Volume Change Behavior of Expansive Soils due to Wetting and Drying Cycles

by

Daniel Curtis Rosenbalm

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Approved July 2013 by the Graduate Supervisory Committee:

Claudia E. Zapata, Chair
Sandra L. Houston
Edward Kavazanjian
Matthew W. Witczak

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ABSTRACT

In a laboratory setting, the soil volume change behavior is best represented by using various testing standards on undisturbed or remolded samples. Whenever possible, it is most precise to use undisturbed samples to assess the volume change behavior but in the absence of undisturbed specimens, remodeled samples can be used. If that is the case, the soil is compacted to in-situ density and water content (or matric suction), which should best represent the expansive profile in question. It is standard practice to subject the specimen to a wetting process at a particular net normal stress ($\sigma_{Net}$). Even though currently accepted laboratory testing standard procedures provide insight on how the profile conditions changes with time, these procedures do not assess the long term effects on the soil due to climatic changes.

In this experimental study, an assessment and quantification of the effect of multiple wetting/drying cycles on the volume change behavior of two different naturally occurring soils was performed. The changes in wetting and drying cycles were extreme when comparing the swings in matric suction. During the drying cycle, the expansive soil was subjected to extreme conditions, which decreased the moisture content less than the shrinkage limit. Nevertheless, both soils were remolded at five different compacted conditions and loaded to five different $\sigma_{Net}$. Each sample was subjected to six wetting and drying cycles.

During the assessment, it was evident from the results that the swell/collapse strain is highly non-linear at low stress levels. The strain-$\sigma_{Net}$ relationship cannot be defined by one single function without transforming the data. Therefore, the dataset needs
to be fitted to a bi-modal logarithmic function or to a logarithmic transformation of $\sigma_{\text{Net}}$ in order to use a third order polynomial fit.

It was also determined that the moisture content changes with time are best fit by non-linear functions. For the drying cycle, the radial strain was determined to have a constant rate of change with respect to the axial strain. However, for the wetting cycle, there was not enough radial strain data to develop correlations and therefore, an assumption was made based on 55 different test measurements/observations, for the wetting cycles.

In general, it was observed that after each subsequent cycle, higher swelling was exhibited for lower $\sigma_{\text{Net}}$ values; while higher collapse potential was observed for higher $\sigma_{\text{Net}}$ values, once the $\sigma_{\text{Net}}$ was less than/greater than a threshold $\sigma_{\text{Net}}$ value. Furthermore, the swelling pressure underwent a reduction in all cases. Particularly, the Anthem soil exhibited a reduction in swelling pressure by at least 20 percent after the first wetting/drying cycle; while Colorado soil exhibited a reduction of 50 percent. After about the fourth cycle, the swelling pressure seemed to stabilize to an equilibrium value at which a reduction of 46 percent was observed for the Anthem soil and 68 percent reduction for the Colorado soil.

The impact of the initial compacted conditions on heave characteristics was studied. Results indicated that materials compacted at higher densities exhibited greater swell potential. When comparing specimens compacted at the same density but at different moisture content (matric suction), it was observed that specimens compacted at higher suction would exhibit higher swelling potential, when subjected to the same $\sigma_{\text{Net}}$. 
The least amount of swelling strain was observed on specimens compacted at the lowest dry density and the lowest matric suction (higher water content).

The results from the laboratory testing were used to develop ultimate heave profiles for both soils. This analysis showed that even though the swell pressure for each soil decreased with cycles, the amount of heave would increase or decrease depending upon the initial compaction condition. When the specimen was compacted at 110% of optimum moisture content and 90% of maximum dry density, it resulted in an ultimate heave reduction of 92 percent for Anthem and 685 percent for Colorado soil. On the other hand, when the soils were compacted at 90% optimum moisture content and 100% of the maximum dry density, Anthem specimens heave 78% more and Colorado specimens heave was reduced by 69%.

Based on the results obtained, it is evident that the current methods to estimate heave and swelling pressure do not consider the effect of wetting/drying cycles; and seem to fail capturing the free swell potential of the soil. Recommendations for improvement current methods of practice are provided.
DEDICATION

To my wife, Kelly, for your unconditional love and support throughout this unforgettable journey.

To my son, Nathan, never give up on your dreams.
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Chapter 1
INTRODUCTION

1.1 Importance of the Study

A study published in 2009, attributed thirteen billion dollars (not adjusted to 2013 dollars) of annual damage to infrastructure in the United States to problems associated with expansive soils (Puppala and Cerato 2009). One-third of the damage was reported to appear in residential and commercial buildings (Wray and Meyer 2004). The remaining damage was done to roadways, bridges, and dams. Prior studies including Jones and Holtz (1973), Krohn and Slosson (1980), and Steinberg (1998) show, as a general trend, that the estimated damage associated with US infrastructure has increased with time. Figure 1.1 shows the trend of estimated damage associated with expansive soils in the United States as well as the increase in population during the same period of time. In the United States alone, the cost to repair structures damaged by expansive soils has been estimated to be twice the combined damages of natural disasters, which include floods, tornadoes, hurricanes, and earthquakes (Jones and Holtz 1973; Jones and Jones 1987; Handy 1995). Despite the alarming high costs associated with repairs to infrastructure associated with expansive soils, the Federal Emergency Management Agency (FEMA) does not have expansive soils on the list of most costly natural disasters. In addition, most insurance companies do not cover damages done by expansive soils and the owner of the property must secure financial capital to repair any damage (Fredlund and Rahardjo 1993; Bonner 1998).
Figure 1.1: Estimated monetary loss caused by expansive soils.

Figure 1.1 shows that as the population increases the costs associated with expansive soils increases. The increase of damage can be attributed to multiple factors such as land development, and the increase of different modal transportation routes, to offset the demand of population growth. The distribution of expansive soil within the continental United States, as recognized in 1972 by National Highway Research Program Report 132, is shown in Figure 1.2. Even though the map is 41 years old, it shows the expansion potential of soil ranging from non-expansive to highly-expansive soils. As shown in this figure, there are expansive soils in every state within the continental United States; with the majority of them located west of the Mississippi river. Despite the widespread occurrence of these soils, the damage caused by the presence of expansive soils is regularly overlooked since it might take years of distress exposure before extensive damage to infrastructure can be observed.
Figure 1.2: Expansive soils distribution within the United States (Witczak 1972).
1.2 Volume Change Behavior of Problematic Soils

Expansive soils are soils that have a potential for shrinkage or swelling under changing moisture conditions (Nelson and Miller 1992). Changes in the moisture conditions of an expansive soil are due to the environmental conditions imposed at the ground surface, vegetation and groundwater table fluctuations. The environmental conditions imposed at the ground surface, may either increase or decrease the moisture content. An increase in the moisture content may be due to precipitation while a decrease in the moisture content may be due to elevated temperatures. During long periods without precipitation, an expansive soil will dry and cause the soil to shrink generally leading to both recoverable and irrecoverable negative volume change. When precipitation occurs after periods of no precipitation, the infiltrated water wets the expansive soil. The recoverability of the volume change in an expansive soil is dependent on the applied stress state and the stress history of the soil. The irrecoverable volume change is known as plastic strain and the recoverable volume change is known as elastic strain. The irrecoverable volume change is due to permanent changes in a metastable soil structure that resulted from the geological deposition of the soil.

In a laboratory setting, the volume change behavior is generally assessed by testing undisturbed or remolded samples for heave and collapse potential from ASTM D4546-08. Remolded samples are used only for assessing positive volume change or heave potential. To assess the heave/expansion potential of an undisturbed sample, the sample is first loaded to a predetermined net normal stress and the sample is inundated with water. The volume change that occurs due to the application of the normal stress and the inundation of water is recorded. The extent of the volume change is dependent on the
initial stress state (i.e. density/moisture content and net normal stress). Expansive soils will exhibit positive volume change (gain in volume or swelling) when water is added to the soil as long as the applied net normal stress does not exceed a threshold pressure usually referred to as the swell pressure (i.e. applied net normal stress to achieve zero volume change).

When undisturbed samples are not collected and an assessment of swell potential is needed for a borrow site (i.e. a location of suitable fill material for use at another site), remolded samples are used in a laboratory setting. The compaction specifications for the borrow material (e.g. moisture content range and minimum percent of the standard Proctor maximum dry density) and maximum swell potential is often governed by code specifications (i.e. city, county, state or engineering specifications). After the compaction specifications are determined, a representative sample of the borrow soil is then remolded, loaded to a specified net normal stress, and inundated with water. If swelling occurs, the volume change is recorded and the sample is then discarded after its final moisture content is obtained. However, this procedure of remolding and subjecting the soil to only one wetting cycle many not accurately assess the field irrecoverable and recoverable volume change behavior of the expansive soil. Therefore, the effects of successive wetting/drying on the mechanical properties of remolded expansive soils should be considered during the design and construction processes.

1.3 Causes of Expansive Soil Behavior

The amount of damage that expansive soils can inflict upon infrastructure has been linked to the type and the amount of clay minerals encountered in the soil (Mitchell and Soga 2005). Researchers have found that most expansive soils are comprised of clay
minerals from the Smectite family (e.g. Montmorillonite) and Illite. The expansion of Smectite and Illite clay minerals is due to the large surface areas and a net negative electrical charge on the face of the clay particle. The negative electrical charge on the particle face will have a significantly larger affinity for water molecules when compared to other soil particles that do not have a net negative charge on the face (Mitchell and Soga 2005). Due to the affinity of water, soils with expansive clay minerals will be very sensitive to seasonal variation of moisture due to precipitation, evaporation from the soil surface, and/or evapotranspiration from vegetation. The hazard created by expansive soils will be the greatest in areas with pronounced wet and dry seasons (Fredlund and Rahardjo 1993). Therefore, infrastructure built in arid to semi-arid regions will be predisposed to expansive soil problems when compared to humid regions that maintain a rather uniform soil moisture condition throughout the year (Handy 1995).

1.4 Comparing Laboratory and Field Expansion Potential

In the last four to five decades, large strides have been made in classifying expansive soils by establishing testing techniques and testing methods to predict swelling or heave characteristics of soils. These techniques and methods usually involve the use of a conventional one-dimensional oedometer devices used to perform one-dimensional consolidation testing. In the standard test method ASTM D 4546-03 “Standard Test Method for One-Dimension Swell or Collapse of Cohesive Soils, the following test procedures are outlined: free-swell test (loading the specimen to a net normal “token” stress of 1 kPa (0.15 psi)), “response to wetting” test, and constant volume test. In the 2008 edition ASTM D 4546-08, the test standard was revised and the constant volume method was removed and replace with a four point method. The four point method uses
four identical compacted or undisturbed samples loaded to different net normal stresses and then inundated with water. The heave/swell or consolidation potential is then plotted versus the applied net normal stress and the swell pressure is obtained. This method has focused on allowing the expansive soil to swell in a one-dimensional oedometer cell filled with distilled water and under various loading conditions. The initial swelling that is achieved has been the focus of interest for many researchers and practitioners. For remolded samples, the soil is generally compacted to simulate field compaction characteristics.

The ASTM D4829-11”Standard Test Method for Expansion Index of Soils” is also used to assess swell potential. In the ASTM D4829-11 standard method, the soil is remolded using a desired compactive energy to achieve 50 percent degree of saturation (by varying the moisture content), then loaded to a net normal “token” stress (net normal “token” stress of 6.9 kPa (1 psi)), and finally inundated with water. In both test procedures, the deformation is recorded and converted to the expansion index.

When there is no option but using compacted expansive soil, it is customary to compact the soil in the field to a density that ranges from 90 to 95 percent of the maximum dry unit weight and moisture content that ranges from optimum moisture content to 2–4 percent above optimum as established by ASTM D698, the standard Proctor Compaction test (ASTM D698 2000) or compaction specification determined by an engineer (National Research Council 1965). This compaction density/moisture condition generally deviates from the equilibrium conditions the soil was at prior to field construction; and therefore, the soil will slowly tend to revert back to an equilibrium condition corresponding to the stress state at which it is subjected (matric suction and net
normal stress) and its current environment (Dye 2008). On the other hand, the remolded specimen used to estimate the swelling characteristics in the laboratory is compacted to the standard conditions used in any response to wetting test. For example, in Arizona, a modified expansion index test is commonly used by practicing engineers. The Arizona Expansion Index test is a one-dimensional swell test performed on reconstituted soil specimens with water content 2% below the optimum water content and at 95% of the maximum dry density as determined by the ASTM D698 standard Proctor compaction test. During the Arizona Expansion Index test, the specimen is subjected to a load of 100 psf (4.8 kPa), saturated and then allowed to swell for a period of 24 hours (Houston et al. 2011). The initial condition used in the lab often differs from the actual compacted conditions in the field. It should be noted that a 100 psf load corresponds to approximately 10 to 12 inches of overburden and therefore, it does not represent a true free swell condition, for the ground surface. It is also evident that the wetting path followed in the laboratory is different to that followed in the field due to the compaction, moisture content, stress history, and wetting and drying cycles.

Several studies have been directed to study the behavior of expansive soils after multiple wetting and drying cycles. These studies include the work of Al-Homoud et al. (1995), Bamsa et al. (1996), Tripathy et al. (2002), Subba Rao and Satayadas (1987), and Lin and Cerato (2013). However, questions still remain regarding how the swell potential of expansive soils, compacted at different initial conditions change when loaded to various net normal stresses and subjected to multiple wetting and drying cycles.
1.5 Volume Change Behavior of Expansive Soils due to Wetting and Drying Cycles

Soils in the field are typically subjected to numerous cycles of wetting and drying due to environmental cyclic conditions. Therefore, one must ask, if using the results obtained from laboratory tests on remolded specimens subjected to only one cycle of wetting is appropriate to determine the heave characteristics (i.e. swell pressure and heave/swell percent) in the field it represents. This leads to the practical lab protocol question, if multiple wetting and drying cycles in a laboratory environment are needed to accurately determine the equilibrium heave characteristics that best duplicate field conditions, if the results are drastically different to those obtained from the current laboratory standard procedures.

1.6 Research Objectives

Based upon these key questions regarding the performance prediction of the current set of lab protocols; it was evident that the behavior of expansive soils in the field might not be properly estimated based on the current laboratory test procedures used in practice. Important parameters such as the overburden pressure, laboratory initial test conditions, and wetting/drying cycles seems to greatly affect the volume change response observed in the field or “under field conditions”. And yet, the impact associated with these parameters have not been properly studied nor documented, as it will be established in Chapter 2.

As such, the main objective of this initial dissertation was to assess the accuracy of the response to the wetting test performed in the laboratory to estimate the swelling characteristics of compacted soil under field conditions. A comprehensive laboratory
testing program was developed in order to investigate the following protocol variables of the standard ASTM test procedures:

1. Assess the effect of the net normal stress on the volume change behavior of compacted expansive soils. The fact that the existing standards of practice rely upon different “token” loads to determine “free swell” was of particular concern.

2. Assess the effects of initial moisture content and density conditions on the swell pressure of a compacted expansive material subjected to not only a single wetting/drying cycle but to multiple cycles were of particular concern.

3. Determine the effects of wetting and drying cycles on the volume change and mechanical behavior of compacted expansive soils as a function of cyclic wetting and drying cycles was of particular concern. The effect on “free swell” and the effect on swell pressure were to be investigated in the study. The analysis was to be performed at a variety of differing net normal stress levels.

In summary, this dissertation focuses upon the comparison of net normal stress and void ratio compacted at different initial conditions for two different expansive soils subjected to multiple wetting/drying cycles and loaded to different net normal stresses. The methodology followed to accomplish the objectives is outlined in the next section.

1.7 Overview of the Methodology Followed to Accomplish the Objectives

In order to accomplish the objectives of this dissertation, the following general tasks were completed:

1. Compile a literature review that described how multiple wetting and drying cycles affect the heave/shrink characteristics (swell pressure and swell percent) of
compacted material. Along with information on the wetting and drying cycle
effect of compacted expansive clay; the literature review was to include previous
research findings on the effect of different initial density/moisture content
conditions on volume change behavior of expansive soils.

2. Develop and conduct a comprehensive laboratory study to determine the swelling
behavior of two different compacted clay materials subjected to different
densities, initial moisture contents, at various net normal stresses, and multiple
wetting and drying cycles.

3. Analyze the results of the comprehensive laboratory study by developing the
following relationships/results obtained for each soil. These tasks were to include:
   a. Percent heave versus net normal stress.
   b. Percent heave versus net normal stress at different number of cycles.
   c. Initial density and initial moisture content versus heave.
   d. Changes in swell pressure at different number of cycles

1.8 Plan of Work

The plan of work is summarized in Table 1.1. The main study comprised swelling
tests performed on two soils subjected to four different net normal stresses, six
consecutive wetting/drying cycles, and five different initial compaction conditions.
Replicates were added only when the goodness-of-fit (via least square error analysis)
through the four net normal stresses versus the percentage of swelling/consolidation was
below 80 percent. The five initial conditions corresponded to a factorial “cube” with a
center run as shown in Figure 1.3.
The cube “edges” corresponded to the following conditions: 90% of the optimum moisture content (OMC) and 90% of the maximum dry density (MDD), 90% of the OMC and 100% of the MDD, 110% of the OMC and 90% of the MDD, and 110% of the OMC and 100% of the MDD. The center run properties were 100% of the OMC and 95% of MDD.

Table 1.1: Factorial Design.

<table>
<thead>
<tr>
<th>Variable</th>
<th>Levels</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Net normal stress, $\sigma_n$</td>
<td>4</td>
<td>1.5kPa, 40%, 80%, and 140% Swell Pressure</td>
</tr>
<tr>
<td>Number of cycles</td>
<td>6</td>
<td>One cycle includes one wetting and one drying processes</td>
</tr>
<tr>
<td>Initial compaction condition</td>
<td>5</td>
<td>% of MDD 90 100 90 100 95 % of OMC 90 90 110 110 100</td>
</tr>
<tr>
<td>Number of soils</td>
<td>2</td>
<td>Anthem PI = 27%, and Colorado PI = 43%</td>
</tr>
</tbody>
</table>

Figure 1.3: Initial compaction conditions of laboratory specimens.

In order to perform the stated analyses, two important questions needed to be addressed: (1) how the radial strain changed throughout the testing sequence; and (2) how
the moisture content changed throughout the test. Therefore, additional testing was needed to answer these two questions. Moreover, the additional points obtained were used to enhance the relationship obtained between swelling and net normal stress. The additional test conditions are shown in Table 1.2. During this testing phase, the samples were removed from the consolidometer cell, and the diameter was measured as well as the mass of the sample. After the measurements were completed, the sample was then placed back into the consolidometer cell and the net normal stress was reapplied. Detailed test procedures are presented in Chapter 6.

<table>
<thead>
<tr>
<th>Soil</th>
<th>Initial Test Compaction Condition</th>
<th>Net Normal Stress (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>%OMC</td>
<td>%MDD</td>
</tr>
<tr>
<td>Colorado</td>
<td></td>
<td></td>
</tr>
<tr>
<td>90</td>
<td>90</td>
<td>54.3</td>
</tr>
<tr>
<td>90</td>
<td>100</td>
<td>8.2</td>
</tr>
<tr>
<td>100</td>
<td>95</td>
<td>8.2</td>
</tr>
<tr>
<td>110</td>
<td>90</td>
<td>54.3</td>
</tr>
<tr>
<td>110</td>
<td>100</td>
<td>54.3</td>
</tr>
<tr>
<td>Anthem</td>
<td></td>
<td></td>
</tr>
<tr>
<td>90</td>
<td>90</td>
<td>21.7</td>
</tr>
<tr>
<td>90</td>
<td>100</td>
<td>8.24</td>
</tr>
<tr>
<td>100</td>
<td>95</td>
<td>4.8</td>
</tr>
<tr>
<td>110</td>
<td>90</td>
<td>21.7</td>
</tr>
<tr>
<td>110</td>
<td>100</td>
<td>4.8</td>
</tr>
<tr>
<td>110</td>
<td>100</td>
<td>8.24</td>
</tr>
</tbody>
</table>

1.9 Report Organization

This project report is organized in the following manner. Chapter 2 presents an extensive literature review of the influence of wetting and drying cycles on the behavior of expansive soils. This chapter also includes a literature review regarding engineering properties associated with expansive soils, how expansive soils are measured by
practitioners, empirical correlations to obtain heave, and factors that affect swelling behavior.

Chapter 3 presents the laboratory testing conducted to obtain the soil properties of the selected soils. The soil properties presented include soil index properties as well as swelling characteristics obtained at 95 percent of the standard Proctor maximum dry density and optimum moisture content. The soil water characteristic curves are also presented.

Chapter 4 presents considerations regarding possible revisions needed to the ASTM D 4546 “Standard Test Method for One – Dimension Swell or Collapse of Cohesive Soils”. A review of the previous ASTM D 4546 standards and how this standard has progressed from its creation to the current edition is also presented. The difference in swell properties determined by the current standard and the revisions that are deemed necessary to be incorporated will be discussed.

Chapter 5 presents the tasks that were needed to accomplish the main study. This chapter also describes additional tasks that were needed, including the creation of the LabVIEW program to collect the data and the calibration of the LVDT’s used to capture the axial deformation.

Chapter 6 presents the results from the radial strain rate tests. The radial strain rates and matric suction (or moisture content) changes with time are presented. These results are consequently used in Chapter 7 for determining how the volumetric strain changes with time.
Chapter 7 presents the results from the main experiment. The analysis for determining the total volume and void ratio calculations are first outlined, followed by the axial and volumetric strain changes with time plots.

Chapter 8 presents the analysis of the laboratory results. The determination of the dry density, moisture content and degree of saturation is outline, followed by the analysis of the effects of the wetting and drying cycles on the behavior of the expansive soils.

Chapter 9 presents the summary and conclusions of the work presented herein. Finally, Chapter 10 presents the recommendations for future work in order to advance both the state of the art and state of the practice related to the characteristics of compacted expansive soils.
Chapter 2

LITERATURE REVIEW

2.1 Importance of this Study

Current engineering practices for determining the volume change behavior of unsaturated expansive soils are mostly based on simplified tests and correlations with index properties. Such practices can lead to uneconomical and distress prone designs. By understanding the volume change behavior of unsaturated expansive soils, it is expected to contribute to more economical and rational design procedures and practices.

It has been shown that the initial dry density, moisture content, and net normal or overburden stress in the field govern the behavior of expansive soils (Al-Homoud et al. 1995; Basma et al. 1996; Chen 1988; Tripathy et al. 2002; Subba Rao, and Satayadas 1987). However, these studies only look at the effects of the wetting/drying cycle at the in-situ stress level or design stress state. It is important to understand that the net normal stresses in the entire soil profile are ever changing due to either consolidation or expansion of the profile when it is subjected to free water, and therefore; it is of interest to study how the expansive material properties respond to varying external stress levels.

This chapter presents a summary of past literature on the definition of expansive soils and expansive soil properties, engineering properties of expansive soils, and volume change associated with wetting and drying cycles of compacted soils. After formally defining expansive soils, the measurable engineering properties of expansive soils are presented. Then the review focuses on the different standards of practice currently used to obtain the swell potential, swell pressure, and the expansion index. Finally, the literature review is focused on studies related to volume change behavior of compacted expansive
soils associated with wetting and drying cycles. This section is sub-divided into two sections. The first sub-section presents previous research findings and measurements; and the second sub-section presents modeling methods created and used by various researchers.

2.2 Engineering Properties of Expansive Soils

In the past 50 to 60 years, researchers have tried successfully and unsuccessfully to identify different unique properties that are soil type and property independent that would aid with classifying expansive soils. The current unique identifiers include the following, in no specific order/discovery: Cation Exchange Capacity (CEC), Specific Surface Area (SSA), Atterberg Limits (plastic, liquid, and shrinkage limits), Clay Fractions ($C_p$), Activity, $C_s$ (Swelling Index), Quantitative Clay Mineralogy, and Free Swell Index. A brief description of each of these properties is presented in the following sections.

2.2.1 Cation Exchange Capacity

The cation exchange capacity or the CEC level is dependent on the isomorphic substitutions that occur with the clay minerals (Mitchell and Soga 2005). The isomorphic substitutions are due to tetrahedral and octahedral sheets containing cations instead of an idealize structure (i.e. aluminum in the places of silicon, magnesium instead of aluminum, etc.). When the isomorphic substitution occurs, multiple cations are replaced with other cations of other valances within the structure to maintain equilibrium within the clay structure. The ability to measure the cation replacement is computed as milliequivalents (meq) per 100 g of clay. The milliequivalents are determined by knowing the atomic weight, and the weight and valance of the element. The CEC value is
a guide to estimate the predominant clay mineral. In addition, the CEC shows how stable
the clay mineral is to isomorphic substitution. When the measureable CEC increases the
isomorphic substitution within the clay mineral also increases. Table 2.1 outlines CEC
values for common clay minerals. As one can see, Kaolinite has the lowest CEC, while
Vermiculite has the highest CEC.

Table 2.1: CEC values for Common Clay Minerals (Mitchell and Soga 2005).

<table>
<thead>
<tr>
<th>Clay Mineral</th>
<th>meq/100 g</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kaolinite</td>
<td>3</td>
</tr>
<tr>
<td>Halloysite</td>
<td>12</td>
</tr>
<tr>
<td>Illite</td>
<td>25</td>
</tr>
<tr>
<td>Vermiculite</td>
<td>150</td>
</tr>
<tr>
<td>Smectite</td>
<td>85</td>
</tr>
<tr>
<td>Chlorite</td>
<td>40</td>
</tr>
</tbody>
</table>

2.2.2 Specific Surface Area

The specific surface area (SSA) is the measure of the surface area of a clay
sample, which is determined by the amount of the polar molecule (i.e. glycol, glycerol, or
ethylene glycol mono-ethyl ether (EGME)) retained under laboratory controlled
conditions and it is then convert to SSA (Mitchell and Soga 2005). The SSA of the
sample is an indication the governing clay mineral within the test sample. Table 2.2
shows different ranges of SSA for common clay minerals.

Table 2.2: SSA values for Common Clay Minerals (Mitchell and Soga 2005).

<table>
<thead>
<tr>
<th>Clay Mineral</th>
<th>SSA (m²/g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kaolinite</td>
<td>10 to 20</td>
</tr>
<tr>
<td>Halloysite</td>
<td>35 to 70</td>
</tr>
<tr>
<td>Illite</td>
<td>65 to 100</td>
</tr>
<tr>
<td>Vermiculite</td>
<td>40 to 80</td>
</tr>
<tr>
<td>Smectite</td>
<td>50 to 800</td>
</tr>
</tbody>
</table>
As shown in Table 2.2, the range of SSA for the different minerals can overlap significantly. For example, if a soil sample has a SSA of 65 m$^2$/g, it can be classified as Halloysite, Illite, Vermiculite, or Smectite. In this case, to define the governing clay mineralogy will require additional testing (i.e. X-ray diffraction).

**2.2.3 Atterberg Limits**

The concept of the Atterberg Limits was introduced to Geotechnical engineering by Arthur Casagrande in 1932. Under Casagrande, a uniform test method was developed to determine the liquid limit (LL), plastic limit (PL), and the plasticity index (PI) of soil. The liquid limit and the plastic limit correspond to different shear strengths of the soils. In 1948, Casagrande used the Atterberg Limits to create a soil classification system, which was then modified by the United Soil Classification System (USCS). Under this system, fines are classified as one of the following: non-plastic (NP), lean silt (ML), highly compressible silt (MH), lean clay (CL), and fat clay (CH). Most of the expansive soil exhibits characteristics of either a CL or CH soil.

Holtz and Kovacs (1981) overlaid different ranges of clay mineralogy on the USCS soil classification, which is shown in Figure 2.1. As one can see, both Kaolinite and Chlorite fall with the ML and MH soil classification ranges while the Illite and Montmorillonite fall within the CL and CH classification ranges.
Figure 2.1: Location of clay mineralogy bands on USCS soil classification (modified after Holtz and Kovacs 1981).

Figure 2.1 is only a guideline for clay mineralogy classification based upon the Atterberg limits. There are a multitude of soils that do not fall within the clay mineralogy groupings that governing the behavior of the soil sample/stratigraphy.

2.2.4 Shrinkage Limit

The shrinkage limit of a soil is the point at which the soil will not exhibit volume change when moisture is removed from the soil sample. Shown in Figure 2.2 is a conceptual drawing of the shrinkage limit (SL).
2.2.5 Clay Fractions

Depending on the classification system the clay fraction size can vary. AASHTO states the clay fraction is determined by the percent passing 0.005 mm while USCS and USDA both state the clay fraction is determined by the percent passing 0.002 mm. When Skempton introduced the idea of activity the clay fraction that he specified is determined by the percent passing 0.002 mm, which is obtained from a hydrometer analysis.

2.2.6 Activity

In 1953 Skempton, introduce the concept of activity. The activity of a soil is defined by the plasticity index divided by the clay fraction, which is shown in Equation 2.2. As the activity increases the higher the swell potential that will occur. Figure 2.2 section 2.4 shows the activity chart that Skempton developed.

\[
A = \frac{PI}{CF - 5}
\]
2.2.7 Swelling Index, $C_s$

The swelling index is less than the compression index, most of the time by a considerable amount. When the soil is non-expansive the swelling index will be less than 0.1 and expansive soils will have a swelling index greater than 0.2 (Mitchell and Soga 2005). Shown in Figure 2.3 is an idealized effective stress void ratio curve for a compressible soil. Where the segment AB, in Figure 2.3, is the initial or virgin compression curve, segment BC is swelling, and segment is CD is the recompression curve. Normally, when a soil does not exhibit a high degree of expansion ($C_s$ less than 0.1) then segment BC is considered rebound. Table 2.3 shows typical swelling indexes for different minerals and sand consolidated under different pore fluids.

![Figure 2.3: Idealized effective stress – void ratio for a compressible soil (Mitchell, J.K. and Soga, K. 2005).](image-url)
Table 2.3: Swelling Indexes for Different Minerals (Olson and Mersi 1970).

<table>
<thead>
<tr>
<th>Mineral</th>
<th>Pore Fluid</th>
<th>Absorbed Cations</th>
<th>Electrolyte Concentration</th>
<th>$e_o$ @ 5kPa</th>
<th>Swelling Index</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>0.01 to 0.03</td>
</tr>
<tr>
<td>Kaolinite</td>
<td>$H_2O$</td>
<td>Na</td>
<td>$1 \times 10^{-4}$</td>
<td>0.95</td>
<td>0.08</td>
</tr>
<tr>
<td></td>
<td>$H_2O$</td>
<td>Na</td>
<td>$1 \times 10^{-4}$</td>
<td>1.05</td>
<td>0.08</td>
</tr>
<tr>
<td></td>
<td>$H_2O$</td>
<td>Ca</td>
<td>1</td>
<td>0.94</td>
<td>0.07</td>
</tr>
<tr>
<td></td>
<td>$H_2O$</td>
<td>Ca</td>
<td>$1 \times 10^{-4}$</td>
<td>0.98</td>
<td>0.07</td>
</tr>
<tr>
<td>Ethyl Alcohol</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>1.1</td>
<td>0.06</td>
</tr>
<tr>
<td>Dry air</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>1.36</td>
<td>0.04</td>
</tr>
<tr>
<td>Illite</td>
<td>$H_2O$</td>
<td>Na</td>
<td>1</td>
<td>1.77</td>
<td>0.37</td>
</tr>
<tr>
<td></td>
<td>$H_2O$</td>
<td>Na</td>
<td>$1 \times 10^{-4}$</td>
<td>2.5</td>
<td>0.65</td>
</tr>
<tr>
<td></td>
<td>$H_2O$</td>
<td>Ca</td>
<td>1</td>
<td>1.51</td>
<td>0.28</td>
</tr>
<tr>
<td></td>
<td>$H_2O$</td>
<td>Ca</td>
<td>$1 \times 10^{-4}$</td>
<td>1.59</td>
<td>0.31</td>
</tr>
<tr>
<td>Ethyl Alcohol</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>1.48</td>
<td>0.19</td>
</tr>
<tr>
<td>Dry air</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>1.46</td>
<td>0.04</td>
</tr>
<tr>
<td>Smectite</td>
<td>$H_2O$</td>
<td>Na</td>
<td>$1 \times 10^{-4}$</td>
<td>5.4</td>
<td>1.53</td>
</tr>
<tr>
<td></td>
<td>$H_2O$</td>
<td>Na</td>
<td>$5 \times 10^{-4}$</td>
<td>11.15</td>
<td>3.6</td>
</tr>
<tr>
<td></td>
<td>$H_2O$</td>
<td>Ca</td>
<td>1</td>
<td>1.84</td>
<td>0.26</td>
</tr>
<tr>
<td></td>
<td>$H_2O$</td>
<td>Ca</td>
<td>$1 \times 10^{-4}$</td>
<td>2.18</td>
<td>0.34</td>
</tr>
<tr>
<td>Ethyl Alcohol</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>1.49</td>
<td>0.1</td>
</tr>
<tr>
<td>Muscovite</td>
<td>Water</td>
<td>--</td>
<td>--</td>
<td>2.19</td>
<td>0.42</td>
</tr>
<tr>
<td></td>
<td>Dry air</td>
<td>--</td>
<td>--</td>
<td>2.29</td>
<td>0.41</td>
</tr>
</tbody>
</table>

2.2.8 Quantitative Clay Mineralogy

Quantitative clay mineralogy is determined by using various computer programs fitting the basal spacing of known clay minerals obtained from extensive X-ray diffraction (XRD) testing to the XRD signal of the soil sample. Depending on the methodology select for analysis, only certain clay minerals are selected to determine the quantitative clay mineralogy; however, in this process, three to four known clay minerals are used to determine the clay composition. When only three to four minerals are selected, some of more/less expansive minerals are left out of the quantification, which can lead to issues when comparing results from other laboratories or determination of the soil composition. Figure 2.4 shows an example of changing the number of minerals from four to twenty and computing the mineral composition of the sample (Shafer, Z. 2013).
2.2.9 Free Swell and Free Swell Index

Free swell is the difference between the final height and the consolidated height divided by the consolidated height. The consolidated height is the height of the sample achieved within a certain amount of time, per the referenced standard, under a prescribed
token load, prior to inundation (or adding water to the sample). The token load for the free swell can range from 1 kPa to 5 kPa (20 psf to 100 psf), which the token load is dependent on the test standard that is used. ASTM D 4546 the token load is 1 kPa and the prescribed token load in Arizona is 5 kPa. ASTM D 4546 states the compactive effort and the moisture content is determined by the laboratory that is running the test, while the Arizona method predetermines the compactive effort and moisture content. The Arizona method requires the moisture content two percent less than the optimum and 95 percent of the maximum dry density.

On the other hand, the free swell index (FSI) is an index property that was proposed by Holtz and Gibbs in 1956 and turned into an ASTM standard, ASTM D5890. The free swell index reflects the potential for expansion of the soil by comparing the ratio of the volume of soil in water to the volume of soil in kerosene. The FSI sample is prepared by oven drying soil passing the #40 sieve and the FSI is expressed by Equation 2.2. The expansion potential for the FSI is shown in Table 2.4.

\[
FSI = \frac{100(V_{water} - V_{kerosene})}{V_{kerosene}}
\]

Where:

\(V_{water}\) is the volume of the soil sample in water

\(V_{kerosene}\) is the volume of the soil sample in kerosene

**Table 2.4: Expansion potential per the Free Swell Index (Mohan 1977).**

<table>
<thead>
<tr>
<th>FSI</th>
<th>Degree of Expansion</th>
</tr>
</thead>
<tbody>
<tr>
<td>&gt;200</td>
<td>Very High</td>
</tr>
<tr>
<td>100 - 200</td>
<td>High</td>
</tr>
<tr>
<td>50 - 100</td>
<td>Medium</td>
</tr>
<tr>
<td>&lt; 50</td>
<td>Low</td>
</tr>
</tbody>
</table>
2.3 Guidelines of Swell Potential Determination Based on Engineering Properties

Various guidelines to determine the potential of swelling of expansive soils by measureable engineering soil properties are presented in this chapter. The most common soil properties used to determine the swell potential of expansive soils include activity, Atterberg limits, clay fractions, colloidal content, plasticity index, probable swell percent, shrinkage limit and the shrinkage index. There are numerous guidelines posted in the literature; however, the ones presented in this section are the major highlights that have been made throughout the past 50 plus years. Regardless of the swell potential classifications, it was found that once the swell potential is considered “medium” or “marginal”, the potential for a geotechnical hazard is significant.

In 1948, Skempton proposed a methodology to classify expanse potential for all types of soil. His methodology uses the percent of clay fraction (percent passing 0.002 mm) and the plasticity index. Shown in Figure 2.5 is the swell potential related to the plasticity index and the clay fraction.
In 1959, The U.S. Bureau of Reclamation (USBR) created a method that used direct correlation of the volume change with the plasticity index, shrinkage limit, and the colloidal content (Holtz 1959). Holtz recorded the volume change of the material from an air-dry state to a saturated state under a 1-psi (7-kPa) surcharge pressure in an odometer apparatus. The swell potential criterion that was determined by Holtz is located in Table 2.5.

**Table 2.5: Swell Potential Criteria per USBR 1959.**

<table>
<thead>
<tr>
<th>Swell Potential</th>
<th>% Swell</th>
<th>Colloidal Content</th>
<th>Plasticity Index</th>
<th>Shrinkage Limit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low</td>
<td>&lt; 10</td>
<td>&lt; 15</td>
<td>&lt; 18</td>
<td>&gt; 15</td>
</tr>
<tr>
<td>Medium</td>
<td>20</td>
<td>13 – 23</td>
<td>15 – 28</td>
<td>10 – 16</td>
</tr>
<tr>
<td>High</td>
<td>30</td>
<td>20 – 31</td>
<td>25 – 41</td>
<td>7 – 12</td>
</tr>
<tr>
<td>Very High</td>
<td>&gt; 30</td>
<td>&gt; 28</td>
<td>&gt; 35</td>
<td>&lt; 11</td>
</tr>
</tbody>
</table>

Prior to the 1959 USBR method for determining the swell potential of questionable soils, Holtz, R.D. and Gibbs, H.J., in 1956, developed a method. This
method was the basis of the USBR method. The Holtz and Gibbs swell potential criteria is shown in Table 2.6.

