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TRIAXIAL RESPONSE OF NATURAL CLAY REINFORCED WITH SAND COLUMNS UNDER PARTIALLY DRAINED CONDITIONS

by
AMJAD ZOUHEIR RAYESS

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AMJAD ZOUHEIR RAYESS

Approved by:

Dr. Shadi Najjar, Associate Professor — Advisor
Civil and Environmental Engineering

Dr. Salah Sadek, Professor — Co-advisor
Civil and Environmental Engineering

Dr. Ghassan Chehab, Associate Professor — Member of Committee
Civil and Environmental Engineering

Date of thesis/dissertation defense: February 03, 2015

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AN ABSTRACT OF THE THESIS OF

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Title: Triaxial Response of Natural Clay Reinforced with Sand Columns under Partially Drained Conditions

Engineers in the field of soil improvement have long referred to the method of constructing sand columns to improve soft clays. This method of soil improvement is relatively more environmentally friendly, economical, faster and practical than other applied techniques. Sand columnar inclusions help accelerate the rate of consolidation, increase the loading/bearing capacity and reduce settlements. Various experimental studies have been conducted to investigate the main characteristics of this composite system. Recently the experimental studies shifted towards reliance on triaxial testing, where the confining pressure, drainage conditions and loading rates could be controlled and varied. A comprehensive assessment of published triaxial tests indicates that almost all tests are conducted using Kaolin clay with no data being gathered for natural clays. In addition, most studies were performed on fully drained or undrained conditions, in contrast to field conditions where the composite system is expected to exhibit partial drainage. The objectives of this study are to (1) conduct conventional drained and undrained triaxial tests to study the performance of natural clay specimens that are reinforced at intermediate and high area replacement ratios (about 18% and 31% respectively) under different confining pressures, (2) compare the results obtained with the results of previous tests performed under same conditions but on Kaolin samples, (3) conduct partially drained triaxial tests on natural clay specimens that are reinforced with sand columns and sheared at different rates of loading under a confining pressure of 100kPa, and (4) compare the results obtained from the partially drained tests with those from conventional drained and undrained tests. The series of triaxial tests will be performed on back-pressure saturated, normally consolidated, natural clay specimens from Achrafieh, Beirut, that are prepared from slurry. The parameters that are varied in the study are the confining pressures, drainage conditions, diameters of the sand columns installed and the loading rates in the partially drained tests (1%, 3%, 5%, 10%, 20% and 60% strain per hour).

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CHAPTER 1

INTRODUCTION AND SCOPE OF WORK

1.1 Introduction

Granular columns are generally installed in deposits of soft clayey soils to improve the clay properties especially when other surface compaction approaches are ineffective. This practical, environmentally friendly technique has been used to accelerate the rate of consolidation of soft clays in response to loading and/or preloading in the form of sand columns or sand drains. Najjar et al. (2010) state that the potential reinforcing effect of granular drains on both the short and long term response of the composite clay/sand system is typically neglected in design given that the role of these columns is generally restricted to facilitating radial drainage. Columnar inclusions in the form of stone columns or vibro-columns have also been used as reinforcing elements to improve the bearing capacity of soft clays.

The performance of the clay/stone columns system has been the subject of extensive experimental and numerical studies that aim at optimizing the design of these systems (Najjar, 2013). Most of the experimental studies are based on 1-g tests that are conducted in one dimensional loading chambers (Hughes and Withers 1974, Narasimha Rao et al. 1992, Muir Wood et al. 2000, Malarvizhi and Ilamparuthi 2004, McKelvey et al. 2004, Ayadat and Hanna 2005, Ambily and Gandhi 2007, Murugesan and Rajagopal 2008, Gniel & Bouazza 2009, Murugeson and Rajagopal 2010, Cimentada et al. 2011, Shahu and Reddy 2011, and Fattah et al. 2011). Recently experimental studies shifted towards reliance on triaxial testing, where the confining pressure, drainage conditions and loading rates could be controlled and varied. Examples of studies where reinforced clays were tested under triaxial conditions include Sivakumar et al. (2004), Black et al. (2006), Black et al. (2007), Najjar et al. (2010), Black et al. (2011), Sivakumar et al. (2011), Maalouf (2012) and Bou Lattouf (2013).

Most previous research studies that were based on triaxial tests studied the undrained behavior of the clay samples, because this behavior generally governs the bearing capacity in the short term. However the assumption of fully undrained conditions may not apply for the case of clays with granular sand columns, since the columns will facilitate radial drainage of the surrounding clay. In fact, the behavior of the system is expected to be partially drained since the granular column will behave as a drained material while the surrounding clay could vary from partially drained to fully drained. The degree of partial drainage will depend on the rate of loading, the permeability of the clay, the spacing and diameter of the sand/gravel columns, and the possibility of smearing of the clay around the column during installation (Najjar 2013). As a result of the partially drained condition, the shear strength of the composite system is expected to be constrained between the lower bound of undrained strength and the upper bound of drained strength.

Current design procedures for problems involving foundations on soft clay deposits that are reinforced with sand/gravel columns lack a systematic approach for quantifying the effect of partial drainage and accounting for it in design. In addition, studies pertaining to the investigation of partially drained behavior are scarce and limited. In what follows is a brief summary of three such studies. The first study was conducted by Juran and Guermazi (1988), the second by Andreou et al (2008) and the third by Bou Lattouf (2013).

Juran and Guermazi (1988) studied the effect of partial drainage of a silty soil sample reinforced with river sand using a modified triaxial cell. The authors conducted tests at a specified rate of shearing while allowing the sand column to drain freely. They also conducted another series of fully undrained tests. The stress-strain response of the reinforced soil was greatly affected by the drainage of the column as seen in Figure 1.1. Juran and Guermazi (1988) reported results indicating that the drainage of the column notably improved the resistance of the reinforced soil to the applied strain and that the freely drained column had a maximum load carrying capacity of about twice that of the undrained column. These findings further reinforce the hypothesis that allowing partial drainage of the composite system, which is more in line with actual field conditions, will improve the shear resistance of the composite.

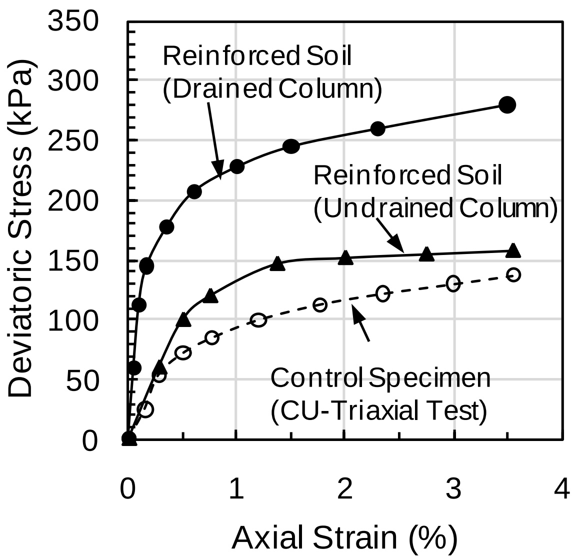


Figure 1.1: Effect of drainage on response of reinforced soil specimens to triaxial compression

Andreou et al. (2008) conducted triaxial compression tests on reinforced Kaolin clay samples. The authors used single columns of Hostun (HF) sand and gravel. Three

(3) series of tests were conducted. These included drained, undrained, and partially drained setups. The comparison of the results from the three series as seen in Figure 1.2 below provided indications of the influence of drainage conditions and rate of loading on the load response. Andreou et al. (2008) measured a reduction in strength when the rate of loading was “accelerated” with drainage allowed (partially drained) relative to slower rates of loading (fully drained). In spite of this decrease in resistance, the measured strength remained higher than that of the reinforced undrained sample. These findings further reinforce the hypothesis that the fully undrained and fully drained strengths constitute lower and upper bounds for the partially drained strength.

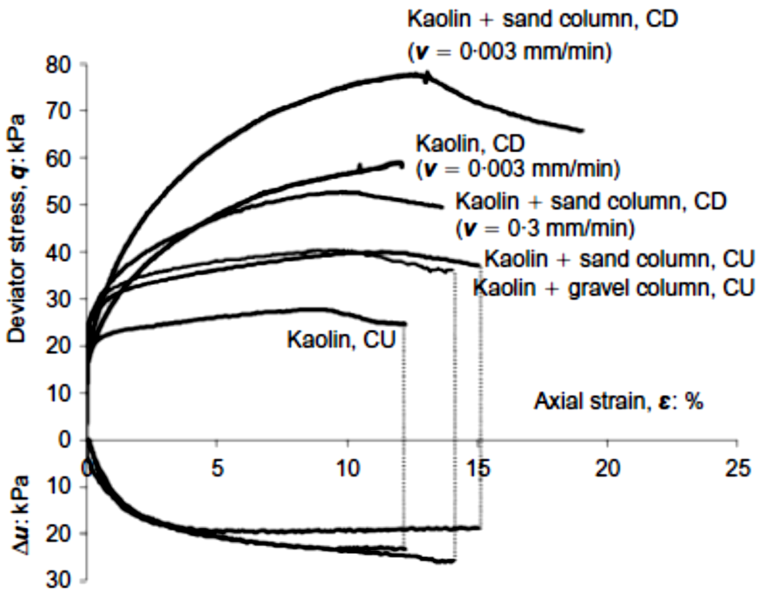


Figure 1.2: Variation of deviator stress and excess pore pressure with axial strain

Najjar et al. (2010), Maalouf (2012), and Bou Lattouf (2013) implemented a comprehensive experimental program that was based on a series of triaxial tests on normally consolidated Kaolin clays that were reinforced with Ottawa sand columns of medium density. The parameters that were varied included the area replacement ratio

(7.9%, 17.8%, and 31.7%), column penetration ratio (0.75 and 1.0), confining pressure (100, 150, and 200 kPa), drainage conditions (undrained, drained, and partially drained) and loading rates (3.5%, 40%, and 80% strain per hour).

The results of the tests reported by Najjar et al. (2010), Maalouf (2012), and Bou Lattouf (2013) indicated an improvement in the undrained and drained shear strength of the sand column reinforced Kaolin clay as the area replacement ratio and column penetration ratio increased. For undrained and drained tests, this increase in strength was coupled with a decrease in pore pressures and volumetric strains, respectively and an increase in the stiffness of the system. In addition to the improvement gained in the drained and undrained performances, Bou Lattouf (2013) reports that for the partially drained tests that were conducted at different rates of loading, the deviatoric stress versus axial strain curves of the specimens were found to lie between two boundaries, the higher one being the fully drained test while the lower boundary being the undrained test. For the partially drained tests, as the shearing rate decreased, the partially drained curves approached that of the fully drained curve. On the other hand, as the shearing rate increased, the partially drained curves got closer to that of the fully undrained curve, without approaching it (see test conducted at a strain rate of 80% per hour). At the fastest shearing strain rate of 80% per hour, results indicated that for Kaolin clay which has relatively high permeability, a considerable degree of partial drainage inevitably occurred from the clay radially through the sand column. The use of kaolin clay proved to be a limitation that will be addressed in this proposed research study.

The major conclusion that could be drawn from the work of Juran and Guermazi (1988), Andreou et al. (2008), and Bou Lattouf (2013) is that the undrained behavior generally underestimates the strength of soft clay reinforced with sand columns, and that partial drainage could significantly affect strength. In addition, BouLattouf (2013) defined a normalized strength improvement index as the ratio of $(\sigma_{d,PD} - \sigma_{d,U})$ to $(\sigma_{d,PD} - \sigma_{d,D})$, where σ_{PD}, σ_U and σ_D are the deviatoric stress measured for the partially drained, undrained (assumed to be a lower-bound for the composite) and drained tests (assumed to be an upper-bound for the composite). This strength improvement index provides a relative measure of the magnitude of strength that could be mobilized in a partially drained test in relation to the minimum and maximum strengths that could be obtained assuming undrained and drained conditions, respectively.

To be able to better understand the behavior of real clays at high shearing rates, there is a need for utilizing clay that has properties that are more aligned with natural clays in the field, which are expected to be less permeable than kaolin clay. As a result, Bou Lattouf recommended the use of a different clay material which allows the specimens to be sheared at a closer rate to real field conditions while allowing drainage of the clay through the sand columns. Another limitation in the previous work on partial drainage is the relatively small area replacement ratios that were utilized in these studies (maximum of 18%). Area replacement ratios in the field are generally between 20% and 35%. It is important to model a system using natural clay that would replicate and represent the actual drainage conditions in the field while maintaining representative area replacement ratios that are typical of field conditions.

1.2 Objectives and Scope of Work

The objectives of this study are to (1) conduct conventional drained and undrained triaxial tests to study the performance of natural clay specimens that are reinforced with medium and dense sand columns at intermediate and high area replacement ratios (about 18% and 31% respectively) under different confining pressures, (2) compare the results obtained with the results of previous tests performed under similar conditions but on Kaolin samples, (3) study the behavior of natural clay specimens that are reinforced with sand columns under partially drained conditions using partially drained triaxial tests that involve different rates of loading under a confining pressure of 100kPa, and (4) establish a relationship between the strength mobilized in the partially drained tests and the corresponding degree of consolidation /drainage.

Two series of triaxial tests were performed on back-pressure saturated, normally consolidated, natural clay specimens from Achrafieh that are prepared from slurry. The parameters that are varied in the study are the confining pressure (100, 150, and 200 kPa), drainage conditions (drained, undrained, and partially drained), area replacement ratio (17.8% and 31.7%), density of the sand columns (medium and dense) and the loading rates in the partially drained tests (1%, 3%, 5%, 10%, 20% and 60% strain per hour). In addition to the two series of tests, conventional tests will be performed to classify and describe the natural clay, including Atterberg Limits, grain size distribution, hydrometer, specific gravity, and 1-D consolidation.

In the first series of tests (Table 1.1), drained and undrained triaxial tests were conducted on slurry-consolidated, back-pressure saturated, Achrafieh clay specimens at confining pressures of 100, 150 and 200kPa. This series of tests includes both control tests (tests conducted on Clay and Sand independently) and tests conducted on clay specimens that are reinforced with sand columns.

Table 1.1: Proposed triaxial soil testing program (Series 1) – Drained and Undrained Tests

Test No.	Confining Pressure σ_3 (kPa)	Type	Diameter of Sand Column (cm)	Drained/ Undrained	Area Replacement Ratio: A_c/A_s (%)	Column Height Penetration Ratio, (H_c/H_s)	Column Height Diameter Ratio, (H_c/D_c)
1	100	Clay	0	D	0	0	-
2			0	U	0	0	-
3		Sand	7.1-D**	D	-	-	-
4			7.1-D	U	-	-	-
5		Composite	3-MD*	U	17.8	1	4.73
6			3-D	D	17.8	1	4.73
7			3-D	U	17.8	1	4.73
8			4-D	D	31.7	1	3.55
9			4-D	U	31.7	1	3.55
10	150	Clay	0	D	0	0	-
11			0	U	0	0	-
12		Sand	7.1-D	D	-	-	-
13			7.1-D	U	-	-	-
14		Composite	3-MD	U	17.8	1	4.73
15			3-D	D	17.8	1	4.73
16			3-D	U	17.8	1	4.73
17			4-D	D	31.7	1	3.55
18			4-D	U	31.7	1	3.55
19	200	Clay	0	D	0	0	-
20			0	U	0	0	-
21		Sand	7.1-D	D	-	-	-
22			7.1-D	U	-	-	-
23		Composite	3-MD	U	17.8	1	4.73
24			3-D	D	17.8	1	4.73
25			3-D	U	17.8	1	4.73
26			4-D	D	31.7	1	3.55
27			4-D	U	31.7	1	3.55

* MD = Medium-Dense Sand (RD = 55%), ** D = Dense Sand (RD = 86%)

The objective of this series of tests is to characterize the undrained and drained stress-strain behavior, volume change behavior, pore pressure generation, shear strength, and stiffness of the "natural clay" when reinforced with sand columns under different test conditions. All samples (including control clay and control sand samples) will have a diameter of 7.1cm and a height of 14.2cm and will be tested at effective confining pressures of 100, 150 and 200kPa.

Two area replacement ratios (17.8% and 31.7%) are targeted in the proposed research. The intermediate area replacement ratio of 17.8% is modeled using a sand column with a diameter of 3cm, while the larger replacement ratio is modeled using a 4cm sand column. Note that the diameter of the clay specimen is 7.1cm. The columns will be fully penetrating in the clay and will be installed using a replacement approach in a non-encased state. The sand columns will be prepared at two relative densities (55% and 86%). The main focus of the proposed research study is on dense granular columns (RD = 86%); however a decision was made to conduct limited tests with the natural clay and medium dense columns (RD = 55%) to allow for direct comparison with the tests conducted on kaolin clay under the same conditions.

In the second series of tests (Table 1.2), partially drained tests will be conducted on 7.1cm-diameter clay specimens that are reinforced with 3cm and 4cm diameter fully penetrating non-encased dense columns. The partial drainage will be enforced by prohibiting drainage from the bottom of the clay and allowing it through the top cap of the triaxial cell. Different shearing rates during the triaxial tests will be conducted to represent relatively quick loading (a strain rate of 60% per hour), relatively slow loading (a strain rate of 1% per hour) and relatively average loading (strain rates ranging from 5%-20% per hour). All the tests will be performed on slurry-consolidated, back-pressure saturated, Achrafieh clay specimens at a confining pressure of 100kPa. The main two objectives of this series of tests are to (1) compare the behavior of partially drained samples to that of fully drained and fully undrained samples under similar conditions, and (2) establish a relationship between the mobilized partially drained strength and the theoretical estimates of the degree of consolidation calculated using finite difference solutions that take into consideration the radial dissipation of pore pressures in the partially drained tests.

Table 1.2: Proposed triaxial soil testing program (Series 2) – Partially Drained Tests

Test No.	Confining Pressure σ_3 (kPa)	Diameter of Sand Column (cm)	Rate of Loading (% strain per hr)	Partially Drained (PD)	Area Replacement Ratio: A_c/A_s (%)	Column Height Penetration Ratio, (H_c/H_s)	Column Height Diameter Ratio, (H_c/D_c)
1	100	3	1	PD	17.8	1	4.73
2		3	3	PD	17.8	1	4.73
3		3	5	PD	17.8	1	4.73
4		3	10	PD	17.8	1	4.73
5		3	20	PD	17.8	1	4.73
6		3	60	PD	17.8	1	4.73
7		4	1	PD	31.7	1	3.55
8		4	5	PD	31.7	1	3.55
9		4	10	PD	31.7	1	3.55
10		4	20	PD	31.7	1	3.55
11		4	60	PD	31.7	1	3.55

1.3 Significance of the Proposed Research

Previous studies that investigated the load response of soft clays reinforced with sand columns under triaxial conditions were performed on Kaolin specimens and not on natural clay. Moreover, these studies mainly addressed the undrained or drained behavior of the composite system and did not take into consideration the effect of partial drainage that is most likely the case in the field. The field behavior of soft clays that are reinforced with sand/stone columns could range from undrained to the drained depending on various variables such as the type, plasticity and permeability of the clay, the area replacement ratio, and the rate of loading. This study constitutes the first comprehensive effort to study the response of “natural” clay in a triaxial setting when reinforced with dense sand columns at practical area replacement ratios under different drainage conditions and rates of loading. The anticipated results will serve to better understand the actual field response of the composite system and could be used as a basis for refining existing design methodologies for granular inclusions in soft clays.

1.4 Organization of Thesis

The thesis consists of the following eight (8) chapters:

Chapter I: Introduction.

Chapter II: A literature review which includes the major experimental and analytical studies related to the reinforcement of soft clays with stone columns.

Chapter III: The properties of the materials used in the testing program presented together with the methodology used in the clay sample preparation and construction of the reinforced and unreinforced sand columns.

Chapter IV: A step by step procedure for operating the automated triaxial equipment presented in a detailed manner.

Chapter V: The results and analysis of drained tests.

Chapter VI: The results and analysis of undrained tests and comparison with Kaolin tests.

Chapter VII: The results and analysis of partially drained tests.

Chapter VIII: Conclusion and recommendations

CHAPTER 2

LITERAURE REVIEW

2.1 Introduction

In this chapter, a brief literature review of the major theoretical and experimental studies performed to investigate the behavior of granular columns is presented, with particular emphasis on studies that investigated the effect of the rate of loading and partial drainage on the response of the composite. The response of clays improved with granular inclusions has been investigated in field and laboratory scale tests (1-g, triaxial, or centrifuge). These tests involved clay specimens reinforced with partially or fully penetrating, encased or ordinary, stone or sand columns that were installed as single columns or as column groups. On the other hand, several research studies involved numerical investigations that were based on finite element models to accomplish the same objective.

One of the earliest experimental works to study the effect of inserting stone columns in weak clays on the bearing capacity and on the rate of settlement was conducted by Hughes and Withers (1974). They used sand columns as reinforcement for the Kaolin clay and their tests were conducted under fully drained conditions. Later, experimental studies evolved to include different drainage conditions and test setups. Examples include the work done by Charles and Watts (1983), Bachus and Barksdale (1984), Juran and Guermazi (1988), NarasimhaRao et al. (1992), Muir Wood et al. (2000), Sivakumar et al. (2004), McKelvey et al. (2004), Ayadat and Hanna (2005), Black et al. (2006, 2007), Ambily and Ghandi (2007), Andreou et al (2008), Black et al (2011) and Sivakumar et al (2011). Other studies involved the use of finite element modeling to examine the effect of sand/stone columns on the stress-strain load response of the reinforced clay including the work of Elshazly et al (2008) and Chen et al (2009).

2.2 State of the Art Review by Najjar (2013)

For a summary of published research papers and reports that focus on the modeling, testing, and analysis of soft clays that are reinforced with sand/stone columns in relation to bearing capacity and settlement considerations, the reader is referred to the paper entitled “A State-of-the-Art Review of Stone/Sand-Column Reinforced Clay Systems” by Najjar (2013). The paper is aimed at summarizing published works on the analysis, testing, and modeling of soft soils that are reinforced with single stone/sand columns and stone/sand column groups. A total of forty-nine papers were reviewed in the study, with the focus being on recent papers that were published after the year 2000. The objectives of the study were to (1) assemble published results from field, laboratory, and numerical investigations of sand/stone columns in clay in one resource to provide future researchers with easy retrieval and access to information and data, (2) present the most common and most recent design approaches and methodologies for ordinary and encased sand/stone columns to act as a resource for designers of clay/stone column systems, and (3) provide a critical assessment/summary of the common findings from the most comprehensive set of published papers and reports on the subject of sand/stone columns in clays.

Based on the review conducted in Najjar (2013), the following main conclusions and findings could be made with regards to the role that sand/stone columns play in reinforcing soft clays:

2.2.1 Mode of Failure

The failure mode of columns that are loaded at their top is characterized by bulging at a distance of 0.5 to 3 column diameters from the top of the column. Under wide loads, the applied loads add to the lateral passive resistance of central columns, thus increasing their load carrying capacity and reducing bulging. Failure of groups is characterized by bulging, bending or shearing. Results from 3D images of failures of column groups from FE analyses indicate that as one moves away from the center of the group, outward bending of the columns increases, with central columns not showing signs of bending. In long columns, deformations are generally concentrated in the upper zones while for shorter columns, the columns tend to punch and penetrate into the soft clay. In triaxial tests, short columns appeared to bulge and penetrate below the reinforced portion of the clay, while fully penetrating columns bulged relatively uniformly along their length. Bulging was found to be more predominant in soft clays than in stiffer clays. Limited finite element results from identical rammed and un-rammed piers indicate that deeper bulging is expected for un-rammed piers compared to rammed piers. When loads are applied to columns that are encased with a geofabric, bulging is reduced significantly in comparison to the unreinforced column. In addition, bulging for encased columns decrease at the surface and is transmitted to greater depths below the top of the column.

2.2.2 Improvement in Bearing Capacity and Stiffness

The inclusion of granular columns in soft clays increases the bearing capacity and the stiffness. Results from field, laboratory, and finite element analyses indicate that as the area replacement ratio increases, both stiffness and strength of the reinforced clay system increase. Results from field and laboratory tests involving identical single columns and column groups tested with foundation loading at the same area replacement ratio indicate that the behavior was found to be similar (group efficiency of about 1.0) indicating that a unit cell concept can simulate the behavior of an interior column in a large group. However, results from limited finite element analyses indicate that the ratio of the settlement of the group to the settlement of an equivalent unit cell was found to range from 0.6 to 2.25 depending on the type of soil and width to length ratio of the foundation, with ratios that are greater than 1.0 indicating that the unit cell settlement is not necessarily the upper bound for settlement prediction. These finite element results were confirmed by laboratory tests which showed that for area replacement ratios between 28% and 40%, the settlement improvement factors measured for the group were about half those measured for the equivalent single columns.

2.2.3 Critical Column Length

For a given area replacement ratio, increasing the length to diameter ratio of the columns results in an increase in the ultimate stress and stiffness. Field, laboratory and finite element results indicate that the column length is significant up to a certain point beyond which increasing the column length will not lead to an increase in strength, although it could still increase the stiffness. The stiffer response observed for longer columns could indicate that longer columns may be more efficient for settlement control. Based on the literature survey, it could be concluded that there is consensus that the optimum length of stone columns for effective load transfer lies between 5 and 8 times the diameter of the column. The critical length may increase as the area ratio increases since the failure mechanism could be pushed deeper below the footing.

2.2.4 Stress Concentration Ratio

The stress concentration factor, defined as the stress in the column to the stress in the surrounding clay, is an important parameter in the design of clay-sand column systems from a stiffness perspective and from an ultimate strength perspective. The literature review indicates that the stress concentration factor for practical field applications involving ordinary granular columns with area replacement ratios ranging from 20% to 40% could range from 2.5 and 5.5. However, the literature review indicates that the stress concentration factor is not a constant for a given reinforced clay system (given area replacement ratio and given column length/penetration) since it depends on the strain level, magnitude of the applied stress relative to the failure stress, time from the application of the load, and drainage conditions. As a result, care should be exercised when arbitrary stress concentration factors are adopted from a specific reference in the literature. In general, results from field tests involving maintained incremental loading indicate that stress concentration factors at small load increments tend to be small upon load application and increase with time. For larger load increments (large displacements and close to failure), the stress concentration factors tend to be constant with time. Results from some laboratory experimental programs indicate that stress concentration factors tend to be larger for small strains at the early stages of loading (McKelvey et al. 2004, Cimentada et al. 2011); however, results from limited field tests (ex. White et al. 2007) showed that the measured stress concentration factors increased from about 3.3 for intermediate levels of loading to about 5.5 at higher levels of loading. These contradicting observations regarding the dependency of the stress concentration factor on load and strain levels could be partially explained by the results presented in Juran and Guermazi (1988), Han and Ye (1991), and Fattah et al. (2011) which show that the stress concentration factor generally increases with load level up to a certain peak after which the stress concentration factor decreases with additional applied load. Results from tests conducted by Fattah et al. (2011) indicated that the stress concentration ratio n reached a peak value at a point located at a stress of q/c_u=2 and then reduced with increasing bearing ratio q/c_u to reach values in the range of 1.2 to 3.1, depending on the area replacement ratio and the L/D ratio.

2.2.5 Drained versus Undrained Loading

The field behavior of clays that are reinforced with granular columns could range from undrained to drained depending on the permeability of the clay, the area replacement ratio, and the rate of loading. Gneill and Bouazza (2009) showed through measurements of the water content of the clay before and after loading that drained, or at least partially drained, loading occurred in the clay immediately surrounding the bulge zone of the column. Studies that targeted the effect of partial drainage indicated that measured deviatoric stresses at failure in triaxial tests were the highest for fully drained conditions, the lowest for undrained conditions, and intermediate for partially drained conditions. It could be concluded that more research is needed to investigate the effect of partial drainage on the load response of clay-sand/stone column systems using field, laboratory, and FE experiments and analyses.

2.2.6 *Effect of Arrangement, Installation Method, Type, and Density of Columns*

The review of the literature indicated that lateral earth pressure coefficients around stone columns were found to depend on the spacing of neighboring columns with back-calculated lower-bound estimates of 1.2, 1.0 and 0.7 and upper bound estimates of 2.0, 1.5 and 1.1 for stone columns in arrangements of 1.20×1.50, 1.75×1.75 and 2.10×2.10m, respectively. Data on the effect of column installation methods on the loads carried by rammed and un-rammed piers from very limited FEM and field studies show contradicting results, with the data from the 3D FEM study by Chen et al. (2009) indicating that the rammed pier capacity could be more than twice that of the ordinary equivalent aggregate pier capacity, while the results from the field study conducted by Stuedlein and Holtz (2012) indicating that vibro-compacted piers exhibited a 15% increase in capacity compared to the tamped piers. With regards to the composition and density of the column material, results from the field study conducted by Bergado et al. (1987b) indicated that the capacity of single rammed aggregate piers that were comprised of sands increased consistently as the density of the sand increased, while the piers in which gravel replaced part or all of the sand exhibited increases in capacity. Along the same lines, the field tests conducted by Stuedlein and Holtz (2012) indicated that columns comprised of uniformly graded gravel exhibited a higher stiffness and load carrying capacity compared to columns comprised of well-graded mixture of gravel, silt, and sand. It should be noted however, that the effects of column material and method of installation witnessed above in the tests conducted by Stuedlein and Holtz (2012) on single columns vanished for tests conducted in the same test program on column groups with an area replacement ratio of about 30%. This led the authors to conclude that the role of the matrix soil in a group loading could be more important than the effects of installation and column type and composition.

2.3 Summary of Studies Involving Partial Drainage

The first investigation dealing with partially drained behavior was conducted by Juran and Guermazi (1988), the second by Andreou et al. (2008) and the third by Bou Lattouf (2013). Juran and Guermazi (1988) studied the effect of partial drainage of a silty soil sample reinforced with river sand using a modified triaxial cell. Andreou et al. (2008) conducted triaxial compression tests on kaolin clay samples reinforced with single columns of Hostun (HF) sand and gravel. Three (3) series of tests were conducted; these included drained, undrained, and partially drained setups. The limited results obtained by Juran and Guermazi (1987) and Andreou et al. (2008) suggest that the assumption of undrained conditions in the clay surrounding sand/gravel columns would lead to an underestimation of the degree of improvement in the shear strength of the clay-sand column system which would be obtained in the field.

Bou Lattouf (2013) performed a series of partially drained tests on Kaolin specimens reinforced with medium dense sand columns, sheared under different shearing rates. For the partially drained tests that were conducted at different rates of loading, the deviatoric stress versus axial strain curves of the specimens were found to lie between two boundaries, the higher boundary being the fully drained test and the lower boundary being the undrained test. Bou Lattouf (2013) reported a limitation for kaolin clay which has relatively high permeability. Even at very high shearing rates, (80% strain per hour), a considerable degree of partial drainage inevitably occurred from the clay radially through the sand column. A summary of the results of the tests that were conducted by Bou Lattouf (2013) at a confining pressure of 100 kPa is presented in Figure 2.1.

Results on Figure 2.1 indicate that for the partially drained tests, as the shearing rate decreases, the partially drained curves approach that of the fully drained curve. On the other hand, as the shearing rate increases, the partially drained curves tend to get closer to that of the fully undrained curve, without approaching it (see test conducted at a strain rate of 80% per hour). The strain rate of 80% per hour was the highest practical shearing rate that could be used in the partially drained tests. Results on Figure 2.1 indicate that for kaolin clay which has relatively high permeability, even at very high shearing rates, a considerable degree of partial drainage inevitably occurred from the clay radially through the sand column. Similar results were obtained for tests conducted with confining pressures of 150 and 200 kPa. The use of kaolin clay proved to be a limitation that will be addressed in this proposed research study.

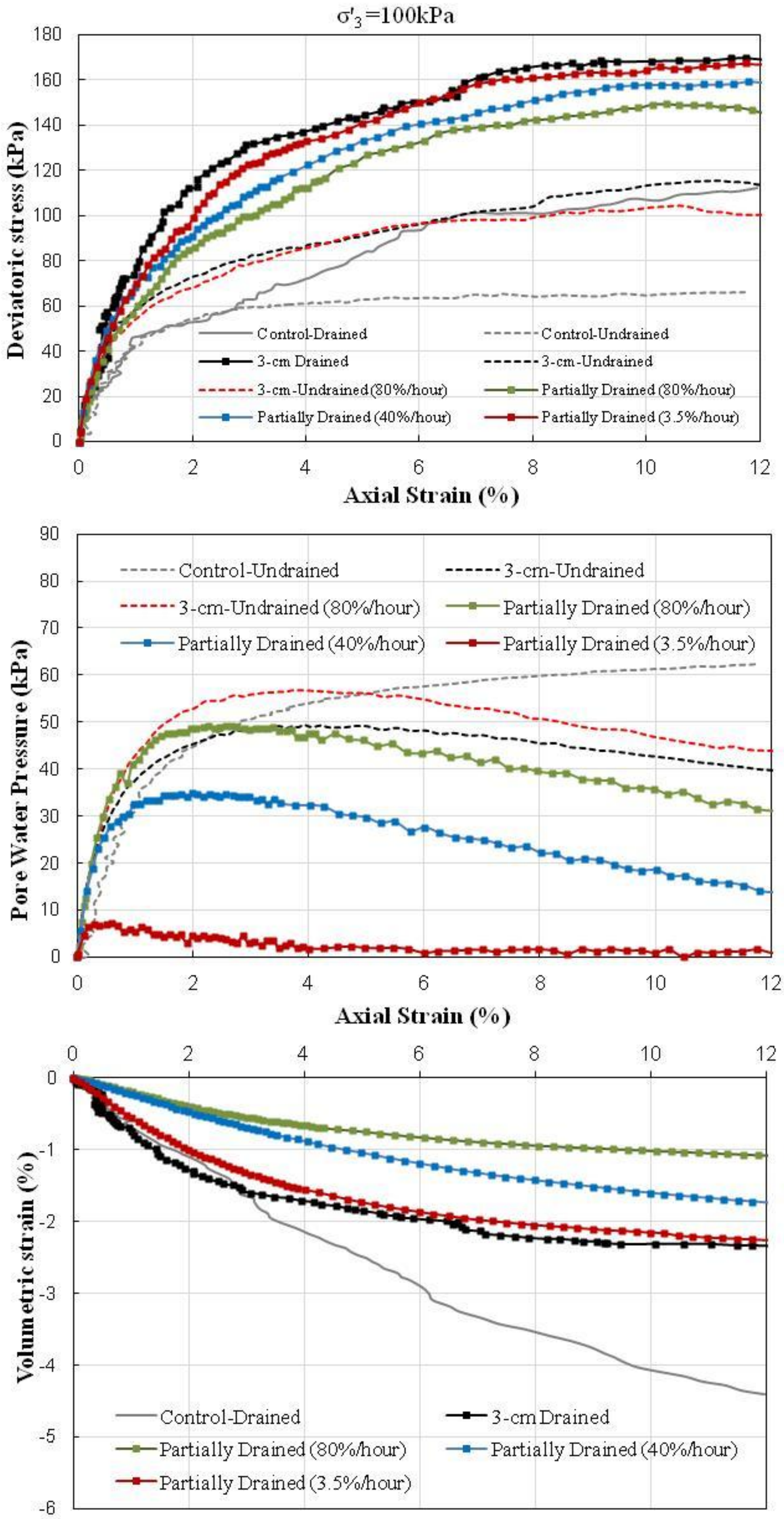


Figure 2.1: Deviatoric stress, pore water pressure, and volumetric strain versus axial strain for reinforced Kaolin specimens at confining pressure of 100 kPa (Bou Lattouf 2013)

The major conclusion that could be drawn from the work of Juran and Guermazi (1988), Andreou et al. (2008), and Bou Lattouf (2013) is that the undrained behavior generally underestimates the strength of soft clay reinforced with sand columns, and that partial drainage could significantly affect strength. BouLattouf (2013) defined a normalized strength improvement index as the ratio of $(\sigma_{d,PD} - \sigma_{d,U})\ to\ (\sigma_{d,PD} - \sigma_{d,D})$, where $\sigma_{d,PD}$ is the deviatoric stress measured for the partially drained tests, $\sigma_{d,U}$ is the stress of the undrained tests (assumed to be a lower-bound for the composite) and $\sigma_{d,D}$ is the stress of the drained tests (assumed to be an upper-bound for the composite). This strength improvement index provides a relative measure of the magnitude of strength that could be mobilized in a partially drained test in relation to the minimum and maximum strengths that could be obtained assuming undrained and drained conditions, respectively.

The strength improvement index for the partially drained tests conducted by Bou Lattouf (2013) is plotted versus the estimated degree of consolidation at failure on Figure 2.2.2. The curves on Figure 2.2.2 indicate that for an average degree of consolidation at failure of 50% (PD tests at strain rate of 80% per hour), the strength improvement index was about 60%, indicating that the partially drained strength was more than half the way between the undrained strength and the drained strength. For an average degree of consolidation at failure of about 75% (PD tests at strain rate of 40% per hour), the strength improvement index was about 80%. The slowest partially drained tests (3.5% per hour) resulted in a theoretical degree of consolidation of about 98%. The strength improvement index for these cases was about 95% indicating more or less complete mobilization of the fully drained strength of the composite.

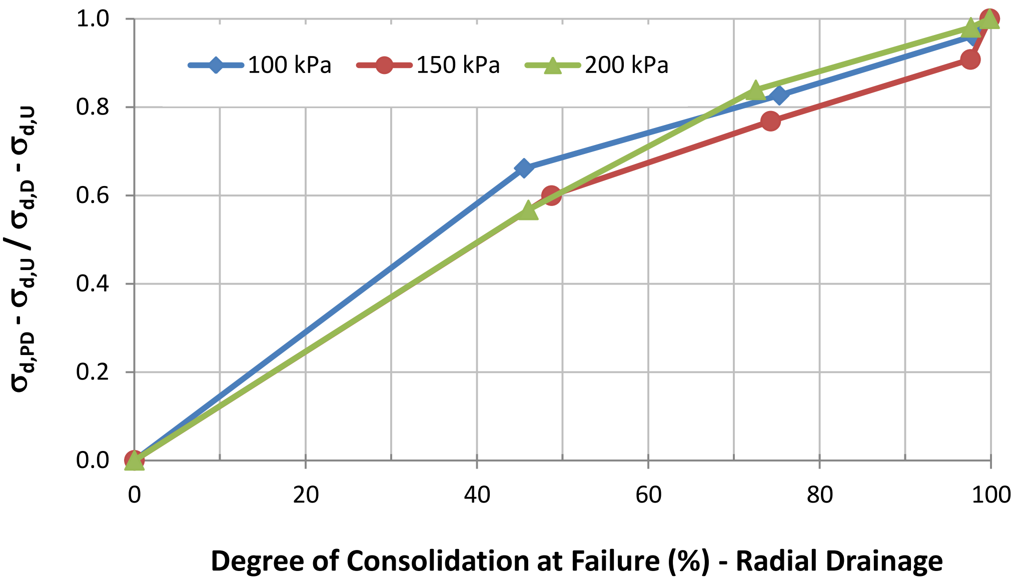


Figure 2.2: Strength improvement index versus degree of consolidation at failure (Bou Lattouf 2013)

The limitations behind using relatively permeable clay like Kaolin clay is clearly illustrated in Figure 2.2.2. Even at the highest shearing rate of 80% per hour, more than 50% consolidation occurred in the clay specimen in the partially drained tests. To be able to better understand the behavior of real clays at high shearing rates, there is a need for utilizing clay that has properties that are more aligned with natural clays in the field, which are expected to be less permeable than kaolin clay. As a result, Bou Lattouf recommended the use of a different clay material which allows the specimens to be sheared at a closer rate to real field conditions while allowing drainage of the clay through the sand columns.

CHAPTER THREE

TEST MATERIALS AND SAMPLE PREPARATION

3.1 Introduction

This chapter describes the properties of the materials used in the testing program and the process of preparation of samples of clay that were reinforced with sand columns. The materials used in the test program include natural clay from Achrafieh and Ottawa sand.

Atterberg limits, sieve analysis, hydrometer analysis, specific gravity and 1-Dimensional consolidation tests (oedometer test) were performed at the soil laboratory at the American University of Beirut in accordance with ASTM standards on natural soil obtained from natural ground from Achrafieh, Beirut, Lebanon in order to determine the soil properties and classification. The results of the consolidation tests were used to determine the coefficient of consolidation of the clay. For Ottawa sand, besides referring to previously performed sieve analysis and relative density tests, additional consolidated drained and consolidated undrained triaxial tests were performed on dense specimens having a relative density of ≈86%.

A detailed description of the process of the sample preparation is presented in this chapter. The process includes the consolidation of the Achrafieh slurry in custom-fabricated consolidometers in addition to preparation and installation of frozen sand columns with varying diameters in pre-augured holes in the specimens.

3.2 Test Materials

3.2.1 Achrafieh Clay

A sufficient reserve of natural clay from Achrafieh was brought from the field to the lab in preserved packages. Around 30 kilograms of this clay were crushed in the lab, oven dried, and then sieved using Sieve No. 20 to achieve a homogeneous and uniform bank of clay powder to be used in the testing program. The steps followed in the process of preparing the clay powder are illustrated in Figure 3.1.

Figure 3.1: Clay obtained from natural ground

Atterberg Limits and Specific Gravity tests were conducted on the clay powder to determine the index properties of the Achrafieh clay. The results are presented in Table 3.1.

Liquid limit (%)	Plastic limit (%)	Plasticity Index	Specific gravity
26.68	15.96	10.72	2.63

Table 3.1: Index properties of Achrafieh clay

To account for any possible changes in the plasticity of the clay due to the recycling, Atterberg limit tests were conducted on the recycled clay at different phases of the project. Results indicated minimal changes in the Atterberg Limits for the recycled clay confirming that no change in plasticity of the clay had occurred as a result of recycling.

The Liquid Limit and the Plasticity Index of the clay were used to classify the soil as per the Unified Soil Classification System (USCS). The clay is shown to be of low plasticity and can be classified as Brown Lean Sandy Clay CL, according to the Unified Soil Classification (Figure 3.2).

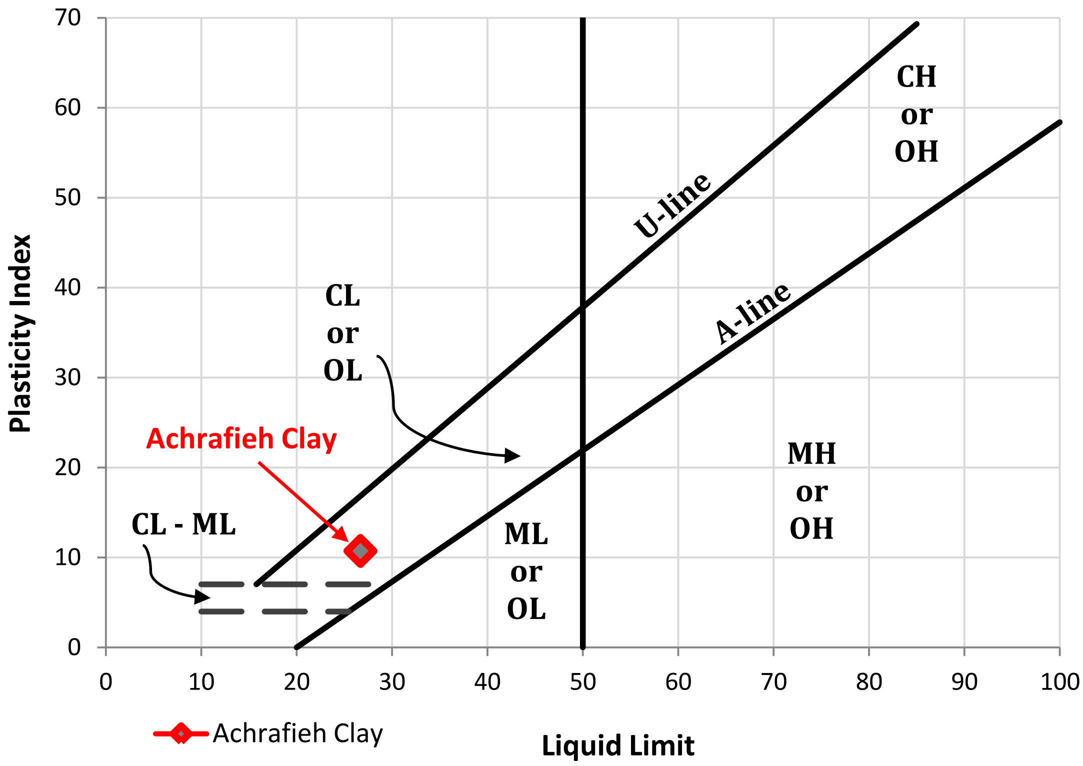


Figure 3.2: Soil classification of Achrafieh clay according to Casagrande's Plasticity Chart

In addition to the Atterberg tests, grain size distribution and hydrometer tests were performed on Achrafieh clay specimens. The results show that this natural clay has a composition of more than 50% fines, while the remaining part is primarily composed of fine sands. The grain size distribution curve as obtained from sieve analysis and hydrometer tests is presented in Figure 3.3.

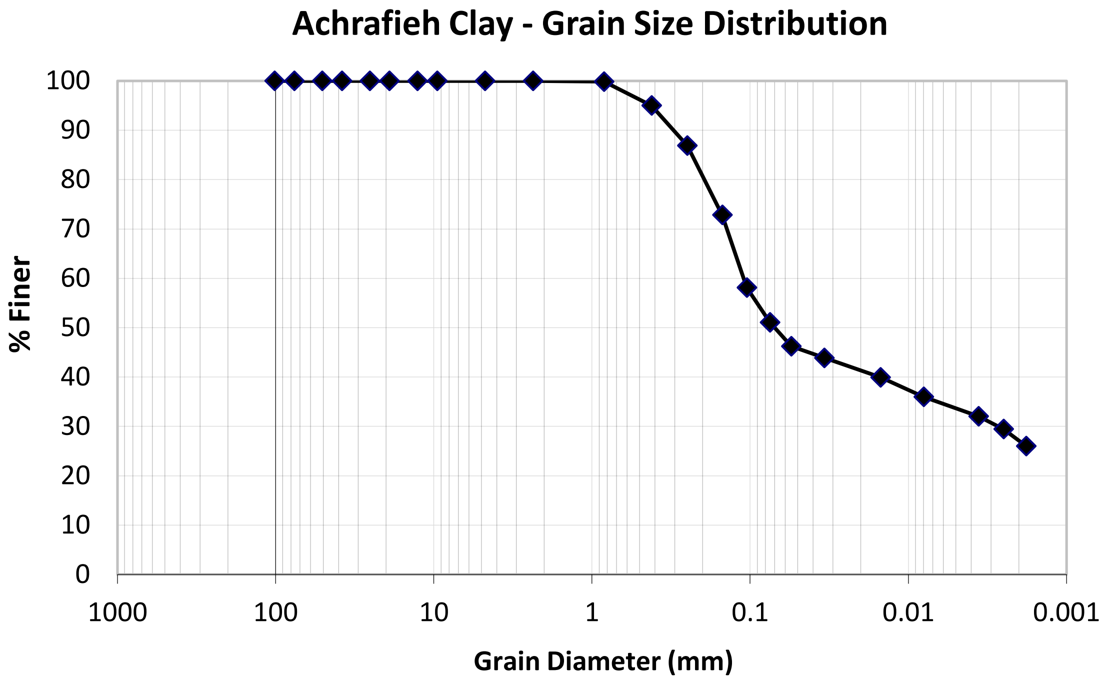


Figure 3.3: Grain size distribution curve of Achrafieh clay

As for the consolidation properties of the Achrafieh clay specimens, these were calculated from a one-dimensional consolidation tests (oedometer test) that was conducted on Achrafieh clay sample having a diameter of 5.00cm and a height of 1.92cm. The test specimen was trimmed from a larger slurry-consolidated sample that was initially consolidated in a prefabricated consolidometer using a vertical effective stress of 100 kPa (see section 3.3.2). The specific gravity, initial water content, and initial void ratio of the slurry-consolidated specimen are presented in Table 3.2. The consolidation test was performed in accordance with the requirements of ASTM 2435. Figure 3.4 shows the variation of the void ratio versus the logarithm of the effective vertical stress (e-log p curve), where the void ratio is defined at the end of each load increment (24 hours from the onset of loading). Based on the e-Log p curve presented in Figure 3.4, the virgin compression (C_c) and the swelling (C_s) slopes are computed as

0.174 and 0.0142, respectively. In addition, the pre-consolidation pressure from the e-log p curve was determined to be ≈96 kPa according to Casagrande's approach.

Table 3.2: Initial properties of 1-dimensional consolidation test specimen of Achrafieh clay

Specific gravity	2.63
Initial water content (%)	21.8
Initial void ratio	0.60
Initial saturation (%)	95.4 (assumed as 100%)

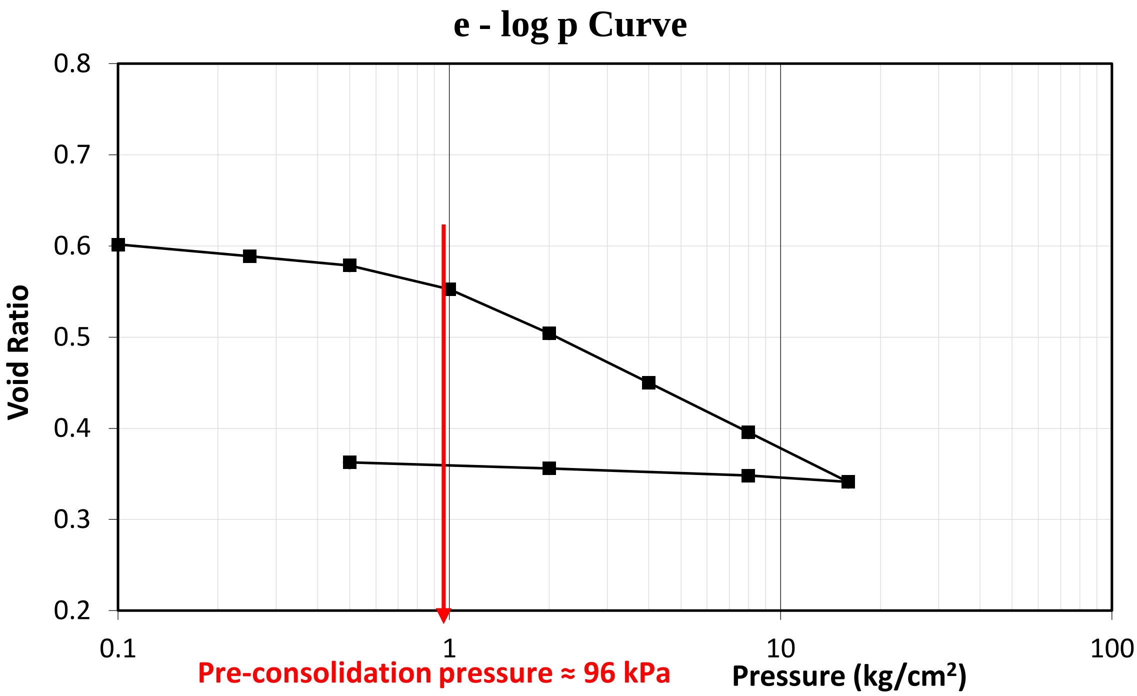


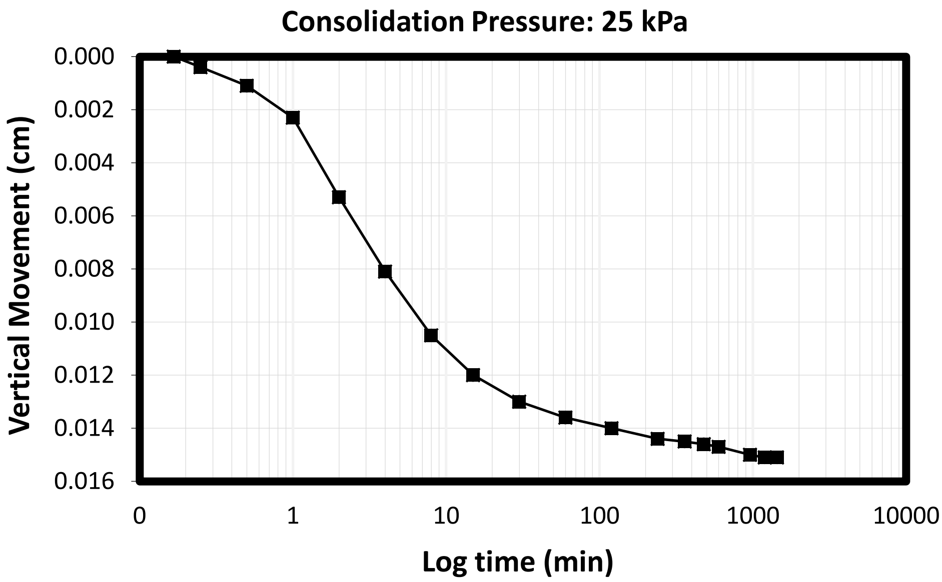
Figure 3.4: e-log P curve for normally consolidated Achrafieh Clay

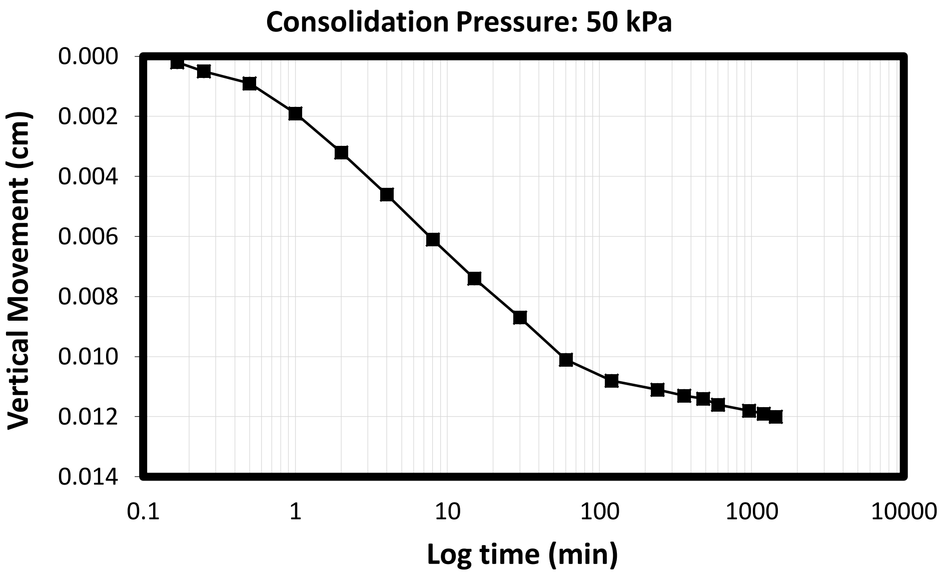
The values of the coefficient of consolidation were calculated based on (t_{50}) that corresponds to the time required to achieve an average degree of consolidation of 50%. According to Casagrande's approach, t_{50} is determined by plotting the vertical settlement of the specimen against the log of time for a given consolidation pressure. The time corresponding to 50% consolidation is considered as t_{50}. The coefficient of consolidation, C_v, can then be obtained as a function of t_{50} such that $C_v = \frac{T*H^2}{t_{50}}$, where T_{50} is the time factor corresponding to an average degree of consolidation of 50% (equal to 0.197), and H is the drainage path which is half the height of the specimen.

