EXPERIMENTAL STUDIES ON SHRINKAGE INDUCED PRESSURE MEASUREMENTS OF FOUR EXPANSIVE SOILS

by

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ABSTRACT

EXPERIMENTAL STUDIES ON SHRINKAGE INDUCED PRESSURE MEASUREMENTS OF FOUR EXPANSIVE SOILS

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Civil structures were damaged by expansive soils due to its large volumetric changes caused by moisture fluctuations. During wet and dry season, the ground heaves up and also shrinks noticeably. The significant amount of curling pressure which is induced by soil shrinkage could cause severe damage to the infrastructure. In the eastern and central Texas, pavements encounter severe cracking and premature loss of serviceability. The maintenance costs, in some cases, are greater than the initial construction costs. Residential dwell structures built on problematic expansive soils have often become distressed due to volume changes associated with seasonal moisture content variations. This thesis research was an attempt to develop a new technique to measure the shrinkage pressure inherently induced inside the matrix of expansive clays. This test is termed here as Shrinkage Induced Pressure (SIP) test.

Laboratory investigations were conducted on soils from different locations, Fort Worth, Dallas, and Paris. These soils are considered as highly plastic and expansive clays. Also, another soil from El Paso which is low expansive clay was studied. Laboratory tests were first conducted included basic soil properties, chemical test and engineering soil tests. Laboratory test results showed that soil from Paris site have more shrinkage potential related to the highest

value of volumetric shrinkage strain at OMC and Wet of OMC condition. Soils from Fort Worth, Dallas, and El Paso show moderate to lower shrinkage strain potentials, respectively.

Comparisons among the results at different orientation and moisture content conditions showed that SIP results especially at LL+10% and with the force sensor (FS) in horizontal orientation provided both repeatable and reliable results. Reliability of the measurement was assessed by evaluating their closeness with the Indirect Tensile Strength (IDT) test results. SIP results showed a good correlation with percents of Montmorillonite in soils which was evaluated using cationic exchange capacity (CEC) values. Volumetric shrinkage and swell behaviors showed similar patterns as SIP data. Overall, this thesis research showed the potential of the SIP measurements using the FS probe can provide repeatable and reliable shrinkage pressure measurements of soils samples prepared at high moisture contents.

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CHAPTER 1

INTRODUCTION

1.1 Introduction

The phenomena of swell and shrink related volume change movements of expansive soil have been a major concern for a long time and these volume changes cause extensive damage to infrastructure. Expansive or swelling soil is problematic due to its large volume changes caused by moisture fluctuations. The ground heaves up considerably during rainy season and shrinks during summer. These swell and shrink behaviors adversely affect the civil infrastructures built above the soil. In the United States, damages from swell and shrink related movements cost close to 6-11 billion dollars per year (Nuhfer et al., 1993).

According to Konrad and Ayad (1997), desiccation cracking forms a significant curling (lifting off) of the polygonal clay blocks. This type of cracking and its related damage to civil infrastructure is receiving increasing attention (Kodikara et al. 2004). Similar to swelling pressure, the significant amount of curling pressure induced by soil shrinkage termed in this thesis as Shrinkage Induced Pressure (SIP) could inflict severe damage to the structures. Hence, it is equally important to thoroughly understand and quantify the shrinkage induced pressure properties.

Kodikara et al. (2004) estimated shrinkage pressure using principles of unsaturated soil mechanics and non-linear elastic theory on numerical simulation models. Due to the difficulty in measuring and non-availability of sensor technology in estimating shrinkage pressures, direct measurement of the shrinkage induced pressure was not achieved. However, at present, due to advancements of technologies, a new sensor termed as ‘Force Sensor’ (FS), which can detect forces acting on a small area on a device, does have potential to measure internal shrinkage induced pressure in soils. Therefore, a testing methodology to directly measure shrinkage

induced pressure (SIP) is developed in this research program, which can possibly lead to new design approach and analysis of civil structures when they are constructed above the expansive soil.

In this thesis, both methodology of measuring shrinkage induced pressure and laboratory evaluation studies attempted on several expansive soils are presented. Also, equipments and sensors used, procedures and results are described in this thesis.

1.2 Research Objectives

This thesis presents a new technique in order to measure the shrinkage pressure inherently induced inside the matrix of expansive clay. The characteristic of expansive soil is the capability of shrink and swell when the soil experiences dry and wet conditions. The behavior of expansive soil can damage structures such as pavements or buildings. For better understanding of the causes of structure damage regarding to shrinkage pressure, this thesis research is attempted with the following specific objectives:

1. To develop shrinkage induced pressure methodology that can measure direct shrinkage induced pressure (SIP) values for all types of soils

2. To find the appropriate orientation of Force Sensor (FS) in providing repeatable measurements in the proposed Shrinkage Induced Pressure (SIP) test

3. To determine the appropriate moisture content for shrinkage induced pressure (SIP) test at four moisture contents (Liquid Limit +10%, Plastic Limit +10%, Wet of Optimum Moisture Content corresponds to 95% of optimum dry unit weight, and Optimum Moisture Content).

4. To comprehend the correlation between SIP results and influence of soil variables including PI (%),CEC values (meq/100gm), volumetric shrinkage strain (%), volumetric swell strain (%), and swell pressure (kPa)

5. To find a correlation between SIP values and tensile strength of soil samples by using Indirect Tensile Strength Tests (IDT) at initial and final condition for different moisture

contents. The results from this study should provide tools and recommendations for shrinkage pressure measurement which lead to development of better infrastructure design approaches.

1.3 Organization of the Thesis

This thesis consists of five chapters.

Chapter 1 provides introduction for the thesis, the significance of the thesis research, objectives of research, and organization of framework for completed research.

Chapter 2 presents a literature review on the behaviors of expansive soils, properties, shrinkage properties and swell properties.

Chapter 3 covers laboratory testing programs to determine soil properties and behavior of expansive soil samples. The soil used in the tests was expansive clay soil from Dallas, Fort Worth, Paris, and El Paso in Texas. The experimental program includes basic soil properties tests, chemical tests, and engineering tests. A summary of the laboratory procedures, equipments used and results are presented in this chapter. Also, the Indirect Tensile Strength Test (IDT) and its results are presented as these represent the tensile strength in soil.

Chapter 4 shows the details of Force Sensor (FS) and the SIP results. Also, different FS orientations and moisture contents were tested. The procedure of Shrinkage Induced Pressure (SIP) Test, calibration curve, influence of soils variables to SIP values, and correlation between SIP values and IDT results are also presented in this chapter.

Chapter 5 presents the important conclusions of Shrinkage Induced Pressure (SIP) test, Indirect Tensile Strength Test (IDT) results, and future recommendations.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

Natural expansive soil is a clayey soil normally found in numerous places worldwide. Expansive soils experience substantial volumetric changes due to moisture content changes from wet and dry seasonal periods. These volumetric changes cause swell and shrinkage movements in soils, which in turn will inflict severe damage to structures built above them (Nelson and Miller, 1992). Expansive clays include high plasticity or high PI clays, clays rich with montmorillonite clay minerals, and shales. It was noted that the expansive soils cause damages to structures, specifically light buildings and pavements that are much greater than the damages caused by other natural disasters like earthquakes and floods (Jones and Holtz, 1973). Various countries in the world, including the United States, Israel, India, South Africa, and Australia, have infrastructure damage problems caused by the movements of expansive soils. These damages are estimated to cost several billions of dollars annually (Nelson and Miller 1992).

Expansive soils owe their characteristics to the presence of swelling clay minerals. During the wet season, the clay minerals absorb water molecules and expand; conversely, as they dry they shrink, leaving large voids in the soil. Swelling clays can control the behavior of practically any type of soil if the percentage of clay is more than about 5 percents by weight (Manosuthikij, 2008). Soils with smectite clay minerals, such as montmorillonite, exhibit the highest swelling properties (Manosuthikij, 2008).

Also, expansive clay soils can be easily recognized in the dry season by the deep cracks in the ground surface as shown in the Figure 2.1. The zone of seasonal moisture content

fluctuations can extend from 3 to 15 ft deep which can result in wider cracks at surface as shown in the Figure 2.2. This creates cyclic shrink/swell behaviors in the upper portion of the soil column, and cracks can extend to greater depths than imagined by most engineers.

Figure 2.1 Expansive clay soils cracking in the dry season
(Rogers et al., 1985)

Many roads constructed on expansive clay subgrade especially in the east and central Texas, USA though over-designed, still experienced severe pavement cracking with short serviceability life periods. The costs of maintenance, in some cases, are even more than their

construction costs. Pavements or roads that are constructed on soft and complex soils have recurring maintenance difficulties (Chen, 1988).

Figure 2.2 Extensive Desiccation Cracking in Soils (Rogers et al., 1985)

Brown (1996) presented a state-of-the-art paper on the use of soil mechanics principles in pavement design. He reviewed the response of clays in the context of the requirements for design. The subgrade soils, in particular expansive soils, should be better considered for both during design and construction of the roads.

Total or differential soil movements caused by swell or shrinkage strains of expansive soils could cause extensive destruction and damage to the highways (Chen, 1988). Contrasting or differential movements produce large moments and shear forces in the structures, which lead to failure in both rigid and flexible pavements. The consequence of these forces not being accounted for in the rigid and flexible pavement design practices result in pavement cracking. These soil movements in highway environment are connected to subgrade moisture distinction conditions, which are influenced by the widening of the right of way, and surroundings of trees close to pavement systems. The latter condition will extract more moisture from the underneath pavements, resulting in the shrinkage problems in soils, thereby causing pavement distress. Other factors including lack of sufficient roadside ditches for drainage, and poor drainage problems around the pavements contribute to these problems.

Damages caused by the pavements include distortion and cracking of pavements in all directions as well as swell related bumps that bring about ride discomforts. The cracks developed in pavements will further allow intrusion of moisture into subsoils, which results in the deteriorating of subsoils and loss of base foundation to properly support pavements. Both magnitude and extent of damages to pavement structures can be extensive, impairing the usefulness of the roads, and fundamentally making them uncomfortable for riding conditions (Manosuthikij, 2008). Maintenance and repairs can be extensive that frequently result in excessive service costs (Manosuthikij, 2008).

The factors influencing the shrink-swell potential of a soil can be divided in three different groups: soil characteristics (clay mineral, plasticity, soil suction and dry density), environmental factors such as climate, groundwater, vegetation and drainage (Nelson and Miller, 1992). The next section describes these in detail with focus on shrinkage characteristics.

2.2 Shrinkage and Swell Properties Variations

2.2.1. Shrinkage Induced Soil Properties

In the present-day practice, expansive soils are mainly characterized based on swell characterization tests. Shrinkage tests are limitedly used in practice. However, it is well known that the shrinkage cracking of expansive soils in the dry environments could lead to enlarged soil heaving in wet conditions. It is because surficial shrinkage cracks will allow much more moisture access into the underlying expansive soils and this result in further heaving. Poor (1974) noted expansive soils that are located in regions where prolonged hot dry periods are followed by cooler and wet periods would cause maximum distress to pavements and structures. Also, Wray and Ellepola (1994) described that large lateral stresses are anticipated when the high PI clays are shrinking. Hence, when characterizing expansive subgrades, it is important to recognize and consider volumetric shrinkage strain potentials and shrinkage induced pressures of soils along with their swell properties.

Researchers and practitioners presently use linear shrinkage strain and Atterberg Limit tests to measure and interpret shrinkage strain potential or cracking behavior of soils. These measurement methods are inefficient since they test low amounts of soils, measure linear strains in rigid wall boxes that restrain warping movements in soils, and they do not address or suggest compaction moisture content levels in the field.

Due to limitations in the linear shrinkage bar test, researchers advised a new test method. A cylindrical compacted soil specimen was prepared and subjected to drying process and then the dimensions of soil samples are measured to determine their volumetric, axial and radial shrinkage strains utilizing digital imaging technology. This test offers numerous advantages over conventional linear shrinkage bar test such as reducing obstruction of boundary conditions on shrinkage, allowing larger amount of soil being tested, and simulating the compaction states of moisture content - dry density conditions. This method was recently

published in an ASTM geotechnical testing journal (Puppala et al., 2004), signifying the importance of this method being accepted by the researchers and practitioners.

Linear shrinkage bar test was conducted to balance the volumetric shrinkage properties and both these results were used to develop correlations between both shrinkage strains. Details of these procedures are presented in the chapter 3.

2.2.1.1 Soil Curling

Soil curling, or lifting off, can occur when soil is dried out in the summer. This is usually a direct result of differential shrinkage. When soil, remarkably expansive soil, is dried out in summer, not only shrinkage and desiccation cracks appear but also soil curling (lifting off) happening as well as presented in Figure 2.3 a. Soil curling is generally affected by the differential shrinkage strain rates occurring down the soil profile during desiccation progression (Kodikara et al., 2004). The ideal curling action indicated on a soil layer as shown in Figure 2.3 b. If the shrinkage strain rate of soil at the top is more than at the bottom, curling warp will lift off at the edges. Additionally, if the strain rate at the bottom is more than at the top, the curling will lift off at the middle (Kodikara et al., 2004).

(a) (b)

Figure 2.3 Example of Soil Curling (a) Curling at the Edges and
(b) Soil Curling on soil layer

Degree of curling is based on the difference of the strain rates between the top and bottom of soil layer and also soil modulus of elasticity during shrinkage process. Though, the crack would take place and relegate the curling deformation (stress relaxation) if an unsteady shrinkage induced stress caused by differential strain rates is greater than the tensile stress of the soil (at that particular moisture content) (Kodikara et al., 2004). As stated before, soil shrinkage and subsequent swelling is significant to characterize since it can have an effect on the strength of the light structures, e. g., pavements and residential structures as shown in the Figure 2.4.

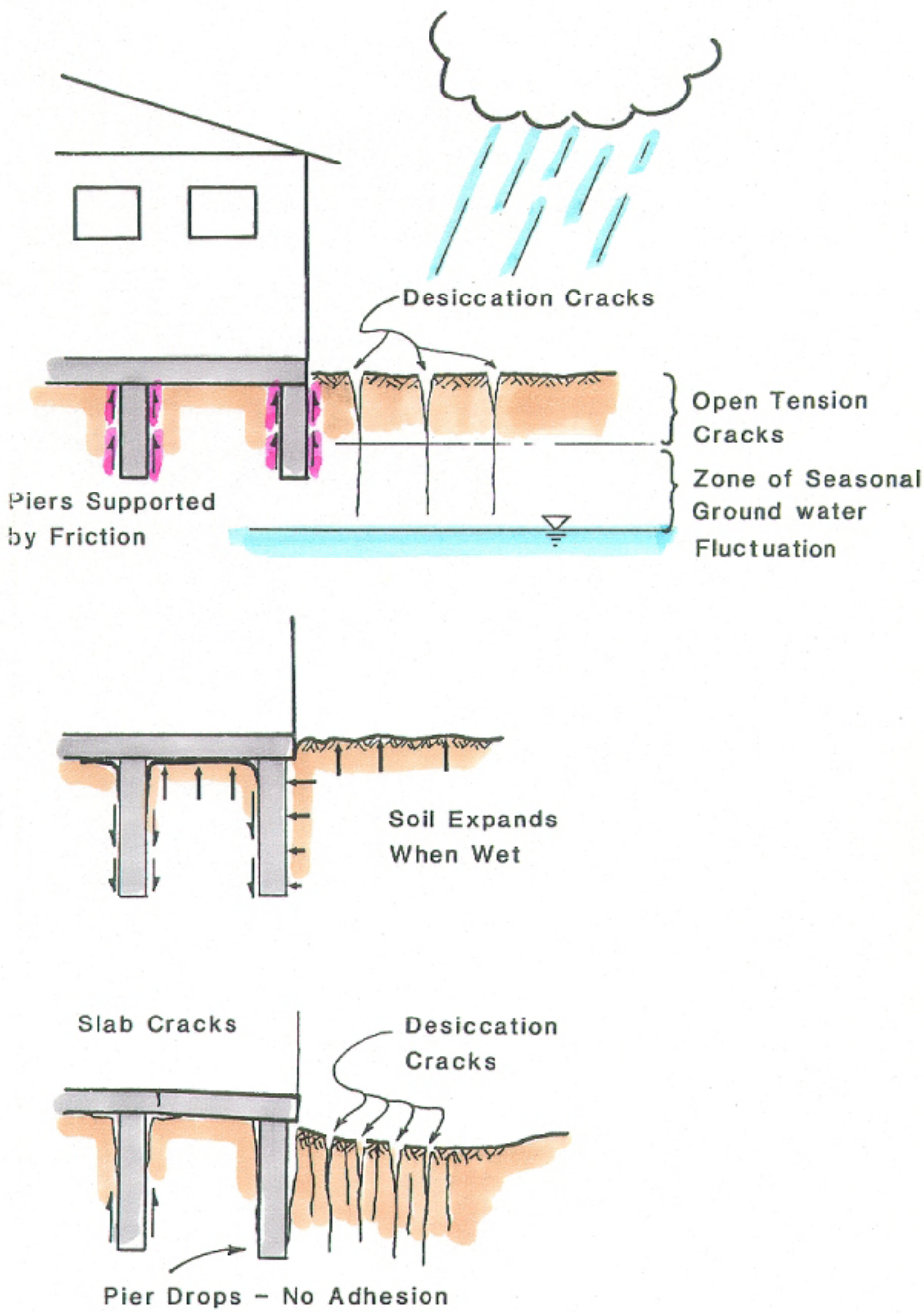


Figure 2.4 Soil swelling and shrinkage movements around light weight structures (Rogers et al., 1985)

2.2.1.2 Shrinkage Pressures

The volume-mass relationships between soil solids, water, and air phases are practical properties in engineering area (Fredlund and Rahardjo, 1993). The shrinkage of soil is associated the structure and mineral in the clay soil, which can lead to cracks and shear in soil. When the soil, especially expansive soil, is dried out along a summer season, the soil shrinks and contracts due to loss of moisture content around the molecular structure. Normally, shrinkage of clayey soil has two main characteristics. The first one is soil contraction then cracks and voids form nearby the shrunk soil. In this case, cracking can occur at any place as the surface of clay soil exposes to the atmosphere. For the second shrinkage characteristics, when the soil is contracted due to surrounding boundary conditions such as wind, heat and humidity conditions. The pressure from soil contraction, called here as shrinkage pressure, can lift the soil, which can lead to structural damage. Schematic of SIP affects a slab on foundation as shown in the Figure 2.5. In this case, the lifted soil (contracting soil) is the result of lateral shrinkage due to the presence of cracks. Consequently, the shrinkage soils curl upwards and move in upward direction. When restrained, the shrinkage soil moves in upward direction, which often results in the distress of infrastructure.

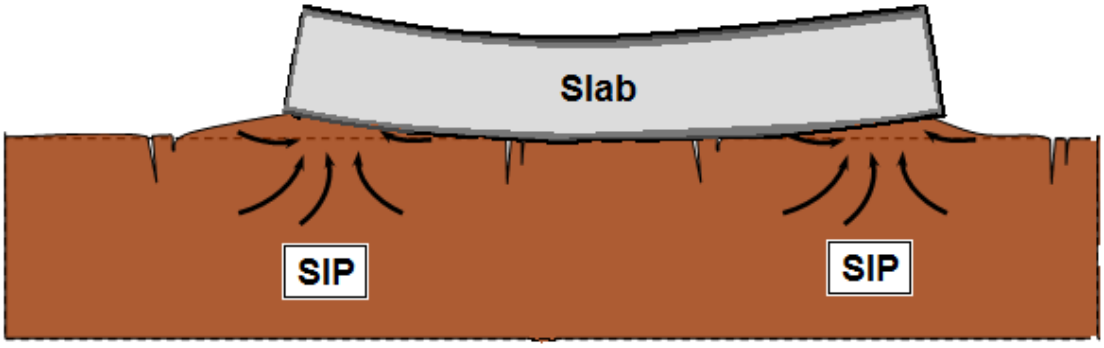


Figure 2.5 Schematic of SIP affects a slab on foundation

From Figure 2.5, the shrinkage pressure can be assumed to be isotropic from surrounding soil particles. As a result, the shrinkage pressure can be assumed to induce uplift pressure of the soil. Thus, a test procedure is developed here by measuring a compression force directly by placing the sensor in a liquid soil mass and then subject soil to contraction or drying. During contraction process, soil exerted shrinkage induced compression forces on the sensor which was captured by the data acquisition system.

While the soil shrinks at a certain pressure, tensile strength of the soil will resist the shrinking action. As long as the tensile strength is more than the shrinkage pressure, soil desiccation would not occur. On the other hand, while the soil is further dried up, their shrinkage pressure will continue to increase. Eventually, the soil cracking will appear once the tensile strength is less than the shrinkage induced pressure.

Numerous researchers have been attempting to measure both shrinkage displacements in the laboratory and in the simulation software. For example, Kodikara et al. (2004) demonstrated modeling of curling in desiccation clay both in the laboratory and simulation models. Chakrabarti and Kodikara (2006) measured specimen's uplifting shrinkage strain from laboratory studies by simulating the finite element program to model this shrinkage pressure. Due to the fact that soil is not rigid in natural state and can be in liquid state unlike concrete or steel specimens, it is difficult to directly measure the shrinkage pressure. The Indirect Tensile Strength (IDT) test has been used to calculate the tensile force generated inside the specimen, which is the closest attempt in the civil engineering field to measure the tensile strength of the soil specimen which indirectly represents the shrinkage induced pressure.

New technologies nowadays can create innovative sensors. Force Sensing Resistors or Force Sensors (FS) is one of them and it consists of a polymer thick film device which exhibits a decrease in resistance with an increase in the force applied to the active area (which is an area of an FS device that responds to normal force with a decrease in resistance). This sensor is

typically used in human touch control of electronic devices. In this thesis research, the use of this sensor to measure shrinkage induced pressure is proposed and evaluated.

2.2.2. Swell Strain Properties

In standard engineering practice, laboratory swell tests were operated in Oedometer type apparatus with low seating pressures. Swell properties such as swell strain and swell induced pressure of expansive soils depends on three factors: (i) soil properties such as compaction or natural moisture content variation, dry density, and plasticity index, (ii) environmental aspects including temperature and humidity conditions and (iii) natural overwhelm pressure conditions. Because of the impact of these factors, several expansive soil characterization methods were developed in the literature (Puppala et al., 2004). These swell methods are primarily based on:

(1) Swell Strain and Pressure Measurements

(2) Plasticity Properties

(3) Other correlations using activity and compaction properties

These characterization methods often create problems for practitioners since swell strain measurements and swell correlations established for certain soil conditions are not suitable for other conditions. Rao and Smart (1980) considered four different correlations using ten different soils and the results showed that none of the correlations were able to match the values. Snethen (1984) reported similar conclusions by testing twenty highly expansive soils based on 17 correlations published in the literature. Regardless of these limitations, the correlations can still be used and provided that they are comprehensively and independently verified for the conditions encountered in the state of Texas.

Formulated by the researchers at The University of Texas at Arlington (UTA), a three-dimensional free swell test not only provided a reasonable representation of the maximum volumetric swell potential and also yielded the reliable and repeatable test results (Punthutaecha et al., 2006). This test was also conducted to examine the maximum vertical,

radial and volumetric swell potentials for both soil types. In the testing, a sample of 4.1-in. diameter and 4.5-in. height was placed between two porous stones at the top and bottom, covered by a rubber membrane, fully inundated with water at both ends and monitored for the vertical and radial swell movement until there was no further significant swell as shown in the Figure 2.6.

Figure 2.6 Three - Dimensional free swell testing setup

2.2.3. Swell Pressure

The swell pressure of expansive soils is commonly determined by restraining the soil specimen from undergoing any volume changes under fully soaked conditions (under constant volume method). The surcharge loads added to the soil specimen to keep it under constant

volume conditions are determined (Figure 2.7). The swell pressure value is estimated from the information of surcharge loads and sample dimensions.