**Table 2.6: Swell Potential Criteria per Holtz and Gibbs 1956.**

<table>
<thead>
<tr>
<th>Swell Potential</th>
<th>% Swell under 1 PSI</th>
<th>Colloid Content</th>
<th>Plasticity Index</th>
<th>Shrinkage Limit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low</td>
<td>&lt; 10</td>
<td>&lt; 17</td>
<td>&lt; 20</td>
<td>&gt; 13</td>
</tr>
<tr>
<td>Medium</td>
<td>20</td>
<td>12 – 27</td>
<td>12 – 34</td>
<td>8 – 18</td>
</tr>
<tr>
<td>High</td>
<td>30</td>
<td>18 – 37</td>
<td>23 – 45</td>
<td>6 – 12</td>
</tr>
<tr>
<td>Very High</td>
<td>&gt; 30</td>
<td>27</td>
<td>&gt; 32</td>
<td>&lt; 10</td>
</tr>
</tbody>
</table>

In 1955, Altmeyer had major criticisms of USBR method for classifying expansive soils and suggested a new method based on correlations between the linear shrinkage, shrinkage limit and the percent swell. His recommendations are shown in Table 2.7.

**Table 2.7: Swell Potential Criteria per Altmeyer 1955.**

<table>
<thead>
<tr>
<th>Linear Shrinkage</th>
<th>Shrinkage Limit</th>
<th>Probable Swell</th>
<th>Degree of Expansion</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 5</td>
<td>&gt; 12</td>
<td>&lt; 0.5</td>
<td>Noncritical</td>
</tr>
<tr>
<td>5 – 8</td>
<td>10 – 12</td>
<td>0.5 – 1.5</td>
<td>Marginal</td>
</tr>
<tr>
<td>&gt; 8</td>
<td>&lt; 10</td>
<td>&gt; 1.5</td>
<td>Critical</td>
</tr>
</tbody>
</table>

In 1965, Ranganathan and Satyanarayana were the first researchers to use the concept of the shrinkage index (i.e. $SI = LL - SL$) (Snethen et al. 1977). Their suggestions for potential swell classification based upon the shrinkage index are shown in Table 2.8.

**Table 2.8: Swell Potential per Ranganathan and Satyanarayan 1965.**

<table>
<thead>
<tr>
<th>Shrinkage Index</th>
<th>Potential Swell Classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 20</td>
<td>Low</td>
</tr>
<tr>
<td>20 – 30</td>
<td>Medium</td>
</tr>
<tr>
<td>30 – 60</td>
<td>High</td>
</tr>
<tr>
<td>&gt; 60</td>
<td>Very High</td>
</tr>
</tbody>
</table>
In 1965 Chen introduced a methodology for classify the swell potential, which Chen was tried to simplified the USBR method. The simplification of the USBR method was by eliminating the hydrometer analysis and replacing the colloidal content with percent passing the 200 sieve. Along with replacing the hydrometer data with sieve data, he also incorporated SPT field data and correlated the data to odometer swell data. Table 2.9 shows the laboratory and field data correlation to the degree of expansion.

<table>
<thead>
<tr>
<th>$P_{200}$</th>
<th>Liquid Limit</th>
<th>SPT (per ft)</th>
<th>Probably Expansion</th>
<th>Degree of Expansion</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 30</td>
<td>&lt; 30</td>
<td>&lt; 10</td>
<td>&lt; 1</td>
<td>Low</td>
</tr>
<tr>
<td>30 – 60</td>
<td>30 – 40</td>
<td>10 – 20</td>
<td>1 – 5</td>
<td>Medium</td>
</tr>
<tr>
<td>60 – 95</td>
<td>40 – 60</td>
<td>20 – 30</td>
<td>3 – 10</td>
<td>High</td>
</tr>
<tr>
<td>&gt; 95</td>
<td>&gt; 60</td>
<td>&gt; 30</td>
<td>&gt; 10</td>
<td>Very High</td>
</tr>
</tbody>
</table>

In 1967, Terzaghi and Peck tried to relate the swell potential of an expansive soil with the plasticity index of the soil. There results were determined by analyzing the results found from Seed, Woodward, and Lundgren 1962. The Terzaghi and Peck swell potential criteria is located in Table 2.10.

<table>
<thead>
<tr>
<th>Plasticity Index</th>
<th>Swell Potential</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 – 15</td>
<td>Low</td>
</tr>
<tr>
<td>10 – 35</td>
<td>Medium</td>
</tr>
<tr>
<td>20 – 55</td>
<td>High</td>
</tr>
<tr>
<td>55 and greater</td>
<td>Very High</td>
</tr>
</tbody>
</table>

In 1967 Raman, V. introduced the swell potential criteria based on the Atterberg Limits and the shrinkage limit. Raman correlated the plasticity index and the shrinkage limit potential swell potential; however, Raman did not include any reported odometer
swelling data. Shown in Table 2.11 is the potential swell potential for the different ranges of plasticity index and the shrinkage index.

**Table 2.11: Swell Potential per Raman 1967.**

<table>
<thead>
<tr>
<th>Plasticity Index</th>
<th>Shrinkage Index</th>
<th>Potential Swell Potential</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 12</td>
<td>&lt; 15</td>
<td>Low</td>
</tr>
<tr>
<td>12 – 23</td>
<td>15 – 30</td>
<td>Medium</td>
</tr>
<tr>
<td>23 – 32</td>
<td>30 – 40</td>
<td>High</td>
</tr>
<tr>
<td>&gt; 32</td>
<td>&gt; 40</td>
<td>Very High</td>
</tr>
</tbody>
</table>

In 1969, Sowers first attempt to describe the potential volume change of an expansive material, Sowers only used the plasticity index. In 1970 Sowers and Sower included the shrinkage limit in describing the potential volume change. The addition of the shrinkage limit increased the accuracy of the prediction. Shown in Table 2.12 is the potential volume change per Sowers and Sowers.

**Table 2.12: Potential Volume Change per Sowers and Sowers 1970.**

<table>
<thead>
<tr>
<th>Shrinkage Limit</th>
<th>Plasticity Index</th>
<th>Potential Volume Change</th>
</tr>
</thead>
<tbody>
<tr>
<td>&gt; 12</td>
<td>&lt; 15</td>
<td>Probably Low</td>
</tr>
<tr>
<td>10 – 12</td>
<td>15 – 30</td>
<td>Probably Moderate</td>
</tr>
<tr>
<td>&lt; 10</td>
<td>&gt; 30</td>
<td>Probably High</td>
</tr>
</tbody>
</table>

In 1973, Dakshanamurthy and Raman proposed another method to predict swell potential using the same methodology that Casagrande outline back in 1948. Nevertheless, Dakshanamurthy and Raman used engineering judgment and posted literature values to generate the swell potential per the plasticity index – liquid limit chart. The chart that Dakshanamurthy and Raman proposed is shown in Figure 2.6.
In 1977, Snethen et al. generated swell potential classification under funding from the FHWA. Snethen et al. performed laboratory testing on multiple soils and compared the results of the swell potential to various author’s swell potential classifications. After the comparing the different swell potential classifications, Snethen et al. generated a new swell potential classification that included the in-situ suction. Shown in Table 2.13 is the swell potential classification that Snethen et al. developed. It is interesting to note that the suction limits that Snethen purposed, for the potential swell classification is near the air entry value for compacted expansive clays.

Table 2.13: Swell Potential Classification per Snethen et al. 1977.

<table>
<thead>
<tr>
<th>Liquid Limit</th>
<th>Plasticity Index</th>
<th>$\psi_{nat}$ tsf (kPa)</th>
<th>Potential Swell</th>
<th>Potential Swell Classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>&gt; 60</td>
<td>&gt; 35</td>
<td>&gt; 4 (383)</td>
<td>&gt; 1.5</td>
<td>High</td>
</tr>
<tr>
<td>50 - 60</td>
<td>25 – 35</td>
<td>1.5 – 4 (144 - 383)</td>
<td>0.5 – 1.5</td>
<td>Marginal</td>
</tr>
<tr>
<td>&lt; 50</td>
<td>&lt; 25</td>
<td>&lt; 1.5 (144)</td>
<td>&lt; 0.5</td>
<td>Low</td>
</tr>
</tbody>
</table>

Figure 2.6: Swell potential per the plasticity chart (Dakshanamurthy and Raman 1973)
Kay 1990 showed good correlation to the shrink-swell response for remolded compacted soils. His correlation for swell potential is based on Liquid Limit. The ranges for the LL correlation are shown in Table 2.14. The limits that Kay proposed are slight different than those proposed by Dakshanamurthy and Raman.

Table 2.14: Swell Potential Classification per Kay 1990.

<table>
<thead>
<tr>
<th>Site Classification</th>
<th>LL Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>S (slightly expansive)</td>
<td>&lt; 20</td>
</tr>
<tr>
<td>M (moderately expansive)</td>
<td>20 – 40</td>
</tr>
<tr>
<td>H (highly expansive)</td>
<td>40 – 70</td>
</tr>
<tr>
<td>E (extremely expansive)</td>
<td>&gt;70</td>
</tr>
</tbody>
</table>

As one can see the, the guidelines for swell potential ranges are different for the different authors as well as the engineering index property that is correlated to the swell potential. Nevertheless, these are the guidelines that have been set forth by the various authors have been used in by different agencies. The underlying factors that have been used for correlation include the Atterberg limits, the shrinkage limit up until the late-seventies and after the late seventies the matric suction of the soil was added. It is interesting to note that when the suction was added, the suction limit that as selected for the high classification is located near the air entry value for expansive clays.

2.4 How Expansive Soil Volume Change is Currently Measured by Practitioners

In practice today, the percentage heave, swell pressure, or the expansion index of the soil are the primary soil index properties that practitioners use to identify expansive soils behavior. The identification of expansive soils normally occurs with proactive practitioner’s that want to identify potential hazards associated with an unknown site. On the other hand, the determination of how to remediate expansive soils normally occurs when an engineer or engineering company did not do their due diligence and identify the
expansive soil and another company or engineer is hired to remediate the issue at hand. Nevertheless, the ASTM methods for determining the percent heave, swell pressure and the expansion index of expansive soils will be outlined in the subsections below.

### 2.4.1 ASTM D 4546 Standard Test Method for One – Dimension Swell or Collapse of Cohesive Soils

The ASTM D 4546 literature is sub-divided into two sections; prior to 2008 and after 2008. In 2008, the standard was completely revised by changing the methodology for determining the one dimensional or collapse of cohesive soils. The one dimension swelling of cohesive soils will be the primary focus of the following subsections.

#### 2.4.1.1 Prior to 2008

ASTM D 4546 presents three different methodologies to assess expansive soils. Each method results in obtaining a swell percentage/heave under a given applied load.

Method A obtains the swell percent/heave from a token load; heave at any given vertical pressure up to the swell pressure, and the load back swell pressure. The token pressure designated per ASTM is 1-kPa or 20-psf. The load back swell pressure is obtained by increasing the load to 5-kPa, and doubling the load thereafter until the original height of the sample is surpassed by the loading sequence. The loading sequence follows ASTM D 2346.

Method B determines the swell/collapse potential under a specified load. This method is known as the “Response to Wetting Test”. Method B states that the sample will be loaded up to the designated load and allow for consolidation to occur. Then the sample in inundated with water and the deformation is recorded. The deformation is recorded is
the swelling/collapse potential is determined. After the deformation stops, the sample load is double per ASTM D 2436 until a specified load is achieved.

Method C determines the swell pressure by increasing the applied pressure on the sample to achieve zero volume change. This method is known as the “Constant Volume Swell Test” or CVST. After the applied pressure becomes stable, the sample is then loaded to higher stress to achieve sample compression. The sample is then unloaded to determine the swelling index, $C_s$. The swell pressure is determined by a modified Casa Grande construction. The Casa Grande Construction to obtain the swell pressure is shown in Figure 2.7. The $C_s$ is applied to the range of stress applied to determine the heave at any given vertical stress value.

In the standard, ASTM D18 subcommittee, state that both method A and C reproduce similar heave values seen in field condition. Method B on the other hand, under represents the heave of field conditions (ASTM D4546 2003). This can be attributed to the consolidation at the stress state at which the sample is tested. The field sample is subjected to the stress state longer than the laboratory sample and has different initial testing conditions as well (i.e. different moisture content, dry density, void ratio, etc.). In addition, the field conditions are ever changing due to the precipitation and evaporation that occurs while the laboratory sample is dependent on the time of sample for the initial conditions.
2.4.1.2 After the 2008 Version

In 2008, the D18 subcommittee completely revised the testing standard, which revised the methodologies to obtain heave and swell pressure that better simulates the one-dimensional wetting-induced volume change behavior of compacted or natural soils in the field (ASTM D 4546 2008). One of the changes in particular, describes wetting-after-loading that the subcommittee states that this test procedure “that is similar to the first-time wetting episode of compacted fills after construction. Nevertheless, the subcommittee decided on three test methodologies to best represent field behavior. With the revision of the testing standard, the subcommittee named all the testing methodologies to describe the testing procedure.

Method A was completely changed it is now known as “wetting-after-loading tests on multiple specimens”. The test method that can be used to measure 1-D wetting induced swelling or collapsing strains over a range of varying vertical stresses. This test requires a four or more identical specimens to be tested at varying applied loads to assess
the heave or collapse at different overburden stresses and obtain the swell pressure of the sample.

Method B likewise was completely changed; it is now known as “single point wetting-after-loading on a single specimen”. The specimen is loaded to the desired overburden stress or design stress and then the sample is wetted. The axial deformation is recorded and the swelling/collapse strain is calculated.

Method C is no longer known as the constant volume swell test; it is now known as “loading-after-wetting test”. After a sample has been subjected to wetting induced collapse/swelling the sample is then loaded to specified stress that would simulate the in-situ addition of a structure or fill material. To accomplish Method C, it requires either Method A or B to be performed first to achieve the wetting induced strain.

As one can see, ASTM D 4546 changed drastically from 2003 to 2008. The changes of this standard were made so that the laboratory behavior would better represent the field behavior. The changes now give an engineer the ability to understand the swelling/collapse strain of a given strata over large ranges of overburden stresses, which is needed in design or litigation after the occurrence of the heave or collapse.

2.4.2 ASTM D 4829-11 Standard Test Method for Expansion Index of Soils

The expansion test was developed in 1960’s in Southern California as a request of the local agencies for a development of a standardized test to measure the expansion of soils (Nelson and Miller 1992). It was adopted by the Uniform Building Code (UBC Standard No. 29-2) and many California practitioners. Then the test became an ASTM testing standard after more studies were performed. The ASTM test is performed by compacting wet soil into 4-in diameter by 1-in height ring at a degree of saturation of 50
percent plus or minus 2 percent (ASTM D4829). The degree of saturation is determined by the moisture content of the soil and the dry density of the soil, which the moisture content and dry density to compact the material into the ring can either be obtained by determining these values from a compaction curve or engineering judgment for moisture content and then the sample is compacted into the ring and the dry density is back calculated and the degree of saturation is determined. If the back-calculated dry density does not obtain a degree of saturation within the limits of the test the operator will adjust the moisture content until the dry density obtains a degree of saturation within the limits.

Prior to compacting the soil into the ring, the soil is passed through the #4 sieve and the soil is mixed with water. The soil and water mixture is supposed to sit for a minimum of 16 hours per the ASTM standard so that soil will have a homogenous moisture content. The soil is compacted in two lifts using a special mold, with 15-blow per lift using a standard 5.5-lb hammer (ASTM D698 hammer), scarify between lifts, and the excess soil is trimmed off to leave a sample with a 1-in height. Once the soil is compacted into the ring, sample is place into consolidometer device with air-dried top and bottom porous stones and allowed to consolidate under a 6.9-kPa or 1 psi load for 10 minutes. The compaction energy exerted on the remold sample is similar to the compaction energy that is exerted during an ASTM D698 standard Proctor test. The sample is inundated with distilled water and the sample is allowed to swell or consolidate taking reading per ASTM 2435 for a minimum of 24 hours or a minimum of 3 hours with the expansion of 0.005-mm/h or 0.0002-in/h (ASTM D4829).
After the test is finished, the final mass is recorded the final moisture content, the final degree of saturation, and the expansion index is determined. The expansion index is determined by the Equation 2.3. The EI of the soil is determined by Table 2.15.

\[ EI = \frac{\Delta H}{H_0} \times 1000 \]

Where

\[ \Delta H = H_2 - H_1 \]

\( H_2 \) is the height at the end of the test

\( H_1 \) is the height after ten minutes of consolidating under the 6.9-kPa load (dry)

\( H_0 \) is the initial height of the sample

<table>
<thead>
<tr>
<th>Expansion Index, ( EI )</th>
<th>Potential Expansion</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 – 20</td>
<td>Very Low</td>
</tr>
<tr>
<td>21 – 50</td>
<td>Low</td>
</tr>
<tr>
<td>51 – 90</td>
<td>Medium</td>
</tr>
<tr>
<td>91 – 130</td>
<td>High</td>
</tr>
<tr>
<td>&gt; 130</td>
<td>Very High</td>
</tr>
</tbody>
</table>

This standard gives the geotechnical testing and design community the ability to measure an expansion index for compacted material; however, it can become a subjective test. The test requires an operator to compact the material at 50 percent plus or minus 2 percent degree of saturation. If the proctor curve of the material is not established, it becomes a trial and error until a degree of saturation falls with 48 to 52 percent saturation. When using the trial and error method the specific gravity of soil solids is normally assumed. It is important to note that a difference of 0.05 in the specific gravity value can change the degree of saturation as much as 1.5 percent. In addition, when using the trial and error method or using a proctor curve there are several different
combinations of moisture content and dry densities that can be potentially used. Shown in Figure 2.8 is the compaction window for the expansion index test with a proctor curve.

Since the compaction energy is similar between the standard test method for expansion index of soils (ASTM D4829) and the standard test method for laboratory compaction characteristics of soil using standard effort (12,400 ft-lbf/ft$^3$) (ASTM D698) the results presented in Rosenbalm (2011) for projected related coefficients of variation for the maximum dry density and optimum moisture content are valid to use. The project related coefficients of variation for cohesive soils were 2.09 percent for the maximum dry density and 7.75 percent for the optimum moisture content (Rosenbalm 2011). With the coefficients of variation the confidence interval can be calculated. The confidence interval is shown in the following equation, Equation 2.4

$$CI_{95\%} = \mu_i \pm 1.96(\mu_i * CV)$$ ...........................................................(2.4)

Where:

$CI_{95\%}$ is the $95\%$ confidence interval

$\mu_i$ is the mean value of either the density or moisture content

CV is the coefficient of variation

One might ask, why the dry density and moisture content are important parameters when the goal of this test is achieving a degree of saturation between 48 to 52 percent? The importance of these two parameters will be outlined in the coming chapter.
2.4.2 Arizona Expansion Index

The current Arizona standard for assessing swell of natural or import soils requires a sample remolded at 95 percent of the Standard Proctor maximum density and two percent less than optimum. This is due to the minimum allowable density and moisture content that occurs in the field for compacted soils. Most soils, in the field, will be compacted at a minimum of 95 percent of the standard Proctor density and around optimum moisture content (i.e. OMC ± 2 percent).

Once the sample is remolded, the sample is subjected to 100 psf surcharge and allowed to consolidate under the load for an hour. After an hour the deformation is recorded and the inundated with water. The sample is then allowed to swell/collapse for a minimum of 24 hours or until less than 0.0002 inch per hour of movement occurs. Finally after the movement has been deemed done, the final deformation value is recorded. The
swell/collapse strain is determined by subtracting the deformation that occurred without water from the final deformation value and dividing that value by the original height. The Arizona Expansion Index states that if a remolded sample swells more than 1.5 percent, that soils cannot be used as import fill.

2.5 Empirical Correlations to Estimate Percent Heave

Presenting in the following subsections are different procedures to estimate the percentage heave. The estimation of the percent heave equations shown, unless noted otherwise are determined at a net normal stress of 6.9kPa or 1.0psi, which lead to issues when wanting to estimate “free-swell” conditions for estimation purposes. The current “free-swell” condition by ASTM D4546 is determined at a net normal stress of 1.0kPa (0.14psi). Even though there are discrepancies, it is still important to discuss the empirical evolution of the estimation of percent heave. Table 2.16 shows some empirical correlations that relate the percent swell/heave to soil index properties.
<table>
<thead>
<tr>
<th>Author</th>
<th>Year</th>
<th>Predictive Equation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Seed et al.</td>
<td>1962</td>
<td>$S = (3.6 \times 10^{-5}) (60) PI^{2.44}$</td>
</tr>
<tr>
<td>Ranganathan and Satyanayan</td>
<td>1965</td>
<td>$S = 0.000413 I_s^{2.67}$</td>
</tr>
<tr>
<td>Nayak and Christensen</td>
<td>1971</td>
<td>$S = 0.0229 PI^{1.45} \left( \frac{C}{w} \right) + 6.38$</td>
</tr>
<tr>
<td>Vijayvergiva and Ghazzaly</td>
<td>1973</td>
<td>$S = 1.1(10)^{0.0833(0.4LL - w + 5.55)}$</td>
</tr>
<tr>
<td>Schneider and Poor</td>
<td>1974</td>
<td>$S = \frac{2}{3} (10)^{0.9 PI/w - 1.19}$</td>
</tr>
<tr>
<td>Weston, D.J.</td>
<td>1980</td>
<td>$S = 0.000195 (LL)^{4.17} (w)^{-2.33}$</td>
</tr>
<tr>
<td>Chen, F.H.</td>
<td>1988</td>
<td>$S = 0.2558(e)^{0.0838(PL)}$</td>
</tr>
<tr>
<td>Basma, A.A.</td>
<td>1993</td>
<td>$S_{100} = 0.00064(PI)^{1.37} (C)^{1.37}$</td>
</tr>
<tr>
<td>Al-Shayea, N.A.</td>
<td>2001</td>
<td>$S = 0.143C \text{ when } C &lt; 20%$</td>
</tr>
<tr>
<td>Rao, A.S., Rao, S.M., and Gangadhara, S.</td>
<td>2004</td>
<td>$S = 4.24\gamma_{di} - 0.47w_i - 0.14q_i - 0.06FSI - 55$</td>
</tr>
<tr>
<td>Yilmaz, I.</td>
<td>2006</td>
<td>$S = 0.155LL - 0.00763CEC - 2.04$</td>
</tr>
<tr>
<td>Zapata et al.</td>
<td>2006</td>
<td>$S_{@100psf} = 0.2014wPI + 1.682$</td>
</tr>
<tr>
<td>Villar, and Lloret</td>
<td>2008</td>
<td>$S = \left( -12.12 \ln(\rho_{do}) + 1.89 \ln(\sigma) \right) \ln(w_o)$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$+ (36.81 \rho_{do} - 53.59)$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$+ (38.27 \ln(\rho_{do}) - 1.25) \ln(\sigma)$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$+ (-149.05 \rho_{do} + 211.42)$</td>
</tr>
<tr>
<td>Buzzi</td>
<td>2010</td>
<td>$S = 3.90 \ln \left( \frac{1}{e_o} \right)^3 \left( \frac{\sigma_o}{\sigma_{wo}} \right) - 15.5$</td>
</tr>
<tr>
<td>Lin and Cerato</td>
<td>2012</td>
<td>$S = -4.64 + 1.55(u_d - u_w)$</td>
</tr>
</tbody>
</table>

Where:

$S$ is the Swell Potential @ 6.9 kPa overburden (unless the overburden is used the predictive equation)

$PI$ is the Plasticity Index

$I_S$ is the Shrinkage Index (LL-SL)
LL is the Liquid Limit
SL is the Shrinkage Limit
C is the clay content
w is the initial water content

$S_{100}$ is the swell percent at 100% of MDD
$(u_a - u_w)$ is the matric suction

$wPI$ is the weighted Plasticity Index or $PI*P_{200}$

2.6 Factors that Affect Swell Potential of Expansive Soils

There are several factors that influence the behavior of expansive soils, which includes the stress path dependencies/overburden stress, soil structure/crumb structure, compactive effort, and climatic effects (Mitchell and Soga 2005; Snethen et al. 1975). Each of these factors, control the behavior of expansive soils differently. Presented in this section are the factors that affect swell potential of expansive soils.

2.6.1 Stress Path Dependencies/Overburden Stress

In 1962, Seed et al. presented their research finds on how changes in net normal or overburden stresses can lead to different results in percent swell. Figure 2.9a clearly shows how the swelling potential changes after unloading a sample. As one can see, three different specimens, compacted to the same initial conditions, were loaded to three different net normal stresses and inundated with water. After the swelling stopped, the samples were then unloaded and allowed to swell again. This process was continued until all the samples were at the same stress level. Their finding show there is stress path dependencies associated with sample swelling.
Figure 2.9b shows stress path dependencies from obtaining a load back swell pressure from starting at different net normal stresses. In this case, the differences, in the load back swell pressure range create a range of swell pressures from 140 kPa to 220 kPa, which is a difference of 70 kPa. Having a difference of 70 kPa in swell pressure could mean an additional 5.0 meters or 16.4 feet of material that would either expand or consolidate dependent on the applied stress state to the soil; therefore knowing the swell pressure is key in predicting/modeling field behavior of expansive soils.
a) Stress path dependencies from Seed et al. 1962.

b) Stress path dependencies from Justo et al. 1984.

Figure 2.9: Stress path dependencies.
It is interesting to note that the constant volume swell test (Method C of ASTM D4546-03) requires an increase in the overburden stress applied to the sample to maintain the same volume. After the applied stress stabilizes, the sample is then loaded to obtain the virgin compression curve and the swell pressure (following the Casa Grande construction). Then the sample is unloaded to obtain the swelling index curve $C_s$ (rebound curve for non-expansive soils) and the $C_s$ is used to obtain the swell potential at any given net normal or overburden stress; however, it is known in the geotechnical engineering community there is a stress path dependency (Seed et al. 1962; Fredlund 1995; Fredlund and Rahardjo 1993).

**2.6.2 Initial Compactive Conditions**

The initial compactive state has a large impact on the swelling percentage and swelling pressure. Figure 2.10 shows two parametric studies of how the swell potential changes when the overburden stress is kept constant and the initial dry density and moisture contents are varied or the initial compactive state is varied. Figure 2.11 shows a similar parametric study of how the varying the initial compactive effort changes the swell pressure and swell percent. The increase of swell potential, shown in Figure 2.11, is attributed to the increase of the dry density (i.e. dry density increases the void ratio decreases). As one can see, there are three different trend presented. The three trends are as follows:

1. As the dry density increases, the swell potential also increases
2. As the moisture content of the sample decreases the swell potential also increases.
3. As the void ratio decrease the swell potential of the soil will increase.
In 1988 Chen stated that out of all the factors that can influence the swell potential (i.e. overburden stress, moisture content, dry density, etc.) only the dry density of the material governs the behavior of swell potential; however, Chen evaluated only one cycle of wetting. His evaluation of multiple swell tests only looked at how a soil will behave when it is compacted at some initial state, loaded to an overburden stress, and inundate the sample with water. It is clearly shown in Figure 2.10 and 2.11 that the swell potential is affected by the dry density; however, the other influences including stress state and moisture content contribute to the swell potential.
Figure 2.10: Effects of compactive effort on swelling

a) Holtz and Gibbs 1956.

2.6.3 Crumb Structure of Soil

The crumb structure of a soil is the collection of different aggregate sizes within the sample. For a re-molded compacted sample, the maximum crumb structure is dictated by the testing standard. For most testing standards the soil passing the No.10 sieve is normally used for re-molding due to the dimensions of the testing apparatus. Nevertheless, it has been shown that the crumb structure or the size of particles used can greatly influence the results (Popescu 1980; Griffiths and Joshi 1990; Cerato et al. 2009). Popescu demonstrated the crumb structure greatly influences the swell potential and pressure. The effect of crumb structure on swell potential and pressure is shown in Figure 2.12. As one can see in Figure 2.12, when the crumb structure of the tested sample decreases, the swell potential and pressure increases, regardless of the dry density. The dry density packs the material closer together thus increasing the inter-particle force (Mitchell and Soga 2005).
Figure 2.12: Effect of crumb structure on swell potential and pressure (Popescu 1980).

Cerato et al. (2009) study the how the maximum particle size influences volume change behavior of compacted soils in the laboratory. Their research showed that the volume change behavior of compacted soils in the laboratory may be significantly different than field behavior due to the clod-size (crumb) distribution as well as the moisture state of the clods. Two different sample sizes were tested; a ring compacted samples passing No. 10 sieve and proctor compacted samples passing No. 4 sieve. The finds of the research showed the proctor compacted samples had generally lower collapse potential and free swell index than the ring compacted samples, while the preconsolidation stress was greater than ring compacted samples. Their reasoning behind the difference are due to the clumps and clods that are generated during moisture conditioning prior to compaction. It was also shown that low cohesive soils or soils with
PI < 10 and cohesionless soils the maximum size and the compactive effort did not affect the results, which indicated that there was no appreciable structure effects.

The crumb structure of soil changes during wetting and drying processes. It is apparent that the crumb structure changes during the drying process. For highly cohesive soils, the drying process causes the clay particles to clump around the larger non-clay particles as the water is displaced (Chertkov 2007). Shown in Figure 2.13, is the aggregation that occurs during the drying process for clay soils.

![Figure 2.13: Aggregate structure of a clay soil with high clay content (Chertkov 2007).](image)

Nevertheless, it has been shown that cracking is dependent on the size of the crumb structure or aggregation of soil particles that form during the drying process (Dasog et al. 1988). As the soil sample dries the soil structure changes. Shiel, et al. (1988) showed as a clay soil is subjected to wetting and drying cycles the crumb structure
of the soil would break down thus increasing the amount of fines in the soil. As the crumb structure breaks down, the aggregated clay lump in Figure 2.13 would break down creating smaller aggregated clay lumps.

2.6.4 Climatic Effects

Depending on the climate that a soil is located, it will influence the water content of a soil to a seasonal depth of wetting/drying. For an expansive soil, the seasonal fluctuations of wetting and drying drive the issue of heave and shrinkage. The seasonal fluctuations that cause the largest changes in heave and shrinkage are located in the semi-arid to arid regions of the world.

During the seasonal fluctuations, the moisture content of the soil will come into pseudo equilibrium. The pseudo equilibrium value can be measured as a suction value (negative pore water pressure) which is related to the moisture content via the soil water characteristic/retention curve or vice versa. Shown in Figure 2.14, is an idealized suction/water content schematic for climatic variations. As one can see, there is a funnel shape that occurs at the surface due to the evaporation and precipitation. The point at which the curves merge can be defined as the regional zone of influence or regional depth of wetting. The suction or moisture content below this point stays constant or has an equilibrium value; however, if construction occurs it can disrupt the regional zone of influence due to various factors.

These factors can include removal of the surficial layer and replacing the surficial layer an engineering fill (can increase the depth of the zone), covering the ground surface with on-grade construction (changes the depth of the zone within the on-grade
construction), and placing engineering fill on the ground surface (potentially raises the zone beneath the engineering fill), just to list a few factors.

![Diagram of suction schematic for climatic variations](image)

**Figure 2.14: Suction schematic for climatic variations (Fredlund and Rahardjo 1993).**

Nevertheless, the seasonal fluctuations are modeled by one of two main models. The first model uses actual climatic data and the unsaturated hydraulic conductivity function along with the SWCC to produce the moisture content/suction value within the soil layer. Normally using these functions require unsaturated modeling software that uses non-linear finite-element analysis to converge on a mathematical answer; however, the mathematical answer that software converges on can be different than actual field conditions. These issues can be remedy by using actual measured soil property values instead of default/estimated values or using mesh refinement if actual soil properties are used.

On the other hand, the other model uses Thornthwaite moisture index (TMI) to model the seasonal fluctuations of wetting and drying, which is a regional equilibrium...
type analysis. The TMI is determined using climatic data, and then the TMI is linked to the suction via soil index properties (PI, P#200). A TMI value of zero corresponds to a region that has equal evapotranspiration to precipitation. A negative TMI corresponds to a region that has a higher evapotranspiration than precipitation. A positive TMI value corresponds to a region that has less evapotranspiration than precipitation.

2.7 Volume Change Associated with Wetting and Drying Cycles of Compacted Soils

In particular, partial or full shrinkage produce different structures (Al Homoud et al., 1995; Basma et al., 1996), which give rise to differences in swelling behavior; so, for example, either amplification or reduction of swelling potential have been observed with increasing number of drying–wetting cycles (Chen, 1965; Popescu, 1980; Pousada, 1984; Dif and Bluemel, 1991; Al Homoud et al., 1995).

2.7.1 Idealized Shape Associated With Volume Change for Expansive Soils

The idealized shape associated with volume change has been topic of debate among researchers for several decades. After the early 1990’s, several researchers start to notice similar trends associated with the wetting and drying or the swelling and shrinkage behavior of expansive soils. The volume change behavior associated with swelling and shrinkage was coined the “Swelling/Shrinkage Characteristic Curve or the SSCC for short (Hanafy 1991). Figure 2.15 shows an idealized SSCC curve for expansive soils. The idealized SSCC requires multiple samples to construct the curve. Each sample is compacted to the same void ratio and moisture content and then sample is allowed swell or shrink depending on which branch of the curve is needed. The swelling portion requires a minimum of three samples. One of the samples is allowed to reach full swelling (e_f); while the other two samples are tested for a period of time and stopped to
measure the moisture content (points e_1 and e_2). On the other hand, the shrinkage curve requires a minimum of three samples as well. One of the samples is allowed to dry to a residual moisture content or a void ratio corresponding to no axial or radial movement (e_d). The other two samples are tested similar to the swelling samples without the access to water, after a period of time the samples are unloaded and the moisture content and the radial measurement are taken (e_3 and e_4). However, one must realize this is the procedural outline to obtain a SSCC for only the initial cycle and loaded at one net normal stress or overburden stress.

Figure 2.15: Idealized SSCC for expansive soils (Hanafy 1991).
Figure 2.16 shows measured SSCC curves published in various journals. As one can see, the shape of the SSCC curve can be idealized by sigmoidal curve. By idealizing the SSCC curve by sigmoidal curve, it can potential be used in modeling software as a constitutive relationship to relate the moisture content of the soil (obtained via the soil water characteristic curve) to the differential movement of the ground surface by normalizing the void ratio to the initial void ratio.
a) Measured SSCC (Ito and Azam 2010).

b) Measured SSCC (Allen 2004).

c) SSCC’s for various compacted conditions (Mirsha et al. 2008).

Figure 2.16: Example of measured SSCC found in literature.
2.7.2 Measurements

In the previous 50 years several researchers studied how expansive soils behave when the expansive soil is inundated or is given access to free water. Up until 30 years ago, researchers started to wonder how expansive soils behave under cyclic wetting and drying conditions. After several decades of wetting and drying research was conducted on expansive soils, suction controlled experiments were introduced. Suction controlled wetting and drying research started to appear in the early 1970’s and have continued to today. It has been shown, by several different researchers, that the results from wetting and drying cycles are repeatable on different samples compacted at the same initial conditions as well as in a single sample (Al-Homoud et al., 1995; Guney et al., 2007; Rao et al., 2000). The way wetting and drying cycles have been measured by researchers over the past 30 years is presented in the following sub-sections. That includes both inundation of the sample and suction controlled measurements.

2.7.2.1 Inundation of Sample (Access to Free Water)

Within in this sub-section, two different topics of measuring volume change behavior, of expansive soils, will be covered. First, the measuring technic associated with measuring the axial strain during inundation and the drying process. Secondly, the measuring technic associated with measuring volumetric strain during inundation and the drying process. Both measuring technics will lead to different answers when radial strains are present in the sample. Measureable radial strains will occur in a one dimensional test during the drying process and when the sample is swelling three dimensionally during the wetting cycle until the sample comes into contact with the confining ring.
2.7.2.1.1 Measuring Only Axial Strain

Most of the posted literature for wetting and drying cycles for compacted expansive clays only measure the axial deformation and axial strain (Al-Homoud et al. 1995; Basma et al. 1996; Day 1994; Dif and Blumel 1991; Doostmahamadi and Moosavi 2009; Guney et al. 2007; Kalkan et al. 2011; Mishra et al. 2008; Osipov et al. 1987; Popescu 1980; Rao and Revanasiddappa 2006; Subba Rao and Satayadas 1987; Sajedi et al. 2008; Tawfiq and Nalbantoglu 2009; Tripathy et al. 2002; Tripathy and Subba Rao 2009; Yazdandoust and Yarobi 2010; Zamenu et al. 2009). The axial strain that is presented was determined by normalizing the axial deformation by the initial compacted height, for cycle comparison. Their studies show empirically that the swell potential begins to converge to an equilibrium swelling potential after 4 to 6 cycles. On the other hand, the shrink potential begins to converge at an equilibrium shrinkage potential after 2 to 4 cycles. The convergence to the equilibrium shrink-swell condition is dependent on the soil type.