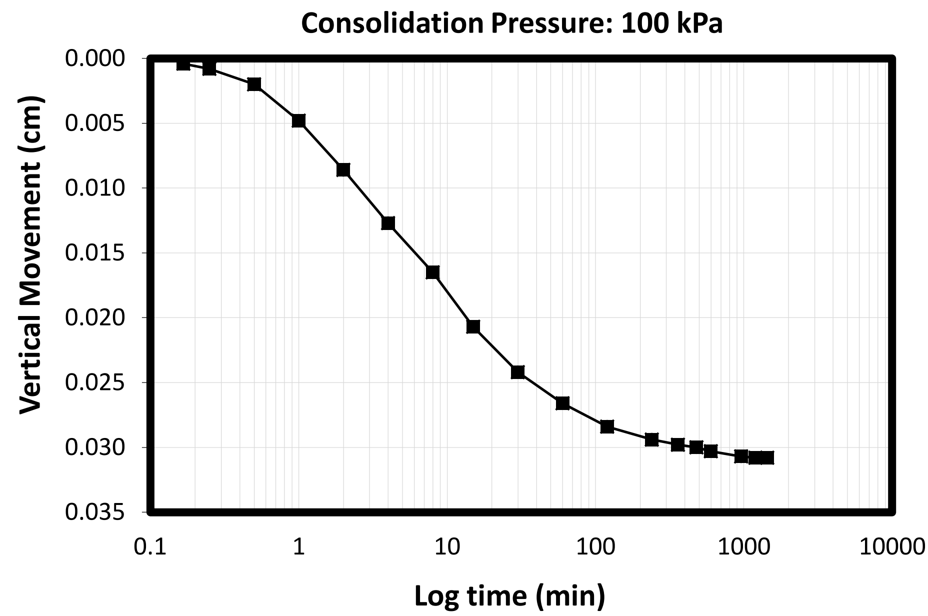
Calculated values for the coefficient of consolidation C_v, are presented as a function of the vertical stress in Table 3.3. Measured time-settlement curves for the typical pressures of 25kPa to 400kPa are also shown in Figure 3.5 for the log-time method.

Table 3.3: Calculated values for the coefficient of consolidation Cv as a function of the vertical stress

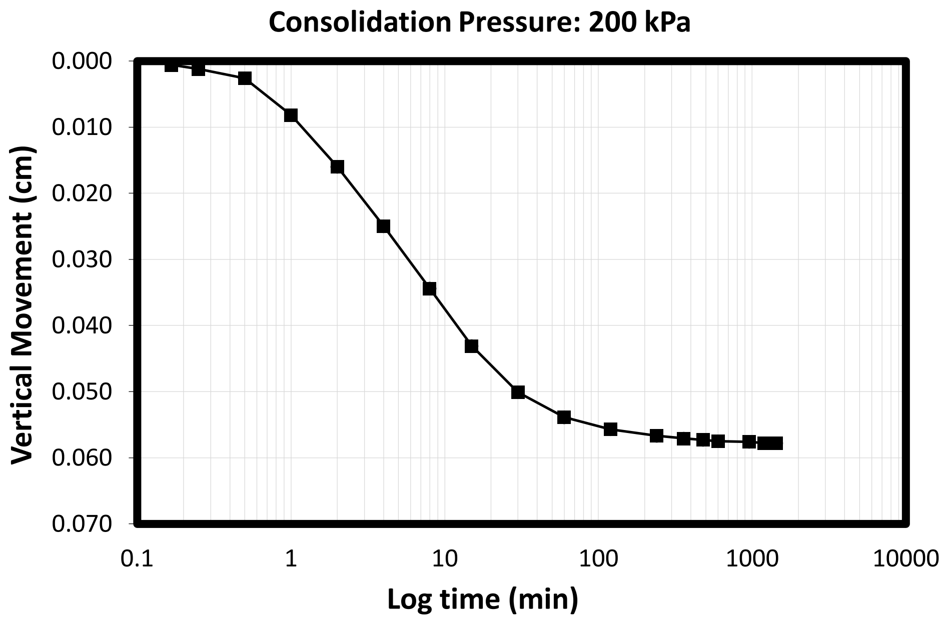
Consolidation pressure (kPa)	Coefficient of consolidation, C_v (cm^2/min)
	From t_{50}
25	0.071
50	0.022
100	0.041
200	0.039
400	0.051
800	0.064
1600	0.065
Average	0.050


Consolidation Pressure: 25 kPa
Vertical Movement (cm)
0.000
0.002
0.004
0.006
0.008
0.010
0.012
0.014
0.016
0
1
10
100
1000
10000
Log time (min)


Consolidation Pressure: 50 kPa
Vertical Movement (cm)
0.000
0.002
0.004
0.006
0.008
0.010
0.012
0.014
0.1
1
10
100
1000
10000
Log time (min)



Consolidation Pressure: 100 kPa
Vertical Movement (cm)
0.000
0.005
0.010
0.015
0.020
0.025
0.030
0.035
0.1
1
10
100
1000
10000
Log time (min)



Consolidation Pressure: 200 kPa
Vertical Movement (cm)
0.000
0.010
0.020
0.030
0.040
0.050
0.060
0.070
0.1
1
10
100
1000
10000
Log time (min)

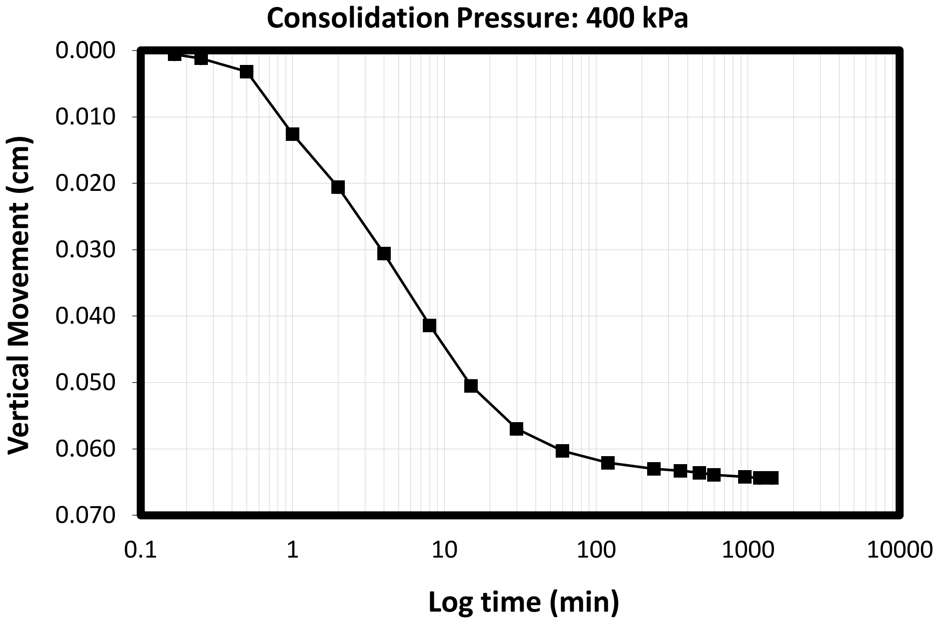


Figure 3.5: Displacement vs. Log time for consolidsation pressures of 25, 50, 100, 200 and 400 kPa

3.2.2 Ottawa Sand

Ottawa Sand was used in the preparation of the sand columns that were installed in the Achrafieh clay specimens. Ottawa sand is a well-known laboratory tested material. Previous soil classification tests conducted by Bou Lattouf (2013) on Ottawa sand indicated that the particles have a mean diameter, D_{50} of 0.34mm, a uniformity coefficient, U_c of 2.3, and a coefficient of curvature, C_c of 0.82. The sand classifies as poorly graded sand (SP) according to the Unified Soil Classification System (USCS). The index properties for Ottawa sand and the sieve analysis results are presented in Table 3.4 and Table 3.5 respectively, while the particle size distribution curve is shown in Figure 3.6.

Table 3.4: Index properties of Ottawa sand

D_{10} (mm)	0.22
D_{30} (mm)	0.3
D_{60} (mm)	0.5
Coefficient of uniformity, U_C, (D_{60}/D_{10})	2.3
Coefficient of curvature, C_c, $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$	0.82
Soil classification (USCS)	SP
Maximum void ratio (e_{max})	0.49
Minimum void ratio (e_{min})	0.75
Specific gravity	2.65

Table 3.5: Sieve analysis results for Ottawa sand

Sieve No.	Diameter (mm)	Weight of retained soil (gm)	Cumulative percent retained (%)	Cumulative percent finer (%)
20	0.84	0	0.0	100.0
40	0.42	223.8	28.0	72.0
60	0.25	464.4	86.2	13.8
100	0.15	87.2	97.1	2.9
140	0.105	18.5	99.5	0.5
200	0.075	1.5	99.6	0.4
pan		2.8	100.0	0.0

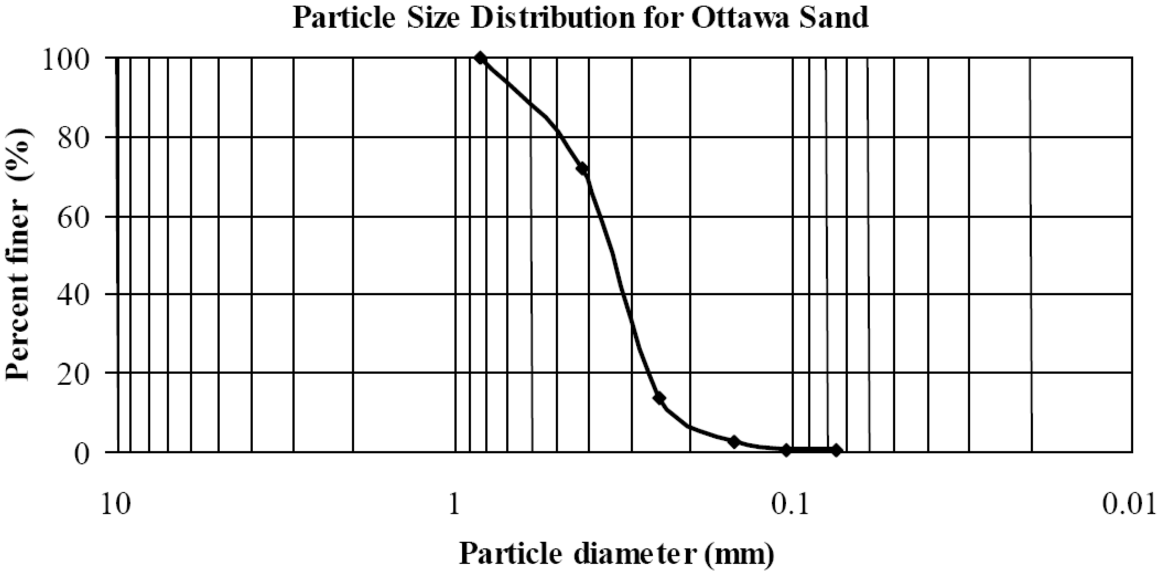


Figure 3.6: Sieve analysis curve for Ottawa sand

With regards to the shear strength of Ottawa Sand, results from consolidated undrained (CU) and consolidated drained (CD) triaxial tests that were conducted on medium-dense (dry density of 16.2kN/m3 corresponding to a relative density of ≈56%, and a void ratio of 0.60) Ottawa sand specimens under confining pressures of 100, 150, and 200kPa indicated that the sand has a friction angle of about 33 degrees in CU-triaxial tests (Maakaroun 2009) and 36 degrees in CD-triaxial tests (Maalouf 2011).
In this study, consolidated undrained (CU) and consolidated drained (CD) triaxial tests were performed on dense Ottawa sand to study the behavior of dense sand used in the reinforced specimens and to obtain the Mohr Coulomb parameters (c’ & Ø’). The specimens tested had a height of 14.2cm and a diameter of 7.1cm and were prepared at a dry density of 17.04kN/m^3, corresponding to a relative density of ≈86%, and a void ratio of 0.53. The tests were also performed under confining pressures of 100, 150 and 200kPa. Results of these tests will be presented in chapters five and six.

3.3 Preparation of Normally Consolidated Achrafieh Clay Samples

3.3.1 Preparation of Achrafieh Slurry

Achrafieh clay powder was mixed with water at a water content of 50%, which is approximately equal to twice the liquid limit of the clay. A mass of 0.5kg of clay was initially mixed with 0.25 liters of water by means of an electric mixer with a capacity of 1.5 liters as shown in Figure 3.7. To ensure proper mixing and homogeneity of the slurry material, the slurry was mixed for two minutes at 200rpm.

Figure 3.7: Electric Mixer for preparing Achrafieh slurry

3.3.2 One-Dimensional Consolidometers

Four 1-D consolidometers were fabricated for the purpose of consolidating the Achrafieh. Each consolidometer consisted of a PVC pipe segment with a height of 35cm with external and internal diameters of 7.3cm and 7.1cm, respectively. The PVC pipe segment was cut longitudinally in the vertical direction into two halves to function as a split mold (Figure 3.12) thus eliminating the need for extruding the soil sample after consolidation. The two PVC sections were held in place using high-strength duct tape which was wrapped around the two cylindrical PVC sections to prevent leakage of slurry and to ensure that lateral strains are negligible during 1-D consolidation under the desired axial load. The advantage behind using a split PVC pipe was to ensure that an undisturbed, relatively soft, normally consolidated clay specimen can be obtained and removed with minimal disturbance after consolidation was achieved.

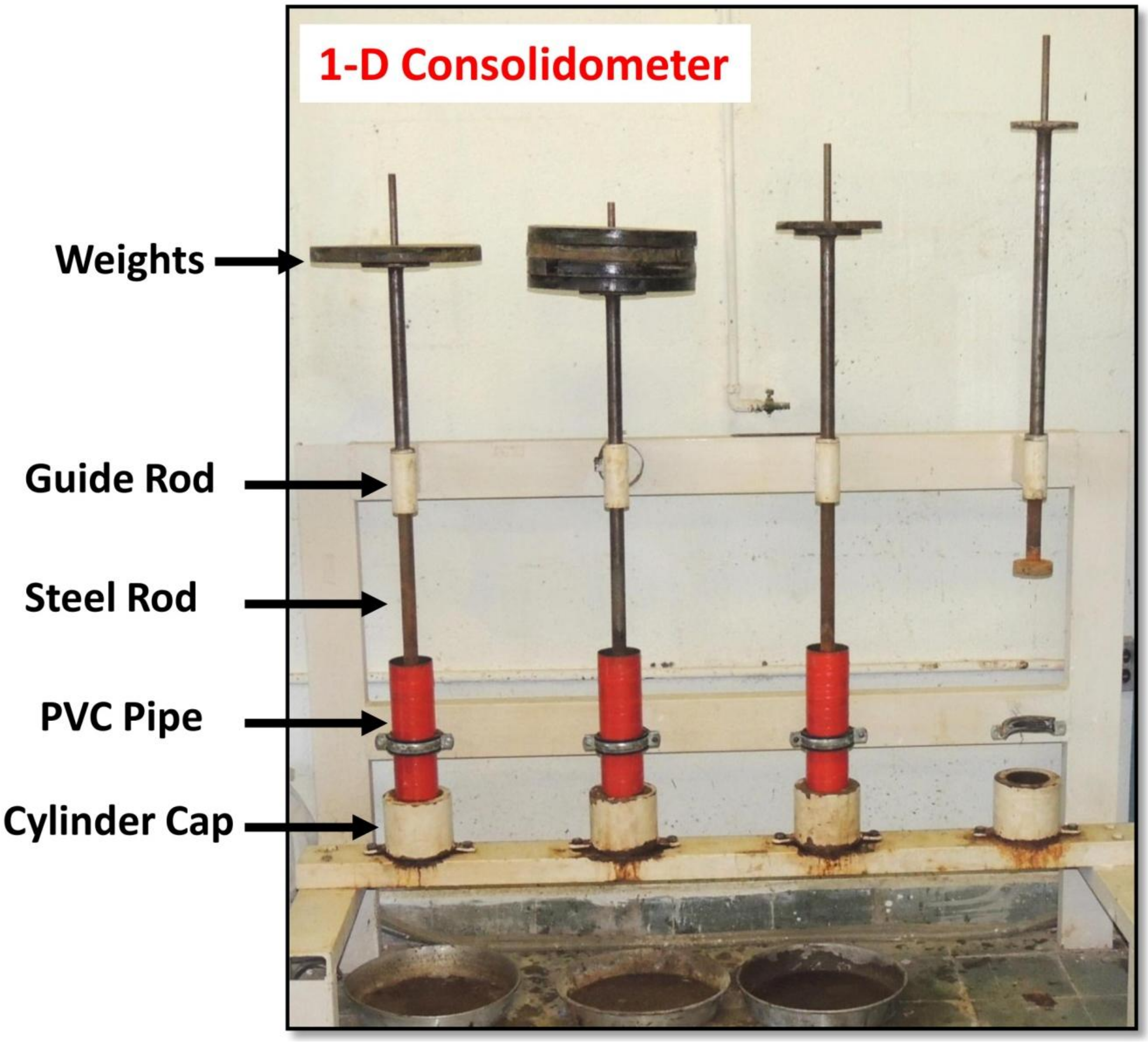


Figure 3.8: Custom fabricated 1-dimensional consolidometers

Figure 3.9: Split PVC pipe and wrapped PVC pipe with duct tape

At its lower end, the PVC pipe segment was fixed in place by means of a hollow steel cylinder with a height of 9cm as shown in Figure 3.10. The stiff and heavy cylinder wraps tightly around the bottom of the PVC segment in order to provide additional lateral confinement and support to the PVC segment during slurry consolidation. The inner walls of the steel cylinder were coated with a thin layer of oil to facilitate the removal of the PVC segment once consolidation was achieved. Moreover, the circumference of the steel rod was coated with a thin layer of grease at the location of the steel rod guide to reduce friction between the steel rod and the guide rod. In addition, a porous stone and a filter paper were placed at the bottom to provide a freely draining boundary at the lower end of the soil specimen.

At its upper end, the soil specimen was loaded with a loading system consisting of dead weights similar to those used in 1-D consolidation tests. The dead weights were seated on a circular steel plate that transferred the load to the top of the soil specimen through a circular steel rod having a diameter of 1cm. A perforated circular steel piston with a diameter of 7.1 cm (same as inner diameter of PVC pipe) was fixed to the bottom of the steel rod to act as a loading plate which transmitted the load to the slurry. The soil was separated from the loading plate with a porous stone and a filter paper to provide a freely draining boundary at the top of the soil specimen. To reduce friction between the perforated loading plate and the PVC segment, the outside periphery of the loading plate was also coated with a thin layer of oil.

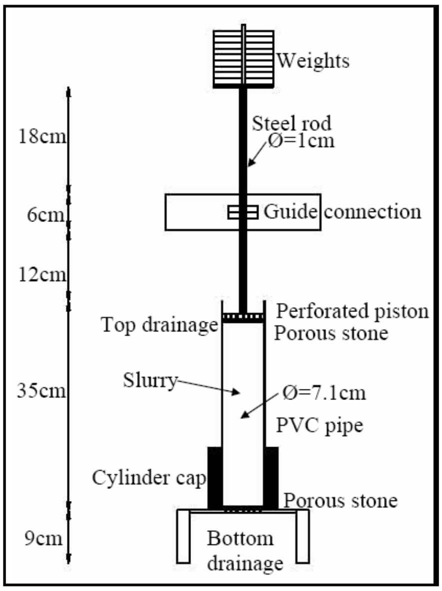


Figure 3.10: Custom fabricated 1-dimensional consolidometer

3.3.3 One-Dimensional Consolidation of Achrafieh Clay

The slurry was poured into the appropriate consolidometer and consolidated under k_o conditions under an effective vertical stress of 100 kPa. With four consolidometers, four clay samples could be prepared simultaneously. Each consolidometer could handle a volume of slurry that is approximately equivalent to two mixed batches of Achrafieh slurry, i.e. one kg of Achrafieh clay mixed with half a liter of water. After pouring the slurry in the appropriate consolidometer (initial specimen height was ≈33cm), the clay was allowed to consolidate under its own weight for a period of 4 hours before loading was started. During 1-D consolidation, drainage was allowed from both ends of the sample through the top and bottom porous stones. Dead weights were then added in stages to the top of the sample, with each weight applied for a specified time period according to the loading sequence shown in Table 3.6 below.

Table 3.6: Loading sequence during 1-D consolidation of Achrafieh slurry

Accumulated weights (Kg)	0.5	1	2	4	8	12	20	30	40
Applied pressure (kPa)	1.25	2.5	5	10	20	30	50	75	100
Duration (Hr)	4	4	24	24	24	24	24	24	24

The consolidation time periods that were allocated to each loading increment were estimated based on the results of the 1-D consolidation test, and then adjusted by trial and error. The objective was to develop a loading sequence which was repeatable, and which resulted in uniform consolidated Achrafieh clay specimens. The typical time duration for primary consolidation for a clay sample under an effective normal stress of 100kPa is approximately 7.5 days.

Additional measures were further taken to help reduce disturbance during sample preparation. These measures included spreading a thin layer of oil over the inner surfaces of the PVC pipes to reduce friction between the Achrafieh specimens and the inner surface of the pipe. This allowed for dismantling the pipe and removing the soil specimen from the consolidometer with minimal disturbance to the soil specimen.

3.3.4 Sample Preparation Prior to Placement in the Triaxial Cell

At the end of the primary consolidation phase, the dead loads were removed and the PVC cylinder was slowly pulled out from the cylindrical cap of the consolidometer. The duct tape surrounding the periphery of the PVC cylinder was unwrapped and the two PVC pieces were dismantled. The clay specimen was then trimmed to a final height of 14.2cm (initial height is about 22 cm) by means of a sharp spatula. Two presoaked porous stones were then placed on the top and bottom of the Achrafieh specimen and the sample is prepared for triaxial testing. Only unreinforced specimens that count as control tests, are wrapped with a presoaked filter paper that has longitudinal perforations in order to accelerate the process of consolidation in the triaxial cell.

Figure 3.11: Procedure for removing clay specimen after the end of primary consolidation phase

A thin rubber latex membrane with a diameter of 7.1cm was then placed on the inside of a cylindrical brass membrane stretcher. To facilitate the placement of the membrane into the stretcher, a thin layer of powder was sprayed over the membrane. Vacuum was then applied to ensure that the membrane adhered well to the inner walls of the stretcher. The stretcher was then positioned around the soil specimen and the vacuum was released. Rubber bands, O-rings, were used to fasten the membrane tightly around the specimen. The specimen was then attached to the base of the triaxial cell and the top drainage tubes were inserted into the holes of the top cap. The triaxial cell was then assembled and the seating piston positioned over the top cap. Figure 3.12 illustrates the above mentioned procedure. Finally, the triaxial cell was placed in the "TruePath" system in preparation for saturation, consolidation, and shearing as will be explained in Chapter#4.

Figure 3.12: Preparation of Achrafieh clay specimen for triaxial testing

3.4 Preparation of Sand Columns

The first step in the preparation of the clay specimens to be reinforced with single sand columns involved the formation of a hole with a diameter of 3cm or 4cm, in the middle of the clay specimen, depending on the area replacement ratio required. For this purpose, a custom-fabricated hand augering apparatus was manufactured in the machine shop. The procedure followed in drilling the holes is presented below.

After dismantling the cylindrical Achrafieh clay specimen from the PVC pipe and trimming it to a final height of 14.2cm, the specimen was wrapped with two lubricated plastic cylindrical PVC tubes making a diameter of 7.1cm and a length of ≈16cm. These tubes were in turn wrapped with duct tape around their circumference. The wrapped specimen was then placed on the augering apparatus that is shown in Figure 3.13 below. Augers with diameters of 3cm or 4cm were connected to the augering machine. During drilling, the vertical alignment of the rotating rod is maintained through the presence of plastic guide plates that are connected to the top and bottom of the steel rod. The penetration of the auger into the specimen is continued in stages until full penetration occurred. Afterwards, a previously prepared frozen sand column with the required diameter was installed in the augered hole.

Figure 3.13: Augering apparatus of Achrafieh clay specimens

Ottawa sand columns with a diameter of 3cm were prepared in a split mold having an inner diameter of 3cm and a height of 16cm, whereas columns with a diameter of 4cm where prepared in a split mold having the same height but with an inner diameter of 4cm. The split mold consists of two PVC sections that are held in place using high-strength duct tape (Figure 3.14). Ottawa sand was placed in the mold in three layers, and every layer was vibrated and tamped for a period of one minute. Prior to placing the sand in the mold, the required column height of 14.2cm, was marked on the inner and outer sides of the mold by means of a marker and the weight of the sand required to reach the desired density to be poured into the column was determined. The dry density of the medium-dense sand columns after vibration was 16.2 kN/m^3 ± 0.1, whereas the dry density for the dense Ottawa sand columns was 17.04 kN/m^3 ± 0.2, irrespective of the pipe diameters.

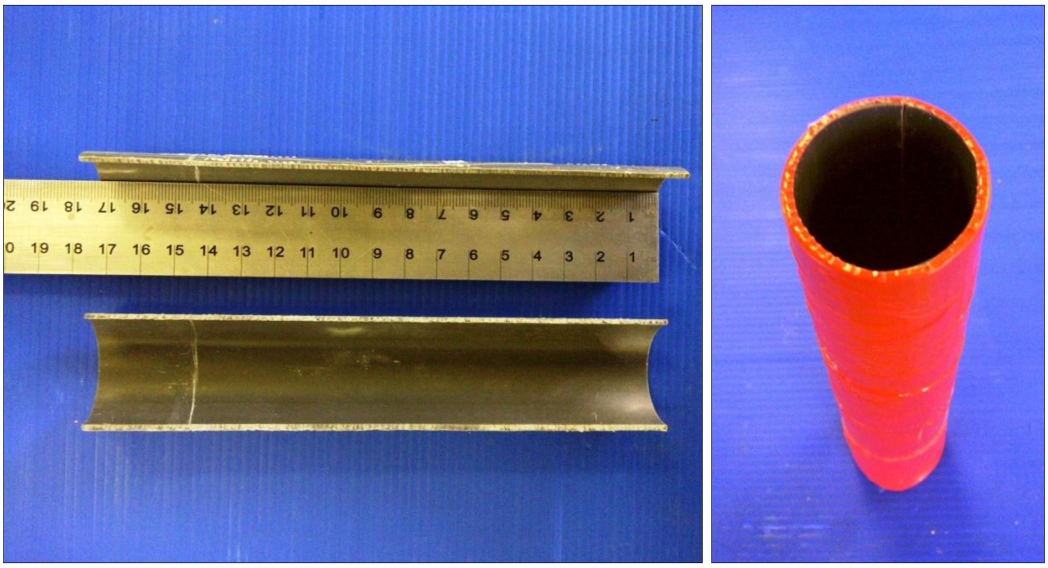


Figure 3.14: Split mold for Ottawa sand columns

After ensuring that the target dry density was achieved, the sand column was soaked with water which was permeated slowly from the top of the column to its bottom. At the end of this process, it was found that the sand columns generally would have a water content of about ≈19%. The column was then placed inside the freezer and left overnight. After freezing, the high strength duct tape surrounding the periphery of the PVC cylinder was unwrapped and the two PVC pieces were dismantled. The frozen sand column was then inserted in the predrilled hole and left to thaw. It is worth noting that this method permitted the preparation of relatively uniform, homogeneous and repeatable sand columns throughout the different phases of the testing program. The process of sand column preparation is illustrated in Figure 3.15.

Although freezing of sand columns is not usually implemented in the field, the idea behind using frozen sand columns in this research is to be able to construct columns with mechanical properties that are repeatable and uniform across the different samples. The friction angle of Ottawa sand depends on the initial density of the column material, which in turn depends on the column diameter. Thus, any variation in the column diameter form one sample to another will lead to variations in the column density and the friction angle of the column material. By constructing frozen columns in which sand particles are compacted outside the Achrafieh clay specimen in a split mold system, the column diameter and density will be uniform and repeatable.

3.5 Summary

Index and compressibility characteristics for the Achrafieh clay were presented in a comprehensive way in this chapter in addition to the presentation of the engineering properties, particle size distribution, and shear strength of Ottawa sand.

Achrafieh clay specimens were prepared from slurry and consolidated in a prefabricated one- dimensional consolidometer after which the Achrafieh specimen was arranged for testing. Step by step methods for preparing frozen sand columns were discussed and pictures and photos were displayed for the purpose of clarifying the sand preparation and the installation processes.

Figure 3.15: Photographs presenting the process of preparing and installing sand columns

CHAPTER FOUR

TRIAXIAL TESTING

4.1 Introduction

This chapter describes the method and steps to be followed in performing consolidated drained, undrained and partially drained tests using the automated triaxial "TruePath" equipment. The step by step approach which describes the process from the initial stage of seating the test specimen to the final stage of shearing the specimen under drained, undrained and partially drained conditions is designed to be a guide for future users of the "TruePath" equipment.

4.2 General Steps in Performing Consolidated Drained (CD), Consolidated Undrained (CU) and Consolidated Partially Drained (PD) Tests

After preparing the Achrafieh clay specimen as described in Chapter 3, the triaxial cell (with the sample inside it) is placed in the automated triaxial "TruePath" system. The main components of the system are presented in Figure 4.1. The "TruePath" system consists of four main parts which are the load frame with the pressure transducer and the deformation sensor, the cell pump which provides the confining cell pressure to the cell chamber, the back/pore pump which provides the back pressure for the specimen and measures the pore water pressure through connecting a pressure transducer to valve#3 (as will be explained in a later stage), and the operating system which allows the user to perform the test and monitor its progress through the screen that displays all the stages of the test.

Each triaxial test consists of four stages which include seating, back pressure saturation, consolidation, and shearing. Each stage is characterized by a series of commands that appear at the top of the screen and guide the user throughout the test. The four tabs, which represent each stage, become active after the specimen and the test data files are created. A specific tab representing a specific stage will become active only after the previous stage is completed.

The following steps describe the detailed procedure to be followed in performing consolidated drained tests (CD) on normally consolidated clay samples.

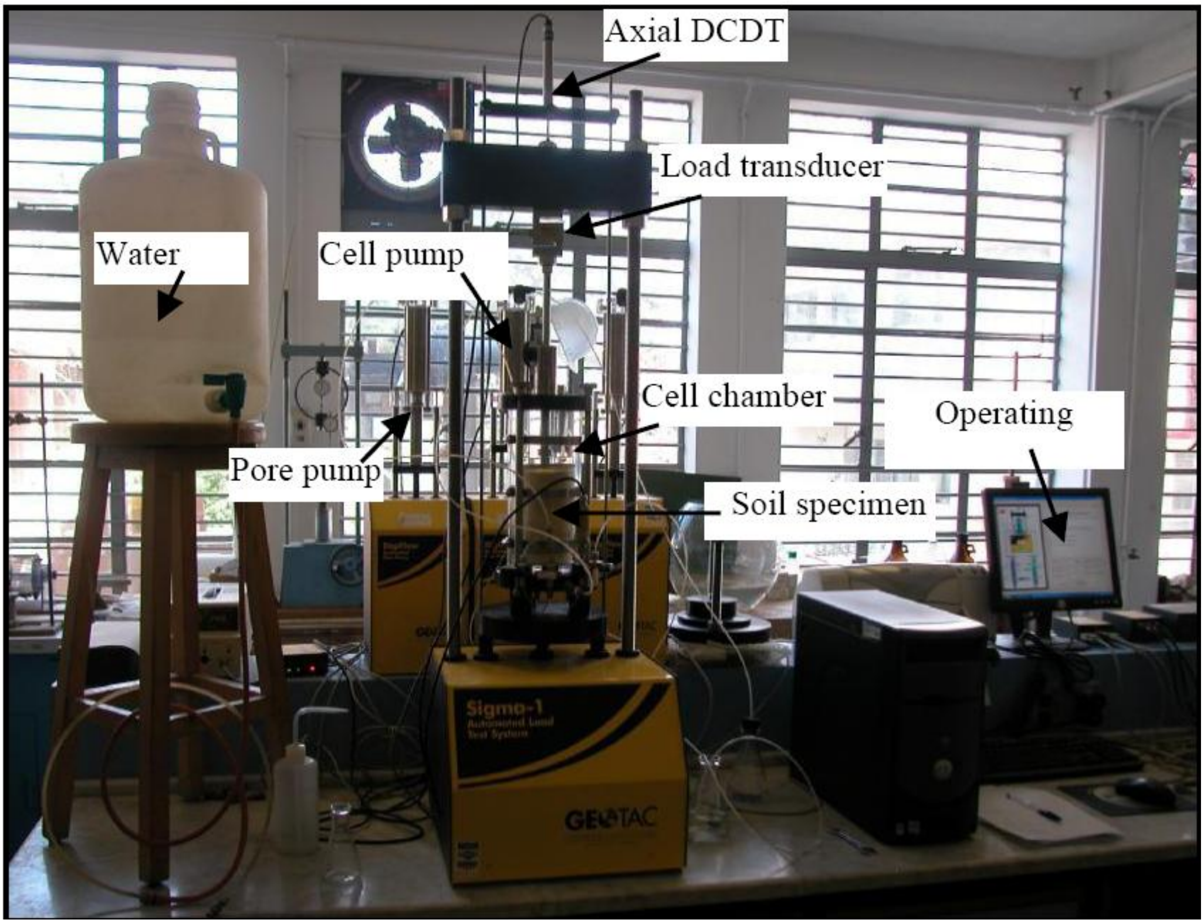


Figure 4.1: Automated triaxial TruePath system

4.3 Creating Specimen and Test Data Files

In order to view the test results while performing the test, the file called "graph initiative" should be deleted prior to the start of the test from the "TruePath" folder which is located under the “program files” folder. The first step in performing a triaxial test involves setting all the sensors and the load transducer readings to zero. This can be achieved by entering the “Set Up” menu and selecting “Sensor”. After highlighting the required sensor or transducer and pressing “Test”, a pop-up window will appear for the selected sensor. On this window, the “Take Zero” button should be pressed so that the sensor reading will indicate the average of ten consecutive readings that are almost zero. This process should be repeated for all the sensors, i.e. pore pressure, back pressure, cell pressure, external load cell and axial DCDT sensors. Figure 4.2 and Figure 4.3 show a step by step procedure for setting the sensors to zero readings.

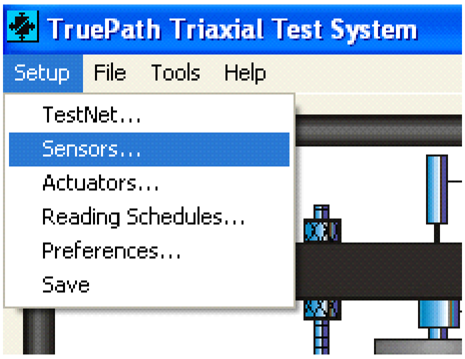


Figure 4.2: Accessing the sensors

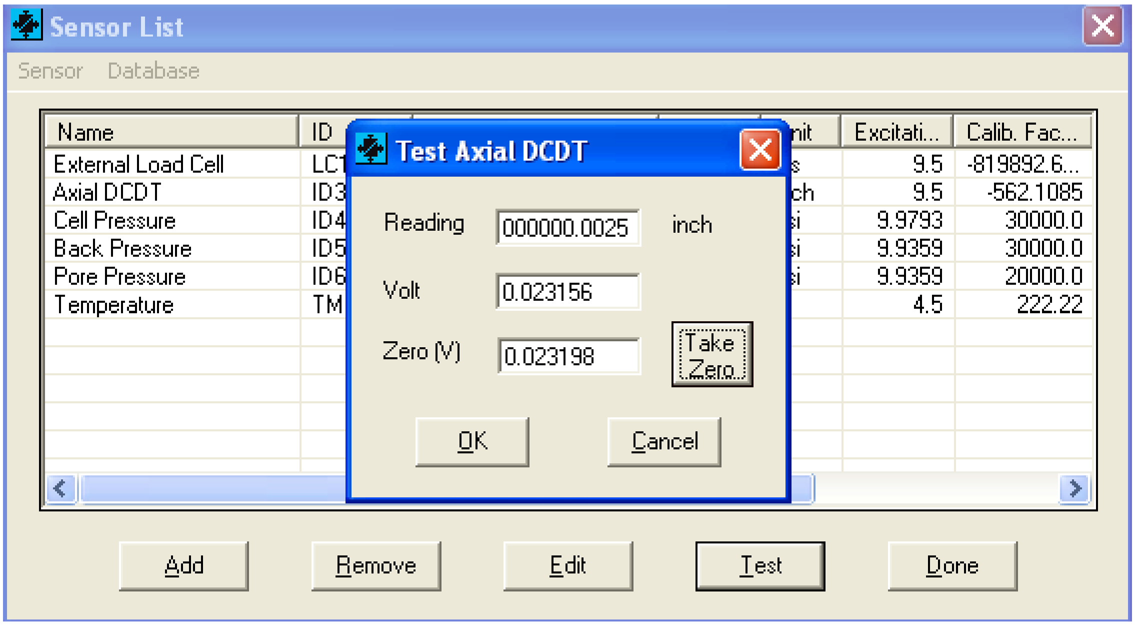


Figure 4.3: Setting the sensors to zero

The second step involves accessing the "File" menu and choosing "Specimen Data" as shown in Figure 4.4. Then the "specimen data" window will appear as shown in Figure 4.5 where the user has to fill the appropriate information which includes the sample height (5.59 inch), sample diameter (2.79 inch), and the sample number and project number. The third step is also initiated from the "File" menu by selecting "Test Data" as shown in Figure 4.4. A window will appear as shown in Figure 4.6 where the user has to enter the control test parameters in the empty spaces.

The input data for the test consists of four categories that are included in one window. The user has to enter the following:

- The value of the target seating pressure which is defined as the seating confining pressure needed to keep the membrane pressed against the specimen during the flushing of the drain lines. A pressure of 10 Psi (≈70kPa) is used for the samples in the testing program.

- The value of the saturation/back pressure that is needed to saturate the sample. A pressure of 45 Psi (≈310 kPa) is chosen for the Achrafieh clay sample to ensure proper saturation.
- The type of consolidation (isotropic in this test program) and the value of the target effective stress at which the sample will be consolidated. Since the test program involves three different confining cell pressures, values of 14.5psi, 21.75psi or 29psi (corresponding to 100, 150 and 200kPa respectively) were input at the target effective stress depending on the confining pressure required to be applied on the Achrafieh clay sample during the consolidation phase. Therefore, prior to each consolidation phase, the effective stress is still 10psi that is the seating pressure applied in the seating phase. Once the consolidation phase is started, the effective stress in increased to values of 14.5, 21.75 or 29psi, depending on the confinement pressure required, and at a stress rate of 200psi per hour to guarantee instantaneous application of the consolidation pressure.
- The drainage conditions which were defined in this testing program to be either "consolidated drained" (CD) for CD and PD tests, or "consolidated undrained" (CU) for CU tests. The loading direction was chosen to be "compression". The maximum vertical effective stress was taken as 150 Psi (≈1036 kPa). The maximum strain was taken as 25%. The strain rate was taken as 0.375%/hr for the drained tests, range of 1%/hr to 60%/hr for the partially drained tests and 1%/hr for the undrained tests. In addition, shearing was set to be terminated when either the maximum stress or the maximum strain is reached.

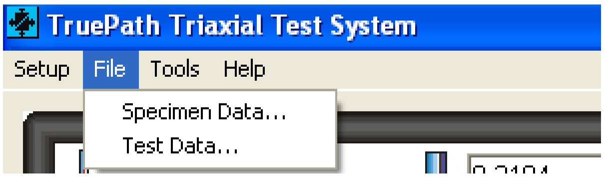


Figure 4.4: Entering file menu to select Specimen Data

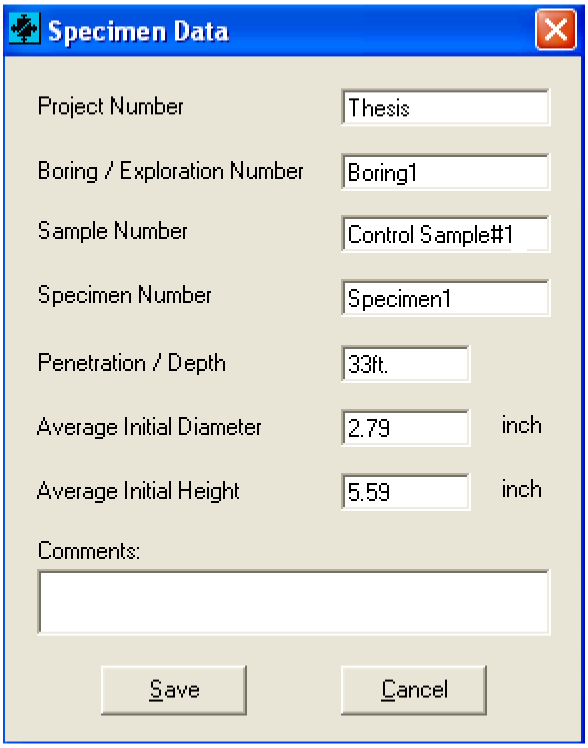


Figure 4.5: Entering the specimen data information

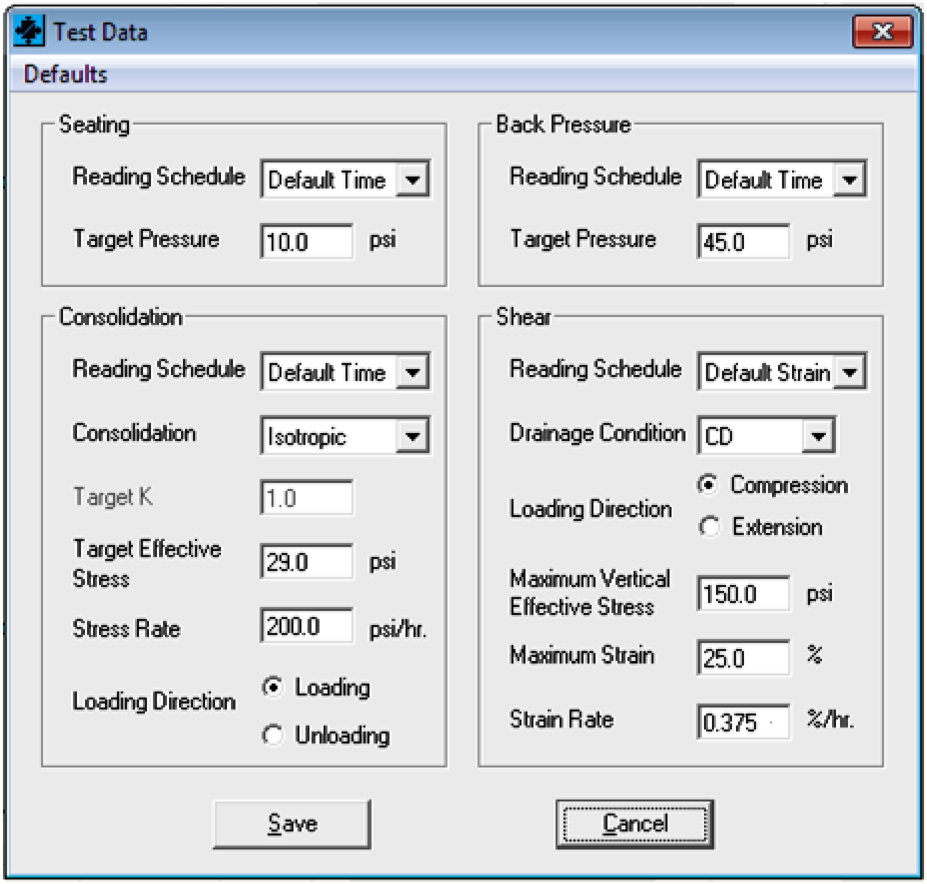


Figure 4.6: Entering the control test parameters

4.4 Seating Stage

After entering the specimen data, the "Seating" tab becomes active. The seating process involves seating the piston, adjusting the external load transducer, filling the cell with water, selecting the cell pressure, flushing the drains, and maintaining the volume of the sample.

4.4.1 Seating the Piston

The process of seating the piston involves locking the piston and minimizing the gap between the piston and the load button using manual control. This is achieved by entering the "Tools" menu, selecting "Manual Mode", pressing on the "Load Frame" and then pressing on the first upward button. When the “Start” button is pressed, the platen will move upward till it reaches the load button. Figure 4.7 and Figure 4.8 below show the sequence followed for reducing the gap between the piston and the load button.

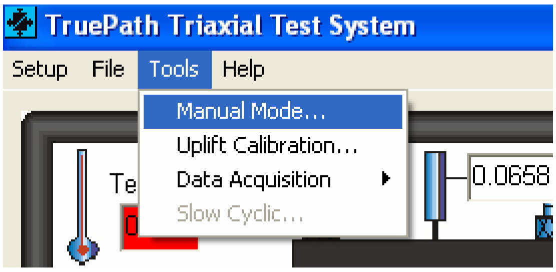


Figure 4.7: Selection for the manual mode

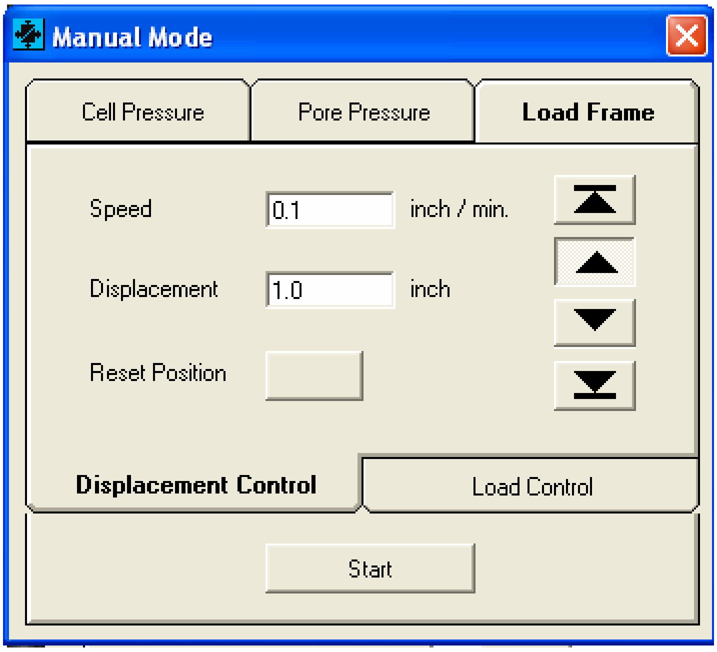


Figure 4.8: Reduction of gap between the piston and the load button

After reducing the gap, the “start” button is pressed as shown in Figure 4.9 and another window will appear. In this window, the “Start” button has to be pressed again and the platen will move upward till it reaches the load button and the platen stops automatically when the load button is seated on the piston.

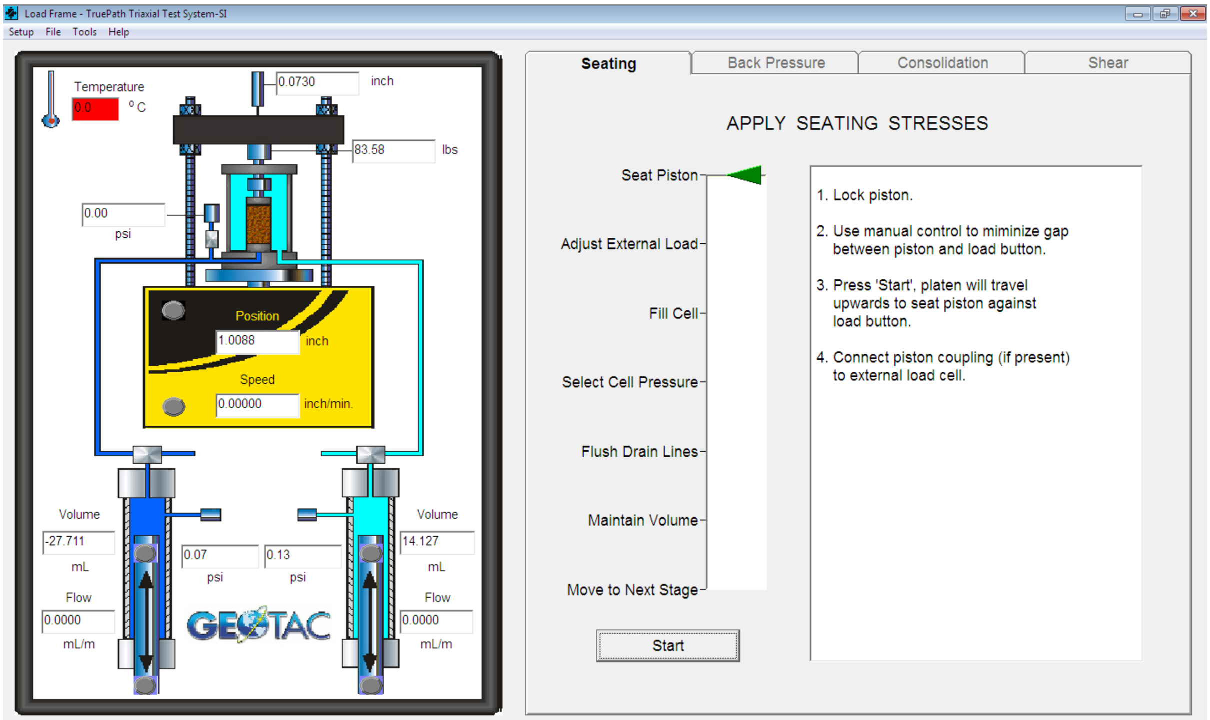


Figure 4.9: Window for seat piston

4.4.2 Adjust the External Load Sensor

When the “Adjust external load” button is pressed followed by pressing the “Start” button, the reading of the load cell becomes almost zero. The piston should be unlocked when the load cell reading approaches zero. Figure 4.10 shows the procedure for adjusting the load.