There are different test methods that can be used to measure swell pressures:

- Conventional consolidation test procedure which yields an upper bound value.
- Method of equilibrium void ratio at different consolidation pressures, which gives the least swell pressure.
- Constant volume method (CV method), which yields an intermediate value. Further details on these test methods are available in Ohri (2003).

Figure 2.7 Swell Test Setup

Soils with swell pressures below 0.4 ksf are regarded as low swelling soils. Soils with swell pressures 7.9 ksf or higher are classified as high swelling soils. Soils with swelling pressures values higher than 40.3 ksf are occasionally encountered in real field conditions

(Peck et al., 1974). The characterization of swell pressure also depends on the overburden pressure conditions from the infrastructure. The swelling pressure of expansive soils decreases with an increase in the overburden pressure and there will be no heave in expansive soils if the overburden pressure is equal to the swell pressure of the expansive soil.

Hence, it is important to consider swell pressures in the design of structures including pavements. For example, while evaluating the stability of rigid pavement systems, it is important to take into account the swell pressures if an expansive soil is encountered as a foundation material. Details of these procedures are presented in the chapter 3.

2.3 Summary

An effort is made here in this chapter to reexamine the previous research studies on expansive soils in different matters for example sources of expansive behaviors, volumetric changes properties and tensile behaviors of the soils. Swell strain behavior and swell pressure properties are well established whereas shrinkage strain behavior is well known. However, shrinkage induced pressure (SIP) measurements is relatively not well researched primarily due to the difficulty in capturing the inherent shrinkage movements in a soil. However, new technological advancements and in particular pressure measurements using tactile and other types of sensors have provided a window of opportunity to measure these pressures. Hence an attempt is made in this research to develop one such measurement technology by using force resistor sensor technology for direct estimation of shrinkage induced pressure. Details of the test setup, calibration, sensor technology are presented in Chapter 4. Additionally, comprehensive laboratory experiments and results as well as analyses are also presented.

CHAPTER 3

LABORATORY STUDIES

For the complete understanding of shrinkage induced pressure in expansive soils, several soils were collected and characterized in this research. Three soil specimens from Fort Worth, Paris, and Dallas districts from Texas with high plasticity index (PI) values were sampled and used in the experimental program. Soil samples from El Paso district with low plasticity index (PI) value are also studied to evaluate shrinkage pressures of low expansive soils. This chapter provides a summary of the laboratory tests, equipments used and test results obtained on these soils. The experimental testing program includes basic soil properties tests, chemical tests, and engineering tests on all four soils from these sites. Also, indirect tensile strength tests (IDT) and their results which are presented in this chapter and these results are later used to correlate with shrinkage pressure measurements.

3.1 Basic Soil Properties

An important part of geotechnical engineering starts with basic soil properties testing and these tests provide physical and plasticity properties. The tests consist of specific gravity test, sieve analysis, hydrometer test, Atterberg limits, and standard Proctor tests. All these tests were performed to measure the basic soil properties. The descriptions of these tests and procedures are presented below.

3.1.1. Sieve Analysis, Hydrometer Tests, and Specific Gravity

For sieve analysis, the basic element of soil classification system is the determination of the amount and distribution of the grain sizes in test materials was decided by using ASTM D 442-98 (2000) procedure (Day, 2001). This sieve analysis method is performed on dry soil particle and also followed the procedure to determine the amount of soils finer than the No. 200 sieve. For hydrometer analysis, the particle distribution for finer particle sizes such as silt and

clay analysis was performed using hydrometer analyses base on Strokes' law (Day, 2001). The idea of the test is that larger soil particle will settle faster than smaller soil particles in the distilled water.

According to ASTM D 5550 (2000), the specific gravity of soil is defined as the ratio of the mass in air of dry solids to the volume of the solids at the same temperature which are given volume of solid or liquid to the mass of an equal volume of water for testing materials.

3.1.2. Atterberg Limit Tests

Atterberg limits are defined as the water contents corresponding to the different silts and clay soil behavior conditions and represent plasticity properties (Day, 2001). Atterberg limit tests were performed to define the properties of soil which is related to consistency of the soil including liquid limit (LL), plastic limit (PL) and shrinkage limit (SL) and these are essential to show a relationship between the shrink-swell potential of the soils with their respective plasticity indices. Upon addition of water, the state of soil changes from dry, semi-solid, plastic and finally liquid states.

According to Lambe and Whitman (2000), Atterberg limits refer to the water contents of the soil at the border lines of various states namely shrinkage (SL), plastic (PL) and liquid (LL) limits, respectively. For this explanation, the liquid limit or LL is defined as the water content at which the soil flows. PL is determined as the water content at which the soil starts to experience the cracking when rolled into a thread approximately 1/8-inch diameter thread corresponding to the behavior change between the plastic states and semisolid states.

The plasticity index (PI) is the difference value between LL and PL values and this index characterizes plasticity nature of the soil. Therefore, representative soil samples from four sites are prepared following the above mentioned procedure and these soils were subjected to Atterberg limit tests to determine LL and PL as per ASTM D 4318-98, 2000.

3.1.3. Standard Proctor Compaction Tests

The testing procedure for compaction relationships is followed to determine the compaction moisture content and dry unit weight relationships of the soils and this method is 'Standard Proctor' test as referred to in the ASTM D 698-98, 2000. This test was used to determine the optimum moisture content of the soil corresponding to a maximum dry unit weight condition. Samples exhibiting high compaction unit weight are best in supporting civil infrastructure since the void spaces and settlements of these soils will be less. This method was performed on all four soils to develop compaction relationships which are used to study engineering properties.

3.2 Chemical Tests

The amount and type of clay minerals have a significant effect on the properties of soil such as swelling and shrinkage conditions (Day, 2001). The three most common clay minerals present in natural soils are Kaolinite, Montmorillonite, and Illite. Therefore, clay mineral in a soil can be identified by chemical test which is described as follow. The Cation Exchange Capacity (CEC) is used here as this value indirectly accounts for the clay mineralogy.

The Cation Exchange Capacity (CEC) is the chemical test to find the quantity of exchangeable cations which are needed to balance the negative charge on the surface of the clay particles. The CEC is presented in milliequivalents per 100 grams of dry. In the first step of test procedure, excess salts in the soil are removed and absorbed cations are replaced by saturating the soil exchange sites with a know species. The amount of the known cations needed to saturate the exchange sites is analytically detected (Nelson and Miller, 1992).

The determination of CEC needs detailed and accurate testing procedures that are not usually done in most soil mechanics laboratories. On the other hand, this test is regularly performed in many agricultural soil laboratories and is an inexpensive test. Typical CEC values of three basic clay minerals are presented in Table 3.1 below. The higher CEC values present

the greater surface activity. Consequently, the CEC increase in soils will indicate swell and shrink potentials of a soil.

Table 3.1 Typical Values of CEC for Three Basic Clay Minerals of Soil (Mitchell, 1976)

Clay Mineral	**CEC (meq/100 gm)**
Kaolinite	3-15
Illite	10-40
Montmorillonite	80-150

3.3 Engineering Tests

This section discusses about the determination of expansive soil movement by testing soil specimens. Engineering tests in this research consist of volumetric shrinkage test, three-dimensional free swell test, and one dimensional pressure swell test.

3.3.1. Volumetric Shrinkage Test

This test method, established at the University of Texas at Arlington (UTA), was developed due to the limitation of linear shrinkage bar test. This test is performed by drying the cylindrical compacted soil specimens and measuring the volumetric, axial and radial shrinkage strains, which this shrinkage test method was published in ASTM geotechnical testing journal (Puppala et al., 2004). To complement the volumetric shrinkage properties and develop correlations between volumetric and linear shrinkage strains, linear shrinkage bar test was also performed. Details and description of these procedures are discussed as below.

According to this geotechnical testing method presented in Puppala et al. (2004), initial compaction conditions and tests were conducted based on three different moisture contents which are optimum moisture content, wet of optimum moisture content, and dry of optimum moisture content. Specimens for each condition were prepared by mixing the dry clay with proper amount of water to prepare close to the designed water contents. After that, the soil specimens were compacted in 2.26 in. diameter and 5 in. height mold as presented in Figure

3.1a, and measured the initial height of the specimen. The soil specimens were cured in the mold at room temperature for 12 hours. Then, the specimens were transferred to an oven at temperature at 220℉ for 24 hours (Figure 3.1, b) to make sure the soil completely dried. The average height and diameter of the shrunk soil specimen are manually measured to determine the decrease in the total volume of soil specimens due to loss of moisture content in soil specimen from initial moisture content condition.

(a) (b)

Figure 3.1 Soil Specimen (a) Before Oven drying
(b) After Oven drying at a temperature of 220℉ for 24 hours

When compared to conventional linear shrinkage bar test, this new 3-D volumetric shrinkage test provides some advantages such as the decrease interference of boundary conditions on shrinkage, larger amount of soil being tested, and simulate compaction states of moisture content - dry density conditions more effectively.

3.3.2. Three-Dimensional Free Swell Testing

To study the expansive soil behavior related to the maximum vertical, radial and volumetric swell potentials of soil, three-dimensional Free Swell testing was conducted (Punthutaecha et al., 2006). Soil specimens of 4.0 in. diameter and 4.6 in. height were prepared at three different moisture conditions, optimum moisture content, Dry of optimum moisture content, and wet of optimum moisture content as shown in the Figure 3.2.

Figure 3.2 Test Setup of Three- Dimensional Free Swell Test
(Dry of OMC, OMC, and Wet of OMC (From left to right))

A soil specimen wrapped in a rubber membrane was placed between two porous stones as presented in Figure 3.3, and placed in to the chamber and then the specimen was subjected to soaking by inundating it with water from both ends. After that, the specimen was monitored for the vertical and radial swell movement at the times of keeping the record by using Dial gauge and PI tape until there was no further significant movement of soil specimen.

Figure 3.3 Three-Dimensional Free Swell Test Setup

To investigate the maximum vertical, diametric and volumetric swell potentials for expansive soil; the three-dimensional free swell test was conducted. It is noted that all tests should be carried out at room temperature and three identical soil specimens should be used for each variable condition as presented in Figure 3.2. Test results obtained from the three-dimension free swell test are typically expressed as a function of swelling time. The results of the swelling strains for three compaction moisture content conditions of the four soils are shown in Table 3.7. Punthutaecha et al. (2006) noted that the three-dimensional free swell test accommodated a reasonable representation of the soil and its maximum volumetric swell potential.

3.3.3. Swell Pressure Test

According to ASTM D4546-96 (2000), the constant swell pressure test was conducted to define the amount of load that should be applied over the expansive soil to resist any volume change in vertical direction. For soil preparation, the soil specimens were compacted at 3 different moisture contents such as optimum moisture content, Dry of optimum moisture content, and wet of optimum moisture content. The specimen diameter divided by the height of specimen should not less than 2.5 (Day, 2010). Thus, if the dimension of the soil specimen is 2.5 in., the thickness is 1.0 in. The specimens have to place in the chamber to make sure that the specimens were fully soaked in the standard consolidation setup.

The consolidation test set up in present study is shown in the Figure 3.4. Two porous stones were placed at the top and bottom of the specimens. A dial gauge was used to monitor changes in specimen's movement. Loads were added in order to maintain original position. Testing was stopped when the dial gauge showed no swell movement significantly for more than two days. The amount of total load applied to the soil specimen was used to calculate its swell pressure. This test is commonly used to identify the maximum swelling pressure of the soil specimen at which no volume change is occurred.

Figure 3.4 Modified Consolidation tests setup

3.4 Indirect Tensile Strength Tests (IDT)

Indirect Tensile Strength (IDT) test is used in this research to measure the tensile strength of soil or rock specimens as presented in the Figure 3.5. The tensile strength is obtained by performing a direct uniaxial tensile test which induces tension directly in pulling the specimen apart. However, direct tensile test on soils is difficult to perform and expensive for regular applications (ASTM D3967- 95a, 2001). The indirect tensile strength or splitting tensile strength test is hence chosen. In this research, the tensile strength of soils is compared with the related shrinkage pressure results. The indirect tensile strength is a representative of the force that breaks the physical bond between soil particles, while the shrinkage induced pressure measures the internal force exerted by the particles that are subjected to drying.

This IDT test was conducted following ASTM D3976, Standard Test method for Indirect Tensile Strength of Intact Rock Core Specimen. For this research, even though the specimen

used in this test is clay, characteristics of tensile force acting to the specimen are still the same as the rock materials.

Figure 3.5 The Indirect Tensile Strength Testing on Dry Soil Specimen

In specimen preparation, the shape of specimens should be a circular disk like specimen, more like a short cylinder. All the specimens are prepared at the same water content and cured in the oven until the specimens are completely dry. Dimensions of specimens vary with the degree of swelling of expansive soils. In addition, the specimens used in this test must have no cracks and should have a ratio of thickness to diameter (t/D) between 0.2 – 0.75 as per ASTM D3976. Prior to the IDT being performed, the side of specimen was prepared such that they should have a smooth surface. Therefore, trimming the side of specimens is needed for unsmooth surfaces. The dry specimens used for this test are shown in the Figure 3.6. Tests were conducted to determine the load (P) that induced a cracking along the vertical lining as shown in the figure. Intact specimens that are dry will provide high tensile strength that is comparable to shrinkage induced pressures that will cause shrinkage cracking.

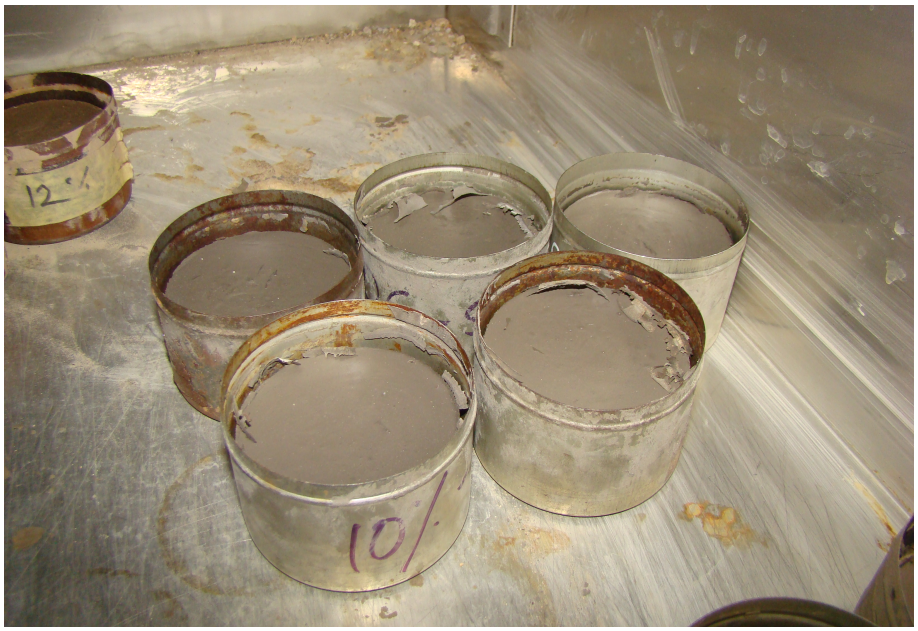


Figure 3.6 Dry Specimens Used in IDT Testing

According to ASTM D3967 (2003), the indirect tensile strength of the specimen can be calculated as per the following Equation:

$$\sigma_t = 2P/\pi LD \qquad (1)$$

Where:

σ_t	=	indirect tensile strength, MPa (psi),
P	=	maximum applied load indicated by testing machine, N (or lbf),
L	=	thickness of specimen, mm (or in.), and
D	=	diameter of the specimen, mm (or in.).

3.5 Laboratory Test Results

3.5.1. Basic Soil Properties Results

Based on the basic soil properties measured in this research, Table 3.2 presents a summary of various physical characteristics of all four soils. Soil samples used in this research were collected from four sites in Fort Worth, Paris, Dallas and El Paso, Texas.

Table 3.2 Basic Soil Properties from Four Sites

Property	**Soil Types**			
	Fort Worth	**Dallas**	**Paris**	**El Paso**
Passing #40(%)	100	100	100	100
Passing #200(%)	85	80	81	88
Specific Gravity	2.7	2.7	2.7	2.7
Liquid Limit (LL, %)	61	77	60	30
Plastic Limit (PL, %)	24	18	23	14
Plastic Index (PI, %)	37	59	37	16
AASHTO Classification	A-7-6	A-7-6	A-7-6	A-6
USCS Classification	CH	CH	CH	CL

Based on the American Association of State Highway and Transportation Officials (ASSHTO) Soil Classification system, Fort Worth, Dallas and Paris soils are classified as A-7-6 and CH as per Unified Soil Classification System (USCS), which means they contain clays of high plasticity with the PI more than 30. Normally, soils that exhibit plastic behavior over wide ranges of moisture content and that have high liquid limits have greater potential for swelling and shrinking. According to expansive soil characterization report given in Table 3.3, Soils from Fort Worth, Dallas and Paris are considered as high swelling and shrinkage soils. El Paso soil is considered as one that exhibits less swelling potential due to low amounts of expansive clay minerals.

Table 3.3 Expansive Soil Classification Based on Plasticity Index (Chen, 1988)

Plasticity Index	**Swelling Potential**
0-15	Low
10-35	Medium
20-55	High
35 and above	Very High

3.5.2. Cation Exchange Capacity (CEC) and Montmorillonite (%) Results

Due to the chemical characteristic, chemical analysis was also introduced in this research in order to justify the causes of expansive soil problems especially swell/shrink behaviors. Tests related soil chemical properties information based on cation exchange capacity was also performed. Cation Exchange Capacity (CEC) is the quantity of exchangeable cations required to balance the negative charge on the surface of clay particles. High CEC values indicate a high surface activity of the clays (Nelson and Miller, 1992). The CEC is desirable to have a comprehensive understanding for better characterization and understanding of the soil mineralogy as well as swell/ shrinkage related characteristics of the soils. In general, the swell potential increases as the CEC increases.

According to Nelson and Miller (1992), clay minerals which have high swell and shrink properties are montmorillonites and some with Illite minerals. The Kaolinite is usually not identified as an expansive soil. Illite can be expansive but generally do not pose significant problem. The test results for cation exchange capacity for all four soil types are shown in Table 3.4. From Table 3.4, Fort Worth, Paris, and Dallas clays have high CEC values which may indicate high amounts of Montmorillonite clay minerals. El Paso clay, low Plasticity Index clay, create noticeably low amount of Montmorillonite minerals.

Table 3.4 Cation Exchange Capacity (CEC) and Montmorillonite (%) Results

Property	Soil Types			
	Fort Worth	Dallas	Paris	El Paso
CEC (meq/100 gm)	117	93.3	133	57
Montmorillonite (%)*	50	41	70	8

* - Based on Model by Chittoori (2008)

3.5.3. Standard Compaction Tests

The water content of soil at which soil is compacted to the maximum dry density condition is known as the optimum moisture content (OMC) and this value is estimated from the standard compaction test procedures. Moreover, this Proctor test data was used to estimate water contents at 95% of maximum dry density conditions, namely Dry of OMC and Wet of OMC which are described in the Figure 3.7 below. Standard Compaction Test or Proctor Compaction tests were performed to establish compaction relationships between moisture content and density of soil.

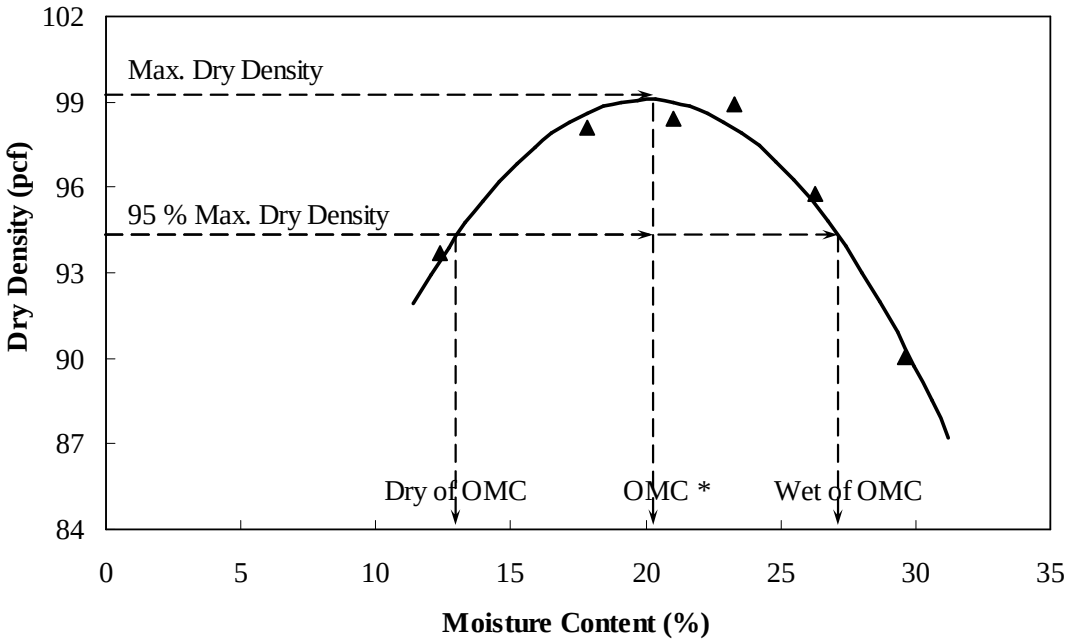


Figure 3.7 Typical Standard Proctor Curve

According to Table 3.5, the results should be recognized that all three moisture content conditions based on Proctor compaction tests consist of Wet of OMC, OMC and Dry of OMC and these conditions are important reference moisture contents for performing the present engineering tests. Other moisture conditions above liquid and plastic limits are also considered. Dallas clayey soil exhibits the highest dry density value among high PI clay group whereas El Paso clays show the highest dry density among four soils site location which indicated better quality of this soil in supporting civil infrastructure.

Table 3.5 Proctor Density Tests Results

Property	**Moisture Condition**	**Soil Types**			
		Fort Worth	**Dallas**	**Paris**	**El Paso**
Moisture Content (%)	**Wet of OMC**	33.0	22.8	33.0	20.0
	OMC	24.0	16.8	23.0	16.5
	Dry of OMC	15.1	10.91	13.0	13.0
Dry Density (pcf)	**Wet of OMC**	86.9	97.76	94.1	106.4
	OMC	91.5	102.9	99.1	112.0
	Dry of OMC	86.9	97.76	94.1	106.4

3.5.4. Volumetric Shrinkage Strain Results

According to the objective of this thesis, Volumetric Shrinkage Strain Test was used for the prediction of soil shrinkage behavior. Test results are expressed in percent values in term of volumetric shrinkage strain. Shrinkage strains in radial and vertical directions were first recorded and used to determine volumetric shrinkage strains which is consist of vertical, radial, and volumetric parameters. Volumetric shrinkage strain test is a better test method than linear shrinkage strain test since volumetric strain was evaluated on tests on soil samples of considerable volume. These results are later correlated to shrinkage pressure test results in Chapter 4. Average shrinkage strain test results are presented in Table 3.6 below.

Table 3.6 Volumetric Shrinkage Strain Results

Moisture Condition	Parameter	Shrinkage Strain (%)			
		Fort Worth	Dallas	Paris	El Paso
Wet of OMC	Vertical	8.43	4.37	8.78	4.28
	Radial	8.87	2.19	9.45	3.55
	Volumetric	23.59	8.51	24.66	10.97
OMC	Vertical	5.29	3.01	4.92	1.86
	Radial	2.47	1.97	4.91	1.77
	Volumetric	12.51	6.79	14.04	5.30
Dry of OMC	Vertical	2.17	1.93	2.41	0.36
	Radial	0.97	1.70	1.46	1.50
	Volumetric	5.22	5.24	6.15	3.33

These results were used in corporate with soil shrinkage prediction for soil behavior in next chapters. As a result, the highest value of volumetric shrinkage strain from Paris site which is considered to have high shrinkage strain behavior. Other clayey soils from Fort Worth, Dallas, and El Paso, respectively showed the rest of the order of shrinkage behavior with low shrinkage potentials were measured for El Paso soil.

