2009

Hydro mechanical behavior of unsaturated soil subjected to drying

Dev Raj Pokhrel
Louisiana State University and Agricultural and Mechanical College

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HYDRO MECHANICAL BEHAVIOR OF UNSATURATED SOIL SUBJECTED TO DRYING

A Thesis
Submitted to the Graduate Faculty of the
Louisiana State University and
Agricultural and Mechanical College
in partial fulfillment of the
requirements for the degree of
Master of Science in Civil Engineering
in
The Department of Civil and Environmental Engineering

By
Dev Raj Pokhrel
B.S., Tribhuvan University, 2002
M.S., The University of Findlay, 2009
December 2009
To my Wife, Son and Daughter
ACKNOWLEDGEMENTS

I am obliged to my advisor Dr. Radhey S. Sharma for his immense help and support with valuable suggestions and guidelines throughout my graduate studies at LSU. His valuable comments, criticism, and encouragement considerably increased my motivation for research. I would like to express my gratitude Dr. Zhi Qiang Deng for being a co-chair in my committee at the last moment, after the move of Dr. Sharma to the West Virginia University. I would like to thank Dr. Murad Yusuf Abu-Farsakh for being a member in my graduate committee.

I would also like to thank and acknowledge all the professors including Dr. Gouping Zhang, Dr. Frank T-C. Tsai, and Dr. Suresh Moorthy who taught and provided me a good education during my studies at LSU.

I would like to thank my friends Sukanta, Binay, Anantha, Luke, Mahendra, Pratima, and Vivek for providing me help and support throughout my study. I am also thankful to all my friends who gave me valuable suggestions, guidance and support in various ways during my stay at LSU.

I have my special gratitude to my family who has helped me a lot with their understanding and love. I would like to express my sincere thanks to my wife, Mrs. Sharda Pokhrel, for her unlimited support, patience, encouragement, and understanding during the hard time with cheerful smile. I am blessed for having two wonderful kids, Abhishekh and Anushree who have been making me energetic all the time. I would like to dedicate this thesis to Sharada, Abhishekh and Anushree. I am grateful to you all.
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NOTATIONS

\( \gamma \) Soil unit weight

\( \gamma_d \) Soil dry unit weight

\( G_{\text{max}} \) Small strain stiffness

\( V_s \) Shear wave velocity

\( \rho \) Soil mass density

\( \rho_d \) Dry density

\( \sigma' \) Effective stress

\( u_a - u_w \) Matric Suction

\( u_a \) Pore air pressure

\( u_w \) Pore water pressure

\( e \) Void ratio

\( L_s \) Length of the specimen in bender elements

\( \text{SWCC} \) Soil water characteristic curve

\( t_s \) Time taken by the shear wave to travel through the \( L_s \)

\( \text{BE} \) Bender element

\( K \) Hydraulic conductivity

\( k_s \) Saturated Hydraulic conductivity

\( K_{sb} \) Hydraulic conductivity for Bentonite soil

\( k_w \) Unsaturated co-efficient of permeability

\( \theta \) Water content

\( \theta_r \) Residual water content

\( \theta_w \) Volumetric water content
S  Degree of saturation
h  Matric suction
S  A sink term
S_{e}  Effective degree of saturation
S_{r}  Residual degree of saturation
a, b  Constant terms
g  Gravitational constant
\lambda  Pore size distribution index
m  Soil water parameter
r  Hydraulic radius
n  Pore size index
\alpha  Bubbling pressure
\nu  Kinematic of viscosity
V_{w}  Volume of water content
V_{b}  Bulk volume
C  Shape factor
L  Pore connectivity parameter

HC_{\text{CritA}} - Absolute value of the allowable minimum allowed pressure head at the soil surface \([L]\) applied to atmospheric boundary.
ABSTRACT

The literature of unsaturated soils in terms of hydraulic and mechanical behavior is reviewed. The hydro-mechanical behavior of unsaturated soil was studied using experimental investigation and numerical simulation of 1D and 2D modeling in HYDRUS software. Low plasticity silty clay (CL) and sand bentonite mixed soil SB (5% and 2% bentonite) specimens were investigated. Soil water characteristics curve (SWCC) tests were carried out in CL sample along with shear wave velocity measurement test using bender element (BE). The influence of moisture variation on specimens was investigated to see its impact on suction and on stiffness of soil. Similarly, the influence of suction on stiffness of soil and influence of density on stiffness of soil were also observed separately. To see the influence of moisture variation on suction (SWCC) and the influence of moisture variation on soil hydraulic properties (specially, hydraulic conductivity), finite element modeling was prepared representing 1D and 2D simulation using HYDRUS software. The results from the experimental investigation were compared with the results from the HYDRUS simulation. From the research, it is concluded that if the moisture content of the soil is decreased, both suction and stiffness modulus of soil is found to be increased. With the decreased moisture content, the initial increasing rate of suction was found higher than the rate of increase of stiffness modulus of the specimen. It was also observed that further decreasing the moisture content to a certain level, stiffness of soil is found to be increased with higher rate than the rate of change of suction.

The available models for prediction of soil water characteristics curve (SWCC) based on grain size distribution and index properties proposed by various researchers were also examined. Recommendations are proposed for future work.
CHAPTER ONE: INTRODUCTION

1.1 General Background

Various studies have shown that the behavior of unsaturated zone has a complex nature. Many geotechnical problems pertaining to soil moisture interaction in vadose zone has been reported in recent years (Fredlund, 1993). There is a growing concern of the study of vadose zone, which has been studied mainly in two aspects: one aspect is geotechnical engineering point of view in which the influence of fluctuation of ground water table on strength characteristics of soil is studied. The second aspect of studying vadose zone is from the geo-environmental point of view. It is reported in various literature that the quality of subsurface environment is being adversely affected by agricultural, industrial, and municipal activities all over the world, so the study of unsaturated zone (vadose zone) is becoming essential.

Traditionally, Geotechnical engineering has been studied using the concept of soil mechanics assuming the earth surface at completely dry condition or fully saturated condition. According to Fredlund and Rahardjo (1993), classical soil mechanics and geotechnical engineering have been studied assuming that soil is either dry (0% saturation) or saturated (100% saturation). In fact, in real geotechnical engineering problems, soil is generally neither fully saturated nor in a dry condition. The two extreme and limiting condition of a soil as dry and saturated condition do not exist all the times, which means, the soil can have degree of saturation between 0% and 100%.

A vast portion of the earth’s surface is subjected to arid and semi-arid climatic conditions. Soils in these regions are dry and desiccated near the ground surface. Even under humid climatic conditions the groundwater table can be well below the ground surface and the soils used in construction are unsaturated in nature. The variation of climatic conditions such as precipitation, temperature, humidity, evaporation and transpiration, the position of ground water table may
fluctuate time to time. The variation in moisture content can lead to change in hydro-mechanical behavior of unsaturated soil.

Various theories have been established for saturated soil, but unsaturated soil properties have not been addressed in details by researchers till date. Unsaturated soil mechanics is relatively a new discipline developed around in the past 40 years. Considerable attention has been paid to develop the fundamentals of unsaturated soil mechanics. In recent years, the effort has been made towards developing models for constitutive behavior of unsaturated soil. Various difficulties associated in this field are:

1) Negative pore water pressure (also called suction) developed in unsaturated zone is difficult to measure.
2) Prediction of unsaturated soil behavior is difficult task.
3) Unsaturated soil testing is costly, time consuming, and difficult to conduct.
4) In comparison to saturated/ dry soil, the theories of unsaturated soil that are developed till date are insufficient and complex to study.

Several researchers such as Bishop and Blight (1963) defined unsaturated soil using a single effective stress principle. The concept of two independent stress states variables was proposed by Jennings and Burland (1961) which are, the net normal stress ($\sigma - u_a$) and matric suction ($u_a - u_w$) ; where $\sigma$ is the total stress, and $u_a$ and $u_w$ are the pore air and pore water pressures, respectively. These independent stress state variables are being widely accepted in the study of unsaturated soil mechanics.

1.2 Problem Statement and Research Background

Geotechnical engineering has traditionally been studied using the principle of mechanics for solving soil strength, seepage and strain analysis. In modern geotechnical science, the stress state
variable approach is becoming the means for transferring unsaturated soil behavior. The behavior of unsaturated soils is of importance in a diverse range of geotechnical and environmental engineering projects.

Arid and semi-arid regions of the world cover more than one third of earth’s surface. Soil in such regions is dry and desiccated near the ground surface. The position of ground water table in such areas may be more than 30 meters below the ground surface depending upon the climatic conditions. Higher negative pore water pressure has been reported for lower moisture content. Various factors including climatic condition such as precipitation, temperature, evaporation, humidity and transpiration affects the fluctuation of ground water table and hence variation in moisture content of the soil. Due to the variation of moisture content on soil mass, the mechanical and other properties are influenced and ultimately geotechnical and geo-environmental projects are affected in a long run. Some researchers such as Ng and Bruce termed the earth’s land surface (unsaturated soil) as hazardous geo-materials to earth structures and earth–supported structures, because, on wetting, by rain or other means, they expand and upon drying, such soils contract and collapse with serious consequences in terms of cost and safety. Swelling clay, collapsing soils, and residual soils are the examples of problematic unsaturated soils in which negative pore water pressure developed after the contact with water, plays a vital role in their mechanical behavior. Given the high cost of damages to buildings, structures, highway and railway embankments, and environmental projects, such as cut-off walls and clay liners at landfill sites caused by unexpected movement of moisture, barrier control mechanism is essential to develop so that considerable reduction in damage of structures and associated cost reduction is possible. Hence the behavior of unsaturated soils is of importance in a diverse range of geotechnical and environmental engineering projects.
1.3 Objective of the Research

The objective of this research is to study the hydro-mechanical behavior of unsaturated soil in two ways.

- To study the influence of moisture content on mechanical behavior using soil water characteristics curve (SWCC) and shear stiffness through experimental investigation,
- To conduct numerical simulations using HYDRUS software and compare the result from HYDRUS with the experimental investigation.

To achieve above mentioned objectives of the research, the tasks of the research work were divided into the followings:

- The influence of moisture content on suction (SWCC)
- The influence of moisture content on stiffness
- The influence of suction on stiffness
- The influence of moisture content on hydraulic conductivity
- The influence of suction on hydraulic conductivity
- The influence of environmental change such as temperature, precipitation, evaporation on soil suction and soil hydraulic properties (water content and hydraulic conductivity)
- The relationship between grain size distribution, suction and hydraulic properties
- The study of stiffness on drying part of SWCC.

1.4 Thesis Outline

This thesis consists of six chapters including this introduction. Following is the summary of the chapter wise outline of the thesis:

Chapter one: Introduction and background of study
Chapter two: Literature review of some mechanical behavior of unsaturated soil, stress state variables, matric suction, stiffness, and soil water characteristics curve

Chapter three: Experimental setup and sample preparation

Chapter four: Numerical modeling using HYDRUS

Chapter five: Discussion of experimental research and HYDRUS simulation

Chapter Six: Conclusions and recommendations
CHAPTER TWO: LITERATURE REVIEW

2.1 Fundamentals of Unsaturated Soil Mechanics

2.1.1 Introduction

After the development of the concept effective stress principle by Karl Tergazi, (in 1936), it is agreed that classical soil mechanics moved from an empirical to a science basis. The soil behavior has been studied using the principle of effective stress which is independent of the soil properties. In the classical approach, soil mechanics is studied on the basis of assumption that soil is either saturated (100% saturation) state or dry (0% saturation) state. The portion of earth surface above water table is considered in dry state while the portion of earth surface below the water table is considered in a saturated state. Various soil engineering analysis and theories have evolved for saturated and dry condition. Most of the design principles were developed after the Tergazi’s effective stress principle came into effect to deal with shear strength, seepage and volume change behavior. According to Fredlund and Rahardjo (1993), the unsaturated soil is generally neither in a totally dry condition nor in a saturated condition, but has a degree of saturation ranging from 0 to 100 percent. Fredlund (1996) also identified that the artificial division of soil mechanics into saturated soil and unsaturated soil is not true because unsaturated soil zone contains the degree of saturation ranging from 0 to 100 percent. The behavior of soil in this zone is difficult to understand, which mainly depends upon the type of soil and degree of saturation. Theories developed for saturated and dry soil conditions cannot be applied directly to the unsaturated soil condition, but the knowledge gained from saturated soils mechanics can be used for unsaturated soil zone. The view of broader unsaturated soil world is shown in Figure 2.1.
In the classical understanding of unsaturated soil, it has been assumed that it contains three phases: soil-solid, water, and air as shown in the Figure 2.2. Later, Fredlund and Morgenstern (1977) recognized that unsaturated soil has four phases instead three i.e. solid, water, air and air water interface (which is also called contractile skin (Paddy, 1969)). Unsaturated soil in this zone is categorized according to the degree of saturation as shown in Figure 2.3. Above the water table, pore water pressure will be negative with respect to atmospheric pressure, but immediately above the water table, a new zone, defined as capillary fringe, where degree of saturation is approximately 100 percentages. The range of thickness of this zone varies from less than 1 to 10 m (Fredlund, 1996).

For simplicity, the soil mechanics can be divided by ground water table. For the portion of earth below the water table, soil behavior is governed by effective stress \((\sigma-u_w)\), whereas the unsaturated soil above the water table is governed by two independent stress variables, net
normal stress \((\sigma - u_a)\) and matric suction \((u_a - u_w)\) (Jennings and Burland, 1962, and Fredlund and Morgenstern, 1977).

![Phase diagram of classical soil mechanics](image)

**Figure 2.2: Phase diagram of classical soil mechanics**

![Unsaturated soil categorization based on degree of saturation](image)

**Figure 2.3: Unsaturated soil categorization based on degree of saturation (Fredlund, 1996)**

### 2.1.2 The Vadose Zone and Climatic Variation

The vadose zone is the representation of unsaturated soil which is the part of earth, lies above ground water table as shown in Figure 2.1 and Figure 2.3. The portion of the earth below the water table, soil is saturated and pore water pressure is always positive. As we move down, pore water pressure increases with the increase of depth. In the unsaturated soil (vadose zone) the pore
water pressure is generally negative with respect to atmospheric (gauge) pressure (Figure 2.4).

Figure 2.4: Illustration of negative pore water pressure in the vadose zone (Fredlund, 1993)

Vadose zone is further classified into three different types based upon degree of saturation. The portion immediately above the water table where the degree of saturation is almost 100 percent called capillary fringe or capillary zone. The range of thickness of this zone depends upon soil type, which is usually less than 10 meter (Fredlund, 1996). Above the capillary fringe, the degree of saturation lies between 20-90 percent depending upon the type of the soil. This layer is also called as two fluid phases because water and air are filled in the void space of soil. As we move upward the soil becomes dryer, the void of soils are totally filled by air and the negative pore water pressure increases to the maximum value, which is also termed as dry soil (Fredlund, 1996). The engineering properties of soils are affected by the movement of water table. The movement of water table is mainly due to the climatic factors such as precipitation, evaporation and transpiration. In the arid or semi arid regions, the ground water table decreases slowly with the time because of high upward flux (evaporation and transpiration) during dry season (see
Figure 2.4). A large portion of the world population (about 60 percent of the world population) is found in the arid regions (Dregne, 1976, and Fredlund, 1976). Most of the civil engineering structures are located on or above the ground water table (within the vadose zone). Because of the climatic and environmental changes, the upward and downward flux (i.e., evaporation, transpiration and precipitation) can make change the movement of moisture up and down in a cycle. This fluctuation of pore water pressure distribution can result in shrinking and swelling of the soil profile over the years. If moisture is extracted from the ground surface (by evaporation or transpiration), the pore water pressure is drawn to the left side of pore water profile as shown in Figure 2.5. Similarly, if moisture enters the ground surface in the form of precipitation, the pore water profile will move to the right side in the surface flux boundary condition as shown in Figure 2.5. This concludes that the upward flux results a gradual drying, cracking and desiccation of the soil mass, whereas downward flux saturates the soil mass. From this discussion (Figure 2.5), it is clear that during the dry periods, the pore water pressure becomes more negative while in the wetting periods, the pore water pressure becomes less negative. The rate of water loss from vadose zone depends upon the permeability of the soil; higher the permeability faster the rate of drying or wetting which results in variation of negative pore water pressure over the time.
2.2 Soil Suction

Soil suction is commonly referred to as the free energy state of soil water (Edlefsen and Anderson, 1943) which can be measured in terms of its partial vapor pressure. Soil suction can be used to evaluate the capability of a soil to attain or hold water. When water enters into unsaturated soils, a part of it is absorbed and stored by the soil. The relationship between total soil suction and the partial pore water vapor pressure is as described by Kelvin’s equation (Spostito, 1981) (Equation 2.1):

\[
 h = -\frac{RT}{v_w \omega_v} \ln \left( \frac{u_v}{u_{v0}} \right) \quad \text{Equation 2.1}
\]

Where,

\( h \) = Soil suction (kPa)

\( R \) = Universal gas constant
T = Absolute temperature (K)

\( \omega_v \) = Molecular mass of water vapor (g/mol)

\( v_{w0} \) = Specific volume of water; inverse of the density of water (m³/kg)

\( u_v \) = Partial pressure of pore water vapor (kPa)

\( u_{v\theta} \) = Saturation pressure of water vapor (kPa)

The term \((u_v/u_{v\theta})\) is called relative humidity, RH (%)

RH can be used to calculate the free energy per unit mass of solution:

\[
E = \frac{u}{\omega_v} = -\frac{RT}{\omega_v} \ln \frac{u_v}{u_{v\theta}} = \frac{RT}{\omega_v} \ln(\text{RH})
\] 

The Equation (2.2) is the free energy per unit mass for a given relative humidity.