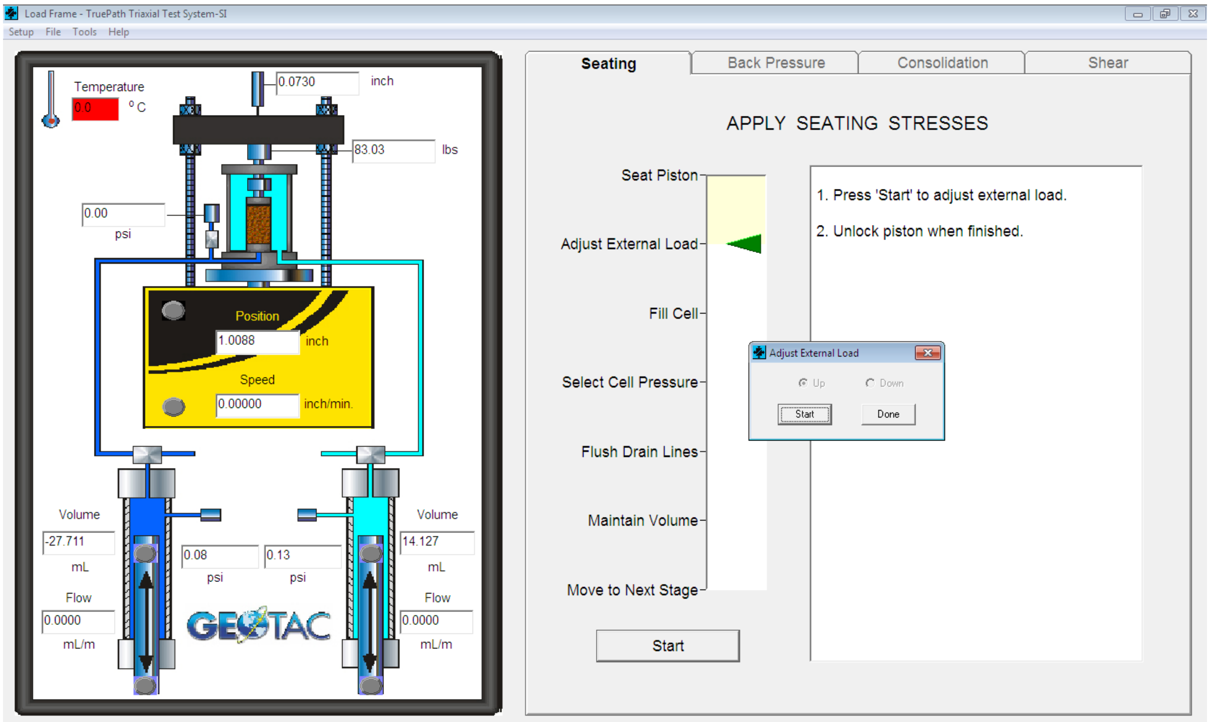


Figure 4.10: Adjustment for the external load transducer

4.4.3 Fill the Cell Chamber with Water

To fill the cell chamber with water, the "Fill Cell" button needs to be clicked and the ventilation air valve should be inserted into the top of the cell as shown in Figure 4.11. Then water should be supplied from an elevated water tank to the bottom quick connect of the cell through a plastic hose with a fitting on its top to allow entrance of the hose into the cell. The air in the cell is displaced by the water and is allowed to escape through the vent port. After filling the cell from the bottom to up, water is allowed to flow out from the air vent port to ensure that all the air was driven out of the cell. The elevated water source should then be closed and the water hose is removed together with the air vent valve. The user can follow the step by step instructions that are displayed on the screen for the purpose of filling the cell with water as shown in Figure 4.12.

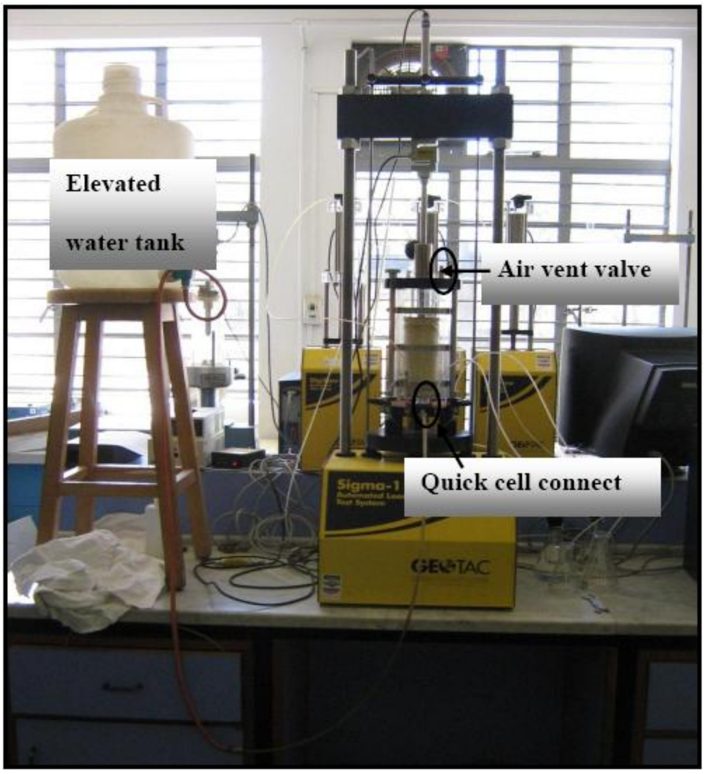


Figure 4.11: Filling the cell chamber with water

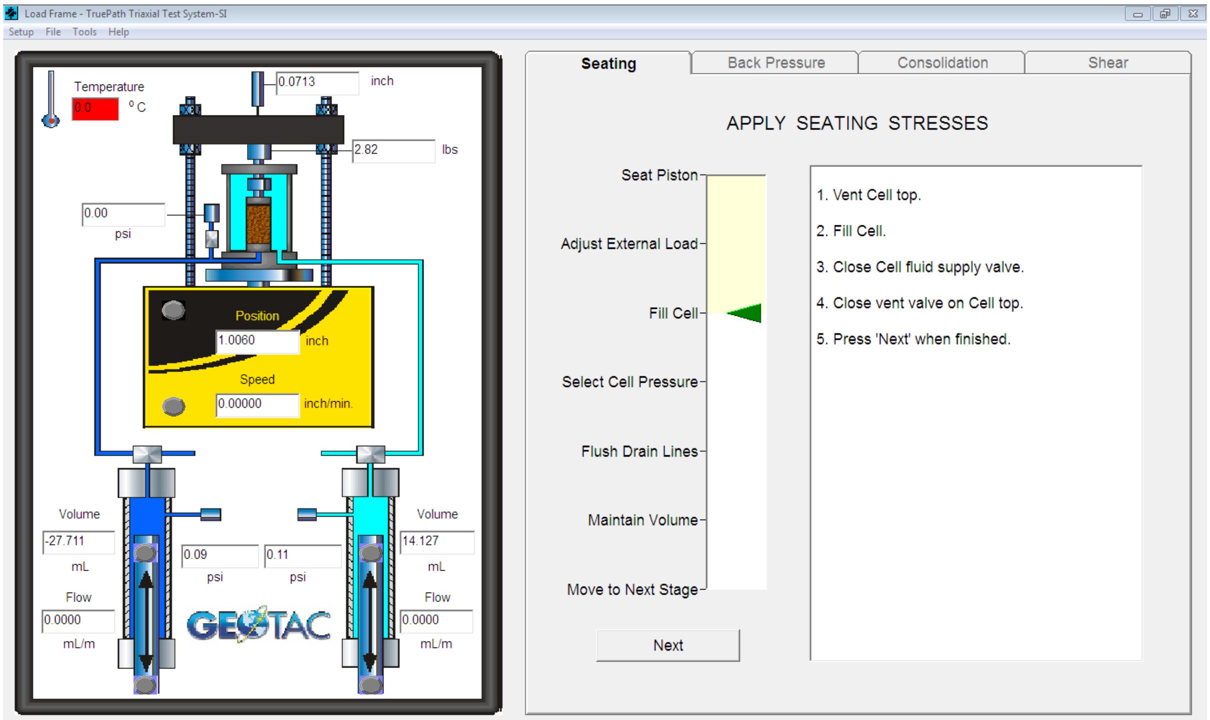


Figure 4.12: Steps for filling the cell chamber with water

4.4.4 Cell Pressure Selection

A small confining pressure of 10 psi (≈70kPa) is exerted on the Achrafieh sample in the Select Cell Pressure step for the purpose of keeping the membrane pressed against the specimen during the drain line flushing. This can be achieved by opening the port valve of the cell pressure and connecting the cell pump pressure line to the cell bottom quick connect as shown in Figure 4.13. The "Start" button should then be pressed to produce a window in which a pressure of 10psi should be entered. After around two minutes, the cell pressure will reach the required value and become stable. When this is achieved the user should press the "Done" button to complete the operation.

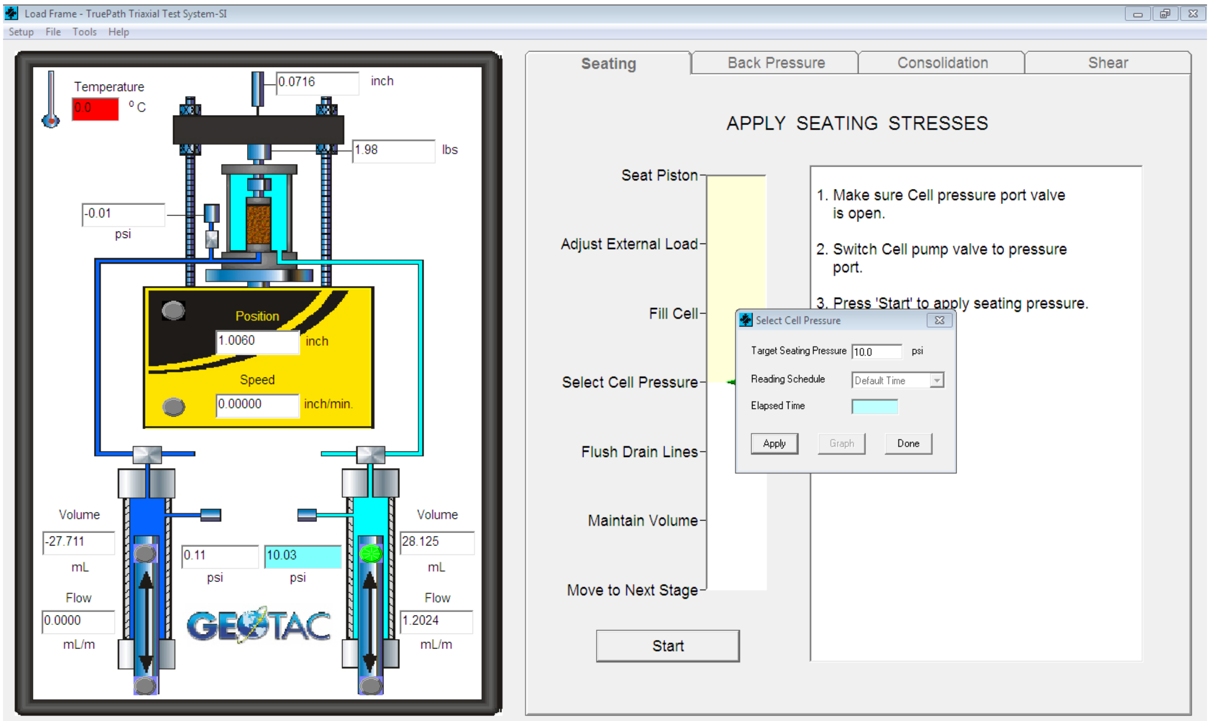


Figure 4.13: Application of initial confining pressure

4.4.5 Flushing the Drains

This step as seen in Figure 4.15 is intended to force water to flow through the top or the bottom drain lines using the pore/back pump in order to expel air bubbles from these drain lines. First the back/pore pump pressure line should be connected to the T fitting as shown in Figure 4.14. Then, overflow tubes should be attached to valves #3 and #4. Water should flow from the back/pore pump into the T fitting through valve#1 and into the container through valve#4 to flush the drains that pass through the upper cap, while water that passes from the back/pore pump to valve#2 passing through valve#3 flushes the drains in the bottom cap. In order to initiate this process, the “Start” button is pressed. Successively, to dislodge completely the air bubbles from the drain lines, the flow can be stopped and restarted simultaneously. After pressing the “Stop” button, valves#1 and #4 are closed and the bottom drain inlet valve #2 and bottom drain vent valve#3 are opened and the same procedure is repeated.

This technique is repeated until no more air bubbles are expelled through the drain lines. It is advised to refill the back/pore pump before completing the flushing step by switching the back/pore pump to the refill container by pressing on “Tools” from the main menu, pressing “Manual Mode”, and selecting “Pore pump”. The “down” arrow is then clicked so that the back/pore pump piston will move downward while water from the container will be drawn into the pump. The pore pump valve should then be returned to the pressure line, and flushing is continued if needed. Finally, the flushing stage should be terminated by closing valves#1 though #4, pressing the “Done” button and dismantling the overflow tubes from valves #3 and #4.

Figure 4.14: Flushing of the drains

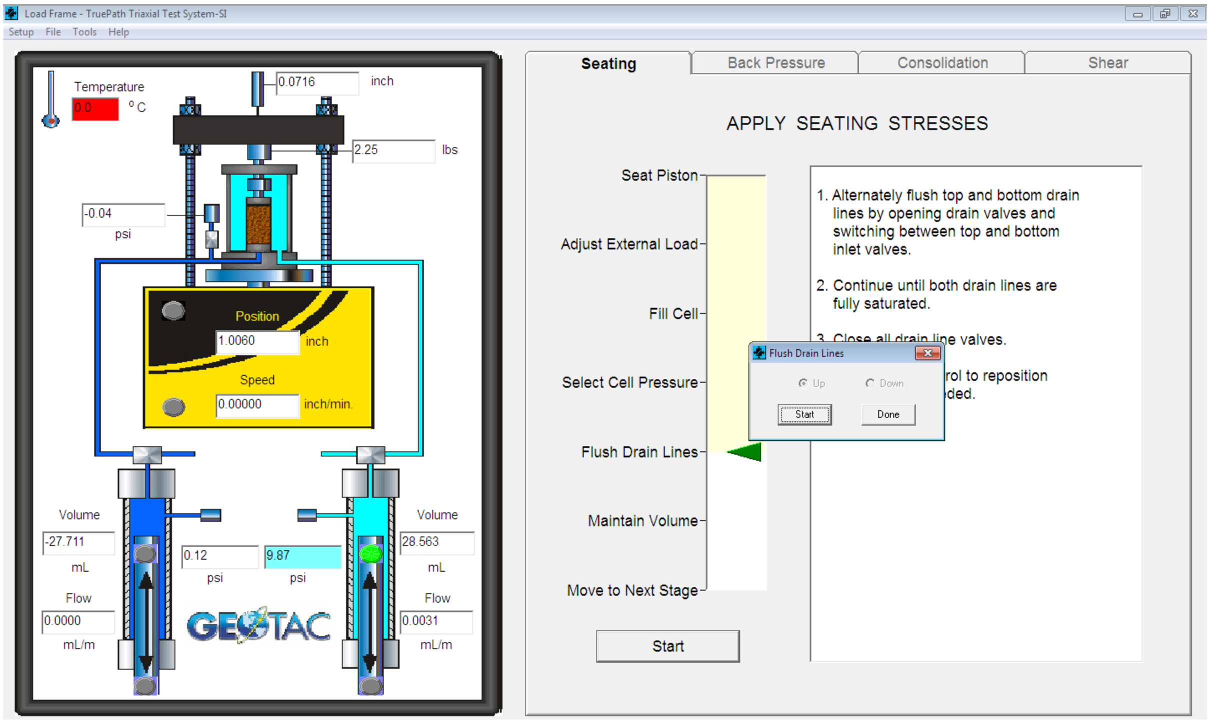


Figure 4.15: Flushing of the drains

4.4.6 Maintain the Volume

The final step in the Seating phase is to maintain the volume and let the sample rest after the flushing of the drains phase is ended. The sample in this stage will be able to relax or consolidate after the seating pressure has been applied earlier, since in this step the drainage valves #1 and #2 will be open. First, the “Maintain Volume” tab should be pressed as shown in Figure 4.16. Next, the “Start” button is pressed and inlet drain valves #1 and #2 are opened. The required confining cell pressure of 10psi, that is the same pressure chosen in the Select Pressure Phase, is then typed in the appropriate space and the “Start” button is pressed. A graph can be displayed to show the variation of the confining pressure with time. Furthermore, a curve showing the volume of water that is drained from the specimen as a function of time can also be displayed on the screen. When water stops draining out from the sample under the specified confining pressure, the maintain volume stage can be terminated. This is done by clicking on the “Stop” button and then on the “Done” button to end the maintain volume stage.

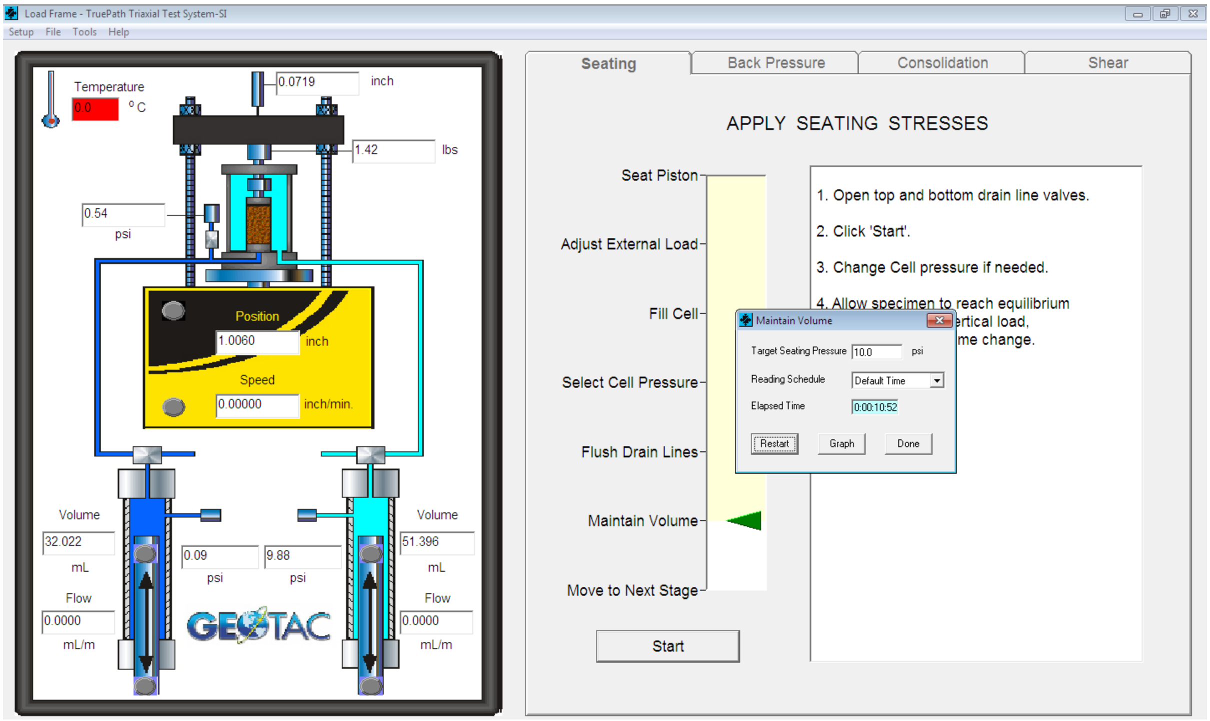


Figure 4.16: Maintain volume

4.5 Back Pressure Saturation Stage

To ensure full saturation of the slurry-consolidated Achrafieh specimen, a back pressure/saturation pressure of 45 psi is applied to the specimen using the back/pore pump. The following steps were followed in the back pressure saturation stage:

- Installing the pore pressure transducer in the inlet of valve #3, then opening drain vent valve#3.
- Check that inlet drain valves #1 and 2 are opened and make sure that the port valve of the bottom pump is opened, while drain valve #4 is closed.
- Input the value of the required saturation pressure of 45psi, effective stress of 10psi, check the tab for Ramp at Specified rate, input the Pressure Rate (dP/dt) of 3psi/minute, and initiate saturation by clicking on the "Start" button as shown in Figure 4.17.
- The value of the back pressure can be checked at the bottom pressure transducer that is displayed on the left side of the screen. Usually the specimen is left for overnight for a period of at least eight to ten hours to saturate.
- To check if the saturation level requested is reached, press on "Pause", and then click on "Check B" to check the B value of the sample. After clicking on "Check B", a pop-up window will appear showing the different steps to be followed as shown in Figure 4.18. First close valves#1 and #2, enter a small increment of cell pressure of 5psi and press "Start". The cell pump will instantaneously increase the cell pressure by 5 Psi, and the pore water pressure should indicate a similar increase of pore water pressure if the sample is completely saturated. The software calculates the B-value and reports its value every 15 seconds on the

screen. During this check, a minimum B-value of 0.95 was generally obtained for tests conducted in this study.

❖ After an acceptable B-value is ensured, click on "Done" and wait till the window for the B value check disappears by itself. When this happens, re-open drain inlet valves#1 and 2, and press on "Done" to end the back pressure saturation stage. If saturation was not achieved using the initial specified back pressure of value 45psi, increase the saturation pressure by a certain increment and repeat the same saturation process.

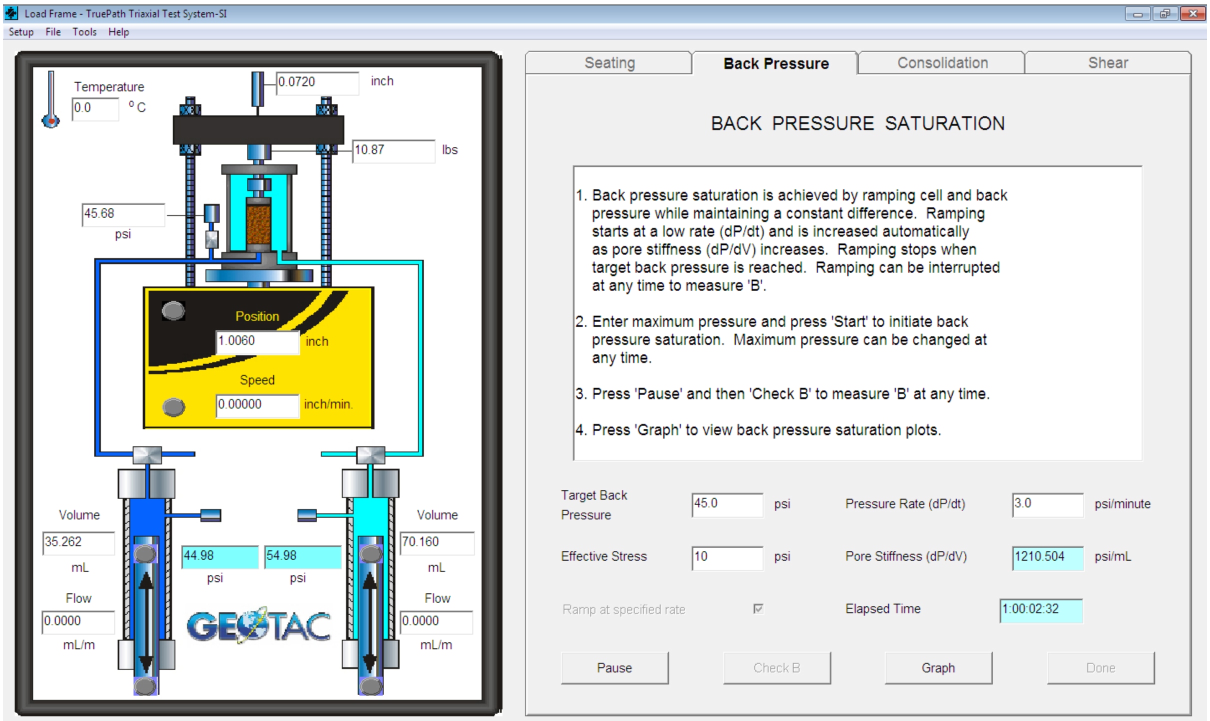


Figure 4.17: Back pressure saturation stage

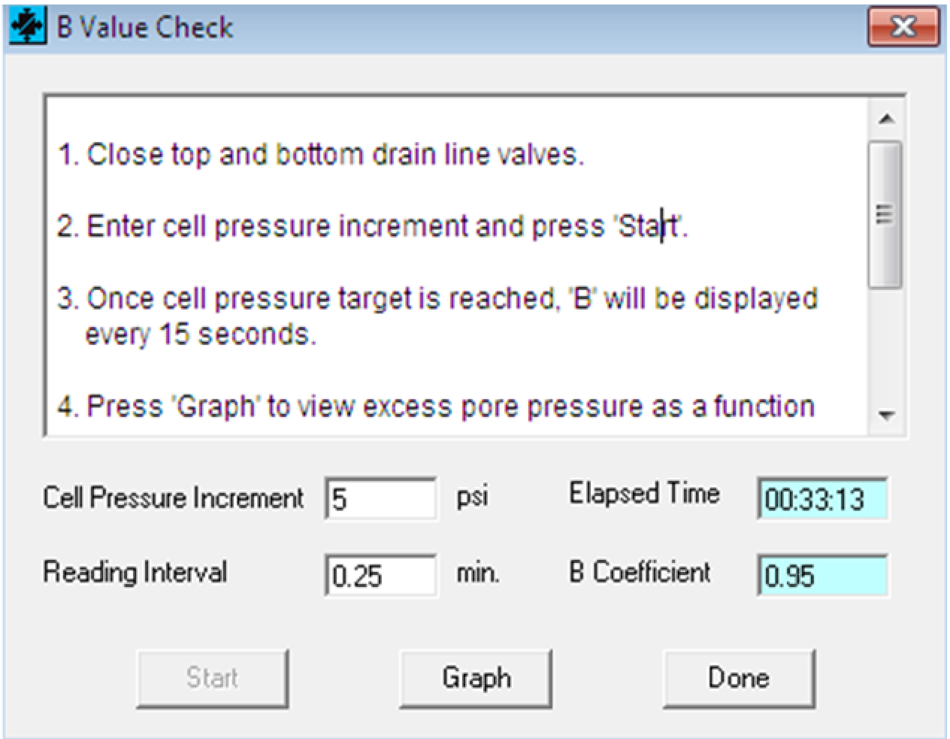


Figure 4.18: Checking B-value

4.6 Isotropic Consolidation Stage

The consolidation stage is initiated by clicking on the "Consolidation" tab. First, the relevant data which includes the effective confining pressures and the stress rate that have been previously entered during the creation of the data test file should be checked. The activated window for isotropic consolidation is shown in Figure 4.19. In this stage, the user can still change the target effective stress and the vertical stress rate, but cannot change the type of consolidation. Once all the input data is verified and consolidation is initiated, consolidation continues until the reading of the pore water volume intake for the pore pump becomes a constant. At this time, the isotropic consolidation stage can be assumed to be completed. An adequate period of at least ten hours is usually needed to consolidate the Achrafieh specimens at confining pressures, however the consolidation phase is not stopped until the change in pore volume stabilizes as can be seen in Figure 4.20.

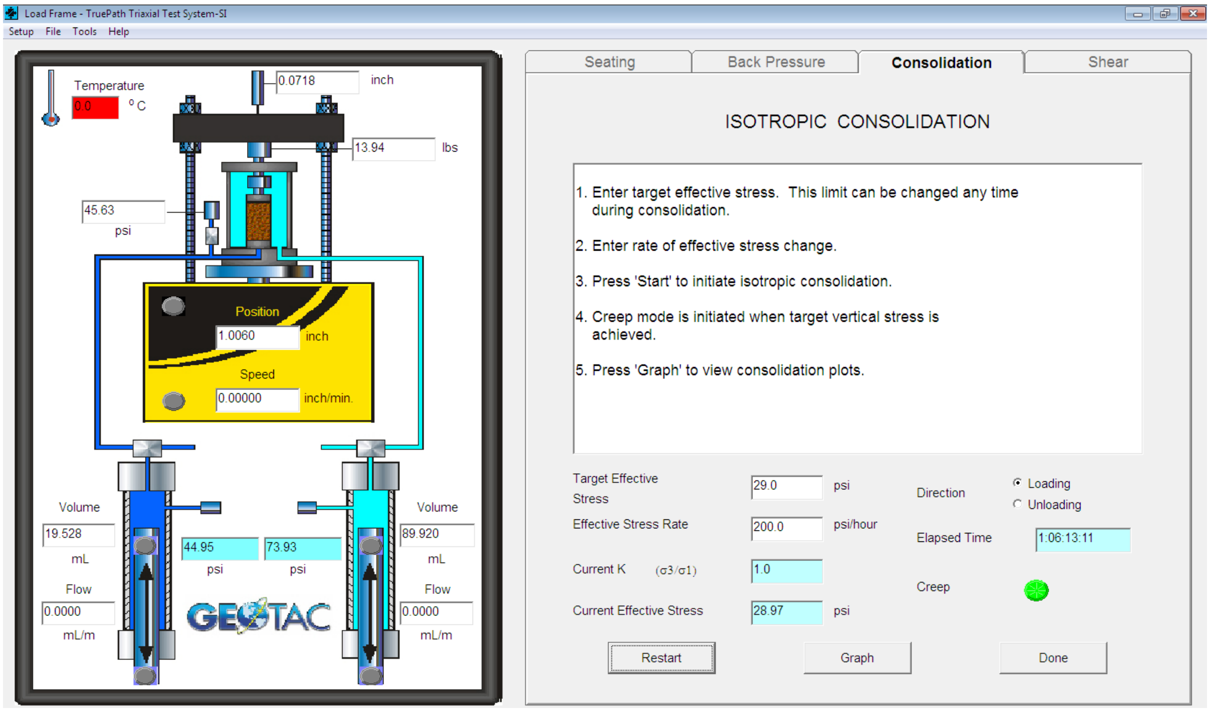


Figure 4.19: Window for isotropic consolidation

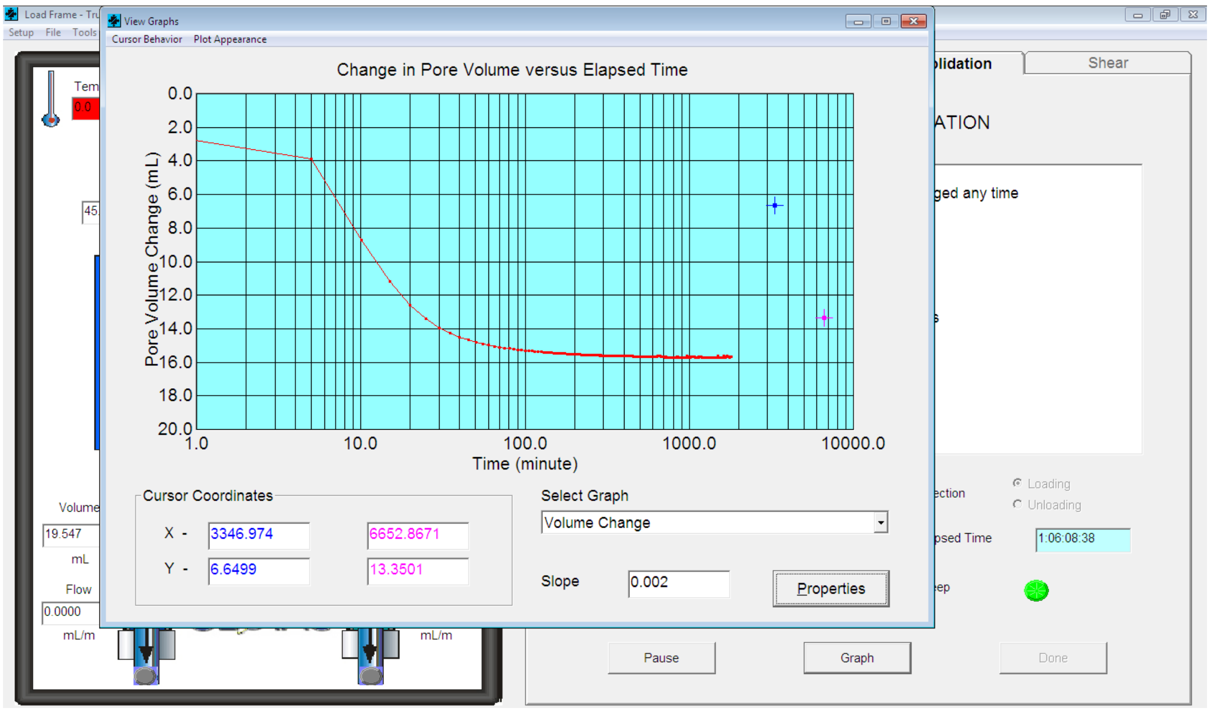


Figure 4.20: Change of volume vs. log Time

4.7 Drained (CD) / Partially Drained (PD) / Undrained (CU) Shearing Stage

At the end of the isotropic consolidation stage, a gap will form between the top cap of the specimen and the bottom of the loading piston. The user has to use the manual controls to close the gap and reestablish contact before starting the shearing stage. Once the window for the "Drained shear" or "Undrained Shear" is activated, the user can change the strain rate. In this research, a value of 0.375%/hour is used for the strain rate for drained shear tests, a value between 1%/hour and 60%/hour for partially drained tests, and a value of 1%/hour for undrained tests.

Once the strain rate is chosen, cell valves#1 and 2 that are connected to the pore pump could have 3 different settings depending on the drainage conditions. If the tests are fully drained, then cell valve #1 and #2 are opened, while if the tests are partially drained, then only cell valve#1 is opened and cell valve#2 is closed. Finally, if the tests are undrained, then both cell valves# 1 and #2 must be closed. In any of the above mentioned drainage conditions, valve#3 between the pore pressure sensor and the pore pump should be checked to be open. The "Start" button is then clicked as shown in Figure 4.21 to initiate shearing. Different curves can be viewed while the test is in progress. For undrained tests, shear stress and pore-water pressure versus axial strain curves can be viewed, whereas for drained and partially drained tests, shear stress and volumetric strain versus axial strain can be viewed as seen in Figure 4.22 and Figure 4.23 below. When the strain reaches a percentage of 25%, the test can be terminated and the software can be exited.

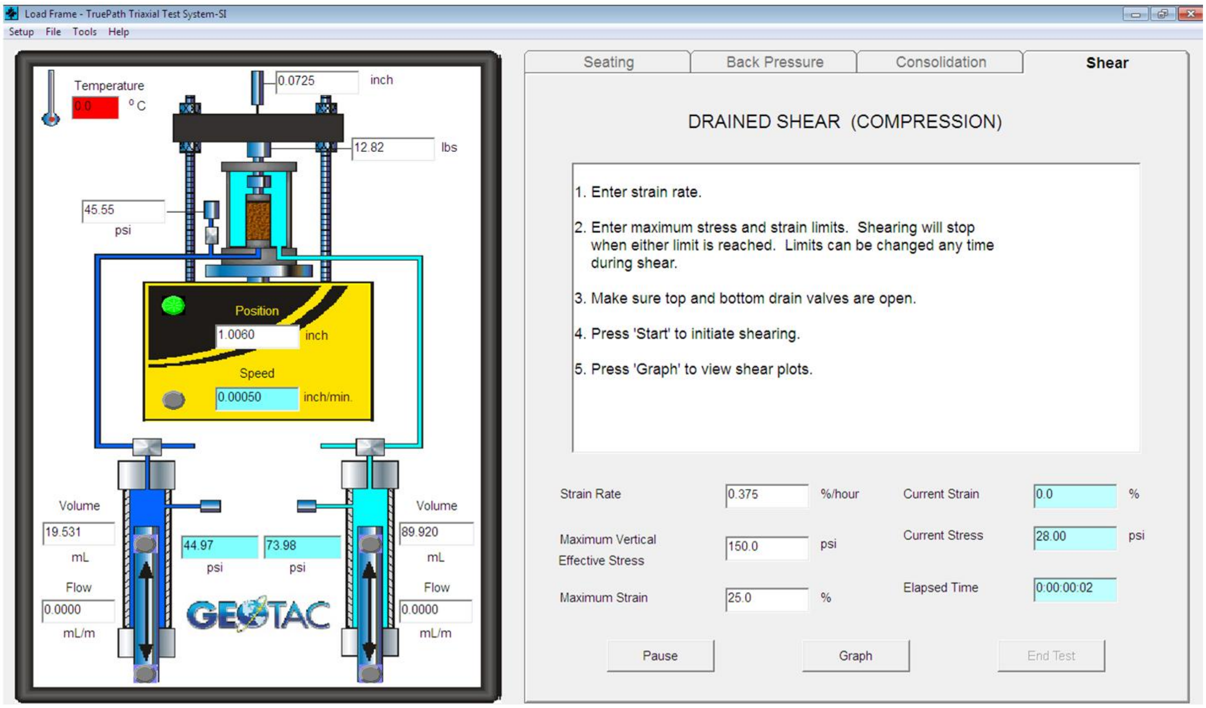


Figure 4.21: Window for drained shear test

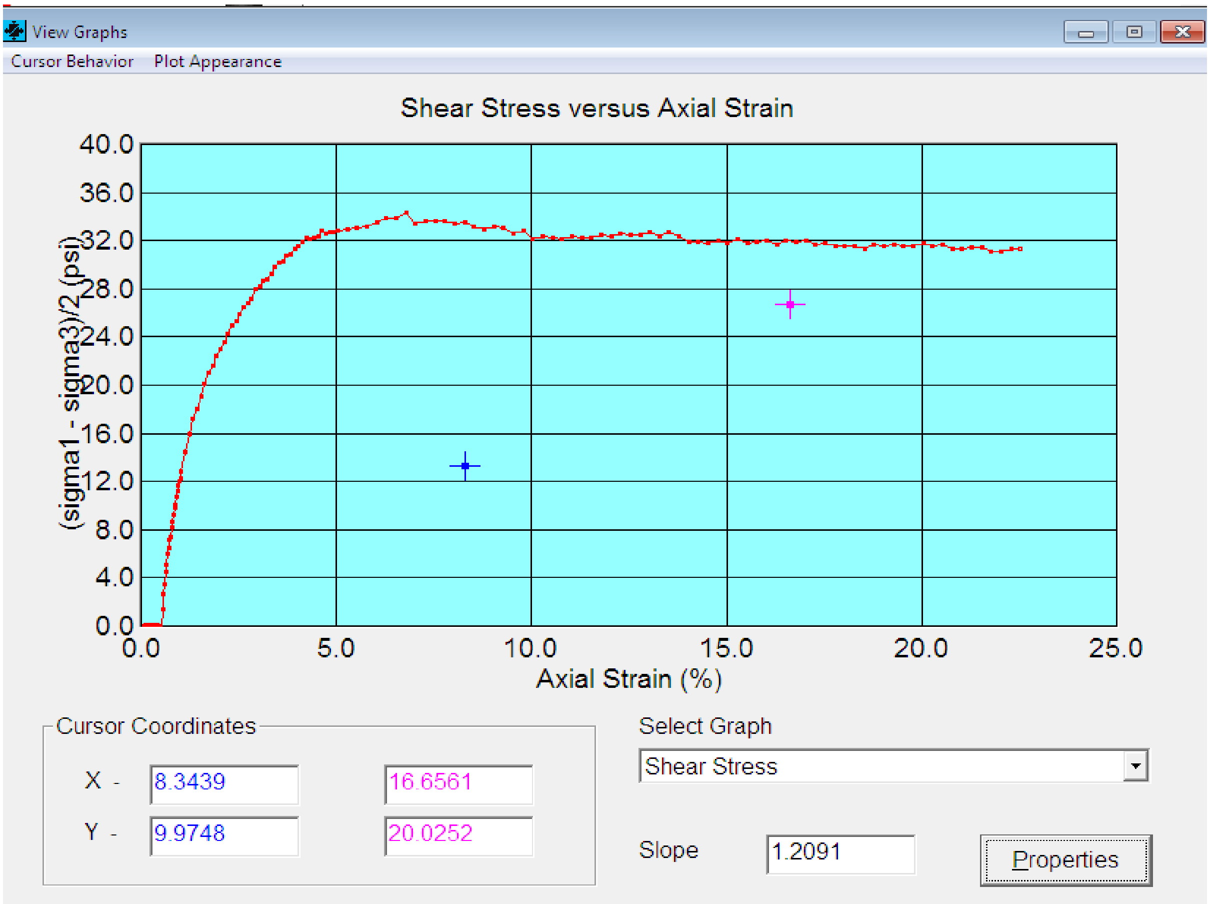


Figure 4.22: Stress strain curves during the test

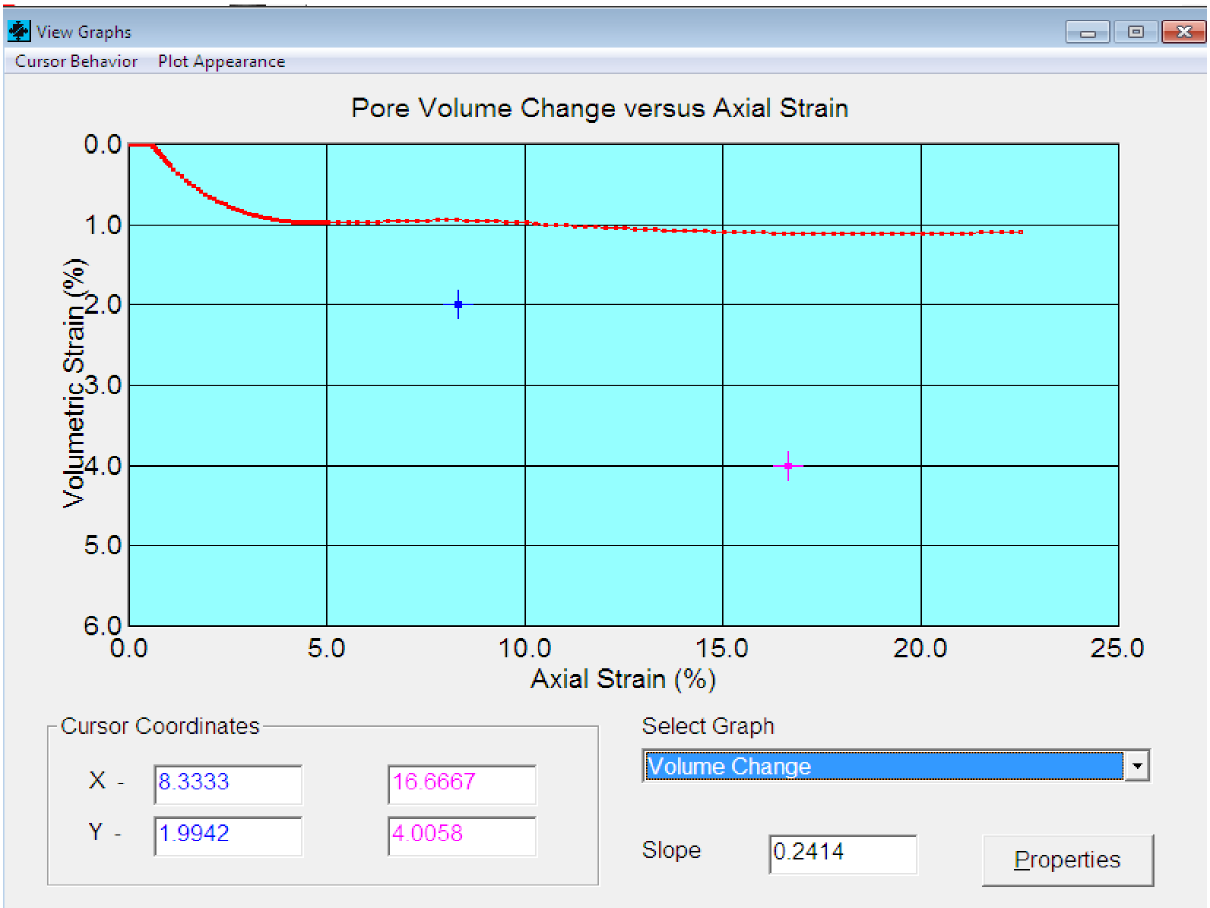


Figure 4.23: Volumetric strain vs. axial strain curve during the test

4.8 Test Tear Down

Test tear down process involves removing pressures from the specimen, the triaxial chamber and the load frame. This can be accomplished through following these steps:

- ❖ Enter the True Path software and lock the cell piston.
- ❖ Select the manual controls, and choose cell pump. After that, choose “Pressure control” as shown in Figure 4.24 and record a value of zero psi for the cell pressure and for the pore pressure, then press start for at each Cell Pressure and Pore Pressure in a fast way. Water will drain out from the cell chamber into the

cell pump to reduce the cell pressure to zero, and from the sample to the pore pump.

- Use the manual control to lower the loading frame platen.
- Connect the top air vent valve and remove the hose from the bottom cell connect and replace it with a tube that discharges water into a container.
- After the water is drained out from the cell, remove the triaxial chamber from the loading frame, and dismantle the cell parts, wash them, and prepare them for another test. Caution is requested while removing the sample from the rubber membrane so as not to disturb. After removing the membrane, photos for the sample can be taken showing the mode of failure.

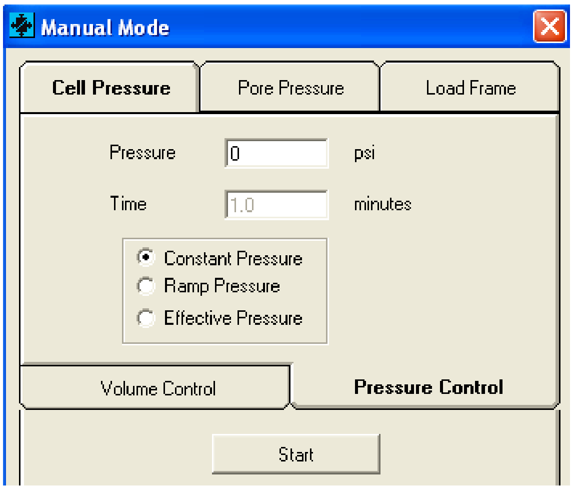


Figure 4.24: Window for unloading stage

4.9 Summary

A comprehensive description for operating the automated triaxial equipment "TruePath" was presented in this chapter in a simple way which includes a step by step procedure with figures and charts that facilitate the understanding of the testing process. The information presented in this chapter will make it easier for any future user to work and operate the "TruePath" equipment especially for samples sheared isotropically under drained, undrained and partially drained conditions. However, reading the manual of the "TruePath" system is crucial and vital in order to complete all the required information that the user should know prior to operating the system.

CHAPTER 5

TEST RESULTS AND ANALYSIS FOR DRAINED TESTS

5.1 Introduction

The automated triaxial test setup "TruePath" by Geotac was used to conduct the consolidated drained CD tests on both control and reinforced clay specimens saturated with a back pressure of 310 kPa. The samples were then isotropically consolidated under confining pressures of 100, 150 and 200 kPa and sheared drained at a strain rate of 0.375%/hr, while measuring the volume change through the drain lines connected to the porous stones at the top and at the bottom of the sample. The measured volume change reflects a global change in the composite sample and do not provide information on local changes in the water content of the sand column and the surrounding clay.

Throughout the tests, the total confining pressure was kept constant as the vertical stress was increased in compression. The Achrafieh clay specimens prepared had a length of 14.2cm and a diameter of 7.1cm, and were reinforced with 3cm and 4cm dense sand columns with relative density of 86.3%, resembling an area replacement ratio of 17.8% and 31.7% respectively.

The test results of consolidated drained tests conducted on 9 Achrafieh clay specimens are presented in this chapter. Composite samples include specimens reinforced with fully penetrating ordinary 3cm and 4cm dense sand columns. The results also include a description of the modes of failure that characterize the behavior of the different test specimens and a detailed analysis of the parameters which are known to affect the load response of clay specimens that are reinforced with sand columns. The effect of these parameters which include the area replacement ratio and confining pressure on the drained shear strength, stiffness, volume change, and effective shear strength parameters of the Achrafieh clay specimens is investigated and highlighted in this chapter.

5.2 Test Results

The test results are presented in the form of deviatoric stress and volumetric strain versus axial strain curves. For the drained tests, results were analyzed at different axial strains including 5%, 10%, 15% and 20%. Therefore, the improvements in the deviatoric stresses σ_d, volumetric strains, secant modulus, and shear strength parameters were assessed at the abovementioned axial strains.

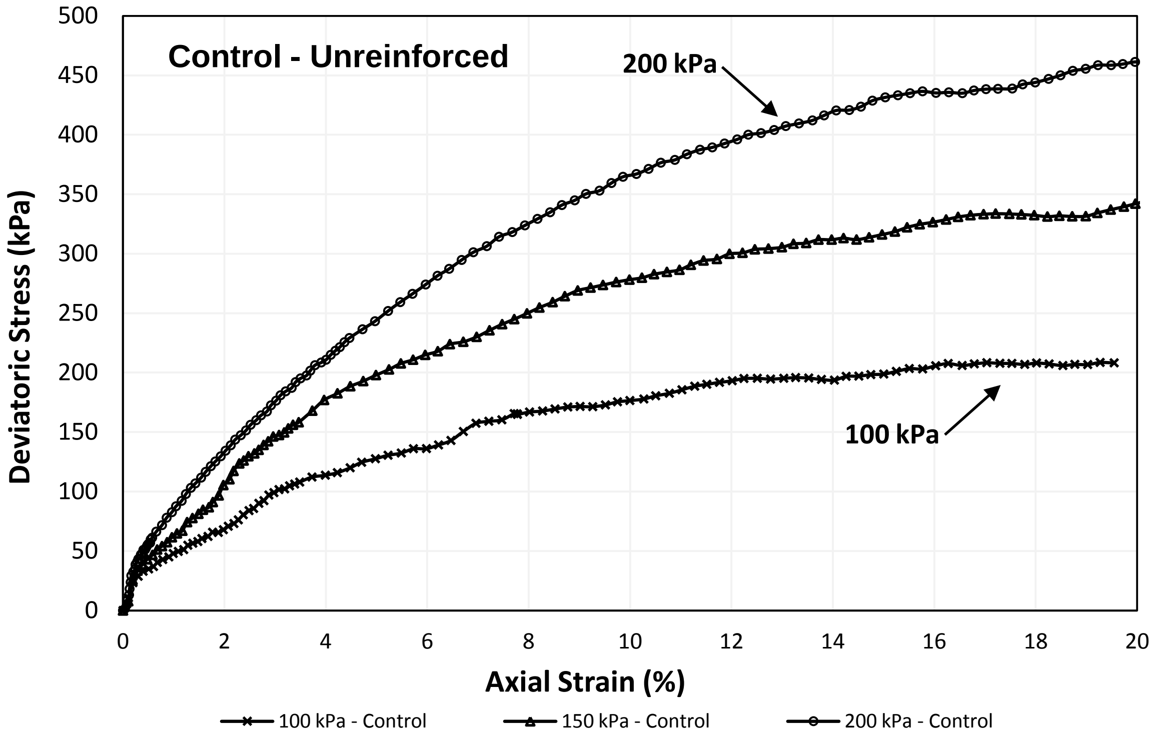
5.2.1 Unreinforced / Control Achrafieh Clay Specimens

Three consolidated drained tests were performed on control, unreinforced Achrafieh clay specimens with lengths of 14.2cm and diameters of 7.1cm, at confining pressures of 100, 150 and 200 kPa. The test results are represented in curves showing the variation of the deviatoric stress and the volumetric strains versus axial strain in Figure 5.1.

For all the confining pressures, the deviatoric stresses increased with axial strains up to the maximum strain of 20% at which the tests were terminated. An investigation of the stress-strain curves and the associated volumetric strain indicate that the tests approached critical state conditions at an axial strain of 20%. At this axial strain, compressive volumetric strains in the order of 4.3%, 5.4%, and 6.5% were observed for the tests conducted at confining pressures of 100, 150, and 200 kPa, respectively.