3.5.5. Three-Dimensional Free Swell Testing

In this Three-Dimension Free Swell Test, three soil samples from four sites for each moisture condition were compacted at three different compaction moisture content conditions (Wet of OMC, OMC, and Dry of OMC) with their corresponding dry densities from a standard Proctor test results as presented in Table 3.5. Table 3.7 presents the average swell strain of the test results from three-dimensional free swell test.

Table 3.7 Volumetric Swell Strain Test Results

Moisture Condition	Parameter	Swell Strain (%)			
		Fort Worth	Dallas	Paris	El Paso
Wet of OMC	Vertical	3.63	6.90	1.43	1.47
	Radial	1.95	2.69	1.51	0.81
	Volumetric	7.71	7.28	7.50	3.11
OMC	Vertical	9.28	10.29	7.37	2.43
	Radial	3.46	3.71	3.60	1.27
	Volumetric	16.97	11.07	15.25	5.04
Dry of OMC	Vertical	14.13	17.44	14.35	4.51
	Radial	4.26	4.63	5.36	1.76
	Volumetric	24.07	19.09	26.93	8.23

At the first eight hours, majority of swell strains in soils was monitored and subsequent swell strains values were continuously recorded until no swell movement was observed. The examples of typical swell characteristic graphs for Paris soils are shown in the Figure 3.8 to 3.10.

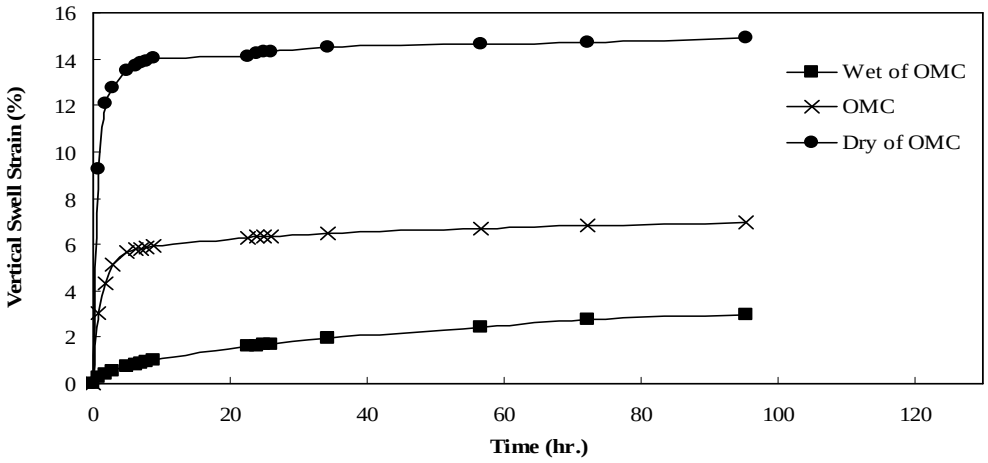


Figure 3.8 Vertical Swell Strains Results for Paris Soils at Three Different Moisture Contents

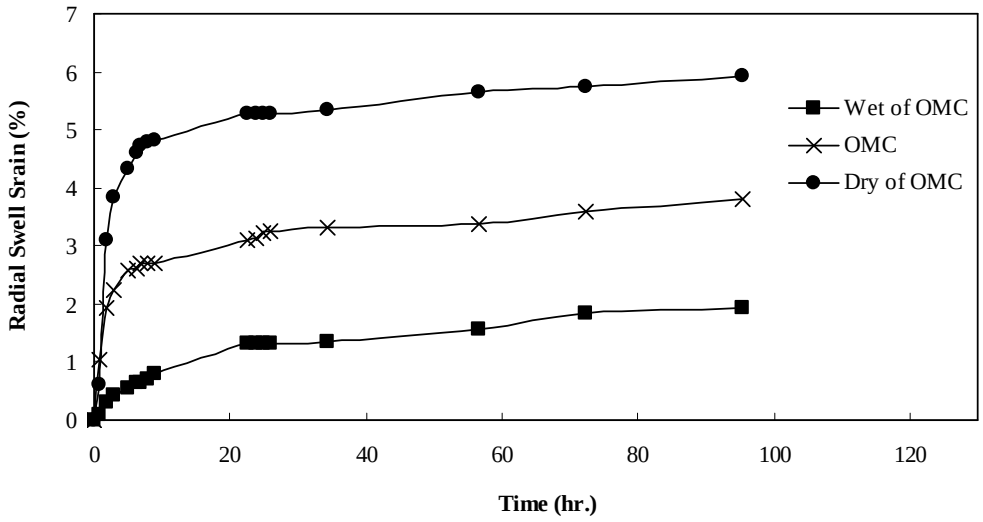


Figure 3.9 Typical Radial Swell Strains Results for Paris Soils at Three Different Moisture Contents

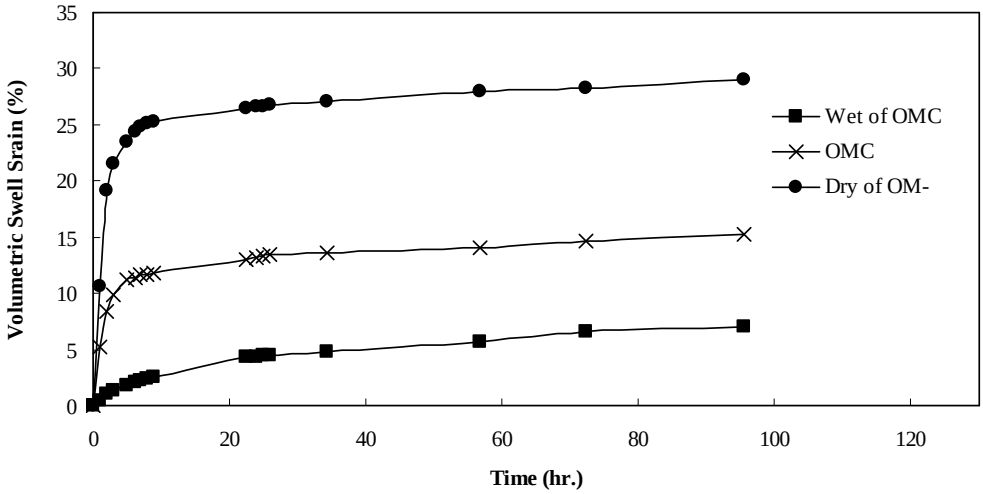


Figure 3.10 Typical Volumetric Swell Strains Results for Paris Soils at Three Different Moisture Contents

Consequently, test results are presented in terms of vertical, radial and volumetric swell strains with time (hour). All three high PI soil types, Fort Worth, Dallas, and Paris clayey soils showed volumetric swell strain at OMC condition more than 10% which is considered as a very high degree of expansion (Chen, 1983). As expected, the El Paso clay exhibited lowest swell strain results because the El Paso clay exhibited lowest Plasticity Index and CEC values. Again, high swell soils have high CEC values which indicate a correlation between chemical and swell properties of the present soils.

3.5.6. Swell Pressure Test

Swell pressure test was conducted in order to measure maximum loads or pressures that soils exhibit in order to maintain original volume of soil. Table 3.8 presents test result for all four soils and three different moisture conditions (Wet of OMC, OMC, and Dry of OMC) in term of ksf (kilo pound per square foot) values. According to Sridharan et al. (1986) and Fredlund and Rahardjo (1993), the constant swell pressure tests were performed as per the following procedures.

Table 3.8 Swell Pressure Test Results of Four Soils

Moisture Condition	**Swell Pressure, ksf (kPa)**			
	Fort Worth	**Dallas[1]**	**Paris**	**El Paso**
Wet of OMC	1.55 (74.21)	NA	1.51 (72.30)	0.23 (11.01)
OMC	2.67 (127.84)	NA	3.37 (161.35)	0.54 (25.86)
Dry of OMC	3.10 (148.43)	NA	3.98 (190.56)	0.78 (37.35)
Field Moisture Content	-	1.28 (61.29)	-	-

Note: NA - Dallas soil was not tested due to insufficient material

As shown in the Table 3.8, for all types of soils the lowest values of swell pressure were found at wet of optimum moisture condition while the highest values were found at dry of optimum moisture condition. This is the reason behind civil infrastructure underlined by the expansive soils experience cracking when those soils are exposed to heavy rain falls following

long dry periods of high temperatures. The trend of this test results revealed the same as other test results as mention above, showing El Paso which represent to low PI clay exhibited lowest swell pressure values than other clays.

3.5.7. IDT Test

In this section, indirect tensile test results that describe the tensile strengths of various soil types at different moisture conditions are presented. Table 3.9 illustrates comparisons between initial and final conditions of Indirect Tensile Strength (IDT) tests.

Table 3.9 Indirect Tensile Strength Test Results in psi, (kPa)

Moisture Conditions	**Types of Specimens**	**Average. of Indirect Tensile Strength psi, (kPa) Initial Condition**	**Average of Indirect Tensile Strength psi, (kPa) Final Condition**	**Standard Deviation psi, (kPa)**
LL+10%	**El Paso**	0	45.44 (313.29)	9.08 (62.66)
	Dallas	0	47.12 (324.87)	4.34 (29.89)
	Fort Worth	0	49.78 (343.22)	4.98 (34.34)
	Paris	0	74.31 (512.25)	18.34 (126.5)
PL+10%	**Dallas**	0	48.73 (335.98)	12.18 (84.01)
	Fort Worth	0	59.56 (410.65)	14.89 (102.7)
Wet of OMC	**Dallas**	2.13 (14.69)	NA	NA
	Fort Worth	2.61 (18.00)	NA	NA
OMC	**Dallas**	3.24 (22.33)	NA	NA
	Fort Worth	4.16 (28.68)	NA	NA

Note: NA- Not able to test them due to soil cracks

The soil specimens used in the IDT tests need to be taken out from the oven about an hour before SIP tests were completed. As shown in the Table, the IDT results indicate tests conducted at LL+10% and PL+10% are equal to zero strength at the original moisture states as these specimens do not show any strength and hence zero IDT values are assigned to the initial compaction moisture content conditions. At final moisture condition, the tensile strength of soil from El Paso, Dallas, Fort Worth, and Paris site at LL+10% are 45.44, 47.12, 49.78, and 74.31 psi, respectively. Both Dallas and Forth Worth soils at Wet of OMC and OMC conditions could not be measured because most of soil samples cracked and failed before testing.

3.6 Summary

This chapter presented basic to engineering soil properties for four different types of soil samples considered for this research. The basic soil tests, CEC measurements, and engineer tests were performed to indentify the soil behaviors of Fort Worth, Dallas, Paris, and El Paso clayey soils. Three soils are considered to have very high degree of expansion and one soil has low degree of swelling. Paris site exhibited the highest value of volumetric shrinkage strain at OMC conditions. Fort Worth, Dallas, and El Paso showed moderate to lower shrinkage strain potentials, respectively. Soils with high amount of cation exchange capacities were shown to undergo high swell and shrinkage strains in the present tests and hence there appears to be a correlation between CEC and volume change properties of soils.

CHAPTER 4

SIP TEST AND ANALYSIS OF TEST RESULTS

4.1 Introduction

Expansive soil is a worldwide problem and many tests and methods that include both indirect and direct measurement have been developed for estimating shrink-swell potentials of expansive soils. Indirect methods involve the use of soil properties and classification schemes to estimate shrink-swell potential while direct methods provide the actual physical measurements of swelling potentials in percentage values.

Most studies deal with swell properties and very few of them use shrinkage properties in the characterization of soils. This Chapter presents the present research in which the Shrinkage Induced Pressure or SIP measurements made directly on the soil samples. This study investigates the appropriate 'force sensor' orientation and how it can be used to determine shrinkage induced pressure in soils. A summary of the Shrinkage Induced Pressure (SIP) test procedure, equipments used, correlation with other soil properties including Indirect Tensile Strength (IDT) test data is presented in this chapter.

4.2 Shrinkage Induced Pressure Test (SIP)

The Shrinkage Induced Pressure (SIP) test is the direct measurement of the shrinkage force of curling soil specimen during drying process when the soil is restrained by a rigid container. Typically soil is subjected to drying and during drying the soil will induce pressure to a sensor attached to rigid walls of the test setup. Unfortunately, it is difficult to know exact point where curling soil will come in touch with the walls of the container. The best measurement can be achieved by placing the force sensor on the location of curling soil that will be expected to lift or raise. This research has focused on this aspect by investigating various configurations of FS probe in the test setup.

A few assumptions are needed in this test. For example, shrinkage force is assumed to be isotropic. As a result, it is easier to measure a shrinkage force directly by placing the sensor in the liquid soil medium and by inducing the soil to compress the sensor when it undergoes drying process. In addition to successful testing and measurements of shrinkage induced pressures (SIPs), studies were performed on different expansive soil types to understand the effects of soil variables on the SIP values. The following describes the sensor details and the test procedure details.

4.2.1 Force Sensor (FS) Features

In the past, researchers used a force sensor termed as a Tactile Sensor for pressure measurements. Tactile sensor was a touch sensor that provides a force measurement when pressed against it. The diameters of tactels are 3.3 mm as presented in Figure 4.1 below.

Figure 4.1 Full Range of Tactile Sensors
(Source: http://www.shadowrobot.com/tactile/overview.shtml, 2010)

This sensor is suitable for various loading applications including thumb and finger induced pressure applications. Figure 4.2 shows one such application on a hand robot model. The tactile are sensitive to loads ranging from 0.1 to 25 N and the temperature range for this sensor is -20 to 50° C (Shadow, 2010). This allows simultaneous control of both delicate

handling and power grasping. The unique construction of Shadow Tactile Sensors allows custom sensors to be built to almost any dimensions. However, this tactile sensor is not suitable for measurement of shrinkage pressure due to the size of sensor. Thus, other sensor technology was considered that is more suitable for shrinkage pressure measurements in wet soils.

Figure 4.2 Tactile Sensors on the Robot Hand
(Source: http://www.shadowrobot.com/tactile/overview.shtml, 2010)

Force sensor (FS) was another innovative technology that has potential to provide direct measurement of soil shrinkage induced pressure. Force sensor is a polymer thick film device which exhibits a decrease in resistance with an increase in force applied to the active surface. The FS is an ultra-thin and flexible printed circuit, which can be easily incorporated into most applications. The FS has a paper-thin construction, durability and force measurement ability. The FS can measure forces between any two surfaces and it is strong enough to stand up to most environments (Tekscan, 2010). The active sensing area is a 0.375 diameter circle at the end of the sensor. The sensors are made of two layers of substrate. This substrate is

composed of polyxester film (Tekscan, 2010). Also, the FS has better force sensing properties, linearity, hysteresis, drift, and temperature sensitivity than any other thin-film force sensors (Tekscan, 2010). A photograph of FS is presented in the Figure 4.3.

Figure 4.3 Force Sensor Model A201 (Tekscan, 2010)

On each layer, a conductive material (silver) is coated, followed by a layer of pressure-sensitive ink (Tekscan, 2010). Bonding agent is then used to shield the two layers of substrate together to form the sensor. The silver circle on top of the pressure-sensitive ink defines the active sensing area. Silver extends from the sensing area to the connectors at the other end of the sensor, forming the conductive leads. FS is finished with a solderable male square pin connector, which allows them to be incorporated into a circuit (Tekscan, 2010). The two outer pins of the connector are active and the center pin is inactive (Tekscan, 2010).

The sensor acts as a variable resistor in an electrical circuit (Figure 4.4). Its resistance is very high, when the sensor is unloaded. The resistance decreases, when a force is applied to the sensor. Sensors should be stored at temperatures in the range of 15℉ (-9℃) to 165℉ (74℃) (Tekscan, 2010). Connecting an ohmmeter to the outer two pins of the sensor connector and then applying a force to the sensing area, one can read the change in resistance which will be used to estimate force. Specifications of the sensor are given Table 4.1.

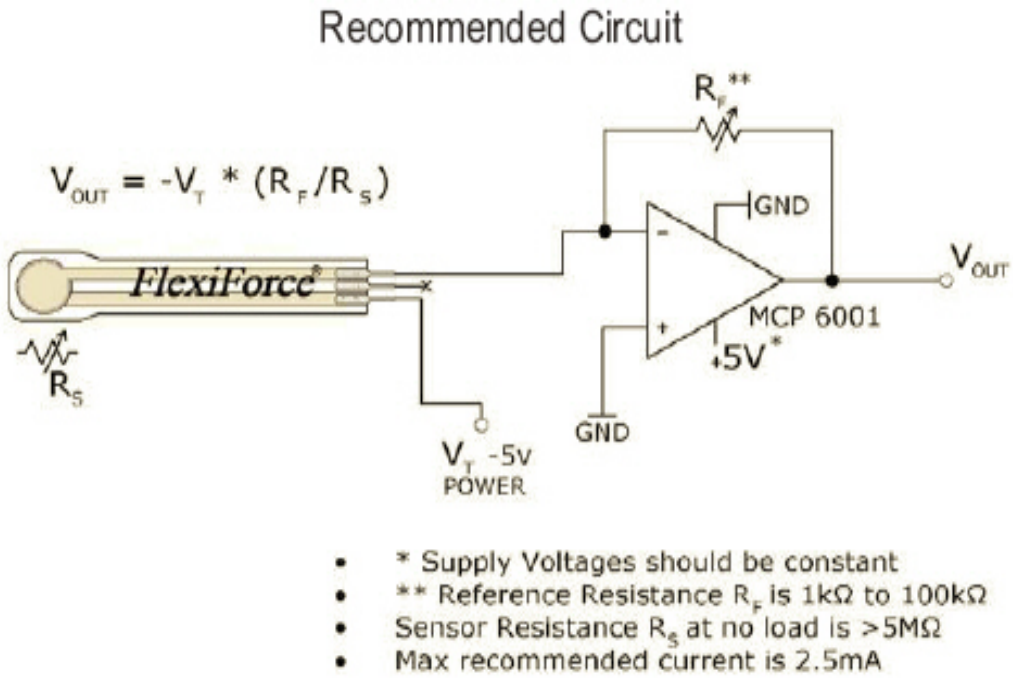


Figure 4.4 Electrical Circuits of Force Sensor Setup (Tekscan, 2010)

The FS probe is flexible enough to allow for non-intrusive measurements in materials. They can be attached to many surfaces, and can be combined with plastic or metal films for greater stiffness or for added protection from abrasion. There are many other ways to integrate the FS into an application. One way is to incorporate this sensor into a force-to-voltage circuit. Calibration is needed to convert the FS output into the appropriate engineering units (Tekscan, 2010). Depending on the setup, an alteration could then be performed to increase or decrease the sensitivity of the sensor. Advantages of using force sensors include: Accuracy and superior linearity results; Wider range of forces; Sensor output is not a function of loading area; High temperature force for measurements values; Greater flexibility sensors; and Durable sensors and cheap.

Table 4.1 Force Sensors (A 201 Model) - Specifications & Features (Tekscan, 2010)

Physical Properties	
Thickness	0.008" (.203mm)
Length	8" (203mm)
	6" (152mm)
	4" (102mm)
	2" (51mm)
Width	0.55" (14mm)
Sensing Area	0.375" diameter (9.53mm)
Connector	3-pin male square pin
Typical Performance	
Linearity Error	<+/-3%
Repeatability	<+/-2.5% of full scale (conditioned sensor, 80% force applied)
Hysteresis	<4.5% of full scale (conditioned sensor, 80% force applied)
Drift	<5% per logarithmic time scale (constant load of 90% sensor rating)
Response Time	<5 microseconds
Operating Temperatures	15℉ to 140℉ (-9℃ to 60℃)
Force Ranges	0-1 lb (4.4 N) & 0-25 lb (110 N) & 0-100 lb (440 N)
Temperature Sensitivity	Output variance up to 0.2% per degree F

Force sensor is available in the current market following force ranges: Sensor A201-1 (0-1 lb. force range); Sensor A201-25 (0-25 lb. force range); and Sensor A201-100 (0-100 lb. force range). Force sensors (FS) were acquired and studied for the first time for the direct measurements of soil shrinkage induced pressures. In this thesis, FlexiForce type force sensor is selected because of its paper thin thickness (0.008 in.) and its high performance, high operating temperature (up to 140 ^{o}F) conditions at which it can be used for present tests.

4.2.2 Force Sensor Calibration

1. It is recommended to warm up the sensors before by placing the maximum (estimated) test load onto the sensor's active area (small grey circle) for approximately 3 seconds. The load should be removed from the sensor and this step should be repeated 4-5 times.
2. Each sensor shall be protected by thin, uniform adhesives, such as Scotch brand double - sided laminating adhesives (see Figure 4.5). Kinks and dents as well as soil moisture at the sensor's active area may result in false triggering and reading.

Figure 4.5 Force Sensor Coated by Texture

3. Both sides of the protected force sensor shall be coated by texture in order to increase adhesion between soil and sensor shown in the Figure 4.6. This is an important step since the soil could shrink away from the sensor and may lose contact between soil and the sensor if the adhesion is inadequate.

Figure 4.6 Active Area for Force Sensor

4. Connect the sensor to the data logger and measure the initial voltage data.
5. Each sensor shall be calibrated by placing different known loads on the sensor active area. The calibration should be conducted at testing temperature condition. The corresponding voltage outputs shall be recorded under each load.
6. Measure the drift error expecting during testing. Drift is the change in sensor output when a constant force is applied over a period of time. If the sensor is kept under a constant load, the resistance of the sensor will continuously decrease, and the output will gradually increase. It is important to take drift into account when calibrating the sensor, so that its effects can be minimized as per the instructions of the Force Sensor manual. Since the drift depends on the time duration that the load has been placed on the sensor, the best way to accomplish this is to put a

(estimated) maximum load in a time frame similar to that which will be used in the testing and then record the drift value.

7. The electrical drift correction shall be applied on the sensor calibration.
8. Calibration curve is finally obtained by correlating forces (or pressures) with corrected voltage outputs as shown in the Figure 4.7 below.

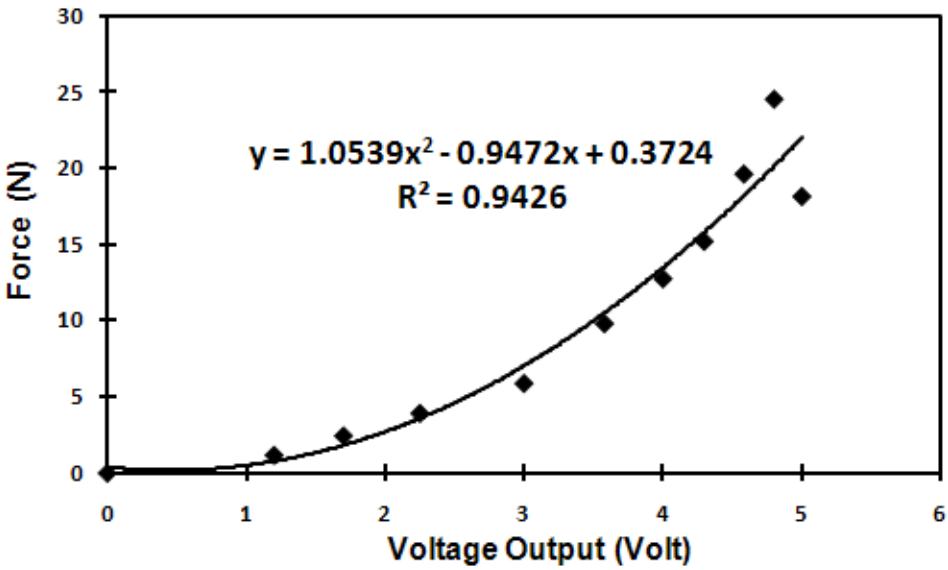


Figure 4.7 Calibration Curve of Force Sensor

During calibration, researchers need to pay attention to the following factors as per the FS manual:

- Apply a calibration load that approximates the load to be applied during system use, using dead weights or a testing device.
- Avoid loading the sensor to near saturation when calibrating. If the sensor saturates at a lower load than desired, adjust the "Sensitivity."
- Distribute the applied load evenly across the sensing area to ensure accurate force readings. Readings may vary slightly if the load distribution changes over the sensing area.