There were two trends that were visible in the results from the 17 different research studies that were found. The 17 different researchers, at a minimum, subjected the expansive soils to a token load of 6.9kPa for the wetting and drying cycles. The first trend showed reduction in swell potential and pressure. The second trend that was seen showed an increase in swell potential and pressure as the wetting and drying cycles progressed. Six of the 17 articles showed a reduction in both the swell pressure and potential (Al-Homoud et al. 1999; Dif and Blumel 1991; Guney et al. 2007; Kalkan et al. 2011; Tripathy and Subba Rao, 2009; Yazdandoust and Yarobi 2010). On the other hand, 11 of the 17 showed an increase of swell pressure and potential with cycles (Basma
et al. 1996; Day 1994; Doostmahammadi and Moosavi 2009; Mishra et al. 2008; Osipov et al. 1987; Popescu 1980; Rao and Revanasiddappa 2006; Subba Rao and Satayadas 1987; Sajedi et al. 2008; Tawfiq and Nalbantoglu 2009; Tripathy et al. 2002; Zamenu et al. 2009). In addition, three of the researchers tested the expansive soils at various overburden stresses (Subba Rao and Satayadas 1987; Tripathy et al. 2002 Tripathy. and Subba Rao 2009). Of those three, one showed reduction in swell pressure and potential. The increase and reduction in swell pressure will be discussed in “Differences in Partial and Full Shrinkage” sub-section. Shown in Figure 2.17 are the results of cyclic wetting and drying cycles performed by various researchers.

Furthermore, it was also shown that as an expansive soil is subjected to wetting and drying cycles, the soil index properties change including the Atterberg Limits and the grain size distribution (Al-Homound et al. 1995; Yazdandoust and Yarobi 2010). Al-Homound et al. showed a reduction in the Atterberg Limits and increase is particle size after multiple wetting and drying cycles. Yazdandoust and Yarobi on the other hand, showed an increase in the Atterberg Limits and decrease in particle size.
Figure 2.17: Changes of swell potential with cycles obtained from various researchers.
2.7.2.1.2 Measuring Volumetric Strain

Very few articles were found that the researchers measured the volumetric strain in the sample before and after the drying process. In 2000, Subba Rao et al. were the first reported researchers to publish measured volumetric strain associated with wetting and drying cycles. Since then very few researchers have recorded volumetric strain throughout the wetting and drying cycles (Tripathy et al. 2002; Tawfiq and Nalbantoglu 2009; Lin and Cerato 2013). Tripathy et al. measured the volumetric differences at the end of the cycles for comparison of axial to volumetric strain for 1-D consolidometer specimens. Their results show that the ratio between volumetric and axial strain is not a 1:1 ratio; therefore, the shrinkage is anisotropic, for both tested soils. In addition to Tripathy et al., Tawfiq and Nalbantoglu (2009) also measured volumetric strains throughout a 1-D consolidometer swell/shrinkage test. Their results also show that the volumetric to axial changes are not 1:1 as well. Figure 2.18 shows the difference in the ratio of volumetric and axial strains.

Lin and Cerato (2013), on the other hand, actually measured volumetric strains on two soils during wetting and drying cycles. The two specimens that were subject to wetting cycles were compacted and placed into a triaxial membrane with a porous stone top and bottom of the sample. The membrane applied a confinement of 7 kPa during the wetting and drying process. Throughout the wetting and drying process, radial and axial measurements were taken and translated into a volumetric strain. Their research also showed the ratio of volumetric and axial strains are not 1:1.
a) Tripathy et al. 2002.

Figure 2.18: Volumetric and axial deformation comparison from literature.
2.7.2.1.3 Differences between Partial and Full Shrinkage

Several different researchers have shown that the swelling paths after drying, for expansive soils, are different (Basma et al. 1996; Subba Rao and Satayadas 1987; Tripathy et al. 2002). The differences are due to drying the sample to a desired moisture content, either the initial moisture content or to a residual moisture content corresponding to a constant mass. For the first case the initial moisture content, it is normally known as partial shrinkage (higher than the shrinkage limit of the soil). The other case drying to a residual moisture content is known as full shrinkage (lower than the shrinkage limit of the soil).

When the soil is dried to the initial moisture content after swelling and rewetted the soil tends to have a reduction in the swell potential and swell pressure. Basma et al Subba Rao and Satayadas and Tripathy et al. showed a reduction in swell pressure and potential after the 2nd cycle when the samples are dried to the initial moisture content. After the 4th or 5th cycle the reduction in swell pressure and potential stabilize to an “equilibrium” value. Figure 2.19 shows results from these three researchers.

On the other hand, when the samples are fully dried to the residual moisture after swelling the soil tends to show an increase in swell potential at light overburden stress and a reduction in swell potential at higher overburden stresses. The increase in swell potential stabilizes after the 4th cycle for the light overburden stresses while the swell potential for the higher overburden stabilizes after the 5th or 6th cycle. The increase in swell potential after multiple cycles is shown in Figure 2.20.
Figure 2.19: Partial shrinkage reported in literature.
Figure 2.20: Partial shrinkage reported in literature.
2.7.2.1.4 Measuring Swell Pressure

Several different researchers report difference in swell pressure after multiple cycles of wetting and drying. In most cases, the swell pressure was determined using the CSVT. After a CSVT sample was tested, it was drained, unloaded and then subsequently dried to the desired moisture content. The desired moisture content for some of the researchers was the initial moisture content and the researchers used the residual moisture content.

The research study by Akcanca and Aytekin used laboratory made samples that consisted of different combinations of sand and bentonite. To assess the swell pressure after multiple cycles of wetting and drying, the samples were tested, unloaded, and dried back to the initial moisture content (partial shrinkage), using an elevated temperature of 40°C. Their researched showed a reduction in swell pressure after each cycle of wetting and drying. After four cycles, the swell pressure converged to an equilibrium value.

The research study by Kalkan et al. compared swell potential and swell pressure of remolded compacted natural material modified by silica fume. The swelling pressure was assessed by drying the sample at room temperature to the initial moisture content (partial shrinkage), under no confinement. Their results showed a reduction in the swell pressure and an equilibrium swell pressure after four cycles.

The research study by Yazdandoust and Yarobi compared the swell potential and swell pressure of remolded compacted natural material modified by polymers. The swell pressure was determined by drying the sample at room temperature to the initial moisture content (partial shrinkage), under no confinement. The results of the swell pressure test
showed a decrease in swell pressure after multiple cycles and the swell pressure stabilized after five cycles.

The research studies by Guney et al. compared the swell potential and swell pressure of remolded compacted natural material modified by polymers. The swell pressure was determined by drying the sample at room temperature to the initial moisture content (partial shrinkage), under no confinement. The results of the swell pressure test showed a decrease in swell pressure after multiple cycles and the swell pressure stabilized after five cycles.

In the four research studies presented above, the reduction in swell pressure can be attributed to the stress history the sample is subjected to during the cyclic wetting and drying. In addition, the samples were dried back to the initial moisture content, which retarded the shrinkage of the material creating a sample with similar volumetrics. It has been proven that when a sample is subjected to wetting and drying cycles the soil properties change thus leading to reduction in swelling (Al-Homound et al. 1995; Yazdandoust. and Yarobi 2010).

The research study by Osipov et al. compared swell potential and swell pressure of undisturbed natural clay samples from the same soil horizon. The swell pressure was determined by using multiple samples similar to Method A of ASTM D4546-08. The results of the swell pressure test showed an increase in swell pressure after multiple cycles. The swell pressure stabilized after four to six cycles depending on the material.

The research study by Doostmahammadi and Moosavi compared the swell potential and swell pressure of natural clay samples from the same soil horizon using the
CVST. The swell pressure was determined by drying the sample at an elevated temperature, to a residual moisture content, and under no confinement. The results of the swell pressure test showed an increase in swell pressure. The increase in swell pressure after multiple cycles is due to drying the sample to the residual moisture content and then rewetting while running the CVST. During each drying process the dry density due to the reduction in volume. In addition during the drying process, the aggregation of the clay particles would increase, which the combination of the higher dry density and particle aggregation created a higher swell pressure.

2.7.2.2 Suction Controlled

Suction controlled experiments require a modified consolidometer apparatus that is capable of applying either a total or matric suction to the specimen. There are three methods of running suction controlled experiments presented in literature. First method requires an axis translation technique. The sample is loaded in an oedometer style pressure cell and the air pressure is changed to generate desired matric suction.

The second method uses a modified oedometer that has salts or acid reservoir that controls the relative humidity within the modified oedometer (Alonso et al. 2005). Then after a period of time, the expansive soil equilibrates with the relative humidity environment or the applied total suction. The time that it takes for equilibrium with the sample is dependent on the applied total suction, and the properties of the soil.

The third method uses a modified oedometer that circulates a specialized solution that applies a total suction to the sample. The sample is covered with a semi-permeable
membrane and the specialized solution is polyethylene glycol (PEG) (Delage et al. 1998). As the PEG solution increases it increases the applied total suction to the sample.

2.8 Comparison of 3-D and 1-D Wetting and Drying Results

There will be a difference in the axial measurements when comparing the wetting results due to the following:

1. The triaxial sample will swell laterally until an equilibrium condition is reached due to the lateral swell pressure of the sample, which is lateral swell pressure is different than the axial swell pressure.

Using a specially designed apparatus, Zeitlen and Komornik (1961) and Komornik and Zeitlen (1965) found swelling pressures to be quite different in the lateral and vertical directions. The ratio of swelling pressures that was recorded can be up to 4 times greater in the lateral direction (Komornik and Zeitlen 1970.). The difference in the swell pressure, changes the principles stresses (Jim 1986). The lateral swell pressure becomes the major principle stress and the vertical swell pressure becomes the minor principle stress. The differences between the major and minor principal stresses create shear stress which could result in failure if it exceeds the shear strength of the soil (Jim 1986).

2. The laterally swelling of the triaxial sample will cause a reduction in the axial strain due to the 3-D effects of the sample. Examples are shown in Figures 2.21 and 2.22 (Al-Mhaidib 1998; Al-Shamrani and Al-Mhaidib 1999; Al-Shamrani 2000).
Figure 2.21: Conceptual differences before and after swelling of a triaxial sample

(Al-Shamrani, M.A. 2000).
a) Difference found by Al-Shamrani and Al-Mhaidib 1999.

b) Differences found by Al-Mhaidib 1998.

Figure 2.22: Differences in triaxial and oedometer axial swell.
3. The 1-D consolidometer sample will have a higher axial swell than the triaxial sample due to the 1-D sample stops swelling in the radial direction after the soil reaches the rigid boundary, which causes the application of the lateral force to the confined soil particles thus leading to axial swelling (Al-Shamrani, M.A. and Al-Mhaidib, A.I. 1999; Jim, C., 1986; Seed et al. 1962).

Swelling of an expansive soil does not necessarily lead to the generation of swelling pressure; it is exerted only when the volume increase of the soil sample is restrained (Seed et al. 1962). Seed et al. also found that the swell pressure tends to dissipate substantially if the sample volume is allowed to increase. In 1986 Jim stated that as the aggregates swell and approach each other due to confinement, the swelling pressure may gradually build up with the closure of voids and the transmission of the volume change across the sample. Swelling is not propagated as strong pressure until most of the large voids are closed. The effectiveness of the subsequent swell pressure depends on the amount of swelling potential left in the sample after the initiation of swelling pressure propagation.

4. Differences in desiccation crack formation during drying.

There is evidence of vertical and horizontal cracks formation during the drying cycle as the shrinkage process occurs and upon wetting cycle the cracks create preferential paths for water to infiltrate the sample. Figure 2.23 shows an example of the formation of vertical and horizontal cracks within the Colorado samples. The generation of the cracking in the drying phase becomes preferential wetting paths during the wetting phase by promoting swelling and closure of the cracks (Abbazadeh 2011).
a) Top view of horizontal cracks that form on the top surface.

b) Side view of the vertical cracks throughout the sample.

Figure 2.23: Shrinkage cracks that occur.
When a drying soil is restrained due to external loads applied, (1-D consolidometer) the external load restricts the volume change, which leads to anisotropic volume change. During the anisotropic volume change, the soil suction can lead to the development of tensile stresses in the restrained direction (Kodikara et al., 1999). Once the tensile stresses exceed the tensile strength of the soil, the soil tends to crack releasing the strain energy developed in the soil. After the soil cracks, the restraints placed on the soil are partially released, which allows the soil to undergo further volume change more isotropically; however, the soil suction can build up to higher tensile forces thus leading to additional cracking in the sample. This phenomenon could describe the field behavior of large desiccation cracks that are found in arid to semi-arid regions, in the superficial upper crustal layer.

On the other hand, when a drying soil is restrained laterally (horizontal directions i.e. from a triaxial test) it tends to rely on the continuity of the structural fabric to maintain zero lateral strain in the soil (Kodikara et al., 1999). Until cracking occurs, in the sample, the sample/soil profile volume change will occur in the vertical direction. Nevertheless, the amount of 1-D shrinkage prior to cracking of the soil/sample is dependent on the stiffness and/or the compressibility of the soil structure (Kodikara et al., 1999). Therefore, as a triaxial sample dries, the boundary condition stay the same in the regards of confinement, which will lead to a consolidation that is induced by shrinkage instead of inducing cracks within the sample. It is possible that the horizontal and vertical cracks are not induced due to the dissipation of tensile forces along the boundary conditions.
Therefore, with the difference shown between a 1-D and 3-D sample, a 1-D sample will be best at representing field behavior above the swell/pressure. The 3-D sample, in contrast, will be best at representing the field behavior below the swell pressure.
Chapter 3
MATERIAL CHARACTERIZATION

3.1 Soil Selection

A total of four soils were selected as candidates for this study. The soils ranged from a clayey sand to a high plastic clay based on the presumed swell potential of the soil types. The presumed swell potential wanted for the study ranged from low, to moderate, to high, to very high swell potential; however, soils with moderate to high swell potentials were eventually selected for the main study. The four soils selected were: Anthem (Arizona), Denver (Colorado), San Antonio (Texas) and San Diego (California). These soils will be referenced as Anthem, Colorado, San Antonio, and San Diego, respectively, in subsequent sections.

3.2 Soil Property Testing Information

The soils were subjected to standardized index tests along with intricate tests. The standardized testing used ASTM standards, ADOT standards and the intricate testing used standardized procedures developed by the work group. In Table 3.1 are the soil tests that were performed on Anthem, Colorado, San Antonio, and San Diego. Most of the soil tests presented in Table 3.1 were performed by the author; the soil tests that were not performed by the author will have an asterisk next to them or a citation referencing the soil properties from other students master’s or PhD’s that worked with the same soil. If the shrinkage was not measured for the soil it was estimated from Equation 3.1. For clayey soils, the shrinkage limit will be determined by subtracting the vertical distance from the A-line from 20.
\[ SL = 20 \pm \Delta p_i \] .................................................................(3.1)

Where:

SL is the shrinkage limit

\( \Delta p_i \) is the vertical distance from the A-Line

The vertical distance from the A-Line can be determined by the following equation:

\[ \Delta p_i = PI - 0.73(LL - 20) \] .................................................................(3.2)

Where:

PI is the plasticity index

LL is the Liquid Limit

**Table 3.1: Soil Test Performed and ASTM Designations.**

<table>
<thead>
<tr>
<th>Soil Test</th>
<th>ASTM Designation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Atterberg Limits</td>
<td>D4318 Standard Test Methods for Liquid Limit, Plastic Limit and Plasticity Index of Soils</td>
</tr>
<tr>
<td>Hydrometer and Sieve Analysis</td>
<td>D422 Standard Test Method for Particle Size Analysis of Soils</td>
</tr>
<tr>
<td>Shrinkage Limit</td>
<td>D4943 Standard Test Method for Shrinkage Factors of Soils by Wax Method</td>
</tr>
<tr>
<td>Soil Suction Determination</td>
<td>D6836 Standard Test Methods for Determination of the Soil Water Characteristic Curve for Desorption Using Hanging Column, Pressure Extractor, Chilled Mirror Hygrometer, or Centrifuge</td>
</tr>
<tr>
<td>Specific Gravity</td>
<td>D854 Standard test methods for specific gravity of soil solids by water Pycnometer</td>
</tr>
<tr>
<td>Standard Proctor Compaction Test</td>
<td>D698 Standard test methods for laboratory compaction characteristics of soil using standard effort (12,400 ft-lbf/ft³).</td>
</tr>
<tr>
<td>Swell Potential</td>
<td>D4546 Standard Test Methods for One-Dimensional Swell or Collapse of Cohesive Soils</td>
</tr>
<tr>
<td>USCS Soil Classification</td>
<td>D2487 Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)</td>
</tr>
</tbody>
</table>
3.3 Soil Properties

In the following subsections are the soil index properties that were measured for the four different soils. The soil index properties that were measured using the ASTM testing standards presented in Table 3.1. In measuring the swell potential, there are multiple interpretations. The interpretations will be explained in Chapter 4, “Revisiting ASTM D4546 Standard”.

3.3.1 Anthem

Shown in Table 3.2, are the basic soil index properties for the San Antonio soil. The USCS classification for this soil is a Lean Clay, which is due to the percent passing a number 200 sieve and the Atterberg Limits. The grain size distribution is shown in Figure 3.1 and the standard proctor curve is shown in Figure 3.2. The average percent heave with two standard deviations, for each point, versus the swell pressure results are shown in Figure 3.3. Each point in Figure 3.3 represents the average of a multiple tests, which the number of tests performed for each stress level is seen in Table 3.3. Figure 3.4 shows the soil water characteristic curve.
Table 3.2: Anthem Soil Index Properties.

<table>
<thead>
<tr>
<th>Particle Size Analysis</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>% Gravel</td>
<td>0.0</td>
</tr>
<tr>
<td>% Sand</td>
<td>11.3</td>
</tr>
<tr>
<td>% Silt</td>
<td>56.5</td>
</tr>
<tr>
<td>% Clay</td>
<td>32.2</td>
</tr>
</tbody>
</table>

Atterberg Limits / Consistency Limits

| Liquid Limit (%)       | 48    |
| Plastic Limit (%)      | 21    |
| Plasticity Index (%)   | 27    |
| Shrinkage Limit Measured (%) | 15 |

Other Index Properties

| Maximum Dry Density g/cm³ (lb/ft³) | 1.715 (107.0) |
| Optimum Moisture Content (%)      | 18      |
| Specific Gravity                  | 2.723   |

Soil Classification

| USCS Classification | CL |

Figure 3.1: Anthem grain size distribution.
Figure 3.2: Anthem proctor curve.

Figure 3.3: Anthem average percent heave versus net normal stress.
Table 3.3: Anthem Average Heave Test Data.

<table>
<thead>
<tr>
<th>Load</th>
<th># of Tests</th>
<th>μ %Swell</th>
<th>σ %Swell</th>
<th>2σ %Swell</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.52</td>
<td>2</td>
<td>7.40%</td>
<td>0.00%</td>
<td>0.00%</td>
</tr>
<tr>
<td>2.56</td>
<td>1</td>
<td>6.10%</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>7.65</td>
<td>2</td>
<td>3.10%</td>
<td>0.28%</td>
<td>0.57%</td>
</tr>
<tr>
<td>14.03</td>
<td>2</td>
<td>2.25%</td>
<td>0.01%</td>
<td>0.01%</td>
</tr>
<tr>
<td>20.86</td>
<td>1</td>
<td>1.80%</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>61.75</td>
<td>2</td>
<td>0.51%</td>
<td>0.21%</td>
<td>0.42%</td>
</tr>
<tr>
<td>79.98</td>
<td>3</td>
<td>0.19%</td>
<td>0.14%</td>
<td>0.28%</td>
</tr>
<tr>
<td>100.42</td>
<td>3</td>
<td>0.04%</td>
<td>0.23%</td>
<td>0.46%</td>
</tr>
<tr>
<td>144.37</td>
<td>2</td>
<td>-0.30%</td>
<td>0.03%</td>
<td>0.06%</td>
</tr>
</tbody>
</table>

Figure 3.4: Anthem soil water characteristic curve (Hashem, 2013).

3.3.2 Colorado

Shown in Table 3.4, are the basic soil index properties for the San Antonio soil. The USCS classification for this soil is a Fat Clay, which is due to the percent passing a number 200 sieve and the Atterberg Limits. The grain size distribution is shown in Figure
3.4 and the standard proctor curve is shown in Figure 3.5. The proctor curve was provided by the NSF expansive soils group as well as the Atterberg Limits of the soil. The average percent heave with two standard deviations, for each point, versus the swell pressure results are shown in Figure 3.6. Each point in Figure 3.6 represents the average of a multiple tests, which the number of tests performed for each stress level is seen in Table 3.5.

<table>
<thead>
<tr>
<th>Table 3.4: Colorado Soil Index Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Particle Size Analysis</strong></td>
</tr>
<tr>
<td>% Gravel</td>
</tr>
<tr>
<td>% Sand</td>
</tr>
<tr>
<td>% Silt</td>
</tr>
<tr>
<td>% Clay</td>
</tr>
<tr>
<td><strong>Atterberg Limits / Consistency Limits</strong></td>
</tr>
<tr>
<td>Liquid Limit (%)</td>
</tr>
<tr>
<td>Plastic Limit (%)</td>
</tr>
<tr>
<td>Plasticity Index (%)</td>
</tr>
<tr>
<td>Shrinkage Limit Measured (%)</td>
</tr>
<tr>
<td><strong>Other Index Properties</strong></td>
</tr>
<tr>
<td>Maximum Dry Density g/cm³ (lb/ft³)</td>
</tr>
<tr>
<td>Optimum Moisture Content (%)</td>
</tr>
<tr>
<td>Specific Gravity</td>
</tr>
<tr>
<td><strong>Soil Classification</strong></td>
</tr>
<tr>
<td>USCS Classification</td>
</tr>
</tbody>
</table>
Figure 3.5: Colorado grain size distribution.

Figure 3.6: Colorado proctor curve.
Figure 3.7: Colorado averaged percent heave versus net normal stress.

Table 3.5: Colorado Averaged Heave Test Data.

<table>
<thead>
<tr>
<th>Load</th>
<th># of Tests</th>
<th>( \mu ) %Swell</th>
<th>( \sigma ) %Swell</th>
<th>( 2\sigma ) %Swell</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.24</td>
<td>3</td>
<td>12.80%</td>
<td>0.20%</td>
<td>0.40%</td>
</tr>
<tr>
<td>3.83</td>
<td>2</td>
<td>10.60%</td>
<td>0.85%</td>
<td>1.70%</td>
</tr>
<tr>
<td>7.65</td>
<td>2</td>
<td>7.83%</td>
<td>0.53%</td>
<td>1.06%</td>
</tr>
<tr>
<td>20.86</td>
<td>2</td>
<td>5.49%</td>
<td>0.18%</td>
<td>0.37%</td>
</tr>
<tr>
<td>61.75</td>
<td>2</td>
<td>3.01%</td>
<td>0.25%</td>
<td>0.49%</td>
</tr>
<tr>
<td>142.74</td>
<td>2</td>
<td>0.99%</td>
<td>0.43%</td>
<td>0.86%</td>
</tr>
<tr>
<td>242.05</td>
<td>3</td>
<td>0.00%</td>
<td>0.24%</td>
<td>0.48%</td>
</tr>
<tr>
<td>382.80</td>
<td>3</td>
<td>-1.71%</td>
<td>0.38%</td>
<td>0.75%</td>
</tr>
</tbody>
</table>
Figure 3.8: Colorado soil water characteristic curve (Hashem, 2013).

3.3.3 San Antonio

Shown in Table 3.6, are the basic soil index properties for the San Antonio soil. The USCS classification for this soil is a Fat Clay, which is due to the percent passing a number 200 sieve and the Atterberg Limits. The grain size distribution is shown in Figure 3.7 and the standard proctor curve is shown in Figure 3.8. The average percent heave with two standard deviations, for each point, versus the swell pressure results are shown in Figure 3.9. Each point in Figure 3.9 represents the average of a multiple tests, which the number of tests performed for each stress level is seen in Table 3.7.
Table 3.6: San Antonio Soil Index Properties.

<table>
<thead>
<tr>
<th>Particle Size Analysis</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>% Gravel</td>
<td>0.0</td>
</tr>
<tr>
<td>% Sand</td>
<td>11.6</td>
</tr>
<tr>
<td>% Silt</td>
<td>34.7</td>
</tr>
<tr>
<td>% Clay</td>
<td>53.8</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Atterberg Limits / Consistency Limits</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Liquid Limit (%)</td>
<td>67</td>
</tr>
<tr>
<td>Plastic Limit (%)</td>
<td>24</td>
</tr>
<tr>
<td>Plasticity Index (%)</td>
<td>43</td>
</tr>
<tr>
<td>Shrinkage Limit Estimated (%)</td>
<td>12</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Other Index Properties</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Dry Density g/cm³ (lb/ft³)</td>
<td>1.609 (100.4)</td>
</tr>
<tr>
<td>Optimum Moisture Content (%)</td>
<td>22</td>
</tr>
<tr>
<td>Specific Gravity</td>
<td>2.795</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Soil Classification</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>USCS Classification</td>
<td>CH</td>
</tr>
</tbody>
</table>

Figure 3.9: San Antonio grain size distribution.
Figure 3.10: San Antonio proctor curve.

Figure 3.11: San Antonio percent heave versus net normal stress.
Table 3.7: San Antonio Averaged Heave Test Data.

<table>
<thead>
<tr>
<th>Load</th>
<th># of Tests</th>
<th>( \mu ) %Swell</th>
<th>( \sigma ) %Swell</th>
<th>2( \sigma ) %Swell</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.00</td>
<td>2</td>
<td>14.20%</td>
<td>1.70%</td>
<td>3.39%</td>
</tr>
<tr>
<td>2.24</td>
<td>1</td>
<td>11.30%</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>3.00</td>
<td>1</td>
<td>10.10%</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>7.00</td>
<td>3</td>
<td>7.33%</td>
<td>1.07%</td>
<td>2.14%</td>
</tr>
<tr>
<td>20.86</td>
<td>2</td>
<td>4.78%</td>
<td>0.04%</td>
<td>0.07%</td>
</tr>
<tr>
<td>61.75</td>
<td>2</td>
<td>1.72%</td>
<td>0.40%</td>
<td>0.81%</td>
</tr>
<tr>
<td>142.74</td>
<td>4</td>
<td>0.35%</td>
<td>0.51%</td>
<td>1.02%</td>
</tr>
<tr>
<td>242.22</td>
<td>2</td>
<td>-0.84%</td>
<td>0.76%</td>
<td>1.53%</td>
</tr>
</tbody>
</table>

3.3.4 San Diego

The basic soil index properties for the San Diego soil are shown in Table 3.8. The USCS classification for this soil is a Sandy Clay, which is due to the percent passing a number 200 sieve and the Atterberg Limits. The grain size distribution is shown in Figure 3.10 and the standard proctor curve is shown in Figure 3.11. The average percent heave with two standard deviations, for each point, versus the swell pressure results are shown in Figure 3.12. Each point in Figure 3.12 represents the average of a multiple tests, which the number of tests performed for each stress level is seen in Table 3.9.
Table 3.8: San Diego Soil Index Soil Properties (modified after Jacquemin, 2011, Abbaszadeh, 2011).

<table>
<thead>
<tr>
<th>Particle Size Analysis</th>
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<tbody>
<tr>
<td>% Gravel</td>
<td>2.1</td>
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<tr>
<td>% Sand</td>
<td>63.1</td>
</tr>
<tr>
<td>% Silt</td>
<td>29.2</td>
</tr>
<tr>
<td>% Clay</td>
<td>5.6</td>
</tr>
</tbody>
</table>

Atterberg Limits / Consistency Limits

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Liquid Limit (%)</td>
<td>41</td>
</tr>
<tr>
<td>Plastic Limit (%)</td>
<td>17</td>
</tr>
<tr>
<td>Plasticity Index (%)</td>
<td>24</td>
</tr>
<tr>
<td>Shrinkage Limit Estimated (%)</td>
<td>11</td>
</tr>
</tbody>
</table>

Other Index Properties

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Dry Density g/cm(^3) (lb/ft(^3))</td>
<td>1.741 (108.7)</td>
</tr>
<tr>
<td>Optimum Moisture Content (%)</td>
<td>17.4</td>
</tr>
<tr>
<td>Specific Gravity</td>
<td>2.722</td>
</tr>
</tbody>
</table>

Soil Classification

<table>
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<th>USCS Classification</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>SC</td>
<td></td>
</tr>
</tbody>
</table>

Figure 3.12: San Diego grain size distribution (modified after Jacquemin, S.C, 2011, Abbaszadeh, M 2011).
Figure 3.13: San Diego standard proctor curve (Jacquemin, S.C, 2011, Abbaszadeh, M 2011).

Figure 3.14: San Diego percent heave versus net normal stress.
Table 3.9: San Diego Averaged Heave Test Data.

<table>
<thead>
<tr>
<th>Load</th>
<th># of Tests</th>
<th>μ %Swell</th>
<th>σ %Swell</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.52</td>
<td>4</td>
<td>8.42%</td>
<td>0.59%</td>
</tr>
<tr>
<td>2.56</td>
<td>4</td>
<td>6.55%</td>
<td>0.34%</td>
</tr>
<tr>
<td>7.65</td>
<td>2</td>
<td>2.86%</td>
<td>0.35%</td>
</tr>
<tr>
<td>20.86</td>
<td>2</td>
<td>2.02%</td>
<td>0.32%</td>
</tr>
<tr>
<td>61.75</td>
<td>2</td>
<td>0.68%</td>
<td>0.08%</td>
</tr>
<tr>
<td>79.36</td>
<td>2</td>
<td>0.58%</td>
<td>0.07%</td>
</tr>
<tr>
<td>101.39</td>
<td>3</td>
<td>0.14%</td>
<td>0.15%</td>
</tr>
<tr>
<td>142.74</td>
<td>2</td>
<td>-0.13%</td>
<td>0.01%</td>
</tr>
<tr>
<td>242.22</td>
<td>3</td>
<td>-0.38%</td>
<td>0.09%</td>
</tr>
</tbody>
</table>
Chapter 4

REVISITING ASTM D 4546 STANDARD

4.1 Introduction

ASTM D 4546 “Standard Test Methods for One-Dimensional Swell or Collapse of Cohesive Soils” is one of many reputable standards that outline how to obtain either swell or collapse strain of soils. Without a standard of this nature, it would create an issue with researchers, practitioner, or state agencies using different methodologies to evaluate how to obtain swell or collapse strain. By having a standard, it standardizes the “most” appropriate methodology to evaluate the potential swell or collapse strain of the soil along with the standardized loading sequences, specimen particle size, and specimen dimensions. Without a standardized specimen particle size, it can lead issues with the results of the swelling or collapse of the material (Noorany and Houston 1995). In 2008 major revisions were made to the standard to best represent field construction conditions. Prior to the 2008 major revisions, the standard focused on obtaining the swell and collapse strain of cohesive soils, but in a different manor and without guidelines on certain parameters. The differences between the 2008 and prior editions/revision will be discussed in the following subsections.

Nevertheless, the primary focus of this standard is to evaluate the free swell, the swell pressure, and the magnitude of the one–dimensional swell or collapse strain of soils (ASTM 4546, 2008). The free swell, swell pressure, and the magnitude of one–dimensional swell or collapse strain at a given net normal stress (or overburden) is important to an engineer when a structure is going to be built upon soils that exhibit
expansion or collapse. If an engineer does not evaluate the magnitude of one-dimensional swell or collapse strain of a soil that exhibit irregular behavior in the presence of water, it can lead to potential damage to the structure. By having a standard that clearly outlines the procedures to measure the free swell, swell pressure, or the magnitude of one-dimensional swell or collapse strain, it gives the engineer the tools that are needed to prevent structural damage and potential lawsuits.

The standard is able to capture the swelling strain; however, even with advances with the standard, there are issues that arise when evaluating swell strain and obtaining a swell pressure. If the current methodologies are used to describe swelling strains, it is still better than prior editions, but it does not capture the entire behavior of expansive soils. Therefore, the work presented herein will discuss how the addition of more testing points will help advance the understanding of swelling strains of expansive soils.

4.2 Difference in ASTM D 4546

In 2008, the most current revision of the ASTM D 4546 was released. Prior to the 2008 revision, the ASTM standard was revised in 1990, 1996 and 2003. The previous revisions (1990 – 2003) included changes such as the addition of the summary of changes, grammatical corrections, addition to ASTM test references, addition of notes, and addition of reporting significant digits.

4.2.1 Prior to 2008

Method A obtains the swell percent/heave from a token load; heave at any given vertical pressure up to the swell pressure, and the load back swell pressure. The token pressure designated per ASTM is 1-kPa or 20-psf. The load back swell pressure is obtained by increasing the load to 5-kPa, and doubling the load thereafter until the original height of the sample is surpassed by the loading sequence. The loading sequence follows ASTM D 2346.

Method B determines the swell/collapse potential under a specified load. This method is known as the “Response to Wetting Test”. Method B states that the sample will be loaded up to the designated load and allow for consolidation to occur. Then the sample in inundated with water and the deformation is recorded. The deformation is recorded is the swelling/collapse potential is determined. After the deformation stops, the sample load is double per ASTM D 2436 until a specified load is achieved.

Method C determines the swell pressure by increasing the applied pressure on the sample to achieve zero volume change. This method is known as the “Constant Volume Swell Test” or CVST. After the applied pressure becomes stable, the sample is then loaded to higher stress to achieve sample compression. The sample is then unloaded to determine the swelling index, $C_s$. The swell pressure is determined by a modified Casa Grande construction. The Casa Grande Construction to obtain the swell pressure is shown in Figure 2.7. The $C_s$ is applied to the range of stress applied to determine the heave at any given vertical stress value.

In the standard, ASTM D18 subcommittee, state that both method A and C reproduce similar heave values seen in field condition. Method B on the other hand,
under represents the heave of field conditions (ASTM D4546 2003). This can be attributed to the consolidation at the stress state at which the sample is tested. The field sample is subjected to the stress state longer than the laboratory sample and has different initial testing conditions as well (i.e. different moisture content, dry density, void ratio, etc.). In addition, the field conditions are ever changing due to the precipitation and evaporation that occurs while the laboratory sample is dependent on the time of sample for the initial conditions.

![Graph of corrected swell pressure per ASTM D4546](image)

**Figure 4.1: Corrected swell pressure per ASTM D 4546 (ASTM D4546 2003).**

### 4.2.2 After the 2008 Version

In 2008, the D18 subcommittee completely revised the testing standard, which revised the methodologies to obtain heave and swell pressure that better simulates the one-dimensional wetting-induced volume change behavior of compacted or natural soils in the field (ASTM D 4546 2008). One of the changes in particular, describes wetting-after-loading that the subcommittee states that this test procedure “that is similar to the
first-time wetting episode of compacted fills after construction. Nevertheless, the subcommittee decided on three test methodologies to best represent field behavior. With the revision of the testing standard, the subcommittee named all the testing methodologies to describe the testing procedure.

Method A was completely changed it is now known as “wetting-after-loading tests on multiple specimens”. The test method that can be used to measure 1-D wetting induced swelling or collapsing strains over a range of varying vertical stresses. This test requires four or more identical specimens (remolded or undisturbed samples with similar densities and moisture contents) loaded to assess the heave or collapse at different overburden stresses and obtain the swell pressure of the sample. Subsequently, the main advantage of using Method A is that it follows the loading and wetting sequence that are most likely encountered in the field (Brackley 1975; El Sayed and Rabba 1986; Schreiner and Burland 1991). The results of Method A can be applied to the case where a building was built while the soil was at its natural moisture condition, and wetting of the soil occurred after completion of construction (Schreiner and Burland, 1991).

Method B likewise was completely changed; it is now known as “single point wetting-after-loading on a single specimen”. The specimen is loaded to the desired overburden stress or design stress and then the sample is wetted. The axial deformation is recorded and the swelling/collapse strain is calculated.

Method C is no longer known as the constant volume swell test; it is now known as “loading-after-wetting test”. After a sample has been subjected to wetting induced collapse/swelling the sample is then loaded to specified stress that would simulate the in-
situ stress as well as a change in stress associated with a structure or fill material. To accomplish Method C, it requires either Method A or B to be performed first to achieve the wetting induced strain.

As one can see, ASTM D 4546 changed drastically from 2003 to 2008. The changes of this standard were made so that the laboratory behavior would better represent the field behavior. The changes now give an engineer the ability to understand the swelling/collapse strain of a given strata over large ranges of net normal stresses (overburden stresses), which is needed in design or litigation after the occurrence of the heave or collapse.

4.3 Non-Linearity in the 2008 version of Method A

For determining the non-linearity of Method A four soils were selected. The four soils include Anthem, Colorado, San Antonio, and San Diego. Anthem is classified as a Lean Clay (CL), Colorado and San Antonio are classified as Fat Clays (CH), and San Diego is classified as a Sandy Clay (SC) per the USCS soil classification system. The soil properties for these four soils are located in Table 4.1.
Table 4.1: Soil Properties.

<table>
<thead>
<tr>
<th>Soil Name</th>
<th>Particle Size Analysis</th>
<th>Atterberg Limits</th>
<th>Other Soil Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>% Gravel (&gt; 4.36 mm)</td>
<td>LL (%)</td>
<td>SL (%)</td>
</tr>
<tr>
<td></td>
<td>% Sand (4.36 - 0.074 mm)</td>
<td>PL (%)</td>
<td>MDD g/cm$^3$</td>
</tr>
<tr>
<td></td>
<td>% Silt (0.074 - 0.002 mm)</td>
<td>PI (%)</td>
<td>OMC (%)</td>
</tr>
<tr>
<td></td>
<td>% Clay (&lt; 0.002 mm)</td>
<td></td>
<td>Specific Gravity</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>USCS Classification</td>
</tr>
<tr>
<td>Anthem</td>
<td>0.0</td>
<td>48</td>
<td>15</td>
</tr>
<tr>
<td>Colorado</td>
<td>0.0</td>
<td>65</td>
<td>12</td>
</tr>
<tr>
<td>San Antonio</td>
<td>0.0</td>
<td>67</td>
<td>12*</td>
</tr>
<tr>
<td>San Diego</td>
<td>2.1</td>
<td>41</td>
<td>11*</td>
</tr>
</tbody>
</table>

* Estimated using Equation 3.1

Method A of the 2008 version requires four or more identical specimens tested at different net normal stress so that the swelling/collapse strain for the material is well defined. Figure 4.2 shows the differences when obtaining results for four, six, and twelve points for the Colorado soil. All twelve points were compacted at optimum and 95 percent of the maximum dry density. In addition, for the six point plot it both the points from the four point (blue diamonds), and the red squares. The all points plot uses both the four point plots (blue diamonds) and the six point plots (red squares) along with the green triangles.
Figure 4.2: Non-linearity associated with ASTM D 4546 Method A for Colorado Soil.

