Evaluation of Testing Methods for Suction-Volume Change of Natural Clay Soils

by

Austin H. Olaiz

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Graduate Supervisory Committee:

Sandra Houston, Chair
Claudia Zapata
Edward Kavazanjian

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ABSTRACT

Design and mitigation of infrastructure on expansive soils requires an understanding of unsaturated soil mechanics and consideration of two stress variables (net normal stress and matric suction). Although numerous breakthroughs have allowed geotechnical engineers to study expansive soil response to varying suction-based stress scenarios (i.e. partial wetting), such studies are not practical on typical projects due to the difficulties and duration needed for equilibration associated with the necessary laboratory testing. The current practice encompasses saturated "conventional" soil mechanics testing, with the implementation of numerous empirical correlations and approximations to obtain an estimate of true field response. However, it has been observed that full wetting rarely occurs in the field, leading to an over-conservatism within a given design when partial wetting conditions are ignored. Many researchers have sought to improve ways of estimation of soil heave/shrinkage through intense studies of the suction-based response of reconstituted clay soils. However, the natural behavior of an undisturbed clay soil sample tends to differ significantly from a remolded sample of the same material.

In this study, laboratory techniques for the determination of soil suction were evaluated, a methodology for determination of the in-situ matric suction of a soil specimen was explored, and the mechanical response to changes in matric suction of natural clay specimens were measured. Suction-controlled laboratory oedometer devices were used to impose partial wetting conditions, similar to those experienced in a natural setting. The undisturbed natural soils tested in the study were obtained from Denver, CO and San Antonio, TX.

Key differences between the soil water characteristic curves of the undisturbed specimen test compared to the conventional reconstituted specimen test are highlighted. The Perko et al. (2000) and the PTI (2008) methods for estimating the relationship between volume and changes in matric suction (i.e. suction compression index) were evaluated by comparison to the directly measured values. Lastly, the directly measured partial wetting swell strain was compared to the fully saturated, one-dimensional, oedometer test (ASTM D4546) and the Surrogate Path Method (Singhal, 2010) to evaluate the estimation of partial wetting heave.

DEDICATION

To my late grandfather, Major Willard R. Lewis, for teaching me at a young age that with hard work and discipline, anything is accomplishable.

To my mother, Tammy J. Lewis, and grandmother, Mary. J. Lewis for the constant support and motivation.

To my Father, Richard Olaiz, for showing me that no obstacle is too big to overcome.

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TABLE OF CONTENTS

CHAPTER Page

CHAPTER Page

LIST OF FIGURES

Figure Page

Figure Page

Figure Page

Figure Page

Figure Page

LIST OF TABLES

Table | Page

1 INTRODUCTION

1.1 The Issue of Expansive Soils

Expansive soils have historically been a leading cause of infrastructure damage in arid and semi-arid regions across the United States and around the world (Liu, 1997). Krohn and Slosson (1980) reported that more than $7 billion worth of infrastructure damage per year is caused by expansive soils in the United States. Recently, the estimation was increased to $11 to $15 billion per year by Wray and Meyer (2004), who studied public infrastructure alone. In spite of much research and improvements to design/building codes, residential and public infrastructure damage due to expansive soils continues.

The shrink/swell response that expansive soils exhibit due to moisture changes, caused by seasonal weather variations or alterations to site drainage due to new development, is the leading cause of expansive clay infrastructure damage (Zhan, Chen, & Ng, 2007; Houston S. , 2014). During a dry season for example, surficial clay layers can shrink in volume causing desiccation cracks. When the wet season arrives, moisture infiltration from rain causes the clay to swell, which can exert significant vertical pressures on any structure above and also result in differential foundation movements. The stress state variable of soil matric suction, which is related to the soil moisture through the soil-water characteristic curve, is used to quantify moisture variations for soil (Fredlund & Morgenstern, 1977).

1.2 Current State of Practice

Methods to predict the magnitude of heave and swelling pressure of expansive soils based on soil suction have been heavily researched in recent decades, in which several advancements have been accepted (Fredlund D. , 1979) (Wray W. , 1984) (McKeen, 1992) (Lytton, Aubeny, & Bulut, 2004) (Singhal, 2010). The majority of these prediction methods use index properties and fully saturated lab testing (i.e. zero suction) in order to obtain swell strain and swell pressure estimations due to difficulty, cost, and duration of testing associated with laboratory suction-controlled evaluation of clay soils. Laboratory suction-controlled studies of expansive clays have been predominantly performed on compacted (reconstituted) specimens due to challenges in establishment of trends resulting from natural soil variability. However, it has been observed that natural clay soils tend to exhibit significantly different responses than reconstituted samples.

1.3 Objectives

The overall objective of this research study was to evaluate and improve on several aspects of the suction-based testing of highly expansive soils; as well as to indicate any significant differences between the mechanical properties of remolded specimens and undisturbed natural specimens. More specifically, the objectives of the study were as follows:

- Improvements and recommendations for SWCC testing of highly plastic clay soils using the oedometer pressure-plate device. A detailed procedure is described for the determination of partial wetting response of the soil specimens, including the suction compression index. The procedure also includes a proposed method for determined the in-situ matric suction of such specimens.

- Comparisons of laboratory methods for suction determination to determine the most viable options for suction measurement for practicing engineers. The WP4-C, oedometer pressure-plate device, and the filter paper method were analyzed.
- Determine the natural "undisturbed" soil response to partial wetting, specifically SWCC shapes and suction compression indices, for comparison to values obtained from currently available estimation methods. Such data on undisturbed soil is very limited, therefore the contribution of these test results are themselves a significant addition to this field of research.
- Evaluate the Surrogate Path Method (Singhal, 2010) for estimation of the partial wetting strain on natural clay soils by comparing the values to directly measured partial wetting strains obtained in the laboratory.

2 GENERAL BACKGROUND REVIEW

2.1 Introduction to Topics Necessary for Understanding this Study

The mechanical response of a highly expansive clay soil is controlled by two separate stress state variables, net normal stress and matric suction. Each stress state variable affects the soil independently, in a highly non-linear fashion. It is important to understand why each stress state must be taken into consideration, and how to control/measure them in a laboratory setting. A review of past and recent literature on unsaturated soils and expansive soils was conducted. The literature review included the swell/shrink mechanism of expansive soils, the soil-water characteristic curve, stress path dependencies, and techniques for laboratory testing methods which allow for control of both unsaturated soil stress state variables.

2.2 Volume Change Properties of Expansive Soils

Volume change should always be accounted for when testing expansive soils. As in conventional saturated soil mechanics, volume-mass relationships are a crucial aspect of unsaturated soil mechanics. Similar to the coefficient of permeability and shear strength, the volume-mass relationship of an unsaturated soil is nonlinear and depends on the two stress state variables, net normal stress and matric suction. Using devices in the lab that allow for measurements of volume change during soil-water characteristic curve determination allow the geotechnical engineer to understand how a soil will change volume in responses to changes in the stress state variables.

The volume of the soil significantly changes as the soil suction varies for a highly plastic soil. The shape of the gravimetric water content vs. soil suction plot is very similar to that of the degree of saturation vs. soil suction curve for the sand soil. For this reason, it is not typically essential to monitor volume change of a nonplastic soil while conduction SWCC testing. This circumstance is opposite for plastic soils however. The gravimetric water content vs. soil suction plot can differ drastically from the degree of saturation vs. soil suction plot for highly plastic clay.

As such, the degree of saturation vs. soil suction must be used when analyzing an expansive soil due to the potential of volume change (Fredlund & Houston, 2013). It is also necessary to track specimen volume change due to the challenge of interpreting the SWCC for the air entry value and residual water content of a clay soil when using a water content or gravimetric water content vs. suction plot. The degree of saturation plot vs. soil suction allows air entry value and residual water content to be determined more directly from the plot.

2.3 Unsaturated Soil Concepts and Soil Suction

Unsaturated soil characteristics and response are dependent on the two stress state variables of net normal stress and matric suction. Net normal stress can be related to the widely understood total concepts for conditions of atmospheric air pressure, however, the understanding of matric suction and impacts of matric suction changes on soil response are not as straightforward. The laboratory measurement and control of matric suction have allowed for advancements in unsaturated soil analysis and design. However, due to relatively long equilibration times, it is generally not feasible for a practicing engineer to conduct unsaturated soil testing. This scenario has led to extensive research in the estimation of the matric suction of soil using common soil index properties (Saxton, Rawls, Romberger, & Papendick, 1986; Zapata, Houston, Houston, & Walsh, 2000; Fredlund, Wilson, & Fredlund, 2002; Fredlund, Wilson, & Fredlund, 2002; Singhal, 2010; Chin, Leong, & Rahardjo, 2010).

Unsaturated soil mechanics deals with soil in the region above the ground water table, as depicted in Figure 1 below. This zone forms a border between the ground water zone and the water in the atmosphere, therefore containing negative pore-water pressures. It is noticeable from Figure 1 that the amount of moisture in the unsaturated zone will be affected by various near-surface circumstances and associated boundary conditions. The amount of moisture in the unsaturated zone can vary because of the depth of ground water and major climatic variations such as rainfall, evaporation, and evapotranspiration.

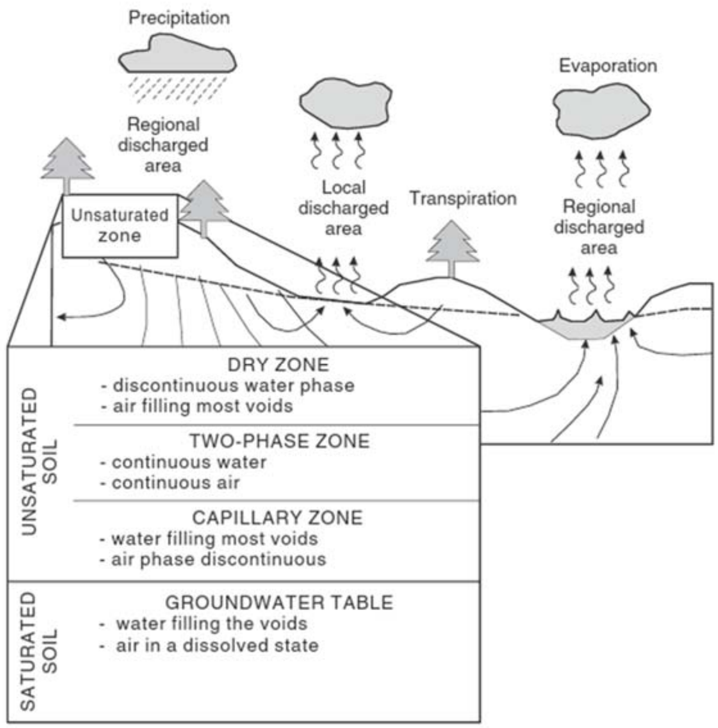


Figure 1: Definitions of Unsaturated Soil Zones (Fredlund, Rahardjo, & Fredlund, 2012)

As these boundary conditions change with time and seasons, so does the amount of moisture within the soil. The changing flux boundary conditions of the unsaturated soil zone often results in unsaturated soil mechanics problems being modeled as partial differential equations and analyzed using finite element models (FEMs). However, in practice, such as with expansive clays, simplifications to boundary conditions and unsaturated soil response are required for practical solutions.

Unsaturated soil mechanics combines two independent variables in order to define the state of stress of a soil element. A second stress state variable of soil suction is introduced and combined with the typical external stress state variable of overburden/structural stress (Fredlund & Morgenstern, 1977). The external stress in unsaturated soil mechanics is referred to as the net normal stress ($\sigma - u_a$), which is the difference between the overburden pressure and the air pressure. The air pressure is typically assumed to be atmospheric (i.e. u_a=0) when dealing with soils in the unsaturated zone (Figure 1). The unsaturated stress state variable that is affected by the amount of moisture with the soil element is matric suction (u_a-u_w). Matric suction is commonly described using the phenomenon of the capillary force (Figure 2). It is the difference between pore air pressure and the pore water pressure across the surface of the meniscus. The matric suction of the capillary tubes shown is inversely proportional to the radius of curvature of the meniscus. Therefore, soils with smaller pores such as silts and clays will result in higher matric suction values.

The matric suction, also referred to as capillary suction, explains how soil is permanently capable of retaining water above the ground water table. The matric suction counteracts the effect of gravity, which is attempting to pull the moisture down to the ground water level. Soil suction has the capability to cause moisture to rise within soil to heights up to nine meters of the free groundwater surface (Terzaghi, 1942). In practice, matric suction becomes the difference between total suction and osmotic suction, and includes soil surface adsorptive forces (Houston, 2017).

The moisture within soil, also referred to as pore-water, generally contains dissolved salts. These dissolved salts can also affect the magnitude of the total soil. The relationship is referred to the osmotic suction of the soil ($\psi_{osmotic}$). The osmotic suction is derived from the partial pressure of the water vapor in equilibrium with a solution identical in composition with the soil-water relative to the partial pressure of water vapor in equilibrium with pure water (Fredlund, Rahardjo, & Fredlund, 2012).

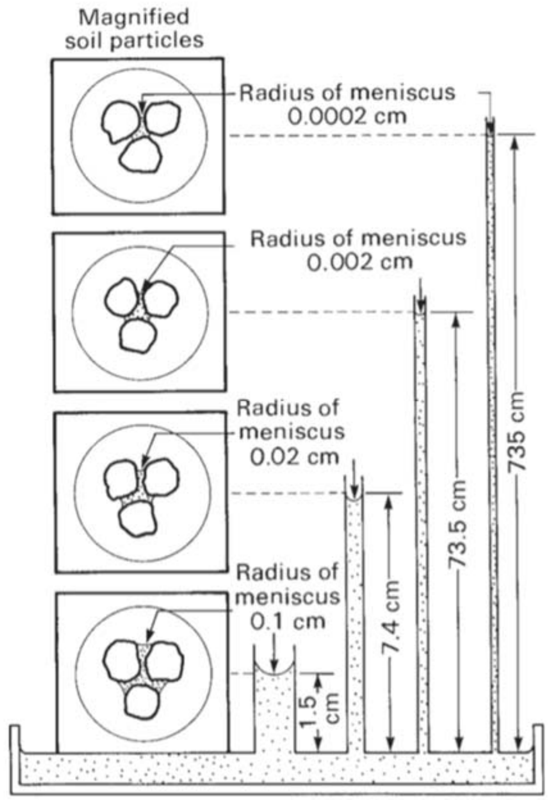


Figure 2: Capillary Tubes Showing Air Water Interface at Varying Heights for Radii of Curvature (Janssen & Dempsey, 1980)

Relative humidity is directly correlated to soil suction, in particular total suction. Total suction is the combination of matric and osmotic suction, therefore the value of the total suction of a saturated soil is theoretically equal to the osmotic suction.

$$\psi_{total} = (u_a - u_w) + \psi_{osmotic} \tag{1}$$

Osmotic suction has been shown to be essentially constant as soil moisture fluctuates (Figure 3) and therefore not as crucial to stress state and response of unsaturated soils (Krahn & Frendlund, 1972).

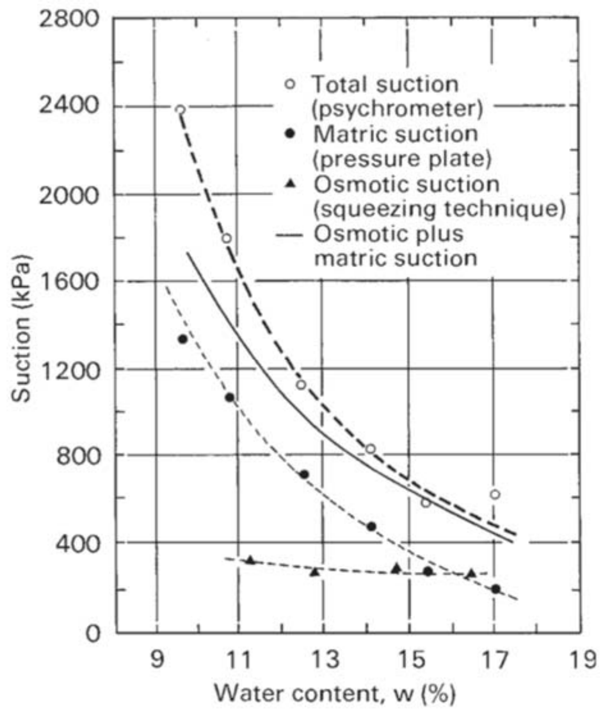


Figure 3: Total, Matric, and Osmotic Suction for Glacial Till (Krahn & Frendlund, 1972)

2.4 Soil Water Characteristic Curve

The soil water characteristic curve (SWCC) provides a relationship between the mass (and/or volume) of water in a soil and the energy state (soil suction) of the water phase (Fredlund, Rahardjo, & Fredlund, 2012). Several unsaturated soil index and engineering properties have become part of the data interpretation of laboratory SWCC tests (Fredlund, Xing, & Haung, 1994).

The soil water characteristic curve is often considered to be the key to analyzing all unsaturated soil mechanics. For example, the coefficient of permeability of an unsaturated soil changes in moisture content can be predicted using the SWCC (Fredlund, Xing, & Haung, 1994). A few other applications of the soil water characteristic curve include a method to predict shear strength (Vanapalli, Fredlund, Pufahl, & Clifton, 1996) and a method to predict the modulus of elasticity of the soil (Oh, Vanapalli, & Puppala, 2009).

The soil water characteristic curve is necessary for the previously described methods of predicting engineering properties the soil due to the changing stress state of the soil as the moisture content changes. The soil water characteristic curve defines the relationship of the change in moisture content to the change in soil suction. Figure 4 below provides of an example of a typical SWCC for a silty soil. There are many different aspects of the SWCC that must be understood. Figure 4 depicts a graph of volumetric water content vs. the log of soil suction in kPa. The SWCC plot generally follows a sigmoidal shape and is most appropriately terminated at maximum theoretical value of soil suction 10^6 kPa. The maximum soil suction value is based on Gibbs free-energy state equation for water vapor (Edlefsen & Anderson, 1943). Volumetric water content is one of the three ways of describing the amount of water in the soil. The volumetric water content is simply a dimensionless version of the water content equal to the ratio of the volume of water to total volume of the soil. The amount of moisture in the soil can also be represented as the gravimetric water content (the water content traditionally used in geotechnical engineering) or as the degree of saturation.

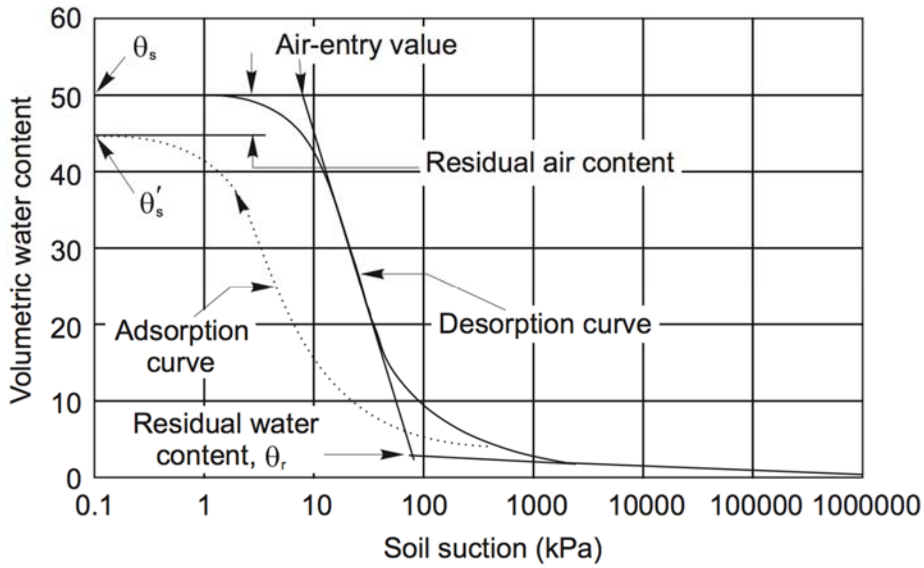


Figure 4: General Soil Water Characteristic Curve for a Silty Soil (Fredlund, Rahardjo, & Fredlund, 2012)

Two key parameters of the SWCC are the air entry value and the residual water content which lie on the desorption curve. The air entry value of the soil corresponds to the difference in air and water pressure (matric suction) at which the largest diameter capillary is drained (Childs, 1940). It is the matric suction value at which air begins to displace the water in the largest pores within the soil. The residual water content is the water content value at which a tremendous suction change is needed to remove additional water from the soil. (Fredlund, Rahardjo, & Fredlund, 2012). The existence of the residual water content value explains why even topsoil from a dry desert climate will always result in a water content greater than zero in a laboratory moisture content test. The air entry value and residual water content are the two main components to the majority of the previously stated empirical methods for unsaturated soil property determination based on the soil water characteristic curve. Because of this, typical lab testing for the SWCC only provides a drying curve.

Similar to the desorption curve, the adsorption curve, commonly referred to as the wetting curve, also contains two important points, the water entry and residual air values. The water entry value is the suction value at which water begins to enter the pores of the soil when moving from very high suction values to lower suction values. It can be defined on the SWCC as the point at which the first break occurs when moving from right to left on the wetting curve (not depicted on Figure 4). The second break when moving from right to left on the adsorption curve defines the residual air value. The adsorption curve is typically more difficult to measure in the lab.

The soil water characteristic curve can be divided into three main sections. These sections are directly correlated to the three sections of unsaturated soil zone illustrated in Figure 1. The first section of the SWCC begins at zero suction and moves along the curve until the air entry value is reached. This section is defined as the boundary effect zone. When the soil is within this suction range is still acts as a saturated soil, similar to a soil within the capillary zone, and conventional saturated soil mechanics may be used for analysis. The second section of the SWCC starts at the air entry value moves along the curve until the residual water content is reached. This section is defined as the transition stage, or two-phase zone. Small fluctuations in moisture content within this stage can have significant effects on the stress state of the soil due to the change in matric suction, therefore requiring nonlinear formulas during analysis. The third section of the SWCC is the residual zone, or commonly referred to as the dry zone. Soils within this zone required drastic changes in soil suction to produce changes in moisture.

The general sigmoidal shape of the soil water characteristic curve (Figure 4) varies with soil type. Soils with higher fines content generally will result in higher suction values at a given water content (Walsh, Houston, & Houston, 1993), which cause a shift in the SWCC (Figure 5). The fines within the soil will result in smaller pore sizes and therefore higher matric suction values due to the capillary effect as previously described.

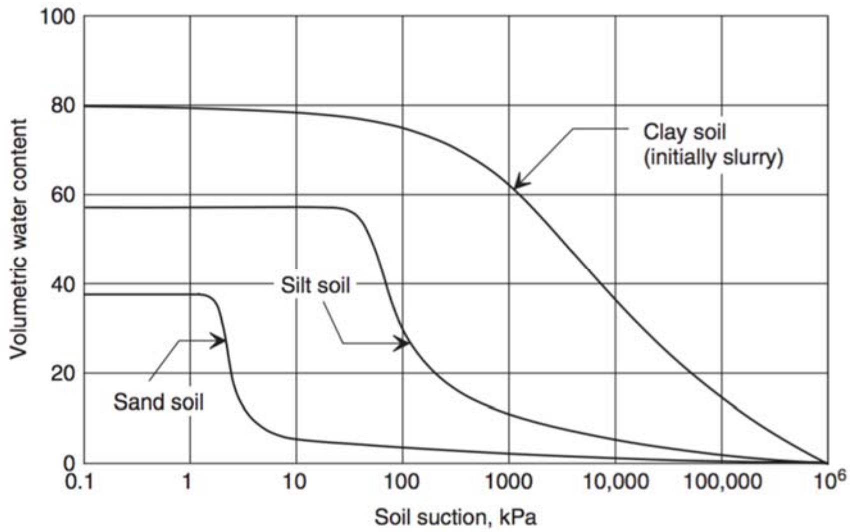


Figure 5: Comparative Desorption SWCCs for Sand, Silt, and Clay Soils (Fredlund, Rahardjo, & Fredlund, 2012)

The effect of plasticity and fines within the soil is also depicted on Figure 6 below. Figure 6 represents typical soil water characteristic curves for soil based on the product of percent fines (the percent passing the #200 sieve) and the PI of the soil (Zapata, Houston, Houston, & Walsh, 2000). This plot may be used for a preliminary unsaturated soil mechanics analysis, as it only requires typical soil laboratory index testing to obtain an approximation of the soil water characteristic curve. Correlations like those depicted in Figure 6 allow the geotechnical engineer to crucial time as developing SWCCs from direct testing can be relatively long and tedious, especially when dealing with highly expansive plastic soils. If the in-situ matric suction of the soil is obtainable, these plots can be used to determine, very approximately, the relative wetness or dryness of the soil.

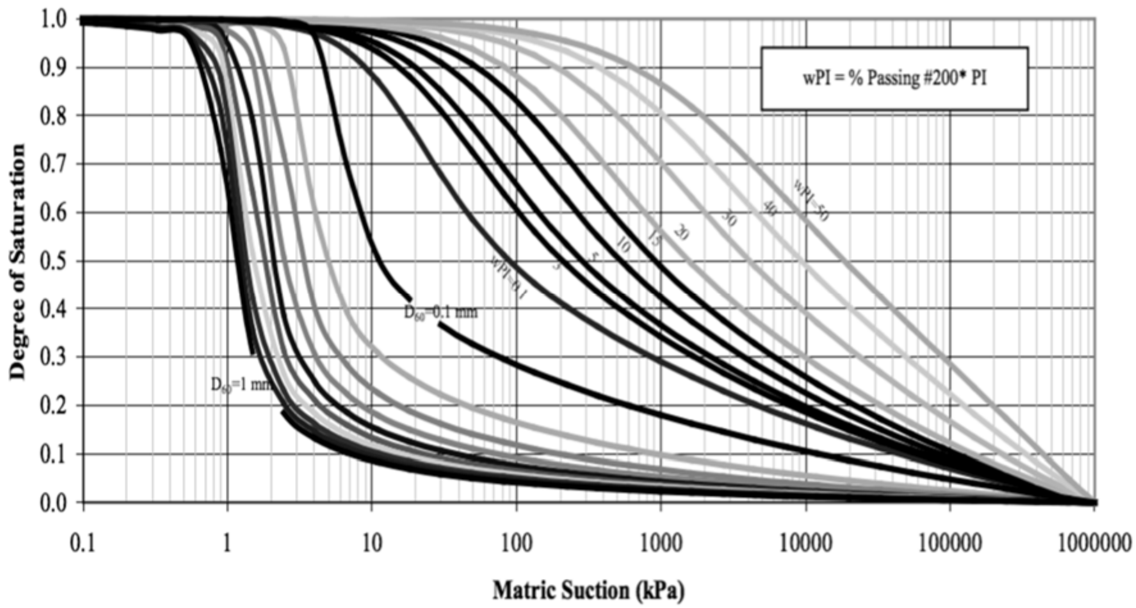


Figure 6: Combined Family of SWCCs for Plastic and Nonplastic Soils (Zapata, Houston, Houston, & Walsh, 2000)

2.4.1. Hysteresis

One of the most important phenomena of unsaturated soil mechanics is the concept of hysteresis as depicted in Figure 4 and Figure 7. A soil will not behave the same as it changes from higher moisture contents to lower moisture contents (desorption path) as it does with a corresponding change from lower moisture contents to higher moisture contents (adsorption path). In other words, a soil will require a higher magnitude of suction to return to its original state if its initially dried to a certain moisture content compared to being initially wetted to this same moisture content. This phenomenon can be explained by the hysteretic effects of the permeability of the soil, which is dependent on the suction stress path.

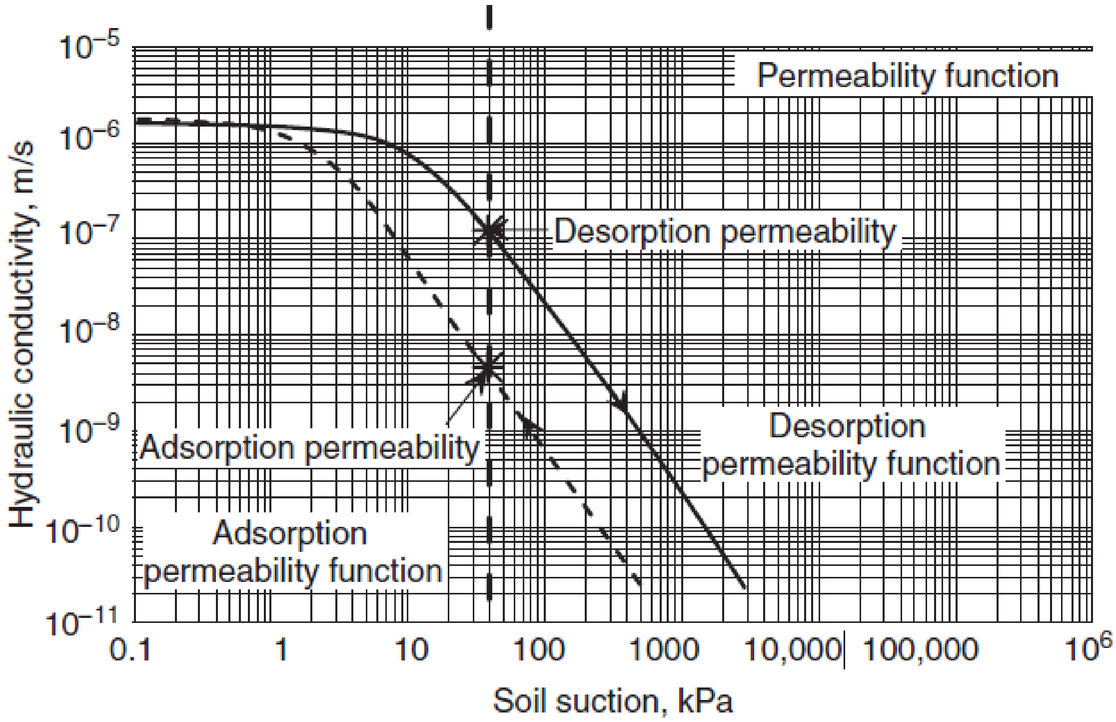


Figure 7: Hysteresis Effects on the Unsaturated Hydraulic Conductivity of a Soil (Fredlund, Rahardjo, & Fredlund, 2012)

Figure 7 illustrates that a soil on the desorption path will exhibit a higher coefficient of permeability than the same soil on the absorption path for a given soil. In other words, water will flow easier through a soil that is experiencing a drying cycle (i.e. increasing matric suction) compared to the same soil that is undergoing a wetting cycle (i.e. decreasing matric suction)

As the soil continues to undergo wetting and drying cycles the hysteresis effects become less significant, although wetting/drying hysteresis is always present. The wetting and drying boundary curves for a given soil represent their initial wetting or drying path from a fully saturated or completely dry state, respectively. Subsequent wetting and drying paths fall between the two boundary curves on what are referred to as scanning curves (Figure 8).

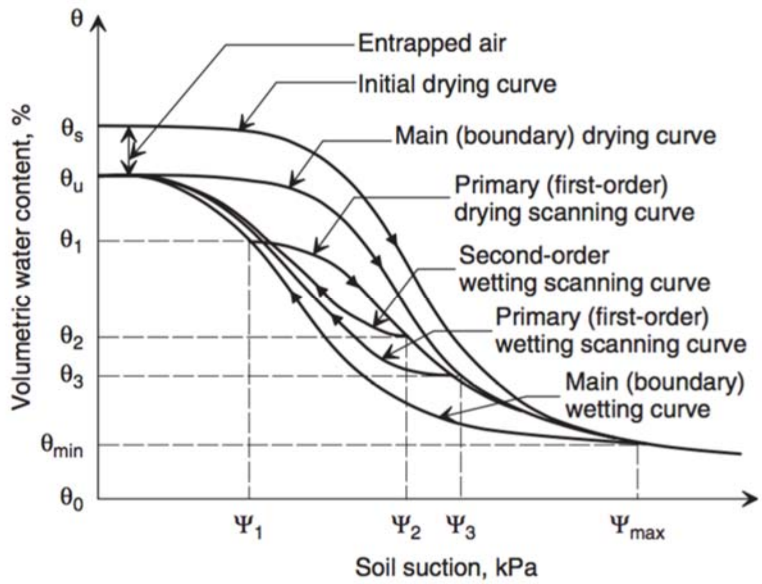
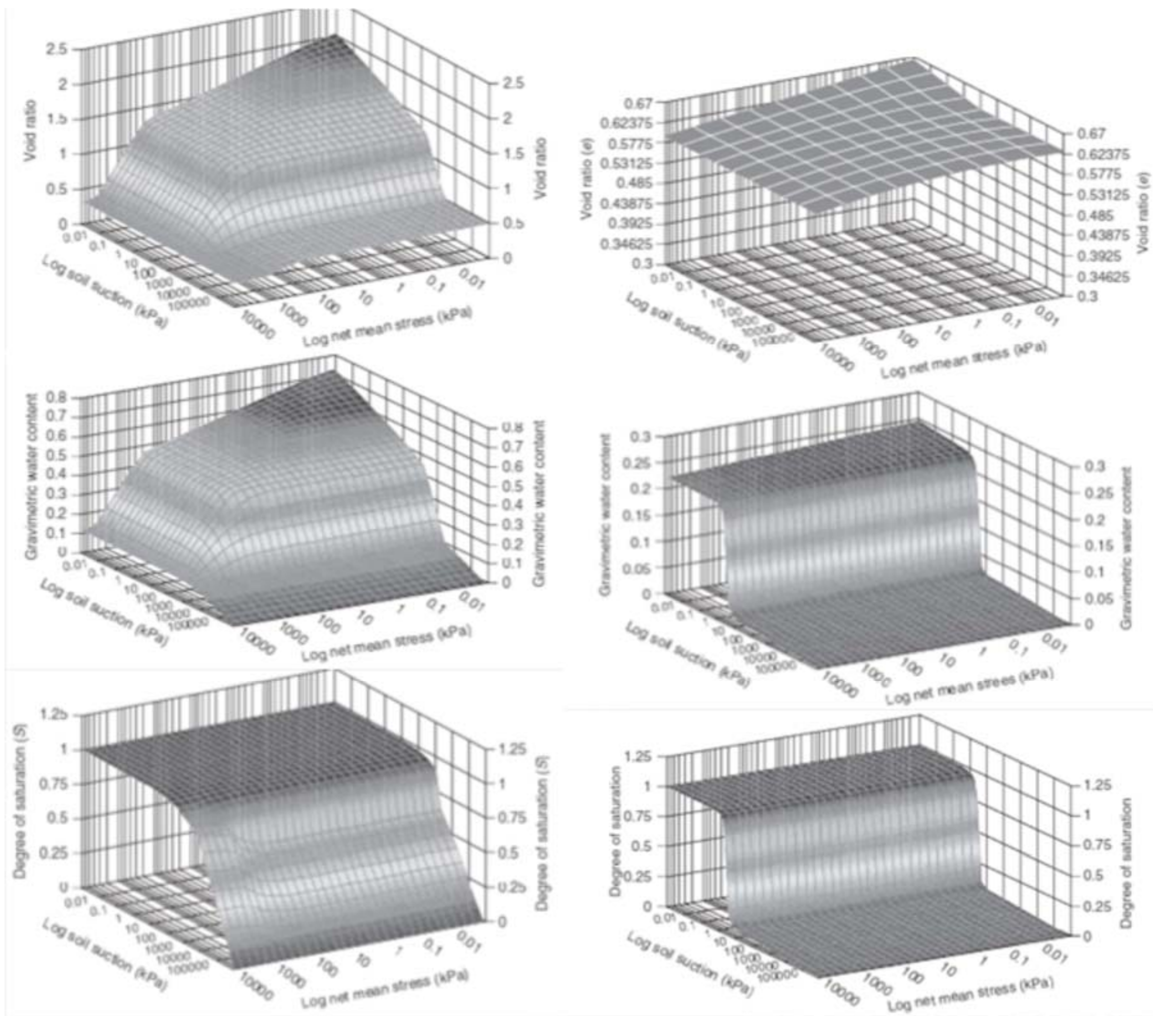


Figure 8: SWCC of Multiple Wetting and Drying Cycles (Fredlund, Rahardjo, & Fredlund, 2012)

Rosenbalm (2013) observed that 6-8 typical field wetting/drying cycles are necessary for a reconstituted soil to reach an equilibrium state, similar to that of a natural soil. Natural soils have experienced numerous amounts of wetting and drying cycles since the beginning of time, and therefore it should be expected that the SWCC shape of a natural soil will fall within the two boundary curves, on a flatter-sloped scanning curve, as depicted in Figure 8.

2.4.2. The Effects of Net Normal Stress and Matric Suction on Volume Change

Volume change should always be accounted for when testing expansive soils. Just as in conventional saturated soil mechanics volume-mass relationships are a crucial aspect of unsaturated soil mechanics. Similar to the coefficient of permeability and shear strength, the volume-mass relationship of an unsaturated soil is nonlinear and depends on the two stress state variables, net normal stress and matric suction. Using devices in the laboratory that allow for measurements of volume change as the soil water characteristic curve is being determined, allows the geotechnical engineer to understand how a soil will change volume in response to changes in the two stress state variables. Three-dimensional plots, like Figure 9a and Figure 9b below, allow for one to visualize how the volume-mass relationship of a soil changes with the soil suction and net normal stress.



(a) (b)

Figure 9: Volume-Mass Surfaces for (a) Highly Plastic Clay (b) Sand Soil (Pham & Frendlund, 2005)

Figure 8a illustrates how the volume of the soil significantly changes as the soil suction varies for a highly plastic soil. The volume change is minimal for the sand soil depicted in Figure 9 (b). The shape of the gravimetric water content vs. soil suction plot is very similar to that of the degree of saturation vs. soil suction curve for the sand soil. For this reason, it is not typically essential to monitor volume change of a nonplastic soil while conducting SWCC testing. This circumstance is opposite for plastic soils, however. The gravimetric water content vs. soil suction plot differs drastically from the degree of saturation vs. soil suction plot for the highly plastic clay, and therefore volume change must be monitored when testing clay soils.

Fredlund, & Houston (2013) studied the effects of modeling high volume-change soils with respect to different moisture variables (i.e. gravimetric moisture content, volumetric moisture content, and saturation). They observed that the air entry value determined from a gravimetric moisture content SWCC will tremendously increase as the volume of the soil changes with respect to changes in suction (Figure 10). This phenomenon indicates the importance of modeling the SWCC of a volume-change soil with respect to the degree of saturation.

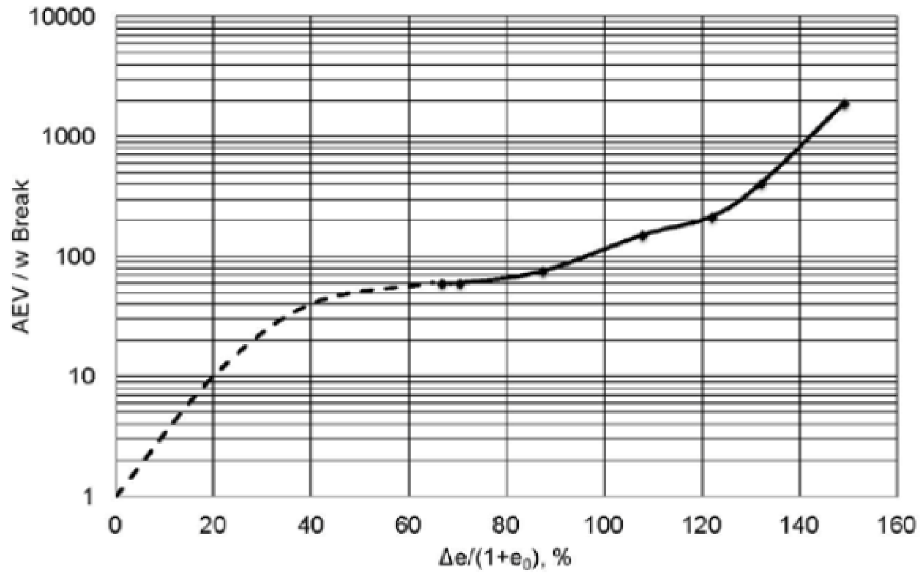


Figure 10: Difference Between the Break in the Gravimetric Water Content SWCC and the Air-Entry Value for Regina Clay (Fredlund & Houston, 2013)

2.5 Suction Compression Index

Soil's volumetric response to changes in matric suction at a given net normal stress is referred to as the suction compression index. The suction compression index is determined in a similar way as a saturated soil's response to changes in effective stress (Nelson & Miller, Expansive Soils: Problems and Practice in Foundation and Pavement Engineering, 1992). The suction compression index represents the change in void ratio for a given change in matric suction under a constant net normal stress condition (Figure 11).

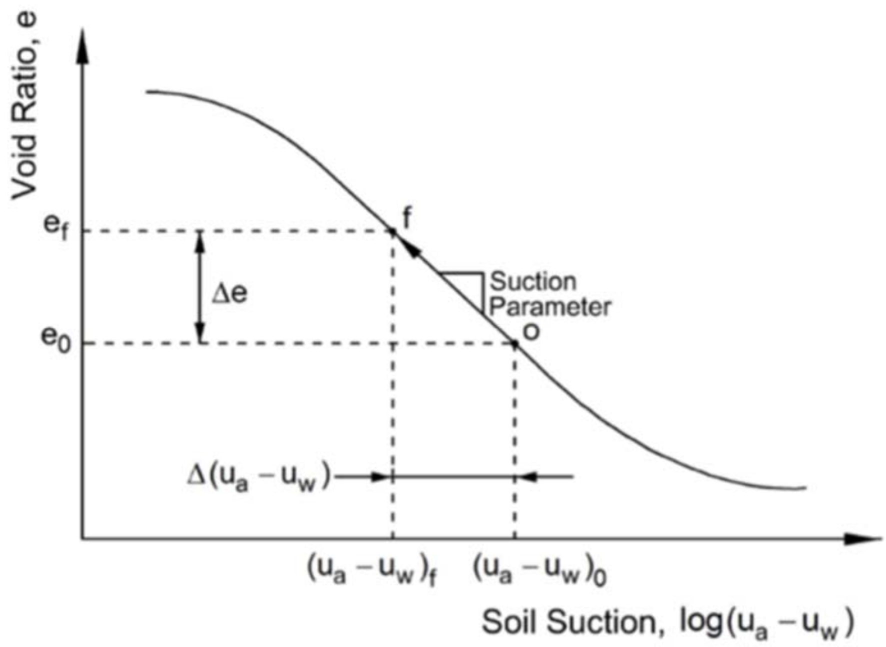


Figure 11: Void Ratio vs. Log Matric Suction (Tu, 2015)

In the general case, unsaturated soil mechanical response is controlled by two stress state variables. However, for expansive clay problems, it is common that changes in void ratio occur in response to changes in matric suction under constant net normal stress conditions. Hence, the suction compression index represents the change in void ratio (or volumetric strain) due to a log cycle change in soil matric suction. Use of the suction compression index to predict soil heave is associated with the design assumption that the net normal stress on the soil will not significantly change post-development on an infrastructure project. Therefore, the key variable in most suction-based ground heave predictions is the suction compression index.

A laboratory testing apparatus, which is able to control/measure net normal stress, matric suction, and vertical deformation is necessary to directly measure values of the suction compression index. However, time necessary for specimen equilibration upon suction change makes such testing unfeasible for most practicing geotechnical engineers. Several methods for estimating the suction compression index have been developed in the last few decades, which encompass index property correlations such as Atterberg Limits, gradation, and mineralogy (McKeen, 1992; Lytton R. L., 1994; Perko, Thompson, & Nelson, 2000; Lytton, Aubeny, & Bulut, 2004) (Lytton R. L., 1994) (Perko, Thompson, & Nelson, 2000) (Covar & Lytton, 2001). The correlations were most commonly developed using results of COLE and CLOD testing. Covar and Lytton (2001) used the mineralogy classification of clay work by Holtz and Kovacs (1981) along with a database of approximately 130,000 surficial soil samples from the Soil Survey Laboratory (SSL) of the National Soil Survey Center to develop their estimation method of the suction compression index. The Covar and Lytton (2001) method was implemented in the Post-Tensioning Institute's Design of Post-Tensioned Slabs-on-Ground 3rd Edition (2004; revised in 2008). The Perko, Thompson, Nelson (2000) and the PTI 3rd Edition (2008) method for estimating the suction compression index will be evaluated further in this study.

3 MATERIAL TESTED IN STUDY

3.1 Soil Samples used in the Study

The soil samples used in this study were from one site in Denver, Colorado and one site in San Antonio, Texas. Both bulk and undisturbed samples were gathered to a depth of 30 feet below existing grade. The following summarizes the sample retrieval efforts:

- Four 30-foot deep boreholes were drilled in close proximity of each other. Two holes were drilled within a pavement-covered area, while the other two holes were drilled in open areas.

- A CME-55 truck mounted drill rig with a 4.5-inch continuous flight auger was used for the borings.
- A Modified California Split Ring Sampler (Drilling World model 67007-01) consisting of 18.0” long split barrel sampler with 2.5” outer diameter, and blunt nosed shoe was used for the undisturbed ring sample retrieval. The Modified California Spilt Ring Sampler has an area ratio of 56% (discussed further in the following section).
- Each ring used to recover and confine the retrieved intact samples was 1.0” in height with a 2.45” inner diameter.

An additional 101 undisturbed soil samples were provided by CTL | Thompson, Inc. out of Denver, CO. These additional soil samples were also retrieved using a sampler with an area ratio of 56%. These CTL Denver samples were used in the evaluation of the WP4-C testing device. All samples were transported to the Arizona State University geotechnical laboratory by CTL Thompson in sealed and moisture controlled containers. The containers were packaged together in such a way that vibrational movements were kept to a minimum.

3.1.1. Sampling Disturbance of Unsaturated Soils

Natural soils in-situ possess a number of characteristics that influence their responses to field loading/unloading and their capacity to transmit water. These characteristics include density, degree of cementation, anisotropy, soil structure, available specific surface area, and stress state. The process of soil sampling infers that the samples will be used for some type of testing, typically in a laboratory. For certain types of testing, stresses that were removed by sampling are reapplied to the test specimen at the beginning of the test. For other tests, stresses different than the original in-situ stresses may be applied initially. For some laboratory testing (e.g. compaction testing) the sample is completely remolded prior to testing, but for other types of testing it is the intent that the sample be as undisturbed as possible. The underlying objective of sampling and testing is to obtain test results and properties that are reasonably applicable to the in-situ mass of soil from which the samples came. Therefore, the effects of sample extraction from a borehole or test pit, transport to the laboratory, and test specimen preparation (such as trimming) are all potentially very important relative to their impacts on the in-situ characteristics listed above. The objective of employing good sampling, transportation and specimen preparation procedures is to obtain final test results that are sufficiently applicable to the field, that appropriate engineering decisions and choices can be made.

In general, sample extraction from a borehole is accomplished with sampling tubes. Hvorslev (1949) is widely credited with pioneering work on sample disturbance and the requirements for obtaining the best undisturbed sample possible. Of course, no sample is completely undisturbed. However, it is possible to get relatively undisturbed samples with thin-walled tubes. These samples are called undisturbed because they are undisturbed relative to auger cuttings and samples extracted by scooping from a test pit with a backhoe, for example, and because the best sampling methods have been used in an effort to preserve in-situ properties (e.g. deformation and shear strength). Hvorslev concluded that an area ratio of the order of 10% is needed to get good undisturbed samples of soft saturated clay. The area ratio, A_r, of the sampling tube is defined as the ratio of the cross-sectional area of the tube wall to the cross-sectional area of the sample inside. Gilbert (1992) suggests that an A_r of 10 to 15% is adequate for obtaining good quality undisturbed samples. Hvorslev also suggests that a sample tube clearance of 0.75 to 1.5% is needed for long samples and a clearance of 0 to 0.5% is adequate for short samples. Gilbert (1992) and Marcuson and Franklin (1979) also state that a fixed piston is an important feature for successful sampling and recovery of soft saturated clays. Hvorslev (1949) also states that the recovery ratio (length of sample obtained to length of tube advancement) should be no more than 1 and no less than about 0.97.

Houston (2014) has discussed at some length disturbance associated with tube sampling of unsaturated soils, as given by the paragraphs that follow.

> A tube sampler with a relatively sturdy shoe is often required to sample highly desiccated, cemented, and often sandy or gravelly unsaturated soils, particularly those encountered in arid climates. A sample tube with the smallest area ratio possible (e.g. Shelby tubes) should be attempted for unsaturated soil sampling to minimize sample disturbance. However, successful sampling of unsaturated soils frequently requires samplers having an area ratio as large as 56%.
>
> For unsaturated soils, although breaking of bonds and densification certainly occur under the cutting edge of the sampler, the disturbance does not propagate laterally as much for typically contractive unsaturated soil as it does for dilative and incompressible soils. Dilative and incompressible soils, such as saturated clays or very dense cohesionless sands, fulfill the conditions of general shear, in which shear straining and bond breaking extend more widely to the sides of the cutting edge of the sample tube. By contrast, unsaturated soils of moderately

low to low density tend to exhibit local shear failure under the cutting edge of the drive tube, as depicted in Figure 12.

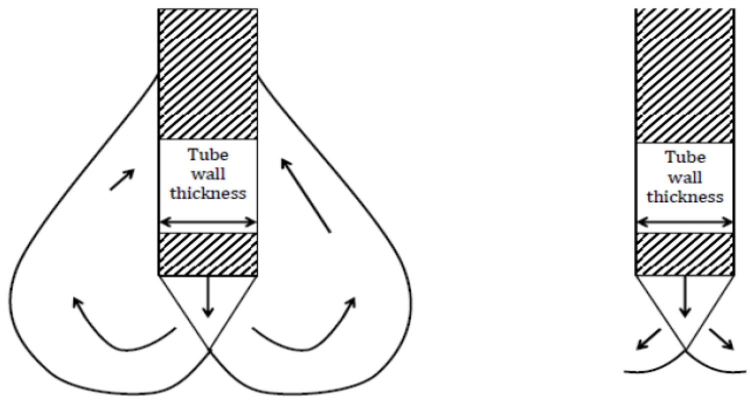


Figure 12: General Shear (a), Local/Punching Shear (b)

The effect of sampling tube area ratio would be expected to decrease with a soil's increased tendency toward compression under load. The local shear behavior of many unsaturated soils limits the extent of remolded/highly disturbed soil entering the tube such that the structure of the soil at the center of the specimen would be expected to be only minimally modified.

