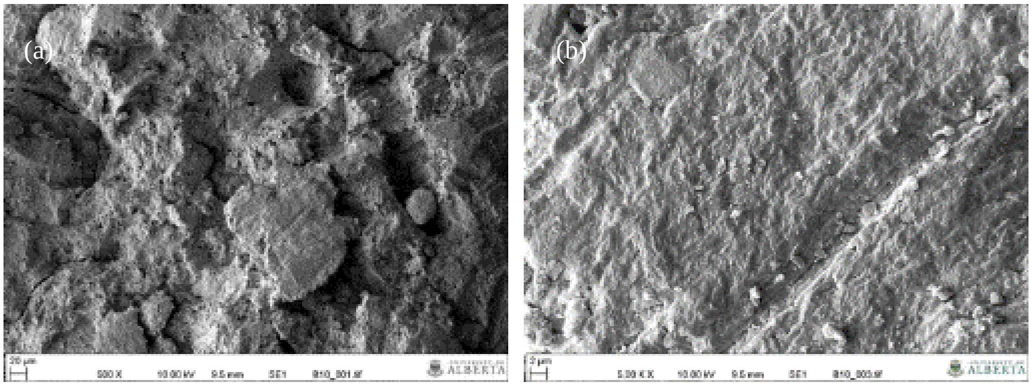


Figure G.5 Photomicrographs of bentonite in unconfined compression test at w=32%, T_c=21 days, (a) A_c=0%, M~500X; (b) A_c=0.25%, M~500X; (c) A_c=0%, M~5,000X; (d) A_c=0.25%, M~5,000X; (e) A_c=0%, M~10,000X; (f) A_c=0.25%, M~10,000X; (g) A_c=0%, M~20, 000X; (h) A_c=0.25%, M~20,000X.



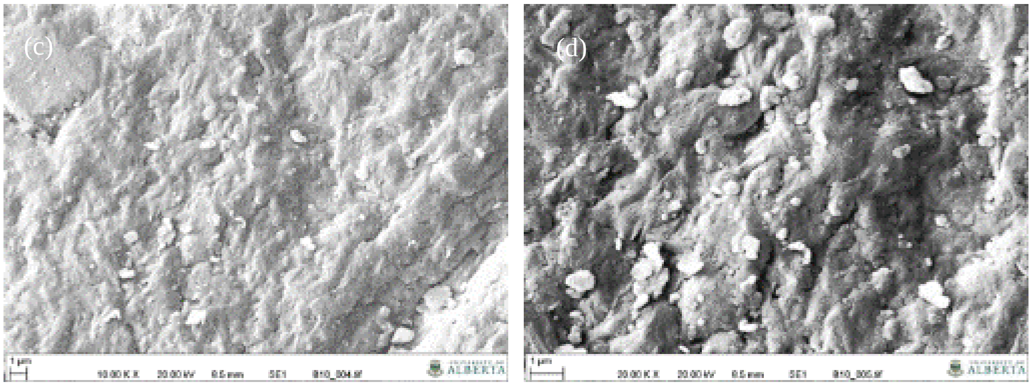
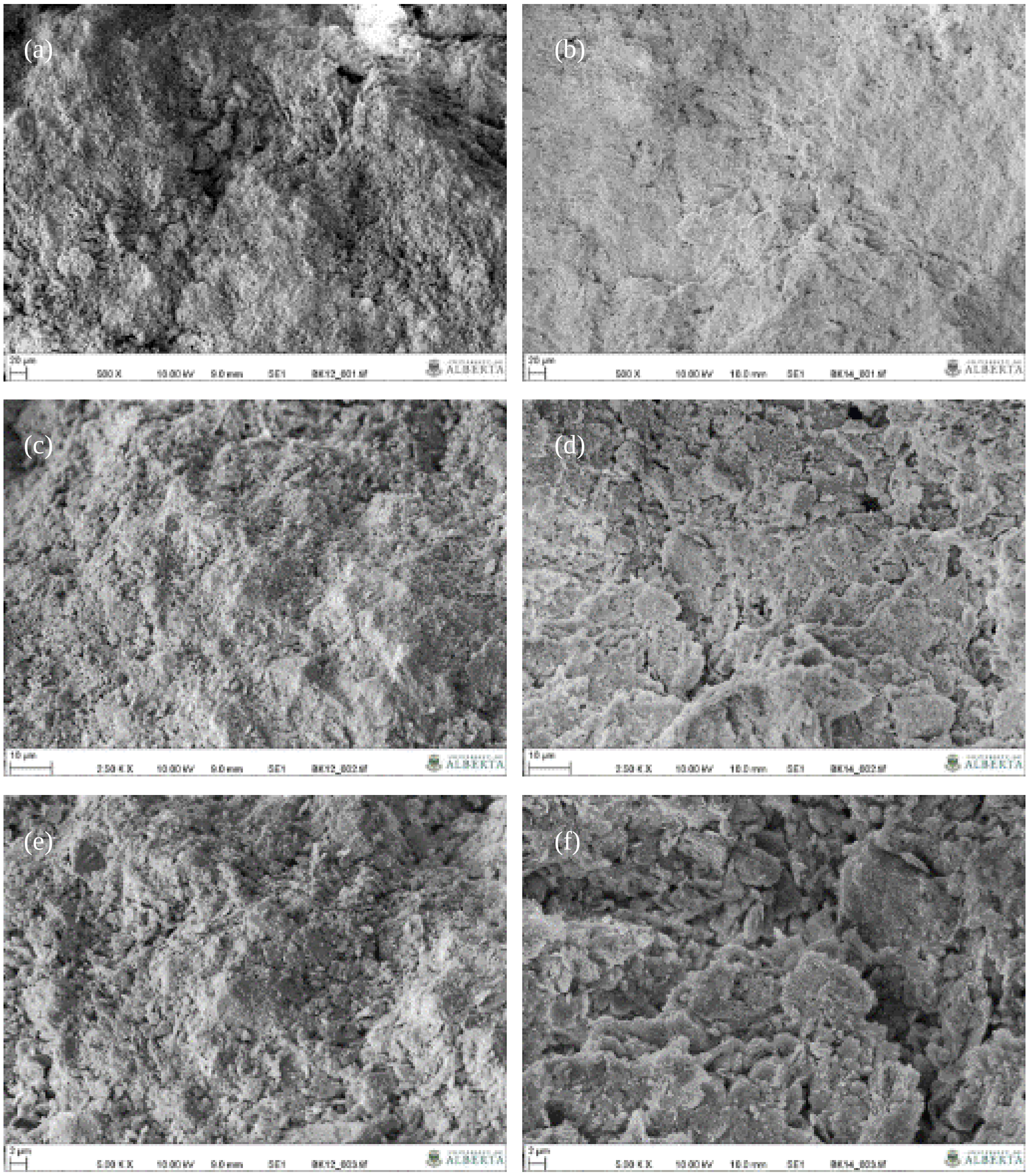


