DETERMINING SUCTION COMPRESSION INDEX OF EXPANSIVE SOILS BASED ON NON-LINEAR SUCTION-VOLUMETRIC STRAIN RELATIONSHIP

By

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DEDICATION

To my grandparents, parents, and sister.

To my wife.

To my daughters Maryam & Sarah.

This achievement would have never been possible without your invaluable support and unconditional love. I cannot put my gratitude to you into words. Period.

This is for you…

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Title of Study: DETERMINING SUCTION COMPRESSION INDEX OF EXPANSIVE SOILS BASED ON NON-LINEAR SUCTION-VOLUMETRIC STRAIN RELATIONSHIP

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Abstract: Expansive soils have always been problematic in many parts of the United States and the world. This is due to the stresses they exert on buildings' foundations, pavements, and other geotechnical structures. In the United States, volumetric changes due to shrinking and swelling soils cause extensive damage which costs billions of dollars annually. In the state of Oklahoma, expansive soil is widespread and the annual maintenance can cost millions of dollars statewide. The climatic conditions in the state easily allow for soil volume changes. This happens due to the wetting and drying cycles which affect the moisture active zone in unsaturated soils. Suction compression index (γ_h) is the key parameter that relates volumetric changes to soil suction changes in unsaturated soils. It is a soil property through which heave in expansive soils can be predicted due to the change in soil suction. It can be determined as the slope of the suction-volumetric strain relationship. Since this relationship is essentially nonlinear, the need for a precise γ_h determination method has always been crucial. The more accurate γ_h is determined, the more accurate soil movements can be predicted and taken care of early in the design stage. Accordingly, more money can be saved from either the repair costs or the initial costs by avoiding over-design. This study proposes an original γ_h testing method. The testing method uniquely incorporates volumetric and suction measurements in a new and practical way utilizing simple digital imaging. This makes it convenient for geotechnical engineering practitioners and laboratories to adopt the testing method. The testing method unprecedentedly integrates statistical modeling for determination of incremental γ_h in order to cover the entire nonlinearity of the suction-volumetric strain relationship. This is done by fitting the S-shaped relationship by a well-known class of statistical functions called Cumulative Distribution Functions (CDF). Incremental γ_h is estimated by estimating the CDF at every suction value. The appropriateness of using these estimates to describe the suction-volumetric strain relationship is evaluated using the Kolmogorov-Smirnov (K-S) goodness of fit test. Furthermore, 95% confidence intervals of the superposed curves are also used to assess the appropriateness of the CDF estimates. Undisturbed soil specimens from three sites in Oklahoma have been tested. The new testing method is compared against other techniques in the literature and proven reliable results.

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INTRODUCTION

1.1 Definition

Suction compression index (γ_h, SCI, or C_h) is a soil property through which heave or shrinkage in expansive soils can be predicted due to the change in suction (i.e. negative pore water pressure). The third edition of the Design of Post-Tensioned Slabs-on-Ground of the Post-Tensioning Institute, PTI (2004), defines the suction compression index (γ_h) as the change in volume related to a change in suction for an intact specimen of soil. It also adds that the change of suction is similar to the change in effective stress in settlement analysis, but has a more complex relationship.

Suction compression index has been addressed by many researchers in the literature among whom are Lytton (1977), McKeen and Nielsen (1978), McKeen (1981), McKeen and Hamberg (1981), McKeen and Lenke (1982), McKeen and Lytton (1984), McKeen (1985), McKeen (1992), Lytton (1994), Perko et al. (2000), Covar and Lytton (2001), Lytton et al. (2005), and Nelson et al. (2015).

It can be represented by Equation 1.1:

$$\gamma_h = \frac{\Delta V/V}{h_f - h_i} \tag{1.1}$$

Where,

γ_h = Suction compression index (slope of suction versus volumetric strain relationship)

$\frac{\Delta V}{V}$ = Volumetric strain, unitless

h_f = Final total suction value, pF

h_i = Initial total suction value, pF

$pF = \log_{10}(kPa) + 1$

1.2 Background

In 1967, the Soil Survey Investigations Report No. 1 of the U.S. Department of Agriculture, USDA (1967), addressed the relationship between suction and volume change by introducing the Coefficient of Linear Extensibility (COLE). This approach was then adopted in civil (geotechnical) engineering and expanded to suction compression index.

Volume change studies may be made using total suction values (McKeen 1981). McKeen added that in practical engineering problems, it is the slope of the volume change versus suction curve that quantifies soil response to moisture changes. Thus the suction compression index, γ_h, is defined as the slope of the volume-total suction curve (Lytton 1977). It is important to highlight that – as per Equation 1.1 – suction values that contribute in γ_h determination are expressed in pF units, which can be determined by taking the common (base-10) logarithm of kilopascal (kPa) units plus value of one ($pF = \log_{10}(kPa) + 1$). The units of pF can be also determined by taking the common logarithm of a height of a water column in centimeters ($pF = \log_{10}(cm_{H2O})$). Despite of the logarithmic nature of suction values in Equation 1.1, the relationship is still not linear in many cases (not idealized). In other words, the relationship is essentially non-linear even when suction is expressed in pF units. Figure 1.1 illustrates the relationship between volume, suction and mechanical pressure. Suction in this figure is not illustrated in logarithmic nature.

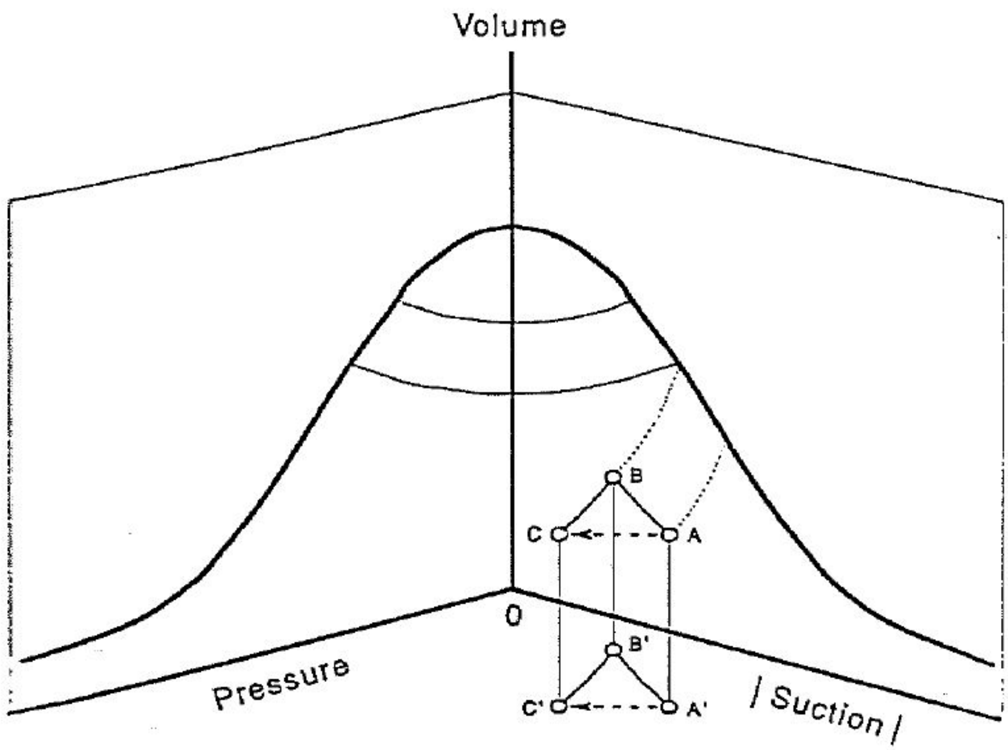

Figure 1.1. Pressure-Suction-Volume Surface for Expansive Soil, after Lytton (1994)

The expanded form of Lytton's volume change equation can be defined as per Equation 1.2:

$$\frac{\Delta V}{V} = -\gamma_h \log_{10}\left(\frac{h_f}{h_i}\right) - \gamma_\sigma \log_{10}\left(\frac{\sigma_f}{\sigma_i}\right) - \gamma_\pi \log_{10}\left(\frac{\pi_f}{\pi_i}\right) \tag{1.2}$$

Where,

$\frac{\Delta V}{V}$ = Volumetric strain

h_f = Final value of matric suction

h_i = Initial value of matric suction

σ_f = Final value of mean principle stress

σ_i = Initial value of mean principle stress

π_f = Final value of osmotic suction

π_i = Initial value of osmotic suction

γ_h = Matric suction compression index (*slope of matric suction versus volumetric strain relationship)*

γ_σ = Mean principle stress compression index (*slope of principle stress versus volumetric strain relationship*)

γ_π = Osmotic suction compression index

In unsaturated soils, the change of matric suction is the key factor that generates the heave and shrinkage (Lytton et al. 2005). While changes in osmotic (solute) suction are rare and the mean principal stress increases only slightly in the shallow zones where most of the volume change takes place (Lytton et al. 2005). Hence, γ_h can be determined as per equation 1.1 in terms of total suction.

In general, soil total suction ($\psi = u + \pi$) is the summation of matric suction (capillary pressure) and osmotic suction (suction due to salts in soil water). Soil total suction is related to relative humidity through the use of Lord Kelvin's equation (Fredlund and Rahardjo 1993). Lord Kelvin's equation (Adamson and Gast 1997) expresses soil total suction as shown in Equation 1.3.

$$\psi = -\frac{RT_k}{v_{w0}\omega_v} \ln\left(\frac{\bar{u}_v}{\bar{u}_{vo}}\right) \tag{1.3}$$

Where,

ψ = Total suction, kPa

R = Universal (molar) gas constant [i.e., 8.31432 J/(mol K)]

T_k = Absolute temperature (K) [i.e., T_k = 273.15 + T]

T = Temperature, °C

v_{w0} = Specific volume of water or the inverse of the density of water ($1/\rho_w$), m^3/Kg

ρ_w = Density of water (i.e., 998.2071 Kg/m^3 at temperature T = 20 °C)

ω_v = Molecular mass of water vapor (i.e., 18.016 Kg/kmol)

$\bar{u}_v$ = Partial pressure of pore-water vapor, kPa

$\bar{u}_{v0}$ = Saturation pressure of water vapor over a flat surface of pure water at the same temperature, kPa

$\frac{\bar{u}_v}{\bar{u}_{v0}}$ or h_r = Relative vapor pressure = relative humidity (RH), decimal

For a temperature of 20 °C, this equation can be reduced as per Equation 1.4.

$$\psi \text{ @ } 20\ °\text{C} = -135{,}045 \ln(RH) \tag{1.4}$$

Soil matric suction (i.e. the negative pore water pressure) can be expressed as per Kelvin's Capillary Model equation as shown in Equation 1.5.

$$u_a - u_w = \frac{2T_s}{R_s} \tag{1.5}$$

Where,

$u_a - u_w$ = Matric suction

u_a = Pore air pressure

u_w = Pore water pressure

T_s = Surface tension at air-water interface

R = Radius of curvature of water meniscus

From chemical thermodynamics principals, soil osmotic (solute) suction is determined by Van ‘t Hoff equation (Metten 1966; Robinson and Stokes 1968; Campbell 1985) as per the following Equation 1.6.

$$\pi = -vRT_kC\phi \tag{1.6}$$

Where,

π = Osmotic suction, kPa

v = Number of ions in solution per one molecule of solute*

R = Universal (molar) gas constant [i.e., 8.31432 J/(mol K)]

T_k = Absolute temperature (K) [i.e., T_k = 273.15 + T]

T = Temperature, °C

C = Sum of the molar concentrations in solution, mol/L

ϕ = Osmotic coefficient [ranges from 0.6 to 1.2]

* ν = 1 for non-ionizing solutes; ν = number of ions per molecule ionizing solutes (ν = 2 for NaCl, KCl, NH_4Cl; ν = 3 for Na_2SO_4, $CaCl_2$, $Na_2S_2O_3$) (Bulut 2001)

Different constitutive equations of heave prediction have different indices similar in nature to suction compression index but not exactly the same. For example, some indices relate volumetric strain to water content instead of suction. Some others use void ratio instead of volumetric strain. These indices have been named differently in the literature based on the way they are determined. Miller et al. (1995) and Nelson and Miller (1992) called it CLOD index while the latter extended the name to both the suction modulus ratio and also index of volumetric compressibility with respect to water content. Shrink–swell modulus has been named by Briaud et al. (2003). Instability index has been named by Mitchell (1980), while Vu and Fredlund (2004) named the volume change index with respect to matric suction.

Fityus et al. (2005) also mentioned the index the shrink-swell (or reactivity) index from which the instability index can be derived. The basic difference between the shrink-swell index and the instability index is that the latter takes into account the lateral confinement and effect of surcharge in the in-situ conditions.

One example of a constitutive equation for heave prediction that is similar to suction compression index – but not exactly the same – is Fredlund's equation (Equation 1.7). The equation was proposed by Fredlund and Morgenstern (1976). Fredlund's and Lytton's equations for volume change have very similar forms. They both consist of the same stress state variables (a suction component and a mechanical stress component) and also same coefficients (material properties) with respect to each of the components.

Fredlund's equation is based on a semi-empirical approach which involves assumptions based on experimental evidence from observing the behavior of many materials (Fredlund et al. 2012; Fredlund and Rahardjo 1993). These assumptions are:

1. Normal stress does not produce shear strain
2. Shear stress does not cause normal strain
3. Shear stress component, τ, causes only one shear strain component, γ.

It is understood that Fredlund's equation is a continuation to the effort originally proposed by Biot (1941). The material properties of Fredlund's equation are based on parameters such as modulus of elasticity, modulus of elasticity with respect to a change in matric suction and Poisson's ratio.

Fredlund's equation is expressed as per Equation 1.7.

$$\epsilon_v = \frac{\Delta V_v}{V_0} = m_1^s \Delta(\sigma_{mean} - u_a) + m_2^s \Delta(u_a - u_w) \tag{1.7}$$

Where,

ϵ_v = Volumetric strain

ΔV_v = Overall volume change of soil element

V_0 = Initial total volume of soil element

m_1^s = Coefficient of volume change with respect to net applied (normal) stress

m_2^s = Coefficient of volume change with respect to matric suction

$\Delta(\sigma_{mean} - u_a)$ = Change in net applied (normal) stress

$\Delta(u_a - u_w)$ = Change in matric suction

$$m_1^s = 3\left(\frac{1 - 2\nu}{E}\right) \tag{1.8}$$

Where,

ν = Poisson's ratio

E = Young's (elastic) modulus

$$m_2^s = \frac{3}{H} \tag{1.9}$$

Where,

H = Modulus of elasticity with respect to a change in matric suction

Another constitutive equation is the one proposed by Hamberg and Nelson (1984). The reduced form of the equation is expressed as per Equation 1.10.

$$\rho = \sum_{i=1}^{n} \frac{(C_w \Delta w)_i}{(1 + e_0)_i} z_i \tag{1.10}$$

Where,

ρ = Total heave

C_w = Clod index

w = Water content

e_0 = Initial void ratio

z = Thickness of a soil layer

i = Increment for each soil layer in a soil profile

$$C_w = \frac{\Delta e}{\Delta w} \tag{1.11}$$

Where,

e = Void ratio

w = Water content

1.3 Problem Statement and Significance

Expansive soils have always been problematic to geotechnical engineers in many parts of the U.S. and the world. This is due to the stresses they may exert on buildings' foundations, pavements and/or other geotechnical structures if the soils happened to expand. In the United States, volumetric changes due to shrinking and swelling soils cause extensive damage, which costs about $7 to $15 billion annually (Nuhfer et al. 1993; Wray and Meyer 2004). In the state of Oklahoma, this type of soil is widespread and the annual maintenance to seal and repair the distress problems caused by it can cost millions of dollars statewide. The climatic conditions of the state help the soil to easily change its volume. Its volumetric change is a function of several factors related to soil suction such as magnitude of suction changes and depth of suction influence (moisture active zone). The severity of stresses produced by those soils is dependent upon the magnitude of its volumetric changes. The suction compression index (γ_h) is the key parameter that relates volumetric changes to soil suction. It can be determined by calculating the slope of the suction-volumetric changes relationship.

It is important to pursue research in determining the suction compression index. It is understood that the more accurate γ_h is determined, the more accurate soil movements can be predicted and taken care of in the design stage, thus more money can be saved from either the maintenance and repair costs or from initial costs by avoiding over-design. In addition, pursuing research in γ_h determination is important to provide geotechnical engineering laboratories a practical and reliable determination method. This is one of the main objectives of this research.

This can be achieved by proposing a new, accurate, relatively fast, inexpensive, and easy testing method. This makes it more practical to be adopted by practitioners. Although, as previously described, suction compression index is the slope of the suction-volumetric changes relationship, it has not been found from the literature review that there is method which is based on regular (or frequent) suction values at different corresponding volumes for a given specimen for γ_h determination. This is what the newly proposed γ_h determination method relies on. The new γ_h testing method comprises both drying and wetting components. An incremental γ_h approach based on statistical modeling is also explained later. The incremental γ_h values from the drying component of the testing method can be statistically determined at different suction values.

Suction compression index can be determined either by drying or wetting with single suction determination (McKeen and Hamberg 1981). They also mentioned examples of each method. For wetting, a soil sample may be inundated in an oedometer ring. Since suction changes between 0 and 33 kPa (2.52 pF) are not accompanied by significant volumetric changes, final suction can be assumed equal to 2.52 pF. By knowing initial suction value and corresponding volumes, γ_h can be determined. For drying, soil sample may be oven-dried. Since changes in soil volume cease when soil reaches the shrinkage limit and since shrinkage limit for clay soils is about 32 MPa (5.5 pF), the final suction value can be assumed 5.5 pF. By knowing initial suction value and corresponding volumes, γ_h can be determined.

The new method includes the following advantages over the examples mentioned by McKeen and Hamberg (1981):

1) It also provides incremental γ_h values from the drying component at different suction values.

2) Wetting suction compression index is not based on just one volume increase measurement (in many cases), therefore the suction-volumetric strain relationship is more representative to soil specimen being tested. It is also not based on assumed final suction value.

3) Wetting suction compression index is not based on soil sample size as small as oedometer ring size which better serves accuracy.

4) Drying suction compression index is based on regular suction values at different corresponding volumes rather than just two values one of which is assumed to be equal to suction at shrinkage limit.

In addition, methods like the CLOD test may take up to 8 weeks (Nevels 2014), the new method is relatively faster which makes is practical.

CHAPTER II

LITERATURE REVIEW

As mentioned in the Introduction chapter, γ_h and other similar indices are calculated in many different ways in the literature. In this chapter, some of the reviewed indices determination methods are presented.

2.1 The COLE Test

Coefficient of Linear Extensibility (COLE) test determines the linear strain of an undisturbed, unconfined sample on drying from 5 psi (33 kPa) suction to oven dry suction 150,000 psi (1000 MPa) (Nelson and Miller 1992). In terms of pF value, this suction range is equivalent to 2.52 pF to 7.00 pF. Soil samples in COLE test can be cylindrical in shape.

The COLE value is expressed as per Equation 2.1:

$$COLE = \frac{L_m - L_d}{L_d} = \frac{L_m}{L_d} - 1 \tag{2.1}$$

Where,

L_m = Moist length of soil clod at suction value of 2.52 pF (33 kPa).

L_d = Oven-dried length of same soil clod considered at suction value of 7.00 pF (1000 MPa).

COLE can also be estimated from laboratory bulk density data and coarse-fragment conversion factor (C_m) as per the following Equation 2.2, USDA (1967):

$$COLE = \left[\frac{1}{C_m \frac{(Db_m)}{(Db_d)} + (1 - C_m)}\right]^{1/3} - 1 \tag{2.2}$$

Where,

C_m = Coarse-fragment conversion factor = $\frac{Moist\ volume\ of\ soil\ particles\ less\ than\ 2\ mm}{Volume\ of\ whole\ soil}$

D_{bm} = Bulk density of the fine-earth fabric (fine-grained soil) at 2.52 pF or 33 kPa

D_{bd} = Bulk density of the fine-earth fabric (fine-grained soil) at oven- or air-dryness.

If there is no coarse material, $C_m = 1$, COLE is referred to as $COLE_f$, and the equation can be simplified to the following as expressed by Equation 2.3, USDA (1967):

$$COLE_f = \left[\frac{Db_d}{Db_m}\right]^{1/3} - 1 \tag{2.3}$$

Where,

γ_{dD} = Dry density of oven dry sample

γ_{dM} = Dry density of sample at 33 kPa (2.52 pF) suction

In 1981, the National Soil Survey Laboratory used Linear Extensibility (LE) as an estimator of clay mineralogy, the ratio of LE to clay content (percentage of soil particles finer than 2 microns) is related to mineralogy as per Table 2.1 (Nelson and Miller 1992):

Table 2.1. Clay Mineralogy According to LE, after Nelson and Miller (1992)

LE/Percent Clay	Mineralogy
>0.15	Smectites (montmorillonite)
0.05-0.15	Illites
<0.05	Kaolinites

The shrink-swell potential can be predicted from COLE values based on Table 2.2:

Table 2.2. Shrink-Swell Classes based on COLE, after Thomas et al. (2000)

Shrink-Swell Class	**COLE Value**
Low	< 0.03
Moderate	0.03 ~ 0.06
High	0.06 ~ 0.09
Very high	> 0.09

Covar and Lytton (2001) also proposed equations for calculation of suction compression index, γ_h, based on COLE value that can be expressed as per Equations 2.4, 2.5, and 2.6:

$$\gamma_h = \left(\frac{\gamma_{(swelling\ case)} + \gamma_{(shrinkage\ case)}}{2}\right) \tag{2.4}$$

$$\gamma_{(shrinkage\ case)} = \left[1 - \frac{1}{\left(\frac{COLE}{100} + 1\right)^3}\right] \tag{2.5}$$

$$\gamma_{(swelling\ case)} = \left[\left(\frac{COLE}{100} + 1\right)^3 - 1\right] \tag{2.6}$$

McKeen and Nielsen (1978) proposed an equation for γ_h determination based on COLE value as well. The equation can be expressed as per Equation 2.7:

$$\gamma_h = -\frac{COLE}{\log\frac{h_f}{h_i}} \tag{2.7}$$

Where,

h_f = Final soil suction value, pF

h_i = Initial soil suction value, pF

McKeen and Nielsen (1978) suggest that if h_f and h_i are assumed to be equal 5.5 pF and 2.53 pF respectively, then Equation 2.7 can be reduced to the following form as per Equation 2.8:

$$\gamma_h = -\,0.337\ COLE \tag{2.8}$$

2.2 The CLOD Test

The CLOD test has been developed at New Mexico Engineering Research Institute as a modification to the COLE test. It may use irregularly shaped specimens which can be obtained from broken soil pieces (Miller et al. 1995). The CLOD test involves coating soil samples with liquid resin. The resin coating allows the flow of water vapor for a drying sample; however, does not allow flow of liquid water if soil sample is inundated in water for a short time. The resin coating is a solution of 1:7 ratio (solute:solvent). The solute is DOW Saran F310 (powder) and the solvent is methyl ethyl ketone (MEK) or acetone.

The basic CLOD test procedure for a drying soil sample as suggested by Hamberg (1985) is as follows:

- Coat the sample (soil clod) with resin and measure its volume. The volume of a soil sample of any shape may be determined by weighing it while submerged underwater on a balance. The reading of the balance, adjusted for the weight of the pan and water, is a direct measurement of buoyant force on the sample. Sample volume can then be determined by Archimedes' principle. (Nelson and Miller 1992).
- Allow sample to dry slowly in air, with periodic volume and weight measurements taken until the sample reaches a constant weight under laboratory humidity conditions.

- Oven dry the sample for 48 hours and take a final volume and weight measurements.

The test procedure allows for determining void ratio and water content values from which the CLOD index, C_w, can be determined. The CLOD index (also known as suction modulus ratio) can be expressed as follows by Equation 2.9 and also shown in Figure 2.1:

$$C_w = \frac{\Delta e}{\Delta w} \tag{2.9}$$

Where,

C_w = CLOD index

Δe = Change in the void ratio

Δw = Change in the water content

Nelson and Miller (1992) expressed heave determination involving CLOD index as per Equations 2.10 and 2.11:

$$\Delta z_i = \frac{\Delta e}{1 + e_o} z_i = \frac{C_w \Delta_w}{1 + e_o} z_i \tag{2.10}$$

Where,

Δz_i = Heave for a uniform layer

C_w = CLOD index

z_i = Layer thickness

Δe = Change in void ratio

Δ_w = Change in water content

e_o = Initial void ratio

$$\rho = \sum_{i=1}^{n} \Delta z_i = \sum_{i=1}^{n} \frac{C_w \Delta_w}{1 + e_o} z_i \tag{2.11}$$

Where,

ρ = Total heave (sum of all increments of heave for each layer)

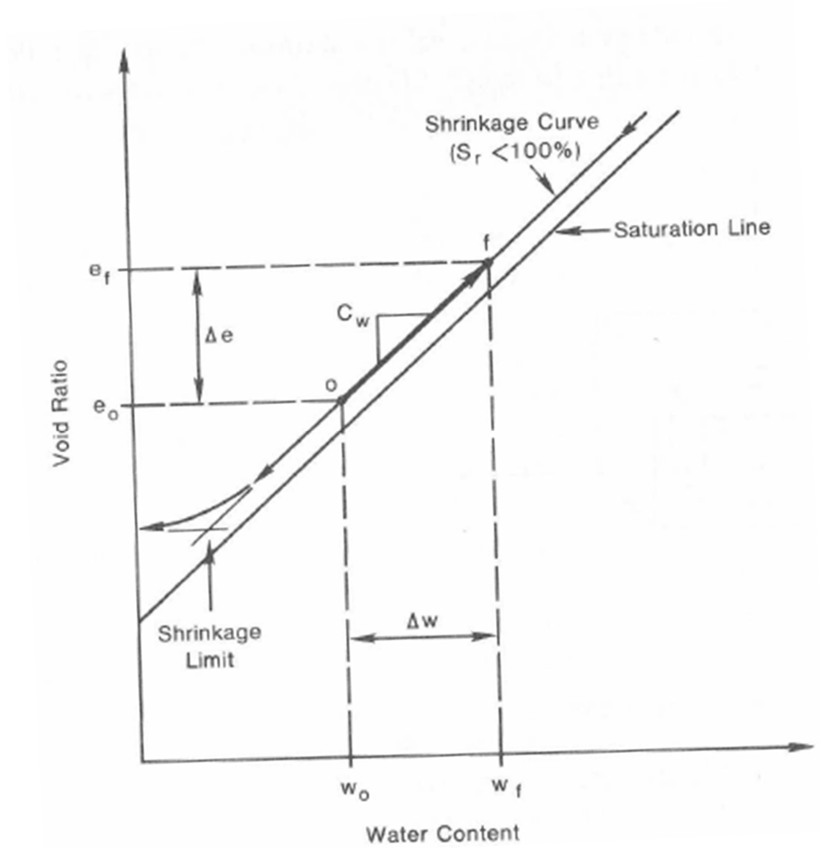

Figure 2.1. Idealized Shrinkage Curve for a CLOD Sample, after (Nelson and Miller 1992)

Hamberg (1985) claims that since void ratio and water content are both directly related to soil suction, the relationship between void ratio and water content is equivalent to the effect of suction on void ratio as long as water content is greater than the shrinkage limit because below the shrinkage limit, changes in water contents are not accompanied by changes in volume, by definition. Other than the void ratio-water content based CLOD test introduced above, there is also a similar explanation introduced in Appendix D of (McKeen 1985) based on a suction measurement and two determinations of bulk density. Moreover, Krosley et al. (2003) proposed using Elmer's Craft Glue as a coating (encasement) material for the CLOD test to replace the resin coating (mixture of DOW Saran F310 and methyl ethyl keton). This is due to the proposed material's non-hazardous nature, low cost, and fast testing time. In addition to the latter advantages, it has been noticed – while performing preliminary trial tests for this study – that the glue proposed by Krosley et al. (2003) is widely available unlike the original DOW Saran F310 that was very challenging to find.

2.3 The Shrink Test-Water Content Method

This method was proposed by Briaud et al. (2003) mainly to estimate the vertical movement of the ground surface for soil that swells and shrinks due to variations in water content. It also comprises a shrink–swell modulus determination through a shrink test. The shrink–swell modulus is similar in definition to the suction compression index. As described by its authors, the fundamental basis of this method is that the water content is directly linked to suction through the soil-water characteristic curve and, since suction is related to the volume change, so is the water content. Therefore, Briaud et al. (2003) states that the use of the water content as a governing parameter is as theoretically appropriate as the use of suction.

One of the parameters calculated from the shrink test is the shrink–swell modulus E_w (which is the slope of the water content versus the volumetric strain relationship) as defined in Equation 2.12 and illustrated in Figure 2.2.

$$E_w = \frac{\Delta w}{\left(\Delta v / v_0\right)} \tag{2.12}$$

Where,

Δw = Change in water content

$\Delta v / v_0$ = Volumetric strain

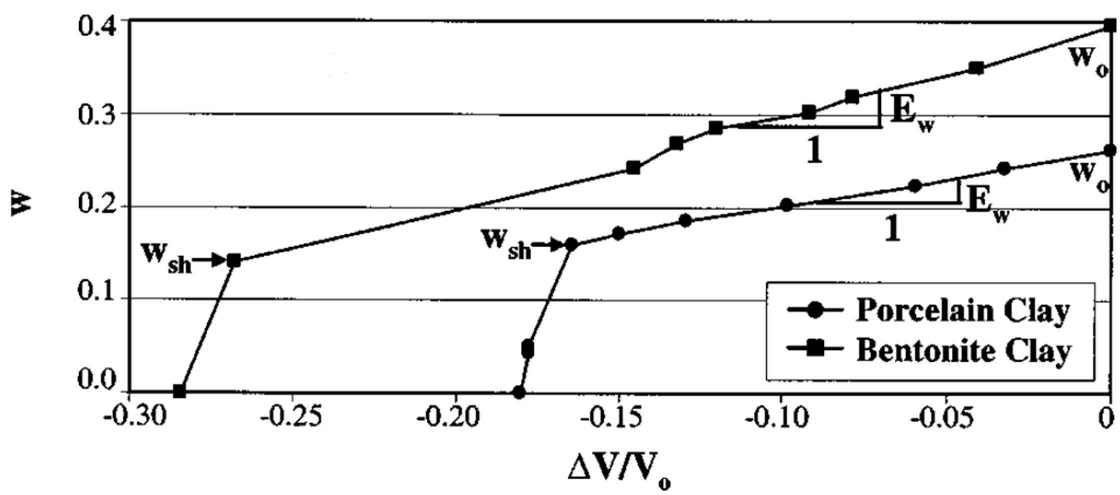

Figure 2.2. Water Content (Δw) versus Volumetric Strain ($\Delta V/V_0$) for the Shrink Test, after Briaud et al. (2003)

The shrink test sample is cylindrical in shape with recommended diameter of 75 mm and height of 150 mm. The sample is tested in a vertical orientation corresponding to the same direction at the site (in-situ vertical direction). Height and diameter measurements are taken with a digital caliper. Readings every hour for the first 8 hours are recommended and two days of readings are usually sufficient (Briaud et al. 2003). Shrink test parameters are shown in Figure 2.3.

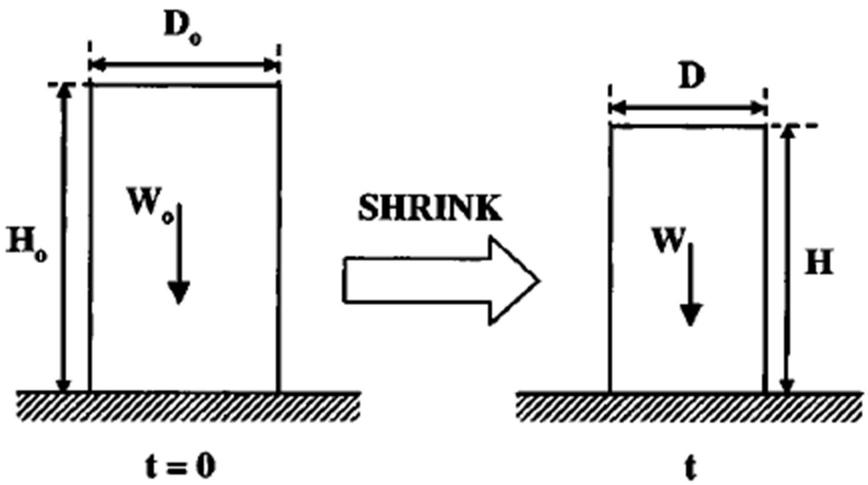

Figure 2.3. Shrink Test Parameters, after Briaud et al. (2003)

A swell test can be used to obtain water content versus volumetric strain curve while soil swells in a consolidometer ring. The swell test leads to a two-point water content versus volumetric strain curve from which shrink–swell modulus E_w can also be obtained (Briaud et al. 2003).