The secant modulus of elasticity E_{sec} for the control Achrafieh clay specimens at axial strains of 1% and 2% were determined at the different confining pressures of 100, 150, and 200 kPa. The secant modulus of elasticity increased with increasing confining pressure and decreased as the axial strain increased from 1% to 2%. Values of $(E_{sec})_{1\%}$ were calculated to be equal to 4808.91, 6267.17 and 8479.56kPa for confining pressures of 100, 150 and 200kPa respectively. Whereas values for $(E_{sec})_{2\%}$ were calculated to be 3425.94, 5314.02 and 6662.39kPa for confining pressures of 100, 150 and 200kPa respectively.

The Mohr Coulomb effective stress failure envelopes for the control specimen at axial strains of 5%, 10%, 15% and 20% are presented in Figure 5.2. The effective cohesion (c') and the effective angle of internal friction (Ø') for the control specimen at the different axial strains are indicated in Figure 5.2. Results indicate that the normally consolidated Achrafieh clay has an effective cohesion c' of zero and an effective friction angle that increases from 23.2 degrees if failure is defined at an axial strain of 5% to 31.2 degrees if failure is defined at an axial strain of 20%.



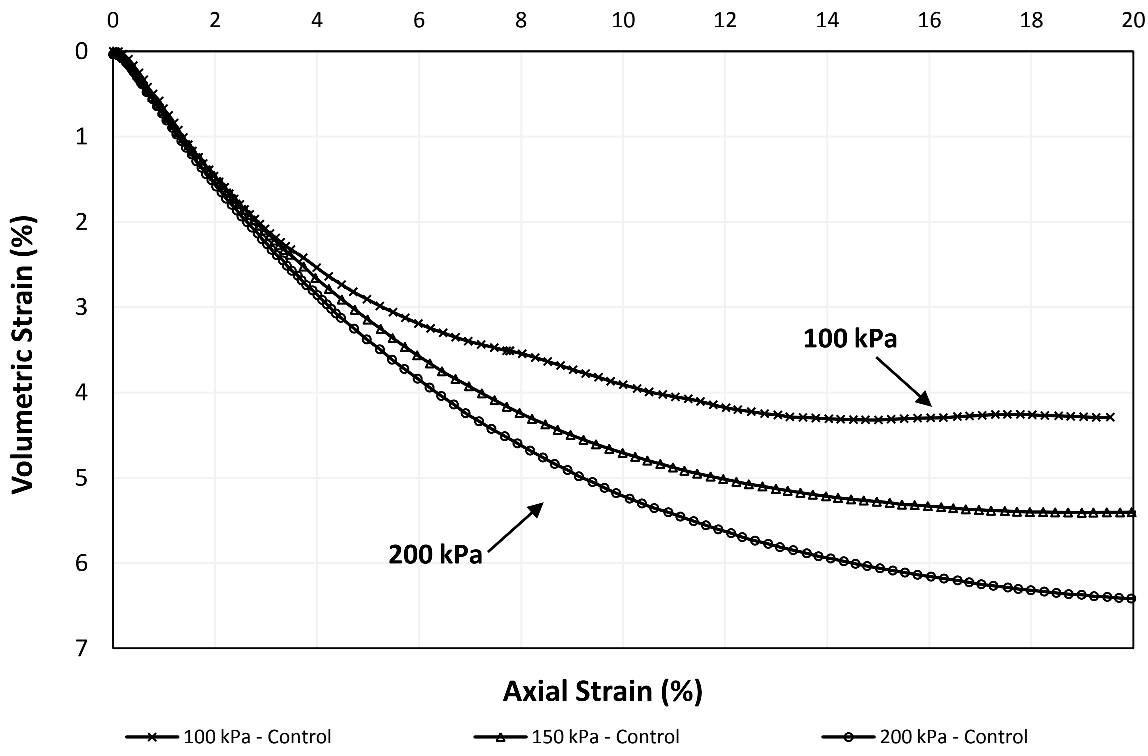


Figure 5.1: Deviatoric stress and volumetric strains versus axial strain for unreinforced/control Achrafieh clay specimens at confining pressures of 100 kPa, 150 kPa, and 200kPa

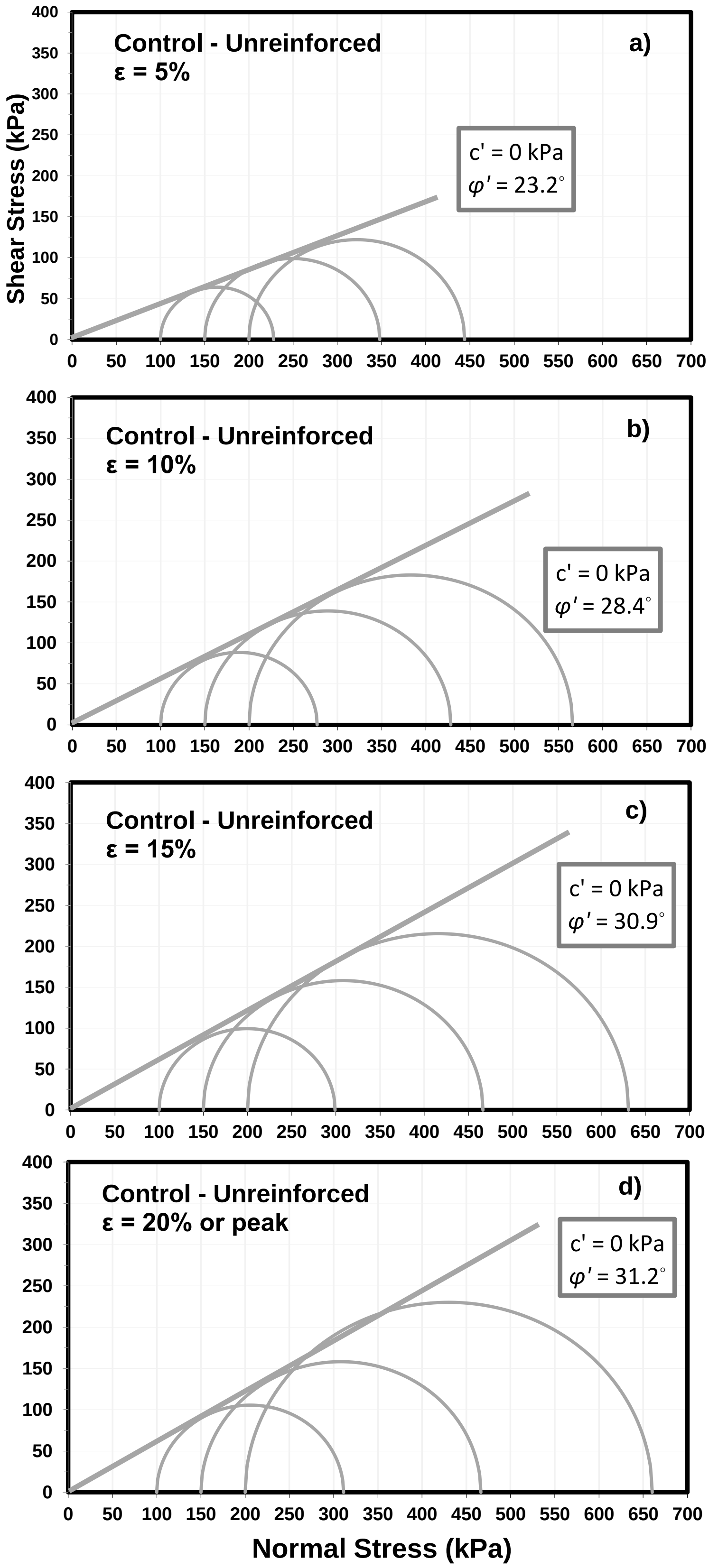


Figure 5.2: Mohr Coulomb effective stress failure envelop for control/unreinforced Achrafieh clay specimens at axial strains of 5, 10, 15 and 20%

5.2.2 Control Ottawa Sand Specimens

Three consolidated drained tests were performed on control, dense Ottawa sand specimens with lengths of 14.2cm and diameters of 7.1cm, at confining pressures of 100, 150 and 200 kPa. The samples had a relative density, D_r, of 86.3%. The test results are presented using curves showing the variation of the deviatoric stress and volumetric strain versus axial strain.

Results on Figure 5.3 indicate that Ottawa sand at a relative density of 86.3% exhibits a strain softening stress-strain relationship for the range of confining pressures used. The peak in the stress-strain relationship is exhibited at axial strains ranging from 4% to 5%. After the peak, the stress-strain curves soften as the sand dilates and approaches critical state conditions. At the maximums axial strain of 20% imposed in these tests, the sand does not reach critical state conditions as indicate by the volumetric strains which were still increasing at 20% strain for all confining pressures. The volumetric strains versus axial strain relationships are typical of the behavior of dense sand specimens that are sheared under drained conditions.

The Mohr Coulomb effective stress failure envelope for the control specimens is shown on Figure 5.4. The effective cohesion (c') and the peak effective friction (Ø') for the control specimen were found to be 0 kPa and 38.5º respectively.

5.2.3 Achrafieh Clay Specimens Reinforced with Sand Columns

Results obtained from the consolidated drained triaxial tests conducted on Achrafieh clay specimens that were reinforced with fully penetrating sand columns with different area replacement ratios are presented in Table 5.1, Table 5.2 and Table 5.3 and in Figure 5.4 to Figure 5.8. The figures also include pictures of the modes of failure and graphs showing the variation of the deviatoric stress and reduction in volumetric strains versus axial strain. The results were analyzed to investigate the effect of relevant parameters such as the area replacement ratio A_c/A_s and confining pressure on the improvement in the drained shear strength and the apparent effective strength parameters of the clay. It should be noted that in all the discussion presented below, it was assumed that the sand column and the surrounding clay act as a single element with homogeneous distributions of stresses and strains.

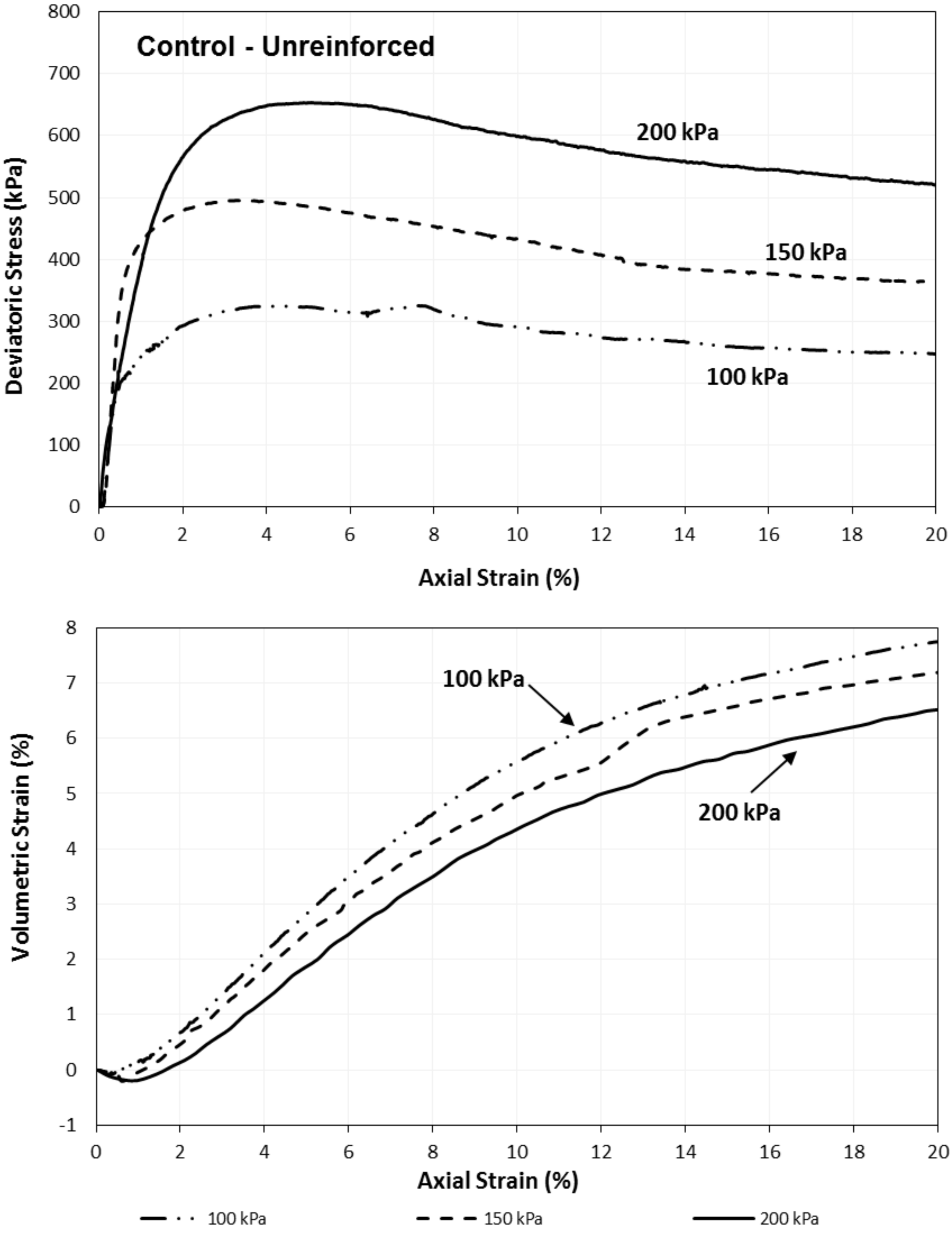


Figure 5.3: Deviatoric stress and volumetric strains versus axial strains for specimens composed of Sand at confining pressures of 100, 150 & 200kPa

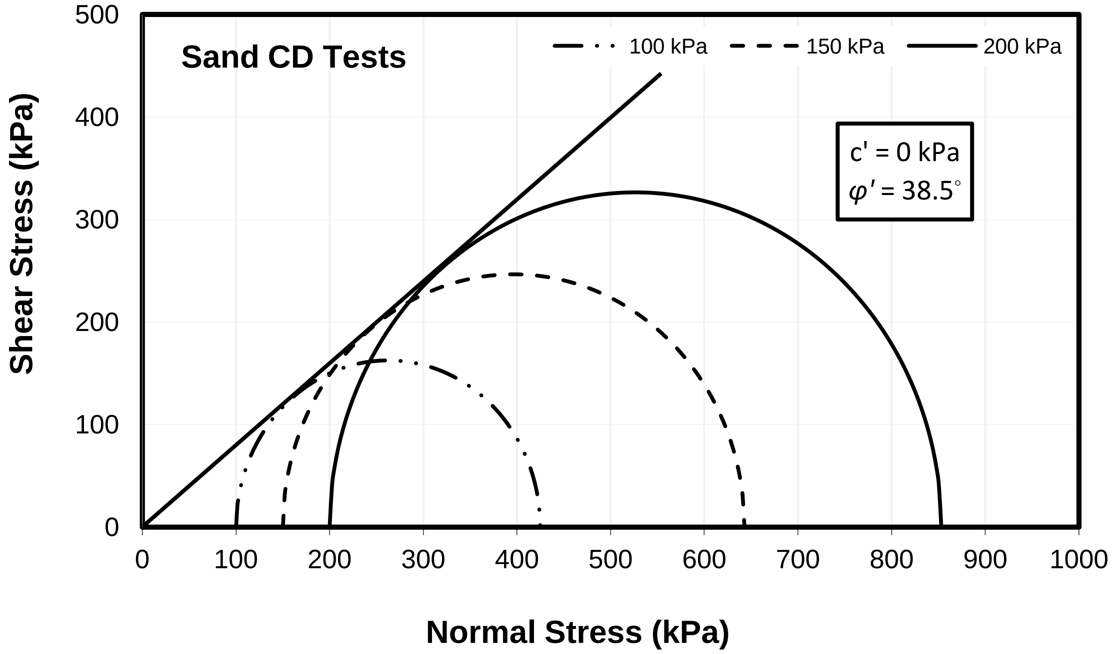


Figure 5.4: Deviatoric stress and volumetric strains versus axial strain for unreinforced/control Achrafieh clay specimens at confining pressures of 100 kPa, 150 kPa, and 200kPa

5.2.3.1 *Mode of Failure*

For consolidated drained control/unreinforced samples, failure was characterized mainly by uniform bulging of the clay specimen along its length with the bulging generally concentrated at the middle portions of the sample (Figure 5.5a). Specimens that were reinforced with sand columns at area replacement ratios of 17.8% (Figure 5.5b) and 31.2% (Figure 5.5c) indicated a similar mode of failure as the control specimens, except that the degree of bulging in the middle of the sample was amplified. To investigate the internal mode of failure of the sand columns, the same test specimens were split along their vertical axes to expose the columns and the surrounding clay. The sections shown in Figure 5.5b and Figure 5.5c indicate that the sand columns exhibited bulging that was in line with the external bulging witnessed in the test specimens. This indicates that the significant bulging that was observed for the reinforced specimens could be directly attributed to the dilation that is expected to occur in the sand columns during shearing. It is worth noting that that no shear planes were observed in the test specimens, even at axial strains in the order of 20%.

5.2.3.2 ***Stress-Strain Behavior***

The variations of the deviatoric stress and the volumetric strain versus axial strain for the composite specimens are presented in Figure 5.6 and Figure 5.7, for samples reinforced with an area replacement ratio of 17.8% and 31.7%, respectively at confining pressures of 100, 150, and 200 kPa. To allow for direct comparison between the response of the control specimens and that of the reinforced specimens, the stress strain curves of the control specimen, the specimen reinforced with an area ratio of 17.8%, and the specimen reinforced with an area ratio of 31.7% were plotted together on Figure 5.8 for each confining pressure, separately. Results on Figure 5.8 lead to the following observations:

1. A systematic change in the shape of the stress-strain relationship was observed in the 3 specimens, with the control specimen showing a typical strain hardening behavior, the reinforced specimen with the high area replacement ratio of 31.7% exhibiting a strain softening behavior, and the reinforced specimen with the intermediate area ratio of 17.8% showing an intermediate stress-strain response.
2. In the relatively low range of axial strains (generally less than 10%), reinforced specimens exhibited an improved stress-strain response compared to the control specimen, in the sense that the deviatoric stress for a given level of strain was

larger for reinforced specimens compared to the control specimen. However, this observation did not hold for the high strain range (10% to 20%), where the deviatoric stresses in the reinforced specimens were found to either decrease (due to strain softening) or stabilize as the axial strains increased, contrary to the deviatoric stresses in the control specimens which continued to increase steadily at high strains.

3. The inclusion of dense columns in the reinforced clay specimens resulted in a reduction in the magnitude of the compressive volumetric strains that were observed in the control clay specimens, with the reduction being significant for specimens reinforced at an area replacement ratio of 31.7% and marginal for specimens reinforced at an area ratio of 17.8%. The reduction in the magnitude of compressive volumetric strains is directly correlated to the dilative tendency of dense sand columns as they are being sheared under drained conditions. This dilative tendency is expected to be more effective as the area replacement ratio increases.

The above three observations are important since they indicate that any improvement in load carrying capacity of clay that is reinforced with dense sand columns is expected to be heavily dependent on the level of strain at which failure is defined. For example, a quick look at the stress-strain curves on Figure 5.8 shows that at an axial strain of 20% (which is a common failure criterion in triaxial tests), reinforcing normally consolidated Achrafieh clay with dense sand columns will not lead to any improvement in the deviatoric stresses at failure, even for a relatively large area replacement ratio of 31.7%. This counter-intuitive observation makes sense since the benefit of adding the sand columns becomes obsolete at large strains due to the tendency of the sand column to strain soften as it is sheared to failure. The real benefit of the column is however clear at relatively low strains (around 5%) where the effect of strain softening of the sand does not exist.

5.2.3.3 *Effect of Sand Columns on the Deviatoric Stress at Failure*

The percent improvement in the drained deviatoric stress at strains of 5%, 10%, 15% and 20%, and are presented in Table 5.2 and plotted versus the initial effective confining pressure in Figure 5.9. As expected, results indicate that the percent improvements in the deviatoric stresses of specimens reinforced with 3-cm diameter dense sand columns were lower than those of the 4-cm dense sand columns. Significant average improvements in the deviatoric stresses were noted at strains of 5% (about 34% for Ac/As = 17.8% and 69% for Ac/As = 31.7%), followed by a major drop in improvements at strains of 10% (about 6% for Ac/As = 17.8% and 21% for Ac/As = 31.7%), and minor to no improvements recorded at strains of 15% and 20%. In fact, at an axial strain of 20%, average reductions in the order of 7% were observed for specimens with Ac/As = 17.8% and almost no improvement was observed for Ac/As = 31.7%).

5.2.3.4 *Effect of Sand Columns on the Drained Secant Modulus*

Secant moduli $(E_{sec})_{1\%}$ and $(E_{sec})_{2\%}$ defined at axial strains of 1% and 2% were calculated for each test by dividing the deviatoric stress measured at axial strains of 1% and 2% respectively by the corresponding strain. The results of the calculated values of $(E_{sec})_{1\%}$ and $(E_{sec})_{2\%}$ are presented in Table 5.3 and plotted in Figure 5.10. As indicated by the test results of Table 5.3, the insertion of fully penetrating dense 3-

cm sand columns significantly increased the stiffness of the unreinforced Achrafieh clay at axial strains of 1% by an average of ≈111% for the three different confining pressures of 100, 150 and 200 kPa, and by an average of ≈83% at axial strains of 2%.

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σ'_3 = 150 kPa - Consolidated Drained Tests

Control - Unreinforced $\quad$ A_c/A_s = 17.8% $\quad$ A_c/A_s = 31.%

Figure 5.5: Photographs showing mode of failure of samples sheared under drained conditions

I.D.	σ_3 (kPa)	Diam. of Sand (cm)	A_c/A_s (%)	Height of Specimen (cm)	Diam. of Specimen (cm)	Deviat. Stress @ 5% (kPa)	Deviat. Stress @ 10% (kPa)	Deviat. Stress @ 15% (kPa)	Deviat. Stress @ 20% (kPa)	Volum. Strain @ 5%	Volum. Strain @ 10%	Volum. Strain @ 15%	Volum. Strain @ 20%
1	100	0	0.00	14.20	7.10	128.00	177.00	199.00	211.26	2.91	3.91	4.32	4.32
2		3	17.85	14.20	7.10	176.00	197.50	205.10	209.70	2.29	3.18	3.71	4.01
3		4	31.74	14.20	7.10	207.70	212.14	212.10	212.14	1.11	1.20	1.31	1.38
4	150	0	0.00	14.20	7.10	198.00	278.20	316.30	316.50	3.15	4.71	5.28	5.41
5		3	17.85	14.20	7.10	254.20	285.60	296.50	307.80	2.41	3.55	4.28	4.68
6		4	31.74	14.20	7.10	324.60	326.70	326.70	326.70	1.28	1.35	1.52	1.60
7	200	0	0.00	14.20	7.10	244.00	366.00	431.00	460.25	3.39	5.21	6.06	6.41
8		3	17.85	14.20	7.10	332.00	379.80	394.20	379.80	2.76	4.12	4.95	5.46
9		4	31.74	14.20	7.10	440.00	457.20	457.20	457.00	1.41	1.45	1.59	1.60

Table 5.1: Results of deviatoric stresses and volumetric strains for Consolidated Drained triaxial tests

I.D.	σ_3 (kPa)	Diam. of Sand Column (cm)	A_c/A_s (%)	Height of Specimen (cm)	Diam. of Specimen (cm)	Increase Deviat. Stress @ 5% (%)	Increase Deviat. Stress @ 10% (%)	Increase Deviat. Stress @ 15% (%)	Increase Deviat. Stress @ 20% (%)	Reduction in Volum. Strain @ 5% (%)	Reduction in Volum. Strain @ 10% (%)	Reduction in Volum. Strain @ 15% (%)	Reduction in Volum. Strain @ 20% (%)
1	100	0	0.00	14.20	7.10	-	-	-	-	-	-	-	-
2		3	17.85	14.20	7.10	37.50	11.58	3.07	-0.74	21.20	18.68	14.12	7.15
3		4	31.74	14.20	7.10	62.27	19.85	6.58	0.42	61.96	69.27	69.73	67.96
4	150	0	0.00	14.20	7.10	-	-	-	-	-	-	-	-
5		3	17.85	14.20	7.10	28.38	2.66	-6.26	-2.75	23.34	24.69	19.01	13.46
6		4	31.74	14.20	7.10	63.94	17.43	3.29	3.22	59.32	71.36	71.29	70.46
7	200	0	0.00	14.20	7.10	-	-	-	-	-	-	-	-
8		3	17.85	14.20	7.10	36.07	3.77	-8.54	-17.48	18.47	20.99	18.25	14.76
9		4	31.74	14.20	7.10	80.33	24.92	6.08	-0.71	58.29	72.18	73.83	75.01

Table 5.2: Improvements in deviatoric stresses and reduction in volumetric strains for Consolidated Drained triaxial tests

I.D.	Confining Pressure σ_3 (kPa)	Diameter of Sand Column (cm)	A_c/A_s (%)	Density of Sand Column	Height of Specimen (cm)	Diameter of Specimen (cm)	E_{sec} @ 1% Axial Strain (kPa)	E_{sec} @ 2% Axial Strain (kPa)	Increase in $(E_{sec})_{1\%}$	Increase in $(E_{sec})_{2\%}$
1	100	0	0.00	-	14.20	7.10	4808.91	3425.94	-	-
2		3	17.85	Dense	14.20	7.10	10081.53	6579.23	109.64	92.04
3		4	31.74	Dense	14.20	7.10	13206.68	8444.22	174.63	146.48
4	150	0	0.00	-	14.20	7.10	6267.17	5314.02	-	-
5		3	17.85	Dense	14.20	7.10	13816.99	9383.34	120.47	76.58
6		4	31.74	Dense	14.20	7.10	20262.62	12886.77	223.31	142.51
7	200	0	0.00	-	14.20	7.10	8479.56	6662.39	-	-
8		3	17.85	Dense	14.20	7.10	17182.08	11882.85	102.63	78.36
9		4	31.74	Dense	14.20	7.10	26023.65	17528.96	206.90	163.10

Table 5.3: Results and Improvements in (E_{sec}) for Consolidated Drained triaxial tests

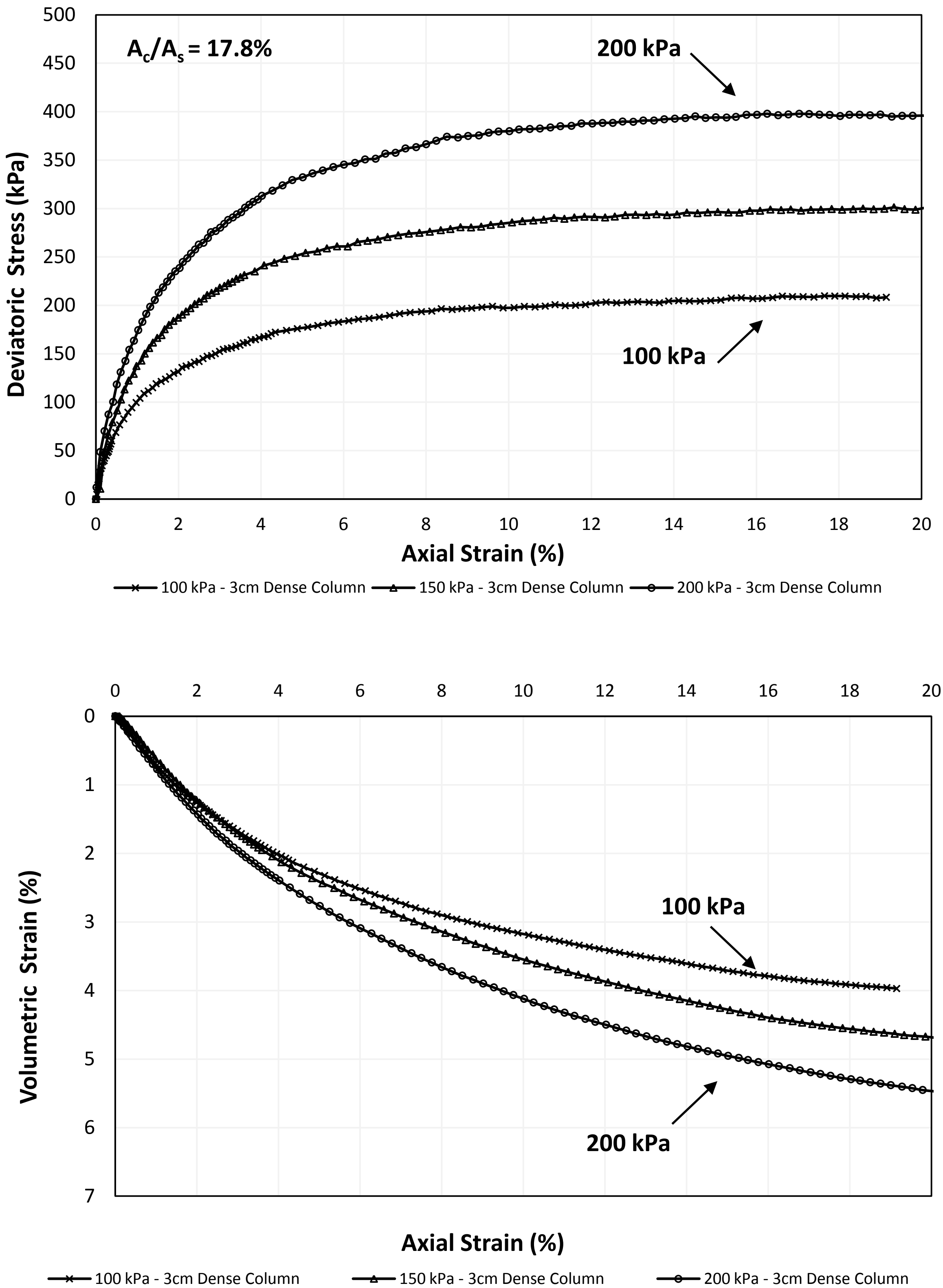


Figure 5.6: Deviatoric stress and reduction in volumetric strains versus axial strain for specimens reinforced with 3cm dense sand columns at confining pressures of 100 kPa, 150 kPa, and 200kPa

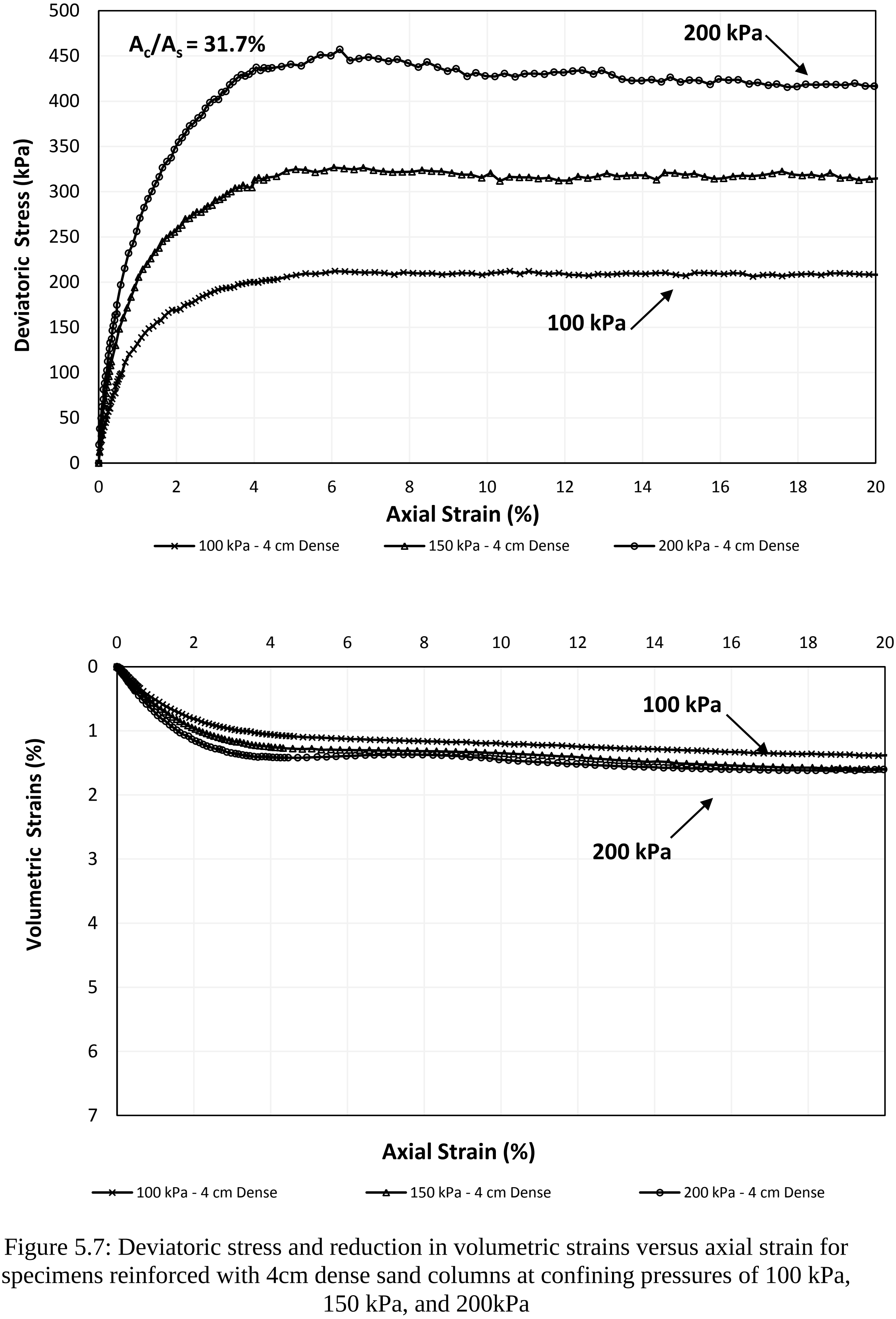


Figure 5.7: Deviatoric stress and reduction in volumetric strains versus axial strain for specimens reinforced with 4cm dense sand columns at confining pressures of 100 kPa, 150 kPa, and 200kPa

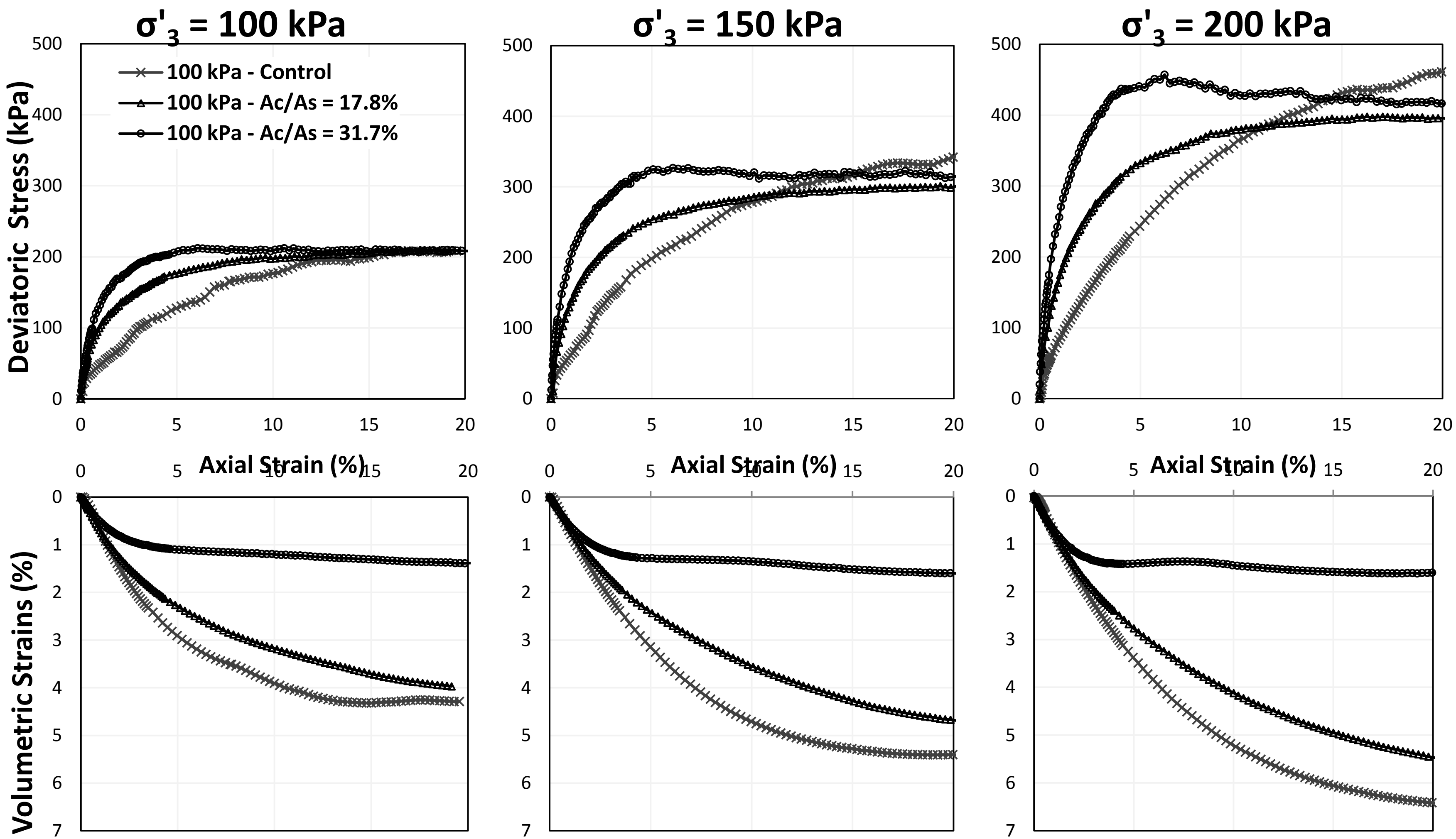
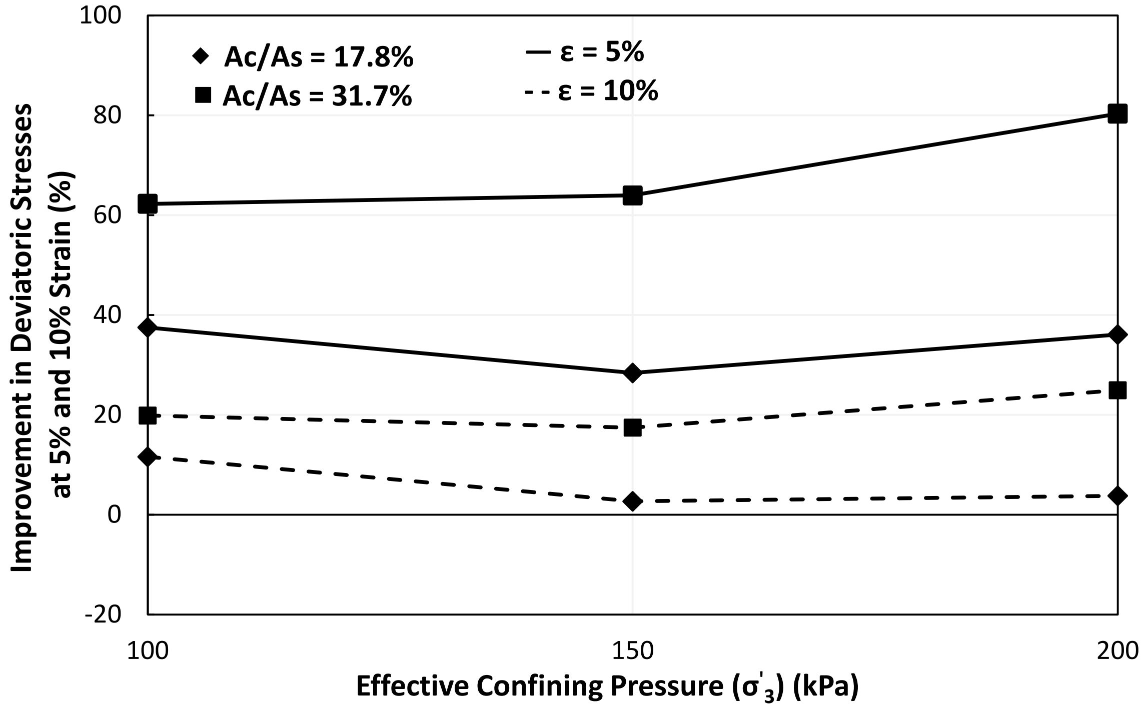


Figure 5.8: Deviatoric stresses & volumetric strains vs. axial strain for reinforced specimens at confining pressures of 100, 150 and 200 kPa (As/Ac=17.8%,As/Ac=31.7%)

As for the specimens reinforced with 4-cm dense sand columns, considerable increases in $(E_{sec})_{1\%}$ and $(E_{sec})_{2\%}$ were noticed with improvements averaging 202% for $(E_{sec})_{1\%}$ and 151% for $(E_{sec})_{2\%}$ for the three different confining pressures of 100kPa, 150kPa and 200kPa respectively. This indicates that for specimens reinforced with 4cm dense sand columns, the columns are successful at attracting significant loads for a given settlement compared to the control specimen and the specimen reinforced at a relatively lower area replacement ratio of 17.8%. The percent improvement in the drained secant modulus at failure for the series of tests are presented in Figure 5.12 and plotted versus the initial effective confining pressures.

The dependency of the drained secant modulus on the strain level was investigated by plotting the variation of Esec with the axial strain for both control and reinforced specimens having area replacement ratios of 17.8% and 31.7% at different effective confining pressures of 100, 150, and 200 kPa as shown in Figure 5.11. The results indicated that the secant modulus for the reinforced and the control specimens decreases as the axial strain increases, reflecting the nonlinearity in the stress-strain response. The specimens exhibited a sharp drop in the secant stiffness for strains that are less than 1% to 2%. After a strain of 2%, the stiffness constantly decreases with axial strain.



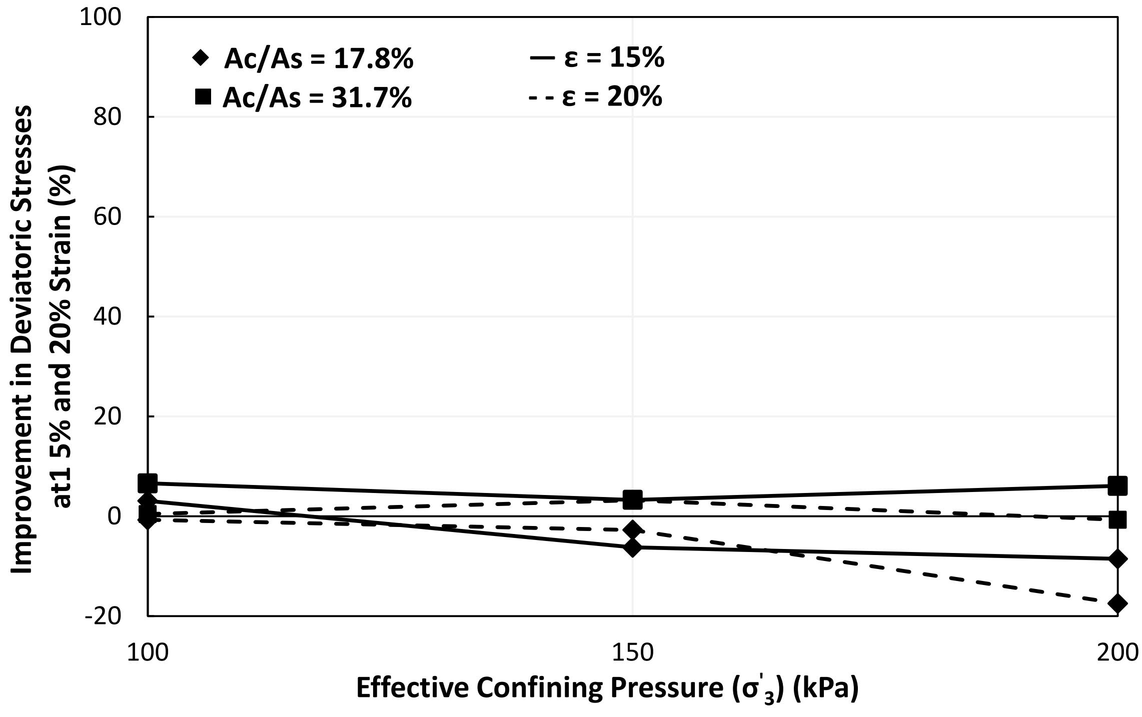
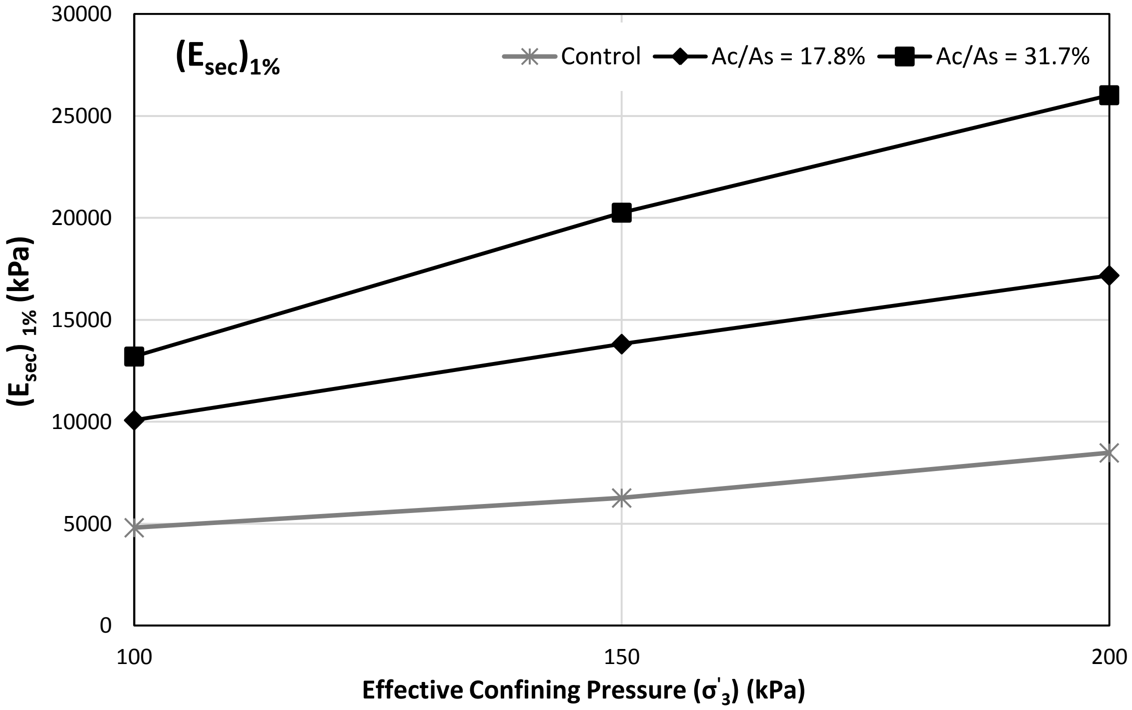


Figure 5.9: Variation of improvement deviatoric stresses with confining pressure (Hc/Hs= 1, As/Ac=17.8%,As/Ac=31.7%, ordinary)



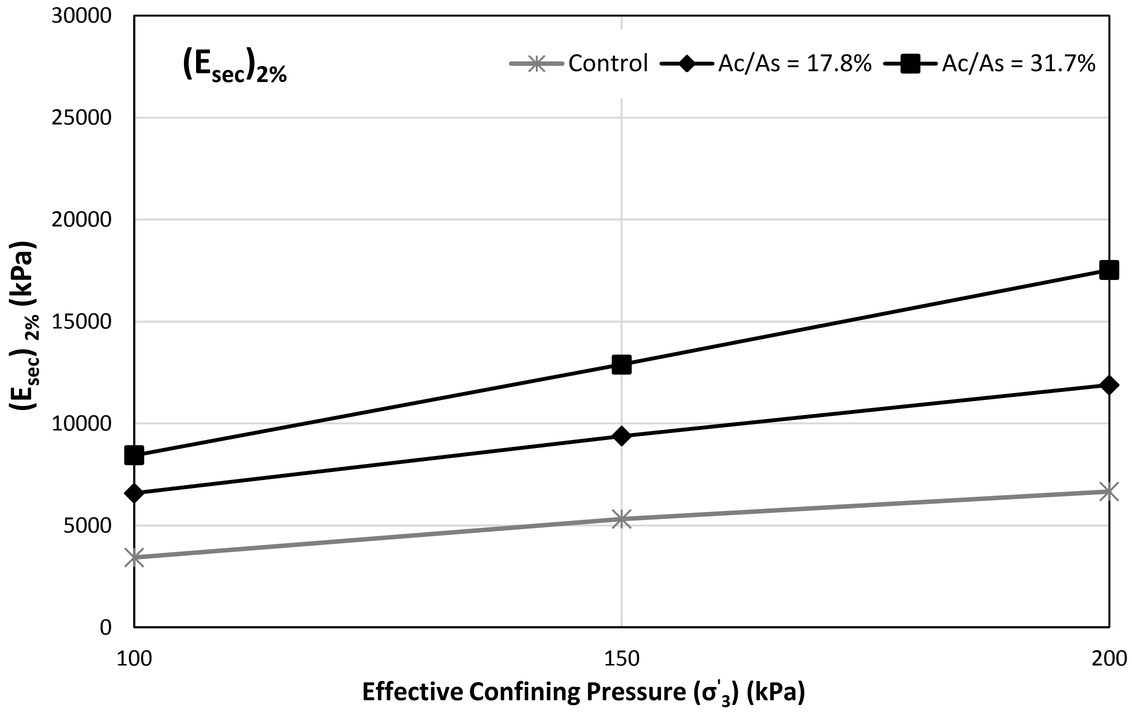
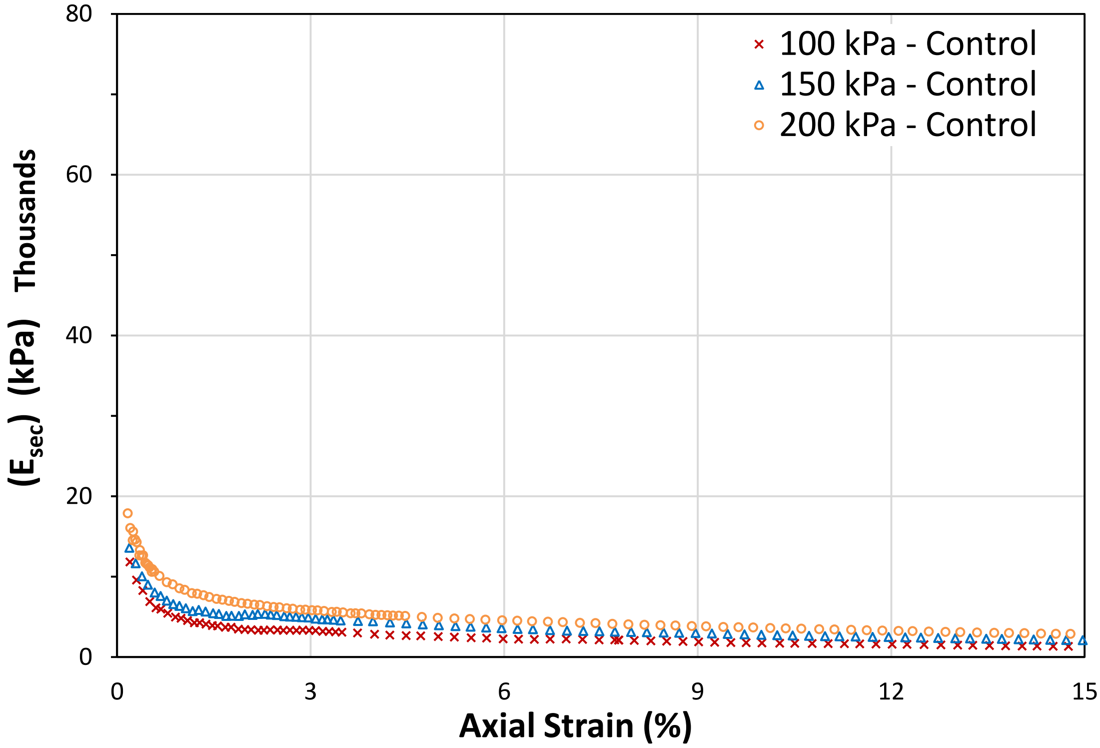
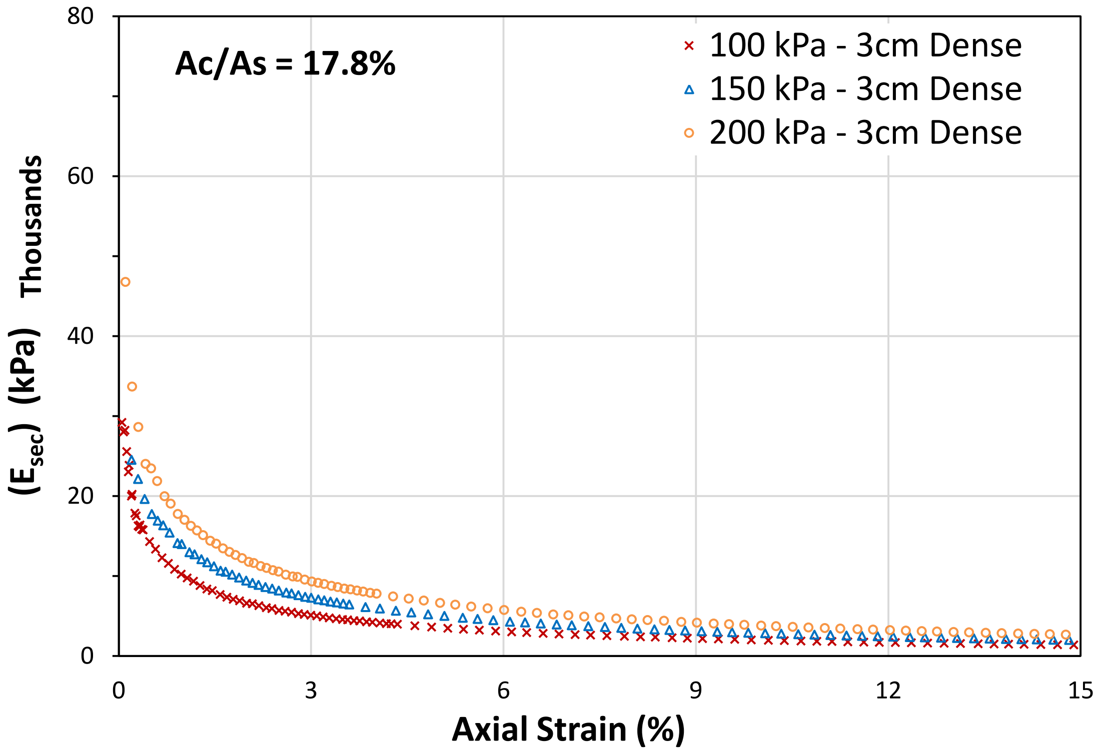