Figure G.6 Photomicrographs of bentonite in unconfined compression test at w=32%, T_c=21 days, (a) A_c=0.025%, M~500X; (b) A_c=0.025%, M~5,000X; (c) A_c=0.025%, M~10,000X; (d) A_c=0.025%, M~20,000X.



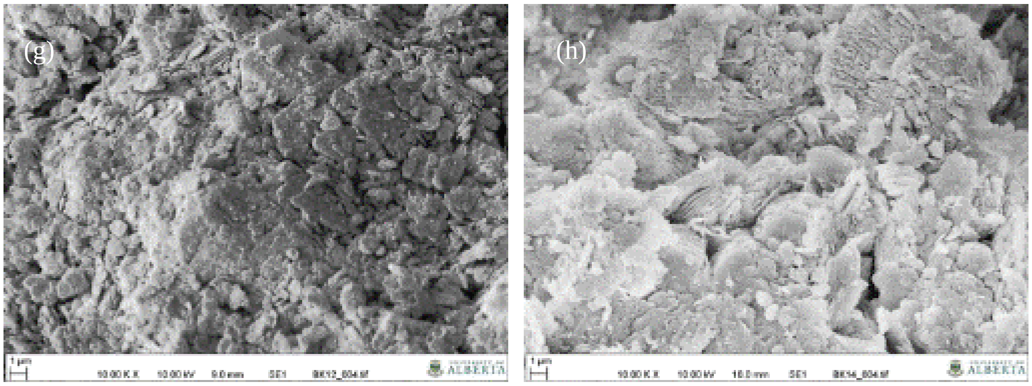


Figure G.7 Photomicrographs of BK20 in unconfined compression test at w=28%, T_c=7 days, (a) A_c=0%, M~500X; (b) A_c=0.25%, M~500X; (c) A_c=0%, M~2,500X; (d) A_c=0.25%, M~2,500X; (e) A_c=0%, M~5,000X; (f) A_c=0.25%, M~5,000X; (g) A_c=0%, M~10,000X; (h) A_c=0.25%, M~10,000X.