![Figure 2.6: Relationship between total suction and RH](image)

2.2.1 Component of Suction: Matric Suction and Osmotic Suction

Total suction has two components: matric suction \((u_a-u_w)\) and osmotic suction \((\pi)\)

\[
h = (u_a - u_w) + \pi
\] 

Where,

\( h \) = total suction,

\( u_a \) = pore air pressure,
\( u_w \) = pore water pressure and \\
\( \pi \) = osmotic suction.

In Equation 2.3, the first term \((u_a-u_w)\) is called matric suction and the second term \(\pi\) is called osmotic suction. The variation of total suction is caused by change in relative humidity (RH) in the soil (Figure 2.6). RH in the soil can vary depending upon the capillary phenomenon (Fredlund and Rahardjo, 1993). Because of the surface tension effect, pore air pressure is generally higher than the pore water pressure (Sharma, 1998). Matric suction is the negative value of difference between pore air pressure \((u_a)\) and pore water pressure \((u_w)\) which is mainly due to the capillary effect across the air-water interface. Matric suction also depends on the curvature of the air-water interface (see Equation 2.4), i.e.

\[
\begin{align*}
  u_a - u_w &= T \left( \frac{1}{r_1} \frac{1}{r_2} \right) \\
\end{align*}
\]

Where,

\( T \) = the surface tension (kN/m)

\( r_1 \) = Radius of air bubble (m)

\( r_2 \) = Radius of water curvature of interface (m), both measured on the air side of the interface.

Osmotic suction is the result from the dissolved ionic concentration in the liquid. It is the function of amount of dissolved salts in the pore of water and is expressed in terms of pressure. The matric suction is only taken into account as a relevant variable in the study of unsaturated soil, assuming that the ionic concentration of liquid in osmotic suction remains unchanged (Alonso et. al, 1987; Fredlund and Rahardjo, 1993).

**2.2.2 Pore Water Pressure and Degree of Saturation in Unsaturated Soil**

According to Sharma (1998), voids in an unsaturated soil can be either water-filled or air-filled. The shape of the voids in soil can explain the condition for water-filled or air-filled voids. To
explain it in a simple way, the shape of voids in soil mass can be compared to the tube with a minimum cross-sectional radius of r. If the radius of the tube is r, the diameter of voids can be taken as 2r. For a hemispherical section, air-water interface filling a tube of radius r, the Equation 2.4 can be expressed as:

$$u_a - u_w = \frac{2T}{r}$$

From Equation 2.5 it is clear that the matric suction \((u_a-u_w)\) is inversely proportional to the radius of the tube.

![Figure 2.7: Air water interface within a soil voids (Sharma, 1998)](image)

According to Wheeler and Karube (1996), pore water in unsaturated soils is categorized into three forms: bulk water, adsorbed water, and meniscus water. Bulk water is the water which is occurred in the completely filled of void space by water. The adsorbed water is tightly holds by the soil particles. The meniscus water occurs at contact of soil particles that does not included in the bulk water. The fundamental influences of pore water and pore air pressure on the behavior of an unsaturated soil depends on the amounts of water in bulk and meniscus forms (Sharma, 1998).

### 2.2.3 Suction Measurement

Soil suction plays an important role in the behavior of unsaturated soils. There are number of devices and techniques available to measure the matric suction. Among the various methods,
filter paper method, thermal conductivity sensors, tensiometers and thermocouple psychrometers are the most popular and widely used methods of measurement of suction in the practice.

2.2.4 Role of Surface Tension and Capillary Rise in Soils Suction

Surface tension is one of the most important properties that affect the matric suction. To explain the behavior of unsaturated soils, Fredlund and Rahardjo (1993) recognized the air-water interface, contractile skin (as explained earlier), as an additional phase, that acts as a stretched membrane between the air and water phases. Air water interface (contractile skin) possesses a property called surface tension, which is due to the inter-molecular forces.

![Figure 2.8: Surface tension effects on soil mass](image)

The inter-molecular attractive force due to cohesion between liquid molecules is responsible for surface tension. The cohesive force at the interior of the soil mass between molecules is shared with all neighboring atoms in all direction (See Figure 2.8). The molecules on the top surface, in Figure 2.8, there are no atoms above the top surfaces; stronger attractive force is experienced on the surface, which is called surface tension.
Figure 2.9: Surface tension in the contractile skin (Fredlund and Rahardjo, 1993)

Because of the surface tension force, the contractile skin behaves like an elastic membrane as shown in Figure 2.9. This behavior can be compared to the inflated balloon which has more pressure to the inside of the balloon than the outside. The pressure difference \((u_a-u_w)\) cause the contractile skin to curve according to Kelvin’s capillary model equation presented in Equation 2.5. According to this equation, as the matric suction increases, radius of contractile skin decreases and when matric suction is zero, the radius of curvature goes to infinity (i.e. flat air water interface exist).

Surface tension is measured in dyne /cm, which means that the force of one dyne requires breaking a film of 1 cm length. Surface tension depends upon temperature (Figure 2.10). At 20°C, water has surface tension 72.8 dyne/cm. Similarly, ethyl alcohol and mercury has 22.3 and 465 dyne /cm respectively.

Surface tension is the important parameter in analyzing the contractile skin in unsaturated soils. This phenomenon can be explained by using the concept of capillary process.
2.2.5 Capillary Rise and Moisture Content in unsaturated soil

The intermolecular attractive force between like molecules is called cohesion, and the intermolecular force between unlike molecules is called adhesion. The adhesive force between water molecules and the wall of a glass tube (as shown in Figure 2.11) are higher than the cohesive force between the molecules of water. This leads to an upward bending of meniscus at the wall of vessel, known as capillary action. Water in the fine pore space will rise above the water table because of the capillary action. Smaller the capillary tube, greater will be the raised height of water, which is measured in terms of suction pressure (negative pressure). Suction pressure is more negative in smaller capillary. The concept of capillary rise in capillary tube can be applied to the unsaturated soil. In an unsaturated soil, air pressure $u_a$ is usually greater than water pressure $u_w$ at the contractile skin as explained by Sharma, 1998. The non uniformity of capillary rise can be seen when a dry column of sandy soil is placed in contact with water as shown in Figure 2.11 and Figure 2.12.

![Figure 2.10: Surface tension- temperature effect](image-url)
Figure 2.11: Rise of capillary tube and pressure distribution

Figure 2.12: Capillary rise on sand column and degree of saturation

Figure 2.12 is the capillary rise of water in sand column with variation of degree of saturation. Figure 2.13 is the capillary rise of water in unsaturated soil zone representing the capillary tube of Figure 2.12. If we monitor the variation of degree of saturation with the height of the soil column caused by capillary rise for a given amount of time, we will obtain the graph of plot as
shown in the Figure 2.12 and 2.13.

**Figure 2. 13: Capillary rise in unsaturated soil**

The degree of saturation is about 100 percent up to the height of \( h_a \). Beyond the height \( h_a \), water occupies only smaller voids and air voids goes increasing if we move upward. During this process, the corresponding degree of saturation goes decreasing, which is less than 100 percent. Hazen (1930) proposed a relationship between height of capillary rise and the effective grain size in Equation 2.6 and 2.7.

\[
hc = \frac{4T}{d \gamma w} \tag{2.6}
\]

\[
h1 = \frac{C}{eD_{10}} \tag{2.7}
\]

Where \( D_{10} \) = effective size (mm)

\( e = \) void ratio

\( C = \) a constant (varies from 10-50 mm²)

### 2.3 Soil Water Characteristics Curve (SWCC)

Soil water characteristics curve (SWCC) is the relationship between water content and suction of soil. It is also called as water retention curve, which means how much water a soil can hold at a
given suction. The degree of saturation ($S_r$) or gravimetric water content ($w$) or volumetric water content ($\theta$) is used to define the soil water characteristics curve. The relation between volumetric water content $\theta$, gravimetric water content $w$, and degree of saturation $S_r$ is given by:

$$\theta (1 + e) = S_r e = w G_s$$ .................................................................2.8

Where, $G_s$ is the specific gravity of soil.

SWCC of a soil plays an important role in defining the hydro mechanical behavior of an unsaturated soil. The study has shown that SWCC is the central relationship describing how soil behaves when it saturates and de-saturates. The shape of SWCC depends upon the type of soil and grain size distribution, void ratio of the soil. Figure 2.14 is the SWCC for different type of soil proposed by Fredlund (1994).

![Figure 2.14: SWCC for various soils (Fredlund, 1994)](image)

SWCC can be used to estimate unsaturated soil property function such as hydraulic conductivity, water storage, shear strength functions, chemical diffusivity, volumetric water content, specific heat and thermal conductivity etc (Fredlund, Wilson, 1993). The general features of water retention curve is as shown in Figure 2.14, in which the volumetric water content, $\theta$, is plotted against the matric suction ($\Psi$). At suction close to zero, a soil is close to saturation (100 % saturation). As volumetric water content ($\theta$) decreases, binding of the water becomes stronger and suction is increased to a great amount with decreasing water content.
2.3.1 Stages of SWCC

The plots of SWCC is divided into three different stages (zones) depending upon the degree of saturation as shown in the Figure 2.15.

If we compare the capillary rise in soil column (Figure 2.12) with Figure 2.15, we will notice three different zone such as capillary zone, de-saturation zone and residual saturation zone which are explained below:

- **Capillary Saturation Zone**: In an unsaturated soil zone, just above the water table, pore water pressure is negative and the soil is in saturated state due to capillary fringe (as explained in section 2.2.2). The suction pressure corresponding to 100 percent degree of saturation is referred as air entry value of particular soil. At this point, the applied suction is higher than the capillary forces and the air starts to enter the soil pores. The air entry value of soil depends upon the soil type and grain size. Various soils have different air entry values.
entry value (see Figure 2.14).

- **De-saturation Zone:** As the degree of saturation continues decreasing, suction increases rapidly, water void in the soil is displaced by the air voids. This process continues until the pore water becomes occluded.

- **Residual Saturation Zone:** In this zone the water is tightly adsorbed into the soil particles. The soil is almost in a dry condition and the degree of saturation is assumed almost to be zero.

### 2.3.2 Measurement of SWCC

Various devices have been developed to measure the SWCC for a given soil. Among them, Fredlund’s device is widely used in practice these days. Figure 2.16 is the typical sketch if Fredlund device used in this research. The detail description for measurement of SWCC is presented in chapter three.

![Figure 2.16: Typical sketch of SWCC device](image)

### 2.4 Relationship among Permeability, Degree of Saturation, and Matric Suction

Fredlund and Rahardjo, (1993) identified that there are two fluid phases in the pore of
unsaturated soil i.e., air and water. At higher degree of saturation, air is in occluded form. At lower degree of saturation, movement of air through the water phase is possible, which is called as diffusion of air through the pore water. Because of the diffusivity of air in to the water, it creates an additional problem in measurement the water content on soil mass. The flow of water in a saturated soil can be described by Darcy’s law, according to which the rate of water flow through soil mass is directly proportional to the hydraulic gradient. Mathematically,

\[ q = k \cdot i \] \hspace{1cm} \text{2.9}

Where,

- \( q \) = flow rate of water
- \( k \) = coefficient of permeability with respect to water phase
- \( i \) = hydraulic head gradient \( (= \frac{\partial h}{\partial y}) \)

Darcy’s law is also holds good for unsaturated soil (Buckingham, 1907; Richards, 1931; Childs and Collis-George, 1950). In a saturated soil, the coefficient of permeability is a function of void ratio (Lambe and Whitman, 1979), however the coefficient of permeability is relatively assumed to be constant when analyzing the problems. In unsaturated soil, the coefficient of permeability depends upon void ratio and degree of saturation of soil. If the change in void ratio in an unsaturated soil is assumed to be small, its effect on the coefficient of permeability may be small, but the effect of change in degree of saturation may be highly significant. So, the coefficient of permeability is mainly described as a function of degree of saturation \( S \), or the volumetric water content \( \theta \).

From the previous section, it is known that the degree of saturation or water content of soil is significantly affected by the change in matric suction. Therefore, the degree of saturation is described as a function of matric suction (see Figure 2.17).
Various semi-empirical equations for the coefficient of permeability have been derived using matric suction versus degree of saturation or soil water characteristic curve (SWCC). Prediction of coefficient of permeability from matric suction versus degree of saturation is described here first, and then, the prediction of coefficient of permeability based on grain size distribution is discussed later.

![Drying curve of SWCC](image)

**Figure 2.17: A Typical sketch for degree of saturation vs. matric suction**

For the first time, coefficient of permeability function based on matric suction and degree of saturation is proposed by Burdine (1952) which is then followed by Brooks and Corey (1964). Based on the relationship between matric suction and degree of saturation, various soils parameters such as air entry value \((u_a-u_w)_b\), residual degree of saturation \((Sr)\), and pore size distribution index \((\lambda)\) are defined. Corey (1954) expressed the effective degree of saturation \((Se)\):

\[
S_e = \frac{S - S_r}{1 - S_r} \\
\text{…………………………………………………………………………………………...2.10}
\]

Where, \(S_e\)=effective degree of saturation

\(S_r\)= residual degree of saturation.

The residual degree of saturation is the degree of saturation at which an increase in matric
suction \((u_a-u_w)\) does not produce any significant change in the degree of saturation. The effective degree of saturation is then calculated by estimating the residual degree of saturation \((Sr)\) and is plotted against the matric suction \((u_a-u_w)\) as shown in Figure 2.18. Air entry value of the soil \((u_a-u_w)\) is the matric suction value from which air starts to enter into the soil, which is also referred to as bubbling pressure (Corey, 1977), from which the maximum pore size in a soil specimen can be measured or estimated.

\[
s_e = \frac{s - s_r}{1 - s_r}
\]

Where, \(\lambda\) = pore size distribution index, the value of \(\lambda\) depends upon the pore size distribution of soil. Larger the range of pore size distribution, smaller will be the value of \(\lambda\).