Figure 5.10: Variation of (Esec)1% and (Esec)2% at confining pressures of 100, 150 and 200 kPa


80
60
40
20
0
(Esec) (kPa) Thousands
0
3
6
9
12
15
Axial Strain (%)
100 kPa - Control
150 kPa - Control
200 kPa - Control


Ac/As = 17.8%
80
60
40
20
0
(Esec) (kPa) Thousands
0
3
6
9
12
15
Axial Strain (%)
100 kPa - 3cm Dense
150 kPa - 3cm Dense
200 kPa - 3cm Dense

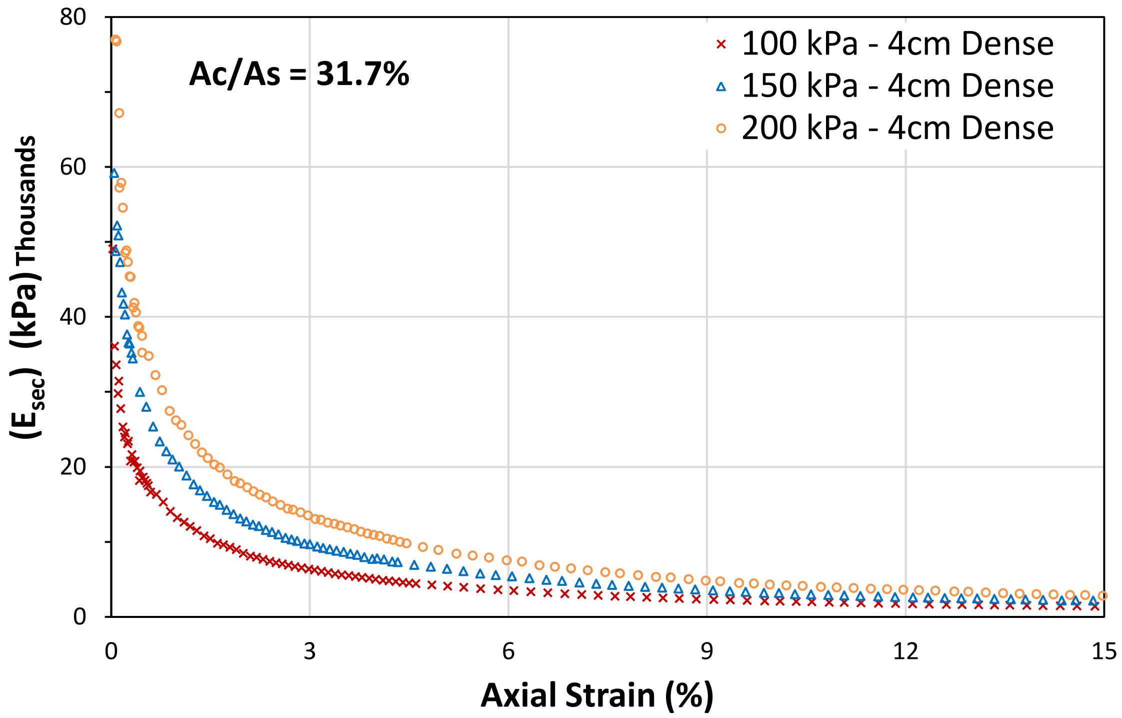


Figure 5.11: Variation of (Esec) with axial strain for control and composite specimens

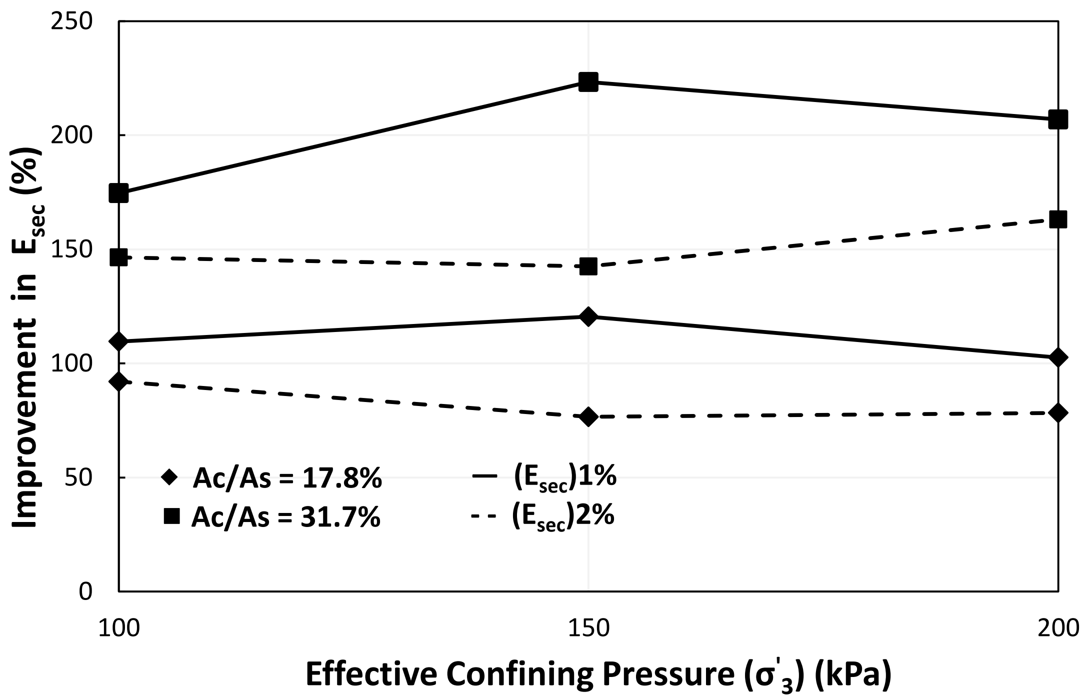


Figure 5.12: Improvements in (Esec)1% and (Esec)2% at confining pressures of 100, 150 & 200 kPa

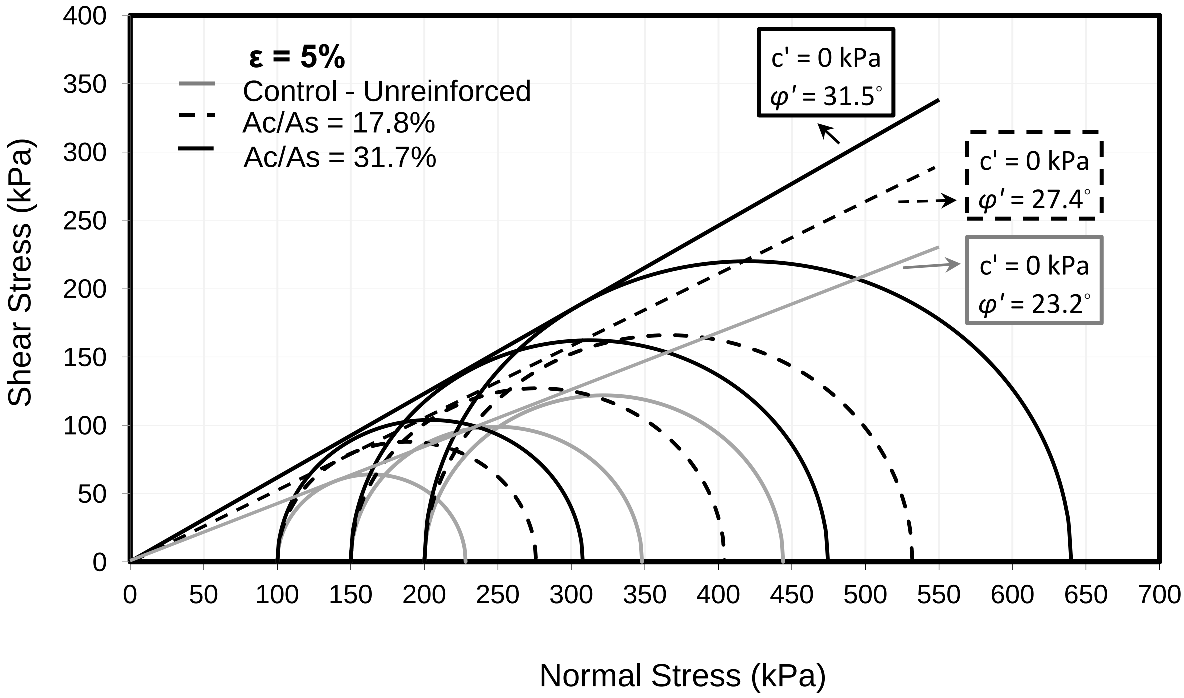
5.2.3.5 ***Effect of Sand Columns on the Effective Shear Strength Parameters***

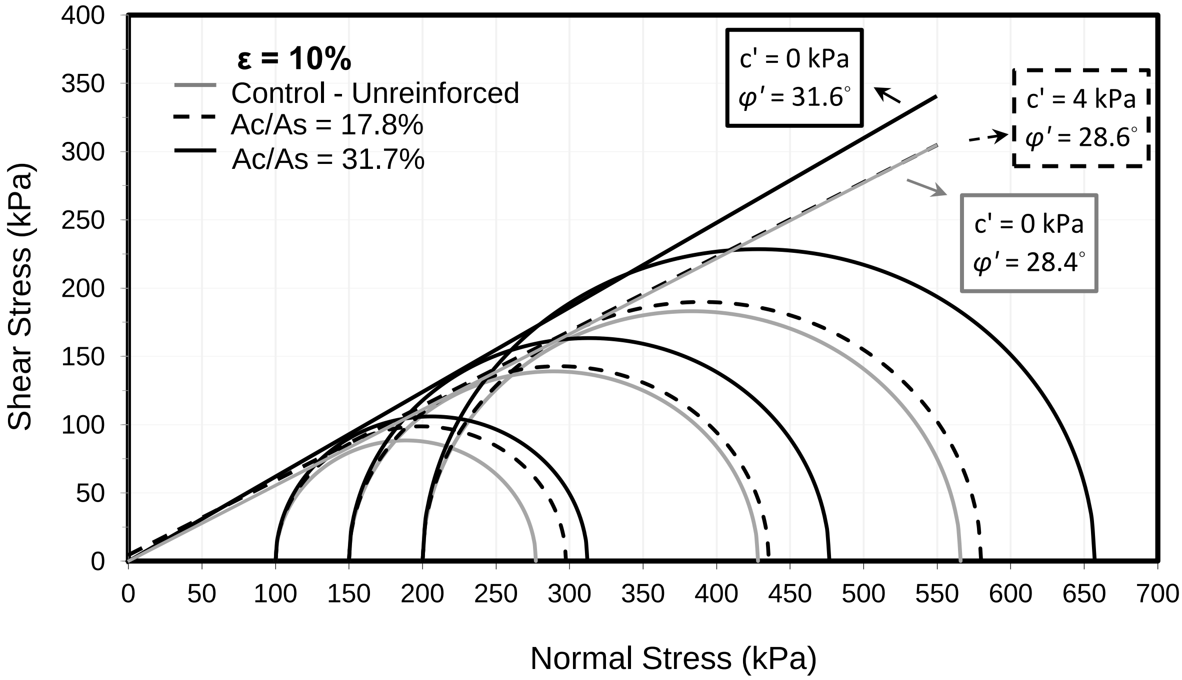
Figure 5.13 shows the effective Mohr-Coulomb envelops corresponding to the unreinforced control specimens, reinforced specimens with fully penetrating 3cm dense sand columns and reinforced specimens with fully penetrating 4cm dense sand columns. In addition, the resulting shear strength parameters c' and ϕ' for the different series of drained tests performed are summarized in Table 5.4. Since the control clay specimens exhibited a strain hardening behavior and for comparison purposes, failure was defined at axial strains of 5%, 10%, 15% and 20%. For each of these assumed failure strains, the deviatoric stresses at failure were taken as the deviatoric stresses corresponding to these strains unless a peak was observed in the stress strain curves before these strains. In these particular cases, the deviatoric stresses at failure were taken as the deviatoric stresses at the peak.

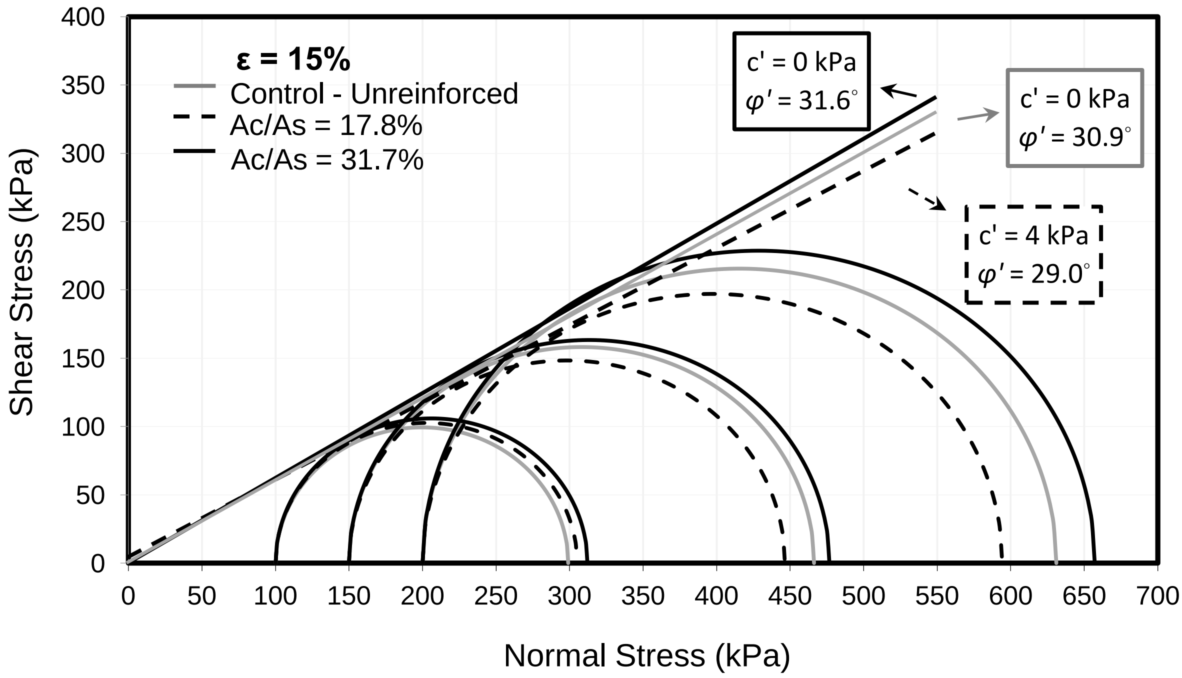
As indicated by the data shown in Table 5.4, the insertion of sand columns increases the drained angle of internal friction (ϕ') of the composite system with respective to the control tests performed on Achrafieh clay samples when failure is defined at the lower strain levels of 5% and 10%, with the increase being most noticeable for samples reinforced with 4-cm dense sand columns where there was an increase of up to 8.3° (at failure strain of 5%), compared to ≈4.2° (at failure strain of 5%) for specimens that were reinforced with 3-cm dense sand columns. For samples reinforced with 4-cm columns, the reported value of ϕ' was about 31.6° for all strain levels, since a peak was observed in these samples at strains lower than 5%. For samples reinforced with 3-cm columns, ϕ' was equal to 27.4° when failure was defined at 5% strain, and increased to 28.6° to 29° when the failure strain was increased above 10%. It should be noted that the control clay exhibited a significant sensitivity to the assumed failure strain level with ϕ' values increasing from 23.2° at an axial strain of 5% to 31.2° at an axial strain of 20%. The drained cohesion c' was found to be negligible or non-existent in both the control and the reinforced clay specimens.

Table 5.4: Effective shear stress failure parameters

Effective Shear Stress Parameters (c', ϕ') for CD tests at axial strains of 5, 10, 15 & 20%								
Area Replacment Ratio A_c/A_s (%)	**ε = 5%**		**ε = 10%**		**ε = 15%**		**ε = 20 % or Peaks**	
	c' (kPa)	**ϕ' (°)**	c' (kPa)	**ϕ'** (°)	c' (kPa)	**ϕ'** (°)	c' (kPa)	**ϕ'** (°)
Control	0	**23.2**	0	**28.4**	0	**30.9**	0	**31.2**
17.80%	0	**27.4**	4	**28.6**	4	**29.0**	4	**29.0**
31.70%	0	**31.5**	0	**31.6**	0	**31.6**	0	**31.6**




ε = 10%
Control - Unreinforced
Ac/As = 17.8%
Ac/As = 31.7%
c' = 0 kPa
φ' = 31.6°
c' = 4 kPa
φ' = 28.6°
c' = 0 kPa
φ' = 28.4°
Shear Stress (kPa)
Normal Stress (kPa)


ε = 15%
Control - Unreinforced
Ac/As = 17.8%
Ac/As = 31.7%
c' = 0 kPa
φ' = 31.6°
c' = 0 kPa
φ' = 30.9°
c' = 4 kPa
φ' = 29.0°
Shear Stress (kPa)
Normal Stress (kPa)

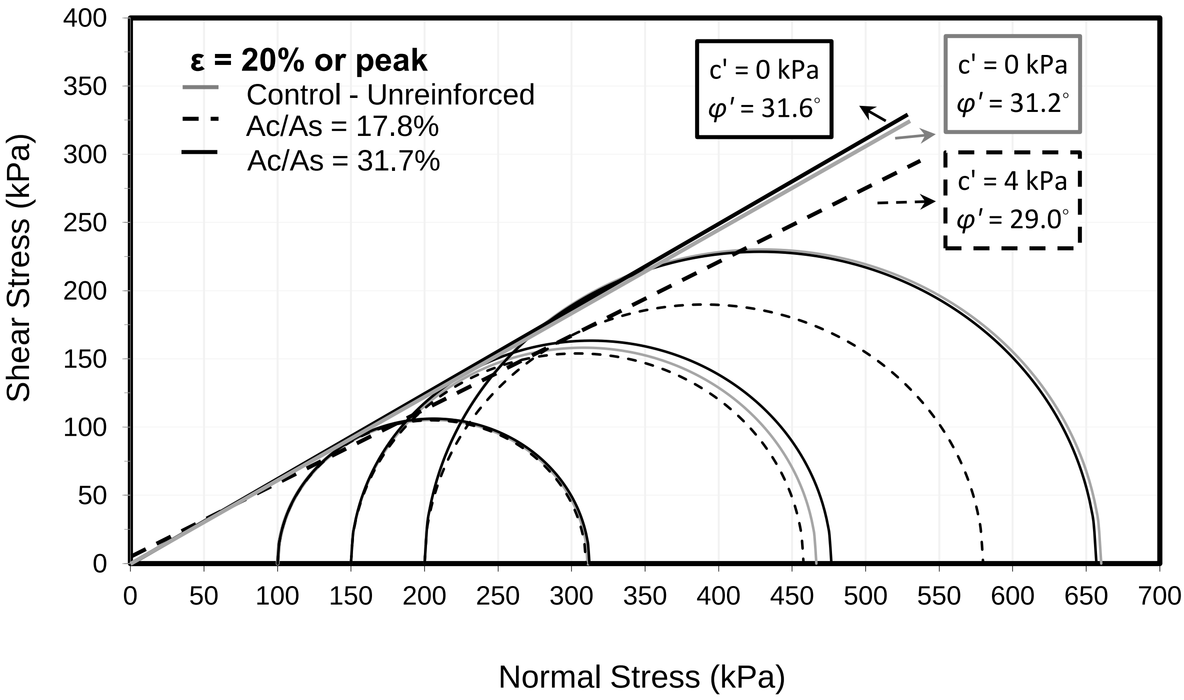


Figure 5.13: Drained failure envelopes for unreinforced and reinforced Achrafieh clay specimens

5.3 Summary of Main Findings

Based on the results of 9 consolidated drained triaxial tests that were conducted in this experimental research on control Achrafieh clay specimens, and on samples reinforced with 3cm and 4cm dense sand columns having a relative density of 86.3%, the following conclusions can be drawn with regards to the effect of sand columns on the drained response of Achrafieh clay:

- For consolidated drained control/unreinforced samples, failure was characterized mainly by uniform bulging of the clay specimen along its length with concentration at the middle portions of the sample. The same mode of failure was identified for reinforced specimens. In addition, no shear plane was observed indicating that failure has most probably occurred by bulging of sample.

- The percent improvement in the deviatoric stress for the series of tests was calculated at strains of 5%, 10%, 15% and 20%. Results indicate that the improvements in the deviatoric stresses of the 3-cm diameter dense sand columns were lower than that of the 4-cm dense sand columns. Significant improvements were noted at strains of 5%, followed by a major drop in improvements at strains of 10% and minor to no improvements recorded at strains of 15% and 20%. The improvements ranged from 28.38% to 37.5% for specimens reinforced with 3cm dense sand columns and 62.27% to 80.33% for specimens reinforced with 4cm dense sand columns at 5% strain.
- Secant moduli $(E_{sec})_{1\%}$ and $(E_{sec})_{2\%}$ defined at axial strains of 1% and 2% were calculated for each test. Results indicate that the insertion of fully penetrating dense 3-cm sand columns significantly increased the stiffness at axial strains of 1% of the unreinforced Achrafieh clay by an average of ≈111% for the three different confining pressures of 100, 150 and 200 kPa, and by an average of ≈83% at axial strains of 2%. As for the specimens reinforced with 4-cm dense sand columns, considerable increases in $(E_{sec})_{1\%}$ and $(E_{sec})_{2\%}$ were noticed with improvements averaging 202% for $(E_{sec})_{1\%}$ and 151% for $(E_{sec})_{2\%}$ for the three different confining pressures of 100kPa, 150kPa and 200kPa respectively.
- The Mohr-Coulomb envelopes were also studied at axial strains of 5%, 10%, 15% and 20%. The insertion of sand columns increased the drained angle of internal friction (ϕ') of the composite system with respective to the control tests

performed on Achrafieh clay samples when failure was defined at 5% and 10% axial strain. For samples reinforced with 4-cm dense sand columns there was an increase of up to 8.3°, compared to 4.2° for specimens that were reinforced with 3-cm dense sand columns at an axial strain of 5%.

CHAPTER 6

TEST RESULTS AND ANALYSIS FOR UNDRAINED TESTS

6.1 Introduction

The automated triaxial test setup "TruePath" by Geotac was used to conduct consolidated undrained CU tests on control and reinforced Achrafieh clay specimens saturated with a back pressure of 310 kPa. The samples were isotropically consolidated under confining pressures of 100, 150 and 200 kPa, then sheared undrained at a strain rate of 1%/hr, while measuring pore water pressure. The measured pore-water pressure reflects a global change in the composite sample and does not provide information on local changes in the pore water pressure in the sand column and the surrounding clay. Throughout the tests, the total confining pressure was kept constant as the vertical stress was increased in compression. The Achrafieh clay specimens prepared had a length of 14.2cm and a diameter of 7.1cm, and were reinforced with 3cm medium-dense Ottawa sand columns with relative density of 55.88%, and 3cm and 4cm dense sand columns with relative density of 86.3%, encompassing an area replacement ratio of 17.8% and 31.7% respectively.

The results for the CU tests that were performed on both control and composite samples are presented in this chapter. Composite samples include specimens reinforced with fully penetrating ordinary 3cm and 4cm sand columns. The results also include a description of the modes of failure that characterize the behavior of the different test specimens and a detailed analysis of the parameters which are known to affect the load response of clay specimens that are reinforced with sand columns. The effects of these parameters which include the area replacement ratio and confining pressure, on the undrained shear strength, stiffness $(E_{sec})_{1\%}$, generation of excess pore-water pressure, and effective shear strength parameters are investigated and highlighted in this chapter.

6.2 Test Results

The test results are presented in the form of deviatoric stress and excess pore-water pressure versus axial strain curves. In the analysis of the test results, failure was defined at an axial strain of 15% unless a peak was observed at smaller strain levels. However, for most tests, a peak occurred at axial strains of ≈ 4-6%.

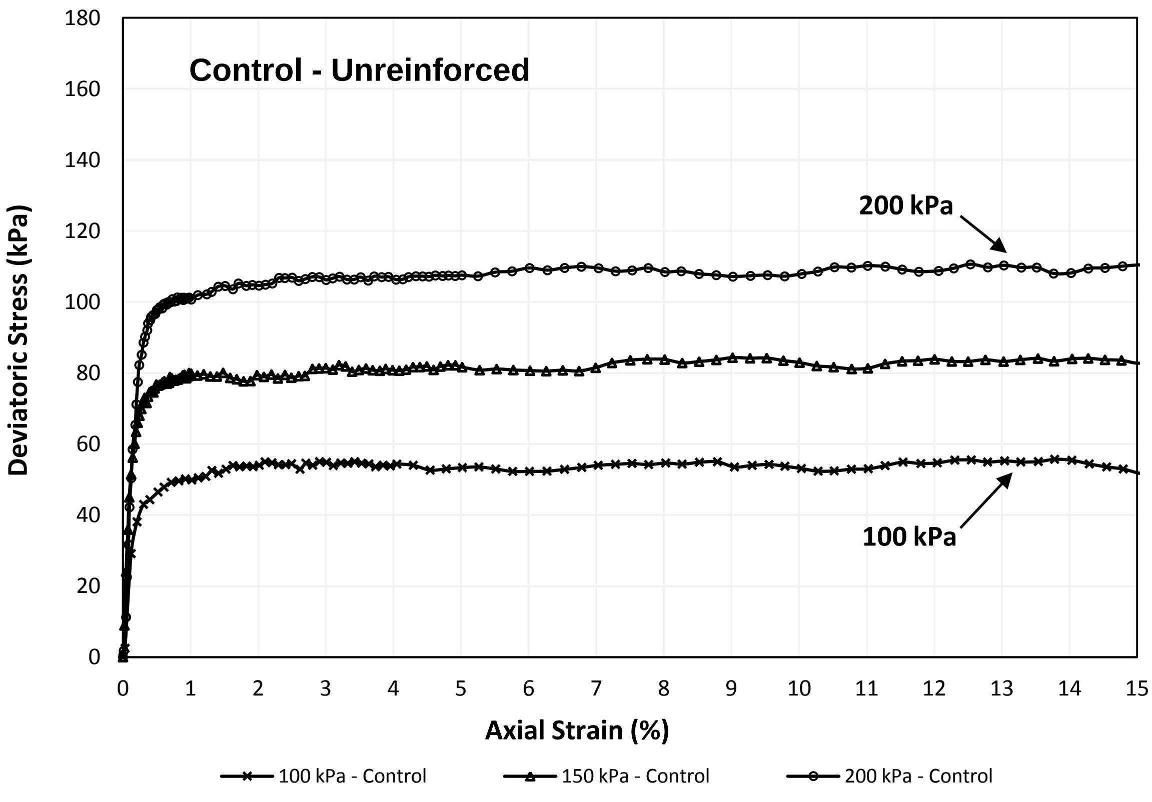
6.2.1 Unreinforced / Control Achrafieh Clay Specimens

Three consolidated undrained tests were performed on control, Achrafieh clay specimens with lengths of 14.2cm and diameters of 7.1cm, at confining pressures of 100, 150 and 200 kPa. The test results are represented in curves showing the variation of the deviatoric stress and excess pore water pressure versus axial strain in Figure 6.1. In addition, the variation of the deviatoric stress, normalized with initial effective confining pressure, is plotted against axial strain and presented in Figure 6.2.

The deviatoric stresses at failure for the control Achrafieh clay specimens were 55.0 kPa, 83.0 kPa, and 108.0 kPa, corresponding to S_u/σ_3 ratios of 0.275, 0.277, and 0.27 respectively, where S_u is the undrained shear strength. The values of S_u/σ_3 ratios are typical of normally consolidated clays prepared from slurry that are sheared in undrained conditions (example Han Lin and Penumadu (2005) and Prashant and Penumadu 2005). The excess pore water pressures at failure were 72.88 kPa, 104.14 kPa, and 136.7 kPa at confining pressures of 100 kPa, 150 kPa, and 200 kPa respectively. Skempton's pore pressure parameter "A", defined as the ratio of the excess pore-water pressure at failure to the deviatoric stress at failure, was equal to 1.33, 1.25, and 1.27 at the corresponding confining pressures, also indicating normally consolidated clay behavior.

For all the confining pressures, the deviatoric stress increased with axial strain and reached a maximum value at an axial strain of 6% to 7%, after which the curve leveled out with further increase in the axial strain. Similarly, excess pore water pressure increased with axial strain and reached a maximum value at approximately the same axial strain values before leveling out.

The secant modulus of elasticity $(E_{sec})_{1\%}$ of Achrafieh clay specimens at an axial strain of 1% was determined at the different confining pressures of 100, 150, and 200 kPa. The modulus of elasticity increased as the confining pressure increased and was equal to 4994 kPa, 7987 kPa, and 10086 kPa, respectively.



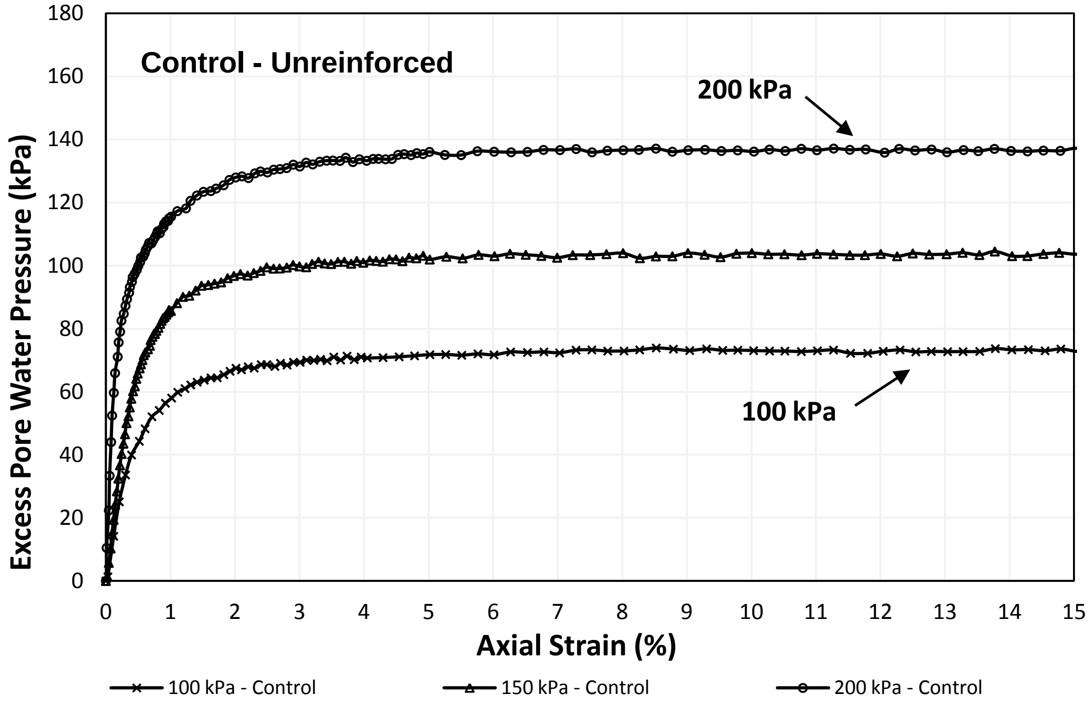


Figure 6.1: Deviatoric stress and excess pore water pressure versus axial strain for unreinforced/control Achrafieh clay specimens at confining pressures of 100 kPa, 150 kPa, and 200kPa

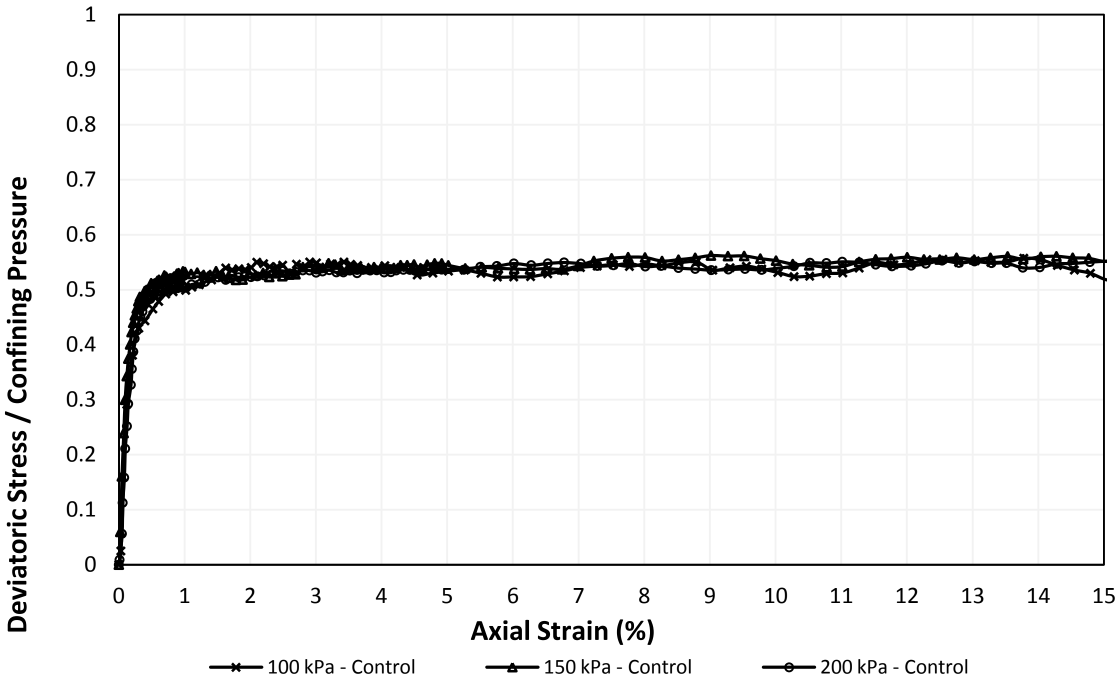


Figure 6.2: Normalized deviatoric stress with confining pressure versus axial strain for unreinforced Achrafieh clay specimens

The Mohr Coulomb effective stress failure envelope for the control specimens is shown on Figure 6.3. The apparent effective cohesion (c') and the effective angle of internal friction (Ø') for the control specimen were 5.0 kPa and 25.1° respectively.

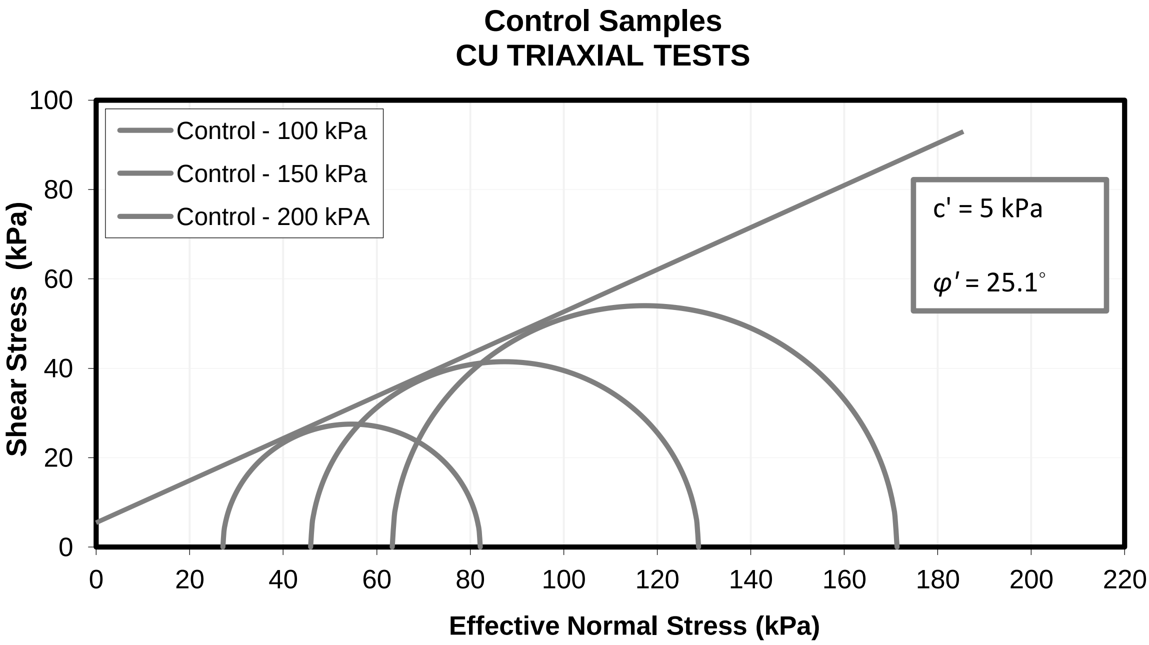


Figure 6.3: Mohr Coulomb effective stress failure envelop for control/unreinforced Achrafieh clay specimens

6.2.2 Control Ottawa Sand Specimens

Three consolidated undrained tests were performed on control, dense Ottawa sand specimens with lengths of 14.2cm and diameters of 7.1cm, at confining pressures of 100, 150 and 200 kPa. The samples had a relative density, D_r, of 86.3%. The test results are presented in curves showing the variation of the deviatoric stress and pore water pressures versus axial strain in Figure 6.4. The Mohr Coulomb effective stress failure envelope for the control specimens at the different axial strains is shown in Figure 6.5. The effective cohesion (c') and the effective angle of internal friction (Ø') for the control specimen at the different axial strains are 0 kPa and 36.5° respectively. As for the mode of failure, an obvious shear plane was observed as seen in Figure 6.6.

It should be noted that friction angle of the sand as obtained from consolidated drained triaxial tests was equal to 38.5 degrees. The reduction in the friction angle in the undrained tests is expected given the generation of negative pore pressures that would increase the effective stresses at failure to more than an order of magnitude compared to the stresses at failure in the drained tests.

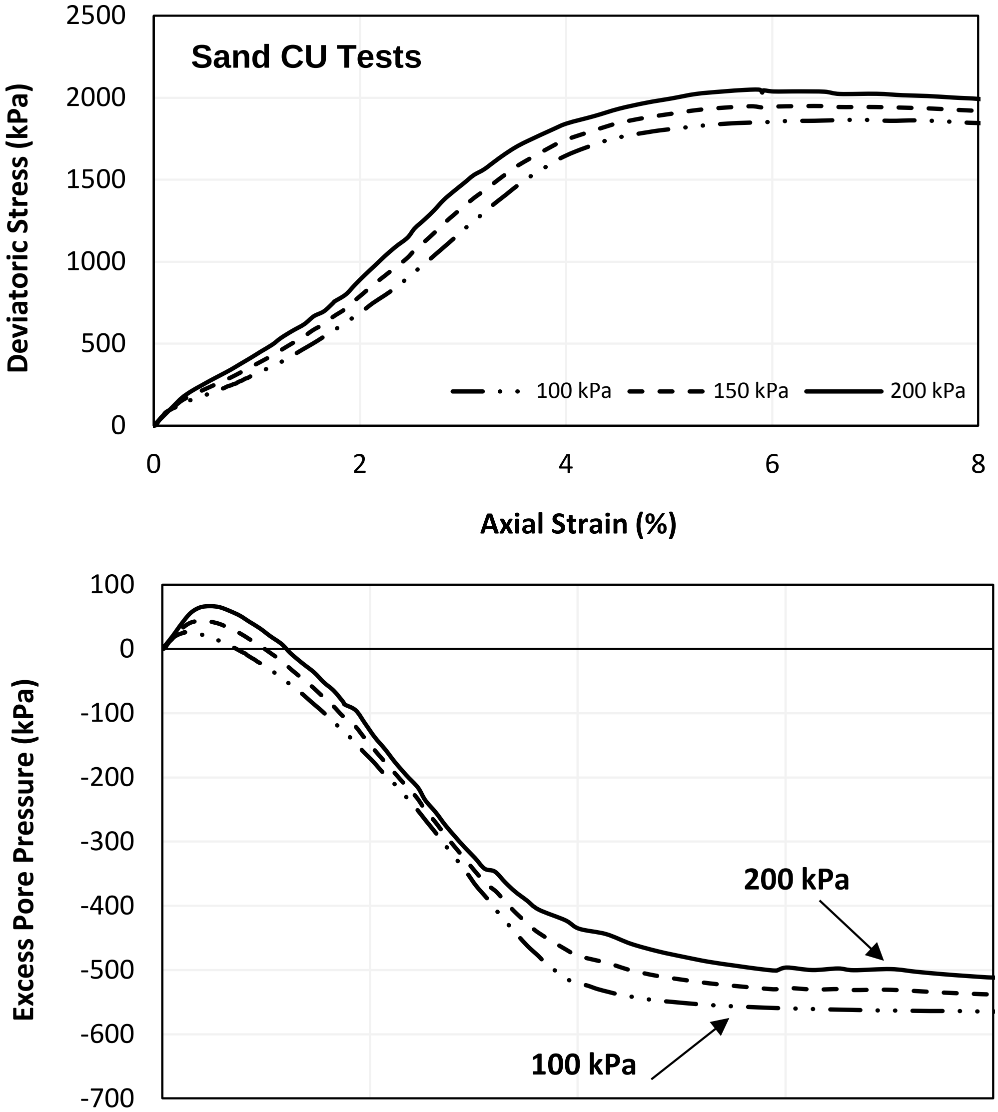


Figure 6.4: Deviatoric stress and Excess pore-pressures vs. axial strain for dense Ottawa sand specimens

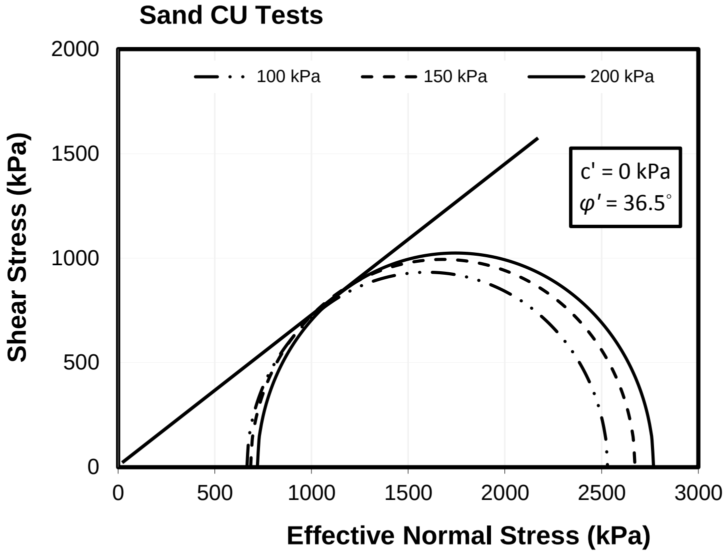


Figure 6.5: Mohr Coulomb envelope for dense Ottawa sand specimens

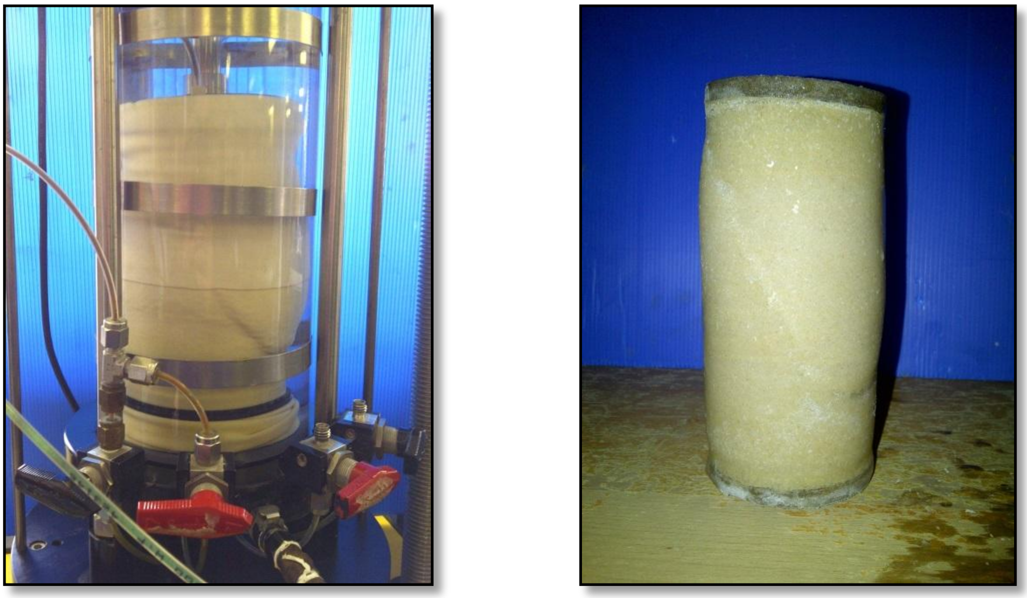


Figure 6.6: Mode of failure for sand columns

6.2.3 Achrafieh Clay Specimens Reinforced with Sand Columns

Results obtained from the consolidated undrained triaxial tests conducted on Achrafieh clay specimens reinforced with fully penetrating sand columns with different area replacement ratios and sand densities are presented in Table 6.1 and in Figure 6.7

to Figure 6.14. The figures include pictures of the modes of failure and graphs showing the variation of the deviatoric stress and excess pore-water pressure versus axial strain. The results were analyzed to investigate the effect of relevant parameters such as density of sand, area replacement ratio A_c/A_s, and confining pressure on the improvement in the undrained shear strength and the apparent effective strength parameters of the clay. It should be noted that in all the discussion presented below, it was assumed that the sand column and the surrounding clay act as a single element with homogeneous distributions of stresses and strains.

6.2.3.1 ***Mode of Failure***

For control samples, failure was mainly characterized by uniform bulging of the clay specimen along its length with the bulging generally concentrated at the middle of the sample. As for specimens reinforced with sand columns having an area replacement ratio of 31.7%, the mode of failure also involved bulging of the specimens. Photographs showing the degree of bulging at different confining pressures are presented in Figure 6.7 to Figure 6.10. It is noted that the bulging severity slightly decreases with increasing confining pressures. Therefore, bulging is more evident in the samples tested at confining pressures of 100 and 150 kPa, but less severe for the confining pressure of 200 kPa. It is also noted that at lower confining pressures, the bulging is uniform across the specimens, whereas at confining pressures of 200 kPa, the bulging seems to be more concentrated at the middle of the sample. To investigate the mode of failure of the sand columns, the same test specimens were split along their vertical axes to expose the columns and the surrounding clay. The sections shown in Figure 6.7 to Figure 6.10

indicate that the sand columns exhibited bulging at different levels, with some samples experiencing uniform bulging and others bulging concentrated at the middle.

6.2.3.2 ***Stress-Strain Behavior***

The variations of the deviatoric stress and pore water pressure versus axial strain for the composite specimens are presented in Figure 6.11, Figure 6.12 and Figure 6.13. For the CU tests performed on the specimens that were reinforced with fully penetrating sand columns having a diameter of 3cm and 4cm, i.e. tests conducted with an area replacement ratio of 17.8% and 31.7% respectively, a peak was observed in all the specimens at a strain that is less than 8%. For these tests, the failure was defined at the measured peak stress. In addition, it was observed that tests performed on specimens having an area replacement ratio of 17.8% experienced stabilization in stress after the peak has been reached unlike the test results for specimens that were reinforced with area replacement ratios of 31.7% that showed a slight decrease in deviatoric stress after the peak stress has been reached. As for the pore water pressure, initially the pressures increased as the deviatoric stresses reached their maximum values. At larger strain levels, the generation of negative pore pressures in the sand columns during shearing resulted in a reduction in the excess pore pressure of the composite samples. This decrease in excess pore water pressures during shearing can be attributed to the higher stiffness and to the dilatational tendency of the sand columns, which are expected to increase as the area replacement ratios and as the density of the sand columns increases. As expected, samples reinforced with 4-cm columns exhibited the sharpest reduction in excess pore pressures at larger strains. The strain where the maximum reduction in excess pore water pressure at this replacement ratio has been mobilized, coincided with the strain at which the peak of deviatoric stress was recorded. For samples reinforced with 3cm sand columns, the leveling off of the deviatoric stresses was achieved as the samples approached critical state conditions.