2.4 The Shrink-Swell Test

Other than the COLE, CLOD and the Shrink Test-Water Content method proposed by Briaud et al. (2003), there is also the shrink-swell test (an Australian practice). Fityus et al. (2005) mention that it is routinely used in Australian geotechnical practice as the principal method for the experimental assessment of the expansive potential of clay soils.

Shrink–swell test results have a coefficient of variation of about 25% and have been considered sufficiently reliable for the design of foundations for lightly loaded structures in Australian practice (Fityus et al. 2004; Walsh and Cameron 1997).

As Fityus et al. (2004) described, the shrink-swell test (AS 1289 7.1.1 1992) comprises both shrinkage and swelling test components, carried out on companion soil samples, initially at their field water contents. The shrinkage component starts with air drying followed by oven drying of an undisturbed sample. The swell component involves an oedometer test in which the undisturbed sample is seated under a nominal 25 kPa load before being inundated with distilled water and allowed to swell until the soil saturates. Shrinkage strain (ε_{sh}) and swell strain (ε_{sw}) are measured in the respective tests and combined to give a shrink–swell (or reactivity) index, I_{ss} as expressed by Equation 2.13:

$$I_{ss} = \frac{\varepsilon_{sh} + {}^{\varepsilon_{sw}}/_{2}}{1.8} \tag{2.13}$$

The denominator of 1.8 is an estimate of the effective range of change in suction in pF units that corresponds to a change in soil volume (Fityus et al. 2004). In other words, the shrink-swell test assumes that the effective range referred to is the range between suction value corresponding to soil field capacity (2.4 pF ≈ 25 kPa) and suction value corresponding to vegetation wilting point (4.2 pF ≈ 1.6 MPa). The shrink-swell test also assumes that the vertical strain in an unconfined swell sample is half of the vertical strain in a confined swell sample.

The swelling strain (ε_{sw}) is calculated (in percentage) by multiplying the final height (dial gauge reading) by 100 and then dividing the result by the initial height of the soil specimen (i.e. the height of the consolidometer ring). The shrinkage strain (ε_{sh}) is calculated (in percentage) by multiplying the final average height of the soil specimen by 100 and then dividing the result by the initial average height. The resulting value is subtracted from 100.

By substituting in Equation 2.13, the unit of I_{ss} is expressed in percentage form as %Strain/pF. Fityus et al. (2004) defines units of pF as log negative (free energy per unit volume of water), and approximated by log negative (hydraulic head in centimeters), or by 1 + log(suction in kPa).

2.4.1 Instability Index

The instability index, I_{pt}, can be derived from the shrink-swell index (Cameron and Walsh 1984). In order to derive the instability index from the shrink–swell index, the index must be adjusted to account for the effects of surcharge and lateral confinement that act on the soil in its in-situ condition (Fityus et al. 2005). This is done by application of a factor, α, usually applied at the time at which ground movements are being estimated. The I_{pt} can be expressed as per Equation 2.14.

$$I_{pt} = \alpha \, I_{ss} \tag{2.14}$$

As explained by Fityus et al. (2005), in a cracked clay soil there is a lack of lateral confinement therefore the α factor is assumed to be equal to a value of one and I_{pt} will be equal to I_{ss}. This is because the latter has already accommodated a lack of confinement by reducing the vertical strain in a confined swell sample to the half. In an uncracked clay soil, the α factor is assumed to be equal to $\left(2-\frac{z}{5}\right)$, where Z is the depth of interest. The value of 2 is considered in order to reverse the reduction of vertical strain in a confined swell sample. The value of $\frac{z}{5}$ is considered in order to take into account linear interpolation of swell in a soil depth of 10 meters from the surface. Observations of soils in parts of Australia suggest that no movement of soil occurs below depth of 10 meters (Fityus et al. 2005). Therefore, by applying α value of $\left(2-\frac{z}{5}\right)$, I_{pt} shall yield to a value of zero at depth of 10 m.

2.5 The Modified Shrink-Swell Test

Lopes (2007) tested the simple linear relationship assumed by the shrink-swell test using a Modified Linear Shrinkage (MLS) test in order to measure strain at various sign posts as shown in Table 2.3. Fifty six clay samples were tested by this method.

Lopes (2007) mentions that Equation 2.13 assumes that the suction versus strain relationship is linear within 2.4-4.2 pF and that the strain from 4.2-5.5 pF is constant as shown in Figure 2.4. Lopes (2007) also mentions that Equation 2.13 assumes that the confined swell is half of the unconfined swell; hence ε'_{sw} is halved to allow for the uniaxial nature of the test.

Table 2.3. Soil Suction Sign Posts, after Lopes (2007)

Suction (pF)	Soil State	References
6.5-7.0+	Oven dry	Cameron, Leeper, Lytton, Uren, Mitchell et al.
6.0	Air dry	Leeper, Lytton, Uren
5.5	Shrinkage limit	McKeen, Mitchell et al.
5.3	10% saturation	Lytton
4.0-4.5	Wilting point	Cameron, Leeper, Lytton, Uren, Wray, et al.
3.2-3.5	Plastic limit	Lytton
2.0-2.5	Field capacity	
1.5-2.0	Swell limit	McKeen
1.0	Liquid limit	McKeen, Lytton

Figure 2.5 shows the typical suction versus strain relationships of the MLS, while in Figure 2.4, lines A and B show two alternate relationships in the shrink-swell test. Line A assumes no significant shrinkage within the suction range of 4.2 pF and 5.5 pF (shrinkage limit) (Lopes 2007). Equation 2.13 follows Line A and ignores the strain from 4.2-5.5 pF, by comparing Figure 2.4 with Figure 2.5, it is clear that there will be different shrink–swell indices (Iss) (Lopes 2007).

Lopes (2007) suggests modifications to the Shrink-Swell test which includes the reduction of the shrinkage component by 30% (as a correction factor).

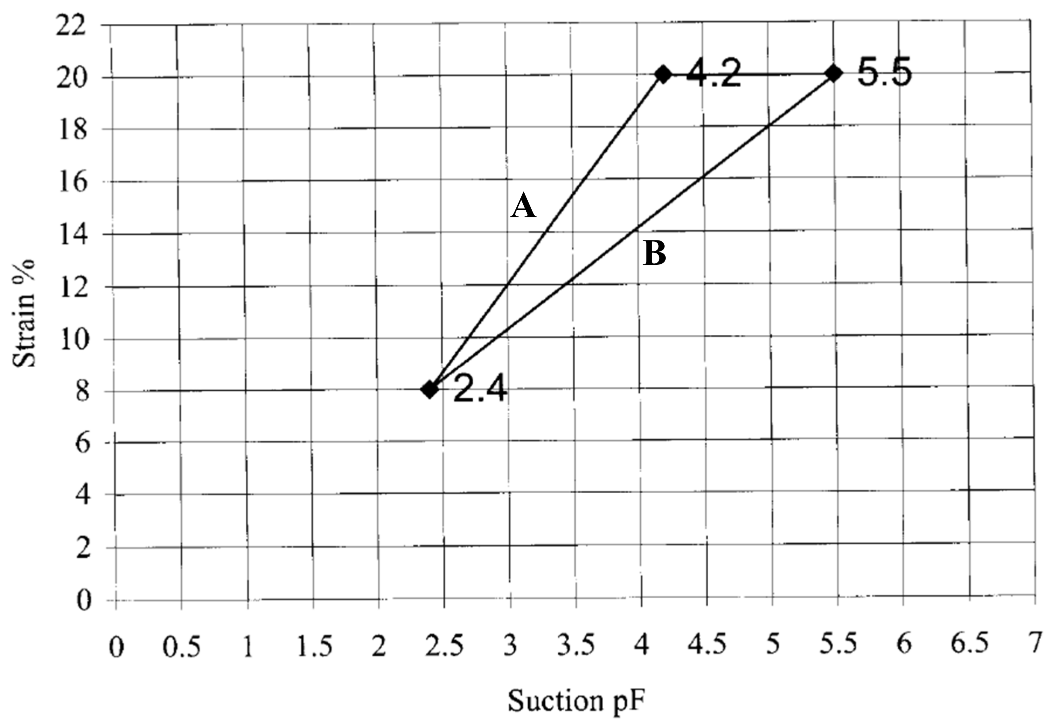

Figure 2.4. Suction/Strain Relationship in Shrink-Swell Test, after Lopes (2007)

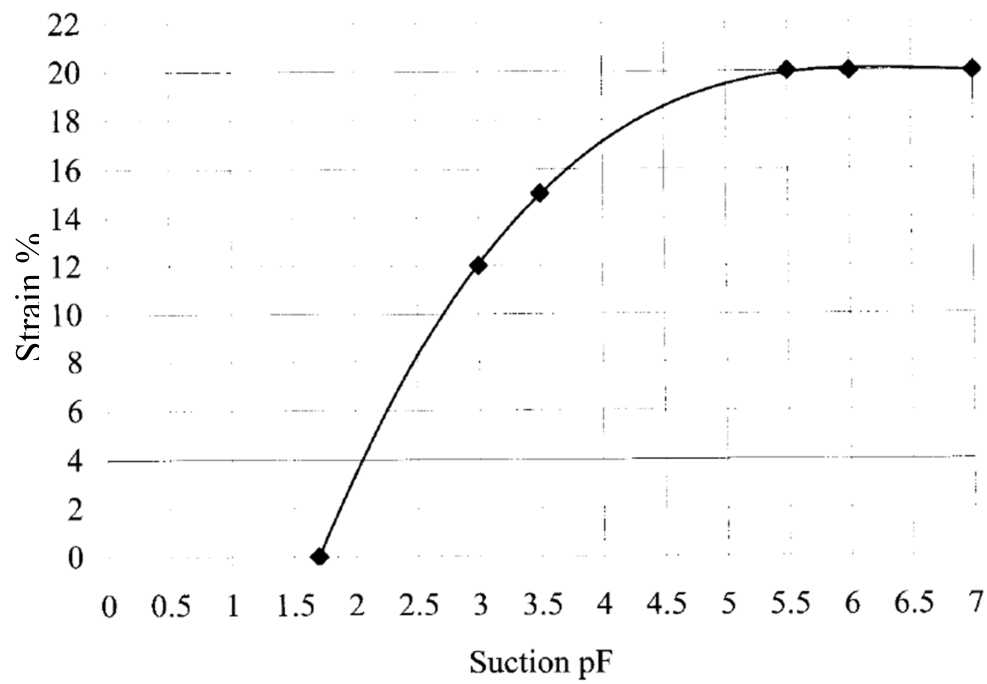

Figure 2.5. Typical Suction/Strain Relationship, after Lopes (2007)

2.6 Soil Classification Properties

Soil index properties have been used in the literature to determine suction compression index and soil expansion potential. Holtz and Gibbs (1956) developed a soil swell classification using index tests as shown in Table 2.4:

Table 2.4. Expansive Soil Classification from Holtz and Gibbs (1956), after Covar and Lytton (2001)

Colloid Content (% minus .0001 mm	Plasticity Index	Shrinkage Limit	Probable Expansion (% Vol)	Degree of Expansion
>28	>35	<11	>30	Very high
20-31	25-41	7-12	20-30	High
13-23	15-28	10-16	10-20	Medium
<15	<18	>15	<10	Low

Pearring (1968) normalized cation exchange capacity (CEC) and plasticity index (PI) to percent fine clay content in order to classify soils as to predominant mineral type. This normalization resulted in two new parameters called activity ratio (A_c) and cation exchange activity (CEA_c) (Covar and Lytton 2001). The two new parameters can be expressed as per Equations 2.15 and 2.16:

$$A_c = \frac{PI\%}{\frac{\% - 2micron}{\% - No.200sieve} \times 100} \qquad (2.15)$$

Where,

PI% = Plasticity index in percent

$\% - 2micron$ = Percent of the portion of the soil which is finer than 2 microns.

$\% - No.200sieve$ = Percent of the portion of the soil which passes the No. 200 sieve.

$$CEA_c = \frac{CEC\,\frac{milliequivalents}{100\ gm\ of\ dry\ soil}}{\frac{\% - 2micron}{\% - No.200sieve} \times 100} \tag{2.16}$$

Where,

CEC = Cation exchange capacity in milliequivalents (meq) per 100 grams of dry soil

The cation exchange capacity (CEC) can be measured with a spectrophotometer or it may be estimated with sufficient accuracy by Equations 2.17 and 2.18 (Lytton et al. 2005; Mojeckwu 1979):

$$CEC \cong (LL\%)^{0.912} \tag{2.17}$$

Where,

LL% = Liquid limit in percent

$$CEC \cong (PL\%)^{1.17} \quad (2.18)$$

Where,

LL% = Plastic limit in percent

Parameters A_c and CEA_c are used in Figure 2.6 developed by Pearring (1968):

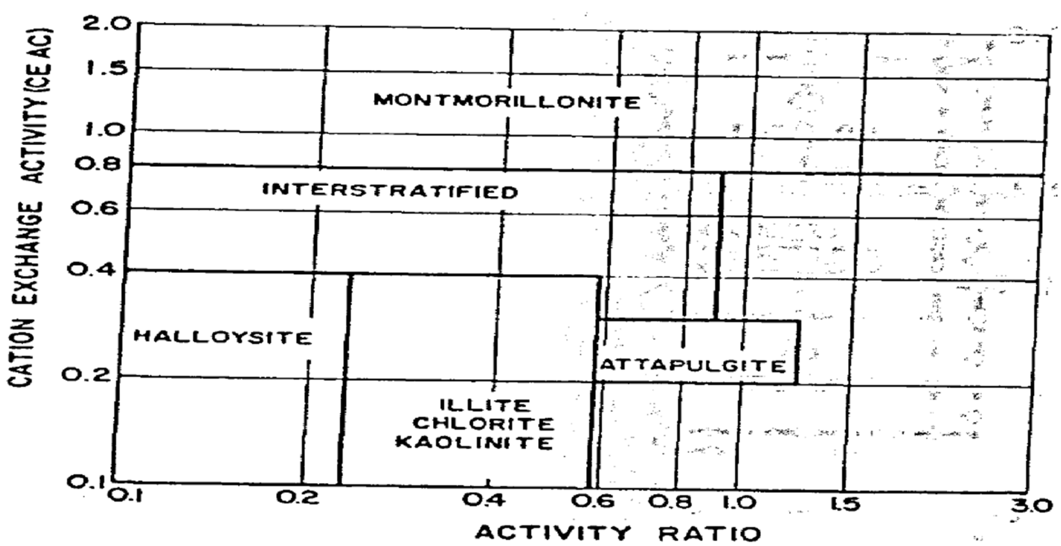

Figure 2.6. Mineralogical Classification (Covar and Lytton 2001 after Pearring 1968)

New Mexico Engineering Research Institute (NMERI) used Pearring's chart to predict a 100% fine clay suction compression index values called volume change guide numbers (γ_o or γ_{100}) without requiring suction tests by correlating COLE and the mineralogical groups (McKeen 1981; McKeen and Hamberg 1981). The resulting chart is shown in Figure 2.7:

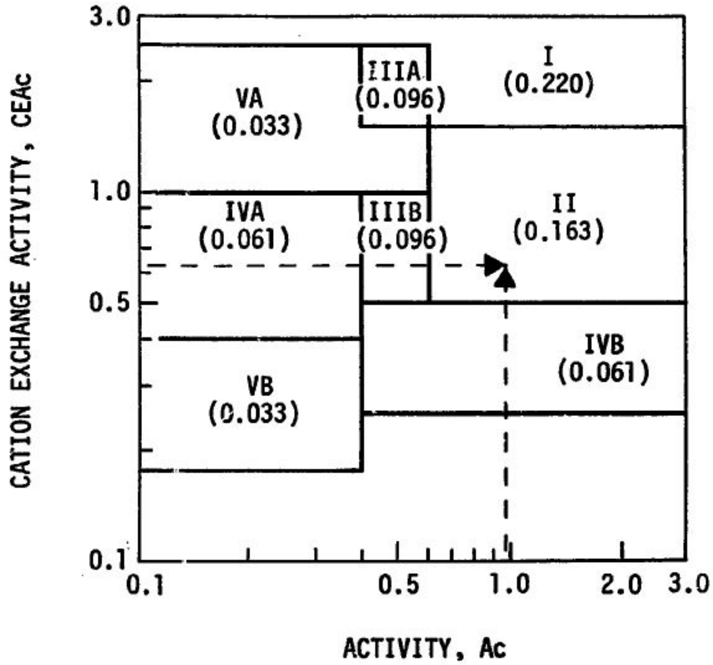

Figure 2.7. Suction Compressibility Prediction, after McKeen (1981)

Suction compression index, γ_h, can then be determined by multiplying γ_o by the clay content as a decimal as per Equation 2.19:

$$\gamma_h = \gamma_o \times \left[\frac{\% - 2micron}{\% - No.200sieve}\right] \tag{2.19}$$

Covar and Lytton (2001) used soil index properties to predict γ_h. They filtered database of Soil Survey Laboratory (SSL) of the National Soil Survey Center (NSSC) to retain only the non-null results for the following 7 tests:

- Liquid limit
- Plastic limit
- Plasticity index
- Cation exchange capacity (CEC)
- Coefficient of linear extensibility (COLE)
- % passing No. 200 sieve
- % passing 2 micron

They performed partitioning on the filtered data (which is about 6400 records) according to their mineralogy which resulted in 8 mineralogical groups. The partitioning was based on Casagrande (1948) and the Holtz and Kovacs (1981) mineral classification chart. This chart is shown in Figure 2.8. Contour charts were developed for the mineralogical groups (also referred to as zones) from which γ_{100} is first predicted (100% fine clay suction compression index value, also called volume change guide number or γ_o) and then adjusted to γ_h as per Equation 2.19. An example of the contour charts for Zone I is shown in Figure 2.9.

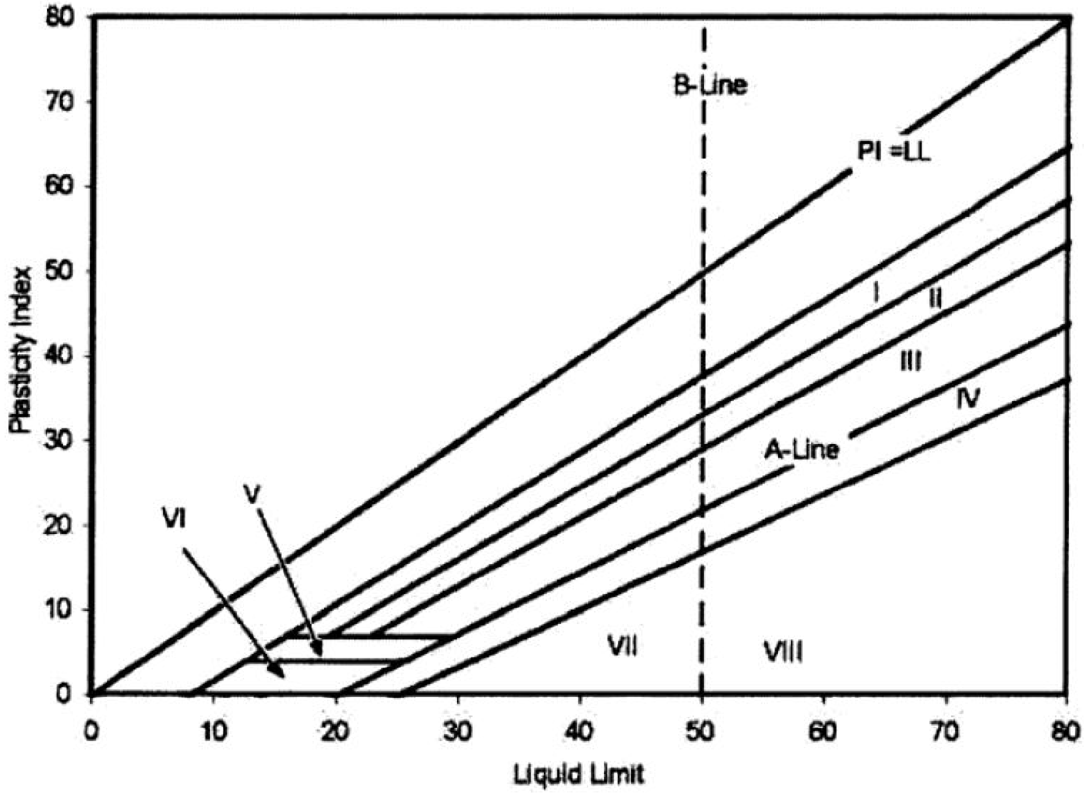

Figure 2.8. Mineral Classification Chart after Casagrande (1948) and Holtz and Kovacs (1981)

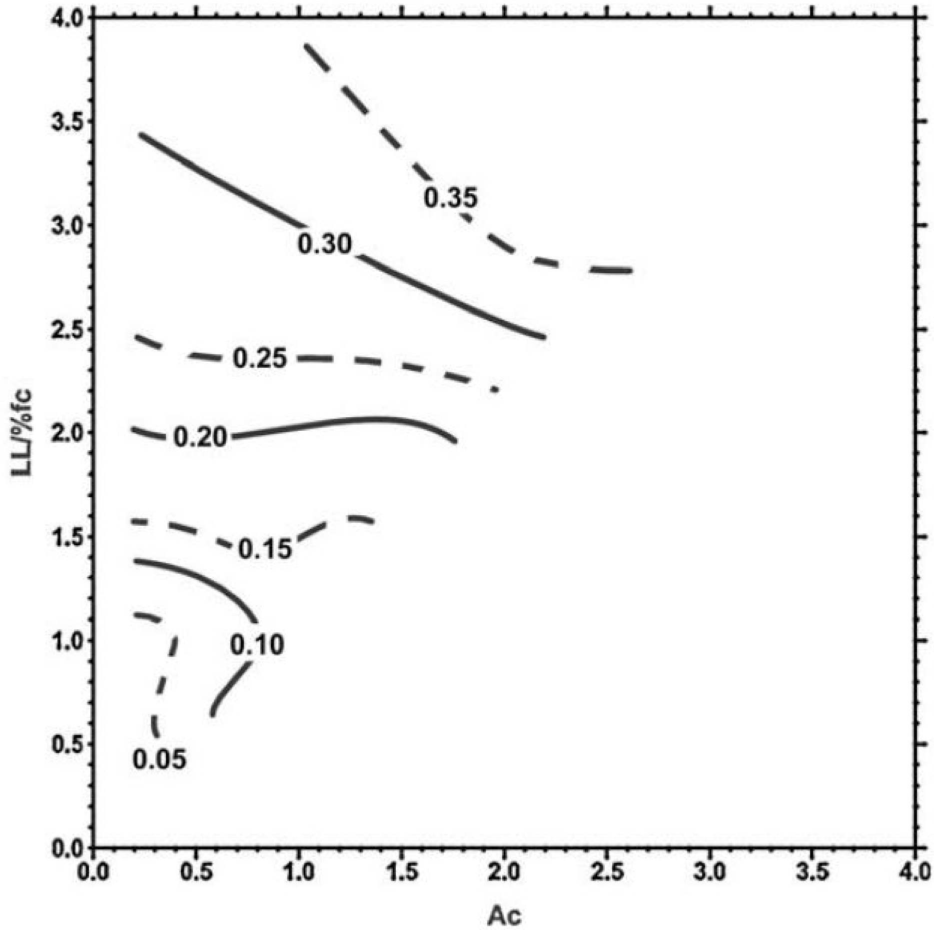

Figure 2.9. Determining γ_{100} for Zone I, after (Covar and Lytton 2001)

2.7 Volume Determination Techniques

In addition to the above presented indices determination methods, it is still beneficial to pay close attention to volume determination techniques and see how they are compared to the proposed digital imaging volume determination technique which is presented in the following chapter.

2.7.1 Sand Displacement

Yeager and Slowey (1996) introduced a simple bulk volume determination method called Ottawa Sand Displacement. Since Ottawa sand is well sorted quartz sand, it is used as the displacement material in this method. Briefly, the mass of sand required to fill an empty container is determined (M_{total} sand). Sand is poured into the empty container to fill an initial layer. Then, the sample is placed inside the container, pouring sand is resumed till the top of the container and surface is leveled. Afterwards the sand is poured into a weighing dish and the mass of the remaining sand is determined ($M_{remaining}$ sand). The mass of the sand displaced by the volume of the sample ($M_{displaced}$ sand) equals (M_{total} sand) minus ($M_{remaining}$ sand). Knowing the density of Ottawa sand, the volume of the sample is determined.

Despite of its simplicity, it is very challenging to adopt this method in the new experimental method due to the following reasons:

1. The number of data points (volumetric strain values) which can be obtained from volume determination of this method is incomparably less frequent than the digital imaging technique from the practical point of view, which accordingly will result in less representative γ_h statistical models.

2. More importantly, this method may perfectly be suitable for rocks – as suggested by Yeager and Slowey (1996) – but when it comes to soils, it becomes very questionable (if not entirely inapplicable) especially those in a moist state. The tested soil specimens range over a wide water content range which allows sand grains to cover the circumference of the specimen if used for volume determination, which will accordingly change the actual volume of specimens and might make it impractical for other measurements. Moreover, soils near saturation are more fragile than they already are, pouring sand on specimens in this state will make them vulnerable for breakage.

2.7.2 Fredlund SWCC Device

On the other hand, Fredlund SWCC device shown in Figure 2.10 is primarily used to obtain a soil-water characteristic curve of a soil specimen. Briefly, it works by placing a soil specimen inside a pressure cell. Water is dissipated from the soil specimen to scaled water columns or absorbed by the soil specimen from the water columns depending on the initial state of the soil specimen and the magnitude of pressure applied. This allows the SWCC to be easily produced by the end of the test. During the test, a dial gauge can be placed over the pressure cell to determine the specimen's heave (or shrinkage). Knowing the fixed diameter of the ring carrying the soil specimen, and by acquiring the dial gauge reading, the specimen's volume can be determined.

Figure 2.10. Fredlund SWCC Device (www.gcts.com)

Beside its time consuming nature, this method is challenging to be adopted in the proposed experimental method from the practical point of view due to the following reasons:

1. Fewer number of data points obtained.

2. The specimens' size used in the digital imaging technique (about 500 cm^3 ~ 1000 cm^3) are much larger than the ones allowed by Fredlund SWCC device (about 70 cm^3) which better serves accuracy.
3. Since specimens used in Fredlund SWCC device are laterally confined in a metal ring, the determined volume using this method will not be accurately reflecting the actual volume change. The linear/vertical change will be exaggerated on the account of null lateral change which will not necessarily be the same as a 3-dimensional volume change.

2.7.3 Photogrammetry and 3D Reconstruction

There are other methods in the literature which are used for volumetric determination. For example, there are commercially available handheld 3D laser scanners which deliver fast and accurate measurements of physical objects. They are also easy to use; however, their cost starts with tens of thousands of dollars. X-ray computed tomography (CT) equipment range between \$200,000 to \$2,000,000 (Zhang et al. 2014).

In 2014, Zhang et al. proposed a photogrammetry-based method to measure unsaturated soil volume changes during triaxial testing by reconstructing a 3D model using calibrated digital camera. Basically, the method works by taking images to the object from different angles as shown in Figure 2.11. Images are taken while having measurement targets (high-contrast dots with special design) attached on the acrylic cell of the triaxial device, loading frame and surface of membrane with soil specimen inside in order to be automatically identified by software. Based on photogrammetry, the camera orientations and acrylic cell shape and location are determined.

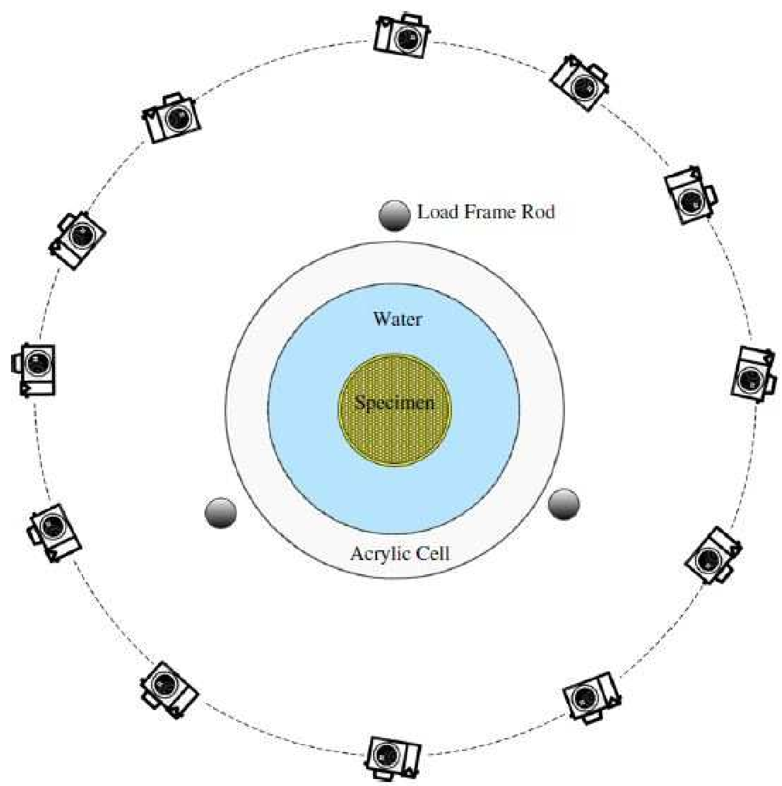

Figure 2.11. Top-View of Camera Positions during Photographing, Zhang et al. (2014)

A disadvantage of this method is being computationally intensive (Zhang et al. 2014). The sophistication of the method might make its practicality challenging. On the other hand, the volumetric determination method within the proposed γ_h testing method in this study is found to be very simple in operation. Simplicity of performing testing methods is crucial for geotechnical engineering laboratories. Moreover – unlike 3D reconstruction –

the newly proposed method does not necessitate the determination of intrinsic and extrinsic parameters known in computer vision as camera calibration.

Furthermore, in the new method, the presence of the test operator in the laboratory is minimum. The new method allows the test operator to utilize the most of their time in other operations while the test is running. The new testing method is explained in details in the following chapter.

2.8 Suction Determination Methods

There are several suction measurement instruments such as: thermal conductivity sensors, tensiometers, thermocouple psychrometers and a chilled-mirror psychrometers (WP4 Dewpoint PotentiaMeter). And there is also the filter paper method.

Each instrument or method has a certain range of soil suction measurement at which it can reliably operate and give accurate measurements. Some instruments can be inserted into undisturbed soil specimens, while others work by extracting portion of soil to be put inside a device. Two suction determination methods – filter paper and chilled-mirror psychrometer – are used in this study and reviewed in the following sub-sections. The first method is mainly relied on while the latter method is used for verification purposes.

2.8.1 Filter Paper Method

The filter paper method is an inexpensive and relatively simple laboratory test method (Bulut 2001). It is reliable for suction values of 50 kPa (2.7 pF) and greater (Bulut and Wray 2005). Basically, ash-free filter papers are put on the top of a soil specimen in a container (e.g. glass jar) with a separating O-ring (for total suction determination) as shown in Figures 2.12 and 2.13. The container is kept for a soil suction-filter paper-air equilibration period. The suction of the filter paper and the specimen in the container should be allowed to come to equilibrium for a minimum of seven days; the seven-day period is sufficient for conditions normally involved in geotechnical engineering, however under many conditions equilibration will be completed more quickly (ASTM D5298-10). The filter papers' water content is calculated after the equilibration period and suction value is then determined using calibration curve of the used filter paper.