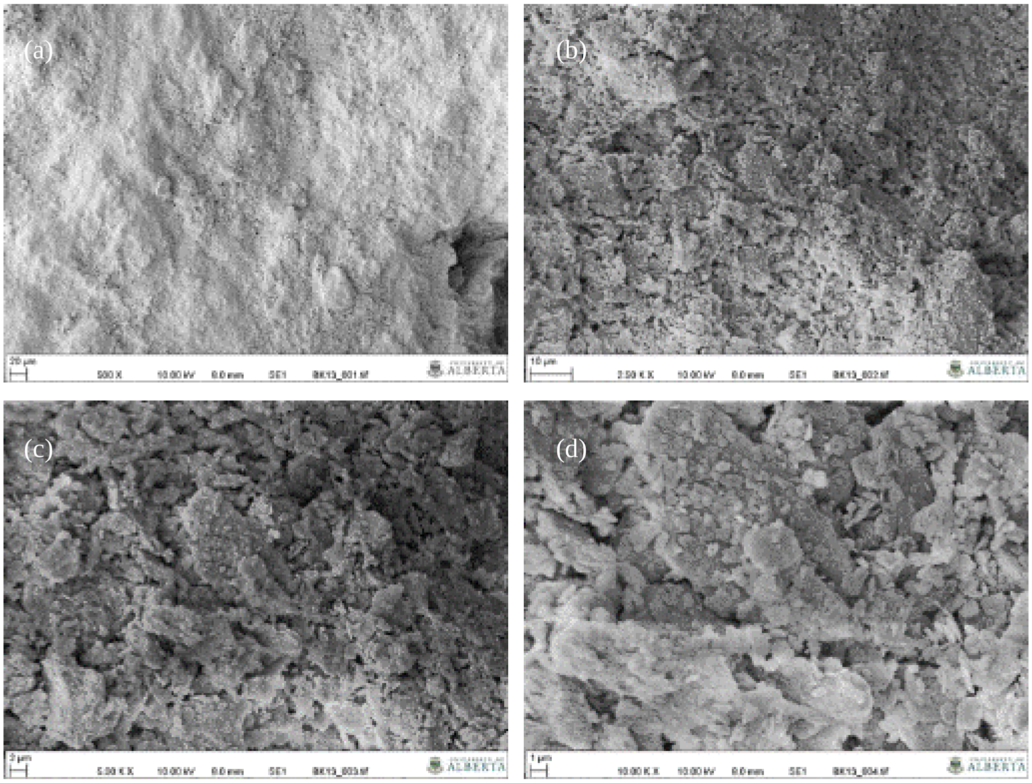
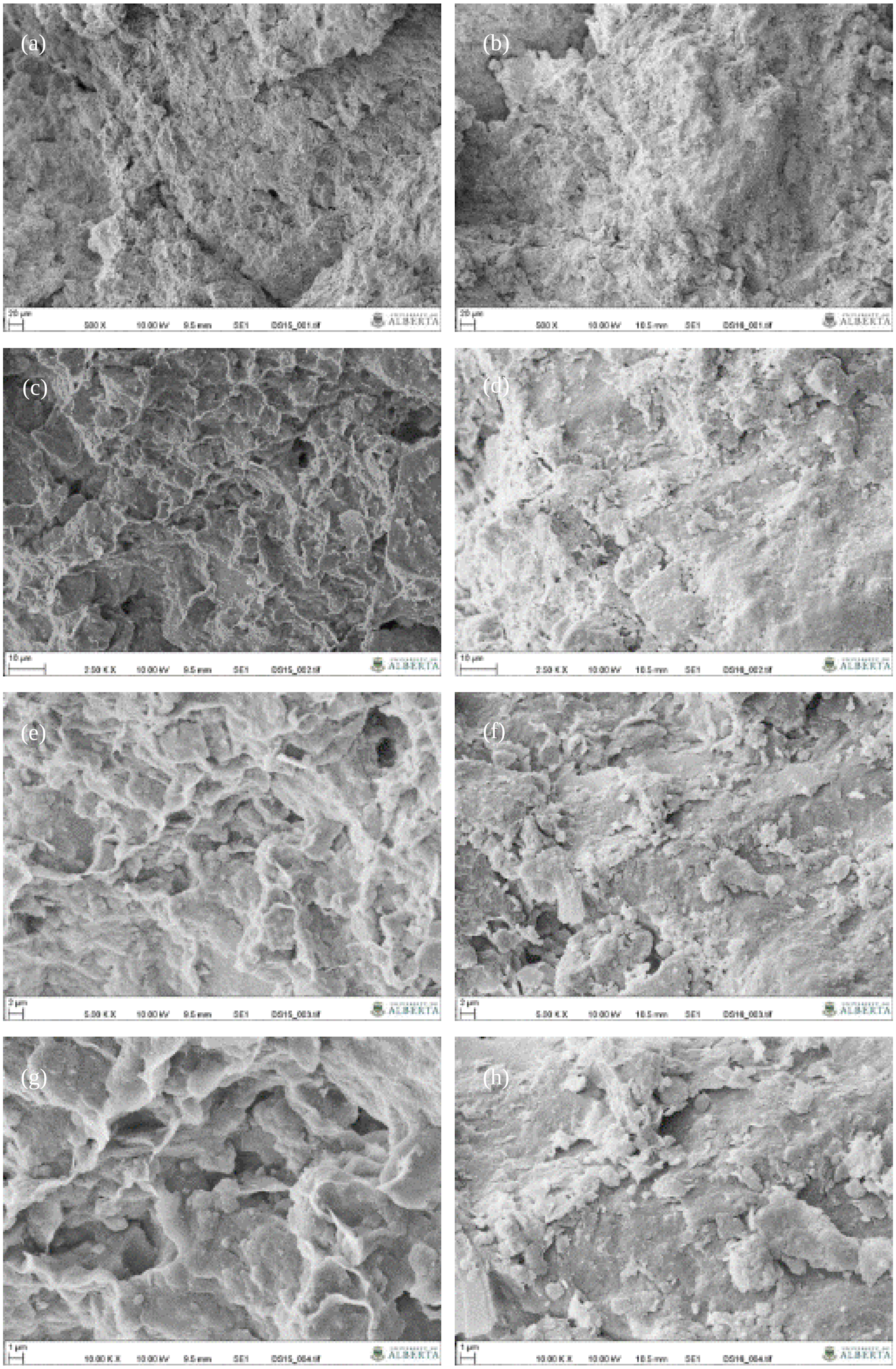