A collapsible soil, for example, is by definition in a contractive state. In fact, a highly collapsible soil serves as the classic example of local shear, in which normal and shear strains tend to be localized under the cutting edge of the sampler. While a collapsible soil is an extreme example of a contractive soil that would be expected to exhibit local shear bearing capacity failure, unsaturated soils, in general, have apparent cohesion due to a combination of matric suction and plasticity, and tend to be contractive under applied net normal stress load and therefore tend to exhibit local shear failure. Studies by the author and her colleagues on sampling disturbance of unsaturated soils support that relatively good quality undisturbed samples of unsaturated soils can be obtained using sampling tubes with area ratios up to 56%. The volume change in response to wetting of tube-sampled unsaturated soils appears to be less affected by sample disturbance than shear strength, but sample disturbance effects on shear strength tend to lead to conservative results.

Houston and El-Ehwany (1991) found in a study of collapsible soils that the contribution of the suction component can become quite significant, especially in soils that have silt or clay particles. Because of their cemented and contractive nature, collapsible soils are not as susceptible to disturbances caused by the use of large-area-ratio samplers or vibrations resulting from hammering. In particular, the type of cementation resulting from soil suction is not easily destroyed. Although sample disturbances are likely to cause decreases in the shear strength of the soils, when the remaining cementation is adequate to maintain the initial dry unit weight, settlement responses are less likely to be significantly affected. This is supported by the results of response-to-wetting tests on undisturbed specimens and completely disturbed and recompacted specimens, at comparable initial dry unit weights, for which total collapse strains (dry plus wetted) were not significantly different (Houston and El-Ehwany, 1991).

Houston and El-Ehwany also reported that the effect of sampler tube area ratio on 1-D response to wetting collapse test results was statistically insignificant up to 56% for collapsible soils with low gravel content. For very lightly cemented soils and high-gravel- content soils a moderately small, but statistically significant, difference in collapse response was observed. To account for sampling disturbance, Houston and Houston (1997) recommended that in the interpretation of 1- D oedometer response to wetting tests on collapsible soils, the field collapse strain for full wetting conditions be taken as the strain from the origin to the wetted curve (corresponding to an essentially flat dry loading curve in Figure 13). This interpretation is supported by the fact that cemented collapsible soils, in their dry in-situ state, exhibit only negligible strain in response to typical structural loading (Houston et al. (1988); Peck et al. (1974). The effect of sampling disturbance on collapsible soils is to break bonds which otherwise would be weakened and therefore lost during wetting, the net result of which is higher dry loading strains as a result of sample disturbance. The overall dry plus wetted strain, however, has been shown to be quite stable (Houston & El-Ehwany, 1991; Feda, 1988; Jasmar & Ore, 1987; Basma & Tuncer, 1992; Munoz-CastelBlanco, Delage, Pereira, & Cui, 2011; Delage, Cui, & Antoine, 2005)

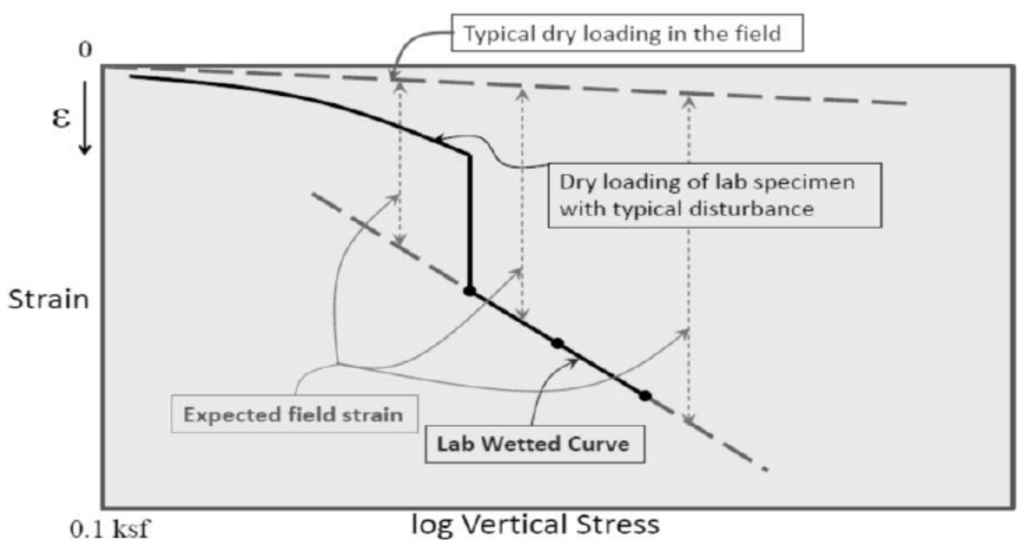


Figure 13:1-D Response to Wetting on Collapsible Soils – Effect of Disturbance

Studies on compacted fills by McCook and Shanklin (2000) showed essentially no difference between drive tube sample dry density and sand cone dry density for compacted fill soils with no reported gravel. Noorany et al. (2000) and Houston et al. (2002) showed that dry density is slightly lower for tube sampling compared to sand cone density determination when gravel content of the soil is 10% to 20%. Houston, et al (2002) attributed the difference in dry density of gravelly soils primarily to the fact that the gravel content of tube samples is, on average, less than the gravel content of field sand cone samples. Houston et al. (2002) compared gravel content between sand cone and drive tube samples, showing average sand cone gravel content of 21.9% compared to a tube-sampled gravel content average of 18.6%. In addition, it is likely that a certain number of rock fragments are hit by the edge of the tube, causing these fragments to rotate and loosen the soil to some extent as it enters the tube. Nonetheless, the tube samples of unsaturated soil containing 10 to 20% gravel have been found to have dry densities within 2 to 8% of companion sand cone specimens (Noorany, et al, 2000; Houston et al, 2002). It is unlikely that the void ratio of the soil matrix and the general structure of the unsaturated soil are significantly altered by tube sampling, even when gravel up to 20% is present in the soil.

In a study on unsaturated expansive clays, Singhal (2010) reports that intentional sample disturbance by remolding leads to some reduction in soil suction, and associated reduction in swell pressure and percent swell. However, partial remolding resulted in much less suction change than thorough remolding, and matric suction did not change appreciably upon remolding for the soils having PI values higher than approximately 45. Figure 14 shows that for higher PI and lower void ratio soils, there is less reduction of matric suction upon remolding. Higher PI soils seem to require higher disturbance effort (breaking up of particles) to fully remold the sample. This, together with the local shear failure issues discussed above, makes it unlikely that tube sampling would have a significant impact on suction of unsaturated clay soils. Further, when unsaturated clay soils are recompressed in the laboratory back to their field net normal stress conditions, effects of sampling disturbance are believed to be largely ameliorated. In support of this position, Singhal et al. (2011) reports that swell pressures of tube-sampled expansive soils, first loaded to field stress level, were found to be, on average, the same as swell pressures obtained on companion specimens where sampling disturbance correction methods proposed by Nelson and Miller (1992) and Fredlund et al. (1980) were applied. Singhal et al. (2011) express the opinion that most of what is perceived as sampling disturbance effects is embodied in the release of stored energy when a sample is removed from the field, and that reapplication of overburden stress before wetting in an oedometer swell test restores most or all of the stored energy lost by sampling.

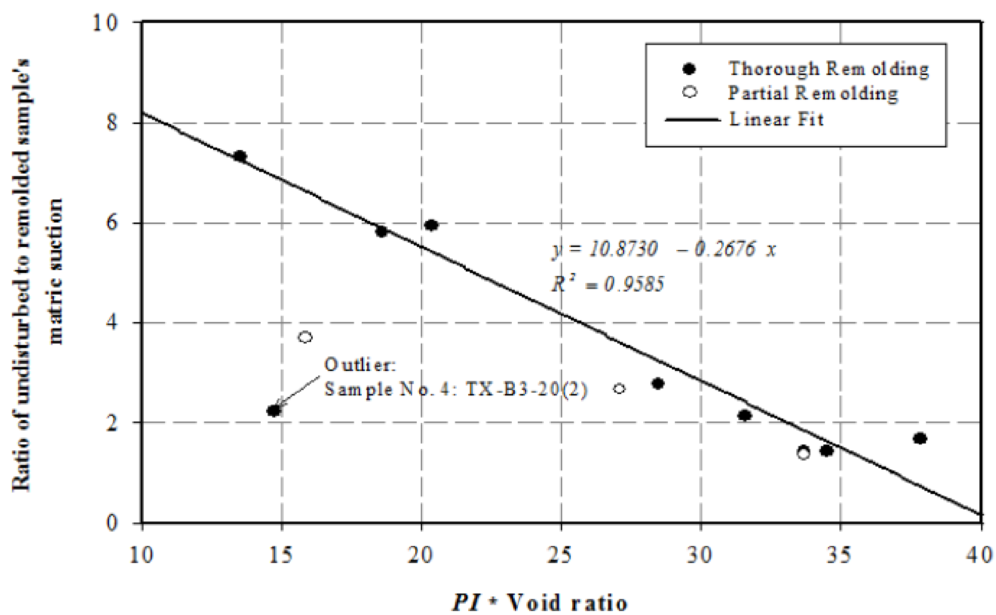


Figure 14: Effect of Remolding on Matric Suction (Singhal, 2010)

In a study by Douthitt et al. (1998), direct shear test cohesion intercept, c, and angle of friction, ϕ, parameters on block samples of unsaturated soils were compared to direct shear parameters for companion thoroughly remolded/disturbed specimens prepared to the same dry density and water content of the block specimens. The results of these direct shear tests are shown in Table 1. Complete and intentional disturbance, well beyond what would be expected during tube sampling, resulted in a reduction of the cohesion intercept, but had little impact on the friction angle of the specimens. Houston et al. (1997) also reported the relative insensitivity of ϕ values to sampling disturbance, as shown in Figure 15 (a) comparing ϕ values for undisturbed specimens to ϕ values for specimens remolded to in-situ dry density and water content. Figure 15 (b), also from Houston et al. (1997), shows that the cohesion intercept, c, is reduced by remolding, but not nearly to the extent of specimen submergence-induced reduction in c. The impact of thorough remolding on shear strength is small compared to the impact of submergence, as shown in Figure 15. The impact of rather thorough remolding of unsaturated soils results in essentially no change in friction angle, ϕ, but some reduction of cohesion intercept (c). Because tube-sampling of unsaturated soils results in considerably less disturbance compared to the intentionally remolding of specimens, shear strength parameters obtained on tube-sampled unsaturated soils would be expected to typically be slightly conservative compared to those of block-sampled specimens or in-situ undisturbed soils. Tube sampling of unsaturated soil shear strength test specimens is considered appropriate for engineering design purposes.

Table 1: Direct Shear Tests on Unsaturated SM Soils (Douthitt, Houston, Walsh, & Houston, 1998)

Sample Description	Site A c (kPa)/ϕ	Site B c (kPa)/ϕ	Site B c (kPa)/ϕ
Dry Block	34/14.5	62/20.5	120/23.4
Remolded at In-Situ Dry Density and wc	14/15.0	27/20.5	45/21.0
Block, Submerged	8/14.0	0/19.0	5/22.0

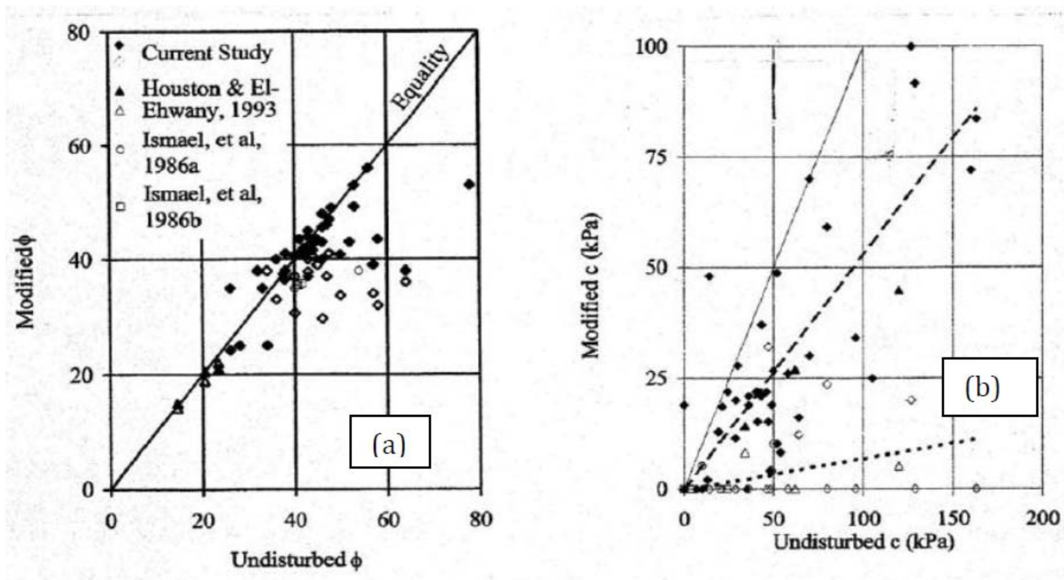


Figure 15: Disturbance and Submergence Effects on (a) Internal Friction and (b) Cohesion (Houston, Walsh, & Houston, Soils Strength Contribution of Soil Suction in Cemented Soil, 1997)

In summary, although studies by the author and others have shown, as expected, some sample disturbance when tube samples are used to collect specimens, overall, research findings support the use of tube sampling in unsaturated soils for geotechnical investigation and estimation of response to wetting and shear strength properties. Given the great inconvenience and/or impracticality of obtaining block specimens, it is recommended for sampling of unsaturated soils that tube samples may be driven, if convenient, and that the wall thickness of the tube be as small as practical, but large enough to successfully sample the soils at the site. It is expected that sampling disturbance of unsaturated soils will, in general, lead to conservative estimates of strength parameters due to an underestimate of cohesion intercept resulting from some bond breaking and some minor reduction in soil suction. Sampling disturbance for response to wetting tests on collapsible soils can be conservatively (but only slightly conservatively) accounted for by taking the full wetting collapse strain for field conditions, the strain from the origin to the wetted curve (dry strain plus strain upon wetting), as suggested by Houston and Houston (1997). For expansive clays, the response to wetting appears to be conservatively estimated by first loading the specimen back to field overburden stress level, and then re-zeroing the LVDT reading before wetting (Singhal, 2010; Singhal et al. 2011).

When representative specimens of new compacted fill soils are required, the best that can be done is to prepare the specimens according to field compaction specifications. Of course, it is quite challenging, if not impossible, to match laboratory gradation precisely to field gradation, and the difficulties are exacerbated when soils are clayey and tend to develop clods and clumps in the field compaction process. Nonetheless, it is advisable in assessment of appropriate field compaction specifications with respect to compacted fill response to wetting that specimens be prepared as close as possible to that which will be acted in the field.

The samples tested as a part of this research were obtained with a Modified California sampler with A_r of 56%. The two most important parameters measured using these samples were the soil suction and the swell strain. Based on the research cited above it appears likely that the suction measured in the laboratory is at least slightly reduced compared to in-situ, but for the more plastic clays in particular (such as the expansive clays of this study) the reduction of suction is likely to be minor. Likewise, the swell strain would be expected to be slightly reduced because swell strain generally diminished with initial soil suction. Compensating errors are judged to be helpful in this regard (Houston W. , 2017) because disturbance reduces soil suction on the one hand, but on the other hand it tends to increase the available specific surface area – thus tending to increase swell. Reloading the specimen back to in-situ overburden stress also promotes a slight densification of the specimen, which also tends to increase swell.

3.1.2. Index Properties and Additional Data of the OPPD Tested Specimens

The partial wetting soil response of 21 intact clay specimens (10 from Denver and 11 from San Antonio) were studied. The soil index properties, in-situ moisture, dry density and net normal stresses were determined by the applicable ASTM standards on the bulk samples and ring trimmings obtained at each sample location. The total suction of each sample was measured using the WP4-C. The following table summarizes the previously described lab efforts for all 21 specimens denoted by their given ID (with D representing a Denver soil and SA representing a San Antonio soil), test boring (TB), and depth.

Table 2: Summary of Index Properties and Additional Data of the Intact Tested Specimens

ID	TB	Depth (m)	USCS	LL	PL	PI	%-#200	w (%)	γ_d (g/cm^3)	σ_{ob} (kPa)	Total Suction (kPa)
D-1	2	1.68	CL	38	16	21	68	10.1	1.79	32.31	3558
D-2	2	3.20	CL	42	16	26	81	9.1	1.56	53.49	3992
D-3	3	1.68	CL	36	12	24	69	9.9	1.75	31.64	5511
D-4	3	4.72	CL	52	24	28	57	23.7	1.40	79.95	834
D-5	3	5.33	CH	64	27	37	85	23.8	1.49	96.60	1450
D-6	3	7.77	CH	58	20	38	89	15.9	1.70	150.04	3475
D-7	5	1.68	CH	56	21	30	64	29.4	1.29	27.44	1075
D-8	5	3.20	CH	65	18	47	92	23.0	1.45	55.71	1400
D-9	5	4.72	CH	65	10	45	94	20.6	1.59	88.53	2271
D-10	5	6.25	CH	55	19	36	83	16.3	1.67	118.47	2194
SA-1	1	1.68	CH	69	15	54	94	29.3	1.45	30.88	390
SA-2	1	3.20	CH	67	16	51	91	21.3	1.43	57.65	526
SA-3	1	4.72	CH	77	20	57	95	29.7	1.40	83.98	1100
SA-4	2	3.20	CH	67	16	51	92	23.2	1.54	59.42	1100
SA-5	2	4.72	CH	82	17	65	92	32.8	1.35	82.84	873
SA-6	3	1.68	CH	67	15	52	91	24.0	1.48	30.02	390
SA-7	3	3.20	CH	58	16	42	91	21.1	1.56	59.18	458
SA-8	3	4.72	CH	81	16	65	94	27.0	1.41	82.69	726
SA-9	4	1.68	CH	66	16	48	87	20.2	1.61	31.74	1553
SA-10	4	3.20	CH	75	17	58	93	19.7	1.61	60.37	514
SA-11	4	4.72	CH	70	16	54	96	29.0	1.40	83.75	915

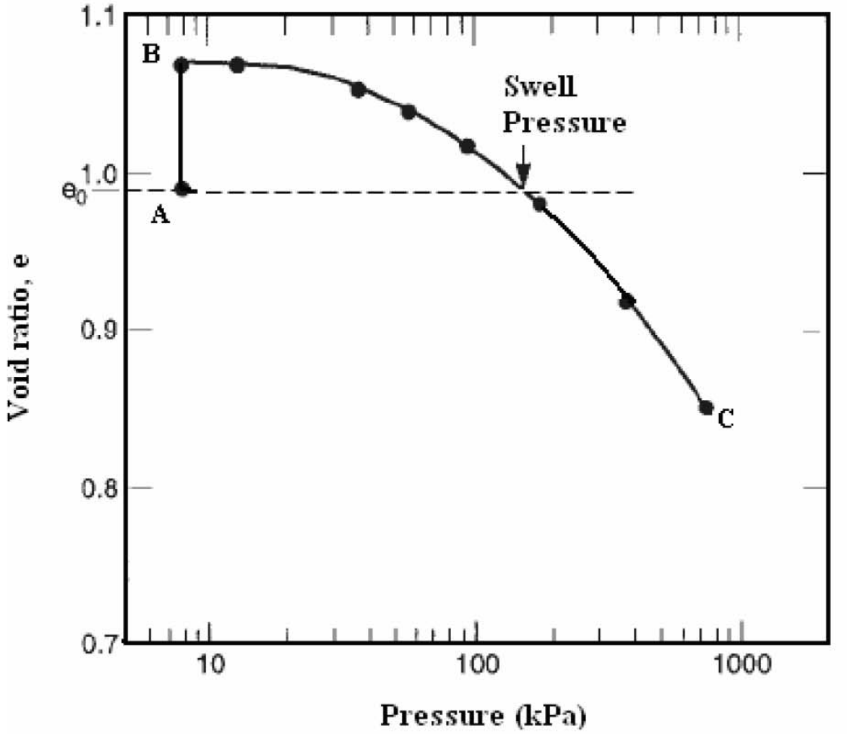
4 FULL WETTING SOIL REPSONSE

4.1 Response to Wetting Test (ASTM D4546, 2014)

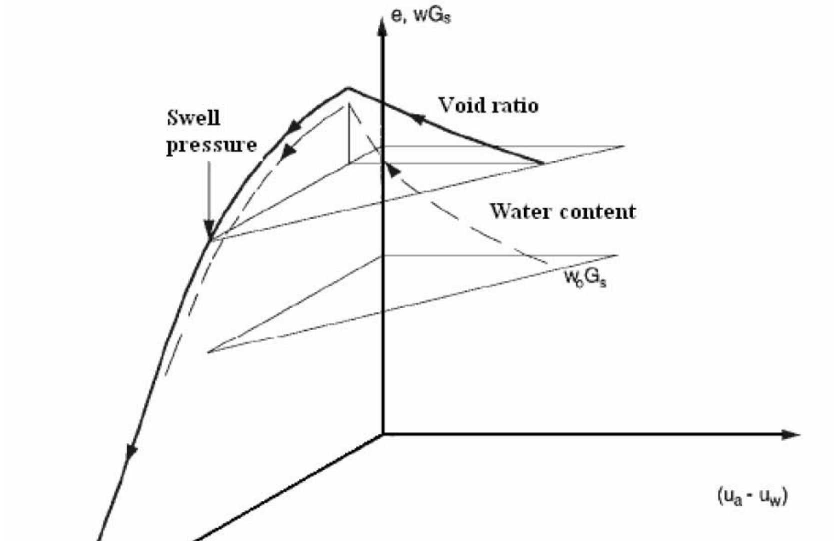
The Standard test method for one-dimensional swell or settlement potential of cohesive soils (ASTM D4546, 2014), typically referred to as the "response to wetting" test, is a simple and straightforward method for obtaining a soil's volumetric response under full inundation (i.e. zero suction). The test is also commonly referred to as a swell test or a collapse test depending on the tested soil type and the direction of volume change. The response to wetting test can also be used to measure the swell pressure of an expansive soil (Brackley, 1973; Justo, Delgado, & Ruiz, 1984; Ajayi, 1987). The current ASTM D4546 (2014) standard encompasses three separate procedures for determining the swell potential, described as:

- Method A – "*wetting-after-loading tests* on multiple reconstituted specimens"
- Method B – "*single-point wetting-after-loading test* on individual intact specimens under in-situ overburden stress or overburden stress plus anticipated structural load"
- Method C –"*loading-after-wetting test* on reconstituted or intact specimens under the desired load"

Methods B and C from ASTM D4546 (2014) allow for determination of the percent swell/collapse of a specimen under full wetting. Method C allows for determination of the swell pressure by compressing a soil which experienced swell after inundation, with incremental loads until the pre-wetting void ratio is returned. This method is commonly referred to as the load-back swell pressure (Figure 16), and is widely used in geotechnical engineering practice for percent swell and swell pressure determination as it only requires a single sample to obtain the desired information.



(a)



(b)

Figure 16: Free Swell Test Results Represented in (a) Conventional Two-Dimensional Manner and (b) on Three-Dimensional Stress Path Plot (Fredlund D. , 1995)

4.2 Constant Volume Oedometer Test

In a constant volume (CV) test, a reconstituted or intact specimen is subjected to a token-load or seating pressure (≈20 psf) in an oedometer apparatus and kept at a constant volume during the inundation process by simultaneously increasing the applied stress. The applied stress at which the specimen no longer has the tendency to swell is considered to be the swell pressure (Figure 17).

Similar to a consolidation test for normally consolidated soils, Fredlund, Hasan, and Filson (1980) proposed that the CV swell pressure must be corrected for sample disturbance. The method of correcting the swell pressure follows the graphical procedure of the Casagrande construction for obtaining the pre-consolidation pressure, illustrated on Figure 17.

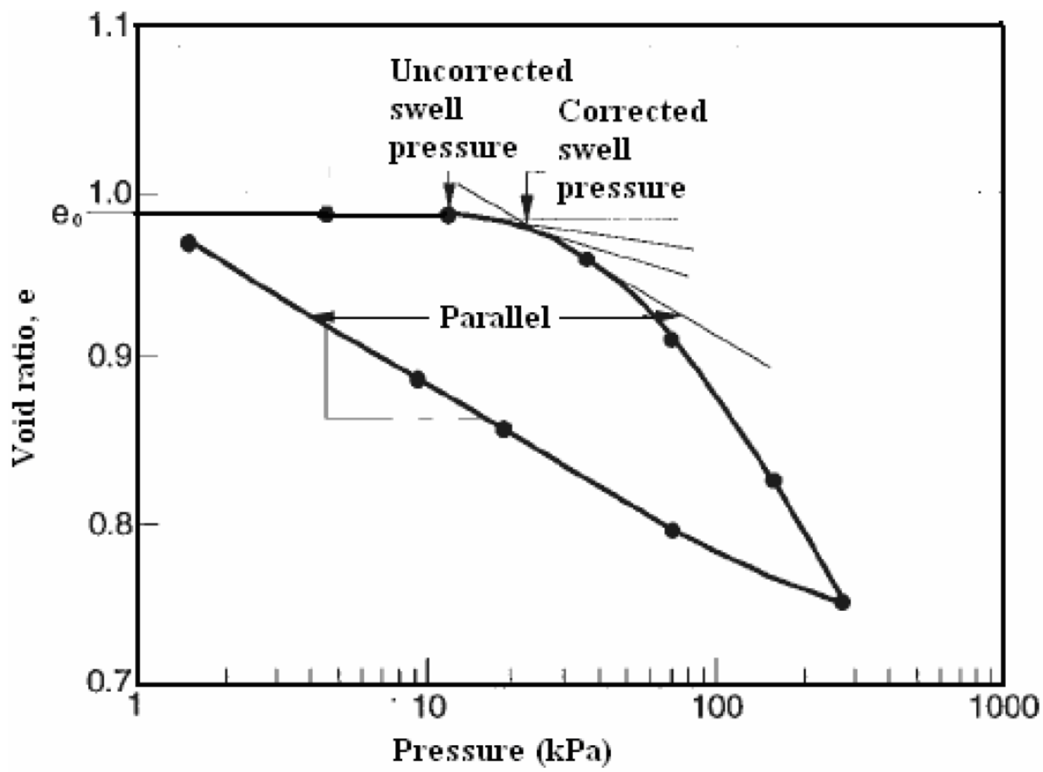


Figure 17: Void Ratio vs. Pressure Plot for Constant Volume Oedometer Test

CV swell pressures that are not corrected for sample disturbance can result in values that are three times less than the corrected value, as observed by Fredlund and Rahardjo (1993) and shown in Figure 18 below.

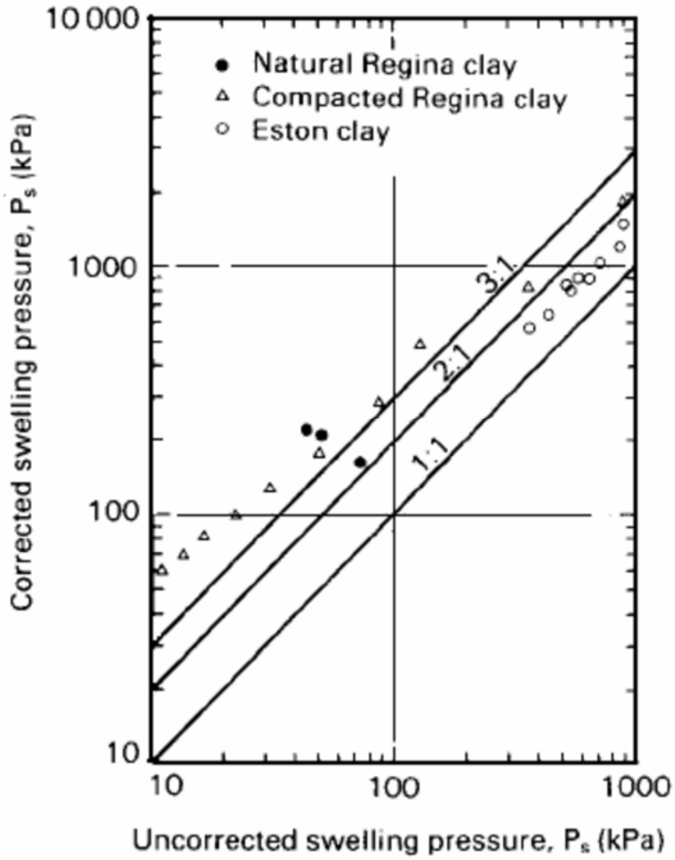


Figure 18: Comparison of "Corrected" and "Uncorrected" Swell Pressure Estimated from CV Test (Fredlund & Rahardjo, 1993)

The CV test is the method preferred by several researchers (e.g., Frydman and Calabresi (1987), Thompson et al. (2006), etc.) because the effects of hysteresis is limited since the soil specimen is kept at a constant volume throughout the duration of the test. However, due to the difficulties associated with running the test correctly, it is not a widely conducted test in civil engineering practice in the United States.

4.2.1. Load-Back Swell Pressure Correction

Several researchers have studied the relationship between the constant volume (CV) swell pressure and the load-back (LB) swell pressure.

Gilchrist (1963) observed that the difference between the CV swell pressure and the LB swell pressure increases as the initial void ratio decreases. The increase in inter-particle resistance (i.e. hysteresis effects) was suggested to be the reason the LB swell pressure tended to exhibit higher values than the CV swell pressure. Together with new experimental data on undisturbed specimens (Figure 19), Singhal (2010) evaluated the relationship of the CV swell pressure to the LB swell pressure from the previous works of Gilchrist (1963), Noble (1966); Lu (1969); Brackley (1975); Nelson and Porter (1980); El Sayed and Rabba (1986); Sridharan et al. (1986); Erol et al. (1987); Khaddaj et al. (1992); Feng et. al (1998); Attom and Barakat (2000); Al-Mhaidib (2006); Thompson et al. (2006); Nelson et al. (2006); and Nagaraj et. al (2009). From Singhal's investigation, the LB swell pressure was found to be approximately 1.4 to 1.5 times the CV swell pressure.

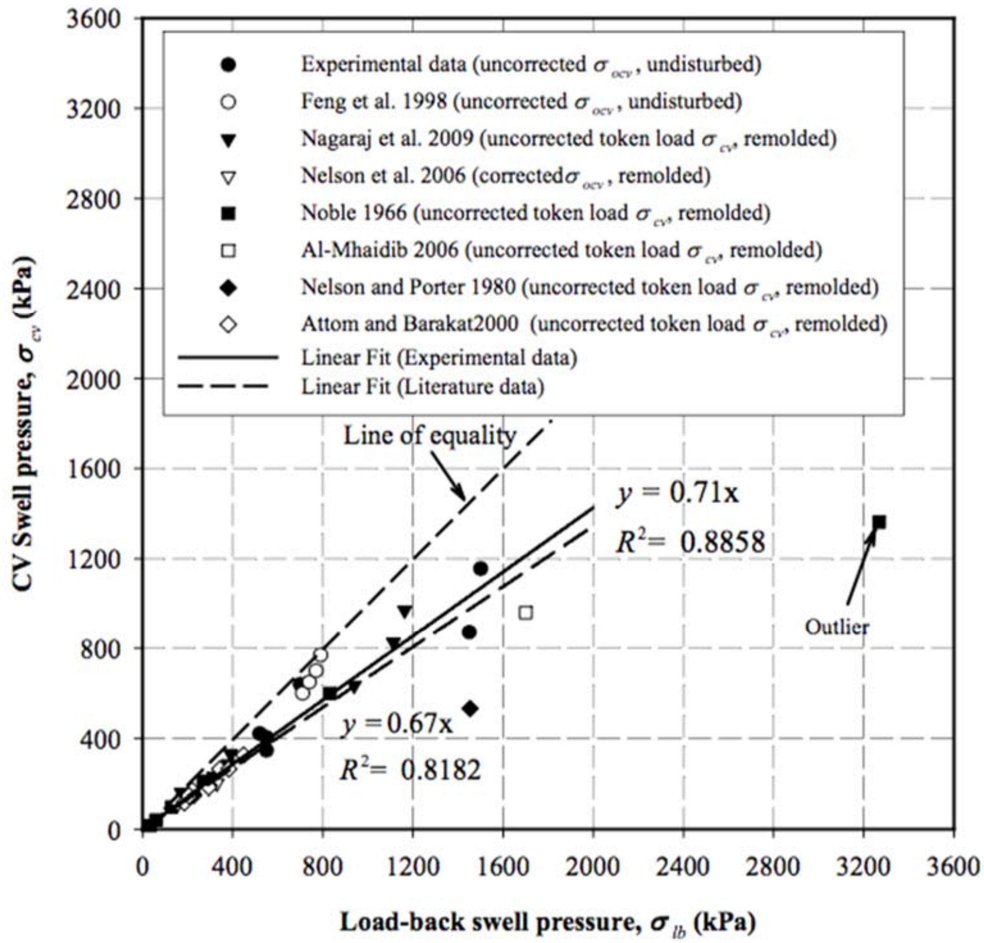


Figure 19: Load-back (LB) vs. Constant Volume (CV) Swell Pressure Relationship (Singhal 2010)

However, issues arise when using a fixed multiplier to convert the LB swell pressure to the CV swell pressure, particularly in samples that exhibit minimal volume change upon inundation, because a conversion factor of 1.4 to1.5 can result in a CV swell pressure that is less than the applied net normal stress (e.g. in-situ net normal stress). This error typically occurs when a soil swells under high net normal stress (e.g. a deep deposit of expansive soil). Using the Singhal (2010) correction factor of 1.41, Table 3 shows the problem of estimation of a swell pressure less than applied net normal stress for one undisturbed sample tested in this study. The samples tested in this study are from varying depths (5.5

feet to 20.5 feet), and therefore the use of the fixed conversion factor will not be appropriate due to relatively high applied overburden stress and limited swell. Rather, an alternative method proposed by Nelson, et al. (2006) is used in this study.

Table 3: Example Calculation of Error Caused by Fixed Conversion Factor of LB Swell Pressure to CV swell Pressure

ID	Depth (ft)	Depth (m)	$\sigma_{ob}(kPa)$	$\varepsilon_{ob}(\%)$	$\sigma_{lb}(kPa)$	$\left(\dfrac{\sigma_{lb}}{\sigma_{cv}}\right)$	$\sigma_{cv}(kPa)$
D-10	20.5	6.2	118.5	0.3	157.7	1.41	111.9

Nelson et al. (2006) also proposed a correlation to obtain the CV swell pressure from the LB swell pressure, which uses a proportionality constant (λ):

$$\sigma_{cv} = \sigma_{ob} + \lambda(\sigma_{lb} - \sigma_{ob}) \quad (2)$$

From the experimental data, Nelson et al. (2006) determined that the proportionality constant lies between 0.5 and 0.7. However, this range of the proportionality constant was based off a limited number of tests on remolded samples (i.e. 2 CV and 4 LB tests). Data from Singhal (2010) on CV and LB swell tests using 6 “companion” undisturbed clay specimens from San Antonio, TX, was used to determine and better approximation for the proportionality constant as it pertains to this study (Table 4). The Nelson et. al (2006) equation was used to back calculate the proportionality constant. The average proportionality constant was determined to be 0.7, which was used in this study to convert the LB swell pressure to CV swell pressures.

Table 4: Determination of Nelson et. al (2006) proportionality constant from Singhal (2010) CV and LB test data

Specimen No	Specimen ID	Test Type	σ_{ob} (kPa)	$(u_a-u_w)_i$ (kPa)	Swell Pressure (kPa)	% Swell	$\left(\frac{\sigma_{lb}}{\sigma_{cv}}\right)$	Back-Calculated (λ)
3	SA-B5-12(2)	CV	71	385	421	0	1.24	0.78
15	SA-B5-12(3)	LB			520	2.93		
16	SA-B5-24(2)	CV	142	1008	640	0	1.09	0.89
17	SA-B5-24(3)	LB			700	2.45		
2	SA-B6-12(2)	CV	71	3977	404	0	1.36	0.70
18	SA-B6-12(1)	LB			550	2.69		
4	SA-B6-20(2)	CV	118	1221	347	0	1.59	0.53
19	SA-B6-20(1)	LB			550	1.83		
6	SA-B6-32(2)	CV	189	2216	1155	0	1.30	0.74
20	SA-B6-32(1)	OS			1500	4.85		
7	SA-B6-24(2)	CV	142	3063	872	0	1.72	0.54
21	SA-B6-24(1)	OS			1500	4.98		
							Average	**0.70**

4.3 Results of ASTM D4546 Testing

The following table (Table 5) summarizes the results of the response to wetting tests on the 21 intact samples. The full wetting strain was determined by inundating the specimen under in-situ net normal stress. The LB swell pressure was also determined and was then corrected to approximate CV values using the previously described Nelson et al (2006) correlation with a proportionality constant of 0.7 from Singhal (2010) data. Note that specimens D-2, D-4, and D-6 compressed when inundated under the corresponding in-situ net normal stress. The plots for each response to wetting test are presented in Appendix A.

Table 5: Summary of Response to Wetting (ASTM D4546) Test Results

ID	σ_{ob} (kPa)	ε_{ob} (%)	σ_{lb} (kPa)	σ_{cv} (kPa)
D-1	32.31	1.45	71.7	59.9
D-2	53.49	-0.99	13.4	25.4
D-3	31.64	1.86	114.7	89.8
D-4	79.95	-0.72	20.0	38.0
D-5	96.60	1.12	241.4	198.0
D-6	150.04	-0.53	37.5	71.3
D-7	27.44	0.2	35.4	33.0
D-8	55.71	1.19	133.8	110.4
D-9	88.53	1.22	215.1	177.1
D-10	118.47	0.28	157.7	146.0
SA-1	30.88	2.17	215.1	159.8
SA-2	57.65	1.85	145.8	119.3
SA-3	83.98	1.42	196.0	162.4
SA-4	59.42	1.48	145.8	119.9
SA-5	82.84	0.55	176.9	148.7
SA-6	30.02	0.7	47.8	42.5
SA-7	59.18	0.42	86.0	78.0
SA-8	82.69	1	176.9	148.6
SA-9	31.74	3.53	358.5	260.5
SA-10	60.37	0.62	112.3	96.7
SA-11	83.75	1.83	337.0	261.0

5 SUCTION-BASED LABORATORY TECHNIQUES

5.1 Filter Paper Test

Filter paper method (FPM) tests for total suction were conducted by CTL Thompson, Inc., an accredited materials laboratory out of Denver, Colorado. The filter paper tests were conducted according to the ASTM D5298 procedure using Whatman filter paper no. 42 to determine the total suction of 101 undisturbed soil specimens from Denver.

5.2 WP4-C

Recently the chilled-mirror Water Potential Meter (WP4-C) was developed by Decagon Devices, Inc. (now Meter, Inc) for determination of total suction of soil samples within minutes, rather than days required for the FPM. The WP4-C (Figure 20) measures water potential by determining the relative humidity of the air above a sample in a closed chamber (conforming to ASTM 6836). Once the sample comes into equilibrium with the vapor in the WP4-C's sealed chamber, the instrument finds relative humidity (i.e. total suction) using the chilled mirror method. This method entails chilling a tiny mirror in the chamber until dew just starts to form on it. At the dew point, the WP4-C measures both mirror and sample temperature with 0.001°C accuracy.

Several testing modes are available with the WP4-C. For purposes of this study, the device was set to precision mode, which allowed for total suction measurements up to 300,000 kPa and as accurate as ±25 kPa (Decagon Devices, Inc., 2011; Leong, Tripathy, & Rahardjo, 2003).

Figure 20: WP4-C Device (Decagon Devices, Inc.)

It is important to note that the sample containers (depicted in Figure 20) are relatively small (0.35-inch height and 1.45-inch diameter) when compared to other typical geotechnical lab testing sample containers.

The effect of sample disturbance on the WP4-C suction measurements was explored in this study. WP4-C total suction measurements were taken on 27 undisturbed samples from the Denver soils. Following the equilibrated suction reading for each specimen, the specimen was removed and manually physically broken down into smaller particles to create a "companion" disturbed specimen. The process was conducted in a time efficient, temperature-controlled manner, in order to limit any change in the samples moisture content. The following plot (Figure 21) illustrates the comparison of the disturbed vs. undisturbed suction values, with the line of equality showing a reference to no difference. In general, the disturbed specimens had somewhat lower suction than the undisturbed specimens. Other researchers have also reported a reduction of soil suction with sample disturbance (Singhal, 2010), however, in this study the effects of sample disturbance on the WP4-C suction values was not found to be highly significant except at low values of suction. The plot of the comparison (Figure 21) illustrates greater impact of disturbance in a low suction range (approximately 150 to 1000 kPa).

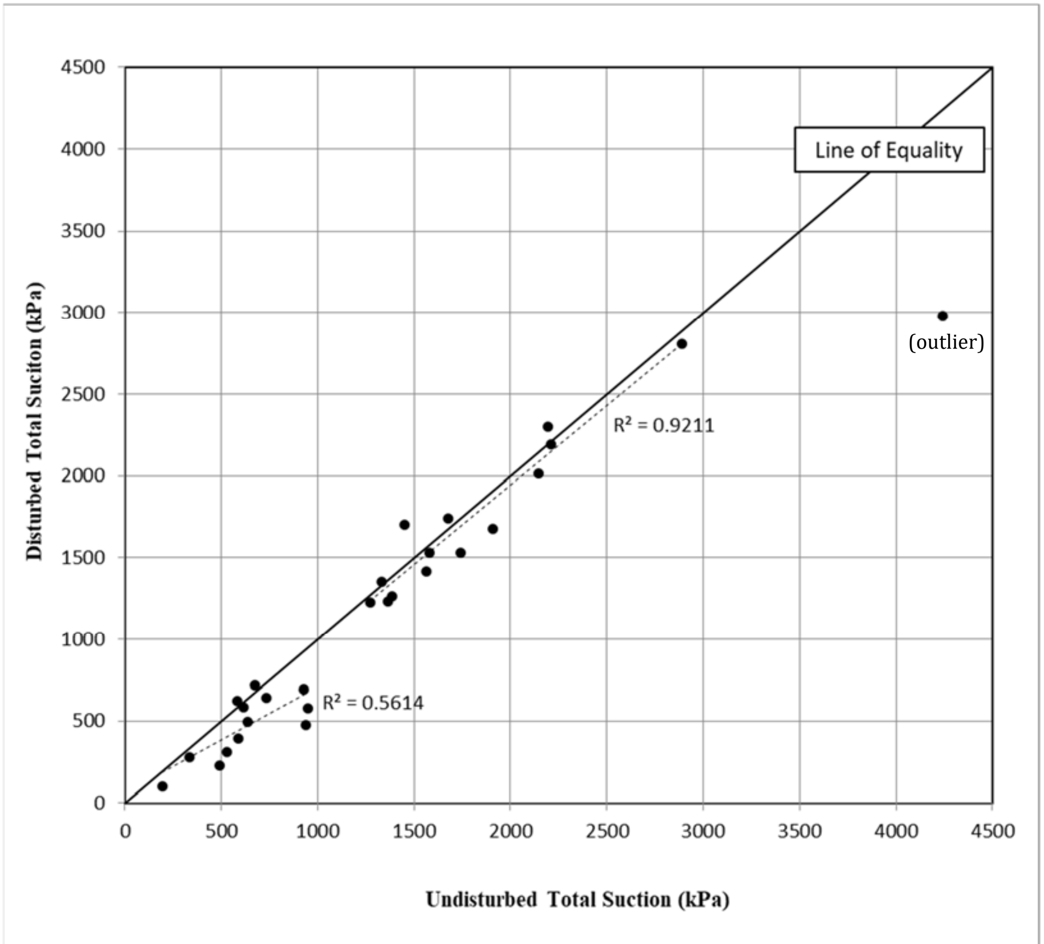


Figure 21: Disturbed vs. Undisturbed Total Suction by WP4-C

5.3 WP4-C to Filter Paper Method Comparison

A comparison of the WP4-C device to the widely accepted filter paper method (FPM) for soil suction determination was conducted to evaluate the feasibility and reliability of the WP4-C in engineering practice.

A previous study comparing a Decagon chilled mirror suction measurement device and the FPM was conducted by Petry and Jiang (2007), but using the original model of the Decagon dew point water potential meter (WP4). The original model (WP4) did not have the precise temperature control of the more recent WP4-C. Seven accredited geotechnical laboratories participated in the Petry and Jiang study. Total suction measurements on three different clay soils from Texas, Missouri, and New Mexico were conducted by each laboratory. Each sample was remolded to specific densities and moisture contents so that all sample variability could be minimized (i.e. "identical samples"). The results from the round robin study concluded that the WP4 total suction measurements closely agreed with the filter paper method total suctions, but the WP4 values tended to be slightly higher. The study also revealed that the filter paper method exhibited a higher between lab variance than the WP4.

An approach similar to that of Petry and Jiang was used in this study, but using the WP4-C rather than the original WP4 device. The filter paper testing was conducted on undisturbed samples by CTL Thompson, Inc., an accredited materials laboratory out of Denver, Colorado. WP4-C total suction tests were performed by the author on companion samples (i.e. from the same sample tube at a given depth) that were shipped to Tempe from Denver in moisture and vibration controlled packaging. Since the WP4-C sample container is much smaller than most undisturbed soil sample rings, a smaller relatively undisturbed specimen was carefully cut out of the larger undisturbed tube sample. The following figure presents a comparison of the total suction results obtained in the study.

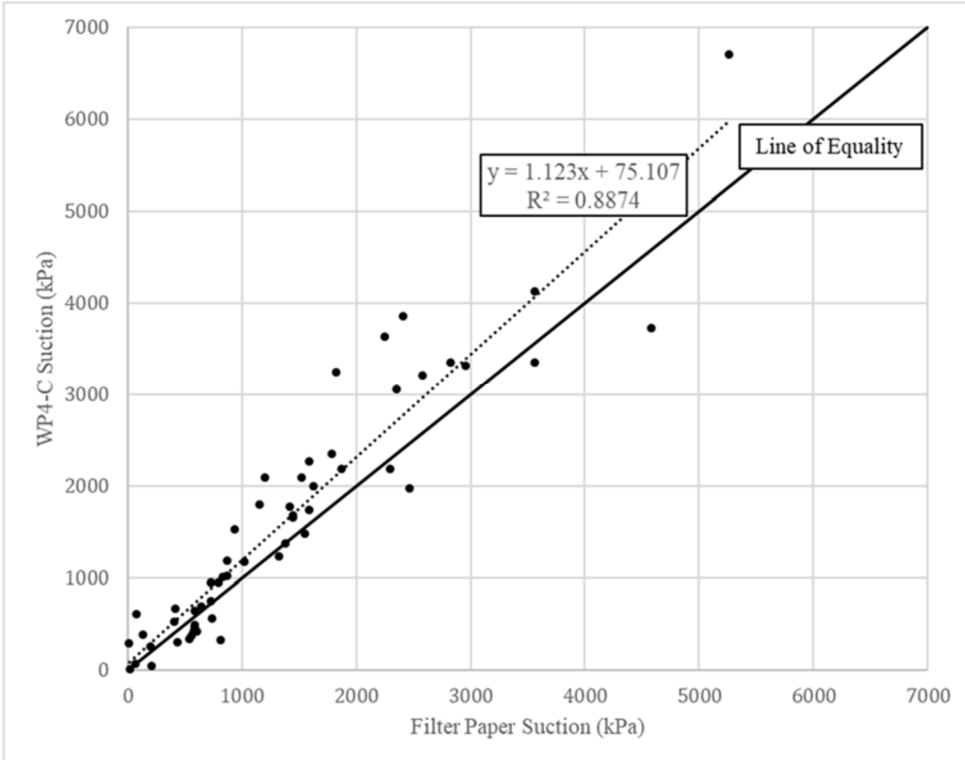


Figure 22: WP4-C vs. Filter Paper Total Suction

The results show a similar trend to the Petry and Jiang (2007) study, in which the WP4 provided slightly higher total suction readings. However, when the moisture contents of each sample from the individual test were compared (Figure 23), the samples tested in the WP4-C were found to be somewhat drier than the samples tested by the filter paper method, which would lead to slightly higher suction values for the WP4-C device. Therefore, the previously described trend that the WP4 produced slightly higher total suction values than the filter paper method, though reasonably supported by the Petry and Jiang study, was not confirmed in this study wherein the match between the WP4-C and FPM suction values was found to be excellent in consideration of the slightly decreased moisture content for the WP4-C specimen. It is well established that a given soil will have higher suction at decreased moisture content.