Figure 2.12. Filter Paper O-Ring

Figure 2.13. Filter Paper Setup

2.8.2 Chilled-Mirror Psychrometer

A standard cup is filled with a piece(s) of soil and inserted into the device shown in Figure 2.14. The device uses a chilled-mirror dewpoint technique to determine the suction value. This method is easy and fast as it usually gives the suction value in about 10 minutes; however, it has a particular suction range over which its measurements can be relied on as other instruments. Chilled-mirror psychrometers operate reliably over a range between about 3.5 pF (316 kPa) and ≈ 6.48 pF (300 MPa).

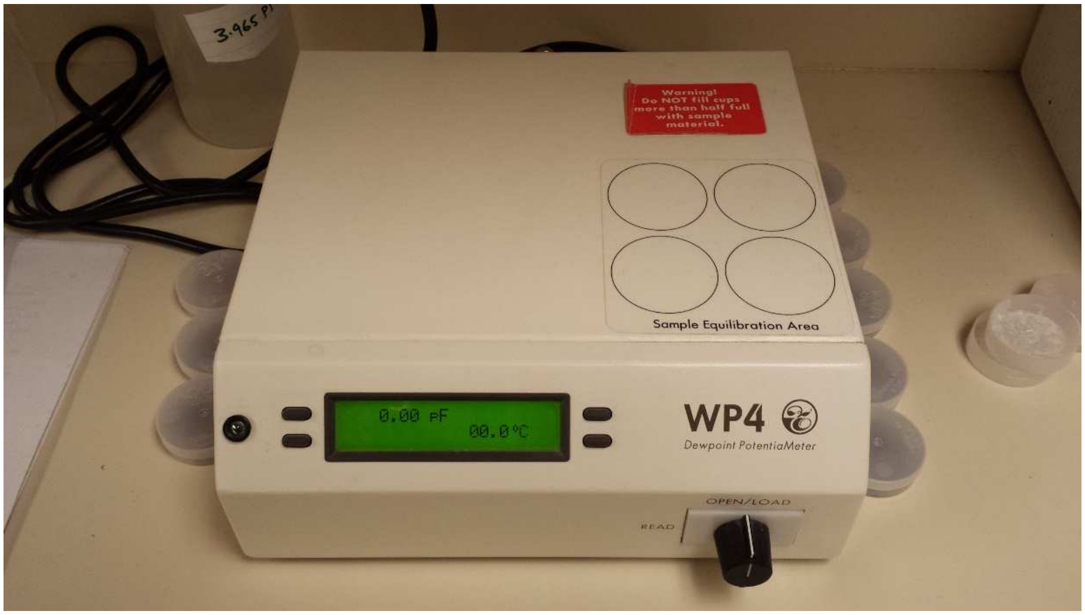

Figure 2.14. Chilled-Mirror Psychrometer Device

2.9 Other Unsaturated Soils Aspects

2.9.1 Suction Distribution in a Soil Profile

It is well known that volume change in expansive soils is function of several factors such as: moisture diffusion rate, magnitude of suction changes, depth of suction influence (moisture active zone) and level of soil expansivity that relates to its mineralogy.

The suction profiles in unsaturated subgrade soils can be illustrated as per Figure 2.15 (Bulut 2001).

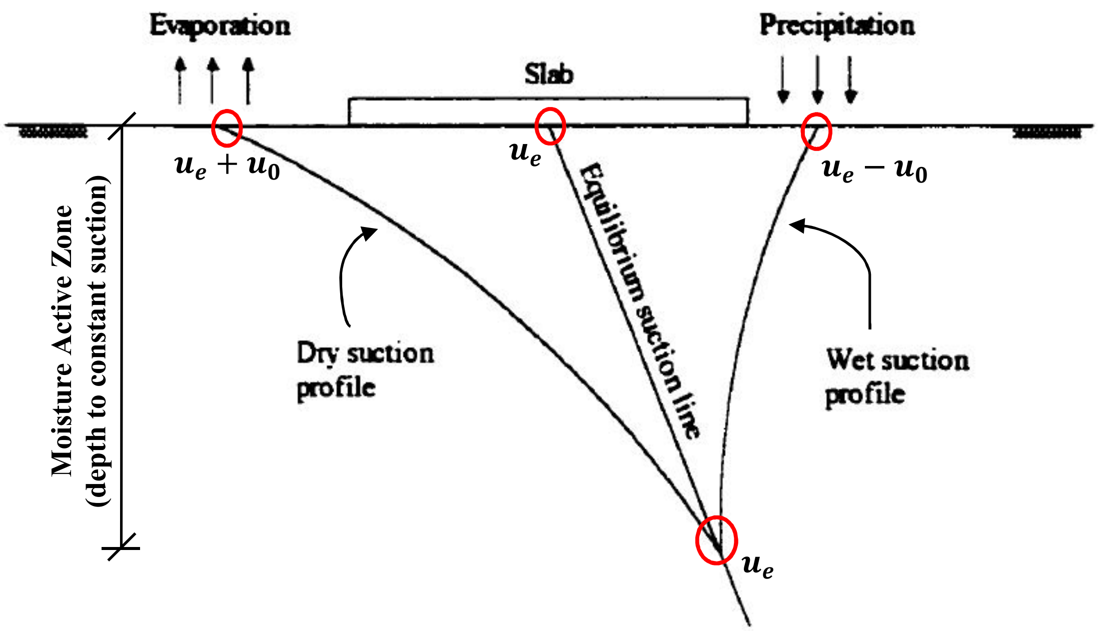

Figure 2.15. Suction Profiles, modified from Bulut (2001)

The equilibrium suction line is the line in a soil profile at which there is no net infiltration nor evapotranspiration. Equilibrium suction point, u_e, can be determined in a soil profile by measuring suction at different depths. It can also be determined from Thornthwaite Moisture Index (TMI), a climatic parameter. Witczak et al. (2006) proposed two regression models for determining matric suction using TMI. One model can be used for granular base materials while the other model can be used for subgrades.

For granular base materials, the model is expressed as per Equation 2.20:

$$h_m = \alpha + e^{[\beta+\gamma(TMI+101)]} \tag{2.20}$$

Where,

h_m = Matric suction

TMI = Thornthwaite Moisture Index

α, β, γ = Constants obtained through the regression process and the values are as shown in Table 2.5.

Table 2.5. Regression Constants for Granular Base Materials Model, after Witczak et al. (2006)

P_{200}	α	β	γ	R^2
0	3.649	3.338	-0.05046	> 0.99
2	4.196	2.741	-0.03824	> 0.99
4	5.285	3.473	-0.04004	> 0.99
6	6.877	4.402	-0.03726	> 0.99
8	8.621	5.379	-0.03836	> 0.99
10	12.18	6.646	-0.04688	> 0.99
12	15.59	7.599	-0.04904	> 0.99
14	20.202	8.154	-0.05164	> 0.99
16	23.564	8.283	-0.05218	> 0.99

Where,

P_{200} = Percent of fine-grained material

For subgrades, the model is expressed as per Equation 2.21:

$$h = \alpha \left[e^{\left[\frac{\beta}{TMI+\gamma}\right]} + \delta \right] \tag{2.21}$$

Where,

h = Matric suction

TMI = Thornthwaite Moisture Index

α, β, γ, and δ = constants obtained through the regression process and the values are as shown Table 2.6.

Table 2.6. Regression Constants for Subgrades Model, after Witczak et al. (2006)

P_{200} or wPI	α	β	γ	δ	R^2
P_{200} = 10	0.300	419.07	133.45	15.00	> 0.99
P_{200} = 50/ wPI = 0.5 or less	0.300	521.50	137.30	16.00	> 0.99
wPI = 5	0.300	663.50	142.50	17.50	> 0.99
wPI = 10	0.300	801.00	147.60	25.00	> 0.99
wPI = 20	0.300	975.00	152.50	32.00	> 0.99
wPI = 50	0.300	1171.20	157.50	27.80	> 0.99

Where,

wPI = product of P_{200} as a decimal and plasticity index as a percentage

Thornthwaite (1948) proposed the original expression for determination of TMI as per Equation 2.22.

$$TMI = \frac{100R - 60D}{PE} \quad (2.22)$$

Where,

R = Annual runoff

D = Annual deficit

PE = Annual net potential for evapotranspiration

Equation 2.22 has been modified by Thornthwaite and Mather (1955) to Equation 2.23.

$$TMI = 100\left(\frac{P}{PE} - 1\right) \quad (2.23)$$

Where,

P = Annual precipitation

Another modification by Witzack et al. (2006) is proposed as per Equation 2.24.

$$TMI = 75\left(\frac{P}{PE} - 1\right) + 10 \quad (2.24)$$

2.9.2 Moisture Diffusion Coefficient and Periodic Surface Suction Change

Historically, it is believed from the literature that Childs and Collis-George (1950) were the first to introduce the coefficient of moisture diffusion, k. They related k to the determination of vertical steady flow rate in unsaturated soils. van Genuchten (1980) introduced a soil-water diffusivity, $D(\theta)$, equation which is related to both the hydraulic conductivity and the slope of the soil-water retention curve (similar to the soil-water characteristic curve). The equation is expressed as per Equation 2.25.

$$D(\theta) = K(\theta) \left| \frac{\partial h}{\partial \theta} \right| \tag{2.25}$$

Where,

$D(\theta)$ = Soil-water diffusivity

$K(\theta)$ = Hydraulic conductivity

∂h = Change in soil water potential (i.e. soil suction)

$\partial \theta$ = Change in water content

A moisture diffusion equation was proposed by Mitchell (1979) that predicts distribution of suction in a soil profile as a function of space coordinates and time. This equation has been widely used in unsaturated soil mechanics. It can also be used to determine the depth of the moisture active zone (McKeen and Johnson 1990; Wray 1998). Moreover, it can be used to predict heave in expansive soils from a known source of moisture or suction change (Bulut 2001). If the suction change due to the effects of climate, drainage and site cover is a periodic function of time (i.e. periodic surface suction change), then the suction at any time at any depth in the soil profile can be determined by solving the diffusion equation. The general nonlinear expression of Mitchell's equation is given as per Equation 2.26:

$$\frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 u}{\partial y^2} + \frac{\partial^2 u}{\partial z^2} + \frac{f(x,y,z,t)}{p} = \frac{1}{\alpha}\frac{\partial u}{\partial t} \tag{2.26}$$

Where,

u = Soil suction

x, y, z = Cartesian coordinates

$f(x,y,z,t)$ = Moisture inflow rate per unit volume

p = Unsaturated permeability parameter

α = Soil moisture diffusivity (moisture diffusion coefficient)

t = Time

Mitchell (1979) simplified the general nonlinear form of the equation to the following linear (one-dimensional) equation by expressing suction in logarithmic units as shown in Equation 2.27.

$$\frac{\partial u}{\partial t} = \alpha \frac{\partial^2 u}{\partial x^2} \tag{2.27}$$

Where,

u = Soil suction in logarithmic units

t = Time

α = Soil moisture diffusivity coefficient

x = Distance

The one-dimensional solution for the periodic surface suction that varies in a sinusoidal manner in response to climatic cycles can be expressed as per Equation 2.28.

$$u(x,t) = u_e + u_0\, e^{\sqrt{\frac{n\pi}{\alpha}}x} \cos\left(2\pi nt - \sqrt{\frac{n\pi}{\alpha}}x\right) \tag{2.28}$$

Where,

u = Soil suction

x = Distance

t = Time

u_e = Equilibrium suction

u_0 = Initial soil suction

n = Frequency (inverse of a period/cycle and is constant for a particular climatic region)

α = Diffusion coefficient of the soil

Equation 2.28 implies that the amplitude of suction at any depth decreases exponentially as a function of moisture diffusion coefficient, α. When the cosine term yields a value of one, maximum and minimum suction boundaries can be obtained.

From the simplified linear (one-dimensional) equation (Equation 2.27), Mitchell (1979) derived equations for determination of drying and wetting diffusion coefficients in the laboratory.

For the drying diffusivity measurements, Equation 2.29 can be used.

$$u(x,t) = u_a + \sum_{n=1}^{\infty} \frac{2(u_0 - u_a)\sin z_n}{z_n + \sin z_n \cos z_n} e^{\frac{z_n^2 \alpha t}{L^2}} \cos\left[\frac{z_n x}{L}\right] \qquad (2.29)$$

Where,

u = Soil suction

x = Distance

t = Time

u_a = Laboratory atmospheric suction

u_0 = Initial soil suction

z_n = Solution of $\cot z_n = z_n/h_e L$

α = Drying diffusion coefficient

L = Length of a cylindrical soil specimen

A z_n parametric study is conducted as a part of this study and presented later in this chapter. The boundary conditions for the drying (evaporation) equation are as per Equations 2.30, 2.31, and 2.32:

$$u(x,0) = u_o \tag{2.30}$$

$$\frac{\partial u(0,t)}{\partial x} = 0 \tag{2.31}$$

$$\frac{\partial u(L,t)}{\partial x} = -h_e[u(L,t) - u_a] \tag{2.32}$$

Where,

h_e = Evaporation coefficient (can be assumed = 0.54 cm^{-1})

For the wetting diffusivity measurements, Equation 2.33 can be used.

$$u(x,t) = u_o + \frac{4(u_s - u_0)}{\pi}\sum_{n=1}^{\infty}\frac{(-1)^n}{2n-1}e^{\frac{-(2n-1)^2\pi^2\alpha t}{4L^2}}\cos\left[\frac{(2n-1)\pi x}{2L}\right] \tag{2.33}$$

Where,

u_s = Soaking suction (can be assumed = 1.75 log kPa)

α = Wetting diffusion coefficient

The boundary conditions for the wetting (soaking) equation are as per Equations 2.34, 2.35, and 2.36:

$$u(x, 0) = u_o \tag{2.34}$$

$$\frac{\partial u(0, t)}{\partial x} = 0 \tag{2.35}$$

$$u(L, t) = u_s \tag{2.36}$$

Mitchell's unsaturated moisture diffusion coefficient, α, can also be determined from both suction compression index, γ_h, and the slope of the *middle* linear portion of suction (in pF) versus water content (gravimetric) curve (soil-water characteristic curve), S, as per Equation 2.37 (Lytton 1994).

$$\alpha = 0.0029 - 0.000162(S) - 0.0122(\gamma_h) \tag{2.37}$$

Where S is a negative value.

More on the application of moisture diffusion coefficient test can be found in (Bulut et al. 2014; Bulut et al. 2013; Bulut et al. 2005).

2.9.3 The z_n Parameter

As shown earlier, the z_n parameter is one of the drying diffusion equation parameters. It can be represented by Equation 2.38.

$$\cot z_n = \frac{z_n}{h_e L} \tag{2.38}$$

Where,

h_e = Evaporation coefficient (can be assumed = 0.54 cm^{-1})

L = Length of a cylindrical soil specimen

As per the parametric study shown in Table 2.7, z_n values range from 1.4 to 28.8. Table 2.7 also shows how sensitive z_n is to different parameters. Three specimen lengths are considered. For each specimen length, four different increment-steps (Δ) are considered. For each increment, 10 π-regions are considered. This concludes 120 different solutions for z_n. For every solution of the 120 solutions, z_n is solved by a trial and error process so that the function $[F = \cot(z_n) - (z_n/h_e L) = 0]$ is satisfied under enough Δ trials.

Table 2.7. z_n Parametric Study

z_n Values												
Region	L = 15 cm				L = 20 cm				L = 25 cm			
	Δ = 0.0001 π	Δ = 0.001 π	Δ = 0.01 π	Δ = 0.1 π	Δ = 0.0001 π	Δ = 0.001 π	Δ = 0.01 π	Δ = 0.1 π	Δ = 0.0001 π	Δ = 0.001 π	Δ = 0.01 π	Δ = 0.1 π
1	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.5	1.5	1.5	1.5
2	4.2	4.2	4.2	4.2	4.3	4.3	4.3	4.3	4.4	4.4	4.4	4.4
3	7.1	7.1	7.1	7.2	7.3	7.3	7.3	7.3	7.4	7.4	7.4	7.4
4	10.1	10.1	10.1	10.1	10.2	10.2	10.2	10.3	10.3	10.3	10.3	10.3
5	13.1	13.1	13.1	13.1	13.3	13.3	13.3	13.3	13.4	13.6	13.6	13.4
6	16.2	16.2	16.2	16.2	16.3	16.3	16.3	16.3	16.4	16.4	16.4	16.4
7	19.2	19.2	19.2	19.3	19.4	19.4	19.4	19.4	19.5	19.5	19.4	19.5
8	22.3	22.3	22.3	22.4	22.4	22.4	22.4	22.5	22.5	22.5	22.5	22.6
9	25.4	25.4	25.4	N/A	25.5	25.5	25.5	25.6	25.6	25.6	25.6	25.7
10	28.6	28.6	28.6	N/A	28.6	28.6	28.6	28.7	28.7	28.7	28.7	28.8

2.9.4 Soil-Water Characteristic Curve (SWCC)

Soil-Water Characteristic Curve (SWCC) is a very important aspect of unsaturated soil mechanics. SWCC is a non-linear relationship which describes the relationship between soil suction and water content (gravimetric or volumetric). Degree of saturation can also be used as a replacement to water content in establishing the SWCC. The inverse of the slope of SWCC ($\Delta h/\Delta w$ or S) is of significant importance as well. The slope inverse is very commonly referred to in the literature as SWCC slope. It is related to several unsaturated soil mechanics parameters such as: suction compression index (γ_h), thornthwaite moisture index (TMI), moisture diffusion coefficient (α), and others. By mentioning SWCC slope, it is meant the slope over a small suction change.

Slope of SWCC ($\Delta h/\Delta w$ or S) can be determined empirically as per Equation 2.39 (Jayatilaka and Lytton 1999; Lytton et al. 2005):

$$S = -20.29 + 0.155(LL) - 0.117(PI) + 0.0684\ (percent\ Fines) \tag{2.39}$$

Where,

S = Slope of SWCC

LL = Liquid limit

PI = Plasticity index

$percent\ Fines$ = Percentage of particle sizes passing the #200 sieve on a dry weight basis

McKeen and Johnson (1990) mentioned that the parameters that indicate the response of soils to suction changes are the best indicators to the moisture diffusion coefficient (α). Therefore, they proposed a multiple linear regression model in order to determine α by investigating the slope of SWCC ($\Delta h/\Delta w$ or S), suction compression index (γ_h), and thornthwaite moisture index (TMI). The model is based on data gathered from sites in the states of Texas, New Mexico, and Mississippi. The model's coefficient of determination (R^2) = 0.999, the standard error of estimate = 0.000062, and the number of observations = 6.

The model is expressed as per Equation 2.40:

$$\alpha = b_0 + b_1(TMI) + b_2\left(\frac{\Delta h}{\Delta w}\right) + b_3\gamma_h \tag{2.40}$$

Where,

b_0 = 0.010134

b_1 = 0.000002

b_2 = 0.05468

b_3 = -0.03509

$\frac{\Delta h}{\Delta w}$ = Suction-water content slope (SWCC slope)

TMI = Thornthwaite moisture index

γ_h = suction compression index

Based on test data from several field monitoring sites, McKeen (1992) proposed an equation for determination of γ_h from $\Delta h/\Delta w$. The equation is basically an expression of a line representing the 85th percentile of the gathered data. The expression is shown as per Equation 2.41:

$$\gamma_h = (-0.02673)\left(\frac{\Delta h}{\Delta w}\right) - 0.38704 \tag{2.41}$$

Based on a statistical linear regression of a population of 69 data points, Perko et al. (2000) also proposed an equation similar to McKeen's equation for γ_h determination from $\Delta h/\Delta w$. Perko et al. (2000) claims that although McKeen's empirical equation fits data points in the midrange, the empirical equation they proposed is more dependable at the extremes as shown in Figure 2.16.

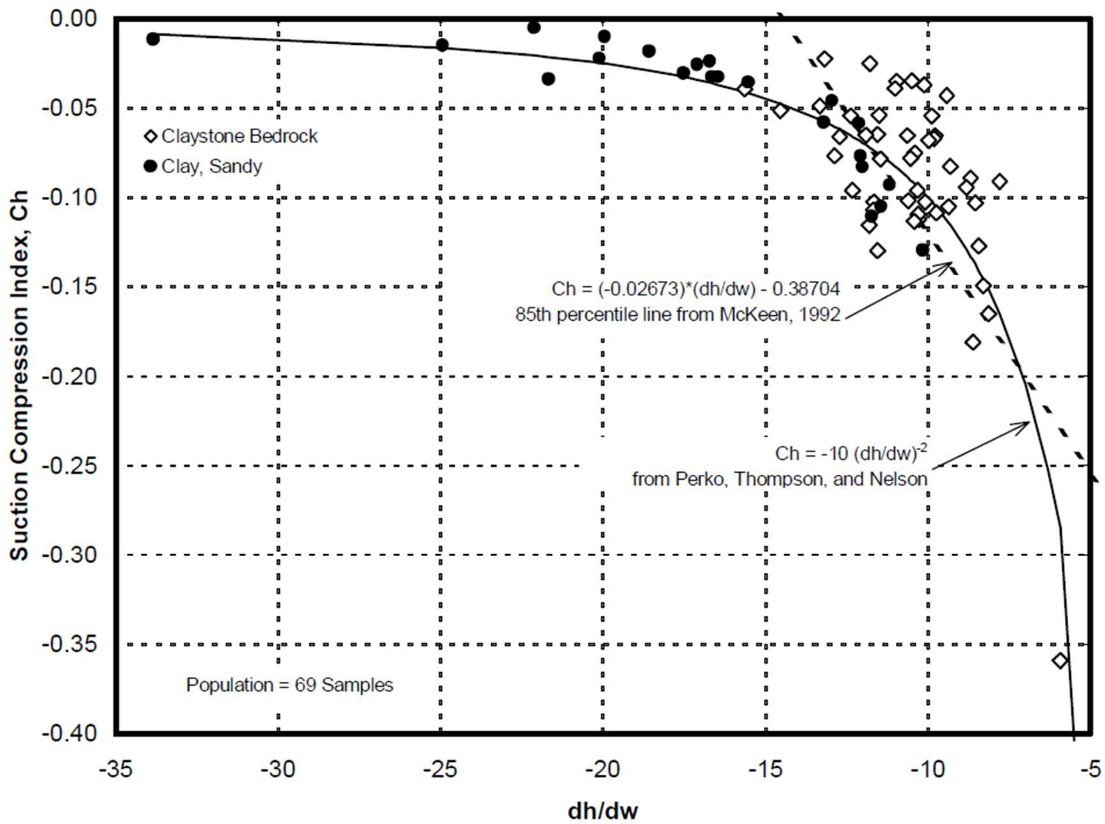

Figure 2.16. Suction to Water Content Ratio and Suction Compression Index Data, after Perko et al. (2000)

The equation of Perko et al. (2000) can be expressed as per Equation 2.42:

$$\gamma_h = (-10)\left(\frac{\Delta h}{\Delta w}\right)^{-2} \tag{2.42}$$

Where,

γ_h = Suction compression index

$\frac{\Delta h}{\Delta w}$ = Suction (in pF) to water content ratio

A Typical SWCC for silty soil and comparative drying SWCCs for different types of soils are shown in Figures 2.17 and 2.18, respectively.

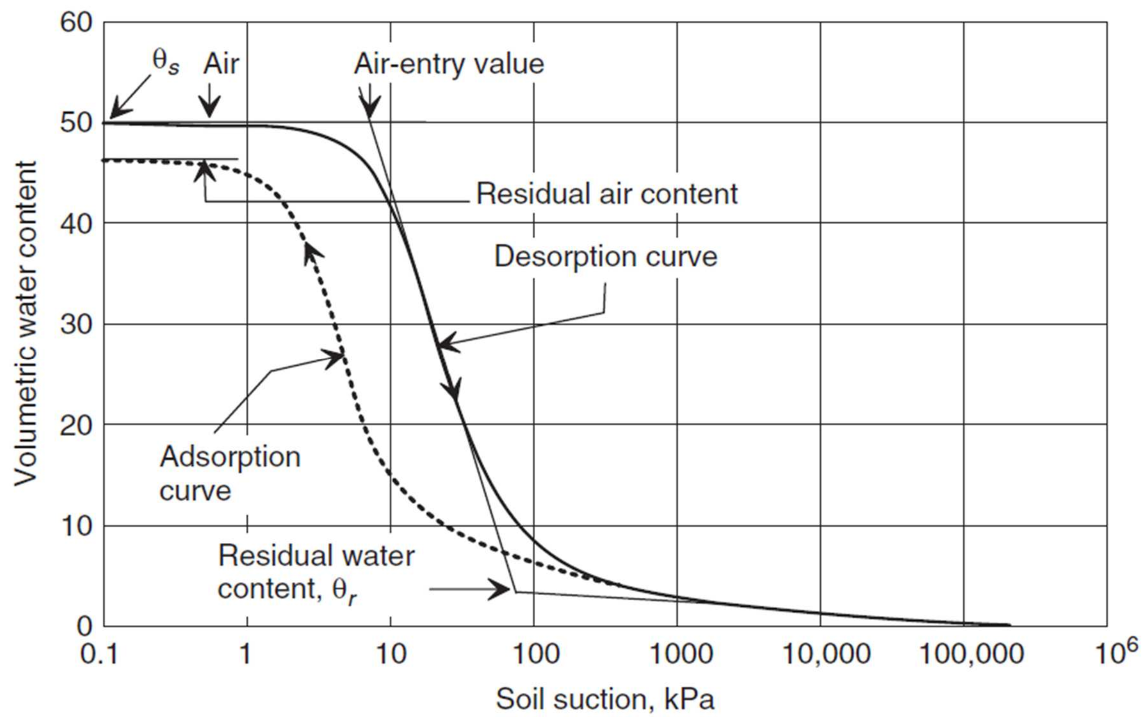

Figure 2.17. Typical SWCC for Silty Soil, after Fredlund et al. (2012)

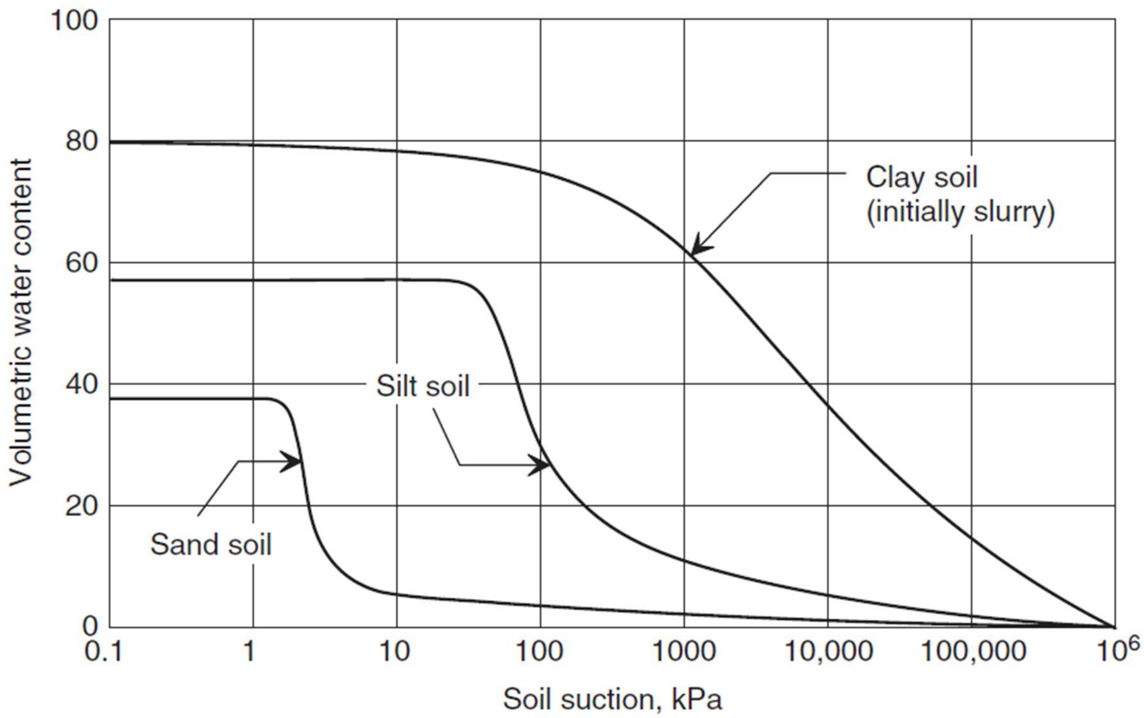

Figure 2.18. Comparative Desorption (Drying) SWCCs for Sandy, Silty, and Clayey Soil, after Fredlund et al. (2012)

The entire soil-water characteristic curve (SWCC) can be determined from an equation proposed by Fredlund and Xing (1994). This equation served as a main cornerstone to this research. The equation can be expressed as per Equation 2.43:

$$W(\psi, a, n, m) = C(\psi)\frac{W_s}{\{\ln[e + (\psi/a)^n]\}^m} \tag{2.43}$$

Where;

W = Water content, decimal

W_s = Water content at saturation, decimal

ψ = Total suction, kPa

a, n, m = Curve fitting parameters (a is in kPa, n and m are unitless)

$C(\psi)$ = Correction function defined as per Equation 2.44

$$C(\psi) = \frac{-\ln(1 + \psi/\psi_r)}{\ln[1 + (10^6/\psi_r)]} + 1 \qquad (2.44)$$

Where;

ψ_r = Suction corresponding to residual water content. Numerical results performed by Fredlund and Xing (1994) show that ψ_r (residual suction) ranges from 1500 kPa to 3000 kPa in general.

2.9.5 Unsaturated Permeability and Unsaturated Flow

The movement of groundwater in soil is mainly governed by its hydraulic gradient. In saturated soils, gravity is the governing factor for movement of groundwater. While in unsaturated soils, suction (capillarity effect) becomes the governing factor as groundwater tends to move towards an area of higher suction (area of more negative pore water pressure). Surface infiltration and evaporation affect the suction gradient along unsaturated soils profile beneath (i.e. along the depth of the moisture active zone).

The unsaturated permeability parameter is related to moisture diffusion coefficient, α, as per the Equation 2.45 under condition that α is assumed to be constant over a small range of suction change.

$$p = \frac{\alpha \gamma_d}{|S| \gamma_w} \tag{2.45}$$

Where,

p = Unsaturated permeability parameter

$|S|$ = Absolute value of the slope of the linear portion of suction versus gravimetric water content curve (soil-water characteristic curve)

γ_d = Dry unit weight of the soil

γ_w = Unit weight of water

α = Moisture diffusion coefficient

On the other hand, Mitchell's unsaturated permeability is related to saturated permeability, k_0, and suction as per Equation 2.46.

$$p = \frac{k_0|h_0|}{0.4343} \tag{2.46}$$

Where,

h_0 = Constant suction value of approximately -100 cm for clays.

Additionally, Laliberte et al. (1966) also related the unsaturated permeability coefficient to the saturated permeability coefficient and suction as per Equation 2.47.

$$k(u) = k_0\left(\frac{u_0}{u}\right)^n \tag{2.47}$$

Where,

$k(u)$ = Unsaturated permeability coefficient

k_0 = Saturated permeability coefficient

u_0 = Constant suction value of approximately -100 cm for clays

u = Soil suction

n = Soil parameter (n = 1 for clay; n = 1~3 for sand)

Unsaturated hydraulic conductivity is function of the vertical steady flow rate in unsaturated soils. From Darcy's Law (Darcy 1856) for groundwater flow in porous media under saturated conditions, the vertical steady flow rate in unsaturated soils can be determined as per the ordinary differential equation shown in Equation 2.48 (Richards 1931; Childs and Collis-George 1950; Gardner 1958).

$$Q = -k(h)\left\{\frac{\partial h}{\partial z} + 1\right\} \tag{2.48}$$

Where,

Q = Unsaturated vertical steady flow rate

$k(h)$ = Unsaturated hydraulic conductivity

h = Soil suction

z = Vertical coordinate

Ross (2003) developed a numerical method for the solution of the equation proposed by Richards (1931). The solution was referred to as the fast method due to its non-iterative nature. Thus, it is relatively faster the other iterative solutions. Varado et al. (2006) tested this numerical solution and mentioned that it is applicable to unsaturated or saturated soils, and homogeneous or heterogeneous soil columns.