Notice when more net normal stresses were added the points deviated from the semi-log regression fit. If the same semi-log regression was used to predict the swell strain at 8.2 kPa, it would over-estimate the swell strain by 3.36 percent or the semi-log regression would predicted a swelling strain of 11.18 percent instead of measured 7.38 percent. Nonetheless, this trend was exhibited for the other three soils that were tested. The non-linearity trends for the other three soils Anthem, San Antonio, and San Diego are shown in Figures 4.3, 4.4, and 4.5, respectively.
Figure 4.3: Non-linear swelling results for Anthem.

Figure 4.4: Non-linear swelling results for San Antonio.
As one can see, if the semi-log regression was used to predict the swelling within a profile for all four soils, it will over/under predict the swelling strain. For Anthem, the over prediction of swelling strain is the highest at 7.65 kPa (159.8 psf) with a difference of 1.2 percent. For San Antonio the over prediction for swelling strain is the highest at 7 kPa (146.2 psf) with a difference of 1.3 percent. For San Diego the over prediction for the swelling strain is the highest at 7.65 kPa (159.8 psf) with 1.69 percent.

However, if more soils are used in the regression, it will then under predict the lighter net normal stresses or near surface strains. Figure 4.6 shows more soils used in the regression analysis, for the Colorado soil. Due to the magnitude of the net normal stress, the larger net normal stresses have more weight in the regression analysis thus leading to the large under prediction at the lighter loads.
Even though the $R^2$ is still above 96 percent for both the 6 point and all point analyses, the near surface swelling strains are under predicted by 1.83 and 2.49 percent for the 6 point and all points analyses respectively. The under prediction at near surface when more data points are used is apparent for the other three soils; therefore, a changing the token load or a transformation of the data is needed to better predict the swelling or collapse strains when multiple data points are used, which the change in token load is outlined in section 4.4 and the data transformation is outlined in section 4.5.

4.4 Semi-Log Bimodal Analysis

As one can see from Figure 4.6 the semi-log regression analysis is highly weighted by the higher net normal stresses and the lighter stresses are not predicted with great accuracy; therefore, another token stress should be used in the analysis. Shown in
Figure 4.7 is the semi-log regression analysis separated for the token stresses and the higher stresses for the Colorado soil. As one can see, the fit for both the token stresses and the higher stresses better represent the data. Using a bimodal semi-log fit to the data set, the largest error associated with the under/over prediction of the swell/collapse strain is 0.75 percent, for the Colorado soil. When comparing to the error associated with the single semi-log regression analysis, the single semi-log regression fit, it was 2.39 percent and the bimodal regression produced an error of 0.75 percent, which reduced the error by a factor of 3.39. Shown in Figures 4.8, 4.9, and 4.10 are similar trends as the Colorado soil for the Anthem, San Antonio, and San Diego soils, respectively. The bimodal regression analyses for Anthem, San Antonio, and San Diego under/over predict the swell strain by a quarter of a percent.

It is evident from these four soils that once the net normal stresses exceeds 7 kPa the swell/collapse strain is semi-log linear, above the swell pressure; therefore the bimodal analysis is only valid within the net normal stresses that were measured. Once the net normal exceeds the swell pressure, normal soil mechanics will govern how the strain changes with the change in stress.
Figure 4.7: Bimodal semi-log regression analysis for Colorado.

Figure 4.8: Bimodal semi-log regression analysis for Anthem.
Figure 4.9: Bimodal semi-log regression analysis for San Antonio.

Figure 4.10: Bimodal semi-log regression analysis for San Diego.
4.5 Third Order Polynomial Analysis

Figure 4.11 shows the third order polynomial fit of the Colorado swelling data with a third order polynomial regression fitted to three different sets of analysis data. The analysis data is subdivided into 4, 6, and all data points.

![Third order polynomial fit for the Colorado soil.](image)

Shown in Figure 4.11, the “6 Points” and “All Points” non-linear regression analyses are very similar for the Colorado soil. Figure 4.12, 4.13, and 4.14 for Anthem, San Antonio, and San Diego, respectively. In addition, all three soils show similar trends as the Colorado soil. When comparing the swelling results for all four soils, the best fit of the third order polynomial requires swelling strains at the three light net normal stresses or token and three stress near the swell pressure loads.
The three light token net normal stresses include a “light”, an “intermediate” and a “heavy” token stress. The “light” token stress would correspond to a stress near 1 kPa, which is minimum stress that is recommended by ASTM D 4546 Method A. An “intermediate” token stress would correspond to a stress near 4.8 kPa, which 4.8 kPa corresponds to driveway/parking lot induced stress. Finally, a “heavy” token load of a stress near 7 kPa corresponds to a stress induced by a lightly load foundation or roadway.

On the other hand, the three stresses nears the swell pressure would include a stress that is at least 60 to 80 percent of the swell pressure and two stresses greater than the swell pressure. A stress of 60 to 80 percent of the swell pressure can range anywhere from 3.5 to 11.5 meters deep in the profile (e.g. the profile is homogenous) and depending on the swell pressure. For Anthem and San Diego this range is 3.5 to 5.1 meters (e.g. Lean and Sandy Clays) while Colorado and San Antonio this range is 6.0 to 11.5 meters (e.g. for Fat Clays). In addition to a stress of 60 to 80 percent of the swell pressure, additional net normal stress, for the other two points are needed to surpass the swelling pressure. The two net normal stresses greater than the swell pressure ideally range from 110 – 130 and 140 – 180 percent of the swell pressure. By having two stresses greater than the swell pressure it fully defines the swelling characteristics of an expansive soil prior to the preconsolidation pressure.
Figure 4.12: Third order polynomial fit for the Anthem soil.

Swell/Collapse Strain = -0.84(Log(σ₃))² + 3.72(Log(σ₃))² - 8.58Log(σ₃) + 8.84%
$R^2 = 1$ (4 Points)

Swell/Collapse Strain = 1.43(Log(σ₃))² + 11.72(Log(σ₃))² - 11.1Log(σ₃) + 9.33%
$R^2 = 0.9998$ (6 Points)

Swell/Collapse Strain = -1.21(Log(σ₃))² + 5.47(Log(σ₃))² - 10.84Log(σ₃) + 9.40%
$R^2 = 0.9978$ (All Points)

Figure 4.13: Third order polynomial fit for the San Antonio soil.

Swell/Collapse Strain = -1.38(Log(σ₃))² + 6.5(Log(σ₃))² - 14.02Log(σ₃) + 14.20%
$R^2 = 1$ (4 Points)

Swell/Collapse Strain = -0.51(Log(σ₃))² + 2.72(Log(σ₃))² - 9.96Log(σ₃) + 14.19%
$R^2 = 0.9996$ (6 Points)

Swell/Collapse Strain = -0.31(Log(σ₃))² + 1.97(Log(σ₃))² - 9.32Log(σ₃) + 14.23%
$R^2 = 0.9992$ (All Points)
Figure 4.14: Third order polynomial fit for the San Diego soil.

4.6 Summary, Conclusions and Recommendations to ASTM D4546

4.6.1 Summary and Conclusions

The non-linearity of the swelling/collapse strain versus net normal stress relationship associated with expansive soils was evident from the test results. Due to the magnitude of the net normal stress, the larger net normal stresses govern the regression analysis thus underpredicting the near surface strains. The non-linearity was accommodated by fitting the data to a bimodal fit and also by using a log transformation model. Out of the two analyses preformed, the use of a third order polynomial seems to be the best option since third order polynomial uses only one equation to describe the swelling data.
Based on the results obtained for the four soils, it was also apparent that if a coefficient of swelling/rebound was used to predict swelling strains; it would not have been effective in predicting strains lower than 7 kPa (or 144 psf). This conclusion can be seen in figures 4.7 through 4.10. The coefficient of swelling/rebound would have used the slope value associated with the “heavier stresses” and those models in all the soils would have underpredicted the free swell (e.g.1 kPa) by 3 to 4.5 percent, depending on the soil used. The underprediction in using the wrong model will result in a 50% error when estimating strains associated with low applied stresses.

To better predict the non-linearity of the swell/collapse strain associated with expansive soils, a log transformation of the net normal stress was proposed. The log transformation allowed a third order polynomial regression fit through the data points; which resulted in a better model to represent the non-linearity observed than the semi-log and the bimodal semi-log regression analyses.

Although the third order polynomial best fit the data when comparing the swelling results for all four soils, the best fit of the third order polynomial requires swelling strains at both ends of the spectrum of stress. The idealized spectrum of stresses includes three light net normal stresses or token stresses and three stresses near the swell pressure loads. Three token stresses that would be used for the fit are as follows: a “light” token stress, an “intermediate” token stress near 4.8 kPa, and a “heavy” token stress.

4.6.2 Recommendations

The following recommendations were derived from the analysis presented:
1) It is recommended to test additional soils (e.g. six to ten soils) to validate the non-linearity trend observed during the testing of the four soils.

2) Increase the minimum number of tests from four to six, using the Method A procedure from the 2008 edition. The 2008 edition was selected as the reference standard since it is the most current ASTM standard edition.

3) To better represent the swell/collapse strain of expansion, it is recommended to test the soil at the following net normal stresses: a light token stress (e.g. 1 kPa), an intermediate token stress (e.g. 5 kPa), a heavy token stress (e.g. 7 kPa), a stress that is 60 – 80 percent of the swell pressure, a stress that is 110 – 130 percent of the swell pressure, and a stress that is 140 – 180 percent of the swell pressure.

4) If resources allow for fewer specimens, it is recommended to test at the heaviest token stress (i.e., 7 kPa instead of 1 kPa), and at stresses near the swell pressure. Then, use the semi-log regression model to predict the strains at the stresses of interest.

5) It is recommended to log transform the swell/collapse strain data with a minimum of six data points (e.g. six different specimens loaded to six different stresses) so that a third order polynomial can be used to predict the strain range within the stresses of interest.

6) Develop new models to describe the coefficient of swelling/rebound for expansive soils subjected to low (token) and high stress levels.
Table 4.2: Recommendations for ASTM D4546.

<table>
<thead>
<tr>
<th>Criteria</th>
<th>Current ASTM D4546</th>
<th>Suggested</th>
</tr>
</thead>
<tbody>
<tr>
<td>Net Normal Stress</td>
<td>Minimum of 4</td>
<td>Minimum of 6</td>
</tr>
<tr>
<td>Net Normal Levels</td>
<td>Token and 3 others of Interest</td>
<td>1.5 kPa, 4.0 kPa, 7.0 kPa, 80% ± 10%, 120% ± 10%, and 140% ± 10% of $\sigma_{SP}$</td>
</tr>
<tr>
<td>Modeling</td>
<td>Semi-log fit or interpolate between large stresses</td>
<td>Use bi-modal natural log fit or third order polynomial fit</td>
</tr>
</tbody>
</table>
Chapter 5
DESIGN OF EXPERIMENT

5.1 Introduction

Out of the four soils that were tested at Arizona State University two soils were selected for additional testing. The additional testing that was chosen is designed to understand how expansive soils behave after multiple wetting drying cycles under various conditions. The soils tested under different applied net normal stresses, at different compactive efforts, and different moisture contents. It has been shown that the initial conditions have the largest effect on the swell potential of the soil; however, it has been shown that wetting and drying cycles will either increase the swell potential and swell pressure or the cycles will decrease the swell potential and the swell pressure (Basma et al. 1995; Al-Houmoud et al. 1996; Tripathy et al. 2002). In either case the different authors only focused on one dry density and the moisture content and varied the net normal stress.

In the main experiment, the soil type, the initial dry density, the initial moisture content, and the net normal stress will be varied. It has been proven that the dry density and moisture content influences the swell potential and swell pressure (Chen 1988; Tripathy et al. 2002). By varying the initial moisture content, the dry density, and the soil type it will show through a full factorial it will show the interaction between variables as well as which variable has the greatest influence on the swell potential and swell pressure.
5.2 Soil Selection

The soil from Denver, Colorado and Anthem Arizona were select for the main experiment. The Colorado soil is classified as a CH and the Anthem soil is classified as a CL by USCS classification. By using these two soils, it allows for the development of a framework to build upon by using additional soils that will range from Sandy Clays to Low plastic Clays and Highly plastic clays. The soil properties for the Colorado and Anthem soils are located in Table 3.4 and 3.2 respectively.

5.3 Initial Compacted Condition Selection

The initial compacted condition selection was based on the using three dry densities and three moisture contents. The percentage of the maximum dry density and percentage of the moisture content and the interaction between the two variables, for the experiment, are shown in Table 5.1.

<table>
<thead>
<tr>
<th>Initial Compacted Condition Combination</th>
<th>% of MDD</th>
<th>% of OMC</th>
</tr>
</thead>
<tbody>
<tr>
<td>Combination #1</td>
<td>100%</td>
<td>110%</td>
</tr>
<tr>
<td>Combination #2</td>
<td>100%</td>
<td>90%</td>
</tr>
<tr>
<td>Combination #3</td>
<td>90%</td>
<td>110%</td>
</tr>
<tr>
<td>Combination #4</td>
<td>90%</td>
<td>90%</td>
</tr>
<tr>
<td>Combination #5</td>
<td>95%</td>
<td>100%</td>
</tr>
</tbody>
</table>

As one can see, there are five combination generated out of the three percentages of maximum dry density and optimum moisture contents. This is due to the design of experiment cube that was developed and a graphical representation is shown in Figure
5.1. The design uses the four edge runs, with a center run to investigate the how the initial condition affect the swell percentage and swell pressure.

![Design of experiment cube.](image)

Figure 5.1: Design of experiment cube.

With the soils selected the difference in ten percent from optimum moisture content generates nearly plus or minus two percent. The densities and moisture contents that are used are shown in Table 5.2.

Table 5.2: Density and Moisture Content for the Experiment for the Two Soils.

<table>
<thead>
<tr>
<th></th>
<th>Denver, Colorado</th>
<th>Anthem, Arizona</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial Compaction</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Condition</td>
<td>Combination #1</td>
<td>Combination #1</td>
</tr>
<tr>
<td>Combination #2</td>
<td>Combination #2</td>
<td>Combination #2</td>
</tr>
<tr>
<td>Combination #3</td>
<td>Combination #3</td>
<td>Combination #3</td>
</tr>
<tr>
<td>Combination #4</td>
<td>Combination #4</td>
<td>Combination #4</td>
</tr>
<tr>
<td>Combination #5</td>
<td>Combination #5</td>
<td>Combination #5</td>
</tr>
<tr>
<td><strong>ρ (g/cm^3)</strong></td>
<td>1.648</td>
<td>1.715</td>
</tr>
<tr>
<td><strong>w%</strong></td>
<td>20.9%</td>
<td>19.8%</td>
</tr>
<tr>
<td></td>
<td>1.648</td>
<td>1.715</td>
</tr>
<tr>
<td></td>
<td>17.1%</td>
<td>16.2%</td>
</tr>
<tr>
<td></td>
<td>1.483</td>
<td>1.544</td>
</tr>
<tr>
<td></td>
<td>20.9%</td>
<td>19.8%</td>
</tr>
<tr>
<td></td>
<td>17.1%</td>
<td>16.2%</td>
</tr>
<tr>
<td></td>
<td>1.566</td>
<td>1.629</td>
</tr>
<tr>
<td></td>
<td>19.0%</td>
<td>18.0%</td>
</tr>
</tbody>
</table>
5.4 Net Normal Stresses Selection

For the main experiment, four net normal stresses were selected to determine the swell pressure and the swell percentage as a function of the net normal stress. The four net normal stresses that were chosen as follows: light “token” stress (1.5 kPa), 40%, 80%, and 140% of the swell pressure. The swell pressure in this instance is based on the initial based on the initial cycle that was completed for the following conditions: 95 percent of the MDD, and OMC. If 140% of the swell pressure does not cause the soil to consolidate, 200% of the swell pressure will be used to obtain consolidation/collapse of the soil. The net normal stresses used for the main experiment are shown in Table 5.3.

<table>
<thead>
<tr>
<th>Stress Level</th>
<th>Stress Level</th>
<th>Colorado</th>
<th>Anthem</th>
</tr>
</thead>
<tbody>
<tr>
<td>Light &quot;Token&quot;</td>
<td>1.52</td>
<td>1.52</td>
<td></td>
</tr>
<tr>
<td>Heavy &quot;Token&quot;</td>
<td>7.00</td>
<td>7.00</td>
<td></td>
</tr>
<tr>
<td>40% σ&lt;sub&gt;Sp&lt;/sub&gt;</td>
<td>100.0</td>
<td>44.0</td>
<td></td>
</tr>
<tr>
<td>60% σ&lt;sub&gt;Sp&lt;/sub&gt;</td>
<td>150.0</td>
<td>66.0</td>
<td></td>
</tr>
<tr>
<td>80% σ&lt;sub&gt;Sp&lt;/sub&gt;</td>
<td>200.0</td>
<td>88.0</td>
<td></td>
</tr>
<tr>
<td>140% σ&lt;sub&gt;Sp&lt;/sub&gt;</td>
<td>350.0</td>
<td>154.0</td>
<td></td>
</tr>
<tr>
<td>200% σ&lt;sub&gt;Sp&lt;/sub&gt;</td>
<td>500.0</td>
<td>220.0</td>
<td></td>
</tr>
</tbody>
</table>

<sup>1</sup>If net normal stress is needed to obtain the swell pressure.

<sup>2</sup>If testing time is available

5.5 Number of Cycle Selection

For the main experiment, a minimum of four cycles of wetting and drying will be accomplished. The time allotment for testing initial compacted condition combinations is 18 days. Although a minimum of four cycles are needed within the 18 days it is possible to complete six cycles. It has been shown in literature that after four to five cycles the swelling becomes asymptotic and the swell pressure starts to stabilize to a constant
pressure or within 10 to 20 kPa of the previous swell pressure of the cycle prior (Tripathy et al. 2002); therefore the goal of the experimental testing is finishing six cycles.

During the wetting cycle, the displacement will be measured with an LVDT and the percent swell or percent consolidation will be determined using the height. After each wetting cycle, the drying cycle will use the displacement measurements with the LVDT; however, once the displacement becomes asymptotic it will be assumed the soil specimen will start to shrink radially. After the drying cycle has finished the consolidometer cells will be disassembled and the soil sample will be measured to have an accurate diameter to compute the volumetric shrinkage and void ratio.

5.6 Cyclic Testing Procedure

The cyclic testing procedure is as follows:

1. Prepare the soil at the testing moisture content. Ideally soil batches will have a mass around 500.0 gram after adding water (small batch size to allow for faster equilibrium).

2. Place the soil in multiple sealable bags and allow the soil to reach equilibrium (The material was placed in a cooler for a minimum of three weeks to allow for the moisture content to reach an equilibrium condition).

3. After allowing for the soil to reach an “equilibrium” state, remove the soil from the multiple sealable bags and compact into the consolidometer rings at the desired initial compacted condition (i.e. at the test dry density multiplied by the test moisture content). If multiple sample are being prepared place each individual ring into a sealable bag so that the sample does gain or loss moisture.
4. Assemble the consolidometer cells and place on the respective consolidometer apparatuses. If the rings are in the sealable bags remove the ring prior to assembly.

5. Set the environmental chamber to 70°F.

6. Load the LabVIEW Program and input the sample information along with test length, file names for the raw data and the test data.

7. Start the LabVIEW Program.

8. Apply the desired net normal stress to the samples and zero the LVDT’s.

9. Click the record button so that the test data will start to record and inundate the samples with water.

10. Once the samples have reached the secondary swell stop the LabVIEW program.

11. Disassemble the consolidometer cells and record the mass of the ring and ring holder.

12. Remove the excess water from the porous stones using air pressure. (if this step is skipped it will increase the time required for sample shrinkage)

13. Reassemble the consolidometer cells and replace onto the consolidometer apparatus.

14. With the LabVIEW program still open, update the sample information including the moisture content, mass of the soil and ring, height of the sample, and file names for both the raw and test data.

15. Start the LabVIEW program.
16. Reapply the desired net normal stress to the samples and zero the LDVT’s and click the record button.

17. Increase the temperature of the environmental chamber from 70°F to 120°F.

18. Allow the samples shrink.

19. Once the displacement changes are relatively constant, stop the LabVIEW program and remove the net normal stress.


21. Decrease the temperature from 120°F to 70°F.

22. Repeat Steps 14 and 15.

23. After 24 hours, stop the LabVIEW program and repeat Steps 11 through 15 and inundate the samples with water.

24. Repeat Steps 10 through 22 until the desired number of cycles has been achieved.

5.7 Preparation for the Experiment

Before the main experiment started, multiple items were procured so that there would be flawless data collection throughout the testing. At the beginning of the baseline testing (95% MDD and OMC), a total of three consolidometer cells were available. Throughout the baseline testing an additional, five consolidometer cells were obtained along with two addition consolidometer apparatuses. In addition to obtaining five consolidometer cells, eight LVDT’s were procured to record the test data while the testing was ongoing; however, with the addition of the LVDT’s it required creating a program to read the raw voltage of the LVDT and convert those values into a displacement value (centimeters). The conversion for the raw voltage output of the
LVDT’s to the displacement required calibration of the system after the program was created.

5.7.1 Creation of LabVIEW Program

The creation of the LabVIEW program was an iterative process. The program version of the program was capable of measuring the raw voltage of three LVDT’s. The next version (version 1.1) of the program incorporated converting the raw voltages into displacement values. Then in version 1.2 incorporated a record option to zero out the LVDT’s and the program also calculated the following soil properties: the initial void ratio, the void ratio throughout the test, and percent swell. After completing several tests and buying an additional three consolidometer cells and LVDT’s, version 2.0 was created. Version 2.0 used the platform created in version 1.2 and expanded it to add in additional LVDT’s; however, there were some issues with this version, and version 3.0 was created to resolve the errors with version 2.0.

Finally version 3.0 was updated to the current version of the program version 4.0, which incorporated the last two consolidometer cells (found in storage) and LVDT’s (purchased). In the current setup the program reads the raw voltage from three myDAQ™. Each myDAQ™ reads the raw voltage from three LVDT’s along with the voltage supplied by the DC power supply. The myDAQ™ read the DC power supply so that each set of LVDT’s can run independently from the other set since the raw voltage from the LVDT’s uses the input DC power to convert the voltage from the LVDT to a displacement value. The use of the DC power to convert the LVDT voltage to a displacement value will be discussed in the next sub-section.
5.7.2 Calibration of LVDT’s

In the earlier versions of the LabVIEW program only one calibration was used since it was not known the DC power supply would fluctuate during the test time. The fluctuation of the DC power supply is not that large but it deemed to create a calibration curve for the LVDT’s across multiple DC power inputs. Nevertheless, in Figure 5.2 shows the original calibration that was used with Versions 1.0, 1.1, 1.2, and 2.0. When Version 3.0 was created and a trial was running, the DC power supply had a massive change in output voltage, which was due to human error; however, this human error showed that the voltage to displacement calibration was not constant across all voltages.

![Figure 5.2: Version 1.0 through 3.0 LVDT cm/V calibration.](image)

\[ y = 0.117264263x \]
\[ R^2 = 100.00\% \]
5.8 Summary and Conclusions

The development of the main experiment design and the steps followed for the preparation of the same were presented in this chapter. It was decided to conduct the experiment on two different soils (CL and CH), by subjecting the specimens to four net normal stresses: 1.5kPa, 40%, 80%, and 120% of the swell pressure. The swell pressure used for the two soils was developed from using the data presented in Chapter 4. It was also decided to compact the specimens at five different initial compaction conditions. These conditions corresponded to the following combination of maximum dry density and optimum moisture content, respectively: 90% of the optimum moisture content and 100% of the maximum dry density, 110% of the optimum moisture content and 90% of the maximum dry density, 110% of the optimum moisture content and 100% of the maximum dry density, and 100% of the optimum moisture content and 95% of the maximum dry density. The optimum moisture content and maximum dry density were determined from ASTM D698 using standard Proctor energy. Finally, the specimens are subjected to six complete wetting and drying cycles. The combination of the desired parameters allowed for the testing of 576 conditions. Additional tests to fully develop the percent swell versus net normal stress relationships, to eliminate variability associated with the testing procedure or to improve correlations were performed as needed. Based on the comprehensive literature review, the parameters chosen to conduct this study are deemed to be the most important variables to understand swell characteristics of compacted expansive soils. The results of this study are presented in Chapter 7, while the analyses of the results are presented in Chapter 8.
Chapter 6

RADIAL STRAIN AND MOISTURE CONTENT DETERMINATION

6.1 Overview

The assessment of radial strain and moisture content variation during the wetting and drying cycles is presented in this chapter. A total of eleven samples were tested for this purpose. During both the wetting and drying cycles, the samples were unloaded; and the diameter and the mass of the sample were recorded. While specimen measurements were taken, the LabVIEW program was still recording the information, which created discontinuities in the data. After the samples were measured and weighed, the specimens were reloaded onto the consolidometer and subjected to the same conditions prior to unloading. In the case of the study corresponding to the wetting cycle, the consolidometer cell was filled with water. Once all the same conditions were applied to the specimen, the LVDT capture in the LabVIEW program was re-zeroed. The LVDT was re-zeroed to allow for easy repair when the discontinuities were removed from the data. The procedure that was used to remove the discontinuities is shown in the next section. A list of the eleven samples that were used for this analysis is presented in Table 6.1. Table 6.1 shows the soil type used in the test, the initial compaction conditions (OMC and MDD), and the applied net normal stress for each of the specimens, and its corresponding swell pressure percentage.
Table 6.1: Radial Strain and Moisture Content Determination Points.

<table>
<thead>
<tr>
<th>Soil</th>
<th>Initial compaction conditions</th>
<th>$\sigma_{Net}$</th>
<th>$%\sigma_{sp}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>%OMC</td>
<td>%MDD</td>
<td></td>
</tr>
<tr>
<td>Colorado</td>
<td>90</td>
<td>90</td>
<td>54.3</td>
</tr>
<tr>
<td></td>
<td>90</td>
<td>100</td>
<td>8.2</td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>95</td>
<td>8.2</td>
</tr>
<tr>
<td></td>
<td>110</td>
<td>90</td>
<td>54.3</td>
</tr>
<tr>
<td></td>
<td>110</td>
<td>100</td>
<td>54.3</td>
</tr>
<tr>
<td>Anthem</td>
<td>90</td>
<td>90</td>
<td>21.7</td>
</tr>
<tr>
<td></td>
<td>90</td>
<td>100</td>
<td>8.24</td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>95</td>
<td>4.8</td>
</tr>
<tr>
<td></td>
<td>110</td>
<td>90</td>
<td>21.7</td>
</tr>
<tr>
<td></td>
<td>110</td>
<td>100</td>
<td>4.8</td>
</tr>
<tr>
<td></td>
<td>110</td>
<td>100</td>
<td>8.24</td>
</tr>
</tbody>
</table>

6.2 Results

At stated earlier, when the samples were removed from the consolidometers, the LabVIEW program was still recording the data causing discontinuity in the data. Shown in Figure 6.1 is an example of the discontinuity created while the sample was removed for the recording of the mass and dimensions. Once the specimen was weighed and all measurements were taken, the specimen was placed back onto the testing equipment and the same conditions were applied to the specimen. The LVDT was the re-zeroed for a new reference point.
To remove the discontinuity in data the following steps were followed:

1. Locate the discontinuity in the data set.
2. Delete the data associated with the discontinuity.
3. Select four displacement measurements prior to the discontinuity.
4. Obtain an arithmetic slope and intercept from the data using linear regression.
5. For the data points that were deleted, fit use the linear trend and generate displacement values.
6. After the discontinuity is fixed, use the last point prior to the reset of the LVDT as the adjustment point and added that value to the subsequent displacement values until the next discontinuity is reached.
7. Follow steps 1 through 6 to repair all the discontinuities in the data set.
Figure 6.2 is the same displacement data shown in Figure 6.1 but the discontinuities in the data were removed and replaced using the steps outlined above. Discontinuity removal process will be used for all 132 tests (e.g. eleven samples, six wetting, and six drying).

![Figure 6.2: Removal of discontinuities from the swelling data.](image)

After eliminating the discontinuity of the data the results obtained are presented in the following subsections. Subsection 6.2.1 will present the results from the Colorado soil and subsection 6.2.2 will cover the results obtained from the Anthem soil.

**6.2.1 Results from Colorado**

The results for the Colorado soils will be presented in the follow manner:

1. Initial compacted condition of 90% OMC – 90% MDD subjected to net normal stress of 54.3 kPa is shown in Figures 6.3 through 6.11.
2. Initial compacted condition of 90% OMC – 100% MDD subjected to net normal stress of 8.2 kPa is shown in Figure 6.12.

3. Initial compacted condition of 100% OMC – 95% MDD subjected to net normal stress of 8.2 kPa is shown in Figure 6.13.

4. Initial compacted condition of 110% OMC – 90% MDD subjected to net normal stress of 54.3 kPa is shown in Figures 6.14.

5. Initial compacted condition of 110% OMC – 100% MDD subjected to net normal stress of 54.3 kPa is shown in Figures 6.15.

The first figure, for initial compacted condition, is the axial strain results from the wetting cycle. The second figure is the radial strain results for the wetting cycles. The third figure is the axial strain results from the drying cycle. The last figure, for each initial compacted condition, is the radial strain results from the drying cycle. Subsequent figures, for the other initial compacted condition, will present a summary of the axial and volumetric strain with cycles similar to Figure 6.11.

Figure 6.3 shows the axial strain associated with the wetting cycles analyzed by the height at the beginning of the cycle. Figure 6.4 shows the axial strain associated with the wetting cycles analyzed by the as compacted height (e.g. the height of the ring). Figure 6.5 shows the volumetric strain associated with the wetting cycles analyzed by the height at the beginning of the cycle. Figure 6.6 shows the volumetric strain associated with the wetting cycles analyzed by the as compacted void ratio. Figure 6.7 shows the axial strain associated with the drying cycles analyzed by the height at the beginning of the cycle. Figure 6.8 shows the axial strain associated with the drying cycles analyzed by
the as compacted height (e.g. the height of the ring). Figure 6.9 shows the volumetric strain associated with the drying cycles analyzed by the height at the beginning of the cycle. Figure 6.10 shows the volumetric strain associated with the drying cycles analyzed by the as compacted void ratio. Shown in Figure 6.11 is the summary of both the wetting and drying cycles axial and volumetric strains for both analyses (i.e. initial compacted conditions and at the beginning of the cycle). Shown in Table 6.2 are results in a tabular form from the data shown in Figure 6.11. The matric suction presented in the table is based on having data from the drying backbone curve only (i.e. only one drying cycle). The matric suction will be slightly different for the other cycles depending on the hysteresis associated with cyclic nature. In each of the figures, the legend will have either a W and a number or a D and a number. The W in corresponds to the wetting cycle and the number corresponds to the cycle. If the legend shows a D and number, the D corresponds to the drying cycle and the number corresponds to the cycle. Tables 6.3, 6.4, and 6.5 present the results in tabular form for Figure 6.12, 6.13, and 6.14, respectively.
Figure 6.3: Wetting axial strain results for Colorado soil compacted at 90% OMC – 90% MDD analyzed by cycle.

Figure 6.4: Wetting axial strain results for Colorado soil compacted at 90% OMC – 90% MDD analyzed by as compacted.
Figure 6.5: Wetting volumetric strain results for Colorado soil compacted at 90% OMC – 90% MDD analyzed by cycle.

Figure 6.6: Wetting volumetric strain results for Colorado soil compacted at 90% OMC – 90% MDD analyzed by as compacted.
Figure 6.7: Drying axial strain results for Colorado soil compacted at 90% OMC – 90% MDD analyzed by cycle.

Figure 6.8: Drying axial strain results for Colorado soil compacted at 90% OMC – 90% MDD analyzed by as compacted.
Figure 6.9: Drying volumetric strain results for Colorado soil compacted at 90% OMC – 90% MDD analyzed by cycle.

Figure 6.10: Drying volumetric strain results for Colorado soil compacted at 90% OMC – 90% MDD analyzed by as compacted.
Figure 6.11: Summarized strain analysis for Colorado compacted at 90% OMC – 90% MDD.

Table 6.2: 90% OMC – 90% MDD – Colorado Summary Data (ID#1).

<table>
<thead>
<tr>
<th>Wetting Cycle Information</th>
<th>Cycle 0</th>
<th>Cycle 1</th>
<th>Cycle 2</th>
<th>Cycle 3</th>
<th>Cycle 4</th>
<th>Cycle 5</th>
<th>Cycle 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>w% (%)</td>
<td>17.1</td>
<td>32.1</td>
<td>30.3</td>
<td>28.5</td>
<td>28.2</td>
<td>26.8</td>
<td>26.8</td>
</tr>
<tr>
<td>%S (%)</td>
<td>54.9</td>
<td>100.0</td>
<td>100.0</td>
<td>97.5</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
</tr>
<tr>
<td>ψ (kPa)</td>
<td>1931.3</td>
<td>0.0</td>
<td>0.0</td>
<td>7.8</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>εa cycle (%)</td>
<td>0.0</td>
<td>1.26</td>
<td>4.48</td>
<td>4.55</td>
<td>4.98</td>
<td>5.50</td>
<td>5.46</td>
</tr>
<tr>
<td>εvol cycle (%)</td>
<td>0.0</td>
<td>1.26</td>
<td>14.91</td>
<td>15.03</td>
<td>13.94</td>
<td>15.64</td>
<td>17.07</td>
</tr>
<tr>
<td>εa = εvol comp (%)</td>
<td>0.0</td>
<td>1.26</td>
<td>-1.27</td>
<td>-2.88</td>
<td>-2.43</td>
<td>-5.38</td>
<td>-4.75</td>
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<table>
<thead>
<tr>
<th>Drying Cycle Information</th>
<th>Cycle 0</th>
<th>Cycle 1</th>
<th>Cycle 2</th>
<th>Cycle 3</th>
<th>Cycle 4</th>
<th>Cycle 5</th>
<th>Cycle 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>w% (%)</td>
<td>32.1</td>
<td>3.2</td>
<td>1.2</td>
<td>1.3</td>
<td>1.9</td>
<td>1.8</td>
<td>1.7</td>
</tr>
<tr>
<td>%S (%)</td>
<td>100.0</td>
<td>15.5</td>
<td>5.9</td>
<td>7.2</td>
<td>10.1</td>
<td>10.9</td>
<td>9.8</td>
</tr>
<tr>
<td>ψ (kPa)</td>
<td>0</td>
<td>41257</td>
<td>188943</td>
<td>145288</td>
<td>87943</td>
<td>76928</td>
<td>92241</td>
</tr>
<tr>
<td>εa cycle (%)</td>
<td>0.0</td>
<td>-8.65</td>
<td>-7.20</td>
<td>-9.07</td>
<td>-8.33</td>
<td>-8.56</td>
<td>-7.04</td>
</tr>
<tr>
<td>εvol cycle (%)</td>
<td>0.0</td>
<td>-16.78</td>
<td>-15.74</td>
<td>-16.12</td>
<td>-16.39</td>
<td>-17.63</td>
<td>-14.69</td>
</tr>
<tr>
<td>εa comp (%)</td>
<td>0.0</td>
<td>-7.33</td>
<td>-8.35</td>
<td>-11.66</td>
<td>-10.49</td>
<td>-13.49</td>
<td>-11.45</td>
</tr>
<tr>
<td>εvol comp (%)</td>
<td>0.0</td>
<td>-15.57</td>
<td>-16.79</td>
<td>-18.51</td>
<td>-18.36</td>
<td>-22.07</td>
<td>-18.74</td>
</tr>
</tbody>
</table>
Figure 6.12: Summarized strain analysis for Colorado compacted at 90% OMC – 100% MDD.

Table 6.3: 90% OMC – 100% MDD – Colorado Summary Data (ID#4).

<table>
<thead>
<tr>
<th>Wetting Cycle Information</th>
<th>Cycle 0</th>
<th>Cycle 1</th>
<th>Cycle 2</th>
<th>Cycle 3</th>
<th>Cycle 4</th>
<th>Cycle 5</th>
<th>Cycle 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>w% (%)</td>
<td>17.1</td>
<td>31.7</td>
<td>31.4</td>
<td>31.4</td>
<td>30.4</td>
<td>30.8</td>
<td>31.0</td>
</tr>
<tr>
<td>%S (%)</td>
<td>71.7</td>
<td>99.8</td>
<td>97.7</td>
<td>96.3</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
</tr>
<tr>
<td>ψ (kPa)</td>
<td>597.8</td>
<td>0.2</td>
<td>0.0</td>
<td>14.7</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>εa cycle (%)</td>
<td>0.0</td>
<td>12.95</td>
<td>14.04</td>
<td>13.74</td>
<td>14.83</td>
<td>15.27</td>
<td>14.24</td>
</tr>
<tr>
<td>εvol cycle (%)</td>
<td>0.0</td>
<td>12.95</td>
<td>28.59</td>
<td>25.04</td>
<td>28.30</td>
<td>27.81</td>
<td>30.91</td>
</tr>
<tr>
<td>εa = εvol comp (%)</td>
<td>0.0</td>
<td>12.95</td>
<td>13.71</td>
<td>14.96</td>
<td>16.66</td>
<td>16.98</td>
<td>18.65</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Drying Cycle Information</th>
<th>Cycle 0</th>
<th>Cycle 1</th>
<th>Cycle 2</th>
<th>Cycle 3</th>
<th>Cycle 4</th>
<th>Cycle 5</th>
<th>Cycle 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>w% (%)</td>
<td>31.7</td>
<td>4.0</td>
<td>0.7</td>
<td>1.6</td>
<td>0.9</td>
<td>4.1</td>
<td>4.1</td>
</tr>
<tr>
<td>%S (%)</td>
<td>99.8</td>
<td>23.6</td>
<td>3.5</td>
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<td>5.2</td>
<td>25.3</td>
<td>21.9</td>
</tr>
<tr>
<td>ψ (kPa)</td>
<td>0</td>
<td>17873</td>
<td>333476</td>
<td>105789</td>
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<td>15433</td>
<td>20911</td>
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<tr>
<td>εa cycle (%)</td>
<td>0.0</td>
<td>-11.09</td>
<td>-10.81</td>
<td>-11.78</td>
<td>-13.73</td>
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<td>εvol cycle (%)</td>
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<td>-20.79</td>
<td>-22.27</td>
<td>-23.84</td>
<td>-22.38</td>
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<tr>
<td>εa comp (%)</td>
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<td>0.49</td>
<td>1.40</td>
<td>0.99</td>
<td>0.53</td>
<td>0.95</td>
<td>2.41</td>
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<tr>
<td>εvol comp (%)</td>
<td>0.0</td>
<td>-10.56</td>
<td>-7.85</td>
<td>-9.32</td>
<td>-9.42</td>
<td>-11.23</td>
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Figure 6.13: Summarized strain analysis for Colorado compacted at 100% OMC – 95% MDD.