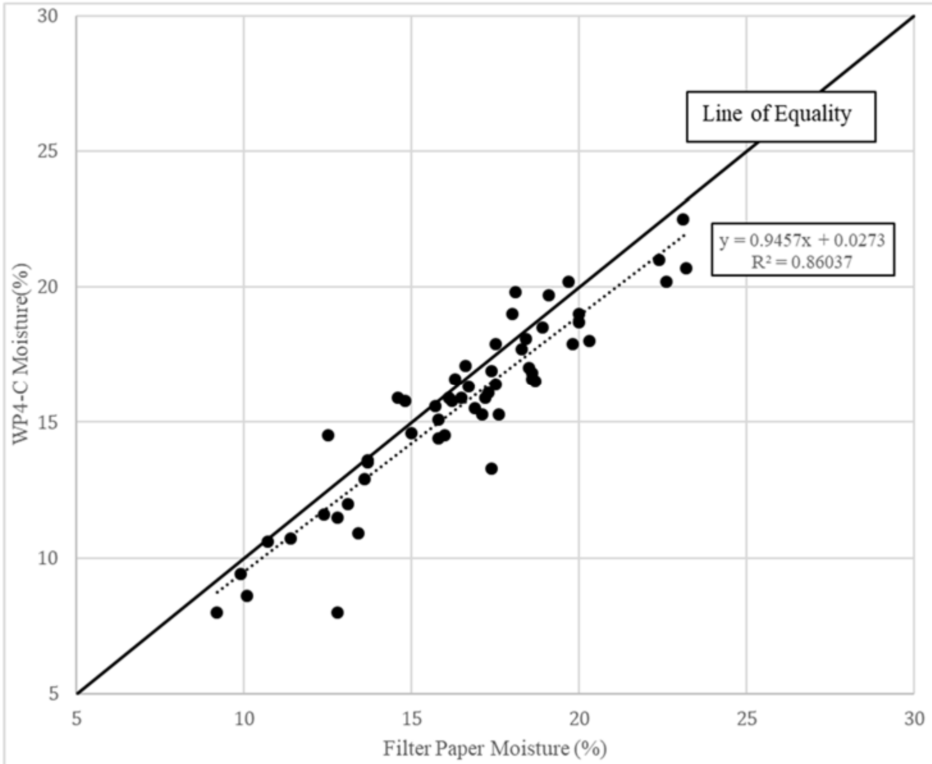


Figure 23: WP4-C vs. Filter Paper Specimen Moisture Content

To aid in the evaluation of how "identical" the undisturbed companion specimens were to each other, the coefficient of variance was computed (Table 6) for each batch of samples used in the test method.

Table 6: Statistical Comparison of Filter Paper Method and WP4-C for Total Suction Measurements

Statistical Variable	Filter Paper			WP4-C		
	w (%)	pF	kPa	w (%)	pF	kPa
Mean	16.5	3.9	1361.1	15.6	4.0	1603.6
Std. Dev.	3.3	0.5	1118.1	3.3	0.5	1332.9
COV (%)	19.73%	13.06%	82.15%	21.23%	12.33%	83.12%

The results of the statistical comparison indicate that the variance between the companion undisturbed samples was very minimal and did not play a significant role in evaluation of the difference between the two testing methods.

The findings of the Petry and Jiang (2007) round robin study, along with comparisons conducted in this study, it is apparent that the WP4-C provides total suction values that are essentially equivalent to those of the widely accepted filter paper method. Since the WP4-C can provide a higher range of total suction measurement in a fraction of the equilibration time, and exhibits less between lab variance (i.e. more "user friendly") than the filter paper method, the WP4-C is recommended as good method for the laboratory determination of soil total suction.

6 DIRECT MEASUREMENT OF PARTIAL WETTING RESPONSE OF NATURAL CLAY SOIL

6.1 Oedometer Pressure-Plate Device

To directly measure the volume change and moisture content change associated with changes in suction at fixed net normal stress the oedometer pressure- plate device (OPPD) can be used. The OPPD uses the axis translation technique (Hilf, 1956), requiring high-air-entry-value (HAEV) ceramic stones for control of suction up to 1500 kPa. The HAEV ceramic stone is used to allow translation of reference for the pore-water pressure from standard atmospheric condition to the final applied air pressure in the chamber by preventing bubbling of air into the water chamber beneath the soil specimen. The SWC-150: Fredlund Soil Water Characteristic Device developed by GCTS in Tempe, Arizona is one of the most common OPPD devices (Figure 24).

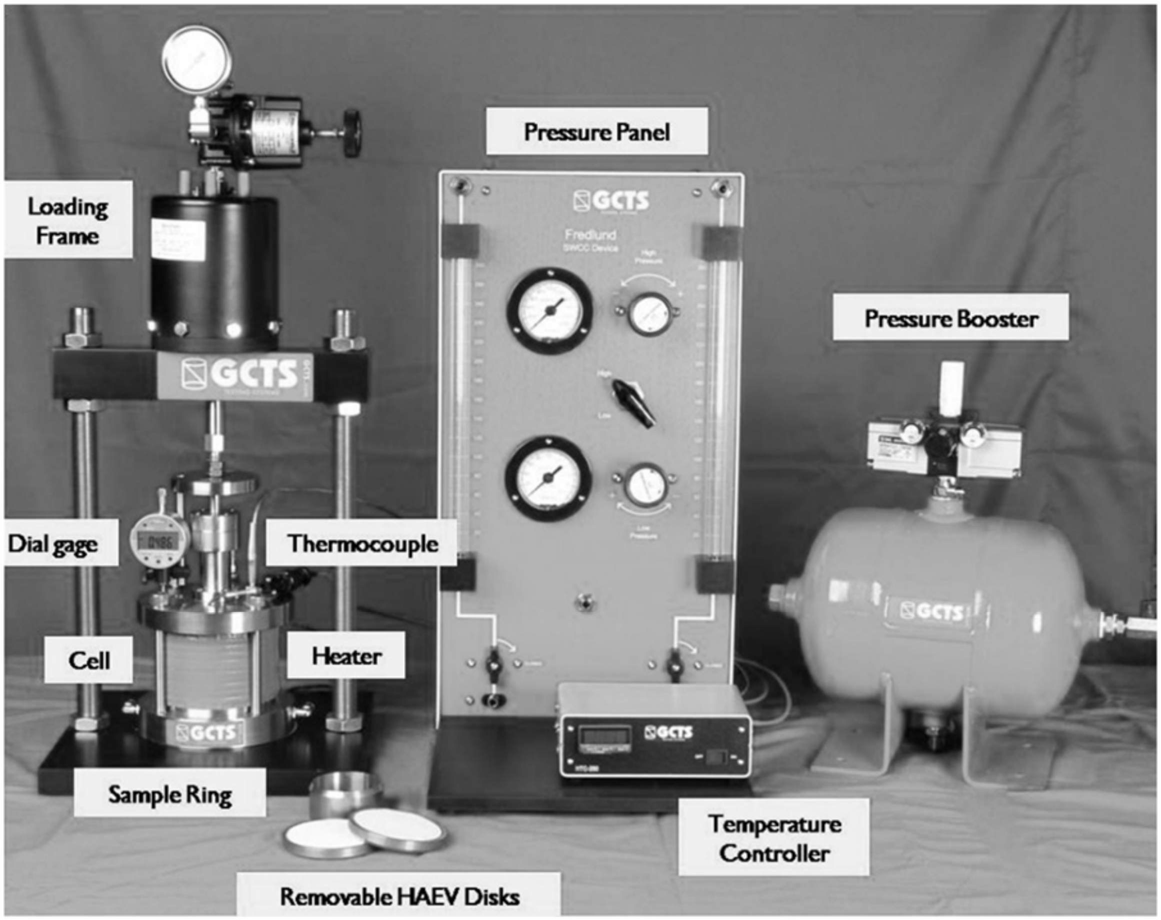


Figure 24: Oedometer Pressure Plate Device (OPPD) (GCTS, Tempe AZ)

The volume/mass of water within the specimen can be recorded at any time using the two manometers on the device's panel. The panel also contains two pressure regulators, one high and one low, for easily adjusting the pressure within the chamber. The load plate above the pressure chamber allows for the net normal stress to be applied to the specimen. The OPPD can be used for in-situ SWCC testing since both stress state variables can be independently applied to undisturbed soil samples.

The main limitation with this OPPD device, as well as all the devices that use the axis translation technique, is the maximum capacity of the high-air-entry ceramic stones. The maximum air-entry value of the stone is currently 15 bar (i.e. 1500 kPa). This limits the measurement or control of matric suctions values to 1500 kPa. This results in higher values of suction on the SWCC curve either being interpolated using empirical equations, or developed from separate total suction measurements. Equilibration times of the axis-translation method also limit the practicality of its use in most commercial geotechnical laboratories. Samples can take from days to weeks to equilibrate in the OPPD, depending on soil type and soil moisture state and the associated unsaturated hydraulic conductivity, which decreases with decreasing soil suction.

6.2 Oedometer Pressure-Plate Device Testing of Natural Highly Plastic Soils

The following detailed procedure outlines the necessary steps and precautions for testing an undisturbed soil's response to partial wetting, induced by changes in matric suction using the oedometer pressure plate device (OPPD) developed by GCTS (Figure 24). The OPPD can be used (up to 1500 kPa) to determine the initial matric suction of the soil specimen (i.e. in-situ matric suction of an undisturbed natural specimen), the soil water characteristic curve (SWCC) of the soil, and the volume change to suction change relationship (i.e. suction compression index) of the tested soil.

There is currently no accepted standard for OPPD testing of undisturbed clay specimens. The following procedure, adapted from the GCTS (2007), outlines the necessary steps as determined by the author. The procedure developed in this study also includes a method for the determination of the in-situ matric suction of the soil,using as an aid the WP4-C device (discussed in more detail later). Note that the undisturbed samples used in the procedure were obtained from the modified California spilt spoon sampler, as reflected in the proposed procedure. Refer to Figure 24 for the names of the different parts of the OPPD system. The heat jacket, thermocouple, and temperature controller shown in Figure 24 were not incorporated in this study.

Step by Step Procedure:

1. Remove the pre-saturated (24-hour) HAEV ceramic stone from the water and carefully pat the surface dry. Record the saturated surface dry (SSD) weight of the stone and place it into the base of the OPPD chamber. Ensure that is secured tightly. A 15-bar HAEV ceramic stone must be used when testing cohesive soils. The ceramic stone must be fully saturated (at least 24 hours) prior to testing.

2. Remove undisturbed soil rings (6), in whole, from sampler tube and determine the in-situ density of the soil.

3. Carefully trim one specimen (i.e. ring) out of the center of the stack and record its mass.

4. Place a small cutting from the sample, preferably a relatively undisturbed specimen trimmed from the larger sample into the WP4-C for determination of the in-situ total suction of the sample. Measure the mass of the remaining soil cuttings and place in the oven for determination of the moisture content.

5. Quickly place the specimen at the center of HAEV ceramic stone located at the base of the OPPD chamber.

6. Place steel confining ring around specimen and a steel loading plate atop the specimen. It is important that porous stones are not used as the load plate as they exhibit matric suction stresses themselves and can alter the moisture content within the specimen. It

is also important that the load plate not be connected to the load shaft of the OPPD. The load plate must be free to move in and out of the specimen's confining ring, while having full contact with the surface of the soil specimen. If the load plate is attached to the shaft above and the specimen is not directly centered below it, the load plate can make contact with the edge of the confining ring, making it no longer in full contact with the soil.

7. Close the OPPD chamber by tightening the four 4.5-inch long socket-head cap screws (SHCS) that seal the cell walls in a cross pattern. It is also recommended that vacuum grease be placed at the location of the O-rings above and below the OPPD chamber walls to help seal the chamber and minimize potential air leaks.

8. Connect tubing from the valves of the manometers to the base of the OPPD chamber; as well as, the tube which connects the pressure outlet from the pressure panel to the roof of the OPPD Chamber. Ensure all tubing is tightly secured.

9. Zero the dial indicator attached to the OPPD load shaft. It is recommended that a conventional (i.e. not electricity dependent) dial be used, a discussion of which is presented later in this report.

10. Calculate the in-situ net normal stress of the specimen by multiplying the in-situ density of the soil by its associated depth. The net normal stress can be applied to the top of the load shaft by a load frame or with the use of dead weights. The weight of the loading plate and other loading apparatus should be taken into account.

- Continuously monitor recompression strain of the sample within the OPPD until equilibrium (in-situ density) has been reached. Calculate in-situ degree of saturation of sample using the current density and moisture content from step 4.
- When testing soils with initial suction values greater than the bubbling pressure of the HAEV ceramic stone (i.e. 1500 kPa), it is crucial to continuously observe the volume change as the specimen will begin to reach equilibrium with the saturated 1500 bar stone below it. The specimen will have the ability to pull water from the capillary pores in the saturated ceramic stone until its matric suction value has dropped below 1500 kPa due to the increase in moisture. Continuous observations during this step are not as crucial when the specimen's initial suction is less than 1500 kPa because the ceramic stone will not give up moisture to the soil.

11. Once it has been determined that the soil is at equilibrium with the applied net normal stress, the manometers and reservoir below the OPPD chamber must be filled with de-aired water. Ensure that the pressure panel is sitting at a level position. Open the valves at the bottom of the manometers and begin to fill the system with water through one of the openings at the top of the pressure panel. Fill until the water levels in the manometer tubes reaches about half way up the tubes.

12. Use the flushing device (ball-pump) to expel any remaining trapped air in the base within the system (i.e manometers, connecting tubes, and OPPD reservoir). Be careful not spill water out of the opposite manometer opening. The flushing process should be

conducted in a pulsing manner. Repeat the flushing process until no air bubbles are visible in the connecting tubes and the manometers. If the water level in the manometers is uneven after the flushing process, there is most likely still trapped air within the reservoir in the OPPD cell base. Repeat flushing until the water level in each manometer is equal.

13. The initial suction of the soil can now be induced into the OPPD cell using the regulator knobs on the pressure panel. The initial matric suction or starting suction for the OPPD test is determined by subtracting an estimated osmotic suction from the WP4-C measured total suction (step 4). The estimated osmotic suction value is discussed later in this report.

    - If the estimated initial matric suction value is greater than 1400 kPa, use 1400 kPa as the starting pressure and skip step 14.

    - Check the system for any air leaks. It is important that there be no air leaks during the test. Check particularly around the O-ring on the bottom of the cell wall and the O-ring at the top plate. A mixture of soapy water can be used to check for leaks

14. Continuously monitor moisture changes (via the manometer tubes) and density changes (vial the dial indicator) and determine degree of saturation if the measurements change. If the current degree of saturation begins to deviate from the in-situ value determined in step 12, adjust chamber pressure (i.e. suction) until the degree of saturation of the sample equilibrates at the in-situ value.

- If the degree of saturation begins to increase, slighty lower the pressure
- If the degree of saturation begins to decrease, slightly raise the pressure

Once equilibrium has been ensured, record the current suction value as it is considered to be equivalent to the in-situ matric suction of the soil specimen. The soil specimen is now considered to be reasonably returned to its in-situ stress states (net normal stress and matric suction). Therefore, this point should be used as the "Zero Strain" reference point in all wetting/drying induced volume change calculations.

15. The first suction change can now be induced. For evaluation of partial wetting volume change, chose at least 3 target suction values less than the initial suction. It is recommended that the gradients of the target suctions be fairly equivalent and cover the full range of suction from the initial value to the lowest desired value. Depending on the pressure to be applied, switch the valve to LOW or HIGH. Use the corresponding regulator knob to apply the pressure in the cell. The pressure compensator on the top plate will automatically equalize the pressure exerted on the piston from within the chamber.
16. Leave the system for equilibration, taking water volume change and density readings on a daily basis.
17. Flushing of diffused air should be completed frequently as diffused air through the ceramic stone and into the water reservoir can restrict the flow of water in and out of the specimen. Padilla, Perera, Houston, Perez, & Fredlund (2012) quantified the air diffusion through the HAEV ceramic disks (Table 7). From their findings, it is recommended that flushing occurs twice or more a day to minimize the air

accumulation in the system.

Table 7: Quantification of Air Diffusion Through High Air-Entry Ceramic Disks (Padilla, Perera, Houston, Perez, & Fredlund, 2012)

Ceramic Disk	Applied Air Pressure (% of Bubbling Pressure)	Average Maximum Air Accumulation In a Day (cm^3)	Frequency of Flushing
1-bar	up to 100	Not Measurable	Once in three days
3-bar	up to 100	Not Measurable	Once in three days
5-bar	up to 100	0.10	Once in two days
15-bar	< 50	0.85	Once a day
	> 50	2.14	Twice or more a day

- Equilibration is considered attained, and the system is ready to receive the next pressure increment when the volume of water in the manometer readings no longer change over a 48-hour period with 15-bar ceramic stones. The time required for the equilibration is discussed later in this thesis.

18. Repeat the same procedure for the remainder of the pressure increments. If a drying cycle is desired, the suction may be increased, and the test continued. At the end of the last pressure increment, take the readings, release the pressure, disassemble the apparatus, and remove the soil specimen.
19. Record the weight of the moist specimen and place the specimen in an oven to dry. Oven dry the soil for at least 24 hours at 110 ºC and record the dry weight in order to determine the final water content.
20. Remove the ceramic stone, bring to a saturated but dry surface condition, and record the weight. The difference between the initial weight and the final weight of the

ceramic stone will indicate if any water has been absorbed or released from the ceramic stone during the test. If the difference is significant, the volume readings may need to be adjusted.

21. Once the dry weight of the soil is available, calculate the initial amount of water in the soil, the initial water content, initial dry density, and initial degree of saturation. Using the initial data, calculate the water released, water content, specimen height, specimen volume, dry density, and degree of saturation for each pressure increment.

6.2.1. Key Aspects of the OPPD Testing of Natural Clay Soils

The time necessary for the soil to equilibrate to changes in matric suction is the main cause of difficulties when using the OPPD to test partial wetting response of a clay soil. Figure 25 below illustrates the effect of both plasticity and initial suction value. The average days/equilibration point for the tested specimens ranged from 9 to 13 days. The whole duration of the test can last several weeks to months depending on the desired number of data points.

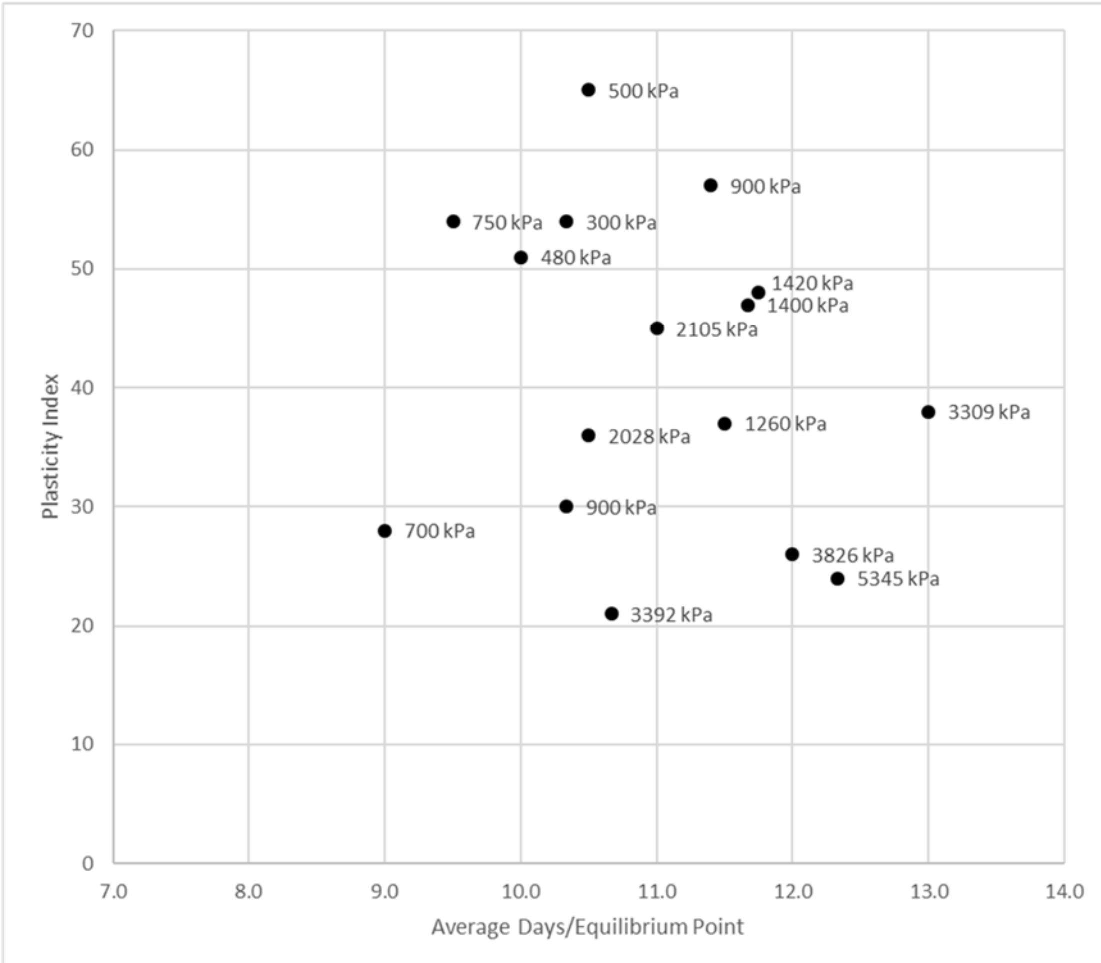


Figure 25: Average Days per OPPD Testing Equilibration Point with Respect to PI and Initial Suction Value

The reason for the individual effects of each plasticity and suction are explained in Figure 26. The relationship between the unsaturated hydraulic conductivity (permeability) and matric suction is plotted for a clean sand (i.e. nonplastic) and a clayey silt (i.e. moderately plastic).

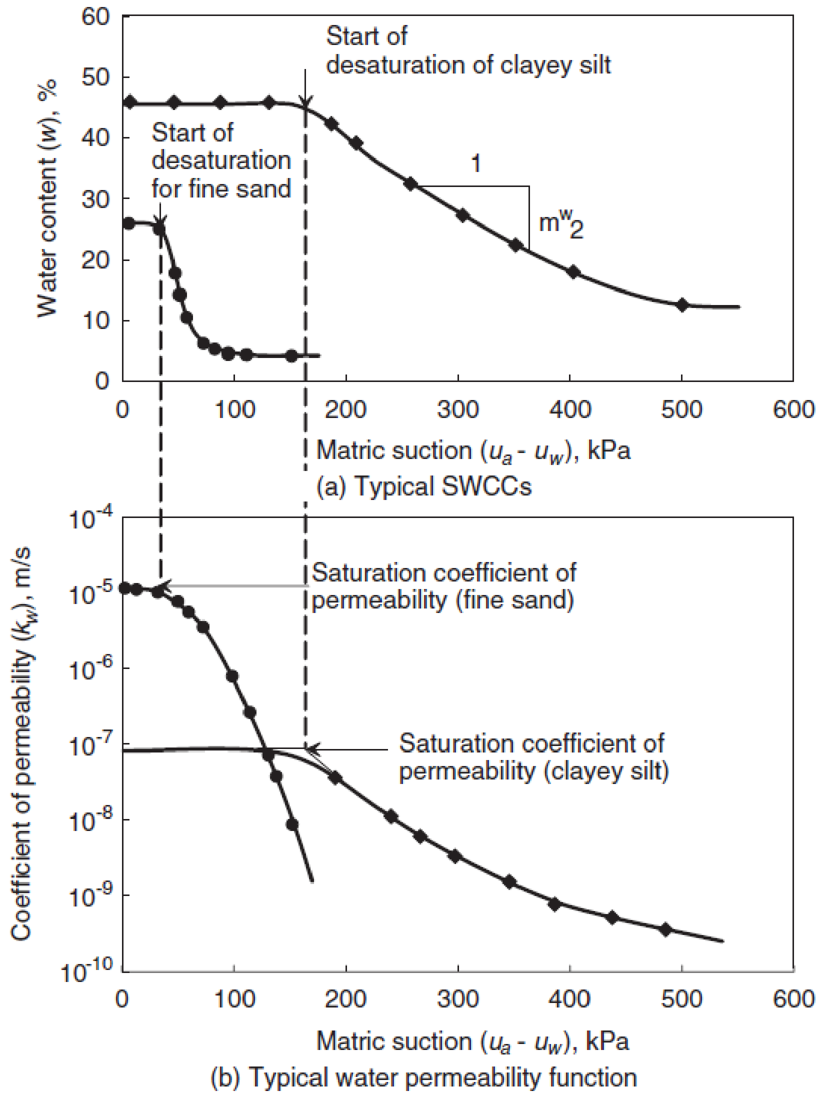


Figure 26: Typical Relationship Between SWCCs (a) and Estimated Permeability Functions (b) for Sand and Clayey Silt (Fredlund, Rahardjo, & Fredlund, 2012)

The figure illustrates the effect of the lower plasticity soil having a higher hydraulic conductivity at a given suction value compared to higher plasticity soil, explaining why the higher plasticity specimens (Figure 25) tended to have longer average equilibration times. Figure 26 (b) also depicts that the hydraulic conductivity for a given soil type will significantly decrease as the soil dries, which explains why the specimens with the higher initial suctions (Figure 25) tended to have longer average equilibration times for a give plasticity range.

The long equilibration times for testing highly plastic soils for partial wetting response using the OPPD makes it typically impractical for a commercial geotechnical engineering project, as a client will not typically want to wait weeks to months for the completion of lab testing. The long equilibration times also increase the chance for mechanical/electrical issues with the OPPD device itself during testing including:

- power loss to any electronic components such as LVDTs, digital dial indicators, and pressure transducers
- leaks within the OPPD system such as in the cell or plastic tubing
- loss of water within the manometer tubes due to evaporation
- buildup of piston friction which acts as an additional stress/resistance on a swelling specimen

It is recommended that the entire OPPD system is setup to run "off the grid", with no electronic components and that all tubing be replaced periodically in between tests. Evaporation in the manometer tubes should be allowed to occur during the test, but the amount of water loss should be monitored and accounted for during the calculations of the specimen's moisture content. It is recommended that a replica OPPD monometer setup be used in order to record the amount of water lost to evaporation (Figure 27).

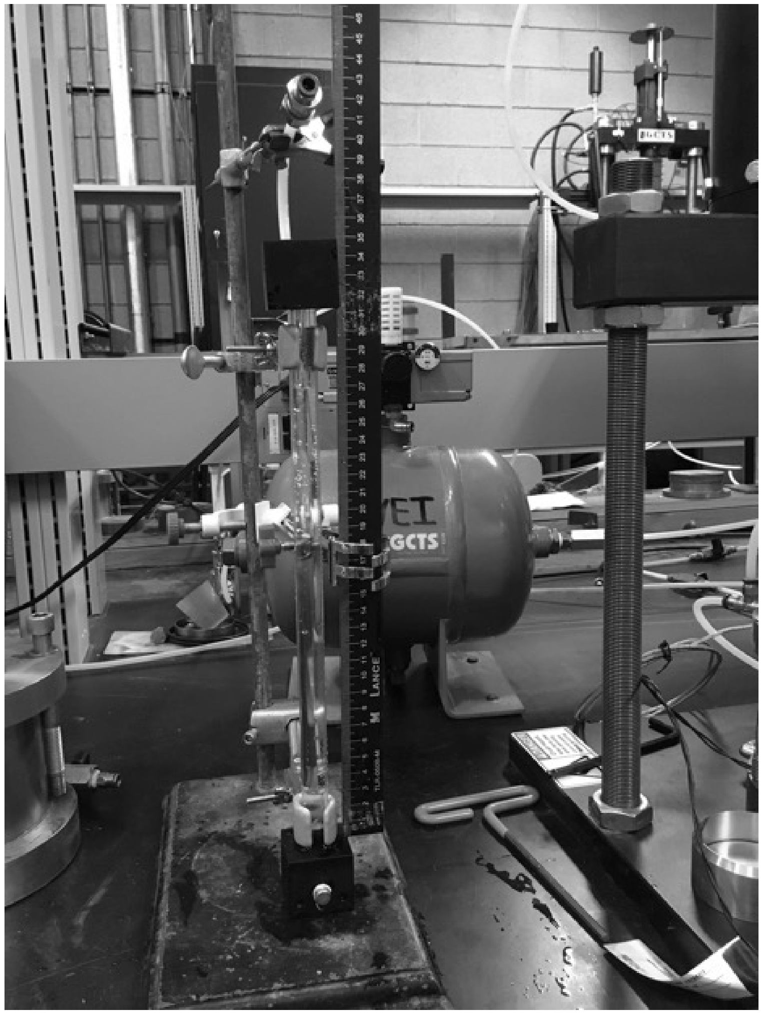


Figure 27: Replica OPPD Manometer Setup

Proper temperature control in the laboratory will aid in the minimization of evaporation of water. It is not recommended that oil be used atop the water within the manometers to limit evaporation. Mixing of the water and oil during the frequent flushing of the trapped air below the chamber will lead to difficulties when trying to read the true manometer levels. Plugs atop the manometers are also not recommended as they can induce unwanted pressure above the water level (i.e. deviation from atmospheric), and evaporation can still occur.

The accuracy of the OPPD manometers, and replica setup for evaporation measurement, is limited to the 1mm changes in the height of the meniscus that can be seen with the human eye. The diameter of the manometer tubes was 9.525mm, resulting in an accuracy of the changes in water content equal to 0.07126 grams. The Arizona State University soil lab used for the OPPD testing in this study averaged 0.07126 grams of water loss to evaporation every 10 days, resulting in an average water loss per day to evaporation of 0.007126 grams.

6.3 In-situ Matric Suction and Osmotic Suction Determination Using WP4-C and OPPD

The previously presented procedure for testing the natural soil response to partial wetting allows for the direct measurement of the in-situ matric suction $(u_a - u_w)$ of the soil (Steps 1-14). Using the measured total suction (ψ_{total}) from the WP4-C and the measured matric suction from the OPPD, a good estimate of the osmotic suction of the soil can easily be calculated by modifying Equation 1.

$$\psi_{osmotic} = \psi_{total} - (u_a - u_w) \tag{3}$$

The suction measurement portion of the procedure alone provides very crucial information that can be useful in various unsaturated soil applications. An understanding of the osmotic suction at a given site allows for the correction of any total suction measurements to matric suction, which plays the main role in the mechanical behavior of the soil. It is important to remember that, due to the limitations of the HAEV ceramic disks, the in-situ matric suction using the previously presented procedure can only be determined directly in the OPPD if matric suction is less than 1500 kPa.

Table 8 presents a summary of the results for the directly measured in-situ matric suction, by the previously proposed method, of the 14 studied undisturbed samples which exhibited an in-situ total suction less than 1500 kPa. The back-calculated osmotic suctions are also presented in the table.

The initial (in-situ) matric suction value is a key component in suction-based partial wetting heave estimation methods. As it pertains to this study, the initial matric suction of the soil is necessary in the use of the Surrogate Path Method (Singhal, 2010), discussed later. For the 7 remaining specimens, of the 21 total, for which the initial matric suction could not be directly determined from the previously proposed OPPD method, the average calculated osmotic suction value from each site was be used, together with WP4-C total suction values, to estimate the initial matric suction.

Table 8: Summary of Measured In-situ Matric Suction and Osmotic Suction

ID	WP4-C Measured Total Suction (kPa)	OPPD Measured Matric Suction (kPa)	Back-Calculated Osmotic Suction (kPa)
D-4	834	700	134
D-5	1450	1260	190
D-7	1075	900	175
SA-1	390	300	90
SA-2	526	480	46
SA-3	1100	900	200
SA-4	1100	1020	80
SA-5	873	500	373
SA-6	390	300	90
SA-7	458	400	58
SA-8	726	650	76
SA-9	1553	1420	133
SA-10	514	440	74
SA-11	915	750	165

The average osmotic suction from the Denver and San Antonio specimens was found to be 166 kPa and 126 kPa, respectively. The overall values ranged from a minimum of 46 kPa to a maximum of 373 kPa, with the lower value occurring likely as a result of errors associated with measurement of total suction in low suction range (estimated by Meter to be plus or minus 50 kPa for the WP4-C when measured total suction is less than 100 kPa). This range of osmotic suction is in reasonable agreement with previously published work by Krahn and Fredlund (Krahn & Frendlund, 1972), in which a glacial till was measured to have an osmotic suction value of approximately 300 kPa (Figure 3), and with Walsh et al., 2009, where osmotic suction for Denver front range clay profiles was found to be about 200 kPa. It is recommended that when estimating an initial matric suction of a soil from its measured total suction, for the previously presented OPPD procedure, a value of 300 kPa be used if previous site specific osmotic suction data is not available. As previously discussed, since it is easier for a soil to dry than it is for it to absorb water, an underestimation of the initial matric suction (i.e. initiating the soil on a wetting path) will cause less hysteresis effects on the specimen, resulting in a more accurate measurement of the in-situ matric suction value.

6.4 Comparison of Full Wetting OPPD Strain to ASTM D4546 Response to Wetting Test

Theoretically, when identical specimens are fully wetted by inundation under the same net normal stress loading conditions, equivalent strains should occur. The rate at which the suction is lowered to zero, either by immediate inundation (ASTM D4546) or methodically by the OPPD, should not have a significant effect on the magnitude of volume change, provided equilibrium is achieved, and therefore the D4546 and OPPD fully wetted tests should return similar swell strains for companion specimens. OPPD specimens that are nearly fully wetted (e.g. 50 kPa matric suction) should return swell strains nearly the same as fully-wetted D4546 tests on companion specimens. However, in dealing with natural clays, true companion specimens are essentially impossible to obtain.

Although the individual undisturbed specimens for response to wetting test and the OPPD testing were extracted from the same sample (e.g. no greater than 1.0 inch vertically apart), several of the specimens exhibited noticeable discrepancies between the two (OPPD and ASTM D-4546) fully wetted strains (Table 9).

Table 9: Summary of Fully Wetted Strain Results for OPPD and ASTM D4546 Testing

ID	$(\varepsilon_{ob})_{D4546}$ (%)	$(\varepsilon_{ob})_{OPPD}$ (%)	$\lfloor(u_a - u_w)_f\rfloor_{OPPD}$ (kPa)
D-1	1.45	0.77	50
D-2	-0.99	0.55	100
D-3	1.86	1.30	50
D-4	-0.72	0.42	50
D-5	1.12	0.83	50
D-6	-0.53	-1.64	25
D-7	0.20	0.3	50
D-8	1.19	1.38	100
D-9	1.22	1.32	100
D-10	0.28	0.95	50
SA-1	2.17	0.93	50
SA-2	1.85	0.94	5
SA-3	1.42	1.75	5
SA-4	1.48	1.82	0
SA-5	0.55	0.53	50
SA-6	0.70	0.50	0
SA-7	0.42	0.83	0
SA-8	1.00	0.52	0
SA-9	3.53	1.63	50
SA-10	0.62	0.80	0
SA-11	1.83	1.37	50

Of the 21 specimens tested, 5 of the specimens exhibited a difference in strain that exceeded 1.0%. In some cases (e.g. Specimens D-2 and D-4) a specimen would compress when fully wetted whereas the "companion" specimen would swell. It is apparent from Table 9 that there can be significant heterogeneity within a given sample tube. Differences in test apparatus and duration of tests were also reviewed as possible contributors to differences between the D4546 and OPPD test results: (1) the weight of the load plate and shaft was not included in applied stress for OPPD tests, however, the additional stress minimal (4.5 kPa), and (2) the possibility of the buildup of piston friction could have affected the swell strain of the OPPD tested specimens as the duration of the tests lasted several weeks, although the presence of excess piston friction was not detected during this study. Thus, sample variability is believed to be the primary reason for differences between fully wetted D4546 test results and essentially fully wetted OPPD test results.

The results in Table 9 depict one of the major complications when studying undisturbed soils. The repeatability of the same result of a given test is difficult to obtain because of the heterogeneity of a natural soil deposit (i.e. very difficult to obtain identical companion samples). Therefore, the concept of identical or "companion" specimens was not used in this study of natural clays.

6.5 Soil Water Characteristic Curves for Natural Clay Soil

SWCC plots were developed for each of the undisturbed soils tested for partial wetting response using the OPPD (refer to Appendix B). Figure 28 below illustrates an SWCC plot for one of the OPPD tested specimens. The complete set of the developed SWCCs is presented in Appendix C.

The degree of saturation is the moisture variable used to model the SWCC since the specimens were highly susceptible to volume change. It is important to note that the SWCC data is presented on the plot as the directly measured points alone. No curve was fitted to the data, and there is currently no accepted method for curve fitting SWCC data located within the boundary wetting and drying curves (i.e. scanning curves). The numerous SWCC curve fitting techniques in current literature were developed to model the SWCC data as a boundary curve (e.g. initial drying and wetting, generally from a slurry). The wPI (Zapata et al., 2000) SWCC for each of the tested specimens was plotted along with the directly measured data points so that a visual comparison to an estimated SWCC curve can be made.

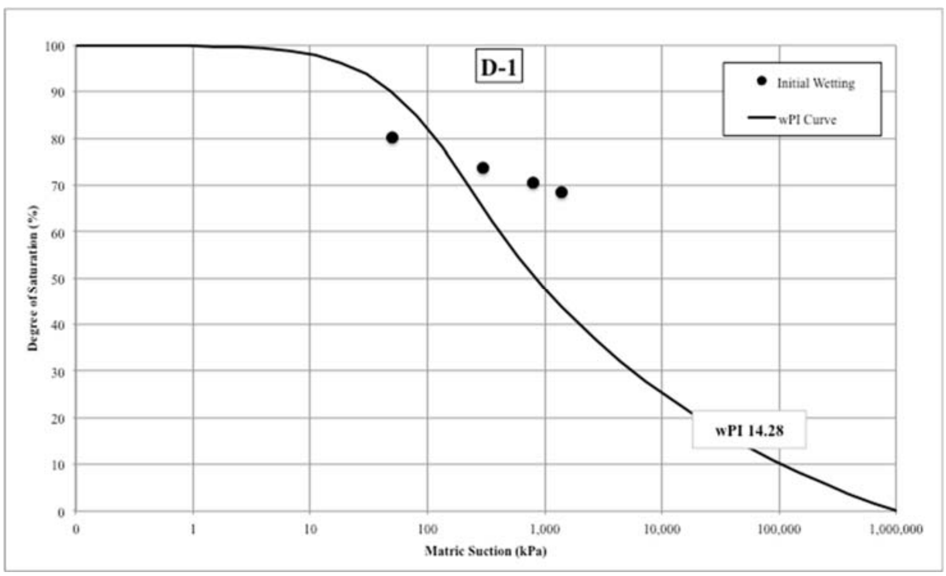


Figure 28: SWCC Data for D-1

The effect of hysteresis on the shape of the natural clay SWCC and the suction compression index was also studied by forcing 4 of the tested specimens through a drying cycle and secondary wetting cycle after the initial wetting phase of the test, as illustrated in Figure 29 below.

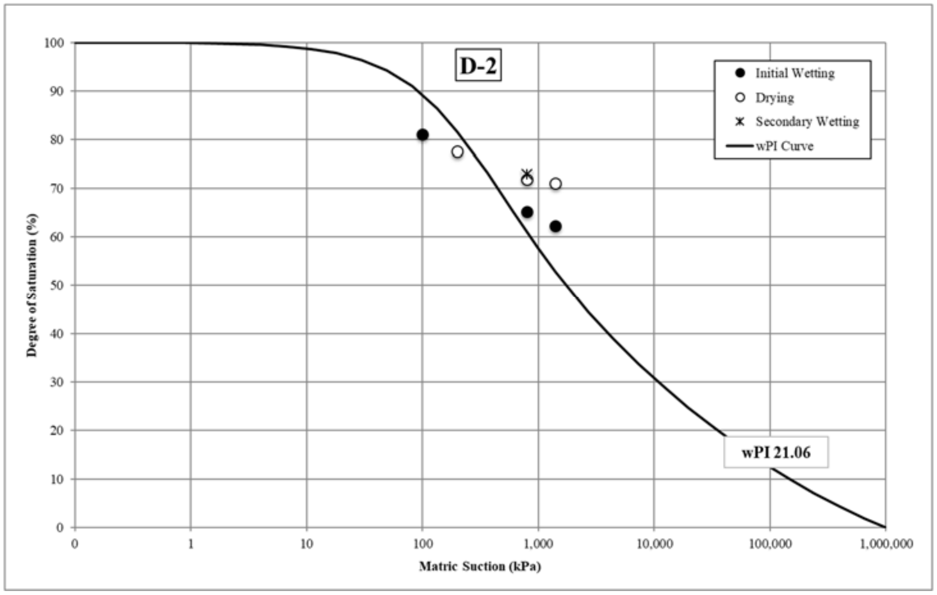


Figure 29: SWCC Data for D-2

From a visual examination of the entire set of developed degree of saturation versus log of matric suction plots (Appendix B), several key observations were made:

- The shape of the natural (undisturbed field specimen) curves, compared to the typically estimated SWCCs, was relatively flatter within the transition zone. This trend was expected due to the understanding that natural soils have experienced numerous wetting/drying cycles over time, so they will not experience as drastic volume changes (and changes in saturation) as suction changes compared to a recompacted specimen. In other words, the shape of the developed natural SWCCs highly resemble scanning curves.
- The determination of the air-entry value (AEV), residual water content value, etc. is not as straight forward when analyzing the developed natural specimen SWCCs. Since it is understood that the natural specimens exhibit a scanning curve path, the breaks between the zones of the SWCC (i.e. saturated zone, transition zone, residual

zone) are not as visually apparent. A scanning curve may intersect and follow the boundary curves as the suction is minimized or maximized. Therefore, it is not recommended to determine the AEV or residual values from the natural, undisturbed specimen SWCC as they can differ significantly from SWCC developed from remolded slurry samples.

- The shapes of the natural drying and secondary wetting SWCCs were similar to those expected by hysteresis scanning curves (Figure 8). However, based on the limited testing of this study, significant differences between the wetting and drying suction compression indices were observed (Figure 83, Figure 85, and Figure 89). The drying path suction compression index, determined under field-appropriate net normal stress, was, on average, 3.5 times greater than the initial wetting path suction compression index. It may be expected that the ratio of the drying suction compression index to the wetting increases with sample depth. For a given initial void ratio, significant decreases in suction are necessary for a deeper soil to exert an uplift force and begin to swell, compared to the relatively small increases in suction that will cause a deeper soil to shrink with the aid of the overburden pressure. On the other hand, the initial void ratio of a specimen at depth would be expected to be lower than the initial void ratio of a shallower specimen of the same soil; a deeper soil specimen at higher confinement would be expected to have a lower initial void ratio and therefore less susceptible to shrinkage upon drying. Thus, more research is required on the comparison of suction compression index (drying) and suction swell index (wetting) because it is expected that drying and wetting suction compression index values, with enough cycles of loading, would begin to approach the same value. This is because field observations support more or less the same suction compression and suction swell index values because cracks in structures tend to open and close about the same amount seasonally after a post-construction pseudo-equilibrium suction state is reached. The observed field results could be due to wetting/drying typically occurring only at shallow depths, where differences between wetting and drying suction compression index values would be expected to be minimal compared to greater depths within the profile.

- The average ratio of the initial wetting suction compression index to the secondary wetting compression index was 1.18. Because the field specimens have undergone numerous cycles of wetting and drying in-situ, it might be expected that the initial wetting and second wetting compression index values would be the same, assuming soil suction values were varied in the laboratory within a range corresponding to field conditions. However, the somewhat higher value of the first wetting suction compression index may be associated with sample disturbance, the effects of which are reduced after multiple loading cycles. To the extent that laboratory suction values were varied over a wider range than those typically experience in the field, hysteresis effects could also account for some of the difference in first and second cycle wetting compression index values. Some remaining sample disturbance could also be a factor.

6.6 Directly Measured Suction Compression Indices of a Natural Clay Soils

From the data obtained by the OPPD testing, the void ratio versus log matric suction relationship was developed for each of the 21 undisturbed specimens. The suction compression index for each specimen was then graphically determined by calculating the slope of a best-fit line to the data within the transition zone. The following table presents the directly measured suction compression index for each specimen. Because these suction compression index values were determined from wetting tests, the term "suction swell index" might be more appropriate. However, suction compression index is the more commonly used term whether determined by wetting or drying.

Table 10: OPPD Directly Measured Suction Compression Indices from Wetting Tests

ID	Suction compression Index*	ID	Suction compression Index*
D-1	0.0068	SA-1	0.0317
D-2	0.0077	SA-2	0.0244
D-3	0.0083	SA-3	0.0320
D-4	0.0088	SA-4	0.0283
D-5	0.0146	SA-5	0.0193
D-6	0.0101	SA-6	0.0124
D-7	0.0067	SA-7	0.0264
D-8	0.0302	SA-8	0.0110
D-9	0.0213	SA-9	0.0211
D-10	0.0128	SA-10	0.0159
		SA-11	0.0298

*Expressed in terms of void ratio

The directly measured suction compression indices will be compared to those obtained by currently accepted estimation methods, in the following section. It is important to note that the suction compression index is expressed in terms of changes in strain or changes in void ratio (Figure 11) caused by a log cycle change in suction. Both volumetric strain and void ratio forms of suction compression index are used in this thesis, dependent on the method at hand.

7 COMPARISON OF OPPD DIRECTLY MEASURED SUCTION COMPRESSION INDICES TO ESTIMATED METHODS

7.1 Perko, Thompson, Nelson (2000) Method for Estimating Suction Compression Index

Perko, Thompson, and Nelson (2000) used CLOD tests to improve on McKeen's (1992) empirical relationship for the suction compression index (γ_h) expressed as:

$$\gamma_h = -0.02673\left(\frac{\Delta h}{\Delta w}\right) - 0.38704 \tag{4}$$

Where Δh is the change in suction associated with the original condition of the sample to the shrinkage limit and Δw is the associated change in water content. CLOD tests were conducted on 69 undisturbed clay soil samples from Denver, CO (Figure 30). It is important to note, although the CLOD tests were performed on undisturbed specimens, the samples were dried in an unconfined state, and therefore not fully representative of field conditions.

The empirical relationship developed from the CLOD tests (shown on Figure 30) was further improved to account for the amount of sand in the soil, similar to that of a "rock correction" in density determination or swell test results. The modified Perko, Thompson, and Nelson (2000) equation for the suction compression index is expressed as:

$$\gamma_h = -\frac{10}{3} PL^2 \left(\frac{e+F}{e+1}\right) \tag{5}$$

Where PL is the plastic limit, e is the void ratio, and F is the percent fines (%-#200). It is important to note that the modified equation is representative of a suction compression index expressed in terms of volumetric strain.

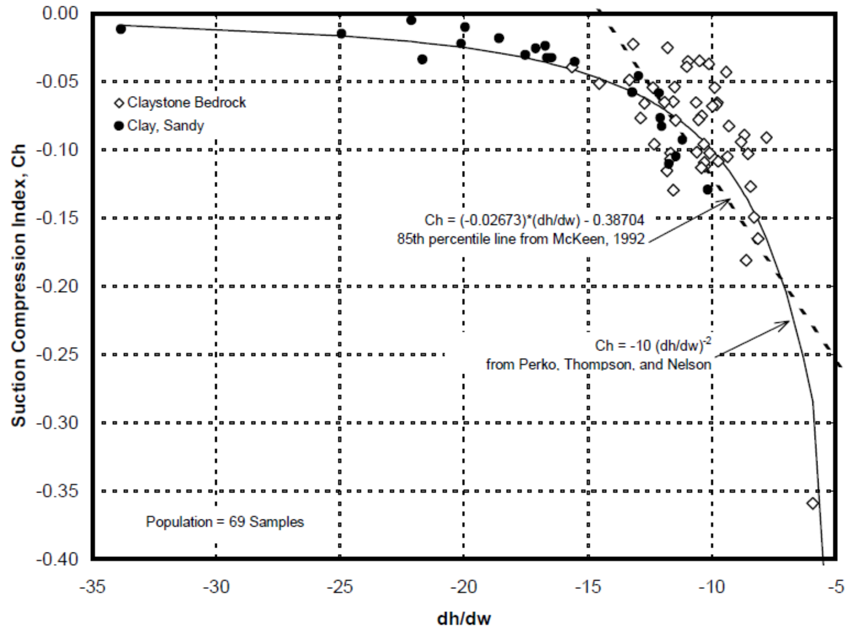


Figure 30: Suction Compression Index vs. Ratio of Change in Suction over change in water content (Perko, Thompson, & Nelson, 2000)

7.2 Perko, Thompson, Nelson (2000) Comparison to Directly Measured Values

The previously discussed Perko, Thompson, and Nelson (2000) method for estimating the suction compression index (strain-based) was compared to the directly measured values of the 21 studied undisturbed samples (Figure 31). From the results, it is observed that the Perko, Thompson, and Nelson (2000) method tends to overestimate, with no apparent trend, the suction compression indices compared to those which were directly measured. It is important to recognize the COLE/CLOD test used in the Perko, et al. (2000) method is a drying test performed under no net normal stress (unconfined). The value of suction swell index and suction compression index would be expected to be more or less the same under conditions of no confining stress, yet the Perko, et al (2000) method results in significantly higher suction compression index values compared to those obtained from the suction-controlled apparatus where field confining stress was applied to the specimen.