4.2.3 SIP Test Equipment

For shrinkage pressure measurement, the following instruments are needed:

1. Force Sensing Resistors (FSR) or Force Sensor (FS) (see Figure 4.8)

Figure 4.8 Force Sensor Model A201

2. Screw terminal and DAQ setup (see Figure 4.9)

Figure 4.9 Set of Drive Circuit

3. Data Logger for recording voltage changes (see Figure 4.10).

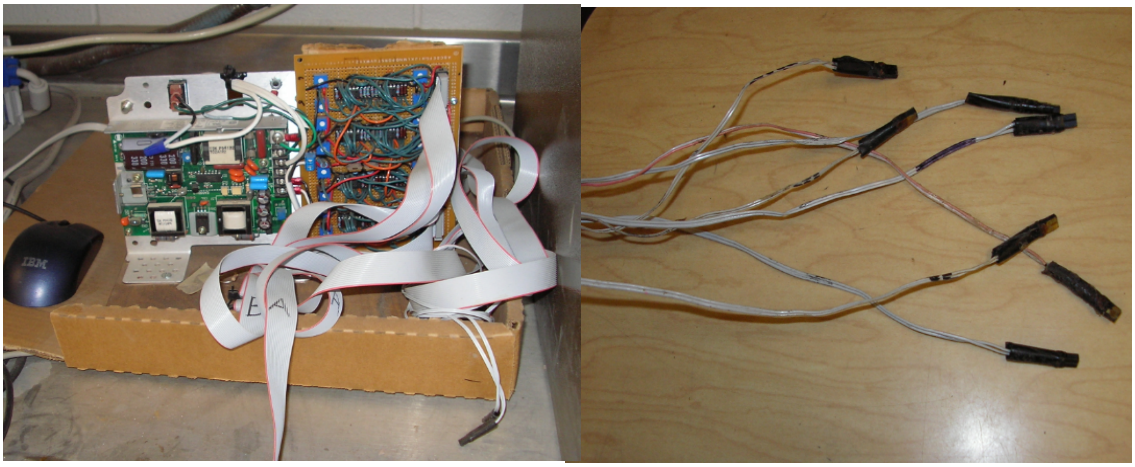


Figure 4.10 Data Logger

4. Oven chamber for drying the soil specimen
5. 3 in. diameter moisture can with a 0.8 in. slit on the side of the can for the placement of force sensor (in horizontal orientation) as shown in Figure 4.11. For vertical orientation SIP tests, no holes are needed.

Figure 4.11 Moisture Can with a Horizontal Slot for FS

6. Lubricating oil

The next section covers the SIP test procedure and test results. Two orientations of sensors are attempted and these are termed as SIPV and SIPH with vertical and horizontal sensor configurations, respectively. The following sections describe these tests in detail.

4.3 SIPV Test – FS Vertical Configuration (SIPV)

Several soils from Dallas, Fort Worth, Paris, and El Paso, Texas were used in the SIP studies. These soils were used to compact and prepare soil specimens at different compaction moisture contents and different FS orientations for SIP measurements. Vertical direction (or orientation) test results are first presented here:

4.3.1 SIPV Test Procedure

The following describes step by step procedure for SIPV measurements.

1. Moisture cans are used as experimental containers. The walls inside the cans are first cleaned.
2. Grease the cans prior to placing the soil such that the soil could undergo free shrinkage without significant cohesive–frictional restraints at the wall boundaries.
3. Place approximately 250 gm of representative dry soil sample which is smaller than 425 μm (sieve no. 40) in the mixing dish.
4. Thoroughly mix the soil with distilled or deionized water at soil's Liquid Limit + 10%.
5. Place the mixture into the containers.
6. Place a force sensor into the mixture in a vertical position. Make sure the contact area of FS is fully submerged into the soil specimen.
7. Provide light compactive tamping around the FS to make sure there are no air bubbles in the mixture and sensor face are fully contact with the mixture. Connect the sensor to data logger system as shown in Figure 4.12.

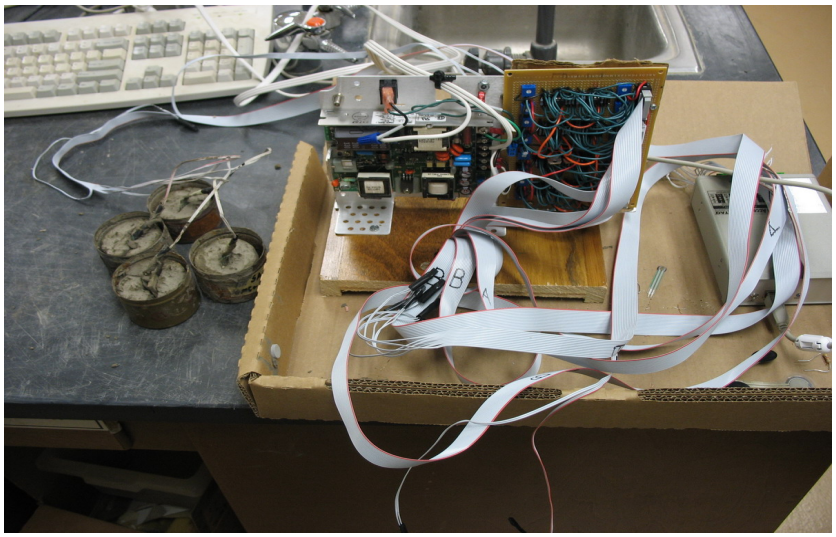


Figure 4.12 Data Logger Setup in Vertical Orientation

8. Handle the tail of the FS device carefully. Otherwise, this can break. The smallest suggested bending radius for the tails of evaluation parts is about 0.1” (2.5 mm). Also, researcher needs to be careful not to overstress the active loading area. This can cause stress on the active area which may result in pre-loading and false readings.
9. Set the data logger system such that it records voltage output from the sensor for every 5 seconds.
10. Set oven temperature at approximately 40 – 45°C (which is a limit of working temperature of the FS sensor). This temperature corresponds to high field temperature conditions of 113°F (45°C).

11. Place the soil container into an oven as shown in the Figure 4.13.

Figure 4.13 Placement soil samples into the oven at temperature 40- 45 ^{0}C

12. Monitor the voltage output until the soil mixture is completely dried up and/or there is no change in voltage output for more than 6 hrs.
13. Use calibration curve to convert output voltage into shrinkage induced pressure value.

4.3.2 Problems Experienced During SIPV Testing

Schematic of the test setup for vertical orientation as presented in the Figure 4.14.

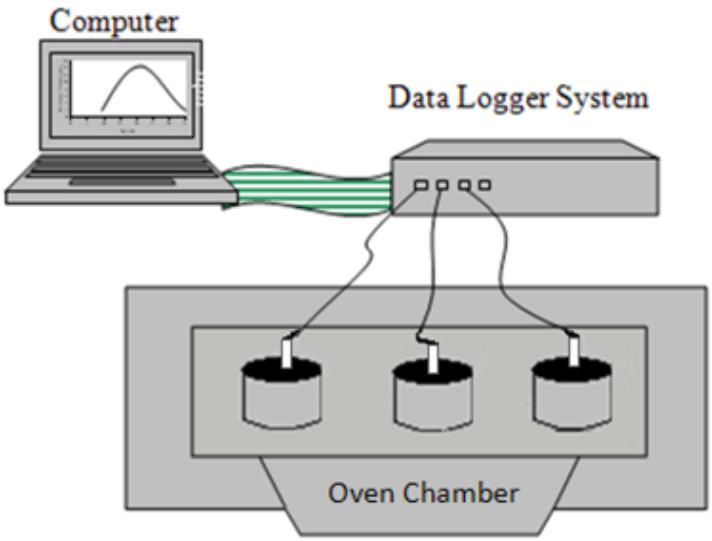


Figure 4.14 Schematic of the Test Setup Used

Table 4.2 presents the SIPV test results from vertical configuration tests on Fort Worth and Paris samples. Based on several tests, it can be mentioned that four out of six tests were successfully conducted. As a result, the percent success of using vertical orientation type SIP tests is only 67% as shown in Table 4.2. Low success was attributed to the sensor placement and soil cracking. However, among the 67% successful tests, still cracking occurred which limited SIP measurements.

Table 4.2 Percent Success of Using Force Sensors at Vertical Orientation

Moisture Contents	**Number of Soil Specimens**		**% Success of using Vertical Orientation**
	Fort Worth	**Paris**	
LL+10%	4/6	4/6	67%

Figure 4.15 shows average shrinkage pressure data from the present SIPV test results. Table 4.3 presents shrinkage pressure results of tested soils. The SIP value generally starts to spike up after 18 hrs of drying and this is attributed to size of the containers used for SIPV testing. If the container is small, it will take less time to dry the soil specimen, which may have resulted in quicker drying and SIP measurements.

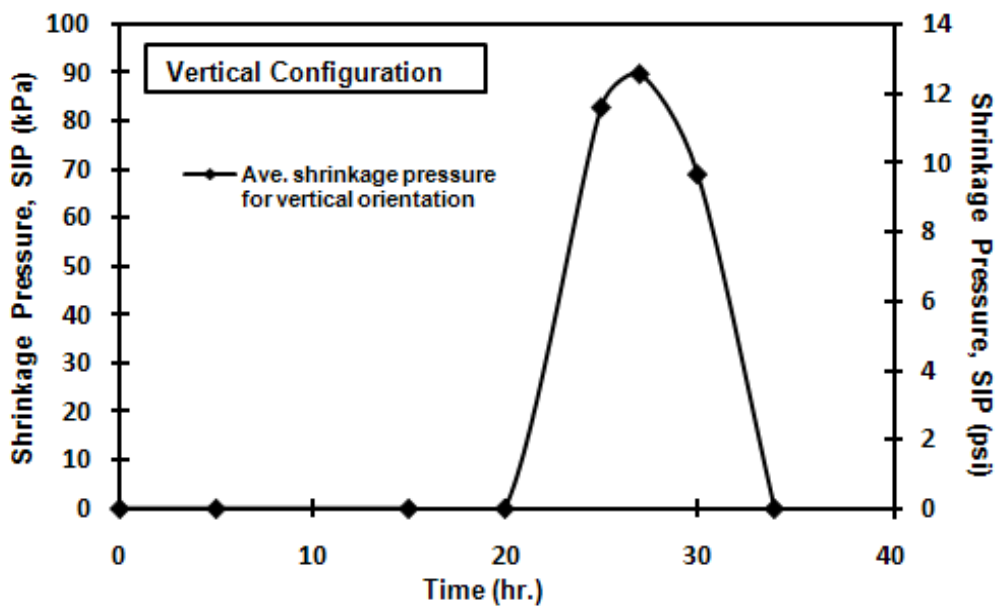


Figure 4.15 Typical Shrinkage Pressure Results of Fort Worth soil at Vertical Orientation SIPV Tests

Table 4.3 Shrinkage Pressure Results of Two Soils Prepared at LL+10% Condition

Soil	Max. Shrinkage Pressure psi, (kPa)	SD psi, kPa	Ave. of Max. Shrinkage Pressure psi, (kPa)
Fort Worth-1	9.97 (68.74)	1.98 (13.65)	12.38 (85.36)
Fort Worth-2	14.79 (101.97)		
Fort Worth-3	12.13 (83.63)		
Fort Worth-4	12.63 (87.08)		
Paris-1	9.31 (62.74)	1.47 (10.14)	11.01 (75.91)
Paris-2	12.71 (87.63)		
Paris-3	10.42 (71.84)		
Paris-4	11.60 (79.98)		

Typically the shrinkage pressure increases and reaches a peak value and beyond the pressure starts decreasing. This is because of loss of contact between drying soil and the sensor contact area as shown in Figure 4.16.

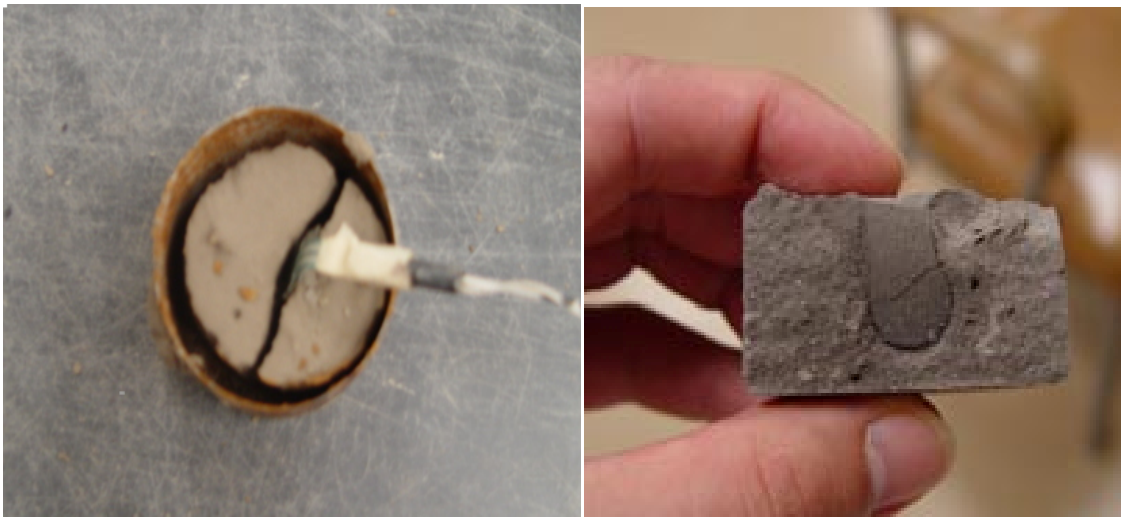


Figure 4.16 Shrinkage Soil Specimen Cracking and Loss of Contact From Soil Cracking

Among the successfully performed, the standard deviation from four test results is relatively high which can be due to variability in thermal changes occurring across the soil specimen. In addition, the presence of vertical sensor may have become as a foreign object in the soil slurry and this resulted in the cracking of the soil along the horizontal axis of the soil specimen as shown in Figure 4.16a. Because of these reasons, SIPH tests with sensor placed in horizontal configuration were conducted and details of these tests are presented in the next section.

4.4 SIP Test – FS Horizontal Configuration (SIPH)

In the following section, horizontally configured FS were used to measure SIP values.

4.4.1 SIPH Test Procedure

4.4.1.1 Specimen Preparation for Wet of OMC and OMC

1. Soil samples were prepared using Static Compaction Test (see Figure 4.17) at designated moisture contents, Wet of OMC and OMC.

Figure 4.17 Soil Specimen Preparations for Wet of OMC and OMC Conditions

2. For each cylindrical soil sample, 3 small samples were trimmed as shown in Figure 4.18.

Figure 4.18 Soil Specimen Preparations for Wet of OMC and OMC Conditions

4.4.1.2 SIPH Test Procedure:

1. The moisture can is lightly covered by grease or lubricating oil at the inside surface in order to allow the soil specimen undergoes free shrinkage without significant adhesion resistant at the contact boundaries with the mold.
2. Approximately 250 gms of representative dry soil sample passing Sieve no. 40 was mixed with water at moisture content of 10% more than its liquid limit (LL) and at PL+10%.
3. The prepared soil mixture was placed into the moisture can and the force sensor was inserted in a horizontal position as shown in the Figure 4.19.

Figure 4.19 SIPH Test in Horizontal Orientation

4. The slotted gap between samples and force sensor shall be sealed with silicone sealant (see Figure 4.20)

Figure 4.20 Sealing of Gaps with Silicone

5. Light tamping shall be applied to the soil mixture to let the air bubbles inside the soil mixture to expel them out and this will ensure full contact of soil mixture with the force sensor.

6. A measuring system shall be setup as shown in Figure 4.21. Several specimens can be tested at the same time with this process.

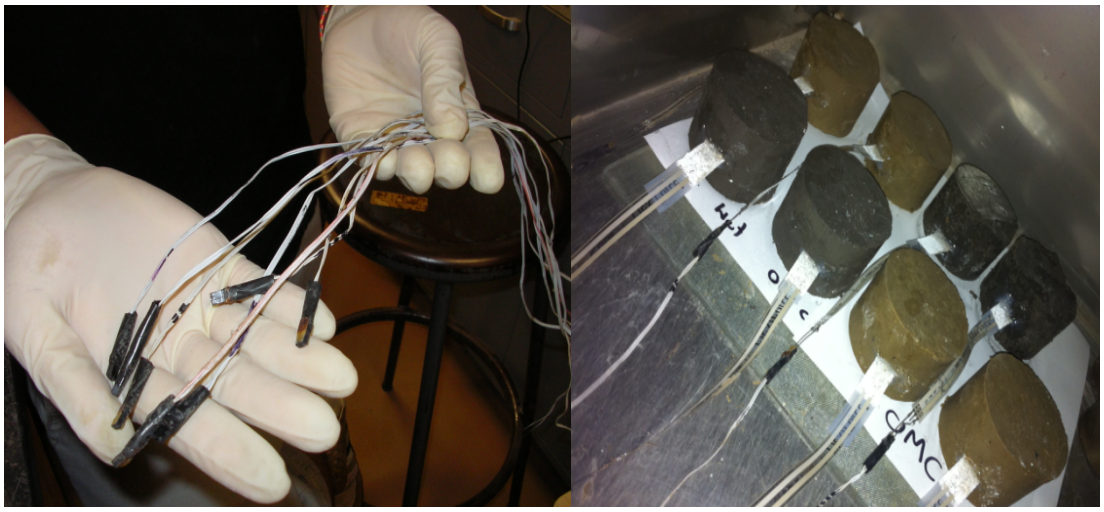


Figure 4.21 Data Logger System and Specimens with Horizontal Probe

7. Data logger system should be set to record voltage output readings for every 5 to 10 seconds. Temperature of the oven shall be set at approximately 40^0C in order to simulate slow drying process.
8. Prepare and place the soil specimens at each moisture contents in the oven chamber (see Figure 4.22). Each soil specimen (include container) shall be weighed every 4-6 hours so that the moisture content changes with time can be established.

Figure 4.22 Soil Specimens in the Oven at Horizontal Orientation for SIPH Tests

9. The voltage output is monitored until the soil mixture is completely dried up or there is no change in voltage readings for more than 6 hours. Figure 4.23 shows the shrunk soil specimens from SIPH tests.

Figure 4.23 Shrunk Soil Specimens after SIPH Tests

Figure 4.24 presents schematic of the test setup used for horizontal orientation SIPH tests.

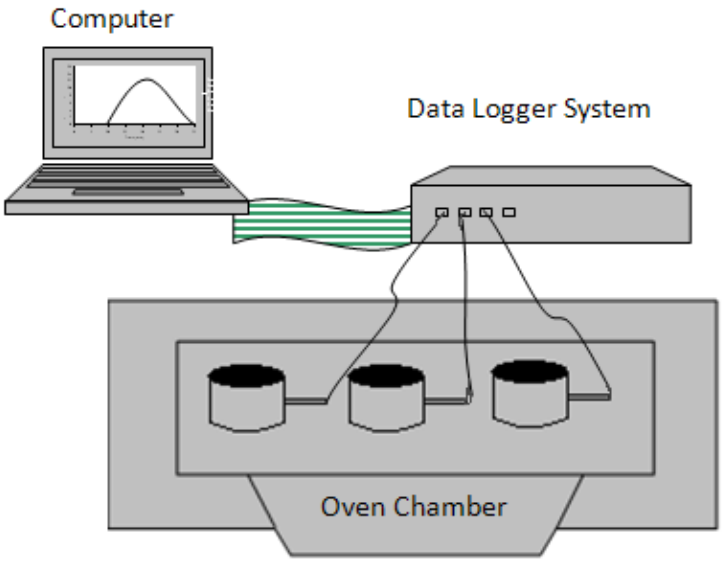


Figure 4.24 Schematic of the Test Setup

After inserting the FS at horizontal orientations, the percent of successful tests performed increased significantly as this number went up close to 87%. Table 4.4 presents these statistics for SIPH tests conducted on twenty three soil specimens from all four site locations.

Table 4.4 Percent Success of Using Force Sensors in Horizontal Orientation

Moisture Contents	**Number of Soil Specimens**				**% Success of using Horizontal Orientation**
	Fort Worth	**Paris**	**Dallas**	**El Paso**	
LL+10%	5/6	5/7	5/5	5/5	87%

Figures 4.25 and 4.26 present the individual shrinkage pressure test data of all soils. These results show very good repeatability of the SIPH test. It is interesting to note that the post

peak trends also showed consistent match with other similar tests. Overall, this indicates that the SIPH method is repeatable and is able to capture SIP values during drying process.

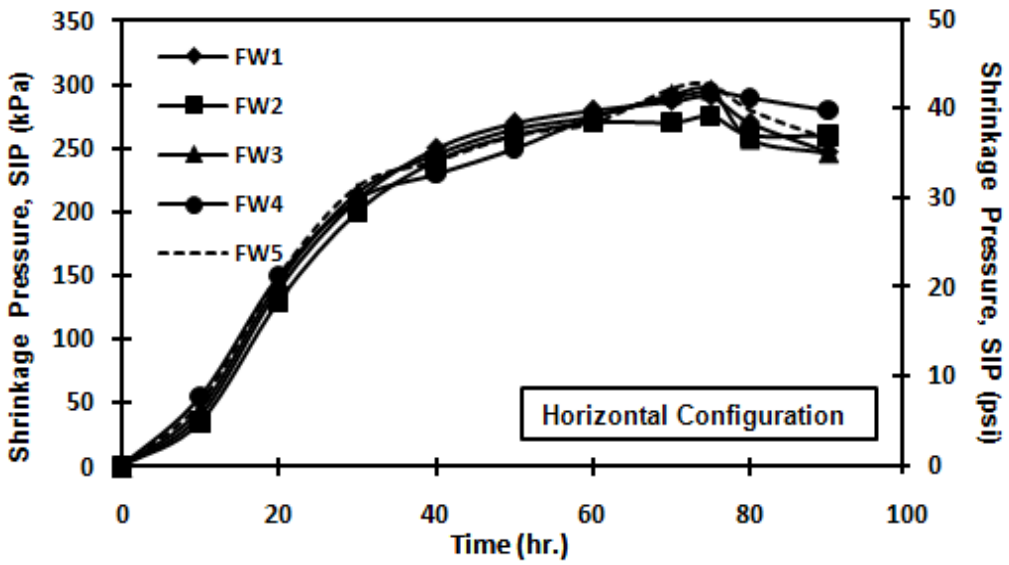


Figure 4.25 Shrinkage Pressure Test Results of Fort Worth Site Soil

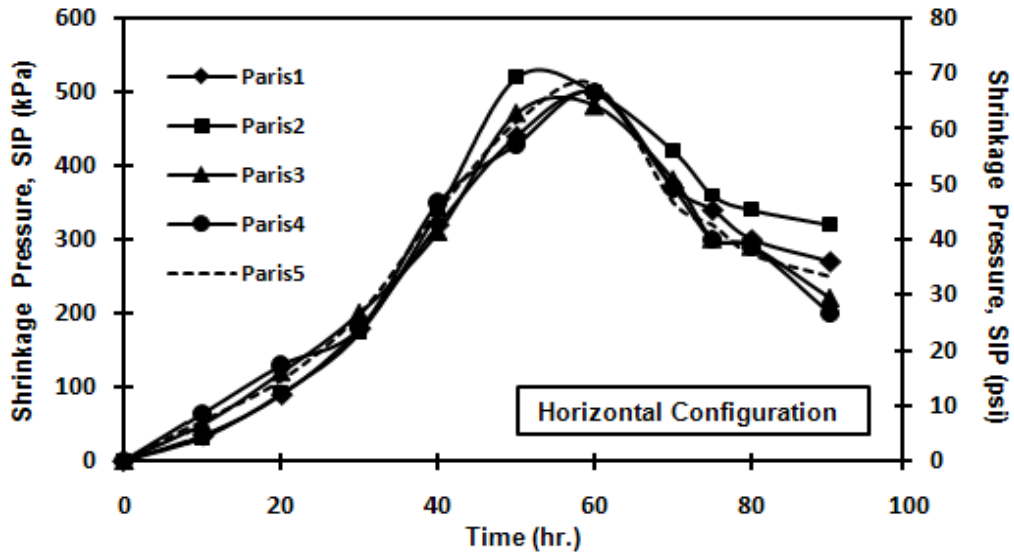


Figure 4.26 Shrinkage Pressure Test Results of Paris Site Soil

The next section describes various other soil variables that are investigated as a part of SIPH measurements. The following section and subsections describe these results.