The relationship between coefficients of permeability with respect to the water phase, \(K_w\), is
obtained as a function of matric suction by substituting the effective degree of saturation \((S_e)\), in the permeability function given by Brooks and Corey, (1964). Several relationships between coefficient of permeability and matric suction have been proposed by several researchers: Gardner, (1958a), Arbhahirama and Kridakorn, (1968), which are summarized in the Table 2.1

<table>
<thead>
<tr>
<th>Equations</th>
<th>Source/ Researcher</th>
<th>Symbol</th>
</tr>
</thead>
<tbody>
<tr>
<td>(k_w = k_s) For ((u_a-u_w) \leq (u_s-u_w)b) [kw = k_s\left(\frac{u_a - u_w}{u_a - u_w}b\right)^\eta]</td>
<td>Brooks and Corey (1964)</td>
<td>(\eta = \text{empirical constant}) (\eta = 2+3\lambda)</td>
</tr>
<tr>
<td>(k_w = \frac{ks}{1 + a\left(\frac{u_a - u_w}{p_w g}\right)^n}) For ((u_a-u_w) &gt; (u_s-u_w)b)</td>
<td>Gardner(1958a)</td>
<td>(a,n = \text{constant})</td>
</tr>
<tr>
<td>(k_w = \frac{ks}{\left(\frac{u_a - u_w}{u_a - u_w}b\right)^n} + 1)</td>
<td>Arbhahirama, Kridakorn (1968)</td>
<td>(n' = \text{constant})</td>
</tr>
</tbody>
</table>

The relation between coefficient of permeability \((k_w)\) and volumetric water content \((\theta_w)\) is described by Buckingham (1907). Later other researchers such as Richards (1931), and Moore (1939) also proposed the similar relation. Similarly, coefficient of permeability function, \(k_w(\theta_w)\) is proposed by Childs and Collis-George (1950). The volumetric water content \((\theta_w)\) can be plotted as a function of matric suction \((u_a-u_w)\), which is also known as SWCC. So, the permeability function, \(k_w(\theta_w)\) can be expressed in terms of matric suction (Marshall, 1958; Millington and Quirk, 1959, 1961). A theoretical relationship between coefficient of permeability and volumetric water content is also expressed using SWCC (Kunze et al., 1968;
Fredlund and Rahardjo, 1993).

From these review it is clear that the flow of water in soils is a function of volume of water present in soil mass. It is because, the coefficient of permeability, $k_w$, is generally assumed to be related to the degree of saturation, $S$, or the volumetric water content, $\theta_w$.

![Figure 2.19: Hysteresis phenomenon in volumetric water content and matric suction for sand (Fredlund and Rahardjo, 1993)](image)

In Figure 2.19, the relationship between $\theta_w$ or $S$ and $k_w$ appear to exhibit a little hysteresis.

![Figure 2.20: A typical plot of Hysteresis phenomenon in permeability and matric suction (Fredlund and Rahardjo, 1993)](image)

It is observed from the research that the degree of saturation or volumetric water content shows significant hysteresis with matric suction. In case of coarse grained soil, the effect of hysteresis
phenomenon is observed which is less effective than fine grained soil (Nielsen and Biggar, 1961; Topp and Miller, 1966; Corey, 1977; Hillel, 1982). Coefficient of permeability also shows significant hysteresis when plotted against matric suction (see Figure 2.20 and Figure 2.21), this part will not be studied in this research.

But, no significant hysteresis in the relationship between water coefficients of permeability versus volumetric water content found (after Fredlund and Rahardjo, 1993).

### 2.5 Permeability Characteristics Based on Pore Size Distribution

Prediction of SWCC from grain size distribution is proposed by various researchers such as Fredlund and Rahardjo (1993), Fredlund et.al, (1997). Several mathematical models were developed. Typical SWCC and grain size distribution curve for a mixture of sand, silt and clay were obtained over 6000 soils sample, Fredlund (1996). SWCC curves were fitted with Fredlund and Xing (1994) equation. Based on the result from their investigations and from the study of prediction of SWCC based on grain size distribution and index properties, Zapata et. al., (2005)
developed a family of SWCCs as shown in Figure 2.22.

Figure 2.22: Grain size distribution and permeability function (Fredlund, 1997)

2.6 Mathematical Model of SWCC (Fredlund and Xing, 1994)

There are several empirical and numerical model developed to describe the SWCC.

Table 2.2 Summary of the model equations proposed for SWCC (after Fredlund, 2000)

<table>
<thead>
<tr>
<th>Source/Researcher</th>
<th>Model equation</th>
<th>Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gardner (1958)</td>
<td>$w = \frac{w_s}{1 + \left(\frac{\psi}{a_g}\right)^{n_g}}$</td>
<td>$a_g, n_g, \Psi=\text{soil suction}$</td>
</tr>
<tr>
<td>Van Genuchten (1980)</td>
<td>$w = \frac{w_s}{{1 + \left(\frac{\psi}{a_{vg}}\right)^{n_{vg}}}^{m_{vg}}}$</td>
<td>$a_{vg}, n_{vg}, m_{vg}$</td>
</tr>
<tr>
<td>Mualem (1976)</td>
<td>$w = \frac{w_s}{{1 + \left(\frac{\psi}{a_m}\right)^{n_m}}^{m_m}}$</td>
<td>$a_m, n_m, m_m=1/(1-n_m)$</td>
</tr>
<tr>
<td>Burdine (1952)</td>
<td>$w = \frac{w_s}{{1 + \left(\frac{\psi}{a_b}\right)^{n_b}}^{m_b}}$</td>
<td>$a_b, n_b, m_b=2/(1-n_b)$</td>
</tr>
<tr>
<td>Fredlund and Xing (1994)</td>
<td>$w = c(\psi)\left[\ln\left[\left(e + \left(\frac{\psi}{a_f}\right)^{n_f}\right)^{m_f}\right]\right]$</td>
<td>$a_f, n_f, m_f, c(\psi)$</td>
</tr>
</tbody>
</table>
Several researchers including Fredlund (2000) reviewed several mathematical equations. First, Gardener (1958) proposed a mathematical model to describe unsaturated coefficient of permeability function and its application to SWCC. Similarly, Burdine (1952) and Maulem (1976) proposed an equation, which was a special case for van Genuchten (1980). The list of various proposed mathematical model describing SWCC are given in the Table 2.2.

In all types of model, the parameter n is the slope of straight line portion of the main drying part of SWCC. Similarly, a, is a relationship to the air entry value of soil referring to the point of inflection along the curve. The model proposed by Fredlund and Xing (1994) gives the result for an entire range of soil suction from 0 to 1,000,000 kPa.

2.7 Correlation between SWCC Curve and Water Permeability Function Curve

According to Fredlund et. al., (2001b), field and lab methods to measure the relationship between water permeability and suction for unsaturated soil is difficult, tedious, and time consuming job.

Figure 2.23: Water Content and Coefficient of Permeability versus Soil Suction (After Fredlund et al., 2001b)
Various semi empirical correlation and models are developed to measure permeability function from SWCC. The comparison between the shape of water permeability function and SWCC were made in the model developed by Fredlund et al., (1994) for sand and clayey silt. It was concluded that the shape of water permeability function curve matches a relationship with the shape of SWCC as shown in Figure 2.23. From his comparison, it was proposed that water permeability for both soils is found relatively constant from zero suction to air entry value. Similarly, for the both soil, water permeability value is found to be decreased rapidly beyond the air-entry value. They also came to the conclusion that water permeability function of unsaturated soil can be predicted using the knowledge of saturated coefficient of water permeability and SWCC (Fredlund et al., 1994). Similar relationship was established by various researchers such as Leong and Rahardjo (1997), Benson and Gribb (1997).

2.8 Effective Stress Approach for Unsaturated Soil

In the explanation of effective stress principle by Terzaghi (1936) as stated “the stress in any point of a section through a mass of soil can be computed from the total principal stresses, $\sigma_1$, $\sigma_2$ and $\sigma_3$, which act at this point. If the voids of the soil are filled with water under a stress, $u$, the total principal stresses consist of two parts. One part, $u$, acts in the water and in the solid in every direction with equal intensity….All the measurable effects of a change in stress, such as compression, distortion and a change in shearing resistance are exclusively due to change in the effective stress”. Mathematical form of this definition in simple form is:

$$\sigma' = \sigma - u_w$$

Where, $\sigma'$ = effective normal stress,

$\sigma$ = total normal stress, and

$u_w$ = pore water pressure.
Equation 2.12 is the definition for the stress state variable for saturated soil. The mechanical behavior of saturated soil is governed by this effective stress equation. Effective stress is equal to total stress minus pore water pressure. Below the ground water table pore pressure is positive, so effective stress is less than total stress. While in the case of above the ground water table, pore water pressure is negative, and net effective stress is greater than total stress. Following equations from 2.13 to equation 2.18 are the summary of the equations developed by various researchers to determine the effective stress developed for unsaturated soils.

(Croney et al., 1958) \[ \sigma' = \sigma - \beta u_w \] .............................2.13
(Bishop, 1959) \[ \sigma' = (\sigma - u_a) + \chi (u_a - u_w) \] .............................2.14
(Aitcheson, 1961) \[ \sigma' = \sigma + \psi p'' \] .............................2.15
(Aitcheson, 1961) \[ \sigma' = \sigma + \beta p'' \] .............................2.16
(Richards, 1966) \[ \sigma' = (\sigma - u_a) + \chi_m (h_m + u_a) + \chi_s (h_s + u_a) \] 2.17
(Aitcheson, 1973) \[ \sigma' = \sigma + \chi_m p''_m + \chi_s p'' \] .............................2.18

Where,
\[ \sigma = \text{Total normal stress}, \]
\[ \sigma' = \text{Effective normal stress}, \]
\[ u_w = \text{Pore water pressure}, \]
\[ u_a = \text{Pore air pressure}, \]
\[ \chi = \text{Parameter for the degree of saturation of the soil}, \]
\[ \psi = \text{A parameter whose values ranges from zero to one}, \]
\[ p' = \text{Pore water deficiency}, \]
\[ \chi_m = \text{Effective stress parameter for matric suction}, \]
\( \chi_m \) = Effective stress parameter for solute suction,

\( h_m \) = Matric suction,

\( p_m \) = Matric suction,

\( h_s \) = Solute suction,

\( p_s \) = Solute suction,

The study of unsaturated soil behavior is complex because of its pore space are partially filled with water and partially filled with air. The effective stress principle developed for saturated soil cannot be applied directly for unsaturated case because the unsaturated soils are dealt as a three phase material (Lambe and Whitman, 1969). Many researchers such as Bishop (1959), Aitchinson (1961) and Jennings (1961), proposed modified forms of effective stress equation for unsaturated soil to include air and water pressure (\( u_a \) and \( u_w \)). The most widely accepted effective stress equation for unsaturated soil was proposed by Bishop (1959) which is given by:

\[
\sigma' = (\sigma - u_a) + \chi (u_a - u_w)
\]

Where, \( \chi \) = a parameter which is a function of degree of saturations

\( \sigma \) = total stress

\( \sigma' \) = effective stress

The magnitude of the parameter \( \chi \) varies between zero for a dry soil and one for a saturated soil.

Coleman (1962) suggested the use of reduced stress variables (\( \sigma_1 - u_a \), (\( \sigma_3 - u_a \)) and (\( u_a - u_w \)) to represent the axial, confining and pore water pressure respectively in triaxial test for unsaturated soil. Bishop and Blight (1963), re-evaluated the use of single valued effective stress equation. They also observed that the change in matric suction (\( u_a - u_w \)) did not always result in the same change in effective stress. Instead, they suggested using independent stress variables (\( \sigma_1 - u_a \)) and (\( u_a - u_w \)) to monitor the volume change behavior of unsaturated soils. This approach of using
independent stress variable was further used by Blight (1965) and Burland (1964, 1965).

2.9 Stress State Variables for Unsaturated Soil

According to Fung (1977), stress variables used for the description of a stress state should be independent of soil properties. This means, two independent stress state variables should be used for dealing with unsaturated soils. While doing so, the effective stress equation was separated into two independent stress variables: \((\sigma-u_a)\) and \((u_a-u_w)\) to describe the mechanical behavior of unsaturated soils.

Further investigations conducted by Jennings and Burland (1962), Matyas and Radhakrishna (1968), Fredlund and Morgenstern (1977), Alonso et al (1987), and Wheeler and Karube (1996) also suggested that single effective stress approach for unsaturated soil does not work and this approach does not explain the volume change and mechanical behavior for unsaturated soils. Later, Bishop and Blight (1963), Blight (1967), and Burland (1964) suggested using the two independent stress variable such as net stress \((\sigma-u_a)\) and matric suction \((u_a-u_w)\) to describe the hydro-mechanical behavior of an unsaturated soil. Further research on volume change behavior of unsaturated soils was conducted by Aitchison and Woodburn (1969), Matyas and Radhakrishna (1968), Barden et al. (1969), and Brackely (1971). They also suggested using the independent stress state variables for unsaturated soil with more clear and justified explanation of unsaturated soil behavior.

Several researchers have suggested that any two out of three possible stress variables namely \((\sigma-u_a)\), \((\sigma-u_w)\) and \((u_a-u_w)\) corresponding to mean net stress, effective stress and matric suction respectively can be used to describe the stress state of an unsaturated soil.

Fredlund and Morgenstern (1977) suggested that any two of the three stress parameters would be sufficient to describe fully the stress state of an unsaturated soil. The possible combinations of
stress state variables can be: \((\sigma - u_a)\) and \((u_a - u_w)\); \((\sigma - u_w)\) and \((u_a - u_w)\); and \((\sigma - u_a)\) and \((\sigma - u_w)\). The parameter \((\sigma - u_a)\) and \((\sigma - u_w)\) are tensor quantities while the third parameter \((u_a - u_w)\) is a scalar quantity, (Sharma, 1998). It is good idea to adopted mean net stress and matric suction as the appropriate stress variables for dealing with unsaturated soil. Similarly, Fredlund and Rahardjo (1993) suggested to use of mean net stress \((\sigma - u_a)\) and matric suction \((u_a - u_w)\) to describe the mechanical behavior of unsaturated soils.

2.10 Shear Modulus and Soil Stiffness

Soil stiffness is a measure of the deformation of a soil over a period of time for an applied load. Shear modulus \(G\) is defined in the Equation 2.20. It is measured in force per unit area \((KN/m^2)\). The stiffness \(E\) is defined as the ratio of the force per unit displacement. It has units of force per unit length \((KN/m)\). The relationship between the modulus and the stiffness for a circular plate with diameter \(B\) can be defined by the Equation 2.20 given by J.L. Briaud, (2001):

\[ G = f \left( \frac{E}{B} \right) \]

Where,

\(G\) is the shear modulus, \(E\) is the stiffness of soil, and \(B\) is the diameter of loading area (base plate) where soil is subjected to deformation. Stiffness is not the properties of soil but depends upon the size of the loaded area whereas shear modulus is the properties of soil (Brianu, 2001).

For example, the stiffness of the elastic material measured with one test will be different for the second test on same material with different loading area. But, the modulus will be same for both the tests on the same material.

Estimating the shear modulus of soil is one of the most difficult parameters because it depends on so many factor such as water content, stress history, particle structure, loading factors, strain level, stress level, etc (Briaud, 2001). The study has shown that the stiffness of soil is highly
non-linear at very small strains and its determination is highly critical in evaluating the strength and deformation of the soil (Atkinson, 2000). The modulus and stiffness are useful in many fields of geotechnical engineering such as settlements of embankments; foundation, tunnels, and movements along the front and back of retaining walls. In this research, using elastic wave propagation, bender elements and shear wave equipment are used to generate and monitor elastic waves to measure the stiffness. In this study, determination of stiffness of soil is presented in a drying curve of SWCC using shear wave velocity.

2.10.1 Determination of Shear Wave Velocity and Shear Modulus Using Bender Elements

In classical method of determining soil stiffness, triaxial and resonant column tests using high precision strain gauges are used. These tests are more tedious and require high skill to perform strains smaller than 0.001%. Recently developed stiffness gauges are most suited to determine stiffness of soils in-situ. To measure the small strain stiffness ($G_{\text{max}}$) of soil at strains lower than 0.0001% in the laboratory, piezo-ceramic elements are widely used, which is based on the principle of wave propagation in soil.