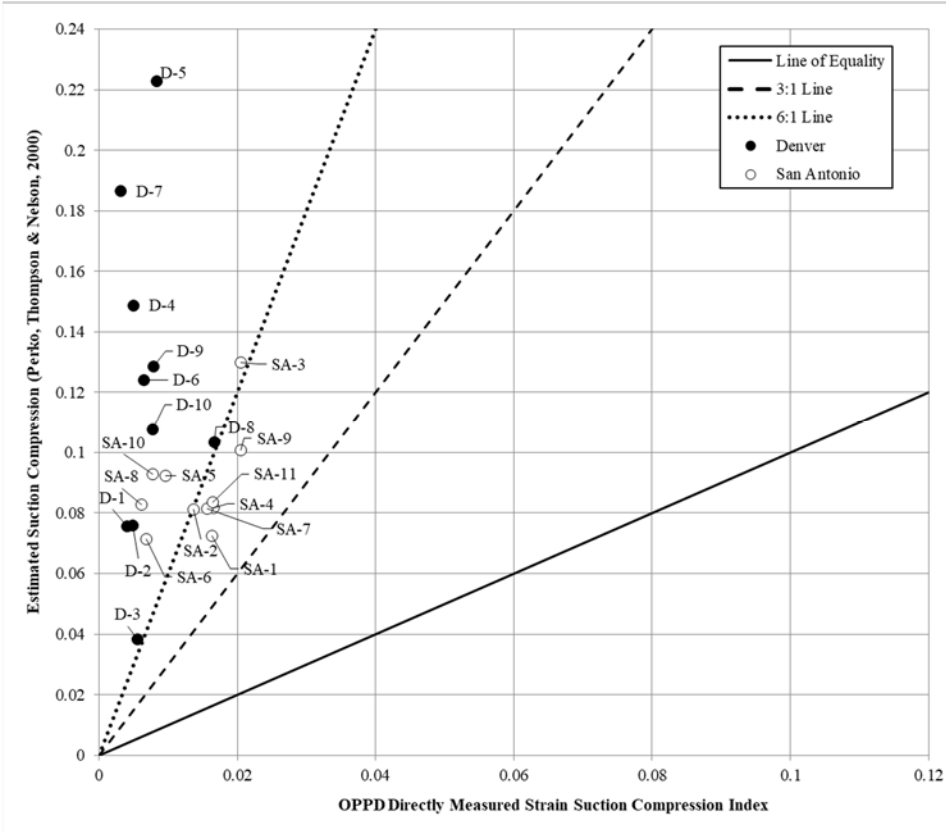


Figure 31: Estimated Suction Compression Index by Perko, Thompson, Nelson (2000) vs. Directly Measured Values

7.3 PTI Method for Estimating Suction Compression Index

The PTI method (2008) is a widely accepted method by practitioners and public agencies for estimation of potential soil movement (shrink/swell) of a soil profile. The method encompasses four different approaches to determining the suction compression index that is used for both wetting and drying paths (Methods A through D), as summarized below:

A. Covar and Lytton (2001) Method

B. Expansion Index Method (ASTM D4829)

C. Consolidation – Swell Pressure Test Method (ASTM D4546 Method C)

D. Overburden Pressure Swell Test Method (ASTM D4546 Method B)

Methods A and D were analyzed as part of this study as they are considered to be the most feasible (and commonly-used) approach for practicing engineers.

VOLFLO, developed by Geostructural Took Kit, Inc. (2012), is a commercially available program of the Post-Tensioning Institute, and widely used by practicing geotechnical engineers. The program automates the PTI (2008) procedure, with the necessary input values, and was used in this study for the determination of estimated suction compression indices in terms of volumetric strain.

7.3.1. Method A: Covar and Lytton (2001)

. Method A uses the previously described work of Covar and Lytton (2001), in which the mean suction compression index (γ_h) is expressed as:

$$\gamma_h = \gamma_0(\% fc)/100 \tag{6}$$

Where *%fc* is percentage of soil passing the No. 200 sieve that is finer than 2 microns and γ_0 is the suction compression index for 100% fine clay content. The percent fine clay can be further defined as:

$$\% fc = \frac{(\% - 2\mu)}{(\% - \#200)} x100 \tag{7}$$

The suction compression index for 100% fine clay content (γ_0) is determined by the following charts which are based on the mineralogy and index properties of the soil.

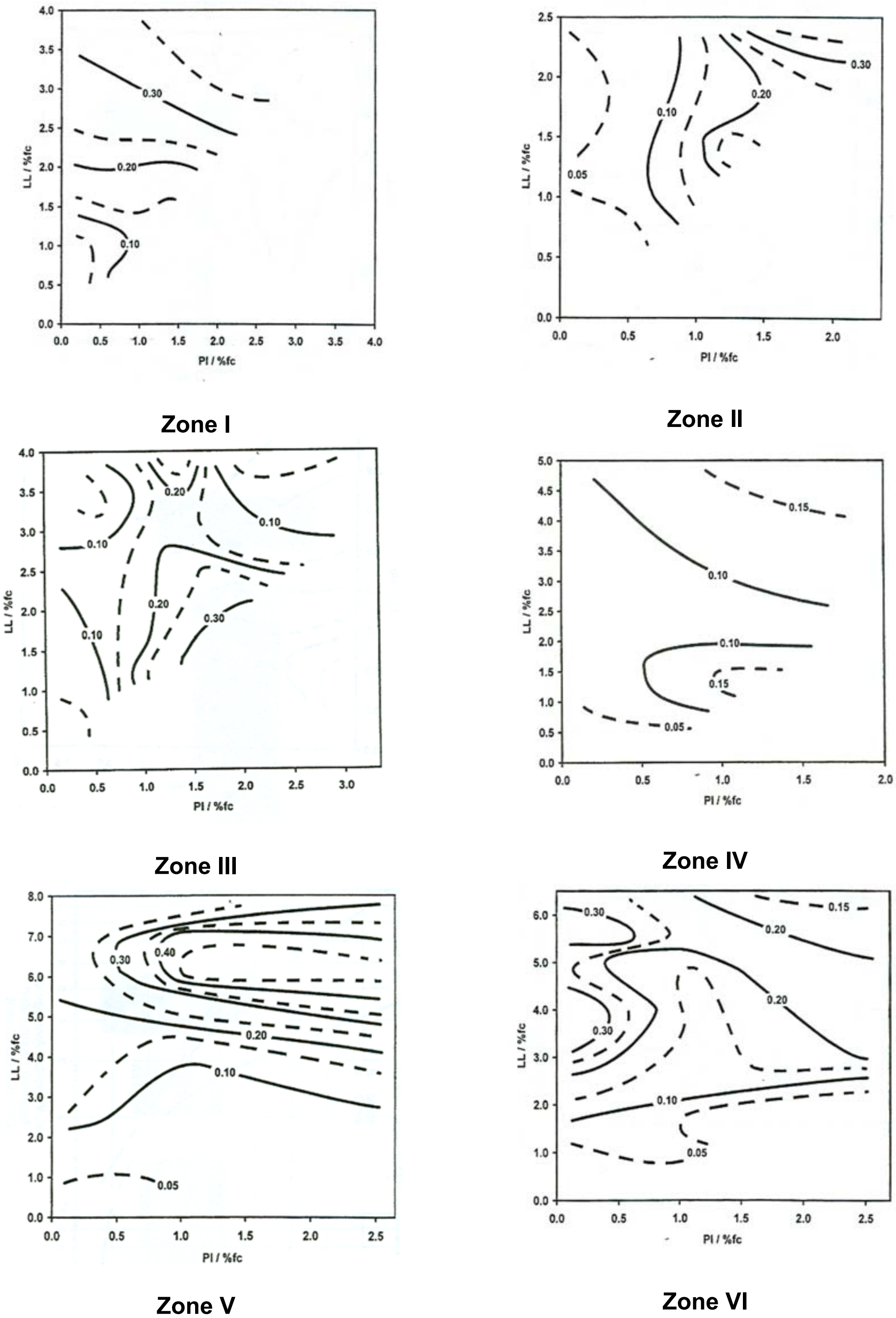


Figure 32: Suction Compression Index Based on Mineralogical Classification and Soil Index Properties (Covar & Lytton, 2001)

Each zone is determined by the following mineralogy chart, which was modified from Holtz and Kovacs (1981).

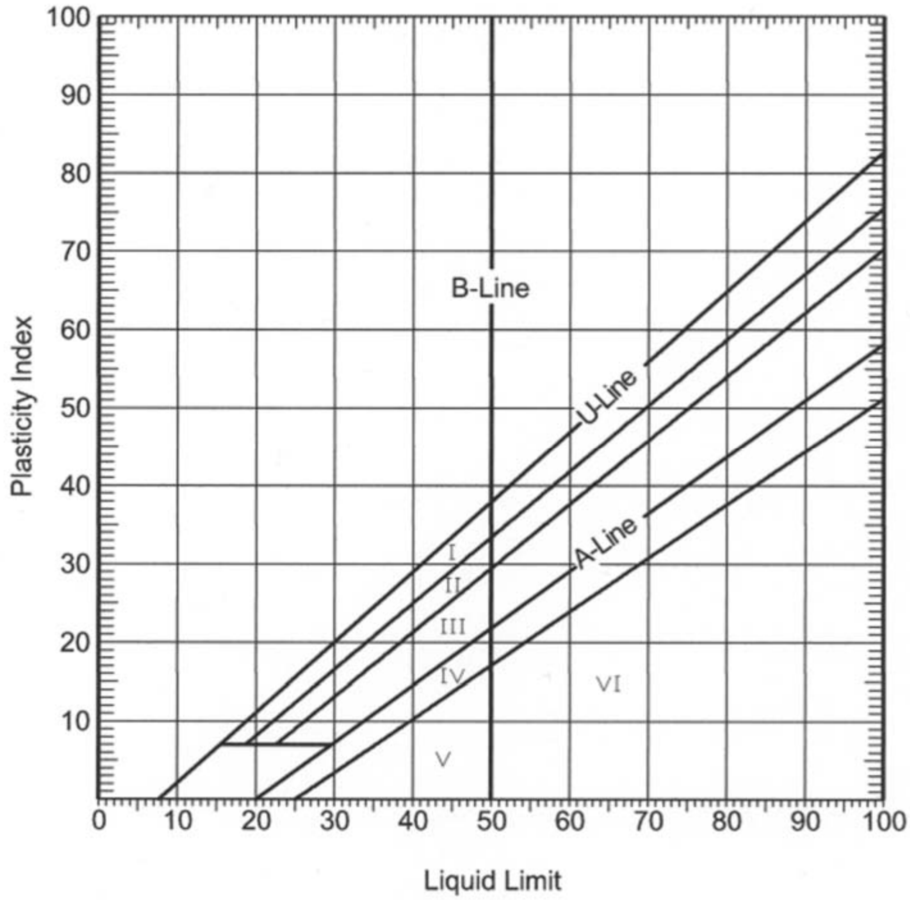


Figure 33: Soil Mineralogical Classification Based on Atterberg Limits (PTI, 2008)

Furthermore, the mean suction compression index is then modified to account for whether the soil is swelling or shrinking. The relationship between the mean, swelling, and shrinking suction compression indices are defined by the following equation and the accompanied plot.

$$(\gamma_h)_{swell} = \gamma_h e^{\gamma_h} \tag{8}$$

$$(\gamma_h)_{shrink} = \gamma_h e^{-\gamma_h} \tag{9}$$

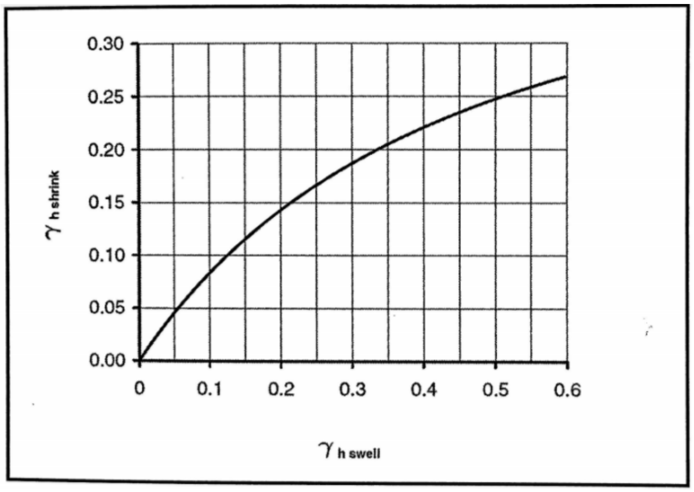


Figure 34: Suction Compression Index Relationship Between Shrinkage and Swelling (PTI, 2008)

It is important to note that from this relationship the swelling suction compression index will also be determined to be greater than the shrinking suction compression index, with differences increasing with increasing swell potential. In contrast, the author would expect the suction compression and suction swell index values to be nearly the same (for low confinement), and perhaps the suction compression index may be somewhat higher than the suction swell index (for greater confinement). However, the shrinkage and swell suction compression index values may also approach the same value at depth because higher net normal stress would tend to compress the soil, decrease the void ratio, and generally make the soil less susceptible to shrinkage.

7.3.2. Method D: Overburden Swell Test

Method D uses the results of the ASTM D4546 Method B, which measures the full wetting strain (ε_{ob}) of a sample under its in-situ overburden stress (σ_{ob}). The suction compression index (γ_h) is in terms of strain and is estimated by:

$$\gamma_h = \frac{\varepsilon_{ob}}{\left(1.7 - \log_{10}\sigma_{ob}\right)} \tag{10}$$

where σ_{ob} is in psi.

There are several underlying assumptions regarding stress state and changes in stress state within the D4546 test that are incorporated into equation 10 (Lytton, 1994; Covar and Lytton, 2001).

7.4 PTI (2008) Method A Comparison to Directly Measured Suction Compression Indices

As previously discussed in Section 7.3.1, The PTI Method A for estimating the suction compression index, follows the procedure laid out by Covar and Lytton (2001). The necessary parameters needed in the estimation are the soil index properties, specifically the Atterberg Limits (LL, PL, & PI), percent fines (%-#200), and percent clay (%-2μ).

The percent clay was only determined for 3 of the 21 undisturbed soil samples used in this study. The lack of clay content values, determined from the hydrometer lab test (ASTM D422), is a common scenario in geotechnical engineering practice. Due to project budgets associated with residential/commercial development, and the laboratory time and effort necessary for a proper hydrometer test, the practicing geotechnical engineer often estimates the clay content for a given soil. However, as discussed in the following section, caution must be taken when estimating the clay content value for implementation into the PTI (2008) Method A, as the suction compression index obtained is sensitive to the clay content, whether the value was directly measured or estimated.

7.4.1. Sensitivity of the PTI (2008) Method A Associated with Clay Content

The following study illustrates the sensitivity of the estimated mean suction compression index as the clay content for a given soil is increased. The estimated mean suction compression index for each of the 21 undisturbed samples used in this study was calculated as the clay content was increased, in 10% increments, from a minimum value to one equivalent with the total fines content (i.e. %fc=100%).

The minimum value was controlled by the limitations of the VOLFLO computer program, which occurs when it cannot interpolate within the gaps on the mineralogical zone charts (Figure 33). This limitation is not significant to this sensitivity study, because it only occurs in typically unrealistic scenarios where a high PI value (ex. 60) is inputted with a low clay content value (ex. 5%), or vice versa.

The VOLFLO program was used to illustrate the relationship between clay content and mean suction compression index for the 21 studied soils. The following figures illustrate the relationship of the suction compression index to the percent fine clay (%fc) from a minimum value to 100% for the Denver and San Antonio samples, respectively. The mineralogy zone determined in the PTI (2008) Method A, is expressed as a separate line/marker, so that any trends (or lack thereof) within or between mineralogy zones could be visualized.

The studied San Antonio samples all fell within Zone 1 (Figure 36). It is apparent from the Figures 32 and 33 that the estimated mean suction compression index value, via the PTI (2008) Method A, is highly sensitive with respect to the clay content and/or percent fine clay. Significant sensitivity is observed in all three of the mineralogical zones that the studied soils fell within (e.g. Zones 1, 2, and 3)

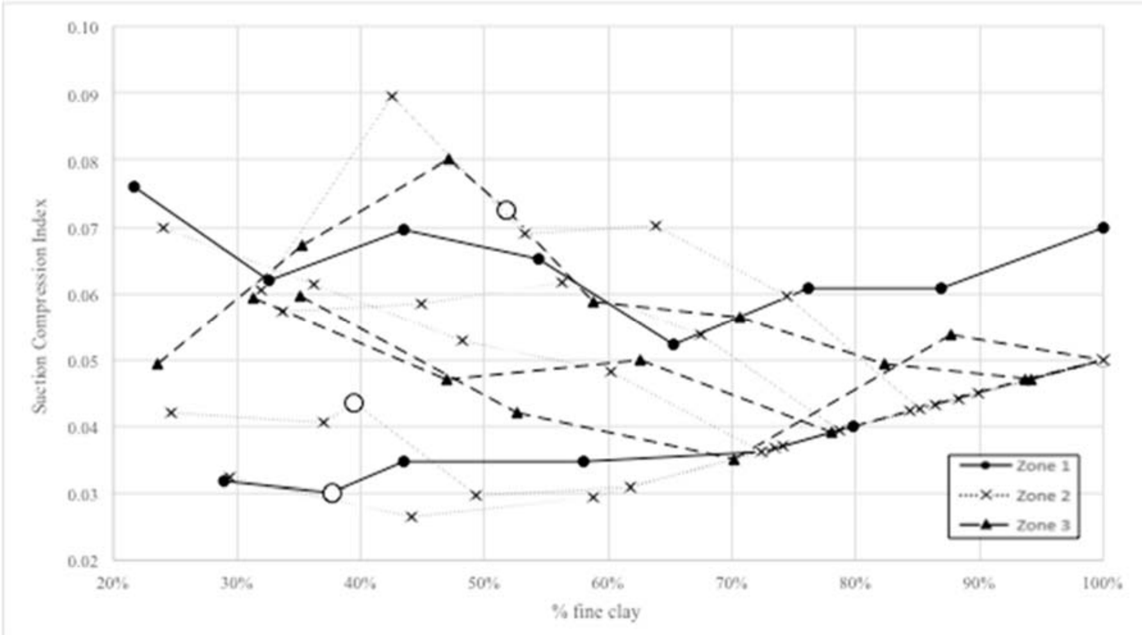


Figure 35: Estimated Mean Suction Compression Index by PTI (2008) Method A vs. %fc for Denver Specimens

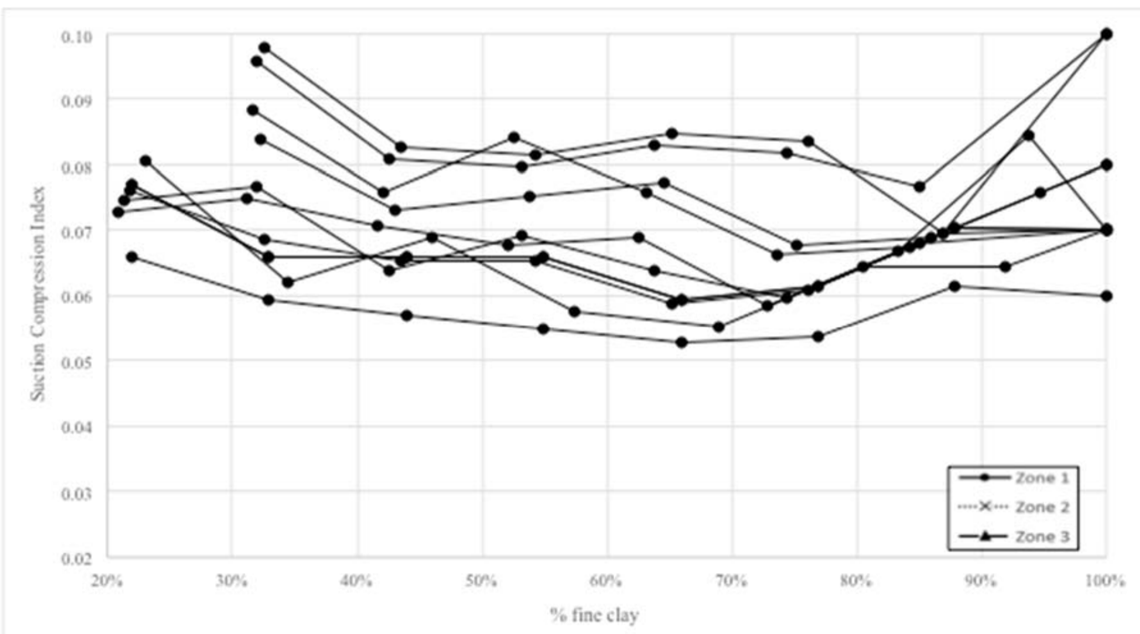


Figure 36: Estimated Mean Suction Compression Index by PTI (2008) Method A vs. %fc for San Antonio Specimens

For 16 of the 21 soils a higher mean suction compression index result was obtained at a %fc value less than 100% compared to lower percent clay values. The estimated mean suction compression indices for the samples with the hydrometer measured clay content values are also plotted on Figure 35 as the larger white points. However, the knowledge of the true clay content does not necessarily increase the accuracy of the estimated mean suction compression index. Due to sensitivity associated with the clay content of the soil, the maximum and minimum suction compression indices with regards to clay content for the 21 samples were compared to the directly measured values. Since the directly measured OPPD suction compression indices were from partial wetting swell tests, the estimated swelling suction compression indices (strain-based) were determined from the previously computed maximum and minimum mean values for comparisons here.

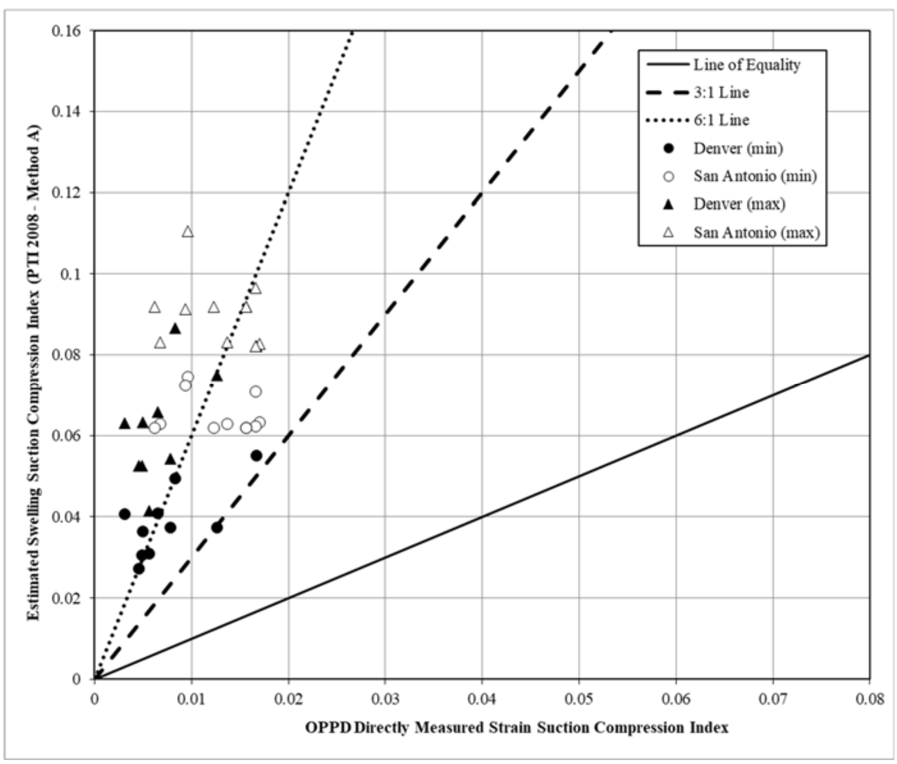


Figure 37: Min. and Max. Estimated Swelling Suction Compression Index by PTI (2008) Method A, with Regards to Clay Content vs. OPPD Directly Measured Values

The results indicate that the PTI (2008) Method A will tend to overestimate the swelling suction compression index for a given soil, no matter the value of the percent fine clay used in the calculation. If the practicing geotechnical engineer errs on the side of caution by using the maximum estimated swelling suction compression index, in relation to percent fine clay, the results may lead to a highly over conservative design, with an average calculated ratio of the estimated to measured suction compression index equal to 9.05. The ratio of the minimum estimated suction compression index to the actual measured value was calculated to be 5.97, indicating that the least conservative approach to this scenario still will overestimate the suction compression index by approximately 6 times, on average. Additional direct testing for suction compression index of undisturbed clay soils, for example using an OPPD, would be required to further refine the estimated over-conservatism of the PTI (2008) Method A.

7.5 PTI (2008) Method D Comparison to Directly Measured Suction Compression Indices

The OPPD directly measured suction compression indices were compared to those obtained using the PTI (2008) Method D, which incorporates the full wetting swell of the specimen under its in-situ net normal stress. The following table summarizes the Method D estimated suction compression index using the D4546 full wetted strain. A generalized, overall average, suction compression index is also presented on the table and was calculated by:

$$\frac{(\varepsilon_{ob})_{D4546}}{\Delta pF} \tag{11}$$

where ΔpF is change in suction from the estimated initial matric suction of the specimen to the final suction at saturation (i.e. zero matric=osmotic).

Table 11: PTI (2008) Method D Computation Summary

ID	Depth (ft)	σ_{ob}		Initial Total Suction		Estimated Osmotic Suction (Table 8)		ΔpF	$(\varepsilon_{ob})_{D4546}$ (%)	γ_h * (PTI, 2008 Method D) (Eq. 10)	$\frac{(\varepsilon_{ob})_{D4546}}{\Delta pF}$
		psi	kPa	pF	kPa	pF	kPa				
D-1	5.5	4.69	32.313	4.56	3558	3.23	166	1.33	1.45	0.0141	0.0109
D-3	5.5	4.60	31.644	4.75	5511	3.23	166	1.52	1.86	0.0179	0.0122
D-5	17.5	14.03	96.604	4.17	1450	3.23	166	0.94	1.12	0.0203	0.0119
D-7	5.5	3.99	27.437	4.04	1075	3.23	166	0.81	0.2	0.0018	0.0025
D-8	10.5	8.09	55.711	4.155	1400	3.23	166	0.93	1.19	0.0150	0.0129
D-9	15.5	12.86	88.526	4.365	2271	3.23	166	1.14	1.22	0.0207	0.0107
D-10	20.5	17.21	118.47	4.35	2194	3.23	166	1.12	0.28	0.0060	0.0025
SA-1	5.5	4.49	30.879	3.6	390	3.11	126	0.49	2.17	0.0207	0.0442
SA-2	10.5	8.38	57.647	3.73	526	3.11	126	0.62	1.85	0.0238	0.0298
SA-3	15.5	12.20	83.985	4.05	1100	3.11	126	0.94	1.42	0.0231	0.0151
SA-4	10.5	8.63	59.415	4.05	1100	3.11	126	0.94	1.48	0.0194	0.0157
SA-5	15.5	12.03	82.837	3.95	873	3.11	126	0.84	0.55	0.0089	0.0065
SA-6	5.5	4.36	30.018	3.6	390	3.11	126	0.49	0.70	0.0066	0.0143
SA-7	10.5	8.60	59.176	3.67	458	3.11	126	0.56	0.42	0.0055	0.0075
SA-8	15.5	12.01	82.694	3.87	726	3.11	126	0.76	1.00	0.0161	0.0131
SA-9	5.5	4.61	31.739	4.2	1553	3.11	126	1.09	3.53	0.0341	0.0324
SA-10	10.5	8.77	60.371	3.72	514	3.11	126	0.61	0.62	0.0082	0.0102
SA-11	15.5	12.17	83.746	3.97	915	3.11	126	0.86	1.83	0.0298	0.0213

*Expressed in terms of strain

Note that specimens D-2, D-4, and D-6 were not included in this comparison as the specimens compressed at full wetting during the ASTM D4546 Response to Wetting Test (Table 5). The generalized suction compression indices (Eq. 11) closely agree with the PTI (2008) Method D estimated values. A similar equation, like the PTI (2008) Method D, which estimates the suction compression index with a set denominator to represent the suction change, is used in the Australian Standard for Residential Slabs & Footings (AS2870, 2011). However, using the true change in suction of the specimen, together with the D4546 swell strain at field stress level, as in generalized suction compression index (Eq. 11), should be more representative of the true soil response.

The PTI (2008) Method D estimated suction compression indices were also compared to the directly measured values. Since several of the OPPD tested specimens used to directly measure the suction compression index were determined not to be acceptable companion samples with the D4546 Response to Wetting specimens, the final OPPD measured strain was used as the full wetting strain variable in the PTI (2008) Method D empirical equation. Figure 38 illustrates the comparisons between the estimated and the directly measured suction compression indices (strain-based). The results indicate that PTI (2008) Method D (Figure 38) tends to more closely estimate the directly measured suction compression index compared to Method A (Figure 37).

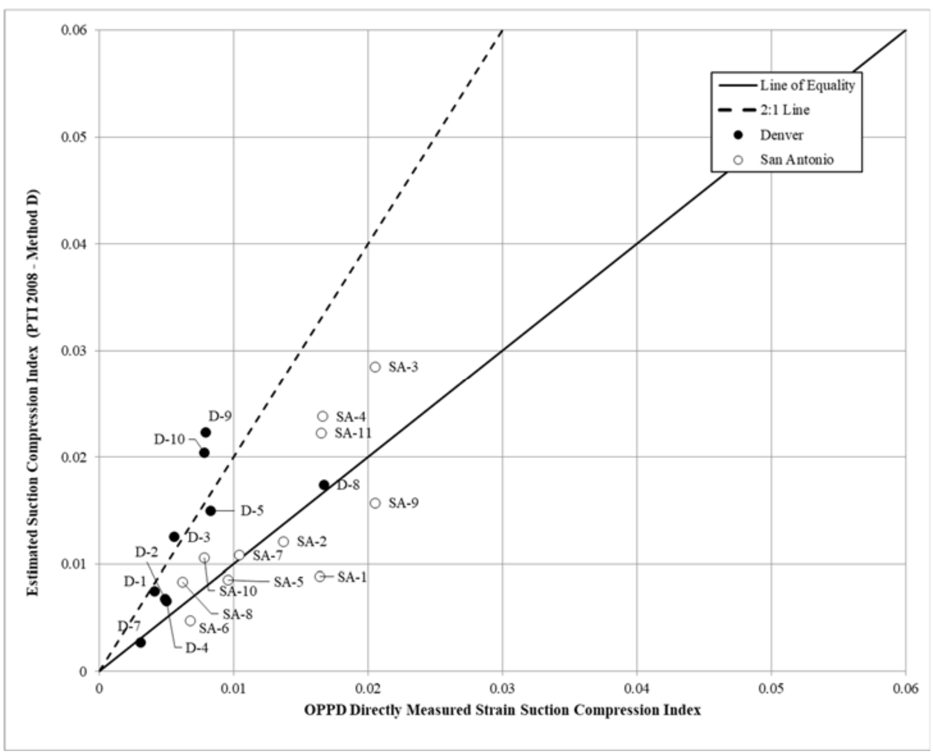


Figure 38: Comparison of PTI (2008) Method D Estimated Suction Compression Indices vs. OPPD Directly Measured Values

7.6 Evaluation of Suction Compression Index Estimation Methods

All three of the explored methods for estimating the suction compression index lead to over conservative results with respect to the OPPD directly measured values. The PTI (2008) Method D showed the closest agreement with the directly measured values. The use of the laboratory measured full wetting strain within the PTI (2008) method D equation aids in the accuracy of the suction compression index estimate. The Perko et al. (2000) and the PTI (2008) Method A, derived from unconfined shrinkage tests, rely on index proprieties alone, without direct testing of response to wetting behavior of the undisturbed soil specimen.

7.7 Discussion of Swelling vs. Shrinking Suction Compression Index

The previously evaluated methods for the suction compression index were derived from unconfined shrinkage, COLE, or CLOD tests. The swelling suction compression index is then determined mathematically from the measured or estimated value (PTI, 2008), and was shown to be greater than the shrinking suction compression index (Figure 34). However, limited data from this study shows an opposite trend in that the measured shrinking suction compression index values were lower than the measured suction swell index values. Although not a focus of this study, 4 of the OPPD intact specimens were forced through a drying and secondary wetting cycle following the initial wetting cycle. A second drying cycle was also conducted on Specimen D-4. The effects from the multiple wetting/drying cycles (i.e. hysteresis) on the suction compression index, in terms of changes in void ratio, are summarized in the table below.

Table 12: Hysteresis Effects on Suction Compression Index

ID	σ_{ob} (kPa)	Wetting/Drying Cycle	Initial Suction (kPa)	Final Suction (kPa)	Suction Compression Index*
D-2	53.49	(1st) $(\gamma_h)_{swell}$	1400	800	0.0077
		(2nd) $(\gamma_h)_{shrink}$	800	1400	0.0259
		(3rd) $(\gamma_h)_{swell}$	1400	800	0.0078
D-4	79.95	(1st) $(\gamma_h)_{swell}$	700	100	0.0088
		(2nd) $(\gamma_h)_{shrink}$	800	1400	0.0499
		(3rd) $(\gamma_h)_{swell}$	800	100	0.0087
		(4th) $(\gamma_h)_{shrink}$	300	1400	0.0153
D-8	55.71	(1st) $(\gamma_h)_{swell}$	1400	800	0.0302
		(2nd) $(\gamma_h)_{shrink}$	800	1400	0.0446
		(3rd) $(\gamma_h)_{swell}$	1400	800	0.0204
D-9	88.53	(1st) $(\gamma_h)_{swell}$	1400	800	0.0213
		(2nd) $(\gamma_h)_{shrink}$	800	1400	0.0397
		(3rd) $(\gamma_h)_{swell}$	1400	700	0.0223

*Expressed in terms of void ratio

From the limited data, the shrinking suction compression index was measured to be greater than the swelling suction compression index. It is believed that this trend is due to the intact sample being tested under field conditions (i.e. confined with in-situ net normal stress applied). It requires more energy for the soil to swell while wetted, given a set suction gradient, than to shrink from drying by the equivalent suction gradient due to the weight of the net normal stress acting on the specimen. However, not enough data was gathered in this study to confidently claim that this trend applies to all soils, and field evidence suggests that the suction compression and suction swell index values may approach essentially the same value after many cycles of wetting and loading in the field. That the swell and shrinkage suction compression index values approach each other after multiple cycles of loading is suggested by the results of specimen D4 of this study where the shrinkage suction compression index approaches the swell suction compression index on the 4th cycle of suction change. Future research efforts should be conducted to further study the relationship between the swelling and shrinking suction compression index.

Table 12 also shows the limited hysteresis effects on the OPPD tested specimens when the secondary wetting cycle was conducted. The suction compression indices from subsequent wetting cycles show good agreement with each other. However, more research efforts should be conducted to further study this trend as well.

8 EVALUATION OF SURROGATE PATH METHOD (SINGHAL, 2010)

Singhal (2010) proposed a method for estimation of partial wetting heave in which the suction compression index does not need to be known. Since the procedure estimates the partial wetting strain, without using the suction compression index, it is referred to the surrogate path method (SPM). The SPM provides a method for mapping the wetting path in the volume change versus log matric suction plane (for a fixed net normal stress) into the volume change versus log net normal stress plane.

8.1 Surrogate Path (SPM) Procedure

Figure 39 below illustrates the 3-dimensional plot of the SPM (Singhal, 2010), where the matric suction (u_a-u_w) and vertical strain (ε) axes are arithmetic and the net total stress (σ-u_a) axis is logarithmic.

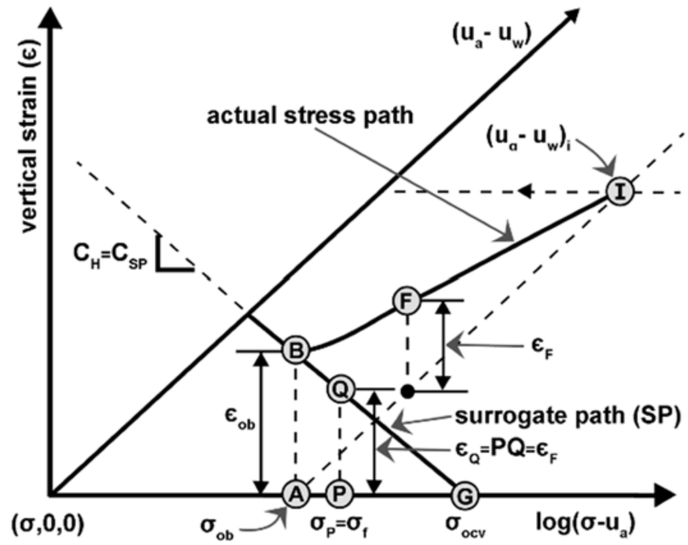


Figure 39: Strain-Based "Equivalence" of Reduction of Suction from $(u_a - u_w)_i$ to Zero (Path IB) to Reduction in Net Normal Stress from σ_{ocv} to σ_{ob} (Path GB, the SP)

Houston and Houston (2017) explain the SPM illustration and procedure as follows. It is important to note that the SPM illustration (Figure 39) is not necessary for the estimation of the partial wetting heave if the outlined computation procedure is followed. However, Figure 39 is discussed in more detail below.

> In Figure 39, Point I is depicted as the initial point for the wetting process where the suction is $(u_a - u_w)_i$. The plane IBA is parallel to the suction axis and perpendicular to the net stress axis. Point B lies in the net total stress plane where the suction is zero (i.e. full wetting). The path (curve) IFB is the suction path for full wetting and produces a full wetting strain, $\varepsilon_{ob} = \varepsilon_{fw}$ (i.e. the strain corresponding to the distance AB in Figure 39). The path IFB is labeled as the Actual Stress Path and is curved, not a straight line. IFB would be somewhat non-linear if suction were plotted on a log scale, and it is even more non-linear given that suction is plotted

arithmetically. For partial wetting, the matric suction does not go all the way to zero but stops at a final condition demonstrated by Point F where the strain is $\varepsilon_{pw} = \varepsilon_F$.

Now consider this wetting path IFB mapped as path GQB in the σ plane, where $(u_a\text{-}u_w) = 0$. The path GQB is the path for wetting to zero matric suction at Point B. The path BQG is established as follows:

i) A test specimen with initial suction $(u_a\text{-}u_w)_i$ and field overburden stress of σ_{ob} is loaded in the laboratory oedometer to σ_{ob} and then submerged. When swelling ceases, the strain is ε_{ob}, which plots as AB in Fig. 3. Note that where structural loads are significant, the applied stress (referred to here as σ_{ob}) should be overburden plus structural load.

ii) A second specimen, as nearly identical as possible to the first specimen, is loaded to a higher stress (preferably 2 to 3 times σ_{ob}, or more), then flooded, and the resultant strain is plotted versus the applied stress. Extrapolation (or interpolation) through points B and the second specimen point on the stress axis is used to approximate the swell pressure, σ_{ocv}, at zero strain (point G). This technique has been widely used to approximate σ_{ocv} (Houston and Nelson, 2010). The subscript "ocv" refers to the swelling pressure for a specimen first loaded to overburden and then subjected to a constant-volume swell pressure measurement. Alternatively, the load-back procedure, with correction, can be used to approximate the constant volume swell pressure, σ_{ocv} (Thompson, 2006; Nelson et al., 2006).

With σ_{ocv} established at point G, the line GB serves as a surrogate path (SP) for the Actual Stress Path. Note that the actual path IFB generates a full wetting strain of ε_{ob} = AB, and the surrogate path (SP) generates the same strain, ε_{ob}, in going from G to B. The objective of the interpolation method is to find an intermediate stress between σ_{ob} and σ_{ocv}, call it σ_p, that produces a strain PQ that is equal to ε_F, the partial wetting strain at Point F. This interpolation is accomplished by using the proportion of suction dissipated by wetting from I to F as a proportionality factor in estimating the final net stress, σ_p, at point P. In other words, R_w is defined as $R_w = (u_a-u_w)_f / (u_a-u_w)_i$ where $(u_a-u_w)_i$ is the initial suction and $(u_a-u_w)_f$ is the final suction. Thus $R_w = 1$ for no wetting and $R_w = 0$ for full wetting, and $(1-R_w)$ = degree of wetting. Then, $\sigma_p = \sigma_{ob} + R_w (\sigma_{ocv} - \sigma_{ob})$. The actual path, I to F, in Figure 39 is replaced with the surrogate path, GQ. The strain PQ at point P (ε_Q) was compared by Singhal (2010) to laboratory suction-controlled measured strain ε_F for numerous cases and an excellent agreement was found for all cases (Table 13).

Table 13: Comparison of SPM-Computed and Directly Measured Partial Wetting Swell Strain for Wetting of Compacted Clay Specimen (Singhal, 2010)

Change in Matric Suction (kPa)	Applied Net Normal Stress (kPa)	Initial Void Ratio	σ_{cv} (kPa)	$(\varepsilon_{pw})_{OPPD}$	$(\varepsilon_{pw})_{SPM}$
1200 to 500	nominal (1 to 3)	0.89	138	0.03	0.022
	25	0.855	29	0.03	0.028
	150	0.848	450	0.034	0.036
1200 to 100	nominal	0.89	138	0.076	0.061
	25	0.855	269	0.071	0.065
	150	0.848	450	0.062	0.07
1200 to 0 (Submerged)	nominal	0.89	138	0.125	0.125
	25	0.855	269	0.087	0.087
	150	0.848	450	0.081	0.081

The SPM interpolation was found to consistently produce results with less than 10% error for heave simulation cases explored by Singhal (2010).

The SPM requires that initial and final suction values in the field be measured or estimated; the SPM does not require that suction-controlled oedometer testing be performed, but rather employs the very familiar oedometer procedure and apparatus. The SPM does not require that the slope of the strain-log suction curve, γ_h, be measured or estimated and problems with the nonlinearity of this curve in the low and high suction range are greatly reduced or eliminated. However, it is noted that the data needed to estimate the suction compression index is readily available from the SPM without measuring or controlling suction. This is because the strain at point F, ε_F, is determined by the SPM and then γ_h in going from $(u_a\text{-}u_w)_i$ to $(u_a\text{-}u_w)_f$ can be back-calculated, if wanted. However, there is no particular need to make this computation because the strains are readily estimated from the SPM for any value of $(u_a\text{-}u_w)_f$ between $(u_a\text{-}u_w)_i$ and zero, and the strains are the ultimate objective.

Singhal (2010) points out that one of the strengths of the SPM is that it is founded on the full-wetting oedometer test and is thus forced to be more or less exactly correct at the extremes of no wetting and full wetting. Singhal also found that the SPM results are not very sensitive to the estimate of σ_{ocv}, and therefore, it is acceptable to use corrected load-back values for σ_{ocv}. The SPM is simply used to provide a reasonable, rational method for interpolation between the extremes – which it does. The SPM is a soil-suction-based approach in that it requires estimates of initial and final suction, and in that the proportionality factor, R_w, is computed from numerical values of suction. However, the SPM is actually a marriage of suction-based approaches and the overburden swell test performed in the familiar oedometer apparatus.

The following summarizes the procedure for the determination of the SPM partial wetting strain (ε_{pw}), in lieu of Figure 39, as laid out by Houston and Houston (2017). The initial matric suction $(u_a\text{-}u_w)_i$ of the soil must be known or estimated. The ratio (R_w) of the initial matric suction to the chosen final matric suction is determined as:

$$R_w = \frac{(u_a - u_w)_i}{(u_a - u_w)_f} \tag{12}$$

The slope of the surrogate path (C_{SP}) is then calculated using the fully wetted oedometer strain (ε_{ob}) under the field net normal stress (σ_{ob}) and the load back swell pressure (,σ_{cv}).

$$C_H = C_{SP} = \frac{\varepsilon_{ob}}{\log\left(\sigma_{cv} / \sigma_{ob}\right)} \tag{13}$$

Next, intermediate stress (σ_p) between σ_{ob} and σ_{cv} is determined by:

$$\sigma_p = \sigma_{ob} + R_w\left(\sigma_{cv} - \sigma_{ob}\right) \tag{14}$$

Lastly, the final partial wetting strain is calculated by:

$$\varepsilon_{pw} = \varepsilon_f = C_H \log\left(\sigma_{cv} / \sigma_p\right) \tag{15}$$

Houston and Houston (2017) have observed that errors in the estimation of the final matric suction value are dampened by the SPM:

> For the above example, if the final suction were actually 200 kPa rather than the estimated 250 kPa, this would represent an error in final soil suction of 20%. If this

final suction error is carried through to the computation of final swell strain, the strain becomes 2.83% instead of the computed 2.58% - an error of about 10%.

8.2 Results of Partial Wetting Heave Estimation by SPM

Similar to the Singhal (2010) comparison study of the SPM to directly measured partial wetting strains (Table 13), undisturbed samples were tested in this study for partial wetting strains using the OPPD and then compared to those estimated by the SPM procedure. If a specimen exhibited compression during full wetting in the OPPD test (i.e. D-6), it was not included in the SPM comparison.

The full wetting oedometer test (ASTM D4546) is typically used in the SPM to determine the slope of the surrogate path (C_H) for each companion specimen. However, several of the full wetting oedometer strains of the specimens tested in this study did not show good agreement with the fully wetted OPPD strains from the same sample location, likely due to field sample variability. Therefore, the concept of companion specimens was disregarded in this this study. Houston and Houston (2017) observe that the slope of the surrogate path (C_H) typically ranges from 2% to 5% for clay soils. In order to evaluate the SPM and dampen the effect of non-companion samples, the average slope of surrogate path ($(C_H)_{avg}$) from the fully wetted oedometer test (ASTM D4546) was determined to be 3.91 and used to estimate swell pressure in the calculations of the partial wetting strains. The OPPD specimens were, in general, taken to low matric suction values of 50 to 100 kPa, and in general, where matric suction was reduced below 50 to 100 kPa very little additional swell was observed. Typically, only about 5 to 10% increase in full wetting swell was observed upon full submergence where full submergence was used at the end of the OPPD test, e.g. a specimen exhibiting 2% swell with reduction to 50 kPa matric suction might exhibit about 2.2% swell for full submergence. Therefore, when specimens were not fully wetted in the OPPD, the largest value of swell strain (e.g., swell strain at 50 kPa matric suction), corresponding to the lowest matric suction used in the test, was used in lieu of the full wetting strain to avoid errors associated with sample variability. The CV swell pressures were estimated (σ'_{cv}) using the average D4546 C_H slope of the surrogate path by the following equation:

$$\sigma'_{cv} = \sigma_{ob}\left(10^{\left(\frac{(\varepsilon_{ob})_{OPPD}}{(C_H)_{avg}}\right)}\right) \tag{16}$$

The estimated partial wetting strain via the SPM ($(\varepsilon_{pw})_{SPM}$) was determined from the specimen's initial matric suction to a lower suction value, at which the corresponding strain was directly measured using the OPPD test ($(\varepsilon_{pw})_{OPPD}$). As previously discussed in Section 6.3, the initial suction for each specimen was either directly measured with the OPPD or back calculated using the average osmotic suction for each site. The results of the comparison of the SPM partial wetting swell strains to the OPPD directly measured partial wetting swell strains are summarized below in Table 14. The difference (d_i) between the SPM result and the directly measured partial wetting result is also presented in the table, which will be used to statistically verify the SPM in the following section.

Table 14: Comparison of SPM-Computed and OPPD Directly Measured Partial Wetting Swell Strains

ID	σ_{ob} (kPa)	σ'_{cv} (kPa)	$(\varepsilon_{ob})_{OPPD}$ (%)	$(u_a\text{-}u_w)_i$ (kPa)	$(u_a\text{-}u_w)_f$ (kPa)	$(\varepsilon_{pw})_{SPM}$ (%)	$(\varepsilon_{pw})_{OPPD}$ (%)	Difference (d_i)
D-1	32.3	50.9	0.8	3392	1400	0.41	0.32	0.09
					800	0.55	0.63	-0.08
					300	0.69	0.62	0.07
D-2	53.5	57.4	0.12	3826	100	0.12	0.12	0.00
D-3	31.6	68.1	1.3	5345	1400	0.85	0.73	0.12
					800	1.03	0.94	0.09
					200	1.23	1.20	0.03
D-4	79.9	102.4	0.42	700	100	0.35	0.42	-0.07
D-5	91.1	148.6	0.83	1260	700	0.32	0.21	0.11
					200	0.67	0.76	-0.09
D-6	150	57.1	-1.64	3309	800	-1.36	-1.95	0.59
					100	-1.61	-1.69	0.08
D-7	27.4	32.7	0.3	900	400	0.16	0.1	0.06
					200	0.23	0.2	0.03
D-8	55.7	125.7	1.38	1400	800	0.46	0.41	0.05
					100	1.23	1.38	-0.15
D-9	88.5	192.8	1.32	2105	1400	0.34	0.1	0.24
					800	0.69	0.81	-0.12
					100	1.23	1.32	-0.09
D-10	112.7	197.3	0.95	2028	1400	0.24	0.32	-0.08
					800	0.51	0.63	-0.12
					200	0.83	0.84	-0.01
SA-1	30.9	53.4	0.93	300	200	0.26	0.21	0.05
					100	0.56	0.81	-0.25
SA-2	57.6	100.3	0.94	480	400	0.12	0.1	0.02
					200	0.48	0.52	-0.04
SA-3	84.0	235.6	1.75	900	500	0.57	0.52	0.05
					200	1.18	1.08	0.10
					100	1.44	1.5	-0.06
SA-4	82.7	241.8	1.82	1020	800	0.26	0.21	0.05
					300	1.06	0.88	0.18
SA-5	82.8	113.2	0.53	500	300	0.19	0.21	-0.02
					200	0.30	0.32	-0.02
SA-6	30.0	40.5	0.51	300	150	0.24	0.21	0.03
SA-7	59.4	96.9	0.83	400	200	0.36	0.47	-0.11
SA-8	60.4	82.0	0.52	650	300	0.26	0.21	0.05
SA-9	31.7	83.0	1.63	1420	800	0.53	0.61	-0.08
					300	1.13	1.12	0.01
SA-10	59.2	94.8	0.8	440	200	0.39	0.32	0.07
SA-11	83.7	187.8	1.37	750	500	0.35	0.32	0.03
					200	0.88	0.89	-0.01

8.2.1. Statistical Analysis of SPM

A student's t-statistic was used to verify that there was no statistical difference between the SPM computed partial wetting strain and the OPPD directly measured partial wetting strain. A paired comparison hypothesis was tested, which uses the average ($\bar{d}$), and standard deviation (s_d) of the difference between the matched pairs, expressed as:

$$\bar{d} = \frac{\Sigma\left(x_{SPM} - x_{OPPD}\right)}{n} \tag{17}$$

$$s_d = \sqrt{\frac{\sum\left(d_i - \mu_d\right)^2}{\left(n-1\right)}} \tag{18}$$

where x_{SPM} is the SPM estimated partial wetting heave value (%), x_{OPPD} is the OPPD directly measured partial wetting heave (%), n is 41 (the number of total comparisons), and d_i is the difference for a specific pair. The null (H_0) and alternative (H_1) hypotheses tested were as follows.

$$H_0 : \mu_d = 0 \tag{19}$$

$$H_1 : \mu_d \neq 0 \tag{20}$$

where μ_d is the true mean of the difference between matched pairs of the SPM computed partial wetting heave and the OPPD directly measured partial wetting heave. The measured average difference between the matched pairs ($\bar{d}$) was assumed to be equivalent to the true mean value (μ_d). The "t" test statistic is computed by:

$$t = \frac{\left(\bar{d} - 0\right)}{s_d}\sqrt{n} \tag{21}$$

If the true mean of the paired differences departs from zero in either direction, the null hypothesis is rejected. The null hypothesis is to be rejected if the absolute value of t-statistic is greater than or equal to the critical value of $t_{\alpha/2;n-1}$, as expressed below:

$$|t| \geq t_{\alpha/2;n-1} \tag{22}$$

where α is the significance level or 1 minus the confidence level. The following table summarizes the hypothesis test, in which a 99% confidence level was used.