Table 6.1: Test results for all CU triaxial tests performed on control and reinforced Achrafieh Clay specimens

Test	Confining Pressure σ_3 (kPa)	Density of Sand Column	Diameter of Sand Column (cm)	Area Replacement Ratio: A_c/A_s (%)	Undrained Shear Strength (kPa)	Excess Pore Water Pressure (kPa)	E_{sec} @ 1% Axial Strain (kPa)	Increase in Undrained Shear Strength (%)	Reduction in Excess Pore Pressure (%)
1	100	-	0	0.00	27.50	72.88	4994.14	-	-
2		Medium-Dense	3	17.85	35.00	69.00	4619.12	27.27	5.32
3		Dense	3	17.85	43.00	69.40	6930.00	56.36	4.77
4		Dense	4	31.74	80.65	32.50	9082.20	193.27	55.41
5	150	-	0	0.00	41.50	104.14	7987.28	-	-
6		Medium-Dense	3	17.85	51.00	101.01	7783.11	22.89	3.01
7		Dense	3	17.85	63.00	103.80	10038.84	51.81	0.33
8		Dense	4	31.74	119.20	55.58	14525.36	187.23	46.63
9	200	-	0	0.00	54.00	136.70	10086.13	-	-
10		Medium-Dense	3	17.85	63.50	135.10	10234.75	17.59	1.17
11		Dense	3	17.85	76.70	141.00	12663.10	42.04	-3.15
12		Dense	4	31.74	145.30	80.00	20825.95	169.07	41.48

A_c/A_s = 17.8% - Medium Dense – Undrained 100, 150 & 200 kPa respectively

Figure 6.7: Photographs showing the mode of failure of specimens reinforced with 3cm meduim dense sand columns sheared under undrained conditions

A_c/A_s = 17.8% - Dense – Undrained 100, 150 & 200 kPa respectively

Figure 6.8: Photographs showing the mode of failure of specimens reinforced with 3cm dense sand columns sheared under undrained conditions

A_c/A_s = 31.7% - Dense – Undrained 100, 150 & 200 kPa respectively

Figure 6.9: Photographs showing the mode of failure of specimens reinforced with 4cm dense sand columns sheared under undrained conditions

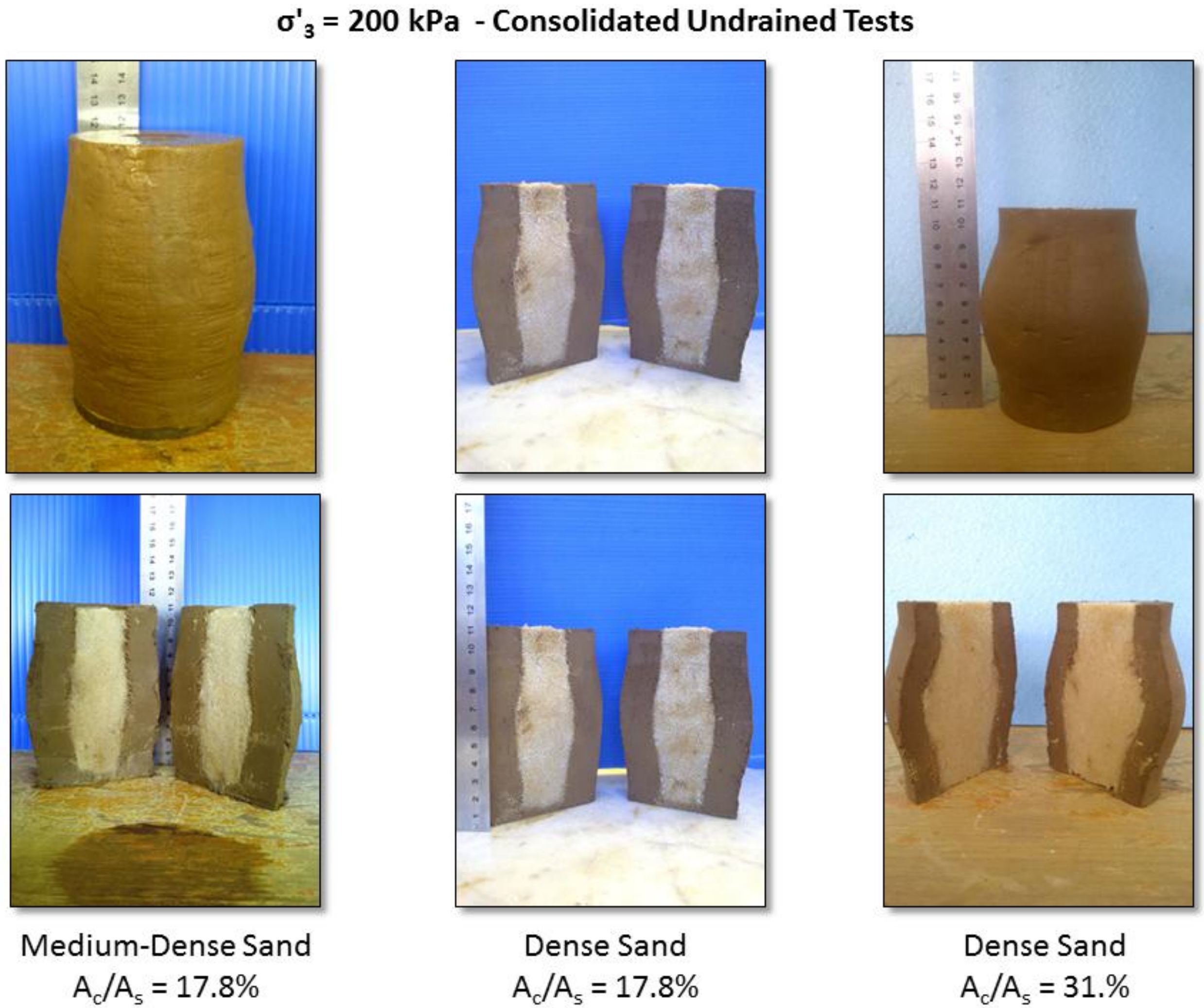
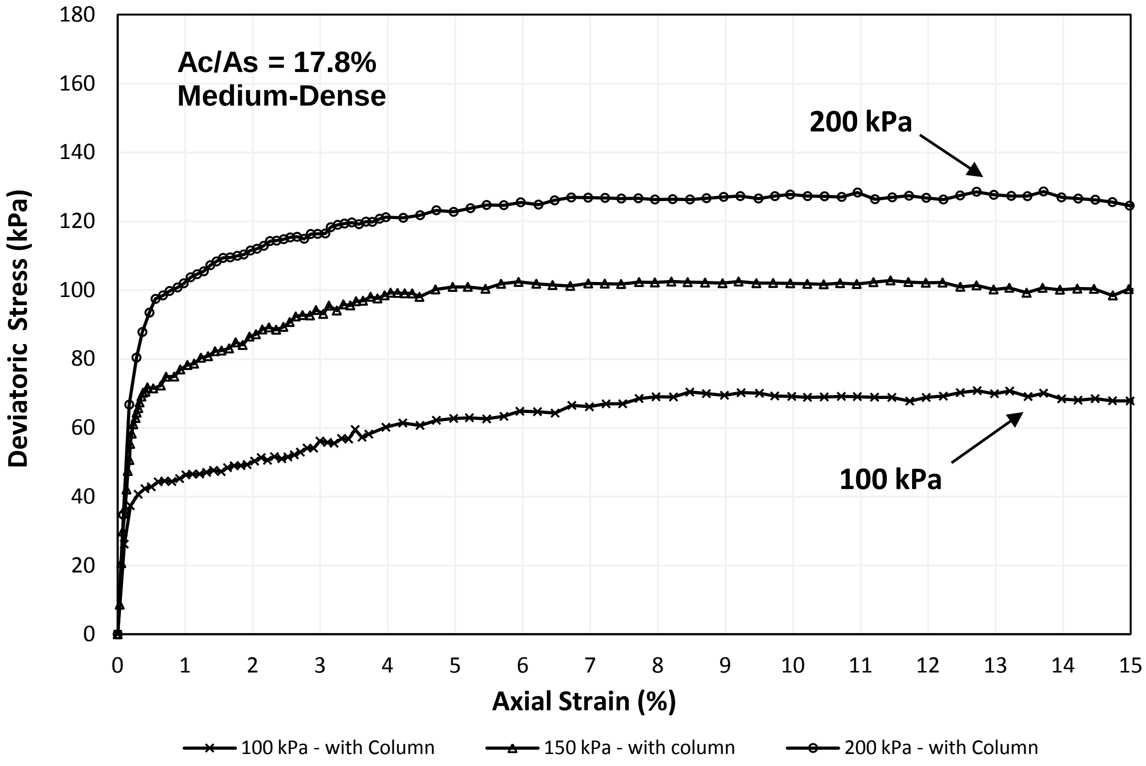


Figure 6.10: Photographs showing the mode of failure of specimens sheared under undrained conditions



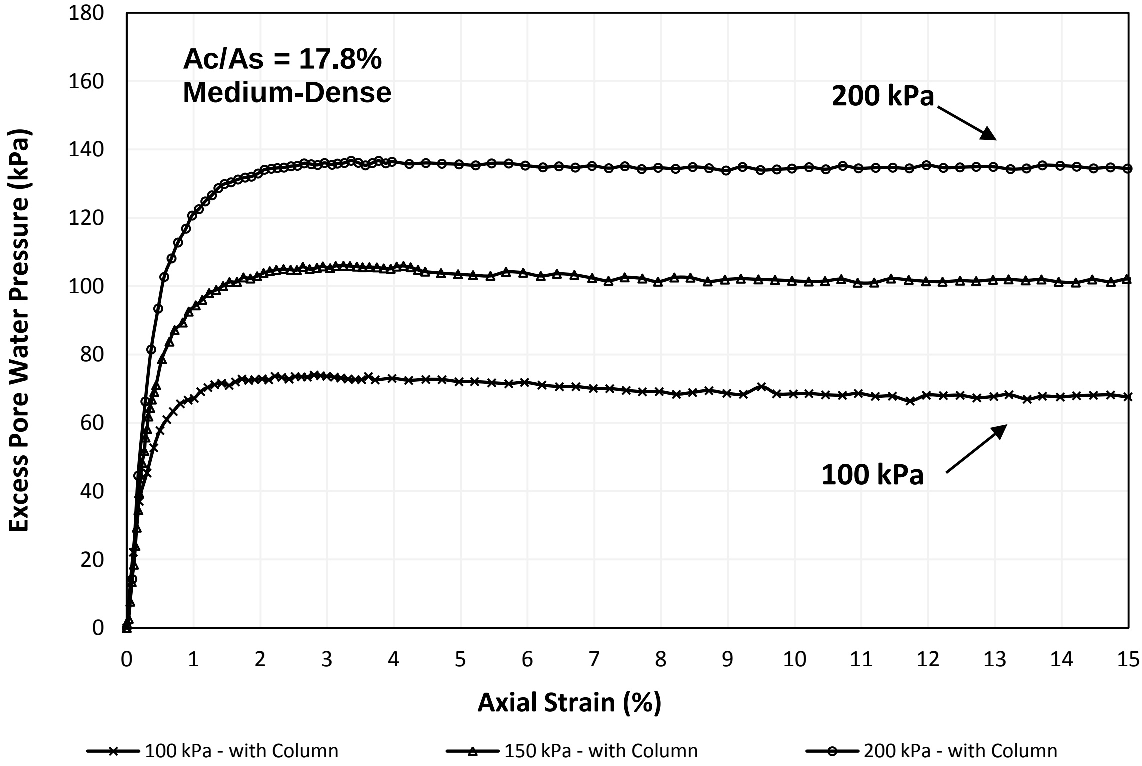
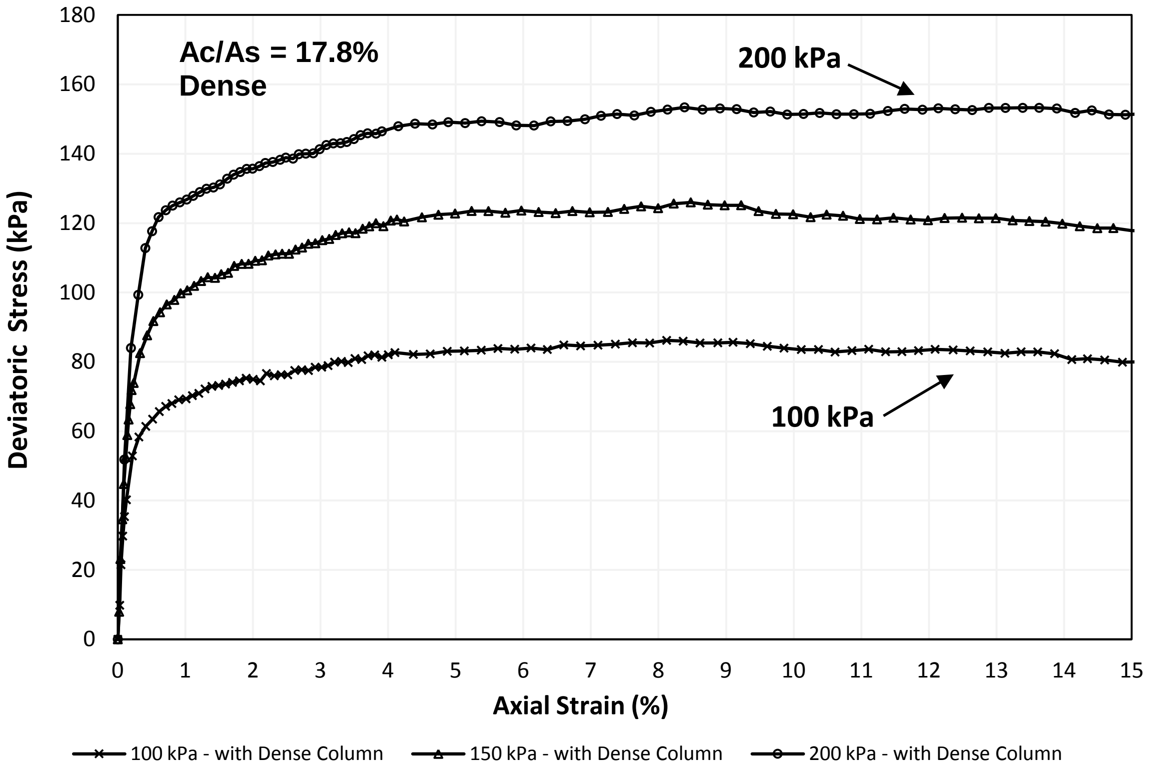


Figure 6.11: Deviatoric stress and pore-water pressure versus axial strain for specimens reinforced with 3cm Medium-Dense sand columns at confining pressures of 100 kPa, 150 kPa, and 200kPa



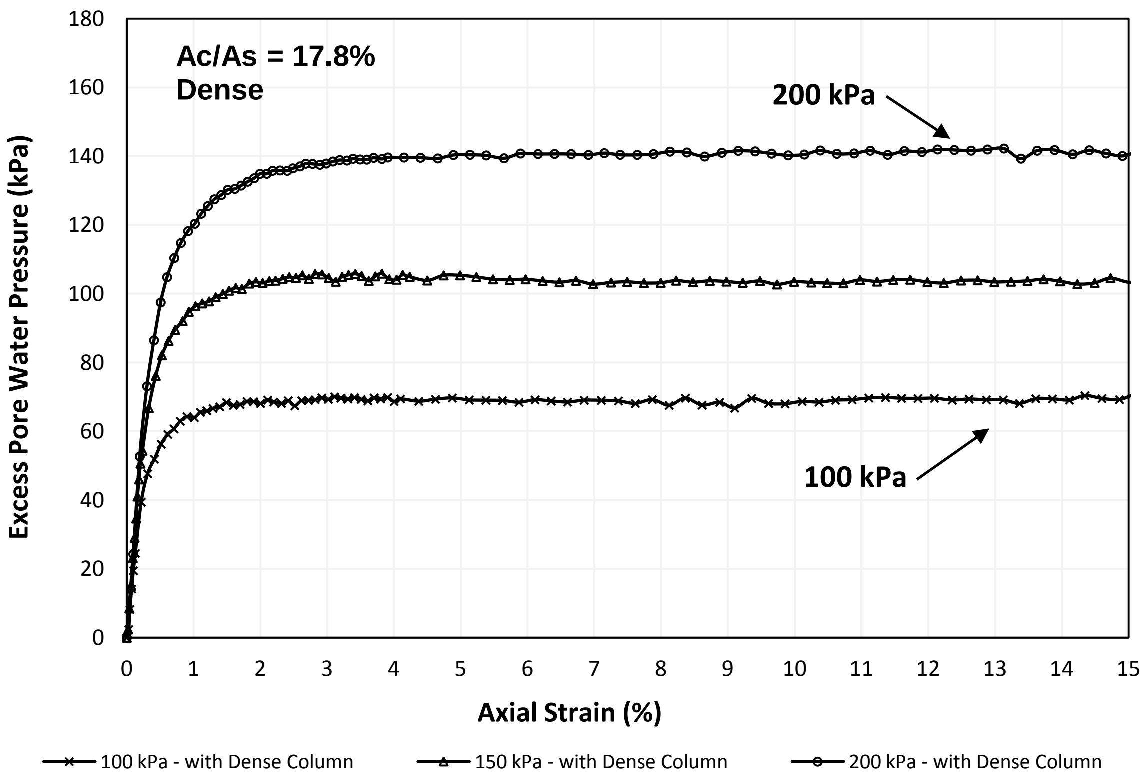


Figure 6.12: Deviatoric stress and pore-water pressure versus axial strain for specimens reinforced with 3cm dense sand columns at confining pressures of 100 kPa, 150 kPa, and 200kPa

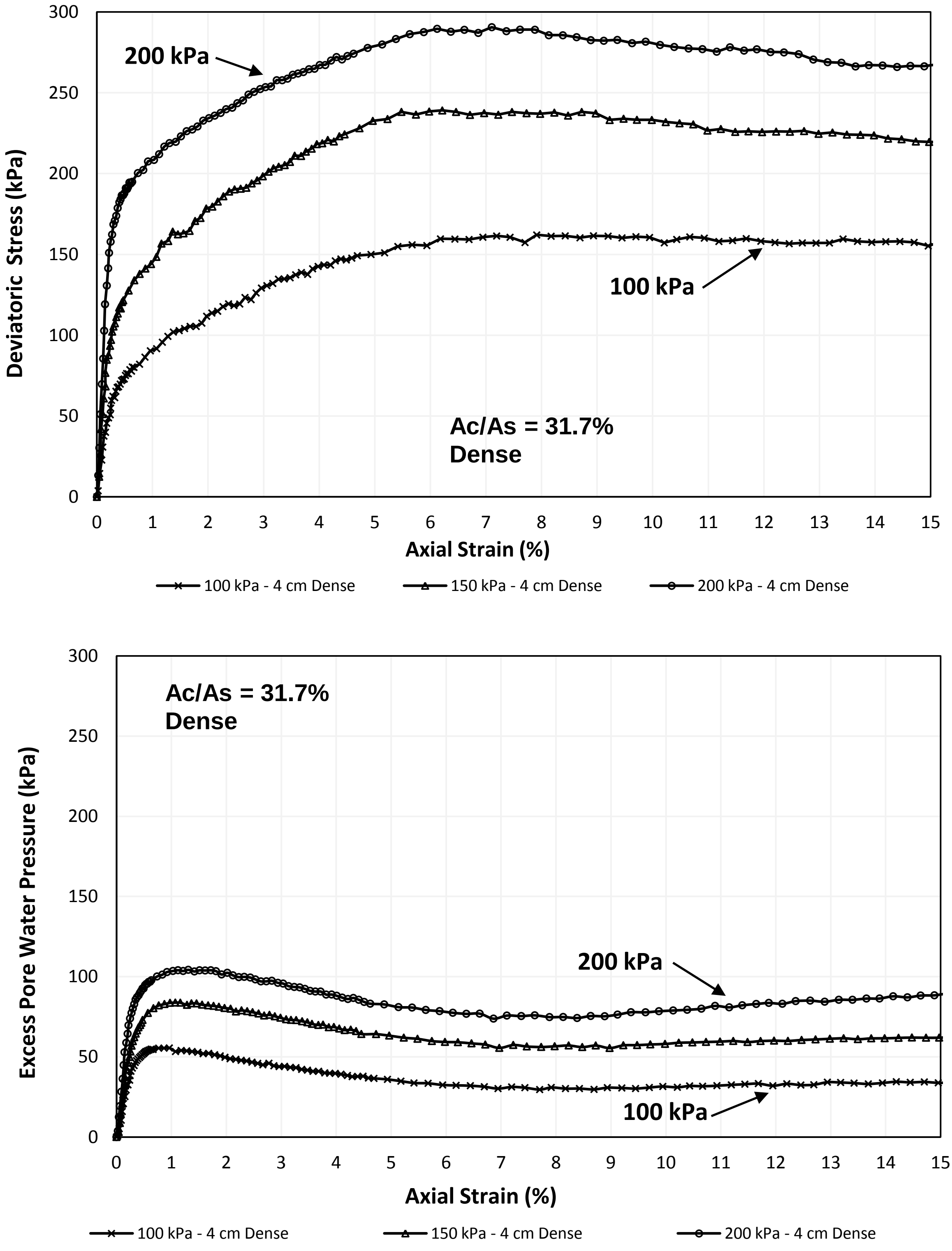


Figure 6.13: Deviatoric stress and pore-water pressure versus axial strain for specimens reinforced with 4cm dense sand columns at confining pressures of 100 kPa, 150 kPa, and 200kPa

Figure 6.14 shows a compilation of deviatoric stress and excess pore pressure curves for control specimens and specimens having area replacement ratios of 17.8%, and 31.7%. The results indicate that significant improvements in the undrained load response were observed particularly for the specimens reinforced with dense sand columns with higher relative density, especially at area replacement ratios of 31.7%, where the pore pressure response showed a significant tendency for the generation of excess negative pore pressures in the reinforced specimens. Specimens reinforced with medium-dense sand columns showed minor improvements in the undrained load response with decreasing improvement as the confining pressures increased. The generation of negative pore pressures resulted in significant increases in the effective confining pressure on the specimens reinforced with 3cm and 4cm dense sand columns, compared to the specimens reinforced with 3cm medium-dense columns and the control clay specimens.

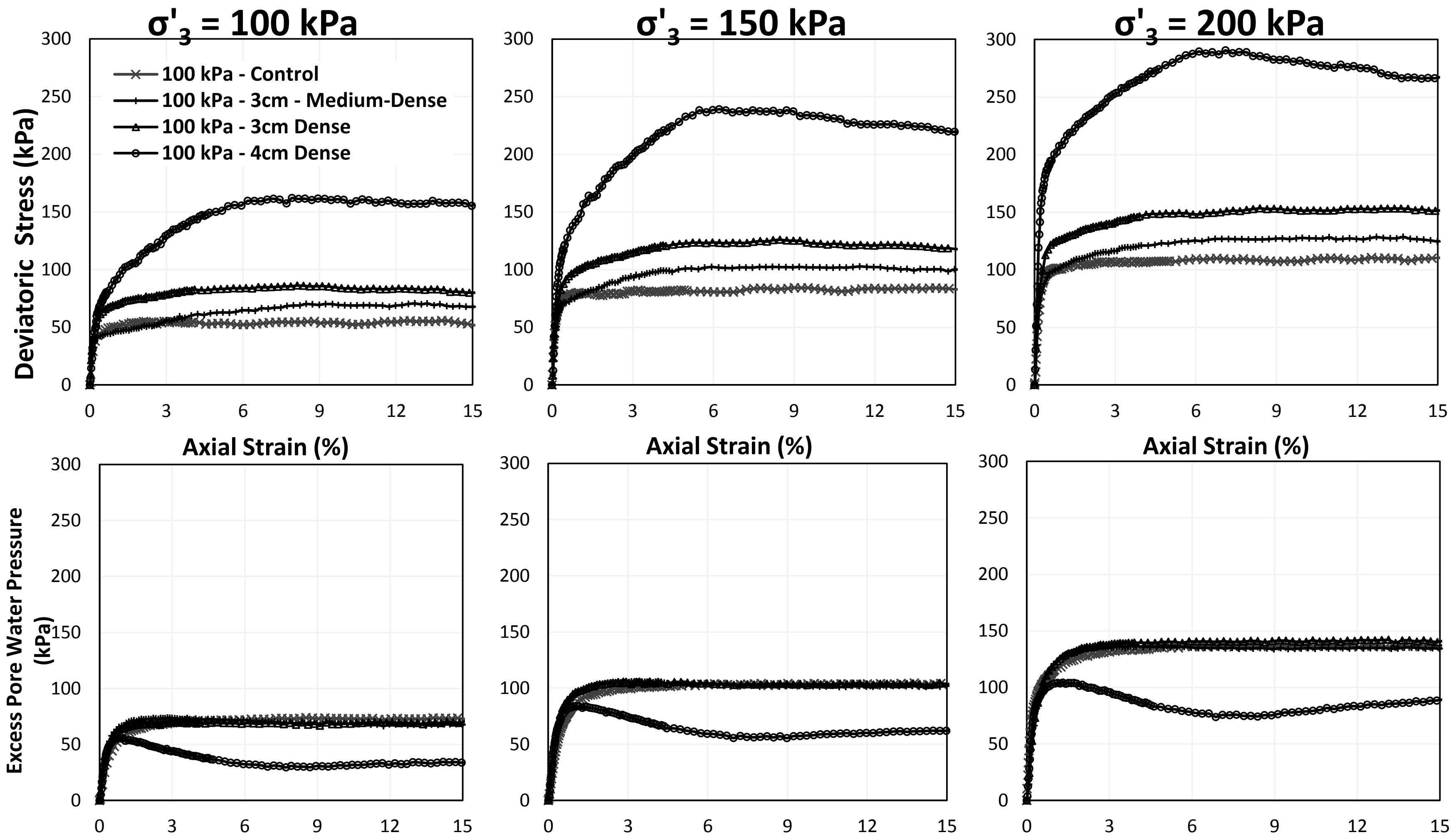


Figure 6.14: Deviatoric stress & pore-water pressure vs. axial strain for reinforced specimens at confining pressures of 100, 150 and 200 kPa

6.2.3.3 *Effect of Sand Columns on the Undrained Shear Strength at Failure*

The percent improvement in the undrained shear strength at failure for the series of tests are presented in Table 6.1 and plotted versus the initial effective confining pressure in Figure 6.15 and Figure 6.16. Results indicate that the use of 3-cm diameter medium-dense sand columns resulted in minor increases in the undrained shear strength. Whereas significant increases in undrained shear strength were observed in specimens reinforced with 4-cm diameter dense sand columns (area replacement ratio=31.7%) with the highest improvement being 193.27% at 150kPa confining pressure.

The improvements ranged from 17.59% to 27.27% for specimens reinforced with 3cm medium-dense sand columns, 42.04% to 56.36% for specimens reinforced with 3cm dense sand columns and 169.07% to 193.27% for specimens reinforced with 4cm dense sand columns. In comparison with the smaller area replacement ratio of 17.8%, the 31.7% resulted in significant improvements in the undrained shear strength.

These results show that the improvement in undrained shear strength was relatively slightly affected by the effective confining pressure, with the percent improvement showing a slight decrease as the initial effective confining pressure increased from 100 kPa to 200 kPa.

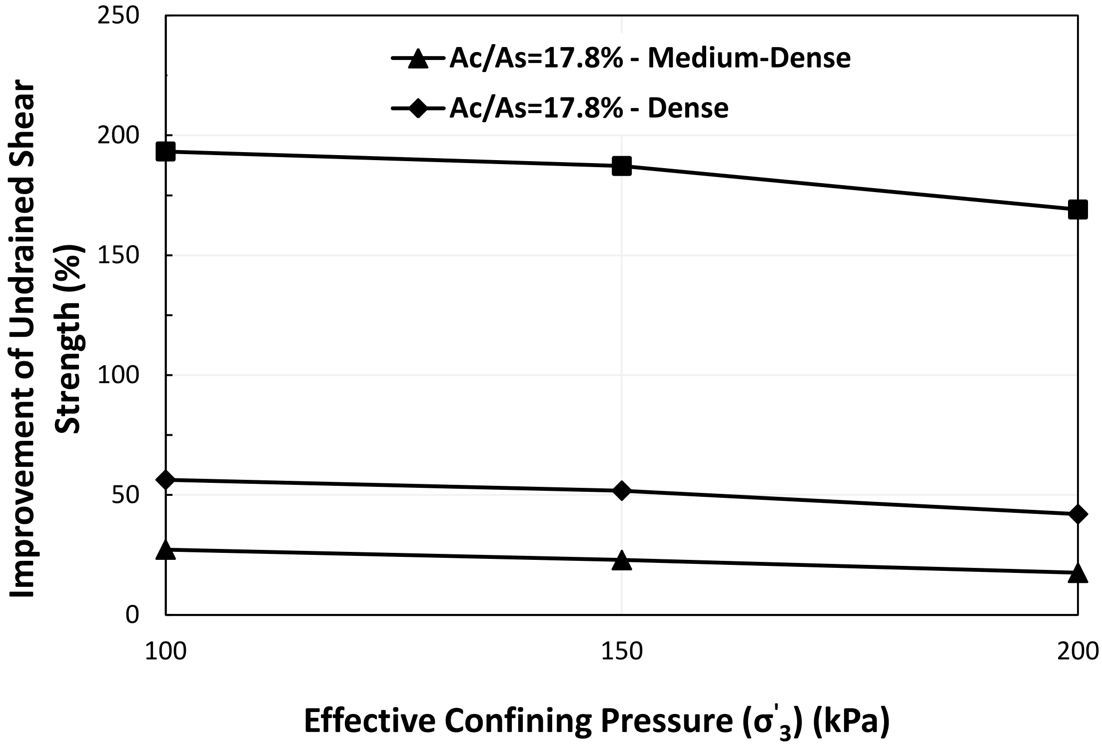


Figure 6.15: Variation of improvement in undrained shear strength with confining pressure (Hc/Hs= 1, As/Ac=17.8%,As/Ac=31.7%, ordinary)

6.2.3.4 *Effect of Sand Columns on Excess Pore-Water Pressure Generation*

An analysis of the results in Table 6.1 indicates that for specimens reinforced with fully penetrating medium-dense sand columns having an area replacement ratios of 17.8%, the reduction in the excess pore-water pressure at confining pressures of 100, 150 and 200 kPa was 5.32%, 3.01% and 1.17% respectively. These results show that minor reductions in excess pore-water pressures were noticed. The same was observed for the specimens having the same area replacement ratio of 17.8% but reinforced with dense sand columns. Minor reductions in excess pore-water pressures were observed, where the reduction in excess pore-water pressures was 4.77%, 0.33% and 0% at confining pressures of 100, 150 and 200 kPa respectively. Hence, it was noticed that the insertion of sand columns with area replacement ratios of 17.8% does not reduce the excess pore-water pressures generated during undrained loading. In addition, the effectiveness in reducing the pore-water pressures decreases with increasing the effective confining pressures.

As for the specimens that were reinforced with dense sand columns resembling an area replacement ratio of 31.7%, a significant reduction in excess pore-water pressures was observed, with the effectiveness also decreasing with increasing the confining pressures. The reduction in excess pore-water pressures for confining pressures of 100, 150 and 200 kPa were 55.41%, 46.63% and 41.48% respectively. This decrease in pore-water pressures is associated with the behavior of the sand columns which tend to dilate and generate negative pore pressures. Figure 6.16 indicates the relationship between the reduction in excess pore-water pressures and the percent improvement in undrained shear strength for all reinforced specimens that were tested under consolidated undrained conditions. Results on Figure 6.16 indicate that for the cases were there were no significant reductions in the excess pore pressures, the improvement in the deviatoric stress at failure due to the addition of columns was relatively small. The significant improvement was only evident when the reduction in excess pore pressures was clearly evident (case of dense columns with high area replacement ratio of about 31.7%).

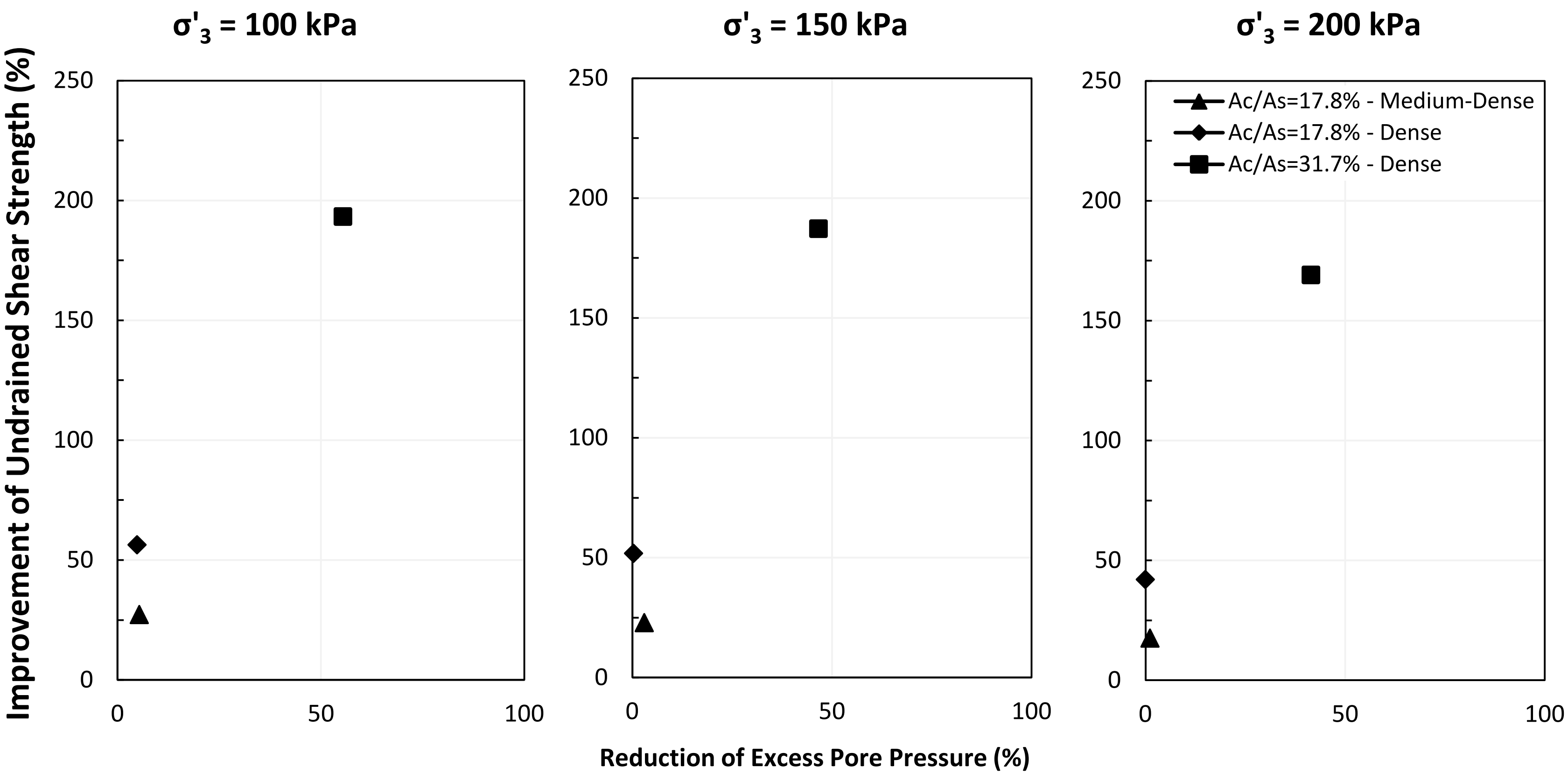


Figure 6.16: Relationship between improvements in undrained shear strength and reduction in excess pore pressure at failure (Ac/As= 17.8%, Ac/As= 31.7)

6.2.3.5 ***Effect of Sand Columns on the Undrained Secant Modulus***

A secant modulus $(E_{sec})_{1\%}$ defined at an axial strain of 1% was calculated for each test by dividing the deviatoric stress measured at an axial strain of 1% by the corresponding strain. The results of the calculated values of $(\mathrm{E_{sec}})_{1\%}$ are presented in Table 6.1 and plotted in Figure 6.17. As indicated by the test results of Table 6.1, the insertion of fully penetrating medium-dense 3cm sand columns slightly decreased the stiffness of the unreinforced Achrafieh clay by an average of ≈2.86% for the three different confining pressures of 100, 150 and 200 kPa.

As for the specimens reinforced with 3cm dense sand columns, $(E_{sec})_{1\%}$ increased from 4994.14 to 6930, from 7987.28 to 10038.84 and from 10086.13kPa to 12663.10kPa for confining pressures of 100, 150 and 200kPa respectively. Thus, the average increase in $(E_{sec})_{1\%}$ was ≈30%.

As for the specimens reinforced with 4cm dense sand columns $(E_{sec})_{1\%}$ increased from 4994.14 to 9082.20, from 7987.28 to 14525.36 and from 10086.13kPa to 20825.95kPa for confining pressures of 100, 150 and 200kPa respectively. Thus, the average increase in $(E_{sec})_{1\%}$ was ≈90%. This indicates that for specimens reinforced with 4cm dense sand columns, more stresses will be distributed along the column length, and consequently less settlement will result. The percent improvements in the undrained secant modulus for the series of tests are plotted versus the initial effective confining pressures in Figure 6.19.

The dependency of the drained secant modulus on the strain level was investigated by plotting the variation of E_{sec} with the axial strain for control and reinforced specimens reinforced with area replacement ratios of 17.8% and 31.7% at different effective confining pressures of 100, 150, and 200 kPa as shown in Figure

6.18. The results indicated that the secant modulus for the reinforced and the control specimens decreases as the axial strain increases, reflecting the nonlinearity in the stress-strain response. The specimens exhibited a sharp drop in the secant stiffness for strains that are less than 1% to 2%. After a strain of 2%, the stiffness decreases with strain at a decreasing rate.

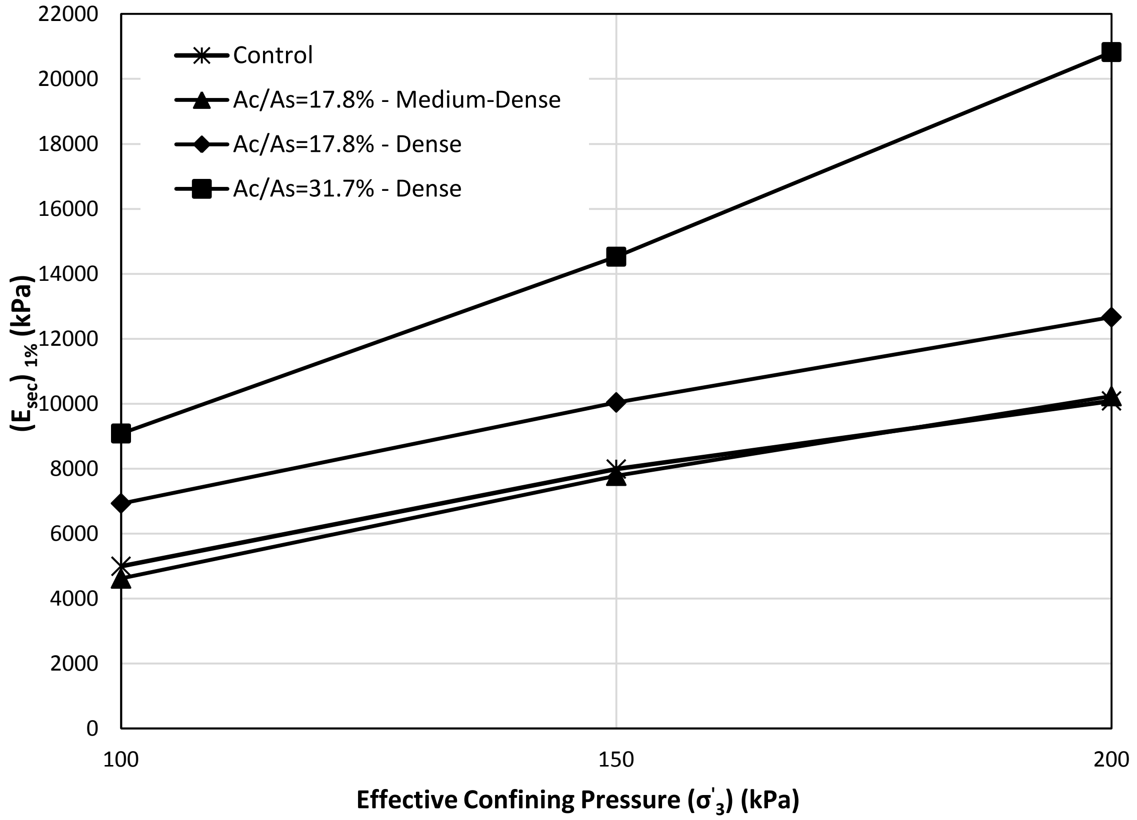
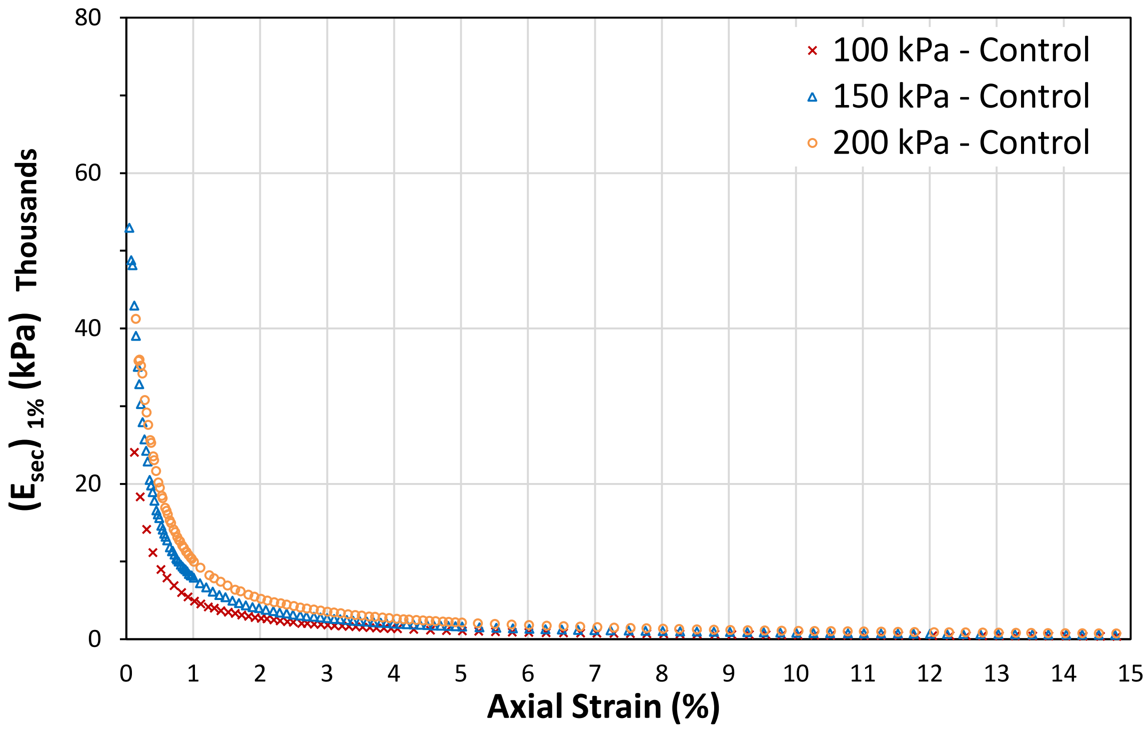
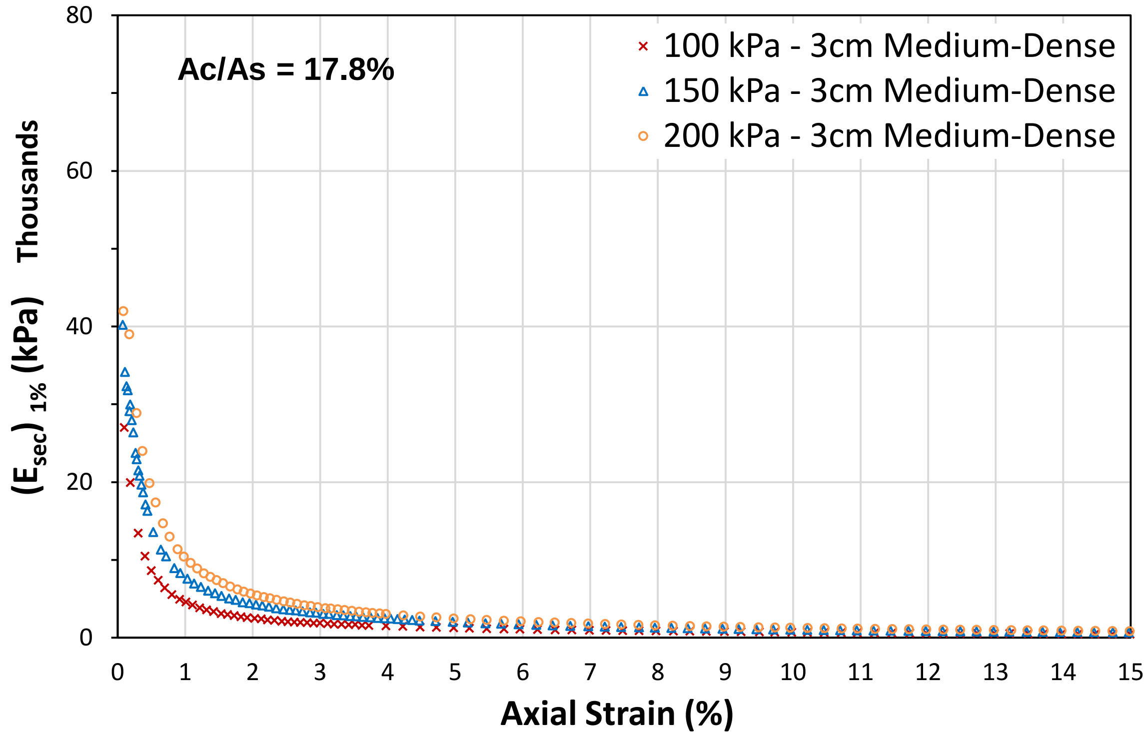


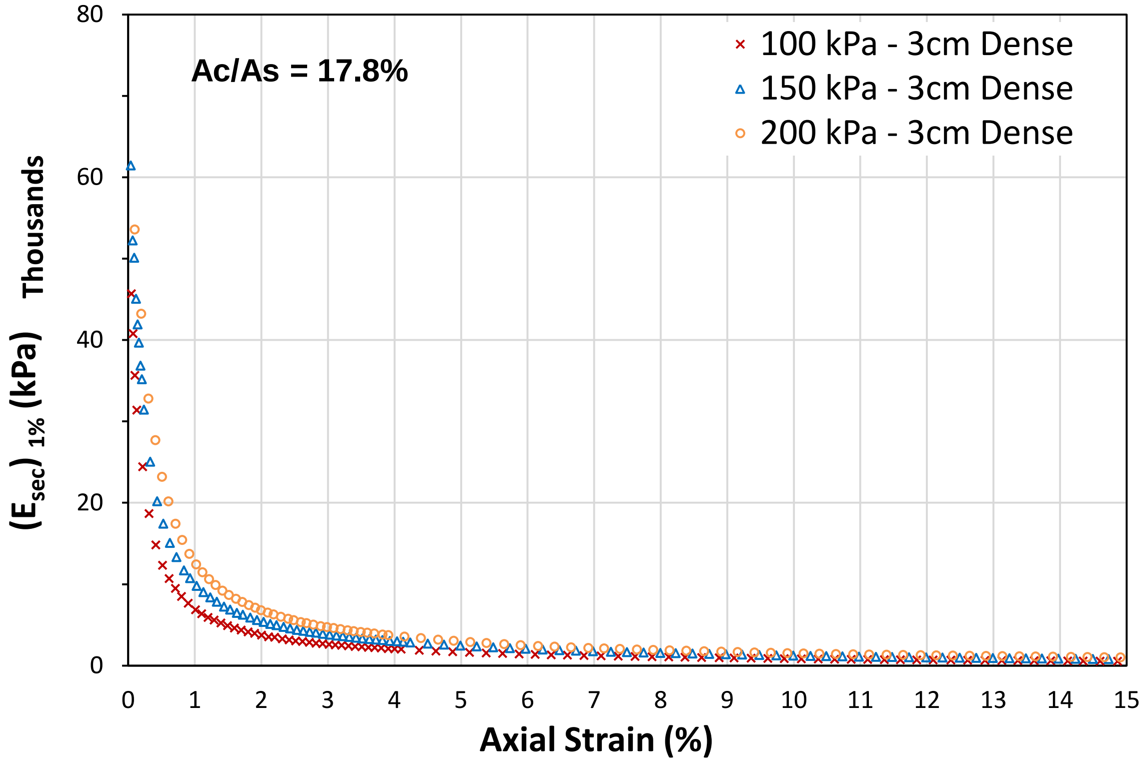
Figure 6.17: Vaiation of (Esec)1% with confining pressure



100 kPa - Control
150 kPa - Control
200 kPa - Control
(E_sec) 1% (kPa) Thousands
Axial Strain (%)



Ac/As = 17.8%
100 kPa - 3cm Medium-Dense
150 kPa - 3cm Medium-Dense
200 kPa - 3cm Medium-Dense
(E_sec) 1% (kPa) Thousands
Axial Strain (%)



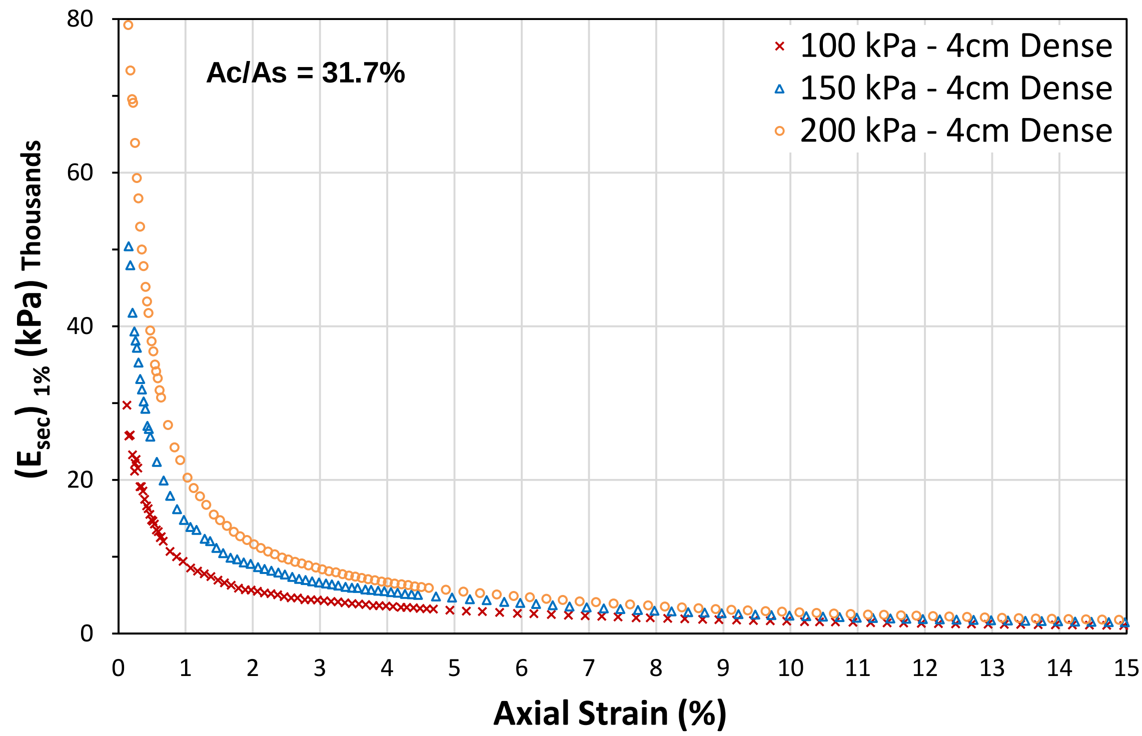


Figure 6.18: Variations of (Esec) with strain for composite specimens

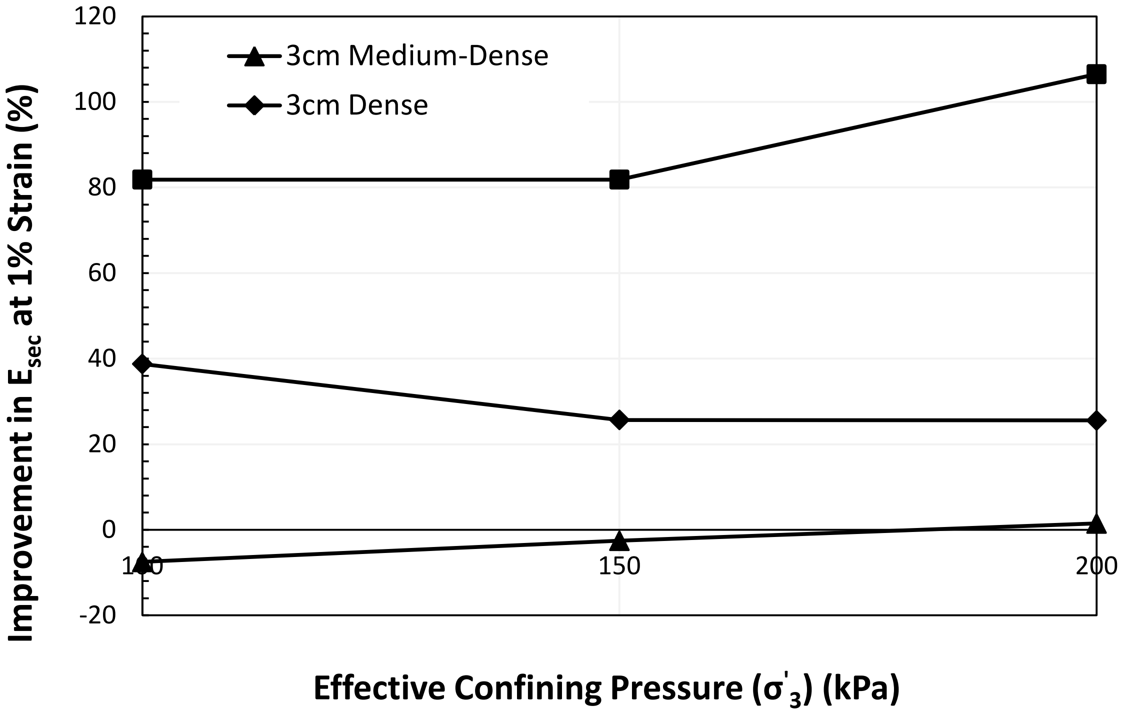


Figure 6.19: Variation of improvement of Esec at 1% strain with varying effective confining pressure

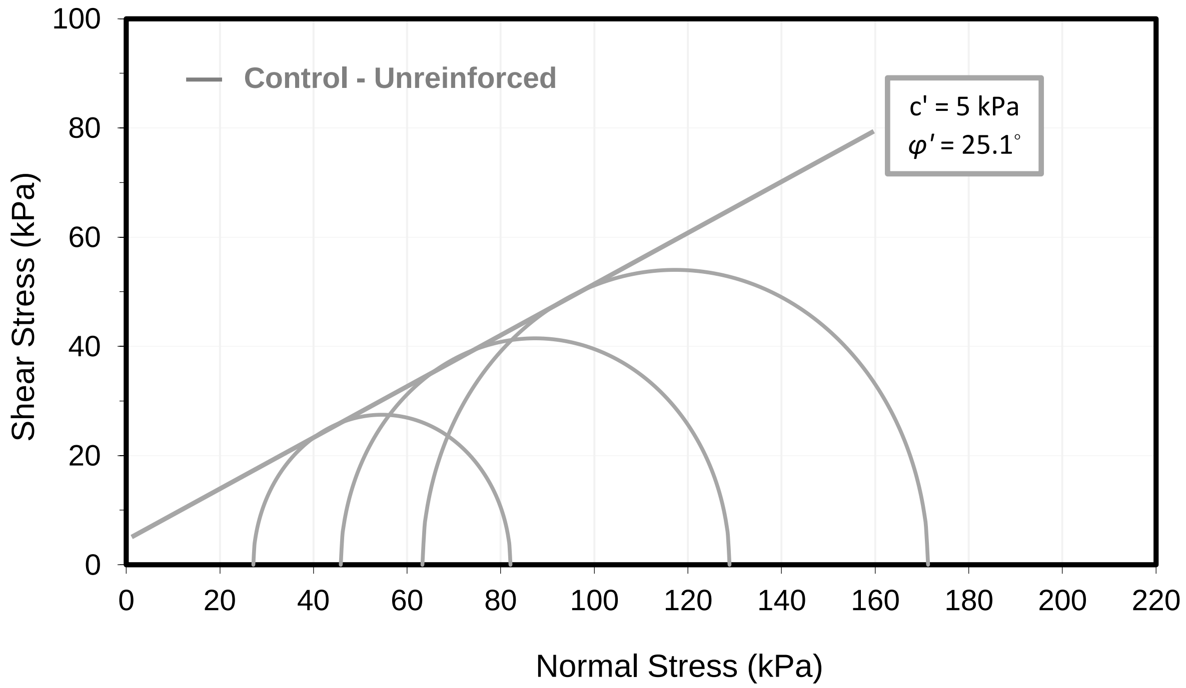
6.2.3.6 ***Effect of Sand Columns on the Effective Shear Strength Parameters***

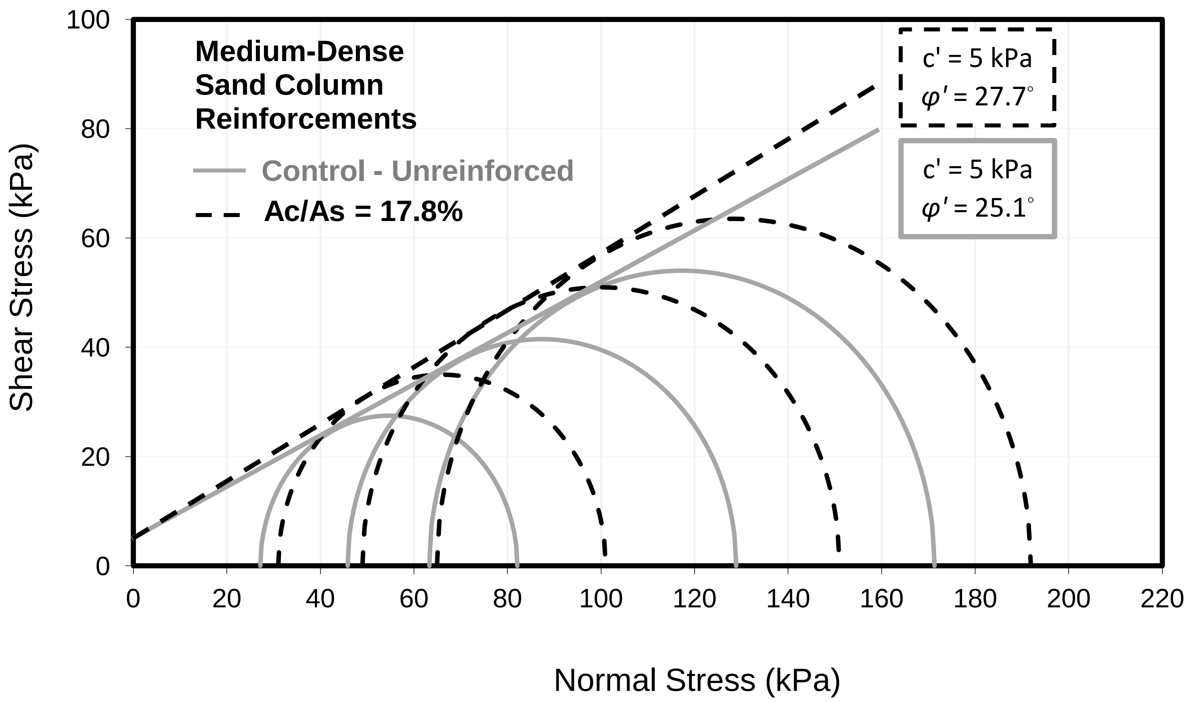
Figure 6.20 shows the effective Mohr-Coulomb envelops corresponding to the unreinforced control specimens, reinforced specimens with fully penetrating 3cm medium-dense sand columns, reinforced specimens with fully penetrating 3cm dense sand columns and reinforced specimens with fully penetrating 4cm dense sand column. In addition, the resulting shear strength parameters c' and ϕ' for the different series of undrained tests performed are summarized in Table 6.2.