The maximum possible vertical water evaporation rate at the surface of a soil column is function in the unsaturated vertical steady flow rate as per Equation 2.49.

$$\int_0^L \partial z = -\int_0^L \frac{1}{\left[\frac{e}{k(h)} + 1\right]} \partial h \tag{2.49}$$

Where,

e = Maximum possible vertical water evaporation rate at the surface of a soil column

$k(h)$ = Unsaturated hydraulic conductivity

L = Full height of a soil column

z = Vertical coordinate

h = Soil suction

CHAPTER III

THE TESTING METHOD: VOLUME AND SUCTION DETERMINATION USING DIGITAL IMAGING

Undisturbed 3-inch diameter Shelby tube soil specimens have been collected from three different sites in the state of Oklahoma in April 2014 in order to be used in the testing method for determination of suction compression index. The first site is Kirkland; which is located east of the city of Enid and northwest of the city of Stillwater (on the southwest bound of intersection of US-412/US-64 and OK-74/OK-15) in Garfield county. The second site is Port; which is located west of the town of Sentinel and west of the city of Oklahoma City as well (on OK-55) in Washita county. The third site is Osage; which is located west of the city of Wagoner and southeast of the city of Tulsa (on OK-51) in Wagoner county. Figure 3.1 is a snapshot which has been taken from Google Earth showing the location of the three sites in the state of Oklahoma.

All tests have been performed in Oklahoma State University laboratories. Index properties tests have been performed as per ASTM standards.

ASTM D4318 has been followed for Atterberg limits tests, while ASTM D422 has been followed for sieve analysis and hydrometer analysis tests.

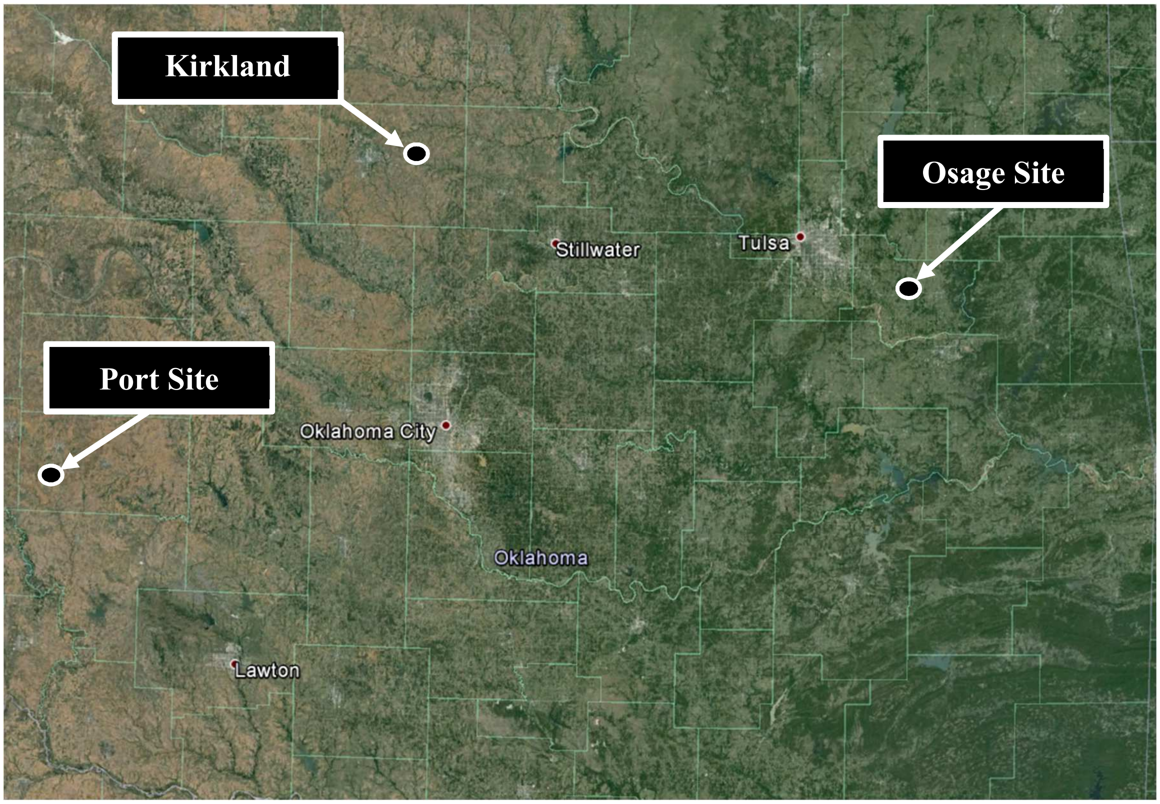

Figure 3.1. Soil Sampling Locations

Ranges of index properties of soil specimens from the three sites are summarized in Table 3.1. The testing method is an integrated method which comprises volume measurements, water content measurements and eventually suction measurements as specimens dry out and gets wet. Measurement methods are described below.

Table 3.1. Index Properties Ranges

	Kirkland	Port	Osage
Liquid Limit (%)	37 ~ 39	46 ~ 50	55 ~ 60
Plastic Limit (%)	19 ~ 23	23	25 ~ 30
Plasticity Index (%)	14 ~ 20	23 ~ 27	28 ~ 30
Percent passing sieve No. 200 (0.075 mm)	9 ~ 32	16 ~ 27	13 ~ 22
Percent passing 2 micron size (clay content)	5 ~ 10	8 ~ 12	8 ~ 12

3.1 Volumetric Changes Monitoring

The volume measurement method, as specimens sit for drying tests, utilizes digital imaging to capture volumetric changes on frequent basis. A high-resolution Canon EOS Rebel T5i DSLR (Digital Single-Lens Reflex) camera with resolution of 18.50 megapixels (*total pixels*) and 18.0 megapixels (*effective pixels*) is used. Its pixel unit is 4.3 µm square and the associated lens kit with it is 18-55mm IS STM. An intervalometer is also used to set the desired imaging frequency. KLONK Image Measurement software is used to take height and diameter measurements of cylindrical soil specimens for volume determination – as shown in Figure 3.2 – at different periods of time. It is expected that the soil specimen geometry does not remain perfectly cylindrical over the drying (or wetting) course. Thus, for every image measurement process, averages of several heights and diameters at different locations on the soil specimen are taken. This takes into account errors that might occur due to geometric irregularities. Also, in order to avoid distortion that may affect specimen measurements, it is preferable that the camera zoom be adjusted in a way that the soil specimen be captured in the middle third of the image frame as illustrated in Figure 3.2. The image measurement software necessitates that a ruler is placed next to the specimen for measurement calibration purposes. Prior to first use, a checkerboard calibration pattern printout with pre-known squares sizes has been captured by the camera from the same angle and distance that will be used for capturing soil specimens. Then the image was measured by the digital image measurement software. This has been performed in order to ensure that the camera measurements are accurate for more than one location in a captured frame.

As for the wetting tests, this same setup can apply. However, since the specimen is kept wrapped in a moist environment, images are not taken continuously and the intervalometer is not used.

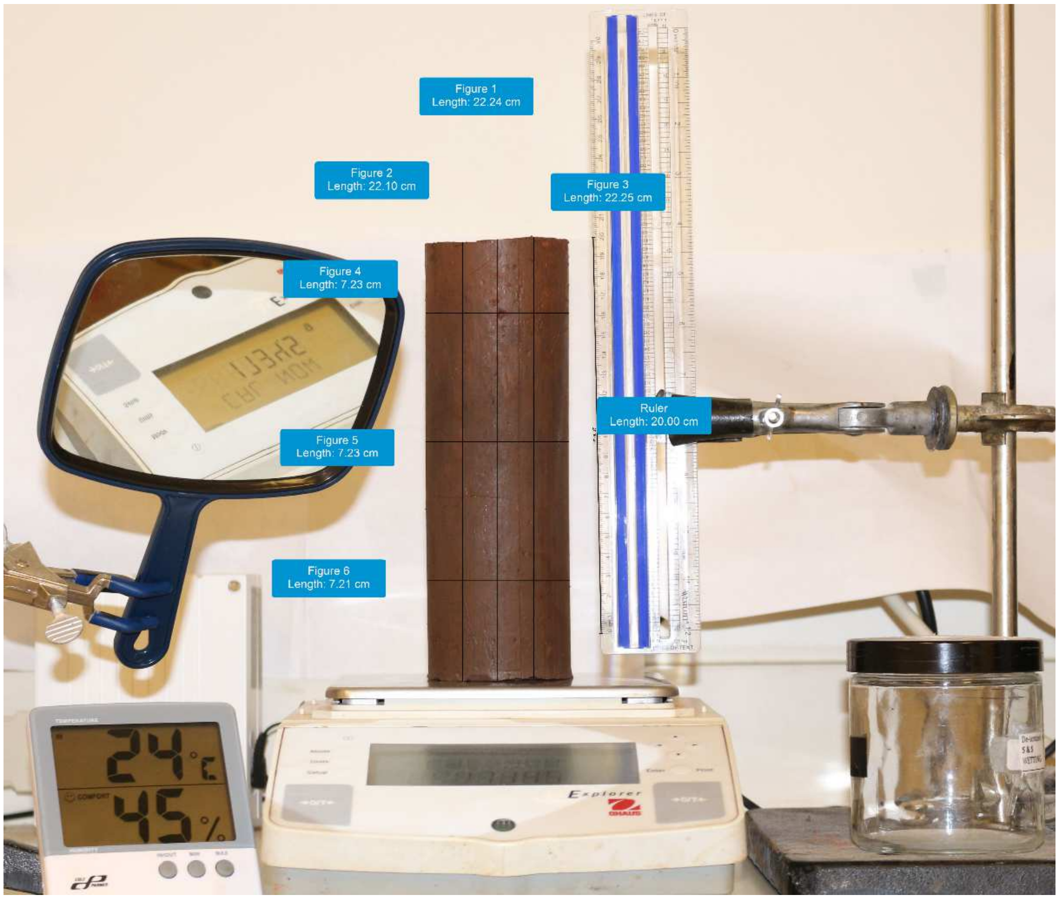

Figure 3.2. Soil Specimen Captured During a Drying Test

3.2 Suction and Water Content Measurements

The drying and wetting testing methods are explained in detail under the following two subsections. Those are the core of the newly developed testing method that leads to determination of the traditional suction compression index (γ_h). Eventually after applying statistical models explained later, an incremental γ_h is determined. The incremental γ_h is a unique and original index that takes into consideration the non-linearity of the suction-volumetric strain relationship. Therefore, it is a representative to the entire relationship between suction and volumetric strain.

3.2.1 Suction and Water Content Measurements for γ_h Determination Based on Drying Test

A moist undisturbed Shelby tube soil specimen is selected for air-drying shrinkage testing at a temperature-controlled environment. Then, the following detailed yet simple steps are followed:

1) Starting from initially moist soil specimen, the specimen is put on a balance.

2) An intervalometer is connected to the high-resolution digital camera and set to a specific time interval (e.g. 30 minutes). Then, image capturing process is started.

3) Over time, the specimen will start shrinking. As the camera takes images for volumetric measurements at specific time intervals (e.g. 30 minutes), the weight of the specimen is recorded in the same frequency as well as shown in Figure 3.2. This will allow for the production of a long series of volume measurements along with its corresponding weights.

4) The test is stopped when volume change seizes. This is challenging to be determined while specimen is still in the testing setup. Thus, this may be translated in terms of time. It is recommended that the test is stopped when in progress for at least 40 hours. Based on the tests conducted in this study, the rate of overall (accumulated) weight loss becomes relatively lower than the beginning of the test after an elapsed time of about 36 to 40 hours. Consequently, water content gives the same observation, thus the volume as well. The drying test duration may vary depending on the tested soil type.

5) The specimen is then cut into pieces and divided into few glass jars as shown in Figure 3.3. It is important to ensure that there is almost no soil is wasted.

Figure 3.3. Preparation for Filter Paper Test

6) Different amounts of de-ionized or distilled water are added to each glass jar. It should be taken into consideration that the added amounts of water should not be excessive so that the maximum water content would not exceed the range of 26% ~ 30%. It has been noticed from the results of tests performed in this study that this water content range is equivalent to 100% degree of saturation. Adding water above 100% degree of saturation to glass jars will affect the process of generating the soil-water characteristic curve (SWCC) referred to in step No. 10. The SWCC is a non-linear relationship which is governed by several parameters. One of which is the water content corresponding to 100% degree of saturation. Having a suction value corresponding to water content higher than 100% degree of saturation will not contribute as a data point towards generating the SWCC.
7) The jars are then sealed using vinyl electrical tape for equilibration as shown in Figure 3.4.

Figure 3.4. Sealed Jar for Equilibrium Prior to Filter Paper Test

8) The filter paper test is started afterwards for suction determination as shown in Figures 3.5 and 3.6.

Figure 3.5. Installing Filter Paper Base for Suction Determination

Figure 3.6. Installing Filter Papers for Suction Determination

9) Soils from the glass jars are carefully poured in a moisture tin for oven-drying as shown in Figure 3.7. This is to determine the water content of each filter paper test performed in the previous step. It is very important that all soils from glass jars are successfully poured in the corresponding moisture tins and that no soils are wasted out of the moisture tin on the laboratory bench. Failure to do so may result in errors that might affect the overall dry weight of the soil specimen, thus the water content value at each selected photo (data point). It may also result in errors that might affect the generation of the soil specimen's soil-water characteristic curve accuracy, consequently the suction value at each selected photo.

Figure 3.7. Carefully Pouring Soil from Glass Jar into Moisture Tin for Oven-Drying

10) Now the soil-water characteristic curve (SWCC) equation can be determined using the water content and suction values which are determined from steps 8 and 9. This equation can be determined as per Fredlund and Xing (1994) as shown in Equation 3.1. The Curve Fitting Toolbox of the software MATLAB has been used to solve for the curve fitting parameters (a, n and m) of the non-linear SWCC equation and determining the optimized sum of squared errors. The toolbox applies Nonlinear Least Squares optimization/minimization method using Levenberg–Marquardt algorithm (LMA). Nonlinear least squares methods involve an iterative improvement to parameter values in order to reduce the sum of the squares of the errors between the function and the measured data points (Gavin

2013). The Levenberg-Marquardt optimization algorithm used in curve-fitting method is essentially a combination of two optimization methods: The Gradient Descent method and the Gauss-Newton method (Gavin 2013).

$$W(\psi, a, n, m) = C(\psi)\frac{W_s}{\{\ln[e + (\psi/a)^n]\}^m} \tag{3.1}$$

Where,

W = Water content, decimal

W_s = Water content at saturation, decimal

ψ = Total suction, kPa

a, n, m = Curve fitting parameters (a is in kPa, n and m are unitless)

$C(\psi)$ = Correction function defined as per Equation 3.2

$$C(\psi) = \frac{-\ln(1 + \psi/\psi_r)}{\ln[1 + (10^6/\psi_r)]} + 1 \tag{3.2}$$

Where,

ψ_r = Suction corresponding to residual water content. Numerical results performed by Fredlund and Xing (1994) show that ψ_r (residual suction) ranges from 1500 kPa to 3000 kPa in general. Lower limit, mid-range and upper limit are attempted to see which value gives the best fit.

11) Weights of oven-dried soils from moisture tins (from step No. 9) are added up altogether. Water content of the shrinking soil specimen can now be easily determined at any selected image time interval over the entire course of the drying test. Few images are selected.

12) By plugging-in the selected water content values determined from step No. 11 into the SWCC equation determined from step No. 10 (Equation 3.1), suction values corresponding to every selected water content value can now be obtained. Using both the suction values and the volume measurements taken by the digital imaging process, a drying suction compression index can be traditionally determined as per Equation 1.1. However, it is important to highlight that an incremental γ_h approach is proposed as explained later in Section 3.4. Incremental γ_h approach will allow for determination of γ_h corresponding to every selected suction value. Consequently, the entire non-linear suction-volumetric strain relationship is covered, not just a linear portion of it, nor an idealized linear relationship.

3.2.2 Suction and Water Content Measurements for γ_h Determination Based on Wetting Test

The wetting test procedure is explained as per the following steps:

1) An initially air-dry soil specimen is placed on a balance for an initial image capturing for weight and volume determination.

2) Due to its impervious and flexible nature, plastic wrap is placed on the laboratory bench as shown in Figure 3.8.

Figure 3.8. Saturated Cloth on the Top of Plastic Wrap

3) A piece of cloth or towel saturated with de-ionized or distilled water is placed over the plastic wrap. Afterwards, the soil specimen is taken off the balance and horizontally placed on the towel as shown in Figure 3.9.

Figure 3.9. Placement of Soil Specimen

4) The soil specimen is gently yet adequately wrapped by both the towel and the plastic wrap from all sides so that the specimen is not subjected to the air as shown in Figure 3.10.

Figure 3.10. Wrapping Soil Specimen by Saturated Cloth and Plastic Wrap

5) As time passes, the towel shall be drying out as the specimen absorbs the water from it. Accordingly, from time to time, the towel shall be gently opened and re-watered – by spraying de-ionized/distilled water – to maintain continuous water transmission to the soil specimen. Then wrapped again.

6) Every few hours, the soil specimen is carefully taken out on the balance for image capturing and returned back. At least one volume-increase image must be secured. It is helpful that this step is repeated as long as the soil specimen can be held (i.e.

is not nearly saturated to an extent that it breaks under its own weight or during handling due to the drop in its shear strength).

7) Subsequently, a wetting γ_h can be determined in the same described manner in the drying test. If the wetting test follows the drying test on the same soil specimen, step No. 5 of the drying test procedures shall be postponed until the wetting test is completed first.

3.3 Testing Parameters

In addition to determining suction compression index based on the newly proposed testing method, comparisons will be made between the resulting incremental γ_h (after application of the statistical models explained later) and γ_h based on COLE value from Equations 2.4, 2.5, and 2.6 proposed by Covar and Lytton (2001) as well as γ_h based on index properties.

The parameters/variables that are measured, calculated, recorded and/or determined in this research are shown as per Figure 3.11 – the flowchart – shown below for each of the three sites:

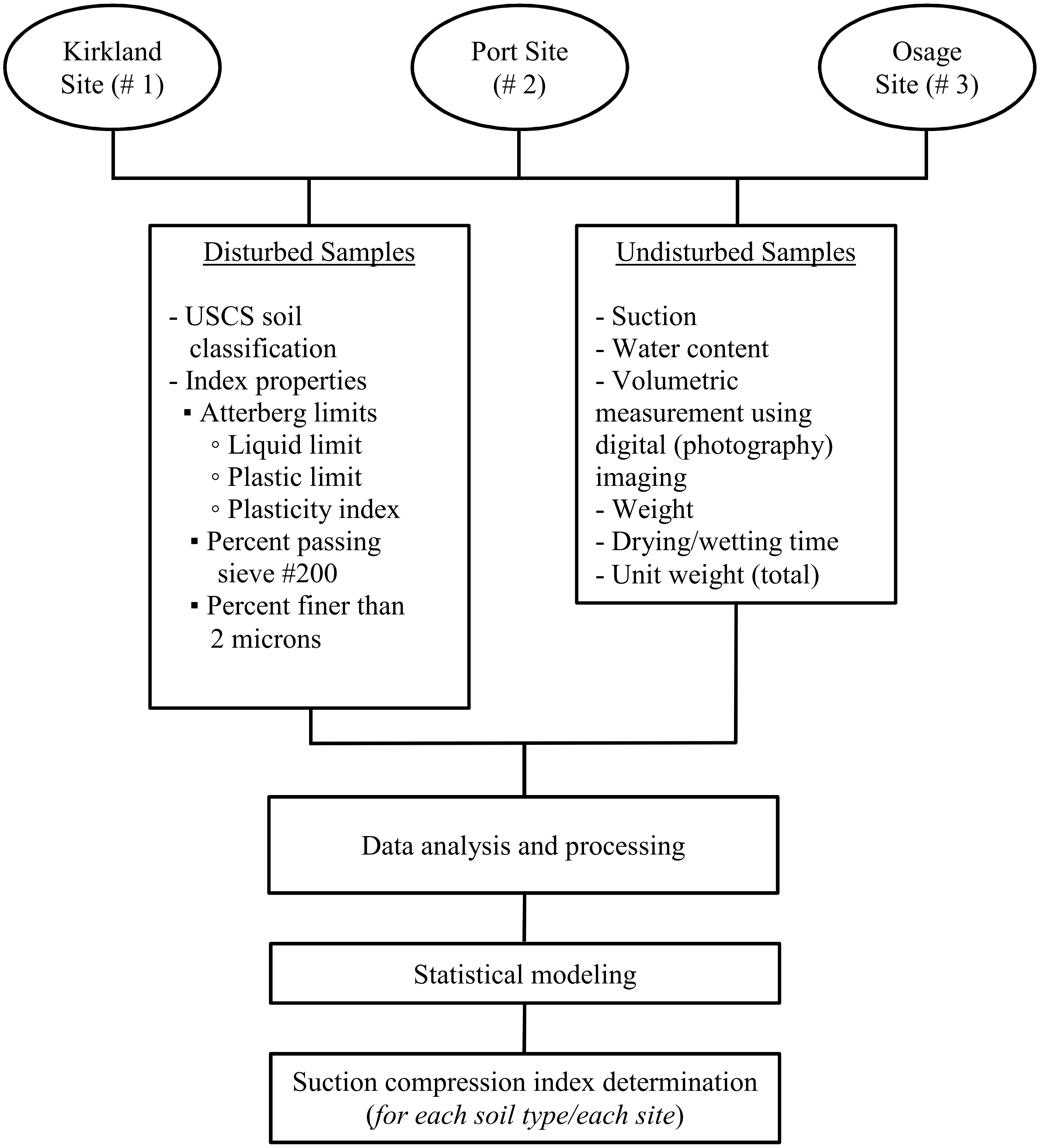

Figure 3.11. Research Framework

3.4 Determination of Incremental Suction Compression Index Using Statistical Models

Since Suction Compression Index (γ_h) is directly related to the suction-volumetric strain relationship, it is important and beneficial that statistical models be developed to reflect this relationship for different types of soils as explained later in this section. From the statistical models, γ_h can be determined in a new and unique method which has not been found to be used in the literature. Traditionally, γ_h is determined by calculating the slope of the linear potion of the suction-volumetric strain relationship which is non-linear in nature or making an assumption that it is a linear relationship and then considering its slope. In this case, only two volumetric strain values and two suction values are used for the slope determination as per Equation 1.1. However, by implementing the statistical models described later in this section, γ_h will be determined in an incremental manner. This means we will be able to determine γ_h at each corresponding suction value along the entire non-linear suction-volumetric strain relationship. This will allow for the use of all suction values instead of neglecting some of them which lie outside the linear portion of the relationship as followed by the traditional approach. Since the newly proposed approach requires many data points than just two or three, this will apply only on the drying component of the testing method. This is because it is quite challenging to secure as such number of data points in the wetting component as in the drying component due to the nature of the test explained in step No. 6 of the wetting test procedures.

From the literature, linearity of relationships similar to suction-volumetric strain relationship is assumed as shown in Figure 3.11 or similar relationships as shown earlier in Figures 2.1 and 2.2. Consequently, only two data points are considered for determination of the calculated indices, while other data points are not taken into consideration. In fact, the presented relationships are not perfectly linear, therefore it is important to take into consideration as many data points as possible especially when some relationships are based on only two data points. The more data points taken into consideration in determination of the index in question, the more accurate and reliable this index becomes specially when the relationship is non-linear.

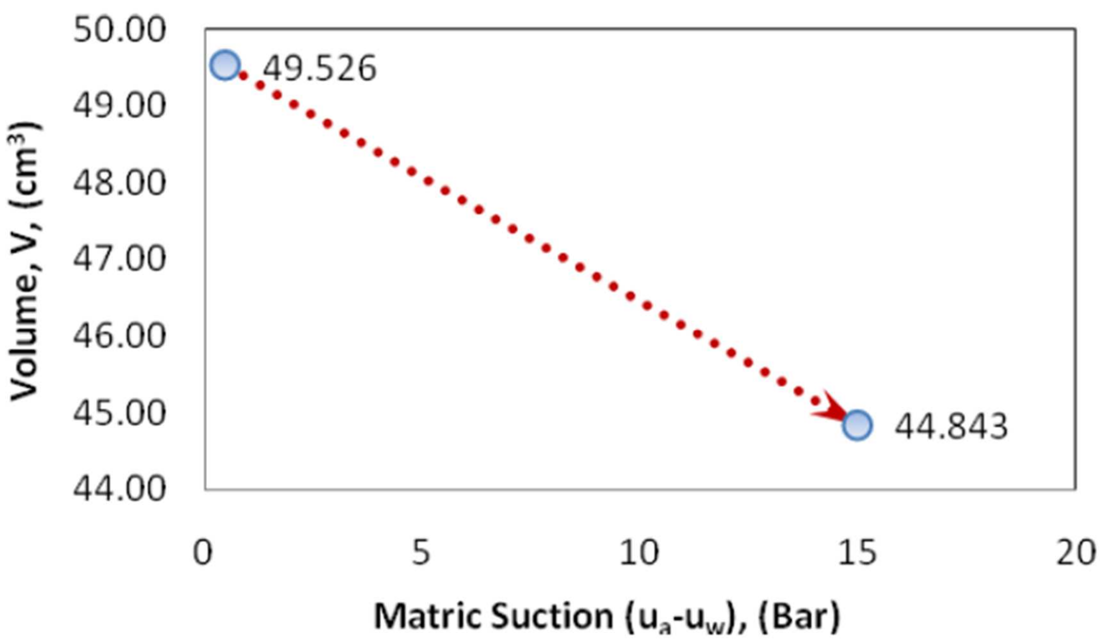

Figure 3.12. Volumetric Change by Changing Suction, after Sahin (2011)

The process of incorporating more data points and determine an incremental suction compression index is implemented as follows:

The first step is to determine the statistical model that best fits the suction-volumetric strain relationship. The test results show that the suction-volumetric strain relationship resembles an S-shaped curve which is very similar to a well-known class of statistical functions called Cumulative Distribution Functions (CDF). The appropriateness of this class of functions in modeling the suction-volumetric strain relationship will be assessed using the Kolmogorov-Smirnov (K-S) goodness of fit test. In this case, the hypotheses being tested are:

$Null\ Hypothesis\ (H_0){:}\ The\ relationship\ can\ be\ described\ using\ a\ CDF$

against

$Alternative\ Hypothesis\ (H_1){:}\ The\ relationship\ cannot\ be\ described\ using\ a\ CDF$

If the P-value resulting from the test exceeds the significance level (α) of 0.05, then the test is considered "statistically insignificant". In other words, the null hypothesis (H_0) cannot be rejected and the suction-volumetric strain relationship can be successfully described by the CDF. In this case (after test assessment proven appropriateness) the suction-volumetric strain relationship can be modeled in the form expressed by Equation 3.3:

$$V^* = F(S) \tag{3.3}$$

Where,

V^* (normalized volumetric strain) = $V/max(V)$

V = Volumetric strain

$max(V)$ = Maximum observed volumetric strain value

S = Soil suction

$F(s)$ = CDF of soil suction at any specific suction value *s*

Because the CDF values range from 0 to 1, the volumetric strain has to be normalized at the very first step. This is to match the same range of the CDF. The normalization effect is reversed back in the last step.

Since suction compression index is the change in volume related to a change in suction, a general form for an initial suction compression index can be derived from Equation 3.3

by taking the derivative of both sides of the equation with respect to soil suction. The resulting form for initial suction compression index is as per Equation 3.4.

$$\gamma_{hi}(s) = \frac{\partial V^*}{\partial S} = f(s) \tag{3.4}$$

Where,

$\gamma_{hi}(s)$ = The value of γ_{hi} at soil suction s

$f(s)$ = The Probability Density Function (PDF) of soil suction at suction s

Subsequently, the value of γ_{hi} can be determined by estimating the density function f in Equation (3.4) using standard density estimation techniques such as the Kernel Density Estimator (KDE). Although there are other standard density estimation techniques such as Splines, KDE is found to be mathematically simpler, widely used, and provides the same efficiency.

The estimation process is implemented using R statistical package. Although there are other powerful statistical packages like SAS, R is a free open-source licensed software that makes it more convenient for others to use if decided to follow this newly proposed incremental γ_h procedure. It is also powerful and compatible with virtually all platforms (Microsoft Windows, Linux, etc…).

The estimated initial suction compression index $\hat{\gamma}_{hi}(s)$ can then be written in the following form expressed by Equation 3.5.

$$\hat{\gamma}_{hi}(s) = \hat{f}(s) \tag{3.5}$$

Finally, in order to arrive at a true suction compression index estimate, $\hat{\gamma}_h(s)$, back-transformation process has to occur. This is done by denormalizing the estimated initial suction compression index $\hat{\gamma}_{hi}(s)$ as per Equation 3.6.

$$\hat{\gamma}_h(s) = \hat{\gamma}_{hi}(s) \times max(V) \tag{3.6}$$

Equation 3.6 can now be utilized for determining suction compression index (γ_h) at each soil suction value. Incremental γ_h has now been successfully determined. This adds flexibility for heave predictions of a particular soil layer in a given soil profile, since only suction compression indices that are relevant to a known suction range in a particular layer can be considered.

Moreover, an average for all γ_h values corresponding to all suction values can be taken for a particular drying test. It can be determined as per Equation 3.7. The averaged γ_h is a single representative index for this particular soil specimen. The averaged γ_h is also a single index that is representative to the entire non-linear suction-volumetric strain relationship, not just a linear portion of it, nor an idealized linear relationship. The R code used in the described statistical modeling for determination of incremental γ_h is available in Appendix A.

$$\hat{\gamma}_{h,avg} = \frac{1}{n}\sum_{i=1}^{n} \hat{\gamma}_h(s_i) \tag{3.7}$$

Where,

i = Number of suction values.

Similarly, in case if more than one test is being performed for a particular site, an average is taken to arrive at a representative index to this particular site.

CHAPTER IV

Results and Discussion

Results of the conducted laboratory tests for specimen Kirkland 1A1 is shown below descriptive analyses and overall discussion. However, the results for the rest of the tested specimens can be found in Appendix B.