Table 15: Hypothesis Test Results for Comparison of SPM to OPPD Directly Measured Values

Number of Samples (n)	41
Degrees of Freedom (n-1)	40
Average of Difference ($\bar{d}$) (%)	0.020
Standard Deviation (s_d) (%)	0.131
Hypothesized Difference (%)	0
Confidence Level (%)	95
Significance level	0.05
t-statistic	0.976
t-critical ($t_{\alpha/2;n-1}$)	2.021
Null Hypothesis ($H_0 : \mu_d = 0$)	Accept

Since the t-statistic for the paired comparison data is less than t-critical, the null hypothesis cannot be rejected (i.e. it is accepted). Therefore, within a 95% level of confidence, there is no statistical difference between the SPM computed partial wetting swell strains and the OPPD directly measured partial wetting swell strains.

9 SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

9.1 Summary

The vast amounts of infrastructure damage caused by expansive soils have led researchers to advance methods of estimating ground heave. Although there are many accepted methodologies, many rely on index property correlations built from laboratory data that does not simulate the natural field conditions of the soil.

This study identified significant differences between partial wetting soil response of a natural undisturbed specimen compared to a reconstituted specimen. Improvements to the oedometer pressure-plate device testing procedure of highly plastic clay soils were discussed including a method for the direct determination of the in-situ matric suction. The improved OPPD procedure allowed for the direct measurement of matric suction on undisturbed specimens, development of the soil water characteristic curves with volume change, the direct measurement of the partial wetting strain, and the determination of the suction compression index for the 21 studied natural soils. The evaluation of the WP4-C total suction measurement device was also conducted in this study by comparing it to the widely accepted filter paper method. Directly measured suction compression index values were compared to existing methods for estimation of suction compression index. Existing methods for estimation of partial wetting strains using initial and final suction values were also compared to directly measured partial wetting strains from the OPPD tests.

9.2 Major Findings

The following summarizes the key contributions of this study:

- The theory that an undisturbed natural soil specimen will exhibit a higher matric suction than a remolded specimen of the same material with equivalent moisture content was verified in this study using the WP4-C (Figure 21). It was observed that specimens with a total suction below 1000 kPa will experience higher effects of disturbance when measuring with the WP4-C.

- For practical purposes, the WP4-C was found to be a highly efficient testing apparatus for the measurement of total suction. The WP4-C showed excellent agreement with the filter paper method, but the quick equilibration time, sample size, and "user friendliness" of the WP4-C make it an ideal total suction measurement tool for practicing engineers.
- The use of the oedometer pressure plate device (OPPD) to test the natural response of a clay soil to partial wetting requires great attentiveness and awareness of sources of potential measurement error. The long equilibration times of such testing comes with high risk of mechanical issues and sample loss. OPPD testing for a natural, undisturbed clay response to partial wetting is not, in general, feasible for practicing engineers due to long equilibration times. However, such lab testing should be continued by researchers so that improvements to current methods of estimating partial wetting soil response can be made.
- When testing undisturbed soil specimens, it must be understood that the concept of identical or companion samples is difficult to achieve due to the heterogeneity of natural soil deposits.
- The shape of the soil water characteristic curve for natural soils differs significantly from reconstituted samples. A natural soil will exhibit a scanning curve shape, while reconstituted samples are expected to create a boundary curve shape. Discrepancies

between values pulled from the SWCC (i.e. AEV, residual water content, and slope in the transition zone) of a natural specimen and those obtained from a reconstituted specimen should be expected.

- The Perko et al. (2000), PTI (2008) Method A, and the PTI (2008) method D tend to overestimate the directly measured undisturbed specimen suction compression index of the soil. However, the PTI (2008) Method D showed the closest agreement to the directly measured, which is believed to be due to the incorporation of the soils full wetting response under field stress conditions. PTI Method D was the method studied that incorporated some aspect of the response to wetting behavior of the undisturbed sample of soil under field-appropriate net normal stress and did not rely solely on index property correlations. Directly measured suction compression index values were compared to these existing methods for estimation of suction compression index, and in general, existing methods based solely on index properties were found to significantly overestimate the suction compression index.
- The PTI (2008) Method A, adapted from Covar and Lytton (2001), was observed to have significant instability with respect to the amount of fine clay (%fc) used in the calculation. Practicing engineers should be aware, when using this method to analyze partial wetting soil response, that slight variations in hydrometer testing can cause significant swings of the calculated shrink/swell values, with no logical trend. Further, the swelling suction compression index values from Method A were found to be overly conservative compared to the directly measured suction compression index values.
- The Surrogate Path Method (Singhal, 2010) for estimation of partial wetting strain

was found to give very good agreement with the OPPD directly measured partial wetting strain values. Since the SPM relies only on the fully wetted oedometer test (ASTM D4546) and does not require the suction compression index to be known, it is the recommended method for estimating partial wetting soil response in practice. Because the Surrogate Path Method (SPM) was found to provide excellent estimates of directly measured partial wetting strains, the SPM could be used to obtain estimates of partial wetting strains that would have been obtained had the long-duration OPPD test been actually performed.

- Although not directly evaluated in this study, Houston and Houston (2017) also describe the SPM procedure for estimating partial wetting strains of compressible/collapsible soils.

Lastly, the results of the OPPD testing of the 21 undisturbed highly plastic soils under natural stress conditions, are in themselves valuable contributions to the field of unsaturated soil mechanics, as such data is very rare due to the previously described difficulties with the lab testing. The developed SWCC plots and void ratio vs. matric suction plots for each of the 21 samples are presented in Appendices B and C.

9.3 Recommendations for Future Work

The results and findings of this study have led to a better understanding of the response to wetting of natural soils when partial wetting occurs, and how this response differs from that of reconstituted samples. Although general trends were observed and cited, the 21 tested soils modeled only a small sample size of the many soil conditions present in nature. It is recommended that direct partial wetting measurements on natural clay soils using the OPPD be continued by researchers to verify and improve on the findings of this study.

9.3.1. Paradigm Switch of Slurry SWCC Test to Index Property Test

It was observed in this study that the natural shape of the SWCC differs significantly from the SWCC obtained from reconstituted/remolded samples, specifically the conventional approach in which a slurry sample undergoes one initial drying cycle. Although such tests are relatively easy to run and highly reproducible, the air-entry, residual, and storage values obtained from such test are not representative of the natural, in-situ behavior of the soil, but rather might be looked at as index parameters for the soil. The possibility of the reconstituted/slurry SWCC test to be looked at as an index test should be discussed further. In lieu of, it is recommended that for natural soils, where volume change response is required, the SWCC should be determined on the sample in its natural condition, and the path of wetting (or drying) should, within field anticipated suction change, be used. Volume change measurements should be used in SWCC testing. Large shifts in mechanical response of the soil (i.e. SWCC and suction compression index) are expected during the first several wetting/drying cycles on both compacted and slurry specimens. Natural field soils undergo a great number of cycles of wetting and drying within certain soil suction ranges, but even with multiple cycles of wetting/drying remolded or compacted specimens are not expected to exhibit the same response as natural field soils. Thus, where the engineering response of natural soils is required, undisturbed specimens should be used in laboratory testing; where fill soil response is required, compacted specimens should be used for laboratory testing.

9.3.2. Development of Full Stress State Surfaces for Natural Soils

The specimens in this study were tested under the field overburden stress only allowing for the determination of the partial wetting strains for a "free field" condition. It is recommended that similar studies be conducted in which the net normal stress includes varying foundation loading. Such testing, will be highly stress path dependent. Different soil response should be expected for soils that are loaded and then wetted, compared to those that are wetted and then loaded. The ability to obtain highly similar (i.e. companion) specimens will be a key factor in such study.

9.3.3. Study of the Relationship Between Swelling and Shrinking Suction Compression Index

It has been accepted from past literature (PTI, 2008), that the swelling suction compression index will be greater than the shrinking suction compression index (Figure 34), however results from the limited tests conducted in this study show the opposite trend. The addition of confinement and field net normal stress on the undisturbed soils tested in this study played a role in this disagreement between trends, since the suction compression index was historically measured by unconfined shrinkage tests with minimal or no net normal stress applied. Additional wetting/drying cycles should be conducted on natural specimens under field net normal stress using the OPPD to further explore this observation, as it is most likely, based on field evidence, that the swell and shrinkage suction compression index are more or less equal for field conditions where there have many cycles of wetting/drying within the range imposed for site-specific circumstances.

REFERENCES

Ajayi, L. (1987). Oedometer tests for swelling or collapsibility potential . *9th African Regional Conference of Soil Mechanics and Foundation Engineering*, (pp. 184-197). Lagos.

Al-Mhaidib, A. (2006). swelling behavior of expansive shale. In *Expansive Soils: Recent advances in characterization and treatment* (pp. 273-288). London: Taylor & Francis Group.

ASTM D2488. (2014). Visual Classification of Soil. *American Standards for Testing and Materials*.

ASTM D4546. (2014). Standard Test Method for one-dimensional swell or settlement potential of cohesive soils.

Attom, M., & Barakat, S. (2000). Investigation of three methods for evaluating swelling pressure of soils. *Enviromental and Engineering Geoscience*, 293-299.

Basma, A., & Tuncer, E. (1992). Evaluation and Control of Collapsible Soils. *Journal of Geotechnical Engineering, 118*, 1491-1504.

Box, J. a. (1962). *Influence of soil bulk density on matric potential.* Soil Science Society of America.

Brackley, I. (1973). Swell pressure and free swell in a compacted clay. *3rd Internation Conference on Expansive Soils*, (pp. 169-176). Haifa, Isreal.

Brackley, I. (1975). Swell under load. *6th African Regional Conference of Soil Mechanics and Foundation Engineering*, (pp. 65-70). Lagos.

Childs, E. (1940). The use of soil moisture characteristics in soil studies. *Journal of Soil Science, 50*, 239-252.

Chin, K., Leong, E., & Rahardjo, H. (2010). A simplified method to estimate the soil-water characteristic curve. *Canadian Geotechnical Journal, 47*, 1382-1400.

Coduto, D. (2011). *Geotechnical Engineering: Principles and Practices.* Pearson.

Covar, A., & Lytton, R. (2001). Estimating soil swelling behaviour using soil classification properties. *Proceedings of Geo-Institute Shallow Foundation and Soil Properties Committee Sessions at the ASCE 2001 Civil Engineering Conference* (pp. 44-63). Houston, Texas: ASCE.

Das, B. M. (2002). Plastic Limit Test. In *Soil Mechanics Laboratory Manual* (Vol. 6, pp. 57-61). New York, NY, United States: Oxford University Press.

Decagon Devices, Inc. (2011). *Effects of sample disturbance on soil water potential.* Pullman, WA: Decagon Devices, Inc.

Delage, P., Cui, Y., & Antoine, P. (2005). Geotechnical problems related with loess deposits in northern France. *International Conference on Problematic Soils*, (p. 24).

Douthitt, B., Houston, W., Walsh, K., & Houston, S. (1998). Effect of wetting on pile skin resistance. *2nd International Conference on Unsaturated Soils*, (pp. 219-224). Beijing.

Edlefsen, N., & Anderson, A. (1943). Thermodynamics of soil moisture. *Hilgardia, 15*, 31-298.

El Sayed, S., & Rabba, S. (1986). Factors affecting behavior of expansive soils in the laboratory and field - a review. *Journal of Geotechnical and Geoenviromental Engineering, 17(1)*, 89-107.

Erol, A., Dhowian, A., & Youssef, A. (1987). Assesment of oedometer methods for heave prediction. *6th International conference of expansive soils*, (pp. 99-103). New Delhi, India.

Feda, J. (1988). Collapse of Loess Upon Wetting. *Engineering Geology, 25*, 263-269.

Feng, M., Gan, J.-K.-M., & Fredlund, D. (1998). A laboratory study of swelling pressure using various test methods. *2nd Internation Conference on Unsaturated Soils*, (pp. 350-355). Beijing, China.

Fredlund, D. (1979). Appropriate concepts and technology for unsaturated soils. *2nd Canadian Geotechnical Colloquium , 16*(1), 121-139.

Fredlund, D. (1995). The prediction of heave in expansive soils. *Canada-Kenya Symposium on Unsaturated Soil Behavior and Applications*, (pp. 105-119). University of Nairobi, Nairobi, Kenya.

Fredlund, D. G., Rahardjo, H., & Fredlund, M. D. (2012). *Unsaturated Soil Mechanics In Engineering Practice.* Hoboken, New Jersey: John Wiley & Sons, Inc.

Fredlund, D., & Houston. (2013). Interpretation of SWCCs when volume change occurs as soil suction is changed. *1st Pan-American Conference on Unsaturated Soils, PanAmUNSAT 2013*, (pp. 15-31). Cartegena de Indias, Colombia.

Fredlund, D., & Morgenstern, N. (1977). Stress state variables for unsaturated soils. *Journal of the Geotechnical Engineering Division, 103*(G75), 441-466.

Fredlund, D., & Rahardjo, H. (1993). *Soil Mechanics for Unsaturated Soils.* Wiley Inter-Science.

Fredlund, D., Hasan, J., & Filson, H. (1980). The prediction of total heave . *4th International Conference on Expansive Soils*, (pp. 1-17). Denver, CO.

Fredlund, D., Hassan, J., & Gilson, H. (1980). The prediction of total heave. *4th International Conference on Expansive Soils*, (pp. 1-17). Denver, CO.

Fredlund, D., Xing, A., & Haung, S. (1994). Predicting the permeability function for unsaturated soils using the soil-water characteristic curve.". *Canadian Geotechnical Journal, 31*(4), 533-546.

Fredlund, M., Wilson, G., & Fredlund, D. (2002). Use of grain-size distribution for the estimation of the SWCC. *Canadian Geotechnical Journal, 39*, 1103-1117.

GCTS Testing Systems. (2007). SWC-150: Fredlund Soil Water Characterstic Device . *1.3*. Tempe, AZ.

Geostructural Tool Kit, Inc. (2012). VOLFLO 1.5. Austin, TX.

Gilbert, P. A. (1992). *Effect of Sampling Disturbance on Laboratory-Measured Soil Properties.* US Army Corps of Engineers.

Gilchrist, H. (1963). *A study of volume change of a highly plastic clay*. Saskatoon: Universtiy of saskatchewan.

Hilf, J. (1956). *An investigation of pore-water pressure in compacted cohesive soils.* PhD Thesis, U.S. Department of Interior, Bureau of Reclamation, Design and Construction Division, Denver, CO.

Holtz, R. D., & Kovacs, W. (1981). Engineering Properties of Expansive Clays. *ASCE, 121*, 641-677.

Houston, S. (2002). Applied Unsaturated Soil Mechanics, State of the Art Report. *UNSAT 2002*, *3*, pp. 1127-1134. Recife, Brazil, Balkema.

Houston, S. (2014). Characterization of Unsaturated Soils: The Imporatnce of Response to Wetting. *GeoCongress.*

Houston, S., & El-Ehwany, M. (1991). Sample Disturbance of Cemented Collapsible Soils. *Journal of Geotechnical Divison, 117*, 731-752.

Houston, S., & Houston, W. (1997). Collapsible Soil Engineering, by S., Unsaturated Soil Engineering Practice,. *GSP*(68), 199-232.

Houston, S., & Houston, W. (2017). Suction-Oedometer Method for Computation of Heave and Remaining Heave. *2nd Pan-American Conference on Unsaturated Soils.* Dallas, TX: ASCE.

Houston, S., Houston, W., & Spadola, D. (1988). Prediction of Field Collapse of Soils Due to Wetting. *Journal of Geotechnical Engineering Division, 1*, 40-58.

Houston, S., Houston, W., Zapata, C., & Lawrence, C. (2001). Geotechnical engineering practice for collapsible soils. *Geotechnical and Geological Engineering, 19*, 333-355.

Houston, S., Walsh, K., & Houston, W. (1997). Soils Strength Contribution of Soil Suction in Cemented Soil. *Solos Nao Saturados*, *1*, pp. 25-34.

Houston, W. (2017, October 16). Personal Communication. (A. Olaiz, Interviewer)

Houston, W., Dye, H., Zapata, C., Perera, Y., & Haraz, A. (2006). Determination of SWCC using One Point Suction Measurement and Standard Curves. *UNSAT2006.* Carefree, AZ: ASCE.

Houston, W., Mahmoud, H., & Houston, S. (1993). A laboratory procedure for partial wetting collapse determination. *Unsaturated Soils*, 55-63.

Hvorslev, M. (1949). *Subsurface Exploration and Sampling of Soils for Civil Engineering Purposes.* Vicksburg, Miss.: U.S. Army Engineer Waterway Experiment Station.

Janssen, D., & Dempsey, B. (1980). Soil-moisture properties of subgrade soils. *Sixtieth Annual Transportation Research Board Meeting.* Washington, DC.

Jasmar, R., & Ore, H. (1987). Hydrocompaction Hazards due to Collapsible Loess in Southeastern Idaho. *23rd Symp. on Engineering Geology and Soils Engineering*, (pp. 461-474).

Justo, J., Delgado, A., & Ruiz, J. (1984). The inflence of stress path in the collapse swellingof soils at the laborartory. *5th International Conference on Expansive Soils*, (pp. 67-71). Adelaide, South Australia.

Khaddaj, S., Lancelot, L., & Shahrour, I. (1992). Experimental study of the swelling behavior of heavily overconsolidated Flandres clay. *7th Internation Conference on Expansive Soils*, (pp. 239-244). Dallas, TX.

Krahn, J., & Frendlund, D. (1972). On total matric and osmotic suction. *Soil Science Journal, 114*(5), 339-348.

Krohn, J., & Slosson, J. (1980). Assesment of expansive soils in the United Sates. *Proceeding of 4th International Conference on Expansive Soils*, (pp. 596-608). Denver, CO.

Leong, E., Tripathy, S., & Rahardjo, H. (2003). Total suction measurments of unsaturated soils with a device using the chilled-mirror dew-point technique. *Geotechnique, 53*(2), 173-182.

Liu, T. (1997). Problems of Expansive Soils in Engineering Construction. *Architecture and Building Press of China, Beijing*.

Lu, Y. (1969). *Swell properties of desiccated Regina Clay.* Saskatoon: Unversity of saskatchewan.

Lytton, R. L. (1994). Prediction of movement in expansive clays. *Vertical and Horizontal Deformations of Foundations and Embankments., 40*, pp. 1827-1845.

Lytton, R., Aubeny, C., & Bulut, R. (2004). *Design procedure for pavements on expansive soils.* Austin, TX: Texas Department of Transportation.

Marcuson, W. F., & Franklin, A. G. (1979). *State of the art of undisturbed sampling of cohesionless soils.* US Army Corps of Engineers.

McCook, D., & Shanklin, D. (2000). NRCS experience with field density test methods including sand-cone, nuclear gage, rubber balloon, druve-cylinder, and CLOD test. *Constructing and Controlling Compaction of Earth Fills*, 72-93.

McKeen, R. (1992). A model for predicting expansive soil behavior. *Proceedings of the 7th International Conference on Expansive Soils*, (pp. 1-6). Dallas, Texas.

Munoz-CastelBlanco, J., Delage, P., Pereira, J., & Cui, Y. (2011). Some Aspects of the Compression and Collapse Behavior of an Unsaturated Natural Loess. *Geotechnique Letters*, 1-6.

Nagaraj, H., Munnas, M., & Sridharan, A. (2009). Critical evaluation of determining swelling pressure by swell-load method and constant volume method. *Geotechnical Testing Journal*, 305-314.

Nelson, J., & Miller, D. (1992). *Expansive Soils: Problems and Practice in Foundation and Pavement Engineering.* New York, NY: John Wiley and Sons, Inc.

Nelson, J., Chao, K., Overton, D., & Nelson, E. (2015). *Foundation Engineering for Expansive Soils.* New York, NY: John Wiley and Sons, Inc.

Nelson, J., Reichler, D., & Cumbers, J. (2006). Parameters for heave prediction by oedometer tests. *4th International Conference on Unsaturated Soils*, (pp. 951-961). Carefree, AZ.

Noble, C. (1966). Swelling measurements and prediction of heavefor a lacustrine clay. *Canadian Geotechnical Journal, 37*, 1252-1264.

Noorany, I., Gardner, W., Corley, D., & Brown, J. (2000). Variability in field density tests. *Constructing and controlling compaciton of earth fills*, 58-71.

Oh, W. T., Vanapalli, S. K., & Puppala, A. J. (2009). Semi-empirical model for the prediction of modulus of elasticity for unsaturated soils. *Canadian Geotechnical Journal, 46*(8), 903-914.

Padilla, J., Perera, Y., Houston, W., Perez, N., & Fredlund, D. (2012). *Quantification of air diffusion through high air-entry ceramic disks.* Tempe, AZ: ASCE.

Peck, R., Hanson, W., & Thornburn, T. (1974). *Foundation Engineering* (Second Edition ed.). Wiley.

Perko, H. A., Thompson, R. W., & Nelson, J. D. (2000). Suction Compression Index Based on CLOD Test Results. *GEO-Denver* (pp. 395-407). Denver, CO: ASCE.

Petry, T. M., & Jiang, C.-P. (2007). Round-Robin Testing of a Dewpoint Potentiameter Versus Filter Paper to Determine. *GEO-Denver 2007.* Denver, CO: ASCE.

Pham, H., & Frendlund, D. (2005). A volume-mass constitutive model for unsaturated soils. *Proceeedings of the fifty-Eighth Canadian Geotechnical Conference, 45*, 443-453.

Rosenbalm, D. (2013). *Volume Change Behavior of Expansive Soils due to Wetting and Drying Cycles.* Tempe, AZ: Arizona State University.

Saxton, K., Rawls, W., Romberger, J., & Papendick, R. (1986). Estimating Generalized soil-water characteristics from texture. *Soil Science Society of America Journal, 50*.

Singhal, S. (2010). *Expansive soil behavior: property measurement techniques and heave prediction methods* . PhD Thesis, Arizona State University, Tempe, AZ.

Singhal, S., Houston, S., & Houston W. (2011). Effects of Testing Procedures on the laboratory Determination of Swell Pressure of Expansive Soils. *ASTM Geotechnical Testing Journal, 34*, 476-488.

Sridharan, D., Rao, A., & Sivapullaiah, P. (1986). Swelling pressure of clays. *Geotechnical Testing Journal, 19(1)*, 24-33.

Terzaghi, K. (1942). Soil moisture: IXa. Soil moisture and capillary phenomena in soils. *Physics of the Earth, IX*, 331-363.

The Institution of Engineers Australia. (2011). *AS2870 - Residential Slabs and Footings.* Australian Standard.

Thompson , R., Perko, H., & Raethamel, W. (2006). Comparison of constant volume swell pressure and oedometer load-back pressure. *4th International Conference of Unsaturated Soils*, (pp. 1382-1393). Carefree, AZ.

Thompson, R. (1997). Evaluation protocol for repair of residences damaged by expansive soils. *Session on Unsaturated Soils, Geo-Logan. 68*, pp. 255-276. Logan, Utah: ASCE.

Tu, H. (2015). *Prediction of the variation of swelling pressure and 1-D heave of expansive soils with respect to suction.* Thesis, Univerisity of Ottawa, Ottawa, Canada.

Vanapalli, S. K., Fredlund, D. G., Pufahl, D. E., & Clifton, A. W. (1996). Model for the prediction of shear strength with respect to soil suction. *Canadian Geotechnical Journal, 33*(3), 379-392.

Walsh, K., Houston, W., & Houston, S. (1993). Evaluation of inplace wetting using soil suction measurements. *Journal of Geotechnical Engineering Division, 119*(5), 862-873.

Wray, W. (1984). The principle of soil suction and its geotechnical engineering applications. *Proceedings of the 5th International Conference on Expansive Soils*, (pp. 114-118). Adelaide, Australia.

Wray, W., & Meyer, K. (2004). Expansive Clay Soil. A widespread and Costly Geohazard. *Geo-Strata*, 24-28.

Zapata, C., Houston, W., Houston, S., & Walsh, K. (2000, August 5-8). Soil-water characterstic curve variability. *Advances in Unsaturated Geotechnics, 99*, 84-124.

Zhan, L., Chen, P., & Ng, C. (2007). Effect of suction change on water content of total volume of an expansive clay. *Journal of Zhejiang University, 5*(5), 699-706.