2.10.2 Wave Propagation in Soils

There are two types of elastic waves: compression waves (P wave), propagating along the longitudinal direction of motion in the medium and shear waves (S wave) propagating perpendicular to the direction of motion in the medium. Usually, P-waves propagate faster than the S-wave, because P- waves have smaller wave lengths and higher frequencies. In general, wave produces small disturbance in the soil and corresponding small changes in their strain levels is measured. In this study, the shear stiffness of soil is analyzed using S wave, in which velocity of propagating wave is calculated using the relation

$$V_s = \sqrt{\frac{G_{\text{max}}}{\rho}}.$$
Where, $V_s$ is the Shear wave velocity, which is the function of the small strain shear modulus ($G_{\text{max}}$), and the density ($\rho$) of soil. Shear wave velocity increases with increase in stresses and particle orientation (denser) and decreases with degree of saturation ($S_r$) as presented by Fratta et al, 2001.

$$V_s = \frac{L}{t}$$ ..............................................................2.22

Where, $V_s$ is the wave velocity, $L$ is the distance between the tips of source and receiver bender elements and $t$ is the travel time. The elastic shear modulus $G_{\text{max}}$ can be determined as,

$$G_{\text{max}} = \rho V_s^2$$ ..............................................................2.23

Where, $\rho$ is the soil mass density and $V$ is the shear wave velocity.

In this research study, the difference in the peak to peak time interval is used to determine the small strain stiffness ($G_{\text{max}}$) of the soil.

### 2.10.3 Factors Affecting Stiffness of Unsaturated soils

Various factors affecting stiffness of unsaturated soil are summarized below which was presented in the literature by Briaud, 2001:

**Stress state**

- The stiffness of unsaturated soil is affected by the stress state, which means how closely the particles are packed. If they are closely packed, the modulus tends to be high, which is also measured by the dry density of the soil.

- Stiffness depends upon the structural organization of particles (as discussed above). This means, how the particles are organized in terms of structural orientation of the soil. It is important to note that two soil samples can have the same dry density but different structures and hence different soil moduli (Briaud, 2001). Higher the dry density, higher is the modulus.
According to Briaud (2001), stiffness of soils depends upon the moisture content of the soil. At low water contents the water binds the particles (especially for fine grained soils) and increases the effective stress between the particles (it is because through the suction and tensile skin of water phenomenon). So, low water contents increases soil moduli. For example, clay shrinks and becomes very stiff when it dries. But, for coarse grained soil, at very low water contents, the compaction is difficult to attain its maximum density. Therefore according to Briaud (2001), in the case of coarse grained soil, very low water contents lead to low moduli. In such cases, modulus increases with the increase of its water content and at the same time the effect of compaction also increases. But, if the water content rises beyond optimum moisture content, modulus decreases (Briaud, 2001).

Many researchers have reported that stiffness depends upon the stress history. Over-consolidated soils have higher moduli than the normally consolidated (NC) soil. Under-consolidated soils have very low moduli (for example, the clay deposited offshore the Mississippi Delta) (Briaud, 2001).

Figure 2.24: Shear stiffness of R1 sand vs. degree of saturation (Sr) (Qian et al., 1993)
**Matric Suction**

According to Briaud, 2001, shear velocity increases with the increase of matric suction. The influence of matric suction on the shear modulus is one of the important characteristics. The water present in the contractile skin exerts negative pore pressure on the particles, thus increasing the shear strength. Shear wave velocity and shear modulus were examined by various researchers including Snachez-Salinero et al., (1986). In this research, the transmitted and received signals of propagating shear wave were captured in the oscilloscope as suggested by Viggiani and Atkinson (1995). Shear wave velocity and shear modulus were calculated using equations 2.22 and equation 2.23. From the research it was observed that shear wave velocity increases with the increase of matric suction as proposed by Briaud (2001). Similar results were reported in the observation of suction vs. initial shear stiffness by Mancuso et al., (2002) (see Figure 2.25). The Equation proposed by Rampello et. al., (1995) is given by,

\[
\frac{G_0}{p_a} = A\left(\frac{p'}{p_a}\right)^n OCR^m
\]

\[\text{.............................................................2.24}\]

**Figure 2.25: Influence of suction to the shear stiffness (Mancuso et al. 2002)**

Figure 2.25 shows the influence of stiffness to increasing suction. Similar results were also reported by Marinho et al., (1995) for clay using bender elements and filter paper for specimens
subjected to drying. The results from the test have observed the similar pattern to the result of saturated soils by Cabarkapa et al., (1999), which was performed in a modified triaxial cell for unsaturated soil in isotropic loading condition at controlled suction.
CHAPTER THREE: EXPERIMENTAL SETUP AND SAMPLE PREPARATION

3.1 Material Properties

3.1.1 Introduction

The soils used for this research were Sand Bentonite mixed soil (5% bentonite, and 2% bentonite, referred here as SB) and dark brown low plasticity silty clay (CL) (referred as CL). All materials SB (2% and 5%) and CL were investigated in the lab to evaluate various parameters such as index properties, saturated hydraulic conductivity, initial moisture content, specific gravity, grain size distribution etc., which were then used in HYDRUS for simulation as initial input data. Similarly, these soils were further tested in the lab to investigate the impact of moisture content (or degree of saturation) to the soil suction, which is also called as soil water characteristics curve (SWCC). The influence of degree of saturation on stiffness, the influence of suction on stiffness and the influence of density on stiffness were also investigated. The results from lab tests were compared with the result from the HYDRUS simulation.

The material, SB, was selected for the lab test because it is widely used for varieties of geotechnical engineering purpose. For example, it is used in many sealing purposes such as excavation pits, retaining walls, landfill contaminant, vertical barriers, ground water control, seepage barrier, tunnel boring works in loose soil and below ground water table, ground water protection, slurry wall during pile-driving, horizontal landfill liners, vertical cut-off walls, tunneling, grouting works, pipe jacking caisson sinking, dam, dykes, river barrages, mineral liner, generation of lateral pressures etc. Sand Bentonite mixed soil (SB), which is used in such application area, is subjected to continuous drying and wetting cycle as water table fluctuates due to various environmental and climatic variations such as evaporation, precipitation and transpiration. So, it is necessary to observe and monitor the effect of variation of moisture
content on hydro mechanical behavior of soil. For this research, two sample were prepared with fine mortar sand containing 5% and 2% of Bentonite to achieve hydraulic conductivity of range $2 \times 10^{-10}$ m/s, which is the standard value adopted mostly in contamination barrier purpose (Ref. Koch, 1989).

### 3.1.2 Properties of Sand Bentonite Mixed Soil (SB)

Bentonites are special type of clay which falls on the category of montomorillonite. Bentonite deposit has two forms, Na-montomorillonite or Ca-montmorillonite or both. They have cat-ion exchange capacity; the swelling behavior due to adsorption of water molecules at interlayer cations and at mineral surface. Sodium bentonite is referred to as swelling clay (Koch. D., 2002), which has single water layer particles containing Na$^+$ as the exchangeable ion. Bentonite has excellent water absorption capacity, which is much higher than ordinary clays. When the sodium bentonite gets saturated, its volume increases approximately 14 times greater than that of its original volume. The sodium bentonite is commercially available in the market in the name of “Natural Gel”.

### 3.1.3 Specific Gravity

Specific gravity is unit less quantity which is defined as a ratio of the mass of unit volume of soil solids to the mass of same volume of water at 20$^\circ$C. Specific gravity of soil solids is used to in geotechnical engineering calculations such as to calculate the phase relationship of soils, degree of saturation etc. Specific gravity of SB (5%) and SB (2%), and CL were calculated in the lab according to ASTM D854 standards, which was found to be 2.68, 2.65 and 2.7 respectively.

### 3.1.4 Particle Size Distribution

Prain-size distribution tests were performed according to ASTM D 422 and ASTM D2487-06 for sieve analysis and hydrometer test respectively. Sieve analysis is used for sand and gravel,
particle sizes greater than 0.075 mm (retained on the No. 200 sieve) and smaller than particle sizes 4.75 mm (Passing through No. 4 sieve). Similarly, the hydrometer analysis is used for particles smaller than 0.075mm (silt and clay), which is based on the principle of sedimentation process. The particle size distribution of sample SB-5%, SB-2% and CL are given in Figure 3.1.

![Particle Size Distribution](image)

**Figure 3.1: Grain-size distribution of Sand Bentonite mixed soil (SB) and CL**

From the particle size distribution curve, Figure 3.1, the coefficient of uniformity ($C_u$) and coefficients of curvature ($C_c$) for soil SB (both 5% and 2%) were calculated as 3.0 and 1.68 respectively. Similarly, the values of $C_u$ and $C_c$ for soil CL were calculated as 64.28 and 2.43 respectively.

According to unified soil classification system, and Cassagrande plasticity chart, the soil was classified as Low Plasticity Silty Clay (CL).

### 3.1.5 Moisture Content

Initial moisture content of SB was carried out using ASTM D 2216-05. Materials were kept in the oven at a constant temperature of 110°C for a period of 24 hours. Volumetric water content of SB 5%, SB 2% and CL were found to be as 0.49, 0.48 and 0.5 cc/cc respectively.
3.1.6 Atterberg Limit
Determination of the liquid limit, plastic limit, and the plasticity index of soils were performed according to test methods given in ASTM D 4318-05. Sand Bentonite mixed soil (SB) is non-plastic material. The liquid limit, plastic limit and plasticity index value of CL were obtained as 49, 29.7, and 19.3 respectively.

3.1.7 Direct shear test
Direct shear tests were performed according to the standard ASTM D 3080 for both sample SB (5% and 2%), and CL. Each sample was sheared for three normal stresses: 1000 psf, 2000 psf and 3000 psf. The friction angle and cohesion for SB (5% and 2%) were obtained as 40°, 42° and 64 psf, and 32 psf respectively. Figure 3.2 and Figure 3.3 is the plot of shear stress vs normal stress for SB.

![Direct Shear Test (5%SB)](image)

**Figure 3.2: Direct shear test for SB (5%)**
Similarly, from the direct shear test for soil CL, the angle of friction was found to be 30 degree and the value of cohesion was found to be 350 psf, the plot of which is shown in Figure 3.4.

3.2 Permeability Test

The permeability test for sample SB and CL is done using falling head permeameter in accordance with the ASTM D 5084-03.
Figure 3.5: Permeability test using falling head permeameter

Figure 3.5 is the general arrangement of falling head permeability test setup. The average coefficient of permeability for sample SB and CL were found to be 2.06E-10 m/sec (5% Bentonite), 1.7E-09 m/sec (2% Bentonite), and 1.6E-7 m/sec for CL sample. Several factors such as grain size distribution, stress history and water content etc affects the permeability of soil. In general speaking, fine grained soils will have lower permeability than coarse grained soils. So, the permeability is in descending order of grain size: Gravel (high permeability), Sand, Silt, Clay, Shale (low permeability).

3.3 Soil Water Characteristics Curve (SWCC)

3.3.1 Methods of Determining SWCC in the Lab

SWCC for a particular soil can be evaluated in the lab using several methods such as volumetric plate extractor, triaxial test, resonant column device etc. The correlation between SWCC and particle size distribution curve can be done to determine the SWCC of particular soil (Zapata, 2005). In this research work, Fredlund SWCC device was used to determine the relationship between degree of saturation or volumetric water content versus suction, which is SWCC. Figure 3.6 is the line diagram of SWCC device developed by Fredlund.
3.3.2 Sample Preparation

A series of laboratory tests were conducted to determine the soil water characteristic curve (SWCC) for soil SB (5% and 2%), and CL. The method for preparation of sample for SB and CL are given in the following section.

- **Sand Bentonite Mixed Soil (SB)**

  Sample is prepared by taking a sand- Bentonite mixed soil (5% and 2% Bentonite, with initial volumetric water content 0.49 and 0.48) in a consolidation steel ring which is then directly placed in a SWCC device for the test. No special method of sample preparation is done for SB soil.

- **Low Plasticity Silty Clay (CL)**

  CL soil was compacted to the standard proctor compaction test at its optimum water content. The specimen was trimmed in to a metallic ring (which is used for consolidation) of size 2.5 inches x
2.1 inches. The specimen was immersed in de-aired water container, after placing two filter papers on top and bottom of the sample, which is then kept in porous stone on both side (top and bottom). It is important to note that the water level should be just below the top of the specimen (about 2mm) so that the entrapped air present inside the void of the specimen could be released during the saturation process. A small weight (say about 50 gm) was placed on top of the specimen so as to increase the water content of the specimen. The specimen was kept in the water for about 24 hours for complete saturation and ready for the test (see Figure 3.7).

![Diagram of specimen saturation process]

**Figure 3.7: Soil specimen saturation process**

Specimen were then tested in the Fredlund SWCC device, which is a simple unsaturated soil testing apparatus for applying matric suctions ranging from nearly zero to 1500 kPa (i.e., 15 bars).

### 3.3.3 Saturation and Mounting of High Air –Entry Ceramic Stone

To achieve the specified air entry value of the ceramic stone, it should be fully saturated before doing SWCC test. The saturation of the high air entry value (500kPa) of ceramic stone was performed using the standard procedure proposed by Fredlund and Rajardjo (1993), and Shivakumar (1993). Ceramic stone was assembled in a metal casing ring using epoxy coated all
around so that a good adhesion can be developed. The stone is then assembled in Fredlund’s SWCC apparatus, water was filled on top of the stone. Next, the cell was closed without any specimen inside on it. It was then subjected to air pressure of less than the air entry value of stone (500 kPa) and water was allowed to pass through it. The rate of water coming out through the stone in the graduated tube was recorded for each hour. The graph is obtained between volume of water coming out and time of measurement in hours as in the Figure 3.8.

![Graph showing Saturation of Ceramic Stone](image)

**Figure 3. 8: Saturation of Ceramic stone**

When the rate of out flow of water reaches to a constant level, as in the Figure 3.9, this shows that the stone is fully saturated and is ready to use for the SWCC.

### 3.3.4 Features of SWCC Device

The Fredlund SWCC Device, used in this research, is a simple unsaturated soil testing apparatus for applying matric suction from nearly zero up to 1500 kPa. This device has various features such as:

- Application of vertical pressure is possible
- Tracking of overall volume changes can be done
• Application of suction up to 1500 kPa (i.e., 15 bars) is possible
• Both drying and wetting curves can be measured
• Dual pressure gauges and regulators for precise pressure control
• Diffused air can be measured and flushed
• Several different high-air-entry-values (HAEV) ceramic stones at 100, 300, 500, and 1500 kPa can be easily applied

Besides such above mentioned features, the device has a more flexibility to use in variable soil condition. For example, the suitable ceramic stone can be selected depending on the soil type to be tested. For this research purpose, 5 Pa (500 kPa) ceramic stone was used. Following is the view of SWCC device used in the lab (Figure 3.9). For each set of pressure increment, water released or absorbed by the soil specimen is measured in the volume tube readings. For soil specimen SB and CL, only the drying test was carried out in this research.

![Figure 3.9: SWCC device](image-url)
3.3.5 Determination of Drying SWCC

After the saturation of specimen and high air entry ceramic stone, (5 bar for this purpose), the saturated surface dry (SSD) weight were measured for both stone and specimen. The SWCC cell was then covered from the top of the device using clean O-ring and high vacuum grease in the grooves of the plate. A quick-connector fitting were inserted into the water volume measuring tubes, which are located on the sides of the bottom plate that holds the two tubes. The two valves of water volume measuring tube were kept opened which is located at the bottom. Ceramic stone was mounted on SWCC device by pressing into the bottom plate of the device. After that, specimen was placed at the middle of the ceramic stone. Disk ring and cell wall were placed on the base of the system. The top plate and bottom plate were attached by tightening the four 4.5 inch long screws. The air pressure source was connected to the pressure panel.

The left tube was filled with de-mineralized water through the opening on the left corner. Both the tubes were kept less than half full with water, and were allowed to equilibrate for some time until the water level on both measuring tube became equilibrium. The initial tube readings were recorded.

Now the selected suction was applied to provide wide range of volume change. The ranges of suction were tried to maintain constant for different set of tests. The water volume change readings were taken on a log cycle time. At the end of the test, the pressure was released and the specimen was weighted. The dry weight of sample was obtained by keeping it in oven dry condition for 24 hours. The weight of the surface dry ceramic stone was also taken after the test. Following calculations were made at the end of the each test.