4.1 Kirkland Site

4.1.1 Specimen Kirkland 1A1

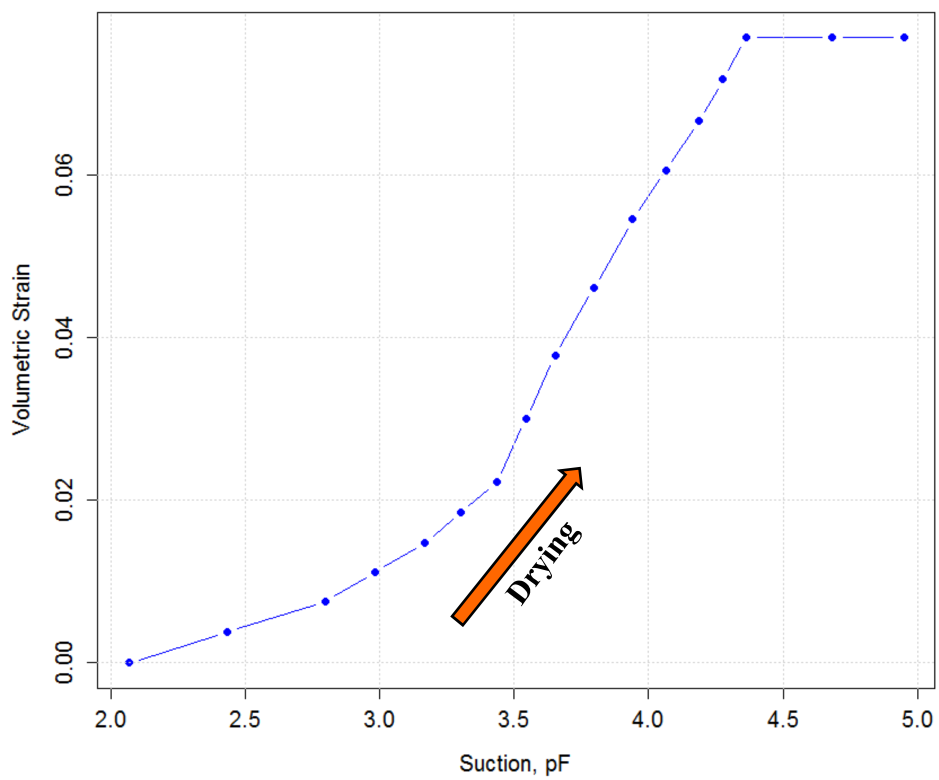

Figure 4.1. Kirkland 1A1: Suction Vs Volumetric Strain – Drying Test

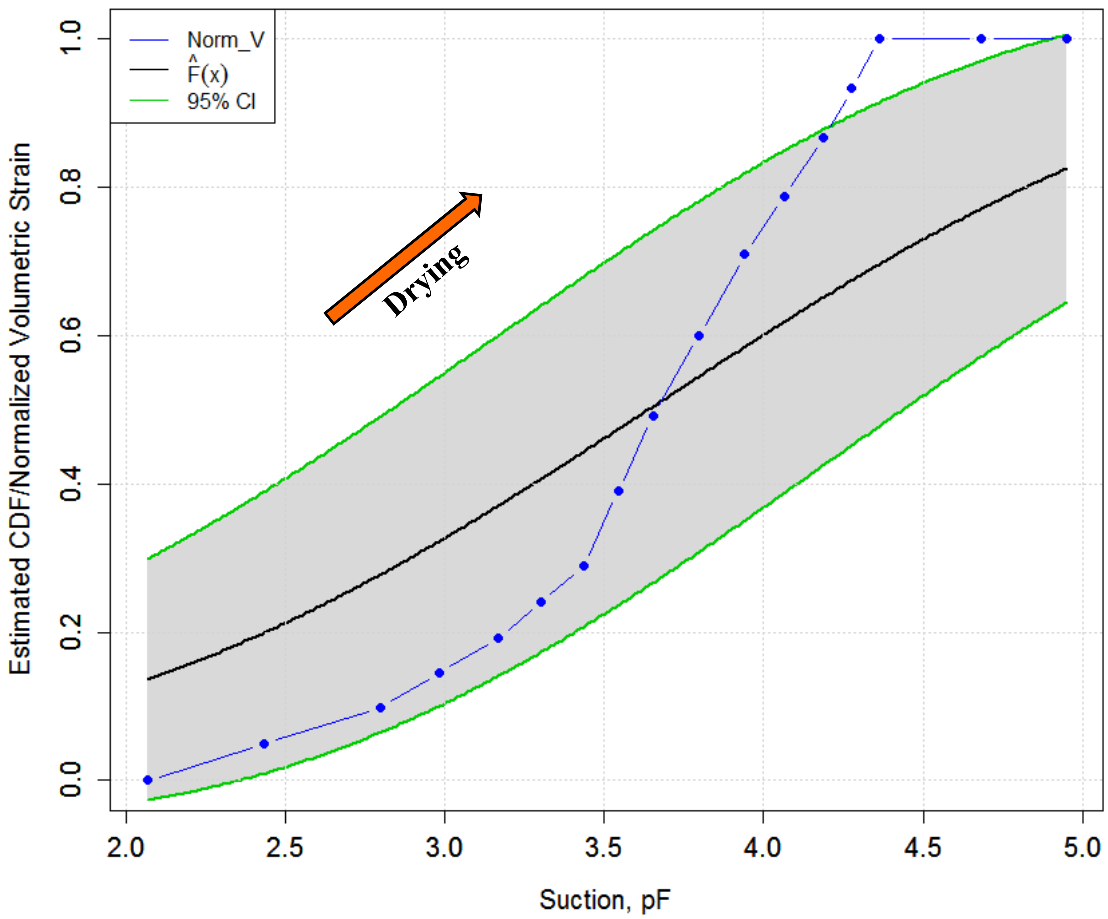

Figure 4.2. Kirkland 1A1: Kernel Estimate for CDF with 95% Confidence Interval – Drying Test

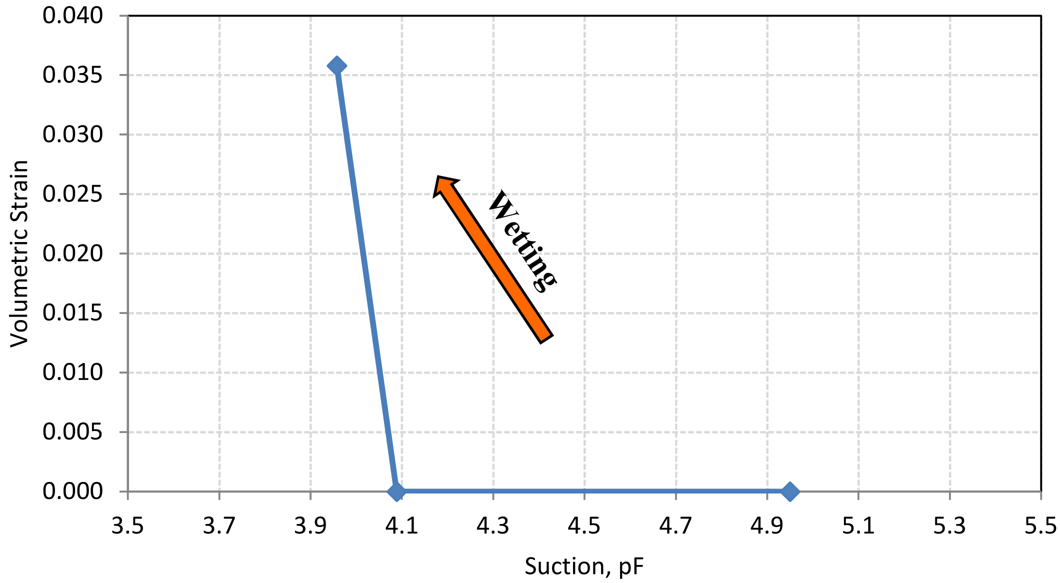

Figure 4.3. Kirkland 1A1: Suction Vs Volumetric Strain – Wetting Test

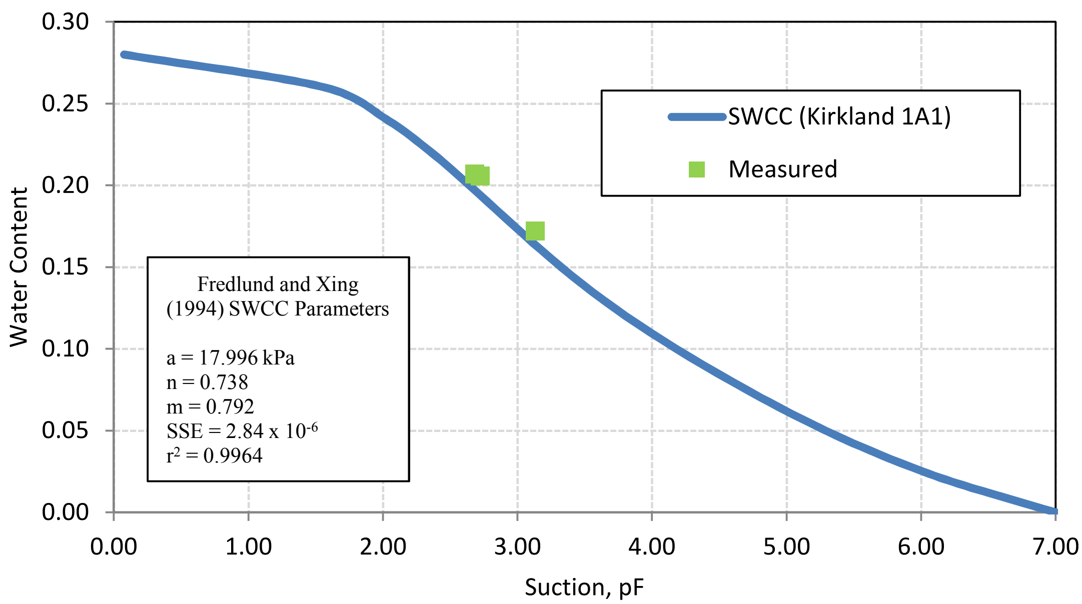

Figure 4.4. Kirkland 1A1 – Soil-Water Characteristic Curve

Table 4.1. Curve Fitting Parameters for Kirkland 1A1 SWCC

Parameters/Error/R^2	**Value**
a	17.99596744 (kPa)
n	0.738411676
m	0.792408544
Optimized/Minimized SSE	0.00000284
R-squared	0.9964

4.2 Goodness of Fit Results from Kolmogorov-Smirnov (K-S) Test

Table 4.2. P-Values for Superposed Curves

Site	**Specimen**	**P-Value**
Kirkland	1A1	0.22160
	2C3	0.14870
	3B2	0.07168
Port	2A2	0.40430
	4A2	0.14600
	6A2	0.08222
Osage	1B2	0.13310
	2C1	0.11220
	3A1	0.29380

4.3 Suction Compression Index Values

4.3.1 Suction Compression Index Values as per the New Method

Table 4.3. Averaged Drying Suction Compression Indices

Site	Specimen	Averaged γ_h Values for Each Test	Averaged γ_h Values for Each Site
Kirkland	1A1	0.030519	0.023821
	2C3	0.011679	
	3B2	0.029266	
Port	2A2	0.032853	0.031551
	4A2	0.032000	
	6A2	0.029799	
Osage	1B2	0.075587	0.068962
	2C1	0.070741	
	3A1	0.060559	

Table 4.4. Averaged Wetting Suction Compression Indices

Site	Specimen	Averaged γ_h Values for Each Test	Averaged γ_h Values for Each Site
Kirkland	1A1	0.272147	0.038036
	2C3	0.041724	
	3B2	0.034347	
Port	2A2	0.018492	0.035646
	4A2	0.040669	
	6A2	0.047777	
Osage	1B2	0.179375	0.257739
	2C1	0.134664	
	3A1	0.459178	

4.3.2 Suction Compression Index Values as per Covar and Lytton 2001 Equations

Table 4.5. Drying Suction Compression Indices (based on Covar and Lytton 2001 Equation)

Site	Specimen	γ_h Values for Each Test	Averaged γ_h Values for Each Site
Kirkland	1A1	0.000897	0.000586
	2C3	0.000353	
	3B2	0.000507	
Port	2A2	0.000276	0.000847
	4A2	0.001357	
	6A2	0.000908	
Osage	1B2	0.001136	0.001749
	2C1	0.001853	
	3A1	0.002257	

Table 4.6. Wetting Suction Compression Indices (based on Covar and Lytton 2001 Equation)

Site	Specimen	γ_h Values for Each Test	Averaged γ_h Values for Each Site
Kirkland	1A1	0.000898	0.000596
	2C3	0.000383	
	3B2	0.000507	
Port	2A2	0.000276	0.000848
	4A2	0.001358	
	6A2	0.000909	
Osage	1B2	0.001137	0.001752
	2C1	0.001856	
	3A1	0.002262	

Table 4.7. Averaged of Drying and Wetting Suction Compression Indices (based on Covar and Lytton 2001 Equation)

Site	Specimen	Averaged γ_h Values for Each Test	Averaged γ_h Values for Each Site
Kirkland	1A1	0.000897	0.000591
	2C3	0.000368	
	3B2	0.000507	
Port	2A2	0.000276	0.000847
	4A2	0.001357	
	6A2	0.000909	
Osage	1B2	0.001136	0.001750
	2C1	0.001855	
	3A1	0.002260	

4.3.3 Suction Compression Index Values as per Covar and Lytton 2001 Contour Charts

Table 4.8. Suction Compression Indices (based on Covar and Lytton 2001 Contour Charts)

Site	Specimen	Averaged γ_h Values for Each Test	Averaged γ_h Values for Each Site
Kirkland	1A1	0.028235	0.025229
	2C3	0.022222	
	3B2	0.028235	
Port	2A2	0.038060	0.041061
	4A2	0.044444	
	6A2	0.040678	
Osage	1B2	0.046512	0.050791
	2C1	0.050562	
	3A1	0.055300	

4.3.4 Suction Compression Index Values as per McKeen's Classification Charts

Table 4.9. Suction Compression Indices (based on Classification Charts from McKeen 1981; McKeen and Hamberg 1981)

Site	Specimen	γ_h Values for Each Test	Averaged γ_h Values for Each Site
Kirkland	1A1	0.045176	0.041414
	2C3	0.033889	
	3B2	0.045176	
Port	2A2	0.042985	0.044594
	4A2	0.047407	
	6A2	0.043390	
Osage	1B2	0.059535	0.061449
	2C1	0.064719	
	3A1	0.060092	

4.3.5 Suction Compression Index Values as per McKeen's Assumed Final Suction Approach

Table 4.10. Suction Compression Indices (based on Final Suction Assumption from McKeen 1992; McKeen and Hamberg 1981)

Site	Specimen	Averaged γ_h Values for Each Test	Averaged γ_h Values for Each Site
Kirkland	1A1	0.022383	0.019319
	2C3	0.014675	
	3B2	0.020898	
Port	2A2	0.017058	0.018974
	4A2	0.015603	
	6A2	0.024262	
Osage	1B2	0.110486	0.086589
	2C1	0.099648	
	3A1	0.049632	

4.4 Discussions

In this study, an accurate, relatively fast, economical, and practical testing method has been proposed to determine suction compression index for expansive soils. This testing method has been explained in detail in Chapter 3. The newly proposed testing method as well as four other techniques in the literature have been applied to soil specimens from three different sites in the state of Oklahoma (Kirkland, Port, and Osage sites). The test results from the new method are reported in section 4.1 through section 4.3, as well as Appendix B. Results from the four other techniques are reported in section 4.3.

Soil suction and volumetric strain are the two elements for determining suction compression index. The suction and volumetric strain for each tested specimen has been measured according to the new testing protocol. Plotting these measures gives insight of the suction-volumetric strain relationship. The resulting plots are displayed in Figures 4.1, B1, B5, B9, B13, B17, B21, B25, and B29. These figures clearly show that the suction-volumetric strain relationship resembles an S-shaped curve.

The fact that the suction-volumetric strain relationship is proven to be nonlinear suggests the necessity of proposing a new approach other than the traditional linear approaches present in the literature. As explained in Chapter 3, a new nonlinear approach presents an incremental suction compression index (γ_h). The incremental γ_h allows for taking advantage of all suction values on the entire nonlinear suction-volumetric strain relationship. Traditionally, γ_h is determined by taking the slope of a linear portion of the relationship. This neglects some of the suction values that lie outside the considered linear portion of the relationship. These neglected suction values are statistically considered important information. Hence, not all information is taken into consideration when γ_h is determined traditionally.

Generally, S-shaped relationships can be fitted by a well-known class of statistical functions called Cumulative Distribution Functions (CDF). Therefore, the incremental γ_h can be estimated by estimating the CDF at every suction value. The appropriateness of using these estimates to describe the suction-volumetric strain relationship is evaluated using the Kolmogorov-Smirnov (K-S) goodness of fit test. Furthermore, 95% confidence intervals of the superposed curves are also used to assess the appropriateness of the CDF estimates.

The results of the K-S tests (P-values) for all tested specimens are tabulated in Table 4.2. It is important to mention that – by default – exact P-values are not obtainable from the K-S test in case of ties (i.e. identical volumetric strain values). For the tested specimens, ties occur when few suction values are recorded above the first value corresponding to final volume. Such values are referred to as tail values. Consequently, tail values are not considered when calculating the P-values from the K-S test. Table 4.2 clearly shows all calculated P-values exceed the 5% significance level which implies that all tests are statistically insignificant and, thus, it is appropriate to use the CDF to model the suction-volumetric strain relationship. The table also shows that the P-values vary from one test to another. This variation is a result of two main factors, namely, number of observations used in each test and proximity of the estimated volumetric strain values to the observed ones. The number of observations in each test depends on occurrence of substantial volume change corresponding to suction change. While the effect of the second factor can be seen from Figures B10, B14, and B30. These figures give additional evidence that the CDF confidently models the suction-volumetric strain relationship for all tests. This is because all of the observed suction-volumetric strain relationships lie inside the 95% confidence intervals. Despite that the suction-volumetric strain relationships of specimens P6A2 and O2C1 do not substantially deviate from a linear form as shown in Figures B17 and B25, respectively, the relationships successfully lie inside the 95% confidence interval as shown in Figures B18 and B26, respectively. P-values of their K-S tests still exceed 5%. This confirms the appropriateness of the CDF in modeling the suction-volumetric strain relationships for these two cases.

From the literature, there are different means for the determination of γ_h. There are also other means for the determination of other indices similar to it in nature to γ_h but not exactly the same. Among the techniques that determine γ_h, there are the following seven techniques:

1. Contour charts which are strictly dependent on soil index properties (Covar and Lytton 2001; Lytton et al. 2005). An example of the contour charts can be seen from Figure 2.9.
2. Classification chart which is dependent on soil index properties (McKeen 1981; McKeen and Hamberg 1981). The chart can be seen from Figure 2.7.
3. Assumption of a final suction value of 5.5 pF and measuring an initial suction value (McKeen and Hamberg 1981; McKeen 1992). This is referred to in Section 1.3.
4. Regression equations which are strictly dependent on a COLE value (Covar and Lytton 2001). These are Equations 2.4, 2.5, and 2.6.

5. A regression equation which is strictly dependent on the slope of the soil-water characteristic curve (SWCC) (McKeen 1992). This is Equation 2.41.
6. Another regression equation which is also strictly dependent on the slope of the SWCC (Perko et al. 2000). This is Equation 2.42.
7. An equation which is dependent on a COLE value as well as two suction values (McKeen and Nielsen 1978). This equation (Equation 2.7) is suggested by McKeen and Nielsen (1978) to be reduced to be strictly dependent on a COLE value (Equation 2.8).

The COLE value is easily determined from the proposed testing method. This necessitates that the dry length of the tested soil specimen is measured when its shrinking volume seizes. Suction at this state is equivalent to an air-dry (shrinkage limit) suction of about 5.0~6.0 pF. Since soil essentially cannot shrink anymore, the air-dry length is considered instead of the length corresponding to oven-dry suction (≈ 7.0 pF).

According to the definition of γ_h explained in Chapter 1, none of the above techniques better serve γ_h determination as the newly proposed testing method in this study does. First of all, by its definition, the COLE test essentially describes the linear strain in the soil rather than volumetric strain. Therefore, techniques No. 4 and No. 7 are not the best for γ_h determination. Other techniques (No. 5 and No. 6) merely depend on the slope of SWCC. They are also not the best for γ_h determination as they lack the volumetric strain measurements. Even though soil index properties are useful in describing different characteristics of the soil, however, techniques No. 1 and No. 2 which are solely dependent on index properties do not comprise volumetric strain measurements nor suction measurements. Finally, technique No. 3 involves only two suction values, one of which is an assumed value as explained in Chapter 1. None of these techniques comprises determination of several suction values at different corresponding volumes for a given specimen, however, the newly proposed testing method in this study does. It can be concluded that the new testing method is the only 'direct' suction compression index determination method. While all others can be described as 'indirect' methods.

Suction compression index is determined for the first four techniques as well as the proposed testing method. Table 4.11 consolidates γ_h values for all the five techniques. 'M' stands for the method type. 'A' stands for the drying incremental γ_h testing method. 'B' stands for the contour charts technique. 'C' stands for Covar and Lytton (2001) drying equation. 'D' stands for the classification chart technique. 'E' stands for the final suction assumption technique. The averaged values of this table are tabulated in Table 4.12.

Table 4.11. Suction Compression Indices According to Five Techniques

M	K 1A1	K 2C3	K 3B2	P 2A2	P 4A2	P 6A2	O 1B2	O 2C1	O 3A1
A	0.030519	0.011679	0.029266	0.032853	0.032000	0.029799	0.075587	0.070741	0.060559
B	0.028235	0.022222	0.028235	0.038060	0.044444	0.040678	0.046512	0.050562	0.055300
C	0.000897	0.000353	0.000507	0.000276	0.001357	0.000908	0.001136	0.001853	0.002257
D	0.045176	0.033889	0.045176	0.042985	0.047407	0.043390	0.059535	0.064719	0.060092
E	0.022383	0.014675	0.020898	0.017058	0.015603	0.024262	0.110486	0.099648	0.049632

Table 4.12. Averaged Suction Compression Indices Per Each Site

M	Kirkland	Port	Osage
A	0.023821	0.031551	0.068962
B	0.026231	0.041061	0.050791
C	0.000586	0.000847	0.001749
D	0.041414	0.044594	0.061449
E	0.019319	0.018974	0.086589

Table 4.13. Averaged Plasticity Index (PI) Values Per Each Site

	Kirkland	**Port**	**Osage**
PI (%)	17.7	24.1	29.6

From Table 4.12, it can be clearly noticed that technique ‘C’ values are considerably lower than all other methods which makes its accuracy questionable. Moreover, it can be noticed that technique ‘E’ yields inconsistent γ_h values. Technique ‘E’ values are almost insensitive for sites Kirkland and Port; however, exceptionally high γ_h value for Osage site. This can be better pictured in Figure 4.5; where Table 4.12 values are plotted versus Table 4.13 values. Consequently, techniques ‘C’ and ‘E’ cannot be considered reliable γ_h determination techniques.

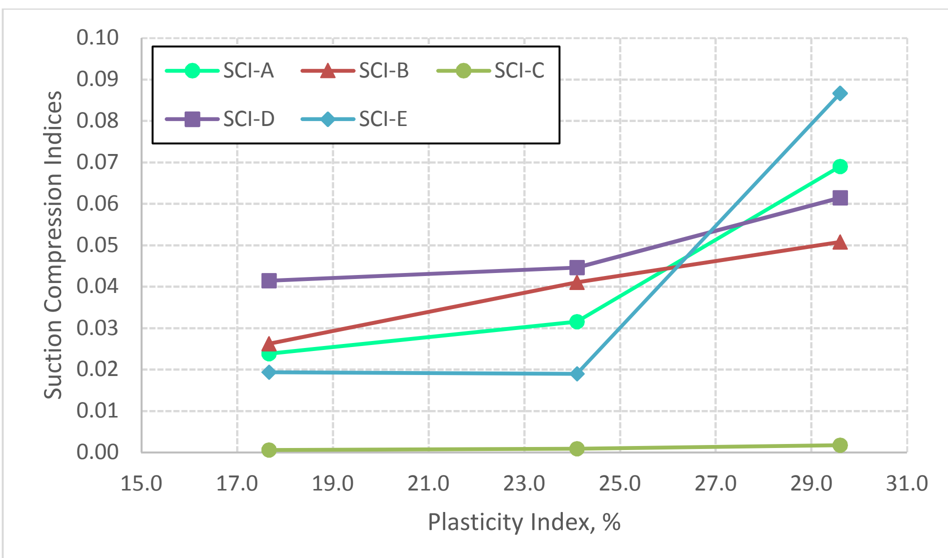

Figure 4.5. Average γ_h Values Vs Average PI Values Per Each Site

Heave prediction calculations have been determined according to Table 4.12. A soil profile is assumed to be of three layers. Each layer is 25 cm thick. Initial suction values for each layer from top to bottom are assumed to be equal to 2.0 pF, 2.5 pF, 3.0 pF, respectively. Final suction values are assumed to be equal to 5.5 pF, 5.0 pF, 4.5 pF, respectively. The used crack fabric factor value is 0.5. The resulting predictions are summarized in Table 4.14 in centimeters of shrinkage.

Table 4.14. Shrinkage Prediction in cm

M	**Kirkland**	**Port**	**Osage**
A	2.2	3.0	6.5
B	2.5	3.8	4.8
C	0.1	0.1	0.2
D	3.9	4.2	5.8
E	1.8	1.8	8.1

In order to put Table 4.14 into perspective, all the indirect methods are referenced to method ‘A’ (the direct method). Table 4.15 shows the shrinkage change in percentage with respect to method ‘A’ and plotted in Figure 4.6 against Table 4.13 values (PI). Techniques ‘C’ and ‘E’ have been excluded from Figure 4.6. It can be clearly noticed that technique ‘D’ over estimates the shrinkage for Kirkland site with an amount of 73.9% of that of technique ‘A’. It can also be noticed that there is an unjustified change in the behavior of technique ‘B’.

Table 4.15. Δ Shrinkage Prediction in %

M	Kirkland	Port	Osage
A	0.0	0.0	0.0
B	10.1	30.1	-26.3
C	-97.5	-97.3	-97.5
D	73.9	41.3	-10.9
E	-18.9	-39.9	25.6

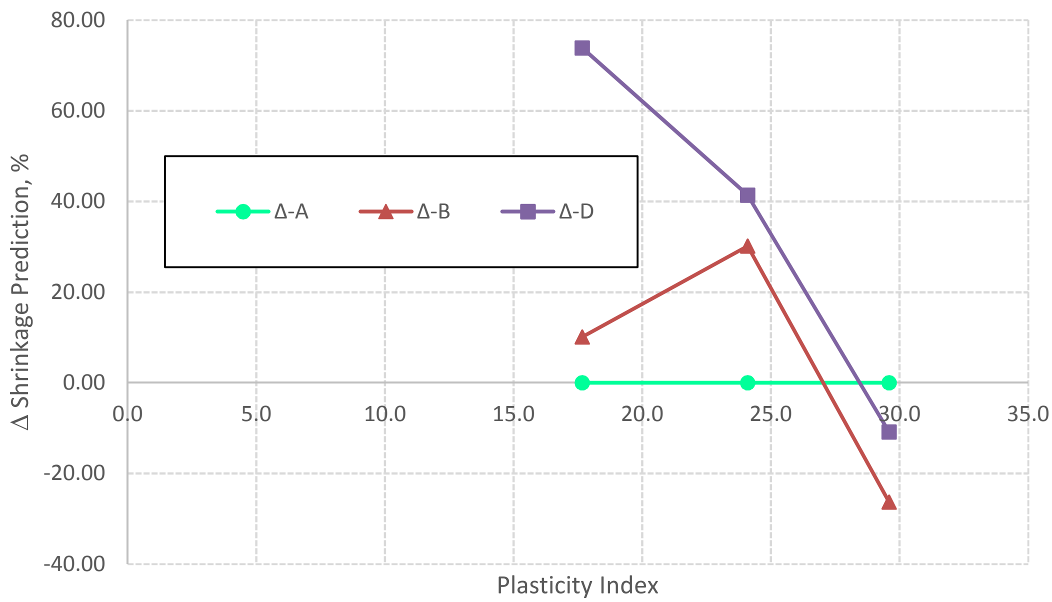

Figure 4.6. Shrinkage Change Values Vs Average PI Values Per Each Site

For the three tested site soils, the degree of volume change has been determined based on plasticity index according to Holtz and Gibbs (1956) as shown in Table 4.16. Table 4.17 shows the degree of volume change determined for techniques 'A', 'B', and 'D' based on γ_h guidelines available from USDA (1972) and McKeen and Nielsen (1978).

For technique ‘A’, it can be observed that the degree of volume change based on based on γ_h guidelines (Table 4.17) is generally overestimated by one class than the one based on plasticity index (Table 4.16). Despite of this observation, it can be seen from Table 4.17 that technique ‘A’ still yields the most consistent classification, while technique ‘D’ shows insensitive results. It can be finally concluded that among all techniques, technique ‘A’ is generally the closest to technique ‘B’ which is adopted by the Post-Tensioning Institute (PTI).

Table 4.16. Degree of Volume Change based on PI (Holtz and Gibbs 1956)

	Kirkland	**Port**	**Osage**
Degree of Volume Change	Low	Moderate	High

Table 4.17. Degree of Volume Change based on γ_h (USDA 1972; McKeen and Nielsen 1978)

	Kirkland	**Port**	**Osage**
Technique ‘A’	Moderate	High	Very High
Technique ‘B’	High	Very High	Very High
Technique ‘D’	Very High	Very High	Very High

4.5 The Wetting Component

In the state of Oklahoma, the drying shrinkage of high-plastic soils is a major problem. As a result, for Oklahoma soils, the research is focused more on the drying approaches than the wetting ones due to the much more importance of drying approaches. Despite of this fact, this study did include research on wetting approach for determination of suction compression index in expansive soils.

The wetting component of the γ_h determination testing method proposed in this study (i.e. the wetting technique) as explained in Chapter 3 is referred to in Table 4.18 as 'A*'. While the wetting γ_h values based on Covar and Lytton (2001) swelling equation is referred to as C*. From Table 4.18, according to technique C*, it has again shown very low and insensitive wetting γ_h values same behavior as in technique 'C'. As for technique A*, it can be noticed that wetting γ_h values are generally acceptable. It can also be noticed that there are few inconsistencies. For specimens Kirkland 1A1 and Osage 3A1 are relatively larger than other specimens tested for the same sites. Additionally, specimen Port 2A2 has shown relatively lower wetting γ_h value than those tested for the same site. The reason behind the discrepancies may be related to the number of volume change data points (observations) which were allowed to be taken. It is understood that the more data points to be secured, the more accurate the result can be obtained. The pointed-out specimens share the same number of allowed volume change observations of two (Figures 4.3, Figure B31, and B11, respectively). Unlike – for example – specimen Port 4A2 which allowed for five volume change observations (Figure 4.11). The number of wetting observations which technique C* allows for soil specimens is found to be critical to some extent. It is also dependent on the type of tested soil.

Table 4.18. Wetting γ_h Values

M	**K 1A1**	**K 2C3**	**K 3B2**	**P 2A2**	**P 4A2**	**P 6A2**	**O 1B2**	**O 2C1**	**O 3A1**
A*	0.272147	0.041724	0.034347	0.018492	0.040669	0.047777	0.179375	0.134664	0.459178
C*	0.000898	0.00383	0.000507	0.000276	0.001358	0.000909	0.001137	0.001856	0.002262

CHAPTER V

CONCLUSIONS

5.1 Conclusions

Suction compression index (γ_h, SCI, C_h) has been determined in the literature based on the slope of an idealized linear suction-volumetric strain relationship. This relationship is essentially nonlinear. Many of the determination techniques are indirect methods. This study proposes a unique testing method for γ_h determination. The new testing method is designed to be more representative for the suction-volumetric strain relationship than the available ones in the literature, relatively fast, economical, and easy. The outcome is a robust method. This makes it practical to be adopted by geotechnical engineering practitioners and laboratories. It mainly requires a high-resolution DSLR (Digital Single-Lens Reflex) camera, a digital image measurement software, a free open-source licensed statistical software, a balance, and an oven. The new testing method has significant advantages over the existing methods in the literature since (1) it yields incremental γ_h representative to the entire nonlinearity of the suction-volumetric strain relationship and (2) it is a direct method.

Unlike many other methods, it directly relies on volumetric strain measurements rather than linear strain measurements or no strain measurements at all. Additionally, it directly relies on suction measurements. Parameters necessary for determination of other indices such as: COLE, CLOD index, shrink-swell modulus, soil-water characteristic curve (SWCC) slope, and crack fabric factor (lateral restraint factor) can also be easily determined from the same testing method. Moreover, if the specific gravity is assumed, soil physical properties such as void ratio, degree of saturation, and porosity can be determined, as well as, unit weight and water content. Furthermore, the testing method provides a wetting γ_h value if enough volumetric strain observations are secured.

The testing method utilizes a high-resolution DSLR camera and a digital image measurement software for taking volume measurements for soil specimens. Accordingly, volumetric strain values are determined. The testing method also comprises water content measurements corresponding to each taken image. Subsequently, suction is determined after running filter paper tests and using soil-water characteristic curve (SWCC) equation proposed by Fredlund and Xing (1994) (Equation 3.1). A unique statistical modeling approach is proposed within the testing method that has never been adopted in other γ_h determination methods available in the literature. The proposed statistical modeling allows for determination of γ_h values corresponding to each suction value on the entire nonlinear suction-volumetric strain relationship. For the first time, this eliminates the need to idealize an essentially nonlinear relationship to be a linear one. Hence, more representative results from an integrated testing method.