APPENDIX

A ASTM D4546 Response to Wetting Results

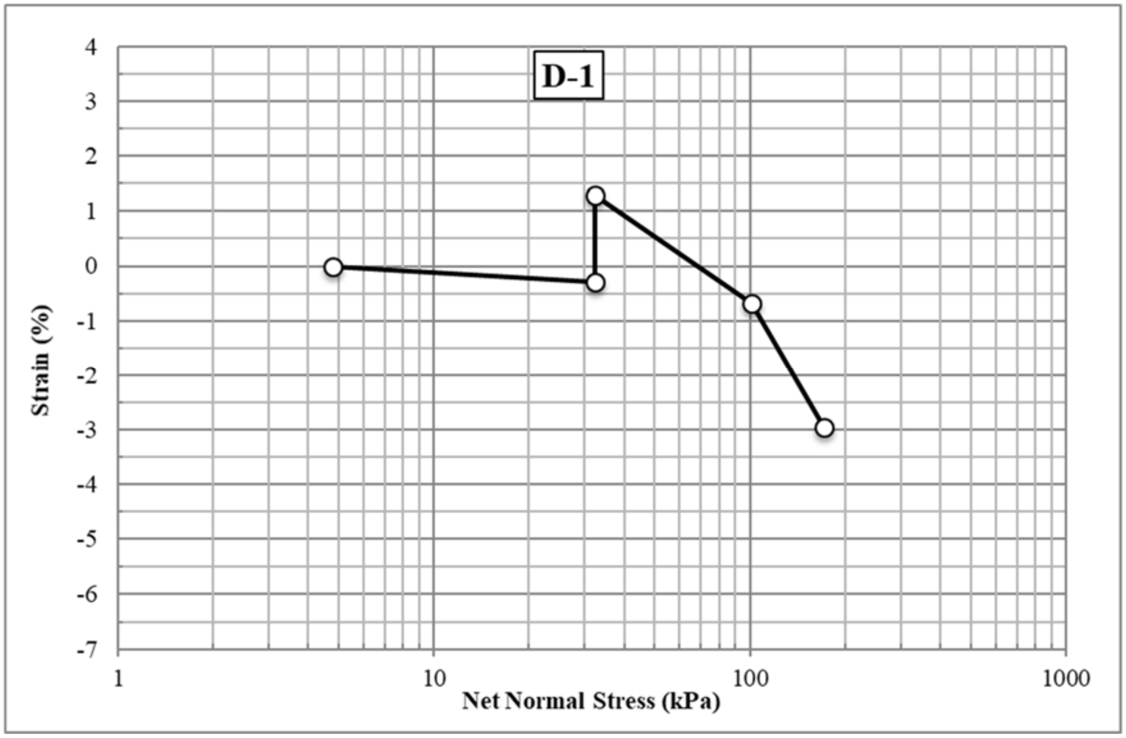


Figure 40: Fully Wetted Oedometer Test (ASTM D4546) for D-1

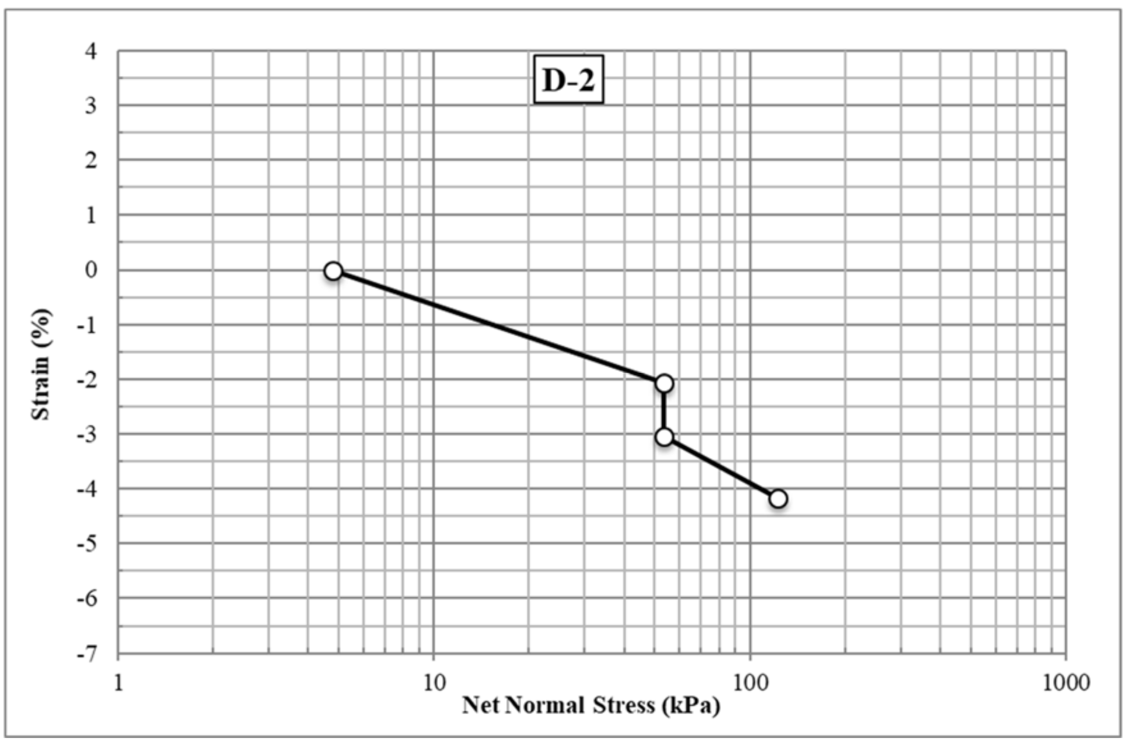


Figure 41: Fully Wetted Oedometer Test (ASTM D4546) for D-2

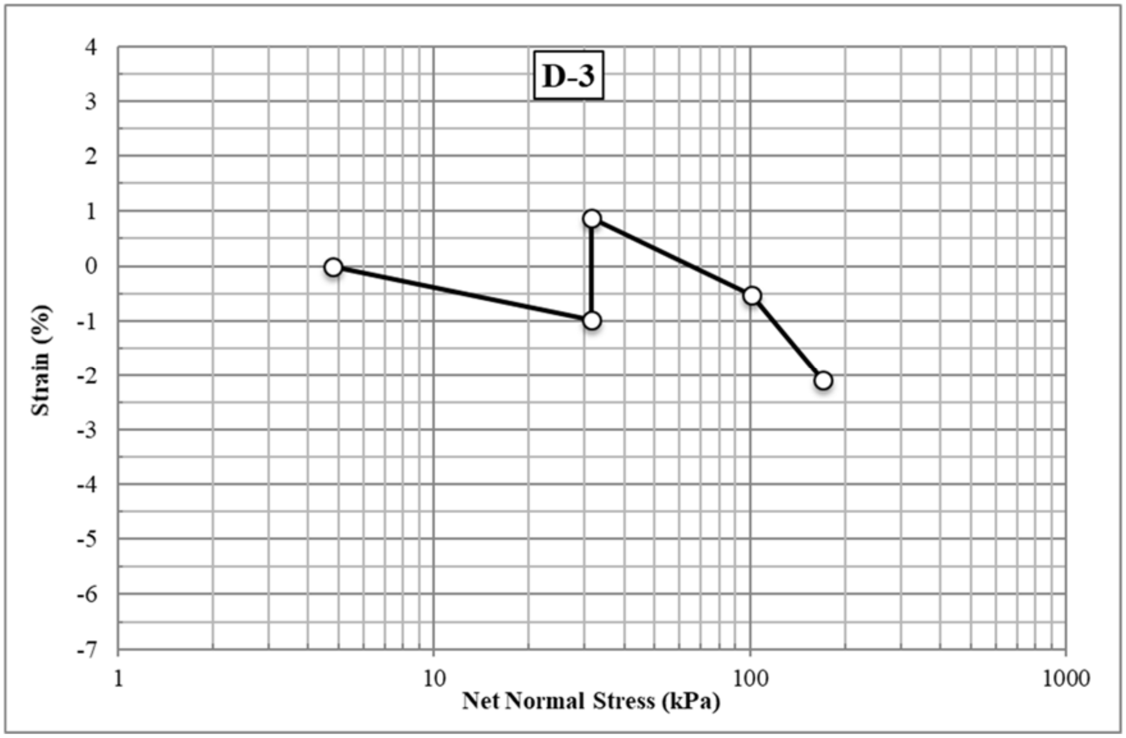


Figure 42: Fully Wetted Oedometer Test (ASTM D4546) for D-3

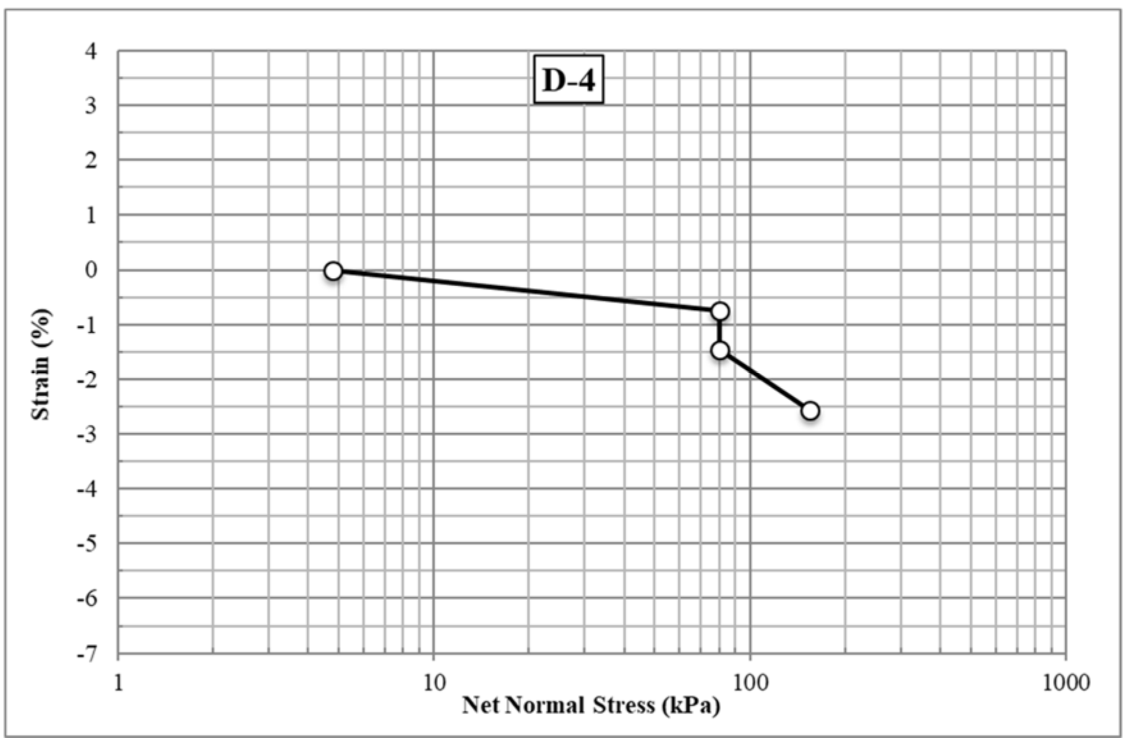


Figure 43: Fully Wetted Oedometer Test (ASTM D4546) for D-4

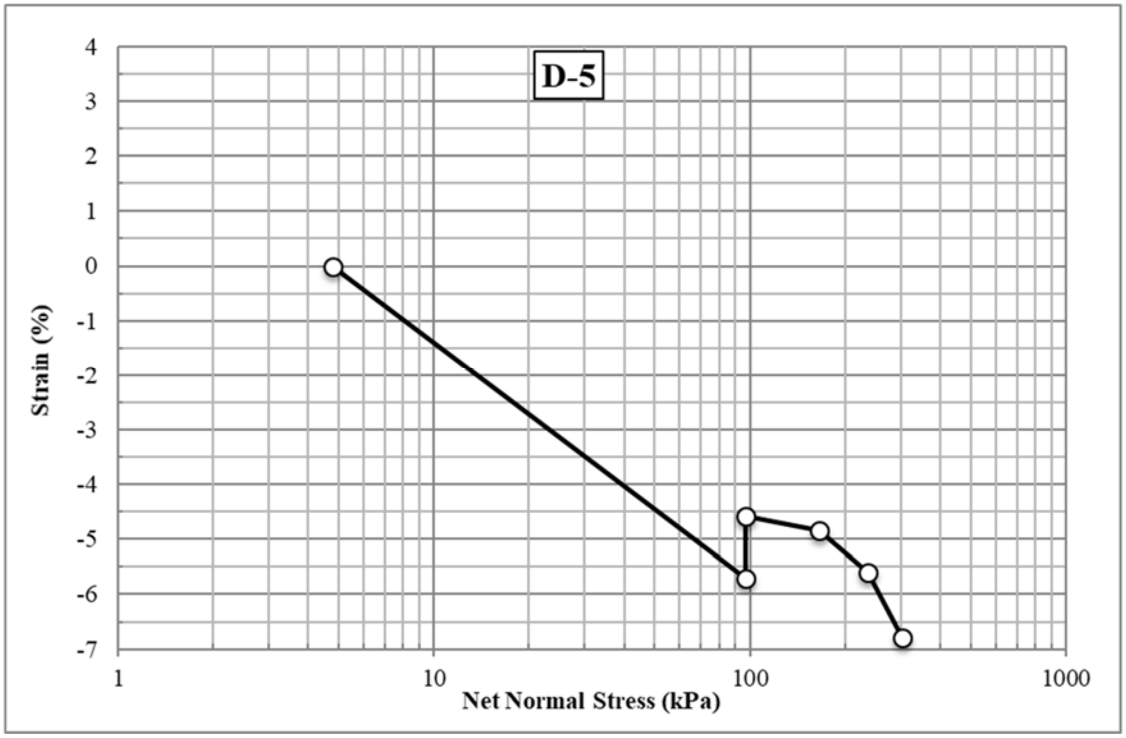


Figure 44: Fully Wetted Oedometer Test (ASTM D4546) for D-5

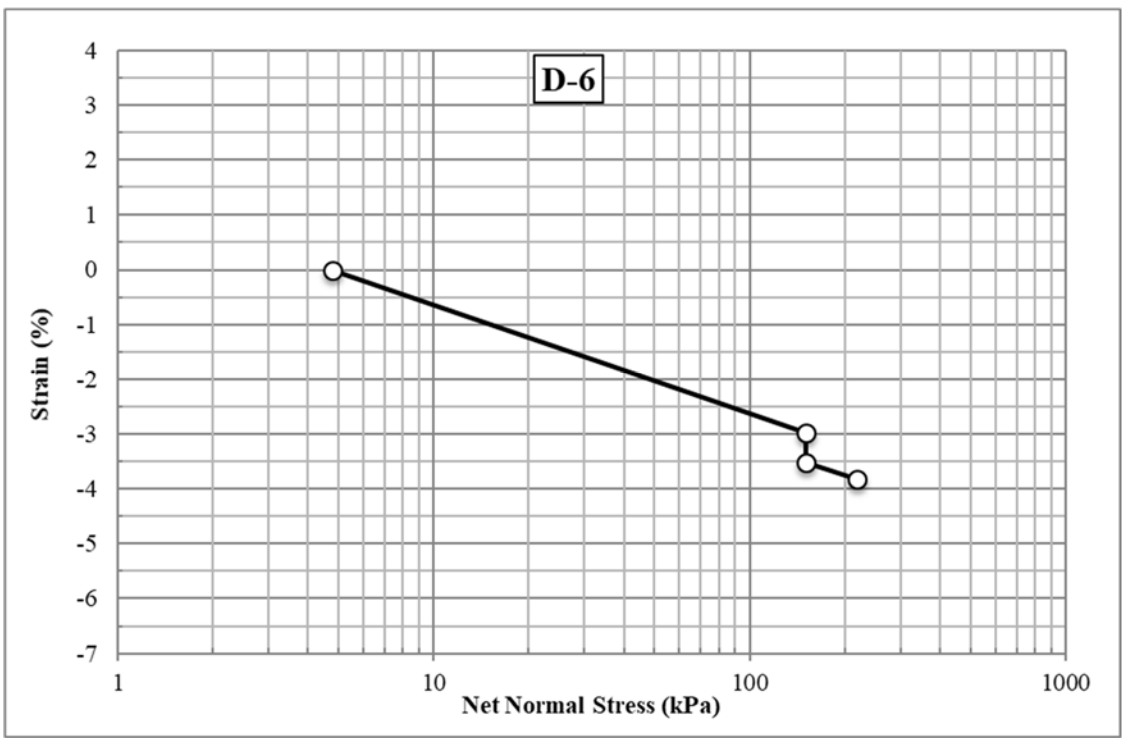


Figure 45: Fully Wetted Oedometer Test (ASTM D4546) for D-6

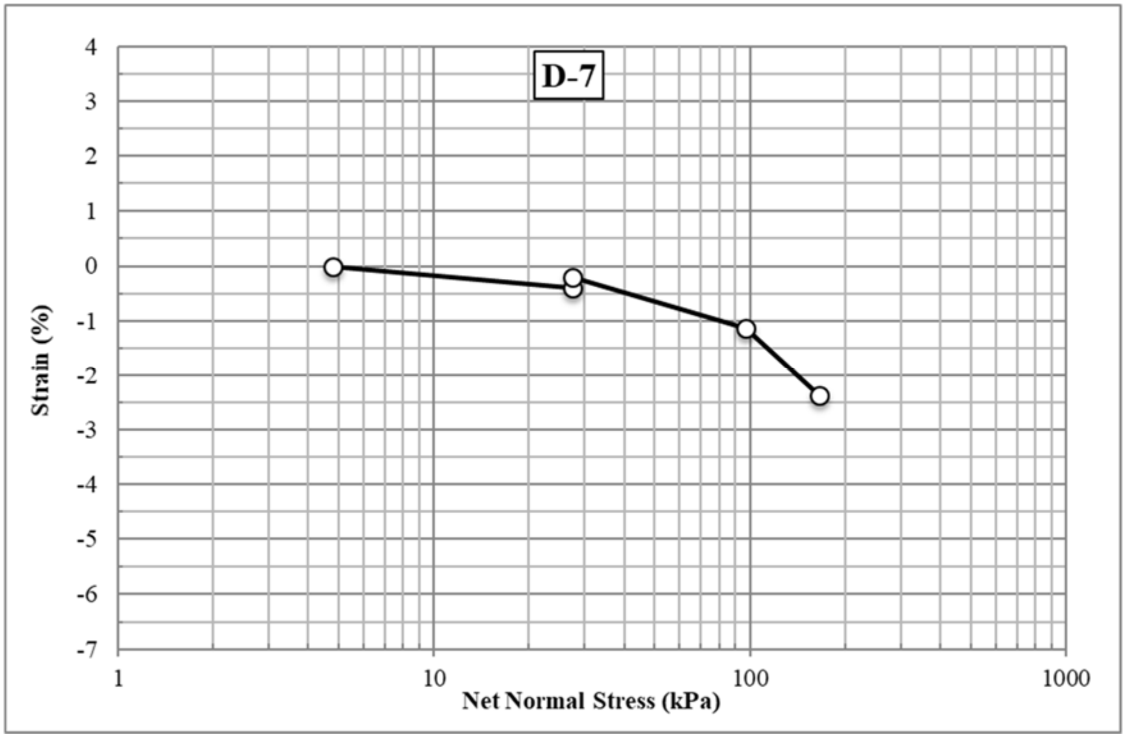


Figure 46: Fully Wetted Oedometer Test (ASTM D4546) for D-7

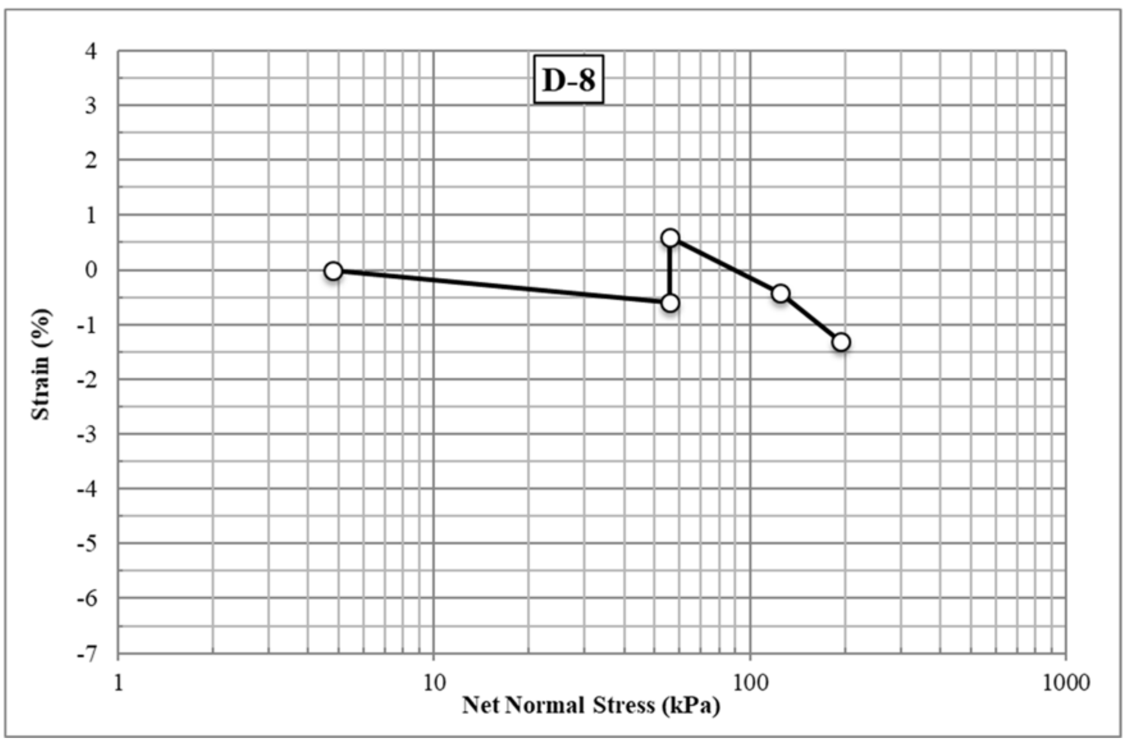


Figure 47: Fully Wetted Oedometer Test (ASTM D4546) for D-8

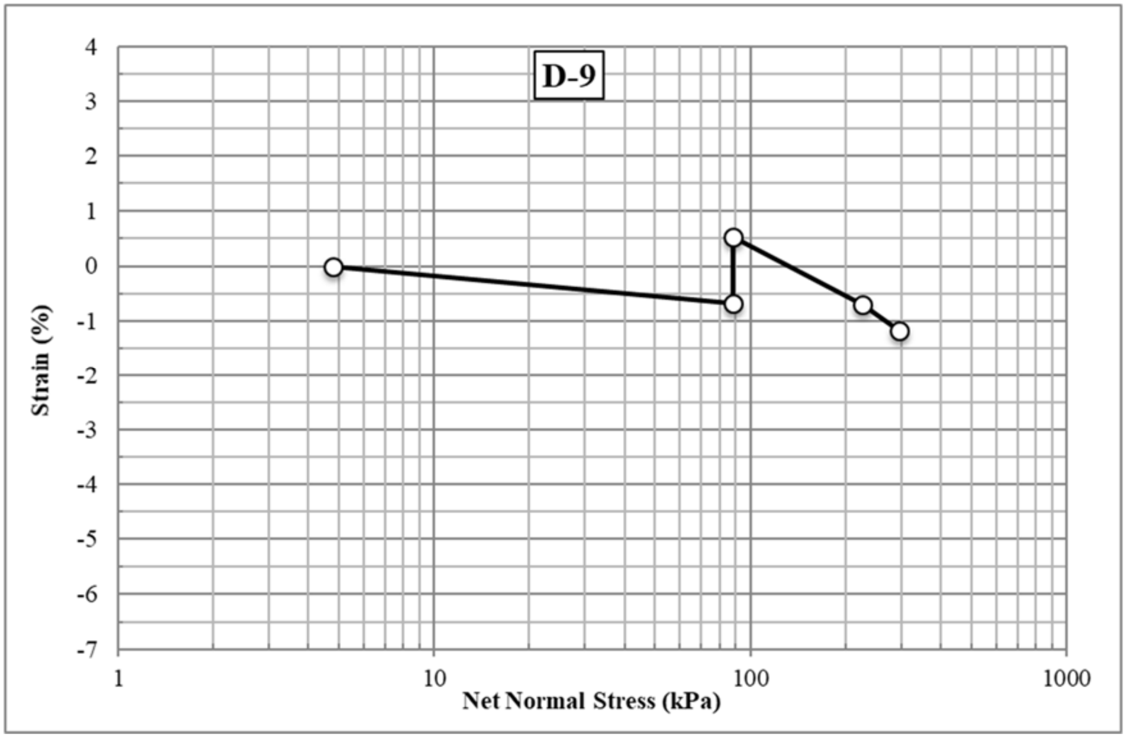


Figure 48: Fully Wetted Oedometer Test (ASTM D4546) for D-9

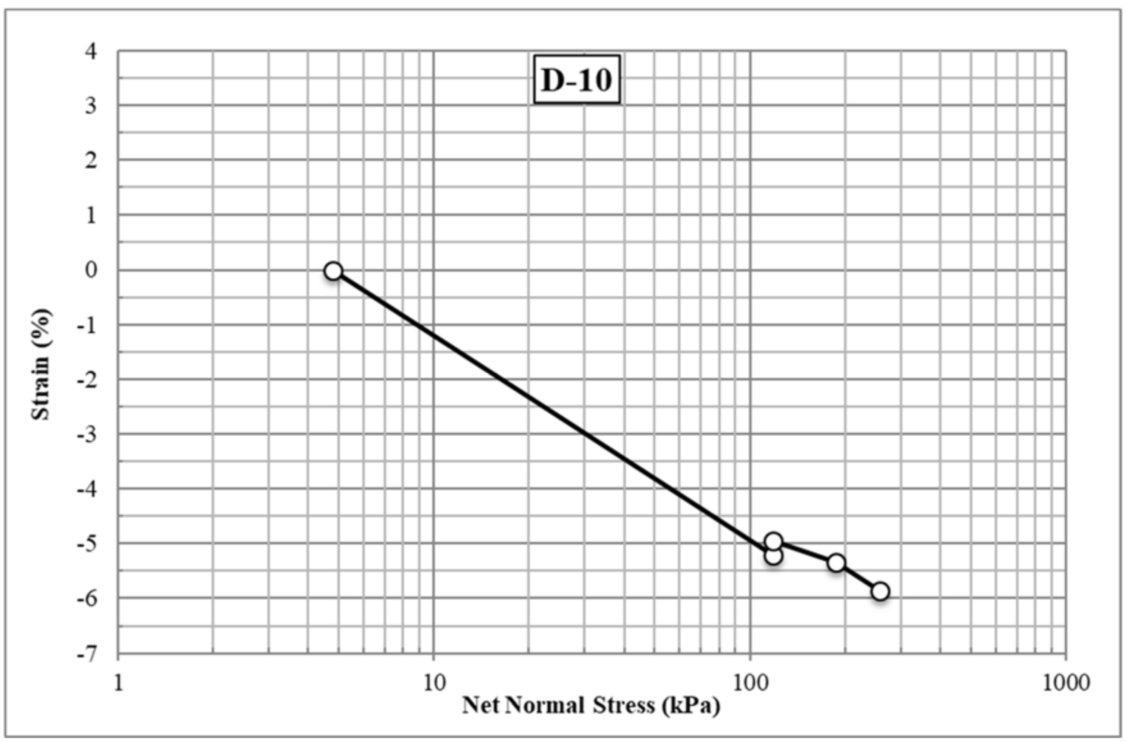


Figure 49: Fully Wetted Oedometer Test (ASTM D4546) for D-10

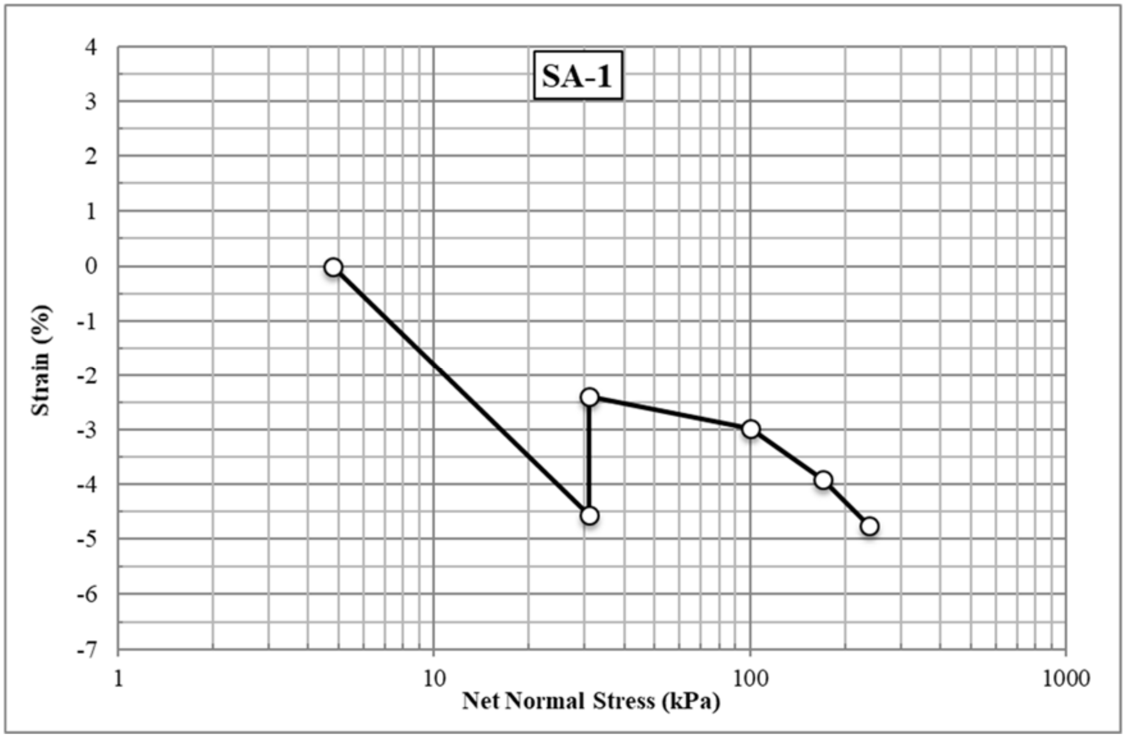


Figure 50: Fully Wetted Oedometer Test (ASTM D4546) for SA-1

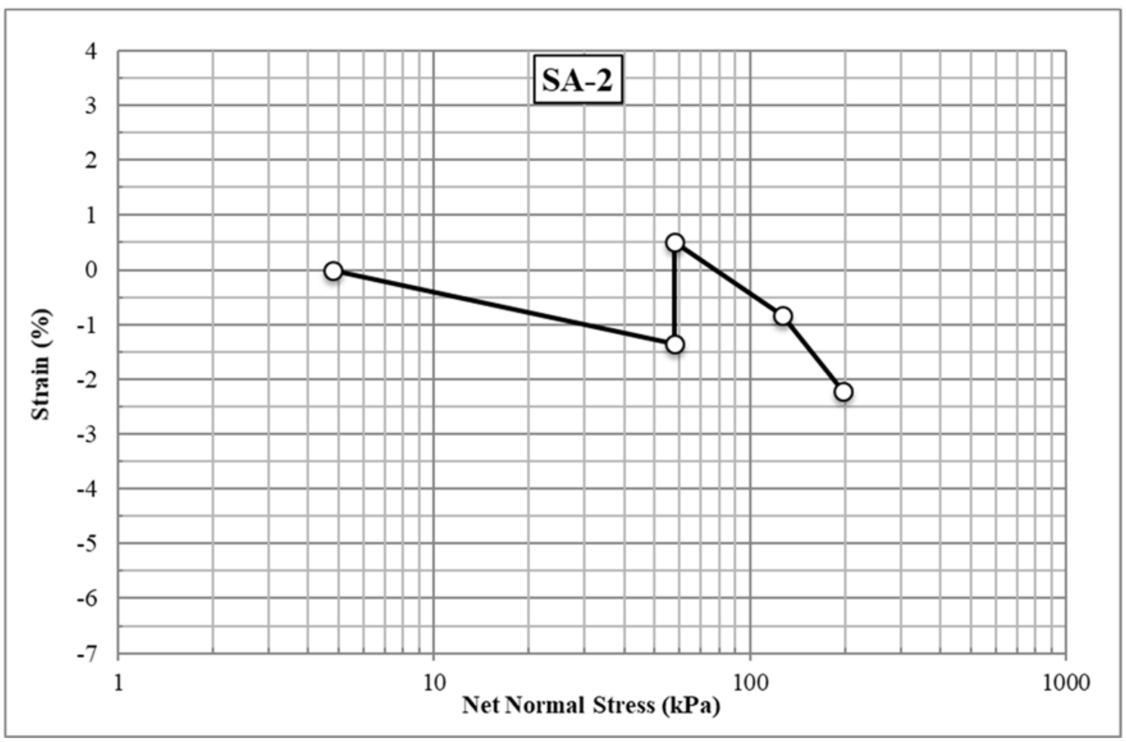


Figure 51: Fully Wetted Oedometer Test (ASTM D4546) for SA-2

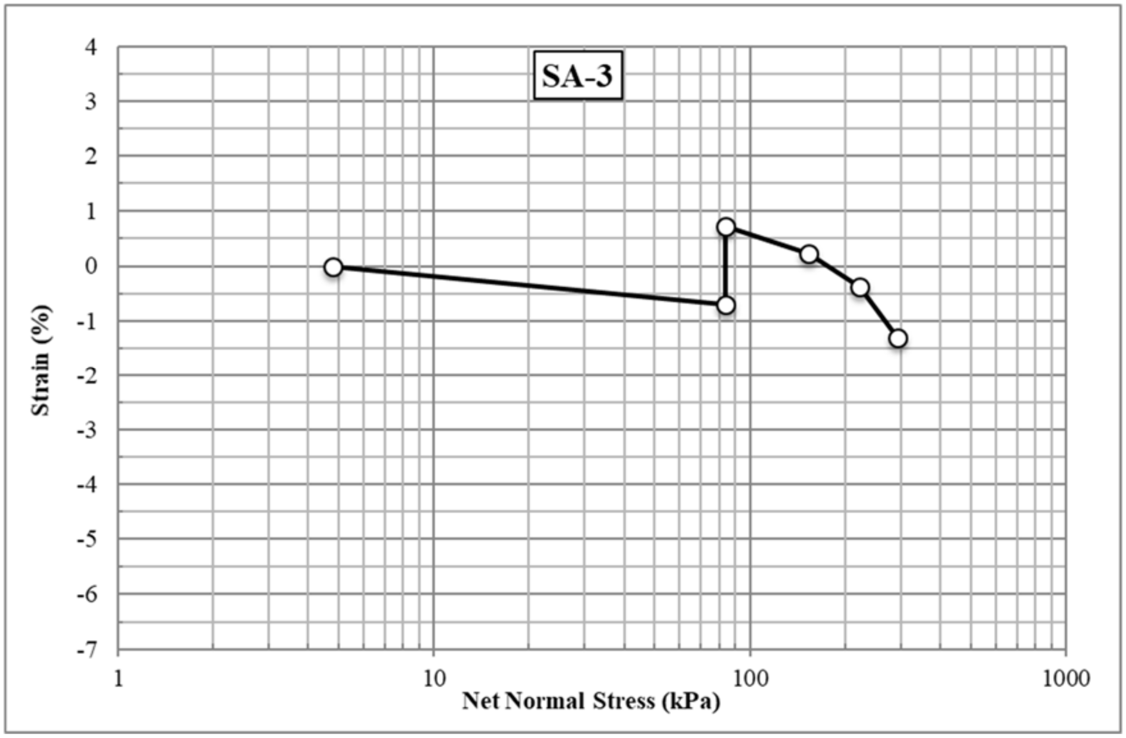


Figure 52: Fully Wetted Oedometer Test (ASTM D4546) for SA-3

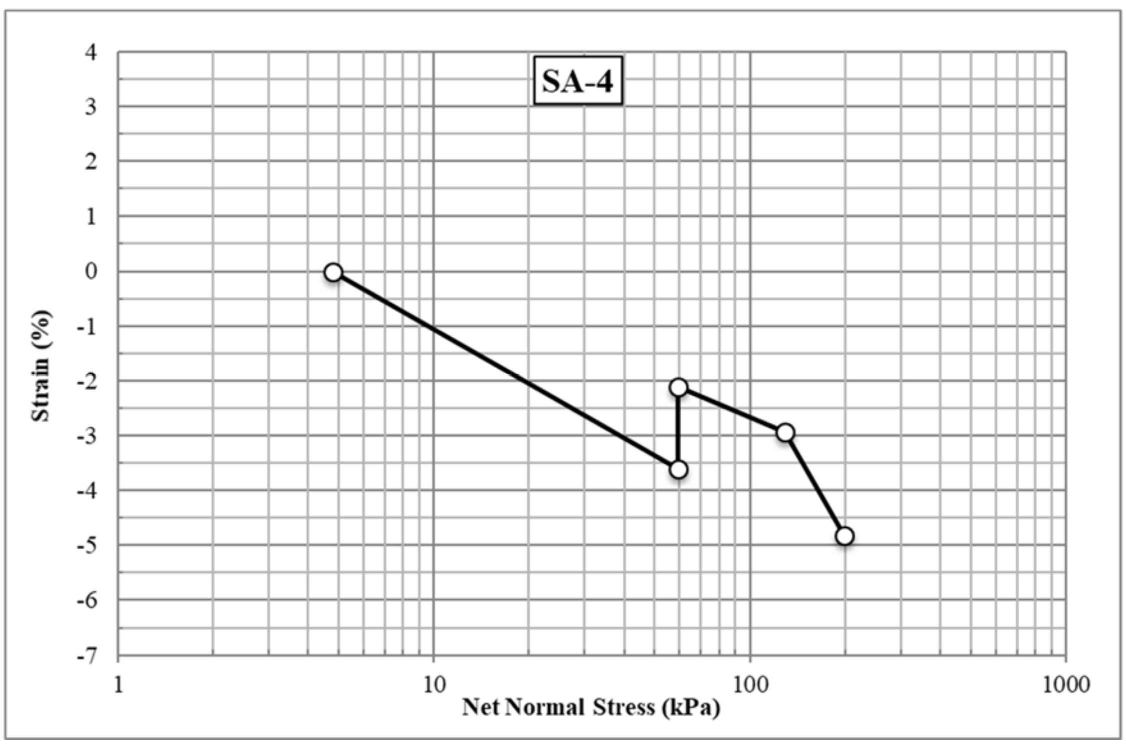


Figure 53: Fully Wetted Oedometer Test (ASTM D4546) for SA-4

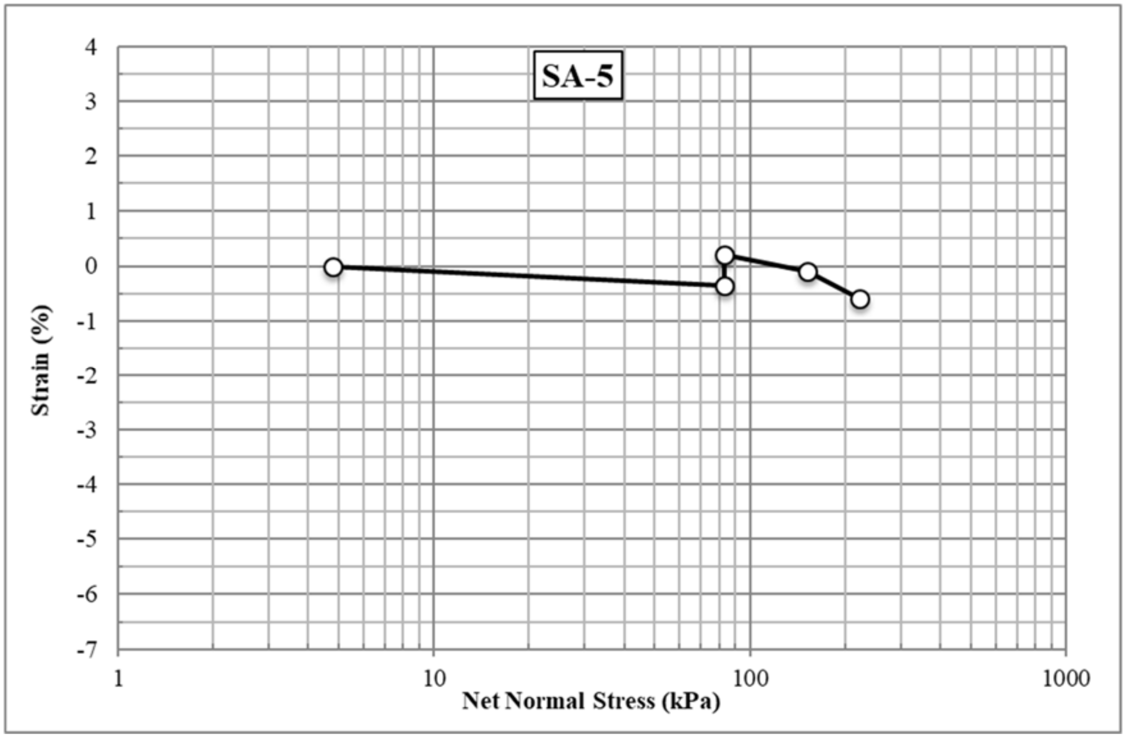


Figure 54: Fully Wetted Oedometer Test (ASTM D4546) for SA-5

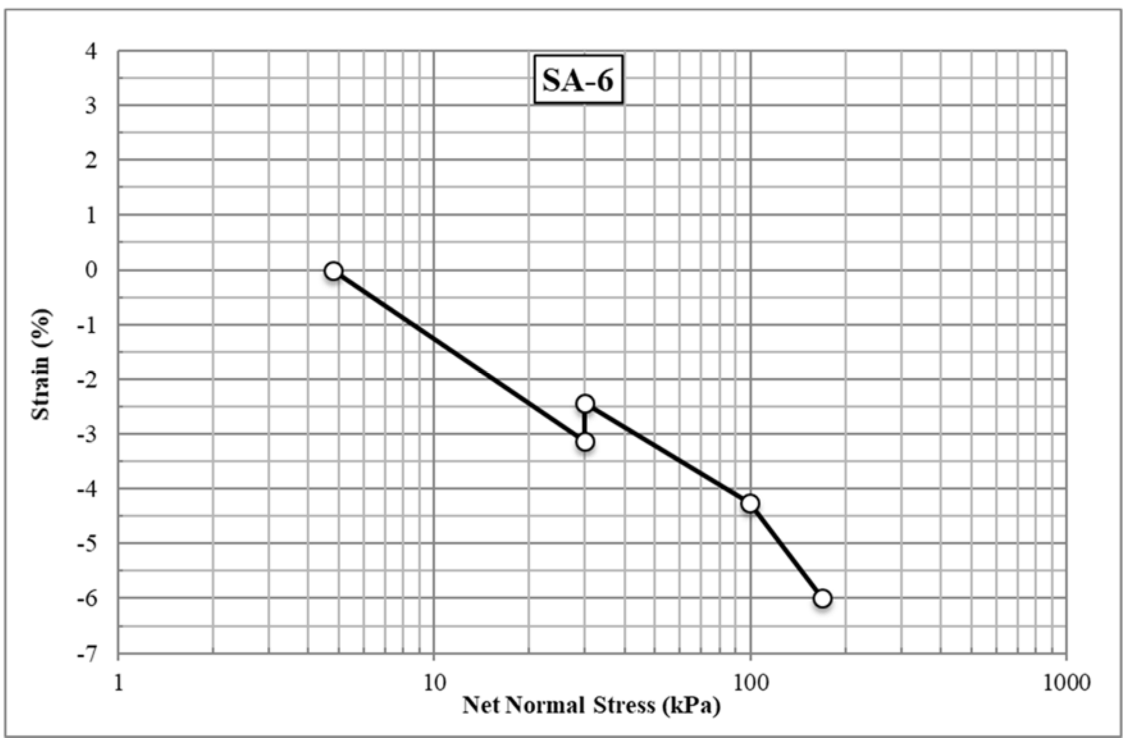


Figure 55: Fully Wetted Oedometer Test (ASTM D4546) for SA-6

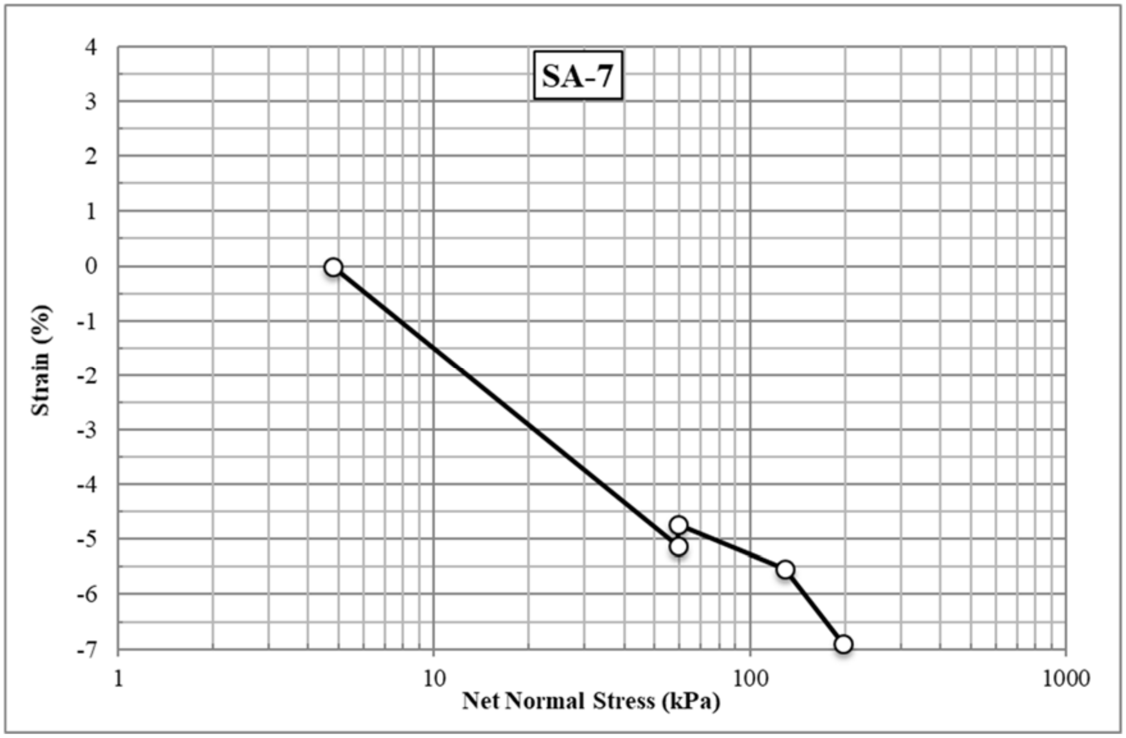


Figure 56: Fully Wetted Oedometer Test (ASTM D4546) for SA-7

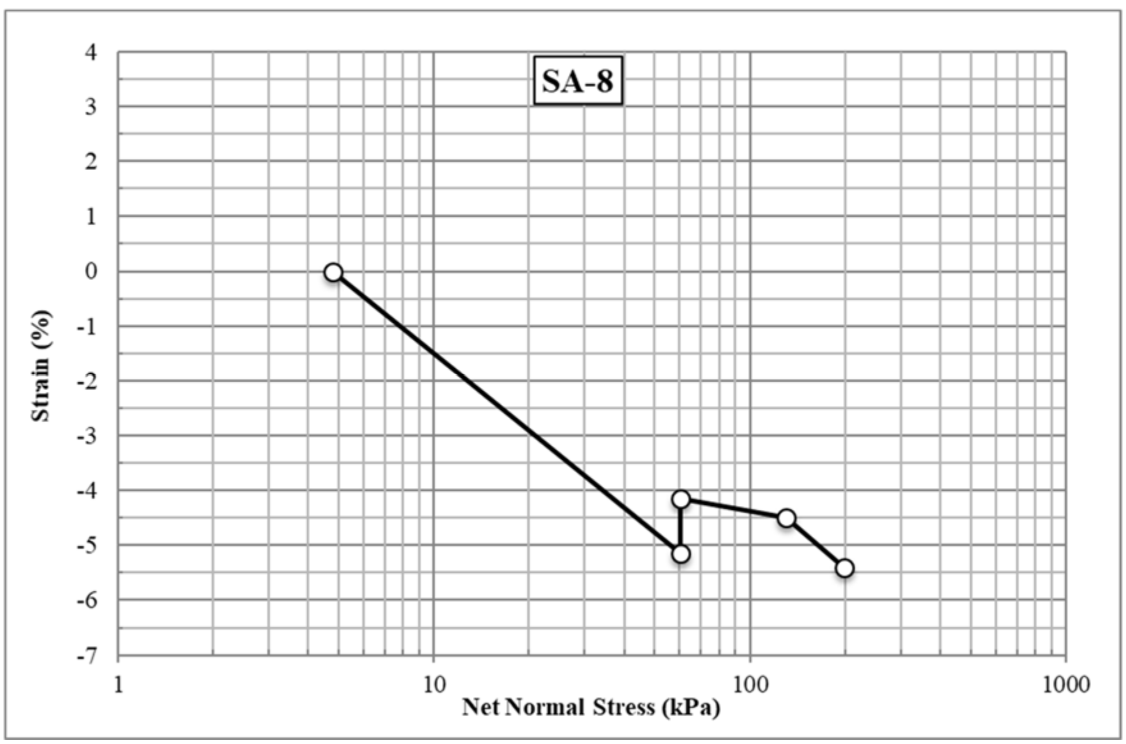


Figure 57: Fully Wetted Oedometer Test (ASTM D4546) for SA-8