4.4.2 Repeatability of SIPH Test and Summary of Test Results

This test was a newly developed one and hence it is necessary to ascertain the quality of test by studying the repeatability of test results. This section describes the present SIPH test results including different soils at various compaction moisture content conditions. The tests at the liquid limit conditions were already performed and these are described in the section 4.4.1 (SIPH Test Procedures by using horizontal orientation). For other moisture content conditions including Plasticity Limit, Wet of OMC and OMC conditions, the tests were also conducted and the results are described in the following.

Figures 4.32 (a – b) to 4.49 (a - b)) present SIPH test results conducted on different types of soil specimens at different moisture contents namely LL+10%, PL+10%, Wet of OMC, and OMC conditions. The correlation between SIP and time (hr.) for all four soils are presented in section a while section b shows the relationship between water content (%) and elapsed drying time period in hrs. Section c describes the percent of water content and SIP values and section d presents the percent change in water content and SIP values. Also, the photographs of the soil specimens after the testing can be seen in Figure 4.27 to 4.31. Table 4.5 presents percent success of all tests conducted in SIPH tests at different test related moisture contents.

Table 4.5 The Percent Success of Using Force Sensors

Moisture Contents	Number of Soil Specimens				% of Success	Rank
	Fort Worth	Paris	Dallas	El Paso		
LL+10%	5/6	5/7	5/5	5/5	87%	1
PL+10%	3/5	-	3/5	-	60%	2
Wet of OMC	1/5	-	1/5	-	20%	3
OMC	1/5	-	0/5	-	10%	4

Figure 4.27 Soil Specimens Dried after the Test at LL+10% Condition

Figure 4.28 El Paso Specimen after Drying

Figure 4.29 The Dallas Specimen after Running the Test

Figure 4.30 The Fort Worth Specimen after Running the Test

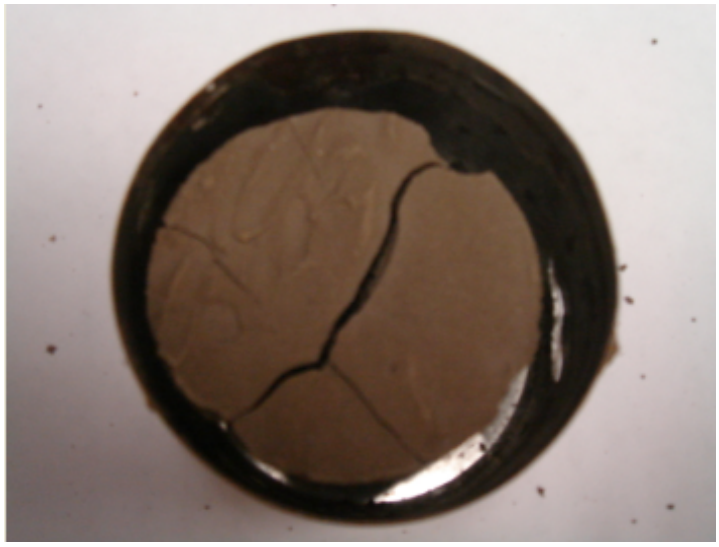
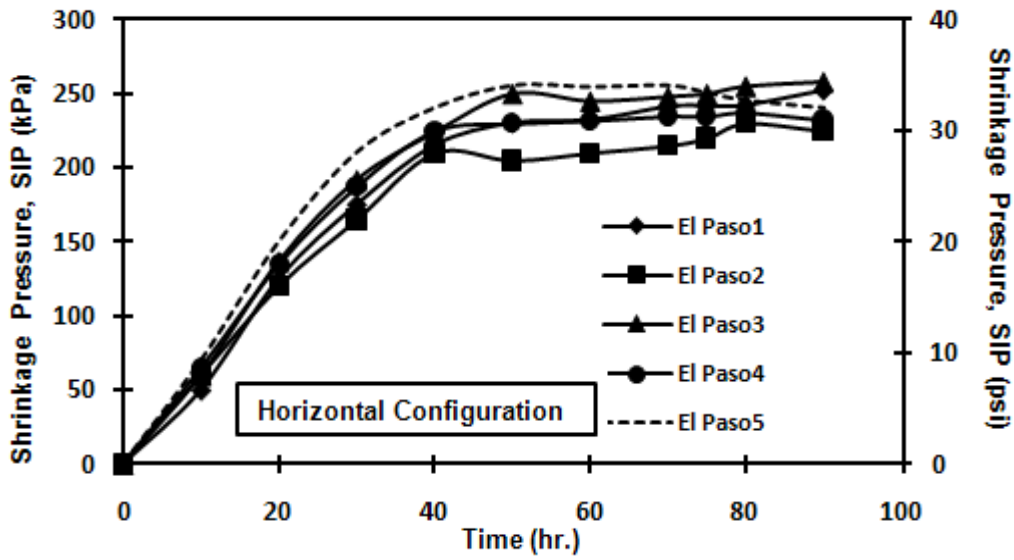
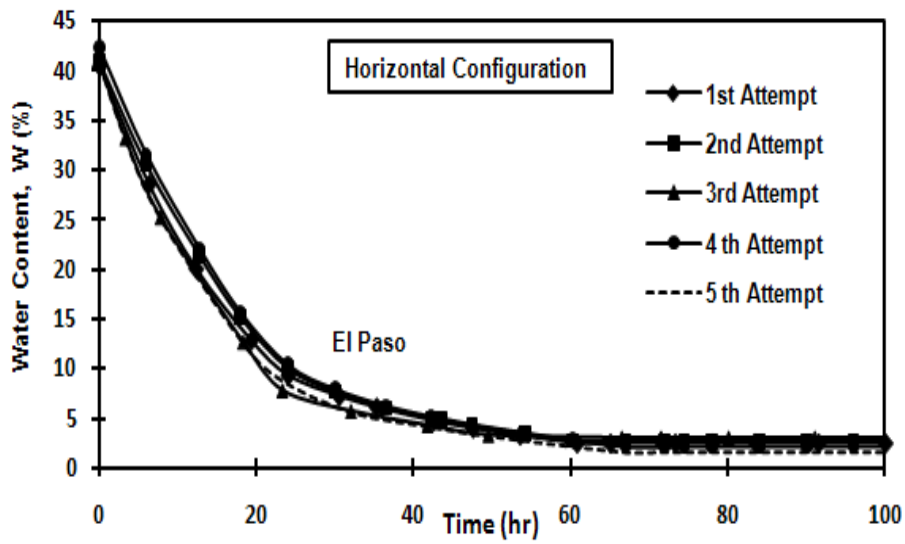


Figure 4.31 The Paris Specimen after Running the Test

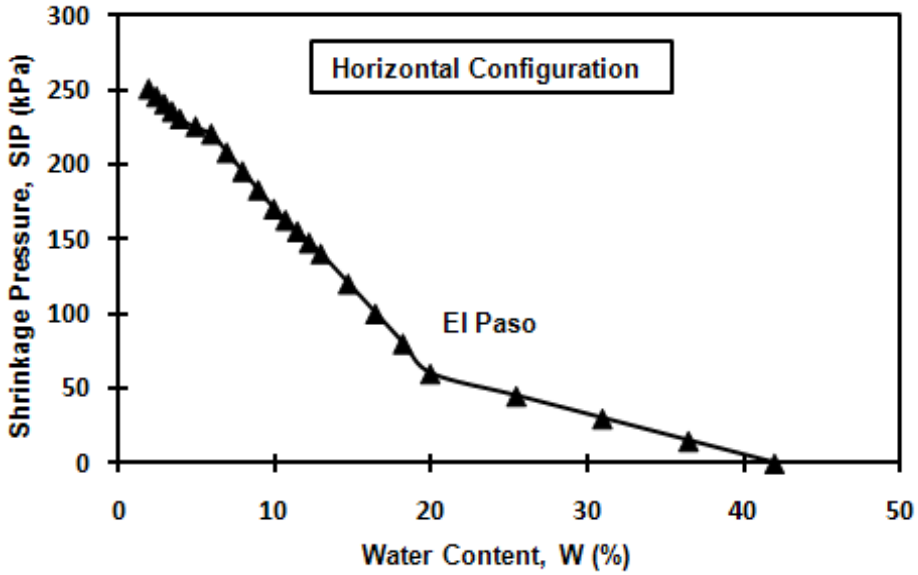


(a)

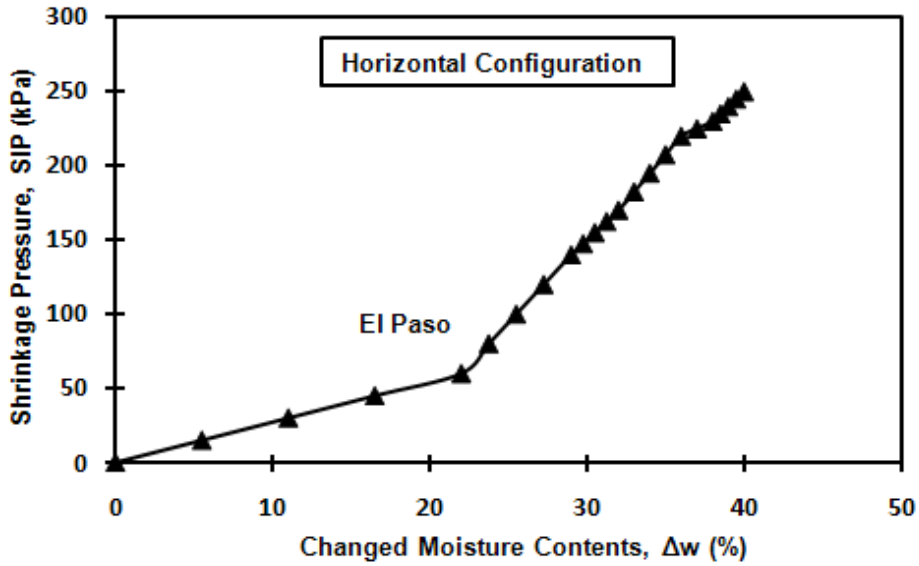


(b)

Figure 4.32 Results of Shrinkage Pressure of the El Paso Soil at LL+10% Condition: a) Time (hr.) versus SIP and b) Time (hr.) versus Water Content (%)

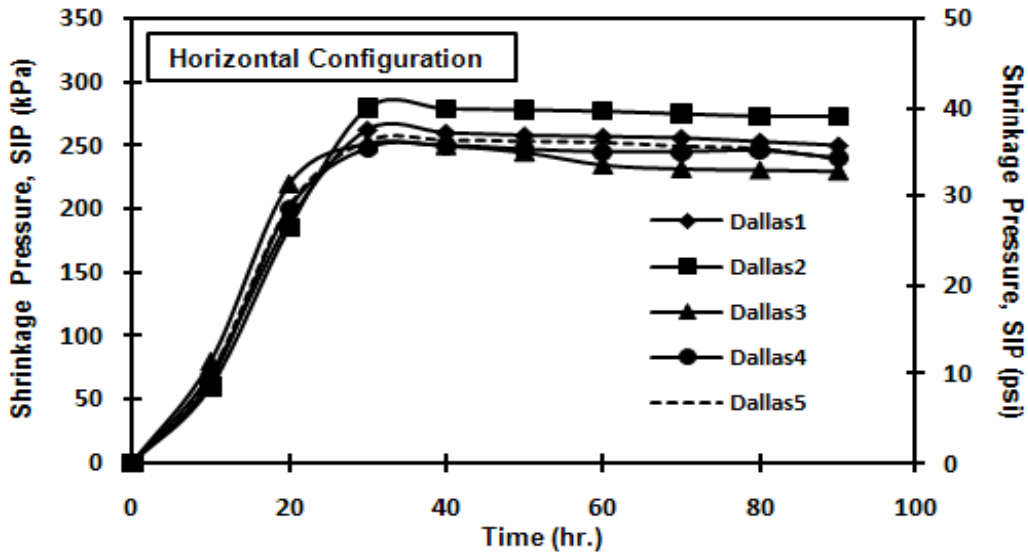


(a)

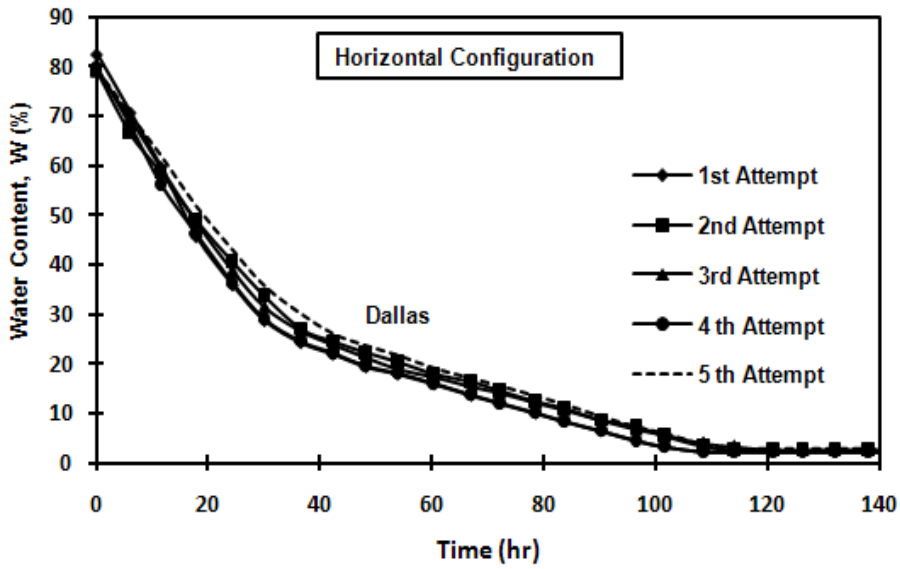


(b)

Figure 4.33 Results of Shrinkage Pressure of the El Paso Soil at LL+10% Condition:
a) SIP versus Water Content (%) and b) The Percent Change of Water Content (%) versus SIP

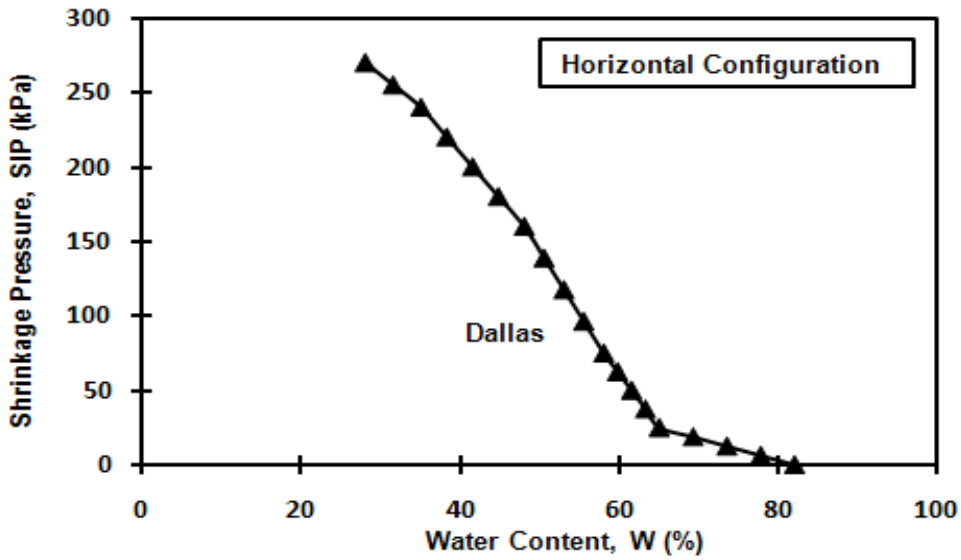


(a)



(b)

Figure 4.34 Results of Shrinkage Pressure of the Dallas Soil at LL+10% Condition: a) Time (hr.) versus SIP and b) Time (hr.) versus Water Content (%)



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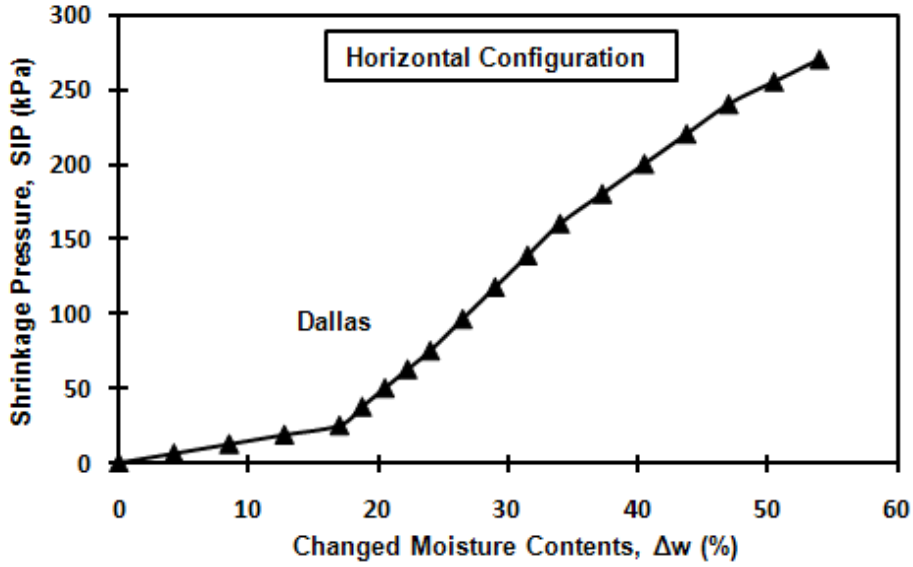
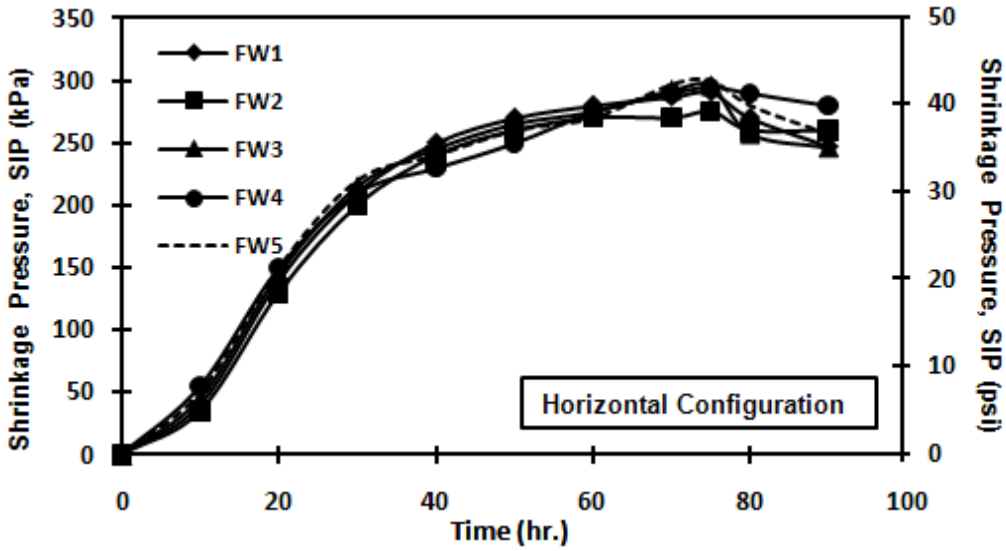


(b)

Figure 4.35 Results of Shrinkage Pressure of the Dallas Soil at LL+10% Condition:
a) SIP versus Water Content (%) and b) The Percent Change of Water Content (%) versus SIP



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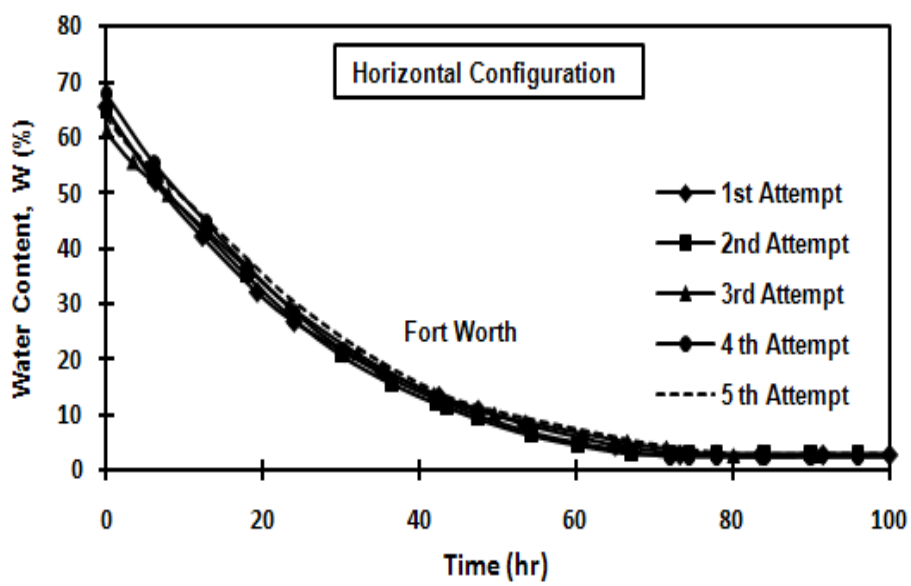
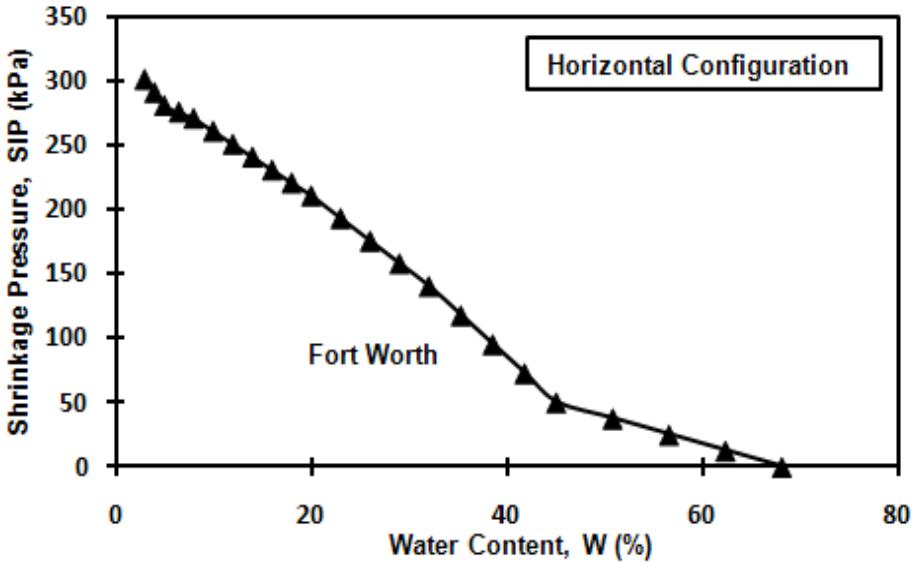
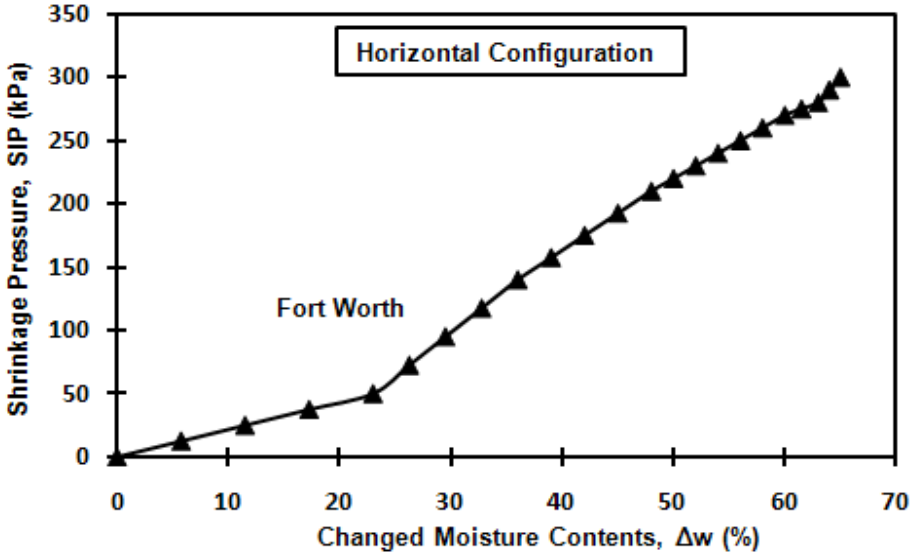