Table 6.4: 100% OMC – 95% MDD – Colorado Summary Data (ID#11).

<table>
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<tr>
<th>Wetting Cycle Information</th>
<th>Cycle 0</th>
<th>Cycle 1</th>
<th>Cycle 2</th>
<th>Cycle 3</th>
<th>Cycle 4</th>
<th>Cycle 5</th>
<th>Cycle 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>w% (%)</td>
<td>19.0</td>
<td>34.3</td>
<td>36.6</td>
<td>36.3</td>
<td>36.6</td>
<td>35.3</td>
<td>33.1</td>
</tr>
<tr>
<td>%S (%)</td>
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<td>11.69</td>
<td>10.85</td>
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Figure 6.14: Summarized strain analysis for Colorado compacted at 110% OMC – 90% MDD.

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<td>$\psi$ (kPa)</td>
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<td>12.4</td>
<td>12.4</td>
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<td>-17.35</td>
<td>-18.40</td>
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Table 6.5: 110% OMC – 90% MDD – Colorado Summary Data (ID#2).
Figure 6.15: Summarized strain analysis for Colorado compacted at 110% OMC – 100% MDD.

Table 6.6: 110% OMC – 100% MDD – Colorado Summary Data (ID#3).

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<th>Cycle 4</th>
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<th>Cycle 6</th>
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<td>w% (%)</td>
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<td>25.8</td>
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<td>100.0</td>
<td>100.0</td>
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<td>0.0</td>
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<td>0.0</td>
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<td>-11.84</td>
<td>-12.76</td>
<td>-10.80</td>
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6.2.2 Results from Anthem

The results obtained for the Anthem soil are presented in the following figures:

1. Initial compacted condition of 90% OMC – 90% MDD, subjected to net normal stress of 21.7 kPa, show in Figure 6.16

2. Initial compacted condition of 90% OMC – 100% MDD, subjected to net normal stress of 8.2 kPa show in Figure 6.17

3. Initial compacted condition of 100% OMC – 95% MDD, subjected to net normal stress of 4.8 kPa show in Figure 6.18

4. Initial compacted condition of 110% OMC – 90% MDD, subjected to net normal stress of 21.7 kPa show in Figure 6.19

5. Initial compacted condition of 110% OMC – 100% MDD, subjected to net normal stress of 4.8 kPa show in Figure 6.20

6. Initial compacted condition of 110% OMC – 100% MDD, subjected to net normal stress of 8.2 kPa show in Figure 6.21

The analysis presented for Colorado soil in the previous section applies to the results obtained for Anthem soil. Additionally, tables 6.7 through 6.12 presents the summary of the data presented in Figure 6.16 through 6.21. These tables also present the moisture content (w%), degree of saturation (S%), and matric suction (ψ) values obtained at the end of each cycle.
Figure 6.16: Summarized strain analysis for Anthem compacted at 90% OMC – 90% MDD.

Table 6.7: 90% OMC – 90% MDD – Anthem Summary Data (ID#7).

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<th>Cycle 4</th>
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Figure 6.17: Summarized strain analysis for Anthem compacted at 90% OMC – 100% MDD.

Table 6.8: 90% OMC – 100% MDD – Anthem Summary Data (ID#5).

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<th>Cycle 4</th>
<th>Cycle 5</th>
<th>Cycle 6</th>
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<td>4.5</td>
<td>5.6</td>
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<td>-1.18</td>
<td>-1.03</td>
<td>-2.91</td>
<td>3.38</td>
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Figure 6.18: Summarized strain analysis for Anthem compacted at 100% OMC – 95% MDD.

Table 6.9: 100% OMC – 95% MDD – Anthem Summary Data (ID#10).

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<td>95.1</td>
<td>95.9</td>
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<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
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<td>0.0</td>
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</tr>
<tr>
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<td>14.06</td>
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<td>5.95</td>
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<td>23.56</td>
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<td>415522</td>
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<td>851154</td>
<td>679670</td>
<td>686074</td>
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<tr>
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<td>-10.08</td>
<td>-11.75</td>
<td>-8.84</td>
<td>-10.57</td>
<td>-9.75</td>
<td>-10.78</td>
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<tr>
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<td>-18.14</td>
<td>-19.82</td>
<td>-16.64</td>
<td>-18.55</td>
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<td>-17.18</td>
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<tr>
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<td>-11.18</td>
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Figure 6.19: Summarized strain analysis for Anthem compacted at 110% OMC – 90% MDD.

Table 6.10: 110% OMC – 90% MDD – Anthem Summary Data (ID#8).

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<th>Cycle 4</th>
<th>Cycle 5</th>
<th>Cycle 6</th>
</tr>
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<td>w% (%)</td>
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<td>27.3</td>
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<td>14.1</td>
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<td>22.6</td>
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<td>93.5</td>
<td>97.3</td>
<td>95.6</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
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<tr>
<td>ψ (kPa)</td>
<td>285.2</td>
<td>13.0</td>
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<td>6.4</td>
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<td>3.92</td>
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<td>2.40</td>
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<td>8.75</td>
<td>6.93</td>
<td>7.64</td>
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<th>Cycle 2</th>
<th>Cycle 3</th>
<th>Cycle 4</th>
<th>Cycle 5</th>
<th>Cycle 6</th>
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<td>3.1</td>
<td>4.6</td>
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<td>0.7</td>
</tr>
<tr>
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<td>921198</td>
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<td>850751</td>
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<td>-5.87</td>
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<td>-9.46</td>
<td>-8.78</td>
<td>-8.40</td>
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<tr>
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<td>-8.76</td>
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Figure 6.20: Summarized strain analysis for Anthem compacted at 110% OMC – 100% MDD.

Table 6.11: 110% OMC – 100% MDD – Anthem Summary Data (ID#9).

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<th>Cycle 4</th>
<th>Cycle 5</th>
<th>Cycle 6</th>
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</thead>
<tbody>
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<td>w% (%)</td>
<td>19.8</td>
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<td>95.8</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
</tr>
<tr>
<td>ψ (kPa)</td>
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<td>0.0</td>
<td>2.7</td>
<td>5.9</td>
<td>0.0</td>
<td>0.0</td>
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<td>6.72</td>
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<td>12.40</td>
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<td>6.72</td>
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<td>23.36</td>
<td>19.74</td>
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<tr>
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<td>15.33</td>
<td>19.94</td>
<td>16.83</td>
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<th>Cycle 3</th>
<th>Cycle 4</th>
<th>Cycle 5</th>
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<td>5.6</td>
<td>2.1</td>
<td>2.3</td>
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<tr>
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<td>635872</td>
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<td>-10.78</td>
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<td>-14.44</td>
<td>-16.36</td>
<td>-17.10</td>
<td>-17.18</td>
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<tr>
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Figure 6.21: Summarized strain analysis for Anthem compacted at 110% OMC – 100% MDD.

Table 6.12: 110% OMC – 100% MDD – Anthem Summary Data (ID#6).

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<th>Cycle 4</th>
<th>Cycle 5</th>
<th>Cycle 6</th>
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<tr>
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<td>100.0</td>
<td>95.3</td>
<td>100.0</td>
<td>100.0</td>
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<td>1.5</td>
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<td>6.4</td>
<td>7.1</td>
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<td>397112</td>
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<td>-9.46</td>
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<td>-9.11</td>
<td>-8.01</td>
<td>-8.82</td>
<td>-6.69</td>
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6.3 Moisture Content and Radial Strain Analysis

Table 6.13 presents the sample ID used in Tables 6.14 through 6.18. Tables 6.14 and 6.15 are the results from the wetting analysis for Colorado and Anthem starting with the second wetting cycle. The mass of the samples for the first wetting cycle were not recorded. By not recording the first cycle of data, it allowed the sample to fully swell so that the results would be very comparable to the main study results. Nevertheless, presented in Tables 6.16, 6.17, and 6.18 are the results from the drying analysis for Colorado and Anthem. Measurements for the last drying cycle (i.e. drying cycle 6) were not presented due to loss of data during file transfer.

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Table 6.14: Wetting Cycles 2 and 3 Measurements.

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<th>t₁ w%</th>
<th>t₂ w%</th>
<th>t₃ w%</th>
<th>D₁</th>
<th>D₂</th>
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<td>6.350</td>
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<tr>
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<td>29.94%</td>
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<td>26.74%</td>
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Using the data from Tables 6.15 and 6.16, the wetting process is best represented by a natural logarithmic curve. The natural logarithmic curve was selected after fitting each individual data set with different curves and using least squares regression analysis to obtain the best fit of the data. Figure 6.22 shows an example of a natural logarithmic fit to the data. The data for the example shown in Figure 6.22 was obtained from the fourth wetting cycle of the 90% OMC – 90% MDD cube point for the Colorado soil (i.e. sample ID #1).

The data presented in Chapter 8 makes use of the natural logarithmic curve for the wetting cycle moisture content determination.

On the other hand, the drying process is best represented by an exponential curve. The exponential curve was selected after fitting the drying data with different curves and
using least square regression analysis to obtain the best fit of the data. Figure 6.23 shows an example of an exponential curve fit to the data. The data for the example shown in Figure 6.23 was obtained from the second drying cycle of the 90% OMC – 90% MDD cube point for the Colorado soil (i.e. sample ID #1). The data that is presented in Chapter 8 will use the exponential curve for the drying cycle moisture content determination. For the drying processes, it uses both normalized time and normalized moisture content; where the normalized time is determined by Equation 6.1 and the normalized moisture content is determined by Equation 6.2.

\[
Normalized \ Time = \frac{t_i}{t_f} \tag{6.1}
\]

Where:

\(t_i\) is the \(i^{th}\) time during the test

\(t_f\) is the final time or the time required to finish the test

\[
Normalized \ w\% = \frac{w_i}{w_o} \tag{6.2}
\]

Where:

\(w_i\) is the \(i^{th}\) moisture content during the test

\(w_o\) is the initial moisture content at the beginning of the test
Figure 6.23: Drying cycle moisture content determination example.

The wetting cycle radial strain measurement, on the other hand, did not have enough data points for the least square regression analysis. The number of data points is attributed to the rate at which the soil samples would swell in the radial direction. During the radial strain assessment, the material would begin to soften and stick to the calipers during the measurements. After finding out the issue with the calipers, the calipers were outfitted with damp filter paper so that the soil would not stick to the calipers during the measurement, which the addition of the filter paper to the calipers was zeroed out prior to measuring. Once the radial strain was obtained from each sample, the samples were allowed a few hours to swell, prior to the next reading. Nevertheless, the time between radial strain readings was dependent on the radial strain rate from the first reading. For some samples, the radial strain would already have reached zero prior to the
measurement; therefore, for the light net normal stresses it is safe to assume the radial strain is zero by five minutes. At heavier net normal stresses, the time that it takes for the radial strain to approach zero varies. The time that it took ranged from five minutes to fifty-five minutes; therefore, it is safe to assume that the heavier stresses will have reached zero radial strain by fifty-five minutes for the first two cycles and the five minutes for each subsequent cycle afterwards.

The drying cycle radial strain measurement, in contrast, had enough data points to accomplish a least square regression analysis. For most of the drying cycles, the soil samples followed the trend presented in Figure 6.24 where the rate of change for the radial strain was relatively constant when compared to the axial strain. Figure 6.24 shows an example of the relatively constant rate of change for the radial strain. The data for this example was obtained from the fifth drying cycle of the 90% OMC – 90% MDD cube point for the Colorado soil (i.e. sample ID #1). Nevertheless, there were a few soils samples that did not follow the relatively constant rate of strain, which an example is shown in Figure 6.25. This example used the data obtained from the fifth drying cycle 90% OMC – 90% MDD cube point for Anthem (i.e. sample ID #7).
Figure 6.24: Relatively constant rate of change for the radial strain.

Figure 6.25: Non-constant rate of change for the radial strain.
The non-constant rate of change shown in Figure 6.25 can be attributed to the tensile cracks that formed in the sample. As the matric suction (i.e. as the moisture content decreases) within the sample increases it create large tensile forces that once the tensile forces exceed the tensile strength the sample will release the tensile stresses in forms of cracks (Kodikara et al., 1999). The samples that exhibited the non-constant rate of change for the radial strain developed larger and larger cracks as the matric suction increased (i.e. sample dried out). Figure 6.26 shows two different samples during the drying cycle; one that exhibited a constant rate of change (i.e. sample ID #6) shown in Figure 6.26a and a sample that exhibited the non-constant rate of change (i.e. sample ID #7) shown in Figure 6.26b. The non-constant rate of change for the radial strain can be attributed to the formation of the cracks. If the cracks were forced closed, it is easy to postulate the rate of change for the radial strain would be relatively constant; therefore, it is safe to assume a constant rate of change for the radial strain for all the specimens. By assuming a constant rate of change for the radial strain produces a slight error when plotting the void ratio versus the moisture content as seen in Figure 6.27.
a) Sample that exhibited constant rate of change for the radial strain.

b) Sample that exhibited non-constant rate of change for the radial strain.

Figure 6.26: Sample comparison for radial strain.
Figure 6.27: Differences when using estimated data for non-constant strain.

6.4 Summary, Conclusions, and Recommendations

6.4.1 Summary and Conclusions

The results of the assessment of the evolution of the radial strain and moisture content changes during cycles of wetting and drying were presented in this chapter. This assessment was necessary and very valuable in order to estimate intermediate volumetric strain results from the main study presented in Chapter 7. The assessment of the moisture content variation within the cycles, on the other hand, was deemed useful for the analysis presented in Chapter 8.

The moisture content variation during the wetting and drying cycles was found to follow two different non-linear paths. The path followed during the wetting cycle was found to be best represented by a natural logarithmic function; while the path followed
during the drying cycle was found to be best represented by an exponential curve. Nevertheless, the data presented in Chapter 8 will only have measured moisture contents at the beginning and the end of test, and these two points will be used to generate the non-linear fit of the data determined in this chapter.

The radial strain variation during the wetting and drying cycles was observed and recorded for the 11 specimens tested. For all samples subjected to the wetting cycle, it was observed that the radial strain became zero within the first five minutes of loading, for light net normal stresses. That made the volumetric strain equal to the axial strain for the wetting cycles. In addition, it was also observed that for higher net normal stresses, the radial strain became zero within the first 55 minutes of loading/inundation for all specimens tested. However, the time it took to reach zero radial strain ranged from 5 to 55 minutes. Nevertheless, it is safe to assume that the radial strain rate will approach zero within the first five minutes of the wetting cycle, for the heavier net normal stress applied.

The radial strain change observed during the drying process allowed enough data points to be recorded; and therefore, it was possible to determine a best fit function through the data by using a least square regression analysis. It was determined that when the radial strain is divided by the axial strain, at any measured point during the cycle, it resulted in nearly a constant ratio. This finding allowed for the use of a constant ratio of the radial to axial strain for the main study analysis presented in Chapter 8.
6.4.2 Recommendations

1. The moisture content variation with respect to time during a wetting cycle can be best represented by a natural logarithmic function fitted through the starting and ending moisture content points. It is recommended to use 0.01 minutes as the starting time.

2. The moisture content variation with respect to time during a drying cycle can be best represented by fitting the starting and ending moisture content points with an exponential function. It is recommended to use 0.01 minutes as the starting time.

3. During a ring-confined oedometer test, the radial strain observed during a wetting cycle became zero within the first 5 minutes of loading/inundation, for light loading. Therefore, it is recommended to assume the radial strain, for lighter loads, approaches zero after five minutes after the test starts. For samples with larger loads, the radial strain observed during a wetting cycle became zero ranging between 5 and 55 minutes of the loading/inundation. Therefore, it is recommended to assume the radial strain, for larger loads, approaches zero prior to 55 minutes.

4. The radial strain variation during a drying cycle was found to be constant with respect to the axial strain. Thus, it is recommended to use a constant rate of change for the radial strain. The radial strain throughout a drying test can be determined by the following equation, Equation 6.3

\[ \varepsilon_{r_i} = \left( \frac{\varepsilon_{r_f}}{\varepsilon_{a_f}} \right) \varepsilon_{a_i} \]  

\[ \text{..........................................................(6.3)} \]
Where:

\[ \varepsilon_{ri} \] is the radial strain at the \( i^{th} \) time during the test

\[ \varepsilon_{rf} \] is the radial strain measured at the end of a test

\[ \varepsilon_{ai} \] is the axial strain at the \( i^{th} \) time during the test

\[ \varepsilon_{af} \] is the axial strain measured at the end of a test
Chapter 7

LABORATORY RESULTS OF EXPANSIVE SOILS SUBJECTED TO WETTING AND DRYING CYCLES

7.1 Overview

This chapter presents the strain results obtained after subjecting two expansive soils to wetting and drying cycles. The specimens were compacted to five different initial conditions and subjected to different net normal stresses. Only the summary of the laboratory results will be presented in this chapter. The analysis of the results will be presented in Chapter 8. The summary figures are presented first followed by tables with tabulated results. The tables present the following data obtained during the testing program: moisture content, degree of saturation, matric suction (based on the first drying cycle), the axial strain obtained during each cycle, and the volumetric strain. This information is presented for each initial compaction condition and for both wetting and drying cycles.

7.2 Colorado Soil

The results obtained from the tests performed on the Colorado soil are presented in the following sections:

1) Results from the initial compacted condition of 90% OMC – 90% MDD (Section 7.2.1)

2) Results from the initial compacted condition of 110% OMC – 90% MDD (Section 7.2.2)
3) Results from the initial compacted condition of 100% OMC – 95% MDD (Section 7.2.3)

4) Results from the initial compacted condition of 90% OMC – 100% MDD (Section 7.2.4)

5) Results from the initial compacted condition of 110% OMC – 100% MDD (Section 7.2.5)

7.2.1 90% OMC – 90% MDD

Figure 7.1 presents the results obtained for the Colorado specimen compacted at 90% OMC and 90% MDD, and subjected to a net normal stress of 1.5 kPa.

Figure 7.1: Strain results for Colorado soil compacted at 90% OMC–90% MDD and subjected to a net normal stress of 1.5kPa.
Figure 7.2: Strain results for Colorado soil compacted at 90% OMC–90%MDD subjected to a net normal stress of 100kPa.

Figure 7.3: Strain results for Colorado soil compacted at 90% OMC–90%MDD subjected to a net normal stress of 251kPa.
Figure 7.4: Strain results for Colorado soil compacted at 90% OMC–90% MDD subjected to a net normal stress of 357 kPa.

Table 7.1: Results for Colorado Compacted at 90% OMC–90% MDD $\sigma_{\text{net}} = 1.5$ kPa.

<table>
<thead>
<tr>
<th>Wetting Cycle Information</th>
<th>Cycle 0</th>
<th>Cycle 1</th>
<th>Cycle 2</th>
<th>Cycle 3</th>
<th>Cycle 4</th>
<th>Cycle 5</th>
<th>Cycle 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>$w_%$ (%)</td>
<td>16.9</td>
<td>40.7</td>
<td>49.2</td>
<td>49.9</td>
<td>48.9</td>
<td>49.7</td>
<td>50.6</td>
</tr>
<tr>
<td>%S (%)</td>
<td>55.6</td>
<td>99.0</td>
<td>99.7</td>
<td>99.6</td>
<td>99.9</td>
<td>99.7</td>
<td>100.0</td>
</tr>
<tr>
<td>$\psi$ (kPa)</td>
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<td>1.7</td>
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<td>0.3</td>
<td>0.0</td>
<td>0.2</td>
<td>0.0</td>
</tr>
<tr>
<td>$\varepsilon_{a\text{ cycle}}$ (%)</td>
<td>0.0</td>
<td>15.84</td>
<td>24.52</td>
<td>23.33</td>
<td>15.92</td>
<td>17.26</td>
<td>17.20</td>
</tr>
<tr>
<td>$\varepsilon_{\text{vol cycle}}$ (%)</td>
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<td>15.84</td>
<td>43.71</td>
<td>46.01</td>
<td>33.36</td>
<td>34.61</td>
<td>35.72</td>
</tr>
<tr>
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<td>15.84</td>
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<td>27.39</td>
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<td>Cycle 3</td>
<td>Cycle 4</td>
<td>Cycle 5</td>
<td>Cycle 6</td>
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<td>6.0</td>
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<td>1.4</td>
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<tr>
<td>%S (%)</td>
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<td>$\varepsilon_{a\text{ cycle}}$ (%)</td>
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<td>-12.76</td>
<td>-12.91</td>
<td>-12.26</td>
<td>-11.14</td>
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<td>-26.52</td>
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<td>-24.13</td>
<td>-23.78</td>
<td>-22.25</td>
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<td>12.95</td>
<td>15.41</td>
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</table>
Table 7.2: Results for Colorado Compacted at 90%OMC–90% MDD $\sigma_{\text{Net}} = 100\text{kPa}$.

<table>
<thead>
<tr>
<th>Wetting Cycle Information</th>
<th>Cycle 0</th>
<th>Cycle 1</th>
<th>Cycle 2</th>
<th>Cycle 3</th>
<th>Cycle 4</th>
<th>Cycle 5</th>
<th>Cycle 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>$w$ % (%)</td>
<td>16.7</td>
<td>29.0</td>
<td>27.2</td>
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<td>25.5</td>
<td>24.4</td>
<td>24.4</td>
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<tr>
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<td>96.6</td>
<td>95.7</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
</tr>
<tr>
<td>$\psi$ (kPa)</td>
<td>1843.7</td>
<td>11.3</td>
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</tr>
<tr>
<td>$\varepsilon_a$ cycle (%)</td>
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<tr>
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<td>0.01</td>
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<td>9.26</td>
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<td>-5.07</td>
<td>-6.87</td>
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</table>

Table 7.3: Results for Colorado Compacted at 90%OMC–90% MDD $\sigma_{\text{Net}} = 251\text{kPa}$.

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<th>Wetting Cycle Information</th>
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<th>Cycle 4</th>
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<tbody>
<tr>
<td>$w$ % (%)</td>
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<td>7.6</td>
</tr>
<tr>
<td>%S (%)</td>
<td>96.8</td>
<td>76.7</td>
<td>40.3</td>
<td>23.1</td>
<td>54.8</td>
<td>23.6</td>
<td>43.8</td>
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<tr>
<td>$\psi$ (kPa)</td>
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<td>393</td>
<td>5025</td>
<td>18745</td>
<td>1945</td>
<td>17983</td>
<td>3961</td>
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<td>$\varepsilon_a$ cycle (%)</td>
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<td>-6.57</td>
<td>-5.38</td>
<td>-5.37</td>
<td>-5.69</td>
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<tr>
<td>$\varepsilon_{\text{vol}}$ cycle (%)</td>
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<td>-6.99</td>
<td>-21.94</td>
<td>-11.68</td>
<td>-9.95</td>
<td>-10.44</td>
<td>-10.24</td>
</tr>
<tr>
<td>$\varepsilon_a = \varepsilon_{\text{vol}}$ comp (%)</td>
<td>0.0</td>
<td>-3.66</td>
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Table 7.2: Results for Colorado Compacted at 90%OMC–90% MDD $\sigma_{\text{Net}} = 100\text{kPa}$.

<table>
<thead>
<tr>
<th>Wetting Cycle Information</th>
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<tbody>
<tr>
<td>$w$ % (%)</td>
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<td>26.9</td>
<td>24.1</td>
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<td>22.1</td>
<td>21.3</td>
<td>21.5</td>
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<tr>
<td>%S (%)</td>
<td>54.0</td>
<td>96.9</td>
<td>91.5</td>
<td>95.5</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
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<tr>
<td>$\psi$ (kPa)</td>
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<td>$\varepsilon_a$ cycle (%)</td>
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<td>2.91</td>
<td>3.50</td>
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<tr>
<td>$\varepsilon_{\text{vol}}$ cycle (%)</td>
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<td>$\varepsilon_a = \varepsilon_{\text{vol}}$ comp (%)</td>
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Table 7.3: Results for Colorado Compacted at 90%OMC–90% MDD $\sigma_{\text{Net}} = 251\text{kPa}$.

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</thead>
<tbody>
<tr>
<td>$w$ % (%)</td>
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<td>13.1</td>
<td>3.9</td>
<td>3.4</td>
<td>4.5</td>
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<td>18.8</td>
<td>26.3</td>
<td>24.2</td>
<td>33.6</td>
</tr>
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<td>$\psi$ (kPa)</td>
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Table 7.4: Results for Colorado Compacted at 90% OMC–90% MDD $\sigma_{\text{Net}} = 357\text{kPa}$.

<table>
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<th>Cycle 4</th>
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<th>Cycle 6</th>
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</thead>
<tbody>
<tr>
<td>$w%$ (%)</td>
<td>16.6</td>
<td>25.3</td>
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<td>21.0</td>
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<td>20.1</td>
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<td>$%S$ (%)</td>
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<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
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<tr>
<td>$\psi$ (kPa)</td>
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<td>8.4</td>
<td>16.7</td>
<td>4.5</td>
<td>0.0</td>
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<tr>
<td>$\varepsilon_{\text{a cycle}}$ (%)</td>
<td>0.0</td>
<td>-5.75</td>
<td>1.15</td>
<td>3.21</td>
<td>3.59</td>
<td>3.93</td>
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<tr>
<td>$\varepsilon_{\text{vol cycle}}$ (%)</td>
<td>0.0</td>
<td>-5.75</td>
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<td>6.54</td>
<td>6.20</td>
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<td>2369</td>
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<td>-8.11</td>
<td>-6.84</td>
<td>-7.29</td>
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<tr>
<td>$\varepsilon_{\text{a comp}}$ (%)</td>
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7.2.2 110% OMC – 90% MDD

Figure 7.5: Strain results for Colorado soil compacted at 110% OMC–90% MDD subjected to a net normal stress of 1.5 kPa.
Figure 7.6: Strain results for Colorado soil compacted at 110% OMC–90% MDD subjected to a net normal stress of 100 kPa.

Figure 7.7: Strain results for Colorado soil compacted at 110% OMC–90% MDD subjected to a net normal stress of 251 kPa.
Figure 7.8: Strain results for Colorado soil compacted at 110%OMC–90%MDD subjected to a net normal stress of 357 kPa.

Table 7.5: Results for Colorado Compacted at 110%OMC–90% MDD $\sigma_{\text{Net}} = 1.5$ kPa.

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<td>37.7</td>
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<td>50.4</td>
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<td>98.8</td>
<td>97.3</td>
<td>94.4</td>
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<td>100.0</td>
<td>100.0</td>
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<td>20.54</td>
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<td>29.82</td>
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<td>14.84</td>
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<td>8.84</td>
<td>9.92</td>
<td>9.44</td>
<td>8.39</td>
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Table 7.7: Results for Colorado compacted at 110\% OMC–90\% MDD \( \sigma_{\text{Net}} = 208\text{kPa} \).
Table 7.8: Results for Colorado Compacted at 110% OMC–90% MDD \( \sigma_{\text{Net}} = 357 \text{kPa} \).

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<td>( % S ) (%)</td>
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<td>100.0</td>
<td>100.0</td>
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Drying Cycle Information

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7.2.3 100% OMC – 95% MDD

![Figure 7.9](image.png)

**Figure 7.9:** Strain results for Colorado soil compacted at 100% OMC–95% MDD subjected to a net normal stress of 1.5kPa.
Figure 7.10: Strain results for Colorado soil compacted at 100% OMC–95% MDD subjected to a net normal stress of 100kPa.

Figure 7.11: Strain results for Colorado soil compacted at 100% OMC–95% MDD subjected to a net normal stress of 208kPa.
Figure 7.12: Strain results for Colorado soil compacted at 100% OMC–95% MDD subjected to a net normal stress of 357kPa.

Table 7.9: Results for Colorado Compacted at 100% OMC–95% MDD $\sigma_{\text{Net}} = 1.5$ kPa.

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Table 7.10: Results for Colorado Compacted at 100% OMC−95% MDD $\sigma_{Net}=100kPa$.

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Table 7.11: Results for Colorado Compacted at 100% OMC−95% MDD $\sigma_{Net}=251kPa$.

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<td>98.8</td>
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<td>2.71</td>
<td>2.76</td>
<td>2.85</td>
<td>2.83</td>
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Table 7.12: Results for Colorado Compacted at 100% OMC–95% MDD $\sigma_{\text{Net}}=357$ kPa.

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<th>Cycle 6</th>
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<td>99.4</td>
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<td>97.0</td>
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<td>0.0</td>
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7.2.4 90% OMC – 100% MDD

Figure 7.13: Strain results for Colorado soil compacted at 90% OMC–100% MDD subjected to a net normal stress of 1.5kPa.
Figure 7.14: Strain results for Colorado soil compacted at 90% OMC–100% MDD subjected to a net normal stress of 53kPa.

Figure 7.15: Strain results for Colorado soil compacted at 90% OMC–100% MDD subjected to a net normal stress of 100kPa.
Figure 7.16: Strain results for Colorado soil compacted at 90% OMC–100% MDD subjected to a net normal stress of 208kPa.

Figure 7.17: Strain results for Colorado soil compacted at 90% OMC–100% MDD subjected to a net normal stress of 357kPa.
Figure 7.18: Strain results for Colorado soil compacted at 90% OMC–100% MDD subjected to a net normal stress of 461kPa.

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<td>100.0</td>
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<td>99.9</td>
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<td>0.0</td>
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<td>5.8</td>
<td>5.4</td>
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Table 7.14: Results for Colorado Compacted at 90% OMC–100% MDD $\sigma_{\text{Net}}=53$ kPa.

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Table 7.15: Results for Colorado Compacted at 90% OMC–100% MDD $\sigma_{\text{Net}}=100$ kPa.

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Table 7.14: Results for Colorado Compacted at 90% OMC–100% MDD $\sigma_{\text{Net}}=53$ kPa.

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<td>$\psi$ (kPa)</td>
<td>649.1</td>
<td>5.3</td>
<td>4.8</td>
<td>45.7</td>
<td>11.4</td>
<td>14.5</td>
<td>0.0</td>
</tr>
<tr>
<td>$\varepsilon_a$ cycle (%)</td>
<td>0.0</td>
<td>3.27</td>
<td>4.86</td>
<td>6.91</td>
<td>8.26</td>
<td>7.50</td>
<td>7.96</td>
</tr>
<tr>
<td>$\varepsilon_{\text{vol}}$ cycle (%)</td>
<td>0.0</td>
<td>3.27</td>
<td>2.07</td>
<td>2.82</td>
<td>1.58</td>
<td>0.32</td>
<td>-0.21</td>
</tr>
<tr>
<td>$\varepsilon_a = \varepsilon_{\text{vol}}$ comp (%)</td>
<td>0.0</td>
<td>3.27</td>
<td>2.07</td>
<td>2.82</td>
<td>1.58</td>
<td>0.32</td>
<td>-0.21</td>
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</table>

Table 7.15: Results for Colorado Compacted at 90% OMC–100% MDD $\sigma_{\text{Net}}=100$ kPa.

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<th>Cycle 5</th>
<th>Cycle 6</th>
</tr>
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<tbody>
<tr>
<td>w% (%)</td>
<td>25.3</td>
<td>15.5</td>
<td>3.8</td>
<td>4.3</td>
<td>3.9</td>
<td>3.4</td>
<td>7.2</td>
</tr>
<tr>
<td>%S (%)</td>
<td>98.0</td>
<td>80.9</td>
<td>23.2</td>
<td>22.7</td>
<td>22.4</td>
<td>20.4</td>
<td>42.4</td>
</tr>
<tr>
<td>$\psi$ (kPa)</td>
<td>5</td>
<td>263</td>
<td>18548</td>
<td>19396</td>
<td>20084</td>
<td>24359</td>
<td>4343</td>
</tr>
<tr>
<td>$\varepsilon_a$ cycle (%)</td>
<td>0.0</td>
<td>-7.68</td>
<td>-7.93</td>
<td>-6.79</td>
<td>-8.12</td>
<td>-7.62</td>
<td>-7.23</td>
</tr>
<tr>
<td>$\varepsilon_{\text{vol}}$ cycle (%)</td>
<td>0.0</td>
<td>-10.52</td>
<td>-12.34</td>
<td>-10.53</td>
<td>-11.45</td>
<td>-11.66</td>
<td>-10.99</td>
</tr>
<tr>
<td>$\varepsilon_a$ comp (%)</td>
<td>0.0</td>
<td>-4.66</td>
<td>-7.73</td>
<td>-3.95</td>
<td>-6.50</td>
<td>-7.33</td>
<td>-7.44</td>
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<tr>
<td>$\varepsilon_{\text{vol}}$ comp (%)</td>
<td>0.0</td>
<td>-7.59</td>
<td>-12.15</td>
<td>-7.80</td>
<td>-9.89</td>
<td>-11.38</td>
<td>-11.19</td>
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Table 7.16: Results for Colorado Compacted at 90% OMC–100% MDD $\sigma_{\text{Net}}=251\text{kPa}$.

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<th>Cycle 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>$w$ (%) (%)</td>
<td>16.8</td>
<td>23.3</td>
<td>22.9</td>
<td>21.9</td>
<td>21.6</td>
<td>21.3</td>
<td>18.8</td>
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<tr>
<td>$%S$ (%) (%)</td>
<td>75.5</td>
<td>100.0</td>
<td>100.0</td>
<td>99.8</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
</tr>
<tr>
<td>$\psi$ (kPa)</td>
<td>435.9</td>
<td>0.0</td>
<td>0.0</td>
<td>0.1</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>$\varepsilon_{a\text{ cycle}}$ (%)</td>
<td>0.0</td>
<td>1.56</td>
<td>3.77</td>
<td>4.13</td>
<td>4.19</td>
<td>4.34</td>
<td>4.27</td>
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<td>$\varepsilon_{\text{vol cycle}}$ (%)</td>
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<td>1.56</td>
<td>7.39</td>
<td>7.14</td>
<td>7.18</td>
<td>7.20</td>
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<td>$\varepsilon_{a=\varepsilon_{\text{vol comp}}}$ (%)</td>
<td>0.0</td>
<td>1.56</td>
<td>-0.90</td>
<td>-0.29</td>
<td>-2.25</td>
<td>-3.91</td>
<td>-5.75</td>
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Drying Cycle Information

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<th>Cycle 3</th>
<th>Cycle 4</th>
<th>Cycle 5</th>
<th>Cycle 6</th>
</tr>
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<tr>
<td>$w$ (%) (%)</td>
<td>23.3</td>
<td>8.5</td>
<td>3.5</td>
<td>3.5</td>
<td>3.8</td>
<td>3.2</td>
</tr>
<tr>
<td>$%S$ (%) (%)</td>
<td>100.0</td>
<td>45.5</td>
<td>20.1</td>
<td>20.3</td>
<td>23.7</td>
<td>20.7</td>
</tr>
<tr>
<td>$\psi$ (kPa)</td>
<td>0</td>
<td>3560</td>
<td>24945</td>
<td>24557</td>
<td>17736</td>
<td>23538</td>
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<tr>
<td>$\varepsilon_{a\text{ cycle}}$ (%)</td>
<td>0.0</td>
<td>-6.46</td>
<td>-5.47</td>
<td>-5.56</td>
<td>-5.39</td>
<td>-5.54</td>
</tr>
<tr>
<td>$\varepsilon_{\text{vol cycle}}$ (%)</td>
<td>0.0</td>
<td>-9.56</td>
<td>-7.84</td>
<td>-8.05</td>
<td>-7.73</td>
<td>-7.79</td>
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<tr>
<td>$\varepsilon_{a=\varepsilon_{\text{vol comp}}}$ (%)</td>
<td>0.0</td>
<td>-2.76</td>
<td>-4.80</td>
<td>-5.84</td>
<td>-7.52</td>
<td>-9.23</td>
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</table>

Table 7.17: Results for Colorado Compacted at 90% OMC–100% MDD $\sigma_{\text{Net}}=367\text{kPa}$.