Soil specimens from three different sites in the state of Oklahoma have been tested. The suction compression indices determined from the new testing method have been compared against results of other techniques from the literature. The other techniques are as follows:

1. Contour charts which are strictly dependent on soil index properties (Covar and Lytton 2001; Lytton et al. 2005). An example of the contour charts can be seen from Figure 2.9. The contour charts technique is adopted by the Post-Tensioning Institute (PTI).
2. Classification chart which is dependent on soil index properties (McKeen 1981; McKeen and Hamberg 1981). The chart can be seen from Figure 2.7.
3. Assumption of a final suction value of 5.5 pF and measuring an initial suction value (McKeen and Hamberg 1981; McKeen 1992). This is referred to in Section 1.3.
4. Regression equations which are strictly dependent on a COLE value (Covar and Lytton 2001). These are Equations 2.4, 2.5, and 2.6.

The γ_h values resulted from the proposed testing method was generally the closest to the one adopted by the PTI. Some results from other techniques showed inconsistency while others showed that they are insensitive to the tested type of soil.

5.2 Recommendations for Future Research

Suction compression index determination techniques based on wetting approach are very limited in the literature. Some of which are based on a single COLE value while others are based on a small soil specimen of the size of a consolidometer ring. Both approaches are based on linear/vertical strain measurements. The proposed testing method considered enhancing these approaches by (1) testing specimens much larger in size and (2) using volumetric strain measurements rather than linear strain ones. However, it might be critical to run. The resulting γ_h values are generally reasonable except for few tests. It was noticed that in those tests the number of volumetric strain measurements was limited. In those tests, the testing condition did not allow except for two measurements which affected the accuracy. Two γ_h values were overestimated while the third one was underestimated.

Despite of the fact that research on high-plastic soils in the state of Oklahoma utilizing drying approaches is much more important than that based on wetting approaches due to the major drying shrinkage problems, it might still be of a benefit to build on the wetting component of the proposed testing method presented in this study. If the wetting component is modified in such a way that the number of wetting observations is increased, this will allow for arriving at more reliable wetting γ_h technique with more consistency.

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APPENDICES

Appendix A – The R Code

The R code used in the described statistical modeling as referred to in Chapter 3, section 3.4, for determination of incremental γ_h, is as follows:

```
#Statistical Analysis#

#Set the working directory - it should be the place where your data is stored#
#Data can be stored in form of a Microsoft Excel spreadsheet#
#The spreadsheet has 2 columns, one for volumetric strain and another for suction#
#Set your directory here in between quotes of the first line of the code, substitute each backslash mark (\) with double backslash marks (\\)#
setwd("")

#Recall required packages#
library(xlsx)
library(sROC)

#Read data from a Microsoft Excel spreadsheet#
dat=read.xlsx("Dry_STAT.xlsx", sheetName="Sheet1", header=TRUE)
dat


#Normalized volume is identified as "v.norm"#
v.norm=dat$v/max(dat$v)
```

```
#Plot v.norm against suction "s" to check the S-curve relationship#
plot(dat$s, v.norm, type="p", col="blue")

#Find the estimated CDF of suction#
s.CDF = kCDF(dat$s,from=min(dat$s), to=max(dat$s), cut=3, na.rm = FALSE)

#Plot the estimated CDF of suction#
plot(s.CDF, alpha=0.05, CI=TRUE, main="Kernel estimate of distribution function")

#Plot v.norm in the same graph with estimated CDF of suction#
par(new=TRUE)
plot(dat$s,v.norm, type="l", col="blue")

#Apply KS test for adequacy of relationship#
ks.test(v.norm, s.CDF$Fhat)

#Perform SCI "index" calculation by calculating the KDE for S#
f.hat=density(dat$s,adjust=1, kernel=c("epanechnikov"))

index.raw=approx(f.hat$x, f.hat$y, dat$s)

index.raw.1=c(unlist(index.raw$y))

#Denormalize index#
index=index.raw.1*max(dat$v)

index

#Calculate a single averaged index
av.index=mean(index)

av.index
```

Appendix B – Test Results

Results of the conducted laboratory tests for the three sites (Kirkland, Port and Osage) are shown below.

6.1 Kirkland Site

6.1.1 Specimen Kirkland 1A1

Results of the very first specimen, Kirkland 1A1, are excluded from Appendix B since they are already included in Chapter 4.

6.1.2 Specimen Kirkland 2C3

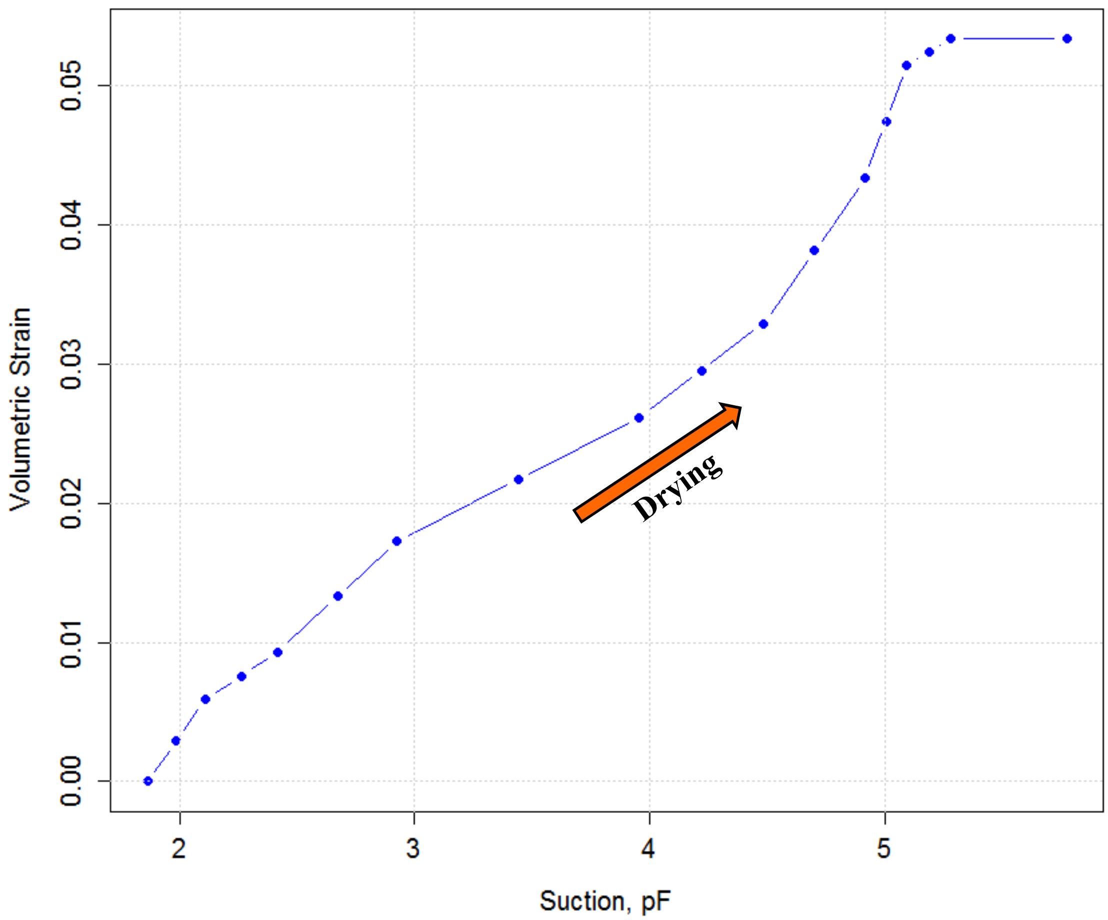

Figure B1. Kirkland 2C3: Suction Vs Volumetric Strain – Drying Test

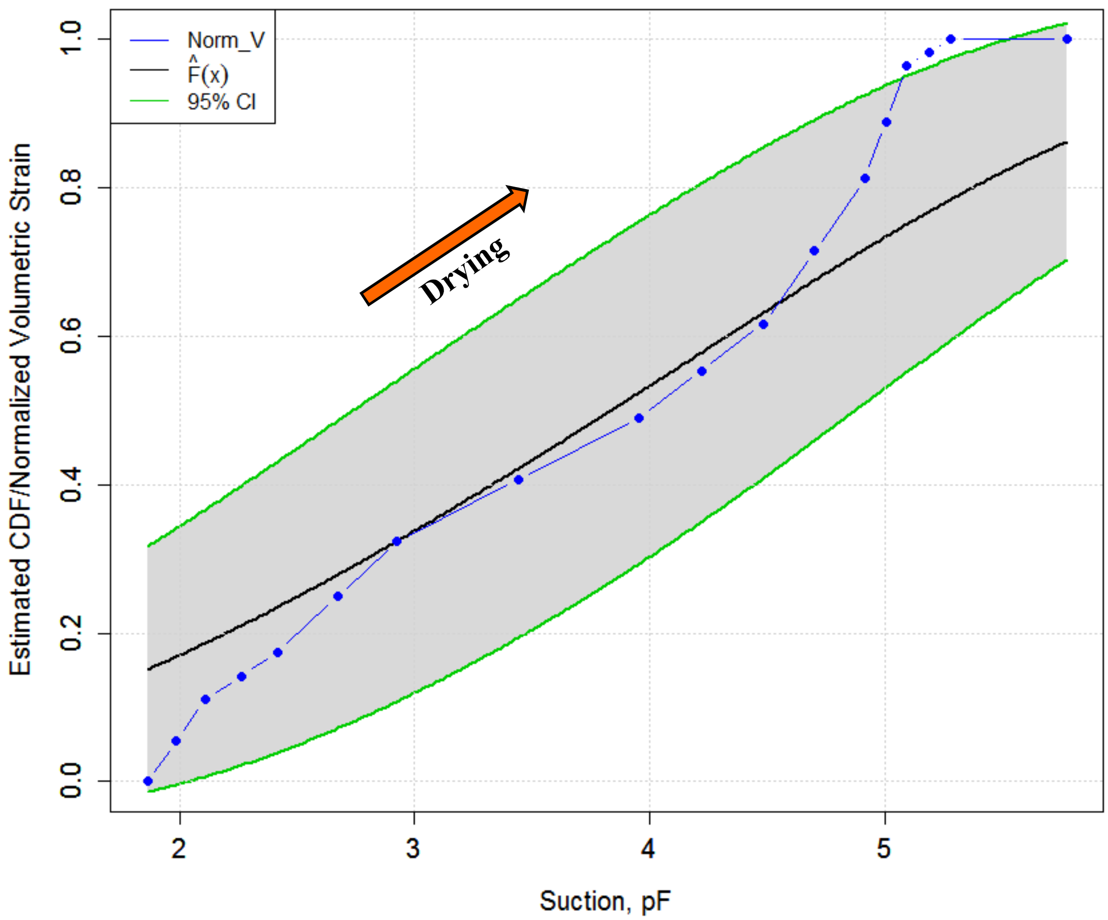

Figure B2. Kirkland 2C3: Kernel Estimate for CDF with 95% Confidence Interval – Drying Test

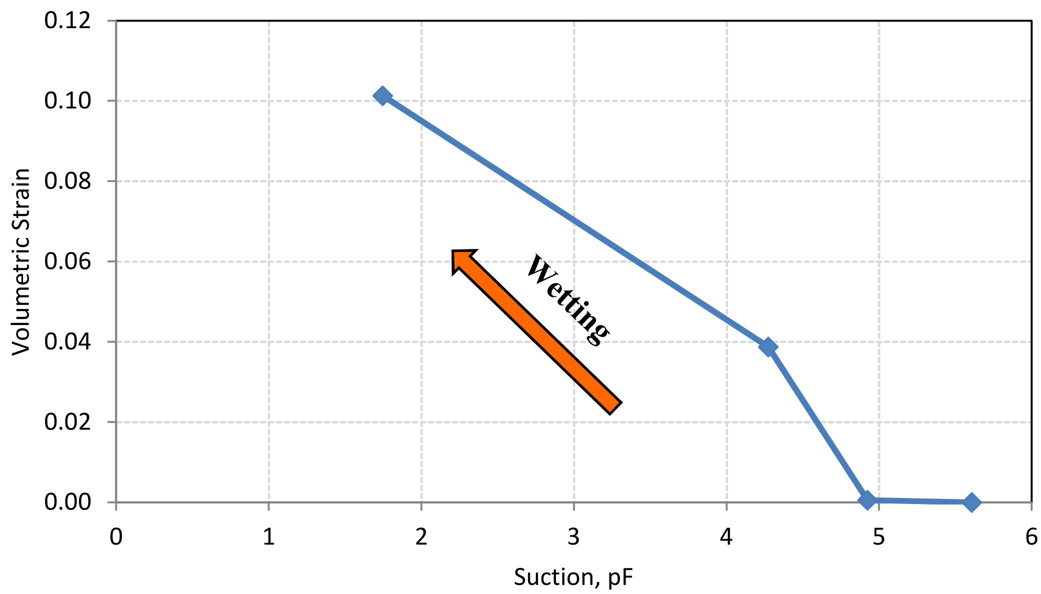

Figure B3. Kirkland 2C3: Suction Vs Volumetric Strain – Wetting Test

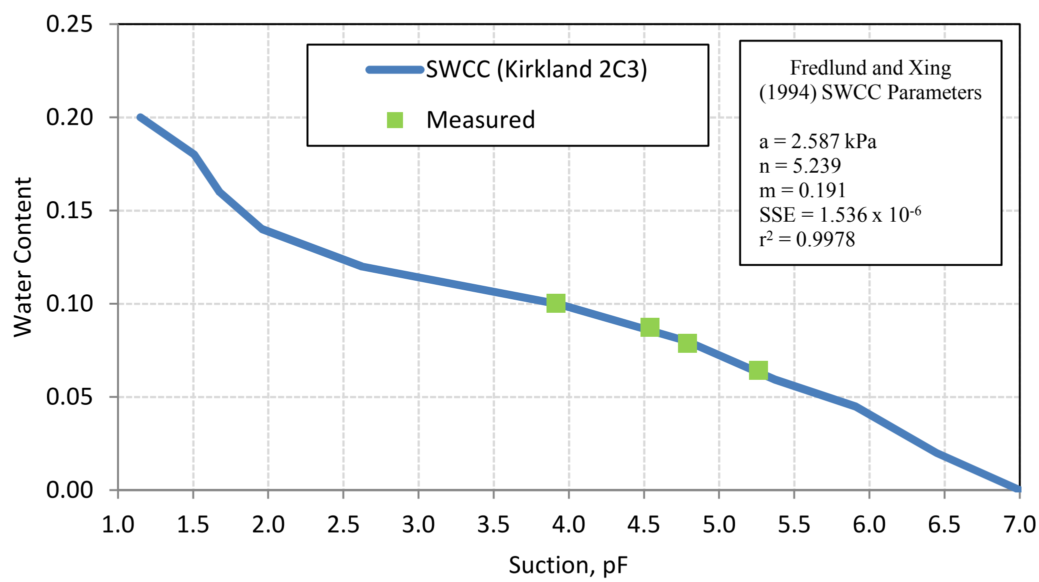

Figure B4. Kirkland 2C3 – Soil-Water Characteristic Curve

Table B1. Curve Fitting Parameters for Kirkland 2C3 SWCC

Parameters/Error/R^2	**Value**
a	2.587 (kPa)
n	5.239
m	0.1907
Optimized/Minimized SSE	0.000001536
R-squared	0.9978

6.1.3 Specimen Kirkland 3B2

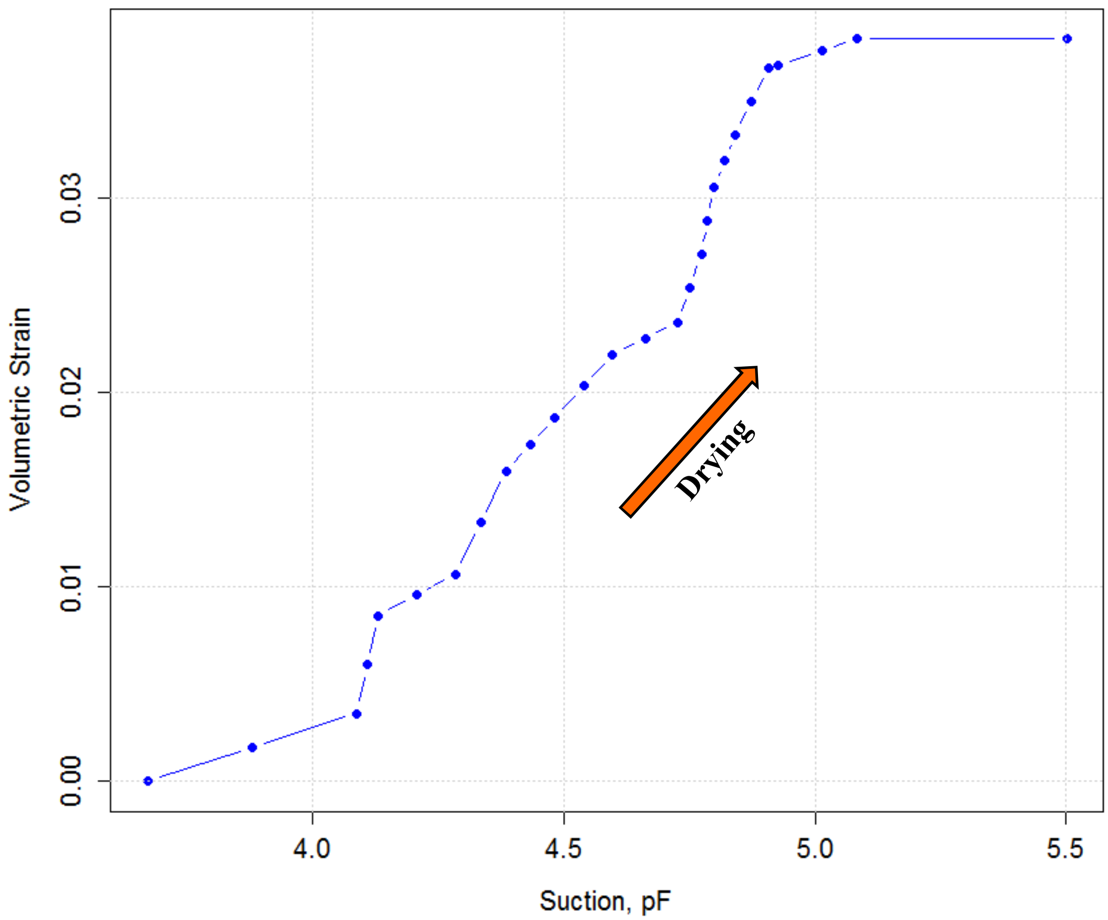

Figure B5. Kirkland 3B2: Suction Vs Volumetric Strain – Drying Test

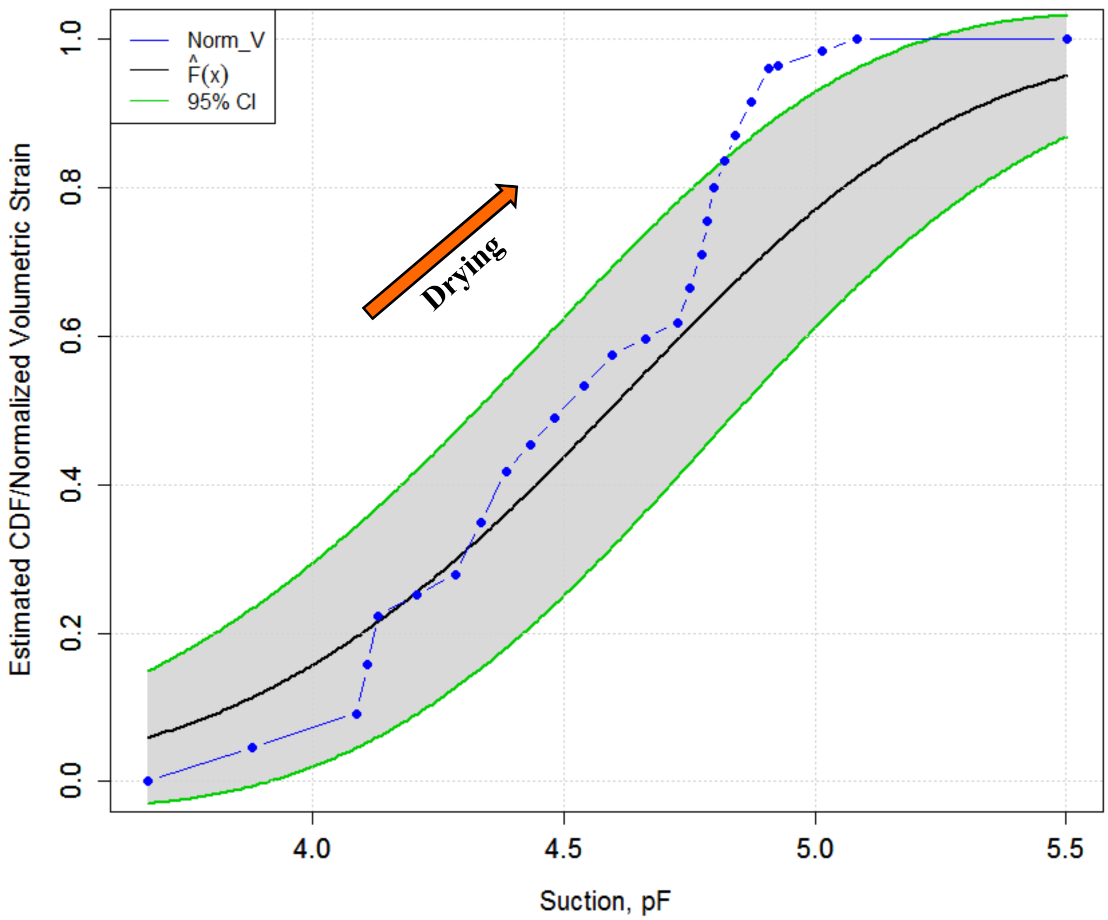

Figure B6. Kirkland 3B2: Kernel Estimate for CDF with 95% Confidence Interval – Drying Test

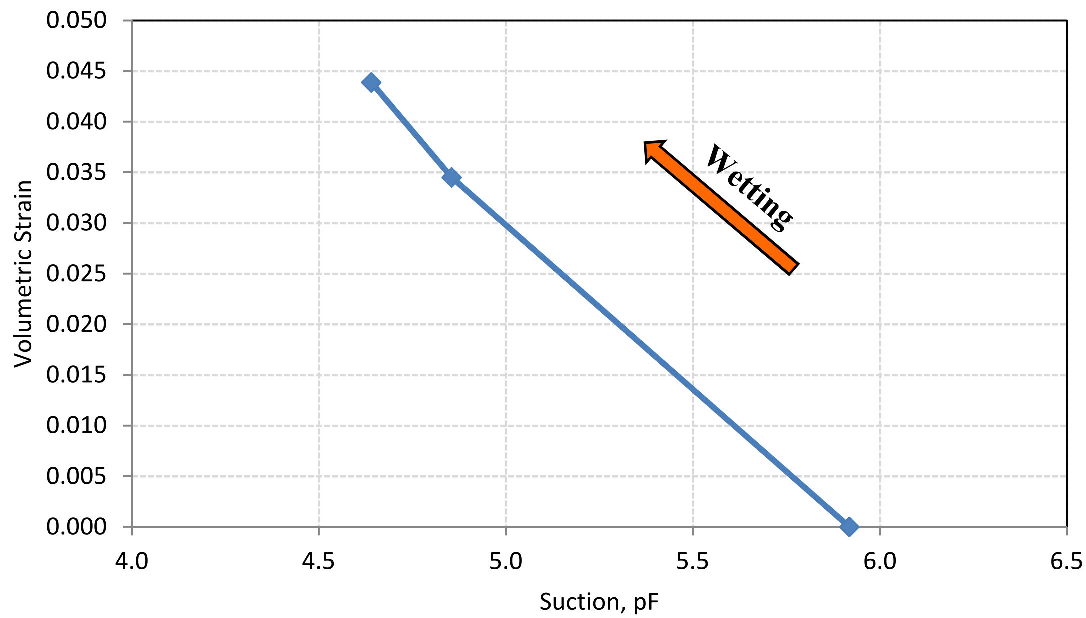

Figure B7. Kirkland 3B2: Suction Vs Volumetric Strain – Wetting Test

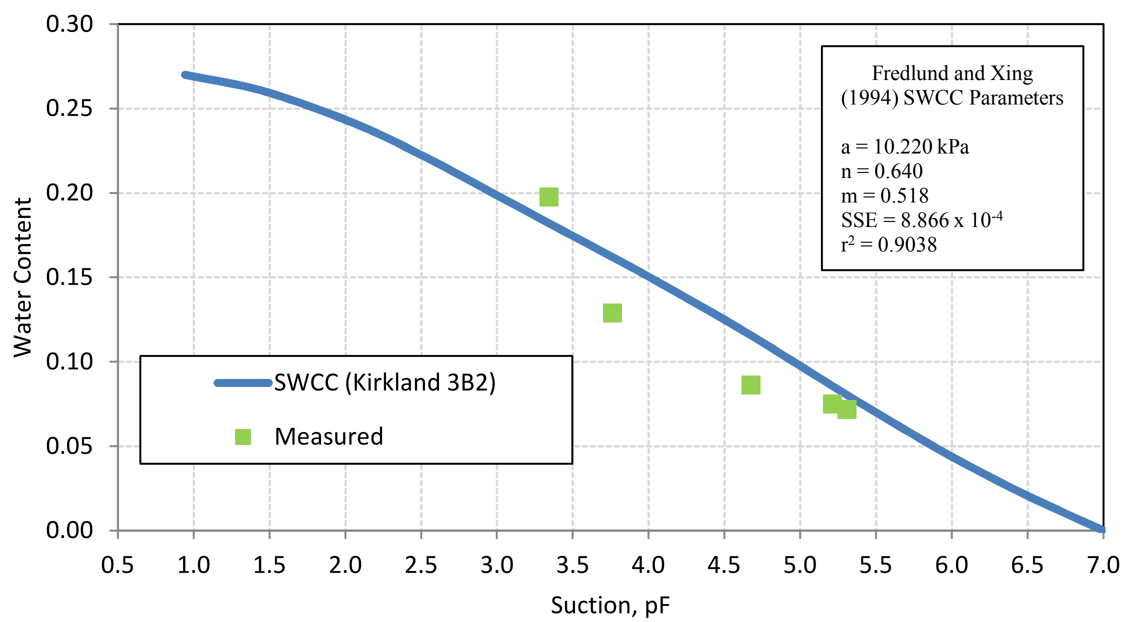

Figure B8. Kirkland 3B2 – Soil-Water Characteristic Curve

Table B2. Curve Fitting Parameters for Kirkland 3B2 SWCC

Parameters/Error/R^2	**Value**
a	10.22 (kPa)
n	0.6404
m	0.5177
Optimized/Minimized SSE	0.0008866
R-squared	0.9038

6.2 Port Site

6.2.1 Specimen Port 2A2

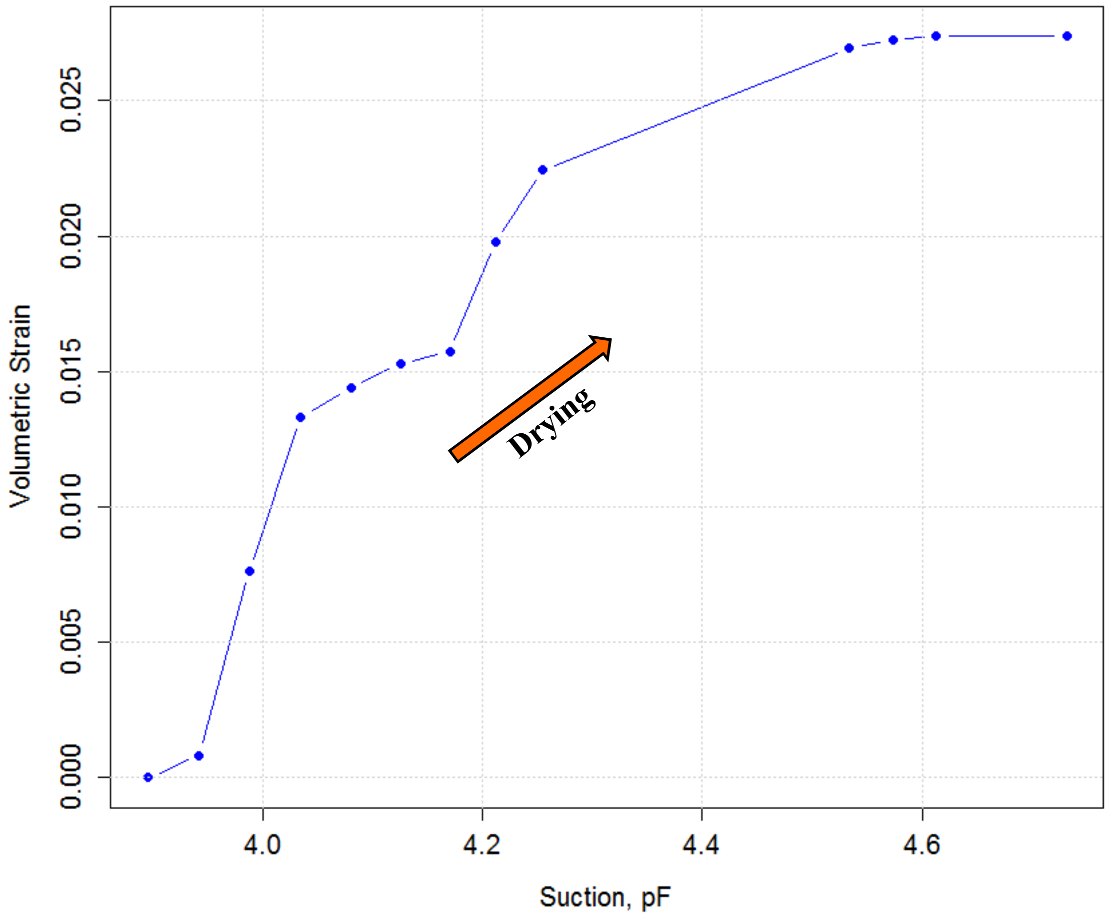

Figure B9. Port 2A2: Suction Vs Volumetric Strain – Drying Test

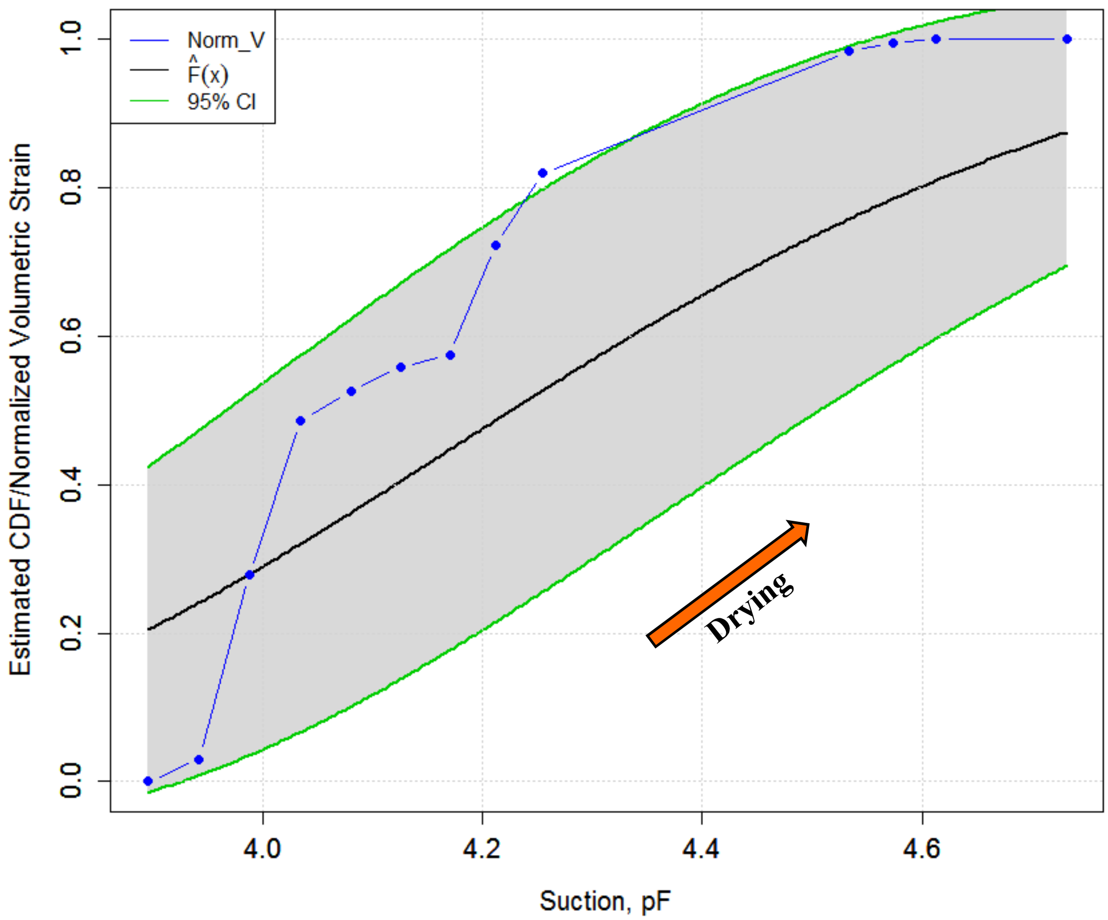

Figure B10. Port 2A2: Kernel Estimate for CDF with 95% Confidence Interval – Drying Test