(b)

Figure 4.36 Results of Shrinkage Pressure of the Forth Worth Soil at LL+10% Condition: a) Time (hr.) versus SIP and b) Time (hr.) versus Water Content (%)

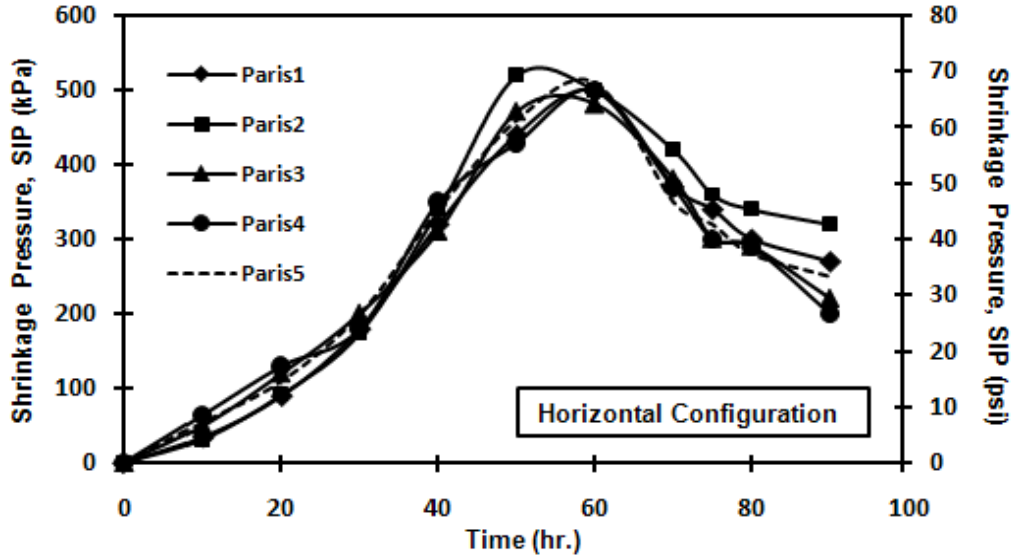


(a)

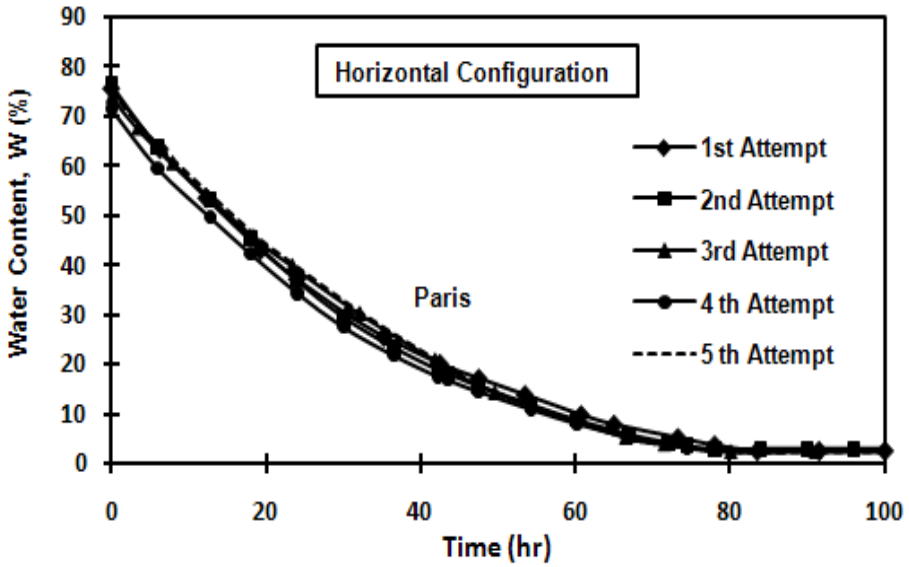


(b)

Figure 4.37 Results of Shrinkage Pressure of the Forth Worth Soil at LL+10% Condition:
a) SIP versus Water Content (%) and b) The Percent Change of Water Content (%) versus SIP

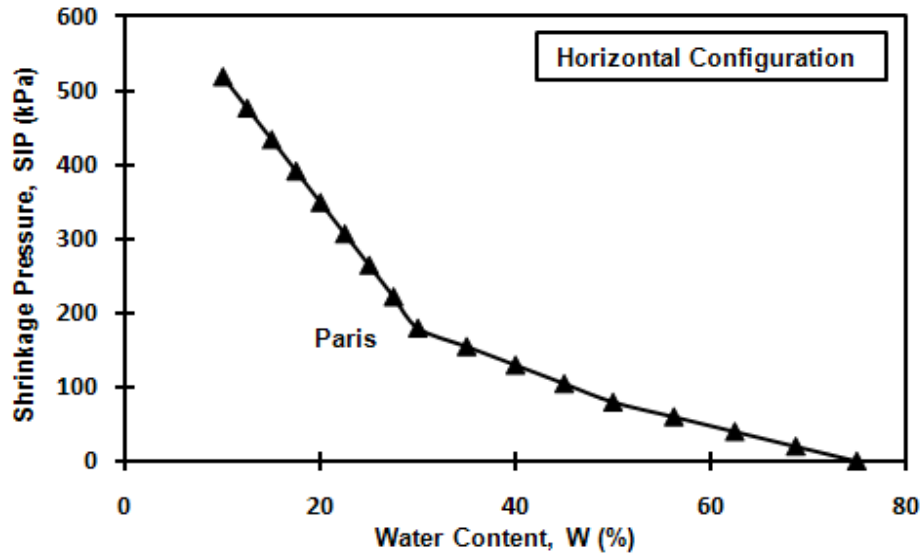


(a)

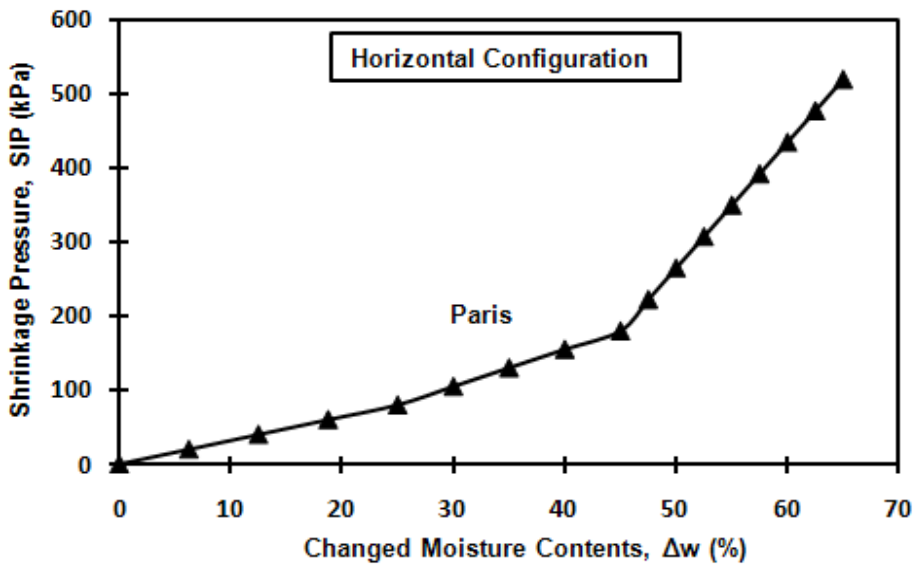


(b)

Figure 4.38 Results of Shrinkage Pressure of the Paris Soil at LL+10% Condition: a) Time (hr.) versus SIP and b) Time (hr.) versus Water Content (%)

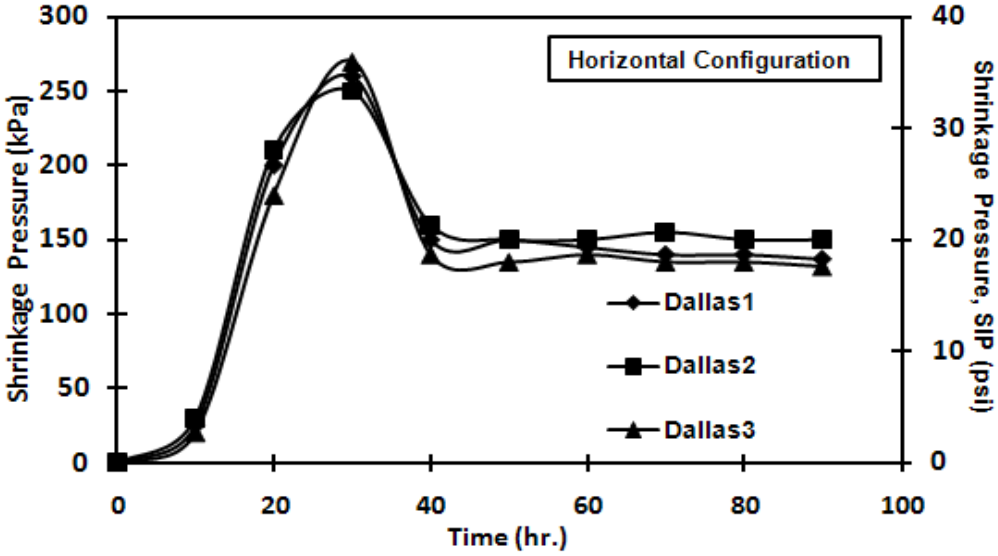


(a)

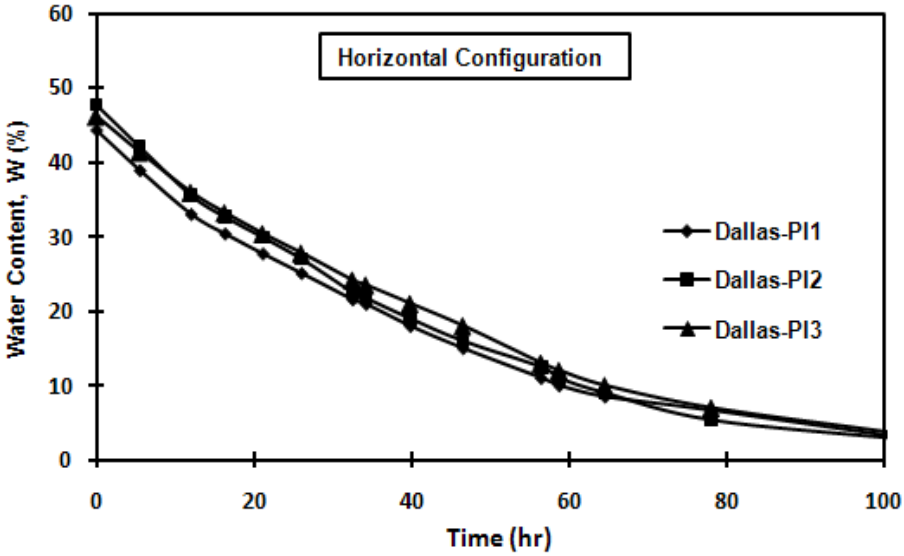


(b)

Figure 4.39 Results of Shrinkage Pressure of the Paris Soil at LL+10% Condition:
a) SIP versus Water Content (%) and b) The Percent Change of Water Content (%) versus SIP

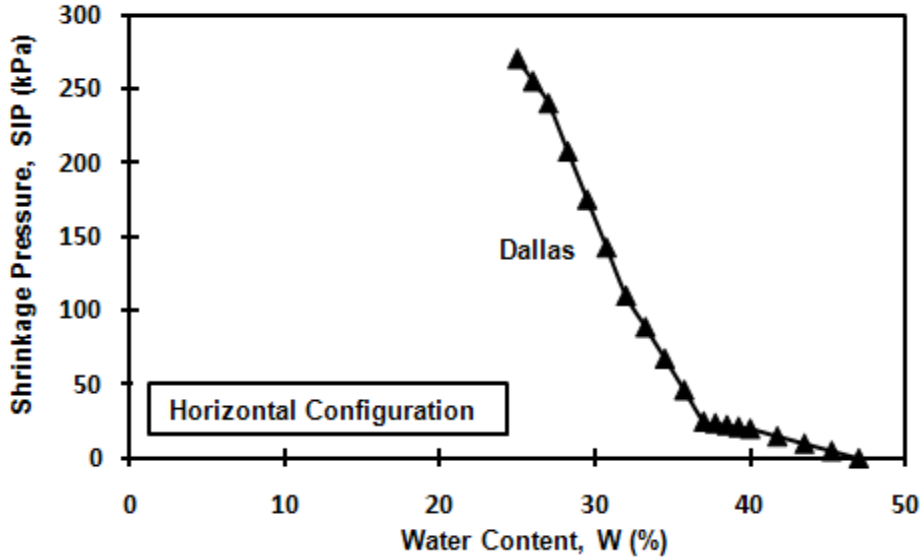


(a)

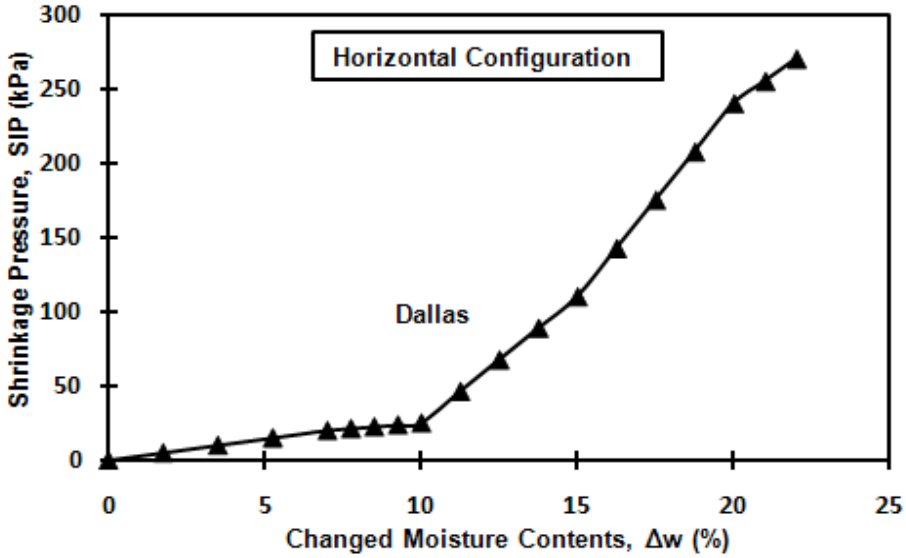


(b)

Figure 4.40 Results of Shrinkage Pressure of the Dallas Soil at PI Condition: a) Time (hr.) versus SIP and b) Time (hr.) versus Water Content (%)

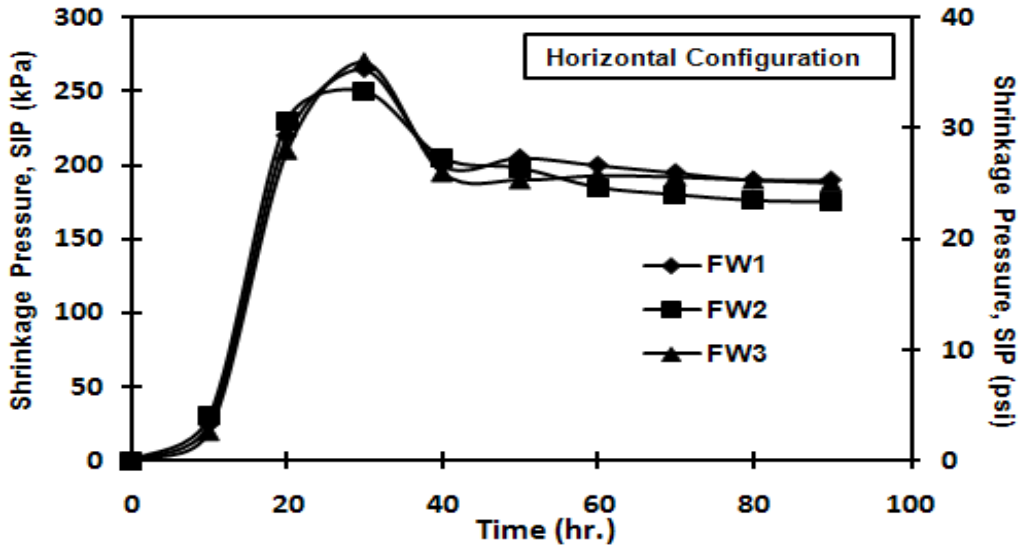


(a)

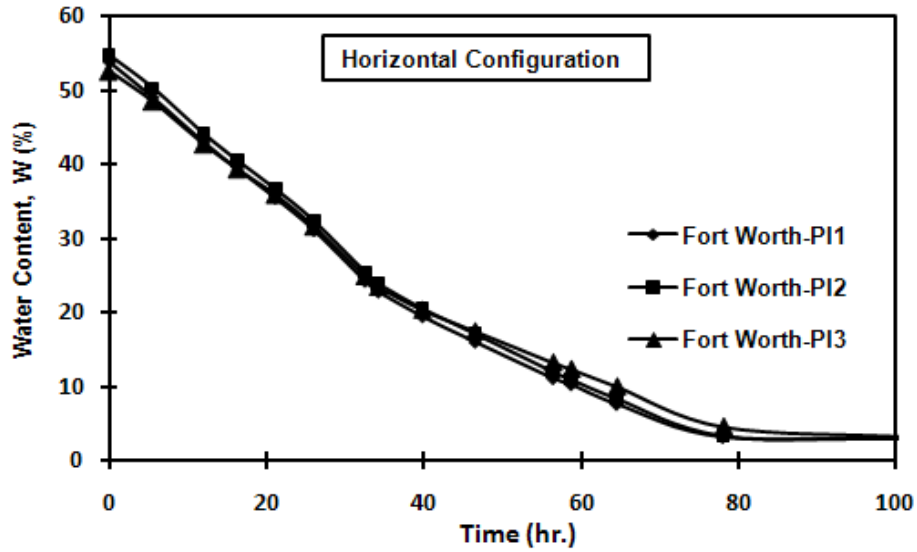


(b)

Figure 4.41 Results of Shrinkage Pressure of the Dallas Soil at PI Condition:
a) SIP versus Water Content (%) and b) The Percent Change of Water Content (%) versus SIP

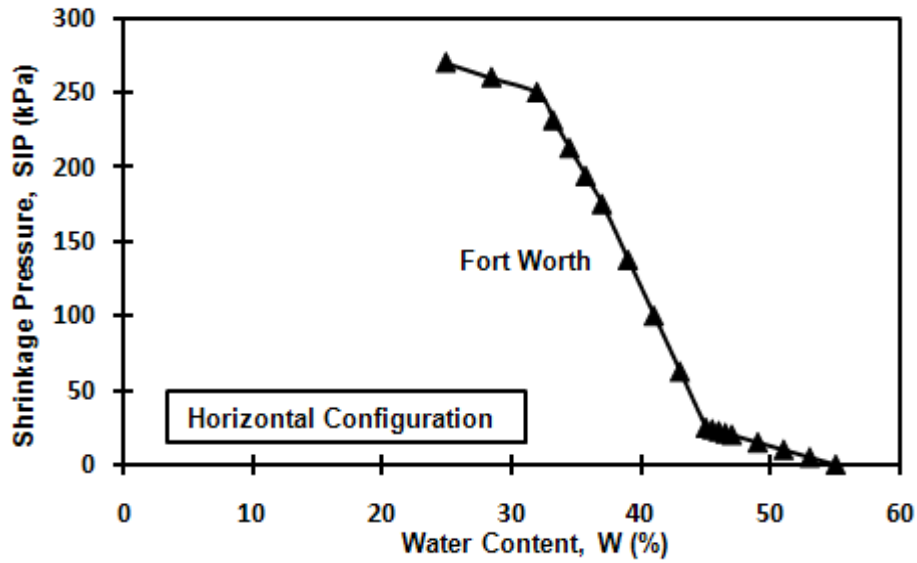


(a)

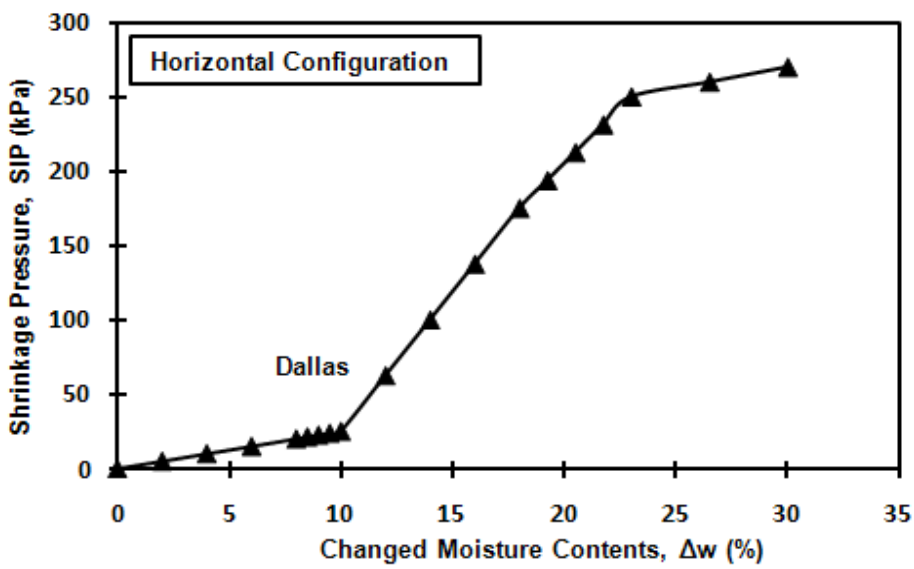


(b)

Figure 4.42 Results of Shrinkage Pressure of the Fort Worth Soil at PI Condition: a) Time (hr.) versus SIP and b) Time (hr.) versus Water Content (%)