As indicated by Table 6.2, the insertion of sand columns increased the effective friction angle (ϕ') of the composite system with respect to the control tests performed on Achrafieh clay samples. This increase is most noticeable for samples reinforced with dense sand columns, irrespective of the area replacement ratios, where the effective friction angle was found to increase from 25.1° to about 32.5°. For the medium dense sand, the effective friction angle was found to increase from 25.1° to 37.7°. The effective apparent cohesion (c') was found to be insensitive to the presence of the sand columns, with a minor reduction (from 5 kPa to 0 kPa) noted for the samples reinforced with dense 4cm sand columns.

Table 6.2: Effective shear stress failure parameters for CU tests

Effective Shear Stress Parameters (c', ϕ') for CU tests			
Area Replacment Ratio A_c/A_s (%)	Desnity of Sand Column	c' (kPa)	ϕ' (°)
Control	-	5	25.1
17.80%	Medium Dense	5	27.7
17.80%	Dense	5	32.2
31.70%	Dense	0	33.6


Control - Unreinforced
c' = 5 kPa
φ' = 25.1°
Shear Stress (kPa)
Normal Stress (kPa)
0
20
40
60
80
100
0
20
40
60
80
100
120
140
160
180
200
220


Medium-Dense
Sand Column
Reinforcements
Control - Unreinforced
Ac/As = 17.8%
c' = 5 kPa
φ' = 27.7°
c' = 5 kPa
φ' = 25.1°
Shear Stress (kPa)
Normal Stress (kPa)
0
20
40
60
80
100
0
20
40
60
80
100
120
140
160
180
200
220

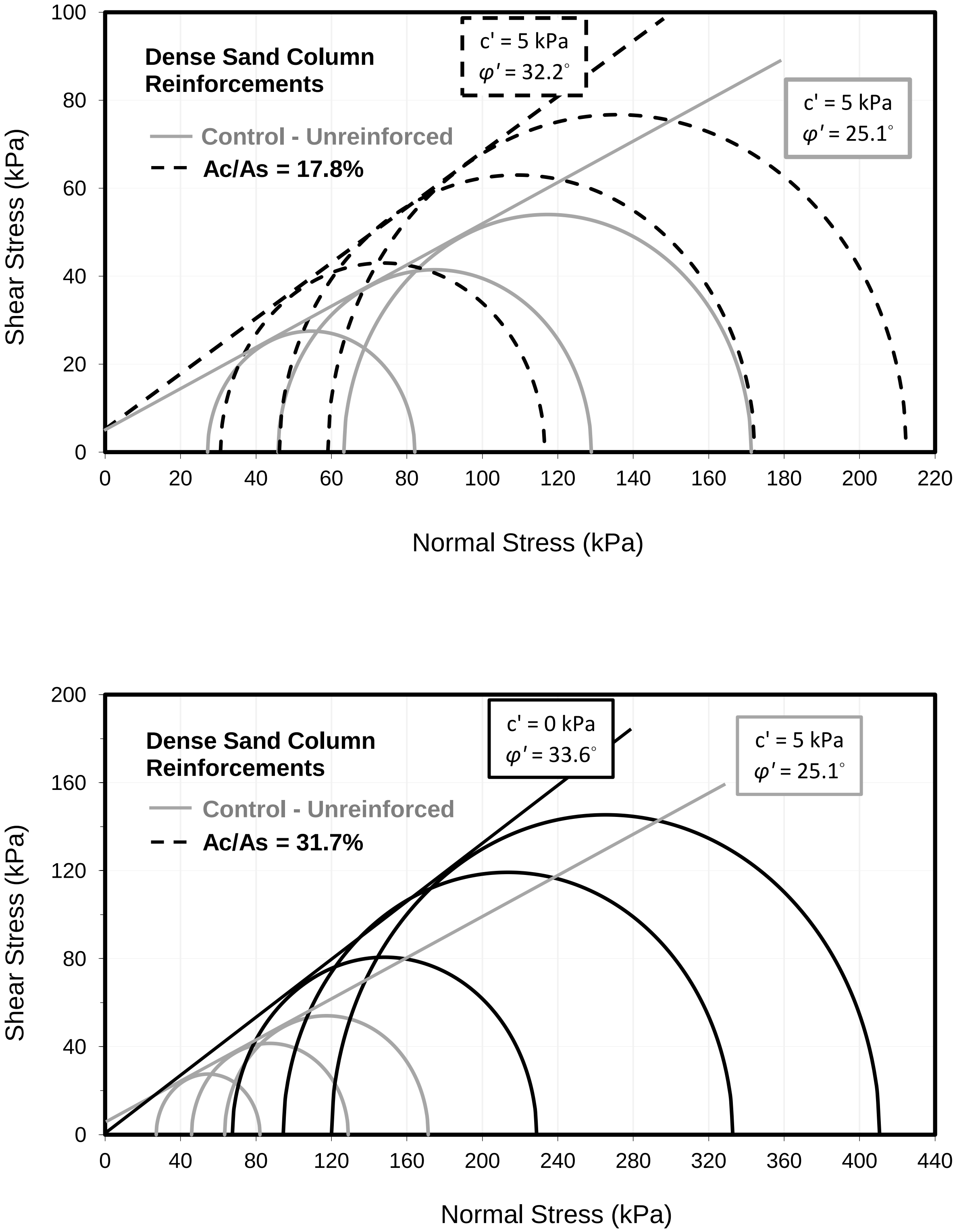


Figure 6.20: Drained failure envelopes for unreinforced and reinforced Achrafieh clay specimens

6.3 Comparison of triaxial tests performed on Achrafieh clay with Kaolin clay for control and samples reinforced with Medium- Dense Sand Columns

A series of consolidated triaxial tests were conducted on Kaolinite clay by Maakaroun (2010), Maalouf (2012) and Bou Lattouf (2013) at the soil laboratory of the American University of Beirut. These tests involved the study of the behavior of Kaolin clay reinforced with medium dense Ottawa sand having a relative density of 55.8% while varying several parameters including effective confining pressure (100, 150, and 200 kPa) and area replacement ratio (17.8% and 31.7%). In addition, all specimens tested had a length of 14.2cm and a diameter of 7.1cm. Therefore a one-to-one comparison of results could be conducted to study the effect of the soil type (kaolinite versus natural Achrafieh clay) on the response of the composite, especially that the same procedures were adopted in these studies and the current study with regards to preparation and testing of samples.

This section presents a comparison between the results for the consolidated undrained triaxial tests performed on Achrafieh clay and the results performed on Kaolin clay. It is worth noting that all the soil specimens were tested in the same soil laboratory, have the same dimensions of 7.1cm diameter and 14.2cm length, using the same triaxial cell, same Ottawa sand, same density for the sand column, same area replacement ratio and following approximately the same experimental procedure. In what follows is a brief comparison of results obtained regarding the deviatoric stress and pore pressures versus axial strain, the mode of failure, degree of improvement and Mohr-Coulomb envelopes of the Kaolin and Achrafieh clay.

6.3.1 *Comparison of Control Tests*

The deviatoric stresses at failure for the control Achrafieh clay specimens were 55.0 kPa, 83.0 kPa, and 108.0 kPa, corresponding to S_u/σ_3 ratios of 0.275, 0.277, and 0.27 respectively, where S_u is the undrained shear strength. Whereas, the deviatoric stresses at failure for the control Kaolin specimens were 64.6 kPa, 84.2 kPa, and 110.2 kPa, corresponding to Su/σ3 ratios of 0.32, 0.28, and 0.27 respectively. The values of S_u/σ_3 ratios for both Achrafieh and Kaolin clay are typical of normally consolidated clays prepared from slurry that are sheared in undrained conditions. The excess pore-water pressures at failure for the Achrafieh clay were 72.88 kPa, 104.14 kPa, and 136.7 kPa at confining pressures of 100 kPa, 150 kPa, and 200 kPa respectively. As for the Kaolin clay, the excess pore water pressures at failure were 61.3 kPa, 95.1 kPa, and 130.9 kPa at confining pressures of 100 kPa, 150 kPa, and 200 kPa respectively. The Skempton's pore pressure parameter "A" was equal to 1.33, 1.25, and 1.27 for the Achrafieh clay and 0.95, 1.12, and 1.19 for Kaolin clay at the corresponding confining pressures, also indicating normally consolidated clay behavior.

For both types of clay, and at all confining pressures, the deviatoric stress increased with axial strain and reached a maximum value at an axial strain of 6% to 7%, after which the curve leveled out with further increase in the axial strain. Similarly, excess pore water pressure increased with axial strain and reached a maximum value at approximately same axial strain values before leveling out.

The secant modulus of elasticity $(\mathrm{E_{sec}})_{1\%}$ of Achrafieh clay specimens at an axial strain of 1% at the confining pressures of 100, 150, and 200 kPa was determined to be 4994.14 kPa, 7987.28 kPa, and 10086.13 kPa, respectively. These values are larger than the values of $(\mathrm{E_{sec}})_{1\%}$ of 4150 kPa, 6092 kPa, and 7637 kPa determined for

Kaolin at the same confining pressures. The increase in the stiffness of Achrafieh clay vs. Kaolin clay is clearly shown in Figure 6.21.

The Mohr Coulomb effective stress failure parameters for both types of clay are representative of normally consolidated clay. The apparent effective cohesion (c') and the effective angle of internal friction (Ø') for the control specimen of Achrafieh and Kaolin clays were 5.0 kPa and 25.08º, and 0 kPa and 26.3º respectively.

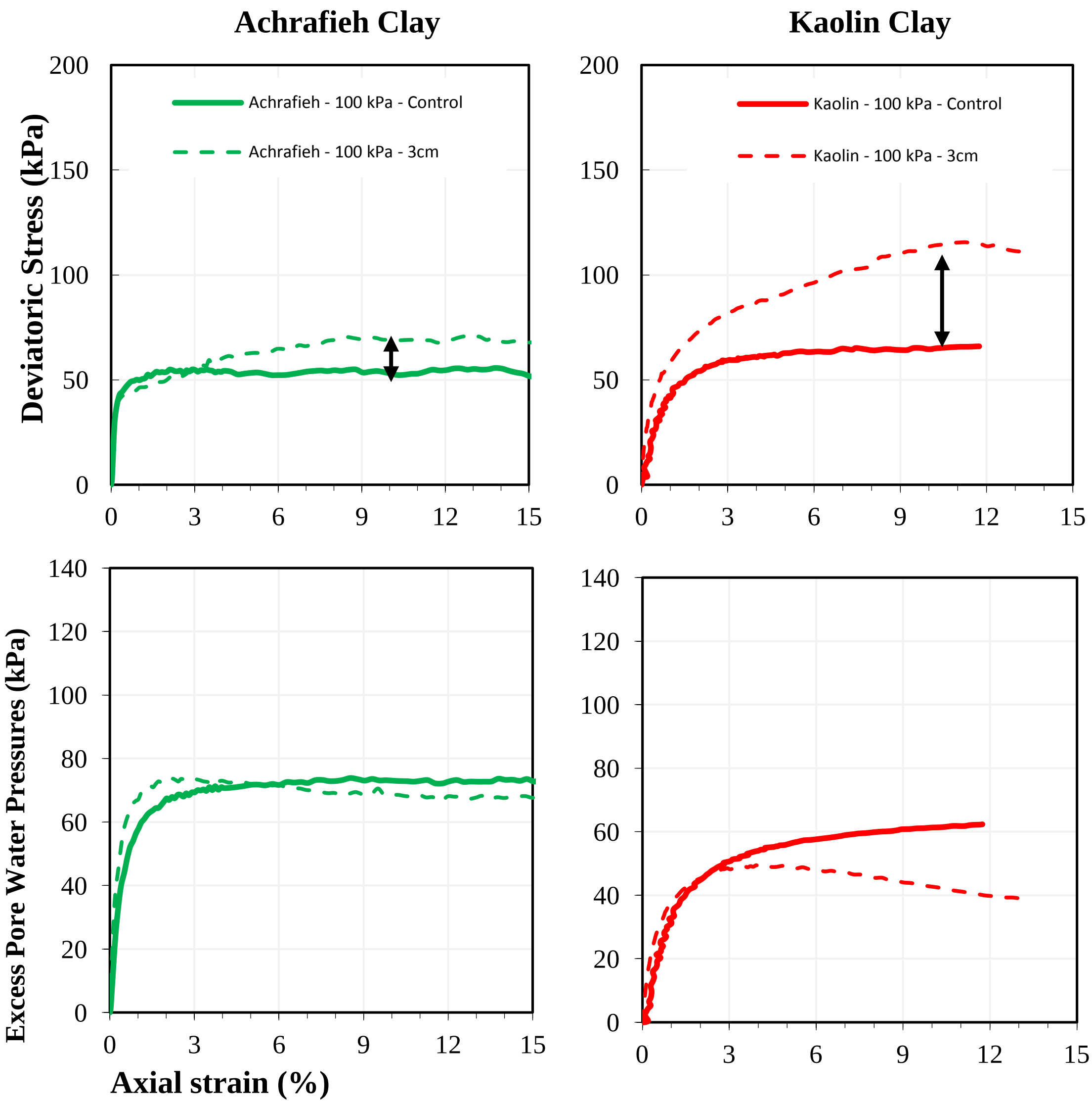
6.3.2 Comparison with Samples Reinforced with 3cm Medium Dense Sand Columns

The variation of the deviatoric stress and pore-water pressures with the axial strain for control and reinforced specimens with 3cm medium dense sand columns corresponding to an area replacement ratio of 17.8% for both types of clays, Achrafieh clay and Kaolin and under confining pressures of 100, 150 and 200kPa are presented in Figure 6.21. The results indicate that the improvement observed in the deviatoric stress at failure in the reinforced Kaolin clay is much more significant than that witnessed in the Achrafieh clay. The increase in undrained shear strength for reinforced Kaolin clay samples under confining pressures of 100, 150 and 200 kPa was 75.5, 75.5 and 67.5% respectively, versus 27.3, 22.9 and 17.6% for reinforced Achrafieh clay samples. The same was noticed in the reduction of pore-water pressures. This difference in the magnitude of the improvement in strength could be explained by the tendency for pore pressure generation that was witnessed in the two soils. For Kaolin samples, there was a reduction in pore-water pressures averaging 30% whereas for Achrafieh samples, the reduction was minimal, around 3%. The major difference between the two clays is that the Achrafieh clay has 50% fines whereas the kaolonite clay has almost 100% fines. It could be that the presence of the "sand" sized portion in the Achrafieh clay reduced the effectiveness of the medium dense sand columns in affecting the tendency for volume change in the reinforced specimens and did not allow for any noticeable reduction in the excess pore pressure. The sand-sized portion in the Achrafieh clay is expected to be in a very loose state due to the method of sample preparation that involved consolidation from a slurry. It should be noted that both types of clay experienced bulging of the clay specimen as the primary mode of failure.

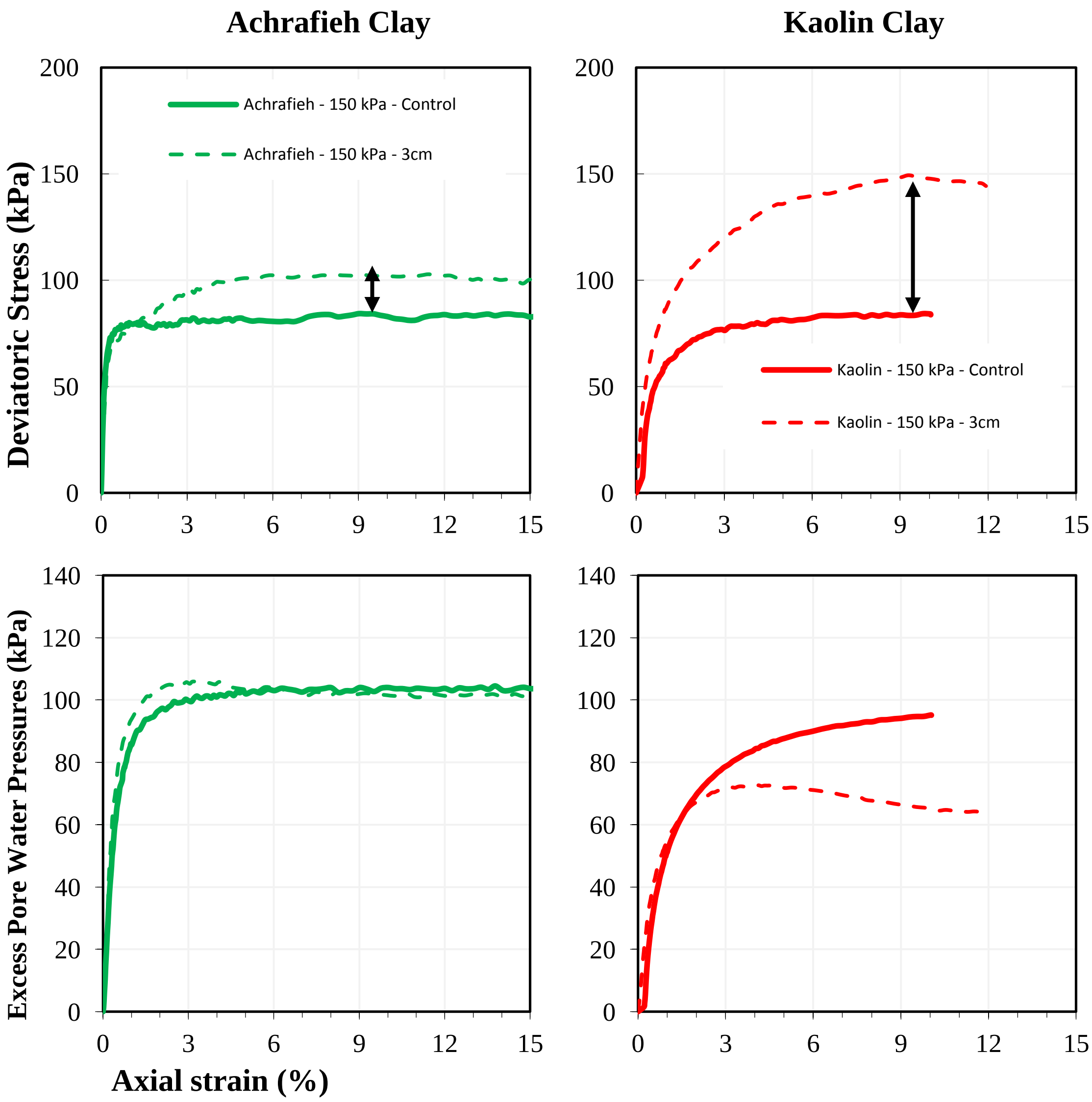
The remarkable improvements in the deviatoric stresses and pore water pressures witnessed by the reinforced Kaolin samples relative to the improvement results of the reinforced Achrafieh samples were also translated in the calculated secant modulus $(E_{sec})_{1\%}$ defined at an axial strain of 1%. The average improvements in $(\mathrm{E_{sec}})_{1\%}$ for reinforced Kaolin samples was 38.3%, compared to an average improvement in $(\mathrm{E_{sec}})_{1\%}$ of -2.86% for Achrafieh reinforced specimens under the same conditions.

As for the shear strength parameters, the insertion of sand columns increased the effective friction angle (ϕ') of the composite system with respect to the control tests performed on Achrafieh clay samples by 2.6°. Whereas, for reinforced Kaolin specimens prepared and sheared under the same conditions, there was a decrease in effective friction angle by 2.7°. However this decrease in (ϕ') was accompanied by an increase in 12 kPa in cohesion for Kaolin sample, whereas the effective apparent cohesion (c') was found to be insensitive to the presence of the sand columns in reinforced Achrafieh clay specimens.

σ'_1 = 100 kPa



$\sigma_1' = 150$ kPa



$\sigma_1' = 200$ kPa

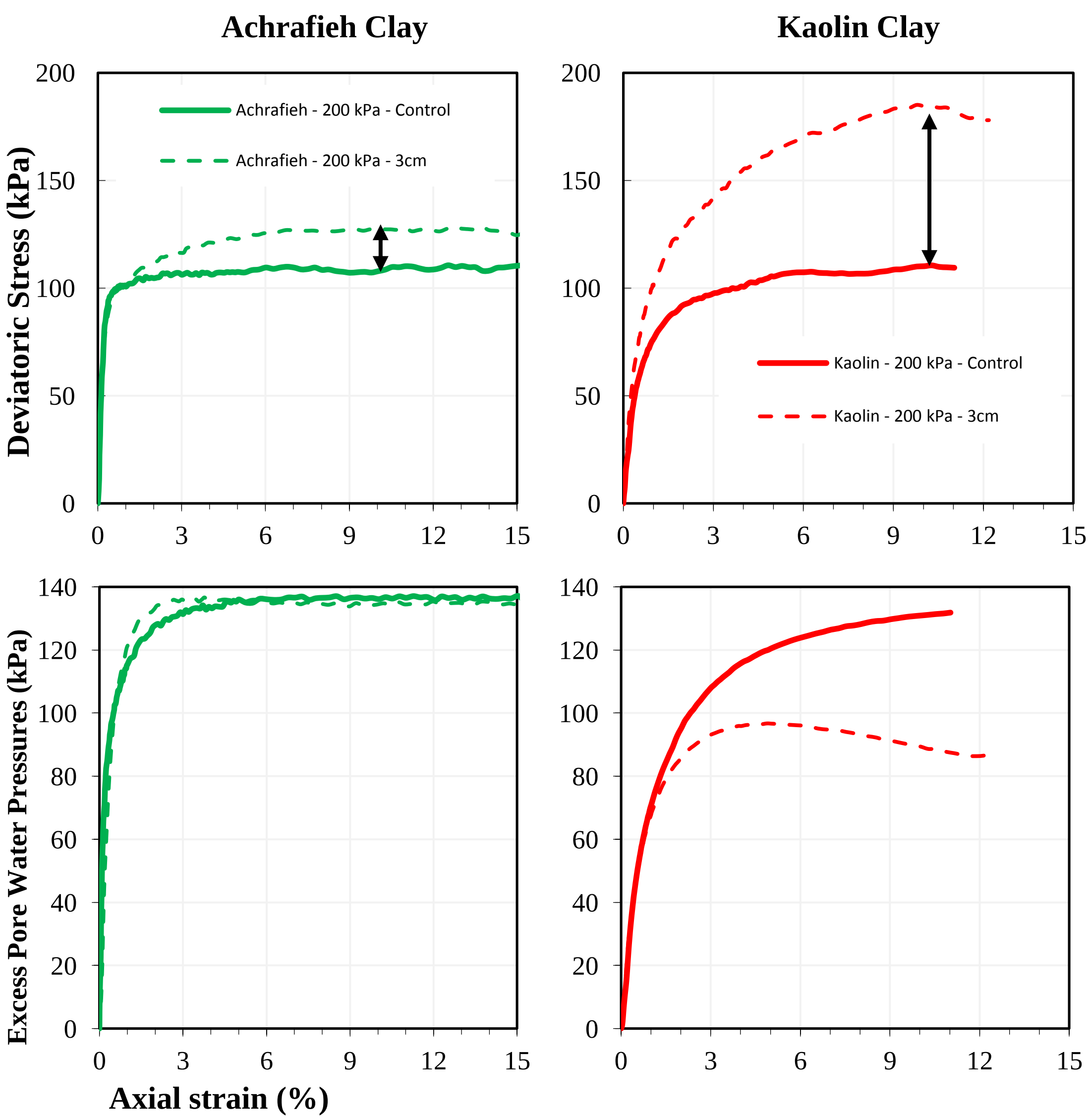


Figure 6.21: Deviatoric stress and excess pore water pressures vs. axial strain for control Achrafieh and Kaolin clay at confining presures of 100kPa, 150kPa and 200kPa

6.4 Summary of Main Findings

Based on the results of 12 consolidated undrained triaxial tests that were conducted in this experimental research on control Achrafieh clay specimens, and on specimens reinforced with 3cm medium dense, 3cm dense and 4cm dense sand columns, the following conclusions can be drawn with regards to the effect of sand columns on the undrained response of Achrafieh clay:

- For consolidated undrained control/unreinforced samples, failure was characterized mainly by uniform bulging of the clay specimen along its length with concentration at the middle portions of the sample. The bulging severity decreases with increasing confining pressure. Therefore, bulging was more evident in the samples tested at confining pressures of 100 kPa and 150 kPa, but was less severe for the higher confining pressure of 200 kPa. This behavior indicates this type of composite system does not fail by punching through the underlying clay. The same mode of failure was identified for reinforced specimens with bulging more obvious in specimens reinforced with dense sand columns.
- Results indicate that the use of 3-cm diameter medium-dense sand columns resulted in minor increase in the undrained shear strength. Whereas significant increase in undrained shear strength was observed in specimens reinforced with 4-cm diameter dense sand columns (area replacement ratio=31.7%) with the highest improvement being 193.27% at 150kPa confining pressure. The improvements ranged from 17.59% to 27.27% for specimens reinforced with 3cm medium-dense sand columns, 42.04% to 56.36% for specimens reinforced with 3cm dense sand columns and 169.07% to 193.27% for specimens reinforced with 4cm dense sand columns. In comparison with the smaller area replacement ratio of 17.8%, the 31.7% resulted in significant improvements in the undrained shear strength.

- An analysis of the results indicates that for specimens reinforced with fully penetrating medium-dense sand columns having an area replacement ratios of 17.8%, the reduction in the excess pore-water pressure at confining pressures of 100, 150 and 200 kPa was 5.32%, 3.01% and 1.17% respectively. These results show that minor reductions in excess pore-water pressures were noticed. The same was observed for the specimens having the same area replacement ratio of 17.8% but reinforced with dense sand columns. Minor reductions in excess pore-water pressures were observed, where the reduction in excess pore-water pressures was 4.77%, 0.33% and 0% at confining pressures of 100, 150 and 200 kPa respectively. Hence, it was noticed that the insertion of sand columns with area replacement ratios of 17.8% does not reduce the excess pore-water pressures generated during undrained loading. In addition, the effectiveness in reducing the pore-water pressures decreases with increasing the effective confining pressures. As for the specimens that were reinforced with fully penetrating dense sand columns resembling an area replacement ratio of 31.7%, a significant reduction in excess pore-water pressures was observed, with the effectiveness also decreasing with increasing the confining pressures. The reduction in excess pore-water pressures for confining pressures of 100, 150 and 200 kPa were 55.41%, 46.63% and 41.48% respectively.
- The insertion of fully penetrating medium-dense 3cm sand columns slightly decreased the stiffness of the unreinforced Achrafieh clay by an average of ≈2.86%. As for the specimens reinforced with 3cm dense sand columns, there was an average increase in $(E_{sec})_{1\%}$ of 30%. The most significant increase observed was for specimens reinforced with 4cm dense sand columns there was

an average increase in $(E_{sec})_{1\%}$ of ≈90%. This indicates that for specimens reinforced with 4cm dense sand columns, more stresses will be distributed along the column length, and consequently less settlement will result.

- The insertion of sand columns significantly increased the drained angle of internal friction (ϕ') of the composite system in respect to the control tests performed on Achrafieh clay samples. This increase is most noticeable for samples reinforced with dense sand columns, irrespective of the area replacement ratios, where increase was up to 8.5°, compared to 2.7° for the specimens that were reinforced with medium-dense sand columns.

CHAPTER 7

TEST RESULTS AND ANALYSIS FOR PARTIALLY DRAINED TESTS

7.1 Introduction

In addition to the consolidated drained and undrained triaxial tests that were performed on Achrafieh clay specimens reinforced with sand columns of various area replacement ratios, a series of tests were performed to study the effect partial drainage on the response of the composite. The objective is to mimic field conditions where radial drainage through the granular column is expected to occur during loading. For typical field rates of load application, it is expected that partial drainage will occur from the clay to the sand columns making the response more of a partially drained response rather than a fully drained or fully undrained response.

To achieve this objective, a series of consolidated partially drained triaxial tests (PD tests) were performed using the same automated triaxial test setup "TruePath" by Geotac. Specimens saturated under a back pressure of 310 kPa, were isotropically consolidated under a confining pressure of 100 kPa. The samples were sheared at different rates of loading (shearing rates of 60%, 20%, 10%, 5%, 3% and 1% strain per hour) while allowing drainage from the drainage lines that are connected to the upper porous stone. Drainage was prohibited from the lower part of the sample to try to enforce a drainage pattern that is mostly radial. The shortest path for drainage of the clay is radial through the sand column, except for the clay at the upper part of the sample where both radial drainage and upward drainage could occur. During the tests, volume change was measured through the drain lines connected to the porous stones at the top of the sample. The measured volumetric strains reflect a global change in the composite sample and do not provide information on local changes in the water content in the sand column and the surrounding clay. Throughout the tests, the total confining pressure was kept constant as the vertical stress was increased in the compression phase.

The partially drained testing program involves specimens reinforced with 3cm sand columns representing an area to replacement ratio of 17.8%, and specimens reinforced with 4cm sand columns corresponding to an area replacement ratio of 31.7%. Shearing rates were varied in order to fully understand the effect of varying shearing rates on the response of the partially drained composite with respect to undrained and drained behavior. For specimens reinforced with 3cm sand columns, partially drained triaxial tests with shearing rates of 60%, 20%, 10%, 5%, 3% and 1% strain per hour were performed. As for specimens reinforced with 4cm sand columns, shearing rates of 60%, 20%, 10%, 5%, and 1% strain per hour were performed. The results include a description of the modes of failure that characterize the behavior of the different test specimens and a detailed analysis of the parameters which are known to affect the load response of reinforced clay specimens.

The test results were also analyzed to establish a relationship between the degree of consolidation that was observed during any given partially drained test and the measured load-carrying capacity. The degree of consolidation was estimated using a finite difference scheme that takes into consideration the rate of load application and assumes radial drainage from the surrounding clay to the columns in the prediction of the rate and extent of dissipation of pore water pressure in the sample. The explicit finite difference solution calculates the pore water pressures at different times, $\mathrm{t} + \Delta\mathrm{t}$, during the shearing stage, and at different distances from the clay, $r + \Delta\mathrm{r}$, for each node sequentially. The average degree of consolidation at different times corresponding to different strains can then be easily computed.

7.2 Test Results

The test results are presented in the form of deviatoric stresses versus axial strain curves, and volumetric strain versus axial strain curves. Failure was defined at the ultimate axial strain achieved during the test.

7.2.1 Achrafieh Specimens Reinforced with Sand Columns

Results obtained from the drained, undrained and partially drained triaxial tests conducted on Achrafieh clay specimens reinforced with fully penetrating 3-cm and 4-cm sand columns, corresponding to area replacement ratios of 17.8% and 31.7% respectively, are presented in Table 7.1 and in Figure 7.1 to Figure 7.4. These figures also include pictures of the modes of failure and graphs showing the variation of the deviatoric stress and volumetric strains with axial strains. The results were analyzed to investigate the effect of drainage conditions on the shear strength and the degree of volumetric strain.

7.2.1.1 ***Modes of Failure***

The mode of failure for specimens reinforced with 3cm dense sand columns that were sheared at the relatively lower strain rates of 1%, 3% and 5%, i.e. at strain rates closer to that of the traditional drained test, was characterized by uniform bulging of the clay specimen and of the sand column (Figure 7.1). This bulging is nearly identical to that observed for fully drained conditions. For samples sheared at higher strain rates, bulging was not uniform across the specimen, with the concentration of the bulging being towards the middle of the sample. This was most significant for the test sheared at the fastest strain rate/hour of 60% strain/hour where the sand column suffered bulging mainly around the middle of the column (Figure 7.1). To investigate the mode of failure of the sand columns, the same test specimens were split along their vertical axes to expose the columns and the surrounding clay. The sand columns showed slight bulging along the same location as the bulging of the clay. Clay specimens that were reinforced with 4cm sand columns (area replacement ratio of 31.7%) also exhibited failure modes that were characterized by excessive bulging that was generally concentrated in the middle of the sample, irrespective of the shearing rate used (Figure 7.2). Samples sheared at higher strain rates exhibited more concentrated and more significant bulging compared to samples sheared at lower strain rates. For the sample sheared at the highest strain rate of 60% strain per hour, results on Figure 7.2 indicate the formation of a shear plane that passed almost laterally in the middle of the sample. The shear plane was evident in both the overall sample and in the internal sand column.

7.2.1.2 ***Stress-strain Behavior***

The variations of the deviatoric stresses and volumetric strains with the axial strains are presented in Figure 7.3 and Figure 7.4 for specimens reinforced with 3cm and 4cm sand columns under a confining pressure of 100 kPa and sheared under various shearing rates. The stress-strain curves of the fully drained and undrained tests that were conducted on control specimens and specimens that were reinforced with 3-cm and 4cm columns are also shown on the same figures for comparison.

For the specimens reinforced with 3-cm diameter columns, results on Figure

7.3 indicate that the stress-strain curves of the partially drained tests are bounded on the low side by the stress-strain curve of the fully undrained test and on the high side by the stress-strain curve of the fully drained test conducted on the reinforced specimens. The stress-strain curves of the partially drained tests gradually move upward towards the fully drained test as the strain rate is decreased from 60% per hour (fastest partially drained test) to 1% per hour (slowest partially drained test). It is interesting to note that even for the fastest test conducted (60% per hour), the stress strain curve was found to be slightly higher than the fully undrained test, indicating that partial drainage must have occurred in the specimen even at this fast rate of loading. These results, which are consistent with the findings of Andreou et al (2008) and Bou Lattouf (2013), could be attributed to the radial drainage that was allowed from the clay to the sand column and up through the upper porous stone in the partially drained tests. Partially drained tests conducted using shearing rates that are greater than 3% axial strain per hour resulted in deviatoric stresses at failure that were smaller than that of the fully drained test, indicating that although partial drainage in these tests occurred, the rate of shear was fast enough to prohibit the dissipation of all the pore pressure generated during shearing.

It should be noted that the stress-strain response of samples that were sheared under partially drained conditions with the fastest strain rates of 20% and 60% per hour showed an early peak in the stress-strain response with a slight strain softening behavior. All other tests (shear rates less than 10% per hour) showed a strain hardening behavior. This could be explained by the fact that at high shearing rates, the low permeability clay surrounding the columns does not have enough time to dissipate the excess pore pressures and is expected to exhibit a more-or-less undrained behavior, whereas the high-permeability sand column is expected to exhibit full drainage. Since the stress-strain response of the sand under fully drained conditions exhibits a peak with strain softening, this is reflected in the composite samples sheared at the highest shearing rates. For the relatively lower shear rates, the tendency for strain softening of the sand column is overcome by the response of the clay surrounding the column which is expected to drain at the relatively slower shear rates. Since the clay has been shown to strain harden under drained conditions, the overall response of the composite for tests sheared at relatively slower shearing rates is to exhibit some strain hardening.

A final note regarding the partially drained tests conducted on samples reinforced with 3-cm columns is that the range in the deviatoric stresses between the fully drained and fully undrained tests is relatively wide. The undrained deviatoric stress at failure is about 86 kPa while the drained deviatoric stress at failure is 210 kPa. This allowed for a significant spread in the data obtained from the partially drained tests which were bounded by the fully drained and undrained tests.

For the specimens reinforced with 4-cm diameter columns, results on Figure 7.4 indicate that the stress-strain curves of the partially drained tests are bounded on the low side by the stress-strain curve of the fully undrained test and on the high side by the stress-strain curve of the fully drained test conducted on the reinforced specimens. However, the tests conducted at the slower strain rates of 1% and 5% resulted in deviatoric stresses at failure that are 1% to 2% higher than that of the fully drained test. These increases in deviatoric stresses at failure in the partially drained tests relative to the fully drained tests will be considered negligible and are attributed to uncertainties in the sample preparation and testing procedures. As a result, the stress-strain response of the fully drained test will still be considered as a practical upper bound to the responses of the partially drained tests.

The main differences in the results of the tests conducted on the samples reinforced with 3-cm columns and 4-cm columns are the following: (1) the range between the stress –strain curves of the fully undrained and fully drained tests is much narrower in the specimens reinforced with 4-cm columns compared to the specimens reinforced with 3-cm column. As a result, the stress-strain curves of the partially drained tests conducted on samples reinforced with 4-cm columns fell in a relatively narrow range, (2) almost all of the tests conducted with 4-cm columns showed stress-strain responses that exhibited a relatively early peak and a more-or-less strain softening response. This is attributed to the fact that the strain softening in the response of the 4-cm sand column dominated the behavior of the composite and concealed any strain hardening that would have been expected as a result of the contribution of the clay to the composite behavior, particularly for samples tests at the slower shearing rates where the clay next to the column is expected to exhibit significant partial drainage, and (3) for specimens reinforced with 3-cm columns, partially drained tests indicated that the stress-strain response started deviating from the fully drained behavior at a strain rate that is as low as 3% per hour, compared to a strain rate of 10% per hour for specimens reinforced with 4cm columns. This is attributed to the faster drainage rate that is expected to exist in specimens reinforced with 4cm columns due to the shorter radial drainage path.

I.D.	Confining Pressure σ_3 (kPa)	Diameter of Specimen (cm)	A_c/A_s (%)	Height of Sand Column (cm)	Column Height Penetration Ratio, (H_c/H_s)	Rate of loading (%strain/hr)	Drained/ Partially drained/ Undrained	Deviatoric Stress at Failure (kPa)	Volumetric Stain at Failure (%)	Reduction in Volumetric Strain (%)
1	100	7.10	0.00	0.00	-	0.25	D	211.26	4.28	-
2		7.10	0.00	0.00	-	1	U	55.71	0.00	-
3		7.10	17.85	14.20	1.00	0.375	D	210.00	4.06	-
4		7.10	17.85	14.20	1.00	0.75	U	86.18	0.00	-
5		7.10	31.74	14.20	1.00	0.375	D	212.14	1.39	-
6		7.10	31.74	14.20	1.00	1	U	162.06	0.00	-
7		7.10	17.85	14.20	1.00	1.00	PD	209.18	3.86	4.93
8		7.10	17.85	14.20	1.00	3	PD	198.70	3.68	9.36
9		7.10	17.85	14.20	1.00	5.000	PD	177.63	3.01	25.86
10		7.10	17.85	14.20	1.00	10.00	PD	165.30	2.65	34.85
11		7.10	17.85	14.20	1.00	20.000	PD	121.70	1.52	62.56
12		7.10	17.85	14.20	1.00	60	PD	110.82	0.41	90.00
13		7.10	31.74	14.20	1.00	1	PD	217.00	1.64	-18.16
14		7.10	31.74	14.20	1.00	5	PD	214.99	1.45	-4.47
15		7.10	31.74	14.20	1.00	10	PD	194.90	0.88	36.74
16		7.10	31.74	14.20	1.00	20	PD	180.20	0.41	70.61
17		7.10	31.74	14.20	1.00	60	PD	176.75	-0.72	152.09

Table 7.1: Test Results for Partially Drained Achrafieh clay

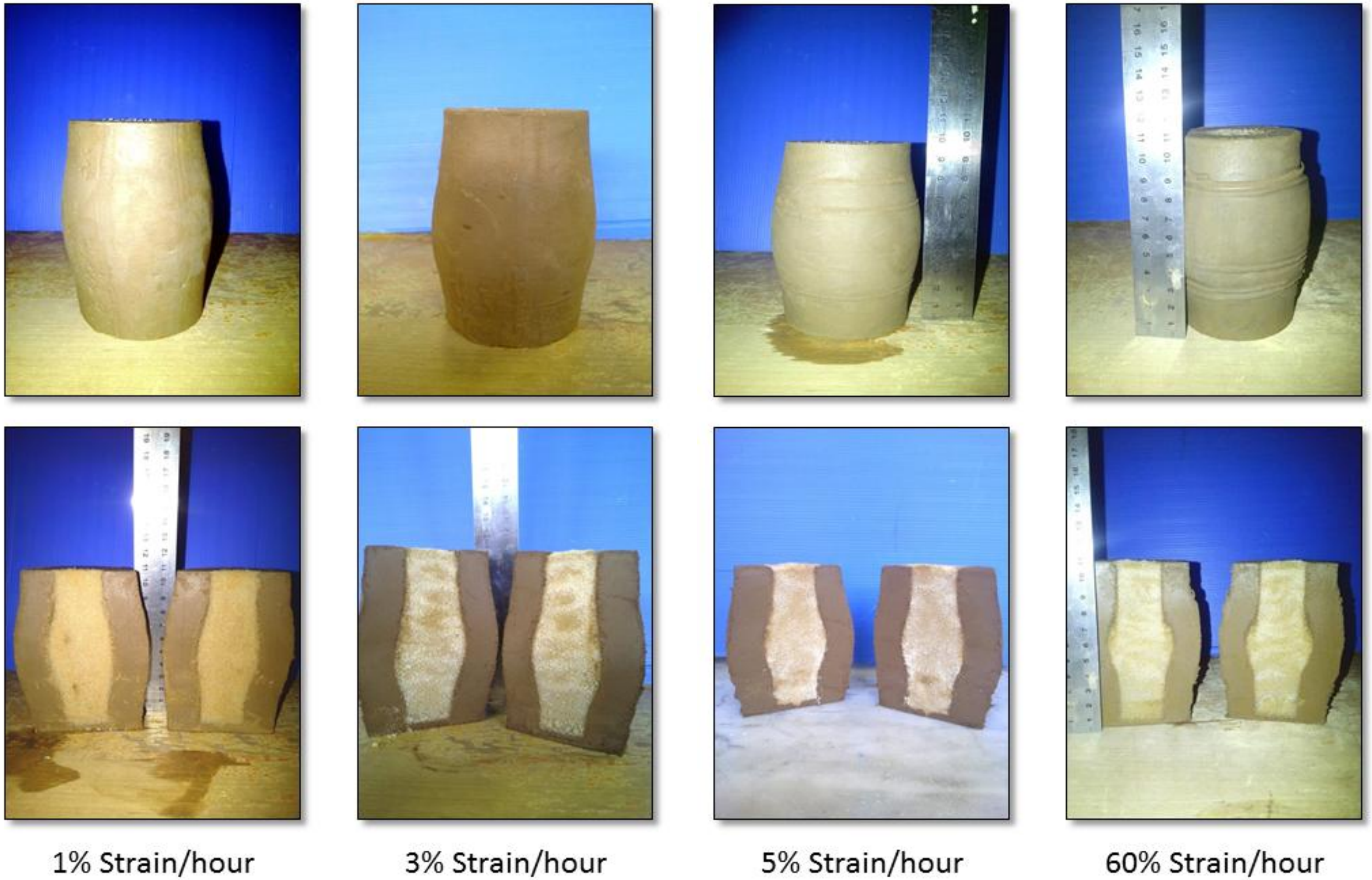


Figure 7.1: External and internal modes of failure of test specimens (Ac/As = 17.8%, σ_3 = 100 kPa)

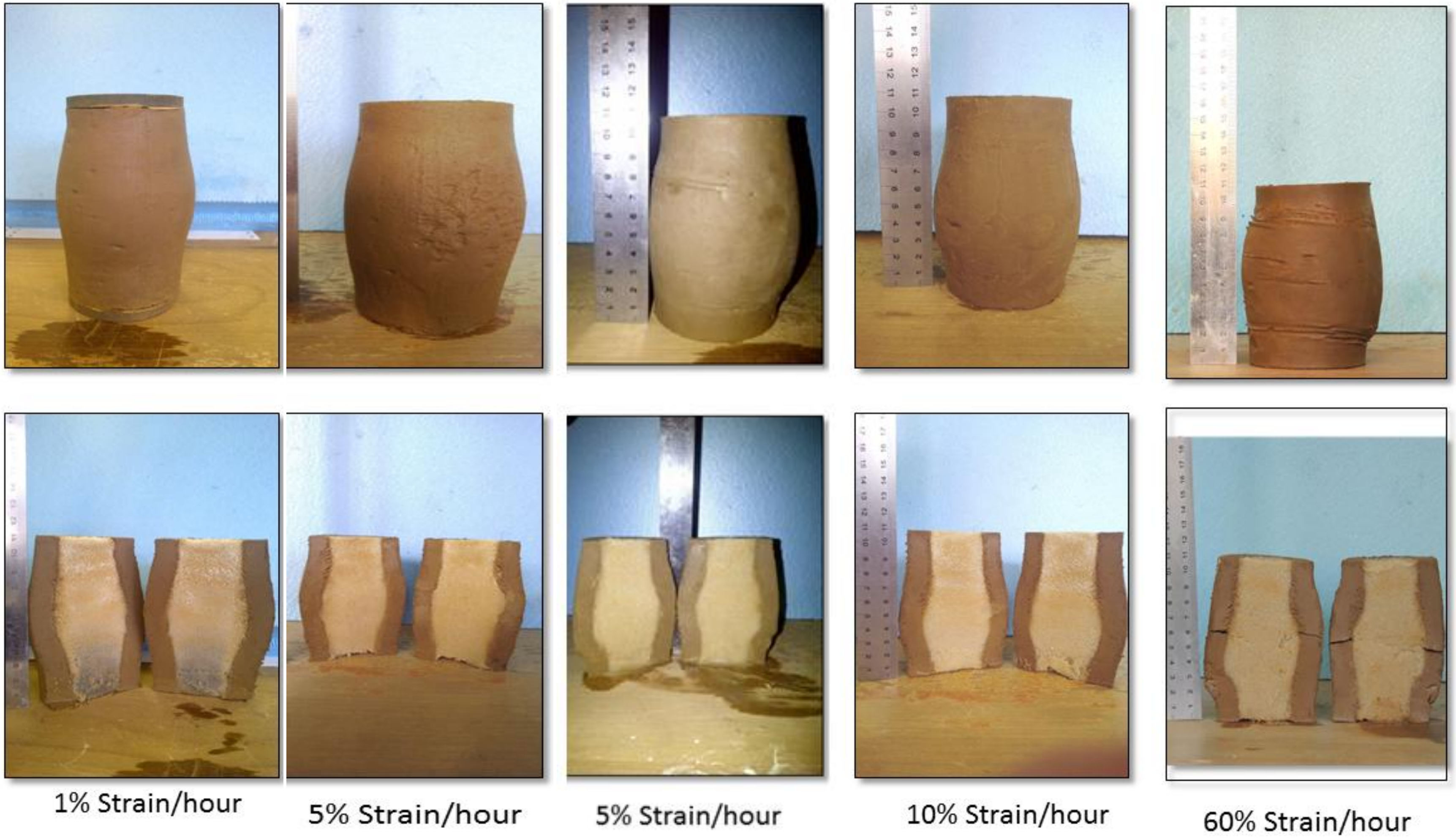


Figure 7.2: External and internal modes of failure of test specimens (Ac/As = 31.7%, σ_3 = 100 kPa)