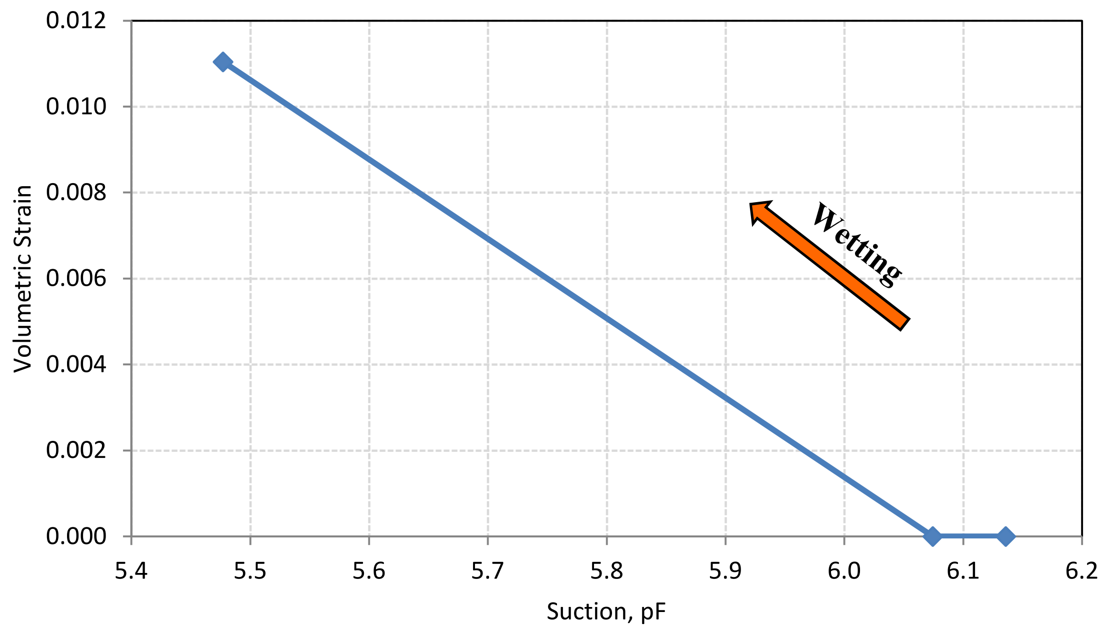

Figure B11. Port 2A2: Suction Vs Volumetric Strain – Wetting Test

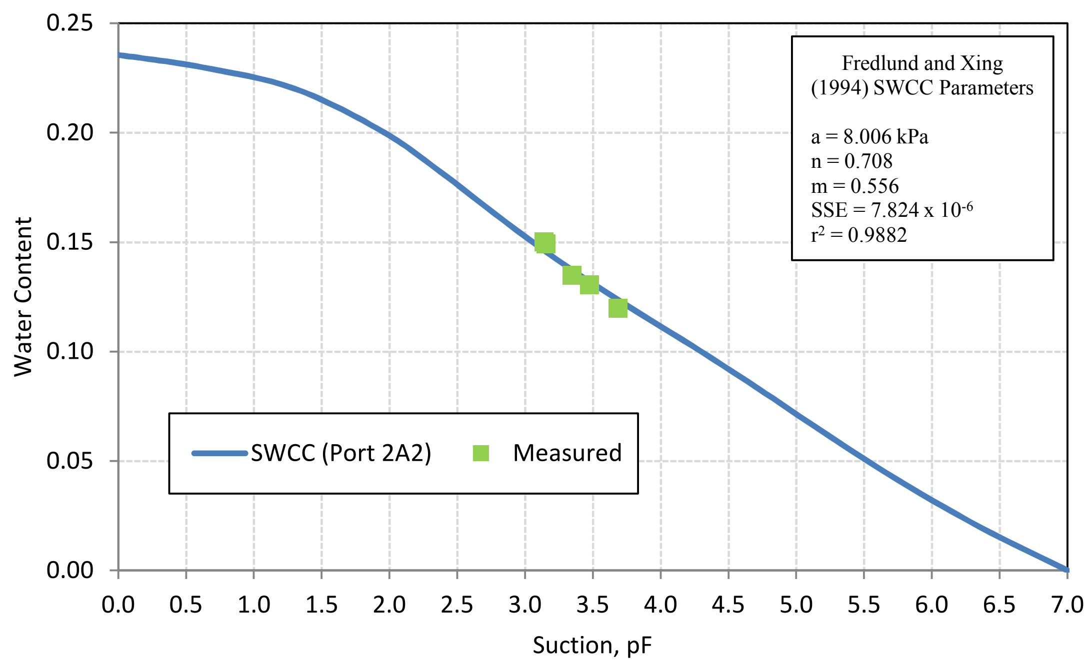

Figure B12. Port 2A2 – Soil-Water Characteristic Curve

Table B3. Curve Fitting Parameters for Port 2A2 SWCC

Parameters/Error/R^2	**Value**
a	8.006 (kPa)
n	0.7077
m	0.5558
Optimized/Minimized SSE	0.000007824
R-squared	0.9882

6.2.2 Specimen Port 4A2

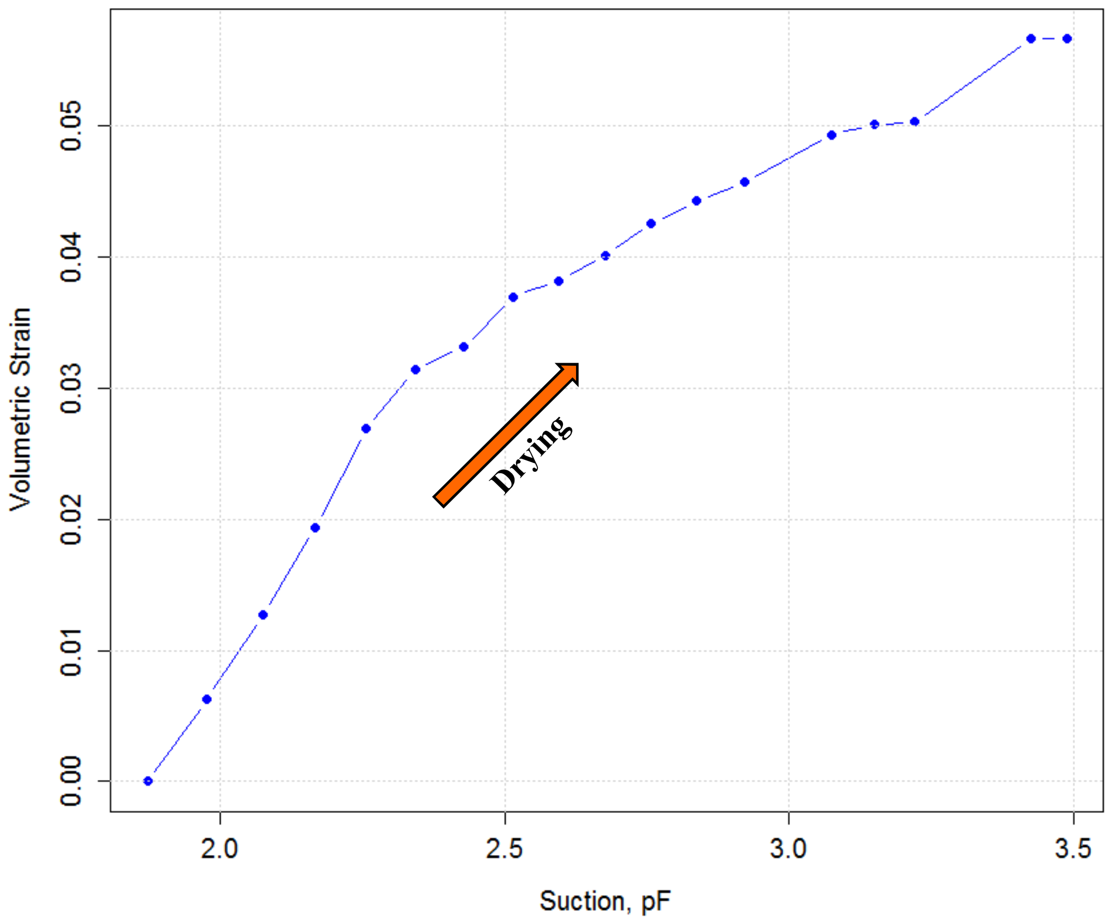

Figure B13. Port 4A2: Suction Vs Volumetric Strain – Drying Test

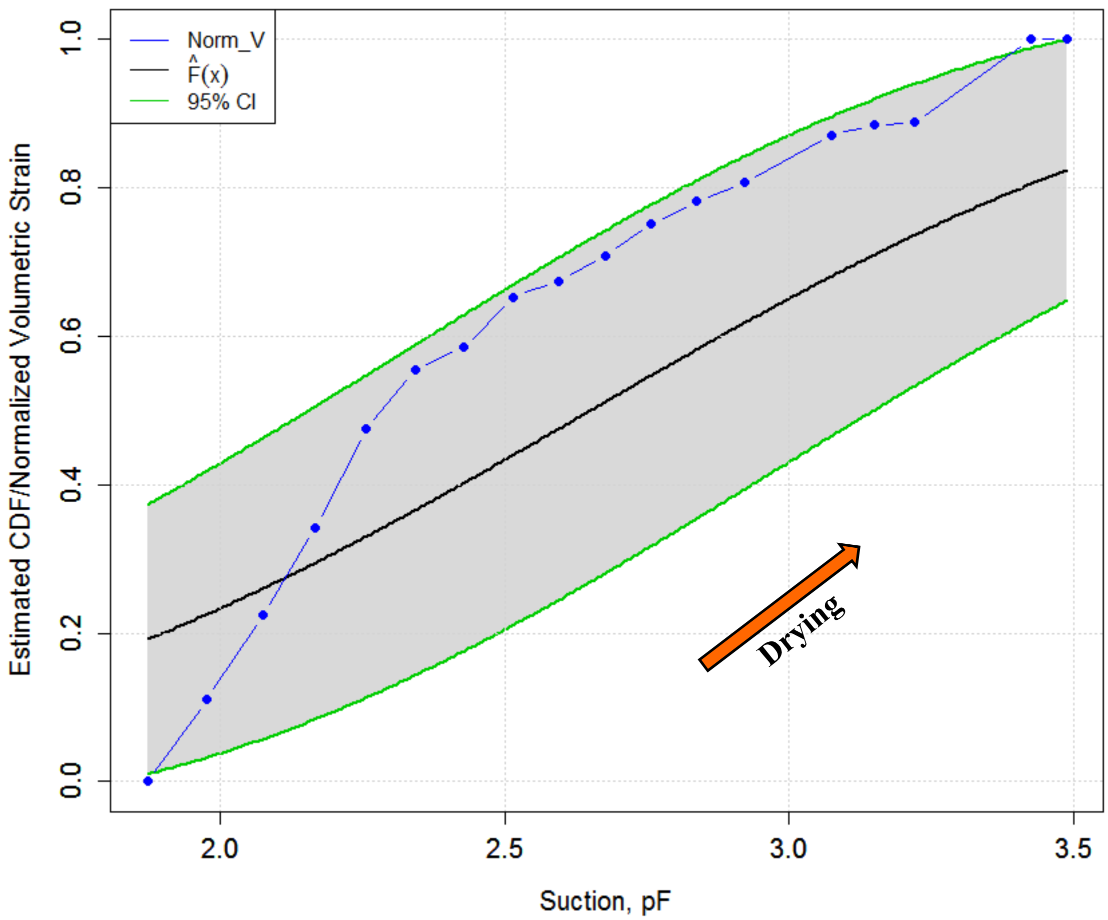

Figure B14. Port 4A2: Kernel Estimate for CDF with 95% Confidence Interval – Drying Test

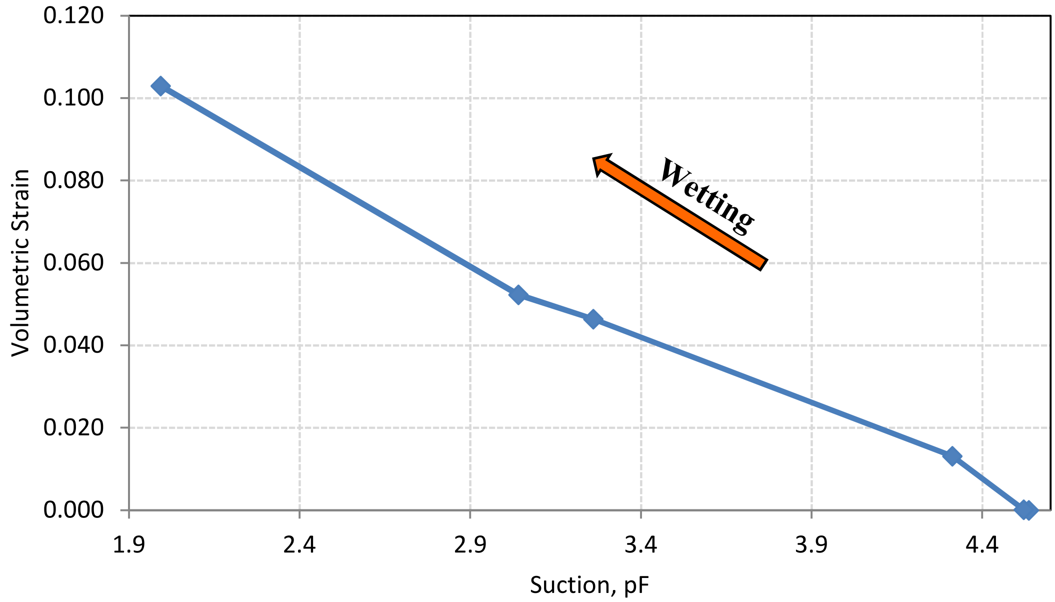

Figure B15. Port 4A2: Suction Vs Volumetric Strain – Wetting Test

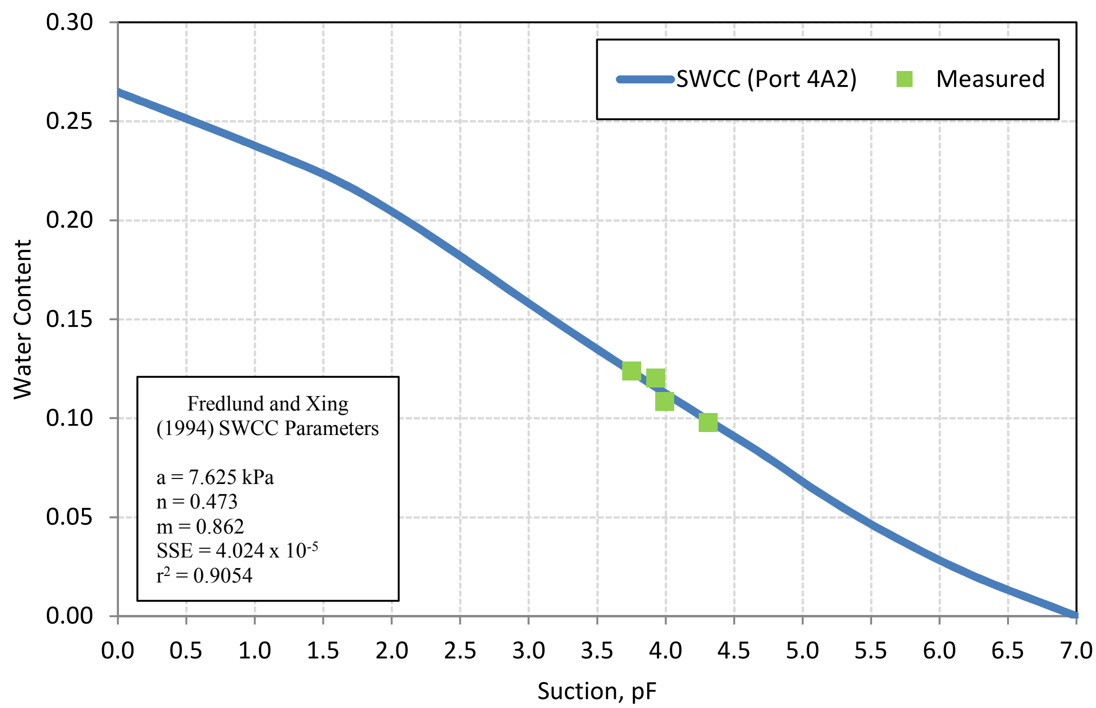

Figure B16. Port 4A2 – Soil-Water Characteristic Curve

Table B4. Curve Fitting Parameters for Port 4A2 SWCC

Parameters/Error/R^2	**Value**
a	7.625 (kPa)
n	0.473
m	0.8615
Optimized/Minimized SSE	0.000040240
R-squared	0.9054

6.2.3 Specimen Port 6A2

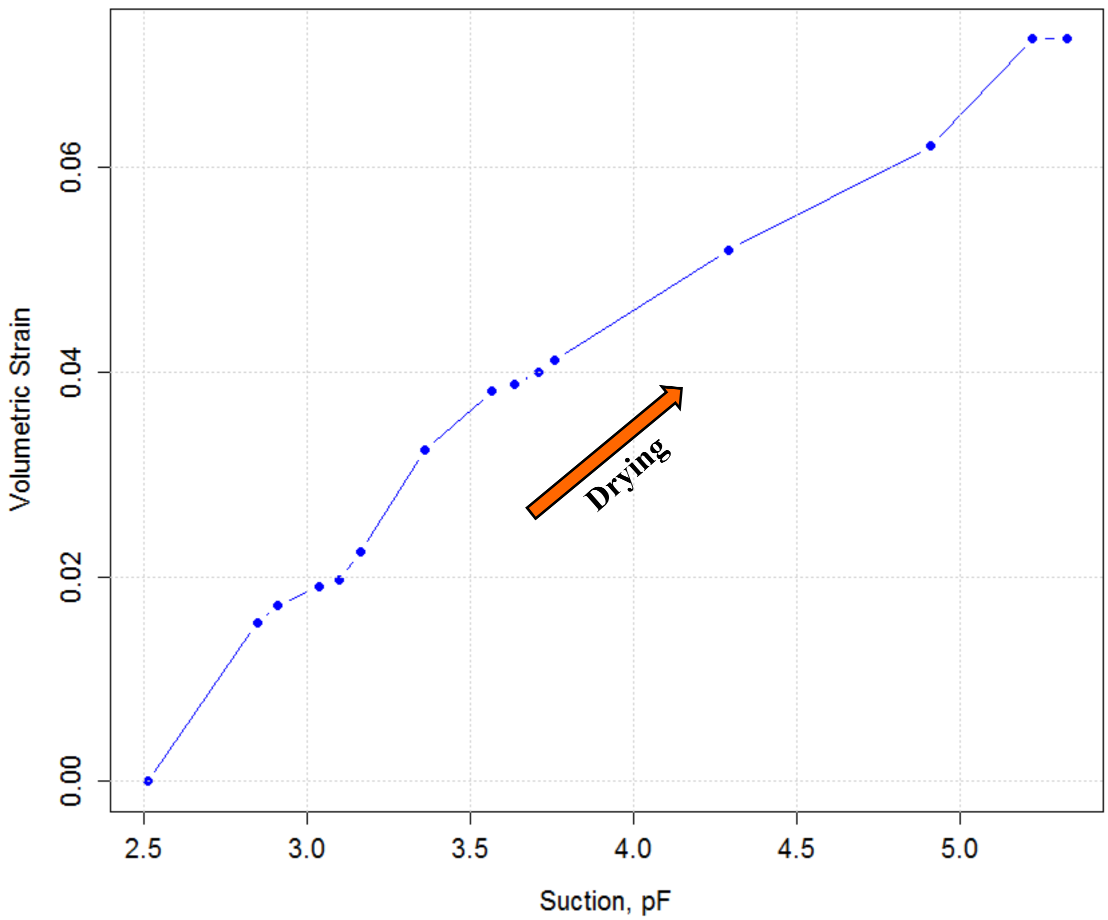

Figure B17. Port 6A2: Suction Vs Volumetric Strain – Drying Test

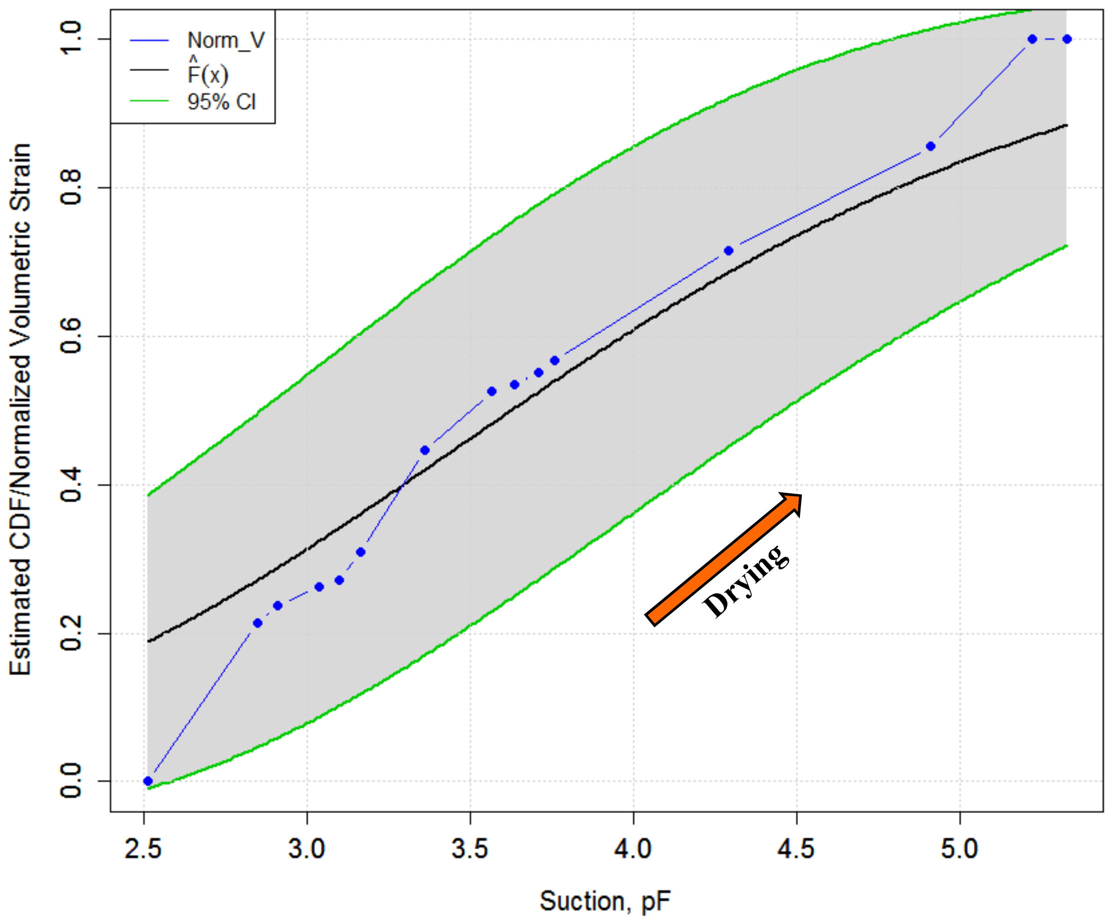

Figure B18. Port 6A2: Kernel Estimate for CDF with 95% Confidence Interval – Drying Test

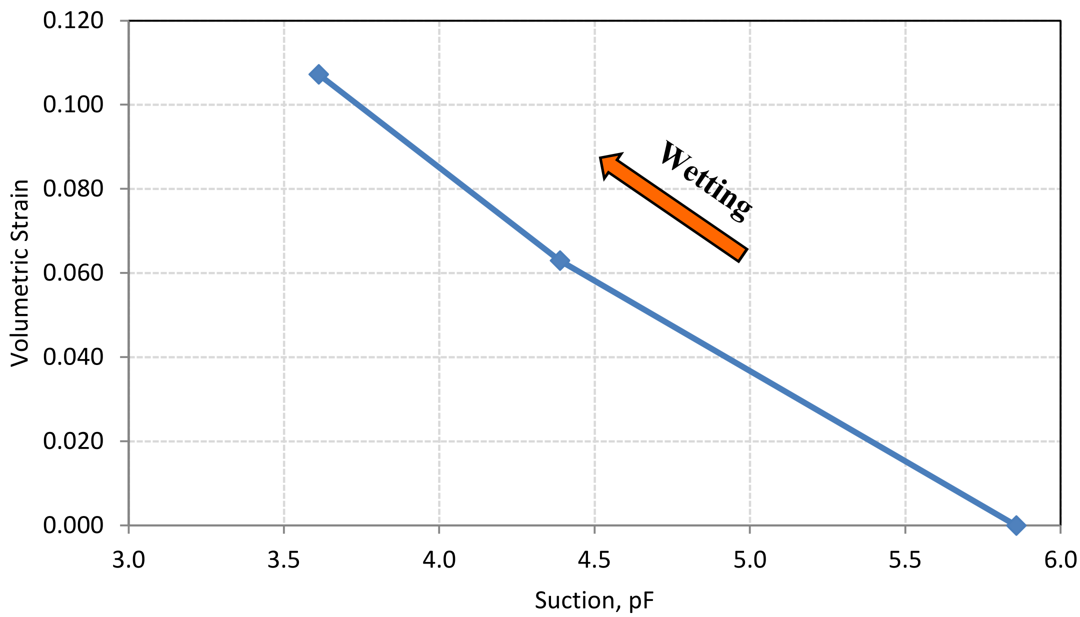

Figure B19. Port 6A2: Suction Vs Volumetric Strain – Wetting Test

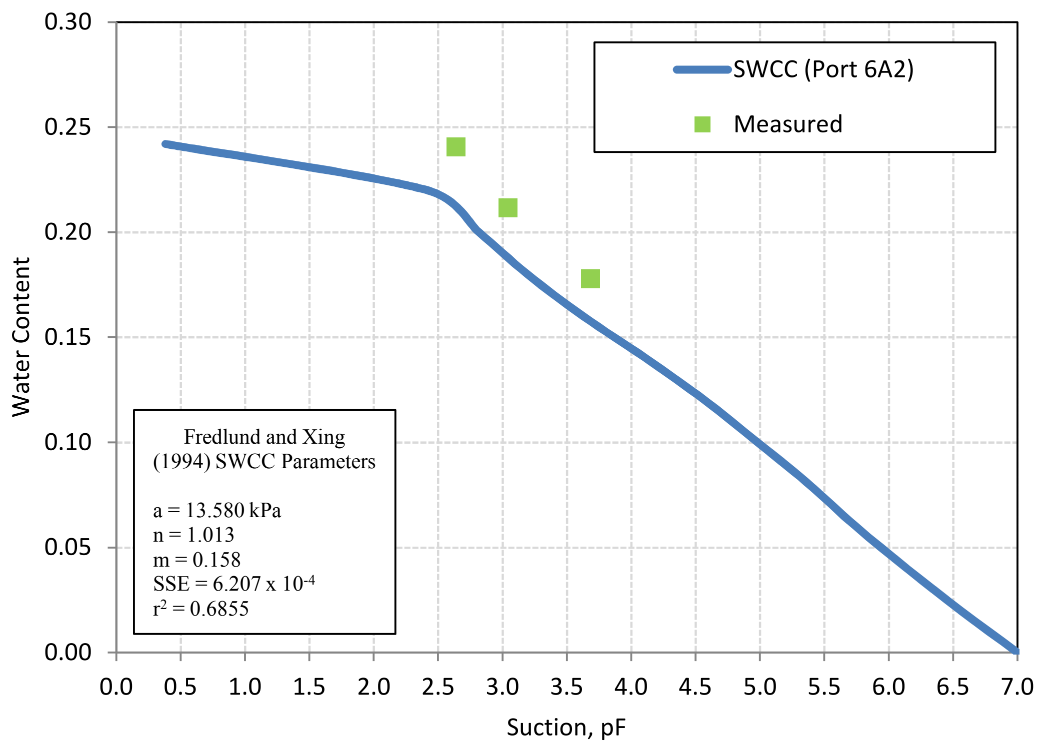

Figure B20. Port 6A2 – Soil-Water Characteristic Curve

Table B5. Curve Fitting Parameters for Port 6A2 SWCC

Parameters/Error/R^2	**Value**
a	13.58 (kPa)
n	1.013
m	0.1576
Optimized/Minimized SSE	0.0006207
R-squared	0.6855