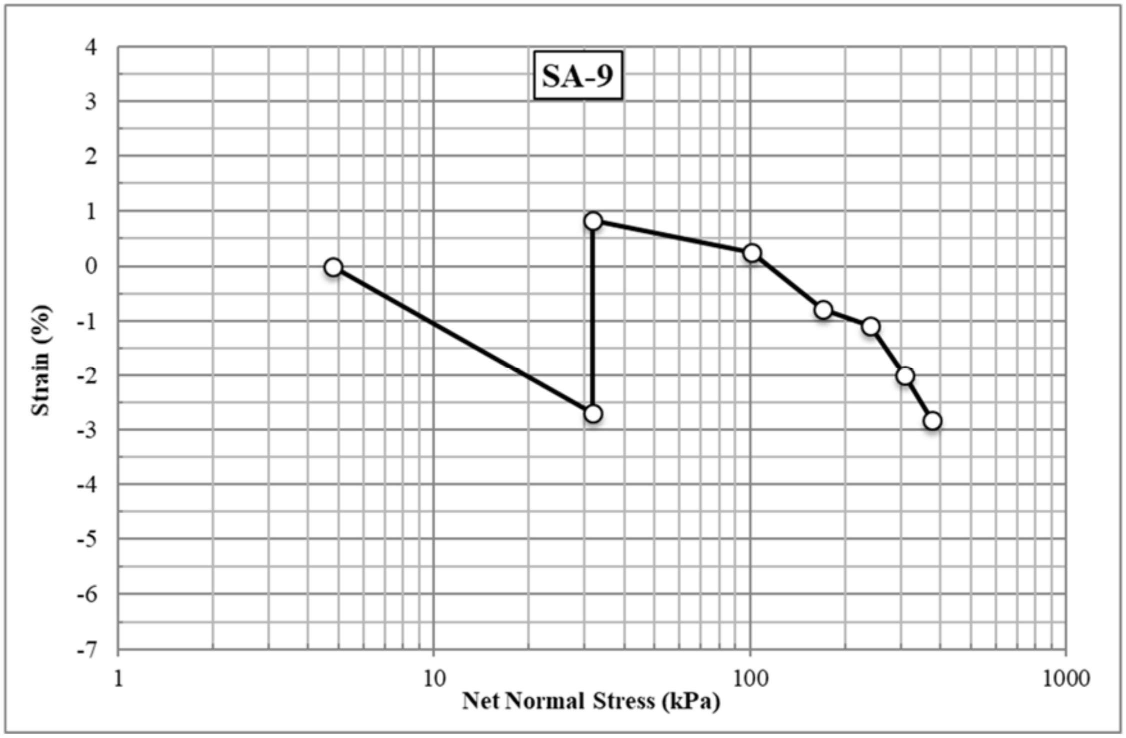


Figure 58: Fully Wetted Oedometer Test (ASTM D4546) for SA-9

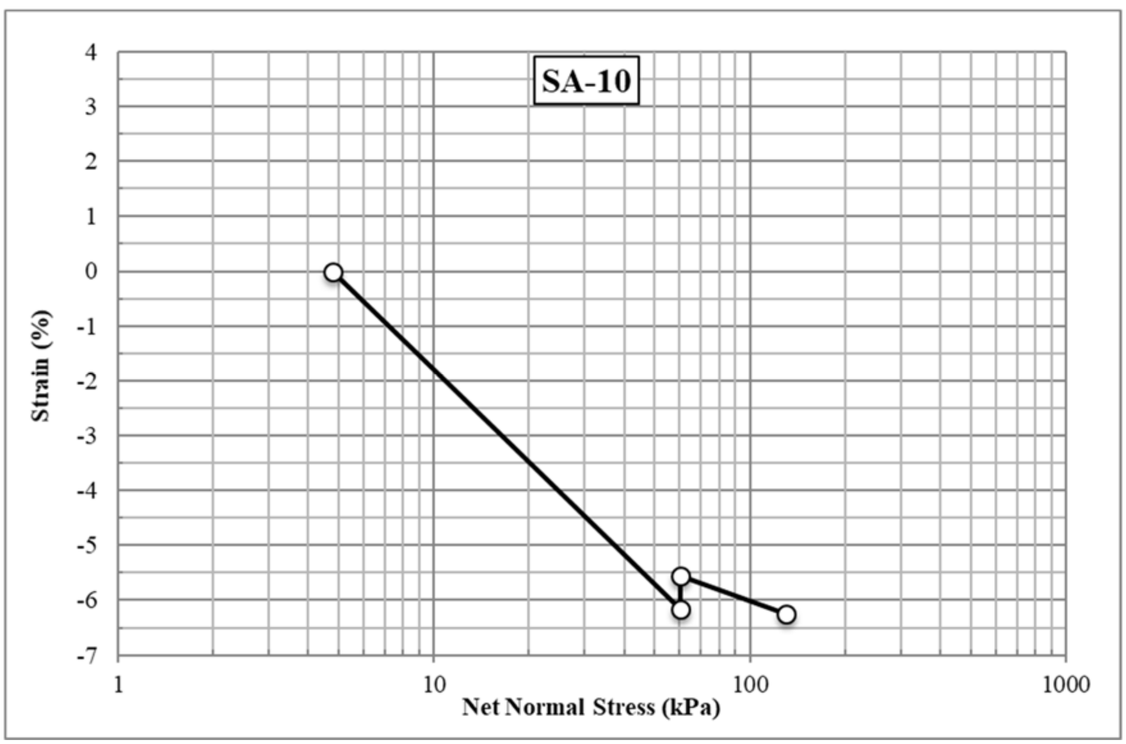


Figure 59: Fully Wetted Oedometer Test (ASTM D4546) for SA-10

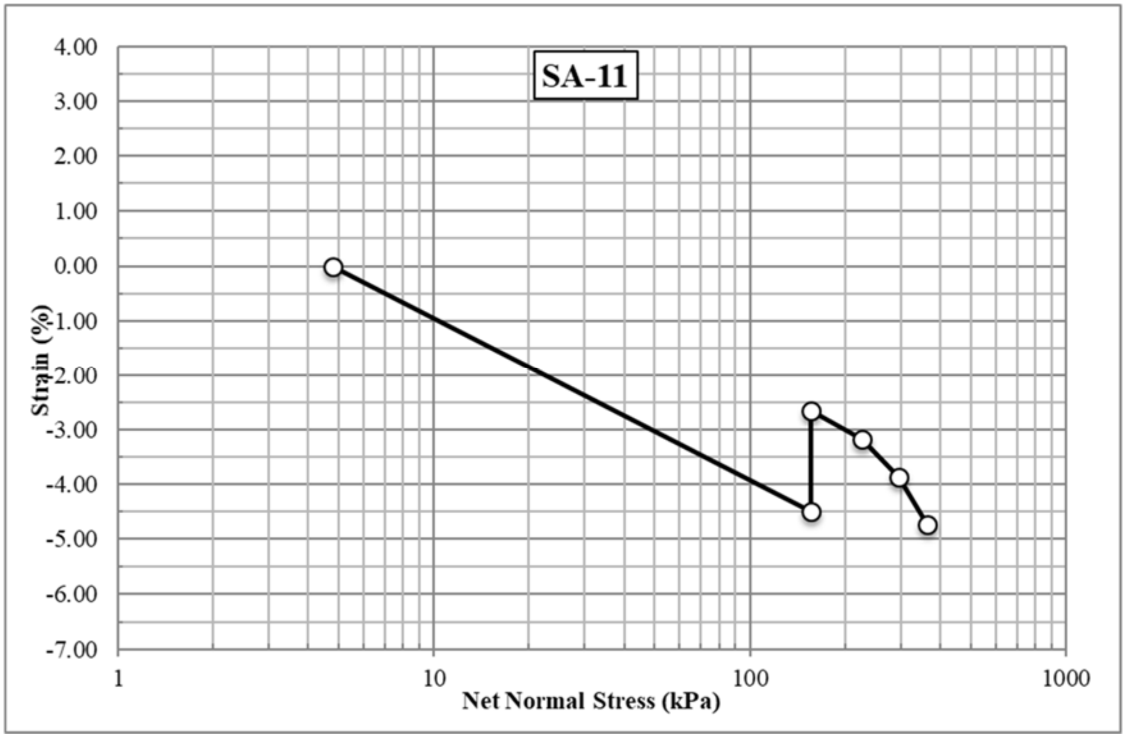


Figure 60: Fully Wetted Oedometer Test (ASTM D4546) for SA-11

APPENDIX

B SWCC Results

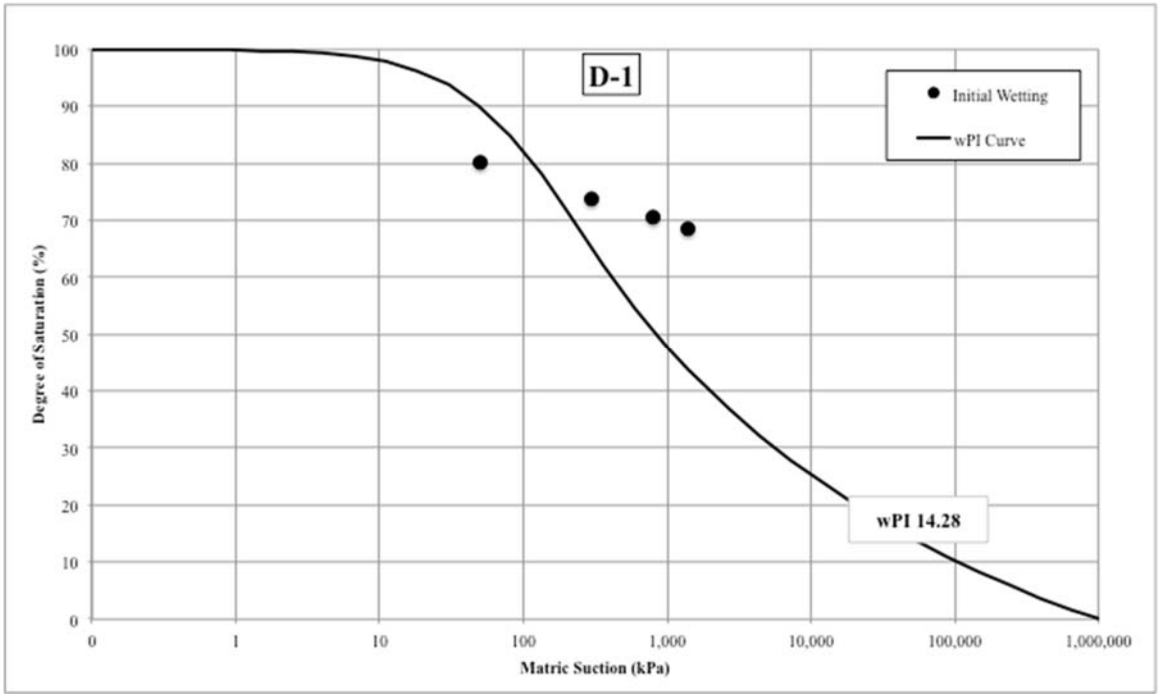


Figure 61: SWCC Data for D-1

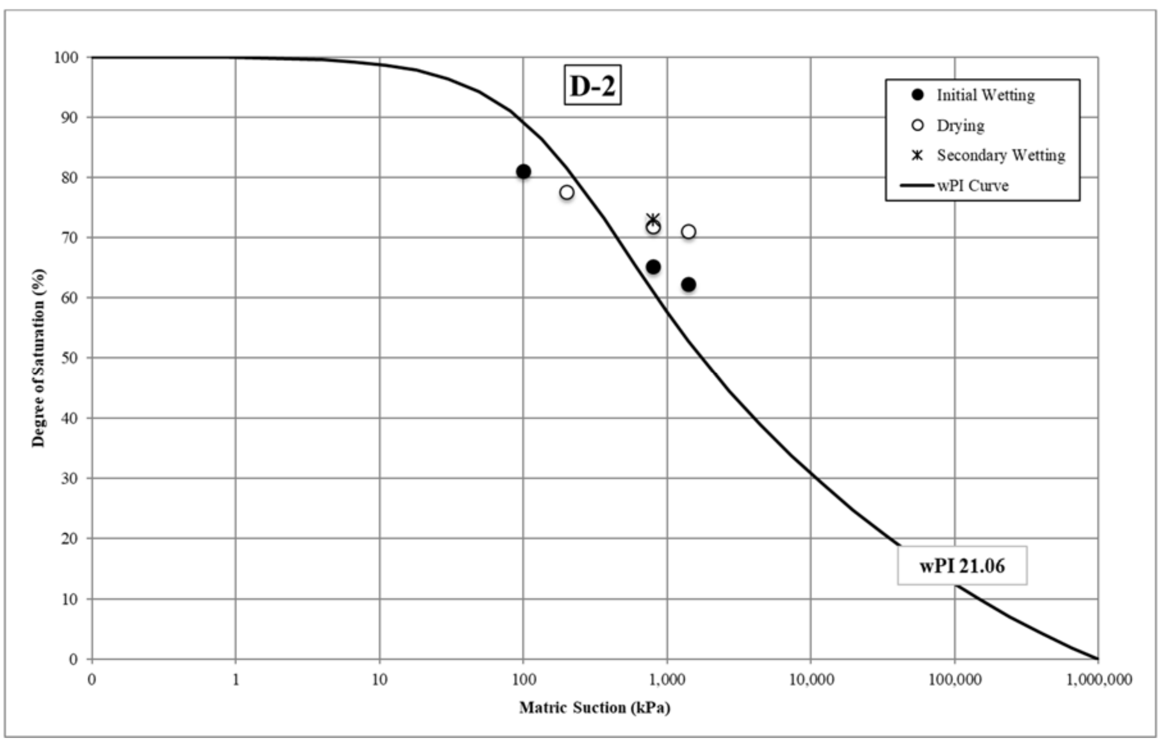


Figure 62: SWCC Data for D-2

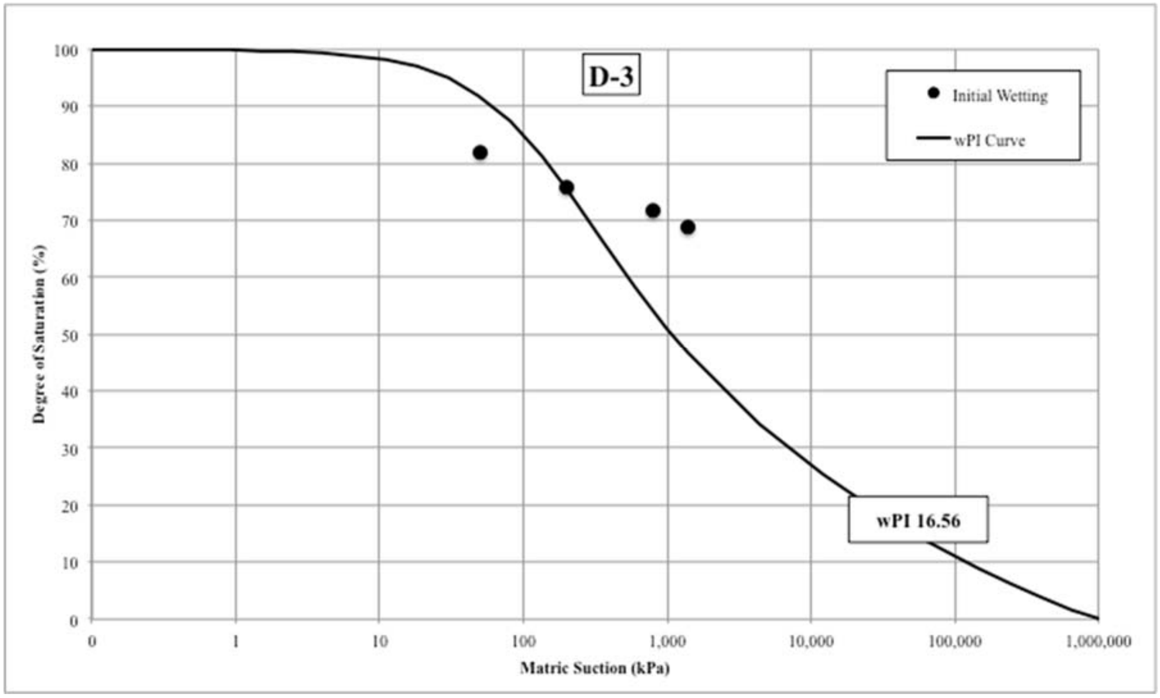


Figure 63: SWCC Data for D-4

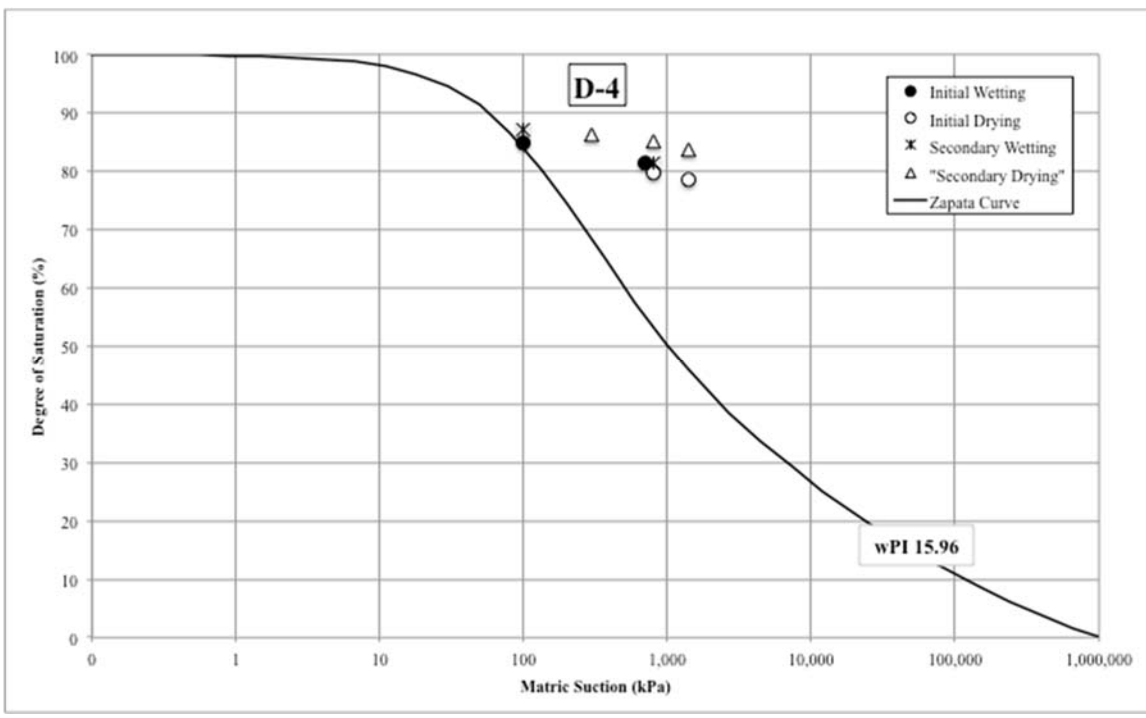


Figure 64: SWCC Data for D-4

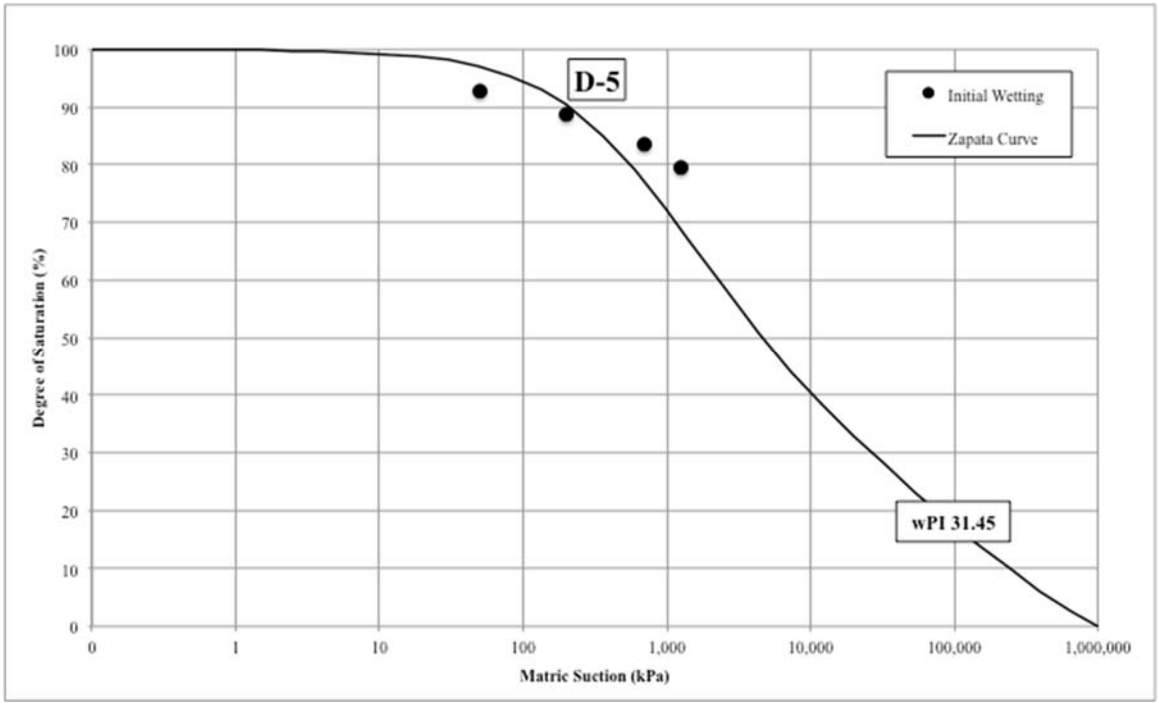


Figure 65: SWCC Data for D-5

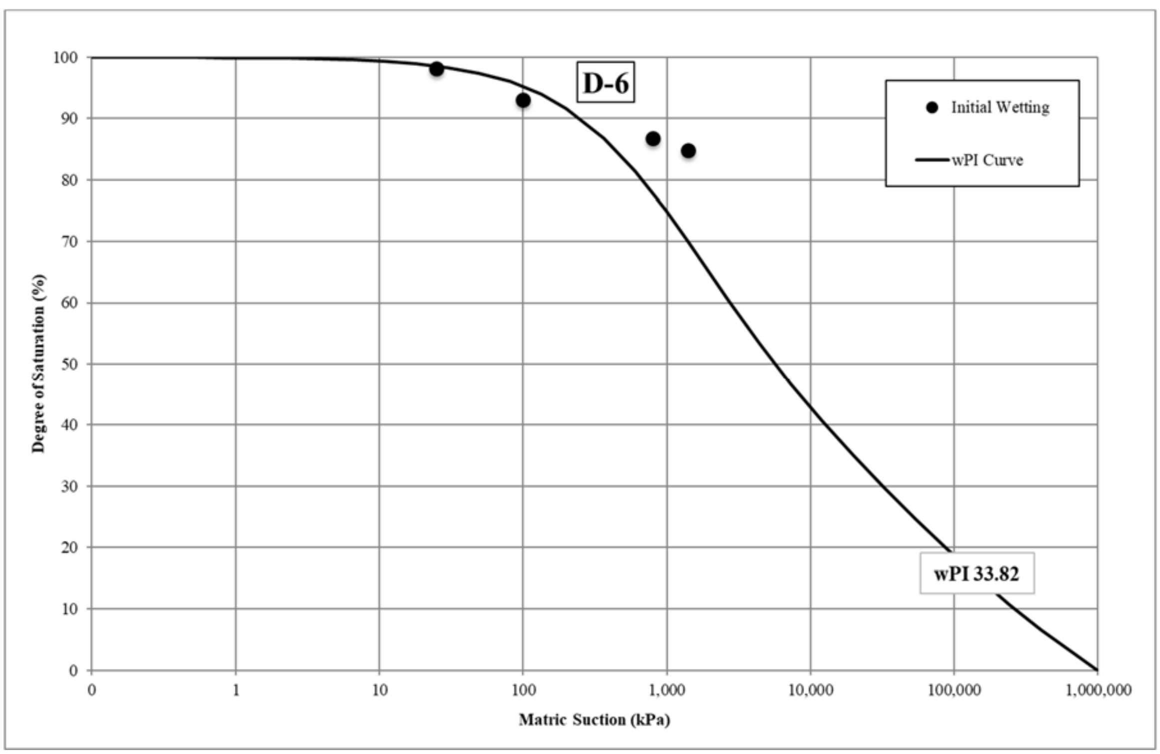


Figure 66: SWCC Data for D-6

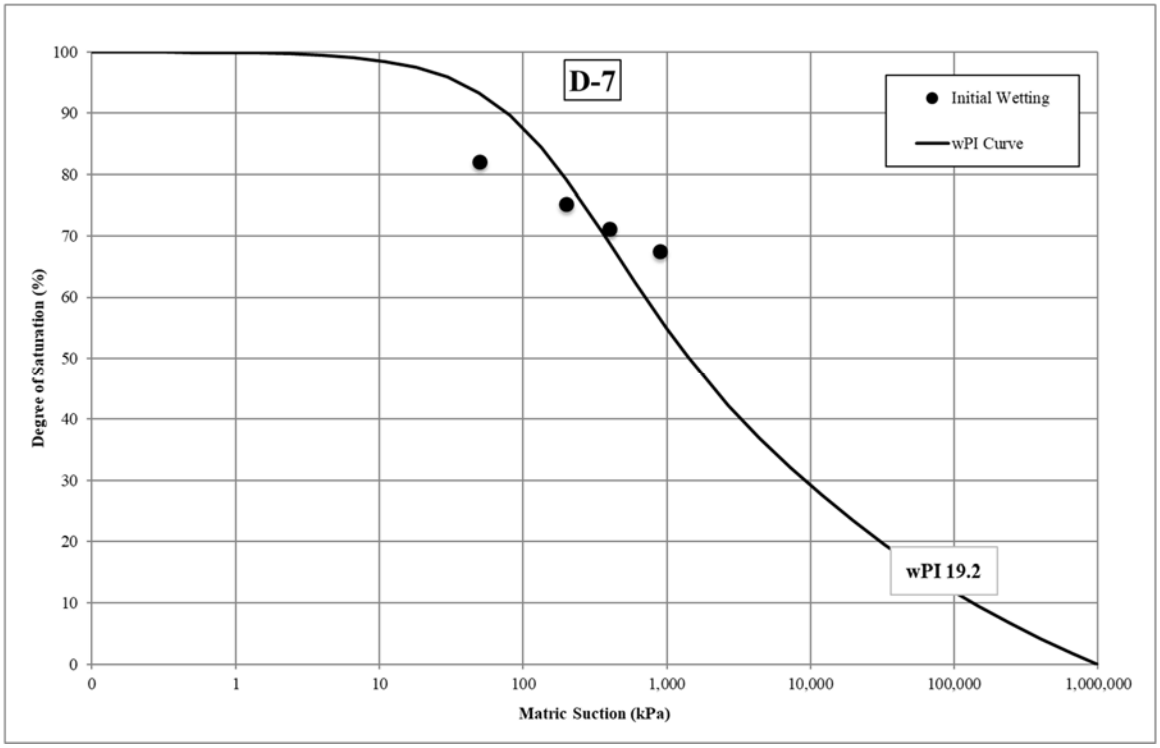


Figure 67: SWCC Data for D-7

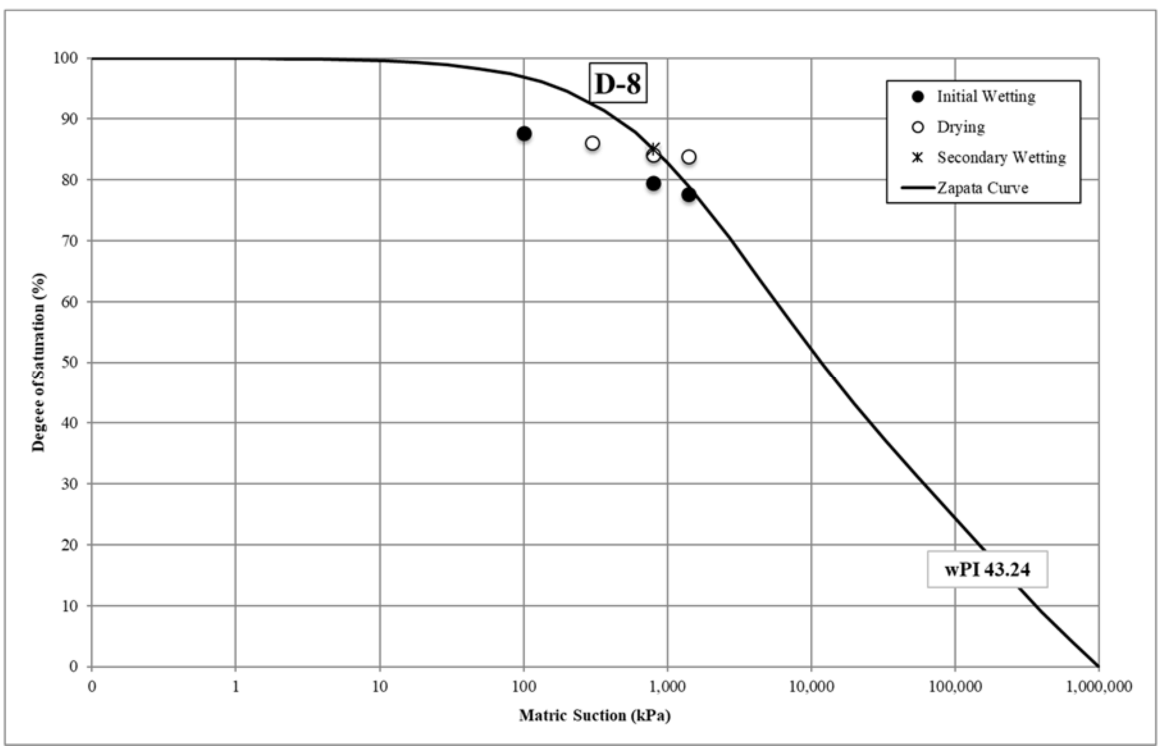


Figure 68: SWCC Data for D-8

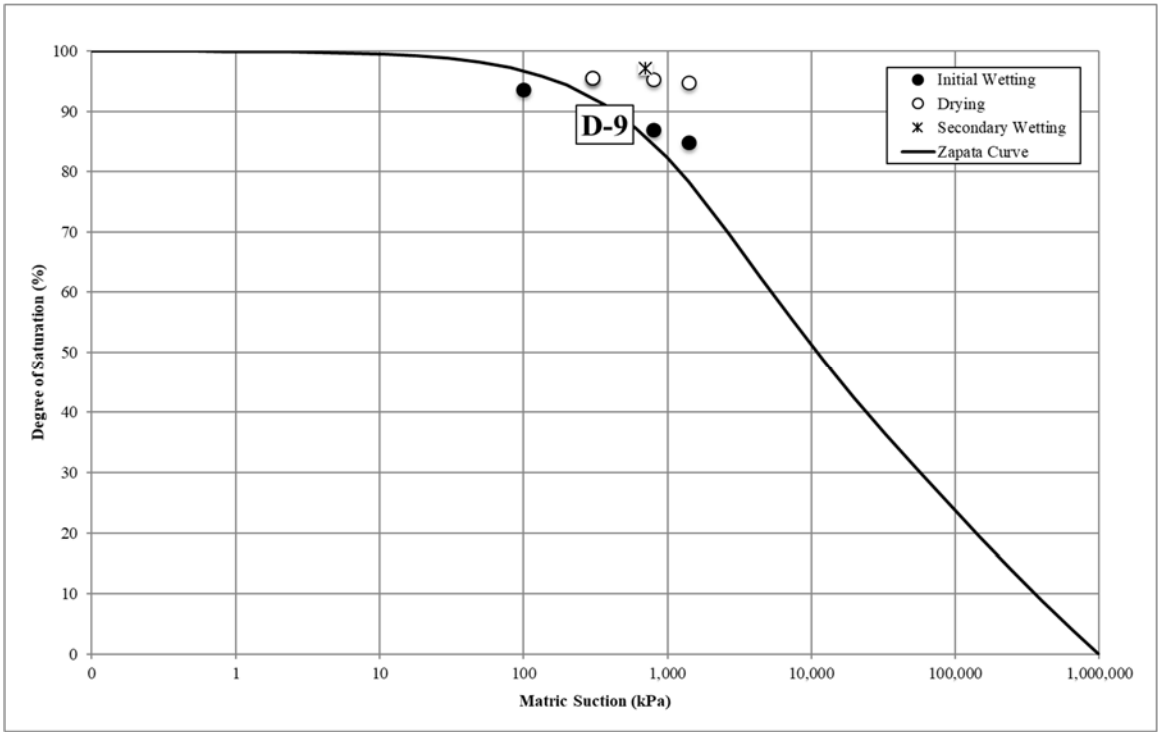


Figure 69: SWCC Data for D-9

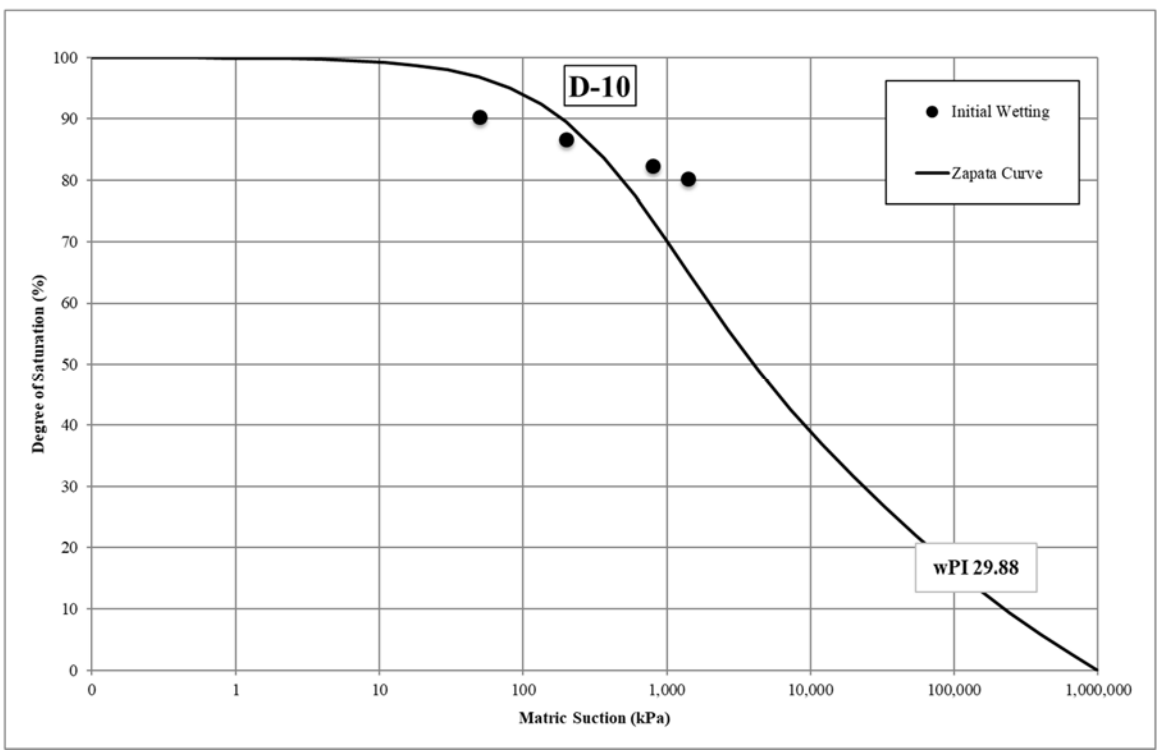


Figure 70: SWCC Data for D-10

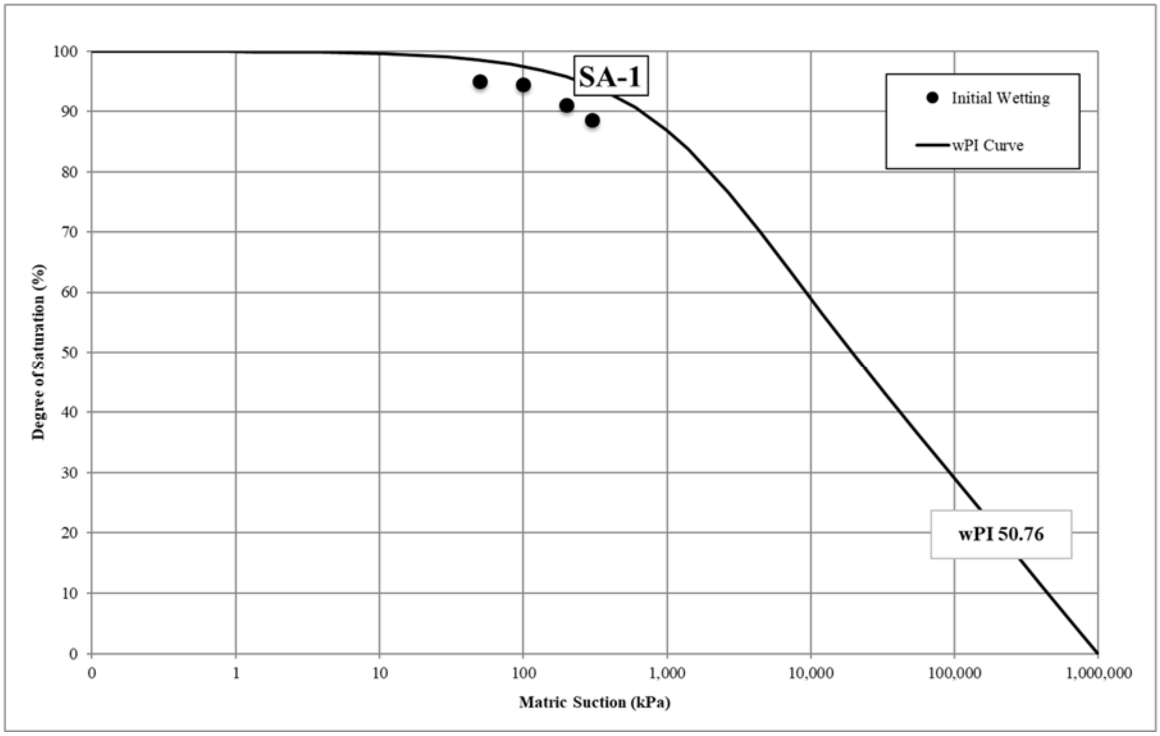


Figure 71: SWCC Data for SA-1

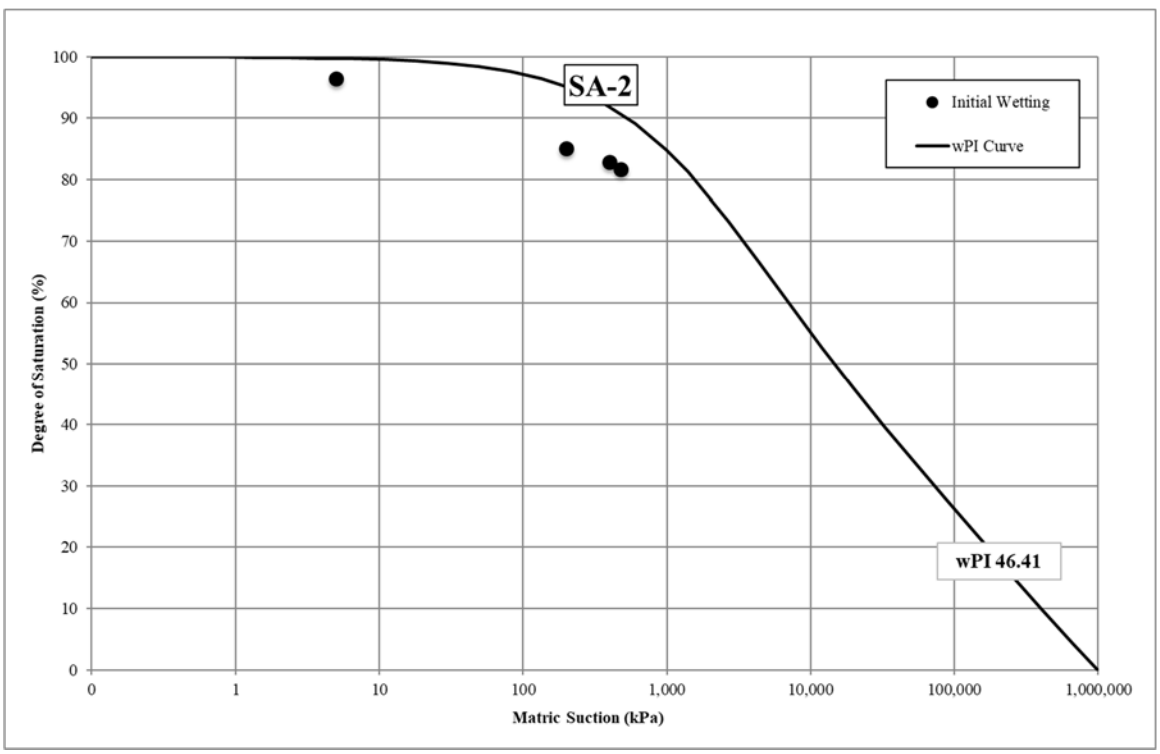


Figure 72: SWCC Data for SA-2

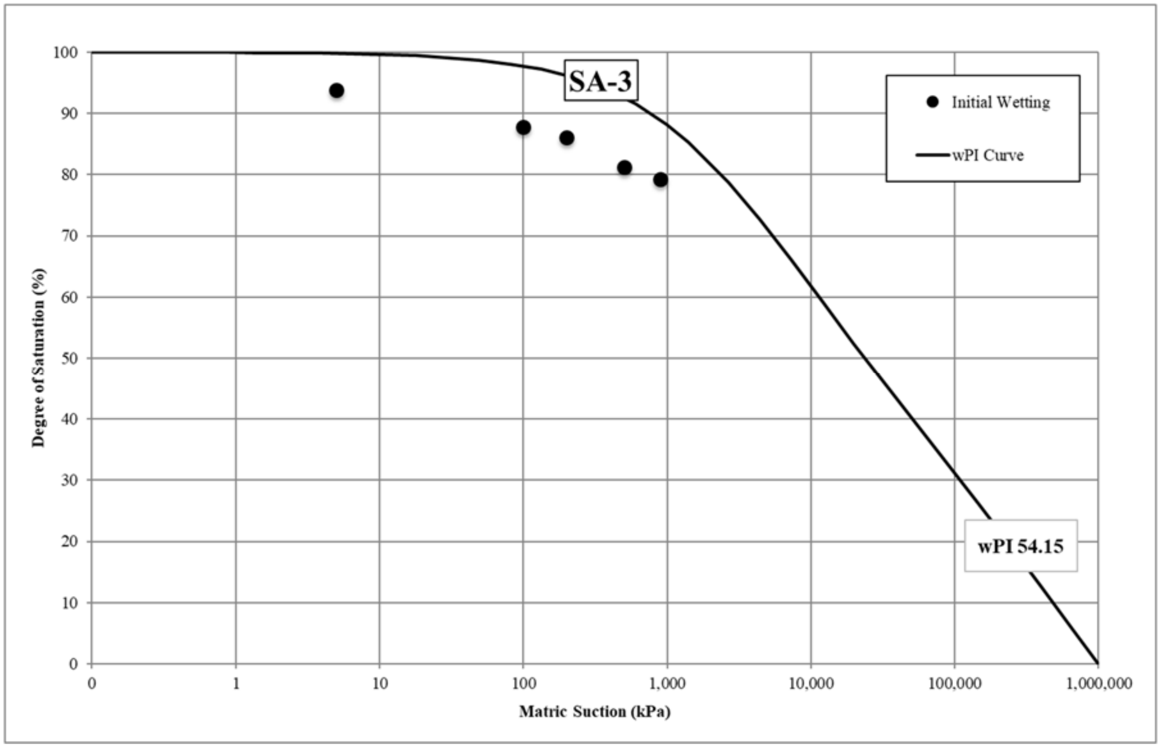


Figure 73: SWCC Data for SA-3

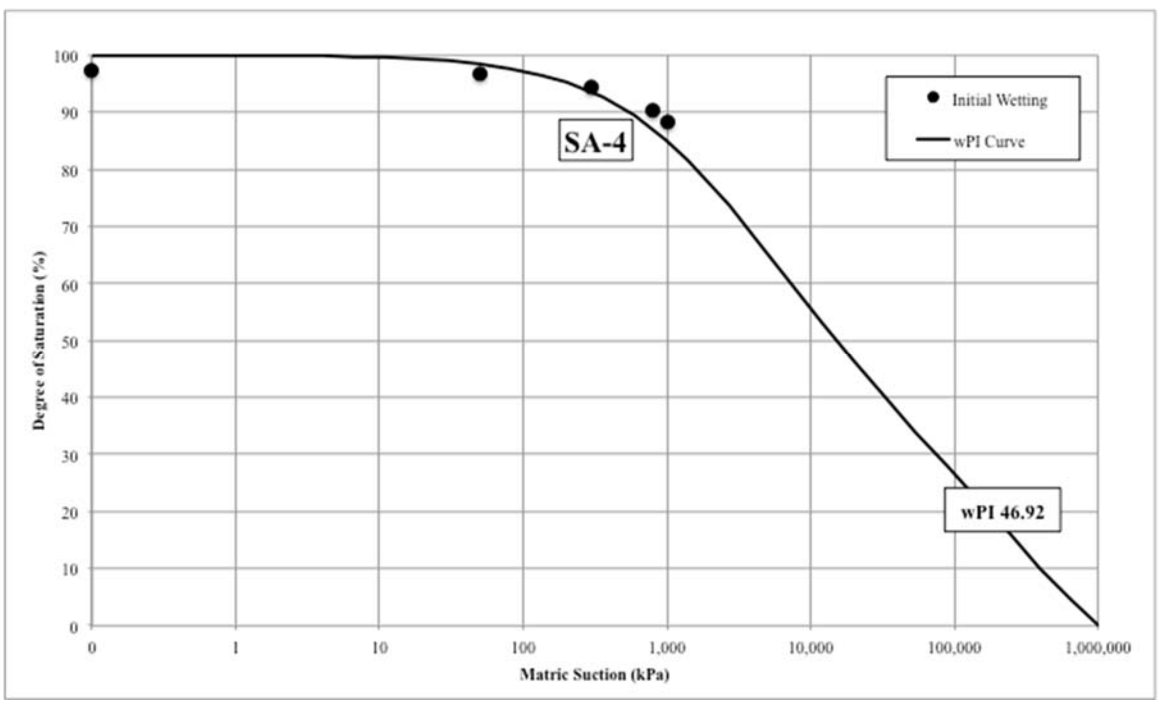


Figure 74: SWCC Data for SA-4

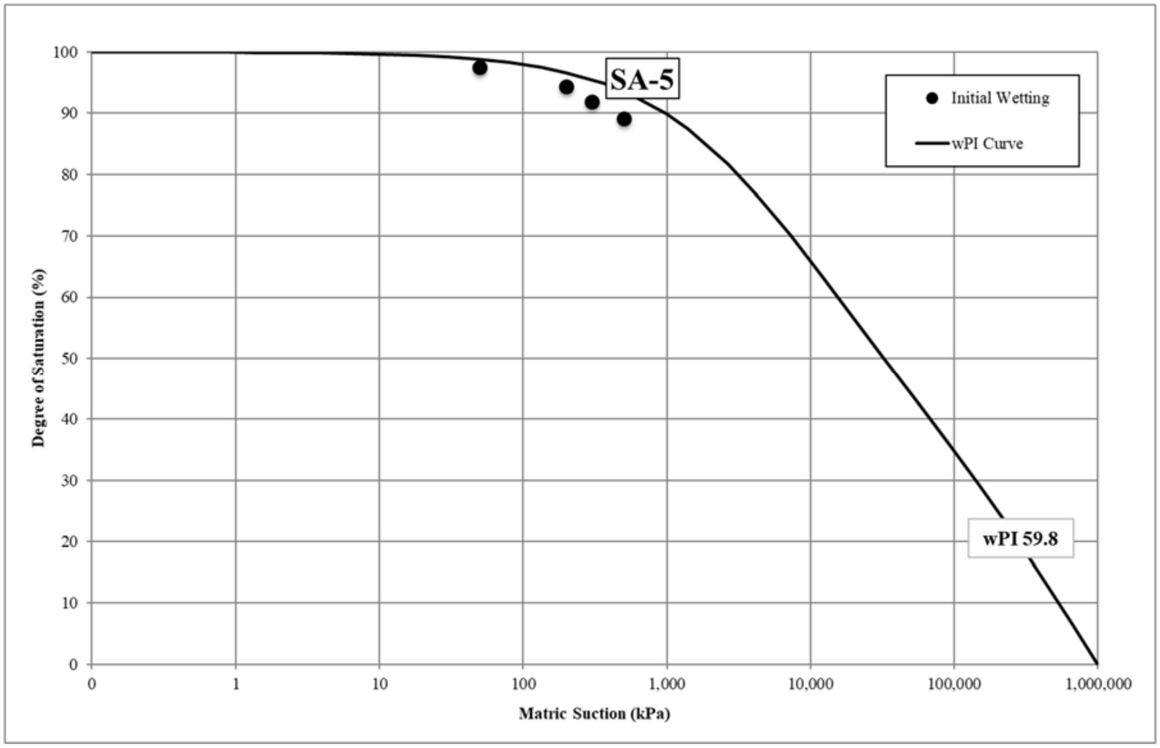


Figure 75: SWCC Data for SA-5

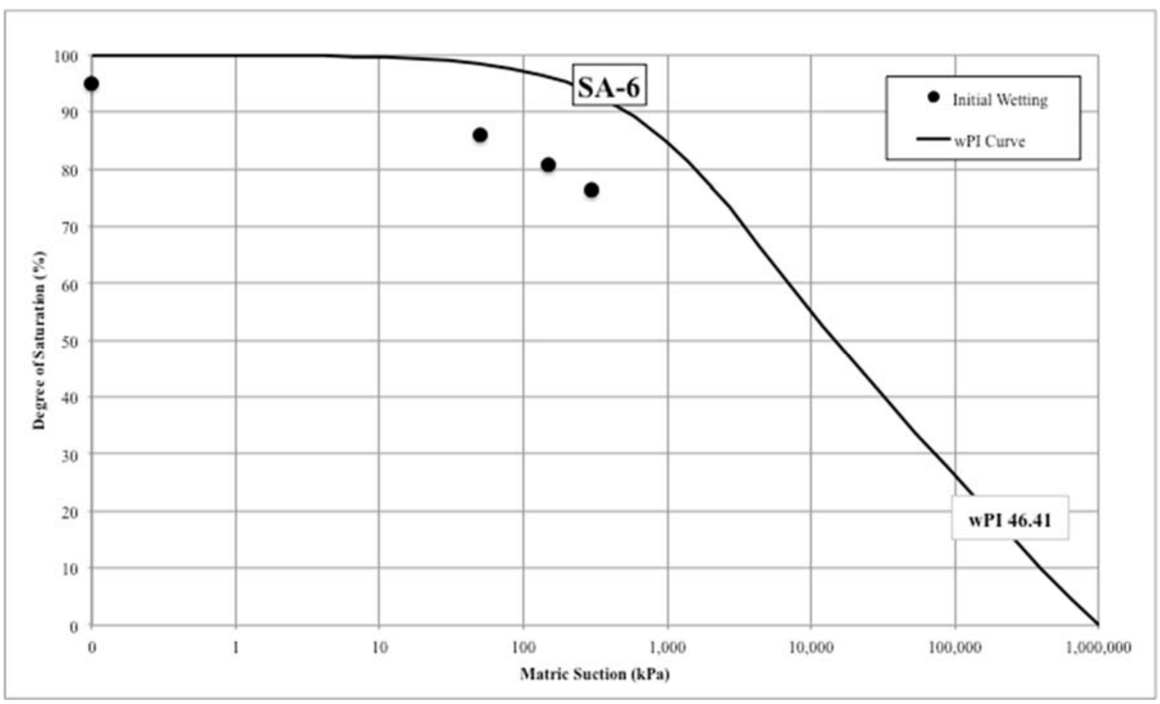


Figure 76: SWCC Data for SA-6

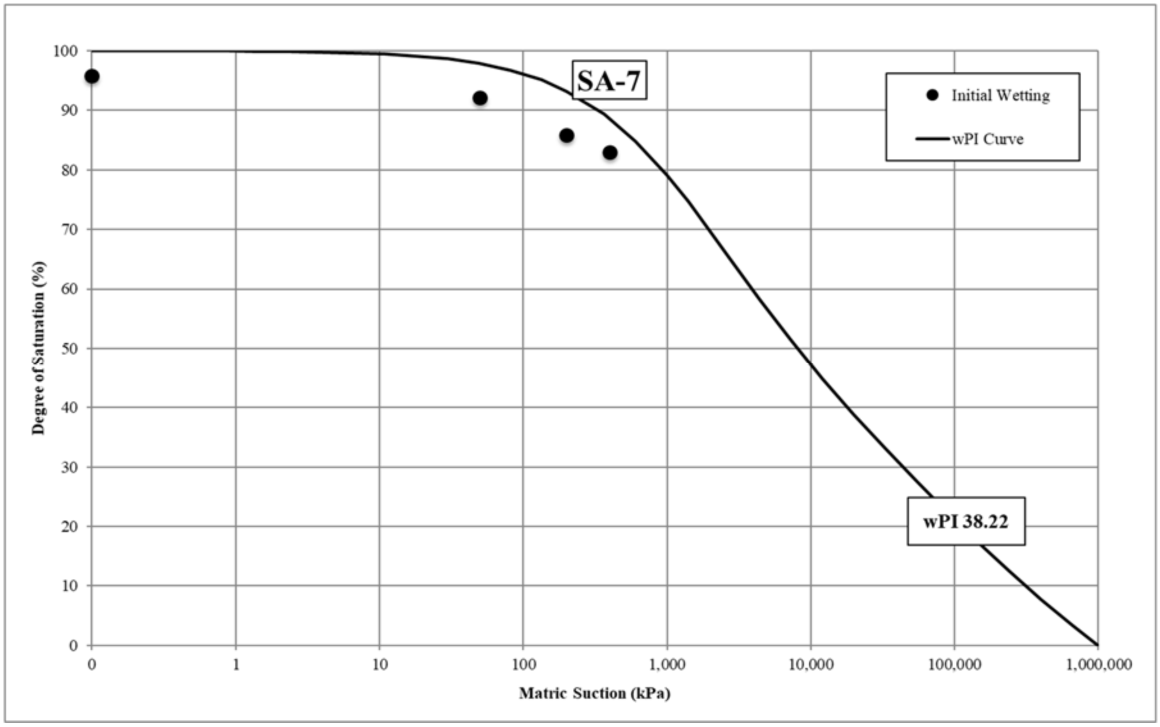


Figure 77: SWCC Data for SA-7

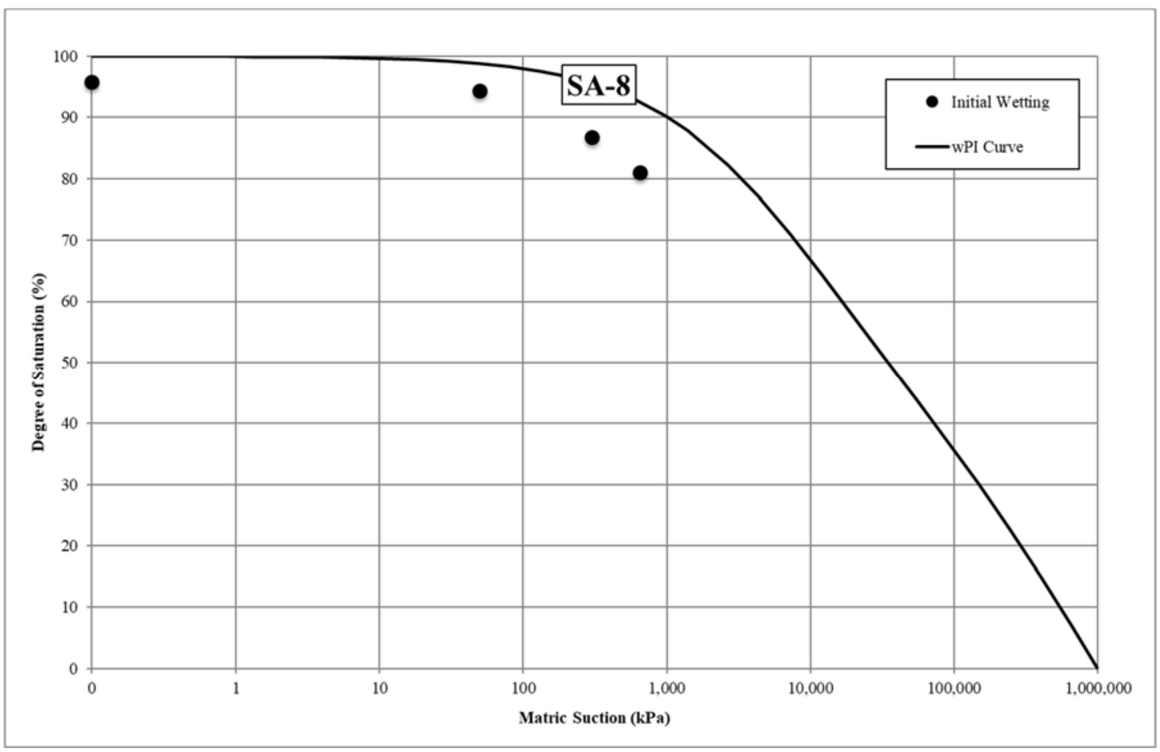


Figure 78: SWCC Data for SA-8

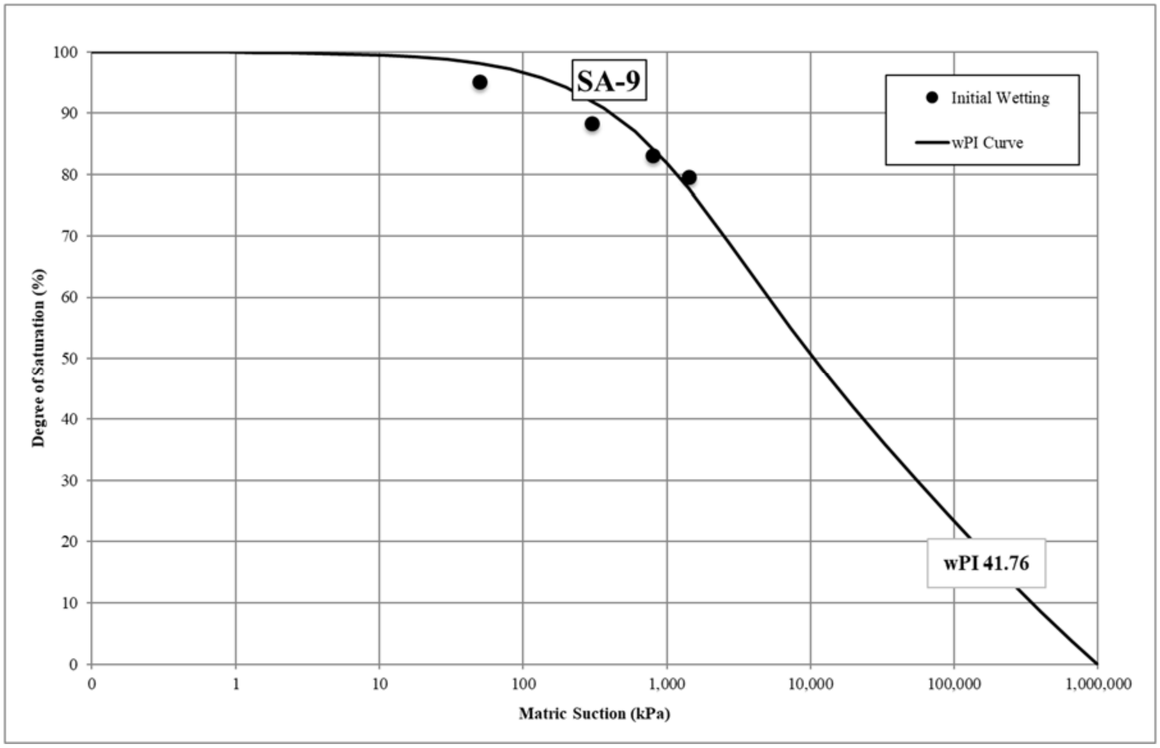


Figure 79: SWCC Data for SA-9

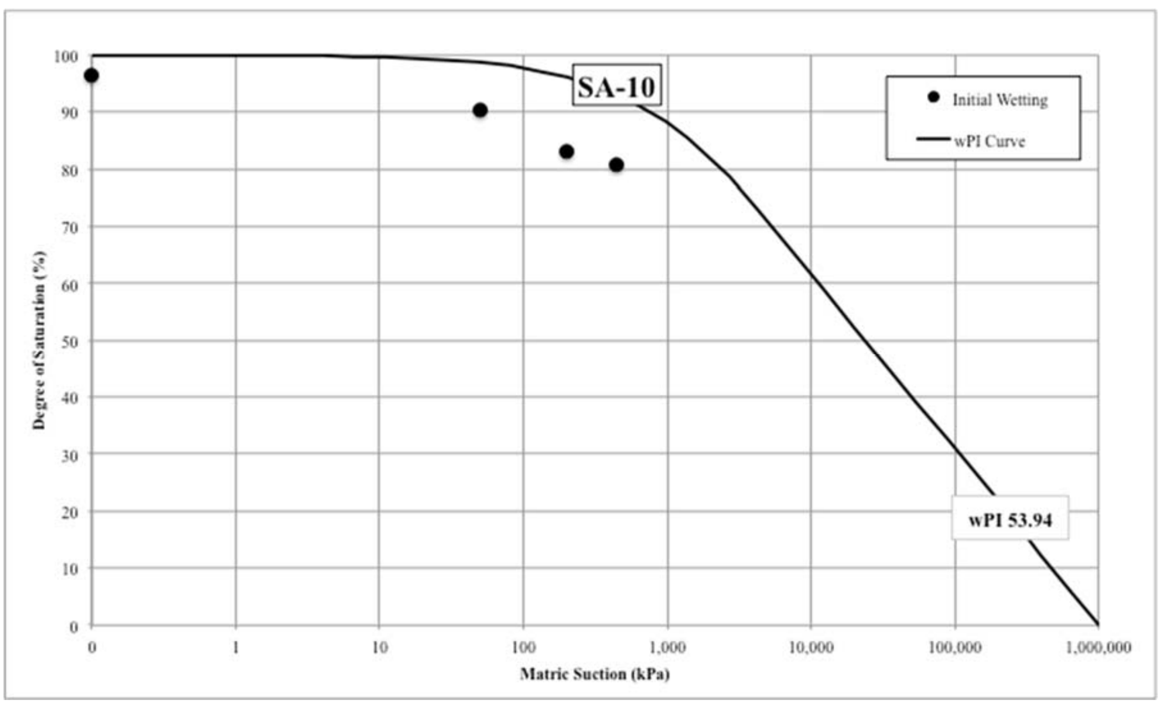


Figure 80: SWCC Data for SA-10

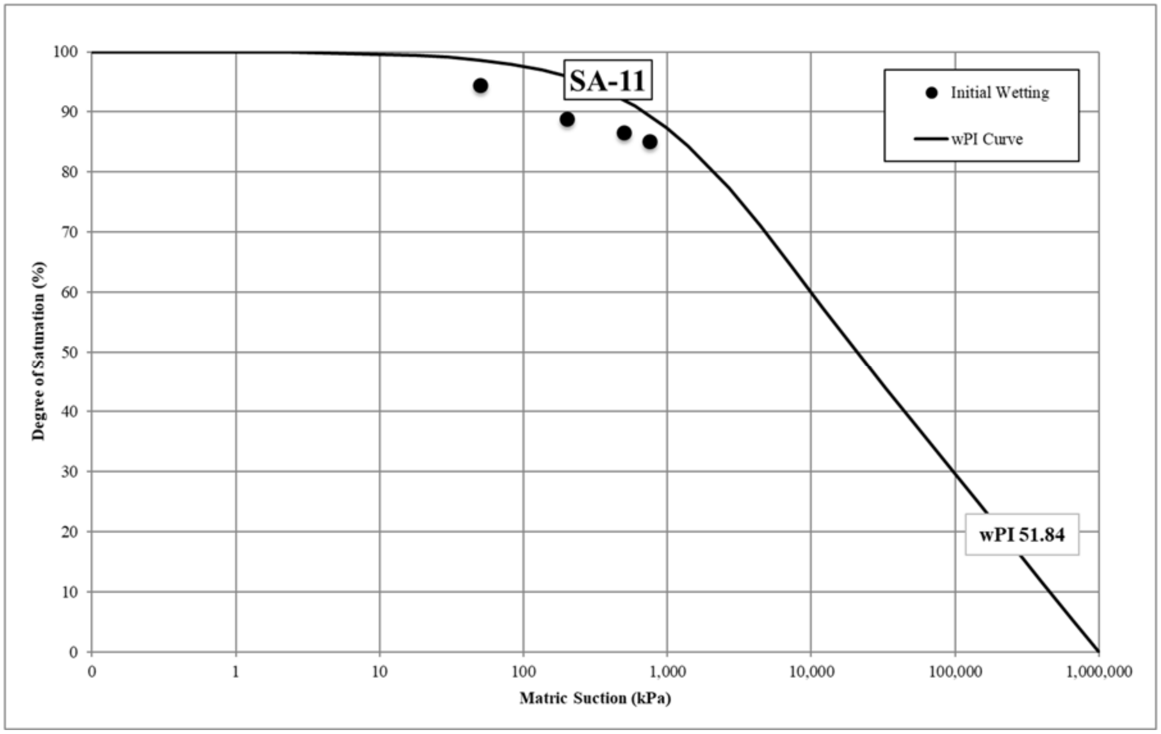


Figure 81: SWCC Data for SA-11

APPENDIX

C Suction Compression Index Results

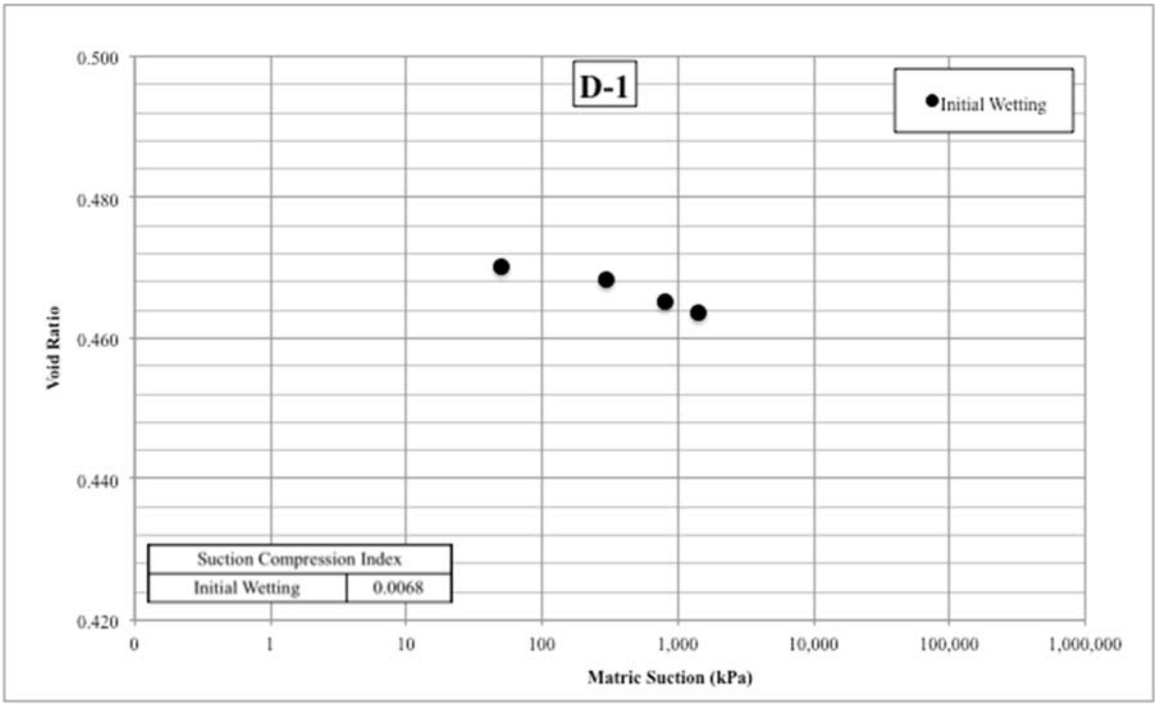