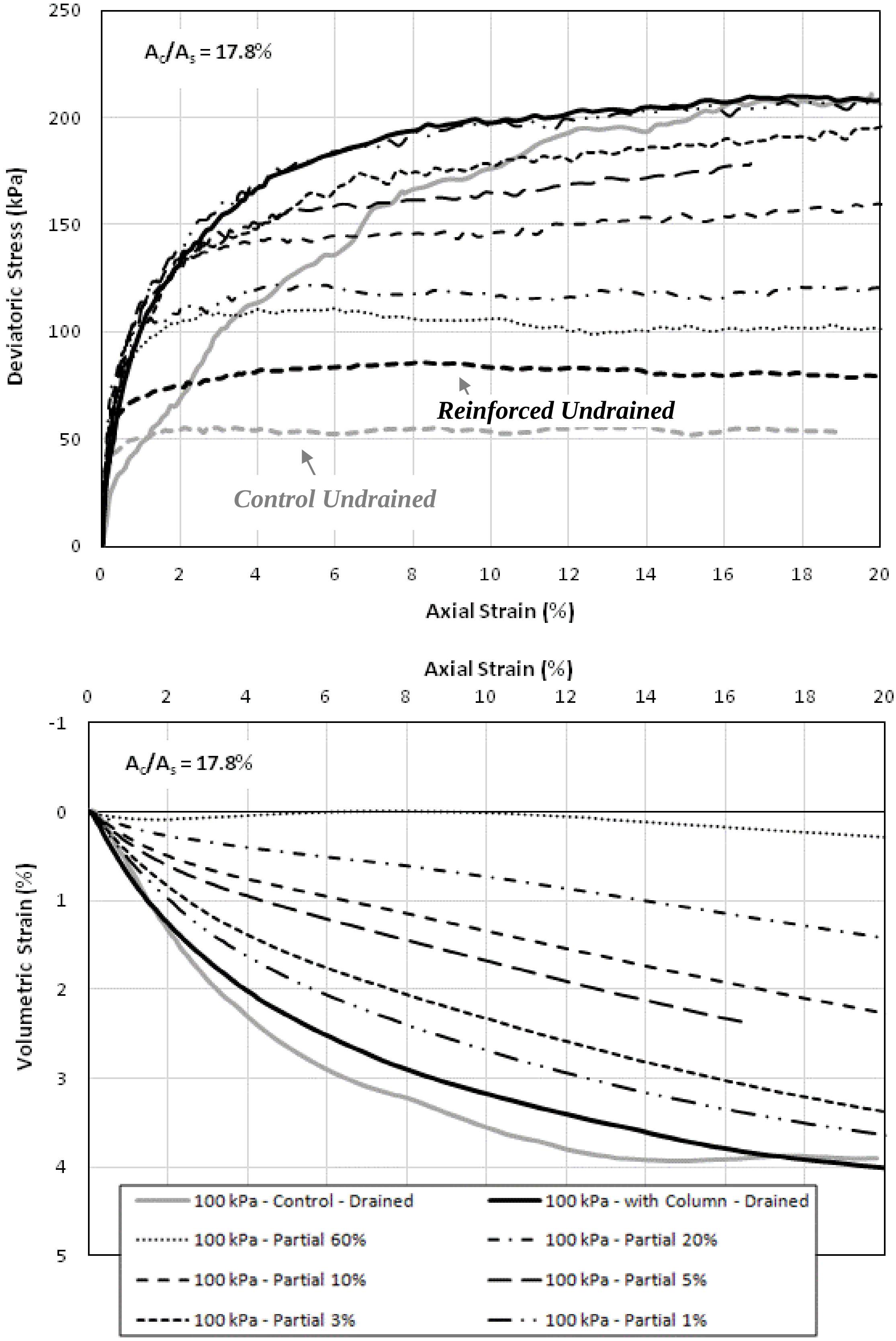


Figure 7.3: Deviatoric stress and volumetric strains vs. Axial Strains for Partially Drained tests of specimens having an area replacement ratio of 17.8%

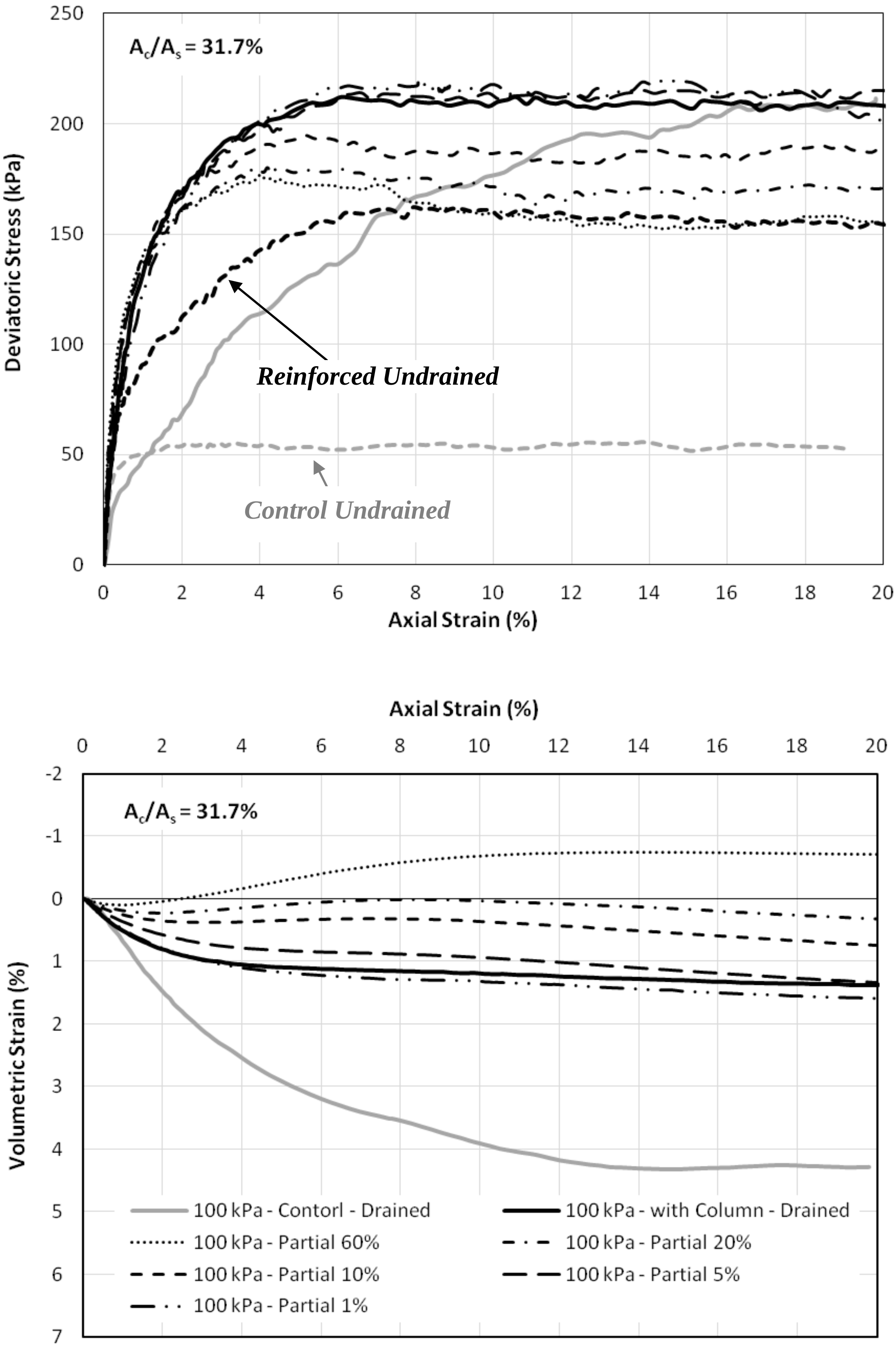


Figure 7.4: Deviatoric stress and volumetric strains vs. Axial Strains for Partially Drained tests of specimens having an area replacement ratio of 31.7%

7.2.2 *Effect of Strain Rate and Drainage Conditions on Volume Change*

For specimens reinforced with 3cm sand columns, the measured volumetric strains for all partially drained tests were contractive (Table 7.1). The negative volumetric strains were reduced significantly with increasing shearing rates in the partially drained tests. For example, the compressive volumetric strain for the reinforced samples decreased from around 4% for the fully drained test to about 0.4% for the partially drained test in which the specimen was sheared at a rate of 60% axial strain per hour. The observed reductions in compressive strains are mainly attributed to the reduction in the compressive volumetric strains in the clay surrounding the column. As the strain rate is increased, the degree of consolidation of the clay surrounding the column is expected to decrease since not enough time is allowed in fast tests for pore pressures in the clay to fully dissipate during the shearing stage. It should be noted that the reported volumetric strains reflect the overall volume change in the sample. A measured volumetric strain that approaches zero does not indicate that the degree of consolidation in the clay is zero. It indicates that enough compressive volumetric strains have occurred in the clay around the column to balance the dilative volumetric strains that will inevitably occur in the freely draining column.

For specimens that were reinforced with 4cm sand columns, volumetric strains for all partially drained and drained tests were contractive except for the partially drained test that was performed at the fastest strain rate of 60% strain per hour. Due to the fast shearing rate and the relatively large sand column diameter, the dilative volume change of the sand column in this very fast test was larger than the compressive volumetric strains that have occurred in the clay surrounding the column.

For illustration, the volumetric strains for the fully drained tests on specimens that were reinforced with 4-cm sand columns were in the order of 1.4%. This overall compressive volumetric strain decreased in the specimens that were sheared at strains rates that exceeded 10% per hour and switched to an overall dilative volumetric strain of about 0.7% for the specimen tested at a strain rate of 60% per hour.

7.2.3 Effect of Partial Drainage on the Measured Strength at Failure

In order to investigate the effect of partial drainage on the load carrying capacity of the reinforced clay specimens in a more general sense, the time to failure ($t_{failure}$) of each test specimen was normalized by (t_{50}), which is defined as the time required for 50% consolidation to occur in the consolidation stage of that test. This ratio of $t_{failure}$ to t_{50} is typically used to determine the shearing rates required for typical consolidated drained tests ($t_{failure}/t_{50}$ is taken at least as 80) and for pore pressures to stabilize in typical consolidated undrained tests (generally $t_{failure}/t_{50}$ is taken at 10). For all partially drained tests conducted in this research, values of t_{50}were determined from the volume change versus logarithm of time relationship obtained from the consolidation stage of that test. Values of (t_{50}) are summarized in Table 7.2. On the other hand, the time to failure $t_{failure}$ should theoretically correspond to the maximum deviatoric stress measured in the triaxial test. In tests that exhibit strain hardening, the time to failure would correspond to the time required for the maximum axial strain to be applied (20% in this research), while in tests that exhibit strain softening, the time to failure should theoretically correspond to the time required for applying the level of strain at which the peak deviatoric stress is observed.

From a practical perspective, one complicating factor in adopting the above theoretical approach in defining the time to failure is the fact that identical specimens that are sheared at different strain rates could exhibit a gradual shift from a strain hardening behavior to a strain softening. For instance, this is witnessed in the specimens that were reinforced with 3-cm columns and sheared at strain rates of 1% and 3% per hour. The stress strain curves of these tests indicate that the test sheared at 1% strain per hour exhibited very minor strain softening, while the sample sheared at a rate of 3% per hour showed very minor strain hardening. If the above theoretical approach for defining $t_{failure}$ is adopted, the specimen sheared at 1% per hour would have a $t_{failure}$ of about 6 hours, while the sample sheared at a rate that is 3 times faster (3% per hour) would have a $t_{failure}$ of 6.67 hours, which does not make sense from a practical standpoint given the similarities in the stress strain responses of the two tests, and the much faster rate used in shearing the sample in the 3% per hour test.

For practical and comparative purposes, a decision was made to define the time to failure in all tests as the time corresponding to the maximum strain reached in these tests (axial strain of 20%). This assumption simplifies the analysis of the tests and helps in clarifying the role of the rate of loading, without the complicating factors that are related to the shape of the stress-strain response. Based on this assumption, all times to failure $t_{failure}$ were calculated and presented in Table 7.2. These calculated values together with the t_{50} values were used to calculate a ratio of $t_{failure}/t_{50}$ for all partially drained and fully drained tests (Table 7.2).

The next step involved investigating the presence of any relationship between $t_{failure}/t_{50}$ and the maximum deviatoric stresses measured in these tests. This relationship is explored in Figure 7.5 which shows the variation of the deviatoric stresses at failure with the ratio of $t_{failure}/t_{50}$ for all partially drained and fully drained tests conducted on specimens reinforced with 3-cm and 4-cm columns. Also plotted on the figure are the deviatoric stresses corresponding to the fully undrained tests on the reinforced specimens and which were shown to act as a practical lower-bound to the strength of the composite. Results on Figure 7.5 indicate that the deviatoric stresses increased systematically as the ratio of $t_{failure}/t_{50}$ increased. These increases seem to level off at an approximate value of $t_{failure}/t_{50}$ of about 10, were the measured deviatoric stresses in the partially drained tests seem to approach the deviatoric stress of the fully drained test, which has been shown to act as a practical upper bound to the strength of the composite.

To generalize the results further, and since the analysis in the previous sections showed that the stress-strain curves for partially drained tests are bracketed by the curves of the fully drained and undrained tests, a normalized strength improvement index is defined as the ratio of $(\sigma_{d,PD} - \sigma_{d,U})\ to\ (\sigma_{d,D} - \sigma_{d,U})$, where $\sigma_{d,PD}$ is the deviatoric stress measured for the partially drained tests, $\sigma_{d,U}$ is the deviatoric stress of the undrained tests (assumed to be a lower-bound strength for the composite) and $\sigma_{d,D}$ is the deviatoric stress of the drained tests (assumed to be an upper-bound strength for the composite). This strength improvement index is analogous to the liquidity index for the soils in the sense that it provides a relative measure of the magnitude of strength that could be mobilized in a partially drained test in relation to the minimum and maximum strengths that could be obtained assuming undrained and drained conditions, respectively.

The strength improvement index $(\sigma_{d,PD} - \sigma_{d,U})$ to $(\sigma_{d,D} - \sigma_{d,U})$ is plotted on Figure 7.6 against $t_{failure}/t_{50}$ for all tests. The results on Figure 7.6 indicate that the normalized stress improvement index that was calculated for all the partially drained tests is strongly correlated to the ratio of $\mathrm{t_{failure}/t_{50}}$. The relationship indicates a strength improvement index of about 30% at a very low value of $t_{failure}/t_{50}$ of about 0.8, 60% to 70% at a relatively low value of $t_{failure}/t_{50}$ of about 4.0, with increasing values of strength improvement index of about 90% for a $t_{failure}/t_{50}$ value of about 8.0. As mentioned earlier, a $t_{failure}/t_{50}$ ratio of about 10 seems to be a practical ratio at which the drained strength is mobilized.

I.D.	Confining Pressure σ_3 (kPa)	Diameter of Sand Column (cm)	A_c/A_s (%)	Rate of loading (%strain/hr)	Drained/ Partially drained/ Undrained	Deviatoric Stress at Failure (kPa)	Drained Shear Strength at Failure (kPa)	Volumetric Stain at Failure (%)	Strength Improvement Index (σd - σU) / (σD-σU)	(Volumetric Strain-PD) / (Volumetric Strain-D)	Degree of Consolidation from Finite Difference	Degree of Consolidation from Finite Difference (%)	Reduction in Volumetric Strain (%)	t_{50} from Consolidation (min)	Time to Failure (min)	Time to Failure / t_{50}
1		0	0.00	0.25	D	211.26	105.63	4.28	-	-	-	-	-	-	-	-
2		0	0.00	1	U	55.71	27.86	0.00	-	-	-	-	-	-	-	-
3		3	17.85	0.375	D	210.00	105.00	4.06	1.0	1.00	-	-	-	-	-	-
4		3	17.85	0.75	U	86.18	43.09	0.00	0.0	0.00	-	-	-	-	-	-
5		4	31.74	0.375	D	212.14	106.07	1.39	1.0	1.00	-	-	-	-	-	-
6		4	31.74	1	U	162.06	81.03	0.00	0.0	0.00	-	-	-	-	-	-
7		3	17.85	1	PD	209.18	104.59	3.86	0.99	0.95	0.87	86.60	4.93	51.00	1200	23.53
8		3	17.85	3	PD	198.70	99.35	3.68	0.91	0.91	0.73	73.30	9.36	51.00	400	7.84
9	100	3	17.85	5	PD	177.63	88.82	3.01	0.74	0.74	0.52	52.30	25.86	60.00	240	4.00
10		3	17.85	10	PD	165.30	82.65	2.65	0.64	0.65	0.38	37.50	34.85	52.00	120	2.31
11		3	17.85	20	PD	121.70	60.85	1.52	0.29	0.37	0.25	25.00	62.56	75.00	60	0.80
12		3	17.85	60	PD	110.82	55.41	0.41	0.20	0.10	0.14	13.80	90.00	50.00	20	0.40
13		4	31.74	1	PD	217.00	108.50	1.64	1.10	1.18	0.98	98.00	-18.16	30.00	1200	40.00
14		4	31.74	5	PD	214.99	107.50	1.45	1.06	1.04	0.82	81.50	-4.47	23.00	240	10.43
15		4	31.74	10	PD	194.90	97.45	0.88	0.66	0.63	0.59	59.30	36.74	25.00	120	4.80
16		4	31.74	20	PD	180.20	90.10	0.41	0.36	0.29	0.40	40.20	70.61	23.00	60	2.61
17		4	31.74	60	PD	176.75	88.38	-0.72	0.29	-0.52	0.21	20.80	152.09	25.00	20	0.80

Table 7.2: Test Results and parameters for Partially Drained Achrafieh clay specimens and analysis of drainage conditions

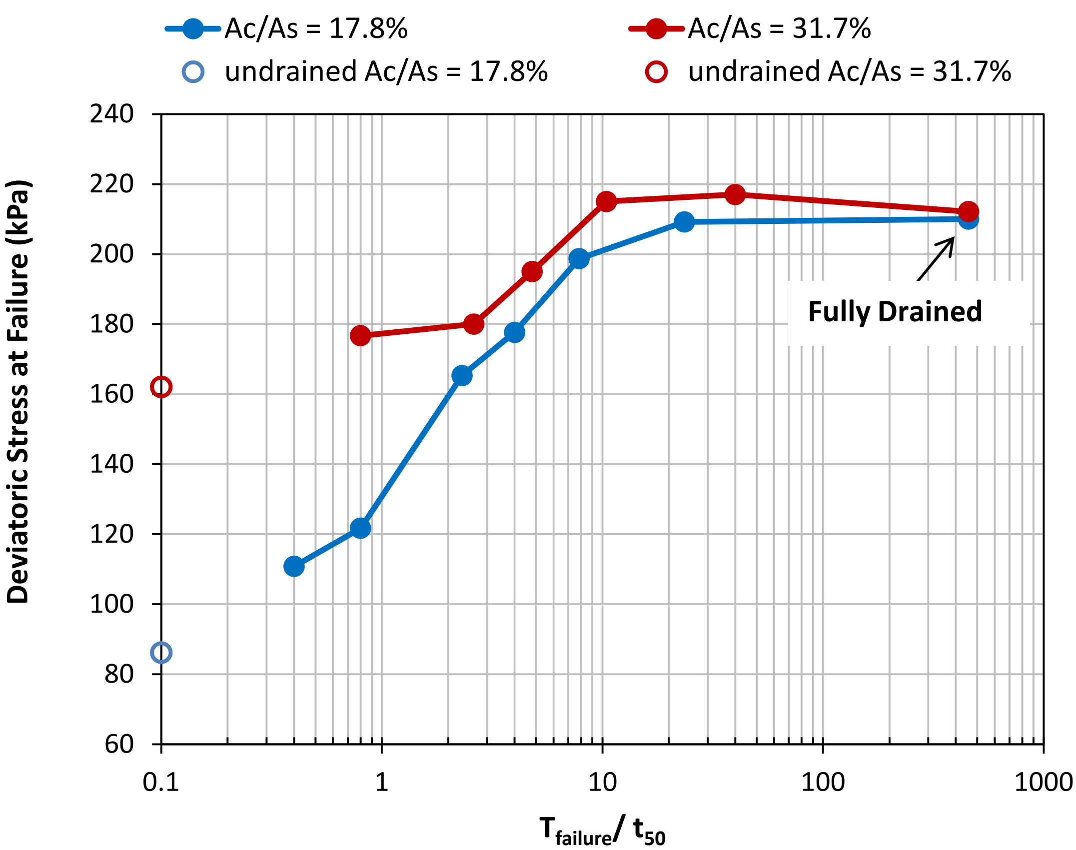


Figure 7.5: Variation of Deviatoric Stress at Failure with $t_{failure}/t_{50}$

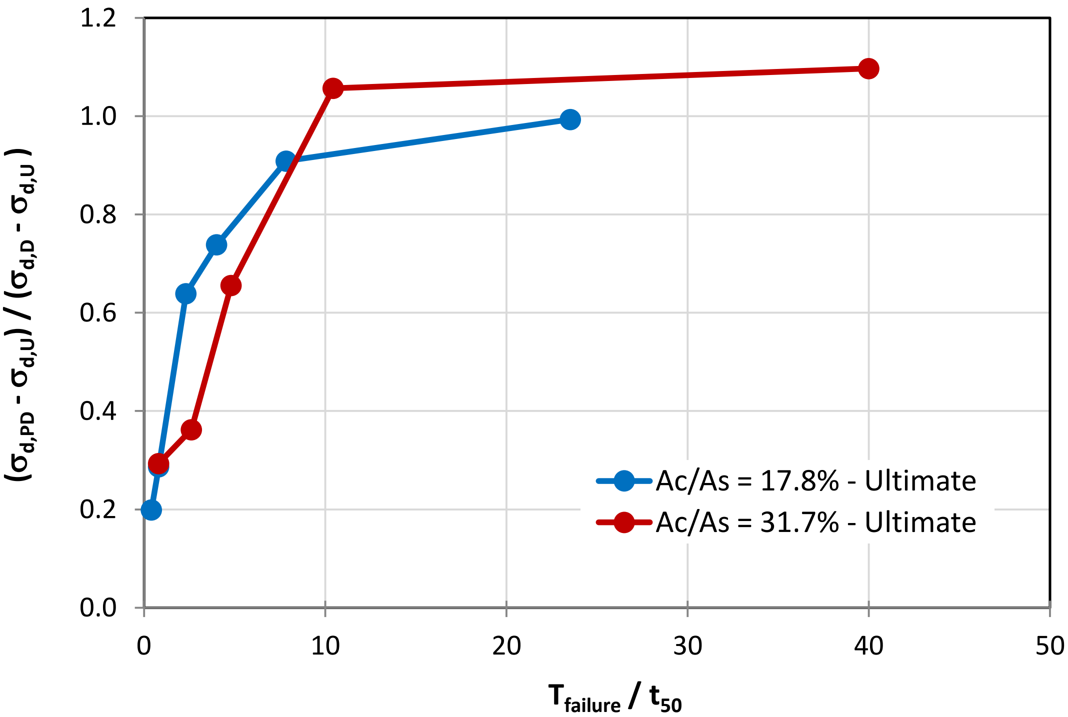


Figure 7.6: Strength improvement index vs. $t_{failure}/t_{50}$

7.3 Relation between the calculated degree of consolidation based on radial drainage and the mobilization of partially drained strength

Although the ratio of $t_{failure}/t_{50}$ seems to be a practical indicator that is correlated to the strength improvement index defined in the previous section, it is expected that the difference in the stress-strain responses for samples sheared at different shearing rates is attributed to the degree of consolidation (U_f) that has occurred in the clay surrounding the column during the test. The degree of consolidation is dictated by the rate and degree of dissipation of pore pressures which is primarily related to the radial drainage that is occurring from the clay to the sand column with minor vertical drainage occurring at the top of the sample through the upper porous stone. The degree of consolidation (U_f) that the sample undergoes at failure in a partially drained test is expected to increase as the shearing rate decreases or as the ratio of $t_{failure}\ to\ t_{50}$ increases.

A realistic estimation of the degree of consolidation (U_f) requires a solution for the coupled radial and vertical consolidation problem which in turn requires a solution for the process of generation and dissipation of pore pressures throughout the test. In this research, a finite difference solution which assumes only radial drainage was used to estimate the degree of consolidation due to the more-or-less time dependent loading that is occurring during the shearing stage. The explicit finite difference solution to the pore pressure dissipation problem is presented in equation 7.1 such that:

$$\boldsymbol{u_{k,j+1}} = \boldsymbol{u_{k-1,}j}(\alpha)\left(\boldsymbol{1} - \frac{\boldsymbol{\Delta r}}{\boldsymbol{2r_k}}\right) + \boldsymbol{u_{k,}j}(\boldsymbol{1} - \boldsymbol{2}\ \alpha) + \boldsymbol{u_{k+1,}j}(\alpha)\left(\boldsymbol{1} + \frac{\boldsymbol{\Delta r}}{\boldsymbol{2r_k}}\right) \tag{7.1}$$

$$coefficient\ \alpha = \frac{\boldsymbol{c_R * \Delta t}}{\boldsymbol{\Delta r^2}} \tag{7.2}$$

Where: $u_{k,j+1}$ is the pore pressure at a point of depth *(k)* and a time *(j+1)*

$u_{k-1,j}$ is the pore pressure at depth *(k-1)* and a time *(j)*

$u_{k,j}$ is the pore pressure at depth *(k)* and a time *(j)*

$u_{k+1,j}$ is the pore pressure at depth *(k+1)* and a time *(j)*

The radial (horizontal) coefficient of consolidation (c_R) which is needed in the finite difference solution to calculate the α coefficient was back calculated from the time required for 50% consolidation to occur during the consolidation phase of each partially drained triaxial test performed in the lab for the course of this study, t_{50}, and assuming radial drainage through the central sand columns. The radial coefficient of consolidation is affected by the radius of the sand column (r_w) the radius of the specimen (r_e) and the ratio between the two such that:

$$\boldsymbol{c_R = \left(d_e^2 * F(n) * \ln\left(\frac{1}{1-U_h}\right)\right) / (8 * t_{50})} \tag{7.3}$$

Where (d_e) is the diameter of the specimen, $F(n)$ a factor depending on n, (U_h) is the degree of consolidation (assumed to be equal 50% since vertical consolidation is neglected), and (t_{50}) is the time to achieve 50% consolidation.

$$\boldsymbol{n = \frac{r_e}{r_w}} \tag{7.4}$$

$$\boldsymbol{F(n) = \left[\left(\frac{n^2}{n^2-1}\right) * \ln(n)\right] - \frac{3n^2-1}{4n^2}} \tag{7.5}$$

$$\boldsymbol{c_R = \frac{T_h d^2}{t_{50}}} \tag{7.6}$$

The use of equation 7.6 allows calculating the values of (c_R) after analyzing the consolidation tests by obtaining t_{50} for specimens reinforced with dense sand columns having area replacement ratios of 17.8% and 31.7%. Results are presented in Table 7.3. The derived (c_R) values were used in the finite difference solution to calculate the degree of consolidation.

Table 7.3: Radial Coefficient of Consolidation

Radial coefficient of consolidation (c_R) (mm/sec^2)		
% Axial Strain per hour	A_c/A_s = 17.8%	A_c/A_s = 31.7%
60	0.047	0.029
20	0.031	0.050
10	0.045	-
5	0.039	0.050
3	0.046	-
1	0.046	0.038
Av	**0.043**	**0.042**

The use of equations 7.1 to 7.5 allows for estimating the degree of consolidation that a partially drained sample undergoes at any time with particular emphasis given to the time at which failure occurs.

The solution calculates the pore water pressures in the clay surrounding the column at different times $(t + \Delta t)$ or $(j + \Delta j)$, during the shearing stage, and at different radial distances from the edge of the sand column clay $(k + \Delta k)$ or $(r + \Delta r)$ for each node sequentially. A grid composed of increments (Δr) of one millimeter was chosen for the specimens. Therefore, for samples having a diameter of 7cm and reinforced with 3cm dense sand columns, the grid composed of twenty increments, whereas for samples reinforced with 4cm dense sand columns, there were fifteen increments. As for average deviatoric stresses, values were measured at time increments (Δt) equal to three seconds, starting from the start of the shearing stage and till the end of the test. Afterwards, the pore pressures at each increment of time were calculated by multiplying the corresponding factored deviatoric stresses in the clay by Skempton's pore pressure coefficient that was obtained by back-calculation from the triaxial tests. The average pore pressures at each increment of time were then calculated.

The average degree of consolidation at different times and strain levels can then be easily computed by subtracting the excess pore pressures that remain in the clay from the total excess pore pressures that were generated up to that time.

The finite difference solution was used to estimate the degrees of consolidation at failure (U_f) for all partially drained tests conducted on specimens reinforced with 3-cm and 4-cm columns. The resulting values of (U_f) are included in Table 7.2.

From a theoretical perspective and as mentioned previously, it is expected that the degree of consolidation (U_f) could provide a stronger indication of the degree of partial drainage in comparison to the ratio of $t_{failure}\ to\ t_{50}$. To test this hypothesis, the strength improvement index for the partially drained tests was plotted versus the computed degree of consolidation in Figure 7.7. The results shown on Figure 7.7 confirm the above hypothesis since they indicate that a relatively strong linear relationship exists between the strength improvement index $(\sigma_{d,PD} - \sigma_{d,U})/(\sigma_{d,D} - \sigma_{d,U})$ and the degree of consolidation at failure U_f. The linear relationship shows a strong correlation between the two parameters with the strength improvement index increasing as the degree of consolidation increases, irrespective of the area replacement ratio used. It should be noted however that the correlation is not perfect since there is scatter in the data around the relatively linear trend.

Theoretically, it could be argued that the expected relationship between the strength improvement index and the degree of consolidation should show that for a given degree of consolidation in the clay surrounding the column (for example U_f = 60%), the corresponding strength improvement index should be of a similar value (for example 0.6). As an extreme, when the degree of consolidation U_f = 100% (indicating full dissipation of pore water pressure in the clay), the strength improvement index should be equal to 1.0 (fully drained conditions govern). The 45 degree line shown in Figure 7.7 represents this theoretical expectation of the behavior. A thorough investigation of the data in Figure 7.7 indicates that the strength improvement index seems to be slightly larger than the equivalent degree of consolidation, with most of the data points falling above the 45 degree line. This deviation from theoretical expectations can be explained by the following two arguments: (1) the solution of the finite difference consolidation problem assumes only radial consolidation. As a result, the estimated degree of consolidation U_f is expected to underestimate to a small degree the magnitude of the true degree of consolidation which is expected to have an additional contribution from vertical drainage at the top of the samples. (2) The finite difference model that was utilized could have been affected by model uncertainty and uncertainty due to the soil parameters used (c_r, A_f, definition of failure, etc...). All these factors could contribute to the degree of scatter observed in Figure 7.7.

In support of point (1) above, it is worth noting that the samples reinforced with the 3-cm diameter columns showed the highest deviation from the 45 degree line with strength improvement indices that were higher than expected, given the estimated degree of consolidation. This is because the contribution of vertical drainage at the top of the specimens to the degree of consolidation is expected to me more significant in specimens with 3-cm columns compared to specimens with 4cm columns. The reason is that the radial drainage distance in specimens reinforced with 3-cm column is longer than the drainage distance for specimens with 4-cm columns, making any contribution to consolidation from vertical drainage more significant relative to the combined degree of consolidation.

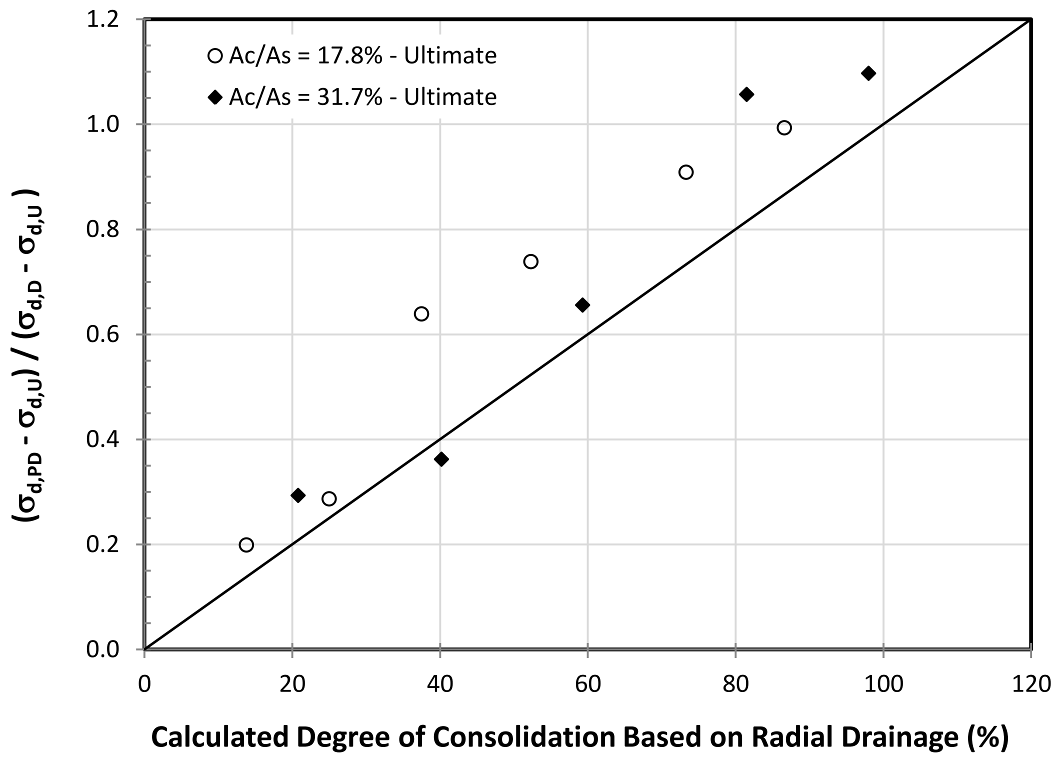


Figure 7.7: Strength improvement index vs. degree of consolidation (Ac/As=17.8% & Ac/As=31.7%)

Another direct measure of the magnitude of drainage (and pore pressure dissipation) in partially drained tests is the measured volumetric strain. It was shown previously that volumetric strains at failure decreased systematically as the strain rate was increased in the partially drained tests. The ratio of the measured volumetric strain for the partially drained tests to that of the fully drained test provides a relative indication of the amount of sample drainage and degree of consolidation that occurred during the partially drained tests compared to the fully drained tests. This ratio that is computed from the results of the triaxial tests directly and is not calculated from any theoretical model is indicative of the degree of consolidation that was achieved in the partially drained tests.

The variation of the strength improvement index with the volumetric improvement ratio (ratio of the volumetric strain measured in the partially drained test to the volumetric strain measured in the fully drained test), is plotted on Figure 7.8. The plot indicates a strong correlation between the strength improvement index and the volumetric improvement ratio, irrespective of the area replacement ratio used. This finding is significant because it indicates that simple measurements of volumetric strains from a triaxial test could provide valuable feedback on the degree of consolidation that has occurred during shearing and could be used as a basis for predicting the relative mobilization of shear strength for the partially drained tests relative to the drained and undrained strengths through the strength improvement index. These results are in accordance with those reported in the testing program that was performed by Bou Lattouf (2013).

The calculated degree of consolidation based on radial drainage was also plotted versus the volumetric improvement ratio for the partially drained tests on Figure 7.9. As expected, the plots indicate that the estimated degrees of consolidation from the finite difference approach are generally smaller than their corresponding volumetric improvement ratios, particularly for samples reinforced with 3-cm columns. This is attributed to the same reasons mentioned above regarding neglecting vertical drainage from the analysis and the uncertainties in the finite difference model for estimating the degree of consolidation.

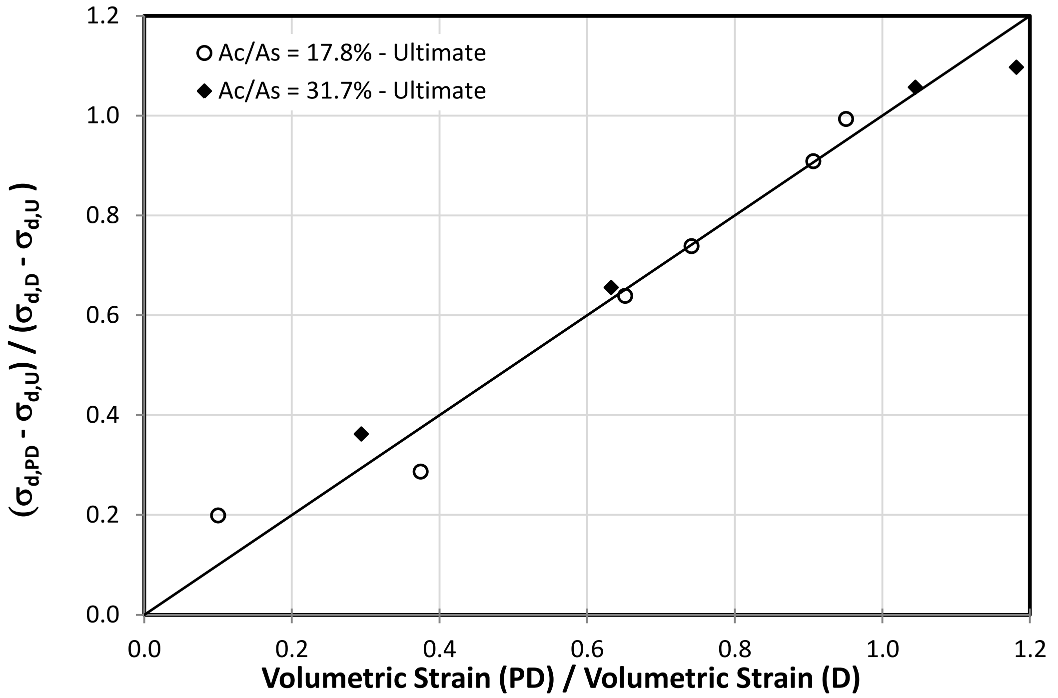


Figure 7.8: Strength improvement index vs. volumetric improvement ratio (Ac/As=17.8% & Ac/As=31.7%)

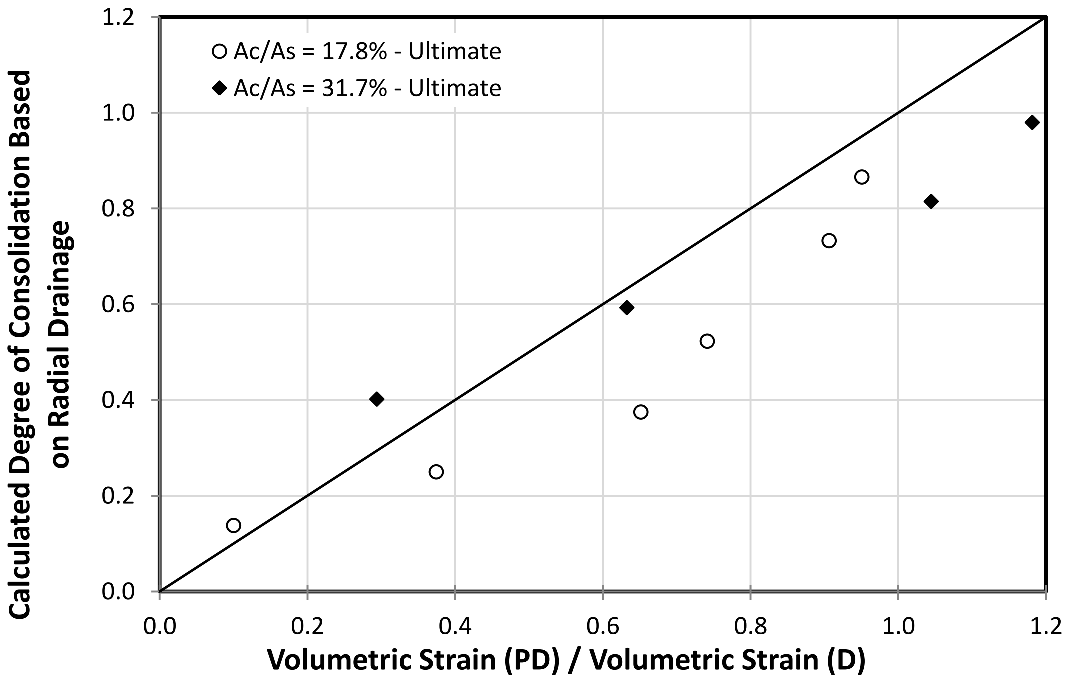


Figure 7.9: Degree of consolidation vs. volumetric improvement ratio (Ac/As=17.8% & Ac/As=31.7%)

7.4 Summary of Main Findings

Based on the results of eleven consolidated partially drained triaxial tests that were conducted in this experimental study, the following conclusions can be drawn regarding the effect of drainage conditions and shearing rates on the drained load response of Achrafieh clay, induced volumetric strains, modes of failure, and the validity of the proposed formula of degree of consolidation using finite difference.

The modes of failure indicate that for tests performed at low shearing rates, the behavior of the partially drained sample will resemble that of the traditional drained sample. The mode of failure is mainly uniform bulging across the whole specimen. As for the partially drained tests performed at fast shearing rates, the behavior of the partially drained sample will resemble the behavior of the undrained one, which also consists of bulging.

The negative volumetric strains were reduced significantly with samples reinforced with specimens having an area replacement ratio of 17.8% and 31.7%. As expected, this reduction in contractive behavior was more significant for tests with faster shearing rates. This higher reduction in contractive behavior for the specimens sheared at a fast strain rate is complemented with higher pore pressures which did not have enough time to dissipate, thus results were closer to that of the traditional undrained tests.

The deviatoric stress versus axial strain curves show that all the partially drained specimens lie between two boundaries, the higher boundary being the fully drained test and the lower boundary being the undrained test. This is consistent with the findings of Andreou et al (2008) and Bou Lattouf (2013). Furthermore, as the shearing rate decreases, the partially drained curves become closer to the fully drained curve.

A strength improvement index along with a theoretical calculated degree of consolidation was compared to the undrained and drained strengths. Results show that the strength improvement, degree of consolidation as calculated from finite difference and the volumetric improvement ratio as computed from the volumetric strain measurements of the partially drained and drained triaxial test could be utilized to predict the strength improvement index for partially drained tests.

CHAPTER 8

CONCLUSIONS AND RECOMMENDATIONS

8.1 Introduction

This chapter includes the main concluding remarks and observations resulting from the drained, undrained, and partially drained triaxial testing programs conducted on 38 Achrafieh clay specimens that were prepared from slurry, consolidated in a prefabricated 1-dimensional consolidometer, and reinforced with fully penetrating ordinary sand columns at different area replacement ratios (Ac/As=17.8% and 31.7%). The data collected from the CD and CU tests highlighted the effect of sand columns on the stiffness, drained and undrained shear strength, the volumetric strain and pore pressure generation and effective shear strength parameters for the reinforced clay. On the other hand, PD tests highlighted the effect of shearing rate and drainage conditions on the stress-strain response and volumetric strains of the reinforced specimens. An effort was also made to compare the load response of the undrained tests performed on Achrafieh clay specimens that are reinforced with medium-dense sand columns to tests conducted by Najjar et al. (2010), Maalouf (2012) and Bou Lattouf (2013).

Recommendations and further research works are also discussed in this chapter.

8.2 Main Conclusions

Based on the results of the consolidated drained, undrained, and partially drained triaxial tests that were conducted in this experimental research study, the following conclusions can be drawn with regards to the effect of drainage conditions on the load response of soft clay, volumetric strains during drained loading, stiffness of reinforced clay, and effective shear strength parameters:

8.2.1 Drained Conditions

- For consolidated drained samples, failure was characterized mainly by uniform bulging of the clay specimen along its length with concentration being at the middle portions of the sample. In addition, no shear plane was observed indicating that failure most probably occured by bulging of the sample.
- The percent improvement in the drained shear strength for the series of tests was observed at strains of 5%, 10%, 15% and 20%. Results indicate that the improvements in the deviatoric stresses of samples reinforced with 3-cm diameter dense sand columns corresponding to an area replacement ratio of 17.8% were lower than that of samples reinforced with 4-cm diameter dense sand columns corresponding to an area replacement ratio of 31.7%. Significant improvements were noted at strains of 5%, followed by a major drop in improvements at strains of 10% and minor to no improvements recorded at strains of 15% and 20%. The improvements ranged from 28.38% to 37.5% for specimens reinforced with 3cm dense sand columns and 62.27% to 80.33% for specimens reinforced with 4cm dense sand columns at 5% strain.
- Secant modulus $(E_{sec})_{1\%}$ and $(E_{sec})_{2\%}$ defined at axial strains of 1% and 2% were calculated for all tests. Results indicate that the insertion of fully penetrating dense 3-cm sand columns significantly increased the stiffness at axial strains of 1% of the unreinforced Achrafieh clay by an average of ≈111% for the three different confining pressures of 100, 150 and 200 kPa, and by an

average of ≈83% at axial strains of 2%. As for the specimens reinforced with 4-cm dense sand columns, additional increases in $(E_{sec})_{1\%}$ and $(E_{sec})_{2\%}$were noticed with improvements leaping to 202% for $(E_{sec})_{1\%}$ and 151% for $(E_{sec})_{2\%}$ for the three different confining pressures of 100kPa, 150kPa and 200kPa respectively.

- The insertion of sand columns increased the drained angle of internal friction (ϕ') of the composite system with respect to the control tests performed on Achrafieh clay when failure was defined at low strains. For samples reinforced with 4-cm dense sand columns there was an increase of up to 8.3°, compared to 4.2° for specimens that were reinforced with 3-cm dense sand columns. As for the drained cohesion (c'), insignificant changes were noticed.

8.2.2 *Undrained Conditions*

- For consolidated undrained control/unreinforced samples, failure was characterized mainly by uniform bulging of the clay specimen along its length with concentration at the middle portions of the sample. The bulging severity decreases with increasing confining pressure. The same mode of failure was identified for reinforced specimens with bulging more obvious in specimens reinforced with dense sand columns.
- Results indicate that the use of 3-cm diameter medium-dense sand columns resulted in minor increases in the undrained shear strength. Whereas significant increases in undrained shear strength were observed in specimens reinforced with 4-cm diameter dense sand columns (area replacement ratio=31.7%) with the highest improvement being 193.27% at 150kPa confining pressure. The

improvements ranged from 17.59% to 27.27% for specimens reinforced with 3cm medium-dense sand columns, 42.04% to 56.36% for specimens reinforced with 3cm dense sand columns and 169.07% to 193.27% for specimens reinforced with 4cm dense sand columns. In comparison with the smaller area replacement ratio of 17.8%, the 31.7% resulted in significant improvements in the undrained shear strength.

- An analysis of the results indicates that for specimens reinforced with fully penetrating medium-dense sand or dense columns having area replacement ratios of 17.8%, the reduction in the excess pore-water pressure was minimal. As for the specimens that were reinforced with fully penetrating dense sand columns resembling an area replacement ratio of 31.7%, a significant reduction in excess pore-water pressures was observed, with the effectiveness also decreasing with increasing the confining pressures. The reduction in excess pore-water pressures for confining pressures of 100, 150 and 200 kPa were 55.41%, 46.63% and 41.48% respectively.
- The insertion of fully penetrating medium-dense 3cm sand columns slightly decreased the stiffness of the unreinforced Achrafieh clay by an average of ≈2.86%. As for the specimens reinforced with 3cm dense sand columns, there was an average increase in $(E_{sec})_{1\%}$ of 30%. The most significant increase observed was for specimens reinforced with 4cm dense sand columns where there was an average increase in $(E_{sec})_{1\%}$ of ≈90%. This indicates that for specimens reinforced with 4cm dense sand columns, more stresses will be distributed along the column length, and consequently less settlement will result.

- The insertion of sand columns increased the drained angle of internal friction (ϕ') of the composite system with respect to the control tests performed on Achrafieh clay samples. This increase is most noticeable for samples reinforced with dense sand columns, irrespective of the area replacement ratios, where the increase was up to 8.5°, compared to 2.7° for the specimens that were reinforced with medium-dense sand columns.

8.2.3 Partially Drained Conditions

- The modes of failure indicate that for tests performed at low shearing rates, the behavior of the partially drained sample resembled that of the traditional drained sample. The mode of failure is mainly uniform bulging across the whole specimen. As for the partially drained tests performed at fast shearing rates, the behavior of the partially drained sample resembled the behavior of the undrained one, which also consists of bulging.
- The compressive volumetric strains witnessed for fully drained tests were reduced in partially drained tests with specimens having an area replacement ratio of 17.8% and 31.7%, particularly as the shearing rates increased. This higher reduction in contractive behavior for the specimens sheared at a fast strain rate is complement indicate that the pore pressures generated in the clay surrounding the column did not have enough time to dissipate, making the response closer to the undrained behavior particularly for specimens sheared at a fast rate.
- The deviatoric stress versus axial strain curves show that all the partially drained specimens lie between two boundaries, the higher boundary being the fully

drained test and the lower boundary being the undrained test. This is consistent with the findings of Andreou et al (2008) and Bou Lattouf (2013). Furthermore, as the shearing rate decreases, the partially drained curves become closer to the fully drained curve.

- A strength improvement index was developed along with a theoretical calculated degree of consolidation. These were compared to the undrained and drained strengths. Results show that the strength improvement, degree of consolidation as calculated from finite difference, and the volumetric improvement ratio as computed from the volumetric strain measurements of the partially drained and drained triaxial tests, could be utilized to predict the strength improvement index for partially drained tests.

8.3 Recommendations

Based on the test results reported in this study, it can be concluded that reinforcement of normally consolidated Achrafieh clays prepared from slurry with dense sand columns can increase the stiffness and shear strength of the soft clay. The degree of improvement in the stiffness and drained shear strength can be enhanced by increasing the area replacement ratio and density of the sand column. Results also show that Achrafieh clay specimens reinforced with medium dense sand columns show limited increase in shear strength and stiffness, unlike the specimens prepared using Kaolin clay and under the same conditions where significant improvements were observed. This indicates that the classification of the clay and its grain size distribution could play a significant role in defining the degree of improvement in the response of the clay-column composite. It is recommended that future studies target this effect in specifically designed research projects.

For practical cases that involve the use of sand columns with similar properties to the sand used in this study (friction angle of about 37.5 degrees) to improve the mechanical properties of normally consolidated clays that have similar index and strength properties to the Achrafieh clay tested in this study ($S_u/\sigma'_v = 0.3$), it can be recommended based on the drained and undrained tests conducted in this study that the clay can be improved with sand columns having a length to diameter ratio of at least 6 and with a preferred area replacement ratio that is greater than 30% to ensure an improvement that is greater than 150% in the undrained shear strength. This high improvement in the undrained shear strength was attributed to a shifting of behavior from the clay to the sand. The improvement in the undrained shear strength at area replacement ratios of 17.8% and 31.7% can be relied on for improving the short term strength of the clay without compromising the long term drained strength of the unreinforced clay.

The series of partially drained tests conducted in this study allowed for a basic understanding of the implications of partial drainage and shearing rate on the volumetric strains and degree of consolidation. The results of this series of tests proved that undrained behavior underestimates the strength of soft clay reinforced with sand columns, and that the partial drainage that occurs can significantly affect strength and volume change. The use of finite difference to estimate the degree of consolidation and the volumetric improvement ratio as computed from the volumetric strain measurements of the partially drained and drained triaxial test could be utilized to predict the strength improvement index for partially drained tests.

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