- Using dry unit weight, initial water content and dry density was calculated.
- Volumetric water content corresponding to each pressure increment was estimated.
- Final water content of the sample was calculated
- Degree of saturation at each successive suction pressure was estimated assuming a constant void ratio (which may not true all the time).
- The degree of saturation for higher suction rate (i.e., above 500 kPa) were estimated using equation proposed by van Genuchetan (1980).

A typical format for calculation of SWCC is given in Table 3.1. A typical plot of SWCC for SB and CL sample is shown in Figure 3.10 and 3.11.

### Table 3.1 Calculation of SWCC

<table>
<thead>
<tr>
<th>Suction (kPa)</th>
<th>Reading in the left volume tube (cc)</th>
<th>Reading in the right volume tube (cc)</th>
<th>Total reading of the volume tubes (cc)</th>
<th>Driven out water (cc)</th>
<th>Wet of wet bentonite (g)</th>
<th>Total volume of Water, Ww (g or cc)</th>
<th>Volumetric Water Content (cc/cc)</th>
<th>Gravimetric Water content (w = Ww/Ws)</th>
<th>Bulk density (γb = W/V)</th>
<th>Dry density (γd = γb / (1+w))</th>
<th>Void ratio (e) = Gs γw / γd -1</th>
<th>Degree of saturation (Sr = wGs/e)</th>
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</table>
Figure 3. 10: A typical Plot of SWCC for SB soil

Figure 3.11: A typical plot of SWCC for CL soil

3.4 Bender Element

3.4.1 Shear Wave Velocity -Piezo Electric Transducers

Piezoceramic bender element is an electro-mechanical transducer, capable of converting mechanical energy into electrical energy. It consists of two thin piezoceramic plates which are bonded together with conducting surface thereby leading to a -type arrangement (referred to as
sandwich). The electrical connections are designed in these elements in such a way that one plate elongates and other plate contracts when supplying electricity. This elongation and contraction of element develops tension and compression resulting generation of electrical signal in the bender element. Such electrical signals are used for analysis of stiffness.

3.4.2 Objective of the Test

The objective of this test was to

- Monitor the shear wave velocity and travel time of the wave signals
- Study the influence of moisture content, suction, and compaction in the stiffness of unsaturated soil

3.4.3 Sample Preparation

To achieve the objectives of the test, bender element of size 12mm x 5mm x 0.5mm were used for this research. The same specimens used for SWCC test for CL were used for this test. The specimens which were subjected to various suctions ranging from zero to 500 kPa in SWCC cell with different moisture content were used in the bender element.

3.4.4 Experimental Setup and Measurement of Shear Wave Velocity

The stiffness characteristics of unsaturated soil at various moisture content along the drying path of SWCC for sample CL was computed using bender element. The experimental setup is as shown in the Figure 3.12. A Kfrohnhte 1450 was used as a function generator which generated the signals and transmitted to the specimen through the bender element. When the specimen was placed between the receiver and transmitter, the electrical signal coming from the functional generator via bender element was converted in to mechanical energy and was transmitted in the form of shear wave to the signal amplifier. The wave got amplified into strong wave where they got stored and displayed in the oscilloscope.
The stored signals were sent to computer monitor. The transmitted and received signals were plotted. The optimum frequency ranging from 5-10 Hz and peak voltage of 20 V was used for this test. The input voltage from a signal generator was sent to transmitter in the form of pulse sine wave as shown in the Figure 3.13.

**Figure 3.12: Setup line diagram of bender element**

**Figure 3.13: Typical sine wave pulse**
Figure 3.14 shows the picture of oscilloscope, function generator, and signal amplifier that were used in the lab.

- The shear wave velocity was calculated using the following relation

\[ V_s = \frac{1}{t} \]

Where, \( v_s \) = shear wave velocity

\( l \) = effective length of specimen

\( t \) = travel time (suggested by Viggiani and Atkinson (1995a))

![Figure 3.14: Instruments used for generating shear wave- a) Agilent 6014 Oscilloscope, b) Krohnhte 3944 Signal Amplifier, and c) Kroohnhte 1400 Function Generator](image)

The shear modulus of soil is calculated by using the relation:

\[ G_{\text{max}} = \rho V_s^2 \]

Where,

\( G_{\text{max}} \) = shear modulus

\( \rho \) = bulk density of the soil and

\( V_s \) = shear wave velocity
CHAPTER FOUR: NUMERICAL MODELING – HYDRUS

4.1 Introduction to HYDRUS

HYDRUS is a software package for simulating water, heat, and solute transport in two and three dimensional variable in saturated and unsaturated porous media. The effect of moisture variation in suction and contaminant transport flow pattern can easily be simulated using this software. The HYDRUS program numerically solves the Richard’s equation for both saturated and unsaturated water flow and solute transport. HYDRUS can handle flow domain characterized by irregular boundaries. It is simple to use and is based on finite element modeling.

4.1.1 Research Modeling

Two models were prepared, one-dimensional and two-dimensional water flows representing the unsaturated field condition were developed for this research purpose. The boundary condition, such as, temperature, precipitation, evaporation and transpiration were applied to simulate in the model. The input data such as index properties, saturated hydraulic conductivity, moisture content, bulk and dry unit weight, and strength parameters which were obtained from the lab test were used in the modeling. For 1-D model, hydraulic behavior of unsaturated soil such as soil suction, hydraulic conductivity and volumetric water content and their relationship were monitored for the period of 173 days (six month) (April 1 to September 30) in which environmental condition were assumed to be relatively dry. The 2-D model was prepared representing vertical cutoff wall in which the relation of suction, moisture content, and other soil hydraulic properties were studied for different climatic region representing Louisiana, Denver and Arizona. The climatic input such as precipitation, temperature and evaporation data were applied taking the average value of recorded data in the corresponding states. This model was simulated for different time period such as 30 days, 6 months, 1 year, 5 year and 10 year to
monitor the hydraulic behavior of unsaturated soil. The various components of HYDRUS are described in the following section.

4.1.2 Graphical User Interface

Graphical user interface is the main program unit in HYDRUS model which defines the overall computational system (see Figure 4.1). This main module has a control on each program and determines which modules are required for a particular application of modeling. The main module contains a project manager and a unit for pre processing and post processing. The pre processing unit contains the information for specifications to run the program. Similarly, the post processing unit consist of a graphical presentation of soil hydraulic properties and other selected variables and their interrelationships including animation, contour and maps.

Figure 4.1: HYDRUS graphical User Interface (HYDRUS 2006, user’s manual)

4.2 Project Manager and Data Management

A command called project manager is used to manage data such as to open, copy and deleting on existing projects. The input and output file in different directory can be managed from this command. This command helps in organizing the projects by giving the information and
description such as name of the project and other necessary description.

Figure 4. 2: Project Manager with the project tab (HYDRUS 2006, user’s manual)

The dialog box for project manager is as shown in the Figure 4.2.

4.3 Geometry information

To solve water flow and solute transport, geometry of model can be defined either in 1D, 2D or in 3D in HYDRUS package. Within the geometry information, we have the facility to select the domain either in a simple geometry with structured finite element mesh or more general geometry having unstructured finite element mesh.

In this research, for 1D simulation, the domain was selected in one dimensional soil profile with 1 cm x 1 cm cross sectional area. The model consist of two layers, top layer (SB material) is 40 cm thick with depth of root zone 30 cm.
The second layer (CL) is extended to a depth of 300 cm. The type of geometry selection window dialog box is shown in Figure 4.3. The detailed geometric cross section of 1D model is shown in Figure 4.4. The model represents the field condition of unsaturated zone.
The 2D modeling of vertical cut-off wall is as shown in Figure 4.4 (b). In this model, the average height of cut-off wall was selected as 50 feet and average width as 30 inch. The water table in the model is considered at a depth of 30 feet from the ground level. The surrounding soil is considered as sand with conductivity 10E-2 cm/sec. SB slurry wall is modeled for 5% bentonite clay with conductivity value of 2E-10 m/s. A protective cover is provided on the top of slurry wall to maintain the moisture contain within the body. The same geometry model is considered for three different climactic geographical regions: arid region (Arizona), semi-arid region (Denver), and saturated region (Louisiana). The relation among soil suction, hydraulic conductivity and volumetric water content were studied in three different zones for various time periods.

![Diagram of 2D cutoff wall with protective cover and sand layer.]

Figure 4.5: A Typical physical model of HYDRUS simulation for 2D cutoff wall

4.4 Flow Parameters

Flow parameter is the unit where various process and criteria are specified and defined before simulating the HYDRUS software. The detailed discussions of such flow parameters are discussed in the following sections.
4.4.1 Main process

The main process dialog window of HYDRUS is as shown in Figure 4.5. The process to be simulated for water flow, solute transport, heat transport and root water uptake transport are defined in this section. For this research purpose, only water flow, and root water uptake are selected.

![Figure 4.5: The main process dialog window (HYDRUS 2006, user’s manual)](image)

4.4.2 Time information

The time information dialog box contains time unit, time discretization, and implementation of boundary conditions that are needed for simulation, are defined under this section. The window dialog box is as shown in the Figure 4.6. The unit of time is selected in days for this simulation purpose.

![Figure 4.6: The main process dialog window (HYDRUS 2006, user’s manual)](image)
4.4.3 Output information

The output information dialog box contains the information about print options, print times and sub-region. An appropriate selection is made depending upon the objective of the model.

4.4.4 Soil Hydraulic Model

Soil hydraulic model is the next important command where various models are defined. It has mainly two parts: hydraulic model and hysteresis. In hydraulic model, a selection is to be done among various six models to define for soil hydraulic analysis. In this research, van Genuchten-Mualem model was selected for no hysteresis condition. The dialog box showing various soil hydraulic models is shown in Figure 4.7.
4.4.5 Water Flow Parameter

The parameter used in various soils hydraulic models, presented in section 4.4.4 are specified in this section. Figure 4.8 is the dialog window which contains the detail of water flow parameters used to define various soil hydraulic models.

Figure 4.8: Soil hydraulic model dialog window (HYDRUS 2006, user’s manual)

Figure 4.9: Flow parameter dialog window (HYDRUS 2006, user’s manual)
In all six models, for example, Brooks and Corey (1964), Van Genuchten (1980), Vogel and Cislerova (1988), Kosugi (1996) and Dunner (1994), various parameters are used such as residual and saturated water content, saturated hydraulic conductivity (Ks), pore connectivity parameter (l) and empirical coefficients (Alfa), and n. The adopted values for such input parameters used in this research are given in the Table 4.1.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>1D modeling</th>
<th>2D modeling</th>
</tr>
</thead>
<tbody>
<tr>
<td>Residual water content (θr)</td>
<td>0.066</td>
<td>0.045</td>
</tr>
<tr>
<td>Saturated water content (θs)</td>
<td>0.35</td>
<td>0.35</td>
</tr>
<tr>
<td>Saturated conductivity (cm/day)</td>
<td>0.00172</td>
<td>2601.72</td>
</tr>
<tr>
<td>n</td>
<td>1.2</td>
<td>2.68</td>
</tr>
<tr>
<td>I</td>
<td>0.5</td>
<td>0.5</td>
</tr>
<tr>
<td>Alpha (α)</td>
<td>0.008</td>
<td>14.5</td>
</tr>
</tbody>
</table>

Table 4.1 Input parameter used in model

<table>
<thead>
<tr>
<th>Textural</th>
<th>θr (L3L-3)</th>
<th>θs (L3L-3)</th>
<th>Α(cm-1)</th>
<th>n(-)</th>
<th>Ks(cmd-1)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand</td>
<td>0.045</td>
<td>0.43</td>
<td>0.145</td>
<td>2.68</td>
<td>712.8</td>
</tr>
<tr>
<td>Loam</td>
<td>0.078</td>
<td>0.043</td>
<td>0.036</td>
<td>1.56</td>
<td>24.96</td>
</tr>
<tr>
<td>Silt</td>
<td>0.034</td>
<td>0.46</td>
<td>0.016</td>
<td>1.37</td>
<td>6</td>
</tr>
<tr>
<td>Clay</td>
<td>0.068</td>
<td>0.38</td>
<td>0.008</td>
<td>1.09</td>
<td>4.8</td>
</tr>
</tbody>
</table>

Table 4.2 Input parameters in Hydrus for analytical function of van Genuchten (1980)

<table>
<thead>
<tr>
<th>Time (day)</th>
<th>Precipitation (cm/day)</th>
<th>Evaporation (cm/day)</th>
<th>Transpiration (cm/day)</th>
<th>hCritA</th>
</tr>
</thead>
<tbody>
<tr>
<td>173</td>
<td>0.02-0.07</td>
<td>0.001--0.003</td>
<td>0.08-0.28</td>
<td>15000</td>
</tr>
</tbody>
</table>

Table 4.3 Input climatic data for time variable boundary condition (for 1D Model)

<table>
<thead>
<tr>
<th>Climatic region</th>
<th>Precipitation (m/yr)</th>
<th>Evaporation (m/yr)</th>
<th>Temperature (Degree celsius)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Louisiana</td>
<td>1.52</td>
<td>0.1</td>
<td>20.5</td>
</tr>
<tr>
<td>Denver</td>
<td>0.3899</td>
<td>0.1</td>
<td>8.33</td>
</tr>
<tr>
<td>Arizona</td>
<td>0.18</td>
<td>2.54</td>
<td>51.67</td>
</tr>
</tbody>
</table>
Figure 4.10: Soil hydraulic parameters for analytical function of van Genuchten (1980)

4.4.6 Time Variable Boundary Condition

Time variable boundary condition includes the parameters such as time, precipitation, evaporation, transpiration which are the input parameters defining time variable boundary condition. The Figure 4.9 is the time variable boundary condition dialog box.

Figure 4.11: Time variable boundary condition dialog box (HYDRUS 2006, user’s manual)
4.5 Finite Element Mesh

Before simulating the model, finite element mesh is generated for the whole domain. The mesh can either be structured or meshgen. The discretization of rectangular domain, hexahedral domain and finite element mesh parameters are shown in the Figure 4.10. For this model, structured FE mesh was selected as shown in Figure 4.11.

Figure 4.12: Finite element mesh generator dialog window (HYDRUS 2006, user’s manual)

![Finite element mesh generator dialog window](image1)

Figure 4.13: Structured FE Mesh for the model 1D and 2D modeling examples of simple rectangular geometry (HYDRUS 2006)

a. 1-D Modeling
b. 2-D Modeling
4.6 Domain Properties, Initial and Boundary Condition

Initial and boundary condition for both water flow and solute transport are defined in this section. Hydraulic factors, root water uptake parameters and possible hydraulic anisotropy, observation nodes etc are specified in domain properties dialog window as shown in Figure 4.12.

![Image of Default Domain Properties dialog window](image1)

**Figure 4.14: Default domain properties dialog window (HYDRUS 2006, user’s manual)**

![Image of Water Flow Initial Condition dialog box](image2)

**Figure 4.15: Water flow initial condition dialog box (HYDRUS 2006, user’s manual)**
For 1D modeling, initial condition for water flow was selected assuming the ground water table at 55 cm below top surface. For 2D modeling, the ground water table was kept at 30 feet (9.14m) from the ground surface. Water flow initial condition was selected in terms of distribution and pressure parameters as shown in the Figure 4.13 of window dialog box. Similar window was used for defining temperature distribution. Figure 4.14 is the typical view of the observation nodes for 1D and 2D model.

![Diagram showing observation nodes and boundary conditions](image)

**Figure 4.16: Typical view of boundary condition and observation node (1D and 2D model)**

### 4.7 Calculation and Graphical Output

After defining each and every parameter in the modeling, the last part is calculation. The current project is saved and is applied for calculation. The calculation time for a project depends upon the allowed time for discretization. For 1D modeling, it took 90 minute to complete the calculation while for 2D modeling; the average time taken was 51 minute. The results of the simulation were obtained in two parts. In the first part, the result was in the form of graphical display i.e., the results were in the form of contour map, isobands, color points, spectral maps, velocity vectors etc. The results could also be displayed in the form of flow animation at a particular time. In the second part of the result, the additional information such as boundary
fluxes, soil hydraulic properties, observation points, pressure head etc were displayed in x-y graphs.