Figure G.8 Photomicrographs of BK20 in unconfined compression test at w=28%, T_c=7 days, (a) A_c=0.025%, M~500X; (b) A_c=0.025%, M~2,500X; (c) A_c=0.025%, M~5,000X; (d) A_c=0.025%, M~10,000X.


(a)
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(b)
20 µm 500 X 10.00 kV 10.5 mm SE1 DS16_001.tif
(c)
10 µm 2.50 K X 10.00 kV 9.5 mm SE1 DS15_002.tif
(d)
10 µm 2.50 K X 10.00 kV 10.5 mm SE1 DS16_002.tif
(e)
2 µm 5.00 K X 10.00 kV 9.5 mm SE1 DS15_003.tif
(f)
2 µm 5.00 K X 10.00 kV 10.5 mm SE1 DS16_003.tif
(g)
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(h)
1 µm 10.00 K X 10.00 kV 10.5 mm SE1 DS16_004.tif

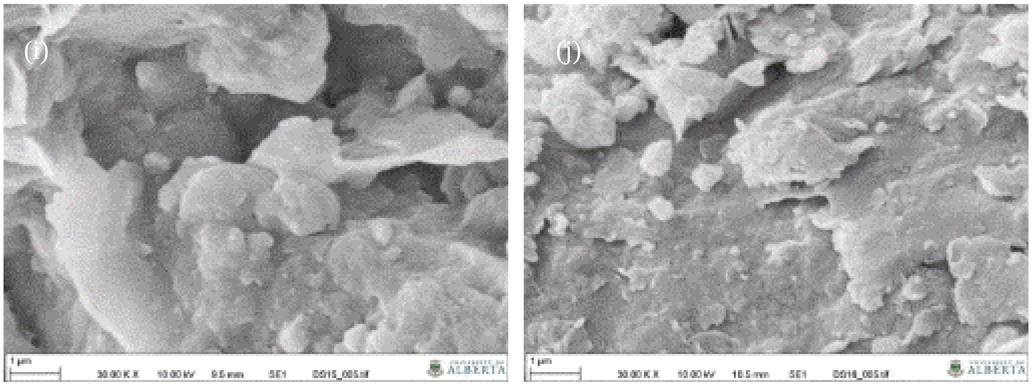

Figure G.9 Photomicrographs of Devon silt in unconfined compression test at w=16.5%, T_c=12 days, (a) A_c=0%, M~500X; (b) A_c=0.05%, M~500X; (c) A_c=0%, M~2,500X; (d) A_c=0.05%, M~2,500X; (e) A_c=0%, M~5,000X; (f) A_c=0.05%, M~5,000X; (g) A_c=0%, M~10,000X; (h) A_c=0.05%, M~10,000X; (i) A_c=0%, M~310,000X; (j) A_c=0.05%, M~30,000X.