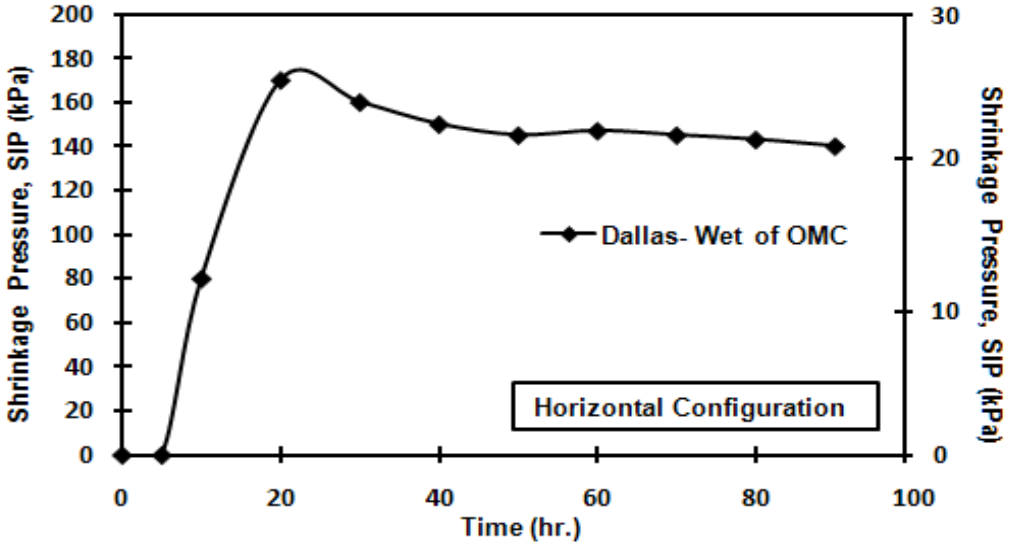


(a)

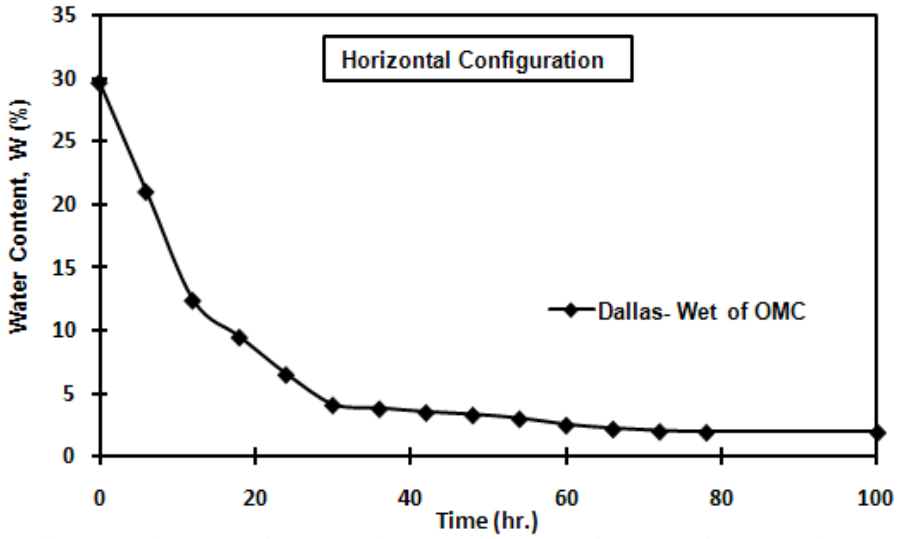


(b)

Figure 4.43 Results of Shrinkage Pressure of the Fort Worth Soil at PI Condition:
a) SIP versus Water Content (%) and b) The Percent Change of Water Content (%) versus SIP

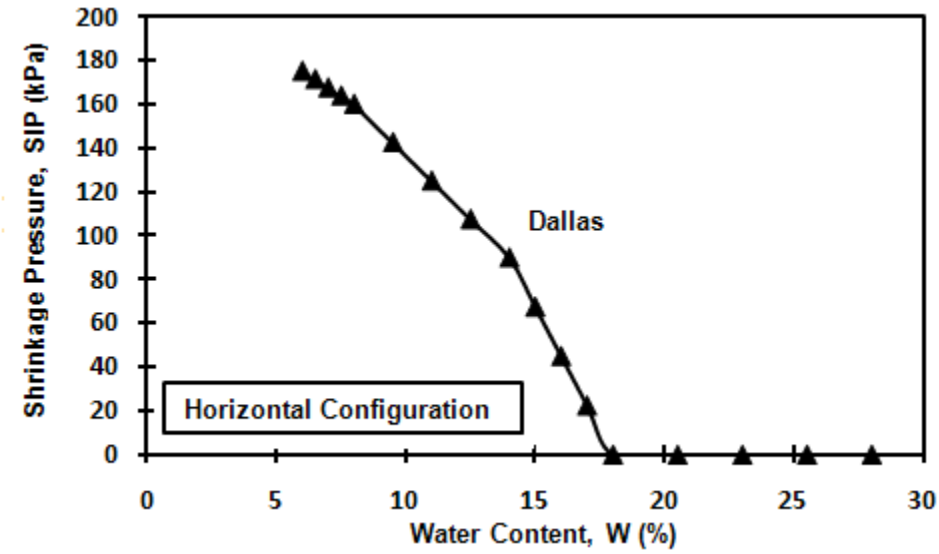


(a)

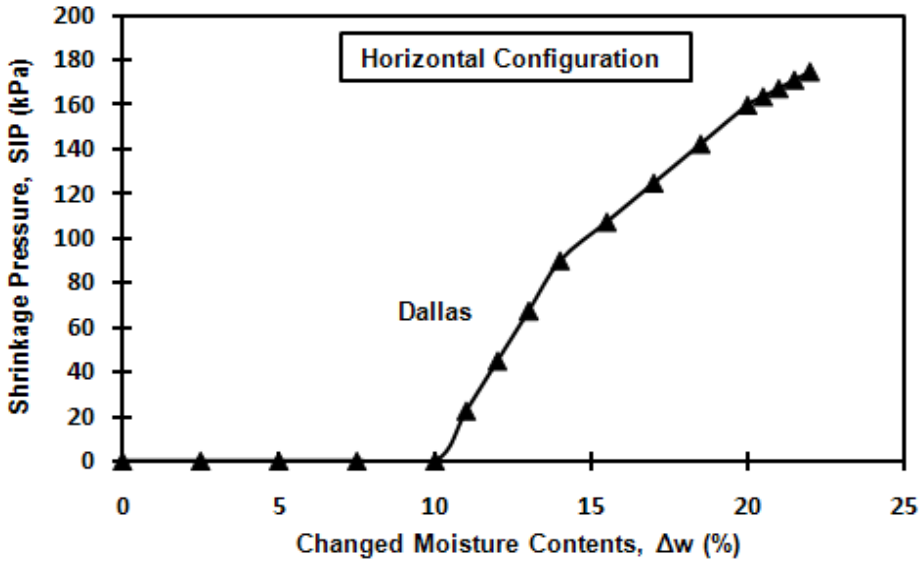


(b)

Figure 4.44 Results of Shrinkage Pressure of the Dallas Soil at Wet of OMC Condition: a) Time (hr.) versus SIP and b) Time (hr.) versus Water Content (%)

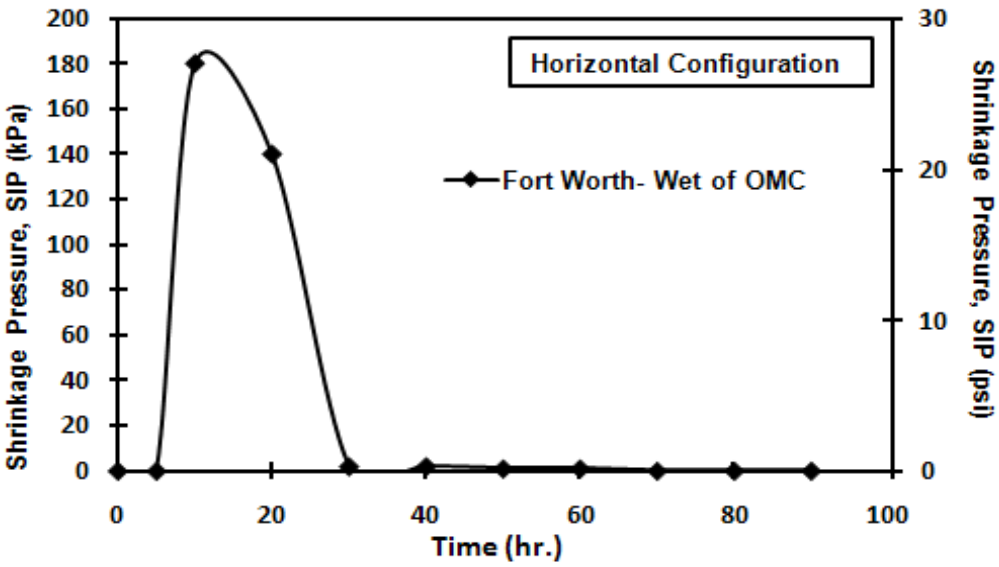


(a)

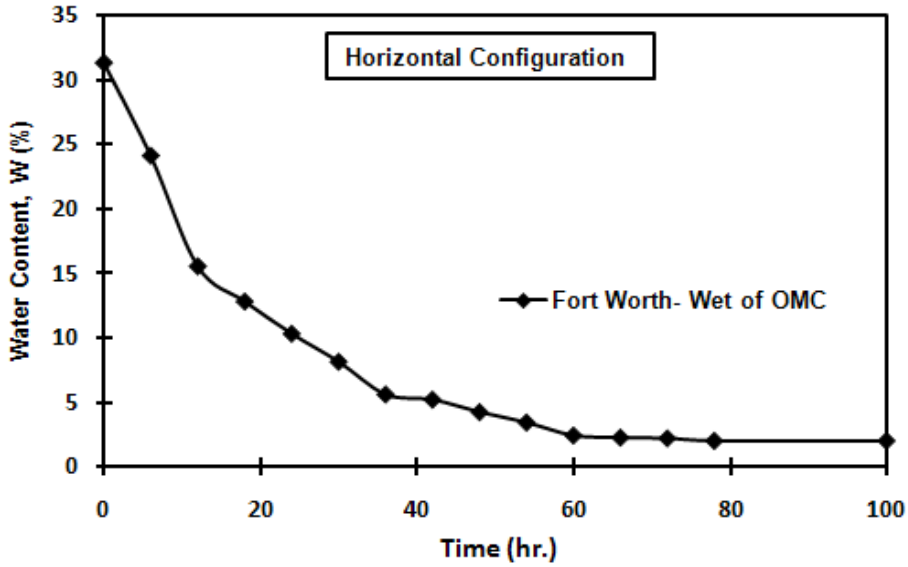


(b)

Figure 4.45 Results of Shrinkage Pressure of the Dallas Soil at Wet of OMC Condition:
a) SIP versus Water Content (%) and b) The Percent Change of Water Content (%) versus SIP

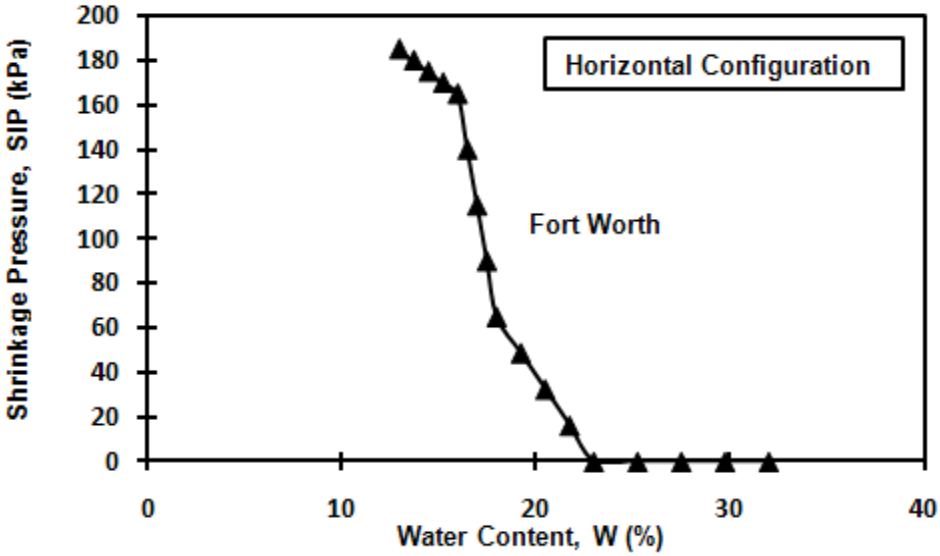


(a)

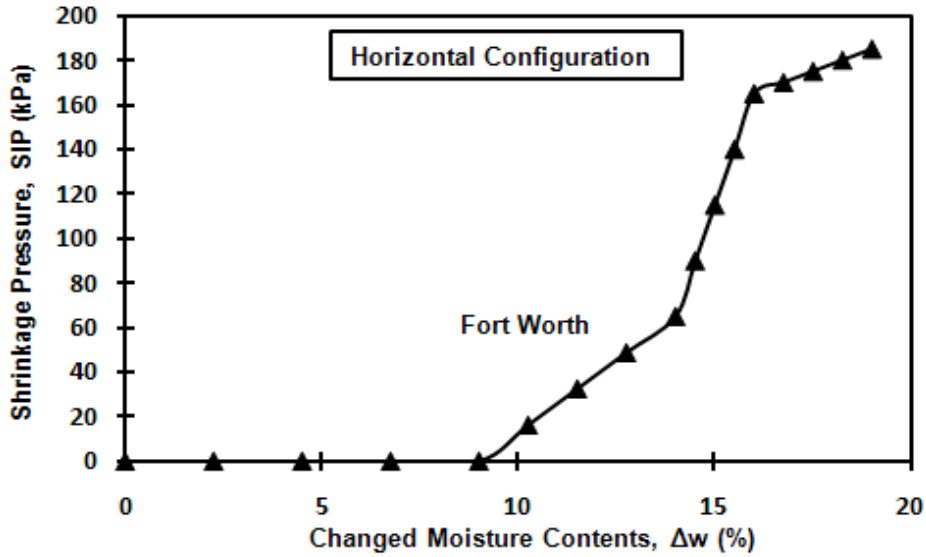


(b)

Figure 4.46 Results of Shrinkage Pressure of the Fort Worth Soil at Wet of OMC Condition: a) Time (hr.) versus SIP and b) Time (hr.) versus Water Content (%)

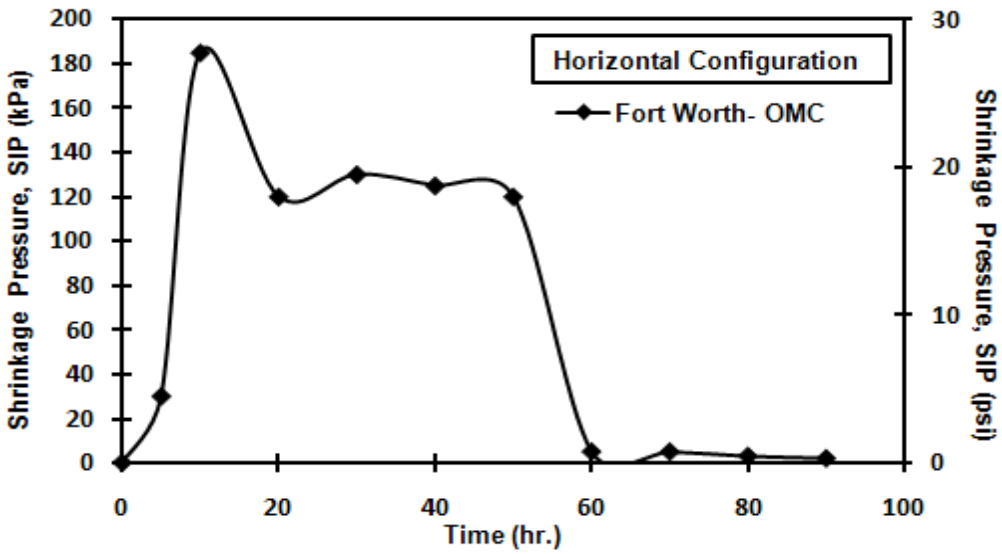


(a)

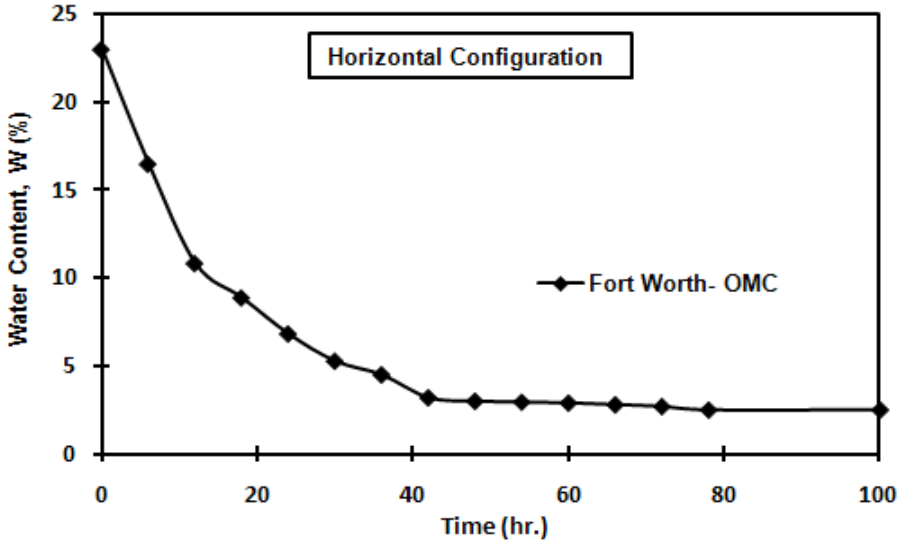


(b)

Figure 4.47 Results of Shrinkage Pressure of the Fort Worth Soil at Wet of OMC Condition: a) SIP versus Water Content (%) and b) The Percent Change of Water Content (%) versus SIP

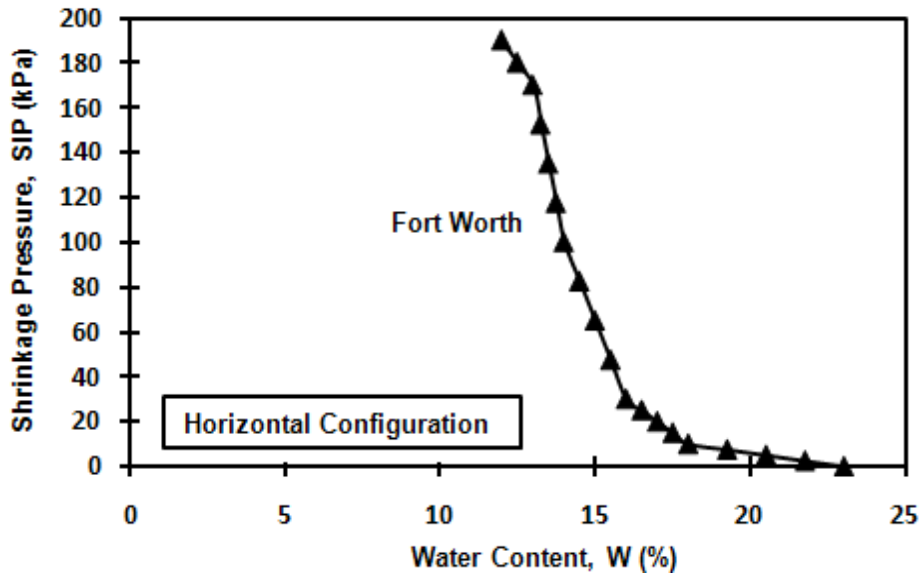


(a)

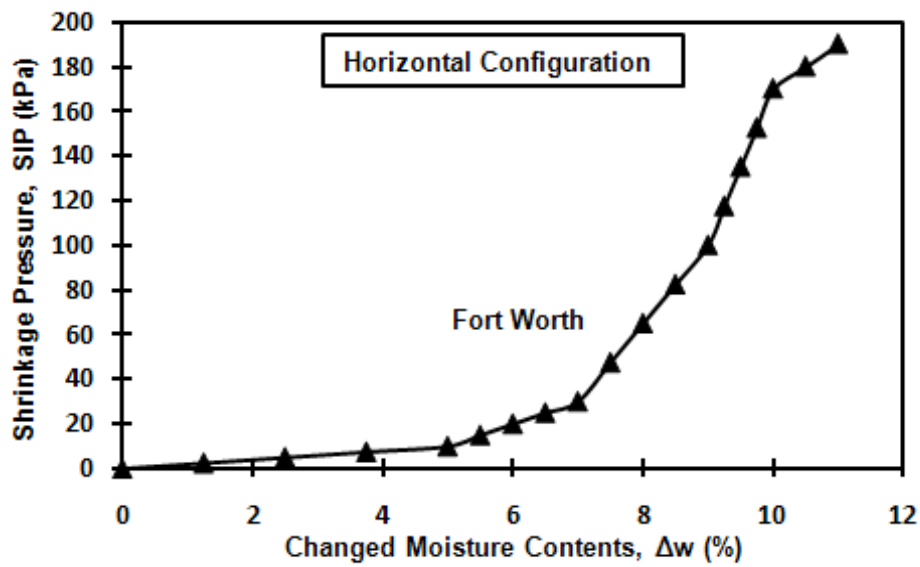


(b)

Figure 4.48 Results of Shrinkage Pressure of the Fort Worth Soil at OMC Condition: a) Time (hr.) versus SIP and b) Time (hr.) versus Water Content (%)



(a)



(b)

Figure 4.49 Results of Shrinkage Pressure of the Fort Worth Soil at OMC Condition:
a) SIP versus Water Content (%) and b) The Percent Change of Water Content (%) versus SIP

After the specimens were prepared and tested in the aforementioned processes, the percent of successes for LL+10%, PL+10%, Wet of OMC, and OMC test conditions are equal to 87%, 60%, 20%, and 10%, respectively as shown in Table 4.6. A few tests could not be completed due to lack of sufficient amounts of those soils for the tests. Both Wet of OMC and OMC conditions could not provide better results because soil specimens crack before the final readings and hence the percent of successful tests are low for these conditions. Also, these specimens are compacted at low moisture content conditions which may have resulted in lesser shrinkage cracking during drying. Nevertheless the SIPH test provided repeatable measurements for other high moisture content conditions which are critical in the assessments of stability of infrastructure built on expansive soils.

Overall, poor repeatability are detected at other compaction moisture content states, which is attributed to lack of moisture contents to adhere the FS to the soils properly. However, this condition is not critical for SIP measurements as soils at this state undergo lesser shrinkage cracks when compared to soils that are wet or close to saturation.

Figure 4.50 presents SIPH results of El Paso, Dallas, Fort Worth, and Paris clays and it can be mentioned that high SIP value is measured for Paris clays and low SIP value is measured for El Paso clay. The Paris soil has the highest shrinkage pressure which is 72.24 psi while Fort Worth, Dallas, and El Paso soils have the shrinkage pressures of 42.14, 40.52, and 36.41 psi, respectively. Differences in these plots are attributed to percent Montmorillonite (MM) in each soil as Paris soil has high MM whereas El Paso has low MM. This explains the significance of MM content in soil and its related shrinkage cracking during drying.