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<th>Cycle 0</th>
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<th>Cycle 3</th>
<th>Cycle 4</th>
<th>Cycle 5</th>
<th>Cycle 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>$w$ (%) (%)</td>
<td>16.8</td>
<td>23.4</td>
<td>23.0</td>
<td>22.1</td>
<td>21.7</td>
<td>21.4</td>
<td>19.0</td>
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<tr>
<td>$%S$ (%) (%)</td>
<td>70.1</td>
<td>97.2</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
</tr>
<tr>
<td>$\psi$ (kPa)</td>
<td>678.0</td>
<td>9.6</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>$\varepsilon_{a\text{ cycle}}$ (%)</td>
<td>0.0</td>
<td>0.19</td>
<td>2.63</td>
<td>3.54</td>
<td>3.28</td>
<td>2.65</td>
<td>3.56</td>
</tr>
<tr>
<td>$\varepsilon_{\text{vol cycle}}$ (%)</td>
<td>0.0</td>
<td>0.19</td>
<td>6.21</td>
<td>6.54</td>
<td>6.24</td>
<td>5.45</td>
<td>6.27</td>
</tr>
<tr>
<td>$\varepsilon_{a=\varepsilon_{\text{vol comp}}}$ (%)</td>
<td>0.0</td>
<td>0.19</td>
<td>-3.46</td>
<td>-3.56</td>
<td>-5.76</td>
<td>-8.06</td>
<td>-8.96</td>
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Drying Cycle Information

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<tr>
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<th>Cycle 4</th>
<th>Cycle 5</th>
<th>Cycle 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>$w$ (%) (%)</td>
<td>23.4</td>
<td>8.6</td>
<td>3.6</td>
<td>3.6</td>
<td>3.9</td>
<td>3.3</td>
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<tr>
<td>$%S$ (%) (%)</td>
<td>97.2</td>
<td>46.2</td>
<td>21.5</td>
<td>20.7</td>
<td>24.5</td>
<td>21.6</td>
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<tr>
<td>$\psi$ (kPa)</td>
<td>10</td>
<td>3385</td>
<td>21771</td>
<td>23512</td>
<td>16526</td>
<td>21516</td>
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<tr>
<td>$\varepsilon_{a\text{ cycle}}$ (%)</td>
<td>0.0</td>
<td>-6.79</td>
<td>-5.62</td>
<td>-4.75</td>
<td>-4.68</td>
<td>-4.23</td>
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<tr>
<td>$\varepsilon_{\text{vol cycle}}$ (%)</td>
<td>0.0</td>
<td>-9.94</td>
<td>-8.13</td>
<td>-7.13</td>
<td>-6.97</td>
<td>-6.44</td>
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<tr>
<td>$\varepsilon_{a=\varepsilon_{\text{vol comp}}}$ (%)</td>
<td>0.0</td>
<td>-5.75</td>
<td>-8.88</td>
<td>-8.01</td>
<td>-10.17</td>
<td>-11.99</td>
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<tr>
<td>$\varepsilon_{\text{vol comp}}$ (%)</td>
<td>0.0</td>
<td>-8.93</td>
<td>-11.31</td>
<td>-10.31</td>
<td>-12.32</td>
<td>-14.02</td>
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</table>
### Table 7.18: Results for Colorado Compacted at 90% OMC–100% MDD \( \sigma_{\text{Net}} = 461 \text{kPa} \).

<table>
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<th>Wetting Cycle Information</th>
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<th>Cycle 3</th>
<th>Cycle 4</th>
<th>Cycle 5</th>
<th>Cycle 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>( w ) (%)</td>
<td>16.9</td>
<td>23.3</td>
<td>21.1</td>
<td>20.5</td>
<td>19.9</td>
<td>19.7</td>
<td>17.2</td>
</tr>
<tr>
<td>( %S ) (%)</td>
<td>68.2</td>
<td>94.8</td>
<td>99.9</td>
<td>99.6</td>
<td>100.0</td>
<td>99.8</td>
<td>90.4</td>
</tr>
<tr>
<td>( \psi ) (kPa)</td>
<td>601.5</td>
<td>0.3</td>
<td>0.0</td>
<td>0.4</td>
<td>0.0</td>
<td>0.1</td>
<td>73.8</td>
</tr>
<tr>
<td>( \varepsilon_a ) cycle (%)</td>
<td>0.0</td>
<td>-0.49</td>
<td>2.27</td>
<td>2.88</td>
<td>2.65</td>
<td>2.87</td>
<td>2.71</td>
</tr>
<tr>
<td>( \varepsilon_{\text{vol}} ) cycle (%)</td>
<td>0.0</td>
<td>-0.49</td>
<td>3.94</td>
<td>4.36</td>
<td>4.50</td>
<td>4.72</td>
<td>4.32</td>
</tr>
<tr>
<td>( \varepsilon_a = \varepsilon_{\text{vol}} ) comp (%)</td>
<td>0.0</td>
<td>-0.49</td>
<td>-6.04</td>
<td>-7.07</td>
<td>-8.26</td>
<td>-8.68</td>
<td>-9.48</td>
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<table>
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<th>Drying Cycle Information</th>
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<th>Cycle 1</th>
<th>Cycle 2</th>
<th>Cycle 3</th>
<th>Cycle 4</th>
<th>Cycle 5</th>
<th>Cycle 6</th>
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</thead>
<tbody>
<tr>
<td>( w ) (%)</td>
<td>23.3</td>
<td>1.7</td>
<td>1.6</td>
<td>1.2</td>
<td>1.0</td>
<td>0.7</td>
<td>1.5</td>
</tr>
<tr>
<td>( %S ) (%)</td>
<td>94.8</td>
<td>9.8</td>
<td>9.1</td>
<td>7.0</td>
<td>6.4</td>
<td>4.2</td>
<td>9.6</td>
</tr>
<tr>
<td>( \psi ) (kPa)</td>
<td>0.3</td>
<td>90917</td>
<td>102321</td>
<td>149763</td>
<td>170092</td>
<td>279558</td>
<td>94710</td>
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<tr>
<td>( \varepsilon_a ) cycle (%)</td>
<td>0.0</td>
<td>-8.01</td>
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<td>-5.24</td>
<td>-5.08</td>
<td>-5.04</td>
<td>-5.10</td>
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<tr>
<td>( \varepsilon_{\text{vol}} ) cycle (%)</td>
<td>0.0</td>
<td>-9.49</td>
<td>-7.03</td>
<td>-6.88</td>
<td>-6.76</td>
<td>-6.50</td>
<td>-6.89</td>
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<tr>
<td>( \varepsilon_a ) comp (%)</td>
<td>0.0</td>
<td>-10.36</td>
<td>-11.40</td>
<td>-11.75</td>
<td>-12.70</td>
<td>-12.95</td>
<td>-14.10</td>
</tr>
<tr>
<td>( \varepsilon_{\text{vol}} ) comp (%)</td>
<td>0.0</td>
<td>-11.79</td>
<td>-12.64</td>
<td>-13.28</td>
<td>-14.25</td>
<td>-14.30</td>
<td>-15.72</td>
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</table>

#### 7.2.5 110% OMC – 100% MDD

![Figure 7.19: Strain results for Colorado soil compacted at 110% OMC–100% MDD subjected to a net normal stress of 1.5kPa.](image)
Figure 7.20: Strain results for Colorado soil compacted at 110% OMC–100% MDD subjected to a net normal stress of 100kPa.

Figure 7.21: Strain results for Colorado soil compacted at 110% OMC–100% MDD subjected to a net normal stress of 208kPa.
Figure 7.22: Strain results for Colorado soil compacted at 110%OMC–100%MDD subjected to a net normal stress of 357kPa.

Table 7.19: Results for Colorado Compacted at 110%OMC–100%MDD $\sigma_{\text{Net}}=1.5\text{kPa}$.

<table>
<thead>
<tr>
<th>Wetting Cycle Information</th>
<th>Cycle 0</th>
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<th>Cycle 3</th>
<th>Cycle 4</th>
<th>Cycle 5</th>
<th>Cycle 6</th>
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<tbody>
<tr>
<td>w% (%)</td>
<td>20.9</td>
<td>36.5</td>
<td>41.6</td>
<td>50.6</td>
<td>48.4</td>
<td>57.5</td>
<td>48.9</td>
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<tr>
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<td>88.0</td>
<td>100.0</td>
<td>99.8</td>
<td>100.0</td>
<td>100.0</td>
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<td>100.0</td>
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<td>0.2</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
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<tr>
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<td>0.0</td>
<td>17.52</td>
<td>27.24</td>
<td>18.56</td>
<td>20.01</td>
<td>20.30</td>
<td>14.71</td>
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<tr>
<td>$\varepsilon_{\text{vol}}$ cycle (%)</td>
<td>0.0</td>
<td>17.52</td>
<td>45.36</td>
<td>40.82</td>
<td>44.96</td>
<td>47.16</td>
<td>41.55</td>
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<td>$\varepsilon_a=\varepsilon_{\text{vol}}$ comp (%)</td>
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<td>17.52</td>
<td>29.70</td>
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<td>42.20</td>
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<th>Cycle 2</th>
<th>Cycle 3</th>
<th>Cycle 4</th>
<th>Cycle 5</th>
<th>Cycle 6</th>
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<tbody>
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<td>w% (%)</td>
<td>36.5</td>
<td>3.8</td>
<td>5.4</td>
<td>4.0</td>
<td>4.6</td>
<td>2.4</td>
<td>1.8</td>
</tr>
<tr>
<td>%S (%)</td>
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<td>22.4</td>
<td>25.8</td>
<td>18.1</td>
<td>18.8</td>
<td>10.5</td>
<td>7.6</td>
</tr>
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<td>30892</td>
<td>28647</td>
<td>82758</td>
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<td>$\varepsilon_a$ cycle (%)</td>
<td>0.0</td>
<td>-14.64</td>
<td>-12.34</td>
<td>-12.91</td>
<td>-12.08</td>
<td>-13.62</td>
<td>-8.46</td>
</tr>
<tr>
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<td>0.0</td>
<td>-25.28</td>
<td>-26.14</td>
<td>-27.63</td>
<td>-28.12</td>
<td>-30.00</td>
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<tr>
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<td>1.00</td>
<td>13.61</td>
<td>17.37</td>
<td>23.61</td>
<td>22.57</td>
<td>29.51</td>
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<td>$\varepsilon_{\text{vol}}$ comp (%)</td>
<td>0.0</td>
<td>-11.59</td>
<td>-4.28</td>
<td>-2.47</td>
<td>1.05</td>
<td>-0.68</td>
<td>10.20</td>
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</table>
Table 7.20: Results for Colorado Compacted at 110% OMC–100% MDD $\sigma_{\text{Net}}$=100kPa.

<table>
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<th>Cycle 2</th>
<th>Cycle 3</th>
<th>Cycle 4</th>
<th>Cycle 5</th>
<th>Cycle 6</th>
</tr>
</thead>
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<td>18.0</td>
<td>27.5</td>
<td>23.3</td>
<td>23.7</td>
<td>23.7</td>
</tr>
<tr>
<td>%S (%)</td>
<td>87.2</td>
<td>100.0</td>
<td>73.8</td>
<td>100.0</td>
<td>99.9</td>
<td>99.8</td>
<td>98.1</td>
</tr>
<tr>
<td>$\psi$ (kPa)</td>
<td>123.7</td>
<td>0.0</td>
<td>503.6</td>
<td>0.0</td>
<td>0.0</td>
<td>0.2</td>
<td>4.8</td>
</tr>
<tr>
<td>$\varepsilon_{a \text{ cycle}}$ (%)</td>
<td>0.0</td>
<td>2.12</td>
<td>6.94</td>
<td>7.77</td>
<td>7.06</td>
<td>6.35</td>
<td>7.87</td>
</tr>
<tr>
<td>$\varepsilon_{\text{vol cycle}}$ (%)</td>
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<td>2.12</td>
<td>13.63</td>
<td>13.57</td>
<td>12.12</td>
<td>11.33</td>
<td>12.78</td>
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<td>0.0</td>
<td>2.12</td>
<td>1.12</td>
<td>1.57</td>
<td>-0.78</td>
<td>-0.72</td>
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Table 7.21: Results for Colorado Compacted at 110% OMC–100% MDD $\sigma_{\text{Net}}$=208kPa.

<table>
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<th>Cycle 6</th>
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<tbody>
<tr>
<td>$w$% (%)</td>
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<td>5.9</td>
<td>5.5</td>
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</tr>
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<td>%S (%)</td>
<td>100.0</td>
<td>32.5</td>
<td>36.2</td>
<td>33.0</td>
<td>33.0</td>
<td>33.8</td>
<td>14.5</td>
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<td>$\psi$ (kPa)</td>
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<td>8679</td>
<td>6664</td>
<td>8389</td>
<td>8394</td>
<td>7898</td>
<td>47208</td>
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<td>$\varepsilon_{a \text{ cycle}}$ (%)</td>
<td>0.0</td>
<td>-8.99</td>
<td>-8.56</td>
<td>-8.71</td>
<td>-7.68</td>
<td>-7.34</td>
<td>-7.31</td>
</tr>
<tr>
<td>$\varepsilon_{\text{vol cycle}}$ (%)</td>
<td>0.0</td>
<td>-14.31</td>
<td>-13.23</td>
<td>-12.80</td>
<td>-11.68</td>
<td>-11.34</td>
<td>-10.77</td>
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<tr>
<td>$\varepsilon_{a \text{ =vol comp}}$ (%)</td>
<td>0.0</td>
<td>-7.05</td>
<td>-7.53</td>
<td>-7.26</td>
<td>-8.39</td>
<td>-7.28</td>
<td>-6.73</td>
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<td>$\varepsilon_{\text{vol comp}}$ (%)</td>
<td>0.0</td>
<td>-12.49</td>
<td>-12.25</td>
<td>-11.42</td>
<td>-12.37</td>
<td>-11.28</td>
<td>-10.20</td>
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<th>Cycle 5</th>
<th>Cycle 6</th>
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<tr>
<td>$w$% (%)</td>
<td>24.1</td>
<td>2.8</td>
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<td>6.0</td>
<td>6.4</td>
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<td>35691</td>
<td>82886</td>
<td>100424</td>
<td>134462</td>
<td>186112</td>
<td>170459</td>
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<td>$\varepsilon_{a \text{ cycle}}$ (%)</td>
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<td>-5.03</td>
<td>-6.12</td>
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<td>-5.95</td>
<td>-5.44</td>
<td>-5.65</td>
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<td>$\varepsilon_{\text{vol cycle}}$ (%)</td>
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<td>-9.12</td>
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<td>$\varepsilon_{\text{vol comp}}$ (%)</td>
<td>0.0</td>
<td>-11.95</td>
<td>-10.69</td>
<td>-11.03</td>
<td>-12.20</td>
<td>-10.91</td>
<td>-12.03</td>
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Table 7.22: Results for Colorado Compacted at 110% OMC–100% MDD $\sigma_{\text{Net}}$=357kPa.

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<tr>
<td>$w$ (%)</td>
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<td>23.6</td>
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<td>20.8</td>
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<td>$%S$ (%)</td>
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<td>99.9</td>
<td>100.0</td>
<td>100.0</td>
<td>99.9</td>
<td>100.0</td>
<td>99.9</td>
</tr>
<tr>
<td>$\psi$ (kPa)</td>
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<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>$\varepsilon_{a\text{ cycle}}$ (%)</td>
<td>0.0</td>
<td>-0.31</td>
<td>2.37</td>
<td>3.26</td>
<td>3.15</td>
<td>3.55</td>
<td>3.48</td>
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<td>$\varepsilon_{\text{vol cycle}}$ (%)</td>
<td>0.0</td>
<td>-0.31</td>
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<td>5.54</td>
<td>5.38</td>
<td>5.26</td>
<td>4.96</td>
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<td>$\varepsilon_{a \text{comp}}$ (%)</td>
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<td>-0.31</td>
<td>-5.67</td>
<td>-6.61</td>
<td>-9.02</td>
<td>-7.57</td>
<td>-8.15</td>
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Drying Cycle Information

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<th>Cycle 4</th>
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<td>1.6</td>
<td>1.3</td>
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<td>16.9</td>
<td>12.4</td>
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<td>10.9</td>
<td>8.0</td>
</tr>
<tr>
<td>$\psi$ (kPa)</td>
<td>0</td>
<td>35027</td>
<td>62210</td>
<td>73986</td>
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<td>124506</td>
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<td>$\varepsilon_{a\text{ cycle}}$ (%)</td>
<td>0.0</td>
<td>-8.13</td>
<td>-6.04</td>
<td>-5.67</td>
<td>-5.64</td>
<td>-4.63</td>
</tr>
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<td>$\varepsilon_{\text{vol cycle}}$ (%)</td>
<td>0.0</td>
<td>-10.84</td>
<td>-8.07</td>
<td>-7.69</td>
<td>-7.16</td>
<td>-5.97</td>
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<td>-11.85</td>
<td>-14.15</td>
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<td>$\varepsilon_{\text{vol comp}}$ (%)</td>
<td>0.0</td>
<td>-11.11</td>
<td>-13.28</td>
<td>-13.74</td>
<td>-15.54</td>
<td>-13.40</td>
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</tbody>
</table>

7.3 Anthem Soil

The result obtained for Anthem soil will be presented as follows:

1) Initial compacted condition of 90% OMC–90% MDD (Section 7.3.1)

2) Initial compacted condition of 110% OMC–90% MDD (Section 7.3.2)

3) Initial compacted condition of 100% OMC–95% MDD (Section 7.3.3)

4) Initial compacted condition of 90% OMC–100% MDD (Section 7.3.4)

5) Initial compacted condition of 110% OMC–100% MDD (Section 7.3.5)

7.3.1 90% OMC – 90% MDD
Figure 7.23: Strain results for Anthem soil compacted at 90% OMC–90%MDD
subjected to a net normal stress of 1.5kPa.

Figure 7.24: Strain results for Anthem soil compacted at 90% OMC–90%MDD
subjected to a net normal stress of 8.2kPa.
Figure 7.25: Strain results for Anthem soil compacted at 90% OMC–90% MDD subjected to a net normal stress of 42kPa.

Figure 7.26: Strain results for Anthem soil compacted at 90% OMC–90% MDD subjected to a net normal stress of 81kPa.
Figure 7.27: Strain results for Anthem soil compacted at 90% OMC–90%MDD subjected to a net normal stress of 144.0 kPa.

Table 7.23: Results for Anthem Compacted at 90% OMC–90%MDD $\sigma_{\text{Net}}=1.5$ kPa.

<table>
<thead>
<tr>
<th>Wetting Cycle Information</th>
<th>Cycle 0</th>
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<tr>
<td>w% (%)</td>
<td>16.2</td>
<td>33.7</td>
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<td>43.4</td>
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<td>42.0</td>
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<td>57.7</td>
<td>100.0</td>
<td>99.8</td>
<td>98.0</td>
<td>99.9</td>
<td>97.1</td>
<td>97.1</td>
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<td>$\psi$ (kPa)</td>
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<td>0.0</td>
<td>1.8</td>
<td>0.0</td>
<td>3.1</td>
<td>3.2</td>
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<td>$\varepsilon_a$ cycle (%)</td>
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<td>8.20</td>
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<td>14.56</td>
<td>13.68</td>
<td>15.62</td>
<td>13.18</td>
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<tr>
<td>$\varepsilon_{\text{vol}}$ cycle (%)</td>
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<td>8.20</td>
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<td>23.90</td>
<td>20.91</td>
<td>21.87</td>
<td>22.15</td>
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<tr>
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<td>0.0</td>
<td>8.20</td>
<td>14.96</td>
<td>21.36</td>
<td>23.75</td>
<td>24.69</td>
<td>23.53</td>
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<table>
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<th>Cycle 4</th>
<th>Cycle 5</th>
<th>Cycle 6</th>
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<tr>
<td>w% (%)</td>
<td>33.7</td>
<td>2.2</td>
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<td>0.2</td>
<td>0.5</td>
<td>1.1</td>
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<tr>
<td>%S (%)</td>
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<td>8.1</td>
<td>3.1</td>
<td>7.8</td>
<td>0.6</td>
<td>1.8</td>
<td>3.7</td>
</tr>
<tr>
<td>$\psi$ (kPa)</td>
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<td>543851</td>
<td>236926</td>
<td>887250</td>
<td>696738</td>
<td>484896</td>
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<tr>
<td>$\varepsilon_a$ cycle (%)</td>
<td>0.0</td>
<td>-3.80</td>
<td>-2.88</td>
<td>-7.70</td>
<td>-3.38</td>
<td>-12.93</td>
<td>-10.29</td>
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<tr>
<td>$\varepsilon_{\text{vol}}$ cycle (%)</td>
<td>0.0</td>
<td>-11.53</td>
<td>-9.79</td>
<td>-13.10</td>
<td>-8.27</td>
<td>-19.30</td>
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<td>12.01</td>
<td>19.56</td>
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<td>5.46</td>
<td>13.52</td>
<td>0.61</td>
<td>2.80</td>
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Table 7.24: Results for Anthem Compacted at 90% OMC–90% MDD $\sigma_{\text{Net}}=8.2\text{kPa}$.

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<td>$w$ (%) (%)</td>
<td>16.2</td>
<td>31.1</td>
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<td>33.8</td>
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<td>%S (%) (%)</td>
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<td>100.0</td>
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<td>0.0</td>
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<td>4.34</td>
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<td>15.05</td>
<td>14.06</td>
<td>14.27</td>
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<td>4.34</td>
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<td>6.45</td>
<td>7.17</td>
<td>6.76</td>
<td>7.91</td>
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<td>Cycle 4</td>
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<tr>
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<td>13.1</td>
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<td>-13.37</td>
<td>-13.58</td>
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<td>$\varepsilon_{\text{vol cycle}}$ (%)</td>
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<td>-16.04</td>
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<td>-14.59</td>
<td>-13.37</td>
<td>-13.58</td>
<td>-10.84</td>
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<td>-4.16</td>
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<td>-0.68</td>
<td>0.37</td>
<td>-1.81</td>
<td>1.87</td>
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<td>-7.47</td>
<td>-6.92</td>
<td>-5.96</td>
<td>-7.67</td>
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Table 7.25: Results for Anthem Compacted at 90% OMC–90% MDD $\sigma_{\text{Net}}=42\text{kPa}$.

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<th>Cycle 4</th>
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<td>27.2</td>
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<tr>
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<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
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<td>0.0</td>
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<tr>
<td>$\varepsilon_{a\text{ cycle}}$ (%)</td>
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<td>0.74</td>
<td>2.36</td>
<td>2.87</td>
<td>1.64</td>
<td>1.98</td>
<td>2.59</td>
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<tr>
<td>$\varepsilon_{\text{vol cycle}}$ (%)</td>
<td>0.0</td>
<td>0.74</td>
<td>5.02</td>
<td>5.69</td>
<td>3.35</td>
<td>3.89</td>
<td>5.47</td>
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<td>$\varepsilon_{a} - \varepsilon_{\text{vol comp}}$ (%)</td>
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<td>-1.45</td>
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<td>1.0</td>
<td>1.1</td>
<td>1.7</td>
<td>1.4</td>
<td>1.3</td>
</tr>
<tr>
<td>%S (%) (%)</td>
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<td>6.8</td>
<td>4.1</td>
<td>4.3</td>
<td>6.7</td>
<td>5.8</td>
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<td>451071</td>
<td>451417</td>
<td>284622</td>
<td>338624</td>
<td>350633</td>
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<td>-3.95</td>
<td>-2.68</td>
<td>-1.85</td>
<td>-1.48</td>
<td>-4.82</td>
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<tr>
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<td>-3.61</td>
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<td>-6.41</td>
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Table 7.26: Results for Anthem Compacted at 90% OMC–90%MDD $\sigma_{Net}=81$kPa.

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<td>99.8</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
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<td>$\psi$ (kPa)</td>
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<td>0.0</td>
<td>0.0</td>
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<td>1.83</td>
<td>1.03</td>
<td>1.91</td>
<td>1.95</td>
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Drying Information

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Table 7.27: Results for Anthem Compacted at 90% OMC–90%MDD $\sigma_{Net}=144$kPa.

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<td>24.7</td>
<td>24.5</td>
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<tr>
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<td>99.8</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
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<td>$\psi$ (kPa)</td>
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<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
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<td>1.05</td>
<td>0.74</td>
<td>1.02</td>
<td>1.38</td>
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<td>2.77</td>
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<td>2.70</td>
<td>3.03</td>
<td>3.00</td>
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<td>-9.59</td>
<td>-10.89</td>
<td>-10.76</td>
<td>-10.83</td>
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Drying Information

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<td>-13.23</td>
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7.3.2 110% OMC – 90% MDD

Figure 7.28: Strain results for Anthem soil compacted at 110% OMC–90% MDD subjected to a net normal stress of 1.5kPa.
Figure 7.29: Strain results for Anthem soil compacted at 110% OMC–90% MDD subjected to a net normal stress of 8.2 kPa.

Figure 7.30: Strain results for Anthem soil compacted at 110% OMC–90% MDD subjected to a net normal stress of 42 kPa.
Figure 7.31: Strain results for Anthem soil compacted at 110% OMC–90% MDD subjected to a net normal stress of 81kPa.

Figure 7.32: Strain results for Anthem soil compacted at 110% OMC–90% MDD subjected to a net normal stress of 144kPa.
Table 7.28: Results for Anthem Compacted at 110% OMC–90%MDD $\sigma_{\text{Net}}=1.5\text{kPa}$.

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<tr>
<td>$w$% (%)</td>
<td>16.2</td>
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<td>100.0</td>
<td>100.0</td>
<td>97.4</td>
<td>100.0</td>
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<td>0.0</td>
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<td>0.0</td>
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<td>15.80</td>
<td>13.79</td>
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<td>18.8</td>
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Table 7.29: Results for Anthem Compacted at 110% OMC–90%MDD $\sigma_{\text{Net}}=8.2\text{kPa}$.

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Table 7.30: Results for Anthem Compacted at 110% OMC–90%MDD $\sigma_{NT}=42$kPa.

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<td>99.8</td>
<td>97.0</td>
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<td>3.6</td>
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<td>-7.11</td>
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Table 7.31: Results for Anthem Compacted at 110% OMC–90%MDD $\sigma_{NT}=81$kPa.

<table>
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<td>0.0</td>
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<td>3.97</td>
<td>3.79</td>
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Table 7.32: Results for Anthem compacted at 110% OMC–90% MDD σ_{Net}=144kPa.

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<td>4.77</td>
<td>3.84</td>
<td>4.64</td>
<td>4.20</td>
<td>3.73</td>
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<td>11.0</td>
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7.3.3 100% OMC – 95% MDD

Figure 7.33: Strain results for Anthem soil compacted at 100% OMC–95% MDD subjected to a net normal stress of 1.5kPa.
Figure 7.34: Strain results for Anthem soil compacted at 100% OMC–95% MDD subjected to a net normal stress of 8.2 kPa.

Figure 7.35: Strain results for Anthem soil compacted at 100% OMC–95% MDD subjected to a net normal stress of 42 kPa.
Figure 7.36: Strain results for Anthem soil compacted at 100% OMC–95% MDD subjected to a net normal stress of 81kPa.

Figure 7.37: Strain results for Anthem soil compacted at 100% OMC–95% MDD subjected to a net normal stress of 144kPa.
Table 7.33: Results for Anthem Compacted at 100% OMC–95% MDD $\sigma_{\text{Net}}=1.5\text{kPa}$.

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<td>100.0</td>
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<td>13.59</td>
<td>11.04</td>
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<td>6.9</td>
<td>4.3</td>
<td>4.1</td>
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Table 7.34: Results for Anthem Compacted at 100% OMC–95% MDD $\sigma_{\text{Net}}=8.2\text{kPa}$.

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Table 7.35: Results for Anthem Compacted at 100% OMC–95% MDD $\sigma_{\text{Net}}=42\text{kPa}$.

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Drying Cycle Information

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<td>421508</td>
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<td>400877</td>
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<td>-4.91</td>
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<td>$\varepsilon_{\text{vol}}$ cycle (%)</td>
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<td>-7.95</td>
<td>-6.89</td>
<td>-6.34</td>
<td>-5.55</td>
<td>-8.02</td>
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<tr>
<td>$\varepsilon_a$ comp (%)</td>
<td>0.0</td>
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<td>-4.69</td>
<td>-3.71</td>
<td>-3.24</td>
<td>-4.22</td>
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<tr>
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<td>-7.77</td>
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Table 7.36: Results for Anthem Compacted at 100% OMC–95% MDD $\sigma_{\text{Net}}=82\text{kPa}$.

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<td>100.0</td>
<td>100.0</td>
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<tr>
<td>$\psi$ (kPa)</td>
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<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>$\varepsilon_a$ cycle (%)</td>
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<td>0.47</td>
<td>4.88</td>
<td>2.28</td>
<td>2.05</td>
<td>2.17</td>
<td>2.26</td>
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<tr>
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<td>0.47</td>
<td>10.13</td>
<td>6.40</td>
<td>6.18</td>
<td>6.31</td>
<td>5.69</td>
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Drying Cycle Information

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<td>110795</td>
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<td>-7.68</td>
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Table 7.37: Results for Anthem Compacted at 100% OMC–95% MDD $\sigma_{\text{Net}}=144$ kPa.

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<td>100.0</td>
<td>100.0</td>
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<td>36.5</td>
<td>0.0</td>
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<td>1.20</td>
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<td>176218</td>
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<td>-10.18</td>
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<td>-11.33</td>
<td>-11.45</td>
<td>-11.68</td>
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7.3.4 90% OMC – 100% MDD

![Figure 7.38: Strain results for Anthem soil compacted at 90% OMC–100% MDD subjected to a net normal stress of 1.5kPa.](image-url)
Figure 7.39: Strain results for Anthem soil compacted at 90% OMC–100% MDD subjected to a net normal stress of 42kPa.

Figure 7.40: Strain results for Anthem soil compacted at 90% OMC–100% MDD subjected to a net normal stress of 82kPa.
Figure 7.41: Strain results for Anthem soil compacted at 90% OMC–100% MDD subjected to a net normal stress of 144kPa.

Figure 7.42: Strain results for Anthem soil compacted at 90% OMC–100% MDD subjected to a net normal stress of 310kPa.
Table 7.38: Results for Anthem Compacted at 90%OMC–100%MDD $\sigma_{Net}=1.5\text{kPa}$.

<table>
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<th>Cycle 4</th>
<th>Cycle 5</th>
<th>Cycle 6</th>
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<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
<td>99.8</td>
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<tr>
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<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.1</td>
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<td>2.16</td>
<td>3.22</td>
<td>2.27</td>
<td>2.89</td>
<td>3.12</td>
<td>2.01</td>
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<td>4.59</td>
<td>6.95</td>
<td>7.70</td>
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<td>3.98</td>
<td>4.38</td>
<td>3.91</td>
<td>3.33</td>
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<td>Cycle 6</td>
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<td>1.2</td>
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<tr>
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<td>5.9</td>
<td>5.5</td>
<td>6.1</td>
<td>6.0</td>
<td>6.7</td>
</tr>
<tr>
<td>$\psi$ (kPa)</td>
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<td>260477</td>
<td>329434</td>
<td>351205</td>
<td>314610</td>
<td>323538</td>
<td>287602</td>
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<td>$\varepsilon_{a,\text{cycle}}$ (%)</td>
<td>0.0</td>
<td>-3.67</td>
<td>-3.28</td>
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<td>-1.88</td>
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<td>-4.92</td>
<td>-3.09</td>
<td>-5.43</td>
<td>-7.03</td>
<td>-8.25</td>
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<td>0.02</td>
<td>0.88</td>
<td>-1.14</td>
<td>-3.27</td>
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Table 7.39: Results for Anthem Compacted at 90%OMC–100%MDD $\sigma_{Net}=42\text{kPa}$.

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<td>100.0</td>
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<td>0.0</td>
<td>0.0</td>
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<tr>
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<td>3.22</td>
<td>2.27</td>
<td>2.89</td>
<td>3.12</td>
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<td>4.59</td>
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<tr>
<td>$\varepsilon_{a,\text{cycle}}$ (%)</td>
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<td>-1.88</td>
<td>-1.74</td>
<td>-3.10</td>
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<tr>
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<td>-5.43</td>
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Table 7.40: Results for Anthem Compacted at 90%OMC–100%MDD $\sigma_{Net}=82kPa$.

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<td>100.0</td>
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<tr>
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<td>36.0</td>
<td>50.8</td>
<td>42.3</td>
<td>34.6</td>
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<tr>
<td>$\psi$ (kPa)</td>
<td>0</td>
<td>8004</td>
<td>2075</td>
<td>4461</td>
<td>9168</td>
<td>11382</td>
<td>11186</td>
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<tr>
<td>$\varepsilon_a$ cycle (%)</td>
<td>0.0</td>
<td>-5.06</td>
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<td>-3.94</td>
<td>-5.75</td>
<td>-6.19</td>
<td>-5.04</td>
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<tr>
<td>$\varepsilon_{vol}$ cycle (%)</td>
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<td>-7.17</td>
<td>-6.39</td>
<td>-7.35</td>
<td>-8.55</td>
<td>-8.95</td>
<td>-7.30</td>
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<tr>
<td>$\varepsilon_{a=vol}$ comp (%)</td>
<td>0.0</td>
<td>-2.68</td>
<td>-0.93</td>
<td>-1.34</td>
<td>-3.69</td>
<td>-3.21</td>
<td>-4.62</td>
</tr>
<tr>
<td>$\varepsilon_{vol}$ comp (%)</td>
<td>0.0</td>
<td>-4.84</td>
<td>-3.83</td>
<td>-4.84</td>
<td>-6.56</td>
<td>-6.07</td>
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Table 7.41: Results for Anthem Compacted at 90%OMC–100%MDD $\sigma_{Net}=144kPa$.

<table>
<thead>
<tr>
<th>Wetting Cycle Information</th>
<th>Cycle 0</th>
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<th>Cycle 2</th>
<th>Cycle 3</th>
<th>Cycle 4</th>
<th>Cycle 5</th>
<th>Cycle 6</th>
</tr>
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<tbody>
<tr>
<td>$w$% (%)</td>
<td>16.2</td>
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<td>20.1</td>
<td>19.9</td>
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<tr>
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<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
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<td>$\psi$ (kPa)</td>
<td>214.5</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>$\varepsilon_a$ cycle (%)</td>
<td>0.0</td>
<td>0.57</td>
<td>1.40</td>
<td>1.67</td>
<td>1.71</td>
<td>1.82</td>
<td>1.81</td>
</tr>
<tr>
<td>$\varepsilon_{vol}$ cycle (%)</td>
<td>0.0</td>
<td>0.57</td>
<td>2.99</td>
<td>3.30</td>
<td>2.74</td>
<td>3.15</td>
<td>3.70</td>
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<td>$\varepsilon_{a=vol}$ comp (%)</td>
<td>0.0</td>
<td>0.57</td>
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<td>-2.45</td>
<td>-2.74</td>
<td>-2.99</td>
<td>-3.39</td>
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<td>Cycle 4</td>
<td>Cycle 5</td>
<td>Cycle 6</td>
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<tr>
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<td>0.9</td>
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<tr>
<td>%S (%)</td>
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<td>6.7</td>
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<td>3.2</td>
<td>5.9</td>
<td>5.1</td>
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<td>$\psi$ (kPa)</td>
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<td>271555</td>
<td>288601</td>
<td>193423</td>
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<td>328356</td>
<td>379555</td>
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<tr>
<td>$\varepsilon_a$ cycle (%)</td>
<td>0.0</td>
<td>-3.81</td>
<td>-3.39</td>
<td>-2.99</td>
<td>-2.98</td>
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<td>$\varepsilon_{vol}$ cycle (%)</td>
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<td>-3.93</td>
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<td>-3.22</td>
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<td>-5.51</td>
<td>-5.87</td>
<td>-6.18</td>
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<tr>
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<td>-6.92</td>
<td>-6.12</td>
<td>-6.73</td>
<td>-7.58</td>
<td>-7.81</td>
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### Table 7.42: Results for Anthem Compacted at 90% OMC–100% MDD $\sigma_{\text{Net}}=310\text{kPa}$.

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<th>Cycle 4</th>
<th>Cycle 5</th>
<th>Cycle 6</th>
</tr>
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<tbody>
<tr>
<td>$w$ (%)</td>
<td>16.2</td>
<td>22.2</td>
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<td>18.4</td>
<td>18.4</td>
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<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
</tr>
<tr>
<td>$\psi$ (kPa)</td>
<td>233.6</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
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</tr>
<tr>
<td>$\varepsilon_a$ cycle (%)</td>
<td>0.0</td>
<td>-0.61</td>
<td>0.76</td>
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<td>1.01</td>
<td>0.91</td>
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<td>0.0</td>
<td>-0.61</td>
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<td>2.70</td>
<td>2.80</td>
<td>2.70</td>
<td>2.60</td>
</tr>
<tr>
<td>$\varepsilon_a=\varepsilon_{\text{vol}}$ comp (%)</td>
<td>0.0</td>
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<td>-5.05</td>
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<td>-6.31</td>
<td>-7.13</td>
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<th>Cycle 2</th>
<th>Cycle 3</th>
<th>Cycle 4</th>
<th>Cycle 5</th>
<th>Cycle 6</th>
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</thead>
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<td>22.2</td>
<td>1.5</td>
<td>1.2</td>
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<td>8.2</td>
<td>8.2</td>
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<td>$\psi$ (kPa)</td>
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<td>249754</td>
<td>203819</td>
<td>222756</td>
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<td>-2.95</td>
<td>-3.03</td>
<td>-2.83</td>
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<td>0.0</td>
<td>-7.06</td>
<td>-4.99</td>
<td>-4.58</td>
<td>-4.64</td>
<td>-4.15</td>
<td>-3.80</td>
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<tr>
<td>$\varepsilon_a$ comp (%)</td>
<td>0.0</td>
<td>-5.95</td>
<td>-8.28</td>
<td>-8.31</td>
<td>-8.71</td>
<td>-9.12</td>
<td>-8.63</td>
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<tr>
<td>$\varepsilon_{\text{vol}}$ comp (%)</td>
<td>0.0</td>
<td>-7.61</td>
<td>-9.78</td>
<td>-9.89</td>
<td>-10.31</td>
<td>-10.18</td>
<td>-9.54</td>
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</table>

#### 7.3.5 110% OMC – 100% MDD

![Figure 7.43: Strain results for Anthem soil compacted at 110% OMC–100% MDD subjected to a net normal stress of 1.5kPa.](image)

subjected to a net normal stress of 1.5kPa.
Figure 7.44: Strain results for Anthem soil compacted at 110% OMC–100% MDD subjected to a net normal stress of 42kPa.