4.8 Objective of the Model Simulation

Two models were prepared to simulate the water flow in a soil profile representing field condition for 1D infiltration model and 2D vertical cutoff wall. These models were simulated for one dimensional and two dimensional flows. The geometry of the domain can be seen as shown in the Figure 4.4a and 4.4b. The boundary condition such as atmospheric data, ground water condition etc. were used for soil hydraulic modeling. Calculations were performed for the period of 173 days (April first to September 30) considering the driest period of the year for 1D model. In the case of 2D modeling, various time period were setup (for example, 30 days, 6 months, 1 year, 5 year and 10 year) representing various climatic zone such as arid, semi arid and saturated zone. Surface boundary condition was applied for precipitation, transpiration, evaporation and transpiration. The boundary condition for bottom part of model was selected for drainage flux ground water relationship. The ground water was initially kept at 55 cm below soil surface for 1D model. In the case of 2 D model, ground water table was kept 30 feet from the ground. The initial moisture profile was assumed to be in equilibrium with the initial ground water level.

The objective of the HYDRUS modeling was to identify

- The variation of suction (head) with water content(\(\theta\))
- The variation of hydraulic conductivity with theta
- The variation of hydraulic conductivity versus head
- The variation of suction with depth (at different observation points)
- The variation of water capacity versus theta and head
- The effect of environmental factors on suction
CHAPTER FIVE: DISCUSSION OF EXPERIMENTAL RESULTS AND HYDRUS SIMULATION

5.1 Introduction
The objective of this research was to study the behavior of unsaturated soil. The study was carried out in two aspects: mechanical behavior and hydraulic behavior. To accomplish the objective, the study was divided into two parts: experimental lab work and numerical simulation using HYDRUS software. The selected materials for the research purpose were sand bentonite mixed soil (SB) (5% Bentonite and 2% Bentonite) and Low plasticity silty Clay (CL). The physical and index properties of these soils were obtained experimentally in the lab. SWCC, direct shear test, and saturated hydraulic conductivity test were conducted on all samples. The climatic data required for HYDRUS simulation such as temperature, precipitation, and evaporation and transpiration rate were obtained from the average value recorded in Louisiana, Denver and Arizona climatic regions. Some of the results such as saturated hydraulic conductivity obtained from the lab work were applied for HYDRUS simulation. The HYDRUS was simulated for 1D and 2D modeling. The result of SWCC from lab work and HYDRUS simulation were compared. 1D modeling was simulated for 6 months period while 2D model was simulated for various time zone period such as 30 days, 6 months, 1 year, 5 year and 10 year period for three different climatic reason representing Louisiana, Denver, and Arizona.

5.2 Grain Size Distribution
As discussed in the literature, there is a relationship between SWCC and grain size distribution of the soil (Perera, Y.Y., Zapata, Z.E et.al, (2005). To correlate the relation between SWCC and grain size distribution and to classify the soil, sieve analysis tests were performed according to ASTM D 422 and ASTM D2487-06 (for sieve analysis and hydrometer test respectively). Sieve
analysis is used for sand and gravel, particle sizes greater than 0.075 mm (retained on the No. 200 sieve) and smaller than 4.75 mm (Passing through No. 4 sieve). Similarly, the hydrometer analysis is used for particles smaller than 0.075mm (silt and clay), which is based on the principle of sedimentation process. The result of grain size distribution of SB (5% and 2%) and CL are presented in Figure 3.1.

5.3 Direct Shear Test

Direct shear test was done in the lab for both soil sample, SB (5% and 2%), and CL to determine the angle of friction and cohesive properties, which are input parameters required in HYDRUS simulation.

From the experiment, the angle of friction was found to be 41.2, 42, and 30 degree for 5% and 2% SB and CL soil respectively. Similarly, the cohesion was found to be 45, 25 and 350 psf for 5% SB, 2% SB and CL soil. The plot of direct shear test for specimen SB and CL is shown in the Figure 3.2, 3.3 and 3.4 respectively.

5.4 Specific Gravity and Index Properties

The test is carried out in the lab using ASTM D854 test procedure to determine specific gravity and index properties of soil specimen SB and CL, which were used in this research. From the experiment, specific gravity of SB and CL were found to be 2.68 (5% SB), 2.65 (2% SB) and 2.7 (CL) respectively. Similarly, plasticity index of CL was determined as 19.0 while sample SB was observed to be non plastic clay.

5.5 Hydraulic Conductivity Test

The saturated hydraulic conductivity of sample SB and CL was determined in the lab using falling head permeability test as described in section 3.2. The average coefficient of permeability for sample was found to be 2.06E-10 m/sec (for 5% SB), 1.7E-9 m/sec (for 2% SB) and 1.6E-7
m/sec (for CL) samples. The permeability of CL was found to be higher than SB. Several factors such as grain size distribution, stress history and water content etc affects the permeability of soil. In general speaking, fine grained soils have lower permeability than coarse grained soils.

5.6 Soil Water Characteristics Curve (SWCC)

Soil water characteristics curve, (SWCC) was determined in the lab using Fredlund’s SWCC device (GCTS SWCC-150). The specimens for SWCC test were prepared for both samples, SB (5% and 2%), and CL, according to the procedure described section 3.3.5. The dimension of the consolidation ring which was used in preparing sample was 6.34 mm diameter, 2.54 mm thickness, and 80.14 mm$^3$ volumes. The detail of calculation for SWCC is shown in the table 5.1 to 5.3.

**Table 5.1 Sample Calculation table for SWCC for SB-5%**

| Dia of sample | 6.343 | mm |
| Thickness of sample | 2.533 | mm |
| Volume of sample (V) | 80.00 | Cc |
| Specific gravity of sample (Gs) | 2.68 | N/A |
| Pressure (psf) | Reading in the left volume tube (cc) | Reading in the right volume tube (cc) | Average reading of the volume tubes (cc) | Driven out water (cc) | Wt of wet bentonite (Ww) (g) | Dry Wt. bentonite (Ws) (g) | Wt of Water (Ww) (g) | V.Water content (w=Ww/V) |
| 0.01 | 110 | 110 | 110 | 0 | 156.25 | 106 | 50.25 | 0.493 |
| 15 | 117 | 119 | 118 | 4 | 152.25 | 106 | 46.25 | 0.488 |
| 40 | 125 | 128 | 126.5 | 4.25 | 148 | 106 | 42 | 0.472 |
| 75 | 140 | 144 | 142 | 7.75 | 140.25 | 106 | 34.25 | 0.43 |
| 100 | 152 | 154 | 153 | 5.5 | 134.75 | 106 | 28.75 | 0.36 |
| 140 | 180 | 182 | 181 | 14 | 120.75 | 106 | 14.75 | 0.18 |
| 160 | 182 | 185 | 183.5 | 1.25 | 119.5 | 106 | 13.5 | 0.17 |
| 190 | 192 | 194 | 193 | 4.75 | 114.75 | 106 | 8.75 | 0.11 |
| 250 | 202 | 202 | 202 | 4.5 | 110.25 | 106 | 4.25 | 0.05 |
| 350 | 209 | 208 | 208.5 | 3.25 | 107 | 106 | 1 | 0.01 |
Figure 5.1: SWCC for SB (5%) (Test #1)

Figure 5.1 is the SWCC of SB-5% sample. Initial volumetric water content of the sample was 0.49 at suction of 0.01 kPa. As the water content of the soil decreases to about 0.46, suction pressure increases rapidly around 70 kPa. So the air entry value of SB soil is observed about 70 kPa in this test. Suction pressure further increases up to 500 kPa at moisture content less than 0.1. Table 5.2 is the calculation of SWCC for the test #2 of the same sample, (SB-5%). The plot of SWCC is shown in Figure 5.2.

Figure 5.2: SWCC for SB (Test #2)
Table 5.2: Sample Calculation for SWCC

<table>
<thead>
<tr>
<th>Dia of sample (Mm)</th>
<th>Thickness of sample (Mm)</th>
<th>Volume of sample (Cc)</th>
<th>Specific gravity of sample (Gs)</th>
<th>Suction (kPa)</th>
<th>Reading in the left volume tube (cc)</th>
<th>Reading in the right volume tube (cc)</th>
<th>Total reading of the volume tubes (cc)</th>
<th>Drive out water (cc)</th>
<th>Wt of wet Bentonite (g)</th>
<th>Wt of Dry. Bentonite (g)</th>
<th>Wt / volume of Water, Ww (g or cc)</th>
<th>Vol. Water Content (cc/cc)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.343</td>
<td>2.533</td>
<td>80.00</td>
<td>2.68</td>
<td>0.1</td>
<td>150</td>
<td>148</td>
<td>149</td>
<td>0</td>
<td>145</td>
<td>104</td>
<td>41.0</td>
<td>0.51</td>
</tr>
<tr>
<td>20</td>
<td>161</td>
<td>159</td>
<td>160</td>
<td>5.5</td>
<td>171</td>
<td>169</td>
<td>170</td>
<td>5</td>
<td>134.5</td>
<td>104</td>
<td>35.5</td>
<td>0.44</td>
</tr>
<tr>
<td>40</td>
<td>171</td>
<td>169</td>
<td>170</td>
<td>5</td>
<td>181</td>
<td>180</td>
<td>180.5</td>
<td>5.25</td>
<td>129.25</td>
<td>104</td>
<td>25.3</td>
<td>0.32</td>
</tr>
<tr>
<td>60</td>
<td>191</td>
<td>191</td>
<td>191</td>
<td>5.25</td>
<td>201</td>
<td>201</td>
<td>201</td>
<td>5</td>
<td>119</td>
<td>104</td>
<td>15.0</td>
<td>0.19</td>
</tr>
<tr>
<td>80</td>
<td>205</td>
<td>205</td>
<td>205</td>
<td>2</td>
<td>213</td>
<td>213</td>
<td>213</td>
<td>4</td>
<td>113</td>
<td>104</td>
<td>9.0</td>
<td>0.11</td>
</tr>
<tr>
<td>1000</td>
<td>218</td>
<td>218</td>
<td>218</td>
<td>2.5</td>
<td>225</td>
<td>224</td>
<td>224.5</td>
<td>3.25</td>
<td>107.25</td>
<td>104</td>
<td>6.5</td>
<td>0.08</td>
</tr>
<tr>
<td>10000</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Volumetric water content corresponding 500 kPa was calculated in the lab and beyond the 500 kPa suction, volumetric water content is estimated using equation 5.1. Table 5.3 is the summary data for SWCC test for 5% bentonite, 2% bentonite SB soil, and for CL soil.
Table 5.3 Data Summary for SWCC for SB and CL

<table>
<thead>
<tr>
<th>Suction (kPa)</th>
<th>Sample 1 (5% SB)</th>
<th>Sample 2 (5%SB)</th>
<th>Sample 3 (2.0%SB)</th>
<th>Sample 4 (2.0%SB)</th>
<th>Sample 5(CL)</th>
<th>Sample 6(CL)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.49</td>
<td>0.49</td>
<td>0.481</td>
<td>0.487</td>
<td>0.5</td>
<td>0.5</td>
</tr>
<tr>
<td>5</td>
<td>0.47</td>
<td>0.48</td>
<td>0.468</td>
<td>0.47</td>
<td>0.49</td>
<td>0.48</td>
</tr>
<tr>
<td>30</td>
<td>0.46</td>
<td>0.45</td>
<td>0.46</td>
<td>0.43</td>
<td>0.47</td>
<td>0.46</td>
</tr>
<tr>
<td>80</td>
<td>0.44</td>
<td>0.44</td>
<td>0.43</td>
<td>0.4</td>
<td>0.41</td>
<td>0.44</td>
</tr>
<tr>
<td>160</td>
<td>0.43</td>
<td>0.41</td>
<td>0.28</td>
<td>0.24</td>
<td>0.22</td>
<td>0.24</td>
</tr>
<tr>
<td>300</td>
<td>0.15</td>
<td>0.1</td>
<td>0.11</td>
<td>0.08</td>
<td>0.2</td>
<td>0.18</td>
</tr>
<tr>
<td>500</td>
<td>0.06</td>
<td>0.07</td>
<td>0.05</td>
<td>0.04</td>
<td>0.13</td>
<td>0.11</td>
</tr>
<tr>
<td>1000</td>
<td>0.04</td>
<td>0.03</td>
<td>0.04</td>
<td>0.02</td>
<td>0.09</td>
<td>0.07</td>
</tr>
<tr>
<td>10000</td>
<td>0.02</td>
<td>0.01</td>
<td>0.01</td>
<td>0.01</td>
<td>0.04</td>
<td>0.03</td>
</tr>
</tbody>
</table>

Figure 5.3 is the plot of suction vs. theta for 5% bentonite mixed SB soil.

![Figure 5.3: Suction vs. Theta for SB (5% bentonite) soil](image)

Both of the plots of 5% SB soil closely follow the similar trend for suction vs. theta graph (SWCC). At theta 0.45, soil exhibits maximum suction, say about 150-200 kPa, which is the air entry value of SB 5% soil.

Figure 5.4 is the plot of SWCC for SB 2% bentonite mixed soil. The air entry value of 2% SB was observed around the range of 90-95 kPa.
Similarly, Figure 5.5 is the plot of SWCC for CL sample used in the research. The air entry value of CL soil was observed around the range of 85-90 kPa.

Figure 5.6 is the plot of SWCC for all samples SB and CL used in the research. From the comparison of SWCC plot, it is obtained that SB soil with 5% bentonite has the highest air entry value while CL sample was found with lowest air entry value.
The volumetric water content for suction higher than 500 kPa was estimated using van Genuchten (1980) to obtain SWCC curve.

$$\theta(h) = \theta_r + \frac{\theta_s - \theta_r}{[1 + (\alpha h)^{\frac{1}{n}}]^m} \text{ (5.1)}$$

The values of parameters in Equation 5.1 are used as: residual volumetric water content, $\theta_r = 0.010$, saturated volumetric water content, $\theta_s = 0.489$, size distribution index, $n = 2.7$, bubbling pressure (inverse of the air-entry value), $\alpha = 0.0045$, pore connectivity parameter, $L = 0.5$, soil water parameter, $m = 1 - 1/n = 0.64$, Matric suction, $h = 1000$ kPa.

5.6.1 Suction Versus Degree of Saturation

Finding the degree of saturation at successive suction is a tedious job in Fredlund SWCC device. It was assumed that void ratio to be a constant at each suctions, which is not realistic; however the degree of saturation at each suctions is calculated assuming a constant void ratio. The plot of matric suction vs. degree of saturation is obtained as shown in the Figure 5.7.
5.6.2 Suction Versus Volumetric Water Content

The results of Suction vs. volumetric water content for soil SB and CL are presented in Figure 5.8.

The plot of SWCC in Figure 5.8 is the average value of 5% and 2% of SB sample and CL. From
the Figure 5.8, it is observed that both sample SB and CL follow a similar kind of trend for volumetric water content vs. suction relationship. It is observed that SB soil has a little more water retention capacity than CL. It might be because of the percentage of bentonite present in SB soil. If we increase the percentage of bentonite in soil, the water holding capacity can be increased. If we see the grain size distribution curve in Figure 3.1, it is clear that the soil SB is poorly graded with higher percentage of sand while soil CL is well graded soil. Poorly graded soil with higher percentage of sand will have lower water retention capacity (see section 5.7.3), however, because of the percentage of bentonite contained in the soil, the water holding capacity of SB was found higher than CL.