Figure 82: Void Ratio vs. Matric Suction for D-1

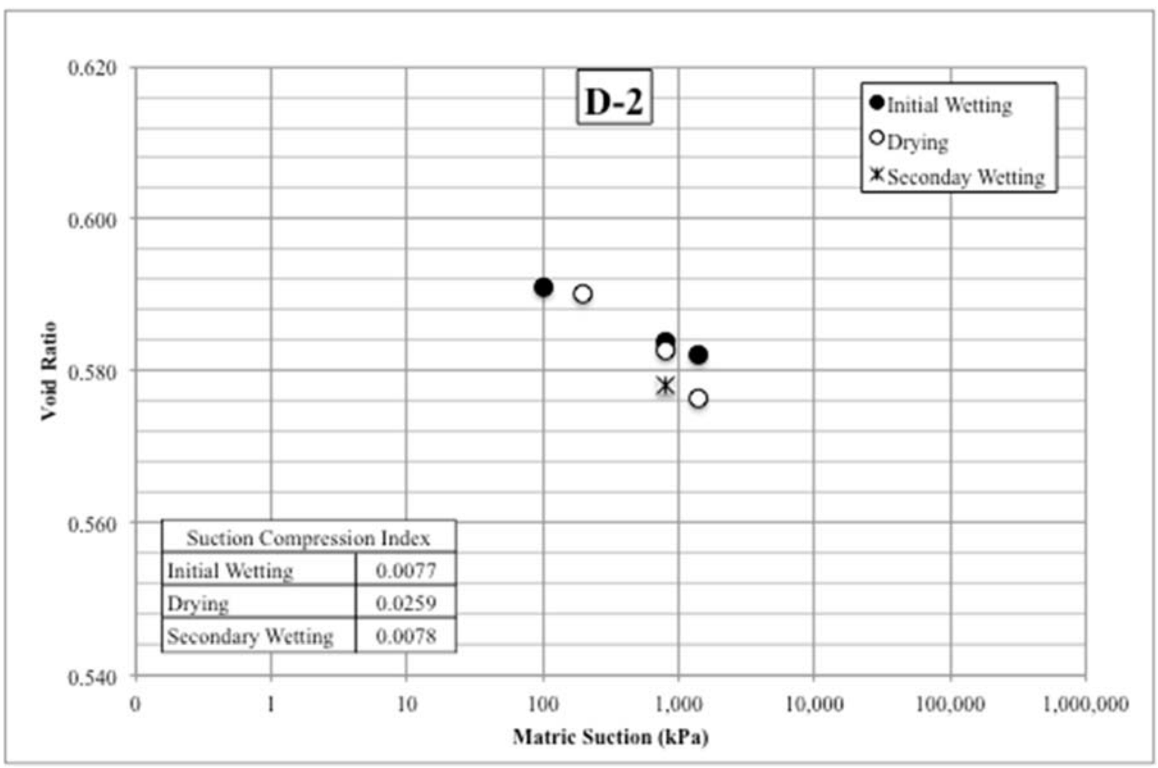


Figure 83: Void Ratio vs. Matric Suction for D-2

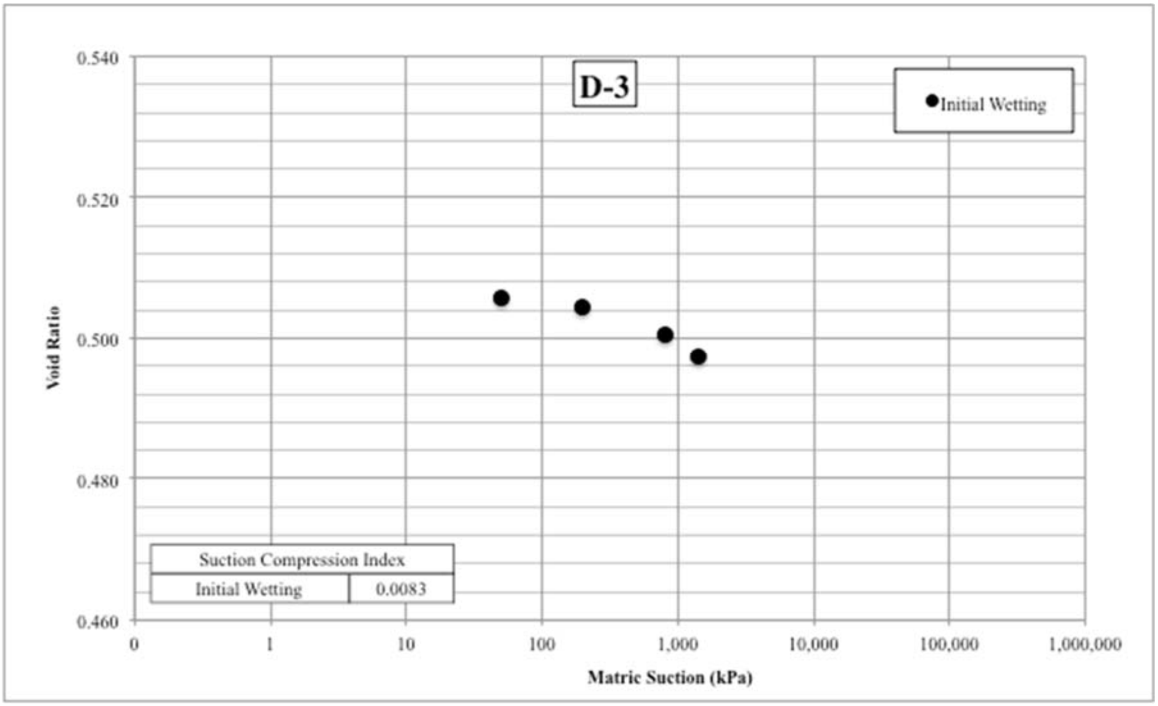


Figure 84: Void Ratio vs. Matric Suction for D-3

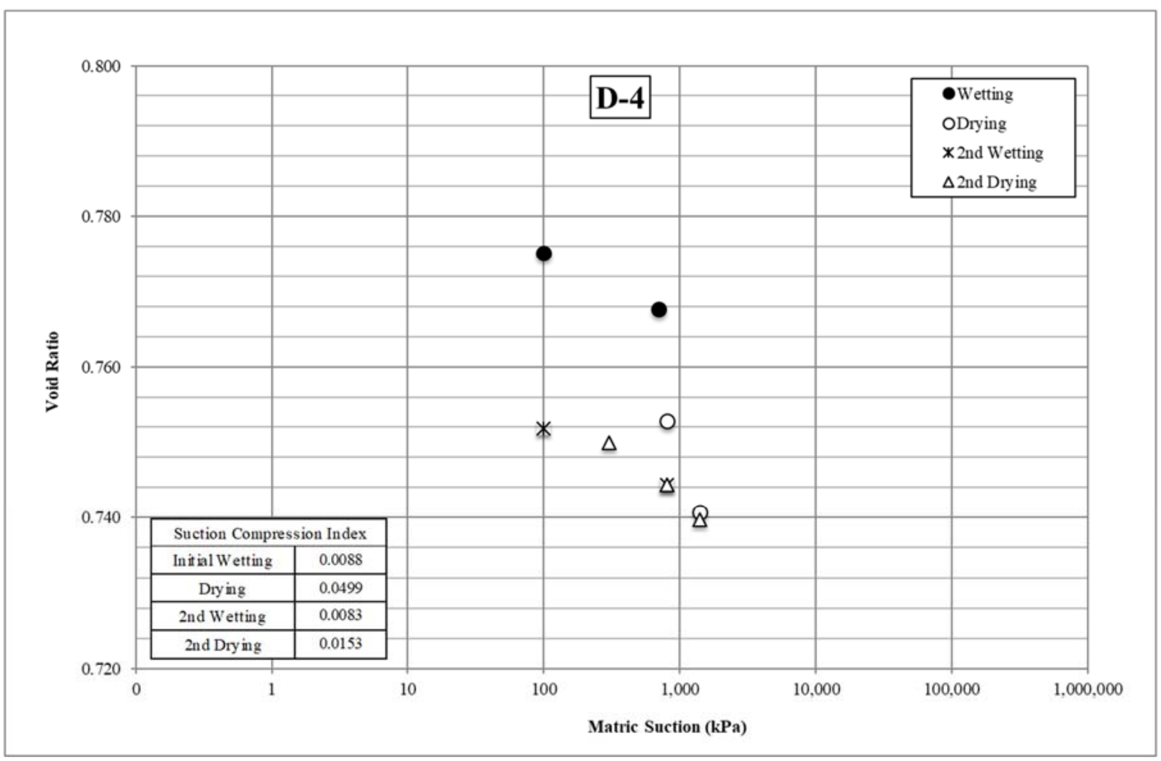


Figure 85: Void Ratio vs. Matric Suction for D-4

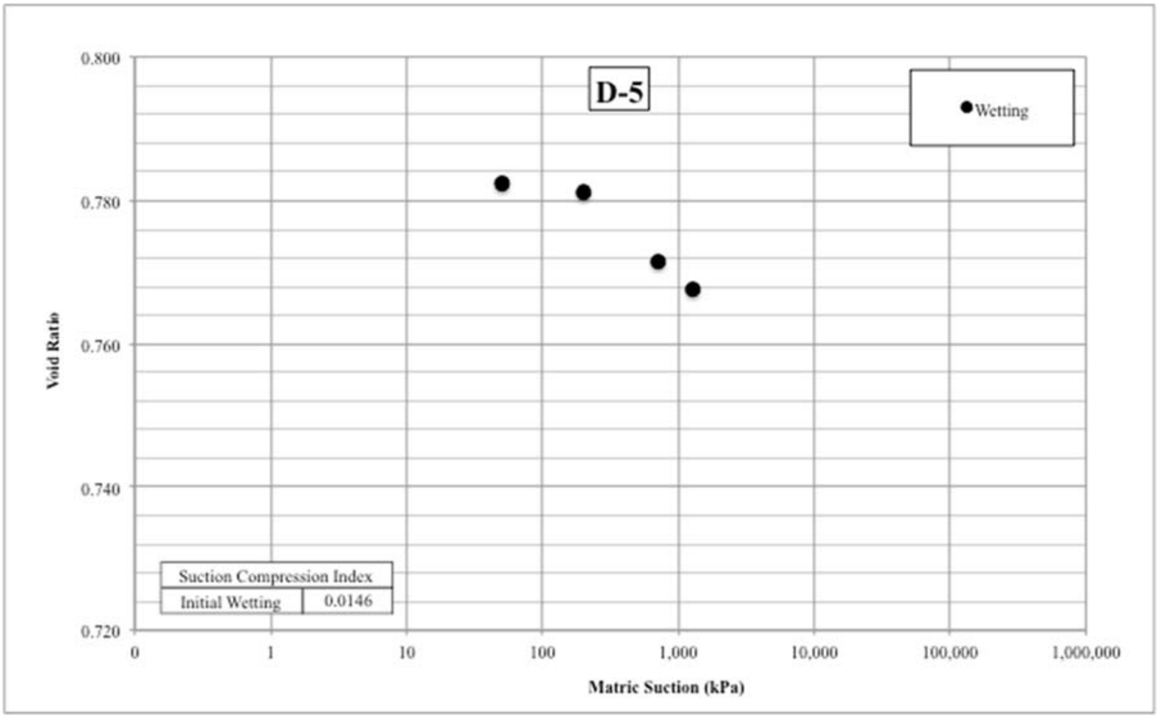


Figure 86: Void Ratio vs. Matric Suction for D-5

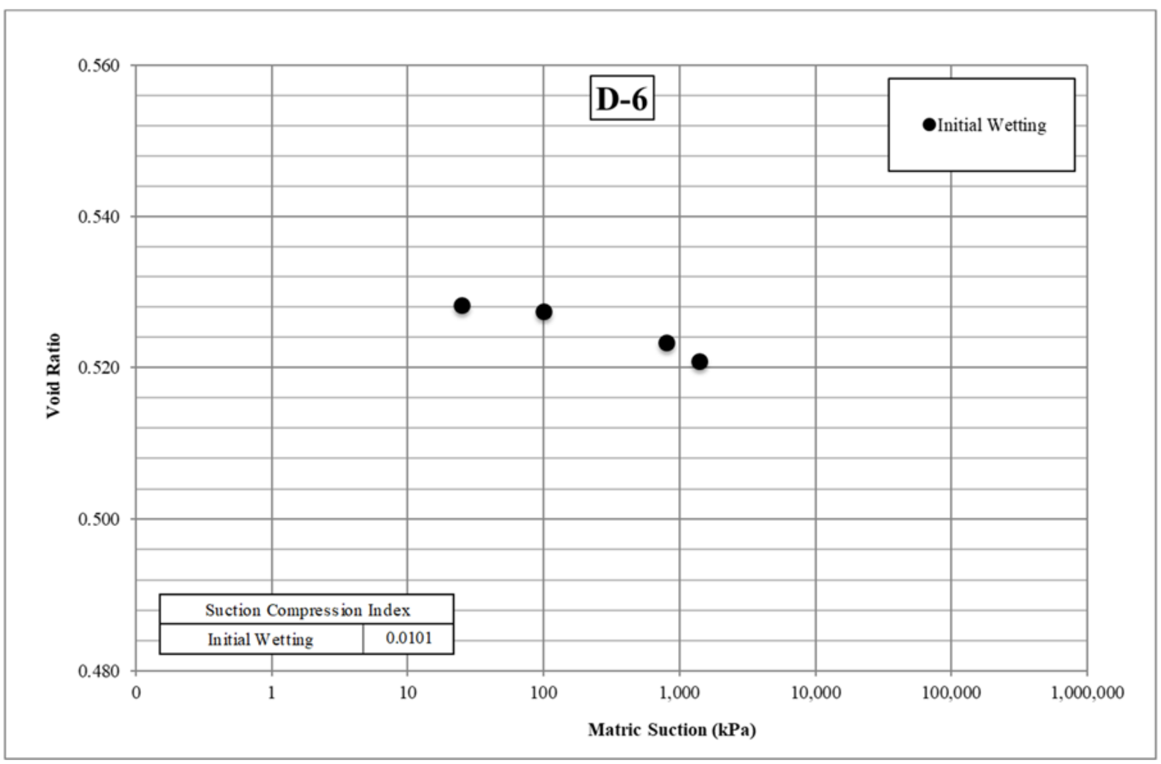


Figure 87: Void Ratio vs. Matric Suction for D-6

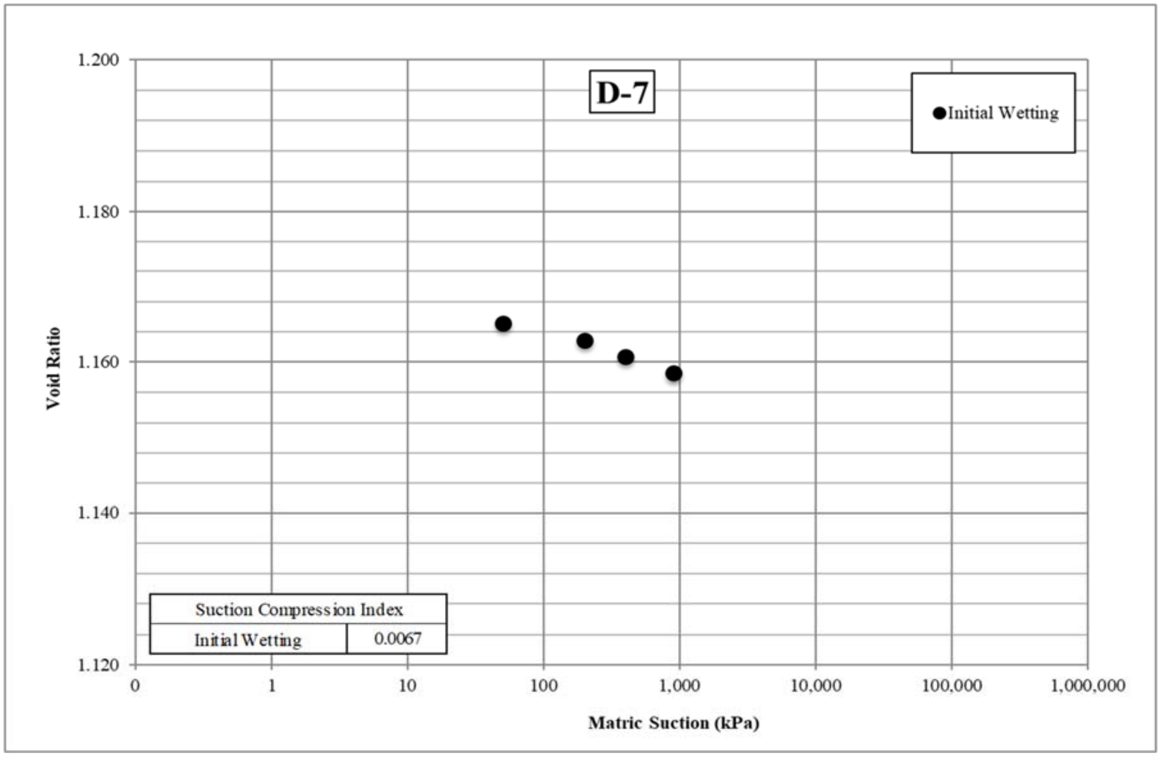


Figure 88: Void Ratio vs. Matric Suction for D-7

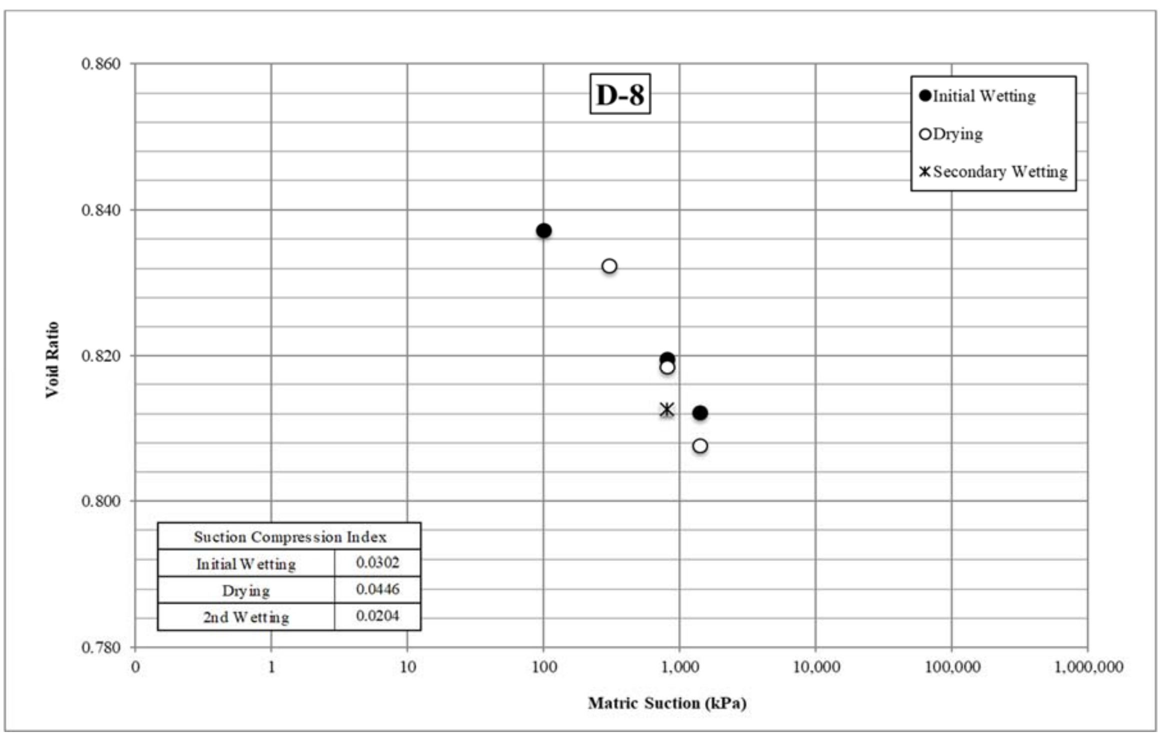


Figure 89: Void Ratio vs. Matric Suction for D-8

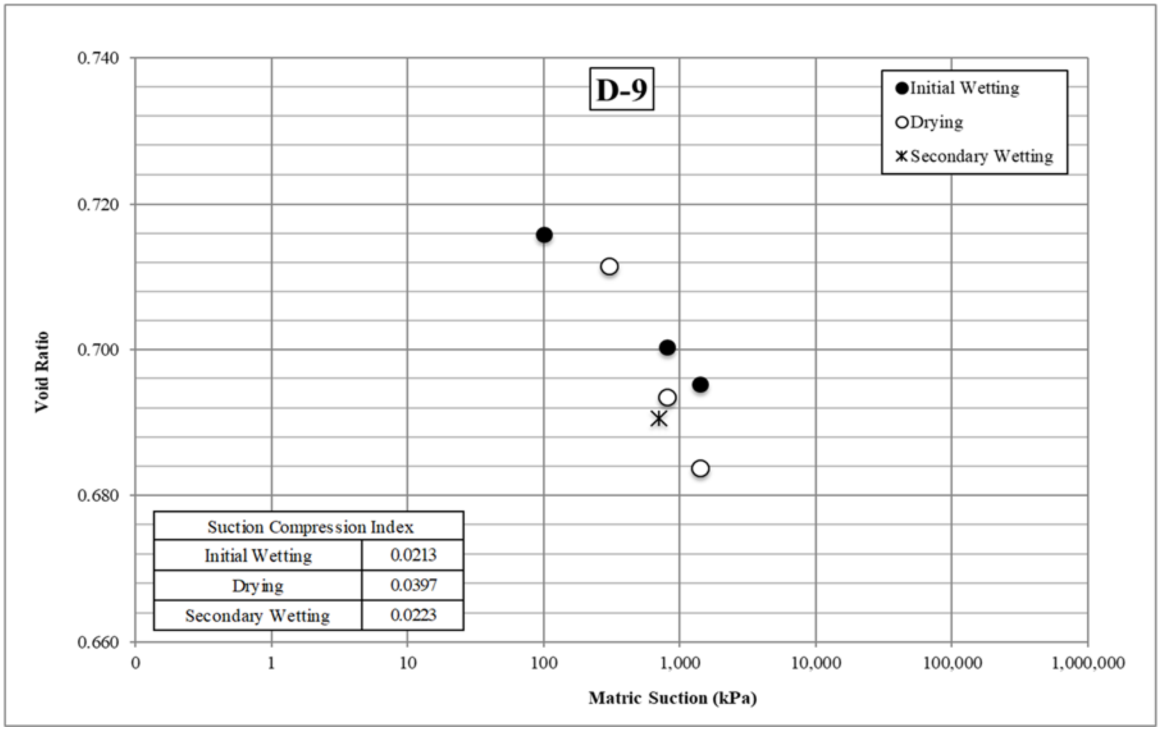


Figure 90: Void Ratio vs. Matric Suction for D-9

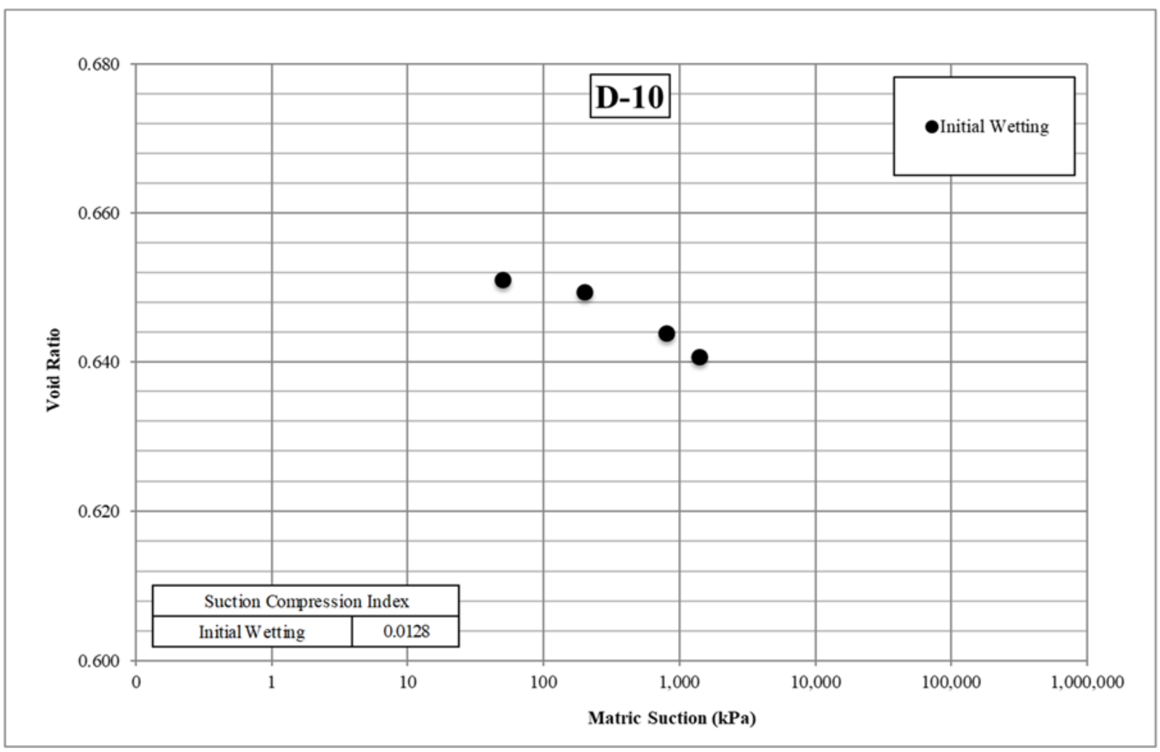


Figure 91: Void Ratio vs. Matric Suction for D-10

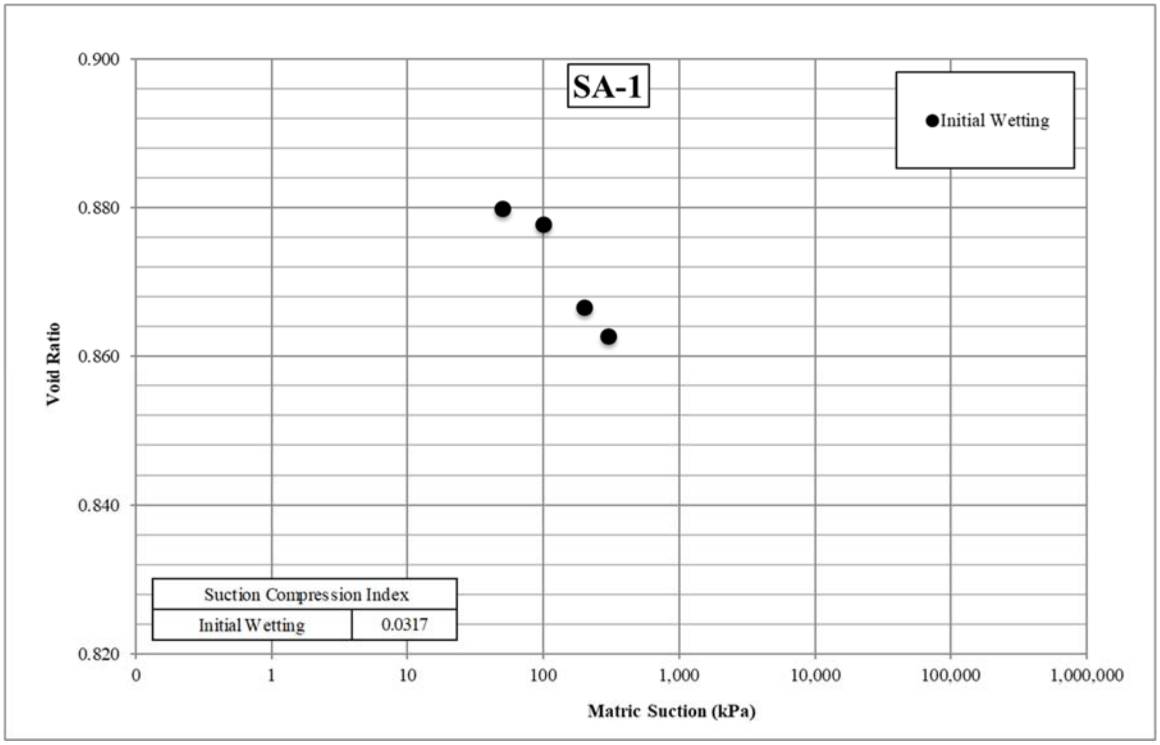


Figure 92: Void Ratio vs. Matric Suction for SA-1

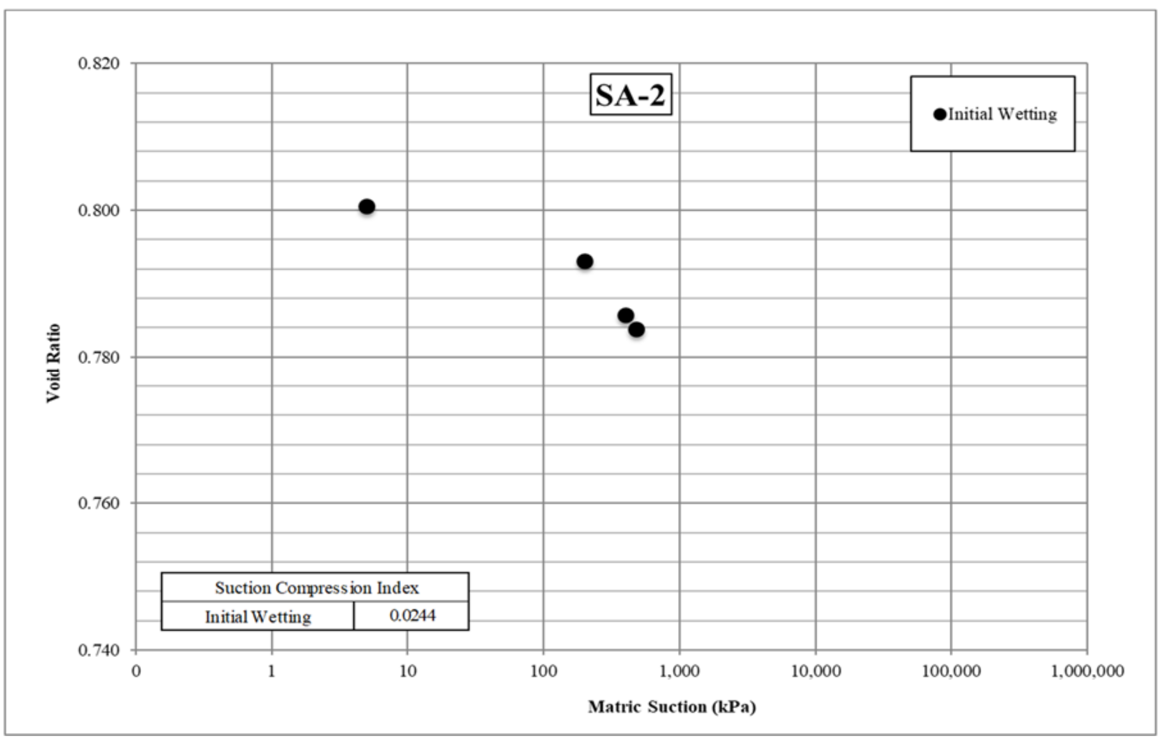


Figure 93: Void Ratio vs. Matric Suction for SA-2

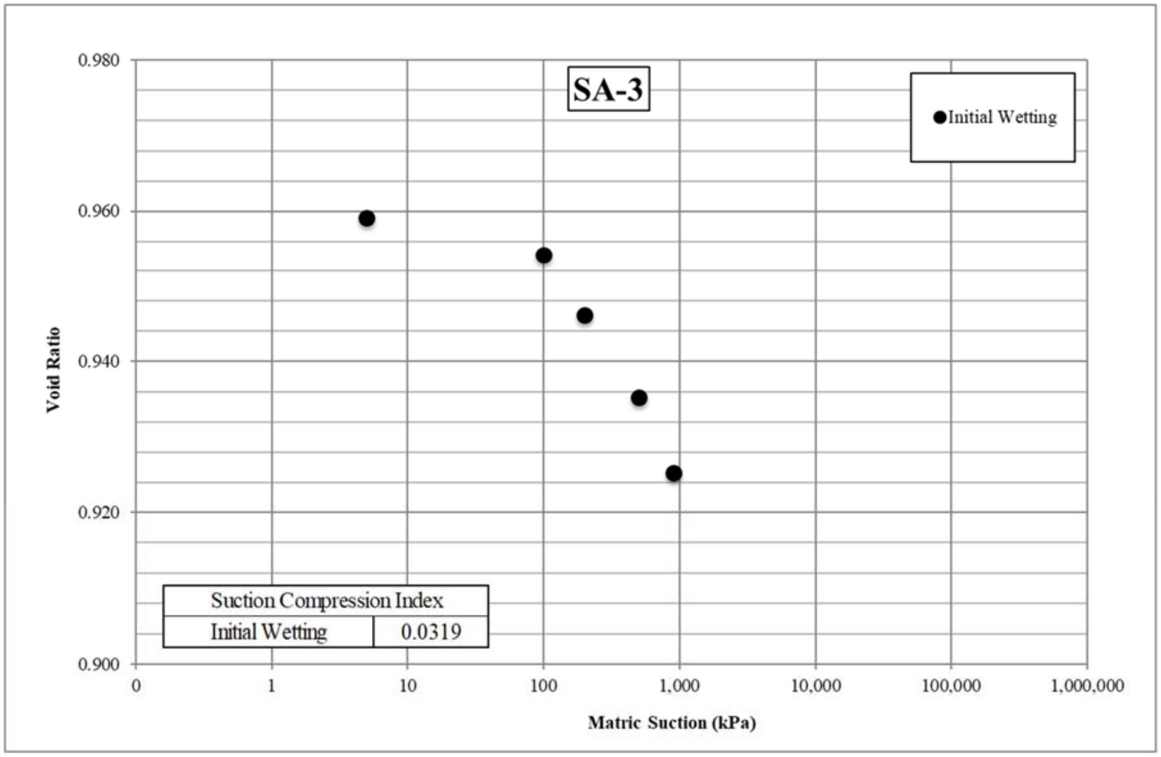


Figure 94: Void Ratio vs. Matric Suction for SA-3

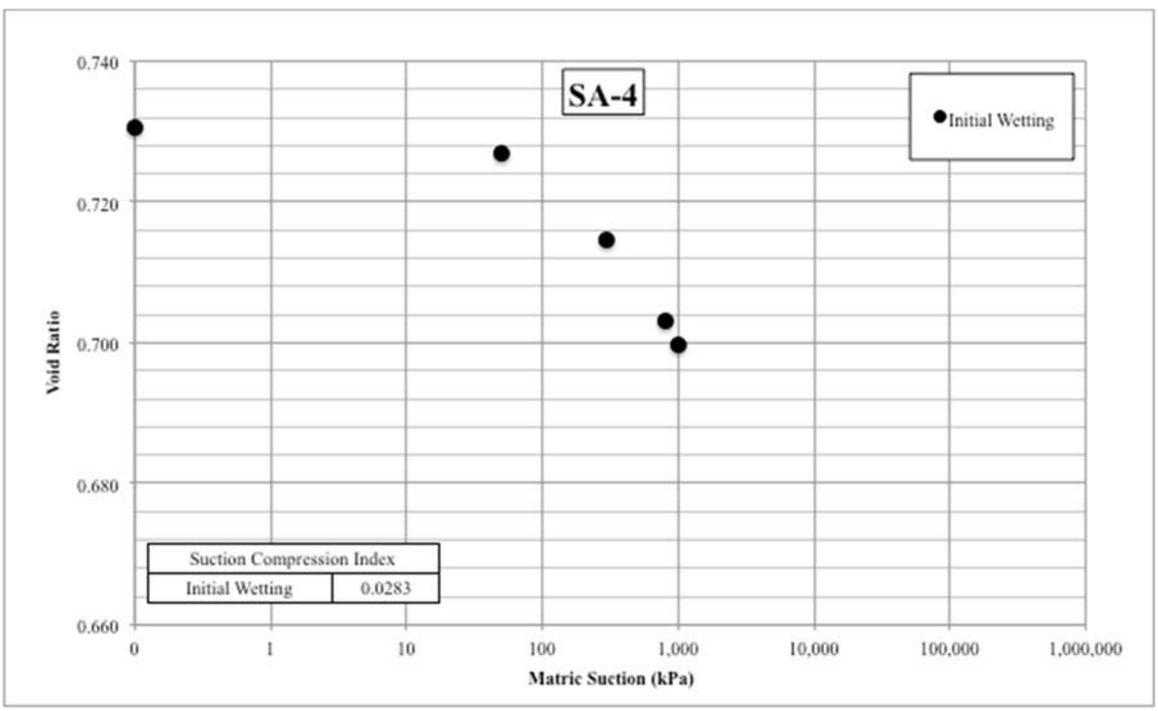


Figure 95: Void Ratio vs. Matric Suction for SA-4

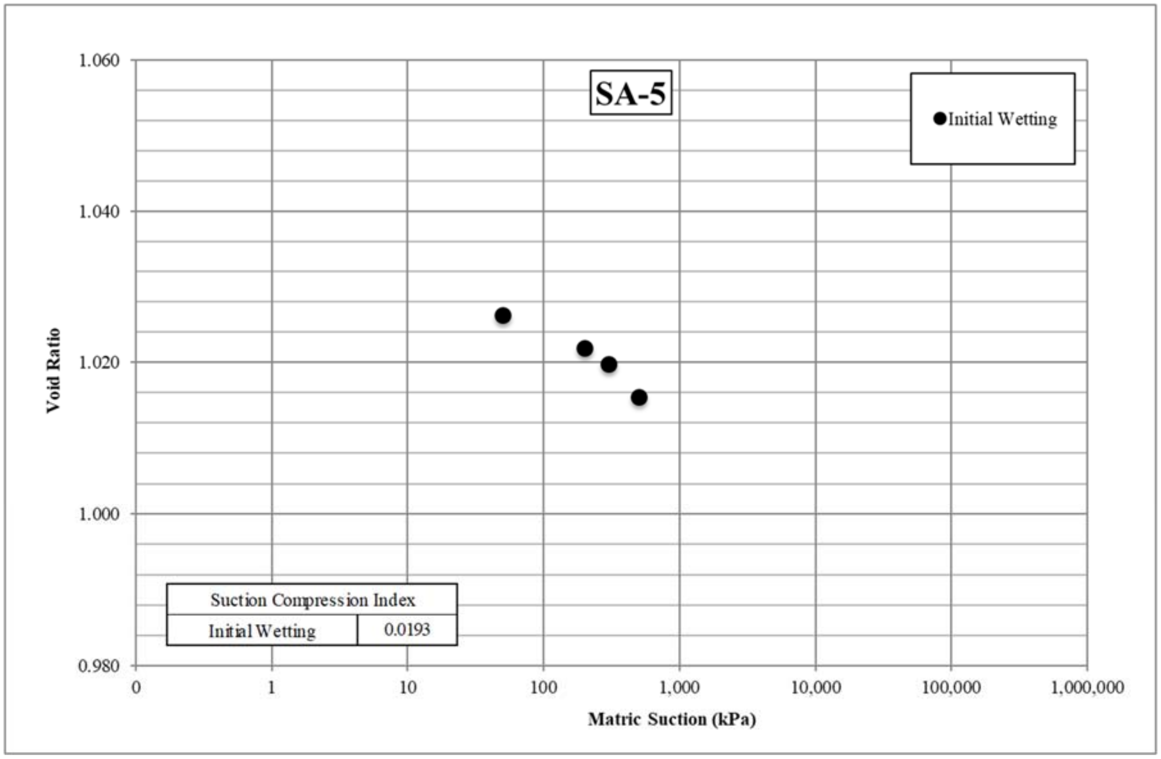


Figure 96: Void Ratio vs. Matric Suction for SA-5

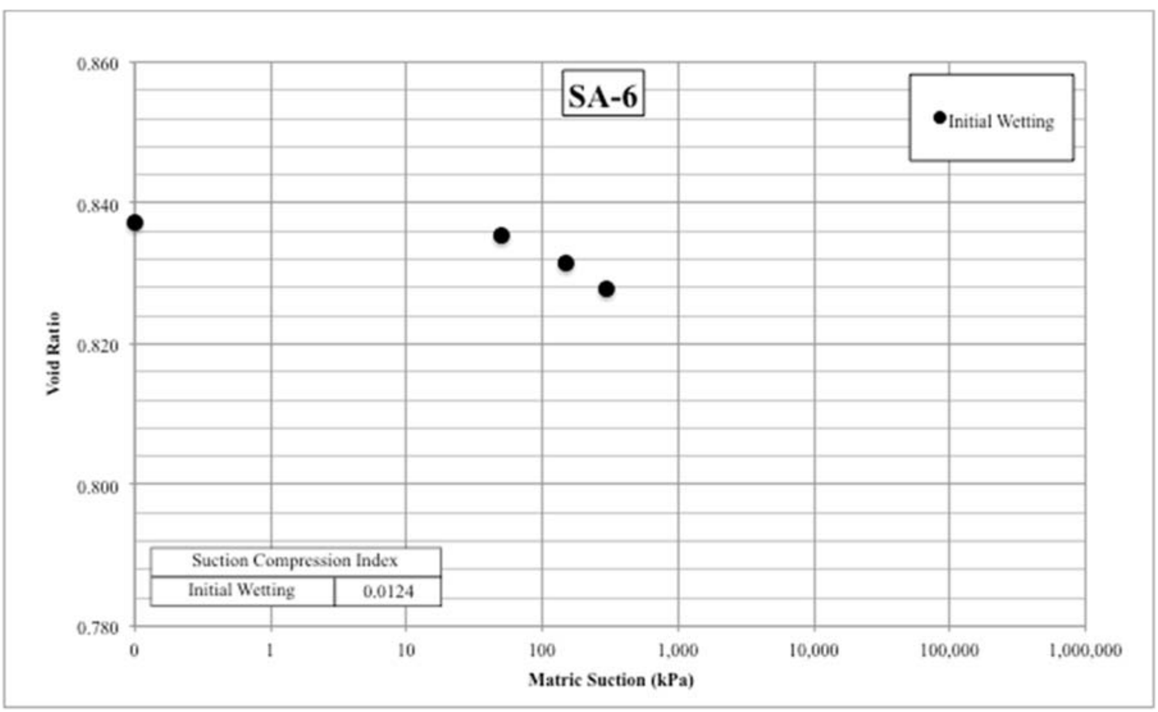


Figure 97: Void Ratio vs. Matric Suction for SA-6

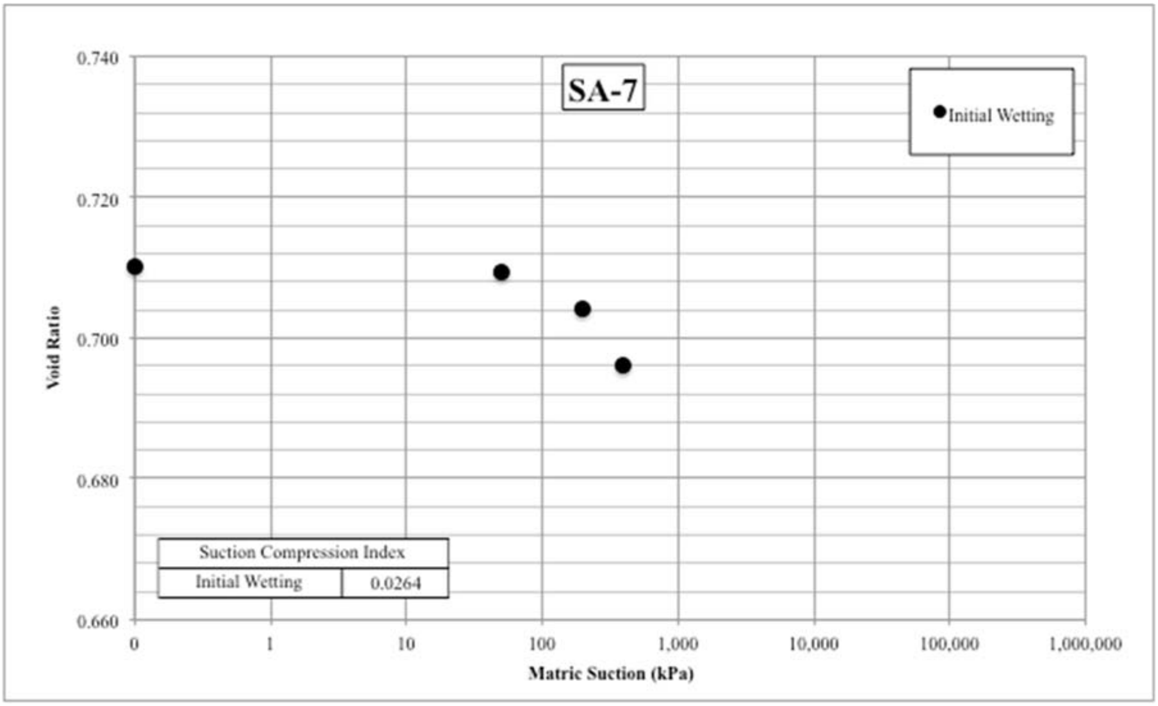


Figure 98: Void Ratio vs. Matric Suction for SA-7

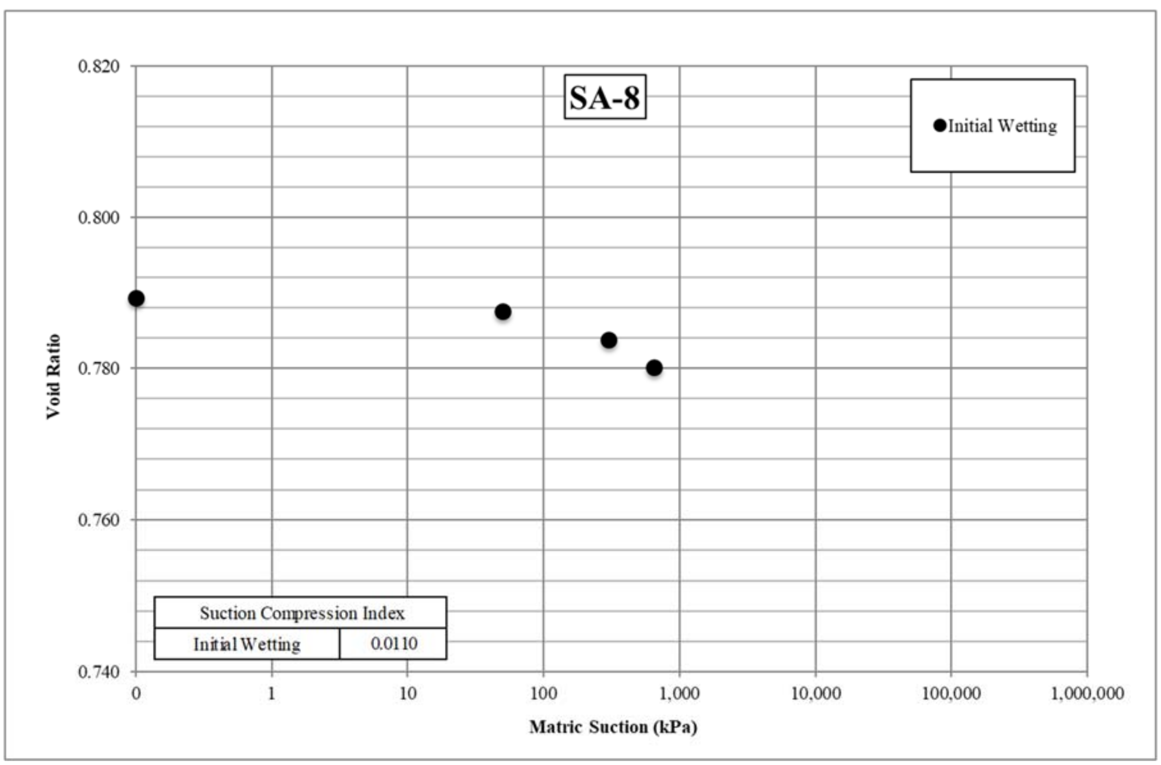


Figure 99: Void Ratio vs. Matric Suction for SA-8

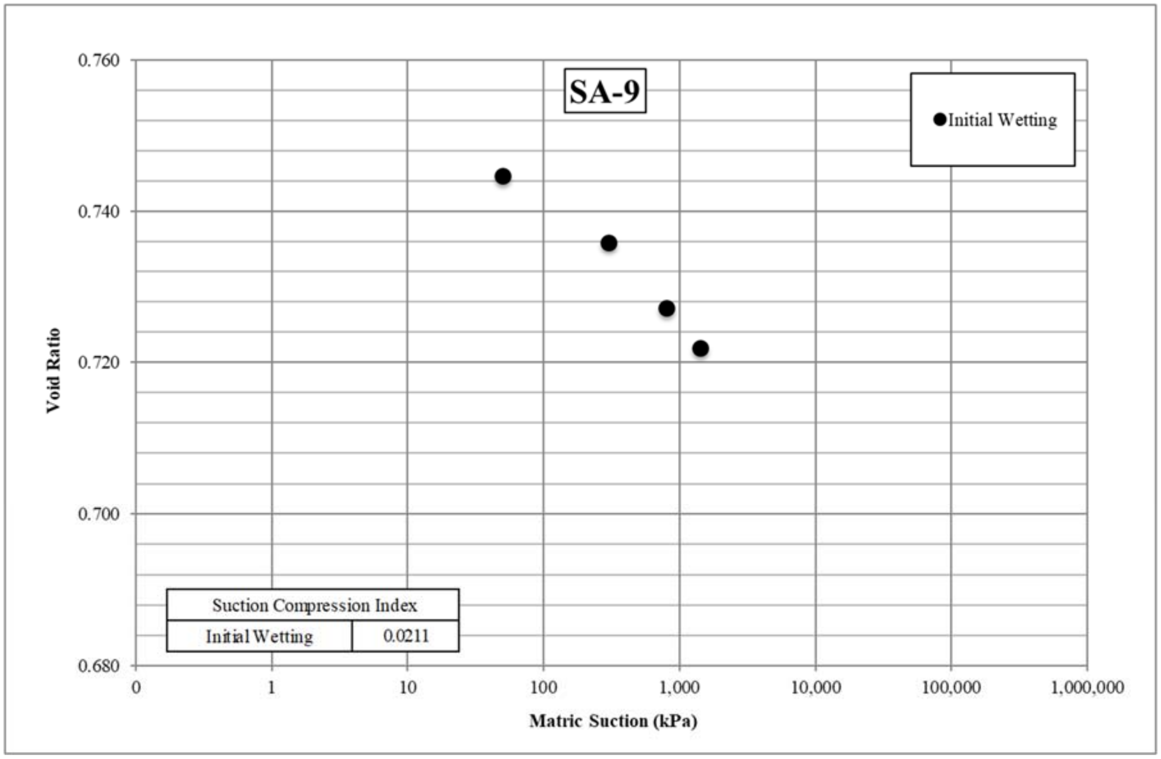


Figure 100: Void Ratio vs. Matric Suction for SA-9

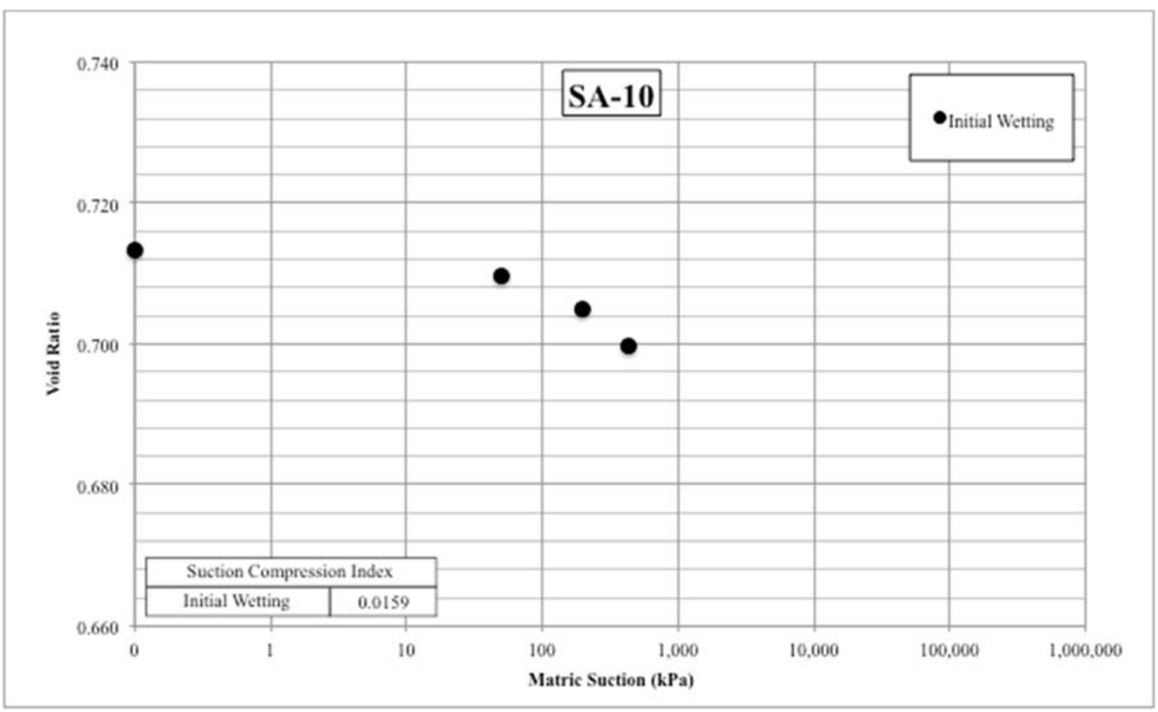


Figure 101: Void Ratio vs. Matric Suction for SA-10

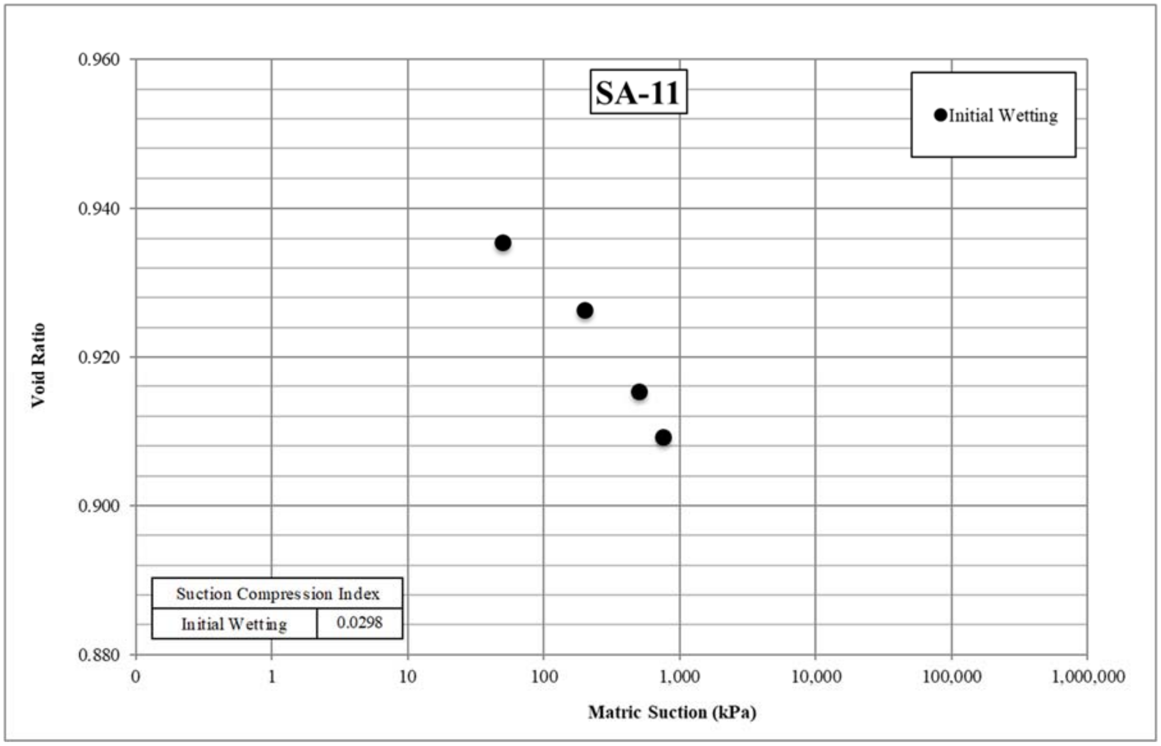


Figure 102: Void Ratio vs. Matric Suction for SA-11