Figure 7.45: Strain results for Anthem soil compacted at 110% OMC–100% MDD subjected to a net normal stress of 82kPa.
Figure 7.46: Strain results for Anthem soil compacted at 110% OMC–100%MDD subjected to a net normal stress of 144kPa.

Table 7.43: Results for Anthem Compacted at 110% OMC–100%MDD $\sigma_{\text{Net}}=1.5$kPa.

<table>
<thead>
<tr>
<th>Wetting Cycle Information</th>
<th>Cycle 0</th>
<th>Cycle 1</th>
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<th>Cycle 3</th>
<th>Cycle 4</th>
<th>Cycle 5</th>
<th>Cycle 6</th>
</tr>
</thead>
<tbody>
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<td>$w$ (%)</td>
<td>16.2</td>
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<td>29.1</td>
<td>34.0</td>
<td>33.0</td>
<td>31.6</td>
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<tr>
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<td>99.9</td>
<td>100.0</td>
<td>100.0</td>
<td>99.9</td>
<td>95.8</td>
<td>100.0</td>
</tr>
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<td>$\psi$ (kPa)</td>
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<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
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<td>8.46</td>
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<td>14.52</td>
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<td>7.08</td>
<td>12.38</td>
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<tr>
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<td>19.81</td>
<td>17.70</td>
<td>21.95</td>
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<td>0.0</td>
<td>8.46</td>
<td>12.38</td>
<td>21.30</td>
<td>19.51</td>
<td>19.62</td>
<td>23.76</td>
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<table>
<thead>
<tr>
<th>Drying Cycle Information</th>
<th>Cycle 0</th>
<th>Cycle 1</th>
<th>Cycle 2</th>
<th>Cycle 3</th>
<th>Cycle 4</th>
<th>Cycle 5</th>
<th>Cycle 6</th>
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<tr>
<td>$w$ (%)</td>
<td>26.8</td>
<td>11.6</td>
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<td>2.1</td>
<td>7.9</td>
<td>4.1</td>
<td>0.8</td>
</tr>
<tr>
<td>%S (%)</td>
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<td>61.9</td>
<td>13.7</td>
<td>9.4</td>
<td>36.0</td>
<td>17.9</td>
<td>3.4</td>
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<tr>
<td>$\psi$ (kPa)</td>
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<td>770</td>
<td>97460</td>
<td>184678</td>
<td>7989</td>
<td>50263</td>
<td>512246</td>
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<td>$\varepsilon_a$ cycle (%)</td>
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<td>-5.76</td>
<td>-5.23</td>
<td>-8.04</td>
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<td>-6.75</td>
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<tr>
<td>$\varepsilon_\text{vol}$ cycle (%)</td>
<td>0.0</td>
<td>-12.42</td>
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<td>-15.95</td>
<td>-15.65</td>
<td>-14.90</td>
<td>-15.07</td>
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<tr>
<td>$\varepsilon_a$ comp (%)</td>
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<td>6.99</td>
<td>11.54</td>
<td>10.81</td>
<td>8.61</td>
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<tr>
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<td>1.94</td>
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<td>5.11</td>
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Table 7.44: Results for Anthem Compacted at 110% OMC–100% MDD $\sigma_{Net}=42kPa$.

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<th>Cycle 2</th>
<th>Cycle 3</th>
<th>Cycle 4</th>
<th>Cycle 5</th>
<th>Cycle 6</th>
</tr>
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<td>100.0</td>
<td>100.0</td>
<td>96.8</td>
<td>99.9</td>
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<tr>
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<td>2.91</td>
<td>3.78</td>
<td>3.75</td>
<td>2.87</td>
<td>3.31</td>
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<tr>
<td>$\varepsilon_{vol,cycle}$ (%)</td>
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Table 7.45: Results for Anthem Compacted at 110% OMC–100% MDD $\sigma_{Net}=82kPa$.

<table>
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<th>Cycle 5</th>
<th>Cycle 6</th>
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<tr>
<td>$%S$ (%)</td>
<td>76.1</td>
<td>98.7</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
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<td>99.9</td>
</tr>
<tr>
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<td>56509</td>
<td>89784</td>
<td>143606</td>
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<td>$\varepsilon_{a,cycle}$ (%)</td>
<td>0.0</td>
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<th>Cycle 4</th>
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<tr>
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<td>89784</td>
<td>143606</td>
<td>134381</td>
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Table 7.46: Results for Anthem Compacted at 110% OMC–100% MDD $\sigma_{Net}=144kPa$.

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<th>Cycle 2</th>
<th>Cycle 3</th>
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<th>Cycle 6</th>
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<tbody>
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<td>$w$% (%)</td>
<td>16.2</td>
<td>21.6</td>
<td>20.3</td>
<td>20.0</td>
<td>19.7</td>
<td>19.9</td>
<td>19.4</td>
</tr>
<tr>
<td>%S (%)</td>
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<td>100.0</td>
<td>98.4</td>
<td>99.9</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
</tr>
<tr>
<td>$\psi$ (kPa)</td>
<td>214.5</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>$\varepsilon_{a , cycle}$ (%)</td>
<td>0.0</td>
<td>-0.10</td>
<td>1.53</td>
<td>1.40</td>
<td>1.74</td>
<td>1.85</td>
<td>1.42</td>
</tr>
<tr>
<td>$\varepsilon_{vol , cycle}$ (%)</td>
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<td>-0.10</td>
<td>5.62</td>
<td>4.65</td>
<td>4.64</td>
<td>4.45</td>
<td>3.79</td>
</tr>
<tr>
<td>$\varepsilon_{a , cycle} = \varepsilon_{vol , comp}$ (%)</td>
<td>0.0</td>
<td>-0.10</td>
<td>-1.71</td>
<td>-2.87</td>
<td>-3.36</td>
<td>-3.00</td>
<td>-3.76</td>
</tr>
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</table>

<table>
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<th>Cycle 0</th>
<th>Cycle 1</th>
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<th>Cycle 4</th>
<th>Cycle 5</th>
<th>Cycle 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>$w$% (%)</td>
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<td>5.1</td>
<td>4.6</td>
<td>2.5</td>
<td>4.9</td>
<td>2.9</td>
<td>1.7</td>
</tr>
<tr>
<td>%S (%)</td>
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<td>30.5</td>
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<td>15.1</td>
<td>29.7</td>
<td>17.5</td>
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<tr>
<td>$\psi$ (kPa)</td>
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<td>18705</td>
<td>80705</td>
<td>14788</td>
<td>59207</td>
<td>154996</td>
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<tr>
<td>$\varepsilon_{a , cycle}$ (%)</td>
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<td>-4.26</td>
<td>-3.70</td>
<td>-3.52</td>
<td>-3.16</td>
<td>-3.21</td>
<td>-3.06</td>
</tr>
<tr>
<td>$\varepsilon_{vol , cycle}$ (%)</td>
<td>0.0</td>
<td>-7.86</td>
<td>-6.72</td>
<td>-6.20</td>
<td>-5.53</td>
<td>-5.41</td>
<td>-5.30</td>
</tr>
<tr>
<td>$\varepsilon_{a , cycle}$ (%)</td>
<td>0.0</td>
<td>-4.29</td>
<td>-5.33</td>
<td>-6.17</td>
<td>-6.30</td>
<td>-6.04</td>
<td>-6.64</td>
</tr>
<tr>
<td>$\varepsilon_{vol , comp}$ (%)</td>
<td>0.0</td>
<td>-7.89</td>
<td>-8.31</td>
<td>-8.78</td>
<td>-8.59</td>
<td>-8.17</td>
<td>-8.80</td>
</tr>
</tbody>
</table>

7.4 Summary and Conclusions

Presented in this chapter were the results of the main experiment. The results include the axial and volumetric strain analyzed by the “as compacted” height/volume and by cycle (i.e. the height/volume at the beginning of the cycle prior to wetting/drying) for both wetting and drying cycles. Chapter 8 will use the information presented in this chapter, for the analysis. On a side note, each figure in this chapter represents at least a month or more of testing time.
Chapter 8

ANALYSIS OF THE LABORATORY RESULTS

8.1 Overview

The analysis of the laboratory results are presented in this chapter. The analysis presented herein will include how the swell/collapse potential changes with cycles, how the swell pressure reduces with cycles, impacts of the initial compacted condition on expansive soil characteristics with cycles, and implications of the initial compacted condition on ultimate heave with cycles. Finally, presented in this chapter will be the summary, conclusions, and recommendations for compacted expansive soils.

8.2 Reduction/Increase in Swell/Heave Potential

In this section, each initial compacted condition will be shown in separate figures. The results from the test are presented first followed by a discussion of the results. The figures are organized as follows: (1) initial compacted condition of 90% OMC – 90% MDD, (2) initial compacted condition of 110% OMC – 90% MDD, (3) initial compacted condition of 100% OMC – 95% MDD, (4) initial compacted condition of 90% OMC – 100% MDD, and (5) initial compacted condition of 110% OMC – 100% MDD. The first five figures, Figures 8.1 through 8.5 are the results from Colorado and the next five figures, Figures 8.6 through 8.10 are the results from Anthem.

Even though in Chapter 4, a third order polynomial was suggested for the best fit, a second order polynomial fit was used for the Colorado results. The Colorado results in some instances, only had two of three proposed token stresses, which if the third order polynomial fit was used it would weigh heavily on the larger stresses creating a non-
realistic swelling strains profile, for some of the sample. In addition results from Anthem only used the second order fit so a comparison between the two soils can be made. The results of the swell pressure analysis are shown in Section 8.3.

Each of the regression equations shown in Figures 8.1 through 8.10 are used to determine the strain at any given net normal stress. Each of these equations were then used to determine the swell pressure, which is presented in Section 8.3. Microsoft Excel Solver was then used to determine the positive root solution for each equation by setting the strain equal to zero.

Figure 8.1: Axial strain changes with cycles for compaction condition of 90% OMC – 90% MDD (Colorado soil).
Figure 8.2: Axial strain changes with cycles for compaction condition of 110% OMC

– 90% MDD (Colorado soil).

Figure 8.3: Axial strain changes with cycles for compaction condition of 100% OMC

– 95% MDD (Colorado soil).
Figure 8.4: Axial strain changes with cycles for compaction condition of 90\% OMC – 100\% MDD (Colorado soil).

Figure 8.5: Axial strain changes with cycles for compaction condition of 110\% OMC – 100\% MDD (Colorado soil).
Figure 8.6: Axial strain changes with cycles for compaction condition of 90%OMC – 90% MDD (Anthem soil).

Figure 8.7: Axial strain changes with cycles for compaction condition of 110%OMC – 90% MDD (Anthem soil).
Figure 8.8: Axial strain changes with cycles for compaction condition of 100% OMC – 95% MDD (Anthem soil).

Figure 8.9: Axial strain changes with cycles for compaction condition of 90% OMC – 100% MDD (Anthem soil).
Figure 8.10: Axial strain changes with cycles for compaction condition of 110% OMC – 100% MDD (Anthem soil).

Figure 8.11: Example of axial strain changes.
As one can see, from Figures 8.1 through 8.10, at the token stresses the swelling strain shows an increase while at larger stresses the swelling strain decreases with each cycle. Shown in Figure 8.11, there is a point at which the swelling strain will stop increasing with cycles and start decreasing, which is denoted with a circle. This point will be deemed the equilibrium point, since the strain at this point seems to fully recover after each wetting cycle. The equilibrium point is not defined during the first wetting cycle. The equilibrium point requires a minimum of two full wetting cycles and full drying cycles for the discovery. Once the soil dries, the soil will recover either all the strain, some of the strain, or the soil will continue to swell/collapse past the strain from the prior cycle, once next wetting cycle occurs. If the sample recovers all the strain and no additional swell occurs, this stress level would then be deemed the equilibrium stress and the strain value would then be deemed the equilibrium point. To the left of this point the swelling strain will continue to increase with cycles. To the right of this point the swelling strain will continue to decrease with cycles. Therefore it possible to postulate this point/stress within the soil profile will exhibit similar strains for both the wetting and drying cycle.

8.3 Changes in Swell Pressure

Each of the regression equations shown in Figures 8.1 through 8.10 are used to determine the strain at any given log transformed net normal stress. In addition, these equations were used determine the swell pressure. Microsoft Excel Solver was then used to determine the positive root solution for each equation by setting the strain equal to zero. The swell pressure data is presented in tabular form in Tables 8.1 and 8.2. Figures
8.11 and 8.12 show the results of the swell pressure with cycles analysis, for all five cube points, for Colorado and Anthem, respectively.

### Table 8.1: Swell Pressure Values for both soils.

<table>
<thead>
<tr>
<th>%OMC</th>
<th>%MDD</th>
<th>Cycle 1</th>
<th>Cycle 2</th>
<th>Cycle 3</th>
<th>Cycle 4</th>
<th>Cycle 5</th>
<th>Cycle 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>90</td>
<td>100</td>
<td>377.4</td>
<td>146.1</td>
<td>163.0</td>
<td>112.8</td>
<td>92.7</td>
<td>84.8</td>
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<tr>
<td>110</td>
<td>100</td>
<td>270.3</td>
<td>117.8</td>
<td>100.8</td>
<td>82.0</td>
<td>60.0</td>
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<tr>
<td>100</td>
<td>95</td>
<td>221.4</td>
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<td>60.9</td>
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<td>46.0</td>
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<td>90</td>
<td>90</td>
<td>94.5</td>
<td>46.6</td>
<td>38.4</td>
<td>36.2</td>
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</tr>
<tr>
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<td>90</td>
<td>97.4</td>
<td>45.0</td>
<td>33.0</td>
<td>32.6</td>
<td>26.4</td>
<td>28.0</td>
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</table>

<table>
<thead>
<tr>
<th>%OMC</th>
<th>%MDD</th>
<th>Cycle 1</th>
<th>Cycle 2</th>
<th>Cycle 3</th>
<th>Cycle 4</th>
<th>Cycle 5</th>
<th>Cycle 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>90</td>
<td>100</td>
<td>185.8</td>
<td>123.9</td>
<td>107.5</td>
<td>103.2</td>
<td>105.5</td>
<td>82.5</td>
</tr>
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<td>110</td>
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<td>138.7</td>
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<td>54.5</td>
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<td>69.4</td>
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<td>90</td>
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<td>110</td>
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<td>16.2</td>
<td>18.1</td>
<td>15.3</td>
<td>14.6</td>
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</tbody>
</table>

#### Figure 8.12: Swell pressure reduction with cycles Colorado.
As one can see, the 90%OMC – 100% MDD generates the highest swell pressure followed by 110% OMC – 100% MDD, 100% OMC – 95% MDD, 110%OMC – 90 MDD, and 90% OMC – 90% MDD, for the first cycle, for both soils. After the first cycle the trend stays the same except the initial compacted conditions of 110% OMC – 90 MDD, and 90% OMC – 90% MDD switch so that the 110% OMC – 90% MDD produces the least amount of swell pressure with each subsequent cycle.

The swell pressure results from Colorado imply that after fourth cycle, for the low initial compacted density (90%MDD), the density reaches an equilibrium value during the wetting and drying cycle leading to similar swell pressures. It was as shown that at the higher initial compacted density (100%MDD) the swell pressure reduces with cycles and starts to approach an equilibrated swell pressure for both condition, which
means the densities of the materials have reached an equilibrated value. The Anthem soil, on the other hand, shows a reduction in swell pressure with cycles but the swell pressure for the two densities do not merge. Since the swell pressure for the two densities 90%MDD and 100%MDD did not merge, it is possible that more cycles are needed to obtain a equilibrated density to correspond to an equilibrated swell pressure.

Nevertheless, Figures 8.11 and 8.12 showed empirically after four to five cycles the swell pressure stabilizes to an equilibrium swell pressure, which is at a minimum 43 percent reduction for Anthem and 69 percent reduction for Colorado. The percent reduction from the initial cycle to the final cycle is presented in Table 8.2 and 8.3 for Colorado and Anthem respectively. The percent reduction is determined by subtracting the initial swell pressure by the $i^{th}$ cycle swell pressure and then dividing by the initial swell pressure. When evaluating the swell pressure from one cycle to the next, the reduction in swell pressure increases with cycles until an equilibrated swell pressure is obtained. After one cycle, Colorado exhibited a minimum reduction of 50 percent and a maximum reduction of 69 percent while Anthem exhibited a minimum reduction of 20 percent and a maximum reduction of 77 percent. The range difference between the two soils is due to the magnitude of the swell pressure.

A percent reduction of 77 percent for Colorado corresponds to a value of 185kPa difference in swell pressure. This difference can affect the amount of additional overburden required to mitigate swell potential within a profile. A difference of 185kPa can correspond to a difference of 9.77 meters of fill or an additional 9.77 meters of a soil profile that can heave when additional water is present. The results are similar for the
Anthem soil. There is a reduction in swell pressure after the first cycle. The maximum reduction for the Anthem soil is 76 percent, which corresponds to a difference of 2.52 meters of either additional fill or 2.52 meters of a soil profile that contributes to the heave.

Table 8.2: Swell Pressure Reduction for Colorado.

<table>
<thead>
<tr>
<th>%OMC</th>
<th>%MDD</th>
<th>Cycle 1</th>
<th>Cycle 2</th>
<th>Cycle 3</th>
<th>Cycle 4</th>
<th>Cycle 5</th>
<th>Cycle 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>90</td>
<td>100</td>
<td>0.0%</td>
<td>61.3%</td>
<td>56.8%</td>
<td>70.1%</td>
<td>75.4%</td>
<td>77.5%</td>
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<tr>
<td>110</td>
<td>100</td>
<td>0.0%</td>
<td>56.4%</td>
<td>62.7%</td>
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<td>77.8%</td>
<td>68.5%</td>
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<td>100</td>
<td>95</td>
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<td>69.1%</td>
<td>72.5%</td>
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<td>50.7%</td>
<td>59.3%</td>
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<td>69.4%</td>
<td>70.8%</td>
</tr>
<tr>
<td>110</td>
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<td>66.1%</td>
<td>66.5%</td>
<td>72.9%</td>
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Table 8.3: Swell Pressure Reduction for Anthem.

<table>
<thead>
<tr>
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<th>Cycle 1</th>
<th>Cycle 2</th>
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<td>56.4%</td>
<td>60.7%</td>
<td>56.1%</td>
</tr>
<tr>
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<td>95</td>
<td>0.0%</td>
<td>40.4%</td>
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<tr>
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<td>73.2%</td>
<td>70.1%</td>
<td>74.7%</td>
<td>75.8%</td>
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</table>

Shown in Figure 8.13, are conceptual locations of the initial compacted conditions for each soil. This figure will be used as a reference for Figures 8.14 and 8.15. Figures 8.14 and 8.15 show the differences in swell pressure for the different initial compacted conditions. In addition to the swell pressure for the initial compacted conditions, Figures 8.14 and 8.15 show the increase in swell pressure from moving from one initial compacted condition to another denoted by an arrow and the increase factor in blue.
Figure 8.14: Conceptual locations for the initial compacted conditions.

Figure 8.15: Differences in swell pressure for the Colorado soil.
Figure 8.16: Differences in swell pressure for the Anthem soil.

Shown in Figures 8.15 and 8.16, the magnitude of the swell pressure, for the initial compacted condition, ranges significantly with each cycle. The largest increase in swell pressure, for initial compacted conditions, is when the initial compacted condition change low density combination to the highest density combination (i.e. from 90%MDD to 100%MDD). The Colorado soil exhibits the highest increase when changing the initial conditions from 90%OMC – 90%MDD to 90%OMC – 100%MDD. The Anthem soil, on the other hand, exhibited the highest increase when changing the initial conditions from 110%OMC – 90%MDD to 110%OMC – 100%MDD.

Nevertheless, after multiple wetting and drying cycles were accomplished the 110%OMC – 90%MDD initial condition showed the greatest reduction in swell pressure.
However, recommendations based upon swell pressure reduction alone only focuses on half the issue. The other half of the issue is the swell potential or heave characteristics change with cycles. It is possible to extrapolate these finding by suggesting that all CL soils will behavior similarly to Anthem and the swell pressure reduction from the first cycle to an equilibrium swell pressure, the equilibrium swell pressure can be reduced by 46 percent of the initial swell pressure. In addition, it is possible to postulate that the findings for the Colorado soil can be extrapolated to all CH soil for swell pressure reduction. The swell pressure reduction from the first cycle to an equilibrium value that could potentially be used for CH soils would reduce the first cycle swell pressure by 68 percent.

![Figure 8.17: Colorado swell pressure contours first cycle.](image)
Figure 8.18: Colorado swell pressure contours sixth cycle.

Figure 8.19: Anthem swell pressure contours first cycle.
Figures 8.17 through 8.20 show how the swell pressure contours change with cycles. Figure 8.17 and Figure 8.19 show how the swell pressure contours begin for the first cycle. These two figures show how on the first wetting cycle the swell pressure is a function of both the dry density and moisture content. As the cycles progress, the swell pressure is no longer a function of both the dry density and moisture content, it becomes a function of only the dry density. This is evident in Figures 8.18 and 8.20. Figure 8.18 shows that as the moisture content changes, the swell pressure is constant at the same dry density. Figure 8.19 shows the swell pressure for Anthem has progressed from being a function of both the dry density and moisture content to being a function of just the dry density. The impact of initial compacted condition on heave characteristic with cycles is discussed in the next section.
8.4 Impact of Initial Compacted Condition on Heave Characteristics with Cycles

In this the impact of the initial compacted condition on heave characteristics with cycles will be assessed. The impacted will compare initial compacted condition with cycle fixing the net normal stress so that the analysis compares the strain associated with each initial compacted condition. Two cases for each soil will be assessed. For the Colorado soil, the initial compacted condition will be compared with cycles for both 1.5 and 100 kPa. For the Anthem soil, the initial compacted condition will be compared with cycles and fixing the net normal stress. The net normal stresses for the Anthem soil comparison includes 42 and 144 kPa. Shown in Figures 8.21 and 8.22 are the impact of the initial compacted conditions for 1.5 and 100 kPa respectively, for the Colorado soil. Shown in Figures 8.23 and 8.24 are the impact of the initial compacted conditions for 42 and 144 kPa, respectively for the Anthem soil.
Figure 8.21: Impact of initial conditions heave characteristics for Colorado subjected to 1.5 kPa net normal stress.

Figure 8.22: Impact of initial conditions heave characteristics for Colorado subjected to 100 kPa net normal stress.
Figure 8.23: Impact of initial conditions heave characteristics for Anthem subjected to 42 kPa net normal stress.

Figure 8.24: Impact of initial conditions heave characteristics for Anthem subjected to 144 kPa net normal stress.
Figures 8.21 through 8.24 show the same general trends. The initial dry density governs the swelling behavior of compacted expansive soils. When comparing the results from within a given density, the specimens that were compacted with higher matric suctions exhibit higher swelling than specimens that were compacted with lower matric suctions. As the cycles progress the difference in swell potential between initial moisture content decreases. The decrease is attributed to the dry density changes during both the wetting and drying cycle. As these cycles progress, the sample slowly approach an equilibrium dry density during both wetting and drying cycle tests. Once the equilibrium dry density is achieved for the specimen, the swelling strains when comparing at the same dry should produce similar strains during the wetting and drying cycle.

In most instances the initial compacted condition of 100%OMC – 90%MDD produced the least amount of swelling as the cycles progressed. Only for the 110%OMC – 90%MDD and the 90%OMC – 90%MDD results cross. The point at which these crossed can be attributed to the time required (cycles required) to reach an equilibrium swell value. Regardless, of the strain shown in the comparison plots, the 90%OMC – 100%MDD produced the highest amount. This is due to the dry density. The dry density governs the swell behavior; however, the matric suction will modify the amount of strain exhibited by the soil profile or specimen. As the matric suction is decreased in the sample, it increases the water in the pores, which when the soil profile or specimen is wetted it decrease the amount of water that can fill the pores. With lower pore volume, it decreases the free water that can interact with the clay mineralogy.
8.5 Implications of the Initial Compacted Condition on Ultimate Heave with Cycles

In this section an example of ultimate heave that can be obtained with the swelling results presented above, which these two soils were subjected to *extreme* matric suction loading. The two soils were subjected extreme matric suction loading that is possible in the field but at near surface conditions. Nevertheless, to achieve this task, one might ask, how does the changes in swell potential and swell pressure effect the overall soil profile. The question will be answer using two profiles, one for the Anthem soil and one for the Colorado soil. The thickness of the profile for the two soils is variable, and depended on the assumption made based on the changes in the soil profile.

The changes in the soil profile are related to the depth to the active zone, applied loading, and initial compacted conditions of the soil. The depth to the active zone is defined as the depth at which a constant equilibrated suction is reached within the profile (Dye et al. 2006). The active zone for Anthem soil is 2.74 meters (9 feet) (Dye et al. 2006). The active zone for the Colorado soil is ranges from 6.1 to 14.0 meters (20.1 to 45.9 feet) (Overton et al. 2006). Using the strain versus net normal stress results presented in section 8.2, the net normal stress can be converted into a depth below the ground surface by dividing the net normal stress by the respected moist unit weight, for each initial compacted condition. In this example, the following assumptions are made:

1) A thick homogenous profile comprised of the same expansive soil placed at the same initial compacted conditions.

2) The depth to the active zone is fully wetted to achieve similar swelling strains as shown in the figures above, for each given cycle.
3) The depth to the active zone is fully dried to achieve similar strains during the drying process.

4) The moist unit weight for the conversion from net normal stress to depth within the profile is constant.

5) No external applied net normal stress.

6) Since it unlikely the soil can dry down to a residual matric suction value deep within the profile, the active zone depth that will be used for Colorado will be 10.4 meters.

A total of three strain profiles were selected to be shown in Figures 8.25 and 8.26. These figures show the strain profiles with depth converted from the net normal stress, for the different initial compacted condition. Figure 8.25 is the strain profiles for the Colorado soil and Figure 8.26 is the strain profile for the Anthem soil.
a) 110\% \text{OMC} – 90\% \text{MDD} \\
b) 90\% \text{OMC} – 90\% \text{MDD} \\
c) 100\% \text{OMC} – 95\% \text{MDD} \\
d) 110\% \text{OMC} – 100\% \text{MDD} \\
e) 90\% \text{OMC} – 100\% \text{MDD}

Figure 8.25: Strain profiles for different initial compaction condition for the Colorado soil.
Figure 8.26: Strain profiles for different initial compaction condition for the Anthem soil.

As one can see, the strain profiles change with cycles. When the strain profile crosses the depth axis or becomes zero that is the depth that the swell pressure occurs for
that particular cycle. For both soils, the initial condition of 110\%OMC – 90\%MDD exhibited largest strain changes with cycles. This is due to the massive reduction in swell pressure after the first cycle, for these soils. The reduction in strain and swell pressure is evident when looking at Figures 8.25 and 8.26. In addition, it is clearly shown that the initial compacted condition of 90\%OMC – 100\%MDD has the large area under the strain curve. The integration under the strain profile will give the ultimate heave/settlement that can occur for the soil profile. The results for the ultimate heave/settlement calculations are shown in Figures 8.27 and 8.28 for the Colorado and Anthem soil, respectively.

![Figure 8.27: Ultimate heave/settlement results for the Colorado soil with cycle.](image-url)
The maximum heave that can occur for the Colorado soil occurs at the first cycle for all initial compacted condition. Anthem, on the other hand, the maximum heave occurred on the third cycle for initial compacted conditions of 90%OMC – 100%MDD, 100%OMC – 95%MDD, and 90%OMC – 90%MDD. Initial compacted condition, 110%OMC – 100%MDD exhibited the largest swelling on the sixth and final cycle. This analysis shows that the first cycle is not always the most harmful to foundations due to the larger heave on the second cycle for Anthem. It is interesting to note that the swell pressure reduced with cycles, which shifts the strain profile negative below the swell pressure. However, the swelling strain grows exponentially with the cycle thus increasing or maintaining the swell the ultimate heave at the ground surface.

Nevertheless, this analysis was based upon using active zoned that reached 2.74 meters for the Anthem soil and an active zoned that reached 10.4 meters into the
homogenous profile. It is possible that the active zone is either too deep or too shallow. In those instances, the results will be very different. If the active zone increases to a shallower depth, it will potentially increase the heave since the strain profile is greater than zero within the region of interest. On the other hand if the active zone is too deep, the profile will exhibit swelling. However, the amount of swelling will become negligible since the integration of the collapse strain will be greater than using a shallower active zone.

8.6 Summary and Conclusions

The swelling strain is recoverable at the light token stresses and is irrecoverable higher stresses. The recoverable or irrecoverable strain can be attributed the changes in void ratio (i.e. the dry density) during the cycle test. As the cycles progressed the material would reach an equilibrium swell or collapse potential due to the preconditioning effect of the net normal stress and the matric suction changes during the test (i.e. the moisture content). Since the net normal stress was fixed, for each sample, the preconditioning effects that the sample exhibited after the first cycle was attributed to the changes in matric suction. The first cycle conditioning, on the other hand, is a function of both the net normal stress and the matric suction.

The changes in the environmental conditions caused the changes in matric suction. When the sample was inundated with water, the pores of the soil would fill up with water releasing the tension on the water thus lowering the matric suction. When water was removed from the system and the temperature increased, it caused an increase in matric suction since water was released from the sample which increased the tension
on the water thus increasing the matric suction. In addition, the increased tension also generated tension cracks within the sample due to the tensile stresses were greater than the tensile strength.

Regardless of the initial matric suction (or moisture content) of the material, the dry density governs the swell characteristics of expansive soils (Chen 1988). However, one can argue that the dry density governs swelling but the matric suction (or moisture content) will also dictate the amount as well, when comparing samples subjected to the same net normal stress. The matric suction (or moisture content) tends to alleviate the swell potential as the matric suction is decreased. The swell potential tends to increase when the matric suction is increased, thus making the swelling potential and pressure function of the compacted moisture content and dry density.

Rao et al. (2004) and Holtz and Gibbs (1956) showed that the as the density increased the swell potential of the soil would increase as well. They also showed that increasing the moisture content for a given density would decrease the swell potential. The results obtained from this laboratory study shows similar trends as Rao et al. (2004) and Holtz and Gibbs (1956). The difference between this study and those two studies, the test specimens were not subjected to multiple wetting and drying cycles.

The effects of wetting and drying cycles were assessed. Several authors have studied this phenomenon prior; however, most of the authors only assessed the effect of swelling potential after multiple wetting and drying cycles using only one initial compacted condition and one net normal stress normally a token load. Six of the 17 researchers showed a reduction in both the swell pressure and potential (Al-Homoud et
al. 1999; Dif and Blumel 1991; Guney et al. 2007; Kalkan et al. 2011; Tripathy and Subba Rao 2009; Yazdandoust and Yarobi 2010). However, from the results obtained from this study, the reduction in swell potential occurs at a particular net normal stress. Stresses smaller than this stress show an increase in swell potential and all stresses larger than this stress show a decrease, which is similar results obtained by these six researchers. In addition, it was shown that the swell pressure reduces with cycles regardless of the initial compacted condition. The initial compacted condition changes the rate at which the reduction occurs.

It was also shown by Tripathy et al. (2002), Basama et al. (1995) and Subba Rao and Satayadas (1987) that there is a reduction in swelling if full wetting and partial drying occurs. The partial drying reduces the amount of strain seen in the next cycle. In this study partial drying was not assessed but what partial drying shows is that if a specimen is subjected to full wetting and full drying, the minimum reduction in swell is observed. However, the strain will increase with full wetting and full drying at lighter stresses. The increase in swelling strain at lower stresses that was reported and convert to ultimate heave might be different when partial drying occurs. Nevertheless, the maximum reduction in swell pressure that is possible will be captured with full wetting and partial drying.

The initial compacted conditions of 110%OMC – 90%MDD will be the best initial compacted conditions suited for expansive soils. If expansive soils are to be used in construction, the compaction specification shallow not be greater than 90% of the maximum dry density obtained from the Standard Proctor. It was shown that increasing
the maximum dry density from 90 to 100 percent of the standard Proctor the swell pressure would increase by a minimum 2.6 times the swell pressure that would have been achieved at 90% MDD. The typical recommendations for expansive soils compaction is as follows: 95% MDD and plus 2% OMC, which interpolating between 110%OMC – 100%MMD and 110%OMC – 90%MDD would increase the swell pressure by a minimum of 1.56 times the swell pressure that would been achieved at 110%OMC – 90%MDD. Therefore, current recommendations for expansive soil compacted needs to be updated and addressed.

It is possible to extrapolate these finding by suggesting that all CL soils will behavior similarly to Anthem and the swell pressure reduction from the first cycle to an equilibrium swell pressure, the equilibrium swell pressure can be reduced by 46 percent of the initial swell pressure. In addition, it is possible to postulate that the findings for the Colorado soil can be extrapolated to all CH soil for swell pressure reduction. The swell pressure reduction from the first cycle to an equilibrium value that could potentially be used for CH soils would reduce the first cycle swell pressure by 68 percent.

The maximum heave that can occur for the Colorado soil occurs at the first cycle for all initial compacted condition. Anthem, on the other hand, the maximum heave occurred on the third cycle for initial compacted conditions of 90%OMC – 100%MDD, 100%OMC – 95%MDD, and 90%OMC – 90%MDD. Initial compacted condition, 110%OMC – 100%MDD exhibited the largest swelling on the sixth and final cycle. This analysis shows that the first cycle is not always the most harmful to foundations due to
the larger heave on the second cycle for Anthem. The swelling strain grows exponentially with the cycle thus increasing or maintaining the ultimate heave at the ground surface.

8.7 Recommendations

1) It was shown, for the two soils, the most ideal initial compacted condition is $110\%\text{OMC} – 90\%\text{MDD}$.

2) Add additional soils to validate the compaction recommendations.

3) It seems the minimum swell pressure reduction from the first cycle to an equilibrium cycle for CL soils would reduce the swell pressure by 46 percent.

4) It seems the minimum swell pressure reduction from the first cycle to an equilibrium cycle for CH soils would reduce the swell pressure by 68 percent.
Chapter 9

SUMMARY AND CONCLUSIONS

9.1 Summary

A 2009 study estimated that expansive soils have been attributed to 13 billion dollars of damage to US infrastructure (Cerrato and Puppala 2009). FEMA does not classify expansive soils as a natural disaster and expansive soils do more damage than all other natural disasters combined. In addition, the damage done by expansive soils is not covered by insurance and the owner of the property has to find appropriate means to pay for the damage. It is possible to mitigate the damage generated by expansive soils by completing appropriate laboratory testing aimed at understanding their behavior. The laboratory testing requires either undisturbed or remolded specimens. In an ideal setting, undisturbed samples should be used for classification purposes as well as determining the swelling characteristics of expansive soils. However, in the absence of undisturbed samples, remolded samples are used. When remolded samples are used, the soil is normally remolded to one density and matric suction that would best represent the field condition. The remolded samples are then subjected to laboratory testing, in which specimens are inundated or subjected to only one wetting cycle. The deformations are then recorded at the end of the wetting cycle, the results of the laboratory tests are determined, and recommendations for design are then made. However, the field condition is ever changing due to changes in the environmental or climatic conditions (i.e. wetting and drying cycles). Environmental factors create a cyclic change in the stress state of the soil represented by changes in the matric suction (i.e. moisture content). Therefore, is it
appropriate to make recommendations for recompacted expansive soils based upon one extreme wetting event?

The main study of this dissertation was aimed at answering the following questions:

1. Do the current standards of practice appropriately address the swelling characteristics of remolded expansive soils?

2. Does the initial compaction condition affect the swell potential and swell pressure after cyclic changes in matric suction?

3. Does the mechanical behavior of expansive soils change after cyclic changes in matric suction when the expansive soil is subjected to different applied net normal stresses?

In order to answer these questions, three objectives were defined. They were:

1. Assess the effect of the net normal stress on the volume change behavior of compacted expansive soils. The fact that the existing standards of practice rely upon different “token” loads to determine “free swell” was of particular concern.

2. Assess the effects of initial moisture content and dry density on the swell pressure of a compacted expansive material subjected to not only a single wetting/drying cycle but to multiple cycles.

3. Determine the effects of wetting and drying cycles on the volume change and mechanical behavior of compacted expansive soils. The effect on “free swell” and the effect on swell pressure were to be investigated in the study. The analysis was to be performed at a variety of differing net normal stress levels.
These objectives were assessed by completing a comprehensive literature review, two companion studies, and the main study.

The literature review, presented in Chapter 2, included the following topics: engineering properties of expansive soils; guidelines of swell potential determination based on engineering properties; how expansive soil volume change is currently measured by practitioners; empirical correlations to predict heave; factors that affect swelling; volume change associated with wetting and drying cycles; and comparison of 3-D and 1-D results.

The soil characterization for the four soils that were used in different capacities is presented in Chapter 3. The soil properties of interest included the grain size distribution, the Atterberg limits, the optimum moisture content and maximum density determined by standard Proctor compaction, the specific gravity of the soil solids, and soil water characteristic curves. Using the grain size distribution and the Atterberg limits, the soils were classified using the unified soil classification system. The four soils were determined to have the following classifications per the USCS: Clayey Sand (1 soil), Lean Clay (1 soil) and a Fat Clay (2 soils). Anthem was classified as a Lean Clay, Colorado was classified as a Fat Clay, San Antonio was classified as a Fat Clay, and San Diego was classified as a Clayey Sand.

The main study consisted in assessing how five different compaction conditions, in conjunction with five different net normal stresses applied to the specimens, affected the swell potential and pressure of expansive soils, during and after cyclic changes in matric suction. The two companion studies included (1) the assessment of the expansive
soil response to differences in the token loads recommended by local and international recognized standards of practice; and (2) the assessment of radial strain and moisture content changes during the cyclic changes in matric suction.