### 5.6.3 SWCC and Grain Size Distribution

Prediction of SWCC based on grain size distribution proposed by Fredlund & Xing (1994) discussed in chapter two. Similarly, Perera, Y.Y., Zapata, Z.E et al (2005) developed a family of SWCC curve for prediction of soil water characteristics based on grain size distribution and index properties. The shape of family of SWCC representing varying index properties of soil presented by Zapata (1999) is similar to the curve given in Figure 5.9, a & b.
Figure 5.9: Family of SWCC (a & b) developed by Zapata in 1999

The Figure 5.9 (a and b) indicates that higher the value of $D_{60}$, the curve lies on left and lower side of family curve. Similarly, higher the value of wPI ($wPI = \%$ passing #200*PI), the curve lies to the right and upper part of the SWCC family curve. The SWCC of any soil which lies between these two extreme left and extreme right sides in the family curve, proposed by Zapata (1999), can be predicted by interpolation. This concept of estimating the SWCC of soil is based on grain size distribution which is similar to the mathematical analysis for estimation of SWCC proposed by Fredlund (1994).

Figure 3.1 is the plot of grain size distribution for sample SB and CL. Sample SB looks poorly graded as described earlier in this chapter while sample CL is somewhat well graded. The value of $D_{60}$ for SB and CL were found to be 0.8 and 0.6 respectively. Also, the value of wPI, calculated using the relation given by Zapata (1999), is found to be 8.35. Now, using these two values of $D_{60}$ and wPI, and comparing with the family curve of SWCC developed by Zapata (1999), SWCC can be predicted for SB and CL. From the graph, it was predicted that SB with 5% and 2% Bentonite mixed soil has higher water retention capacity than CL, similar kind of
result were also obtained experimentally in the lab. Figure 5.10 is the plot of SWCC for various type of soil observed by Fredlund (1994), which was based on the mathematical model. This plot is similar to the family of SWCC presented by Zapata (1999). For example, Sand has higher value of $D_{60}$ in comparison to loam clay, medium clay and peat. So, the SWCC of sand lies to the left side in the plot of family of SWCC. This plot also indicated that sand has very low water retention capacity. The air entry value of sand is also low in comparison to the other soils. The general concept is, higher the grain size, lower will be the capillary rise of water in the tube. The opposite phenomenon can be seen in case of peat soil. Because of the well graded grain size distribution in peat, pore space are very small, the capillary rise will be higher in such soil and hence air entry value of such soil will be higher in comparison to the sandy soil. Such soil will have high water retention capacity. Even with the decrease of water content at considerable amount in the SWCC cell, there will be slow rate of increase of suction, and after certain level of water content, suction increases in a rapid growth (see Figure 5.10).

![Figure 5.10: SWCC for various clay (Fredlund, 1994)](image)

5.7 Measurement of Shear Wave Velocity Using Bender Element

The shear wave velocity was measured in the lab using Bender Elements (BE) as described in
A series of total nine tests were carried out in the lab to measure the travel time and velocity of propagation of wave through a known length of BE. The influence of suction on stiffness, the influence of water content on stiffness and the effect of compaction on stiffness of soil was observed during the shear wave measurement. The experiment for shear wave velocity was done side by side with the SWCC test. The first set of BE was prepared using the specimen used for SWCC test. The specimen prepared for SWCC test were first subjected to different suction pressure in SWCC cell, which were then used in BE. At each successive suction pressure, once the desired pressure increment was achieved in the SWCC cell, the specimen was taken out from the cell and the same cell was used for BE to measure the shear wave velocity. While doing so, there might be chance of getting some errors. One possibility of error would be of getting drier of the sample during the process of taking out the specimen from the SWCC cell and transferring it to the shear wave equipment. Another possibility of arising error would be release of suction pressure from the specimen which was applied in the SWCC cell. When the sample is opened to the atmosphere from the closed cell of SWCC, the pressure on the specimen may get released and hence may not truly represent the stress level while measuring shear wave velocity. However, considering the error effect is negligible, the experiments were carried out and the influence of moisture content, suction and density on stiffness were analyzed for CL sample.

5.7.1 Influence of Moisture Content on Matric Suction and Stiffness

After the completion of set of test of SWCC and shear wave velocity, calculations were done to analyze the behavior and relation of moisture content, matric suction and stiffness of soil.

Figure 5.11 is the plot showing transmitted and received signals from oscilloscope during shear wave measurement test. Channel 1 is a transmitted signal and channel 2 is a received signal.
Travelling time was determined measuring peak to peak time distance of received and transmitted signal, which is widely adopted developed by Viggiani and Atkinson (1995). Shear wave velocity was calculated by dividing the length of BE specimen by travel time.

Table 5.4: Influence of Moisture Content, Compaction and Suction on Soil Stiffness

<table>
<thead>
<tr>
<th>Suction (kPa)</th>
<th>Reading in the left volume tube (cc)</th>
<th>Reading in the right volume tube (cc)</th>
<th>Total reading of the volume tubes (cc)</th>
<th>Driven out water (cc)</th>
<th>Wt of wet bentonite (g)</th>
<th>Wt or volume of Water, Ww (g or cc)</th>
<th>Volumetric Water Content (cc/cc)</th>
<th>Time (mSec)</th>
<th>Velocity (m/s)</th>
<th>Gmax (MPa)</th>
<th>Density (ρ) KN/m³</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1</td>
<td>150</td>
<td>148</td>
<td>149</td>
<td>0</td>
<td>145</td>
<td>104</td>
<td>41.0</td>
<td>0.51</td>
<td>0.21</td>
<td>12.34</td>
<td>2.71</td>
</tr>
<tr>
<td>20</td>
<td>161</td>
<td>159</td>
<td>160</td>
<td>5.5</td>
<td>139.5</td>
<td>104</td>
<td>35.5</td>
<td>0.44</td>
<td>0.19</td>
<td>13.05</td>
<td>2.91</td>
</tr>
<tr>
<td>40</td>
<td>171</td>
<td>169</td>
<td>170</td>
<td>5</td>
<td>134.5</td>
<td>104</td>
<td>30.5</td>
<td>0.38</td>
<td>0.19</td>
<td>13.69</td>
<td>3.09</td>
</tr>
<tr>
<td>60</td>
<td>181</td>
<td>180</td>
<td>180.5</td>
<td>5.25</td>
<td>129.25</td>
<td>104</td>
<td>25.3</td>
<td>0.32</td>
<td>0.18</td>
<td>14.23</td>
<td>3.21</td>
</tr>
<tr>
<td>80</td>
<td>191</td>
<td>191</td>
<td>191</td>
<td>5.25</td>
<td>124</td>
<td>104</td>
<td>20.0</td>
<td>0.25</td>
<td>0.16</td>
<td>15.83</td>
<td>3.81</td>
</tr>
<tr>
<td>140</td>
<td>201</td>
<td>201</td>
<td>201</td>
<td>5</td>
<td>119</td>
<td>104</td>
<td>15.0</td>
<td>0.19</td>
<td>0.15</td>
<td>16.89</td>
<td>4.16</td>
</tr>
<tr>
<td>170</td>
<td>205</td>
<td>205</td>
<td>205</td>
<td>2</td>
<td>117</td>
<td>104</td>
<td>13.0</td>
<td>0.16</td>
<td>0.14</td>
<td>18.76</td>
<td>5.05</td>
</tr>
<tr>
<td>200</td>
<td>213</td>
<td>213</td>
<td>213</td>
<td>4</td>
<td>113</td>
<td>104</td>
<td>9.0</td>
<td>0.11</td>
<td>0.11</td>
<td>23.03</td>
<td>7.35</td>
</tr>
<tr>
<td>300</td>
<td>218</td>
<td>218</td>
<td>218</td>
<td>2.5</td>
<td>110.5</td>
<td>104</td>
<td>6.5</td>
<td>0.08</td>
<td>0.08</td>
<td>31.66</td>
<td>13.58</td>
</tr>
</tbody>
</table>

Figure 5.11: Graph showing signals from oscilloscope for BE
For the series of nine tests, travel time and shear velocity was calculated at corresponding suction values of 0.1, 20, 40, 60, 80, 140, 170, 200, and 300 kPa. Finally, shear modulus was calculated using the relation, $G_{\text{max}} = \rho \cdot V^2$. The data for calculation in Table 5.4 were analyzed to see the influence of moisture content on stiffness of the soil. The plot of stiffness versus moisture content was made as shown in the Figure 5.13. It is concluded that moisture content has great influence on shear stiffness. It was found that if the moisture content on soil is increased, stiffness of soil was found to be decreased as in Figure 5.13.

![Figure 5.12: Picture taken from oscilloscope showing signals for BE](image1)

![Figure 5.13: Plot of stiffness vs. moisture content](image2)
Figure 5.14: Plot of Stiffness vs. Degree of Saturation

Similar kind of plot was reported by Ng et al., 2000 as described in the literature review chapter. Similarly, the influence of matric suction on stiffness was studied. The plot of stiffness versus metric suction is developed from the table 5.4 as shown in Figure 5.15. From the result of plot in Figure 5.15, it was observed that stiffness of soil is found increased with the increase of suction.

Figure 5.15: Plot of stiffness vs. Suction

At lower value of suction, the rate of increase of stiffness is found to be slower, but after certain level of suction (in this case, around 150 kPa) the stiffness increased with higher growth rate.
Similarly, the density of soil at different suction is calculated from the Table 5.4 which is then analyzed to see if there was any influence of density on stiffness of the soil.

![Density vs Shear Stiffness](image)

**Figure 5.16: Plot of Shear Stiffness vs. Density**

The plot of Shear stiffness vs. density of soil at different suction is plotted as shown in Figure 5.16. As more water coming out from the soil specimen, bulk density goes decreasing while dry density continues increasing. With decreasing trend of density of soil specimen, the increasing trend of soil stiffness was observed from the result of plot in Figure of 5.16.

Shear stiffness was calculated using the equation:

\[ G_{max} = \rho \cdot v_s^2 \]

Where, \( \rho \) is the bulk density of soil, depends upon function of degree of saturation, void ratio, and specific gravity of the soil. The study has concluded that shear stiffness decreases with the increase of density of soil and vice versa. In other words, if more water coming out of the soil sample, dry density increases but bulk density goes decreasing, and the stiffness of soil increases.
In the above equation, shear stiffness has direct relationship with density. Shear stiffness decreases even with the increase of density; it is because when the density increases, shear velocity decreases (Figure 5.17). The square of velocity results dramatic reduction in stiffness in the equation $G_{\text{max}}=\rho*V^{2}$.

5.7.2 Stiffness Along the Drying Curve of SWCC

SWCC and Shear wave (using bender element) test were carried out simultaneously to study the behavior of unsaturated soil in terms of the influence of water content, matric suction and density of soil on stiffness. The influence of moisture content on suction and shear stiffness was observed as shown in Figure 5.18. The influence of moisture content on suction is already described in SWCC. As the moisture content decreases, matric suction increases rapidly as shown in the Figure 5.17.

Similar trend of influence of water content was observed in the case of shear stiffness too. As the value of theta (volumetric water content) decreases to some value (say 0.2 in this case), shear stiffness increases with a slow rate. After the moisture content reaches beyond 0.2, shear stiffness increases rapidly as in the Figure 5.18.
Figure 5. 18: Stiffness along the drying curve of SWCC

It is concluded that decreasing the water content increases both shear stiffness and matric suction.

5.8 Result from the HYDRUS Simulation

The behavior of unsaturated soil was studied using experimental lab work and numerical modeling using HYDRUS software. Hydraulic behavior of unsaturated soil was observed in the HYDRUS modeling. The result from the HYDRUS was compared with the experimental lab work. The detail description about the HYDRUS simulation is presented in the chapter four. The result of HYDRUS simulation (for 1D and 2D modeling) in relation to hydro-mechanical behavior of unsaturated soil is presented in the following sections.

5.8.1 Relation Between Water Content (Theta), and Suction - SWCC

From the literature review in chapter two, it is known that environmental factors such as precipitation, temperature, evaporation and transpiration lead change to the moisture content of soil in unsaturated zone. It is also known that decreasing the water table in earth surface causes decrease in water content and consequently increases negative pore water pressure (which is also
called Matric suction). To monitor the influence and behavior of moisture content in unsaturated soil, HYDRUS is simulated for 1D and 2D model as described in the chapter four. The result from HYDRUS simulation for 1D modeling in terms of matric suction vs. water content (theta) for material SB and CL is obtained as in the Figure 5.19.

![Figure 5.19: HYDRUS result for suction vs. theta (1D modeling)](image)

Figure 5.19 shows how matric suction is influenced by the water content in the model. It was observed that both materials SB and CL are influenced by water content in a similar manner. As the water content decreased, matric suction increases linearly. SB sample showed higher air entry value (approximately 100 kPa) than the CL sample (which is about 90 kPa). Similar trend of results were established in the study of closed form equation for predicting the hydraulic conductivity of unsaturated soil by M. TH. Van Genuchten (1980).

The results obtained from the experimental investigation of SWCC in the lab were found similar to the result from HYDRUS. In general, Bentonite clay exhibits high water retention capacity than other type of clay. Bentonite is, therefore, widely used as a water barrier material such as in cutoff wall (for more information about application of SB, see chapter two). In this research
model (1D modeling), SB was observed to have more water retention capacity than the CL; this might be because of the percentage of bentonite contained in the sample. Figure 5.20 is the comparison plot of suction vs. theta for lab test result with HYDRUS (1D) simulation. The plot of SWCC for SB with 5% bentonite exhibits higher level of air entry value than other sample. The result from HYDRUS simulation closely matches with the results from the lab.

![Suction Comparison](image)

**Figure 5.20: Comparison of SWCC**

The effect of grain size distribution in SWCC is also reflected in the result of HYDRUS. As we discussed in section 5.7.3 that higher the value of D$_{60}$, SWCC curve of that material lies in the left side of family curve. From the result of HYDRUS, and the result of lab test, SWCC of SB (both 5% and 2%) is found to the right side of CL with higher range of air entry value than that of CL; which was the similar trend of result as proposed by Zapata (1999). The range of air entry value of SB (5% and 2%) and CL obtained in the lab was found approximately similar to the results obtained from 1D HYDRUS simulation.
2D Modeling

Similarly, the result of 2D modeling for HYDRUS simulation of suction vs. theta is presented through Figure 5.21 to Figure 5.25. Figure 5.21 is the plot of suction vs. theta for 30 days simulation for three climatic regions: Louisiana, Denver and Arizona. The environmental input data for HYDRUS simulation for different regions were given as in the Table 4.4.

![Suction VS. Theta(30 Days)](image)

**Figure 5. 21: Suction vs. Theta for HYDRUS simulation for 2D modeling (30 days)**

The result of simulation for suction vs. theta plotted in Figure 5.21 to Figure 5.25 shows that Arizona climatic soil has the highest air entry value resulting higher level of SWCC curve and Louisiana climatic soil is found to be the lowest air entry value with lower level of SWCC curve for all simulated period of time (for example, 30 days, 6 months, 1 year, 5 year and 10 year). For the 30 days simulation period, initial value of theta was observed to be a nearest to 0.45 cc/cc at suction less than zero kPa. With the decrease of theta to 0.05, suction value is raised to higher than 1000 kPa. The air entry value of SB (5%) soil for all simulation was found to be approximately 90 kPa for Arizona climatic soil, 85 kPa for Denver, and 70 kPa for Louisiana climatic soil. These air entry value and SWCC curve are closely equal to the results to the 1D
modeling.

**Figure 5. 22: Suction vs. Theta for HYDRUS simulation for 2D modeling (6 months)**

Theta vs. suction curve in Figure 5.22 is the result from simulation of 2D modeling for 6 months period of time representing for Louisiana, Denver and Arizona climatic region.

**Figure 5. 23: Suction vs. Theta for HYDRUS simulation for 2D modeling (1 year)**
Figure 5.23 is the SWCC plot representing for LA, Denver and AZ climatic regions for 2D modeling for 1 year period of simulation.

**Figure 5.23: Suction vs. Theta for HYDRUS simulation for 2D modeling (5 year)**

Figure 5.24 is the plot of SWCC for three climatic regions of 2D simulation for 5 year period of simulation.

**Figure 5.24: Suction vs. Theta for HYDRUS simulation for 2D modeling (5 year)**

Figure 5.25 is the plot of SWCC for three climatic regions of 2D simulation for 10 year period of simulation.