6.3 Osage Site

6.3.1 Specimen Osage 1B2

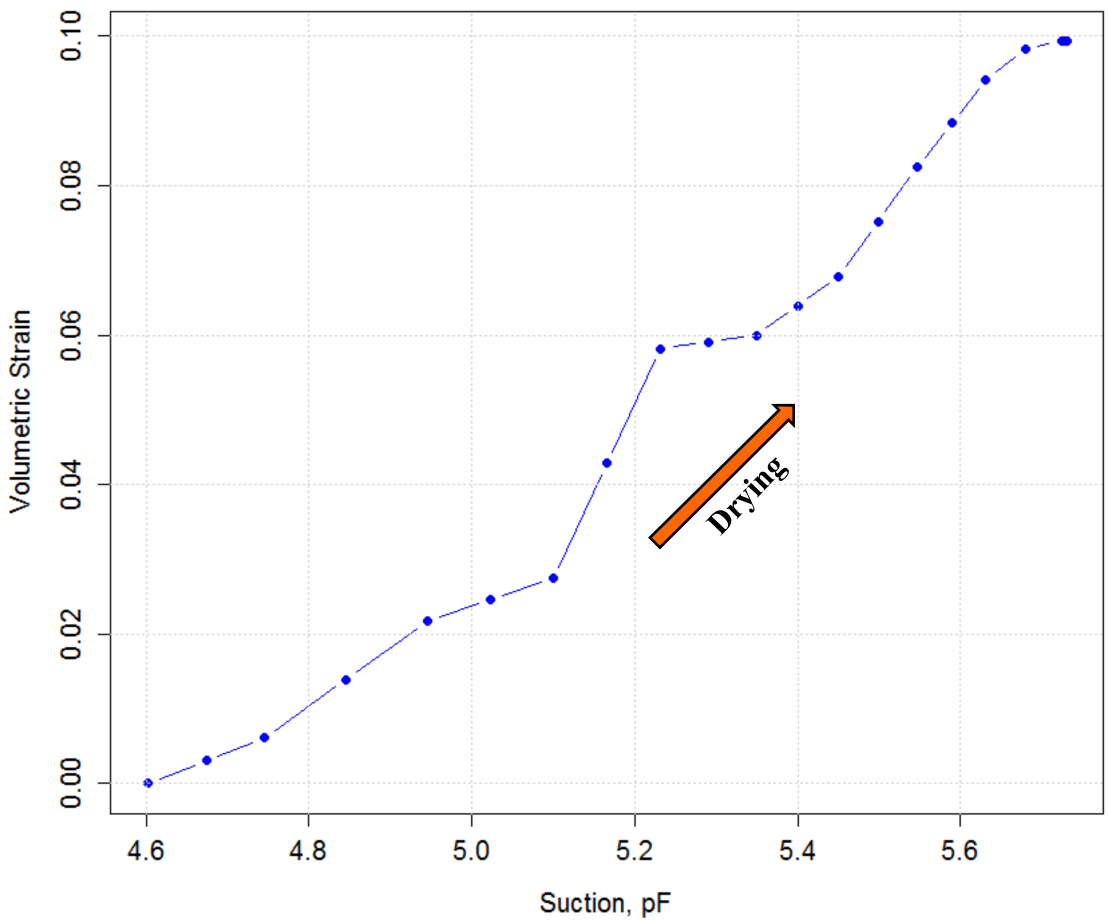

Figure B21. Osage 1B2: Suction Vs Volumetric Strain – Drying Test

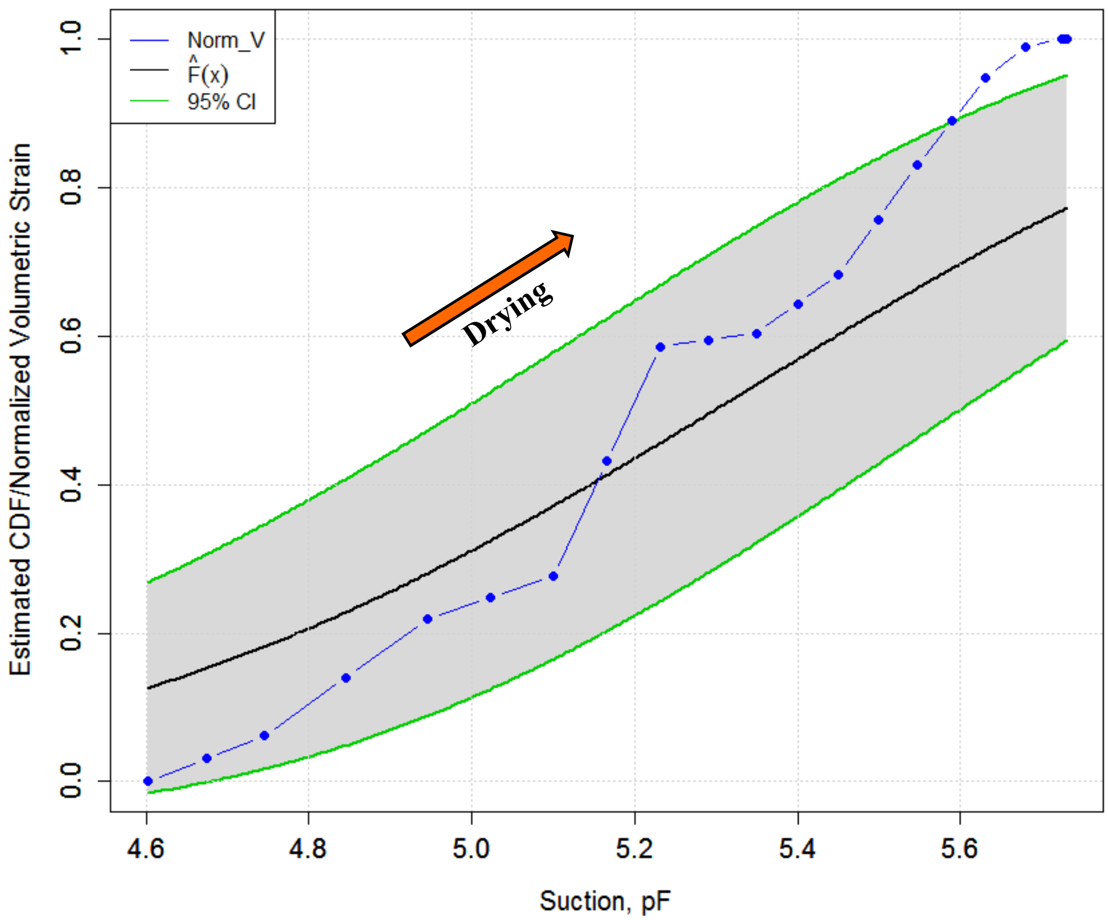

Figure B22. Osage 1B2: Kernel Estimate for CDF with 95% Confidence Interval –

Drying Test

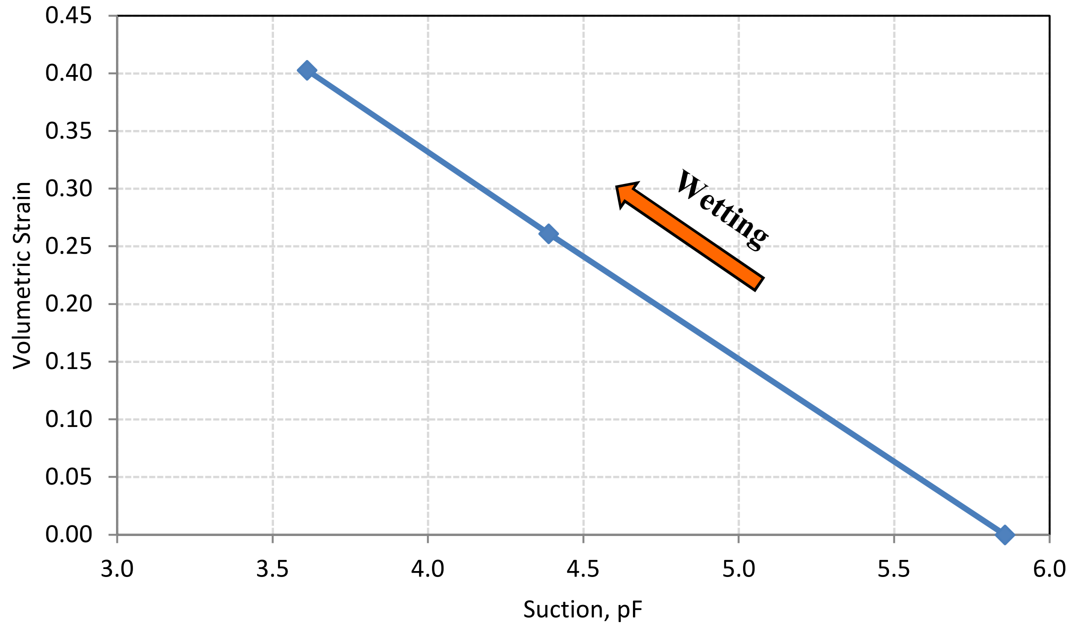

Figure B23. Osage 1B2: Suction Vs Volumetric Strain – Wetting Test

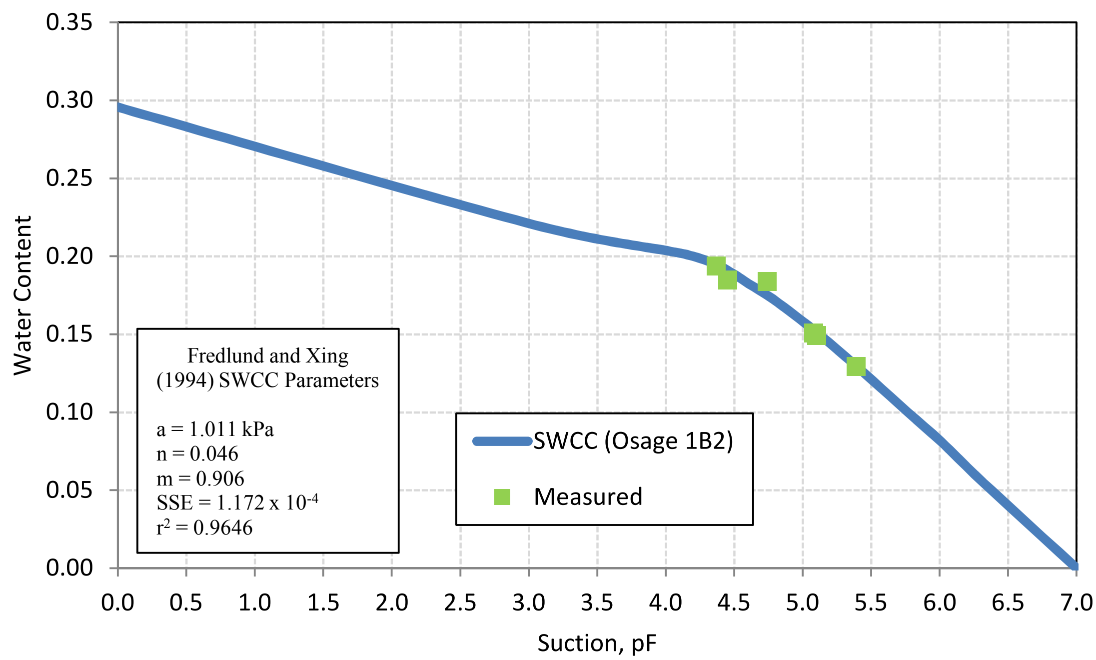

Figure B24. Osage 1B2 – Soil-Water Characteristic Curve

Table B6. Curve Fitting Parameters for Osage 1B2 SWCC

Parameters/Error/R^2	**Value**
a	1.011 (kPa)
n	0.0459
m	0.9057
Optimized/Minimized SSE	0.0001172
R-squared	0.9646

6.3.2 Specimen Osage 2C1

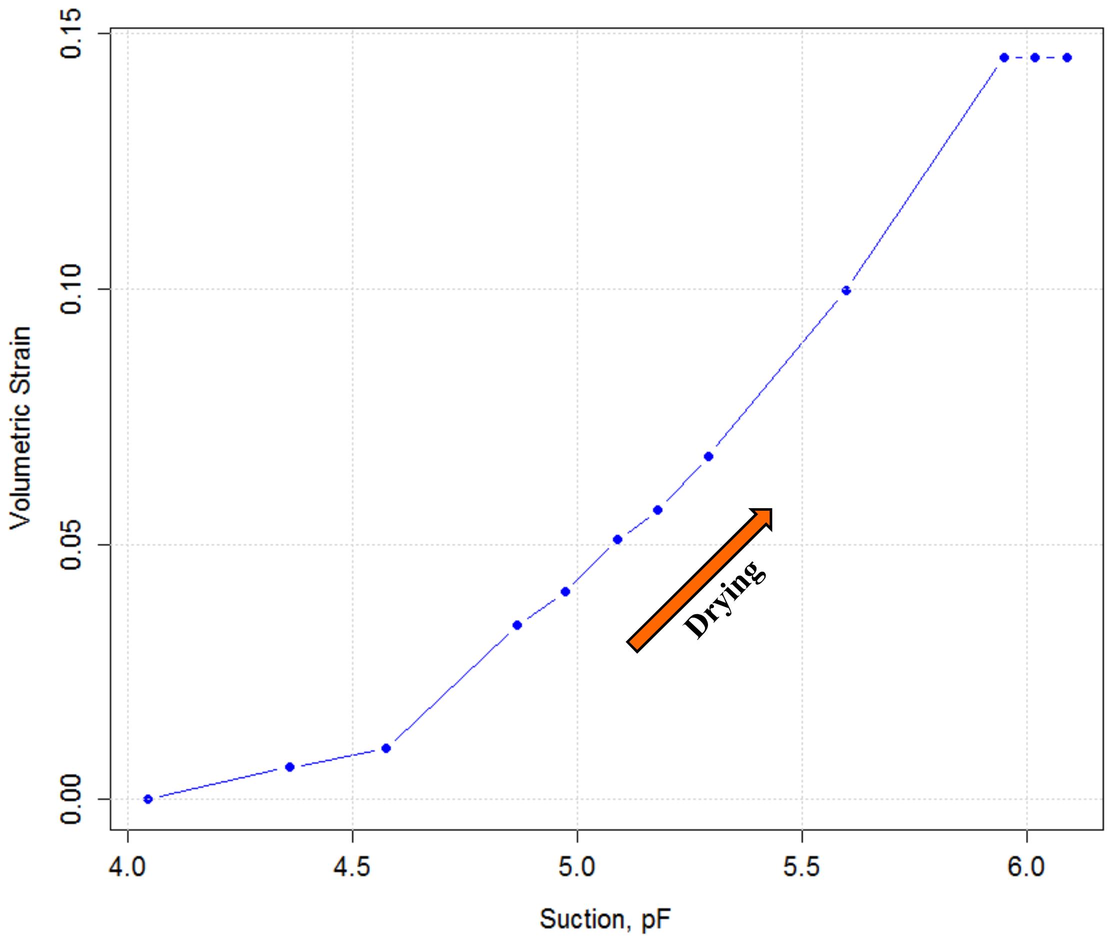

Figure B25. Osage 2C1: Suction Vs Volumetric Strain – Drying Test

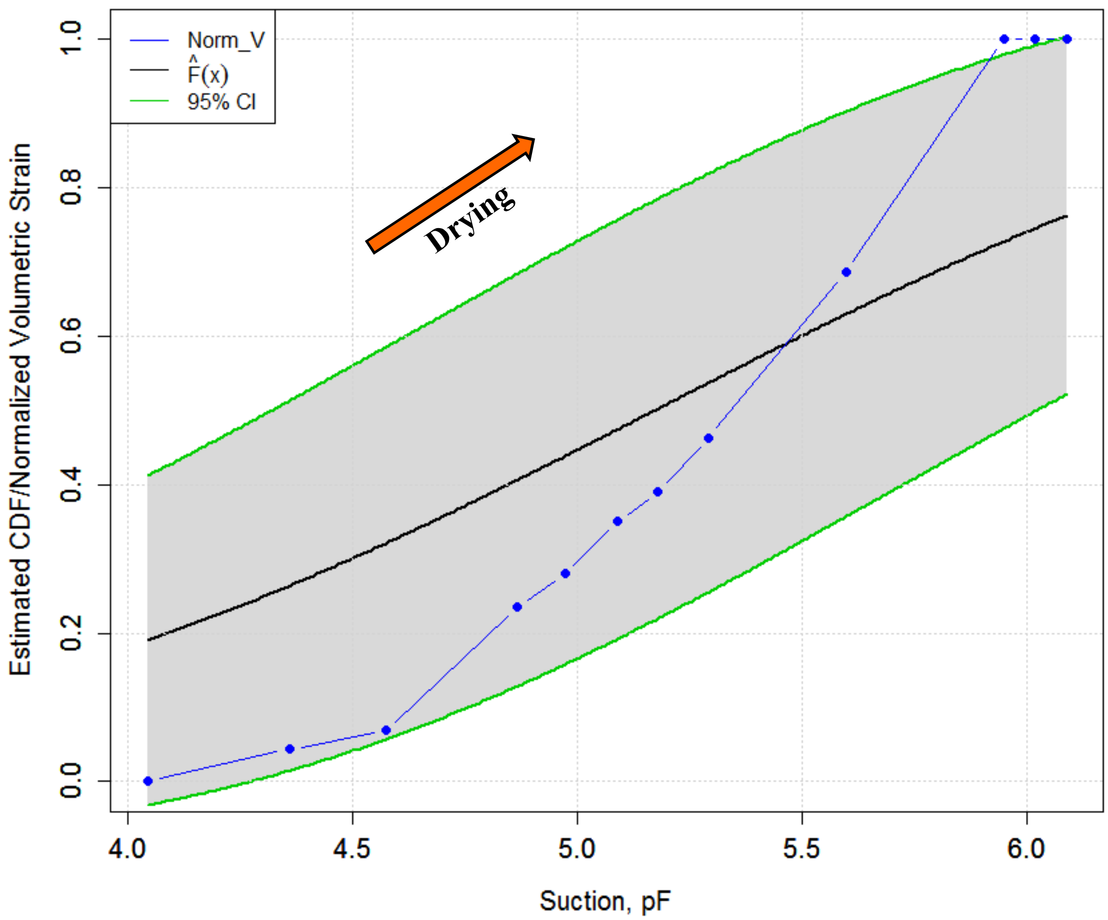

Figure B26. Osage 2C1: Kernel Estimate for CDF with 95% Confidence Interval – Drying Test

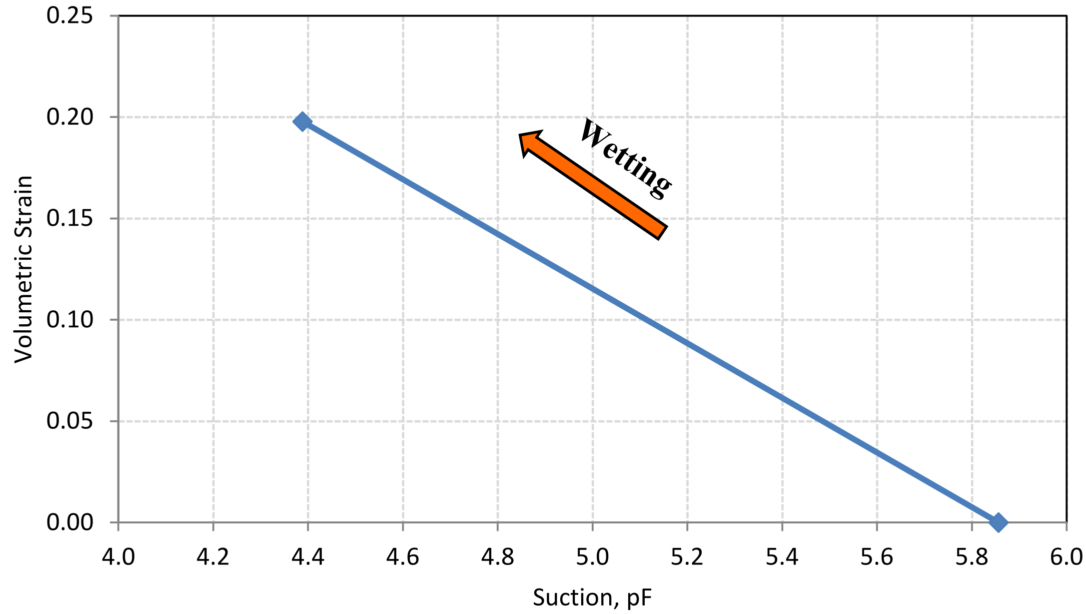

Figure B27. Osage 2C1: Suction Vs Volumetric Strain – Wetting Test

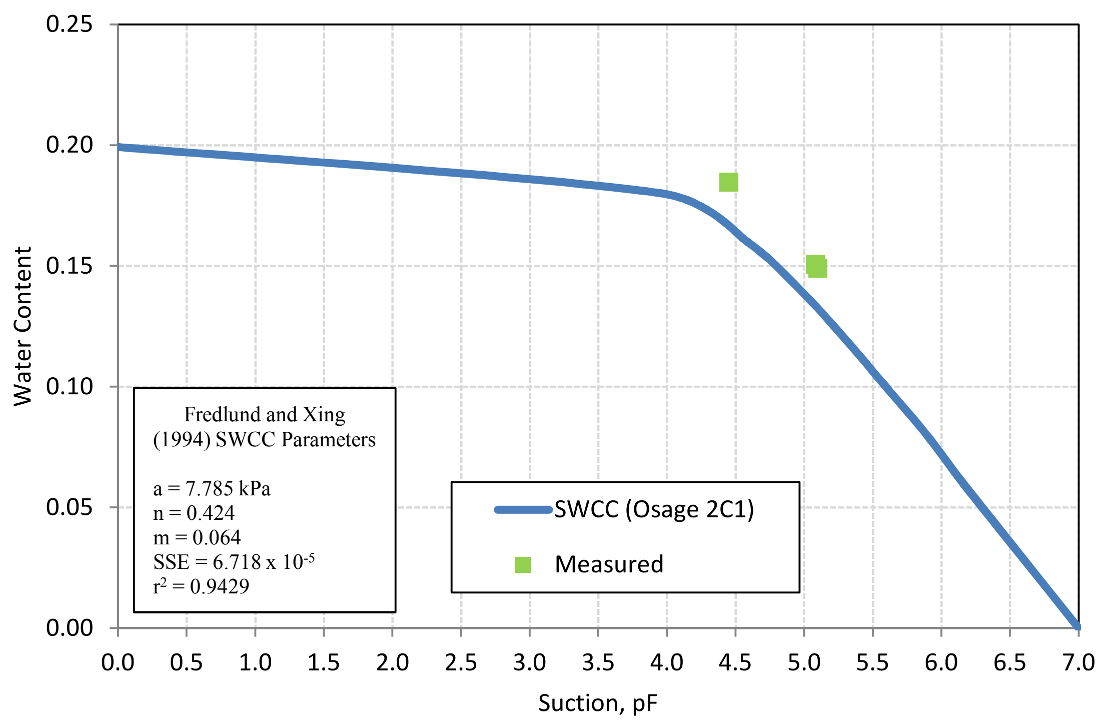

Figure B28. Osage 2C1 – Soil-Water Characteristic Curve

Table B7. Curve Fitting Parameters for Osage 2C1 SWCC

Parameters/Error/R^2	**Value**
a	7.785 (kPa)
n	0.4243
m	0.06404
Optimized/Minimized SSE	0.00006718
R-squared	0.9429

6.3.3 Specimen Osage 3A1

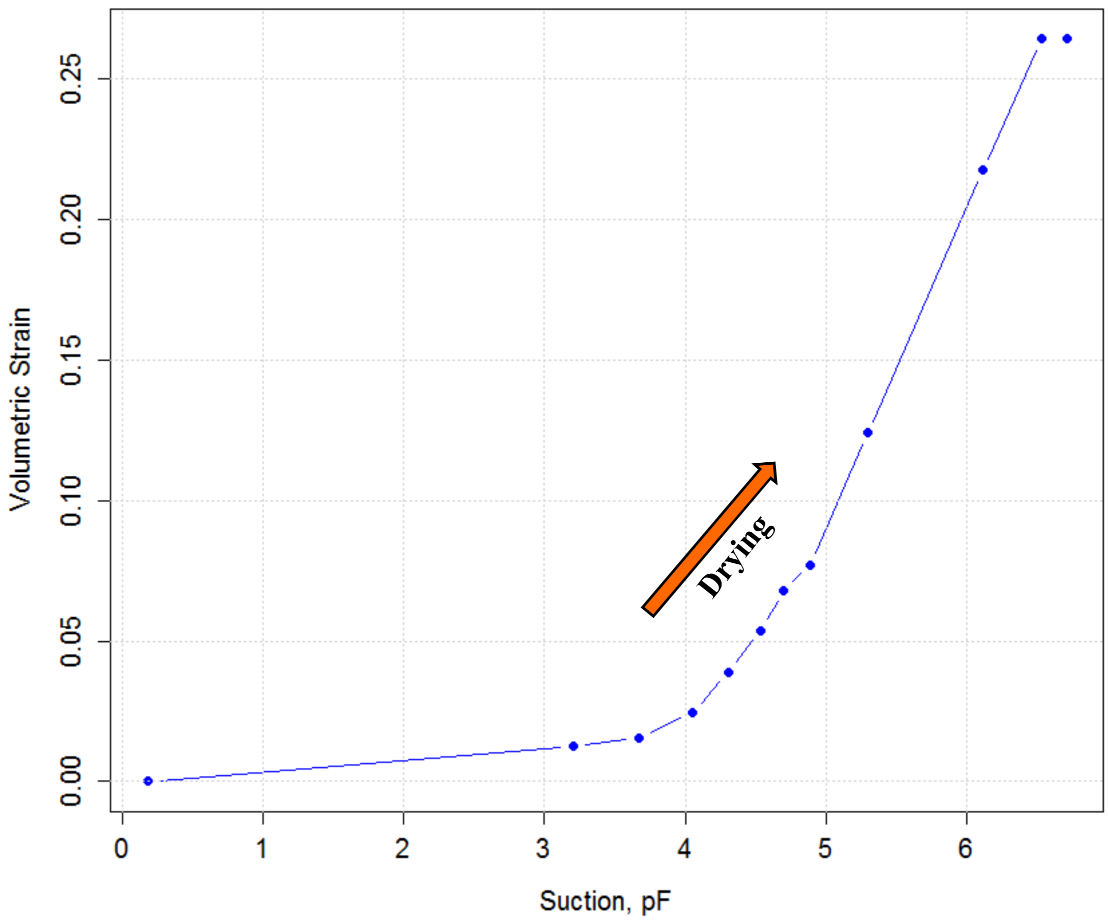

Figure B29. Osage 3A1: Suction Vs Volumetric Strain – Drying Test

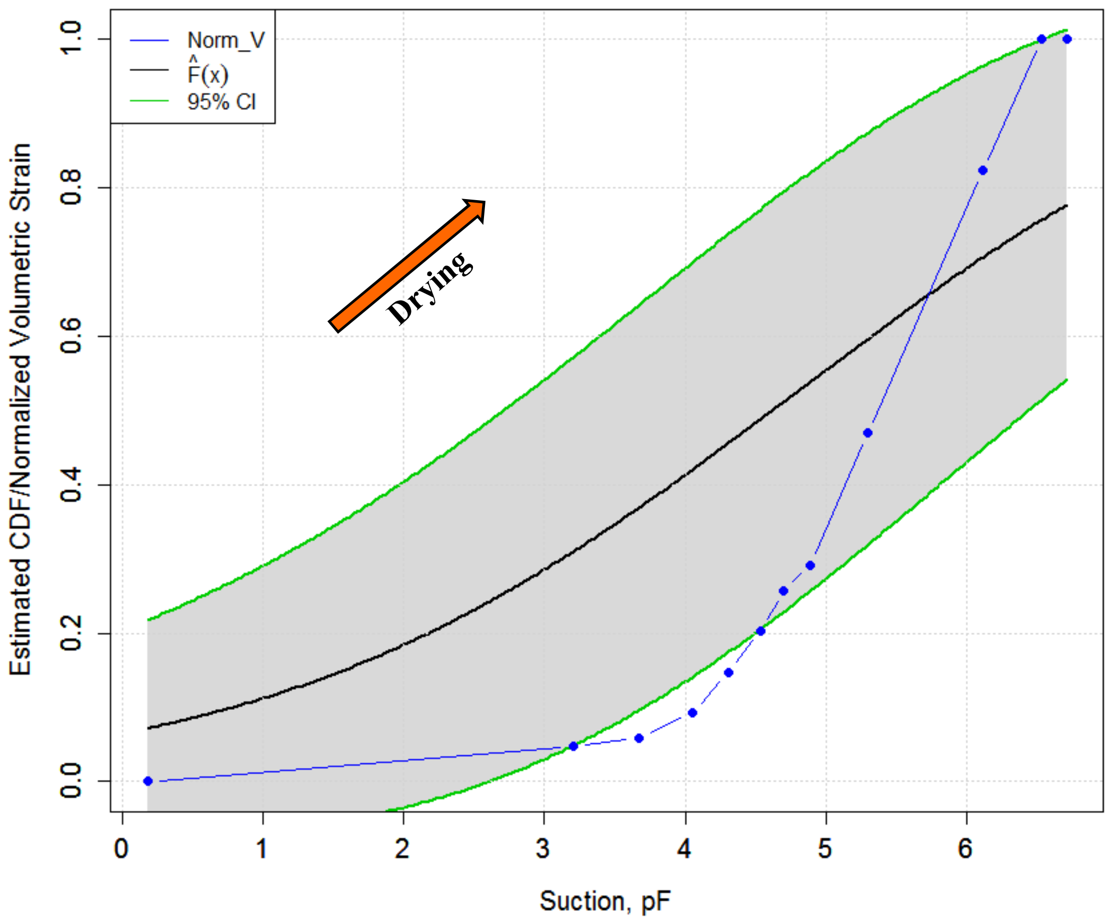

Figure B30. Osage 3A1: Kernel Estimate for CDF with 95% Confidence Interval –

Drying Test

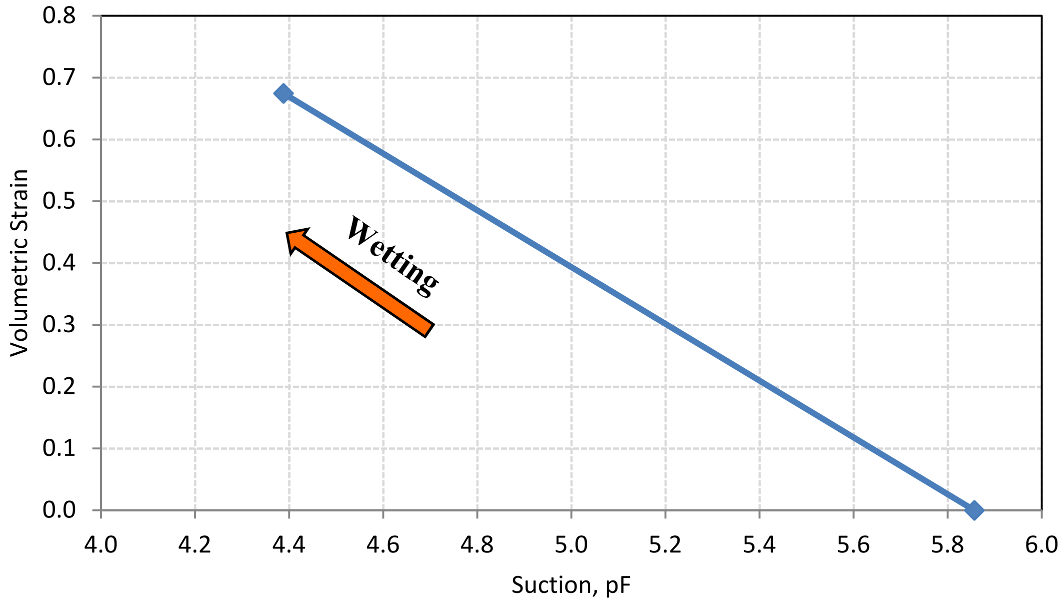

Figure B31. Osage 3A1: Suction Vs Volumetric Strain – Wetting Test

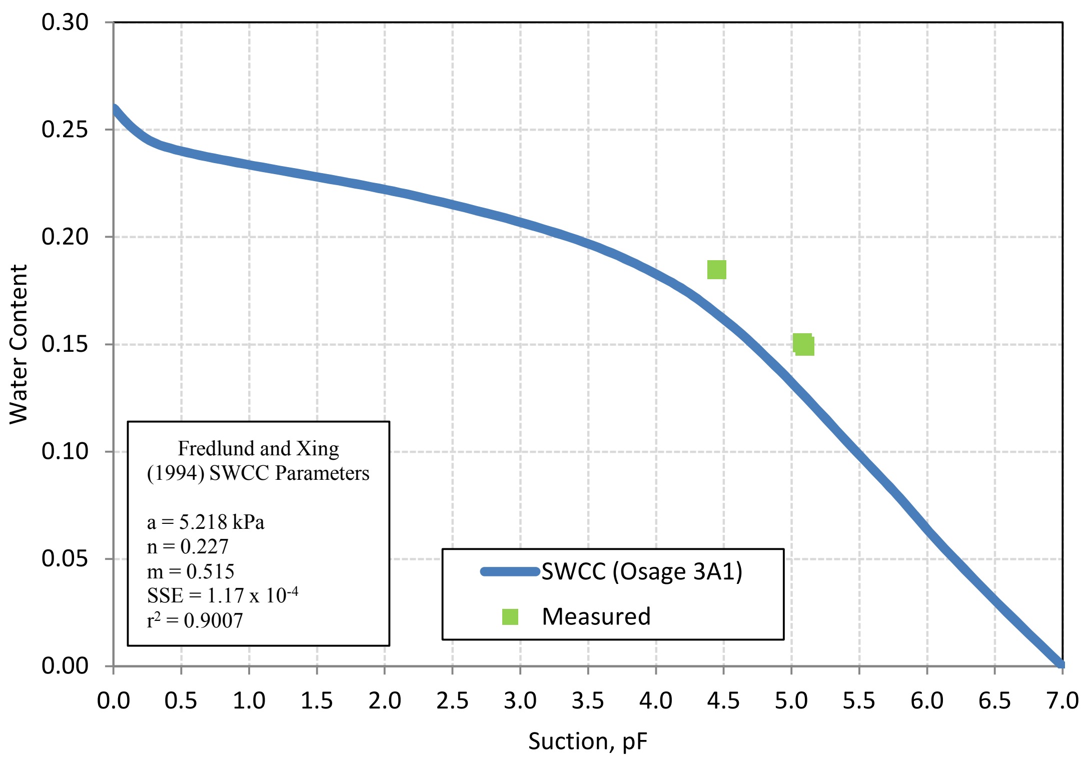

Figure B32. Osage 3A1 – Soil-Water Characteristic Curve

Table B8. Curve Fitting Parameters for Osage 3A1 SWCC

Parameters/Error/R^2	**Value**
a	5.218 (kPa)
n	0.2269
m	0.515
Optimized/Minimized SSE	0.000117
R-squared	0.9007

VITA

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