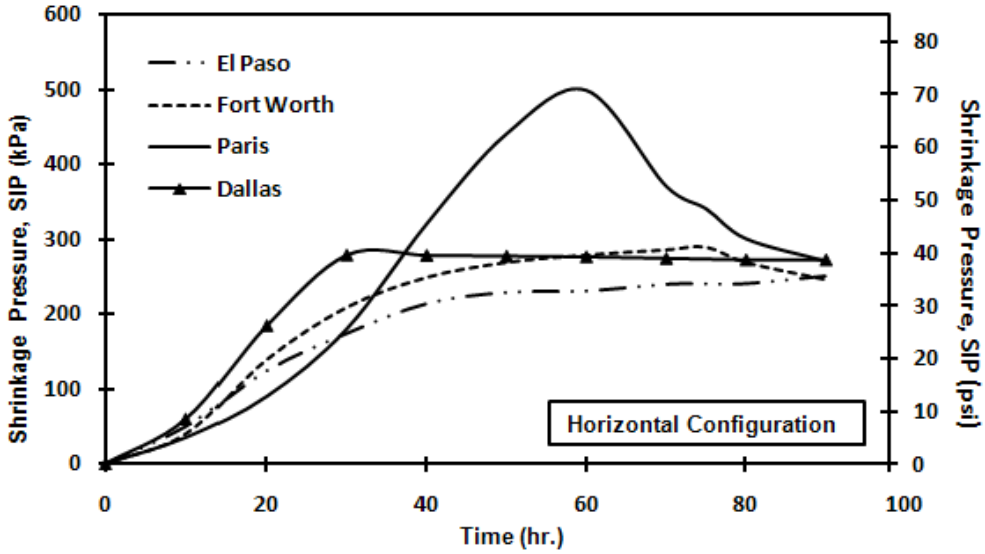


Figure 4.50 Comparisons of SIPs among Expansive Soil from Dallas, El Paso, Fort Worth, and Paris at LL+10% Condition

Table 4.6 Summary of Shrinkage Pressure Results from Each Soil

Moisture Cond.	**Soil Types psi, (kPa)**							
	El Paso	**SD**	**Dallas**	**SD**	**Fort Worth**	**SD**	**Paris**	**SD**
LL+10%	36.41 (251.03)	2.92 (20.08)	40.52 (279.38)	4.05 (27.92)	42.14 (290.54)	6.32 (43.57)	72.24 (498.06)	6.65 (45.85)
PL+10%	NA	NA	36.42 (251.04)	6.19 (42.69)	38.43 (264.97)	6.53 (45.04)	NA	NA
Wet of OMC	NA	NA	25.38 (174.99)	NA	28.29 (195.05)	NA	NA	NA
OMC	NA	NA	27.53 (189.81)	NA	NA	NA	NA	NA

Note: SD -Standard deviation in psi, (kPa)

4.4.3 Influence of Soil Variables

4.4.3.1 Soil Properties

For characterization, it is desirable to have a comprehensive understanding of the soil mineralogy as well as volume change related characteristics of the soils. Thus, several factors including PI (%), CEC values (meq/100gm), volumetric shrinkage strain (%), volumetric swell strain (%), and swell pressure (kPa) are correlated with the measured SIP values and these results are presented in Figures 4.51 to 4.55.

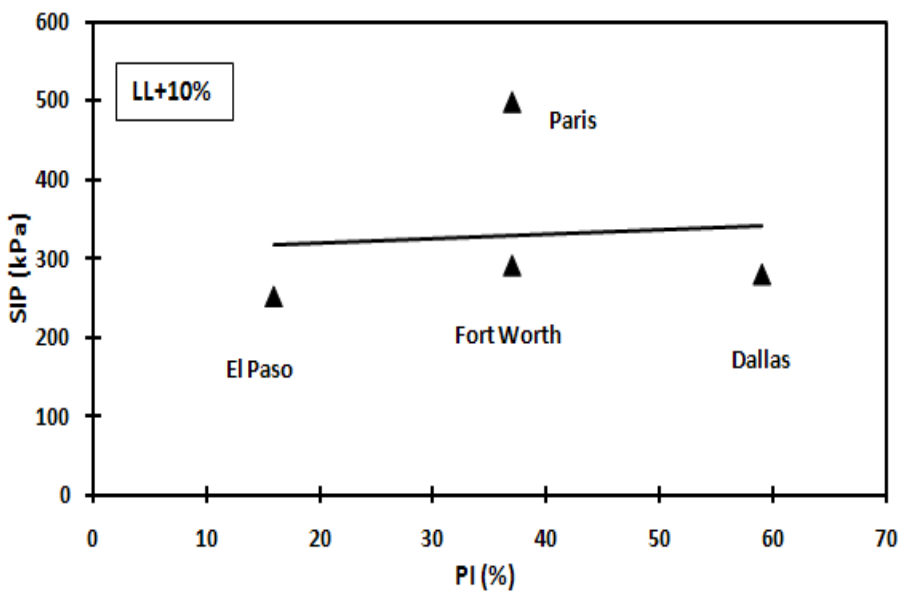


Figure 4.51 Correlation between PI (%) and SIP (kPa)

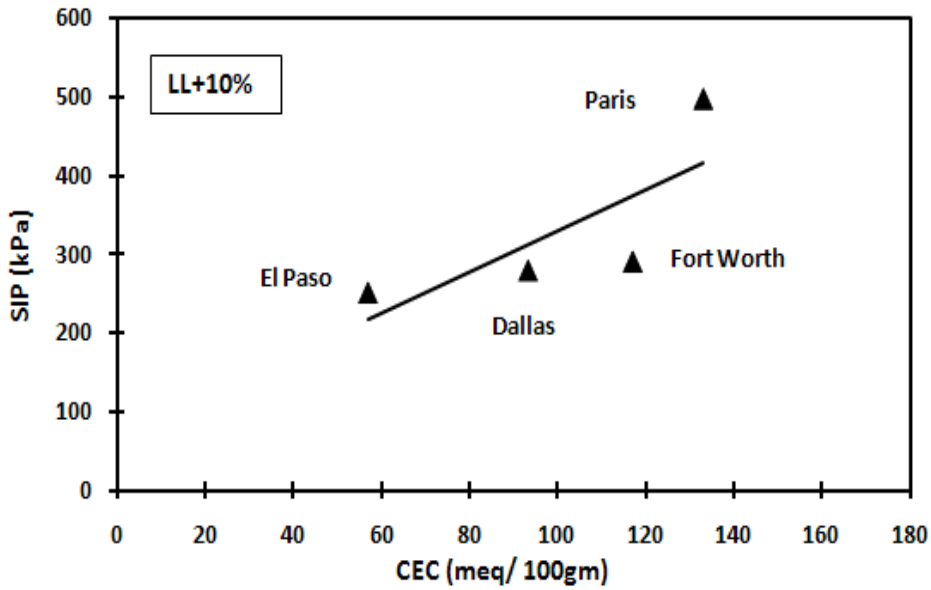


Figure 4.52 Correlation between CEC (meq/100gm) and SIP (kPa)

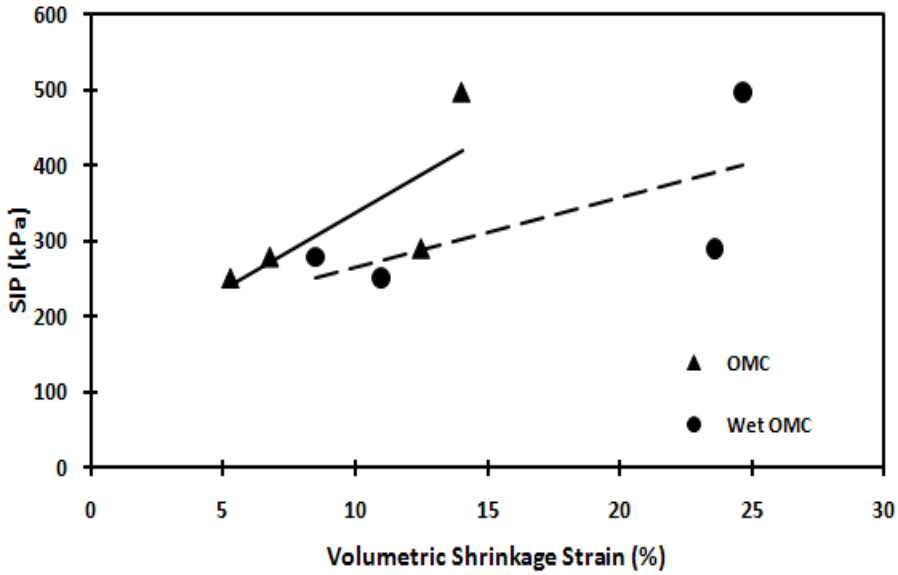


Figure 4.53 Correlation between Volumetric Shrinkage Strain (%) and SIP (kPa)

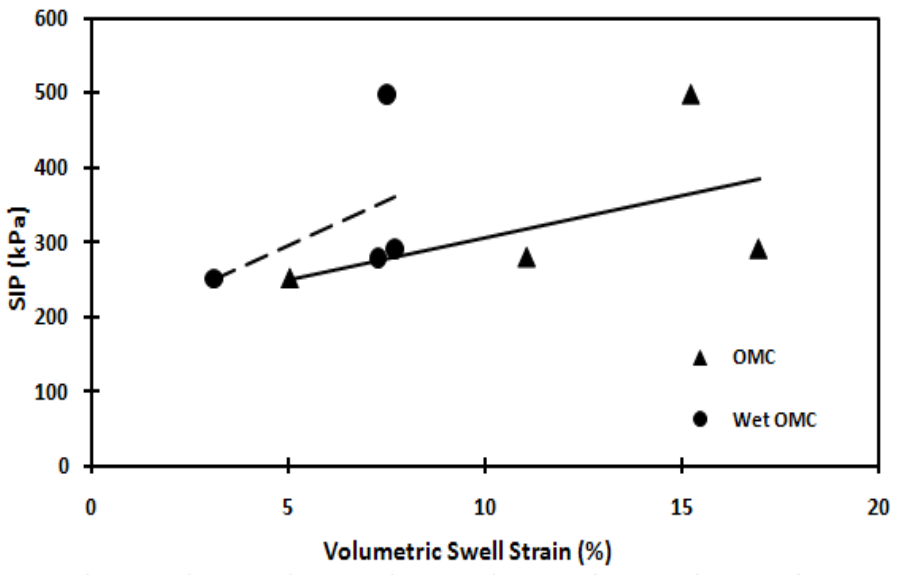


Figure 4.54 Correlation between Volumetric Swell Strain (%) and SIP (kPa)

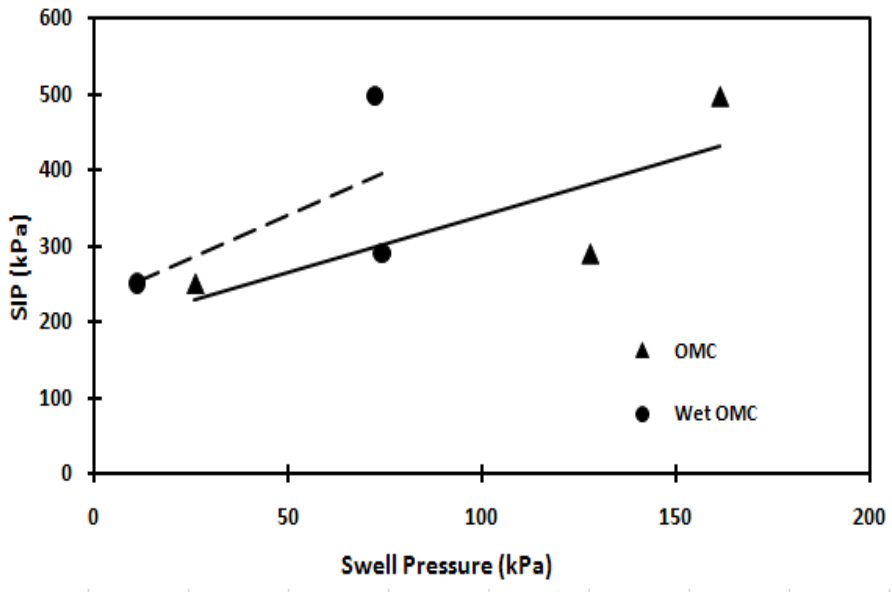


Figure 4.55 Correlation between Swell Pressure (kPa) and SIP (kPa)

From Figure 4.51 (PI vs SIP results), the PI values show influence on SIP measurements at LL conditions. For engineering tests, volumetric swell and shrinkage strains measured at OMC and Wet of OMC conditions are correlated with corresponding SIP values. It should be noted that there are some problems of SIPH tests at these moisture content conditions. These results show that higher variability which is expected due to difficulty in measuring SIP values at these conditions. Interestingly, the CEC value which is an indicator of MM presence in soils is correlated well with SIP value. Since this study is performed only on fours soils, it is difficult to explain all trends in these results. Nevertheless, the results show that this property can be correlated with soil properties that indicate swell and shrink characteristics.

4.4.4 Correlation with Indirect Tensile Strength (IDT)

The shrinkage induced pressure is a direct method to determine pressure from shrinkage type conditions in soils. In the past, the indirect tensile strength is used to determine the tensile strength of soil by breaking the soil specimens perpendicular to the compressive load direction. IDT results of each soil at different moisture contents were determined as per ASTM D3967-95a and these results were compared with SIP results as shown in table 4.7. An attempt is also made to develop correlations between SIP and IDT values.

Table 4.7 The Comparisons of the Results of Shrinkage Induced Pressure (SIP) Test and Indirect Tensile Strength (IDT) Test

Moisture Cond.	Soil Types	Average of Maximum SIP psi, (kPa)	Standard Deviation (SIP) psi, (kPa)	Average of IDT Final Cond. psi, (kPa)	Standard Deviation (IDT) psi, (kPa)	(SIP/ IDT) x 100 (%)
LL+10%	El Paso	36.41 (251.03)	2.92 (20.08)	45.44 (313.29)	9.08 (62.66)	80.13
	Dallas	40.52 (279.38)	4.05 (27.92)	47.12 (324.87)	4.34 (29.89)	85.60
	Fort Worth	42.14 (290.54)	6.32 (43.57)	49.78 (343.22)	4.98 (34.34)	84.65
	Paris	72.24 (498.06)	6.65 (45.85)	74.31 (512.25)	18.34 (126.5)	97.21
PL+10%	Dallas	36.42 (251.04)	6.19 (42.69)	48.73 (335.98)	12.18 (84.01)	74.74
	Fort Worth	38.43 (264.97)	6.53 (45.04)	59.56 (410.65)	14.89 (102.7)	64.52
Wet of OMC	Dallas	25.38 (174.99)	NA	NA	NA	NA
	Fort Worth	28.29 (195.05)	NA	NA	NA	NA
OMC	Fort Worth	27.53 (189.81)	NA	NA	NA	NA
	Dallas	NA	NA	NA	NA	NA

Note: NA- The tests were inapplicable due to soil specimens cracked at the final moisture content.

The test results show that the percent ratio between SIP and IDT is below 100%, which means the actual SIP values are lower than the indirect tensile strengths of tested soils. The weak point of this comparison analysis is the standard deviation of IDT results is high at plastic moisture content condition. The percent ratio of SIP to IDT test results for LL + 10% is about 86.9%. Figure 4.56 presents comparisons between SIP values and IDT values. Another advantage of SIP test is that this test can be performed at high moisture content conditions including wet of optimum moisture content condition. The IDTs can be performed at OMC conditions and not at high moisture content conditions. Also, SIP values provide direct shrinkage pressure from volume change related shrinkage conditions and hence more appropriate for the design of civil structures on expansive soils that undergo inherent volume change fluctuations from seasonal changes.

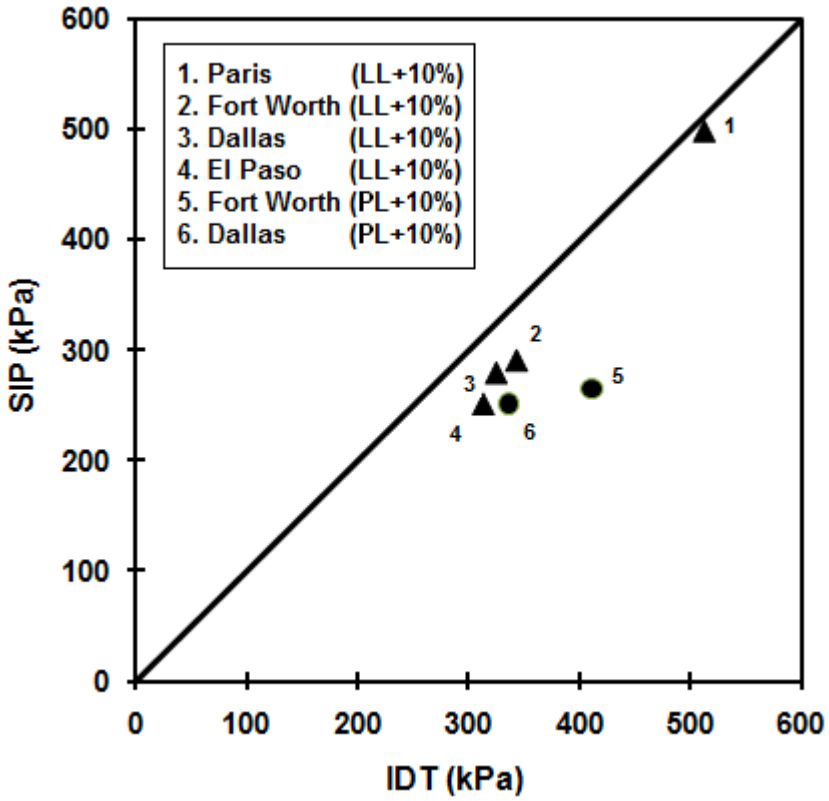


Figure 4.56 Comparisons between IDT (kPa) and SIP (kPa)

4.5 Summary

This chapter presents the reviews of force sensor (FS) specification used in this thesis and also details of a new shrinkage pressure measurement by using Shrinkage Induced Pressure (SIP) test. Both vertical orientation (SIPV) and horizontal orientation (SIPH) of force sensor in the reliable measurement of SIP values is studied and validated. A horizontal configuration (SIPH) is recommended for SIP tests on expansive soils.

Different compaction moisture content conditions, LL+10%, Wet of OMC, OMC, and PL+10% of four soils were studied and these results are used to compare the SIP values with various swell, shrinkage and PI properties. Also, from comparisons of SIP results with IDT results, it is observed that SIP results at LL+10% provided the ratio between SIP to IDT results close to 80% or higher. Several advantages of SIP measurements over IDT measurements and various application areas are also described.

CHAPTER 5

SUMMARY OF FINDINGS AND FUTURE RESEARCH DIRECTIONS

5.1 Introduction

Swell and shrinkage behaviors of soils related to movements occur in the soils underneath structure based on moisture content fluctuations in wet and dry seasons. These soil behaviors adversely affect the civil infrastructure built above the soil. According to Nahlawi and Kodikara (2002), the significant amount of curling or shrinkage pressure which is induced by soil shrinkage could cause severe damage to the structures. In this study, three highly expansive soils from Paris, Dallas, and Fort Worth in Texas were tested and compared with a moderate expansive soil from El Paso, Texas.

The main objective of this research is to develop a shrinkage induced pressure test procedure to determine shrinkage induced pressure in order to better understand the shrinkage behavior in soils. This thesis presents this new technique or measurement procedure for direct shrinkage pressure measurements inherently induced inside the matrix of expansive clay soil. This objective was fully accomplished as a part of the experimental program with four different soils and several variables fully investigated with this new method.

In the past, due to non-availability of sensor technology, shrinkage of soil was not studied. However, at present, there is a new sensor called Force Sensor (FS), which can detect forces acting on a small area of the device. Thus, this research investigated the study of using FS for shrinkage induced pressure measurements in a test termed as Shrinkage Induced Pressure (SIP) test. Position in which the FS can be used for repeatable measurements is also investigated. For reliability of the test results, the SIP data is correlated with tensile strength measurements obtained from Indirect Tensile Strength (IDT) test.

The following conclusions are obtained from the analysis of test results presented in Chapter 4. These conclusions are based on the majority of the trends noted in the present data. These conclusions may still be valid for other similar types of soils and more studies are also needed to further advance the shrinkage characterization area that is relatively under-researched.

5.2 Summary of Findings

The following lists the major conclusions obtained from this research:

1. Shrinkage Induced Pressure (SIP) test was successfully developed and the methodology was demonstrated for measuring SIP values for various soil types.

2. In order to evaluate both repeatability and reliability of SIP measurements, similar tests were conducted on identical soil samples. Initially the results showed high variability when FS was used in vertical configuration as discussed in SIPV tests. Hence, subsequent tests were planned with horizontal configuration. Based on a few other tests performed on this configuration, it is determined that the horizontal configuration or direction is the appropriate orientation for SIP test using Force Sensor (FS). Horizontal orientation SIP test (SIPH) provided good consistency and repeatable test results.

3. Reliability measurements are difficult to evaluate as no other methods are reported that provide direct measurements of shrinkage pressures. Hence, an attempt is made to correlate shrinkage induced pressure with indirect tensile strength or IDT that is also an indicator of tensile loading that result in shrinkage vertical cracking. Based on the comparisons of the present soil test results, both SIP and IDT values are comparable within the range of moistures tested from Wet of OMC to LL+10%. It should be noted that IDT were performed on the soil samples prepared at Wet of OMC to LL+10% after oven drying. More tests at Wet of OMC condition are still needed to have adequate results to check consistency and reliability of the test data. Overall, it can be mentioned that the SIP test measurement of shrinkage pressure

is well correlated with the IDT test results on shrunk soil specimens prepared at high moisture condition.

4. Based on the SIP tests conducted on four soils, it can be mentioned that test results at all moisture content conditions (LL+10%, PL+10%, Wet of OMC, and OMC) are different and the success of SIP test was also varied with the respect to test moisture content. Overall, tests were successful when tests were conducted at LL+10% moisture content condition as these tests provided better consistency and repeatability of the data, compared with other conditions.

5. From the SIP results conducted on all four soils at LL+10% condition at horizontal FS orientation show that the Paris soil has exhibited highest SIP value of 72.24 psi. Fort Worth, Dallas, and El Paso soils have the next higher shrinkage pressures 42.14, 40.52, and 36.41 psi, respectively. Also, the soil that exhibited highest SIP value had the highest percentage of Montmorillonites in soils. This reconfirms that the higher the finer fraction, higher the SIP value.

6. Lastly, other soil variables including Plastic Index (PI), volumetric shrinkage and swell behavior also provided good correlation with the SIP data, reconfirming the procedure provided reliable results that follow certain trends with the soil types.

5.3 Future Research

1. Soil samples from different locations and soil samples with different percents of Montmorillonites are needed to be tested for further validation of the findings and for the development of a SIP test based model for better estimating of SIP values in soils.

2. Investigations on other parameters of soils which are potentially related to expansive soil volume changes especially shrinkage behavior and cracking initiation are needed.

3. SIP values can be another parameter along with swelling pressures that can be used for better design in civil structure on expansive soils in order to prevent damage of the structure due to volume change of soils. This will further enhance the understanding of soil behavior. Also, there is a need to incorporate unsaturated soil mechanics related to soil suction on the shrinkage pressures of expansive soils.

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BIOGRAPHICAL INFORMATION

Flt. Lt. Varagorn Puljan was born in Nakhon Sawan, THAILAND. He graduated from Royal Thai Airforce Academy (RTAFA), Bangkok, THAILAND, with a Bachelor's Degree in Civil Engineering in 2005. After receiving a Bachelor's Degree in Civil Engineering from the RTAFA, he decided to become an instructor at Thai Royal Airforce Academy and after working as instructor for 3 years, he obtained and won the Royal Thai Government Scholarship for continuing Master's Degree to study aboard. He has long been greatly interested in Geotechnical Engineering as it would provide an insight picture into the foundation of infrastructure. This would particularly be useful for the Air Force as well as to the RTAFA as it is still in high demand for more specialists in this field.

In 2008, he pursued master program in the Department of Civil Engineering at the University of Texas at Arlington (UTA), Arlington, Texas with Geotechnical Engineering as the major area of research. At UTA, he performed research in studies on expansive soils behaviors under the guidance of Prof. Anand J. Puppala and successfully defended his thesis in November 2010. During the course of his study, he worked in various research areas related to field instrumentation, shrinkage pressure, Indirect Tensile Test (IDT), and expansive soils behaviors.