The design of the experiment needed to answer the objectives of this dissertation was presented in Chapter 5. It was decided to conduct the experiment on two soils: a CL soil from Anthem (AZ) and a CH soil from Denver (CO). The specimens were subjected to four net normal stresses: 1.5kPa, 40%, 80%, and 140% of the swell pressure, which was obtained for the initial compacted condition of 95 percent of the maximum dry density and optimum moisture content. It was also decided to compact the specimens at five different initial compaction conditions. These conditions corresponded to the following combination of optimum moisture content (OMC) and maximum dry density (MDD):

- 90% of the OMC and 100% of the MDD
- 90% of the OMC and 90% of the MDD
- 110% of the OMC and 90% of the MDD
- 110% of the OMC and 100% of the MDD
- 100% of the OMC and 95% of the MDD

The optimum moisture content and maximum dry density were determined based on the ASTM D698 using standard Proctor energy. Finally, the specimens were subjected to six complete wetting and drying cycles. The combination of the desired parameters allowed for the testing of 576 conditions. Additional tests were performed as needed to fully develop the percent swell versus net normal stress relationships, to
eliminate variability associated with the testing procedure, and/or to improve correlations. Based on the comprehensive literature review, the parameters chosen to conduct this study were deemed to be the most important variables to understand swell characteristics of compacted expansive soils.

9.2 Conclusions

9.2.1 Comparison of Non-Linearity in Swelling Strains Associated with a Single Cycle

As part of the laboratory study, the effect of different token loads (specified in various standard methods of practice) on swelling potential were addressed in Chapter 4. It was evident from the results that the swelling/collapse strain is highly non-linear at token loads (low net normal stresses) and the strain-net normal stress relationship cannot be defined by one single function without submitting the data to log-transformation. Based on these results, several functions were attempted to best fit the data. Results showed that when a logarithmic function is used, the larger net normal stresses govern the regression analysis, resulting in the underprediction of near surface strains. Therefore, the dataset needs to be either subdivided into two groups so that a logarithmic function can be used (bi-modal fit); or use a third order polynomial fit through the data when plotted in a semi-log plot.

It was apparent that at about 7 kPa, the swelling strain rate changed. Therefore, if the bi-modal logarithmic fit is desired, it is recommended that the strain values corresponding to net normal stresses less than 7 kPa should be fit to the first natural logarithmic fit; while the strains at higher stress levels should be fit to the second natural
logarithmic fit. It should be noted that if a single natural logarithmic function is used (instead of the proposed bi-modal model), swelling at 1 kPa were underpredicted by at least 3 to 4.5 percent for the soils tested in the study, which corresponds to a swell potential estimation error of 50 percent. On the other hand, if a polynomial fit is used, either a second or third order polynomial function can be used to best describe the data. When using a second or third order polynomial function the results of the swelling strain net normal stress needs to be analyzed using a semi-log polynomial fit where the net normal is plotted in the log scale. A second order polynomial is recommended when five or less data points are available. Conversely, a third order polynomial fit is recommended when there are six or more strain-net normal stress data points available.

It was observed that at least six swelling strain versus net normal stress data points were necessary in order to eliminate most of the uncertainty associated with the fitted function. The data points that resulted in the best estimate of swell potential comprised of light net normal stresses (i.e. net normal stresses less than 7 kPa) and heavy net normal stresses (i.e. net normal stresses near the swelling pressure). The light net normal stresses included values such as 1 kPa, 4 kPa, and 7 kPa. The heavy net normal stresses included stresses that correspond to 80, 120, and 140 percent of the swell pressure. It appears that the swell strain could be modeled with confidence, when using these six prescribed net normal stresses.

9.2.2 Moisture Content and Radial Strain Changes with Cycles

The results of the assessment of the evolution of the radial strain and moisture content changes during cycles of wetting and drying were presented in Chapter 6. This
assessment was necessary and very valuable in order to estimate intermediate or instantaneous volumetric strain results, for the main study. The assessment of the moisture content variation within the cycles, on the other hand, was deemed useful for the analysis presented in Chapter 8. The moisture content variation during the wetting and drying cycles found to follow two different non-linear paths. The path followed during the wetting cycle was found to be best represented by a natural logarithmic function; while the path followed during the drying cycle was found to be best represented by an exponential curve.

The radial strain variation during the wetting and drying cycles was observed and recorded for the 11 specimens tested. For all of the samples subjected to the wetting cycle, it was observed that for light net normal stresses, the radial strain became zero within the first five minutes of loading. That made the volumetric strain equal to the axial strain for the wetting cycles. It was also observed that for higher net normal stresses, the radial strain became zero within the first 55 minutes of loading/inundation for all specimens tested. The radial strain change observed during the drying process allowed for enough data points to be recorded; and therefore, it was possible to determine a best fit function through the data by using the least square regression analysis. Based on these results, it was observed that when the radial strain was divided by the axial strain, at any measured point during the cycle, it resulted in nearly a constant ratio.

The results of the main experiment are shown in Chapter 7. The results included the axial and volumetric strain analyzed by the “as compacted” height/volume and by each cycle (i.e. the height/volume at the beginning of the cycle prior to wetting/drying)
for both wetting and drying cycles. These strain results were obtained by analyzing the primary swell or collapse from each test result. The primary swell or collapse was analyzed by using the methodology outlined in ASTM D4546; where the strain/displacement was plotted versus the log time.

9.2.3 Swelling Strain Changes with an Increase in Cycles

The swelling strain is recoverable at the light token stresses and is irrecoverable at the higher stresses. The recoverable or irrecoverable strain can be attributed to the changes in void ratio (i.e. the dry density) during the cycle test. It was observed that as the cycles progressed, the material reached an equilibrium swell or collapse potential due to the preconditioning effect of the net normal stress and the matric suction (i.e. the moisture content) changes during the test. Since the net normal stress was fixed for each specimen, the preconditioning effects that the sample exhibited after the first cycle was attributed to the changes in matric suction. It should be noted that the first cycle conditioning, on the other hand, is a function of both the net normal stress and the matric suction.

The changes in the environmental conditions caused the changes in matric suction. When the sample was inundated with water, the pores of the soil would fill up with water releasing the tension on the water and thus, lowering the matric suction. When water was removed from the system and the temperature increased, it caused an increase in matric suction due to the increase in the tension on the water. In addition, the tension increase also generated tension cracks within the sample due to the tensile stresses being greater than the tensile strength.
Regardless of the initial matric suction (or moisture content) of the material, the dry density governs the swell characteristics of expansive soils (Chen 1988). However, one can argue that the dry density governs swelling but the matric suction (or moisture content) will also dictate the amount as well, when comparing samples subjected to the same net normal stress. As the matric suction decreases, the swell potential decreases; thus, making the swell potential and swelling pressure functions of the compacted moisture content as well as the dry density.

Rao et al. (2004) and Holtz and Gibbs (1956) showed that as the compaction dry density increased, the swell potential of the soil increased as well. These authors also showed that increasing the compaction moisture content, for a given dry density, would decrease the swell potential. The results obtained from this laboratory study showed similar trends as Rao et al. (2004) and Holtz and Gibbs (1956). The difference between this study and those two studies was that those two studies tested specimens that were not subjected to multiple wetting and drying cycles.

In this study, the effects of wetting and drying cycles were also assessed. Several authors have studied this phenomenon before. However, most of the authors only assessed the effect of swelling potential after multiple wetting and drying cycles using only one initial compaction condition and one net normal stress (generally at a token load). Six of the 17 researchers showed a reduction in both the swell pressure and potential as the number of cycles increased (Al-Homoud et al. 1999; Dif and Blumel 1991; Guney et al. 2007; Kalkan et al. 2011; Tripathy and Subba Rao 2009; Yazdandoust and Yarobi 2010). However, the results obtained from this study indicate that the
reduction in swell potential occurs at a particular net normal stress. Stresses smaller than this net normal stress, show an increase in swell potential while all stresses larger than this net normal stress, show a decrease in swell potential. Results (swell pressure reduction) obtained by these six researchers seems to agree with the results obtained from this study. However it was shown in this research, the swell potential did not decrease with cycles once the applied net normal stresses was lighter than a certain stress level.

It was also shown by Tripathy et al. (2002), Basama et al. (1995) and Subba Rao and Satayadas (1987) that there is a reduction in swelling if full wetting and partial drying occurs. The partial drying reduces the amount of strain observed in the next cycle. In this study, partial drying was not assessed. However, it is safe to conclude that if a specimen is subjected to full wetting and full drying, the minimum reduction in swell pressure is observed. The minimum reduction in swell pressure will be attributed to the magnitude of the swell strain changes with net normal stress. Partial drying with full wetting shows a decrease in swell potential at the lower stress levels, which will decrease the slope of the strain-stress curve thus lowering the point at which zero strain occurs. Also, the strain will increase with full wetting and full drying at lighter stresses. The increase in swelling strain at lower stresses was reported and converted to ultimate heave. It should be noted that these results might be different when partial drying occurs.

The swelling strain results from this study compares well with the results published in literature. Tripathy et al. (2002), Dif and Blumel (1991), and Subba Rao and Satayadas (1987) showed the strain will increase with full wetting and full drying at lighter stresses. In addition, these researchers showed the strain will decrease with full
wetting and drying at larger net normal stresses. Subba Rao and Satayadas (1987) showed that the large changes in strain associated with the wetting/drying cycles are due to the starting degree of saturation or density and matric suction at the beginning of the cycle.

9.2.4 Swell Pressure Changes with an Increase in Cycles

The results from the laboratory tests showed empirically after four to five cycles the swell pressure stabilizes to an equilibrium swell pressure. The equilibrium swell pressure had a minimum reduced from the initial cycle to the equilibrium cycle, for all five initial compaction conditions, of 43 percent for the Anthem soil and 69 percent for the Colorado soil. When evaluating the swell pressure from one cycle to the next, the reduction in swell pressure increases with cycles until an equilibrated swell pressure is obtained. After one cycle, it was shown that Anthem exhibited a minimum reduction of 20 percent and a maximum reduction of 77 percent, for all five initial compaction conditions. In addition the Colorado soil exhibited a minimum reduction of 50 percent and a maximum reduction of 69 percent, for all five initial compaction conditions. The difference between the two soils is due to the magnitude of the swell pressure and soil type.

There is an evolution in swell pressure changes with an increase in cycles. On the first cycle, it was shown that the swell pressure is a function of both the compacted dry density as well as the matric suction. As the cycles progress the swell pressure becomes more of a function of the dry density and the matric suction has little to no effect on the swell pressure results. These results were shown for both the Anthem and Colorado soil. The Anthem soil seemed to be more of a function of the moisture content than the
Colorado soil when comparing the results of the last cycle or cycle number six. The Colorado soil, conversely, showed by the sixth cycle the swell pressure was constant for the same density regardless of the matric suction. Nevertheless, when comparing the first cycle, both soils showed similar swell pressure contours around the initial compaction conditions that were tested.

When comparing the results of the equilibrated swell pressure to the initial cycle swell pressure the reduction is quite significant. The maximum reduction (77%) exhibited by the Anthem soil would corresponds to a difference of 45 kPa. This difference corresponds to 2.52 meters of either additional fill or the soil within the profile that contributes to the consolidation or collapse. The results are similar for the Colorado soil. The maximum reduction (69%) exhibited by the Colorado soil corresponds to a value of 185kPa difference in swell pressure. A difference of 185kPa can correspond to a difference of 9.77 meters of additional consolidation or collapse when evaluating only the difference of the swell pressure.

In addition, it was shown that the swell pressure reduces with cycles regardless of the initial compacted condition. The initial compaction condition changes the rate at which the reduction occurs. The reduction associated with each initial compaction condition can be attributed to the initial dry density. The highest reduction in swell pressure occurs at the highest matric suction when evaluating the same initial low dry density (90%MDD). On the other hand, when evaluating the same initial high dry density (100%MDD) the lowest matric suction had the highest reduction. It seems in both cases
the initial condition that had the lowest reduction in swell pressure was closer to the equilibrated condition.

As stated earlier, these laboratory results were obtained using full wetting and full drying. If full wetting and partial drying results were obtained for the same soils, the swelling strains would be lower. Having lower swelling strain would lower swell pressure value. Therefore, it seems that the swell pressure reduction that is exhibited by full wetting and full drying will produce the minimum reduction in swell pressure with each subsequent cycle.

No literature has been published, at this time, evaluating the swell pressure changes with cycles using multiple sample method. Therefore, a comparison between the results found in this study cannot be compared to results from other authors since the published data focuses on constant volume swell pressure only. Guney et al. (2007), Kalkan et al. (2011), Yazdandoust and Yarobi (2010), and Basma et al. (1996) showed a decrease in the constant volume swell pressure with an increase in full wetting and partial drying cycles. For these authors the samples were dried back to their initial moisture contents. After the samples were dried the samples were then subjected to the constant volume swell test again until a certain number of cycles were achieved. It was shown that from these results the equilibrium swell pressure required four to five cycles, which is similar to the results presented in this research.

It was also shown that the constant volume swell pressure increased with cycles. Basma et al. (1996) and Doostmahammadi and Moosavi (2009) showed an increase in swell pressure with an increase in cycles. The increase in swell pressure can be attributed
to the start of the testing condition. The samples were fully dried or dried past the shrinkage limit and then the constant volume swell test was then performed on the sample, which had a different height from the previous cycle. If the sample was allowed to swell back to the original volume, from the initial condition, it is possible the full wetting and full drying constant volume swell test would provide the same trend as the partial shrinkage constant volume swell tests. In addition, these results show the constant volume swell pressure equilibrates around the fourth or fifth wetting cycle.

Singhal (2010) compared the swell pressure results using the different methodologies to obtain swell pressure. Singhal showed compacting a soil to the same initial conditions and subjected to different net normal stresses, the load back swell pressure would be slightly different due to the stress dependency of the material. In addition to the remolded samples, Singhal tested swell pressures of intact samples. The intact samples showed

Even though the only results that were documented in literature were results from constant volume swell tests, the partial shrinkage results showed a reduction in swell pressure with each cycle similarly to the results shown in this study. In addition, the percent reduction from the first cycle to the second cycle was similar for the results presented in this study as well as the results presented by Basma et al. (1996), Guney et al. (2007) and Yazdandoust and Yarobi (2010). The percent reduction in swell pressure from the first cycle to the second cycle ranged from 18 percent to 60 percent, which is similar to the two soils tested in this study. Therefore, it seems that if a compacted expansive soil is subjected to one wetting cycle prior to construction, the reduction that is
seen will range from 18 to 60 percent depending on the initial compaction condition (i.e. density and matric suction).

**9.2.5 Remolded and Undisturbed Comparison**

Naturally occurring or undisturbed expansive soils have been subjected to several wetting and drying cycles over the centuries, which creates equilibrated pore geometry and equilibrated soil structure. When the expansive soils are sampled, and then remolded it changes the original structure of the soil. Remolded samples are considered disturbed samples where the original soil structure was destroyed. The remolding process causes the destruction of the original soil structure by modifying the pore spacing and geometry. The modification alters the suction of the soil as well as the mechanical behavior when comparing the behavior of remolded samples to undisturbed expansive samples (Mitchell and Soga 2005; Seed et al., 1961). Nonetheless, remolded samples are used to estimate the settlement or heave of compacted fills, retaining structures, and highway subgrades (Attom et al. 2006). Remolded samples are used for these applications since the original soil structure is destroyed in the field during the construction process.

The laboratory testing on remolded samples will equilibrate to the conditions that the sample is subjected to, which will not create similar pore geometry or soil structure that was developed over the centuries. These inconstancies between the results lead to the over/underprediction of field behavior when using remolded samples. Therefore, remolded samples should never be used to quantify or estimate field behavior of intact or undisturbed expansive soils subjected to different net normal stresses and matric suction loading (Singhal 2010).
If the field condition requires remolding, then the results from the laboratory testing will be more appropriate in estimating the field behavior. With the current standards of practice, the results of the laboratory only are valid when estimating heave/collapse when the profile is fully wetted. Current standards of practice do not address how the behavior of expansive soils when subjected to different net normal stresses or matric suction loading. Therefore, a comprehensive standard of practice is needed to assess the cyclic nature of remolded expansive soils.

9.2.6 Practicality of the Results

The results presented in this study were results obtained from using remolded expansive soils and subjecting the remolded samples to extreme drying conditions that can occur in near surface conditions. Expansive soils in the field will not undergo the large suction changes that were seen in this study below near surface conditions.

It was shown that as the cycles increased at light stress levels, the remolded samples would swell significantly with each cycle, which creates large increase in swelling strains at the near surface. In addition, it was shown that at higher stress levels, remolded samples would collapse or compress significant with each cycle, which creates large increase in the collapse strain. When the strain results are used to determine the swell pressure with each cycle, it showed the swell pressure would decrease, which is counterintuitive. Common belief about the behavior of expansive soils is that as the swell strain increases the swell pressure will increase as well. The increase in both swelling strain and swell pressure is recorded throughout literature by various authors. However, this phenomenon is of both factors increasing is based upon a single wetting cycle. As a
remolded expansive soil is subjected to cyclic changes in the matric suction, the dry
density of the material will attempt to approach an equilibrium condition. This
equilibrium condition for large stresses will cause the material to collapse or compress
under loading thus generating negative strain.

Dif and Blumel (1991), Tripathy et al. (2002), and Subba Rao and Satayadas
(1986) showed that as a remolded sample is subjected to large net normal stress and
cyclic change the swell potential will decrease. In addition, these authors also showed an
increase in swell potential at the light stresses. When the results of these tests are
analyzed using the original height/volume as the initial reference point, they show similar
trends were the swell pressure decreases with cycles as the swell potential at the light net
normal stresses increase significantly. These results from these authors are similar to the
results presented in this study. Therefore, the impact on recompacted expansive soil
behavior is governed by the magnitude of the net normal stress after it is subjected to
cyclic changes in the net normal stress.

Nonetheless, the example, presented in Chapter 8, showed that the ultimate
heave/settlement that could be obtained if these conditions of extreme drying and full
wetting could occur within the soil profile that was remolded throughout to the depth of
the analysis. The Anthem soil profile would heave, for most of the initial conditions, and
settle with the Colorado profile, for most of the initial conditions. As the cycles
progressed the Anthem profile saw an increase in heave while the Colorado profile
showed a decrease in heave. The initial compaction condition of 110 percent of the
optimum moisture content and 90 percent of the standard Proctor maximum dry density

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showed the highest reduction in heave with an increase in cycles. For the Anthem soil after six cycles of wetting and drying the profile would exhibit almost no movement, once it was fully wetted (results of the 110%OMC–90%MDD). The Colorado soil on the other hand, exhibited settlement after each cycle, which by the sixth cycle the reduction in ultimate heave would be 685 percent reduction from the initial cycle ultimate heave calculation.

9.2.7 Compaction Guidelines

Finally it was demonstrated that the initial compacted conditions of 110%OMC – 90%MDD would be the best initial compaction conditions suited for expansive soils. If expansive soils are to be used in construction, it appears that the compaction shall not exceed 90% of the maximum dry density obtained from the standard Proctor. It was shown that by increasing the maximum dry density from 90 to 100 percent of the standard Proctor, the swelling pressure increased by a minimum of 2.6 times. It should be noted that typical recommendations for expansive soils compaction call for 95% MDD and plus 2% OMC, which, by interpolation of the results obtained represents an increase of the swell pressure by a minimum of 1.6 times of that achieved at 110%OMC – 90%MDD. Therefore, current recommendations of expansive soil compaction needs to be updated and addressed.

The results from this research compares well with published literature. Rao and Revanasiddappa (2006) state similar conclusions. Rao and Revanasiddappa (2006) tested one soil at the same initial density but at different moisture contents and subjected the samples to multiple wetting and drying cycles. They recommended that compacting at
two percent higher than the optimum moisture content will mitigate most of the swell potential of expansive soils. It was also shown in the swell potential contour plot generated by Holtz and Gibbs (1956) at two percent greater than the optimum moisture content the swell potential will be less the swell potential generated at lower moisture contents. Ito (2009) showed that compacting at higher moisture contents and lower densities, it will mitigate swell potential and swell pressure.
Chapter 10

RECOMMENDATIONS FOR FUTURE WORK

10.1 Recommendations for State of the Practice

The recommendations for the state of the practice will focus on the testing standards that are currently used by practitioners to assess the swelling/collapse characteristics of expansive soils. The current testing standards used in Arizona include ASTM D 4546, ASTM D 4829, and the Arizona Expansion Potential. Each recommendation will focus on issues that were apparent from the laboratory testing performed. In addition, a recommendation for an improved testing standard is discussed.

10.1.1 Develop a New Standard Test for Assessing Environmental Conditions on Swell or Collapse Behavior of Cohesive Soils

Compacted expansive soils are subjected to varying environmental conditions in the field and current laboratory test procedures do not address them properly. Soils in the field are subjected to various environmental conditions that include wetting and drying (changes in suction loading) under various external loading conditions (i.e. different net normal stresses). Current standards only assess strain after one wetting cycle from an initial compacted condition that corresponds to either current or future field compaction characteristics. Therefore, the swell or collapse behavior of cohesive soils expected in the field should be assessed under different environmental loading conditions. It is expected that by using an improved test method as that proposed below, will allow for a more reliable prediction of swelling strain, swelling pressure with cycle, and the ability to assess the ultimate heave associated with the given soil.
A new standard method should carefully address the following criteria:

1) *Number of net normal stress levels*: the minimum number of net normal stress levels should be five but six would be more appropriate.

2) *Net normal stress levels*: the net normal stress levels should include:
   
   a. 1.5 kPa
   
   b. 4.0 kPa (if six samples)
   
   c. 7.0 kPa
   
   d. 80% ± 10% of swell pressure
   
   e. 120% ± 10% of swell pressure
   
   f. 140% ± 10% of swell pressure

3) *Number of cycles*: the minimum number of cycles recommended is four cycles but six would be more appropriate.

4) *Initial compacted condition*: the initial compacted condition should correspond to the estimated field condition. However, the lower the compaction dry density and the higher the water content, the less swelling potential should be expected after a number of wetting/drying cycles.

5) *Modeling moisture content changes*: The moisture content change with respect to time can be modeled as follows:
   
   a. The wetting cycle can be adjusted to a natural logarithmic function by using only the initial and final moisture content data points. An initial time of 0.01 minute can be used.
b. The drying cycle can be adjusted to an exponential decay function using only the initial and final moisture content data points. An initial time of 0.01 minute for the initial moisture content can be used.

6) Radial strain: The radial strain changes over time can be assessed as follows:
   a. The radial strain for the drying cycle can be approximated by a constant rate of change when compared to the axial strain, at any given time. The rate of change is determined by dividing the final radial strain by the final axial strain.
   b. The radial strain for the wetting cycle is more difficult to assess and more testing to determine the approximated rate is needed. However, it seems that the radial strain approaches zero after 5 minutes of inundation for lightly loaded specimens. The radial strains for larger net normal stresses need more evaluation.

7) Strain potential versus Net normal stress: It appears that the strain potential versus the net normal stress should be model by using a semi-log third order polynomial function to fit the data. This model will allow for the assessment of the swell strain data that was not collected during laboratory testing.
### Table 10.1: Recommendations for New Testing Standard.

<table>
<thead>
<tr>
<th>Criteria</th>
<th>Suggested</th>
</tr>
</thead>
<tbody>
<tr>
<td>Net normal stress</td>
<td>Minimum of 6</td>
</tr>
<tr>
<td>Number of cycles</td>
<td>Minimum of 4 cycles</td>
</tr>
<tr>
<td>Initial compacted condition</td>
<td>Expected field compacted condition</td>
</tr>
<tr>
<td>Wetting moisture content</td>
<td>Use a natural logarithmic function</td>
</tr>
<tr>
<td>Drying moisture content</td>
<td>Use an exponential function</td>
</tr>
<tr>
<td>Wetting radial strain</td>
<td>Assume fully healed sample within 55 minutes</td>
</tr>
<tr>
<td>Drying radial strain</td>
<td>Constant rate of change between axial and radial</td>
</tr>
<tr>
<td>Net normal levels</td>
<td>1.5 kPa, 4.0 kPa, 7.0 kPa, 80% ± 10%, 120% ± 10%, and 140% ± 10% of $\sigma_{SP}$</td>
</tr>
<tr>
<td>Modeling</td>
<td>Use a semi-log third order polynomial fit</td>
</tr>
</tbody>
</table>

10.1.2 ASTM D 4546 “Standard Tests Method for One-Dimension Swell or Collapse of Cohesive Soils” Method A Recommendations”

The recommendations for ASTM D 4546 are as follows:

1) Increase the minimum number of tests from four to six for Method A of 2008 edition to better represent the swelling/collapse characteristics of a cohesive profile.

![Different semi-log regression analyses for Colorado.](image)
Even though the $R^2$ is still above 96 percent for both the 6 point and all point analyses, the near surface swelling strains are under predicted by 1.83 and 2.49 percent for the 6 point and all points analyses respectively. As one can see from Figure 10.1 the semi-log regression analysis is highly weighted by the higher net normal stresses and the lighter stresses are not predicted with great accuracy; therefore, another token stress should be used in the analysis. Shown in Figure 10.2 is the semi-log regression analysis separated for the token stresses and the larger stresses for the Colorado soil. As one can see, the fit for both the token stresses and the higher stresses better represent the data. Using a bimodal semi-log fit to the data set, the largest error associated the under/over prediction of the swell/collapse strain was 0.75 percent, for the Colorado soil.

![Figure 10.2: Bimodal semi-log regression analysis for Colorado.](image)

2) Change how the swelling/collapse strains are modeled.
To better understand the swell/collapse strain of expansive soils use the following stresses: a light token stress (e.g. 1 kPa), an intermediate token stress (e.g. 5 kPa), heavy token stress (e.g. 7 kPa), a stress that is 80±10, 120 80±10, and a stress that is 140±10 percent of the swell pressure. Figure 10.1 shows how the swelling strain can be overpredicted when only four net normal stresses are used.

3) Use a semi-log third order polynomial fit of the swell/collapse strain data with a minimum of six data points (i.e. six different specimens loaded to six different stresses) can be utilize to predict the strains within the stresses of interest.

Figure 10.3 show third order polynomial fit of the Colorado soil. By using a third order polynomial fit, it allows for an appropriate use of a single non-linear function to describe the swelling strain at any given net normal stress within the tested points of interest.

**Figure 10.3: Third order polynomial fit for the Colorado soil.**
4) Change the recommended token stress from 1 kPa of 7 kPa allowing the use of a natural logarithmic function to predict the strains within the stresses of interest that were not tested.

Figure 10.1 and Figure 10.2 show the difference in using 1 kPa and 7 kPa as the token stress for the different analyses. As one can see the difference between the two points are substantial.

5) Develop new models to describe the coefficient of swelling/rebound for expansive soils at token stresses and higher stresses.

Figure 10.2 shows the difference in swelling strain when the non-linearity of the data is recognized. The coefficient of swelling/rebound for the expansive soil shown in Figure 10.2 would correspond to the swell/collapse strain equation for the larger stresses. The implication of using this coefficient would correspond to underpredicting the swell strain at 1 kPa by 5.7 percent. An underprediction of 5.7 leads to an error in estimating swelling potential by 50%. Therefore, to better represent the swell potential of an expansive soil use a bimodal coefficient of swell/rebound. Table 10.2 shows the possible changes that would improve ASTM D4546 for remolded expansive soils.

<table>
<thead>
<tr>
<th>Criteria</th>
<th>Current ASTM D4546</th>
<th>Suggested</th>
</tr>
</thead>
<tbody>
<tr>
<td>Net Normal Stress</td>
<td>Minimum of 4</td>
<td>Minimum of 6</td>
</tr>
<tr>
<td>Net Normal Levels</td>
<td>Token and 3 others of Interest</td>
<td>1.5 kPa, 4.0 kPa, 7.0 kPa, 80% ± 10%, 120% ± 10%, and 140% ± 10% of $\sigma_{SP}$</td>
</tr>
<tr>
<td>Modeling</td>
<td>Semi-log fit or interpolate between large stresses</td>
<td>Use bi-modal natural log fit or third order polynomial fit</td>
</tr>
</tbody>
</table>
10.1.3 ASTM D 4829 “Standard Test Method for Expansion Index of Soils”

ASTM D 4829 is an index test that assesses the expansion potential of soils by compacting the soil to 50 percent degree of saturation. The 50 percent degree of saturation is achieved by compacting damp soil in a specialized mold using roughly standard Proctor energy to achieve density. The compacted moisture content and density is then used to calculate the degree of saturation; however, if the degree of saturation is higher or lower than the recommended range, water is either added or subtracted to achieve moisture content along with the compacted density that falls within the desired window. The degree of saturation can vary by two percent.

It was shown in this research that it is possible to have different specimens compacted within two percent of each other and obtain varying results. For the Colorado soil, the initial compacted conditions of 110% OMC–90% MDD, 100% OMC–95% MDD, and 90% OMC–100% MDD corresponded to 68.5, 70.5, and 72 percent, respectively. If the desired degree of saturation was 70 and token stress was 1.5 kPa all three initial compacted conditions would be valid test specimens by ASTM D 4829 testing standards. For the Anthem soil, the desired degree of saturation was 72 and the token stress was 1.5. Shown in Figure 10.4 and 10.5 are the comparison of the swell potential and pressure results for the Colorado soil and Anthem soil respectively. All three points would have similar classifications. However, the swell pressure is quite different, for each initial compacted conditions.
Figure 10.4: Colorado soil EI example.

Figure 10.5: Anthem soil EI example.
Even though ASTM D4829 recommends a degree saturation of 50 percent, a practitioner should understand that the expansion potential classifications will be similar. The end result of the force exerted by the soil profile will be different. The differences in swell pressure, on the other hand, will change the recommendations of required overburden, cut-and-fills, and etcetera. Therefore, practitioners should abandon ASTM D4829 for index testing and use more appropriate means of assessing heave similarly to ASTM D 4546 since ASTM D 4546 remolds the sample at initial compaction conditions that are similar to field compaction conditions.

10.1.4 AZ Method for Swelling Assessment Recommendation

The current Arizona standard for assessing swell potential of natural or imported soils require a sample remolded at 95 percent of the standard Proctor maximum dry density and two percent less than optimum. This is due to the minimum allowable density and moisture content that occurs in the field for compacted soils. Most soils, in the field, will be compacted at a minimum of 95 percent of the standard Proctor density and around optimum moisture content (i.e. OMC ± 2 percent).

Nevertheless, it is possible to obtain a compacted section that has a density that corresponds 100 percent of the standard Proctor density and moisture content less than optimum and within the compaction window. For those conditions, it was shown from this research that they will generate the highest possible swell potential and pressure; therefore the Arizona Method should be re-evaluated by increasing the density from 95 to 100 percent of the standard Proctor maximum dry density.
10.1.5 Compaction Recommendations

The initial compacted conditions of 110%OMC – 90%MDD will be the best initial compacted conditions suited for expansive soils. If expansive soils are to be used in construction, the compaction specification shall not be greater than 90% of the maximum dry density obtained from the standard Proctor. It was shown that increasing the maximum dry density from 90 to 100 percent of the standard Proctor the swell pressure would increase by a minimum 2.6 times the swell pressure that would have been achieved at 90% MDD. The typical recommendations for expansive soils compaction is as follows: 95% MDD and plus 2% OMC, which interpolating between 110%OMC – 100%MMD and 110%OMC – 90%MDD would increase the swell pressure by a minimum of 1.56 times the swell pressure that would been achieved at 110%OMC – 90%MDD. Therefore, current compaction recommendations for compacted expansive soils need to be updated and addressed.

10.2 Recommendations for State of the Art

10.2.1 Creation of a Correlation of Swell Potential and Pressure with Cycles

Using more soils along with the two that were tested in the laboratory analysis create a correlation of swell potential and pressure changes with an increase in cycles. By creating a correlation with an increase of cycles, it will give practitioners the ability to adjust the swelling results that were obtained from laboratory testing to best represent field conditions.
10.2.2 Complete a Partial Drying Study on the Same Soil

Several studies have shown there is a difference between full wetting and drying and full wetting and partial drying. However, these studies only focused on one initial compacted condition and one net normal stress. This study, on the other hand, focused only on extreme environmental changes that created full wetting and full drying to assess the volume change behavior expansive soils. Nevertheless, the matric suction variation that occurs in the field is far less than that seen in the laboratory results that were obtained. Therefore, another study needs to be performed to assess the volume change potential of the same two expansive soils subjected to partial drying or less matric suction variation and compare the results.

10.2.3 Repeat the Analysis Using Intact Samples

This study needs to be completed on intact samples or undisturbed samples to assess the volume change behavior of natural expansive soils that have not been subjected to compactive effort. Undisturbed samples have been subjected to wetting and drying cycles over several centuries. When intact samples are collected from the field, they are normally subjected to one wetting in the laboratory. However, this is a snap shot of the behavior that can occur when it is wetted from that initial point. Therefore, multiple cycles are needed to assess if the behavior of intact samples will change with cycles or if one wetting and drying cycle will assess the volume change behavior.
10.2.4 Repeat the Analysis Using Recompacted Samples Subjected to Field Conditions

Intact recompacted expansive soils are subjected to field conditions or environmental changes (changes in matric suction loading). However, when these soils are designed they are only subjected to one wetting cycle. This research has shown that compacted expansive soils change with wetting/drying cycles. Therefore, intact recompacted expansive soils need to be collected to assess how the field condition change the swell potential and pressure. By collecting intact recompacted samples, it will allow for comparison of the results obtained from this research to determine how many cycles a compacted expansive soil has to be subjected to before an equilibrium condition is reached.

10.2.5 Expansive Soils Database

An expansive soils database is needed so that correlations can be made to current test methods results to the results obtained from soils that are subjected to multiple wetting and drying cycles. However, to infer useful correlations for practitioners, it requires a large sample population of naturally occurring recompacted soils. Currently there are two soils within the continental United States that have been subjected to multiple wetting and drying cycles and the results are presented herein. However, these two naturally occurring expansive soils represent soils from the same geographical region. Additional soils minimum of 2 from each geographical region should be collected and analyzed to add to the current database of soils. The properties of interest should include grain size distribution, Atterberg Limits, soil water characteristic curves,
mineralogy, specific surface area, concentrations of salts, sulfates, CEC, and SARs. These soils should then be tested at the same initial compaction conditions proposed in Chapter 5.

### 10.2.6 Obtain In-situ Samples and Compare to Compacted Samples

In addition to testing compacted samples, the collection and comparison of undisturbed samples will help with correlating how many cycles are needed on compacted specimens to replicate field behavior. This will also help with refining the estimation of swell potential with cycles for compacted expansive soils. However, in-situ samples, from the same site, should be collected throughout the year (i.e. before and after massive environmental changes). By collecting in-situ samples after large environmental changes, it will also help assess how the cyclic nature of in-situ changes.

If intact samples from the field cannot be collected, it is possible to create a small scale model that can be used to subject compacted expansive soils to environmental loading. Once the soil was equilibrated to the environmental condition of interest, the scale model then could be sampled and subjected to additional testing. The sampling intervals would be dependent on the equilibrated condition for assessment. After sampling, the small scale model then could be subjected to additional environmental changes prior to the next sampling interval.

### 10.2.7 Increase the Initial Compacted Conditions and Testing Scope

An increase in the testing initial conditions is needed. The increase should include different initial matric suctions as well as different initial densities. These different densities and matric suctions would be determined by using both modified Proctor energy
and non-standard Proctor energy. The non-standard Proctor energy would have an energy between the modified energy of 56,000 ft-lb/ft$^3$ and 12,400 ft-lb/ft$^3$ or an energy around 30,000 ft-lb/ft$^3$. To achieve this Proctor energy, it would require the modified Proctor hammer, four inch diameter mold, three lifts, and 22 blows.

After obtaining the two additional Proctor curves, test points around those optimum conditions would then be used for the ratio of MDD and OMC. The initial compacted conditions that should be tested include the five that were used in this study as well including 80 and 120 percent of the optimum moisture content as well as 85 and 105 percent of the maximum density. With the initial compacted conditions and the additional initial compacted conditions, it would create the testing conditions that are shown in Figure 10.6. Figure 10.6 shows the initial compaction conditions that should be included to make the best recommendations for compacting expansive soils in the field.

Even though compacting at a low density and a high moisture content to mitigate the swell potential and pressure, the strength of the material needs to be assessed. After the material is subjected to wetting and drying cycles, the strength of the material can be assessed by using direct shear measurements. The strength of the material needs to be assessed on first, third, and final cycle.
10.2.8 Determine if there is Validity to the Equilibrium Point

It was shown that there is a possible equilibrium point from the regression that was preform on the data. This equilibrium point needs to be determined if it exists since that point is at which the soil will recover back to its swelling strain from the previous cycle. If the expansive soil recovers back to a similar volume either in a swollen or collapse state, any structure loaded to this point should exhibit similar uplift due to swelling and similar consolidation due to collapse. During wetting events, the structure would heave uniformly and as the soil dries, the structure would uniformly consolidate. By knowing the shrinkage and swelling potential of expansive soils, it will allow designers to incorporate appropriate measures to mitigate potential swelling or collapse.

Figure 10.6: Increase in initial compaction condition testing points.
10.2.9 Development of a New Testing Device

A device should be created that is able to capture the radial strain during the wetting or drying cycle so that an accurate volume calculation is possible for a 1-D consolidometer sample or a restrained sample. By creating a new testing device it will allow for the expansion of other testing capabilities. During the wetting cycle the radial strain approaches zero quicker than most of the axial displacement, which makes it impossible to correlate the radial strain throughout the wetting cycle. In addition, having a device that is capable of measuring the radial strain during the test would avoid removing the sample from the consolidometer for measurements. When the samples are removed it is possible to lose material from the assembly and disassembly. The loss of material will cause changes in the dry density, which then can affect the swell potential during the next wetting cycle. Therefore, having a device that can measure radial movement but confine the sample once it reaches the original diameter is key for making more reliable decisions concerning how expansive soils expand and shrink in a 1-D consolidometer.
References