**Figure 5.25: Suction vs. Theta for HYDRUS simulation for 2D modeling (10 year)**
From the above plots of SWCC, the value of theta was found to be decreased from initial volumetric water content of 0.44 to 0.35 in the first 6 months and afterwards, the value of theta was remained constant till 10 years period of simulation. The overall trend of suction vs. theta was found to be constant for all simulated period of time (for example, 30 days to 10 year period). This means, time has not shown significance impact on suction vs. theta relationships for the 2D modeling simulation for given period of time.

The plot of suction vs. theta for SB (5% and 2%) and CL from experimental investigation was compared with the HYDRUS simulation. The comparison study was done for various plots of SWCC which is presented in Figure 5.26. From this comparative study, the results from the experimental investigation were also found similar to the result of HYDRUS simulation for 2D modeling as in the case of 1D.

**Figure 5.26: Comparison of Suction vs. Theta for lab and HYDRUS for 2D modeling**

In all the above plots of suction vs. theta, SB soil with 5% bentonite soil has found to be the highest air entry value (greater than 100 kPa). The similar results were also obtained in the case of 1D modeling of HYDRUS.
5.8.2 Influence of Climatic Variation on SWCC

The study has shown that the climatic changes such as variation in temperature, precipitation, and evaporation etc. have a great impact on the suction pressure. 1D and 2D model is simulated in HYDRUS with different initial and boundary condition. The effect of climatic and environmental variation is monitored for a period of 173 days in the case of 1D modeling. Figure 5.27 is the plot of atmospheric boundary head with respect to time, which shows the variation of head in different course of time period in 1D simulation. The maximum suction (about 160 cm head) is observed during the period of 115 days. With the change in atmospheric boundary condition, there is cyclic change in suction head as shown in the Figure 5.27.

![Time vs Atmospheric Boundary Head](image)

Figure 5.27: Atmospheric Boundary Head

During the simulation, temperature was kept constant inside the model, which means a constant temperature was kept in all observation points. It is done so, because the variation of temperature gradient at different observation point was difficult to define. The variation of atmospheric boundary head in Figure 5.27 is due to the atmospheric condition such as evaporation, transpiration and precipitation effects.
5.8.3 Variation of Water Content (Theta) With Depth

The variation of suction for a given initial and boundary condition was observed at five different observation points in the simulation of 1D modeling. Figure 5.28 is the plot of variation of theta with respect to time line at different observation points. It is observed from the simulation that water content (theta) at observation point one (i.e., at node one, the top part of the model) has lower water content (approx. 0.34) than the other points (0.38 at node five). Water content was found to be increased linearly with depth. With the passage of time, a great reduction in value of theta for node one was observed. For example, the water content for node one was initially 0.34 which reached to 0.19 (approx) within the first 40-50 days of start of simulation and then it remained constant (see Figure 5.29). For other nodes, smaller reduction of water content was observed in comparison to the node one. As we know that the reduction in water content in the soil increases suction pressure.

Figure 5.28: Variation of Theta over the time at different observation point (1D modeling)

In the case of 2D modeling, theta remained constant for all observation nodes in all three
regions; Louisiana, Denver and Arizona for all range of time period of simulation. (see Figures 5.29 to Figure 5.31).

Figure 5.29: Variation of Theta over the time at different observation point (2D modeling)
Figure 5.29 is the plot of change of volumetric water content at different depth of observation points with respect to time representing Louisiana climatic region, which was simulated for 30 days period of time.

Figure 5.30: Variation of Theta over the time at different observation point (2D modeling)
Figure 5.30 is the plot of time vs. volumetric water content representing Denver climatic region which was simulated for 30 days period of time.

![Theta vs. Time (AZ-30 Days)](image)

Figure 5.31: Variation of Theta over the time at different observation point (2D modeling)

In all Figures 5.29 to 5.31, the variation of theta over the time at different observation points were found constant.

5.8.4 Variation of Suction at Different Depth of Observation Point

It is observed form the 1D simulation that there is a fluctuation of water content at different observation points of the model. The maximum fluctuation occurred at node one. The variation of suction over the time is shown in Figure 5.32. Initially, the suction was almost zero. As the reduction in moisture content (theta) started, simultaneous increase in suction was observed.
The maximum suction of about 80 kPa was observed within the 50 days of simulation. The effect of moisture variation on suction is clearly seen from these two Figures 5.28 and Figure 5.32.

In the case of 2D modeling, the variation of suction with time is simulated for different period of time (for example, 30 days, 6 months, 1 year, 5 year and 10 years) for three different climatic regions such as Louisiana, Denver and Arizona. The suction recorded in seven observation nodes of 30 days simulation for Louisiana climatic region is plotted in the Figure 5.33. Similar plot for different climatic area for different time of simulation period is presented in the Figure 5.34 to Figure 5.40.
Figure 5.33: Suction measurement at observation node (LA-30 days)

Figure 5.33 is the plot of variation of suction at different observation node with respect to time for 30 days simulation for LA climatic region.

Figure 5.34: Suction measurement at observation node (LA-1 year)

Figure 5.34 is the plot of change of suction over the time representing LA climatic region which was simulated for 1 year period of time.
Figure 5.35: Suction measurement at observation node (LA-5 years)

Figure 5.35 is the plot of variation of suction over the period of time for LA climatic region which was simulated for 5 year periods of time.

Figure 5.36: Suction measurement at observation node (LA-10 years)
Figure 5.36 is the plot of suction over the time period for LA which was simulated for 10 year period of time.

Figure 5.37: Suction measurement at observation node (Denver-30 Days)

Figure 5.37 is the plot of suction over the time period for Denver which was simulated for 30 days period of time.
Figure 5.38: Suction measurement at observation node (Denver-6Months)

Figure 5.38 is the plot of suction over the time period for Denver which was simulated for 6 months period of time.

Figure 5.39: Suction measurement at observation node (Denver-1 Year)
Figure 5.39 is the plot of suction over the time period for Denver which was simulated for 1 year period of time.

![Figure 5.39: Suction measurement at observation node (Denver-5 Year)](image)

Figure 5.40: Suction measurement at observation node (Denver-5 Year)

Figure 5.40 is the plot of suction over the time period for Denver which was simulated for 5 year period of time.

5.8.5 Influence of Water Content on Hydraulic Conductivity
The influence of water content on hydraulic conductivity is also observed in the HYDRUS for 1D and 2D model simulation. Figure 5.41 is the plot of hydraulic conductivity vs. theta for 1D modeling. It is observed from the plot that hydraulic conductivity has direct relationship with moisture content. With the increased water content (Theta), the hydraulic conductivity is found to be increased. But, moisture content and suction has found opposite relationship, higher the moisture content, lower will be the suction and vice versa. Similar kinds of results were also proposed in Mualem-van Genuchten model by Schaap, and Leij, (2000).
Figure 5.41: Hydraulic conductivity vs. water content for 1D modeling

The result for hydraulic conductivity vs. theta for 2D modeling is also presented in Figure 5.42 through Figure 5.47. Hydraulic conductivity is found decreased with the decreasing moisture content. Similar kind of result is obtained in all 2D simulation of varying time zone and climatic region.

Figure 5.42: Relation between conductivity and theta LA-30 Days (2D modeling)
Figure 5. 43: Relation between conductivity and theta Dnv-30 Days (2D modeling)

Figure 5.43 is the plot of hydraulic conductivity vs. volumetric water content representing Denver climatic region for 2D modeling which was simulated for 30 days period of time.

Figure 5. 44: Relation between conductivity and theta AZ-30 Days (2D modeling)

Figure 5.44 is the plot of hydraulic conductivity vs. volumetric water content representing AZ climatic region for 2D modeling which was simulated for 30 days period of time.
Figure 5.45: Relation between conductivity and theta LA-10 year (2D modeling)

Figure 5.45 is the plot of hydraulic conductivity vs. volumetric water content representing LA climatic region for 2D modeling which was simulated for 10 years period of time.

Figure 5.46: Relation between conductivity and theta Dnv-10 year (2D modeling)

Figure 5.46 is the plot of hydraulic conductivity vs. volumetric water content representing Denver climatic region for 2D modeling which was simulated for 10 years period of time.
Figure 5.47: Relation between conductivity and theta AZ-10 year (2D modeling)

Figure 5.47 is the plot of hydraulic conductivity vs. volumetric water content representing AZ climatic region for 2D modeling which was simulated for 10 years period of time. All the above plots of conductivity vs. theta showed that with the increase of water content (theta), the value of conductivity was also found increased. Similar kind of trend was observed in all climatic soil such as Louisiana, Denver and Arizona for wide range of simulation time period (for example, from 30 days to 10 years).

5.8.6 Influence of Head (Suction ) on Hydraulic Conductivity

The influence of suction (pressure head) on hydraulic conductivity was simulated in 1D and 2D model. Figure 5.48 is the plot of suction vs. conductivity. It was observed that with the increase of suction, hydraulic conductivity decreases dramatically. It is because; suction is influenced highly with water content, higher the suction resulted lower the water content. Reduction in water content resulted reduction in hydraulic conductivity of soil which is observed in the 1D modeling of HYDRUS simulation, which is shown in the Figures from 5.48 to Figure 5.50.
Figure 5.48: Hydraulic conductivity vs. suction (1D modeling)

Figure 5.48 is the plot of conductivity vs. suction for 1D modeling.

Figure 5.49: Log Hydraulic conductivity vs. suction (1D modeling)

Figure 5.49 is the plot of conductivity vs. suction for 1D modeling which is plotted in log scale.

 Similar kind of plot for conductivity vs. suction for 2D modeling was observed in three different climatic regions such as Louisiana, Denver and Arizona.
Similar kind of plot was developed by van Genuchten (1980) during the application of either Mulaem theory or Burdine theory to the Brook and Correy model of the soil water retention curve.

**5.8.7 Influence of Water Content on Water Capacity**

As we know that water capacity is the amount of water that a soil can store that is available for use of plants. The influence of water content on water capacity in the simulation of 1D modeling was observed as shown in Figure 5.51. Water capacity increases with the increase of water content, after reaching its optimum point, beyond that water capacity starts decreasing. The relation of water capacity is not that much important from the geotechnical engineering point of view rather it has more application to the agricultural/plant science. However, geotechnical engineers should be aware about this relationship and would be helpful while working in Geo-environmental field.
Figure 5.51: Water content vs water capacity

Figure 5.51 is the plot of water content vs. water capacity for soil SB and CL.

5.8.8 Influence of Head on Water Capacity

From the simulation, it was also observed the relationship between water capacity and suction.

Figure 5.35 is the plot of suction vs water capacity.

Figure 5.52: Plot of Log (h) vs water capacity

Figure 5.52 is the plot of water capacity vs. suction plotted in log scale.
CHAPTER SIX: CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE WORK

6.1 Introduction

The study of hydro mechanical behavior of unsaturated soil was performed in two ways: experimental investigation and numerical modeling using HYDRUS. The influence of moisture content on suction and shear modulus in a drying path of SWCC was observed. The relationship among suction, bulk density, stiffness and moisture variation was investigated. The result from the simulation of numerical modeling using HYDRUS in relation to the behavior of unsaturated soil with moisture variation was studied. The conclusions are presented below.

6.2 Conclusions

The hydro mechanical behavior of unsaturated soil was studied in two specimens: low plasticity clay (CL) and sand bentonite mixed soil (5% and 2% bentonite, referred here as SB). The particle size distribution, specific gravity, plasticity index, direct shear test and saturated hydraulic conductivity tests were conducted to classify and obtain the important properties of soil which were used in this research. The climatic data for HYDRUS simulation were obtained from average climatic data recorded in Louisiana, Denver and Arizona. Other soil properties required for simulation were obtained in the lab.

Conclusions of this research work are summarized as follows:

- The influence of moisture content on suction (SWCC) was determined on three specimen: bentonite-sand mixed soil (5% and 2% SB soil), and low plasticity clay (CL). All specimens showed that with the decrease of moisture content, suction was found to be increased. SB with higher percentage of bentonite (5%) was found higher water retention capacity than SB with 2% bentonite and CL specimen; it is because, bentonite exhibits more water retention properties.
• The influence of moisture content on soil stiffness was determined on CL sample. It was observed that if the moisture content on specimen decreased, stiffness of the soil was found to be increased.

• It is concluded that if the moisture content of the specimen is reduced due to various climatic conditions such as evaporation, transpiration etc, suction value on such specimen is found to be increased.

• The influence of suction on stiffness of soil was observed. It was found that if the suction on soil increased, the corresponding stiffness of soil was also found to be increased, but the increment was not in the same fashion.

• The influence of moisture content on hydraulic conductivity was simulated using HYDRUS software. From the simulation of 1D and 2D modeling, it was observed that with the decreased moisture content, hydraulic conductivity was also found to be decreased.

• The influence of suction on hydraulic conductivity was simulated both in 1D and 2D modeling of HYDRUS. From the simulation, it was found that if moisture content of the soil decreased, suction was found to be increased for all type of soil specimens SB (5% and 2%) and CL. With the increased suction, the hydraulic conductivity of soil was found to be decreased.

• The influence of environmental changes in suction of soil was studied. It is concluded that during the dry season, higher temperature resulted higher evaporation and transpiration rate which ultimately lead to decrease the moisture content of soil in the earth. The decreased moisture content resulted higher rate of suction pressure and consequently resulted reduction in hydraulic conductivity.
• The relationship between grain size distribution and soil hydraulic properties was also observed for all soils, SB and CL. From the grain size distribution curve, CL was observed to be better graded than SB. The value of D_{60} for SB was found to be higher (0.8) than the CL (0.6). The plot of SB was found to the lower part of SWCC family distribution chart, while CL was found to the higher and right side of the chart. Such prediction of SWCC based on grain size distribution was also verified in the result of HYDRUS 1D and 2D simulation.

• The study of stiffness in a drying part of SWCC was carried out. In SWCC test, by the application of suction pressure, water from the specimen was released there by reducing the moisture content in the specimen. It was observed that if the water content of soil decreased, bulk density was found to be decreased, but the dry density of soil was found to be increased. The shear wave velocity was found to be increased with the decreasing bulk density of the soil. And, shear modulus was found to be increased with decreasing bulk density. Shear modulus and suction both were found to be increased with the decreasing water content of the soil, but the trend of increasing of suction and shear modulus was not in the same fashion. Suction was found to be increased rapidly even with the little moisture reduction in the specimen. After certain level of moisture content, the rate of increment of suction was found at a slow rate. The opposite behavior was found in the case of shear modulus.

6.3 Future Recommendation

Followings are some of the recommendation proposed from this study:

• It was not possible to record and measure the deformation of specimen at each applied
suctions on Fredlund’s SWCC device. Therefore, it was not possible to measure the void ratio and hence the degree of saturation at successive suction. SWCC and shear wave velocity measurement were carried out side by side to monitor the stiffness of soil at each applied suction. While doing so, there could be chance of accumulation of errors while taking out the sample from the SWCC cell; it might not represent the true suction pressure while taking out the specimen from the SWCC cell for blender element test. So, if the SWCC cell can be modified in such a way that shear wave velocity can be measured simultaneously at each successive suction of SWCC in a single unit, would be attained more accurately and precisely for studying hydro-mechanical behavior of unsaturated soil.

- The study of unsaturated soil behavior is a complex job. It requires advanced triaxial apparatus so that most of the unsaturated soil test such as SWCC, volume change behavior, shear wave velocity measurement, shear behavior etc. can be experimented in a single unit so that the errors associated with each separate test could be minimized. Further studies for development of advanced unsaturated triaxial apparatus are proposed to monitor unsaturated soil behavior.

- The wetting part of SWCC was not investigated in this research because it was difficult to carry out experiment for wetting part on Fredlund’s device.
REFERENCES


VITA

Dev Raj Pokhrel was born in Nepal. He earned his undergraduate degree in Civil Engineering from Tribhuvan University, Nepal. After working several years in public and private sectors in the Civil Engineering industries based in Nepal, he started his M.S. studies in Environmental Management at the University of Findlay, Ohio in Spring 2005. In pursuit of higher studies in Civil and Environmental Engineering, he joined LSU in the Spring of 2008. He is a candidate for the Master of Science in Civil Engineering degree for Fall 2009.